研 究 速 報

Comparison of Analytical Fragility Curves for RC Bridge Piers Designed by Using Japanese Old and Recent Seismic Design Codes

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1. INTRODUCTION

To estimate damage levels (slight, moderate, extensive and complete) of highway bridge piers due to earthquakes, we need a set of fragility curves, which predict the damage level for a given level of ground motion indices (e.g., PGA, PGV and SI). Based on the actual damage data of highway bridges from the 1995 Hyogoken-Nanbu (Kobe) earthquake, a set of empirical fragility curves[1] were developed. However, these fragility curves do not consider structural parameters and structural responses. Considering structural parameters and variation of input ground motion, an analytical approach[2] was adopted to construct such kind of fragility curves. In this study, a hypothetical bridge is considered. Then its pier is modeled using the old (1964) and recent (1998) seismic design codes[3] for highway bridges in Japan. From a static analysis the yield force and yield displacement of the pier is obtained for the both cases. Then the damage analysis of the RC bridge pier is performed using the strong ground motion records that were chosen from the Hyogoken-Nanbu earthquake. The damage indices are obtained from a nonlinear dynamic response analysis and finally using these damage indices and ground motion indices, the analytical fragility curves are constructed for the both cases. Then the fragility curves are compared to see the damage behavior of the piers due to seismic action.

2. STATIC ANALYSIS

Bridge piers were designed by using the old (1964) and recent (1998) seismic design codes[3] for highway bridges in Japan. The pier model, cross-section, concrete and steel models that are used in this study are shown in Figure 1. The yield and ultimate capac-

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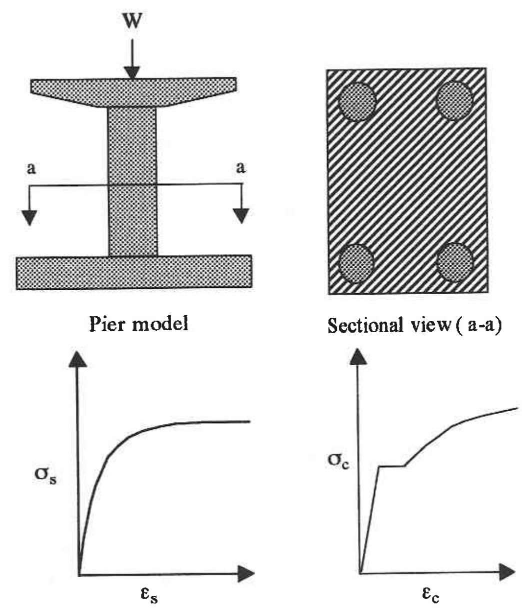

Fig. 1　Pier model, cross-section, concrete and steel model

ity of the pier is obtained following the procedure that is given in the seismic design codes for highway bridges in Japan. However, the moment-curvature relationship for each cross section is obtained using the program Response-2000[4]. Finally, the force-displacement relationship at the top of the bridge pier is obtained from moment-curvature diagram. The moment-curvature and force-displacement diagrams are shown in Figures 2 and 3. From the sectional analysis it is found that the flexural failure governs the failure mode in case of the 1998 pier while the shear failure governs the failure mode in case of the 1964 pier.

3. STRONG MOTION RECORDS

For a nonlinear dynamic response analysis input ground motion records were taken from the Hyogoken-Nanbu earthquake. The earthquake records (acceleration time histories) were chosen on the basis of maximum PGA (peak ground acceleration) and maximum PGV (peak ground velocity) values. A total of 50 original acceleration time histories were chosen from the 1995 Hyogoken-Nanbu earthquake. However, in order to get sufficient damage data in case of extensive and complete damage cases, the original input ground motions were scaled up by 1.5 and 2 times as well as their original scale (1.0). Hence, for nonlinear dynamic response analysis total acceleration time histories are taken as one hundred and fifty. The distribution of PGA and PGV for 150 acceleration time histories is shown in Figure 4.

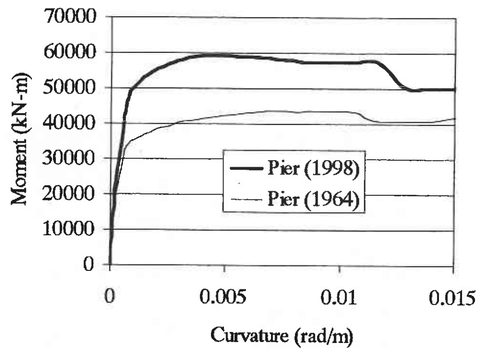

Fig. 2 Moment-curvature diagram of the cross-section at the base of the bridge pier

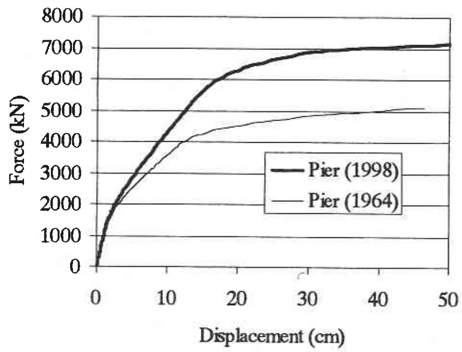

Fig. 3 Force-displacement relationship at the top of the bridge pier

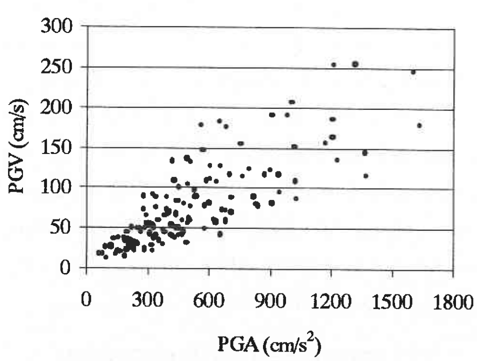

Fig. 4 Distribution of PGA and PGV for the original and scaled records from the Kobe earthquake

4. DYNAMIC ANALYSIS

For the dynamic response analysis the pier is modeled as a single degree of freedom (SDOF) system. A bilinear hysteretic model was idealized and the post yield stiffness[5] is taken as 10% of the secant stiffness of the pier with 5% damping ratio. The yield stiffness of the pier for the both cases is obtained from static analysis. In this case, assuming a bilinear idealization the yield stiffness of the pier is obtained using the yield force and yield displacement. The analytical model of the pier and bilinear hysteretic model are shown in Figure 5. After performing the nonlinear dynamic response analysis the ductility demand at the top of the bridge pier is obtained and the relationship between the PGA and the ductility factor is shown in Figure 6. The ductility factors thus obtained are used for the damage assessment of the bridge pier.

5. DAMAGE ANALYSIS

For the damage assessment of the bridge pier due to the seismic action Park-Ang[6] damage index was used in this study. The damage index DI is given by

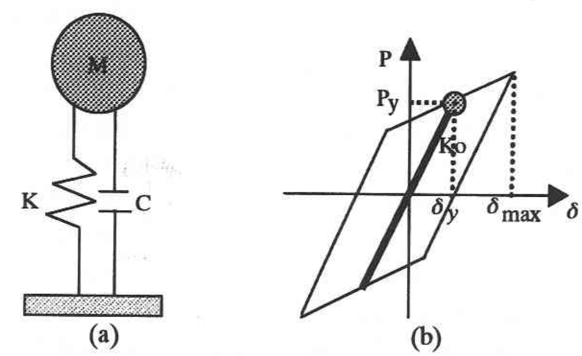

Fig. 5 (a) Analytical model of the bridge pier (SDOF system) and (b) bilinear hysteretic model

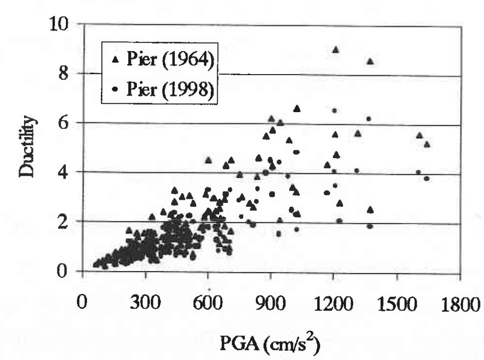

Fig. 6 Relationship between PGA and ductility of the bridge pier

$$DI = \frac{\mu_d + \beta \cdot \mu_h}{\mu_u} \quad \cdots\cdots (1)$$

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

$$\mu_h = E_h / E_e \quad \cdots\cdots (2)$$

where E_h and E_e are the cumulative hysteretic and elastic energy of the pier. The damage index of the bridge pier is obtained using the relationship given in Eq. (1) and the relationship between PGA and damage index is shown in Figure 7. After obtaining the damage indices for the given input ground motion, it is then calibrated to get the relationship between the damage index and damage rank. This calibration is done using the method that is proposed by Ghobarah et al.[7] Table 1 shows the relationship between the damage index and damage rank. Then PGA and PGV values for each damage rank are obtained using this relationship. Figure 8 shows the distribution of PGA and damage rank for 1964 and 1998 bridge piers.

Table 1　Relationship between the damage index and damage rank

Damage Index	Damage Rank	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

6. FRAGILITY CURVES

For each damage rank we have one data set, i.e., PGA and DI. Based on these data, fragility curves for the bridge pier 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

$$P_R = \Phi\left[\frac{\ln PGA - \lambda}{\zeta}\right] \quad \cdots\cdots (3)$$

$$P_R = \Phi\left[\frac{\ln PGV - \lambda}{\zeta}\right] \quad \cdots\cdots (4)$$

where, Φ is the standard normal distribution, λ and ζ are the mean and standard deviation of ln *PGA* and ln *PGV*. 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 paper for the 1964 bridge pier with respect to PGA is shown in Figure 9. Finally

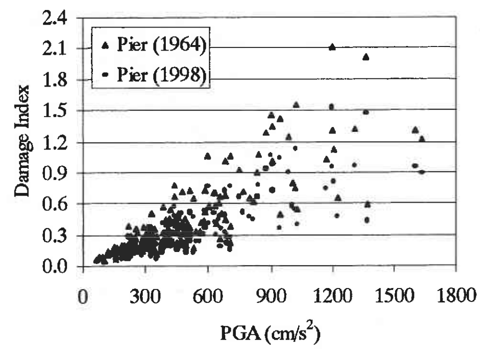

Fig. 7　Relationship between PGA and damage index of the bridge pier

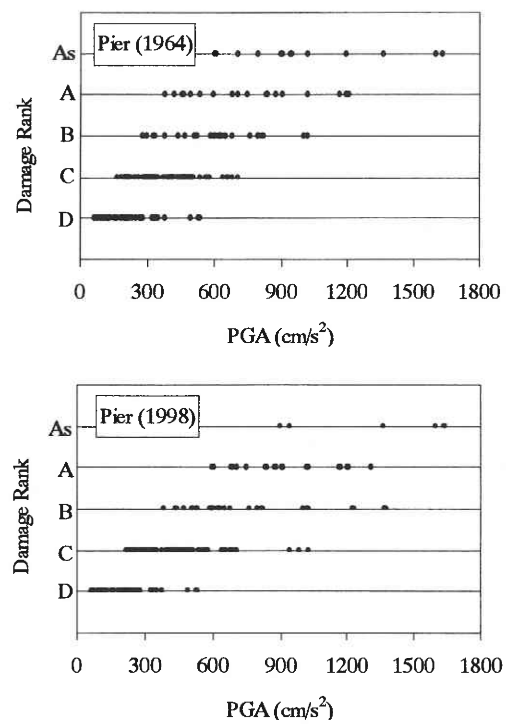

Fig. 8　Distribution of PGA and damage rank for the both 1964 and 1998 bridge piers

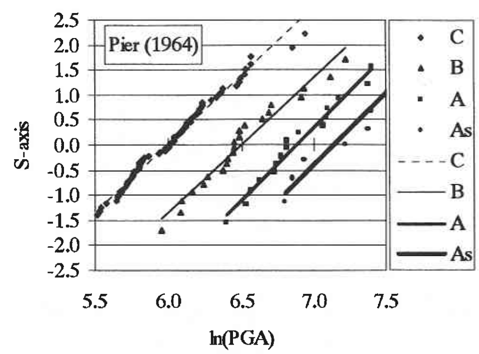

Fig. 9　Lognormal probability paper for the 1964 bridge pier with respect to PGA

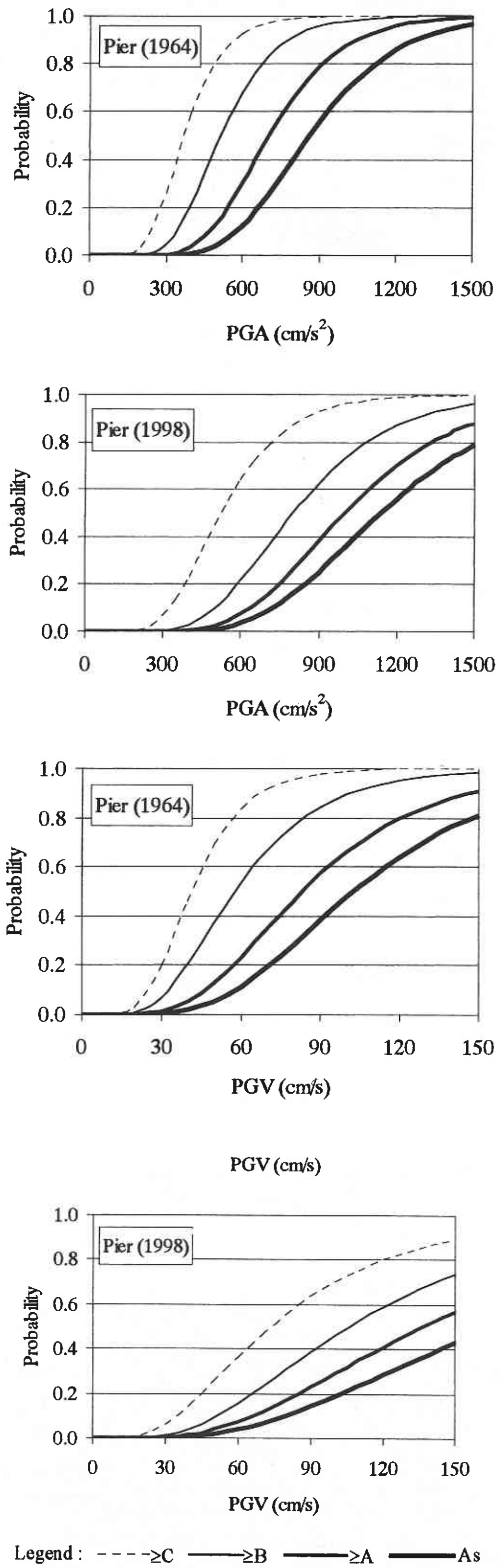

Fig. 10 Fragility curves for RC bridge piers with respect to PGA and PGV from the records of the Kobe earthquake

the fragility curves for damage ranks are constructed using these two parameters. The fragility curves for the both 1964 and 1998 bridge piers with respect to PGA and PGV are shown in Figure 10.

7. CONCLUSIONS

Analytical fragility curves were constructed for a pier designed by the 1964 code and for a pier designed by the 1998 code. For the both cases the input acceleration time histories were chosen from the Hyogoken-Nanbu earthquake. In this case the actual as well as the scaled acceleration time histories were applied to the bridge piers for nonlinear dynamic response analysis. Then the analytical fragility curves were obtained for both PGA and PGV values. After obtaining the analytical fragility curves the performance of the bridge piers as well as the effects of code provisions for highway bridge piers were investigated. It is found that the probability of occurrence of a damage rank is higher in the case of the pier designed by the 1964 code than for the pier designed by the 1998 code. This implies that the pier designed by 1998 code performs well against the seismic action than the pier designed by the 1964 code. The fragility curves thus constructed can be used for damage estimation of highway bridge piers due to earthquakes.

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