IDENTIFICATION OF BRIDGE FRAGILITY PARAMETERS USING SEISMIC DAMAGE DATA OBTAINED FROM EXPERIMENTAL STUDY

A Thesis in
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by
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ABSTRACT

The study describes a procedure to identify bridge fragility parameters utilizing its vibration response recorded during experimental study. For this purpose, bridge damage data observed in a near full-scale shake table experiment is utilized. The bridge was tested under a sequence of earthquake ground motions with increasing intensities. Low and high amplitude tests were performed in series to observe the seismic performance of the bridge starting from yielding to complete failure. In the present study, recorded bridge acceleration during high amplitude tests is utilized and further analyzed to evaluate the degraded performance of the bridge after each high amplitude test. This is done by using extended Kalman filtering (EKF) technique as a tool. The degraded performance of the bridge after each run is measured in terms of effective stiffness of the bridge at pier ends. In parallel, finite element (FE) model of the same bridge is developed in order to perform time history analysis under a set of earthquake ground motions with various hazard levels. Before applying the ground motions, the FE model is updated with the effective stiffness of the bridge obtained from EKF after each high amplitude test. This is important to numerically simulate the post-damaged condition of the bridge and to quantify the gradual progression of bridge damage when subjected to earthquake ground motions in sequence. After each time history analysis, bridge response is obtained in terms of the rotation at bridge pier ends. Thus obtained response from time history analyses is used for fragility curve development. The change in fragility parameters represents the progressive damage of the bridge when subjected to ground motions with incremental intensity. Statistical uncertainty of the fragility curves is measured in terms of 90% confidence interval.
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Chapter 1 Introduction

1.1. Problem Statement

Bridge safety and serviceability are major concerns to civil engineers. Experience showed that bridges are the most vulnerable components of highway transportation systems. Performance of a highway transportation network under extreme natural disasters greatly depends on the performance of the constituent bridges. During the most recent major earthquake, 1994 Northridge earthquake in California, seven major freeway bridges in the neighborhood of Los Angeles collapsed and a number of bridges suffered from major damage. This resulted in a significant functional degradation of the Los Angeles area highway transportation network. Similar scenario is observed during Hurricane Katrina in 2005; several bridges were destroyed making the highway transportation system in Louisiana, Mississippi and New Orleans totally disrupted. Figure 1-1 shows snapshots taken from these two extreme events. Hence to minimize such negative consequences of extreme disasters, it is important to evaluate bridge performance under future scenario events and assess the expected risk of highway transportation systems under regional hazards.

Development of bridge fragility curves is one of the most convenient ways to express bridge performance under external loading. Fragility curves represent the failure probabilities of bridges at a damage level under certain loading intensity. These curves can be developed by utilizing bridge damage data obtained from numerical
analysis, experimental research and/or past events. In analysis and experiment, bridge failure in a particular performance level (such as minor, moderate and major damage for seismic analysis of bridges) is determined by comparing bridge response with a threshold value corresponding to that performance level. These threshold values representing bridge performance levels are not constant; they may vary depending on the type of loading (e.g., seismic, high wind, flood, etc.), bridge attributes and configurations (e.g., straight, curved, skewed bridges), and qualitative definitions of bridge performance. In case of past events, bridge failure at different performance levels is decided based on the subjective judgment of post-event reconnaissance groups. Thus obtained bridge failure and non-failure data from various sources are analyzed statistically and fragility parameters are estimated. In this process, it is desirable to have more failure and non-failure data to avoid any statistical uncertainty in the developed fragility curves.

Figure 1-1. Bridge Failure during extreme events; (a) collapse of a connector bridge between I5 and SR 14 during the Northridge earthquake\(^1\) (b) failure of Bay St. Louis Bridge in US 90 during Hurricane Katrina\(^2\)

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\(^1\) [Link](http://www.smate.wwu.edu/teched/geology/eq-CA-Northridge2.html)

\(^2\) [Link](http://mceer.buffalo.edu/publications/bulletin/06/20-01/02katrinasem.asp)
Numerous studies (Shinozuka et al. 2000, 2003, 2007; Yang and Pan 2007; Nielson and DesRoches 2007; Ghanem and Ferro 2006; Hwang et al. 2001; Basoz and Mander 1999; Loh and Tou 1995; Loh and Chung 1993; Hoshiya and Saito 1984; Yun and Shinozuka 1980) have been performed over the last few decades to numerically simulate bridge performance under seismic excitation and to develop seismic fragility curves of bridges. In contrast, availability of bridge fragility curves generated using experimental results is rather scarce. This is because large-scale experiments involving entire bridge models are expensive and demanding; component-level testing is more common for this purpose. Even if large-scale experimental results exist, bridge fragility analysis utilizing the recorded response has not been studied yet. For example, Johnson et al. (2006) performed shake table tests of a near-full scale bridge model. Bridge response recorded from this experiment is processed to gain desired knowledge on the progressive damage of the bridge under seismic excitations. However, recorded data has not been analyzed to identify bridge fragility characteristics. Indeed, such fragility curves are important not only to characterize the seismic vulnerability of bridges having similar attributes and configurations, but also to investigate how bridge fragility characteristics change after the strike of an earthquake event.
1.2. Research Objective

Objective of this present study is to identify bridge fragility parameters utilizing its vibration response recorded in an experimental study. Bridge damage data observed in the near-full scale shake table experiment (Johnson et al. 2006) is utilized for this purpose. In this experiment, the bridge was tested under a sequence of earthquake ground motions with increasing intensities. Low and high amplitude tests were performed in series to observe the seismic performance of the bridge starting from yielding to complete failure. In this present research, recorded bridge acceleration during high amplitude tests are utilized and analyzed further to evaluate the degraded performance of the bridge after each high amplitude test and draw fragility curves with degradation in stiffness. Extended Kalman Filtering (EKF) technique is used for this purpose. The degraded performance of the bridge after each run is measured in terms of effective stiffness at bridge pier ends. This made it possible to quantify the degraded performance of the bridge after it experiences a seismic event.

In general, experiment provides limited data. Therefore to avoid the statistical uncertainty in fragility analysis, numerical study is performed in parallel to simulate the seismic response of the same bridge. Finite element model of the bridge is developed for this purpose. Time history analyses are performed under a set of ground motions with various hazard levels. Before applying seismic ground motions as loading, the bridge model is updated each time with the effective stiffness obtained by analyzing experimental data through EKF. This numerically simulates the gradual progression of bridge damage when the bridge is subjected to sequential ground motions with
increasing intensities. Additionally, such updating calibrates the numerical model with the real experimental bridge model. After each time history analysis, bridge response is obtained in terms of the rotation at bridge pier ends (at locations where the effective stiffness values are measured through EKF). Thus, obtained response from time history analyses shows the progressive damage of the bridge when subjected to ground motions with incremental intensity.

Physical damage states of the bridge are decided by comparing obtained bridge response from time history analysis with threshold limits that represent different seismic damage states of the bridge. These threshold limits are given in Banerjee and Shinozuka (2008a). Upon deciding the damage states, fragility parameters of the bridge are determined. Hence, this research establishes a procedure of utilizing structural vibration response recorded from dynamic testing and/or from embedded sensors to determine its fragility characteristics under ambient vibration or under natural hazards.

The statistical uncertainty of fragility curves are estimated by developing 90% confidence intervals of these curves. Monte Carlo simulation technique is used for this purpose. This uncertainty may be prominent due to insufficient damage data based on which fragility curves are generated.

The research includes following tasks; Task 1: Review of existing literature, Task 2: Identification of bridge effective stiffness from vibration measurement and development of numerical bridge model, Task 3: Fragility analysis and quantification of uncertainty. These tasks are described in Chapter 2, 3 and 4, respectively.
1.3. Assumptions

Key assumptions that are applied to the present study include:

1) In numerical analysis and EKF program, a modal damping ratio of 5% is taken for all models of vibration of the bridge. This is appropriate for initial pre-yield condition of the structure. Beyond yielding, equivalent damping should be calculated based on the ductility level.

2) Nonlinearity is considered only at the ends of bridge piers. Rest part of bridge piers are considered to be elastic.

3) The uncertainty analysis considers only the statistical uncertainty of fragility curves. Uncertainties from other sources such as parameter and model uncertainty are not analyzed here.
Chapter 2 Review of Existing Literature

To review existing literature relevant to the present research, this task is divided into three subtasks; (i) literature review on the quantification of post-event structural capacity, (ii) review of experimental research, (iii) review of bridge fragility analysis, and (iv) review of literature on uncertainty quantification for structural analysis. These subtasks are described below.

2.1. Review of Literature on the Quantification of Post-event Structural Capacity

Quantification of structural damage and the evaluation of post-event structural capacity can be done by calculating the effective stiffness values of structural elements. This also facilitates the identification of damage locations in the structure. Extended Kalman filtering (EKF) method is an effective tool for this purpose (Grewal and Andrews 2008). This technique has been extensively used over last three decades for structural damage identification (Yun and Shinozuka 1980, Hoshiya and Saito 1984, Loh and Chung 1993, Loh and Tou 1995, Maruyama and Hoshiya 2001, Ghanem and Ferro 2006, Yang and Pan 2007, Soyoz and Feng 2008, Zhou et al. 2008). Through EKF, the state of a dynamic system can be estimated from its vibration measurement data. The technique is essentially a set of mathematical equations that filters observed measurements (may be associated with noise and inaccuracies) to obtain true state of the structure. This requires state transition and observation models which may be either linear or nonlinear. For a multi-degree of freedom system, the basic equation of motion can be expressed as (Welch and Bishop 2001)
\[ M \ddot{u}(t) + C(t) \dot{u}(t) + K(t)u(t) = -MI\ddot{u}_g \]  

(2.1)

where \( M \) is the mass matrix, \( C(t) \) is the damping matrix, \( K(t) \) is the stiffness matrix, \( I \) is the influence vector and \( \ddot{u}_g \) is the acceleration of input ground motion. \( \ddot{u}(t), \dot{u}(t), \) and \( u(t) \) represent relative structural response. Absolute acceleration \( (\dddot{u} + \ddot{u}_g) \) and ground acceleration \( (\ddot{u}_g) \) can be obtained from measurement, mass matrix can be calculated from structural design drawings, \( C(t) \) is considered to be Rayleigh-type damping which has linear relation with mass and stiffness matrices. The stiffness matrix \( K(t) \) is regarded as the indicator of structural damage as it represents the structural capacity (or stiffness) at a time instant \( t \). In presence of structural response (obtainable for experimental research or real-time structural health monitoring), the objective would be to identify \( K(t) \) to know the present state of the structure.

EKF defines an extended state vector as

\[
x(t) = [u(t), \dot{u}(t), \psi(t)]^T
\]

(2.2)

where \( \psi(t) \) is the extended state. In this present study, it represents the stiffness of the upper and lower portions of bridge piers. If \( \hat{x}_{k|k} \) represents the estimated state (where \( \hat{\cdot} \) denotes estimation) at time step \( k \), the error covariance can be written as

\[
P_{k:k} = E[(x_k - \hat{x}_{k|k})(x_k - \hat{x}_{k|k})^T]
\]

(2.3)

EKF determines the optimal estimate of \( \hat{x}_{k|k} \) that minimizes the error covariance \( P_{k:k} \). There are two conceptual phases in EKF; the prediction phase and the correction phase (Figure 2-1). In the prediction phase, the initial values of state estimate \( \hat{x}_{k-1|k-1} \) and error covariance \( P_{k-1:k-1} \) are projected to get a priori estimates of \( \hat{x}_{k|k-1} \) and \( P_{k:k-1} \). Then in the correction phase, these a priori estimates are filtered
using measurements of $k^{th}$ time step to get \textit{a posteriori} estimates $\hat{x}_{k|k}$ and $P_{k|k}$.

Eventually, these $\hat{x}_{k|k}$ and $P_{k|k}$ are used as initial estimates for the following time step $(k+1)$.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{image}
\caption{A complete picture of the operation of EKF}
\end{figure}

Therefore, the mathematical formulation of EKF can be performed in following five steps:

\textit{Step 1:} Predicting state:
\[ \hat{x}_{k|k-1} = \Phi_{k-1} \hat{x}_{k-1|k-1} \]  \hspace{1cm} (2.4)

\textit{Step 2:} Predicting covariance:
\[ P_{k|k-1} = \Phi_{k-1} P_{k-1|k-1} \Phi_{k-1}^T + Q_{k-1} \]  \hspace{1cm} (2.5)

\textit{Step 3:} Computing Kalman gain:
\[ G_k = P_{k|k-1} H_k^T (H_k P_{k|k-1} H_k^T + R_k)^{-1} \]  \hspace{1cm} (2.6)
Step 4: Correcting state:
\[
\hat{x}_{k|k} = \hat{x}_{k|k-1} + G_k[y_k - H_k \hat{x}_{k|k-1}]
\]  
(2.7)

Step 5: Correcting covariance:
\[
P_{k|k} = (I - G_k H_k) P_{k|k-1}
\]  
(2.8)

The terms \( Q, R, \Phi, y \) are discussed in following paragraphs.

The dynamic system can be defined as
\[
\dot{x}(t) = f(x, t) + w(t)
\]  
(2.9)

where \( w(t) \) is the system noise which is taken as zero mean Gaussian distribution with covariance \( Q(t) \). The measurement model can be represented as
\[
y(t) = h(x, t) + v(t)
\]  
(2.10)

where \( y(t) \) is the measurement and \( v(t) \) is the measurement noise. \( v(t) \) is considered to be zero mean Gaussian distribution with covariance \( R(t) \). Initial state and error covariance of the structure are represented as
\[
E[x(t_0)] = x_0
\]  
(2.11)

\[
E[(x(t_0) - x_0)(x(t_0) - x_0)^T] = P_0
\]  
(2.12)

For continuous time, the following will be satisfied.
\[
E[w(t)w^T(t)] = Q(t)
\]  
(2.13)

\[
E[v_jv_k^T] = R_k \delta_{jk}
\]  
(2.14)

where \( j \) indicates discrete time.

Linearized measurement matrix \( H_k \) can be obtained as
\[
H(x, t) = \frac{\partial h(x, t)}{\partial x}
\]  
(2.15)
Linearized state matrix $F_k$ (for calculating the state transition matrix) can be obtained as

$$F(x, t) = \frac{\partial f(x, t)}{\partial x} \quad (2.17)$$

$$F_k = F(\hat{x}_{k|k-1}, t_k) \quad (2.18)$$

State transition matrix $\Phi_{k-1}$ can be obtained as

$$\Phi_{k-1} = I + \int_{t_{k-1}}^{t_k} F(\hat{x}_{k|k-1}, t) \Phi(t, t_{k-1}) \, dt \quad (2.19)$$

The last equation is expressed as

$$\Phi(t_k, t_{k-1}) = \exp(\Delta t \cdot F_{k-1}), \quad \text{Constant } F \forall t \in [t_{k-1}, t_k] \quad (2.20)$$

This is approximated as

$$\Phi(t_k, t_{k-1}) = I + \Delta t \cdot F_{k-1} \quad (2.21)$$

Also, the process noise can be discretized as

$$Q_{k-1} = \Delta t \cdot Q(t_{k-1}) \quad (2.22)$$

After initializing $x_0$, $P_0$, $Q$, and $R$ in Equations (2.11)-(2.14), state vector $x$ is obtained at each time step. The state vector contains information of stiffness values. Thus, stiffness can be obtained at each time step.

### 2.2. Review of Experimental Research

The present study uses the vibration measurement data of a 2-span reinforced concrete bridge under earthquake ground motions. The data is obtained from large-scale shaking table tests conducted at the University of Nevada, Reno (Johnson et al. 2006).
Results of this experimental study included bridge response (acceleration time histories and rotational and translational deformations) and the nature of bridge failure from yielding to complete collapse under ground motions with increasing intensity levels. Figure 2-2 shows the bridge model and experimental plan. Figure 2-3 shows the instrumentation plan (only the locations of accelerometers). Total height of this specimen was 3.28 m from the bottom of the footing block to the top of the superstructure. Bridge spans, each 9.14 m long, were supported on three pier bents with the tallest one at middle. Clear heights of these bents were 1.83 m, 2.44 m and 1.52 m. Each bent consisted of two piers of the same cross-sectional and material properties. The bridge deck was a solid slab post-tensioned in both the longitudinal and transverse directions of the bridge. The axial force was estimated to be 209.2 kN in Bent 1 and 3, and 182.2 kN in Bent 2. Additional details of the bridge and the experiment are given in Johnson et al. (2006).

Figure 2-2. Experimental model of a 2-span reinforced concrete bridge (Johnson et al. 2006)
The ground motion time history recorded during the 1994 Northridge earthquake at the Century City Country Club was used in the experimental program to calculate input ground motions. During experiment, input motions were applied in the transverse direction of the bridge as the end conditions of the bridge in the longitudinal direction were not modeled. Both low and high amplitude tests were performed. No damage was observed during low amplitude tests. Therefore, it was assumed that the bridge responded in elastic range during low amplitude tests. Following that, nine high amplitude tests were performed. Ground motion intensities during these high amplitude tests were increased gradually to observe bridge damage from yielding to complete collapse.

Bridge vibration response measured during high amplitude tests T12, T13, T14, T15, T17 and T18 are used in this study. Peak ground accelerations (PGAs) of...
these tests and a summary of observed bridge damage are reported in Table 2-1. During these tests, progressive damage observed at the top of one of the piers at Bent 3 is shown in Figure 2-4 to Figure 2-9 (Johnson et al. 2006). At test 19, rebar buckled in one of the piers in Best 3. This was regarded as the failure of the bridge.

Table 2-1: Input PGAs and corresponding damage in each test, in general

<table>
<thead>
<tr>
<th>Test</th>
<th>Target PGAs (g)</th>
<th>Damage in bridge piers</th>
</tr>
</thead>
<tbody>
<tr>
<td>T12</td>
<td>0.075</td>
<td>No damage</td>
</tr>
<tr>
<td>T13</td>
<td>0.15</td>
<td>No damage</td>
</tr>
<tr>
<td>T14</td>
<td>0.25</td>
<td>Hairline cracks</td>
</tr>
<tr>
<td>T15</td>
<td>0.50</td>
<td>Minor cracking or spalling</td>
</tr>
<tr>
<td>T17</td>
<td>1.00</td>
<td>Minor cracking and significant spalling occurred</td>
</tr>
<tr>
<td>T18</td>
<td>1.33</td>
<td>Significant spalling occurred</td>
</tr>
</tbody>
</table>

Figure 2-4. Damage at the top of one of the piers at Bent 3 after T15 (east side)
Figure 2-5. Damage at the top of one of the piers at Bent 3 after T17 (east side)

Figure 2-6. Damage at the top of one of the piers at Bent 3 after T18 (east side)

Figure 2-7. Damage at the top of one of the piers at Bent 3 after T15 (west side)
The original elastic stiffness of the bridge piers can be obtained from the design drawing given in Johnson et al. (2006). Banerjee and Shinozuka (2008a) developed the moment-rotation relationships for the piers. For this purpose, input data such as effective pier height, pier diameter, concrete cover, number and diameter of longitudinal reinforcing bars, and diameter and spacing of spiral reinforcement were obtained from the pier reinforcement plan as shown in Figure 2-10 (Johnson et al. 2006). Figure 2-11 and Figure 2-12 show the moment-rotation relationships of individual piers in three
bents (Banerjee and Shinozuka 2008a). From these figures the original elastic stiffness of bridge piers can be obtained by calculating the slope in the linear elastic region, which is indicated as $k_{\text{elastic}}$ in the figures. Note that the moment curvature relationship of piers in Bent 1 and 3 are different that those in Bent 2 due to the variation of above design values.

![Figure 2-10. Cross section of the piers (Johnson et al. 2006)](image1)

![Figure 2-11. Moment-rotation relationship of each pier in Bent 1 and Bent 3 (Banerjee and Shinozuka 2008a)](image2)
2.3. Review of Literature on Establishing Fragility Curves

Seismic fragility curves of bridges express the probability of failure of the bridge in a damage state under certain ground motion intensity. A comprehensive summary on the bridge fragility curve development can be found in Banerjee (2007). Following Shinozuka et al. (2000), fragility curves are developed here in the form of two parameter lognormal distribution function. Distribution parameters, referred to as fragility parameters, are median $c$ and log-standard deviation $\zeta$. These parameters can be determined using the maximum likelihood method. The likelihood function can be expressed as (Shinozuka et al. 2000)

$$L = \prod_{i=1}^{N} \left[ F(a_i) \right]^{x_i} \left[ 1 - F(a_i) \right]^{1-x_i}$$

(2.23)

where $x_i$ is equal to 1 if the damage state occurs in the simulation with earthquake intensity $a_i$, and $x_i$ is equal to 0 if the damage doesn’t occur. In the likelihood function,
$F(a_i)$ represents the probability that a structure will fall in a damage state when subjected to motion intensity expressed by PGA=$a_i$. The maximum likelihood estimates are obtained by solving the equations shown below using an optimization algorithm (Shinozuka et al. 2000):

$$\frac{d\ln L}{dc} = \frac{d\ln L}{d\zeta} = 0$$  \hspace{1cm} (2.24)

$$F(a_i) = \Phi\left[\frac{\ln (a_i/c)}{\zeta}\right]$$  \hspace{1cm} (2.25)

2.4. Review of Literature on Uncertainty Quantification for Structural Analysis

Evaluation of structural performance is important for structural risk management under extreme events. However, this process involves large uncertainty due to the uncertainty of parameters affecting both the structural capacity and demand. Research has been conducted on analysis of these parameters uncertainty.

To evaluate the contributions of different sources of uncertainty in bridge fragility modeling under seismic effect, Mackie and Nelson (2009) investigated a unified analytical fragility method using both simulation and closed form methods. In the study, it was found that the uncertainty of bridge properties, as-built properties and damage state definitions can be sorted into epistemic uncertainty. The result showed that considering all bridge parameters in demand analysis as random may result in an over-estimation of the damage fragility uncertainty.

To select an eligible level of uncertainty treatment while balancing the simulation and computational effort, Padgett and DesRoches (2007) evaluated the
modeling parameters which significantly affect the seismic performance of retrofitted bridges. They compared the importance of uncertainty in these parameters finding that the fragility estimation may be simplified by screening modeling parameters preliminarily. In the study, a retrofitted multi-span simply supported steel girder bridge was chosen as the analytical model. As a result the author concluded that the most important modeling parameters include loading direction, gross geometry in association with the bridge properties such as abutment stiffness, damping ratio and mass. Besides, the uncertainty in ground motion and gross geometry tend to dominate those associated with modeling parameter variation. Prudent definition of these parameters could tolerate deterministic treatment of many other parameters whose variation is less influential for overall fragility.

Research has also been conducted concerning parameter uncertainty of structures other than bridge. The method, however, can provide an insight into fragility derivation of bridge. Thermouet al. (2004) established the relative effect of strong-motion variability and random structural parameters on fragility curves based on the shake-table test result of a three story ordinary moment resisting reinforced concrete frame. The author pointed that the variation of parameters from the model material properties is far less important than the variation of ground motion.
Experimental study provides limited bridge failure data. To analyze bridge seismic vulnerability in the form of fragility curves, numerical simulations are performed to simulate bridge seismic damage data after each event of high amplitude tests. Three-dimensional finite element models of the same experimental bridge at pre- and post-damaged conditions (i.e., before and after high amplitude tests) are developed in SAP2000 Nonlinear (Computer and Structures 1995) and analyzed under a suite of earthquake ground motions with various hazard levels. Obtained bridge response is used for fragility curve development. The result is used further for the uncertainty analysis. Figure 3-1 represents a flow chart showing the research steps and provides a clear step-by-step guide of this study.
Experimental bridge response (acceleration) after i\textsuperscript{th} high amplitude test

EKF

Effective stiffness at pier ends after i\textsuperscript{th} test

Update SAP model with effective stiffness

Time history analysis

Ground motion from (i+1)\textsuperscript{th} high amplitude test

Bridge response from SAP

Experimental bridge response from (i+1)\textsuperscript{th} test

Model verification

60 SAC notions

Bridge response from SAP

Fragility curves at different damage levels after i\textsuperscript{th} test

Repeat this process for i=12, 13, 14, 15, 17, 18

Bridge fragility characteristics

Statistical uncertainty of fragility curves

Figure 3-1. Flowchart of research process in this study
3.1. Identification of Effective Stiffness of the Experimental Bridge after High Amplitude tests

To quantify the performance degradation of the bridge after each high amplitude test, present study identifies the effective stiffness of the bridge at both ends of bridge piers where damage was primarily observed during these tests. The effective stiffness is the secant property of a structure which pertains to its nonlinear response under dynamic loading (Priestley et al. 1996). Figure 3-2 illustrates the definition of effective stiffness. In this study, the effective stiffness is expressed as dimensionless effective stiffness, which can also be called as ratio stiffness. The ratio stiffness is the ratio of effective stiffness over original elastic stiffness \( (k_0 \text{ or } k_{\text{elastic}}) \). This original elastic stiffness can be obtained from moment-rotation curves of bridge piers as presented in Figures 2-11 and 2-12. This effective stiffness expresses the extent of degradation of bridge piers. Lower ratio stiffness indicates higher degradation of stiffness.

![Effective stiffness and dimensionless stiffness](image)

Figure 3-2. Effective stiffness and dimensionless stiffness
Extended Kalman filtering (EKF) method is used here to analyze bridge vibration measurements from experiment and to calculate bridge effective stiffness at both ends of piers. A program on EKF is developed in Matlab (Mathworks Inc. 2008) which contains a finite element (FE) lumped mass model of the experimental bridge. During T12, T13, T14, T15, T17 and T18, time histories of bridge acceleration recorded at installed accelerometers and ground motion time histories recorded at the shake table level are used as input to the EKF program. Final outcome from this analysis is the effective stiffness of the bridge in a dimensionless format (i.e., effective stiffness/initial elastic stiffness).

Prior to the analysis, all vibration measurements are filtered through butterworth-type low-pass filter of 20 Hz to eliminate noise from recorded signals. Figures 3-3 to 3-8 show ground motion time histories used in high amplitude tests before and after filtering.

**Figure 3-3.** Original and filtered ground motions of Test 12
Figure 3-4. Original and filtered ground motions of Test 13

Figure 3-5. Original and filtered ground motions of Test 14
Figure 3-6. Original and filtered ground motions of Test 15

Figure 3-7. Original and filtered ground motions of Test 17
In Matlab program, bridge damping is assumed to be Rayleigh damping. Relation between Rayleigh damping coefficients $\alpha$ and $\beta$ and modal damping ratio ($\xi$) can be expressed as

$$C = \beta M + \alpha K$$  \hspace{1cm} (3.1)

$$\xi = \frac{\beta}{2\omega_i} + \frac{\alpha \omega_i}{2}$$  \hspace{1cm} (3.2)

where $C$ is damping matrix, $M$ is mass matrix, $K$ is stiffness matrix and $\omega_i$ is the frequency at $i$th mode of vibration. Since the actual $\xi$ value is not available for each modal frequency, the calculation of actual $\alpha$ and $\beta$ is impossible. However from the experiment observation, the frequencies of the bridge at first 3 modes of vibrations are evaluated to be 2.93Hz, 3.70Hz and 13.7Hz (Soyoz and Feng 2008). The third mode has a high frequency and hence, it has less influence on the model performance. It was also observed that after T14, the second and third modes were no longer visible (Soyoz and...
Feng 2008). In a separate set of analysis, it is observed that the effective stiffness of the bridge is not very much sensitive to $\alpha$ and $\beta$. So the present study assumed 5% damping ratio in first and second modes of vibration. Therefore the Rayleigh damping coefficients are calculated as $\alpha=0.0024$, $\beta=1.027$ with equation (3.2). These values are further used in EFK program.

Depending on initial values of parameters used for EFK such as measurement noise covariance $R$ and initial error covariance $P_0$, identified effective stiffness of the bridge at pier ends may vary. The optimal combination of $R$ and $P_0$ is selected here by adjusting their values so that the bridge effective stiffness at pier ends realistically represents physical damage observed in the bridge during the experiment. From trial analyses with a range of $R$ values (having mean = 300), it is found that effective stiffness of the bridge is not sensitive to $R$. Hence, $R = 300$ is considered for EKF. To analyze the sensitivity of error covariance $P_0$ on effective stiffness, 8 sets of $P_0$ (6 values per set for top and bottom of bridge piers in Bent 1, 2 and 3) are taken and obtained stiffness values are represented in Figures 3-9 to 3-14. These stiffness values are compared with observed bridge damage at different locations. Comparison indicates that among six chosen sets, $P_0 = \{0.005, 0.001, 0.005, 0.001, 0.005, 0.001\}$ provides the closest match between effective stiffness and observed damage. Table 3-1 shows the effective stiffness of the bridge (as a ratio to the initial elastic stiffness) obtained from EKF using $R = 300$ and $P_0 = \{0.005, 0.001, 0.005, 0.001, 0.005, 0.001\}$. 
Figure 3-9. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.001 at Bent 1
Figure 3-10. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.001 at Bent 2
Figure 3-11. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.001 at Bent 3
Figure 3-12. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.002 at Bent 1.
Figure 3-13. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.002 at Bent 2.
Figure 3.14. Comparison of 4 sets of $P_0$ when the values of bottom part is 0.002 at Bent 3.
Table 3-1: Dimensionless stiffness (effective stiffness/initial elastic stiffness) at bridge pier ends as obtained from EKF

<table>
<thead>
<tr>
<th>Bent</th>
<th>Top</th>
<th>Bottom</th>
<th>Bent</th>
<th>Top</th>
<th>Bottom</th>
<th>Bent</th>
<th>Top</th>
<th>Bottom</th>
<th>Bent</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
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<td>R=300, P₀={.005, .001, .005, .001, .005, .001}</td>
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<td>Bent 2</td>
<td>Bent 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>T12</td>
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<td></td>
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</tr>
<tr>
<td>Bottom</td>
<td>1.0000</td>
<td>1.0000</td>
<td>0.9991</td>
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<td>Bottom</td>
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</table>
3.2. Numerical Bridge Model with Identified Effective Stiffness

In this task, the finite element model of the bridge is developed in SAP 2000. Schematic view of the model is shown in Figure 3-15. Figure 3-16 shows the 3D view of the model. Plastic hinges are likely to emerge during seismic excitations at both top and bottom of each pier ends. This is modeled in SAP2000 by introducing nonlinear links at all the pier ends. Effective stiffness, yielding strength and post yield stiffness ratio are all defined in the link members to simulate the nonlinear performance of the pier ends. The rest part of each pier is modeled with linear element. Initial pre-damaged stiffness of these nonlinear links is obtained from moment rotation relations presented in Figures 2-11 and 2-12. Six additional FE models of the bridge are developed that simulate bridge condition after high amplitude tests T12, T13, T14, T15, T17 and T18. This is done by updating the pre-damaged elastic stiffness of the nonlinear links with effective stiffness identified through EKF (Table 3-1). Hence, these numerical models can actually represent degraded state of the bridge after it experiences seismic events equivalent to those used in high amplitude tests.

Figure 3-15. 2D profile of the SAP bridge model
3.2.1. Verification of Numerical Bridge Model

Prior to the time history analysis, developed bridge models are verified by comparing bridge response obtained from numerical simulations with that recorded during experimental study. Analyses are performed under ground motions that are used in the experimental program during high amplitude tests T12, T13, T14, T15, T17 and T18. After each analysis, time histories of bridge response in terms of transverse displacement at bridge pier tops and accelerations at locations of installed accelerometers are recorded. Figures 3-17 and 3-18 show the acceleration response at the midpoint of bridge deck (accelerometer number 7) during high amplitude test T14.
and T15. These figures show excellent agreement between the measured and numerically simulated bridge response.

![Acceleration History at: accelerometer #7 Test 14](image1)

**Figure 3-17. Acceleration response at the midpoint of the deck during Test 14**

![Acceleration History at: accelerometer #7 Test 15](image2)

**Figure 3-18. Acceleration response at the midpoint of the deck during Test 15**

The displacement and rotation of each pier obtained from SAP are compared with those recorded in the experimental study after each high amplitude test. Tables 3-2 and 3-3 show the maximum and minimum rotation of bridge pier ends as obtained from numerical analysis and experiment, respectively. The maximum transverse displacements of the bridge obtained from numerical analysis and experiment are given
in Tables 3-4 and 3-5, respectively. Comparison of result presented in these two tables (Tables 3-4 and 3-5) indicates that the simulated response compares reasonably well with the measured response. However, the difference between them is significant in some locations of the bridge. Linear regression analysis is performed as a following step to establish a relation between measured and simulated displacement response of the bridge.
### Table 3-2: Rotation of each pier end obtained from SAP2000 model

<table>
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<th>Bent#</th>
<th>Link#</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T17</th>
<th>T18</th>
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<td>0.003668</td>
<td>0.00443</td>
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<td>B1T2</td>
<td>08Min</td>
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<tr>
<td>B1B1</td>
<td>01Max</td>
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<td>0.003812</td>
<td>0.004144</td>
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Table 3-3: Rotation of each pier end recorded from experiment (Johnson et al. 2006)

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<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T17</th>
<th>T18</th>
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<td>0.00425</td>
<td>0.006</td>
<td>0.0095</td>
<td>0.0195</td>
<td>0.023</td>
</tr>
<tr>
<td>B1B1</td>
<td>-0.0014</td>
<td>-0.005</td>
<td>-0.006</td>
<td>-0.0145</td>
<td>-0.01</td>
<td>-0.023</td>
</tr>
<tr>
<td>B1B2</td>
<td>0.00135</td>
<td>0.0044</td>
<td>0.0065</td>
<td>0.0105</td>
<td>0.021</td>
<td>0.024</td>
</tr>
<tr>
<td>B1B2</td>
<td>-0.00165</td>
<td>-0.00455</td>
<td>-0.0055</td>
<td>-0.013</td>
<td>-0.011</td>
<td>-0.0265</td>
</tr>
<tr>
<td>B2T1</td>
<td>0.00075</td>
<td>0.002</td>
<td>0.00265</td>
<td>0.006</td>
<td>0.0075</td>
<td>0.017</td>
</tr>
<tr>
<td>B2T1</td>
<td>-0.00065</td>
<td>-0.00155</td>
<td>-0.0019</td>
<td>-0.004</td>
<td>-0.0095</td>
<td>-0.018</td>
</tr>
<tr>
<td>B2T2</td>
<td>0.0007</td>
<td>0.0016</td>
<td>0.00215</td>
<td>0.00485</td>
<td>0.006</td>
<td>0.014</td>
</tr>
<tr>
<td>B2T2</td>
<td>-0.0006</td>
<td>-0.0015</td>
<td>-0.00185</td>
<td>-0.0055</td>
<td>-0.013</td>
<td>-0.025</td>
</tr>
<tr>
<td>B2B1</td>
<td>0.00095</td>
<td>0.0022</td>
<td>0.0027</td>
<td>0.007</td>
<td>0.013</td>
<td>0.024</td>
</tr>
<tr>
<td>B2B1</td>
<td>-0.0006</td>
<td>-0.002</td>
<td>-0.00265</td>
<td>-0.0065</td>
<td>-0.0085</td>
<td>-0.018</td>
</tr>
<tr>
<td>B2B2</td>
<td>0.0011</td>
<td>0.0025</td>
<td>0.00305</td>
<td>0.0065</td>
<td>0.0115</td>
<td>0.022</td>
</tr>
<tr>
<td>B2B2</td>
<td>-0.00055</td>
<td>-0.0017</td>
<td>-0.00225</td>
<td>-0.0055</td>
<td>-0.007</td>
<td>-0.0155</td>
</tr>
<tr>
<td>B3T1</td>
<td>0.00115</td>
<td>0.00245</td>
<td>0.00385</td>
<td>0.0085</td>
<td>0.0125</td>
<td>0.032</td>
</tr>
<tr>
<td>B3T1</td>
<td>-0.00055</td>
<td>-0.00175</td>
<td>-0.00335</td>
<td>-0.0095</td>
<td>-0.013</td>
<td>-0.033</td>
</tr>
<tr>
<td>B3T2</td>
<td>0.00125</td>
<td>0.00285</td>
<td>0.00495</td>
<td>0.016</td>
<td>0.015</td>
<td>0.035</td>
</tr>
<tr>
<td>B3T2</td>
<td>-0.00105</td>
<td>-0.00245</td>
<td>-0.00415</td>
<td>-0.0115</td>
<td>-0.0165</td>
<td>-0.0435</td>
</tr>
<tr>
<td>B3B1</td>
<td>0.00145</td>
<td>0.00305</td>
<td>0.005</td>
<td>0.013</td>
<td>0.0175</td>
<td>0.043</td>
</tr>
<tr>
<td>B3B1</td>
<td>-0.00115</td>
<td>-0.0032</td>
<td>-0.005</td>
<td>-0.0185</td>
<td>-0.018</td>
<td>-0.0405</td>
</tr>
<tr>
<td>B3B2</td>
<td>0.00115</td>
<td>0.0027</td>
<td>0.0047</td>
<td>0.012</td>
<td>0.0175</td>
<td>0.0465</td>
</tr>
<tr>
<td>B3B2</td>
<td>-0.00135</td>
<td>-0.0032</td>
<td>-0.005</td>
<td>-0.0175</td>
<td>-0.0155</td>
<td>-0.034</td>
</tr>
<tr>
<td>MAX</td>
<td>0.00175</td>
<td>0.005</td>
<td>0.0065</td>
<td>0.0185</td>
<td>0.021</td>
<td>0.0465</td>
</tr>
</tbody>
</table>
Table 3-4: Displacement of each pier end obtained from SAP2000 model (mm)

<table>
<thead>
<tr>
<th>Bent#</th>
<th>joint#</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T17</th>
<th>T18</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1max</td>
<td>22</td>
<td>6.02</td>
<td>16.47</td>
<td>19.86</td>
<td>38.71</td>
<td>43.08</td>
<td>80.17</td>
</tr>
<tr>
<td>B1min</td>
<td>22</td>
<td>-6.35</td>
<td>-16.84</td>
<td>-18.28</td>
<td>-31.33</td>
<td>-48.19</td>
<td>-70.5</td>
</tr>
<tr>
<td>B2max</td>
<td>15</td>
<td>3.13</td>
<td>7.74</td>
<td>9.86</td>
<td>29.57</td>
<td>48.08</td>
<td>76.18</td>
</tr>
<tr>
<td>B2min</td>
<td>15</td>
<td>-3.12</td>
<td>-8.42</td>
<td>-12.9</td>
<td>-27.32</td>
<td>-36.16</td>
<td>-70.28</td>
</tr>
<tr>
<td>B3max</td>
<td>11</td>
<td>3.33</td>
<td>9</td>
<td>13.96</td>
<td>30.03</td>
<td>58.29</td>
<td>72.79</td>
</tr>
<tr>
<td>B3min</td>
<td>11</td>
<td>-3.57</td>
<td>-10.56</td>
<td>16.99</td>
<td>-32.34</td>
<td>36.38</td>
<td>-68.13</td>
</tr>
</tbody>
</table>

Table 3-5: Displacement of each pier end obtained from experiment (mm)

<table>
<thead>
<tr>
<th>Bent#</th>
<th>T12</th>
<th>T13</th>
<th>T14</th>
<th>T15</th>
<th>T17</th>
<th>T18</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1max</td>
<td>5.9</td>
<td>16.2</td>
<td>19.1</td>
<td>39.7</td>
<td>33</td>
<td>70.6</td>
</tr>
<tr>
<td>B1min</td>
<td>-4.8</td>
<td>-14.4</td>
<td>-19.2</td>
<td>-28.8</td>
<td>-50.3</td>
<td>-57.8</td>
</tr>
<tr>
<td>B2max</td>
<td>4.3</td>
<td>10.9</td>
<td>14.2</td>
<td>31.7</td>
<td>36.1</td>
<td>69.8</td>
</tr>
<tr>
<td>B2min</td>
<td>-4</td>
<td>-9.3</td>
<td>-11.8</td>
<td>-28.3</td>
<td>-49.4</td>
<td>-85.5</td>
</tr>
<tr>
<td>B3max</td>
<td>3.1</td>
<td>7.9</td>
<td>12.6</td>
<td>37.1</td>
<td>36.2</td>
<td>73.9</td>
</tr>
<tr>
<td>B3min</td>
<td>-3.1</td>
<td>-7.3</td>
<td>-12</td>
<td>-26.8</td>
<td>-33.8</td>
<td>-83.9</td>
</tr>
</tbody>
</table>
3.2.2 Regression Analysis with Measured and Simulated Displacement

In linear regression analysis, it is assumed that variable $Y$ is dependent on an independent variable $X$. The linear relation between these two variables can be expressed as (Ang and Tang 1975).

$$ E(Y|X = x) = \beta x + \alpha $$

where $\alpha$ and $\beta$ are constants. Variance of $Y$ may be independent (or constant) or a function of $x$. A constant variance of $Y$ is taken when all data ($x$ and $y$) has consistent degree of scatter. However, in cases when the scattergram of the data shows a significant variation in the degree of scatter with $x$, variance of $Y$ is taken as a function of $x$ which may be expressed as

$$ Var(Y|x) = \sigma^2 g^2(x) $$

where $g(x)$ is a predetermined function and $\sigma$ is an unknown constant. In this case, it is assumed that data points in regions of small variance will have more weight on the regression equation than those in regions of large variance. Therefore, weights of the data are expressed as the inverse proportion of the variance which is

$$ w'_i = \frac{1}{Var(Y|x_i)} = \frac{1}{\sigma^2 g^2(x_i)} $$

The squared error is

$$ \Delta^2 = \sum_{i=1}^{n} w'_i (y_i - \alpha - \beta x_i)^2 $$

Hence, the coefficients of regression line $\alpha$ and $\beta$ can be obtained by minimizing squared error with respect to $\alpha$ and $\beta$. These values become

$$ \alpha = \frac{\sum w_i y_i - \beta \sum w_i x_i}{\sum w_i} $$

(3.7)
and
\[ \beta = \frac{\sum w_i (\sum y_i x_i) - \sum w_i x_i (\sum y_i)}{\sum w_i (\sum x_i^2) - (\sum x_i)^2} \]  \hfill (3.8)

where
\[ w_i = \sigma^2 w_i' = \frac{1}{g^2(x_i)} \]  \hfill (3.9)

An unbiased estimate of the unknown \( \sigma^2 \) is
\[ s^2 = \frac{\sum w_i (y_i - \alpha - \beta x_i)^2}{n-2} \]  \hfill (3.10)

Hence an estimate of the conditional variance is
\[ s_{y|x}^2 = s^2 g^2(x) \]  \hfill (3.11)

and
\[ s_{y|x} = sg(x) \]  \hfill (3.12)

In the present study, bridge displacement obtained from experiment is assumed to be actual displacement (i.e., variable \( X \)). Displacement obtained from numerical analysis is assumed to be variable \( Y \). From 36 values of the maximum and minimum displacement obtained at 3 bent tops during 6 high amplitude tests, parameters of two projected regression lines and standard deviation(s) are calculated. The regression equation is calculated to be
\[ y = 0.9958x - 0.3511 \]  \hfill (3.13)

with an unbiased standard deviation \( s = 0.209 \). Therefore the upper and lower bounds of regression line are \( y = 1.205x \) and \( y = 0.786x \), respectively. The regression line with bounds is shown in Figure 3-19. This shows that bridge response from numerical analysis is in good agreement with that measured during the experiment.
Figure 3-19. Regression analysis with measured and simulated bridge displacement.
Chapter 4 Fragility Analysis and Quantification of Uncertainty

4.1 Development of Fragility Curves of the Bridge

Time history analysis of the bridge is performed under 60 ground motion time histories. These motions were originally developed for the area of Los Angeles, CA for the purpose of Federal Emergency Management Agency (FEMA) SAC steel project\(^3\). The motions are categorized in 3 sets, each having 20 records of horizontal ground accelerations derived either from historical recordings or from simulations. Each set (of 20 records) is scaled linearly so as to have return periods of 2500 yrs, 475 yrs and 72 yrs (exceedance probabilities respectively of 2, 10 and 50% in 50 yrs).

To identify degraded fragility characteristics of the bridge after six high amplitude tests, all six numerical models of the bridge with effective stiffness are analyzed under 60 ground motion time histories. Thus, six independent sets of 60 time history analyses are performed. From each analysis, rotational time histories of the bridge are recorded at all pier ends. These response quantities are used to express bridge seismic performance when it is partially degraded due to the strike of earthquakes similar to those used in high amplitude tests.

Measured rotations are converted to rotational ductility \(\mu\). By definition, rotational ductility \(\mu\) is the ratio of rotation of the bridge pier to the yield rotation measured at the same location. The yield and ultimate rotation values for bridge piers are obtained from the initial moment rotation relationships (Figures 2-11, 2-12). Thus, \(\mu_{ult}\) corresponding to the ultimate state is calculated to be equal to 14.10 for piers in

\(^3\)http://nisee.berkeley.edu/data/strong_motion/sacsteel/ground_motions.html
Bent 1, 3 and 14.67 for piers in Bent 2. Rotational ductility at the yield state ($\mu_y$) is equal to 1.0 for all piers. Beyond yielding and before ultimate state, three intermediate states of bridge damage namely minor (or slight), moderate and major (or extensive) damage (definition follows HAZUS 1999 physical descriptions of bridge seismic damage) are considered. Threshold rotational ductility at these intermediate damage states are taken from a previous study on the same bridge model (Banerjee and Shinozuka 2008a). In this literature, bridge damage was categorized into four damage levels starting from no damage to major damage based on the nature of damage observed in bridge piers during high amplitude tests. This categorization followed the qualitative seismic damage state descriptions for bridges given in HAZUS (1999). Table 4-1 lists guidelines provided in Banerjee and Shinozuka (2008a) to categorize observed bridge damage from the experimental study. Thus threshold rotational ductility values for minor, moderate and major damage are given as

\[
\begin{align*}
\mu < 3.14; & \quad \text{No damage} \\
3.14 \leq \mu < 5.90; & \quad \text{Minor damage} \\
5.90 \leq \mu < 9.42; & \quad \text{Moderate damage} \\
\mu \geq 9.42; & \quad \text{Major damage and beyond}
\end{align*}
\]
Table 4-1: Definition of bridge damage states (Banerjee and Shinozuka 2008a)

<table>
<thead>
<tr>
<th>Damage states</th>
<th>Description of physical damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Damage</td>
<td>No damage in bridge piers</td>
</tr>
<tr>
<td>Minor Damage</td>
<td>Height of the pier flaking or spalling ≤ 80mm or exposure of pier reinforcement</td>
</tr>
<tr>
<td>Moderate Damage</td>
<td>Height of the pier spalling &gt; 80mm</td>
</tr>
<tr>
<td>Major Damage and Beyond</td>
<td>Significant spalling to buckling of reinforcing bar(s)</td>
</tr>
</tbody>
</table>

In fragility analysis, rotational ductility at bridge pier ends is taken as signature representing bridge seismic performance at different damage states. These values are calculated by dividing absolute maximum rotations from rotational time histories with yield rotations obtainable from moment rotation relationships. It should be noted here that piers in Bent 1 and 3 have yield rotation different from that in Bent 2. Therefore under a particular ground motion, twelve rotational ductility values are obtained at both ends of six bridge piers. The minimum amongst these twelve rotational ductility values is chosen for the fragility analysis. Bridge damage state under a ground motion is decided by comparing the rotational ductility value with the threshold limit for a damage state. For example, if rotational ductility is calculated to be more than 3.14 and less than 5.90 under a particular ground motion, the bridge is regarded to suffer from minor damage due to this event. If the rotational ductility exceeds 5.90, the bridge is no longer in minor damage state; it goes to a higher damage state.

Fragility curves at minor, moderate and major damage states of the bridge are developed using the maximum likelihood method as described in Section 2.3. The curves are shown in Figures 4-1 to 4-3. Corresponding median values are given in
Table 4-2. The log-standard deviation is taken as 0.5 for all fragility curves to avoid crossing of any two curves in the same figure. By doing so, bridge fragility characteristics can be compared in terms of the median value of fragility curves.

Table 4-2: Median ground motion values for each damage state after each test

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Median Value in g</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T12</td>
</tr>
<tr>
<td>minor</td>
<td>0.5428</td>
</tr>
<tr>
<td>moderate</td>
<td>1.1579</td>
</tr>
<tr>
<td>major</td>
<td>2.051</td>
</tr>
</tbody>
</table>

Figure 4-1. Bridge fragility curves at minor damage state after high amplitude tests
Figure 4-2. Bridge fragility curves at moderate damage state after high amplitude tests

Figure 4-3. Bridge fragility curves at major damage state after high amplitude tests

These fragility curves represent bridge vulnerability under seismic ground motions. As these curves move towards left (i.e., median value decreases), bridge
seismic vulnerability increases. It is evident from these figures that bridge vulnerability increases after each high amplitude test. This is quite obvious because the effective stiffness of the bridge degrades in a progressive manner as high aptitude tests proceeded. During the experiment, not much degradation was observed in test T12 to T14. This reflects properly in bridge fragility characteristics. For all three damage states, median fragility parameters do not have significant change from T12 to T14. Following T15, bridge effective stiffness degrades to a large extent so as the fragility curves. After T17 and T18, bridge seismic performance in major damage changes rapidly indicating a very high probability of having bridge failure at higher damage state. The similar phenomenon is observed during the experiment. After T18, more than one bar buckled in bridge piers, which was regarded as the failure of the bridge.

4.2 Quantification of Uncertainty in Fragility Curves

Obtained seismic fragility characteristics of the bridge largely depend on available damage data. Insufficient data may introduce statistical uncertainty in fragility curves. To quantify the statistical variations of these fragility parameters, 90% confidence intervals (corresponding to 5% and 95% statistical confidence) of fragility curves are developed by performing Monte Carlo simulation (Banerjee and Shinozuka 2008b). In the first step, 512 values of ground motion intensity parameter $a^*$ and corresponding failure probability of the bridge $b^*$ are generated. It is considered that $a^*$ and $b^*$ are uniformly distributed in the range of 0-1.0g and 0-1, respectively. For a particular set of $a^*_i$ and $b^*_i$ ($i = 1$ to 512), bridge damage condition $(x^*_i)$ ata damage state
\( k \) (1 for minor, 2 for moderate, 3 for major damage) is obtained as follows (Banerjee and Shinozuka 2008b):

At minor damage

\[
x_{i1}^* = \begin{cases} 
1, & \text{if } \Phi \left[ \frac{\ln(a_i^*/c_2)}{0.05} \right] < b_i \leq \Phi \left[ \frac{\ln(a_i^*/c_1)}{0.05} \right] \\
0, & \text{otherwise}
\end{cases}
\] (4.1)

At moderate Damage

\[
x_{i2}^* = \begin{cases} 
1, & \text{if } \Phi \left[ \frac{\ln(a_i^*/c_3)}{0.05} \right] < b_i \leq \Phi \left[ \frac{\ln(a_i^*/c_2)}{0.05} \right] \\
0, & \text{otherwise}
\end{cases}
\] (4.2)

And at major Damage

\[
x_{i3}^* = \begin{cases} 
1, & \text{if } b_i \leq \Phi \left[ \frac{\ln(a_i^*/c_3)}{0.05} \right] \\
0, & \text{otherwise}
\end{cases}
\] (4.3)

where \( c_1, c_2, \) and \( c_3 \) are median fragility parameters of the bridge at minor, moderate and major damage states (Table 4-2). This process is repeated for all 512 sets of \( a^* \) and \( b^* \) and 512 values of \( x_{i1}^* \) are obtained.

In the second step, fragility analysis is performed with 512 values of \( x_{i1}^* \) and \( \alpha^* \) as described in section 2.3 and a set of median fragility parameters \( c_k^* \) is obtained at minor, moderate and major damage states of the bridge.

In the third step, the entire process described in first and second steps is repeated for 500 times to generate 500 values of \( c_k^* \). These realizations are plotted in lognormal probability papers as shown in Figures A-1 to A-18, which indicate that the simulated values fit satisfactorily in lognormal distribution function. From these figures, median fragility parameters (\( c \)) corresponding to 0.95, 0.50 and 0.05 exceedance probabilities (i.e., for 5%, 50% and 95% statistical confidence) can be obtained which are given in
Table 4-3. Obtained fragility curves with 90% confidence band are shown in Figures 4-4 to 4-21. It can be seen from these figures that the statistical variations of fragility curves at any particular damage state are not significant. Relatively higher variation is observed at higher damage states than lower damage states. This is mainly due to less number of observed bridge failure at higher damage states than lower damage states when 60 ground motions are applied.
Table 4-3: Median fragility parameters with 5%, 50% and 95% statistical confidence

<table>
<thead>
<tr>
<th>Damage States</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>5%</td>
<td>50%</td>
<td>95%</td>
</tr>
<tr>
<td></td>
<td>T12</td>
<td>0.577</td>
<td>0.542</td>
<td>0.509</td>
</tr>
<tr>
<td></td>
<td>T13</td>
<td>0.577</td>
<td>0.542</td>
<td>0.509</td>
</tr>
<tr>
<td>Minor Damage</td>
<td>T14</td>
<td>0.558</td>
<td>0.526</td>
<td>0.496</td>
</tr>
<tr>
<td></td>
<td>T15</td>
<td>0.557</td>
<td>0.526</td>
<td>0.497</td>
</tr>
<tr>
<td></td>
<td>T17</td>
<td>0.379</td>
<td>0.355</td>
<td>0.333</td>
</tr>
<tr>
<td></td>
<td>T18</td>
<td>0.247</td>
<td>0.230</td>
<td>0.214</td>
</tr>
<tr>
<td>Moderate Damage</td>
<td>T12</td>
<td>1.255</td>
<td>1.163</td>
<td>1.077</td>
</tr>
<tr>
<td></td>
<td>T13</td>
<td>1.255</td>
<td>1.163</td>
<td>1.077</td>
</tr>
<tr>
<td></td>
<td>T14</td>
<td>1.184</td>
<td>1.104</td>
<td>1.030</td>
</tr>
<tr>
<td></td>
<td>T15</td>
<td>1.033</td>
<td>0.968</td>
<td>0.908</td>
</tr>
<tr>
<td></td>
<td>T17</td>
<td>0.710</td>
<td>0.672</td>
<td>0.636</td>
</tr>
<tr>
<td></td>
<td>T18</td>
<td>0.557</td>
<td>0.525</td>
<td>0.495</td>
</tr>
<tr>
<td>Major Damage</td>
<td>T12</td>
<td>2.461</td>
<td>2.081</td>
<td>1.760</td>
</tr>
<tr>
<td></td>
<td>T13</td>
<td>2.461</td>
<td>2.081</td>
<td>1.760</td>
</tr>
<tr>
<td></td>
<td>T14</td>
<td>2.461</td>
<td>2.081</td>
<td>1.760</td>
</tr>
<tr>
<td></td>
<td>T15</td>
<td>2.461</td>
<td>2.081</td>
<td>1.760</td>
</tr>
<tr>
<td></td>
<td>T17</td>
<td>1.316</td>
<td>1.217</td>
<td>1.126</td>
</tr>
<tr>
<td></td>
<td>T18</td>
<td>0.818</td>
<td>0.769</td>
<td>0.723</td>
</tr>
</tbody>
</table>
Figure 4.4. Bridge fragility curves with 90% confidence interval at minor damage state after T12

Figure 4.5. Bridge fragility curves with 90% confidence interval at moderate damage state after T12
Figure 4-6. Bridge fragility curves with 90% confidence interval at major damage state after T12

Figure 4-7. Bridge fragility curves with 90% confidence interval at minor damage state after T13
Figure 4-8. Bridge fragility curves with 90% confidence interval at moderate damage state after T13.

Figure 4-9. Bridge fragility curves with 90% confidence interval at major damage state after T13.
Figure 4-10. Bridge fragility curves with 90% confidence interval at minor damage state after T14

Figure 4-11. Bridge fragility curves with 90% confidence interval at moderate damage state after T14
Figure 4-12. Bridge fragility curves with 90% confidence interval at major damage state after T14

Figure 4-13. Bridge fragility curves with 90% confidence interval at minor damage state after T15
Figure 4-14. Bridge fragility curves with 90% confidence interval at moderate damage state after T15

Figure 4-15. Bridge fragility curves with 90% confidence interval at major damage state after T15
Figure 4-16. Bridge fragility curves with 90% confidence interval at minor damage state after T17

Figure 4-17. Bridge fragility curves with 90% confidence interval at moderate damage state after T17
Figure 4-18. Bridge fragility curves with 90% confidence interval at major damage state after T17

Figure 4-19. Bridge fragility curves with 90% confidence interval at minor damage state after T18
Figure 4-20. Bridge fragility curves with 90% confidence interval at moderate damage state after T18

Figure 4-21. Bridge fragility curves with 90% confidence interval at major damage state after T18
Chapter 5 Summary and Conclusions

This study developed a procedure to determine fragility characteristics of a structure under ambient vibration or earthquake through the use of structural vibration response recorded from dynamic testing or real-time monitoring. Acceleration response of a two-span reinforced concrete bridge recorded from a large-scale shake table experiment is analyzed here through Extended Kalman filter (EKF) and post-event effective stiffness of the bridge at pier ends is evaluated. This effective stiffness is taken here as a measure of performance degradation of the bridge due to seismic events. Numerical simulations are performed to generate bridge damage data after it experiences earthquakes equivalent to those used in the experiment. Seven finite element models of the bridge are developed that simulate pre- and post-damaged conditions of the bridge. Bridge model at pre-damaged condition contains the initial design properties. This model is updated with effective stiffness of the bridge obtained from EKF and numerical bridge model with post-damaged conditions are generated. Accuracy of these numerical models is verified by comparing bridge response from numerical analysis and experimental program. Comparison shows an excellent agreement between bridge response obtained from numerical simulation and experimental observation.

Time history analyses of the bridge are performed under 60 ground motion time histories and bridge damage conditions at minor, moderate and major damage states are recorded. Fragility analysis is performed to generate bridge fragility curves that represent the failure probability of the bridge at a certain damage state under seismic
ground motions. Statistical uncertainty of these fragility curves are obtained by performing Monte Carlo simulations. This uncertainty is measured in terms of 90% confidence interval that corresponds to 5% and 95% statistical confidence levels. Results show that statistical variations of median fragility parameters are not significant at any damage state.

Fragility curves show the increase of bridge seismically vulnerable due to sequential seismic events. Therefore, for nonlinear performance evaluation of an existing structure under external loading, consideration of its initial (i.e., pre-damaged) condition may overestimate the actual capacity of the structure.

5.1. Research Significance

The research outlines a unique methodology that utilizes vibration measurements of structures for their condition assessment and fragility characterization. The numerical technique adopted here allows the development of fragility curves based on a limited set of data recorded during experiment or real-time monitoring. Such fragility curves represent vulnerability of structures at their present state under similar events and are useful for the evaluation of structural risk and reliability under similar future events.
References


Appendix: Lognormal probability plot of fragility parameter (c)

Figure A-1. Probability plot for c in minor damage after Test12

Figure A-2. Probability plot for c in minor damage after Test13
Figure A-3. Probability plot for \( c \) in minor damage after Test14

Figure A-4. Probability plot for \( c \) in minor damage after Test15
Figure A-5. Probability plot for c in minor damage after Test17

Figure A-6. Probability plot for c in minor damage after Test18
Figure A-7. Probability plot for c in moderate damage after Test12

Figure A-8. Probability plot for c in moderate damage after Test13
Figure A-9. Probability plot for c in moderate damage after Test14

Figure A-10. Probability plot for c in moderate damage after Test15
Figure A-11. Probability plot for c in moderate damage after Test17

Figure A-12. Probability plot for c in moderate damage after Test18
Figure A-13. Probability plot for c in major damage after Test1.

Figure A-14. Probability plot for c in major damage after Test13.
Figure A-15. Probability plot for c in major damage after Test14

Figure A-16. Probability plot for c in major damage after Test15
Figure A-17. Probability plot for c in major damage after Test17

Figure A-18. Probability plot for c in major damage after Test18