Seismic Fragility Analysis of Bridge Pier

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Abstract: Bridges are classified as lifeline structures as they need to be functional in an earthquake event. The performance-based analysis of the existing bridges is important for the stakeholders. Information on seismic performance in terms of fragility of existing bridges in the country can provide valuable information to the decision-makers. This study focuses on the development of seismic fragility curve, which is the probability of exceedance of a defined damage parameter of the bridge pier under a given ground motion intensity and development of damage index function of bridge piers. Existing bridge piers are considered and peak ground acceleration is taken as ground motion intensity measure and drift at the pier top level is considered as the damage parameter.

1. Introduction

Due to an increase in the frequency of seismic events, there is a need for the evaluation of the existing lifeline structures for their seismic response. Bridges are one of the lifeline structures that are most likely to be affected during any seismic event. The bridge consists of two parts i.e. superstructure and substructure. Bridge pier is a part of the substructure of any bridge which is responsible for the transfer of load from the superstructure to the foundation. Past earthquakes (1985 Mexico; 2001 Bhuj; 2005 Islamabad; 2006 Sikkim; 2011 South Island; 2015 Nepal) have revealed that bridge pier is the most susceptible part of the bridge, failure of which can lead to failure of the complete bridge. Thus, the seismic response of bridges greatly depends on the seismic behaviour of its piers.

The seismic fragility curve represents the probability of exceedance of the defined damage state parameter with the variation of intensity of Peak Ground Acceleration (PGA). Such fragility curves can be used to extract information about the seismic performance of a bridge, loss estimation due to expected damage in an earthquake, emergency preparedness, and response after an earthquake, pre and post-earthquake rehabilitation tasks, etc. Number of studies [1-9] have been done for the development of the seismic fragility curve of bridge. Many studies [4][12] are conducted on the definition of damage states required for the development for fragility curves. Damage Index Function (DIF) is a function relating the variation of damage states with PGA for a given probability of exceedance. These curves are also capable of estimating the structural damage state for a particular pier.

In this paper the seismic fragility curves are developed based on an accepted methodology [11] for selected existing bridge piers. This study also focuses on the development of DIF of each of the selected piers.

2. Methodology

A Probabilistic Seismic Demand Model (PSDM), which is a relationship between the drift and PGA, is required to obtain from a Probabilistic Seismic Demand Analysis (PSDA). PSDA involves performing
a series of dynamic time history analysis on the given structural model to estimate the probability of exceedance of structural demand. In the present study, the demand parameter \( D \) is selected as drift percentage of bridge piers subjected to a series of accelerograms. As per [11], PSDM for the selected bridge pier can be represented through a power-law expression given as:

\[
D = a( PGA )^b
\]  

(1)

where ‘\( a \)’, ‘\( b \)’ = constant coefficients, PGA = Peak Ground Acceleration. The drift percentage is known to be log normally distributed about the median with a standard deviation of \( \beta(D/PGA) \) [11]. The values of \( a, b, \beta(D/PGA) \) are calculated after performing a series of nonlinear time-history analyses and then performing the regression analysis on the data obtained.

Seismic fragility curve represents the probability of exceeding the seismic demand parameter \( D \) for the given performance level (\( C \)) for the given PGA. As given by [12] fragility curve can be drawn for each of the predefined damage states in closed form as:

\[
P( C − D ≤ 0/PGA) = \phi\left(\frac{\ln\left(\frac{C}{D}\right)}{\sqrt{\beta^2 (\frac{D}{PGA}) + \beta_c^2 + \beta_m^2}}\right)
\]  

(2)

where \( \beta_c, \beta_m = \) dispersion in capacity and modelling respectively. ‘\( D \)’, ‘\( C \)’ is the median of the drift ratio demand and capacity at chosen performance level respectively, ‘\( \phi \)’ represents the standard normal distribution function. Combining equations (1 and 2), we can rearrange the equation 1 to be:

\[
P( C − D ≤ 0/PGA) = 1 − \phi\left(\frac{\ln(c) − \ln(a( PGA )^b)}{\sqrt{\beta^2 (\frac{D}{PGA}) + \beta_c^2 + \beta_m^2}}\right)
\]  

(3)

The value of \( \beta_c \) is assumed to be 0.25 based on the moderate construction quality. Damage index function is a function capable of predicting the damage index for the pier when subjected to a series of accelerograms. It can be represented by a Weibull distribution function with a cumulative probability distribution. The equation is as follows:

\[
F(s) = 1 − e^{-\left(\frac{s}{\alpha}\right)^\beta}
\]  

(4)

where \( F(s) \) represents damage index, ‘\( s \)’ represents PGA, \( \alpha, \beta \) represents the Weibull distribution factors.

3. Details of the Bridge

Three bridge piers are selected from an existing four-span simply supported slab-girder highway bridge with carriageway width of 10.5 m (overall width 12.9 m). The deck consists of a concrete slab having 230 mm thickness resting on 1.015 m deep steel-I beams. The bridge is located in seismic zone V (seismic zone factor i.e. \( Z = 0.36 \)). The selected piers are circular in shape with a diameter of 1.8 m. The total height of the piers (pier cap to the top of foundation) is 11.977 m and the piers rests on an isolated sloped foundation of size 10.20 m x 10.20 m. The characteristic strength of concrete and steel used for the pier is M35 and Fe500 respectively.

3.1. Seismic weight

The seismic weight at the top of piers are about 437t (for pier 1), 425t (for pier 2), 413t (for pier 3) and they are applied as lumped mass at top of pier. This mass is taken as combination of self-weight plus 20% of live-load of superstructure on either side and the self-weight (50%) of pier.

3.2. Structural Modelling

The bridge pier selected is modelled and analysed in FEM software [15]. The pier is modelled as an equivalent single degree of freedom system (SDOF). The element type used to model pier is inelastic force-based frame element and the section is divided into 400 fibre sections i.e. (fibre approach). The point of maximum curvature is found to be at a depth of 3.6 m below the ground level. Hence the piers are fixed at the depth of 3.6m below ground level. The base is assumed to be fixed at an unsupported length of 7.8m from top for pier 1, 6.0 m for pier 2 and 4.0m for pier 3.
The random parameters considered for all three bridge piers are characteristic strength of concrete, yield strength of rebars, global damping ratio (Table 1). Latin hypercube sampling (LHS) approach which is a technique to generate samples of random variables from a multi-dimensional distribution. This method is utilised to generate the 44 sets of samples of random parameters. Using the generated sets of random parameters 44 models are developed. The accelerograms used in this paper are collected from [14], each having unit PGA.

**Table 1. Random variable Details**

| Random parameters             | Mean     | COV (%) | Source                  |
|-------------------------------|----------|---------|-------------------------|
| Concrete compressive Strength | 45.0 MPa | 21      | Ranganathan (1999)      |
| Yield strength                | 581.5 MPa | 10      | Ranganathan (1999)      |
| Global damping ratio          | 5%       | 40      | Davenport and Carroll (1986) |
4. **Probabilistic Seismic Demand Model**

Non-linear dynamic analysis is performed on the developed models in which each model is subjected to a PGA ranging from 0.1g to 1.0g. Maximum drift percentage for each model is recorded and a graph is plotted between drift percentage and the applied ground motions for the selected model of bridge piers (Figure 2a, 2b and 2c). Then after performing the regression analysis on the obtained data, power-law expression (i.e. Eq. 1) is developed for the selected piers (Table 2). The PSDM model obtained will give the most probable maximum drift ratio for a given PGA.

![Figure 2. PSDM model for Pier 1, 2 & 3.](image)

**Table 2. PSDM models**

| PIER      | PSDM                        | $R^2$ | $\beta$ | $D_{PGA}$ |
|-----------|-----------------------------|-------|---------|-----------|
| Pier 1 (7.825 m) | $3.5937(PGA)^{1.4533}$     | 0.8041| 0.4713  |           |
| Pier 2 (6.0 m)  | $1.0863(PGA)^{1.2433}$     | 0.9169| 0.2442  |           |
| Pier 3 (4.0 m)  | $1.1563(PGA)^{1.5954}$     | 0.6595| 0.3669  |           |

5. **Seismic Fragility Curve**

The obtained PSDMs are utilised to develop seismic fragility curve as per Eq. 3. Drift ratios at five damage states, namely almost no, slight, moderate, extreme and complete, are taken from Dutta and Mander (1999) as shown in Table 3. Figures 3-7 represents the fragility curves obtained for the five different damage states. Pier 1 is found to more vulnerable for all damage states compared to that of other piers. This can be attributed to the high slenderness ratio of the pier 1 compared to that of other piers. Damage index values for the piers are computed based on drift (%) limits and shown in Table 3. Complete collapse of the structure is considered to be at a damage index of 0.9, and based on this, damage index for other damage states are computed.
### Table 3. Damage state parameter limits

| Damage State | Description                       | Drift (%) limits (S_c) | Damage Index |
|--------------|-----------------------------------|-------------------------|--------------|
| Almost no    | First yield                       | 0.5                     | 0.09         |
| Slight       | Spalling, cracking of cover       | 0.7                     | 0.13         |
| Moderate     | Anchorage loss                    | 1.5                     | 0.27         |
| Extreme      | Beginning of pier collapse        | 2.5                     | 0.45         |
| Complete     | Collapse of pier                  | 5.0                     | 0.90         |

#### Figure 3. Fragility curves at first yield ($S_c = 0.5\%$)

#### Figure 4. Fragility curves at spalling of cover ($S_c = 0.7\%$)

#### Figure 5. Fragility curves at anchorage loss ($S_c = 1.5\%$)
Figure 6. Fragility curves at Incipient collapse ($S_c = 2.5\%$)

Figure 7. Fragility curves at complete collapse ($S_c = 5.0\%$)

6. **Damage Index Function**

The fragility curves shown in figures 3-7 are transformed to damage index function using Weibull distribution. The damage index function is a function of damage index versus PGA which is contour corresponds to a probability of exceedance of 50% for every damage state. Pier 1 is found to have more damage index compared to other piers for a given PGA value. The procedure demonstrated in this study can be utilised to develop the fragility curves and damage functions of any type of existing bridges.

Figure 8. Damage index vs. PGA (for probability of exceedance of 50%)
7. Conclusion
In this present study, existing bridge piers are considered and their seismic fragility curves have been developed for five performance levels. It is concluded that as the height of the pier increases the probability of failure of pier also increases. The same trend can be observed from the results of damage index function. It also indicates that as the height of the pier increases it is more likely to be damaged in an earthquake event.

The information obtained from the damage index function can be used for damage cost evaluation, disaster preparedness etc. Though the present study considered only the damage of the most important component of the bridge, pier, it can be extended by considering failure of other components such as bearing, connection between superstructure and substructure, diaphragm abutment, expansion joints etc.

8. References
[1] Mackie K and Stojadinovic B 2001 Probabilistic seismic demand model for the California highway bridges J. Bridge Eng. 6(6) 468–81
[2] Neilson B G and DesRoches R 2007 Seismic fragility methodology for highway bridges using a component level approach Earthq. Eng. Struct. Dyn. 36(6) 823–39
[3] Pan Y, Agarwal A K and Ghosn M 2007 Seismic Fragility of continuous steel highway bridges in New York State J. Bridge Eng. 12(6) 689–99
[4] Banerjee S and Shinozuka M 2008 Mechanic quantification of RC bridge damage states under earthquake through Fragility analysis Probab. Eng. Mech. 23(1)12–22
[5] Seo J and Linzell D G 2012 Horizontally curved steel bridge seismic vulnerability assessment Eng. Struct. 34 21–32
[6] Ranganathan R 1999 Structural reliability analysis and design (Mumbai, India: Jaico)
[7] Dimitrakopoulous E G and Paraskeva T S 2015 Dimensionless fragility curves for rocking response to near-fault excitation Earthq. Eng. Struct. Dyn. 44(12) 2015–33
[8] Monteiro R, Delgado R and Pinho R 2016 Probabilistic seismic assessment of RC bridges: part 1-uncertainty model Structures 5 258–73
[9] Rogers LP and Seo J 2017 Vulnerability sensitivity of curved precast-concrete-I girder bridges with various configurations subjected to multiple ground motions J. Bridge Eng. 22(2) 04016118
[10] C A Cornell, F Jalayer, R O Hamburger and D A Foutch 2002 The Probabilistic basis for the 2000 SAC/FEMA steel moment frame guidelines J. Struct. Eng. 128(4) 526–33
[11] Shome N and C A Cornell 1999 Probabilistic Seismic Demand Analysis of Nonlinear Structures (Stafford University)
[12] Celik O, and B R Ellingwood 2010 Seismic fragilities for the non-ductile reinforced concrete frames – Role of aleatoric and epistemic uncertainties Struct. Saf. 32(1) 1–12
[13] A Dutta and J B Mander 1999 Seismic fragility analysis of highway bridges Proceeding of the Center-to-Center project workshop on Earthquake Engineering in Transportation Systems (Tokyo)
[14] Haselton C B, A S Whittaker, A Hortacsu, J W Baker, J Bray and D N Grant 2012 Selecting and scaling earthquake ground motions for performing response-history analyses Proc. 15th World conf. on Earthquake Engineering
[15] Seismosoft 2018 SEISMOSTRUCT – A computer programme for static and dynamic nonlinear analysis of framed structure
[16] IRC 6:2015 Standard specifications and code of practice for road bridges-section II- Loads and Stresses (Fourth Revision)