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Improving the behavior of buckling restrained braces through obtaining optimum steel core length

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Abstract. Concentric braced frames are commonly used in steel structures to withstand lateral forces. One of the drawbacks of these systems is the possibility that the braces are buckled under compressive loads, which leads to sudden reduction of the bearing capacity of the structure. To overcome this deficiency, the idea of the Buckling Restrained Brace (BRB) has been proposed in recent years.

The length of a BRB steel core can have a significant effect on its overall behavior, since it directly influences the energy dissipation capability of the member. In this study, numerical methods have been utilized for investigation of the optimum length of BRB steel cores. For this purpose, BRBs with different lengths placed into several two-dimensional framing systems with various heights were considered. Then, the Response History Analysis (RHA) was performed, and finally, the optimum steel core length of BRBs and its effect on the responses of the overall system were investigated. The results show that the shortest length where failure does not occur is the best length that can be proposed as the optimum steel core length of BRBs. This length can be obtained through a formula which has been derived and verified in this study by both analytical and numerical methods.

Keywords: buckling restrained braces; steel core; optimum length; seismic loads; failure

1. Introduction

Concentric braced frames are commonly used in steel structures to withstand lateral forces. The most important issue concerning these systems is the behavior of the braces under compressive loads. Generally, the load-deformation curves of the braces can proceed to the level beyond the yield point under tensile loads, which indicates appropriate performance of the braces. However, the braces may be buckled under compressive loads before reaching the yield level. To overcome this deficiency, the idea of the Buckling Restrained Brace (BRB) has been proposed in recent years. In this way, the buckling of the brace is prevented through insertion of a steel core into a metal tube which is filled with a filler material such as concrete. Thus, the load-deformation curve of the brace can also proceed to the level beyond the yield point under compressive loads; that is, the brace will show a symmetric hysteresis behavior under both tensile and compressive loadings.

The idea of using steel braces protected by an outer mechanism was first introduced by Wakabayashi *et al.* (1973). Thereafter, further developments were made by researchers such as Fujimoto *et al.* (1988), Nagao and Takahashi (1990), and Nakamura *et al.* (2000). However, Watanabe *et al.* (1988) introduced the BRB in its typical current form. Later on, Black *et al.* (2002) performed a large-scale testing of BRBs and applied cyclic reversal loading to evaluate the lateral capacity of BRBFs. Mazzolani *et al.* (2009) carried out an experimental investigation on improvement of the seismic responses of a two-story one-bay reinforced concrete structure equipped with BRBs. In a recent attempt, Razavi *et al.* (2014) conducted research including experimental tests and numerical analyses to investigate the effect of reducing the length of the BRB steel core on its overall performance.

It can be seen that many studies have been conducted on the properties of the BRB components. Determination of the optimum steel core length of BRBs can be considered as one of such studies. A recent case has been studied experimentally by Mirtaheri *et al.* (2011) on a single-brace sample under cyclic loading. Hoveidae *et al.* (2015) investigated the seismic behavior of short-core all-steel buckling restrained braces. The results showed that the SCBRB system is partially able to reduce the story and residual story drifts in the braced frames. Jiang *et al.* (2015) studied the overall performance of buckling-restrained braces through refined finite element (FE) model via considering the contact interaction between the core and external restraining members that led to propose the recommended values of core width-to-thickness ratio, core thickness. Talebi *et al.* (2015) studied the effects of size and type of filler material through a three-dimensional numerical analysis on the performance of buckling restrained braces at fire. The study showed the premier fire performance of BRB with metal filler material in the gap than concrete as well as by increasing the size of the gap. Kim and Choi (2004) suggested reinforcing H-shaped braces with non-welded cold-formed stiffeners to restrain flexure and buckling through a finite element analysis. Wu

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and Mei (2015) studied the buckling mechanism of the steel core of buckling-restrained braces. The results indicate that increasing axial load affects the development of buckling mode. Also, the results led to obtaining the formulae of the maximum contact force and the maximum bending moment of the restraining member. Experimental tests were carried out on the reduced-core length BRB (RCLBRB) specimens including detachable casings to investigate the influence of variable core clearance and the local detailing of casings on the cyclic performance of RCLBRB specimens. The results showed the strain sustainability up to a core strain of 4.2% and nearly the same strength-adjustment factors for the RCLBRB specimens and conventional BRBs as noticed in the past studies (Pandikkadavath *et al.* 2016). Mirtaheri *et al.* (2017) investigated the Local and global buckling condition of all-steel buckling restrained braces. An experimental investigation had been conducted on the reduced-core length BRB (RCLBRB) specimens to evaluate their hysteretic and overall performance under gradually increased cyclic loading by (Muhamed *et al.* 2016). In order to obtain the limiting yielding core lengths of BRBs, a modified approach based on Coffin-Manson relationship and the higher mode compression buckling criteria has been proposed in this study (Pandikkadavath *et al.* 2017). Response modification factor of the frames braced with reduced yielding segment BRB had been investigated by Fanaei *et al.* (2014). Full-scale inelastic cyclic static tests of all-steel dismountable buckling restrained braces (BRBs) applied to an existing damaged reinforced concrete (RC) building was investigated by Della Corte *et al.* (2015).

In continuation of this study, the suitable steel core length of a BRB can be investigated by placing it in a frame under seismic loading. This leads to a discovery of the real behavior of such systems. Performing such experiments on BRBFs requires high costs and highly advanced equipment for simulation of the seismic loads. Furthermore, such an experiment is unique to testing the BRBF system, which requires many tests to be conducted together. Therefore, in this study, the issue is intended to be discussed through numerical methods. For this purpose, a number of BRBs with different lengths placed into several two-dimensional framing systems with various heights were considered. Then, the Response History Analysis (RHA) was performed, and finally, the optimum steel core length of BRBs and its effect on the responses of the overall system were investigated. This paper finally presents the following contributions:

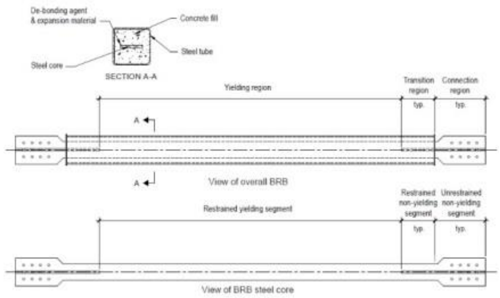
a) Development of a new relationship called

b) Performing a set of incremental dynamic analyses to provide some statistics of the data to investigate the sensitivity of behavior of the BRB system to the ground motion types.

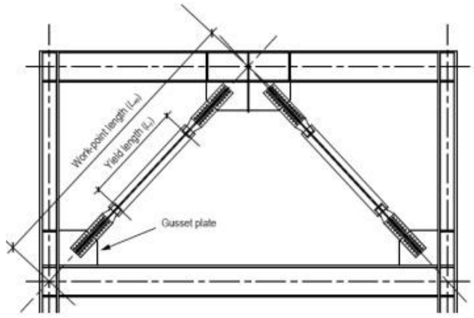
c) Play the groundwork for any future experimental research to verify and expand the new developed relationship.

2. General description

Generally, a BRB consists of a steel core enclosed with a restraining mechanism and an unbonding material which removes or minimizes the shear force transfer between the steel core and the restraining part. In addition, the steel core comprises three different parts, namely, a yielding encased core, an elastic or non-yielding encased core and an elastic outer connection. The yielding encased core is designed and detailed in order to dissipate input energy through yielding under seismic loads. The term “non-yielding encased core” implies that this part of the member must be designed and detailed in such a way that no plastic deformation or strain is induced within the component. Proper stiffening or local cross section increasing may be appropriate techniques to avoid local yielding within the member. The elastic outer connection must also remain elastic during seismic loading, and is used to connect the BRB to the frame members. Both non-yielding segments are responsible solely for load transfer from the frame members to the yielding steel core, in which energy dissipation and damping take place. As the configuration shown in Fig. 1 illustrate, BRBFs resist lateral loads as vertical trusses in which the axes of the members are aligned concentrically at the joints. Although the global geometric configuration of a BRBF is very similar to a conventional Concentrically Braced Frame (CBF), the members, connections, and behavior of BRBFs are distinctly different from those of Ordinary Concentrically Braced Frames (OCBFs) and Special Concentrically Braced Frames (SCBFs). The key difference is the use and behavior of the Buckling- Restrained Brace (BRB) itself.



(a)



(b)

Fig. 1 Schematic Fig. of a BRB: (a) Common BRB assembly, (b) Typical BRBF configurations

According to the studies conducted by researchers in the past, the performance of BRBFs can be improved against laterally-exerted loads by limiting the steel core length of BRBs to a percentage of the total length of the brace.

In other words, the length of a BRB steel core could have a significant effect on its overall behavior since it directly influences the energy dissipation of the member.

Modified BRBs with shorter lengths may be called damper BRBs, because they function as if a typical damper is utilized along each brace member. Furthermore, BRB could be effectively utilized as a damper to dissipate seismic input energy, particularly when used as a fuse within the brace in a frame. These damper (BRBs) are fabricated and tested in previous study both experimentally and analytically conducted by the authors.

Researchers (Mirtaheri *et al.* 2011) tested specimens with four different yielding encased core lengths to obtain information about optimized configuration. They found that Two important factors should be considered when BRBs with such short lengths are to be designed, namely, energy dissipation (or energy dissipation efficiency) and occurrence of low cyclic fatigue. Their efforts indicate that Specimen with moderate length (1 m) not only dissipated energy efficiently but also was able to sustain imposed loading up to the last loading cycle.

Obviously, as the core length decreases, the energy dissipation of the brace increases in an overall manner. This is easily justified since not only brace susceptibility to local and global buckling decreases, but also the core undergoes significant plastic deformations, and therefore, hysteretic energy dissipation inherently increases. On the other hand, as the core length decreases, susceptibility to low-cyclic fatigue also increases due to the tendency of the plastic strains to accumulate.

For investigation of the effect of low-cyclic fatigue on the structural components, Coffin-Manson relationship is mostly referred to in classical approaches (ASTM 2011). It simply states a linear relationship between the log of the number of constant amplitude cycles to failure, N_f, and the log of the strain amplitude experienced in each cycle, ε_i. Eq. (1) shows the exponential form of Coffin-Manson relationship.

$$\varepsilon = \varepsilon_0 \left(N_f \right)^c \tag{1}$$

In Eq. (1), ε_0 and c are material parameters which can be obtained through a series of experimental tests with constant amplitude loading cycles. However, during seismic excitations, it is unlikely that a component is subjected to constant amplitude cycling. As a consequence, a Miner's cycle counting index has been proposed to accumulate damage upon seismic loading in Eq. (2) (Fisher *et al.* 1988).

$$D = \sum \frac{N_i}{N_{fi}} \tag{2}$$

In Eq. (2), the damage of each amplitude of cycling is estimated through division of the number of cycles at that amplitude N_i by the number of cycles at that amplitude necessary to cause failure N_{fi}, and overall damage due to low-cyclic fatigue is estimated by linearly summing the damages for all of the amplitudes of the deformation cycles considered.

If index D exceeds unity due to application of the seismic loads, the member will fail. According to the above discussion, it is predictable that as core length decreases, index D increases and gets closer to unity. Therefore, it can be concluded that the shortest length where failure does not occur is the best length that can be proposed as the optimum steel core length of BRBs. Accordingly, Mirtaheri *et al.* (2011) derived a formula for the optimum steel core length of BRBs. To this aim, they combined the Coffin-Manson relationship and Miner's cycle counting index, and then, involved the FEMA-450 loading protocol provisions in the resultant combined equation (FEMA-450 2003). The formula is as follows

$$L_{opt} = \frac{2\lambda^{|c|}}{\varepsilon_0} \Delta_{bm} \quad , \quad \lambda = 7 + 4(0.5)^{\left|\frac{1}{c}\right|} + 2(1.5)^{\left|\frac{1}{c}\right|} \tag{3}$$

Where L_{opt} is the optimum steel core length of the BRB, ε_0 and c are material parameters, which were mentioned before, and Δ_{bm} is the amplitude of cycling concerning an allowable drift of the story.

3. Analytical study

For achievement of a more accurate approach in calculating the optimum steel core length of BRBs, a formula is proposed in this section which is based on the seismically exerted loads. The formula is expected to provide an accurate length equal to that obtained under real seismic loading. The process of deriving the formula is as follows:

Eq. (1) can be rewritten in the following form

$$\varepsilon = (\varepsilon_0 - \varepsilon_m)\left(N_f \right)^c \tag{4}$$

Where ε_m is the average strain, incorporated by Duggan *et al.* (1979) in Coffin-Manson relationship. Substituting Eq. (4) in Eq. (2) gives

$$D = \sum N_i \left(\frac{\varepsilon_i}{\varepsilon_0 - \varepsilon_{mi}} \right)^{\left|\frac{1}{c}\right|} \tag{5}$$

The strain experienced in each cycle i

$$\varepsilon = 2\frac{\Delta}{L} \tag{6}$$

Where Δ is the cycle amplitude and L is the length of the yielding encased steel core. Substituting Eq. (6) in Eq. (5) gives

$$D = \sum N_i \left(\frac{2\Delta_i}{L\varepsilon_0 - 2\Delta_{mi}} \right)^{\left|\frac{1}{c}\right|} \tag{7}$$

As noted earlier, if index D exceeds unity due to application of the seismic loads, the member will fail. Therefore, to prevent the failure of the member, index D must be less than unity

$$\sum N_i \left(\frac{2\Delta_i}{L\varepsilon_0 - 2\Delta_{mi}} \right)^{\left|\frac{1}{c}\right|} \le 1 \tag{8}$$

Eq. (8), which can be named Mirtaheri-Sehat-Nazeryan (MSN) formula, has no closed-form solutions, and therefore, it generally needs to be evaluated numerically. Accordingly, the optimum length must be obtained through a series of trial and error steps until the left-hand side of the equation experiences the nearest value to one.

It should be mentioned that Δ_i is the ith cycle amplitude, which is equal to BRB deformation demand in the ith cycle. Since the deformation demand history of the BRB does not contain complete closed cycles in seismic loading, in order to use Eq. (8), complete closed cycles should be formed by means of a cycle counting method as an effective approach. Here, Rainflow counting method is used for this purpose.

4. Numerical study

Numerical studies were carried out to attain more robust insights into the optimum steel core length of BRBs. To this end, first, five buildings with 4, 6, 8, 10 and 12 stories were modelled and designed as full-scaled structures. Using the bracing forces obtained from the above models, BRB dampers were designed based on the FEMA-450 provisions (FEMA-450 2003). Then, three earthquake records were selected as per Iranian Seismic Code 2800 for performing the RHA. In the next step, an arbitrary bracing frame was extracted from each building, and the BRB dampers were utilized by being placed with different lengths into the frame. Then, the new framing system was modelled in the OpenSees software, and the RHA was performed. Finally, the behavior of the BRBs was investigated, and their optimum steel core length was determined. The detailed description of each of these stages will be discussed in the following sections.

4.1 Design of the buildings

In order to obtain a reasonable model of the frame specimens, first, five buildings with 4, 6, 8, 10 and 12 stories were modelled and designed as full-scaled structures. All of these buildings had three spans in each of the two orthogonal directions. The width of each span was 5 m, and the height of each story was 3 m. In order to prevent any irregularity effect on the design procedure, the buildings were modelled symmetrically. The lateral resisting system of the buildings was a bracing system with simple connections in one direction and a moment resisting frame in the other direction. The braces were located in the middle span of adjacent frames, and the array of braces was in the form of inverted V, which is the most common array in bracing systems with BRB dampers. Fig. 2 shows the plan view of these buildings.

The material used in the modeling of the structural elements was steel ST-37, for which the minimum yielding stress is F_y=2400 kg/cm^2. Element sections were selected from among W-sections for the columns and beams, and HSS-sections for the braces. The ceiling system of the buildings was crossbars, in which the concrete slab had a thickness of 5 cm. The seismic design of the buildings was performed according to Iranian Seismic Code 2800 (2014). In this way, the buildings were designed for a typical site in Tehran, which represents a high seismic zone in Iran. The buildings were assumed to be located on soil type B (the average shear wave velocity to a depth of 30 m would be 360-750 m/s). In addition, the code UBC97-ASD was used for the structural designing (UBC97-ASD 1997).

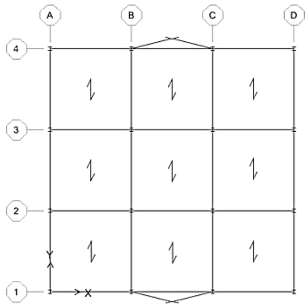


Fig. 2 Plan view of buildings

4.2 Design of the BRB dampers

The BRB dampers were designed using the bracing forces obtained from the models described in the previous section. All of the dampers had rectangular cross sections with aspect ratios of 1 to 10. The properties of the steel utilized in modeling the dampers were derived from a series of coupon tests which had been conducted by Mirtaheri *et al.* (2011). Table 1 summarizes the key parameters of the steel material used in the modeling of the dampers.

In order to obtain the dimensions of the steel core sections, it should be considered that the required axial strength of the brace (P_{nsc}) shall not exceed the design strength of the BRB steel core (φP_{ysc}) in Eq. (9), in which φ=0.9 (FEMA-450 2003)

$$P_{nsc} \leq \varphi P_{ysc} \tag{9}$$

The design strength of the BRB steel core is calculated as Eq. (10)

$$P_{ysc} = F_y A_{sc} \tag{10}$$

Where F_y is the specified minimum yield stress of the type of steel being used, and A_{sc} is the cross-sectional area of the steel core.

The BRB dampers were designed using the bracing forces obtained from the models described in the previous section. All of the dampers had rectangular cross sections with aspect ratios of 1 to 10. The properties of the steel utilized in modeling the dampers were derived from a series of coupon tests which had been conducted by Mirtaheri *et al.* (2011). Table 1 summarizes the key parameters of the steel material used in the modeling of the dampers.

4.3 Selection of the earthquake records

According to Iranian Seismic Code 2800 (2014), three earthquake records need to be selected for performing the RHA in two-dimensional systems. The selected records must represent the real ground motion characteristics of the site where the structures are constructed. Table 2 gives a

Table 1 Material parameters of steel (Mirtaheri *et al.* 2011)

Yield stress (MPa)	Yield strain	Ultimate stress (MPa)	Strain corresponding to ultimate stress	Ultimate strain
297.5	0.0022	449.8	0.0182	0.21

Table 2 Some of the characteristics of the earthquake records

Earthquake	Date	Station	Magnitude	Type	Areas affected	Max. intensity	PGA
Chi Chi	21-Sep-99	TCU045	7.6 Mw	-	Taiwan	-	0.499
Imperial Valley	15-Oct-79	CERRO-PRIETO	6.4 Mw	Strike-slip	Southern California	IX	0.168
Loma Prieta	17-Oct-89	ANDERSON-DAM DOWNSTREAM	6.9 Mw	Oblique-slip	Central Coast (California)	IX	0.260

brief description of the characteristics of the selected records.

The selected records were filtered to remove unwanted frequencies, and scaled to simulate the real seismic loading.

4.4 Modeling and analysis

As the final stage, the frame specimens were modelled and analyzed using the OpenSees software. To this end, frame 1 was extracted from each building, and the BRB dampers were utilized by being placed with different lengths into the frame. Then, the new framing system was modelled in the OpenSees software, and the RHA was performed. Finally, the behavior of the BRBs was investigated, and their optimum steel core length was determined. The OpenSees modelling process was as follows:

First, the required variables including the frame, earthquake characteristics, and applied loads were defined. Then, the node, material, section and element properties were also defined. All of the structural elements were modelled using the steel material, fiber section and nonlinear beam-column elements. In the next step, gravity and lateral loading were defined. Subsequently, a static analysis was performed under the gravity loads. Then, the parameters of dynamic analysis were set up, and the final dynamic analysis was performed for extraction of the desirable outputs that will be explained in the following section.

5. Incremental nonlinear dynamic analysis (IDA)

IDA is one of the well-known approaches to evaluate the structural performance level under a suite of seismic ground motions. IDA is able to estimate limit-state capacity and seismic demand by performing a series of nonlinear time history analyses under a suite of multiple scaled accelerogram records of earthquake ground motion acceleration. In IDA method, the intensity of selected ground motion is incrementally increased until the intended limit state seismic capacity of the global structural system is achieved. (Kishore *et al.* 2017).

Besides, it contains plotting an intensity measure (i.e., first mode spectral acceleration, Sa) versus a damage measure (maximum inter-story drift ratio).

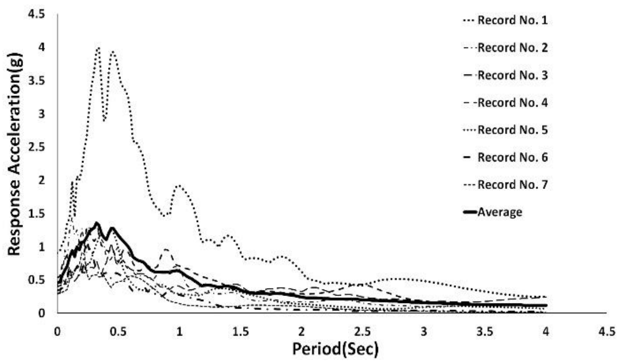


Fig. 3 Variation of spectral acceleration with period of structure

Table 3 Some of the characteristics of the earthquake records

Record No.	Earthquake	Station	Year	Magnitude (Richter)	d (km)	PGA (g)	Duration (s)
1	Chuetsu-Oki	Kashiwazaki Npp, unit 1 : ground surface	2007	6.8	11.0	0.909	69.99
2	El Mayor Cucapah	Ritto	2010	7.2	13.71	0.39	129.995
3	El Mayor Cucapah	Cerro Prieto Geothermal	2010	7.2	11.0	0.288	99.99
4	El Mayor Cucapah	Michoacan De Ocampo	2010	7.2	16.0	0.538	99.995
5	Loma Prieta	Gilroy Array #4	1989	6.93	14.34	0.419	39.99
6	Morgan Hill	Gilroy Array #4	1984	6.19	11.54	0.349	39.99
7	Northwest China-03	Jiashi	1997	6.1	17.73	0.3	59.98

In this study, maximum inter-story drift ratio and first mode spectral acceleration (Sa (T1,5%)) was considered as damage measure and intensity measure, respectively.

In order to more accurate evaluation of optimum steel core length of buckling restrained braces, the two dimensional Incremental Dynamic analyses of 4-story models subjected to strong ground motions which are shown in Table 4 were carried out, using OpenSees (2012). The Giuffre-Menegotto-Pinto steel material (OpenSees STEEL02 material 2012) with isotropic strain hardening was used for the calibration of all metals.

Columns were fixed at the base. Nonlinear beam–column elements with fiber sections were used to model beams, columns, braces and BRB dampers.

The entire brace comprises of three different parts, namely, a yielding encased core, an elastic or non-yielding encased core and an elastic outer connection. Because of only the yielding encased core part of each brace, would be experienced nonlinear behavior, considering the greater amounts of Young’s modulus (E) and section area for two parts of non-yielding encased core and elastic outer connection lead to avoid local yielding within these members.

Fig. 3 shows the elastic acceleration spectrum (Sa) of the seven selected records. The details of the selected earthquake ground motion data are represented in Table 3 show the details of the selected ground motion data.

The effect of the gravity framing system. And low cycle fatigue failure of BRBs and beam-column-brace elements was neglected in the models. Also, the ultralow cycle fatigue was neglected in the analyses discussed herein.

Inherent damping was modelled as Rayleigh damping by setting the critical damping ratio to 2% at the fundamental and third modes of the structure. A leaning column was used to model second order effects. All the analyses were conducted in conformance with the FEMA P-695 methodology (FEMA P-695 2009).

Because of source to site distance of records in greater than 10 km, Effects of Near-Fault Ground Motions were neglected.

6. Results

In view of all of the above numerical studies, four types of output information were obtained: the axial deformation response history of the BRBs, axial load response history of the BRBs, drift history of the stories, and, finally, periods of the vibration modes. These outputs needed to be processed in such a way that the desired results would be obtained. The results will be presented below:

1) The first issue which was investigated in this study was the verification of Eq. (3) under the real conditions. For this purpose, the optimum length arising from Eq. (3) was examined as compared with the optimum length in the real conditions. Table 4 shows the responses of the 4-story frame under Imperial Valley earthquake record to the various lengths of the BRBs.

As can be observed in Table 4, once the core length of the BRBs is less than 11 percent of the brace length, the maximum story drift adopts a high value, which implies the failure of the BRBs. Therefore, the shortest length where failure does not occur is 11 percent of the brace length. This length is the optimum steel core length of the BRBs under the real conditions. The optimum lengths corresponding to the other frame specimens were also obtained similarly. Fig. 4 illustrates the optimum lengths corresponding to all the frame specimens as a percentage of the actual brace length. On the horizontal axis in Fig. 4, C, I and L represent the applied earthquakes, namely, Chi Chi, Imperial Valley, and Loma Prieta, respectively. It is apparent from the Fig. 4 that the maximum value of optimum length concerns the 12-story frame under Imperial Valley earthquake record, and is 15 percent of the brace length. Since the brace length is 3.90 m for a conventional 5×3 panel, the above optimum length will be 0.59 m.

On the other hand, the optimum length obtained from Eq. (3) was taken as 1.05 m as reported by Mirtaheri *et al.* (2011). It should be noted that the number 1.05 m is based on Δ_{bm}, corresponding to the maximum allowable drift of 2%.

2) As previously mentioned, the optimum length of 1.05 m was calculated based on Δ_{bm}, corresponding to the maximum allowable drift of 2%, which means that the frame was assumed to reach the maximum allowable drift. However, the maximum drifts obtained from numerical analysis were not necessarily equal to the maximum allowable drift of 2%. Thus, since Eq. (3) is a function of the BRB deformation demand, it is preferable that the optimum length is calculated in terms of the deformation demand developed in the BRB. This leads to a more realistic insight into the relationship between the optimum

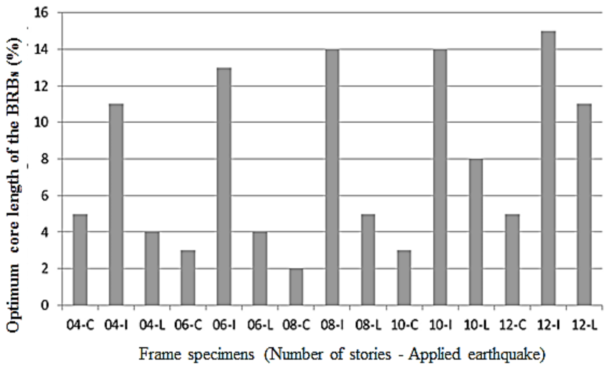


Fig. 4 Optimum core length of the BRBs used in the frame specimens

length arising from Eq. (3) and that in the real conditions. These two optimum lengths are illustrated in Fig. 5.

3) For verification of the equation given in Section 3, the optimum length arising from Eq. (8) was examined similarly to that in the real conditions. For this purpose, one can obtain the optimum length in the real conditions as described in the previous section, and that arising from Eq. (8) as per the relevant calculations resulting from the analysis. Fig. 6 gives a comparison between these two optimum lengths.

It can be seen in Fig. 6 that in most cases, the difference between the two curves is less than 25 percent of the optimum length in the real conditions. The difference of 25% is much smaller than that obtained in the previous part, so it can be stated that Eq. (8) has an approximately acceptable accuracy, and can be considered as an accurate equation in analysis and design applications.

According to Fig. 7, line $L_{opt}/L=1$ implies a case in which the core length of the BRBs is equal to their corresponding optimum length. Therefore, the points of the curve meeting the above line will correspond to the optimum length. As can be seen, this occurs only for the case of the optimum length obtained in the previous part, after which the curve shows a descending trend, which indicates that the structure moves away from its optimum state. This also applies to the other frame specimens. Similarly, it can be demonstrated that the optimum length obtained in the previous part is indeed the best length that can be proposed as the optimum steel core length of the BRBs. Where N is the number of steps addressing a certain core length of the BRBs. It should be mentioned that the first step concerns the case of the optimum length obtained in the previous part. Based on Table 5, the dimensionless curve of N-L_{opt}/L can be drawn as shown in Fig. 7.

Fig. 8 shows the summarized IDA curves (50% fractiles) determined on the basis of IDA analysis for the 4-story model. BRB damper gave very encouraging results in terms of maximum inter-story drift ratios.

4) Under seismic loading, changing the core length of a BRB would result in a change in its deformation demand and, subsequently, in the related optimum core length. As a consequence, there is an optimum length corresponding to each core length of a BRB which is used in a frame under

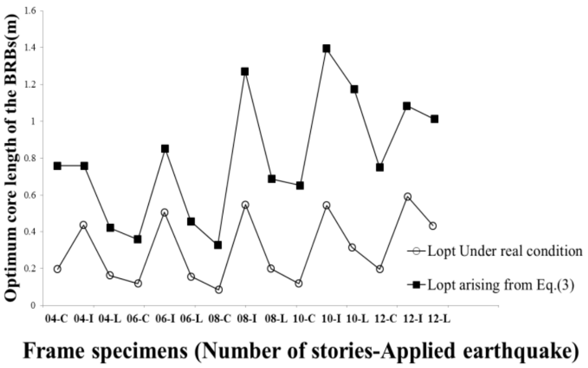


Fig. 5 comparison between numerical and analytical optimum core length

Table 4 Responses of the 4-story frame under Imperial Valley earthquake record

BRB length (percentage of the brace length)	Time period (sec)	Maximum story drift	Maximum deformation demand of BRB (m)	Dissipated energy (kJ)
99	0.766	0.0130317	0.0483841	383.05
90	0.751	0.0132448	0.0474622	388.01
80	0.734	0.0129655	0.0455549	395.27
70	0.717	0.0129419	0.0454415	403.7
60	0.700	0.0129424	0.0453667	414.78
50	0.682	0.0129557	0.0444504	428.17
40	0.664	0.0120124	0.0405014	441.47
30	0.645	0.0113847	0.0364143	459.35
20	0.626	0.0105376	0.0302922	479.75
11	0.607	0.00953293	0.0225469	502.01
10	0.605	0.3571890	1.4806200	460.95

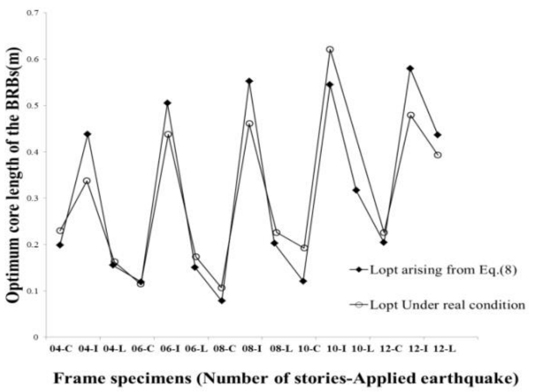


Fig. 6 comparison between numerical and analytical optimum lengths

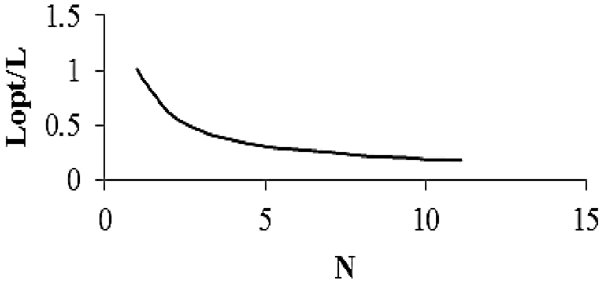


Fig. 7 Dimensionless curve of N-L_{opt}/L

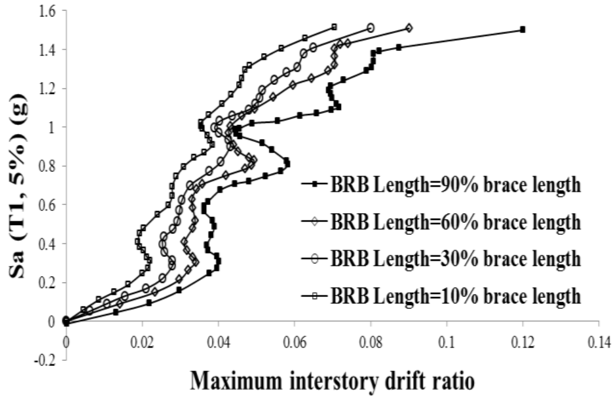


Fig. 8 summarized IDA curves (50% fractiles)

Table 5 Performed and optimum core length of the BRBs

N	1	2	3	4	5	6	7	8	9	10	11
L (%)	4	10	20	30	40	50	60	70	80	90	99
L (m)	0.156	0.390	0.781	1.171	1.562	1.953	2.343	2.734	3.124	3.515	3.866
L_{opt} (m)	0.156	0.239	0.345	0.421	0.468	0.527	0.584	0.601	0.629	0.653	0.684

seismic loading. In this part, the seismic behavior of the frame specimens is examined in terms of these optimum lengths. The optimum lengths were calculated using Eq. (8). The core length of the BRBs and their corresponding optimum length concerning the 4-story frame under the Loma Prieta earthquake record are given in Table 5.

7. Conclusions

In the present study, it has been attempted to investigate the optimum steel core length of BRBs through performing numerical methods. Five 2D finite element models of bracing frames comprising BRB dampers with various lengths were subjected to three earthquake records, and were analyzed to obtain information regarding the optimized state of steel core length of BRBs. Based on the results provided by the numerical studies, the following conclusions may be reached:

• For finding a way to obtain an optimum length for BRB steel cores, the equation offered by Mirtaheri *et al.* (2011) was examined under the real conditions applied in this study. Accordingly, it was concluded that the optimum length arising from the above equation satisfies the limitation of non-failure of the BRBs under seismic loads, and it is perfectly on the safe side. This confirms the validity of the mentioned equation, but it is not adequate, and the relevant studies should be developed to obtain a more accurate approach.

• In order to improve the previous equation proposed by Mirtaheri *et al.* (2011), another equation called MSNN equation was derived in the present paper, which is based on seismically exerted loads. Based on the conducted investigations, the equation has an approximately acceptable accuracy, and can be considered as an accurate equation for to be performed in analysis and design guidelines.

• In order to investigate the behavior of BRBs used in a typical structure, some investigations were made, in which the core length of the BRBs were compared with their corresponding optimum core length. Once again, it was demonstrated that the optimum length obtained in this paper is indeed the best length that can be proposed as the optimum steel core length of BRBs, after which, as core length increases, the structure moves away from its optimum state.

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