THE INFLUENCE OF SEISMIC SOURCES ON SEISMIC VULNERABILITY ASSESSMENT OF RC BRIDGES

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ABSTRACT
This paper focuses on a probabilistic fragility analysis for two groups of bridges: simply supported and continuous bridges subjected to reverse and strike-slip faults respectively. 3-D nonlinear analyses were performed for each set of bridge samples by considering the displacement ductility of the piers. To perform the fragility curves, uncertainties due to the presence of lap splices in columns and superstructure type in terms of continuous or simply supported spans were considered. The results show that the simply supported bridges perform consistently better from a seismic perspective than continuous bridges.

Keywords: Fragility curves, simply supported & continuous bridges, seismic sources.

INTRODUCTION
Recently continuous bridges with expansion joins become more popular alternative to common bridges. Even though the construction of continuous bridge are growing in recent decades, there are still several deficiencies in different countries in existing regulations and technical issues (Denton, 2012). Although evaluation of seismic performance of bridges with different type of superstructures is provided by previews researchers (Nicknam, 2011; Varum, 2011; Jara, 2012; Lin, 2015; Ramanathana, 2015; Mosleh, 2016), few studies to compare the seismic performance of continues bridges and simply support due to different seismic sources are carried out (Choine, 2015).

Continuous bridges have some advantages, namely (a): elimination of the unseating superstructure problem, (b): elimination of expansion joints associated expansion bearings which leads to decrease in structure life and maintenance costs, (c): avoidance in corrosion problem from the water which run-off from superstructure to substructure through expansion joints (Mistry, 2005), (d): elimination of the unseating superstructure from substructure. In general, continuous bridges provide lower operating costs and more durability performance. Therefore seismic vulnerability of continuous bridges is necessary.

Seismic vulnerability of highway bridges is usually performed by constructing fragility curves with a probabilistic analysis. Fragility curves relate strong motion severity to the probability of reaching or exceeding a certain limit state (Razzaghi, 2014). In the past decades fragility functions have been employed by researches for buildings; however less investigation was performed for the study of bridges. Therefore, development of fragility curves, particularly in some specific classes of bridges should be a high priority in the research activities.
The main objective of this study is to determine analytical fragility curves for two classes of bridges in Iran subjected to different seismic sources. First, nonlinear time-history analyses of 3-D analytical models are performed. Next, the selection of earthquake ground motion records from two seismic sources and definition of damage limit states are presented. Finally, the results conduct to obtain fragility curves by fitting a curve to a lognormal distribution function. Then, the proposed fragility curves are compared with two different seismic sources: reverse and strike-slip.

**BRIDGE CHARACTERISTICS**

Fragility curves are performed for two groups of concrete bridges based on different earthquake databases. Two major bridge classes are analysed: simply supported structures (SSB) on elastomeric bearings at the abutments and column bents and continuous bridges (CB). In the following, a brief explanation is presented for each group of bridge classification.

Fig. 1 presents the schematic drawings of sample bridges in the longitudinal and transverse directions. Note that different bridge lengths and column heights are also included in the models. Table 1 presents overall dimensions of the bridges and structural attributes for each bridge classification.

![Fig. 1 - General characteristics of (a) simply supported bridge, (b) continuous bridge, (c) transverse view of simply supported bridge and (d) transverse view of continuous bridge](image-url)
Table 1 - The overall dimensions of the bridges for 2 bridge classification

<table>
<thead>
<tr>
<th>Bridge classes</th>
<th>Column Height (H_col), (m)</th>
<th>Column section</th>
<th>Longitudinal steel ratio (%)</th>
<th>Deck width (W), (m)</th>
<th>Span length (L), (m)</th>
<th>Number of spans</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB</td>
<td>4-8</td>
<td>Circular (D=1.0)</td>
<td>1.125</td>
<td>22.5</td>
<td>20</td>
<td>4</td>
</tr>
<tr>
<td>SB</td>
<td>6-10.5</td>
<td>Circular (D=1.2-1.3)</td>
<td>1.06-1.56</td>
<td>12-16</td>
<td>20-32</td>
<td>4-6</td>
</tr>
</tbody>
</table>

Structural components are considered to create 3-D analytical bridge models. Generally, superstructure and substructure can be considered as structural components. Superstructure is composed of cast-in place reinforced concrete girders. Bent system and abutments constitute the substructure of the bridge. Elastomeric bearings are located between the substructure and superstructure as an isolation unit. Link elements modelled as fixed springs are used to locate the exact position of the bearings. Rigid elements are utilized at the rigid zone of the cap beams, columns and superstructure end connections. One of the important issue which should be considered is selecting the realistic value for the stiffness of rigid members. By employing low stiffness value for rigid elements, the analytical models can be incorrect. If a large elastic stiffness is selected, some problem in terms of numerical convergence may be appeared. To minimize numerical problem, according to Wilson (Wilson, 2002), the stiffness of rigid elements should not be 100 times over than adjacent elements. Therefore, stiffness of the rigid elements is specified accordingly.

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 are depends on the accuracy of the material properties which are considered. Bilinear steel material model with kinematic hardening based on Caltrans recommendation (Caltrans, 2013) is utilized to model reinforcing steel bars. The confined and unconfined concrete strength parameters for the columns are estimated using the approach described by Mander et al. (Mander, 1988).

P-delta effects are included in the model to consider the second order effects in columns. For the response-history analysis of the bridges the mass- and stiffness-proportional Rayleigh damping coefficients for the first two modal periods are calculated. Frame elements are utilized to model columns and the nonlinear behaviour of the columns is considered with a concentrated plasticity model by assigning plastic hinges at both column ends, as recommended in Caltrans code (Caltrans, 2004). The nonlinear behaviour of plastic elements at the column ends are described by considering moment-curvature diagram in SAP2000. The behaviour of the column is assumed to be linear outside the plastic hinge length.

Elastic springs in the longitudinal and transverse directions are employed to model the abutments and backfill soil due to the Caltrans recommendation (Caltrans, 2013). Between the superstructure and substructure components elastomeric bearings are located without any dowel or connecting device as shown in Fig.2. Bearings locate under each of the concrete girder of the superstructure. The vertical stiffness for bearing is provided by the internal steel plates, referred to as shims, and reduce the lateral bulging of the bearing as well. To model the vertical and lateral bearing stiffness, spring elements are selected, using the as proposed by Priestley et al. (Priestley, 1996b).
GROUNDED MOTION SELECTION

To develop fragility curves one of the noteworthy components needed are the earthquake ground motions. The selection of suitable ground motions is crucial to obtain reliable fragility curves. The seismic hazard level of the earthquake ground motions can be represented by different ground motion intensity measures. The essential point in choosing the suitable intensity measure could be a certain level of correlation with the seismic damage of bridges. In this study, several ground motions are considered to perform the fragility curves. Existing ground motion intensities can be directly calculated from ground motion records, such as peak ground acceleration (PGA). Priestley et al. (Priestley, 2007), noted that synthetic records typically have a longer duration comparing with real earthquake ground motion data. Bommer and Acevedo (Bommer, 2004) mentioned that the use of real earthquake databases, as an input to dynamic analysis of structures, is more realistic than artificial records. Naeim and Lew (Naeim, 1995) pointed that there are some several problems in terms of uncontrolled use of synthetic records in seismic design which can lead to overestimate displacement demands and energy input and consciously mislead the expected performance of the structure. In this study, in order to avoid such problems synthetic ground motions are not employed. However, real earthquake ground motions are considered which show the seismic potential of the investigated region. In this study PGA is obtained directly from earthquake record databases without any additional information. Baker and Cornell (Baker, 2006) use geometric mean of the intensity measure of the two horizontal components of ground motion for hazard analysis. In this study, two horizontal orthogonal components are employed to perform the analysis. On the other hand, geometric mean of the intensity measures of the two horizontal components of the ground motion is calculated to obtain the intensity measure of each ground motion. Since the recorded ground motion from past earthquakes in Iran is not sufficient to develop fragility curves, recorded ground motions from other regions with similar seismic potential to Iran, and faulting mechanisms, are considered to generate fragility curves.

GEOLOGICAL AND SEISMOLOGICAL FEATURE OF IRAN

The Iranian plateau is located between the Arabian plate in the south which moves at 2.1-2.5 cm/yr and the Eurasian plate in the north. The study of earthquake mechanisms along active fault systems in Iran implies dominance of strike-slip faulting and reverses faulting. Fig. 3
shows the distribution of reverse and strike-slip faults, particularly in south-western, center and northern regions of Iran. Due to the high density of active faults in Iran and the inaccuracy of the macro-seismic data of the area, the source of some of the earthquakes have been related to more than one fault. Therefore, the development of studies on the seismic vulnerability of bridges based on different seismic sources is an important issue (Berberian, 1994).

Nonlinear dynamic analyses consider 20 unscaled reverse fault signals and 20 recorded in strike-slip sources from Iran and other countries, which similar seismic potentiality and faulting mechanisms. Fig. 4 displays response spectra of the selected ground motions (5%damping) reverse and strike-slip faults.

Fig. 3 - (a): Strike-slip, (b): reverse fault in Iran. (http://earthquake.usgs.gov/earthquakes/world/iran/gshap.php)

Fig. 4 - Response spectra of the selected ground motions (5%damping) for (a): reverse and (b): strike-slip faults
ANALYTICAL FRAGILITY CURVES

Four damage states are considered namely: slight, moderate, extensive and damage control as described by Hwang et al. (Hwang, 2001). Displacement ductility is selected as a seismic performance indicator in order to quantify damage states (Hwang, 2001; Mosleh, 2015). Tables 2 and 3 present the different damage states for simply supported and continuous bridges, respectively. Table parameters are as follows:

$L_p$ is the plastic hinge length estimated according to Priestley et al. (Priestley, 1996a).

$\varphi_1$: 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.

$\varphi_2$: is the curvature correspondent to the relative displacement of a column when the vertical reinforcing bars at the bottom of the column reach the yield.

$\varphi_2$ and $\varphi_4$: is the curvature of a column when $\varepsilon_c = 0.002$ or $\varepsilon_c = 0.004$ respectively for the column with or without lap splice. Note that: $\varepsilon_c$ is the compressive strain at the concrete column.

$\mu_1$: is the first limit state corresponding to a first yield displacement ductility ratio equal to 1.

$\mu_y$: is the yield displacement ductility ratio.

$\mu_2$ and $\mu_4$: are the displacement ductility ratio corresponding to $\varepsilon_c = 0.002$ and $\varepsilon_c = 0.004$ respectively for the columns.

$\mu_2\max = \mu_2+3$ and $\mu_4\max = \mu_4+3$.

Table 2 - Limit states for simply support bridges (SB)

<table>
<thead>
<tr>
<th>Column shape</th>
<th>Col. height</th>
<th>Direction</th>
<th>$L_p$ (m)</th>
<th>$\varphi_1$</th>
<th>$\varphi_2$</th>
<th>$\varphi_4$</th>
<th>$\mu_1$</th>
<th>$\mu_y$</th>
<th>$\mu_2$</th>
<th>$\mu_2\max$</th>
<th>$\mu_4$</th>
<th>$\mu_4\max$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6</td>
<td>Longi.</td>
<td>0.70</td>
<td>3.05 E-03</td>
<td>3.91 E-03</td>
<td>5.50 E-03</td>
<td>1.30 E-02</td>
<td>1.29</td>
<td>1.46</td>
<td>4.46</td>
<td>2.26</td>
<td>5.26</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Trans.</td>
<td>0.40</td>
<td>3.05 E-03</td>
<td>3.91 E-03</td>
<td>5.50 E-03</td>
<td>1.30 E-02</td>
<td>1.29</td>
<td>1.50</td>
<td>4.50</td>
<td>2.52</td>
<td>5.52</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>Longi.</td>
<td>1.20</td>
<td>4.09 E-03</td>
<td>5.42 E-03</td>
<td>4.30 E-03</td>
<td>1.30 E-02</td>
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<td>1.23</td>
<td>4.23</td>
<td>1.71</td>
<td>4.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Trans.</td>
<td>0.70</td>
<td>4.09 E-03</td>
<td>5.42 E-03</td>
<td>4.30 E-03</td>
<td>1.30 E-02</td>
<td>1.33</td>
<td>1.20</td>
<td>4.20</td>
<td>1.82</td>
<td>4.82</td>
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<tr>
<td></td>
<td>10.5</td>
<td>Longi.</td>
<td>1.09</td>
<td>2.60 E-03</td>
<td>3.41 E-03</td>
<td>4.80 E-03</td>
<td>1.20 E-02</td>
<td>1.31</td>
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<td>4.47</td>
<td>2.29</td>
<td>5.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Trans.</td>
<td>0.68</td>
<td>2.60 E-03</td>
<td>3.41 E-03</td>
<td>4.80 E-03</td>
<td>1.20 E-02</td>
<td>1.31</td>
<td>1.51</td>
<td>4.51</td>
<td>2.52</td>
<td>5.52</td>
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</tbody>
</table>
### Table 3 - Limit states for continuous bridges (CB)

<table>
<thead>
<tr>
<th>Column shape</th>
<th>Col. height</th>
<th>( L_p ) (m)</th>
<th>( \varphi_1 )</th>
<th>( \varphi_y )</th>
<th>( \varphi_2 )</th>
<th>( \varphi_4 )</th>
<th>( \mu_1 )</th>
<th>( \mu_y )</th>
<th>( \mu_2 )</th>
<th>( \mu_{2max} )</th>
<th>( \mu_4 )</th>
<th>( \mu_{4max} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4</td>
<td>0.43</td>
<td>4.30 E-03</td>
<td>4.60 E-03</td>
<td>4.70 E-03</td>
<td>1.10 E-02</td>
<td>1</td>
<td>1.07</td>
<td>1.08</td>
<td>4.08</td>
<td>1.93</td>
<td>4.93</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>0.45</td>
<td>4.30 E-03</td>
<td>4.60 E-03</td>
<td>4.70 E-03</td>
<td>1.10 E-02</td>
<td>1</td>
<td>1.07</td>
<td>1.08</td>
<td>4.08</td>
<td>1.70</td>
<td>4.70</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.53</td>
<td>4.30 E-03</td>
<td>4.60 E-03</td>
<td>4.70 E-03</td>
<td>1.10 E-02</td>
<td>1</td>
<td>1.07</td>
<td>1.08</td>
<td>4.08</td>
<td>1.63</td>
<td>4.63</td>
</tr>
</tbody>
</table>

### RESULTS AND CONCLUSIONS

Fig. 5 shows results of fragility curves for two bridge classifications subjected to reverse and strike slip faults for different damage states. Both bridge classifications are more vulnerable to reverse fault signals. Since the maximum amplitude of the mean SA response spectrum of strike slip fault accelerograms is smaller than the mean SA value of reverse fault records (Fig. 4), greater demands from the bridges subjected to the reverse fault records are obtained. Fig. 5 shows that simply supported bridges performs consistently better comparing to continuous bridges. Note that in CB bridges the superstructure is monolithic with the substructure and consequently superstructure transfers more demands to substructure elements in seismic loading.

![Fig. 5 - Fragility curves subjected to reverse and strike-slip fault for different damage limit states for (a): simply support and (b): continues bridge classification](image-url)
Fig. 6 shows the fragility curves as function of presence or not of lap splice for different damage states for simply supported and continuous bridges. The graphs present extensive and collapse limit states, because the presence of lap splice has more effect on these limit states comparing to slight and extensive states. The result shows that the column with lap splice are more vulnerable than the one without lap splice, therefore as pointed out in different codes and guidelines using lap splice in the critical location of columns in longitudinal reinforcement is not permitted (FHWA, 2006; AASHTO, 2012; Caltrans, 2013). However bridges designed with old codes frequently have lap splices near the base of column, which make them more seismically vulnerable.

Fig. 6 - Fragility curves in function of the presence or not of lap splice in columns (a): SSB-LS3&LS4-reverse fault, (b): SSB-LS3&LS4-strike slip fault, (c): CB-LS3&LS4-reverse fault, (d): CB-LS3&LS4-strike slip fault

REFERENCES


