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Structural behavior of conventional and buckling restrained braced frames subjected to near-field ground motions

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Abstract. In this study, nonlinear dynamic analyses were performed in order to evaluate and compare the structural response of different type of moment resisting frame buildings equipped with conventional braces (CBs) and buckling restrained braces (BRBs) subjected to near-field ground motions. For this, the case study frames, namely, ordinary moment-resisting frame (OMRF) and special moment-resisting frame (SMRF) having two equal bays of 6 m and a total height of 20 m were utilized. Then, CBs and BRBs were inserted in the bays of the existing frames. As a brace pattern, diagonal type with different configurations were used for the braced frame structures. For the earthquake excitation, artificial pulses equivalent to Northridge and Kobe earthquake records were taken into account. The results in terms of the inter-story drift index, global damage index, base shear, top shear, damage index, and plastification were discussed. The analysis of the results indicated a considerable improvement in the structural performance of the existing frames with the inclusion of conventional and especially buckling-restrained braces.

Keywords: buckling-restrained brace; conventional brace; earthquake; frames; structural response; performance characteristics

1. Introduction

One of the great concerns for engineers is the lateral displacement of structural buildings and to control this lateral displacement, different techniques have been used in the design stages. Among them, the braces as a lateral load resisting system have shown a considerable improvement in that area to resist lateral loads such as an earthquake. However, conventional braces (CBs) exhibit buckling deformation when loaded with large compression forces (Martinelli *et al*. 1998, FEMA-450 2003) and show unsymmetrical hysteresis behavior in tension and compression, and typically the load resisting capacities are reduced when loaded monotonically in compression or cyclically, as shown in Fig. 1 (Qiang 2005, Asgarian and Amirhesari 2008). In order to overcome this problem, many research efforts have been conducted (Wakabayashi *et al*. 1973, Kimura *et al*. 1976, Mochizuki *et al*. 1979, Fujimoto *et al*. 1988, Nagao *et al*. 1988, Black *et al*. 2004, Park *et al*. 2012, Zhao *et al*. 2014) and as a result, a new type of brace called buckling restrained brace (BRB) with a perfect nonlinear behavior such as symmetrical hysteresis behavior, large energy dissipation, dissipation, and significant ductility has been developed by providing lateral

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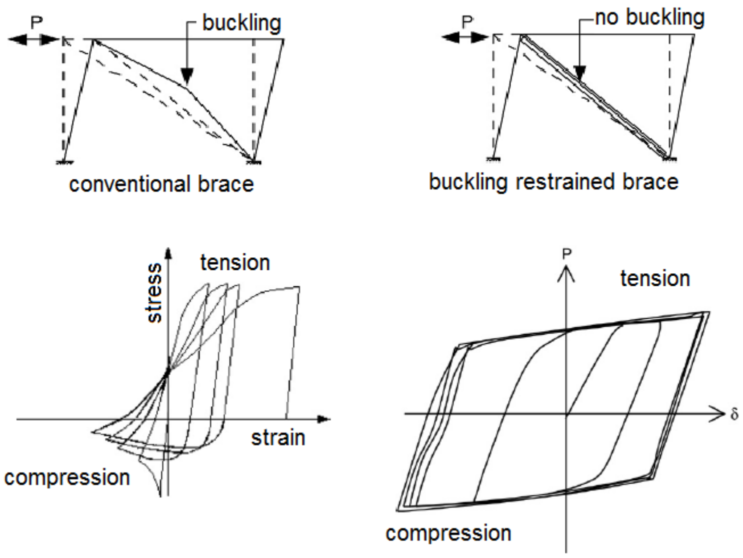


Fig. 1 Behavior of conventional brace and buckling restrained brace (Qiang 2005)

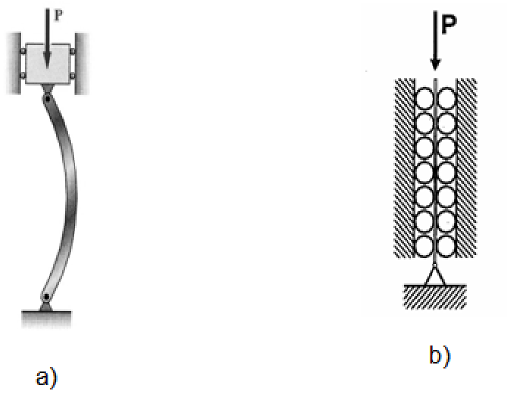


Fig. 2 Schematic diagram of a) conventional brace and b) buckling restrained brace (Hussain *et al*. 2006)

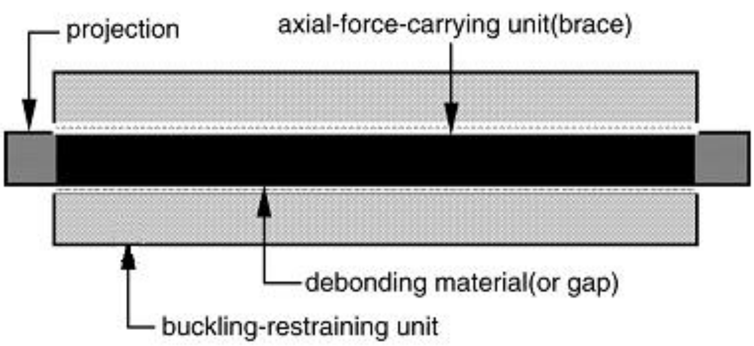
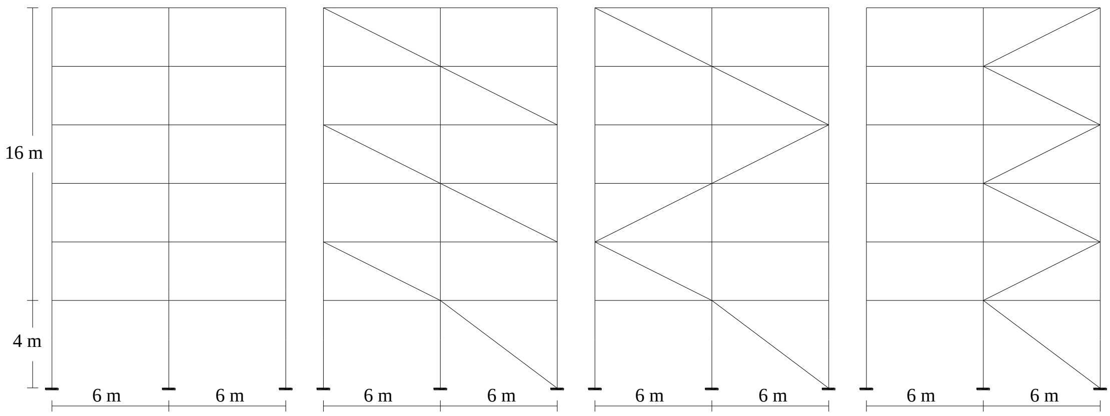


Fig. 3 Composition of typical buckling-restrained brace (Qiang 2005)



(a) (b) (c) (d)

Fig. 4 Elevation view of a) unbraced frame (Tirca *et al*. 2003) and braced frames: b) conf-1,c)conf-2, and d) conf-3 under investigation

support to ordinary braces that prevent buckling deformation, as shown in Fig. 2.

In the design of the buckling restrained braces, the required axial strength and flexural rigidity are taken into consideration. As shown in Fig. 3, the design of buckling restrained braces basically consists of four components such as:i) Steel core member that provides the required axial strength, ii) Projection part connecting the brace and connection, iii) Encasing unit or buckling restraining unit that provides the flexural rigidity and prevent the brace from buckling, and iv) A debonding material or a gap between the brace core and inner filler material, so that the brace can slide easily inside restraining unit (Qiang 2005).

In the literature, several researches have been conducted for evaluating the seismic behavior of conventionally braced frames (CBFs) and/or buckling restrained braced frames (BRBFs) subjected to strong ground motions (Sabelli *et al*. 2003, Kim and Seo 2004,Kim *et al*. 2004, Lee and Bruneau 2005, Kumar *et al*. 2007, Asgarian and Amirhesari 2008, Di Sarno and Elnashai 2009, Mahmoudi and Zaree 2010, Güneyisi 2012, Rezvani and Asgarian 2012, Abdollahzadeh and Banihashemi 2013, Lee *et al*. 2013, Gu *et al*. 2014). For example, Sabelli *et al*. (2003) conducted a research effort on behavior of frames with buckling restrained braces in chevron configuration to identify improved design procedures and code provisions. Asgarian and Amirhesari (2008) presented the results of analytical study carried out to compare the seismic behavior of 4 and 12 story frames with conventional braces and buckling restrained bracesunder strong ground motion. Kumar *et al*. (2007) investigated the effect of tailoring the strength and stiffness of BRBs on the performance of steel frames. DiSarno and Elnashai (2009) performed an analytical study to investigate and compare the seismic performance of special concentrically braced frames (SCBF), buckling restrained braced frame (BRBF), and mega braced frame (MBF). In the study of Güneyisi (2012), by development of fragility curves, the seismic reliability of the steel moment resisting framed building retrofitted with buckling restrained braces were investigated. In the study of Gu *et al*. (2014), the effect of buckling restrained brace constitutive parameters on seismic structural response of a buckling restrained braced non-moment resisting steel frame were examined as a case study. Moreover, they presented the efficiency of usingdirect differentiation method over finite difference method for the derivation of the response sensitivitiesto the material constitutive parameters.However, few studies have investigated the seismic performance of CBFs and/or BRBFs under the effect of near field ground motions (Tirca and Tremblay 2004, Ren *et al*. 2008). For instance, in the study of Tirca and Tremblay (2004), seismic behavior of zipper braced steel frames under three different seismic ground motions: regular crustal earthquakes,near field earthquakes and long duration subduction zone earthquakeswere examined. In the another study forwarded by Ren *et al*. (2008), damage of a high-rise steel frame structure with buckling restrained braces under near field ground motions was evaluated based on fuzzy mathematics theory. Therefore, further analytical researcheswould be beneficial in investigating the effectiveness of conventional and buckling restrained braces under near field ground motions.

Within this context, the main objective of this study is to compare the seismic performance of different type of steel moment resisting framed buildings, equipped with conventional braces (CBs) and buckling-restrained braces (BRBs) subjected to near-field ground motions. For this, six-story ordinary moment resisting frame (OMRF) and special moment resisting frame (SMRF) were selected as case study buildings. Then, CBs and BRBs were introduced in the bays of the existing frames. In the strengthening of the OMRF and SMRF, diagonal braces with three different configurations were taken into account. The performance of the frames were analyzed through nonlinear time history analyses by considering the inter-story drift index, global damage index, base shear, top shear, damage index, and plastificationin the frames.

2. Model definition

The analyzed structural models utilized in this study were two different six story moment resisting steel framed structures. The first one was an ordinary moment resisting frame (OMRF) and the second one was a special moment resisting frame (SMRF) (AISC 2005).They were first designed by Tirca *et al*. (2003) and modified in the current study considering the lateral stiffness insufficient to satisfy code drift limitations. The basic geometry of the unbraced frame (UF) systems consists of two bays with a span length of 6.0 m. The story height is 4.0 m at ground level and 3.2 m in other stories. The frames were assumed to have uniform mass distribution and the columns and beams were built with IPE and HEB profiles, respectively. The column and beam sections varies along the height of the frames.

To evaluate the effectiveness of conventional and buckling restrained braces on the structural behavior of the frames, these braces were inserted to the unbraced frames (UFs) with three different bracing configurations, namely configuration-1 (denoted as conf-1), configuration-2 (denoted as conf-2), and configuration-3 (denoted as conf-3). Fig. 4 shows the elevation view of unbraced and braced framesconsidered. The cross-sectional area of the cores of the BRBs in conf-1of both OMRF and SMRF were designed such that the inter-story index in these frames with buckling restrained braces coincides with the target inter-story drift index for the limit state of life safety. Then, for the purpose of comparison, the same cross-sectional areas were used in the other bracing configurations.

The performance of unbraced frames and various braced frames subjected to near field ground motions was investigated through nonlinear dynamic analyses by using SAP 2000 Nonlinear version 14.0, which is a general purpose structural analysis program (CSI 2009). The columns and beams were modeled with frame element and braces were modeled with nonlinear link (NLLink) element. In modeling, the material nonlinearities of the analytical frames; for the beam and column members lumped plasticity approach, which is characterized by addition of discrete nonlinear

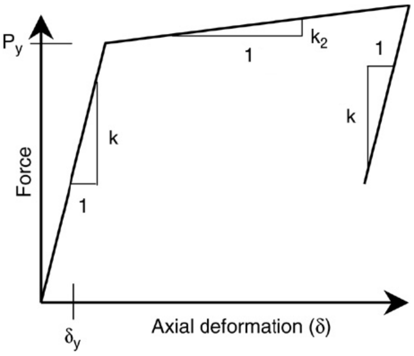


Fig. 5 Constitutive model of BRBs (Kumar *et al*. 2007)

moment rotation hinges at predetermined locations, was used. Thus, the nonlinear behavior of the beam and column members was defined at concentrated plastic hinges,properties of which were defined in accordance with FEMA 273 (1997).

For modeling the nonlinear behavior of BRBs, the nonlinear link members having the elasto-plastic force deformation property as shown in Fig. 5 (Kumar *et al*. 2007) was used. The effective axial stiffness in the elastic and post elastic range was evaluated by using the equations given below:

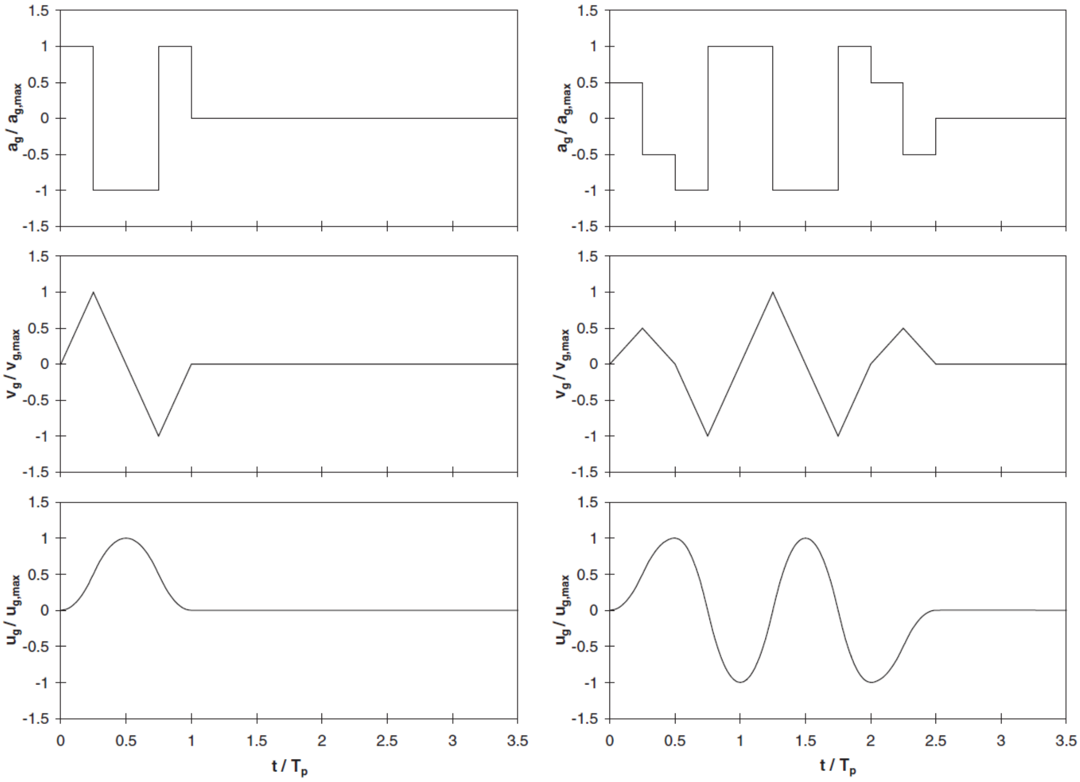
$$k = \frac{AE}{L} \quad \delta < \delta_y \tag{1}$$

$$k_2 = \frac{AE_t}{L} \quad \delta \geq \delta_y \tag{2}$$

where$A$ is the cross-sectional area of the BRB, $E$is the modulus of elasticity, $L$is the length of BRB, $E_t$is the modulus of elasticity of steel after yielding,and$\delta_y = f_y A/k$ (Kumar *et al*. 2007).

3. Near field ground motions

Ordinary earthquake ground motionsrequire smaller demand in comparison to nearfield ground motions. This assumptionis valid especially for the near field ground motion time histories of fault-normal component(Somerville 1998)withforward rupture directivity. In the near field ground motions with forward rupture directivity, fault rupture propagates toward the site at a speed close to the shear wave velocity, andmost of the seismic energy reaches within a short time at the beginning of the earthquake. However, the fault normal component of the ground motions with backward directivity aredescribed by low amplitude pulse, long duration of motion and long period(Somerville*et al*. 1997).As a result, fault normal component of the near field ground motions with a forward rupture directivity, which can be characterized by large amplitude pulses, is distinguished by its capability of causing severe damage to structures.Simulating near-fault ground



a) b)

Fig. 6 Acceleration, velocity, and displacement time histories of the pulses a)P2 and b)P3 (Alavi and Krawinkler 2004))

motion in the forward directivity region with simple pulse modelsin velocity and displacement time history with reasonable accuracy greatly aid the process of analysis and design of the structuressubjected to near field ground motions (Alavi and Krawinkler 2004).

Many pulse shapes have been developed by the researchers, but the three basic pulse shape and commonly used pulses are: half pulse (P1), full pulse (P2), and multiple pulse (P3) (Alavi and Krawinkler 2004). In Fig. 6, the acceleration, velocity, and displacement time history for the pulse P2 and P3 are illustrated. The pulse P1 which is not presented in the Fig. 6 is the first half of pulse P2. These artificial pulses are defined by two parameters: the pulse period T_p and the intensity of the earthquake that is either the maximum ground acceleration or the maximum ground velocity. These parameters are related to each other as seen in the following equation(Alavi and Krawinkler 2004):

$$v_{gmax} = a_{gmax}\frac{T_p}{4} \tag{3}$$

In the study of Tirca *et al*. (2003), the authors showed that the equivalent pulse P2 adequately represented the Northridge and Kobe near field earthquake motions.Therefore, in the present study, the same artificial spectra generated by Tirca *et al*. (2003) were employedby utilizing pulse shape P2 with the maximum ground acceleration of 0.4g and a pulse period (T_p) of 1.4s and 0.9s to simulate the Northridge and Kobe earthquakes, respectively.

4. Results and discussion

In this section, the results for unbraced frames (UFs), conventional braced frames (CBFs), and buckling restrained braced frames (BRBFs) obtained from nonlinear dynamic analyses are given and discussed comparatively. In the present study, a total of 28 different cases were taken into account and the structural performance of unbraced and braced frame systems having different type of brace and configuration under the effect of near field ground motions were evaluated.

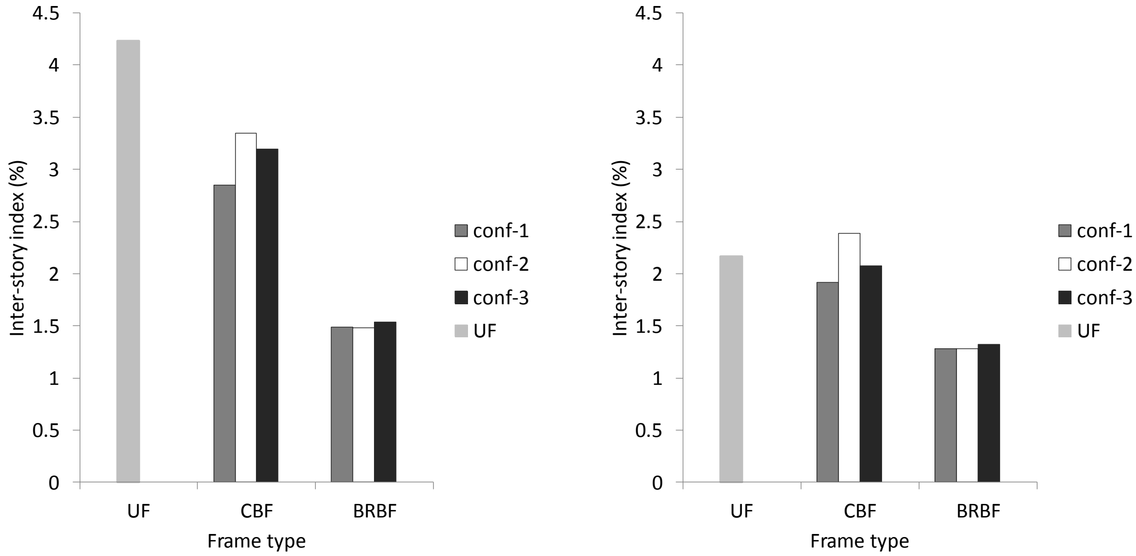
4.1 Inter-story index

The maximum inter-story drift divided by the story height is defined as the maximum inter-story index. In this study, the acceleration pulses representing Northridge and Kobe near field ground motions were scaled at 0.4g considering high seismic effects. The maximum inter-story index of both OMRF and SMRF with and without CBs and BRBs subjected to equivalent pulses to Northridge and Kobe earthquakes were assessed. The maximum inter-story index for CBFs and BRBFs with different brace configuration andframe type under pulse accelerations with different pulse periods are given in Figs.7-8. Comparison of the maximum inter-story index of the frames indicated that this index obtained for CBFs was considerably greater than that for BRBFs. According to SEAOC (1999), for Life Safety (LS) performance state, the inter-story index limit is recommended as 1.5%. Asseen in the response plots, both unprotected and CBFs did not meet SEAOC limitations and in some case, itwas even two times higher than the limit (i.e., OMRF-CB subjected to pulse with T_p=1.4 s). Furthermore, generally a better performance was observed in CBFs and especially BRBFs compared to unbraced frames. Since the structural response of the frames were sensitive to the pulse period value T_p, for all types of frames,the maximum inter-story indexes obtained under the earthquake excitations with a high pulse period were greater than the ones obtained under earthquake excitations with low pulse period.

It was also observed from the figures that there was a slight difference between the inter-story indexes of the frames with different configurations. However, some brace configurations performedbetter than the others. For example, the inter-story index in the braces in the second configuration (conf-2) wasgreater compared to the other configurations (conf-1 and conf-3). This holds particularly true in the case of CBFs due to thedifferent load carrying capacity of the conventional bracesin compression and tension. However, the differences in the inter-story index for different configurations in the case of BRBFs were very small since BRBs had the same load carrying capacity in both compression and tension.

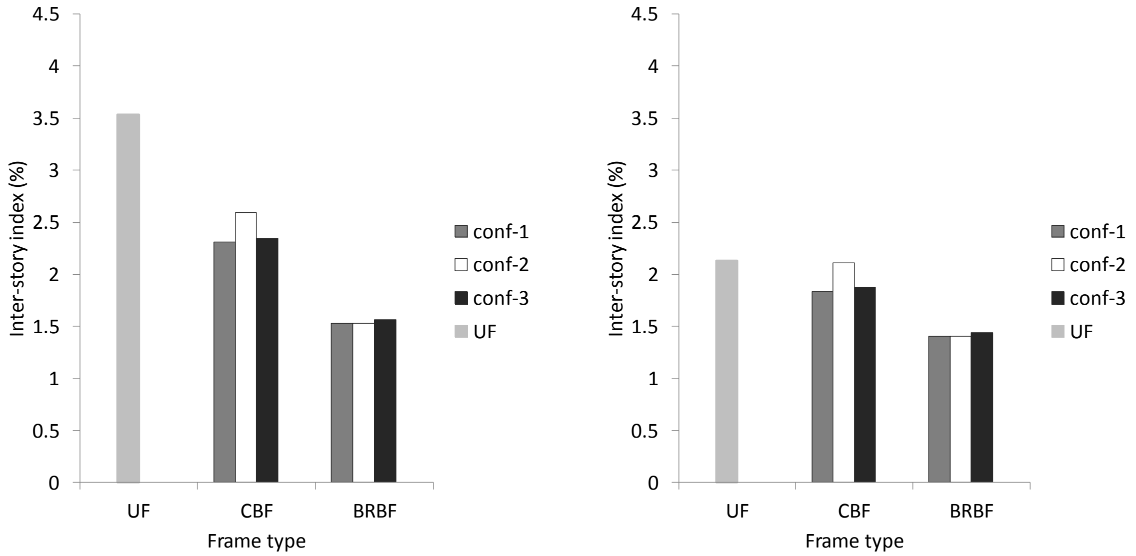
4.2 Global damage index

The ratio of the roof displacement over the total height of the building is defined as the global damage index. The global damage index was assessed for both CBFs and BRBFs subjected to artificial acceleration pulses representing Northridge and Kobe earthquakes with pulse period of



a) b)

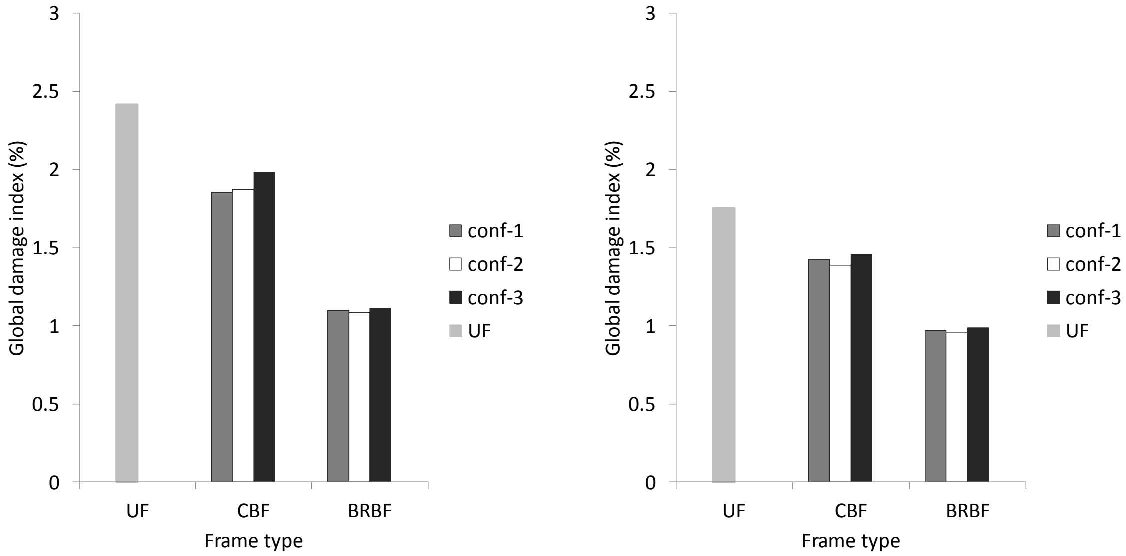
Fig. 7 The maximum inter-story index for OMRF under pulseswith a)T_p=1.4 s and b) T_p=0.9 s



a) b)

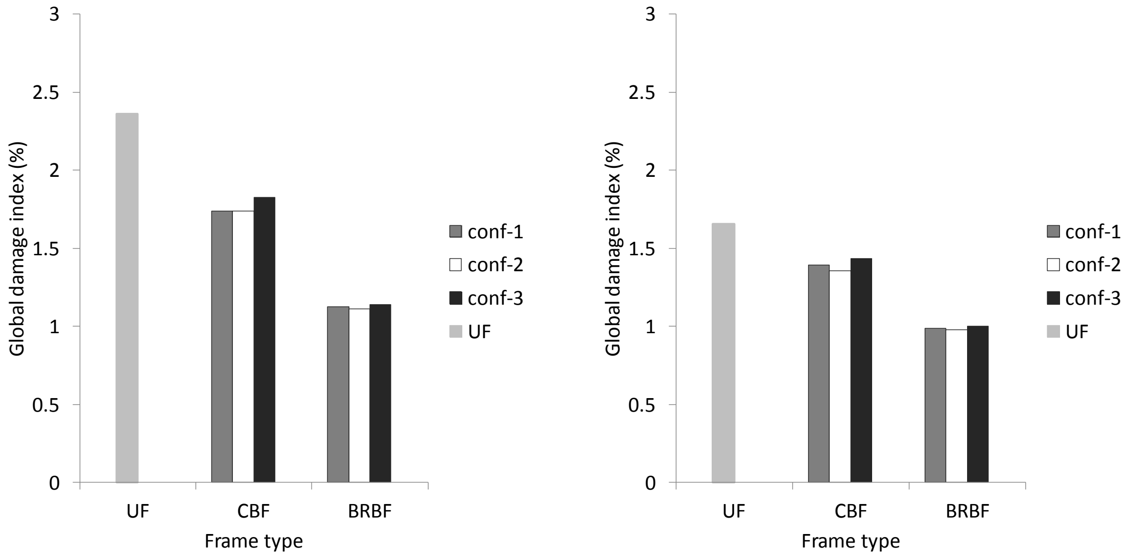
Fig. 8 The maximum inter-story index for SMRFunder pulses with a) T_p=1.4 s and b) T_p=0.9 s

1.4s and 0.9s, respectively. Figs.9-10 compare the global damage index obtained for CBFs and BRBFs with different configurationof braces. Comparison of global damage index of the frames revealed that the global index for the frames with CBs was greater than that for the ones with BRBs, and the brace with CBs showed better performance in comparison to the unbraced frames. The use of CBs resulted in reductions of 15-25%. However, the use of BRBs resulted in further



a) b)

Fig. 9 The global damage index for OMRF under pulses with a) T_p=1.4 s and b) T_p=0.9 s



a) b)

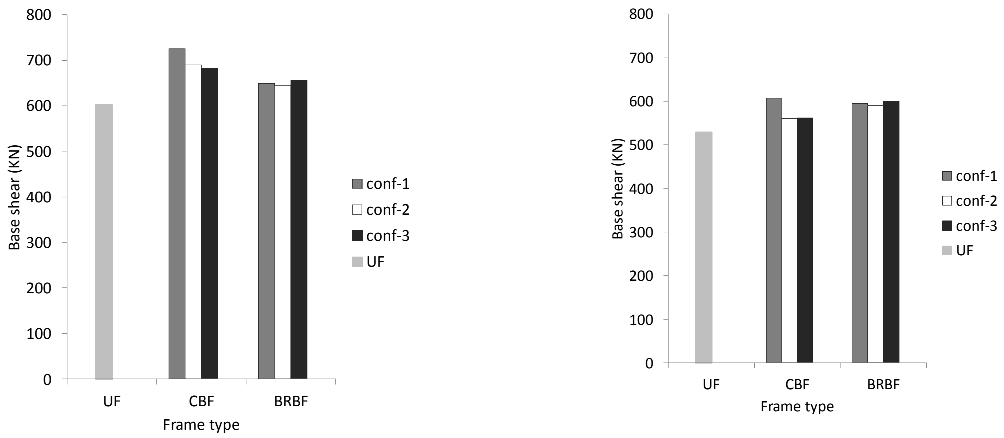
Fig. 10 The global damage index for SMRF under pulses with a) T_p=1.4 s and b) T_p=0.9 s

reductions of 39-54%. It was also observed that these global deformations depend mainly upon the characteristics of earthquake ground motions, especially frequency content.

Similar to the inter-story index results, because of the sensitivity of structural response to the pulse periodT_p, as the pulse periods become larger, greater global damage index was introduced.From these results, it was pointed out that the various configurations had a small effect on global performance of the structures, especially in the case of BRBs. That was due to similar behavior of BRBs in both compression and tension. However, due to distinct load carrying capacity of CBs in compression and tension, the difference in the global damage index was more noticeable among the frames with CBs.

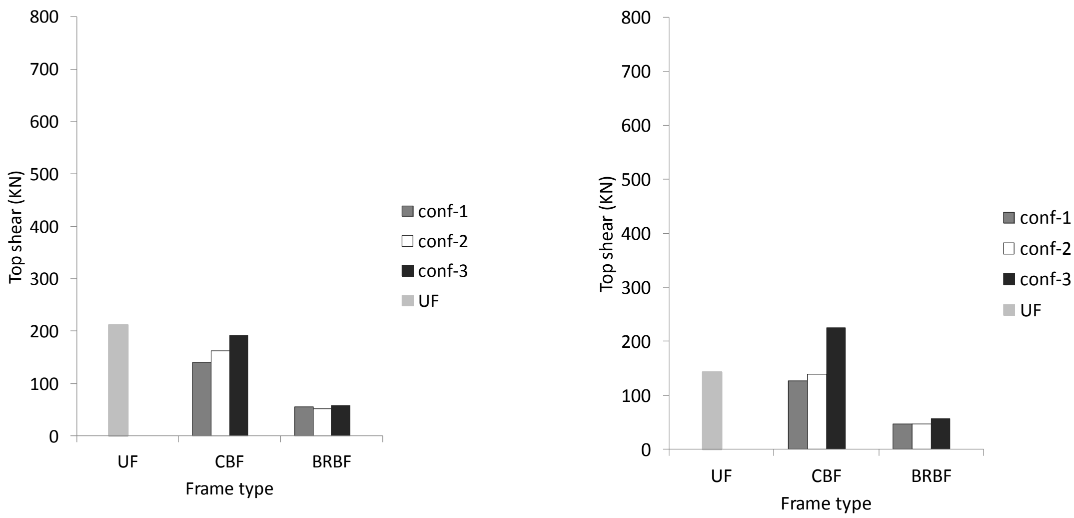
4.3 Base shear and top shear

The base shear and top shear were evaluated for both OMRF and SMRF with CBs and BRBsunderpulse accelerations equivalent to Northridge and Kobe earthquakes. The base shear and top shear attained for these frames are shown in Figs. 11-14.



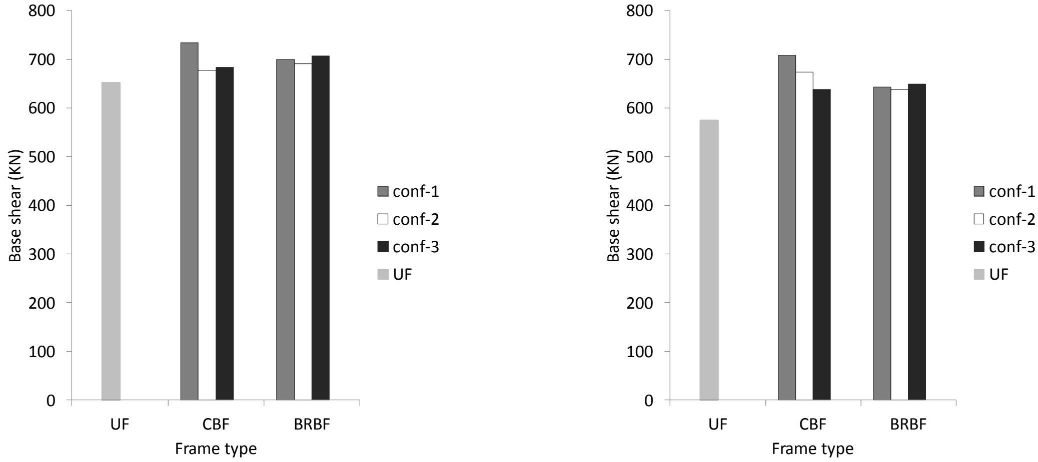
a) b)

Fig. 11 The base shear for OMRF under pulses with a) T_p=1.4 s and b) T_p=0.9 s



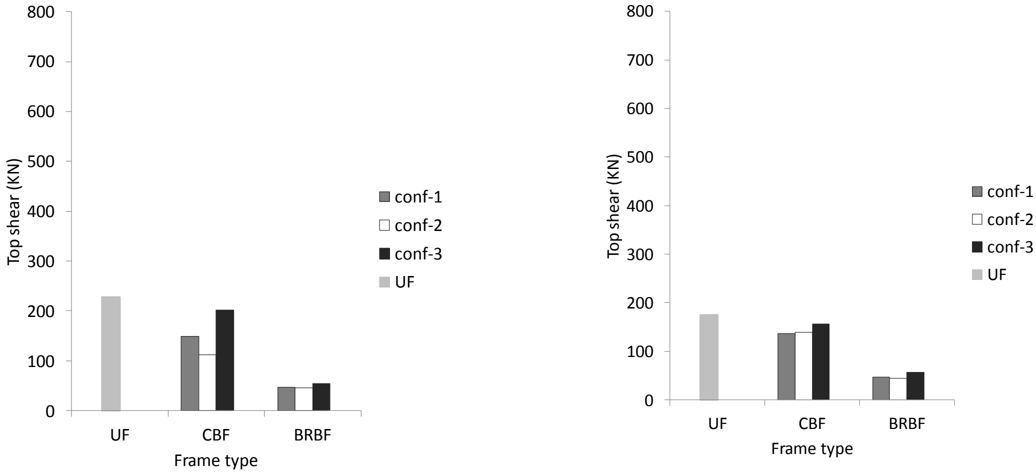
a) b)

Fig. 12 The top shear for OMRF under pulses with a) T_p=1.4 s and b) T_p=0.9 s



a) b)

Fig. 13 The base shear of SMRF under pulses with a) T_p=1.4 s and b) T_p=0.9 s



a) b)

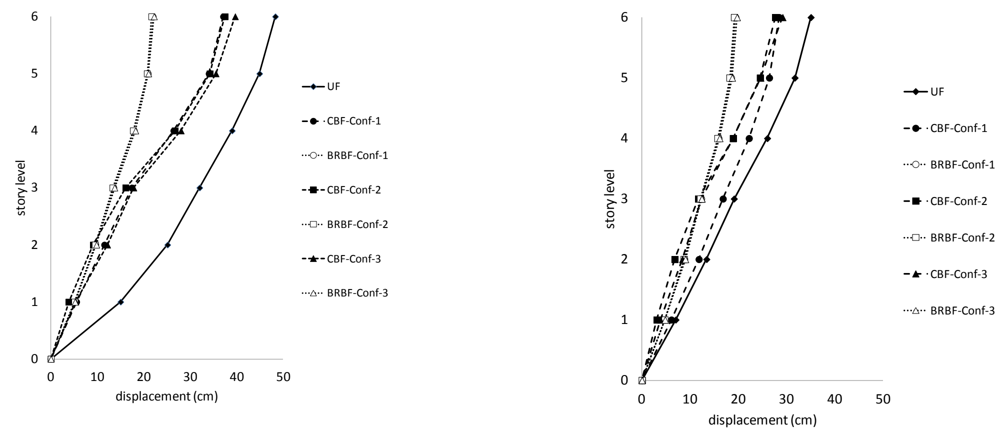
Fig. 14 The top shear of SMRF under pulses with a) T_p=1.4 s and T_p=0.9 s

Strength and stiffness characteristics of the frames, earthquake characteristics such as peak ground acceleration, earthquake type affect the variation of base shear forces. Position of the fundamental period of the frame with respect to earthquake acceleration spectrum defines this variation in the elastic stage. Furthermore, the base shear and top shear distribution are affected significantly by the ratio of the fundamental period of vibration of the frame to the pulse period. Putting the severity of pulse P2 with T_p=1.4 s in perspective with T_p=0.9 s;because of the value of the velocitypulseperiod, larger base shear was observed in the former pulse.

The total base shear increased in the presence of braces, but the columns were not influenced so much by this increment, because most of the shear forces were supported by the braces. As explained in detail in the section 4.6. Plastification in the frames, most of the plastic hinges was formed in the braces.However, in the case of CBFs, some plastic hinges were developed in the columns because when conventional braces exhibited buckling deformation their strength was degraded and then most of the forces required to be supported by the columns. Furthermore, the total top shear was decreased in the presence of braces. However, for some cases (i.e., OMRF-CB), itwas even increased.

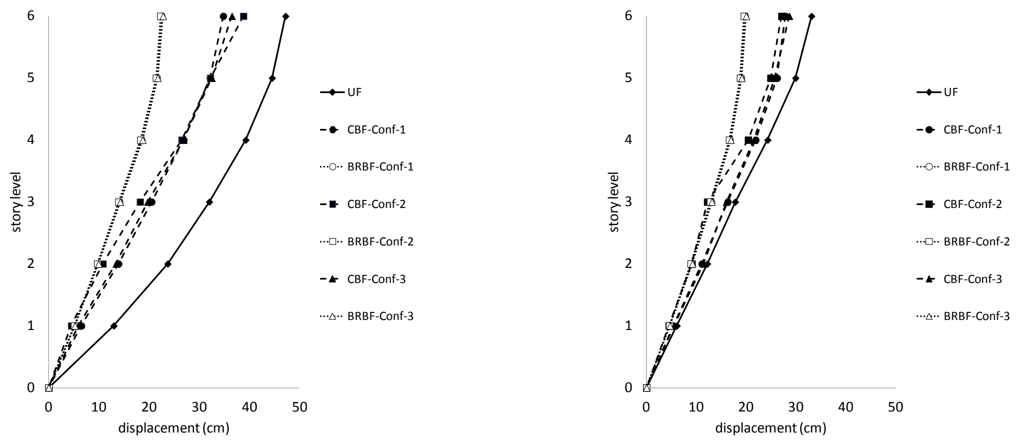
4.4 Variation of story displacement

Figs. 15-16 show the deflected shape of OMRFs and SMRFs at various circumstances at the instance corresponding to the maximum roof displacement. Both BRB and CB considerably



a) b)

Fig. 15 The deflected shape at the maximum roof displacement for OMRF under pulse with a)T_p=1.4 s and b) T_p=0.9 s



a) b)

Fig. 16 The deflected shape at the maximum roof displacement for SMRF under pulse with a)T_p=1.4 s and b) T_p=0.9 s

Table 1 Effective damage index (D_{eff}) of bracings

Sample no.	Designation of frame	Compression			Tension		
		Story	Damage index	LS limit	Story	Damage index	LS limit
1	OMRF-Tp0.9-BRB-Conf-1	1	0.45	0.67	4	0.39	0.67
2	OMRF-Tp0.9-BRB-Conf-2	1	0.45	0.67	4	0.41	0.67
3	OMRF-Tp0.9-BRB-Conf-3	1	0.45	0.67	2	0.43	0.67
4	OMRF-Tp0.9-CB-Conf-1	1	7.11	2.00	4	0.63	0.67
5	OMRF-Tp0.9-CB-Conf-2	4	8.96	3.92	5	0.56	0.67
6	OMRF-Tp0.9-CB-Conf-3	4	7.72	3.92	5	0.59	0.67
7	OMRF-Tp1.4-BRB-Conf-1	1	0.51	0.67	4	0.48	0.67
8	OMRF-Tp1.4-BRB-Conf-2	1	0.50	0.67	4	0.46	0.67
9	OMRF-Tp1.4-BRB-Conf-3	1	0.51	0.67	2	0.47	0.67
10	OMRF-Tp1.4-CB-Conf-1	1	10.49	2.00	4	0.94	0.67
11	OMRF-Tp1.4-CB-Conf-2	4	12.60	3.92	5	0.78	0.67
12	OMRF-Tp1.4-CB-Conf-3	4	11.95	3.92	5	0.76	0.67
13	SMRF-Tp0.9-BRB-Conf-1	1	0.48	0.67	1	0.43	0.67
14	SMRF-Tp0.9-BRB-Conf-2	1	0.48	0.67	1	0.42	0.67
15	SMRF-Tp0.9-BRB-Conf-3	1	0.48	0.67	2	0.46	0.67
16	SMRF-Tp0.9-CB- Conf-1	1	6.91	3.25	1	0.56	0.67
17	SMRF-Tp0.9-CB- Conf-2	4	7.95	3.11	2	0.50	0.67
18	SMRF-Tp0.9-CB- Conf-3	2	7.03	5.00	2	0.54	0.67
19	SMRF-Tp1.4-BRB- Conf-1	1	0.53	0.67	2	0.45	0.67
20	SMRF-Tp1.4-BRB- Conf-2	1	0.52	0.67	4	0.45	0.67
21	SMRF-Tp1.4-BRB- Conf-3	1	0.53	0.67	2	0.50	0.67
22	SMRF-Tp1.4-CB- Conf-1	1	8.43	3.25	2	0.77	0.67
23	SMRF-Tp1.4-CB- Conf-2	4	9.78	3.11	2	0.65	0.67
24	SMRF-Tp1.4-CB- Conf-3	1	9.27	3.25	3	0.64	0.67

decreased the value of the maximum roof displacement and corresponding story displacement compared to unbraced frames, especially in the case of BRBs, more uniform response of the frame along the height of the structure was observed. Moreover, it was pointed out that the differences between the performance of BRB and CB frames was much more apparent under ground motion with greater pulse period (T_p=1.4 s).

Similarly, the difference in performance of BRBFs and CBFs was less noticeable in ground motions with low pulse period (i.e., T_p=0.9 s) since both CBs and BRBs having the same initial elastic stiffness provided similar behavior in the elastic stage. However, during severe earthquake excitations, most of the structures were expected to undergo inelastic deformation such as in the case of ground motion with high pulse period (i.e., T_p=1.4 s) and the difference in the performance of BRBFs and CBFs became obvious due to the buckling deformation and strength degradation exhibited by CBs.

4.5 Variation of damage index

The damage index is a parameter for indication of damage state of a structure under seismic effects. To describe the state of damage that can be observed in a structure by a single value on a determined scale in the form of a damage index is attractive, and in the literature there are various types ofdamage indexes based on response considering the maximum deformation, cumulative damage or both of them (Ghobarah *et al.* 1999). In this study, the ductility damage index for the braces was used. The ductility damage index (D) was defined as given in the equation below

(Kumar *et al*. 2007):

$$D = \frac{\varepsilon_{max}}{\varepsilon_u} \tag{4}$$

For example, for BRBs, conservatively the ultimate strain was taken as $12\varepsilon_y$and the maximum strain for the calculation of damage index was taken as $12\varepsilon_y$,$10\varepsilon_y$, $8\varepsilon_y$ and $\varepsilon_y$which corresponds to damage index value of 1.0, 0.83, 0.67, 0.083 for ultimate failure, collapse prevention (CP), life safety (LS),and immediate occupancy (IO) performance states, respectively(Kumar *et al*. 2007, FEMA273 1997).

Since a story collapse is considered as a global collapse of the frame, the effective damage index of the frame, D_{eff}, was taken as the maximum of D_i (the damage index of the ith story) as given in the equation below (Kumar *et al*. 2007):

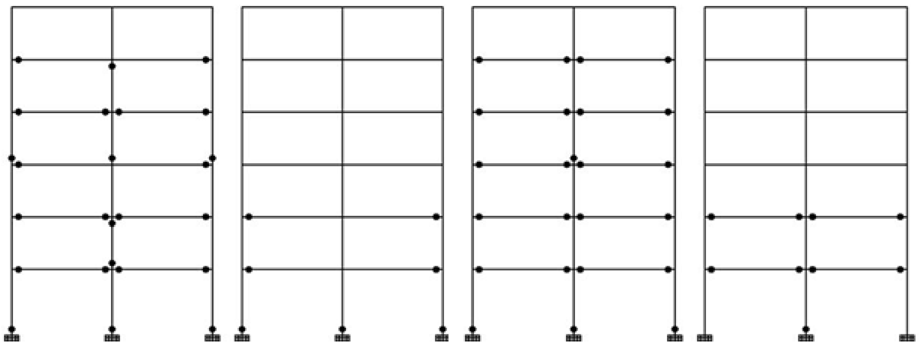
$$D_{eff} = maximum\ of\ (D_i) \tag{5}$$

Comparing the value of effective damage indexes for both CBs and BRBs given in Table 1, it was noticed that in both tension and compression, this index obtained was greater for the formerthan later, and they exceeded the admissible value especially when they were under compression loading. This may be explained as CBs typically buckled under compression force and exhibitedlarge stiffness and strength degradation when loaded cyclically or monotonically in compression. However, due to the improved nonlinear behavior of BRBs, the damage indexes for the frames with BRBs were within the limit.

4. 6 Plastification in the frames

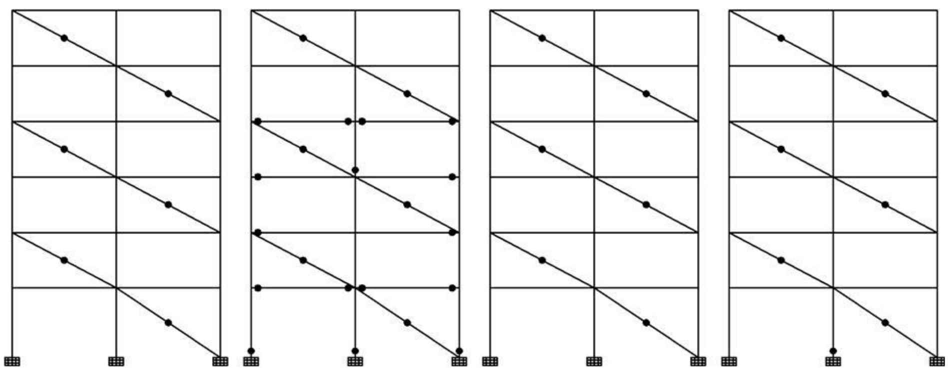
From the nonlinear dynamic analysis, the location of plastic hinges forthe frames equipped with CBs and BRBs were evaluated. The plastification observed in the OMRF and SMRF systems under artificial pulses simulating Kobe and Northridge earthquakes are given in Figs. 17-23. Comparing the location and number of the plastic hinges given in the figures, it was observed that due to buckling and then strength and stiffness deterioration of the CBs, most of the critical forces transmitted to the structural members, and they entered the inelastic range of deformation. On contrary, in the case of BRBs, because of their perfect nonlinear behavior and absorbing more energy in the inelastic range, most of the structural members remained in the elastic range and plastic hinges were concentrated in the BRBs, which could be easily replaced after the earthquake. Due to the sensitivity of the structural response with the pulse period value T_p, the ground motions with a smaller pulse period T_p resulted in less damage.

As shown from the figures, it was pointed out that damage in the case of pulse with T_p=1.4 s was more compared to the pulse with T_p=0.9 s. In addition to these, when the performance of SMRFs with OMRFs were compared, it was observed that SMRFs were performing better than OMRFs. In the case of SMRFs, most of the plastic hinges were concentrated in the beam elements, and most of the columns remained in the elastic stage; however, in the case of OMRFs, many column elements entered the inelastic range that might lead to catastrophic failure.



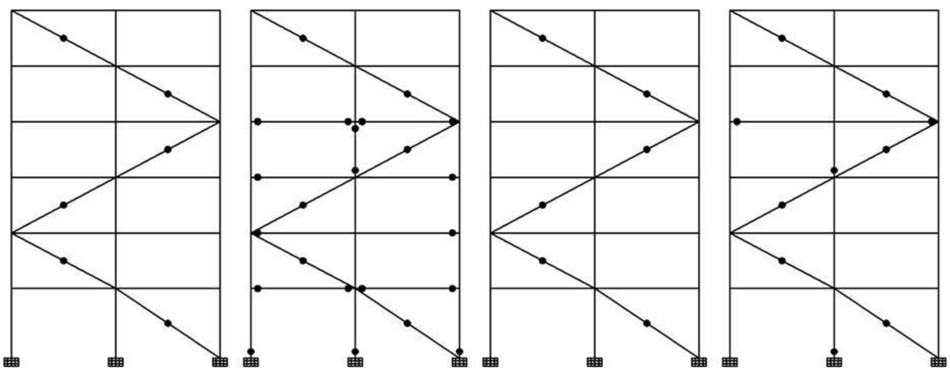
a) b) c) d)

Fig. 17 The plastic hinge formations for OMRFsunder pulses with a) T_p=1.4s, b) T_p=0.9s and for SMRFsunder pulses with c)T_p=1.4s and d) T_p=0.9 s



a) b) c) d)

Fig. 18 The plastic hinge formation for OMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-1)



a) b) c) d)

Fig. 19 The plastic hinge formation for OMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-2)

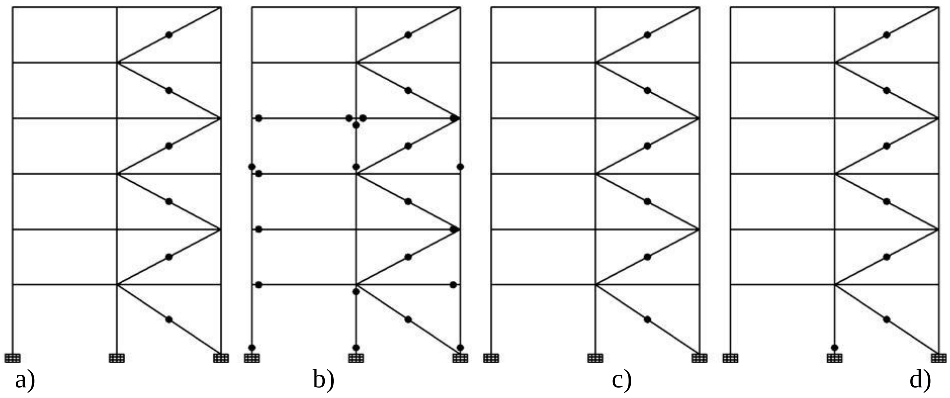


Fig. 20 The plastic hinge formation for OMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-3)

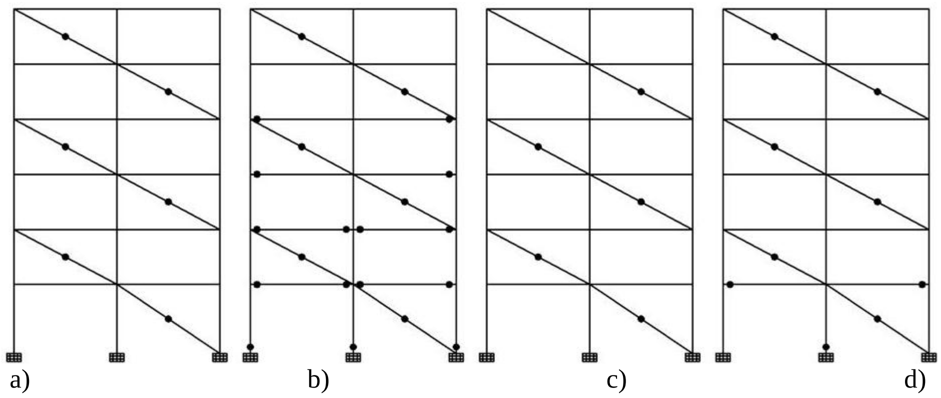


Fig. 21 The plastic hinge formation for SMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-1)

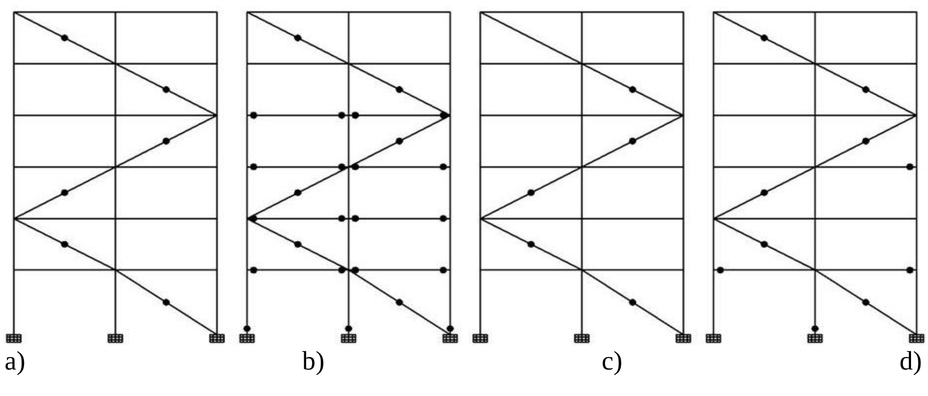


Fig. 22 The plastic hinge formation for SMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-2)

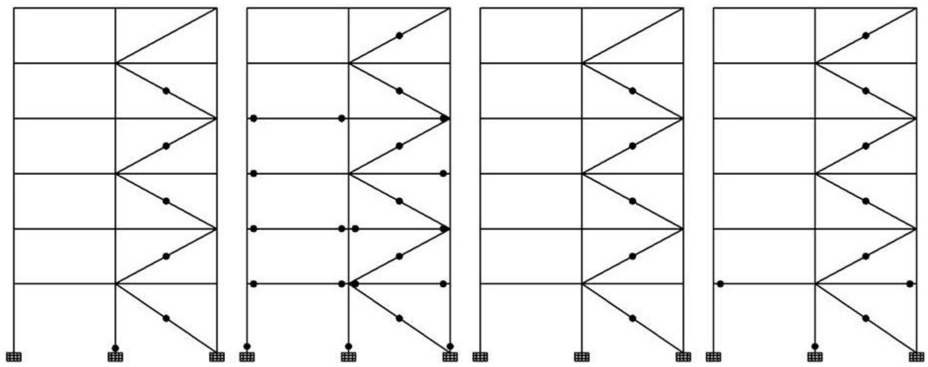


Fig. 23 The plastic hinge formation for SMRFs with a) BRB and b)CB under pulses with T_p=1.4s; c)BRB and d) CB under pulses with T_p=0.9s (conf-3)

5. Conclusions

The analytical study described herein investigated the seismic performance assessment of different type of steel moment resisting frame buildings (OMRFs and SMRFs) equipped with diagonal conventional braces (CBs) and buckling restrained braces (BRBs) subjected to near-field ground motions. By comparing the analysis and results presented in this paper, the following conclusions can be drawn:

- Comparing the performance of SMRFs with OMRFs, it was observed that SMRFs were performing better than OMRFs.In the case of SMRFs, most of the plastic hinges were concentrated in the beam elements, and most of the columns remained in the elastic stage. However, in the case of OMRFs, many column elements entered the inelastic range of deformation that might result in a failure.
- BRBFs provided smaller inter-story drift index compared to CBFs. The results of the nonlinear dynamic analysis indicated that BRBs were more effective since the reduction of the inter-story drifts with respect to the original frames was on average equal to 50%. However, in the case of CBFs, this reduction wasabout 13%.The effect of the brace configuration was pronounced in the response of CBFs whereas in the BRBFs close response for all configurations was observed.
- BRBs dissipated more energy as compared to CBs due to their nonbucklingbehavior, which provides high cyclic ductility and a symmetric response under compression and tension forces.On contrary, due to the buckling of the conventional braces before reaching the maximum yield strength of the brace, the energy dissipated by CBs decreased and unsymmetrical hysteresis behavior developed.
- The effective damage index for CBFs was found to be considerably higher than that for BRBFs. Due to the enhanced nonlinear response of BRBs, the damage index for the frames with BRBs were generally satisfied the life safety performance limit.
- The use of BRBs in strengthening the existing frames resulted in more uniform response along the height of the structure. Moreover, with the use of BRBs, the frames remained generally in the elastic stage and plastification occurred in the BRBswhich might be easily replaced after the

earthquake. Unlikely, the utilization of CBs resulted in inelastic deformations not only in the braces, but also in the other structural members.

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