Earthquakes and Structures, *Vol. 8, No. 6 (2015) 1349-1362*
DOI: http://dx.doi.org/10.12989/eas.2015.8.6.1349

Nonlinear dynamic response of reinforced concrete building retrofitted with buckling restrained braces

Esra Mete Güneyisi*, Osman Tunca and Ibrahim Azez

Department of Civil Engineering, Gaziantep University, 27310, Gaziantep, Turkey

(Received August 16, 2013, Revised September 23, 2014, Accepted December 16, 2014)

Abstract. This paper presents an analytical study aimed at evaluating the effectiveness of using buckling-restrained braces (BRBs) in mitigating the seismic response of a case study 6 storey reinforced concrete (RC) building. In the design of the BRBs with non-prismatic cross-sections, twelve combinations of α and β design parameters that influence the strength and stiffness of the BRBs, respectively, were considered. The response of the structure with and without BRBs under earthquake ground accelerations were evaluated through nonlinear dynamic analysis. Two sets of ground motions representative of the design earthquake with 10% and 50% exceedance probability in fifty years were taken into account. By comparing the structural performance of the original and buckling restrained braced structures, it was observed that the use of the BRBs were very effective in mitigating the seismic response as a retrofit scheme. However, the selection of the strength and stiffness parameters of the BRBs had considerable effect on the response characteristics of RC structures. For instance, by increasing the value of α and by decreasing the value of β of the buckling-restrained braces, the maximum deformation demand of the structures increased.

Keywords: buckling restrained brace; nonlinear dynamic analysis; reinforced concrete building; seismic response

1. Introduction

In the last few decades, there have been several studies on innovative approaches in order to better protect or strengthen the structures under the effect of external dynamic forces. The idea behind these innovative approaches which especially focus on the materials and systems is to limit the inelastic deformations in other structural members by dissipating the energy in itself (Symans *et al*. 2008, Housner *et al*. 1997, Soong *et al*. 1997, Soong *et al*. 2002, Bergami and Nuti 2013, Karalis and Stylianidisa 2013, Güneyisi and Şahin 2014). In this study, among these innovative approaches, buckling-restrained braces (BRBs), which have the advantages of low cost, ease of production, and installation (Kanaji *et al*. 2003, Farhat *et al*. 2009), were investigated.

In buckling restrained braces, as shown in Fig. 1 (Clark *et al*. 2000), there is a core that provides the axial strength to dissipate energy and a restraining section that provides the flexural rigidity to avoid buckling. An insert filler material such as infill concrete, mortar or grout is used to fill the space between the core and restraining section. In order to prevent excessive shear stress

*Corresponding author, Associate Professor, E-mail: eguneyisi@gantep.edu.tr

http://www.techno-press.org/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

transfer that may occur when the brace is under compression, an unbonding material is also placed between the core and insert filler material. In BRBs, the aim is to obtain a yielding core that can deform longitudinally under tension and compression forces independent from buckling. Since lateral and local buckling behaviour is restricted, high cyclic ductility is attainable (Sabelli *et al.* 2003). Based on these design approaches, the buckling-restrained braces provides stable hysteresis behaviour having approximately the same axial yield force in tension and compression, as given in Fig. 2 (Clark *et al.* 2000). In the last few decades, many researchers have investigated the hysteresis behaviour of different designs of buckling restrained braces (Sabelli *et al.* 2003, Watanabe *et al.* 1988, Wada *et al.* 1998, Merritt *et al.* 2003a, b, Koetaka *et al.* 2006, Iwata and Murai 2006, Usami *et al.* 2008, Tsai *et al.* 2008).

Even though the use of buckling restrained braces is desirable for seismic design of new buildings for their higher ductile behaviour; in the last decade its use for strengthening and rehabilitation of existing structures gain considerable attention (Ash and Bartoletti 2009, Di Sarno and Manfredi 2010, Güneyisi 2012, Güneyisi and Ameen 2014, Della-Corte *et al.* 2015). Within this context, the aim of this analytical study is to introduce buckling restrained braces designed with different stiffness and yield load as a seismic upgrading scheme for damage prevention capacity and to evaluate the upgrading effectiveness of using them in a reinforced concrete structure.

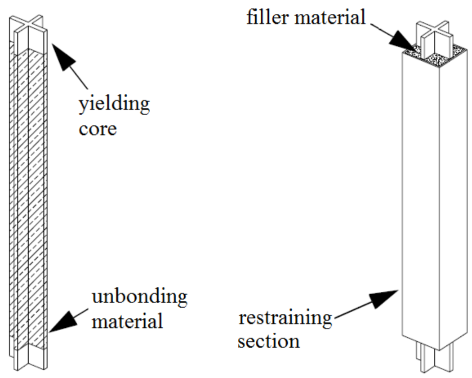


Fig. 1 Schematic view of a buckling resistant unbounded brace (Clark *et al.* 2000)

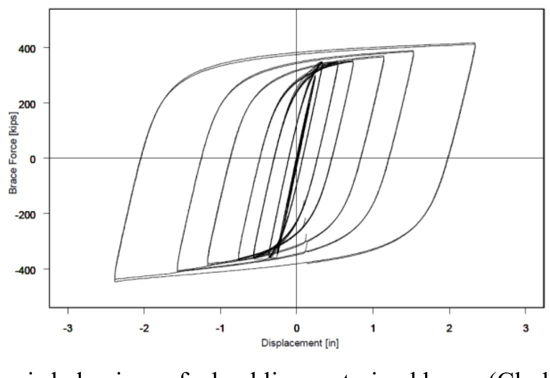


Fig. 2 Hysteresis behaviour of a buckling restrained brace (Clark *et al.* 2000)

2. Description of sample structures

2.1 Original frame

An exterior frame of a six storey reinforced concrete (RC) building was selected in order to compare the seismic response of the original structure with the updated seismic response of the structure after the addition of buckling restrained braces with different design properties. A building which consists of four bays in each direction and which is regular in shape and symmetric in plane was chosen in order to carry out the analysis on two-dimensional models which ease the interpretation of the results of analysis. Typical floor plan and elevation view of the selected building were given in Fig. 3. The material properties such as the uniaxial compressive strength of concrete and the yield strength of both longitudinal and lateral reinforcement were assumed to be 16 MPa and 220 MPa, respectively. The dimensions of the columns varied with the storey height, as shown in Table 1 while all the beams had the same cross-sectional properties as 250×500 mm. For these structural members, the reinforcement details were given in Table 1. Moreover, for the potential plastic hinge zones, the spacing of the lateral reinforcement was taken as 150 mm. In modeling of the frame, as gravity loads, dead load and live load were taken into consideration. In the calculation of the dead load, weight of the structural members and weight of the masonry infill walls were included. The live load was taken as 2.0 kN/m^2, which is typical for residential buildings.

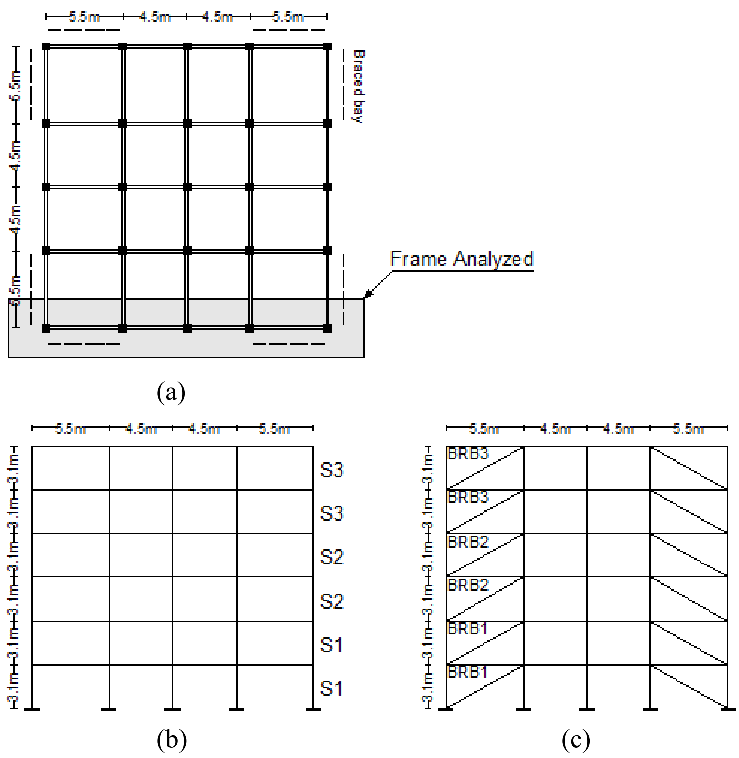


Fig. 3 A layout for (a) a floor plan and (b) elevation of the original frame, and (c) elevation of the buckling restrained braced frames

Table 1 Properties of columns in the original frame and total core area for buckling restrained braces in BRBFs

Storey Level	Original Frame			Buckling Restrained Braced Frame	
	Column	Dimensions (mm)	Reinforcement	Buckling Restrained Brace	Total Core Area (mm^2)
1	S1	400×400	8ϕ16	BRB1	600
2	S1	400×400	8ϕ16	BRB1	600
3	S2	350×350	8ϕ14	BRB2	450
4	S2	350×350	8ϕ14	BRB2	450
5	S3	300×300	4ϕ16	BRB3	200
6	S3	300×300	4ϕ16	BRB3	200

The analytical model of the frames including nonlinear properties of the structural members was obtained by using SAP 2000 Nonlinear version 14.0 which is a general purpose structural analysis program (CSI 2011). Lumped plasticity approach was utilized and the nonlinearity was taken into account by adopting plastic hinges with hysteretic relationships based on FEMA-356 (FEMA 356) to each end of the beam and column members. For the column members, axial force and biaxial moment hinges (PMM) and for the beams flexural moment hinges (M3) were considered.

2.2 Buckling restrained braced frames

In this study, buckling restrained braces with different design properties were utilized as a means to update the seismic response of the original structure, and the effectiveness of the buckling restrained braces were investigated, comparatively.

In the study of Kalyanaraman *et al.* (1998), it is shown that for the buckling restrained braces having non-prismatic cores, by changing the reduced area of the core and by changing the length of the core with reduced area, it is possible to obtain wide range of stiffness and strength for the buckling restrained braces. They also showed that by using this property, it is possible to attain the desired inter-storey drift and energy absorption in the structures. Furthermore, in the study of Kumar *et al.* (2007), the parameters related to hysteretic behaviour of the buckling restrained braces such as the stiffness in the elastic and post-elastic range, are given in terms of the reduced area of the core and the length of the core with reduced area. The ratio of the reduced area of the core to the total area of the section is denoted with α ($\alpha = A_{reduced}/A_{total}$), and the ratio of the length of the core with reduced area, to the total length of the brace is denoted with β. The hysteretic behaviour of the buckling restrained brace shown in Fig. 4 is defined in terms of α and β by using Eqs. (1) and (2) (Kumar *et al.* 2007)

$$k_1 = \frac{AE}{l\left(\frac{\beta}{\alpha}+(1-\beta)\right)} \text{ for } \delta < \delta_y \tag{1}$$

$$k_2 = \frac{AE}{l\left(\frac{\beta}{\alpha\frac{E_t}{E}+(1-\beta)}\right)} \text{ for } \delta \geq \delta_y \tag{2}$$

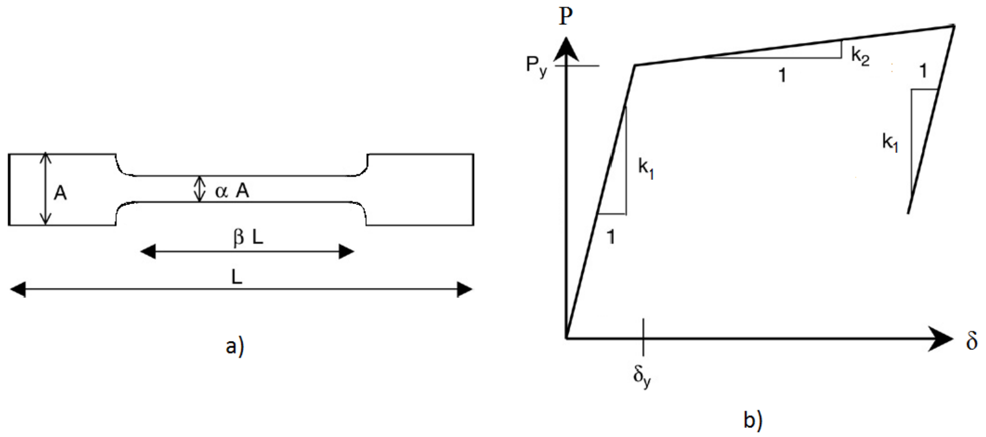


Fig. 4 (a) Core element and (b) hysteretic behaviour of buckling restrained braces (Kumar *et al.* 2007)

Table 2 Free vibration periods of the original and buckling restrained braced frames

Frame System		α	β	Period (s)		
				1st	2nd	3rd
Original Frame	OF	-	-	0.70	0.26	0.15
Buckling Restrained Braced Frames	BRBF100100	1.00	1.00	0.57	0.22	0.13
	BRBF075100	0.75	1.00	0.59	0.23	0.13
	BRBF050100	0.50	1.00	0.62	0.23	0.14
	BRBF025100	0.25	1.00	0.66	0.25	0.14
	BRBF075075	0.75	0.75	0.59	0.22	0.13
	BRBF050075	0.50	0.75	0.61	0.23	0.14
	BRBF025075	0.25	0.75	0.65	0.24	0.14
	BRBF075050	0.75	0.50	0.58	0.22	0.13
	BRBF050050	0.50	0.50	0.60	0.23	0.13
	BRBF025050	0.25	0.50	0.63	0.24	0.14
	BRBF075025	0.75	0.25	0.57	0.22	0.13
	BRBF050025	0.50	0.25	0.59	0.22	0.13
	BRBF025025	0.25	0.25	0.61	0.23	0.14

In these equations, A is the total area of the cross section, E is the modulus of elasticity whereas E_t is the modulus of elasticity after yielding, δ is the displacement, and $\delta_y = f_y \alpha A / k_1$ is the yield displacement.

In the current study, buckling restrained braces with different hysteretic behaviour defined following the approach stated by Kumar *et al.* (2007) were introduced into the side bay of the original frame, as shown in Fig. 3. In the retrofitting projects, the area of the reduced core is generally selected based on the desired seismic performance level of the building. If a force based design approach is followed, the reduced core area is designed to resist some portion of the design lateral load. If a displacement based approach is followed, the design process focus on determining the reduced core area that satisfy the qualitative description of the seismic performance state (Ruiz-Garcia and Teran-Gilmore 2010). In the current study, the effectiveness of BRBs with different strength and stiffness properties on mitigation of seismic displacement demand of a reinforced concrete structure was investigated. Thus, the total area of the core with respect to storey level was kept constant for the frames with non-buckling braces as shown in Table 1, while the reduced area of the core and the length of the core with reduced area were varied to obtain BRBs with different strength and stiffness characteristics.

For the value of α which is the reduced area of the core to the total area, 0.25, 0.50, 0.75 and for the value of β which is the ratio of the length of the core with reduced section to the total length, 0.25, 0.50, 0.75, 1.0 were considered. Therefore, 12 combinations of these α and β values were utilized in buckling restrained braced frames (BRBFs). The set of BRBFs assessed in this study including their modal properties are summarized in Table 2. In the analytical model, the hysteretic behaviour of the buckling restrained braces was modeled with nonlinear link (NLLink) members.

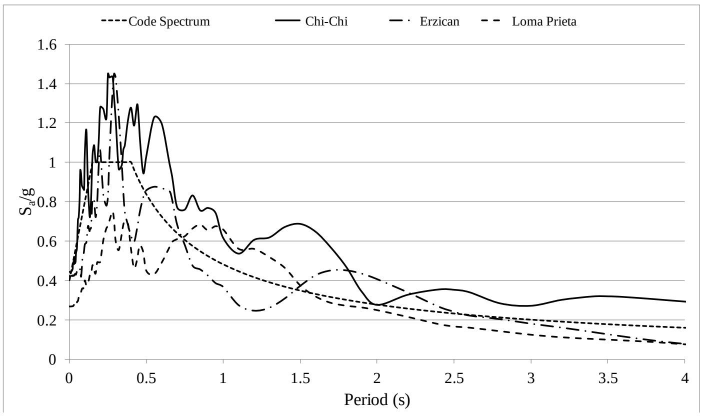
3. Nonlinear dynamic analysis

In order to see effects of different structural characteristics of buckling restrained braces on seismic response of the structures, the seismic behaviour of the unbraced and buckling restrained braced frames were investigated under earthquake ground accelerations. For this, nonlinear dynamic analysis was performed. In nonlinear time history analysis, analytical models consisting the structural members as described in the previous sections were subjected to earthquake ground accelerations.

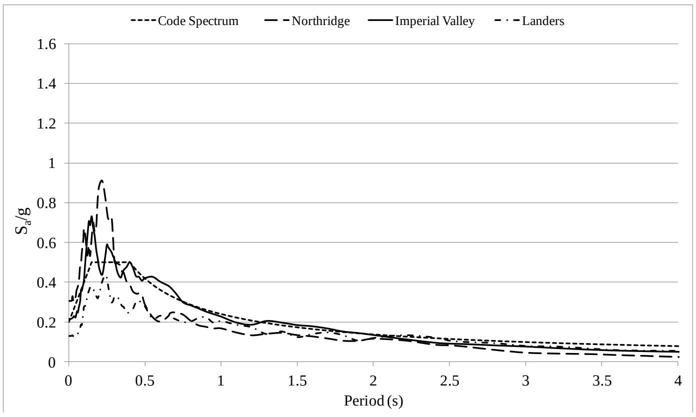
For nonlinear dynamic analysis of the frames, a set of natural ground accelerations generated as spectrum compatible were utilized (PEER 2011). Two levels of seismic hazard for the design code spectrum were considered such as: 10% and 50% probability of exceedance in 50 years period. The comparison between the design code spectrum and elastic spectra of the scaled natural ground accelerations are given in Fig. 5. Moreover, the characteristic properties of the natural ground motions such as the magnitude (Mw), the peak ground acceleration (PGA), the peak ground velocity (PGV), peak ground displacement (PGD), and characteristics of the site where acceleration recorded are listed in Table 3.

Table 3 Characteristics of the selected ground accelerations

Seismic Hazard Level	Earthquake Record	Year	Magnitude (M_w)	V_{s30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)	Scale Factor
10% probability of exceedance in 50 years	Chi Chi	1999	7.62	504.4	0.443	139.20	146.42	2.0
	Erzincan	1992	6.69	274.5	0.420	45.29	16.52	1.0
	Loma Prieta	1989	6.93	594.5	0.267	52.96	13.95	2.1
50% probability of exceedance in 50 years	Northridge	1994	6.69	336.2	0.600	31.07	10.46	0.5
	Imperial Valley	1905	6.53	202.3	0.430	55.33	32.82	0.5
	Landers	1992	7.28	271.4	0.128	19.00	9.25	1.0



(a)



(b)

Fig. 5 Elastic spectral accelerations of the ground motions scaled for seismic hazard of (a) 10% probability of exceedance in 50 years and (b) 50% probability of exceedance in 50 years

4. Results and discussion

As one of the global seismic performance parameter, maximum roof displacement demands for the frames were obtained and the variation of maximum roof displacement demand of the original and buckling restrained braced frames are demonstrated in Fig. 6. As seen in Fig. 6, for all BRBFs, the maximum roof displacement demand obtained was less than that of the original frame under all earthquake ground motions. The conducted comparative analyses illustrated that the variation of the α and β values had effect on the maximum roof displacement demand such as the maximum roof displacement of the BRBFs decreased with the increase in α value, generally increased with the increase in β values. For example, under Chi-Chi earthquake, the maximum roof displacement of the original frame was 3.4, 2.7, 2.2, and 2.0 times the maximum roof displacement demand of the BRBFs with α=0.50 and β=0.25, 0.50, 0.75 and 1.0, respectively. The ratio of the maximum roof displacement demand of the original frame to that of BRBFs with a constant β=0.50 and varying α values of 0.25, 0.50 and 0.75 became 2.2, 2.7 and 2.9, respectively. However, when subjected to earthquakes which have 50% probability of exceedance, these ratios reduced such as under Imperial-Valley earthquake, the maximum roof displacement of the original frame was 2.5, 2.3, 2.0 and 1.9 times the maximum roof displacement demand of the BRBF050025, BRBF050050, BRBF050075, and BRBF050100, respectively. As seen in Fig. 6, under Northridge earthquake, the BRBF025100 had slightly higher maximum roof displacement demand than the OF which may be due to characteristics of the earthquake acceleration and increase in the stiffness of the frame with BRBs.

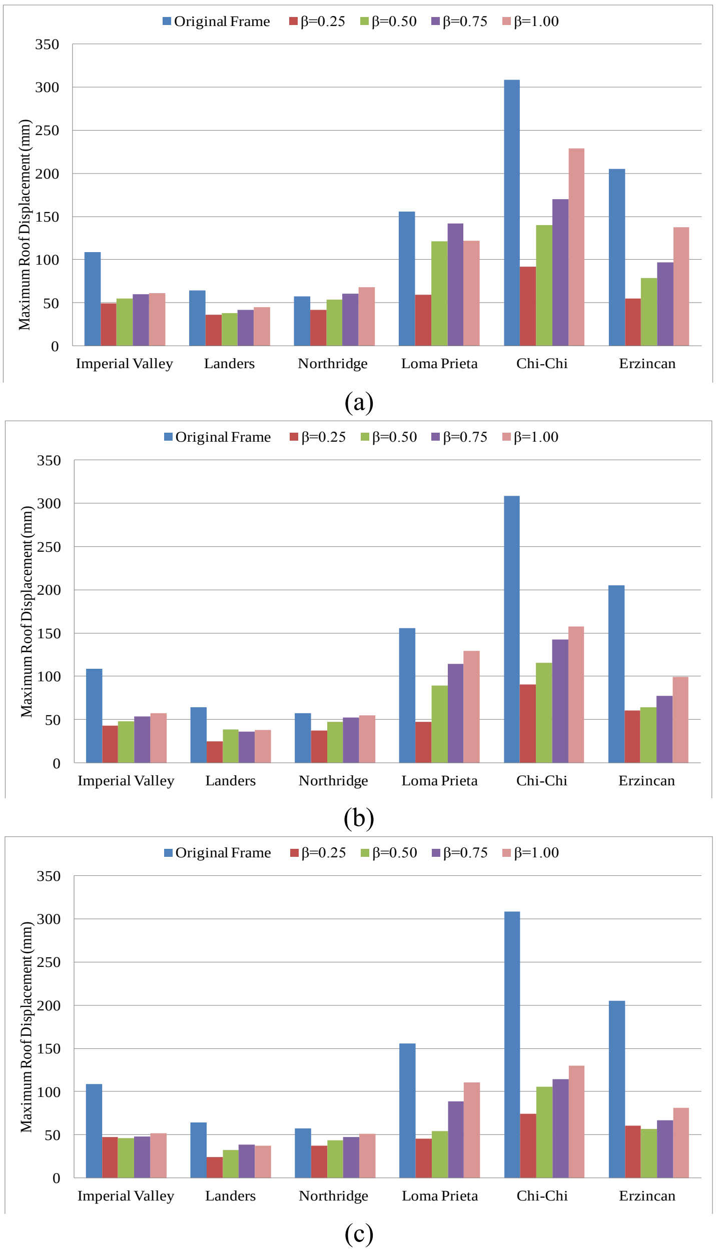
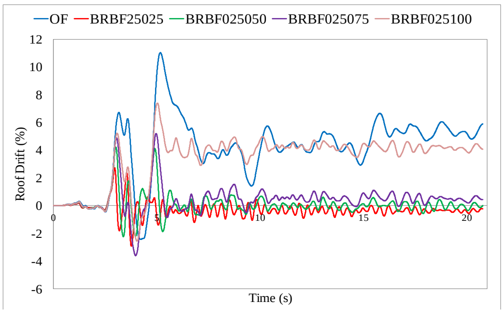
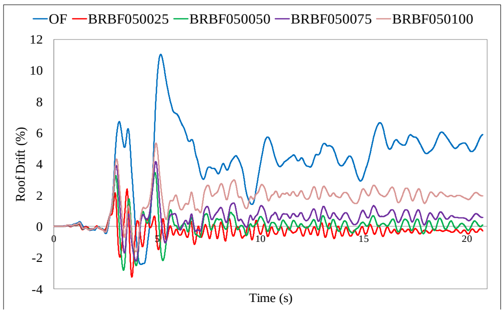


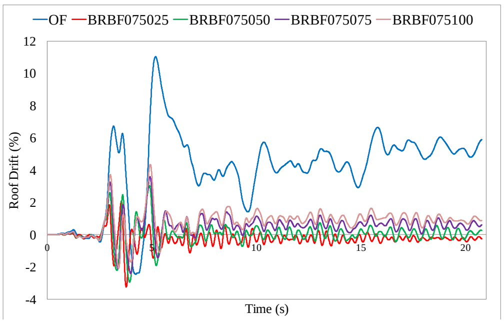
Fig. 6 Variation of maximum roof displacement demand for the original and BRBFs with α value of (a) 0.25, (b) 0.50, and (c) 0.75



(a)



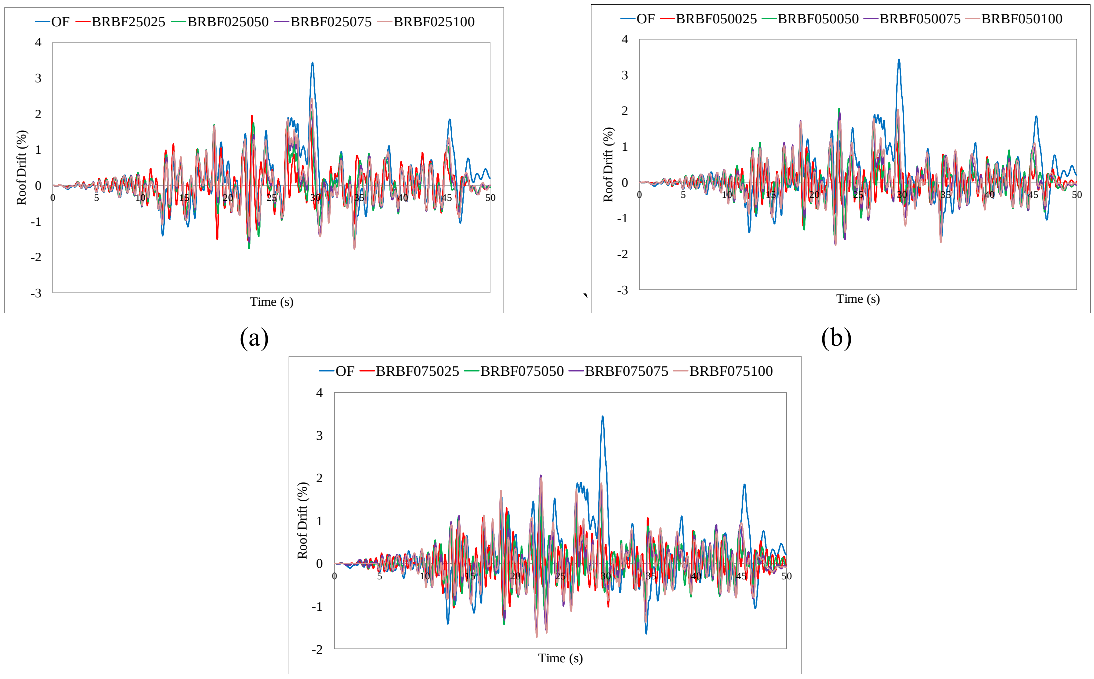
(b)



(c)

Fig. 7 Roof drifts obtained for the original and BRBFs with α value of (a) 0.25, (b) 0.50, and (c) 0.75 under Erzincan earthquake

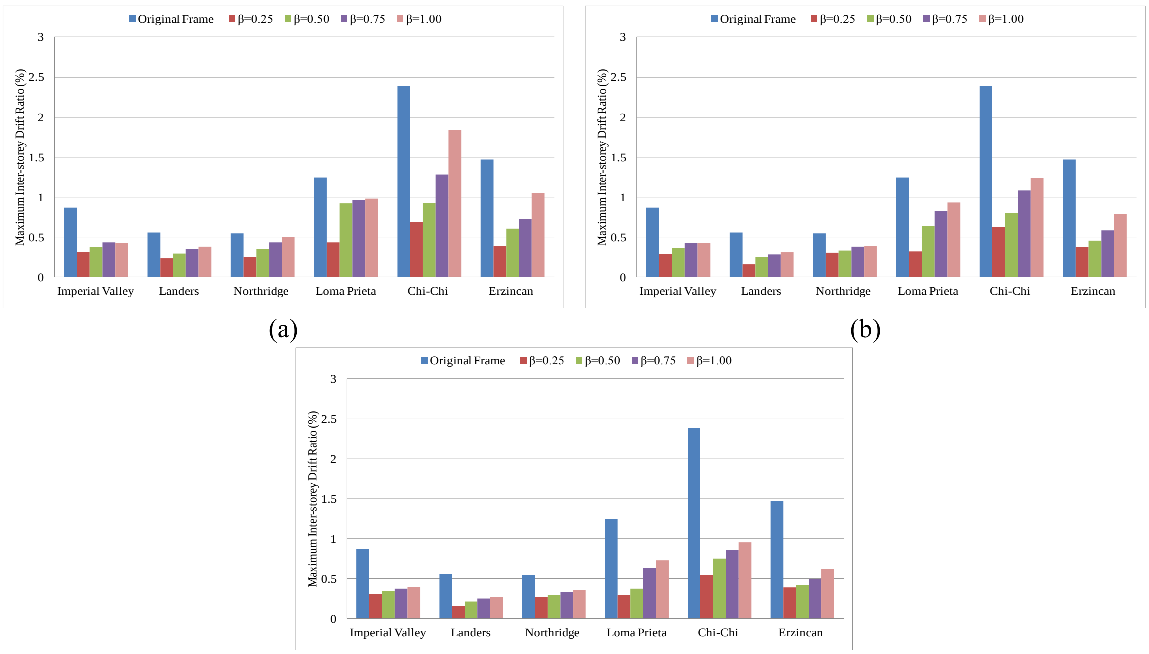
The time history response of the roof drifts of the original and BRBFs subjected to Erzincan and Landers earthquakes are presented in Figs. 7 and 8, respectively. When the response histories obtained under Erzincan and Landers earthquakes were compared, it was observed that the buckling restrained braces were more effective in reducing roof drifts when subjected to Erzincan earthquake acceleration which has 10% probability of exceedance. Similarly, significant residual drifts were examined in the original frame and in some of the buckling restrained braced frames when the earthquake accelerations with 10% probability of exceedance in 50 years were considered. Furthermore, with the increase in α value, more reduction in the roof drift history of the BRBFs was observed and it was much more pronounced especially for the case of β=1.0. For instance, under Erzincan earthquake, the reduction in the maximum roof displacement demand of the BRBF075100 with respect to the original frame was about 60%, whereas 33% to 51%



(a) (b)

(c)

Fig. 8 Roof drifts obtained for the original and BRBFs with α value of (a) 0.25, (b) 0.50, and (c) 0.75 under Landers earthquake



(a) (b)

(c)

Fig. 9 Variation of maximum inter-storey drift ratio for the original and BRBFs with α value of (a) 0.25, (b) 0.50, and (c) 0.75

reduction was obtained for BRBF025100 and BRBF050100, respectively. In addition to this, it was noticed that for a constant α value, the decrease in β value resulted in a reduction of the roof drift response history of the BRBFs. For BRBF025025, BRBF025050, BRBF025075, and BRBF025100, approximately 73%, 62%, 53%, and 33% reduction in the maximum roof drift with respect to the original frame was obtained.

Maximum inter-storey drift ratio obtained at each storey level for the original and BRBFs with α value of 0.25, 0.50, and 0.75 are given in Fig. 9. As seen from the Fig. 9, the maximum inter-storey drift ratio considerably decreased with the increase in the α value. The maximum inter-

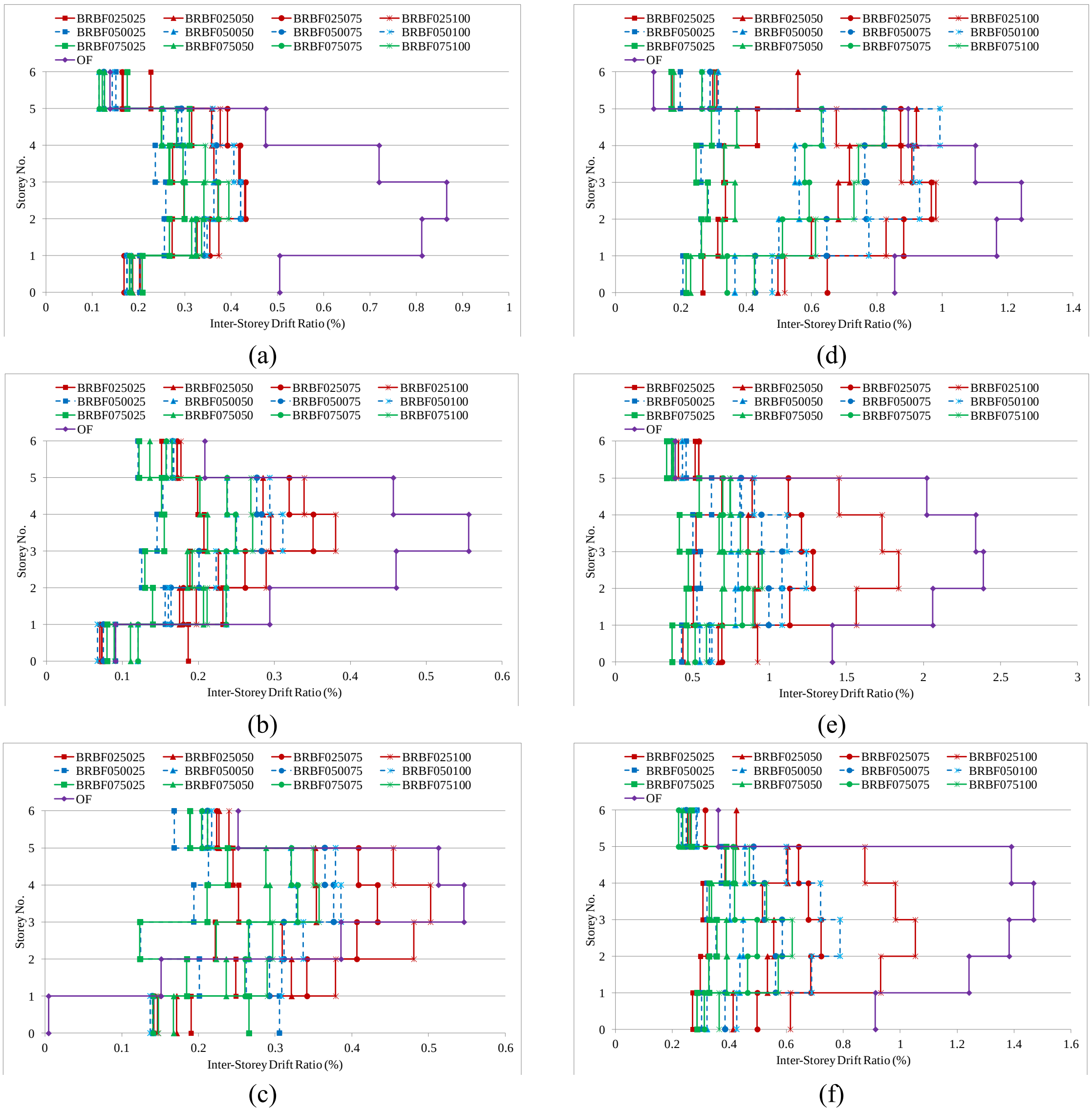


Fig. 10 Maximum inter-storey drift ratios for the original frame and BRBFs subjected to (a) Imperial Valley, (b) Landers, (c) Northridge, (d) Loma Prieta, (e) Chi-Chi, and (f) Erzincan earthquake accelerations

storey drift ratio decreased with the use of buckling restrained braces for all α and β values, and this reduction became more evident for the earthquakes with 10% probability of exceedance in 50 years period. Therefore, the results verified the beneficial effects of buckling restrained braces in reducing the seismic deformation demand.

The results of inter-storey drift ratios presented in Fig. 10 illustrated that buckling restrained braces were more effective for the earthquakes with 10% probability of exceedance, and thus they became more efficient in the inelastic range. Moreover, with the use of buckling restrained braces, generally more uniform distribution of inter-storey drifts along the height of the structure was attained, but the maximum inter-storey drift distribution through the height of the structure was changing, depending mainly upon the α and β values and also the ground acceleration used. The design values of α and β parameters of the buckling restrained braces significantly influenced the level of seismic response of the BRBFs. Moreover, it was observed that for a constant α value, the maximum inter-storey drift ratio at each storey level increased with the increase in β value, and for a constant β value, the maximum inter-storey drift ratio decreased with the increase in α value.

It was reported that a properly designed buckling restrained braces could accommodate inelastic deformations without permitting undesirable modes of failure (Sabelli and López 2004) and under strong ground accelerations; they could experience axial strains which are approximately 20 times their yield strain (Usami *et al.* 2005). In this study, conservatively, 12 times the yield strain was considered as the strain capacity of the buckling restrained braces and the maximum axial strain demand of the buckling restrained braces were checked. As an example, the normalized load deformation behaviour of buckling restrained braces obtained under Erzincan earthquake acceleration are given in Fig. 11. From this cyclic behaviour of buckling restrained braces, it was observed that with the decrease in α values, axial strain demand of the buckling restrained braces had a tendency to increase.

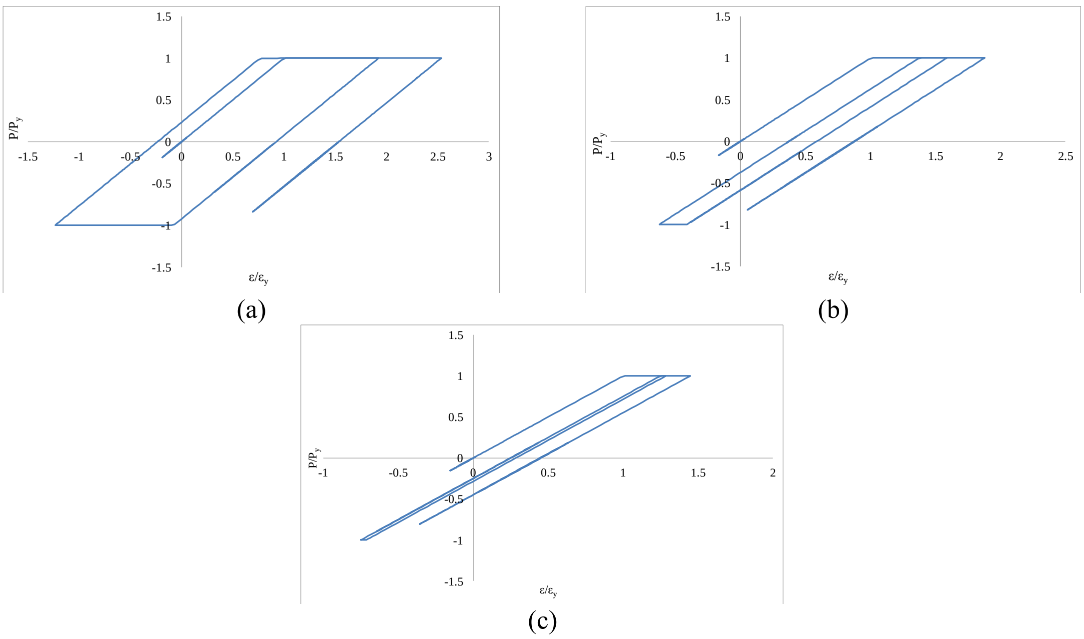


Fig. 11 Load deformation behaviour of BRBFs with β=1.0 and (a) α=0.25, (b) α=0.50, and (c) α=0.75 under Erzincan earthquake

5. Conclusions

This analytical study demonstrates that for improving the seismic behaviour of reinforced concrete structures, the buckling restrained braces can be a viable solution. It was observed that in the design of the buckling restrained braces with non-prismatic core sections, the selection of the α and β parameters affected the seismic response of the structures. For instance, the increase in the α value and decrease in the β value decreased the seismic deformation demand of the BRBFs. However, the change in the maximum deformation demand of the frames due to β value was small especially under the earthquakes with 10% probability of exceedance. Moreover, for all buckling restrained braces, the maximum strain demand was less than the limit since with the inclusion of the buckling restrained braces, the frames became stiffer. Because of these reasons, any value of β in the design of the buckling restrained brace could be utilized in case of providing enough rigidity for the reinforced concrete structure. Moreover, the buckling restrained braces were more effective in reducing the seismic response of the original frame when subjected to earthquakes with 10% probability of exceedance in 50 years since their contribution to seismic energy dissipation increased in the inelastic range.

References

Ash, C. and Bartoletti, S. (2009), "Seismic rehabilitation of an existing braced frame hospital building by direct replacement with buckling-restrained braces", *ATC & SEI 2009 Conference on Improving the Seismic Performance of Existing Buildings and Other Structures*, San Francisco, California, United States.

Bergami, A.V. and Nuti, C. (2013), "A design procedure of dissipative braces for seismic upgrading structures", *Earthq. Struct*., **4**(1), 85-108.

Clark, P., Kasai, K., Aiken, I.D. and Kimura, I. (2000), "Evaluation of design methodologies for structures incorporating steel unbonded braces for energy dissipation", *Proceedings, New Zealand Society for Earthquake Engineering, 12th World Conference on Earthquake Engineering*, Auckland, New Zealand.

Computers and Structures Inc. (2011), *SAP 2000 Advanced 14.0.0., Structural Analysis Program*, Berkeley, CA.

Della-Corte, G., D'Aniello, M. and Landolfo, R. (2015), "Field testing of all-steel buckling restrained braces applied to a damaged reinforced concrete building", *J. Struct. Eng.*, ASCE, **141**(1), D4014004.

Di Sarno, L. and Manfredi, G. (2010), "Seismic retrofitting with buckling restrained braces: Application to an existing non-ductile RC framed building", *Soil Dyn. Earthq. Eng.*, **30**(11), 1279-1297.

Farhat, F., Nakamura, S. and Takahashi, K. (2009), "Application of genetic algorithm to optimization of buckling restrained braces for seismic upgrading of existing structures", *Comput. Struct.*, **87**(1-2), 110-119.

FEMA 356 (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington DC.

Güneyisi, E.M. (2012), "Seismic reliability of steel moment resisting framed buildings retrofitted with buckling restrained braces", *Earthq. Eng. Struct. Dyn*., **41**(5), 853-874.

Güneyisi, E.M. and Şahin, N.D. (2014), "Seismic fragility analysis of conventional and viscoelastically damped moment resisting frames", *Earthq. Struct*., **7**(3), 295-315.

Güneyisi, E.M. and Ameen, N. (2014), "Structural behavior of conventional and buckling restrained braced frames subjected to near-field ground motions", *Earthq. Struct*., **7**(4), 553-571.

Housner, G.W., Bergman, L.A., Caughey, T.K., Chassiakos, A.G., Claus, R.O., Masri, S.F., Skelton, R.E., Soong, T.T., Spencer, B.F. and Yao, J.T.P. (1997), "Structural control: past, present and future", *J Eng. Mech*., ASCE, **123**(9), 897-971.

Iwata, M. and Murai, M. (2006), "Buckling-restrained brace using steel mortar planks: performance

evaluation as a hysteretic damper", *Earthq. Eng. Struct. Dyn*., **35**(14), 1807-1826.
Kalyanaraman, V., Sridhara, B.N. and Ramachandran, B. (1998), "A sleeved bracing system for earthquake resistant design of tall buildings", *Proceedings of the 11th Symposium on Earthquake Engineering*, University of Roorkee, Roorkee, India.
Kanaji, H., Takada, Y. and Mino, T. (2003), "Study on cost reduction of a long span truss bridge retrofitted with hysteretic dampers", *Proceedings of Japan road conference*.
Karalis, A.A. and Stylianidisa, K.S. (2013), "Experimental investigation of existing R/C frames strengthened by high dissipation steel link elements", *Earthq. Struct*., **5**(2), 143-160.
Koetaka, Y., Byakuno, Y. and Inoue, K. (2006), "Experimental verification of design criteria of knee brace damper", *Proceedings of the 8th Taiwan Korea Japan Joint seminar on earthquake engineering for building structures*.
Kumar, G.R., Kumar, S.R.S. and Kalyanaraman, V. (2007), "Behaviour of frames with non-buckling bracings under earthquake loading", *J. Constr. Steel Res.*, **63**(2), 254-262.
Merritt, S., Uang, C.M. and Benzoni, G. (2003a), "Subassemblage testing of CoreBrace buckling-restrained braces", University of California, San Diego, Report No. TR-2003=01.
Merritt, S., Uang, C.M. and Benzoni, G. (2003b), "Subassemblage testing of Star Seismic buckling-restrained braces", University of California, San Diego, Report No. TR-2003=04.
PEER (2011), Users manual for the PEER Ground motion database application.
Ruiz-Garcia, J. and Teran-Gilmore, A. (2010), "Seismic performance of existing steel buildings retrofitted with buckling-restrained braces", *14th European Conference on Earthquake Engineering*, Ohrid, Republic of Macedonia.
Sabelli, R. and López, W. (2004), "Design of buckling-restrained braced frames, modern steel construction", *North American Steel Construction Conference*, Long Beach, California.
Sabelli, R., Mahin, S. and Chang, C. (2003), "Seismic demands on steel braced frame buildings with buckling restrained braces", *Eng. Struct.*, **25**(5), 655-666.
Soong, T.T. and Dargush, G.F. (1997), *Passive Energy dissipation systems in structural engineering*, John Wiley & Sons, West Sussex, England.
Soong, T.T. and Jr Spencer, B.F. (2002), "Supplemental energy dissipation: state-of-the-art and state-of-the practice", *Eng. Struct.*, **24**(3), 243-259.
Symans, M.D., Charney, F.A., Whittaker, A.S., Constantinou, M.C., Kircher, C.A., Johnson, M.W. and McNamara, R.J. (2008), "Energy dissipation systems for seismic applications: current practice and recent developments", *J. Struct. Eng*., ASCE, **134**(1), 3-21.
Tsai, K.C., Hsiao, B.C., Wang, K.J., Weng, Y.T., Lin, M.L., Lin, K.C., Chen, C.H., Lai, J.W. and Lin, S.L. (2008), "Pseudo-dynamic tests of a full scale CFT/BRB frame Part I: Specimen design, experiment and analysis", *Earthq. Eng. Struct. Dyn.*, **37**(7), 1081-1098.
Usami, T., Ge, H.B. and Kasai, A. (2008), "Overall buckling prevention condition of buckling restrained braces as a structural control damper", *Proceedings of the 14th world conference on earthquake engineering*, Beijing, China.
Usami, T., Lu, Z.H. and Ge, H.B. (2005), "A seismic upgrading method for steel arch bridges using buckling-restrained braces", *Earthq. Eng. Struct. Dyn.*, **34**(4-5), 471-496.
Wada, A., Saeki, E., Takeuchi, T. and Watanabe, A. (1988), "Development of unbonded brace", *Nippon Steel Corporation Building Construction and Urban Development Division*, Tokyo, Japan.
Watanabe, A., Hitomi, Y., Yaeki, E., Wada, A. and Fujimoto, M. (1988), "Properties of braces encased in buckling-restraining concrete and steel tube", *Proceedings of 9th world conference on earthquake engineering*, Kyoto, Tokyo.

CC