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Evaluation of Seismic Performance Factors for RC Buildings Retrofitted by EBF with Single and Dual Vertical Links

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ABSTRACT

In this paper, seismic performance of RC buildings retrofitted by EBF with single and dual vertical links was investigated. In this procedure, various editions of Standard No 2800 were used. For this purpose, three RC intermediate moment frames of 4, 8 and 12 stories were modeled and designed by Standard No. 2800, second edition. In order to assessment of these buildings under modified loads, they were reloaded according to the Standard No. 2800, third and fourth editions. The results showed that the stress ratios in most of the columns were larger than one. So, the buildings were retrofitted by eccentrically braced frames with single and dual vertical links and their seismic performance were evaluated through the non-linear static analyses. In continue, response modification factors of these systems were calculated. The results show that these systems increase the stiffness and control the displacement of retrofitted buildings but lead to a significant reduction of the ductility. According to the obtained results in this research, the average of the response modification factors for each type of links was almost three.

KEYWORDS: Seismic performance factors, eccentrically braced frames, single and dual vertical links.

1. INTRODUCTION

A quick look at the process of the country's constructions in the past indicates that a large percentage of existing reinforced concrete buildings are not strong enough against earthquake or they have not been basically and appropriately analyzed, designed and made to resist against the forces caused by earthquake. Since the analytical and design standards and building codes are always changing and being completed, in one hand, lack of resources and space and on the other hand, the costs of deterioration and reconstruction caused the evaluation of the current status of buildings according to the modern standards and a wide range of researches to be done in the field of strengthening the existing structures in recent years. Many researchers have presented many analytical models for both horizontal and vertical link beam that each of them has some drawbacks and advantages which will be briefly referred to in the following.

In 2009, Shayanfar et al. [1] experimentally and analytically investigated the seismic behavior of eccentrically brace frames with composite vertical links. In 2012, Mozafarei et al [2] investigated the resisted reinforced concrete buildings with eccentric braces with vertical single link, and got to the results that by adding the V-EBF braces to concrete buildings, the seismic loads applied to columns become axial, therefore it decreases the stress ratio in the columns to less than one and centralization of seismic loads causes transfer of plastic hinges from columns to beams. Due to the same reason, more than 90 percent of columns were safe of failure and damage.

In 1983, Popov and Hjelmstad [3] suggested a finite element model using the stress formulation. The major weakness of this method is consideration of the strain hardening. In a dynamic analysis, strain hardening is a very important factor due to the cycle behavior of link beam. Another weakness of this method is that the division of the link into a large number of elements in order to minimize the errors causes this model to be inefficient in nonlinear analysis in the eccentrically braced frames.

In 2000, Ghobarah and Abou Elfath [4] investigated the seismic performance of the non-ductile reinforced concrete buildings which were improved by the steel eccentric braces with vertical links. The three lines model which had been suggested by Qobara and Radman was used in order for nonlinear analysis of the link beam. This model had been prepared to be used in DRAIN-DX2 software. Finally, the results showed the significant influence of these braces in decreasing the damage index compared to the other systems.

In this paper, in particular we deal with retrofitting the 4, 8 and 12-storey reinforced concrete buildings by eccentrically braced frames (EBF) with single and dualvertical links and assess the seismic performance of the buildings according to the third and fourth edition of Iranian seismic code (standard No 2800) [5]. Ultimately, the coefficients of seismic performance (response modification coefficient, deflection amplification factor and the over strength factor) of the retrofitted buildings are obtained and compared through these systems.

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2. Modeling

One method that recently has received considerable attention is using of the eccentrically braced frames (EBF) with single and dual link beam for retrofitting the buildings. In this study, three intermediate reinforced concrete buildings with 4, 8 and 12-storey were modeled in SAP2000 software [6]. The selected plan is a square with the dimensions of 15m*15m (figure 1). The height of all floors is 3.2 meters. The length of vertical link beam in the eccentric braces was considered equal to 50 centimeters, the dead loads of the floors are 600 kg/m^2 and the live loads of the floors are 200 kg/m^2. Also the dead load of the roof was considered equal to 500 kg/m^2 and its live load is equal to 150 kg/m^2. Sections of beams and columns in the models were designed for the concrete compressive strength equal to 250 kg/cm^2 and for the yielding strength of bending bars equal to 4000 kg/cm^2 according to the ACI 318-99 [7]. The ST37 typical mild steel was used for the link beams. The desired buildings have been designed according to the Iranian code of practice for seismic design (standard No 2800, the 2ndedition). The dimensions of beams and columns were designed so that the stress ratio becomes larger than one for considering the created uncertainty in concrete resistance due to the passage of time. In the following, in order to control the designed buildings according to the newer codes, the seismic coefficients were calculated according to the 3rdedition of Standard No 2800, and they were applied to the buildings. The results of analyses indicated that the columns in the new situation don't meet the seismic requirements of the 3rdedition of Standard No 2800 and the stress ratios in most of the columns were larger than one and they require retrofitting. in order to retrofit these buildings, we added the eccentric braces with single and dual vertical link to the buildings according to the figure (2) so that the typical mild steel was used for the link beams in both of the systems and they were designed in the best conditions of stress ratio for the link beams and the stress ratio in columns was reduced to the allowed limit (less than one) after design. After that, the buildings were reloaded and redesigned according to the 4th edition of Standard No 2800. In continue, the structures were analyzed through the nonlinear static analysis and their seismic performance was evaluated. The results obtained from the edition change of Standard No 2800 were investigated.

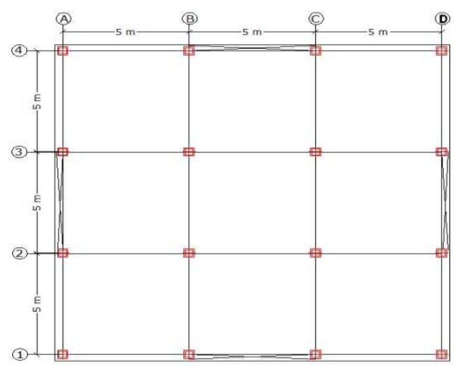
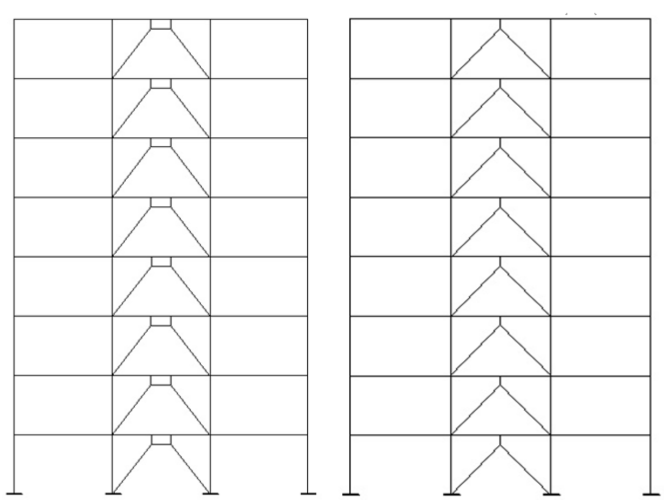


Fig. 1.the plan of the retrofitted and investigated buildings



(b) **(a)**

Fig. 2.(a) The 8-storey building with single vertical link beam, (b) the 8-storey building with dual vertical link beam

3. Investigation of the differences of the 2nd, 3rdand 4thedition of Standard No 2800

The seismic response coefficient is determined in accordance with Eq.1:

$$C = \frac{ABI}{R} \tag{1}$$

Where A is the basic design acceleration, B is the building response coefficient, I is the importance factor and R is the response modification factor.

Compared to the 2ndedition, in the 3rdedition of Standard No2800, the calculation of all of the four parameters have been changed in some cases and moreover, the 4thedition of the Standard No 2800 has been also changed in some of the above coefficients compared to the 3rdedition. We will explain it in details in order to clear the exact origin of the created changes in the structures due to the change of edition of Standard No 2800.

3-1. Basic design acceleration (A)

The basic design acceleration is determined based on the region of constructing the structure and the seismic hazard level of the area. In the 3rdedition of the Standard 2800, this coefficient is constant in most areas of Iran. It has been added for a few cities. Also, this parameter has been increased for 14 cities such as Zabol, Shahre Reza, Qorveh, etc. The majority of these areas have been changed from the moderate seismic zone to the high-risk seismic zone. In the 4thedition of Standard No 2800, this coefficient is constant in most of areas, and it has been changed in about 42 cities. Urumia, Kerman, Boushehr, Baneh, Naqan, etc. can be referred as their most populous ones. The majority of these areas have been changed from the moderate seismic area to the high-risk seismic one or from the high-risk seismic zone to the very high-risk seismic one. The basic design acceleration has not been changed between the 2ndand 3rdedition of Standard No 2800 based on the construction area of the existing structures in this study which located in Qaemshahr city from Mazandaran province and in both of the editions it has a relatively high risk of earthquake.

3-2. Building response coefficient

This parameter indicates the way that the building responds to the ground motion and all of its equations have been changed in various editions of Standard No 2800. The building response coefficient in the second edition was determined based on the type of frame system, height of the structure, the effect of infilled frames in structures and the site soil properties .This parameter is calculated through the Eq.2 in 2nd edition of Standard No 2800.In the 3rdedition all of the above items are considered, moreover, the site soil type is also considered in the calculations with a higher influence and the response coefficient is calculated regarded to the equations 3. Also in the 4thedition all of the items existing in the third edition are considered and an addition to them, two parameters of N and S_0have been added which the coefficient N (modification coefficient of the response spectrum) is calculated regarded to the paragraph a and b that have been indicated in the following. Also, the formula $B=B_1N$ is used in order to obtain the response coefficient in the 4thedition of Standard No 2800. Where the B_1 coefficient is obtained from the equations 6.

$$B = 2.5\left(\frac{T_0}{T}\right)^{\frac{2}{3}} \qquad \le 2.5 \tag{2}$$

$$\begin{aligned} B &= 1 + S\left(\frac{T}{T_0}\right) && 0 \le T \le T_0 \\ B &= S+1 && T_0 \le T \le T_s \\ B &= (S+1)\left(\frac{T_s}{T}\right)^{\frac{2}{3}} && T_s \le T \end{aligned} \tag{3}$$

a- for areas with high or very high seismic risk

$$\begin{aligned} N &= 1 && T < T_s \\ N &= \frac{0.7}{4 - T_s}(T\text{-}T_s)+1 && T_s < T < 4\sec \\ N &= 1.7 && T > 4\sec \end{aligned} \tag{4}$$

b- for areas with moderate or low seismic risk

$$\begin{aligned} N &= 1 && T < T_s \\ N &= \frac{0.4}{4 - T_s}(T\text{-}T_s)+1 && T_s < T < 4\sec \\ N &= 1.4 && T > 4\sec \end{aligned} \tag{5}$$

The equations (4) and (5)have been indicated for the soil type II and have been shown in figure 3.

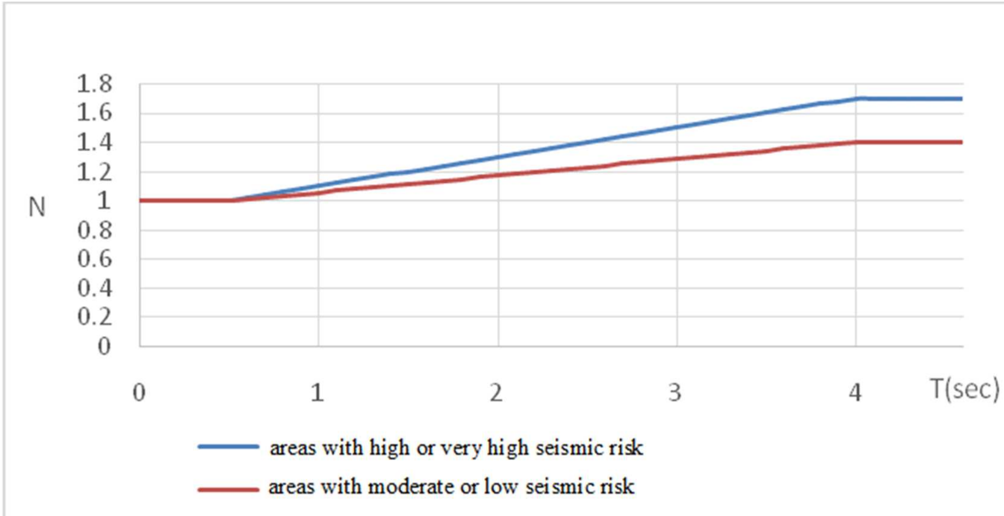


Fig. 3. The diagram of parameter N for areas with high or very high seismic risk and for areas with moderate or low seismic risk of the Standard No 280, 4th edition

$$
\begin{aligned}
&B_1 = S_0 + (S - S_0 + 1)\left(\frac{T}{T_0}\right) && 0 < T < T_0 \\
&B_1 = S+1 && T_0 < T < T_s \qquad (6)\\
&B_1 = (S+1)\left(\frac{T_s}{T}\right) && T > T_s
\end{aligned}
$$

Where T is the fundamental period of structures and the coefficients T_0, T_s, S and $S_0$are determined from Table (2-2) of the Standard No 2800, 4th edition.

In this research, the buildings have been assumed to be located in a high-risk seismic zone. Figure 4 shows the building response coefficient in accordance with 2nd, 3rd and 4th editions of Standard No 2800 for areas with high and very high seismic risk and soil type II that it refers to the very dense soil.

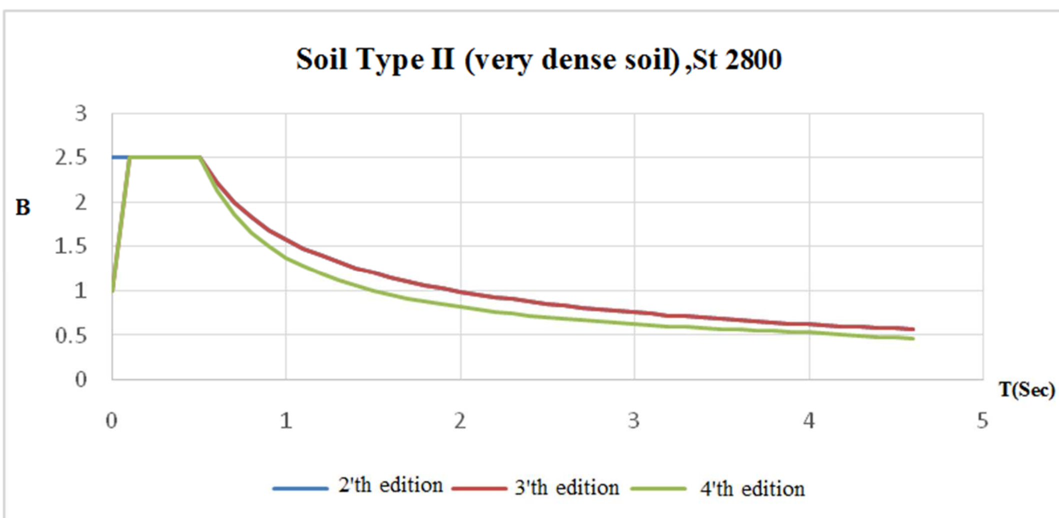


Fig. 4. The diagram of building response coefficients for areas with high and very high seismic risk based on the 2nd, 3rdand 4theditions of the Standard No 2800

3-3. Importance Factor (I)

Compared to the 2ndedition, in the 3rdedition of the Standard No 2800 just the importance factor of one risk category of the buildings has changed and it is as following: the buildings which their usability is especially important after the earthquake and delay in using them indirectly causes increase in damages and casualties like: hospitals and clinics, fire stations, water supply centers and facilities, power plants and powerhouses, Airport Towers, Telecommunication centers, Radio and TV centers, Security facilities, help centers and in general, all the buildings which their use is influential in rescue and relief. Buildings and facilities which their damage causes massive spread of toxic substances and hazardous chemicals in short time and long time in the environment are considered from these category of the buildings. In the 4thedition of the Standard No 2800 the importance coefficient has not changed compared to the 3rdedition.

3-4. Behavior factor of the building (R-factor)

The behavior coefficient of the building includes the impacts of factors such as ductility, degree of indeterminacy and the over strength of the structure. This coefficient is determined based on the seismic force-resisting system. Compared to the 2nd edition, several systems have been added to the list in the 3rdand 4theditions of Standard No 2800. Also, the values of R factor of systems have been changed in various editions of Iranian seismic code. Table (1) shows the R factors for the buildings of the current study.

Table (1): The values of behavior coefficient based on the 2nd, 3rd and 4theditions of Standard No 2800 for the intermediate reinforced concrete moment frames

H(m)	Versio4 (4th Edition of Standard No2800)	Version3 (3rd Edition of Standard No2800)	Version2 (2nd Edition of Standard No2800)	Seismic Force-Resisting System	Structural System
50	5	7	8	Intermediate reinforced concrete moment frame	Moment-resisiting frame system

In the figure (5), the values of seismic response coefficient has been indicated based on the 2nd, 3rdand 4theditions of Standard No 2800 for the 4, 8 and 12-storey buildings designed in the current study.

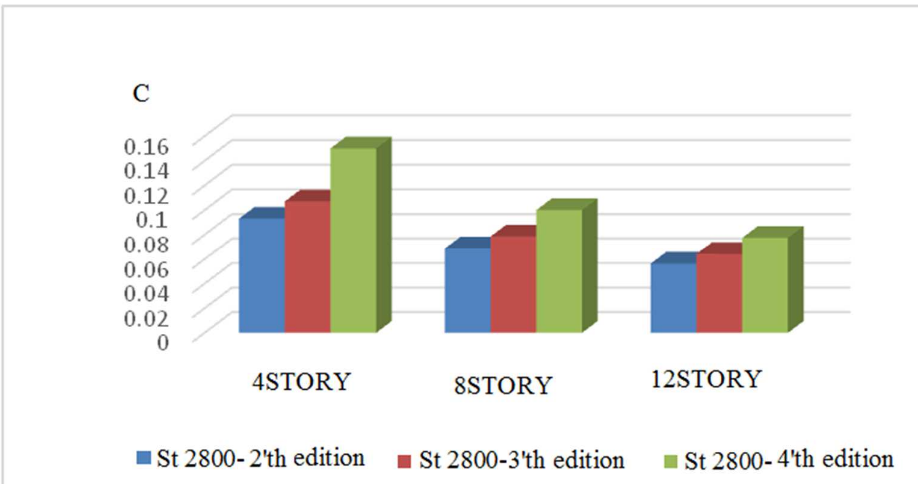


Fig. 5. The values of seismic response coefficient

4. Nonlinear Static analysis

In method of nonlinear static analysis, the performance of structure is evaluated in the situation of maximum response of the structure which faced design earthquake. In order to reach this situation, we firstly determine the relation between base shear force and lateral displacement of the control node which is located at the center of mass at the roof. This relationship is appeared in the form of a curve so-called the capacity curve or pushover curve and also this type of analysis is called pushover analysis. After obtaining the capacity curve, a point is determined on the capacity curve which is compatible with the demand displacement of the design earthquake. The mentioned point is called the performance point (PP) and its corresponding roof displacement is called demand displacement or target displacement.

In the nonlinear static method, the lateral load is monotonically increased until a target displacement is exceeded. When the lateral load is increased, the deformations and internal forces are continually considered. This method is similar to the linear static analysis with one difference:

- The nonlinear behavior of each of the members and components of the structure are entered in the analysis.
- The earthquake effect is estimated according to the deformation instead of applying certain load.

In nonlinear static analysis, the nonlinear behavior model is determined for each of the components of the structure in multi-linear form or simplified two-linear form. During the analysis, when the lateral load is gradually increased, deformations and the internal forces of all of components are calculated and compared with their capacity. This method is more complex than the linear static analysis, but its obtained results show the real behavior of the structure better and present more useful information for designing. Unlike the linear analysis methods, in this method, due to considering the nonlinear behavior of the materials, the obtained internal forces are almost equal to the expected values under the design earthquake.

5. Analysis and Investigation of Results

As mentioned, in this study, the reinforced concrete moment frame buildings designed by Standard No 2800 (second edition) have been re-evaluated based on the 3rdedition of this code and due to the weakness of some of members, these buildings were retrofitted by EBF with single and dual vertical links; moreover the coefficients of seismic load have been calculated based on the 4thedition of Standard No 2800 and according to the section 3 of this study and they were applied on the buildings in order to investigate the possible weakness points in members. In table (2), the weak members and also the percentage of decrease in stress due to retrofitting are indicated.

Table 2. The values of stress ratio in columns for (a) 4-storey building, (b)8-storey building and (c) 12-storey building,

Column ID	Initial Stress Ratio before Retrofitting		Stress Ratio after Retrofitting			
			EBF with Single Link		EBF with Dual Link	
	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4
1B-4	1.122	1.443	0.705	0.863	0.518	0.587
1B-3	1.119	1.544	0.647	0.866	0.407	0.522
1B-2	1.157	1.79	0.802	1.212	0.47	0.709
1B-1	1.151	1.72	0.707	1.174	0.598	1.021

(a)

Column ID	Initial Stress Ratio before Retrofitting		Stress Ratio after Retrofitting			
			EBF with Single Link		EBF with Dual Link	
	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4
1B-8	1.022	1.237	0.674	0.776	0.589	0.661
1B-7	1.116	1.416	0.564	0.69	0.412	0.476
1B-6	1.2	1.522	0.567	0.706	0.383	0.427
1B-5	1.237	1.608	0.637	0.842	0.447	0.499
1B-4	1.225	1.575	0.848	1.187	0.55	0.775
1B-3	1.142	1.511	0.927	1.305	0.732	0.939
1B-2	1.09	1.299	0.827	1.164	0.735	0.96
1B-1	0.957	1.103	0.949	1.392	0.94	1.341

(b)

Column ID	Initial Stress Ratio before Retrofitting		Stress Ratio after Retrofitting			
			EBF with Single Link		EBF with Dual Link	
	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4	St No 2800, Version3	St No 2800, Version4
1B-12	1.09	0.794	0.824	0.824	0.857	0.834
1B-11	0.99	1.126	0.698	0.8	0.619	0.643
1B-10	1.13	1.386	0.635	0.743	0.48	0.59
1B-9	0.995	1.215	0.574	0.657	0.405	0.522
1B-8	1.24	1.532	0.683	0.835	0.428	0.62
1B-7	1.106	1.355	0.665	0.732	0.615	0.62
1B-6	1.168	1.394	0.833	0.932	0.83	0.836
1B-5	1.171	1.455	0.864	1.168	0.764	0.835
1B-4	1.06	1.262	0.953	1.285	0.949	0.922
1B-3	1.04	1.062	0.91	1.002	0.981	0.958
1B-2	0.979	0.891	0.932	1.026	0.986	0.996
1B-1	0.994	0.887	0.959	1.289	0.998	1.245

(c)

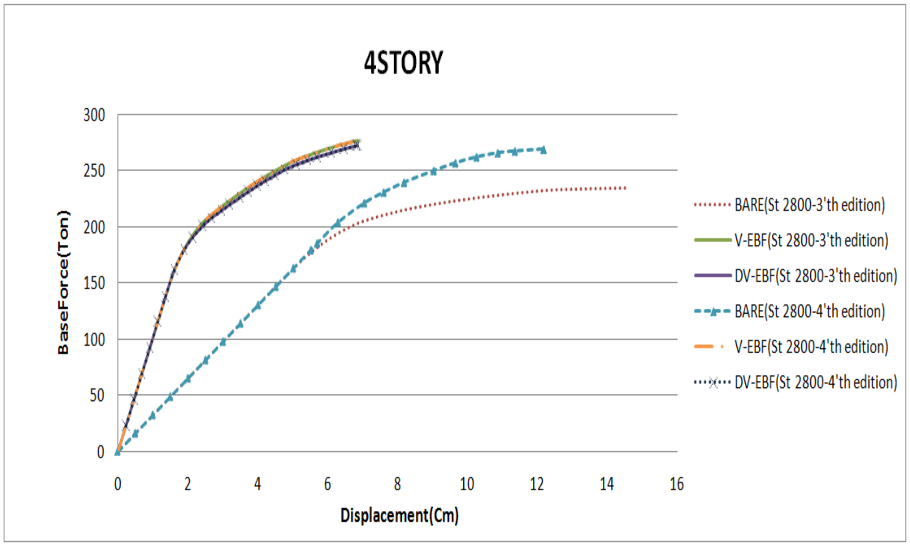


Fig. 6. Base shear – lateral displacement curves of the studied4-storey buildings

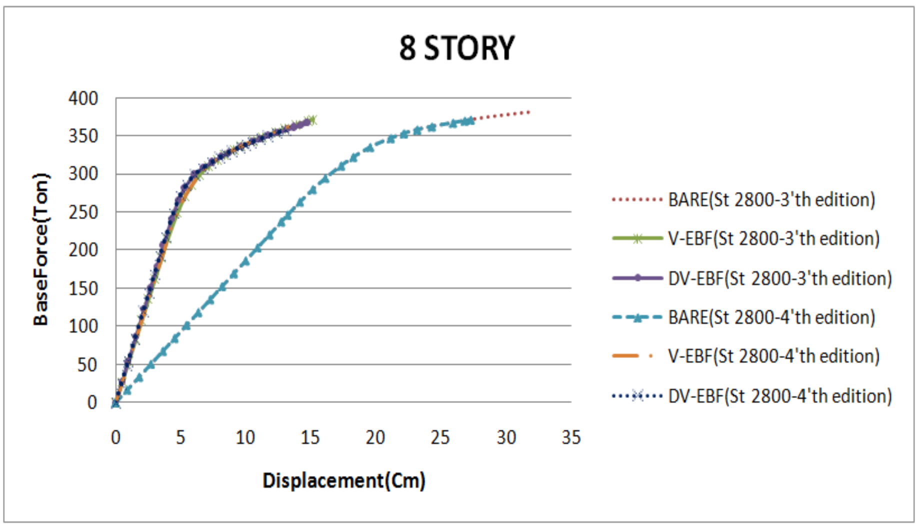


Fig. 7. Base shear – lateral displacement curves of the studied8-storey buildings

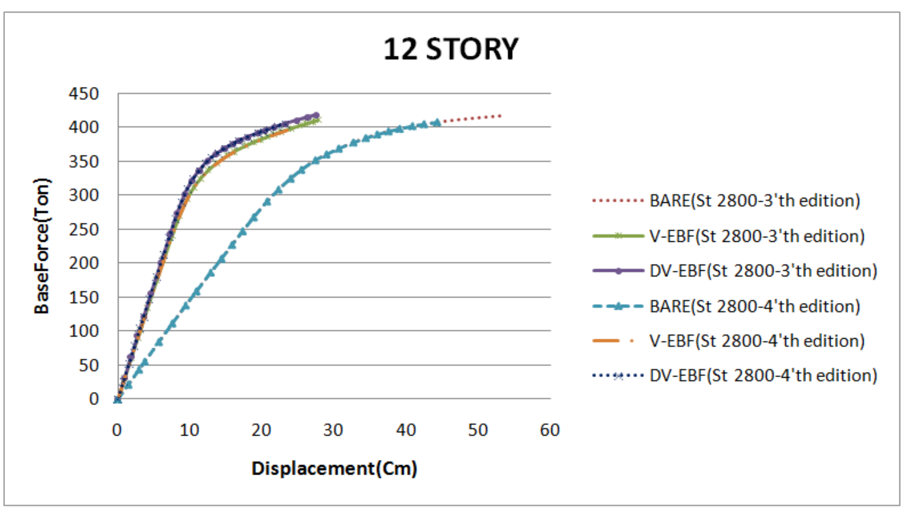


Fig. 8. Base shear – lateral displacement curves of the studied 12-storey buildings

6.Evaluation of response modification coefficient (R Factor)

The main aim in seismic design of buildings is based on the fact that the building's behavior remains in linear range against small earthquakes and tolerates the structural and nonstructural damages along with saving general stability against the forces caused by intense earthquakes. Due to the same reason, in design codes the desired seismic resistance against the earthquake is usually lower or in some cases very lower than the lateral resistance required for maintaining the structure stability within the elastic range in an intense earthquake. So, when moderate and intense earthquakes occur, the behavior of structures enters the inelastic range and their designing requires an inelastic analysis; but regarded to the high costs of this method and complication of the inelastic analysis and the ease of elastic method, the conventional analysis and designing methods are done according to the elastic analysis of the structure and the decreased force of the earthquake. Decrease in resistance of the structure from the required elastic resistance is mostly done through the resistance decrease coefficients. So far, some methods have been suggested for calculating the R factor which each of them has considered the effect of some of the influential factors. One of the equations presented for R is the relationship which includes three factors of ductility, over strength and degree of indeterminacy, as follows:

$$R = \frac{Ve}{V} = \Omega_0\ R_\mu Y \tag{7}$$

$$R_\mu = \frac{\delta_e}{\delta_y} = \frac{V_e}{V_y} \tag{8}$$

In these equations, $\Omega_0$is the over strength factor, shall be calculated in accordance with Eq. 9. R_μ is the ductility-based R factor, Y is the redundancy factor determined according to the attitude of design codes for design stress (i.e. yield stress and allowable stress). Y shall be taken as 1.0 if the LRFD method is used to design. The term V is the maximum base shear, if the system remained entirely linearly elastic for design earthquake. V_y represents the yield strength for the idealized pushover curve and V is the design base shear of the real structure. δ_e and δ_y are lateral displacements corresponding to the V_e and V_y, respectively. The deflection amplification factor (C_d) is determined in accordance with Eq. 10.

$$\Omega_0 = \frac{V_y}{V} \tag{9}$$

$$C_d = \frac{\delta}{\delta_e}\ R \tag{10}$$

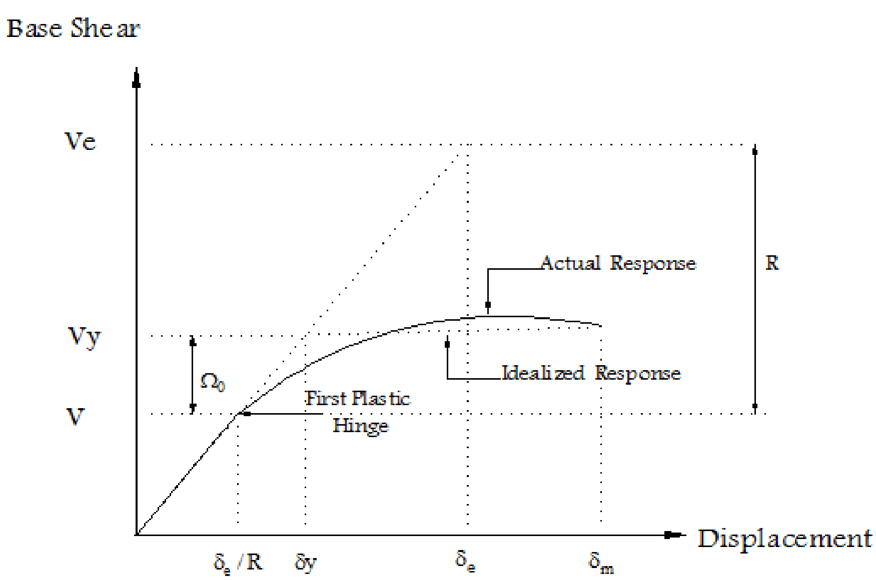


Fig. 9. Idealized nonlinear static pushover curve

Figure 9 illustrates all parameters that are used in the equations. In this study the modeled buildings have been designed accordance to the LRFD design method. In order to calculate the seismic performance factors, the capacity curve of all of the studied buildings was obtained and it was replaced with an idealized relationship in accordance with FEMA 356[8].Table (3) shows the values of seismic performance factors for the studied buildings. Also, the R factors have been shown in figure10.

Table 3. The values of seismic performance factors for the studied buildings

	Standard 2800	Frame type	Ω_0	R_μ	R	C_d
4 STORY	3'th edition	VEBF	1.62	2.41	3.91	3.53
		DVEBF	1.562	2.35	3.68	3.6
	4'th edition	VEBF	1.456	2.34	3.41	3.46
		DVEBF	1.387	2.34	3.24	3.48
8 STORY	3'th edition	VEBF	1.357	2.32	3.15	2.77
		DVEBF	1.282	2.2	2.82	2.85
	4'th edition	VEBF	1.264	2.21	2.83	2.45
		DVEBF	1.187	2.09	2.48	2.48
12 STORY	3'th edition	VEBF	1.563	2.37	3.7	2.65
		DVEBF	1.381	2.31	3.19	2.83
	4'th edition	VEBF	1.1503	2.12	2.44	2.36
		DVEBF	1.126	2.07	2.34	2.4

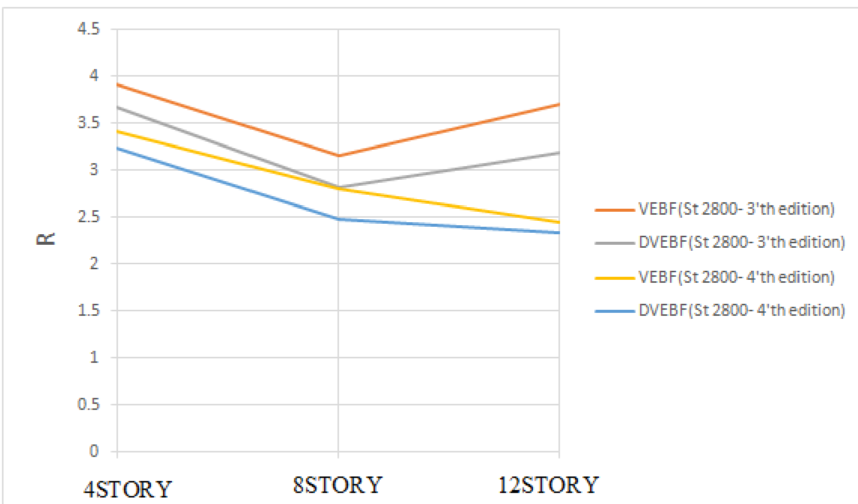


Fig. 10. The R factors for the studied buildings

7. Conclusion

In this study, the following brief results have been obtained from assessment of the seismic performance of the reinforced concrete buildings retrofitted by the eccentric braces with single and dual vertical links:

1. with adding the eccentric brace with vertical link beam to the reinforce concrete buildings, the stress ratios in the members are significantly decreased and it gets smaller than one in the weak columns; this decrease is 36% and 52% for the single and dual vertical links, respectively. Also the stress ratio in the buildings with single link which have been designed by the 4thedition of the Standard 2800 increases 32% and it increases 24% for the buildings with dual link compared to the 3rdedition. Therefore, having the shear behavior, the eccentric braces with the single and dual vertical links perform well against the earthquakes and it is advised to use them in retrofitting the reinforced concrete buildings.

2. The base shear –displacement values of the buildings with single and dual link beam designed by the 4thedition of the Standard No2800 has respectively decreased 4% and 8% compared to the 3rdedition.

3. the values of R factor for the braced buildings with the single and dual link beam which have been designed according to the 4thedition of the standard No 2800 have increased 20% and 17%, respectively compared to the 3rdedition. Also, the value of deflection amplification factor for the braced buildings with single and dual link beam which have been designed according to the 4thedition of the Standard No 2800 decrease 8% and 10%,respectivelycompared to the 3rd edition.

4. Using these systems plays an influential role in increasing the stiffness and controlling the displacement; but they extremely decrease the ductility so that in this study the average value of response modification coefficient for both types of the single and dual link has approximately been 3.0.

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