2003 Pacific Conference on Earthquake Engineering

Fragility Curves for Seismically Retrofitted Concrete Bridges

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ABSTRACT: This study presents the development of fragility curves of the Caltrans bridges. The bridges were seismically strengthened following the 1994 Northridge earthquake by the retrofitting of steel jacketing of bridge columns and restrainers at expansion joints. Monte Carlo simulation is performed to study nonlinear dynamic responses of the bridges before and after retrofit. Fragility curves in this study are represented by lognormal distribution functions with two parameters (fragility parameters consisting of median and log-standard deviation) and developed as a function of peak ground acceleration (PGA). The sixty ground acceleration time histories for Los Angeles area developed for FEMA SAC project are used for the dynamic analysis of the bridges and a computer code is developed to calculate hysteretic parameters of bridge columns before and after steel jacketing. The effect of retrofit is expressed in terms of the increase of the median value of the fragility curve for retrofitted bridge from that of the bridge before retrofit. The comparison of fragility curves of the bridges before and after column retrofit demonstrates that the improvement of the bridges with steel jacketing on the seismic performance is excellent for the damage states defined in this study.

1 INTRODUCTION

Several recent destructive earthquakes, particularly the 1989 Loma Prieta and 1994 Northridge earthquakes in California, as well as the 1995 Kobe earthquake in Japan, have caused significant damage to a large number of highway structures that were seismically deficient. The investigation on these negative consequences gave rise to serious discussions about seismic design philosophy and extensive research activity on the retrofit of existing bridges as well as the seismic design of new bridges. This study attempts to present an approach for the assessment of older bridges retrofitted by steel jacketing of columns having substandard seismic characteristics and restrainers installed at expansion joints to prevent bridges spans from unseating. The main objective of the study is to evaluate the effects of column retrofit with steel jacketing for increasing the ductility capacity of circular bridge columns.

2 RETROFIT OF CONCRETE COLUMN

Concrete columns commonly lack of flexural strength, flexural ductility and shear strength, especially in the older bridges. The main causes of these structural inadequacies are lap splices in critical regions and/or premature termination of longitudinal reinforcement.

A number of column retrofit techniques, such as steel jacketing, wire pre-stressing and composite material jacketing, have been developed and tested. Although advanced composite materials and other methods have been recently studied, the steel jacketing has been widely applied to bridge retrofit as the most common retrofit technique.

2.1 Steel jacketing

An experiment was performed to investigate the retrofit of circular columns with steel jacketing (Chai et al. 1991). In this experiment, for circular columns, two half shells of steel plate rolled to a radius slightly larger than the column are positioned over the area to be retrofitted and are site-welded up the vertical seams to provide a continuous tube with a small annular gap around the column. This gap is grouted with pure cement. It is typical that the jacket is cut to provide a space of about 5 cm between the jacket and any supporting member. It is for the jacket to avoid the possibility to act as compressing reinforcement by bearing against the supporting member at large drift angles. It is noted that the jacket is effective only in passive confinement and the level of confinement depends on the hoop strength and stiffness of the steel jacket.

2.2 **Moment Curvature Relationship for Confined Concrete**

It was also observed that confinement of the concrete columns can be improved if transverse reinforcement layers are placed relatively close together along the longitudinal axis by restraining the lateral expansion of the concrete (Chai et al. 1991). It makes it possible for the compression zone to sustain higher compression stresses and much higher compression strains before failure occurs. Unfortunately, however, it is not applicable to existing bridges to enhance the performance of columns by adding transverse reinforcement layers.

The effect of confinement is to increase the compression strength and ultimate strain of concrete as illustrated in Figure 1 (Priestley et al. 1996). Many different stress-strain relationships have been developed for confined concrete. Most of these are applicable under certain conditions. A recent model applicable to all cross-sectional shapes and all levels of confinement is used for the analysis (Priestley et al. 1996).

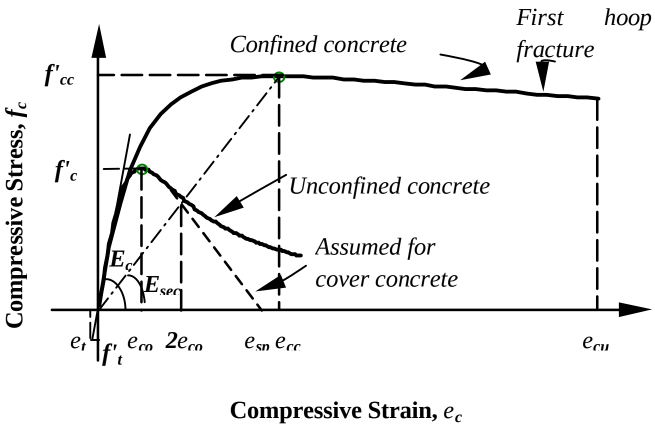


Figure 1. Stress-strain model for concrete in compression.

3 BRIDGE ANALYSIS

3.1 **Description of bridges**

Two (2) sample bridges used for analysis are shown in Figures 2-3. Bridge 1 has the overall length of 242 m with five spans with an expansion joint. This bridge is supported by four columns of equal height of 21 m. Each column has a circular cross section with a 2.4 m diameter. The deck has a 3-cell concrete box type girder section 13 m wide and 2.1 m deep.

Bridge 2 has the overall length 226 m with five spans and two expansion joints. The bridge is supported by different columns in height. Each column has an oblong type cross section. The superstructure deck has a width of 20.7 m and a depth of 2.6 m.

A column is modelled as an elastic zone with a pair of plastic zones at each end of the column. Each plastic zone is then modelled to consist of a nonlinear rotational spring and a rigid element depicted in Figure 4. The plastic hinge formed in the bridge column is assumed to have bilinear hysterestic characteristics. The expansion joint is constrained in the relative vertical movement, while freely allowing horizontal opening movement and rotation. The closure at the joint, however, is restricted by a gap element when the relative motion of adjacent decks exhausts the initial gap width of 2.54 cm.

The opening is also restricted by a hook element when the relative motion exhausts the initial slack of 1.3 cm.

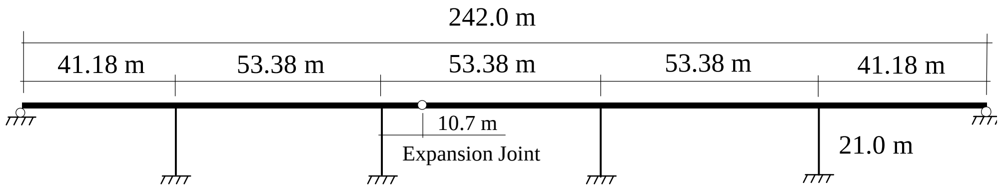


Figure 2. Elevation of Bridge 1.

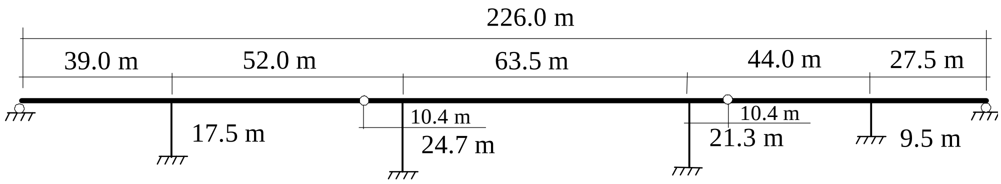


Figure 3. Elevation of Bridge 2.

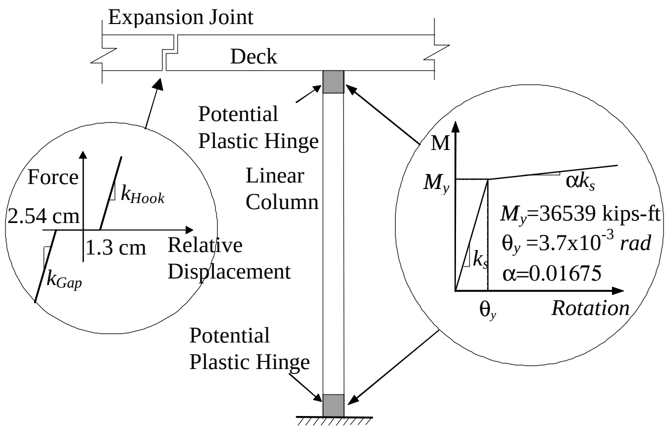


Figure 4. Nonlinearities in bridge model.

3.2 Thickness of steel jacket

The thickness of steel jacket is calculated from the following equation (Priestley et al. 1996).

$$t_j = \frac{0.18(e_{cm} - 0.004)Df'_{cc}}{f_{yj}e_{sm}} \tag{1}$$

where e_{cm} is the strain at maximum stress in concrete, e_{sm} is the strain at maximum stress in steel jacket, D is the diameter of circular column, f'_{cc} is the compressive strength of confined concrete and f_{yj} is the yield stress of steel jacket.

3.3 Analysis

Nonlinear time history analysis has been performed using a computer code SAP2000 Nonlinear for the example bridges under sixty (60) Los Angeles earthquake time histories (selected for the FEMA SAC project) to develop the fragility curves before and after the column retrofit with steel jackets, and with and without restrainers at expansion joints. The parameter used to describe the nonlinear structural response in this study is the ductility demand. The ductility demand is defined as θ/θ_y, where θ is the rotation of a bridge column in its plastic hinge and θ_y is the corresponding rotation at the yield point.

3.4 **Moment-curvature curves**

Nonlinear response characteristics associated with the bridge are based on moment-curvature curve analyses taking axial loads as well as confinement effects into account. The moment-curvature relationship used in this study for the nonlinear spring is bilinear without any stiffness degradation. Its parameters are calculated using a computer code (Kushiyama 2002) according to the equations mentioned in Section 2.2 (Priestley et al. 1996).

Moment-curvature curves for a column of Bridge 1 are plotted in Figure 5. The result shows that the curve after retrofit gives a much better performance than that before retrofit by 2.7 times based on curvature at the ultimate compressive strain.

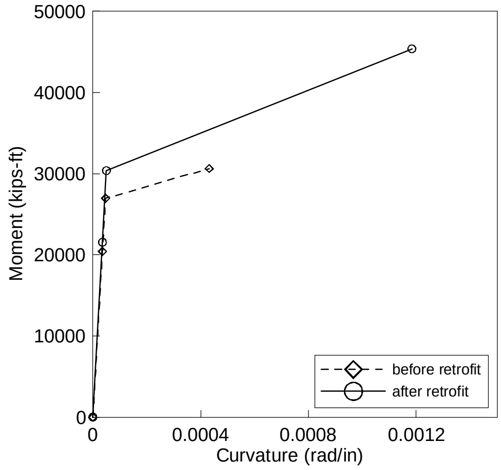
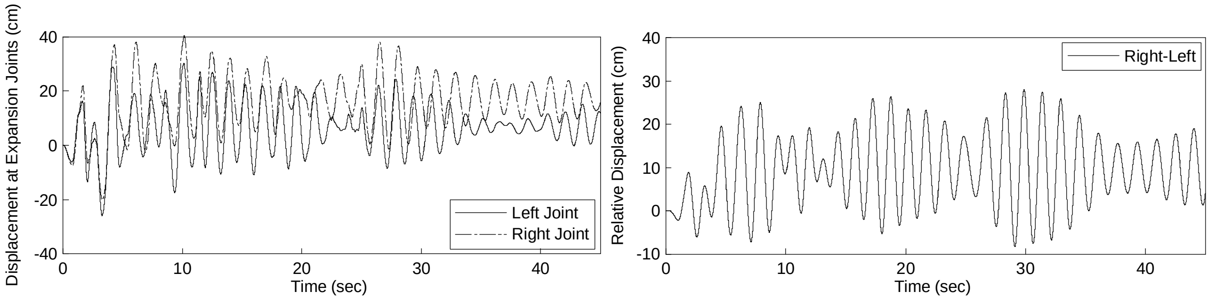


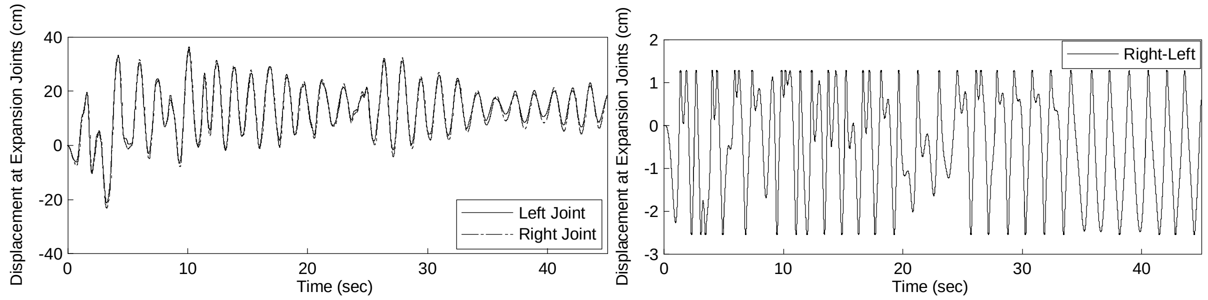
Figure 5. Moment-curvature curves for column of Bridge 1.

3.5 **Responses of bridge**

Typical responses at expansion joints and column bottom end of Bridge 1 are plotted in Figures 6-7.

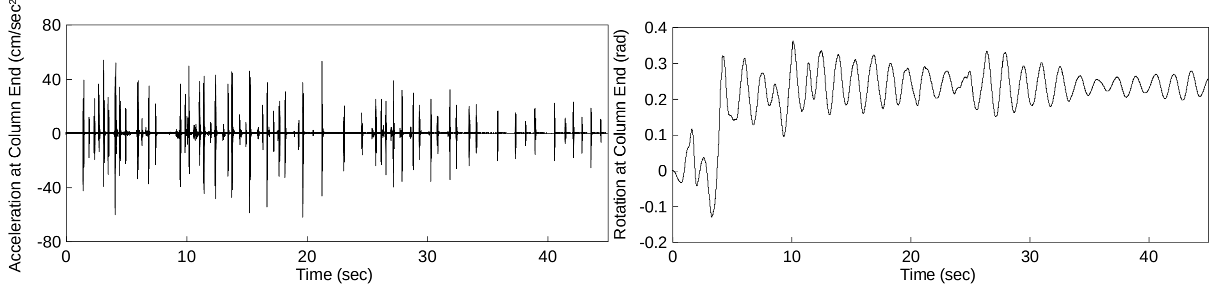


(a) without gap and hook elements

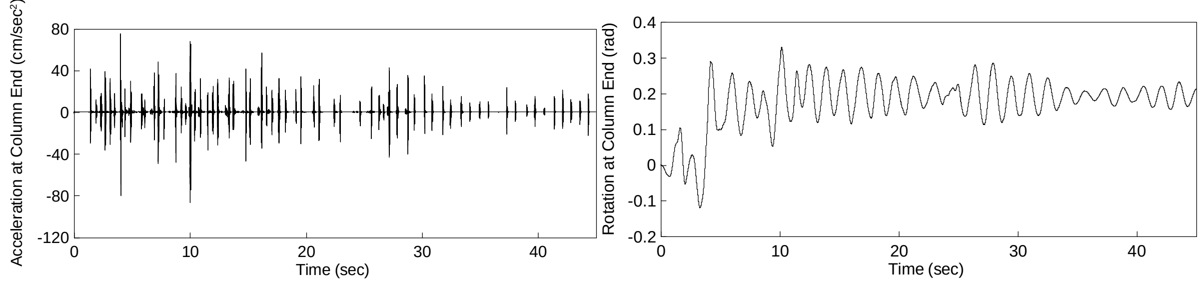


(b) with gap and hook element

Figure 6. Displacement at expansion joints of Bridge 1



(a) before column retrofit with steel jacket



(b) after column retrofit with steel jacket

Figure 7. Responses at column ends of Bridge 1.

It is reasonable to expect that the rotation after retrofit is generally smaller than before, while the accelerations do not necessarily behave that way and can be quite different each other. It is noted that such higher fluctuations in acceleration response appear because opening and closing of the expansion joints are restricted by nonlinear spring elements referred as k_{Gap} and k_{Hook} in Figure 4.

4 FRAGILITY ANALYSIS

It is assumed that the fragility curves can be expressed in the form of two-parameter lognormal distribution functions, and the estimation of the two parameters (median and log-standard deviation) is performed with the aid of the maximum likelihood method. A common log-standard deviation, which forces the fragility curves not to intersect, can also be estimated (Shinozuka et al. 2002).

4.1 Damage states

A set of five (5) different damage states are introduced (Dutta & Mander 2002). Table 1 displays the description of these five damage states and the corresponding drift limits for a typical column. For each limit state, the drift limit can be transformed to peak ductility demand of the columns for the purpose of this study. Table 1 lists the values for the cases before and after retrofit of two (2) sample bridges.

Table 1. Damage states and peak ductility demand of columns.

Damage state	Description	Drift Limits	Bridge 1		Bridge 2	
			before	after	before	after
Almost no	First yield	0.005	1.00	1.00	1.00	1.00
Slight	Cracking, spalling	0.007	1.36	1.83	1.25	2.07
Moderate	Loss of anchorage	0.015	2.81	5.16	2.23	6.37
Extensive	Incipient column collapse	0.025	4.63	9.31	3.46	11.74
Complete	Column collapse	0.05	9.16	19.70	6.54	25.16

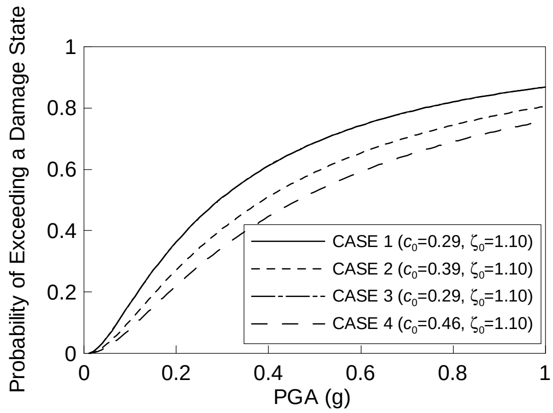
4.2 Fragility curves and effect of retrofit

The moment-curvature curves and the fragility curves for two (2) sample bridges associated with those damage states are plotted in Figures 8-9 as a function of peak ground acceleration, in order to compare and highlight how much bridge retrofit can improve structural behaviour on the seismic performance, along with Tables 2-3 showing the number of damaged bridges respectively.

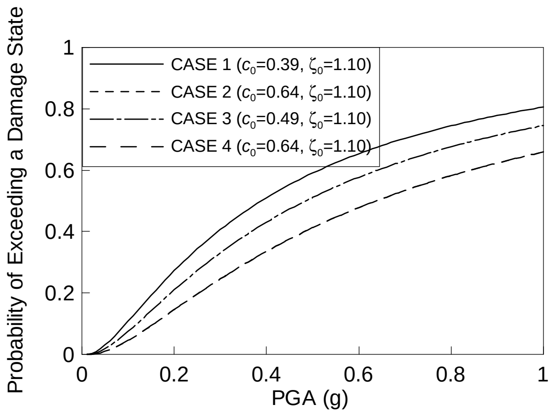
Table 2. Number of Damaged Bridge 1.

sample size=60

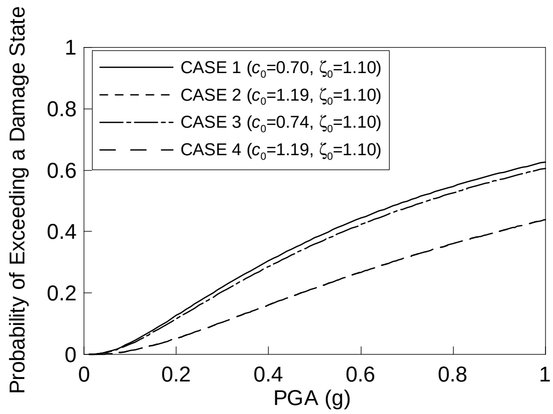
Damage States	Case1	Case2	Case3	Case4
Almost No	50	46	50	45
Slight	46	38	44	38
Moderate	35	20	33	20
Extensive	24	9	24	7
Collapse	11	1	9	1



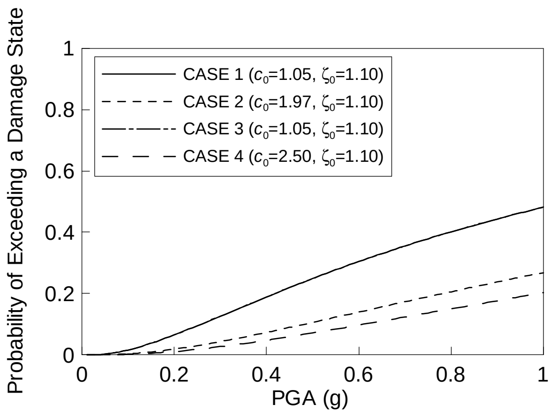
(a) Almost No Damage



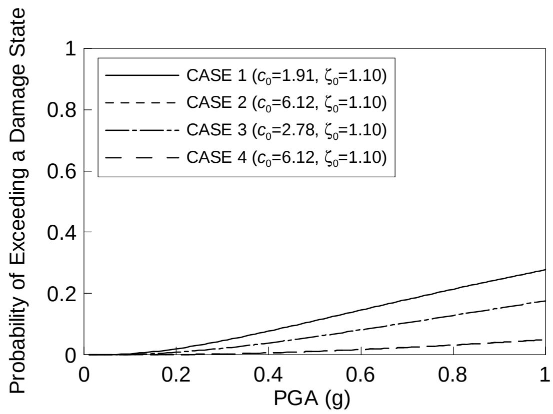
(b) Slight Damage



(c) Moderate Damage



(d) Extensive Damage



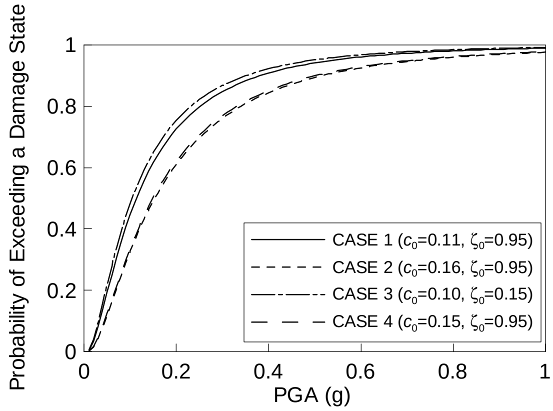
(e) Complete Collapse

Figure 8. Fragility Curves of Bridge 1.

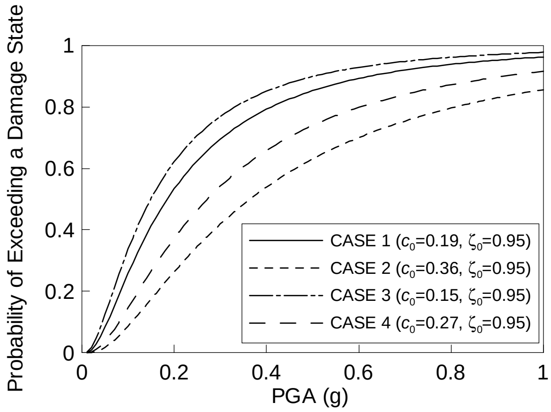
Table 3 .Number of Damaged Bridge 2.

sample size=60

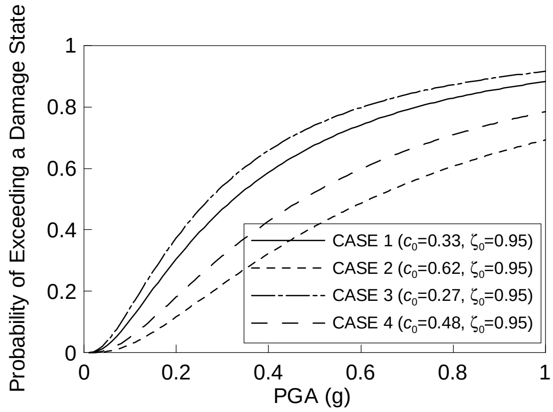
Damage States	Case1	Case2	Case3	Case4
Almost No	59	57	58	55
Slight	56	44	55	51
Moderate	47	28	51	36
Extensive	41	17	42	22
Collapse	28	4	36	8



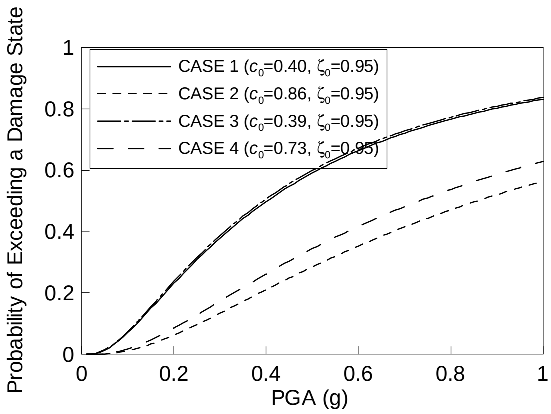
(a) Almost No Damage



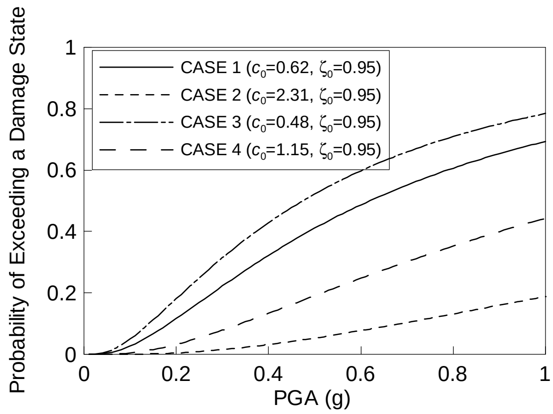
(b) Slight Damage



(c) Moderate Damage



(d) Extensive Damage



(e) Complete Collapse

Figure 9. Fragility Curves of Bridge 2.

Case 1 is before steel jacketing and without restrainers, case 2 after steel jacketing and without restrainers, case 3 before steel jacketing and with restrainers, and case 4 after steel jacketing and with restrainers. It is noted that bridge with jacketed columns is expected to be less vulnerable to ground motion than bridge with the columns not jacketed and therefore we expect that the group of these fragility curves should not theoretically intersect.

It shows that the effect of column retrofit on the seismic performance is excellent explaining that the bridges are less fragile up to 320% for Bridge 1 and 373% for Bridge 2 after retrofit compared with the case without retrofit in terms of the median values, while the effect of restrainers at expansion joints is found to be negligible for Bridge 1 and even adversely affects on the column responses for Bridge 2. However, this observation might not always apply, depending on the details of specific bridge characteristics.

The damage state of a bridge in this study is defined in terms of the maximum value of the peak ductility demands sustained by all the column ends. In this context, comparison between fragility curves in Figures 8-9 indicates that the bridge is less susceptible for damage to the ground motion after column retrofit than before. In fact, the simulated fragility curves in this study demonstrate that, for all levels of damage states, the median fragility values after column retrofit are larger than the corresponding values before retrofit. If the number of times each bridge suffers from a certain state of damage is counted, it is smaller when bridge is subjected after column retrofit than before it to the sixty (60) different ground motions introduced in Section 3.3 and used for the damage simulation. The number is listed in Table 2-3 for before and after retrofit to Bridge 1 and Bridge 2.

5 CONCLUSION

The computed analytical fragility curves corresponding to these damage states appear to make intuitive sense relative to the bridge's design, retrofit and performance in past seismic events. The following conclusions can be made on the results of this study.

1) The simulated fragility curves after column retrofit with steel jacketing show excellent improvement (less fragile) compared to those before retrofit by as much as 3.73 times based on median PGA values simulated.

2) The effect of restrainers at expansion joints is found to be negligible or even adversely affects on the column responses.

3) After column retrofit, the number of damaged bridges is substantially decreased especially for severe damage states.

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