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**RAPID POST-EARTHQUAKE
DISPLACEMENT-BASED
ASSESSMENT METHODOLOGY
FOR BRIDGES – PHASE I**

Prepared by:

Ana Paula Bona, Ph.D. Candidate
Jungmin Lee, Ph.D. Student
Mervyn Kowalsky, Ph.D., Principal Investigator
Ashly Cabas, Ph.D., Principal Investigator

Prepared for:

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Facilities (AKDOT&PF)



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Constructed Facilities Laboratory
2414 Campus Shore Drive
North Carolina State University
Raleigh, NC 27695-7533
Tel: (919) 513-1733
Fax: (919) 513-1765
Email: cfl@ncsu.edu
Web Site: www.cfl.ncsu.edu

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Final Report

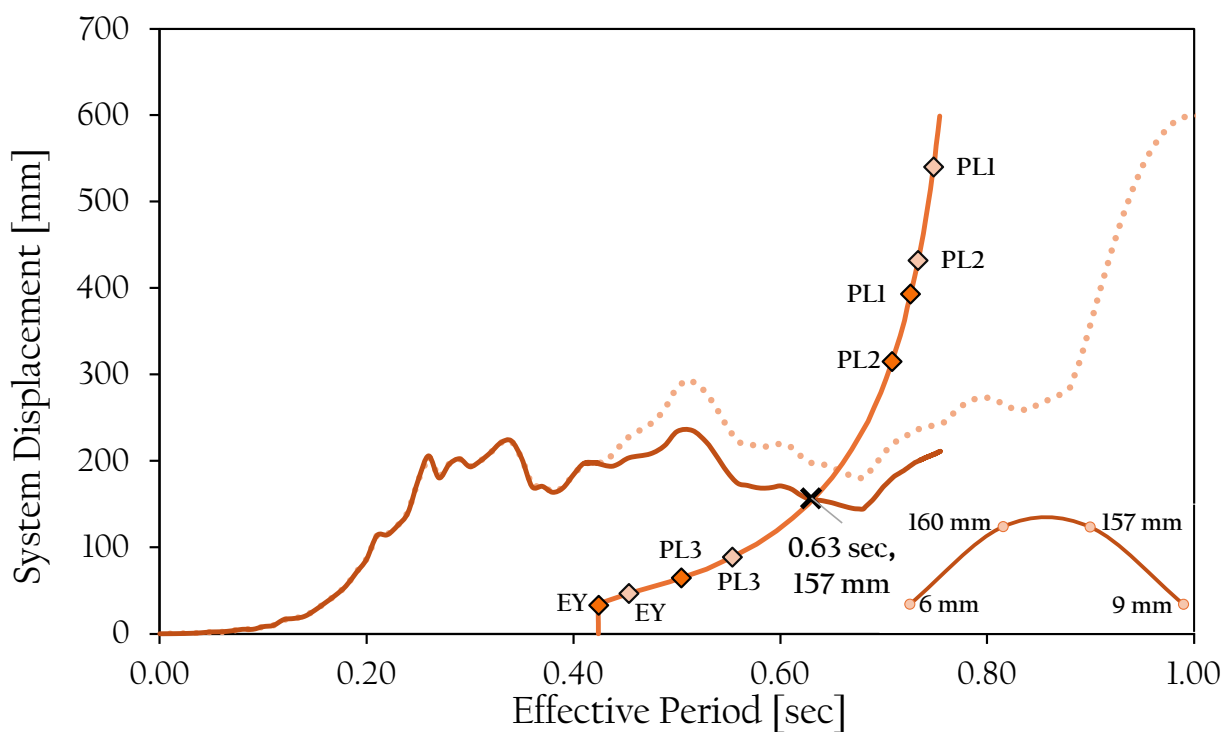
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FINAL REPORT



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Ana Paula Bona, Ph.D. Candidate

Jungmin Lee, Ph.D. Student

Mervyn Kowalsky, Ph.D., Principal Investigator

Ashly Cabas, Ph.D., Principal Investigator

Raleigh, North Carolina

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METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply by	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
<u>LENGTH</u>					<u>LENGTH</u>				
in	inches	25.4		mm	mm	millimeters	0.039	inches	in
ft	feet	0.3048		m	m	meters	3.28	feet	ft
yd	yards	0.914		m	m	meters	1.09	yards	yd
mi	Miles (statute)	1.61		km	km	kilometers	0.621	Miles (statute)	mi
<u>AREA</u>					<u>AREA</u>				
in ²	square inches	645.2	millimeters squared	cm ²	mm ²	millimeters squared	0.0016	square inches	in ²
ft ²	square feet	0.0929	meters squared	m ²	m ²	meters squared	10.764	square feet	ft ²
yd ²	square yards	0.836	meters squared	m ²	km ²	kilometers squared	0.39	square miles	mi ²
mi ²	square miles	2.59	kilometers squared	km ²	ha	hectares (10,000 m ²)	2.471	acres	ac
ac	acres	0.4046	hectares	ha					
<u>MASS (weight)</u>					<u>MASS (weight)</u>				
oz	Ounces (avdp)	28.35	grams	g	g	grams	0.0353		oz
lb	Pounds (avdp)	0.454	kilograms	kg	kg	kilograms	2.205		lb
T	Short tons (2000 lb)	0.907	megagrams	mg	mg	megagrams (1000 kg)	1.103	short tons	T
<u>VOLUME</u>					<u>VOLUME</u>				
fl oz	fluid ounces (US)	29.57	milliliters	mL	mL	milliliters	0.034	fluid ounces (US)	fl oz
gal	Gallons (liq)	3.785	liters	liters	liters	liters	0.264	Gallons (liq)	gal
ft ³	cubic feet	0.0283	meters cubed	m ³	m ³	meters cubed	35.315	cubic feet	ft ³
yd ³	cubic yards	0.765	meters cubed	m ³	m ³	meters cubed	1.308	cubic yards	yd ³
Note: Volumes greater than 1000 L shall be shown in m ³									
<u>TEMPERATURE (exact)</u>					<u>TEMPERATURE (exact)</u>				
°F	Fahrenheit temperature	5/9 (°F-32)	Celsius temperature	°C	°C	Celsius temperature	9/5 °C+32	Fahrenheit temperature	°F
<u>ILLUMINATION</u>					<u>ILLUMINATION</u>				
fc	Foot-candles	10.76	lux	lx	lx	lux	0.0929	foot-candles	fc
fl	foot-lamberts	3.426	candela/m ²	cd/cm ²	cd/cm ²	candela/m ²	0.2919	foot-lamberts	fl
<u>FORCE and PRESSURE or STRESS</u>					<u>FORCE and PRESSURE or STRESS</u>				
lbf	pound-force	4.45	newtons	N	N	newtons	0.225	pound-force	lbf
psi	pound-force per square inch	6.89	kilopascals	kPa	kPa	kilopascals	0.145	pound-force per square inch	psi
These factors conform to the requirement of FHWA Order 5190.1A *SI is the symbol for the International System of Measurements									

TABLE OF CONTENTS

LIST OF TABLES	iii
LIST OF FIGURES	iv
ABBREVIATIONS	vii
LIST OF SYMBOLS	ix
ACKNOWLEDGMENTS	xii
AUTHORSHIP NOTE.....	xii
EXECUTIVE SUMMARY	xiii
1 Chapter 1: Introduction	1
1.1 Motivation.....	1
1.2 Objectives	4
1.3 Expected Outcomes	5
1.4 Report Layout	6
2 Chapter 2: Literature Review.....	9
2.1 Post-Earthquake Assessment Analysis Engines	9
2.1.1 HAZUS	13
2.1.2 ShakeCast.....	21
2.1.3 Nonlinear Time History Analysis (NLTHA).....	28
2.1.4 Direct Displacement-Based Assessment.....	29
2.1.5 Summary of Analysis Engines.....	41
2.2 Bridge Performance Limit States.....	43
2.2.1 Strain Limits.....	44
2.2.2 Converting Limit States Strains to Member Deformations	46
2.3 Ground-Motion Characterization.....	48
2.3.1 Existing Ground Motion Models to Predict Spectral Displacements	50
2.3.2 Site Parameter, V_{S30} , to Characterize Local Site Response	56
2.3.3 Ground-Motion Modification to Obtain Input Motions at the Reference Site	60
3 Chapter 3: Direct Displacement-Based Assessment Method on Multi-Span Bridge Inventories.....	66
3.1 Bridge Data Collection	67
3.2 Subsurface Data Collection.....	68
3.3 Estimation of Bridge Pushover Curve	69
3.4 Generation of Bridge Performance Spectra	72
3.5 Seismic Demand Characterization.....	74

3.6	Calculation of Variable Demand Spectra	78
3.7	Estimation of Final Displacement.....	81
4	Chapter 4: Rapid Post-Earthquake Assessment Implementation.....	83
5	Chapter 5: Performance Limit State Uncertainty.....	106
6	Chapter 6: Conclusions and Future Work.....	116
6.1	Future Work - Phase II.....	118
	REFERENCES	127
	Appendix A Post-Earthquake Assessment of Additional Bridges.....	133
	Appendix B User Guide – DDBA MATLAB Code	208

LIST OF TABLES

Table 1. HAZUS grouping and fragility relationships for three bridge classes (Modified after FEMA, 2022)	17
Table 2. Median values for the proposed fragility functions vs. HAZUS. (In PSA 1.0 sec) (Source: Biasi et al. (2017))	28
Table 3. Performance Limit States evaluated and their concrete and reinforcing steel strain limit	45
Table 4. Site and geology information at selected bridges	69
Table 5. Equations for obtaining values of spectral reduction factor.	80
Table 6. Database built for Bridge 610.	85
Table 7. Database built for Bridge 646.	85
Table 8. Summary of moment-curvature values for Bridges 610 and 646.	87
Table 9. Summary of force-displacement values for the bilinear relationship for Bridges 610 and 646.	88
Table 10. Summary of limit state displacement values for each pier column for Bridge 610 and 646.	89
Table 11. Epicentral distances and proxy-based V_{S30} at the selected bridges from Geyin and Maurer (2022)	90
Table 12. Summary of the system displacement and total base shear for the four performance limit states for both piers of Bridge 610.	92
Table 13. Summary of the system displacement and total base shear for the four performance limit states for both piers of Bridge 646.	92
Table 14. Summary of the effective period and system displacement for the four performance limit states for both piers of Bridge 610.	96
Table 15. Summary of the effective period and system displacement for the four performance limit states for both piers of Bridge 646.	96
Table 16. Computed scaling factors regarding the target displacement at $T = 1$ sec	98
Table 17. Final Performance information regarding Bridge 610 and 646, including priority of inspection between both bridges	104
Table 18. Variable parameters with lower and upper-limit values	107
Table 19. Mean and standard deviation values of the limit state displacements.	114
Table 20. The number of nearby station and input parameters to estimate spectral displacement in each bridge	134
Table 21. Summary of bridges chosen for post-earthquake assessment.	135
Table 22. Inspection priority and performance limit states achieved by nine bridges.	136

LIST OF FIGURES

Figure 1. Intensity Map from 2014 M6.0 Napa Valley, CA earthquake generated by ShakeMap. (Source: Worden et al., 2020)	11
Figure 2. A schematic of the basic ShakeMap ground motion interpolation scheme (Source: Worden & Wald, 2016).....	12
Figure 3. Hazus Earthquake Model Methodology Schematic (Source: FEMA, 2022)	15
Figure 4. HAZUS bridge classification scheme.	16
Figure 5. Example of fragility curves (the use of the terms weak, medium, and strong shaking is applied here for simplicity, in the actual curves, quantitative values of spectral response are used). (Source: FEMA, 2001)	17
Figure 6. Observed and estimated number of simply supported bridges. (Source: Ranf et al. ,2007)	19
Figure 7. System and component fragilities for selected bridge classes for moderate damage state (a) two-span and (b) three span. (Source: Mangalathu et al., 2017).....	20
Figure 8. Proposed bridge classification scheme (Source: Mangalathu et al., 2017)	20
Figure 9. Facility type editing page of the ShakeCast administrator interface. (Source: Turner et al., 2009)	22
Figure 10. ShakeCast web interface for bridge inspection priority list. (Source: modified from Turner et al., 2009).....	23
Figure 11. ShakeCast flowchart indicating ShakeMap data, user's inventory, and notifications. (Source: Lin & Wald, 2008)	24
Figure 12. System and component level fragility curves for post 1990 MSCC-BG bridges with multi-column bent equipped with modular joint seal assembly system for BSST-0 and BSST-3. (Source: (K. N. Ramanathan, 2012))	26
Figure 13. Comparison of the developed adjustment factor with the HAZUS proposed modification factor for post-1990 design era. (SKL, SKM, SKH, SKVH, SKEx are low, medium, high, very high, and extreme skew angles, respectively). (Source: Soleimani, 2017) ...	27
Figure 14. Fundamentals of Direct Displacement-Based Design. (Adapted from Priestley et al., 2007)	30
Figure 15. Flowchart representation of a proposed DDBA process for an SDOF system.	31
Figure 16. Equivalent SDOF representation of a multi-span bridge under transverse displacement for DDBA application.	32
Figure 17. Flowchart for Direct Displacement-Based Assessment of multi-span bridges (Source: Şadan et al., 2013).....	33
Figure 18. Δ_{sys} , $THA\Delta_{sys}$, $DDBA$ ratios for 27 bridges at (a) yield limit state, (b) serviceability limit state, and (c) damage control limit state. (Source: Kong, 2017)	35
Figure 19. Relationship between median values of $\Delta_{sys} \cdot D$ and ΔH (Source: Kong, 2017)	37
Figure 20. Limit state displacement over demand (LS/D) for the seven Alaskan bridges, using the DDBA analysis method and IDA to verify the results for (a) yield limit state, (b) serviceability limit state, (c) damage control limit state. (Source: Cetinkaya, 2019)	39
Figure 21. Mean displacement ratio between NLTHA and DDBA for 3 different types of hysteretic model per DDBA ductility level for both tectonic regimes.	41
Figure 22. Equivalent curvature distribution for a single bending bridge pier (Adapted from Priestley et al., 2007)	47

Figure 23. Map of Alaska showing the fault and interface sections used in the 2023 Alaska National Seismic Hazard Model. Epicenters (red dots) of recent significant earthquakes with moment magnitude (M) are included for reference (after Powers et al. 2024).....	49
Figure 24. Spatial variability in earthquake ground motions (indicating peak ground acceleration, PGA) induced by 2018 Anchorage earthquake (Cabas et al., 2021)	50
Figure 25. Example of spectral displacements based on the AB07 model: $M = 7$, Reverse fault, Soft soil, $R_{jb} = 2$ km (adapted from Akkar & Bommer, 2007)	52
Figure 26. Example of spectral displacements obtained from the F04 model with $r = 5$ km, $\Delta\sigma = 3$ MPa and M_w of 7 (red) and 6.5 (blue).	53
Figure 27. Example of spectral displacements based on the CB08 model: $M_w = 7$, $R_{rup} = 54.8$ km, $V_{S30} = 290$ m/s.	54
Figure 28. Proxy-based V_{S30} map for Alaska after (Geyin & Maurer, (2022)). Most locations fall within site classes D ($180 < V_{S30} < 360$ m/s) and C ($360 < V_{S30} < 760$ m/s).	59
Figure 29. The location of measured and estimated V_{S30} (adapted from Thornley et al., 2021)..	60
Figure 30. The intention of V_S - $\Delta\kappa_0$ corrections with a candidate ground motion nearby the site (bridge) of interest.....	62
Figure 31. Framework of V_S - $\Delta\kappa_0$ correction (after Cabas & Rodriguez-Marek, 2017)	62
Figure 32. Flowchart of the proposed methodology.....	67
Figure 33. (a) Displacement profile and (b) Displaced shape related to the example stated above	70
Figure 34. Flowchart to calculate total base shear and system displacement of a specific displacement profile for an MDOF bridge system.	72
Figure 35. Flowchart of the procedure to obtain the performance spectra	74
Figure 36. Framework to estimate the seismic demand incorporating the site effects	76
Figure 37. Proposed GMM selection workflow for Alaska. Note that the NGA-West2 GMMs can also be used in case of shallow crustal regimes. The GMMs shown in that branch of the workflow correspond to available GMM that directly provide values of SD instead of PSA. Conversions to SD using the natural angular frequency could be used in conjunction with PSA values from NGA-West2 models as well.....	77
Figure 38. Flowchart of the procedure to obtain the variable demand spectra.....	81
Figure 39. Illustration of the process for combining performance spectra and variable demand spectra to determine the final system displacement, representing the structure's response to the specified seismic demand.	82
Figure 40. Elevation view of Bridges 610 and 646. (Courtesy of AKDOT&PF)	85
Figure 41. Elevation view of typical pier section of Bridge 610 and 646. (Courtesy of AKDOT&PF).....	85
Figure 42. Force-displacement relationship of the abutments	86
Figure 43. Moment-curvature relationship for bridges 610 and 646.....	87
Figure 44. Force-displacement relationship for bridges 610 and 646.	88
Figure 45. Locations of the earthquake epicenter and selected bridges for these example application of the proposed rapid assessment framework.	90
Figure 46. Pushover Curve of Bridge 610 and 646 with the four performance limit states for each pier column.	93
Figure 47. Displaced Shape Progression for Bridge 610 including first and last interaction values.	94

Figure 48. Displaced Shape Progression for Bridge 646 including first and last interaction values.	94
Figure 49. Performance Spectra of Bridge 610 and 646 with the four performance limit states for each pier column.	97
Figure 50. The location of selected stations for the ground motion scaling	99
Figure 51. 5% damped elastic displacement spectra for locations of bridge 610 and 646.	99
Figure 52. Variable Demand Spectra and 5%-damped spectra for both 610 and 646 bridges.	100
Figure 53. Flowchart illustrating the procedure for calculating a single point on the Variable Demand Spectra corresponding to a given effective period.	102
Figure 54. Bridge 610 final performance with the final system displacement and effective period identified.	103
Figure 55. Bridge 646 final performance with the final system displacement and effective period identified	104
Figure 56. Cumulative Distribution Function from the performance limit states curvatures for Bridge 610.	109
Figure 57. Cumulative Distribution Function from the performance limit states curvatures for Bridge 646.	109
Figure 58. Cumulative Distribution Function from the performance limit states displacements for Bridge 610.	112
Figure 59. Cumulative Distribution Function from the performance limit states displacements for Bridge 646.	112
Figure 60. Rapid Assessment Application Flowchart.	126
Figure 61. The location of the selected earthquake event and bridges	134
Figure 62. Elevation view of generic pier with the representation of parameters used in the MATLAB code	211
Figure 63. Command Window output	215
Figure 64. Visual summary of DDBA steps provided by the DDBA MATLAB code.	217

ABBREVIATIONS

AB07	Ground Motion Model proposed by Akkar and Bommer (2007)
ALR	Axial Load Ratio
CB08	Ground Motion Model proposed by Campbell and Bozorgnia (2008)
CIIM	USGS Community Internet Intensity Map
CSUS	Central and Southeastern United States
DDBA	Direct Displacement-Based Assessment
DDBD	Direct Displacement-Based Design
DYFI	"Did You Feel It?" Platform
EDP	Engineering Demand Parameter
EHS	Elastic Halfspace
EMS	Effective Mode Shape
F04	Ground Motion Model proposed by Faccioli et al. (2004)
FEMA	U.S. Federal Emergency Management Agency
GM	Ground Motion
GMICE	Ground Motion Intensity Conversion Equations
GMM	Ground Motion Model
GMRotI50	Geometric Mean Horizontal Component
HVSR	Horizontal-to-Vertical Spectral Ratio
IDA	Incremental Dynamic Analysis
IEA	Iterative Eigenvalue Analysis
IM	Ground Motion Intensity Measure
IPE	Intensity Prediction Equation
KBCG20	Ground Motion Model proposed by Kuehn et al. (2020)
MDOF	Multi Degree of Freedom
ML	Machine Learning
MMI	Mercalli Modified Intensity
NCSU	North Carolina State University
NDOT	Nevada Department of Transportation
NIBS	National Institute of Building Sciences
NLTHA	Nonlinear Time History Analysis
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PSA	Pseudo-Spectral Acceleration
PSBAH20	Ground Motion Model proposed by Parker et al. (2020)
PSDM	Probabilistic Seismic Demand Model
QWL	Quarter Wavelength Method
RC	Reinforced Concrete

RCFST	Reinforced Concrete Filled Steel Tube
SD	Spectral Displacement
SDOF	Single Degree of Freedom
SRA	Site Response Analysis
SRI	Square-root-impedance Method
TF	Transfer Function
USGS	United States Geological Survey
VDS	Variable Demand Spectra
WSDOT	Washington State Department of Transportation

LIST OF SYMBOLS

A_{1100}	Median estimate of PGA on a reference rock outcrop ($V_{S30} = 1100$ m/s)
A_g	Gross area of the column cross-section
A_{st}	Longitudinal reinforcement area
$b_1 - b_{10}$	Coefficients depending on the damping ratio
d_{bl}	Diameter of longitudinal reinforcement
D_c	Diameter of column
d_{max}	Peak ground displacement for Faccioli et al. (2004)
C_{atn}	Attenuation correction factor
C_{imp}	Impedance correction factor
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
f	Frequency
F	Total base shear distributed to the inertia mass locations
f'_{cc}	Compression strength of the confined concrete
F_N and F_P	Normal and reverse faulting earthquake
f_y	Longitudinal reinforcing steel yield strength
f_{ye}	Expected longitudinal steel yield strength
f_{yh}	Transverse steel yield strength
f_{yhe}	Expected transverse steel yield strength
f_u	Ultimate strength of reinforcing steel
H	Column height
I_y	Superstructure flexural inertia – strong axis transverse section
K_{eff}	Effective stiffness
K_{sec}	Secant stiffness
L_p	Analytical plastic hinge length
L_{sp}	Strain penetration length
m_i	Inertial mass associated with each i_{th} location
m_{eff}	Effective mass
M_w	Moment magnitude
n	Number of inertial mass locations
P	Axial load acting at the section centroid
Q	Quality factor
r	Hypocentral distance
R_ξ	Spectral reduction factor
R_{jb}	Joyner-Boore distance
R_{rup}	Rupture distance

s	Spacing of transverse steel
S_d	Spectral displacement
S_{dv}	Scaled spectral displacement
S_s and S_A	Binary variables taking values of 1 for soft and stiff soil sites (otherwise, 0)
T_c	Period when the maximum displacement is reached
T_{eff}	Effective period
$\bar{V}$	V_s over a given depth
V_b	Total base shear
V_i	Force in the columns and abutments
V_R	V_s at the reference depth
V_S	Shear wave velocity
V_{S30}	Time-averaged shear-wave velocity in the top 30 m deposit
$\bar{Z}$	Average seismic impedance
$Z_{2.5}$	Depth to the 2.5 km/s shear-wave velocity horizon
Z_R	Seismic impedance at the reference depth
Z_{TOR}	Depth to the top of the coseismic rupture plane

Greek Letters

γ	Proportional constant depending on the soil condition
δ_i	Displacement of each pier
δ_{crit}	Higher displacement in a displacement profile
δ_{n_i}	Normalized displacement
$\Delta_{\kappa 0}$	Difference between the surface and the base rock
Δ_{σ}	Stress drop
Δ_{ls}	Limit state displacement
Δ_y	Equivalent yield displacement
Δ_{sys}	System displacement
$\Delta_{sys_{rev}}$	Revised system displacement
ε_{su}	Maximum reinforcement strain
κ	Decay parameter
$\kappa_{0,surf}$	κ_0 at surface
$\kappa_{0,rock}$	κ_0 at the base rock
μ_{Δ}	Displacement ductility
ξ_{eq}	Equivalent viscous damping
ξ_{min}	Small strain damping
ξ_{sys}	System equivalent viscous damping
$\bar{\rho}$	Average density over a given depth
ρ_R	Density at the reference depth
ρ_v	Transverse volumetric steel ratio
ϕ_{ls}	Limit state curvature
ϕ_y	Equivalent yield curvature

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EXECUTIVE SUMMARY

This report presents the results of Phase I of a research project focusing on developing a rapid and reliable method for assessing the post-earthquake performance of multi-span bridges. Prompt structural evaluations are critical in the aftermath of a strong earthquake, particularly in regions where bridges function as lifelines for emergency response. Traditional assessment methods tend to fall at two extremes: fast but with greater uncertainty, or higher potential for accuracy at the expense of time- and resource-intensive nonlinear analyses. This research addresses the need for an approach that balances speed, robustness, and practicality for real-world implementation.

The proposed method is based on the Direct Displacement-Based Assessment (DDBA) framework, which simplifies complex structural systems into equivalent single-degree-of-freedom models and relies on spectral displacement as the intensity measure. This choice enables more meaningful correlations with structural damage than acceleration-based intensity measures typically used in existing tools.

Phase I of this research, which is presented in this report, had four primary objectives: (1) to develop the DDBA approach for rapid post-earthquake evaluation of reinforced concrete bridges; (2) to quantify uncertainty in structural performance limit states due to material and detailing variability; (3) to define representative seismic demands using existing ground-motion models applicable to Alaska's tectonic settings; (4) to outline a roadmap for developing a software tool that integrates structural and site data for automated assessment.

The framework was applied to a set of bridges from the AKDOT&PF inventory, demonstrating its utility in predicting displacement demands, estimating performance limit states, and supporting

inspection prioritization. A sensitivity analysis further explored how variability in structural properties affects performance thresholds, offering insights for future probabilistic applications.

The results of this phase lay the groundwork for Phase II, which will involve the development of a user-oriented software platform to support both real-time assessments following earthquakes and scenario-based planning. This tool will enable transportation agencies to make informed, timely decisions on bridge safety and inspection strategies, enhancing post-event response capabilities and long-term infrastructure resilience.

CHAPTER 1: INTRODUCTION

1.1 Motivation

Immediately following a potentially damaging earthquake, it is crucial to accurately assess the predicted performance of affected structures to expedite decision-making and mitigate potential losses. In particular, bridges in highly seismic regions serve a significant post-earthquake role as lifelines, providing alternative routes for emergency vehicles. Hence, transportation agencies managing extensive bridge inventories are expected to rapidly assess the performance of multiple structures, ensuring public safety and accelerating the restoration of bridges. The prioritization of inspections becomes especially challenging in regions where bridges are sparsely distributed and often located far from major urban centers. Deploying engineers to all bridge sites for structural inspections can be time-consuming, often requiring multiple weeks. Thus, a rapid and reliable evaluation method is necessary to determine structural conditions and optimize resource allocation in the aftermath of a strong seismic event.

To assess post-earthquake structural performance, various evaluation methodologies have been developed, each balancing accuracy, efficiency, and practicality. One approach involves characterizing bridge behavior through fragility functions, which provide a simplified yet efficient means of assessing multiple structures with similar characteristics. This method typically relies on high-level input parameters, enabling a more rapid assessment compared to more detailed analytical approaches. However, while the expedited nature of this process facilitates timely decision-making, it comes at the cost of greater uncertainty and reduced specificity of damage. Additionally, these tools typically rely on ground-motion intensity measures (IMs) based on acceleration, such as Peak Ground Acceleration (PGA) and Pseudo-Spectral Acceleration (PSA). However, PGA has been identified as

the IM with the lowest correlation to peak tensile strain (Thangjitham, 2023). Therefore, since material strain is a reliable indicator of damage, we can infer that the relationship between acceleration-based IMs and structural damage is questionable. Notable examples of such approaches include ShakeCast, developed by USGS, and HAZUS, developed by FEMA, both of which help prioritize inspections on a regional scale.

At the other end of the spectrum are time-domain analyses, such as nonlinear time history analysis (NLTHA), which capture the structure's response to seismic excitations as a function of time. Unlike simplified methods, NLTHA explicitly accounts for material and geometric nonlinearities, enabling the simulation of effects such as yielding, cracking, and stiffness degradation. While these methods offer the potential for high damage specificity and reduced uncertainty, they are computationally demanding, time-intensive, highly sensitive to modeling assumptions, and require extensive input information regarding the structure and the seismic demand. Improper application can lead to erroneous outcomes, potentially compromising the reliability of post-earthquake assessments.

To address the limitations of the previously discussed assessment methodologies, this study proposes an alternative technique for the rapid post-earthquake evaluation of bridges. The proposed approach is based on the Direct Displacement-Based Assessment (DDBA) framework, which facilitates structural performance evaluation by representing an inelastic multi-degree-of-freedom (MDOF) system as an equivalent single-degree-of-freedom (SDOF) system. Moreover, the approach relies on spectral displacement as an IM, since it is recognized as a more robust and reliable predictor of seismic performance. By leveraging these principles, DDBA aims to enhance both the efficiency and uncertainty of post-earthquake assessments. The method seeks to bridge the gap between the computational precision of NLTHA and the practicality of fragility-functions-based approaches, ensuring that its predictive capabilities remain as robust as possible within the limitation posed by

equivalent linearization techniques. Additionally, the framework is designed to provide results that are both technically rigorous and accessible to practitioners.

A robust characterization of the ground-motion is a key aspect of post-earthquake assessment. Earthquake ground-motion characteristics are influenced by the seismic source, and the properties of the Earth's crust through which seismic waves propagate. Near-surface geologic materials can exert a particularly significant influence on the main characteristic of a ground-motion, namely amplitude, frequency content and duration (Borcherdt, 1994; Kramer, 1996; Seed & Idriss, 1971). This phenomenon is known as site effects, and past earthquakes have shown that it can exacerbate damage to the built environment. "Because earthquake-induced ground motions are a complex seismic signal, it is often desirable to describe them through parameters, or summary statistics, referred to as ground-motion intensity measures" (Baker et al., 2021).

Portions of Alaska have a relatively dense network of strong ground-motion recording stations, even supplemented by instrumented structures and vertical arrays (e.g., Anchorage). However, most of the state's critical infrastructure is not located nearby a seismic recording station. Moreover, Alaska is in a complex and highly active tectonic environment (i.e., shallow crustal and subduction events) with unique geologic structures that greatly influence the spatial variability of ground motions in the region. Thus, the characterization of ground-motion in the context of post-earthquake evaluations in Alaska requires development of a framework to identify alternative pathways based on the available information about the event and the site of interest.

Different intensity measures (IMs) can be helpful in capturing the characteristics of a ground-motion, from peak amplitude parameters like peak ground acceleration (PGA) to response spectral values or Fourier spectral values. In this work, we will focus on spectral displacements as indicators of ground-motion severity. In the absence of nearby ground-motion records after an event, semi-empirical

ground-motion models (GMMs) can be used to provide a characterization of the distribution of ground-motion intensity at locations of interest. The outcome of a GMM is not a ground-motion time series, but a ground-motion IM, such as pseudo-spectra acceleration or spectral displacements. USGS uses GMMs in combination with recorded ground-motion to generate ShakeMaps (Wald et al, 2003) after notable earthquakes in the US and around the world. However, the incorporation of site-specific effects is necessarily simplified in the generation of ShakeMaps. In this work, we explore different methodological frameworks to incorporate site effects on ground motions in Alaska as well as their uncertainty.

1.2 Objectives

The research presented in this report has four key objectives:

1. Develop a DDBA approach for the rapid post-earthquake evaluation of multi-span RC bridges.

This approach relies on new spectral representations of both structural capacity and seismic demand, which, when intersected, determine the bridge's performance under a given earthquake scenario. These representations provide a continuous depiction of structural behavior, enabling the rapid determination of bridge lateral displacement and the extent of structural damage following a seismic event.

2. Investigate the uncertainty associated with performance limit states for integration into a probabilistic framework. This study examines how variation in material properties and detailing (e.g., transverse bar spacing), influences limit states displacement of a single RC circular column within a bridge pier. The quantified uncertainties in the limit state displacements can be incorporated into the final step in the rapid post-earthquake assessment process, where the structural response is estimated and the extent of structural damage is predicted, to establish a reliable inspection priority list.

3. Establish efficient ground-motion characterization tailored to Alaska's seismicity and geology for rapid assessment. Sufficient ground-motion records are not available across the whole state to solely rely on nearby recordings after an earthquake to characterize the seismic demand at each bridge of interest. Regional GMMs to estimate spectral displacements in Alaska do not exist. Hence, existing GMMs are explored to estimate spectral displacements directly or indirectly depending on the tectonic setting (i.e. shallow crustal or subduction) and the available site metadata (i.e. the time-averaged shear wave velocity in the top 30 m, V_{S30}). Direct modifications of recorded ground-motion to account for differences in site effects between the recording station and the site of interest (e.g., Cabas & Rodriguez-Marek, 2017) are also evaluated.
4. Outline an implementation plan for Phase II, which will include the development of a software application designed for both post-evaluation and planning purposes. The software tool will be capable of deploying the DDBA methodology connected to a bridge database offering enough structural information as well as subsurface characterization information to run a deterministic assessment. The rapid assessment is initiated by the occurrence of a seismic event (or by the exploration of a hypothetical earthquake scenario) of engineering significance, providing results on structural performance.

1.3 Expected Outcomes

The anticipated outcomes of Phase I and the ongoing Phase II of this project will be directly applicable to bridge design practice, providing an immediately deployable tool for AKDOT&PF engineers. This research will allow engineers to efficiently and accurately estimate potential damage levels in multi-span bridges relying on robust IMs and detailed structural information. Specifically, in post-earthquake scenarios, it can assist in developing an inspection priority list, which is important for large states such as Alaska. Furthermore, this approach has broader applications in seismic planning,

enabling scenario simulations that would be able to inform decision-makers about the potential impacts of specific events on the bridge system so that vulnerabilities may be identified.

The other anticipated outcome for the ground-motion characterization is a better estimation of spectral displacements incorporating local site effects from Phase I and the ongoing Phase II. This report will provide a workflow for the GMM selection process to predict spectral displacements for different tectonic settings and the ability to capture site effects and the associated uncertainty. Furthermore, a modification of a framework for simple, first-order modifications to ground-motion records (to account for differences in V_s and attenuation properties after Cabas & Rodriguez-Marek, 2017) is proposed to estimate spectral displacements when ground-motion recordings are not available at (or nearby) bridge locations of interest.

1.4 Report Layout

Chapter 1 introduces the research, outlining the motivation behind the study. It also presents the research objectives and describes the overall organization of the report.

Chapter 2 presents a review of the most widely recognized post-earthquake assessment analysis engines. It also introduces the performance limit states referenced in this report, describing them in terms of the corresponding observed damage levels in a reinforced concrete column, along with the associated material strain values. Additionally, the chapter outlines the methods employed to convert strain measurements into member displacements. Existing GMMs and available site parameters are described with a focus on GMMs that directly provide spectral displacements and most recently developed models for subduction zones (i.e., the Next Generation Attenuation models, NGA-Sub (Bozorgnia et al., 2022)). Also, simple modifications to ground-motion records to account for site effects are also presented following the framework proposed by Cabas & Rodriguez-Marek (2017).

Chapter 3 outlines the proposed post-earthquake assessment method, detailing the seven steps involved in determining the final performance of a bridge under a seismic event. These steps include bridge data collection, subsurface data collection, estimation of the bridge pushover curve, generation of the bridge Performance Spectra, characterization of the seismic demand, calculation of Variable Demand Spectra, and estimation of the final displacement.

Chapter 4 presents a case study application of the proposed methodology using two bridges selected from the AKDOT&PF inventory: Bridge 610 and Bridge 646. The analysis incorporated key modeling assumptions and simplifications, including the use of scaled ground motions from the 2018 Anchorage Alaska earthquake, idealized abutment behavior and the treatment of reinforced concrete-filled steel tube (RCFST) columns as reinforced concrete (RC) columns, in line with the current scope and limitations of the study.

Chapter 5 investigates the uncertainty associated with the performance limit states defined in Chapter 2 by employing a deterministic approach that systematically varies key design parameters, including material properties and detailing aspects. The analysis considers a multi-column pier configuration to capture the influence of axial load redistribution under seismic overturning moments. Columns from Pier 1 of Bridge 610 and Pier 1 of Bridge 646 are used as representative case studies.

Chapter 6 summarizes the main conclusions of this report, which represents Phase I of the research project. It also outlines the objective and scope of the upcoming Phase II and provides a detailed implementation plan for the tool designed to support rapid post-earthquake assessment of bridges within the AKDOT&PF inventory.

Finally, the previous chapters are complemented with two appendices. Appendix A presents the results of the post-earthquake assessment conducted on nine AKDOT&PF bridges, as described in Chapter 4. The analysis includes specific assumptions and simplifications, such as modeling

abutments with elasto-plastic behavior and treating RCFST columns as RC columns with pile-supported footings, excluding the steel tube contribution. Each bridge has a dedicated section with detailed results, including structural database information, moment-curvature and force-displacement relationships, system pushover curves, displacement progression, Performance Spectra, Variable Demand Spectra, and the final performance limit state of each pier. Additionally, Appendix A provides an inspection priority list for the nine bridges. The methodology used follows the procedure outlined in Chapter 3 and is illustrated through examples in Chapter 4. Appendix B includes the user guide of the DDBA MATLAB Code provided.

CHAPTER 2: LITERATURE REVIEW

2.1 Post-Earthquake Assessment Analysis Engines

This section provides a comprehensive literature review of the most widely recognized seismic assessment analysis engines. The discussion begins with an overview of ShakeMap, a tool that defines seismic hazard parameters adopted in various assessment methodologies, including ShakeCast and HAZUS. Subsequently, the primary analysis engines are examined in detail, with a critical evaluation of their capabilities and inherent limitations.

ShakeMap

The capability to obtain more than just moment magnitude and epicenter location within minutes of an earthquake event was considered impractical approximately three decades ago. While these two parameters provided some level of insight, they were insufficient for conducting a reliable and rapid post-earthquake assessment of affected regions (Wald et al., 2003). Hence, influenced by the lack of further information, the TriNet Project – collaboration between the California Institute of Technology, the California Division of Mines and Geology, and the USGS – proposed a framework for generating “shake” maps for earthquakes of M3.0 or greater in Southern California. Following the successful implementation of this initiative, the project was expanded both nationally and globally, ultimately leading to the development of the ShakeMap system. ShakeMaps are rapid-response ground motion maps employed across a wide range of applications, including emergency response, loss estimation, and engineering analyses. The system provides various levels of information, making it accessible not only to specialists but also to the general public.

The ShakeMap system is a stand-alone software and a passive consumer of seismic data, implying that the system does not hold a data acquisition component (Worden et al., 2020). Instead, seismic

data recorded at monitoring stations are collected by third-party entities and subsequently transmitted to the ShakeMap system, whereupon the receipt of this information automatically triggers the generation of a ShakeMap. The threshold moment magnitude for triggering this process varies by region, ranging from M3.5 in the Western United States to M5.4 for global seismic events (Wald et al., 2022).

ShakeMap generation can be triggered by two primary sources: seismic recording stations, which provide ground motion data, and community-based observations submitted through the “Did You Feel It?” (DYFI) platform. DYFI, formally known as the USGS Community Internet Intensity Map (CIIM), is a worldwide web-based system designed to collect earthquake intensity reports from the general public. These reports, which characterize the Modified Mercalli Intensity (MMI) felt during the event, are spatially correlated with the respondents’ locations to enhance ShakeMap coverage. This approach enables the generation of ShakeMaps for earthquakes with magnitudes as low as M2.0. The integration of IMs from DYFI is particularly valuable in regions with limited seismic instrumentation, where such observations often serve as the primary or sole data source.

Furthermore, according to Worden et al. (2020), the parametric data collected from seismic stations supporting the ShakeMap system typically includes MMI, PGA, Peak Ground Velocity (PGV), and 5%-damped PSA at 0.3, 1.0, and 3.0 second periods. From these IMs, it is possible to estimate ground motion amplitudes using a suitable Ground Motion/Intensity Conversion Equation (GMICE). Worden et al. (2020) analyzed approximately 200,000 MMI and ground motion observations in California to develop probabilistic, reversible relationships between ground motion parameters and IMs. These relationships enable the derivation of ground motion data from MMI observations and vice versa, and are now used to generate ShakeMaps, increasing the robustness of the outputs. Figure

1 illustrates a typical intensity-based ShakeMap alongside the corresponding relationships between PGA, PGV, and MMI for a specific seismic event.

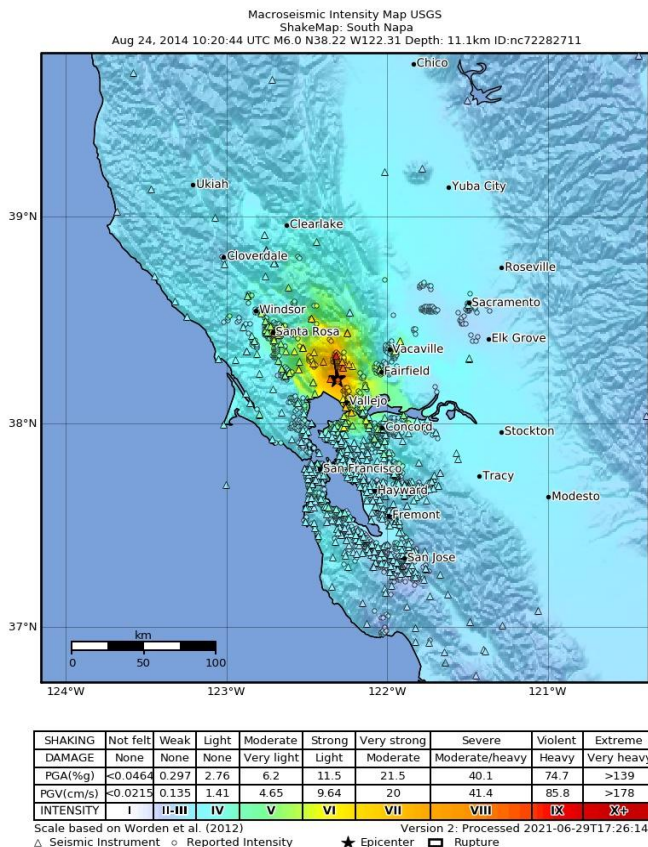


Figure 1. Intensity Map from 2014 M6.0 Napa Valley, CA earthquake generated by ShakeMap. (Source: Worden et al., 2020)

According to Worden et al. (2020), once all necessary data have been received, the ShakeMap processing system can estimate ground motion at any location within the grid through interpolation. This capability is particularly critical in regions with low seismic station density, where significant gaps in recorded shaking data may exist. ShakeMap performs these estimations by computing a weighted average of three distinct data inputs. The first input consists of direct observations of ground motion or intensity, obtained either from seismic instrumentation or community-reported DYFI data. The second input comprises converted observations, where one type of measurement is transformed

into another using GMICEs (e.g., $MMI=f(PGV)$ or $PGA=g(MMI)$). The third input consists of estimated values derived from GMMs and Intensity Prediction Equations (IPEs).

To generate the final ground motion maps, ShakeMap first removes the effects of near-surface amplification from all three data sources and standardized the dataset to represent shaking condition at bedrock (Worden & Wald, 2016). The system then interpolated the processed data onto a uniform computational grid. Finally, site-specific amplifications are applied to each grid point based on the V_{s30} , as provided by the system operator. A schematic representation of this interpolation process is illustrated in Figure 2.

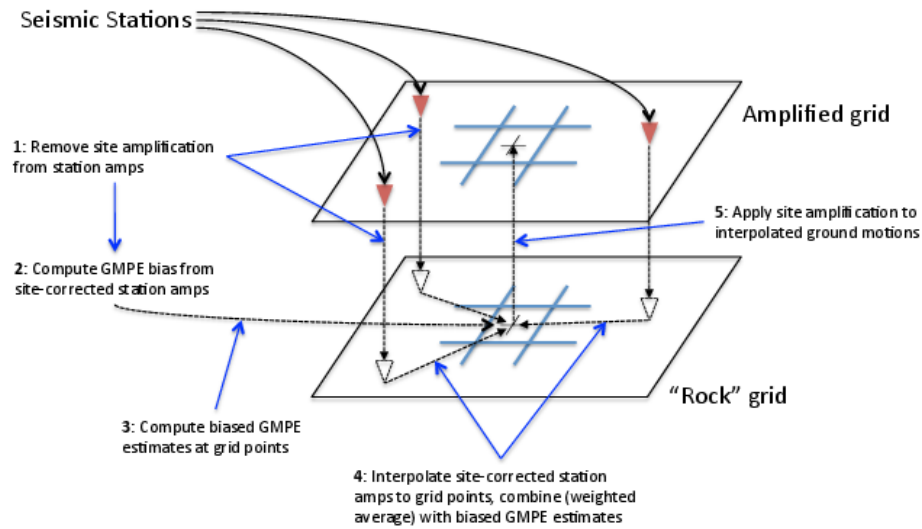


Figure 2. A schematic of the basic ShakeMap ground motion interpolation scheme (Source: Worden & Wald, 2016)

Once data processing is complete, ShakeMap can rapidly generate regression plots (how PGA, PGV, and PSA change with earthquake magnitude and distance), seismic station records (MMI, PGA, PGV, and epicentral distance), and grid-based or contour representations of instrumental intensity, PGA, PGV, and PSA. The peak values plotted correspond to the maximum recorded values from the two horizontal motion components. However, with appropriate input data and configuration settings,

ShakeMap can generate maps incorporating various IMs components, such as rotation-independent measures like RotD50 (the 50th percentile of response spectra) according to (Worden et al., 2020).

Regarding the timing of ShakeMap generation, in most regions of the United States, the system can determine the epicenter location typically within 5 to 10 minutes following a seismic event and generate the initial ShakeMap shortly thereafter. Globally, ShakeMap production can take up to 15 minutes. However, as additional data continues to be received over the subsequent hours, updated ShakeMaps with improved accuracy are subsequently generated.

As previously discussed, ShakeMaps serves multiple applications in post-earthquake assessments. Existing assessment methodologies, such as FEMA's HAZUS and ShakeCast, continue to benefit from the increasing accuracy and efficiency of ShakeMaps. These systems will be examined in greater detail later in this chapter.

2.1.1 HAZUS

Despite advancements in earthquake characterization, predicting the specific consequences from ground motions remains a significant challenge for the scientific community. To address this issue, in 1997, the National Institute of Building Sciences (NIBS) and the U.S. Federal Emergency Management Agency (FEMA) developed a software tool designed to support the development of strategies for mitigating earthquake hazards. Initially named Hazard U.S., the tool focused exclusively on earthquake modeling. However, in 2004, an expanded version, HAZUS Multi-Hazard (HAZUS-MH), was released, incorporating additional hazard models for tsunamis, floods, and hurricanes. HAZUS-MH is a standardized risk modeling methodology capable of estimating hazard-induced losses and is currently employed to support both pre- and post-event decision-making processes across the United States.

The Hazus Earthquake Estimation Methodology (hereafter referred to as HAZUS) is designed to quantify and spatially map the risk of physical damage, economic loss, and social impacts associated with seismic events. The HAZUS risk assessment process consists of four key steps: (1) identifying hazards, (2) evaluating the inventory of exposed assets, (3) estimating potential losses, and (4) assessing mitigation strategies. In pre-event applications, the output generated by HAZUS analyses are used to develop contingency plans aimed at reducing the severity of earthquake impacts. The system runs multiple earthquake scenarios to enhance the accuracy of predictive analyses.

In a post-event context, HAZUS serves as a critical tool for the rapid assessment of earthquake-induced losses, facilitating efficient emergency response and recovery operations. The system's estimations include damage assessments for buildings and infrastructure, functional disruptions to critical facilities, and projections of economic losses and casualties. To optimize the prediction of damage and losses under both pre- and post-earthquake conditions, HAZUS integrates a nationwide inventory of buildings, facilities, transportation networks, and utility systems with earthquake hazard data. This integration enables the system to establish relationships between IMs and infrastructure damage, as described by Remo & Pinter (2012). Utilizing this information, HAZUS can project the spatial and structural extent of disaster impacts. A schematic representation of HAZUS inputs and outputs is presented in Figure 3.

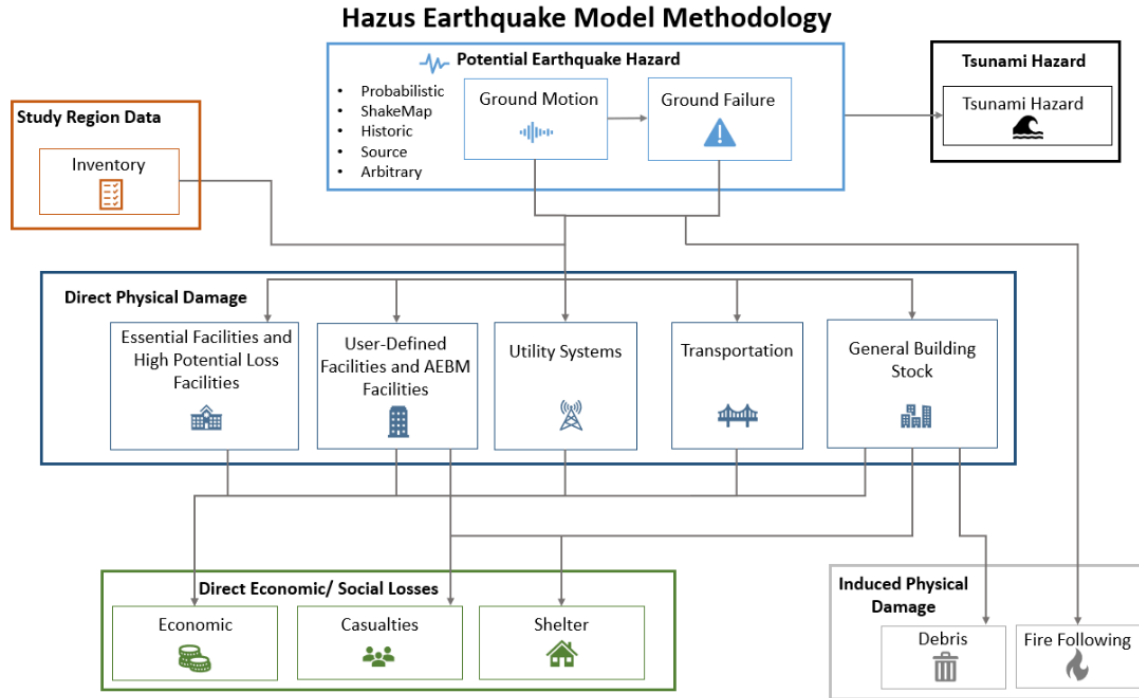


Figure 3. Hazus Earthquake Model Methodology Schematic (Source: FEMA, 2022)

HAZUS is recognized for its documentation on the classification of buildings and bridges, along with corresponding seismic vulnerability assessments, as noted by Mangalathu et al. (2017). The bridge classification system employed by HAZUS is based on the National Bridge Inventory (NBI), which is a standard database maintained by the Federal Highway Administration (FHWA) that summarizes metadata from over 600,000 highway bridges in the United States. According to FEMA (2022), the classification of bridges within HAZUS is determined by multiple parameters, including (1) the seismic design code in effect at the time of construction, (2) the number of spans, (3) structural type, (4) pier type, (5) abutment type, (6) bearing type, and (7) span continuity. Additionally, the length of the longest span is considered in the classification process. Based on these parameters, HAZUS categorizes bridges into 28 distinct classes, facilitating more accurate seismic vulnerability assessments.

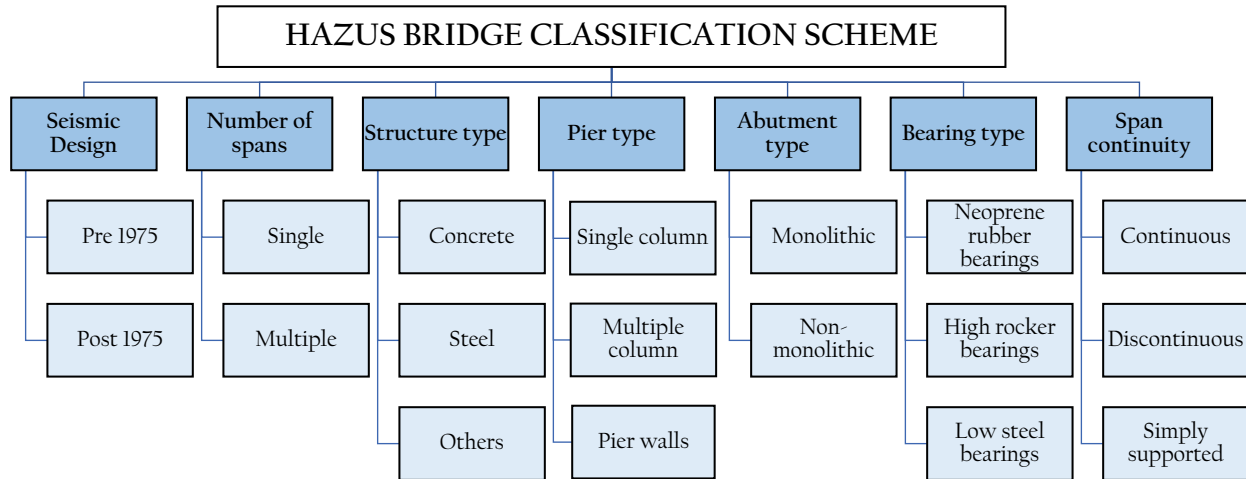


Figure 4. HAZUS bridge classification scheme.

For each bridge class, HAZUS provides associated fragility curves which are lognormal probability functions that represent the likelihood of a structure reaching or exceeding specific damage states for a given peak response – within HAZUS, this parameter is the PSA at 1.0 seconds. The fragility curves incorporated in HAZUS were initially proposed by Mander & Basöz (1999), who developed these relationships for several common “standard bridge” types in the United States. These curves were derived using structural attributes that include the number of spans, structural type, year of construction, skew, and bridge length. The proposed fragility curves were validated against observed damage data from the 1994 Northridge and 1989 Loma Prieta earthquakes.

In Figure 5, it is possible to observe the characteristics of a fragility curve. According to Kircher et al. (2006), these lognormal functions consider the variability associated with the capacity curve properties, damage states, and ground shaking. HAZUS works with the following damage states: slight, moderate, extensive, and complete. The damage states are associated with performance outcomes, such as the level of structural damage and type of repairs required.

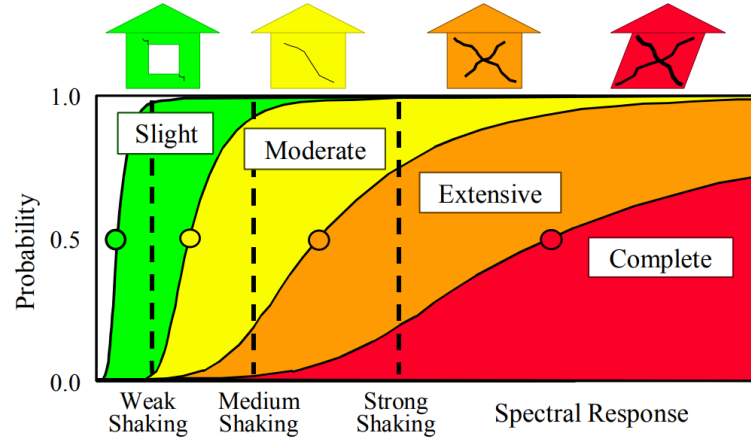


Figure 5. Example of fragility curves (the use of the terms weak, medium, and strong shaking is applied here for simplicity, in the actual curves, quantitative values of spectral response are used). (Source: FEMA, 2001)

Rather than storing the complete fragility curve, HAZUS provides, for each curve, the ground motion intensity corresponding to a 50% probability of reaching or exceeding a specific damage state. These values are represented in Figure 5 by the circular markers on each curve. In addition to the median value, HAZUS also includes the associated dispersion factor (standard deviation), as presented in Table 1. Regarding bridge classifications, HWB20 refers to prestressed concrete bridges with a single-column bent and a box girder (conventional design); HWB21 represents prestressed concrete bridges with a single-column bent and a box girder (seismic design). HWB28 encompasses all remaining bridge types that do not fall within the predefined categories by the HAZUS classification, including wooden bridges.

Table 1. HAZUS grouping and fragility relationships for three bridge classes (Modified after FEMA, 2022)

Class	PSA (1.0 sec) for Damage Functions due to Ground Shaking (g)				
	Slight	Moderate	Extensive	Complete	Dispersion
HWB20	0.35	0.45	0.55	0.80	0.60
HWB21	0.60	0.90	1.30	1.60	0.60
HWB28	0.80	1.00	1.20	1.70	0.60

Regarding ground motion data, HAZUS employs ShakeMap as a deterministic source for both real and scenario-based earthquakes. According to Wald et al. (2020), advancements in ShakeMap ground shaking data have enhanced HAZUS' ability to incorporate the most up-to-date ShakeMap grids of peak ground motion, thereby improving the reliability of pre-event predictions and facilitating rapid post-event assessments.

As outlined by FEMA (2022), HAZUS was designed to accommodate both basic and advanced analyses, categorized into three levels of data refinement. The default analysis level relies on regional earthquake hazard models, predefined inventory data, and general damage estimations. However, more precise results can be achieved by integrating high-resolution data, including a detailed inventory of essential infrastructures and region-specific ground motion parameters with site amplification considerations.

Ranf et al. (2007) conducted an evaluation of HAZUS' default fragility curves for slight damage, comparing them to bridge damage reports from the 2001 Nisqually earthquake. The study combined ShakeMap-derived ground motion data with the WSDOT bridge inventory and HAZUS fragility functions. As depicted in Figure 6, HAZUS overestimated the number of simply supported bridges sustaining damage, estimating 149 affected structures compared to 22 observed in post-earthquake assessments. Conversely, for the category encompassing all the other bridge types not classified within the 27 predefined categories, HAZUS underestimated the number of damaged bridges by 425%.

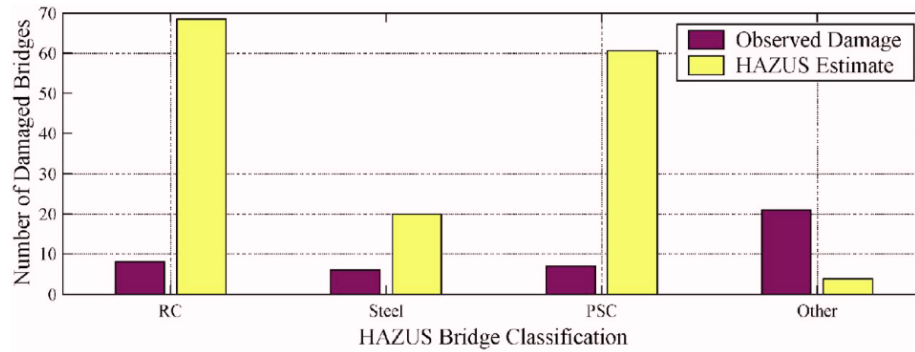


Figure 6. Observed and estimated number of simply supported bridges. (Source: Ranf et al. ,2007)

Mangalathu et al. (2017) proposed enhancements to the HAZUS bridge classification system through case studies of box-girder bridges in California. In addition to incorporating modifications suggested by previous researchers, the study examined the influence of the number of spans (two, three, and four spans) on structural behavior. The analysis focused on bridges with the following characteristics: pre-1971 design era, seat-type abutments, two-column bents, and circular column cross-sections.

In HAZUS, these bridge types are collectively classified under a single category (multiple spans). However, the study revealed that two-span bridges exhibit greater vulnerability compared to three- and four-span configurations. The median fragility curve values for three- and four-span bridges were found to be 10-15% higher than those of two-span bridges, indicating greater seismic resilience. Figure 7 illustrates the system and component fragilities, highlighting the distinctions in vulnerability between two- and three- span bridges.

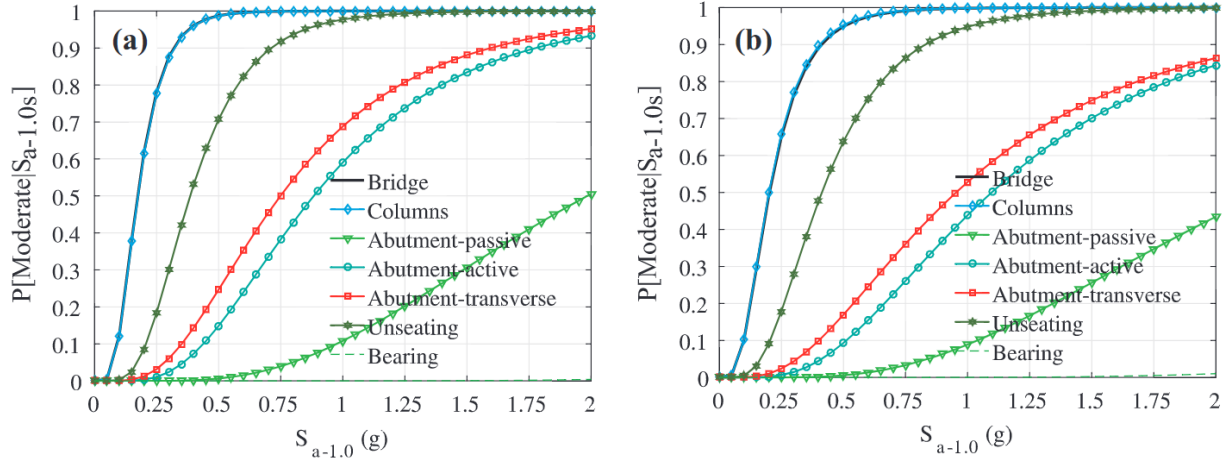


Figure 7. System and component fragilities for selected bridge classes for moderate damage state (a) two-span and (b) three span. (Source: Mangalathu et al., 2017)

On the other hand, the difference in the median fragility value between three-span and four-span bridges was approximately 3%, indicating that their fragility curves are statistically similar. As a result, rather than categorizing bridges simply as single-span or multiple-span structures, the authors proposed a refined classification system that distinguishes between one-span, two-span, and more-than-two-span bridges. Figure 8 illustrates the proposed classification scheme.

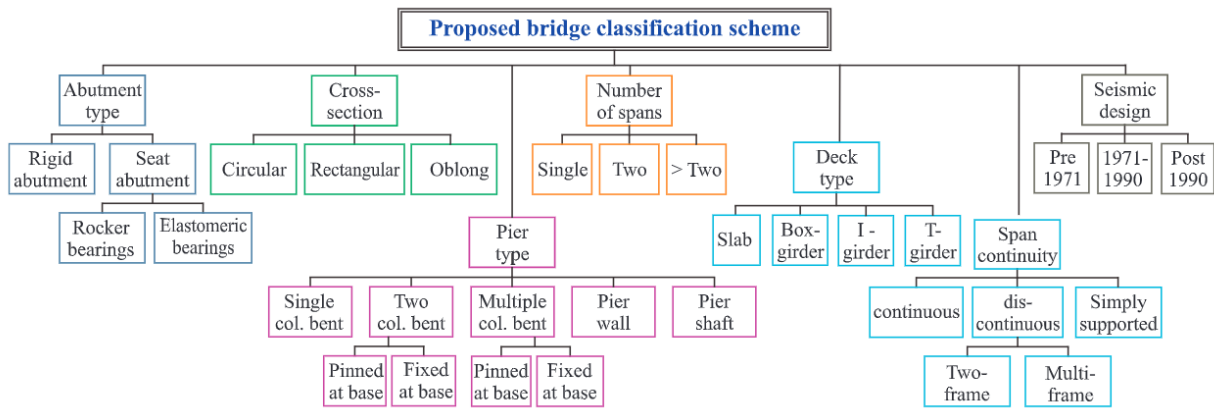


Figure 8. Proposed bridge classification scheme (Source: Mangalathu et al., 2017)

2.1.2 *ShakeCast*

Recognizing the need for a web-based application capable of automatically integrating with ShakeMap to expedite post-earthquake assessment, the USGS developed the first version of ShakeCast (ShakeMap Broadcast) in 2003. ShakeCast is a freely available post-event software analysis tool widely used by transportation agencies, utilities, and other large organizations. According to Biasi et al. (2017), ShakeCast automatically retrieves ground motion data from ShakeMap, comparing it against predefined capacity estimates of the user's facilities (e.g., bridges and buildings), and provides notifications regarding potential damage assessment within its inventory. In addition to real-time seismic events, ShakeCast can also generate and deliver assessments for scenario-based earthquakes.

Since ShakeCast relies exclusively on the ground motion data produced by ShakeMaps, its accuracy is directly influenced by the density of seismic station coverage. As noted by Lin & Wald (2008), the tool yields more precise results in regions with extensive seismic monitoring networks, such as California, Washington, Oregon, Nevada, Utah, and Alaska.

Organizations utilizing ShakeCast assign personnel to specific roles: “users,” who have access to personal profiles and can update contact and notification preferences, and “administrators,” who manage all ShakeCast functionalities. To initiate ShakeCast operations, the administrator must input the organization's facility inventory into the system. For each facility, in addition to its name, description, and location, the user must define its structural capacity through a fragility function.

For bridges, the administrator can select a predefined bridge classification to automatically assign standard fragility settings. These default fragility functions are based on the bridge damage models developed by Mander & Basöz (1999) and are categorized according to the NBI and the HAZUS classification system. Alternatively, rather than selecting a predefined classification, administrators

can manually adjust the fragility function parameters for each facility within the ShakeCast system. Figure 9 illustrates the ShakeCast administrator interface, where users can modify fragility metrics, as well as set lower and upper thresholds for each damage level.

Damage Level	Low Limit	High Limit	Metric
Damage Unlikely or Slightly	0	25	Peak Ground Acceleration (%g)
Moderate Damage Possible	25	71	Peak Ground Acceleration (%g)
Extensive Damage Possible	71	155	Peak Ground Acceleration (%g)
Complete Damage Possible	155	99999	Peak Ground Acceleration (%g)

Figure 9. Facility type editing page of the ShakeCast administrator interface. (Source: Turner et al., 2009)

Regarding ground motion metrics, ShakeCast utilizes one of the IMs provided by ShakeMap, including the MMI, PGV, PGA, and PSA at 0.3, 1.0, and 3.0 seconds. As noted by Turner et al. (2009), similar to HAZUS, ShakeCast does not store complete fragility functions. Instead, it retains only the parameters corresponding to the 50% probability of exceedance value. The number of damage levels is predefined but can be modified by the administrator. By default, these levels align with those used in HAZUS: slight, moderate, extensive, and complete damage.

ShakeCast generates hierarchical lists and maps of structures and facilities that are most likely to be affected by an earthquake by integrating ShakeMap ground motion data with the fragility functions of the inventory. The system prioritizes these structures strictly based on predicted damage levels,

arranging them in descending order of inspection priority. Figure 10 presents an example of ShakeCast output, illustrating its application to an area near Chino Hills, which experienced the M6.7 Chino Hills earthquake in July 2008. The figure displays bridges ranked by priority including their locations and associated intensity measures. Additionally, the accompanying map enables users to identify bridge locations and access relevant structural information.

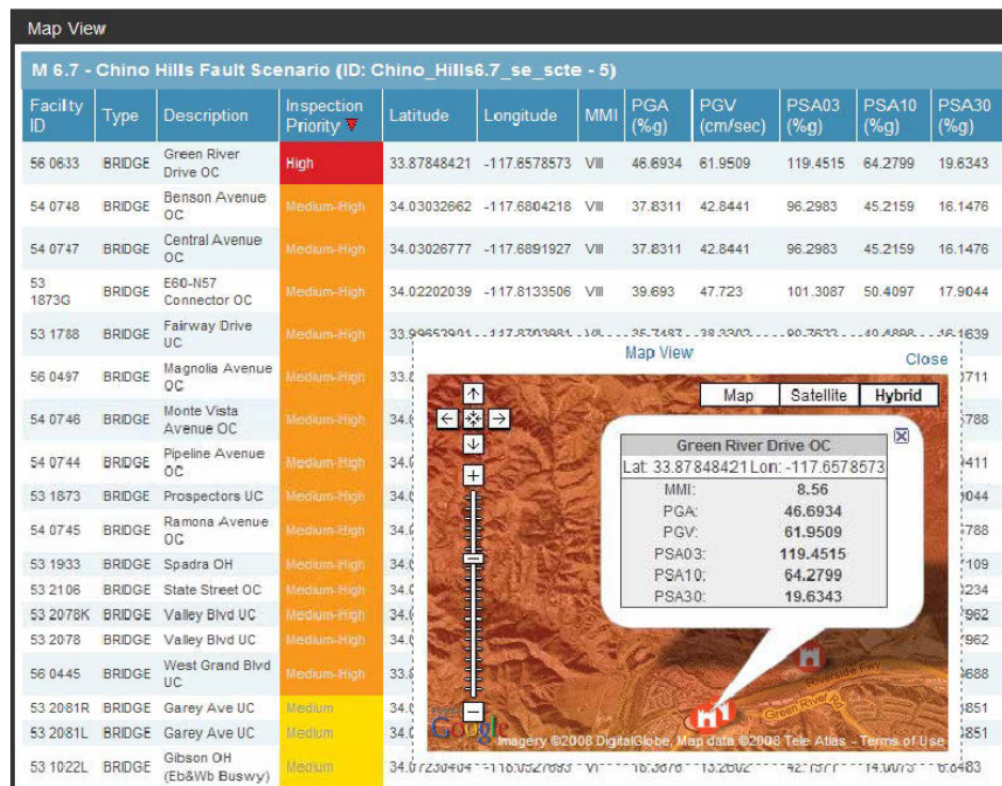


Figure 10. ShakeCast web interface for bridge inspection priority list. (Source: modified from Turner et al., 2009)

Lin & Wald (2008) indicate that users can configure damage state thresholds for each facility and designate a profile that will be automatically notified based on the assigned inspection priority level. Typically, notifications are sent via email or message to inspectors, operators, and other personnel responsible for facility management within their organization. According to Turner et al. (2009),

ShakeCast is capable of distributing these notifications within 15 minutes of an earthquake. Figure 11 presents a flowchart illustrating the operational process of ShakeCast.

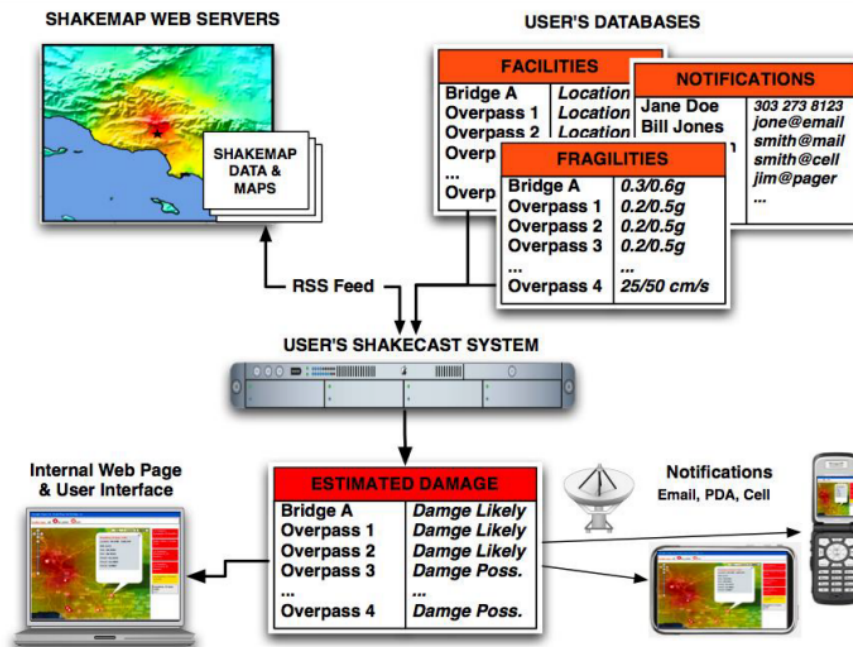


Figure 11. ShakeCast flowchart indicating ShakeMap data, user's inventory, and notifications. (Source: Lin & Wald, 2008)

Several studies have examined the integration of ShakeCast with state Department of Transportation (DOT) bridge inventories, including those of Caltrans (California) and NDOT (Nevada), to assess the system's accuracy and refine fragility functions for bridges. (Turner et al., 2009, Mohammed et al., 2016, Biasi et al., 2017). The following sections provide further details on the implementation of ShakeCast within these DOTs.

California Department of Transportation (Caltrans)

Wald et al. (2008) identified Caltrans as a key testbed and significant user of the ShakeCast system. However, as previously mentioned, the fragility functions derived from HAZUS and utilized in ShakeCast have several limitations. Notably, they do not account for variations in bridge geometric parameters such as the number of spans, span length, column height, bridge curvature and skew, and

material parameters, such as concrete compressive strength, reinforcing steel yield strength, and soil stiffness. To address these limitations, researchers have developed more representative fragility functions tailored to Caltrans bridges, which may be integrated into the ShakeCast framework (DesRoches et al., 2012; K. Ramanathan et al., 2015; Soleimani, 2017; Zheng, 2021). These advanced fragility functions, termed “Generation-2 Fragility” (g2F) models, are based on the California inventory and provide a more comprehensive assessment of structural vulnerability.

Unlike HAZUS fragility functions, which primarily consider column behavior, g2F models account for multiple bridge components, including abutment seat and shear key displacement, foundation translation and rotation, and deck displacement. According to Ramanathan et al. (2015), bridge components are classified into primary and secondary categories, each with its own fragility curve. Primary components, such as columns and abutments, are those whose extensive or complete damage would necessitate bridge closure. In contrast, damage to secondary components does not directly lead to closure. While component-specific fragility functions enhance the understanding of bridge performance, a comprehensive system-level fragility function is essential for practical implementation. According to Lin & Wald (2012), each individual component behavior is aggregated into a single function, which serves as a basis for prioritizing post-earthquake inspections.

Ramanathan (2012) was among the researchers who extensively contributed to the development of g2F models. The study focused on multi-span continuous concrete box girder (MSCC-BG) bridges, which constitute approximately 37% of California’s bridge inventory. Figure 12 presents a comparison between Bridge System State 0 (BSST-0), representing minor damage, and Bridge System State 3 (BSST-3), indicating complete damage, highlighting the contribution of individual component fragilities to the overall system fragility. Notably, at the initial damage state, columns are not the critical structural component, contrary to the assumptions made by HAZUS. However, at the

complete damage state, column fragility becomes the primary factor influencing the overall structural response of the bridge system.

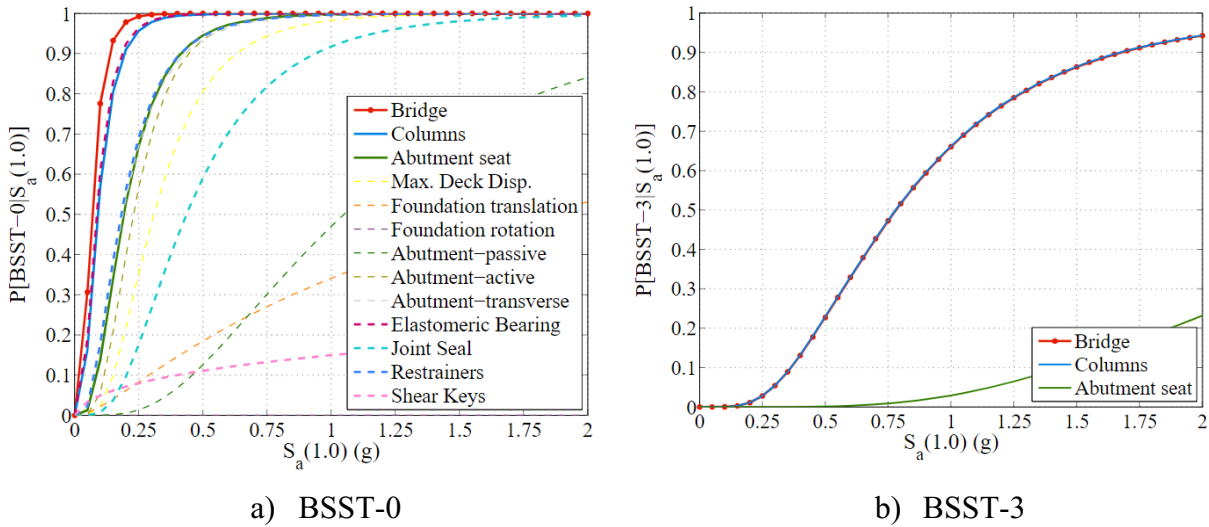


Figure 12. System and component level fragility curves for post 1990 MSCC-BG bridges with multi-column bent equipped with modular joint seal assembly system for BSST-0 and BSST-3. (Source: (K. N. Ramanathan, 2012))

Soleimani (2017) expanded upon previous research on g2F models by incorporating the effects of geometric irregularities – such as skew angle, unbalanced frame stiffness, and tall column bents – on the seismic performance of concrete box-girder bridges in California. Tall column bents are defined in this study as bents where the average column heights are higher than 1.5 times the average column height of the regular bridges. The study highlighted that irregular bridges may be more susceptible to earthquake induced damage. Among the three irregularities considered, skew angle is the only parameter for which HAZUS provides a single modification factor. Figure 13 presents a comparison between the modification factor used in HAZUS and those computed in the study. The results indicate that for bridges with low skew angles (0° - 15°), the computed factors align well with those used in HAZUS. However, for bridges classified under SKVH (very high skew angles, 30° to 45°) and SKEx

(extreme skew angles, 60° to 77°) categories, HAZUS employs lower modification factors, which, according to Soleimani (2017), leads to an overestimation of the probability of damage.

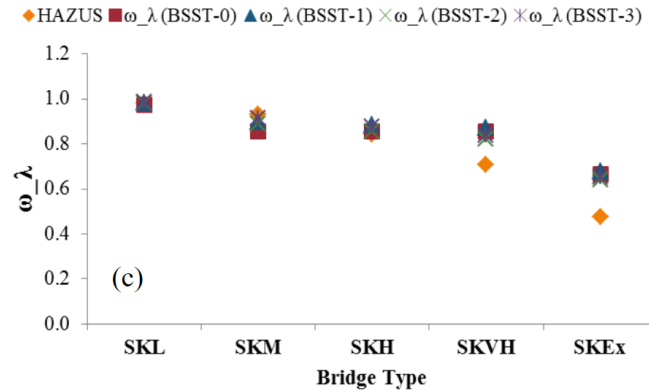


Figure 13. Comparison of the developed adjustment factor with the HAZUS proposed modification factor for post-1990 design era. (SKL, SKM, SKH, SKVH, SKEx are low, medium, high, very high, and extreme skew angles, respectively). (Source: Soleimani, 2017)

Nevada Department of Transportation (NDOT)

Biasi et al. (2017) conducted a ShakeCast case study to assess the seismic vulnerability of bridges in Nevada using two distinct methodologies. The first approach utilized ShakeMap to generate 112 realistic earthquake scenarios, which were then applied through ShakeCast to the 1,831 bridges in Nevada's bridge inventory. The second method employed hazard curves from the USGS National Seismic Hazard Map (NSHM) to determine the return periods of earthquakes anticipated to cause extensive bridge damage. The authors proposed lower structural capacities than those used in ShakeCast for five bridge types classified under HAZUS (HWB10, HWB11, HWB15, HWB22, and HWB23). Table 2 presents the median values of fragility functions proposed by the authors alongside those utilized by HAZUS. These proposed values were derived from existing studies, consultations with NDOT in planning retrofit strategies. As a result of this study, ShakeCast has been implemented and is now operational at NDOT, facilitating rapid post-earthquake assessment of bridges exposed to significant seismic hazards.

Table 2. Median values for the proposed fragility functions vs. HAZUS. (In PSA 1.0 sec) (Source: Biasi et al. (2017))

HAZUS class	NBI class	Year built	Source	Damage states		
				Moderate	Extensive	Complete
HWB10	205	<1983	Proposed	0.18 g	0.27 g	0.36 g
			HAZUS	0.90 g	1.10 g	1.50 g
HWB11	205	≥ 1983	Proposed	0.26 g	0.60 g	0.87 g
			HAZUS	0.90 g	1.10 g	1.50 g
HWB15	402	<1983	Proposed	0.32 g	0.41 g	0.51 g
			HAZUS	0.75 g	0.75 g	1.10 g
HWB22	605	<1983	Proposed	0.18 g	0.27 g	0.36 g
			HAZUS	0.90 g	1.10 g	1.50 g
HWB23	605	≥ 1983	Proposed	0.26 g	0.60 g	0.87 g
			HAZUS	0.90	1.10	1.50

2.1.3 Nonlinear Time History Analysis (NLTHA)

Nonlinear time history analysis (NLTHA) is the most computationally intense analysis employed for response prediction and requires satisfying dynamic force equilibrium in the time domain. NLTHA results can be challenging to interpret, making the process time-intensive due to the sensitivity of structural response to the characteristics of the ground motion used as seismic input (Ramanathan, 2012). NLTHA often serves as a validation tool for simpler assessment methods, as demonstrated in studies (Martinez & Kowalsky, 2023; Pettorruso & Quaglini, 2024). The effective application of NLTHA necessitates careful selection of modeling parameters, as modeling choices significantly influence results (Martinez & Kowalsky, 2023). Therefore, analysts must be well-trained in its implementation, and results should be independently verified using simplified methods.

Pettorruso & Quaglini (2024) investigated the effectiveness of simplified seismic assessment methods for straight, multi-span bridges by comparing them to NLTHA results. The study focused on three bridge archetypes common in the Italian bridge stock: two simply supported bridges (with varying stiffness, pile heights, and horizontal capacities), and one continuous-deck bridge with highly elastic piles. Five analysis procedures recommended by design codes were evaluated: Equivalent Static

Analysis (ESA), Modal Pushover Analysis (MPA), Response Spectrum Analysis (RSA), Linear Time History Analysis (LTHA), and NLTHA.

The comparative analysis revealed important performance trends. ESA, while the most approximate method, showed potential as an efficient tool for preliminary assessment at the portfolio level, particularly for simply supported bridges. RSA offered the best balance between computation efficiency and accuracy for structures expected to remain elastic or undergo limited nonlinear behavior, such as under weak and moderate earthquakes. MPA demonstrated good agreement with NLTHA for the detailed assessment of critical individual bridges, though caution is needed for simply supported systems where reactions may be underestimated. LTHA, while more refined, was not recommended for practical use due to its high computational demand without significant improvements in accuracy, emphasizing the need to balance accuracy with practicality in everyday engineering applications.

2.1.4 Direct Displacement-Based Assessment

Direct Displacement-Based Design (DDBD), proposed by Priestley et al. (2007), is a method for designing structures to achieve a given performance limit state under a seismic hazard. The approach is based on the Substitute-Structure Method, developed by Güllkan & Sozen (1974), which simplifies the design process by representing an inelastic MDOF system as an equivalent SDOF system with equivalent elastic properties. Reinforced concrete structures, when subjected to strong ground motions, undergo a decrease in stiffness and a rise in energy dissipation, significantly influencing their dynamic response (Güllkan & Sozen, 1974). Consequently, the equivalent SDOF system is defined by its secant stiffness at maximum displacement and by the level of equivalent viscous damping (which represents both viscous and hysteretic damping).

Figure 14 illustrates the DDBD fundamentals for a multi-span bridge.

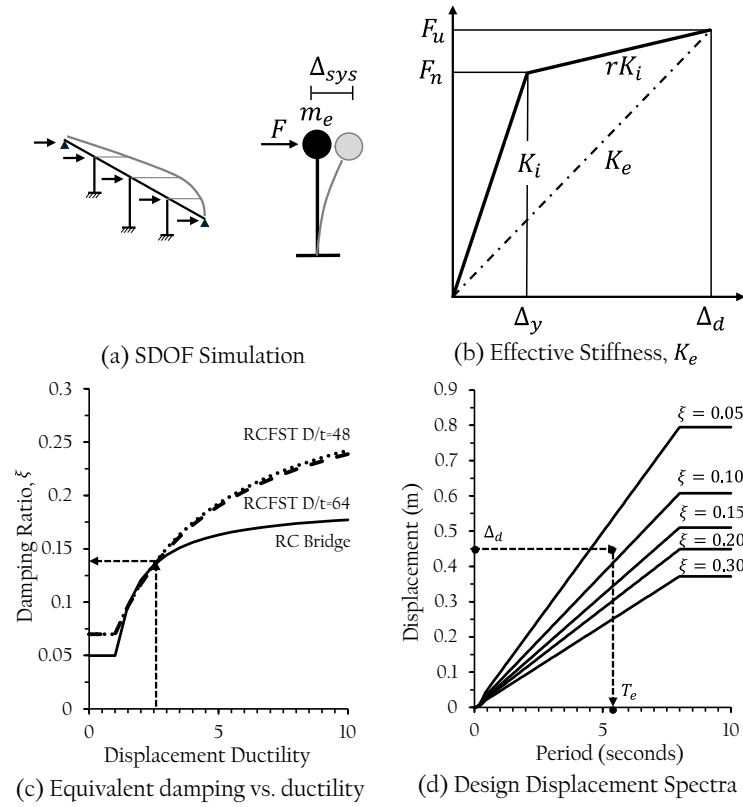
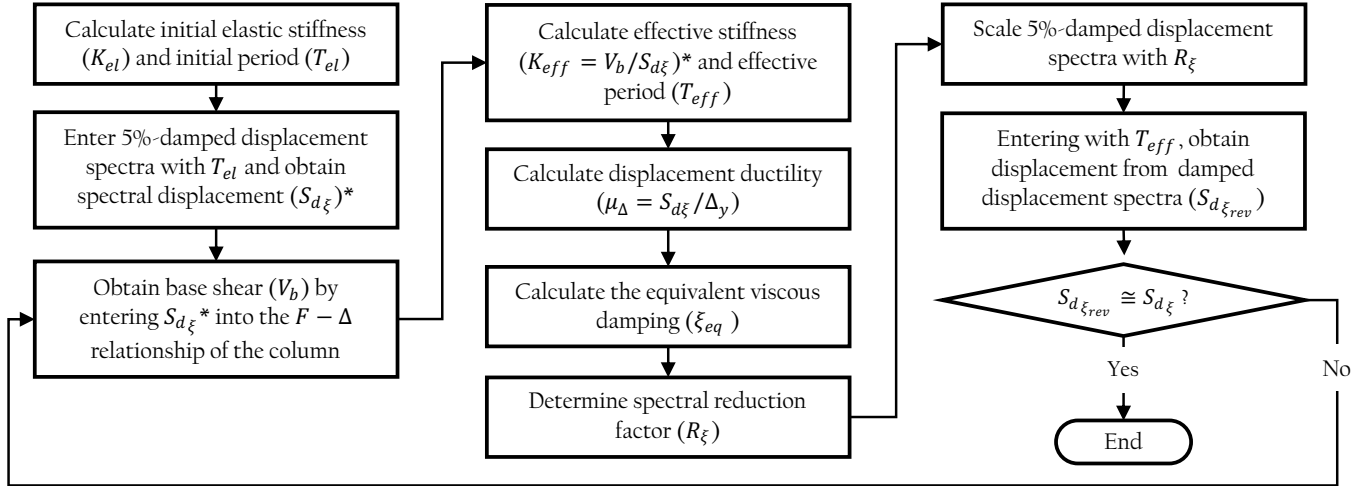


Figure 14. Fundamentals of Direct Displacement-Based Design. (Adapted from Priestley et al., 2007)

A variation of DDBD, known as Direct Displacement-Based Assessment (DDBA), can be implemented to obtain the performance of a structure under a defined seismic demand expressed in the form of response spectra. While DDBD defines the required strength for a structure to achieve the predetermined target displacement, DDDBA presents a different challenge. Here, the final displacement remains unknown, which triggers an iterative process, necessitating multiple adjustments until convergence is reached to determine the structure's final properties. The flowchart for a DDDBA for SDOF is shown in Figure 15.



* For the first iterative cycle, $S_{d\xi} = S_{d5\%}$

Figure 15. Flowchart representation of a proposed DDBA process for an SDOF system.

The DDBA flowchart presented in Figure 15 follows an iterative process, beginning with the initial properties of the SDOF system, such as elastic stiffness and initial period. Using the 5%-damped displacement spectra, the spectral displacement corresponding to the initial period is obtained. The next step involves determining the base shear by inputting the spectral displacement value into the force-displacement relationship of the column. This allows for the calculation of effective stiffness and effective period. Subsequently, the displacement ductility of the system is determined using the spectral displacement of the column's yield displacement. Based on damping-ductility relationships, the equivalent viscous damping and the spectral reduction factor are derived. The 5%-damped displacement spectra is then scaled by the spectral reduction factor and used to obtain a new spectral displacement corresponding to the effective period. If this revised displacement is sufficiently close to the value from the previous iteration, the DDBA process is considered complete; otherwise, additional iterations are performed.

When analyzing an MDOF system, extra complexities arise. In the case of a transverse analysis of continuous deck bridges, the displacement of the columns is influenced by the connection provided between columns by the superstructure, according to Kowalsky (2002). The superstructure guides the

bridge into a particular displacement profile when subjected to lateral seismic excitation. This profile is mainly determined by the columns' stiffness and the abutment restraint, in addition to span length and total length of the bridge.

To be able to represent a multi-span bridge as an equivalent SDOF system, we are required to find the equivalent properties of the system. Figure 16 shows these properties in more detail. The displacement profile is represented as a system displacement (Δ_{sys}), the mass of the superstructure, the abutments, and the columns are depicted as effective mass (m_{eff}), the base shear forces of individual piers are translated to total base shear (V_b). Additionally, the SDOF has effective stiffness (K_{eff}), an effective period (T_{eff}), and an equivalent viscous damping (ξ_{eq}).

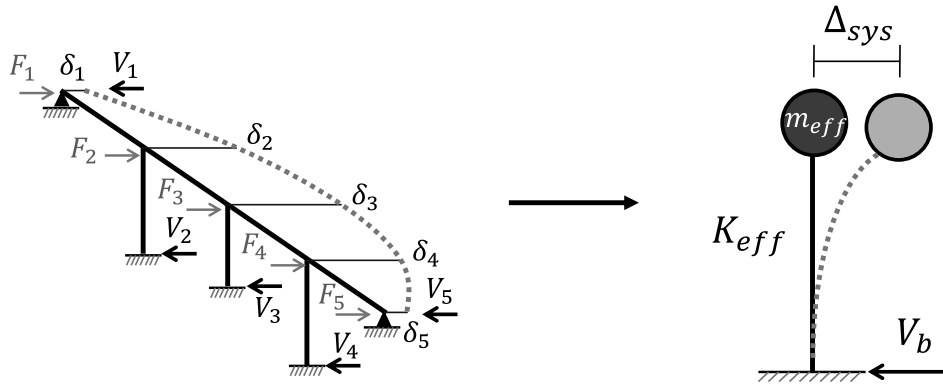


Figure 16. Equivalent SDOF representation of a multi-span bridge under transverse displacement for DDBA application.

Şadan et al. (2013) extended the DDBA approach for application to multi-span reinforced concrete bridges with single-column piers. The methodology consists of four main steps: (1) collecting structural information to derive a bilinear force-displacement curve for each pier, (3) generating the displacement shape in the transverse direction, (3) determining the equivalent SDOF properties for the MDOF system, and (4) conducting the bridge assessment. Figure 17 presents the flowchart of the DDBA approach.

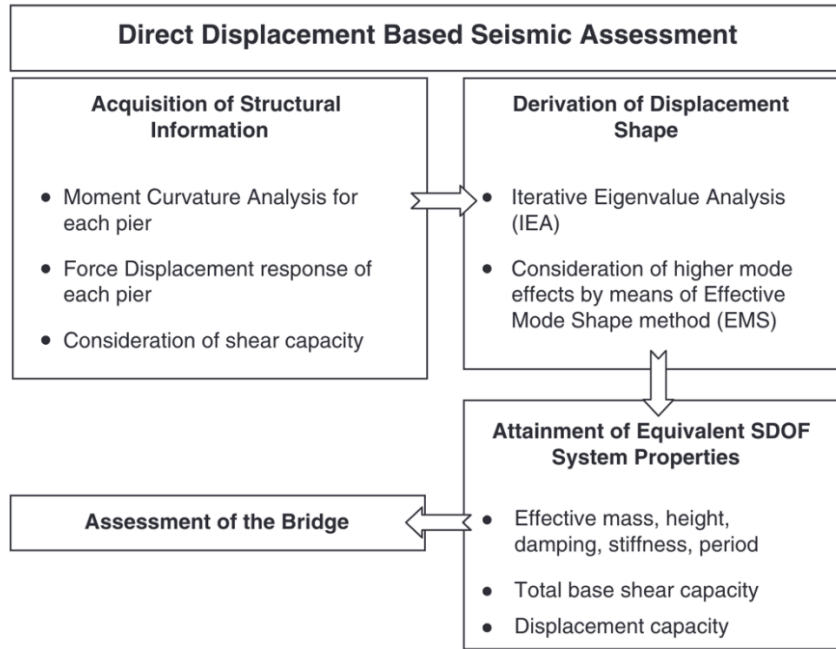


Figure 17. Flowchart for Direct Displacement-Based Assessment of multi-span bridges (Source: Şadan et al., 2013)

In Step 1, the moment-curvature response of critical sections is determined. According to Şadan et al. (2013), a bilinear representation provides satisfactory results for this approach. The plastic hinge method is employed to obtain the force-displacement response of each pier, with the response being influenced by the fixity conditions between the pier and the deck.

Step 2 involves an iterative procedure known as Iterative Eigenvalue Analysis (IEA) to determine the displacement profile of the bridge in the transverse direction. If higher mode effects significantly influence the seismic response of the structure, an additional step must be incorporated at the conclusion of the IEA procedure. This additional step is the Effective Mode Shape (EMS) method, proposed by Kowalsky (2002). According to Şadan et al. (2013), the EMS method is implemented when the mass participation ratio in the orthogonal direction, at the final stage of the IEA process, is below 65-70% of the total mass.

Step 3 consists of converting the MDOF system into an equivalent SDOF system. This process involves determining the displacement capacity, equivalent mass, shear resisted by each pier, equivalent viscous damping, effective stiffness, and effective period.

Finally, Step 4 focuses on the bridge assessment, which follows a pass/fail criterion based on the capacity-to-demand (C/D) ratio. The demand is defined using the elastic displacement spectra specified in seismic design codes.

Şadan et al. (2013) applied the approach to nine strain bridges with deck supported on bearings and four straight bridges with monolithic pier-deck connections. The analysis results were compared to the results obtained using a force-based approach, with both methodologies validated through Incremental Dynamic Analysis (IDA). It is important to note that the approach presented by the authors included certain limitations related to bridge characteristics, such as its applicability limited to single-pier structures, the assumption of pinned transverse directions at the abutments, and the consideration that abutments remain elastic.

The comparison between DDBA and IDA results indicated that, among the 13 bridges analyzed, nine exhibited an error margin between 0-5%, three had errors ranging from 5-10%, and one had an error between 15-20%. In contrast, for the force-based approach, only two bridges had errors within the 0-5% range, two fell within 5-10%, one ranged between 15-20%, and eight exhibited errors exceeding 20%. Based on these findings, the authors concluded that the DDBA procedure is a reliable assessment method and can be applied to bridges with different typologies, with only minor modifications to the procedure.

Kong (2017) applied the DDBA approach to 27 bridges with varying configurations, derived from an existing bridge, BN 0547, in the AKDOT&PF database. Key material properties, including concrete compressive strength, reinforcement yield stress, longitudinal bar diameter, and transverse volumetric

steel ratio, were kept constant. All bridges featured six girders supported on bearings, while all columns had a diameter of 2 meters and were assumed to be fixed at the base and monolithically connected to the superstructure.

The study considered several variables, including column height (7, 11, 13, and 17 meters), span length (30-50 meters), number of spans (3, 4, and 6), and longitudinal steel reinforcement ratio (1%, 2%, and 3%). All 27 structures were subjected to ground motions from the 1999 Chichi Earthquake, while six randomly selected bridges (Bridges 4, 7, 10, 23, 24, and 27) were additionally subjected to ground motions from the 1994 Northridge Earthquake. The assessment evaluated performance levels corresponding to yield, serviceability, and damage control limit states. To validate the predictions obtained from the rapid DDBA approach, a NLTHA was conducted. Figure 18 presents the ratio $\Delta_{sys,THA}/\Delta_{sys,DDBA}$ of all 27 bridges across the considered limit states.

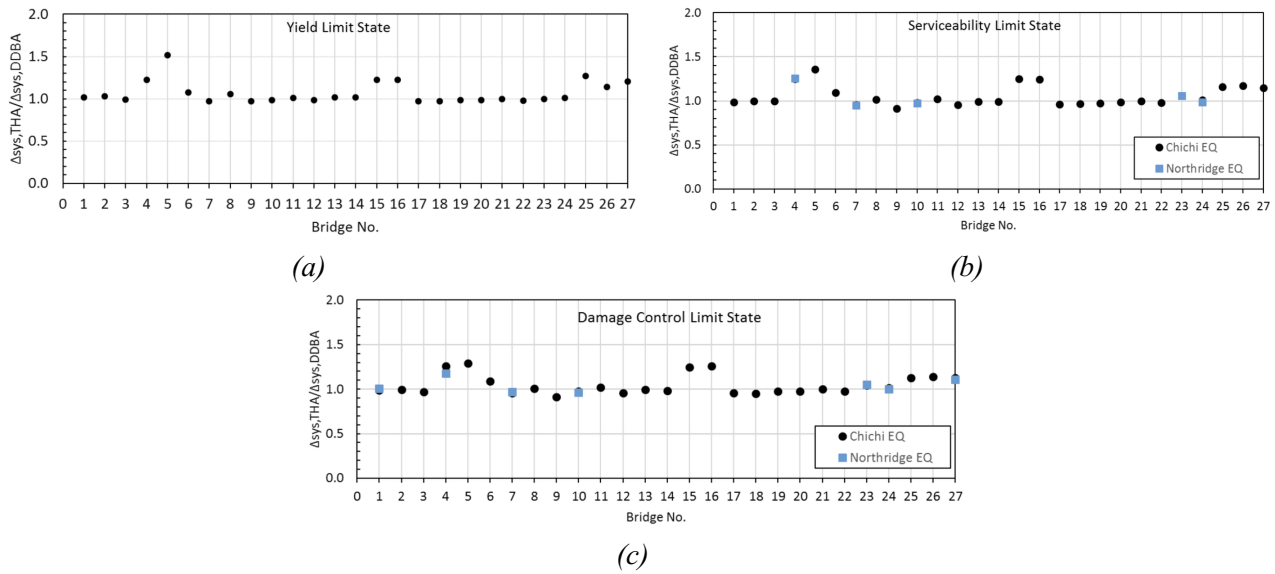


Figure 18. $\Delta_{sys,THA}/\Delta_{sys,DDBA}$ ratios for 27 bridges at (a) yield limit state, (b) serviceability limit state, and (c) damage control limit state. (Source: Kong, 2017)

According to Kong (2017), a strong correlation was observed between the system displacements obtain from DDBA and NLTHA for most bridges, particularly concerning the yield limit state.

Bridges exhibiting high displacement ratios were primarily asymmetric structures (Bridges 4, 5, 6, 15, 16, and 25). Additionally, Bridges 26 and 27, which, despite being symmetric, exhibited high ratios due to their longer lengths. Nonetheless, for symmetric and slightly asymmetric bridges, the displacement patterns obtained using the DDBA approach aligned well with those derived from the NLTHA.

Furthermore, Kong (2017) developed a bridge inventory considering typical parameter ranges, such as column height, span length, and the number of spans. The DDBA method was applied to determine the target displacement for each bridge within the inventory. Based on the analysis results, fragility curves corresponding to the yield limit state were generated, and the parameters that most significantly influenced the seismic fragility of the bridges were identified.

For the bridge inventory, both symmetric and asymmetric four-span bridges were analyzed. Kong (2017) reported that column heights in Alaska typically range from 3 to 23 meters. Considering a four-span bridge where each column height is assigned one of 20 integer values within this range, a total of 8,000 different bridge configuration were possible. Consequently, the DDBA approach was applied to all the bridges with varying geometric properties. The study found that the maximum column height difference ($\Delta H = H_{max} - H_{min}$) has a significant effect on the system displacement results. Figure 19 illustrates the relationship between ΔH and the median values of $\Delta_{sys} \cdot D$.

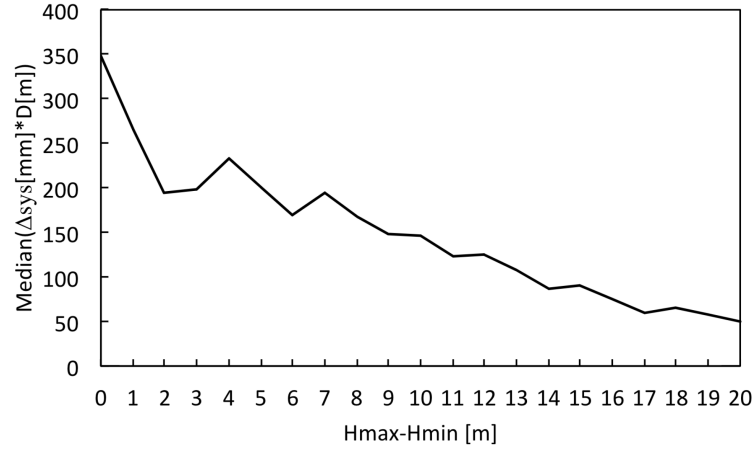


Figure 19. Relationship between median values of $\Delta_{sys} \cdot D$ and ΔH (Source: Kong, 2017)

The negative slope of the observed trend indicates that larger ΔH values are associated with lower $\Delta_{sys} \cdot D$. This outcome aligns with expectations, as higher ΔH values suggest increased structural irregularity, often characterized by the presence of a short column (e.g., 3 meters) at one of the piers. The system displacement is largely governed by the behavior of the shortest column, as displacement is a function of column height. Consequently, shorter columns exhibit smaller displacements, thereby constraining the overall system displacement.

However, it is important to acknowledge the limitations of this study. The analysis is restricted to flexible patterns in which the abutments are fully restrained, with no definition of their stiffness. Additionally, only three limit states are considered in the analysis. These constraints should be considered when generalizing the findings to other structural configurations.

Cetinkaya (2019) applied the DDBA methodology to seven bridges in Alaska (BN 547, 596, 597, 610, 639, 1391, and 1903), comprising three reinforced concrete bridges and four reinforced concrete-filled steel tube (RCFST) continuous bridges. The analysis was conducted exclusively in the transverse direction under the assumption that the superstructure and cap beam behave rigidly, without considering transverse abutment restraint, as stated by the author. The study evaluated three

performance limit states – yield, serviceability, and damage control – using seismic demand data from the 1964 Alaska Earthquake (M9.2), obtained from ShakeMap. The performance level of each bridge was estimated based on the ratio between the limit state displacement and the seismic demand (LS/D). Furthermore, IDA was employed as a nonlinear analysis method to validate the DDBA results. Seven ground motions were matched to each acceleration response spectrum at the respective bridge sites using wavelet theory (Montejo & Suarez, 2013). Figure 20 presents a comparison of LS/D results obtained through DDBA and IDA for the three performance limit states. A preliminary analysis suggests that DDBA is an effective and reliable assessment method, as its results closely align with those obtained from the more computationally intensive IDA approach.

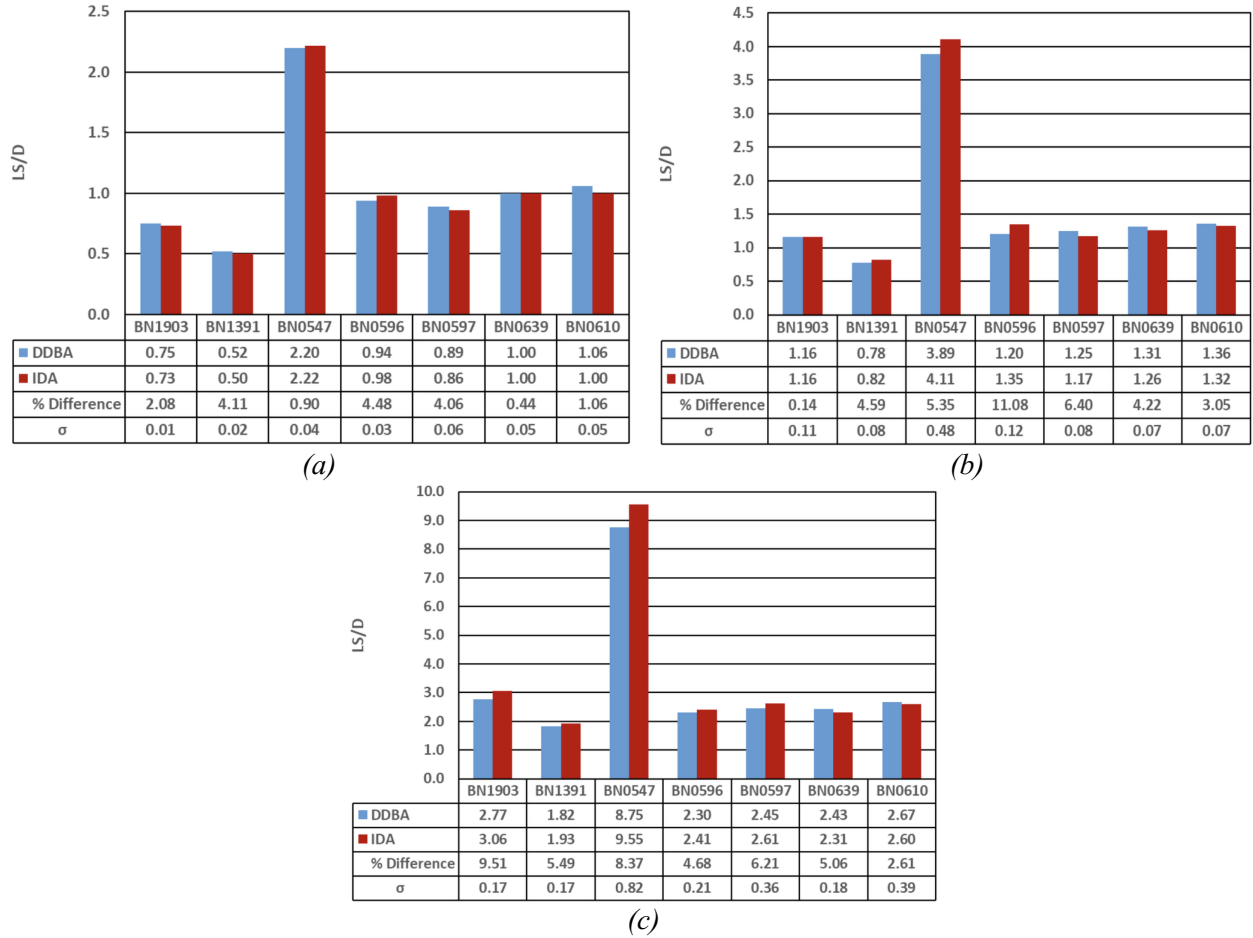


Figure 20. Limit state displacement over demand (LS/D) for the seven Alaskan bridges, using the DDBA analysis method and IDA to verify the results for (a) yield limit state, (b) serviceability limit state, (c) damage control limit state. (Source: Cetinkaya, 2019)

Based on the analysis of the graphs in Figure 20, it can be concluded that none of the bridges reached the damage control limit state ($LS/D > 1$). Bridge BN 1391 attained the serviceability limit state, while four bridges (NB 1903, 0596, 0597, and 0639) only reached the yield limit states.

Out of the 21 comparisons between IDA and DDBA results (considering three limit states for each of the seven bridges), only 8 (38.1%) exhibited a difference of 5% or more. Notably, 5 of these discrepancies occurred in the damage control limit states. Furthermore, the standard deviation of results increased progressively, ranging from 0.01 to 0.06 in the yield limit state, 0.07 to 0.48 in the serviceability limit state, and 0.17 to 0.82 in the damage control limit state. This increasing variability

can be attributed to the displacement demand of the seven ground motion records, whose inherent variability grows as the limit state period goes from yield to damage control.

However, certain limitations should be considered when interpreting these results. The analysis is restricted to rigid superstructures, which does not capture the response of flexible bridge systems. Additionally, abutment restraints were not considered, meaning that the deformation shape was governed exclusively by the relative stiffness of the columns. These constraints should be acknowledged when extending the findings to other bridge configurations.

Martinez & Kowalsky (2022) conducted a comparative analysis of displacement demands on reinforced concrete bridge columns using the DDBA and NLTHA methodologies for both shallow crustal and subduction tectonic regimes. The study incorporated the 3-period and 22-period representations from the 2018 USGS Seismic Hazard Model to determine the displacement demands. For NLTHA, the demands were derived by spectrally matching records of five design spectra from each tectonic regimes.

A total of 288 columns were analyzed, with variations in diameter, aspect ratio, longitudinal and transverse steel ratios, and axial load ratios. The boundary conditions assumed that the columns were fixed at the base while free to rotate and translate at the top. Figure 21 presents the mean response displacement ratios between NLTHA and DDBA across different ductility levels for both tectonic regimes.

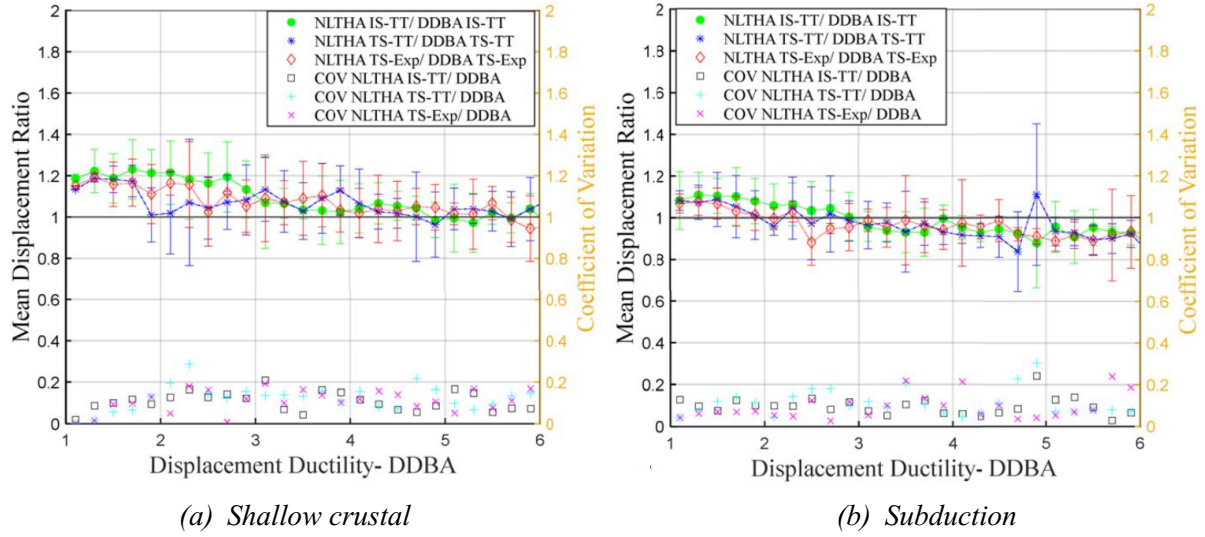


Figure 21. Mean displacement ratio between NLTHA and DDBA for 3 different types of hysteretic model per DDBA ductility level for both tectonic regimes.

For shallow crustal earthquakes, it is possible to observe that the DDBA underestimates the displacement demands for ductility levels below 4. However, at higher ductility levels, the mean ratio converges to a unity, indicating that the DDBA effectively captures the nonlinear response of reinforced concrete circular columns under such conditions (Martinez & Kowalsky, 2022). In contrast, for subduction earthquakes, the DDBA also underestimates displacement demands, though to a lesser extent than in shallow crustal tectonic regimes and primarily for $\mu_{\Delta} < 3$. At a higher ductility level, however, the DDBA tends to overestimate displacement demands.

2.1.5 Summary of Analysis Engines

Post-earthquake bridge assessment methodologies must balance accuracy, efficiency, and practicality. One common approach uses fragility functions to estimate structural damage probabilities based on intensity measures such as PGA and PSA. While computationally efficient, these methods introduce uncertainty due to their generalized nature and limited damage specificity.

HAZUS, developed by FEMA, is a standardized risk assessment tool that estimates losses from earthquakes and other disasters. It enables pre- and post-event analyses but relies on simplified two-

dimensional methods and generic fragility functions, limiting its precision. Despite its low computational demand, HAZUS primarily evaluates column vulnerability and classifies damage into broad categories, reducing its applicability for detailed structural assessments.

ShakeCast, developed by the USGS, improves post-earthquake evaluations by integrating ground motion data with facility inventories. Used by agencies like Caltrans, it allows users to adjust fragility functions and define damage states, improving assessment accuracy. However, it still relies on median fragility values and predefined intensity measures, limiting its predictive capability compared to advanced methodologies.

NLTHA explicitly models material and geometric nonlinearities, capturing effects such as yielding and stiffness degradation. This approach provides the most detailed assessments but requires extensive input data, high computational effort, and is sensitive to modeling assumptions, making it impractical for rapid evaluations.

The DDBA method offers a flexible alternative, balancing effort and accuracy. It allows scalable post-earthquake evaluations, adjusting analysis detail based on available structural data. While requiring more effort than HAZUS and ShakeCast, DDBA significantly improves damage estimation without the computational demands of NLTHA, making it a practical choice for rapid assessments.

In summary, assessment methods vary in complexity and accuracy. HAZUS and ShakeCast offer quick, regional evaluations with limited precision, while NLTHA provides detailed insights at a high computational cost. DDBA emerges as an effective middle ground, enhancing post-earthquake assessments through adaptability and efficiency.

2.2 Bridge Performance Limit States

Performance limit states are defined for bridge members to assess the damage level a bridge will achieve under a seismic event. Well-designed ductile elements, such as bridge columns built after 1995 in Alaska, are expected to pass through several limit states prior to reaching their ultimate capacity: concrete cracking, longitudinal steel yielding, cover concrete crushing, confinement steel yielding, longitudinal bar buckling, and fracture of the previously buckled longitudinal bar. This work will consider the three performance limit states proposed by AASHTO Guidelines for Performance-Based Seismic Design of Highway (2023) and an additional four performance limit states for reinforced concrete columns undergoing nonlinear response. The level of observable damage expected by each one is as follows:

1. **First Yield:** yielding of the extreme tension reinforcement.
2. **Fully Operational:** crushing of concrete cover may occur, and reinforcing bars have likely yielded, but residual cracking widths are small (less than one millimeter).
 - Full use of the bridge is expected immediately following an earthquake.
3. **Operational:** minor inelastic response and narrow flexural cracking in concrete. Exposed reinforcing steel, but not visibly buckled.
 - The bridge is damaged but accessible by emergency vehicles and potentially repairable with or without restrictions on traffic flow.
4. **Damage Control I:** yielding of confinement steel.
 - This limit state represents the demarcation where significant intervention is needed.
5. **Damage Control II:** buckling of longitudinal bars and significant damage to the core concrete.

- The exceedance of this limit state is related to the high costs of structural repair that may exceed the replacement cost of the bridge or its components.
6. **Life Safety:** permanent offsets and cracking. Exposed, buckled, and possibly some fractured reinforced steel.
 - The potential for bridge collapse is minimized during an earthquake but perhaps requires replacement afterward.
 7. **Near Collapse:** the structural damage is severe, and life safety is at risk, but structural collapse is prevented.

The *fully operational*, *operational*, and *life safety* limit states are defined by the AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges (2023), and are also known as PL3, PL2, and PL1 respectively.

It is important to note that the Performance Limit States mentioned in this section are related exclusively to RC columns. During the development of Phase II of this project, limit states related to RCFST columns will be defined and studied in more depth.

2.2.1 Strain Limits

Even though qualitative definitions are important for visual inspection, engineering demand parameters (EDP) must be provided to accurately assess the bridge's structural response under seismic loading. Goodnight et al. (2016) identified steel and concrete strains as effective EDPs. For that reason, empirical expressions were developed to characterize the material behavior of each performance limit state in terms of strain limits. The expressions are presented in Table 3.

Table 3. Performance Limit States evaluated and their concrete and reinforcing steel strain limit

Limit State	Concrete Strain Limit	Reinforcing Steel Strain Limit
AASHTO Guidelines (2023)		
Fully Operational (PL3)	0.004	0.010
Operational (PL2)	$0.004 + 1.4\rho_v f_{yh} \varepsilon_{su} / f'_{cc}$	$0.8 \cdot \varepsilon_{s,bb2}$
Life Safety (PL1)	$1.4 \cdot (0.004 + 1.4\rho_v f_{yh} \varepsilon_{su} / f'_{cc})$	$\varepsilon_{s,bb2} = 0.032 + \frac{790\rho_v f_{yhe}}{E_s} - 0.14 \cdot P / f'_{ce} \cdot A_g$
Additional Performance Limit States		
First Yield	0.002	ε_y
Damage Control I	$0.009 - 0.3 \cdot A_{st} / A_g + 3.9 \cdot f_{yhe} / E_s$	-
Damage Control II	$0.004 + 1.4\rho_v f_{yh} \varepsilon_{su} / f'_{cc}$	$\varepsilon_{s,bb1} = 0.03 + \frac{700\rho_v f_{yhe}}{E_s} - 0.1 \cdot P / f'_{ce} \cdot A_g$
Near Collapse	-	0.07

For the parameters given in Table 3, ρ_v is the transverse volumetric steel ratio, which is calculated as $4 \cdot \frac{A_{sp}}{d_s \cdot s}$, where A_{sp} is the area of one transverse reinforcement, d_s is the distance center to center of transverse reinforcement, and s is the spacing of transverse reinforcement; f_{yh} is the transverse steel yield stress, ε_{su} is the maximum reinforcement strain (taken as 0.12), f'_{cc} is the compression strength of the confined concrete, A_{st} is the longitudinal reinforcement area, A_g is the gross area of the section, f_{yhe} is the expected transverse steel yield stress, E_s is the modulus of elasticity of the steel, and P is the axial load acting at the section centroid.

The concrete strain limits for Operational, Damage Control II, and Life Safety are derived from the Mander Confined Concrete Model (Mander et al., 1988). The model is based on the principle that the strain energy capacity of the transverse steel is equal to the strain energy stored in the concrete core (Mander et al., 1988). For the Operational and Damage Control II limit states, a factor of 1.0 is applied, representing the lower-bound compressive strain limit. However, to address the conservatism in the equation, the Life Safety limit state incorporated a factor of 1.4, representing an upper-bound

compressive strain limit. The concrete strain limit for Damage Control I is based on the Goodnight et al. (2016) Model.

The steel strain limit for Damage Control II is established according to the Goodnight et al. (2016) Model. Furthermore, the steel strain limits for Operational are derived from the Goodnight et al. (2016) Model. The expression for the Operational limit state is based on the statistical data obtained from this study, in which less than 10% of reinforcing bars are expected to buckle. This threshold corresponds to approximately 80% of the limit observed at the Life Safety limit state.

It is important to note that while this study focuses on seven selected limit states, the methodology presented in this report can be extended to incorporate additional limit states, provided that corresponding concrete and steel strain limits are defined.

2.2.2 Converting Limit States Strains to Member Deformations

Once the limit state strains are defined, their corresponding curvature can be obtained. Modern bridge columns, designed as ductile elements, are intended to develop plastic hinges at predetermined locations to dissipate energy under earthquake loading. The curvatures at the limit states associated with bridge columns are converted to top displacements by applying an idealization of curvature distribution, such as the Plastic Hinge Method proposed by Priestley et al. (2007). The method specifies an analytical plastic hinge length L_p in which strain and curvature are treated as equal to the maximum value located in the column base, according to Priestley et al. (2007). The width of the block is equal to the plastic curvature at the critical section, as shown in Figure 22.

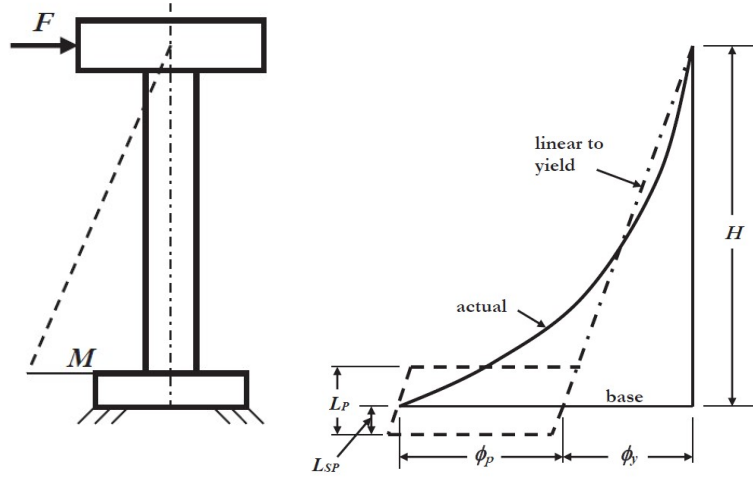


Figure 22. Equivalent curvature distribution for a single bending bridge pier (Adapted from Priestley et al., 2007)

The top displacement at the at the equivalent yield limit state can be obtained by applying the following expression:

$$\Delta_y = C_1 \phi_y (H + L_{sp})^2 \quad (1)$$

Where C_1 is a constant that depends on end fixity. $C_1 = 1/3$ for a rigid base and pinned top case. $C_1 = 1/6$ for a rigid base and top case. ϕ_y is the equivalent yield curvature of the section, and for circular columns, it is taken as approximately $\phi_y = 2.25\varepsilon_y/D_c$. H is the height of the column and D_c is the diameter of the column. L_{sp} is the strain penetration length equal to $L_{sp} = 0.022f_{ye}d_{bl}$, where f_{ye} is the expected yield strength (in MPa) of the longitudinal reinforcement and d_{bl} is their diameter (in mm).

For other limit states beyond yielding, the top displacement can be obtained by:

$$\Delta_{ls} = \Delta_y + (\phi_{ls} - \phi_y)L_p H \quad (2)$$

The parameter L_p , plastic hinge length, can take different forms depending on the controlling strain limits:

- For limit states governed by compressive strain limits, L_p is given by:

$$L_p = L_{pc} = kL_c + L_{sp} \geq 2L_{sp} \quad (3)$$

Where $k = 0.2(f_{ye}/f_{ue} - 1) \leq 0.08$, and L_c represents the length from the critical section to the point of contraflexure in the member.

- For limit states governed by tensile strain limits, L_p is given by:

$$L_p = L_{pt} = L_{pc} + \gamma D_c \quad (4)$$

Where $\gamma = 0.4$ for bidirectional loading, and equal to 0.3 for unidirectional loading, for limit states controlled by tensile strain limits.

By linking material strain limits to member deformations, the approach enables consistent damage assessment and displacement predictions.

2.3 Ground-Motion Characterization

Alaska is one of the most seismically active areas in the world (AEC, last accessed June 6, 2025). More earthquakes occur in Alaska in a given year than in any other region of the US. In the past century the State of Alaska has experienced devastating earthquakes from shallow crustal and subduction zone seismic sources (Powers et al., 2024). Alaska experiences deformations on the overriding North American plate and continental transform in the Fair-Queen Charlotte Fault zone. Subduction events typically occur in the Aleutian-Alaska subduction zone (see Figure 23). Because of various types of earthquakes induced by the complexity of the tectonic environment, engineers should select proper GMMs to estimate the seismic demand (i.e., spectral displacement) when ground-motion records nearby or at the site of interest are not available.

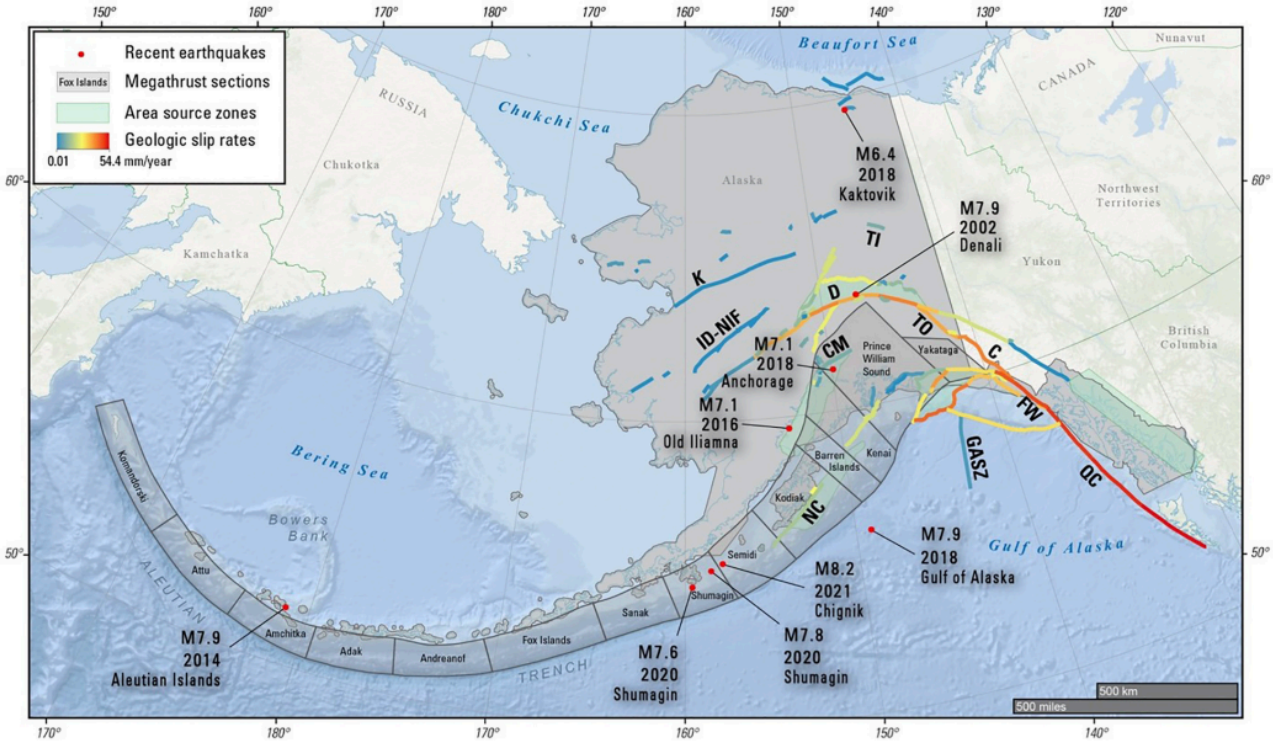


Figure 23. Map of Alaska showing the fault and interface sections used in the 2023 Alaska National Seismic Hazard Model. Epicenters (red dots) of recent significant earthquakes with moment magnitude (M) are included for reference (after Powers et al., 2024)

Shallow and deep geologic structures can contribute to the spatial variability of earthquake ground shaking. For example, Figure 24 shows the spatial variability of PGA induced by the 2018 M7.1 Anchorage earthquake (Cabas et al., 2021).

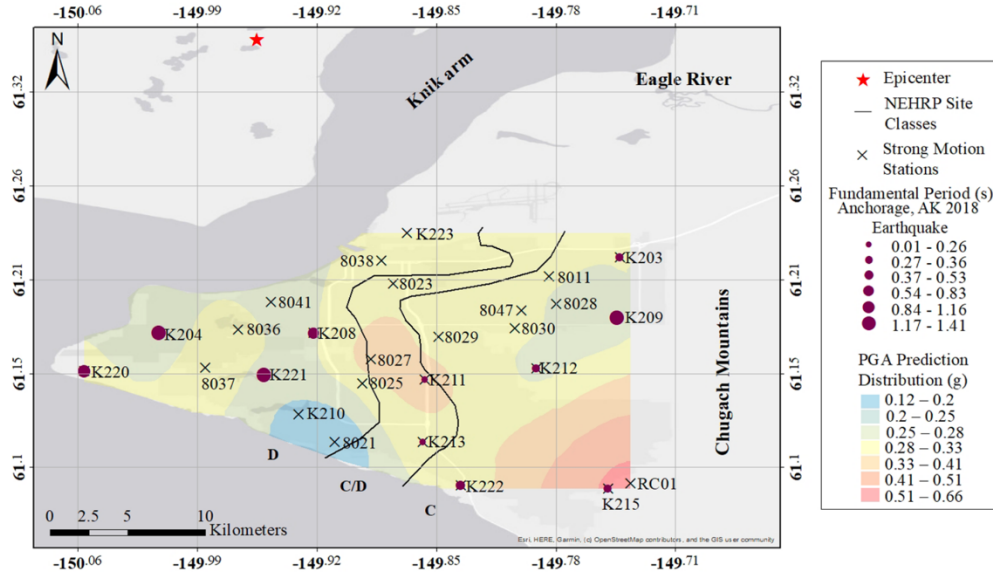


Figure 24. Spatial variability in earthquake ground motions (indicating peak ground acceleration, PGA) induced by 2018 Anchorage earthquake (Cabas et al., 2021)

In this work, when ground-motion records are not available at or nearby the site of interest, we examine appropriate existing GMMs to predict spectral displacements appropriate for different tectonic regimes and incorporating site effects. The commonly used site parameter, V_{S30} is used (either when measured in situ or from proxy-based methods) to account for site effects in the selected GMMs. Furthermore, for cases when a recording station exists nearby the site of interest, but the records correspond to different site conditions than the ones at the site of interest, we examine a modification/adjustment framework of ground-motion as an alternative for ground-motion characterization incorporating site effects.

2.3.1 Existing Ground Motion Models to Predict Spectral Displacements

We identify five appropriate GMMs to predict spectral displacements (SDs) from the diverse tectonic regime (i.e. shallow crustal and subduction zones) and site conditions in Alaska: Akkar & Bommer (2007; AB07), Faccioli et al. (2004; F04), Campbell & Bozorgnia (2008; CB08), Kuehn et al. (2020; KBCG20), Parker et al. (2020; PSBAH20), and the NGA-West2 GMMs. The first two GMMs (i.e.

AB07 and F04) provide SD values directly. CB08 used statistical regressions based on pseudo-spectral accelerations (PSAs), and later converted PSAs to SDs using angular frequency to provide SD values:

$$PSA = \omega^2 SD$$

Where ω is the natural angular frequency of the single degree of freedom system. The KBCG20 and PSBAH20 models correspond to NGA-Sub (Bozorgnia et al., 2022) project and can be used for subduction zone events. The NGA-West2 GMMs (e.g., BSSA14, CY14, etc; Boore et al., 2014; Chiou & Youngs, 2014) are appropriate for shallow crustal seismicity (Bozorgnia et al., 2014).

Akkar and Bommer (2007)

The Akkar & Bommer (2007; here after referred to as AB07) model was developed for shallow crustal earthquakes. The model provides the geometric mean spectral displacement (SD) and peak ground acceleration (PGA). Scaling factors are considered to obtain spectral ordinates at damping values other than 5 % (see Figure 25). The main parameters in AB07 are moment magnitude (M_w), Joyner-Boore distance (R_{jb}), a binary variable defining soil conditions and another variable for fault type. The limits of application of the AB07 model includes: (1) moment magnitudes between 5 and 7.6 and (2) distances up to 100 km. The AB07 model provides SDs up to 4 sec period due to the contamination of the recordings with long-period noise beyond that threshold. Also, site effects are not considered beyond the distinction capture between soft and stiff soils used in their parameterization (i.e., S_S and S_A).

The equation for the geometric mean SD is shown below:

$$\log[SD(T, \xi)] = b_1 + b_2 \mathbf{M} + b_3 \mathbf{M}^2 + (b_4 + b_5 \mathbf{M}) \log \sqrt{R_{jb}^2 + b_6^2} + b_7 S_s + b_8 S_A + b_9 F_N + b_{10} F_R$$

where M = Moment magnitude, R_{jb} = Joyner-Boore source -to-site distance (km), S_S and S_A = binary variable for soft and stiff soil (taking values of 0 or 1), F_N and F_R = normal or reverse faulting earthquake, coefficient $b_1 - b_{10}$ depending on the desired damping ratio.

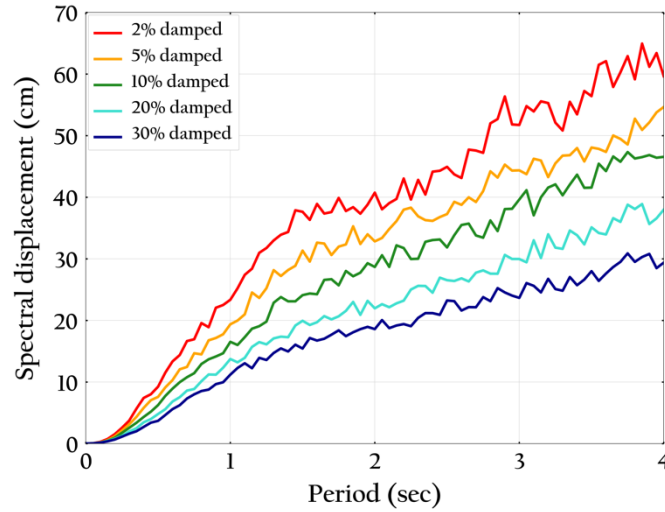


Figure 25. Example of spectral displacements based on the AB07 model: $M = 7$, Reverse fault, Soft soil, $R_{jb} = 2$ km (adapted from Akkar & Bommer, 2007)

Faccioli et al. (2004)

The Faccioli et al. (2004; hereafter referred to as F04) model is a bilinear-shape model and was developed for shallow crustal event by multiple events from different regions (see Figure 26). The main parameters in F04 are magnitude (M_w), hypocentral distance (r), and stress drop ($\Delta\sigma$). The applicability of the F04 model is limited to: (1) moment magnitudes less than 7.5 and greater than 5.5 (2), source to site distances from 10 to 50 km, and (3) stress drops between 1 MPa and 10 MPa. Compared with the AB07 model, the F04 model provides response spectral values up to longer oscillator periods (i.e. 10 sec). Site effects are not considered in the F04 model though.

The functional form of the F04 GMM is as follows:

$$\log_{10} d_{max} = -4.46 + \frac{1}{3} \log_{10} \Delta\sigma + M_w - \log_{10} r$$

$$T_c = 1.0 + 2.5(M_w - 5.7) \text{ second}$$

where T_c = Period when the maximum displacement is reached, $d_{\max}$ = the peak ground displacement, r = hypocentral distance, $\Delta\sigma$ = stress drop.

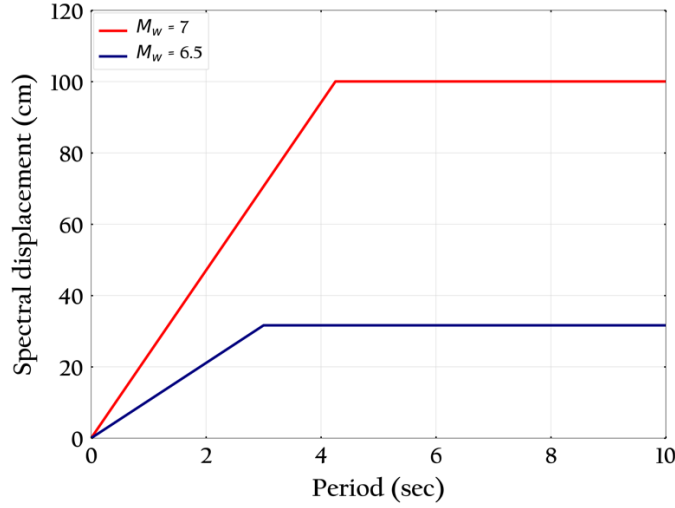


Figure 26. Example of spectral displacements obtained from the F04 model with $r = 5$ km, $\Delta\sigma = 3$ MPa and M_w of 7 (red) and 6.5 (blue).

Campbell and Bozorgnia (2008)

The Campbell & Bozorgnia (2008; hereafter referred to as CB08) is a GMMs developed for shallow crustal tectonic regimes. The CB08 model provides the geometric mean horizontal component (GMRotI50; Boore et al, 2006) of peak ground acceleration (PGA), peak ground velocity (PGV), and PSAs. Input parameters include moment magnitude (M_w), rupture distance (R_{rup}), Joyner-Boore distance (R_{jb}), faulting indicator, slip angle, dip angle, the time-averaged shear-wave velocity in the top 30 m of the site profile (V_{s30}), the depth to the top of the coseismic rupture plane (Z_{TOR}), the depth to the 2.5 km/s shear-wave velocity horizon ($Z_{2.5}$), and the median estimate of PGA on a reference rock outcrop (A_{1100}). The model applicability is limited to: (1) $4.3 \leq M_w \leq 7.9$, (2) $0.1 \text{ km} \leq R_{\text{rup}} \leq 199 \text{ km}$. The CB08 model predicts spectral accelerations for long-period ground-motion (up to $T =$

10 s). Site response terms are included in the CB08 model. Conversion from PSAs to SDs is required using the natural angular frequency. Figure 27 shows an example of spectral displacements obtained using the CB08 model for a moderate to large magnitude event at a soft site condition.

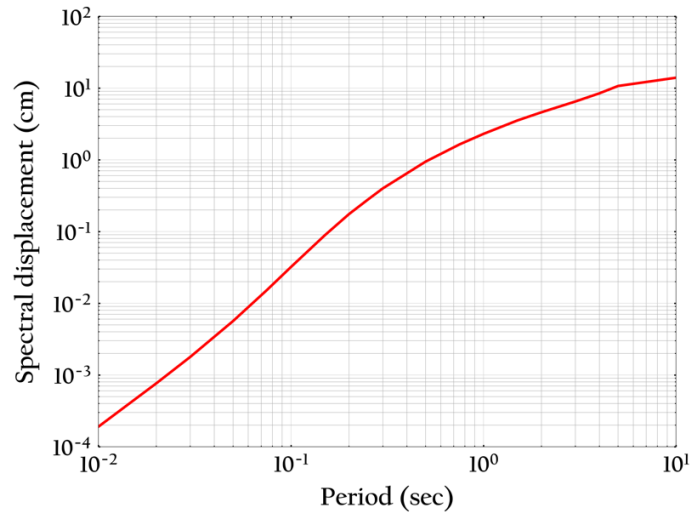


Figure 27. Example of spectral displacements based on the CB08 model: $M_w = 7$, $R_{rup} = 54.8$ km, $V_{S30} = 290$ m/s.

Kuehn et al. (2020)

The Kuehn et al. (2020; hereafter referred to as KBCG20) model is a partially non-ergodic GMM for subduction events developed as a part of the NGA-Subduction project (Kuehn et al., 2020, NHR3; <https://www.risksciences.ucla.edu/nhr3/nga-subduction/gmtools>; last accessed November 2022). The terms “partially non-ergodic” refer to the ability of the model to capture path- and/or site-specific effects. The ergodic assumption implies that temporal variations in earthquake ground motions is replaced by spatial variations for a given parameterization of a rupture scenario (e.g., M_w , R_{rup} and V_{S30}). The KBCG20 model estimates the median ground-motion response spectra for the ground motion pair rotated over all horizontal orientations (Bozorgnia et al., 2012), and it is referred to as SaRotD50 (Boore, 2003). Ground-motion scaling effects account for differences in moment magnitude, source-to-site distances, site amplification, basin amplification (mostly affecting long-

period motion), adjustable event type (interface and intraslab events), and different subduction zone regions. The KBCG20 model can be applicable to estimate peak ground motion parameters and 5 %-damped PSAs with a period range from 0.01 to 10 s. The KBCG20 model accounts for linear site amplification by V_{S30} scaling.

The model applicability includes: (1) $5 \leq M_w \leq 9.5$ for interface events and $5 \leq M_w \leq 8.5$ for intraslab events, (2) $10 \text{ km} \leq R_{\text{rup}} \leq 1000 \text{ km}$, (3) $Z_{\text{TOR}} \leq 50 \text{ km}$ for interface events and $Z_{\text{TOR}} \leq 200 \text{ km}$ for intraslab events, (4) $150 \leq V_{S30} \leq 1500 \text{ m/sec}$, (5) basin (sediment) depth depending on regional applicability. The KBCG20 model provides PGA and 5 %-damped PSA, so conversion from PSAs to SDs are required similar as the CB08 and PSBAH20 models.

Parker et al. (2020)

The Parker et al. (2020; PSBAH20) is a semi-empirical GMM for subduction events. The model was also developed as a part of the NGA-Sub project (Parker et al., 2022, NHR3; <https://www.risksciences.ucla.edu/nhr3/nga-subduction/gmtools>; last accessed November 2022). The PSBAH20 model provides RotD50 values of PGA and PSA. The PSBAH20 model captures observed differences in source and path effects for interface and intraslab events. Regionalized GMM components include the model constants, anelastic attenuation, magnitude-scaling break point, linear site response, and sediment depth terms. Input parameters in the model include earthquake magnitude, rupture distance, hypocentral depth, the time-averaged shear-wave velocity in the top 30 m of the site profile, the depth to the 2.5 km/s shear wave velocity isosurface, among other. Similarly to the KBCG20 model, a global version of this model can be applied with larger epistemic uncertainty for locations where regional factors are not defined. Further, the interface and intraslab models are applicable only to sites in the forearc region of subduction zones.

Thresholds for the applicability of the model applicability include: (1) $4.5 \leq M_w \leq 9.5$ for interface events and $4.5 \leq M_w \leq 8.5$ for intraslab events, (2) $20 \text{ km} \leq R_{\text{rup}} \leq 1000 \text{ km}$ for interface events and $35 \text{ km} \leq R_{\text{rup}} \leq 1000 \text{ km}$ for intraslab events, (3) $Z_{\text{hyp}} \leq 40 \text{ km}$ for interface events and $Z_{\text{hyp}} \leq 20 - 200 \text{ km}$ for intraslab events, (4) $150 \leq V_{S30} \leq 2000 \text{ m/sec}$, (5) basin (sediment) depth depending on regional applicability. The PSBAH20 model can be applied only to sites in the forearc region of subduction zones. The PSBAH20 model provides PGA and 5 %-damped PSA, so conversion from PSAs to SDs are required similar to the CB08 and KBCG20 models.

2.3.2 Site Parameter, V_{S30} , to Characterize Local Site Response

The time-averaged shear wave velocity of the top 30 m of deposit (V_{S30}) is (1) widely used as a proxy for site effects, (2) forms the current basis of seismic site class in the building code, (3) underpins site amplification function, and (4) is a required input to many empirical GMMs as the site term (Geyin & Maurer, 2023). Values of V_{S30} can be obtained via proxy-based methods or through invasive or noninvasive field investigation techniques (e.g., downhole arrays or surface wave testing) .

Proxy-based V_{S30}

Proxy-based V_{S30} models are build by combining various site metadata, including the V_s conditions (i.e., V_s profiles and V_{S30}) and secondary information (i.e., surface geology, topographic gradient/slope, and geomorphic terrain categories (Ahdi, Stewart, Kwak, et al., 2017)). Proxy-based V_{S30} provides estimated values commonly based on topographic slope (D. J. Wald & Allen, 2007) or informed by geology and other parameters (e.g., Thompson et al., 2014; Geyin & Maurer, 2023), so uncertainties in proxy-based V_{S30} values can be large and should be quantified by a standard deviation (σ).

Proxy-based V_{S30} values can be obtained from different models of proxy-based. (i.e., USGS Mosaic, Heath et al. (2020) last accessed in 2022, Ahdi, Stewart, Kwak, et al., 2017, and Geyin & Maurer, 2022, 2023). Ahdi et al. (2017a) developed a regional proxy-based model compiling a V_S profile database with additional sites with measured V_{S30} available in Alaska as a part of the NGA-Subduction project. V_{S30} data in Alaska was compiled from 126 measured V_S profiles (Ahdi et al, 2017a). Most of the V_{S30} data are clustered in Anchorage, Fairbanks, Seward, Valdez, and areas affected by the 2002 Denali earthquake. The main challenge to develop a proxy-based V_{S30} model in Alaska is the lack of data across geologic formations in the state, so alternative approaches are suggested for future research directions, namely (1) validating/calibrating PNW (the Pacific Northwest Region of North America) models for application in Alaska for geologic conditions for which V_S data is available, and/or (2) adopting models from other regions for similar geologic conditions in Alaska that lack V_S data. In this project, we pursue the second option and identify global and/or regional models that can inform the spatial distribution of V_{S30} values in Alaska.

USGS (Heath et al., 2020) provides a code-based hybrid global map (called USGA Mosaic) so that new regions can be added (or removed) to update or revise the V_{S30} grid. The final V_{S30} raster grid is hosted on the ArcGIS server (accessible via <https://earthquake.usgs.gov/data/vs30/>). The USGS Mosaic originates from various data sources, including National and sub-national geological surveys, regional seismic network operators, and similar agencies outside of the United States. Direct quantifications of the uncertainties in the USGS Mosaic are currently not available.

Geyin & Maurer (2022, 2023) developed a set of U.S. national V_{S30} models and maps based on remote sensing and machine learning (ML) techniques (accessible from https://doi.org/10.17603/ds2-80d8-9m83_v1; Geyin & Maurer, 2022). However, there are still limitations regarding the ML-base model (e.g. the lack of mechanistic links between prediction variables and targets). Geyin & Maurer (2022)

model uses various above-ground geospatial variables and V_{S30} data sources. 7081 V_{S30} measurements (including 15 V_{S30} measurements also documented in Ahdi, Stewart, Kwak, et al. (2017) and McPhillips et al. (2020) in Alaska) were used for the model development. 17 above-ground geospatial variables were included: (1) depth of bedrock (Shangguan et al., 2017), (2) depth of groundwater (Fan et al., 2013), (3) geologic unit (Horton et al., 2017), (4) consolidation state (Horton et al., 2017), (5) measured distance to river (Lehner & Grill, 2013), (6) compound topographic index (Verdin, 2017), (7) classified geomorphologic phenotype, (8) topographic slope, (9) topographic position index, (10) profile curvature, (11) tangential curvature, (12) roughness, (13) terrain ruggedness index, (14) vector ruggedness measure, (15) landform entropy, (16) landform uniformity, (17) landform Shannon index (Amatulli et al., 2018). The results for the predictive V_{S30} model (Geyin & Maurer, 2022) were blended with the D. J. Wald & Allen (2007) topographic slope map.

Geyin & Maurer (2023) provides two maps (called Model 1 and Model 2) due to undesirable transitions at a few state boundaries and different map geologies from different state source maps. Model 1 contains V_{S30} predictions across the contiguous United States, and Model 2 provides V_{S30} predictions across all 50 US States (including Alaska) and Puerto Rico. The standard deviation of the kriged residual is available for each model. The standard deviations of Model 1 and Model 2 are equal to 0.218 and 0.264, respectively. Figure 28 shows their proxy-based V_{S30} map of Model 2 for Alaska.

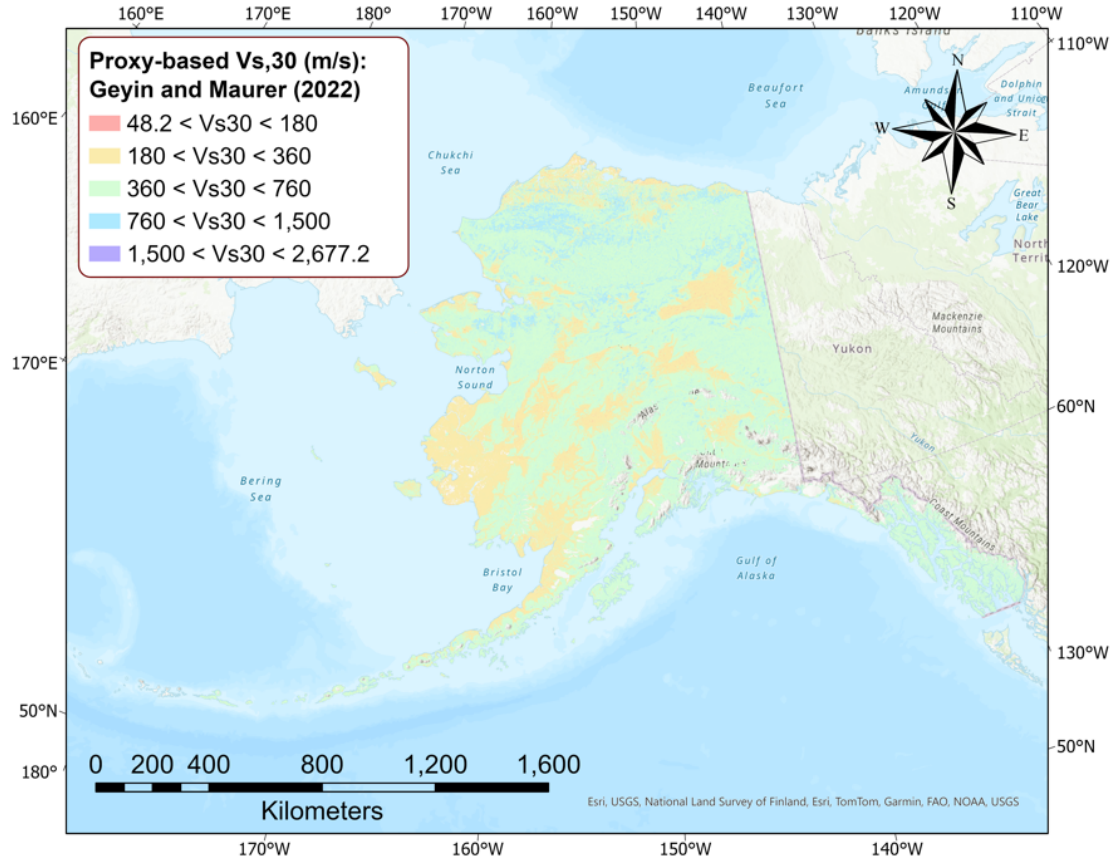


Figure 28. Proxy-based V_{S30} map for Alaska after (Geyin & Maurer, (2022)). Most locations fall within site classes D ($180 < V_{S30} < 360$ m/s) and C ($360 < V_{S30} < 760$ m/s).

Measured versus Estimated V_{S30}

Very limited measured V_{S30} data in Alaska is available. Dutta et al. (2000) reported 36 V_{S30} values, McPhillips et al. (2020) provided Dutta et al. (2000) 15 V_{S30} values, and more recently Thornley et al (2021) characterized McPhillips et al. (2020) 55 sites using measured and estimated V_{S30} values. The most recent measured/estimated V_{S30} can be obtained from Thornley et al. (2021). Values of V_{S30} at 55 sites were estimated based on horizontal-to-vertical spectral ratio (HVSr) from three-component ground motions (two horizontal and one vertical motions). 35 strong ground-motion stations were utilized to define the relationship between HVSr and site response in the study.

However, similar to the previous efforts in the region, most of the estimated V_{S30} values were in Anchorage (see Figure 29).

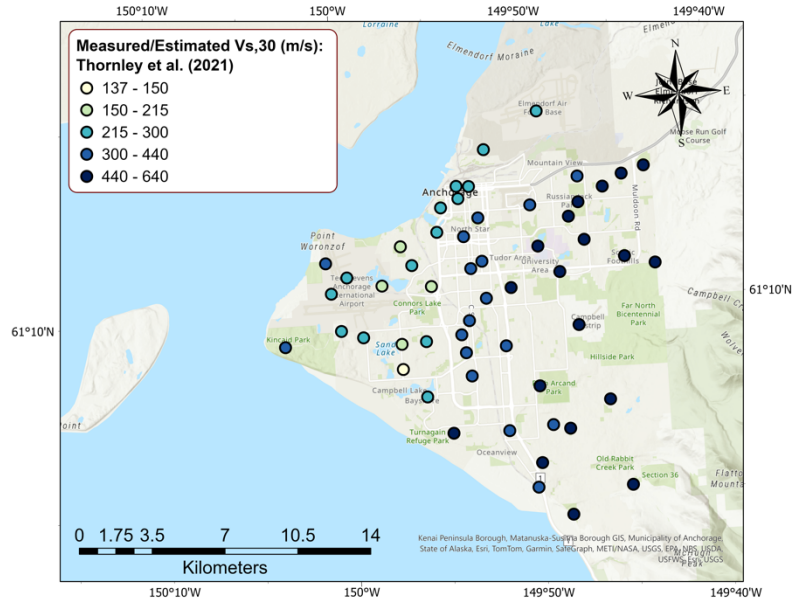


Figure 29. The location of measured and estimated V_{S30} (adapted from Thornley et al., 2021)

The aforementioned V_{S30} values can be used in GMMs presented in this report to account for site effects at multiple bridge sites across the state of Alaska. One important consideration is the level of uncertainty in V_{S30} values, particularly outside of Anchorage.

2.3.3 Ground-Motion Modification to Obtain Input Motions at the Reference Site

Seismic site response analyses (SRA) are commonly used to understand wave propagation phenomena numerically and estimate surface motions that may directly affect the stability of structures (Cabas et al., 2017). SRA can evaluate the effects of local geologic conditions which affects the amplitude of seismic waves (i.e., amplifying or deamplifying ground motions), frequency content, and duration (Seed & Idriss 1971; Borchardt 1994; Kramer 1996). Thus, understanding local site conditions and performing SRA can be important for a seismic design process (Rathje et al., 2010).

When one-dimensional (1-D) equivalent-linear SRA are conducted, recorded ground motions at the reference depth are used as input motions. In general, earth materials below the reference depth are assumed to behave as an elastic material, and the numerical boundary used is referred to as an elastic halfspace (EHS). Input ground motions are often assigned to the EHS boundary. This assumption of the EHS indicates that the material below the reference depth is sufficiently stiff to ensure that no other strong impedance contrasts occur, underneath implying that some of the energy of the downward-propagating seismic waves is absorbed by the bedrock (Cabas et al., 2021). As a result, SRA could not explicitly capture the effects of wave propagation below the reference depth. These effects must be represented by the selected input motions (Cabas & Rodriguez-Marek, 2017).

Selecting input motions at the reference depth is a crucial step in SRA. Current protocols recommend that recorded ground motions can be selected at a station located at bedrock which is exposed at the ground surface (Kramer, 1996). These motions, called rock outcropping motions (i.e., bedrock motions), can be the input motions, and the bedrock as the reference site can be defined based on V_{S30} . The most commonly used V_{S30} value for the reference site is the NEHRP site classification B/C (i.e., $V_{S30} = 760$ m/s). However, rock outcropping motions are not widely available; thus, in practice SRA is often used to derive the input motions using a deconvolution approach.

Cabas & Rodriguez-Marek (2017) proposed a framework to implement V_S - $\Delta\kappa_0$ corrections to obtain bedrock motions from surface motions (i.e., a simple alternative to deconvolution approaches; see Figure 30 and 31). Two main correction factors are included in the V_S - $\Delta\kappa_0$ correction, i.e., an impedance correction factor (C_{imp}) and an attenuation correction factor (C_{atn}), which were based on the square-root-impedance (SRI) method (Joyner et al., 1981; Boore 2013). These factors can be estimated based on site information such as V_S and density profiles at the sites of interest. The V_S - $\Delta\kappa_0$ correction framework only accounts for first order linear elastic site effects.

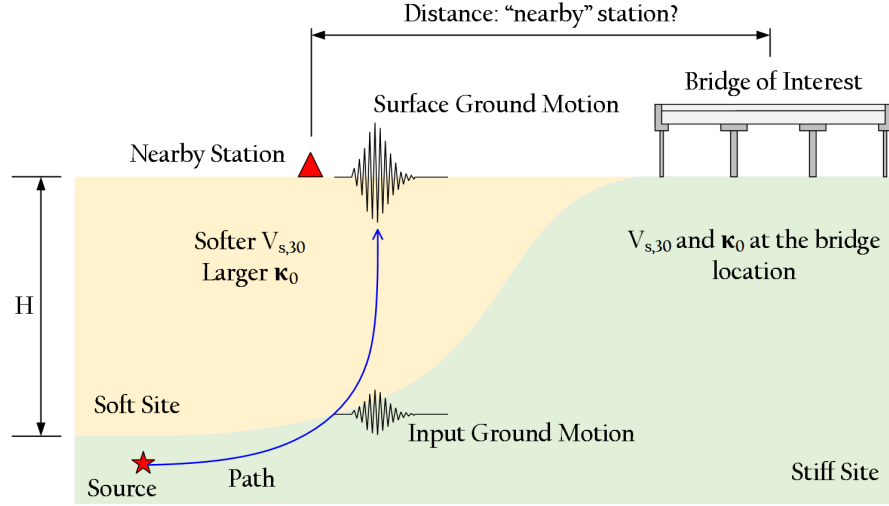


Figure 30. The intention of V_s - $\Delta\kappa_0$ corrections with a candidate ground motion nearby the site (bridge) of interest

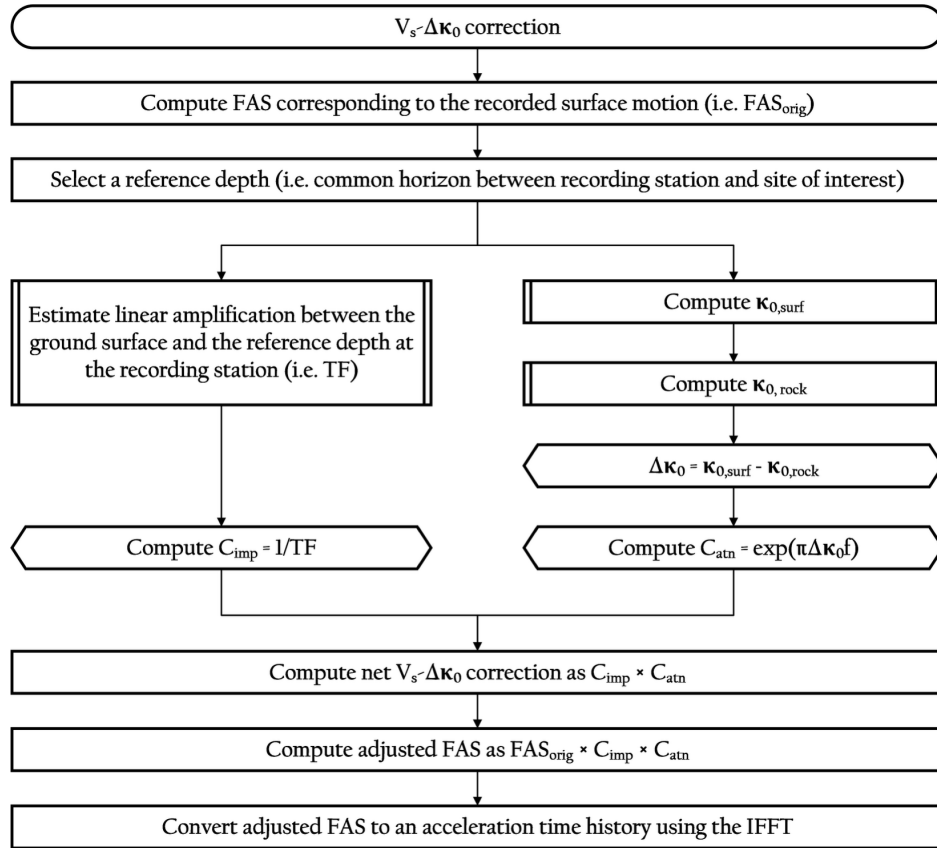


Figure 31. Framework of V_s - $\Delta\kappa_0$ correction (after Cabas & Rodriguez-Marek, 2017)

Impedance correction factor, C_{imp}

“The impedance correction factor (CF_{imp}) can be defined by estimating the linear amplification (i.e., the transfer function, TF) between the ground surface and the selected reference depth at the station where the surface motion to be adjusted was recorded. Such reference depth should be conveniently chosen as a common horizon between the station and the site of interest” (Cabas & Rodriguez-Marek, 2017). The C_{imp} can be computed by the reciprocal of transfer function (TF), a ratio of amplitudes at any two points in the soil layer (e.g. at ground surface and reference site). The equation of TF is as follows,

$$TF = \left(\frac{Z_R}{\bar{Z}} \right)^{\frac{1}{2}} = \left(\frac{\rho_R \cdot V_R}{\bar{\rho} \cdot \bar{V}} \right)^{\frac{1}{2}}$$

in which Z_R = the seismic impedance at the reference depth (the product of the density and seismic wave propagation velocity of the medium); $\bar{Z}$ = the average seismic impedance; ρ_R = the density at the reference depth; V_R = the V_S at the reference depth; $\bar{\rho}$ = the average density over a given depth; and $\bar{V}$ = the V_S over a given depth. Each frequency depending on the depth can be defined by the quarter wavelength (QWL) method, which was initially proposed by Joyner et al. (1981). More details on its implementation can be found in Boore (2003). Finally, C_{imp} is obtained as follows,

$$C_{imp} = \frac{1}{TF}$$

Attenuation correction factor, C_{atn}

The attenuation correction factor, C_{atn} , is computed using the spectral decay parameter, kappa (κ ; Anderson & Hough (1984). Its site-specific component, κ_0 , is commonly understood as a site term that captures the attenuation experienced by seismic waves as they travel through near-surface

materials (Ktenidou et al., 2015). κ_0 can be computed using records at the ground surface (i.e., κ_0 at surface ($\kappa_{0,surf}$)), at depth (i.e., κ_0 at the base rock ($\kappa_{0,rock}$)). Additionally, the contribution from a given sedimentary column ($\Delta\kappa_0$) can be estimated as follows (Cabas & Rodriguez-Marek, 2017):

$$\kappa_{0,surf} = \Delta\kappa_0 + \kappa_{0,rock}$$

C_{atn} represents the seismic attenuation that the seismic waves experience between the base rock to the ground surface. Using $\Delta\kappa_0$, the C_{atn} can be calculated as below,

$$C_{atn} = \exp(-\pi\Delta\kappa_0 f)$$

where f is frequency.

Campbell (2009)

Estimating $\Delta\kappa_0$ corresponding to the soil column at a site of interest is a function of the quality factor, Q , and shear wave velocity, V_s (Campbell, 2009). The quality factor, is directly related to small-strain damping, ξ_{min} . Approaches to estimate ξ_{min} include: (1) laboratory test-based ξ_{min} (e.g., Darendeli 2001), and (2) assuming that Q is proportional to V_s (Cabas et al., 2017, Campbell 2009). Values of $\Delta\kappa_{meas}$ were proposed by (Cabas et al., (2017) using the small-strain damping (ξ_{min}) and the quality factor calculated with a proportionality constant, γ , depending on the site conditions (Silva & Darragh, 1995) and shear wave velocity, V_s :

$$\Delta\kappa_{meas} = \int_0^H \frac{1}{\gamma V_s^2(z)} dz$$

in which V_s is shear wave velocity in each layer; and γ refers to a proportional constant depending on the soil condition (soft soil = 0.007, hard soil = 0.029).

Modifications to ground motion records using the $V_S\text{-}\Delta\kappa_0$ correction are proposed herein when ground motions from the event of interest are not available at the bridge site or nearby. When enough subsurface characterization is available (namely V_{S30} values), this adjustment framework can be used to incorporate first order site effects (i.e., impedance and attenuation effects) into the characterization of the seismic demand of bridges in Alaska.

CHAPTER 3: DIRECT DISPLACEMENT-BASED ASSESSMENT METHOD ON MULTI-SPAN BRIDGE INVENTORIES

To accurately predict the performance of a specific bridge following an earthquake event, it is essential to obtain detailed information about both the structure and the seismic event. For the structural component, it is crucial to obtain the force-displacement response of key bridge elements, such as columns, abutments, and superstructure.

The proposed method compares the capacity of the bridge and the seismic demand. The capacity of the structure is represented by the pushover curve and corresponding performance spectra, and the seismic demand is represented by the variable demand spectra. The intersection between performance spectra and variable demand spectra represents the performance point.

The performance spectra characterize the reduction of stiffness experienced by structures as they are pushed into the nonlinear range. Meanwhile, the variable demand spectra quantifies the increase in energy dissipation, and corresponding reduction in demand. The variable demand spectra is obtained by including the properties of the performance spectra into the 5%-damped displacement spectra. The derivation of both the performance spectra and the variable demand spectra will be explained in detail in this chapter. A graphical representation of this procedure is provided in Figure 32.

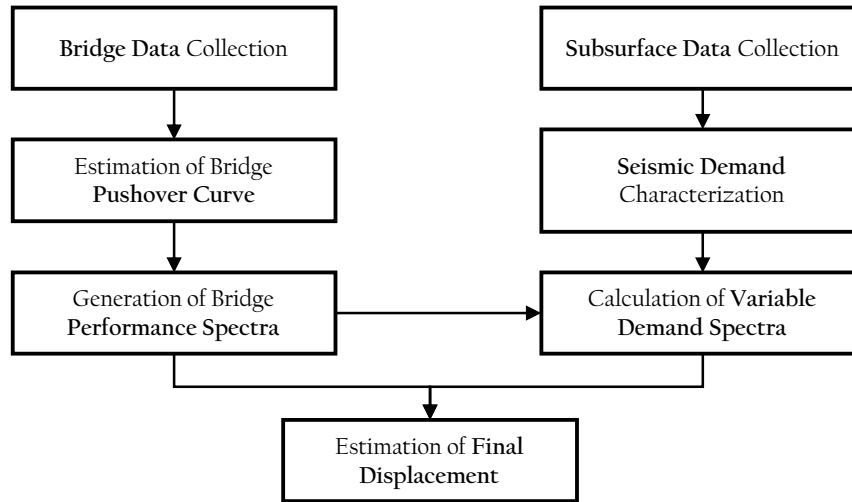


Figure 32. Flowchart of the proposed methodology.

In subsequent sections, each of the steps of the methodology are described in detail.

3.1 Bridge Data Collection

Bridge data collected in this step is used for **(1)** Calculation of the force-displacement response of columns, **(2)** Demand characterization as impacted by bridge location relative to the earthquake source under consideration, **(3)** Other bridge information necessary for calculation of performance spectra, variable demand spectra, and conduct of the analysis.

(1) For calculation of the force-displacement response of columns, section properties – such as section diameter, cover to longitudinal bars, quantity and diameter of longitudinal reinforcement, transverse reinforcement diameter, transverse steel spacing – **must** be obtained. Additionally, member information, such as column height, and material properties such as concrete compressive strength, reinforcing steel yield and ultimate strength are necessary. Alternatively, if the force-displacement response of the columns is directly provided, specifying at least two points – equivalent yield and ultimate – may be sufficient, eliminating the need to include detailed geometric and reinforcement

characteristics in the database. While this simplification expedites database construction, it is important to note that it may impact the accuracy of strain-based displacement limit state estimations.

(2) For demand characterization, bridge location (latitude and longitude) must be identified.

(3) For calculation of performance spectra, variable demand spectra, and analysis, it is important to collect information such as number and length of spans, number of columns per pier, cap beam length, superstructure moment of inertia around the vertical axis and weight per length. Additionally, equivalent yield strength and displacement of the abutments are essential.

3.2 Subsurface Data Collection

Subsurface data collected in this step is used for incorporation of site effects into the characterization of the seismic demand of bridge sites of interest. Shear wave velocity measurements available in Alaska were described in Chapter 2. Unfortunately, only 106 (36+15+55; if all papers characterized independent sites; fewer sites if we have the same site characterized by multiple people) individual V_s profiles have been measured in the state of Alaska with most of those being located in Anchorage and Fairbanks. Some geotechnical reports at bridge locations were kindly provided by AKDOT personnel in early stages of this project, but these reports need to be digitized and not always provide deep V_s profiles at the locations of interest. Geologic maps of the state of Alaska were obtained from Wilson et al. (2012) and preferred V_{s30} values at recording stations from the seismic networks managed by the USGS and the Alaska Earthquake Center were reviewed. Table 4 represents site and geology information at selected bridges. Soil and geotechnical engineering data that are not conveyed within the geotechnical reports will be collected from previous studies such as Martirosyan et al. (2002), Ahdi, Stewart, Ancheta, et al. (2017), Ahdi, Stewart, Kwak, et al. (2017), and Geyin and Maurer (2022).

Table 4. Site and geology information at selected bridges

Bridge Number	Proxy-based V_{S30} [m/s]	Geologic age	Geologic units
526	329.6	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
537	458.4	Early Jurassic (Jhg)	Plutonic rocks of Hidden terrane
595	361.9	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
610	389.2	Upper Cretaceous (Kaf)	Chugach flysch
639	401.8	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
646	410.7	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
654	530.9	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
663	252.5	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
1208	312.0	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
1350	282.7	Quaternary (Qs)	Unconsolidated surficial deposits, undivided
2081	289.3	Quaternary (Qs)	Unconsolidated surficial deposits, undivided

3.3 Estimation of Bridge Pushover Curve

Once the force-displacement response of each column and abutment has been determined from *Step 1: Bridge Data Collection*, the overall structural behavior of the bridge can be represented by a pushover curve. This curve plots the system displacement of the MDOF structure against the total base shear (as defined in Figure 16). As the pushover curve progresses, each point on the curve corresponds to a distinct stage in the bridge's response, characterized by a unique displacement profile. These profiles capture how the structure deforms under increasing lateral load.

As the bridge displacement profile changes, its equivalent properties – such as displacement, stiffness, mass, and base shear – vary accordingly. The displacement profile represents the actual displacements experienced by the bridge. The *displaced shape*, introduced here, is derived by normalizing these

displacements with respect to the pier with maximum displacement (δ_{crit}), providing a dimensionless representation of the displaced shape. For example, in a two-span bridge, if both abutments displace 10 mm, and pier 1 columns displace 200 mm, then the displaced shape would follow a 0.05 - 1.00 - 0.05 pattern. The difference between displaced shape and displacement profile is shown in Figure 33.

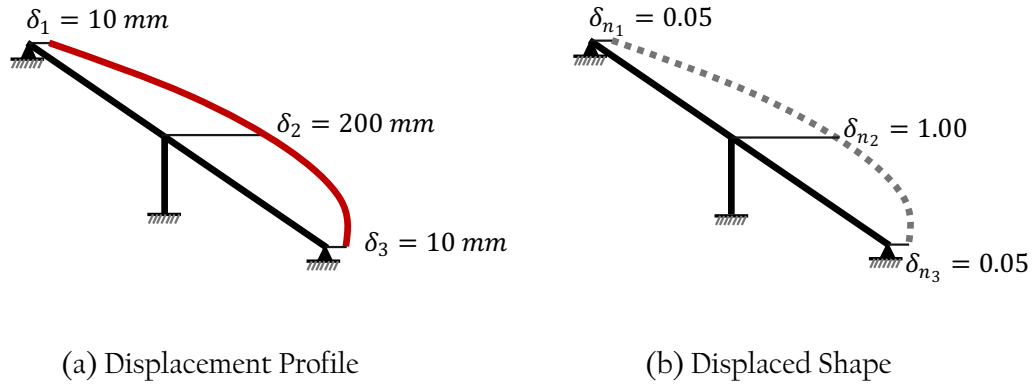


Figure 33. (a) Displacement profile and (b) Displaced shape related to the example stated above

However, accurately determining the displacement profile associated with each step in the pushover curve analysis requires an iterative process. This process is detailed in the following paragraphs.

- A displacement for the critical column in the bridge (δ_{crit}) must be assumed and a displaced shape with normalized displacements (δ_{n_i}) must be selected. Based on these assumptions, the displacement profile – and consequently the displacement of each column and abutment (δ_i) – can be obtained by the following:

$$\delta_i = \delta_{crit} \cdot \delta_{n_i} \quad (5)$$

Given that the inertial mass of each pier is known, the system displacement (Δ_{sys}) can be calculated as follows:

$$\Delta_{sys} = \frac{\sum_{i=1}^n m_i \delta_i^2}{\sum_{i=1}^n m_i \delta_i} \quad (6)$$

where n is the number of inertial mass locations, and m_i is the inertial mass associated with each i^{th} location.

- The next step is to compute the force (V_i) in the columns and abutments corresponding to their displacement (δ_i), based on their force-displacement response. Using the obtained force and displacement values, the secant stiffness (K_{sec_i}) of each column and abutment can be determined as follows:

$$K_{sec_i} = \frac{V_i}{\delta_i} \quad (7)$$

- The total base shear (the equivalent base shear for the SDOF system) is obtained by:

$$V_b = \sum_{i=1}^n V_i \quad (8)$$

- Then, the total base shear is distributed to the inertia mass locations, as proposed by M. J. N. Priestley et al. (2007):

$$F_i = V_b \cdot \frac{m_i \delta_i}{\sum_{i=1}^n m_i \delta_i} \quad (9)$$

- The next step involves performing a structural analysis to determine a new static displaced shape. This is done by applying F_i to the corresponding i^{th} inertia mass locations and assigning the associated secant stiffness K_{sec_i} as springs to these locations. A revised value of system displacement ($\Delta_{sys_{rev}}$) is obtained.

Finally, the displacements Δ_{sys} and $\Delta_{sys_{rev}}$ are compared. If convergence is not achieved, a new interaction begins, with the new displaced shape from the structural analysis serving as the updated assumed shape. This iteration process continues until convergence is reached, at which point the system displacement and total base shear can be determined. A graphical representation of the process is shown in Figure 34.

This procedure corresponds to a single point on the pushover curve. For that reason, to increase precision of the analysis, this iteration must be repeated for a sufficiently large number k of displacement profiles.

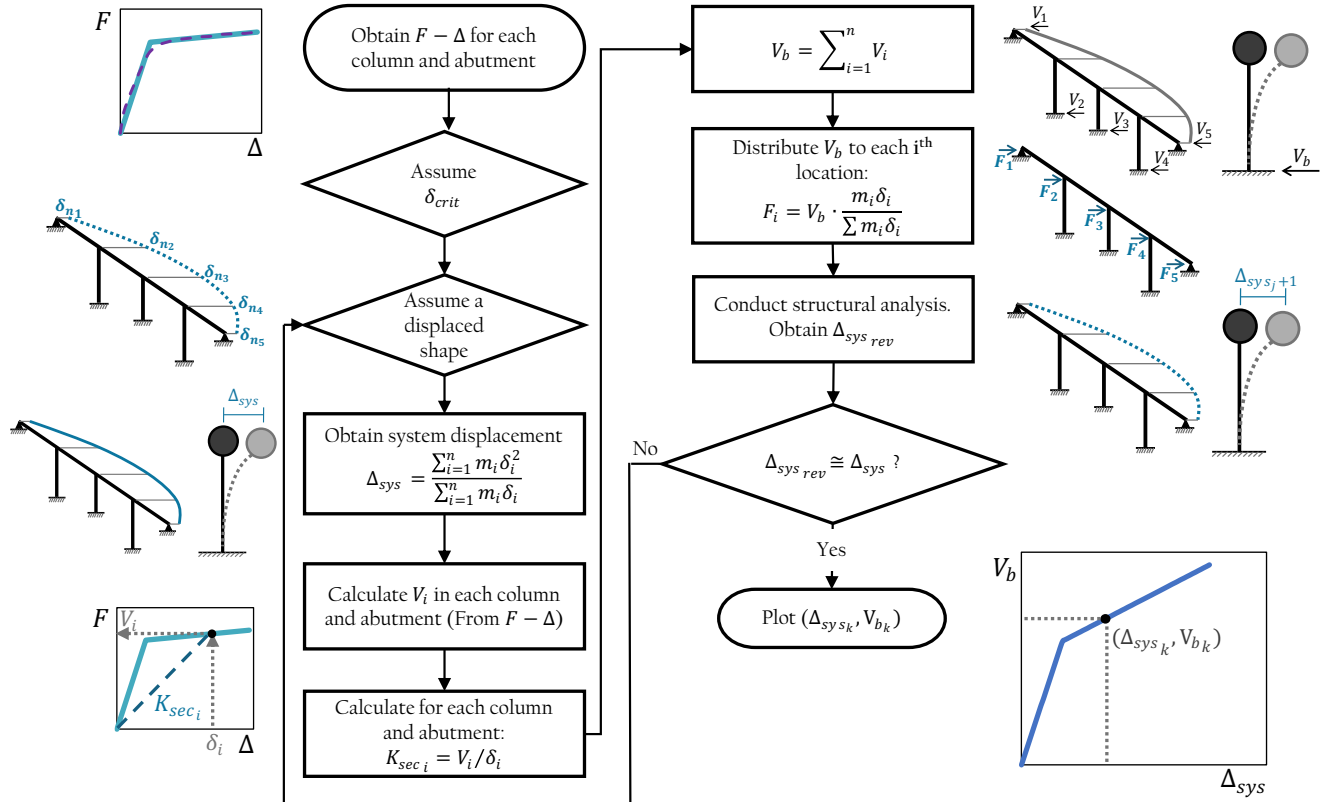


Figure 34. Flowchart to calculate total base shear and system displacement of a specific displacement profile for an MDOF bridge system.

3.4 Generation of Bridge Performance Spectra

This step results in a graphical representation of the bridge's performance. The performance spectra is defined as a unique structural representation, where the variation of the effective period (T_{eff}) of an equivalent SDOF system is plotted as a function of its system displacement (Δ_{sys}).

For all k pair of values obtained from the previous step (Δ_{sys} and V_b), an effective stiffness related to the system can be calculated as follows:

$$K_{eff_k} = \frac{V_{b_k}}{\Delta_{sys_k}} \quad (10)$$

Additionally, from the system displacement and displaced shape, the effective mass (m_{eff}) of the system is obtained from the following expression:

$$m_{eff_k} = \frac{\sum_{i=1}^n m_i \delta_i}{\Delta_{sys_k}} \quad (11)$$

The calculation of effective mass accounts for the mass participating in the inelastic first mode of vibration, and the force equilibrium between the real MDOF system and idealized SDOF system, according to M. J. N. Priestley et al. (2007).

With effective mass and effective stiffness, one is able to obtain the effective period (T_{eff}) as follows:

$$T_{eff_k} = 2\pi \sqrt{\frac{m_{eff_k}}{K_{eff_k}}} \quad (12)$$

Then, we have a pair of values T_{eff_k} and Δ_{sys_k} . Plotting all the discrete pairs of values results in the performance spectra. A graphical representation of the outlined procedure is shown in Figure 35.

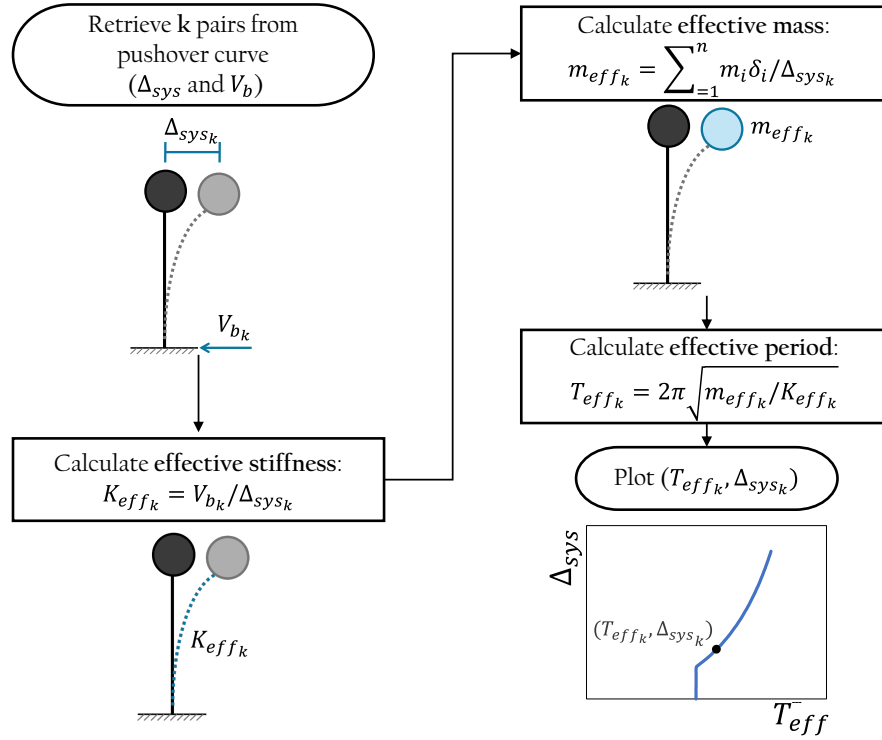


Figure 35. Flowchart of the procedure to obtain the performance spectra

3.5 Seismic Demand Characterization

Figure 36 represents the workflow to characterize the seismic demand within the proposed rapid assessment framework. In this step, the seismic demand would represent a real or hypothetical earthquake scenario. Information such as moment magnitude (M_w), site-to-source distance and local site conditions (e.g., V_{s30}) are critical to consider. In this study, we characterize the seismic demand via 5%-damped elastic displacement response spectra considering site effects.

For a given seismic event and bridge in the inventory, engineers first determine whether ground motion records at the site are available. When recorded ground motions at (or nearby) the site are available, the displacement response spectra can be computed directly from the records (particularly if the nearby station shares the same geology as the site of interest) by solving the equation of motion

for the original recording (or adjusted record via V_S - $\Delta\kappa_0$ corrections if enough subsurface characterization is available).

When ground motion recordings from nearby stations are not available (which is the case for 808 of 1403 bridges in Alaska; <https://gis.data.alaska.gov/datasets/AKDOT::bridges-akdot-1/about>, last accessed September 10th, 2024), the PGA, PGV, and PSA at 0.3, 1, and 3 second can be obtained from the corresponding ShakeMap generated by the USGS, or the implementation of an appropriate GMM using proxy-based V_{S30} (if a measured value is not available, which is the case in most of the state except for Anchorage), or from a physics-based broadband ground-motion simulations. Unfortunately, the latter option is still an area of research, and the main elements required for the simulation (i.e., crustal velocity models and source rupture models, Baker et al., 2021) of Alaska events are not readily available.

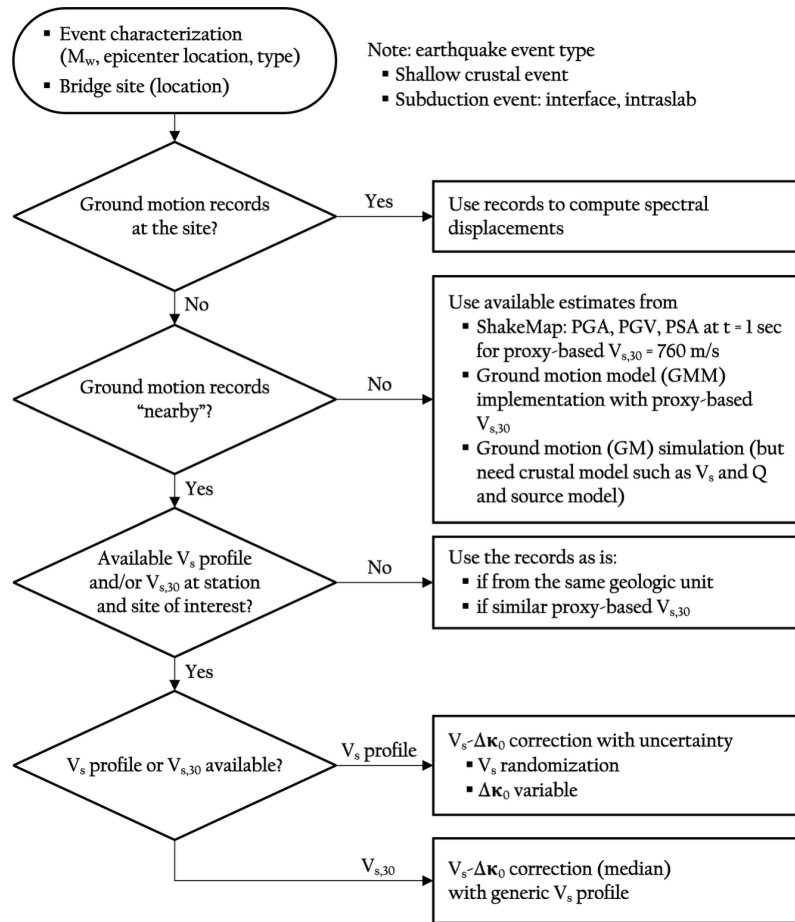


Figure 36. Framework to estimate the seismic demand incorporating the site effects

GMMs for shallow crustal and subduction events are proposed to estimate spectral displacements (i.e., the seismic demand) in accordance with each tectonic regime. In Alaska, because both shallow crustal and subduction events can take place, GMMs for both types of event are considered. Additionally, local site conditions should be considered because the complexity of shallow and deep geologic structures in Alaska can contribute to the variability of the amplitude, frequency content, and duration of ground motions (Seed & Idriss, 1971; Borchardt, 1994; Kramer, 1996; Cabas et al., 2021). Figure 37 shows a flowchart depicting the proposed ground-motion model characterization

process when a GMM is needed. The GMMs are categorized by the type of seismicity (i.e. shallow crustal and subduction zone events) and the site parameter, V_{S30} that is commonly used for GMMs.

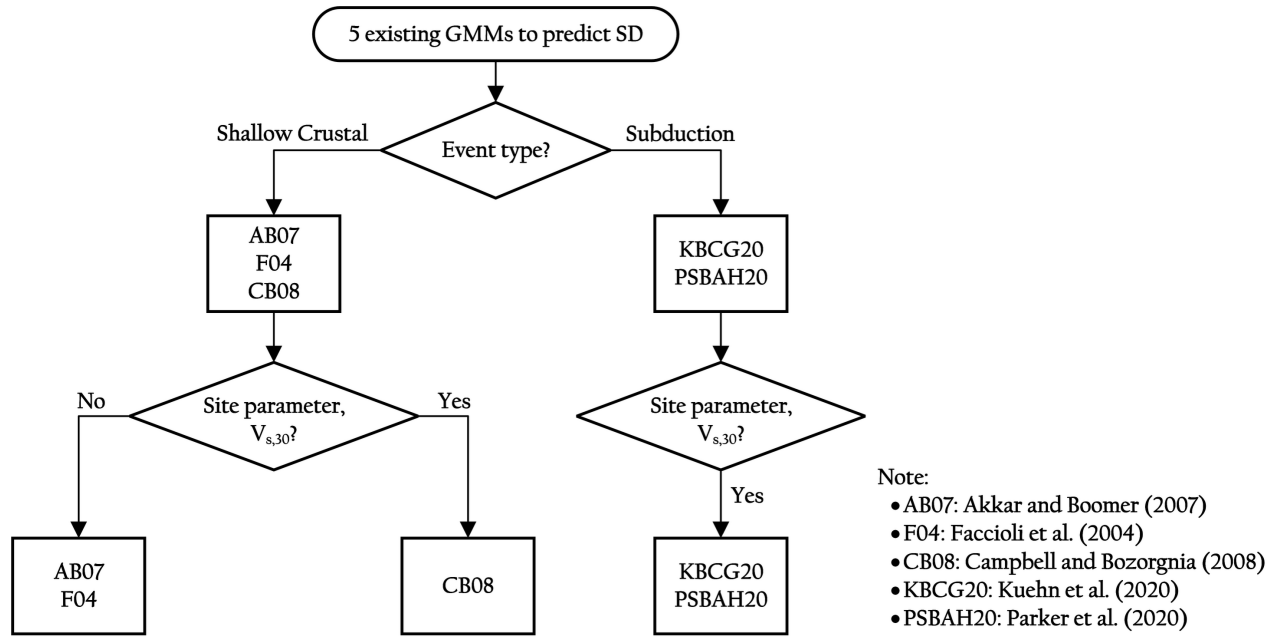


Figure 37. Proposed GMM selection workflow for Alaska. Note that the NGA-West2 GMMs can also be used in case of shallow crustal regimes. The GMMs shown in that branch of the workflow correspond to available GMM that directly provide values of SD instead of PSA. Conversions to SD using the natural angular frequency could be used in conjunction with PSA values from NGA-West2 models as well.

When V_{S30} is included as a site parameter in any of the GMMs shown in Figure 36, using a measured value at the site of interest is preferred. However, proxy-based values (of the nature described in Chapter 2) can provide a reasonable estimation of V_{S30} in the absence of data. The GMMs shown in that branch of the workflow correspond to available GMM that directly provide values of SD instead of PSA. Conversions to SD using the natural angular frequency could be used in conjunction with PSA values from NGA-West2 (Bozorgnia et al., 2014) models as well.

When engineers can obtain recorded ground motions at nearby stations, but site metadata, such as V_S profiles or V_{S30} at such stations and the nearby site of interest, are not available, the ground-motion

recording can be used assuming that the bridge location and the nearby recording station share the same geologic conditions and/or have similar proxy-based V_{S30} . “Nearby” stations are commonly described with a separation distance of 10 km, which is commonly used for studies of spatial correlation of ground motions (e.g., Jayaram & Baker, 2009). If local geologic conditions at the bridge site and the nearby recording station are not similar (as deemed by the analyst), using the nearby ground-motion as-recorded may lead to bias in the analysis of the bridge.

If enough subsurface data are available at the bridge site of interest and the nearby station, V_S - $\Delta\kappa_0$ correction can be implemented to account for potential differences in the site response of both sites and obtain an unbiased seismic demand at the bridge location (e.g., Jayaram & Baker, 2009). The V_S - $\Delta\kappa_0$ correction framework with uncertainty can only be implemented when V_S profiles are available. Uncertainty will be computed by estimating randomized V_S profiles via Toro’s randomization method (Toro, 1995, 2022) and using multiple values of $\Delta\kappa_0$. If a full V_S profile is not available, but values of V_{S30} are measured or can be estimated, the V_{S30} can be used to estimate a generic V_S profile at the site of interest. Only the median response spectra will be computed, and uncertainties will not be considered.

3.6 Calculation of Variable Demand Spectra

This step introduces a new representation of seismic demand. Building upon the previous step, in which the 5%-damped displacement spectra was obtained, this formulation accounts for the nonlinear behavior of certain structural components, such as columns, in the MDOF system. As lateral deformations increase, these elements exhibit higher ductility demands. The displacement ductility of a member is obtained as:

$$\mu_{\Delta_i} = \frac{\delta_i}{\Delta_{y_i}} \geq 1.0 \quad (13)$$

where Δ_{y_i} is the equivalent yield displacement of the member.

Given the displacement ductility for each structural member, its corresponding equivalent viscous damping (ξ_{eq_i}) can be determined using Equation (14) (M. J. N. Priestley et al., 2007). This equation assumes elastic tangent stiffness damping ratio of 0.05 and the Takeda Thin hysteretic rule, which effectively represents the response of RC bridges with well-confined sections.

$$\xi_{eq_i} = 0.05 + 0.444(\mu_{\Delta_i} - 1)/\mu_{\Delta_i}\pi \quad (14)$$

Once the equivalent viscous damping of each element is estimated, the value representing the entire system can be obtained by weighing the member damping in proportion to the work done by each member in accordance with the recommendations of M. J. N. Priestley et al. (2007).

$$\xi_{sys_k} = \frac{\sum_{i=1}^n V_i \delta_i \xi_{eq_i}}{\sum_{i=1}^n V_i \delta_i} \quad (15)$$

Each k pair of values from the system's pushover curve will have a system equivalent viscous damping, therefore, there are k pairs of values (T_{eff}, ξ_{sys}). Thus, once we enter the 5% displacement spectra with T_{eff_k} and retrieve a value of spectral displacement (S_{d_k}), this value needs to be scaled by its proper system equivalent viscous damping. That process is done as follows:

$$S_{d_{vk}} = R_{\xi_k} \cdot S_{d_k} \quad (16)$$

Where $S_{d_{vk}}$ is the scaled spectral displacement, and R_{ξ_k} is the spectral reduction factor. The following table shows some of the equations suggested by the literature to be used for obtaining the spectral reduction factor based on the purpose of this research:

Table 5. Equations for obtaining values of spectral reduction factor.

Proposed by:	Spectral Reduction Factor:	
Eurocode 8 (1994) ¹	$R_{\xi} = \sqrt{7/(2 + \xi)}$	$0.2 \text{ s} < T < 6 \text{ s}$
Eurocode 8 (2004) ²	$R_{\xi} = \sqrt{10/(5 + \xi)} \geq 0.55$	$0.2 \text{ s} < T < 0.75 \text{ s}$
Kong & Kowalsky (2016) ³	$R_{\xi} = \left(\frac{12}{7+\xi}\right)^{0.5} - \frac{M\sqrt{\xi}}{500} + \frac{7.6-M}{3+30\left(\frac{12}{7+\xi}\right)^{1.5}} \log_{10} 0.75$	$0.2 \text{ s} < T < 0.75 \text{ s}$
	$R_{\xi} = \left(\frac{12}{7+\xi}\right)^{0.5} - \frac{M\sqrt{\xi}}{500} + \frac{7.6-M}{3+30\left(\frac{12}{7+\xi}\right)^{1.5}} \log_{10} T$	$0.75 \text{ s} \leq T < 10 \text{ s}$
AASHTO (2011) ⁴	$R_{\xi} = (5/\xi)^{0.4}$	

^{1,2,3,4}: Values of damping ratio must be expressed in percentage.

³: M is the moment magnitude of the event, and T is the structure's period.

As we can observe, for three models, the damping ratio is the only variable (AASHTO, 2011; Eurocode 8, 1994; Eurocode 8, 2004). However, it is known that spectral reduction factors are function of rupture distance, moment magnitude, and period of the structure.

A study done by Kong & Kowalsky (2016), illustrated that the Eurocode 8 (1994) model was shown to be reasonable for large magnitude events; however, error increases as the magnitude decreases, particularly with longer period structures. Overall, the Kong & Kowalsky (2016) model showed good agreement across a moment magnitude range of 5.5 to 8.

Once the scaled spectral displacement is calculated, a new k pair of values (T_{effk}, S_{dv_k}) is obtained.

A variable demand spectra is then defined as a graphical representation of the demand spectra wherein each period is associated with a damped value, that is the resultant of a 5%-damped displacement spectra scaled by its correspondent spectral reduction factor. A graphical representation of the procedure is shown in Figure 38.

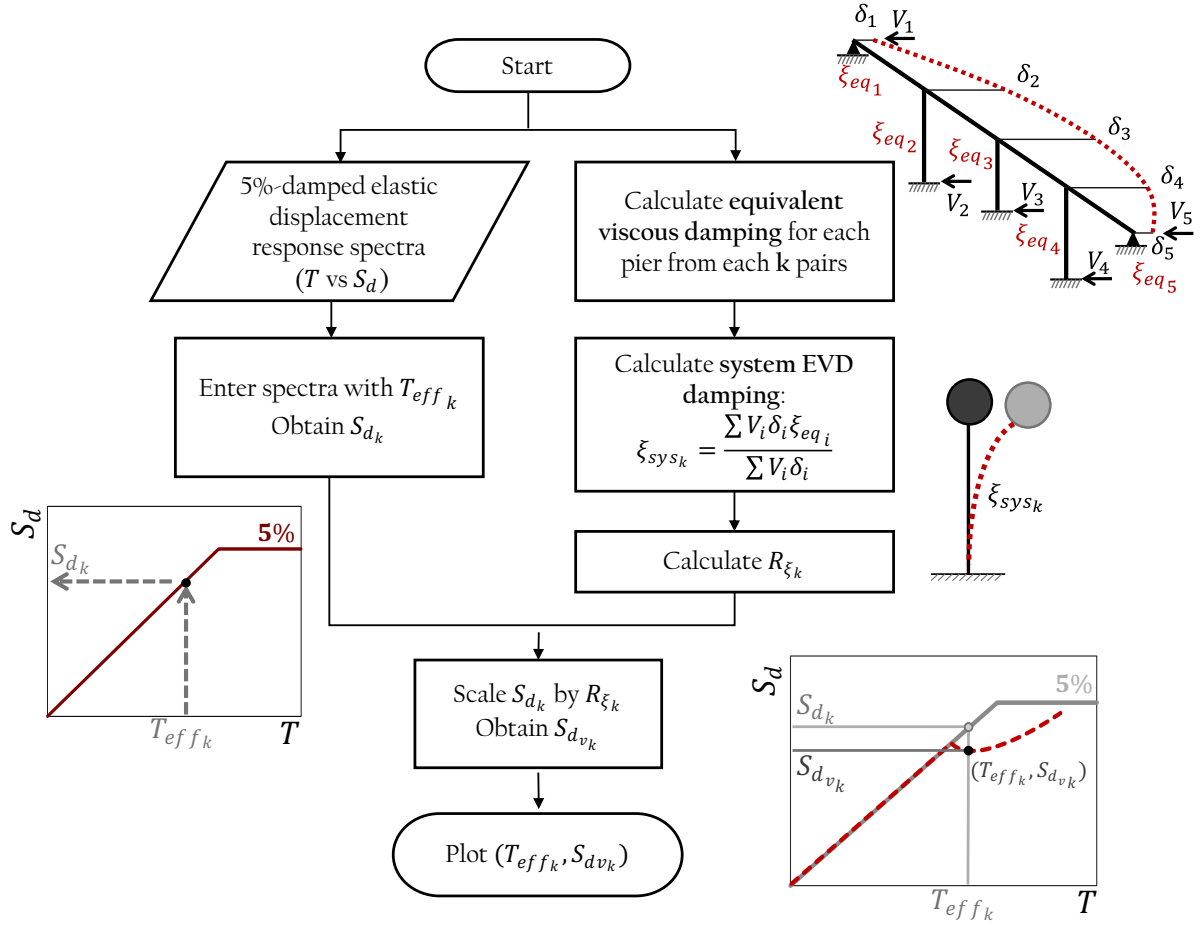


Figure 38. Flowchart of the procedure to obtain the variable demand spectra.

3.7 Estimation of Final Displacement

In this final step, the bridge's final displacement under the specific seismic demand is predicted. This displacement corresponds to the intersection of the performance spectra and the variable demand spectra, representing the system's expected response to the given seismic conditions, as shown in Figure 39.

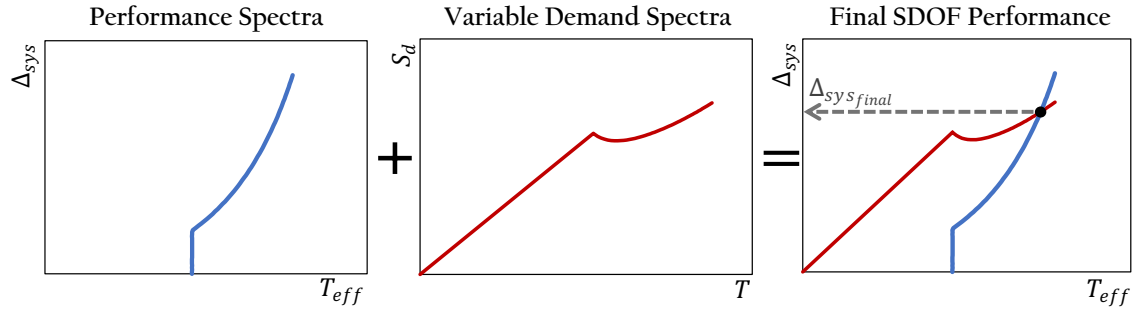


Figure 39. Illustration of the process for combining performance spectra and variable demand spectra to determine the final system displacement, representing the structure's response to the specified seismic demand.

As previously mentioned, the system displacement is obtained through Equation (6) which assumes an equivalence in work between the real MDOF and equivalent SDOF system (Priestley et al., 2007). In order to obtain the displacement in each bridge pier, the final displaced shape must be scaled by the final system displacement.

CHAPTER 4: RAPID POST-EARTHQUAKE ASSESSMENT IMPLEMENTATION

In this chapter, we present an example application of the proposed methodology using two bridges from the AKDOT&PF inventory: 610 and 646. An additional nine bridges are analyzed in Appendix A as part of the broader assessment process.

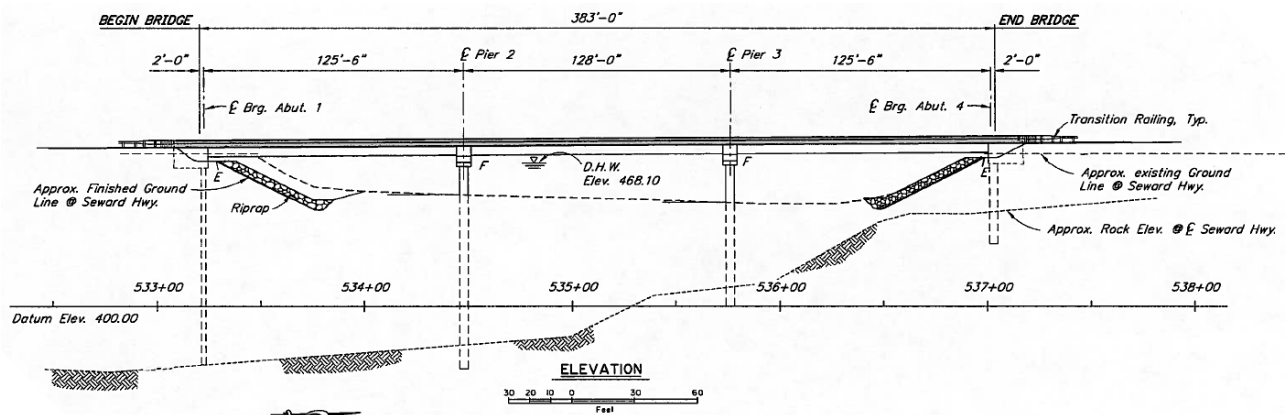
It is important to highlight that certain assumptions and simplifications were made to the structures and input ground motions during the analysis. The abutments were assumed to have an elasto-plastic behavior, with the yield displacement being 40 mm and the yield strength, 28400 kN. Although all eleven bridges chosen from the database have RCFST columns, they were treated as RC columns with pile-supported footings in this study. Consequently, the contribution of the steel tube was excluded from the analysis. The column height was defined as the distance from the bottom of the cap beam to ground level, plus an additional length equal to twice the column's diameter. This accounts for the region between ground level and the expected location of maximum bending moment, typically within one to two times the diameter, as described by Priestley et al. (1996). Refer to Figure 62 for visual guidance. Additionally, the cross-section details are the same for all columns in the bridge. Assumptions pertaining to the ground motions used in the example analyses are described in detail below.

1. Bridge Data Collection

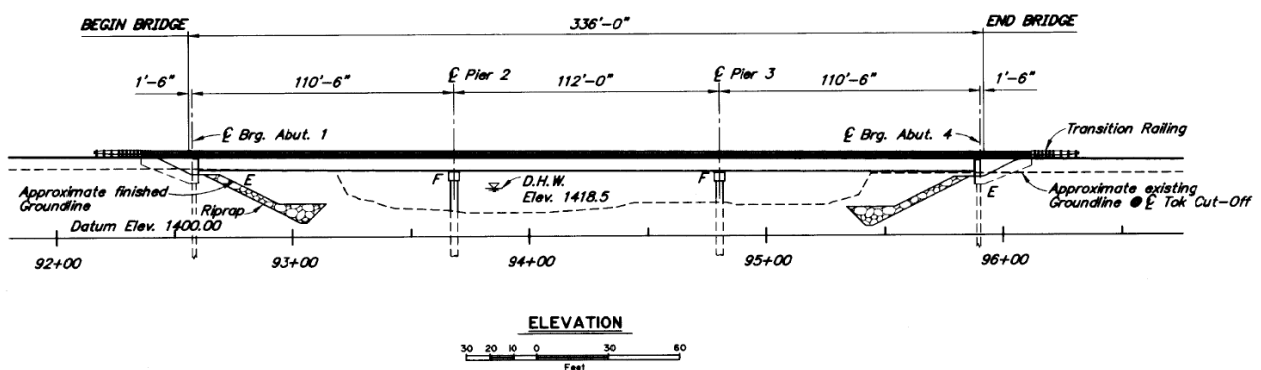
In this step, a comprehensive database is developed for each of the two selected bridges. The database includes the following information: bridge identification number; geographic location (latitude and longitude); number and length of spans; materials properties, such as the yield strength of longitudinal and transverse reinforcement, ultimate strength of longitudinal reinforcement, concrete compressive strength, and concrete modulus of elasticity. Superstructure properties are also recorded, including

the moment of inertia in the transverse direction and the linear weight of the superstructure. Substructure data are obtained, including the number of columns per pier, cap beam length, column heights in each pier, columns diameter, cover to longitudinal reinforcement, diameter of longitudinal and transverse reinforcement, and spacing of transverse reinforcement. Finally, abutment characteristics are captured, including their yield strength and associated displacement.

Following, the elevation view of the longitudinal profile of the bridges as well as of the typical pier sections are shown in Figure 40 and Figure 41, respectively.



(a) Bridge 610



(b) Bridge 646

Figure 40. Elevation view of Bridges 610 and 646. (Courtesy of AKDOT&PF)

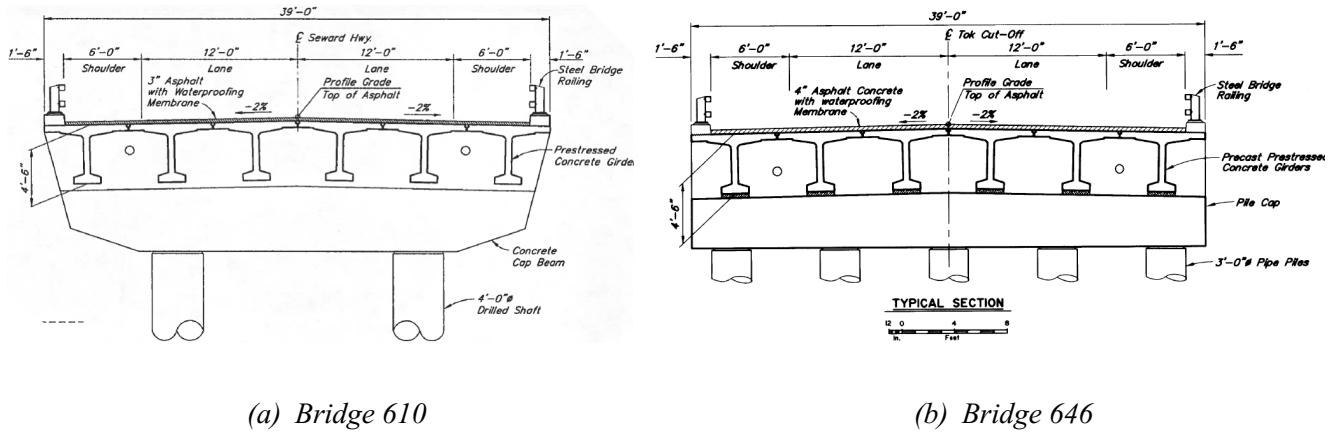


Figure 41. Elevation view of typical pier section of Bridge 610 and 646. (Courtesy of AKDOT&PF)

Following, the database built for Bridges 610 and 646 are presented in Table 6 and Table 7, respectively.

Table 6. Database built for Bridge 610.

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude	Longitude	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	fc (MPa)	Iy (m4)	Weight SS (kN/m)
610	60.4357	-149.37	3	38.25	39.01	38.25	0	420	420	600	28	47.05	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	2	12	7.03	8.31	0	117	50.8	20	32.26	15.875	76.2	28400	40

Table 7. Database built for Bridge 646.

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude	Longitude	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	fc (MPa)	Iy (m4)	Weight SS (kN/m)
646	62.3014	-145.31	3	36.68	34.14	36.68	0	420	420	600	28	48.02	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	5	11.9	6.25	4.83	0	0.88	50.8	16	32.26	15.875	76.2	28400	40

As previously stated, the abutment behavior is assumed to be elasto-plastic. The yield displacement and yield force were kept constant across all bridges in the study. No constraints were imposed on abutment displacement. While this simplification does not fully reflect real-world behavior, it is considered acceptable for the purposes of the present research. The assumed abutment behavior is illustrated in Figure 42, where displacement is allowed to increase indefinitely beyond the yield point.

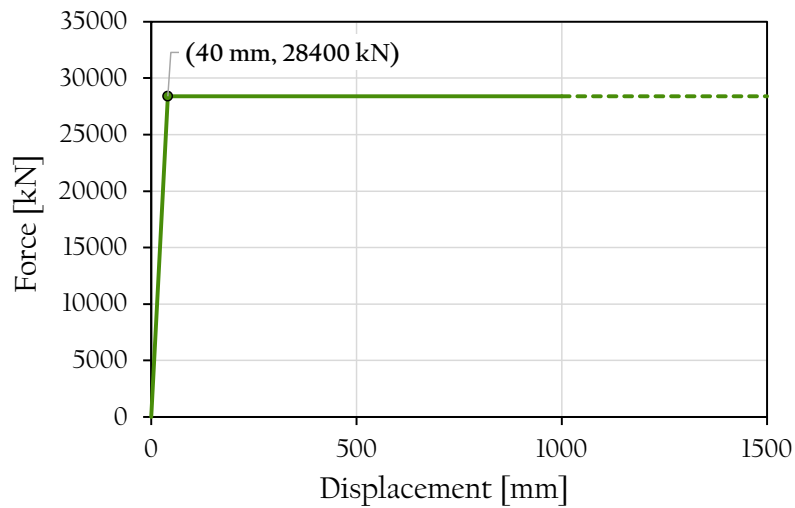


Figure 42. Force-displacement relationship of the abutments

Using this information, the moment-curvature relationship for the columns at each pier can be determined. Although the cross-section geometry is consistent across all columns within a given bridge, variations in column height from one pier to another result in slight differences in the axial load considered during the moment-curvature analysis, as it is computed based on the total height of each column. The moment-curvature relationship for the columns at each pier of both bridge is presented in Figure 43.

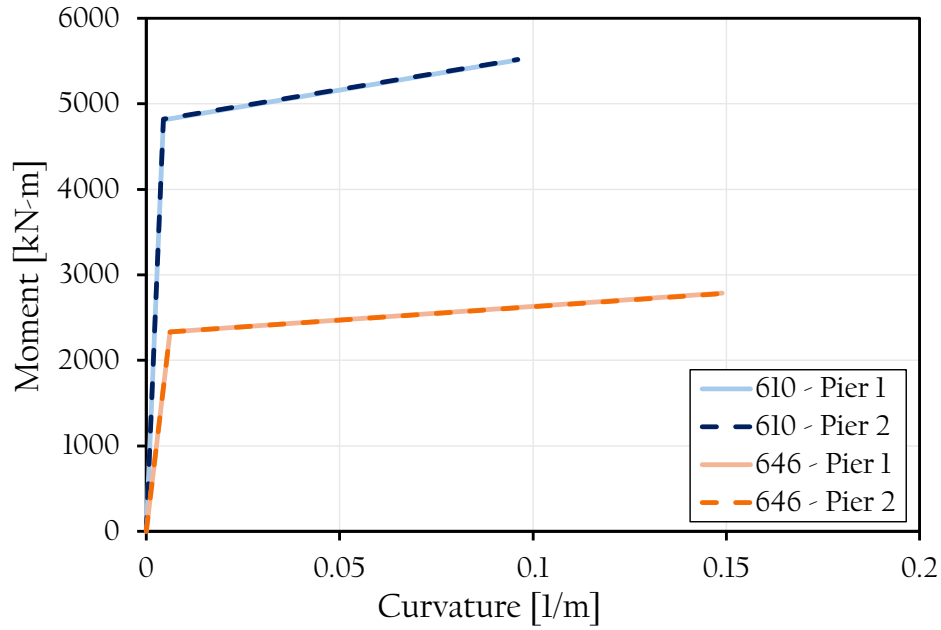


Figure 43. Moment-curvature relationship for bridges 610 and 646.

A summary of the values for the bilinear moment-curvature relationships for bridges 610 and 646 are shown in Table 8.

Table 8. Summary of moment-curvature values for Bridges 610 and 646.

Bridge 610		Bridge 646	
Pier 1			
Curvature [1/m]	Moment [kN-m]	Curvature [1/m]	Moment [kN-m]
0	0	0	0
0.0044	4811	0.0062	2334
0.0961	5513	0.1489	2786
Pier 2			
0	0	0	0
0.0044	4821	0.0062	2330
0.0960	5517	0.1491	2781

Once the moment-curvature relationship is established, the force-displacement response for each column is derived using the Plastic Hinge Method proposed by Priestley et al. (2007), as described in Chapter 2. The resulting force-displacement curves for the columns at each pier of both bridges are presented in Figure 44. Four performance limit states are considered in the analysis - Equivalent

Yield (EY), Fully Operational (PL3), Operational (PL2), and Life Safety (PL1) – and are represented by diamond markers on the plots in that respective order. The corresponding concrete and steel strain limits associated with each limit state are detailed in Chapter 2.

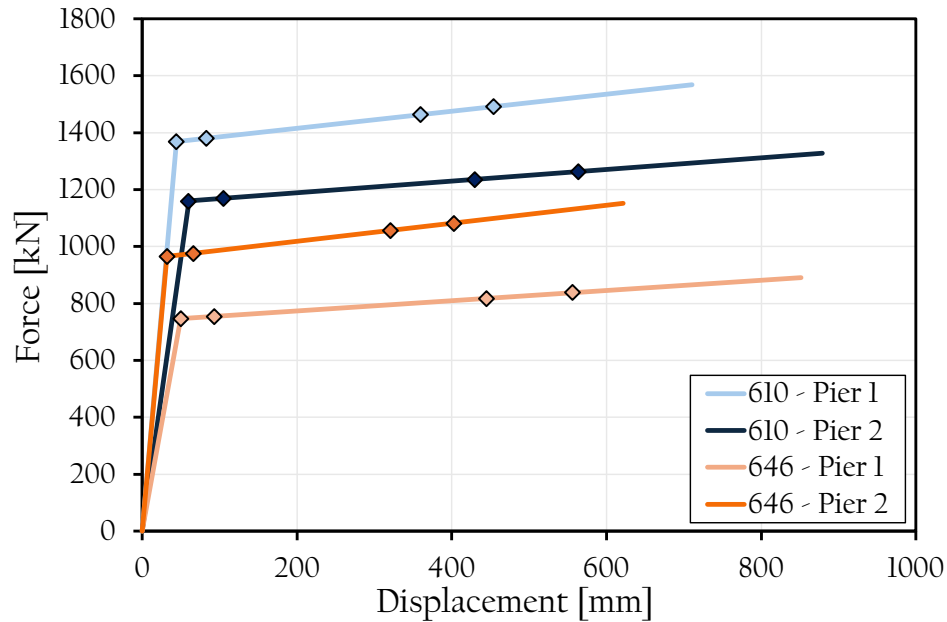


Figure 44. Force-displacement relationship for bridges 610 and 646.

A summary of the values for the bilinear force-displacement relationships for bridges 610 and 646 are shown in Table 9.

Table 9. Summary of force-displacement values for the bilinear relationship for Bridges 610 and 646.

Bridge 610		Bridge 646	
Pier 1			
Displacement [mm]	Force [kN]	Displacement [mm]	Force [kN]
0	0	0	0
44	1369	50	747
711	1569	852	892
Pier 2			
0	0	0	0
60	1160	32	965
879	1328	622	1151

A summary of the displacement values for the four performance limit states chosen for this example for bridges 610 and 646 are shown in Table 10.

Table 10. Summary of limit state displacement values for each pier column for Bridge 610 and 646.

	Bridge 610		Bridge 646	
	Pier 1 Columns	Pier 2 Columns	Pier 1 Columns	Pier 2 Columns
	[mm]	[mm]	[mm]	[mm]
Equivalent Yield (EY)	44	60	50	32
Fully Operational (PL3)	83	105	93	66
Operational (PL2)	360	430	445	321
Life Safety (PL1)	454	564	556	403

2. Subsurface Data Collection

Figure 45 shows the locations of the selected earthquake event and bridges (i.e., 610 and 646) for this example application. More information about the characteristics of the selected seismic events and the corresponding seismic demand will be provided in later sections of this report. The corresponding epicentral distances and the proxy-based V_{S30} values at the selected bridge sites are shown in Table 11.

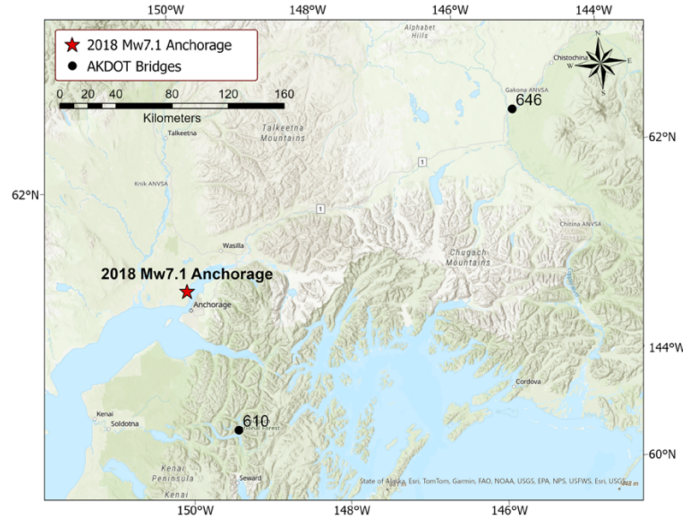


Figure 45. Locations of the earthquake epicenter and selected bridges for these example application of the proposed rapid assessment framework.

Table 11. Epicentral distances and proxy-based V_{S30} at the selected bridges from Geyin & Maurer (2022)

Bridge	Epicentral distance (km)	Proxy-based V_{S30} (m/s; Geyin and Maurer, 2022)	Geologic age
610	105.64	389.2	Upper Cretaceous (Kaf)
646	265.81	410.7	Quaternary (Qs)

There are no measured V_S profiles at the selected sites, which is why only proxy-based V_{S30} values are provided in Table 14. Both bridge sites are located in stiff soil conditions or NEHRP site class C. No additional geotechnical data are available at the study sites.

3. Estimation of Bridge Pushover Curve

To illustrate the procedure, we evaluate the system displacement and total base shear corresponding to the Operational (PL2) performance limit state achieved by the Pier 1 columns from Bridge 646. As described in Chapter 3, the initial step in estimating a specific point on the bridge pushover curve involves determining the displaced shape, which is obtained through an iterative process. For the purpose of this example, we assume that the iteration to determine the displaced shape corresponding

to the Operational limit state achieved by Pier 1 columns of Bridge 646 has been completed. The results are as follows:

$$0.056 - 1.000 - 0.994 - 0.054$$

Considering that $\Delta_{PL2,Pier\ 1} = \delta_2 = 445\ mm$ for Pier 1 columns, the displacement profile is given as follows:

$$25\ mm - 445\ mm - 442\ mm - 24\ mm$$

With that, we then calculate the system displacement:

$$\Delta_{sys} = \frac{\sum_{i=1}^n m_i \delta_i^2}{\sum_{i=1}^n m_i \delta_i}$$

$$\Delta_{sys} = \frac{3301.2 \cdot (25\ mm)^2 + 6532 \cdot (445\ mm)^2 + 6496 \cdot (442\ mm)^2 + 3301.2 \cdot (24\ mm)^2}{3301.2 \cdot 25\ mm + 6532 \cdot 445\ mm + 6496 \cdot 442\ mm + 3301.2 \cdot 24\ mm}$$

$$\Delta_{sys} = 432\ mm$$

The total base shear is calculated as follows:

$$V_b = V_{ab_1} + 5V_{col1} + 5V_{col2} + V_{ab_2}$$

$$V_b = 17750\ kN + 5 \cdot (818\ kN) + 5 \cdot (1111\ kN) + 17040\ kN$$

$$V_b = 44437\ kN$$

The same calculation was performed for all piers of both bridges, therefore, Table 12 and Table 13 summarize the system displacement and total base shear values corresponding to the performance limit states achieved by the columns at each pier and bridge.

Table 12. Summary of the system displacement and total base shear for the four performance limit states for both piers of Bridge 610.

Performance Limit States	Bridge 610			
	Pier 1 Columns		Pier 2 Columns	
	System Displ. [mm]	Total Base Shear [kN]	System Displ. [mm]	Total Base Shear [kN]
Equivalent Yield (EY)	45	7462	58	8965
Fully Operational (PL3)	82	10584	102	11936
Operational (PL2)	353	28613	421	33138
Life Safety (PL1)	446	34792	552	41876

Table 13. Summary of the system displacement and total base shear for the four performance limit states for both piers of Bridge 646.

Performance Limit States	Bridge 646			
	Pier 1 Columns		Pier 2 Columns	
	System Displ. [mm]	Total Base Shear [kN]	System Displ. [mm]	Total Base Shear [kN]
Equivalent Yield (EY)	47	12617	33	10115
Fully Operational (PL3)	89	16072	65	14100
Operational (PL2)	432	44437	315	34709
Life Safety (PL1)	540	53380	393	41269

The pushover curves for both bridges are presented in Figure 46. The figure also includes the four performance limit states identified for each pier's columns.

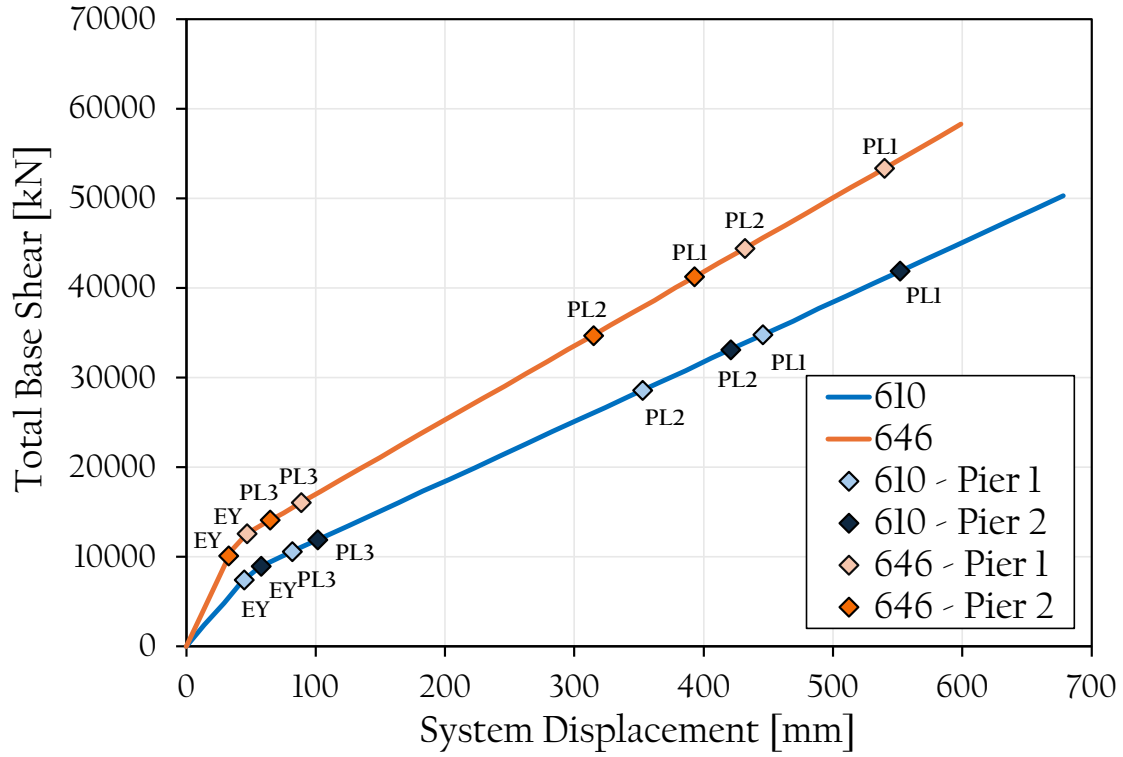


Figure 46. Pushover Curve of Bridge 610 and 646 with the four performance limit states for each pier column.

As part of the generation of the bridge pushover curve, the displaced shapes corresponding to the system displacements are obtained. Figure 47 and Figure 48 illustrate the progression of the displaced shape throughout the iterations – light gray representing the initial iteration values and black the final iteration. Figure 47 corresponds to Bridge 610, while Figure 48 shows the results for Bridge 646.

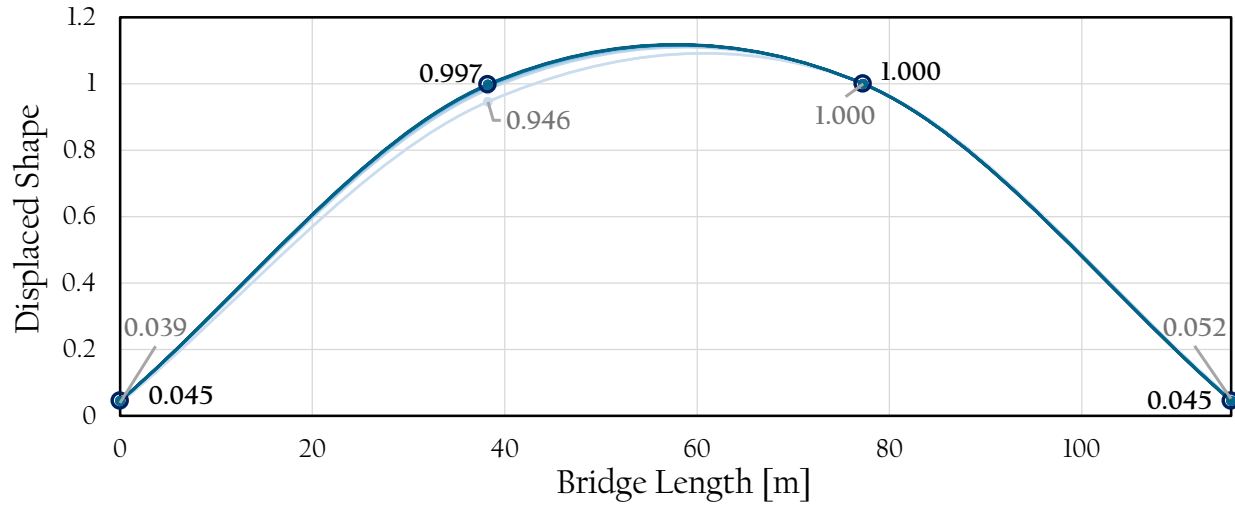


Figure 47. Displaced Shape Progression for Bridge 610 including first and last interaction values.

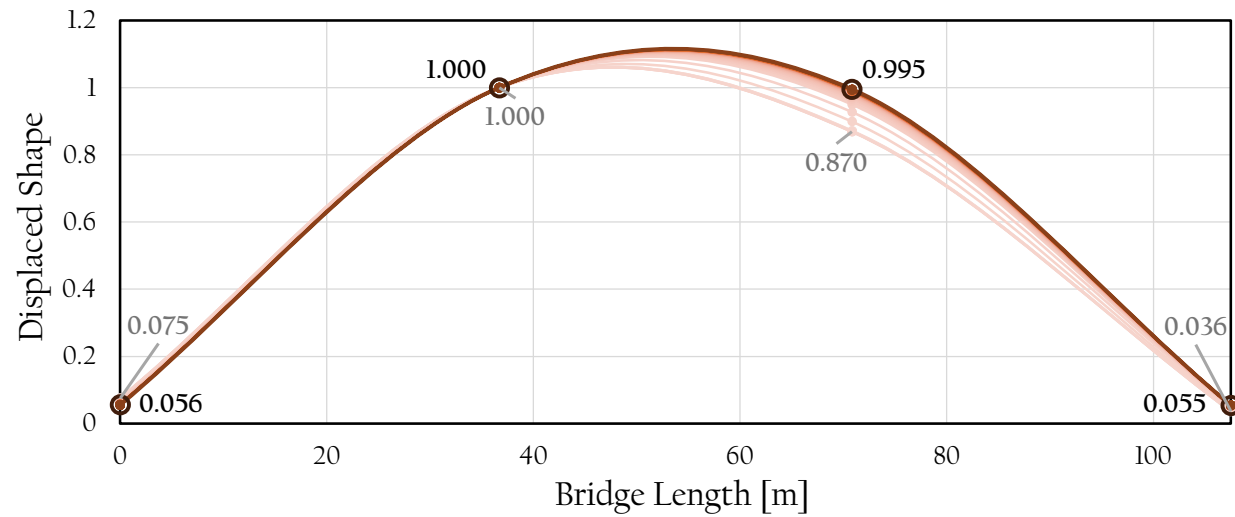


Figure 48. Displaced Shape Progression for Bridge 646 including first and last interaction values.

As expected, the pier with the tallest columns exhibits the highest displacements throughout the pushover analysis. For Bridge 610, this corresponds to Pier 2, and for Bridge 646, to Pier 1. Towards the final stages of the pushover analysis, the variation between successive displaced shapes becomes minimal. The displacement values for the shorter piers also begin to gradually converge towards to normalized value of 1.0, reaching 0.997 for Bridge 610 and 0.995 for Bridge 646.

4. Generation of Bridge Performance Spectra

In this step, we will obtain the bridge's performance through a graphical representation. We evaluate the effective period and the system displacement corresponding to the Operational (PL2) performance limit state achieved by the Pier 1 columns from Bridge 646.

The first step is to obtain the effective stiffness (K_{eff}) through the system displacement and total base shear information, that is:

$$K_{eff} = \frac{V_b}{\Delta_{sys}} = \frac{44437 \text{ kN}}{432 \text{ mm}} = 103 \text{ kN/mm}$$

The second step is to obtain the effective mass (m_{eff}) of the system that is obtained as follows:

$$m_{eff} = \frac{\sum_{i=1}^n m_i \delta_i}{\Delta_{sys}}$$
$$m_{eff} = \left(\frac{1000 \text{ N}}{1 \text{ kN} \cdot 9.81 \text{ m/s}^2} \right) \frac{5939731 \text{ kN}}{432 \text{ mm}}$$
$$m_{eff} = 1.4 \cdot 10^6 \text{ kg}$$

The third step is to obtain the effective period (T_{eff}) as follows:

$$T_{eff} = 2\pi \sqrt{\frac{m_{eff}}{K_{eff}}} = 2\pi \sqrt{\frac{1.4 \cdot 10^6 \text{ kg}}{103 \text{ kN/mm}}}$$
$$T_{eff} = 0.733 \text{ sec}$$

The same calculation was performed for all piers of both bridges, therefore, Table 14 and Table 15 summarize the effective period and system displacement values corresponding to the performance limit states achieved by the columns at each pier and bridge.

Table 14. Summary of the effective period and system displacement for the four performance limit states for both piers of Bridge 610.

Performance Limit States	Bridge 610			
	Pier 1 Columns		Pier 2 Columns	
	Effective Period [sec]	System Displ. [mm]	Effective Period [sec]	System Displ. [mm]
Equivalent Yield (EY)	0.587	45	0.621	58
Fully Operational (PL3)	0.678	82	0.713	102
Operational (PL2)	0.857	353	0.870	421
Life Safety (PL1)	0.874	446	0.886	552

Table 15. Summary of the effective period and system displacement for the four performance limit states for both piers of Bridge 646.

Performance Limit States	Bridge 646			
	Pier 1 Columns		Pier 2 Columns	
	Effective Period [sec]	System Displ. [mm]	Effective Period [sec]	System Displ. [mm]
Equivalent Yield (EY)	0.454	47	0.424	33
Fully Operational (PL3)	0.554	89	0.505	65
Operational (PL2)	0.733	432	0.708	315
Life Safety (PL1)	0.748	540	0.726	393

The performance spectra for both bridges are presented in Figure 49. The figure also includes the four performance limit states identified for each pier columns.

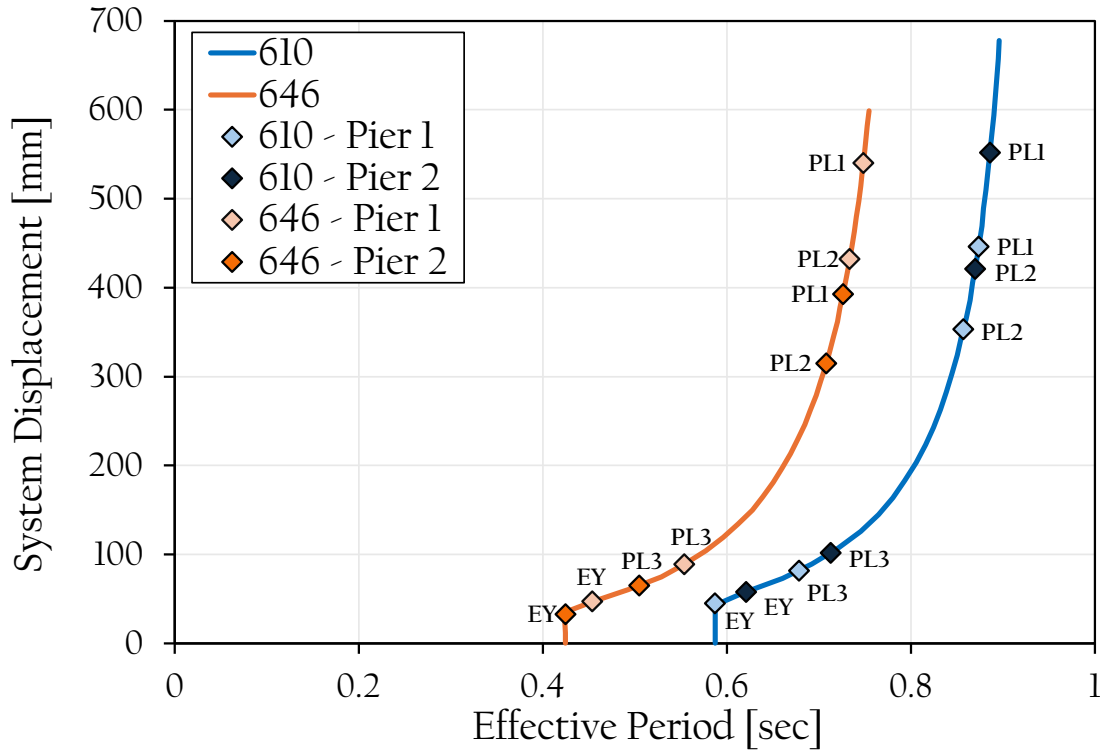


Figure 49. Performance Spectra of Bridge 610 and 646 with the four performance limit states for each pier column.

5. Seismic Demand Characterization

Ground-motion recordings are used to characterize the seismic demand for these example applications. The selected earthquake event is the 2018 M_w 7.1 Anchorage earthquake, located approximately 12 km north of downtown Anchorage. The 2018 Anchorage earthquake is an intraslab event within the northeastern part of the Alaska-Aleutian subduction zone. The focal depth of the earthquake is 46.7 km. In this step, 5%-damped elastic displacement spectra are computed using ground-motion records from the 2018 Anchorage, AK earthquake (Cabas et al., 2021). Actual ground motion records at the closest stations from the earthquake epicenter were selected to provide an example application using strong ground shaking (Figure 50). Linear scaling was also implemented to increase the amplitude of recorded ground motions and to compute variable demand spectra that would be more informative in the assessment of higher levels of ductility in the structure. The

candidate stations include the Port Access Bridge (NSMP8043), Central (NSMP8038), Government Hills Elementary School (K223), the Hilton Hotel (NSMP2716), and the Delaney Part Downhole Array (DPDA; NSMP8040) (Cabas et al., 2021). The ground motion pair at the Station 8043 is selected because the largest PGA observed from the 2018 event was recorded there (i.e., PGA = 0.44 g). The east-west component of the ground motion is used for the ground motion scaling. The target intensity measure used for the linear scaling is a spectral displacement at an oscillator period of 1s between 400 and 600 mm (Kowalsky, personal communication). Thus, scaling factors are computed and listed in Table 15. Scaled SDs obtained with the target IM of 400 mm and 600 mm at an oscillator period of 1 second will be used to analyze variable demand spectra for bridge 610 and 646, respectively (see Figure 51). The red line represents the spectrum corresponding to the scaled spectrum for Bridge 610 (i.e., target IM = 400 mm). The scaled spectrum for Bridge 646 (i.e., target IM = 600 mm) is shown as the blue line.

Table 16. Computed scaling factors regarding the target displacement at $T = 1$ sec

The target spectrum at $T = 1$ sec (mm)	Scaling factor
400	3.9
600	5.8

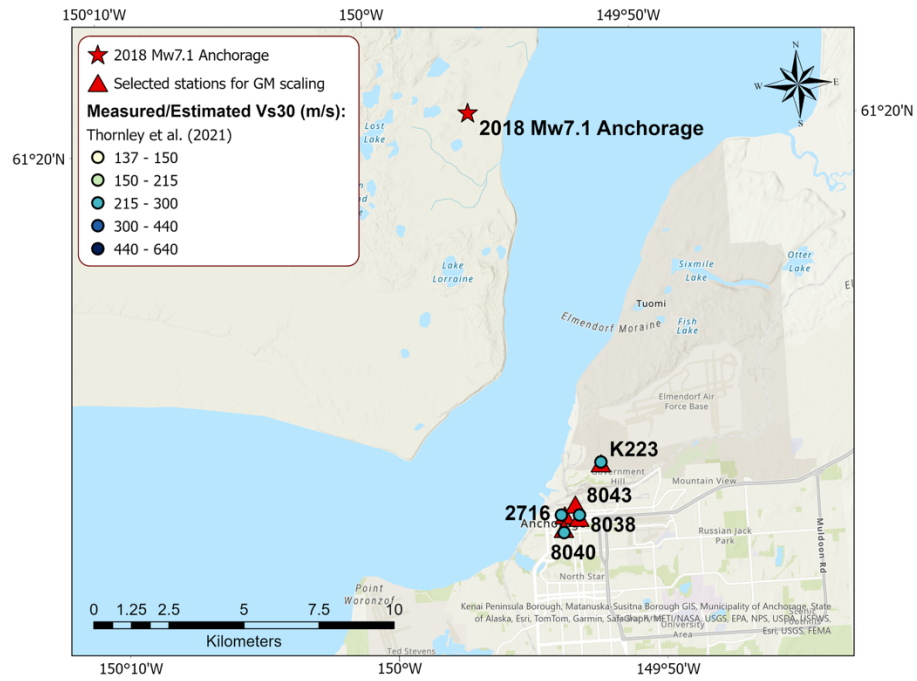


Figure 50. The location of selected stations for the ground motion scaling

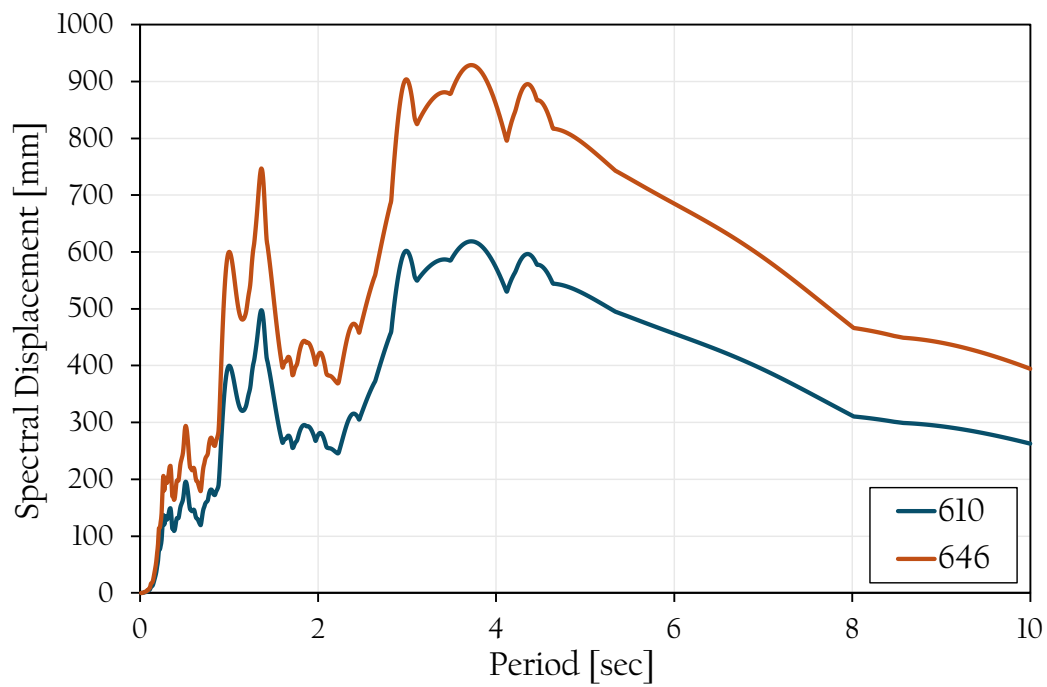


Figure 51. 5% damped elastic displacement spectra for locations of bridge 610 and 646.

6. Calculation of Variable Demand Spectra

In this step, the Variable Demand Spectra for the two bridges are obtained. Figure 52 illustrates the resultant Variable Demand Spectra and the corresponding 5%-damped displacement spectra for each bridge, within the period range of 0 to 1 second. The spectral reduction factor (R_{ξ}) was computed using the equation proposed by Eurocode 8 (1994), proposed in Table 5. The plots reveal that the Variable Demand Spectra starts to diverge from the 5%-damped spectra at $T = 0.587 \text{ sec}$ for bridge 610 and at $T = 0.424 \text{ sec}$ for bridge 646. These values correspond to the elastic period of the respective structures. Beyond these points, the increasing period indicates that the bridge columns have yielded, entering the inelastic range. In this stage, structural behavior becomes nonlinear, with increasing ductility and a corresponding rise in the equivalent viscous damping.

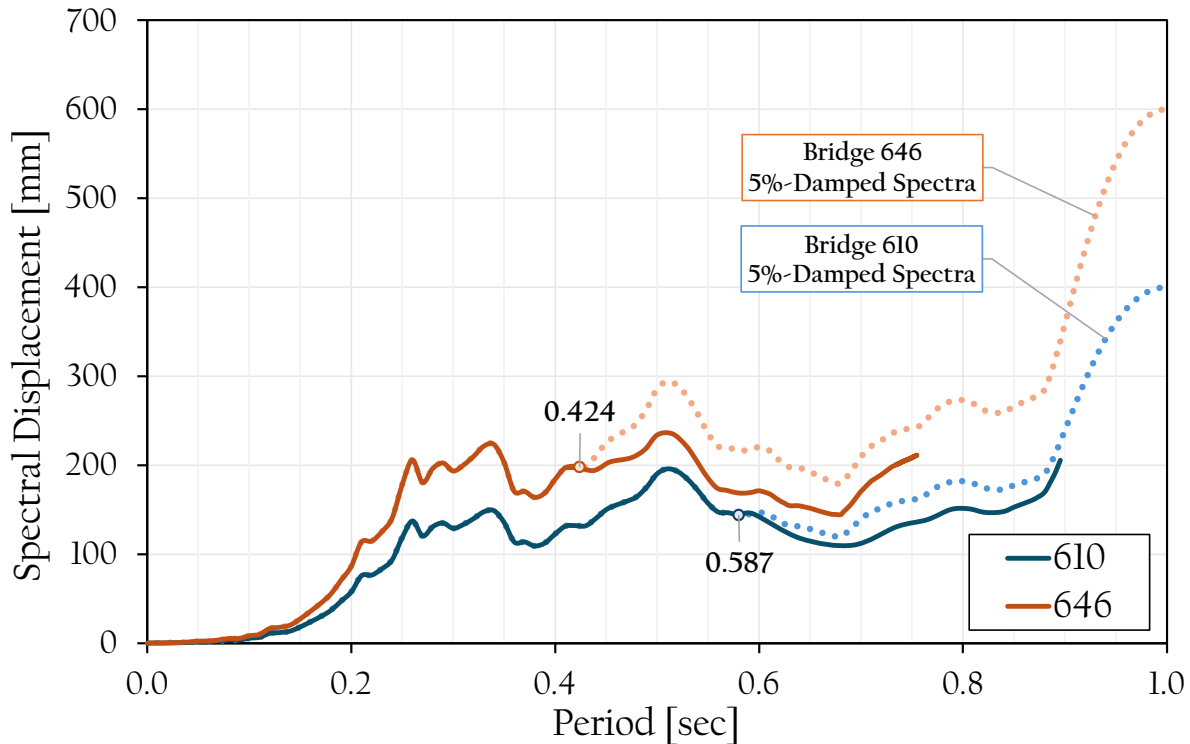


Figure 52. Variable Demand Spectra and 5%-damped spectra for both 610 and 646 bridges.

Figure 53 illustrates this process for the computation of a single point on the Variable Demand Spectra for Bridge 646. It follows the same flowchart steps presents in Figure 38 and detailed in Chapter 3. For Bridge 646, an effective period of $T_{eff} = 0.554 \text{ sec}$ was selected, corresponding to a $\Delta_{sys} = 89 \text{ mm}$ and $V_b = 16072 \text{ kN}$. The value of $S_d = 236 \text{ mm}$ is obtained by entering the 5% damped displacement spectra with the value of T_{eff} . Then, the final value of scaled spectral displacement (S_{d_v}) is obtained by $S_{d_v} = S_d \cdot R_\xi = 236 \text{ mm} \cdot 0.907 = 214 \text{ mm}$.

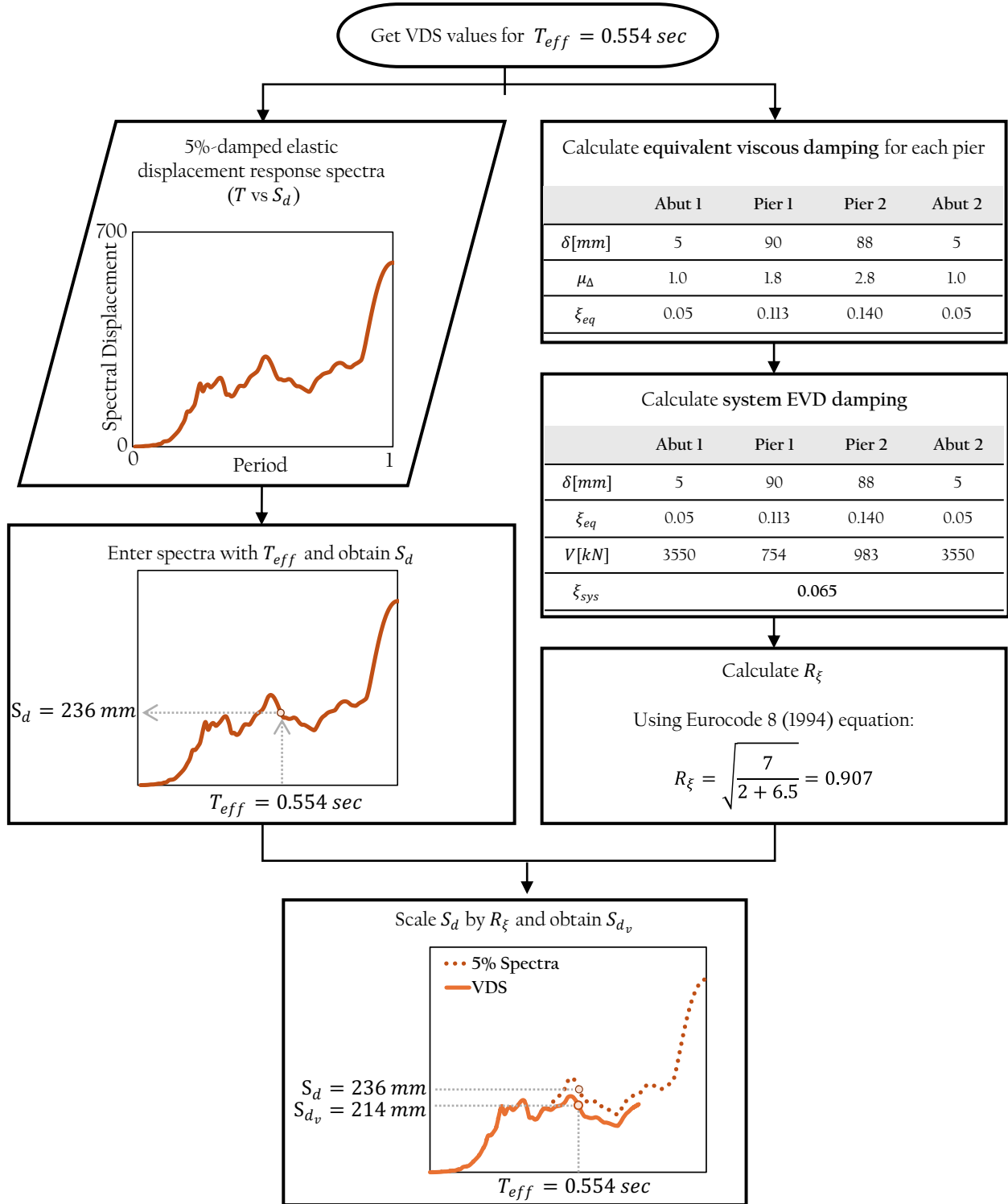


Figure 53. Flowchart illustrating the procedure for calculating a single point on the Variable Demand Spectra corresponding to a given effective period.

7. Estimation of Final Displacement

In this final step of the assessment, the seismic performance of each bridge is determined by the intersection point between the Performance Spectra and the Variable Demand Spectra. This intersection defines the effective period and system displacement the bridge achieves under the seismic demand. As illustrated in Figure 54 for Bridge 610 and Figure 55 for Bridge 646, these parameters have been clearly identified. From both figures, it can be observed that both bridges achieve the Fully Operational (PL3) performance limit state across both piers.

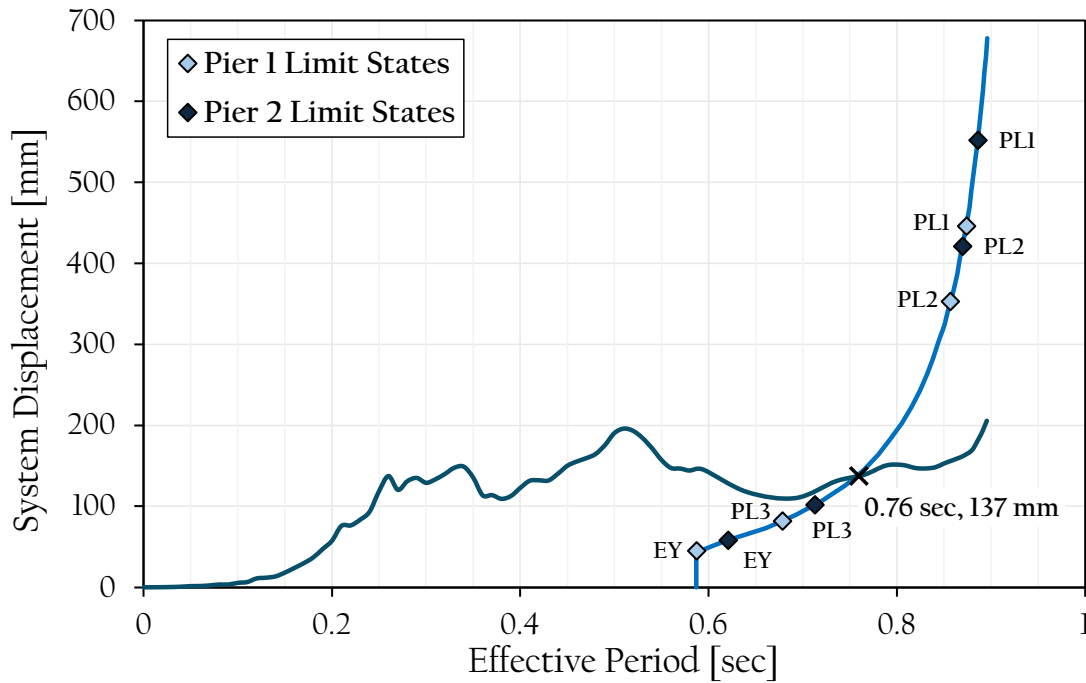


Figure 54. Bridge 610 final performance with the final system displacement and effective period identified.

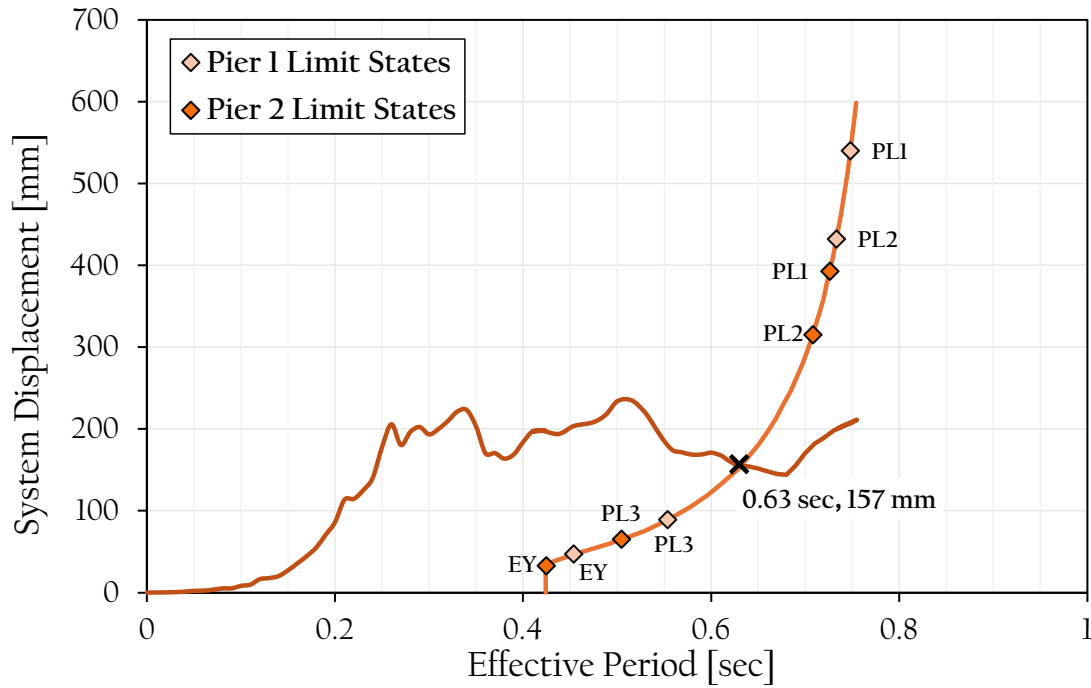


Figure 55. Bridge 646 final performance with the final system displacement and effective period identified

The displaced shape corresponding to the system displacement for Bridge 610 is: 0.043 – 0.986 – 1.000 – 0.043, while for Bridge 646 it is: 0.056 – 1.000 – 0.981 – 0.056. With both the displaced shape and system displacement known, the displacement profile for each bridge can be derived using the system displacement equation detailed in Chapter 3. These results, along with the highest performance limit state reached by the columns in each pier, are presented in Table 17.

Table 17. Final Performance information regarding Bridge 610 and 646, including priority of inspection between both bridges

	Bridge 610	Bridge 646
Final System Displacement [mm]	137	157
Final Displaced Shape	0.043 – 0.986 – 1.000 – 0.043	0.056 – 1.000 – 0.981 – 0.056
Displacement in Each Pier [mm]	6 – 138 – 140 – 6	9 – 160 – 157 – 9
Column's Ductility	Pier 1 – 3.1 Pier 2 – 2.3	Pier 1 – 3.2 Pier 2 – 5.0
Highest Performance Limit State Achieved by the Columns	Pier 1: Fully Operational (PL3) Pier 2: Fully Operational (PL3)	Pier 1: Fully Operational (PL3) Pier 2: Fully Operational (PL3)

The next step in a rapid post-earthquake assessment, particularly when working with a large bridge inventory, is the generation of an inspection priority list. This process must consider factors such as the highest performance limit state achieved by the columns, the functional importance of the route, bridge location, age, and other relevant parameters. When multiple bridges exhibit the same highest performance limit states, additional criteria must be considered by the DOTs to ensure a reliable and effective prioritization for inspection.

CHAPTER 5: PERFORMANCE LIMIT STATE UNCERTAINTY

In the previous chapter, it was shown how the intersection between the Performance Spectra and the Variable Demand Spectra provided a final value of the bridge's system displacement, which was subsequently translated into pier-level and column-level displacements. These displacements were then used to identify the performance limit states achieved by each column. In order to obtain the limit state displacements, nominal material properties were assumed, and detailing parameters were considered as per design specifications. However, in practice, these parameters may vary due to construction-related factors – such as deviations in spiral reinforcement spacing – or due to the inherent variability in material properties.

In this chapter, a deterministic approach is employed to examine the uncertainty of the performance limit states defined in Chapter 2 for reinforced concrete circular columns, by systematically varying design parameters, including material properties and detailing aspects. The analysis also incorporates a multi-column pier configuration, where overturning moments influence the axial load distribution among the columns, thereby affecting their seismic behavior. To illustrate this study, columns from Pier 1 of Bridge 610 and Pier 1 of Bridge 646 are used as representative case studies.

The variable parameters chosen to perform this analysis was concrete compressive strength (f'_c), reinforcing steel yield strength (f_y and f_{yh}), reinforcing steel ultimate strength (f_u), spacing of transverse steel (s), and axial load ratio (ALR), which is calculated as $P/f'_c A_g$. Each parameter was assigned lower and upper bounds to reflect realistic variations that may occur due to construction practices. Table 18 presents the five variable parameters with their lower and upper-limit values chosen.

Table 18. Variable parameters with lower and upper-limit values

Parameters	Lower-Limit	Upper-Limit
Concrete Compressive Strength (f'_c)	28 MPa	36.4 MPa
Steel Yield Strength (f_y and f_{yh})	420 MPa	470 MPa
Steel Ultimate Strength (f_u)	$1.25f_y$	$1.6f_y$
Spacing of Transverse Steel (s)	76.2 mm	90 mm
Axial Load Ratio (ALR)	0.10 (Bridge 610)	0.14 (Bridge 610)
	0.06 (Bridge 646)	0.10 (Bridge 646)

Concrete compressive strength is bounded by a lower limit of the nominal design value (28 MPa) and an upper limit equal to the expected strength, defined as $f'_{ce} = 1.3 \cdot f'_c = 36.4 \text{ MPa}$. The steel yield strength is constrained between the nominal Grade 60 value (420 MPa) and the expected yield strength of 470 MPa (~ 68 ksi), as typically assumed for Grade 60 steel in accordance with AASHTO Guide Specifications (2011). The ultimate steel strength is limited by a minimum tensile-to-yield (T/Y) ratio of 1.25 and a maximum of $1.6 \cdot f_y$. Reinforcement spacing is constrained by a lower limit of 76.2 mm (3 inches), following the design specifications, and an upper limit of 90 mm (~ 3.5 inches).

The axial load ratio for the bridge columns is similarly constrained. For Bridge 610, the ratio ranges from 10% to 14%, while for Bridge 646, it ranges from 6% to 10%. These limits were established based on an analysis of axial load variation in columns belonging to a two-column pier (Bridge 610) and a five-column pier (Bridge 646). Depending on the direction of the applied seismic loading, the overturning moment induces additional axial forces in the columns – either in compression or tension – which, when combined with the gravity-induced axial load, results in a range of total axial load ratios. This variation directly influences the columns' seismic response and it's therefore essential to capture in the analysis.

Given that five parameters were considered, each with two extreme values, a total of 32 combinations were generated. Accordingly, 32 moment-curvature analyses and force-displacement responses were performed, resulting in 32 curvatures and displacements values associated with each selected performance limit state. This process produced a deterministic dataset that was subsequently used to construct cumulative distribution functions (CDFs), enabling a probabilistic characterization of each performance limit state under varying construction and demand conditions. The CDF describes the probability that a random variable X takes on a value less than or equal to x :

$$F_X(x) = P(X \leq x), \text{ for all } x \in \mathbb{R}$$

Although the primary interest lies in the displacement values corresponding to the performance limit states, the behavior of the CDFs for the associated curvature values also provides meaningful insights into the structural response. The resulting curvature's CDFs for each performance limit states are presented below. Figure 56 illustrated the CDFs for Bridge 610, while Figure 57 shows those for Bridge 646.

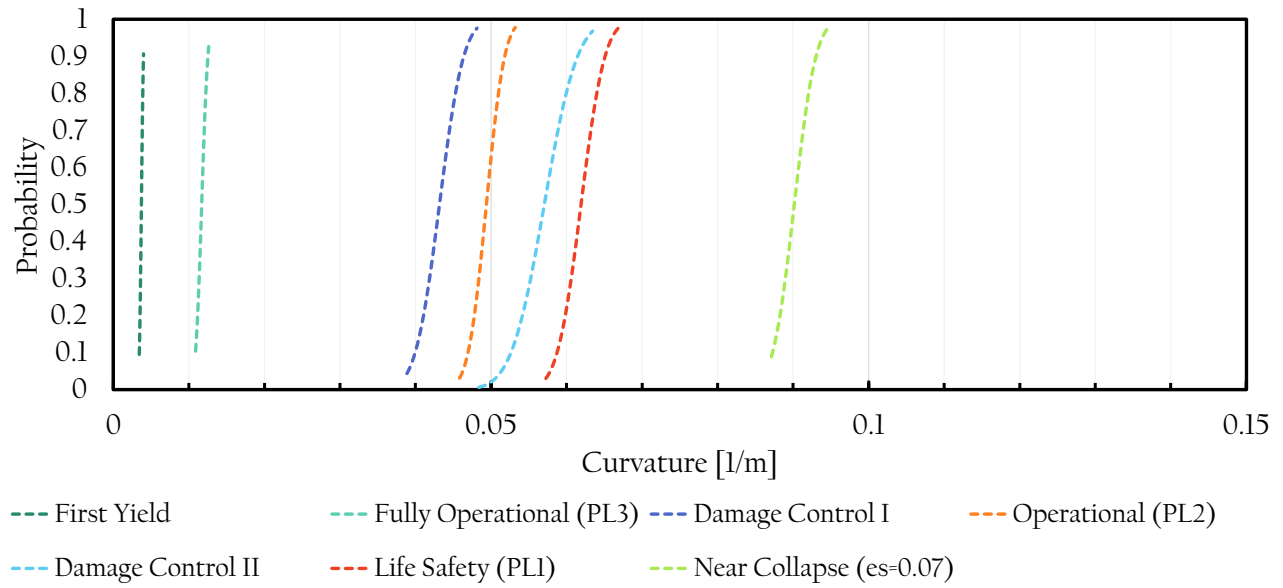


Figure 56. Cumulative Distribution Function from the performance limit states curvatures for Bridge 610.

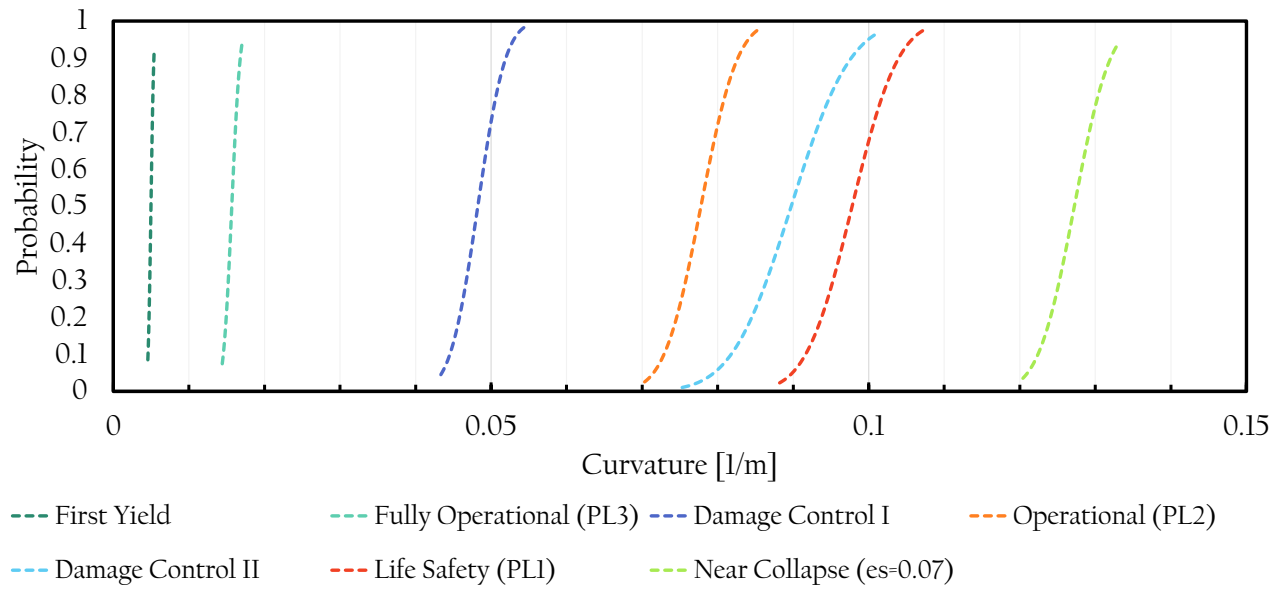


Figure 57. Cumulative Distribution Function from the performance limit states curvatures for Bridge 646.

The behavior of the CDF for each performance limit state is influenced by different combinations of material and design parameters. For the First Yield limit state, the variability is low, as the yield curvature $[\phi_y = 2.25(f_y/E_s)/D_c]$ depends only on one varying parameter, f_y . Similarly, for the Fully Operational (PL3) limit state, the variability remains limited, since the strain limits used to

define this state are not directly influenced by any of the parameters being varied in the study. The small variability observed in this case results from how the progression of curvature is affected by changes in material properties and axial load.

The definition of the Damage Control I limit state is influenced by only one varying parameter, f_{yh} . Nevertheless, it exhibits greater variability than the First Yield limit state, as it corresponds to a point already within the plastic region of the response, where the effects of material nonlinearity and interaction between parameters become more pronounced.

Operational (PL2) and Life Safety (PL1) limit states exhibit CDF with similar shapes. The primary difference between them lies in a shift along the curvature axis, with the Life Safety limit state corresponding to higher curvature values. This shift occurs due to differences in the strain limits used to define each state: the concrete strain limit for Life Safety is 1.4 times greater than that for Operational, while the reinforcing steel strain limit is increased by a factor of 1.25 ($1/0.8$), as observed in Table 3. These proportional increases lead to a consistent shift in the curvature thresholds, while maintaining the overall distribution pattern.

Among all the performance limit states studied, Damage Control II exhibits the highest variability. This is due not only to the fact that it is influenced by multiple varying parameters, such as spacing (through ρ_v), f'_c , f_y , and axial load, but also because it represents a stage of behavior where the governing strain limit can shift depending on the specific combination of parameters. In some cases, the concrete strain limit controls the definition of the limit states; in others, it is the reinforcing steel strain limit that governs. This contrasts with the other performance limit states, for which the controlling material remained consistent despite changes in the input parameters.

The Near Collapse limit state presents relatively low variability, as the strain limit used in its definition is not directly influenced by any of the varying parameters in the study. The variability observed is therefore intrinsic, arising from indirect effects of changes in material properties and axial load on the progression of the curvature. Although the limit state lies deep into the plastic region – where nonlinear behavior is more significant – the strain thresholds remain fixed, which helps contain the overall dispersion in the curvature values.

Having characterized the variability of curvature values for each performance limit state, the analysis was extended to displacements, which are the primary interest in the displacement-based assessment. The resulting displacement CDFs for each performance limit state are presented below. Figure 58 illustrated the CDFs for Bridge 610, while Figure 59 shows those for Bridge 646.

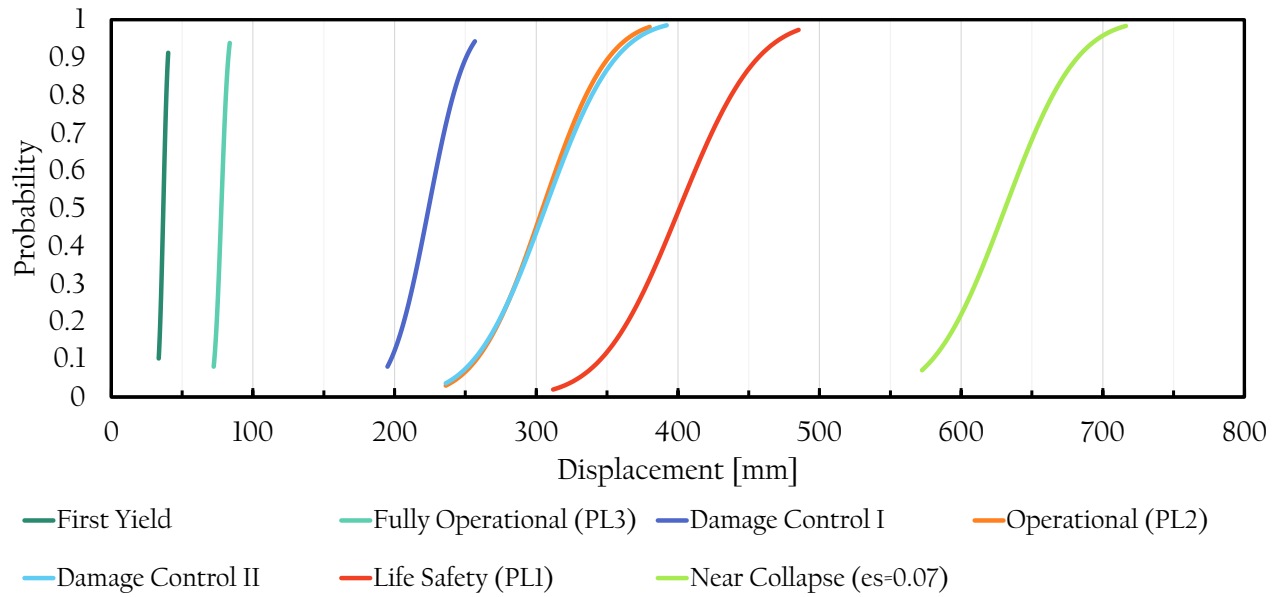


Figure 58. Cumulative Distribution Function from the performance limit states displacements for Bridge 610

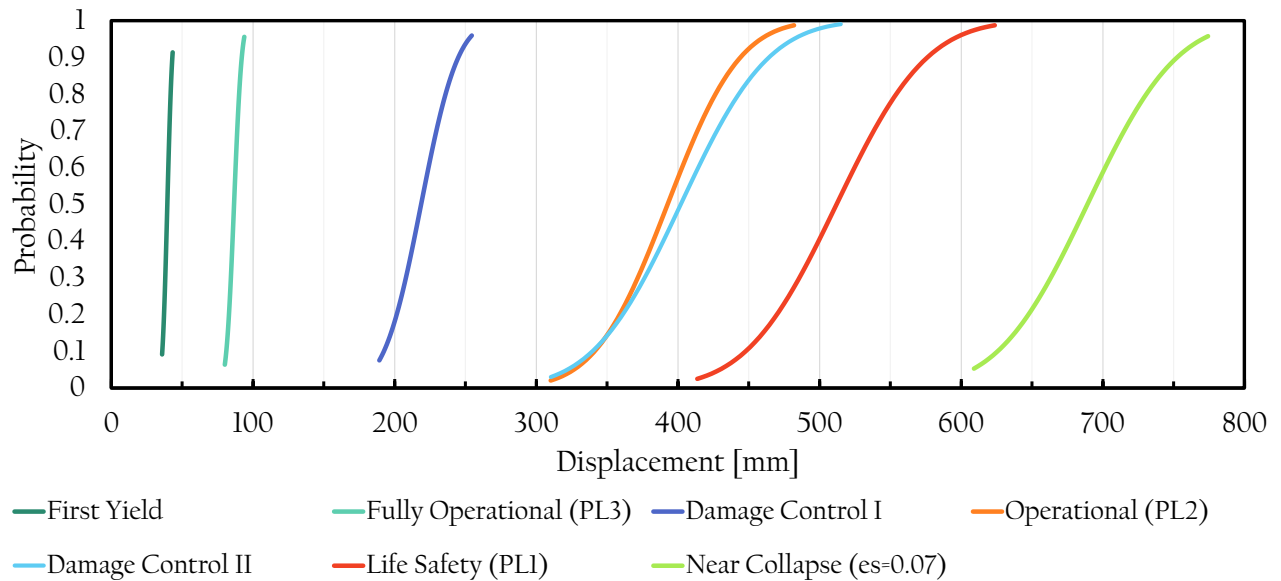


Figure 59. Cumulative Distribution Function from the performance limit states displacements for Bridge 646

It is important to note that the transition from curvature to displacement is governed by the Plastic Hinge Method, as described in Section 2.2.1. Among the parameters varied in this study, only f_y and f_u directly affect the plastic hinge calculation. Consequently, the variability observed in the displacement-based limit states is partially attributable to changes in these two parameters. Moreover,

the method uses different expressions to compute the plastic hinge length depending on whether the governing strain limit is in the concrete or the reinforcing steel, which introduces additional variability into the displacement results.

For a given curvature, the governing material (concrete or steel) may not affect the curvature value itself; however, when converting curvature to displacements, the resulting value is sensitive to which strain governs due to the different plastic hinge length formulations. This distinction helps explain the differences observed between curvature- and displacement-based CDFs. For instance, the Operational (PL2) and Life Safety (PL1) limit states show distinct behavior in curvature, but their displacement CDFs are much more similar, especially for Bridge 610, where many results align. This convergence arises from the change in the governing strain type when transitioning from curvature to displacement, reinforcing the influence of the plastic hinge length definition on the overall response.

To further quantify the variability observed in the performance limit state displacements, Table 19 summarizes the statistical parameters derived from the CDFs, including the mean, standard deviation, and coefficient of variation (COV) for each limit state. These metrics offer a clearer picture of the sensitivity of each performance threshold to the variations in material and design parameters and provide a basis for comparing the relative consistency or dispersion of the structural response across different performance levels.

Table 19. Mean and standard deviation values of the limit state displacements.

Performance Limit State	Bridge 610			Bridge 646		
	Mean [mm]	Standard Deviation [mm]	COV [%]	Mean [mm]	Standard Deviation [mm]	COV [%]
First Yield	36.84	1.99	5.39	39.48	2.19	5.54
Fully Operational (PL3)	77.90	3.35	4.30	86.95	4.12	4.74
Damage Control I	225.82	18.40	8.15	221.81	19.40	8.75
Operational (PL2)	308.00	42.88	13.92	396.14	51.20	12.92
Damage Control II	314.08	46.50	14.80	412.60	61.01	14.79
Life Safety (PL1)	398.64	51.59	12.95	518.75	62.53	12.05
Near Collapse	644.33	42.93	6.66	691.70	49.23	7.12

Based on the assessment results presented in Chapter 4, the maximum displacement of the first pier columns was found to be 138 mm for Bridge 610 and 160 mm for Bridge 646. As shown in Figure 58 and Figure 59, these displacement values lie entirely to the right of Fully Operational (PL3) curve, indicating a probability of 1.0 for achieving this limit state. Additionally, since the displacement values also lie entirely to the left of Damage Control I curve, this higher limit state has no probability of being reached. At Fully Operational (PL3), some crushing of concrete cover and yielding of reinforcement may occur, but residual cracks remain limited, and the bridge is expected to remain fully functional immediately after an earthquake.

In this study, the input variables were assumed to have equal probabilities of reaching their lower or upper bound values. For example, the steel yield strength was considered equally likely to be 420 MPa or 470 MPa. However, it is well established that, for Grade 60 steel, both the yield strength and ultimate strength follow a beta distribution, as reports by Mirza & Macgregor (1979). A similar assumption was made for concrete compressive strength, where equal probabilities were assigned to values such as 28 MPa and 36.4 MPa. In reality, however, material property distribution for concrete is not uniform and is govern by the statistical behavior of their mechanical characteristics. Therefore,

future studies on the uncertainty of performance limit states should incorporate the actual probabilistic distribution of material properties to achieve a more realistic and reliable evaluation of uncertainty.

CHAPTER 6: CONCLUSIONS AND FUTURE WORK

During Phase I of this study, a Direct Displacement-Based Assessment (DDBA) approach was developed for the rapid post-earthquake evaluation of multi-span RC bridges. The proposed approach relies on new spectral representation for both structural capacity (Performance Spectra) and seismic demand (Variable Demand Spectra). The intersection of these spectra allows for the direct estimation of a bridge's seismic performance under specific earthquake scenarios. This framework offers a continuous and comprehensive depiction of structural behavior, facilitating the rapid assessment of damage level following a seismic event. As a demonstration of the proposed methodology, modified versions of Bridge 610 and 646 from AKDOT&PF database were selected as case studies.

To strengthen the proposed assessment framework, this study also investigated the sensitivity of performance limit states to variations in material properties and detailing. While initial analyses used nominal values, a deterministic approach was applied to assess how changes in concrete compressive strength, steel properties, reinforcing spacing, and axial load ratio influence column displacement capacities. A multi-column pier configuration was considered to account for axial load redistribution due to overturning moments. Case studies from Bridge 610 and 646 demonstrated how such variability affects column-level performance, supporting the integration of uncertainty into post-earthquake evaluations.

Early limit states such as First Yield and Fully Operational (PL3) showed low variability, while limit states deeper into the plastic range, such as Damage Control I (COV = 8.15% and 8.75%) and especially Damage Control II (COV = 14.80% and 14.79%), exhibited increasing dispersion due to nonlinear effects and multiple controlling parameters. Operational (PL2) and Life Safety (PL1) had similar CDF shapes, due to consistent curvature shifts reflecting proportional increases in strain

limits. Near Collapse showed relatively low variability, with COV equals to 6.66% and 7.12%, as its fixed strain thresholds limited the impact of material variation.

Seismic hazard characterization in Phase 1 focused on examining available subsurface characterization data in the region to better design pathways for the incorporation of the effects of local geologic conditions into the seismic demands of bridges. Because there are only a few instrumented bridges that can provide strong ground motion records at the site of interest after a seismic event, alternative approaches for the characterization of the seismic demand were examined. Existing ground motion models (GMMs) for spectral displacements are very limited, particularly when considering the diverse seismicity of Alaska. Thus, appropriate GMMs were considered for Alaska, including those developed to predict PSA values and efforts focused on the development of simple, yet effective workflows to incorporate site effects into estimated spectral displacements at the bridge locations of interest. The framework to estimate the seismic demand incorporating site effects is multifaceted to accommodate different levels of subsurface characterization available at the sites as well as their uncertainty. A simple modification scheme for recorded ground motions at stations nearby the bridge of interest was also proposed to account for potential differences in site response (i.e., based on differences in seismic impedance – the product of density and shear wave velocity- and attenuation – dissipation of energy-). The case studies demonstrated the implementation of the workflow assuming that nearby records are available from stations that shared local geologic conditions with the bridge sites of interest. Practical considerations for the selection of appropriate GMMs were also provided for the far more common case when no records are available at or nearby the site of interest. The proposed V_S - $\Delta\kappa_0$ correction framework for recorded ground motions is also included as an alternative approach to incorporate site effects when more detailed subsurface information is available (e.g., a full V_S profile instead of a proxy-based V_{S30} value only). It is noted

that this last adjustment approach would be more applicable in cities like Anchorage and Fairbanks where the density of seismic stations and availability of subsurface data are more significant.

6.1 Future Work - Phase II

The results from Phase I are applicable to bridges with relatively simple configurations, specifically those with circular reinforced concrete columns, pile-supported footings, and no skew or horizontal curvature. However, these characteristics do not reflect the broader AKDOT&PF bridge inventory, which includes many structures with multi-column RCFST piers, varying skew angles, and curved alignments. Additional complexities must also be addressed, particularly in the treatment of abutment fixity conditions. In Phase I, abutments were modeled as elasto-plastic elements without a defined displacement limit, allowing unrestricted lateral deformation. This assumption oversimplified real-world behavior.

The seismic demand (i.e., spectral displacements) used as input into DDBA can be characterized using ground motion models (GMMs) or ground motion recordings at stations. Because in Alaska there are many bridge sites with no nearby recording station, GMMs become an effective alternative to characterize the seismic demand within a rapid assessment framework. However, there are no ground motion models based on spectral displacements developed for subduction zones. In addition, site metadata, such as measured V_{S30} and/or V_S profiles, is not readily available for the whole state of Alaska.

Notably, the threshold to define “nearby” stations is not clearly identified, as it depends on both local and regional spatial variability of geologic structures. More research is needed to better understand what makes a nearby station in Alaska so that records from one station can be used as is or slightly modified to characterize the seismic demand at bridge sites of interest across the state.

It is also essential to benchmark the proposed methodology against existing tools, comparing performance across existing scenarios and using consistent modeling assumptions. This comparative evaluation will help situate the proposed approach within the spectrum of available methods in terms of accuracy, computational efficiency, and operational readiness.

Therefore, the objectives for Phase II are:

1. Develop more robust characterization of the multi-span bridges, considering higher complexity structures.

Phase II will incorporate the effects of skew angles other than zero and the effects of superstructure curvature, both of which significantly influence the seismic response. Approximately 30% of AKDOT&PF post-2004 multi-span bridges exhibit nonzero skew angles. Furthermore, abutment fixity conditions will be explicitly considered. In current AKDOT&PF practice, break-away abutment shear keys are designed to fracture under a prescribed lateral force to prevent damage to abutment piles. Once these shear keys fail, the bridge superstructure is permitted to slide laterally while vertical support is maintained. Although some residual friction remains, the structure's displaced shape is then governed primarily by the columns and the superstructure itself. This change affects the *system's pushover curve*, which must be updated accordingly.

Phase II will also incorporate substructure types that better reflect real inventory characteristics. While Phase I only considered circular RC columns on pile-supported footings, RCFST piers will be introduced in this next phase. As noted by Brown et al. (2015), RCFST are often used as oversized shafts, with a larger diameter below grade than above, although some designs maintain constant diameter. These piers may be pinned-head – where hinging occurs below grade – or fixed-head, with hinging forming first at the column top. RCFST behavior is highly sensitive to soil-structure interaction (SSI), especially in Alaska, where frozen soil conditions can significantly

modify foundation fixity. To accurately reflect this, Phase II will implement soil condition variability by integrating ambient temperature data into the assessment logic. This could include automation through the National Weather Service data over the two weeks prior to the event to infer likely subsurface conditions.

Moreover, Phase II will incorporate the axial load variation between columns in multi-column pier configurations. The behavior of the structural member is altered, and the capacity of the member will need to be expressed in terms of the uncertainty of the direction of the load. The consideration of a multi-column pier will modify how the *system's pushover curve* is obtained and its shape, since the demand in each column will change.

Finally, Phase II will incorporate findings from the ongoing AKDOT&PF-funded research project, *Impact of Response Spectra Definitions and Direct Displacement-Based Design Simplification for Multi-Span Bridges*. The outcomes of this research will directly inform how seismic input is defined within the rapid assessment framework. It is currently envisioned that users will be able to select the RotD definition most appropriate for their application. The corresponding seismic input will then be adjusted using relationships developed as part of the ongoing study.

2. Develop more robust characterization of the seismic demand at bridge sites with very little (or no) subsurface and ground motion data, including uncertainty estimation.

Seismic demand characterization in Phase II will expand the analytical frameworks considered in Phase 1, such as estimating spectral displacements from either ground motion models or modified recorded ground motions including site effects. Thus, three main takeaways will be considered in Phase II: (1) the threshold to define a nearby station to be “close enough”, (2) geospatial interpolation of ground motions where no records are available nearby, and (3) improvement of adjustment schemes for ground motion records in highly uncertain environments.

The proximity threshold to deem a nearby station to be “close enough” will be investigated to provide a reliable characterization of the seismic demand at the bridge site of interest. We will consider various scenarios depending on the density of seismic stations, such as the closest source of recorded ground motion for multiple bridges or one bridge with multiple nearby stations.

Geospatial interpolation schemes, such as Kriging, will be performed to improve geospatial interpolation of ground motions where no records are available nearby. The application of the geospatial interpolation will provide a more robust characterization of ground shaking and its associated uncertainties to capture the local site effects in ground motion models. This task will contribute to represent a “nearby” site and ground motion that can be used in post-earthquake rapid evaluation.

Adjustment schemes for ground motion records will be improved to account for path and site effects in highly uncertain environments. This work will be addressed by quantifying the uncertainties in correlation factors proposed as part of V_S - $\Delta\kappa_0$ correction framework proposed by Cabas and Rodriguez-Marek (2017). V_S - $\Delta\kappa_0$ adjustments will be developed using generic V_S profile and randomized V_S profiles when the limited geotechnical information is available. It will help capture expected variability in situ. Further, the impact of the uncertainty in κ_0 will be investigated.

3. Compare and contrast the methodology proposed with existing rapid assessment methods using existing scenarios and appropriate modeling

As part of Phase II, the accuracy and reliability of the proposed rapid assessment methodology will be evaluated by comparing its results with two established tools: (1) a tool that offers a quick but coarse evaluation of structural behavior, and (2) a more detailed and computationally

demanding one. The tools selected to represent each category are: (1) USGS's ShakeCast, and (2) NLTHA, serving as the benchmark for more precise structural performance analysis.

Two type of cases will be used to complete this task:

- Real Bridges and Hypothetical Earthquake Scenarios

A select group of bridges from the AKDOT&PF inventory will be used for the analysis. These bridges will be subjected to a specific set of seismic scenarios. The performance predictions generated by the proposed methodology will be compared to results from ShakeCast and NLTHA. The comparison will focus on critical response metrics such as displaced shape, peak column displacement, and performance limit states achieved. The eleven bridges presented in this study (two throughout Chapter 4 and nine in Appendix A) will serve as the controlled set for validation. Although most of these structures include RCFST piers, the same bridge geometries will be reanalyzed with RC piers to test the methodology's adaptability.

- Real Bridges and Real Earthquake Scenario

In addition to hypothetical cases, the methodology will be applied to real bridges that experienced the 1994 Northridge Earthquake. Eight bridges were documented and analyzed by M. J. Priestley et al. (1994), using "as-built" plans and basic elastic methods, due to the limited tools available at the time. For this study, those same bridges will be reassessed using the proposed approach and results will be compared against both the reported damage observations and analytical findings from the original study. This comparison will provide a practical benchmark for evaluating how well the proposed rapid assessment methods align with observed real-work structural performance. The evaluation will focus on the displaced shape, peak column displacement, and performance limit states reached.

By comparing the proposed approach to both fast and detailed tools across real and hypothetical scenarios, Phase II will establish how well the methodology balances speed, accuracy, and practicality for seismic assessment of bridge structures.

4. Develop a software tool to conduct a rapid post-earthquake assessment of bridge within the AKDOT&PF inventory.

During Phase II, there will be a development of a software application capable of deploying the DDBA methodology developed during Phase I and II into a user-friendly application interface. The tool would be connected to a bridge database offering enough structural information and a site database (offering subsurface characterization information) to run a deterministic assessment. The software will also incorporate user input details for bridges and/or sites not included in the database. The rapid assessment is initiated by the occurrence of a seismic event (or by the exploration of a hypothetical earthquake scenario) of engineering significance.

Figure 60 presents the flowchart related to the application tool. The steps are explained below:

1. Trigger – an earthquake occurred in a region

The first step is the triggering of analysis by a seismic event. In the case of the specific application tool that will be developed, a region needs to be delimited, and a moment magnitude threshold needs to be defined to select relevant seismic events for the analysis.

2. ShakeMap Info

Once an earthquake occurs, USGS generated (within minutes) maps of ground motion and shaking intensity. Additionally, information of the epicenter (latitude and longitude), moment magnitude, and type (shallow crustal, subduction or intraslab) are provided. For the generation of the map, MMI, PGA, PGV, and PSA (0.3, 1.0, and 3.0 sec) are produced through

interpolation of seismic stations recordings and community perception (inputs in the “Did You Fell It” website). The generation of a ShakeMap will serve as a trigger for the application tool to run. *(It is important to note that in case of the need to run a fictitious earthquake scenario, the trigger and ShakeMap generation will not be considered, since the seismic event will be input into the system).*

3. Ground Motion Selection

In this step, the question “does the bridge have a recorded ‘nearby’ motion?” will need to be answered. During Phase II, more details about the definition of “nearby” will be obtained.

3a. Adjust nearby motion: if the answer to the previous question turns out positive, then the nearby motion will need to be adjusted following the proposed aforementioned schemes.

3b. ShakeMap: if the answer to the previous question is negative, then the information collected from the ShakeMap obtained in Step 2 will be used.

3c. However, if for some reason a ShakeMap is not available for a specific seismic event, then the application will obtain latitude, longitude, and the moment magnitude of the event. With this limited information, the code will choose between a list of GMMs that would be most appropriate for the event.

4. 5% Damped Displacement Spectra

During this step, the product of Step 3 will be used to obtain 5% damped displacement spectra.

In this step, a spectra will be generated for each bridge location.

Static Portion of the Analysis

There will be a static portion of the analysis that will contain a bridge database and site database. This will work independently of the seismic event under consideration.

In the static portion, the user will be able to insert information regarding the bridge. Additionally, for each bridge site, geologic maps, geophysical data, and geotechnical data will be available. This information will be combined to obtain the following:

System's Pushover Curve: a computational code will be implemented to obtain the system's pushover curve in terms of system displacement and total base shear.

Performance Spectra: a computational code will be implemented to obtain the performance spectra in terms of effective period and system displacement.

5. Variable Demand Spectra

During this step, the 5% damped displacement spectra and the Performance Spectra generated for each bridge will allow the generation of the Variable Demand Spectra, which is presented in terms of period and (variable damped) spectral displacement. And a computational code will be implemented to obtain the spectra.

6. "Final" Bridge Performance

During this step, the Performance Spectra and Variable Demand Spectra are combined to obtain the value where both curves intersect. From the system displacement, the displacement of each one of the abutments and piers is obtained. Once the final displacements are obtained, the performance limit states and level of damage of each pier column is identified.

7. Inspection Priority List

With the information from Step 6, an inspection priority list can be built in terms of the level of damage expected at each bridge. The displacement in each pier and abutment, the performance limit state achieved by each member, and a map with bridge location and color defining its priority of inspection will be the results of the list.

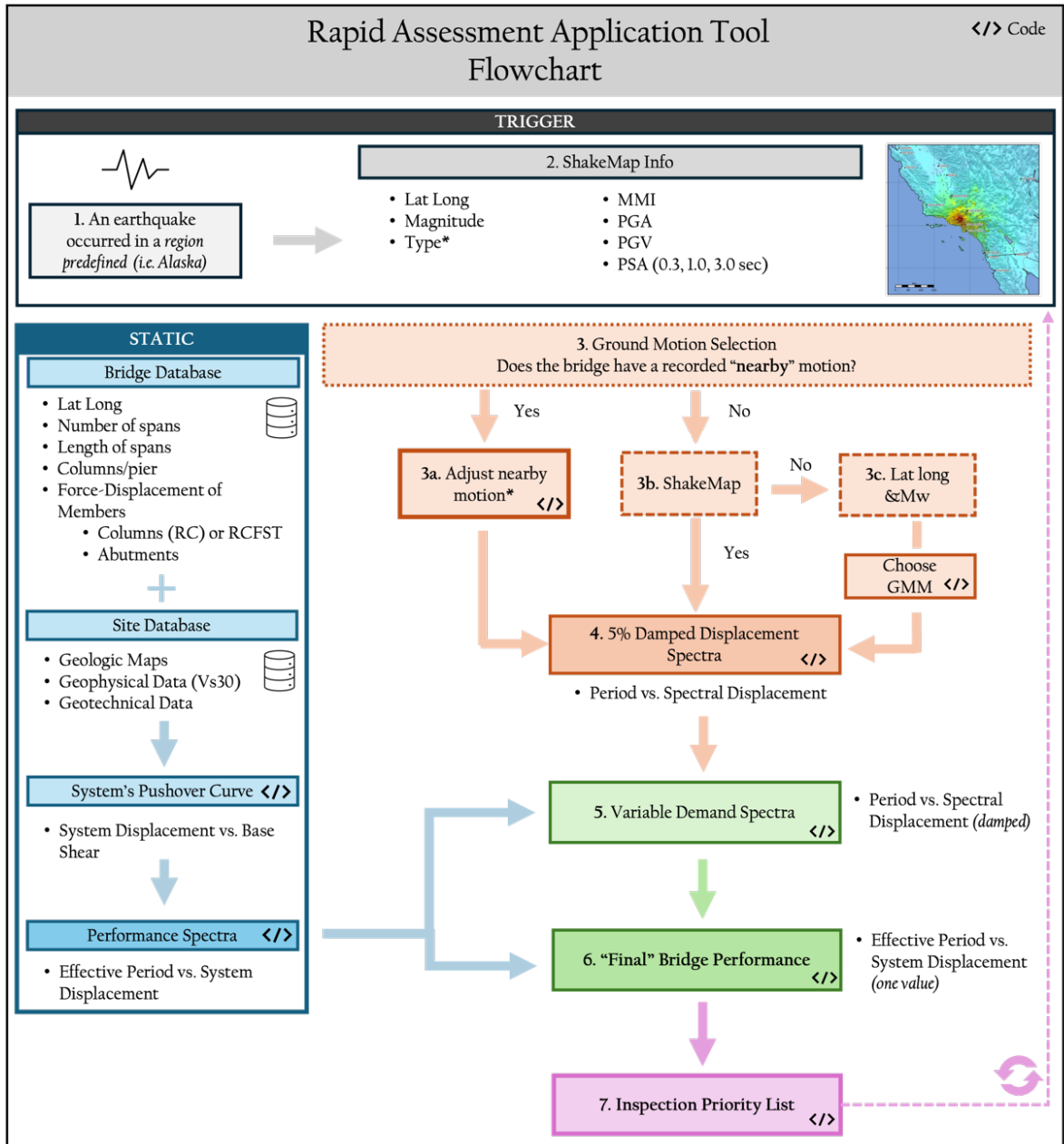


Figure 60. Rapid Assessment Application Flowchart

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APPENDIX A

POST-EARTHQUAKE ASSESSMENT OF ADDITIONAL BRIDGES

This appendix presents the results of the post-earthquake assessment conducted on nine bridges. As described in Chapter 4, the selected bridges are part of the AKDOT&PF inventory. However, for the purposes of this analysis, certain assumptions and simplifications were made regarding the structural configurations. The abutments were modeled with elasto-plastic behavior, with the yield displacement being 40 mm and the yield strength, 28400 kN, as shown in Figure 42. The abutment displacement was not limited. Although the bridges have RCFST columns, they were treated as RC columns with pile-supported footings in this study due to the scope and limitations at its current stage. Accordingly, the contribution of the steel tube was excluded from analysis. The column height was defined as the distance from the bottom of the cap beam to the ground level, plus twice the column diameter. Additionally, the cross-section details are the same for all columns in the bridge.

The seismic demand the bridges are under is identified as 1964 M_w 9.2 Great Alaska earthquake in the Prince William Sound region of Alaska. The earthquake rupture started propagating at the focal depth of 25 km approximately (<https://earthquake.usgs.gov/earthquakes/events/alaska1964/>). The locations of the earthquake and assessed bridges are shown in Figure 61. “Nearby” stations are within a radius of 10 km (Jayaram & Baker, 2009) from the bridges to find ground motion records. Bridge 1350 and 2081 have the largest number of nearby stations because those bridges are in Anchorage (see Table 20).

Ground motion models are used to estimate spectral displacements because we could not obtain ground motion records of the 1964 Alaska earthquake at the nearby stations. PSBAH20 (Parker et al,

2020), one of the NGA-Subduction GMMs, estimates spectral displacements in each bridge (NHR3; <https://www.risksciences.ucla.edu/nhr3/nga-subduction/gmmmodels-excel>; last accessed November 2022). The interface model is considered as the earthquake event type. Table 20 describes epicentral distances between the event and bridges and proxy-based V_{S30} (Geyin & Maurer, 2023) as the site parameter of PSBAH20 model.

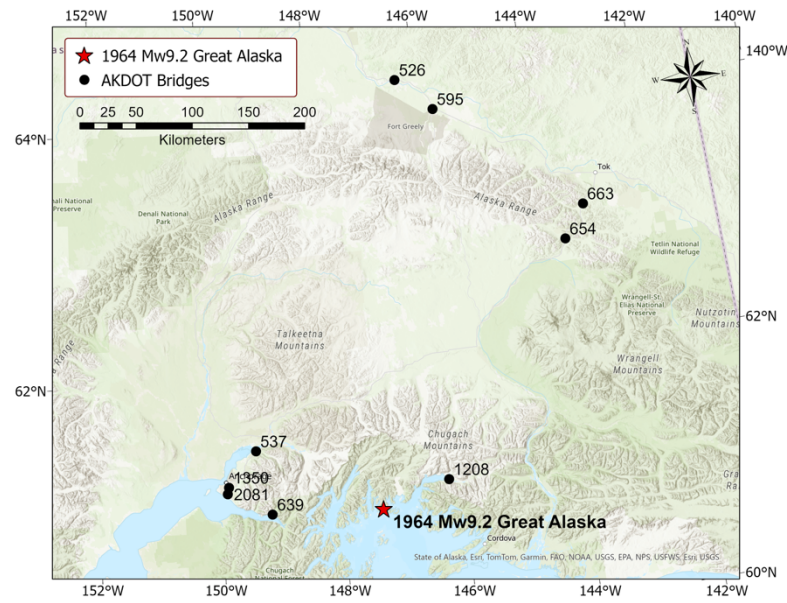


Figure 61. The location of the selected earthquake event and bridges

Table 20. The number of nearby station and input parameters to estimate spectral displacement in each bridge

Bridge Number	Number of Nearby Stations	Epicentral Distance from the Earthquake Event [km]	Proxy-based V_{S30} [m/s]	Geologic age
526	0	376.54	329.6	Quaternary (Qs)
537	1	32.59	458.4	Early Jurassic (Jhg)
595	2	368.94	361.9	Quaternary (Qs)
639	0	61.01	401.8	Quaternary (Qs)
654	0	366.21	530.9	Quaternary (Qs)
663	1	396.91	252.5	Quaternary (Qs)
1208	1	201.55	312.0	Quaternary (Qs)
1350	36	18.87	282.7	Quaternary (Qs)
2081	29	24.39	289.3	Quaternary (Qs)

Each of the nine bridges – summarized in Table 21 – has a dedicated section in this appendix, providing detailed assessment results. These include:

- Structural database information
- Moment-curvature and force-displacement relationships
- System’s Pushover Curve
- Displacement Progression
- Performance Spectra
- 5% damped Displacement Spectra
- Variable Demand Spectra
- Final performance limit state achieved by each pier

The methodology used to generate these results follows the procedure outlined in Chapter 3 and is further illustrated through examples in Chapter 4.

Table 21 provides an overview of the assessed bridges and indicates the corresponding page number for each detailed evaluation.

Table 21. Summary of bridges chosen for post-earthquake assessment.

Bridge Number	Structure Name	Number of Main Spans	Number of Columns/pier	Page Number
526	Banner Creek	2	4	137
537	Eklutna River	2	3	147
595	Jarvis Creek	2	4	153
639	Glacier Creek	2	4	161
654	Slana River	2	4	169
663	Tok River	3	4	177
1208	Lowe River North Channel	2	6	183
1350	Campbell Creek	2	4	190
2081	C St UC at O’Malley, EB	2	4	200

As a key outcome of the rapid post-earthquake assessment methodology is the generation of an inspection priority list. This list ranks the nine evaluated bridges according to the urgency of inspection and indicated the performance limit state reached by each structure. For detailed information on the observed damage levels at the pier level, please refer to Section 2.2 of this report. Table 22 presents the resulting inspection priority on the earthquake scenario considered.

Table 22. Inspection priority and performance limit states achieved by nine bridges.

Priority	Bridge #	Performance Limit State Achieved	
		Pier 1 Columns	Pier 2 Columns
1	639	Fully Operational (PL3)	-
2	663	Equivalent Yield	Equivalent Yield
	1208	Equivalent Yield	-
	2180	Equivalent Yield	-
3	537	Elastic Behavior	-
	526	Elastic Behavior	-
	595	Elastic Behavior	-
	654	Elastic Behavior	-
	1350	Elastic Behavior	-

Among the assessed structures, only Bridge 639 reached the Fully Operational (PL3) performance limit state, making it the highest priority for inspection. Bridges 663, 1208, and 2180 reached the Equivalent Yield, suggesting that their piers experienced inelastic deformations. These bridges are therefore assigned to a second-level priority. The remaining bridges—537, 526, 595, 654, and 1350—maintained elastic response under the seismic demand, and are thus considered lower priority for immediate post-earthquake inspection.

1. Bridge 526

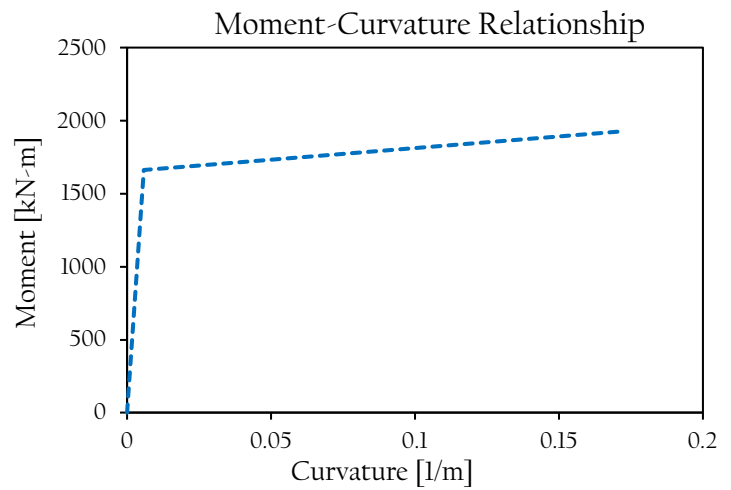
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	f'c (MPa)	Iy (m4)	Weight SS (kN/m)
526	64.2892	-146.352	2	27.74	27.74	0	0	420	420	600	28	72.49	180

Substructure						Substructure						Abutments	
RC or RCFST	# Columns/pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long Bars	Long Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	15.5	13.3	0	0	0.88	50.8	12	28.65	15.875	76.2	28400	40

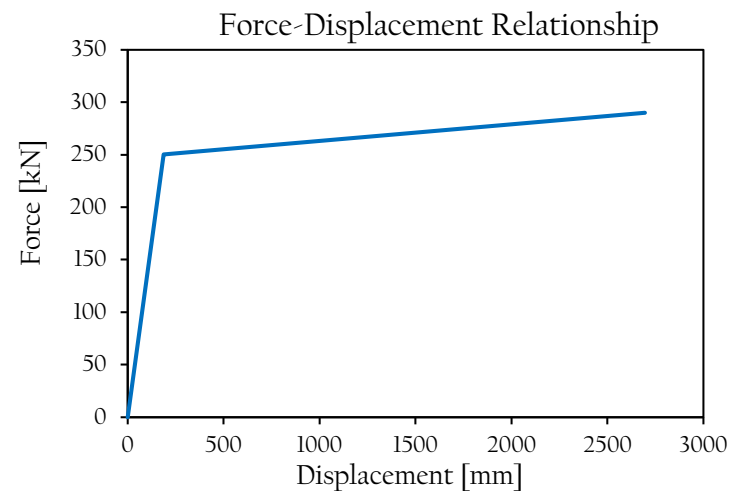
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0058	1663.8
0.1724	1927.7



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
186.65	250.20
2694.75	289.88



- Performance Limit States – Displacements

Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	142
Equivalent Yield	187
Fully Operational (PL3)	328
Operational (PL2)	1204
Life Safety (PL1)	1483

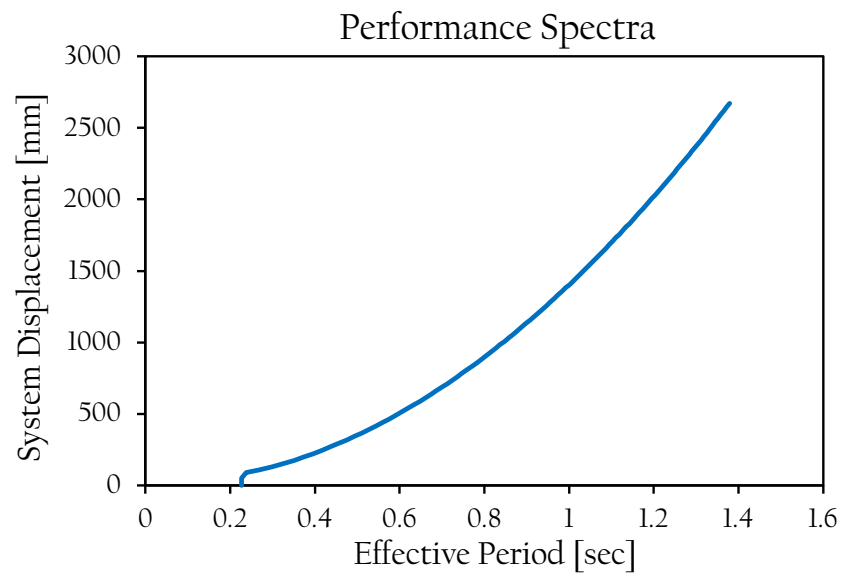
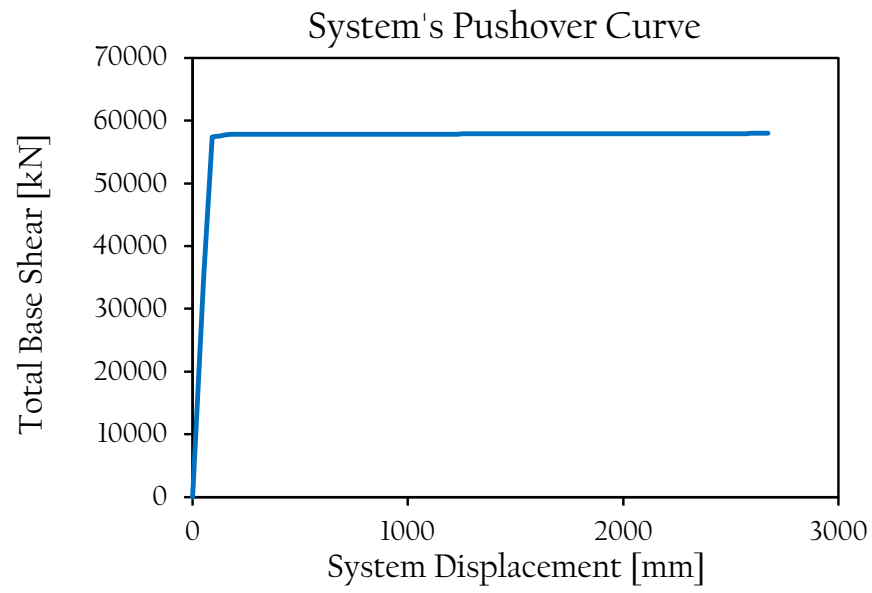
- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.2259	0	0	0
52	35532	0.2259	24.79	62.73	24.79
90	57379	0.23864	49.24	107.95	49.24
109	57487	0.27032	73.97	128.22	73.97
131	57606	0.30006	98.72	150.32	98.72
153	57729	0.32621	123.47	173.32	123.47
177	57801	0.35206	148.2	196.86	148.2
200	57803	0.37516	172.93	220.76	172.93
224	57804	0.39769	197.67	244.87	197.67
249	57806	0.41977	222.42	269.12	222.42
273	57808	0.43989	247.18	293.47	247.18
297	57809	0.4591	271.95	317.91	271.95
322	57811	0.47825	296.73	342.41	296.73
346	57812	0.49593	321.52	366.96	321.52
371	57814	0.51368	346.32	391.57	346.32
396	57815	0.53083	371.13	416.2	371.13
420	57817	0.54678	395.95	440.87	395.95
445	57818	0.5629	420.78	465.56	420.78
470	57820	0.57857	445.62	490.28	445.62
494	57822	0.59321	470.47	515.03	470.47
519	57823	0.60809	495.34	539.79	495.34
544	57825	0.62261	520.22	564.58	520.22
569	57826	0.6368	545.1	589.39	545.1
594	57828	0.65067	570	614.21	570
618	57829	0.66371	594.91	639.06	594.91
643	57831	0.67703	619.84	663.92	619.84
668	57833	0.69008	644.77	688.79	644.77

693	57834	0.7029	669.72	713.68	669.72
718	57836	0.71548	694.67	738.59	694.67
743	57837	0.72784	719.64	763.51	719.64
768	57839	0.73999	744.62	788.45	744.62
793	57840	0.75196	769.61	813.4	769.61
818	57842	0.76372	794.62	838.36	794.62
843	57844	0.77531	819.63	863.34	819.63
868	57845	0.78673	844.66	888.33	844.66
893	57847	0.79798	869.7	913.34	869.7
918	57848	0.80908	894.75	938.36	894.75
943	57850	0.82002	919.81	963.39	919.81
968	57852	0.83081	944.88	988.44	944.88
993	57853	0.84148	969.96	1013.49	969.96
1018	57855	0.852	995.06	1038.57	995.06
1043	57856	0.8624	1020.17	1063.65	1020.17
1068	57858	0.87267	1045.29	1088.75	1045.29
1093	57859	0.88282	1070.42	1113.86	1070.42
1118	57861	0.89286	1095.56	1138.98	1095.56
1143	57863	0.90278	1120.72	1164.11	1120.72
1169	57864	0.91298	1145.88	1189.26	1145.88
1194	57866	0.92269	1171.06	1214.42	1171.06
1219	57867	0.93229	1196.25	1239.59	1196.25
1244	57869	0.9418	1221.45	1264.78	1221.45
1269	57871	0.9512	1246.67	1289.97	1246.67
1294	57872	0.96052	1271.89	1315.18	1271.89
1320	57874	0.97011	1297.13	1340.4	1297.13
1345	57875	0.97925	1322.38	1365.64	1322.38
1370	57877	0.9883	1347.63	1390.88	1347.63
1395	57879	0.99726	1372.91	1416.14	1372.91
1421	57880	1.00651	1398.19	1441.41	1398.19
1446	57882	1.01531	1423.48	1466.69	1423.48
1471	57883	1.02405	1448.79	1491.99	1448.79
1497	57885	1.03304	1474.11	1517.29	1474.11
1522	57887	1.04162	1499.44	1542.61	1499.44
1547	57888	1.05013	1524.78	1567.94	1524.78
1573	57890	1.0589	1550.13	1593.28	1550.13
1598	57891	1.06728	1575.5	1618.64	1575.5
1623	57893	1.07558	1600.87	1644	1600.87
1649	57895	1.08415	1626.26	1669.38	1626.26

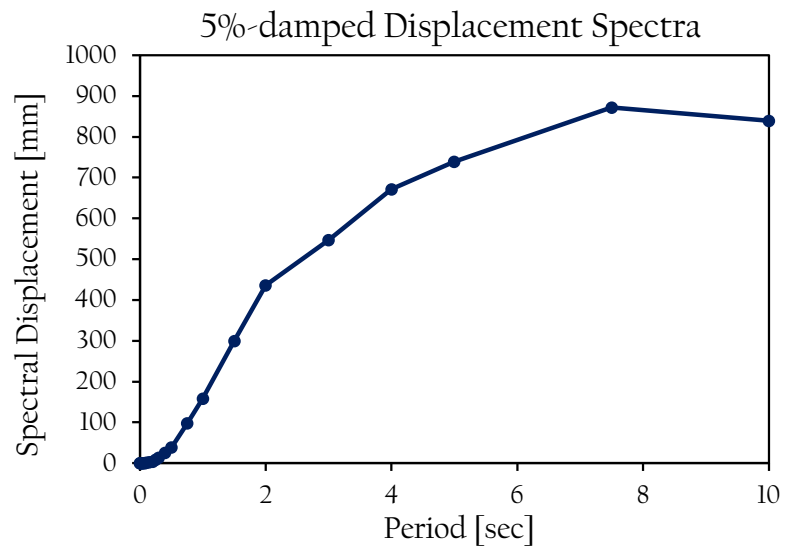
1674	57896	1.09233	1651.66	1694.77	1651.66
1699	57898	1.10044	1677.07	1720.17	1677.07
1725	57899	1.10882	1702.5	1745.59	1702.5
1750	57901	1.11681	1727.93	1771.01	1727.93
1776	57903	1.12506	1753.38	1796.45	1753.38
1801	57904	1.13294	1778.84	1821.9	1778.84
1827	57906	1.14107	1804.31	1847.36	1804.31
1852	57908	1.14883	1829.79	1872.84	1829.79
1878	57909	1.15686	1855.28	1898.32	1855.28
1903	57911	1.16452	1880.79	1923.82	1880.79
1929	57912	1.17244	1906.3	1949.33	1906.3
1954	57914	1.17999	1931.83	1974.85	1931.83
1980	57916	1.1878	1957.37	2000.38	1957.37
2005	57917	1.19527	1982.92	2025.93	1982.92
2031	57919	1.20297	2008.49	2051.48	2008.49
2056	57920	1.21034	2034.06	2077.05	2034.06
2082	57922	1.21795	2059.65	2102.63	2059.65
2108	57924	1.22552	2085.25	2128.22	2085.25
2133	57925	1.23275	2110.86	2153.82	2110.86
2159	57927	1.24022	2136.48	2179.44	2136.48
2184	57929	1.24736	2162.11	2205.07	2162.11
2210	57930	1.25476	2187.76	2230.71	2187.76
2236	57932	1.26209	2213.41	2256.36	2213.41
2261	57933	1.26912	2239.08	2282.02	2239.08
2287	57935	1.27638	2264.76	2307.69	2264.76
2313	57937	1.28359	2290.45	2333.38	2290.45
2338	57938	1.2905	2316.16	2359.08	2316.16
2364	57940	1.29763	2341.87	2384.79	2341.87
2390	57942	1.30473	2367.6	2410.51	2367.6
2416	57943	1.3118	2393.34	2436.24	2393.34
2441	57945	1.31854	2419.09	2461.99	2419.09
2467	57946	1.32554	2444.85	2487.74	2444.85
2493	57948	1.33248	2470.62	2513.51	2470.62
2519	57950	1.33939	2496.41	2539.29	2496.41
2544	57951	1.34601	2522.2	2565.08	2522.2
2570	57953	1.35285	2548.01	2590.89	2548.01
2596	57955	1.35965	2573.83	2616.7	2573.83
2622	57956	1.36643	2599.66	2642.53	2599.66
2648	57958	1.37317	2625.51	2668.37	2625.51

2674	57959	1.37988	2651.36	2694.22	2651.36
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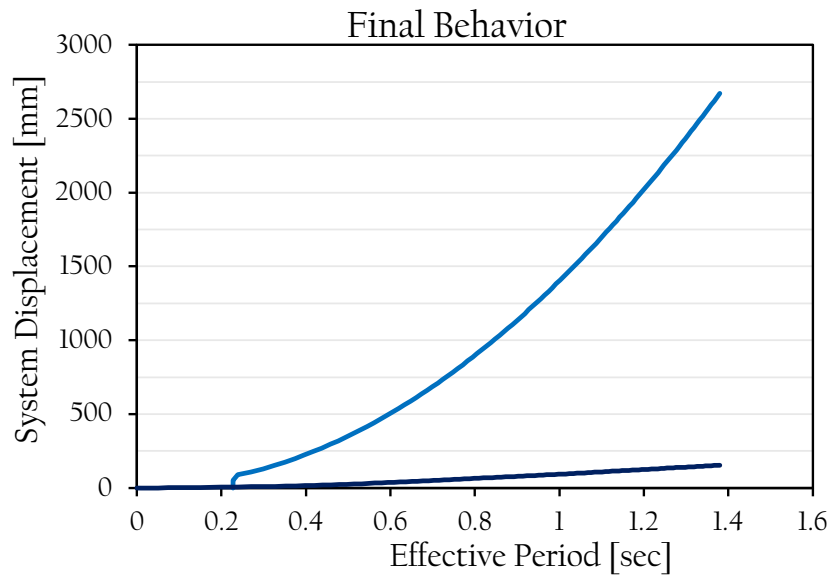
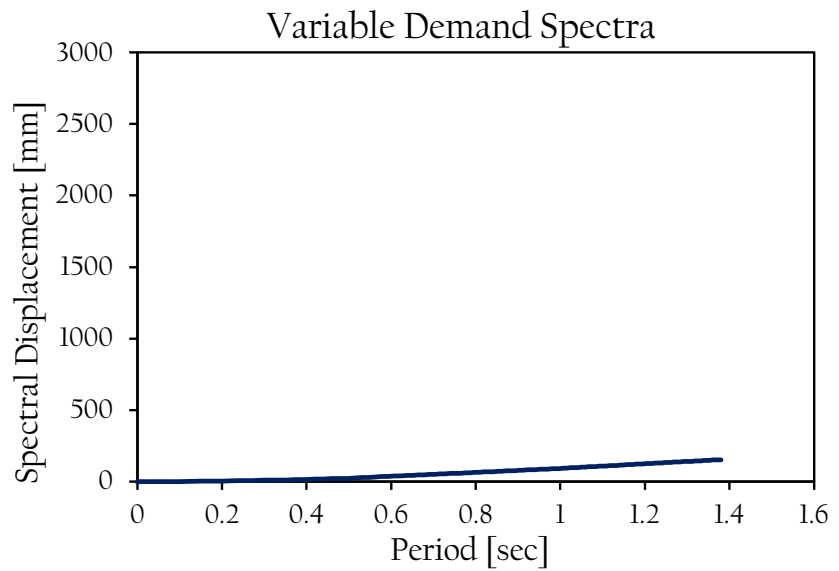
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.006
0.02	0.026
0.03	0.059
0.05	0.178
0.075	0.467
0.1	0.891
0.15	2.130
0.2	4.462
0.25	8.365
0.3	13.446
0.4	25.214
0.5	39.187
0.75	98.194
1	158.674
1.5	299.811
2	436.245
3	547.527
4	670.949
5	739.135
7.5	871.419
10	839.568



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.006
0.02	0.026
0.03	0.059
0.05	0.178
0.075	0.467
0.1	0.891
0.15	2.13
0.2	4.462
0.2386	6.38
0.2703	7.539
0.3001	9.109
0.3262	10.806
0.3521	12.51
0.3752	14.019
0.3977	15.501
0.4198	17.23
0.4399	18.827
0.4591	20.35
0.4783	21.872
0.4959	23.273
0.5137	25.475
0.5308	27.823
0.5468	30
0.5629	32.201
0.5786	34.336
0.5932	36.327
0.6081	38.351
0.6226	40.323
0.6368	42.25
0.6507	44.133
0.6637	45.901
0.677	47.706
0.6901	49.476
0.7029	51.213



0.7155	52.917
0.7278	54.593
0.74	56.238
0.752	57.865
0.7637	59.498
0.7753	61.107
0.7867	62.692
0.798	64.253
0.8091	65.794
0.82	67.312
0.8308	68.81
0.8415	70.289
0.852	71.749
0.8624	73.192
0.8727	74.617
0.8828	76.025
0.8929	77.417
0.9028	78.793
0.913	80.21
0.9227	81.556
0.9323	82.888
0.9418	84.206
0.9512	85.511
0.9605	86.804
0.9701	88.135
0.9793	89.402
0.9883	90.657
0.9973	91.901
1.0065	93.336
1.0153	94.764
1.024	96.18
1.033	97.64
1.0416	99.03
1.0501	100.411
1.0589	101.834
1.0673	103.191
1.0756	104.537
1.0841	105.926
1.0923	107.252

1.1004	108.567
1.1088	109.926
1.1168	111.222
1.1251	112.559
1.1329	113.837
1.1411	115.156
1.1488	116.414
1.1569	117.716
1.1645	118.957
1.1724	120.241
1.18	121.465
1.1878	122.731
1.1953	123.941
1.203	125.191
1.2103	126.386
1.218	127.62
1.2255	128.846
1.2328	130.019
1.2402	131.23
1.2474	132.387
1.2548	133.586
1.2621	134.776
1.2691	135.914
1.2764	137.091
1.2836	138.26
1.2905	139.38
1.2976	140.537
1.3047	141.687
1.3118	142.833
1.3185	143.927
1.3255	145.06
1.3325	146.186
1.3394	147.306
1.346	148.379
1.3528	149.488
1.3597	150.591
1.3664	151.69
1.3732	152.782
1.3799	153.871

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	6
T_{eff} [sec]	0.23
Displacement Profile	0 mm – 1 mm – 0 mm
Column's Ductility	Pier I: < 1.0
Highest Limit State Achieved	Pier I: Elastic Behavior

2. Bridge 537

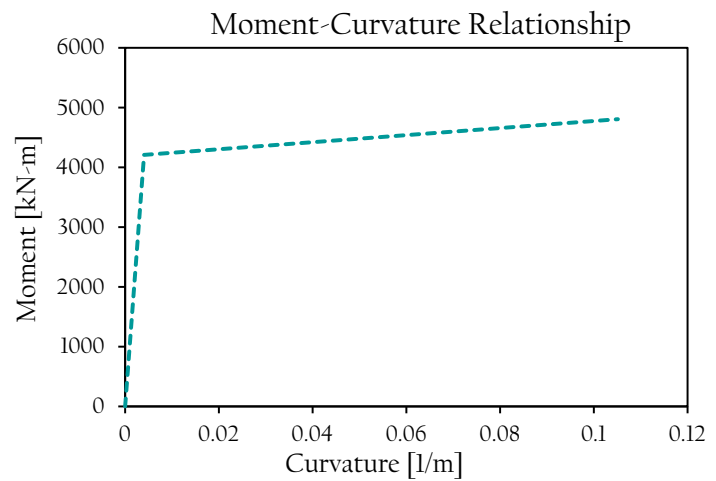
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	f'c (MPa)	Iy (m4)	Weight SS (kN/m)
537	61.4498	-149.3699	2	38.56	38.55	0	0	420	420	600	28	84.31	180

Substructure						Substructure						Abutments	
RC or RCFST	# Columns/pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	3	12.8	13.9	0	0	1.22	50.8	16	32.26	15.875	76.2	28400	40

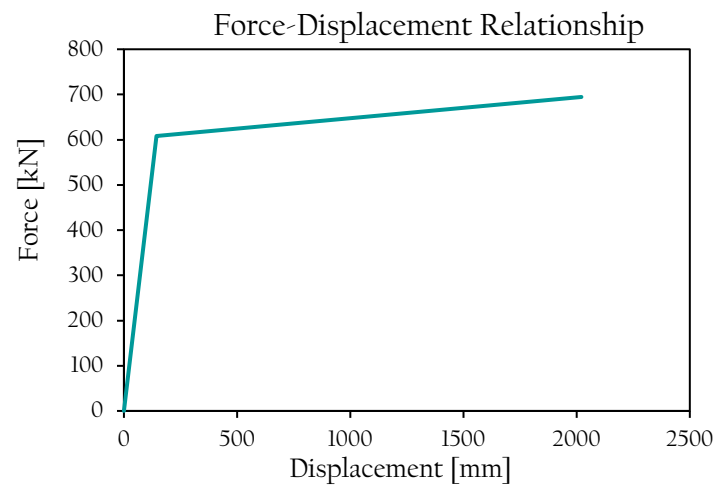
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0041	4210
0.1052	4809



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
144.25	607.88
2020.85	694.42



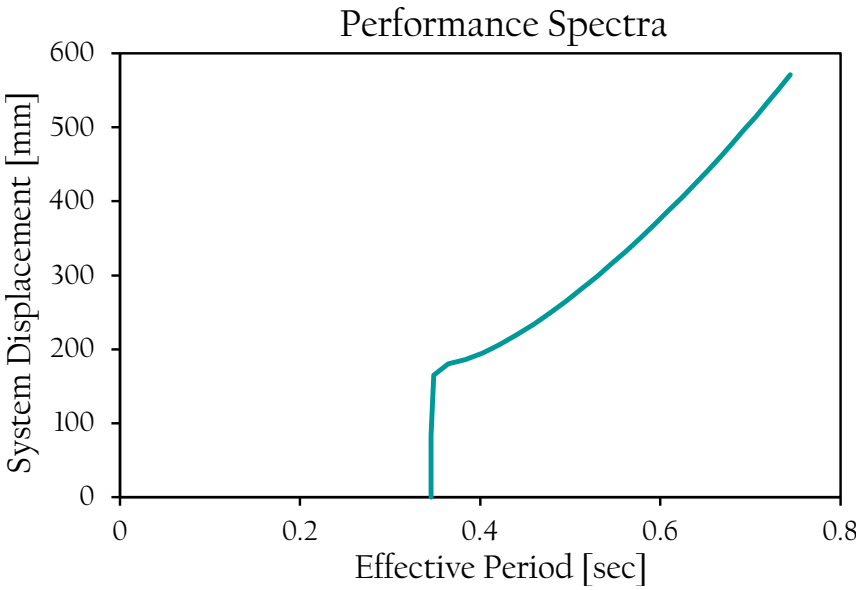
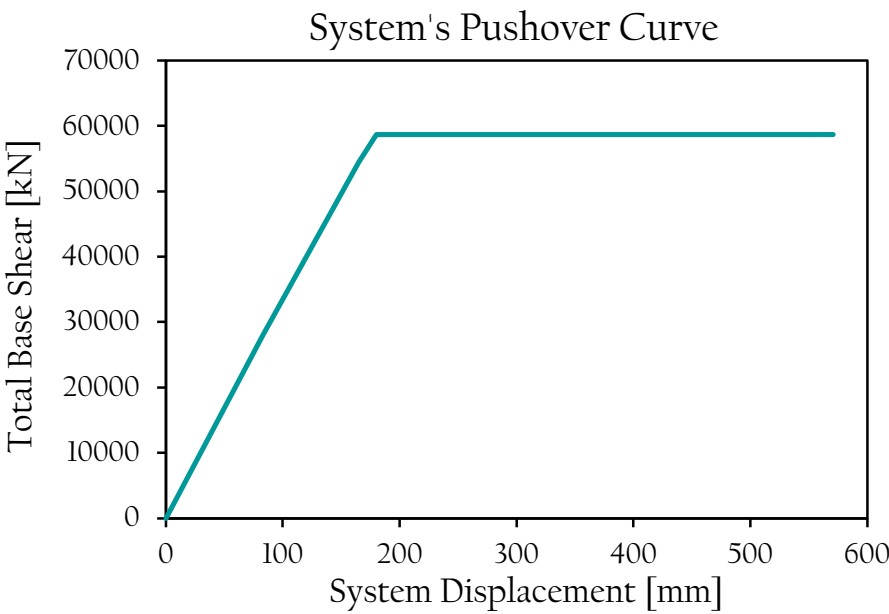
- Performance Limit States – Displacements

Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	108
Equivalent Yield	144
Fully Operational (PL3)	268
Operational (PL2)	852
Life Safety (PL1)	1047

- System's Pushover Curve & Performance Spectra

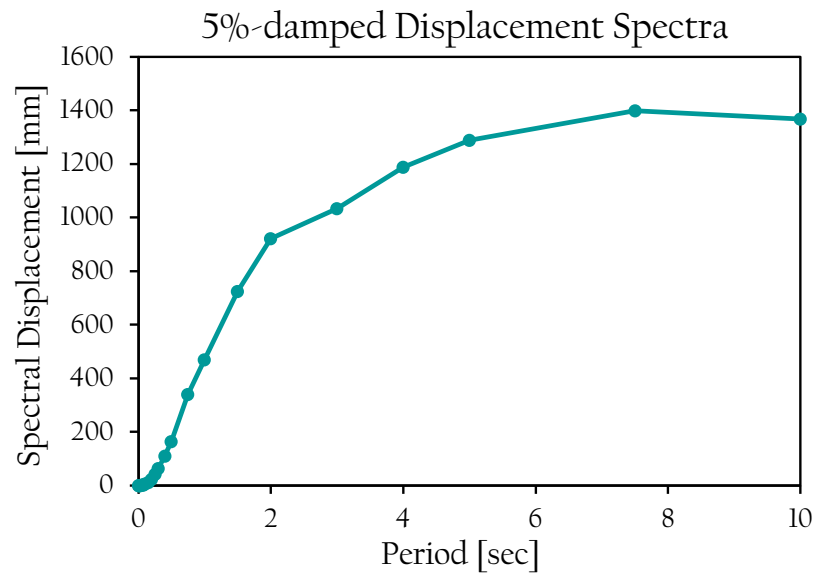
System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.34533	0	0	0
83	27919	0.34533	18.81	95.49	18.81
165	54429	0.34865	37.04	188.36	37.05
180	58633	0.36476	55.74	210.49	55.84
186	58634	0.3838	74.6	220.97	74.73
195	58636	0.40281	93.48	233.26	93.65
206	58638	0.42146	112.37	246.92	112.57
220	58640	0.44125	131.26	261.61	131.5
234	58642	0.45948	150.16	277.1	150.43
249	58644	0.47745	169.06	293.18	169.37
265	58647	0.49531	187.97	309.74	188.31
282	58649	0.51319	206.89	326.67	207.26
299	58651	0.53026	225.81	343.89	226.22
316	58654	0.54664	244.75	361.36	245.18
333	58656	0.56243	263.69	379.02	264.16
351	58659	0.5785	282.64	396.85	283.14
369	58661	0.59407	301.6	414.82	302.13
387	58664	0.60918	320.57	432.92	321.14
405	58666	0.62387	339.56	451.11	340.15
423	58669	0.63818	358.65	469.41	359.08
441	58671	0.65214	377.66	487.78	378.11
460	58674	0.6665	396.69	506.21	397.16
478	58676	0.67983	415.72	524.71	416.21
497	58679	0.69357	434.77	543.27	435.28
515	58681	0.70635	453.83	561.88	454.37
534	58684	0.71955	472.9	580.54	473.46
552	58687	0.73184	491.99	599.24	492.57

571	58689	0.74457	511.09	617.99	511.69
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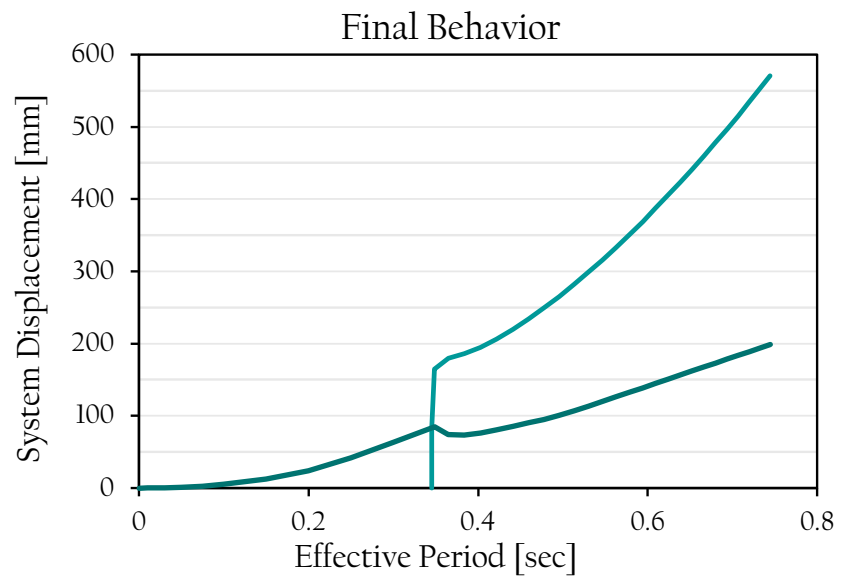
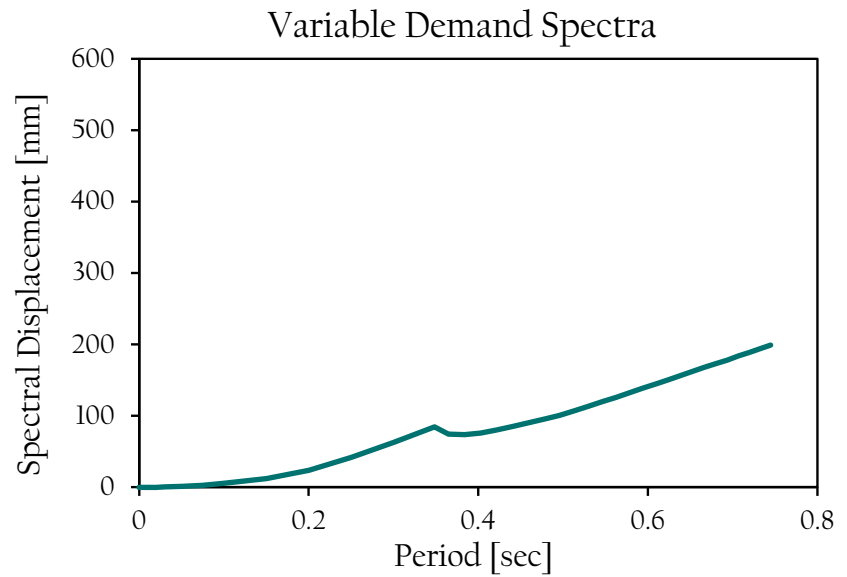
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.031
0.02	0.128
0.03	0.311
0.05	1.037
0.075	2.794
0.1	5.349
0.15	12.415
0.2	23.671
0.25	41.841
0.3	62.995
0.4	109.652
0.5	163.252
0.75	339.239
1	469.379
1.5	724.680
2	920.959
3	1033.852
4	1187.966
5	1287.790
7.5	1399.184
10	1367.993



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.031
0.02	0.128
0.03	0.311
0.05	1.037
0.075	2.794
0.1	5.349
0.15	12.415
0.2	23.671
0.25	41.841
0.3	62.995
0.3486	84.927
0.3648	74.3
0.3838	73.514
0.4028	75.961
0.4215	80.171
0.4412	85.304
0.4595	90.19
0.4774	95.172
0.4953	100.244
0.5132	106.78
0.5303	113.5
0.5466	119.954
0.5624	126.181
0.5785	132.554
0.5941	138.732
0.6092	144.728
0.6239	150.564
0.6382	156.25
0.6521	161.803
0.6665	167.533
0.6798	172.838
0.6936	178.324
0.7063	183.414
0.7196	188.689



0.7318	193.587
0.7446	198.676

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	84
T_{eff} [sec]	0.35
Displacement Profile	19 mm – 97 mm – 19 mm
Column's Ductility	Pier I: < 1.0
Highest Limit State Achieved	Pier I: Elastic Behavior

3. Bridge 595

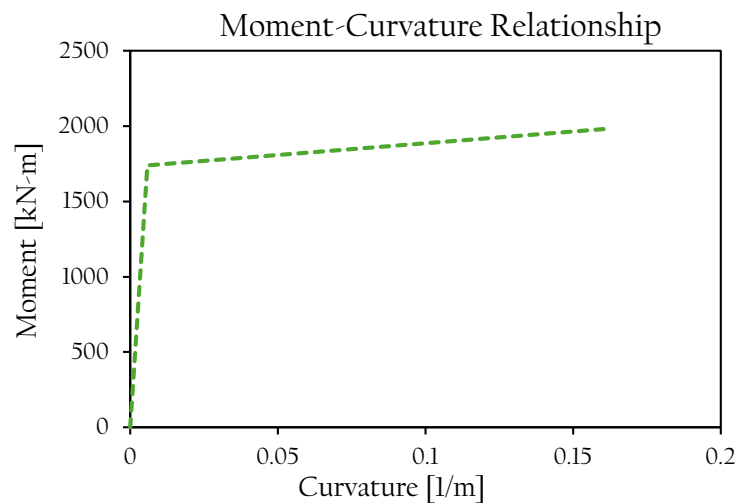
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	f'c (MPa)	Iy (m4)	Weight SS (kN/m)
595	64.0232	-145.7252	2	36.65	36.65	0	0	420	420	600	28	52.76	180

Substructure						Substructure						Abutments	
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	13.1	5	0	0	0.88	50.8	12	28.65	15.875	76.2	28400	40

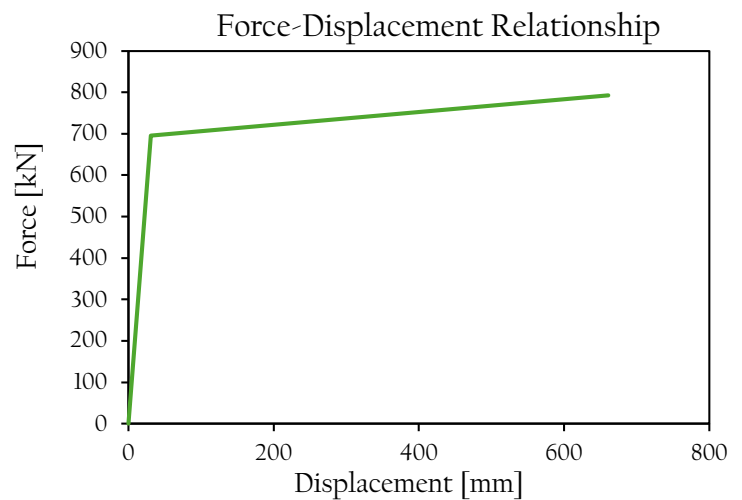
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0059	1740
0.1613	1982



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
30.96	695.86
660.83	792.73



- Performance Limit States - Displacements

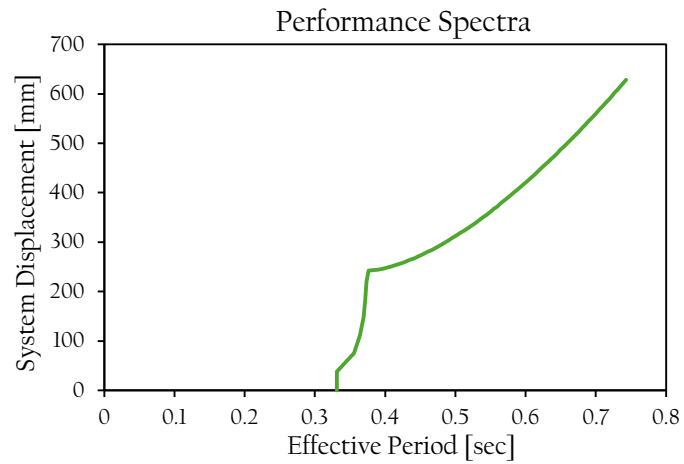
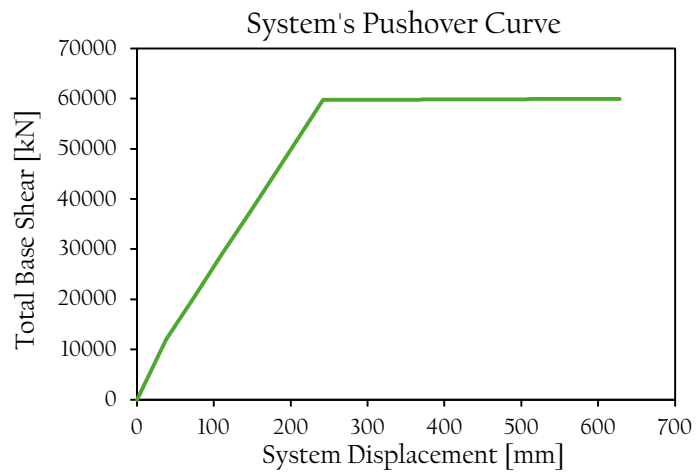
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	24
Equivalent Yield	31
Fully Operational (PL3)	67
Operational (PL2)	307
Life Safety (PL1)	386

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.33153	0	0	0
38	12054	0.33153	6.52	42.77	6.52
75	20576	0.35566	12.51	83.77	12.51
111	29073	0.36372	18.47	124.64	18.47
148	37563	0.36935	24.43	165.47	24.43
184	46050	0.37186	30.39	206.28	30.39
220	54536	0.37358	36.35	247.06	36.35
242	59732	0.37665	42.43	272.69	42.43
243	59733	0.38315	48.73	274.51	48.73
244	59735	0.38935	55.05	276.56	55.05
246	59736	0.39605	61.37	278.89	61.37
248	59738	0.40247	67.7	281.44	67.7
251	59739	0.40943	74.05	284.23	74.05
254	59741	0.41612	80.4	287.23	80.4
257	59743	0.42254	86.77	290.43	86.77
260	59745	0.42872	93.14	293.82	93.14
264	59747	0.43548	99.51	297.4	99.51
267	59750	0.44118	105.9	301.14	105.9
271	59752	0.44749	112.29	305.04	112.29
275	59755	0.4536	118.69	309.09	118.69
280	59757	0.46033	125.09	313.29	125.09
284	59760	0.46606	131.5	317.59	131.5
288	59763	0.47162	137.92	322.03	137.92
293	59765	0.47783	144.34	326.59	144.34
298	59768	0.48388	150.77	331.25	150.77
303	59771	0.48978	157.21	336.02	157.21
308	59774	0.49554	163.65	340.88	163.65

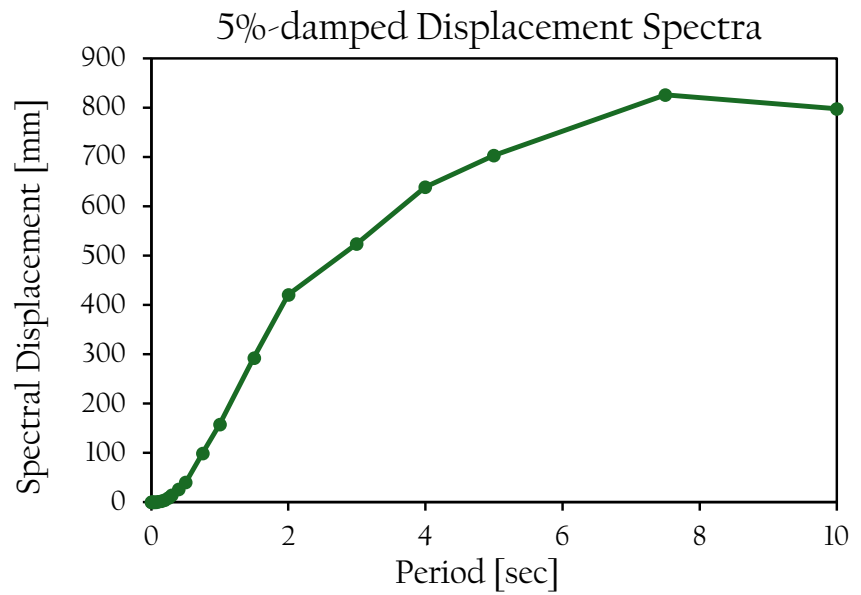
313	59777	0.50117	170.09	345.82	170.09
318	59780	0.50667	176.54	350.85	176.54
323	59783	0.51206	183	355.96	183
328	59787	0.51734	189.46	361.14	189.46
333	59790	0.52251	195.93	366.39	195.93
339	59793	0.52837	202.4	371.72	202.4
344	59796	0.53336	208.88	377.09	208.88
350	59800	0.53902	215.36	382.52	215.36
355	59803	0.54383	221.85	388	221.85
361	59806	0.54933	228.35	393.53	228.35
367	59810	0.55474	234.85	399.11	234.85
372	59813	0.55932	241.36	404.74	241.36
378	59817	0.56458	247.87	410.41	247.87
384	59820	0.56977	254.39	416.12	254.39
390	59824	0.57489	260.91	421.86	260.91
395	59827	0.57922	267.44	427.65	267.44
401	59831	0.58422	273.98	433.47	273.98
407	59835	0.58916	280.52	439.33	280.52
413	59838	0.59404	287.06	445.21	287.06
419	59842	0.59886	293.62	451.13	293.62
425	59846	0.60363	300.18	457.08	300.18
431	59849	0.60836	306.74	463.05	306.74
437	59853	0.61303	313.31	469.06	313.31
443	59857	0.61765	319.89	475.08	319.89
449	59860	0.62223	326.47	481.14	326.47
456	59864	0.62745	333.06	487.22	333.06
462	59868	0.63194	339.66	493.32	339.66
468	59872	0.63638	346.26	499.44	346.26
474	59875	0.64079	352.86	505.59	352.86
480	59879	0.64515	359.48	511.75	359.48
487	59883	0.65015	366.1	517.94	366.1
493	59887	0.65443	372.72	524.15	372.72
499	59891	0.65868	379.35	530.37	379.35
505	59895	0.6629	385.99	536.61	385.99
512	59898	0.66774	392.64	542.88	392.64
518	59902	0.67189	399.29	549.16	399.29
524	59906	0.676	405.94	555.45	405.94
531	59910	0.68073	412.6	561.76	412.6
537	59914	0.68478	419.27	568.09	419.27

544	59918	0.68944	425.95	574.44	425.95
550	59922	0.69343	432.63	580.8	432.63
556	59926	0.69739	439.32	587.17	439.32
563	59930	0.70195	446.01	593.56	446.01
569	59933	0.70586	452.71	599.96	452.71
576	59937	0.71036	459.42	606.38	459.42
582	59941	0.71421	466.13	612.81	466.13
589	59945	0.71865	472.85	619.26	472.85
595	59949	0.72245	479.58	625.72	479.58
602	59953	0.72683	486.31	632.19	486.31
608	59957	0.73058	493.05	638.67	493.05
615	59961	0.73491	499.79	645.17	499.79
621	59965	0.73861	506.54	651.67	506.54
628	59969	0.74289	513.3	658.19	513.3



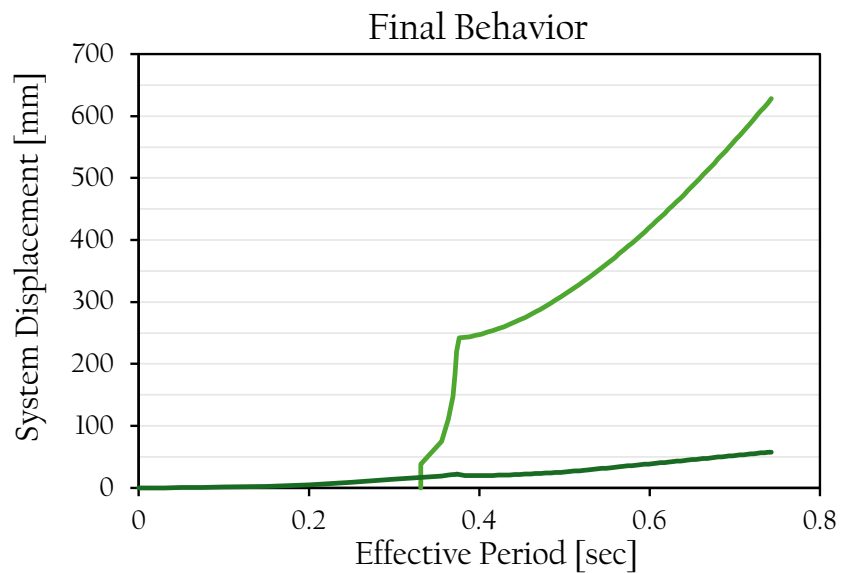
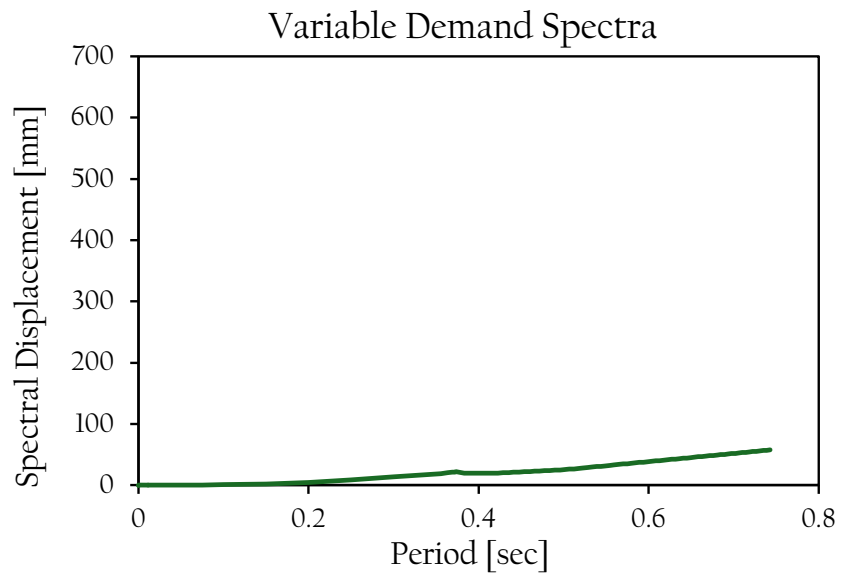
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.007
0.02	0.028
0.03	0.064
0.05	0.197
0.075	0.518
0.1	0.986
0.15	2.334
0.2	4.805
0.25	8.893
0.3	14.130
0.4	26.207
0.5	40.443
0.75	99.509
1	158.249
1.5	293.053
2	420.932
3	524.140
4	638.826
5	703.759
7.5	825.714
10	797.757



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.007
0.02	0.028
0.03	0.064
0.05	0.197
0.075	0.518
0.1	0.986
0.15	2.334
0.2	4.805
0.25	8.893
0.3	14.13
0.3557	19.097
0.3637	20.226
0.3693	21.086
0.3719	21.553
0.3736	21.894
0.3767	21.28
0.3831	20.134
0.3893	19.57
0.3961	19.368
0.4025	19.362
0.4094	19.586
0.4161	19.857
0.4225	20.157
0.4287	20.471
0.4355	20.872
0.4412	21.197
0.4475	21.598
0.4536	21.995
0.4603	22.462
0.4661	22.847
0.4716	23.226
0.4778	23.671
0.4839	24.108
0.4898	24.537



0.4955	24.959
0.5012	25.441
0.5067	26.17
0.5121	26.883
0.5173	27.582
0.5225	28.267
0.5284	29.052
0.5334	29.711
0.539	30.47
0.5438	31.106
0.5493	31.843
0.5547	32.567
0.5593	33.173
0.5646	33.878
0.5698	34.573
0.5749	35.258
0.5792	35.831
0.5842	36.5
0.5892	37.161
0.594	37.814
0.5989	38.46
0.6036	39.099
0.6084	39.731
0.613	40.356
0.6176	40.975
0.6222	41.588
0.6275	42.292
0.6319	42.892
0.6364	43.487
0.6408	44.077
0.6452	44.662
0.6501	45.335
0.6544	45.909
0.6587	46.478
0.6629	47.043
0.6677	47.696
0.6719	48.251
0.676	48.802
0.6807	49.439

0.6848	49.982
0.6894	50.61
0.6934	51.144
0.6974	51.675
0.702	52.29
0.7059	52.814
0.7104	53.42
0.7142	53.936
0.7186	54.535
0.7224	55.044
0.7268	55.635
0.7306	56.137
0.7349	56.721
0.7386	57.217
0.7429	57.793

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	17
T_{eff} [sec]	0.33
Displacement Profile	1 mm – 8 mm – 1 mm
Column's Ductility	Pier I: < 1.0
Highest Limit State Achieved	Pier I: Elastic Behavior

4. Bridge 639

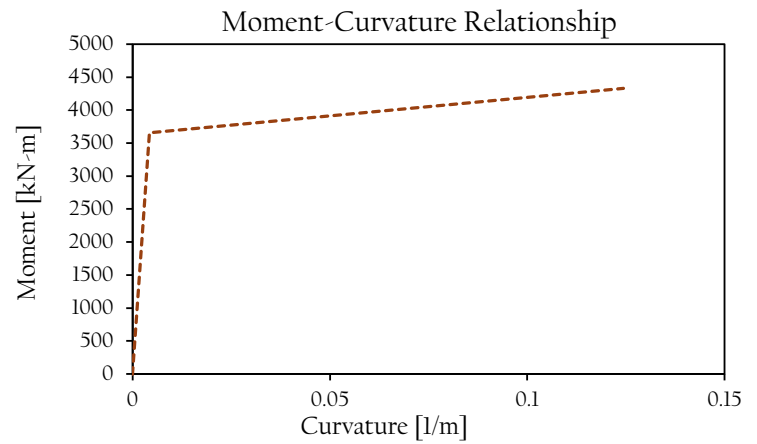
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				f _y (MPa)	f _y _h (MPa)	f _u (MPa)	f _c (MPa)	I _y (m ⁴)	Weight SS (kN/m)
639	60.9387	-149.1674	2	36.12	36.12	0	0	420	420	600	28	61.43	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	11	5.2	0	0	1.17	50.8	16	32.26	15.875	76.2	28400	40

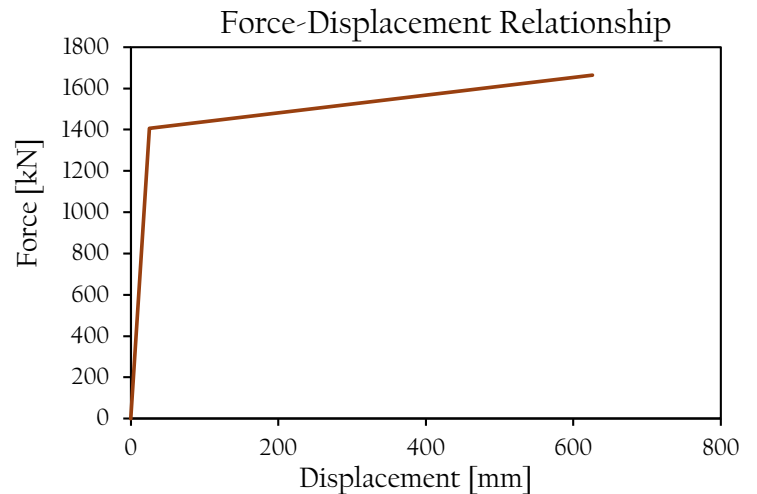
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0043	3659
0.1243	4327



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
25.49	1407.15
625.68	1664.35



- Performance Limit States – Displacements

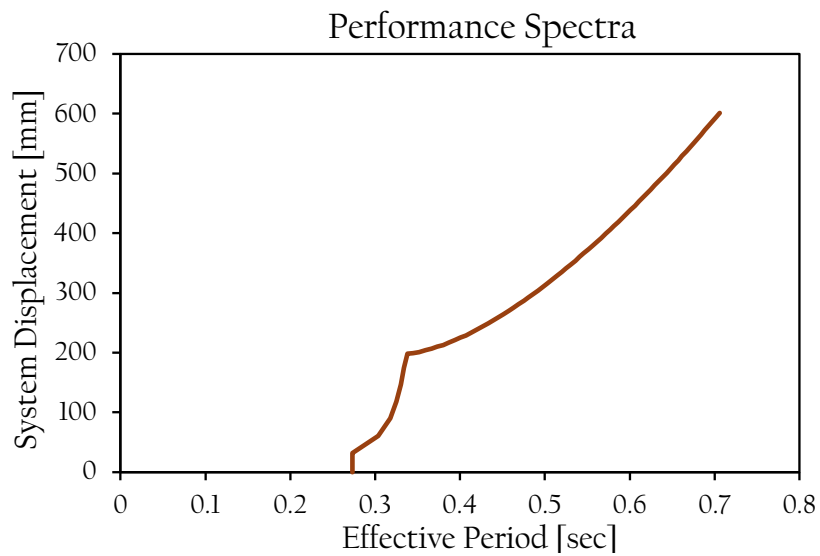
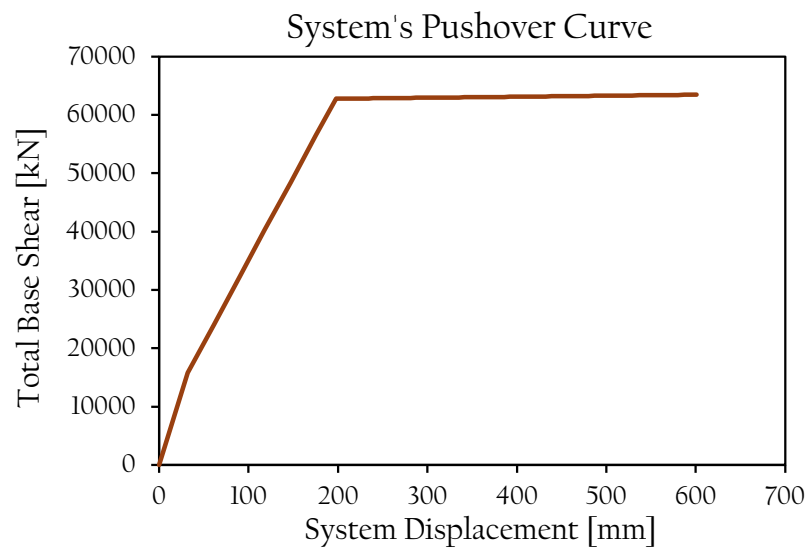
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	18
Equivalent Yield	25
Fully Operational (PL3)	64
Operational (PL2)	231
Life Safety (PL1)	288

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.27298	0	0	0
32	15763	0.27298	7.12	36.62	7.12
61	24028	0.30343	12.9	69.7	12.9
90	32194	0.31776	18.62	102.28	18.62
118	40326	0.32476	24.3	134.7	24.3
147	48444	0.33051	29.98	167.04	29.98
175	56554	0.33362	35.65	199.35	35.65
198	62770	0.33812	41.46	224.51	41.46
199	62773	0.34467	47.82	226.47	47.82
201	62777	0.35172	54.22	228.75	54.22
204	62781	0.35929	60.64	231.3	60.64
207	62786	0.36651	67.09	234.16	67.09
210	62792	0.37338	73.56	237.29	73.56
213	62797	0.37993	80.05	240.67	80.05
217	62804	0.38705	86.56	244.29	86.56
221	62810	0.39388	93.08	248.12	93.08
225	62817	0.40043	99.62	252.16	99.62
229	62824	0.40672	106.18	256.38	106.18
234	62832	0.41365	112.75	260.78	112.75
239	62840	0.42035	119.33	265.34	119.33
244	62848	0.42684	125.93	270.05	125.93
249	62856	0.43312	132.54	274.9	132.54
254	62865	0.43922	139.16	279.87	139.16
259	62873	0.44516	145.8	284.96	145.8
264	62882	0.45093	152.45	290.16	152.45
270	62891	0.4574	159.12	295.46	159.12
275	62901	0.46288	165.8	300.85	165.8

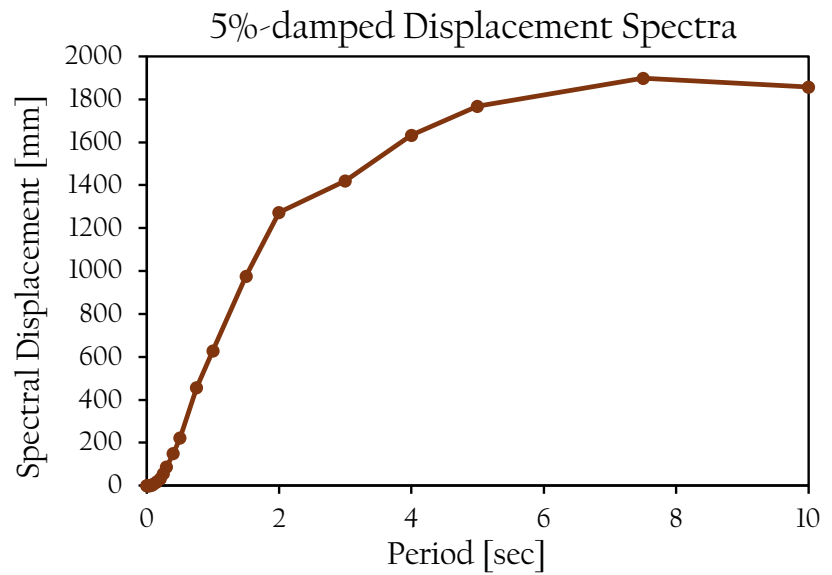
281	62910	0.46908	172.49	306.34	172.49
287	62920	0.47513	179.19	311.9	179.19
292	62929	0.48025	185.91	317.55	185.91
298	62939	0.48609	192.64	323.26	192.64
304	62949	0.49181	199.39	329.05	199.39
310	62959	0.49743	206.15	334.89	206.15
316	62969	0.50296	212.92	340.8	212.92
322	62979	0.50839	219.71	346.77	219.71
329	62990	0.51452	226.51	352.79	226.51
335	63000	0.51979	233.32	358.86	233.32
341	63011	0.52497	240.15	364.98	240.15
347	63021	0.53008	246.99	371.15	246.99
354	63032	0.53588	253.85	377.36	253.85
360	63042	0.54085	260.72	383.61	260.72
366	63053	0.54576	267.61	389.91	267.61
373	63064	0.55134	274.51	396.26	274.51
379	63075	0.55613	281.43	402.64	281.43
386	63086	0.56158	288.36	409.05	288.36
392	63097	0.56625	295.3	415.49	295.3
399	63108	0.57158	302.26	421.97	302.26
406	63119	0.57686	309.24	428.48	309.24
412	63131	0.58137	316.23	435.03	316.23
419	63142	0.58653	323.23	441.6	323.23
426	63153	0.59164	330.25	448.21	330.25
432	63165	0.59601	337.29	454.84	337.29
439	63176	0.60103	344.34	461.51	344.34
446	63187	0.60599	351.4	468.2	351.4
453	63199	0.6109	358.48	474.92	358.48
460	63211	0.61577	365.58	481.67	365.58
466	63222	0.61993	372.69	488.44	372.69
473	63234	0.62472	379.81	495.24	379.81
480	63245	0.62946	386.96	502.07	386.96
487	63257	0.63416	394.11	508.92	394.11
494	63269	0.63882	401.28	515.79	401.28
501	63281	0.64345	408.47	522.69	408.47
508	63293	0.64803	415.68	529.62	415.68
515	63305	0.65257	422.89	536.56	422.89
522	63317	0.65708	430.13	543.53	430.13
529	63329	0.66156	437.38	550.52	437.38

537	63341	0.66662	444.64	557.54	444.64
544	63353	0.67102	451.92	564.57	451.92
551	63365	0.67539	459.22	571.63	459.22
558	63377	0.67972	466.53	578.71	466.53
565	63389	0.68403	473.86	585.81	473.86
572	63401	0.6883	481.2	592.94	481.2
580	63414	0.69314	488.56	600.08	488.56
587	63426	0.69735	495.94	607.25	495.94
594	63438	0.70154	503.33	614.43	503.33
601	63450	0.70569	510.73	621.64	510.73



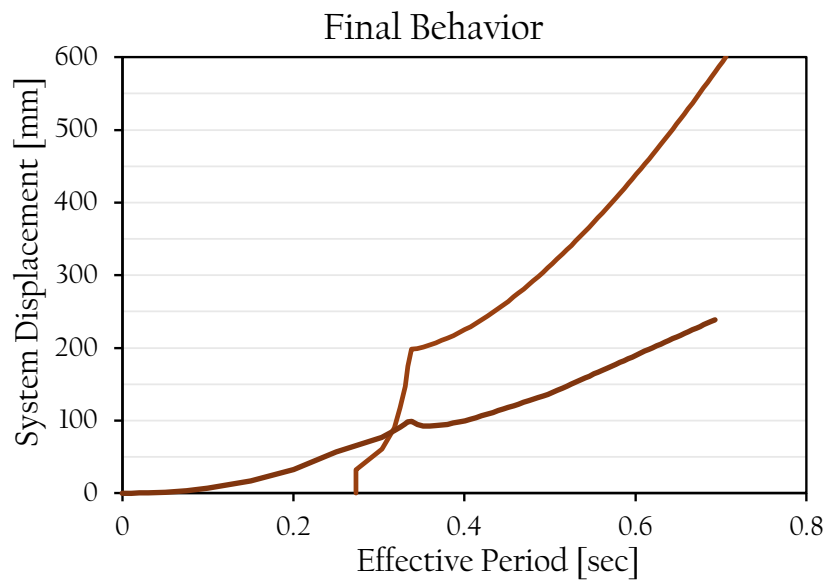
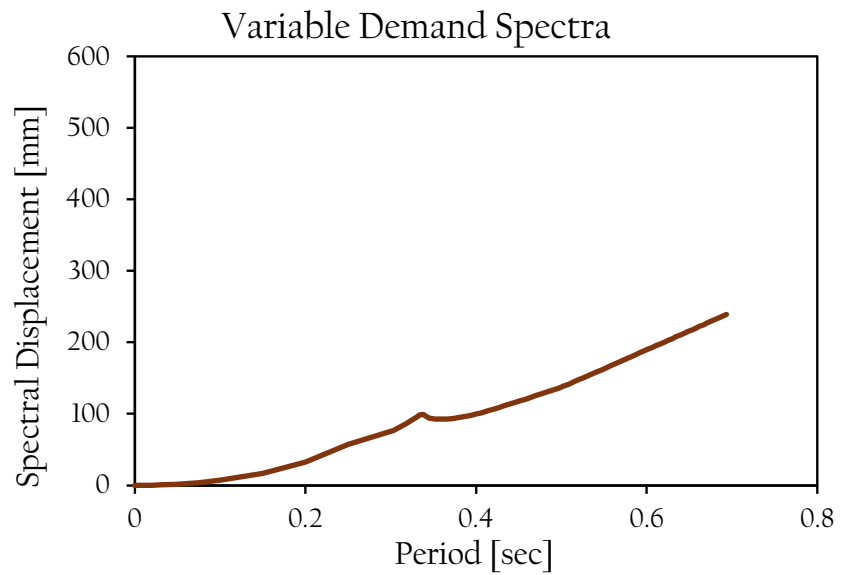
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.041
0.02	0.166
0.03	0.401
0.05	1.325
0.075	3.567
0.1	6.935
0.15	16.532
0.2	32.410
0.25	57.140
0.3	86.880
0.4	150.014
0.5	223.699
0.75	457.382
1	629.224
1.5	977.112
2	1272.266
3	1420.587
4	1632.395
5	1768.281
7.5	1899.338
10	1856.136



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.041
0.02	0.166
0.03	0.401
0.05	1.325
0.075	3.567
0.1	6.935
0.15	16.532
0.2	32.41
0.25	57.14
0.3034	77.039
0.3178	85.975
0.3248	91.191
0.3305	95.616
0.3336	98.395
0.3381	99.028
0.3447	94.327
0.3517	92.559
0.3593	92.413
0.3665	92.897
0.3734	93.74
0.3799	94.794
0.387	96.349
0.3939	97.962
0.4004	99.627
0.4067	101.702
0.4136	104.165
0.4203	106.581
0.4268	108.947
0.4331	111.264
0.4392	113.53
0.4452	115.751
0.4509	117.926
0.4574	120.45
0.4629	122.536



0.4691	124.969
0.4751	127.356
0.4803	129.33
0.4861	131.64
0.4918	133.913
0.4974	136.151
0.503	138.714
0.5084	141.543
0.5145	144.773
0.5198	147.515
0.525	150.215
0.5301	152.881
0.5359	155.939
0.5409	158.534
0.5458	161.094
0.5513	164.04
0.5561	166.537
0.5616	169.417
0.5662	171.856
0.5716	174.673
0.5769	177.458
0.5814	179.815
0.5865	182.545
0.5916	185.245
0.596	187.531
0.601	190.181
0.606	192.805
0.6109	195.402
0.6158	197.975
0.6199	200.156
0.6247	202.686
0.6295	205.195
0.6342	207.681
0.6388	210.145
0.6434	212.589
0.648	215.013
0.6526	217.418
0.6571	219.804
0.6616	222.171

0.6666	224.864
0.671	227.194
0.6754	229.506
0.6797	231.802
0.684	234.081
0.6883	236.344
0.6931	238.918

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	84
T_{eff} [sec]	0.31
Displacement Profile	17 mm – 96 mm – 17 mm
Column's Ductility	Pier 1: 3.8
Highest Limit State Achieved	Pier 1: Fully Operational (PL3)

5. Bridge 654

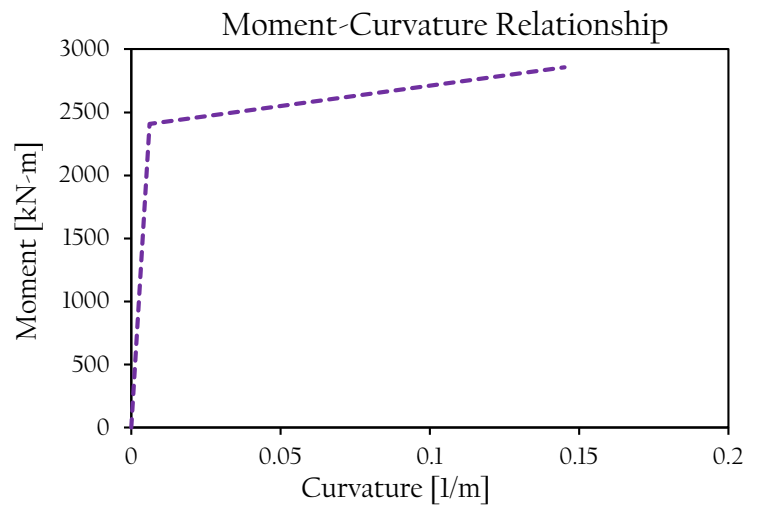
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				f _y (MPa)	f _y h (MPa)	f _u (MPa)	f _c (MPa)	I _y (m4)	Weight SS (kN/m)
654	62.856	-143.6968	2	36.68	36.68	0	0	420	420	600	28	47.73	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	11.9	4.95	0	0	0.88	50.8	16	32.26	15.875	76.2	28400	40

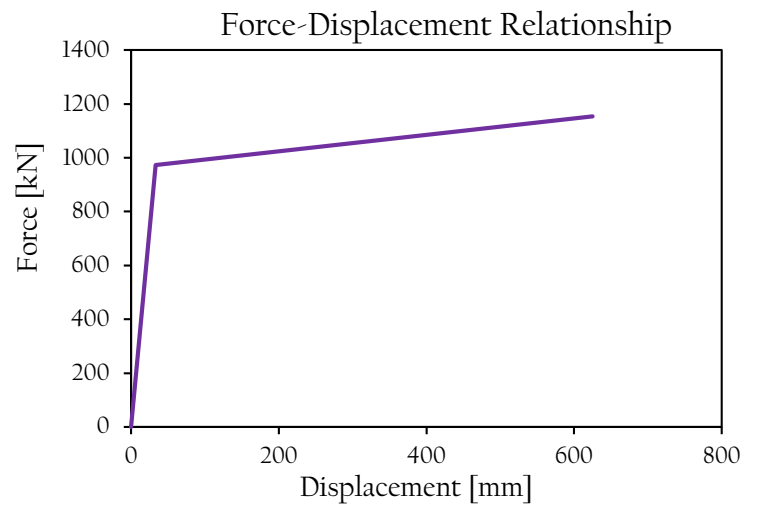
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0062	2409
0.1452	2856



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
33.37	973.22
625.09	1154.18



- Performance Limit States – Displacements

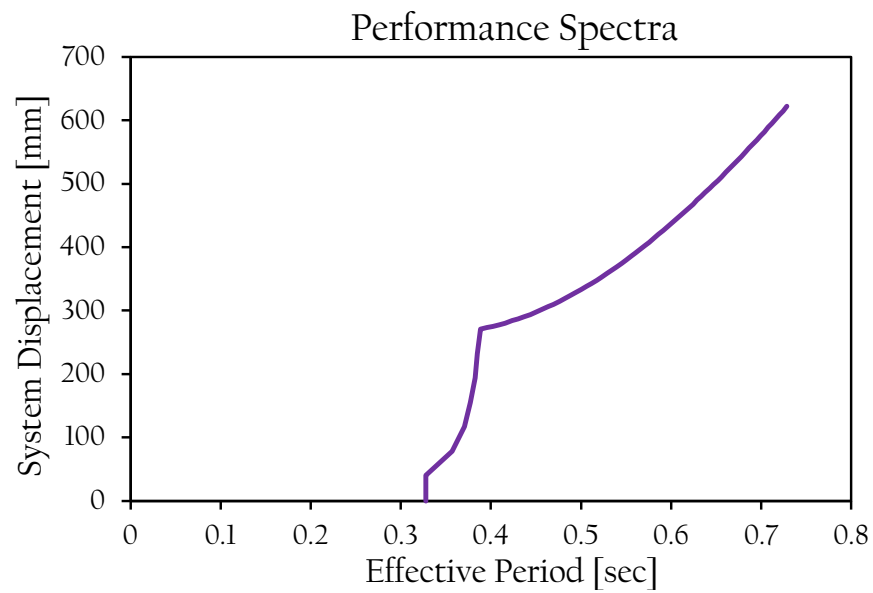
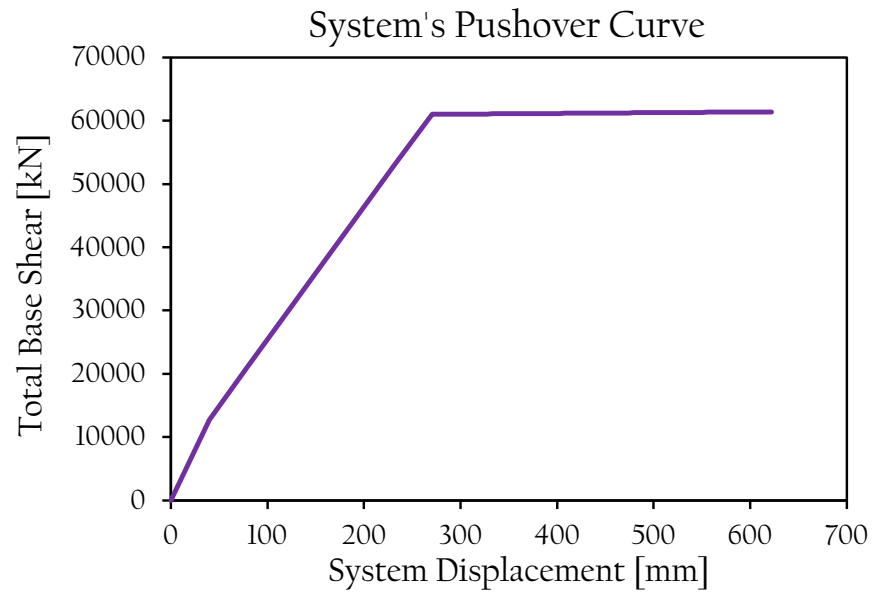
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	25
Equivalent Yield	33
Fully Operational (PL3)	66
Operational (PL2)	340
Life Safety (PL1)	425

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.32796	0	0	0
40	12752	0.32796	6.23	44.34	6.23
78	20856	0.35703	11.9	87.25	11.9
117	28934	0.37088	17.55	129.96	17.55
155	37002	0.3773	23.2	172.62	23.2
194	45066	0.38237	28.84	215.25	28.84
232	53129	0.38503	34.48	257.86	34.48
271	61019	0.38837	40.13	299.59	40.13
273	61020	0.39538	46.22	300.97	46.22
275	61022	0.40218	52.34	302.55	52.34
278	61024	0.40951	58.48	304.37	58.48
280	61027	0.41588	64.64	306.42	64.64
284	61030	0.42352	70.82	308.69	70.82
287	61033	0.43019	77.02	311.16	77.02
290	61036	0.43664	83.23	313.83	83.23
294	61039	0.44364	89.46	316.67	89.46
298	61043	0.45042	95.7	319.72	95.7
302	61047	0.45699	101.95	322.94	101.95
306	61051	0.46336	108.22	326.33	108.22
310	61056	0.46953	114.51	329.87	114.51
315	61060	0.47627	120.8	333.57	120.8
319	61065	0.48208	127.11	337.41	127.11
324	61070	0.48847	133.42	341.39	133.42
328	61075	0.49394	139.75	345.49	139.75
333	61080	0.50001	146.09	349.72	146.09
338	61085	0.50593	152.45	354.06	152.45
343	61091	0.5117	158.81	358.51	158.81

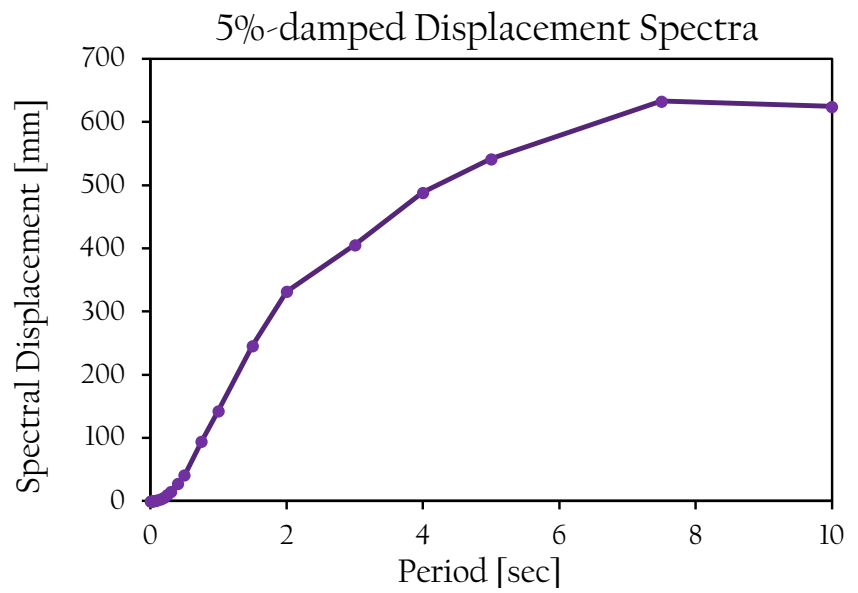
348	61096	0.51734	165.18	363.06	165.18
353	61102	0.52285	171.57	367.71	171.57
359	61108	0.52897	177.96	372.45	177.96
364	61114	0.53424	184.37	377.28	184.37
369	61120	0.53941	190.78	382.19	190.78
375	61126	0.54519	197.21	387.18	197.21
380	61132	0.55015	203.65	392.24	203.65
386	61138	0.55573	210.1	397.37	210.1
391	61145	0.5605	216.56	402.58	216.56
397	61151	0.56591	223.02	407.84	223.02
403	61157	0.57123	229.5	413.17	229.5
408	61164	0.57575	235.99	418.55	235.99
414	61171	0.58091	242.49	423.99	242.49
420	61177	0.586	249.01	429.49	249.01
426	61184	0.59101	255.53	435.03	255.53
432	61191	0.59596	262.06	440.62	262.06
438	61198	0.60084	268.6	446.26	268.6
444	61205	0.60566	275.16	451.94	275.16
450	61212	0.61041	281.72	457.67	281.72
456	61219	0.61511	288.3	463.44	288.3
462	61226	0.61976	294.88	469.24	294.88
468	61233	0.62435	301.48	475.09	301.48
474	61240	0.62889	308.08	480.97	308.08
480	61248	0.63338	314.7	486.89	314.7
486	61255	0.63783	321.33	492.84	321.33
493	61262	0.64288	327.97	498.83	327.97
499	61270	0.64723	334.62	504.85	334.62
505	61277	0.65154	341.28	510.89	341.28
511	61284	0.65581	347.95	516.99	347.95
518	61292	0.66067	354.64	523.1	354.64
524	61299	0.66486	361.33	529.24	361.33
530	61307	0.66901	368.04	535.4	368.04
537	61315	0.67375	374.75	541.59	374.75
543	61322	0.67783	381.48	547.81	381.48
550	61330	0.68249	388.22	554.05	388.22
556	61337	0.6865	394.97	560.32	394.97
563	61345	0.69108	401.73	566.62	401.73
569	61353	0.69502	408.5	572.93	408.5
576	61361	0.69954	415.28	579.27	415.28

582	61368	0.70342	422.07	585.64	422.07
589	61376	0.70787	428.88	592.02	428.88
595	61384	0.71168	435.69	598.43	435.69
602	61392	0.71607	442.52	604.86	442.52
609	61400	0.72042	449.36	611.31	449.36
615	61408	0.72415	456.21	617.79	456.21
622	61416	0.72845	463.07	624.28	463.07



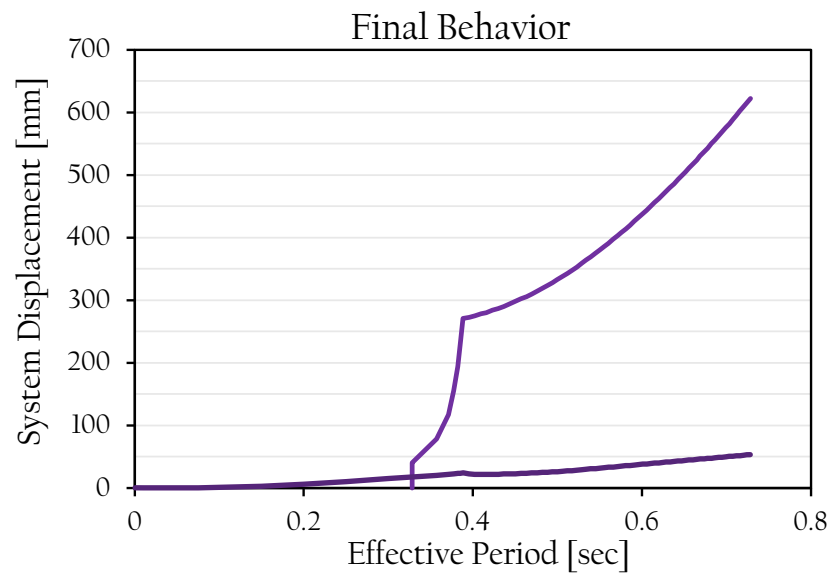
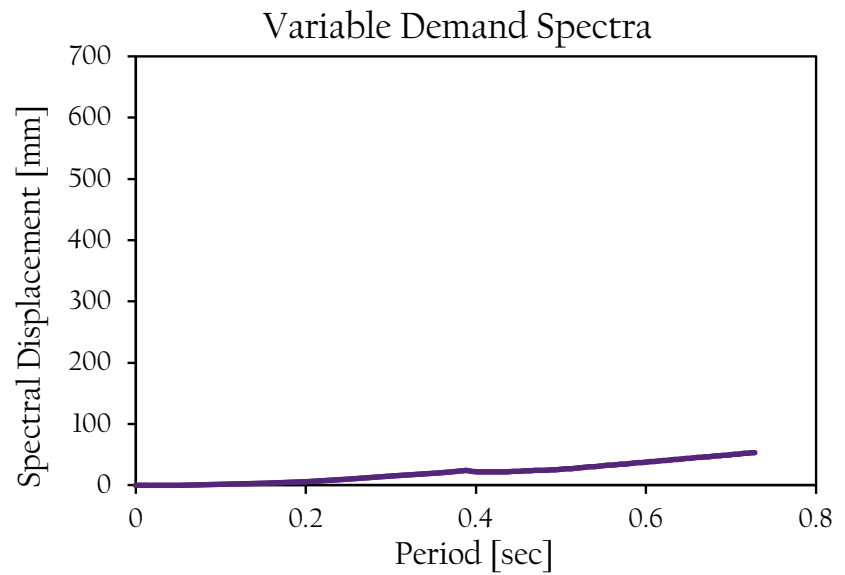
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.008
0.02	0.034
0.03	0.082
0.05	0.267
0.075	0.709
0.1	1.326
0.15	2.996
0.2	5.721
0.25	10.140
0.3	15.311
0.4	27.360
0.5	40.985
0.75	94.791
1	142.988
1.5	245.757
2	331.612
3	405.616
4	488.322
5	541.749
7.5	633.152
10	624.285



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.008
0.02	0.034
0.03	0.082
0.05	0.267
0.075	0.709
0.1	1.326
0.15	2.996
0.2	5.721
0.25	10.14
0.3	15.311
0.357	19.778
0.3709	21.524
0.3773	22.512
0.3824	23.315
0.385	23.811
0.3884	24.282
0.3954	22.74
0.4022	22.011
0.4095	21.792
0.4159	21.699
0.4235	21.875
0.4302	22.051
0.4366	22.274
0.4436	22.598
0.4504	22.938
0.457	23.289
0.4634	23.645
0.4695	24.002
0.4763	24.425
0.4821	24.778
0.4885	25.193
0.4939	25.538
0.5	25.942
0.5059	26.635



0.5117	27.312
0.5173	27.974
0.5228	28.621
0.529	29.353
0.5342	29.974
0.5394	30.582
0.5452	31.275
0.5501	31.86
0.5557	32.53
0.5605	33.094
0.5659	33.743
0.5712	34.381
0.5758	34.918
0.5809	35.538
0.586	36.15
0.591	36.753
0.596	37.348
0.6008	37.935
0.6057	38.515
0.6104	39.088
0.6151	39.655
0.6198	40.214
0.6244	40.768
0.6289	41.316
0.6334	41.857
0.6378	42.394
0.6429	43.009
0.6472	43.534
0.6515	44.055
0.6558	44.571
0.6607	45.163
0.6649	45.67
0.669	46.171
0.6737	46.749
0.6778	47.242
0.6825	47.81
0.6865	48.295
0.6911	48.855
0.695	49.331

0.6995	49.882
0.7034	50.352
0.7079	50.895
0.7117	51.357
0.7161	51.893
0.7204	52.424
0.7242	52.876
0.7284	53.401

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	17
T_{eff} [sec]	0.33
Displacement Profile	1 mm – 8 mm – 1 mm
Column's Ductility	Pier I: < 1.0
Highest Limit State Achieved	Pier I: Elastic Behavior

6. Bridge 663

- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	f c (MPa)	Iy (m4)	Weight SS (kN/m)
663	63.1092	-143.2944	3	36.68	34.14	36.68	0	420	420	600	28	46.61	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	11.9	4.04	4.22	0	0.88	50.8	16	32.26	15.875	76.2	28400	40

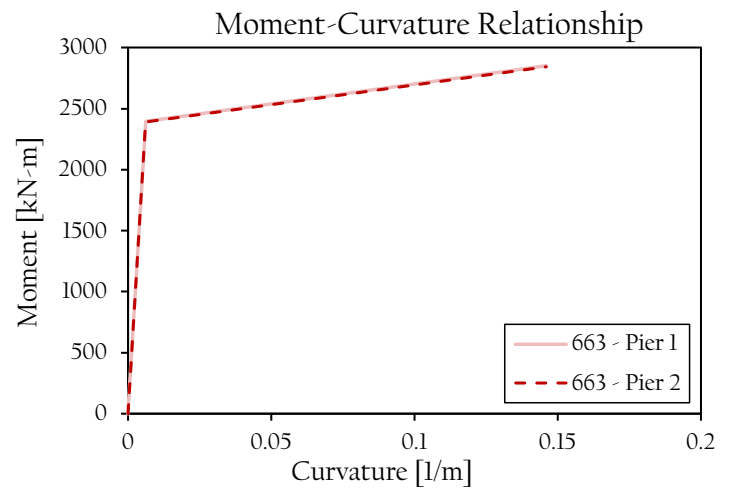
- Moment-Curvature Relationship

Pier 1

Curvature [1/m]	Moment [kN-m]
0	0
0.0062	2397
0.1459	2854

Pier 2

Curvature [1/m]	Moment [kN-m]
0	0
0.0062	2390
0.1461	2843



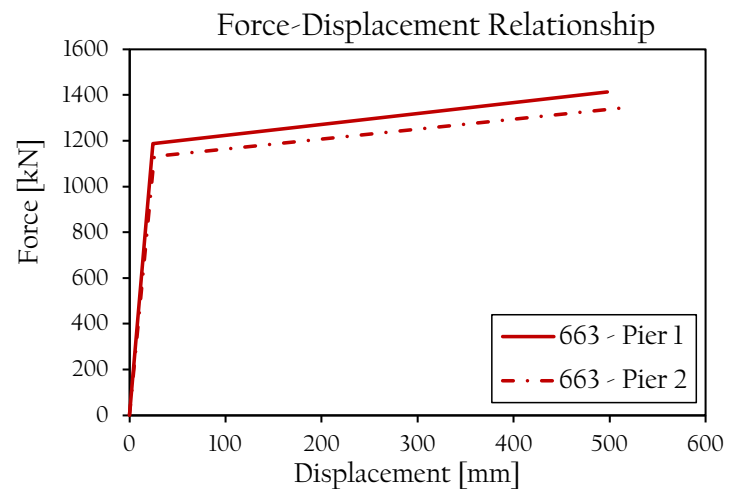
- Force-Displacement Relationship

Pier 1

Displacement [mm]	Force [kN]
0	0
24.13	1186.61
497.66	1412.82

Pier 2

Displacement [mm]	Force [kN]
0	0
25.59	1132.45
522.94	1347.49

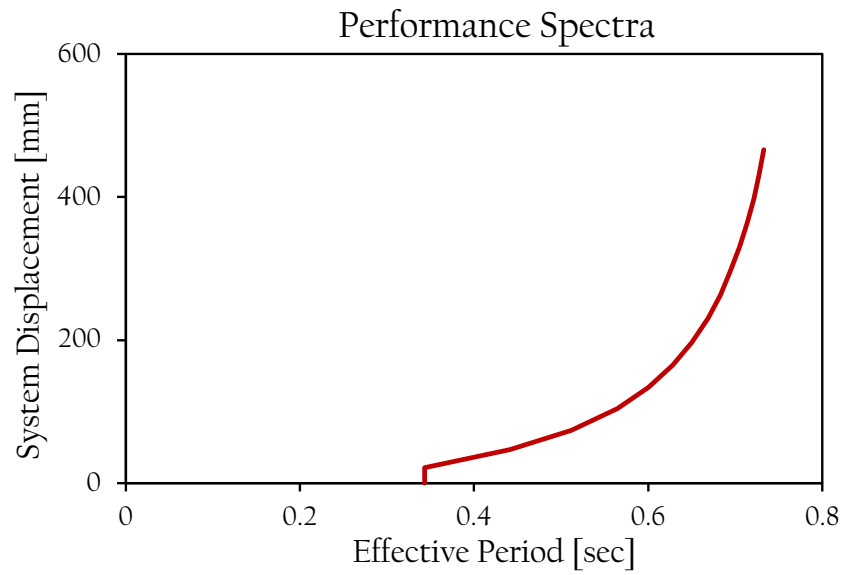
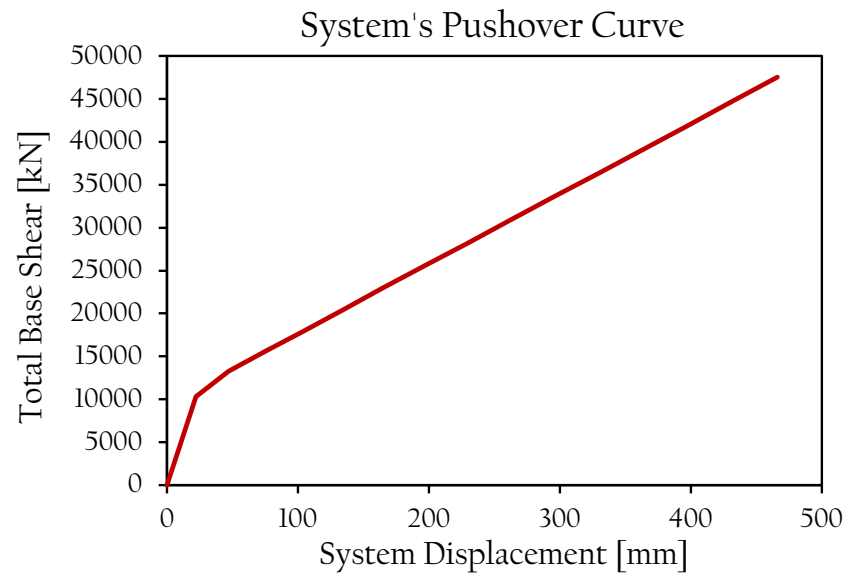


- Performance Limit States – Displacements

Performance Limit States	Columns Pier 1 Displacement [mm]	Columns Pier 2 Displacement [mm]
First Yield	18	19
Equivalent Yield	24	26
Fully Operational (PL3)	51	54
Operational (PL2)	267	281
Life Safety (PL1)	335	352

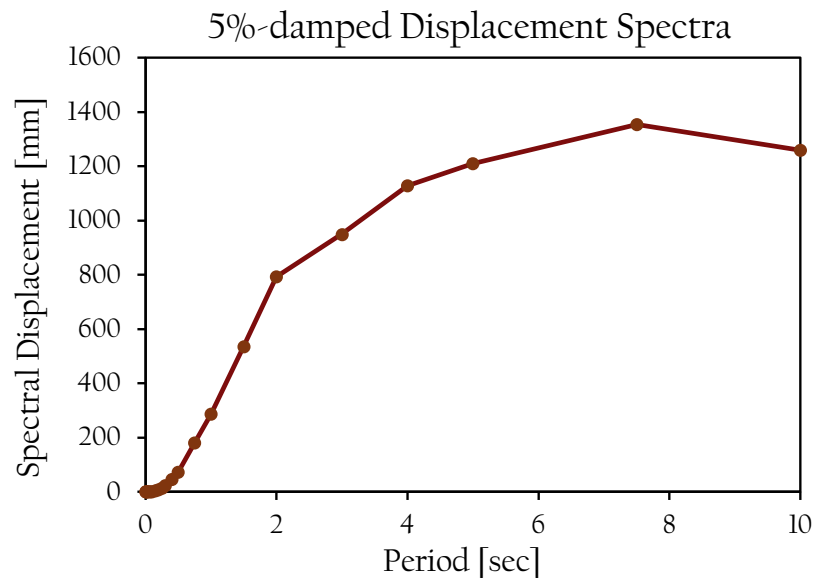
- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression			
			Abutment 1 [mm]	Pier 1 [mm]	Pier 2 [mm]	Abutment 2 [mm]
0	0	0.34304	0	0	0	0
22	10315	0.34304	1.26	21.98	22.78	1.5
47	13256	0.44149	2.67	47.57	48.08	2.82
74	15508	0.51182	4.19	75.94	76.43	4.32
104	17900	0.56458	5.8	106.02	106.5	5.93
134	20390	0.60035	7.47	137.32	137.8	7.6
165	22954	0.6278	9.19	169.52	170.01	9.33
197	25573	0.64985	10.95	202.42	202.91	11.09
230	28235	0.66822	12.74	235.84	236.35	12.88
263	30931	0.68267	14.55	269.7	270.22	14.69
296	33654	0.69429	16.38	303.9	304.43	16.52
330	36401	0.70486	18.22	338.39	338.93	18.37
364	39166	0.71366	20.08	373.11	373.66	20.23
398	41947	0.72107	21.95	408.03	408.59	22.1
432	44741	0.72739	23.82	443.11	443.69	23.98
466	47547	0.73283	25.71	478.34	478.93	25.87



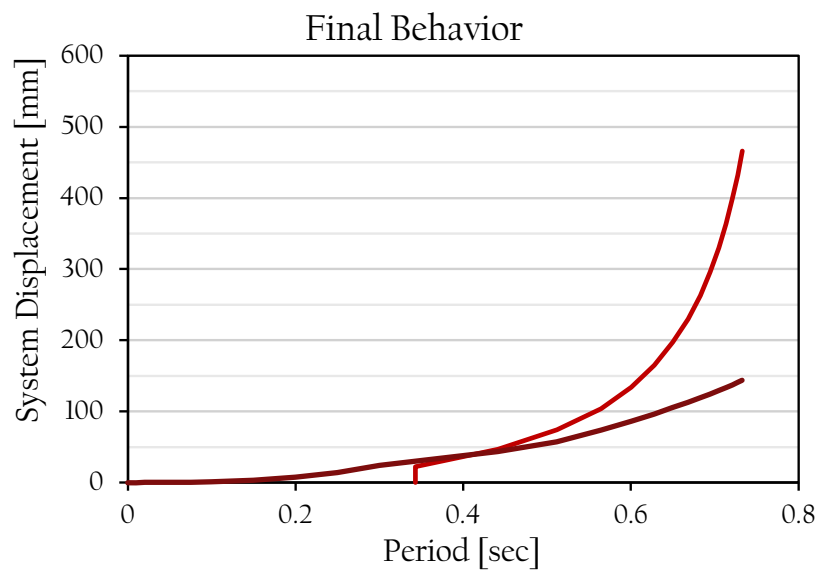
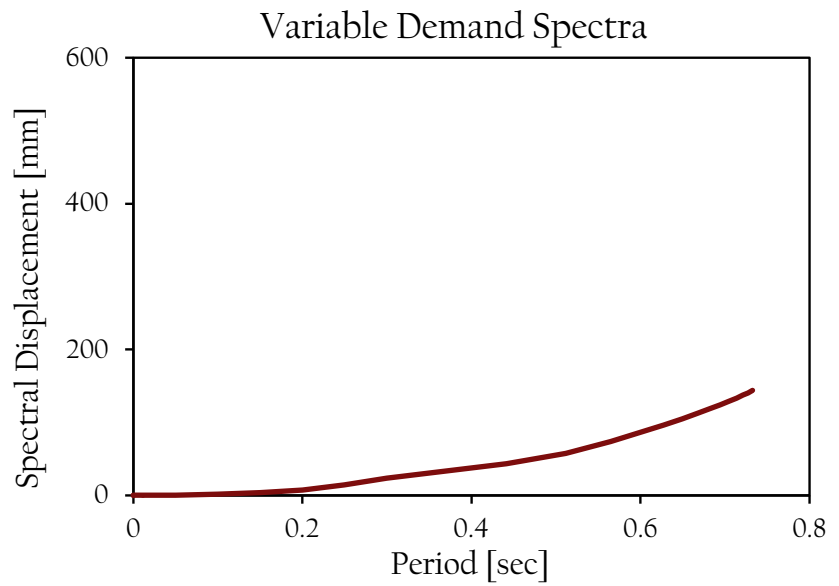
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.010
0.02	0.040
0.03	0.090
0.05	0.266
0.075	0.692
0.1	1.351
0.15	3.381
0.2	7.423
0.25	14.225
0.3	23.877
0.4	45.358
0.5	72.441
0.75	180.279
1	286.704
1.5	535.487
2	793.603
3	949.061
4	1128.964
5	1209.587
7.5	1354.133
10	1259.744



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.01
0.02	0.04
0.03	0.09
0.05	0.266
0.075	0.692
0.1	1.351
0.15	3.381
0.2	7.423
0.25	14.225
0.3	23.877
0.4415	43.53
0.5118	57.18
0.5646	73.994
0.6003	86.337
0.6278	96.512
0.6499	105.187
0.6682	112.766
0.6827	119.074
0.6943	124.413
0.7049	129.357
0.7137	133.64
0.7211	137.386
0.7274	140.691
0.7328	143.626
0.7211	137.386
0.7274	140.691
0.7328	143.626



- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	41
T_{eff} [sec]	0.42
Displacement Profile	2 mm – 41 mm – 42 mm – 3 mm
Column's Ductility	Pier 1: 1.7 Pier 2: 1.6
Highest Limit State Achieved	Pier 1: Equivalent Yield Pier 2: Equivalent Yield

7. Bridge 1208

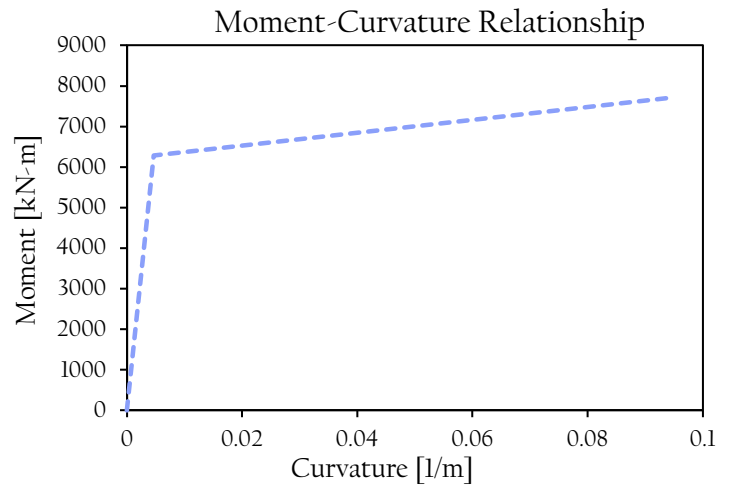
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				fy (MPa)	fyh (MPa)	fu (MPa)	f'c (MPa)	Iy (m4)	Weight SS (kN/m)
1208	61.0861	-146.2097	2	38.05	38.05	0	0	420	420	600	28	144.56	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	6	15.5	7.21	0	0	1.17	50.8	20	43	15.875	76.2	28400	40

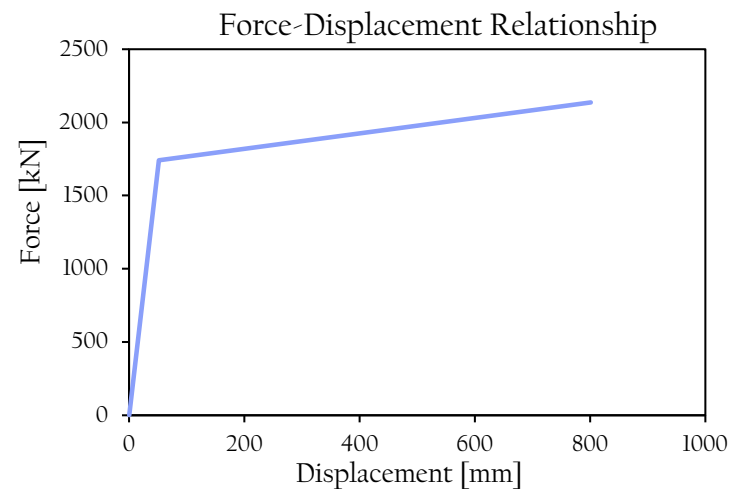
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0046	6280
0.0936	7701



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
51.33	1741.98
801.35	2136.18



- Performance Limit States – Displacements

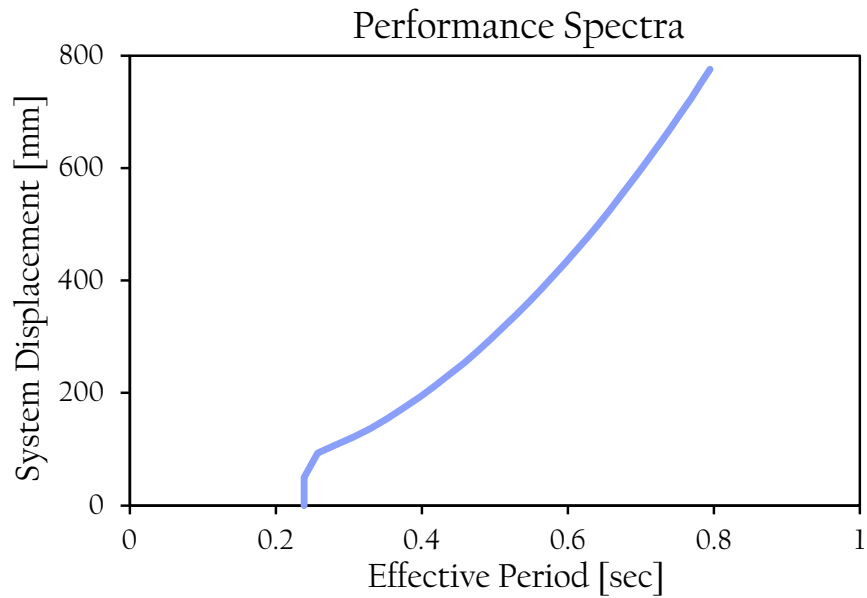
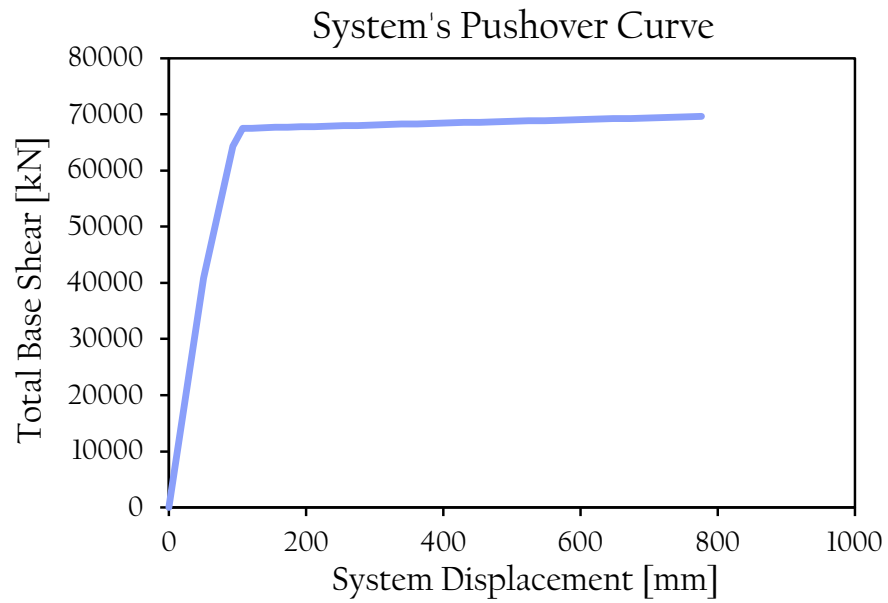
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	37
Equivalent Yield	51
Fully Operational (PL3)	103
Operational (PL2)	428
Life Safety (PL1)	532

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.23471	0	0	0
27	22894	0.23471	11.55	31.89	11.55
46	38488	0.23603	19.74	54.88	19.74
65	49143	0.24716	27.19	78.06	27.19
85	59710	0.25578	34.58	100.96	34.58
100	67465	0.26241	42.27	118.93	42.27
104	67481	0.27335	51.13	124.03	51.13
109	67500	0.28442	60.27	130	60.27
115	67521	0.29568	69.43	136.56	69.43
121	67543	0.30606	78.61	143.59	78.61
129	67566	0.31819	87.81	150.99	87.81
136	67590	0.32844	97.03	158.69	97.03
144	67615	0.33936	106.28	166.63	106.28
152	67641	0.34978	115.55	174.78	115.55
160	67667	0.35979	124.85	183.1	124.85
169	67694	0.37054	134.18	191.57	134.18
177	67721	0.37984	143.55	200.18	143.55
186	67749	0.3899	152.94	208.92	152.94
195	67777	0.39966	162.38	217.75	162.38
204	67805	0.40915	171.85	226.67	171.85
213	67833	0.41839	181.37	235.69	181.37
222	67862	0.42739	190.92	244.78	190.92
231	67891	0.43619	200.51	253.96	200.51
240	67920	0.44478	210.14	263.21	210.14
250	67949	0.4541	219.82	272.54	219.82
259	67979	0.46232	229.54	281.93	229.54
269	68009	0.47126	239.3	291.38	239.3

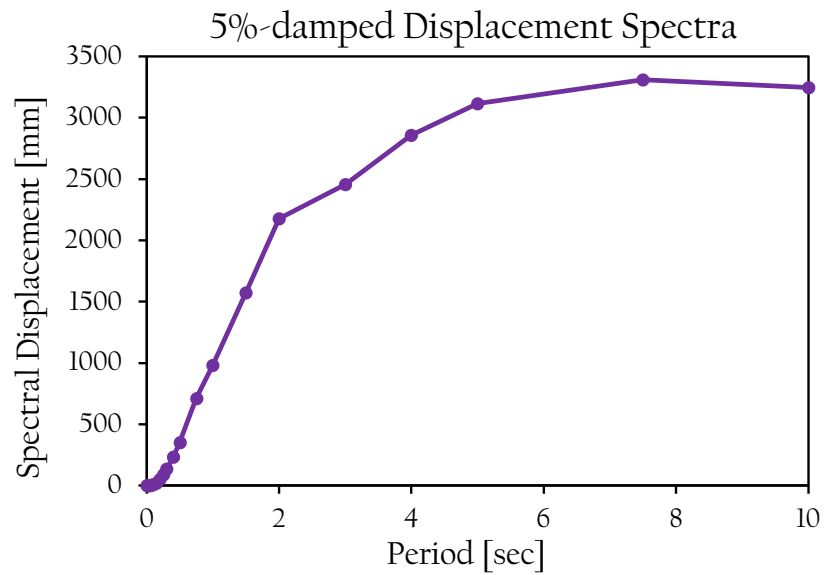
278	68039	0.47915	249.1	300.9	249.1
288	68069	0.48775	258.95	310.48	258.95
297	68100	0.49534	268.84	320.13	268.84
307	68130	0.50364	278.78	329.83	278.78
317	68161	0.51178	288.76	339.59	288.76
327	68192	0.51979	298.79	349.41	298.79
337	68223	0.52766	308.86	359.28	308.86
347	68254	0.53541	318.98	369.21	318.98
357	68286	0.54303	329.14	379.19	329.14
367	68317	0.55054	339.35	389.22	339.35
377	68349	0.55794	349.6	399.32	349.6
387	68381	0.56523	359.9	409.46	359.9
397	68413	0.57242	370.24	419.66	370.24
407	68446	0.57951	380.63	429.9	380.63
418	68478	0.58721	391.07	440.2	391.07
428	68511	0.5941	401.55	450.55	401.55
439	68544	0.60159	412.08	460.95	412.08
449	68577	0.60831	422.66	471.4	422.66
460	68610	0.61561	433.28	481.9	433.28
470	68643	0.62216	443.94	492.45	443.94
481	68676	0.62929	454.66	503.05	454.66
491	68710	0.63568	465.42	513.7	465.42
502	68744	0.64264	476.22	524.4	476.22
513	68778	0.64951	487.08	535.14	487.08
524	68812	0.65631	497.98	545.94	497.98
535	68846	0.66303	508.92	556.79	508.92
546	68880	0.66968	519.91	567.68	519.91
556	68915	0.67564	530.95	578.63	530.95
567	68949	0.68215	542.04	589.62	542.04
579	68984	0.68918	553.17	600.66	553.17
590	69019	0.69554	564.34	611.75	564.34
601	69054	0.70184	575.57	622.89	575.57
612	69090	0.70807	586.84	634.07	586.84
623	69125	0.71425	598.15	645.3	598.15
635	69161	0.72092	609.51	656.58	609.51
646	69196	0.72698	620.92	667.91	620.92
657	69232	0.73297	632.38	679.29	632.38
669	69268	0.73946	643.88	690.71	643.88
680	69304	0.74533	655.42	702.18	655.42

692	69341	0.7517	667.02	713.7	667.02
703	69377	0.75747	678.65	725.26	678.65
715	69414	0.76372	690.34	736.88	690.34
727	69451	0.76991	702.07	748.54	702.07
738	69487	0.77552	713.85	760.24	713.85
750	69524	0.78161	725.67	771.99	725.67
762	69562	0.78763	737.54	783.79	737.54
774	69599	0.79361	749.45	795.64	749.45



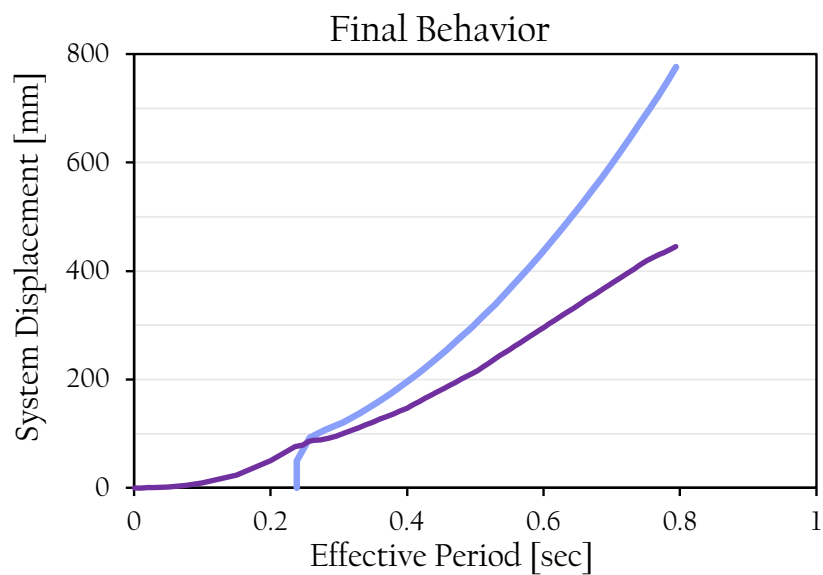
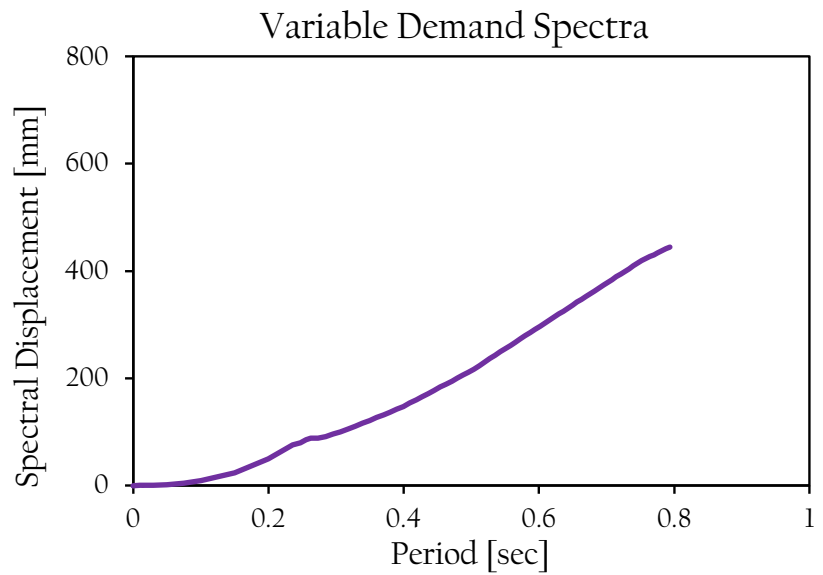
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.057
0.02	0.229
0.03	0.547
0.05	1.770
0.075	4.753
0.1	9.530
0.15	24.020
0.2	49.981
0.25	87.988
0.3	137.148
0.4	234.428
0.5	352.938
0.75	711.406
1	982.702
1.5	1573.159
2	2178.064
3	2455.193
4	2857.555
5	3115.122
7.5	3309.349
10	3245.973



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.01
0.02	0.04
0.03	0.09
0.05	0.266
0.075	0.692
0.1	1.351
0.15	3.381
0.2	7.423
0.25	14.225
0.3	23.877
0.4415	43.53
0.5118	57.18
0.5646	73.994
0.6003	86.337
0.6278	96.512
0.6499	105.187
0.6682	112.766
0.6827	119.074
0.6943	124.413
0.7049	129.357
0.7137	133.64
0.7211	137.386
0.7274	140.691
0.7328	143.626
0.7211	137.386
0.7274	140.691
0.7328	143.626



- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	85
T_{eff} [sec]	0.26
Displacement Profile	35 mm – 102 mm – 35 mm
Column's Ductility	Pier I: 2.0
Highest Limit State Achieved	Pier I: Equivalent Yield

8. Bridge 1350

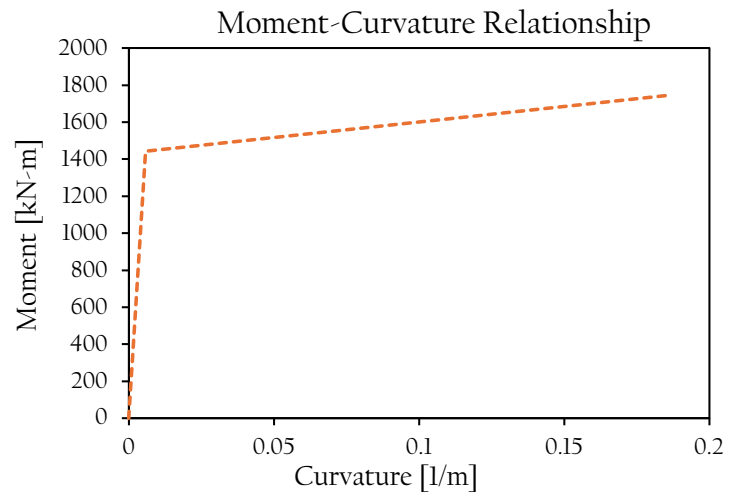
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				f _y (MPa)	f _y _h (MPa)	f _u (MPa)	f _c (MPa)	I _y (m ⁴)	Weight SS (kN/m)
1350	61.1754	-149.8594	2	26.97	26.97	0	0	420	420	600	28	58.27	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	12.9	9.53	0	0	0.88	50.8	12	25.4	25.4	76.2	28400	40

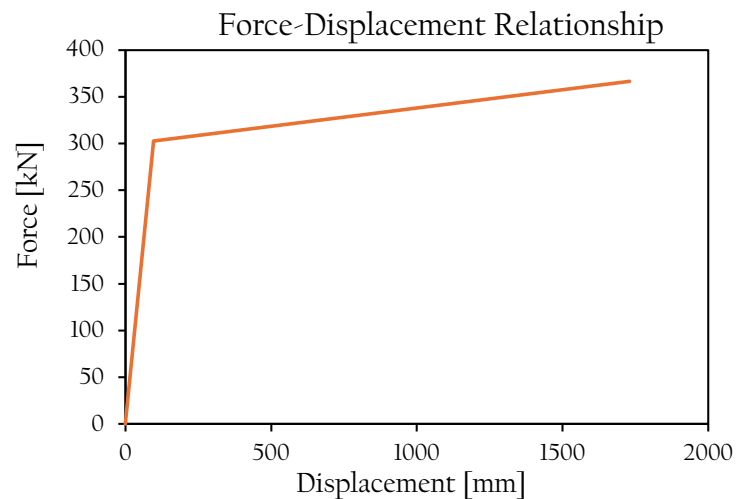
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0057	1442
0.1861	1746



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
96.59	302.71
1730.40	366.34



- Performance Limit States – Displacements

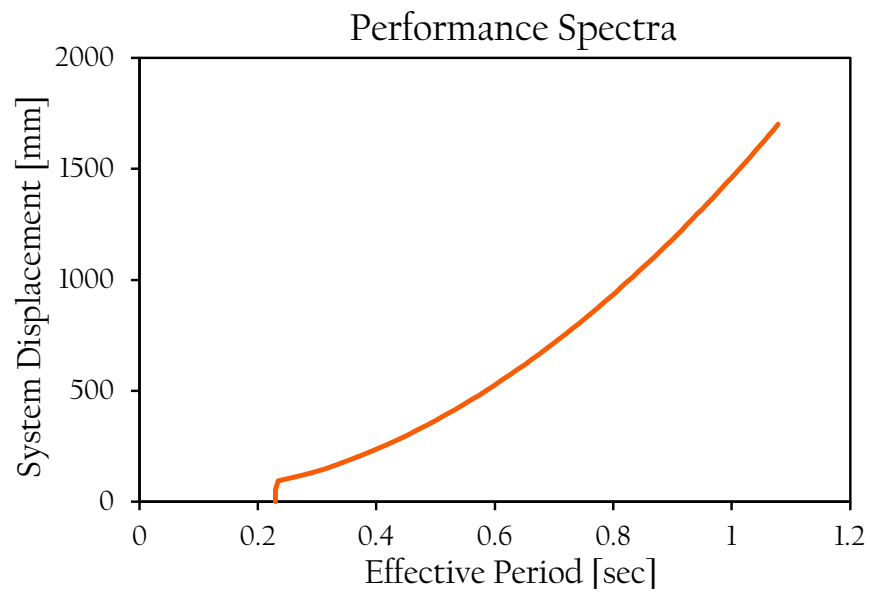
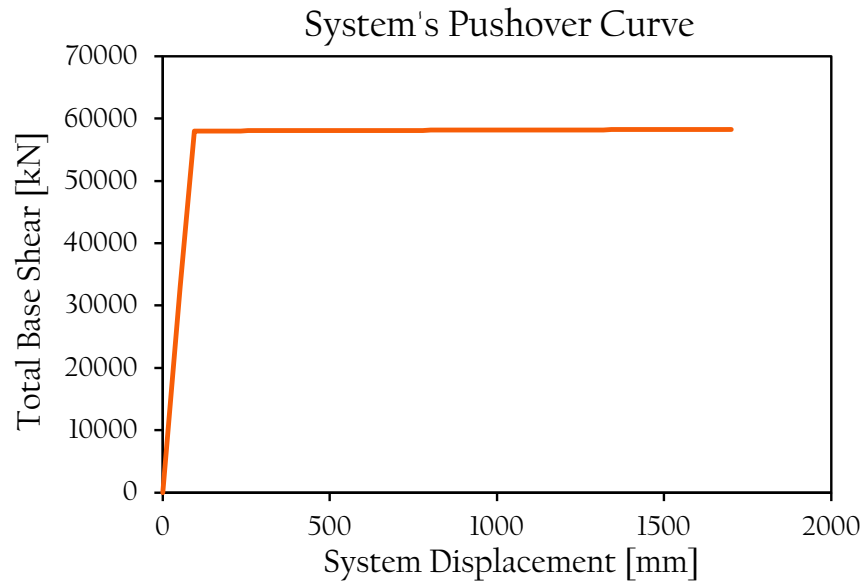
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	73
Equivalent Yield	97
Fully Operational (PL3)	188
Operational (PL2)	1070
Life Safety (PL1)	1321

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.22969	0	0	0
38	23577	0.22969	16.21	45.03	16.21
74	46334	0.22864	31.85	88.49	31.85
97	58014	0.23835	47.61	116.35	47.61
107	58016	0.25706	63.59	128.33	63.59
120	58018	0.27656	79.57	141.52	79.57
133	58020	0.29405	95.55	155.49	95.55
147	58022	0.31117	111.53	169.94	111.53
162	58025	0.32813	127.52	184.74	127.52
177	58027	0.34409	143.52	199.79	143.52
192	58029	0.35922	159.53	215.02	159.53
207	58032	0.37365	175.54	230.4	175.54
223	58034	0.38835	191.57	245.88	191.57
238	58037	0.40163	207.6	261.45	207.6
254	58039	0.41526	223.65	277.1	223.65
269	58041	0.42765	239.7	292.81	239.7
285	58044	0.44043	255.77	308.57	255.77
301	58046	0.45284	271.84	324.37	271.84
317	58049	0.46489	287.93	340.22	287.93
333	58051	0.47664	304.03	356.1	304.03
349	58054	0.48809	320.14	372.01	320.14
364	58056	0.49859	336.25	387.95	336.25
380	58059	0.50953	352.39	403.92	352.39
396	58061	0.52024	368.53	419.92	368.53
412	58064	0.53072	384.68	435.94	384.68
428	58066	0.541	400.85	451.98	400.85
444	58069	0.55108	417.02	468.04	417.02

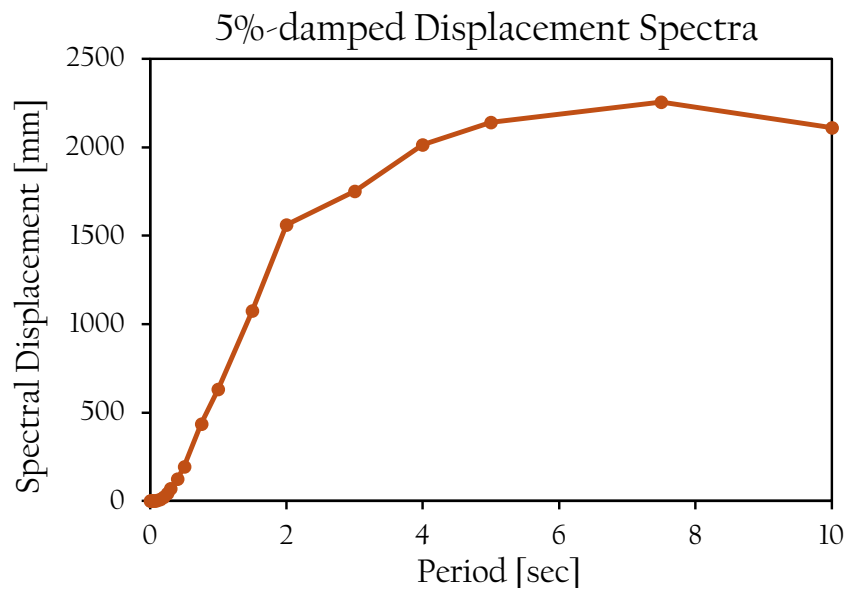
461	58071	0.56159	433.21	484.12	433.21
477	58074	0.5713	449.41	500.22	449.41
493	58076	0.58085	465.62	516.34	465.62
509	58079	0.59024	481.85	532.47	481.85
525	58081	0.59948	498.08	548.63	498.08
541	58084	0.60857	514.33	564.79	514.33
557	58086	0.61754	530.58	580.98	530.58
574	58089	0.62691	546.85	597.18	546.85
590	58091	0.63561	563.13	613.4	563.13
606	58094	0.64419	579.43	629.63	579.43
622	58096	0.65266	595.73	645.87	595.73
639	58099	0.66153	612.05	662.13	612.05
655	58101	0.66977	628.37	678.41	628.37
671	58104	0.67791	644.71	694.7	644.71
687	58107	0.68595	661.07	711	661.07
704	58109	0.6944	677.43	727.32	677.43
720	58112	0.70224	693.8	743.65	693.8
736	58114	0.71001	710.19	760	710.19
753	58117	0.71816	726.59	776.35	726.59
769	58119	0.72576	743	792.73	743
786	58122	0.73374	759.42	809.11	759.42
802	58124	0.74117	775.85	825.51	775.85
818	58127	0.74852	792.3	841.92	792.3
835	58130	0.75625	808.76	858.34	808.76
851	58132	0.76346	825.22	874.78	825.22
868	58135	0.77105	841.7	891.23	841.7
884	58137	0.77812	858.2	907.7	858.2
901	58140	0.78556	874.7	924.17	874.7
917	58142	0.7925	891.22	940.66	891.22
934	58145	0.7998	907.75	957.16	907.75
950	58147	0.80662	924.28	973.68	924.28
967	58150	0.81379	940.84	990.2	940.84
983	58153	0.82048	957.4	1006.74	957.4
1000	58155	0.82754	973.97	1023.29	973.97
1016	58158	0.83412	990.56	1039.86	990.56
1033	58160	0.84107	1007.16	1056.43	1007.16
1049	58163	0.84754	1023.77	1073.02	1023.77
1066	58166	0.85437	1040.39	1089.62	1040.39
1083	58168	0.86114	1057.03	1106.24	1057.03

1099	58171	0.86747	1073.67	1122.86	1073.67
1116	58173	0.87414	1090.33	1139.5	1090.33
1133	58176	0.88076	1107	1156.15	1107
1149	58179	0.88694	1123.68	1172.82	1123.68
1166	58181	0.89347	1140.37	1189.49	1140.37
1183	58184	0.89994	1157.08	1206.18	1157.08
1199	58186	0.90599	1173.79	1222.88	1173.79
1216	58189	0.91238	1190.52	1239.59	1190.52
1233	58192	0.91871	1207.26	1256.31	1207.26
1249	58194	0.92464	1224.01	1273.05	1224.01
1266	58197	0.9309	1240.78	1289.8	1240.78
1283	58199	0.93711	1257.55	1306.56	1257.55
1300	58202	0.94328	1274.34	1323.33	1274.34
1317	58205	0.94941	1291.14	1340.12	1291.14
1333	58207	0.95515	1307.95	1356.91	1307.95
1350	58210	0.9612	1324.77	1373.72	1324.77
1367	58212	0.96722	1341.6	1390.54	1341.6
1384	58215	0.97319	1358.45	1407.37	1358.45
1401	58218	0.97913	1375.31	1424.22	1375.31
1418	58220	0.98504	1392.18	1441.08	1392.18
1434	58223	0.99056	1409.06	1457.95	1409.06
1451	58226	0.99639	1425.95	1474.83	1425.95
1468	58228	1.0022	1442.86	1491.72	1442.86
1485	58231	1.00796	1459.77	1508.62	1459.77
1502	58233	1.0137	1476.7	1525.54	1476.7
1519	58236	1.0194	1493.64	1542.47	1493.64
1536	58239	1.02506	1510.6	1559.41	1510.6
1553	58241	1.0307	1527.56	1576.36	1527.56
1570	58244	1.0363	1544.53	1593.33	1544.53
1587	58247	1.04188	1561.52	1610.31	1561.52
1604	58249	1.04743	1578.52	1627.29	1578.52
1621	58252	1.05294	1595.53	1644.3	1595.53
1638	58255	1.05842	1612.55	1661.31	1612.55
1655	58257	1.06388	1629.59	1678.33	1629.59
1672	58260	1.06931	1646.64	1695.37	1646.64
1689	58263	1.0747	1663.69	1712.42	1663.69
1706	58265	1.08008	1680.76	1729.48	1680.76



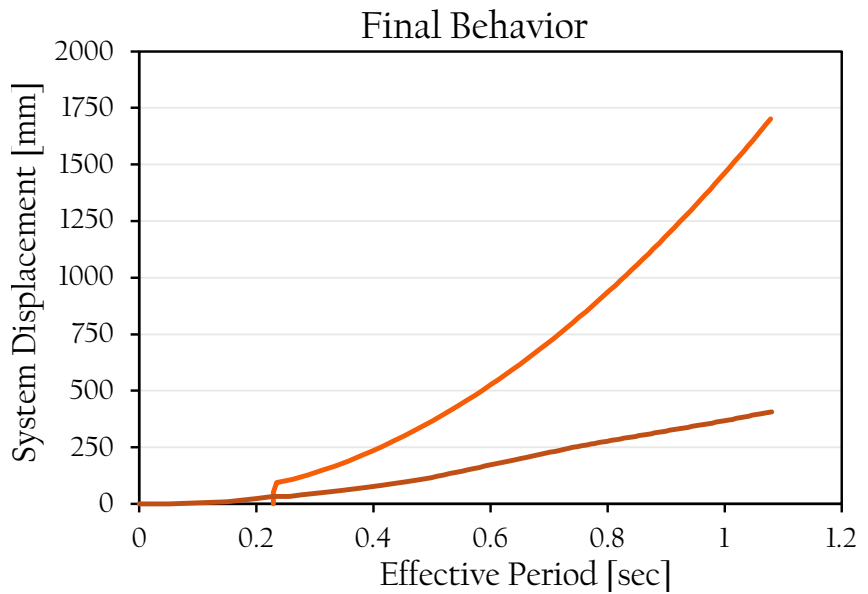
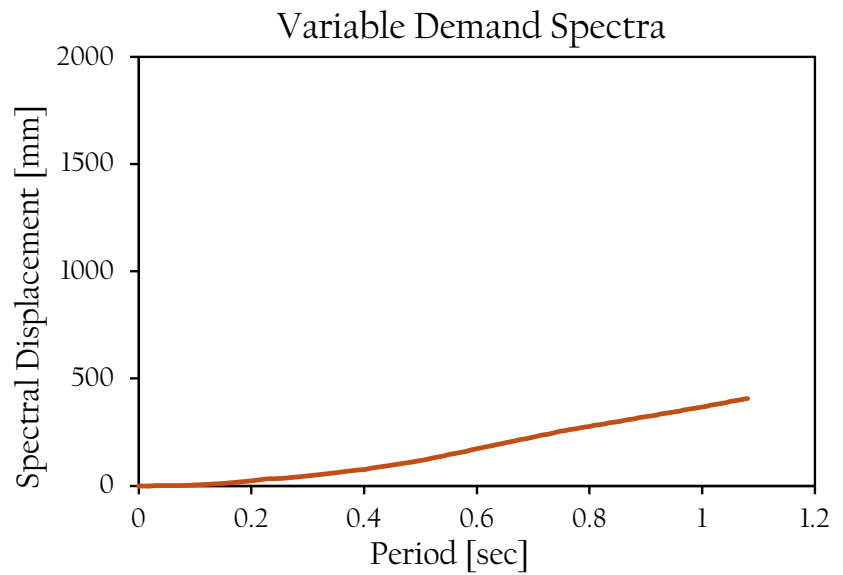
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.028
0.02	0.113
0.03	0.260
0.05	0.798
0.075	2.092
0.1	4.171
0.15	10.658
0.2	22.902
0.25	41.675
0.3	68.940
0.4	123.582
0.5	195.158
0.75	435.074
1	632.375
1.5	1073.710
2	1560.230
3	1752.251
4	2014.926
5	2141.783
7.5	2255.685
10	2109.433



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.028
0.02	0.113
0.03	0.26
0.05	0.798
0.075	2.092
0.1	4.171
0.15	10.658
0.2	22.902
0.2286	33.656
0.2384	32.425
0.2571	34.484
0.2766	39.758
0.2941	44.662
0.3112	49.648
0.3281	54.702
0.3441	59.478
0.3592	64.024
0.3736	68.374
0.3883	72.852
0.4016	77.044
0.4153	82.628
0.4276	87.668
0.4404	92.89
0.4528	97.955
0.4649	102.874
0.4766	107.664
0.4881	112.331
0.4986	116.594
0.5095	122.446
0.5202	128.369
0.5307	134.163
0.541	139.84
0.5511	145.402
0.5616	151.209



0.5713	156.56
0.5808	161.821
0.5902	166.99
0.5995	172.078
0.6086	177.084
0.6175	182.016
0.6269	187.181
0.6356	191.967
0.6442	196.683
0.6527	201.338
0.6615	206.222
0.6698	210.754
0.6779	215.226
0.686	219.644
0.6944	224.292
0.7022	228.604
0.71	232.871
0.7182	237.357
0.7258	241.529
0.7337	245.916
0.7412	249.999
0.7485	254.037
0.7563	257.666
0.7635	260.907
0.771	264.32
0.7781	267.5
0.7856	270.85
0.7925	273.971
0.7998	277.261
0.8066	280.327
0.8138	283.559
0.8205	286.57
0.8275	289.751
0.8341	292.712
0.8411	295.841
0.8475	298.755
0.8544	301.832
0.8611	304.887
0.8675	307.733

0.8741	310.743
0.8808	313.726
0.8869	316.509
0.8935	319.452
0.8999	322.371
0.906	325.099
0.9124	327.977
0.9187	330.835
0.9246	333.507
0.9309	336.327
0.9371	339.132
0.9433	341.915
0.9494	344.68
0.9551	347.265
0.9612	349.996
0.9672	352.712
0.9732	355.408
0.9791	358.088
0.985	360.754
0.9906	363.243
0.9964	365.875
1.0022	368.615
1.008	371.53
1.0137	374.432
1.0194	377.313
1.0251	380.178
1.0307	383.031
1.0363	385.865
1.0419	388.682
1.0474	391.489
1.0529	394.277
1.0584	397.049
1.0639	399.812
1.0693	402.556
1.0747	405.286
1.0801	408.006

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	34
T_{eff} [sec]	0.23
Displacement Profile	13 mm – 36 mm – 13 mm
Column's Ductility	Pier I: < 1.0
Highest Limit State Achieved	Pier I: Elastic Behavior

9. Bridge 2081

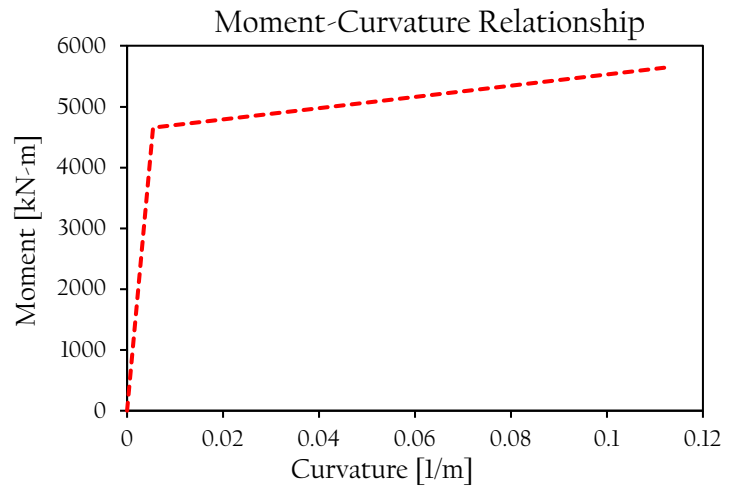
- Database Info

Bridge #	Location		Spans					Materials				Superstructure	
ID	Latitude (°)	Longitude (°)	# of Spans	Span Length (m)				f _y (MPa)	f _y _h (MPa)	f _u (MPa)	f _c (MPa)	I _y (m4)	Weight S (kN/m)
2081	61.1233	-149.8864	2	33.62	33.62	0	0	420	420	600	28	42.39	180

Substructure											Abutments		
RC or RCFST	# Columns/ pier	Cap Beam Length (m)	Column Height (m)			Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)	Yield Strength (kN)	Yield Displacement (mm)
RC	4	11.9	7.68	0	0	1.02	50.8	30	32.26	15.875	76.2	28400	40

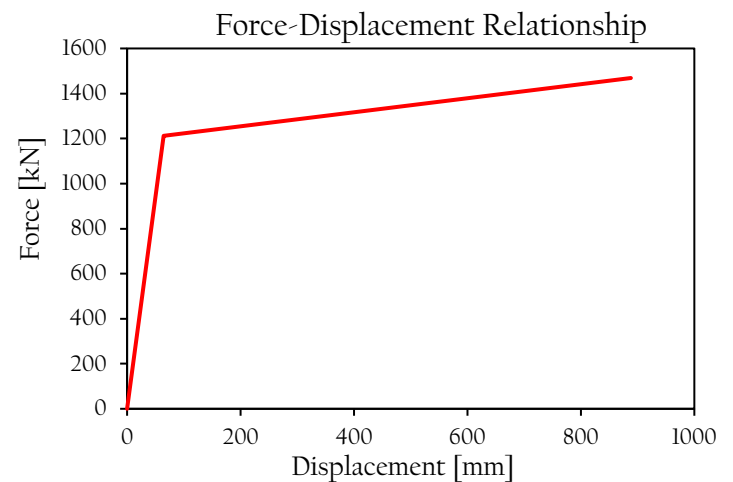
- Moment-Curvature Relationship

Curvature [1/m]	Moment [kN-m]
0	0
0.0055	4655
0.1120	5642



- Force-Displacement Relationship

Displacement [mm]	Force [kN]
0	0
64.04	1212.28
887.91	1469.23



- Performance Limit States – Displacements

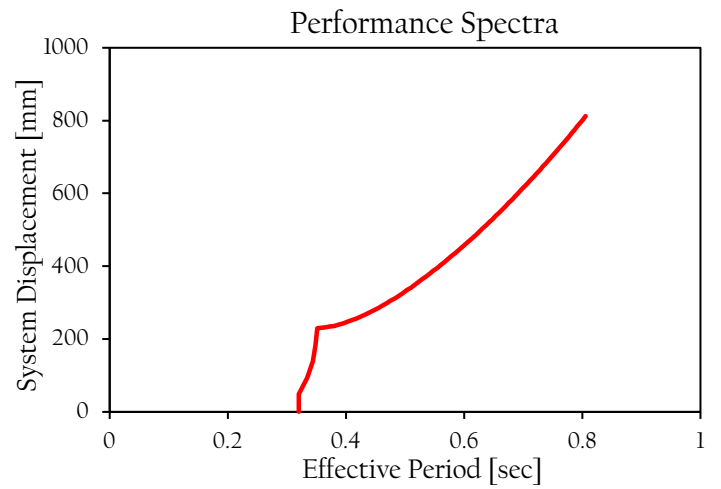
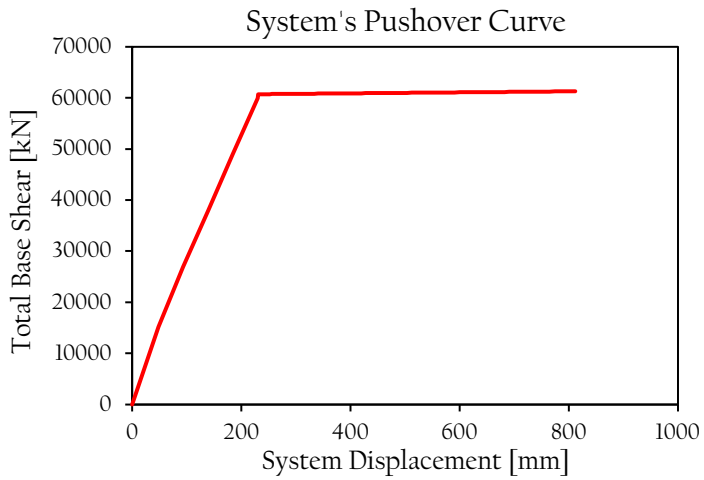
Performance Limit States	Columns Pier 1 Displacement [mm]
First Yield	47
Equivalent Yield	64
Fully Operational (PL3)	104
Operational (PL2)	449
Life Safety (PL1)	607

- System's Pushover Curve & Performance Spectra

System Displacement [mm]	Total Base Shear [kN]	Effective Period [sec]	Displacement Progression		
			Abutment 1 [mm]	Pier 1 [mm]	Abutment 2 [mm]
0	0	0.3202	0	0	0
48	15194	0.3202	8.5	54.02	8.5
93	26882	0.33454	16.29	105.05	16.29
139	37916	0.3441	24.02	156.09	24.02
184	48938	0.34833	31.75	207.09	31.75
230	59956	0.35176	39.47	258.07	39.47
231	60708	0.35709	47.74	263.71	47.74
232	60711	0.3646	56.09	266.5	56.09
234	60714	0.37238	64.47	269.73	64.47
236	60717	0.37967	72.88	273.39	72.88
239	60721	0.38729	81.31	277.43	81.31
243	60726	0.39527	89.76	281.84	89.76
247	60730	0.40284	98.23	286.56	98.23
252	60735	0.41083	106.71	291.61	106.71
256	60740	0.41764	115.22	296.95	115.22
262	60746	0.42574	123.74	302.54	123.74
267	60751	0.43272	132.28	308.38	132.28
273	60757	0.44022	140.83	314.43	140.83
279	60763	0.44745	149.4	320.68	149.4
285	60770	0.45444	157.99	327.11	157.99
292	60776	0.46199	166.59	333.71	166.59
298	60782	0.46854	175.2	340.47	175.2
305	60789	0.47568	183.83	347.36	183.83
312	60796	0.48264	192.47	354.38	192.47
319	60803	0.48942	201.13	361.53	201.13
326	60810	0.49604	209.81	368.78	209.81
334	60817	0.50327	218.49	376.14	218.49

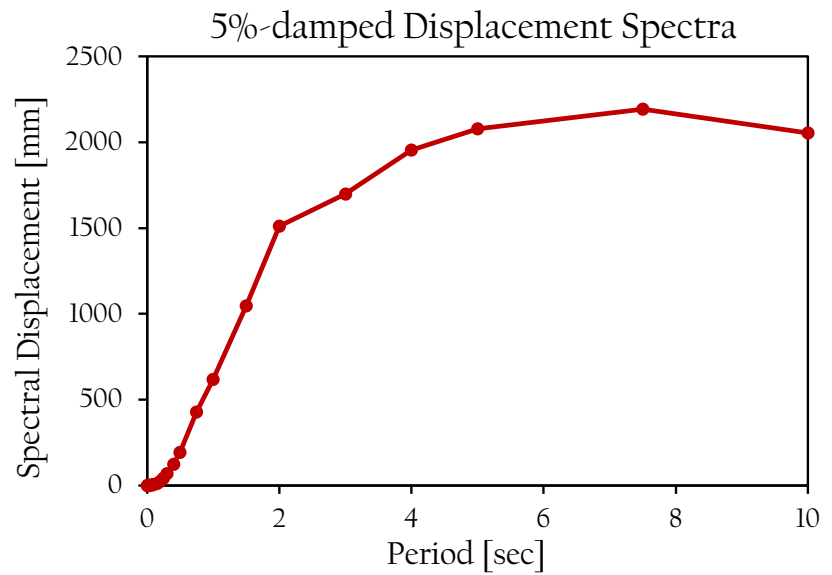
341	60824	0.5096	227.2	383.59	227.2
349	60832	0.51654	235.92	391.13	235.92
356	60839	0.52262	244.65	398.76	244.65
364	60847	0.52931	253.4	406.48	253.4
372	60854	0.53588	262.16	414.25	262.16
380	60862	0.54235	270.94	422.09	270.94
387	60869	0.548	279.74	430	279.74
395	60877	0.55426	288.55	437.96	288.55
403	60885	0.56043	297.38	445.99	297.38
412	60893	0.5672	306.22	454.07	306.22
420	60901	0.57319	315.08	462.2	315.08
428	60908	0.5791	323.95	470.38	323.95
436	60916	0.58493	332.84	478.61	332.84
444	60924	0.59069	341.75	486.89	341.75
453	60933	0.59703	350.67	495.21	350.67
461	60941	0.60264	359.61	503.57	359.61
470	60949	0.60883	368.57	511.97	368.57
478	60957	0.61431	377.54	520.41	377.54
487	60965	0.62037	386.53	528.89	386.53
495	60973	0.62572	395.53	537.41	395.53
504	60982	0.63164	404.55	545.96	404.55
512	60990	0.63688	413.59	554.54	413.59
521	60998	0.64269	422.65	563.16	422.65
530	61007	0.64843	431.72	571.81	431.72
538	61015	0.65351	440.8	580.49	440.8
547	61024	0.65915	449.91	589.2	449.91
556	61032	0.66473	459.03	597.94	459.03
565	61041	0.67025	468.17	606.72	468.17
574	61049	0.67573	477.32	615.52	477.32
583	61058	0.68116	486.49	624.35	486.49
591	61066	0.68596	495.68	633.2	495.68
600	61075	0.69129	504.88	642.09	504.88
609	61084	0.69658	514.1	651	514.1
618	61092	0.70182	523.34	659.93	523.34
627	61101	0.70702	532.6	668.9	532.6
636	61110	0.71218	541.87	677.89	541.87
645	61118	0.7173	551.16	686.9	551.16
654	61127	0.72237	560.46	695.94	560.46
664	61136	0.72796	569.78	705	569.78

673	61145	0.73295	579.12	714.08	579.12
682	61154	0.7379	588.48	723.19	588.48
691	61163	0.74282	597.85	732.33	597.85
700	61171	0.74771	607.24	741.48	607.24
710	61180	0.75309	616.65	750.66	616.65
719	61189	0.7579	626.08	759.87	626.08
728	61198	0.76267	635.52	769.09	635.52
737	61207	0.76742	644.98	778.34	644.98
747	61216	0.77265	654.45	787.61	654.45
756	61225	0.77732	663.94	796.9	663.94
765	61234	0.78197	673.45	806.21	673.45
775	61243	0.78709	682.98	815.54	682.98
784	61252	0.79168	692.52	824.9	692.52
794	61261	0.79673	702.09	834.28	702.09
803	61271	0.80125	711.66	843.68	711.66
812	61280	0.80574	721.26	853.09	721.26



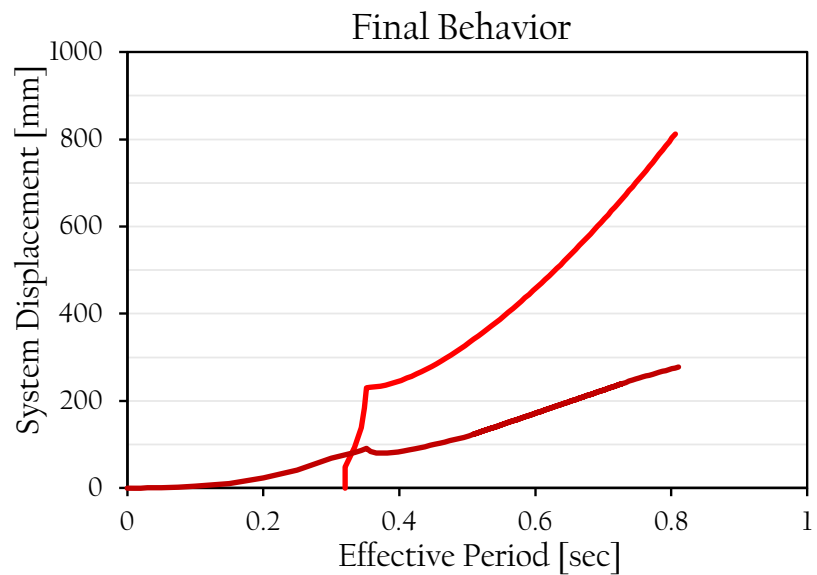
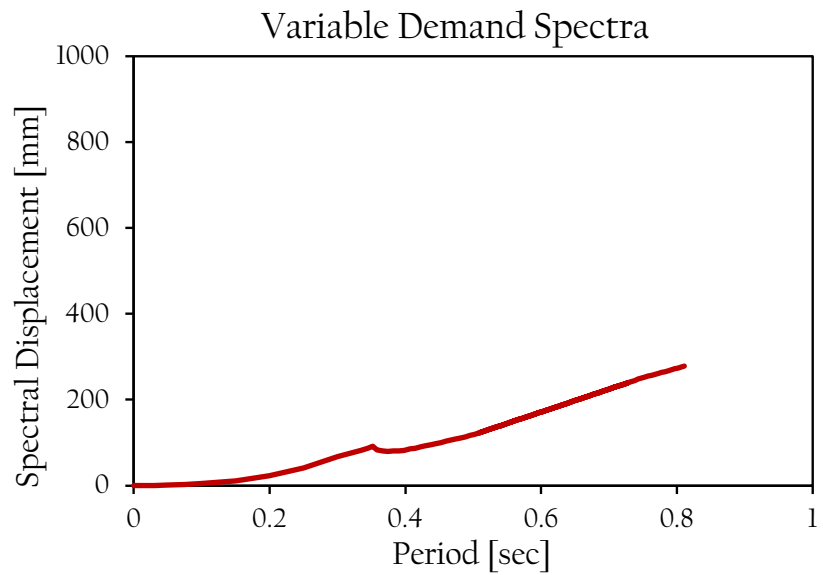
- 5%-damped Displacement Spectra

Period [sec]	Spectral Displacement [mm]
0	0.000
0.01	0.028
0.02	0.113
0.03	0.260
0.05	0.801
0.075	2.103
0.1	4.182
0.15	10.634
0.2	22.713
0.25	41.284
0.3	67.997
0.4	121.775
0.5	191.822
0.75	426.667
1	619.103
1.5	1047.003
2	1512.193
3	1698.718
4	1953.440
5	2078.181
7.5	2192.404
10	2054.018



- Variable Demand Spectra & Final Behavior

Period [sec]	Spectral Displacement [mm]
0	0
0.01	0.028
0.02	0.113
0.03	0.26
0.05	0.801
0.075	2.103
0.1	4.182
0.15	10.634
0.2	22.713
0.25	41.284
0.3	67.997
0.3345	81.766
0.3441	86.188
0.3483	88.732
0.3518	90.961
0.3579	83.328
0.3662	80.65
0.374	79.806
0.3821	80.13
0.3905	81.183
0.3985	82.452
0.4069	84.876
0.4141	87.06
0.4225	89.943
0.4298	92.388
0.4376	95.129
0.445	97.792
0.4522	100.382
0.46	103.247
0.4667	105.696
0.474	108.423
0.4811	111.083
0.488	113.682
0.4948	116.223
0.5021	119.344



0.5085	122.721
0.5155	126.452
0.5217	129.685
0.5285	133.275
0.5351	136.795
0.5416	140.257
0.5473	143.255
0.5536	146.608
0.5598	149.906
0.5667	153.545
0.5727	156.743
0.5786	159.896
0.5845	163.01
0.5903	166.081
0.5966	169.485
0.6023	172.475
0.6085	175.796
0.614	178.717
0.6201	181.962
0.6254	184.816
0.6314	187.992
0.6366	190.781
0.6425	193.892
0.6482	196.969
0.6533	199.673
0.659	202.692
0.6645	205.678
0.6701	208.638
0.6756	211.568
0.681	214.474
0.6858	217.028
0.6912	219.885
0.6965	222.715
0.7017	225.52
0.7069	228.305
0.7121	231.064
0.7172	233.8
0.7223	236.519
0.7279	239.517

0.7329	242.188
0.7378	244.839
0.7428	247.471
0.7476	250.083
0.753	252.67
0.7578	254.763
0.7626	256.842
0.7674	258.906
0.7726	261.191
0.7773	263.227
0.7819	265.25
0.7871	267.49
0.7917	269.487
0.7967	271.698
0.8012	273.668
0.8057	275.624
0.8107	277.794

- Final Limit State Achieved

Parameters	Final Values
Δ_{sys} [mm]	81
T_{eff} [sec]	0.33
Displacement Profile	14 mm – 91 mm – 14 mm
Column's Ductility	Pier 1: 1.4
Highest Limit State Achieved	Pier 1: Equivalent Yield

APPENDIX B

USER GUIDE – DDBA MATLAB CODE

DISCLAIMER

The code provided is distributed *as is*, without any express or implied warranties regarding its correctness, accuracy, or suitability for any particular purpose. No guarantee is made that the code is error-free or adheres to any specific standard. Use of this code is entirely at your own risk. It must not be used in any application where errors could result in loss of property, personal injury, or other significant consequences. The authors disclaim all liability, direct or indirect, arising from the use or misuse of this code.

INPUT FILE - DATABASE

The user must provide all the inputs of geometry and material in the Microsoft Excel file named *DATABASE-INPUT.xlsx*.

• Bridge Identification and Location

Bridge #	Location	
ID	Latitude (°)	Longitude (°)
646	62.30142	-145.307288

ID: AKDOT&PF Bridge Number

Latitude and Longitude: bridge location (°).

• Spans Description

Spans				
# of Spans	Span Length (m)			
3	36.68	34.14	36.68	0

of Spans: Total number of spans.

Span Length: Length of each bridge span (m). (If less than 4 spans, insert “0” in additional cells).

• Material Properties and Superstructure Properties

Materials					Superstructure	
f_y (MPa)	f_{yh} (MPa)	f_u (MPa)	E_c (kN/m ²)	f_c (MPa)	I_y (m ⁴)	Weight SS (kN/m)
420	420	600	33000000	28	48.02	180

f_y : Yield strength of longitudinal reinforcing steel (for columns) (MPa).

f_{yh} : yield strength of transverse reinforcing steel (for columns) (MPa).

f_u : Ultimate strength of reinforcing steel (for columns) (MPa).

E_c : Concrete’s modulus of elasticity (for columns and superstructure) (kN/m²).

f_c : Compressive concrete strength (for columns) (MPa).

I_y : Superstructure flexural inertia – strong axis transverse section (m⁴).

Weight SS: Distributed dead load on bridge (kN/m).

- **Substructure**

Substructure					
RC or RCFST	# Columns/pier	Cap Beam Length (m)	Column Height (m)		
RC	5	11.9	6.25	4.83	0

Substructure					
Section Diameter (m)	Cover to Long Bars (mm)	Quantity of Long. Bars	Long. Bars Diameter (mm)	Trans. Reinf. Diameter (mm)	Spacing of Trans. Steel (mm)
0.88	50.8	16	32.26	15.875	76.2

RC of RCFST: Choice of substructure type.

Columns/pier: Number of columns per pier.

Cap Beam Length: Distance between center of extreme columns in the pier (m). Refer to Figure 62 for visual guidance.

Column Height: Height of columns in each pier (m). (*If less than 3 piers, insert “0” in additional cells*). For pile-supported footing, the column height is measured from the underside of the cap beam to the top of the foundation. For RCFST columns, the column height is the distance from the underside of the cap beam to ground level, then add twice the column diameter. Refer to Figure 62 for visual guidance on both cases.

Section Diameter: Diameter of columns (m).

Cover to Long. Bars: Cover to outside of longitudinal bars (mm).

Quantity of Long. Bars: Quantity of longitudinal reinforcing bars.

Long. Bars Diameter: Diameter of longitudinal reinforcing bars (mm).

Trans. Reinf. Diameter: Diameter of transverse reinforcing bars (mm).

Spacing of Trans. Steel: Center to center distance of transverse reinforcing bars (mm). See Figure 62 for clarification.

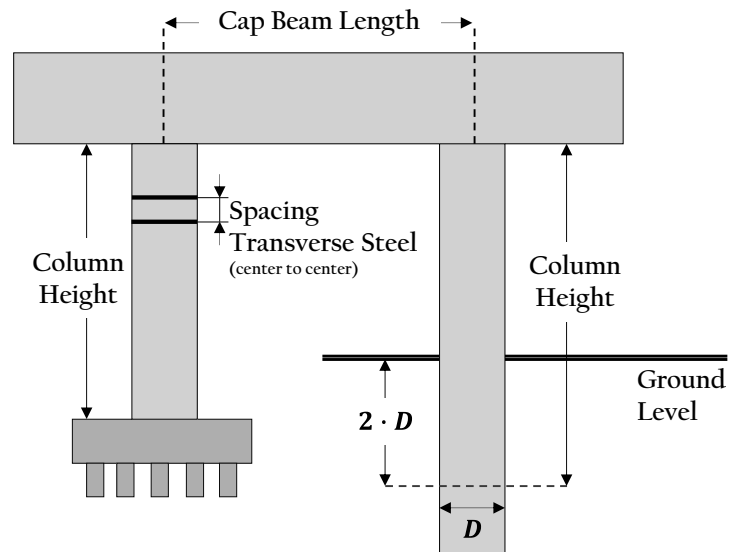


Figure 62. Elevation view of generic pier with the representation of parameters used in the MATLAB code

• Abutments Details

Abutments	
Yield Strength (kN)	Yield Displacement (mm)
28400	40

Refer to Figure 42 for the following abutment definitions:

Yield Strength: Yield strength of abutments (kN).

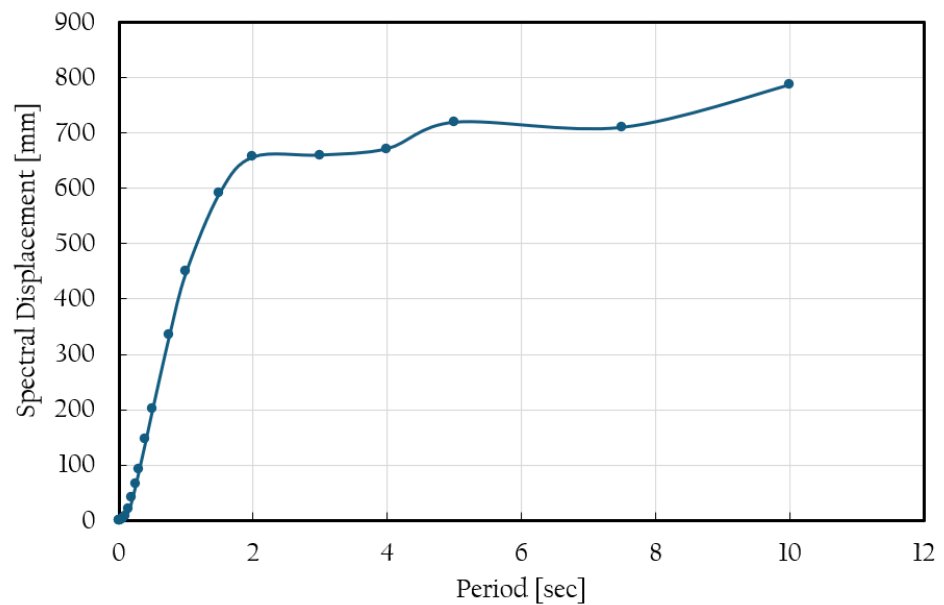
Yield Displacement: Yield displacement of abutments (mm).

INPUT FILE – SEISMIC DEMANDS

The code provides two options for defining seismic demands:

- 1. Manual Input of Displacement Spectra:** the user can manually input 5%-damped displacement spectra data (i.e., period and spectral displacement). This information must be entered in the Microsoft Excel file named *EVENT-INPUT.xlsx*. There is no limit to the size of the vectors.

Period [s]	Sd [mm]
0.000	0.000
0.010	0.048
0.020	0.212
0.030	0.532
0.050	1.592
0.075	3.691
0.100	7.202
0.150	20.346
0.200	41.149
0.250	66.155
0.300	92.582
0.400	146.483
0.500	202.024
0.750	334.949
1.000	449.097
1.500	591.512
2.000	656.180
3.000	659.515
4.000	671.152
5.000	718.791
7.500	709.490
10.000	786.174



- 2. Automated Calculation via Ground Motion Model:** alternatively, the user can specify the moment magnitude and site-to-source distance directly in the code. The seismic demands will be automatically calculated using the GMM developed by Faccioli et al. (2004).

It is important to note that the objective of Phase II is to integrate all the Ground Motion Models mentioned in the report in the MATLAB Code.

ANALYSIS INPUT IN MATLAB

The code allows the user to define some analysis parameters. The command “SEISMIC” permits the user to choose the method of definition of seismic demand for the analysis, either manually inputting the 5% damped displacement spectra through the input file (SEISMIC=1) or obtaining it through the Faccioli et al. (2004) GMM (SEISMIC=2). If the latter is chosen, moment magnitude (M_w) and site to source distance (r) need to be defined. The command “dsf” permits the user to choose the spectral reduction factor based on Eurocode 8 (1994) (dsf=1), Kong & Kowalsky (2016) (dsf=2), AASHTO Guide Spec (2011) (dsf=3), or Eurocode 8 (2004) (dsf=4). The spectral reduction factors mentioned are explained in Chapter 3.

```
%-----START OF USER's INPUT-----

%%% I. INPUT and OUTPUT DATA %%%

outputfile = '646.xlsx'; %name of excel output file
inputfile='database-input.xlsx'; %name of excel input file with bridge database

%%% II. SEISMIC DEMAND %%%
%5% Damped Displacement Spectra
SEISMIC =1; %1= Manually input
%2= GMM from Faccioli et al (2004)

if SEISMIC ==2
    Mw=7; %Moment Magnitude
    r=10; %site-to-source distance, in km
end

if SEISMIC ==1
    inputfile_s='event-input.xlsx'; %name of excel input file with ground motion data (5% damped)
end

dsf=1; %1 for Eurocode 8 (1994)
%2 for Kong and Kowalsky (2016)
%3 for AASHTO(2011)
%4 for Eurocode 8 (2004)

%-----END OF USER's INPUT-----
```

OUTPUT FROM MATLAB

Upon completion of the analysis, the DDBA Code provides a summary of results both in MATLAB *Command Window* and through a summary figure. It is important to note that if the seismic demand exceeds the structural capacity – resulting in column failure – the only output generated is a message in the Command Window stating: “*The seismic demand pushed the bridge beyond its allowable displacement capacity.*”.

Command Window Output

It provides a summary of initial information, such as column heights and span lengths, and number of columns per pier. Additionally, it also provides information regarding the seismic event: if it was manually input, it will display “Manually Input”, if the Faccioli et al. (2004) GMM was used, the moment magnitude and site-to-source distance are displayed. Following, the final behavior of the bridge is shown, such as final effective period, final system displacement, final and yield pier displacements, and displacement ductility demands. *It is important to note that the first and last value for the displacements and ductility correspond to the abutments.* Additionally, the performance limit states that the columns of each pier achieved are displayed. As shown in the following figure.

```

-----SUMMARY & FINAL VALUES-----
-----Initial Information:
Column Heights [m] = 6.25
Column Heights [m] = 4.83
Column Heights [m] = 0.00
Span Lengths [m]= 36.68
Span Lengths [m]= 34.14
Span Lengths [m]= 36.68
Span Lengths [m]= 0.00
Number of Columns/pier= 5
Seismic Event - Manually Input
-----Final Behavior:
Effective Period [s] = 0.68
Final System Displacement [mm] = 240.00
Final Pier Displacements [mm] = [14 248 245 13]
Yield Pier Displacements [mm] = [40 50 32 40]
Displacement Ductility = [0.4 5 7.7 0.3]

column1_ls =

      'Columns Pier 1 Fully Operational (PL3)'

column2_ls =

      'Columns Pier 2 Fully Operational (PL3)'

```

Figure 63. Command Window output

Visual Summary of DDBA Steps

The output consists of eight graphs:

1. **Force-Displacement Response per Pier Column:** Displays both the full and bilinear force-displacement curves for each pier column, along with the corresponding performance limit states: First Yield, Fully Operational (PL3), Operational (PL2), Life Safety (PL1), and Near Collapse.
2. **System Pushover Curve:** Shows the pushover curve for the bridge system, with all performance limit states clearly identified.
3. **Performance Spectra:** Presents the performance spectra with the same performance limit states indicated.
4. **Bridge Elevation Profile:** Provides a schematic elevation view of the bridge.

5. **5% Damped Displacement Spectra:** Displays the 5% damped displacement response spectra.
6. **Variable Demand Spectra:** Shows the variable demand spectra in red, with the 5% damped displacement spectra overlaid as a green dashed line. *(Note: This graph uses a different period range compared to the previous one, and the axes are scaled differently.)*
7. **Final Bridge Performance:** Combines the performance spectra and the variable demand spectra, with their intersection representing the final performance point of the bridge.
8. **Final Displacements:** illustrates the yield and final displacements of each pier and abutment. Please note that this figure is schematic and does not represent the actual displaced shape of the structure; it is intended solely to visualize the relative displacement magnitudes achieved by each component. As shown in the following figure.

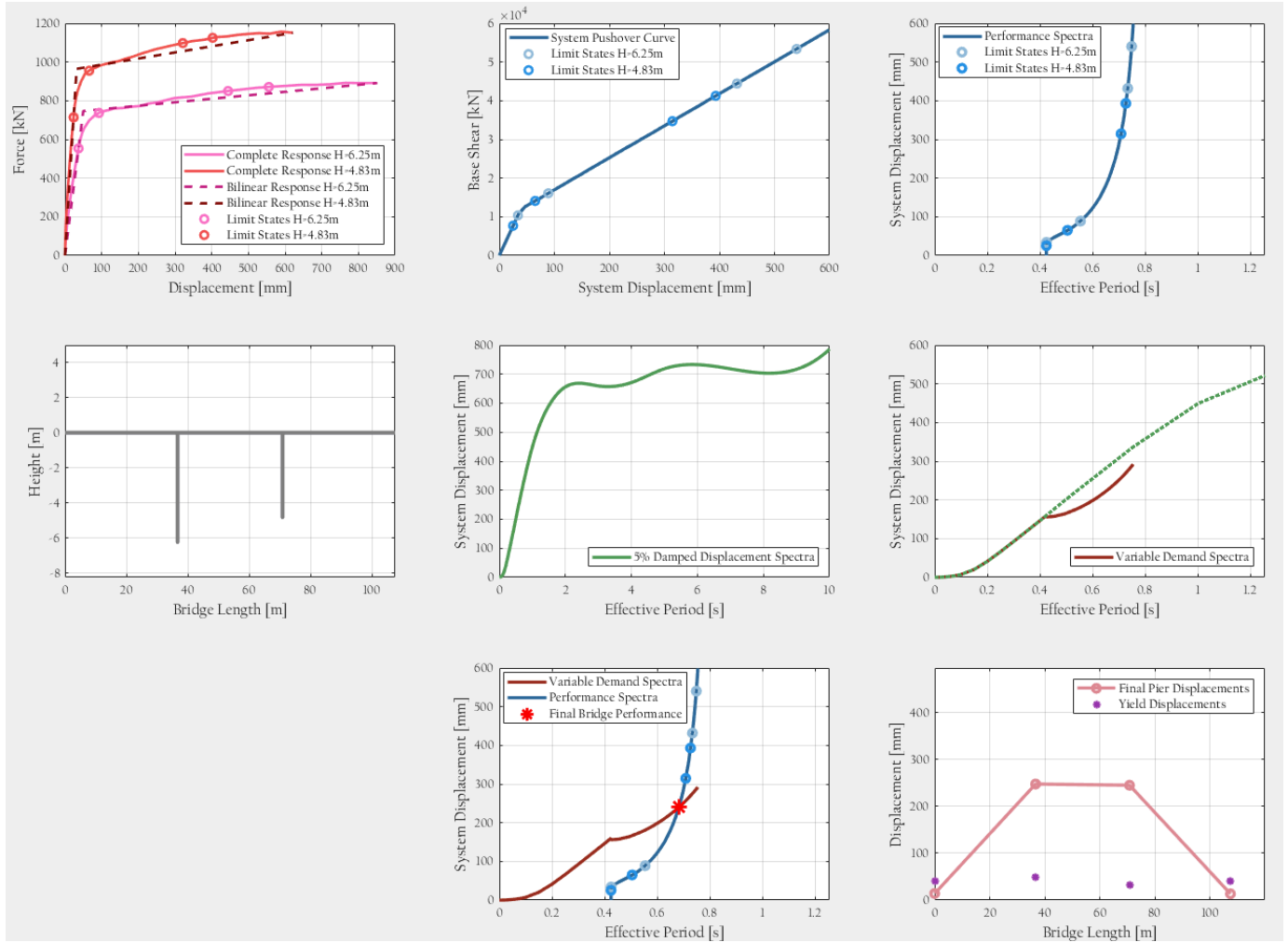


Figure 64. Visual summary of DDBA steps provided by the DDBA MATLAB code.

EXPORTED OUTPUT FILE

The code generates a Microsoft Excel output file containing six sheets that summarize the DDBA process for the structure. The first sheet, titled “**Summary**”, provides an overview of the highest performance limit states reached by the pier columns—ranging from first yield and equivalent yield to fully operational (PL3), operational (PL2), and life safety (PL1). It also includes the final displacements of both abutments and piers, as well as the displacement ductility and yield displacements. Additionally, it presents the displacements of each pier column corresponding to each performance limit state. As shown in the following figure.

Columns Pier 1 Fully Operational (PL3)			
Columns Pier 2 Fully Operational (PL3)			
Pier Displacements [mm]			
abutment 1	pier 1	pier 2	abutment 2
14	248	245	13
Displacement Ductility			
0.4	5	7.7	0.3
Yield Displacements [mm]			
40	50	32	40
Performance Limit States	Columns Pier 1 [mm]	Columns Pier 2 [mm]	
First Yield	37	24	
Equivalent Yield	50	32	
Fully Operational	93	66	
Operational	445	321	
Life Safety	556	403	

Summary – Example Bridge 646

The second sheet, titled “**5% Spectra**”, provides the values of period and spectral displacement corresponding to the 5%-damped displacement spectra either manually inputted, or generated by Faccioli et al. (2004) GMM.

5%-Damped Displacement Spectra	
T [sec]	Sd [mm]
0	0
0.01	0.04819962
0.02	0.212165602
0.03	0.531983095
0.05	1.592188721
0.075	3.691464882
0.1	7.202106644
0.15	20.3457424
0.2	41.14927638
0.25	66.15511419
0.3	92.58162492
0.4	146.4825891
0.5	202.0243164
0.75	334.9489214
1	449.0970198
1.5	591.5122624
2	656.1797662
3	659.5149557
4	671.1521494
5	718.7908885
7.5	709.4898099
10	786.1737731

5% Spectra – Example Bridge 646

The third sheet, titled “**Displacement Progression**”, presents the displacement values for each pier and abutment at every point along the pushover curve. The fourth sheet, titled “**Pushover Curve**”, presents the system displacement and total base shear values.

abutment 1 [mm]	pier 1 [mm]	pier 2 [mm]	abutment 2 [mm]
0	0	0	0
1.93	25.79	22.42	0.94
3.22	49.97	47.47	2.49
4.72	77.73	75.37	4.05
6.36	107.76	105.42	5.7
8.08	138.95	136.6	7.42
9.84	170.98	168.6	9.18
11.64	203.64	201.23	10.97
13.47	236.79	234.34	12.79
15.32	270.32	267.83	14.63
17.19	304.16	301.62	16.48
19.07	338.25	335.66	18.35
20.96	372.55	369.9	20.23
22.86	407.01	404.32	22.12
24.77	441.62	438.88	24.02
26.69	476.36	473.56	25.92
28.61	511.2	508.34	27.83
30.54	546.13	543.22	29.74
32.48	581.14	578.18	31.66
34.41	616.22	613.21	33.58

Displacement Progression

System Displacement [mm]	Total Base Shear [kN]
0	0
24	7377
47	12636
75	14875
104	17295
134	19808
165	22389
197	25021
229	27692
262	30394
295	33121
328	35868
362	38631
395	41409
429	44198
463	46996
497	49804
531	52619
565	55440
599	58267

Pushover Curve

The fifth sheet, titled “**Performance Spectra**”, presents the effective period and system displacement values. The sixth sheet, titled “**VDS**”, presents the Variable Demand Spectra information: effective period and spectral displacement values.

Effective Period [sec]	System Displacement [mm]
0.42401	0
0.42401	24
0.45416	47
0.52856	75
0.5771	104
0.61202	134
0.63873	165
0.66015	197
0.67652	229
0.69069	262
0.70205	295
0.71135	328
0.72008	362
0.7265	395
0.73283	429
0.7383	463
0.74304	497
0.74721	531
0.75089	565
0.75415	599

Performance Spectra

Effective Period [sec]	Spectral Displacement [mm]
0	0
0	0
0.01	0.048
0.02	0.212
0.03	0.532
0.05	1.592
0.075	3.691
0.1	7.202
0.15	20.346
0.2	41.149
0.25	66.155
0.3	92.582
0.4	146.483
0.4542	157.668
0.5286	173.286
0.5771	189.323
0.612	203.751
0.6387	216.433
0.6601	227.64
0.6765	237.024
0.6907	245.563
0.7021	252.885
0.7114	259.236
0.7201	265.239
0.7265	270.116
0.7328	274.839
0.7383	279.064
0.743	282.859
0.7472	286.291
0.7509	289.353
0.7542	291.985

VDS