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Technical Note

Evaluating the overstrength of concentrically braced steel frame systems considering members post-buckling strength

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Abstract

Inelastic deformation of structural components is generally acceptable in seismic design. In such behavior, the strength of structures increases while plastic hinges are formed in members frequently. The strength revealed during the formation of plastic hinges is called "overstrength". Overstrength is one of the important parameters in the seismic design of structures. The present study tries to evaluate the overstrength of the concentrically steel braced frames (CBFs), considering reserved strength, because of members post-buckling. As such, a static nonlinear (pushover) analysis has been performed on the model buildings with single and double bracing bays, different stories and brace configurations (chevron V, invert V and X-bracing). It has been realized that the number of bracing bays and the height of buildings have a low effect on reserve strength due to brace post-buckling. However, these parameters have a profound effect on the overstrength factor. These results indicate that the overstrength values for CBFs, proposed in seismic design codes, need to be modified.

Keywords: Concentrically steel braced frames, Overstrength factor, Post-buckling strength, Response modification factor.

1. Introduction

Normally, the preliminary design in most of the buildings is based on equivalent static forces specified by the governing building codes. The height-wise distribution of these static forces (and therefore, stiffness and strength) seems to be based implicitly on the elastic vibration modes. However, structures do not remain elastic during severe earthquakes and they are expected to undergo large nonlinear deformations [1]. Many seismic codes permit a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength (overstrength) and the capacity to dissipate energy (ductility), which are incorporated in structural design through a response modification factor [2].

Steel concentric braced frames (CBFs) are one of the lateral load resisting systems, especially for structures constructed in high seismic regions. The worklines of CBFs essentially intersect in some points [3]. In CBFs, steel braces improve the lateral strength and stiffness of the structural system and participate in seismic energy dissipation by yielding in tension and buckling inelastically in compression [4]. Consequently, the cyclic axial response of the bracing members, which are expected to undergo tension deformations beyond yield and compression deformations into the post-buckling range, represent the most crucial aspect of the seismic response of a braced frame system [5].

Several researchers have investigated the factors that may have contributed to the observed overstrength. Osteraas and Kraeinkler [6] conducted a detailed study of overstrength concentric braced frames designed following the allowable stress design provisions with seismic loads per UBC seismic zone 4 and soil type S2. Finding overstrength factors of CBFs, Rahgozar and Humar [7] showed that the main parameter controlling these factors in braced frame structures is the slenderness ratio of bracing members. Performing pushover analyses, Kim and Choi [2] evaluated the overstrength, ductility and response modification factors of the chevron type concentric braced frames with diverse stories and span lengths. The studies carried out by Disarno and Elnashai [8] clarify that in CBFs with stainless steel braces and columns, the increase in overstrength is about 40% with respect to the configuration in mild steel. According to Davaran and Hoveidae [9] the type of mid-connection detail of X concentric braced frame could improve the response modification factor and the overstrength factor to about 28% and 5% respectively, more than the one with common mid-connection detail.

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Previous studies did not consider the effect of bracing bays and reserve strength after the buckling of the braces. Considering brace post-buckling strength, the present study has focused on the evaluation of the overstrength factor of CBFs, loaded by Iranian Earthquake Resistance Design Code (Standard No. 2800) [10] and designed according to part 10 of the Iranian National Building Code, steel structure design [11]. Here, the nonlinear static pushover analysis was conducted by considering cyclic behavior of bracing members in life safety structural performance level as suggested by FEMA-356 [3].

2. Cyclic Behavior of the Brace

Bracing systems are well-known solutions for providing sufficient lateral strength and stiffness in steel frameworks [12]. In normal buildings, bracing members are expected to buckle in compression and yield in tension once subjected to a reverse cyclic loading [13]. The severity of the cyclic loading depends on the slenderness ratio of the brace [14]. Several researchers have tried to investigate the cyclic behavior of bracing members hence; the experimental and analytical studies demonstrate the distinctive hysteretic response of axially loaded members. This is characterized by the gradual reduction in compressive resistance as well as deterioration of stiffness in tension, with loading cycles of increasing deformation amplitude. These investigations also identified the improved stiffness and energy dissipation capabilities provided by relatively rigid end connections. In comparison with nominally pinned conditions, rigid connections cause plastic hinges to form at the member ends, in addition to that at mid-length, leading to improved inelastic performance [5].

The CBF response to earthquake loading depends mainly on the asymmetric axial resistance of the bracing members [15], which has a complex cyclic inelastic behavior due to the influence of the following physical phenomena: yielding in tension, buckling in compression, post-buckling deterioration of compressive load capacity, deterioration of axial stiffness with cycling, low-cycle fatigue fractures at plastic hinge regions, and the Bauschinger effect. These factors complicate the formulation of efficient analytical models that are capable of accurately simulating the inelastic behavior of steel braces. Nevertheless, practical and reliable analytical tools are essential for the transition from current prescriptive seismic codes to performance based design specifications, which require accurate predictions of inelastic limit states up to structural collapse [16].

According to Ikeda and Mahin [17], frame element models which have been used to simulate the inelastic behavior of steel braces can alternatively be classified as finite element, phenomenological and physical theory models. Despite the difficulty of determining input data, phenomenological models have been widely used for nonlinear seismic analyses.

Figure 1 shows the ideal load-deflection of steel bracing members, suggested by FEMA-356 [3]. The horizontal axial represents the deflection (axial displacement) and the vertical axial shows the members internal forces (tension or compression). The yield or buckling occurs in point B. In the compression case, BC is related to elastic buckling and CE represents inelastic buckling. The strength that happened from point B to point - in which the first member reaches to the life

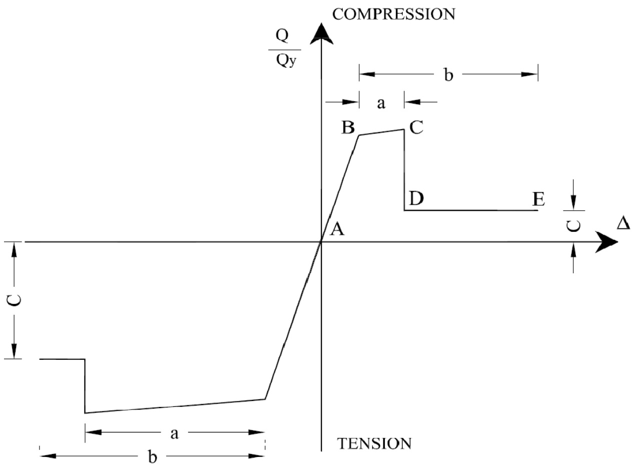


Fig. 1. Generalized force-deformation relation for steel brace elements (FEMA-356) [3].

safety case - is post-buckling strength.

Structural performance level, life safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injuries as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic considerations this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy [3].

3. Overstrength Factor

Observations during earthquakes have shown that building structures could take the forces considerably larger than those that they were designed for. This is explained by the presence of such structures with significant reserve strength not accounted for in design [7]. Overstrength helps structures stand safely not only against severe tremors but reduces the elastic strength demand as well. This objective is performed using force reduction factor by several codes of practice [18]. Figure 2 represents the base-shear versus roof displacement relation of a structure, which can be developed by a nonlinear static analysis. The design overstrength factor (R_{sd}) and post-buckling overstrength factor (R_{sp}) are defined as follows:

$$R_{Sd} = \frac{V_y}{V_d} \tag{1}$$

$$R_{SP} = \frac{V_u}{V_y} \tag{2}$$

According to Figure 2, V_d is the design base shear of the building, V_y is the base shear corresponding to the first yield observe in the frame and V_u is the base shear in relevance to the first life safety performance in structural members.

In this paper the overstrength factor of the frames were computed using Equations 1 and 2 based on the analysis results. The overstrength factors shown in Equations 1 and 2 are based on the use of nominal material properties applied. The actual overstrength factor (R_s), should consider the contribution from some other effects [19]:

$$R_S = R_{Sd}.R_{SP}.R_1.R_2.... \tag{3}$$

In Equation 3, R_1 is used to account for the difference between actual static yield strength and nominal static yield strength. For structural steel, a statistical study shows that the value of R_1 may be taken as 1.05 [20]. Parameter R_2 may be used to consider the increase in yield stress as a result of strain rate effect during an earthquake. For the strain rate effect, a value of 1.1 or a 10% increase could be used [21]. The current study uses steel type St-37 for all structural members. It considers parameters R_1 and R_2 equal to 1.05 and 1.1, taking into account R_{Sm}=1.155 as the material overstrength factor. Other parameters such as nonstructural component contributions and variation of lateral force profile could be included once reliable data is available.

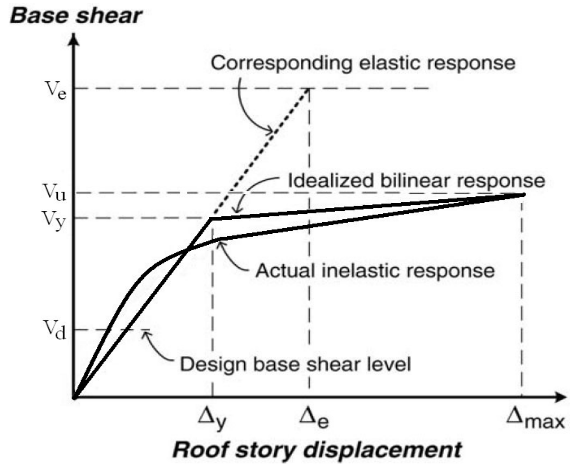


Fig. 2. General structural response.

For CBFs, various codes present numerical values of the overstrength factor. For instance, the overstrength factor for CBFs is equal to 2 in IBC [22], AISC [23], FEMA-450 [24] and Iranian National Building Code (steel structure design) [11] and equal to 2.2 in UBC [25].

4. Structural Models

4.1. Design of Model Structures

To evaluate the overstrength factor of CBFs, 30 building models with 3, 5, 7, 10 and 12 stories with a bay length of 5m were designed. For this structural model, three different bracing types (X, chevron V and chevron-Inverted V) were considered. The height of every model structure was fixed to 3.2m. Figure 3 shows the plan of the model structures and the type of braces located in single and double bays.

The gravity load of 5.5 KN/m^2 and 2 KN/m^2, was used for dead and live load, respectively. For member design subjected to earthquake, equivalent lateral static forces were applied on all the story levels. These forces were calculated following the provisions stated in the Iranian Earthquake Code (Standard No. 2800) [10]. The design base shear was computed as follows,

$$V = CW \rightarrow C = \frac{ABI}{R} \tag{4}$$

where, V is the base shear of the structure, C is the base shear ratio and W is the equivalent weight of the structure. A × B is the design spectral acceleration, expressed as the fundamental period of structure T and soil type, I is the importance factor and R is the response modification factor. The importance factor of I = 1, preliminary response modification factors of R = 6 and seismic zone factor of A = 0.35 were considered for frame design. All beam - column connections were assumed to be pinned at both ends as frames were not designed to be

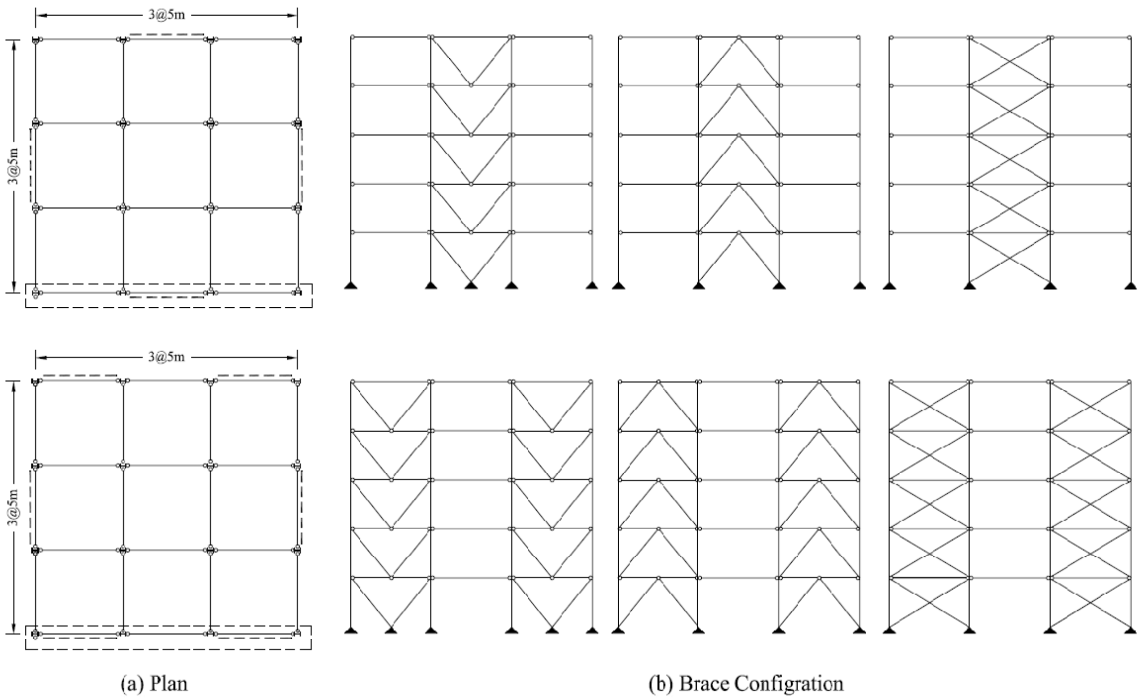


Fig. 3. Configuration of model structures.

moment resistant. The braces were also designed to sustain 100 percent of the lateral load.

The models were designed keeping in view the part 10 of Iranian national code [11]. To ensure that vertical bracing columns have enough strength to resist the forces transferred by bracing elements. Iranian Standard No.2800 [10] has instructions to design vertical bracing columns for the following load combinations:

(a) Axial compression according to:

$$P_{DL} + 0.8P_{LL} + 2.8P_E \le P_{SC} = 1.7F_a A \quad (5)$$

(b) Axial tension according to:

$$0.8P_{DL} + 2.8P_E \le P_{ST} = F_y A \quad (6)$$

The maximum lateral story displacement ($\overline{\Delta}_M$) limit was selected based on the Iranian Standard Code No. 2800 [10] as follows:

for frames with a fundamental period less than 0.7 s:

$$\overline{\Delta}_M < 0.025H \quad (7)$$

for frames with a fundamental period more than 0.7 s:

$$\overline{\Delta}_M < 0.02H \quad (8)$$

in which 'H' is the story height.

4.2. Pushover Analysis

Nonlinear static (pushover) analysis is a simplified analysis procedure that can be useful for estimating seismic demands and providing valuable information about the locations of structural weaknesses and failure mechanisms in the inelastic range [26]. To evaluate the overstrength factor, the inelastic pushover analysis is generally used. Pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral forces. The selection of an appropriate lateral load distribution is an important step within the pushover analysis [27]. Pushover analyses were carried out to evaluate the buckling and post-buckling limit state by progressively increasing the lateral story forces proportional to the fundamental mode shape. The post-yield stiffness of the beams, columns and braces was assumed to be 2% of the initial stiffness. The phenomenological model presented in FEMA-356 [3], was used for modeling nonlinear behavior of braces (Fig.1). The post-buckling residual compression force is set to be 20% of the buckling load as given in Tables 5-7 of FEMA-356 [3].

5. Results

Figures 4 through 6 show nonlinear static pushover analysis results in terms of base shear-roof displacement for different bracing types (inverted V, chevron V and X-type). Figures 7 and 8 show the variation of overstrength for different types of bracing configuration. In Tables 1 through 3 the design overstrength factor, post-buckling overstrength factor and overstrength factor of braced frames are shown. It can be seen that the overstrength factor decreases as the height of the building increases. On the other hand, the overstrength factors increase as the number of bracing bays increase. This is because of the limitation on slenderness and allowable axial stress reduction for braces in design codes seismic provisions.

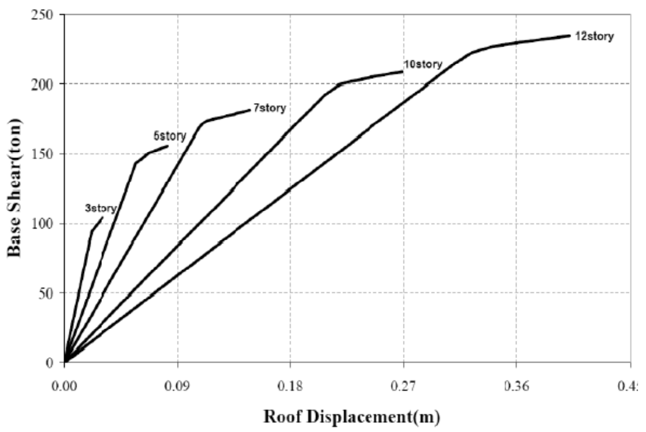
Post-buckling overstrength factors have a constant value for each type of brace frame. The number of bracing bays and the height of the building have no affect on this factor, approximately. It is concluded that X-shape bracing's post-buckling strength is more than chevron's. In X-shape bracings, the tension member stands against lateral load after the buckling of compression member while in chevron shapes it does not.

6. Conclusion

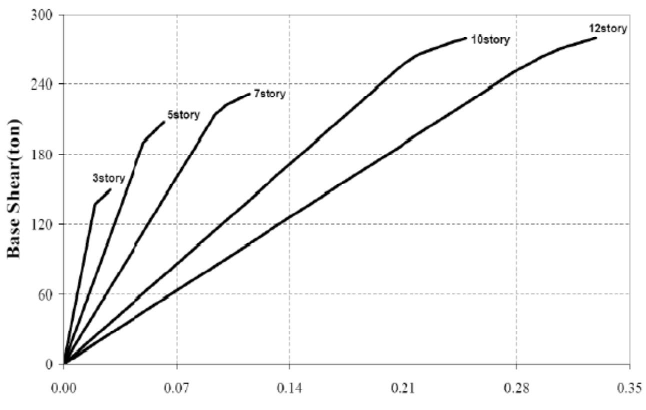
Considering reserved strength because of brace Post-Buckling, this paper assesses the overstrength factor of the 30 concentrically steel braced frame systems in life safety structural performance level. For this purpose, the static nonlinear (pushover) analysis has been performed on buildings with single and double bracing bays and various stories and different brace configurations. The model structures were designed for relatively large seismic loads and the beam-column connections were assumed to be pinned so that the seismic load was resisted mainly by the braces.

The results of this study can be summarized as follows:

1. The overstrength factors increase with the decrease of

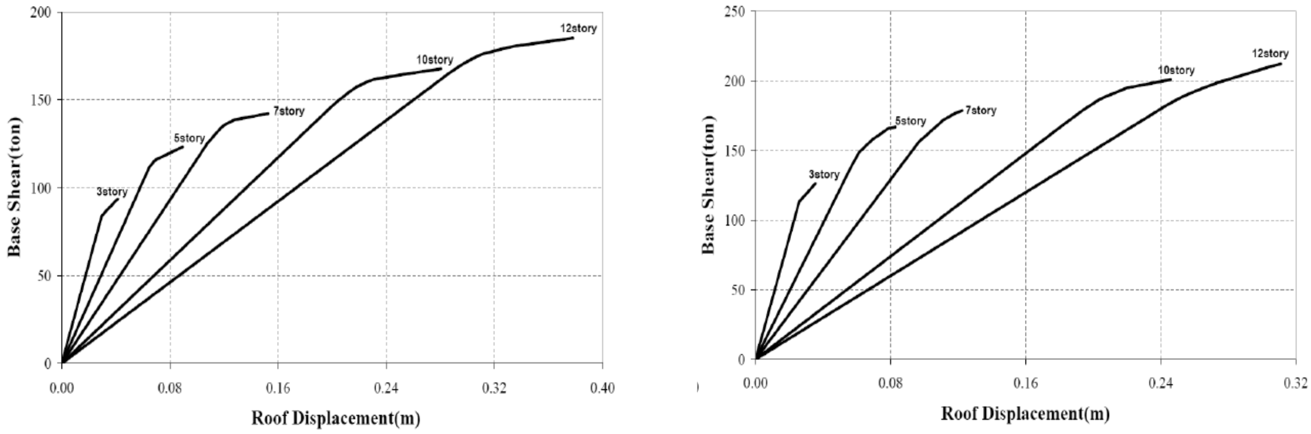


(a) Single Bay Brace frames



(b) Double Bays Brace frames

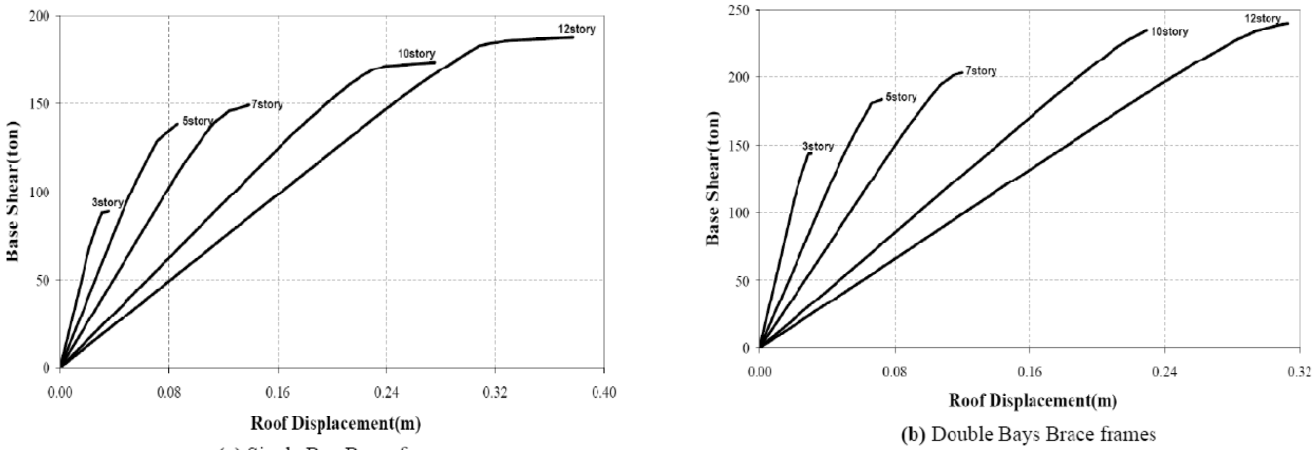
Fig. 4. Roof displacement-base shear curve for conventional invert V-brace.



(a) Single Bay Brace frames

(b) Double Bays Brace frames

Fig. 5. Roof displacement-base shear curve for conventional chevron V-brace.



(a) Single Bay Brace frames

(b) Double Bays Brace frames

Fig. 6. Roof displacement-base shear curve for conventional X-brace

Table 1. Overstrength factor of CBFs with chevron invert V-brace

No. story	Single bay brace frame				Double bays brace frame			
	R_{sd}	R_{sp}	R_{sm}	R_s	R_{sd}	R_{sp}	R_{sm}	R_s
3	3.34	1.10	1.155	4.24	4.81	1.09	1.155	6.09
5	3.01	1.08	1.155	3.75	4.00	1.10	1.155	5.08
7	2.98	1.08	1.155	3.72	3.77	1.09	1.155	4.74
10	2.83	1.07	1.155	3.51	3.72	1.09	1.155	4.70
12	2.79	1.09	1.155	3.50	3.35	1.11	1.155	4.29

Table 2. Overstrength factor of CBFs with chevron V-brace

No. story	Single bay brace frame				Double bays brace frame			
	R_{sd}	R_{sp}	R_{sm}	R_s	R_{sd}	R_{sp}	R_{sm}	R_s
3	2.95	1.12	1.155	3.82	4.04	1.12	1.155	5.22
5	2.35	1.10	1.155	2.98	3.16	1.11	1.155	4.05
7	2.22	1.12	1.155	2.88	2.78	1.14	1.155	3.67
10	2.20	1.10	1.155	2.80	2.66	1.11	1.155	3.41
12	2.16	1.11	1.155	2.78	2.50	1.13	1.155	3.27

Table 3. Overstrength factor of CBFs with that have X-brace

No. story	Single bay brace frame				Double bays brace frame			
	R_{sd}	R_{sp}	R_{sm}	R_s	R_{sd}	R_{sp}	R_{sm}	R_s
3	2.54	1.33	1.155	3.61	3.83	1.32	1.155	5.86
5	2.27	1.29	1.155	3.38	3.05	1.26	1.155	4.46
7	2.07	1.27	1.155	3.05	2.75	1.31	1.155	4.16
10	2.07	1.22	1.155	2.92	2.54	1.35	1.155	3.96
12	2.06	1.20	1.155	2.86	2.49	1.28	1.155	3.67

structure height and increase in the number of bracing bays.

2. The number of bracing bays and the height of the building have a low affect on post-buckling overstrength factors.

3. Code's seismic provisions for brace member design have a profound effect on the CBFs overstrength factors.

4. The obtained post-buckling overstrength factors for CBFs in type V, inverted V and X with single and two bracing bays are 1.11, 1.08 and 1.28, respectively.

5. The overstrength factor for concentrically steel braced frames in type V, inverted V and X with single bracing bay are evaluated as 2.90, 3.75 and 3.10, respectively.

6. The overstrength factor for concentrically steel braced

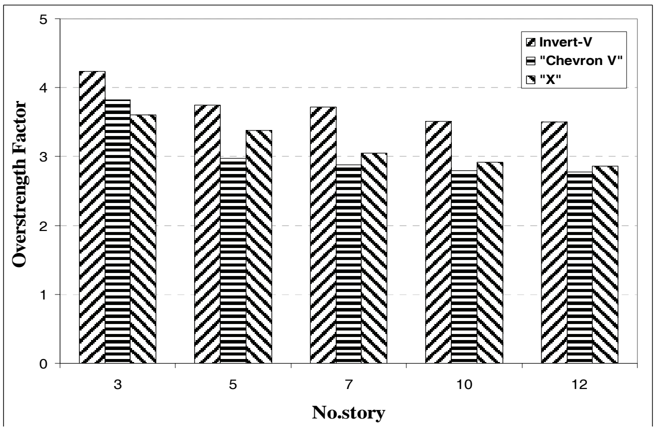


Fig. 7. Overstrength factors of single bay CBFs

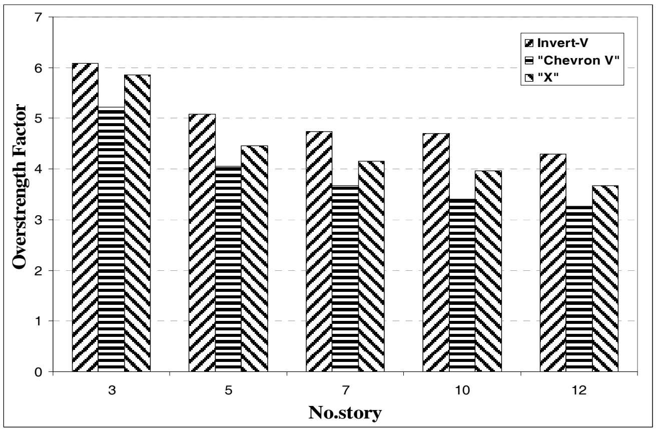


Fig. 8. Overstrength factors of double bays CBFs

frames in type V, inverted V and X with two bracing bay are evaluated as 3.80, 4.80 and 4.20, respectively.

7. Codes present constant value of overstrength factor for CBFs, however, the overstrength factors evaluated in this paper have different values for brace configuration types, the number of bracing bays and building height. Therefore, the results indicate that the overstrength factors proposed in seismic codes need to be modified for concentrically steel braced frame systems.

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