SEISMIC VULNERABILITY ASSESSMENT FOR CONCRETE BRIDGES BY DEVELOPING FRAGILITY CURVES

Araliya Mosleh\(^{1(*)}\), Humberto Varum\(^{2}\), José Jara\(^{3}\), Mehran S. Razzaghi\(^{2}\)

\(^{1}\)Department of Civil Eng., Faculty of Engineering, University of Aveiro, Aveiro, Portugal.
\(^{2}\)CONSTRUCT-LESE, Department of Civil Eng., Faculty of Engineering, University of Porto, Porto, Portugal.
\(^{3}\)Department of Civil Eng., Faculty of Engineering, Universidad Michoacana de San Nicolas de Hidalgo, Morelia, Mexico.
\(^{(*)}\)Email: a_mmosleh@yahoo.com

ABSTRACT
This work focuses on the development of analytical fragility curves for ordinary highway bridges subjected to reverse and strike-slip fault mechanisms and classified in terms of the column height, reinforcement’s yield strength and concrete compressive strength. The results show that the column height have significant effect on seismic vulnerability of ordinary highway concrete bridges. Additionally, the reverse fault records produce larger bridge demands than those of the bridges subjected to the strike-slip accelerograms, increasing the seismic vulnerability as well.

Keywords: Seismic vulnerability, concrete bridges, fragility curves.

INTRODUCTION
Seismic vulnerability assessment of the highway bridges located in areas of high seismic hazards plays an important role for the safety of transportation systems. One of the most critical issue in pre-earthquake planning, and post-earthquake response of the transportation system is qualitative and quantitative assessment of the seismic risk in the highway bridge systems. During the past decades several bridges damaged due to the occurrence of earthquakes (Eshghi, 2004; Nicknam, 2011; Yang, 2015), therefore, the expected seismic performance of bridges attracted several researchers during the last decades (Jara, 2011; Varum, 2011; Lin, 2015). The action of earthquake loads on highway infrastructure systems has typically investigated by previous researched due to economic losses and closure time (Basoz, 1997; Mackie, 2005; Luna, 2008; Liao, 2010; Padgett, 2010; Zhou, 2010). Such assessments produce valuable knowledge about a number of important effects of earthquakes in terms of effect on the regions’ economy, traffic disruption of the transportation system and post-earthquake response and recovery (Bruneau, 2003). Nations’ freight economy on highway bridges combined with awareness of the seismic hazard in the region and appropriate considerate of their seismic response and vulnerability are the important issues in risk assessment. Fragility curves relate strong motion severity to the probability of reaching or exceeding a certain limit state. These curves found widespread use in probabilistic seismic risk assessment of highway bridges. A single intensity measure, such as peak ground acceleration (PGA), or spectral acceleration by considering the geometric mean of the two horizontal components of the ground motions can be used as seismic intensities. Fragility curves have a current and potential future to be employed as an application including: (a):
emergency response such as priority in bridge inspection, (b): design support and performance based earthquake engineering and (c): planning support like traffic impacts from scenario earthquakes, or cost effectiveness of alternate bridge retrofit strategies and additional seismic retrofit needs.

To perform the seismic vulnerability assessment of concrete bridges, probabilistic approaches and fragility curves can be utilized. A great number of highway bridges around the world does not meet the seismic detailing requirements recommended in current codes and guidelines (Caltrans, 2013b). To evaluate the seismic fragility of bridges different approaches are used by previous researchers such as judgmental or expert-based fragility curve (Rossetto, 2003), empirical fragility curves based on site damages due to the post-earthquake damage statistical data (Shinozuka, 2000b; Yazgan, 2015), analytical fragility curve methods such as elastic spectral method (Hwang, 2000), nonlinear static analysis (Moschonas, 2009), nonlinear time history analysis (Ramanathan, 2012; Yang, 2015; Mosleh, 2016) and incremental dynamic analysis (Billah, 2013). Although nonlinear response history analysis is the most computational time-consuming method, but it is the most reliable as well (Shinozuka, 2000b).

This study determines the seismic vulnerability of existing old concrete bridges. The research work presents statistical analysis, classification of concrete bridges, ground motion selection, damage state definition, real construction practices, results of nonlinear dynamic analyses and finally the fragility curves to assess the seismic vulnerability of common concrete bridges subjected to reverse and strike-slip faults.

CLASSIFICATION OF BRIDGES

Fragility curves are performed for ordinary highway bridges in Iran constructed after the 1980s in order to assess their seismic vulnerability. A general understanding of ordinary highway bridges in Iran constructed based on old codes in terms of structural properties, as well as their seismic behavior, is essential to develop fragility curves. Since existing bridges have their own characteristics based on structural properties therefore, each bridge has different seismic behavior. Hence, it is difficult to evaluate in detail the seismic performance of each bridge in a large inventory subjected to earthquake actions. Although each bridge has its own structural characteristics, they have some similarities in some features. Based on that, the bridges in this study are grouped into different classes in terms of their structural characteristics, which are expected to have similar seismic response for a specific class. Bridge classification helps to peruse each type of bridge in detail instead of investigating all bridge samples individually in the inventory. The selected 56 bridges are defined as Ordinary Standard Bridges, as per Caltrans (Caltrans, 2013a), due to the following properties these bridges:

- Span lengths inferior to 90m
- Constructed with normal weight concrete girder, and column or pier elements
- Horizontal members are supported on conventional bearings
- Without no-standard components such as; dropped bent caps, integral bent caps terminating inside the exterior girder, outrigger bents; offset columns; isolation bearings or dampers
- Foundations supported on spread footing or pile cap with piles
- Soil is not susceptible to liquefaction, lateral spreading, or scour
Figure 1 presents a histogram of the investigated structural characteristics. Table 1 shows the bridge classes selected for this research.

![Histogram of structural characteristics](image)

**Table 1 - Selected bridge classes**

<table>
<thead>
<tr>
<th>Column height (m)</th>
<th>Column section (circular)</th>
<th>Longitudinal steel bar (%)</th>
<th>Deck width (m)</th>
<th>Span length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16-21</td>
<td>D=1.4</td>
<td>0.91</td>
<td>12-16</td>
<td>20-32</td>
</tr>
</tbody>
</table>

In terms of material properties bridges are classified based on three real cases with column heights in the range of 10.5–21 m, and variation of material properties, which produced eighteen bridge samples. The compressive strengths of concrete is 20, 25, and 30 MPa and yield strength of the steel ranges between 300 and 400 MPa.

**ANALYTICAL MODELING OF TYPICAL BRIDGES AND LIMIT STATES**

To perform analytical fragility curves, numerical analyses of the bridge samples are formed by considering a proper structural component model. Nonlinear behavior of the structure is obtained directly from the nonlinear stress-strain relationship of concrete and steel; therefore the reliability of nonlinear bridge members depends on the accuracy of the material properties considered. Reinforcing steel bars are modeled utilizing bilinear steel material model with kinematic hardening according to Caltrans recommendation (Caltrans, 2013a). For each real case study yield and ultimate strength with real values obtained from experimental tests was employed. Modulus of elasticity is 200 GPa, nominal yield strain ($\varepsilon_y$) and expected yield strain ($\varepsilon_{ye}$) are 0.0021 and 0.0023, respectively. Ultimate tensile strain ($\varepsilon_{su}$) which is bar size dependent is 0.12. For the confined concrete previous researchers developed different stress-strain relationships (Kent, 1971; Bazant, 1976; sheikh, 1980; Mander, 1988). Some of the proposed methods have restriction in range of applicability (e.g., circular or rectangular...
sections); however the method suggested by Mander et al. (Mander, 1988) applies to all section shapes. Note that tensile strength of concrete members are neglected. Strain compressive strength and ultimate strain capacity at unconfined concrete are 0.002 and 0.005, respectively. In this study the confined and unconfined concrete strength parameters for the columns are estimated using the approach described by Mander et al. (Mander, 1988).

85-90 percent of the bridge total mass is constituted by the mass of superstructure. To estimate the actual seismic behavior of the bridges, vertical rigid elements are utilized between superstructure mass and the substructure components in the analytical model.

The model includes the P-delta effects to consider the increase of seismic demands in the columns. Mass and stiffness-proportional Rayleigh damping coefficients are determined considering the first two modal periods, and the hysteretic damping is included. Elastic–perfectly plastic idealization generated with the SAP2000 program (Computers and Structures Inc. CSI. SAP2000 V-14, 2009) was considered. Seismic damage is classified using four damage states, as described by Hwang et al. (Hwang, 2001; Mosleh, 2015). In order to quantify damage states (as describe in Table 2), the relative displacement ductility ratio of a column is used. In this table $L_p$ is the plastic hinge length estimated according to Priestley et al. (Priestley, 1996), $\phi_1$ and $\phi_y$ are the curvature correspondent to the relative displacement of a column when the vertical reinforcing bars at the bottom of the column reach the first yield and yield respectively. $\phi_3$ is the curvature of a column when $\epsilon_c = 0.004$, $\mu_c1$, denotes the first limit state corresponding to a first yield displacement ductility ratio equal to 1. The second damage state, $\mu_c2$, represents the yield displacement ductility ratio. The displacement ductility corresponding to the third damage state ($\mu_c3$) is the displacement ductility ratio corresponding to $\epsilon_c = 0.004$ for the columns, where $\epsilon_c$ is the compressive strain at the concrete column. Finally, $\mu_c4$ can be calculated as $\mu_c3 + 3$.

<table>
<thead>
<tr>
<th>Column shape</th>
<th>Column height</th>
<th>Direction</th>
<th>$L_p$ (m)</th>
<th>$\phi_1$</th>
<th>$\phi_y$</th>
<th>$\phi_3$</th>
<th>$\mu_c1$</th>
<th>$\mu_c2$</th>
<th>$\mu_c3$</th>
<th>$\mu_c4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>1.5</td>
<td>E-03</td>
<td>2.35</td>
<td>0.1</td>
<td>1.1</td>
<td>1.31</td>
<td>2.27</td>
<td>5.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transvers</td>
<td>0.9</td>
<td>E-03</td>
<td>2.35</td>
<td>0.1</td>
<td>1.1</td>
<td>1.31</td>
<td>2.44</td>
<td>5.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1.78</td>
<td>E-03</td>
<td>3.18</td>
<td>0.1</td>
<td>1.0</td>
<td>1.25</td>
<td>1.79</td>
<td>4.79</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transvers</td>
<td>1.07</td>
<td>E-03</td>
<td>3.18</td>
<td>0.1</td>
<td>1.0</td>
<td>1.25</td>
<td>1.89</td>
<td>4.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1.9</td>
<td>E-03</td>
<td>2.37</td>
<td>0.1</td>
<td>1.2</td>
<td>1.31</td>
<td>2.21</td>
<td>5.21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transvers</td>
<td>1.1</td>
<td>E-03</td>
<td>2.37</td>
<td>0.1</td>
<td>1.2</td>
<td>1.31</td>
<td>2.35</td>
<td>5.35</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
SELECTION OF GROUND MOTION

A total of 40 ground motion (20 records of reverse faults and 20 records belong to strike slip sources) recorded in Iran and other regions having similar faulting mechanisms and seismic potential are selected without applying any scaling to represent the record-to-record variability. Each ground motion have two horizontal orthogonal components. Table 3 shows important characteristics of the selected earthquakes. Two horizontal orthogonal components are considered in the nonlinear time history analyses. On the other hand the intensity measure of each ground motion is determined by calculating the geometric mean of the intensity measures of the two horizontal components of the ground motion. Each bridge was subjected to two orthogonal horizontal components of the ground motions. Since each bridge is analyzed twice by each ground motion record (two horizontal orthogonal components) to obtain the maximum response, a total of 80 analyses for each bridge are performed.

Table 3 - Some important parameters of the selected earthquake ground motions

<table>
<thead>
<tr>
<th>Earthquake belong to reverse fault</th>
<th>Year</th>
<th>$M_w$</th>
<th>PGA</th>
<th>Earthquake belong to strike slip fault</th>
<th>Year</th>
<th>$M_w$</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chi-Chi-0.1125g</td>
<td>1999</td>
<td>7.62</td>
<td>0.1125</td>
<td>Morgan Hill-0.0983g</td>
<td>1984</td>
<td>6.19</td>
<td>0.0983</td>
</tr>
<tr>
<td>Chi-Chi-0.1348g</td>
<td>1999</td>
<td>7.62</td>
<td>0.1348</td>
<td>Parkfield-0.469g</td>
<td>2004</td>
<td>6.0</td>
<td>0.469</td>
</tr>
<tr>
<td>Chi-Chi-0.1748g</td>
<td>1999</td>
<td>7.62</td>
<td>0.1748</td>
<td>Parkfield-0.602g</td>
<td>2004</td>
<td>6.0</td>
<td>0.602</td>
</tr>
<tr>
<td>Chi-Chi-0.2058g</td>
<td>1999</td>
<td>7.62</td>
<td>0.2058</td>
<td>Manjil-0.5051g</td>
<td>1990</td>
<td>7.4</td>
<td>0.5051</td>
</tr>
<tr>
<td>Chi-Chi-0.2595g</td>
<td>1999</td>
<td>7.62</td>
<td>0.2595</td>
<td>Morgan Hill-0.2814g</td>
<td>1984</td>
<td>6.19</td>
<td>0.2814</td>
</tr>
<tr>
<td>Chi-Chi-0.3643g</td>
<td>1999</td>
<td>7.62</td>
<td>0.3643</td>
<td>Morgan Hill-0.3426g</td>
<td>1984</td>
<td>6.19</td>
<td>0.3426</td>
</tr>
<tr>
<td>Chi-Chi-0.5283g</td>
<td>1999</td>
<td>7.62</td>
<td>0.5283</td>
<td>Kobe-0.7105g</td>
<td>1995</td>
<td>6.9</td>
<td>0.7105</td>
</tr>
<tr>
<td>Chi-Chi-0.0823g</td>
<td>1999</td>
<td>7.62</td>
<td>0.0823</td>
<td>Imperial Valley-0.176g</td>
<td>1979</td>
<td>6.53</td>
<td>0.176</td>
</tr>
<tr>
<td>Northridge-0.2148g</td>
<td>1994</td>
<td>6.69</td>
<td>0.2148</td>
<td>Duzce-0.1445g</td>
<td>1999</td>
<td>7.14</td>
<td>0.1445</td>
</tr>
<tr>
<td>Northridge-0.3908g</td>
<td>1994</td>
<td>6.69</td>
<td>0.3908</td>
<td>Victoria-0.5722g</td>
<td>1980</td>
<td>6.33</td>
<td>0.5722</td>
</tr>
<tr>
<td>Northridge-0.4673g</td>
<td>1994</td>
<td>6.69</td>
<td>0.4673</td>
<td>Parkfield-0.2934g</td>
<td>1966</td>
<td>6.19</td>
<td>0.2934</td>
</tr>
<tr>
<td>Northridge-0.4898</td>
<td>1994</td>
<td>6.69</td>
<td>0.4898</td>
<td>Landers-0.1407g</td>
<td>1992</td>
<td>7.28</td>
<td>0.1407</td>
</tr>
<tr>
<td>Northridge-0.5102g</td>
<td>1994</td>
<td>6.69</td>
<td>0.5102</td>
<td>Landers-0.3733g</td>
<td>1992</td>
<td>7.28</td>
<td>0.3733</td>
</tr>
<tr>
<td>Northridge-0.5908g</td>
<td>1994</td>
<td>6.69</td>
<td>0.5908</td>
<td>Kobe-0.0765g</td>
<td>1995</td>
<td>6.9</td>
<td>0.0765</td>
</tr>
<tr>
<td>Sanfernando-0.2994g</td>
<td>1971</td>
<td>6.61</td>
<td>0.2994</td>
<td>Duzce-0.2101g</td>
<td>1999</td>
<td>7.14</td>
<td>0.2101</td>
</tr>
<tr>
<td>Whittier Narrows-0.3408g</td>
<td>1987</td>
<td>5.99</td>
<td>0.3408</td>
<td>Duzce-0.7367g</td>
<td>1999</td>
<td>7.14</td>
<td>0.7367</td>
</tr>
<tr>
<td>Capemendocino-0.1668g</td>
<td>1992</td>
<td>7.01</td>
<td>0.1668</td>
<td>Parkfield-0.271g</td>
<td>2004</td>
<td>6.0</td>
<td>0.271</td>
</tr>
<tr>
<td>Capemendocino-0.4244g</td>
<td>1992</td>
<td>7.01</td>
<td>0.4244</td>
<td>Imperial Valley-0.166g</td>
<td>1979</td>
<td>6.53</td>
<td>0.1661</td>
</tr>
<tr>
<td>Tabas-0.3505g</td>
<td>1978</td>
<td>7.4</td>
<td>0.3505</td>
<td>Duzce-0.1174g</td>
<td>1999</td>
<td>7.14</td>
<td>0.1174</td>
</tr>
<tr>
<td>Tabas-0.8128g</td>
<td>1978</td>
<td>7.4</td>
<td>0.8128</td>
<td>Kojaeli-0.1387g</td>
<td>1999</td>
<td>7.51</td>
<td>10.75</td>
</tr>
</tbody>
</table>

FRAGILITY CURVES

Figs. 2 and 3 present the result of fragility curves in terms of material properties subjected to reverse and strike slip fault due to different damage states. The results show that the response of bridge is more sensitive to steel yield strength than concrete compressive strength. outcome is consistent with the bridge responses observed by previous researches (Pan, 2010a).
Fig. 2 - Fragility curves based on different $f_c$: (a) LS-1, (b) LS-2, (c) LS-3, (d) LS-3

Fig. 3 - Fragility curves based on different $f_y$: (a) LS-1, (b) LS-2, (c) LS-3, (d) LS-3
Fig. 4 shows fragility curves for selected bridge classification subjected to reverse and strike-slip fault. The results show that bridges are more vulnerable to reverse fault records than to strike fault accelerograms. One reason could be that maximum amplitude of the mean SA response spectrum of reverse records is greater than the mean SA value of a strike-slip fault.

RESULTS AND CONCLUSIONS

This study focuses on the development of analytical fragility curves for the highway bridges in Iran. In this study, the most common bridges in Iran constructed in 1980s are selected and classified in terms of span length, deck width and number of girders. The parameters considered for grouping the bridge structures are the column height, reinforcement yield strength, and concrete compressive strength. Nonlinear time history analyses using 3-D models were conducted for each set of bridge samples subjected to earthquake ground motions with different intensities. The selected family of seismic records originates in strike-slip and reverse fault seismic sources. The maximum absolute ductility demand determines the damage limit state of the bridge column for each seismic record.

We also analyze the influence of the earthquake mechanism on the seismic vulnerability of the bridges, using as a performance parameter the displacement ductility demand of the piers. To calculate the maximum seismic response of the bridge components obtained from nonlinear analyses, the engineering demand parameters are also considered. Based on this parameter, damage states were established to determine fragility curves that evaluate the probability of reaching or exceeding the structural capacity as a function of a seismic intensity under each ground motion record, which is represented by the intensity measures of PGA. A cumulative lognormal probability distribution is utilized to present the probability of exceeding a certain damage limit state in recent studies. Consequently, the fragility curves are generated for each bridge class based on each limit state and intensity measure. For each curve the coefficients of determination are computed due to the investigated intensity measures. Based on the results, the following conclusions can be drawn:

The fragility curves of common highway bridge classes’ transportation system in Iran can be employed for post-earthquake emergency response plans pre-earthquake preparedness plans.
The results show that column height had a noteworthy effect on the seismic vulnerability of the bridges, however material properties, especially concrete compressive strength, did not have a particular impact on the seismic vulnerability of the structures.

Additionally, pre-1990 bridges subjected to reverse fault records were more vulnerable than the ones subjected to the strike-slip fault accelerograms.

REFERENCES


