13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1163

IMPACT OF RETROFIT ON THE SEISMIC FRAGILITY OF A REINFORCED CONCRETE STRUCTURE

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SUMMARY

An assessment of seismic fragility was conducted for a five-story reinforced concrete (RC) frame building representative of 1980s construction in the Mid-America region. The structural response was predicted using nonlinear response analysis and synthetic ground motions. The performance of the unretrofitted structure is presented in terms of fragility relationships that relate the probability of exceeding a performance level to the earthquake intensity. Fragility relationships for several possible intervention techniques and performance levels are compared to those for the unretrofitted structure. A reduction in the seismic fragility is demonstrated through the use of shear walls and RC column jackets.

INTRODUCTION

Earthquakes are of concern to cities in the Central United States (U.S.) because of the history of seismic activity around the New Madrid Seismic Zone (NMSZ). Three major earthquakes took place during the winter of 1811-1812 with body-wave magnitudes of 7.35, 7.2, and 7.5. The epicentral locations for these earthquakes are near New Madrid, Missouri and are the center of the NMSZ. This study focuses on predicting the expected seismic performance of a reinforced concrete (RC) building in the Central U.S. characteristic of office buildings constructed in that area during the mid-1980s. The seismic performance of the unretrofitted structure is quantified in terms of fragility relationships that relate the probability of exceeding a particular performance level to the earthquake intensity. The seismic fragility relationships for various intervention techniques are then developed and compared to the fragility relationships for the unretrofitted structure. This study is part of a Mid-America Earthquake (MAE) Center research program focusing on consequence minimization, which contributes to the development of a new Consequence Based Engineering paradigm. The findings of this study provide information about the expected seismic performance of a common type of structure in Mid-America, as well as the potential to minimize the expected damage for varying earthquake intensities through retrofit.

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DESCRIPTION OF CASE STUDY BUILDING

A case study building was designed according to the codes used in St. Louis, Missouri during the early 1980s, prior to St. Louis' assignment to Seismic Zone 2 of the *Building Officials and Code Administrators (BOCA) Basic/National Code 1987* (BOCA [1]). Several engineers with design experience in the St. Louis area provided information for use in selecting a prototype structure by responding to questionnaires (Hart [2]). The five-story RC case study building has a moment frame system not specially detailed for ductile behavior. The floor system is composed of a flat slab and perimeter moment resisting frames with spandrel beams. Figures 1 and 2 provide a typical floor plan and elevation view of the case study structure. Design load requirements were taken from the ninth edition of the *BOCA Basic/National Code 1984* (BOCA [3]), in which St. Louis is considered to be in Seismic Zone 1. It should be noted that Memphis, Tennessee was also assigned to Seismic Zone 1, based on the map given in the 1984 BOCA code. The perimeter frames were designed to resist the full design lateral load based on design practices that were common and generally accepted during the 1980s. The structural member design follows the provisions of the American Concrete Institute (ACI) *Building Code Requirements for Reinforced Concrete (ACI 318-83)* (ACI Comm. 318 [4]). The material properties are a concrete compressive strength of 28 MPa and a steel reinforcement yield strength of 410 MPa.

ANALYTICAL MODEL

General

ZEUS-NL is a finite element structural analysis program developed for nonlinear dynamic, conventional and adaptive push-over, and eigenvalue analysis (Elnashai et al. [5]). The program can be used to model two-dimensional and three-dimensional steel, RC and composite structures, taking into account the effects of geometric nonlinearities and material inelasticity. The program uses the fiber element approach to model these nonlinearities, where the cross-sections are divided into fibers monitoring the confined concrete section, the unconfined concrete cover and the steel reinforcement.

Overall Building Model

A two-dimensional analytical model was used, which is adequate for the regular floor plan of the case study building (see Figure 3). The model takes advantage of the building's symmetry such that only half of the structure is analyzed. One exterior frame and two interior frames, oriented along the short direction of the building, are linked with rigid truss elements such that only lateral forces and displacements are transmitted between frames. Rigid zones were used to define the joint regions, so that inelastic behavior is monitored outside the joint. The horizontal dimension of the rigid zone within each joint was specified to be equal to the column width. The rigid zone height was set equal to the spandrel beam depth for joints around the perimeter of the building; and equal to the slab depth, not including the additional thickness due to the shear capital, for interior slab-column joints. Members were divided such that a Gauss point monitors the member section just outside the joint region. To refine this model, a second node was added near the joint at 91 cm from each column face. To produce the appropriate initial static member forces, additional nodes were used along the horizontal members for application of the self-weight as a series of equivalent point loads. The lumped mass element (Lmass) was used to include the seismic dead weight as a lumped mass at each column joint for the dynamic analysis.

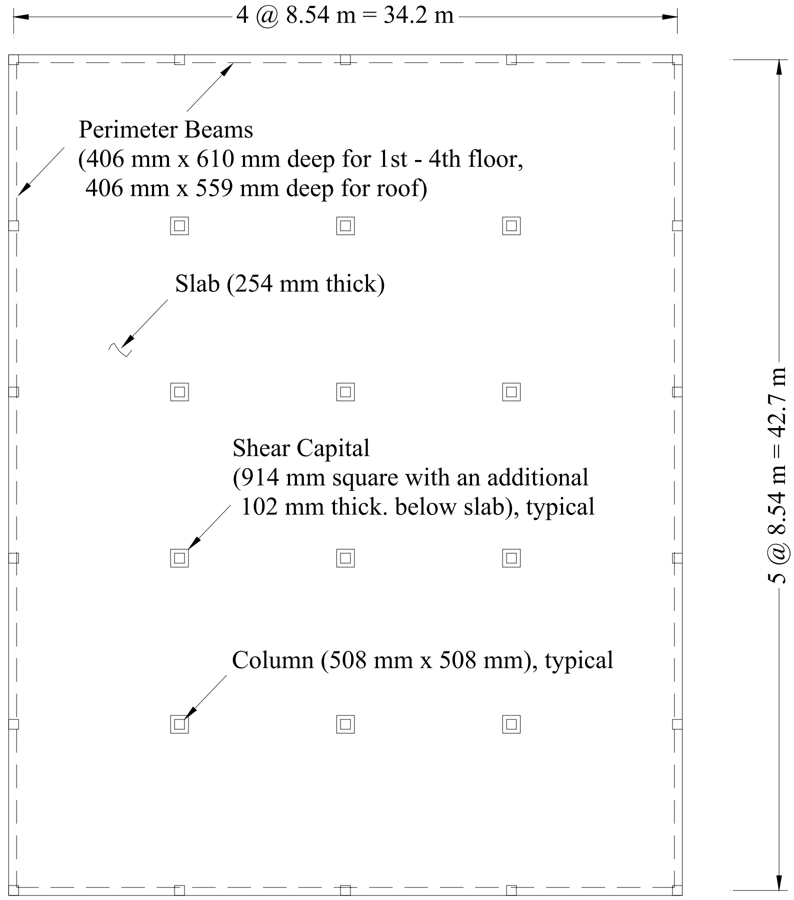


Figure 1. Plan view of case study building

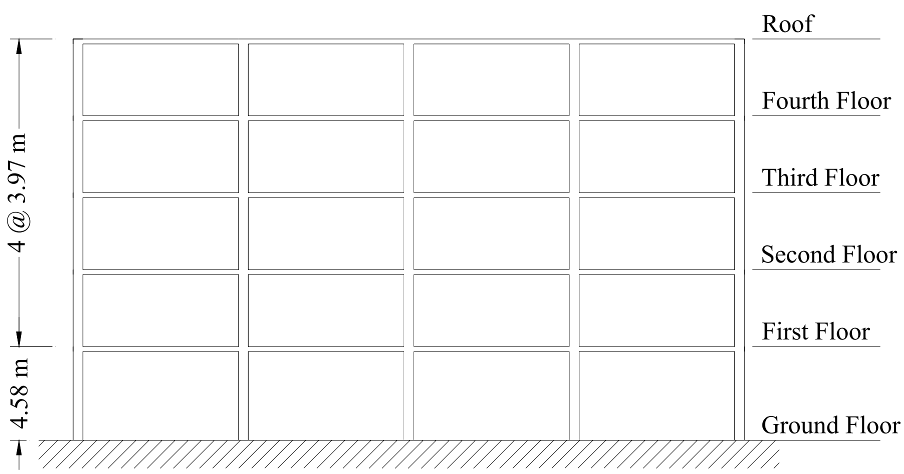


Figure 2. Elevation view of case study building

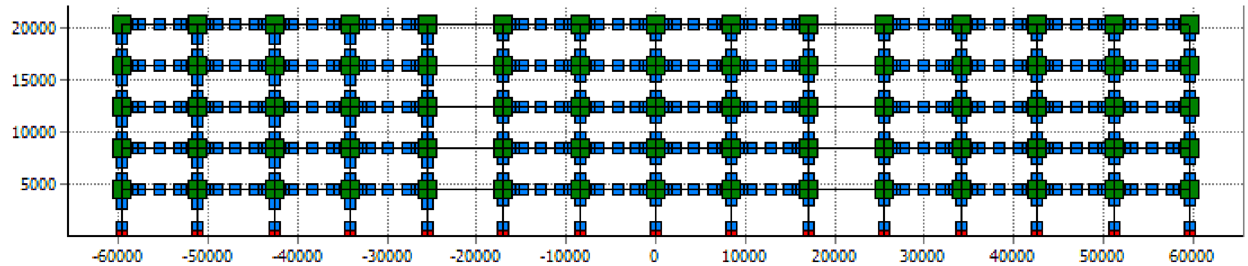


Figure 3. Model of case study building used in ZEUS-NL analysis (units in mm)

Modeling of Individual Members

A cubic elasto-plastic three-dimensional element (cubic) was used to model the columns, beams, slabs and rigid connections. The joint element with uncoupled axial, shear and moment actions (joint) was used to model the joints as rigid. It is possible to model bond-slip using the joint element, but this was not included in this analysis. The cross-sections of the column members were described using the RC rectangular section (rcrs), while the cross-sections of the beam and slab members were defined using the RC T-section (rcts). The bilinear elasto-plastic material model with kinematic strain hardening (stl1) was used for the reinforcement and rigid connections, and the uniaxial constant confinement concrete material model (conc2) was used for the concrete.

GROUND MOTION RECORDS

The ground motions used for the nonlinear time history analyses are suites of synthetic records developed by Wen and Wu [6]. Each suite contains ten ground motions whose median response (based on a lognormal distribution) corresponds to the specified return rate and location. Return rates of 475 years (10% probability of exceedance in 50 years) and 2475 years (2% probability of exceedance in 50 years) were used for St. Louis, Missouri and Memphis, Tennessee. These ground motions were based on representative soil conditions for each city. The ground motion characteristics are provided in Tables 1 and 2 and the response spectra plots are shown in Figure 4. To reduce the computation time, the ground motions were shortened for the analysis at the time point when the energy reaches 95% of the total energy imparted by the acceleration record, based on the procedure developed by Trifunac and Brady [7]. The duration of the shortened records for analysis ranged between approximately 10 to 60 seconds.

Table 1. Characteristics of St. Louis synthetic ground motions

10% in 50 years						2% in 50 years					
Record ID	PGA	Duration (s)	Body Wave Mag.	Focal Depth (km)	Epic. Distance (km)	Record ID	PGA	Duration (s)	Body Wave Mag.	Focal Depth (km)	Epic. Distance (km)
l10_01s	0.13g	41.0	6.0	2.7	76.4	l02_01s	0.23g	150	8.0	17.4	267
l10_02s	0.10g	81.9	6.9	9.3	202	l02_02s	0.25g	150	8.0	9.1	230
l10_03s	0.09g	81.9	7.2	4.4	238	l02_03s	0.83g	20.5	5.4	2.1	28.7
l10_04s	0.11g	41.0	6.3	9.8	252	l02_04s	0.25g	81.9	7.1	5.5	253
l10_05s	0.13g	41.0	5.5	2.9	123	l02_05s	0.19g	150	8.0	17.4	254
l10_06s	0.11g	41.0	6.2	7.7	208	l02_06s	0.24g	81.9	6.8	5.8	225
l10_07s	0.10g	81.9	6.9	1.7	194	l02_07s	0.24g	150	8.0	33.9	196
l10_08s	0.12g	41.0	6.2	27.6	175	l02_08s	0.24g	150	8.0	9.1	261
l10_09s	0.11g	41.0	6.2	6.5	221	l02_09s	0.25g	150	8.0	9.1	281
l10_10s	0.08g	81.9	6.9	2.7	237	l02_10s	0.54g	41.0	5.9	4.4	47.7

Table 2. Characteristics of Memphis synthetic ground motions

10% in 50 years						2% in 50 years					
Record ID	PGA	Duration (s)	Body Wave Mag.	Focal Depth (km)	Epic. Distance (km)	Record ID	PGA	Duration (s)	Body Wave Mag.	Focal Depth (km)	Epic. Distance (km)
m10_01	0.06g	41.0	6.3	5.2	121.0	m02_01	0.44g	150	8.0	25.6	147.6
m10_02	0.08g	41.0	6.4	6.7	57.5	m02_02	0.33g	150	8.0	33.9	186.1
m10_03	0.07g	41.0	6.8	18.1	125.1	m02_03	0.36g	150	8.0	25.6	163.2
m10_04	0.07g	41.0	6.8	2.1	92.4	m02_04	0.32g	150	8.0	9.1	169.6
m10_05	0.11g	41.0	6.2	27.0	107.1	m02_05	0.48g	150	8.0	9.1	97.6
m10_06	0.05g	150	6.2	3.2	41.2	m02_06	0.42g	150	8.0	17.4	117.6
m10_07	0.07g	41.0	6.5	11.5	58.8	m02_07	0.37g	150	8.0	17.4	119.2
m10_08	0.09g	20.5	6.5	23.9	129.1	m02_08	0.29g	150	8.0	9.1	145.7
m10_09	0.09g	20.5	6.3	9.5	166.4	m02_09	0.34g	150	8.0	9.1	170.5
m10_10	0.06g	41.0	6.8	8.7	35.6	m02_10	0.41g	150	8.0	17.4	187.7

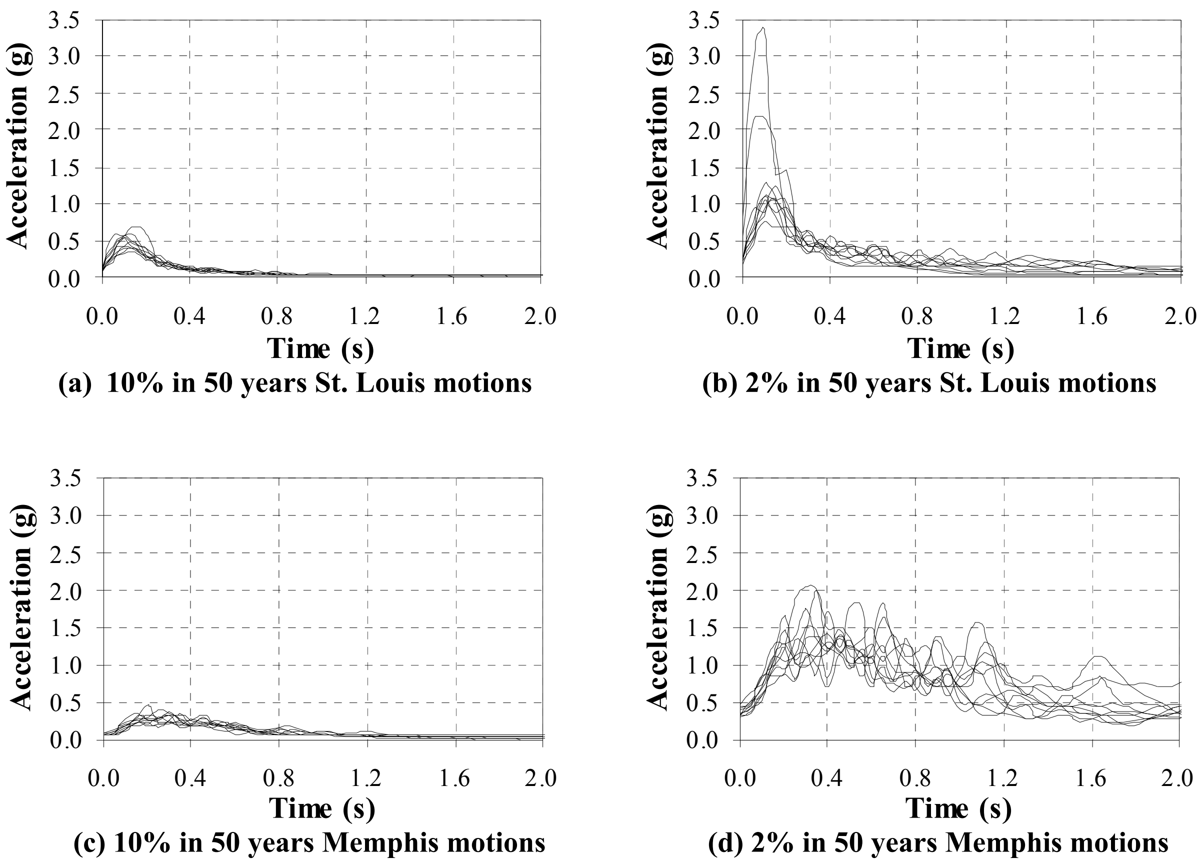


Figure 4. Response spectra for synthetic ground motion sets (two percent damping)

EVALUATION OF UNRETROFITTED BUILDING

FEMA 356 Criteria

The *Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356)* (ASCE [8]) performance criteria were used to assess the seismic performance of the case study building based on the ZEUS-NL response analysis. FEMA 356 provides analytical procedures and criteria for the performance-based evaluation of existing buildings and for designing seismic rehabilitation alternatives. Performance *levels* describe limitations on the maximum damage sustained during a ground motion, while performance *objectives* define the target performance level to be achieved for a particular intensity of ground motion. Structural performance levels in FEMA 356 include Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). Structures at CP are expected to remain standing, but with little margin against collapse. Structures at LS may have sustained significant damage, but still provide an appreciable margin against collapse. Structures at IO should have only minor damage. In FEMA 356, the *Basic Safety Objective (BSO)* is defined as LS performance for the Basic Safety Earthquake 1 (BSE-1) earthquake hazard level and CP performance for the BSE-2 earthquake hazard level. BSE-1 is defined as the *smaller* of an event corresponding to 10% probability of exceedance in 50 years (10% in 50 years) and 2/3 of BSE-2, which is the 2% probability of exceedance in 50 years (2% in 50 years) event. Both global-level (drift) limits and member-level (plastic rotation) limits are provided to assess structural performance. However, the global-level limits are intended to illustrate the overall structural response for a given performance level, while the member-level limits are intended for evaluation of specific structural components.

Global-Level Evaluation

St. Louis, Missouri

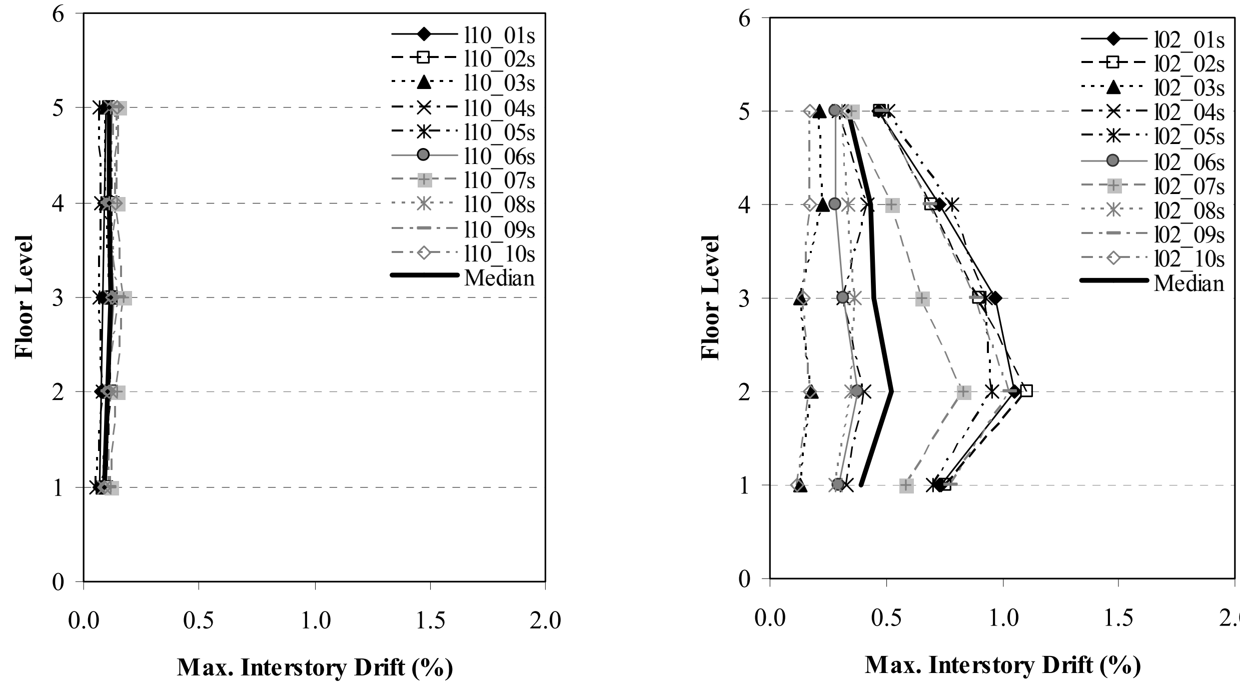
The ZEUS-NL model was used to evaluate the response of the case study building for the St. Louis ground motion records. Figure 5 provides maximum interstory drifts for all motions. For an approximate global assessment, FEMA 356 provides limiting drift values for RC frame structures as 1, 2 and 4 percent for the IO, LS and CP performance levels, respectively. The median maximum interstory drift values for both suites are below 1 percent, indicating that the structural performance is well within the BSO described in FEMA 356, based on the global response.

Memphis, Tennessee

The maximum interstory drifts from the ZEUS-NL analysis with the Memphis motions are shown in Figure 6. Based on an approximate global-level performance evaluation, the structure meets the BSO of LS for the 10% in 50 years Memphis event. In this case, the median drift values are well below the LS limit of 2 percent. For the 2% in 50 years event, the median drift values (ranging from 0.9 to 2.9 percent) are within the CP limit of 4 percent, indicating that the BSO objective for this event is also met.

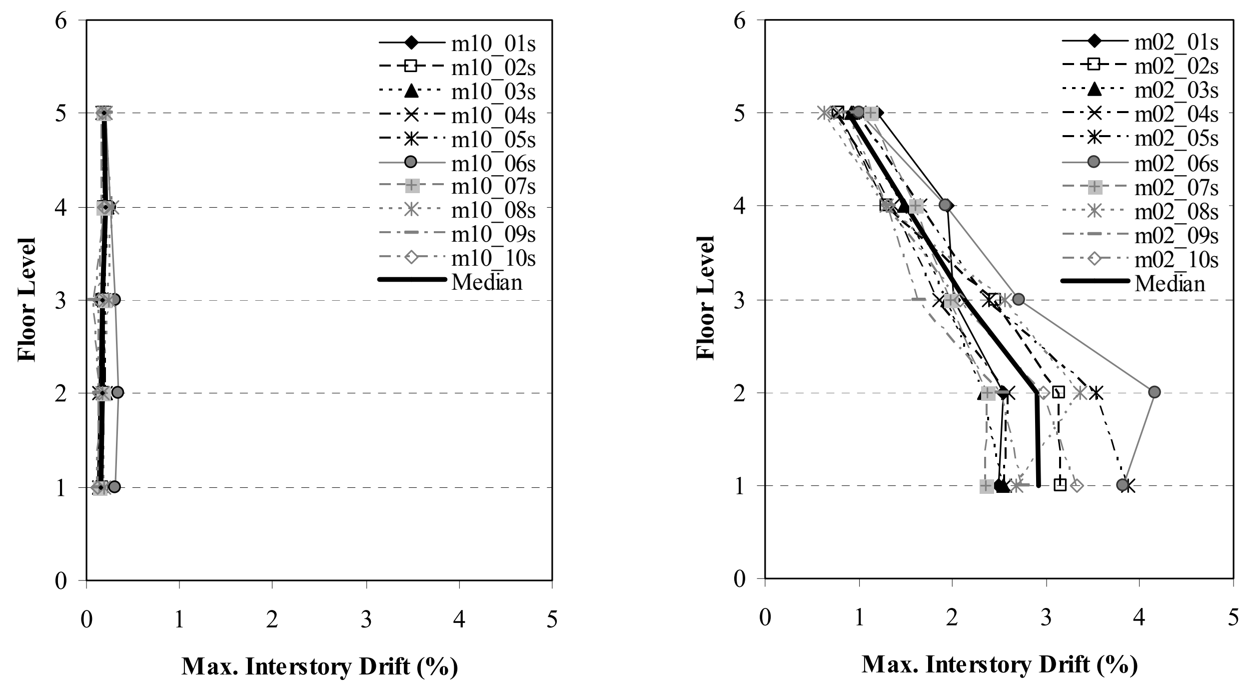
Member-Level Evaluation

FEMA 356 provides member-level criteria for three performance levels: IO, LS and CP. The applicable plastic rotation limits for the structural components of the case study building are summarized in Table 3. For the 10% in 50 years Memphis event, no plastic rotations occurred and so the BSO of LS for this recurrence interval was satisfied. The member-level performance evaluation for the 2% in 50 years Memphis event is summarized in Table 4. Note that the joints are modeled as rigid and so no plastic joint rotations are predicted. Cases where the BSO of CP for this recurrence interval is not met are noted with bold font. Limits for plastic rotation are exceeded in column, beam, and slab members. According to this evaluation, the first and second stories are the most vulnerable and significant damage is expected.



(a) 10% in 50 years **(b) 2% in 50 years**

Figure 5. Maximum interstory drift values for St. Louis motions



(a) 10% in 50 years **(b) 2% in 50 years**

Figure 6. Maximum interstory drift values for Memphis motions

Table 3. FEMA 356 Plastic Rotation Limits for the Unretrofitted Case Study Building

Story	Performance Level	Beams	Columns	Beam-Column Joints	Slabs and Slab-Column Joints
1	IO	0.00500	0.00418	0	0.00550
	LS	0.0100	0.00418	0	0.00825
	CP	0.0100	0.00518	0	0.0110
2	IO	0.00500	0.00453	0	0.00550
	LS	0.0100	0.00453	0	0.00825
	CP	0.0100	0.00553	0	0.0110
3	IO	0.00500	0.00481	0	0.00550
	LS	0.0100	0.00481	0	0.00825
	CP	0.0153	0.00581	0	0.0110
4	IO	0.00500	0.00500	0	0.00550
	LS	0.0100	0.00500	0	0.00825
	CP	0.0161	0.00600	0	0.0110
5	IO	0.00500	0.00500	0	0.000500
	LS	0.0100	0.00500	0	0.000750
	CP	0.0157	0.00600	0	0.00100

Table 4. Maximum plastic rotations for 2% in 50 years Memphis motions

Story	Median Ground Motion	Beams	Columns	Slabs
1	m02_09s	**0.0179**	**0.0286**	**0.0179**
2	m02_10s	**0.0168**	**0.0222**	**0.0127**
3	m02_10s	0.0110	**0.0175**	0.00768
4	m02_03s	0.00487	**0.0112**	0
5	m02_09s	0	0.00507	0

Shear failures are not monitored in the ZEUS-NL analysis. Additional calculations for the 2% in 50 years Memphis event indicate that the median maximum base shear does not exceed the available column shear strength. However, punching shear failures are expected at the first and second floor levels based on the gravity shear ratio and interstory drift demand, as determined from the prediction model proposed by Hueste and Wight [9].

RETROFIT STRATEGIES

Retrofit Techniques

Because of the deficiencies determined from the FEMA 356 member-level evaluation, several retrofit techniques were evaluated for the case study structure with a goal of modifying different structural response parameters. The three selected techniques include addition of shear walls, addition of RC column jackets, and confinement of the column plastic hinge regions using externally bonded steel plates. The addition of shear walls is a common seismic retrofit technique that leads to an increase in the global stiffness and strength of the structure. In this study, 410 mm thick shear walls were added to the two central bays of the exterior frame (see Figure 7). Two layers of #6 (US) reinforcing bars at 305 mm spacing were modeled in the shear wall. The selected shear wall characteristics are based on previous research by Pincheira and Jirsa [10] and meet the requirements of ACI 318-02 (ACI Comm. 318 [11]). To model the shear walls, the RC flexural wall section (rcfws) in the ZEUS-NL library was used.

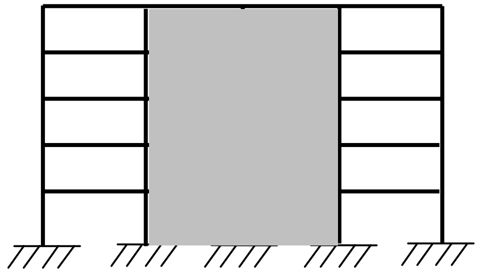


Figure 7. Retrofit 1: shear walls added to exterior frame

Based on the FEMA 356 member-level evaluation, the columns had the most deficiencies in meeting the BSO of CP for the 2% in 50 years Memphis event. Therefore, column jacketing was selected as the second seismic retrofit technique. The columns that did not satisfy the FEMA member-level criteria were retrofitted with additional RC column jackets. The locations of the jacketed members are shown in Figure 8. The dimensions and reinforcement of the jackets were determined by satisfying the LS global-level drift limit. Figure 9 shows typical details of the jacketed columns. The RC jacket rectangular section (rcjrs) in ZEUS-NL was used to model the jacketed members.

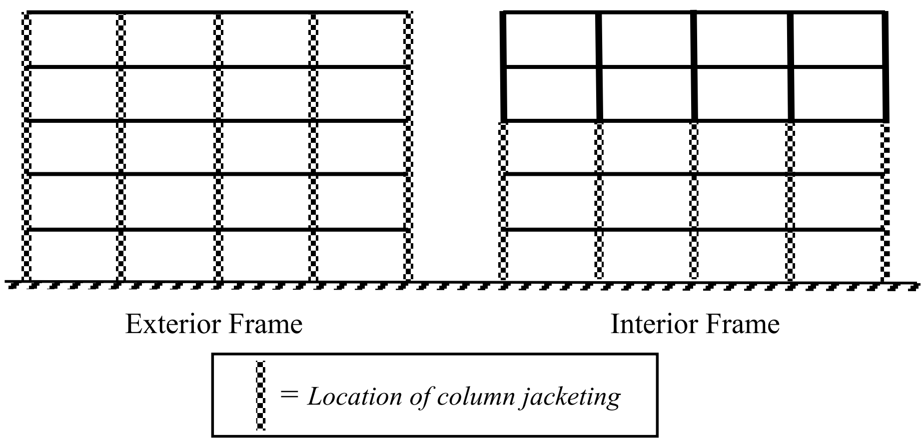
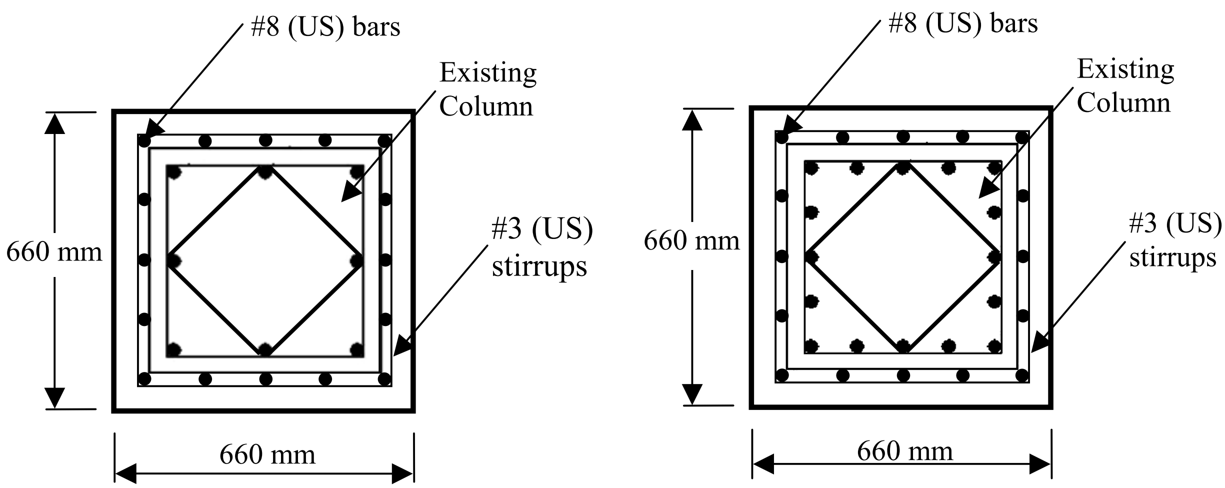


Figure 8. Retrofit 2: addition of RC column jackets



(a) 1st - 5th Story for Exterior Frame
2nd - 3rd Story for Interior Frame

(b) 1st Story for Interior Frame

Figure 9. Cross-sectional details of RC column jacket retrofit

The third retrofit technique is the addition of external steel plates to confine the plastic hinge zones at the column ends to increase ductility. To model the confinement of the column plastic hinge zones, the confinement factor (*k*), developed by Mander et al. [12] for rectangular concrete sections with axial compression forces, was increased. The confinement factor corresponds to the ratio of the confined compressive strength to the unconfined compressive strength of the concrete. To select an appropriate value of *k* for the confined region, the FEMA transverse reinforcement requirements for ductile column detailing were used, and the corresponding confinement factor *k* of 1.3 was adopted. This may be compared to the unretrofitted columns, for which the transverse reinforcement corresponds to a confinement factor *k* of 1.02. The external steel plates were assumed to be applied over a 910 mm length at the column ends indicated in Figure 10. This length was selected to exceed the expected flexural plastic hinge length of 625 mm for the first story columns based on Equation 1 from Paulay and Priestley [13].

$$L_p = 0.15 d_b f_y + 0.08L \tag{1}$$

where:

L_p = Plastic hinge length (inches)

d_b = Longitudinal bar diameter (inches)

f_y = Yield strength of reinforcement (ksi)

L = Member length (inches)

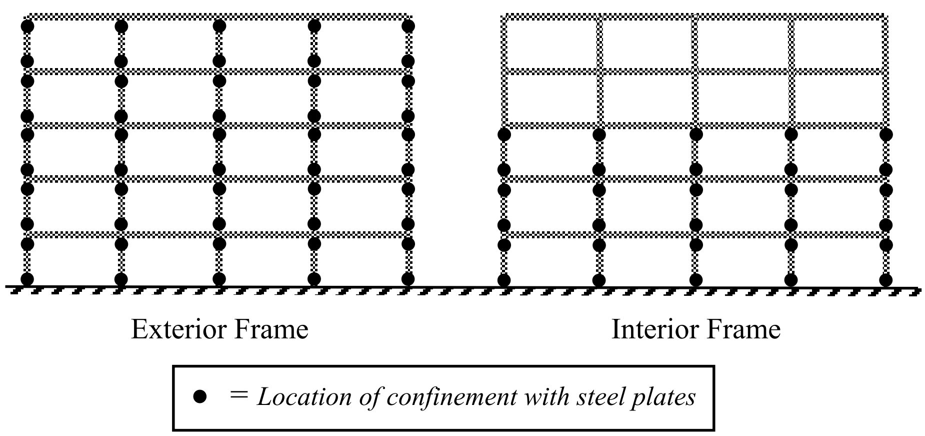


Figure 10. Retrofit 3: confinement of column plastic hinge zones

FRAGILITY ANALYSIS

Seismic Fragility Analysis

Methodology

In this study, the objective of the seismic fragility analysis was to assess the effectiveness of retrofit by estimating the reduction in the probability of exceeding a certain limit state, as compared to the unretrofitted structure. To develop the desired fragility curves, several parameters were needed, including structural characteristics, earthquake intensities, and uncertainties for capacity and demand. The seismic demand was determined from the twenty synthetic Memphis ground motions summarized in Table 2. The fragility curves for the unretrofitted and retrofitted case study building were derived using the relationship given in Equation 2 from Wen et al. [14].

$$P(LS/S_a) = 1 - \Phi\left(\frac{\lambda_{CL} - \lambda_{D/S_a}}{\sqrt{\beta_{D/S_a}{}^2 + \beta_{CL}{}^2 + \beta_M{}^2}}\right) \tag{2}$$

where:

$P(LS/S_a)$	=	Probability of exceeding a limit state given the spectral acceleration at the fundamental period of the building
Φ	=	Standard normal cumulative distribution function
λ_{CL}	=	ln(median drift capacity for a particular limit state), where drift capacity is expressed as a percentage of the story height
λ_{D/S_a}	=	ln(calculated median demand drift given the spectral acceleration), where demand drift is determined from a fitted power law equation
β_{D/S_a}	=	Uncertainty associated with the fitted power law equation used to estimate demand drift $= \sqrt{\ln(1+s^2)}$
β_{CL}	=	Uncertainty associated with the drift capacity criteria, taken as 0.3 for this study [14]
β_M	=	Uncertainty associated with analytical modeling of the structure, taken as 0.3 for this study [14]
s^2	=	Square of the standard error $= \dfrac{\sum\left[\ln(Y_i) - \ln(Y_p)\right]^2}{n-2}$
Y_i and Y_p	=	Observed demand drift and power law predicted demand drift, respectively, given the spectral acceleration
n	=	Number of sample data points for demand

Unretrofitted Case Study Building

To demonstrate the above methodology, the unretrofitted case study building is considered. Figure 11 provides the relationship between maximum interstory drift and the corresponding spectral acceleration for both the 10% in 50 years and the 2% in 50 years Memphis motions. A total of twenty points are plotted, where each data point represents the demand relationship for one ground motion record. The spectral acceleration (S_a) for a given ground motion record is the value corresponding to the fundamental period of the structure based on cracked section properties (T_1 = 1.62 s) and 2 percent damping. The drift demand value is the maximum interstory drift determined during the nonlinear time history analysis of the structure when subject to that ground motion record. The best-fit power law equation is also provided in the graph. This

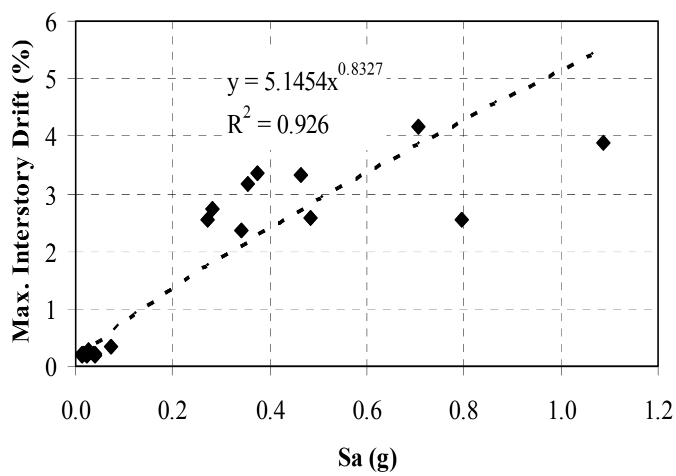


Figure 11. Demand drift versus spectral acceleration for Memphis motions

equation is used to describe the demand drift when constructing the fragility curves for the unretrofitted structure. The corresponding value of s^2 for the unretrofitted case is 0.144, which gives a β_{D/S_a} value of 0.367.

In addition to describing the demand drift, the drift capacity must also be defined for each limit state (or performance level). As a starting point for this study, the FEMA 356 global drift limits were used to describe the IO, LS and CP performance levels; which are 1, 2 and 4 percent, respectively, for a RC frame structure. As noted earlier, FEMA 356 also provides member-level plastic rotation limits that are more specific to the detailing and demands for specific structural components. Because this structure contains an interior slab-column frame system, punching shear drift limits were also considered to establish an upper bound drift limit for CP. The gravity shear ratio of the interior slab-column connections in the case study structure is 0.29 at the floor levels and 0.39 at the roof level. Using the model proposed by Hueste and Wight [9], the corresponding interstory drift limits at which punching shear is predicted at the interior slab-column connections are 2.9 percent and 1.6 percent. Because the maximum interstory drift values during the nonlinear dynamic analyses occurred at the lower stories, a punching shear drift limit of 2.9 percent was selected for the CP limit state. This value was used instead of the 4 percent limit for CP given by FEMA 356. The resulting fragility curves for the unretrofitted building are provided in Figure 12.

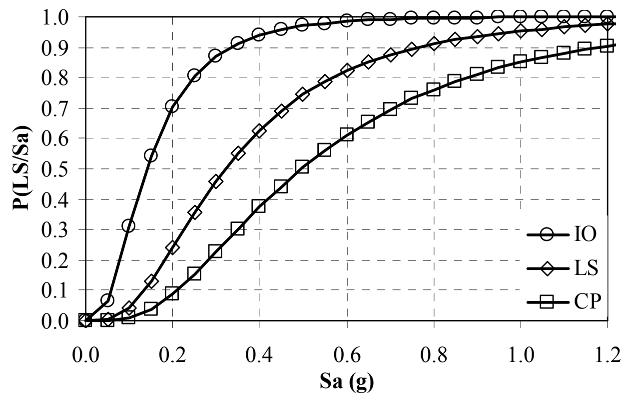
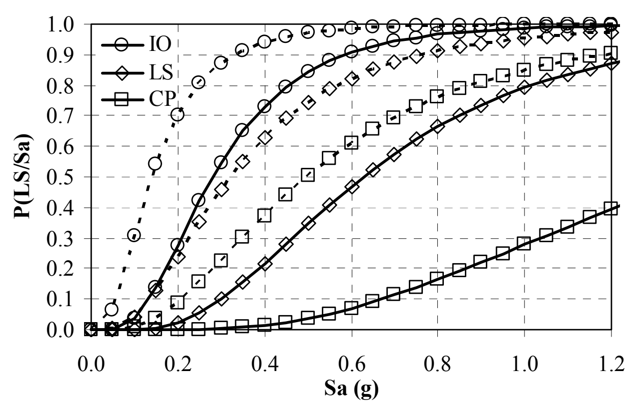


Figure 12. Fragility curves for the unretrofitted case study building

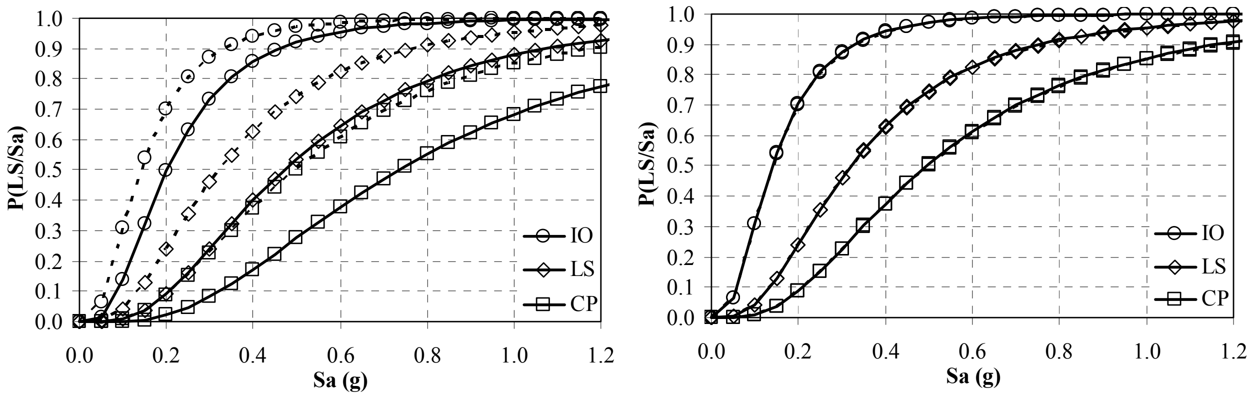
Retrofitted Case Study Building

The fragility curves developed using the three retrofit techniques are provided in Figure 13. For comparison, the fragility curves for the unretrofitted structure are represented on each graph with dotted lines. Based on the global drift limits of FEMA 356, the IO, LS and CP performance levels are defined differently for concrete wall elements; with drift limits of 0.5, 1 and 2 percent, respectively. Therefore, these values were used to define drift capacity for the shear wall retrofit fragility curves. As shown in Figures 13a and 13b, the addition of shear walls and RC column jackets were effective in decreasing the probability of exceeding each limit state. However, for the case of confining the plastic hinge zones (Retrofit 3), the fragility curves for each limit state are the same as those for the unretrofitted structure (Figure 13c). This is because the same global-level capacity drift limits are used for both the unretrofitted and Retrofit 3 structures. In addition, the demand drifts are nearly the same because the added confinement of Retrofit 3 does not modify the global structural response.



(a) Retrofit 1: shear walls added to exterior frame

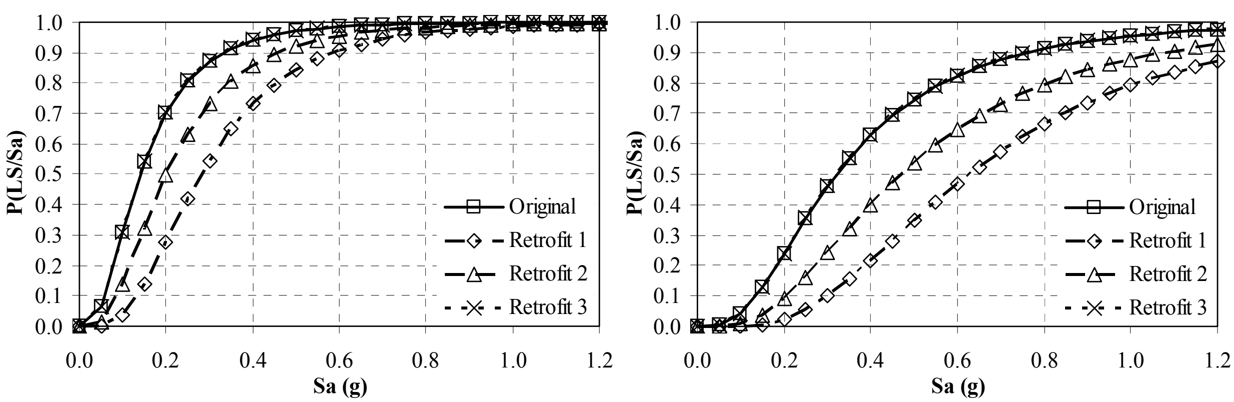
Figure 13. Fragility curves for the retrofitted case study building



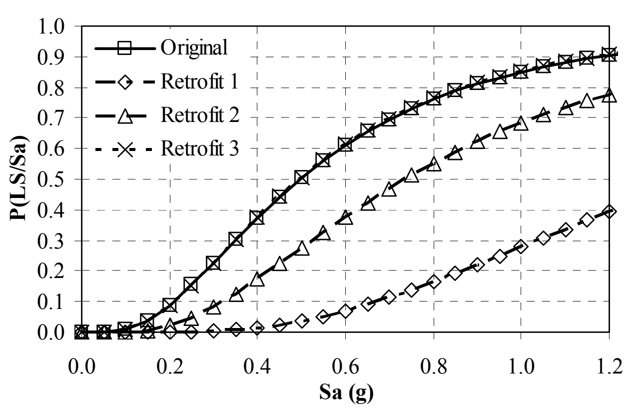
(b) Retrofit 2: addition of RC column jackets (c) Retrofit 3: confinement of column plastic zones

Figure 13. Fragility curves for the retrofitted case study building (continued)

Figure 14 shows comparisons of the fragility curves for each limit state. The shear wall retrofit (Retrofit 1) provides the greatest reduction in the probability of exceedance, while the column jacketing retrofit (Retrofit 2) is the second most effective. Because of the similarity in structural response and capacity limits for the unretrofitted building and Retrofit 3, no change in fragility was observed due to confining the column plastic hinge zones. This underscores the need to consider capacity drift limits based on member-level deformation limits. Further research is aimed at developing refined structure-specific capacity drift limits, based on member-level criteria, and corresponding fragility curves.



(a) Immediate Occupancy (IO) (b) Life Safety (LS)



(c) Collapse Prevention (CP)

Figure 14. Comparisons of fragility curves for each limit state

SUMMARY AND CONCLUSIONS

Nonlinear analyses were conducted for a prototype five-story reinforced concrete (RC) frame office building designed for mid-1980s code requirements in the Central United States. The FEMA 356 performance criteria were applied to determine whether the predicted response of the building meets the suggested Basic Safety Objective (BSO). It was found that the predicted response for the St. Louis ground motions was within the BSO limits. For the Memphis ground motions, different outcomes occurred when the global-level performance criteria (drift) were used versus the member-level performance criteria (plastic rotation). Based on the drift limits, the predicted building response meets the BSO for both the 10% in 50 years and the 2% in 50 years events. However, an evaluation using the member-level limits indicated that the member response is not within the limits of the BSO for the 2% in 50 years event.

Three retrofit techniques were applied to the case study structure: addition of shear walls, addition of RC column jackets, and confinement of the column plastic hinge regions using externally bonded steel plates. To assess the effectiveness of retrofit, fragility curves were developed for the unretrofitted and retrofitted structures. Twenty ground motions for the 10% in 50 years and the 2% in 50 years Memphis events were used to determine the relationship between demand drifts and spectral acceleration. As a first step, the FEMA 356 global drift limits were used to define drift capacity for the Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) performance levels (limit states). Punching shear drift limits were also considered to establish an upper bound drift capacity limit for CP.

The addition of shear walls and RC column jackets led to a decrease in the probability of exceeding each limit state. However, for the case of confining the plastic hinge zones, the fragility curves for each limit state are the same as those for the unretrofitted structure because the capacity drift limits do not account for increased member ductility. Further research is aimed at developing refined structure-specific capacity drift limits, based on member-level criteria, and the corresponding fragility curves.

It must be noted that this evaluation is specific to the characteristics of this structure. Additional studies are needed to characterize the expected seismic performance of vulnerable structures and to develop effective seismic rehabilitation techniques that meet the selected performance objectives.

ACKNOWLEDGMENTS

The authors wish to acknowledge the National Science Foundation and the University of Illinois who funded this research through the Mid-America Earthquake Center (NSF Grant Number EEC-9701785). The financial support provided by the Civil Engineering Department at Texas A&M University, where this research was conducted, is also appreciated. The authors also wish to express their appreciation to Jason Hart for his contribution in defining the case study building. The opinions expressed in this paper are those of the authors and do not necessarily reflect the views or policies of the sponsors.

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