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Performance-based plastic design for seismic rehabilitation of high rise frames with eccentric bracing and vertical link

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Abstract. A large number of available concrete buildings designed only considering gravity load that require seismic rehabilitation because of failure to meet plasticity criteria. Using steel bracings are a common type of seismic rehabilitation. The eccentric bracings with vertical link reduce non-elastic deformation imposed on concrete members as well as elimination of probable buckling problems of bracings. In this study, three concrete frames of 10, 15, and 20 stories designed only for gravity load have been considered for seismic improvement using performance-based plastic design. Afterwards, nonlinear time series analysis was employed to evaluate seismic behavior of the models in two modes including before and after rehabilitation. The results revealed that shear link can yield desirable performance with the least time, cost and number of bracings of concrete frames. Also, it was found that the seismic rehabilitation can reduce maximum relative displacement in the middle stories about 40 to 80 percent. Generally, findings of this study demonstrated that the eccentric bracing with vertical link can be employed as a suitable proxy to achieve better seismic performance for existing high rise concrete frames.

Keywords: concrete frame; seismic parameters; shear link; eccentric bracing; finite element; rehabilitation

1. Introduction

High rise buildings are being increasingly designed and constructed due to rapid increase in population of cities and consequently high demand for their settlement. Moreover, aging is another problem associated with available tall buildings that should be rehabilitated carefully to provide safety of their residents. Earthquake can be considered amongst the most natural disasters threatening safety and life of people. Thus, these buildings embedding many people in should be resistant enough to guarantee life of them. There are many factors affecting on a resistant system against lateral loads such as stiffness, structural flexibility, and potential to absorb high energy by cyclic behavior or hysteresis system. Marthong (2019) evaluated rehabilitation of reinforced concrete beam-column connections using epoxy resin injection and galvanized steel wire mesh. The results were promising to employ the proposed materials their performance for rehabilitation was satisfactory. Existing lateral resistant systems including seismic separators, buckling-restrained braced frame, steel shear wall systems, and reinforced polymeric materials have some disadvantages and limitations (e.g., problems of economic justification, implementation, and demanding large space). In this regard, eccentrically frames braced frames (EBFs) have been introduced and designed as a suitable proxy to overcome the abovementioned problems. These frames were mainly designed to incorporate the advantages of moment resisting frame and concentrically braced frame lateral load resisting systems into a single structural system (Azad and Topkaya 2017). Therefore, they are expected to serve as a structure with high seismic performance (ability to damp high energy of earthquakes) and also to represent high elastic stiffness. Using EBFs instead of moment resisting frame is more cost effective because it decreases steel demand significantly. Moreover, many successful applications of EBFs for high seismicity regions such as Japan and United States demonstrate efficiency and reliability of them (Foutch 1989, Okazaki *et al*. 2009, Richards 2004).

Azad and Topkaya (2017) provided a thorough review on research dealing with application of eccentrically braced frames from different points of view. In this regard, many experimental and numerical studies have been taken under consideration covering a wide variety of issues associated with the performance of this type of frames. Ruiz-García *et al*. (2018) investigated seismic behavior of steel EBFs under soft-soil seismic sequences by means of analytical models. The results revealed nonuniform distribution of hysteretic energy along-height of the links. This feature of EBFs indicates that the energy dissipating capacity of the shear links is not fulfill completely. Bozkurt, Kazemzadeh Azad and Topkaya (2018) through an experimental study investigated the low-cycle behavior of shear links in EBFs. They have investigated behavior of 14 samples under constant-amplitude cycles as well as proportional and arbitrary loading histories to find link fracture under earthquake-induced loading histories. They argued and evaluated the existing standard related to shear links against findings of the study. Considering structural responses to

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near fault and far fault ground motions, it was found that the effects of near field earthquakes on the displacement of the structure are greater than the distal earthquake (Hajali *et al*. 2018). Dealing with the far field earthquakes (referred to the long distance narrow band and long duration excitations) with low frequency motions, the tuned mass damper is effective for the structures as the period is closer to/ or longer than the natural period of a structure (Cao and Li 2019). Additionally, they stated that the effectiveness of the damper reduces as the motion period shortens. Thus, its efficiency for near fault (pulse like near fault) ground motions is greatly less than for far field one. Fakhraddini *et al*. (2019) proposed a simple empirical equation to estimate the peak floor displacement patterns Performance-based seismic design of EBFs. To derive the relationship, four EBFs with various link to span length ratio has been examined for testing purposes. Comparing the results related to displacement patterns obtained from the proposed formulae with those of exact nonlinear dynamic analysis demonstrates efficiency of the proposed formulae.

Some of existing buildings constructed before 1990 may does not meet seismically criteria as the modern seismic code came into wide use after that time. Therefore, they may subject to remarkable deficiencies in their overall structural systems or members (Al-Dwaik and Armouti 2013). Therefore, their failures may cause serious casualties or problems if they do not be treated appropriately. Rehabilitation can be considered as a suitable remedy to improve performance of such structures against probable seismic issues. In brief, structural rehabilitation can be defined as application or employment of any procedures and techniques to improve and upgrade safety of the buildings with minimum disturbance to the residents considering time and cost. It has attracted attention of many researchers and engineers in which different techniques and approaches have been introduced and applied. In this interval, EBFs with vertical link have been taken more popularity due to their advantages in terms of required space, time, cost and capability to improve the structure seismic performance. Al-Dwaik and Armouti (2013) investigated effect of seismic rehabilitation to improve seismic performance for a 5-story concrete frame. To do that, they employed two methods of the addition of eccentric steel bracing and the addition of reinforced concrete column jackets. Linear and nonlinear dynamic analysis was conducted along with using synthetic ground motion to evaluate seismic performance of each rehabilitation strategy. The results demonstrated that the eccentric shear link bracing is superior over (former strategy) over the later strategy in terms of performance and cost. Furtado *et al*. (2017) assessed efficiency of different strengthening methods for an existing building using numerical analyses. In this regard, four strengthening techniques including addition of reinforced concrete shear walls, addition of steel bracings with and without shear link and reinforced concrete column jacketing have been examined to attenuate the soft story like response. The results showed that the addition of steel bracing outperform the other techniques for reducing the maximum first-story drift. Wu *et al*. (2018) demonstrated that tie beam and column strengthening can improve the seismic performance

Table 1 Characteristics of beams and columns

Dimension ID	Section name	Heights (cm)	Width (cm)	Placing reinforcing bars
1	B40×50	40	50	-
2	B40×60	40	60	-
3	B40×65	45	65	-
4	B40×70	50	70	-
1	C40×40	40	40	8φ20
2	C45×45	45	45	12φ20
3	C50×50A	50	50	16φ20
4	C50×50B	50	50	20φ20
5	C55×55A	55	55	24φ20
6	C55×55B	55	55	24φ22
7	C60×60	60	60	28φ22
8	C65×65	65	65	24φ25
9	C70×70	70	70	24φ28
10	C75×75	75	75	28φ28

of frames on two ground levels, significantly. Mohsenian and Mortezaei (2018) evaluated reliability and multi-level response reduction factor of the structural systems equipped with vertical links by using pushover and incremental dynamic analysis. The results revealed efficiency of the system with vertical links in terms of seismic energy dissipation and ductility. Apart from the abovementioned studies, there are several studies addressing efficiency of EBFs for improving performance of existing buildings (Castro *et al*. 2018, Simpson and Mahin 2017, Valente and Milani 2018, Vetr *et al*.2017, Mohsenian and Nikkhoo 2019). However, less efforts were devoted to explore whether efficiency of the rehabilitation approach for high rise concrete frames are the same as for the middle and short buildings.

The main objective of this study is therefore to investigate effect of EFBs to improve seismic behavior of high rise concrete frames. In this regard, three concrete frames with 15, 20 and 25 stories designed for gravity load presented by code ACI-318-99 are taken under consideration for rehabilitation. Using nonlinear analyses of historical time series under 7 scaled far-field earthquakes for life safety (LS) and collapse prevention (CP) in OpenSees as a powerful finite element package, seismic performance of the models are investigated. The results for two modes of original structure with and without the eccentric bracing with vertical links are evaluated following the available standards and code No. 356. Moreover, the performance-based plastic design is implemented for rehabilitation purpose. Rest of the paper is organized as: section 2 presents the methodology and materials. Model verification, earthquake time series and shear link simulation are discussed in section 3. The results are discussed and analyzed in section 4. The conclusions are presented in the last section.

2. Materials and methods

2.1 Model development

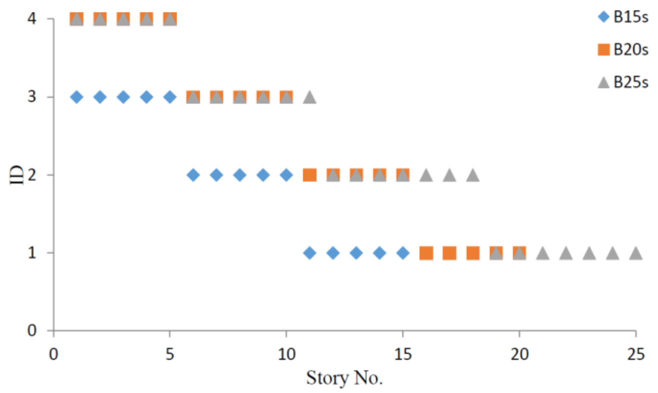


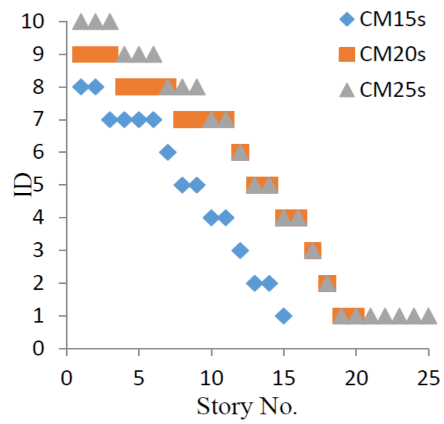
Fig. 1 Dimension ID of beams for different frames and stories

The modeling procedure is established by selection of three concrete frames of 15, 20 and 25 stories in which each story has 3.6 m height with three constant spans of 5 m. The models are implemented in a two dimensional system where the lateral load resisting system of the frames includes concrete moment-resisting frame with medium ductility. For concrete and steel, the modulus of elasticity was considered as 2.291289×10^5 kg/cm^2 and 2×10^6 kg/cm^2, respectively. Live load for residential buildings was assumed as 200 kg/m^2 and the distributed load imposed on the floor was set as 100 kg/m^2. Also the coefficient of this load for mass association to determine seismic mass was considered as 1. Finally, the live load on the roof and dome was given as 150 kg/m^2. Table 1 gives characteristics of the concrete sections for beams and columns design. It is noticed that the letters B and C in the following table represent beam and column respectively.

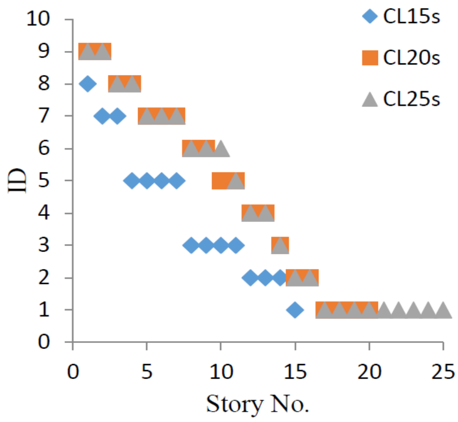
In Table 1, letters B and C stand for beam and column respectively. These sections are numbered in ascending order for further investigations and for sake of simplicity. As it can be observed from Table 1, the sections are employed with the height from 40 to 50 cm for beams and from 40 cm to 70 cm for columns. Similarly, width of the beams and columns are in range of [50-70] and [50-75]. For columns with larger sections, reinforcing bars are placed in which larger columns are supported with armors with higher dimensions. Fig. 1 presents specifications of the beams applied for different stories in the concrete frames. The latter word “s” in the Figs. 1 and 2 denote the story number.

Fig. 1 depicts that the taller concrete frames requires larger beams in which for 25 stories frame, the lower floors requires larger beams (B40*B70) while for the 15 stories frame it reduces to ID 3 indicating a narrower beam (B40*B65). In a similar way, the plot is repeated for the middle (CM) and lateral (CL) columns in Fig. 2.

Following Figs. 2(a)-(b) and Table 1, dimensions of the columns employed for the modeling procedure in each frame can be found easily. After finding dimensions of the beams and columns, the frames can be introduce to the numerical model (here means finite element model implemented in OpenSees) for further simulations and investigations. In this study, the computational models of concrete frames have been developed two dimensionally



(a) Middle columns

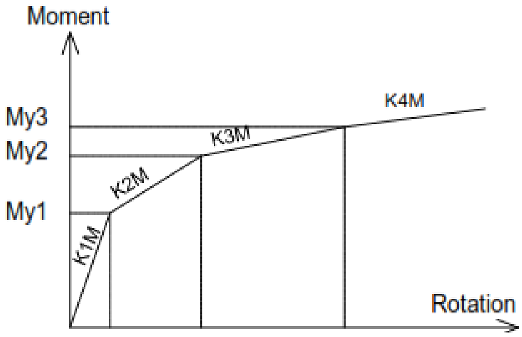


(b) Lateral columns

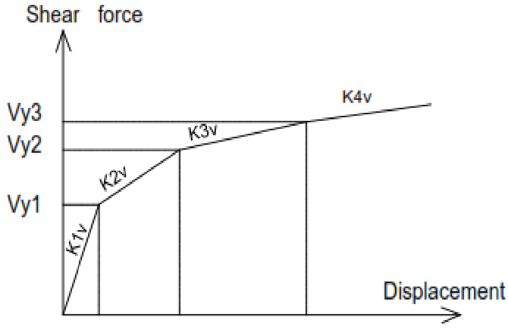
Fig. 2 Dimension ID of different frames and stories

implemented with OpenSees software framework (McKenna *et al*. 2015). OpenSees is a finite element based framework created by the National Science Foundation-sponsored pacific earthquake engineering center to simulate behavior of structural and geotechnical systems under seismic loads. It was proven by previous studies that OpenSees can be served as a powerful tool for simulation and analysis of seismic behavior of structures (Bao *et al*. 2008, Dolšek and Fajfar 2008, Haselton *et al*.2010, Lu *et al*. 2015). This study is not aimed to delve into basic concepts and formulations embedded in the software and only the important points related to this research are briefly mentioned.

As a finite element based software, OpenSees has the capability to estimate large displacement behavior of space frames under both static and dynamic loads that contain geometric linear behavior and also non-elastic behavior of materials. To simulate the elements in nonlinear domain, all the connections for concrete structure members such as beams, columns and base story nodes are considered as fixed connections. Moreover, it is assumed that the columns bear all the vertical loads. Regarding the behavior of rigid diaphragm in roofs, in dynamical analysis, the mass in each story level is equally distributed among the nodes linking beams and columns. The material has been selected as uniaxial Material which is capable to consider axial force and moment to satisfy one dimensional stress-strain relationship. In the software setup, rigid Diaphragm,



(a) Moment-rotation



(b) Shear force-displacement

Fig. 3 Spring relationships used in steel connection

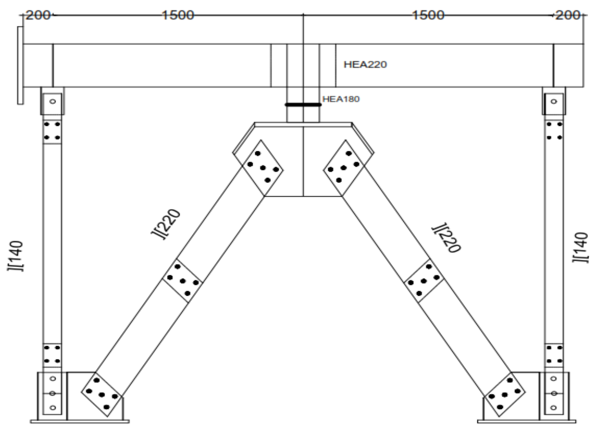


Fig. 4 Schematic layout of the frame with the bracing (Vetr 1998)

Table 2 Characteristics of the frame members with vertical link (Vetr 1998)

Members	Features	P_y (KN)	M_y (KN-cm)	V_y (KN)	Length of the link (cm)
Bracings	2UNP220	2244	16140	-	-
Columns	2UNP140	1224	5704	-	-
Beam	HEA320	3720	48840	-	-
Vertical link	HEA280	-	33360	299	30

Concrete02, and Steel02 were selected to determine rigid diaphragm, concrete and steel types. The beams and columns were set in nonlinear mode with five sections. All the sections in the model were considered as fiber model and also Gauss-Lobatto quadrature law was employed for integration purpose in the element nodes.

2.2 Simulation and validation of shear link

In recent years, many efforts have been made to investigate link models for non-elastic static and dynamic analyses of eccentric bracing systems. In this study, to simulate shear link, Richards model has been taken under consideration in which the steel connection is as a beam linear element with 6 nonlinear springs at the end (Richards, 2004). These springs have the rotational and translational motions where 3 double torsion springs are employed to simulate non-elastic bending behavior of plastic hinge at the end of connection. Moreover, three double translational springs are applied to model shear non-elastic behavior of the web beam by means of a multi-linear function as illustrated in Fig. 3. In this regard, the vertical steel connection is shown as two torsion and translational springs at the fix ends of the link. Each spring has been designed and developed to derive required equations in Figs. 3(a)-(b). One spring gives moment-rotation relationships while the other provides displacement-lateral force relationships. These relationships can be expressed as Eqs. (1)-(6).

$$V_{y1} = V_y \; ; \; M_{y1} = M_y \tag{1}$$

$$V_{y2} = 1.5V_y \; ; \; M_{y2} = 1.03M_y \tag{2}$$

$$\mathrm{V_{y3}} = 2.0\mathrm{V_y} \; ; \; \mathrm{M_{y3}} = 1.06\mathrm{M_y} \tag{3}$$

where the stiffeners can be computed as

$$K_{2V} = 0.100K_{1V} \; ; \; K_{2M} = 0.03K_{1M} \tag{4}$$

$$K_{3V} = 0.003K_{1V} \; ; \; K_{3M} = 0.015K_{1M} \tag{5}$$

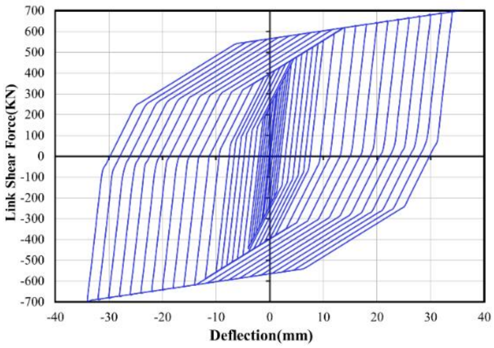
$$K_{4V} = 0.007K_{1V} \; ; \; K_{4M} = 0.002K_{1M} \tag{6}$$

Dealing with any numerical simulation, model validation is a key step toward achieving reliable results and special care should be taken for this stage. To do that, it is common to compare the results of the preliminary model with those of the observed or experimental data in order to tune the model parameters. The current research employs experimental results of Vetr model to validate and compare the numerical simulations (Vetr 1998). The experiment was implemented with an eccentric bracing frame with one span vertical link for a single story. Fig. 4 illustrate a schematic layout of the experimental frame. The steel was selected as ST37 with allowable stress of $F_y = 2400\,\mathrm{kg/cm^2}$ and ultimate stress of $F_u = 4000\,\mathrm{kg/cm^2}$.

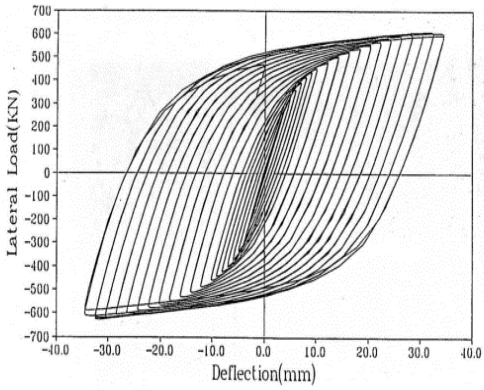
In Table 2, details of frame and link beam applied for the experimental purpose are presented. The connection of beam and column was done by a simple type. The columns were connected to their underneath roller support by welding. More descriptions on the experimental model can be found in the reference.

Subsequent to modeling of eccentric bracing frame with vertical link of the first sample in OpenSees, loading and displacement controlling, results of link shear force against deflection are presented in Fig. 5.

According to Figs. 5(a)-(b), it can be observed that the results of the numerical model show a good agree of similarity with those of obtained from the experimental



(a) OpenSees



(b) Experiments of M. Vetr (1998)

Fig. 5 Results of hysteresis plot

model. This agreement between results of numerical model and experimental model proves reliability and efficiency of the simulations performed using the software.

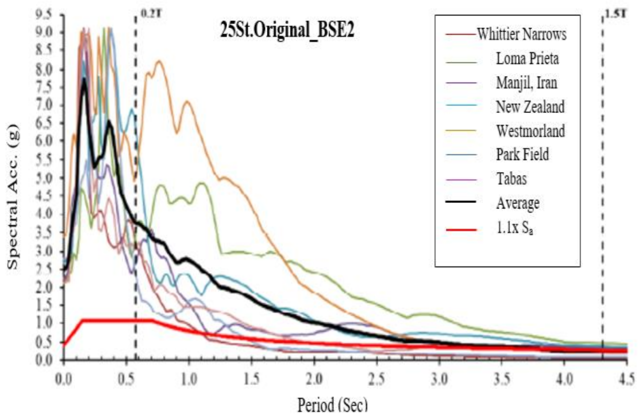
2.3 Specifications of the selected earthquakes

Selected earthquakes to analyze non-linear time series have similar characteristics such as richter magnitude in a range of 4.5 to 8, plate distance to the fault about 20 to 60 kilometer, speed of shear wave considering location of frame construction (site class=*D*) as 600 to 1200 ft/s and the maximum acceleration was between 0.2 g to 2 g. Specifications of 7 selected earthquakes are given in Table 3. As this study employs two dimensional models, only the earthquake records in *x* direction are presented in the following table. To carry time series analysis for the 7 selected earthquakes in performance level of life safety and progressive collapse for two frames in original and improved form, the accelerations are scaled for two hazard level of 1 and 2 with probability of 2 and 5 percent during 50 year of useful life of structure, respectively.

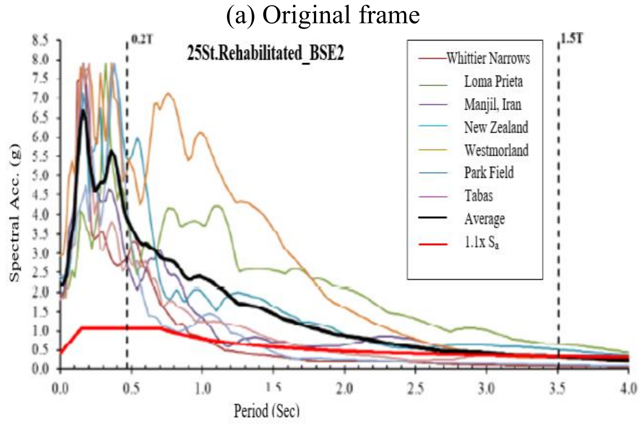
Records of different accelerators have been scaled for each frame separately. This process was computed for both original frame and also the frame with eccentric bracing in which it was employed along with vertical link to improve seismic performance of the concrete frame. However, the results are presented in Fig. 6 only for the frame with 25 stories. For the other frames including those with 15 and 20 stories, the trend and pattern is roughly the same but the magnitude of the spectral acceleration has lower values compared with the 25 story frame. The figure illustrates spectral acceleration for hazard level 2 while for the level 1 the magnitude of the acceleration is lower.

Table 3 Characteristics of the applied accelerators

Accelerator	Station	PGA (m/s^2)		Distance from fault (km)
		X direction	Y direction	
Whittier Narrows	90079 Doweney-Birchdale/180	0.243g	0.299g	56.8
Loma Prieta WVC	CDMG 58235 Saratoga-W Valley Coll.	0.255g	0.332g	23.7
Manjil, Iran	BHRC 99999 Abbar	0.515g	0.496g	40.43
New Zealand A-MAT	9999 Matahina Dam	0.256g	0.344g	24.23
Westmorland	5169Westmorland Fire Sta/90	0.368g	0.496g	35.6
Park FieldTMB	CDMG 1438 Temblor pre-1969	0.357g	0.272g	26.1
Tabas DAY	Dayhook	0.328g	0.406g	27.0



(a) Original frame



(b) Rehabilitated frame

Fig. 6 Spectral acceleration against period

As observed from Figs. 6(a)-(b), the acceleration graph versus period has higher peaks here representing spectral acceleration for the original frame prior to be supplied with bracing. Moreover it has higher value of 1.5T where *T* represents period in second. The highest magnitude for the averaged acceleration reaches 7.5 for the original frame while the magnitude decreases when the frame is supplied with the eccentric bracing and vertical link.

In this research, performance-based plastic design (PBPD) was used for seismic rehabilitation of high rise concrete frames with eccentric bracing and vertical link. In

Table 4 Link beam sections for different conditions

Story	15-story frame				20-story frame				25-story frame			
	BSE-2		BSE-1		BSE-2		BSE-1		BSE-2		BSE-1	
	Sec.	($ØV_n$)	Sec.	($ØV_n$)	Sec.	($ØV_n$)	Sec.	($ØV_n$)	Sec.	($ØV_n$)	Sec.	($ØV_n$)
25									IPE 140	85	IPE 100	52
24									IPE 180	125	IPE 140	85
23									IPE 220	171	IPE 180	125
22									IPE 240	197	IPE 200	148
21									IPE 270	237	IPE 220	171
20					IPE 200	148	IPE 160	105	IPE 270	237	IPE 220	171
19					IPE 270	237	IPE 200	148	IPE 300	285	IPE 240	197
18					IPE 300	285	IPE 240	197	IPE 300	285	IPE 240	197
17					IPE 330	332	IPE 270	237	IPE 330	332	IPE 270	237
16					IPE 360	385	IPE 300	285	IPE 330	332	IPE 270	237
15	IPE 180	125	IPE 160	105	IPE 400	462	IPE 300	285	IPE 330	332	IPE 270	237
14	IPE 240	197	IPE 200	148	IPE 400	462	IPE 330	332	IPE 400	462	IPE 300	285
13	IPE 300	285	IPE 240	197	IPE 400	462	IPE 330	332	IPE 400	462	IPE 300	285
12	IPE 330	332	IPE 270	237	IPE 450	570	IPE 360	385	IPE 400	462	IPE 360	385
11	IPE 360	385	IPE 300	285	IPE 450	570	IPE 360	385	IPE 400	462	IPE 360	385
10	IPE 360	385	IPE 300	285	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462
9	IPE 400	462	IPE 330	332	IPE 450	570	IPE 400	462	IPE 500	687	IPE 400	462
8	IPE 400	462	IPE 330	332	IPE 450	570	IPE 400	462	IPE 500	687	IPE 400	462
7	IPE 400	462	IPE 330	332	IPE 500	687	IPE 400	462	IPE 500	687	IPE 400	462
6	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 500	687	IPE 400	462
5	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 550	824	IPE 450	570
4	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 550	824	IPE 450	570
3	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 550	824	IPE 450	570
2	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 550	824	IPE 450	570
1	IPE 450	570	IPE 360	385	IPE 500	687	IPE 400	462	IPE 550	824	IPE 450	570

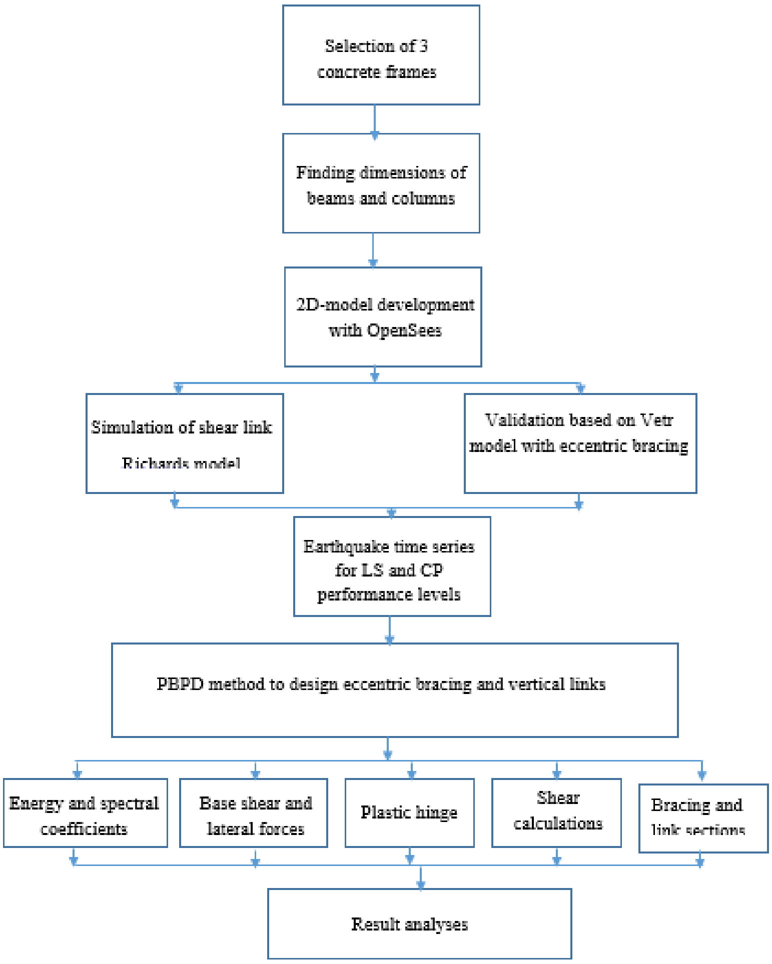
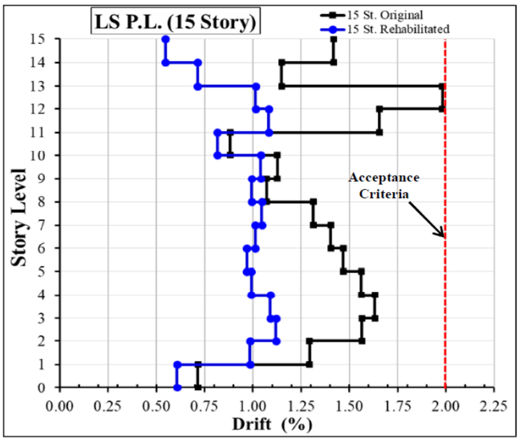
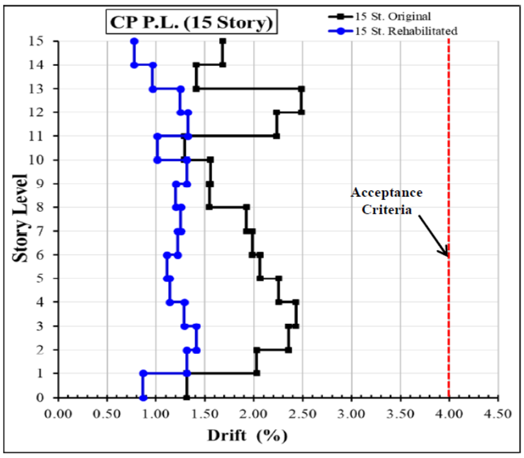


Fig. 7 Flow chart of the study considering PBPD for seismic rehabilitation

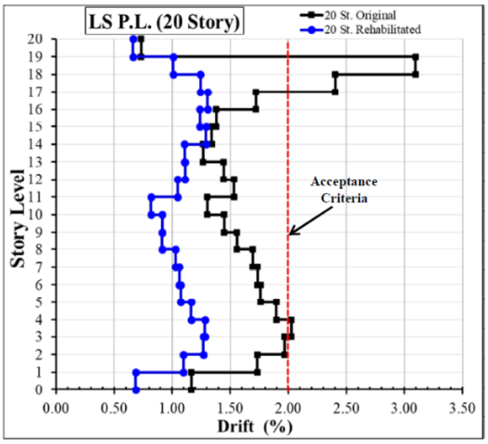


(a) LS

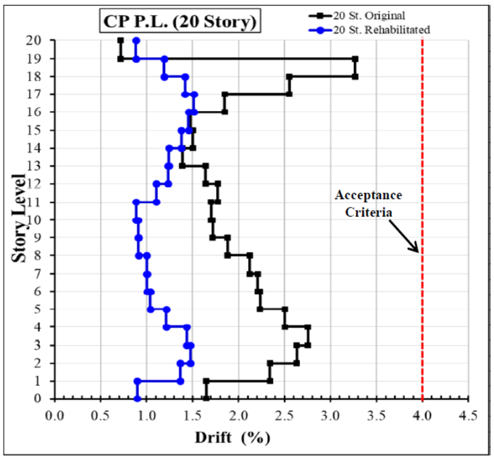


(b) CP

Fig. 8 Relative drift for 15 story frame for two hazard levels



a) LS



(b) CP

Fig. 9 Relative drift for 20 story frame for two hazard levels

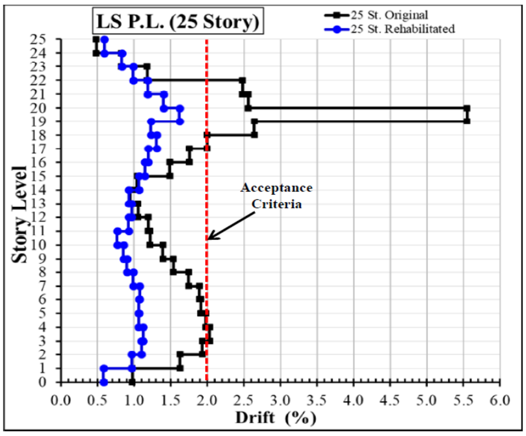
this method, the designed shear for the desired hazard level is computed as the required work to push the structure monotonically up to the design target drift which is equal to the elastic input energy using inelastic response spectra for elastic-plastic single degree of freedom systems (Goel *et al*. 2010, Liao 2010). Thus, the new distribution of force is carried out according to maximum shear of stories obtained from non-linear analysis. Considering displacement of target and desired yielding mechanism and energy balance relationships, the base shear can be derived. To compute distribution of base shear in the height, the method proposed by Lee *et al.* (2004) is used in which a=0.75. Afterwards, bracings are designed considering base shear and its distribution along the structure height. The link beam sections in each story for three frames and for two hazard levels are presented in Table. 4.

Flow chart of performance-based plastic design (PBPD) for seismic rehabilitation of high rise frames with eccentric bracing and vertical link is presented in Fig. 7.

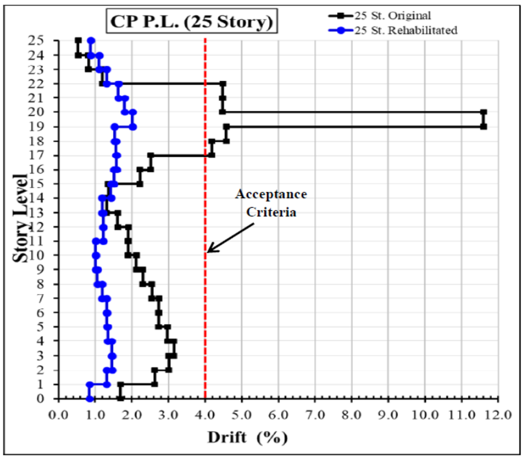
3. Results and discussion

Using OpenSees software, the research proceeded for concrete frames in two modes of original and rehabilitated under scaled earthquakes. Moreover, accurate analysis for nonlinear time series was carried by the software to find relatively maximum displacement in the middle stories where the frame is subjected to a definite earthquake in different performance level. Deformation of the frames is computed when relatively maximum displacement in the middle stories is occurred. This is obtained from analyses of nonlinear time series for the selected earthquakes. Figs. 8 to 10 illustrates relative deformations for the original and improved frames by means of the proposed seismic rehabilitation method. These computations was repeated for 7 selected earthquakes and two hazard level (BSE-1 and BSE2) where represent life safety (LS) performance level and collapse prevention (CP), respectively. Also, in these figures, relatively maximum allowable displacement based on code FEMA356 for performance level of CP and LS with the values of 2% and 4% are depicted.

Generally, the figures show that concrete frames in their original form for both performance levels have a completely nonlinear distribution of relatively displacement along the height of structure. The relatively maximum displacement for the 15, 20 and 25 story frames are happened in the 12st, 15st, and 18st level, respectively. In other words, with increasing in the number of stories of the original mode, the levels with the maximum drift moves toward the upper levels. As observed from the figures, the maximum drift in the middle stories significantly decreases when eccentric



(a) LS



(b) CP

Fig. 10 Relative drift for 25 story frame for two hazard levels

bracing with vertical link are added to the structure for rehabilitation purpose. Generally speaking, it can be derived that eccentric bracing with vertical link is an efficient method to improve seismic performance of the concrete frames. The results are valid for both hazard levels and for frames with different levels ranging from short to tall buildings. The results illustrated in Figs. 8 to 10 indicates that the original frames have frequently higher values of drift in which it decrease when the structures supplied with the eccentric bracing. Maximum and minimum drifts for the 15, 20, and 25 story frames before and after rehabilitation for LS and CP performance levels and their difference are given in Table 5.

According to Table 5 it can be obtained that the eccentric bracing with vertical link decrease maximum drift for all the frames ranging from short to tall buildings. However, for taller frames, the bracing performs more efficiently in a way the maximum drift decreases in a higher rate than those of medium and short frames. Moreover, with increasing in the earthquake intensity considering performance level of CP, the relative decrease in maximum drift will increase. Thus, with increasing in in the stories and earthquake magnitude, performance of the vertical link and its efficiency increases and this type of link is more influential to improve seismic performance of the existing buildings.

Table 5 Relative decrease in maximum drift for rehabilitated frame compared to original frames

Frame-mode	LS performance level			CP performance level		
	Level	Max. drift	Decrease (%)	Level	Max. drift	Decrease (%)
15s -original	12	1.98	43.43	13	2.486	68.79
15s-rehabilitated	3	1.12		15	0.776	
20s -original	19	3.096	58.33	19	3.267	53.69
20s-rehabilitated	15	1.290		17	1.513	
25s -original	20	5.553	70.84	20	11.598	82.63
25s-rehabilitated	20	1.619		20	2.015	

The main reason for this improvement is because of transformation of bigger shear forces to the vertical link when strong earthquakes are happening. Therefore, large non-elastic deformations are concentrated in the shear link and remarkably perform better than the elastic deformation mode (shear link deformation in small earthquakes) for energy absorbing and damping of lateral forces. Moreover, it was found that 15, 20 and 25 story frames with eccentric bracing and vertical link are performing as an intelligent system with increasing strength and lateral stiffness than original mode. Furthermore, the drifts alongside of the height are uniformed and make it capable to decrease maximum drift about 40 to 80 percent. It can be observed that eccentric bracing with vertical link has a great consistency from structural point of view in any seismic level.

Evaluation of plastic rotation capacity of beams and columns is an important issue to avoid structural brittle collapse and also to improve its seismic behavior. In this regard, after each nonlinear static analysis, moment curvature was obtained and subsequently final or ultimate curvature ($\emptyset_u$) was computed for the points of the elements under consideration. Finally, plastic rotation of beams and columns are calculated using Eqs (7)-(8).

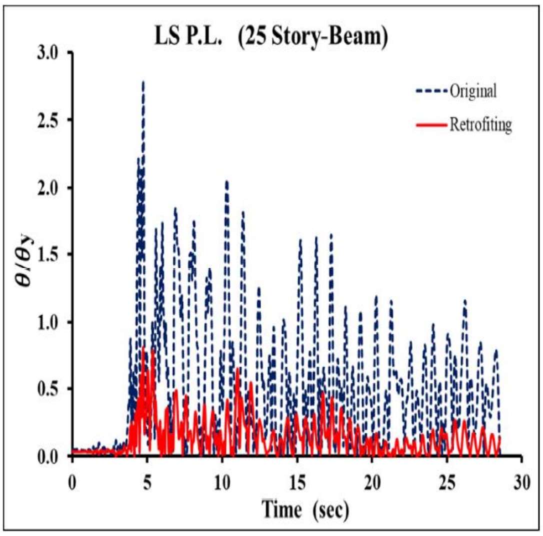
$$\theta_P = (\emptyset_u - \emptyset_y)L_P \tag{7}$$

where θ_P= plastic rotation, L_P=length of the plastic hinge and $\emptyset_y$=yielding curvature that can be expressed as

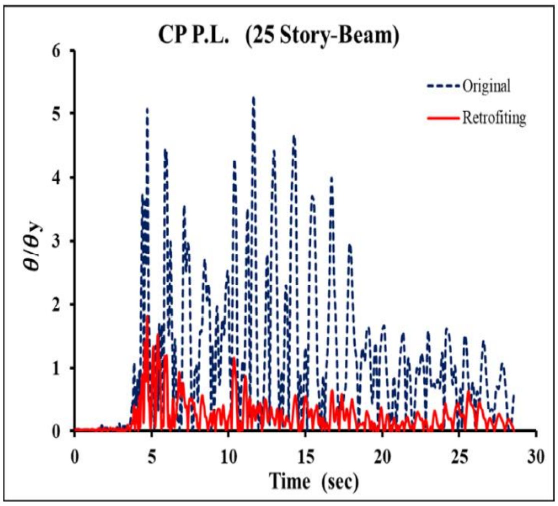
$$\emptyset_y = \frac{M_y}{E_c I_{cr}} \tag{8}$$

where M_y, E_c, and I_{cr} represent elastic moment, concrete elasticity module, and critical inertia moment, respectively. Further details about computation of the abovementioned parameters can be found in Zou and Chan (2005) and Priestley *et al.* (1996). For the 25 story frame, the results representing the ratio of $\frac{\theta}{\theta_y}$ for frames before and after rehabilitation are computed. These results are illustrated in Figs. 11 and 12 for critical beam and column of the frame, respectively.

From Figs. 11 and 12, it can be inferred that rehabilitation by the means of eccentric bracing with the vertical link improves performance of the frames compared to the original case for both hazard levels. Moreover, this improvement can be observed for the whole period (30

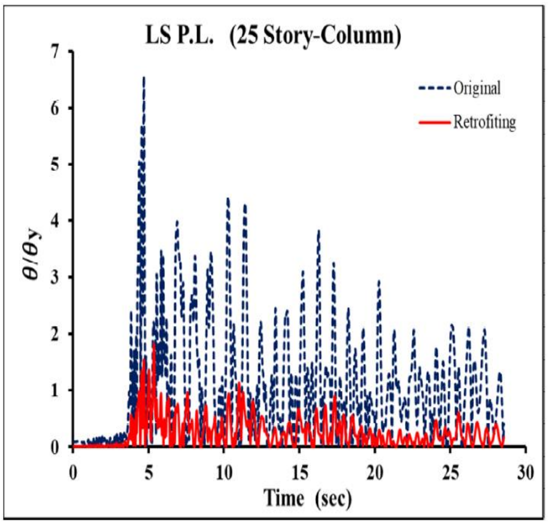


(a) LS

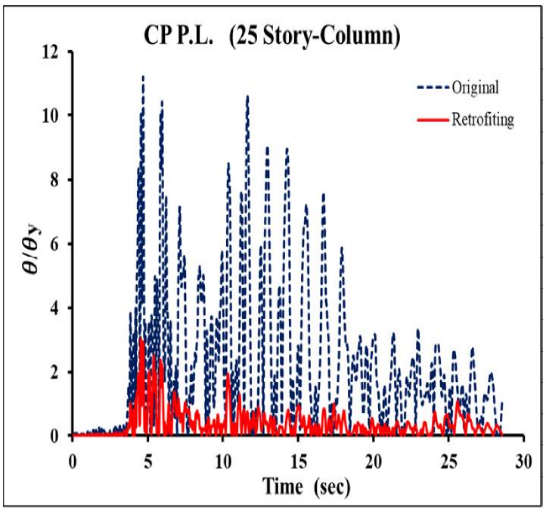


(b) CP

Fig. 11 The ratio of θ/θ_y for critical beam of 25 story frame and for two hazard levels



(a) LS



(b) CP

Fig. 12 The ratio of θ/θ_y for critical column of 25 story frame and for two hazard levels

sec.). For both beam and column sections the bracing has improved performance of the frames, remarkably. According to the figures, it can be derived that the relative decrease in $\frac{\theta}{\theta_y}$ of the critical beams and for the improved frames compared with those of the original ones is about 71 and 66 percent when computed for LS and CP modes, respectively. Similarly, the results for the critical columns provides a relatively decrease about 72 and 73 percent. These decreases imply remarkable improvement in ductility of concrete members because the shear link damper performs as a fuse to absorb the earthquake horizontal force and the plastic behavior of structures yield desired performance with the least damage to them.

4. Conclusions

The current research investigated efficiency and performance of eccentric bracing for three frames of 15, 20, and 25 stories to improve their seismic behavior. To implement the numerical model, OpenSees software using finite element approach was employed and the seismic behavior was simulated using nonlinear time series of 7 far field earthquake records. The results related to maximum displacement in the middle stories for the hazard levels of life safety (LS) and collapse prevention (CP) were analyzed. Main findings of the study can be summarized as the following remarks.

- Original concrete frames for both performance levels have non-uniform distribution of relative displacement along the structure height. Therefore, it can be shown that plastic behavior of the concrete frames indicating their hysteresis behavior against seismic load are distributed along the frame height non-uniformly.
- With increasing in the structure stories from 15 to 25, the story with maximum relative displacement moves toward the upper levels. On the other hand, the other stories have frequently lower capability to damp the imposed energy to the structure. Thus, the original

buildings (before rehabilitation) lacks of a lateral load resistant system to represent a hysteresis behavior and subsequently fails to make uniform deformation under energy entry to the structure.

• In case of supporting the frame with eccentric bracing and vertical link, the non-uniform distribution of relative displacement in the middle levels varies along the height remarkably. This variation is carried out by decreasing the ratio of maximum to minimum displacement about 50 to 90 percent. Moving from 15 to 25 story frames and also from LS to CP performance level, the abovementioned ratio will increase.

• The concrete frames with eccentric bracing and vertical link performs as an intelligent system leading to increase in strength and lateral stiffness compared to original frames. In this way, the structure is expected to experience a uniform distribution of displacements along the height. Moreover, the maximum displacement happening in the middle stories decreases about 40 to 80 percent for different performance levels.

• The proposed rehabilitation approach decreases the ratio of rotation of plastic hinge to elastic limit rotation for both columns and beams significantly. This decrease is in a range of 51 to 71 percent for the critical beams of 15 to 25 story frames and the corresponding decrease for columns are about 64 and 73 percent, respectively.

Findings of this study demonstrate that applying eccentric bracing with vertical links improves seismic efficiency and performance significantly. The method can be successfully employed to enhance ductility of concrete frames and life safety and also to prevent probable collapse of building due to near field and far field ground motions even though its efficiency may be less for the near field motions. The method is consistent and compromising in terms of plastic rotation and maximum displacement features as well as its time and cost efficiency. Exploring efficiency of the proposed method for near field ground motions can be considered as the future direction of the study.

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