Response Modification Factor of Dual Moment-resistant Frame with Buckling Restrained Brace (BRB)

Gh. Abdollahzadeh
Department of Civil Engineering, Faculty of Engineering,
Babol Noshirvani Technology University, Iran

MR. Banihashemi
Ph.D. candidate, Babol Noshirvani Technology University, Iran

S. Elkaee
M. S, Tarbiat Modaress University, Tehran, Iran

M. Esmaeelnia Amiri
Ph.D. candidate, Babol Noshirvani Technology University, Iran

SUMMARY:
In this paper, overstrength, ductility and response modification factor of buckling restrained braced frames (BRBF) dual system were evaluated. To do so, building with various stories and different bracing configuration including diagonal, X, chevron (V and inverted V) bracing were considered. Static pushover analysis, nonlinear incremental dynamic analysis and linear dynamic analysis have been performed using SAP2000 software. The effects of some parameters influencing response modification factor, including the height of the building and the type of bracing system, were investigated. In this paper seismic response modification factor for each of BRBF dual systems has been determined separately and tentative value of 10.4 has been suggested for allowable stress design method.

Keywords: ductility, R factor, buckling resistant brace, nonlinear dynamic analysis, pushover analysis.

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 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. As a matter of fact, many seismic codes permit a reduction in design loads, taking advantage of the fact that the structures possess significant reserve strength (overstrength) and capacity to dissipate energy (ductility), which are incorporated in structural design through a response modification factor. In fact, the response modification factor (R) effects the capability of a structure to dissipate energy through inelastic behavior. The current study intends to characterize important aspects of the hysteretic behavior of different structural systems undergoing inelastic response during severe earthquake incidents. 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). The work lines of CBFs essentially intersect at points [FEMA, 2000]. The steel braces improve the lateral strength and the stiffness by inelastic deformation during an earthquake that leads to seismic energy dissipation [Davaran A, Hoveidae N, 2009]. Studies show that the lateral response of CBFs is mainly dominated by inelastic behavior of bracing members [Annan CD, Youssef MA, 2009]; hence these members are subjected to alternating tension and compression loads once CBFs are exposed to the earthquake loading [Broderick

BM, 2008]. It is through the post-buckling hysteresis behavior of bracing members and upon cyclic loading that the braced frames yield and dissipate energy [Lee K, Bruneau, 2005]. However, the energy dissipation capacity of a steel braced structure is limited due to the buckling of the braces [Kim J, Seo, 2004]. Considering this limitation, efforts have been made to develop new CBF systems with stable hysteretic behavior, significant ductility as well as large energy dissipation capacity. One such CBF system with an improved seismic behavior is the buckling restrained braced frame (BRBF) that enhances not only the energy dissipation capacity of a structure rather 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. (Mahmoudi and Zaree ,2010) indicated that the response modification factor of BRBF were higher than CBF, also they founded that the number of bracing bays and height of buildings have had greater effect on the response modification factors. (Asgarian and Shokrgozar, 2009) used both the pushover and the nonlinear incremental dynamic analyses to evaluate overstrength, ductility and response modification factors of BRBFs with two bracing bays. Considering cyclic behavior of bracing members in life safety structural performance level as suggested by FEMA-356 , 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) and designed in accordance with part 10 of Iranian National Building code, steel structure design (MHUD,2009) and seismic provision of (AISC ,2005). To acquire those behavioral factors, the nonlinear static pushover analyses, nonlinear dynamic analyses, linear dynamic analyses were conducted.

2. BUCKLING RESTRAINED BRACED FRAMES

With respect to the conventional concentrically braced frames, since much of the potential difficulties arise from differences between tensile and compression capacity of the brace as well as the degradation of brace capacity under compressive and cyclic loading, a considerable research has been conducted to develop braces with ideal elasto plastic behavior . The idea of buckling restrained brace (BRB) frames was borne out of need to enhance the compressive capacity of braces without affecting its stronger tensile capacity in order to produce a symmetric hysteretic response. The BRB is composed of a ductile steel core, designed to yield during tension and compression both. To prevent the buckling phenomenon, the steel core is first placed inside a steel casing before it is being filled with mortar or concrete. Prior to mortar casting, an unbinding material or a very small air gap is left over between the core and mortar to minimize or possibly eliminate the transfer of axial force from steel core to mortar and the hollowness of structural section components of BRB (Fig. 1). Thus, the core in BRB under both tension and compression can undergo a considerable yielding, and absorb energy unlike conventional bracing. On the other hand, the basic structural framework in BRBF is designed to remain elastic and all of the seismic damage occurs within the braces. Fig. 2 shows a comparison of a typical hysteresis curve of typical conventional bracing and the buckling restrained bracing.

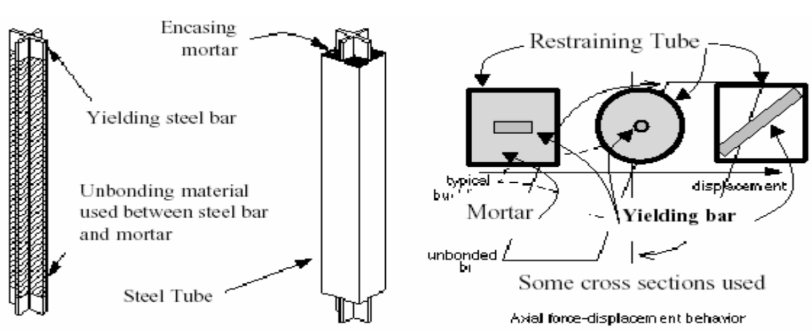

Figure 1. Some schematic details used for buckling restrained braces (Sabelli R, Mahin S, Chang C).

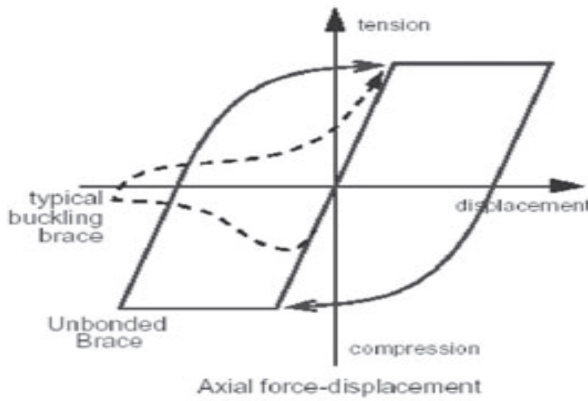

Figure 2. Behavior of conventional brace versus buckling-restrained brace (Kumar GR, Kumar SRS, Kalyanaraman V. (2007)).

3. RESPONSE MODIFICATION FACTOR

Structures elastic analysis under earthquake can create base shear force and stress which are so noticeably bigger than real structure response. The structure can absorb quiet a lot of earthquake energy and resists when it enters the inelastic range of deformation. Oerstrength in structures is related to the fact that the maximum lateral strength of a structure generally exceeds its design strength. Hence, seismic codes reduce design loads, taking advantage of the fact that structures possess overstrength and ductility. In fact the response modification factor includes inelastic performance of structure and indicates over strength and ductility of structure in inelastic stage (FEMA,2005). As it is shown in Fig. 3, usually real nonlinear behavior is idealized by a bilinear elasto perfectly plastic relation. The yield force of structure is shown by V_y and the yield displacement is Δy. In this figure V_e (Vmax) correspond to the elastic response strength of the structure. The maximum base shear in an elasto perfectly behavior is V_y (BHRC,2005). The ratio of maximum base shear considering elastic behavior V_e to maximum base shear in elasto perfectly behavior V_y is called force reduction factor, The overstrength factor is defined as the ratio of maximum base shear in actual behavior V_y to first significant yield strength in structure Vs.

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

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

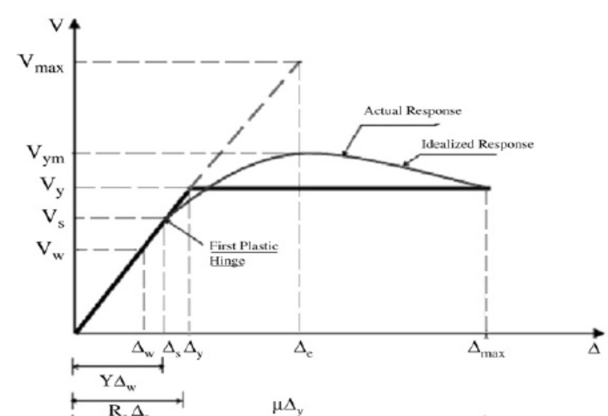

Figure 3. General structure response (Uang CM,1991).

The concept 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 over-strength factor was proposed to establish R for buckling restrained braced frames in SEAOC (Davaran A, Hoveidae N,2009). In this paper over-strength factor of the frames were 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 asR_{S0}, the actual over strength factor Rs which can be used to formulate R should consider the beneficial contribution of some other effects (BHRC,2005):

$$R_s = R_{S0} F_1 F_2 \dots F_n. \tag{3}$$

In this equation, F1 is used to account for difference between actual static yield strength and nominal static yield strength. For structural steel, a statistical study shows that the value of F1 may be taken as 1.05. Parameter F2 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 CM. (1991. In this paper the steel type St-37 was used for all structural members. Parameters F1 and F2 equal to 1.05 and 1.1 were considered taking into 1.155 as material overstrength factor. Other parameters can also be included when reliable data is available. These are included to the parameters such as nonstructural component contributions, variation of lateral force profile. To design for allowable stress method, the design codes decrease design loads from V_s to V_w. This decrease is done by allowable stress factor which is defined as:

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

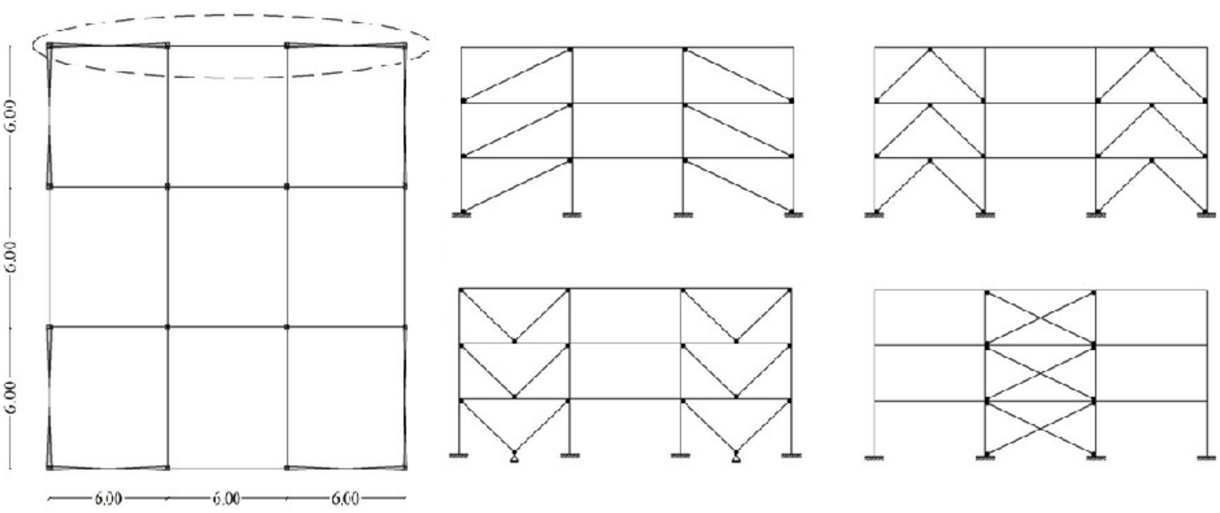

Figure 4. Configuration of Model Structure

The response modification factor, therefore accounts for the ductility and overstrength of the structure and the difference in the level of stresses considered in its design. It is generally expression the following form taking into accounts the above mentioned conceptions,

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

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

Formula (5) is the seismic response modification factor in ultimate strength design method and formula (6) is seismic response modification factor in allowable stress design method (BHRC,2005).

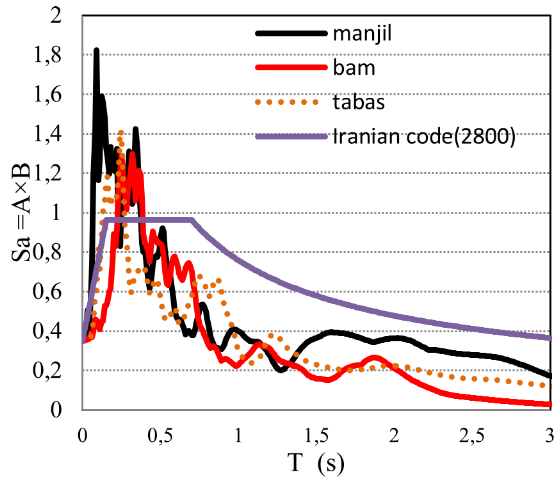

Figure 5. Variation of spectral acceleration with period of structure

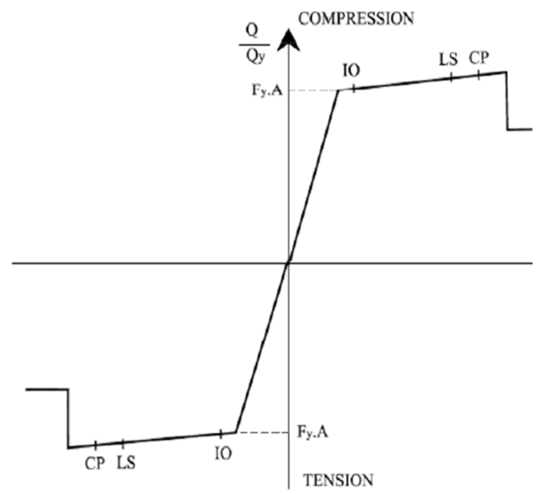

Figure 6. Generalized force-deformation relation for steel brace elements (FEMA-356).

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

4. DESIGN OF MODEL STRUCTURES

To evaluate the overstrength, ductility, and the response modification factors of buckling restrained braced frames 3, 5, 8, 12 and 15 story building with the bay length of 6 m and 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(Annan CD, Youssef MA, El Naggar MH,2009) and Iranian National Building Code, part 10, steel structure design (MHUD,2009). Fig.4. show the typical configuration of the models used. The story height of the models was considered as 3 m. 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 Iranian Earthquake Code (Standard No. 2800).(BHRC,2005). The dead and live loads of 600 and 200Kg/m2, respectively, were used for gravity load.
The design base shear was computed as follows:

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

In which V is 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 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 were considered for frame design. In designing dual system, all

beam to column connections were assumed to be rigid at both ends as frames were designed to be moment resisting. The moment frames were also designed to sustain 25% of the lateral load and the Braces were designed to sustain 100 percent of the lateral load also the braces were assumed to be pinned at both of the ends. Allowable stress design method was used to design frame members in accordance to part 10 of Iranian national code. To ensure that vertical bracing columns have enough strength to resist the force transferred from bracing elements; Iranian Standard No. 2800 (Annan CD, Youssef MA, El Naggar MH,2009) has instruction to design vertical bracing columns for the following load combinations:
(a) Axial compression according to:

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

(b) Axial tension according to:

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

In which Fa is allowable compressive stress, F_y is the yield stress, A is 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 design tensile and compression strength of column, respectively (Annan CD, Youssef MA, El Naggar MH,2009).

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 (MHUD,2009).. For modeling of the members in nonlinear range of deformation, following assumptions were assumed. In dual system all of the frame member, beam and column were considered as rigid-ended but the braces were considered as pinned-ended. For design member the w section and plate section were considered for (beam, column) and braces, respectively. To evaluate behavior factors, the nonlinear static analyses (pushover), nonlinear dynamic analyses and linear dynamic have been done. There for to do these analyses the nonlinear behavior of members suggested by FEMA-356(FEMA,2005). For buckling restrained braces, the model presented in Tables 5-7of FEMA-356 (FEMA,2005) 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, Qy 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 5ΔC (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 7ΔT (ΔT is the axial deformation at expected tensile yielding load). For determining R-factor and it's components (overstrength, reduction ductility) we have to stop the nonlinear analyses, there for the failure criteria was selected based on Iranian standard code No.2800 that explain in continues.

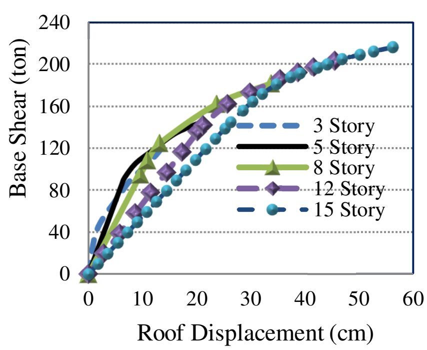

Figure 7. Roof displacement-base shear curve for Dual System that have diagonal brace

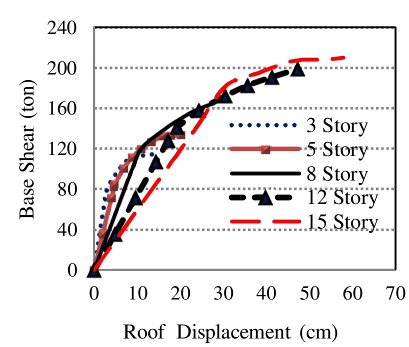

Figure 8. Roof displacement-base shear curve for Dual System that have inverted V brace.

6. RESPONSE MODIFICATION FACTOR

In this paper, two factors Rs and Rμ have been calculated as followed

6.1 Overstrength Factor (Rs)

To calculateV_y, the Incremental Nonlinear Dynamic Analysis of the models subjected to strong ground motions match with the design spectrum was carried out. The response spectra and the design spectrum are shown in Fig. 6. In these analysis by the use of time history of tabas, manjil and bam earthquake (Table 1), their PGA's with several try and errors had changed in a way that the gained time history resulted in the structure reaching to a one of following failure criteria. The maximum nonlinear base shear of this time history is the inelastic base shear of structure (AISC,2005). To gain the base shear related to the first plastic hinge formation in structure Vs, the pushover analysis was carried out by 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 has 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 was selected based on the Iranian Standard Code No. 2800 as follows (AISC,2005)

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

$$\Delta M < 0.025H \tag{10}$$

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

$$\Delta M < 0.02H \tag{11}$$

In which `H' is story height.

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 per 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 was not occur, the nonlinear dynamic analysis was stopped and the last scaled earthquake base shear was selected as ultimate limit state.

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

No. story	Tabas ground motion			Manjil ground motion			Bam ground motion			Vy(avg(Ton))
	PGA(g)	limit state	Vy (Ton)	PGA(g)	limit state	Vy(Ton)	PGA(g)	limit state	Vy(Ton)	
3	0.92	Drift2.5%	75.5	0.87	Drift2.5%	77.6	0.76	Drift2.5%	76	76.3
5	0.88	Drift2.5%	91.4	1.02	Drift2.5%	81.57	0.82	Drift2.5%	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 analyses, respectively

No.Story	Vs(Ton)	Ve (tabas (Ton))	Ve(manjil)(Ton)	Ve (bam)(Ton)	Ve(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

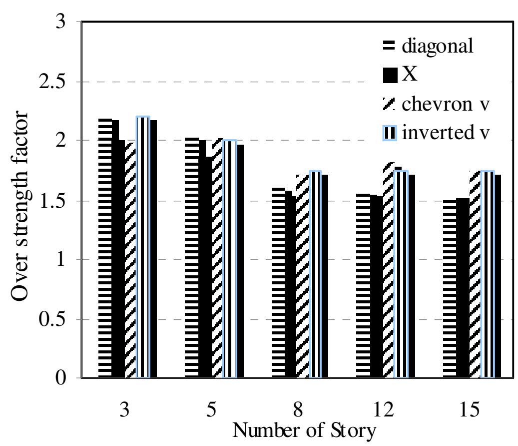

Figure 11. Overstrength factor of dual system with buckling resistant brace

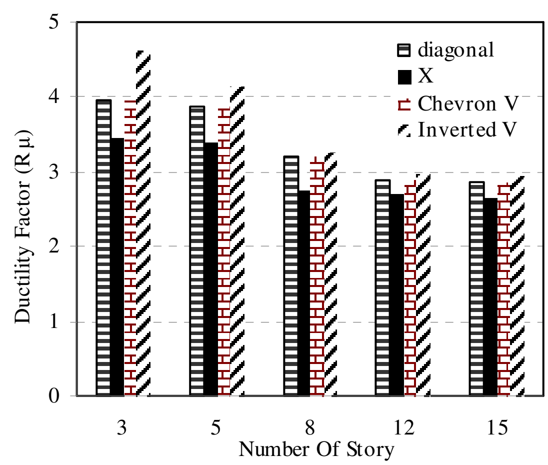

Figure 12. Ductility reduction factor of dual system with buckling resistant brace

6.2 R_{μ} Calculation

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 (AISC. BHRC,2005).

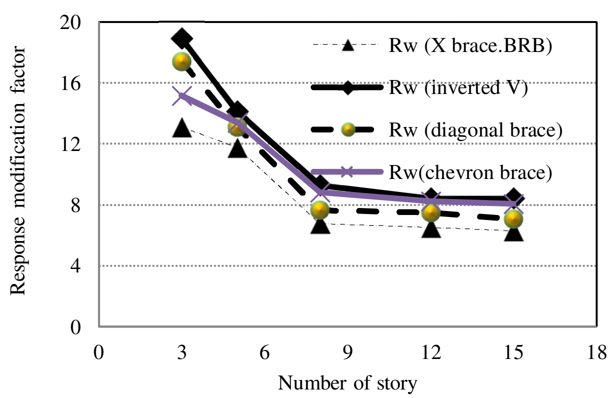

Figure 13. Number of story-Response modification factor

6.3 Tentative Response Modification Factor

Figs. 7 through 8 show nonlinear static pushover analysis result in terms of base shear-roof displacement for diagonal braces, inverted V. Fig. 9 shows the result of drift story of nonlinear dynamic analysis for 8 story dual system with Chevron V BRB in related to failure criteria were defined. In Figs. 10, incremental dynamic analysis results were compared with the static pushover curve in terms of roof displacement-base shear for 8 story dual system with chevron V brace. This figures shows that the incremental dynamic analysis form 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 also shows maximum elastic base shear, V_e, resulted from linear dynamic analysis under above-mentioned time histories, also to calculate γ according to equation.4, this table shows the base shear related to first hinge plastic that obtain of pushover analyses . Finally, in Tables 4 and 5 allowable stress factor overstrength factor, ductility factor and response modification factor of dual system with buckling resistant brace are shown. 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 was calculated statistically as followings: dual system with Diagonal buckling restrained braces, inverted V, Chevron and X, R_w= 10.54, 11.8, 10.7, 8.56 respectively.

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

No.Story	γ	Rs0	Rs	Rμ	Rw	No.Story	γ	Rs0	Rs	Rμ	Rw
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	γ	Rs0	Rs	Rμ	Rw	No.Story	γ	Rs0	Rs	Rμ	Rw
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

It was observed that the response modification factor depends on the type of bracing configuration. Fig. 13 and 14 show variation of overstrength and ductility factor for difference type of bracing configuration. It can be seen that ductility factor decreases more rapidly compare to overstrength factor as the number of story increases.

6.4. Effect of number of stories on response modification factor

The response modification factor for different type of bracing configuration, presented in Fig. 13. It can be observed that the response modification factor decreases as the height of building increases. This result was apparent in all type of bracing configuration. In dual system with buckling restrained braced with increasing number of stories the ductility of structure decrease. The decrease in ductility factor causes to decrease the response modification factor.

7. CONCLUSION

The overstrength, ductility and response modification factors of the 20 dual system with buckling-restrained braced with various stories No. and type of bracing were evaluated by performing static pushover, linear dynamic and incremental nonlinear dynamic analysis. 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.
Codes give constant value of response modification factors for dual system with BRB. However, the response modification factors, evaluated in this study, have different values for brace con-figuration types, number of story height. Consequently, results indicate that the response modification factors proposed in seismic codes need to be modified for dual system with BRB In the general state, the overstrength factor and force reduction factor resulted from ductility for buckling restrained braced frames (BRBF) are suggested as 2 and 3.33, respectively.
Response modification factor for dual system with buckling restrained braced (BRB) are suggested as 10.47 for allowable stress method.
The over strength and ductility factors are decreased as the number of stories is increased

REFERENCES

AISC (2005).. Seismic provisions for structural steel buildings. Chicago (IL): American Institute of Steel Constructions;

Annan CD, Youssef MA, El Naggar MH.(2009), Experimental evaluation of the seismic performance of modular steel-braced frames, Journal of Engineering Structures; 31(7):1435-46.

Asgarian B, ShokrgozarHR,(2009),BRBF response modification factor. Journal of Constructional Steel Research;65(2):290-8.

Broderick BM, Elghazouli AY, Goggins J,(2008),Earthquake testing and response analysis of concentrically-braced sub-frames. Journal of Constructional Steel Research; 64(9):997-1007.

BHRC(2005) . Iranian code of practice for seismic resistance design of buildings:(2005) ;Standard no. 2800. 3rd ed. Building and Housing Research Center.

Davaran A, Hoveidae N, (2009),Effect of mid-connection detail on the behavior of X- bracing systems. Journal of Constructional Steel Research: 65(4):985-90.

FEMA. (2000). Prestandard and commentary for the seismic rehabilitation of building. FEMA-356. Federal Emergency Management Agency, Washington, DC;

Lee K, Bruneau M,(2005) ,Energy dissipation of compression members in concentrically braced frames: review of experimental data. Journal of the Structural Engineering; 131(4):552-9.

Mahmoudi M, Zaree M, (2010) Evaluating response modification factors of concentrically braced steel frames Journal of Constructional Steel Research; 66: 1196-1204

MHUD. (2009) Iranian national building code (part 10): steel structure design, Tehran (Iran). Ministry of Housing and Urban Development;

Mazzolani FM, Piluso V (1996),Theory and design of seismic resistant steel frames, E& FN Spon, London,

Kim J, Seo Y,(2004) Seismic design of low-rise steel frames with buckling-restrained braces. Journal of Engineering Structures; 26(5):543-51.

Kiggins S, Uang CM.(2006)Reducing residual drift of buckling-restrained braced frames as a dual system. Journal of Engineering Structures; 28(11): 1525-32.

Kumar GR, Kumar SRS, Kalyanaraman V. .(2007); Behaviour of frames with non buckling bracings under earthquake loading. Journal of Constructional Steel Research;63(2):254-62.

Rahai AR, Alinia MM.(2008),Performance evaluation and strengthening of concrete structures with composite bracing members. Journal of Construction & Building Materials;22(10):2100-10.

Sabelli R, Mahin S, Chang C. Seismic demands on steel braced frame buildings with buckling-restrained braces. Journal of Engineering Structures 2003; 25(5):655-66.

Uang CM. (1991);Establishing R (or Rw) and Cd factor for building seismic provision.Journal of Structure ngineering;117(1):19-28