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Implications for Expected Bridge Performance Considering Bearing Rotation and Ground
Motion Duration

By

Maria Camila Lopez Ruiz

A dissertation submitted in partial satisfaction of the
requirements for the degree of
Doctor of Philosophy
in
Engineering – Civil and Environmental Engineering
in the
Graduate Division
of the
University of California, Berkeley

Committee in charge:

Professor Tracy C. Becker, Chair

Professor Khalid M. Mosalam

Professor James Demmel

Summer 2024

Implications for Expected Bridge Performance Considering Bearing Rotation and Ground
Motion Duration

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Maria Camila Lopez Ruiz

Abstract

Implications for Expected Bridge Performance Considering Bearing Rotation and Ground Motion Duration

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Doctor of Philosophy in Engineering - Civil and Environmental Engineering

University of California, Berkeley

Professor Tracy C. Becker, Chair

The optimal performance of critical bridge infrastructure plays a key role in upholding the functionality and resilience of transportation networks following significant seismic events. As such reliable methods for evaluating and quantifying bridge performance are essential. This work aims to quantify the effect on structural performance of two analysis considerations not included in conventional evaluation methods. The two cases studied are (1) the effect of ground motion duration correlation to ground motion intensity and (2) the effect of accounting for isolation bearing end-plate rotations.

The first step in a performance-based evaluation is obtaining site-specific ground motions for input into nonlinear structural models. Site-specific ground motions are generally selected based on magnitude and distance for the structure of interest. While others have studied the effect of duration, the correlation with ground motion models has not been included. Researchers have found that duration and spectral acceleration are negatively correlated, which means that the target duration should decrease when the intensity of the ground motion scenario increases. As such, the current state of practice may be overestimating the damage probability for structures in high seismic zones. This work compares the effect of the duration correlation for a major bridge and a typical highway overpass.

The following step in the analysis is to develop a numerical model that can accurately capture the structural response. The case of interest in the second part of this work is the behavior of isolation bearings subject to rotational demands, as is the case for bearings placed on top of flexible bridge piers, a condition observed in the major bridge used as a case study. This is relevant because a typical design assumes no rotation of bearing end-plates and past research has shown that bearing end-plate rotation leads to a reduction in the horizontal stiffness of isolation bearings. To capture the changes in bearing behavior due to rotation, the study utilizes a hybrid simulation experiment to inform and improve numerical modeling.

Fragility curves are developed for a moderate and a major damage state based on pier drift and bearing shear strain for both studies, the effect on bridge performance is evaluated in each case

by comparing the median probability of damage to that of baseline fragilities developed using traditional methods. It is shown that considering ground motion duration correlation to intensity leads to a reduction in the predicted median damage for the major bridge and the typical bridge, and the effect is dependent not only on the correlation parameters but also on the amount of degradation included in the numerical model. A low correlation coefficient (small changes in predicted duration) may lead to large changes in performance if the model includes additional degrading elements like the lead rubber bearings in the major bridge. By contrast, it is revealed that significant changes in the bearing shear and rotational stiffness, when considering bearing rotation, resulted in minimal impact on performance when measured by damage fragility functions.

Keywords: Earthquake Engineering, Fragility Assessment, Ground Motion Duration, Bearing Rotation, Bridges.

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1. Introduction

1.1 Overview and Motivation

Critical bridge infrastructure is an integral component of transportation networks that facilitate access to emergency services and communication following earthquakes. Its continued functionality is crucial for effective disaster response and recovery. As such, reliable and robust methods for evaluating bridge performance are imperative. This work examines the impacts of specific hazard selection and structural modeling assumptions on bridge performance criteria.

Performance-based earthquake engineering (PBEE) has gained popularity as it connects structural demand parameters to performance goals such as life-safety, economic impacts, and functionality. (Moehle and Deierlein, 2004). Through this connection, PBEE enables the development of decision-making frameworks that quantify the likelihood of exceeding performance criteria at different seismic intensity levels. The methodology encompasses site-specific probabilistic hazard analysis, structural analysis, damage analysis, and loss analysis. This study leverages the PBEE methodology up to the damage analysis stage to evaluate the performance of a major bridge and a typical highway overpass. Each stage of the framework influences the resulting prediction of the performance of the structure. Thus, the accuracy of each individual section is essential for the accuracy of the structural performance predictions. As such, there is constantly emerging research in the many areas encompassed by the methodology. This study focuses on the effects on structural bridge performance of two cases: (1) accounting for ground motion duration in the hazard analysis, and (2) accounting for isolation bearing rotation in the structural model.

The outcome of the analysis in this work is damage fragility functions. The damage fragility functions link ground motion intensity to the probability of structural damage states. Several researchers have contributed to the creation of standard procedures for estimating the parameters of fragility functions (i.e. Porter et al. (2007), Baker (2015), Baker et al. (2021)). The early development of fragility functions in earthquake engineering was used for the seismic assessments of nuclear power plants (Kennedy et al., 1980, Kennedy and Ravindra, 1984). Fragility functions can be used to determine the effects of different analysis considerations on damage prediction by examining the shift of the fragility curve compared to a baseline case.

One key analysis consideration examined is the effect of ground motion duration. While it is well understood that long-duration ground motions can impose many cycles on a structure, leading to cumulative damage when nonlinear behavior is considered (Raghunandan and Liel, 2013), existing procedures for site-specific hazard analysis frequently neglect the effects of ground motion duration, and current codes do not include requirements to account for the effects. However, previous work on the effect of ground motion duration typically only compared the response to short and long-duration sets, with no comment on what was likely to occur. In collaborative work, Slate Geotechnical Consultants developed a ground motion duration correlation to spectral acceleration, showing that duration and spectral acceleration are negatively

correlated (Lopez Ruiz et al., 2024 - in submission). This means that the target ground motion duration should be smaller when selecting records at high intensities. Ignoring this correlation could lead to an overestimation of damage probabilities at high-intensity measures, especially for fragility curves which are often derived from investigating the response to increasing larger ground motions. This study addresses this gap by examining how the effect of ground motion duration correlation in the hazard analysis affects the predicted performance of two case study bridges modeled in 3D, a major bridge (partially isolated with lead rubber bearings) with a natural period of 1.6 seconds and a typical highway overpass with a natural period of 0.5 seconds. The performance is measured by developing damage fragility curves based on two ground motion selection methodologies, one with traditional selection methods and one accounting for the duration correlation.

Another analysis consideration is the detailed modeling of complex bridge components such as the rotational behavior of bearings. While the behavior of isolation bearings is well understood in typical design applications with parallel end-plate conditions, the effects of rotational demands introduced by flexible supports, like flexible bridge piers, have not been thoroughly investigated. Previous studies have shown that rotation of the bearing end-plates leads to a reduction in the horizontal stiffness of isolation bearings (Crowder and Becker, 2017, Ravari et al., 2012). This change in the bearing's mechanical behavior could alter the structural response of bridges with bearings on top of flexible piers, as is the case for the major bridge in this study.

While testing has been done to explore bearing performance with rotation under quasistatic cyclical loading, no tests have explored the interaction of the structural performance with the rotation of the bearing. As such, hybrid simulation with model updating was selected as a testing method. Hybrid simulation is a testing methodology that combines numerically modeled and physically tested subassemblies of the structure (Nakashima et al., 1992). Model updating was used to enhance hybrid simulation by using the physical experimental data to update the numerical substructure elements of similar characteristics, improving the overall accuracy and reducing the extent of the experimental setup (Kwon and Kammula, 2013). The hybrid simulation test includes one lead rubber bearing tested in 3 degrees-of-freedom: axial, shear, and rotation. At each time step, the experimental response is leveraged to update other numerically modeled bearings. To accommodate the experimental setup, the study uses a 2D model of the major bridge.

To evaluate the effect of bearing rotation on the overall bridge performance, a fully numerical model is used to develop fragility curves. Based on experimental data collected during a hybrid simulation, a numerical model is proposed that captures bearing coupling behavior in shear and rotation. This model captures the rotational behavior of the bearings in a fully numerical model of the major bridge. Fragility curves are developed for two damage states and presented to measure the effect on bridge performance of considering bearing rotational behavior in the structural model.

This study found that accounting for ground motion duration correlation to intensity results in predicting a significantly lower median damage probability for both the major bridge and the typical highway overpass when compared to conventional hazard selection methods. This effect is influenced not only by the correlation parameters but also by the degradation included in each

bridge model. Conversely, considerable changes in the bearing behavior due to rotational demands had a limited effect on the estimated damage probability.

In summary, this research focuses on evaluating the effect of ground motion duration and bearing rotation on bridge performance as measured by damage fragility curves. The effect of ground motion duration is studied for two cases (1) a major bridge and (2) a typical highway overpass. The performance effects of bearing rotation are evaluated for the major bridge. The ground motion duration and the bearing rotation studies are compared to show how each case has a different influence on structural performance. Together, these investigations provide a holistic view of the various parameters influencing bridge performance in seismic events, offering valuable contributions to the field of earthquake engineering.

1.2 Objectives

The goal of this study is to quantify the impact of different analysis assumptions on bridge damage performance predictions, focusing on the correlation of ground motion duration to the intensity and rotational behavior of isolation bearings. To accomplish this, this study has the following objectives:

1. Examine if considering duration correlation changes the predicted performance of two bridges when compared to a traditional incremental dynamic analysis.
2. Investigate the behavior of lead rubber bearings when subject to flexible conditions in a bridge system.
3. Evaluate changes in the system-level performance of the bridge resulting from bearing rotational behavior.
4. Compare the effect of ground motion duration and bearing rotation on the performance of a major bridge.

1.3 Organization

The dissertation comprises two overarching topics, ground motion duration correlation and bearing rotation, and their effect on bridge performance is organized as follows:

Chapter 1 outlines the objectives, scope, and research significance of the project. It also provides a general overview of the topics investigated in this study and the factors driving the need for such research.

Chapter 2 delves into the effects of ground motion duration on bridge performance for a major bridge and a typical highway bridge. A detailed description for each case study includes a ground motion selection, structural analysis using OpenSees, selection of damage analysis criteria, and development of fragility curves to determine performance. The procedure includes a performance prediction based on current hazard selection practices as a baseline and a performance prediction based on a novel hazard selection methodology that accounts for ground motion

duration correlation to intensity. Finally, the influence of ground motion duration on performance is compared for the two bridges.

Chapter 3 investigates the impact of bearing rotation on the structural performance of a major bridge. A hybrid simulation experiment was conducted to determine the bearing behavior due to axial, shear, and rotational demands. The component-level findings from the experimental program are used to propose a bearing model and determine system-level changes in bridge performance as measured by damage fragility functions. The analysis compared the performance of a baseline case, which used traditional modeling assumptions for the bearing, and the performance obtained by using the proposed model, which captures bearing rotational behavior for the major bridge

Chapter 4 compares the performance implications of ground motion duration and bearing rotation for the major bridge. Then it summarizes and highlights the key findings presented in this study and provides recommendations for future development in this line of research.

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2. Bridge Performance Considering Ground Motion Duration

This chapter is based on the PEER report (under review) by Lopez Ruiz, M.C, Largent, M., Becker, T.C., Watson-Lamprey, J. (2024). Portions of the report completed by other authors have been reduced.

2.1 Abstract

Construction or detailed evaluation of transportation infrastructure requires site-specific ground motions for input into nonlinear structural models. These site-specific ground motions are generally selected based on the magnitude, distance, and site condition of the recorded ground motion. A preliminary study performed by Slate Geotechnical Consultants found that duration and spectral acceleration are negatively correlated for large-magnitude earthquakes recorded on soft-soil sites ($V_{s30} < 360$ m/s). This means that if the target spectral acceleration is above average, then the average duration associated with that ground motion should be below average for the scenario magnitude and distance. This ground motion duration correlation implies that the current state of the practice is overestimating the probability of collapse for structures. This project aims to improve the characterization of ground motion duration and evaluate how the improved characterization affects bridge fragilities used in the Performance-Based Earthquake Engineering (PBEE) framework. Numerical models for two prototype structures, a major bridge and a typical highway overpass, are developed using OpenSees (McKenna et al., 2010). Using these models, the effect of ground motion duration is evaluated quantitatively following the PEER PBEE methodology.

2.2 Introduction

The PEER performance-based earthquake engineering (PBEE) methodology has increasingly been used to evaluate the seismic risk to structural systems and link the use of structural demand parameters, typically used by designers, to desirable performance goals such as life-safety, economic losses and post-earthquake downtime (Moehle and Deierlein, 2004). Agencies and researchers have used PBEE to develop decision-making plans based on probabilistic analysis techniques such as fragility curves, which describe probabilities of exceeding design or performance criteria at different seismic intensity levels. The PBEE methodology begins by using the location and structural design characteristics (D) to do a site-specific probabilistic hazard analysis to find the probability of an intensity measure (IM). The resulting hazard information, sometimes in the form of ground motion time series, is used for structural analysis to determine the probability of an engineering demand parameter (EDP), like drifts or stresses, given the intensity measures. Then a damage analysis is conducted to relate the EDPs to a set of damage measures (DM). Finally, a loss analysis relates the DMs to a decision variable (DV) such as repair costs or downtime. Each stage of the framework influences the resulting prediction of the

performance of a structure, e.g., a bridge, and, thus, the accuracy of the hazard analysis is a basis for the accuracy of the structural performance predictions. This study uses the PBEE methodology up to the damage analysis stage as shown in Figure 2.1 to evaluate the effects of accounting for ground motion duration in the hazard analysis on the structural performance of two cases study bridges: (1) a major bridge, and (2) a typical highway overpass.

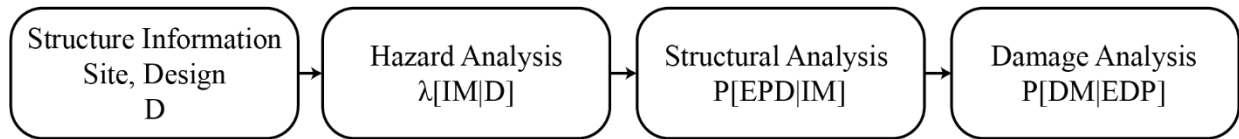


Figure 2.1. PBEE Methodology schematic up to damage analysis.

Current procedures for typical site-specific hazard analysis do not account for the effects of ground motion duration, and there are no requirements in design codes. Long-duration ground motions, however, can subject a structure to a large number of cycles and can result in accumulated damage when nonlinear behavior is present. Effects of ground motion duration on structural performance have been of interest to researchers for some time. While some early papers have found mixed results regarding the significance of ground motion duration on the structural response (Shome et al., 1998, Tremblay, 2003, Iervolino et al., 2006, Hancock and Bommer, 2007), recent literature has demonstrated that duration has significant effects on structural response when there is consideration for duration measure, material degradation, and damage measure. Raghunandan and Liel (2013) found that ground motion duration has a measurable effect on the collapse capacity of reinforced concrete buildings, with longer duration leading to higher collapse probability when accounting for degradation in the numerical model. Barbosa et al. (2017) found that long-duration ground motions in steel frames tend to induce larger damage than short-duration ground motions at high values of spectral acceleration based on a combined damage index that considers deformation and hysteretic energy dissipated. Chandramohan et al. (2016) investigated the effect of duration on the collapse capacity of a concrete bridge pier. They used two sets of records, both spectrally matched to the same response spectrum but with different average durations. They found that the collapse capacity was reduced by 17% for the concrete pier when long-duration ground motions were used. Hassan and Billah (2020) found that long-duration ground motions lead to higher damage of isolated bridges because the residual displacement increases.

Hancock and Bommer (2006) provide a thorough review of studies involving ground motion duration effects on structural damage. They found that the effect of duration is dependent on the damage metric used such as peak response, cumulative damage, or energy dissipated. The adequacy of a damage metric is dependent on the failure mode that is expected from the structure. While cumulative damage measures are recommended to capture all failure modes, peak response is simpler to implement and is often adequate if the failure mode expected is plastic hinge formation. Using peak responses results in higher expected damage when long duration ground motions are considered.

In cases where duration is deemed to be significant for the structural response, the state of the practice is to check the duration of the selected records against existing duration prediction models. Current duration prediction equations (DPE) describe the mean and variability of a duration metric (for a certain amount of energy released) given a set of predictor variables such as moment magnitude (M_w) and distance (R_{rup} , R_{jb} , R_x , etc.). These methods, such as Abrahamson and Silva (1996), Bommer et al. (2009), and Afshari and Stewart (2016), consider these predictor variables from the earthquake but do not consider the correlation between spectral acceleration and duration. Section 2.3 of this study presents the development of a ground motion duration correlation to spectral acceleration model that shows that duration and spectral acceleration are negatively correlated. This means that if the target spectral acceleration is above average (i.e. the ground motion is above the median deterministic scenario) then the average duration associated with that ground motion should be below average for the scenario magnitude and distance. Thus, the current state of the practice may be overestimating the probability of collapse for structures by introducing long-duration ground motions at high intensity measures. This is particularly true when methods such as incremental dynamic analysis, in which ground motions are scaled to larger and larger intensities, are used to generate damage or collapse fragility curves.

The main objective of this chapter is to answer two questions. (1) Does considering duration correlation in ground motion selection appreciably change the hazard input for analysis? (2) Does considering duration correlation change the predicted performance of bridges especially when doing incremental dynamic analysis? Section 2.4 and 2.5 focus on developing the fragility curves, with and without considering duration correlation, for the major bridge and typical overpass following the PBEE methodology steps.

The major bridge has lead rubber bearing isolators on top of flexible piers, is located in Southern California and has a natural period of around 1.6 seconds. This bridge was selected because it can be affected by near fault conditions which are related to ground motion duration. The typical highway overpass is a single-pier, two-span bridge located in Northern California with a period of about 0.5 seconds. A site-specific hazard analysis is done to obtain two sets of ground motions; one ignoring duration correlation and implementing an incremental dynamic analysis (IDA) (FEMA-P695, 2009, Vamvatsikos and Cornell, 2002), and one considering the duration correlation in the scaling using correlation coefficients presented in this chapter. Then a structural model is developed capturing material deterioration which is a critical feature to understand the effect of duration. Finally, damage states are determined based on pier drift and bearing shear strain to develop structural fragility curves for both sets of ground motions and used to compare the predicted performance at moderate and major damage levels.

2.3 Ground Motion Duration Correlation

Slate Geotechnical Consultants found that duration and spectral acceleration are negatively correlated. This means that when the target spectral acceleration increases, the average duration of the ground motions selected for that hazard level should be lower than the average duration of the original scenario. This subsection provides a brief overview of the process used to develop the correlation relationship between ground motion duration and spectral acceleration and summarizes the ground motion selection methodology. For additional details see Lopez Ruiz et al. (2024 - in submission).

2.3.1 Correlation

This section summarizes the procedure and data used to develop the ground motion duration correlation coefficients to pseudo-spectral acceleration (PSA). The relationship between PSA and ground motion duration for both $D_{5-75\%}$ (5-75% of normalized Arias Intensity) and $D_{5-95\%}$ (5-95% of normalized Arias Intensity) was developed using four ground motion models (GMMs) and three duration prediction equations (DPE). They used the NGA-West2 ground motion database (Ancheta et al., 2014) to calculate the PGA and duration residuals. The NGA-West2 database encompasses data from active crustal regions including California, Taiwan, Europe, China, and Japan. The duration correlation relationship used data from about 600 earthquakes and a total of 21,539 records. The database features records that vary in moment magnitude (M_w), distance and V_{s30} with ranges of 2.99 to 7.9 for M_w , 0.05 to ~1500 km for distance, and ~90 to 2100 m/s for V_{s30} , with most data falling between 200 to 500 m/s. Figure 2.1 shows the database M_w vs distance, to provide a visual representation of the amount of data used to develop the correlation parameters.

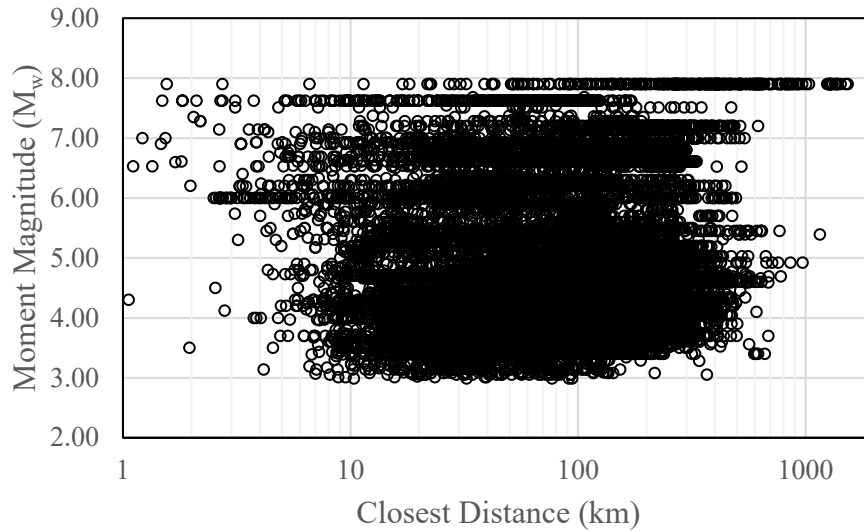


Figure 2.2 Magnitude Distance plot of available data in the NGA-West2 database

The correlation coefficient, ρ , was then calculated using Equation 2.1:

$$\rho = \frac{n \sum xy - \sum x \sum y}{\sqrt{(n \sum x^2 - (\sum x)^2)(n \sum y^2 - (\sum y)^2)}} \quad (2.1)$$

where:

x is the intraevent residual for the GMM;

y is the intraevent or total residual of the duration and;

n is the number of samples.

The correlation coefficient was calculated for four GMMs (ASK14, BSSA14, CB14, and CY14) and three DPEs (AS96, B09, and AS16) with two duration measures ($D_{5-75\%}$ and $D_{5-95\%}$) resulting in a total of 24 correlation sets for periods between 0.01 seconds and 10 sec. See references for each GMM and DPE in Table 2.1 and Table 2.2, respectively

Table 2.1 GMMs used to develop correlation coefficients

GMM	Reference
ASK14	Abrahamson et al. (2014)
BSSA14	Boore et al. (2014)
CB14	Campbell and Bozorgnia (2014)
CY14	Chiou and Youngs (2014)

Table 2.2 DPEs used to develop correlation coefficients

DPE	Reference
AS96	Abrahamson and Silva (1996)
B09	Bommer et al. (2009)
AS16	Afshari and Stewart (2016)

To facilitate the implementation of the correlation coefficients, the correlations are fit to an equation. The authors used a slightly modified version of the functional form from Baker and

Jayaram (2008) and applied a least squares regression to solve for the coefficients. The functional form of the equation is given in Equations 2.2 through 2.6:

$$C_1 = 1 - \cos\left(\frac{\pi}{2} - x_1 \ln\left(\frac{T_{max}}{\max(T_{min}, x_2)}\right)\right) \quad (2.2)$$

$$C_2 = \begin{cases} 1 - x_3 \left(1 - \frac{1}{1 + e^{x_4 T_{max}^{x_5}}}\right) \left(\frac{T_{max} - T_{min}}{T_{max} - x_5}\right) & \text{if } T_{max} < x_7 \\ 0 & \text{otherwise} \end{cases} \quad (2.3)$$

$$C_3 = \begin{cases} C_2 & T_{max} < x_1 \\ C_1 & \text{otherwise} \end{cases} \quad (2.4)$$

$$C_4 = C_1 + x_8(\sqrt{C_3} - C_3)(1 + \cos\left(\frac{\pi T_{min}}{x_1}\right)) \quad (2.5)$$

$$\rho = \begin{cases} \rho_0 \max(C_1, C_2, C_4) & T < T_3 \\ \rho_0 \min(C_1, C_2, C_4) & \text{otherwise} \end{cases} \quad (2.6)$$

*if C_1 , C_2 , or $C_4 > 1$ they are ignored in the min and max of Equation 2.6

where:

x_1 through x_8 are defined for each model in Table 2.3;

T_{max} and T_{min} are found by comparing T and T_2 ;

T_2 is the period at which the correlation coefficient reaches its maximum magnitude (per Table 2.1);

T is the period of interest for the user, and;

ρ_0 is the correlation coefficient at T_2 ;

Equations 2.2 through 2.5 are directly from Baker and Jayaram (2008) while the piecewise function in Equation 2.6 was slightly modified by the authors to fit the correlation coefficients found. The curve fitting described was performed for all 24 correlation sets obtained from Equation 2.1.

It is common practice for the four GMM relationships to be equally weighted, and as such the average was obtained by performing a regression on the correlation sets for the four GMMs. Figure 2.3 and Figure 2.4 show the normalized correlation coefficient curves obtained from the average of the four GMMs for the three DPEs considered as a function of period for D5-75% and D5-95% respectively. To obtain the correlation coefficient at T , the normalized coefficient needs to be multiplied by ρ_0 . Note that the trends of the normalized coefficients for the three DPEs are similar when the four GMM comparisons are given the same weight. The short periods have the

strongest correlation, and the coefficient decreases as the period increases until it reaches a value of zero.

It should be noted that this correlation is only applicable for shallow crustal earthquakes due to the use of the NGAWest2 database. This correlation may not be applicable to subduction earthquakes which are typically greater in magnitude and duration.

Table 2.3 Coefficients for the average correlation coefficients for the three DPE's

Coefficient	AS96			B09			AS16		
	D _{5-75%}	D _{5-95%} (T ≤ 0.2 sec)	D _{5-95%} (T > 0.2 sec)	D _{5-75%}	D _{5-95%} (T ≤ 0.2 sec)	D _{5-95%} (T > 0.2 sec)	D _{5-75%}	D _{5-95%} (T ≤ 0.2 sec)	D _{5-95%} (T > 0.2 sec)
T ₂	0.01	0.2		0.01	0.2	0.2	0.01	0.01	0.01
T ₃	0.4			10	0.4		10	0.4	
ρ ₀	-0.2726	-0.1558		-0.2807	-0.2090		-0.5217	-0.5008	
x ₁	0.270	0.1752	0.4075	0.4499	0.1175	0.4000	0.477	0.08	0.3266
x ₂	0.110	0.07	0.1297	0.1795	0.07	0.165	0.25	0.12	0.2
x ₃	0.15	0.0883		0.094	0.0676		0.1077	0.07	
x ₄	300	538.1269		853.357	847.9955		345.3948	100	
x ₅	9.00	16.4533		26.42	26.011		17.3465	5	
x ₆	0.015	0.0199		0.0199	0.02		0.0252	0.0099	
x ₇	0.60	0.1476		0.2	0.2		0.2001	0.0951	
x ₈	0.60	0.7719	0.4495	0.321	1.0000	-0.1091	0.0225	-0.7	-0.5393

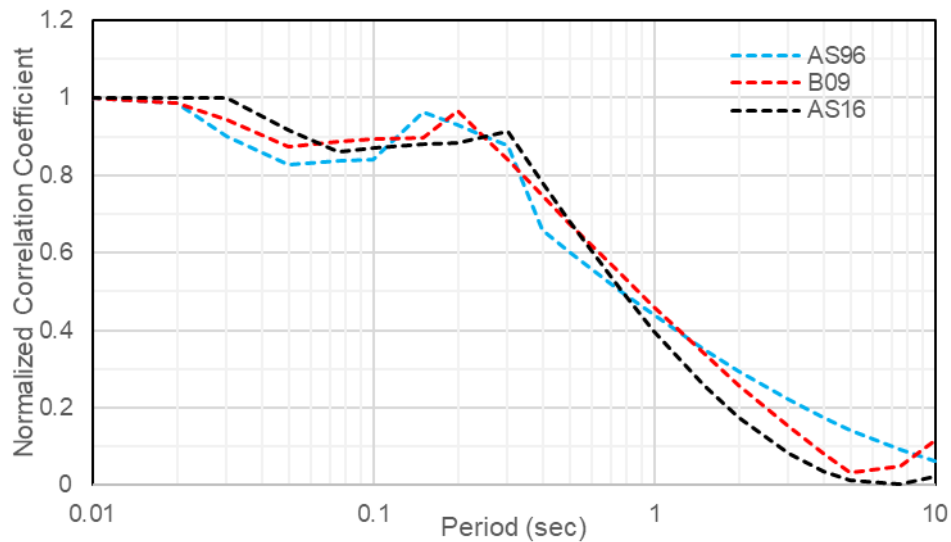


Figure 2.3 Comparison of three DPE curves of the normalized correlation coefficients for D5-75%

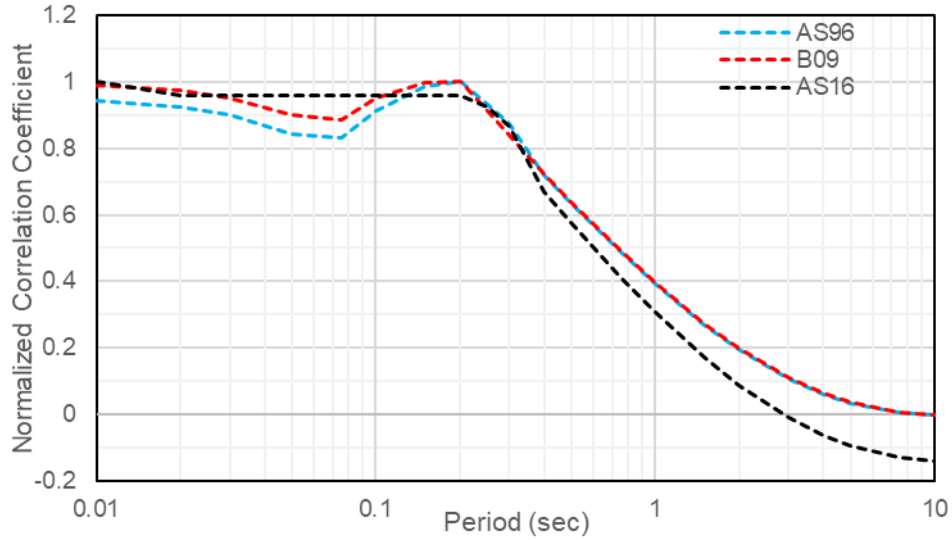


Figure 2.4. Comparison of three DPE curves of the normalized correlation coefficients for D5-95%

2.3.2 Hazard Analysis and Ground Motion Selection

To evaluate the impact of the duration correlation coefficients on the damage probability, multiple suites of ground motions were selected using traditional ground motion selection methods and implementing the proposed correlation coefficients. To select the ground motions, a probabilistic seismic hazard analysis (PSHA) was performed for the two bridges considered in this study, the major bridge in Southern California and the typical highway overpass in Northern California. PSHA is used to find the probability of different shaking intensities and typically contains four stages: (1) identifying seismic sources that contribute to the hazard at the site of interest, (2) characterizing the seismic source by finding the distribution of earthquakes magnitude and distance, (3) selecting GMMs, and (4) Computing the seismic hazard through a probabilistic analysis.

The seismic source characterization is based on UCERF3 (Field et al., 2013), a multi-year project that provides an authoritative estimate of the magnitude, location, and frequency of potentially damaging earthquakes in California. UCERF3 addresses issues such as relaxing fault segmentation assumptions and accommodating multi-fault ruptures. It includes all possible earthquake ruptures above a magnitude of 5 and offers two alternative fault models representing the epistemic uncertainty in fault system geometry. The four GMMs from NGAWest2 listed in

Table 2.1 were used and assigned equal weight to compute the probability distribution of ground motion intensity as a function of variables like moment magnitude, distance, faulting mechanism and others. The PSHA calculations were executed using the computer program Haz45.2 developed by Norm Abrahamson¹ and modified by Slate to include the Third Uniform

¹ Abrahamson, N. (2017). "HAZ" v45.2. Contributors: Hale, C.; Kottke, A.; Hsieh, P.S.; Murphy, D.G. and Gregor, N.J.

California Earthquake Rupture Forecast (UCERF3) (Field et al., 2013). This program was validated as part of the Pacific Earthquake Engineering Research (PEER) Center Probabilistic Seismic Hazard Analysis Code Verification Project (Hale et al., 2018).

The deaggregation results from the PSHA result in magnitude and distance ranges used to select seven suites of seed ground motions with consideration of overall spectral shape and scaling factor. Each suite consists of eleven ground motions of two-component seed records at their as-recorded orientations. The first suite was selected using non-correlated durations, magnitude, distance, and other secondary earthquake parameters. The other six suites were selected based on the predicted duration parameters for the uniform hazard spectra scaled to factors of 2, 2.5, 3, 3.5, 4, and 4.5. Details about the use of the duration correlation parameters and criteria used to sort, prioritize, and select seed time histories, are discussed in Section 2.4 and Section 2.5.

The final selected set of seed time histories are spectrally matched, over a period range of PGA to 5.0 sec, to their corresponding Uniform Hazard Spectra (UHS) using RSPMatch.2 By performing spectral matching, the difference between the suites of records can be isolated to those of their non-stationary properties like duration. The 2018 release of RSPMatch, used in the study, allows for the component-to-component variability of the two horizontal components to be maintained. After the spectral matching process, each record was baseline corrected, which ensures that the acceleration, velocity, and displacement return to zero at the end of the record. Finally the resulting response spectra were calculated for each record component.

² 2018 release, executable received from N. Abrahamson via email November 12, 2018

2.4 Major Bridge

The first bridge considered is based on the design and location of the San Diego-Coronado bridge which underwent a structural seismic retrofit in 1999. As part of this retrofit, lead rubber bearings (LRBs) were installed on 13 of its piers while the remaining piers have rigid connections to the deck. The Coronado bridge has spans of about 200 ft over the isolated span, and the piers have heights of up to 160 ft. This bridge was selected for the case study as a major bridge due to its long spans and tall piers making it more flexible than a typical highway bridge. The major bridge with flexible piers has a natural period of around 1.6 seconds. The location of the bridge is shown in Figure 2.5. An overview photo of the bridge and a close-up of a top of a pier with a lead rubber bearing are shown in Figure 2.6.

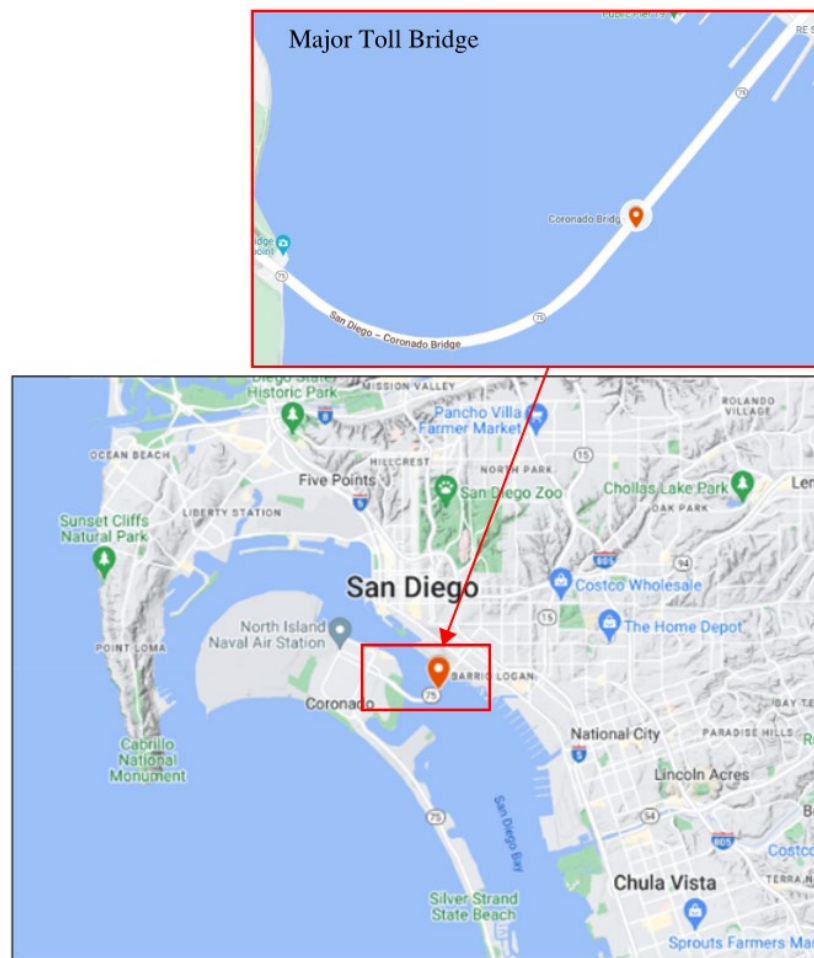


Figure 2.5 Major bridge location in Southern California showing curved span on plan view.³

³ Google Maps. 2023. [Coronado bridge] [Online]. Available: <https://www.google.com/maps/@32.6950075,-117.1601968,14.5z?entry=ttu> [Accessed May 23, 2023].

The PBEE methodology is followed through to the damage analysis phase to evaluate the performance of two bridge models to compare the performance of each structure using more traditional IDA and the proposed hazard selection methodology using a correlated spectral duration and acceleration. Fragility curves for each case are presented to compare the expected bridge performance under both ground motion selection methods.

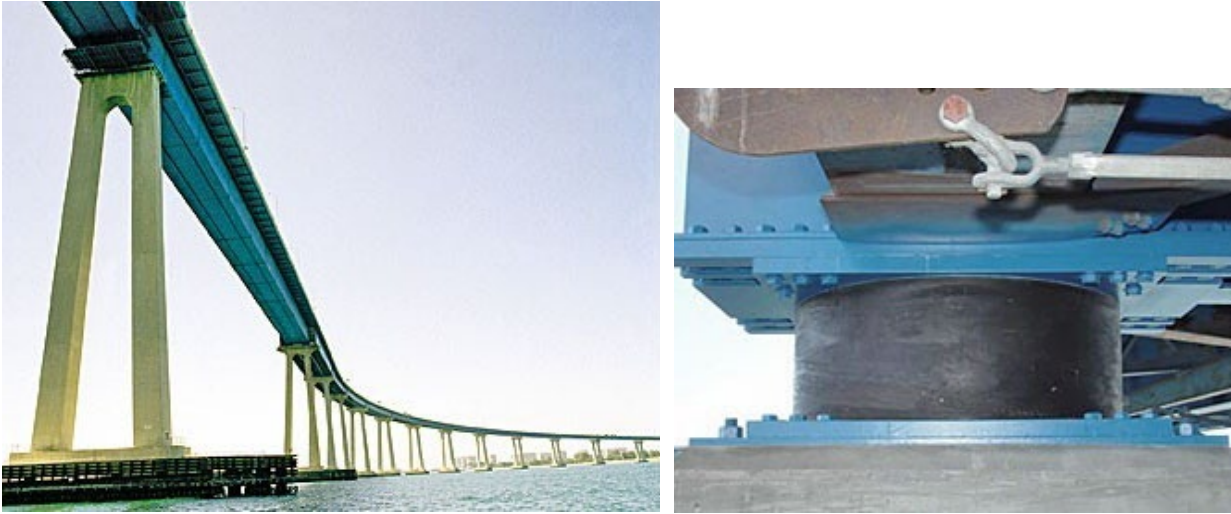


Figure 2.6 Major bridge picture of flexible piers and top of pier lead rubber bearing.⁴

2.4.1 Ground Motion Selection

2.4.1.1 Hazard Analysis

The UHS for a return period of 2,475 years (corresponding to a 2% probability of exceedance in 50 years) was developed using the method outlined in Section 2.3 and is shown in Figure 2.7. Figure 2.8. depicts the deaggregation of the PSHA results for a V_{S30} of 300 m/s for the period of the bridge (1.6 seconds). The major bridge is dictated by a Mw 7.1 at 1 km according to the deaggregation results. These values were used in the deterministic analysis to calculate the ε for the duration correlation analysis (discussed later).

⁴ Forell | Elsesser Engineers. San Diego Coronado Bay Bridge Seismic Retrofit [Online]. Available: <https://forell.com/projects/transportation/coronado-bay-bridge-seismic-retrofit/> [Accessed May 23, 2023].

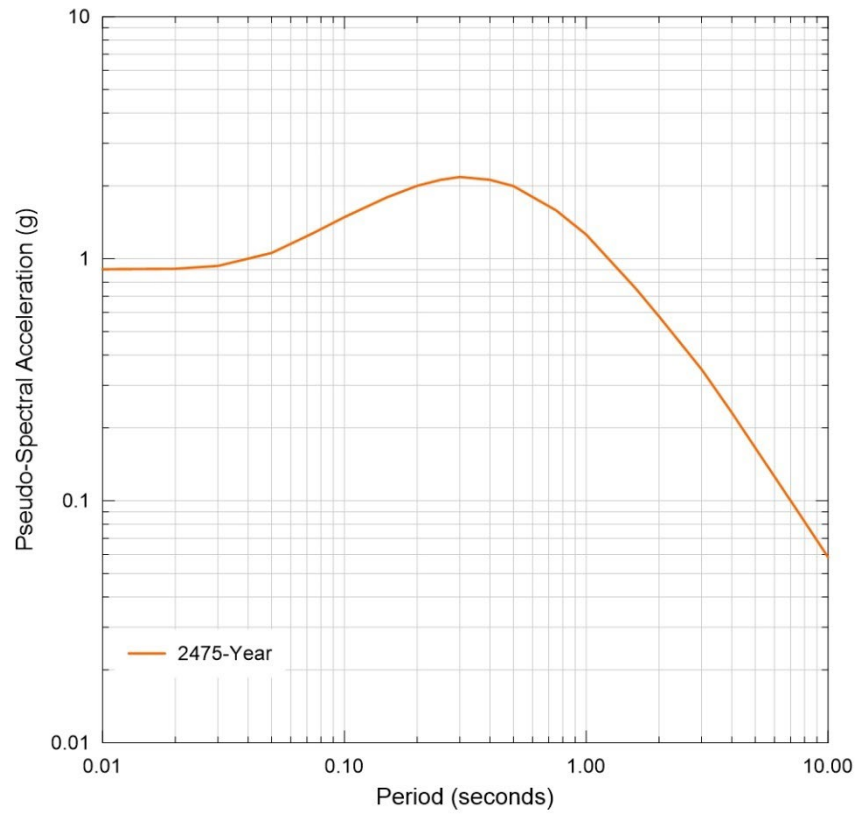


Figure 2.7. Uniform hazard spectra for a return period of 2,475 years (corresponding to a 2% probability of exceedance in 50 years, Maximum Considered Earthquake) for the major bridge location.

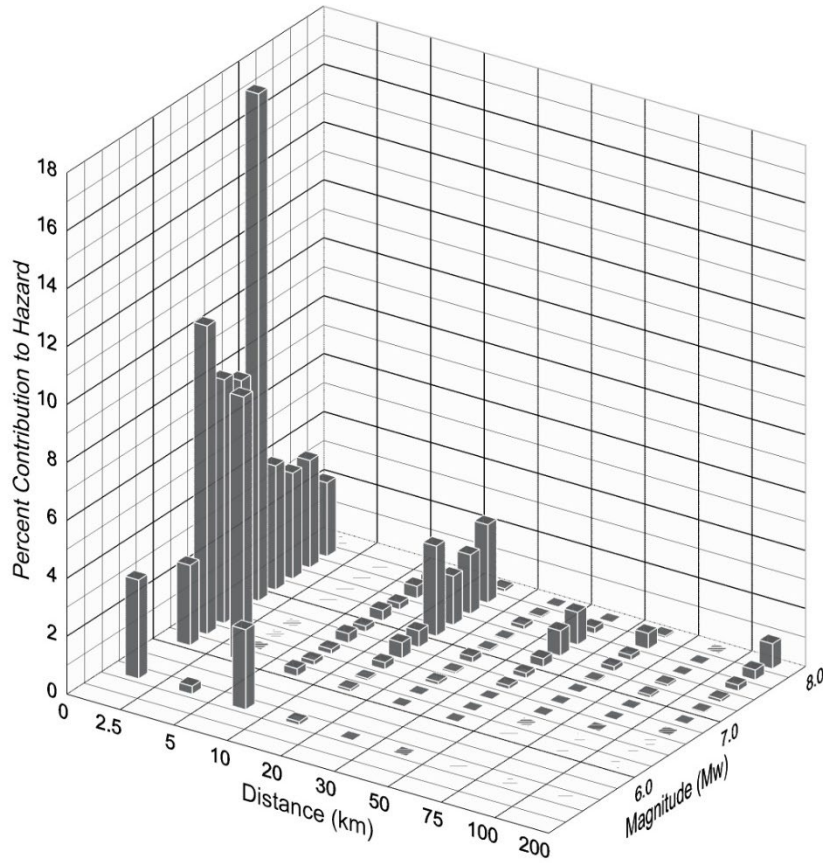


Figure 2.8. 2475-year return period hazard deaggregation at a period of 1.6 second showing near fault contribution.

2.4.1.2 General Selection of Seed Time Series

The specific earthquake source parameters that represent the dominant earthquake scenario in the seismic hazard were used in the selection of seed time series. Criteria used to sort, prioritize, and select seed time series for the bedrock analysis spectra from the PEER Center NGA-West2 Ground Motion Database included the following for all suites:

- Magnitude (M_w): 7.0 – 8.6;
- Rupture distance: 0 – 50 km;

2.4.1.3 Selection of Non-correlated Suite

The first suite was selected based on the mean duration values from the DPEs and the other parameters listed above. For the site the mean duration values from the DPEs are as follows:

- D₅₋₇₅: 6.0 seconds
- D₅₋₉₅: 13.0 seconds

The suite using these duration bounds is shown in Table 2.4.

Table 2.4 Non-correlated suite of time histories

RSN	Earthquake Name	Station Name	Mw	D _{5-75%} (sec)	D _{5-95%} (sec)
828	Cape Mendocino	Petrolia	7.0	6.5	17.7
1158	Kocaeli, Turkey	Duzce	7.5	6.1	11.8
1509	Chi-Chi, Taiwan	TCU074	7.6	13.3	19.7
1787	Hector Mine	Hector	7.1	7.6	11.7
3748	Cape Mendocino	Ferndale Fire Station	7.0	6.2	13.6
3934	Tottori, Japan	SMN002	6.6	6.7	15.6
3965	Tottori, Japan	TTR008	6.6	6.2	18.5
4031	San Simeon, CA	Templeton - 1-story Hospital	6.5	3.3	10.3
5818	Iwate, Japan	Kurihara City	6.9	6.5	15.1
6890	Darfield, New Zealand	Christchurch Cashmere High School	7.0	9.0	20.0
8063	Christchurch, New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4

2.4.1.4 Selection of Correlated Suites

The next six suites for each site were selected based on correlated duration bounds and scaled versions of the UHS. The correlation coefficient at the fundamental period of each bridge was used and applied through Equation (2.7), below.

$$\mu_{dur|Sa} = \mu_{dur} + \varepsilon \rho_{os} \sigma_{dur} \quad (2.7)$$

where,

μ_{dur} is the mean duration from the DPE;

σ_{dur} is the standard deviation of the DPE;

ρ_{os} is the correlation coefficient at the period of the bridge (calculated in Section 2.3);

ε is the number of standard deviations of the scaled UHS (at the period of the bridge) from the median ground motion; and

$\mu_{dur|Sa}$ is the correlated mean duration.

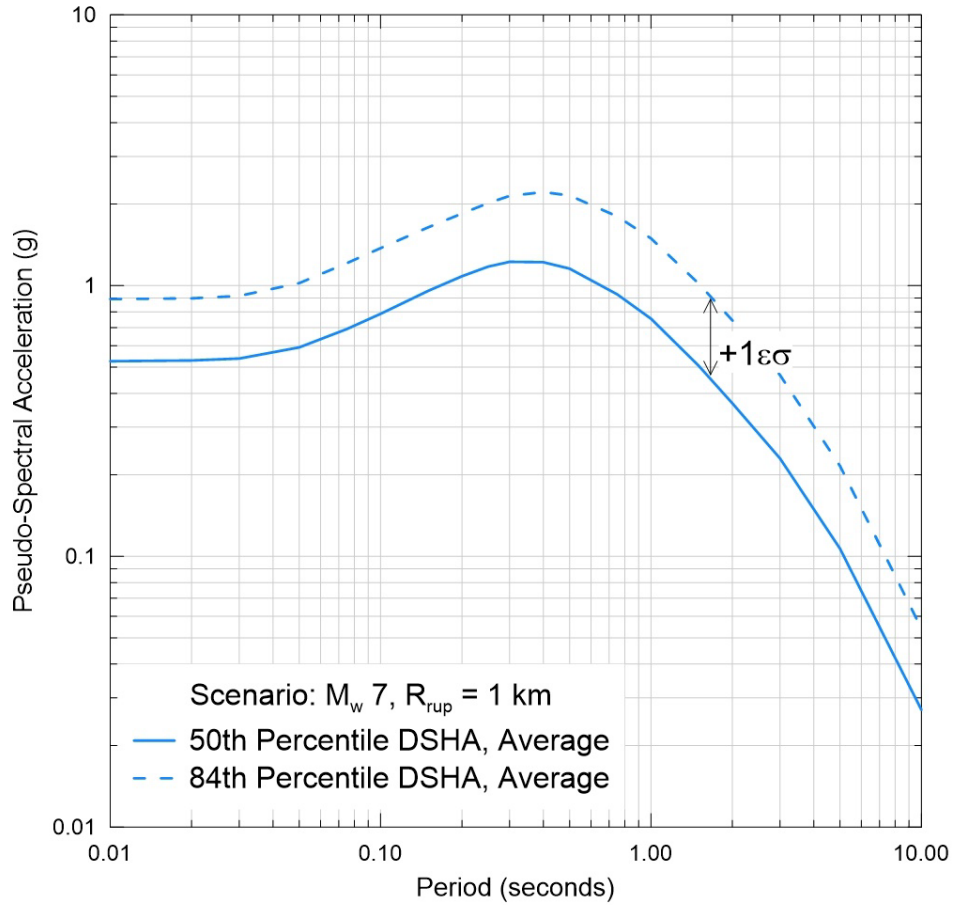


Figure 2.9. Deterministic spectra and depiction of epsilon for the Major Bridge site

ϵ is calculated using the deaggregation results from the PSHA to calculate a deterministic median pseudo-spectral acceleration and standard deviation using the four NGA-West2 GMMs (shown in Figure 2.9). An example of one ϵ is depicted between the median and 84th percentile deterministic spectra. The UHS (and scaled UHS) are compared to this deterministic scenario to calculate the number of standard deviations from the median to get the actual ϵ .

To develop the fragility curves scale factors of 2, 2.5, 3, 3.5, 4, and 4.5 were applied to the UHS, the correlated $D_{5-75\%}$ and $D_{5-95\%}$ were calculated, and additional time histories selected. The values of ϵ used for the correlated duration can be found in Table 2.5 and the correlated duration values are in Table 2.6. The time series for each of the scale factors and sites are outlined in Table 2.7 through Table 2.12. The main focus of this selection process was the duration values, so the magnitudes and other parameters of the suites are not as close to the deaggregation results. The difference between the mean duration used for the uncorrelated suite and the mean durations used for the correlated suite increases as scale factor increases.

Table 2.5. ϵ values (number of standard deviations from deterministic median ground motion) for the Major Bridge (1.6 sec)

Scale Factor	ϵ
1	0.68
2	1.67
2.5	1.99
3	2.25
3.5	2.47
4	2.67
4.5	2.83

Table 2.6 Correlated Duration values for the Major Bridge site

Major Bridge		
Scale Factor	D _{5-75%} (sec)	D _{5-95%} (sec)
1	5.79	12.78
2	5.49	12.46
2.5	5.39	12.36
3	5.32	12.28
3.5	5.25	12.21
4	5.20	12.16
4.5	5.15	12.12

Table 2.7 Suite of time histories for the UHS scale factor of 2, Major Bridge

RSN	Earthquake Name	Station Name	Mw	D _{5-75%} (sec)	D _{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
1116	Kobe, Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe, Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli, Turkey	Duzce	7.5	6.1	11.8
1492	Chi-Chi, Taiwan	TCU052	7.6	5.7	16.7
3748	Cape Mendocino	Ferndale Fire Station	7.0	6.2	13.6
4874	Chuetsu-oki, Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki, Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
6962	Darfield, New Zealand	ROLC	7.0	7.1	11.4
8063	Christchurch, New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4
8118	Christchurch_ New Zealand	Papanui High School	6.2	5.9	13.9

Table 2.8 Suite of time histories for the UHS scale factor of 2.5, Major Bridge

RSN	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
1116	Kobe_ Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe_ Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli_ Turkey	Duzce	7.5	6.1	11.8
1492	Chi-Chi_ Taiwan	TCU052	7.6	5.7	16.7
3748	Cape Mendocino	Ferndale Fire Station	7.0	6.2	13.6
4229	Niigata_ Japan	NIGH12	6.6	5.4	10.1
4874	Chuetsu-oki_ Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki_ Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8063	Christchurch_ New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4
8118	Christchurch_ New Zealand	Papanui High School	6.2	5.9	13.9

Table 2.9 Suite of time histories for the UHS scale factor of 3, Major Bridge

RSN	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
1116	Kobe_ Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe_ Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli_ Turkey	Duzce	7.5	6.1	11.8
1492	Chi-Chi_ Taiwan	TCU052	7.6	5.7	16.7
3964	Tottori_ Japan	TTR007	6.6	4.9	12.7
4229	Niigata_ Japan	NIGH12	6.6	5.4	10.1
4874	Chuetsu-oki_ Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki_ Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8063	Christchurch_ New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4
8118	Christchurch_ New Zealand	Papanui High School	6.2	5.9	13.9

Table 2.10 Suite of time histories for the UHS scale factor of 3.5, Major Bridge

Record Number	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
1116	Kobe, Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe, Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli, Turkey	Duzce	7.5	6.1	11.8
1492	Chi-Chi, Taiwan	TCU052	7.6	5.7	16.7
3964	Tottori, Japan	TTR007	6.6	4.9	12.7
4117	Parkfield-02, CA	Parkfield - Fault Zone 15	6.0	5.3	11.7
4228	Niigata, Japan	NIGH11	6.6	3.9	12.2
4874	Chuetsu-oki, Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki, Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8063	Christchurch, New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4

Table 2.11 Suite of time histories for the UHS scale factor of 4, Major Bridge

Record Number	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
1111	Kobe, Japan	Nishi-Akashi	6.9	4.5	11.2
1116	Kobe, Japan	Shin-Osaka	6.9	4.5	11.6
1158	Kocaeli, Turkey	Duzce	7.5	6.1	11.8
1492	Chi-Chi, Taiwan	TCU052	7.6	5.7	16.7
3964	Tottori, Japan	TTR007	6.6	4.9	12.7
4117	Parkfield-02, CA	Parkfield - Fault Zone 15	6.0	5.3	11.7
4228	Niigata, Japan	NIGH11	6.6	3.9	12.2
4874	Chuetsu-oki, Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki, Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8063	Christchurch, New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4

Table 2.12 Suite of time histories for the UHS scale factor of 4.5, Major Bridge

Record Number	Earthquake Name	Station Name	Mw	D _{5-75%} (sec)	D _{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
768	Loma Prieta	Gilroy Array #4	6.9	5.4	14.8
1111	Kobe, Japan	Nishi-Akashi	6.9	4.5	11.2
1116	Kobe, Japan	Shin-Osaka	6.9	4.5	11.6
1158	Kocaeli, Turkey	Duzce	7.5	6.1	11.8
3964	Tottori, Japan	TTR007	6.6	4.9	12.7
4117	Parkfield-02, CA	Parkfield - Fault Zone 15	6.0	5.3	11.7
4228	Niigata, Japan	NIGH11	6.6	3.9	12.2
4874	Chuetsu-oki, Japan	Oguni Nagaoka	6.8	5.6	12.2
4894	Chuetsu-oki, Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8063	Christchurch, New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4

2.4.2 Structural Analysis

A 2D schematic of the bridge profile is shown in Figure 2.10 including modeling details. However, it is important to note that a full 3D model was developed using OpenSees (McKenna et al., 2010) to capture the behavior due to the 90-degree bend on the span and considering bidirectional ground motion loading. Each pier consists of two reinforced concrete columns. The first 14 piers (see Figure 2.10 for numbering) are isolated using four LRBs located on top of each capping beam. The pier height and span length along the centerline are listed in Table 2.13.

The pier dimensions increase with increasing height but maintain an assumed reinforcement ratio of 1%. To simplify the model, piers with similar geometric characteristics are grouped together. The piers were divided into 7 sets with geometric characteristics summarized in Table 2.14 including Column Width, B ; Column Depth, D ; number of longitudinal bars at top and bottom (#14), number longitudinal bars at the sides (#14), and plastic hinge length, L_p . The sets are Pier 2 through Pier 6, Pier 7 – Pier 11, Pier 12 and 13, and Pier 14 – 17, Pier 18 and 21, Pier 19 and 20, and Pier 22 – 26. A schematic of column section is provided in Figure 2.11. The piers are modeled using beam-with-hinges elements with Concrete02 and Steel02 materials to capture the nonlinear behavior of piers and the concrete degradation of the pier with cyclic loading. A confined core and unconfined concrete fiber section was defined. The compressive strength of concrete is assumed to be 5 ksi and the expected yield strength of the steel rebar is defined as 69 ksi. The plastic hinge length was calculated using Priestley and Park (1987) equation based on pier length (L) and reinforcement diameter (D_{bl}) and adapted by Caltrans (2019): P-Delta effects are considered for all piers to account for geometric nonlinearity.

$$L_p = 0.08L + 0.15f_y D_{bl} \geq 0.3f_y D_{bl} \quad (2.8)$$

Contrary to buildings, bridges are designed with a “strong beam – weak column” philosophy. As such, damage to bridges is often concentrated on the formation of plastic hinges in the pier while the deck remains elastic. Given the knowledge of this design practice, the pier caps and deck are modeled using elastic beam elements. The deck masses are lumped at the top of each bearing node, and the pier beam cap masses are lumped at the top of each pier. A consistent mass matrix is formulated for the distributed mass of the piers along their height. The structural damping is estimated using Rayleigh damping proportional to mass and the tangent stiffness with coefficients based on the first two modes and a damping ratio of 3% per Caltrans (2019) Section 4. The boundary conditions are simplified by assuming fixed pier bases and a pinned abutment connection. The model ends at pier 26 where the approach ramps begin.

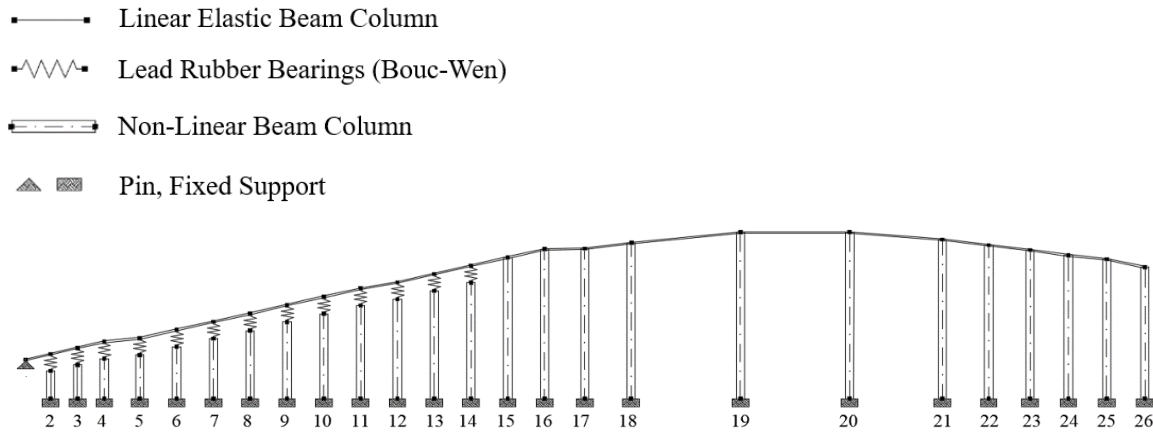


Figure 2.10 Major bridge model in profile view.

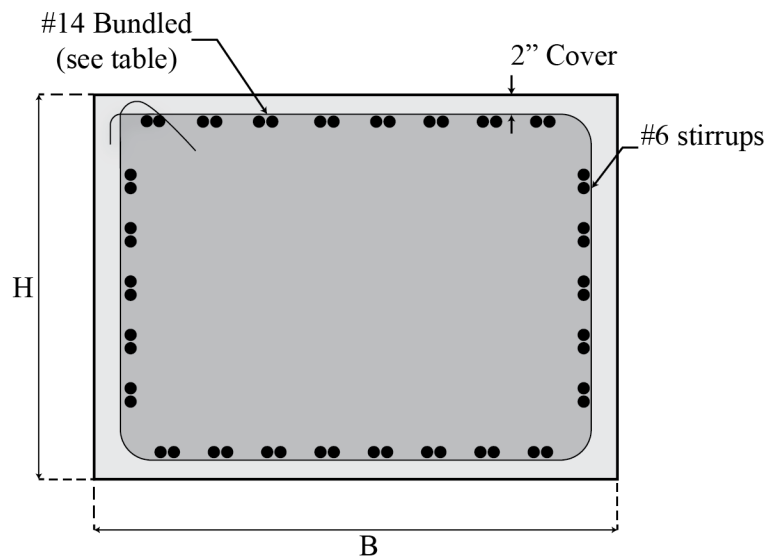


Figure 2.11 Major bridge pier cross-section.

The design of the lead rubber bearings is the same across all piers. The geometric and mechanical properties are summarized in Table 2.15. The LRB is modeled using a two node link element with a hysteretic model including energy degradation in the shear and linear behavior in axial and rotational degrees-of-freedom. Using the geometric properties the mechanical properties of the isolator are calculated following Kelly and Konstantinidis (2011). The vertical and rotational stiffness, K_v and K_θ , are calculated using Equation 4.3 and 4.4 respectively, where E_c is the instantaneous compression modulus of rubber, A is the loaded rubber area, t_r is the total thickness of rubber, and El_{eff} is the bending stiffness of the annular bearing. The hysteretic behavior of the bearing including degradation is shown in Figure 2.12. The OpenSees hysteretic material parameters that control hysteretic degradation is $damage_2$, δ_A , and has a value of 0.005.

$$K_v = \frac{E_c A}{t_r} \quad (2.9)$$

$$K_\theta = \frac{El_{eff}}{t_r} \quad (2.10)$$

Table 2.13 Pier properties.

Pier	Height (ft)	Span Length (ft)
1	-	150
2	28.0	162
3	35.0	162
4	42.6	215'-3"
5	50.4	222'-4"
6	65.3	222'-4"
7	75.7	222'-4"
8	85.7	222'-4"
9	96.4	222'-4"
10	106.8	222'-4"
11	117.2	222'-4"
12	125.1	222'-4"
13	135.5	222'-4"
14	145.9	222'-4"
15	156.2	222
16	166.7	242
17	167.5	280
18	174.6	660
19	187.9	660
20	187.9	560
21	178.8	280
22	171.9	250
23	165.7	231
24	159.0	231
25	154.0	231
26	144.0	-

Table 2.14 Column properties.

Pier Set	B (ft)	D (ft)	N_{l_topbot}	N_{l_side}	L_p (in)
Piers 2 – 6	8.5	8.0	16	8	80
Piers 7 – 11	9.0	9.0	20	8	130
Piers 12 – 13	10.5	11.0	28	10	150
Pier 14 – 17	11.0	12.0	30	14	160
Pier 18 & Pier 21	11.5	13.5	34	16	160
Pier 19 – 20	14.5	14.5	36	32	160
Pier 22 – 26	11.0	12.5	30	14	160

Table 2.15 Lead Rubber Bearing Properties.

Bearing Property	
Bearing Diameter, $D_{bearing}$ (in)	45.0
Lead core diameter, D_{lead} (in)	9.0
Number of layers, n	20
Total rubber thickness, t_r (in)	15.0
Yield stiffness, K_1 (kip/in)	51.25
Post-yield stiffness ratio, α (kip/in)	0.119
Characteristic strength, Q_d (kip)	87.75
Vertical Stiffness, K_V (kip/in)	3,664
Rotational Stiffness, K_θ (kip-in)	321,200

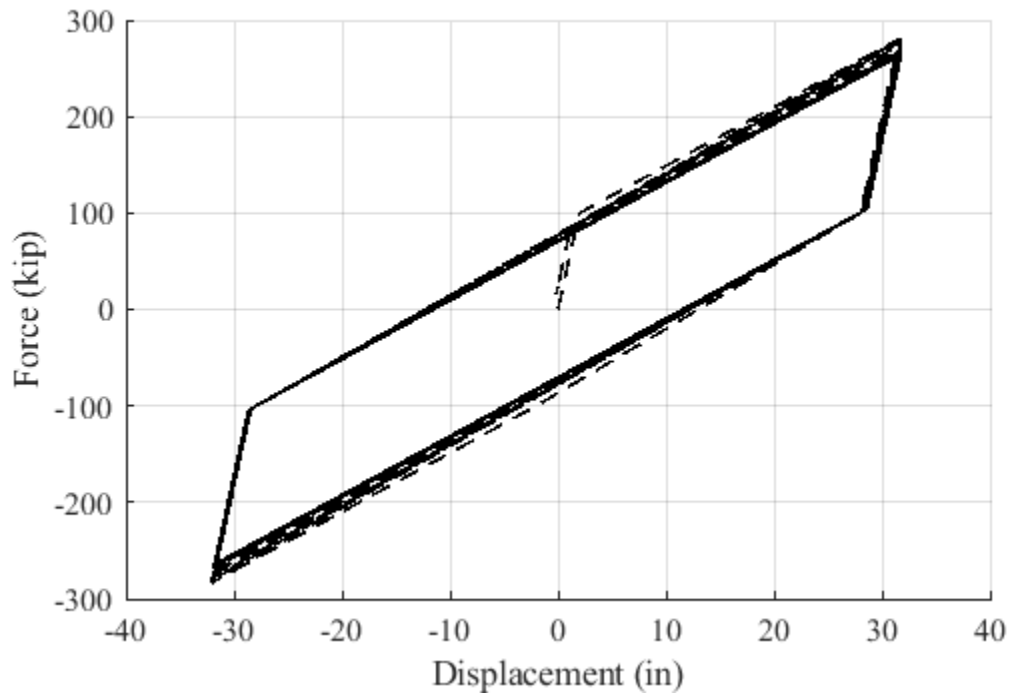


Figure 2.12 Lead rubber bearing model hysteresis.

2.4.3 Damage Analysis

The purpose of the damage state fragility curves developed in this study is to evaluate the effect of incorporating ground motion duration in the selection of ground motions in IDA. Thus, the fragility curves are not presented as a prediction of the exact bridge performance expected. To determine the damage state of the bridge, the damage states of the components, specifically the piers and isolation bearings, were assessed. Caltrans (2019) Section 1.3 suggests dividing performance into five damage states. This study presents results for two damage states of interest: DS-3 and DS-5, Moderate Damage and Major Damage, respectively. Caltrans defines expected performance; at DS-3 bridge repair is likely, but bridge replacement is unlikely, while at DS-5 bridge replacement is likely. For concrete piers, at DS-3 extensive cracks and spalling of the concrete is expected, while at DS-5 the onset of compressive failure of the core concrete is expected.

Currently there is no formal definition for bearing damage states; for the purpose of this study two bearing damage states are defined. The moderate damage state (DS-3) is defined as the bearing shear strain that leads to reaching the buckling limit under the isolator axial load using the overlapping area method (Buckle and Liu, 1994), which for this design is 235% shear strain. Calculating the stability limit using this method has been shown to lead to a conservative estimate of the stability limit (Weisman & Warn, 2012). As such, the major damage state (DS-5) is defined as the bearing shear strain under which there is no remaining overlap between the top and bottom of the bearing, which occurs at the displacement equal to the bearing diameter, which for this design is 300% shear strain. Table 2.17 shows the bearing damage states considered for analysis.

Previous studies on bridge column damage correlate engineering demand parameters with observed damage states. Vosooghi and Saiidi (2010) tested 30 reinforced concrete bridge columns and proposed five damage states. They reported statistics for each damage state for six response parameters: maximum drift ratio, residual drift ratio, frequency ratio, inelasticity index, maximum longitudinal steel strain, and maximum transverse steel strain. They found that piers tested under near-field conditions were able to sustain larger drifts before reaching each damage state than the rest of the columns. Vosooghi and Saiidi (2012) tested two more specimens and averaged the results for all 32 columns in the database to develop fragility curves for the six response parameters. A more recent study (Saini and Saiidi, 2014) expanded the existing database to a total of 38 columns and reported updated mean and logarithmic standard deviation for all six response parameters proposed by Vosooghi and Saiidi (2010) as well as updated damage fragility curves.

This study focuses on generating damage fragility curves using the maximum drift ratio of each column, which is easily obtained from OpenSees analysis, to classify the damage state of the individual piers. Since bidirectional ground motions were considered, the maximum displacement was calculated as the peak resultant displacement of each element. While updated parameters from the Vosooghi and Saiidi 2012 and 2014 studies expand the bridge column database, the lumped statistics do not include the difference in damage under near-field motions. As the major bridge is predominantly affected by near-field ground motions, the fragility curves developed using the data reported for columns tested under near-field motions by Vosooghi and Saiidi (2010) are used. Table 2.16 shows the pier damage states considered for analysis.

The damage fragility curves are found by counting the percent of the ground motions at each spectral acceleration scale under which the damage state occurs across any of the components. This is a conservative approach since it equates overall bridge performance to individual component performance. The fragility curves are obtained using the maximum likelihood fit per Baker (2015). The reason for using the maximum likelihood method instead of the typical method of moments estimator used to fit a lognormal distribution to IDA analysis is that like the case of the multiple stripes analysis (MSA), the suite of the ground motions selected using the correlated duration may not always present the IM values associated with 100% probability of a given damage state. As described by Baker (2015), the function fits a lognormal cumulative distribution function (CDF) to the observed probability of collapse data using optimization on the likelihood function for the data. The lognormal cumulative distribution function is most commonly used due to its simplicity, established precedent in the field of earthquake engineering, minimal knowledge requirements, and frequent fit to engineering data.

Table 2.16 Pier Damage Limits.

Damage State	Description	Drift Ratio Threshold
DS-1	Minimal Damage: Flexural Cracks	0.029
DS-2	Minor Damage: minor spalling and possible shear cracks	0.041
DS-3	Moderate damage: extensive cracks and spalling	0.059
DS-4	Moderate damage: visible lateral and/or longitudinal reinforcing bars	0.109
DS-5	Major Damage: imminent failure, onset of compressive failure of core concrete	0.131
DS-6	Failure/Collapse	-

Table 2.17 Bearing Damage Limits.

Damage State	Description	Shear Strain Threshold
DS-3	Moderate Damage: bearing bucking limit at isolator axial load based on overlapping area.	235%
DS-5	Major Damage: imminent failure	300%

2.4.4 Bridge Performance

2.4.4.1 *Non-Correlated Suite of Ground Motions – Fragility curves*

The observed fractions of reaching damage state DS-3 are plotted against the scale factor of each suite of ground motions, along with the curve fitted to the data for each element in Figure 2.13, Figure 2.14 and Figure 2.15 for the isolation bearings, the isolated piers, and the non-isolated piers, respectively. Figure 2.13 shows that the bearings on mid-height piers are the first to reach DS-3. For instance, the bearing on Pier 6 reaches the median damage probability at a scale factor of 1.09, while the bearing on Pier 2 reaches the same damage probability at a scale factor of 2.37.

The fragility curves of the piers for DS-3, shown in Figure 2.14, tend to increase with increasing pier height. The increase in damage probability with pier height is seen by the shift of the fitted curves to the left while maintaining a similar dispersion. For the tall piers, DS-3 is reached at smaller scale factors. The isolated piers reach the median damage at scale factors ranging from 3.3 to 5.3. Comparing these values with the ones obtained from the bearing elements on Figure 2.13, shows that the isolation layer is performing as intended by reducing the pier demands and controlling the response. In Figure 2.15 the same plot is repeated for the non-isolated piers. While these have a larger damage probability than the isolated piers, reaching the median damage probability at scale factors ranging from 3.3 to 3.9, the LRB fragilities continue to control the performance.

The trend in results for the element-level fragility curves are similar for DS-5, and the figures are included in Appendix B. The overall bridge damage is determined based on the maximum damage of any element at each intensity level. Figure 2.16 shows the data points for observed bridge damage with the fitted logarithmic fragility curves. Using the non-correlated motions, the bridge reached the median probability of DS-3 and DS-5 at scale factors of 1.09 and 1.19 respectively.

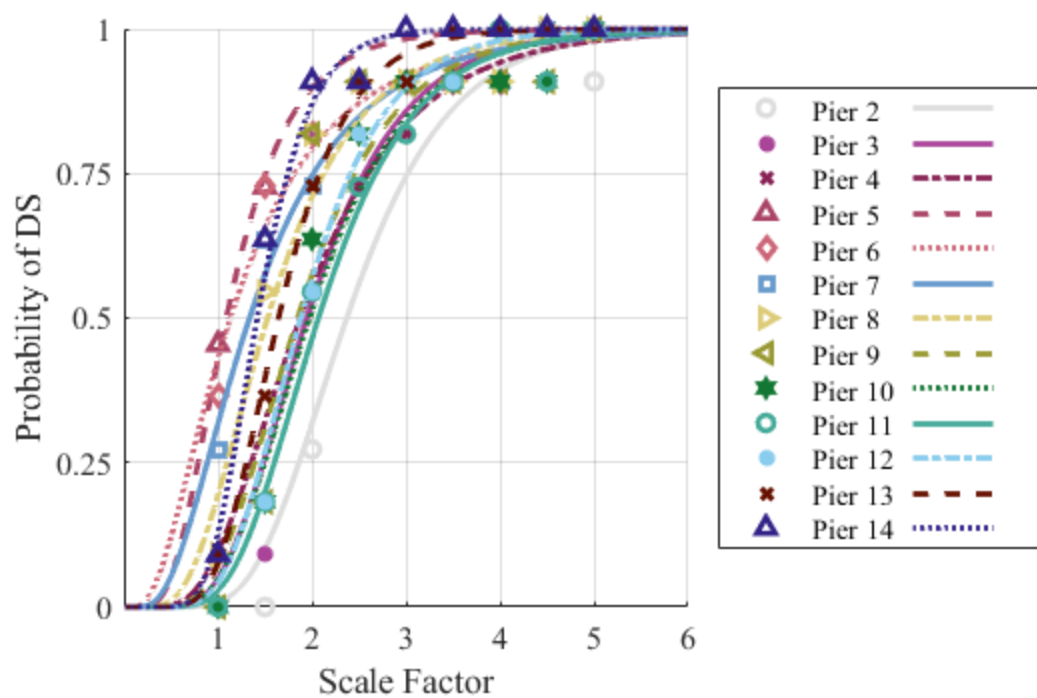


Figure 2.13 DS-3 fitted fragility curves for bearings under non-correlated ground motions.

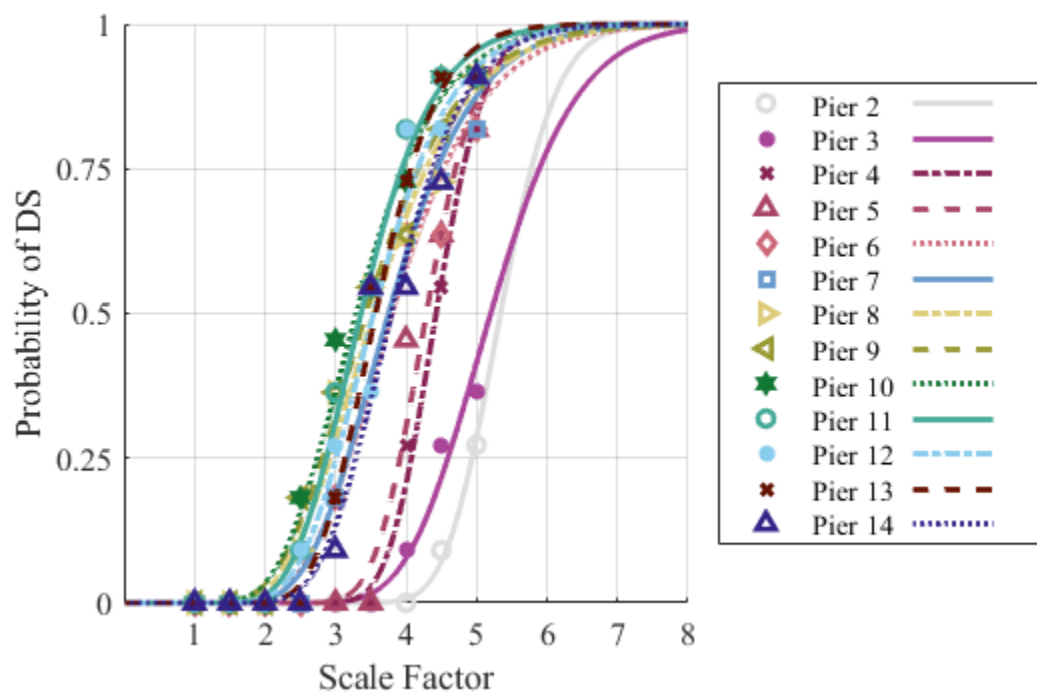


Figure 2.14 DS-3 fitted fragility curves for isolated piers under non-correlated ground motions.

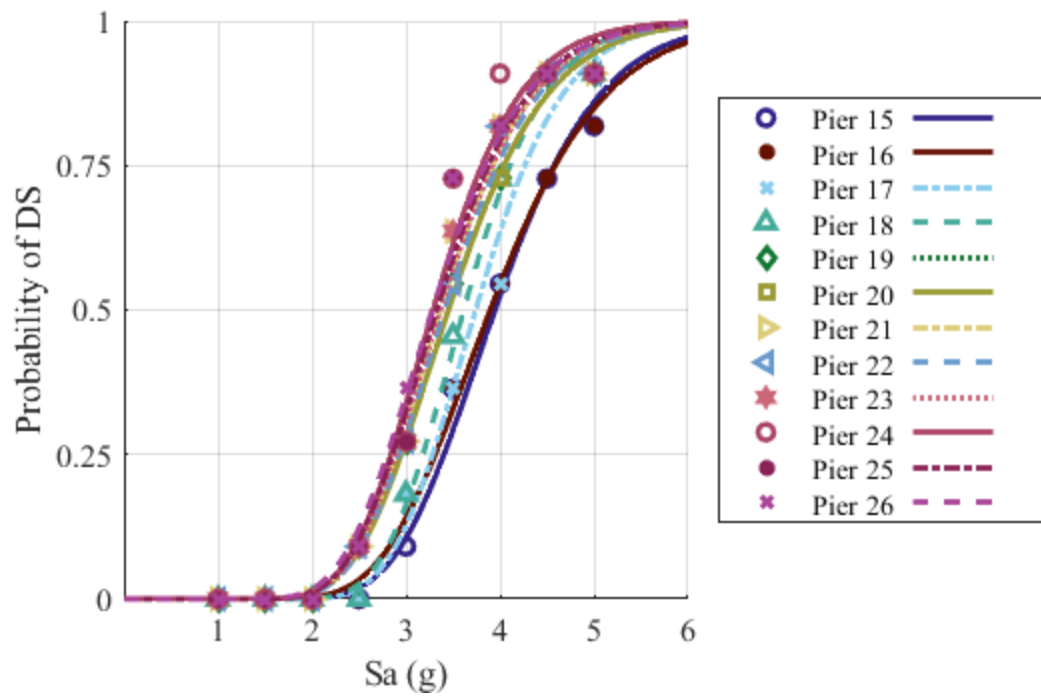


Figure 2.15 DS-3 fitted fragility curves for non-isolated piers under non-correlated ground motions.

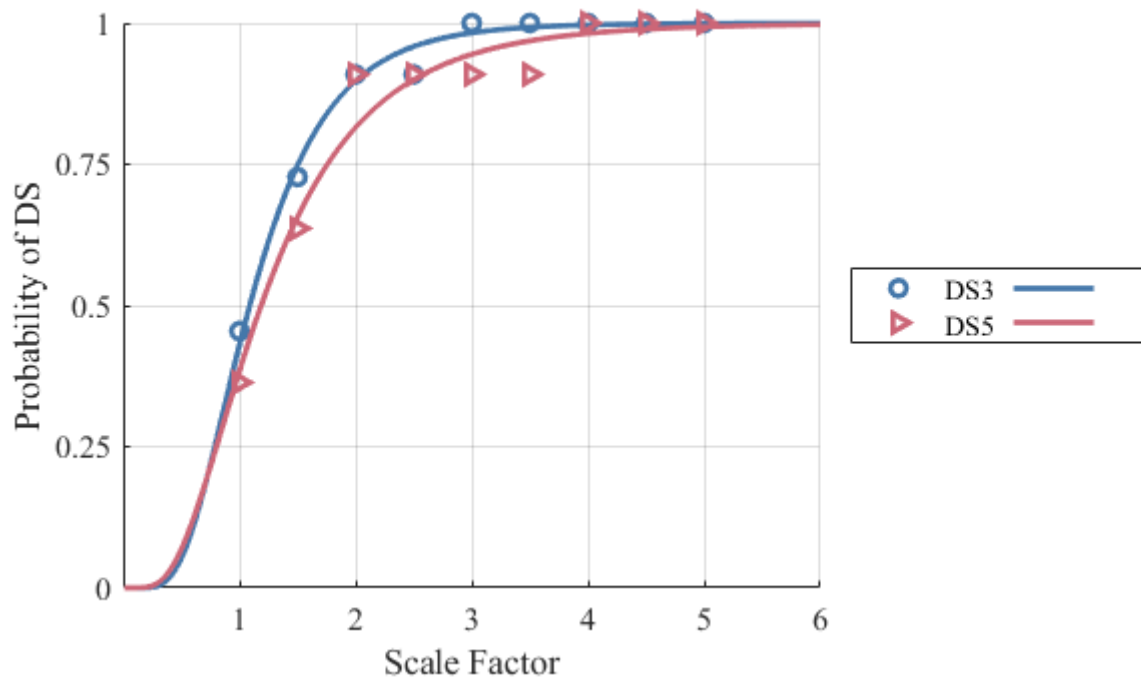


Figure 2.16 Major bridge fitted fragility curves for DS-3 and DS-5 under non-correlated ground motions.

2.4.4.2 Correlated Suite of Ground Motions – Fragilities

For the fragility curves generated using the duration correlated motions, similar trends are observed. There are slight shifts of the fragility curves to the right indicating higher performance is predicted with reduced ground motion duration. In this case the isolation continues to control the response. The bearings observed fractions of reaching DS-3 along with fitted fragility curves are shown in Figure 2.17 reaching the median probability at scale factors of 1.52 and 2.57. Here the bearings mid-height piers 5, and 6 are set apart with a higher damage probability and having a different dispersion from other piers. The fragility curves for the overall bridge are shown in Figure 2.18, the bridge reached the median probability of DS-3 and DS-5 at scale factors of 1.59 and 1.90 respectively. The fragility curves obtained using the duration correlated records presents a slightly larger dispersion than the curves from the non-correlated records. Additional fragility curves are shown in the Appendix B.

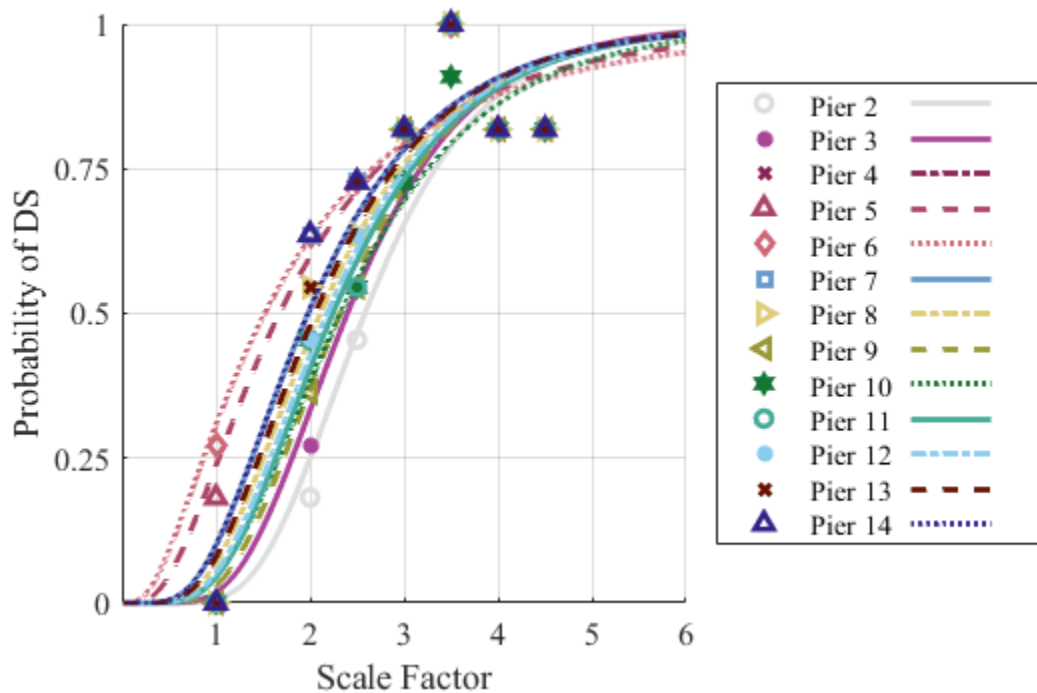


Figure 2.17 DS-3 probability for bearings on each isolated pier with fitted fragility curves under correlated ground motions.

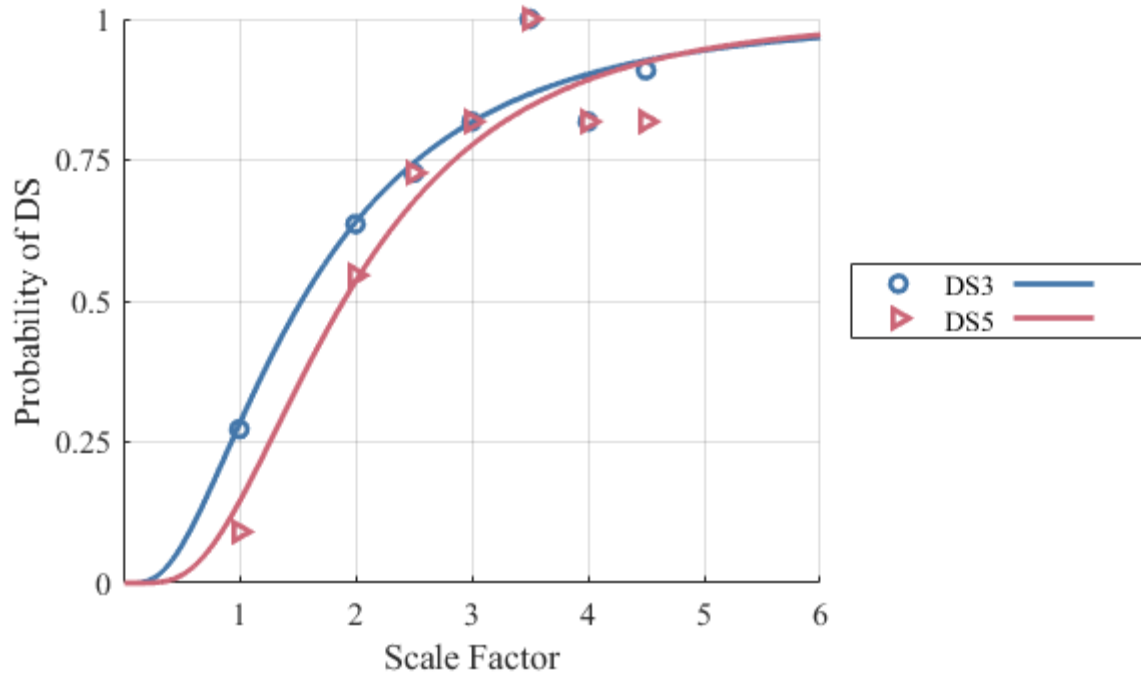


Figure 2.18 Major bridge damage probability with fitted fragility curves for DS-3 and DS-5 under correlated ground motions.

2.4.4.3 Major Bridge Performance Comparison

Comparing the fragility curves developed for both ground motion scaling methodologies, Figure 2.19 shows the fragility curves obtained with the correlated (C) and the non-correlated (NC) data sets for bearings on piers 3, 5 and 14 at DS-3. Here it is seen that the different hazard selection methodologies have different effects on each element. The bearings on Piers 3, 5, and 14, in Figure 2.19, show that under the non-correlated IDA analysis, the median probability of the bearings reaching DS-3 occurs at scale factors of 1.95, 1.09 and 1.42, respectively. However, the fragility curves considering the duration correlated ground motions reach a median DS-3 probability at scale factors of 2.40, 1.68 and 1.98, respectively. These indicate a shift of the median DS-3 fragility curves by 18.8%, 35.1%, and 28.3%. Similarly, Figure 2.20 shows the correlated (C) and the non-correlated (NC) sets for bearings on piers 3, 5 and 14 at DS- 5. The shift in the median DS-5 for bearings on pier 3, 5 and 14 is 9.0%, 38.9% and 20.7%, respectively.

The overall bridge fragility curves obtained from both methodologies are compared in DS-3 and DS-5 and shown in Figure 2.21. The overall performance compared in Figure 2.21 considering all the bridge piers and bearings shows a change in the median probability of 29.2% at DS-3 and 37.4% at DS-5 with changes in dispersion from the non-correlated to the correlated data set. These results show the median probabilities of reaching DS-3 and DS-5 in many cases are lower when the ground motion duration correlation is considered in the analysis.

The average number of cycles at set displacement ranges is calculated and averaged for all 11 ground motions on each suite at scale factors of 1.0, 2.5, and 4.0 to show the effect the duration correlation has on the number of cycles the bearings experienced. The averaged cycle count is presented in Figure 2.22 and Figure 2.23 showing the number of cycles observed in the non-correlated and the correlated cases. The figures show a reduction in the displacement cycles under the duration correlated records as the scale factor increases. This finding supports the original assumption that the number of loading cycles is reduced when ground motion duration correlation is accounted for. This reduction is not the same across elements. For instance, bearing 14 in Figure 2.23 show a larger difference in the number of cycles when compared to bearing 7 in Figure 2.22.

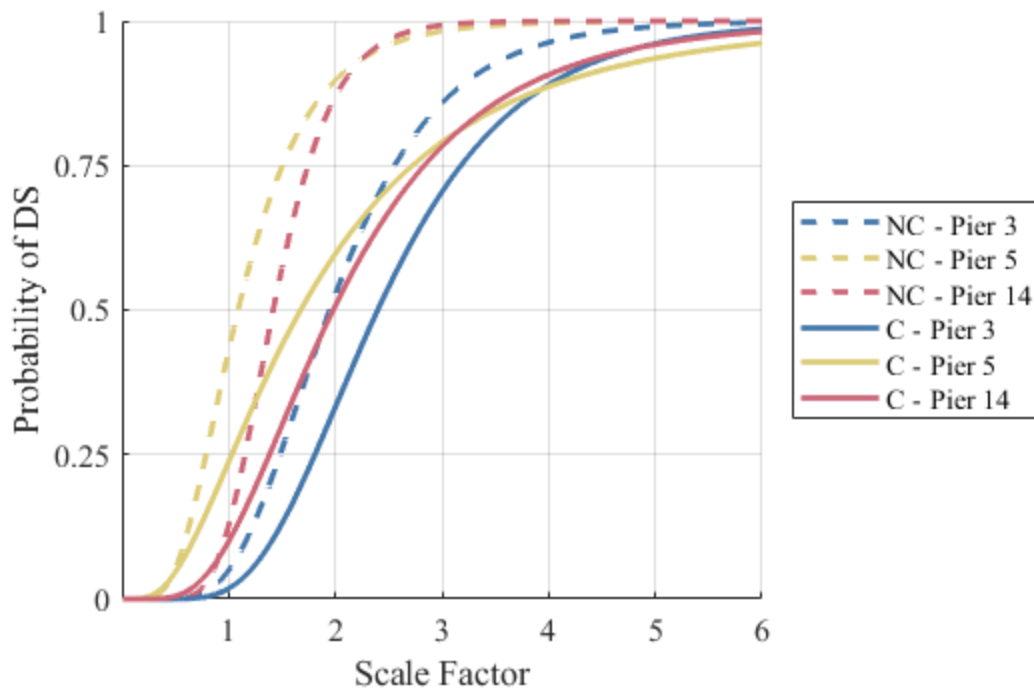


Figure 2.19 Major bridge bearings 3, 5, and 14 fragility curves comparison for DS-3.

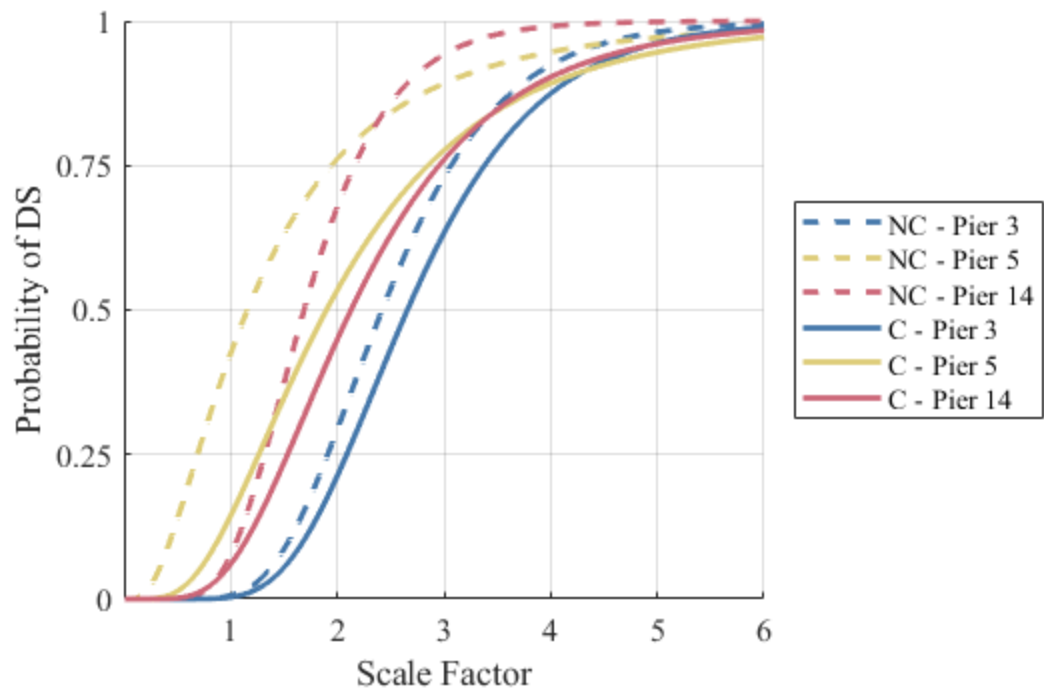


Figure 2.20 Major bridge bearings 3, 5, and 14 fragility curves comparison for DS-5.

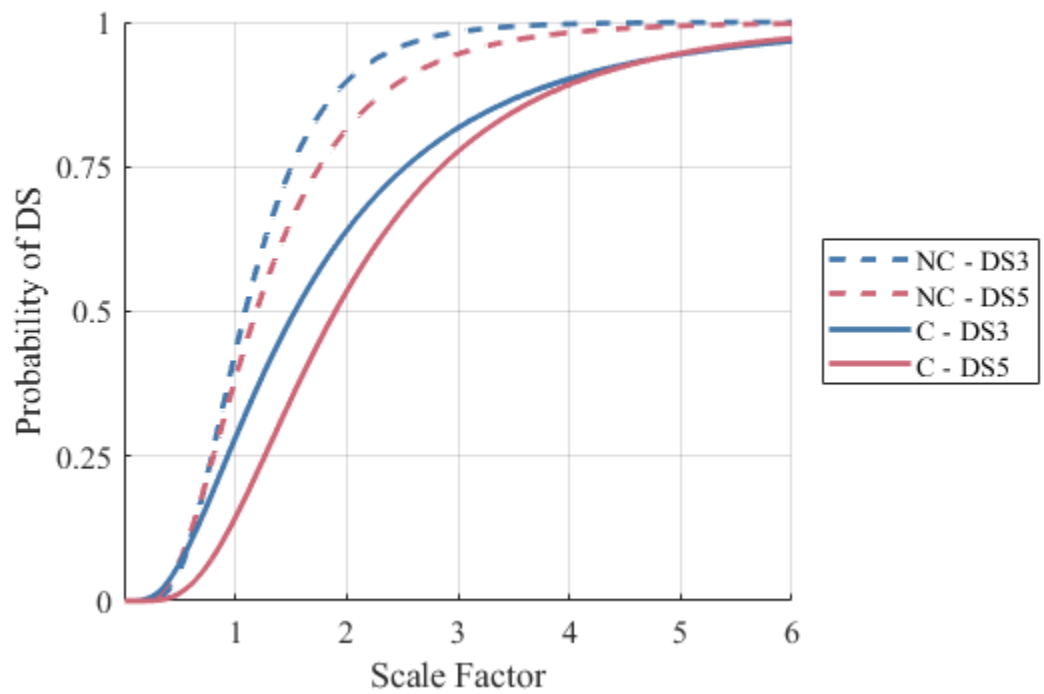


Figure 2.21 Major bridge fragility curves comparison.

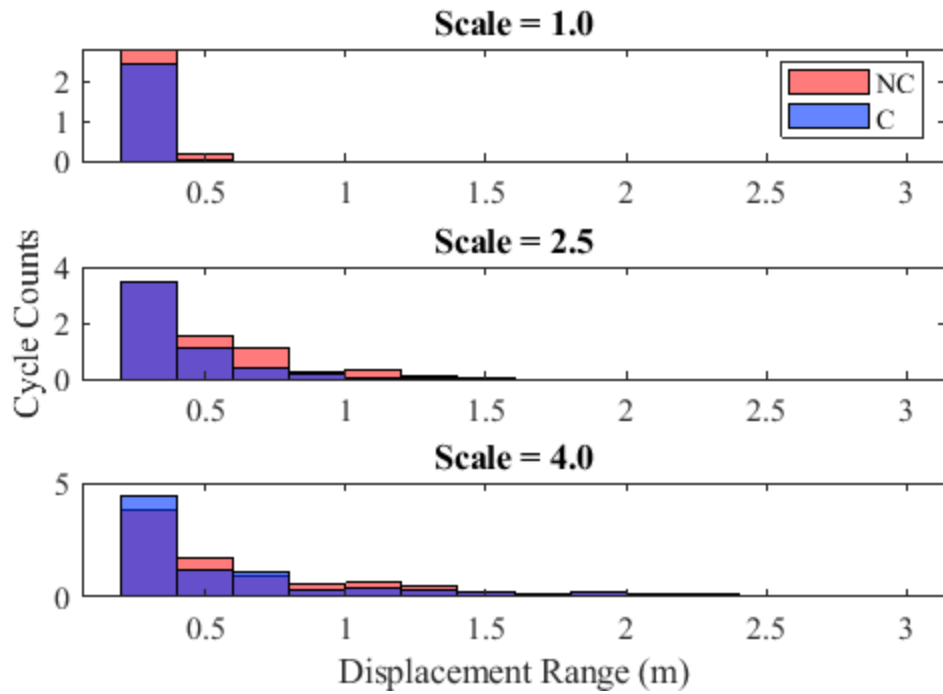


Figure 2.22 Comparison of average number of cycles for bearing 7 over suites of non-correlated (NC) and correlated (C) records at scale factors of 1.0, 2.5, and 4.0.

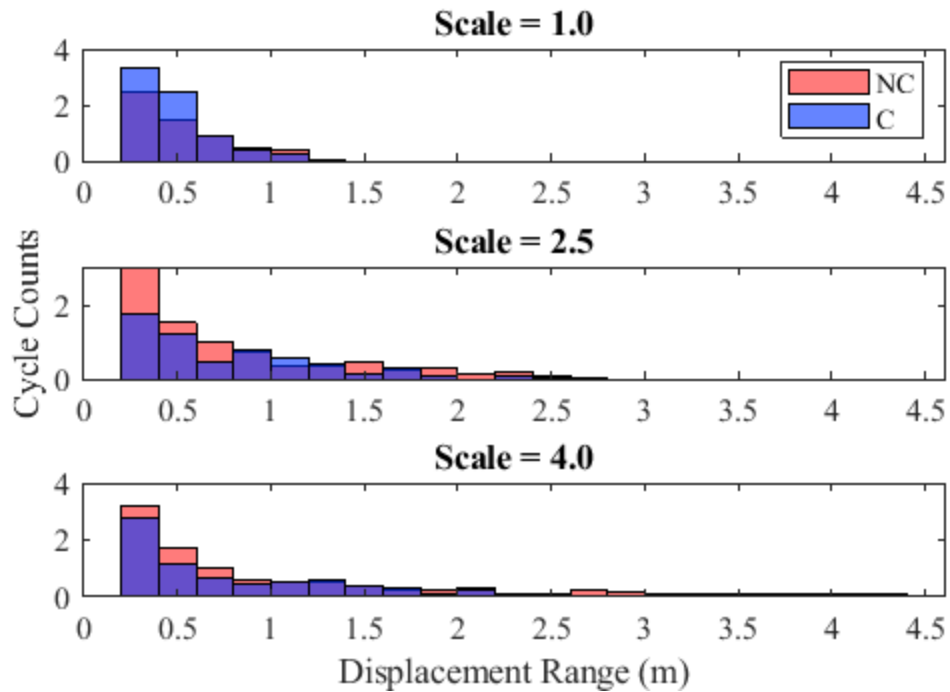


Figure 2.23 Comparison of average number of cycles for bearing 14 over suites of non-correlated (NC) and correlated (C) records at scale factors of 1.0, 2.5, and 4.0.

2.5 Typical Highway Overpass

For the second case study, a typical highway overpass was selected as these are commonly designed structures and, thus, any changes to understanding of their performance has wide ranging consequences. The overpass has a first period of roughly 0.5 seconds and is located in Northern California, shown in Figure 2.24. This second case study aims to capture different correlation changes due to the change in structural period, particularly for a short period bridge, while the pervious chapter explored a long period bridge. The same methodology as followed in Section 2.4 is used to examine the effect of including duration correlation in the performance analysis.

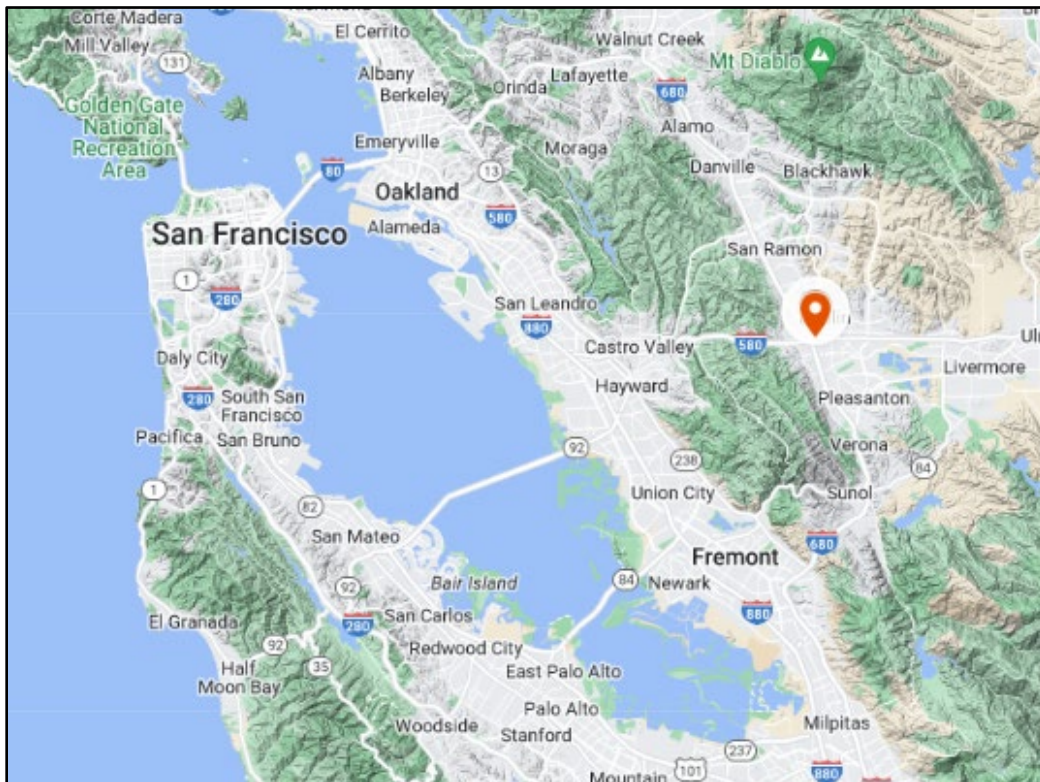


Figure 2.24 Major bridge location in Southern California showing curved span on plan view.⁵

2.5.1 Gound Motion Selection

2.5.1.1 Hazard Analysis

The UHS for a return period of 2,475 years (corresponding to a 2% probability of exceedance in 50 years) was developed using the method outlined in Section 2.3 and shown in Figure 2.25. Figure 2.26 are the design spectrum and the deaggregation of the PSHA results for a V_{S30} of 250 m/s for

⁵ Google Maps, 2023

the period of the bridge (0.35 seconds). The median magnitude and distance were found to be Mw 7.0 at 10 km. These values were used in the deterministic analysis to calculate the ε for the duration correlation analysis (mentioned later).

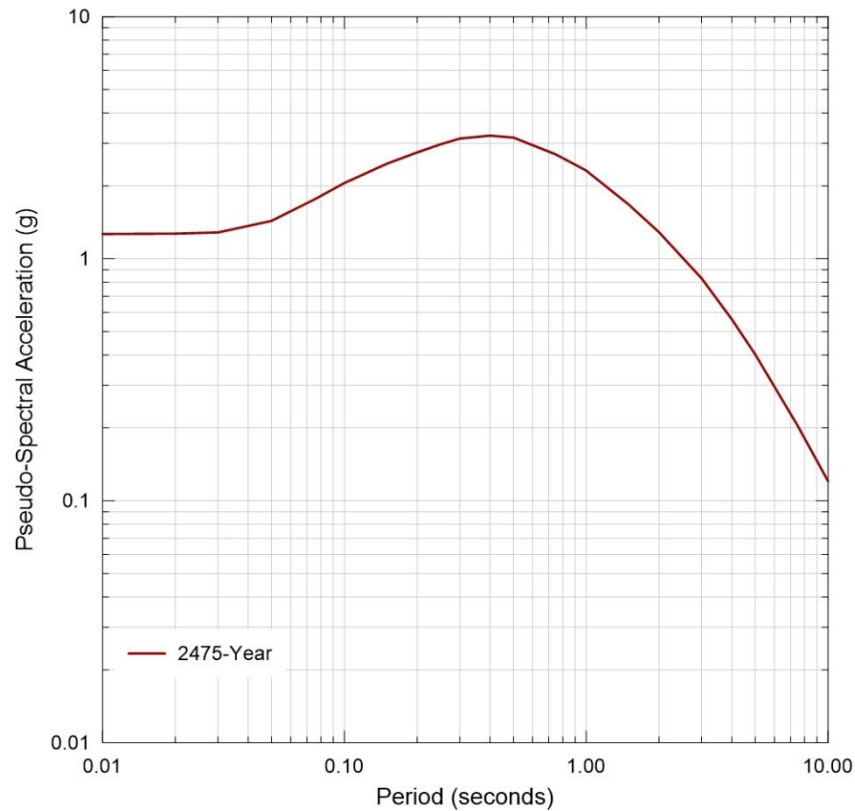


Figure 2.25. Uniform hazard spectra for a return period of 2,475 years (corresponding to a 2% probability of exceedance in 50 years, Maximum Considered Earthquake) for the highway overpass location.

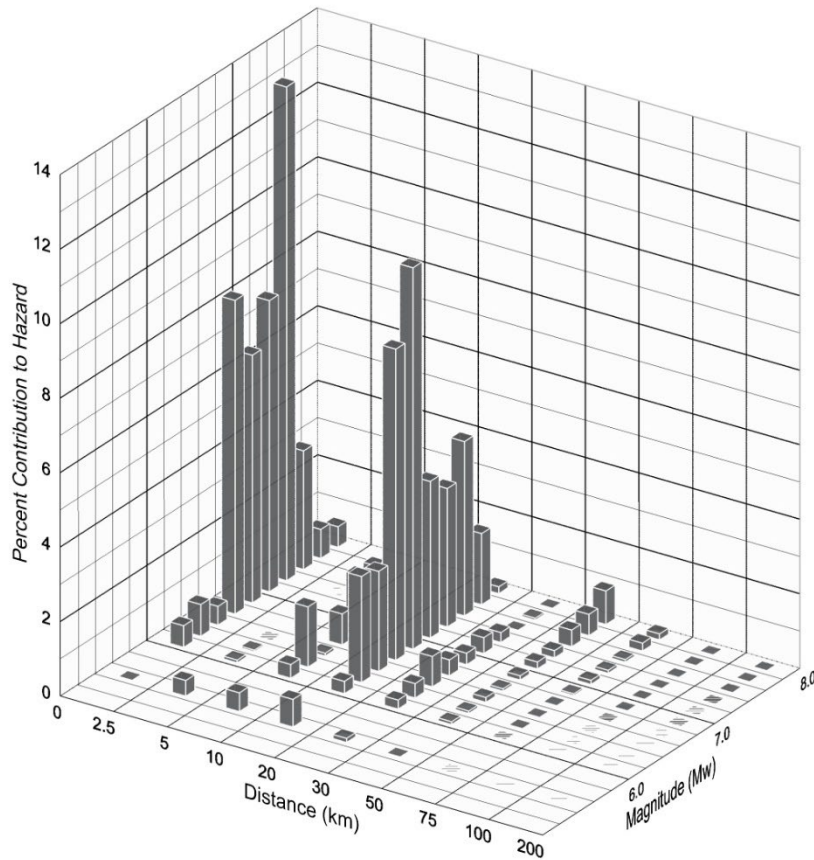


Figure 2.26. 2475-year return period hazard deaggregation at a period of 0.35 second showing near fault contribution.

2.5.1.2 General Selection of Seed Time Series

The specific earthquake source parameters that represent the dominant earthquake scenario in the seismic hazard were used in the selection of seed time series. Criteria used to sort, prioritize, and select seed time series for the bedrock analysis spectra from the PEER Center NGA-West2 Ground Motion Database included the following for all suites:

- Magnitude (M_w): 7.0 – 8.6;
- Rupture distance: 0 – 50 km;

2.5.1.3 Selection of Non-correlated Suite

The first suite was selected based on the mean duration values from the DPEs and the other parameters listed above. For the two sites the mean duration values from the DPEs are as follows:

- D₅₋₇₅: 7.3 seconds
- D₅₋₉₅: 15.51 seconds

The suite using these duration bounds is shown in Table 2.18.

Table 2.18. Non-correlated suite of time histories

RSN	Earthquake Name	Station Name	Mw	D _{5-75%} (sec)	D _{5-95%} (sec)
728	Superstition Hills-02	Westmorland Fire Sta	6.5	11.9	23.5
806	Loma Prieta	Sunnyvale - Colton Ave.	6.9	9.9	25.3
827	Cape Mendocino	Fortuna - Fortuna Blvd	7.0	11.3	18.7
900	Landers	Yermo Fire Station	7.3	10.9	18.9
1193	Chi-Chi, Taiwan	CHY024	7.6	13.8	27.0
1244	Chi-Chi, Taiwan	CHY101	7.6	13.5	30.4
2114	Denali, Alaska	TAPS Pump Station #10	7.9	9.1	29.5
4859	Chuetsu-oki, Japan	Mitsuke Kazuiti Arita Town	6.8	17.8	29.4
6911	Darfield, New Zealand	HORC	7.0	7.0	9.5
6953	Darfield, New Zealand	Pages Road Pumping Station	7.0	11.2	22.0
8606	El Mayor-Cucapah, Mexico	Westside Elementary School	7.2	10.1	25.3

2.5.1.4 Selection of Correlated Suites

ε is calculated using the deaggregation results from the PSHA to calculate a deterministic median pseudo-spectral acceleration and standard deviation using the four NGA-West2 GMMs (shown in Figure 2.27). ε is depicted between the median and 84th percentile deterministic spectra. The UHS (and scaled UHS) are compared to this deterministic scenario to calculate the number of standard deviations from the median, to get the actual ε .

To develop the fragility curves scale factors of 2, 2.5, 3, 3.5, 4, and 4.5 were applied to the UHS, the correlated D_{5-75%} and D_{5-95%} were calculated, and additional time histories selected. The values of ε used for the correlated duration can be found in Table 2.19 and the correlated duration values are in Table 2.20. The time histories for each of the scale factors and sites are outlined in Table 2.21 through Table 2.26. The main focus of this selection process was the duration values, so the magnitudes and other parameters of the suites are not as close to the deaggregation results. The difference between the mean duration used for the uncorrelated suite and the mean durations used for the correlated suite increases as scale factor increases.

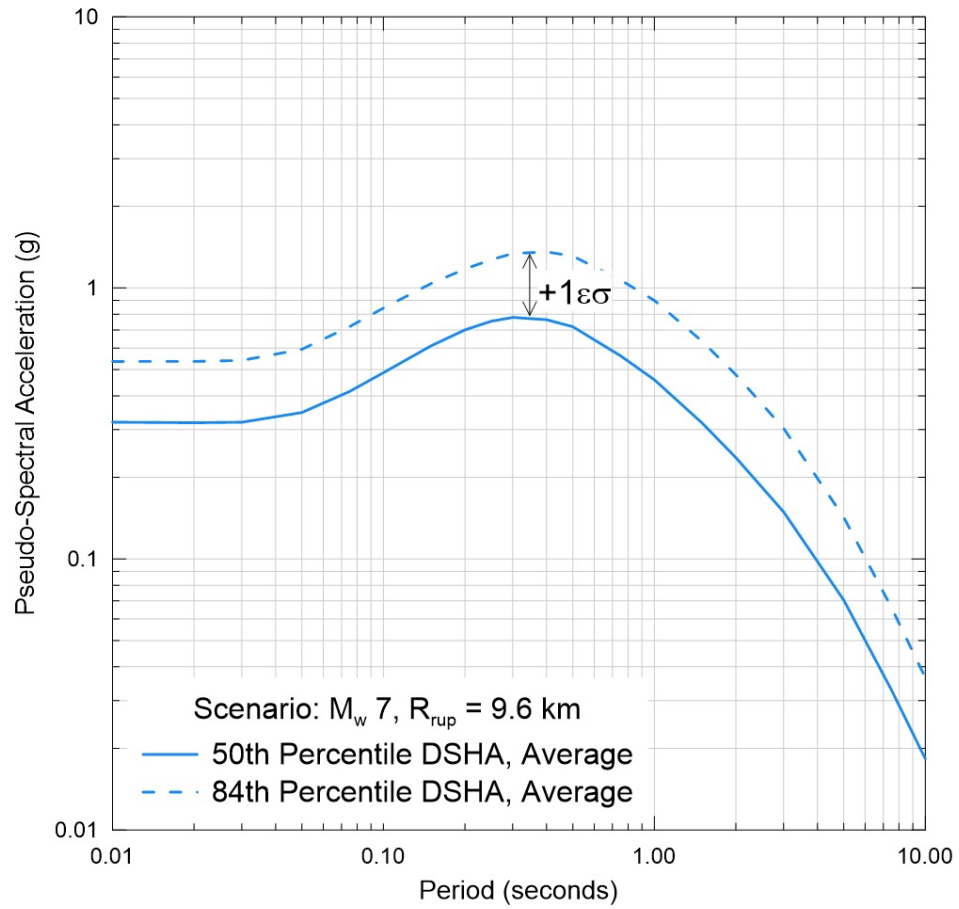


Figure 2.27. Deterministic spectra and depiction of epsilon for the highway overpass

Table 2.19. ϵ values (number of standard deviations from deterministic median ground motion) for the Highway Overpass (0.35 sec)

Scale Factor	ϵ
1	2.53
2	3.77
2.5	4.17
3	4.50
3.5	4.77
4	5.01
4.5	5.22

Table 2.20 Correlated Duration values for the Highway Overpass Site

Highway Overpass		
Scale Factor	D_{5-75%} (sec)	D_{5-95%} (sec)
1	5.02	12.14
2	4.20	10.80
2.5	3.96	10.40
3	3.78	10.09
3.5	3.64	9.83
4	3.52	9.62
4.5	3.41	9.43

Table 2.21 Suite of time histories for the UHS scale factor of 2, Highway Overpass

RSN	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
778	Loma Prieta	Hollister Differential Array	6.9	4.2	13.3
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
1116	Kobe_Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe_Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli_Turkey	Duzce	7.5	6.1	11.8
2650	Chi-Chi_Taiwan-03	TCU116	6.2	4.9	12.3
3748	Cape Mendocino	Ferndale Fire Station	7.0	6.2	13.6
4894	Chuetsu-oki_Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8123	Christchurch_New Zealand	Christchurch Resthaven	6.2	4.4	11.2

Table 2.22 Suite of time histories for the UHS scale factor of 2.5, Highway Overpass

RSN	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
173	Imperial Valley-06	El Centro Array #10	6.5	5.3	12.8
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
982	Northridge-01	Jensen Filter Plant Administrative Building	6.7	4.0	12.5
1116	Kobe_ Japan	Shin-Osaka	6.9	4.5	11.6
1120	Kobe_ Japan	Takatori	6.9	6.0	11.3
1158	Kocaeli_ Turkey	Duzce	7.5	6.1	11.8
1197	Chi-Chi_ Taiwan	CHY028	7.6	4.8	8.7
2650	Chi-Chi_ Taiwan-03	TCU116	6.2	4.9	12.3
4894	Chuetsu-oki_ Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8	6.0	10.4
8123	Christchurch_ New Zealand	Christchurch Resthaven	6.2	4.4	11.2

Table 2.23 Suite of time histories for the UHS scale factor of 3, Highway Overpass

RSN	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
778	Loma Prieta	Hollister Differential Array	6.9	4.2	13.3
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
982	Northridge-01	Jensen Filter Plant Administrative Building	6.7	4.0	12.5
1063	Northridge-01	Rinaldi Receiving Sta	6.7	4.3	9.1
1116	Kobe_ Japan	Shin-Osaka	6.9	4.5	11.6
1158	Kocaeli_ Turkey	Duzce	7.5	6.1	11.8
1197	Chi-Chi_ Taiwan	CHY028	7.6	4.8	8.7
2650	Chi-Chi_ Taiwan-03	TCU116	6.2	4.9	12.3
3746	Cape Mendocino	Centerville Beach_ Naval Fac	7.0	4.6	10.6
4115	Parkfield-02_ CA	Parkfield - Fault Zone 12	6.0	4.0	9.7

Table 2.24 Suite of time histories for the UHS scale factor of 3.5, Highway Overpass

Record Number	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
982	Northridge-01	Jensen Filter Plant Administrative Building	6.7	4.0	12.5
1063	Northridge-01	Rinaldi Receiving Sta	6.7	4.3	9.1
1116	Kobe_ Japan	Shin-Osaka	6.9	4.5	11.6
1197	Chi-Chi_ Taiwan	CHY028	7.6	4.8	8.7
2650	Chi-Chi_ Taiwan-03	TCU116	6.2	4.9	12.3
3746	Cape Mendocino	Centerville Beach_ Naval Fac	7.0	4.6	10.6
3943	Tottori_ Japan	SMN015	6.6	3.7	9.9
4115	Parkfield-02_ CA	Parkfield - Fault Zone 12	6.0	4.0	9.7
8123	Christchurch_ New Zealand	Christchurch Resthaven	6.2	4.4	11.2

Table 2.25 Suite of time histories for the UHS scale factor of 4, Highway Overpass

Record Number	Earthquake Name	Station Name	Mw	D_{5-75%} (sec)	D_{5-95%} (sec)
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
982	Northridge-01	Jensen Filter Plant Administrative Building	6.7	4.0	12.5
1063	Northridge-01	Rinaldi Receiving Sta	6.7	4.3	9.1
2650	Chi-Chi_ Taiwan-03	TCU116	6.2	4.9	12.3
2655	Chi-Chi_ Taiwan-03	TCU122	6.2	3.3	9.0
3746	Cape Mendocino	Centerville Beach_ Naval Fac	7.0	4.6	10.6
3943	Tottori_ Japan	SMN015	6.6	3.7	9.9
4115	Parkfield-02_ CA	Parkfield - Fault Zone 12	6.0	4.0	9.7
4131	Parkfield-02_ CA	Parkfield - Vineyard Cany 1W	6.0	4.3	8.7
8123	Christchurch_ New Zealand	Christchurch Resthaven	6.2	4.4	11.2

Table 2.26 Suite of time histories for the UHS scale factor of 4.5, Highway Overpass

Record Number	Earthquake Name	Station Name	Mw	D _{5-75%} (sec)	D _{5-95%} (sec)
181	Imperial Valley-06	El Centro Array #6	6.5	5.3	11.5
802	Loma Prieta	Saratoga - Aloha Ave	6.9	4.1	9.4
803	Loma Prieta	Saratoga - W Valley Coll.	6.9	4.1	11.1
982	Northridge-01	Jensen Filter Plant Administrative Building	6.7	4.0	12.5
1085	Northridge-01	Sylmar - Converter Sta East	6.7	3.8	7.4
2655	Chi-Chi_ Taiwan-03	TCU122	6.2	3.3	9.0
3744	Cape Mendocino	Bunker Hill FAA	7.0	3.6	13.4
3746	Cape Mendocino	Centerville Beach_ Naval Fac	7.0	4.6	10.6
3943	Tottori_ Japan	SMN015	6.6	3.7	9.9
4115	Parkfield-02_ CA	Parkfield - Fault Zone 12	6.0	4.0	9.7
4131	Parkfield-02_ CA	Parkfield - Vineyard Cany 1W	6.0	4.3	8.7

2.5.2 Structural Analysis

The typical highway bridge model is based on a typical Caltrans bridge located in Northern California. The bridge has a single concrete pier and a concrete box girder deck of two spans supported by concrete seat abutments. A schematic of the bridge, reflecting the numerical modeling layout in OpenSees, is shown in Figure 2.28. The deck is modeled using an elastic-beam column element and the pier is modeled using a beam with hinges element to capture the nonlinear behavior of the pier. A cross-section of the pier is presented in Figure 2.29. The concrete and steel reinforcement materials are modeled using Concrete02 and Steel02 to capture material degradation. The concrete compressive strength is 4 ksi, while the steel expected yield strength is defined as 69 ksi. A fiber section including core confinement, unconfined cover and longitudinal steel bars is defined for the plastic hinge section of the pier. The length of the plastic hinge, L_p , is calculated using Equation (2.7) proposed by Priestley and Park (1987).

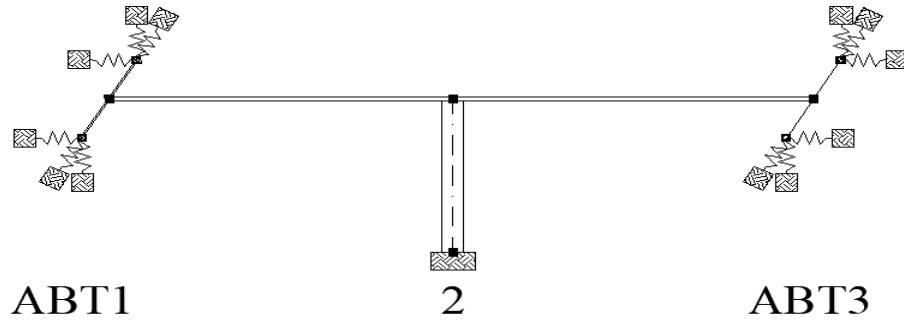


Figure 2.28 Typical highway bridge damage model

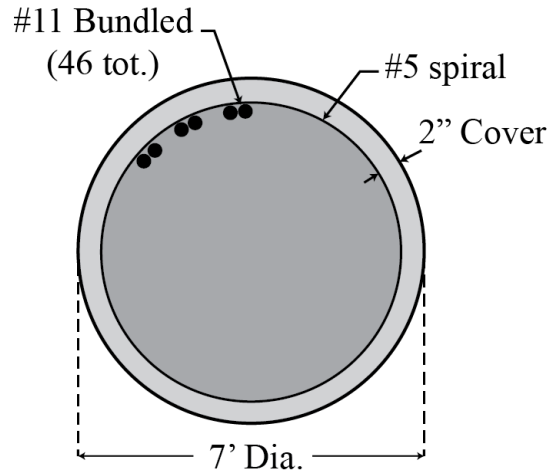


Figure 2.29 Typical highway bridge pier cross-section.

The abutments are modeled using a simplified abutment model as described in Caltrans (2019) Section 6. The abutment backwall is modeled with a rigid beam perpendicular to the span, supported by three springs at each end of the beam. The longitudinal behavior is modeled using two materials, a compression-only springs in parallel with an elastic perfectly-plastic gap spring and an elastic spring based on the bearings, combined in parallel. The transverse behavior of the abutments is modeled using an elastic-perfectly plastic material by modifying the longitudinal stiffness and load capacity using wall effectiveness and participation coefficients, $C_L = 2/3$, and $C_W = 4/3$ (Maroney and Chai, 1994). The vertical behavior is modeled with a compression-only elastic spring and a very large stiffness. The abutment modeling parameters are calculated following Caltrans (2019) Section 6 and equations are replicated here for reference. The stiffness and yielding force of the longitudinal spring, K_{abl} and P_{abl} , is calculated using Equation 5.1 and 4.2, where w_{bw} and h_{bw} are the width and the height of the abutment backwall, and R_{sk} is the skew reduction factor due to the skew angle, θ . The Transverse stiffness and yielding force, K_{abt}

and P_{abt} , are calculated using Equation 5.4 and 5.5. Table 2.27 shows the summary of model parameters and a schematic of the abutment springs is presented in Figure 2.30 for clarity.

$$K_{abl} = w_{bw} (5.5h_{bw} + 20)R_{sk} \quad (2.11)$$

$$P_{abl} = w_{bw} \left(\frac{5.5h_{bw}^{2.5}}{1+2.37h_{bw}} \right) R_{sk} \quad (2.12)$$

$$R_{sk} = e^{\frac{-\theta}{45}} \quad (2.13)$$

$$K_{abt} = 1 / 3 \times C_L \times C_W \times K_{abl} \quad (2.14)$$

$$P_{abt} = 1 / 3 \times C_L \times C_W \times P_{abl} \quad (2.15)$$

Table 2.27 Typical Highway Bridge Properties.

Bearing Property	
Backwall width, w_{bw} (ft)	47.3
Backwall Length, h_{bw} (ft)	5.5
skew angle, θ (deg)	25
Skew reduction factor, R_{sk}	0.64
Longitudinal bearing stiffness, K_{ab_brg} (kip/in)	57.6
Longitudinal gap, Δ_{gap} (in)	1.5
Longitudinal stiffness, K_{abl} (kip/in)	131,500
Longitudinal yield force, P_{abl} (kip)	425,200
Transverse stiffness, K_{abt} (kip/in)	39,030
Transverse yield force, P_{abt} (kip)	126,000
Plastic hinge length, L_p (in)	20

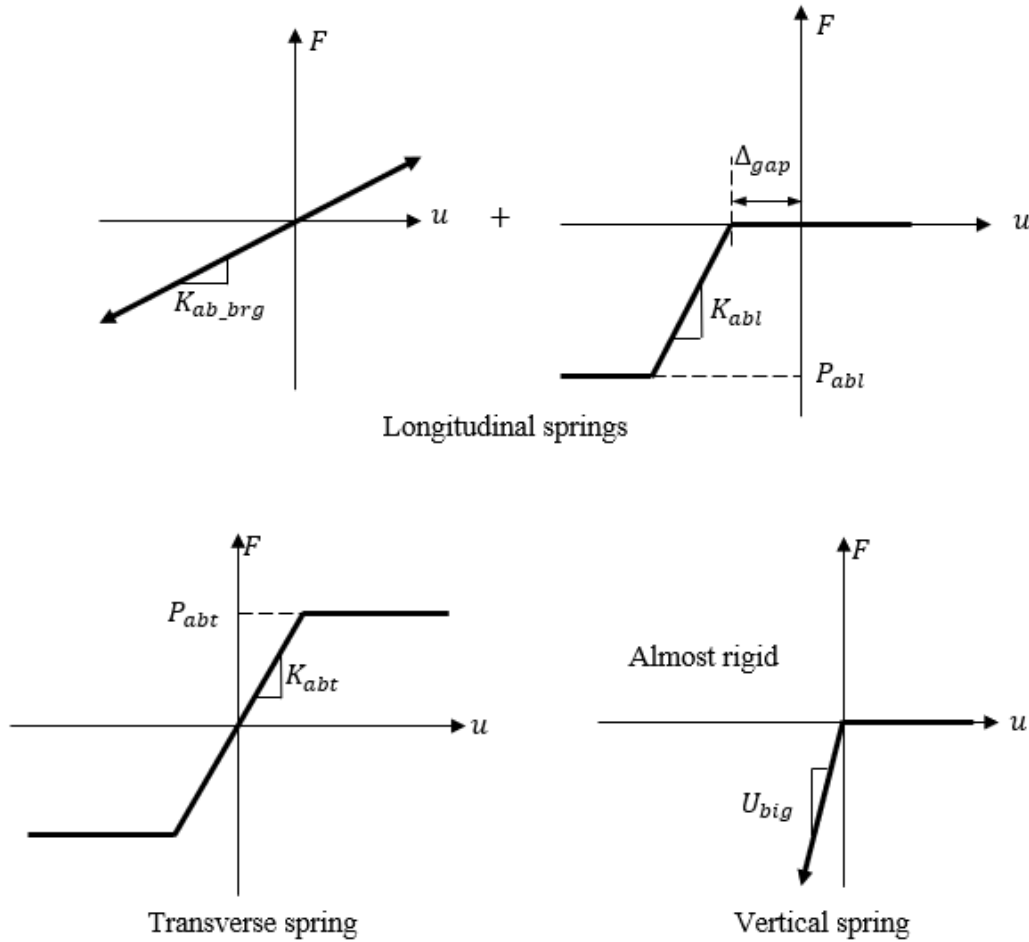


Figure 2.30 Typical highway bridge – Abutment model springs.

2.5.3 Typical highway bridge performance

The performance of the typical highway overpass is evaluated by the same process described in Chapter 2.4.3 by calculating the damage probability for pier 1. The overall bridge performance for the typical highway bridge focuses solely on the pier drift for the purpose of comparing both ground motion selection methodologies. The typical bridge pushover curve of the overpass bridge, with the occurrence of damage states is shown in Figure 2.31, showing the pier strength degradation with increasing drift. The observed fractions of damage states resulting from the numerical analysis of both ground motion sets are presented along with the fitted fragility curve for the bridge pier. Figure 2.32 shows the fragility curves resulting from the non-correlated data set while Figure 2.33 shows the pier fragility curves resulting from the correlated ground motions. In the case of the correlated ground motions, there is high number of observations of DS-3 even at a scale of 1.0, as such there is not an adequate spread of data to generate a fragility curve, and the curve shown should not be used to derive statistics of performance.

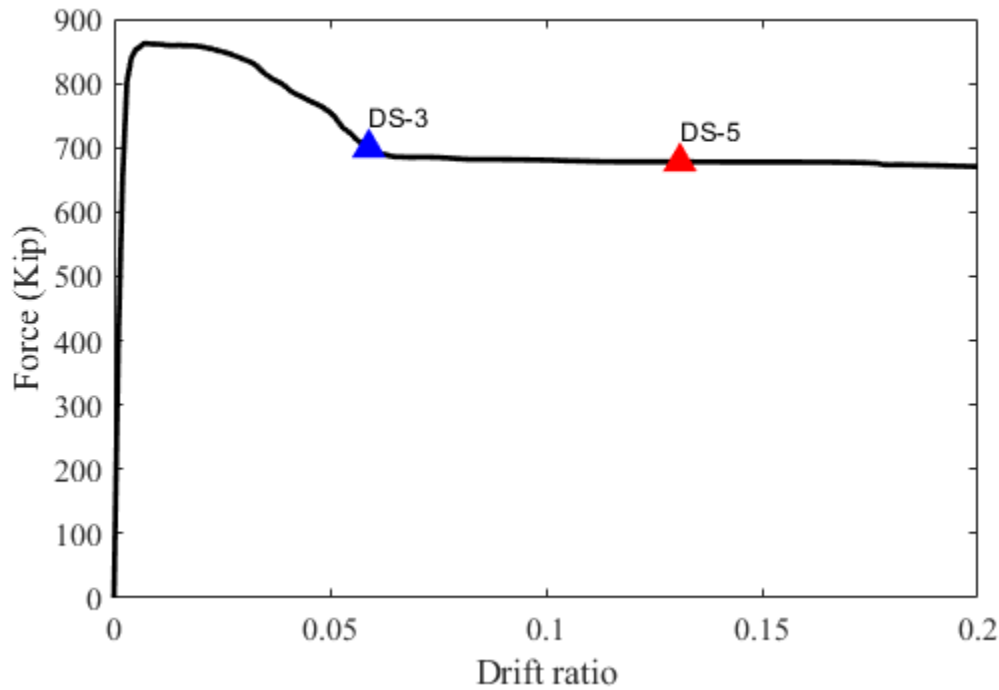


Figure 2.31 Typical highway bridge pushover curve showing DS-3 and DS-5.

A comparison of the fragility curves developed for both ground motion sets is presented in Figure 2.34. For DS-5, the analysis considering the duration correlated ground motions leads to reaching the median damage probability at a scale factor of 1.68 as compared to a scale factor of 1.39 for the non-correlated IDA analysis. This shift is a 17.3% increase in the intensity expected to reach the median DS-5 probability and shows that considering ground motion duration correlation in the scaling reduces the damage expected in the structure for the typical highway overpass. For DS-3, however, due to the lack of data to get an accurate dispersion of the correlated set no significant changes to the median probability of collapse can be observed. The average number of cycles is calculated over the non-correlated and correlated suites at scale factors of 1.0, 2.0, and 3.0 and presented in Figure 2.35. Here there is a clear reduction in the number of cycles the pier experiences with increasing scale factor. As such, the reduction in damage probability from the non-correlated to the correlated data set is attributed to this reduction in the number of cycles the structure experiences.

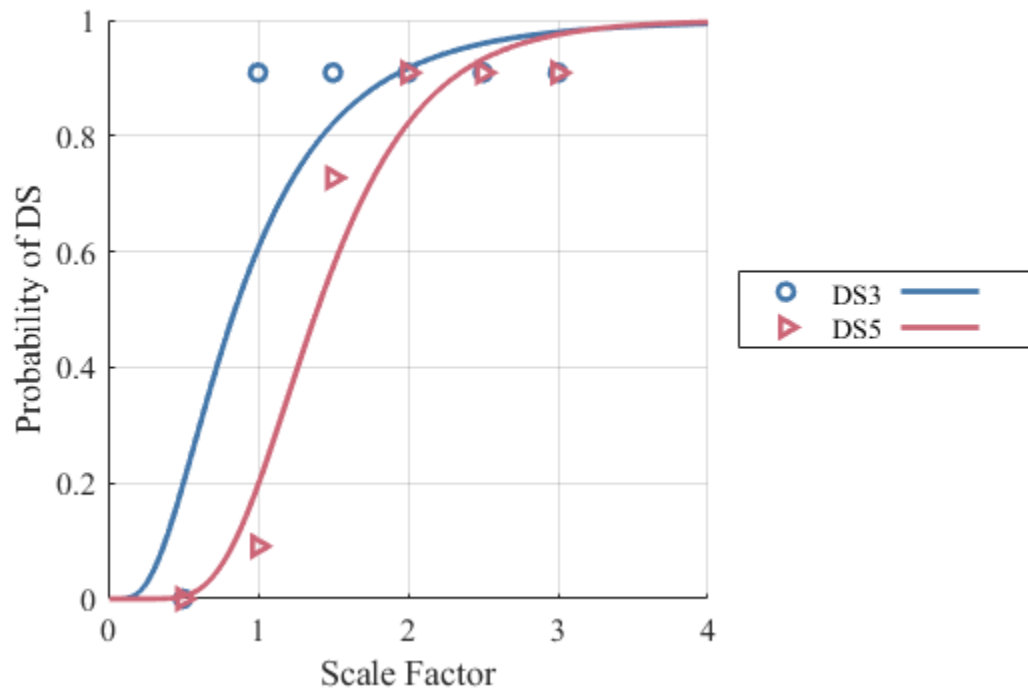


Figure 2.32 Typical bridge, DS-3 and DS-5 probability with fitted fragility curves under non-correlated ground motions.

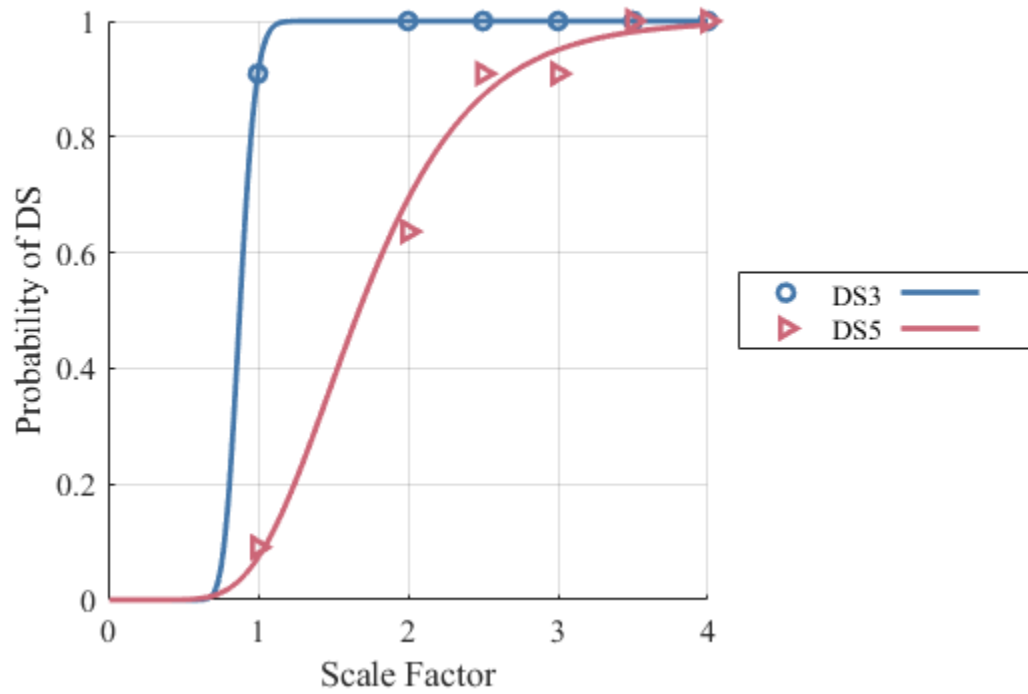


Figure 2.33 Typical bridge, DS-3 and DS-5 probability with fitted fragility curves for under correlated ground motions.

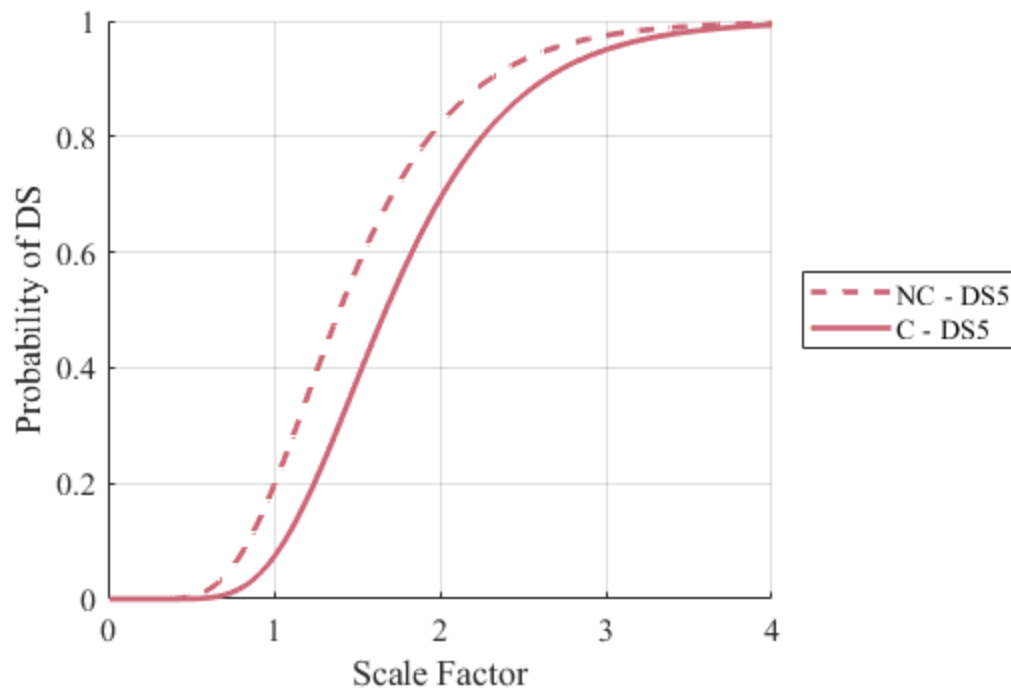


Figure 2.34 Typical bridge damage fragility curves for DS-5 under non-correlated and correlated ground motions.

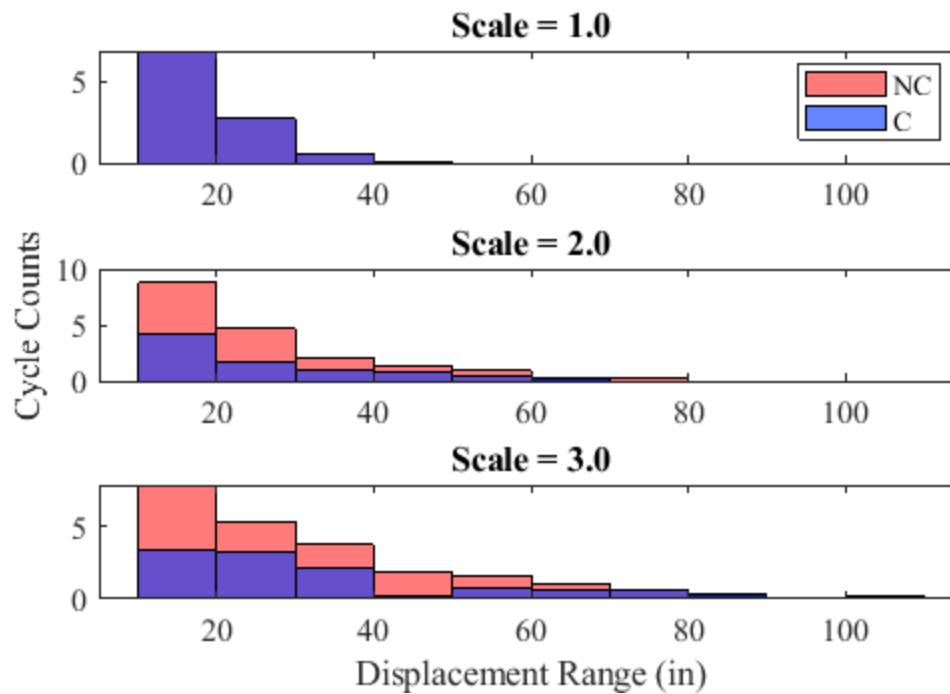


Figure 2.35 Comparison of average number of cycles under non-correlated (NC) and correlated (C) records at scale factors of 1.0, 2.0, 3.0.

2.6 Conclusions

This study focuses on developing fragility curves, with and without considering duration correlation, for a major bridge with a period of 1.6 seconds and a typical overpass with a period of 0.5 seconds. The first objective is to determine if considering duration correlation in ground motion selection appreciably changes the hazard input for analysis. Given this hazard change, the second goal is to evaluate changes in the fragility damage curves for the two bridges.

Ground motion duration is negatively correlated to intensity in the hazard analysis. The duration correlation is largest at shorter structural periods and decreases with increasing structural period. Multiple suites of ground motions were selected both with and without accounting for duration correlated to spectral acceleration. The first suite was chosen using magnitude, distance, other secondary earthquake parameters, and non-correlated durations and then scaled directly with values of 2, 2.5, 3, 3.5, 4, and 4.5. The second set of ground motions was formed by selecting six additional suites based on scaled hazard spectra considering the duration correlation. The bridge performance was compared using the fragility curves resulting from these sets of ground motions to assess the effect of correlation coefficients on the probability of reaching DS-3 and DS-5 for a major bridge and a typical highway overpass located in Southern California and in Northern California.

The performance of the bridges was evaluated by comparing the fragility curves of the model subject to the non-correlated and the duration correlated ground motions. The major bridge resulted in the median probability of reaching DS-3 and DS-5 at intensity values 29.2% and 37.4% larger when subject to the correlated suite of ground motions than those with the non-correlated suite. The overall bridge performance is based on a single worst-case component. However, the figures in Section 2.4.4.3 show that other individual components follow the same trend and generally have lower damage probability when duration correlation is accounted in the hazard selection process. The performance of the typical highway bridge was evaluated by comparing the fragility curves for the model subject to the non-correlated suite of ground motion and the duration correlated ground motions. From the fragility curves, the median probability of DS-5 using the correlated ground motions occurs at a 17.3% higher scale factor than traditional IDA. This shift in the median probability shows that the non-correlated traditional IDA leads to an underestimation of the bridge performance at the major damage state.

The difference in the duration correlation for both bridges can be observed by comparing the correlated duration values in Table 2.6 and Table 2.20. The major bridge has a small reduction in duration with each increasing scale, due to a large fundamental period, with a -0.66 seconds difference in D_{5-95} from a scale of 1.0 to scale of 4.5. The typical highway overpass, however, has a larger reduction of -2.71 seconds in D_{5-95} from a scale of 1.0 to a scale of 4.5. The effect of this difference in duration correlation can be directly observed in the structural response by comparing Figure 2.22 and Figure 2.23 of the major bridge with Figure 2.35 of the typical highway bridge. The difference in the number of cycles is significantly larger in the case of the typical highway overpass.

The case studies of both the major bridge and the typical highway overpass indicate a reduction in the damage probability when considering duration correlations. This suggests IDA without considering duration correlation underestimates the performance of both structural systems. A summary of the effects of both case studies is summarized in Table 2.28. Both sites have similar seismicity and soil type but differ in period and structural behavior. While the longer period of the bridge leads to smaller changes in duration when considering the correlation, it has lead rubber bearings which have strength deterioration with larger number of cycles. This results in both bridges having a change in performance. The typical highway overpass is influenced by the large duration correlation at its period while the major bridge with even a small correlation is heavily influenced by its structural behavior. This study shows that, while the magnitude of effects of incorporating ground motion duration correlation is influenced by both period and structural response, it is nontrivial in location with large ground motions.

Table 2.28 Increase in median DS intensity measure when duration correlation is considered for both case study bridges.

Damage state	Major bridge	Typical bridge
DS-3	29.2%	-
DS-5	37.4%	17.3%

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3. Bridge Performance Considering Bearing Rotation

This chapter is based on Lopez Ruiz, M.C., Cheng, M., Becker, T.C. (2024) (In preparation)

3.1 Abstract

Most existing numerical models for isolation bearings assume parallel end-plate conditions, neglecting bearing rotations. This assumption is not necessarily true in the presence of flexible framing members such as tall piers in bridges which can lead to rotational demands. There is limited understanding of bearing rotation behavior and coupling between shear and rotation; however, experimental testing of the interaction of the bearings with flexible framing in a large structural system is often cost-prohibitive. Thus, a comprehensive experimental study was undertaken using large-scale hybrid simulation. The study focuses on a multi-span bridge, incorporating seismic isolation lead rubber bearings atop tall piers. A one-third-scale lead rubber bearing was used as the experimental element. The remaining components of the bridge, including bearings on twelve other piers, were numerically modeled. Real-time model updating was employed during testing to ensure an accurate representation of the shear and rotational stiffness of the remaining bearings. The study addresses the challenges of conducting a large-scale hybrid simulation with loading in three degrees of freedom and model updating. The bearings show a reduction in the effective shear stiffness properties when rotation is present. However, the changes in the bearing properties result in minimal changes in the system-level bridge performance.

3.2 Introduction

Isolation bearings are commonly used in bridges in regions with mid to high seismicity. Their local behavior and its effect on bridge performance are well understood under typical design applications and loading. While typical installations of rubber bearings maintain nominally parallel top and bottom end plates, some applications, such as bridges with tall piers or buildings with column-top isolation, may violate this typical assumption. Potential rotational demands from flexible piers introduced in the bearings are shown exaggerated in Figure 3.1. Through analytical studies, Mitoulis (2015) found that rubber bearings can have nonnegligible rotational demands due to pier cap rotations and Xu et al. (2021) showed that considering permanent initial rotation led to increased horizontal bearing displacements in a truss bridge.

Rotational behavior and its coupling with shear, is not well understood for rubber bearings. Stanton (2008) conducted studies on rubber bridge bearings, finding that a linear estimation of rotational stiffness is sufficient for low rotations expected in typical conditions. Studies on seismic isolation bearings include He et al. (2012), Ishii et al. (2017), and Moghadam and Konstantinidis (2021), which all investigated the moment-rotational behavior by applying cyclic loading at pre-applied constant shear strains or rotations. They found that bearing rotational stiffness increases

with increasing vertical load but decreases with increasing shear strain, and there is a non-linear moment-rotation relationship at large rotations. Crowder and Becker (2017) and Darlington and Becker (2021) tested rubber bearings with combined simultaneous translation and rotation. The tests found that rotation of the bearing end-plates leads to a reduction in the horizontal stiffness of isolation bearings. While these tests applied demands closer to those seen in the field, they were cyclical, and rotation and translation were applied exclusively in-phase.

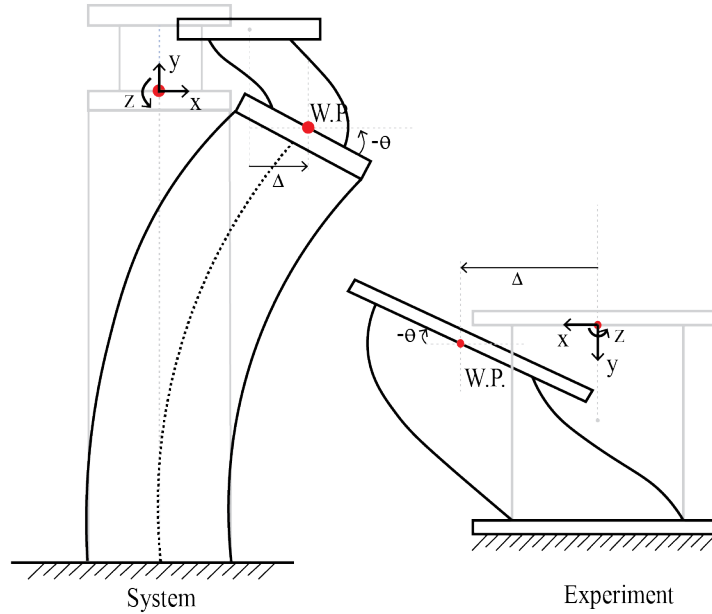


Figure 3.1. Bearing rotation shear and rotation demands due to flexible pier showing the work point (W.P.) in system and experimental setup.

While multiple tests on bridges with sliding isolation have been conducted (e.g. Mosqueda et al. (2004), Marin-Artieda et al. (2009)), system-level experiments of bridges isolated with rubber bearings are limited. Monzon et al. (2016) tested a large-scale lead-rubber bearing isolated bridge with a curved span and showed that in-plane deck rotation due to the curved girder induces increased shear demands in the isolators. However, no experimental tests have explored seismic input representing combinations of both shear and rotational demands that would arise from flexible framing bounding the bearing.

To investigate (1) the change in the behavior of the rubber bearings when placed on tall, flexible piers and (2) any potential resulting changes in the behavior of the bridge, the system-level performance is best evaluated. As such, a hybrid simulation with model updating was selected for the test. Hybrid simulation is a testing methodology that combines numerically modeled and physically tested subassemblies of a structure (Nakashima et al., 1992). Hybrid simulation has been used to explore the behavior of isolated structures assuming parallel end-plate conditions, for example midlevel-isolated buildings with shake table testing (Schellenberg et al., 2017). Sarebanha et al. (2019) performed a hybrid simulation test of a nuclear power plant by testing a single full-scale bearing to represent all the isolators or a portion of the isolators. They

found that the choice of what the physically tested bearing represents (e.g. a single bearing, a group of bearings, or all bearings in the isolation system) can influence the overall prediction of structural behavior. Model updating (Kwon and Kammula, 2013), which applies the instantaneous model parameters identified from the physical specimen to update numerically simulated elements with the same design, provides a solution to this issue.

Several researchers have conducted hybrid simulations with model updating at small-scale for various systems, for example a one-bay frame updating rotational spring parameters of one column (Hashemi et al., 2014), a five-story steel frame with the first story tested experimentally and used to update the upper stories (Ou et al., 2020), and a one-bay frame with updating of a nonlinear model of the frame (Song and Dyke, 2014). For larger-scale experiments, most controlled and updated a single degree-of-freedom (DOF) (Shao et al., 2016, Mei et al., 2018). There is a lack of larger-scale hybrid simulation experiments with model updating that incorporate multiple DOFs with complex element behavior that is not necessarily understood a-priori, providing challenges in terms of complexity of hybrid simulation experimental setup and control.

The hybrid experiment in this paper expands on these limitations by conducting a hybrid simulation with a large-scale, three-DOF experiment, leveraging model updating. The test is based on a major California bridge which features seismic isolation on half of its piers. The experimental substructure is the isolation bearing located on the tallest pier. The design of the isolation bearings at each pier is identical. However, as the height of piers increases, the demand for each bearing varies from one to the next, resulting in different demands for each isolation bearing. This combination makes the bridge a good candidate for model updating. During the hybrid simulation tests, two representative ground motions were used to evaluate the performance of the bearing, bridge, and overall testing methodology. Model updating was used to learn both the shear and rotational behavior of the bearing, making the application unique as there are limited applications of multi DOF model updating.

3.3 Hybrid Simulation Experiment

The outline of the hybrid simulation is shown in Figure 3.2. The bridge is a major bridge in southern California (same as presented in Chapter 2) with 13 isolated piers and 12 non-isolated piers. Piers 2 through 14 have four identical lead-rubber bearings (LRB) on top of a capping beam over two reinforced concrete columns. These piers increase in height from 10.7 to 48.8 meters, and the column dimensions increase with increasing pier height. The properties of the LRBs are summarized in Table 3.1. When the loading conditions are very similar on each bearing, it may be appropriate to assume that all elements have identical responses to a single physically tested bearing. For this case study, however, the demands are different due to the changing flexibility of the piers and the response would not be accurately captured by assuming an identical response. Thus, in a hybrid simulation without model updating, this would require testing 13 bearings simultaneously.

Leveraging model updating, for this test, a single bearing was used. The bearing at the top of pier 14 was used for the experimental subassembly as the pier flexibility leads to higher shear and rotational demands that the updating algorithm can use to learn bearing model parameters. The data from the experimental test was used to update the modeling parameters for the bearings on piers 2 – 13. The model-updating algorithm proposed by Cheng and Becker (2021) was adopted for the experimental program which uses a weighted adaptive constrained unscented Kalman filter (WACUKF). A detailed examination of the performance of the model updating in this hybrid experiment can be found in Cheng et al. (2024) .

For the numerical subassembly, a 2D model of the bridge was developed using OpenSees (McKenna et al., 2010). The concrete piers are modeled using beam-with-hinges elements. At the hinges, a fiber section was defined with Concrete02 and Steel02 materials to capture the nonlinear behavior of piers including strength degradation with cyclic loading. The pier caps and deck are assumed to be elastic. The deck masses are lumped at the top of each bearing node, and the pier beam cap masses are lumped at the top of each pier. A consistent mass matrix is formulated for the distributed mass of the piers along their height. The structural damping is estimated using Raleigh damping coefficients based on the first two modes and a damping ratio of 0.03 per Caltrans (2019). The boundary conditions are simplified by assuming fixed pier bases and a pinned abutment connection.

The physical subassembly of a single LRB was tested in the University of California, Berkeley Structures Laboratory. A 1/3rd length scale was used for the physical test, and the model bearing properties are in Table 1. The experimental set up was designed to test the bearing in three in-plane DOFs: shear, axial and rotation with loads applied at the working point shown in Figure 3.1 (note that the figure shows positive shear and negative rotation). The experimental setup is shown in Figure 3.3. Darlington and Becker (2021) found that rotational stiffness and the rotation shear coupling is related to the total rotation of the bearing, regardless of whether it is applied at the top or bottom plate. This finding allowed for a simplified set up keeping one end of the bearing fixed while applying shear and rotation only to one end plate. Three actuators are used in the experiment; one in the horizontal direction with a capacity of 534kN and stroke of 46 cm (120 kip,

18 in stroke) and two actuators in the vertical direction with a capacity of 534kN and stroke of 15 cm (120kip, 6 in stroke) to apply axial load and rotation. Figure 3.4, shows the bearing subjected to combined shear and rotation loads.

A schematic of the hybrid simulation communications protocol is shown in Figure 3.5. The hybrid simulation starts by assuming an initial stiffness of the bearing in shear and rotation and applying a displacement command (u) to the experimental element in OpenFresco (Schellenberg et al., 2009). OpenFresco contains the experimental setup and handles communications between the numerical subassembly commands (u) and the physical response (f). OpenFresco sends the command to Simulink which hosts a continuous signal generator with a predictor-corrector algorithm that is sent to the controller. An MTS FlexTest was used to control the actuators in the physical experiment, interfacing with the MTS DOF Control package to handle the geometric transformations from the actuator coordinate system to the global work point coordinate system used in the numerical analysis. Finally, the model updating algorithm, written in Python, reads in the response of the physical subassembly and fits the experimental data with an updated set of bearing parameters. These new ‘instantaneous’ parameters are returned to OpenSees and used in the numerical model in the following time step. Additional details about the experimental communication setup and the integration of model updating are available in Cheng (2023).

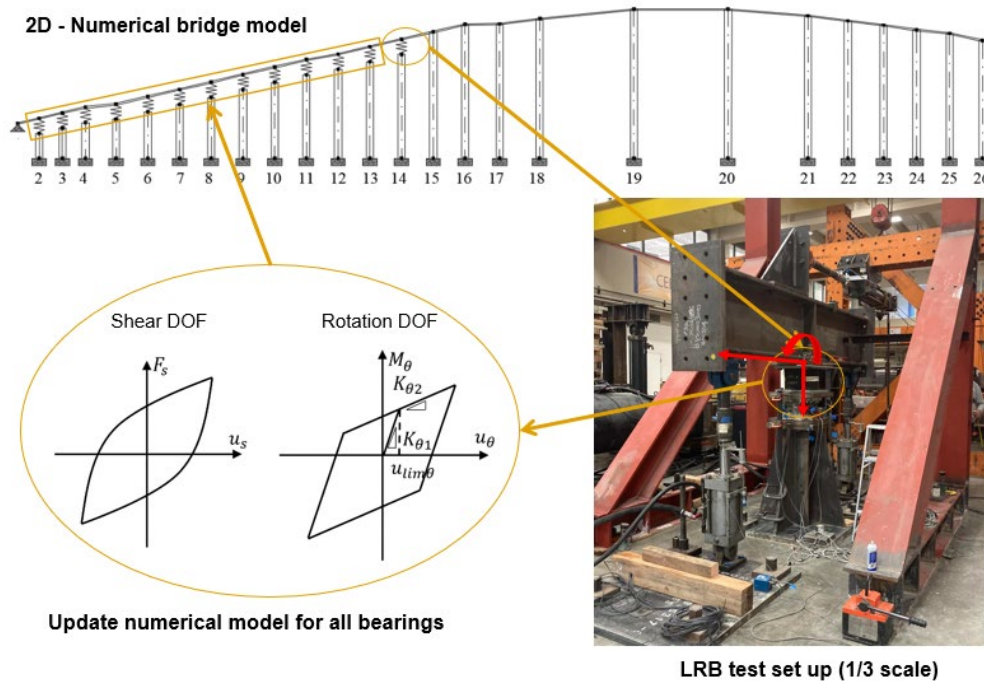


Figure 3.2. Hybrid simulation schematic including the numerical subassembly, the physical subassembly and the model updating flow.

Table 3.1. Bearing Geometry.

Bearing Parameter	Prototype	Experimental model (1/3 scale)
Bearing diameter (mm)	1143.0	381.0
Lead core diameter (mm)	228.6	76.2
Number of layers	20	20
Total rubber thickness (mm)	381.0	127.0
Shim thickness (mm)	18.96	6.35
Rubber shear modulus (MPa)	0.41	0.41

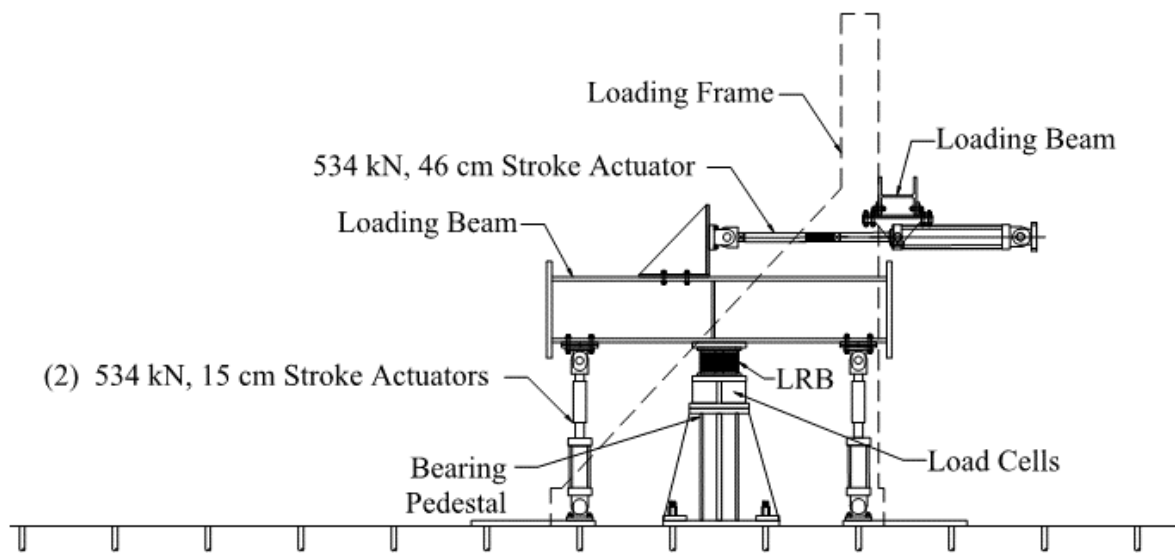


Figure 3.3. 3 DOF hybrid simulation experimental set up.

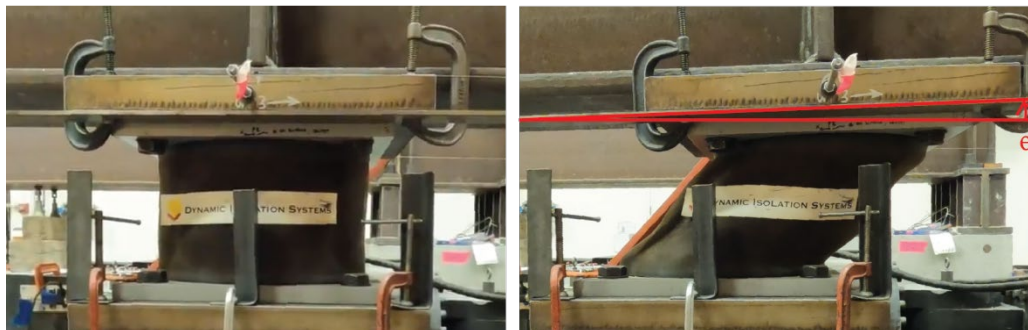


Figure 3.4. LR bearing during the experiment. (left) Undeformed bearing (right) bearing with a positive rotation of 0.016 radians, and negative shear strain 150%.

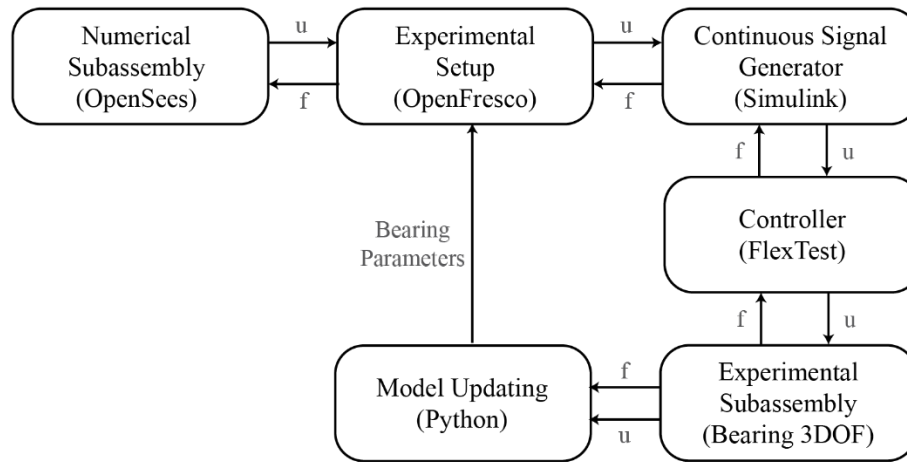


Figure 3.5. Control loop for hybrid simulation experiment.

3.4 Bearing Numerical Model

A variety of elastomeric bearing models, capable of including bearing rotations, have been developed. The simplest rotation model, of course, uses a linear decoupled spring for the rotational stiffness. This has been adopted by researchers such as Kumar et al. (2014) when rotational demands are not expected to be large. Another option, multiple spring models have been developed that include rotational behavior; however, their focus is primarily on bearing stability under typical parallel end-plate boundary conditions (Nagarajaiah and Ferrell, 1999, Iizuka, 2000, Han et al., 2013, Han and Warn, 2015). These spring models simulate LRB behavior with a combination of two or more linear or nonlinear springs under coupled degrees-of-freedom (DOFs). The spring models can simulate the hysteretic features like stiffness degradation and interaction between different DOFs. Later, models were extended to capture support rotation. Based on experimental tests Vemuru et al. (2014), proposed a new nonlinear empirically based equation for the rotational and shear springs using higher order terms of lateral displacement; these equations require two calibration parameters. Rastgoo Moghadam and Konstantinidis (2017b) proposed a spring model to find the effect of end-plate rotation on the critical shear force; the model relies on modification factors calibrated using an FEM model (e.g. Rastgoo Moghadam and Konstantinidis (2017a)) or experimental data. Ishii et al. (2017) proposed a model capturing the interaction between shear and end-plate rotation using a group of parallel vertical springs. Their study explored the moment-rotation relationship for different shear strains and rotations, finding a nearly bi-linear behavior in the moment-rotation relationship and highlighting the coupling between bearing shear, rotation, and axial response. Generally, these models are based on complex calculations of the spring states and require iterative solution algorithms for analysis, often resulting in implementation or convergence challenges. Many of them also rely on experimentally calibrated parameters, as noted above.

There are also matrix-based numerical models to capture rotation. He et al. (2012) proposed a matrix model representing the displacement-force relationship with a polynomial equation to calculate the bearing horizontal stiffness based on the current rotational stiffness and axial load. Crowder and Becker (2017) developed a stiffness matrix model for natural rubber bearings. While the model captures the shear stiffness changing over the typical operating shears, its performance under extreme displacement is less accurate. These models consider coupling and are easy to use, but they still lack the ability to capture a full range of hysteretic features, such as energy dissipation, and have limitations on their range of validity.

To capture the rotational bearing behavior as well as the rotation-shear interaction in the numerical subassembly, a bearing model capable of representing this behavior is required. To implement model updating, it is desirable that the model have easy-to-update parameters. Models capable of describing this behavior often use complex combinations of springs (Vemuru et al., 2014, Ishii et al., 2017, Rastgoo Moghadam and Konstantinidis, 2017b) with many parameters and are often not formulated for implementation within larger system-level models. Instead, an alternative bearing model which compromises on complexity and ease of use is used. The model is described briefly here and discussed in detail in (Cheng et al., 2024).

In the shear DOF, a Bouc-Wen model (Park et al., 1986) was modified with a cross term for rotational influence, resulting in

$$F_s = \underbrace{[\alpha_1 g(k_x, u_x) + (1 - \alpha_1) q_{yield} z_x]}_{\text{Bouc-Wen Model}} + \underbrace{CT_s u_\theta}_{\text{rotation coupling term}} \quad (3.1)$$

where F_s and k_x are the shear force and stiffness; α_1 is the ratio of the post yield and elastic stiffness; q_{yield} is the characteristic strength; and z_x is the hysteretic displacement. The interaction with the rotational DOF is captured using the added cross term, CT_s , multiplied by the angle of rotation, u_θ .

The rotational DOF follows a bilinear model, based on the behavior shown by Ishii et al. (2017), with a separate cross-term for shear influence.

$$M_\theta = \underbrace{(K_{\theta 1} - K_{\theta 2}) u_{lim\theta}}_{\text{first stiffness}} + \underbrace{K_{\theta 2} u_\theta}_{\text{second stiffness}} + \underbrace{CT_\theta u_s}_{\text{shear coupling term}} \quad (3.2)$$

where M_θ is the rotational moment. The first term is an elastic perfectly-plastic model where $u_{lim\theta}$ is the elastic deformation limit taken as a constant value equal to 0.0003 rads based on quasi-static test results, and $K_{\theta 1}$ and $K_{\theta 2}$ are the pre and post yield stiffnesses. The interaction with the horizontal DOF is captured by the cross term, CT_θ , multiplied by the shear displacement, u_s .

The axial DOF was modeled simply as a linear spring with axial force, $F_v = k_v u_v$, and axial stiffness, $k_v = \frac{E_c A}{T_r}$, where u_v is the axial displacement, E_c is the elastic modulus, T_r is the total rubber thickness, and A is the area of the bearing. The parameters for the axial load are not updated because the axial load behavior does not change significantly under the loading. The parameters updated during the experiment are $\alpha_1, k_s, q_{yield}, CT_s, K_{\theta 1}, K_{\theta 2}, CT_\theta$. The initial estimate of the bearing parameters was obtained by using the LRB properties to calculate the mechanical properties of the isolator theoretically following (Kelly and Konstantinidis, 2011) and validated with small displacement, quasi-static initial tests. The initial estimates of the parameters are shown in Table 3.2 along with the final bearing parameters obtained from experimental data discussed later.

Table 3.2. Bearing Parameters

Bearing Parameter	Initial Estimate	Final Fitting (GM1 Scale = 2.0)	Change
k_s (N/m)	$8.98 \cdot 10^6$	$5.61 \cdot 10^6$	-37.5%
q_{yield} (N)	$3.90 \cdot 10^5$	$2.82 \cdot 10^5$	-27.8%
α_1	0.119	0.172	+44.7%
CT_s (N)	-	$-2.61 \cdot 10^5$	-
$K_{\theta 1}$ (N-m)	$3.63 \cdot 10^7$	$7.40 \cdot 10^8$	-
$K_{\theta 2}$ (N-m)	$3.63 \cdot 10^7$	$6.32 \cdot 10^7$	-13.0%
CT_{θ} (N)	-	$-5.00 \cdot 10^4$	-

3.5 Experimental Results

A site-specific hazard analysis was done to obtain a set of ground motions spectrally matched to the 2475-year return period earthquake event (2% probability of exceedance in 50 years). Two representative ground motions were selected for experimental testing then scaled with factors of 1.0, 1.5, and 2.0. Ground motion 1 (GM1) is the Cape Mendocino, USA, 1992 earthquake recorded at the Petrolia station with a moment magnitude of 7.0 and ground motion 2 (GM2) is the Iwate, Japan, 2008 earthquake recorded at the Kurihara city station with a moment magnitude of 6.9.

3.5.1 Hybrid Simulation Performance

The complexity of the 3 DOF control in the setup limited the time scale for the testing. The maximum error of the hybrid simulation test under GM2 at a scale of 1.0 at two different time scales was compared. At a time scale = 5.2, the peak shear and rotation DOF error were 1.6 mm and $7\text{e-}4$ rad, respectively; while at a time scale = 17.3, they were 0.46 mm and $2.8\text{e-}4$ rad (reductions of 71% and 60%). Figure 3.6 shows experimental data at these time scales. The smaller time scale exhibits significantly more noise, indicating increased error from the control system. In contrast, the longer time scale results in a smooth displacement curve, representative of better control of the experimental test. As such, a time scale of 17.3 was selected for the remainder of the experiments presented in this paper.

Model updating has the benefit of increasing the accuracy of the experiment while maintaining a reduced experimental setup by testing a single bearing. Figure 3.7 and Figure 3.8 show the error tracking in the experimental control program, comparing two experiments with and without model updating under GM1 at a scale of 2.0 and GM2 at a scale of 2.0, respectively. The model updating algorithm used in the experiments takes approximately 0.002 seconds per integration step ($\frac{1}{4}$ of the time of the numerical analysis). Here, when model updating is included in the experiment, the error remains low with no change in the maximum shear error and a very small change in the maximum rotation error as compared with no model updating. Note that the shear and rotational demands remain out-of-phase (positive displacement and negative rotation as shown in Figure 1), as can be seen in Figure 3.7 and Figure 3.8.

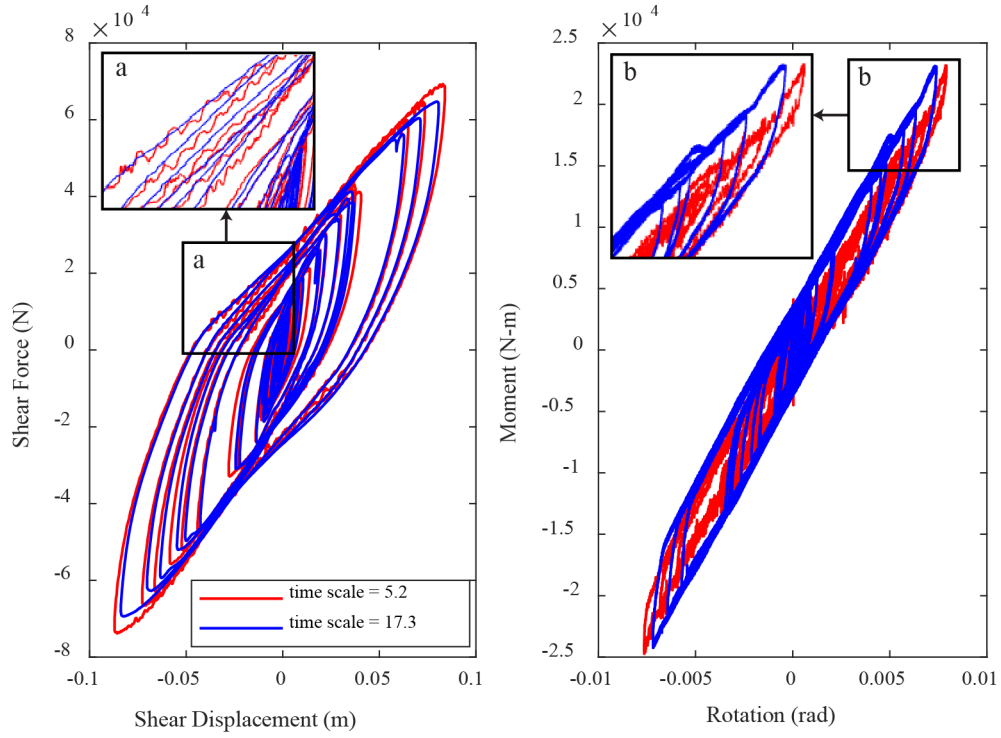


Figure 3.6. Comparison of experimental results for GM2, scale factor of 1.0 with time scale = 5.2 and 17.3

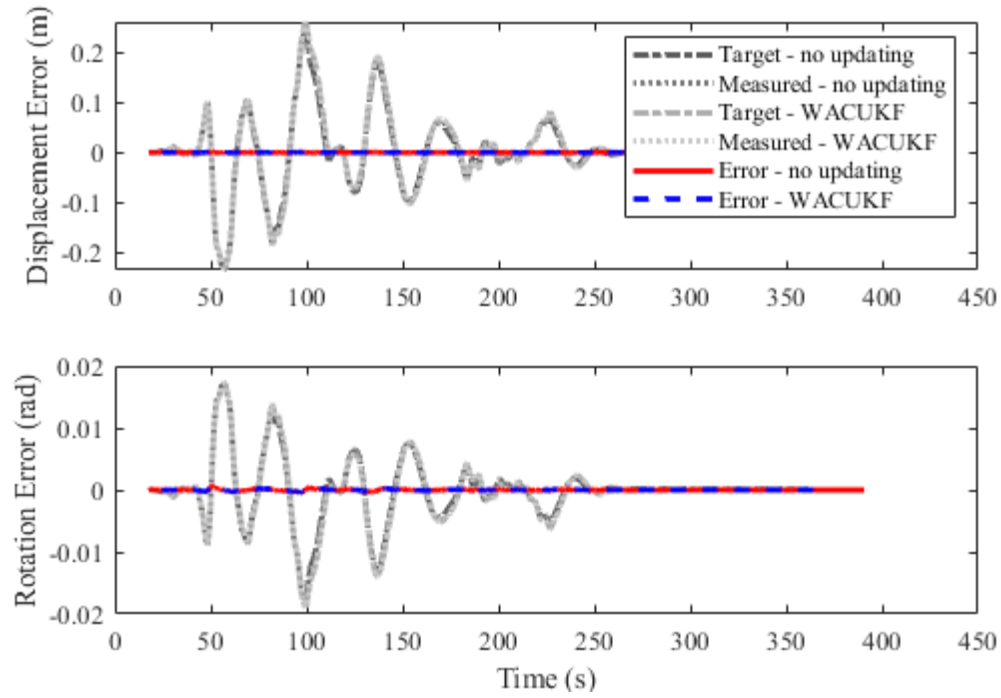


Figure 3.7. Error Tracking under GM1 and scale factor of 2.0 for experiment with and without model updating.

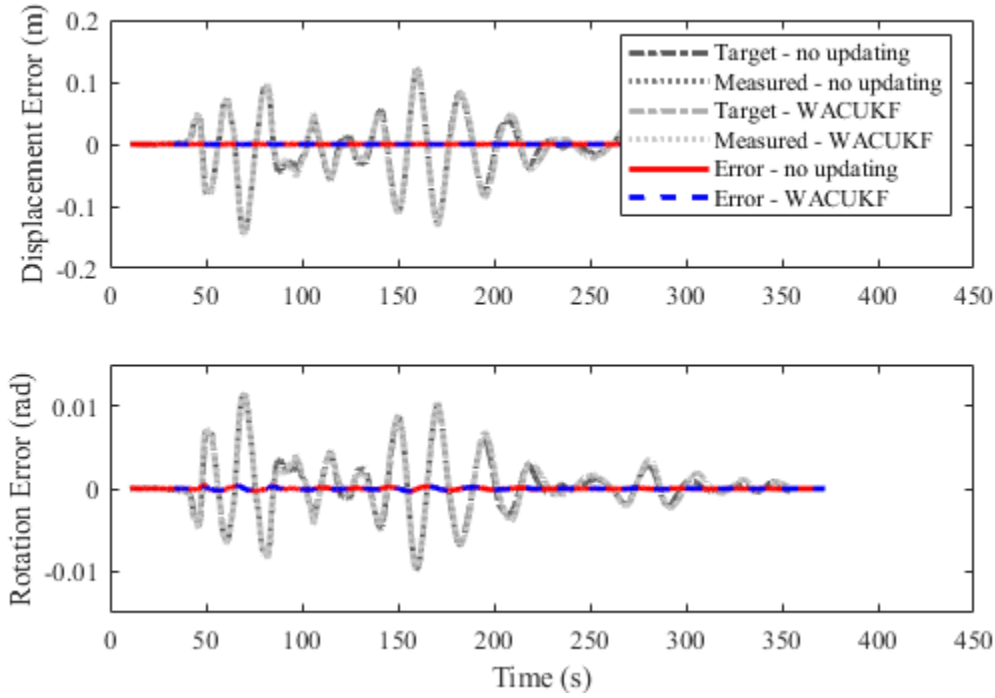


Figure 3.8. Error Tracking under GM2 and scale factor of 2.0 for experiment with and without model updating.

3.5.2 Bearing and System Performance

To assess the accuracy of the bearing response prediction and the effectiveness of model updating a detailed comparison is presented using the bearing model obtained from experimental tests. The initial response prediction of the bearing is obtained using the initial parameters from bearing geometry. The model updating algorithm results in two additional sets of data fittings: the instantaneous fitting, which progressively changes during the experiment by updating the model parameters instantaneously, and the final fitting, which uses only the parameters fit at the end of the experiment to reflect the capacity of the learned parameters for future modeling. The initial and final predicted responses are compared with the experimentally recorded response. The experimental results for GM1 and GM2 at a scale of 2.0 are plotted in Figure 3.9, showing the shear and rotational hysteresis at full scale. GM1 had shear and rotational demands of ± 762 mm. and ± 1.09 degree, respectively. GM2 had smaller shear and rotational demands of ± 381 mm. and ± 0.68 degree, respectively. The ability of the final fitting results to capture the experimental data behavior clearly contrasts with the initial model prediction, which overestimates the damping in shear and the rotational stiffness. The change in shear and rotation parameters are listed in Table 3.2, showing a decrease in shear stiffness, characteristic strength, and rotational post-yield stiffness and an increase in the shear post-yield stiffness ratio and the initial rotational stiffness.

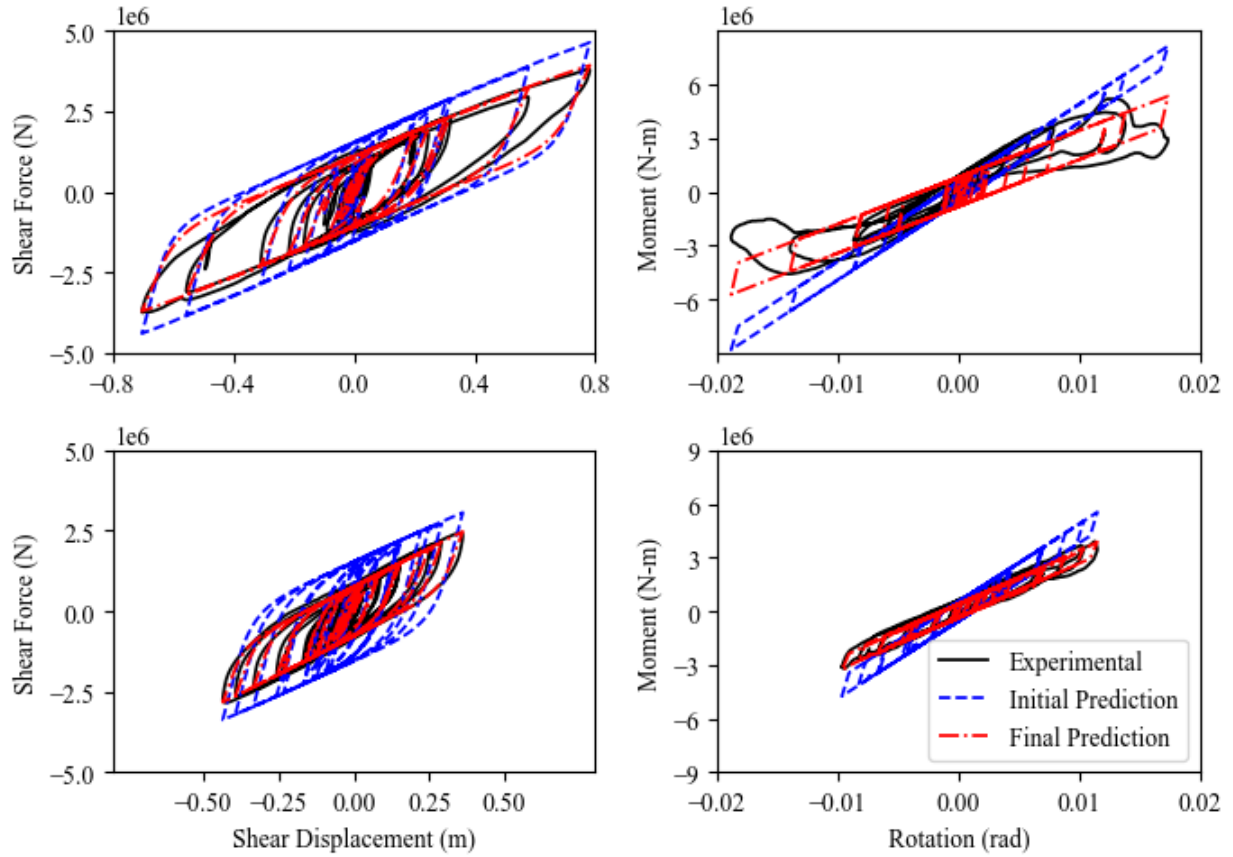


Figure 3.9. Shear and rotational behavior with fitted parameters for GM1 and GM2 at scale factor of 2.0

The effects of model updating are evaluated at a system level by comparing the structural response from hybrid simulation with and without model updating as shown in Figure 3.10 for GM1 and GM 2 with a scale factor of 2.0. The bearing shear and rotational demands increase with increasing pier height; as such, the largest difference in the structural response is that of piers 10, 11, and 12. Note that since these results are directly from the hybrid simulation, the ratio for pier 14, and adjacent pier 13, are very close to 1 because bearing 14 is the experimental element. The test without model updating predicted lower values for pier drift, bearing shear strain, and rotation by 20.0%, 18.3%, and 15.6%, respectively, while smaller changes in base shear and moment demands of 8.9% and 2%. The test without model updating results in lower values for pier drift, bearing shear strain, and rotation because the updated estimate has a lower bearing effective stiffness in shear and rotation as shown in Figure 3.9. Thus, the assumptions for the bearing numerical model parameters directly affect the system-level response.

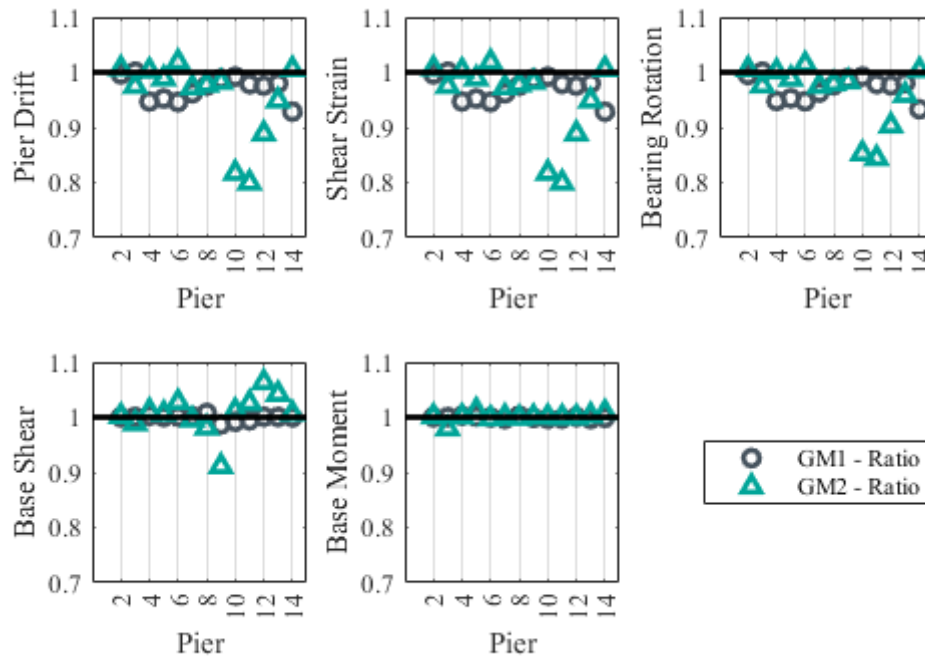


Figure 3.10. Structural response ratio of hybrid simulation test without model updating to with model updating for GM1 and GM2 scale of 2.0.

Another consideration is the selection of the physical specimen to be tested. The experiments presented up to this point used bearing 14 as the physical element since this is the bearing on the tallest pier and has the highest displacement demands. However, to explore the consequences of using a different element in the experimental subassembly, a second case uses bearing 7 as the experimental element, which, being on a mid-height pier, presents lower displacement demands. Figure 3.11 shows the response ratio under GM1 and GM2 for a hybrid simulation using bearing 7 to that of using bearing 14 as the physical element. This shows that in the pier drift, bearing shear strain and rotation are up to 17% lower for shorter piers like pier 4 while up 13% higher for taller piers like pier 14.

To examine the reason for the system changes we can compare the hysteretic response resulting from model updating and validation data for both cases. The validation data was obtained by recording the force-displacement data of the numerical bearings during the hybrid simulation and applying the displacement record in a displacement-controlled test on the bearing to obtain a force response. This process results in two hysteric loops that can be compared to evaluate how well the model updating captures the actual response. Figure 3.12 shows the shear and rotation hysteresis for the instantaneous fitting, the final fitting, and the validating data in the two cases (1) using bearing 14 as the physical element and validating the numerical model of bearing 7, and (2) using bearing 7 as the physical element and validating the numerical model of bearing 14, using GM2 at a scale of 2.0 for both cases. In case 1, the model updating was able to learn the parameters from the experimental response of bearing 14 and provides as good fit to the validation data of bearing 7. However, for case 2, the model updating results in a higher effective shear stiffness and

lower damping than the validation data for bearing 14. The accuracy of the model updating algorithm can be quantified by comparing the equivalent damping ratio between the validation data and the updated model discussed in depth in Cheng et al. (2024). The second case estimates an effective damping ratio 16.22% and 52.01% lower than the validation data; the findings are summarized in Table 3.3. This significant underestimation of the damping ratio can lead to the increase in bearing shear strain, rotation, and pier drift observed in Figure 3.11. Using the element with the most extreme behavior, in this case, bearing 14, the model updating can more accurately capture the mechanical properties of other elements with lower demands and, as such, leads to better estimation of the overall structural response.

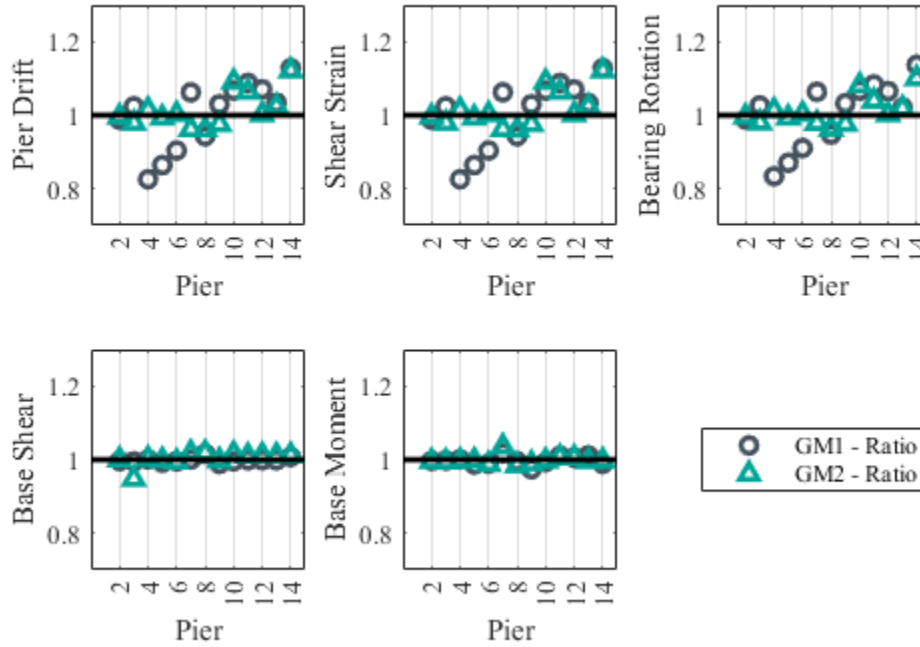


Figure 3.11. Structural response ratio of hybrid simulation test using bearing 7 to test using bearing 14 for GM1 and GM2 scale of 2.0.

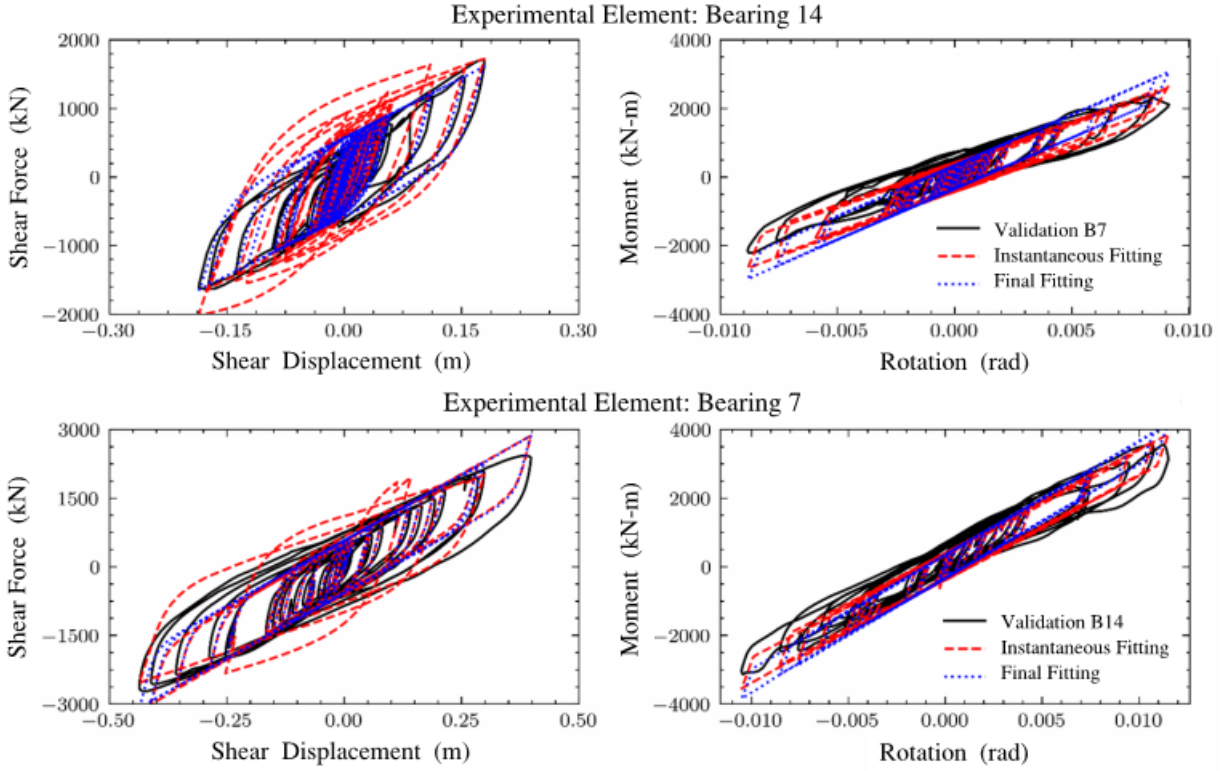


Figure 3.12. Validation data comparison for using different experimental elements:
Case (1) bearing 14, Case (2) bearing 7

Table 3.3. Percent difference in equivalent damping ratio

	Case 1 (Experimental B14)	Case 2 (Experimental B7)
Shear	7.8%	-16.22%
Rotation	-12.11%	-52.01%

3.6 Extension to Bridge Performance

To evaluate the overall system behavior, two fully numerical models of the bridge are compared. In the first model, the bearings are represented using a Bouc-Wen model in shear and a linear spring in rotation using the initial estimated parameters (Table 3.2). In the second model, the proposed bearing model including a nonlinear rotation and coupling in the shear and rotational DOFs is used. The proposed model uses the final fitting parameters, learned from the hybrid simulation experiments of GM1 at a scale of 2.0 since the bearings experienced the most extreme behavior in that case. A sample bearing hysteresis is presented in Figure 3.13, comparing the numerical response of bearing 14 in the Bouc-Wen with linear rotation model to the proposed model including updated parameters along with the experimental response for reference. The shear behavior shows a reduction in effective stiffness and energy dissipation compared to the Bouc-Wen model with linear rotation due to the rotational coupling, while the rotational DOF follows a similar tangential stiffness to the linear estimation with an increase in energy dissipation due to the nonlinear response as compared to the linear approximation. The effective shear stiffness at maximum displacements during the test decreases by 15% when the proposed model (Cheng et al., 2024) and updated parameters are used.

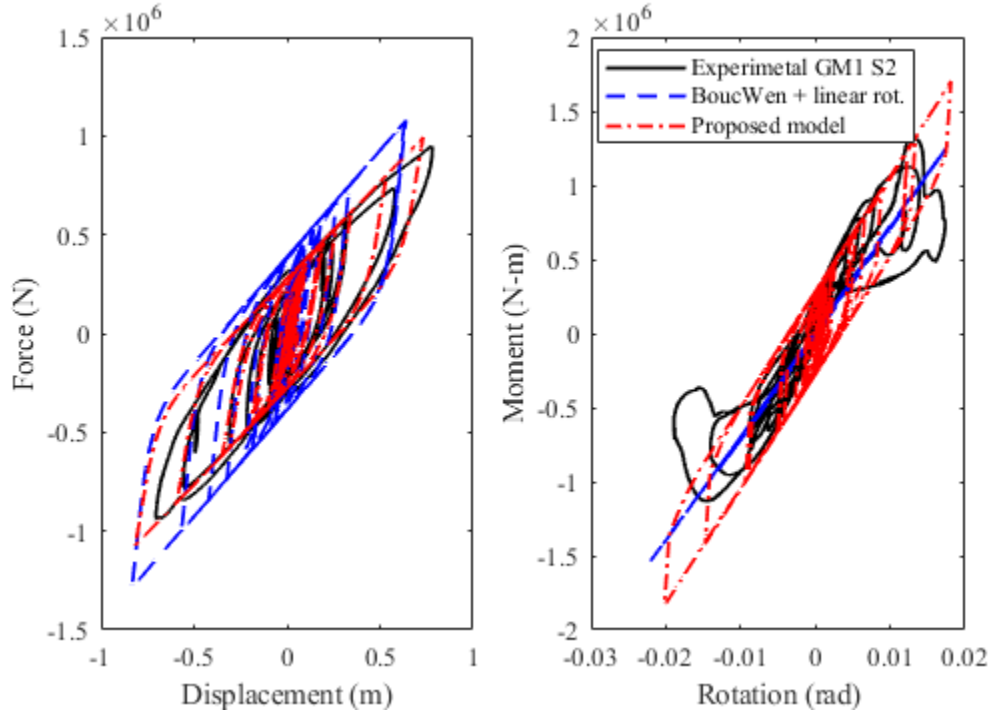


Figure 3.13. The hysteretic behavior of bearing 14 comparing the Bouc-Wen with linear rotation model and proposed model with parameters fitted using GM1 at a scale of 2.0.

Figure 3.14 shows the pier and bearing responses from the Bouc-Wen model with linear rotation to the proposed model with final fitting parameters. For GM1 and GM2 at a scale of 2.0, the Bouc-Wen with linear rotation model tends to overestimate the total bearing rotation with the largest value 25% higher than the proposed model. The response ratio of pier drift and bearing shear strain, however, varies along the piers with the Bouc-Wen with linear rotation model tending to overestimate the response of the short piers and underestimate the response of the taller piers. In both cases, the base shear and moment remain similar for all piers.

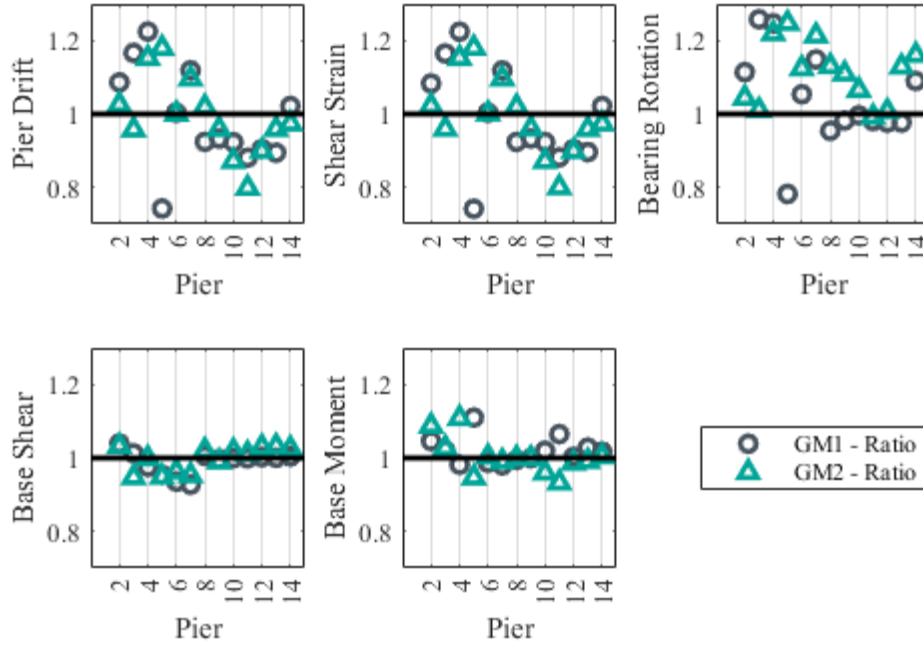


Figure 3.14. Structural response ratio of the Bouc-Wen with linear rotation model to the proposed model with final fitting parameters.

An incremental dynamic analysis is performed in both cases to determine the fragility curves for a moderate damage (DS-3) and a major damage (DS-5) of the bridge. A suite of 11 ground motions, shown in Table 3.4, selected and spectrally matched for a hazard with 2% probability of exceedance in 50 years, is then scaled to factors of 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, and 4.0. The suite includes the ground motions used in the experimental program and are labeled with (GM1 and GM2). The damage states of the piers are defined based on the work of Vosooghi and Saiidi (2010). For the piers, DS-3 indicates extensive cracks and spalling of the concrete, while DS-5 is defined as the onset of compressive failure of the concrete core. The median maximum drift ratio is 0.059 at DS-3 and 0.131 at DS-5. For the LRBs, there is no formal definition of damage states in the literature. For this investigation, bearing stability limits are used. The moderate damage level (DS-3) is defined as the bearing buckling limit calculated by using the overlapping area method (Buckle and Liu, 1994) which for this design is 235% shear strain.

Weisman and Warn (2012) found that the overlapping area method for calculating bearing instability was a conservative estimate. As such, the major damage state (DS-5) is defined as the isolator shear strain where the displacement is equal to the diameter of the bearing and there is no rubber overlap, 300% strain.

Table 3.4. Suite of ground motion records used in analysis including those used in experiments (GM)

RSN	Earthquake Name	Station Name	Mw	D 5-75 (sec)	D 5-95 (sec)
828 (GM1)	Cape Mendocino	Petrolia	7	6.5	17.7
5818 (GM2)	Iwate_ Japan	Kurihara City	6.9	6.5	15.1
1158	Kocaeli_ Turkey	Duzce	7.5	6.1	11.8
1509	Chi-Chi_ Taiwan	TCU074	7.6	13.3	19.7
1787	Hector Mine	Hector	7.1	7.6	11.7
3748	Cape Mendocino	Ferndale Fire Station	7	6.2	13.6
3934	Tottori_ Japan	SMN002	6.6	6.7	15.6
3965	Tottori_ Japan	TTR008	6.6	6.2	18.5
4031	San Simeon_ CA	Templeton - 1-story Hospital	6.5	3.3	10.3
6890	Darfield_ New Zealand	Christchurch Cashmere High School	7	9	20
8063	Christchurch_ New Zealand	Christchurch Botanical Gardens	6.2	4.8	11.4

The maximum drift ratio and shear strain values for each damage state were used as a threshold to determine observed fractions of damage at each intensity value. Then, a lognormal cumulative distribution function (CDF) was fit to the data to determine a fragility function for each element. The entire bridge is assumed to reach a damage state if any one element reaches that damage state at each intensity level. The fragility curves for both bridge models are compared in Figure 3.15. The bridge model with Bouc-Wen with linear rotation reached the median probability

of DS-3 and DS-5 at intensity values of 1.36 and 1.79, respectively. The bridge model incorporating the proposed bearing model, with parameters fitted to the GM1 experimental data, reached the median probability of DS-3 and DS-5 at intensity values of 1.34 and 1.73, respectively. These results show that the overall change in performance is small. This small increase in damage probability, more noticeable at the element level in Figure 3.15, shown by the shift to the left of the fragility curves when rotation is considered, indicates that coupling of shear and rotation has a small effect on overall bridge performance for this case study.

On a component level there are significant changes to displacements and shear strains as shown in the structural response in Figure 3.14, where the response ratio represents individual component changes. However, the fragility curves show small changes because they are dependent on triggering a threshold displacement which is not as sensitive as the individual component results. Additionally, in the case of the overall bridge fragility shown in Figure 3.16, since it is assumed to be controlled by the worst-case component (pier 14 in this case) it is consistent with Figure 3.14 to have insignificant changes in shear and pier drift. Finally, while the effective shear stiffness is reduced when rotational demands are introduced, reducing the energy dissipation in shear, the nonlinear rotational behavior dissipates more energy compared to the Bouc-Wen with linear rotation model. As such, the changes in bearing behavior result in a very similar performance for the whole system as measure by the fragility curves.

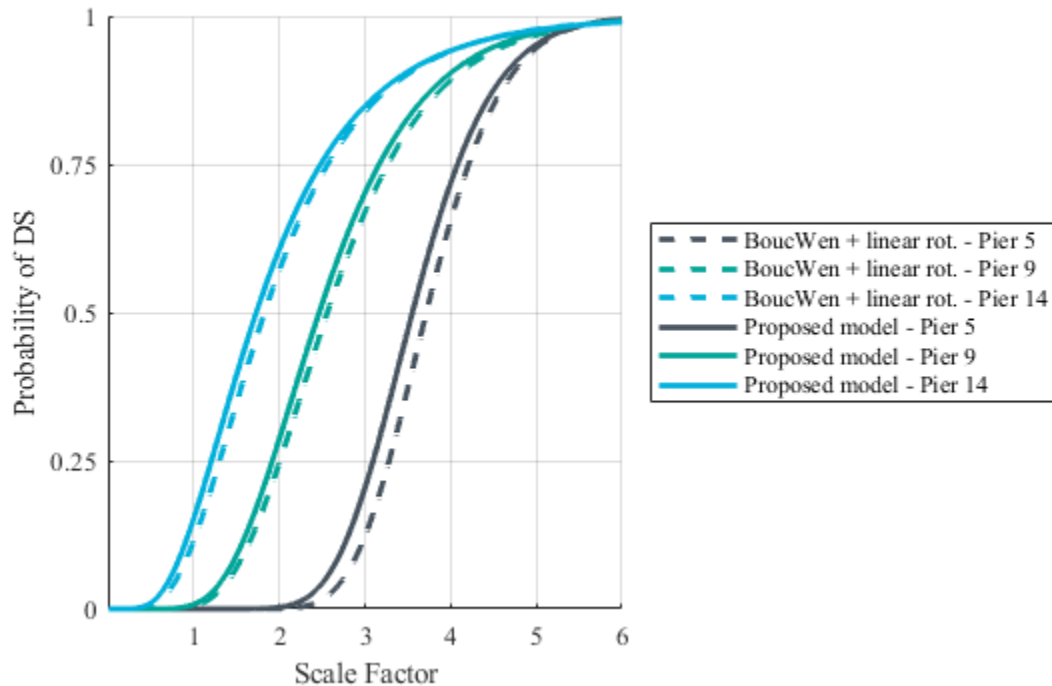


Figure 3.15. Comparison of fragility curves for bearings on pier 5, 9, and 14 at DS-5.

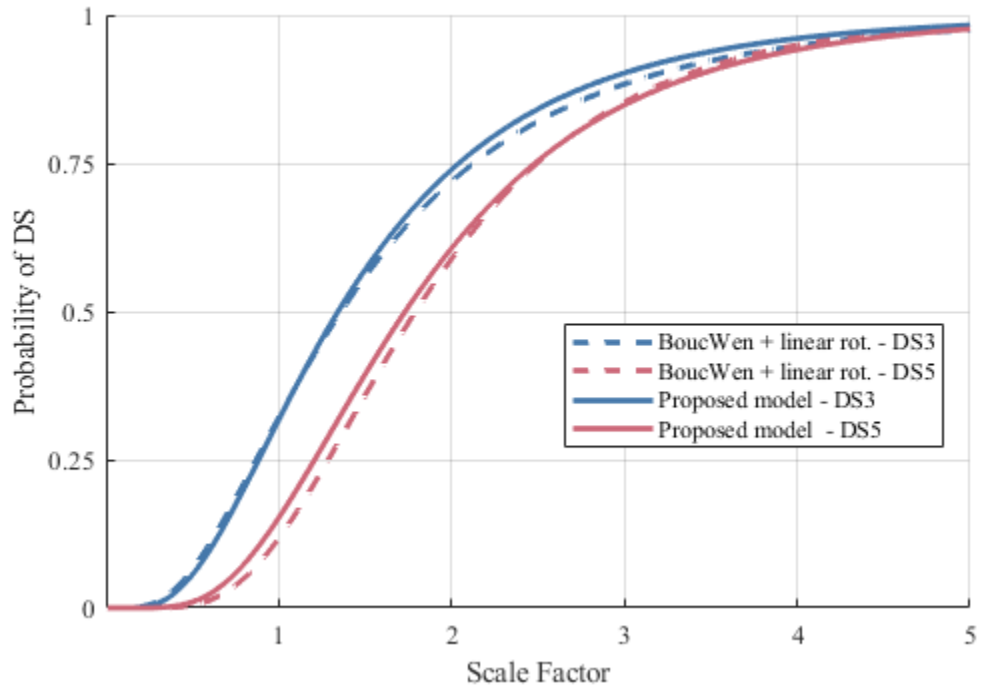


Figure 3.16. Comparison of bridge fragility curve at DS-3 and DS-5.

3.7 Conclusions

The major bridge evaluated is partially isolated with lead rubber bearings over 13 of its piers. Top-of-column isolated sections with tall piers introduce flexible boundary conditions that are contrary to the typically assumed parallel end-plate conditions of isolators. While there are existing models that try to capture bearing rotational behavior many are still highly complex and rely on parameters calibrated experimentally while still lacking coupling behavior between different degrees of freedom. This paper discussed the implementation of an innovative hybrid simulation experiment with model updating and with three DOF testing capabilities. The experimental program proposed was able to overcome the modeling challenges of LRB isolators and accurately capture the complex behavior due to combined shear, axial and rotational demands.

The results of the experiments show that model updating significantly improved accuracy of the experimental results. Additionally, the component behavior changes captured by the model updating resulted in larger values of expected pier drift, bearing shear strain, and rotation. While there were significant changes in the bearing's component behavior, the overall bridge performance remains almost the same for both cases at DS-3 and DS-5. This is attributed to the fact that while there was decrease in the stiffness and energy dissipated through shear, there was an increase in the energy dissipation in the rotational DOF due to the nonlinear behavior. These two changes counter each other in this case study resulting in similar damage fragility curves. However, such a difference in the component response could alter the vulnerability and performance expectations of other bridges. Finally, the experimental program was able to capture changes in structural behavior that are not captured by traditional modeling with rigid end-plate conditions.

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4. Conclusions and Future Research

4.1 Discussion and Comparison

The structural performance evaluation using PBEE in this study involves inputs from hazard analysis, structural analysis, and damage analysis decisions. Research in the many areas involved is constantly evolving. The effects on the structural performance of the major bridge presented in Section 2.4 for ground motion correlation and in Section 3.6 for the bearing rotational behavior provide a basis to compare the effect of both inputs.

The baseline fragility functions for the ground motion duration study, presented in Chapter 2, are the results for the non-correlated dataset. The baseline fragility functions for the bearing rotation study, presented in Chapter 3, are the results for the Bouc-Wen with linear rotation model. These baseline fragility functions are obtained from the same set of ground motions and assume a similar bearing model. For the non-correlated dataset, the major bridge reaches the median damage probability of DS-3 and DS-5 at scale factors of 1.09 and 1.19, respectively. For the Bouc-Wen with linear rotation model, the major bridge reaches the median damage probability of DS-3 and DS-5 at scale factors of 1.36 and 1.81, respectively.

Note that the estimated baseline damage fragility functions for the major bridge (the dashed fragility lines) shown in and Figure 4.1 present a significant difference, 24.7% for DS-3 and 52.1% for DS-5, despite using the same suite of ground motions. The ground motion duration correlation study in Section 2.4 uses a 3D model that includes bi-directional ground motions as well as a nearly 90-degree bend along the span of the bridge; as such, the estimated structural demands are larger and result in a larger damage probability. The bearing rotation study in Section 3.6 used a 2D model of the major bridge to estimate the structural response. The reduced model was necessary to have an achievable experimental test setup. As such, it includes ground motions in a single direction and ignores the nearly 90-degree bend along the span which results in the lower damage probability observed.

Comparing the damage fragility functions developed for the ground motion duration correlation and the bearing rotation, shown in Figure 4.1, reveals distinct impacts on the bridge performance. When accounting for duration correlation, the major bridge reaches the median damage probability for DS-3 and DS-5 at scale factors 29.2% and 37.4% higher than the baseline, respectively. In contrast, when accounting for bearing rotational behavior, the major bridge reaches the median damage probability for DS-3 and DS-5 at scale factors 1.5% and 4.0% lower than the baseline, respectively. These results show evidence that the performance of the major bridge presented in this investigation is more sensitive to the changes in hazard analysis when considering ground motion duration, while it exhibits only marginal changes when considering bearing rotation. The changes for each study are summarized in Table 4.1. It is worth noting that this result is consistent with findings by others; Lee and Mosalam (2006) showed that ground motion uncertainties are more significant than uncertainties in structural parameters at high intensity levels based on a PEER PBEE testbed of a USC building.

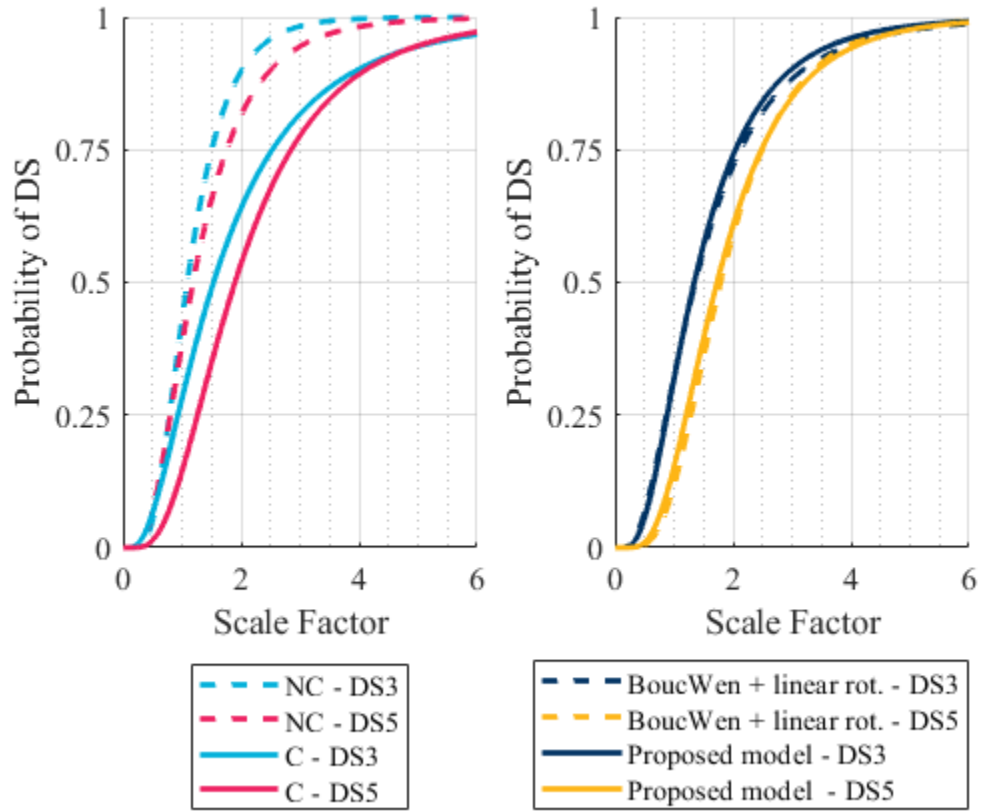


Figure 4.1. Comparison of bridge fragility curves at DS-3 and DS-5. (left) Duration correlation study, (right) bearing rotation study

Table 4.1 Change in median DS intensity measure when ground motion duration and bearing rotation are considered for the major bridge.

Damage state	Duration Correlation	Bearing Rotation
DS-3	29.2%	-1.5%
DS-5	37.4%	-4.0%

4.2 Summary

The structural performance evaluation using PBEE in this study involves inputs from hazard analysis, structural analysis, and damage analysis decisions. There is constantly emerging research in the many areas involved. A major bridge in Southern California with seismic isolation bearings on top of flexible piers and a typical highway bridge in Northern California were used to evaluate the effect of distinct hazard inputs and structural modeling assumptions on structural performance.

Chapter 2 considers the influence of ground motion duration correlation to intensity in the hazard analysis. Traditional hazard analysis methods like IDA do not consider the correlation of ground motion duration to intensity as it directly scales the entire record, potentially resulting in long-duration records at high-intensity levels. Long-duration ground motions, however, can subject structures to a larger number of cycles and potentially result in accumulated damage, especially when nonlinear behavior is present. By considering ground motion duration in the hazard analysis, this study aims to improve the accuracy of performance predictions for the major bridge and the typical highway overpass.

Multiple suites of ground motion records were chosen using both the state of practice for accounting for duration and using the ground motion duration correlation to spectral acceleration models developed by Slate Geotech Lopez Ruiz et al. (2024 - in submission). The first suite was chosen using non-correlated durations, magnitude, distance, and other secondary earthquake parameters. The first set consists of directly scaling ground motions in the non-correlated suite. The second set of ground motions was formed by selecting six additional suites based on a uniform hazard spectrum scaled to different scale factors considering the duration correlation proposed by Slate Geotechnical Consultants. The bridge performance was compared using the fragility curves resulting from these ground motions to assess the impact that correlation coefficients have on the probability of reaching DS-3 and DS-5 for the major bridge.

The performance of the bridge was evaluated by comparing the fragility curves of the model subject to the non-correlated and the duration correlated ground motions. For both DS-3 and DS-5 the median probability of attaining a damage state was observed at a higher intensity measure for the suite of ground motions considering a duration correlation when compared to the standard IDA. The mean probability of reaching DS-3 for the non-correlated and the correlated shows a 29.2% increase in expected performance. While the median probability of reaching DS-5 happens at an intensity value 37.4% larger in expected performance under traditional analysis. This shift in the median value of fragility curves shows that the standard IDA leads to an underestimation of the bridge performance.

The second case study investigates the effects of accounting for changes in isolation bearing behavior due to rotation of end-plates during seismic loading. Previous studies have shown that the rotation of bearing end-plates leads to a reduction in the horizontal stiffness of isolation bearings. The study utilizes preliminary results from a hybrid simulation experiment to capture changes in bearing behavior due to rotational demands. Based on the analysis results, damage fragility curves are developed for moderate and major damage states and compared to the traditional analysis using an IDA and assuming parameters based on parallel end-plate conditions.

The performance of the bridge considering bearing rotation was evaluated by comparing the fragility curves for the structural model with updated bearing parameters to the original estimate. From the fragility curves, the median probability of DS-3 and DS-5 happen at a 1.5% and 4.0% lower IM, respectively. This indicates a reduction in the bridge performance when bearing rotations are considered.

A summary of the effects of both case studies is summarized in Table 4.1. This study shows that the changes in the hazard analysis due to ground motion duration have a larger impact on bridge performance than the bearing rotation effects. By examining the two case studies, this study enhances our understanding of the major bridge performance assessment within the PBEE framework. It sheds light on the influence of ground motion duration and bearing rotation on structural damage expected. The findings contribute to more accurate performance predictions and inform decision-making for bridge performance analyses.

4.3 Future Research and Recommendations

While this study provides valuable insights into the implications on bridge performance of selecting different hazard and modeling assumptions, further research is needed for generalized results. Some future work is suggested:

1. Both the 3D and the 2D numerical models make significant simplifications in the modeling of the abutment and foundations. While these serve well their function for comparison purposes in this study, they can be improved to more reliably determine the performance expected for each case
2. Regarding the methodology's damage analysis stage, the bearing damage metrics could be significantly improved with experimental data and the development of fragility curves specific to isolation bearings.
3. For the bearing rotation investigation discussed in Chapter 3, the conclusions are specific to the major bridge selected for this study. It is expected that the influence of bearing rotational behavior on bridge performance will be different for other configurations. As such, other archetype bridges should be developed and evaluated to determine the influence of bearing rotation on performance.
4. The proposed bearing model in Section 3.4 was developed with a capacity of 3 degrees-of-freedom. Future work could expand this model to 6 DOF to account for bearing rotational behavior in 3D bridge models.

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Appendix A. Ground Motion Tables

The tables here show a summary of the ground motion records used in the correlated and non-correlated suites for analysis at each hazard level.

Table A.1 Non-correlated suite of ground motions for 2475-year hazard. .

RSN	Earthquake Name	Station Name	Mw
828	Cape Mendocino	Petrolia	7
1158	Kocaeli_ Turkey	Duzce	7.5
1509	Chi-Chi_ Taiwan	TCU074	7.6
1787	Hector Mine	Hector	7.1
3748	Cape Mendocino	Ferndale Fire Station	7
3934	Tottori_ Japan	SMN002	6.6
3965	Tottori_ Japan	TTR008	6.6
4031	San Simeon_ CA	Templeton - 1-story Hospital	6.5
5818	Iwate_ Japan	Kurihara City	6.9
6890	Darfield_ New Zealand	Christchurch Cashmere High School	7
8063	Christchurch_ New Zealand	Christchurch Botanical Gardens	6.2

Table A.2 Correlated suite of ground motions for 2475 -year scale of 2.0 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
1116	Kobe_ Japan	Shin-Osaka	6.9
1120	Kobe_ Japan	Takatori	6.9
1158	Kocaeli_ Turkey	Duzce	7.5
1492	Chi-Chi_ Taiwan	TCU052	7.6
3748	Cape Mendocino	Ferndale Fire Station	7
4874	Chuetsu-oki_ Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki_ Japan	Kashiwazaki NPP_ Unit 1: ground surface	6.8
6962	Darfield_ New Zealand	ROLC	7
8063	Christchurch_ New Zealand	Christchurch Botanical Gardens	6.2
8118	Christchurch_ New Zealand	Papanui High School	6.2

Table A.3 Correlated suite of ground motions for 2475 -year scale of 2.5 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
1116	Kobe Japan	Shin-Osaka	6.9
1120	Kobe Japan	Takatori	6.9
1158	Kocaeli Turkey	Duzce	7.5
1492	Chi-Chi Taiwan	TCU052	7.6
3748	Cape Mendocino	Ferndale Fire Station	7
4229	Niigata Japan	NIGH12	6.6
4874	Chuetsu-oki Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki Japan	Kashiwazaki NPP Unit 1: ground surface	6.8
8063	Christchurch New Zealand	Christchurch Botanical Gardens	6.2
8118	Christchurch New Zealand	Papanui High School	6.2

Table A.4 Correlated suite of ground motions for 2475 -year scale of 3.0 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
1116	Kobe Japan	Shin-Osaka	6.9
1120	Kobe Japan	Takatori	6.9
1158	Kocaeli Turkey	Duzce	7.5
1492	Chi-Chi Taiwan	TCU052	7.6
3964	Tottori Japan	TTR007	6.6
4229	Niigata Japan	NIGH12	6.6
4874	Chuetsu-oki Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki Japan	Kashiwazaki NPP Unit 1: ground surface	6.8
8063	Christchurch New Zealand	Christchurch Botanical Gardens	6.2
8118	Christchurch New Zealand	Papanui High School	6.2

Table A.5 Correlated suite of ground motions for 2475 -year scale of 3.5 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
1116	Kobe Japan	Shin-Osaka	6.9
1120	Kobe Japan	Takatori	6.9
1158	Kocaeli Turkey	Duzce	7.5
1492	Chi-Chi Taiwan	TCU052	7.6
3964	Tottori Japan	TTR007	6.6
4117	Parkfield-02 CA	Parkfield - Fault Zone 15	6
4228	Niigata Japan	NIGH11	6.6
4874	Chuetsu-oki Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki Japan	Kashiwazaki NPP Unit 1: ground surface	6.8
8063	Christchurch New Zealand	Christchurch Botanical Gardens	6.2

Table A.6 Correlated suite of ground motions for 2475 -year scale of 4.0 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
1111	Kobe Japan	Nishi-Akashi	6.9
1116	Kobe Japan	Shin-Osaka	6.9
1158	Kocaeli Turkey	Duzce	7.5
1492	Chi-Chi Taiwan	TCU052	7.6
3964	Tottori Japan	TTR007	6.6
4117	Parkfield-02 CA	Parkfield - Fault Zone 15	6
4228	Niigata Japan	NIGH11	6.6
4874	Chuetsu-oki Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki Japan	Kashiwazaki NPP Unit 1: ground surface	6.8
8063	Christchurch New Zealand	Christchurch Botanical Gardens	6.2

Table A.7 Correlated suite of ground motions for 2475 -year scale of 4.5 hazard.

RSN	Earthquake Name	Station Name	Mw
173	Imperial Valley-06	El Centro Array #10	6.5
768	Loma Prieta	Gilroy Array #4	6.9
1111	Kobe_ Japan	Nishi-Akashi	6.9
1116	Kobe_ Japan	Shin-Osaka	6.9
1158	Kocaeli Turkey	Duzce	7.5
3964	Tottori_ Japan	TTR007	6.6
4117	Parkfield-02_ CA	Parkfield - Fault Zone 15	6
4228	Niigata_ Japan	NIGH11	6.6
4874	Chuetsu-oki_ Japan	Oguni Nagaoka	6.8
4894	Chuetsu-oki_ Japan	Kashiwazaki NPP Unit 1: ground surface	6.8
8063	Christchurch New Zealand	Christchurch Botanical Gardens	6.2

Appendix B. Major Bridge – Non-Correlated Sets Fragility Curves.

Supplementary fragility curves to section 4 are presented resulting from the non-correlated data set and the correlated data set.

Appendix B.1 Non-correlated sets.

The plots show the observed fractions of reaching a damage state against the scale factor of each suite of ground motions, along with the curve fitted to the data for each element. Included here fragility curves for isolated piers and non-isolated piers at DS-5.

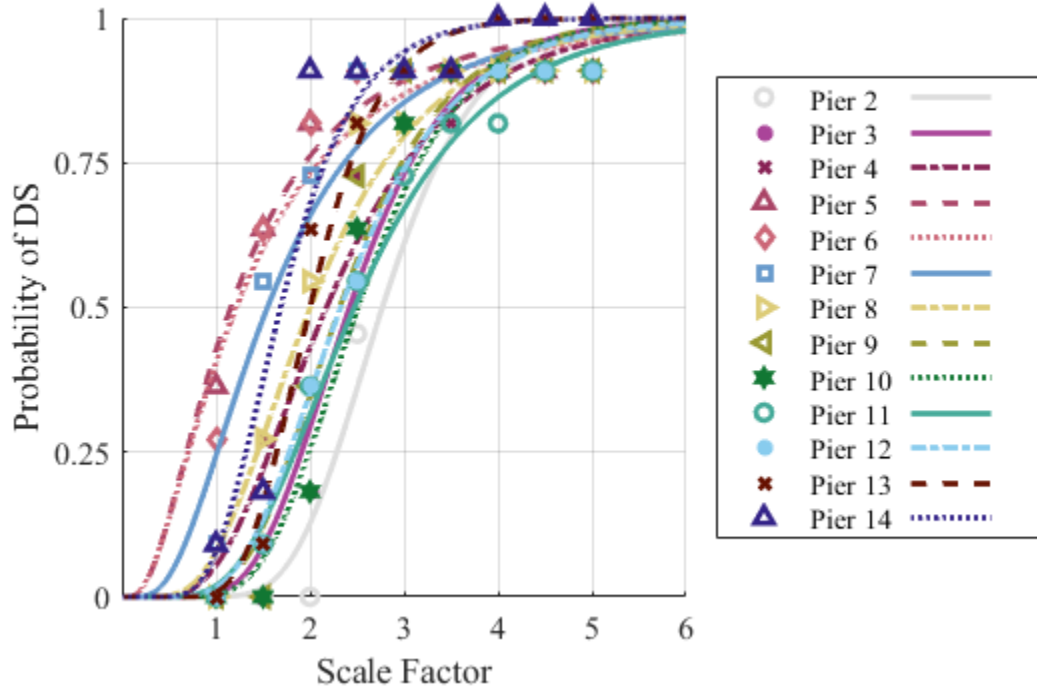


Figure B.1 DS-5 probability for bearings on each isolated pier with fitted fragility curves under non-correlated ground motions.

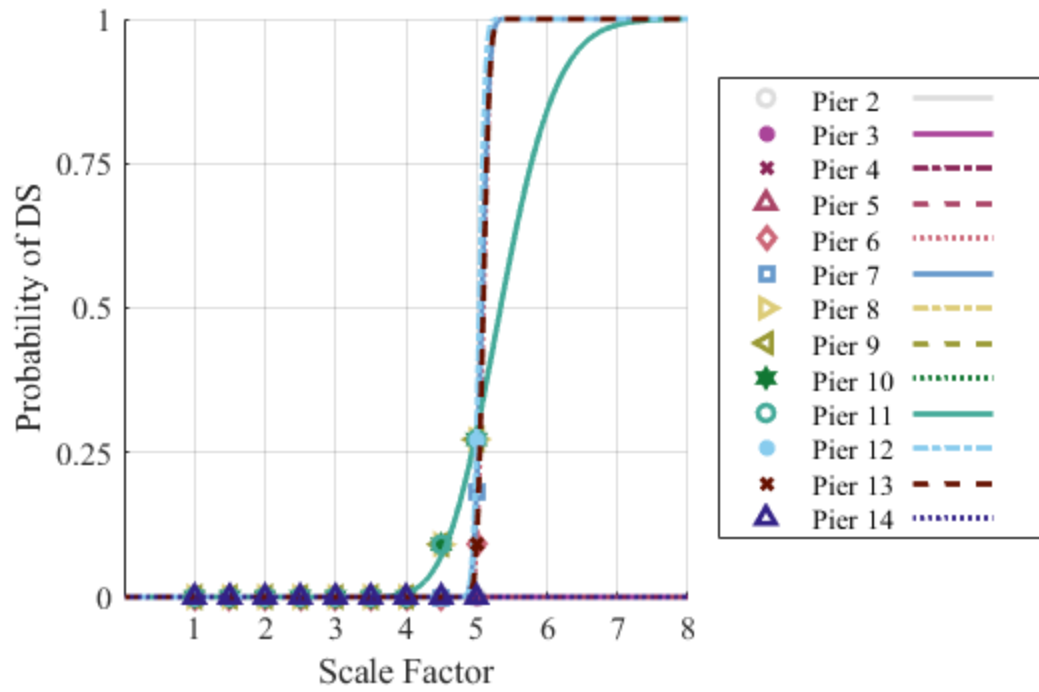


Figure B.2 Major bridge, isolated piers damage probability with fitted fragility curves for DS-5 under non-correlated ground motions.

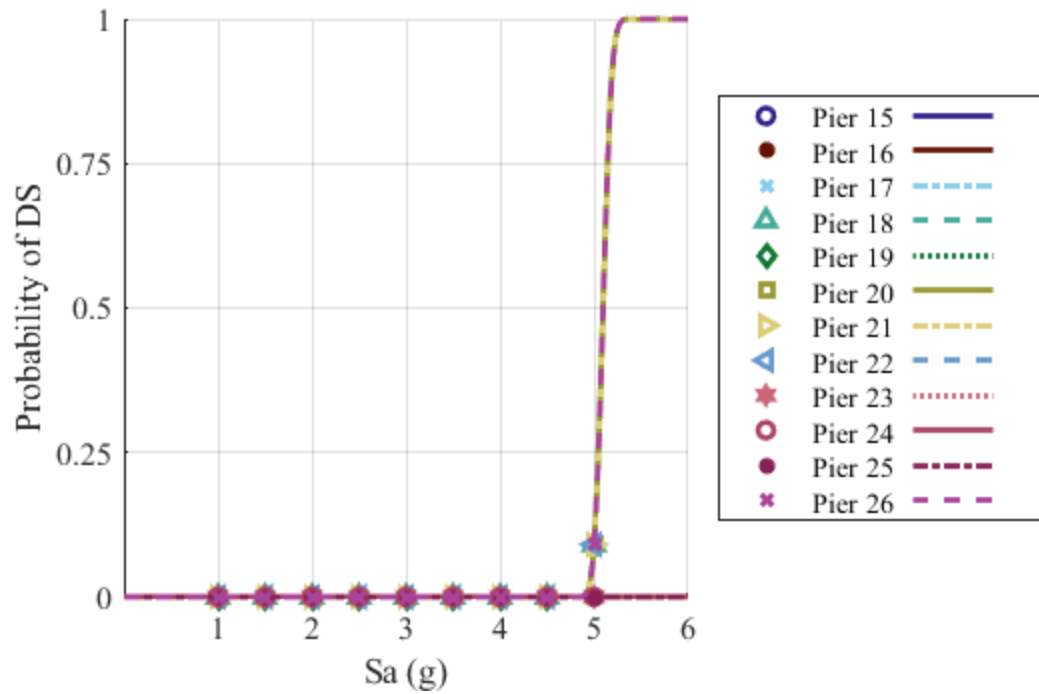


Figure B.3 Major bridge, non-isolated piers damage probability with fitted fragility curves for DS-5 under non-correlated ground motions.

Appendix B.2 Correlated Sets.

The plots show the observed fractions of reaching a damage state against the scale factor of each suite of ground motions, along with the curve fitted to the data for each element. Included here are fragility curves for isolated piers and non-isolated piers at DS-3 and all fragility curves for DS-5.

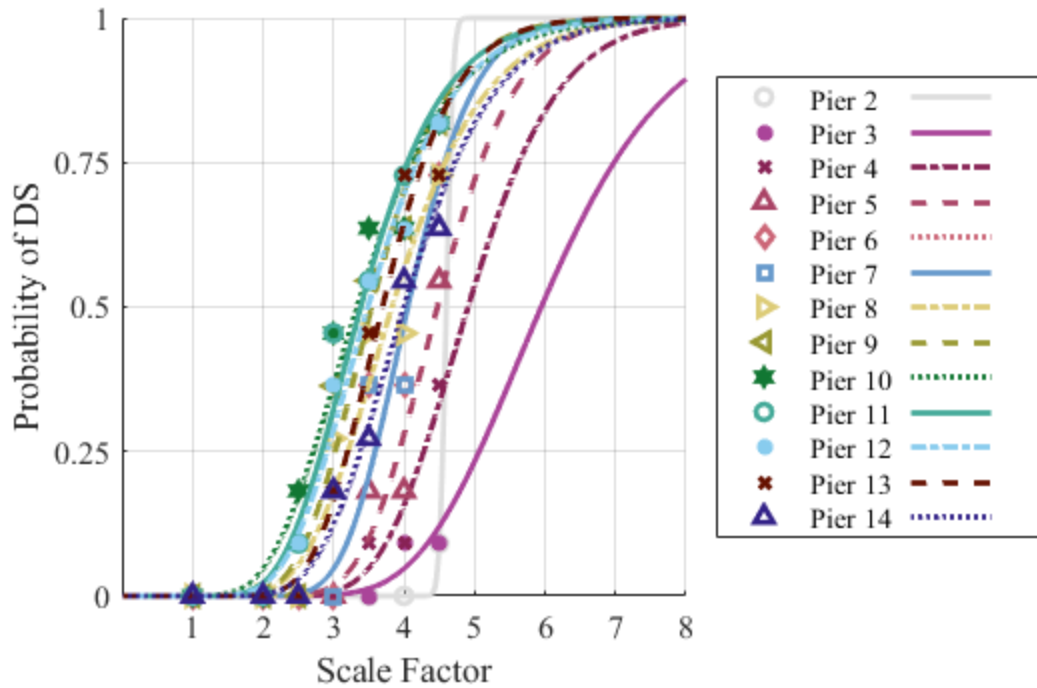


Figure B.4 Major bridge, isolated piers damage probability with fitted fragility curves for DS-3 under correlated ground motions.

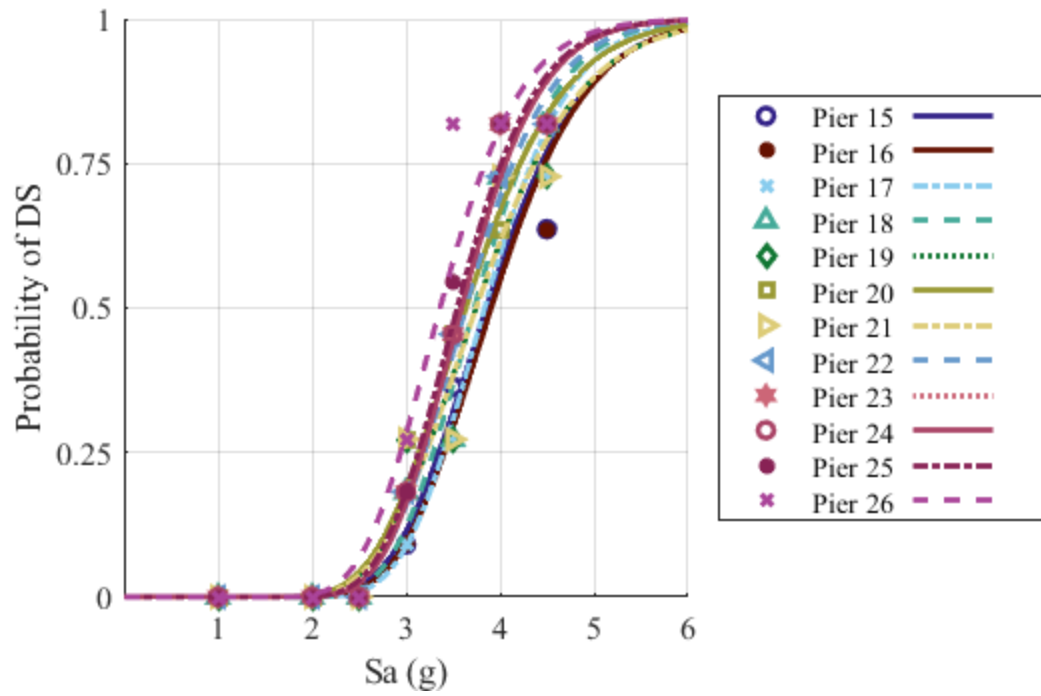


Figure B.5 Major bridge, non-isolated piers damage probability with fitted fragility curves for DS-3 under correlated ground motions.

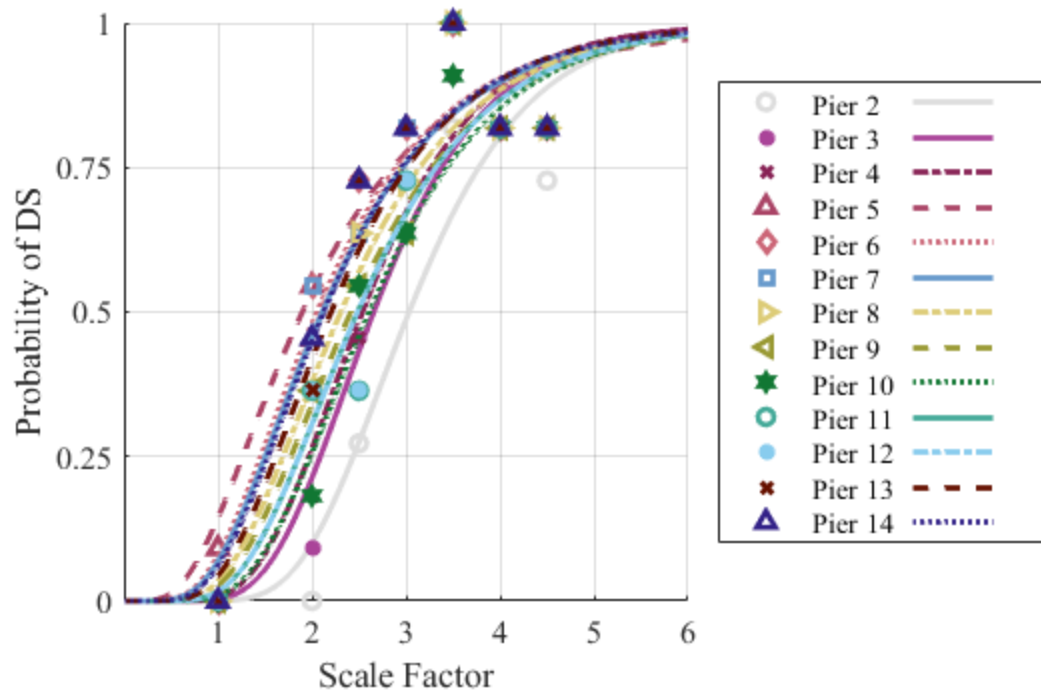


Figure B.6 Major bridge, bearings on each isolated piers with fitted fragility curves for DS-5 under correlated ground motions.

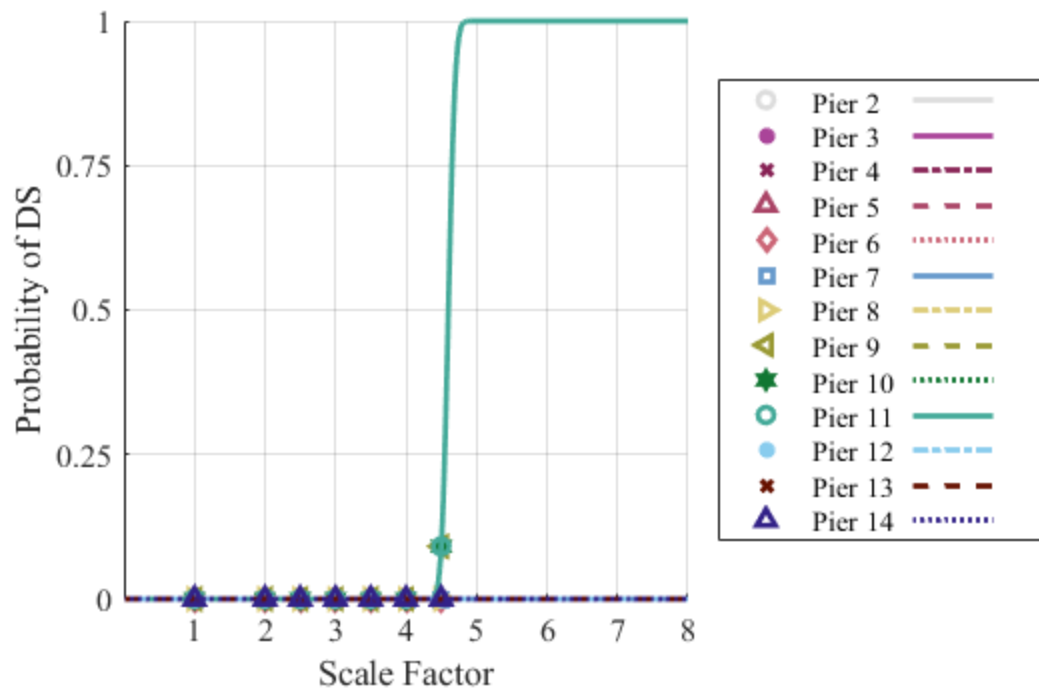


Figure B.7 Major bridge, isolated piers damage probability with fitted fragility curves for DS-5 under correlated ground motions.

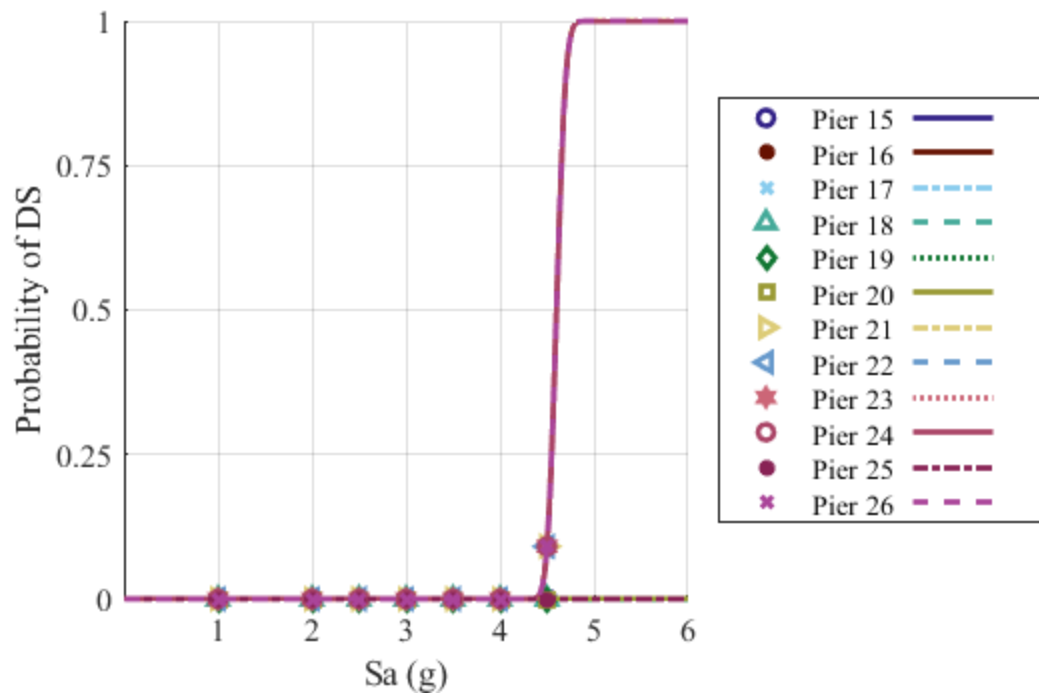


Figure B.8 Major bridge, non-isolated piers damage probability with fitted fragility curves for DS-5 under correlated ground motions.