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Seismic performance assessment of steel building frames equipped with a novel type of bending dissipative braces

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Abstract. The seismic performance of steel frames equipped with a particular type of bending dissipative braces (BDBs) having U elements, which has recently been introduced and tested by the authors, is investigated. For this purpose, two structural systems, i.e., simple and dual steel building frames, both with diagonal BDBs and different number of stories, are considered. After providing a design method of this new BDB, the detailed structural models are developed in the OpenSees platform to perform nonlinear dynamic analyses. Seismic performance factors like ductility, overstrength, response modification and deflection amplification factors are calculated using incremental dynamic analysis (IDA). In addition, to assess the damage probability of the structural models, their seismic fragilities are developed. The results show high energy dissipation capacity of both structural systems while the number of U elements needed for the bracing system of each story in the moment frames are less than those in the corresponding non-moment (simple) frames. The average response modification and deflection amplification factors for both structural schemes are obtained about 8.6 and 5.4, respectively, which are slightly larger than the corresponding recommended values of ASCE for the typical buckling-restrained braces (BRBs).

Keywords: bending dissipative brace; U-shaped element; seismic factors; nonlinear dynamic analysis; seismic fragility

1. Introduction

The typical solution for seismic resistant steel structures is traditionally based on the use of concentric steel members which are located into the frame mesh in the form of single bracing, cross bracing, chevron bracing or any other concentric bracing configurations. Although such systems possess high lateral stiffness and strength for resisting lateral loads, some drawback has to be taken into account, concerning the unfavorable hysteretic performance resulted from the buckling of compression members under severe excitations, which generally leads to a poor dissipation capacity of the whole system.

One way to improve the performance of traditional cross bracing systems against the buckling phenomena and the consequent low dissipation capacity can be obtained by the use of some particular bracing systems with similar hysteretic behavior in both tension and compression (Gioncu and Mazzolani 2013). In particular, the buckling-restrained braces (BRBs) (Wada *et al*. 1998, Black *et al*. 2004, Xie 2005, Takeuchi and Wada 2017, Xu *et al*. 2018) and the bending dissipative braces (BDBs) (Kelly *et al*. 1972, Gray *et al*. 2012, Aghlara and Tahir 2018, Taiyari *et al*. 2019a) are two examples of this family. BRBs are based on the concept of restraining lateral displacement of the core element when submitted to axial compression by using external boxes. More in detail, they can be classified into two types of so-called "unbounded brace" and "all-steel brace" while both dissipate the input energy by the inelastic behavior of the buckling-restrained core element in tension and compression (Gioncu and Mazzolani 2013, Takeuchi and Wada 2017). As an alternative, the typical characteristic of BDBs is to dissipate the earthquake energy by the inelastic bending of some considered devices. Although the term "BDB" has recently been suggested by the authors for this type of braces (Taiyari *et al*. 2019a), a very few special cases of this category have already been introduced by other researchers. Kelly *et al*. (1972) presented the first idea of using the plastic bending capacity of steel devices by proposing some steel plates, which bend perpendicularly to the axis of the diagonal brace. The axial force in the brace leads the devices to the plastic range, and thus the input energy is dissipated. Aghlara and Tahir (2018) introduced bar fused dampers by changing the steel plates of the flexural devices of Kelly *et al*. (1972) to a number of bolts.

During the last decades, the cyclic behavior of U-shaped devices has been experimentally tested and numerically evaluated by some researchers (Aguirre and Sanchez 1992, Dolce *et al*. 1996, Deng *et al*. 2013). Their outputs showed stable hysteretic behavior of the device with low fatigue and negligible strength and stiffness degradation as well as the simplicity in design and fabrication. Allowance for high displacements with insignificant degradation can be regarded as one of the distinguishing features of these devices. Their application was mainly oriented to seismic

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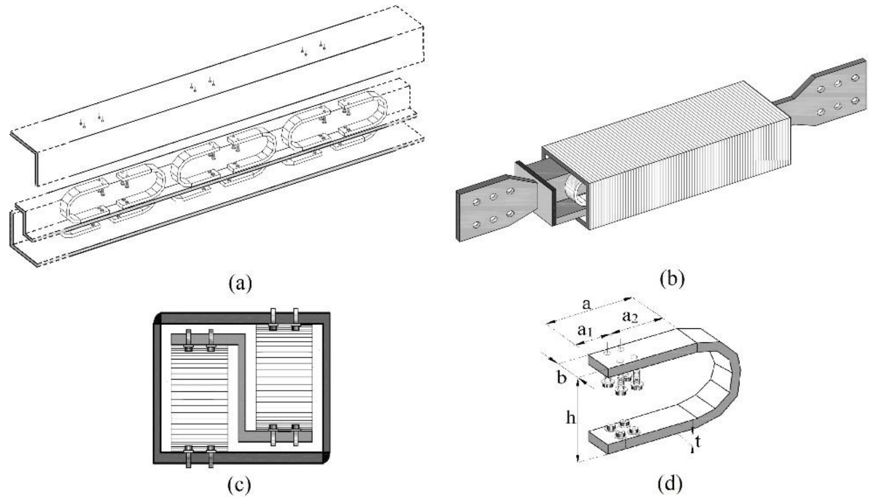

Fig. 1 Proposed BDB: (a) inside view (when six pairs of U elements are used); (b) outside view; (c) cross-section; (d) single U element

response reduction of bridges in the form of both dissipative device and base isolation system. Bagheri *et al*. (2015) investigated the application of U elements in building frames as a passive damper system and compared their seismic responses to the friction dampers. The U-shaped devices in their studies are used between the floor beam and chevron brace, which corresponds to the typical installation configuration of metallic passive energy dissipative devices in the framed structures.

More recently, Taiyari *et al*. (2019a) presented a new type of BDBs with the use of U elements within the diagonal braces. In this configuration, a rolling-bending motion of the steel U-shaped elements in the plastic range of deformation can dissipate the input energy to the system. By utilizing a different number of U elements, the energy dissipation capacity of the proposed bracing system can be adjusted in a broad range which can cover the seismic demand of almost all framed structures. The introduced brace has a similar performance to the classical BRBs because the low yield U elements prevent buckling of the brace. Three experimental tests have been done in the above-mentioned reference to demonstrate the efficiency of the proposed brace. It was found that the thickness and height of the U devices strongly affected the behavior of the proposed bracing system in a nonlinear manner. It was also shown that for practical applications, the Ramberg-Osgood model could be used to approximate the hysteretic behavior of the proposed system.

Additional aspects of the new BDB, like its effect on the seismic response of multi-story frames, are still not investigated and it seems to be a significant step in understanding its efficiency in structural applications. The main object of this paper is to evaluate the seismic behavior of steel building frames equipped with the new BDB proposed by the authors in their previous work (Taiyari *et al*. 2019a). For this purpose, two structural configurations, i.e., non-moment frames with bending dissipative braces (NFBDBs), and moment frames with bending dissipative braces (MFBDBs) are considered and their seismic performance factors, namely, response modification factor (and its components, including overstrength and ductility factors) and deflection amplification factor are evaluated. For each configuration, three models with a different number of stories are created in OpenSees software and the incremental dynamic analysis (IDA) using a number of earthquake records are performed. Finally, the fragility functions are also obtained to investigate their damage potential under seismic loads.

2. Characteristics of BDBs

The new BDB system proposed by Taiyari *et al*. (2019a) is based on the U-shaped steel strips placed inside the diagonal braces. In this configuration, the brace axial force causes U devices to be under a rolling-bending motion; so, they can provide a great dissipation capacity through their plastic flexural behavior. Experimental and numerical studies done by the authors on this bracing system have shown its stable hysteretic behavior. The schematic view of this brace which has three distinct parts (U-shaped, Z-shaped, and box elements) is depicted in Fig. 1. Any desired number of pairs of U-shaped steel strips can be placed between the box and Z-shaped elements to achieve the target capacity. This bracing system is connected to the frame at one end with the box element and at the other end with its Z-shaped part. The relative movement between Z and box elements results in a rolling-bending motion in the U-shaped steel strips that are connected to both Z and box elements. During the action, the plastic deformations and consequently the energy dissipation are not concentrated in one part of the U elements because the yielded part of each U element moves along the curved plate due to the relative longitudinal movement of the two opposite ends. This can be mentioned as one of the advantages of using U elements in this bracing system.

The box and Z parts of the BDBs must be designed so as to remain in the elastic range during the plastic deformation of U devices. In addition, to avoid local and global buckling within the box and Z parts, their buckling capacities must

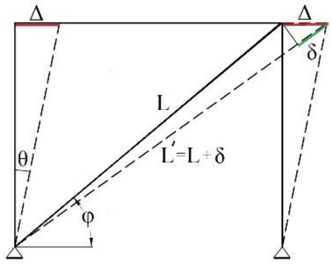

Fig. 2 Deformed shape of the brace

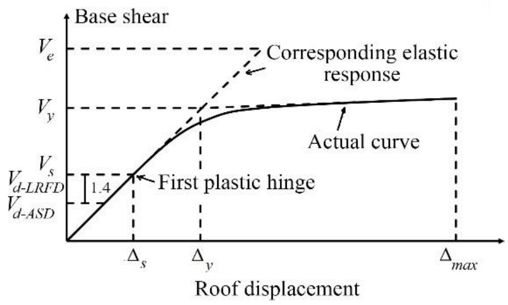

Fig. 3 Typical capacity curve of a structure

be set to be larger than the requested BDB strength. Accordingly, this bracing system can also be assumed as a special type of BRBs.

Fig. 2 depicts the lateral deformation of a one-bay building frame equipped with a diagonal brace like the proposed one. The axial displacement of the brace (δ) with the initial length and angle of L and φ, respectively, in both tension and compression, can be related to the inter-story drift ratio (θ) using Eq. (1)

$$\delta = \frac{\theta L \sin(2\varphi)}{2} \tag{1}$$

This displacement in the BDB governs the design of the upper and lower straight parts of U elements (a_2 in Fig. 1(d)) because it is mainly related to the deformation capacity of the U elements, which, in turn, is supplied by the segment of length a_2. Because of the parallel configuration of U elements in the brace, the required number of them can be calculated by dividing the total capacity (strength or stiffness) requested to each brace by the capacity of the single U element. Consequently, if each pair of U elements has a strength F_0, the total number of them (n_i) needed in the bracing system of the ith building story with story shear V_i is

$$n_i = \frac{V_i}{\cos(\varphi)\, F_0/2} \tag{2}$$

3. Seismic performance factors

Most seismic design codes consider a decrease in seismic design loads to take advantage of the fact that the structural systems possess a considerable amount of reserve strength (called overstrength) and capacity to dissipate input energy (called ductility). This is incorporated in the design of structures through a coefficient called response modification factor R in the US and behavior factor q in Europe. On the other hand, a structural system designed with the mentioned reduced forces must tolerate inelastic deformations. The maximum inelastic deflection that may occur during an earthquake event can be estimated through an elastic analysis using a deflection amplification factor. Such seismic performance factors can be obtained or evaluated by using a nonlinear procedure in the form of either static pushover or incremental dynamic analysis (IDA).

The pushover analysis is the simplest way for evaluating the seismic performance factors. Obtained results based on this method are influenced by a chosen lateral load pattern (FEMA-356 2000). However, the IDA procedure can give more reliable results. During the last decade, several studies are conducted to evaluate the seismic performance factors of different structural systems based on IDA results (Kim *et al*. 2009, Mahmoudi and Abdi 2012, Louzai and Abed 2015, Fanaie and Shamlou 2015).

Fig. 3 shows a typical capacity curve of a structural system, which can be obtained by either the pushover or IDA procedure. In this figure, Δ_y and V_y are the yield displacement and yield strength, respectively; Δ_{max} is the maximum displacement; V_e represents the force level that should be developed in the structure if it remains elastic during the design earthquake; V_s corresponds to the formation of the first plastic hinge and Δ_s is the corresponding displacement. According to the Load and Resistance Factor Design (LRFD) method, V_s and Δ_s correspond to the design base shear V_d and its displacement Δ_d, respectively; whereas they are reduced by a factor of about 1.4 when the Allowable Stress Design (ASD) method is used (Uang 1991).

According to Fig. 3, the seismic performance factors are calculated as follows (Uang 1991)

$$R = R_\mu R_s \tag{3}$$

$$R_\mu = \frac{V_e}{V_y} \tag{4}$$

$$R_s = \frac{V_y}{V_d} \tag{5}$$

$$C_d = \frac{\Delta_{max}}{\Delta_d} \tag{6}$$

where R_μ, R_s, R, and C_d represent the ductility, overstrength, response modification and deflection amplification factors, respectively.

4. Design and modeling of structures

4.1 General design assumptions

As analysis models, building frames having three, six, and nine stories, each with three bays of 5 m and story

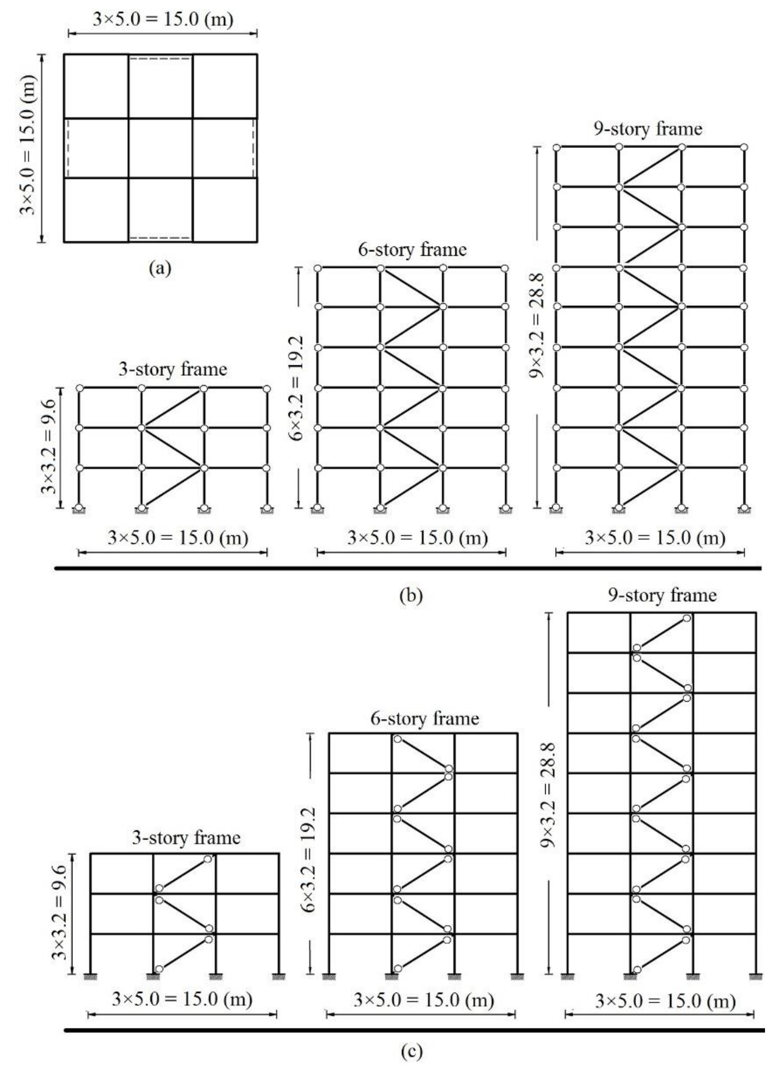

Fig. 4 Frame models: (a) plan view, (b) elevation view of NFBDBs, (c) elevation view of MFBDBs

height of 3.2 m (see Fig. 4) are designed based on AISC (2010) provisions. Recently, more sophisticated design procedures have been proposed for concentrically braced frames in simple and dual systems (see e.g., Giugliano *et al*. 2011, Longo *et al*. 2016), which can be applied to non-moment and moment frames with BDBs; however, in this study we prefer to use a simple and code-compliant design to assess the seismic performance of our proposed system. The gravity dead and live loads of all floors are taken to be 5 and 2 kN/m^2, respectively. The following parameters are assumed to calculate the earthquake design load according to ASCE 7 (2010): Importance Factor I_e = 1, Seismic Design Category D and Site Class D (stiff soil). The response modification factor R and the displacement amplification factor C_d are iteratively obtained starting from the values of 8 and 5, respectively, which are given for BRBs in the US code. The Los Angeles response spectrum is used with S_{DS} = 1.12 g and S_{D1} = 0.63 g, where S_{DS} and

Table 1 Designed sections of the 6-story models

Story level	NFBDB scheme			MFBDB scheme		
	Interior columns	Exterior columns	Beams	Interior columns	Exterior columns	Beams
6	W8×18	W8×10	W10×33	W8×18	W8×24	W10×33
5	W8×18	W8×15		W8×28	W8×24	
4	W8×28	W8×15		W8×28	W8×24	
3	W8×48	W8×18		W8×48	W8×31	
2	W8×48	W8×21		W8×48	W8×31	
1	W8×58	W8×24		W8×58	W8×31	

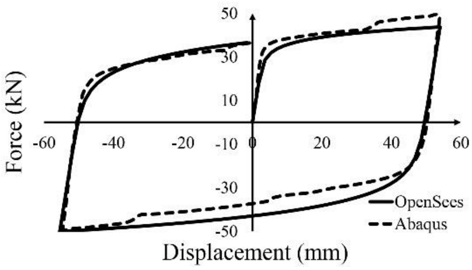

Fig. 5 The RO model of OpenSees for the hysteretic behavior of the BDB with a pair of the U elements (2U) in comparison to the Abaqus simulation

S_{D1} are the site design spectral accelerations at 0.2 s and 1.0 s, respectively. The designed sections of 6-story frames are listed in Table 1.

A 2D finite element model of each building frame is built in OpenSees platform to perform structural analyses. Both geometric and material nonlinearities are taken into account in the analyses. Herein, beam and column elements are modeled using the *forceBeamColumn* option in OpenSees, where the fiber sections are characterized by the inelastic behavior of steel S275 with an elastic modulus of 205 GPa, yield strength of 275 MPa and ultimate strength of 430 MPa. The geometric nonlinearity is considered by including the P-delta effect.

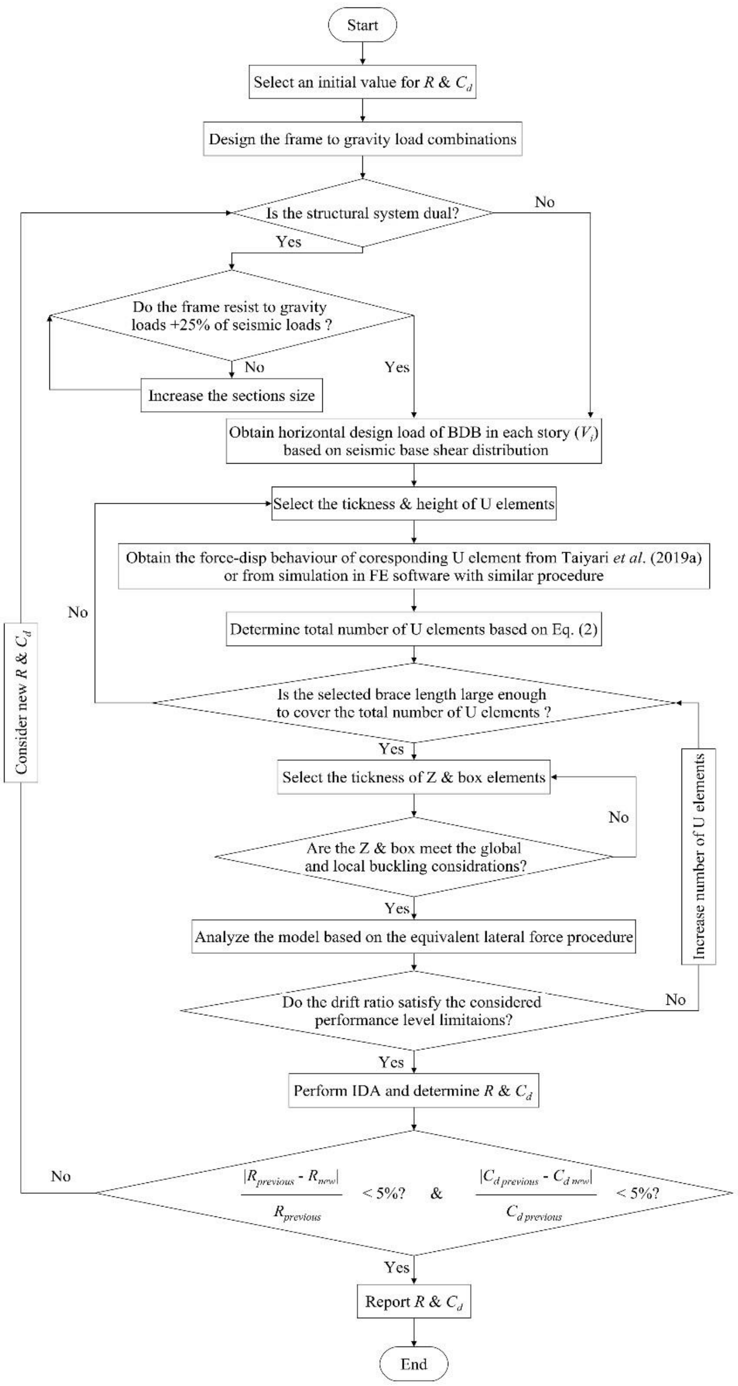

Fig. 6 The process of obtaining the seismic factors of the structural models with BDBs

4.2 Design of braces

Basic design criteria of BDBs were discussed in Section 2. According to Eq. (1), the required axial displacement capacity of the braces is calculated to be about 55 mm based on the inter-story drift ratio demand of 2%, as usually given for life safety (LS) performance level.

Enhancing the seismic behavior of the designed structures, the material of the braces has been chosen with smaller strength than in the other members: it is steel S235 with an elastic modulus of 205 GPa, yield strength of 235 MPa and ultimate strength of 360 MPa.

The unique behavior of the BDB under cyclic loadings was studied and discussed in the previous work of the authors (Taiyari *et al*. 2019a), and a simplified analytical model was proposed based on the Ramberg-Osgood (RO) model. Herein, the RO relation according to Eq. (7) is selected from the options of the OpenSees software in order to simulate the BDBs behavior via the *twoNodeLink* element.

$$\delta = \frac{F}{K} + a\left(\frac{F}{F_y}\right)^{\eta} \tag{7}$$

where F represents the axial force, δ is the corresponding displacement, K is the initial stiffness, F_y is the yield force, a and η represent the model parameters.

For setting-up the force-displacement relationship of the braces in OpenSees software, the values of F_y, K (or $\delta_y = F_y/K$), η, and a can be evaluated by fitting the RO equation with reliable numerical data. This fitting procedure can be done through an algorithm presented by Sireteanu *et al*. (2014a, b) based on the cyclic behavior of a pair of U-shaped elements with a required displacement capacity (herein 55 mm), which can be obtained experimentally or numerically. The FE simulations in Abaqus software and their calibrations to the corresponding experimental data were presented for several U-devices with a displacement capacity of 150 mm by Taiyari *et al*. (2019a). The similar Abaqus modeling procedure is adopted here to obtain the cyclic behavior and RO parameters of the considered U device. The geometrical properties of the device used, according to the definition of Fig. 1, are as follows: h = 120 mm, t = 15 mm, b = 60 mm (2×60 = 120 mm for a pair of U elements), and a = 85 mm. The value of a represents the length needed for the displacement demand (a_2 = 55 mm) plus the length of the connected part (a_1 = 30 mm). The resulted hysteretic behavior of the BDB with a pair of the U elements is shown in Fig. 5 while the corresponding RO parameters are F_y = 35 kN, K = 8504 kN/m, α = 4, and η = 10. Also, $F_0 = F_{@55\ mm}$ is 49 kN.

Fig. 6 represents the process of obtaining seismic factors for the assumed structural models with BDBs. As already mentioned, in order to design the building frame models, the values of 8 and 5 are initially assumed for the response

Table 2 Ground motion records used in this study

ID No.	Earthquake	Station	Year	M	PGA (g)	Site class (NEHRP)	Fault type
1	Northridge	Beverly Hills - Mulhol	1994	6.7	0.52	D	Thrust
2	Northridge	Canyon Country-WLC	1994	6.7	0.48	D	Thrust
3	Duzce, Turkey	Bolu	1999	7.1	0.82	D	Strike-slip
4	Hector Mine	Hector	1999	7.1	0.34	C	Strike-slip
5	Imperial Valley	Delta	1979	6.5	0.35	D	Strike-slip
6	Imperial Valley	El Centro Array #11	1979	6.5	0.38	D	Strike-slip
7	Kobe, Japan	Nishi-Akashi	1995	6.9	0.51	C	Strike-slip
8	Kobe, Japan	Shin-Osaka	1995	6.9	0.24	D	Strike-slip
9	Kocaeli, Turkey	Duzce	1999	7.5	0.36	D	Strike-slip
10	Kocaeli, Turkey	Arcelik	1999	7.5	0.22	C	Strike-slip
11	Landers	Yermo Fire Station	1992	7.3	0.24	D	Strike-slip
12	Landers	Coolwater	1992	7.3	0.42	D	Strike-slip
13	Loma Prieta	Capitola	1989	6.9	0.53	D	Strike-slip
14	Loma Prieta	Gilroy Array #3	1989	6.9	0.56	D	Strike-slip
15	Manjil, Iran	Abbar	1990	7.4	0.51	C	Strike-slip
16	Superstition Hills	El Centro Imp. Co.	1987	6.5	0.36	D	Strike-slip
17	Superstition Hills	Poe Road (temp)	1987	6.5	0.45	D	Strike-slip
18	Cape Mendocino	Rio Dell Overpass	1992	7.0	0.55	D	Thrust
19	Chi-Chi, Taiwan	CHY101	1999	7.6	0.44	D	Thrust
20	Chi-Chi, Taiwan	TCU045	1999	7.6	0.51	C	Thrust
21	San Fernando	LA - Hollywood Stor	1971	6.6	0.21	D	Thrust
22	Friuli, Italy	Tolmezzo	1976	6.5	0.35	C	Thrust

modification and deflection amplification factors, respectively. According to ASCE 7 (2010), the moment frame of a dual system should be capable of resisting at least 25% of the seismic design force while the combination of the moment frame and bracing system provides the total seismic resistance. However, in the simple building frames, the entire seismic force is to be resisted by the braces.

Considering a pair of U elements with the assumed geometrical and mechanical characteristics in this section, the preliminary estimate for the total number of U elements needed in the bracing system of the *i*th building story can be obtained using Eq. (2). Then, it can be increased if the inter-story drift ratio of the analyzed model exceeds its limit (2% for LS performance level). After satisfying all the design requirements, IDA analysis (as will be discussed in the next section) is performed to obtain the seismic performance factors (i.e., R and C_d). The obtained factors are then compared to the initially assumed ones; if a satisfactory agreement is not reached, the iterative process will repeat until an acceptable convergence is achieved.

5. Earthquake records and IDA procedure

The far-field earthquake record set recommended by FEMA-P695 (2009) for nonlinear dynamic analyses includes 22 component pairs of horizontal ground motions recorded at sites greater than or equal to 10 km from fault rupture. Here, because of the 2D nature of the structural models, only the largest component of each pair in terms of peak ground acceleration (PGA) is used for the IDA procedure. Table 2 shows the main characteristics of the selected records.

The seismic behavior of the proposed bending dissipative braced frames is investigated through the incremental dynamic analysis (IDA) which represents a reliable methodology to estimate the performance of structures under earthquake loads (Vamvatsikos and Cornell 2002). It involves subjecting a structural model to one (or more) earthquake record(s), in which each one is scaled to several levels of intensity (e.g., PGA), then producing one (or more) response curve(s) in terms of intensity level. In order to calculate the seismic performance factors from the dynamic capacity curves, the structural base shear and its corresponding roof displacement are obtained at each intensity level. It is clear that these capacity curves have no

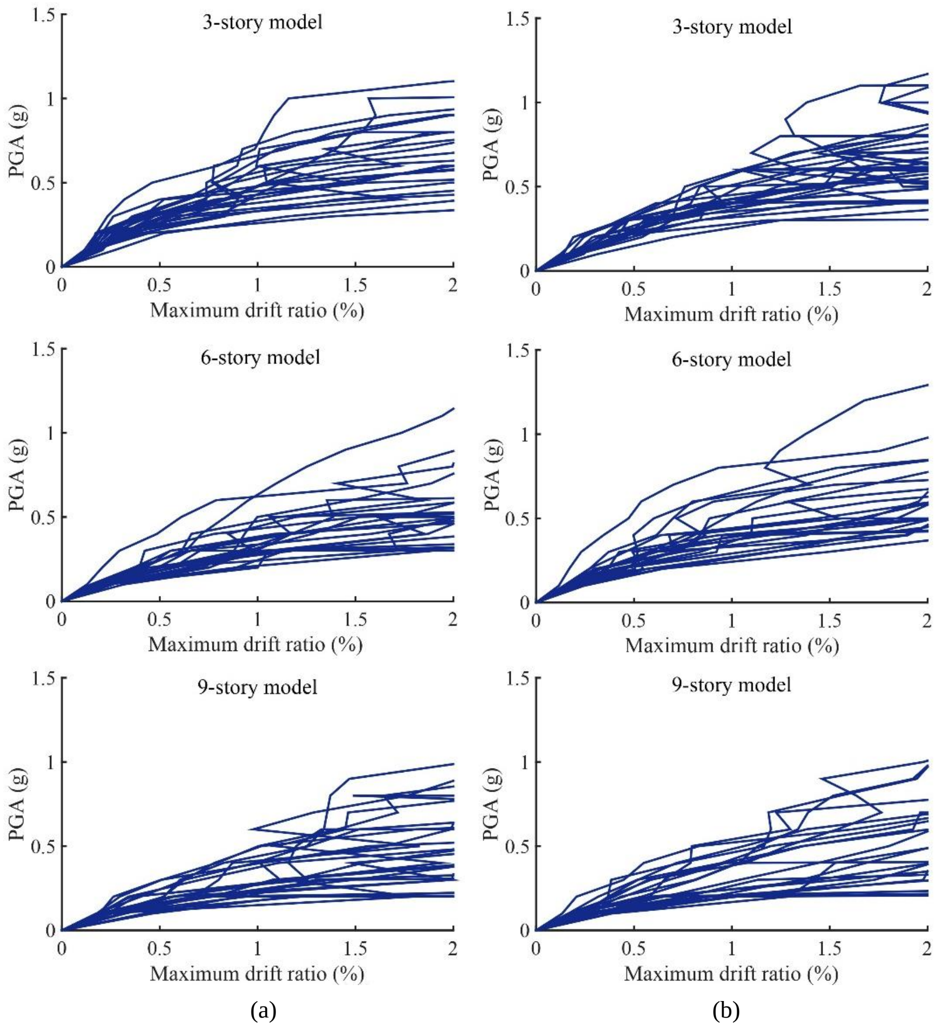

Fig. 7 The IDA curves at the final step of design process; (a) NFBDB and (b) MFBDB models

dependency on the choice of intensity measure; thus the PGA is selected here in order to characterize the IDA curves. The chosen earthquake records are scaled to several intensity levels with an increment of 0.1 g, starting from a PGA of 0.1 g up to reach the predefined limit state of damage. Herein, four different limit states of damage are assumed: (i) maximum inter-story drift ratio of 2% (corresponding to the life safety performance level), (ii) maximum brace displacement capacity (55 mm as discussed in section 4.2), (iii) ultimate strength of steel materials, and (iv) global dynamic instability of the frame.

6. Results and discussion

The resulted IDA curves corresponding to the final step of the repeating process described in Fig. 6 are shown in Fig. 7 and the corresponding base shear – roof displacement curves of the models are depicted in Fig. 8. In both Figs. 7 and 8, the curves are continued until reaching one of the predefined failure criteria. The seismic factors resulted from each earthquake record are computed through Eqs. (3)-(6), then the averaged values of the results of the 22 records are reported for each model. In these equations, V_e and V_y correspond to the maximum base shears of each model subjected to the last IDA scaled ground motion record when assuming linear elastic behavior and actual nonlinear behavior, respectively.

The obtained overstrengths, ductility factors, response modification factors, and deflection amplification factors are compared in Fig. 9 between NFBDB and MFBDB systems with different number of stories. Numerical values of the factors are also listed in Table 3. It can be observed that for both NFBDB and MFBDB models, the overstrength factor increases when the number of stories increases, whereas the trend is almost reversed in the case of the ductility factor. Considering the results of this study, the MFBDBs in comparison to the NFBDBs have overstrength and ductility a little higher and smaller, respectively.

The values of the response modification factor *R* lie in the range of 8.0 to 8.9 for all models of NFBDB and MFBDB with different heights. The average values of overstrength, ductility, response modification, and deflection amplification factors for NFBDBs with different number of stories are 3.0, 2.8, 8.6, and 5.4, respectively, and the corresponding values for MFBDBs are 3.4, 2.6, 8.6, and 5.4, respectively. Obviously, the same averaged value of the response modification factor (= 8.6) as well as deflection

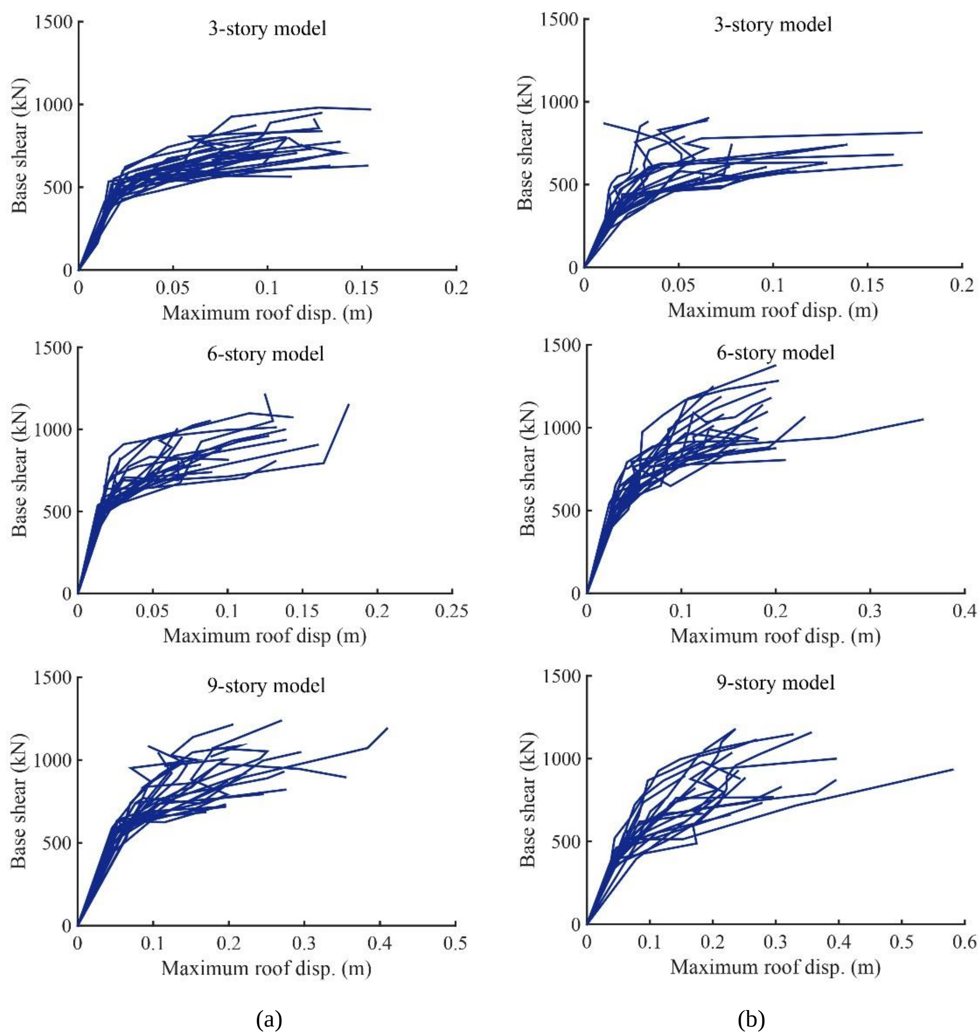

Fig. 8 The base shear – roof displacement curves of; (a) NFBDB and (b) MFBDB models

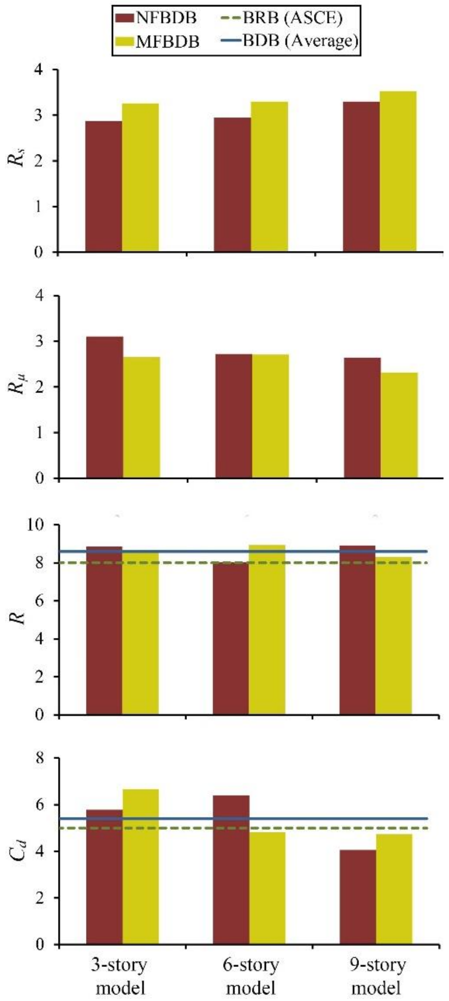

Fig. 9 Comparison of the seismic factors for NFBDB and MFBDB models with different number of stories

amplification factor (= 5.4) is obtained for both NFBDB and MFBDB models.

Comparing the resulted values of R and C_d for bending dissipative braced frames to the recommended values of ASCE 7 (2010) for the steel buckling-restrained braced frames (R = 8 and C_d = 5), it can be seen that the seismic factors are obtained only about 8% higher than the code recommended values for the similar system. This comparison is also indicated in Fig. 9 via solid and dashed lines.

The final number of U elements needed for the bracing system of each story in NFBDB and MFBDB configurations is given in Table 4. The U elements are placed in two rows inside the box element of the brace (see Fig. 1); so the total number of U elements of each diagonal brace in Table 4 is always an even number. As an example, detail of the brace in the first floor of the 9-story building model in NFBDB configuration, which contains the highest number of U elements, is illustrated in Fig. 10. The required length of the Z part and the whole brace are 4950 + 55 = 5005 mm and 4950 + 2 × 55 + 2 × 130 = 5320 mm, respectively, while the available diagonal distance in the models is 5936 mm.

Despite the fact that the response modification and the deflection amplification factors of both assumed structural systems are similar, the number of U-elements are different; i.e., each NFBDB needs more elements than the corresponding MFBDB. It can also be observed that the difference between the total number of U elements in the two configurations increases with increasing the number of stories. In the cases under consideration, the total number of U elements in NFBDB is 16%, 50%, and 98% higher than the corresponding one in MFBDB for the three, six and nine-story models, respectively.

The damage potential of a structural system can be

Table 3 Obtained and average values of the seismic factors for NFBDB and MFBDB models

Seismic factors	NFBDB				MFBDB			
	3-story	6-story	9-story	Average	3-story	6-story	9-story	Average
R_s	2.9	2.9	3.3	3.0	3.3	3.3	3.5	3.4
R_μ	3.1	2.7	2.6	2.8	2.6	2.7	2.3	2.6
R	8.8	8.0	8.9	8.6	8.6	8.9	8.3	8.6
C_d	5.8	6.4	4.1	5.4	6.7	4.8	4.7	5.4

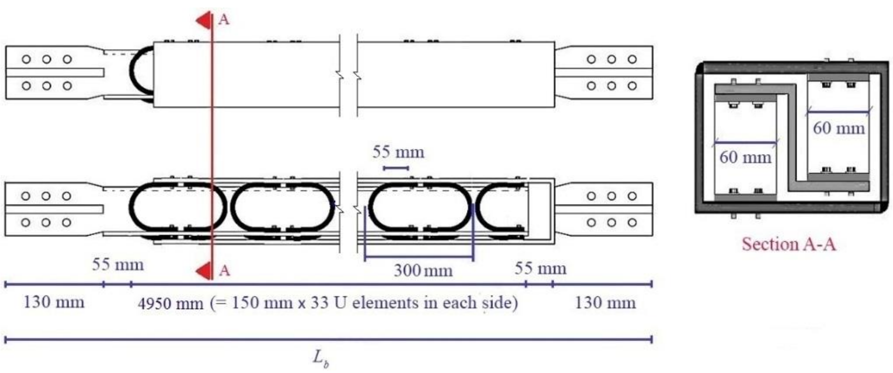

Fig. 10 Detail of the brace in the first story of the 9-story NFBDB which has 66 U elements

Table 4 Number of U elements in each bracing system

Story level	NFBDB scheme			MFBDB scheme		
	3-story	6-story	9-story	3-story	6-story	9-story
9			16			8
8			30			14
7			42			20
6		18	52		12	26
5		36	58		20	32
4		42	64		28	32
3	24	50	66	22	34	32
2	42	54	66	36	36	34
1	50	56	66	42	38	34
Total No.	116	252	460	100	168	232

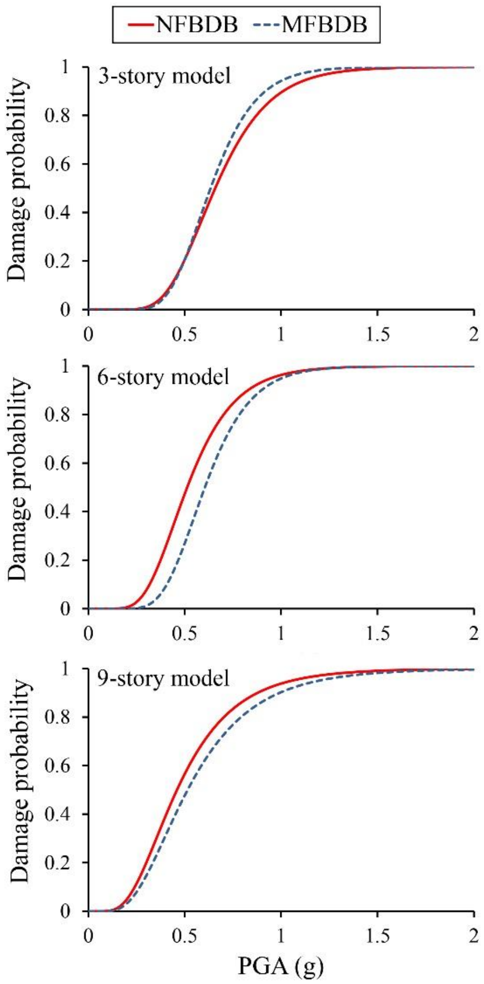

Fig. 11 Comparison of fragility curves for NFBDB and MFBDB models with different number of stories

evaluated by computing the probability of collapse or other limit states of interest as a function of earthquake intensity measure (e.g., PGA), which is called fragility function. In order to capture the nonlinear dynamic behavior of a structure, which is needed to estimate its fragility curve, one modern and effective method is using the IDA procedure. To see the mathematical framework of this approach, one can refer to Taiyari *et al*. (2019b). The fragility curves based on the IDA results of the analyzed models are obtained and depicted in Fig. 11. It is seen that, apart from the 3-story building models in which the damage probabilities do not differ widely between NFBDB and MFBDB, the damage potential of MFBDB configuration is less than that of NFBDB scheme. In both configurations, the damage probability generally increases as the number of stories of the models increases.

7. Conclusions

The performance of steel building structures with a particular type of bending dissipative braces in both non-moment and moment frames was investigated in this paper. The nonlinear incremental dynamic analyses were conducted on the structural models having different number of stories and their structural responses were evaluated. As a result, the seismic behavior factors, namely, overstrength, ductility factor, response modification factor, and deflection amplification factor were obtained. In order to assess the structural damage potential, their fragility functions were calculated and compared with each other.

The advantages of BDBs have been already shown in a previous paper (Taiyari *et al*. 2019a). This paper represents an extension of this activity to demonstrate the reliability of the BDB system in multi-story structures, which are examined in both NFBDB and MFBDB configurations for the sake of completeness. The analysis of the main differences between the two investigated configurations leads to the following remarks:

- For the given design requirements, the total number of U elements in the NFBDB configuration is 16%, 50%, and 98% higher than the corresponding one in the MFBDB configuration for the three, six and nine-story models, respectively.
- In both NFBDB and MFBDB configurations, the overstrength factor increases when the number of stories increases. The trend is almost reversed for the ductility factor.
- The comparison of the overstrength and ductility factors between NFBDB and MFBDB configurations shows that while the overstrength factor is a little higher in the MFBDB models, the ductility factor is often higher in the NFBDB ones.
- The average values of overstrength, ductility factor, response modification factor, and deflection amplification coefficient for NFBDB configurations are about 3.0, 2.8, 8.6, and 5.4, respectively.
- The average values of overstrength, ductility factor, response modification factor, and deflection amplification coefficient for MFBDB configurations are about 3.4, 2.6, 8.6, and 5.4, respectively. It can be seen that for both configurations, the average values of the response modification factor are the same. This is also valid for the deflection amplification factor.
- The obtained values of R and C_d for the proposed

BDB system are slightly larger than the code recommended values for a similar system, i.e., BRB.

- The comparison of the fragility functions indicates that the damage potential generally increases as the number of stories increases in both NFBDB and MFBDB configurations. In addition, the damage potential of the structures in the MFBDB configuration is less than that of the NFBDB configuration for the 6 and 9-story models.

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