DEVELOPMENT OF ANALYTICAL FRAGILITY CURVES FOR RC BRIDGE PIERS USING STRONG MOTION RECORDS

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To estimate damage levels of highway bridge piers due to earthquake ground motion, 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). In this study, based on PGA and PGV values, earthquake ground motion records were chosen from the 1995 Hyogoken-Nanbu and the 1994 Northridge earthquakes. Considering structural parameters and structural responses, the nonlinear dynamic response analysis for RC bridge piers was performed using these records. The damage indices were obtained for the both events and finally the fragility curves for the RC bridge piers were constructed assuming a lognormal distribution.

Key Words: *strong ground motion, PGA, PGV, dynamic analysis, ductility demand, damage index, fragility curves*

1. INTRODUCTION

To estimate damage levels (slight, moderate, extensive and complete) of highway bridge pier due to earthquake ground motion, we need a set of fragility curves, which predict the damage level for a given level of ground motion indices (e.g., PGA, PGV, SI). Based on the actual damage data of highway bridges from the 1995 Hyogoken-Nanbu (Kobe) earthquake, a set of empirical fragility curves[1)] have already been developed. However, these fragility curves do not consider structural parameters. In this study, an analytical approach has been adopted to construct fragility curves considering structural parameters and variation of input motion. First, a hypothetical bridge is considered. Then the pier is modeled using the seismic design codes for highway bridges. Here, both Japanese old (1964) and recent (1998) seismic design codes[2)] for highway bridges are considered. 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 are chosen from the Hyogoken-Nanbu and the Northridge earthquakes. The damage indices are obtained from a nonlinear dynamic response analysis and finally using these damage indices and ground motion indices, fragility curves are constructed assuming a lognormal distribution.

2. STATIC ANALYSIS

Though the pier is taken as a hypothetical one, for better understanding its detailing and all the required parameters are taken from the recent seismic design code (1998) for highway bridges in Japan. The pier model and its cross section are shown in **Fig. 1**. The yield and ultimate capacity of the pier is obtained following the procedure that is given in the seismic design codes for highway bridges. However, the moment-curvature relationship for each cross section is obtained using the program Response-2000[3)]. Finally, the force-displacement relation at the top of the bridge pier is obtained from moment-curvature diagram. In addition, another pier is also designed using the old seismic design code (1964) for highway bridges to observe the behavior of the old pier against the seismic action. The moment-curvature and force-displacement diagrams are shown in **Figs. 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

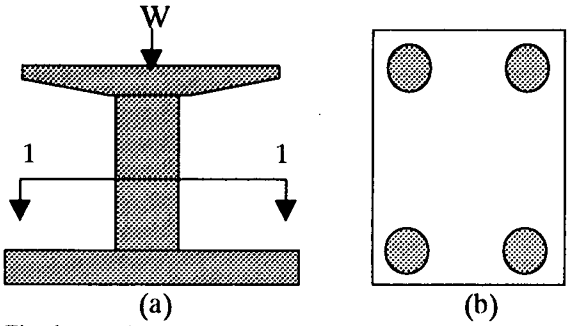

Fig. 1 (a) Bridge pier model and (b) cross-section at 1-1

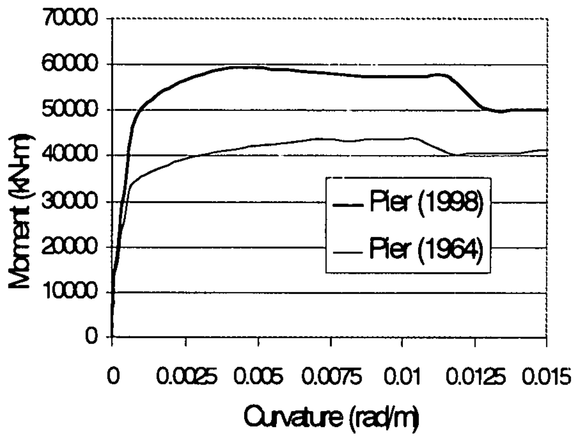

Fig. 2 Moment-curvature diagram of the cross section at the base of the bridge pier.

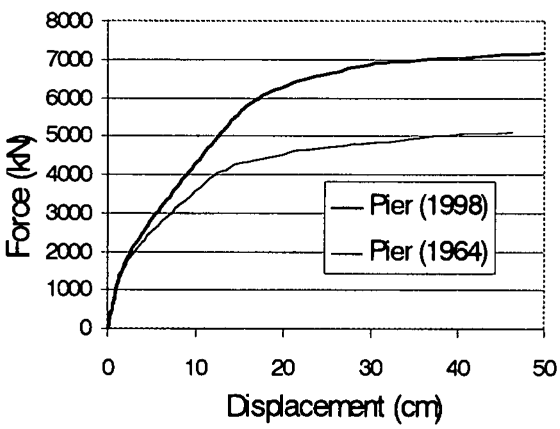

Fig. 3 Force-displacement relationship at the top of the bridge pier.

Hyogoken-Nanbu and the Northridge earthquakes. The earthquake records for the both events were chosen on the basis of PGA (peak ground acceleration) and PGV (peak ground velocity) values. A total of 50 acceleration time histories were chosen for the both events. 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). The distribution of PGA and PGV values for the both events are shown in **Fig. 4**.

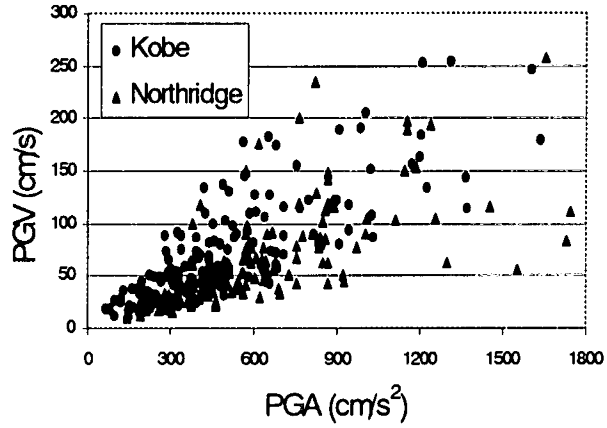

Fig. 4 Distribution of PGA and PGV for the two events (150 records each)

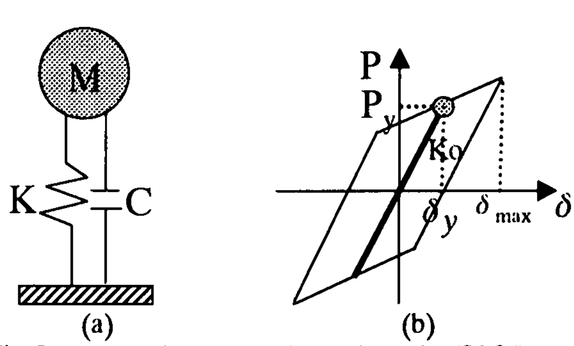

Fig. 5 (a) Analytical model of the bridge pier (SDOF system) and (b) bilinear hysteretic model

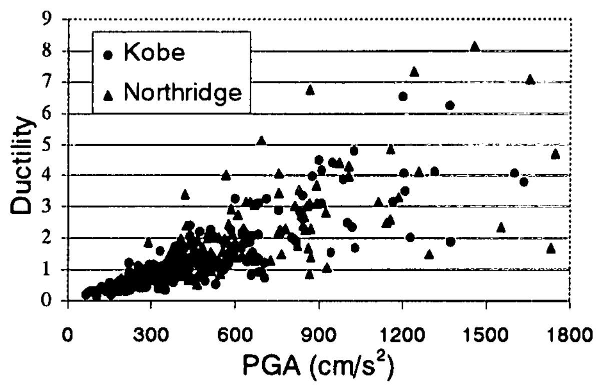

Fig. 6 Relationship between PGA and ductility for 1998 pier.

4. DYNAMIC ANALYSIS

For the dynamic response analysis the pier is modeled as a SDOF (single degree of freedom) system. A bilinear hysteretic model was idealized and the post yield stiffness[4] is taken as 10% of the secant stiffness of the pier with 5% damping ratio. The analytical model of the pier and bilinear hysteretic model are shown in **Fig. 5**. After performing the nonlinear dynamic response analysis the ductility demand at the top of the bridge pier was obtained and the relationship between the PGA and the ductility factor is shown in **Fig. 6**. The ductility factor thus obtained was used for the damage assessment of the bridge pier.

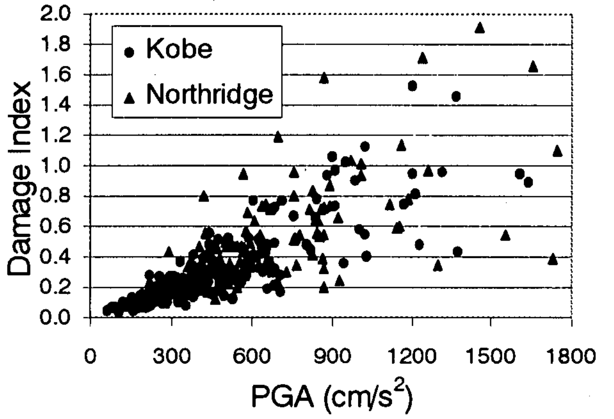

Fig. 7 Relationship between PGA and damage index for 1998 bridge pier

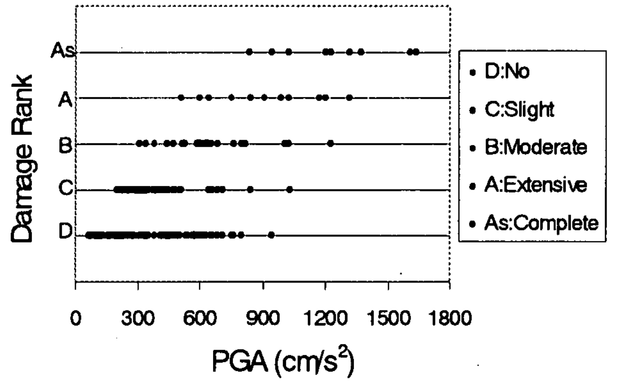

Fig. 8 Distribution of PGA and damage rank due to the Kobe earthquake for the 1998 pier.

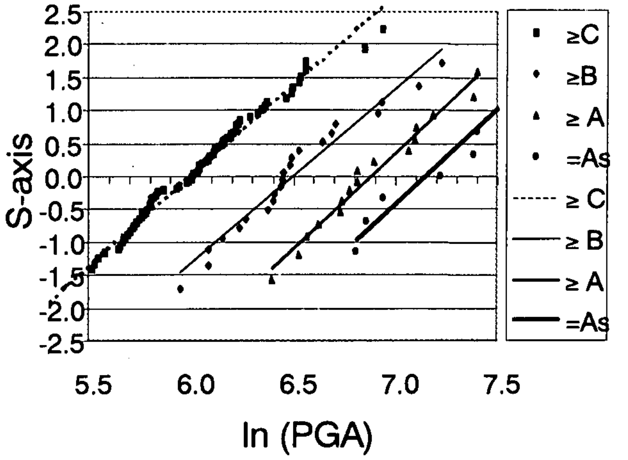

Fig. 9 Lognormal probability paper for PGA of the Kobe earthquake and 1998 bridge pier.

5. DAMAGE ANALYSIS

For the damage assessment of the bridge pier due to the seismic action Park-Ang[5)] damage index was used in this study. The damage index DI is given by

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

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

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

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 that is given in Eq. (1) and the relationship between PGA and damage index is shown in **Fig. 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.[6)]. **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 due to the Kobe earthquake for the 1998 pier.

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 were 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

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

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

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 PGA of the Kobe earthquake and the 1998 pier is shown in **Fig. 9**. Finally the fragility curves for damage ranks were constructed using these two parameters. The fragility curves for the both events and for both 1998 and 1964 piers are shown in **Figs. 10 to 13**.

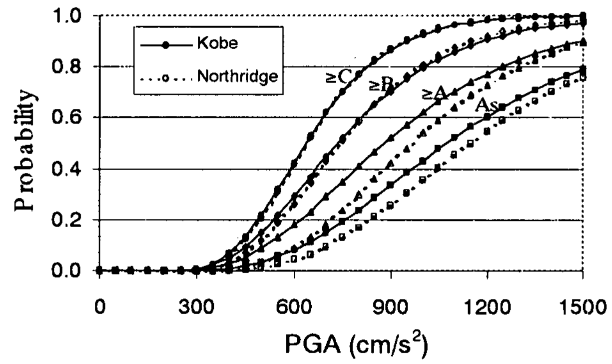

Fig. 10 Fragility curves for RC bridge pier (designed by 1998 code) for PGA.

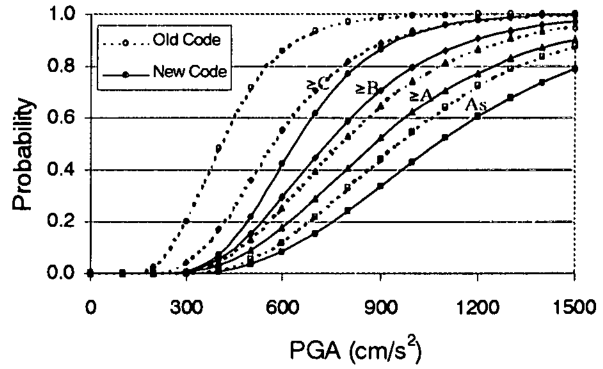

Fig. 12 Fragility curves for RC bridge pier (designed by 1998 and 1964 codes) for PGA of Kobe earthquake.

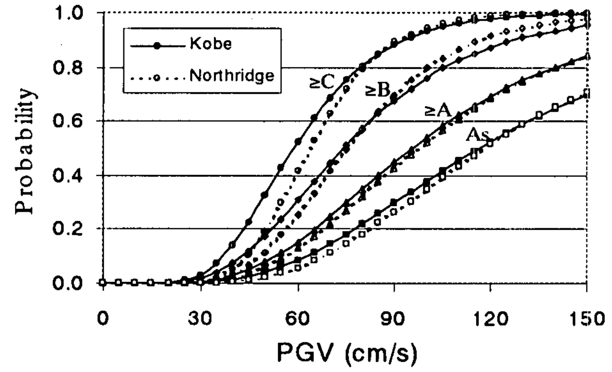

Fig. 11 Fragility curves for RC bridge pier (designed by 1998 code) for PGV.

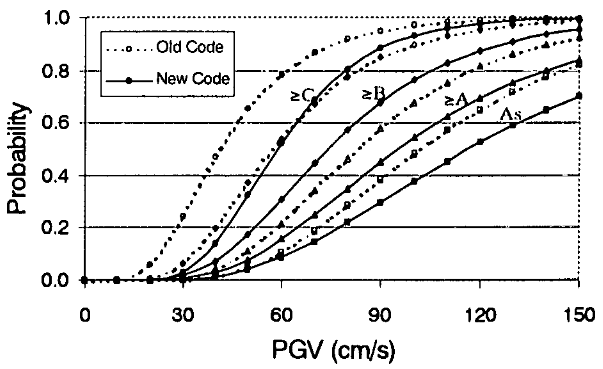

Fig. 13 Fragility curves for RC bridge pier (designed by 1998 and 1964 codes) for PGV of Kobe earthquake.

7. RESULTS AND DISCUSSIONS

After obtaining the analytical fragility curves for the bridge piers, the characteristics of Hyogoken-Nanbu and the effects of code provisions for highway bridge piers were investigated. It is found that the probability of occurrence of a damage rank is similar for the Hyogoken-Nanbu and Northridge earquakes. It is also found that the probability of occurrence of a damage rank is higher in the case of the pier that is designed by 1964 code than for the pier designed by 1998 code. In this study only a hypothetical bridge structure is considered. However, to get better ideas we need to consider a real structure in a future research.

8. CONCLUSIONS

The analytical fragility curves for a bridge piers were obtained using actual and scaled records from the Hyogoken-Nanbu and Northridge earthquakes. It is found that both the earthquake events show a very similar damage characteristics to the pier. It is also found that the pier that is designed by 1998 code performs well against the seismic action than the pier designed by 1964 code.

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