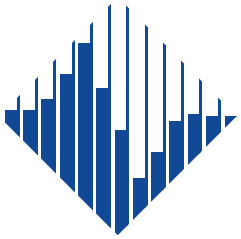

PACIFIC EARTHQUAKE ENGINEERING
RESEARCH CENTER

Aftershock Seismic Vulnerability and Time-Dependent Risk Assessment of Bridges

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PEER Report No. 2019/04
Pacific Earthquake Engineering Research Center
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PEER 2019/04
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ABSTRACT

The time-dependent seismic risk of bridges is assessed while accounting for the effect of aftershocks and the uncertainty in the damage state after a mainshock event. To achieve this, a Markov risk-assessment framework is adopted to account for the probabilistic transition of the bridge structure through different damage states and time-dependent aftershock hazard. The methodology is applied to three typical California bridge configurations that differ only based on their era of design and construction. Era 11, Era 22, and Era 33 designations are used for the three bridges, which are designed and detailed to reflect pre-1971, 1971–1990 and post-1990 construction. In addition to mainshock-only evaluations (used as a benchmark to quantify the additional risk posed by aftershocks), pre-mainshock (to account for the uncertainty in the occurrence of mainshock and aftershock events) and post-mainshock (which are based on a conditioning mainshock event and bridge damage state) seismic risk assessments are performed. To support these assessments, a set of 34 pairs of ground motions from as-recorded mainshock–aftershock sequences is assembled. Sequential nonlinear response history analyses (including incremental dynamic analyses) are used to obtain the response demands when the structural models of all bridges are subjected to mainshock-only records or mainshock–aftershock record-pairs. Physical damage in both the mainshock and mainshock–aftershock environments is defined using the following mutually exclusive and collectively exhaustive limit states: Intact, Slight, Moderate, Extensive, and Complete. For both the pre- and post-mainshock assessments, the additional risk caused by aftershock hazard is found to be higher for the older bridges (i.e., Era 11 and Era 22) and more severe conditioning damage states. A direct correlation between the bridge's age and the increase in seismic risk due to aftershock hazard was also observed for the pre-mainshock assessment. It is suggested that the proposed methodology be used to make informed decisions regarding the appropriateness and timing of bridge closures (partial and complete) following a seismic event while considering aftershock hazard.

ACKNOWLEDGMENTS

This work was funded under the PEER TSRP Fund No. 1137-NCTRHB from the State of California's Transportation Division-State Highway Fund, via the University of California Office of the President. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the sponsors, the Pacific Earthquake Engineering Research Center (PEER), or the Regents of the University of California.

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1 Introduction

Bridges are an essential part of transportation systems, and the mobility of emergency responders in the event of a seismic event is highly dependent on a functioning transportation network. Determination of the structural integrity and functionality of earthquake-damaged bridges is a critical part of post-event response and recovery. Due to their high rate of occurrence within a short time window after the causative mainshock, aftershocks could further hamper the functional restoration of bridges and, in addition, threaten their structural integrity after their lateral load-carrying capacity is reduced under the mainshock.

Currently, the California Department of Transportation (Caltrans) uses a set of system-level damage states as the basis for classifying the post-earthquake operability of bridges. The damage states are based on HAZUS [FEMA 2012] classifications (minor, moderate, extensive, and complete), and each one is assigned a "likely post-event traffic state" [Mangalathu et al. 2016; Mangalathu 2017]. Despite HAZUS being the primary tool used as the basis for post-earthquake decisions regarding the partial or complete closure of bridges, the extent to which knowledge of residual structural capacity and time-dependent aftershock hazard and risk, and how these factors inform these damage-traffic-state relationships is unclear.

Although significant advances have been made in estimating bridge seismic vulnerability and risk under mainshock events (e.g., Mackie and Stojadinovic [2001]; Nielson [2005]; Mackie and Stojadinović [2006]; Banerjee and Shinozuka [2008]; Zhang et al. [2008]; Vosooghi and Saiidi [2010]Ramanathan [2012]; Mangalathu et al. [2017]; Soleimani et al. [2017]; Mangalathu and Jeon [2018]; and Mangalathu et al. [2018]), research to quantify the time-dependent vulnerability and risk in the aftershock environment is still in its infancy. A few studies have been carried out to estimate the vulnerability of bridges that have been damaged by mainshocks [Franchin and Pinto 2009; Alessandri et al. 2013; Ghosh et al. 2015; Jeon et al. 2016; and Omranian et al. 2018].

Franchin and Pinto [2009] suggested an approach to determine when to open or close mainshock-damaged bridges in the presence of aftershock hazard by estimating the transition probability from a mainshock damage state to collapse state. The authors combined mainshock and aftershock fragilities with probabilistic seismic hazard analysis (PSHA) to assess the collapse risk, which was used as the basis for closure decisions; this methodology was applied in a case study of a viaduct bridge in Italy.

Alessandri et al. [2013] proposed a methodology for evaluating the feasibility of continuing bridge operations after a mainshock while accounting for the effect of aftershocks. The damage state of the bridge following the mainshock is estimated by combining the analytical fragility curves with information gleaned from *in situ* visual inspections. The authors proposed usability criteria for the bridge based on the expected aftershock risk. For California bridges, Jeon et al. [2016] investigated the effect of successive earthquakes on mainshock-damaged bridges. The study concluded that the additional damage to columns caused by aftershocks is more pronounced when the bridge undergoes higher mainshock damage states. The authors also evaluated the effect of various retrofitting strategies for mitigating aftershock vulnerability.

Recently, Omranian et al. [2018] investigated the effect of skew angle and ground-motion orientation on the fragility curves of skewed bridges under mainshocks and aftershocks. The study revealed that the skew angle and angle of incidence have a significant influence on the aftershock fragility curves of a California bridge. Note, to date the aftershock studies for bridges in California have been limited to two-span bent bridges, and there have been no investigations on the effect of time-dependent aftershock hazard on the seismic risk. As noted in Mangalathu [2017], single-column bent bridges occupy a significant portion of the California inventory. Although several studies have highlighted the time-dependent nature of seismic hazard in the post-mainshock environment and its impact on the seismic risk evolution for buildings, none have been conducted on the time-dependent seismic risk of typical California bridges in the post-mainshock environment. The specific objectives of the research reported herein was a follows:

1. Select mainshock and aftershock ground motions for single-site time-dependent risk assessment of California bridges.
2. Investigate the seismic risk in California highway bridges under mainshock-aftershock sequences using sequential nonlinear response history analyses (RHAs). The first ground motion in the sequence will be used to induce a specified level of damage in the structure, and the second will be used to evaluate the response of the mainshock-damaged bridge. Post-mainshock states ranging from intact to imminent collapse are considered. These states will be used to define the possible damaged condition of the bridge after being subjected to mainshock or aftershock ground motions. The results from nonlinear RHAs will be used to populate the elements of a set of Markov transition matrices. These Markov matrices are central to the time-dependent bridge seismic risk analysis.
3. Explore the effect of construction era on the time-dependent seismic risk of California bridges, which can be grouped into three categories based on the adopted design philosophy: constructed before 1971 (Era 11, hereafter), constructed between 1971 through 1990 (Era 22, hereafter) and constructed after 1990 (Era 33, hereafter). To date, no studies have been carried out on the effect of these design eras on aftershock risk for bridges located in California .

The report is organized as follows: Chapter 2 outlines the ground-motion selection for mainshock and aftershock performance assessment. The seismic hazard evaluation (mainshock and aftershock) of a selected site in California is also presented. Chapter 3 outlines the proposed framework, and the nonlinear structural modeling approach is presented in Chapter 4. The results from the seismic risk evaluation are detailed in Chapter 5.

2 Ground-Motion Selection and Probabilistic Seismic Hazard Analysis

2.1 INTRODUCTION

The best way to obtain reasonable estimates of a structure's seismic response is to perform nonlinear RHAs using a representative set of ground motions based on the site of interest. The objective of the current study is to evaluate the seismic performance of individual bridges when they are subjected to a major mainshock event and subsequent aftershocks. As such, it is necessary to first recognize the likely differences in the attributes of mainshock and aftershock ground motions, and then select a representative group of record pairs.

Significant differences in the frequency content of mainshock and aftershock ground motions have been reported in literature (Chiou and Youngs [2008]; Ruiz-García and Negrete-Manriquez [2011]; and Abrahamson et al. [2013]). Several factors affect the characteristics of aftershock ground motions. Aftershocks are generally smaller in magnitude [Shcherbakov and Turcotte 2004] and, for the same site, have larger source-to-site distances (possibly as a result of having a smaller rupture area) than their causative mainshocks. Second, even when source and path differences are considered, there is evidence of a mild correlation between the characteristics of mainshock and aftershock records belonging to the same sequence [Boore et al. 2014]. Selecting mainshock-aftershock (MS–AS) record pairs from the same sequence is considered "ideal" because the inherent relationship between the sequential ground motions will be explicitly captured. When considering other constraints that might rise in selecting ground motions for sequential RHAs (e.g., matching target magnitude (**M**), distance (R), and V_{s30} to target a specific ε value from hazard deaggregation), it may not be possible to find an adequate number of record pairs from the same sequence. Selection of MS–AS record pairs that do not necessarily belong to the same sequence or pairs of mainshock-only records (which are usually much more readily available compared to aftershock records) would allow access to a broader pool of ground motions, which has obvious benefits, but at the cost of possibly not preserving the within-sequence ground-motion correlations [Boore et al. 2014].

A few studies have investigated differences in the dynamic response of structures subjected to mainshock-mainshock (MS–MS) versus MS–AS ground-motion sequences. Goda [2012] compared the ductility demands imposed by MS–MS and MS–AS sequences. The MS–

MS sequences were selected such that the distribution of the magnitudes of the events producing the second motion in the pair would match the aftershock magnitude distribution predicted by Omori's law [Utsu et al. 1995]; the effects of different rupture distances were not considered. A third foreshock–MS–MS sequence was also considered (originally proposed by Hatzigeorgiou and Beskos [2009]) in which the foreshock and aftershock records are scaled versions of the mainshock records (by a factor of 0.85). The probability distribution of the peak ductility demands developed for a set of single-degree-of-freedom (SDOF) structures with different periods showed slight differences between responses obtained from the MS–AS record pairs and the MS–MS sequences generated based on Omori's law; the triad sequence produced significantly higher peak ductility demands. In a separate study, Goda [2015] compared the collapse performance of a two-story wood-frame building under both MS–MS and MS–AS ground-motion pairs. In contrast to the earlier study [Goda 2012], the MS–MS sequence used the same records in the second event as in the first. Not surprisingly, the MS–MS sequence produced higher collapse probabilities than the one with MS–AS records. Ruiz-García [2012] conducted a similar study considering two low- and mid-rise steel frames, and reached the same conclusion.

The difference in the frequency content of mainshock and aftershock ground motions has also been investigated in the development of recent ground-motion models (GMMs). Using a systematic approach to compare ground motions that allows for controlling for source and site characteristics (e.g., **M** and R), Boore et al. [2014]) examined the correlation of event terms from parent mainshocks and their children aftershocks using the PEER NGA-West 2 database [Chiou et al. 2008]. A mild correlation between the event terms was observed. No adjustment for aftershocks was included in the final functional form of their GMM as the difference between the event terms of mainshocks and average of event terms from aftershocks was practically zero and independent of magnitude. For the original NGA project [Chiou et al. 2008], functional forms used in some of the GMMs included a term to distinguish mainshocks from aftershocks. Abrahamson et al. [2013] found that the median spectral values of aftershocks at short periods were smaller than those from similar mainshocks, whereas the aftershock spectral ordinates were larger at longer periods (> 0.75 sec).

As shown in Figure 2.1, Chiou and Youngs [2008] reached a similar conclusion, but the transition from lower to higher spectral ordinates (of aftershocks relative to mainshocks) was about 2.0 sec. Such a relationship between S_a values of mainshocks and aftershocks needs to be viewed in the context of the different **M** and R expected for these two event types. Whereas aftershock demands will generally be smaller (due to lower **M** and larger R), their spectral shape is different from that of mainshocks; see Figure 2.1. Chiou and Youngs also found that the style of faulting for aftershocks had a smaller influence on the predicted spectral values when compared to mainshocks, but the depth to top of rupture (Z_{TOR}) had a stronger influence on the predictions made for aftershocks.

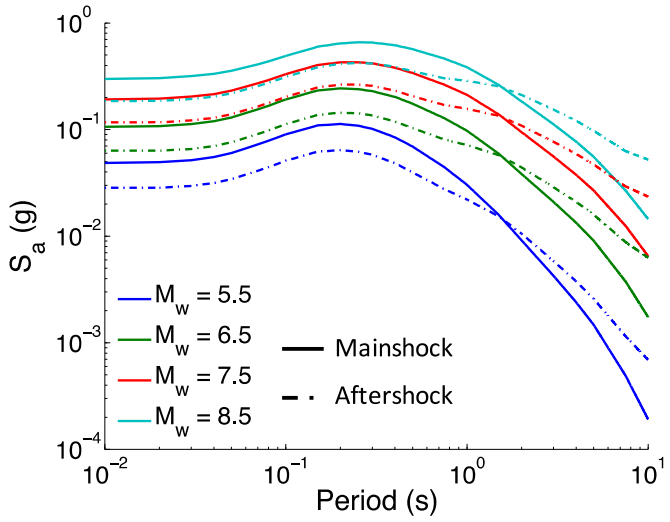

Figure 2.1 **Comparing the spectra of mainshock and aftershock records with similar magnitudes obtained from the Chiou and Youngs [2008] GMM.**

2.2 RECORD PAIRS FOR RESPONSE HISTORY ANALYSIS

Given the previously mentioned measurable differences in the frequency content of mainshock and aftershock ground motions and their implications for RHA, the mainshock–aftershock record pairs in this study are selected from ground motions that are specifically identified through the use of time and distance windowing algorithms, as the actual seismic sequences of major earthquakes [Knopoff and Gardner 1972; Wooddell and Abrahamson 2014]. The ground-motion pairs are from the Class 1 (mainshock) and Class 2 (aftershock) records of the Imperial Valley 06, Northridge, Livermore, Coalinga, Landers, Mammoth Lake, Chalfant Valley, Whittier Narrows, Darfield, Umbria Marche, and Chi-Chi earthquakes, which are available in the PEER-NGA-West2 database [Chiou et al. 2008]. A magnitude-dependent time window and a distance threshold of 40 km measured in terms of the centroidal Joyner–Boore distance [Wooddell and Abrahamson 2014] was used to identify the aftershock ground motions. The magnitudes of the events that produced the selected ground motions range from 5.80 and 7.62 for mainshocks, and 5.01 and 6.46 for aftershocks. Pulse-like ground motions were excluded from the set of 34 record pairs. Table 2.1 summarizes the properties of the records in each ground-motion sequence. The response spectra of the mainshock and aftershock ground motions are shown in Figures 2.2 and 2.3, respectively.

Table 2.1 Attributes of the mainshock-aftershock ground-motion sequences.

Sequence ID	Event name	Mainshock ground motion			Aftershock ground motion		
		M_w	R (km)	V_{s30} (m/sec^2)	M_w	R (km)	V_{s30} (m/sec^2)
1	Imperial Valley 06	6.53	22	242	5.01	24	237
2	Northridge	6.69	12	546	5.20	20	508
3	Northridge	6.69	9	356	5.93	22	450
4	Northridge	6.69	20	450	5.28	18	379
5	Northridge	6.69	10	706	5.28	14	409
6	Livermore	5.80	15	378	5.42	28	517
7	Coalinga	6.36	24	275	5.09	24	467
8	Landers	7.28	24	354	6.46	35	297
9	Mammoth Lakes 01	6.06	13	537	5.69	14	537
10	Chalfant Valley 02	6.19	22	371	5.44	24	303
11	Whittier Narrows	5.99	10	267	5.27	22	316
12	Whittier Narrows	5.99	20	321	5.27	24	297
13	Whittier Narrows	5.99	15	371	5.27	23	271
14	Whittier Narrows	5.99	18	301	5.27	21	347
15	Whittier Narrows	5.99	11	329	5.27	17	325
16	Whittier Narrows	5.99	17	241	5.27	19	345
17	Whittier Narrows	5.99	25	303	5.27	34	318
18	Whittier Narrows	5.99	19	412	5.27	29	400
19	Whittier Narrows	5.99	7	969	5.27	17	371
20	Whittier Narrows	5.99	0	401	5.27	12	545
21	Umbria Marche	6.00	19	401	5.50	36	492
22	Darfield	7.00	44	638	6.20	85	638
23	Darfield	7.00	31	255	6.20	67	561
24	Darfield	7.00	52	551	6.20	62	484
25	Chi-Chi	7.62	33	348	6.20	36	379
26	Chi-Chi	7.62	19	492	6.20	23	543
27	Chi-Chi	7.62	39	573	6.20	41	492
28	Chi-Chi	7.62	83	250	6.20	91	541
29	Chi-Chi	7.62	53	789	6.20	60	573
30	Chi-Chi	7.62	47	453	6.20	51	278
31	Chi-Chi	7.62	48	438	6.20	53	573
32	Chi-Chi	7.62	50	614	6.20	92	411
33	Chi-Chi	7.62	60	484	6.20	77	573
34	Chi-Chi	7.62	82	496	6.20	97	489

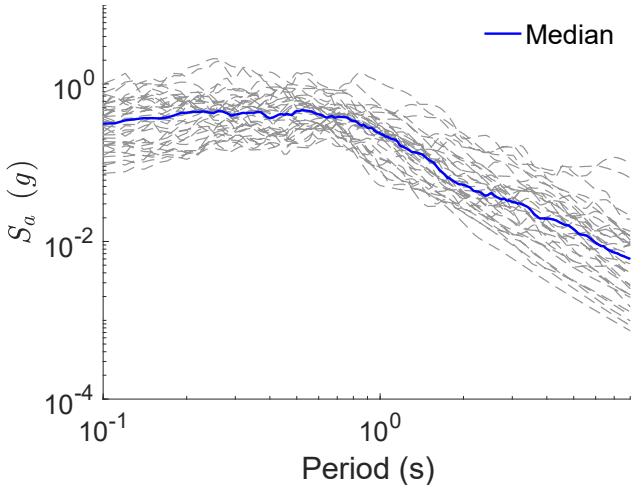

Figure 2.2 Individual and median response spectra for the mainshock ground motions.

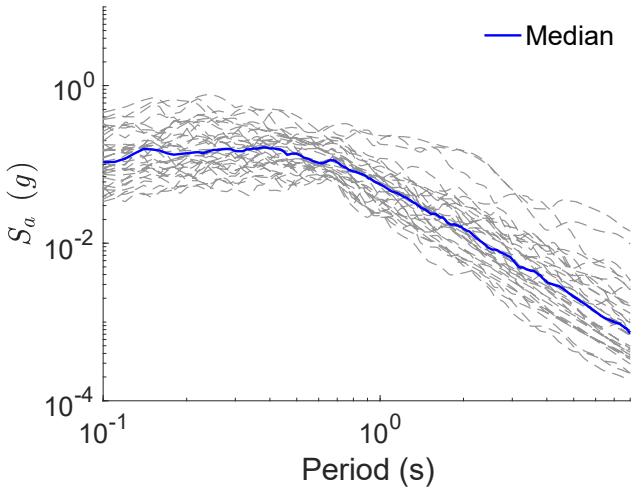

Figure 2.3 Individual and median response spectra of aftershock ground motions.

The intensity measure (IM) used to represent and scale the ground motions is their 5%-damped spectral acceleration at a period of 1 sec. Numerous prior studies have been dedicated to proposing ground-motion selection guidelines for use in nonlinear dynamic analysis, e.g., Kalkan and Chopra [2010] and ATC [2017]. These guidelines have mostly been focused on the selection criteria that would result in a less-disperse and unbiased estimate of the seismic performance of the structure being examined while minimizing the computational effort associated with the nonlinear dynamic analysis. While there are conflicting recommendations in terms of the impact of (**M**), (R), and V_{s30} of ground motions without forward-directivity effects on the outcomes of nonlinear dynamic analysis [Bommer and Acevedo 2004; Shome et al. 1998], the spectral shape parameter (ε) of a ground motion is often recognized as the parameter that has the most significant effect on structural performance [Baker and Cornell 2006; Haselton et al. 2009]. Prior studies of earthquakes that occur in California have suggested that ground motions with long return periods usually have a peaked spectral shape, with a positive ε value that ranges between 1

and 2 [Haselton et al. 2009]. If the ground motion set used in RHA is selected without properly accounting for this peaked spectral shape, the seismic performance of the examined structure could be significantly underestimated.

To ensure that the spectral shape of the ground motions used in the RHAs reflect the site of interest, the 34 record pairs were selected such that their ε values would match the ε value of the location of the bridges obtained from seismic hazard deaggregation [Bazzurro and Cornell 1999].

2.3 PROBABILISTIC SEISMIC HAZARD ANALYSIS

To develop mainshock hazard curves, conventional PSHA [Kramer et al. 1996] is performed while the aftershock hazard curves are generated using the approach suggested by Yeo and Cornell [2009]. Probabilistic seismic hazard analysis and aftershock PSHA (APSHA) are performed using source and magnitude models that are simpler than the USGS models but still account for all the sources that contribute to the seismic hazard at the site of interest. Seismic hazard analyses were performed for a high seismicity site in Southern California (33.996, -118.162). The parameters that define the spatial distribution of the earthquake magnitudes and the time-dependent aftershock rate are adopted from the California model suggested by Reasenberg and Jones [1989]. The Boore and Atkinson [2008] ground-motion prediction equation (GMPE) was used to obtain the statistical distribution of the IM conditioned on the magnitude and distance.

Following the work of Ramanathan [2012], the spectral acceleration at a period of one second, $Sa_{1.0s,}$ is adopted as the IM. The magnitude of the largest aftershock is assumed to be equal to that of the largest mainshock. The minimum magnitude is taken as 5 as events with smaller magnitudes are not likely to induce notable damage in bridge structures. The relationship that forms the basis of conventional (mainshock) PSHA is shown in Equation (2.1). v_i is the rate of occurrence of seismic events with magnitudes larger than 5.0 for source i and $G\left(IM > im \middle| M, R, \varepsilon\right)$ is an indicator function whose value is 1.0 if $IM > im$ and 0.0 otherwise. When performing APSHA, v_i is replaced with μ_{AS} for each time interval *k* from Equation (3.4).

$$\lambda_{IM}\left(IM > im\right) = \sum_{i=1}^{N_{AS}} v_i \int\int\int_{M,R,\varepsilon} G\left(IM > im \middle| M, R, \varepsilon\right) f_M(m) f_R(r) f_\varepsilon(\varepsilon) dM dR d\varepsilon \qquad (2.1)$$

The mainshock hazard curve shown in Figure 2.4 is developed for all faults contributing to the total seismic hazard at the site, whereas the aftershock hazard curve is only for the single fault that is the primary source of mainshock hazard at the assumed location. The APSHA hazard curve is limited to a single fault because the concurrent occurrence of multiple mainshocks that would trigger aftershocks on different faults is unlikely. The APSHA hazard curve is developed for a time window of one year starting immediately after the occurrence of the mainshock. A

quick comparison between the PSHA and APSHA hazard curves in Figure 2.4 reveals substantially higher aftershock hazard compared to the mainshock hazard.

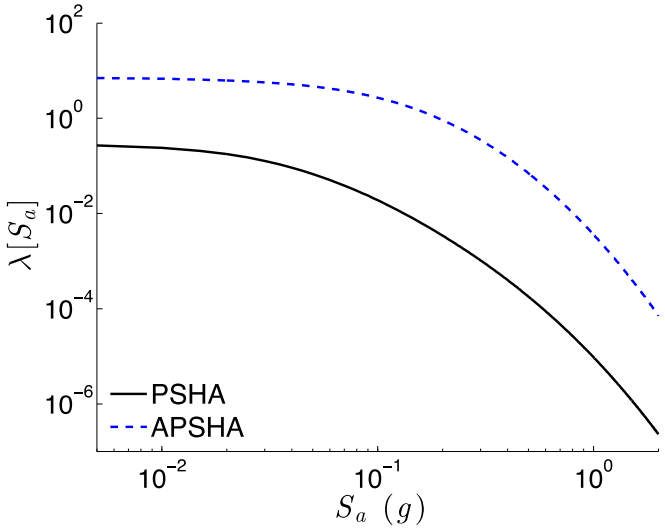

Figure 2.4 **Mainshock and aftershock (for 1 year after the occurrence of the mainshock) seismic hazard curves for *T* = 1.0 sec.**

3 Markov-Based Risk Assessment Framework

The current version of the performance-based earthquake engineering (PBEE) framework assumes that the state of a structure prior to the occurrence of a seismic event is known (usually the structure is assumed to be in the intact state), and the structure will always return to its intact state before the next earthquake occurs. When performing seismic risk analysis under major mainshock events, this is a reasonable assumption as the time interval between major mainshock events is usually long enough to allow for affected structures to be restored to their intact state.

In the post-mainshock environment, structures that have sustained some level of structural damage are usually not repaired during the short time period immediately following the mainshock when the rate of occurrence of aftershocks is highest. This, coupled with the accumulation of structural damage under multiple post-mainshock events, adds to the uncertainty in determining the state of a structure before being subjected to each subsequent aftershock. As such, evaluating the seismic performance of structures subjected to sequential seismic events requires explicit consideration of the uncertainty in the state of the structure after being subjected to any of the events in a cluster comprising a mainshock and subsequent aftershocks.

The probability of a bridge transitioning from one damage state to the next can be evaluated using a Markov process [Yeo and Cornell 2009]. A key assumption in the Markov process is that the state of the bridge under event i in the sequence is dependent only on its state under the event $i-1$, and is independent of its states under the events that precede event $i-1$. The uncertainty in the state of the bridge is considered through the Markov transition matrix [Equation (3.1)], whose elements represent the probability of transitioning between the r pre-defined limit states.

$$\prod = \begin{bmatrix} P_{11} & P_{12} & \cdots & P_{1r} \\ 0 & P_{12} & \cdots & P_{2r} \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & P_{rr} \end{bmatrix} \tag{3.1}$$

P_{ij} in Equation (3.1) is the probability that the bridge transitions from damage state i (in response to the previous event) to damage state j when subjected to a new event. The Markov

transition matrix (Π) has an upper triangular form since no repair measures are assumed to have taken place to restore the bridge to a less severe damage state. Diagonal elements in Equation (3.1) represent a "no-limit-state-transition" when the bridge is subjected to a seismic event.

To calculate the $p_{ij}, j > i$ terms of the transition matrix Π in Equation (3.1) during a time window (t_0, t_m) where t_0 is measured from the occurrence of the causative mainshock, the time window can be broken into small intervals such that the probability of occurrence of more than one aftershock during each interval is negligible. With this assumption, the P_{ij} terms for a single seismic source in each interval (t_{m-1}, t_m) of the time window (t_0, t_1) can be calculated:

$$P_{ij}(t_{m-1}, t_m) = P(N_{AS} = 1) \int \left[P_{ij}^{DS}\left(EDP > edp_j \middle| IM\right) - P_{ij+1}^{DS}\left(EDP > edp_{j+1} \middle| IM\right) \right] f_{IM}(im) d\,im \qquad (3.2)$$

where $P(N_{AS} = 1)$ is the probability of occurrence for a single aftershock during each time interval (t_{m-1}, t_m), and $P_{ij}^{DS}\left(EDP > edp_j \middle| IM\right) - P_{ij+1}^{DS}\left(EDP > edp_{j+1} \middle| IM\right)$ is the probability that the bridge would experience damage state *j* under the current seismic event given that it has already sustained damage state *i* when subjected to the preceding seismic event. $f_{IM}(im)$ is the source-specific probability density function of the intensity measure (*IM*) that links the response of the bridge to the seismic hazard at its location and can be obtained through PSHA.

It is usually assumed that the occurrence of seismic events can be characterized with a Poisson distribution. As such, $P(N_{AS} = 1)$ in Equation (3.2) can be calculated using Equation (3.3), where the mean rate of aftershocks during the time interval (t_{m-1}, t_m) $[\mu_{AS}(t_{m-1}, t_m)]$ can be obtained using Equation (3.4). More specifically, Equation (3.4) is used to calculate the mean number of aftershocks on a seismic source with minimum and maximum magnitudes of M_0 and M_m, and is obtained by combining the modified Omori's law for the daily rate of aftershocks and Gutenberg-Richter's relationship for the magnitude distribution [Reasenberg and Jones 1989; Yeo and Cornell 2009]; M_m is usually taken as the mainshock's magnitude. *a* and *b* are constants that characterize the magnitude distribution and *c* and ρ are constants that define the temporal decay in the number of aftershocks.

$$P_2(N_{AS} = 1) = \mu_{AS}(t_{m-1}, t_m) e^{-\mu_{AS}(t_{m-1}, t_m)} \qquad (3.3)$$

$$\mu_{AS}(t_{m-1}, t_m) = \left[10^{a+b(M_m - M_0)} - 10^a\right] \frac{(t_m + c)^{1-p} - (t_{m-1} + c)^{1-p}}{1-p} \qquad (3.4)$$

The decline in aftershock seismic hazard with the elapsed time since the occurrence of the mainshock [Utsu et al. 1995] means that the Markov transition matrix of Equation (3.1) will be non-stationary, with elements that can be calculated through Equation (3.5).

$$\Pi(t_{m-1},t_m)=\begin{bmatrix} P_{11}(t_{m-1},t_m) & P_{12}(t_{m-1},t_m) & \cdots & P_{1r}(t_{m-1},t_m) \\ 0 & P_{22}(t_{m-1},t_m) & \cdots & P_{2r}(t_{m-1},t_m) \\ \vdots & & \ddots & \vdots \\ 0 & 0 & \cdots & 1 \end{bmatrix} \tag{3.5}$$

At time step k after the occurrence of the mainshock, the cumulative probability that the structure is in damage state j—given that it has undergone damage state i under the mainshock—is equal to the element on row i and column j of matrix P^k in Equation (3.6).

$$P^k=\Pi_{m=1}^{k}\,\Pi(t_{m-1},t_m) \tag{3.6}$$

Equations (3.1) through (3.6) provide a probabilistic framework to assess the seismic performance of bridges in the post- mainshock environment. The framework can be easily extended to perform seismic risk assessment due to sequential events in the pre-mainshock environment following the same logic and steps mentioned earlier in this section.

Unlike the post-mainshock risk evaluation, the pre-mainshock environment considers the uncertainty in the occurrence of the mainshock and associated damage to the structure. This uncertainty can be incorporated into the Markov process by multiplying the aftershock limit state transition matrix in Equation (3.6) by a vector of p_i^{MS} values as shown in Equation (3.7). The vector of $\lambda_{i,n}^{MS}$ values represents the mean annual rate of the structure experiencing damage state $i,\ i=1,\cdots,r$ under the mainshock ground motions. The summation in Equation (3.7) accounts for all the seismic sources (N_s) that contribute to the seismic hazard at the site of interest.

$$\lambda_{\mathrm{Pre}MS}^{m}=\sum\nolimits_{n=1}^{N_s}\left[\left(\lambda_{1,n}^{MS},\ldots\lambda_{r,n}^{MS}\right)\Pi_{m=1}^{k}\,\Pi_n(t_{m-1},t_m)\right] \tag{3.7}$$

Each element of the $\lambda_{\mathrm{Pre}MS}^{m}$ vector obtained from Equation (3.7) can be used as the input for a Poisson (or exponential) distribution to calculate the limit state exceedance probabilities over any desired time window.

As shown in Equation (3.8), the $\lambda_{1,n}^{MS}$ terms can be obtained by subjecting the bridge to mainshock ground motions to induce the desired target level damage $\left[P_{MS}^{DS}\left(EDP>edp_i\middle|IM\right)-P_{MS}^{DS}\left(EDP>edp_{i+1}\middle|IM\right)\right]$)], where λ_{IM}^{n} is the mean annual frequency of exceedance of the *IM* and can be obtained through mainshock seismic hazard analysis.

$$\lambda_{i,n}^{MS}=\int\left[P_{MS}^{DS}\left(EDP>edp_i\middle|IM\right)-P_{MS}^{DS}\left(EDP>edp_{i+1}\middle|IM\right)\right]d\lambda_{IM}^{n}(im) \tag{3.8}$$

4 Bridge Nonlinear Structural Modeling and Limit State Definitions

4.1 INTRODUCTION

The aftershock risk assessment framework presented in Chapter 2 is implemented on a set of two-span single-column bent bridges with seat abutments. Two-span box-girder bridges make up a significant portion of California bridge inventory [Mangalathu 2017], and three bridges from different design eras (Era 11, Era 22 and Era 33) were considered.

4.2 NUMERICAL MODELING

The following section presents the modeling considerations for various bridge components, which can generally be categorized based on whether they are located in the superstructure or substructure. The superstructure (bridge deck) typically remains elastic during an earthquake and is therefore modeled using elastic beam–column elements; see Figure 4.1.

4.2.1 Substructure

4.2.1.1 Bents

The bents are modeled using a combination of displacement-based beam–column elements and rigid links to facilitate moment and force transfer between the members of the bent. The finite-element discretization of a single column bent is shown in Figure 4.2.

4.2.1.2 Columns

Columns are one of the most seismically vulnerable components in a bridge. In fact, most of the past earthquake-induced bridge failures have been attributed to column damage. Displacement-based beam–column elements with fiber-defined cross sections were used to model the columns; see Figure 4.2. Fiber cross sections have the distinct advantage of incorporating unique material properties for different locations across a member's cross section. For instance, confined concrete is used to represent the concrete behavior in the core section of the column, while unconfined concrete is used to represent the unconfined cover concrete. The Chang and Mander [1994] model is used to define the monotonic stress–strain curves of confined and unconfined concrete.

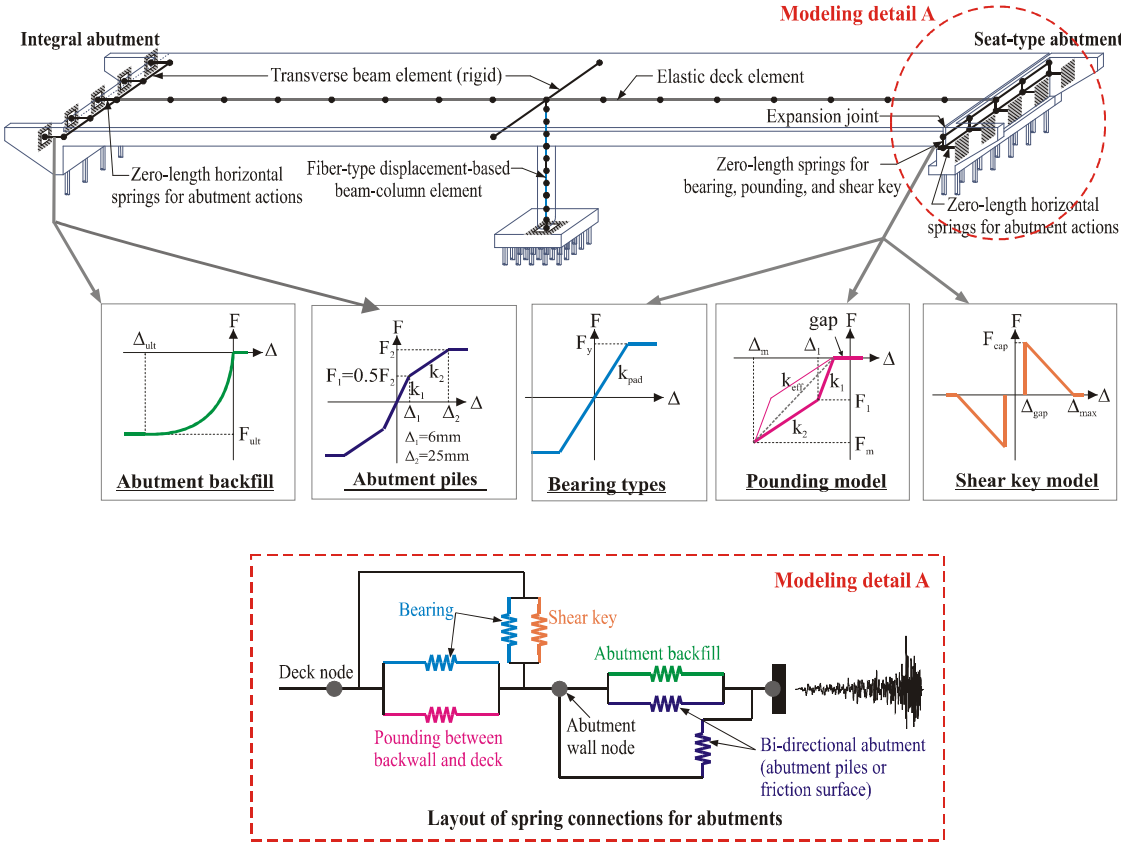

Figure 4.1 Numerical modeling of various bridge components.

4.2.1.3 Abutments

Abutments can be classified into two basic types: diaphragm abutments and seat abutments [Ramanathan 2012]. Diaphragm abutments are cast as being monolithic with the superstructure. As they engage the backfill soil during seismic action, they provide a good source of energy dissipation and reduce the likelihood of span unseating. Seat abutments provide a bearing support to the superstructure, which is restrained longitudinally by the abutment backwall and transversely by the piles and the shear key. When the deck is in contact with the abutment backwall in the longitudinal direction, the bridge's stiffness and resistance to seismic action increases. As the superstructure moves away from the abutment, the primary resistance comes from the bearing pads, making it susceptible to unseating. The backwall of the seat abutment is typically designed to fail under impact and passive response before damaging forces are transmitted to the lower portion of the abutment.

Abutment response to seismic loading is dependent on pressure from the earthquake and the structure. The pressure of soil on the abutment is due to the longitudinal response of the bridge deck and includes passive and active resistance. Passive resistance is provided by the fraction between the backfill soil and pile surface (depending on the abutment footing type). It develops when the abutment moves toward the backfill soil. These frictional forces contribute to

the active resistance, which is activated when the abutment moves away from the backwall soil. The passive response of the abutment backwall is simulated using the hyperbolic soil model proposed by Shamsabadi and Yan [2008]. The model is based on experimental testing of bridge abutments conducted at the University of California Los Angeles with 5.5-ft-high backwalls and typical non-cohesive and cohesive backfill soils. The test results were then extended to develop closed-form solutions for the abutment backfill soil response for a range of backwall heights based on a series of analyses using the limit-equilibrium method that implements mobilized logarithmic-spiral failure surfaces coupled with a modified hyperbolic soil stress–strain behavior.

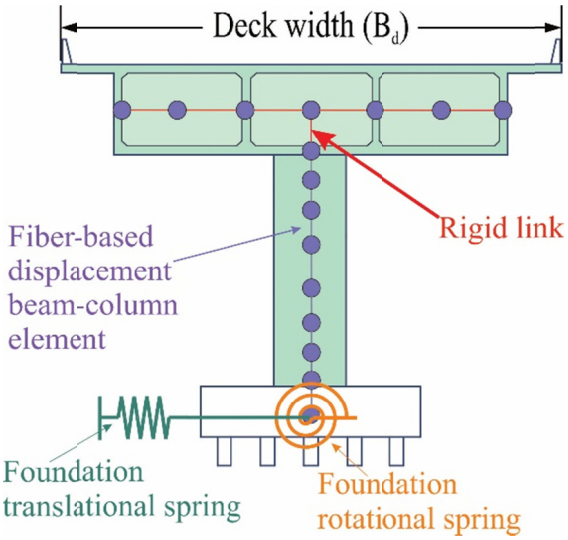

Figure 4.2 Finite-element discretization of a single-column bent.

4.2.1.4 Abutment Piles

Abutment piles provide longitudinal and transverse stiffness to the abutments. The trilinear force-deformation response of the pile, along with the associated modeling parameters, are presented in Figure 4.1. The initial yield parameters (Δ_1, F_1) are determined based on the design recommendations of the Caltrans 2014 draft of bridge design aids on *Permissible Horizontal Loads for Standard Plan and Steel HP Piles* [Mangalathu 2017]. The plastic yielding parameters (Δ_2, F_2) are calculated based on the results from modeling various pile systems simulated in LPILE [Mangalathu 2017]. The pile hysteretic behavior is captured using the *Hysteretic* material in OpenSees with the hysteretic parameters *pinchX* and *pinchY* set to 0.75 and 0.5, respectively [Ramanathan 2012].

4.2.1.5 Elastomeric Bearings

Elastomeric bearings decouple the superstructure from the substructure, which makes the superstructure susceptible to large deformations. The elastomeric bearing is assumed to be elasto–plastic, and the yield force, F_y, is obtained by multiplying the normal force by the

coefficient of friction. The force-deformation response of the elastomeric bearing is shown in Figure 4.1.

4.2.1.6 Shear Keys

Shear keys help restrain the relative transverse movement between the deck and the bridge abutments. Earthquake-induced shear key can result from one of four mechanisms: shear friction, flexure, shear, and bearing [Megally et al. 2002]. The shear-key designs are categorized as isolated (emerging designs) or non-isolated (conventional designs) [Mangalathu 2017]. Since the isolated shear key is a new type of design and does not exist in the current inventory, this study will only focus on the non-isolated shear keys. The nonlinear model of the shear key is depicted in Figure 4.3. F_{cap} denotes the capacity of the shear key, which is computed as the product of the dead-load reaction and the acceleration [Caltrans 2012]. Megally et al. [2002] conducted a series of experiments on shear keys and found that $\Delta_{max} - \Delta_{gap}$ equal to 3.5 in. is the deformation at which the capacity of the shear key essentially degrades to zero.

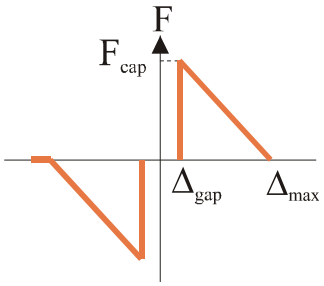

Figure 4.3 Shear key model.

4.2.1.7 Pounding

Seismic pounding refers to impact between bridge decks, between the deck and the abutment, or between the adjacent decks in a multi-frame bridge in the longitudinal direction. Impact occurs when the relative displacement between adjacent decks or the deck and the abutment exceeds the gap separating them. Significant pounding damage was observed at the I-5/SR-14 interchange during the 1994 Northridge, California, earthquake [Muthukumar and DesRoches 2006]. The contact element developed by Muthukumar and DesRoches [2006] was used to model the pounding between the superstructure and abutments. This material model explicitly accounts for the loss of hysteretic energy; see Figure 4.4. The maximum deformation, Δ_m, is assumed to be 1.0 in. The yield deformation, Δ_1, is assumed to be $0.10\Delta_m$. The stiffnesses, k_1 and k_2, are recommended to be 1022.3 kip/in./ft and 351.755 kip/in./ft, respectively.

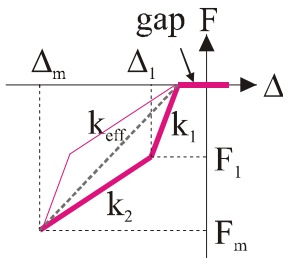

Figure 4.4 Analytical model of pounding between the deck and abutment back wall.

4.2.1.8 Foundation

The foundation transmits service and ultimate loads from the structure to the underlying soil. Foundations can be classified as either shallow or deep. As the name implies, for shallow foundations, the loads from the structure are transferred to the underlying soil at shallow depths. Deep foundations (piles) are used when soil conditions are not favourable to shallow foundations. The type of bridge foundation is determined based on various factors, e.g., the condition of the soil, overhead clearance, existing utilities, and proximity to existing facilities such as buildings and railroads [Mangalathu 2017]. For the bridges considered in the current study, foundations are modeled using elastic translational and rotational springs placed at the base of the column.

The numerical models for the various components are integrated to produce a global analytical model of the bridge. Displacement-based beam–column elements are used to model the columns. Translational and rotational springs are added to the base of the column to simulate the behavior of the footing. Zero-length elements capturing the response of the abutment backfill soil under bi-directional loading (abutment piles or frictional surface) are connected in parallel and to the transverse deck elements in the case of diaphragm abutments. Bearing-pad elements and pounding elements are also modeled with zero-length springs and are connected in parallel to the abutment springs in the case of seat abutments.

4.3 CASE STUDY BRIDGE CONFIGURATIONS

Figure 4.5 shows the details of the selected bridge and the associated geometric properties. The bridge has a deck width of 9.9 m with a span length of 30.5 m, and is supported by seat abutments with cast-in drilled concrete piles (with stiffness per deck width of 116 KN/m). The abutments are 7.5 m high , and elastomeric bearings are used to support the superstructure. Following the work of Mangalathu [2017], the coefficient of friction and stiffness of the bearing per deck are estimated as 0.30 and 1060 N/mm/m, respectively. The diameter of the column is 1524 mm with a longitudinal reinforcement ratio of 0.02 (36 No. 11 bars). The transverse reinforcement in the Era 11 bridge includes No.5 bars with a pitch of 300 mm. In Era 22 and Era 33, the spacing of the transverse reinforcement is 90 mm and 75 mm, respectively.

The concrete compressive strength in the superstructure and column is 33.0 MPa, and the yield strength of the reinforcement is 465 MPa. The mass density of the concrete and damping

ratio is assumed as to be 2400 kg/m^3 and 4.5%, respectively. The translational and rotational stiffness of the foundation is 300 kN/mm and 4.5 × 10^9 N-m/rad. For the Era 11 bridge, the gap between the abutments and the deck is 21 mm. For the same bridge, the gap between the shear key and the superstructure is 12.5 mm. The shear keys at the abutments are designed to resist a force of 1g times the superstructure dead-load reaction at the abutments. Interested readers are directed to Mangalathu [2017] for an evaluation of the stiffness and strength properties of various bridge components.

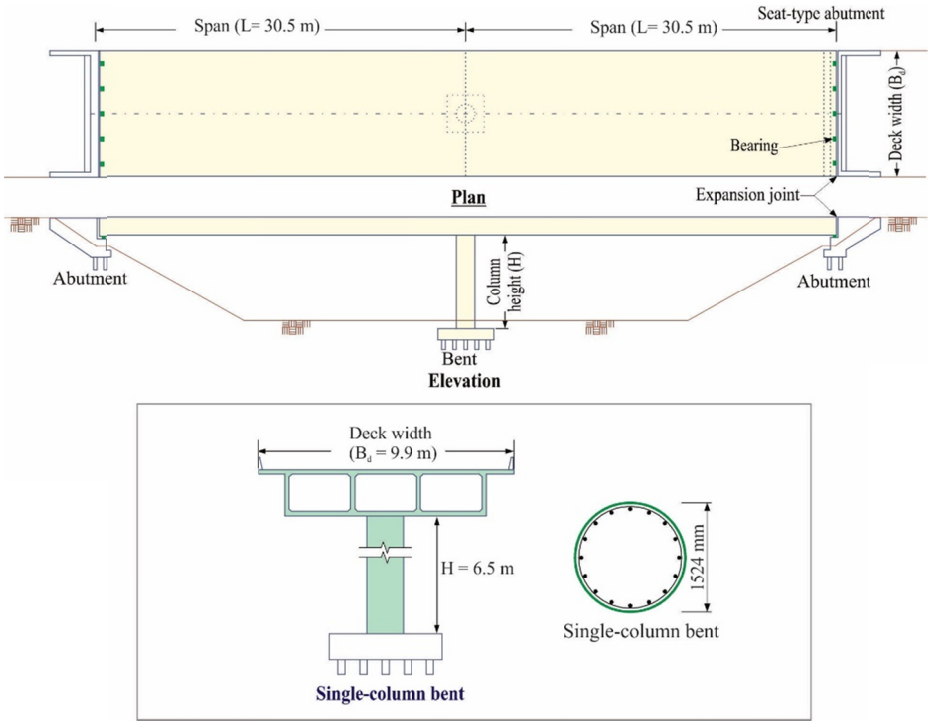

Figure 4.5 Schematic illustration of the bridge layout.

4.4 LIMIT STATES: DEFINITIONS

The description of the bridge column limit states are provided in Table 4.1. The column curvature ductility, which is used to link response demands to the physical damage descriptions, is assumed to follow a lognormal distribution with the median values and dispersions shown in Table 4.1. The dispersion, β, is taken as a constant (0.35) for all eras [Ramanathan 2012; Mangalathu et al. 2016; Jeon et al. 2016; and Mangalathu 2017].

Table 4.1 Limit state descriptions for bridge columns [Mangalathu 2017].

Limit state	Column state	Column damage	Column repair	Column curvature ductility			
				Median value			Dispersion (β)
				Era 11	Era 22	Era 33	
Slight	None or aesthetic	EQ-related minor cracking	Seal and paint	0.8	1.0	1.0	0.35
Moderate	Minor repairs needed	Minor spalling of cover concrete	Epoxy inject, minor removal/patch	2.0	5.0	5.0	0.35
Extensive	Major repairs needed, but function intact	Exposed core, confinement yield	Major removal/patch. Add jacket.	5.0	8.0	11.0	0.35
Complete	Irreparable damage, function compromised	Bar bucking, large drift, core crushing	Remove/Replace column (or bridge)	8.0	11.0	17.0	0.35

5 Demonstration of the Proposed Framework

5.1 MAINSHOCK AND AFTERSHOCK FRAGILITY CURVES

Incremental dynamic analyses are performed on the bridge structural models, and the response demands are used to develop fragility curves. The demand at each IM is convolved with the capacity estimates to compute the probability of limit state exceedance at that IM. The fragility curve parameters (median and log-standard-deviation) are then obtained by applying the maximum likelihood method [Baker 2015]. The aftershock fragility curves are generated by performing sequential nonlinear RHAs, where the bridge models are subjected to the mainshock-then-aftershock ground motions. Within this sequence, the mainshock ground motions are scaled to target specific damage levels: Slight, Moderate, and Extensive as described in Table 5.1.

Figure 5.1 shows how the probability of exceedance for the Complete limit state is affected by mainshock damage. More specifically, Figure 5.1(a), 5.1(b), and 5.1(c) compare the aftershock fragility for the Complete limit state under Slight, Moderate, and Extensive mainshock damage, respectively. As shown in Figure 5.1(a), if Slight damage occurs under the mainshock, the effect on aftershock vulnerability for the Complete limit state is negligible for the Era 33 and Era 22 bridges. In contrast, if Era 11 bridge incurs Slight damage following the mainshock, the median spectral capacity for the Complete limit state is reduced by about 7% compared to when the bridge is intact following the mainshock. If the damage caused by the mainshock is Moderate or Extensive, the increase in vulnerability is measurable for all three Eras. For the Moderate damage state, (Figure 5.1b), the median spectral intensity corresponding to Complete damage is reduced by 17%, 9% and 5% compared to the Intact state (the bridge remains undamaged when subjected to mainshock), for Eras 11, 22, and 33, respectively. For Extensive mainshock damage, the reduction relative to the Intact state is 23%, 11%, and 7% for Eras 11, 22, and 33, respectively.

Table 5.1 shows the median and dispersion values for the aftershock fragility curves corresponding to the four damage states when the bridge is in the Intact, Slight, Moderate, or Extensive damage states immediately following the mainshock.

Table 5.1 Median (μ) and Dispersion (β) for mainshock and aftershock limit state fragility functions.

Design eras	Damage states	Main shock fragility parameters		Aftershock fragility parameters (no damage under mainshock)		Aftershock fragility parameters (slight damage under mainshock)		Aftershock fragility parameters (moderate damage under mainshock)		Aftershock fragility parameters (extensive damage under mainshock)	
		μ	β	μ	β	μ	β	μ	β	μ	β
Era 11	Slight	0.093	0.588	0.095	0.510	NA	NA	NA	NA	NA	NA
	Moderate	0.238	0.513	0.241	0.504	0.208	0.503	NA	NA	NA	NA
	Extensive	0.609	0.422	0.617	0.431	0.554	0.446	0.522	0.446	NA	NA
	Complete	0.870	0.343	0.873	0.347	0.807	0.390	0.727	0.388	0.674	0.334
Era 22	Slight	0.117	0.479	0.117	0.479	NA	NA	NA	NA	NA	NA
	Moderate	0.562	0.384	0.562	0.384	0.540	0.450	NA	NA	NA	NA
	Extensive	0.804	0.355	0.804	0.355	0.788	0.393	0.772	0.350	NA	NA
	Complete	1.004	0.335	1.004	0.335	0.983	0.364	0.914	0.323	0.890	0.358
Era 33	Slight	0.118	0.469	0.118	0.469	NA	NA	NA	NA	NA	NA
	Moderate	0.562	0.383	0.562	0.383	0.544	0.418	NA	NA	NA	NA
	Extensive	1.003	0.334	1.003	0.334	0.994	0.320	0.984	0.321	NA	NA
	Complete	1.338	0.308	1.338	0.308	1.323	0.310	1.272	0.2994	1.247	0.377

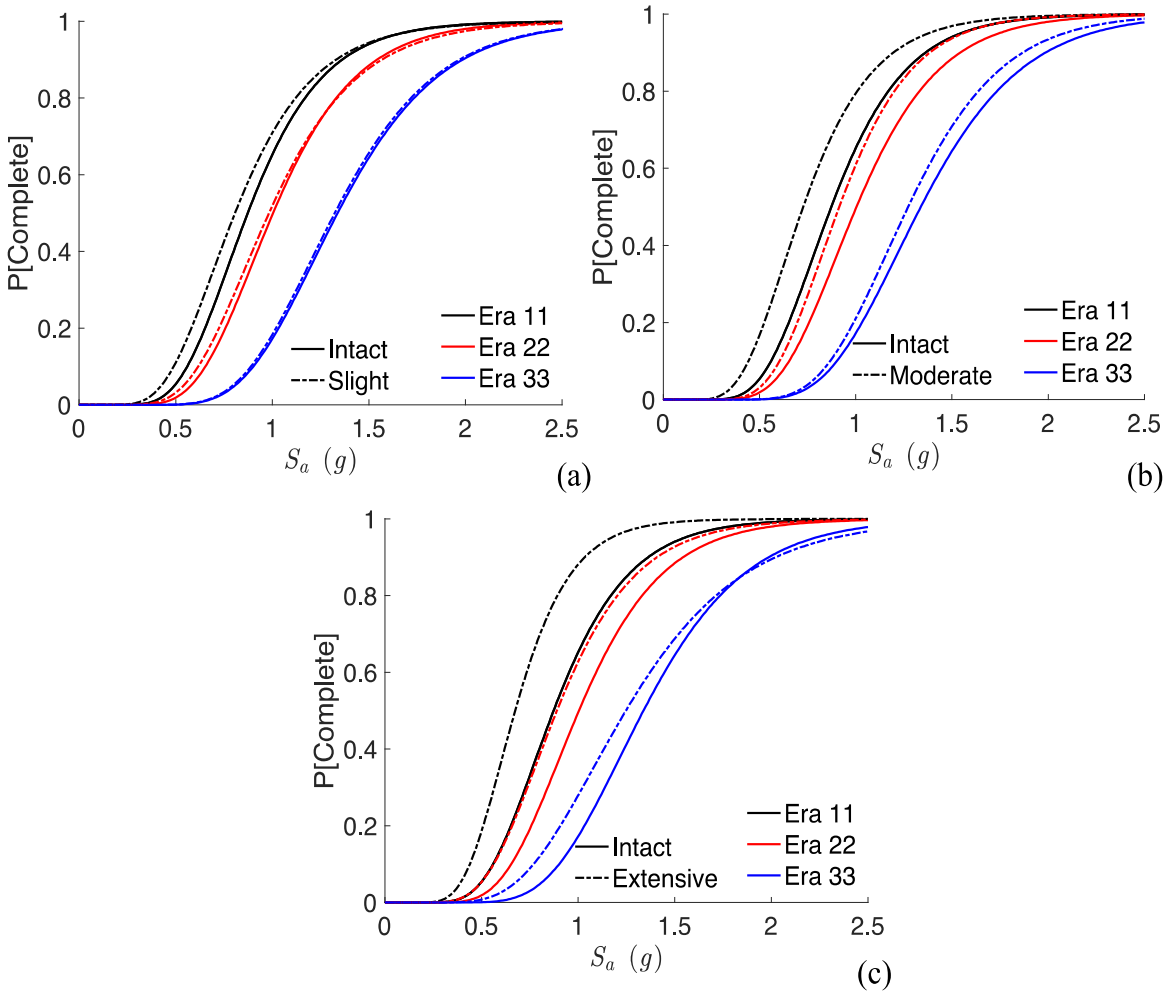

Figure 5.1 **Comparing mainshock and aftershock fragility curves corresponding to the (a) Slight, (b) Moderate and (c) Extensive damage states under mainshock.**

5.2 Assessing Bridge Aftershock Seismic Risk in the Pre- and Post-Mainshock Environments

This section implements the framework presented in Chapter 3 to assess the seismic risk of the three bridges in the pre- and post-mainshock environment. For the latter, aftershock risk is assessed assuming that the mainshock has occurred, and the damaged state of the bridge is known. A period of 30 days following the mainshock is considered; the likelihood that repairs would occur during this time is not considered. A time increment of 0.001 day is used for the Markov chain analysis since the occurrence of more than one aftershock within such a small duration is highly unlikely. Similar to the underlying assumption used to develop the aftershock hazard curve (Chapter 2), the limit transition probabilities are obtained using Equation (3.5), assuming that the first event in the sequence occurred on the fault that has the highest

contribution to the mainshock seismic hazard; in this case it is the Los Angeles section of the Puente Hills fault.

Figure 5.2 shows the time-dependent aftershock limit state occurrence probabilities obtained when a post-mainshock assessment is performed on the Era 11 bridge. Figures, 5.2(a), 5.2(b), and 5.2(c) are plotted for a different conditioning damage state, i.e., Intact, Slight, and Moderate, respectively. Overall, the limit-state occurrence probabilities decrease—for the conditioning state—and increase sharply —for states more severe than the conditioning state—immediately following the mainshock and converge to a constant value after a few days. The initial rise (or fall) is attributed to a high rate of occurrence of aftershock events in the early days following the mainshock. This observation is explained by the exponential decline in the rate of aftershock events during the days following the mainshock.

Figure 5.2(a) shows that when the Era 11 bridge is in the Intact state following the mainshock, the probability that it remains there drops off (from a value of 1.0) to about 0.5 within days following the event. The probability of Slight or Moderate damage occurring within the same duration is approximately 0.45 (50% of which is attributed to each of the two states), and the probability of more severe damage is negligible. As the severity of the conditioning states increases [e.g., Slight and Moderate as shown in Figure 5.2(b) and Figure 5.2(c), respectively], the likelihood of transitioning to a worst limit state increases. For example, compared to the Intact conditioning state, the likelihood of experiencing the Complete limit state within 30 days of the mainshock increases by 60%, 27%, and 16% in the Era 11, Era 22, and Era 33 bridges when they sustain Moderate damage immediately following the mainshock. The conditional limit-state occurrence probabilities 30 days following the mainshock for all three bridges are summarized in Tables 5.2 to 5.4.

Table 5.2 Conditional limit-state occurrence probabilities for the Era 11 bridge 30 days after the mainshock.

Mainshock \ Aftershock	Intact	Slight	Moderate	Extensive	Complete
Intact	8.E-01	2.E-01	2.E-02	3.E-04	2.E-05
Slight		1.E+00	3.E-02	5.E-04	5.E-05
Moderate			1.E+00	6.E-04	8.E-05
Extensive				1.E+00	7.E-05
Complete					1.E+00

Table 5.3 Conditional limit-state occurrence probabilities for the Era 22 bridge 30 days after the mainshock.

Mainshock \ Aftershock	Intact	Slight	Moderate	Extensive	Complete
Intact	9.E-01	1.E-01	3.E-04	3.E-05	6.92E-06
Slight		1.E+00	6.E-04	4.E-05	1.09E-05
Moderate			1.E+00	3.E-05	1.04E-05
Extensive				1.E+00	1.84E-05
Complete					1.E+00

Table 5.4 Conditional limit-state occurrence probabilities for the Era 33 bridge 30 days after the mainshock.

Mainshock \ Aftershock	Intact	Slight	Moderate	Extensive	Complete
Intact	9.E-01	1.E-01	3.E-04	6.E-06	7.E-07
Slight		1.E+00	5.E-04	5.E-06	8.E-07
Moderate			1.E+00	5.E-06	9.E-07
Extensive				1.E+00	3.E-06
Complete					1.E+00

Next, the seismic risk assessment is carried out in the pre-mainshock environment (i.e., the mainshock–aftershock assessment) using the framework presented in Chapter 3. This type of evaluation accounts for the uncertainty in the occurrence of both mainshocks and aftershocks. Considering a 50-year service life and the aftershock hazard assumed insignificant beyond 30 days after the mainshock, the damage state probability at each time step during the life span of the structure is obtained using an exponential distribution with input rates calculated using Equation (3.7). An underlying assumption in using the exponential distribution to calculate service-life seismic risk is that the bridge is repaired between clusters of mainshock–aftershock events. The low mean annual frequency of exceedance of the limit states (λ_{LS}) summarized in Table 5.5 validates this assumption. More specifically, a low λ_{LS} would allow repairs to be completed, and the bridge returned to the Intact state before potential triggering of a limit-state transition in response to the next cluster of events occurs.

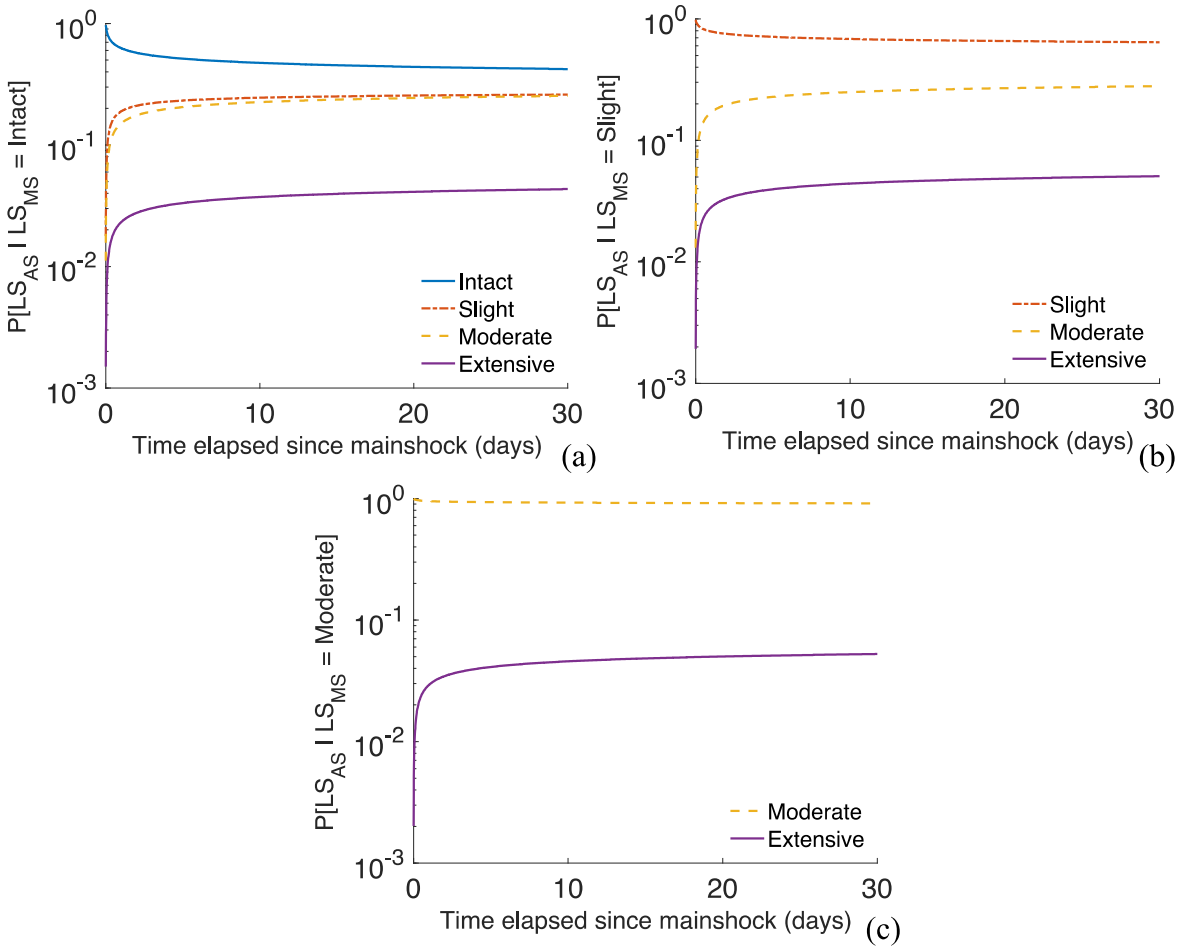

Figure 5.2 **Conditional time-dependent limit-state occurrence probabilities in the post-mainshock environment for the Era 11 bridge in the (a) Intact; (b) Slight; and (c) moderate damage states immediately following the mainshock.**

Figure 5.3 compares the time-dependent occurrence probabilities for the Slight and Complete limit states for the mainshock-only and mainshock–aftershock assessments. For the former, the effect of aftershocks is not included in the risk analysis. Table 5.6 summarizes the limit state occurrence probabilities at the end of the assumed 50-year service life. The results in Figure 5.3 and Table 5.6 suggest that neglecting the effect of aftershocks leads to a significant underestimation of the service damage for all three bridges. Additionally, the differences between the mainshock-only and mainshock–aftershock assessment results increases with the severity of the limit state and is more significant for the Era 11 bridge, where the 50-year Slight damage probability increases by 25% when aftershocks are considered. For Extensive damage, the 50-year occurrence probability increases by factors of 3.0, 2.5, and 2.0 for the Era 11, Era 22, and Era 33 bridges, respectively.

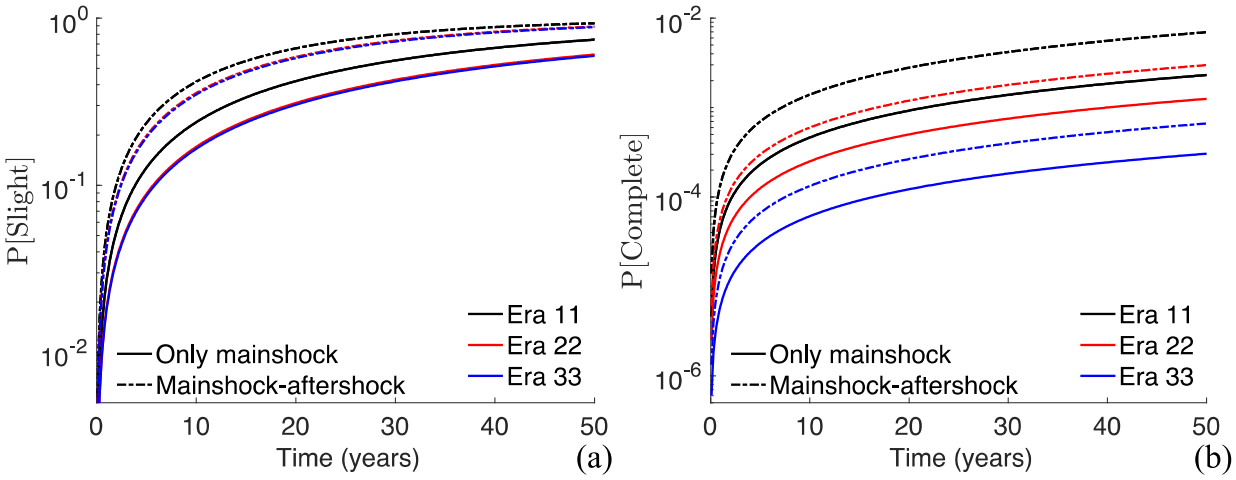

Figure 5.3 **Comparing the time-dependent mainshock-only and mainshock–aftershock occurrence probabilities for the (a) Slight and (b) Complete limit states for all three bridges.**

Table 5.5 **Mean annual frequency of limit-state exceedance in the pre-mainshock environment.**

Bridge type	$\lambda_{LS}\left(\times 10^{-3}\right)$			
	Slight	Moderate	Extensive	Complete
Era 11	53.9	11.9	0.6	0.1
Era 22	44.1	0.6	0.1	0.1
Era 33	43.0	0.6	0.0	0.0

Table 5.6 **Limit state occurrence probabilities at the end of the assumed 50-year service life.**

Scenario	Bridge	Limit state occurrence probability			
		Slight	Moderate	Extensive	Complete
Mainshock-only	Era 11	0.744	0.191	0.009	0.002
	Era 22	0.606	0.010	0.002	0.001
	Era 33	0.595	0.012	0.001	0.000
Mainshock-aftershock	Era 11	0.932	0.447	0.027	0.007
	Era 22	0.890	0.028	0.005	0.003
	Era 33	0.883	0.031	0.002	0.001

6 Summary and Conclusions

The need for functional bridges increases after a major seismic event because of the role they play in supporting the mobility of first responders. As such, the decision of whether to close a bridge temporarily or permanently after it has been damaged in response to a seismic event is critical. Although this decision is usually influenced by a myriad of factors, ideally the time-dependent risk of further damage caused by aftershocks should be considered.

This study presents a seismic risk assessment for three bridges designed during different eras subjected to the mainshock-only (aftershocks are not considered), aftershock (i.e., conditioned on the known state of the bridge following the mainshock) and mainshock–aftershock (i.e., uncertainty in both the mainshock and aftershock is considered) hazard. The time-dependent nature of aftershock seismic hazard and the uncertainty in the state of the structures in the post-mainshock environment is considered through the use of a Markov-chain framework.

The assessment is performed for three typical California bridges that were built during different design/construction eras: constructed before 1971 (Era 11), constructed between 1971 through 1990 (Era 22), and constructed after 1990 (Era 33). Nonlinear structural models were developed for the three bridges. Based on an assumed location in Southern California, 34 pairs of mainshock and aftershock ground motions were selected for sequential nonlinear RHAs. Seismic vulnerability in the mainshock and aftershock environments was represented using four limit states, denoted as Slight, Moderate, Extensive, and Complete based on the bridge column damage and repair state. Fragility curves were generated initially considering the Intact state and varying level of mainshock damage (as defined by the aforementioned limit states).

For all three design eras, the results from the vulnerability assessment revealed Slight damage under the mainshock had minimum impact on the aftershock performance. For the Era 11 bridge, the effect on aftershock fragility was considerable under more severe forms of mainshock damage (e.g., when the bridge is in the Moderate and Extensive states following the mainshock). The Markov chain-modeling approach was used to perform seismic risk assessments in the pre- and post-mainshock environments. The non-conditioning limit state probabilities for the Era 11 bridge increased sharply within the first few days of the mainshock and plateaued soon after. This observation reflects the high rate of aftershock hazard drop-off

following the mainshock. For the same type of assessment, compared to when the bridge is intact, the probability of Complete damage within 30 days following the mainshock increased by 60%, 27%, and 16% when the Era 11, Era 22, and Era 33 bridges, respectively, are subjected to Moderate mainshock damage.

As noted earlier, the mainshock–aftershock risk assessment considered the uncertainty in the occurrence of both mainshock and aftershocks. Based on an assumed 50-year in-service period and a comparison between the results from the mainshock-only and mainshock–aftershock assessments, the lifetime risk of damage was found to be significantly underestimated if aftershock hazard is ignored. This is especially true for the more severe damage states and older bridges. For instance, the probability of extensive damage within a 50-year period increased by a factor of 3.0 for the Era 11 bridge and 2.0 for the Era 33 bridge. For the Era 11 bridge, accounting for aftershock hazard only increased the 50-year probability of Slight damage by 25%.

Note that the results presented in the current study are for a single bridge configuration (e.g., single column bent and seat abutments) and a hypothetical site. Future studies should consider other bridge configurations (e.g., multi-column bents) and design attributes.

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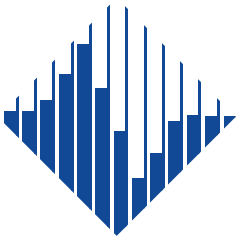

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ISSN 2770-8314
https://doi.org/10.55461/UUUE2614