# Assessment of Seismic Retrofitting Techniques of RC Structures Using Fragility Curves

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***Abstract*— In order to study the efficacy of several seismic retrofitting techniques on the seismic performance of existing multistory reinforced concrete structures, finite element analysis is conducted using the finite element package, SeismoStruct. First, three models are considered; 4-story, 8-story and 12-story reinforced concrete framed structures designed according to Saudi Building Code (2007) for vertical loads and seismic forces for 0.2-second and 1.0-second response spectral accelerations of 0.21g and 0.061g, respectively. Two conventional retrofitting techniques are considered to upgrade the structures to withstand seismic forces for 0.2-second and 1.0-second response spectral accelerations of 0.66g and 0.23g, respectively. These techniques are increasing the dimension of structural elements and attaching concentric steel braces at the middle bay of each story to the existing reinforced concrete frames. One more innovative retrofitting technique relying on adding passive control devices is considered in the analysis. Incremental dynamic analysis using records of twelve artificial and historic earthquakes is carried out. Fragility curves are developed for all original and retrofitted cases considering five different performance levels for the sake of the assessment of the effectiveness of different retrofitting techniques. Based on the results obtained, retrofitting existing reinforced concrete framed structures by adding concentric steel braces are the best technique that enhances their seismic performance, compared to other techniques.**

***Index Terms*—retrofitting, incremental dynamic analysis, reinforced concrete frames, steel braces, fragility curves**

## I. Introduction

After major earthquakes and the subsequent devastating damage occurring to structures, building codes and method of seismic design are considered for revisions and modifications. This may result in the necessity of strengthening and retrofitting of existing structures. There are many well-known seismic retrofitting techniques for structures. These techniques can be categorized into two groups [1]:

- Conventional methods, based on improving the strength, stiffness and ductility of the structure.
- Innovative response modification methods, which aim to reducing the effect of seismic forces on structures.

Conventional methods include techniques such as increasing the lateral stiffness of structural systems through increasing the dimensions of reinforced concrete columns, adding reinforced concrete infill walls to the structural system and adding braces to the existing reinforced concrete frames [2], [3]. These methods can be easily designed and applied using conventional construction techniques. However, excessive increase in lateral stiffness may lead to larger earthquake forces and lower ductility, which affects the structural performance under earthquakes. In some cases, there is a need to heavy demolition and construction work [4].

Different bracing systems may be used in retrofitting structures [5], [6]. Another retrofitting method was proposed [1], which is a system of a rectangular steel housing frame with chevron braces and a yielding shear link connected between the braces and the frame. Reference [7] investigated the seismic reliability of a six story reinforced concrete building retrofitted using eccentric steel braces through fragility analysis. They examined the effectiveness of using D, K, and V types of eccentric steel braces in retrofitting the building.

Innovative techniques include installing passive control devices such as dampers or seismic isolation devices in the building. These devices help dissipating energy during earthquakes, which enhances structural response and reduces seismic forces transmitted to the structural system. [8], [9]. When used for seismic retrofitting, innovative techniques usually do not require heavy demolition or construction work. However, they are generally costly to be used for retrofitting ordinary buildings [10].

Some researchers look for new materials to be used to enhance structural response under earthquakes. Reference [11] analyzed the possibility of the application of stainless steels for seismic retrofitting of steel structural systems for multi-story structures. Reference [12] assessed the efficiency of external fiber reinforced polymer, FRP, reinforcement retrofitting using three experimental investigations. They studied the influence of axial compressive loading, shape of the reinforced short column, presence of FRP bars and FRP reinforcement on the performance index of the columns.

Performance-based concepts may be used in order to assess the enhancement of structural performance using different seismic retrofitting techniques. The performance levels are commonly interpreted in terms of interstory

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drift ratios. Reference [13] suggested values of maximum interstory drift ratio for each performance level for different structural systems. For systems rather than that with masonry shear walls, the values of maximum interstory drift ratios for performance levels; Operational (OP), Immediate Occupance (IO), Damage Control (DC), Life Safety (LS) and Collapse Prevention (CP) are 0.005, 0.01, 0.015, 0.02 and 0.025, respectively. Although the seismic risk is different from a place to another, but the definition of performance level, which depends on the desired damage status of a building after a specific earthquake is almost the same. In KSA, no specific values of interstory drift ratio are specified for different performance levels in Saudi Building Code [14]. However the design story drift values are selected to match the values in other international provisions, such as FEMA 450 [15], for all structural systems. Therefore same definition of damage state can be used but at different earthquake intensities. Accordingly, the values suggested by [13] are used in this research.

Fragility curves are always used in the seismic risk analysis [16], [17]. Also, they have been used as a tool for assessment of retrofitting option [18]. Fragility analysis was used as a tool to assess retrofitting effectiveness of structures [7]. Also, the fragility curves were used in the decision-making process for structures [19].

In this research, finite element analysis is utilized, using SeismoStruct [20], to study the effectiveness of seismic retrofitting of existing multistory reinforced concrete framed structures. Three structural models are considered; 4-story, 8-story and 12-story reinforced concrete framed structures with a limited lateral stiffness. Two conventional retrofitting techniques are utilized: increasing the dimension of structural elements and adding concentric steel braces at the middle bay of each story to the existing reinforced concrete frames. One more innovative retrofitting technique relying on adding passive control devices is considered in the analysis. Incremental dynamic analysis using records of twelve artificial and historic earthquakes is carried out in the analysis. Fragility curves are developed for all original and retrofitted cases considering five different performance levels for the sake of the assessment of the effectiveness of different retrofitting techniques.

## II. Structural Models

### A. Models

The structures considered in this research represent typical mid-rise reinforced concrete residential framed structures in KSA. Three structural models are selected; 4-story, 8-story and 12-story moment-resisting frames. The structures are three-bay frames with first story height of 5 m, while the height of the rest of the floors is 3 m each. The bay width is 5 m and the frames are 4 m apart. The compressive strength of concrete is 300 Kg/cm2, while the yielding stress of reinforcing steel is 3600 Kg/cm2. Structural models are shown in Fig. 1. The soil class is assumed D, which is stiff soil with shear wave velocity, VS, ranging from 180 to 370 m/s. The structures are classified as low hazard buildings, with importance factor I = 1. According to the modal analysis, the natural time period is 0.66, 0.88 and 1.06 seconds for 4-story, 8-story and 12-story structural models, respectively.

First, the structural models were designed according to the Saudi Building Code (2007) for vertical loads and seismic forces for 0.2-second and 1.0-second response spectral accelerations of 0.21g and 0.061g, respectively. The seismic design equivalent lateral forces at each floor are calculated from the relationship.

$$F_x = C_{VX} V \tag{1}$$

where V is the total base shear and $C_{VX}$ is vertical distribution factor calculated from:

$$C_{VX} = \frac{W_x * h_x^k}{\sum W_i * h_i^k} \tag{2}$$

$w_i$ and $w_x$ = the portion of the total gravity load of the structure ($W$) located or assigned to level $i$ or $x$, $h_i$ and $h_x$ = the height (m) from the base to Level $i$ or $x$, $k$ is an exponent related to the structure period. For structures having a period of 0.5 sec or less, $k$ = 1.0 while for structures having a period between 0.5 and 2.5 seconds, $k$ is calculated by linear interpolation between 1.0 and 2.0. For ordinary moment-resisting frames, the seismic response modification coefficient, R, is taken to be 4. The details of the design sections of beams and columns for the three original structures models are summarized in Table I.

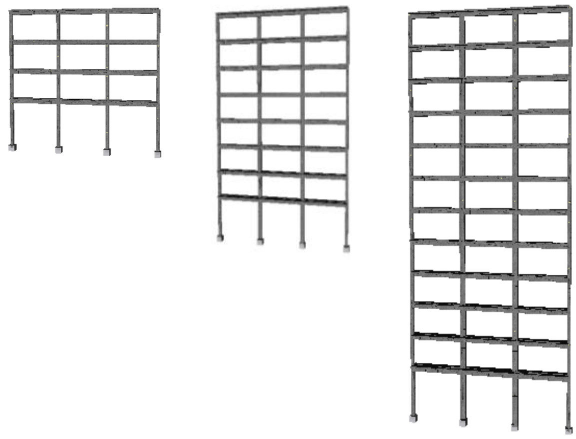


Figure 1. Models of original (not retrofitted) structures (case 1)

### B. Retrofitting Techniques

Two retrofitting techniques are used. The first technique is to jacket the columns and increase their reinforcing bars in order to withstand larger seismic forces through (0.2-second and 1.0-second response spectral accelerations of 0.66g and 0.23g, respectively). These spectral accelerations may be specified when changing the building code, after poor performance during a recent earthquake or when there is a need to enhance structural performance under future earthquakes. The structural models are redesigned considering the new seismic forces and the details of new cross sections of columns are tabulated in Table II (case 2).

TABLE I. DETAILS OF SECTIONS OF BEAMS AND COLUMNS OF ORIGINAL FRAMES (CASE 1)

| Model | Beams | Columns | |
|---|---|---|---|
| | | External | Internal |
| 4-story | B1 | C1 | C1 |
| 8-story | B1 | C1 | C2 |
| 12-story | B2 for 1st floor<br>B1 for the rest | C1 | C3 |

TABLE II. DETAILS OF SECTIONS OF BEAMS AND COLUMNS OF RETROFITTED FRAME (CASE 2)

| Model | Columns | |
|---|---|---|
| | External | Internal |
| 4-story | C3 | C4 |
| 8-story | C3 | C4 |
| 12-story | C3 | C5 |

The second technique is to add concentric steel braces to the existing reinforced concrete frames in order to retrofit the structure to withstand same seismic forces as in case 2 (0.2-second and 1.0-second response spectral accelerations of 0.66g and 0.23g, respectively). The braces are added at each floor of the structural models in the middle bay of the frames. Same cross sections of beams and columns of original models are maintained while the added braces are designed to carry the increase in the seismic forces. The models with the added braces are shown in Fig. 2. The brace models and their cross sections are tabulated in Table III and Table IV (Case 3). The cross sections and reinforcement of beams and columns used in all models are tabulated in Table V and Table VI, respectively.

## III. INCREMENTAL DYNAMIC ANALYSIS

### *A. Methodology*

The structural models were subjected to the selected twelve ground motions using SeismoStruct. Under each ground motion, nonlinear time history analyses were conducted while scaling the peak ground acceleration, PGA, of chosen ground motion incrementally every 0.10g, until structural instability is obtained or up to PGA = 1.0g. The relationship between the maximum interstory drift ratio and the corresponding PGA was obtained, which creates the IDA curves for a certain structure under the specified ground motion.

TABLE III. DETAILS OF RETROFITTED MODELS USING BRACES (CASE 3)

| Structural Model | Brace model |
|---|---|
| 4-story | Brace 2 for 1st floor<br>Brace 1 for the rest |
| 8-story | Brace 3 for 1st floor<br>Brace 1 for the rest |
| 12-story | Brace 3 for 1st floor<br>Brace 2 for 2nd to 5th floor<br>Brace 1 for the rest |

TABLE IV. BRACES SECTIONS

| Brace Model | Section | Area (cm2) |
|---|---|---|
| Brace 1 | W6x12 | 22.90 |
| Brace 2 | W8x18 | 33.94 |
| Brace 3 | W12x30 | 56.71 |

TABLE V. BEAM SECTIONS

| Beam Model | Dimensions cm x cm | Tension reinf. | Comp. reinf. | Stirrups (/m) |
|---|---|---|---|---|
| B1 | 25 x 50 | 4 ϕ 16 | 2 ϕ 12 | 5 ϕ 10 |
| B2 | 25 x 60 | 6 ϕ 16 | 2 ϕ 14 | 5 ϕ 10 |

TABLE VI. COLUMNS SECTIONS

| Column Model | Dimensions cm x cm | Reinf. | Stirrups (/m) |
|---|---|---|---|
| C1 | 30 x 30 | 6 ϕ 16 | 5 ϕ 10 |
| C2 | 30 x 40 | 8 ϕ 16 | 5 ϕ 10 |
| C3 | 30 x 50 | 10 ϕ 16 | 5 ϕ 10 |
| C4 | 30 x 60 | 18 ϕ 16 | 5 ϕ 10 |
| C5 | 30 x 70 | 18 ϕ 16 | 5 ϕ 10 |

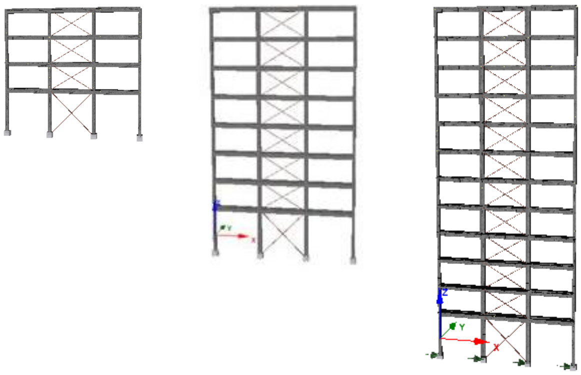


Figure 2. Models of retrofitted structures using steel braces (case 3)

TABLE VII. CHARACTERISTICS OF GROUND MOTIONS USED

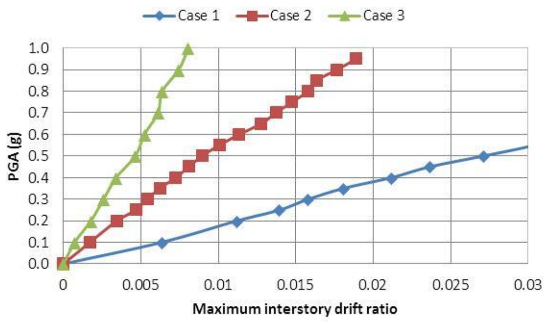
| No. | Ground Motion | Location | PGA (g) |
|---|---|---|---|
| 1 | Artificial | By SeismoStruct | 0.436 |
| 2 | ChiChi | Taiwan | 0.808 |
| 3 | Loma Prieta | Corralitos station, USA | 0.799 |
| 4 | Loma Prieta, | Emeryville station, USA | 0.250 |
| 5 | Friuli | Italy | 0.479 |
| 6 | Hollister | City Hall station, USA | 0.120 |
| 7 | Kocaeli | Sakaria station, Turkey | 0.628 |
| 8 | Kern County | Taft Lincoln School Tunnel, CA, USA | 0.179 |
| 9 | San Fernando | 8244 Orion Blvd, Los Angeles, CA USA | 0.134 |
| 10 | Imperial Valley | EL Centro, USA | 0.349 |
| 11 | Northridge | Arleta and Nordhoff Fire Station, USA | 0.344 |
| 12 | Parkfield | Cholame, Shandon, CA, USA | 0.275 |

### *B. Ground Motions*

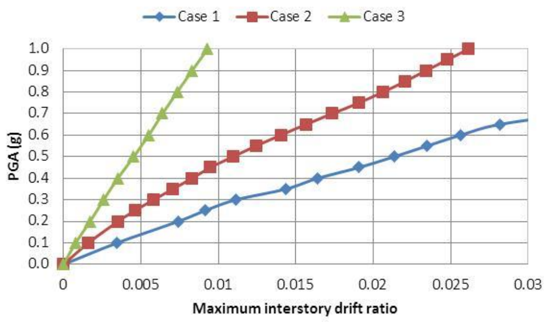
An appropriate set of ground motions is required to perform incremental dynamic analysis. For mid-rise buildings, ten to twelve ground motions are required in order to provide good estimation of seismic demand [21]. These ground motions can be selected from real records of earthquakes or can be generated artificially. Real records are more realistic since they include all ground motions characteristics such as amplitude, frequency, duration, energy content, number of cycles and phase [22]. No ground motions were recorded in the past in KSA. Accordingly, in this analysis, twelve records of ground motions were selected to perform the nonlinear time history analysis of the chosen structures; one is artificial and the rest are real records of historical earthquakes. The characteristics of these ground motions are presented in Table VII.

### C. Results

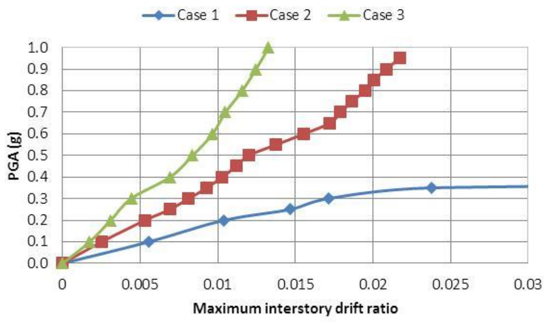
The IDA curves developed for the three structural models considering the original case and the retrofitting using the two different techniques were presented in details [23]. The average IDA curves of these structural models considering different cases are shown in Fig. 3.




(a) 4-story




(b) 8-story




(c) 12-story

Figure 3. Average IDA curve for structural models

According to the obtained results, there is an enhancement in structural response using both retrofitting techniques for 4-story, 8-story and 12-story structural models. Using steel braces (case 2) has better effect on structural performance than column jacketing (case 3). For earthquakes with PGA of 0.2g, which are considered relatively weak ground motions, the original structures (case 1) experienced interstory drift ratios higher than 0.005 under 12, 5 and 8 ground motions out of 12 for 4-story, 8-story and 12-story structural models, respectively. However, in case 2, this value of interstory drift ratio was exceeded for the same intensity of ground motions under 2, 3 and 4 ground motions out of 12 for 4-story, 8-story and 12-story structural models, respectively. The enhancement in structural response in case 2 was much better so that the 0.005 interstory drift ratio value was exceeded under one ground motion only for 12-story structural model and was not exceeded for the two other models. This value of interstory drift ratio is considered as operational performance level, which means continuous service with negligible structural and non-structural damage (SEAOC 2000).

For relatively stronger earthquakes with PGA of 0.4g, structural models in case 1 experienced interstory drift ratios higher than 0.005 under 12, 10 and 11 ground motions out of 12 for 4-story, 8-story and 12-story structural models, respectively. However, in case 2, this value of interstory drift ratio was exceeded for the same intensity of ground motions under 10, 8 and 9 ground motions out of 12 for 4-story, 8-story and 12-story structural models, respectively. Same value of interstory drift ratio was exceeded 3, 4 and 7 ground motions out of 12 for 4-story, 8-story and 12-story structural models, respectively for case 3. According to these results, the effect of retrofitting using steel braces has better effect on the enhancement in 4-story structural model than that obtained for 8-story and 12-story structural models. Accordingly, mega braces (extended over more than one floor) may be a better choice for retrofitting higher structures.

## IV. Fragility Curves

Fragility curves are always used in the seismic risk analysis. The fragility curves are considered useful tools for predicting the extent of probable damage. The fragility curves can be used in retrofitting decisions, estimating of casualties and economic losses, and finally the disaster response planning, which is the most important objective.

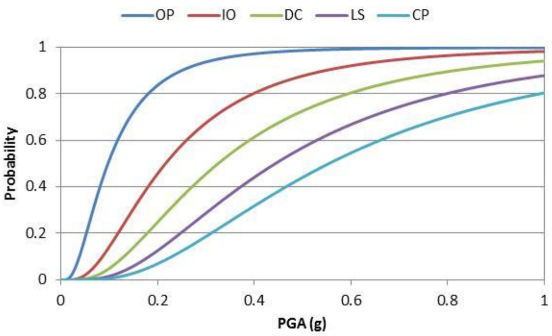
Fragility curves are lognormal functions which express the probability of reaching or exceeding a specific damage state. They can be developed in terms of a seismic parameter, such as spectral acceleration, spectral displacement, peak ground velocity and PGA. Since PGA was the parameter used in developing the incremental dynamic analysis in this research, the PGA was selected to be the corresponding parameter in developing the fragility curves.

The cumulative distribution functions was calculated by dividing the number of data points that reached or exceeded a particular damage state by the number of data points of the whole sample [24]. The conditional probability of a structure to reach or exceed a specific damage state, D, given the peak ground acceleration, PGA, is defined by:

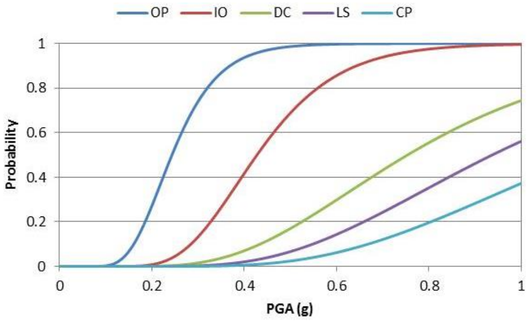
$$P[\frac{D}{PGA}] = \Phi(\frac{\ln(PGA) - \mu}{\sigma}) \tag{3}$$

where: $\Phi$ is the standard normal cumulative distribution function; $\mu$ and $\sigma$ are the mean value and standard deviation of the natural logarithm of PGA at which the building reach the threshold of a specific damage state or performance level, D. Lognormal functions with two parameters (μ and σ) were fitted for different performance levels; OP, IO, DC, LS and CP, associated with 4-story, 8-story and 12-story structures for the original case, case 1, and retrofitted cases, case 2 and case 3. The whole set of fragility curves are shown in Fig. 4, Fig. 5 and Fig. 6. From these figures, the following observations are noticed:

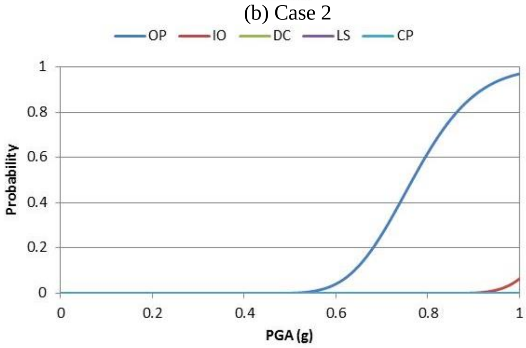
- When exposed to weak ground motions with PGA = 0.2g, the probability of reaching or exceeding the OP performance level is 84.1%, 28.0% and 0.0% for the 4-story structure for case 1, case 2 and case 3, respectively. For 8-story structure, the probability of reaching or exceeding the OP performance level is 51.0, 22.5% and 0.0% for case 1, case 2 and case 3, respectively. For the 12-story structure, the probability of reaching or exceeding the OP performance level is 69.8%, 34.3% and 0.0% for three cases respectively. Considering the DC performance level, probability of reaching or exceeding this performance level is 25.3%, 0.1% and 0.0% for the 4-story structure, 11.9%, 0.54% and 0% for the 8-story structure and 23.3%, 4.7% and 0% for the 12-story structure for case 1, case 2 and case 3, respectively.
- Similar trend is obtained when exposed to relatively strong ground motions with PGA = 0.4g. The probability of reaching or exceeding the OP performance level is 97.3%, 93.8% and 0.0% for the 4-story structure for case 1, case 2 and case 3, respectively. For 8-story structure, the probability of reaching or exceeding the OP performance level is 81.6, 66.3% and 0.0% for case 1, case 2 and case 3, respectively. For the 12-story structure, the probability of reaching or exceeding the OP performance level is 91.1%, 76.7% and 0.0% for three cases respectively. Considering the DC performance level, probability of reaching or exceeding this performance level is 61.5%, 7.1% and 0.0% for the 4-story structure, 40.0%, 9.2% and 0% for the 8-story structure and 51.1%, 22.8% and 0% for the 12-story structure for case 1, case 2 and case 3, respectively.
- Good enhancement is obtained when jacketing the columns while great enhancement is reached by adding concentric steel braces for all structural models for different performance levels.




(a) Case 1

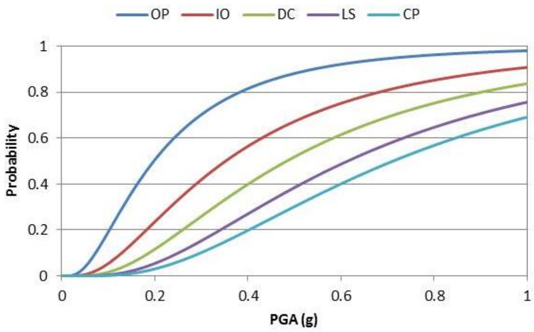



(b) Case 2

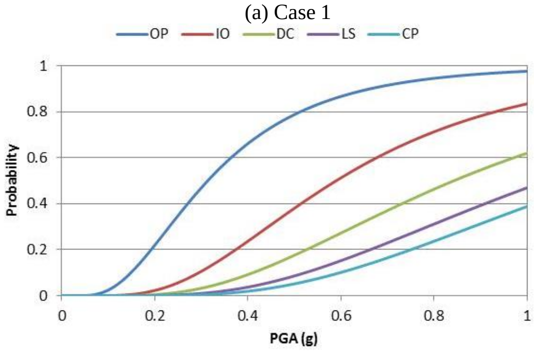



(c) Case 3

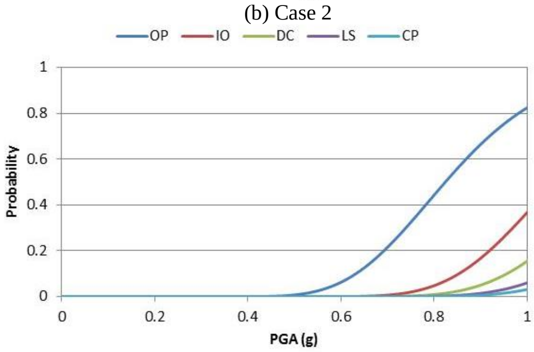
Figure 4. Fragility curves for the 4-story structural model




(a) Case 1




(b) Case 2

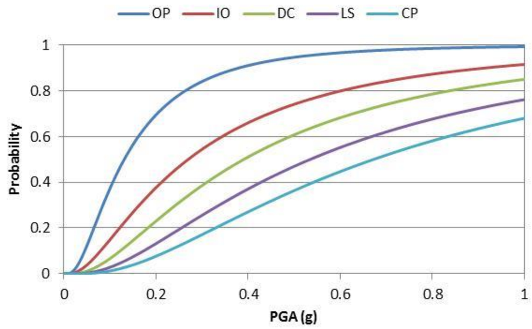



(c) Case 3

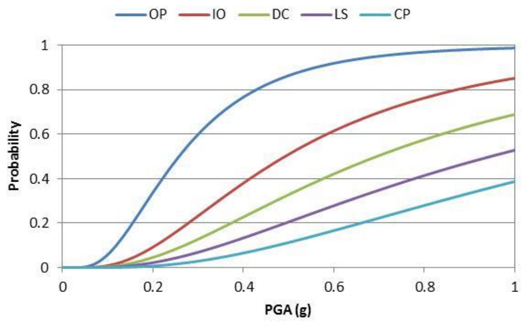
Figure 5. Fragility curves for the 8-story structural model

## V. Innovative Technique Using Passive Control Devices

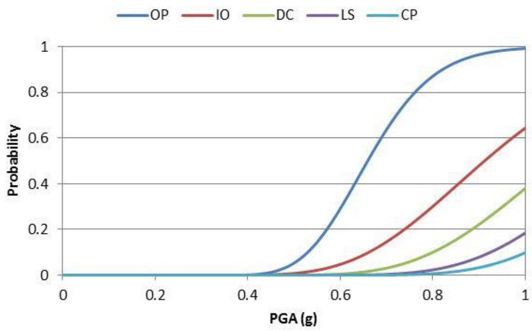
In SeismoStruct, dampers are normally modeled using link elements with adequate response curves that may be able to characterize the force-displacement relationship of a given damper. However, in those cases where velocity dependence is important, this dashpot element may be employed instead with a linear force-velocity relationship. The dashpot is a single-node damping element, which may be employed to represent a linear dashpot fixed to the ground. The dashpot accounts for the relative motion with respect to the ground. In this multistory structural model, it is not accurate to link all dampers used to the ground. Instead, the passive control devices are modeled using diagonal elements with mass proportional damping of a parameter equals 0.10. Fig. 7 shows the configuration of these dampers for the 8-story structure. The IDA curves for the 8-story structure retrofitted using passive control devices are developed using the same twelve ground motions used in the previous analysis (Fig. 8). Fig. 9 shows the developed fragility curves for this structure considering same performance levels.




(a) Case 1




(b) Case 2




(c) Case 3

Figure 6. Fragility curves for the 12-story structural model

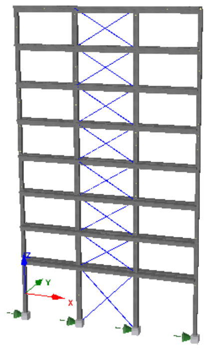


Figure 7. 8-story structure retrofitted using passive control devices

When exposed to weak ground motions with PGA = 0.2g, the probability of reaching or exceeding the OP performance level is reduced from 51.0% in the original 8-story structure (case 1) to 0.0% when retrofitted using passive control devices. When exposed to relatively strong ground motions with PGA = 0.4g, the probability of reaching or exceeding the OP performance level is dropped from 81.6% to 14.6% while the probability of reaching or exceeding the DC performance level is dropped from 40.0% to 1.1% when retrofitting with passive control devices. Similar enhancement is obtained considering the CP performance level, where the probability is reduced from 19.9% to only 0.3%. Moreover, if subjected to ground motions with PGA=0.6g, the probability of reaching or exceeding OP, IO, DC, LS and CP is 29.2%, 4.8%, 0.6%, 0% and 0%, respectively.

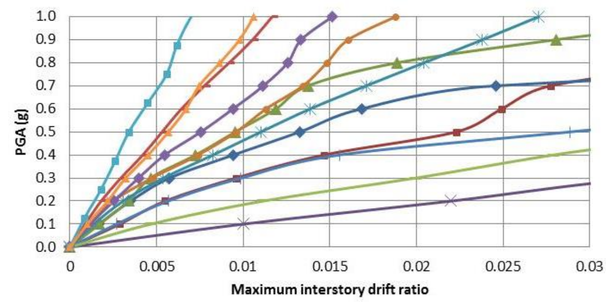



Figure 8. IDA for 8-story structure retrofitted using passive control devices

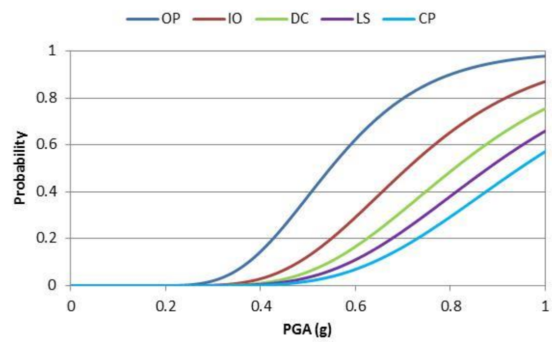



Figure 9. Fragility curves for 8-story structure retrofitted using passive control devices

Compared to other retrofitting techniques, the enhancement in seismic performance when using passive control devices is better than that obtained when jacketing the columns. However, adding concentric steel braces provide the best enhancement in structural response under earthquakes.

## VI. Conclusion

Three structural models are selected to investigate the effectiveness of several retrofitting techniques including using column jacketing, steel braces and passive control devices on the structural seismic performance. These models are for typical 4-story, 8-story and 12-story reinforced concrete residential structures in KSA. The analysis is conducted using incremental dynamic analysis utilizing twelve ground motions by SeismoStruct. Analytical fragility curves considering five different performance levels are presented for the three models retrofitted by the different retrofitting techniques. According to the results obtained, the following conclusions are achieved:

- Developed fragility curves for the structural models in different cases considered give a clear picture on the effect of different retrofitting techniques on the structural response under earthquakes.
- Retrofitting techniques using column jacketing, steel braces and passive control devices enhance the structural response of multistory reinforced concrete framed structures. Better enhancement can be attained by using concentric steel braces than other techniques in almost all cases. Structures with smaller height can be more efficiently retrofitted if compared to higher structures in case same retrofitting technique is utilized.

Selecting the appropriate retrofitting technique depends on the architectural aspects of the structure, the extent of performance enhancement needed and the common construction practice and technologies available in the region.

Retrofitting using different types of steel braces will be included in the future research, such as K, D and V types. Also, mega braces will be considered as an alternative brace configuration to be considered and compared among other brace types with the techniques used in this paper. Different passive control devices with various configuration and arrangement may be considered and compared in terms of their effect on structural response enhancement.

## Acknowledgment

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## References

[1] C. Durucan and M. Dicleli, “Analytical study on seismic retrofitting of reinforced concrete buildings using steel braces with shear link,” *Engineering Structures*, vol. 32, no. 10, pp. 2995-3010, 2010.

[2] T. Endo, A. Okifuji, S. Sugano, T. Ayashi, T. Shimizu, and K. Takahara, “Practices of seismic retrofit of existing concrete structures in Japan,” in *Proc. 8th World Conference on Earthquake Engineering*, 1984, vol. 1, pp. 469-476.

[3] A. Ghobarah, M. El-Attar, and N. M. Aly, “Evaluation of retrofit strategies for reinforced concrete columns: A case study,” *Engineering Structures*, vol. 22, no. 5, pp. 490-501, 2000.

[4] J. M. Kelly, “A seismic base isolation: Review and bibliography,” *Soil Dynamics and Earthquake Engineering*, vol. 5, no. 4, pp. 202-216, 1986.

[5] L. D. Sarno and A. S. Elnashai, “Bracing systems for seismic retrofitting of steel frames,” *Journal of Constructional Steel Research*, vol. 65, pp. 452-465, 2009.

[6] L. D. Sarno and G. Manfredi, “Seismic retrofitting with buckling restrained braces: Application to an existing non-ductile RC framed building,” *Soil Dynamics and Earthquake Engineering*, vol. 30, no. 11, pp. 1279-1297, 2010.

[7] A. E. Ozel and E. M. Guneyisi, “Effects of eccentric steel bracing systems on seismic fragility curves of mid-rise R/C buildings: A case study,” *Structural Safety*, vol. 33, no. 1, pp. 82-95, 2011.

[8] F. Mazza, M. Mazza, and A. Vulcano, “Displacement-based seismic design of hysteretic damped braces for retrofitting in-elevation irregular r.c. framed structures,” *Soil Dynamics and Earthquake Engineering*, vol. 69, pp. 115-124, 2015.

[9] R. Han, Y. Li, and J. van de Lindt, “Seismic risk of base isolated non-ductile reinforced concrete buildings considering uncertainties and mainshock–aftershock sequences,” *Structural Safety*, vol. 50, pp. 39-56, 2014.

[10] S. Ahmad, F. Ghani, and R. Adil, “Seismic friction base isolation performance using demolished waste in masonry housing,” *Construction Building Materials*, vol. 23, no. 1, pp. 146-152, 2009.

[11] L. D. Sarno, A. S. Elnashai, and D. A. Nethercot, “Seismic retrofitting of framed structures with stainless steel,” *Journal of Constructional Steel Research*, vol. 62, no. 1-2, pp. 93–104, 2006.

[12] G. Promis and E. Ferrier, “Performance indices to assess the efficiency of external FRP retrofittingof reinforced concrete short columns for seismic strengthening,” *Construction and Building Materials*, vol. 26 , no. 1, pp. 32-40, 2012.

[13] Q. Xue, C. W. Wu, C. C. Chen, and K. C. Chen, “The draft code for performance-based seismic design of buildings in Taiwan,” *Engineering Structures*, vol. 30, pp. 1535-1547, 2008.

[14] Saudi Building Code, SBC 301: Loads and Forces Requirements. Riyadh, KSA, Saudi Building Code National Committee, 2007.

[15] Federal Emergency Management Agency, FEMA 450, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, Part 1: Provisions, Washington DC, 2003.

[16] Y. E. Ibrahim and M. M. El-Shami, “Seismic fragility curves for mid-rise reinforced concrete frames in Kingdom of Saudi Arabia,” *IES Part A: Civil and Structural Engineering*, vol. 4, no. 4, pp. 213-223, 2011.

[17] Y. E. Ibrahim, O. Shallan, A. Elshihy, and M. Selim, “Seismic performance evaluation of residential structures in Egypt,” *Advanced Materials Research*, vol. 919-921, pp. 945-950, 2014.

[18] S. H. Kim and M. Shinozuka, “Development of fragility curves of bridges retrofitted by column jacketing,” *Probabilistic Engineering Mechanics*, vol. 19, no. 1-2, pp. 105-112, 2004.

[19] R. J. Williams, P. Gardoni, and J. M. Bracci, “Decision analysis for seismic retrofit of structures,” *Structural Safety*, vol. 31, pp. 188–196, 2009.

[20] SeismoStruct Ver. 7.0.0. (2014). SeismoSoft, Earthquake Engineering Software Solutions, Italy. [Online]. Available: http://www.seismosoft.com

[21] N. Shome and C. A. Cornell, “Probabilistic seismic demand analysis of nonlinear structures,” Ph.D. dissertation, Stanford, Stanford University, 1999.

[22] M. Rota, A. Penna, and G. Magnes, “A methodology for deriving analytical fragility curves for masonry buildings based on stochastic nonlinear analyses,” *Engineering Structures*, vol. 32, pp. 1312–1323, 2010.

[23] Y. E. Ibrahim, “Seismic retrofitting of mid-rise reinforced concrete framed structures,” *Earthquake Resistant Engineering Structures X, WIT Transaction on the Built Environment*, vol. 152, 2015.

[24] M. Shinozuka, F. Kim, T. Uzaua and T. Ueda, Statical Analysis of Fragility Curves. Technical Report MCEER-03-0002, Department of Civil and Environmental Engineering, University of Southern California, 2003.

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