Earthquake-Resistant Performance of a Steel Frame Model with Inverse-Chevron Buckling Restrained Braces

Yuan Xu, Hong Guan, Hassan Karampour & Yew-Chaye Loo
Griffith School of Engineering, Griffith University Gold Coast Campus Queensland4222, Australia

Xuejun Zhou
School of Civil Engineering, Shandong Jianzhu University, PR China

ABSTRACT: Beam-column connections of conventional braced steel frames are generally designed to have considerable stiffness for seismic resistance purposes. However, brittle failures of such connections and braces are often observed under severe earthquakes, which may potentially incur significant costs for replacement or repair. In this paper, a new steel frame system with bolted beam-column connections and buckling restrained braces (BRBs) is proposed to meet the challenges of the conventional frames. The key feature of this system is that seismic damages to the main structural members can be prevented by the BRBs which are capable of resisting all the lateral seismic loads without having risks of connection failure, whilst the beams and columns are responsible to carry the gravity loads. In this study, an OpenSees finite element model is developed and verified against published experimental tests of a 3-story single-bay frame with configuration of chevron and inverse-chevron BRBs. Subsequently, a 9-story 3-bay frame model taking the form of the proposed system is studied using OpenSees. In this model, typical type of inverse-chevron BRBs located in the middle bay is considered. The earthquake-resistant performance of the model under different seismic intensities is analysed using the time-history method. Numerical results indicate that the proposed system performs elastically under frequent earthquakes. When exposed to severe earthquakes, the main structural components are well within their capacities and the BRBs are found to display considerable energy dissipation capacity. The outcomes of this study confirm the effectiveness of the newly proposed steel frame system in resisting earthquakes.

1 INTRODUCTION

1.1 *Background*

In conventional seismic design of steel structures, the main focus is typically towards developing braced systems with high ductility and stiffness which are mostly exemplified by the rigidity of the beam-column connections. However, the conventional rigid connections usually require on-field welding which lacks construction quality assurance and is prone to industrial accidents (Faella et al. 1999). In this proposition, it is evident that the construction speed, precision and labor cost, are the primary factors influencing the construction quality. In addition to the problem encountered in welded connections, the huge cost of repairing or replacing damaged structural members after a severe earthquake is another major challenge (Sabelli et al. 2003). Since the conventional seismic design of steel frame structures intends to dissipate the earthquake energy by means of inelastic deformation of structural members or corresponding damage to the beam-column connections, the individual structural members designed by this principle may fail inevitably after being shaken by a design-level earthquake. Subsequently, significant expense would be required to repair or replace the damaged structural components.

1.2 *The proposed system*

In order to address the aforementioned issues, a new prefabricated steel system (Xu et al. 2015) with bolt-connected beams and columns is proposed herein. The potential damage to the main structural members due to earthquake ground motions can be prevented by introducing lateral-load resisting braces to withstand all the lateral seismic loads. Under a design-level earthquake, structural damage is limited to the lateral-load-resisting braces while the main bolt-connected members remain elastic. After the earthquake, the damaged lateral-load-resisting members can be rapidly replaced at a reasonable cost. In addition to the abovementioned advantages, bolted connections have demonstrated huge benefit in reducing the potential risks associated with low construction quality and industrial accidents.

The energy dissipation capacity of a conventional steel-braced structure subjected to earthquake loads is limited due to the low buckling capacity of the braces in compression. It has been shown that the energy dissipation or damage-prevention capacity of a steel framed structure can be greatly enhanced by employing buckling-restrained braces (BRBs) (Xie 2005, Black et al. 2002), which also offer an improved inelastic deformation capacity. Consequently, the demands for inelastic deformation of other structural members will be greatly reduced.

2 NUMERICAL MODEL AND VERIFICATION

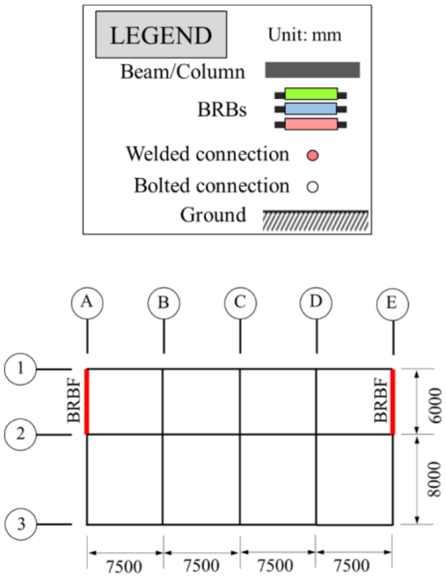
2.1 *General*

In order to investigate the feasibility of the proposed frame system with BRBs, dynamic time-history analysis method, which has the advantages of providing informative and accurate solutions for seismic resistance study, is used herein to examine the seismic behaviour of the frame model under earthquakes. The finite element analysis software OpenSees is employed and the model is created based on a hybrid loading test (Lin et al. 2012) conducted on a buckling restrained braced frame (BRBF) at the Taiwan National Center for Research on Earthquake Engineering. The numerical results obtained from the OpenSees model are compared with the experimental results to verify the accuracy the modelling approach.

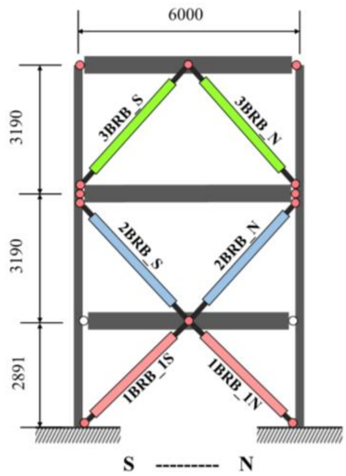
2.2 *The hybrid loading test (Lin et al. 2012)*

In Lin's (Lin et al. 2012) hybrid experiment, the prototype frame was based on a three-story office building in the city of Los Angeles. The building has two spans on side walls and four spans on front and lee walls (see Figure 1(a)). A single-bay of this building was chosen for the test and its elevation is shown in Figure 1(b), with BRBs in chevron and inverse-chevron configurations. In this BRBF, apart from the bolted beam-column connections in the first floor, all the other BRB-column and beam-column connections were welded. The time-history method was used in the test to study the seismic resistance of this BRBF.

Note that two hybrid loading tests were conducted in Lin's (Lin et al. 2012) experimental study. In the first test, local bulging was observed in one of the first-story BRBs after the commencement of the test, but the overall stability of the BRBF was maintained. The second hybrid test was conducted under the same ground motion yet in reversed direction, after the BRBs in the first story were replaced. Note also that the push-pull actions to the BRBF were only performed in the first test to reduce the residual displacements. Therefore, comparisons with the first test are undertaken herein to verify the modelling approach.



(a) Plan view of the building



(b) Elevation of the test frame

Figure 1. Details of the building and the test frame (all dimensions are in mm).

2.3 *The frame model*

The input earthquake for the test frame is adopted from LA03 ground accelerations with a PGA of 530 gal. A damping ratio of 5% is considered when scaling the ground accelerations. In the numerical simulation, all the parameters including the structural member sizes and seismic conditions incorporated in the OpenSees model are identical to those of the test frame. Both the welded and bolted connections are assumed to have approximately zero rotational stiffness.

In the model, the displacement-based beam-column elements with fiber sections are used to simulate the beams and columns. In order to account for the material non-linearity, "steel 01" material type with elasto-plastic bilinear response is selected for the beams and columns. To capture the geometric non-linearity, P-δ effects in the columns and BRBs are considered. Truss elements are used to model BRBs, and “steel 02” material type with similar hysteretic behaviour in tension and compression (Uriz & Mahin 2008) is employed to model the BRBs. For the pin connections exhibiting zero rotational stiffness, a pair of slave and master nodes is defined at the corresponding beam-column connections. These nodes are connected with zero-length elements which share the same translational degrees of freedom.

2.4 *Comparison between experimental and numerical results*

The inter-story drift ratios obtained from Lin’s experiment and the present simulation are compared in Figure 2. The comparison shows some discrepancies between the numerical and experimental results. A possible reason for this might be due to the assumption of the structural stiffness in the numerical models, where all the connections were assumed to exhibit pin behavior whereas in the experiment the connection stiffness was not neglected.

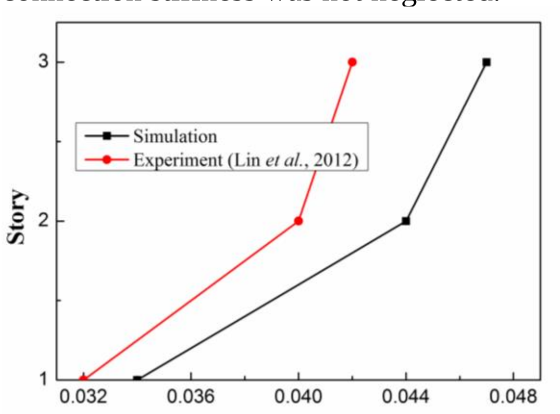
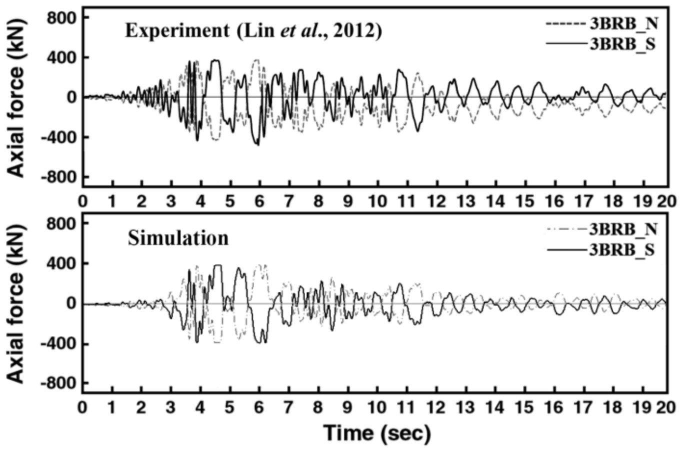


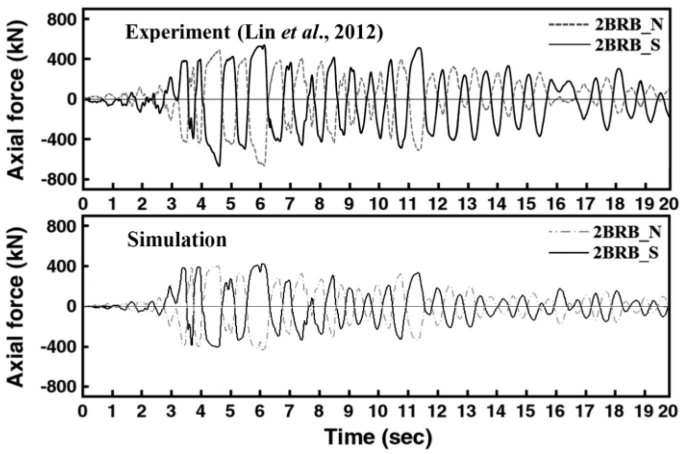
Figure 2. Inter-story drift comparison.

The axial force time histories of the BRBs are presented in Figure 3 where the notations are indicated in Figure 1. From Figure 3, it can be observed that the axial forces calculated from the present simulation of BRBs in the 2nd and 3rd stories are close to those measured in the experiment. However, following the unexpected bulging in one of the 1st story BRBs (i.e., 1BRB_1S), the experimental axial force in this BRB tends to be smoothed out after about 15 seconds (Figure 3(c)).

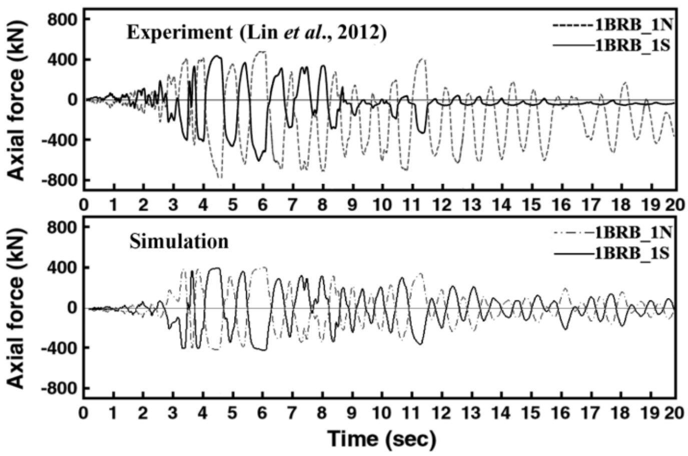
The hysteretic loops for the north BRBs (BRB_N) and south BRBs (BRB_S) are shown in Figures 4 and 5, respectively, where the axial force versus the core plate strain relationship is plotted for individual BRBs. Note that the core plate in a BRB refers to as the bearing component designed to resist the full axial force developed in the bracing. Figures 4 and 5 indicate that, except for the damaged BRB in the 1st story (1BRB-1S in Figure 5), the simulation results agree reasonably well with the experimental records. The minor discrepancies in the comparison may be caused by the additional frictional resistance existing in the core plate which is not considered in the simulation. Notwithstanding, the comparison confirms the accuracy of the numerical modelling approach.



(a) 3rd story BRBs (3BRB_N/S represents the BRB in 3rd story noted in Figure 1, the same for all other figures)



(b) 2nd story BRBs



(c) 1st story BRBs

Figure 3. Axial force time histories of the BRBs.

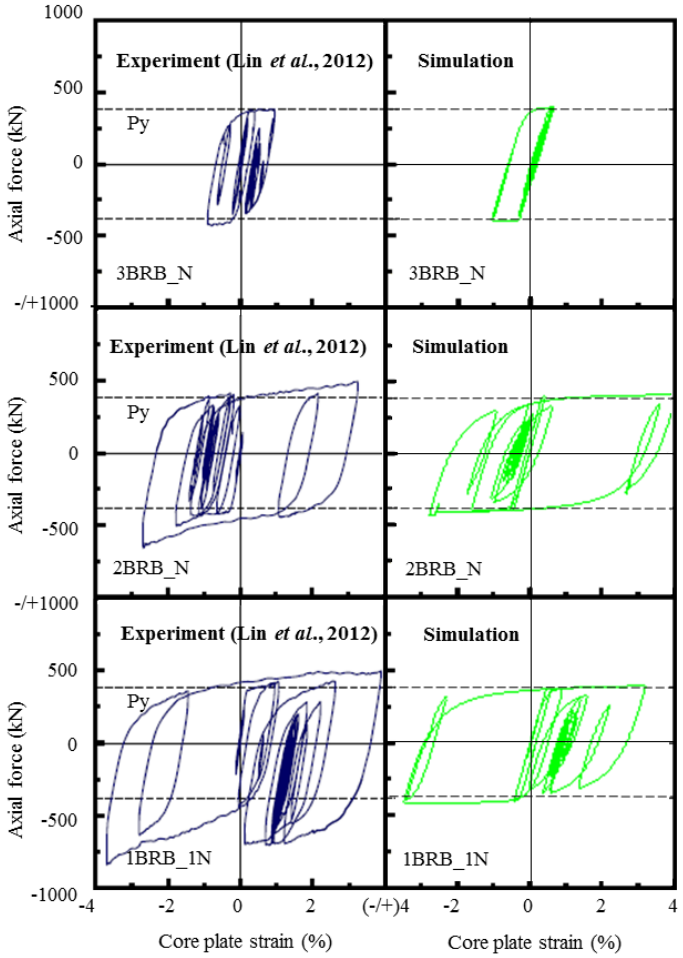


Figure 4. Hysteretic loops of the north BRBs.

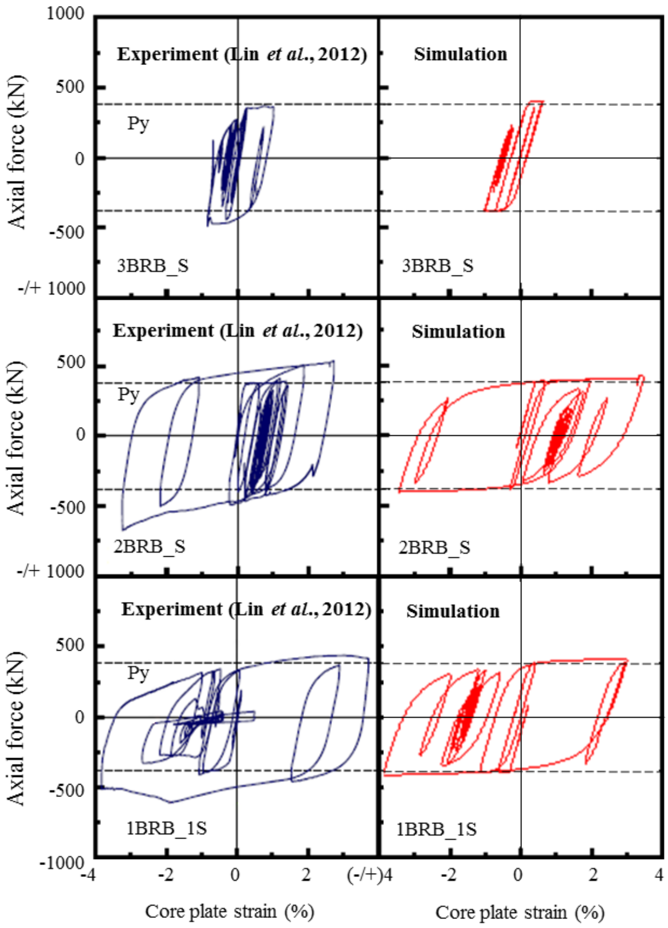
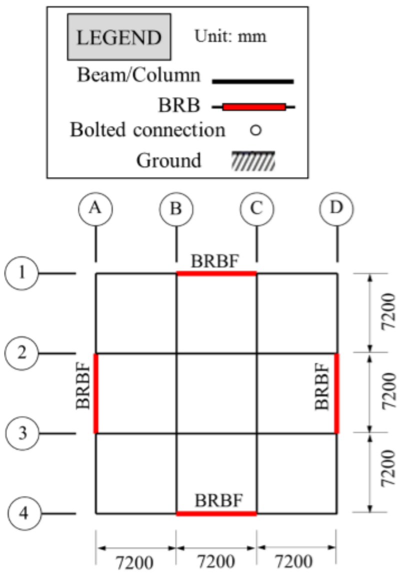


Figure 5. Hysteretic loops of the south BRBs.

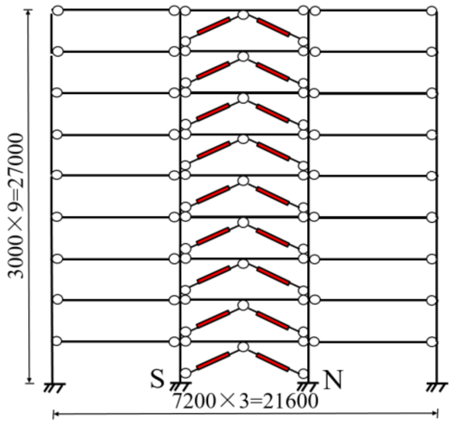
3 NUMERICAL ANALYSIS OF A 9-STORY BRBF

3.1 *Modelling details*

Using the previously verified numerical modelling approach, a 9-story steel frame is studied herein to testify the reliability of the proposed system with bolted beam-column connections and BRBs. In accordance with the current Chinese seismic design provision (GB50011 2008), the frame model is designed for a highly seismic location where the seismic fortification intensity is 8°, and the design basis earthquake acceleration value is 0.3g (g represents the gravity acceleration). The design earthquake group is 2 and the construction field belongs to Site-class II. The structural safety is specified as the second class, and the design working life is 50 years. The plan and elevation of the model are shown in Figure 6. The total height of the frame is 27m with a typical 3m story height. Its plan dimension is 21.6m×21.6m comprising of 3 bays in each direction. All columns are assumed to be fixed to the ground. As the structure is symmetrical about all axes, a 2D model is created to represent the structure. The yield strength of the steel beams and columns is 235MPa. The cross-sectional area of the BRB is 8960mm^2, and its yield strength is 160MPa. Beams and columns adopt 300×200×10×16 and 400×400 ×22×25 I-shaped steel sections, respectively.



(a) Plan view



(b) Elevation

Figure 6. 9-story steel frame.

One recorded earthquake ground motion from the 1940 El-Centro Earthquake and an artificial seismic wave A1 are considered in the modelling. The ground motions, which are scaled to a value of peak ground acceleration (PGA) of 110gal (1gal=1cm/s^2) (representing frequent earthquake) and 510gal (severe earthquake), are used as the seismic action on the structure horizontally after the static and modal analyses are conducted. A damping ratio of 2% is adopted for the non-linear dynamic time-history analysis.

3.2 *Modelling results*

Figure 7 shows the envelope of the inter-story drift ratios of the structure under the action of two seismic ground motions with different earthquake intensities. It can be seen that maximum drifts occur at the 6th - 7th story when the structure suffers frequent earthquake. Under severe earthquake, the seismic response is found to be more intense at lower stories. Due to the various hysteretic behaviors of the seismic wave input, the response of the entire structure, especially when the PGA is 510gal, is more pronounced under A1 ground motion. It can be further noticed that the predicted inter-story drifts of the structure are less than the specified upper bound values, i.e., 1/250 under frequent earthquakes and 1/50 under severe earthquakes (vertical blue lines shown in Figure 7), as specified by the Chinese code (GB50011 2008), indicating a reasonable level of the lateral stiffness of the structure.

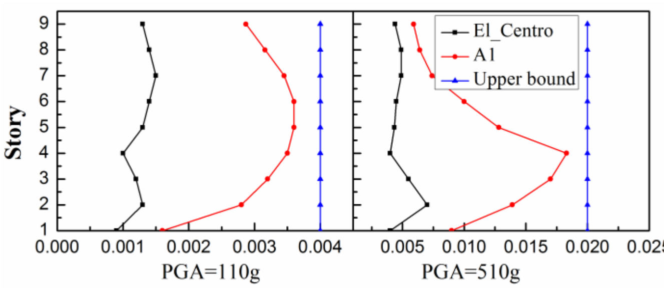


Figure 7. Inter-story drift ratios of the 9-story frame.

The structural member behaviour under frequent and severe earthquakes is investigated through the time-dependent stress and strain response of the columns and BRBs. Due to the structural symmetry, the typical columns labeled with "S" in Figure 6 (b) and the left-hand side BRBs are selected. Since the behaviours of the structural members under both ground motion actions are quite similar, only the stress-strain relationship under El-Centro earthquake is studied herein, with results being summarised in Tables 1-2 and Figure 8. Note that in Tables 1 and 2, the values in brackets represent the principal stress and strain ranges for the BRBs. It can be observed from the two tables and Figure 8 that both the columns and BRBs exhibit elastic behaviour under frequent earthquake; when the structure suffers severe earthquake, the columns still perform elastically even partial BRBs have entered into the plastic stage. When the structure is under frequent earthquake, the overall stress level is very low in the columns and BRBs. However, the stress level increases substantially when the structure is under severe earthquake. For the columns, large stresses and strains occur in lower stories under both earthquake intensities. The principal stress developed in the 1th story is found to almost reach the yield stress, while the stress ratio decreases gradually at higher stories. Furthermore, only the 1st, 2nd and 3rd story BRBs enter into the plastic stage completely, while the 4th and 5th story BRBs yield partially. On the other hand, the BRBs in the 6th, 7th and 8th stories are still in elastic stage without contributing effective energy dissipation capacity.

It can be further observed from the numerical results that the BRBs on the 1st, 2nd and 3rd stories exhibit excellent hysteretic and ductile behaviour, thereby playing the most effective roles in dissipating seismic energy. On the contrary, the BRBs on the 4th and 5th stories demonstrate relatively lower hysteretic energy capacity. Although not showing in Figure 8, the BRBs on the 6th and 8th stories are found not to completely enter into the plastic stage.

Table 1. Principal stress and strain ranges for typical columns.

Story	PGA=110gal		PGA=510gal	
	σ range (MPa)	ε range ($\times10^{-3}$)	σ rang (MPa)	ε range ($\times10^{-3}$)
1	(-73, 72)	(-0.35, 0.35)	(-234, 291)	(-1.1, 1.4)
3	(-49, 46)	(-0.24, 0.22)	(-116, 160)	(-0.78, 0.56)
5	(-35, 34)	(-0.17, 0.17)	(-100, 99)	(-0.49, 0.48)
7	(-14, 15)	(-0.07, 0.07)	(-46, 55)	(-0.22, 0.26)
9	(-3,3)	(-0.02,0.01)	(-9, 8)	(-0.04, 0.04)

* The stress and strain ranges listed in this table are all in linear relationship

Table 2. Principal stress and strain ranges for typical BRBs.

Story	PGA=110gal		PGA=510gal	
	σ range (MPa)	ε range ($\times 10^{-3}$)	σ rang (MPa)	ε range ($\times 10^{-3}$)
1	(-70, 64)	(-0.3, 0.3)	--	--
3	(-65, 65)	(-0.3, 0.3)	--	--
5	(-37, 39)	(-0.18, 0.19)	--	--
7	(-44, 44)	(-0.22, 0.21)	(-127, 135)	(-0.7, 0.6)
9	(-26, 26)	(-0.12, 0.12)	(-87, 76)	(-0.4, 0.4)

* The stress and strain ranges listed in this table are all in linear relationship

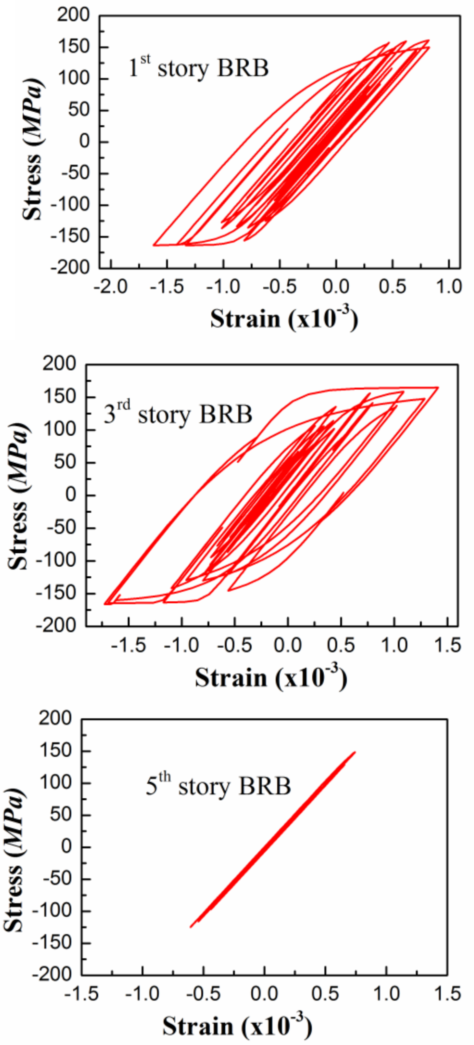


Figure 8. Principal stress-strain relationship for typical BRBs in the 1st, 3rd, and 5th stories.

4 CONCLUSION

Overall, the numerical simulation results reveal that the proposed steel frame (BRBF) with bolted beam-column connections and BRBs performs well in energy dissipation. Most BRBs are developed into the plastic stage and exhibit excellent hysteretic and ductile behaviour in resisting the lateral forces. In other words, the BRBs play important roles in absorbing energies against seismic activities. It can be initially concluded that the BRBFs can be used in the areas with a fortification intensity of 8°. From the economic point of view, the BRBs on the top stories can be designed to have smaller cross section, leading to more economic design and more uniform contribution from all the BRBs to the overall structural performance.

Other types and arrangement of BRBs will be considered in the future study to further optimize the earthquake-resistant performance of this type of structural system.

5 ACKNOWLEDGMENTS

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