

# Analytical Case Study of Seismic Performance of Retrofit Strategies for Reinforced Concrete Frames: Steel Bracing with Shear Links Versus Column Jacketing

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## ABSTRACT

The effectiveness of seismic retrofitting applied to enhance seismic performance is assessed for a five-storey reinforced concrete (RC) frame building structure as built in Jordan in mid 80s. The response of the structure is evaluated using nonlinear static and dynamic analysis with synthetic ground motion records for rock base.

FEMA 356 criteria are used to evaluate the seismic performance of the case study building. Two approaches are used for seismic evaluation: global-level evaluation (drift values) and member-level evaluation using three performance levels (immediate occupancy, life safety and collapse prevention). Based on the seismic evaluation results, two possible retrofit techniques are applied to improve the seismic performance of the structure, including the addition of RC column jackets and the addition of eccentric steel bracing. SAP 2000 is used to perform linear and nonlinear dynamic analysis, whereas plastic hinge analysis is performed by Response 2000.

This study shows that adding new structural elements as steel members to an existing RC building proves to be effective in enhancing performance and reducing cost than adding RC elements. Even more, the eccentric bracing proves to outperform the column jacketing drift limit, plastic hinge limit and cost effectiveness.

**KEYWORDS:** RC frame retrofitting, Seismic performance, Nonlinear analysis, Shear link, Bracing, Jacketing, Plastic hinge.

## INTRODUCTION

Modern seismic codes did not come into wide use until the 1990s. Therefore, some of the existing structures that were built before that time are probably seismically unsafe if checked by today's seismic standards, and that check will be specially significant if the structure is of high importance (historical buildings, schools, military barracks, ...etc). These structures might have significant deficiencies in their overall

structural system or in the members' reinforcement or detailing, thus not providing an adequate lateral support to satisfy seismic demand. Therefore, it is necessary to evaluate these structures and improve their seismic resistance / performance by retrofitting techniques of their systems if found vulnerable.

Since the purpose of any retrofit technique is to upgrade the seismic safety of the building to the desired level with the best and most appropriate techniques in a short time with minimum disturbance to the building residents, the retrofitting cost will be a vital criterion in retrofitting system selection; hence a

cost comparison is represented for both retrofitting schemes.

This study makes use of synthetic ground motions that were developed for rock base and provides an evaluation of a representative structure using both *Prestandard and Commentary Criteria for the Seismic Rehabilitation of Buildings* (FEMA 356) and other quantitative parameters as drift, ductility and stiffness.

### Scope and Research Objectives

The objectives of this study are to evaluate the seismic performance of a typical 1980s RC frame building in Jordan using the FEMA 356 performance criteria and to determine the effectiveness of various seismic retrofit techniques. Both FEMA 356 global-level and member-level limits are assessed for Basic Safety Objectives (BSO) performance levels. In addition, drift limits are set and compared to the structural drift values. In order to compute global structural parameters, such as stiffness, ductility and capacity curves, nonlinear static (pushover) analysis and nonlinear dynamic (time-history) analysis are conducted for the RC structure. For the time-history

analysis, three synthetic ground motion records developed by (Armouti, 2008) are used.

Two retrofit techniques are implemented: column RC jacketing and eccentric shear link bracing as shown in Figure 1.

### Case Study Building

- Building Description

The case study building is a five storey RC beam-column building with an overall height of 20.0 m and two-way slab floors. Each storey is 4.0 m high. The building is essentially rectangular in shape and is 42.5 m long by 34.0 m wide. The bay size is 8.5 m by 8.5 m. The building is a frame system that is not detailed for ductile behavior and is not designed to accommodate any seismic action. Figure 2 provides a typical floor plan and elevation view of the structure.

The case study building is designed according to the provisions of *American Concrete Institute Building Code Requirements for Reinforced Concrete, ACI 318-08*.

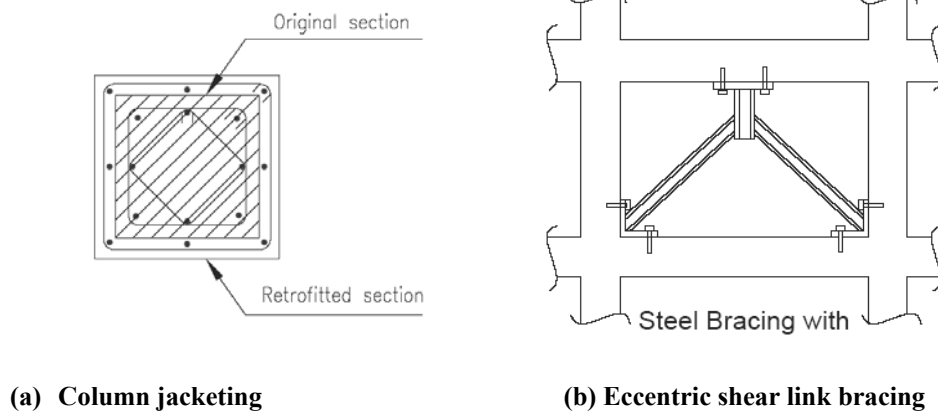
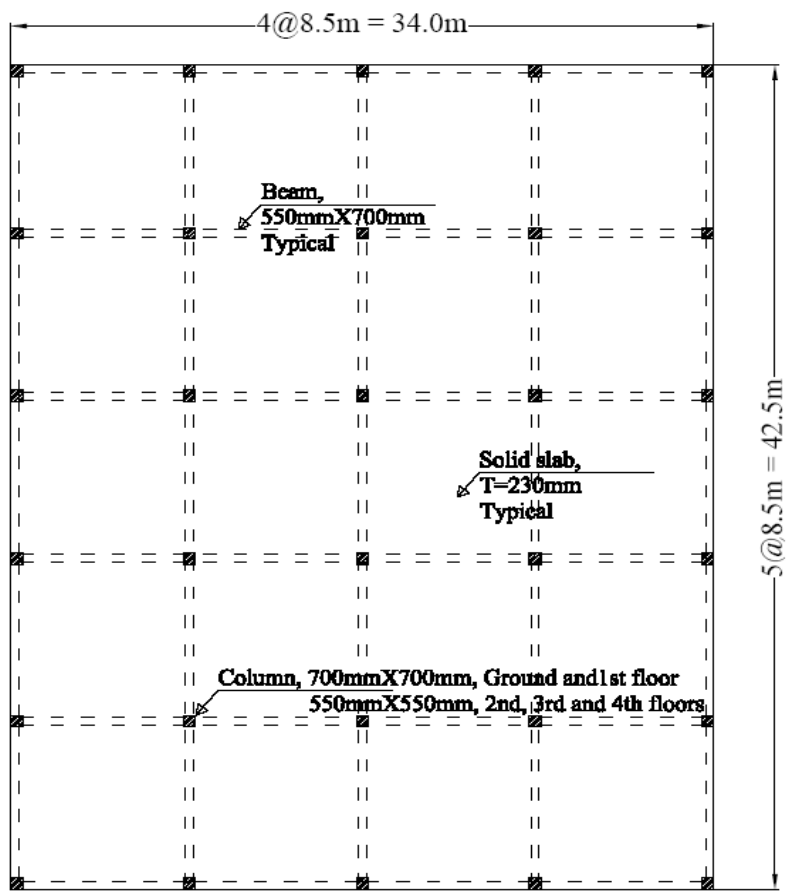
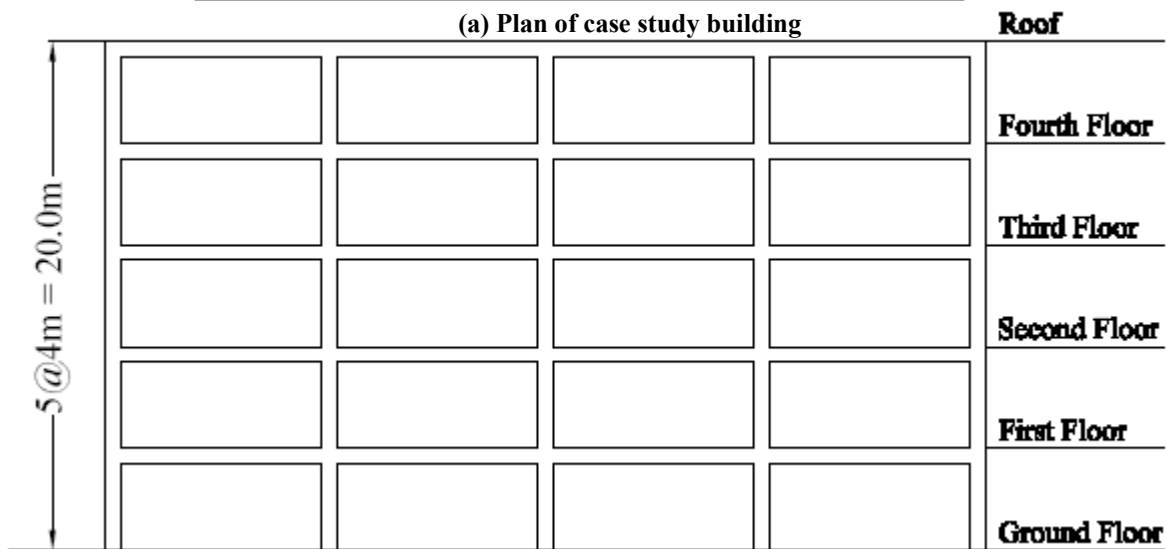


Figure 1: Proposed retrofit techniques



(a) Plan of case study building



(b) Elevation view of case study building

Figure 2: Case study building

Design load requirements are based on *American Society of Civil Engineers Standards: "Minimum Design Loads for Buildings and Other Structures" ASCE 7-05*. Wind load is based on *Uniform Building Code* provisions "UBC 97". The self-weight of reinforced concrete is assumed to be  $24.0 \text{ kN/m}^3$  and the compressive strength is  $30.0 \text{ MPa}$ , a partition loading of  $1.0 \text{ kN/m}^2$  is considered as an area load. For the exterior frames, a cladding loading of  $0.75 \text{ kN/m}^2$  is applied to each perimeter beam as a uniform load based on the vertical tributary area. The design live load for this office building is  $2.4 \text{ kN/m}^2$  on each floor. The roof live load is calculated as the larger value of the roof load and snow load. The wind load is applied as a uniform load distributed vertically on the windward and leeward sides of the building and horizontally on the building's roof. The yielding strength of the reinforcing steel is  $420 \text{ MPa}$  (Grade 60).

- **Modeling of the Case Study Building**

Structural analysis of the building is conducted using SAP 2000, V 14.2.2. Beams and columns are designed based on the results of structural analysis using the non-seismic factored load combinations listed by the ACI 318.

The perimeter beams are  $550 \text{ mm}$  wide by  $700 \text{ mm}$  deep. The two-way solid slab is  $230 \text{ mm}$  thick. The columns are  $700 \text{ mm} \times 700 \text{ mm}$  for the ground and 1<sup>st</sup> floors and  $550 \text{ mm} \times 550 \text{ mm}$  for second, third and fourth floors. Figure 3 shows the reinforcement in the column, beam and slab sections.

### **Retrofitting I: Jacketing**

The first retrofit strategy consists of adding RC jackets. This method is selected to strengthen the columns in order to meet the applied seismic demand for the case study building. Hence the size of the RC jackets and the amount of reinforcement are determined based on seismic demand for the original case study building as shown in Figure 4.

### **Retrofitting II: Bracing**

In eccentrically braced frames (EBFs), forces are

transferred to the brace members through bending and shear forces developed in the ductile steel link. The link is designed to act as a fuse by yielding and dissipating energy while preventing buckling of the brace members.

In RC frames, the concrete beams are incapable of performing as a ductile link for the steel bracing system that is inserted in the frame bays. A vertical steel shear link may be introduced by the Y- bracing pattern shown in Figure 5. The vertical shear link is attached to the beam of the RC frame. This connection should have sufficient capacity to ensure effective transmission of forces when subjected to seismic loads. Figure 5 shows details of the link connection of the eccentric bracing system inserted in the exterior bays of the RC frame building. The link connection is located at mid-span of the RC beam, and it is connected to steel plates which are anchored to the RC beam. The force in the brace is transmitted to the shear link using an end plate.

The vertical shear link is assumed to act as a cantilever. The brace members are assumed to be pin-connected to the vertical link while the link itself is considered fixed to the RC beam. The brace members provide negligible constraint to the link end against rotation. The critical length of a vertical cantilever shear link is half the length of a steel link that has fixed connections at both ends with reverse curvature and equal end moments. The critical length,  $e_{crit}$ , is defined as follows:

$$e_{crit} = \frac{M_u}{V_u} \quad \text{Eq. (1)}$$

where  $e_{crit}$  is the maximum cantilever shear link length,  $M_u$  and  $V_u$  are the ultimate end moment and shear force for a link with a well stiffened web.

Based on experimental data (Ghobarah and Abou Elfath, 2000), the formula for calculating the length of a cantilever link to ensure that the link yields primarily in shear is:

$$e_{crit} = \frac{2b_f t_f}{t_w} \quad \text{Eq. (2)}$$

where  $b_f$  and  $t_f$  are the width and thickness of the flange and  $t_w$  is the web thickness of a wide flange section link.

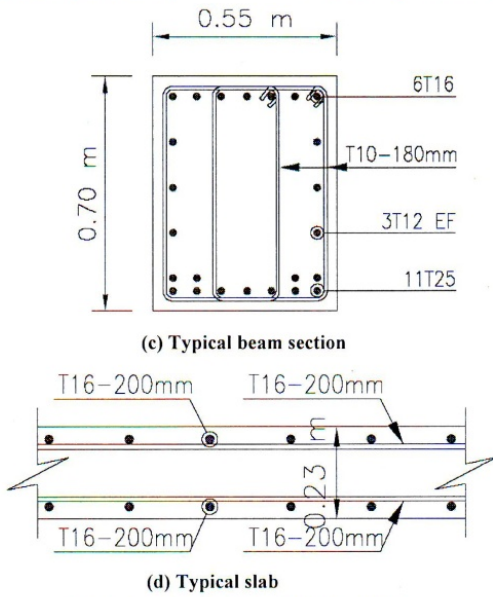
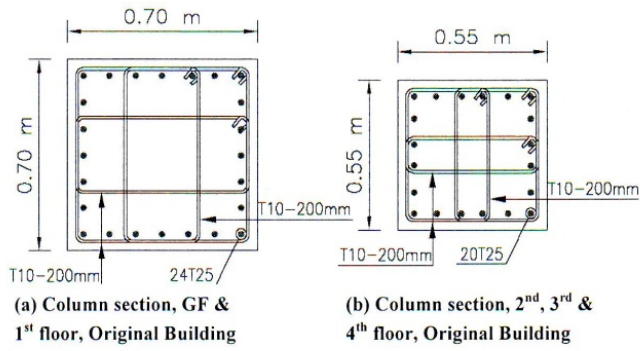


Figure 3: Original building sections details

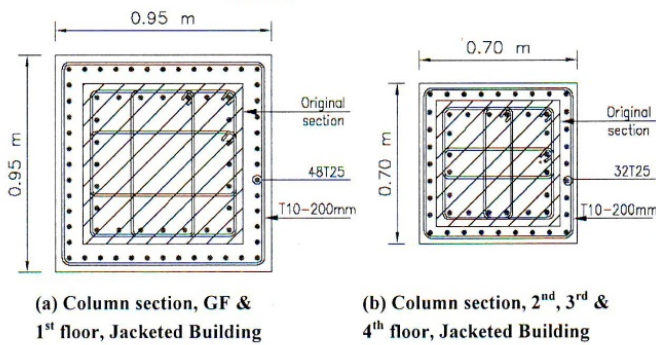


Figure 4: Jacketed building column

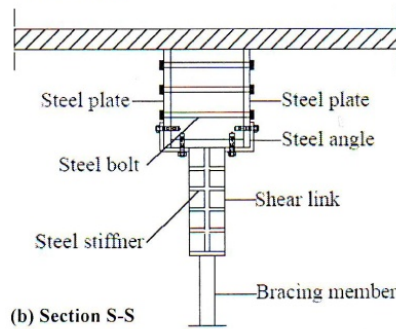
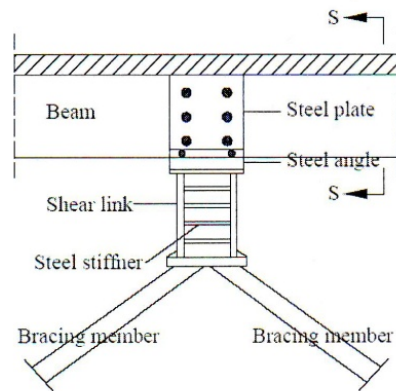


Figure 5: Connection details of the vertical steel link

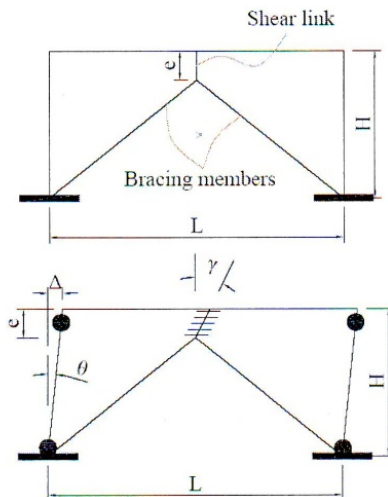


Figure 6: Plastic hinge mechanism of a reinforced concrete frame rehabilitated using a vertical shear link

The maximum link deformation angle  $\gamma_{max}$ , shown in Figure 6, that can be achieved by the shear link, is dependent on the link detailing. The link deformation angle is defined as the link lateral displacement over the link length for both single and double curvature cases. Shear links with closely spaced web stiffeners exhibit relatively large ultimate deformation angles under the effect of cyclic loading. The ultimate link deformation angle is defined as the maximum deformation angle developed by the link before the occurrence of considerable strength deterioration due to severe flange and web buckling of the link. It was found that the ultimate link deformation angle ( $\gamma$ ) for well stiffened shear links may approach 0.1 rad (Ghobarah and Abou Elfath, 2000).

Steel brace members in EBFs should be designed to ensure that they will behave elastically when subjected to an earthquake loading. The brace member should be designed as a compression member with its axial load capacity depending on the plastic strength of the steel link.

Figure 6 shows a plastic mechanism of an eccentrically braced reinforced concrete frame provided with a vertical steel link. In this figure, small solid circles indicate plastic flexural hinges and the cross-hatched lines indicate a plastic shear hinge. The storey displacement is denoted  $\Delta$ , where  $H$  is the storey height,  $L$  is the frame span,  $e$  is the link length,  $\theta$  is the storey drift angle ( $\theta = \Delta/H$ ) and  $\gamma$  is the link deformation angle. For the RC frame shown in Figure 6, the deformation angle of the link can be estimated approximately as  $\gamma = \theta(H/e)$ . In EBFs,  $e$  is normally much smaller than  $L$  and  $H$  and therefore severe deformation demands are placed on the link.

For the case of the reinforced concrete frame shown in Figure 6, the maximum storey drift angle,  $\theta_{max}$ , is calculated as:

$$\theta_{max} = \frac{e_{crit}\gamma_{all}}{H} \quad \text{Eq. (3)}$$

where  $\gamma_{all}$  is the allowable link deformation angle and  $e_{crit}$  is the maximum shear link length calculated using Eqs. (1) and (2).

### Shear Link Model

Steel links are subjected to high levels of shear forces and bending moments in the active link regions. In the analysis of the performance of links, elastic and inelastic deformations of both shear and flexural behaviors have to be taken into consideration. Ghobarah et al. (2000) modeled the link as a linear beam element with six non-linear rotational and translational springs at each end. Three rotational bilinear springs were used to represent the flexural inelastic behavior of the plastic hinge at the link end represented by the multilinear function shown in Figure 7. Three translational bilinear springs are used to represent the inelastic shear behavior of the link web represented by the multilinear function shown in Figure 8.

The values of  $M_y$  and  $V_y$  are considered equal to  $M_p$  and  $0.9V_p$ , respectively (Ghobarah et al., 2000), where the moment-rotation and the shear force-lateral displacement relationships of the steel link shown in Figures 7 and 8 are given as:

$$\begin{aligned} V_{y1} &= V_y & M_{y1} &= M_y \\ V_{y2} &= 1.06 V_y & M_{y2} &= 1.03 M_y \\ V_{y3} &= 1.12 V_y & M_{y3} &= 1.06 M_y \end{aligned} \quad \text{Eq. (4)}$$

The values of the stiffness are:

$$\begin{aligned} K_{2v} &= 0.03 K_{1v} & K_{2M} &= 0.03 K_{1M} \\ K_{3v} &= 0.015 K_{1v} & K_{3M} &= 0.015 K_{1M} \\ K_{4v} &= 0.002 K_{1v} & K_{4M} &= 0.002 K_{1M} \end{aligned} \quad \text{Eq. (5)}$$

The values of  $K_{1M}$  and  $K_{1v}$  can be calculated as:

$$K_{1M} = 3EI/e \quad \text{Eq. (6)}$$

$$K_{1v} = GA_{web}/e \quad \text{Eq. (7)}$$

where  $E$  is Young's modulus of steel,  $I$  is the moment of inertia of the link cross-section,  $G$  is the modulus of rigidity of steel and  $A_{web}$  is the area of the web of the link section.

The properties of the shear link and the brace members are calculated using Young's modulus,  $E = 200,000$  MPa, and steel yield stress,  $f_y = 350$  MPa. The

vertical steel link section (W 200x46) is selected to satisfy Eq. (2). The brace members are selected as

round hollow sections (HCS 168X12),  $r = 55.4$  mm and  $KL/r = 100.5$ .

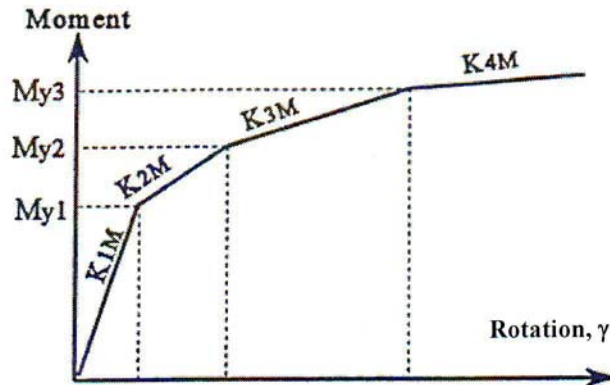


Figure 7: Moment-rotation relationship of a steel link (Ghobarah and Abou Elfath, 2000)

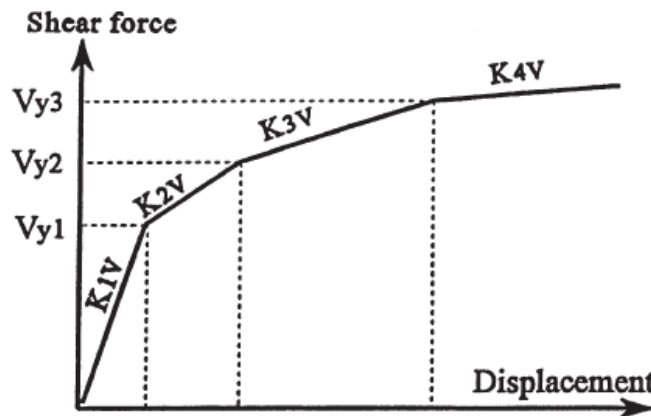


Figure 8: Shear force-shear deformation relationship of a steel link (Ghobarah and Abou Elfath, 2000)

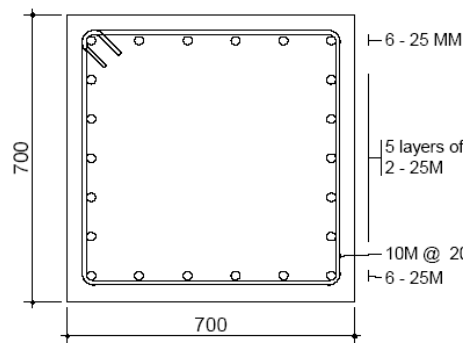


Figure 9: Column section properties in RESPONSE 2000 (Response output image)

**Plastic Hinge**

In order to perform pushover analysis for the original and the retrofitted buildings, capacity analysis is performed for each original and jacketed section. To model the capacity curve for each section, RESPONSE 2000 is used. Capacity curve for four values of axial

loads (2000, 500, 0, -1500 kN) with a moment increment is modeled for each column, only a moment increment capacity curve is modeled for beams.

An example for capacity analysis is stated here; a full plastic hinge analysis for the 700x700 column is shown next. The details of the column section as modeled in RESPONSE 2000 are shown in Figure 9.

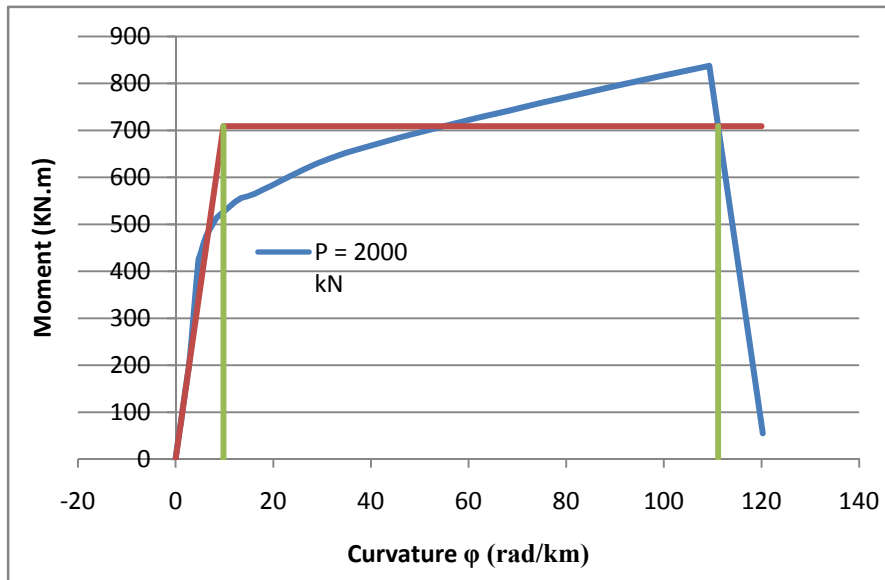


Figure 10: Column capacity curve for P=2000 kN

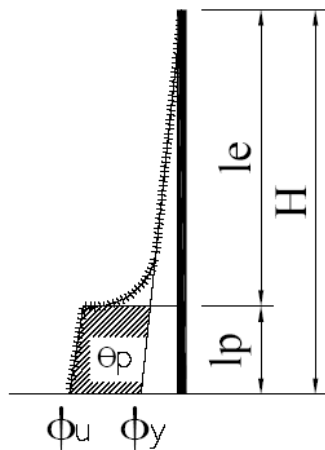


Figure 11: Distribution of curvature along the length of a cantilever member (single curvature)

By solving for each axial load, an exact moment-curvature curve is plotted. To extract the values of

yielding and ultimate curvatures, an approximate bilinear curve is established such that the extracted and



added energies are equal, as shown in Figure 10, where  $\phi_y = 9.79 \text{ rad/km}$ ,  $\phi_u = 111.12 \text{ rad/km}$  and  $M_u = 709.2 \text{ kN.m}$ .

The plastic hinge properties for the column section are calculated to be properly modeled in SAP 2000. Other properties such as  $l_p$ ,  $l_e$ ,  $\theta_y$  and  $\theta_u$  are explained and calculated below.

Referring to (Armouti, 2008), the plastic hinge length

$$l_p = 0.5 d + 0.05 Z \quad \text{Eq. (8)}$$

$$= 0.5 * 635 + 0.05 * 4000 / 2$$

$$= 517.5 \text{ mm}$$

where  $l_p$  is the plastic hinge length,  $d$  is the column section depth and  $Z$  is the shear span; i.e., the distance from critical section to the point of contraflexure.

Figure 11 illustrates the plastic hinge concept, where the plastic rotation  $\theta_p$  of a plastic hinge is given by the relationship  $\theta_p = (\phi_u - \phi_y) l_p$  (Makarios, 2005).

In the case study building, the columns are under double curvature, as shown in Figure 12, where:

$$l_e = L - 2 * l_p = 4000 - 2 * 517.5 = 2965 \text{ mm}$$

To calculate the yielding rotation, (Armouti, 2008) suggested that:

$$\theta_y = \phi_y * l_e / 2 \quad \text{Eq. (9)}$$

$$= 9.79 * 2965 * 10^{-6} / 2 = 0.0145 \text{ rad}$$

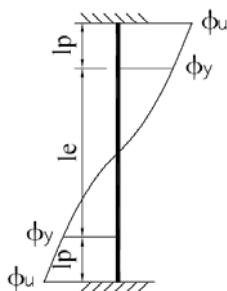


Figure 12: Plastic hinge curvatures and length for a column subjected to double curvature action

and to calculate the ultimate rotation, a formula will be extracted from Table 1, which is suggested by (Inel et al., 2004).

By referring to the simple model, it can be noticed that:

$$\theta_u = \Delta_u / L, \text{ and } \Delta_u = \Delta_y + \Delta_p \quad \text{Eqs. (10 and 11)}$$

$$\Delta_p = \theta_p(L - 0.5 L_p), \text{ and } \theta_p = (\phi_u - \phi_y)L_p \quad \text{Eqs. (12 and 13)}$$

By solving for  $\theta_u$ :

$$\theta_u = [(\phi_u - \phi_y)(l_p - 0.5 l_p/L)] + \phi_y L / 3 \quad \text{Eq. (14)}$$

$$\theta_u = 0.062 \text{ rad}$$

By referring to Table 6-8 in FEMA 365, the acceptance criteria for the section can be obtained. Accordingly, number of values can be calculated:

$$\frac{P}{A_g f'_c} = 0.185 \quad \frac{V}{b_w d \sqrt{f'_c}} = 0.0251$$

By interpolation, the acceptance criteria, in terms of rotation, become:

IO (rad)	LS (rad)	CP (rad)
0.004	0.014	0.019

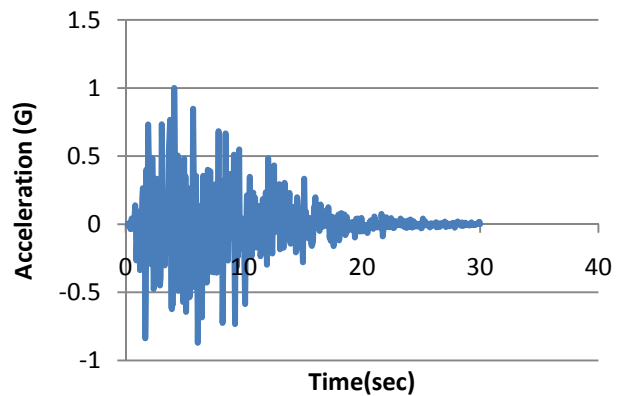


Figure 13: Ground motion record R2.nsa

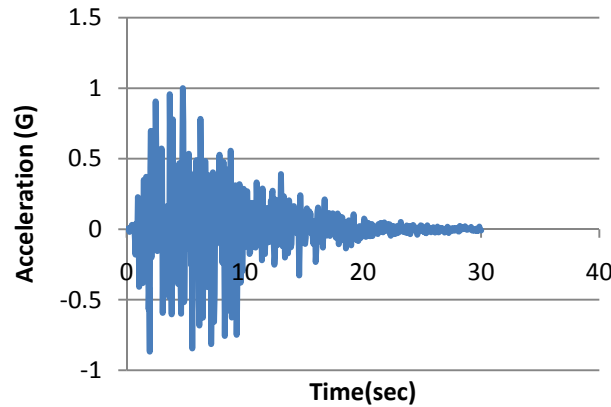


Figure 14: Ground motion record R3.nsa

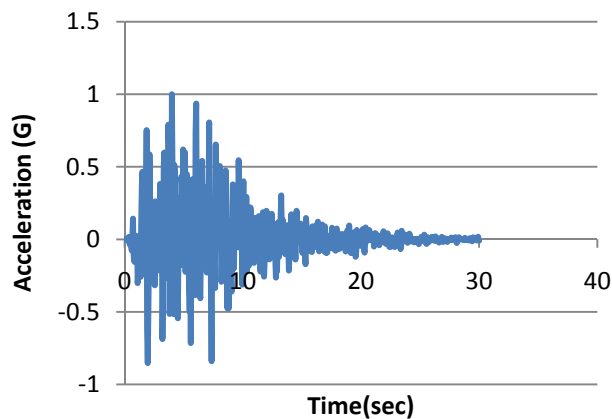


Figure 15: Ground motion record R4.nsa

### Ground Motion Records

Due to scarcity of actual earthquake records representative of various site conditions, synthetic records are accepted as an alternative to actual records (Armouti, 2003). The model building in this study is subjected to a set of three ground motion records R2.nsa, R3.nsa and R4.nsa, which are representative of records found in rock sites. Figure 13 to Figure 15 show the accelerogram of these records which are synthetic records adapted from (Armouti, 2003).

Extension to other sites might be studied in future work. The ground motion record that induced the maximum building response is adapted as the time-history function defined in SAP 2000, which is R3.nsa in this case.

### RESULTS AND DISCUSSION

- Dynamic Analysis

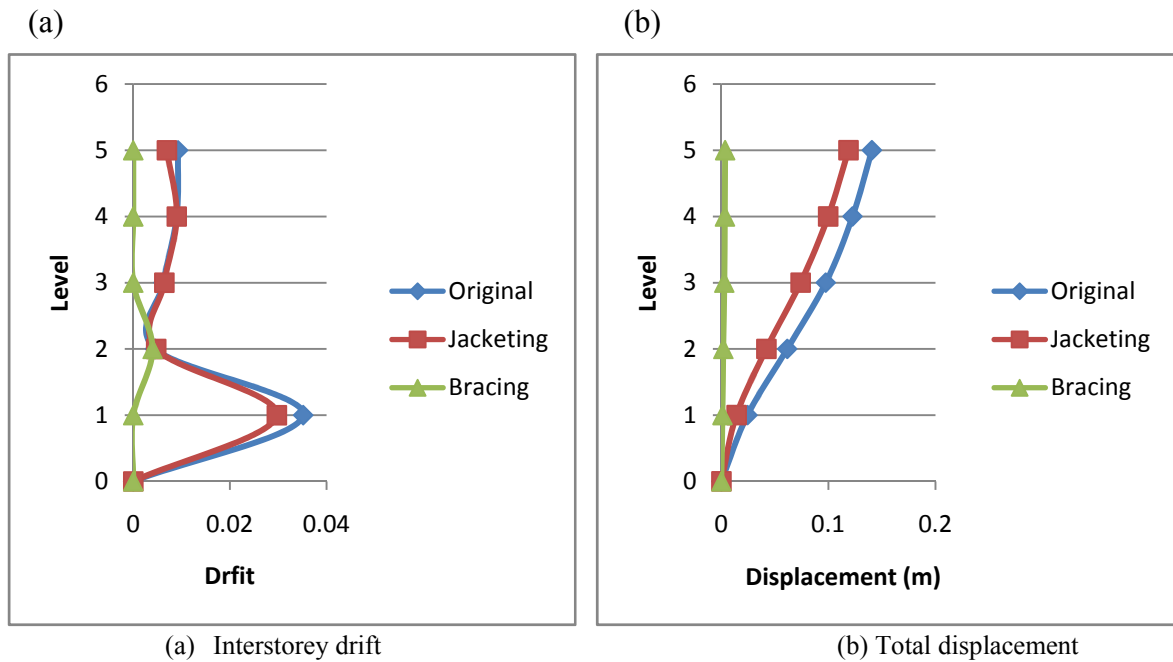
The dynamic behavior of the retrofitted case study building is investigated using the maximum of the three computer generated earthquake records, which is R3.nsa. As shown in Figure 16, for the shear link bracing retrofit, the drift of the first storey is substantially reduced due to the complete change in the lateral support system from the RC columns to shear links. In addition, a shift in the maximum drift location occurred, which is shifted from the first floor in the original building to the second floor in the braced building. This shift can be explained by looking at the bracing layout in Figure 23, which shows that the first floor is enhanced by adding a brace to each bay of that

floor; hence, the lateral stiffness of that storey is considerably increased.

**Table 1. Definition of deformation indices using available models**

Deformation Index	Simple Model	Lehman Model	Fardis Analytical Model	Fardis Empirical Model	Priestley Model
$\Delta_{y,flexure}$	$\phi_y L^2/3$	$\phi_y L^2/3$	$\phi_y L^2/3$	$\phi_y L^2/3$	$\phi_y(L+0.15f_y d_b)^2/3$
$\Delta_{y,shear}$	NA	$V_y L/(0.4E_{c,sec}0.8A_g)$	0.00275L	0.00275L	$\Delta_{conc, shear} + \Delta_{truss, shear}^{(1)}$
$\Delta_{y,slip}$	NA	$\phi_y L f_y d_b / 8 v f_c'$	$\epsilon_y f_y d_b L / (5 f_c' (d - d'))$	$\epsilon_y f_y d_b L / (5 f_c' (d - d'))$	Included in $\Delta_{y,flexure}$
$\theta_p$	$(\phi_u - \phi_y) L_p$	$(\phi_u - \phi_y) L_p$	$(\phi_u - \phi_y) L_p^{(2)}$	$\theta_u - \Delta_y / L$	$(\phi_u - \phi_y) L_p$
$\theta_u$	$\Delta_u / L$	$\Delta_u / L$	$\Delta_u / L$	$^{(1)}$	$\Delta_u / L$
$\Delta_p$	$\theta_p (L - 0.5 L_p)$	$\theta_p (L - 0.5 L_p)$	$\theta_p (L - 0.5 L_p)$	$\Delta_u - \Delta_y$	$\theta_p (L - 0.5 L_p)$
$\Delta_u$	$\Delta_y + \Delta_p$	$\Delta_y + \Delta_p$	$\Delta_y + \Delta_p$	$\theta_u L$	$\Delta_y + \Delta_p$
$L_p$	0.5H	$0.5L(M_u - M_n) / M_n + 1.2(f_u - f_y) * d_b / 4 v f_c'$	$0.026L + 0.13H + 0.02 f_y d_b$	NA	$0.08L + 0.022 f_y d_b$

<sup>(1)</sup> Details for these values can be found in [Inel, 2004]



**Figure 16: Comparison between maximum deformation of retrofit schemes**

The maximum interstorey drift for the RC column jacketing retrofit is also slightly reduced, at the first, second and fifth stories; however, for the third and fourth stories the maximum interstorey drift increased slightly, and that is where the column section (and the jacketing section) are reduced.

By looking at the overall drift profiles, a similarity between the shape of the original structure drift profile and the jacketed building drift profile is noticed. In addition, both of them exceed the 0.02 drift limit (0.035 for original and 0.029 for jacketed), while the bracing reduced the maximum structure drift to be 0.005.

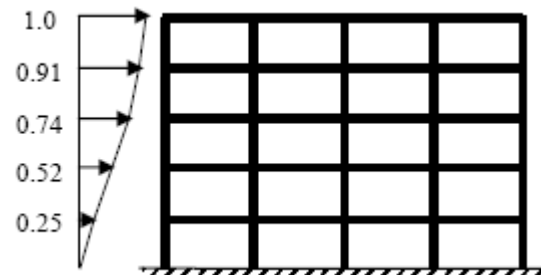
Table 2 shows the maximum link deformation angle,  $\gamma$ , for the bracing case, reached due to the application of R3.nsa. The ratio between the maximum link deformation angle and the storey drift angle,  $\theta$ , is calculated and is presented in the table. The analytical ratio is found to be close to the ratio calculated by Eq. (3) ( $\gamma/\theta = H/e = 6.67$ ). This indicates that the ratio can be approximately considered independent of the loading condition. The difference between the  $\gamma/\theta$  ratio calculated using Eq. (3) and the values obtained from the dynamic analysis is because the axial deformations of the brace member and the RC frame members are neglected in the derivation of Eq. (3).

**Table 2. Shear link deformation angle**

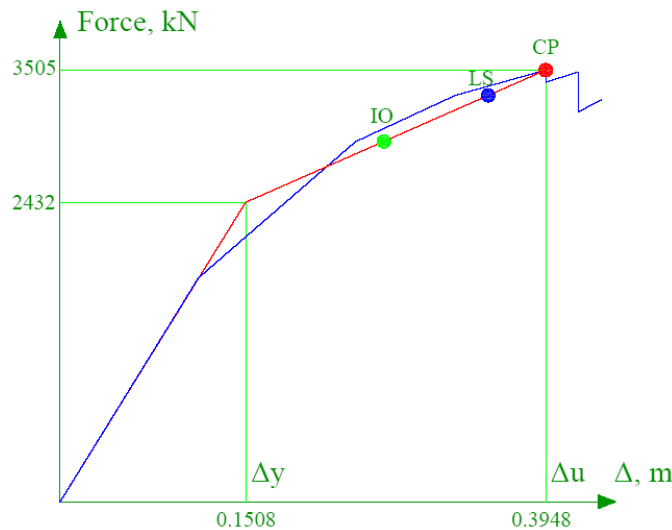
Theoretical		Analytical	
$\gamma$ (rad)	$\gamma/\theta$	$\gamma$ (rad)	$\gamma/\theta$
0.1	6.7	0.03	6.0

• Pushover Analysis

Pushover analysis results' evaluation is considered as member-level evaluation for the structure. It is conducted with an inverted triangular unit load pattern for the three structures. The inverted triangular load pattern is based on the first mode shape from an eigenvalue analysis for the original structure. Figure 17 shows the applied load pattern.



**Figure 17: Inverted triangle load pattern for pushover analysis**



**Figure 18: Pushover curve for the original structure**

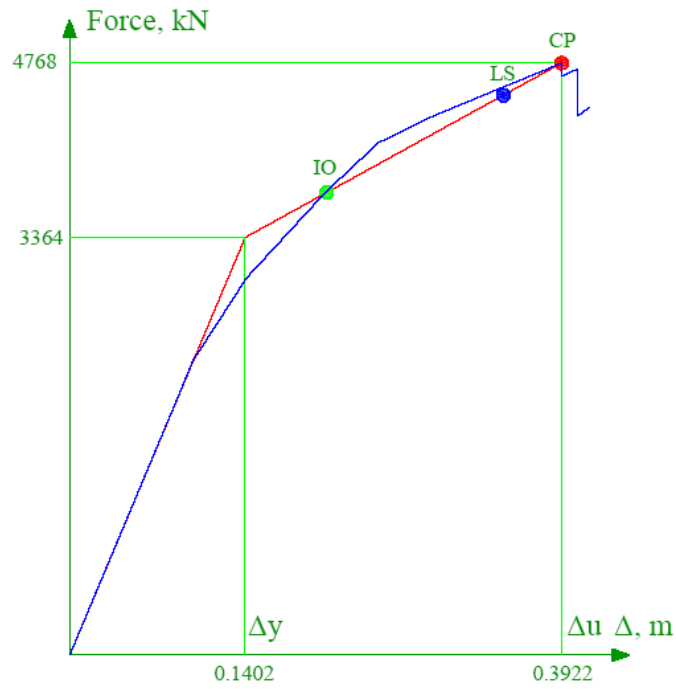


Figure 19: Pushover curve for the column jacketing structure

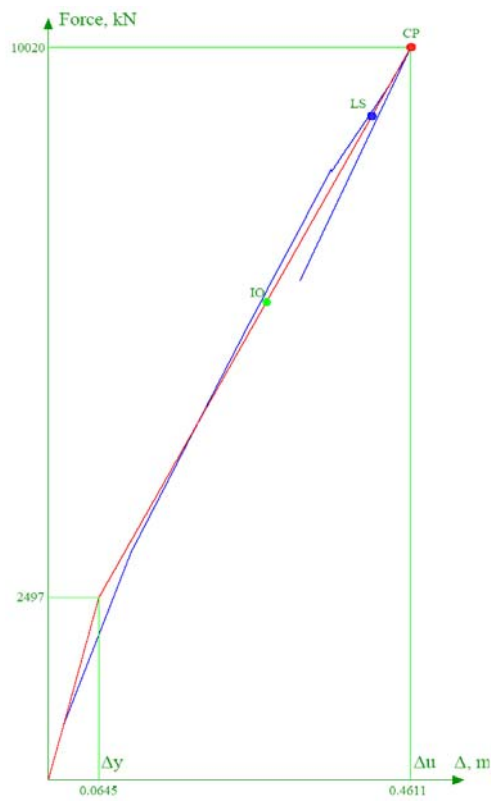


Figure 20: Pushover curve for the braced structure

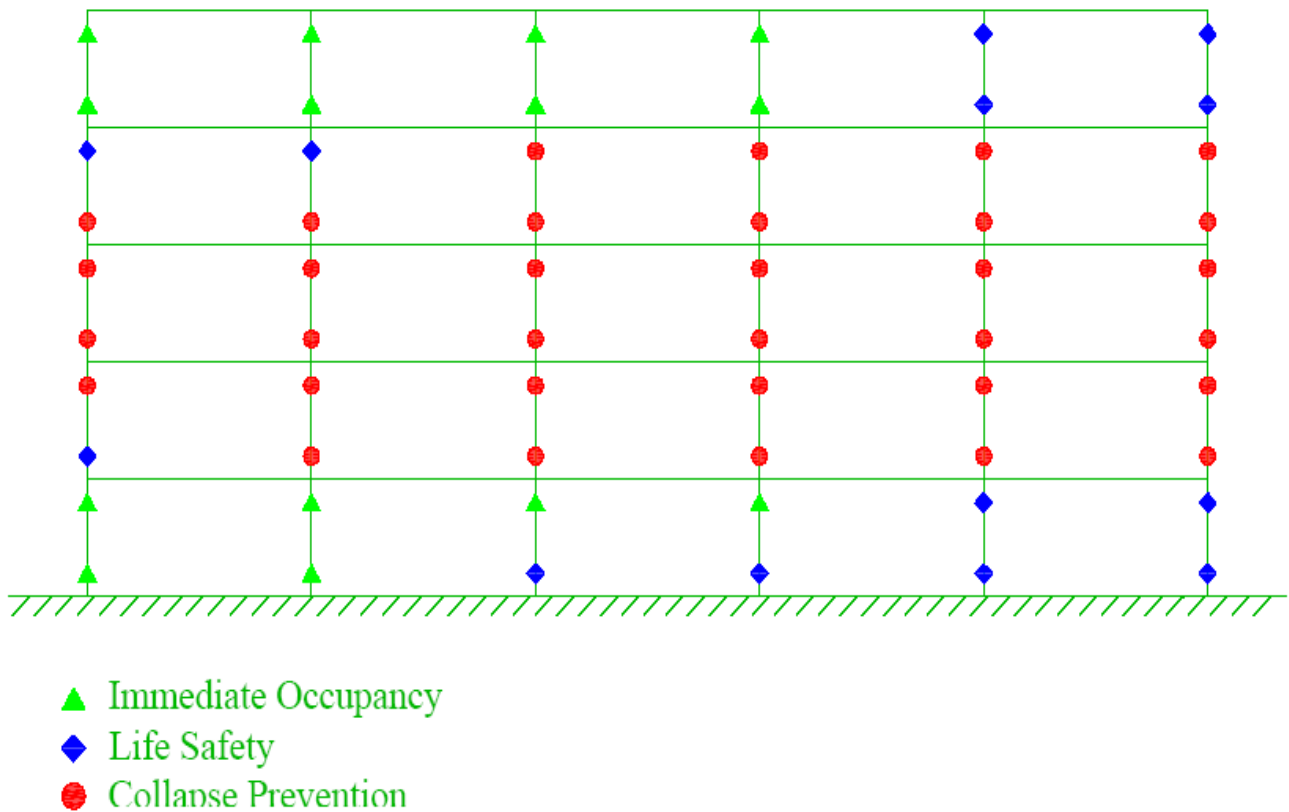


Figure 21: Locations where plastic hinge limits are exceeded for the original structure

Figures 18 to 20 show how the base shear is related to the building displacement for each retrofit case. Ductility enhancement is measured for both techniques using ductility capacity,  $\mu_c$ , as a parameter, where (Armouti, 2008):

$$\mu_c = \Delta_u / \Delta_y \quad \text{Eq. (15)}$$

for the original structure:  $\mu_c = 2.6$

for the jacketed structure:  $\mu_c = 2.8$

for the braced structure:  $\mu_c = 7.15$

Even though, both techniques provide a ductility enhancement. It can be seen that it is a negligible enhancement for column jacketing, while it is a large enhancement for frame bracing (almost three times the original ductility).

Column jacketing provides a 36% increase of the maximum ultimate force compared to the original

structure; while bracing provides a 185% increase in the ultimate force. Hence, both techniques provide a strength enhancement.

Elastic stiffness for each of the three structures, which is designated by the letter K, is the slope of the elastic part of the pushover curve. The value of K for each structure is:

$$K_{\text{original}} = 2432 / 0.1508 = 16127 \text{ kN/m}$$

$$K_{\text{jacketing}} = 3364 / 0.1402 = 23994 \text{ kN/m}$$

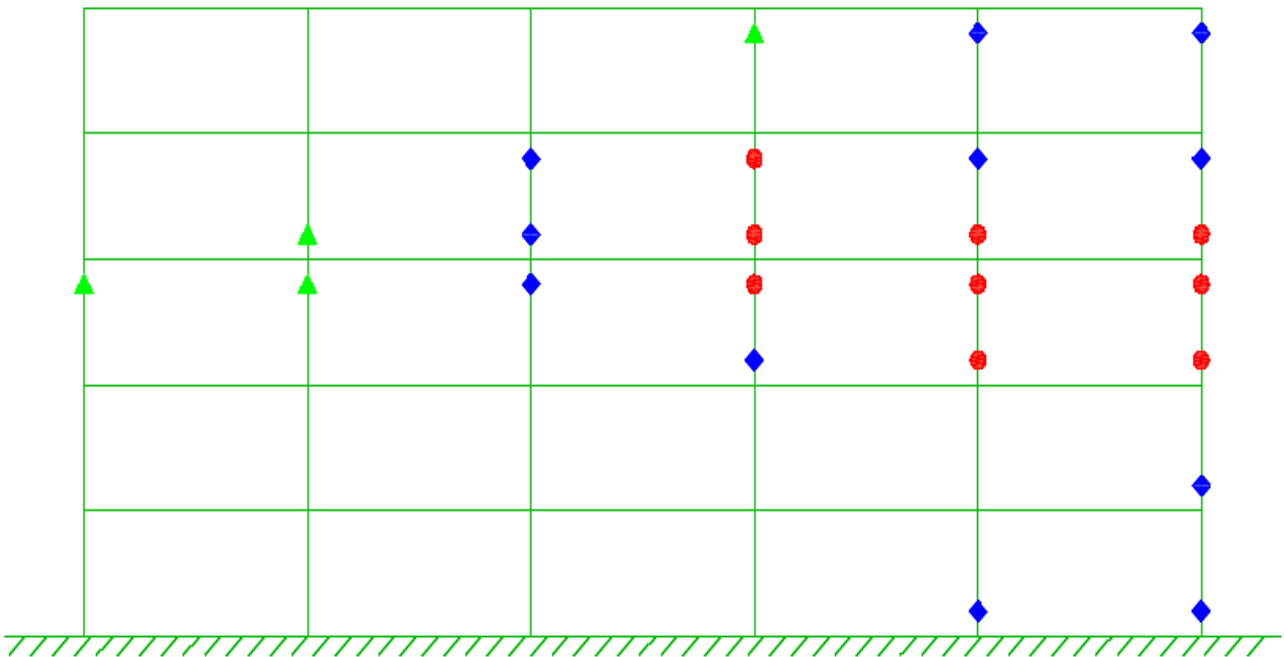
$$K_{\text{bracing}} = 2497 / 0.0645 = 38713 \text{ kN/m}$$

It can be noticed that jacketing enhances the stiffness by 49% and that bracing enhanced the stiffness by 140%.

Figures 21 to 23 show the locations of inelastic behavior in the unretrofitted structure and retrofitted structures where the plastic rotations exceed the Basic

Safety Objectives (BSO) limit for IO, LS and CP performance levels under the applied ground motion. Locations where rotations exceed the FEMA 356 member-level criteria for each limit state are shown with different geometric shapes. Although the retrofits do not meet the FEMA 356 criteria for some members, these figures demonstrate the relative improvement after applying each retrofit technique.

In the original structure, Figure 21, plastic hinges formed in all columns, where 55% of the plastic hinges exceed the CP limit because of insufficient column ductility, 21.5% exceed the LS limit because of insufficient column strength and 23.5% exceed the IO limit because of insufficient column stiffness (Beth et al., 2007).



**Figure 22: Locations where plastic hinge limits are exceeded for the column jacketing retrofitted structure**

Column jacketing, Figure 22, provides enough strength and stiffness to eliminate some of the LS hinges and most of the IO hinges, but does not gain enough ductility to eliminate most of the CP hinges. Adding an eccentric shear link bracing to the structure, Figure 23, eliminates most of the CP hinges (ductile behavior), some IO and LS hinges are still exceeded, but in a lower percentage than for jacketing.

### Cost Analysis

A cost comparison between the two retrofitting schemes is held, which is an additional aspect that should be considered in any construction process, in

order to select an appropriate feasible retrofitting scheme.

- Material cost for both schemes is calculated and compared in Table 3. The final costs show that the column jacketing materials' cost is almost five times that of bracing materials. Notice that jacketing is placed on all columns, whereas bracing is induced only in outer frames.
- In bracing scheme, only eighteen footings have to be resized (the perimeter footings), which are the footings attached to the columns supporting the bracing members. In column jacketing, all of the thirty footings have to be resized due to the axial

- load and moment altered values for each column.
- In bracing scheme, only the outer frames need to be worked on, which is more practical than the column jacketing retrofit, in which all columns in all floors need to be roughened, surrounded by the formwork and cast. In bracing scheme, the brace members

need to be attached to the concrete frame at three points, where appropriate Hilti chemicals and bolts can be used as shown in Figure 24, and that means less work done, less time and less labor to do the work.

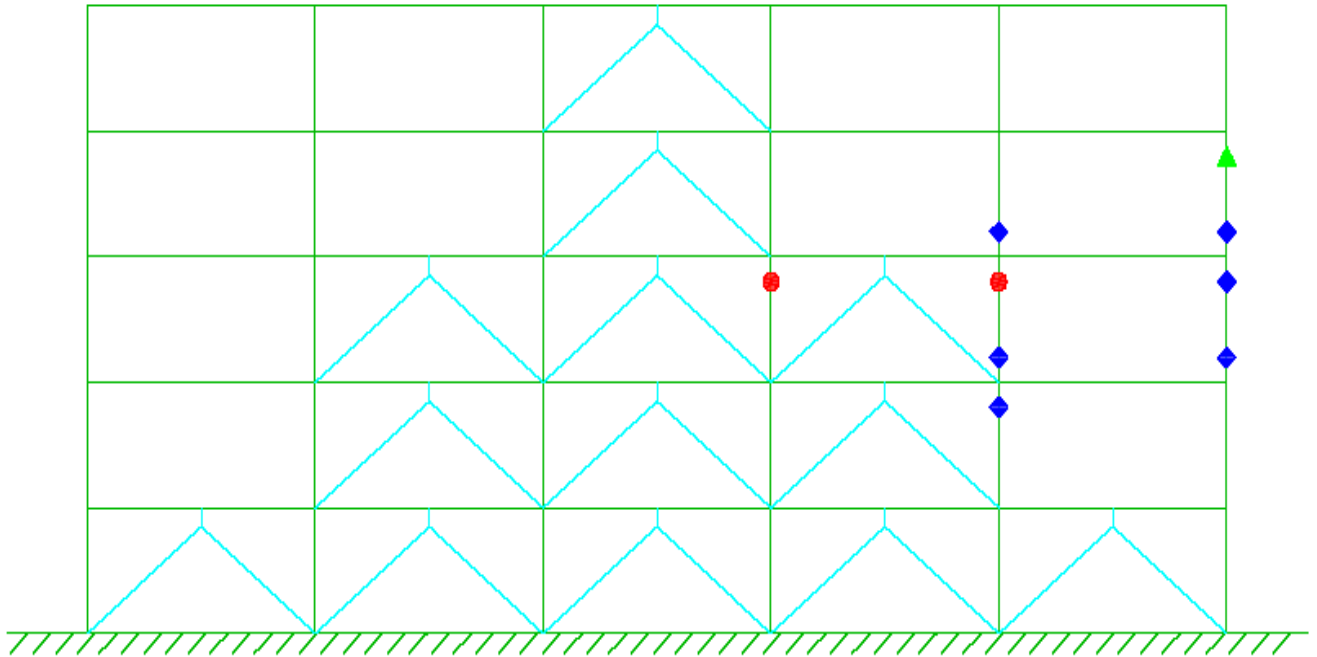
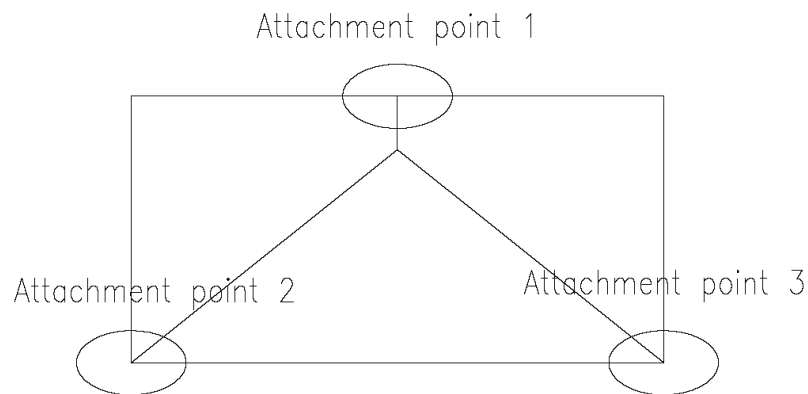


Figure 23: Locations where plastic hinge limits are exceeded for the eccentric brace retrofitted structure

Table 3. Material cost comparison for both retrofitting techniques

Item	Jacketing	Bracing
Concrete (m <sup>3</sup> )	166.5	-
Item cost (USD/m <sup>3</sup> )	80.0	-
Cost (USD)	13320.0	-
Reinforcing bars (kN)	888.0	-
Item cost (USD/ kN)	70.0	-
Cost (USD)	62157.3	-
Structural steel (kN)	-	172.2
Item cost (USD/ kN)	-	78.0
Cost (USD)	-	13428.1
<b>Total costs (USD)</b>	<b>75477.3</b>	<b>13428.1</b>





**Figure 24: Eccentric shear link bracing is attached to the concrete frame at points 1, 2 and 3**

### CONCLUSIONS

The following conclusions can be drawn from the results of this study:

1. When designed to meet the minimum safety requirements for the seismic demand, eccentric shear link bracing brings the drift value below its limit and eliminates almost all the CP plastic hinges, while column jacketing does not satisfy the drift criteria and very small number of CP plastic hinges.
2. Both retrofit techniques affect the structure stiffness. In bracing scheme, the main effect is on the lower stories, in jacketing the effect is on the entire structure.
3. Eccentric shear link bracing enhances the structure ductility and strength more than column jacketing.
4. In any structure, if a large drift value is located in a certain storey, bracing might be concentrated for that storey to affect that specific drift value.
5. Column jacketing increases the overall building stiffness, but that is accompanied by an increase in the seismic forces induced in the building, which means that a change in the stiffness of the structure does not necessarily affect the drift values, but will also affect the displacements and the overall lateral forces.
6. Cost and time of installment of the shear link bracing are lower than for column jacketing by all means. This is an important advantage for bracing retrofit.

In this study, eccentric shear link bracing turns out to be the better choice for retrofitting mid-rise office buildings. It provides a better performance in both dynamic and static nonlinear analysis and a better value costwise. While still in Jordan column jacketing is the default method of retrofitting, a steel bracing should be considered and studied in the future for structures that need seismic retrofitting.

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