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Determination the Response Modification Factors of Buckling Restrained Braced Frames

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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. Relying on this, many seismic design codes led to reduce loads. The present paper tries to evaluate the response modification factors of buckling restrained braced frames (BRBFs) utilized for rehabilitation of steel frames. Since, the response modification factor depends on ductility and overstrength, the static nonlinear analysis has been performed on building models including single and double bracing bays, multi-floors and different brace configurations (chevron V, invert V). The BRBFs values for factors such as ductility, overstrength, force reduction due to ductility and response modification have been assessed for all the buildings. The results showed that the response modification factors for BRBFs have high values. It was found that the number of bracing bays and height of buildings have had greater effect on the response modification factors.

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Keywords: buckling restrained braced frames; ductility factor; overstrength factor; response modification factor.

1. Introduction

Normally, the preliminary design in most of the buildings is based on equivalent static forces specified by the governing building codes. The height wise distribution of these static forces 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 Moghaddam et al. (2005). As a matter of fact,

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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 Kim and Choi (2005). In fact, the response modification factor (R) reflects 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). Studies show that the lateral response of CBFs is mainly dominated by inelastic behavior of bracing members Annan et al. (2009). Hence; 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, 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 an structure rather decreases the demand for inelastic deformation of the main structural members.

The response modification factors of BRBFs have been the subjects of investigations by various researchers. Sabelli et al (2003) have presented a series of models with chevron BRBs which designed and analyzed once subjected to tremors representing various seismic hazard levels. They found that the BRBF response was not sensitive to R factors in the range of 6 to 8. Kiggins and Uang (2006) found that the buckling restrained brace with steel moment frames not only reduces residual story drifts rather leads 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 generates rigid structures but the BRB system produces a concurrent suitable rigidity, ductility and maximum overstrength factors for structures hence; 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 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 (2000), the current paper intends to evaluate and compare the overstrength, ductility and response modification factors of BRBFs. 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 were conducted.

2. Buckling Restrained Braced Frames

In order to resist lateral forces due to wind and earthquakes, CBFs are considered to be one the most efficient structural systems in steel construction. In regular buildings, bracing members are expected to buckle in compression and yield in tension once subjected to a reverse cyclic loading. The response of CBFs to significant earthquake loading depends strongly on the asymmetric axial resistance of the bracing members Broderick et al. (2008), which has a complex cyclic inelastic behavior (Fig.1). The idea of buckling restrained brace (BRB) frames, utilized for rehabilitation of steel 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.2). 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 Sabelli et al. (2003). Fig.1 shows a comparison of a typical hysteresis curve of typical conventional bracing and the buckling restrained bracing.

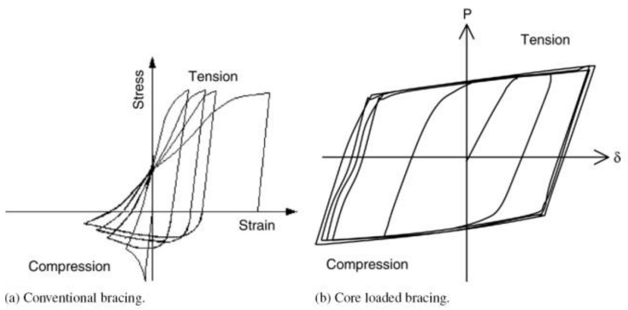

Figure 1. Difference in energy dissipation between conventional bracing and buckling restrained bracing under cyclic loading

3. Response Modification Factor

In the force-based seismic design procedures, the response modification factor (R) is the one used to reduce the linear elastic response spectra to the inelastic ones. In other words, response modification factor is the ratio of strength required to maintain the structural elasticity. Fig.3 represents the base shear versus roof displacement relation of a structure, which can be developed by a nonlinear static analysis. In this figure real nonlinear behavior is idealized by a bilinear elasto-plastic relation. The response modification factor is determined as follows:

$$R = R_{\mu} . R_S \qquad (1)$$

Where, Rμ is a reduction factor due to ductility and RS is the overstrength factor.

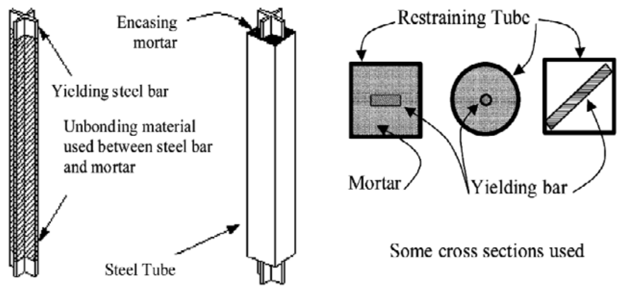

Figure 2. Some schematic details used for buckling restrained braces

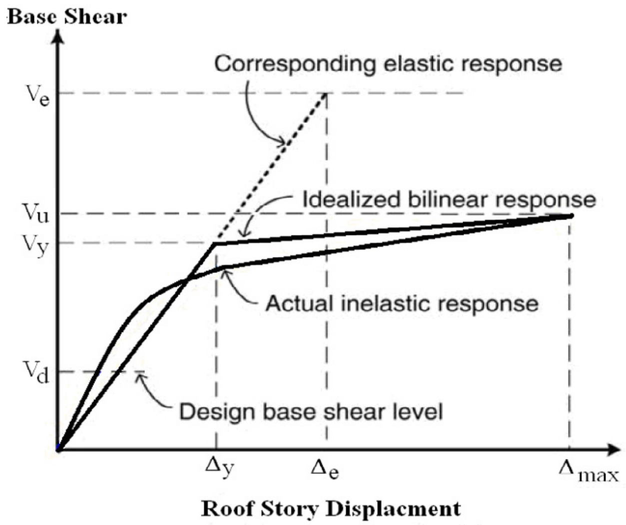

Figure 3. General structure response

3.1. Reduction factor due to ductility

Rμ is a parameter that measures the global nonlinear response of a structure, due to the hysteretic energy. Several proposals have been put forward for Rμ. In a simple version of the N2 method proposed by Fajfar (2002), the reduction factor Rμ is written as:

$$R_\mu = (\mu - 1)\frac{T}{T_c} + 1(T < T_c)$$

$$R_\mu = \mu\,(T \geq T_c) \tag{2}$$

Where, T is the fundamental period, T_C is the characteristic of ground motion equal to 0.5 for the soil type II that has been considered here based on the Iranian Earthquake Resistance Design Code Standard No. 2800 (BHRC 2005) and μ is the structural ductility factor defined as:

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \tag{3}$$

Where, Δ_{max} is the maximum displacement for the first life safety performance in structure and Δ_y is the yield displacement observed there.

3.2. Overstrength factor

As observed in some of the intermittent quake incidents, it seems building structures could take the forces considerably larger than those were designed for. The presence of significant reserve strength that was not accounted in design, explains this phenomenon. Overstrength helps structures stand safely not only against sever tremors but reduces the elastic strength demand, as well. This object is performed using the force reduction factor (Mahmoudi 2003). The design overstrength factor (R_{sd}) is defined as follows:

$$R_{Sd} = \frac{V_u}{V_d} \tag{4}$$

Here, V_d is the design base shear in the building and Vu is the base shear in relevance to the first life safety performance in structural members (Fig.3). In this equation, the overstrength factor is based on the applied nominal material properties. Meanwhile, the actual overstrength factor should consider the help of some other effects (Asgarian and Shojrgozar 2009):

$$R_S = R_{Sd}.R_1.R_2 \dots \tag{5}$$

In the Eq.5, R_1 accounts for the difference between actual and nominal static yield strengths. Based on a statistical study, for structural steel, the value of R_1 may be put as 1.05 (Asgarian and Shojrgozar 2009). During an earthquake phenomena, parameter R_2 may be used to know the yield stress under the strain rate effect. For that matter, to account for the strain rate effect, a value of 1.1 (an increased of 10%) could be used (Asgarian and Shojrgozar 2009). The current study has used steel type St-37 for all structural members. It considers parameters R_1 and R_2 equal to 1.05 and 1.1, taking into account R_{sm}=1.155 as material overstrength factor. Other parameters such as nonstructural component contributions, variation of lateral force profile could be included once a reliable data is available.

4. Structural Models and Analysis

Use To assess the overstrength factor, reduction factors due to ductility, and the response modification factors, some 20 BRBFs with 3, 5, 7, 10 and 12 stories as well as a bay of 5 m long were selected. For BRBFs two bracing types (chevron V and chevron-inverted V) were considered. The height of every model structure was fixed to 3.2 m. Fig.4 shows the plan of the model structures, and the type of braces located in single and double bays.

The dead and live loads of 5.5 and 2 KN/m2, respectively, were used for gravity. To compute the seismic design base shear, parameters such as importance factor of I =1, seismic zone factor of A = 0.35, soil type II considered based on the Iranian Earthquake

Resistance Design Code BHRC, (2005) and the response modification factor R=8 was used based on the seismic provision of AISC. All beam to column connections were assumed to be pinned at both ends as frames were not designed to be moment resisting. The braces were also designed to sustain 100 percent of the lateral load.

The models were designed keeping in view the Iranian Earthquake Resistance Design Code Standard No. 2800 (BHRC 2005), Iranian National Building Code, part 10, steel structure design (MHUD 2009) and The buckling restrained braces were designed by seismic provision of AISC (2005). Table1 indicates detains about the structural members selected for the seven story model frame with invert V BRB.

To evaluate behavioral factors, nonlinear static (pushover) analysis is performed by subjecting a structure to monotonically increasing lateral forces with an invariant height-wise distribution. For that purpose, SNAP-2DX (Rai et al 1996) program is used. The analysis was conducted using life safety structural performance level for both tension and compression brace behavior presented in Table 5–7 of FEMA-356 (2000). In Fig.5, Q, Qy and Δ are the generalized component load, expected strength and component displacement, respectively. The post-yield stiffness of beams, columns and braces was initially assumed to be 2%.

5. Results

Figs.6 and 7 show the nonlinear static pushover analysis results for BRBFs type (inverted V and chevron V type) for single and double bracing bays. Figs.8 shows variation in response modification factor for different type of BRBFs configuration.

Tables 2 and 3 show the overstrength factor (R_S), reduction factor due to ductility (R_μ) and response modification factor (R) of BRBFs.

According to Figs.6 and 7, initial stiffness of BRBFs remains the same with the increase in the height of the building.

6. Conclusion

Paper has evaluated the factors such as overstrength, reduction due to ductility, and the response modification of 20 BRBFs considering life safety structural performance levels. As such, a static nonlinear (pushover) analysis was performed on the model buildings with single and double 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.

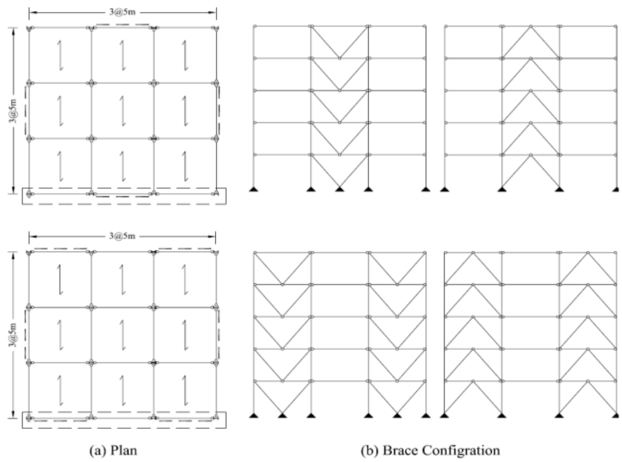

Figure 4. Configuration of model structures

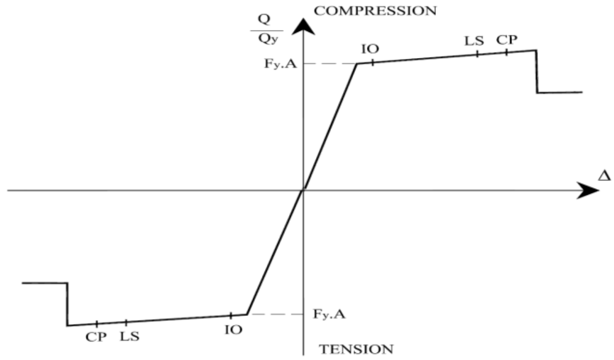

Figure 5. Generalized force-deformation relation for BRB elements (FEMA-356)

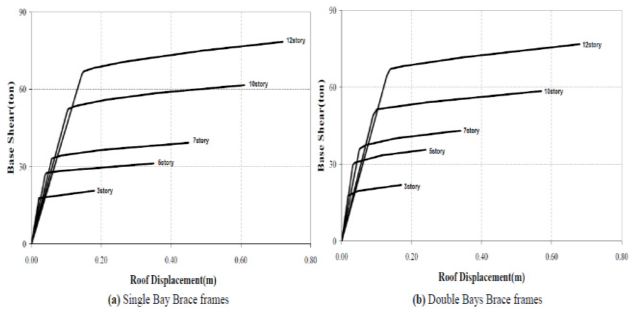

Figure 6. Roof displacement-base shear curve for buckling restrained invert-V brace

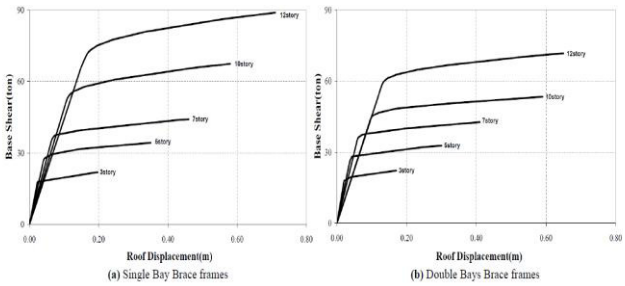

Figure 7. Roof displacement-base shear curve for buckling restrained V brace

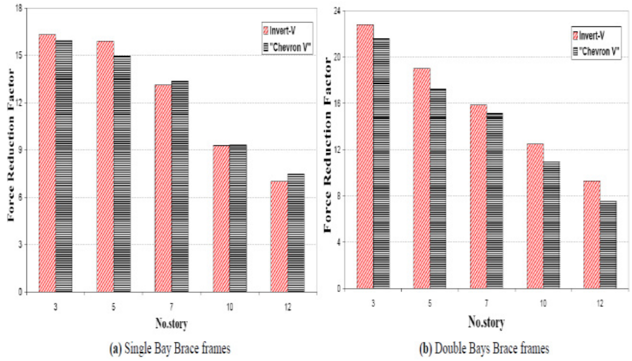

Figure 8. Response modification factors for BRBFs

The results of this study can be summarized as follows:

(1) It was observed that the overstrength and response modification factors of BRBFs decrease with an increase in the height of buildings. However, the reduction factors due to ductility have varied quantity for different numbers of stories. Also, the overstrength and response modification factors increase with an increase in the number of bracing bays but there is no obvious variation on the reduction factors due to ductility.

(2) In BRBFs, because of brace energy dissipating capacity in tension and compression (Fig.5), the maximum roof displacement (Δ_{max}), and reduction factors due to ductility have higher values that cause these parameters to have main effect on response modification factors.

(3) The overstrength factors for different types of BRBFs with single and double bracing bay are 1.90 and 2.40, respectively. The type of brace configuration in BRBFs has no effect on overstrength factors.

(4) The obtained reduction factors due to ductility for different type of BRBFs with single and double bracing bays are varies between 4.7 and 8 for.

(6) The obtained response modification factors for different type of BRBFs with single bracing bay varies between 7 and 16 and for double bracing bays between 8 and 22. In BRBFs, to calculate final seismic response modification factors the models were designed and analyzed again to observe convergence on their final seismic response modification factors.

(7) Codes give constant value of response modification factors for BRBFs. However, the response modification factors, evaluated in this study, have different values for brace configuration types, number of bracing bays and buildings height. Consequently, results indicate that the response modification factors proposed in seismic codes need to be modified for BRBFs.

Table 1. Sectional properties of seven story model structures with single bay invert V BRB

Story	Interior col.	Exterior col.	Buckling restrained brace	Beam
1	Box150×150×10	Box150×150×10	PL50×15	IPE360
2	Box150×150×10	Box150×150×10	PL50×15	IPE360
3	Box150×150×10	Box150×150×10	PL50×18	IPE360
4	Box250×250×15	Box150×150×10	PL50×20	IPE360
5	Box250×250×15	Box150×150×10	PL50×20	IPE360
6	Box300×300×20	Box150×150×10	PL50×20	IPE360
7	Box300×300×20	Box150×150×10	PL60×20	IPE360

Table 2. Response modification factor of BRBFs that have chevron invert-V brace

No. story	Single bay brace frame					Double bays brace frame				
	RSd	RSm	RS	Rμ	R	RSd	RSm	RS	Rμ	R
3	2.09	1.155	2.41	6.76	16.30	2.96	1.155	3.41	6.68	22.85
5	1.54	1.155	1.78	8.94	15.90	2.25	1.155	2.60	7.31	19.00
7	1.50	1.155	1.74	7.54	3.12	2.04	1.155	2.36	6.72	15.89
10	1.35	1.155	1.57	5.91	9.27	1.73	1.155	2.00	6.25	12.51
12	1.21	1.155	1.40	5.02	7.02	1.54	1.155	1.78	5.25	9.34

Table 3. Response modification factor of BRBFs that have chevron V brace

No. story	Single bay brace frame					Double bays brace frame				
	RSd	RSm	RS	Rμ	R	RSd	RSm	RS	Rμ	R
3	2.19	1.155	2.53	6.30	15.93	2.93	1.155	3.39	6.38	21.62
5	1.60	1.155	1.85	8.07	14.96	2.10	1.155	2.43	7.09	17.23
7	1.59	1.155	1.84	7.25	13.34	1.90	1.155	2.19	6.91	15.13
10	1.47	1.155	1.70	5.49	9.33	1.52	1.155	1.76	6.24	10.98
12	1.36	1.155	1.58	4.75	7.50	1.28	1.155	1.48	5.11	7.56

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