Comparison of Empirical and Analytical Fragility Curves for Highway Bridge Piers

by

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ABSTRACT

Fragility curves correlate the probability of structural damage due to earthquakes as a function of ground motion indices (e.g., PGA, PGV). Based on the actual damage data of highway bridges from the 1995 Hyogoken-Nanbu (Kobe) earthquake, a set of empirical fragility curves was constructed. However, the empirical fragility curves do not consider the structural parameters and variation of input ground motion. In this study, an analytical approach was adopted to construct fragility curves for highway bridge piers. A typical bridge structure was considered and its piers were designed using the seismic design codes in Japan. Based on PGA and PGV, earthquake ground motion 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 assuming a lognormal distribution. The fragility curves constructed following this approach were then compared with the empirical fragility curves.

INTRODUCTION

The actual damages to highway systems from recent earthquakes have emphasized the need for risk assessment of the existing highway transportation systems. The vulnerability assessment of bridges is useful for seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of highway systems. During a post-earthquake recovery period, the structural vulnerability assessment can assist in decision making, such as whether a bridge should be open to traffic immediately after an earthquake, it should be demolished or repaired, and if there are several bridges to be repaired in what order should they be repaired or replaced. Hence, it is important to know the degree of damages of the highway bridge structures due to earthquakes. To estimate a damage level (slight, moderate, extensive, and complete) of highway bridge structures, fragility curves are found to be useful tool. Fragility curves show the relationship between the probability of highway structure damages and the ground motion indices. They allow estimating a damage level for a known ground motion index.

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[1)] was 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 specific bridge structures.

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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 were constructed. The fragility curves developed following this approach were 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.*[1)] developed a set of empirical fragility curves based on the actual damage data from the Hyogoken-Nanbu earthquake and showed the relationship between the damages occurred to the expressway bridge structures and the ground motion indices. In the empirical approach, a total of two hundred and sixteen (216) bridge structures of the Japan Highway Public Corporation (JH) were taken into account among which around fifty percent of the bridges were constructed during the period 1964. The damage data of the JH expressway structures due to the Hyogoken-Nanbu earthquake were collected and the ground motion indices along the expressways were estimated based on the estimated strong motion distribution using Kriging technique. The damage data and ground motion indices were related to each damage rank. Using a certain range of ground motion indices, the damage ratio was obtained. Finally, using the damage ratio for each damage rank, the empirical fragility curves for the expressway bridge structures were constructed assuming a lognormal distribution. The empirical fragility curves obtained following this approach do not consider structural parameters and variation of input ground motion and may not be applicable for specific bridge structures.

In this study, we consider an analytical approach to construct the fragility curves for bridge piers of specific bridges. The piers are designed using the 1964 and 1998 seismic design codes[6)] for highway bridges in Japan. The yield stiffness of the piers is obtained performing the sectional and static analysis.[7)] A nonlinear dynamic response analysis of the piers is performed using this yield stiffness 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 value for the selected records ranges from 65.96 to 818.01 cm/s^2 and PGV value ranges from 11.10 to 127.00 cm/s. **Figure 1** shows the distribution of PGA and PGV for the selected 50 acceleration time histories. Using these acceleration time histories as input ground motion, the damage indices[9)] 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 schematic diagram for constructing the analytical fragility curves is shown in **Fig. 2**. 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.

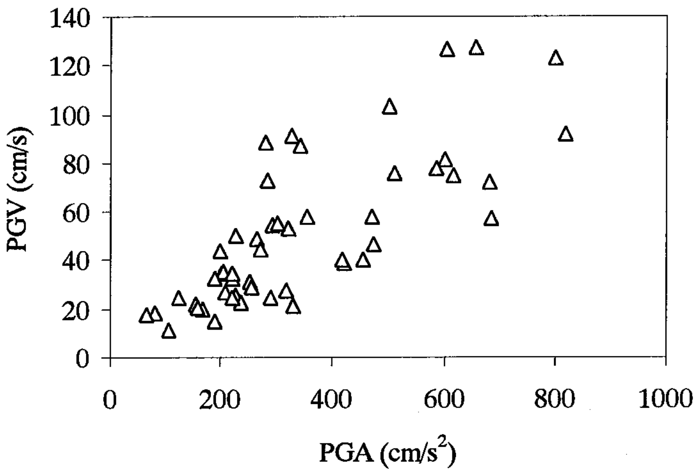


Fig. 1 Distribution of PGA and PGV of the 1995 Hyogoken-Nanbu earthquake records

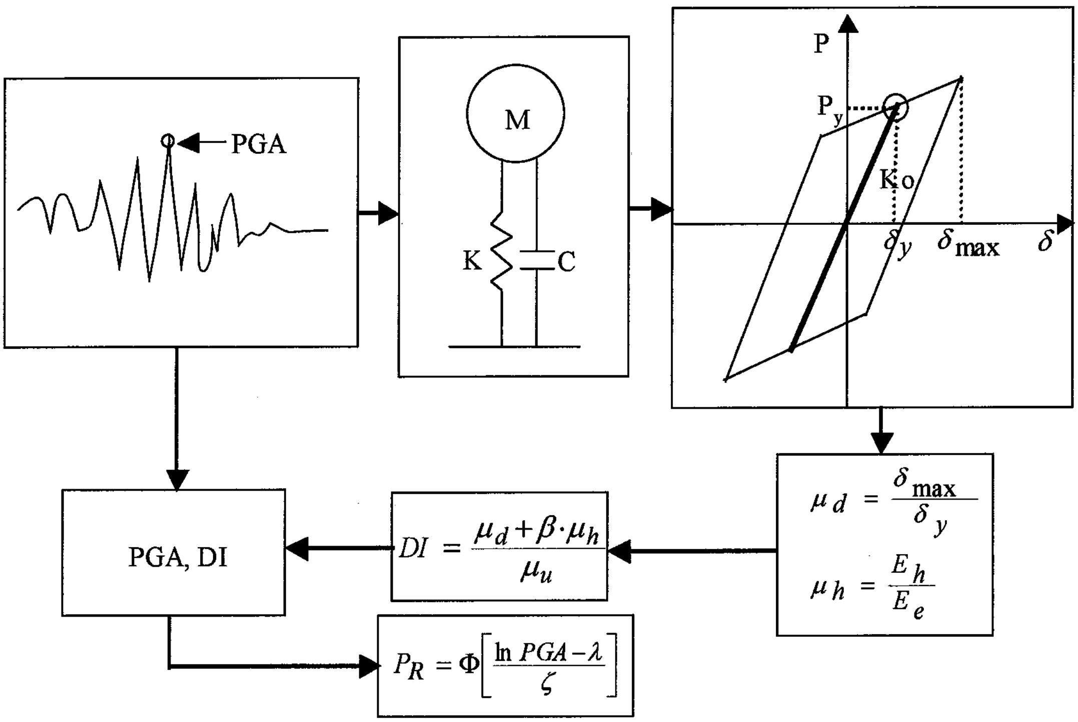


Fig. 2 Schematic diagram for constructing the fragility curves for RC bridge piers

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 and 1998 seismic design codes[6)] in Japan and are named as 1964 and 1998 piers. A Static analysis of the bridge piers is performed to obtain the yield stiffness and ultimate ductility of the piers.

The moment-curvature analysis is carried out to obtain the moment-curvature diagram for each cross-section of the bridge piers. The moment-curvature diagrams of the cross-sections are used to obtain the force-displacement relationship at the top of the bridge piers. The moment-curvature diagram can be obtained using the same procedure given in the "Seismic Design Codes for Highway Bridges in Japan". However, for simplicity, the moment-curvature diagram for each cross-section was obtained using the program Response-2000.[7)]

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. The cross-sectional properties for a 1964 pier used for the Response-2000 program is given in **Fig. 3**. The moment-curvature diagrams obtained for the cross-sections at the base level of the piers are shown in **Fig. 5(a)**.

To obtain the force-displacement relationship, the pier is divided into N slices (50 slices are recommended in the design codes) along its height. The moment-curvature diagram for each cross-section is obtained using the procedure that is mentioned above. For moment-curvature diagrams, it is mainly focused on three sections, namely section at the top level, section at one-third level from the bottom of the pier and section at the bottom level. This is because the configuration of the reinforcement at these levels is different. Using the moment-curvature diagrams for the all sections, the force-displacement relationship at the top of the bridge pier is obtained. **Figure 4** shows the numerical evaluation of the flexural and shear components of the displacement. The steps for obtaining the force-displacement curve are as follows:

1. Divide the pier into N slices along its height.
2. Obtain the moment-curvature diagram for each cross-section.
3. Apply the horizontal force P at the top of the bridge pier.
4. Obtain the bending-moment diagram of the pier for the applied force P.
5. Get the curvature and shear strain by interpolation for each cross-section using the bending moment and moment-curvature diagrams.
6. Calculate the displacement δ using the following equation:

$$\delta = \sum_{i=1}^{N} \left(\phi_i \times dy \times d_i + \gamma_i \times dy\right) \tag{1}$$

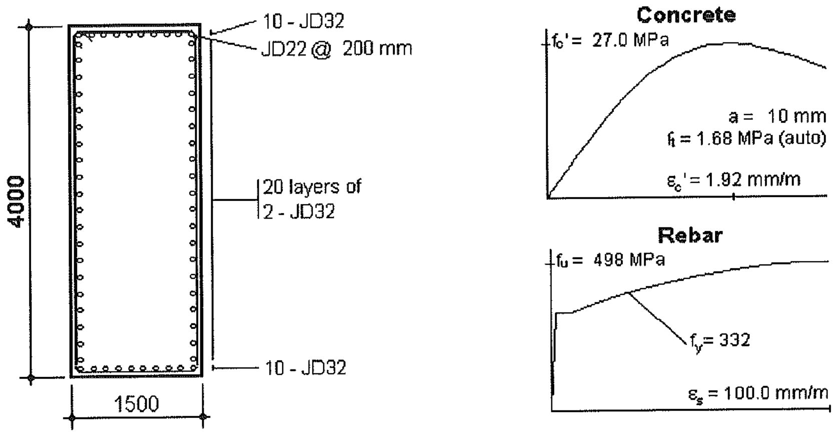


Fig. 3 Cross-sectional properties for the 1964 pier used for sectional analysis

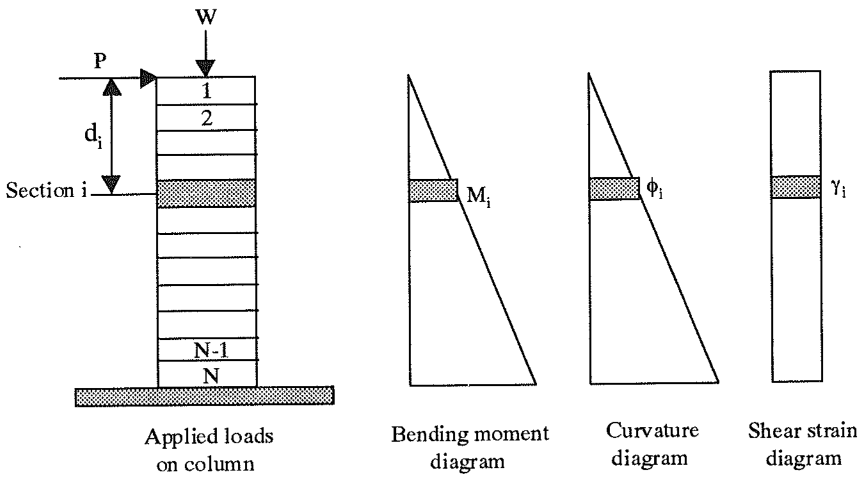
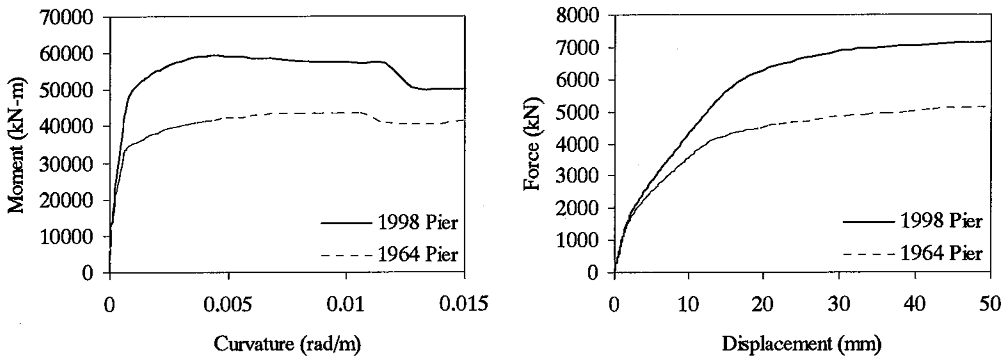


Fig. 4 Numerical evaluation of flexural and shear components of displacement

where δ is the displacement, N is the number of cross-sections, ϕ_i is the curvature, dy is the width of each cross-section, d_i is the distance from the top of the pier to the c.g. of each cross-section, and γ_i is the shear strain.

7. In a similar way, apply several forces P and obtain the corresponding displacements δ. Finally, using these two values, obtain the force-displacement curve at the top of the bridge pier.

Figure 5(b) is the plot of the force-displacement relationship at the top of the bridge piers. The yield force and yield displacement for the 1964 pier are obtained as 3920.53 kN and 1.68 cm, respectively.



(a) Monument curvature diagrams

(b) Force-displacement curves

Fig. 5 Moment-curvature diagrams and force-displacement relationship of the bridge piers

For the 1998 pier, these values are 5381.48 kN and 1.91 cm, respectively. The ductility capacities for the two piers are obtained as 4.94 and 6.01 respectively.

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[8] was taken as 10% of the secant 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 from the nonlinear dynamic response analyses. 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 piers.

For the damage assessment of the bridge piers, Park-Ang's[9] damage index was used in this study. The damage index *DI* is expressed as

$$DI = \frac{\mu_d + \beta \cdot \mu_h}{\mu_u} \tag{2}$$

where μ_d and μ_u are the displacement and 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

$$\mu_h = E_h / E_e \tag{3}$$

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 done using the method that was proposed by Ghobarah *et al.*[10] **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

Table 1 Relationship between the damage index and damage rank[10]

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 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		6.36	0.29	6.46	0.30	6.67	0.29	6.87	0.32
Analytical	1964 Pier	5.62	0.28	6.48	0.24	6.68	0.23	6.88	0.23
	1998 Pier	6.14	0.28	6.87	0.20	7.05	0.17	7.23	0.16

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

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	1964 Pier	3.65	0.58	4.54	0.47	4.72	0.44	4.93	0.40
	1998 Pier	4.21	0.54	4.92	0.44	5.12	0.41	5.31	0.36

Note: C = Slight damage, B = Moderate damage, A = Extensive damage, As = Complete damage.

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

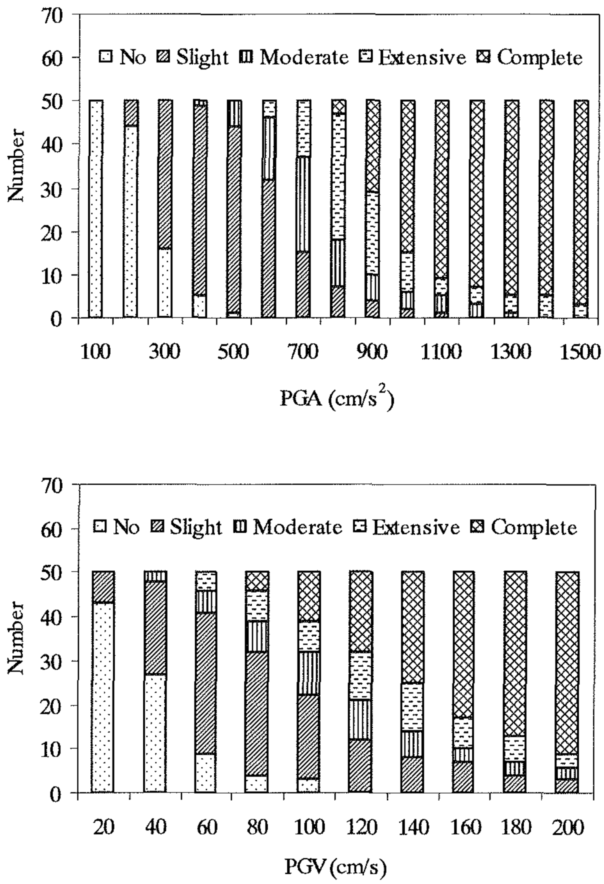


Fig. 6 Number of occurrence of each damage rank for the 1964 pier in different excitation levels

PGV were obtained from the relationship of PGA and PGV of the selected records in order to maintain the compatibility. **Figure 6** shows the number of occurrence of each damage rank for each excitation level for the 1964 pier. The numbers are shown 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.

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

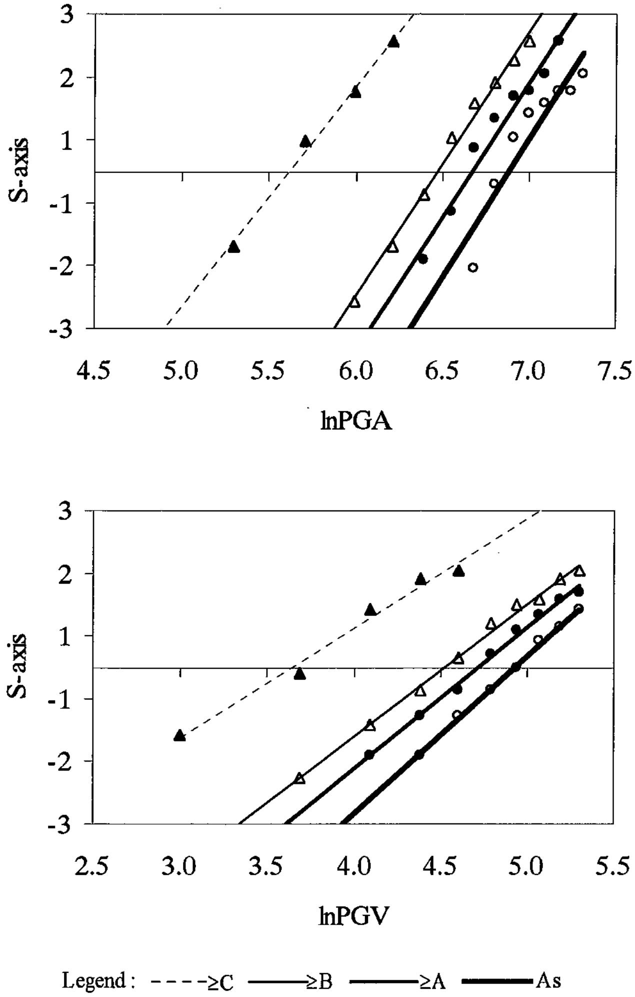


Fig. 7 Lognormal probability papers for PGA and PGV of the Kobe earthquake for the 1964 pier

$$P_R = \Phi\left[\frac{\ln X - \lambda}{\zeta}\right] \tag{4}$$

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 1964 bridge pier with respect both PGA and PGV are shown in **Fig. 7**. 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[1)] were also shown for a comparison.

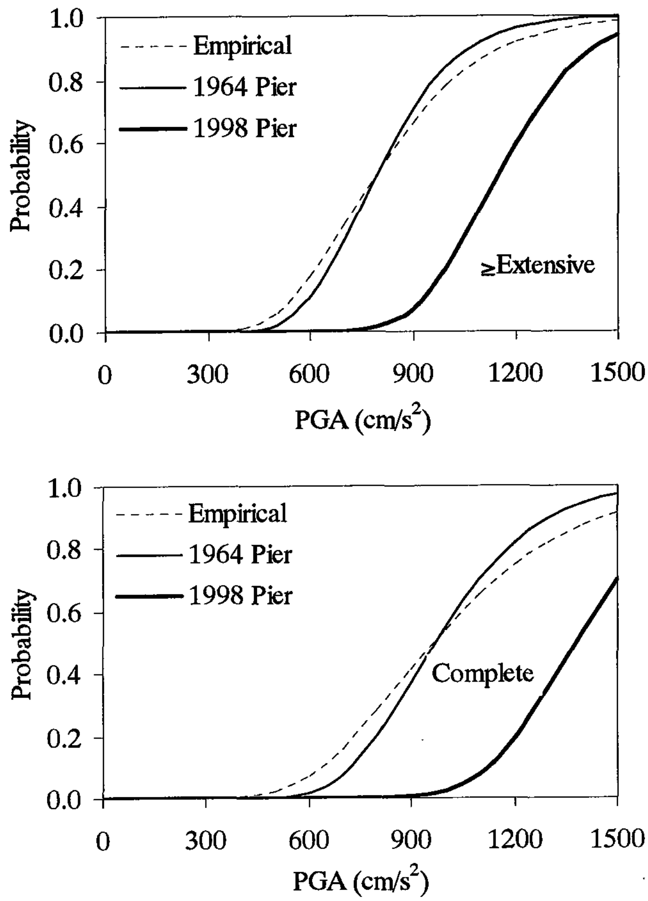


Fig. 8 Fragility curves for RC bridge piers with respect to PGA from the records of the Kobe earthquake

Figures 8 and **9** show the plots of the empirical and analytical fragility curves for the 1964 and 1998 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 (**Fig. 8**) of the 1964 bridge pier shows a very similar level of damage probability with respect to PGA. However, with respect to PGV (**Fig. 9**) some difference is observed between the two. It can also be seen that the empirical and analytical fragility curves (**Fig. 8** and **9**) of the 1998 bridge pier show a very different level of damage probability with respect to both PGA and PGV. As the level of PGA and PGV increases, the analytical fragility curves of the 1998 bridge pier show a lower level of damage probability comparing to the empirical fragility curves. Although, only two pier models 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.

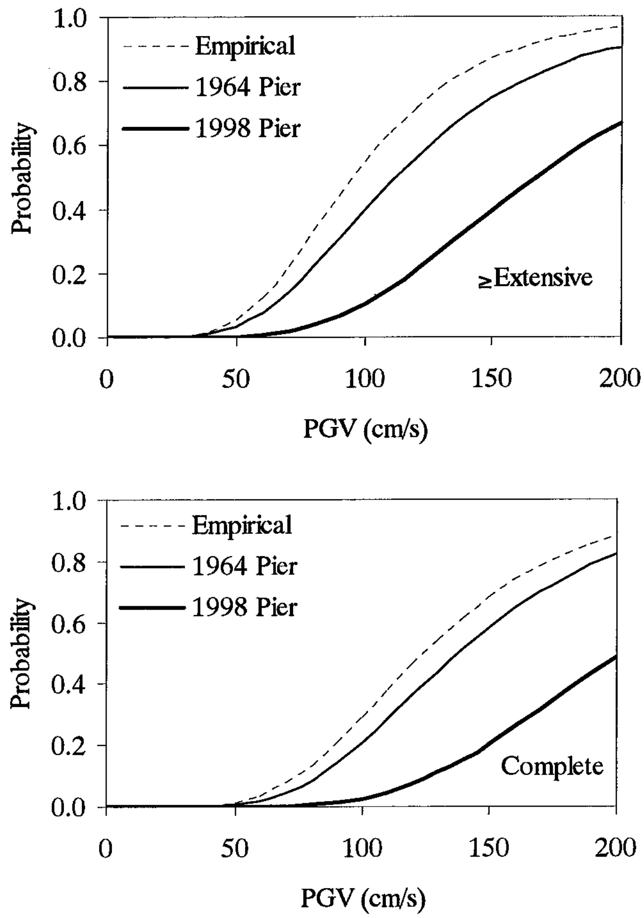


Fig. 9 Fragility curves for RC bridge piers with respect to PGV from the records of the Kobe earthquake

CONCLUSIONS

An analytical method to construct the fragility curves for the piers of a specific bridge was presented. The analytical fragility curves for the piers designed by the 1964 and 1998 Japanese highway bridge codes 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. Good agreement was observed between the empirical and analytical fragility curves of the 1964 bridge pier. However, the empirical fragility curves showed a very different level of damage probability comparing to the analytical fragility curves of the 1998 bridge pier. 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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