COMPARISON OF EMPIRICAL AND ANALYTICAL FRAGILITY CURVES FOR RC BRIDGE PIERS IN JAPAN

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Abstract

This paper describes an analytical method to construct fragility curves for highway bridge piers considering both structural parameters and variation of input ground motion. A typical bridge structure was considered and its piers were designed using the seismic design codes in Japan. Based on PGA and PGV, earthquake records were selected from the 1995 Hyogoken-Nanbu earthquake. Using the records as input ground motion, nonlinear dynamic response analyses were performed and the damage indices for the RC bridge piers were obtained. Using the damage indices and ground motion indices, fragility curves for the bridge piers were constructed. The fragility curves constructed following this approach were then compared with the empirical fragility curves.

Introduction

The 1995 Hyogoken-Nanbu (Kobe) earthquake, which is considered as one of the most damaging earthquakes in Japan, caused severe damage to expressway structures in Kobe area. Based on the actual damage data from the earthquake, a set of empirical fragility curves (Yamazaki *et al*., 1999) were constructed. The empirical fragility curves give a general idea about the relationship between the damage levels of the highway structures and the ground motion indices. These fragility curves may be used for damage estimation of highway bridge structures in Japan. However, the empirical fragility curves do not specify the type of structure, structural performance (static and dynamic) and variation of input ground motion and may not be applicable for estimating the level of damage probability for a specific bridge structure.

The objective of this study is to develop analytical fragility curves considering structural parameters and variation of input ground motion. In this study, we consider a RC bridge structure. The piers of the bridge structure are designed using the seismic design codes for highway bridges in Japan. Using the strong motion records from the Hyogoken-Nanbu earthquake, the damage indices of the bridge piers are obtained from a nonlinear dynamic response analysis. Then, using the obtained damage indices and the ground motion indices, the fragility curves for the bridge piers are constructed. The fragility curves developed following this approach are compared with the empirical fragility curves. The method may be used in constructing fragility curves for a class of bridge piers, which do not have enough earthquake experience.

Development of Fragility Curves

Yamazaki *et al*. (1999) developed a set of empirical fragility curves based on the actual damage data from the Hyogoken-Nanbu earthquake. In this paper, we consider an analytical approach to construct the fragility curves for bridge piers of specific bridges. A nonlinear dynamic response analysis of the piers is performed and the piers are modeled as a single-degree-of-freedom (SDOF) system.

For a nonlinear dynamic response analysis, strong motion records were selected from the 1995 Hyogoken-Nanbu earthquake. A total of fifty (50) acceleration time histories were taken on the basis of Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV). PGA for the selected records ranges from 65.96 to 818.01 cm/s^2 and PGV ranges from 11.10 to 127.00 cm/s. Using these acceleration time histories as input ground motion, the damage indices (Park and Ang, 1985) of the bridge piers are obtained from the nonlinear analysis. Finally, using the obtained damage indices and the ground motion indices, the analytical fragility curves for RC bridge piers are constructed. The fragility curves obtained by following this approach consider both structural parameters and variation of input ground motion. The steps for constructing the analytical fragility curves are as follows:

1. Select the earthquake ground motion records.
2. Normalize the PGA and PGV of the selected records to different excitation levels.
3. Make an analytical model of the structure.
4. Obtain the stiffness of the structure.
5. Select a hysteretic model for the nonlinear dynamic response analysis.
6. Perform the nonlinear dynamic response analysis using the selected records.
7. Obtain the ductility factors of the structure.
8. Obtain the damage indices of the structure for each excitation level.
9. Calibrate the damage indices for each damage rank.
10. Obtain the number of occurrence of each damage rank for each excitation level and get the damage ratio.
11. Construct the fragility curves using the obtained damage ratio and the ground motion indices for each damage rank.

Static Analysis

To obtain the analytical fragility curves for RC bridge piers, a typical bridge structure is considered. The bridge model taken in this study is rather simple. The length of each span of the bridge is 40m and the width is 10m. The height of each pier is 8.5m. The cross-section of each pier is 4m by 1.5m. The total weight is 5163 kN, calculated as the weight of the superstructure (deck and girder) and weight of the substructure (pier). The weight of the superstructure is 4416 kN and self-weight of the pier is 1494 kN. The piers are designed by using the 1964 seismic design code in Japan and are named as 1964 pier. For sectional analysis, yield strength of steel (σ_{sy}) and compressive strength of concrete (σ'_c) are taken as 332 and 27 MPa, respectively. The longitudinal and tie reinforcement is taken as 0.09 and 1.03 percent, respectively. Both sectional and static pushover analyses are performed using the program Response-2000 (Evan and Michael, 1998). From the sectional analysis, it is found that shear failure governs the failure mode. The yield force and yield displacement for a typical pier calculated as 3920.53 kN and 1.68 cm, respectively. The ductility capacity is obtained as 4.94.

Dynamic Analysis

To perform dynamic response analysis, the piers are modeled as a single-degree-of-freedom (SDOF) system. A bilinear hysteretic model was considered and the post yield stiffness (Kawashima and Macrae, 1993) was taken as 10% of the yield stiffness of the pier with 5% damping ratio. The yield stiffness of the piers is obtained using the yield force and yield displacement. The ductility demand at the top of the bridge pier is obtained. The ductility is defined as the ratio of the maximum displacement (obtained from the nonlinear dynamic response analysis) to the yield displacement (obtained from the static analysis). The ductility factors thus obtained are used to evaluate the damage of the bridge piers.

For the damage assessment of the bridge piers, Park-Ang (1985) damage index was used in this study. The damage index *DI* is expressed as

$$DI = \frac{\mu_d + \beta \cdot \mu_h}{\mu_u} \tag{1}$$

where μ_d is the displacement ductility, μ_u is the ultimate ductility of the bridge piers, β is the cyclic loading factor taken as 0.15 and μ_h is the cumulative energy ductility defined as

$$\mu_h = E_h / E_e \tag{2}$$

where E_h and E_e are the cumulative hysteretic and elastic energy of the bridge piers. The damage indices of the bridge piers are obtained using equation (1). The obtained damage indices for the given input ground motion are then calibrated to get the relationship between the damage index (DI) and damage rank (DR). This calibration is conducted using the method that was proposed by Ghobarah *et al.* (1997). Table 1 shows the relationship between the damage index and damage rank. It can be seen that each damage rank has a certain range of damage indices. The damage rank ranges from slight to complete. Using the relationship between DI and DR, the number of occurrence of each damage rank is obtained. These numbers are then used to obtain the damage ratio for each damage rank.

To count the number of occurrence of each damage rank, PGA and PGV for the selected records were normalized to different excitation levels. For instance, PGA were normalized from 100 to 1500 cm/s^2 having fifteen (15) excitation levels with equal intervals. Then the ground motion records are applied to the structure to obtain the damage indices. Using these damage indices, the number of occurrence of each damage rank is obtained for each excitation level. Finally, using the numbers, the damage ratio is obtained for each damage rank. Same procedure is also applied for PGV. In case of PGV, it was normalized from 20 to 200 cm/s having ten (10) excitation levels with equal intervals. It should be noted that the ranges for PGV were obtained from the relationship of PGA and PGV of the selected records in order to maintain the compatibility. Figure 1 shows the number of occurrence of each damage rank for each excitation level with respect to both PGA and PGV. It can be seen that as the excitation level increases the number of occurrence of slight damage decreases, whereas the number of occurrence of complete damage increases.

Damage Index (DI)	Damage Rank (DR)	Definition
0.00<DI≤0.14	D	No Damage
0.14<DI≤0.40	C	Slight Damage
0.40<DI≤0.60	B	Moderate Damage
0.60<DI<1.00	A	Extensive Damage
1.00≤DI	As	Complete Damage

Table 1. Relationship between the damage index and damage rank (Ghobarah *et al.*, 1997).

Event	Damage Rank							
	DR≥C		DR≥B		DR≥A		DR=As	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
Empirical	6.36	0.29	6.46	0.30	6.67	0.29	6.87	0.32
Analytical	5.62	0.28	6.48	0.24	6.68	0.23	6.88	0.23

Table 2. Parameters of fragility curves for RC bridge piers for PGA.

Event	Damage Rank							
	DR≥C		DR≥B		DR≥A		DR=As	
	λ	ζ	λ	ζ	λ	ζ	λ	ζ
Empirical	4.10	0.35	4.27	0.36	4.55	0.40	4.82	0.40
Analytical	3.65	0.58	4.54	0.47	4.72	0.44	4.93	0.40

Table 3. Parameters of fragility curves for RC bridge piers for PGV.

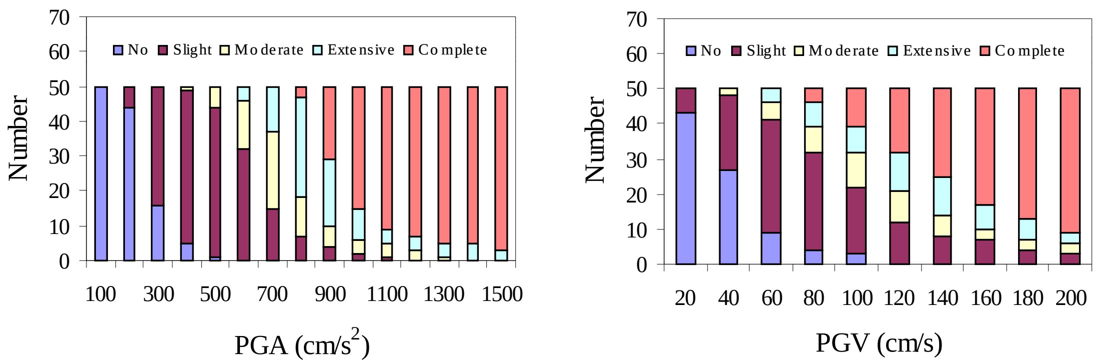

Figure 1. Number of occurrence of each damage rank in different excitation levels.

Fragility Curves

For each damage rank we have one data set, i.e., PGA and damage ratio and similarly PGV and damage ratio. Based on these data, fragility curves for the bridge piers are constructed assuming a lognormal distribution. The fragility curves are constructed using both PGA and PGV values. For the cumulative probability P_R of occurrence of the damage equal or higher than rank R is given as

$$P_R = \Phi\left[\frac{\ln X - \lambda}{\zeta}\right] \tag{3}$$

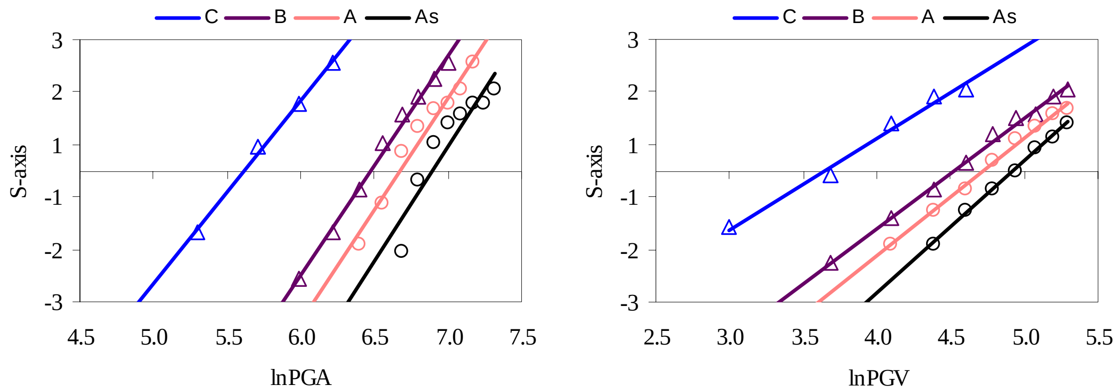

Figure 2. Lognormal probability papers with respect to both PGA and PGV.

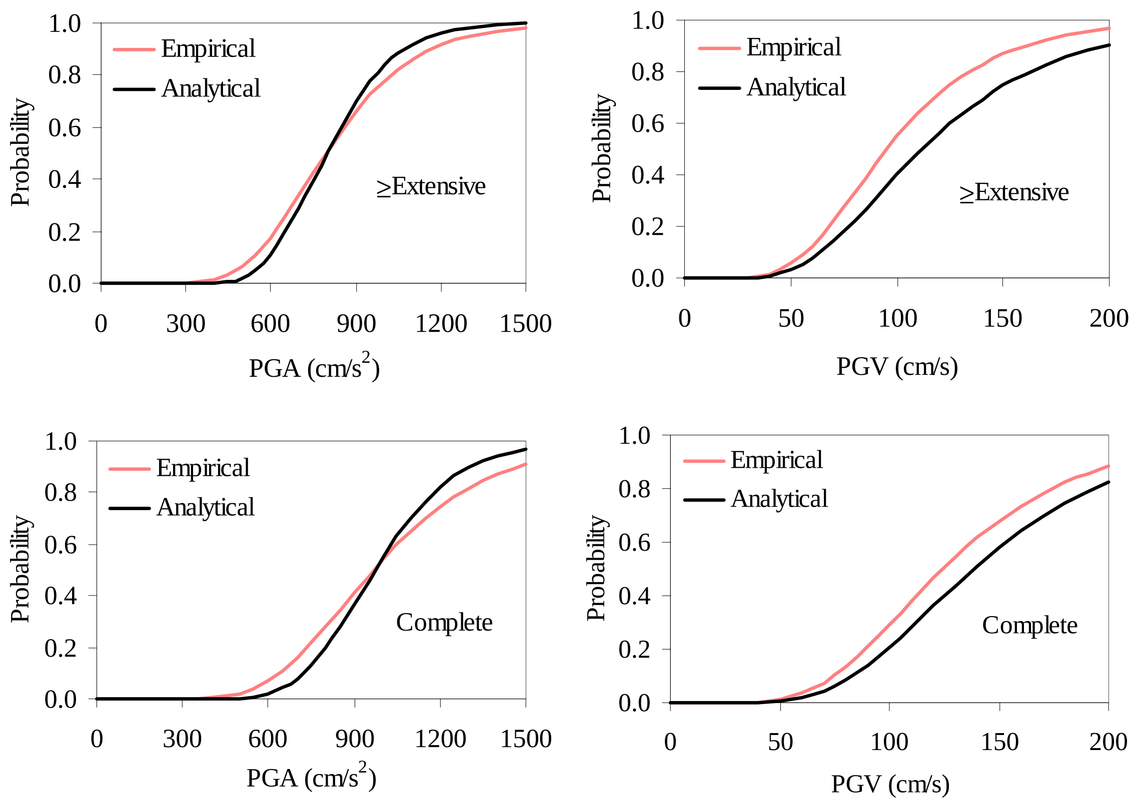

Figure 3. Fragility curves for RC bridge piers with respect to both PGA and PGV from the records of the Kobe earthquake.

where Φ is the standard normal distribution, X is the ground motion indices (PGA and PGV), λ and ζ are the mean and standard deviation of $\ln X$. Two parameters of the distribution (i.e., λ and ζ) are obtained by the least square method on a lognormal probability paper. The lognormal probability papers for the bridge piers with respect both PGA and PGV are shown in Figure 2. The parameters λ and ζ for $\ln X$ that are obtained using the lognormal probability papers are given in Tables 2 and 3, respectively. In the tables, the parameters λ and ζ for the empirical fragility curves (Yamazaki *et al.*, 1999) were also shown for a comparison.

Figure 3 shows the plots of the empirical and analytical fragility curves for the 1964 Japanese bridge piers. Note that there are five damage ranks that are shown in Table 1. For simplicity, the fragility curves only for extensive and complete damage cases are shown in the plots. One can see that the empirical and analytical fragility curves (Figure 3) show a very similar level of damage probability with respect to PGA. However, with respect to PGV some difference is observed between the two. Although only one pier model and one set of earthquake records are used in this study, the method presented herein is useful to demonstrate the effects of structural parameters and input motion characteristics on fragility curves.

Conclusions

An analytical method to construct the fragility curves for the piers of a specific bridge was presented. The analytical fragility curves for a pier designed by the 1964 Japanese highway bridge code were constructed with respect to both PGA and PGV using the records from the 1995 Kobe earthquake. The obtained analytical fragility curves were compared with the empirical ones from the Kobe earthquake and good agreement was observed. However, empirical fragility curves can not introduce various structural parameters and characteristics of input motion, and they require a large amount of actual damage data for a certain class of structures. Hence, the analytical method employed in this study may be used in constructing the fragility curves for a class of bridge structures, which do not have enough earthquake experience.

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