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Evaluating the displacement amplification factors of concentrically braced steel frames

Mussa Mahmoudi* and Mahdi Zaree

Abstract

According to seismic design codes, nonlinear performance of structures is considered during strong earthquakes. Seismic design provisions estimate the maximum roof and story drifts occurring during major earthquakes by amplifying the drifts computed from elastic analysis at the prescribed seismic force level with a displacement amplification factor. The present study tries to evaluate the displacement amplification factors of conventional concentric braced frames (CBFs) and buckling restrained braced frames (BRBFs). As such, static nonlinear (pushover) analysis and nonlinear dynamic time history analysis have been performed on the model buildings with single and double bracing bays, and different stories and brace configurations (chevron V, invert V, and X bracing). It is observed that the displacement amplification factors for BRBFs are higher than that of CBFs. Also, the number of bracing bays and height of buildings have a profound effect on the displacement amplification factors. The evaluated ratios between displacement amplification factors and response modification factors are from 1 to 1.12 for CBFs and from 1 to 1.4 for BRBFs.

Keywords: Buckling restrained braced frame, Concentrically steel braced frame, Displacement amplification factor, Ductility factor, Overstrength factor

Introduction

It is well recognized that most disasters due to moderate or severe earthquake ground motions are caused by the failure of civil engineering facilities, many of which were presumed to have been designed and constructed to provide protection against natural hazards. Much of the damages and collapse of structures during severe earthquakes primarily occurred due to excessive displacement in stories. In force-based seismic design, the force demand is generally determined on the basis of the structural linear response. Studies show that structures designed by modern seismic code procedures are likely to undergo large cyclic deformations in the inelastic range when subjected to a severe tremor. In current seismic codes, design base shears are calculated by reducing the elastic to the inelastic strength demands using the response modification factor (R) (Lee et al. 2004). Similarly, the displacement demand of a structure is estimated by multiplying a linear displacement response by the displacement amplification factor (C_d). The displacement amplification factor is the structural response parameter most widely employed for evaluating the inelastic performance of structures. C_d is also the parameter explicitly or implicitly used in most common design procedures.

There are several systems that can be used effectively to provide resistance to seismic lateral forces. Conventional concentric braced frame (CBF) and buckling restrained braced frame (BRBF) are the most efficient and common structural systems in steel construction to resist lateral forces, especially for structures in highly seismic regions. The use of concentric braces in framed structures offers an attractive system for seismic resistance, primarily due to their efficiency in providing lateral stiffness, hence limiting inter-story as well as overall lateral deformations (Goggins et al. 2006). In other words, because of complete truss action, steel braces improve the lateral strength and stiffness of the structural system and participate in seismic energy dissipation by deforming inelastically during an earthquake (Davaran and Hoveidae 2009).

Lateral displacements on structural buildings have been of great concern for engineers. Several researchers

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have tried to investigate the displacement amplification factors of structural systems. Uang and Maarouf (1994) have discussed the effects of building and predominant earthquake ratios, types of yield mechanisms, and structural overstrength on the displacement amplification factor. Kim and Choi (2004) showed that the structural displacements decrease with the increase in BRB stiffness. They also found that the story-wise distribution of BRB, in proportion to the story drifts and story shears, results in better structural performance. Mahmoudi (2004) evaluated the displacement amplification factor and proposed a value to estimate the maximum lateral structural displacement, without using the nonlinear analysis. He also calculated the ratio of displacement amplification factor and response modification factor. In their study, Kiggins and Uang (2006) found that the use of BRB with steel moment frames will reduce residual story drifts and permanent deformations which can conversely lead to obtaining larger value of response modification factor.

Studies show various behavior factors for concentric braced frames (Mahmoudi and Zaree 2010). Furthermore, based on the previous design provisions, codes give constant value of displacement amplification factor for conventional CBFs and BRBFs, which do not consider the structure characteristic (number of stories and bracing bays). To overcome this inadequacy, the present paper has also focused on the evaluation of displacement amplification factor and its relation to the response modification factor of both CBFs and BRBFs. Here, the nonlinear static pushover analysis and nonlinear dynamic time history analysis were conducted by considering the behavior of members in life safety structural performance level as suggested by the Federal Emergency Management Agency (FEMA)-356 (2000).

Concentric braced frames

Bracings as lateral load-resistant system are one of the most commonly used methods to resist lateral loads such as earthquake. The braced frame response to earthquake loading depends mainly on the asymmetric axial resistance of the bracing members (Broderick et al. 2008). Conventional steel bracings dissipate considerable energy yielded under tension, but buckle without much energy dissipation in the compression range of cyclic loading (Kumar et al. 2007). If buckling of steel brace is restrained and the same strength is ensured both in tension and compression, the energy absorption of the brace will be markedly increased, and the hysteretic property will be simplified. Considering limited ductility and energy dissipation capacity of conventional CBF systems, efforts were made to develop new systems with stable hysteretic behavior, significant ductility, and large energy dissipation capacity. One such system with an improved seismic behavior is the BRBF. A typical BRB consists of a yielding steel core encased in a mortar-filled steel hollow section to restrain buckling, non-yielding and buckling restrained transition segments, and non-yielding and unrestrained end zones (Figure 1) (Sahoo and Chao 2010). Axial forces in BRBs are primarily resisted by steel cores which are laterally braced continuously by the surrounding mortar and steel encasement to avoid their buckling under compressive loads. This allows the steel core to yield in tension and compression, thereby significantly increasing the energy dissipation capacities of BRBs as compared to conventional steel braces (Figure 2) (Sahoo and Chao 2010).

Methods

Structural model and design

To evaluate the displacement amplification factor and its relation to response modification factor, 30 conventional CBFs and 20 BRBFs with 3, 5, 7, 10, and 12 stories as well as a bay length of 5 m were designed. Three different bracing types (i.e., X, chevron V, and chevron inverted V) for conventional CBFs and two bracing types (chevron V and chevron inverted V) for BRBFs were considered. The height of every model structure was fixed at 3.2 m. Figure 3 shows the plan of the model structures with the braces located in single and double bays. The model buildings were designed to take into account the Iranian Earthquake Resistance Design Code (Standard No. 2800) (BHRC 2005) and Iranian National Building Code, part 10, steel structures design (MHUD 2009). The buckling restrained brace members were designed according to the seismic provision of AISC (2005). The beam-column connections were assumed to be pinned so that the seismic load was resisted mainly by braces.

For brace designs, the double channel sections and the plate sections were used for CBFs and BRBFs, respectively. The effective length factor (K) considered for brace design is 0.5 for X braces, 1 for V and inverted V conventional concentric braces, and 0 for buckling restrained braces. Meanwhile, the IPB sections were used for the column in 3- and 5-story buildings, and the box sections were preferred in 7-, 10-, and 12-story buildings. Table 1 presents details about the structural members selected for the seven-story model frame with inverted V braces.

Displacement amplification factor

Both structural and nonstructural damages observed during earthquake ground motions are primarily produced by lateral displacements. Thus, the estimation of lateral displacement demands is of significant importance in performance-based design methods (Hajirasouliha and Doostan 2010). According to modern seismic design provisions, building structures undergo inelastic deformation during severe earthquakes. Therefore, these provisions

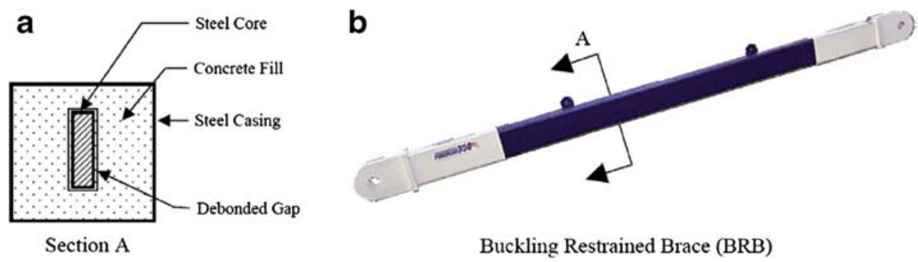

Figure 1 Typical BRB element. From Hussain et al. (2006).

permit a designer to reduce the elastic seismic force demand through a response modification factor. The response modification factor is the ratio required to maintain the structure from elastic to the inelastic design strength. Since reduced seismic force is used in the design, computed displacements from an elastic analysis are amplified in order to estimate the actual deformations following a severe earthquake. The displacement (or drifts), calculated through structural analysis, is not the real one; rather, it is less than the maximum structural displacement during strong tremors. The seismic design provisions estimate the maximum roof displacement and story drifts by augmenting the elastic analysis of displacement amplification factor (C_d) (Uang and Maarouf 1994):

$$\Delta_{\max} = \Delta_W \times C_d \tag{1}$$

where Δ_{max} is the maximum inelastic displacement (roof or story drifts), Δ_W is the displacement calculated by elastic analysis, and C_d is the displacement amplification factor.

Figure 4 represents the structural relations of a base shear and roof displacement, which can be developed by

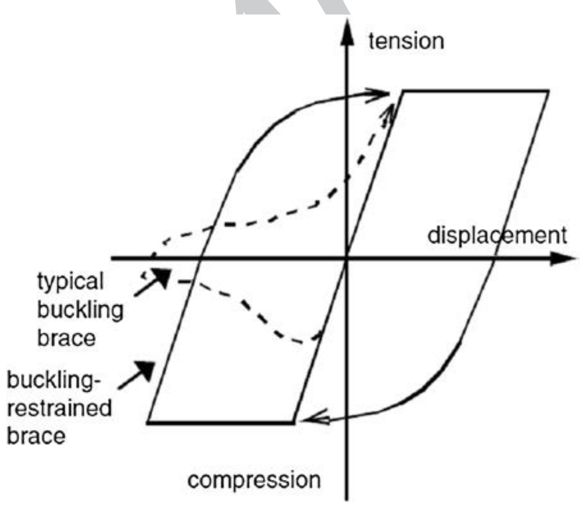

Figure 2 Axial force displacement behavior of braces. From AISC (2005).

a nonlinear analysis. In this figure, real nonlinear behavior is idealized by a bilinear elasto-plastic relation. Displacement amplification factor (C_d) and response modification factor (R) are determined as follows (Uang and Maarouf 1994):

$$C_d = \mu \times R_S \tag{2}$$

$$R = R_\mu \times R_S \tag{3}$$

where R_μ is a reduction factor due to ductility, R_S is the overstrength factor, and μ is the structural ductility factor defined as follows:

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \tag{4}$$

where, according to Figure 4, Δ_{max} is the maximum displacement for the first life safety performance in the structure, and Δ_y is the yield displacement observed in the structure.

Several formulas of reduction factor due to ductility have been proposed by previous researchers such as Riddell (1989), Nassar and Krawinkler (1992), Miranda (1993), and Fajfar (2002). In the simple version of the N2 method proposed by Fajfar (2002), R_μ is written as follows:

$$\begin{aligned} R_\mu &= (\mu - 1)\frac{T}{T_C} + 1 (T < T_C) \\ R_\mu &= \mu (T \geq T_C) \end{aligned} \tag{5}$$

where T_C is the characteristic period of the ground motion, and T is the fundamental period.

The strength revealed during the formation of plastic hinges is called overstrength, which is one of the important parameters in seismic design of structures. The overstrength factor R_S is defined by Mahmoudi and Zaree (2011) as follows:

$$R_S = \frac{V_u}{V_W} \times R_1 \times R_2 \tag{6}$$

According to Figure 4, V_W is the design base shear of the building, and V_u is the base shear with relevance to

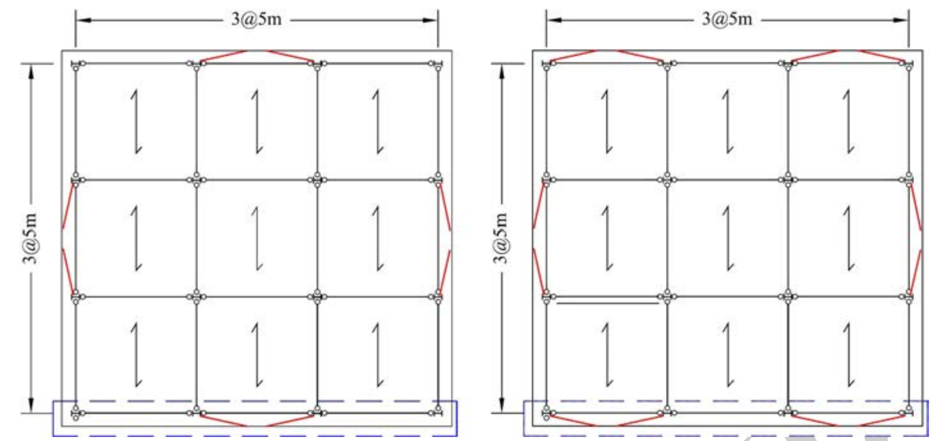

Figure 3 **Plan views of buildings and brace locations in the frames.**

Table 1 Sectional properties of seven-story model structures with inverted V braces

Braced type	Number of story	Interior column	Exterior column	Braces	Beam
Conventional CBFs					
Single bay brace frame	1	Box 150 × 150 × 10	Box150×150×10	2UNP120	IPE360
	2	Box 150 × 150 × 10	Box150×150×10	2UNP160	IPE360
	3	Box 150 × 150 × 10	Box 150 × 150 × 10	2UNP180	IPE360
	4	Box 250 × 250 × 15	Box 150 × 150 × 10	2UNP200	IPE360
	5	Box 250 × 250 × 15	Box 150 × 150 × 10	2UNP200	IPE360
	6	Box 300 × 300 × 20	Box 150 × 150 × 10	2UNP220	IPE360
	7	Box 300 × 300 × 20	Box 150 × 150 × 10	2UNP220	IPE360
Double bays brace frame	1	Box 150 × 150 × 10	Box 150 × 150 × 10	2UNP100	IPE360
	2	Box 150 × 150 × 10	Box 150 × 150 × 10	2UNP120	IPE360
	3	Box150 × 150 × 10	Box 150 × 150 × 10	2UNP120	IPE360
	4	Box150 × 150 × 10	Box 150 × 150 × 10	2UNP140	IPE360
	5	Box 200 × 200 × 15	Box 200 × 200 × 15	2UNP160	IPE360
	6	Box 200 × 200 × 15	Box 200 × 200 × 15	2UNP160	IPE360
	7	Box 250 × 250 × 15	Box 250 × 250 × 15	2UNP160	IPE360
BRBFs					
Single bay brace frame	1	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×15	IPE360
	2	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×15	IPE360
	3	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×18	IPE360
	4	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×20	IPE360
	5	Box 200 × 200 × 15	Box 150 × 150 × 10	PL50×20	IPE360
	6	Box 200 × 200 × 15	Box 150 × 150 × 10	PL50×20	IPE360
	7	Box 200 × 200 × 15	Box 150 × 150 × 10	PL60×20	IPE360
Double bays brace frame	1	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×10	IPE360
	2	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×10	IPE360
	3	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×10	IPE360
	4	Box 150 × 150 × 10	Box 150 × 150 × 10	PL50×10	IPE360
	5	Box 150 × 150 × 10	Box150 × 150 × 10	PL50×12	IPE360
	6	Box 200 × 200 × 15	Box 200 × 200 × 15	PL50×12	IPE360
	7	Box 200 × 200 × 15	Box 200 × 200 × 15	PL50×12	IPE360

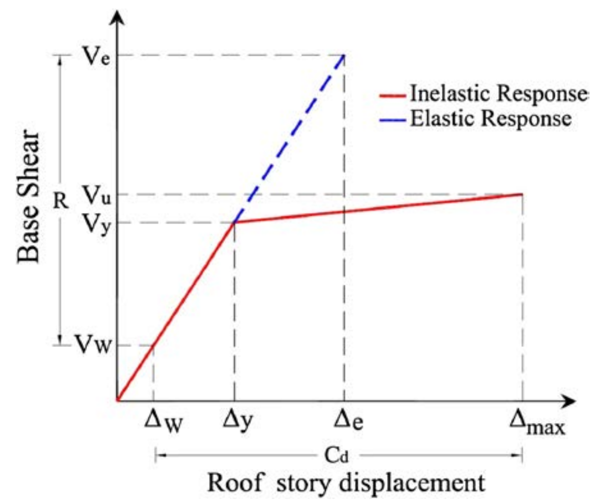

Figure 4 General structure response.

the first life safety performance of the structural members. R_1 accounts the difference between actual and nominal static yield strengths. For structural steel, a statistical study shows that the value of R_1 may be taken as 1.05 (Schmidt and Bartlett 2002). The parameter R_2 may be used to consider an increase in yield stress as a result of strain rate effect during an earthquake. For the strain rate effect, a value of 1.10 or a 10% increase could be used (Uang 1991).

To confirm displacement amplification factor (C_d) obtained from pushover analysis, Equation 7 was used for dynamic analysis:

$$C_d = \frac{\Delta_{max}}{\Delta_W} \times R_1 \times R_2 \times R_{SP} \tag{7}$$

where Δ_W is the design displacement, and R_{SP} is the post-buckling overstrength factor for CBFs. The post-buckling 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 (Mahmoudi and Zaree 2011).

Considering Equations 2 and 3, the ratio of C_d and R is thus:

$$\frac{C_d}{R} = \frac{\mu \times R_S}{R_\mu \times R_S} = \frac{\mu}{R_\mu} \tag{8}$$

According to Equation 8, it is feasible to evaluate the ratio of μ/R_μ instead of C_d/R. For building frame systems, various codes present the numerical values of the ratio between C_d and R. For instance, this ratio for concentrically steel braced frames is from 0.5 to 1 in NEHRP (1994) and IBC (2000) and is equal to 0.7 in the Iranian Earthquake Resistance Design Code (Standard No. 2800) (BHRC 2005).

Table 2 Characteristics of earthquake ground motions

Record	Year	Peak ground acceleration(*g*)	Duration(s)
El Centro	1940	0.348	53.7
Naghan	1977	0.723	5
Tabas	1978	0.915	25

Nonlinear analysis

Most structures experience inelastic deformations when subjected to severe earthquake ground motions. Therefore, the nonlinear behavior of structures should be taken into account to have accurate estimation of deformation demands. The extensive set of nonlinear analyses of the model buildings presented opportunities for the investigation of a large number of different response characteristics. Different performance criteria were also defined to verify structural or nonstructural elements under various performance levels. Structural performance level life safety (LS) is considered for the C_d assessment carried out in the present study. In the LS performance, the structure, or any part of it, does not collapse, retaining integrity and residual load capacity after the earthquake. The structure is significantly damaged and may have moderate permanent drifts but retains its full vertical load bearing capacity and sufficient residual lateral strength and stiffness to protect life even during strong aftershocks. Thus, nonlinear static (pushover) analysis and nonlinear time history analysis were used at life safety structural performance level. To do so, the SNAP-2DX (Rai et al. 1996) program was used.

Nonlinear static (pushover) analysis

Pushover analyses provide information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis. Furthermore, the ability of nonlinear static procedures to predict the maximum roof displacement caused by ground

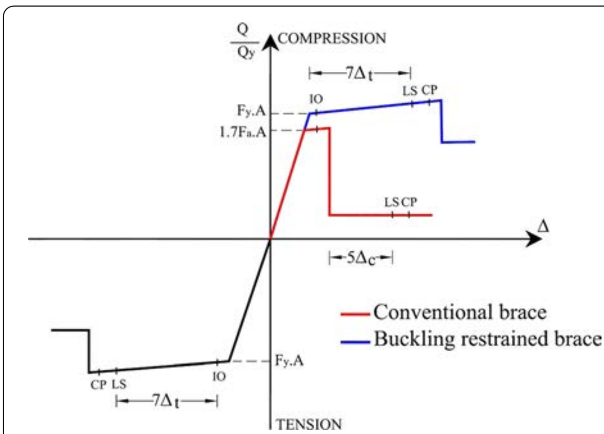

Figure 5 Generalized force-deformation relation for steel brace elements (FEMA-356). From Hajirasouliha and Doostan (2010).

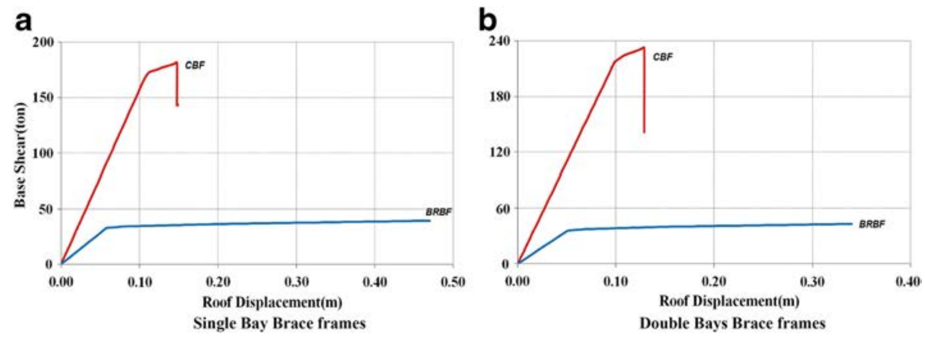

Figure 6 Roof displacement-base shear pushover curve for a seven-story inverted V braced frames. (**a**) Single bay brace frames. (**b**) Double bay brace frames.

Table 3 Displacement amplification factors for concentric braced frames result from pushover analysis

Brace type	Number of story	Single bay brace frame						Double bays brace frame					
		μ	R_S	R_μ	C_d	R	C_d/R	μ	R_S	R_μ	C_d	R	C_d/R
Conventional CBFs													
Chevron inverted V	3	1.45	4.24	1.31	6.15	5.55	1.11	1.53	6.09	1.35	9.31	8.22	1.13
	5	1.44	3.75	1.44	5.40	5.40	1.00	1.26	5.08	1.26	6.40	6.40	1.00
	7	1.39	3.72	1.39	5.17	5.17	1.00	1.25	4.74	1.25	5.92	5.92	1.00
	10	1.30	3.51	1.30	4.56	4.56	1.00	1.19	4.70	1.19	5.59	5.59	1.00
	12	1.29	3.50	1.29	4.51	4.51	1.00	1.18	4.29	1.18	5.06	5.06	1.00
Chevron V	3	1.48	3.82	1.32	5.65	5.04	1.12	1.39	5.22	1.26	7.25	6.58	1.11
	5	1.38	2.98	1.38	4.11	4.11	1.00	1.34	4.05	1.34	5.43	5.43	1.00
	7	1.42	2.88	1.42	4.08	4.08	1.00	1.28	3.67	1.28	4.69	4.69	1.00
	10	1.35	2.80	1.35	3.78	3.78	1.00	1.26	3.41	1.26	4.29	4.29	1.00
	12	1.32	2.78	1.32	3.66	3.66	1.00	1.26	3.27	1.26	4.05	4.05	1.00
X brace	3	1.68	3.61	1.49	6.06	5.38	1.13	1.50	5.86	1.34	8.79	7.85	1.12
	5	1.51	3.38	1.51	5.10	5.10	1.00	1.44	4.46	1.44	6.28	6.28	1.00
	7	1.52	3.05	1.52	4.64	4.64	1.00	1.45	4.16	1.45	6.03	6.03	1.00
	10	1.48	2.92	1.48	4.32	4.32	1.00	1.37	3.96	1.37	5.42	5.42	1.00
	12	1.42	2.86	1.42	4.06	4.06	1.00	1.37	3.67	1.37	5.03	5.03	1.00
BRBFs													
Chevron inverted V	3	9.47	2.41	6.76	22.82	16.30	1.40	9.33	3.41	6.68	31.80	22.85	1.39
	5	8.94	1.78	8.94	15.90	15.90	1.00	7.31	2.60	7.31	19.00	19.00	1.00
	7	7.54	1.74	7.54	13.12	13.12	1.00	6.72	2.36	6.72	5.89	5.89	1.00
	10	5.91	1.57	5.91	9.27	9.27	1.00	6.25	2.00	6.25	2.51	2.51	1.00
	12	5.02	1.40	5.02	7.02	7.02	1.00	5.25	1.78	5.25	9.34	9.34	1.00
Chevron V	3	8.77	2.53	6.30	22.18	15.93	1.39	8.89	3.39	6.38	30.13	21.62	1.39
	5	8.07	1.85	8.07	14.96	14.96	1.00	7.09	2.43	7.09	17.23	17.23	1.00
	7	7.25	1.84	7.25	13.34	13.34	1.00	6.91	2.19	6.91	15.13	15.13	1.00
	10	5.49	1.70	5.49	9.33	9.33	1.00	6.24	1.76	6.24	10.98	10.98	1.00
	12	4.75	1.58	4.75	7.50	7.50	1.00	5.11	1.48	5.11	7.56	7.56	1.00

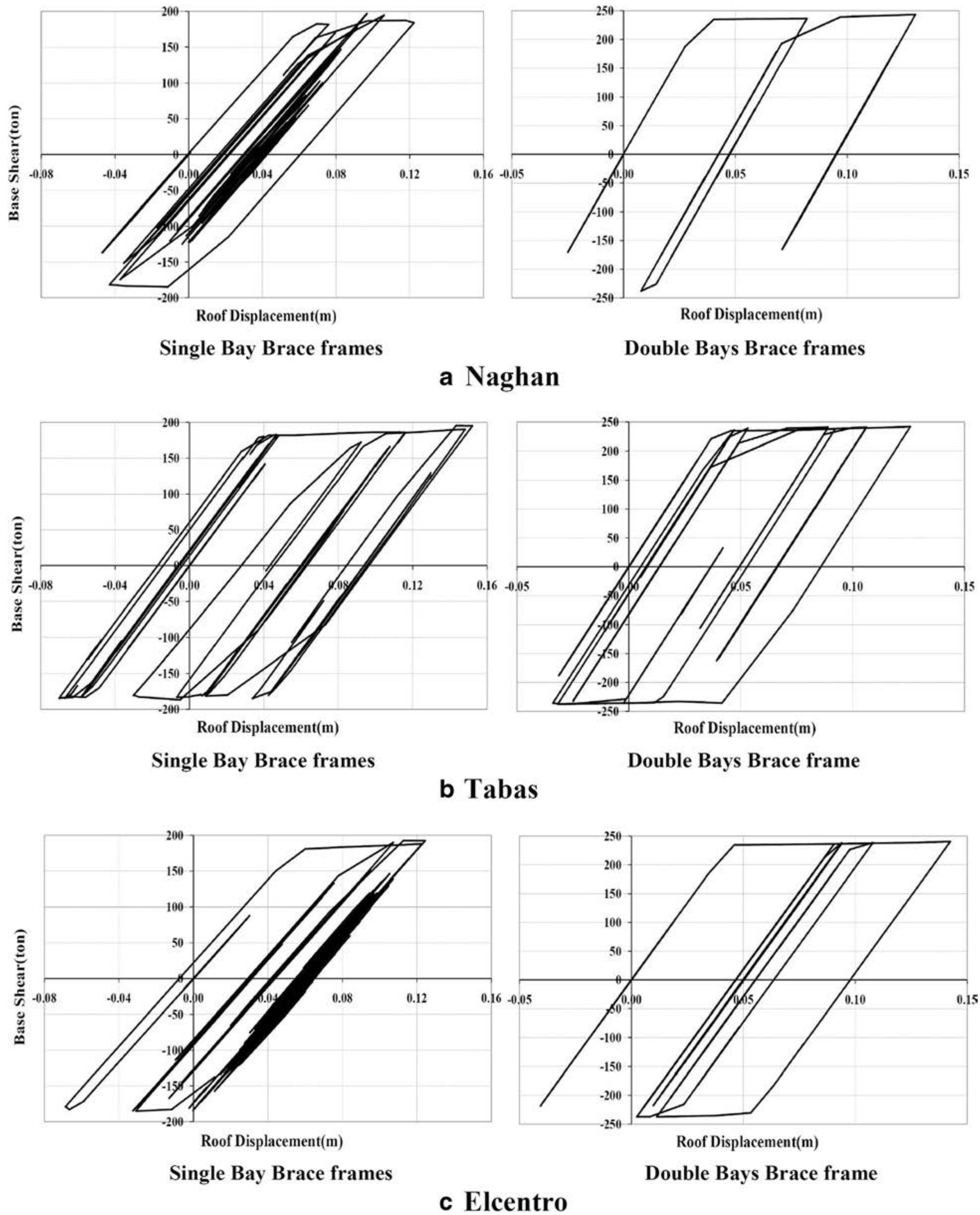

Figure 7 Nonlinear time history curves for seven-story conventional inverted V CBFs. (**a**) Naghan, (**b**) Tabas, and (**c**) El Centro earthquakes.

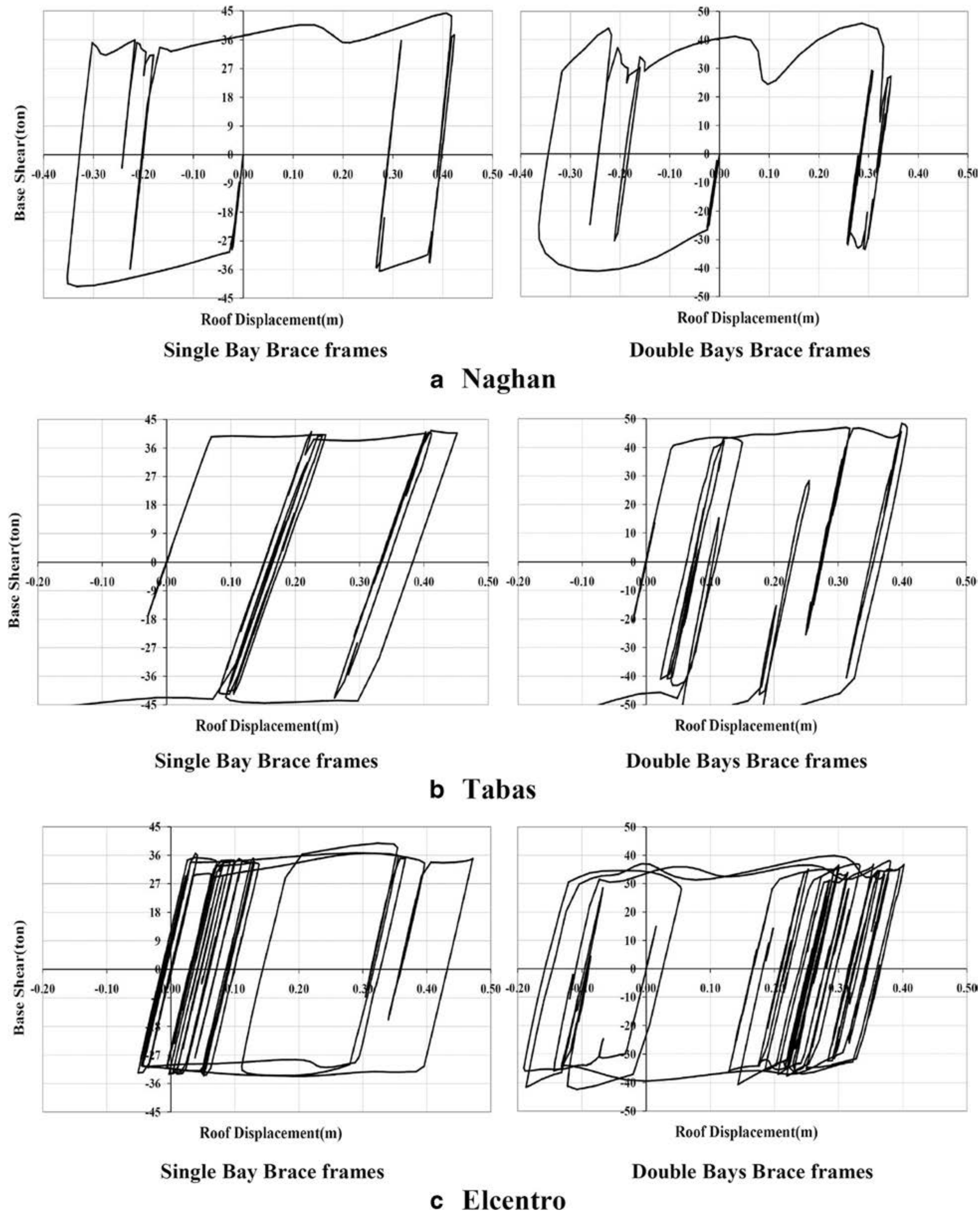

Figure 8 Nonlinear time history curves for seven-story inverted V BRBFs. (**a**) Naghan, (**b**) Tabas, and (**c**) El Centro earthquakes.

Table 4 Displacement amplification factors for concentric braced frame results from dynamic analysis

Brace type	Number of story	Single bay brace frame			Double bay brace frame		
		Δ_W(mm)	Δ_{max}(mm)	C_d	Δ_W(mm)	Δ_{max}(mm)	C_d
Conventional CBFs							
Chevron inverted V	3	10	48	6.03	4	31	9.27
	5	20	82	5.24	14	72	6.55
	7	36	134	4.84	27	127	6.03
	10	77	279	4.64	60	247	5.29
	12	113	370	4.21	89	330	4.75
Chevron V	3	11	47	5.45	6	35	7.32
	5	25	87	4.26	19	78	5.21
	7	45	139	3.87	32	123	4.86
	10	86	255	3.70	73	231	3.97
	12	125	346	3.46	101	314	3.86
X brace	3	8	34	6.21	6	34	8.95
	5	20	75	5.54	15	67	6.60
	7	37	121	4.83	28	117	6.18
	10	77	230	4.39	62	216	5.17
	12	125	333	3.93	90	298	4.90
BRBFs							
Chevron inverted V	3	13	248	21.87	7	199	32.36
	5	31	420	15.40	19	340	20.14
	7	42	460	12.57	32	406	14.69
	10	91	708	8.99	71	691	11.26
	12	154	956	7.15	117	915	9.06
Chevron V	3	13	229	20.98	8	208	29.3
	5	33	407	14.34	21	327	17.96
	7	42	506	13.98	40	491	15.13
	10	91	991	8.82	74	725	10.28
	12	152	943	7.14	128	840	7.56

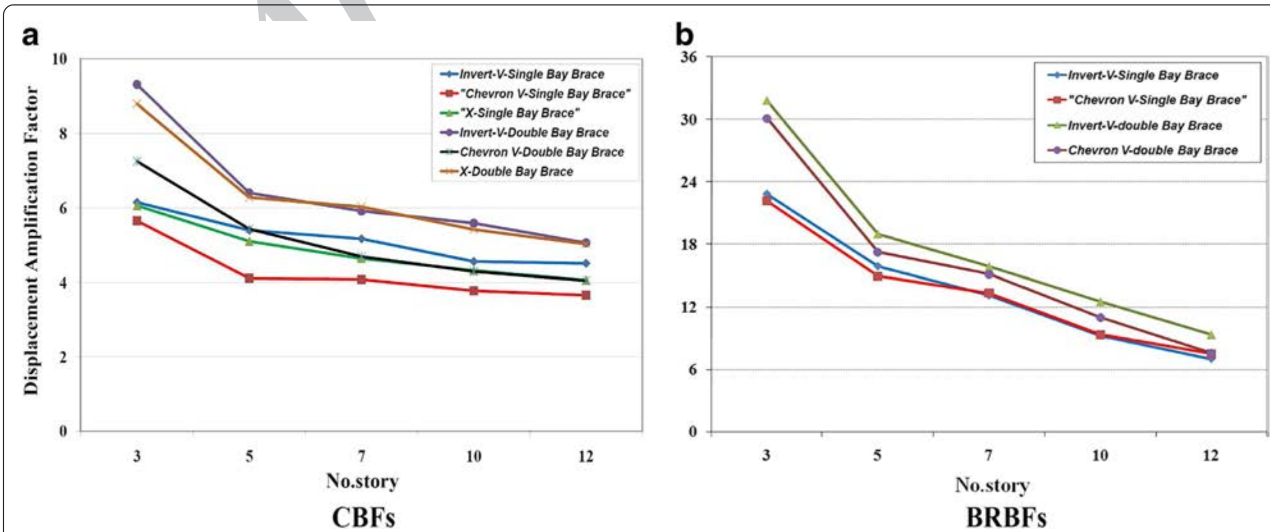

Figure 9 Displacement amplification factors for concentric braced frames. (**a**) CBFs and (**b**) BRBFs.

motion for CBFs was emphasized (Moghaddam and Hajirasouliha 2006). To achieve overstrength factor (R_S), ductility factor (μ), reduction factor due to ductility (R_μ), and finally displacement amplification factor (C_d) and its relation to response modification factor (R), the pushover analysis was carried out by subjecting a structure to monotonically increasing lateral forces with an invariant height-wise distribution.

Nonlinear time history analysis

Nonlinear time history analysis of a detailed analytical model is perhaps the best option to estimate deformation demands (Hajirasouliha and Doostan 2010). Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions, and limitations of current pushover procedures must be identified. Thus, nonlinear time history analyses have been performed to confirm the adequacy of static (pushover) analyses. Nonlinear dynamic analyses were carried out by employing suites of time history of the El Centro, Naghan, and Tabas earthquake matching with the design spectrum. The properties of the records used for this study are summarized in Table 2.

Modeling nonlinear behavior of braces, a generalized force-deformation relation was used for steel brace element as suggested by FEMA-356 (2000) (Figure 5). For buckling restrained braces, the model presented in Tables five, six, and seven in FEMA-356 (2000) were considered for both tension and compression behavior (Figure 5). The post-yield stiffness of beams, columns, and braces was initially assumed to be 2%. In Figure 5, Q, Q_y, and Δ are the generalized component load, expected strength, and component displacement, respectively. For conventional brace in compression, the residual strength after degradation is 20% of the buckling strength, and life safety plastic

Table 5 Comparison of displacement amplification factors for concentric braced frames

Brace type	Number of story	Single bay brace frame		Double bay brace frame	
		Pushover analysis	Time history analysis	Pushover analysis	Time history analysis
Conventional CBFs					
Chevron inverted V	3	6.15	6.03	9.31	9.27
	5	5.40	5.24	6.40	6.55
	7	5.17	4.84	5.92	6.03
	10	4.56	4.64	5.59	5.29
	12	4.51	4.21	5.06	4.75
Chevron V	3	5.65	5.45	7.25	7.32
	5	4.11	4.26	5.43	5.21
	7	4.08	3.87	4.69	4.86
	10	3.78	3.70	4.29	3.97
	12	3.66	3.46	4.05	3.86
X brace	3	6.06	6.21	8.79	8.95
	5	5.10	5.54	6.28	6.60
	7	4.64	4.83	6.03	6.18
	10	4.32	4.39	5.42	5.17
	12	4.06	3.93	5.03	4.90
BRBFs					
Chevron inverted V	3	22.82	21.87	31.80	32.36
	5	15.90	15.40	19.00	20.14
	7	13.12	12.57	5.89	14.69
	10	9.27	8.99	2.51	11.26
	12	7.02	7.15	9.34	9.06
Chevron V	3	22.18	20.98	30.13	29.3
	5	14.96	14.34	17.23	17.96
	7	13.34	13.98	15.13	15.13
	10	9.33	8.82	10.98	10.28
	12	7.50	7.14	7.56	7.56

deformation Δ_{LS} is equal to $5\Delta_C$ (Δ_C is the axial deformation at expected buckling load). On the other hand, for conventional brace in tension and buckling restrained brace, the life safety plastic deformation Δ_{LS} is equal to $7\Delta_T$ (Δ_T is the axial deformation at expected tensile yielding load). Based on earlier tests, the compression strength of BRB was assumed to be 10% larger than the strength in tension (Clark 2000). In this paper, the compression strength for BRB is considered equal to the tension strength. To capture the greatest demands on braces and beams, flexible beams were used.

Results and discussion

Base shear vs. maximum roof displacement nonlinear pushover analysis for a seven-story inverted V CBF and BRBF with single and double bracing bays are shown in Figure 6. As it is shown, the stiffness of CBF is higher than BRBF, whereas the ductility for BRBF is higher compared to CBF. Table 3 shows the displacement amplification factors (C_d) and its relations to response modification factors (R) for both CBFs and BRBFs. Figures 7 and 8 show nonlinear dynamic time history analysis results of the El Centro, Naghan, and Tabas earthquakes for a seven-story inverted V CBF and BRBF with single and double bracing bays. Table 4 shows the displacement amplification factors (C_d) for both CBFs and BRBFs assessed from nonlinear dynamic time history analysis.

The ductility in CBFs has lower values due to deterioration in strength and degradation of stiffness due to brace buckling in cyclic loading. Thus, it can be said that the overstrength factors have main effects on displacement amplification factors for CBFs. The number of bracing bays and structure height have an effect on overstrength and thus on C_d. On the other hand, these factors as such have no obvious result on ductility, so changing the structure height and the number of bracing bay has no effect on the ratio C_d/R.

In BRBFs, because of brace energy dissipation capacity in tension and compression, the ductility has high values and becomes the main parameter to determine displacement amplification factors. Also, structure height has a profound effect on ductility, so for BRBFs, variation in the number of stories has obvious impression on displacement amplification factors and the ratio C_d/R. On the other hand, structure characteristic cause little variation in overstrength factors and so in C_d and the ratio C_d/R.

Variation in displacement amplification factors for different types of concentric braced frames are shown in Figure 9. The comparison of pushover and nonlinear dynamic analysis displacement amplification factors are shown in Table 5. According to the results, pushover analysis provides good predictions of seismic demands for concentrically braced steel frames.

Conclusions

This paper assesses the displacement amplification factor (C_d) and the ratio between C_d and R factor of 30 conventional CBFs and 20 BRBFs in life safety structural performance level. For this purpose, the nonlinear static (pushover) analysis and nonlinear dynamic time history analysis have been performed on the buildings with single and two bracing bays, various stories, and different buckling restrained brace and conventional brace configurations. The beam-column connections were assumed to be pinned so that the seismic load was resisted mainly by braces. The results of this study can be summarized as follows:

- The displacement amplification factors increase with the decrease of structure height and the increase in the number of bracing bays. However, the number of bracing bays has no effect on the ratio C_d/R in both CBFs and BRBFs. The ratio between C_d and R factor is from 1 to 1.12 and 1 to 1.4 for CBFs and BRBFs, respectively.
- The displacement amplification factors for CBFs in type V, inverted V, and X are evaluated as 4.40, 5.20, and 4.90 for single bracing bay and as 5.40, 6.80, and 6.60, respectively, for double bracing bays.
- The obtained displacement amplification factors for different types of BRBFs with single bracing bay vary from 7 to 22.50, and for double bracing bays, these are from 8 to 31.
- The structure height in CBFs has no effect on ratio C_d/R, but in BRBFs, this has an effect on the ratio C_d/R because of ductility variation. Thus, in ductile brace frame systems (high ductility) and stiff buildings (low fundamental period), the ratio C_d/R is higher than 1 when the fundamental period (T) is lower than the period of the ground motion (T_C).

Competing interests
The authors declare that they have no competing interests.

Authors' contributions
MM proposed the subject of the research and participated in the sequence alignment. He also controlled the results. ZM carried out the structural analysis (linear and nonlinear) and data processing and participated in the sequence alignment. All authors read and approved the final manuscript.

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