RELIABILITY-BASED DESIGN OF SEISMIC RETROFIT FOR BRIDGES

by

Kenneth J. Fridley, Professor and Head

and

Zhiyuan Ma, Former Graduate Research Assistant

Department of Civil, Construction, and Environmental Engineering
The University of Alabama
Tuscaloosa, AL 35487-0205

Submitted to:

The Washington State Department of Transportation

February 2007

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENTS CATALOG NO
WA-RD 664.1		

4. TITLE AND SUBTITLE	5. REPORT DATE
Reliability-Based Design of Seismic Retrofit for Bridges	February 2007
	6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)	8. PERFORMING ORGANIZATION REPORT NO.
Kenneth J. Fridley and Zhiyuan Ma	

9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. WORK UNIT NO.
The University of Alabama Dept of Civil, Construction, & Environmental Engineering Tuscaloosa, AL 35487-0205	11. CONTRACT OR GRANT NO.

12. COSPONSORING AGENCY NAME AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
Research Office Washington State Department of Transportation Transportation Building, MS 47372 Olympia, WA 98504-7372 Kim Willoughby, Project Manager, 360-705-7978	Final Research Report 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This research focused on developing reliability-based seismic retrofit assessment procedures for highway bridge columns. Fragility curves were developed to assess the relative performance of various retrofit methods considering several limit states. Analytical fragility curves are developed from scaled earthquake records and structural response models. A simplified, single-degree-of-freedom (SDOF) structural model was assumed to reasonably represent the structural response of the bridge columns for this research. The structural configuration, geometry, and properties of the bridge columns came from Washington State Department of Transportation standard bridge designs. Scaled earthquake data were used to calculate the displacement history during the prescribed earthquakes, and structural displacement, ductility and dissipated energy, were determined to calculate damage indices (DI). Various retrofit methods were selected from the literature. For the retrofitted cases, different degrees of increase in mass, stiffness, damping, and ductility were taken from the literature, over those of the un-retrofitted structure.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
WSDOT, reliability, fragility, bridge columns, seismic retrofit	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None		

DISCLAIMER

The contents of this report reflect the view of the author, who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Transportation Commission, Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This research focused on developing reliability-based seismic retrofit assessment procedures for highway bridge columns. Fragility curves were developed to assess the relative performance of various retrofit methods considering several limit states. For this research, an analytical approach was adopted in which fragility curves are developed from scaled earthquake records and structural response models. A simplified, single-degree-of-freedom (SDOF) structural model was assumed to reasonably represent the structural response of the bridge columns for this research. The structural configuration, geometry, and properties of the bridge columns came from Washington State Department of Transportation standard bridge designs. Scaled earthquake data were used to calculate the displacement history during the prescribed earthquakes, and structural displacement, ductility and dissipated energy, were determined to calculate damage indices (DI). Various retrofit methods were selected from the literature. For the retrofitted cases, different degrees of increase in mass, stiffness, damping, and ductility were taken from the literature, over those of the un-retrofitted structure. Hence, modified (retrofitted) structural properties were used to repeat the calculations for the retrofitted cases. The developed fragility curves were found to be well-represented by a lognormal distribution. To provide an approximate design approach, the design values of the materials were assumed in the analysis versus assuming material values to be represented as random variables. The so-called "deterministic" curves were found to be slightly non-conservative, but may be useful considering the reduced computational time. This research considered 5 cases: (1) the un-retrofitted (or as-built) case, (2) quarter-height steel jacketing, (3) half-height steel jacketing, (4) full-height steel jacketing, and (5) full-height composite jacketing. Four performance levels were considered for each case: (1) slight damage curve, (2) moderate damage curve, (3) extensive damage curve, and (4) collapse curve. In total, a

suite of twenty fragility curves was developed (5 cases × 4 performance levels each). This study indicates that the fragility analysis approach may be used to rationally compare and select optimal retrofit strategies based on target design performance levels.

INTRODUCTION

Many structures, including bridges, built before 1970 are not adequately detailed for seismic actions. This means bridge structures have historically been vulnerable to seismic loading, with a large number of examples of damage occurring to both superstructure and substructure elements and, in some cases, complete and catastrophic collapse. The turning point event in changing structural design philosophy was the 1971 San Fernando earthquake. That event caused severe damage to major lifelines, including the transportation lifeline. Bridges, an important component of the transportation lifeline, suffered major damage. This prompted almost all the state departments of transportation in earthquake prone areas to upgrade their design specifications and structural details to resist earthquake action, as did the Federal Highway Administration (FHWA). However, even with this, there are still many existing older bridges that were designed and built prior to 1971. It is critical that these bridges be upgraded to current seismic design standards in regions with strong earthquake potential. While many methods and approaches are available to accomplish this, methods to systematically and rationally compare these methods and select the optimal method are lacking, particularly in terms of target performance levels. The aim of this research is to explore one such method based on fragility analyses.

BACKGROUND

Seismic hazards include ground shaking, fault rupture, soil liquefaction and lateral or vertical ground movements. Through previous research (e.g., Priestley, 1987; Xiao & Ma, 1997), columns in many existing bridges been shown to typically have the following potential problems:

1. Undependable flexural capacity due to poor details in longitudinal lap splices;
2. Insufficient ductility due to improper transverse confinement;
3. Insufficient shear strength; and
4. Improper details and insufficient strength in the column/footing and column/superstructure joints.

The related damage of bridges observed in recent earthquakes may generally be categorized into particular classifications, including (1) foundation/abutment damage due to soil failure or movement; (2) collapsed or unseated spans due to bearing failures or inadequate support widths; and (3) significant damage due to inadequate strength or ductility of columns from inadequate development of reinforcement, insufficient confinement reinforcement, and poorly detailed transverse reinforcement (Cooper et al., 1994; Priestley & Seible, 1991).

For bridges, there are many strategies for improving their resistance to seismic hazards. Approaches include (1) strengthening; (2) isolation; (3) increasing ductility; (4) improved displacement allowance; and (5) improved energy dissipation. However, some effective and available retrofit strategies (e.g., jacketing) for existing bridge substructures mainly include strengthening and increasing ductility. This study focuses on comparing retrofit strategies, specifically for jacketing, which may be an appropriate retrofit strategy for bridge columns.

DEVELOPMENT OF FRAGILITY CURVES

A fragility curve displays the conditional probability that a structure surpasses some defined limit state at different levels of load or other actions. For seismic fragility, the curves represent the probability of seismic damage at various levels of ground shaking, which is described for the purposes of this research in terms of peak ground acceleration (PGA).

Yamazaki et al. (1999) developed a set of empirical fragility curves based on the actual damage data acquired from the 1995 Hyogo-ken Nanbu (Kobe) earthquake. Shinozuka et al. (2000) presented both empirical and analytical approaches for fragility curves. Kim and Shinozuka (2003) then developed fragility curves for concrete bridges retrofitted by column steel jacketing. The fragility curves were expressed in the form of a two-parameter lognormal distribution function with the estimation of the two parameters performed per an optimization algorithm, and it could be achieved through ground motion records and seismic structural response analyses.

SEISMIC RETROFIT METHODS FOR HIGHWAY BRIDGE COLUMNS

Steel Jacketing

As one of the column retrofit methods, steel jacketing has been proven to be very effective in improving column strength and ductility (Chai et al., 1991; Priestley et al., 1994; Xiao et al., 1996). Steel jacketing is a term used to describe the external encasement of columns by prefabricated steel shells welded in situ. Depending on the type of column (rectangular and circular), the jacket is typically either elliptical or circular. In the case of circular columns, the jacket is fabricated slightly oversized and the gap between the jacket and column is filled with cement-based grout to ensure composite action between the jacket and column. Extensive experimental research has shown that steel jacketing is effective in enhancing the flexural capacity and ductility, as well as the shear capacity of the column. Flexural tests on 0.4:1 scale models of circular bridge columns (Chai et al., 1991) indicated that retrofit of pre-1971 columns with cylindrical jackets corresponded to a 3.1% volumetric confining ratio enabled the column to develop ductile behavior. Stable hysteretic loops were obtained up to displacement ductility factors or corresponding to drift ratios exceeding 5%. Steel jacketing also prevented bond failures that might otherwise develop in "as-built" circular columns detailed with inadequately lapped longitudinal reinforcement.

The steel jacket may be idealized conservatively as a series of independent closely spaced peripheral hoops with thickness and spacing equal to the jacket thickness. The steel jacket will normally be required over the full height of the column if the shear strength enhancement for the columns is needed. For example, full-height steel jacketing is often times used retrofitting the squat column. However, for the typical bridge column type (i.e., tall prismatic circular and relatively slender columns, which account for a considerable portion of the bridge columns), full-height steel jacketing may attract excessive seismic force because of its greatly increased stiffness. In this case, using partial-height steel jackets is appropriate. In this research, a quarter-height steel jacket and half-height steel jacket were used as partial-height jacketing cases to compare the full-height steel jacketing.

Composite Jacketing

Although steel jacketing has been widely used in the United States, other alternatives to improve the retrofitting process for the vast number of existing, structurally deficient bridges both in the United States and throughout the world are being sought. One of the key goals is to ease construction and increase the strength-weight ratio (Xiao & Ma, 1997). Based on this fact, advanced composite materials have been recognized for their potential and have been applied to retrofit existing bridge. Compared with steel jacketing, the general expectations from composite retrofit system include lightweight and high stiffness- or strength-to-weight ratios. Several composite-jacketing systems had been developed and validated in laboratory or field conditions. Matsuda et al. (1990) first tested a system for bridge pier retrofit using unidirectional carbon fiber sheets wrapped longitudinally and transversely in the potential plastic hinge region. Priestley and Seible (1991) experimentally evaluated another composite wrapping system using E-glass fiber, which is more economical than carbon fiber. Saadatmanesh et al. (1994) proposed a wrapping technique using glass fiber composite straps for column retrofit. Finally Seible et al. (1995) experimentally validated a carbon fiber system that uses an automated machine to wrap carbon bundles to form a continuous jacket.

These composite retrofit measures can be categorized as in-situ fabricated jacketing that involves hand or automated machine placement of epoxy saturated glass or carbon-based fabrics on the surface of existing concrete (Xiao et al., 1996). An in-situ fabricated jacket can match the shape of an existing concrete column, which is an advantage over the steel jacket. Three model columns with composite jackets were constructed and tested by Xiao et al. (1995), each layer of the jackets was prefabricated with unidirectional glass fiber sheets and two-part epoxy. The tests showed that using prefabricated composite jacketing could effectively delay structural brittle failure, significantly improve hysteretic responses and increase ductility of the retrofitted columns. Further research (Priestly et al., 1994; Xiao et al., 1999) indicated that prefabricated composite jacketing systems are advantageous over steel jacketing that may stiffen a column by over 30% and thus may result in attracting excessive earthquake force by the steel jacketed columns. In this study, full-height composite jacketing was considered as a comparison to steel jacketing.

SEISMIC DAMAGE EVALUATION PROCEDURE

A nonlinear dynamic response analysis of the column is performed with a single-degree-of-freedom (SDOF) representation. Material uncertainties were assumed to follow the normal distribution for concrete compressive strength and the lognormal distribution for steel yield strength. The structural ultimate displacement, δ_u, was assumed to be 6 times the yield displacement, δ_y, for elasto-plastic (bilinear) model response (Guzman et al., 2002).

For the purposes of this research, the peak ground acceleration (PGA) was selected to represent the ground motions intensity, although there are other intensity measures such as peak ground velocity and spectral acceleration. For the nonlinear analysis, 30 ground motion records were chosen from each of ten earthquake recording stations. Using a time-history scaling procedure, the selected PGA values were scaled to different target levels (i.e., immediate occupancy, life safety and collapse prevention), and ranged from 0.05 g to 1.5 g. Using these time histories as input ground motion records, the damage indices of the bridge columns were calculated from the dynamic analysis. The different damage levels or scales, which are related to the damage indices, are also obtained. The probabilities of damage are based on 300 records, hence they were derived according to each PGA, respectively. The fragility curves for highway bridge columns were then constructed considering both structural model parameters and various ground motion records. The steps to accomplish this can be expressed as follows: (1) select ground motion records and modeling the SDOF system; (2) scale ground motion records to different PGA values; (3) dynamic analysis to obtain structural response (i.e., displacement) history; (4) calculate damage indices; (5) model retrofitted structure by increasing stiffness, mass, damping, steel and concrete strength; 6) repeat Step 3 and Step 4 to obtain retrofitted damage indices; (7) derive probability of structural damage according to scaled PGA; (8) plot fragility curves; (9) select design PGA values from seismic hazard maps; and (10) compare seismic retrofit methods.

FRAGILITY ANALYSIS FOR UN-RETROFITTED CASE

Bridge and Column Details

To obtain fragility curves analytically, a "typical" bridge was assumed as provided by the Washington State Department of Transportation (WSDOT). The length of each span was 36 m (120 ft), the weight per unit length of the bridge deck was assumed to be 190 kN/m (13 kips/ft), and then the weight of the bridge superstructure was 6840 kN (1560 kips). The height of the typical column was 7.62 m (25 ft) with a diameter of 1.68 m (5.5 ft). The column was assumed to have a unit weight of 23.57 kN/m^3 (0.15 kips/ft^3) thus the total weight of the column was calculated as 390 kN (89 kips). The total weight of the modeled structure is 7230 kN (1649 kips). From the WSDOT column design requirements, the longitudinal reinforcing and the tie reinforcing steel ratio were taken as 1.5% and 0.1%, respectively.

Material Uncertainties

In general, material strengths were considered as random variables, while other parameters like structural geometries, mass, or unit weight was considered deterministic (Tantala & Deodatis, 2002). In traditional design, it is assumed that the values of the yield strength f_y (and hence ultimate strength f_u) of the reinforcing steel and the nominal compressive strength of concrete f_c' accurately represented the true material properties. However, in assessing the fragility for an existing or new structure, the exact values of these material property constants are typically not known and, therefore, are considered random variables. Accordingly, the parameters of the moment-curvature envelope (yield moment, yield rotation, and post-yield stiffness) are therefore functions of these random variables.

For tall buildings, it is assumed that both steel and concrete strengths follow normal distributions (Ellingwood et al., 1980). The first two rows of Table l shows the related statistical

Table 1. Distributions of Material Uncertainties

Random Variables	Symbol	Nominal	Mean	COV	Distribution	Reference
Concrete Compressive Strength	f_c'	20.7 Mpa	23.4 Mpa	18.0%	Normal	Ellingwood et al.
Grade 60 Steel yield Strength	f_y	414 Mpa	465 Mpa	9.8%	Normal	(1980)
Concrete Compressive Strength	f_c'	27.6 Mpa	31 Mpa	20.0%	Normal	Shinosuka et al.
Grade 40 Steel yield Strength	f_y	276 Mpa	336 Mpa	10.7%	Lognormal	(2000)
AX Concrete Compressive Strength	f_c'	41.4 Mpa	43.4 Mpa	20.0%	Normal	This Research
A15 Steel Yield Strength	f_y	345 Mpa	393 Mpa	11.0%	Lognormal	(2003)

parameters of material properties, where COV is the coefficient of variation and is equal to the percent ratio of the standard deviation to the mean value. For non-building structures such as RC bridge columns or piers, Shinosuka et al. (2000) utilized the values provided in rows three and four of Table 1. Note here that Grade 40 reinforcing steel was adopted instead of Grade 60. Hence the COV of the concrete compressive strength varies from 15% to 20%, and the COV of steel yield strength varies from 6% to 11%. In this research, the COV of 20% for f_c' and 11% for f_y were assumed. Based on the assumed column materials, which includes an intermediate grade steel ASTM A-15 with nominal f_y 345 MPa (50 ksi) and AX class concrete with nominal f_c' 41.4 MPa (6000 psi), the distributions and their statistical parameters were assumed as shown in Table 1, rows five and six.

Dynamic Analysis

The bridge column was assumed, for the purpose of this research, to be adequately represented as a single-degree-of-freedom (SDOF) model to perform the required dynamic analyses. An elasto-plastic, idealized hysteretic response was considered. The pre-yield stiffness was calculated as 28.74 kN/m (164 kips/in.) considering second order effects and the post-yield stiffness was assumed to be zero with a 5% damping ratio. To calculate the maximum displacement, numerical methods, as well as linear interpolation, were used with a time step 0.1

seconds. For various selected time histories, each maximum structural displacement (or peak drift for the model) was derived. Then, the ductility demand at the top of the bridge column was obtained. The ductility is defined as the ratio of the structural maximum displacement to yield displacement. The latter was taken as the ratio of yield strength to stiffness and calculated as 2.22 mm (0.875 in.).

The dissipated hysteretic energy during the seismic response was also determined. This can be expressed as a function of time *t*, including both viscous damping and yielding energy as:

$$E_h = E_d(t) + E_y(t) = \int_0^t c[\dot{x}(t)]^2\, dt + \int_0^t \dot{x} f_s(x, \dot{x})\, dt - \frac{[f_s(t)]^2}{2k} \tag{1}$$

where E_h is dissipated hysteretic energy, E_d is damping energy, E_y is yielding energy, f_s is resisting force equivalent to static force, c is damping coefficient taken as 0.85 for the structural model, and x is structural relative displacement.

Damage Index (DI) and Damage Scale (DS)

For structural damage assessment, Park-Ang (1985) defined the damage index (*DI*), which has been adopted in many recent studies (e.g., Guzman & Ishiyama, 2003; Karim & Yamazaki, 2000; Zhu & Ni, 2001), as follows:

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

where μ_d is the is placement ductility, μ_u is the ultimate ductility, β is the cyclic loading factor and taken as 0.15, and μ_h is the cumulative energy ductility defined as:

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

where E_h and E_e are defined as the hysteretic and elastic energy, respectively.

The damage indices of the bridge column were obtained using Equations (1), (2), and (3). To establish the relationship between damage indices and damage levels, damage scales (DS) were adopted. There are various definitions of existing damage scales (Rossetto & Elnashai, 2001). The damage scale proposed by Ghobarah et al. (1997) was used in this study as shown in Table 2. The damage scale then ranges from slight damage to collapse, and it can be seen that each damage scale has a certain range of damage indices.

Table 2. Definitions for Damage Scale

Damage Index (DI) Values	Damage Scale (DS) Definitions
$0.00 < DI \leq 0.14$	No Damage
$0.14 < DI \leq 0.40$	Slight Damage
$0.40 < DI \leq 0.60$	Moderate Damage
$0.60 < DI \leq 1.00$	Extensive Damage
$DI \geq 1.00$	Collapse

Probability of Damage vs. PGA

For each of the ten selected earthquake station records, 30 time histories were developed. These ground motion records came from one earthquake (Satsop, WA Sunday, June 10, 2001 06:19 AM PDT -- 11.2 miles N. of Satsop, WA, Magnitude 5.0). The number of occurrences of each damage scale was then used to obtain the probability of damage. PGA values from the selected records were scaled to different excitation levels. In this study, PGA values were scaled from 0.05 g to 1.5 g providing thirty-four (34) excitation levels. Note that these target levels are not on even steps. 0.05 g - 0.1 g level step is 0.01 g, 0.1 g - 1.5 g level step is 0.05 g.

Figure 1 shows the number of occurrence of various damage scales in 34 target levels, as well as the related probability of occurrence. Based on the above results, the cumulative probability of damage for this earthquake was obtained with respect to PGA. Four series of points were plotted, which represent the probability of slight damage, moderate damage,

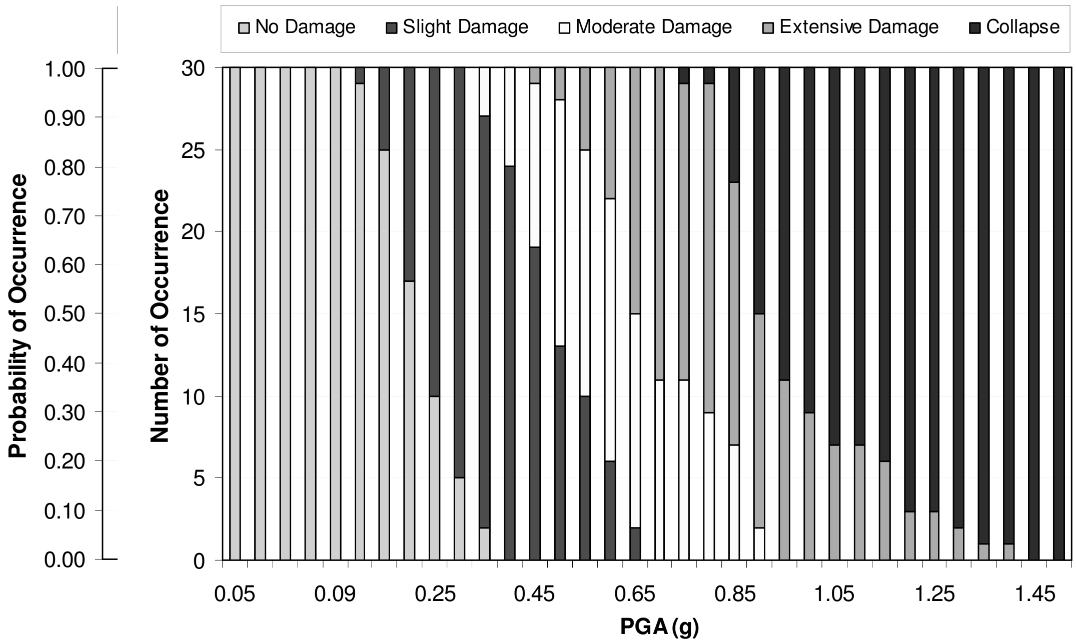


Fig. 1. Number and probability of occurrence vs. PGA for Satsop EQ, 2001

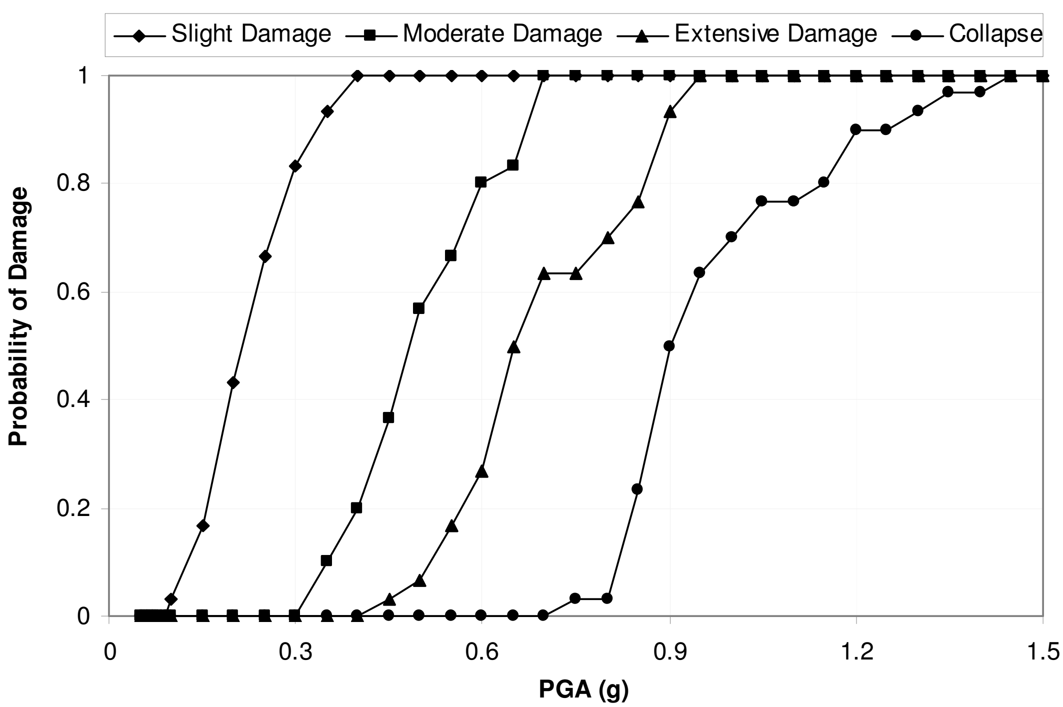


Fig. 2. Probability of damage vs. PGA for Satsop EQ, 2001

extensive damage, and collapse versus PGA, respectively. The data points for one such analysis are shown in Figure 2.

FRAGILITY CURVES FOR UN-RETROFITTED CASE

To fully develop the fragility curves, the procedure outlined for the Satsop earthquake was extended to 9 additional earthquakes, therefore, a total of 300 ground motion records (10 earthquakes × 30 stations each) were selected and adopted as computer input to obtain more points of probability of occurrence versus PGA. Additionally, to realize material uncertainties, random number generators were employed. In this research, 100 realizations (which means the computer program would run 100 times for each set of simulations, and totally 100 randomizations × 300 ground motion records = 30,000 points per PGA value would obtained) were assumed for each case (including both unretrofitted and retrofitted bridge columns) to achieve more points. Figure 3 shows an example of material-randomized points (of probability of damage versus PGA). In Figure 3, the various points represent the random variables of the material uncertainties, or specifically, they represent the results from 100 sets of simulation (100 pairs of f_y and f_c'), and one earthquake (Satsop EQ, 2001).

From the above plotted points, it was assumed that these points might follow lognormal distribution. Shinozuka et al. (2000) suggested that the assumed function could be expressed as follows:

$$P_d = F(a) = \Phi\left[\frac{\ln(a/c)}{\zeta}\right] \tag{4}$$

where P_d is cumulative probability of damage and is a function of PGA a, $\Phi[\cdot]$ represents standard normal distribution function, and c and ζ are two lognormal distribution parameters that satisfy the following equation:

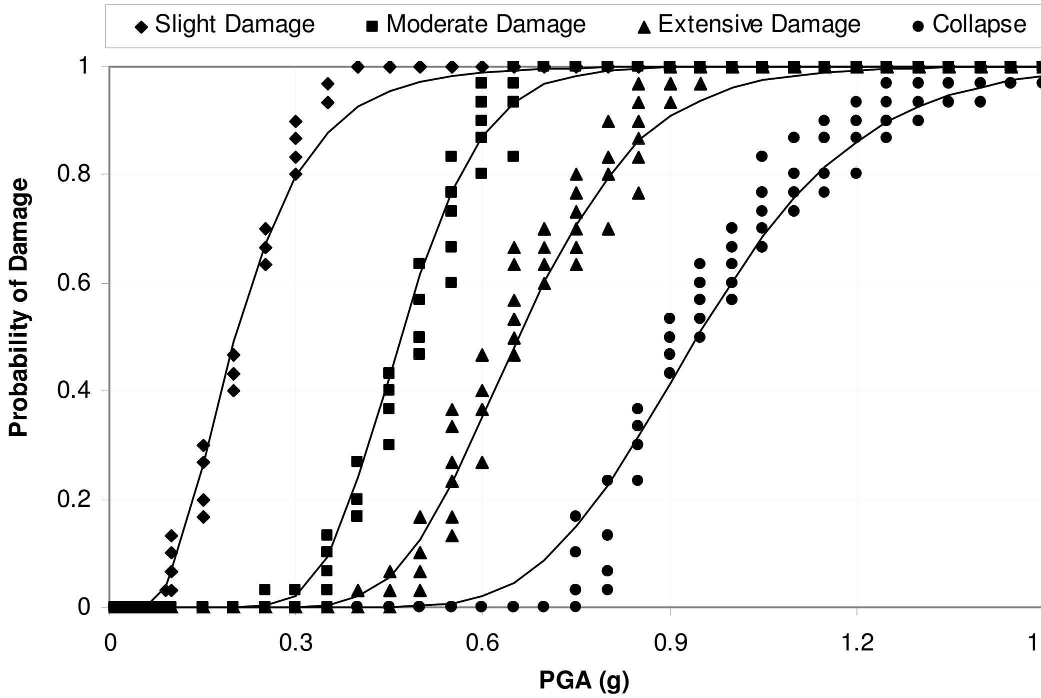


Fig. 3. Material randomized fragility curves for unretrofitted case for Satsop EQ, 2001

$$\frac{d(\ln L)}{d(c)} = \frac{d(\ln L)}{d(\zeta)} = 0 \qquad (5)$$

This computation could be complemented by using least squares method on a lognormal probability paper or by implementing straight-forward optimization algorithm. The calculated two parameters of the Equation (4) for the plotted points in Figure 3 are shown in Table 3 as follows.

Table 3. The Estimated Parameters of Lognormal Distribution for Unretrofitted Case

Damage Scales (DS)	Estimated Parameters	
	c	ξ
Slight Damage	4.95	0.47
Moderate Damage	2.14	0.26
Extensive Damage	1.52	0.24
Collapse	1.06	0.22

Using the obtained two parameters, the lognormal distribution functions can be plotted, which represent the fragility. The finalized fragility curves (only for Satsop EQ, 2001) for unretrofitted bridge columns with respect to different ground excitation (PGA) levels, with material uncertainties that concrete strength f_c' follows a mean of 43.4 MPa and standard deviation of 8.68 MPa normal distribution; steel yield strength f_y follows a mean of 393 MPa and standard deviation 43.23 MPa lognormal distribution, then are plotted as in Figure 3. From the figure, the four damage scales (slight, moderate, extensive, and collapse) were compared and then an evaluation for "as-built" structural seismic performance could be obtained.

To provide an approximate design approach, the design values of the materials were assumed in the analysis instead of assuming them to be represented as random variables. In this simplified procedure, the SDOF model was considered without material uncertainties, and the structural seismic response is based on its design material properties. Figure 4 shows this

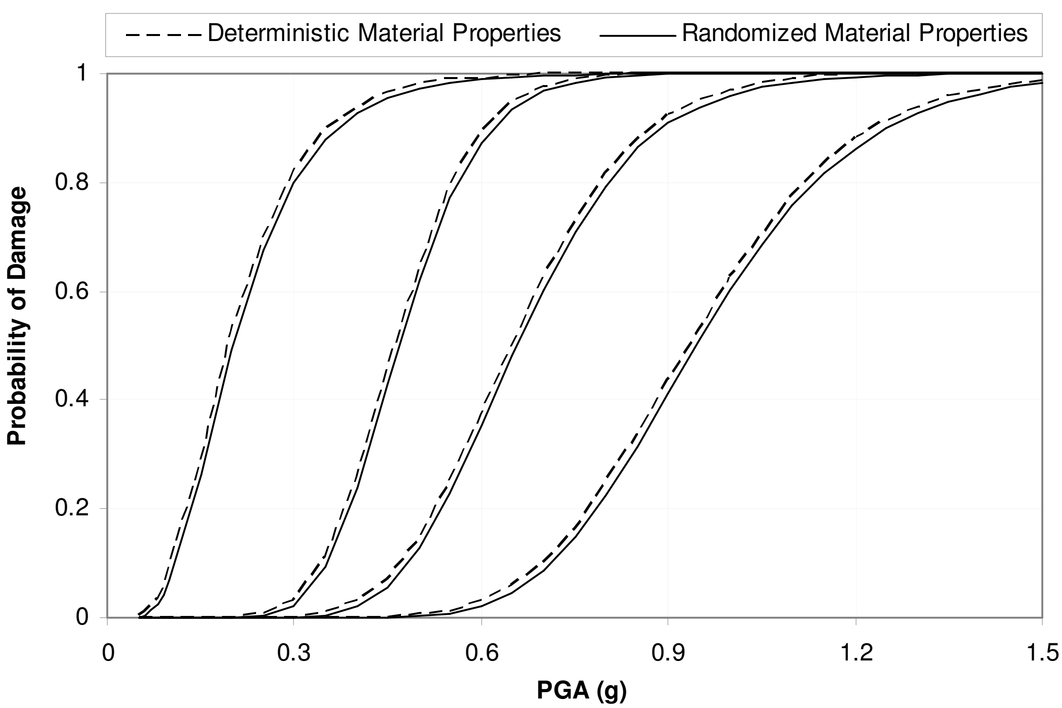


Fig. 4. Deterministic vs. randomized fragility curves

comparison. It is also acknowledged that the points' plotting indicates that 100 random number generators might be adequately stable since more than 100 times simulations would result in similar curves. These so-called "deterministic" curves are slightly conservative, which may be useful for fragility analysis procedure demonstration, or could be relatively effective considering required computational time.

FRAGILITY ANALYSIS FOR RETROFITTED CASES

Structural Model Simplification and Generalization

In the previous un-retrofitted case, the bridge column was considered as an SDOF model. For the four retrofitted cases, there were two classifications: one is full height jacketing (both steel and composite) column; the other is partial height jacketing (quarter and half-height) column. The former could still be considered as an SDOF model; the latter, however, should be appropriately represented as a multi-degree-of-freedom (MDOF) model. To simplify the computational procedure, the partial height jacketing column (MDOF) was considered as a generalized SDOF model. That is, the column displacement field was represented by a single shape function $\psi(z)$, and the main material properties generalized damping, stiffness, and mass were expressed as follows:

$$D^* = \int_0^h c(z)[\psi(z)]^2 dz + \sum_i c_i [\psi(h_i)]^2 \tag{6}$$

$$K^* = \int_0^h EI(z)[\psi''(z)]^2 dz + \int_0^h k(z)[\psi(z)]^2 dz + \sum_j k_j [\psi(h_j)]^2 \tag{7}$$

$$M^* = \int_0^h m(z)[\psi(z)]dz + \sum_k m_k \psi(h_k) \tag{8}$$

where $c(z)$, $EI(z)$, and $m(z)$ are distributed damping, stiffness, and mass, respectively; c_i,

$k(z)$, and m_k are localized damping, rigidity, and mass, respectively; h is column height, h_j is the height of the localized center. The retrofitted (partial height jacketing) structure then was transferred to an SDOF model with generalized material properties.

Increased Structural Properties

It is worth noting that for retrofitted cases, especially for jacketing in this research, fragility analysis follows the similar computational procedure as that of the unretrofitted case. The difference is simply that the retrofitted structural properties change appropriately to some specific degree. Literature indicates that steel or composite jacketing will increase the structural properties (i.e., mass and stiffness) and hence can highly improve structural seismic performance. Experiments on circular flexural columns indicate that the increase in lateral stiffness was about 10-15% for a jacket length of twice the column diameter, and extension of the steel jacket to full height of the column was often not necessary for flexural retrofit (Chai et al., 1991). Xiao, Priestley, and Seible (1994) mentioned that the effect of lateral stiffening by the steel jacket might become more pronounced when the jacket length was extended to the full height of the column, which may increase the stiffness of a column by over 30%. In this study, full-height steel jacketing was analyzed as comparison to partial height steel jacketing or full-height composite jacketing. Tests for composite jacketing compared behavior to the "as-built" model column, and indicated that there was essentially no initial stiffness increase in the retrofitted columns (Xiao et al. 1999). To simplify the retrofit cases, four specific types of jacketing were considered in this study: (1) quarter-height steel jacketing, (2) half-height steel jacketing, (3) full-height steel jacketing, and (4) full-height composite jacketing. An appropriate assumption considering model generalization was implemented that both quarter and half height steel jacketing belongs to partial height jacketing, in which the stiffness increases were assumed to be 4% and 12%, respectively. Similarly, for the full-height steel jacketing, the stiffness increase was assumed to be 30%, and the full-height composite jacketing would have 1% stiffness increase. The other adjusted structural properties, such as yield and ultimate strength, came from acquired experimental data, and the geometrical properties like mass and dimensions were obtained from mathematic calculation and the previous generalization.

Considering additional material uncertainties, that is, the steel jackets were using A40 plate steel, which were assumed to follow a lognormal distribution with a mean value of 283 MPa (41 ksi) and a COV value of 11%, the increased structural properties (input parameters of the model) of the retrofitted cases were shown in Table 4. Note that using 11.1 mm (7/16 in.) thick A40 plate steel casing with 6.4 mm (1/4 in.) thick grout infill for the steel jacketing; 3-layer (each layer thickness 3.2 mm or 1/8 in.), 9.6 mm (3/8 in.) thick prefabricated glass fiber composite wrapping with adequate adhesive for the composite jacketing.

Table 4. Increased Structural Properties for Retrofitted Cases

Structural Properties	Unretrofitted Case	Quarter-Height Steel Jacketing	Half-Height Steel Jacketing	Full-Height Steel Jacketing	Full-Height Composite Jacketing
k, k^*	164 kips/in	+ 3.2%	+ 10.9%	+ 30%	+ 1%
m, m^*	4.272 k-sec^2/in	+ 0.18%	+ 0.44%	+ 0.93%	+ 0.25%
c, c^*	2.647 k-sec/in	+ 1.4%	+ 3.7%	+ 9.6%	+ 0.13%
f_y	143.5 kips	+ 67%	+ 129%	+ 140%	+ 11%
δ_y	0.875 in	+ 58%	+ 114%	+ 120%	+ 24%
δ_u	5.25 in	+ 21%	+ 66%	+ 47%	+ 153%
T	1.104 sec	- 1.1%	- 3.1%	- 7.9%	- 0.13%

Fragility Curves for Retrofitted Cases

Following a similar analysis and computational procedure as the fragility analysis for un-retrofitted case, with the structural input parameters from Table 4, the two calculated parameters of the lognormal distribution for each retrofitted case are shown in Table 5. Then, the finalized fragility curves for retrofitted cases are plotted in Figure 5.

Table 5. The Parameters of Lognormal Distribution for Retrofitted Cases

Damage Scales (DS)	Slight Damage		Moderate Damage		Extensive Damage		Collapse	
Parameters for Lognormal Distribution	c	ζ	c	ζ	c	ζ	c	ζ
Quarter-Height Steel Jacketing	3.90	0.40	1.80	0.21	1.43	0.20	1.05	0.21
Half-Height Steel Jacketing	3.22	0.32	1.72	0.18	1.28	0.20	0.96	0.17
Full-Height Steel Jacketing	2.53	0.26	1.49	0.16	1.16	0.18	0.89	0.15
Full-Height Composite Jacketing	2.25	0.23	1.39	0.15	1.09	0.17	0.84	0.14

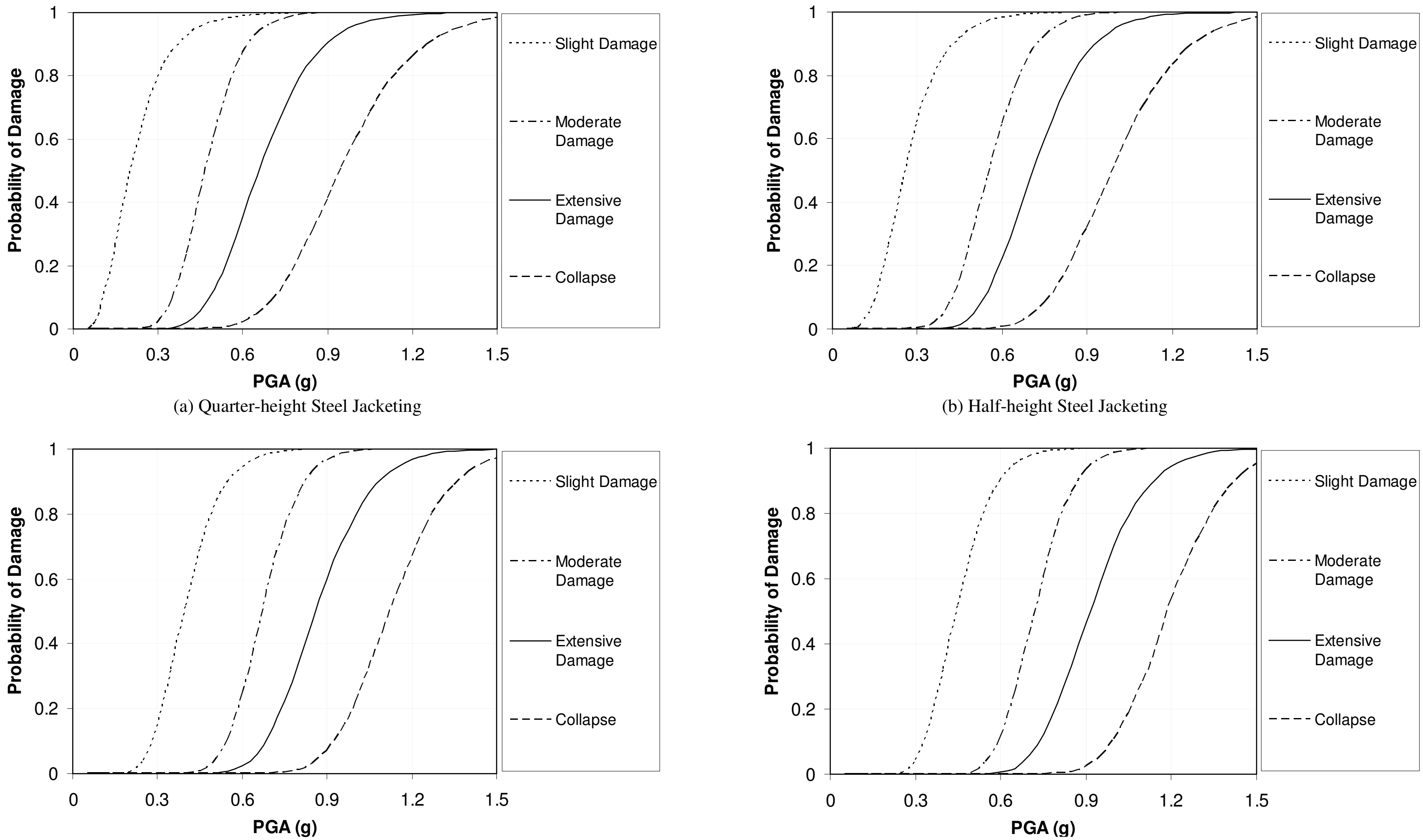
SEISMIC RETROFIT STRATEGIES ASSESSMENT

Seismic Retrofit Methods Comparison

From the fragility curves obtained in Figure 5, various retrofit methods, as well as the previous unretrofitted case, could be compared. The comparison is based on different damage levels, i.e., slight damage, moderate, extensive damage, and collapse. For each damage level, there are four retrofitted cases (quarter-height steel jacketing, half-height steel jacketing, full-height steel jacketing, and full-height composite jacketing) and one unretrofitted case. Figure 6 shows this comparison with respect to PGA.

Design PGA and Acceptable Probability of Damage

The United States Geological Survey (USGS) presented various seismic hazard maps throughout the United States. One of them is the National Seismic Hazard Mapping Project. The project produces US national maps showing earthquake ground motions that have a specified probability of being exceeded in 50 years. Although these ground motion values (PGA or PGV) can be used to assess relative hazard between sites, it is primarily adopted for reference in structural design for earthquake resistance. In this research, design PGA values were



(a) Quarter-height Steel Jacketing

(b) Half-height Steel Jacketing

(c) Full-height Steel Jacketing

(d) Full-height Composite Jacketing

Fig. 5. Fragility curves for retrofitted cases

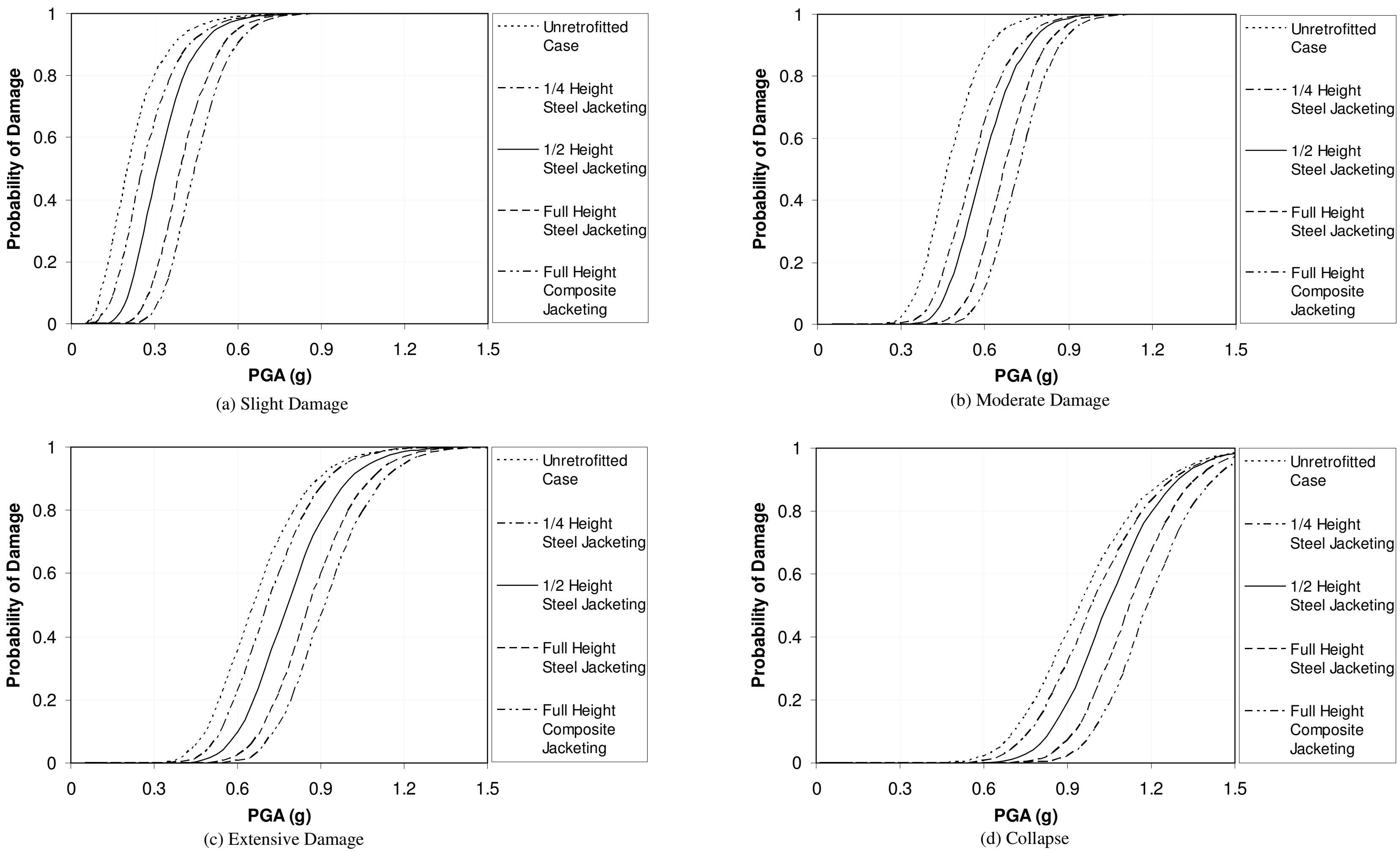


Fig. 6. Fragility-based comparison of various retrofit methods for different damage levels

conservatively assumed with regard to sites or stations from the seismic hazard maps. A design PGA values 0.3 g for structural slight damage level, 0.5 g for moderate damage level, 0.6 g for extensive damage level, and 0.7 g for collapse damage level were assumed to appropriately evaluate the structural seismic performance.

An acceptable probability of damage is an important index to express structural performance demand. Acceptable probability of damage varies a considerable range since it depends on structural type and demand, site location, and importance level of structure. It represents the allowable or maximum level of probability with respect to specified damage level. For the purpose of this research, various acceptable probability of damage for the highway bridge columns was assumed for each damage level. Additionally, an assumption was made that the target probability of failure is 0.02 for all the limit states.

Fragility-Based Seismic Retrofit Methods Evaluation

Evaluation Regarding Target Probability of Failure

As it was said previously, a target probability of failure was assumed to be 2% (or 0.02) based on typical structural reliability analysis. This target probability of failure could be used to determine the maximum acceptable PGA values with respect to various seismic hazard levels, and also, various retrofit methods. Figure 7 demonstrates this determination. It can be seen that in Figure 7, the horizontal dash above *x*-axis (PGA) represents the target probability of failure, which is constantly 0.02 for all cases. The intersections of the dash and all the fragility curves are different limit states. Hence the maximum acceptable PGA values for each case can be obtained by observing the horizontal coordinates of all the intersections. Table 6 shows this observation. Note that the term "seismic hazard level" is not identical with term "structural damage level," both of them, however, have some correlations and responsibility.

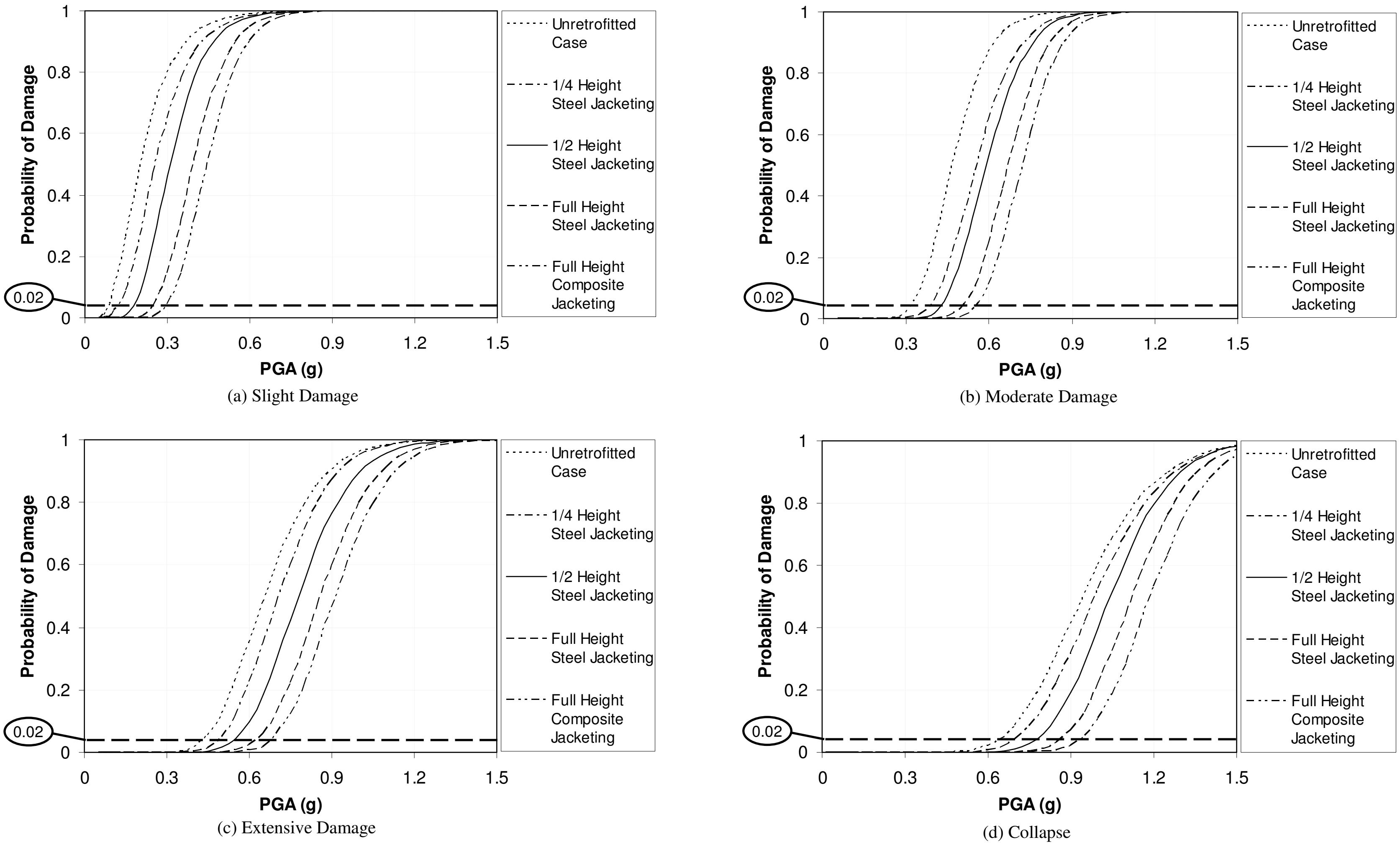


Fig. 7. Maximum acceptable PGA values based on target probability of failure

Table 6. Maximum Acceptable PGA Valuesunder Target Probability (0.02) of Failure

Seismic Hazard Levels	50% / 50 Year	20% / 50 Year	10% / 50 Year	2% / 50 Year
Unretrofitted Case	0.08 g	0.32 g	0.42 g	0.65 g
1/4-Height Steel Jacketing	0.12 g	0.39 g	0.49 g	0.69 g
1/2-Height Steel Jacketing	0.18 g	0.44 g	0.54 g	0.78 g
Full- Height Steel Jacketing	0.24 g	0.51 g	0.62 g	0.86 g
Full-Height Composite Jacketing	0.29 g	0.56 g	0.68 g	0.93 g

Evaluation Regarding Design PGA or Acceptable Probability of Damage

The evaluation shown in Figure 8 focuses on design PGA values, various probabilities of damage hence can be obtained from the fragility curves. In Figure 8(a), the vertical dash represents the assumed design PGA, which is 0.3 g for slight damage level. The vertical coordinates of the intersections of the dash and the fragility curves are probability of damage for each retrofit method. It can be seen that with design PGA value is 0.3 g, the approximate probability of slight damage for unretrofitted case is 0.8; for quarter-height steel jacketing is 0.65; for half-height steel jacketing is 0.45; for full-height steel jacketing is 0.15; and for full-height composite jacketing is 0.05. Similarly, Figure 8(b)/(c)/(d) show the assessment for moderate damage, extensive damage, and collapse, respectively. Figure 9 is another evaluation based on assumed acceptable probability of damage. In this figure, acceptable probability of damage is determined (assumed) initially, and then each maximum acceptable probability of damage is obtained from the figure. In Figure 9(a), the acceptable probability of slight damage was assumed to be 0.6, similar to target probability of failure, the horizontal dash represents the acceptance level (or limit state), and each horizontal coordinate of the intersections is the maximum acceptable PGA values for each specified retrofit methods (and, unretrofitted case).

It can be seen that in Figure 9(a), the maximum acceptable PGA for unretrofitted case is 0.23 g; for quarter-height steel jacketing is 0.28 g; for half-height steel jacketing is 0.34 g; for

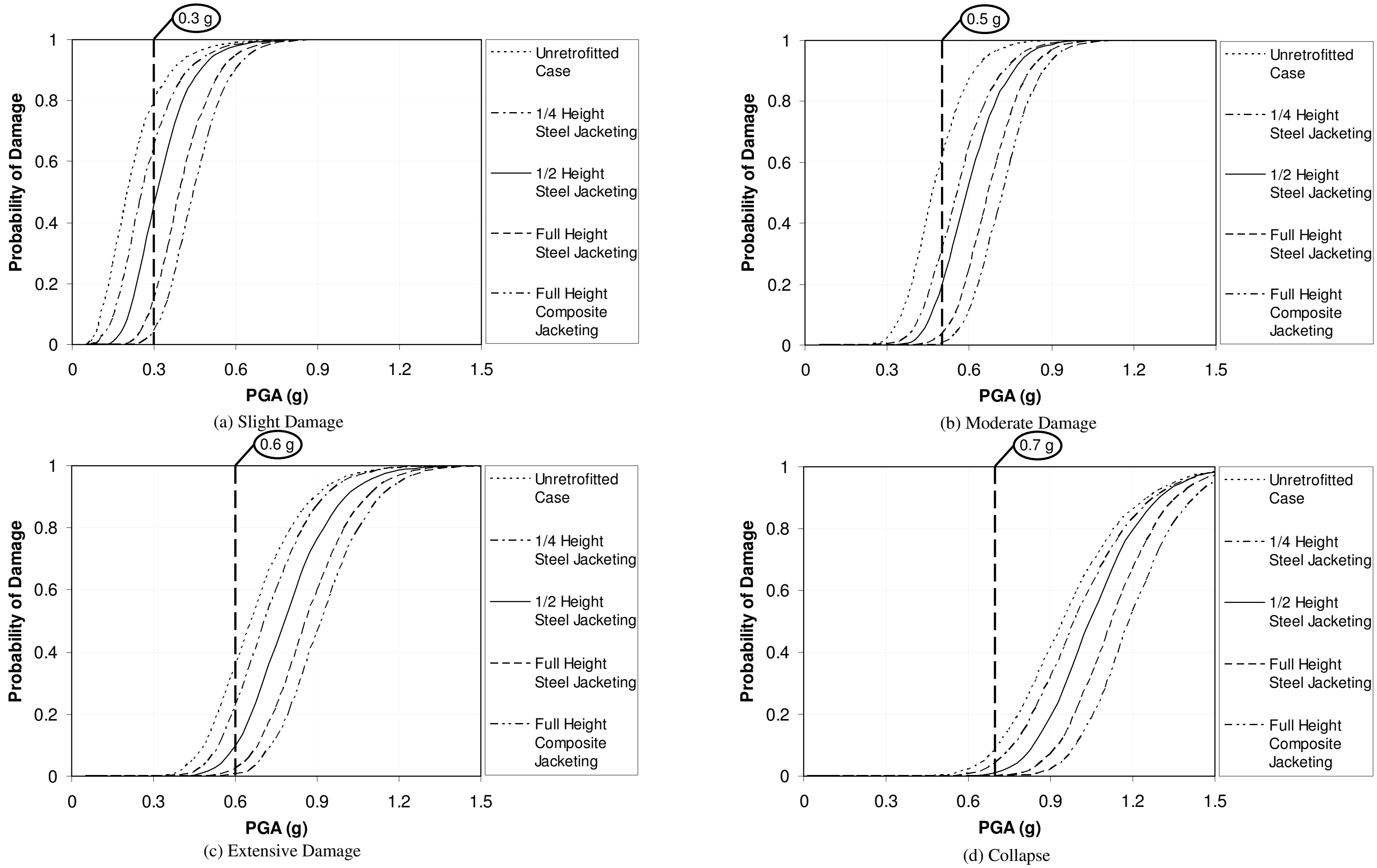
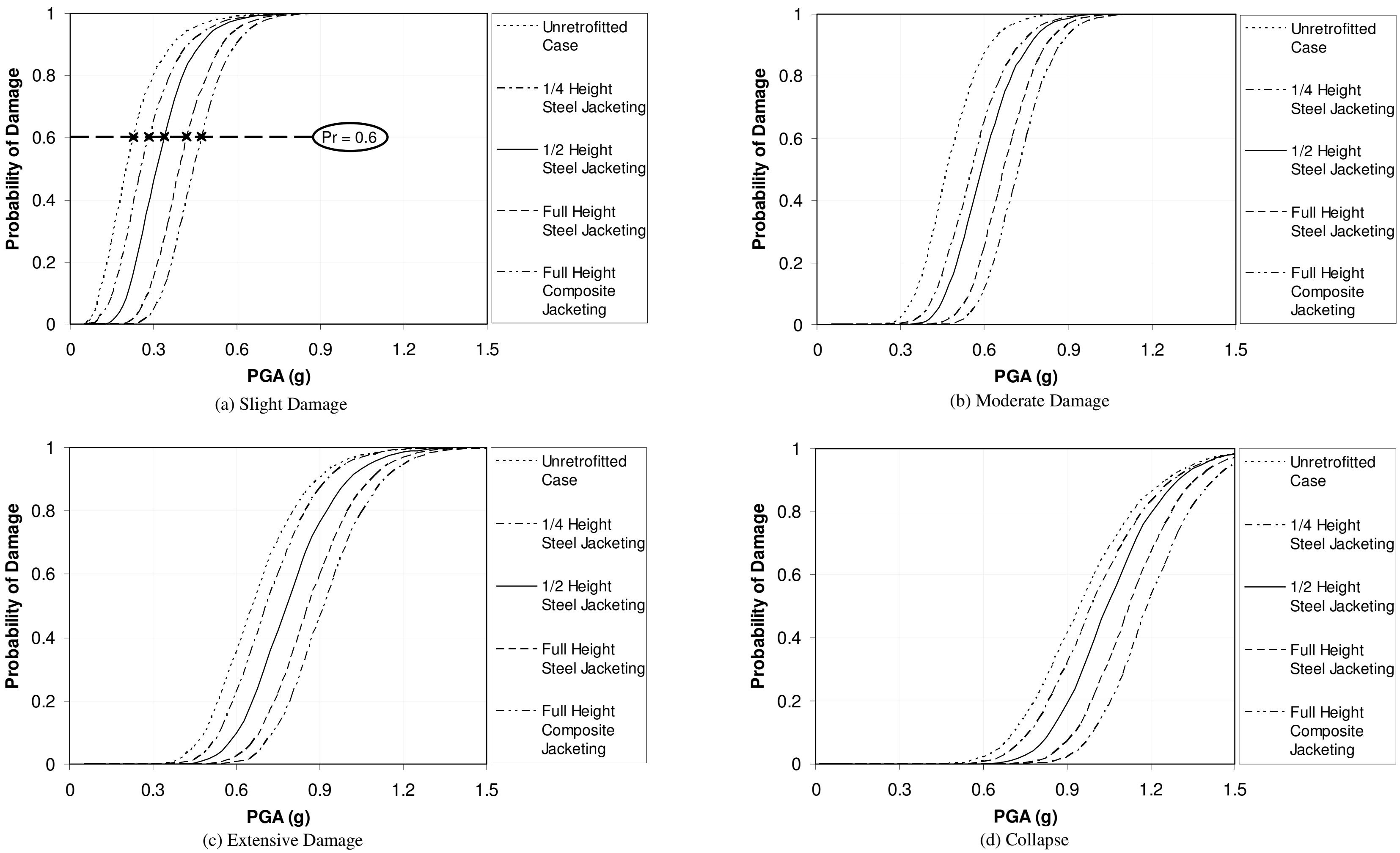


Fig. 8. Probability of damage based on design PGA values



(a) Slight Damage

(b) Moderate Damage

(c) Extensive Damage

(d) Collapse

Fig. 9. Maximum acceptable PGA values based on assumed probability of damage

full-height steel jacketing is 0.42 g; and for full-height composite jacketing is 0.47 g. This indicates that under this acceptable probability of damage (0.6), if the ground motion shaking intensity is greater than 0.23 g (PGA > 0.23 g), the "as-built" column will be in failure; similarly, if PGA > 0.28, 0.34, 0.42, and 0.47 g, the quarter-height steel jacketed, half-height steel jacketed, full-height steel jacketed, and full-height composite jacketed column will be in failure, respectively.

Retrofit Strategies Comparison and Optimal Selection

In Figure 10, five extensive-damage-level fragility curves were plotted and compared, and the horizontal straight line marked by arrow represents acceptable probability of damage (it was only assumed values in this research), and the vertical straight line marked by the arrow represents design PGA. It can be concluded that the intersection (point) of the two lines represents the limit state that satisfies both design PGA and acceptable probability of damage.

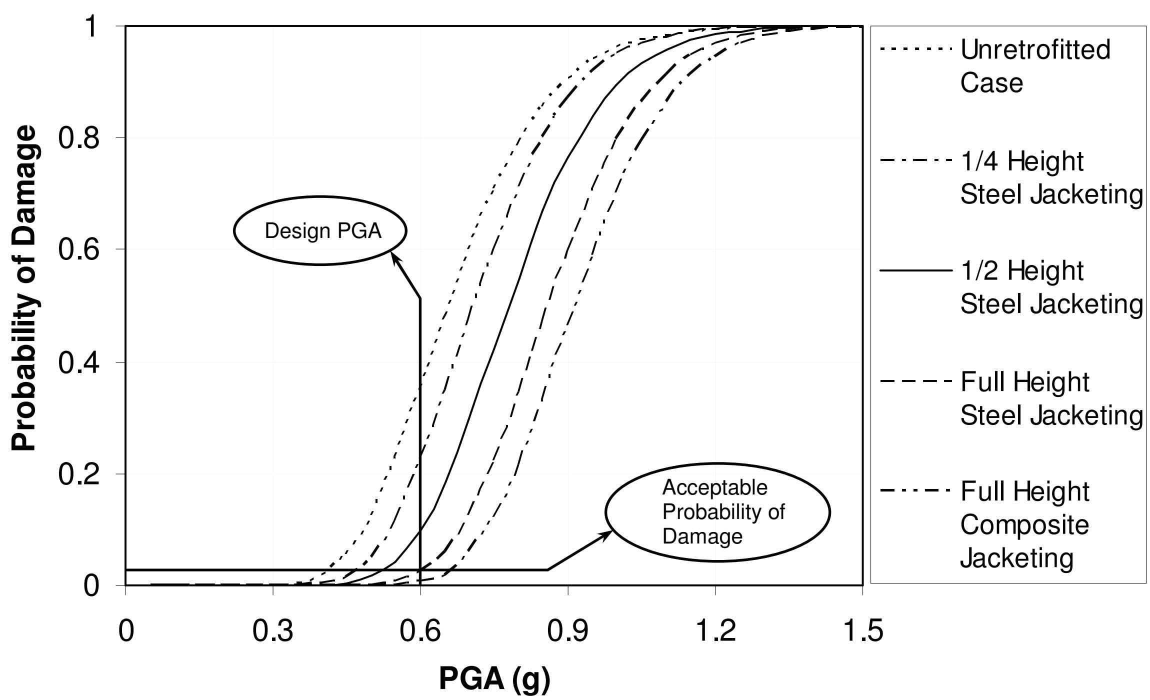


Fig. 10. Seismic retrofit methods evaluation and optimal selection

Considering the five cases (one original and four retrofitted), if some of the cases are above this point, then they are not adequate to satisfy the structural demand. For example, an assumption with design PGA value is 0.6 g was made, in which the probability of extensive damage for un-retrofitted case is 0.36, for quarter-height steel jacketing is 0.37, for half-height steel jacketing is 0.10, for full-height steel jacketing is 0.02, for full-height composite jacketing is 0.009, approximately. Another assumption, with 0.02 allowable probability of extensive damage for the bridge column, was also made. Then it can be seen from Figure 10 that the first three cases, un-retrofitted, quarter-height steel jacketing, half-height steel jacketing, are above the intersection point. Only full-height steel/composite jacketing are below (or on) the point. Hence a conclusion might be drawn that the former three cases were not adequate for structural demand, while full-height steel and composite jacketing could perform well during seismic hazard. If the design PGA and/or acceptable probability of damage were changed, the allowable choices would be changed. For example, design PGA is 0.4 g, then the half-height steel jacketing could be also one of the selections; Compared to the above two assumed cases, if the acceptance level is 0.005 because of some crucial cases (e.g., nuclear plant or hospital), and the design PGA 0.7, full-height composite jacketing might be the only choice.

In case that there are several retrofit methods that meet the structural requirements, to optimally select the best choice, other factors like economy should be considered. Typically the best selection from these adequate methods might be one curve that is the closest to the intersection point.

CONCLUSIONS

In total, a suite of twenty fragility curves is obtained in this study. By examining all the curves, as well as seismic design maps, the various retrofit methods can be compared and evaluated. This research indicates that the fragility analysis approach may be used to rationally and systematically evaluate, compare, and select optimal retrofit strategies based on target design performance levels.

LITERATURE CITED

Chai, Y. H., (1991). "Steel jacketing of circular reinforced concrete bridge columns for enhanced flexural performance." Ph.D. thesis, University of California, San Diego, California.

Chai, Y. H., Priestley, M. J. N., & Seible, F. (1991a). "Flexural retrofit of circular reinforced concrete bridge columns by steel jacketing—experimental studies." *Structural System Research Project*, *Report No. SSRP-91/06*, Dept. of Applied Mechanics and Engineering Science, University of California, San Diego, California.

Chai, Y. H., Priestley, M. J. N., & Seible, F. (1991b). "Flexural retrofit of circular reinforced concrete bridge columns by steel jacketing—COLRET—a computer program for strength and ductility calculation." *Structural System Research Project*, *Report No. SSRP-91/07*, Dept. of Applied Mechanics and Engineering Science, University of California, San Diego, California.

Chai, Y. H., Priestley, M. J. N., & Seible, F. (1991c). "Seismic retrofit of circular bridge columns for enhancing flexural performance." *ACI Structural Journal*, *88*(5), 572-584.

Cooper, J. D., Friedland, I. M., Buckle, I. G., Nimis, R. B., & Bobb, N. M. (1994). "The Northridge earthquake: Progress made, lessons learned in seismic resistant bridge design." *Public Roads*, *58*(1), 27-36.

Ellingwood, B., R., Galambos, T. V., MacGregor, J. G., & Cornell, C. A. (1980). "Development of a probability based load criterion for American National Standard A58." *NBS Special Publication 577*, Washington D. C., National Bureau of Standards.

Ghobarah, A., Aly, N. M., & El-Attar, M. (1997). "Performance level criteria and evaluation." *Proc.*, *International Workshop on Seismic Design Methodologies for the next Generation of Codes*, Bled, Slovenia, 207-215.

Guzman, P. D., & Ishiyama, Y. (2003). "Damage index as a criterion in evaluating seismic performance of building structures." *Proc.*, *2003 Annual Meeting of Architectural Institute of Japan*, Nagoya, Japan.

Karim, K. R., & Yamazaki, F. (2000). "Comparison of empirical and analytical fragility curves for RC bridge piers in Japan." *Proc.*, *8th ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability*, Notre Dame, Indiana.

Kim, S.-H. & Shinozuka, M. (2003). "Development of fragility curves of bridges retrofitted by column jacketing", *Journal of Probabilistic Engineering Mechanics*, special Issue dedicated to *4th International Conference on Computational Stochastic Mechanics*.

Matsuda, T., Sato, H., Fujiwara, H., & Higashira, N. (1990). "Effect of carbon fiber reinforcement as a strengthening measure for reinforced concrete bridge piers." *Proc.*, *1st U.S.–Japan Workshop on Seismic Retrofit of Bridges*, Public Works Research Institute, Ministry of Construction, Tsukuba Science City, Japan.

Park, Y. J., & Ang, A. H-S. (1985). "Seismic damage analysis of reinforced concrete buildings." *Journal of Structural Engineering*, *ASCE*, *111*(4), 740-757.

Priestley, M. J. N., (1987). "Damage of the I-5/I605 separator in the Whittier earthquake of October 1987." *Earthquake Spectra*, *4*(2), 389-405.

Priestley, M. J. N., & Seible, F. (1991). "Seismic assessment and retrofit of bridges." *Structural System Research Project*, *Report No*. *SSRP-91/03*, M. J. N. Priestley and F. Seible, University of California, San Diego, California.

Priestley, M. J. N., Seible, F., Xiao, Y., & Verma, R. (1994a). "Steel jacket retrofitting of reinforced concrete bridge columns for enhanced shear strength—Part 1: Theoretical consideration and test design." *ACI Structural Journal*, *91*(4), 394-405.

Priestley, M. J. N., Seible, F., Xiao, Y., & Verma, R. (1994b). "Steel jacket retrofitting of reinforced concrete bridge columns for enhanced shear strength—Part 2: Experimental result." *ACI Structural Journal*, *91*(5), 537-551.

Priestley, M. J. N., Verma, R., & Xiao, Y. (1994c). "Seismic shear strength of reinforced concrete columns." *Journal of Structural Engineering*, *ASCE*, *120*(8), 2310-2329.

Rossetto, T., & Elnashai, A. (2001). "Derivation of vulnerability functions for RC buildings based on observation data." *Safety Assessment for Earthquake Risk Reduction (SAFERR) Research Training Network*, European Commission, Landon, U.K.

Saadatmanesh, H., Ehsani, M. R., & Li, M. W. (1994). "Strength and ductility of concrete columns externally reinforced with fiber composite straps." *ACI Structural Journal*, *91*(4), 434-444.

Seible, F., Hegemier, G. A., & Innamorato, D. (1995). "Developments in bridge column jacketing using advance composites." *Proc*., *National Seismic Conference on Bridges* *and Hwy*., Federal Highway Administration and California department of Transportation, San Diego, California.

Shinozuka, M., Feng, M. Q., Lee, J., & Naganuma, T. (2000). "Statistical analysis of fragility curves." *Journal of Engineering Mechanics*, *ASCE*, *126*(12), 1224-1231.

Tantala, M. W., & Deodatis, G. (2002). "Development of seismic fragility curves for tall buildings." *Proc*., *15th ASCE Engineering Mechanics Conference*, Columbia University, New York, NY.

Xiao, Y., Martin, G. R., Yin, Z., & Ma, R. (1995). "Bridge column retrofit using snap-tite composite jacketing for improved seismic performance." *Structural Engineering Research Program*, University of Southern California, Los Angeles, California.

Xiao, Y., Martin, G. R., Yin, Z., & Ma, R. (1996). "Seismic retrofit of existing reinforced concrete bridge columns using a prefabricated composite jacketing system." *Proc*., *1st International Conference on Composite in Infrastructure*, University of Arizona at Tucson, Tucson, Arizona, 903-916.

Xiao, Y., & Ma, R. (1997). "Seismic retrofit of RC circular columns using prefabricated complete jacketing." *Journal of Structural Engineering*, *ASCE*, *123*(10), 1357-1364.

Xiao, Y., Wu, H., & Martin, G. R. (1999). "Prefabricated composite jacketing of RC columns for enhanced shear strength." *Journal of Structural Engineering*, *ASC*E, *125*(3), 255-264.

Yamazaki, F., Hamada, T., Motoyama, M., & Yamauchi, H. (1999). "Earthquake damage assessment of expressway bridges in Japan." *Optimizing Post-Earthquake Lifeline System Reliability*, *ASCE Proceedings of 5th U.S. Conference on Lifeline Earthquake Engineering*, Seattle, Washington, 361-370.

Zhu, X., & Ni, Y. (2001). "Strength reduction factor spectra based on structural damage performance." *Proc., Earthquake Engineering Frontiers in the New Millennium, China-US Millennium Symposium on Earthquake Engineering*. Peking, P. R. China