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INVESTIGATING OF SEISMIC PARAMETERS OF RC FRAMES REHABILITATED BY ECCENTRICALLY BRACING WITH VERTICAL LINK

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ABSTRACT

Most of existing concrete structures do not have suitable seismic performance due to various reasons, therefore they need seismic rehabilitation. One of the seismic rehabilitation method in structural level is using of steel bracing. New investigation of steel bracing can be referred to eccentrically bracing with single vertical link. This method of rehabilitation provides many advantages such as increasing in ductility, stiffness, lateral resistance, architectural compatibility, low weight, and the fewest changes in primary structural system. In this paper, two existing 3- and 9-story RC frames are assessed on the basis of FEMA356. Eccentrically bracing with single vertical link is used for seismic rehabilitation of these frames. The results of nonlinear time history analysis based on the maximum inter-story drift, maximum roof displacement and plastic rotation in critical elements of original and rehabilitated frames for two performance levels of Life safety (LS) and Collapse Prevention (CP) are presented. The results indicate that single vertical link can lead structures to the desired performance level with minimum cost and braced span number.

KEYWORDS

Seismic rehabilitation, RC frames, eccentrically bracing, Time history, Shear link

INTRODUCTION

The possibilities of severe earthquakes due to natural geology conditions of structure site on one hand and the design and construction of many RC (Reinforced Concrete) buildings that seismic loading criteria are not observed or due to changes of these criteria, their seismic loading are underestimated on the other hand, make the seismic evaluation and rehabilitation essential for these buildings. Recent earthquakes around the world have shown that non-ductile (gravity load-designed) RC structures are so vulnerable to the earthquake that it causes severe damage or complete collapse. Various methods have been proposed for the seismic rehabilitation of RC structures, that each of them has their own advantages and disadvantages. One of the common methods among researchers is applying of new structural members as steel braces to RC structures [1-8]. Steel bracing are often used for seismic retrofitting of RC buildings, in contrast, while they are subjected to strong ground motions, the buckling of the braces leads to loss of lateral stiffness and strength of the structural system [9]. Thus, seismic retrofitting of RC buildings with steel bracing that may lead buckling cannot be a reliable retrofitting solution. Using of eccentrically bracing with vertical link, not only eliminates the probable buckling, but also leads to reduction of large inelastic deformations of RC members. Accordingly, this study is motivated to a seismic rehabilitation system that is capable of dissipating the earthquake input energy without buckling of braces. In recent years, seismic rehabilitation method of using the eccentrically bracing with vertical link as an additional energy dissipation element has been widely used. Ghobarah and Abou Elfath (2001) examined the distribution effect of eccentrically bracing along the height of the RC frames in cases of stories drift and damage indexes [10]. Seismic assessment of RC structures that were rehabilitated by eccentrically bracing with single vertical link was studied by Durucan and Dicleli (2010) [11]. Results show that plastic deformations of structural components were less, compared to conventional approaches. According to the research carried out by Sahoo and Rai (2010), the effect of changing the vertical link material from steel to aluminium is discussed and the results showed a significant increase in the energy absorption capability of seismic rehabilitated non-ductile RC frames using eccentrically bracing with single vertical link [12]. The failure modes of the connections between vertical link with slab, and the braces with RC columns and the beams by a series of full-scale experiments on RC building rehabilitated by eccentrically braces with vertical link were assessed by Mazzolani (2008) [13]. This method also provides many advantages such as increasing in ductility, stiffness, lateral resistance, architectural compatibility, low weight, and the fewest change in primary structural system. Since, the approach of seismic rehabilitation by usage of eccentrically bracing with vertical link has not been done before, more studies about this system are required. The Proposed Seismic Rehabilitation System (PSRM), as it is shown in Figure 1, can be applied in various configurations which including: (a) link and braces are directly connected to the RC members through steel plates by bolts and epoxy grouting, (b) the link is connected to steel beam collector, which is attached to the concrete beam, and the rest of the members are connected to the RC members via steel plates or (c) link and braces are housed in a rectangular steel frame (housing frame) where the steel frame is connected to the RC members by bolts and epoxy grouting.

Research Outline

In this paper, the basis of seismic rehabilitation method by usage of eccentrically bracing with single vertical link is studied. In order to evaluate the seismic performance of PSRM, two existing non-ductile RC frames, 3 and 9 stories are selected according to Ref. [14] and they will be seismically evaluated according to FEMA 356 [15] after numerical modelling in OpenSees software [16]. The PSRM is determined after the seismic evaluation of frames and then the frames are designed based on the desired performance level. A performance based approach that includes nonlinear static analysis and response spectrum analysis is used for the seismic rehabilitation design of the frames which is considered in this study. Then, numerical modelling of single vertical link is done in OpenSees according to the respective experimental results. After that, Nonlinear Time History (NLTH) analysis are conducted to assess the maximum inter-story drifts, maximum roof displacement, plastic rotations of critical members and deformed shapes of frame at the instant of maximum stories drifts in two different seismic performance levels (Life Safety and Collapse Prevention PLs). Then, the results are compared with restrictions of FEMA 356.

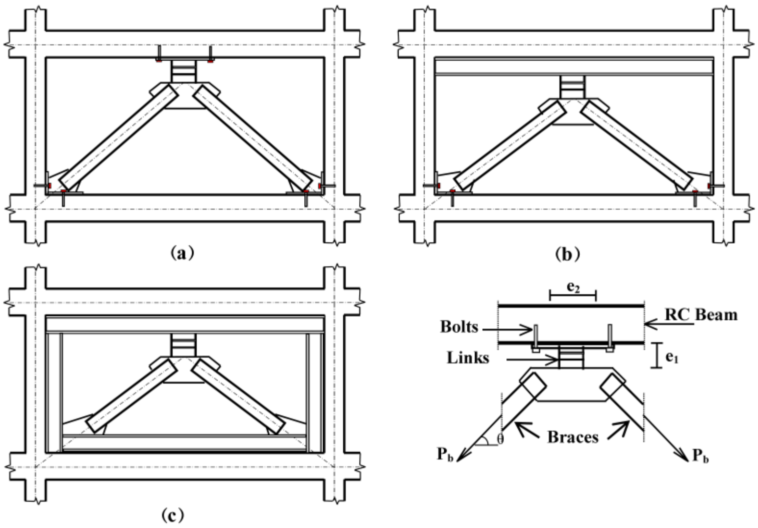

Fig. 1 – The configurations details of proposed seismic rehabilitation method: (a) PSRM-1; (b) PSRM-2; (c) PSRM-3

Details of the Considered Frames

Analytical models to evaluate PSRM are two 2D frames of non-ductile RC office buildings, 3 and 9 stories that are designed according to the ACI code [17] by considering only the gravity loads. The design concrete strength is 21 MPa and the design steel strength is 300 MPa and the modulus of elasticity is 200000 MPa. The design dead load and live load for the frames are taken as 35 kN/m and 12 kN/m respectively. The building mass due to the weight of all structural and nonstructural elements is equal to 945kN/floor. The frame elevations are shown in Figure 2 and the structural details of frames are given in Table 1.

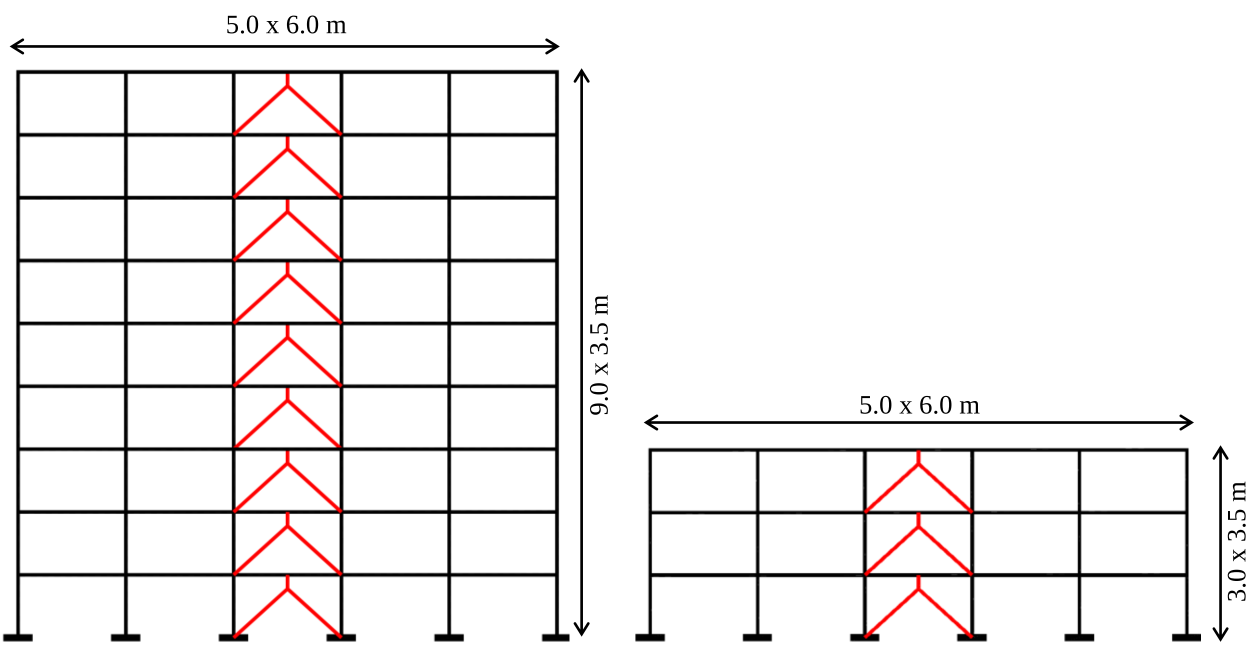

Fig. 2 – The RC seismic rehabilitated frames with the PSRM

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Tab. 1 - Structural details of 3 and 9 story RC frames

Story		exterior column		interior column		interior beam		
		Size	Steel	Size	Steel	Size	Top Steel	Bottom Steel
3 Story	1-3	300x300	4Φ19	400x500	8Φ19	250x600	5Φ19	2Φ19
9 Story	1-3	500x500	8Φ22	600x600	8Φ25	250x600	5Φ19	2Φ19
	4-6	400x400	8Φ19	500x500	8Φ22	250x600	5Φ19	2Φ19
	7-9	300x300	4Φ19	400x500	8Φ19	250x600	5Φ19	2Φ19

Design of Proposed Seismic Rehabilitation System

Seismic rehabilitation objectives which are used in this paper consist of basic and optimal objectives. In basic and optimal objectives, the frames should achieve the Performance Levels (PLs) of Life Safety (LS) and Collapse Prevention (CP). In LS PL, low or repairable structural and non-structural damage is expected for moderate earthquake excitations (10% possibility of exceedance in 50-year). In CP PL, irreparable or hardly repairable structural and nonstructural damage will be happened but collapse is not expected for major earthquake excitations (2% possibility of exceedance in 50-year). In FEMA 356, the allowable criteria of PLs are mainly defined by plastic rotation limits for the RC members.

In order to design the rehabilitation details of the frames; a performance based approach is used, that is mainly based on the equal energy dissipation principle. In this method, the monotonic energy dissipation capacities of the frames based on the roof displacement are calculated and compared in two regions of linear elastic region which is obtained from Response Spectrum (RS) analyses and nonlinear inelastic region which is obtained from Nonlinear Static Pushover (NLSP) analyses. The difference in area between the two regions of elastic and inelastic of base shear force versus roof displacement curve is equal to the required additional energy that should be absorbed by the PSRM [11].

The steel characteristics which are used for the design process of PSRM have yield strength of 250 MPa and the modulus of elasticity of 200000 MPa. Braces and shear links details of PSRM for frames are available in Table 2.

Tab. 2 - Details of PSRM for 3 and 9 story RC frames

Story		Braces	Frame	Single -VL	
		Section	Section	Section	e_1 (mm)
3 Story	1	2UNP140	IPE240	IPE400	820
	2	2UNP120	IPE220	IPE360	760
	3	2UNP120	IPE220	IPE330	700
9 Story	1-3	2UNP140	IPE270	IPE450	860
	4-6	2UNP140	IPE240	IPE400	760
	7-9	2UNP120	IPE220	IPE360	700

PSRM Details Design

At first, the number of brace-link system and the shear strength that is required at each floor, Q_{Li}, is calculated based on seismic rehabilitation design procedure. In order to prevent lateral strength and stiffness degradation that is associated with braces buckling, the shear link is designed in such a way that yields before brace buckling. Shear yielding provides more effective energy dissipation than flexural yielding [18] and hence, it is appropriate for the design of vertical link in the PSRM. The shear yield strength, *Vy*, of I section according to AISC 2010 [19] is given by:

$$V_y = 0.6F_yA_w \tag{1}$$

Where F_y is the yield strength of steel, and A_w is the cross-section area of the web of the link. Setting $V_y = Q_{Li}$, the cross-section area of the web at i story is obtained by $A_{wi} = \frac{Q_{Li}}{0.6F_y}$. Then an I section with the calculated web area, A_w, is chosen. The middle stiffeners for the link web are designed based on AISC 2010. For vertical shear link that its moment diagram is shown in Figure 3, in order to obtain the conditions of shear plastic hinge formation before flexural plastic hinge formation due to the unequal moments at both ends of the link, the Equation 2 for determining the length of shear link, (h_s) is proposed by Eurocode 8 [20].

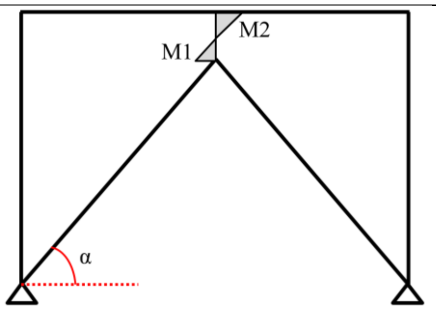

Fig. 3 – Moment diagram of frame with single vertical link

$$h_s \leq \frac{0.8 \times (1+k)M_P}{Q_{Li}} \quad ; \quad k = \frac{M_2}{M_1} \quad ; \quad M_2 \leq M_1 \tag{2}$$

Where, M_1 and M_2 are the unequal moments at both ends of the shear link and M_p is the plastic moment of shear link. According to proposed details in the region of the compression bracing buckling, the axial tensile and compressive forces of the both tension and compression braces are equal to the buckling load, P_b. Consequently, in order to prevent the buckling of the compression braces, the sum of the horizontal components of the buckling loads of the two braces must be larger than the yielding strength of the link multiplied by the over-strength factor (φ_s). Thus:

$$2P_b \cos\alpha \geq \varphi_s Q_{Li} \tag{3}$$

In Equation 3, α is the angle that the braces make with horizontal line. By solving P_b from the above equation, the required buckling strength of the brace can be obtained as:

$$P_b = \frac{\varphi_s Q_{Li}}{2 \cos\alpha} \tag{4}$$

The braces are selected in order to have a minimum buckling capacity (P_b).

Verification of Single Vertical Link

According to Ref. [21], the shear link was modelled as a linear element with six nonlinear rotational and translational springs at each end. So that, 3 rotational bilinear springs and 3 translational bilinear springs were used to represent the inelastic flexural bahavior of plastic hinge and inelastic shear behaviour of the link web located at the end of the link that is represented by the multilinear functions shown in *Figure 4*. For numerical modeling of vertical link in this paper, the model of Ref. [22] is chosen; this model is same as the model of Ref. [20], but with different stiffness values for shear springs. The values of M_y and V_y are considered equal to M_P and $0.9V_P$, respectively. The relations between the moment-rotation and the shear-displacement of shear link with the stiffness values are shown in Figure 4. The values of $\mathrm{K_{1M}}$ and $\mathrm{K_{1V}}$ are respectively equal to $\frac{3\mathrm{EI}}{\mathrm{e}}$ and $\frac{\mathrm{GA_{web}}}{\mathrm{e}}$. In which, 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.

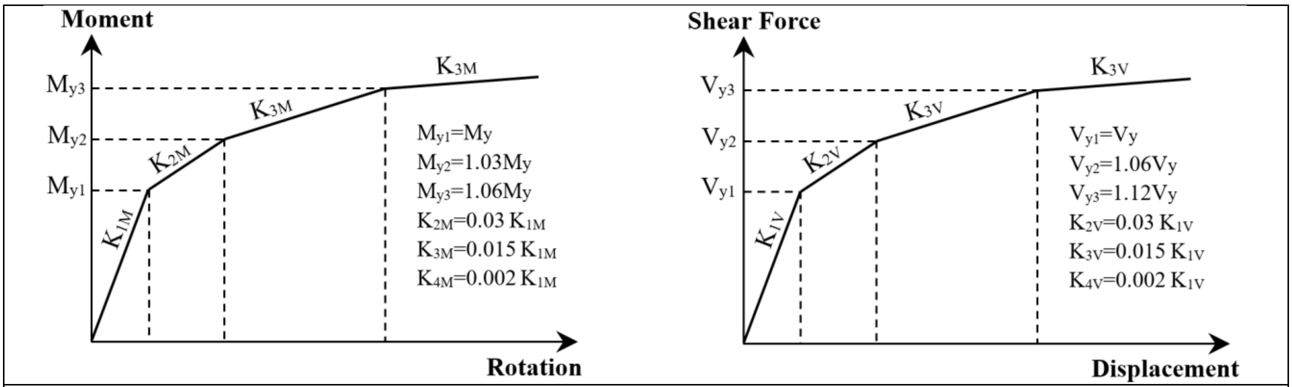

Fig. 4 – Moment–rotation and shear force–shear deformation relationships of the steel link

For comparing the results of numerical modelling of single vertical link with experimental results, the experimental model of Ref. [23] is chosen. The experimental model is a single story, single span frame with eccentrically braces with single vertical link. The design steel is ST37 with yield strength of 240 MPa, ultimate strength of 420 MPa and the modulus of elasticity of 200000 MPa as it is shown in Figure 5. The connections of beam to column, the braces to column and the columns to the floor is simple. The connection of single vertical link to beam and braces is welded on the basis of fix connection type. After the numerical analysis of considered frame under the loading according to the Figure 6, the hysteretic curve is formed as it is shown in Figure 7. By comparing this curve with hysteretic curve of experimental study, appropriate compatibility is observed that result in accuracy of modelling approach of single vertical link with behavioural model of Ref. [22].

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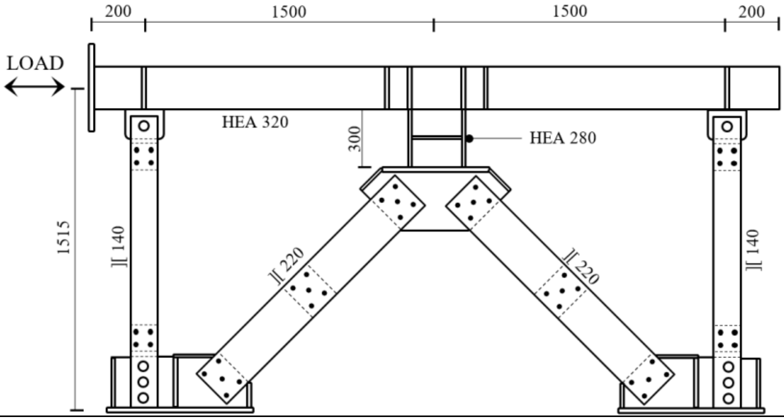

Fig. 5 - Details and configuration of eccentrically braced frame with single vertical link in the laboratory

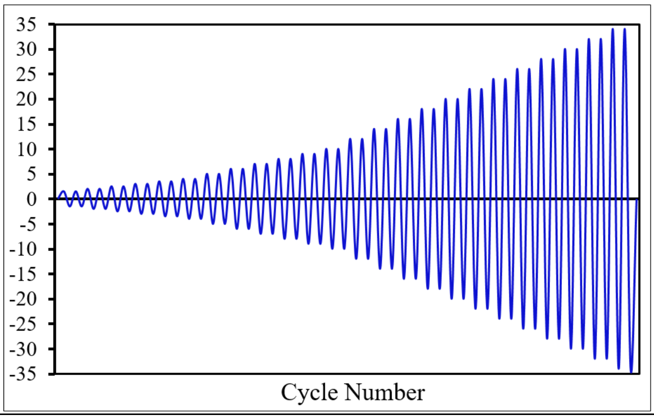

Fig. 6 - Loading protocol of eccentrically braced frame with single vertical link in the laboratory

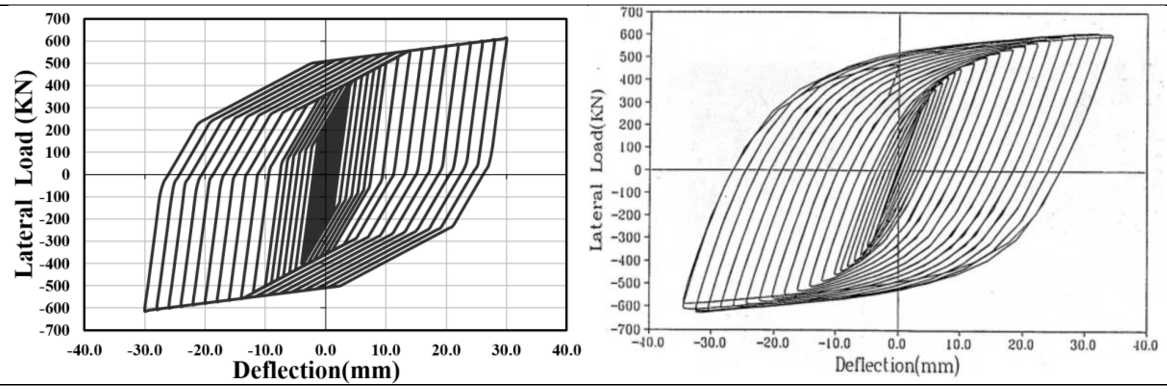

Fig. 7 - Hysteresis curves: (a) obtained from experimental work (b) obtained from modelling in OpenSees

Nonlinear Dynamic Analysis (Time History)

In this section, the seismic behaviour of the original frames and seismic rehabilitated frames (configuration of PSRM-3) under the ground motions is discussed. The main objectives of this study are the assessment of the maximum drifts, the changes of ductility and the energy dissipation. Behaviour of all beams and columns of frames are controlled by bending and shear forces, respectively. The allowable value of beam's plastic rotation to form nonlinear hinge at LS

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and CP PLs are equal to 0.02 and 0.025, and these amounts for column are equal to 0.005 and 0.01, respectively, also the allowable amount of shear link's plastic rotation are 0.11 and 0.14 radian for LS and CP PLs. The gravity load combination for combining with the seismic load is equal to $Q_G = 1.1(Q_D + Q_L)$, where Q_D is the dead load and Q_L is equal to 25% of the non-reduced design live load.

In order to calculate the plastic rotation of nonlinear hinges of RC members, firstly, the amount of ultimate curvature of each fiber section of the elements must be determined by results of moment-curvature, which are obtained via NLSP analysis and then the Equation 5 is used:

$$\theta_P = (\emptyset_U - \emptyset_y) \times L_P \tag{5}$$

Where θ_P is the plastic rotation, $\emptyset_U$ is the ultimate curvature, $\emptyset_y$ is the yield curvature that is defined according to Ref. [24]:

$$\emptyset_y = \frac{M_y}{E_C I_{cr}} \tag{6}$$

$$\text{for Beams}: \; M_y = 0.5 f_C \, B \, kd \left(\frac{kd}{3} - d'\right) + f_y \, Bd \, (d - d')\rho \tag{7}$$

$$\text{for Columns}: \; M_y = \frac{f_y \, B \, kd}{2n_{SC}} \left(\frac{D}{2} - \frac{kd}{3}\right) \frac{k^2}{1-k} \tag{8}$$

$$k = \sqrt{(\rho + \rho')^2 \, n_{SC}{}^2 + \left(\rho + \rho' \frac{d'}{d}\right) n_{SC}} \; - \; (\rho + \rho') n_{SC} \tag{9}$$

Where M_y is the yield moment, E_C is the modulus of elasticity of concrete, I_{cr} is the critical moment of inertia, that is equal to $0.5I_g$ where I_g is the moment of inertia of RC section without crack, ρ is the tensile steel ratio, ρ' is the compression steel ratio and also $n_{SC} = \frac{E_S}{E_C}$ is the proportion of the modulus of elasticity of steel to the modulus of elasticity of concrete. f_y is the yield strength of the tension steel and *d* is the effective depth, which is equal to the distance from the extreme compression fiber to the centroid of the tension steel, *d'* is equal to the distance from the extreme compression fiber to the centroid of the compression steel, B is section width. *k* is the neutral axis depth factor at the first yield and $n_{sc} = E_s/E_c$ where E_c and E_s are the moduli of elasticity of the concrete and the steel, respectively.

The assumption length of plastic hinge (L_P) is defined according to Ref. [25]:

$$L_P = 0.08L + 0.022 \, f_{ya} \, d_{bl} \tag{10}$$

Where, L is the considered element length in mm, f_{ya} is the yield strength of bars in MPa, d_{bl} is the diameter of bar in mm.

Tab. 3 - Details of selected earthquakes

EQ. NO	Year	Earthquake	Recording Station	PGA (g)	Vp (m/s)	EQ. Scale Factor			
						3St. BSE-1	3St. BSE-2	9St. BSE-1	9St. BSE-2
1	1987	Whittier Narrows	90079 Doweney-Birchdale/180	0.299	0.378	1.34	1.89	1.48	2.18
2	1989	Loma Prieta WVC	CDMG 58235 Saratoga-W Valley Coll.	0.332	0.625	2.02	2.84	2.09	3.07
3	1990	Manjil, Iran	BHRC 99999 Abhar	0.496	0.4378	1.01	1.42	1.04	1.54
4	1987	New Zealand A-MAT	99999 Matahina Dam	0.293	0.2107	1.68	2.36	2.18	3.2
5	1981	Westmorland	5169Westmorland Fire Sta/90	0.496	0.344	1.57	2.21	1.83	2.69
6	1966	Park FieldTMB	CDMG 1438 Temblor pre-1969	0.357	0.215	2.24	3.15	2.61	3.84
7	1987	Tabas DAY	9102 Dayhook	0.406	0.265	1.68	2.36	2.00	2.94

The selected earthquakes have similar characteristics such as: magnitudes from 4.5 to 8 Richter scale, and the shear wave velocity according to the site soil type is classified as type D and this velocity is equal to 182.88 to 365.76 m/s and the selected range of the maximum acceleration is between 0.2g-2g. The elastic acceleration response spectrum with the damping of 5% for each ground motion and the average of 7 ground motions are shown in Figure 8.

Design Spectrum (DS) for two earthquake hazard levels, BSE-1 and BSE-2 (Basic Safety Earthquake), is calculated according to ASCE 2010 [26]. At first, each of these ground motions are scaled for the two earthquake hazard levels (BSE-1 and BSE-2). Following ASCE 2010, the maximum difference in the range of 0.2T-1.5T between the average value of the seven ground motions with the 1.4 times of the DS should be equal to 10%. Details of 7 earthquakes and their scale factors for earthquake hazard levels are shown in Table 3. Also, the calculated DS for the two earthquake hazard levels including: BSE-1 and BSE-2 and the acceleration response spectra for the 7 ground motions for both frames of 3 and 9 story are shown in Figure 9.

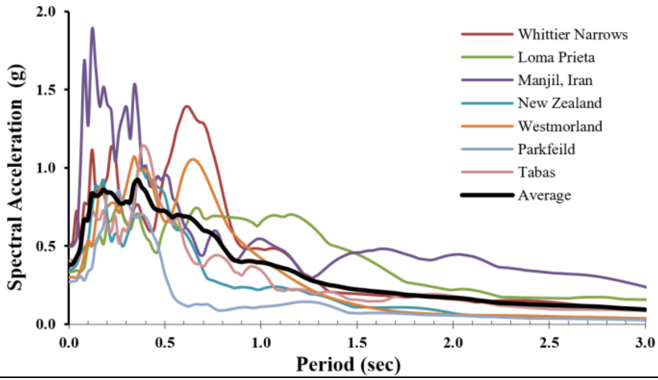

Fig. 8 - The acceleration spectrum for the selected earthquakes

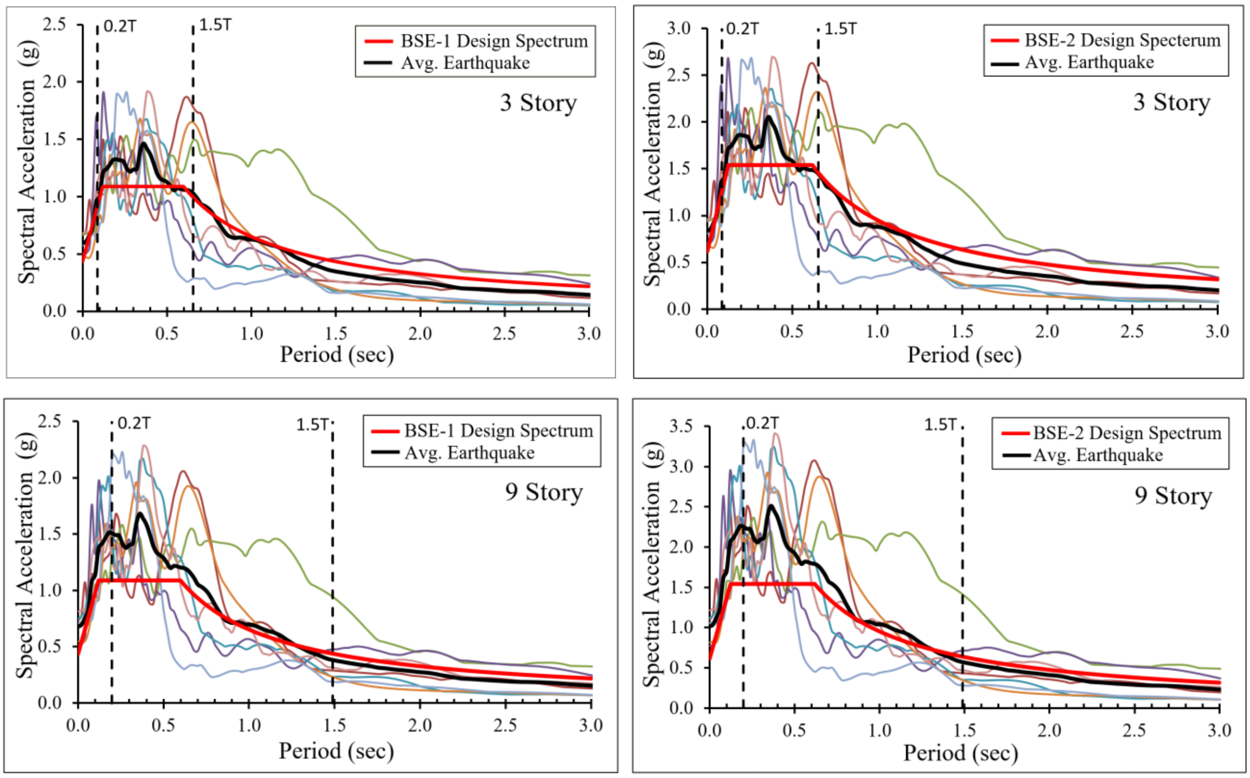

Fig. 9 - The site special acceleration spectrum and the selected earthquakes acceleration spectrum scaled to the site special acceleration spectrum

The maximum drifts of the original and seismic rehabilitated frames which are obtained from NLTH analysis for each performance levels, are shown in Figure 10. The frames rehabilitated with PSRM indicate more appropriate response than the original frames at the both performance levels. Moreover, regardless of the earthquakes characteristic (Such as frequency content) that is used in the analysis, the PSRM indicates more stable response. It means that the PSRM presents more uniform or similar response for all selected earthquakes.

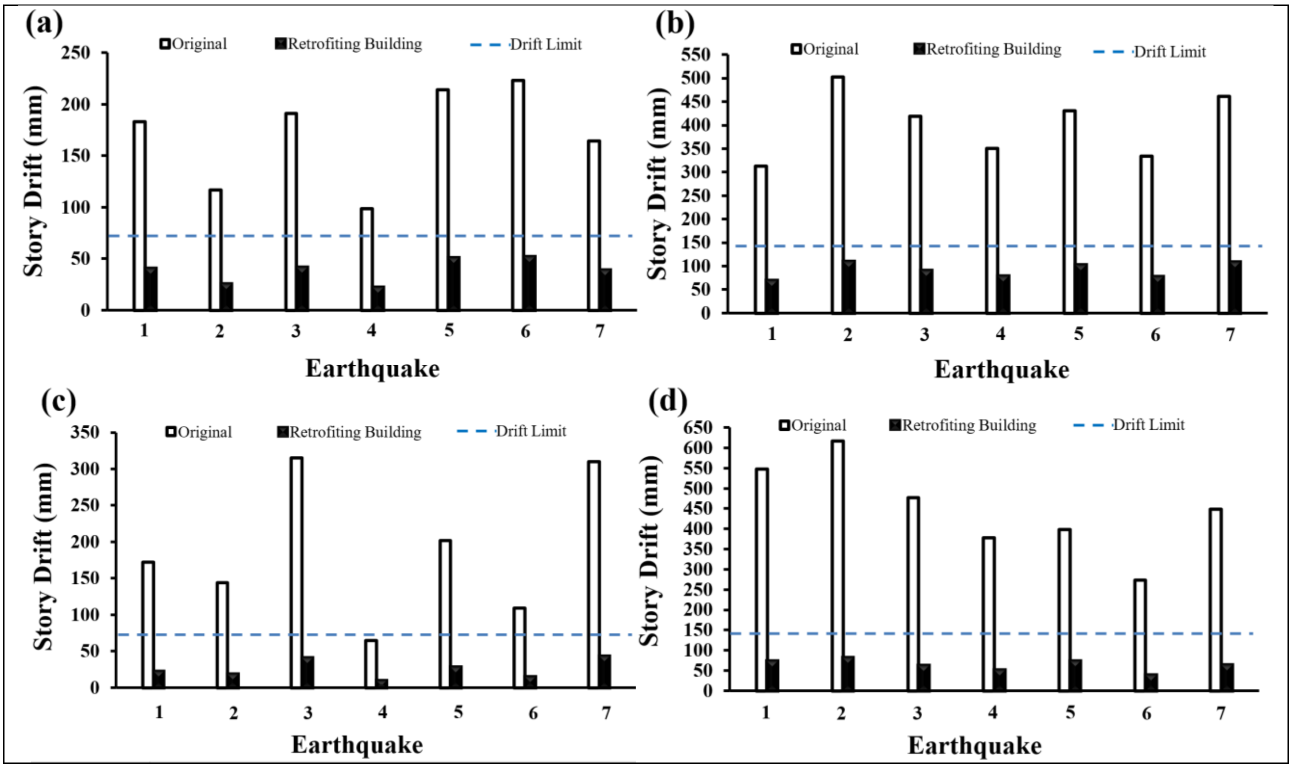

Fig. 10 - The maximum story drift for the original and seismic rehabilitated frames under NLTH analysis; (a) 3 story frame at LS PL. (b) 3 story frame at CP PL. (c) 9 story frame at LS PL. (d) 9 story frame at CP PL.

In order to evaluate the performance of the seismic rehabilitated frames with PSRM in comparison with the original frames, the maximum inter-story displacement in the frame's height is shown in Figure 11. The deformations of frames are obtained at the time of the maximum stories drift. The reduction amount of the average maximum stories drift under seven selected earthquakes for the seismic rehabilitated frames of 3 and 9 story in comparison with the original frames, in case of the LS PL are equal to 77% and 86%, in case of the CP PL are equal to 72% and 81% respectively. This indicates the relatively uniform behaviour of the PSRM in decreasing the story drift in various performance levels. Consequently, damages which are created by probable earthquake are severely reduced. Moreover, the frames equipped with the PSRM have the uniform lateral deformation pattern. So, the amount of energy dissipation will have a better distribution at the height of frames.

In order to assess the ductility improvement of RC members in the frames with PSRM, the critical beam and column of the first story (the critical beam and column at both frames) are selected and analysed under nonlinear time history method by the earthquake number 5, to compare the formed plastic hinge rotation to the elastic one(θ/θ_y) in the original case with the rehabilitation case. As it is shown in Figure 12, the reduction of the maximum ratio of (θ/θ_y) for 3 and 9 story rehabilitated frames in comparison with the original frames for beam are equal to 6.47, 9.25 and for column are equal to 5.17, 7.03 respectively, that indicates the significant improvement in ductility behaviour.

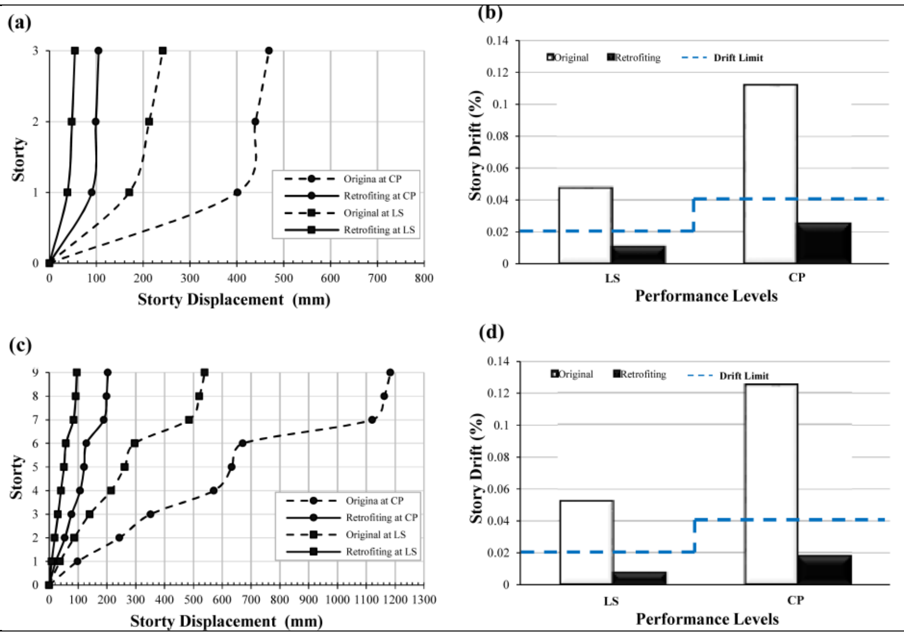

Fig. 11 - (a) The maximum inter-story drifts along the height of the 3 story frame, (b) the average inter-story drift for the 3 story frame, (c) The maximum inter-story drifts along the height of the 9 story frame, (d) the average of inter-story drift for the 9 story frame

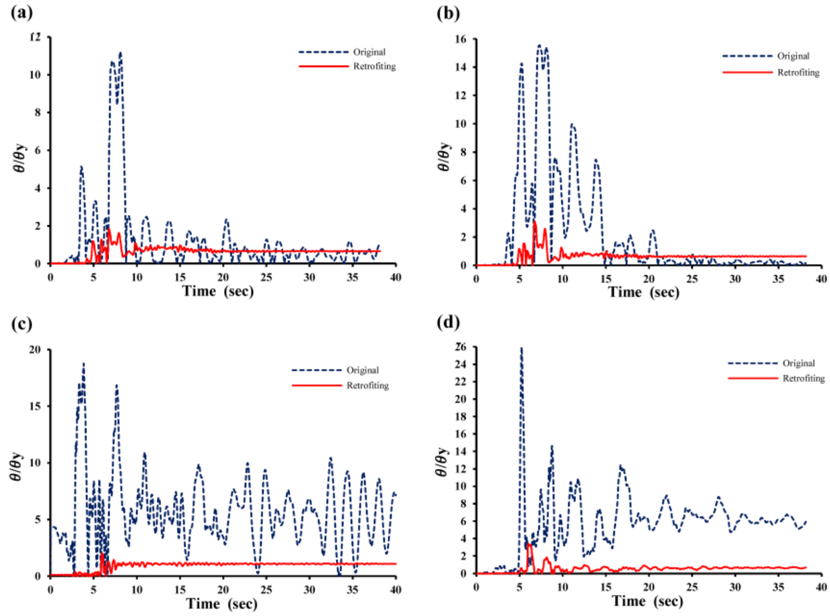

Fig. 12 - The ratio comparison of the ultimate rotation to the yield rotation for the critical elements of original and seismic rehabilitated frames under NLTH analysis for CP PL, (a) critical beam for 3 story frame, (b) critical column for 3 story frame, (c) critical beam for 9 story frame, (d) critical column for 9 story frame

RESULTS

In this paper, a new seismic rehabilitation method based on the performance is used for the seismic rehabilitation design of frames to ensure the satisfactory performance of the rehabilitated structures with the PSRM. The efficiency of this method was evaluated by performing the nonlinear time history analysis on two 3 and 9 story RC frames according to FEMA356. The results of NLTH have shown that the distribution of inter-story drift in the height of frames have more non-uniform behaviour by increasing the story level and it is because of the gravity loads that is governed in design of low rise frames in comparison with high rise frames. So the application of PSRM in low rise frames not only have no significant effect in the uniform distribution of inter-story drift but also decrease the inter-story drift. It is observed that in case of using the PSRM, a significant increase in ductility of RC components is achieved during the earthquakes. In the frames rehabilitated by PSRM, due to increasing in ductility, stiffness and lateral load strength capacity of frames, the damage indices have small values that unlike the original cases of frames which experience the significant structural damages.

REFERENCES

[1] Badoux M, Jirsa JO., 1990. Steel bracing of RC frames for seismic retrofitting. Journal of structural engineering, 116(1): 55–74.

[2] Bush TD, Jones EA, Jirsa O., 1991. Behaviour of RC frame strengthened using structural steel bracing. Journal of structural engineering,**117**(4):1115–26.

[3] Rodriguez M, Park R., 1991. Repair and strengthening of reinforced concrete buildings for seismic resistance. *Journal of earthquake spectrum,* **7**(3): 439– 459.

[4] Nateghi-Alahi F., 1995. Seismic strengthening of eight-story RC apartment building using steel braces. *Journal of engineering structure,* **6**(17): 461-455.

[5] Masri AC, Goel SC., 1996. Seismic design and testing of an RC slab–column frame strengthened by steel bracing. *Journal of earthquake spectra,* **12**(4): 645–666.

[6] Maheri MR, Sahebi A., 1997. Use of steel bracing in reinforced concrete frames. *Journal of engineering structure,* **19**(12): 1018–24.

[7] Maheri MR, Kousari R, Razazan M., 2003. Pushover tests on steel X-braced and knee-braced RC frames. *Journal of engineering structure,* **25**(13): 1697–1705.

[8] Ferraioli M, Avossa AM, Malangone P., 2006. Performance based assessment of RC buildings strengthened with steel braces. *Proceedings of the 2nd International Congress*, Naples, Italy.

[9] Dicleli M, Mehta A., 2009. Seismic retrofitting of chevron-braced steel frames based on preventing buckling instability of braces. Journal of Structural Stability and Dynamics, **9**(2):333-56.

[10] Ghobarah A, Abou Elfath H., 2001. Rehabilitation of a reinforced concrete frame using eccentric steel bracing. *Journal of engineering structure,* **23**:745–755.

[11] Durucan C, Dicleli M., 2010. Analytical study on seismic retrofitting of reinforced concrete buildings using steel braces with shear link. *Journal of engineering structure,* **32**: 2995-3010.

[12] Sahoo DR, Rai DC., 2010. Siesmic strengthening of non-ductile reinforced concrete frames using aluminum shear links as energy-dissipation devices. *Journal of engineering structure,* **32**:3548-3557.

[13] Mazzolani FM., 2008. Innovative metal systems for seismic upgrading of RC structures. Journal of constructional steel research, **65**(7): 882-895.

[14] Elfath A, Mohamed H., 1998. *Rehabilitation of non-ductile reinforced concrete buildings using steel systems*. PhD Thesis, University of McMaster, Hamilton.

[15] Federal Emergency Management Agency (FEMA356). *Prestandard and commentary for the seismic rehabilitation of buildings.* Washington DC, 2000.

[16] Mazzoni S, McKenna F, Scott MH, Fenves GL. Open system for earthquake engineering simulation (OpenSees). User command language manual, version 1.7.3. Richmond, CA (USA): Pacific Earthquake Engineering Research Center, University of California, Berkeley; 2006.

[17] American Concrete Institute (ACI 318-63). *Building code requirement for reinforced concrete*. Farmington Hills, MI. 1963.

[18] Kasai K, Popov EP., 1968. General behaviour of WF steel shear link beams. *Journal of structural engineering,* **112**(2): 362-82.
[19] American Institute of Steel Construction (AISC 341-10). *Seismic Provisions for Structural Steel Buildings.* Chicago, IL. 2010.
[20] European Committee for Standardization (EC8). *Design of structures for earthquake resistance*. Brussels, B. 2003.
[21] Ramadan T, Ghobarah A., 1995. Analytical model for shear-link behaviour. *Journal of structural engineering,* **121**(11):1574-1580.
[22] Richards PW., 2004. *Cyclic stability and capacity design of steel eccentrically braced frames*. PhD Thesis, Univ. of California, San Diego.
[23] Vetr MG., 1997. *Seismic behaviour, analysis and design of eccentrically braced frames with vertical shear links*. PhD Thesis, University of Technology Darmatadt, W. Germany.
[24] Zou XK, Chan CM., 2005. Optimal seismic performance-based design of reinforced concrete buildings using nonlinear pushover analysis. *Journal of structural engineering,* **27**:1289–1302.
[25] Priestley MJN, Seible F, Calvi GM., 1996. *Seismic design and retrofit of bridges*. John Wiley & Sons, ISBN: 978-0-471-57998-4, New York.
[26] American Society of Civil Engineers (ASCE/SEI 7). *Minimum Design Loads for Buildings and Other Structures*. Reston, VA. 2010.