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The use of steel X-bracing combined with shear link to seismically upgrading reinforced concrete frames

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Abstract

This study examines the feasibility of utilizing steel concentrically X-bracing rigidly connected to shear link to retrofit seismically defected reinforced concrete (RC) frames. It involves examining the lateral behavior of two hypothetical frames strengthened using this proposed system, a single story-single bay RC frame, and a five-story RC office building. For the former, both elastic and inelastic push-over analyses were performed to examine the lateral behavior of strengthened and original RC frames, while the original and strengthened five-story frames were analyzed under the action of monotonically increasing lateral loads and nonlinear time histories. In detailing the strengthening scheme, procedure complying with the capacity design concept was adopted whereby the shear links were detailed to yield prior to RC frames structural elements to reduce the demand on them. The stiffnesses, strengths, and plastic hinging formation patterns for original and strengthened frames under the action of different loading conditions were determined and evaluated. Story shears and maximum story displacements were recorded and examined. It was concluded that this system is feasible for seismically strengthening the defected RC frames. Guidelines for detailing this scheme were provided within the context of this research study.

Keywords: Concrete gravity designed frames, Shear links, X-bracing, Seismic strengthening, Push-over analysis, Time histories

Introduction

Seismically defected reinforced concrete (RC) frame structures designed without ductile detailing represent a considerable hazard when experiencing medium or severe earthquakes. They develop brittle failure due to (1) inadequate transverse reinforcement within the critical regions of beams and columns, (2) absence of hoops for shear into the joint panel areas, (3) lack of confinement for the columns laps splice areas, and (4) sliding of longitudinal reinforcement bars of beams in joint areas [1–4].

In the past decade, various types of intervention were proposed to improve seismic behavior of these defected frames [5]. These interventions were classified into two groups. The first group involves the conventional intervention techniques that aim towards enhancing the strength, stiffness, or ductility of the structures by adding RC shear walls or jacketing the columns and beams of the RC frames. In spite that these techniques have the advantages of being easily designed and executed using traditional construction methods, they have some technical and practical disadvantages such as increasing the structures stiffness and weight leading to attraction of higher seismic forces. Also, they involve heavy demolition and construction works which will halt the building from functioning for long time. The second group involves innovative intervention techniques that aimed at alleviating the effect of seismic forces on structures such as increasing buildings' ductility without significantly affecting their strengths or stiffnesses, reducing building stiffnesses, controlling buildings damages, or the utilization of any suitable combination of these indicated methods [6].

One of those innovative techniques is the introduction of "damage-controlled buildings" [7]. In this technique, the retrofitted structure is equipped with auxiliary structural system parallel to the original non-ductile structural systems. This auxiliary system possesses both high stiffness, with minimum weight, to control building lateral deformation without increasing its mass, but provides the capability of dissipating severe earthquake input energy through inelastic action. Consequently, damage in this case confines only to the auxiliary structure, while the original system behaves almost elastically under severe earthquakes.

Steel bracing is considered one of those damage-controlled techniques. It is preferred for seismic upgrading of RC frames due to its high strength to weight ratio, simplicity in construction with minimal disruption to occupants, and alteration to buildings configuration and functions and rapid construction [8–15]. Retrofitted frames with concentric braces (CBF) shown on Fig. 1 resist lateral loads through truss action in which braces are subjected to tension and compression. They provide high levels of monotonic stiffness and strength,however, they often exhibit strength and stiffness degradation during their plastic cyclic response due to buckling of braces in compression [16]. To overcome this defect, two other load lateral resisting systems were proposed. The first is the use of buckling restraint braces instead of traditional braces. These braces yield in both tension and compression, but without brace buckling [8]. The other system is employing eccentric bracing (EBF) [17]. In this system, frames respond to earthquake loads through a mix of truss and flexure actions in system. Consequently, this system combines the advantages of braced frames and moment resisting frames with shear links being the

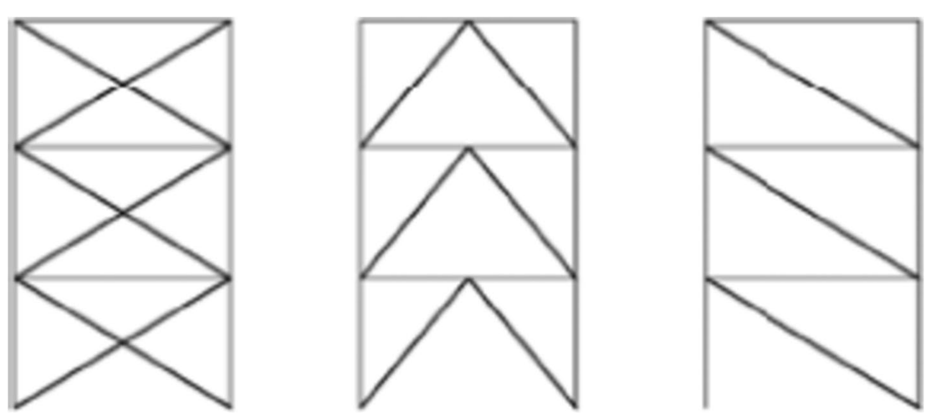

Fig. 1 Concentric steel bracing system

sacrificial elements in this system [18, 19]. However, EBF links are either currently placed as shown in Fig. 2a within the floor beams which, in case of defected RC frames, are incapable of performing as ductile links and consequently need to be jacketed [20] or placed as vertical link attached to the floor beams mid spans utilizing “inverted Y” bracing system as shown in Fig. 2b which requires strengthening beams to sustain expected excessive moments [21].

Another comparable system was the utilization of concentrically braced frames combined with shear panels as shown in Fig. 3 [22, 23], in this system, defected frames are equipped with concentric X-braces connected in series with rectangular sacrificial shear panels. The braces are designed to remain elastic during seismic events, while the shear panels are sized and configured to dissipated ample energy through plastic deformation [24]. Like eccentric bracing, this system enjoys the benefits of both conventional braced frames by limiting the buildings’ story drift and that of moment resisting frame by enhancing buildings ductility through dissipating large amount of energy in shear panels. However, in concentrically braced frame system combined by shear panels, the shear panels should be proportionally dimensioned to the frame bay dimension,otherwise, the system will be unstable and has no role in withstanding lateral loads.

Methodology

In this study, a new system to retrofit defected RC frames was proposed by replacing the shear panels in previously described system by shear links that are rigidly connected to the bracing members as schematically shown on Fig. 4.

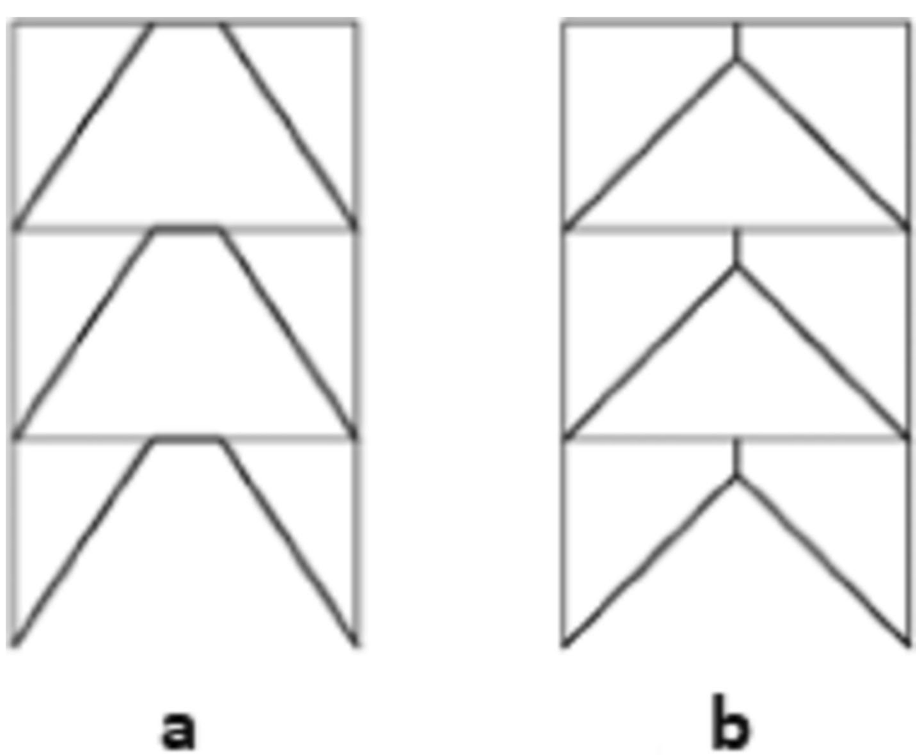

Fig. 2 Eccentric steel bracing system

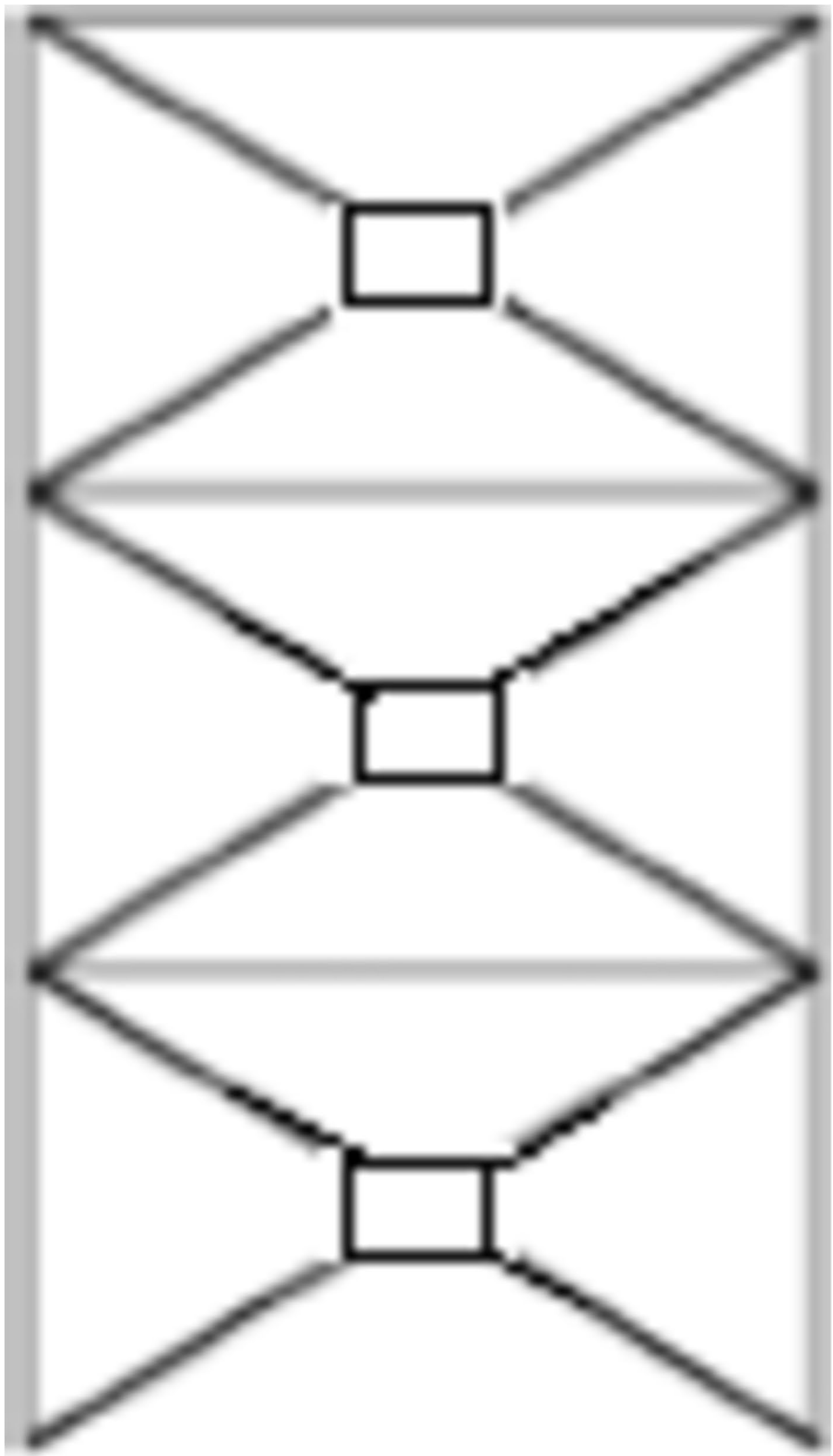

Fig. 3 Concentric steel bracing system with shear panels

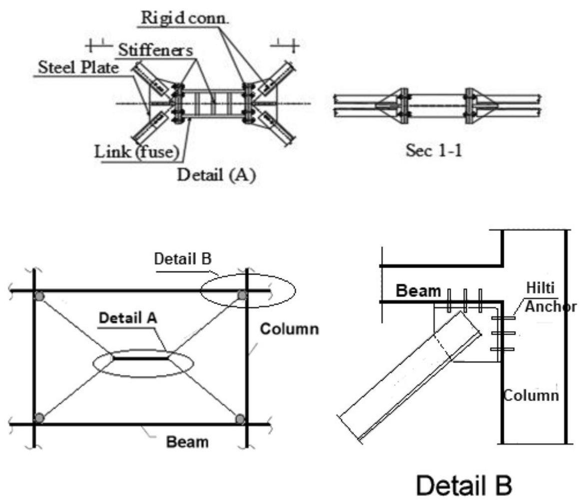

Fig. 4 Schematic overview to the dissipative system

The four short brace members transfer the lateral displacements arising from the earthquake loads on the frame to the shear link. If the shear link is detailed to have strength weaker than the adjacent brace members, it will limit the seismic demand on the bracing members and buildings' concrete elements. At the same time, the bracing with the shear link will enhance slightly the building stiffness and strength for resisting lateral loads without the need for increasing the building weight, thus avoiding the attraction of higher lateral forces. It is worth to mention here that for ease of construction and limiting the straining actions in brace members, the brace members are connected to concrete frame through pin connections and connected rigidly to the shear link as shown in Fig. 4.

Structural detailing for the retrofitting scheme

Conceptually, this proposed scheme can be visualized as a hybrid system that derives its stiffness from the truss action and its ductility from inelastic deformation of link member. The added stiffness due to truss action provided by the brace members which remain in elastic state (no buckling or yielding) can limit the lateral drift of the building to prevent damage to the non-ductile concrete frame members. However, such added stiffness should not be excessive to avoid attracting higher lateral earthquake forces to the building. Meanwhile, the link can be designed to be the fuse that dissipates the earthquake input energy without damaging the bracing members. Such requirements can be assured through utilizing capacity design producer by estimating the ultimate capacity of the link and proportioning the brace members in a way that the inelastic activity is confined to the link.

In this regard, from examining the free body diagram of forces acting on the link shown on Fig. 5, it can be concluded that the brace members can be designed roughly for compressive, or tensile (P_b) forces equal to 1.2 the ultimate shear strength of the link taking into consideration the material overstrength factor (R_y) and brace angle α. Consequently, the design axial compressive or tensile force for each brace member can be calculated as follows:

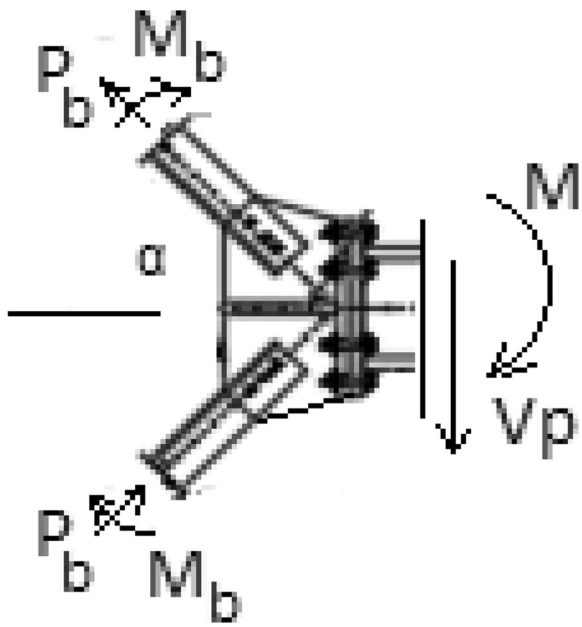

Fig. 5 Freebody diagram for forces acting on link and brace members

$$P_b = (1.2 R_y V_p)/(2 \sin\alpha) \tag{1}$$

where α is the angle the that the brace making with the horizontal and V_p is link ultimate shear strength and is given by the following:

$$V_p = f_y/\sqrt{3}\, t_w\, h_w \tag{2}$$

where f_y is the yield strength of steel, t_w is link web thickness, and h_w is link web [25].

Recognizing that the connection of the link to the brace member in this configuration should be rigid, otherwise, the system will not contribute to resisting the applied lateral forces, each brace member should be designed also for a moment (M_b) equal to M/2 where M is the moment developed at link ends associated with shear yielding of the link. The magnitude of (M_b) can be calculated as follows:

$$M_b = 1.2\, V_p\, R_y\, e/4 \tag{3}$$

In addition, there will be shear acting on the brace member, but without any significance.

To ensure that the link will yield mainly in shear and to minimize the magnitude of the link end moments to avoid increasing the brace members cross-sectional areas and its inertia, the link length (e) should be less than 1.6 M_p/V_p where M_p is the link ultimate moment capacity and is equal to $R_y\, f_y\, b_f\, t_f\, (h_w + t_f)$, where b_f is link flange width, t_f is link flange thickness, and h_w is the web height. Popov et al. [26] suggested that for the link behavior to be dominated by shear yielding, its length should be ranging between 0.15 and 0.2 the frame bay span. Such selection will also ensure that the link length is within practical limits for fabrication purposes.

Regarding the ductility of this proposed scheme, examining the energy dissipation mechanism (collapse mechanism) shown in Fig. 6 indicates that large shear deformation to the link is accompanied by small deformation from the column which in turn minimizes the demand on the RC frame. As can be noted, the small lateral story drift of a single bay θ which is given by the following:

$$\Theta = \Delta/\mathrm{h} \tag{4}$$

where Δ is the relative lateral deformation of the bay and *h* is the story height results in large link shear deformation angle (γ). This deformation angle can be calculated as follows:

$$\gamma = (\mathrm{L}-\mathrm{e})\theta/\mathrm{e} = (\mathrm{L}-\mathrm{e})\,\Delta/(\mathrm{e\,h}) \tag{5}$$

Consequently, for instance, if *e* is taken as 0.2 L or 0.15 L, the link angle of rotation at yield will be roughly about 4θ or 5.67θ, respectively. In other words, the link rotates about 4 or 5.67 time the rotation of the column depending on link length, which implies that large skewness from the links which is the ductile element is associated with small lateral deformation from the columns which are non-ductile elements.

Consequently, for design purpose, the designer can initially use code clauses to calculate the new or updated seismic forces acting on the building and then equip the concrete frame with this scheme and design them for strength requirements and code drift limitations. Following that, the brace members and the link can be proportional to satisfy the previously described ductility requirements.

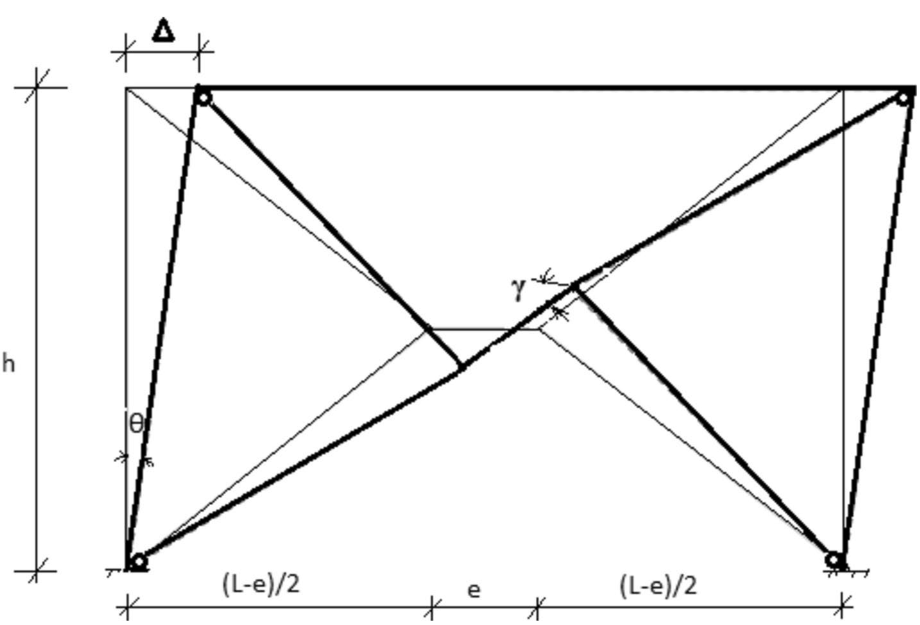

Fig. 6 Collapse mechanism for the proposed scheme

Results and discussion of the structural behavior of retrofitted structures

Retrofitted single story frame

To understand the behavior of this proposed retrofitting scheme on frames in both elastic and inelastic states, initially, a pilot study was conducted considering unbraced hypothetical single-story RC (reinforced concrete) frame. This RC frame comprised of two 4000 mm (H) high columns, spaced at 6000 mm (L), hinged at their bases, and having square cross-sectional area of 400 × 400 mm with reinforced 4T25 reinforcement bars. At their tops, they are connected by a beam having 600-mm depth and 120-mm width reinforced with 2T16 top and bottom reinforcement bars. The frame's concrete cylindrical strength was assumed to be 25 MPa and the reinforced bar to have 420 MPa yield strength. The frame was subjected at its top to concentrated lateral load (F) of intensity 25 KN. The frame's lateral stiffness and resulting straining actions experienced by its structural elements in elastic range were recorded as shown in Fig. 7 and listed in Table 1.

For augmenting the elastic stiffness of this frame and enhancing its structural response under action of lateral loads, this frame was provided as shown in Fig. 8 with arbitrary shear link having, I shape cross-section comprises of two flanges (100 × 20 mm) and web (250 × 4 mm) and set of X-brace members having cross-sectional area formed from

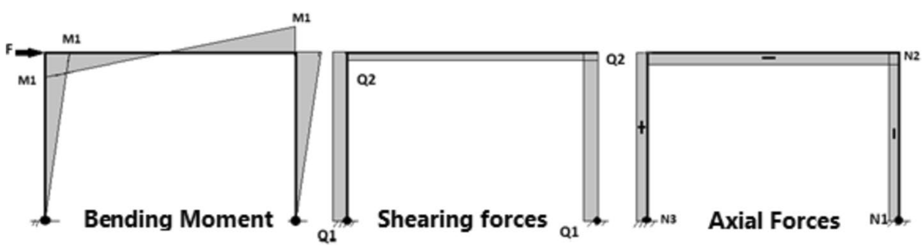

Fig. 7 Straining actions developed in unbraced concrete frame

Table 1 Straining actions for concrete elements in original and strengthened frames

Case study	Stiffness (KN/m)	Axial forces (KN)		Shear forces (KN)		Bending moment (KN.m)	
		Column	Beam	Column	Beam	Column	Beam
Original frame	2609	− 16.34 (N1) + 16.34 (N3)	− 12.25 (N2)	12.25 (Q1)	16.34 (Q2)	49.11 (M1)	49.11 (M1)
Strength frame A	13625	− 8.55 (N1) + 10.9 (N3)	− 10.83 (N2)	2.32 (Q1)	3.12 (Q2)	9.29 (M1)	9.29 (M1)
Strength frame B	30656	− 7.10 (N1) + 10.56 (N3)	− 10.0 (N2)	1.00 (Q1)	1.32 (Q2)	4.01 (M1)	4.01 (M1)

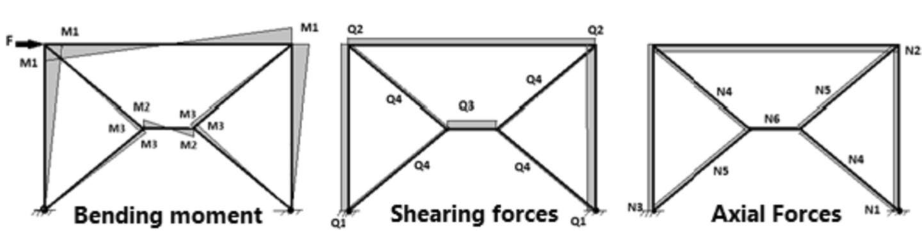

Fig. 8 straining actions developed in braced concrete frames

Table 2 Straining actions for brace and link members in strengthened frames

Case study	Axial forces (KN)		Shear forces (KN)		Bending moment (KN.m)	
	Brace	Link	Brace	Link	Brace	Link
Strengthened frame A	− 13.7 (N4) + 10.02 (N6)	− 2.83 (N6)	1.27 (Q4)	13.23 (Q3)	3.96 (M3)	7.96 (M2)
Strengthened frame B	− 16.32 (N4) + 10.71 (N6)	− 4.42 (N6)	1.04 (Q4)	15.03 (Q3)	3.37 (M3)	6.80 (M2)

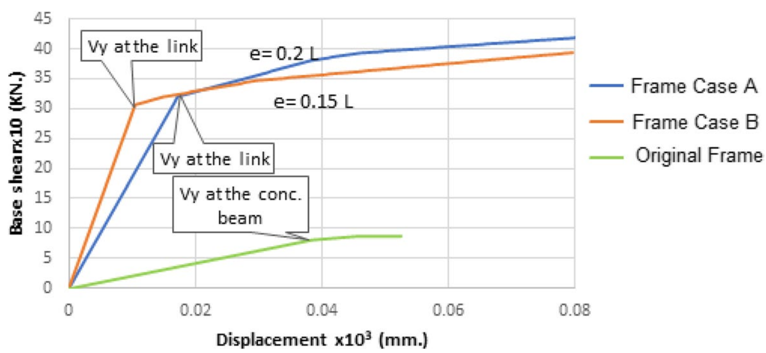

Fig. 9 Frames base shear versus lateral deformation relationships

two angles back-to-back (150 × 150 × 15). The brace members were detailed according to previously described approach to satisfy the capacity design requirements. The link length was taken in once (case A) equal to 1.2 m (0.2 the frame span) and in other case (case B) equal to 0.9 m (0.15 the frame span). As can be observed from examining Table 1, the stiffness of the strengthened frames in cases A and B were 5.22 and 11.75 that of the unbraced original frame. Thus, adopting such strengthening scheme increased frames lateral stiffness which in turn limits their lateral drift, but without creating very stiff system that can attract higher lateral forces.

The straining actions developed in strengthened frames' members under the action of same lateral load (25 KN) applied for the unreinforced frame is shown in Fig. 8. As can be noted, the utilization of this strengthening scheme reduced the bending moments acting on the RC columns and beams for strengthened frames cases A and B, respectively, to 19% and 8% of that for the original frame. Also, axial and shear forces acting on columns and beams were reduced drastically as shown in Table 1. This in turn reduced the demand on those non-ductile elements during earthquake events. On the other hand, for strengthened frames, it can be noted that reducing frames shear link length increases the normal forces acting on brace members and the normal and shear forces acting on shear links under the application of same lateral force as shown in Table 2.

To examine the nonlinear inelastic behavior of these frames, the three frames were analyzed using push-over technique. Figure 9 shows the frames' base-shear versus lateral deformation relationships. As can be noted adding the steel X-bracing system combined with shear link increased significantly both the stiffness and strength of the frames. For frame case A, the strength at first yield occurred at lateral load (F) equal to 312.8 KN, while for frame case B, the strength at first yield occurred at lateral load (F) equal to

310 KN. These yield strengths are significantly high compared to that of original frame which yielded at lateral load equal to 80.26 KN. Furthermore, by observing, the collapse mechanism was noted that in the original frame, the first plastic hinge formed at the right side of the beam followed by another plastic hinge formation at the top of the left column making the frame unstable. In case of frames A and B, the hinges formed at the shear links first, and then at the top of the left side column, but no collapse mechanism was formed.

It is worth to mention here that this study was performed using the nonlinear version of finite element package SAP2000 [27]. The nonlinear behavior of the concrete frame elements was represented by utilizing concentrated plasticity hinges at the potential yielding points. A hinge property is a named set of nonlinear properties that can be assigned to points along the length of one or more frame elements. A capacity drop occurs for a hinge when it reaches a negative-sloped portion of its force–displacement curve during pushover analysis as shown in Fig. 10. In this figure, the five points given (A, B, C, D, and E) are used to define the hinge rotation behavior of RC members following the criteria given by FEMA 356 [28] and adopted by SAP2000. According to this criterion, no plastic deformation occurs until point B where the hinge yields. So, the line A–B represents the elastic behavior of the element. This is followed by a yield plateau or strain hardening behavior until point C which represents the ultimate capacity of the hinge. After point C, the hinge's force capacity immediately drops to point D which corresponds to the residual strength of the hinge. Point E represents the ultimate displacement capacity of the hinge after which the total failure of the hinge is reached at point E.

For modelling shear link in this study, the multilinear analytical model proposed by Ramadan and Ghobarah [29] was used to simulate the nonlinearity of the shear links

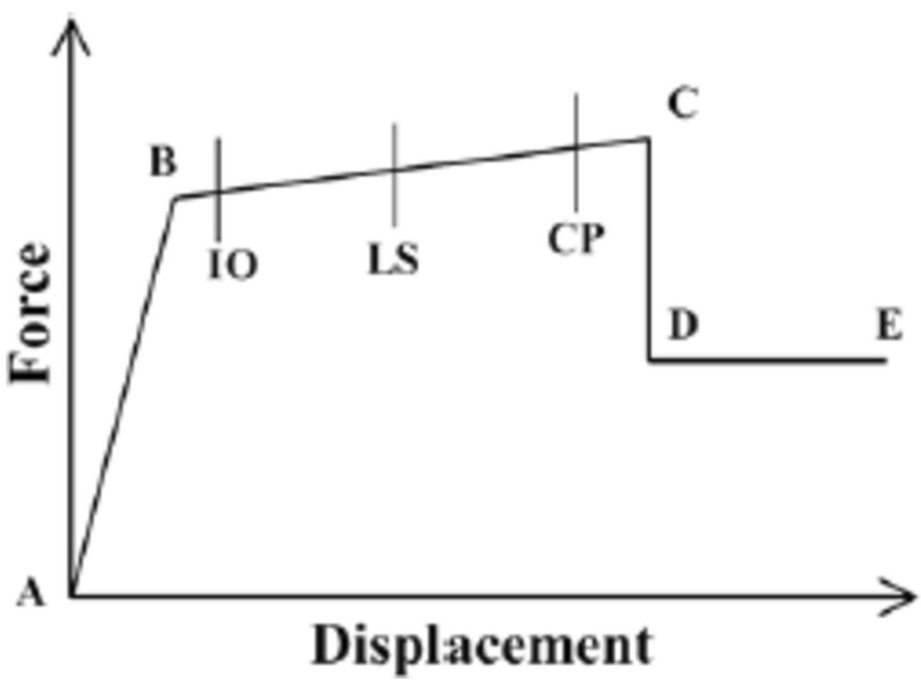

Fig. 10 Plastic hinge behavior according to FEMA 356

as shown on Fig. 11. In this figure, "e" is the shear link length, "G" is the shear modules, "Vp" is the nominal shear resistance of links, and "γ" is the inelastic link rotations angle. It is to be noted that this analytical model is only applicable for well stiffened shear links and it does not consider the axial force that may act on the link. For the bracing members, they were selected as previously indicated to be always elastic. Consequently, they were modeled as frame elements with release to the moment at the joints connecting the concrete frame.

Retrofitted five-story frame

Description of the five-story frame

To evaluate the feasibility of adopting such retrofitting technique for enhancing the seismic behavior of low-rise concrete gravity frames, a hypothetical five-story residential building having a total height of 17 m and a plan area 13.5 × 18 m as shown in Fig. 12 was considered. The building was assumed to be located at Sharm Al-Sheikh city, Egypt, which has a maximum peak ground acceleration of intensity 0.25 g. The building is symmetrical about its main axes and comprising of four equal bays in E-W direction and three bays in N-S direction. The structure was designed mainly to support gravity and wind loads according to old provisions of concrete Egyptian code [30] with no consideration to seismic forces. In this regard, the concrete moment frames along axes A and D were utilized to resist wind loads in E-W direction, while for resisting wind in N-S direction, the concrete moment frames along axes 1 to 5 were considered. The building cylindrical concrete strength was assumed to be 25 MPa, and the steel reinforcement is grade 40/60 with yield strength of 400 MPa. The design dead loads consisted of the structure self-weight, weight of floor finish

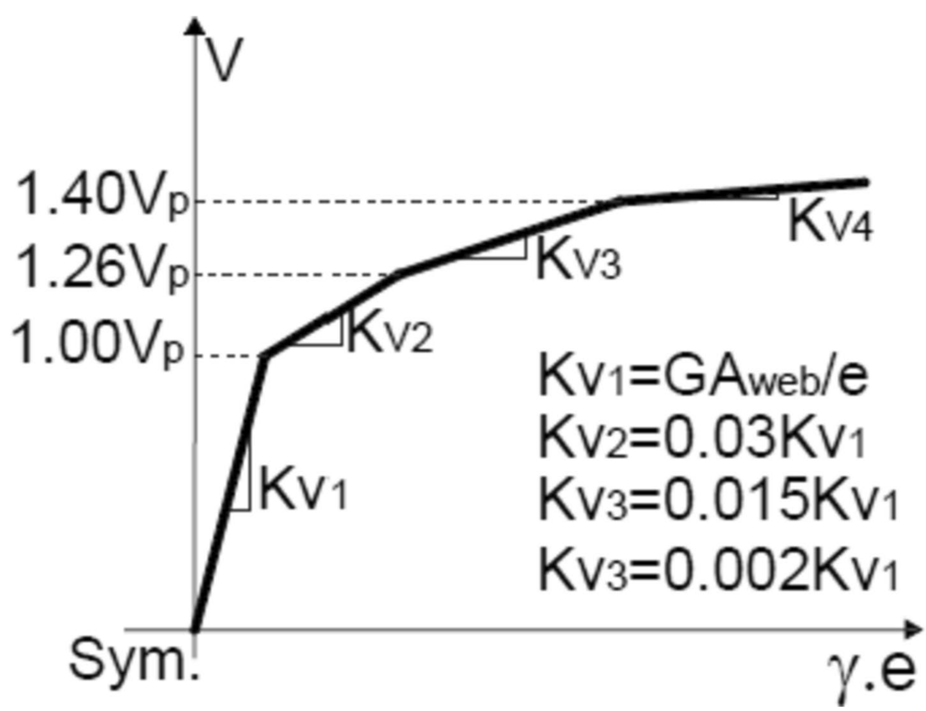

Fig. 11 Multilinear link model

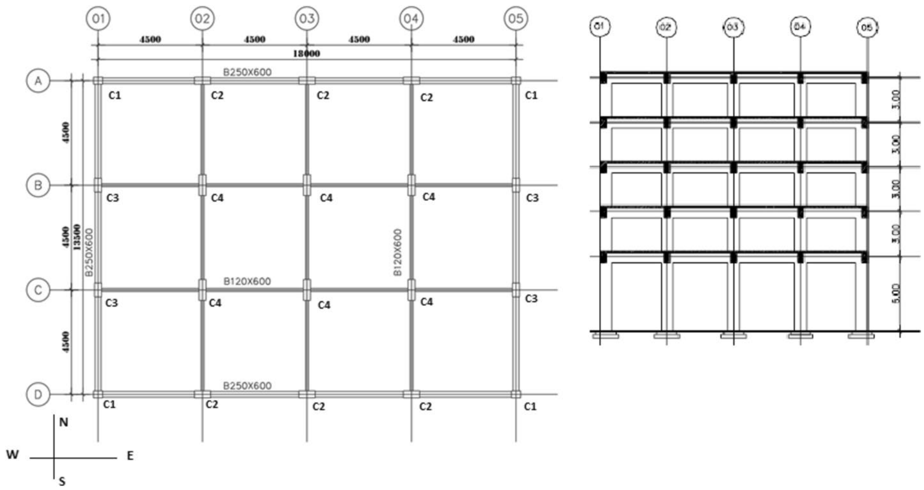

Fig. 12 Typical floor plan and elevation for the 5-story building

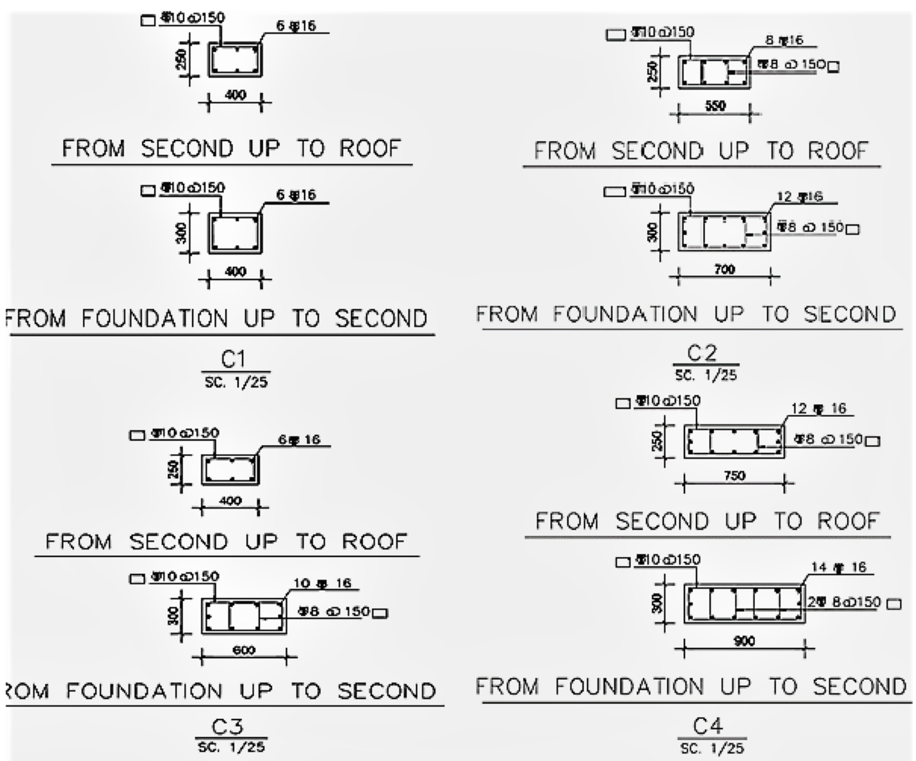

Fig. 13 Details of the reinforced concrete columns for the 5-story building

having an intensity of 2 KN/m^2, weight of internal partitions, and loads of exterior masonry cladding. The design live load was taken as 2 KN/m^2 uniformly distributed over floor area. Figure 13 shows the building's columns concrete cross-sections and their reinforcement. Regarding the beams, all interior beams were assumed to 120×600 mm, while for all exterior beams were assumed to be 250×600 mm.

Evaluating the structural integrity of this building against seismic forces in E-W direction using the provision of the new the Egyptian code [31] assuming that the building is of adequate ductility with force reduction factor, R, equal to 5, indicated that the building is unsafe. All columns at ground floor violated extensively code requirements forming a story mechanism and was unable to sustain the base shear which is equal to 320 KN. Consequently, it was decided to strengthen the building in E-W direction by adding X-braces accompanied by shear links within the exterior frames along axis A in each floor with the bay pounded by Axes O3 and O4.

To model analytically the building for inelastic analyses, it was simulated as a series of planar frames in each direction connected at each floor level by fictitious rigid links representing the effect of rigid floor diaphragm. The analyses were performed using the computer code SAP2000. Due to building symmetry, only half of the building was considered for the analyses. Consequently, in E-W direction, the building was modeled utilizing only two frames located along axes A (exterior frame) and B (interior frame) as shown in Fig. 14. The masses were lumped at beam column joints. The concrete beams and columns were as described previously, with beams having only moment (M3) hinges, whereas columns have axial load and moment (P-M3) hinges. The steel braces were simulated using beam-column element with moment release at the end connected to concrete elements and rigid end to that connected to shear links. Shear link was modeled as previously described.

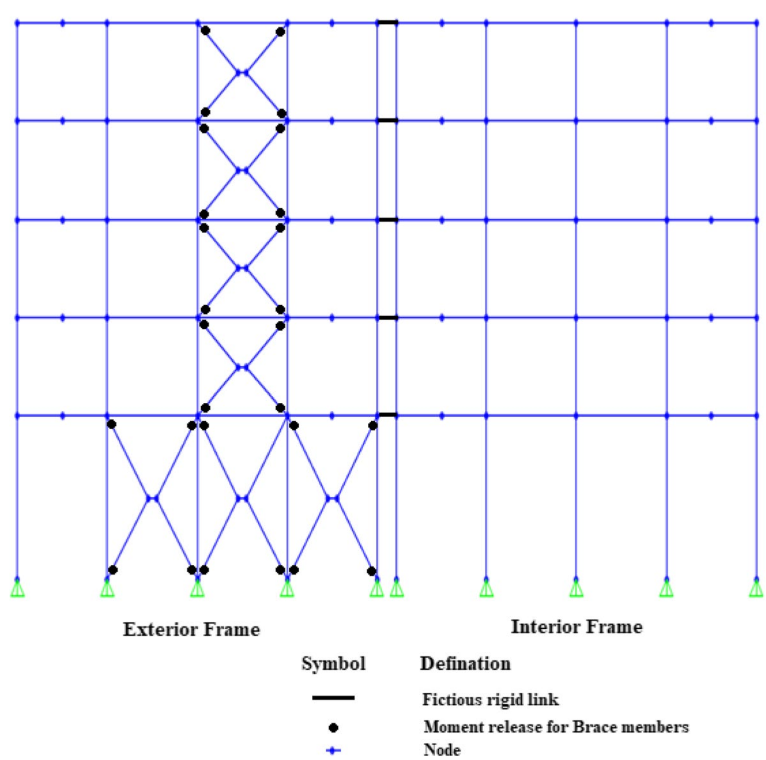

Fig. 14 Idealized finite element model for analyzed 5-story frame

Table 3 Cross-sectional area for brace and shear link members

Floor no	Shear link cross-section	Bracing cross-section
Ground floor	I-section 400 × 250/4 × 40	2 angles back-to-back 150 × 15
1st floor	I-section 400 × 250/4 × 40	2 angles back-to-back 50 × 5
2nd floor	I-section 400 × 250/4 × 40	2 angles back-to-back 50 × 5
3rd floor	I-section 400 × 250/4 × 40	2 angles back-to-back 50 × 5
4th floor	I-section 400 × 250/4 × 40	2 angles back-to-back 50 × 5

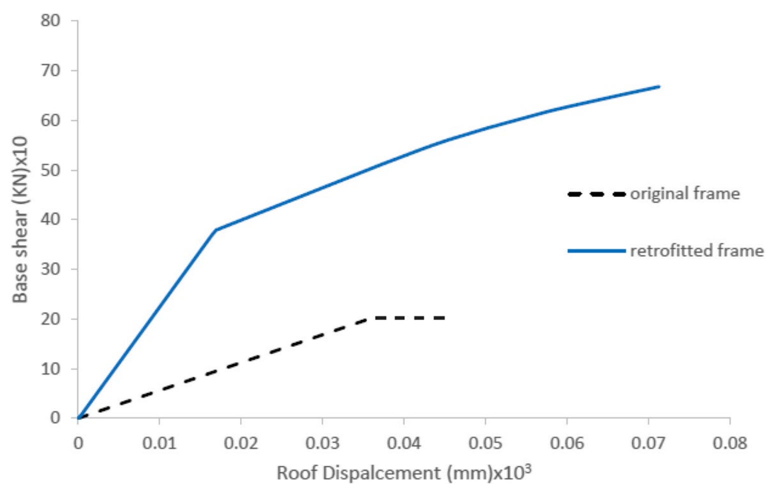

Fig. 15 Base shear versus roof lateral displacement for the 5-story building in E-W direction

Analyzing strengthen frames

To ensure safety of the building against earthquake in E-W direction, the building was equipped as shown on Fig. 14 with steel X-braces combined with steel shear links within the bay bounded between axes 03/04 along the building height. To avoid having high tensile axial forces induced at column bases or possible foundation uplift, the retrofitting braces at the ground floor were spread over three bays as shown in Fig. 14 [13]. Table 3 lists the cross-sectional areas and the bracing utilized for the building in this direction.

The considered structure was analyzed both statically for push-over and dynamically for different time histories. Regarding the push-over analysis, in addition to static gravity loads, a lateral static load having an inverted triangular loading pattern simulating the building 1st mode shape was applied as increasingly static monotonic load. The relationship between roof lateral displacement versus applied base shear are determined and plotted in Fig. 15.

As can be noted, the retrofitted building has an initial stiffness about 3.94 times that of the original structure. Also, it was able to resist the ultimate design base up to 360 KN without any signs of yielding which exceeded the design base shear (320 KN). Consequently, retrofitting the frame ensured its safety against minor and moderate earthquakes. At 360 KN base shear, yielding started within the shear links of the ground floors, and the whole structure showed yielding sings at 380.7 KN base shear exhibiting overstrength of about 1.19. The first yielding of concrete element started

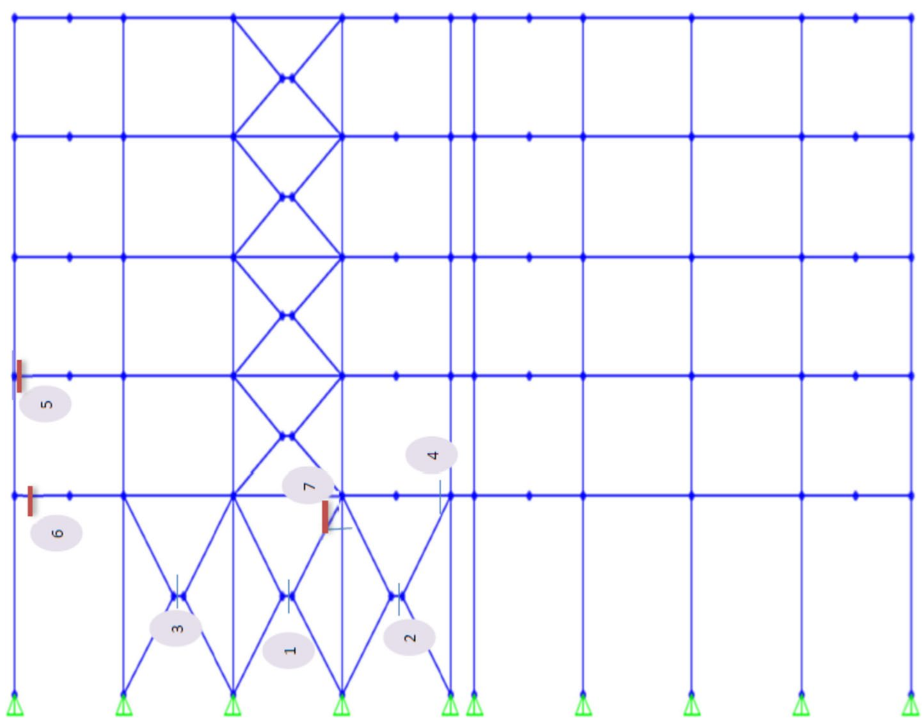

Fig. 16 plastic hinging distribution for the 5-story building

Table 4 characteristics of considered time histories

ID	Earthquake	Date	a/v	Category	Mag	Station name
1	Banja Luka Yugoslavia	1981	2.31	High	6.1	Seism. Station, Banja Luka
2	Park field California	1966	1.7	High	5.6	Cholame, Shandon No. 5
3	San Francisco California	1957	1.67	High	5.25	State Bldg., S.F
4	Imperial Valley California	1940	1.04	Intermediate	6.6	El Centro
5	Kern County California	1952	1.01	Intermediate	7.6	Taft Lincoln School Tunnel
6	San Fernando California	1971	1.01	Intermediate	6.4	3838 Lankershim Blvd., L.A
7	Long Beach California	1933	0.37	Low	6.3	Subway Terminal,.A
8	Lower California	1934	0.77	Low	6.5	El Centro

at ground floor beams at a base shear equal to 480 KN, with corresponding lateral roof displacement of 34 mm. Figure 16 shows the progress of plastic hinges within the strengthened frame up to roof lateral deformation of 40 mm. According to the Egyptian code, the maximum expected inelastic lateral displacement of this building is about 49 mm, but subsequent analyses using eight different time histories indicated that the target displacement based on averaging the maximum roof deformations is 34.4 mm. At such deformation, no yielding was recorded in the concrete elements except at hinge marked “4.”

In addition to push-over analysis, the original and strengthened five-story buildings were analyzed for different eight-time histories. The selected time histories were having different ratios for peak ground acceleration to peak ground velocity (variable

Table 5 Number of plastic hinges formed in both original and retrofitted structures due to different time histories

Ear ID	Original structure		Retrofitted structure		
	Hinges in columns	Hinges in beams	Hinges in columns	Hinges in columns	Hinges in links
1	0	7 (Gr. floor)	0	5 (2nd and 3rd floors)	3 (Gr. floor)
2	0	0	0	0	0
3	0	7 (Gr. floor)	1 (3rd floor)	2 (3rd and 4th floors)	3 (Gr. floor)
4	2 (Gr. floor)	20 (Gr. and 1st floors)	0	12 (Gr., 1st, 2nd, and 3rd floors)	3 (Gr. floor)
5	2 (Gr. floor)	22 (Gr. and 1st floors)	2 (3rd floor)	18 (Gr., 2nd, 3rd, and 4th floors)	3 (Gr. floor)
6	4 (Gr. floor)	21 (Gr. and 1st floors)	4 (3rd and 4th floor)	20 (Gr., 1st, 2nd, and 3rd floors)	3 (Gr. floor)
7	0	10 (Gr. floor)	0	1 (3rd floor)	3 (Gr. floor)
8	0	0	0	0	0

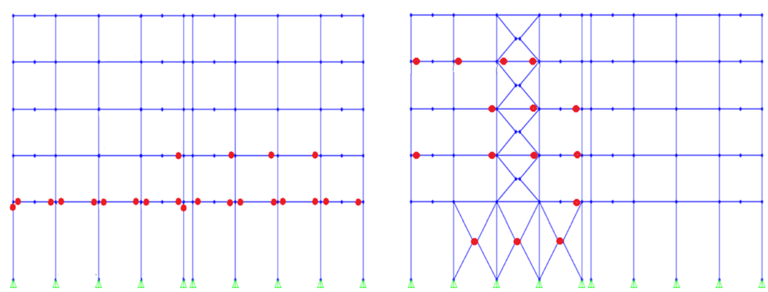

Fig. 17 Formed plastic hinges in structures due to Imperial Valley earthquake

a/v ratios) to cover wide spectrum of possible earthquakes [32]. Table 4 lists the characteristics of these earthquakes. As can be noted, three earthquakes were having high a/v ratios, another three earthquakes were possessing intermediate a/v ratios, and the last two earthquakes were having low a/v ratios. All time histories were scaled to have a maximum acceleration of 0.25 g corresponding to that expected at Sharm Al-Sheikh area.

The different time histories were applied to both the original and retrofitted structure. Table 5 summarizes the number of formed hinges within each frame and their locations. Examining this table indicated that the time histories with intermediate a/v ratios (earthquakes with ID 4, 5, and 6) were the most damaging earthquakes to the 5-story buildings, while those with low a/v ratio (earthquakes with ID 7&8) produced the minimum damage. Also, it showed that for original structure, the damage due to different earthquakes was concentrated mainly in the columns of ground floor and beams of ground and first floors, while for retrofitted frames, in addition to expected damage at active links located within ground floors, the damage to beams and columns spread all over the building height with no damages to columns at ground floor.

Examining Figs. 17, 18 and 19 which show the locations of formed plastic hinges marked as red dots due to Imperial Valley, Kern County, and San Fernando

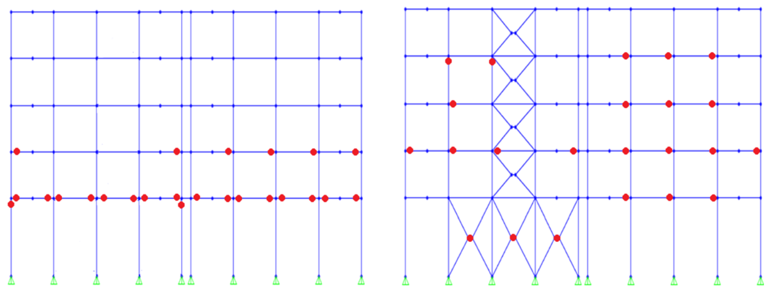

Fig. 18 Formed plastic hinges in structures due to Kern County earthquake

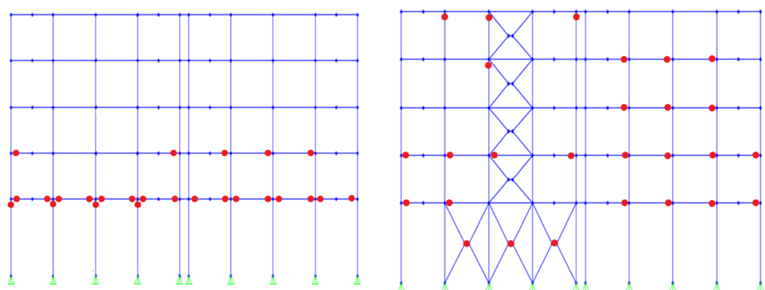

Fig. 19 Formed plastic hinges in structures due to San Fernando earthquake

earthquakes for both original and retrofitted structures indicated that the number of formed plastic hinges in the nonductile concrete elements of the retrofitted building is less compared to those formed in original structure. Consequently, it can be concluded that retrofitting the building with steel X-bracing system combined with shear link improved drastically the building seismic response. Also, it can be noted that the formed plastic hinges in retrofitted frames spread over the frames' height, while those exhibited by original frames concentrated in ground and first floors. This can be further proved by examining Figs. 20, 21 and 22 which show that the recorded maximum story displacement occurred under the action of each of the eight earthquakes for these frames at each floor. As can be observed, the ground floor in original structure experienced under the action of all considered earthquakes significant lateral displacement indicating that most of the damages concentrated in frames' ground and first floor nonductile concrete elements which can result in frames collapse, while for strengthened frames, such lateral displacement at ground and first floors was controlled reducing the demand drastically concrete elements. For instance, the lateral drift determined within the ground floor during El Centro and kern county earthquakes was about 1/66 for original structure and 1/171 for retrofitted ones. Obviously, the original structures with such extreme drift are expected to experience severe damage, especially that it is not seismically detailed, while with retrofitting frames, the displacement within this ground floor became more controlled and lateral drift was limited.

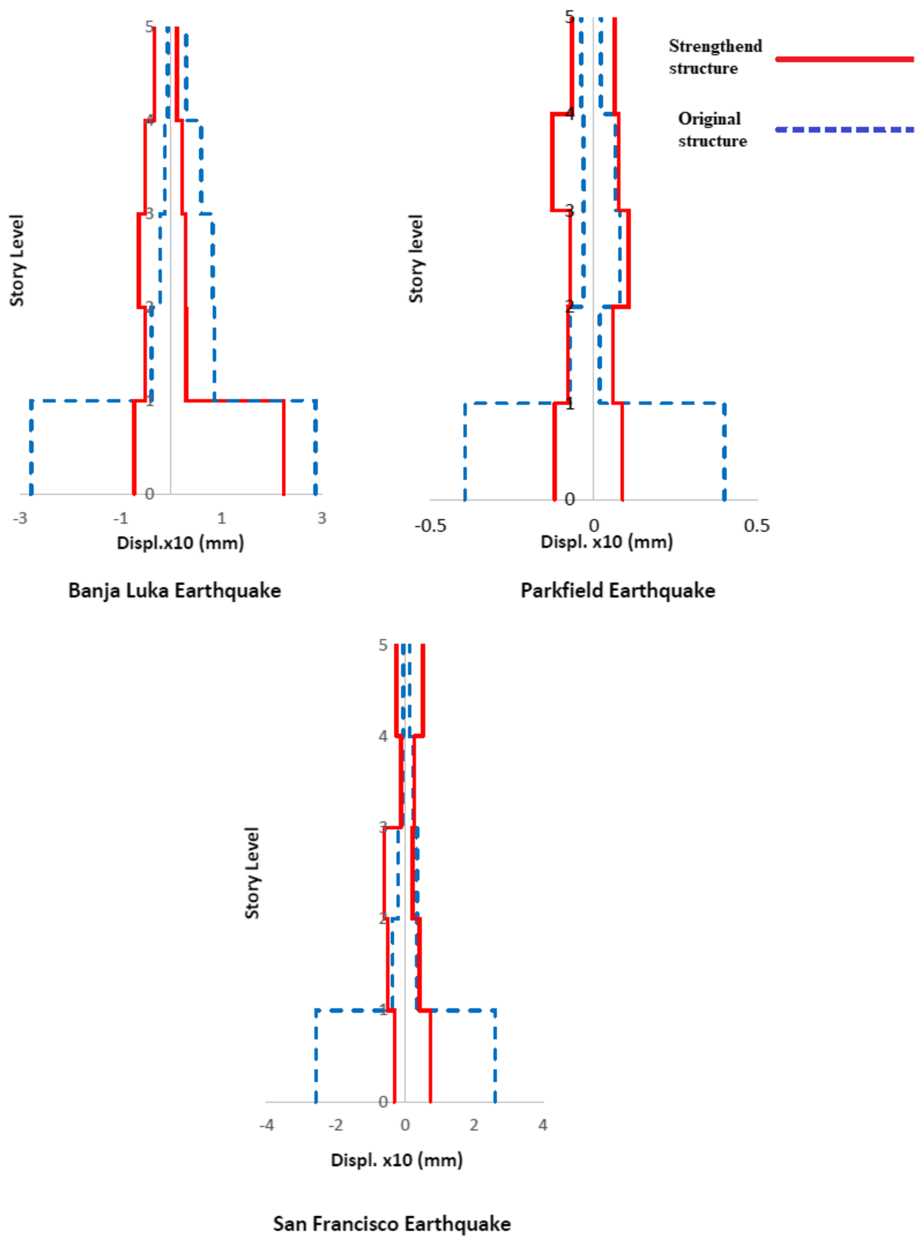

Fig. 20 Story maximum lateral displacement due to earthquakes having high a/v ratio

With respect to attracted base shear, Table 6 lists the maximum and minimum base shear recorded for each frame due to the different earthquakes. As can be noted, adding bracing to frames increased the building stiffness, which led to attracting higher lateral forces. However, the increase in building lateral stiffness was controllable (the increase in base shear was between 2 to 3.8 times that of original structures).

Conclusions

This research study examined the rationality and effectiveness of employing steel X-bracing combined with shear link to seismically upgrading seismically defected reinforced concrete frames. The proposed retrofitting system combined the

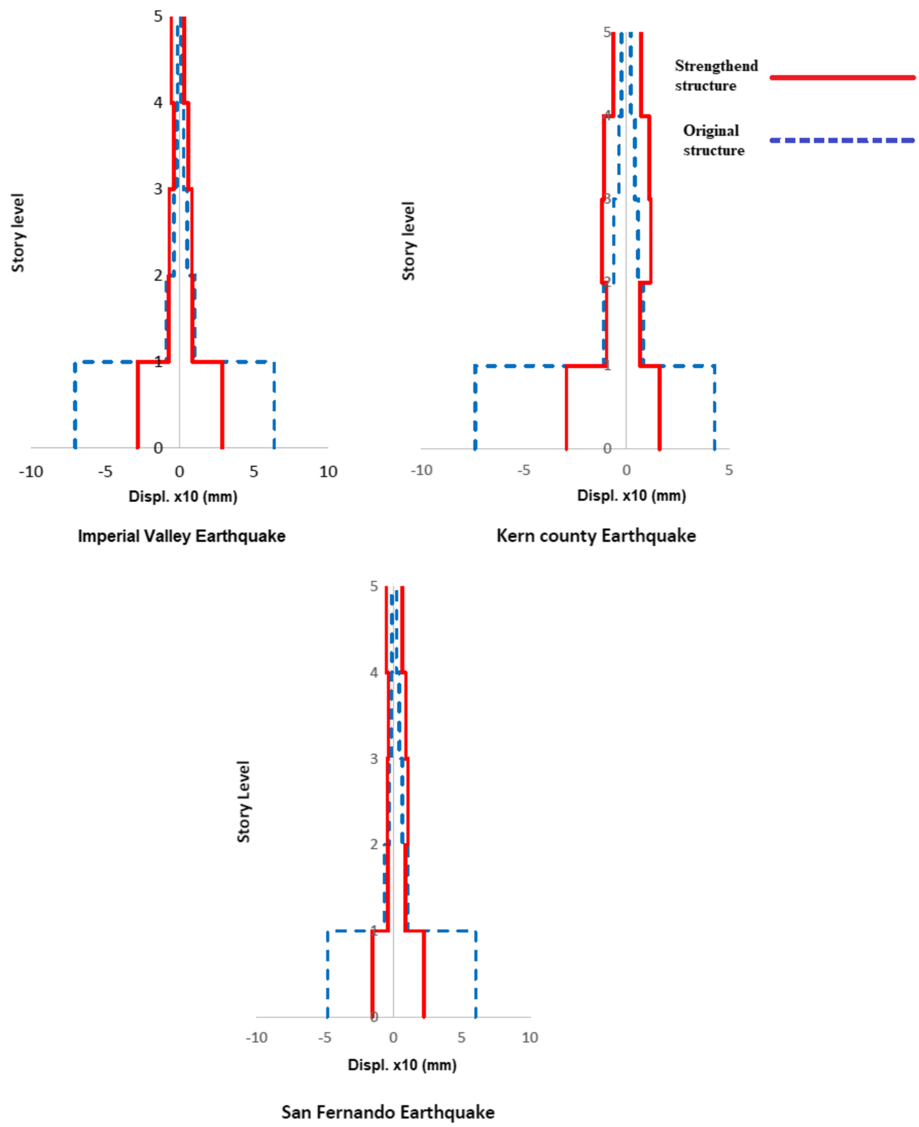

Fig. 21 Story maximum lateral displacement due to earthquakes having intermediate a/v ratio

advantageous of employing concentric bracing to increase, in a controllable manner, the lateral stiffness of defected frames to limit their lateral drift and utilizing shear links to provide the required ductility and earthquake energy dissipation in stable hysteretic behavior. Several nonlinear static and dynamic analyses were carried out on defected frames and retrofitted ones. The research showed that this scheme provided several benefits to frames seismic response. Those benefits involved the following:

1. Limiting lateral drift of defected RC frames at ground and first floors prevented the formation of weak story failure mechanism.
2. Reducing the number of formed plastic hinges and distributing them over the frames height instead of concentrating at lower floors as in the case of original frames.

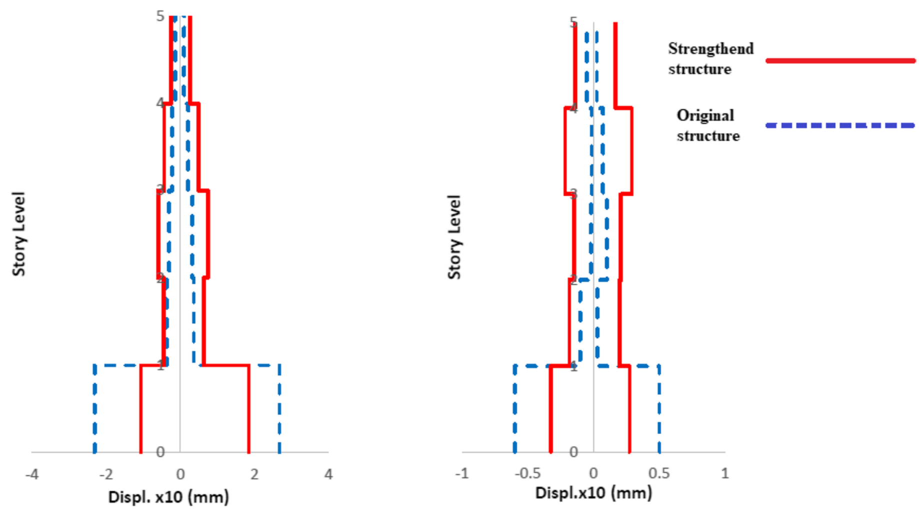

Fig. 22 Story maximum lateral displacement due to earthquakes having low a/v ratio

Table 6 Maximum and minimum base shear due to different earthquakes

Ear ID	Original structure		Retrofitted structure	
	Max. base shear (KN)	Min. base shear (KN)	Max. base shear (KN)	Min. base shear (KN)
1	200	−160	420	−380
2	50	−40	100	−70
3	130	−130	480	−430
4	320	−310	610	−690
5	330	−290	580	−520
6	320	−300	580	−560
7	150	−180	470	−540
8	100	−60	240	−230

3. Reducing the demands on structural elements of RC frames by preventing them from yielding prior to yielding of shear links.
4. The proposed system can be installed easily and quickly, thus eliminating the need for disrupting the occupant of existing structures. Also, it is light in weight and does not require strengthening to concrete elements.
5. After earthquake, this system can be replaced easily, if experienced inelastic actions.

In addition, guidelines were presented for designing and detailing this system. Its fundamental concept is based on adopting capacity design by ensuring that yielding/buckling strength of bracing must be more than the strength of the link, so it does not yield and damage confined to shear links.

Abbreviations

RC Reinforced concrete
CBF Concentrically braced frame
EBF Eccentrically braced frame

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Authors' contributions
A.O. interpreted the results and draw the graphs and wrote the paper. A. F. performed the finite element analysis and achieved the results. All authors read and approved the final manuscript.

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Declarations

Competing interests
The authors declare that they have no competing interests in this section.

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