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Response modification factor of dual moment-resistant frame with buckling restrained brace (BRB)

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Abstract. Response modification factor is one of the seismic design parameters to consider nonlinear performance of building structures during strong earthquake, in conformity with the point that many seismic design codes led to reduce the loads. In the present paper it's tried to evaluate the response modification factors of dual moment resistant frame with buckling restrained braced (BRB). Since, the response modification factor depends on ductility and overstrength; the nonlinear static analysis, nonlinear dynamic analysis and linear dynamic analysis have been done on building models including multi-floors and different brace configurations (chevron *V*, invert *V*, diagonal and *X* bracing). The response modification factor for each of the BRBF dual systems has been determined separately, and the tentative value of 10.47 has been suggested for allowable stress design method. It is also included that the ductility, overstrength and response modification factors for all of the models were decreased when the height of the building was increased.

Keywords: *R*-factor; ductility and overstrength factors; dual moment resistant frame with BRB

1. Introduction

The preliminary design in most of the buildings is normally based on equivalent static forces specified by the governing building codes. The height-wise distribution of these static forces seems to be based implicitly on the elastic vibration modes. However, the structures do not remain elastic during severe earthquakes and they are expected to undergo large nonlinear deformations (Moghaddam *et al*. 2005) as a matter of fact, many seismic codes permit a reduction in design loads, having the 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 (Kim and Choi 2005). In fact, the response modification factor (*R*) affects the capability of a structure to dissipate energy through inelastic behavior. The current study intends to characterize the important aspects of hysteretic behavior of different structural systems undergoing inelastic response, during severe earthquake incidents (Mitchel *et al*. 2003). Steel concentric braced frame (CBF) is one of the efficient and commonly used lateral load resisting systems, especially in the structures of high seismic regions (or moderate to high seismic prone zone, FEMA-356.2 2003) .The steel braces improve the lateral

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strength and the stiffness by inelastic deformation during an earthquake that leads to seismic energy dissipation (Davaran and Hoveidae 2009). Studies show that the lateral response of CBFs is mainly dominated by inelastic behavior of bracing members (Annan *et al*. 2009), hence these members are subjected to alternating tension and compression loads when CBFs are exposed to the earthquake loading (Broderick *et al*. 2008). It is because of the post-buckling hysteresis behavior of bracing members and based up on the cyclic loading that cause the braced frames yield and dissipate the energy (Lee and Bruneau 2005). However, the energy dissipation capacity of a steel braced structure is limited due to the buckling of the braces (Kim and Seo 2004). Considering this limitation, some efforts have been made to develop new CBF systems with stable hysteretic behavior, significant ductility as well as large energy dissipation capacity. A system such as CBF with an improved seismic behavior is the buckling restrained braced frame (BRBF) that not only enhances the energy dissipation capacity of a structure but also decreases the demand for inelastic deformation of the main structural members. The behavioral or response modification factors of CBFs and BRBFs have been the subjects of investigations by various researchers. Maheri and Akbari (2003) also investigated the response modification factors of steel braced reinforced concrete frame. They found that the response modification factor would be significantly increased by adding steel *X* and knee braces, and also the number of stories would be the predominant variable. DiSarno *et al*. (2008) found that the overstrength was being increased about 33% in CBFs by stainless steel braces and columns. Thus it will be ineffectual to use stainless steel in CBF beams. According to Davaran and Hoveidae (2009), the type of mid-connection detail of *X* concentric braced frame could improve response modification factor to about 28% more than the one with common mid-connection detail. Sabelli *et al*. (2003) presented a series of models with chevron BRBs which were designed and analyzed once subjected to tremors representing various seismic hazard levels. They found that the BRBF response was not influenced by R factors in the range of 6-8. During doing other researches, Mahmoudi and Zaree (2010)indicated that the response modification factor of BRBF was higher than CBF, they also found that the number of bracing bays and the height of buildings had more influence on the response modification factors. Di Sarno and Manfredi (2010) Found that the seismic performance of typical reinforced concrete (RC) of the existing framed structures that retrofitted with buckling resistant brace, demonstrated that both global and local lateral displacements were notably reduced after the seismic retrofit of the existing system. The computed inter storey drifts were 2.43% at Collapse prevention limit state (CP) and 1.92% at Life Safety limit state (LS) for modal distribution of lateral forces, or Conversely for the retrofitted structure, the estimated values of inter storey drifts (d/h) were halved; the maximum d/h were 0.84% at CP (along the Y-direction) and 0.65% at LS (yet along the Y-direction). Furthermore, lateral drifts are uniformly distributed along the height; but the damage localizations were inhibited, especially at ultimate limit states, i.e., LS and CP.

Kiggins and Uang (2006) found that the buckling restrained brace with steel moment frames not only reduced residual story drifts but also led to a larger value of response modification factor, comparing concrete structures. Having concentric steel bracings with those having BRB systems, Rahai and Alinia (2008) found that the concentric *X* bracing laterally creates rigid structures but the BRB system produces a concurrent suitable rigidity, ductility and maximum overstrength factors for structures; so it confirms a better performance of the BRB system in the nonlinear range. Asgarian and Shokrgozar (2009) used both the pushover and the nonlinear incremental dynamic analysis to evaluate overstrength, ductility and response modification factors of BRBFs with two bracing bays and various story. Finally the results showed that the response modification factor had been decreased as the height of building had been increased. Chang and Chiu (2011) showed that the BRBs could provide a high level of confidence, ensuring the building to achieve the performance objectives of immediate occupancy and life safety.

Considering cyclic behavior of bracing members in life safety structural performance level, as suggested by (FEMA-356 2003), the current paper intends to evaluate the overstrength, ductility and response modification factors of dual moment resistant frame with buckling restrained brace (BRBF). The model buildings were loaded by (Iranian Earthquake Resistance Design Code (Standard No. 2800 2005) and designed in accordance with part 10 of Iranian National Building code, steel structure design (Iranian national building code (part 10) 2009) and seismic provision of AISC (2005). To acquire those behavioral factors, the nonlinear static pushover analysis, nonlinear dynamic analysis and linear dynamic analysis were conducted.

2. Buckling restrained braced frames

Concentric Braced Frames (CBFs) are the most efficient lateral load resisting systems. In former earthquakes, including 1994 Northridge and 1995 Kobe events, a significant number of this type of structures suffered extensive damage, required extensive repair and upgrading work (Rai and Goel 2003). Under seismic loading, the bracing members undergo large deformations in the post-buckling range that cause large reversed cyclic rotation at the plastic hinges formed in the brace members and the connections at the end (Rai and Goel 2003). In recent years, researchers have focused on a relatively new type of bracing system named “Buckling-Restrained Braces” in order to overcome some of the problems with ordinary braces. In this type of braces, load deflection characteristic of member is a stable one and the member is unable to carry compressive load up to tensile yield strength (Asgarian and Amirhesari 2008). On the contrary to ordinary braces, buckling-restrained braces are able to achieve stable, balanced hysteretic behavior and substantial ductility by accommodating compression yielding before the onset of buckling (Black *et al*. 2002). Typical buckling-restrained braces (Fig. 1) consist of a central yielding steel member that carries the entire axial load of the braces, confined by steel sleeve-only or a combination of steel sleeve and concrete outer member which provides the brace flexural, and also the buckling

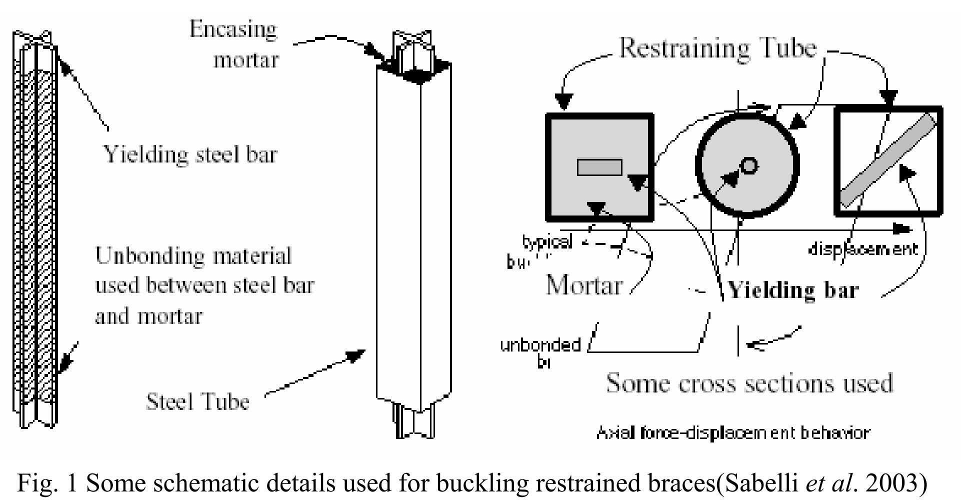


Fig. 1 Some schematic details used for buckling restrained braces(Sabelli *et al*. 2003)

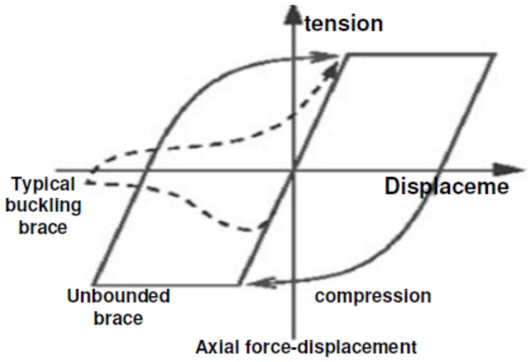


Fig. 2 Behavior of conventional brace versus buckling-restrained brace(Ravi Kumar *et al*. 2007)

resistance. Buckling restrained braces consist of ductile steel core which is in yielding position in tension and compression. The steel core is put inside a steel sleeve and filled with concrete, cement grout or any other inter filler in order to prevent buckling under compression. The bond between the core and the filler is broken to ensure that no part of the axial force is transferred to sleeve. The projection of core beyond the sleeve is designed so that the core dose not yield or buckle locally in this region (Black *et al*. 2002, Tsai *et al*. 2004b). The behavior of buckling restrained braces under monotonic compression and cyclic loading has been studied by (Kalyanaraman *et al*. 2003). The main advantage of these braces is to increase buckling mode and high compressive strength capacity in comparison with the ordinary braces (Tsai *et al*. 2004a). So, the buckling criterion in these braces will stand after yield criterion of the steel core. This criterion causes noticeable ductility increase in this system, compared to ordinary braced frames (Uang and Nakashima 2003). Fig. 2 shows a comparison of a typical hysteretic curve of an ordinary bracing and a buckling restrained bracing.

3. Response modification factor

Structures elastic analysis under earthquake can create base shear force and stress, which are significantly bigger than real structure response. The structure can absorb quiet a lot of earthquake energy, and it resists, when entering the inelastic range of deformation. Overstrength in structures depends on the fact that the maximum lateral strength of a structure generally exceeds its design strength. Therefore, seismic codes reduce design loads considering the fact that the structures possess overstrength and ductility. In fact the response modification factor includes inelastic performance of structure, and indicates the over strength and ductility of structure in inelastic stage (ATC 1995). Mazzolani and Piluso (1996) addressed several theoretical approaches such as maximum plastic deformation, energy and low cycle fatigue approaches to compute response modification factor. As it is shown in Fig. 3, the real nonlinear behavior is usually idealized by a bilinear elastic perfectly plastic relation. The yield force of structure is shown by V_y and the yield displacement is Δy. In this figure V_e ($V_{\max}$) correspond to the elastic response strength of the structure. The maximum base shear in an elastic perfectly plastic behavior is V_y (Uang 1991). The ratio of maximum base shear considering elastic behavior, V_e, to maximum base shear in elastic perfectly plastic behavior, V_y, is called force reduction factor

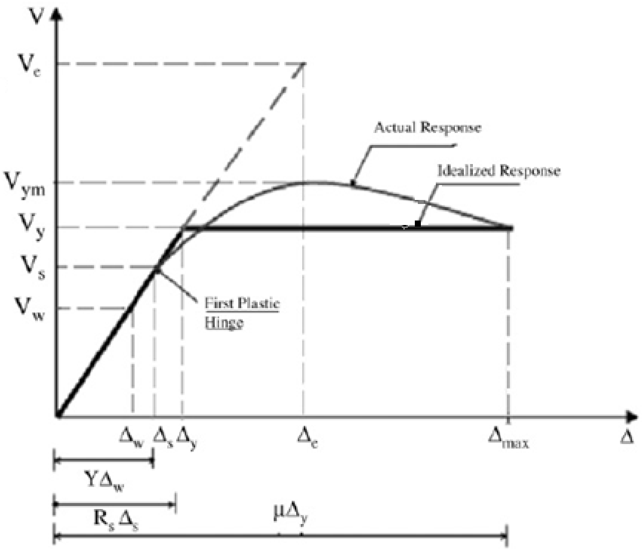


Fig. 3 General structure response (Uang 1991)

$$R_\mu = \frac{V_e}{V_y} \tag{1}$$

The overstrength factor is defined as the ratio of maximum base shear in idealized behavior, V_y, to first significant yield strength in structure, V_s

$$R_s = \frac{V_y}{V_s} \tag{2}$$

The concepts of over-strength, redundancy and ductility, which are used to scale down the earthquake forces, need to be clearly defined and expressed in quantifiable terms. The value of (2) for overstrength factor is proposed to establish R for buckling restrained braced frames in (SEAOC 2001). In this paper, overstrength factor of the frames is computed using Eq. (2) based on analysis results. The over-strength factor shown in Eq. (2) is based on the use of nominal material properties. Denoting this overstrength factor as R_{so}, the actual over strength factor R_s, which can be used to formulate R should consider the beneficial contribution of some other effects (Uang 1991)

$$R_s = R_{so}\mathrm{F}_1\mathrm{F}_2\cdots\mathrm{F}_n \tag{3}$$

In this equation, F_1 is used to account for the difference between actual static yield strength and nominal static yield strength. For steel structures, a statistical study shows that the value of F_1 may be taken as 1.05 (Schmidt and Bartlet 2002). Parameter F_2 may be used to consider the increase in yield stress, as a result of strain rate effect during an earthquake excitation. A value of 1.1, a 10% increase, to account for the strain rate effect, could be used (Uang 1991). In this paper the steel type St-37 is used for all structural members. Parameters F_1 and F_2 equal to 1.05 and 1.1 are considered 1.155 as material overstrength factor. Other parameters can also be included when reliable data is available. They are the parameters such as non structural component contributions, variation of lateral force profile. To design in allowable stress method, the design codes decrease design loads from V_s to V_w. This decrease is done by an allowable stress factor which is defined as

$$Y = \frac{V_s}{V_w} \tag{4}$$

Therefore, the response modification factor, accounts for the ductility and overstrength of the structure and the differences in the level of stresses considered in its design. It is generally the expression of the following form that takes the above mentioned conceptions into accounts

$$R = \frac{V_e}{V_s} = \frac{V_s}{V_y} \times \frac{V_y}{V_s} = R_\mu \times R_s \tag{5}$$

$$R_w = \frac{V_e}{V_w} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times \frac{V_s}{V_w} = R_\mu \times R_s \times Y \tag{6}$$

Eq. (5) is the seismic response modification factor in the ultimate strength design method, and Eq. (6) is the seismic response modification factor in allowable stress design method (Uang 1991).

4. Design of model structures

To evaluate the overstrength, ductility, and the response modification factors of buckling restrained braced frames, the 3, 5, 8, 12 and 15 story buildings with the bay length of 6 m and the four different bracing types (*X*, chevron *V*, chevron-Inverted *V* and diagonal Types) were designed as per the requirement of Iranian Earthquake Resistance Design Code and Iranian National Building Code, part 10, Steel Structure Design. Fig. 4 shows the typical configuration of the used models. The story height of the models was considered as 3 m. For member design subjected to the earthquake, the equivalent lateral static forces were applied on all the story levels. These forces

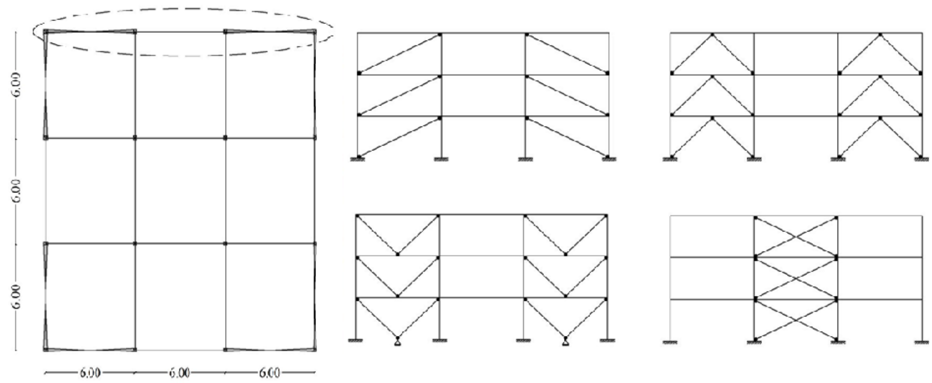


Fig. 4 Configuration of model structure

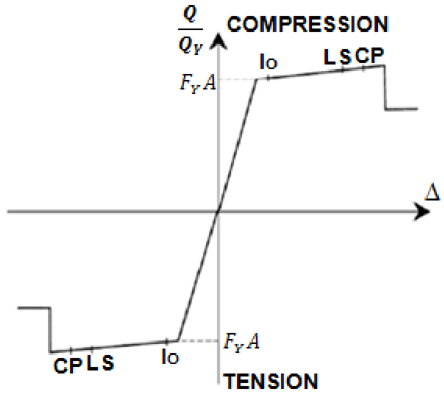


Fig. 5 Generalized force-deformation relation for steel braces elements (FEMA-356 2000).

were calculated following the provisions stated in Iranian Earthquake Code (Standard No. 2800). The dead and live loads of 600 and 200 Kg/m^2, respectively, were used for the gravity load.

The design base shear was computed as follows

$$V = CW \rightarrow C = \frac{ABI}{R} \tag{7}$$

In which V is the base shear of structure, C is seismic coefficient and W is the equivalent weight of the structure. $A*B$ is the design spectral acceleration, expressed as the ratio of gravitational acceleration, for the fundamental period of structure T and the soil type (Fig. 6), I is the importance factor and R is the response modification factor. The importance factor of I = 1, preliminary response modification factors of R = 8 and seismic zone factor of A = 0.35 are considered for frame design. In designing dual system, all beams to column connections are assumed to be the rigid at both ends as frames are designed to be the moment resisting frames. The moment frames are also designed to sustain 25% of the lateral load and the braces are designed to sustain 100 percent of the lateral load, and also the braces are assumed to be pinned at the both ends. Allowable stress design method is used to design frame members in accordance with part 10 of Iranian National Building Code. To ensure that vertical bracing columns are strong enough to resist the force transferred from bracing elements; Iranian Standard No. 2800, has the instruction to design vertical bracing columns for the following load combinations

(a) Axial compression according to

$$P_{DL} + 0.85P_{LL} + 2.8P_E < P_{SC} = 1.7F_a A \tag{8}$$

(b) Axial tension according to

$$0.85P_{DL} + 2.8P_E < P_{ST} = F_y A \tag{9}$$

In which F_a is allowable compressive stress, F_y is the yield stress and A is the area section of column. P_{DL}, P_{LL}, P_E, are axial load from dead, live and earthquake load, respectively, and P_{SC}, P_{ST} are the design tensile and compression strength of column, respectively.

5. Modeling the structure in software

The computational model of the structures was developed using the modeling capabilities of the software framework of SAP 2000. For modeling of the members in nonlinear range of deformation, the following assumptions were assumed. In dual system, all of the frame members, beams and columns were considered as rigid-ended, but the braces were considered as pinned-ended. For the design member, the w section and plate section were considered for (beams, columns) and braces, respectively. To evaluate behavior factors, the nonlinear static analysis (pushover), nonlinear dynamic analysis and linear dynamic have been done. There for, to do these analyses, the nonlinear behaviors of members are suggested by FEMA-356. For buckling restrained braces, the model presented in Tables 5-7 of FEMA-356 were considered for both tension and compression behavior (Fig. 5). The post-yield stiffness of beams, columns and braces was initially assumed to be 2%. In Fig. 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 buckling strength, and life safety plastic deformation ΔLS is equal to $5\Delta C$ (ΔC is the axial deformation at expected buckling load). Whereas, for conventional brace in tension and buckling restrained brace, the life safety plastic deformation ΔLS is equal to $7\Delta T$ (ΔT is the axial deformation at expected tensile yielding load). For determining R-factor and its components (overstrength, reduction ductility), we have to stop the nonlinear analysis based on failure criteria selected in accordance with Iranian Standard Code No. 2800, is explained as follows;

Table 1 Ground motion data

Earthquake	year	PGA (g)	Duration (S)
BAM	2003	0.42	80
TABAS	1978	0.862	33
MANJIL	1990	0.41	46

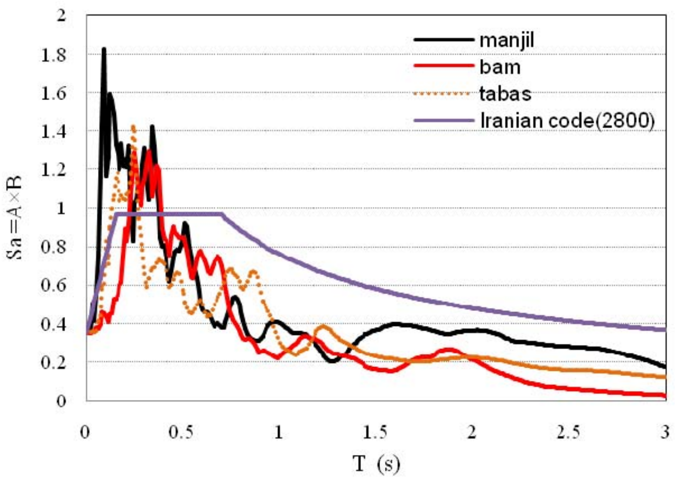


Fig. 6 Variation of spectral with period of structure

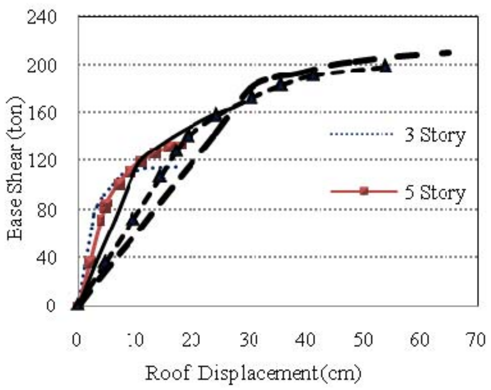


Fig. 7 Roof displacement-base shear curve

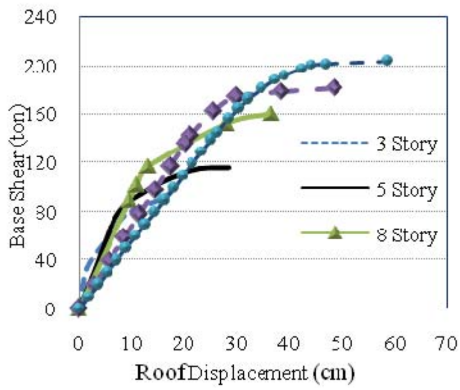


Fig. 8 Roof displacement-base shear curve

For dual systems that have diagonal brace for dual systems that have inverted V brace;

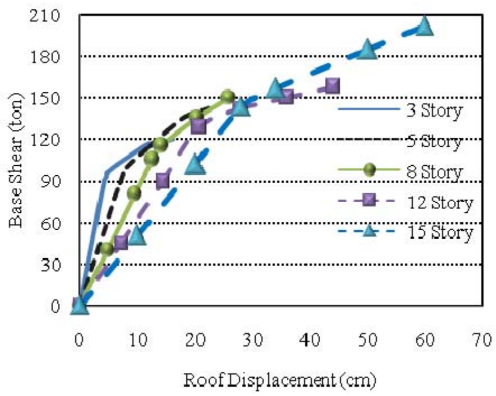


Fig. 9 Roof displacement-base shear curve

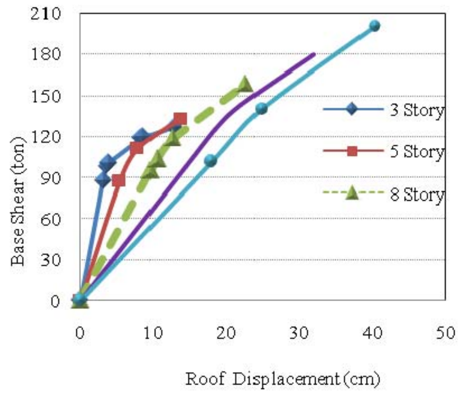


Fig. 10 Roof displacement-base shear curve

For dual systems that have X brace for dual systems that have chevron V brace;

6. Determining the response modification factor

In this paper, two factors R_s and R_μ have been calculated as follows;

6.1 Over strength factor (R_s)

To calculate V_y, the Incremental Nonlinear Dynamic Analysis (IDA) of the models subjected to the strong ground motions matched with the design spectrum was carried out. The response spectra and the design spectrum are shown in Fig. 6. In these analysis by using the time history of

Tabas, Manjil and Bam earthquakes (Table 1), their PGA's with several tries and errors had been changed in a way that the gained time history resulted in the structure reaching to one of the following failure criteria. The maximum nonlinear base shear of this time history is the inelastic base shear of structure (Mwafy and Elnashai 2002). To gain the base shear related to the first plastic hinge formation in structure V_s, the pushover analysis was carried out by the progressively increasing lateral forces, proportional to the fundamental mode shape. It means that the linear ultimate limit of structure in nonlinear static analysis and nonlinear dynamic analysis have been considered the same. Finally the material over-strength factor of 1.155 was considered for actual over-strength factor. The failure criteria were defined in two following levels:

6.1.1 The relative displacement between the floors

The maximum relative story displacement limit (ΔM) is selected based on the Iranian Standard Code No. 2800 as follows [18];

(a) For the frames with the fundamental period less than 0.7 s

$$\Delta M < 0.0025H \tag{10}$$

(b) For the frames with the fundamental period more than 0.7 s

$$\Delta M < 0.025H \tag{11}$$

In which 'H' is the story height. It should be noted that in according with the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800), the expected live safety performance for typical buildings under design earthquake was selected. So in this research, the response modification factor is evaluated for life safety level.

6.1.2 Forming failure mechanism and frame instability

To determine the ultimate limit which was defined by the maximum inter-story drift ratio as discussed, it is necessary to make sure that the frame has kept its stability. In the case of story mechanism or overall mechanism happening in a frame under earthquake event, if the inter-story limit does not occur, the nonlinear dynamic analysis will be stopped and the last scaled earthquake base shear will be selected as ultimate limit state.

To calculate R_μ the nonlinear dynamic analysis and linear dynamic analysis were carried out. By the use of nonlinear dynamic analysis and try and error on PGA of earthquake time histories, the nonlinear base shear V_y was calculated as described. Then by linear dynamic analysis of the structure under the same time history the maximum linear base shear V_e was calculated; and finally the ductility reduction factor was evaluated (Uang 1991, Mwafy and Elnashai 2002).

6.2 Tentative response modification factor

Figs. 7 to 10 show nonlinear static pushover analysis result in terms of base shear-roof displacement, for different bracing types (diagonal braces, inverted *V*, *V* type and *X*). Fig. 11 shows the result of inter drift story of nonlinear dynamic analysis for 8 stories dual systems with Chevron *V* BRB related to the failure criteria are defined. In Fig. 12, incremental dynamic analysis results are compared with the static pushover curve in terms of roof displacement-base shear for 8 story dual system with chevron *V* brace. These figures show the incremental dynamic analysis, form the upper bond of the static pushover results. In Table 2 the ultimate base shear, V_y, maximum acceleration and limit state resulted from nonlinear dynamic analysis are shown under Tabas, Manjil and Bam events for inverted *V* braced frames. Table 3 shows the maximum elastic base shear, V_e, resulted from linear dynamic analysis under above-mentioned time histories; and it also shows the base shear related to the first hinge plastic that is obtained from pushover analysis to calculate γ according to Eq. (4). Then the average of V_e and V_y for each model is computed and finally, response modification factor, R_w, is computed for the models, based on Eq. (6). As the primary frames were designed based on preliminary response modification factor, and their tentative values were evaluated, a repeat on response modification factor calculation was performed considering the latest values. To calculate final seismic response modification factors, the models were amended and designed based on new response modification factors. Then,

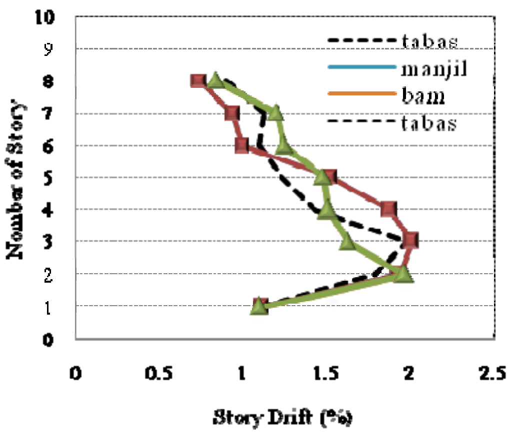


Fig. 11 Drift story related to failure criteria for 8-story dual system with Chevron *V* (BRB)

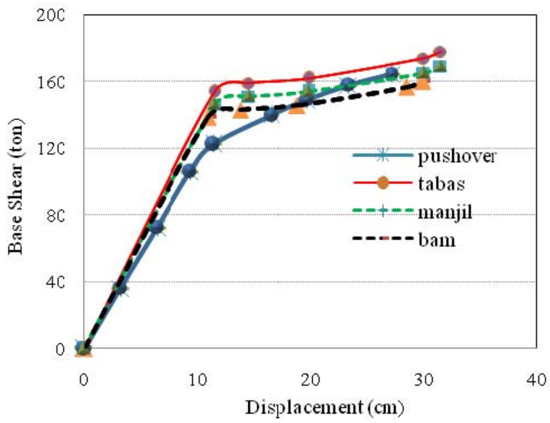


Fig. 12 Comparative of nonlinear dynamic and static push over roof displacement-base shear curve 8 story dual system with chevron *V* (BRB)

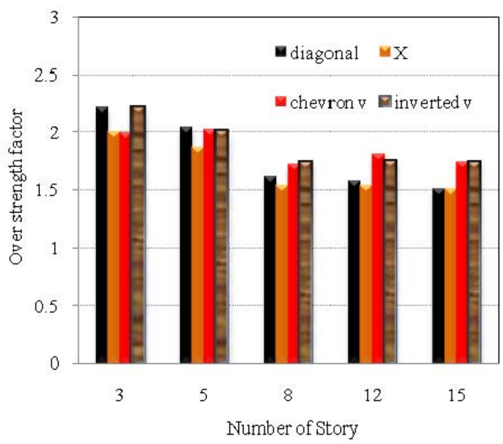


Fig. 13 Overstrength factor of dual systems with buckling resistant brace

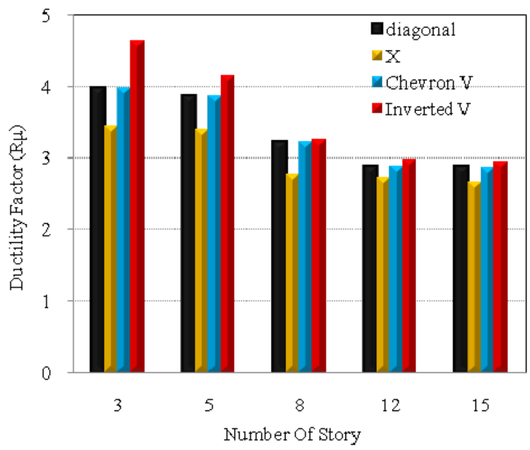


Fig. 14 Ductility reduction factor of dual systems with buckling resistant brace

Table 2 Nonlinear maximum base shear, PGA and extend point for dual system with chevron V brace

No.	Tabas ground motion			Manjil ground motion			Bam ground motion			V_y (avg) (ton)
Story	PGA (g)	Limit state	V_y (ton)	PGA (g)	Limit state	V_y (ton)	PGA (g)	Limit state	V_y (ton)	
3	0.92	Drift 2.5%	75.5	0.87	Drift 2.5%	77.6	0.76	Drift 2.5%	76	76.3
5	0.88	Drift 2.5%	91.4	1.02	Drift 2.5%	81.57	0.82	Drift 2.5%	85.5	86.2
8	1.07	Drift 2%	120.2	1.07	Drift 2%	124	1.12	Drift 2%	117.5	120.6
12	1.12	Drift 2%	174	1.07	Drift 2%	142	1.18	Drift 2%	172	162.7
15	1.12	Drift 2%	177.2	1.12	Drift 2%	181	1.21	Drift 2%	188	182

Table 3 Linear maximum base shear of dual system with chevron V brace and base shear related to first hinge plastic that obtain of dynamic and pushover analysis, respectively

No. story	V_s (ton)	V_e (Tabas (ton))	V_e (Manjil)(ton)	V_e (Bam)(ton)	V_e (avg)(ton)
3	40.3	285	265	302	284
5	47.8	362	318	342	340.6
8	71.36	423	376	358	385.7
12	100.33	616	426	465	502.3
15	115.4	527	614	504	548.3

according to the mentioned procedure, all models were analyzed and their final seismic response modification factors were calculated. Finally, these converged values for allowable stress factor, overstrength factor, ductility factor and response modification factor of dual system with buckling resistant brace are shown in Tables 4 and 5. It can be seen that the overstrength factors, ductility factors and response modification factors decrease as the height of building increases. Response modification factor for different type of bracing configuration is calculated statistically as follows:

Dual system with Diagonal buckling restrained braces, inverted V, Chevron and X, R_w = 10.54, 11.8, 10.7, 8.56, respectively.

It's observed that the response modification factor depends on the type of bracing configuration. Figs. 13 and 14 show the variation of overstrength and ductility factor for different types of bracing configuration. It can be seen that ductility factor decreases more rapidly comparing to overstrength factor as the number of story increases.

6.3 Effect of number of stories on response modification factor

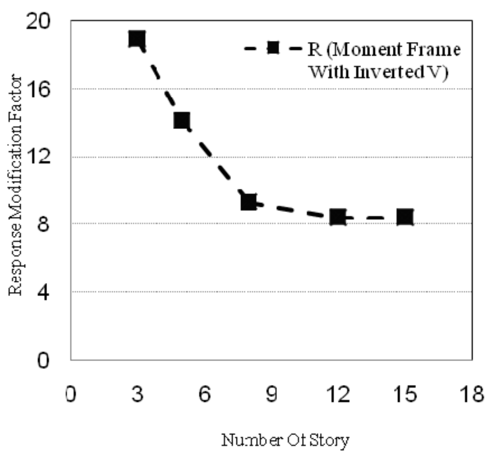
The response modification factor for different types of bracing configuration is presented in Fig. 15. It can be observed that the response modification factor decreases as the height of building increases. This result is apparent in all types of bracing configuration. In dual system with buckling restrained braced with increasing number of stories, the ductility of structure decreases. The decrease in ductility factor causes to decrease the response modification factor. This result is the same as the one that was obtained by Asgarian and Shokrgozar (2009).

Table 4 Left to right, response modification factor of dual system with chevron *V* and inverted *V* brace

No. story	Left					No. story	Right				
	γ	R_{s0}	R_s	R_μ	R_w		γ	R_{s0}	R_s	R_μ	R_w
3	1.92	1.99	2.341	3.97	15.16	3	1.85	1.913	2.21	4.63	18.9
5	1.73	2.015	2.37	3.87	13.4	5	1.7	1.737	2.006	4.14	14.1
8	1.6	1.717	2.02	3.21	8.81	8	1.64	1.509	1.743	3.25	9.27
12	1.58	1.811	2.13	2.88	8.25	12	1.62	1.516	1.751	2.97	8.4
15	1.608	1.743	2.05	2.87	8.05	15	1.64	1.509	1.743	2.94	8.4

Table 5 Left to right, response modification factor of dual system with *X* and Diagonal brace

No. story	Left					No. story	Right				
	γ	R_{s0}	R_s	R_μ	R_w		γ	R_{s0}	R_s	R_μ	R_w
3	1.9	1.9975	2.35	3.45	13.12	3	1.93	1.92	2.21	4.08	17.4
5	1.85	1.87	2.2	3.4	11.77	5	1.84	1.77	2.04	3.49	13.1
8	1.6	1.53	1.8	2.76	6.75	8	1.54	1.4	1.62	3.07	7.64
12	1.57	1.53	1.8	2.71	6.5	12	1.6	1.36	1.57	2.99	7.5
15	1.58	1.5045	1.77	2.66	6.3	15	1.6	1.3	1.504	2.93	7.06



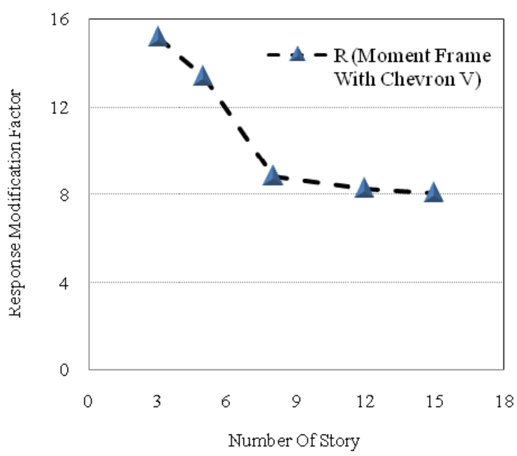


Fig. 15 Continued

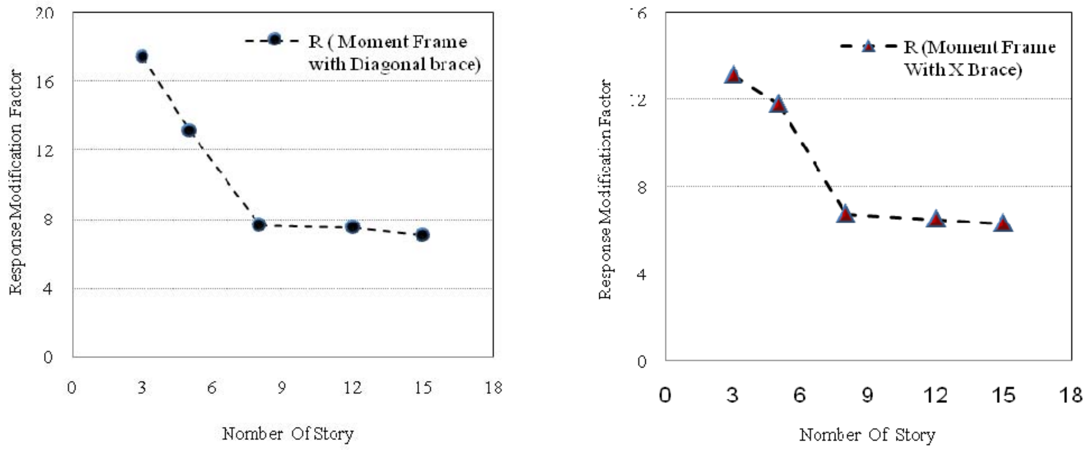


Fig. 15 Number of story-response modification factor

7. Conclusions

The overstrength, ductility and response modification factors of the 20 dual system with buckling-restrained braced with various stories No. and the type of bracing were evaluated by performing static pushover, linear dynamic and incremental nonlinear dynamic analysis. The result of the study can be summarized as follows;

- The obtained allowable stress factor for dual system with buckling restrained braces in type *V*, inverted *V*, *X* and diagonal are, respectively, 1.8, 1.69, 1.7 and 1.702.
- The obtained ductility factor for dual system with buckling restrained braces in type *V*, inverted *V*, *X* and diagonal are, respectively, 3.4, 3.65, 3 and 3.31.
- The obtained overstrength factor for dual system with buckling restrained braces in type *V*, inverted *V*, *X* and diagonal are, respectively, 2.18, 1.9, 2 and 1.8.
- In the general state, the overstrength factor and force reduction factor and response modification factor resulted from buckling restrained braced frames (BRBF) are suggested as 2 and 3.33 and 10.47 respectively.
- The over strength and ductility factors are decreased as the number of stories is increased.

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