

---

Article

# Steel Frames Exposed to Severe Ground Motions: Use of Viscous Dampers and Buckling Restrained Braces to Dissipate Earthquake Induced Energy

Osman Hansu<sup>1,\*</sup>, Esra Mete Güneysi<sup>2</sup>

<sup>1</sup> Department of Civil Eng., Gaziantep University, 27310, Gaziantep, Turkey; ohansu@gantep.edu.tr

<sup>2</sup> Department of Civil Eng., Gaziantep University, 27310, Gaziantep, Turkey; eguneyisi@gantep.edu.tr

\* Correspondence: ohansu@gantep.edu.tr

**Abstract:** This study addresses an alternative use of viscous dampers (VDs) associated with buckling restrained braces (BRBs) as the innovative seismic protection devices. For this, 4, 8 and 12 storey steel frames were designed with 6.5 m equal span length and 4 m storey height. Thereafter, the VDs and BRBs were placed over the height of each frame considering three different configurations. The structures were modeled using SAP2000 finite element program and evaluated by the nonlinear time history analyses subjected to the six natural accelerograms (1976 Gazlı, 1978 Tabas, 1987 Superstition Hills, 1992 Cape Mendocino, 1994 Northridge and 1999 Chi-Chi). The structural response of the structures with and without VDs and BRBs were studied in terms of variation in the displacement, interstorey drift, absolute acceleration, maximum base shear, time history of roof displacement. The results clearly indicated that the application of VDs and BRBs had remarkable improvement in the earthquake performance of the case study frames by reducing the local/global deformations in the main structural systems and satisfied the serviceability.

**Keywords:** Innovative systems; Ground motion; Steel Frame; Nonlinear analysis; Viscous damper.

---

## 1. Introduction

Recent earthquake regulations in the world have promoted the structural design with sufficient ductility in the earthquake prone zones. Thus, the ductile design of structures becomes the main goal of engineers to dissipate the earthquake induced energy without permanent damage or overall collapse. Earthquake forces generate stresses that need to be resisted by the frames in the buildings. When a structure undergoes strong ground motions, the conventional frames assume substantial lateral deformations so that structural and non-structural damages occur compromising the structural integrity. Steel braces are integrated within the frames to prevent such failures in the steel structures [1–5]. Even though they seem to improve the lateral stiffness of the structures, steel braces have very limited ductility under cyclic tension and compression. After buckling in the concentrically braced frames, they have unsymmetrical hysteresis behavior associated with substantial strength degradation, thus being unable to dissipate the earthquake energy [6,7].

With the advances in the engineering technology, the conventional concentrically braced systems have been replaced by more innovative remedies. Buckling restrained braces (BRBs) have been developed to overcome buckling of the traditional braces and they have been proposed for use in seismic protection [8-10]. Similarly, the viscous dampers (VDs) have been utilized as a passive energy dissipating device for seismic protection of the structures. Both BRBs and VDs are capable of controlling the deformations developed in steel frames by dissipating the energy or increasing the stiffness. Hence, they have been involved in improving the seismic performance of the overall structures [11-14].

BRBs are designed so that no buckling occurs on the bracing provided by a sufficient lateral support. In contrast to the conventional braces, they exhibit more stable and symmetrical hysteretic behavior throughout the floor height under cyclic compression and tension generated by the earthquakes [15-20]. Because of such superior properties, BRBs have been the subject of various studies in the literature. For example, Kumar et al. [4] conducted a parametric study to compare the seismic behavior of moment resisting and non-moment resisting frames designed with non-buckling bracing systems. Lin et al. [21] evaluated the seismic performance of eccentrically braced frames and buckling restrained braced frames in comparison with that of ordinary moment resisting frames. Deulkar et al. [22] searched the effects of BRB with varying length and core area on the design of five storey steel frames. Both of the variables studied seemed to be quite effective on type of the braces. Moreover, Asgarian and Amirhesari [7] compared the seismic behavior of conventionally braced and buckling restrained braced frames with three bays. Braces in the form of split X, inverted V, chevron V, and diagonal compression were placed within the inner bay only. A better performance was observed for the frames with inverted-V buckling restrained braces. In the study of Di Sarno and Elnashai [23], the seismic performances of special concentrically braced frame, buckling restrained braced frame, and mega braced frame were investigated. It was reported that the mega bracing was the most cost-effective bracing systems.

The use of VDs in the seismic protection of the structures relies on a general approach in decreasing the negative effects of the earthquakes. Its primary function is to reduce the structural response by dissipating the energy within the dampers, thus the potential damage in the framing system decreases significantly during an earthquake [24,25]. They are integrated to the frames in different positions so that the energy dissipation is ensured all over the structure. Utilizing VDs as a seismic protection device has been investigated previously in some researches in the literature. For example, Chang et al. [26] evaluated the performance of three-storey steel frames with and without VDs by shaking table tests. They found that VDs could effectively reduce the structural response and ductility demand. Xu [27] also conducted shaking table tests to monitor the dynamic response of a 1/5 scale reinforced concrete structure with VDs. In the study of Xu et al. [28], seismic performance of the eight-storey reinforced concrete frame with VDs was investigated by using numerical simulations. Dicleli and Mehta [13] compared the seismic performance of steel chevron braced frames (CBFs) with and without viscous dampers as a function of intensity and ground motion characteristics. Multi-storey steel frames with a single bay were analyzed to report that the poor seismic performance of the original frames significantly improved by installing VDs into CBFs by maintaining their elastic behavior. Moreover, SaiChethan et al. [29] carried out a numerical assessment of a twenty storey reinforced concrete building. The results confirmed that a significant reduction in the responses such as displacements would be possible with the introduction of fluid VDs. Prasad and Mazumder [30] investigated the seismic response of a set of steel buildings with and without VDs. They were installed within the inner bay for the energy dissipation. VDs were reported to reduce the displacements generated by the seismic loads, which in turn decreased the amount of steel needed for the overall stability. The study conducted by Balkanlou et al. [31] demonstrated that the structure with dampers could be designed optimally to justify the cost spent for the use of dampers.

Previous studies on the BRBs and VDs have focused on their uses in the form of split X, inverted V, concentric, etc by generally overlooking the position of them within the frames. As mentioned above, BRBs or VDs were generally installed within the inner bays of the model frames. Moreover, the comparative assessment of them under similar conditions has not taken adequate attention. In order to justify the benefit of using BRBs and VDs in seismic protection, more researches on the modeling approach of such systems seem to be necessary. Considering this fact, a comparative study on the use of VDs and BRBs installed within the outer-bays, inner-bays, or all-bays of the steel structures with different heights is presented in this paper. For this, the contributions of the BRBs and

VDs in improving the seismic performance of the structures were examined by means of the nonlinear time history analysis under different ground motions. The response parameters such as storey displacement, interstorey drift ratio (IDR), roof drift ratio (RDR), acceleration on the storey, base shear, and time history of roof displacement were evaluated and discussed accordingly.

## 2. 2. Details of Steel Frames, Modeling and Analysis

In this parametric study, a set of three different frames are employed to investigate the seismic protection capability of BRBs and VDs. For this, 4, 8, and 12-storey bare structures have been designed as steel moment resisting frames (MRFs) in accordance with Eurocode-3 and Eurocode-8 [32,33]. The storey height and bay width of the frames are equal to 4 and 6.5 m, respectively. The bare structures have columns of HEA sections and beams of IPE sections. The fundamental periods of 4, 8, and 12-storey frames were obtained as 0.64, 1.15, and 1.54 s, respectively. Fig. 1 presents the elevation views of bare frames. Moreover, the placement of the BRBs and VDs throughout the each frame elevation is given in Fig. 1.

The gravity loading on the floor system is considered as the additional dead loads of 2 kN/m<sup>2</sup> and live loads of 3 kN/m<sup>2</sup>. The yield stress and the post yield stiffness ratio of the steel material are assumed to be equal to 235 MPa and 0.03, respectively [34,35]. The lumped plasticity approach is followed for the nonlinear behavior of the frames. Therefore, the nonlinearity considered in the numerical models of the beams and columns are restricted by the plastic hinges assigned at the end of the frame members as described in FEMA-356 [36]. A series of nonlinear time-history analyses have been performed using SAP 2000 [37] via direct integration method with the P-Delta effect included. Mass-and-stiffness proportional damping is also taken into account in the analysis with 5% damping ratio.

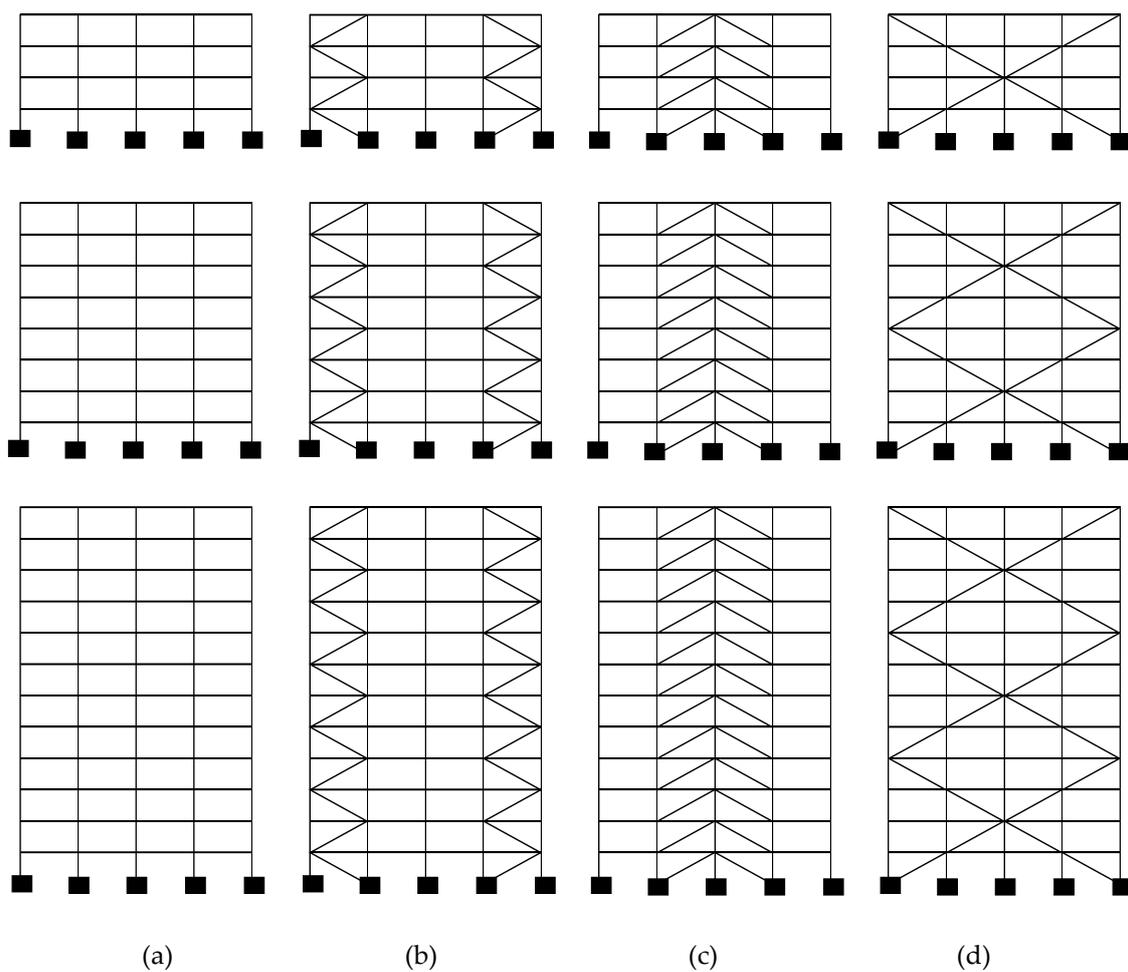
A detailed view of buckling restrained brace (BRB) section used in this study is given in Fig. 2 [4]. The structural elements under the earthquake load have been ensured to remain within the total load core area to which they are exposed. The non-prismatic axial rigidity equations are considered to be elastic and post-elasticity as  $k$  and  $k_2$ , respectively [4].

$$k = \frac{A \times E}{l \times \left( \frac{\beta}{\alpha} + (1 - \beta) \right)} \text{ for } \delta < \delta_y \quad (1)$$

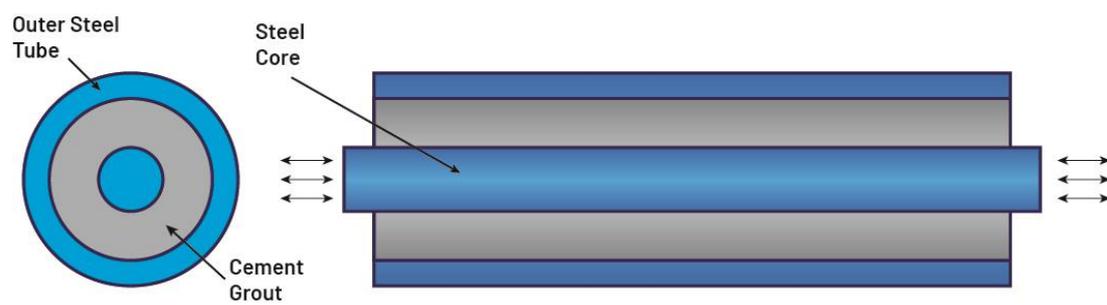
$$k_2 = \frac{A \times E}{l \times \left( \frac{\beta}{\alpha} + (1 - \beta) \right)} \text{ for } \delta \geq \delta_y \quad (2)$$

$$\delta = f_y \alpha A / k \quad (3)$$

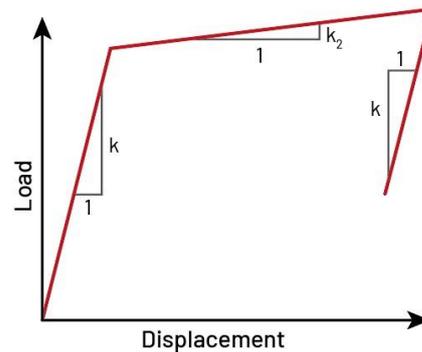
Where,  $A$  is the total cross-sectional area,  $E$  is the modulus of elasticity,  $l$  is the length of beam member,  $\alpha$  is the ratio of the decreased cross sectional area to the whole area,  $\beta$  is the ratio of the length of the decreased core to the whole length, and  $\delta_y$  is the deformation after yielding. In the nonlinear analyses, non-buckling bracings are modelled as nonlinear link (NL-Link) elements with a uniaxial plasticity property as given in Fig. 3.



**Figure 1.** (a) Frame without brace (F-NO BRACE), (b) Frame with brace in outer bays (F-BRB or VD-OUTER BAYS), (c) Frame with brace in inner bays (F-BRB or VD-INNER BAYS), and (d) Frame with brace in all bays (F-BRB or VD-ALL BAYS).

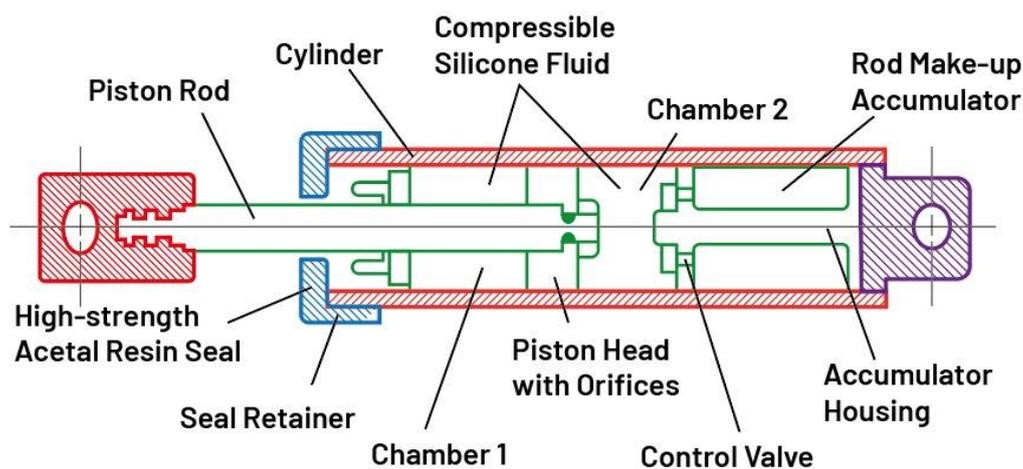


**Figure 2.** Section view of BRB



**Figure 3.** Uniaxial Plasticity properties of bracing

Another seismic protection system used in the current study is the viscous dampers (VDs). Such devices were originally developed for military applications and were later used for various applications that contribute to shock and vibration isolation such as energy absorbing buffers, channel lock buffers, and offshore oil pillar suspension. Liquid VDs work on the principle of flow of viscous liquid through the holes [30]. The VDs typically consist of a perforated piston head located in a cylinder filled with a highly viscous liquid, usually a silicone compound or a similar type of oil. When the piston head moves in the liquid, the energy is distributed by directing the liquid into the damper. The fluid in the cylinder is virtually incompressible, and when the damper is subjected to a compression force, the volume of fluid in the cylinder decreases as a result of the piston rod region movement. A reduction in volume results in restoring force. The section view of the VD is given in Fig. 4 [30]. The damper used in the analysis is a Taylor damper device of reference no RT50DH50 having the properties of the VD as the damper coefficient of 310 kNs/m, damper exponent of 0.52, stiffness of 30 kN/mm, and weight of 42 kg [30].



**Figure 4.** Section view of the VD

The frames were considered to be located in a seismically active area and they were evaluated by using various ground motions extracted from the Pacific Earthquake Engineering Research (PEER) Centre [38] strong motion database with the following characteristics; the rupture distance ( $R_{rup}$ ) is lower than 8.5 km; magnitude of the earthquake ( $M_w$ ) is between 6.54 and 7.62; and Peak ground acceleration (PGA) is higher than 0.4g.

The properties of the six earthquake records, namely Cape, Gazlı, Northridge, Hills, Chi-Chi and Tabas are given in Table 1. Moreover, they were scaled based on ASCE 7-10 [39]. The frames with and without BRBs and VDs were examined by means of the non-linear time history analysis under the given earthquakes.

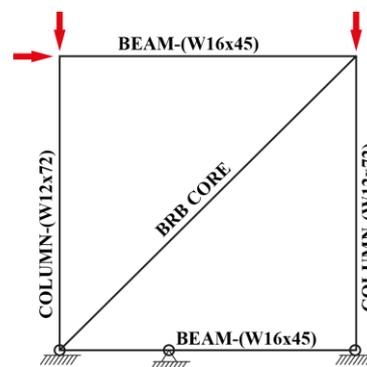
**Table 1.** Properties of the ground motion accelerations

Name	Year	$M_w$	$R_{jb}$ (km)	$R_{rup}$ (km)	$V_{s30}$ (m/s)	PGA (g)	PGV (cm/s)
Cape	1992	7.01	0	8.2	712.8	0.66	82.1
Gazlı	1976	6.8	3.9	5.5	659.6	0.72	65.39
Northridge	1994	6.69	0	5.3	441	0.84	122.7
Hills	1987	6.54	0.9	0.9	348.7	0.41	106.74
Chi-Chi	1999	7.62	0.6	0.6	305.9	0.82	127.8
Tabas	1978	7.35	1.8	2	766.8	0.80	118.29

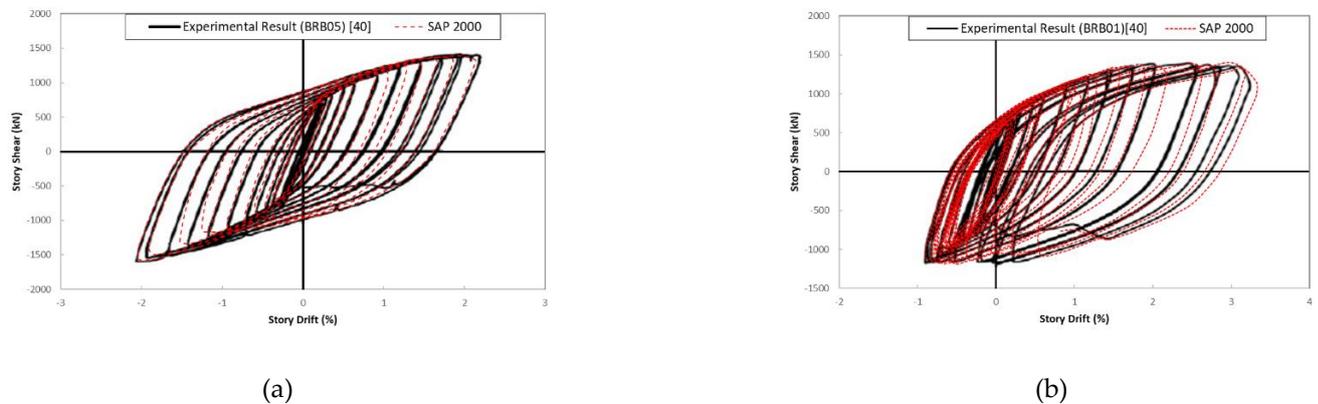
$M_w$ : Magnitude;  $R_{jb}$ : Distance of surface projection;  $R_{rup}$ : Distance of rupture;  $V_{s30}$ : Average shear velocity over 30 m; PGA: Peak ground acceleration; PGV: Peak ground velocity.

### 3. Verification of Analytical Models

In this section, the SAP 2000 model is verified through the related experimental results. For this, the studies of Palmer et al. [40] and Christopoulos [41] were utilized. They conducted the experimental work to address the performance of BRB connections in realistic framing systems and to develop a design methodology which ensures the ductility of BRB frame systems. The test program included the full-scale, single-bay, single-story planar BRB frames with geometry of 3.68 m column spacing and story height as shown in Figure 5 [41]. The structural sections were used for the beams (W16 × 45) and columns (W12 × 72). The BRBs had a total length of 3.6 m, a 19 × 162 mm rectangular core plate with a length of 2.34 m, and a 250 × 250 × 6 mm steel tube restrainer casing with infill grout. A constant axial force of 780 kN was applied to both columns which simulated gravity load from upper stories and limited column uplift. The experimental results are presented in Figure 6 as the story-shear force vs. drift response to represent the hysteretic behaviors of the frames with BRB05 and BRB01, respectively. Similarly, the numerical modeling of the frames with the aforementioned properties was performed through SAP 2000 in line with the experimental program. The results obtained from SAP 2000 are also presented in Figure 6. Comparisons between the experimental results with those obtained from the SAP 2000 models indicated that the developed SAP 2000 models were able to simulate the hysteretic behavior of the BRB-steel frames with a good accuracy.



**Figure 5.** Geometry of the tested frame in the experimental studies [41]



**Figure 6.** Verification of SAP 2000 model against the experimental results of a) BRB05 and b) BRB01 [40,41]

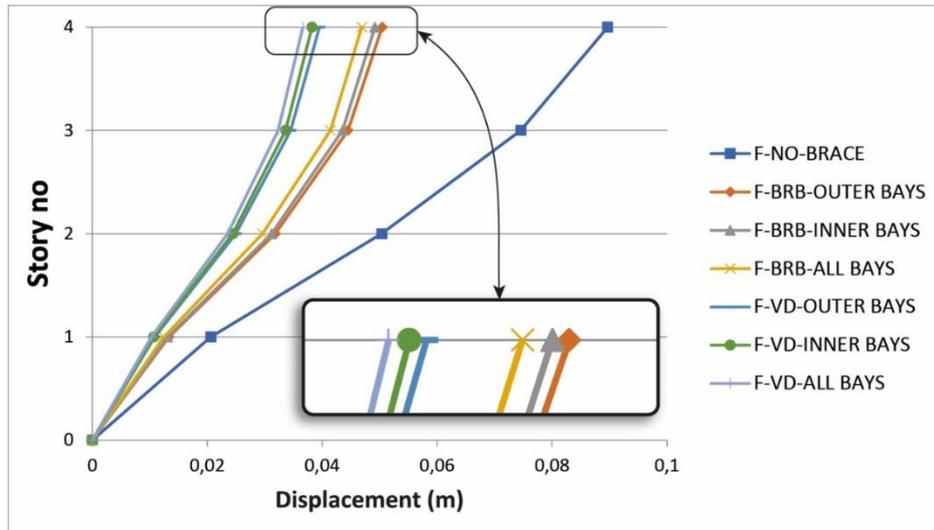
## 4. Results and Discussion

### 4.1. Variation of displacement with storey level

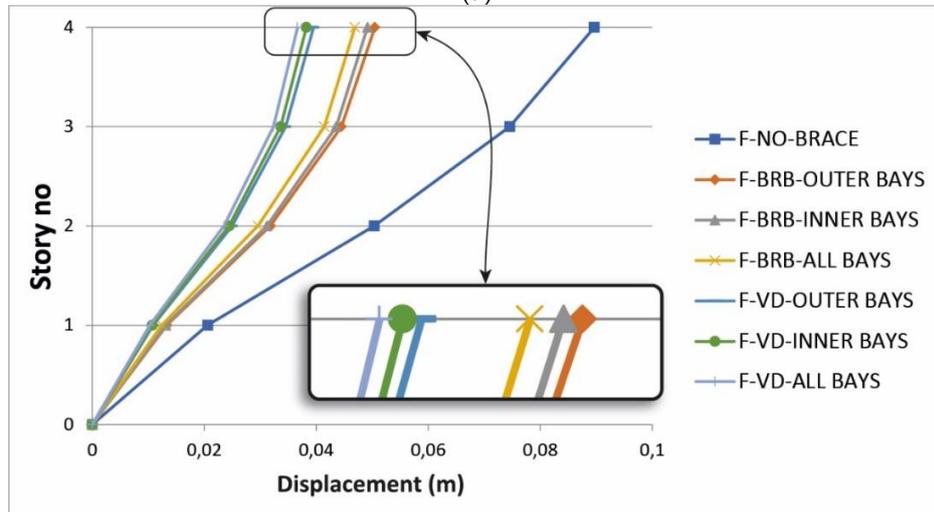
The variations of displacements against storey levels of 4-storey structures with and without BRBs and VDs in different configurations are depicted in Fig. 7. In 4-storey bare structures, the Tabas earthquake gives the highest displacement of 22 cm among the other earthquakes. The BRBs and VDs added frames show relatively better performance against the bare frames. On the other hand, the BRBs and VDs included frames indicates the highest displacement difference with respect to the bare frame as 15 cm for the same earthquake record. Moreover, the effect of Northridge earthquake on BRBs and VDs added frames are very limited in terms of the displacement since the structures with BRBs and VDs provide almost similar results.

Fig. 8 presents the variation of displacement against storey level for 8-storey buildings. Similar to 4-storey structures, in BRB and VD added 8-storey structure, the variation of displacement obtained from the Northridge earthquake yield the highest value. The Chi-Chi earthquake, however, shows the highest displacement for the bare frame. For the same record, in some cases, the difference in the displacements of the bare and upgraded frames with the BRBs and VDs reaches nearly 35 cm. Moreover, the effects of using BRBs and VDs on the displacement are clearly seen in the higher stories, the BRBs and VDs seem to be more favorable in decreasing the lateral displacement. More importantly, it is observed that the VDs shows better performance compared to the BRBs with the same configuration when the reduction in lateral displacement is taken into account.

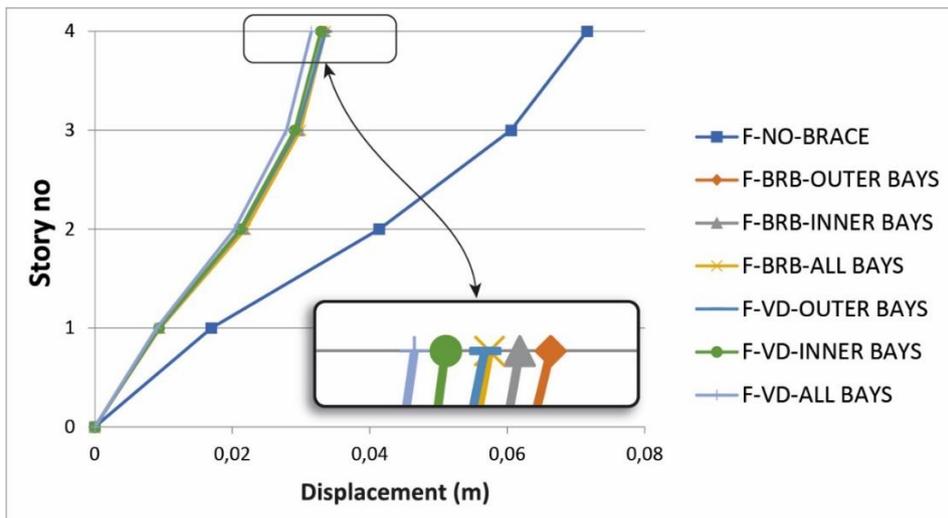
The variation of displacement vs. storey level for 12-storey buildings is shown in Fig. 9. In the case of the frames with BRB and VD systems, the Chi-Chi earthquake appears to be more effective on the variation of displacement. When the seismic protection of BRB or VD is provided, the smallest difference between displacements of bare and the upgraded frames is obtained under the Cape earthquake. The lowest lateral displacement demand belongs to the frames with VDs in all bays. Among the upgraded frames, however, the frames with BRBs in outer bays have the highest displacement.



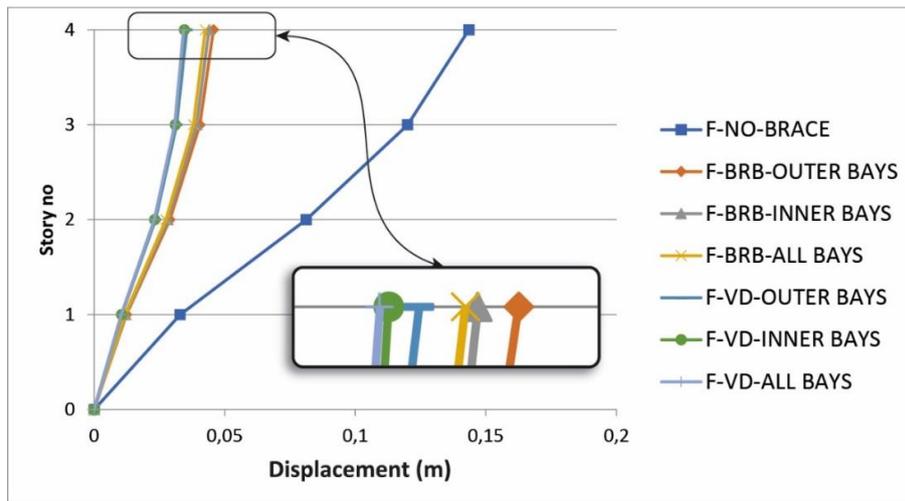
(a)



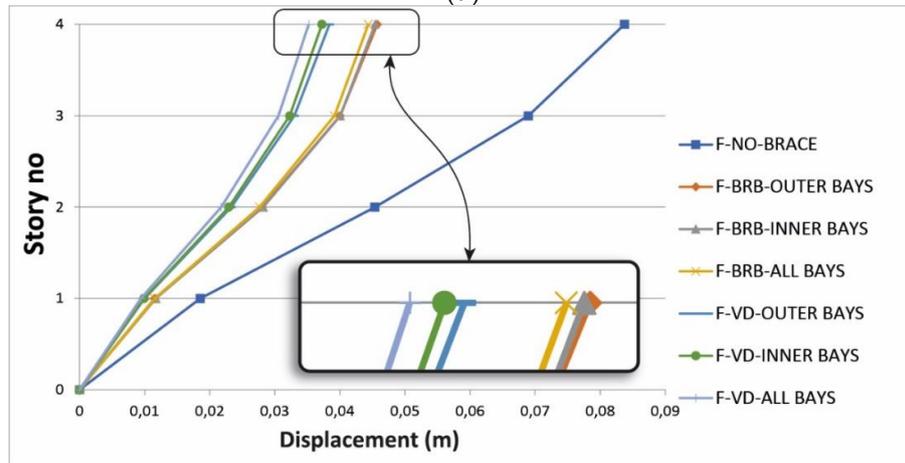
(b)



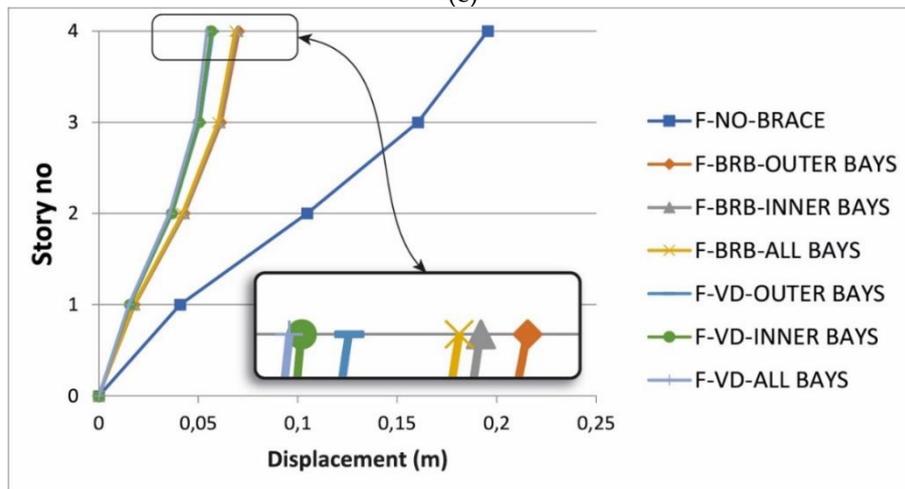
(c)



(d)

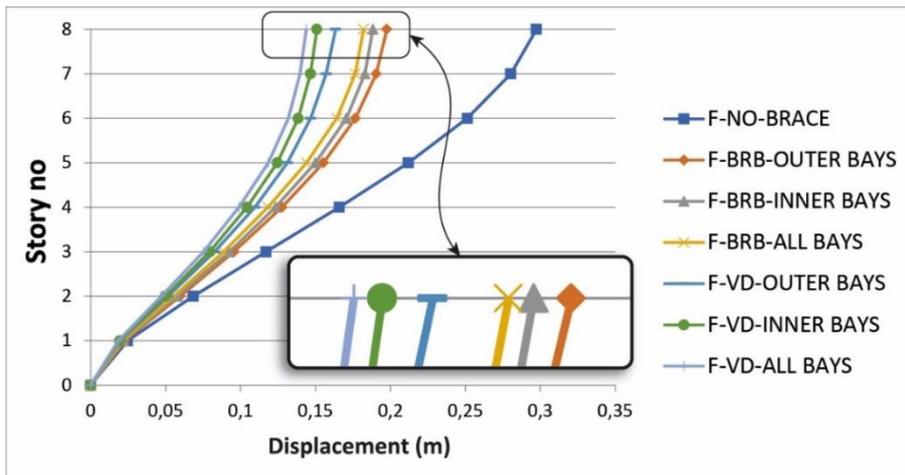


(e)

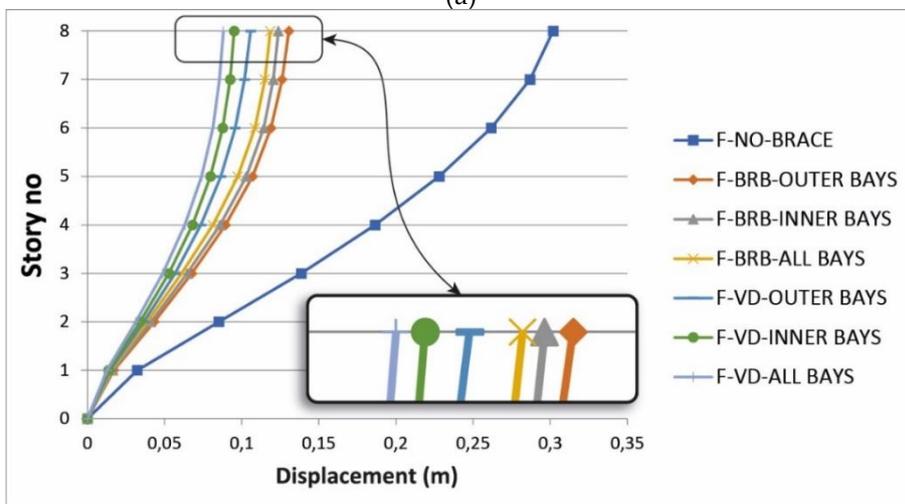


(f)

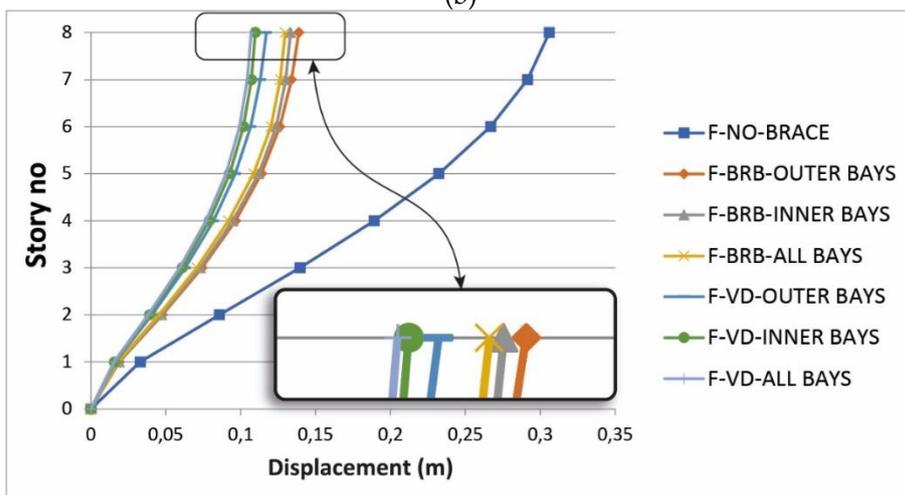
Figure 7. Variation of displacement against story level of 4-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



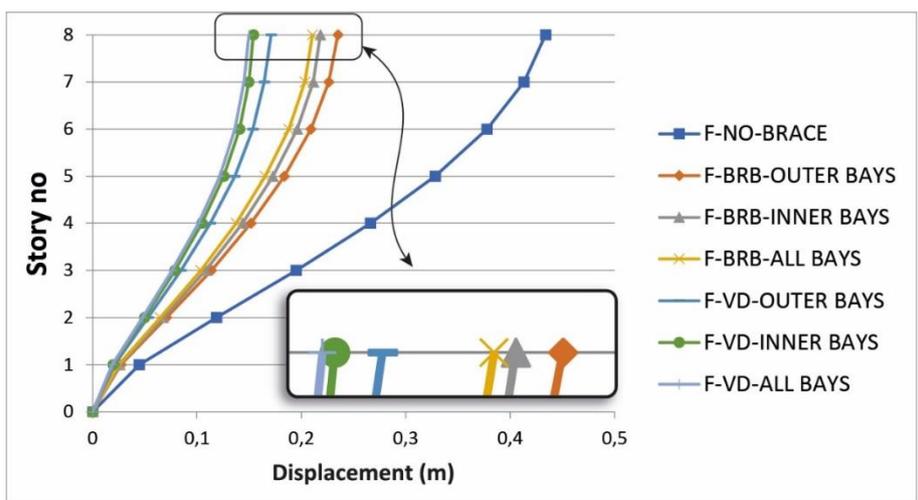
(a)



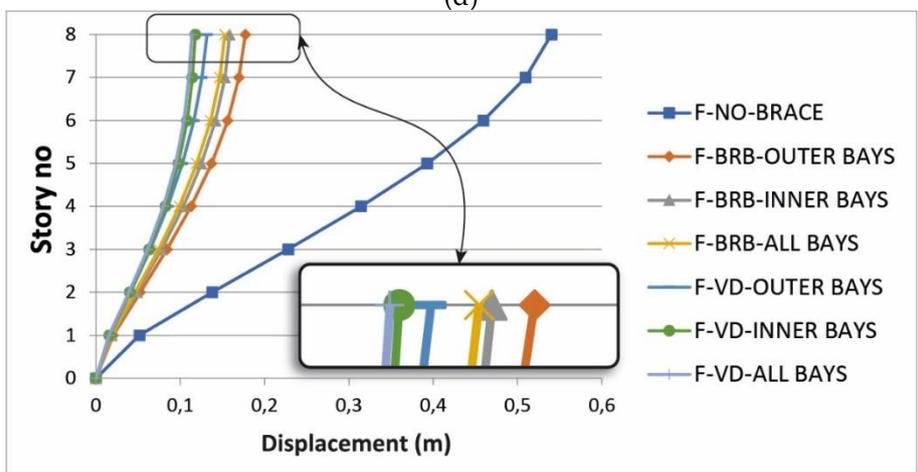
(b)



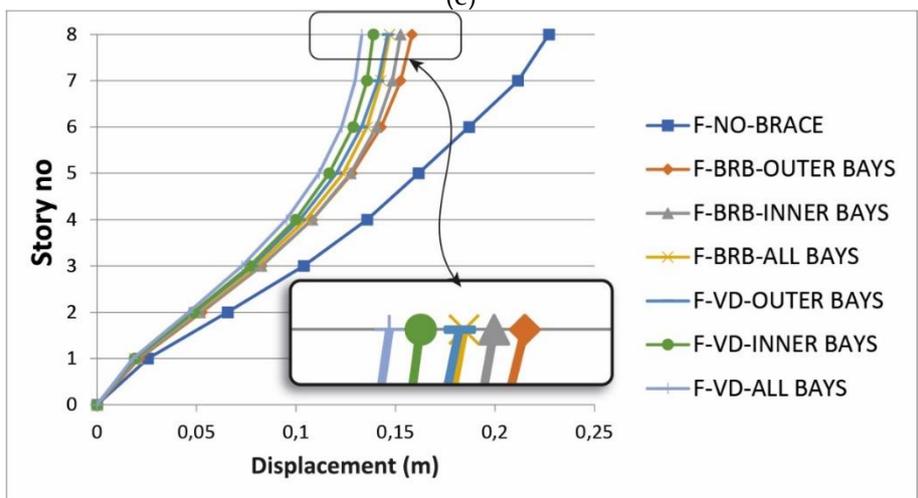
(c)



(d)



(e)



(f)

Figure 8. Variation of displacement against story level of 8-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.

#### 4.2. Variation of interstorey drift ratio with storey level

Fig. 10 illustrates the variation of interstorey drift ratio vs. storey level for 4-storey buildings. When the interstorey drift ratios of the bare structure is examined, the minimum interstorey drift ratio is calculated as 0.68% under the Gazli earthquake while the maximum interstorey drift ratio of 1.65% is acquired under the Tabas earthquake. In the case of BRB or VD integrated 4-storey structures, the highest interstorey drift ratio belongs to the Tabas earthquake. However, the BRBs and VDs systems in the 4-storey structure shows excellent performance against the bare frame behavior, regardless of the ground motion characteristics. The effect in decreasing the interstorey drift ratio is much less favorable under the Northridge earthquake. The performance of BRBs in reducing this parameter is lower compared to that of the VDs. This section may be divided by subheadings. It should provide a concise and precise description of the experimental results, their interpretation, as well as the experimental conclusions that can be drawn.

Fig. 11 shows the variation of interstorey drift ratio against storey level for 8-storey buildings. A similar trend can be observed in the bare frame systems under the Gazli and Tabas earthquakes whereas it is varied under the other earthquake records. The former and the latter yield the maximum interstorey drift ratios of 2.32% and 0.65%, respectively. Moreover, with the use of BRBs and VDs, the interstorey drift becomes remarkably less. Unlike 4-storey frames, the maximum interstorey drift ratio belongs to the third floor in that case. The configuration and type of the seismic protection devices have also marked effect on this parameter. Obviously, the systems with all-bays lead to the highest reduction in the interstorey drift ratio, irrespective of the type of bracing, especially under the Cape earthquake. When the type is considered, however, VDs appear to have more positive effect.

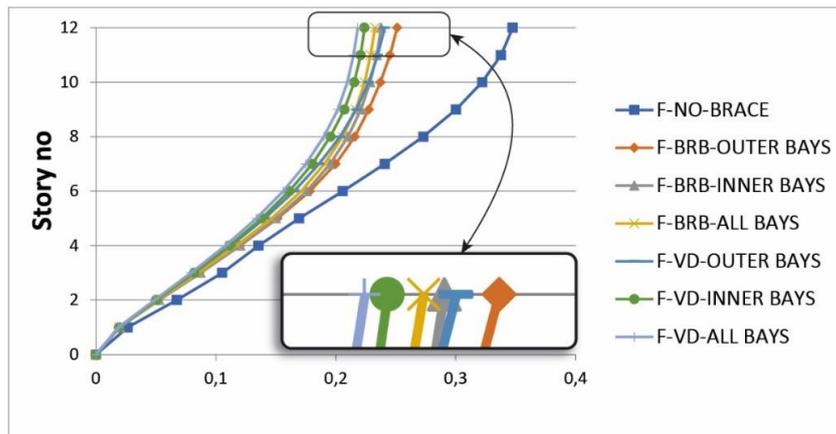
Fig. 12 presents the variation of interstorey drift ratio with storey level for 12-storey buildings. For the bare frame, the Chi-Chi earthquake causes the maximum interstorey drift ratio of 2.30% while the minimum one of 1.10% is achieved under the Cape earthquake. The variation of interstorey drift in the case of 12-storey frames well agrees that seen in both 4 and 8-storey frames. It is observed that the structures with BRBs and VDs have much more uniform interstorey drift variation with the increase in the storey level. The lowest response is achieved by using VDs in all-bays configuration, followed by the inner and outer-bays patterns. Similarly, the use of BRBs in all-bays system is measured to be more effective in diminishing the interstorey drift ratio.

#### 4.3. Maximum interstorey drift ratio (IDR) and Maximum Roof Drift Ratio (RDR)

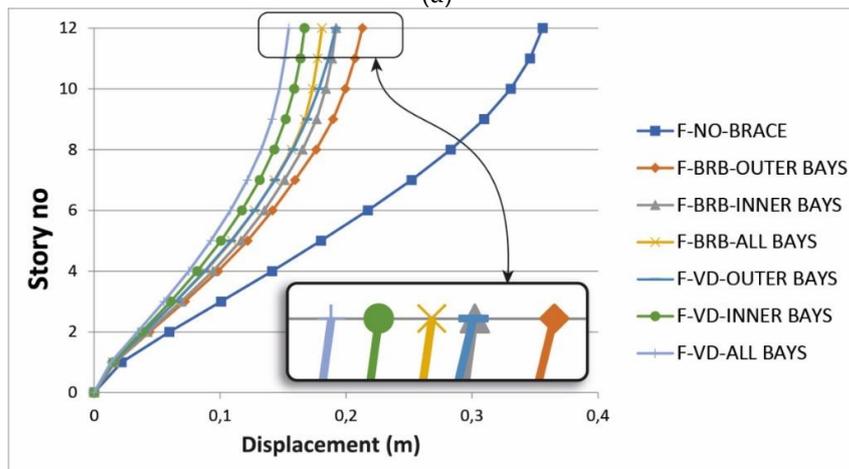
When the studies in the literature are examined, the maximum interstorey drift ratio (IDR) is generally accepted as one of the basic criteria to determine the general seismic performance. Moreover, IDR is considered as the ultimate limit state boundary condition during the design of a structure. Therefore, this parameter is quite essential when evaluating structural systems under lateral forces. FEMA 273 [42] and Vision 2000 [43] suggest the following limits for the performance levels: (a) Immediate Occupancy (IO) with 1% drift, (b) Life Safety (LS) with 2% drift, (c) Collapse Prevention (CP) with 2.5% drift, and (d) Near Collapse (NC) with 3%. Fig. 13 presents the maximum interstorey drift ratios for the 4, 8 and 12-storey structures. It is observed that the IDR values of 1.6%, 2.25%, and 1.45% are monitored in the case of 4, 8, and 12-storey bare frames, respectively, each of which being dominated by the Tabas, Chi-Chi, and Northridge earthquakes, respectively. It is evident that none of the bare buildings in this study satisfy the immediate occupation limit state. When the BRBs and VDs are integrated to the 8 and 12-storey frames, the dominating earthquakes shift to the Cape and the Hills earthquakes, respectively while there is no change in the prevailing earthquake in the case of 4-storey frame. Moreover, all of the structures considered fall within the limit state of immediate occupancy. The change in BRB or VD configurations has rather little effect on the maximum IDR of 4 and

8-storey structures while this effect become more evident with raising the storey level as observed in 12-storey structure.

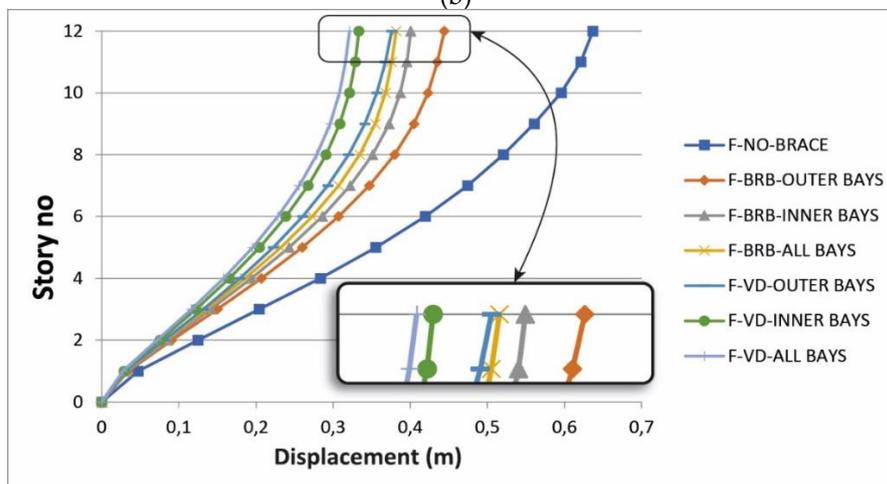
Roof drift ratio (RDR) has become a significant factor in the assessment of structural performance similar to the IDR. RDR values of the structural systems are shown in Fig. 14. It is observed that there is a similar trend between the variation of RDR and IDR for the case study frames under the earthquakes. The type of the seismic protection devices and their configurations over the frame seems very influential in reducing the RDR values.



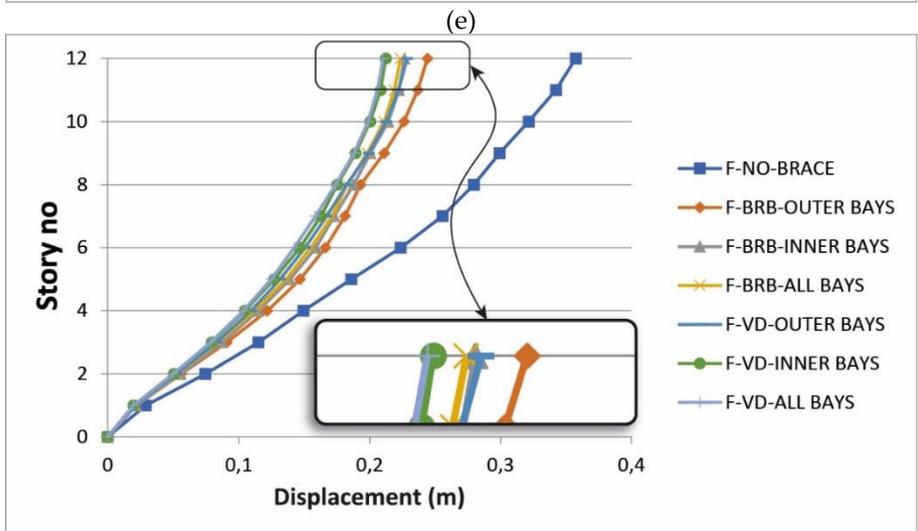
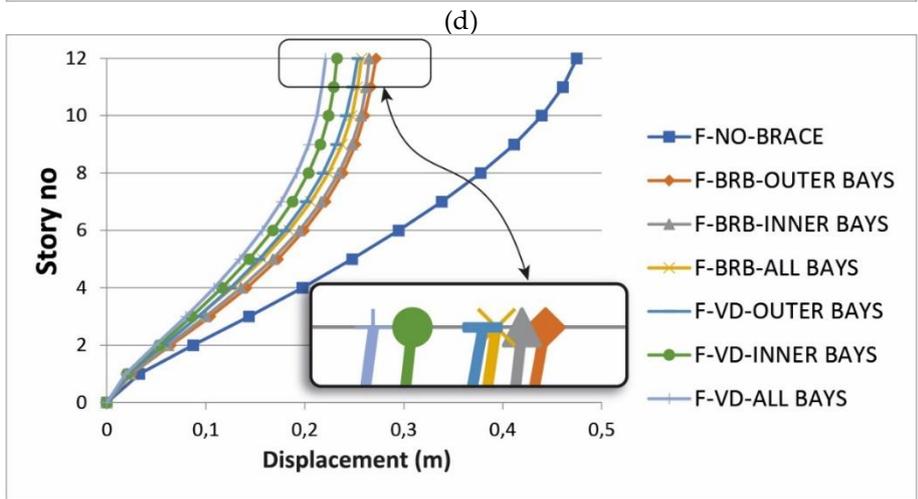
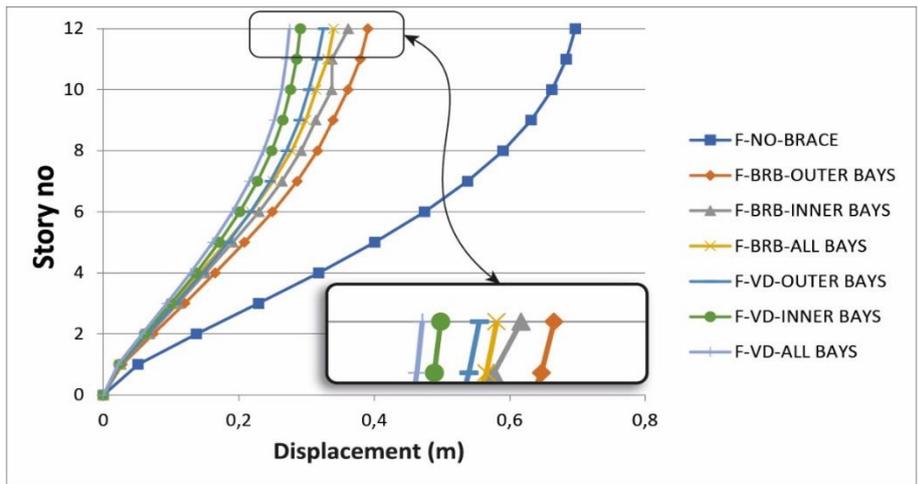
(a)



(b)

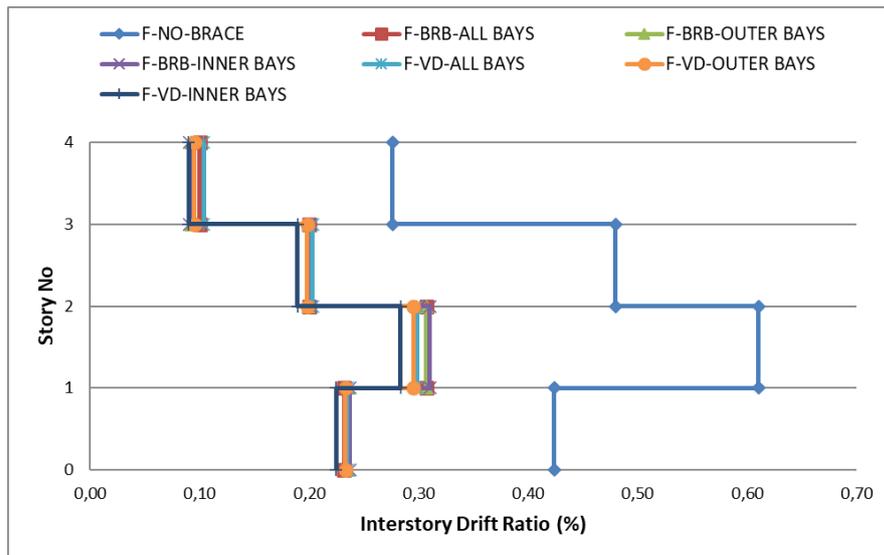


(c)

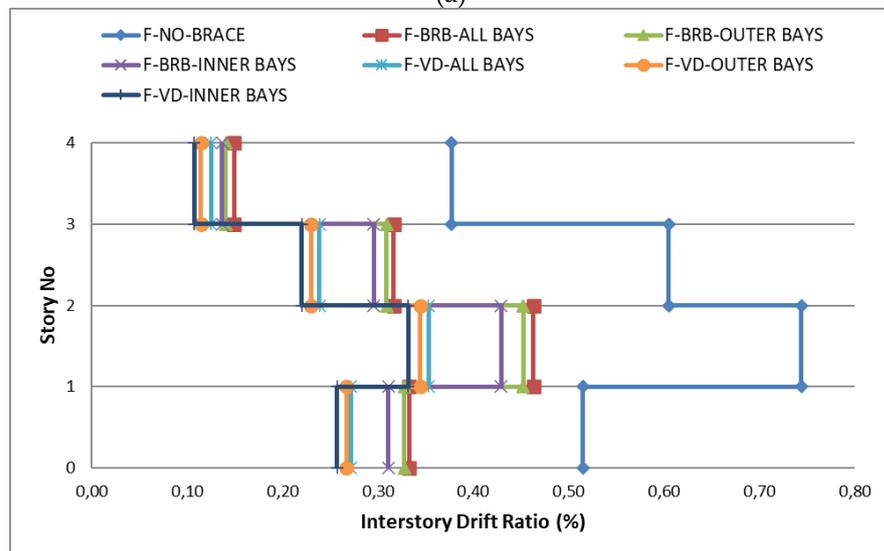


(f)

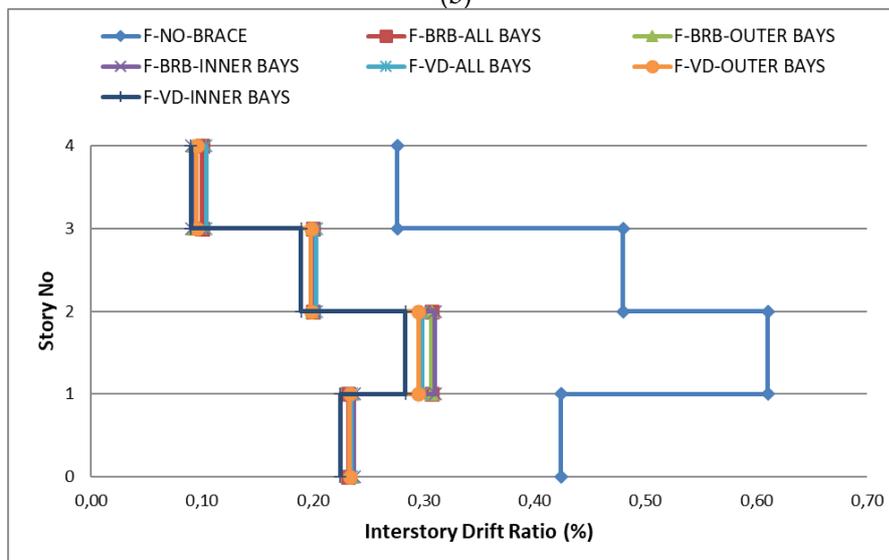
Figure 9. Variation of displacement against story level of 12-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



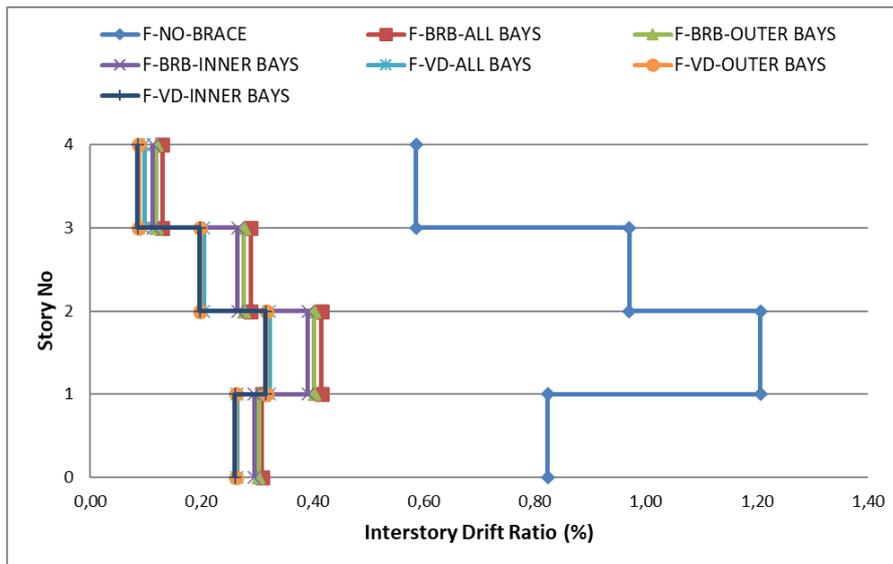
(a)



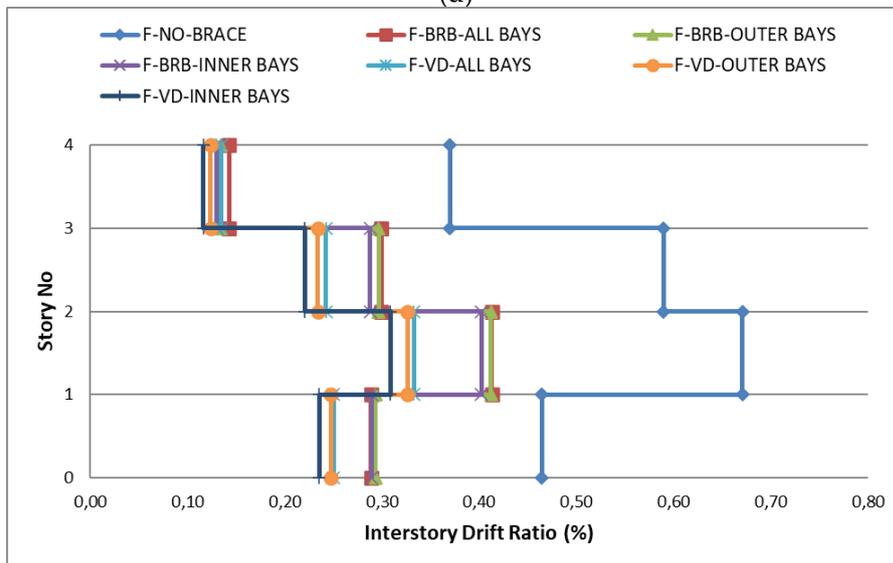
(b)



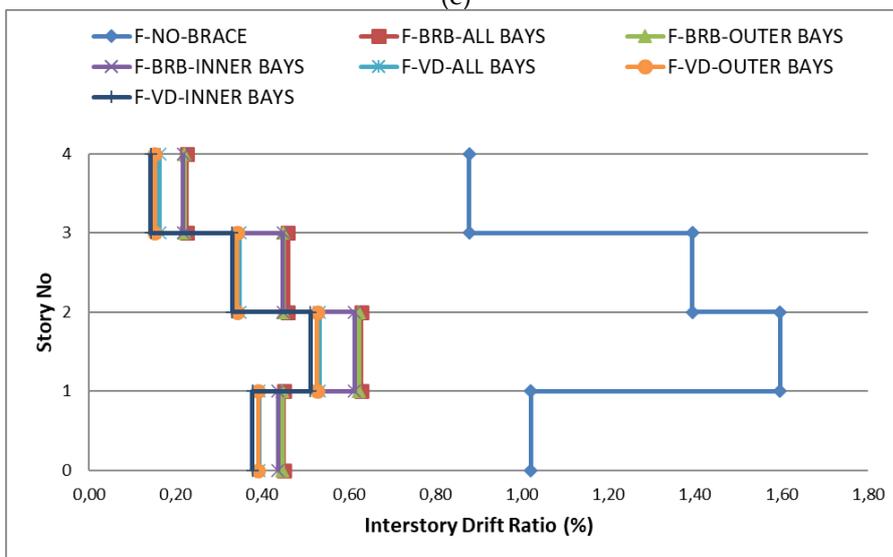
(c)



(d)

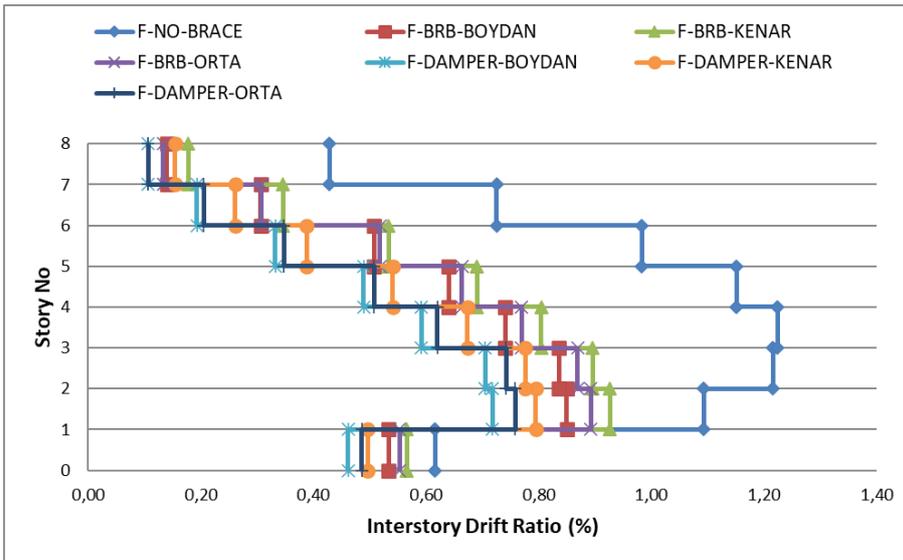


(e)

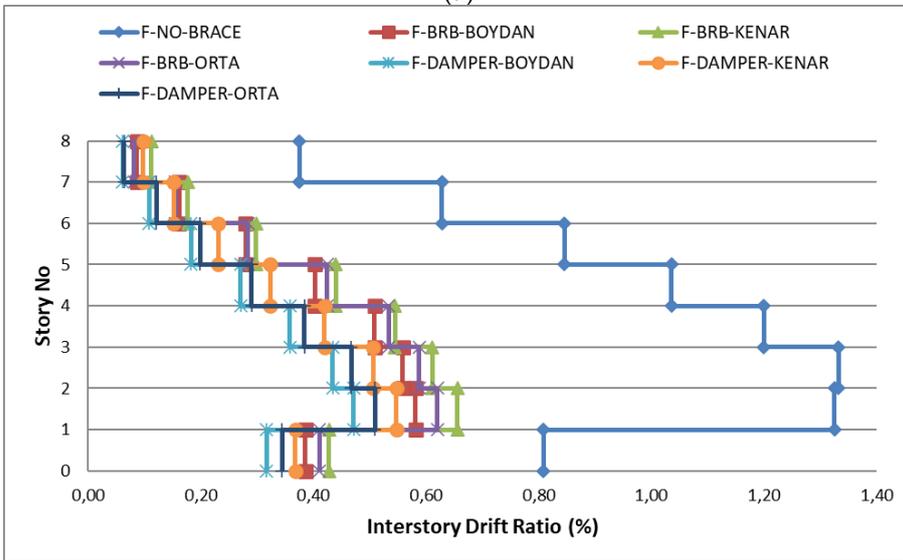


(f)

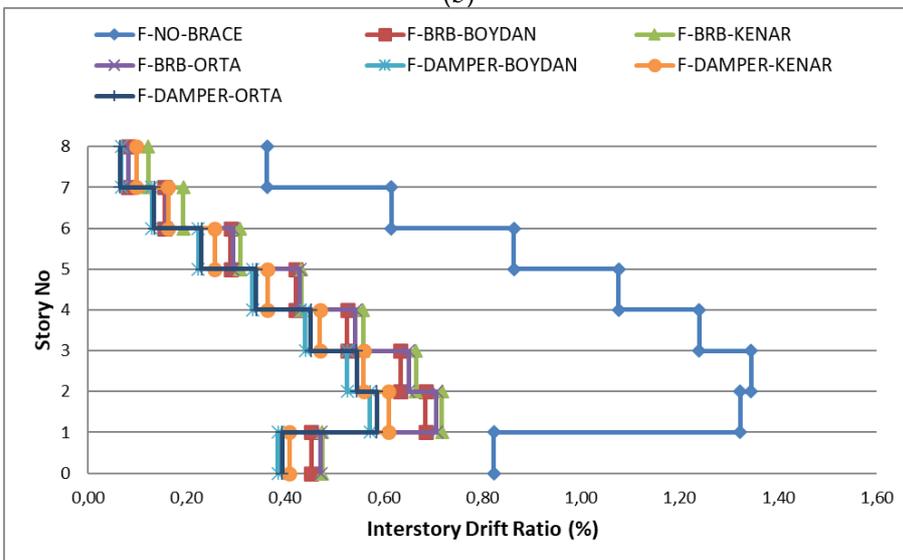
Figure 10. Variation of inter story drift ratio against story level of 4-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



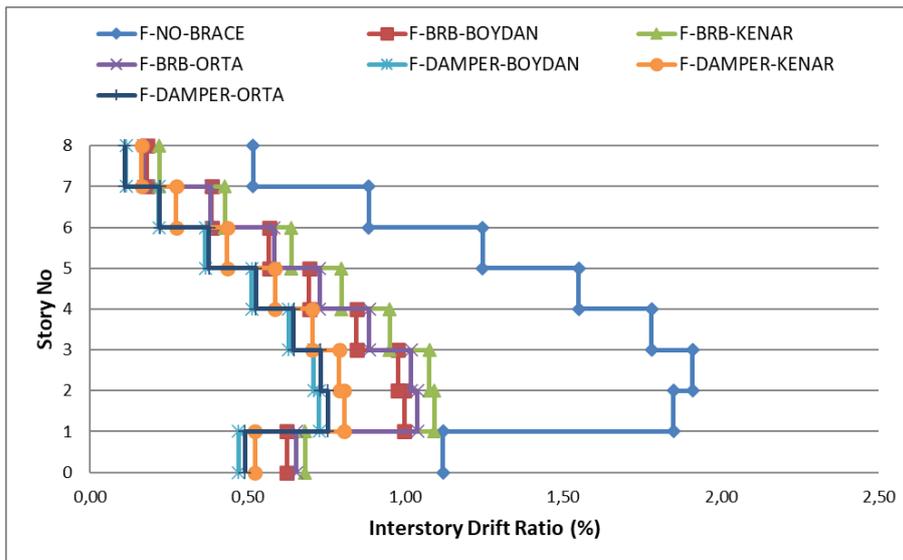
(a)



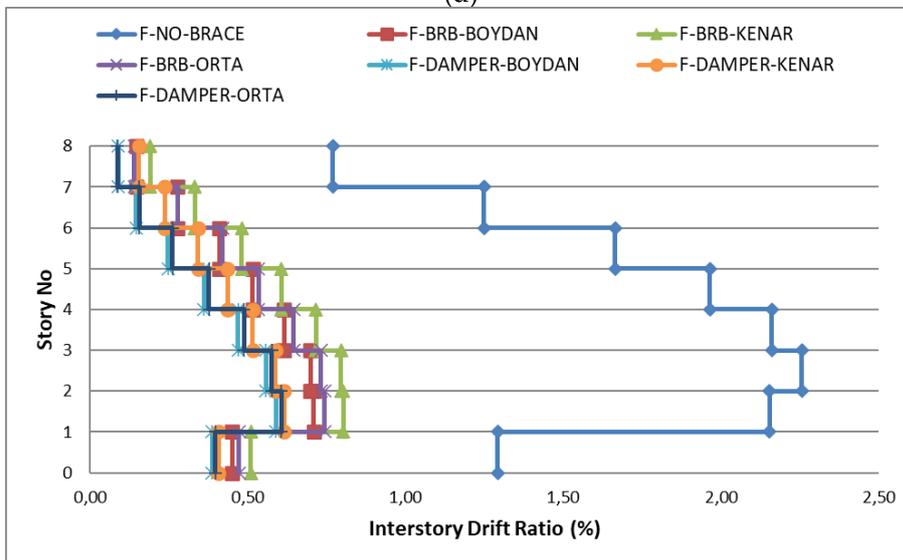
(b)



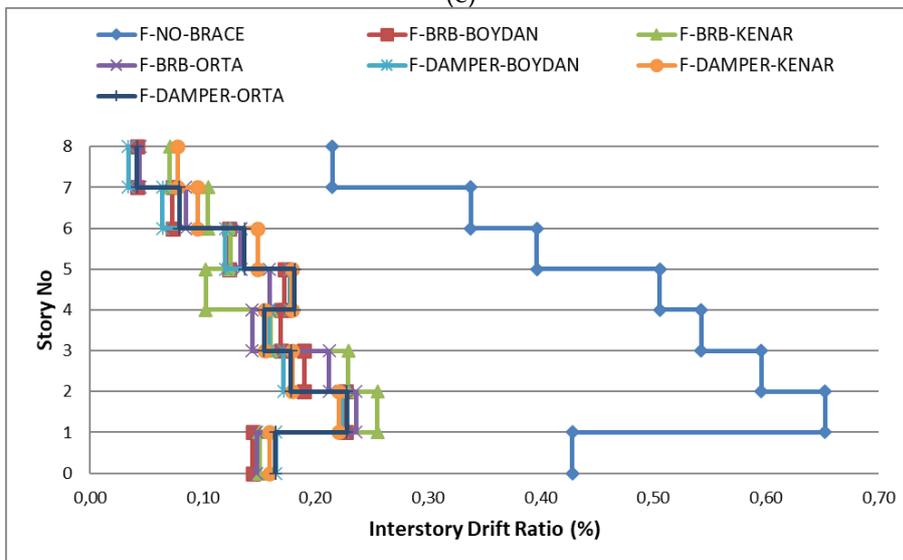
(c)



(d)

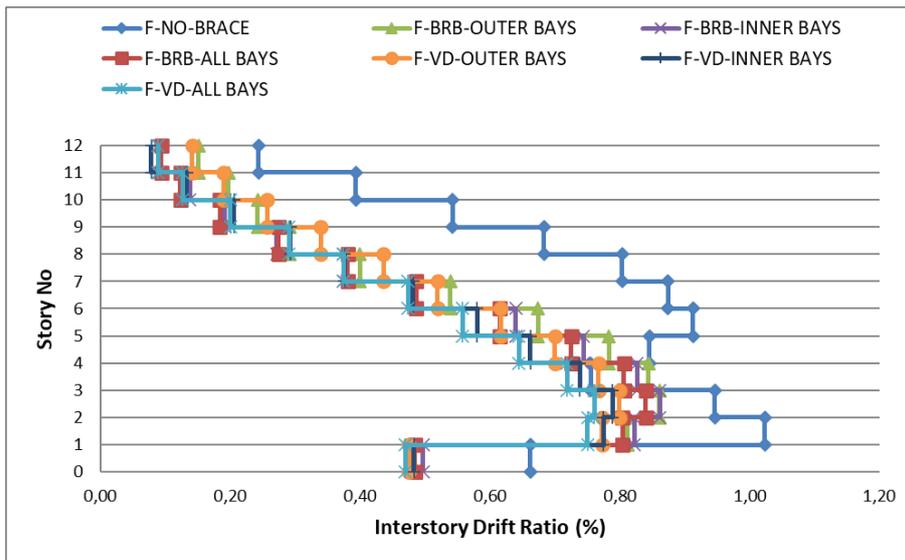


(e)

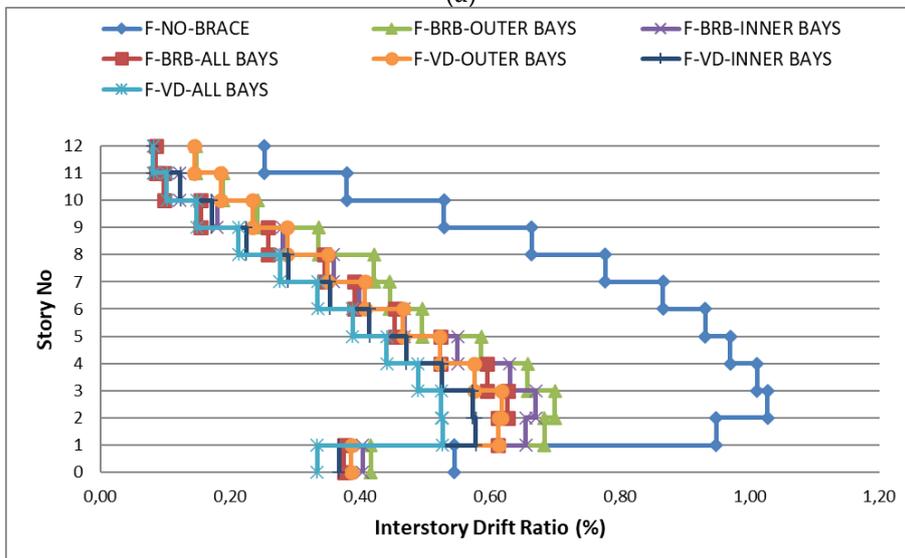


(f)

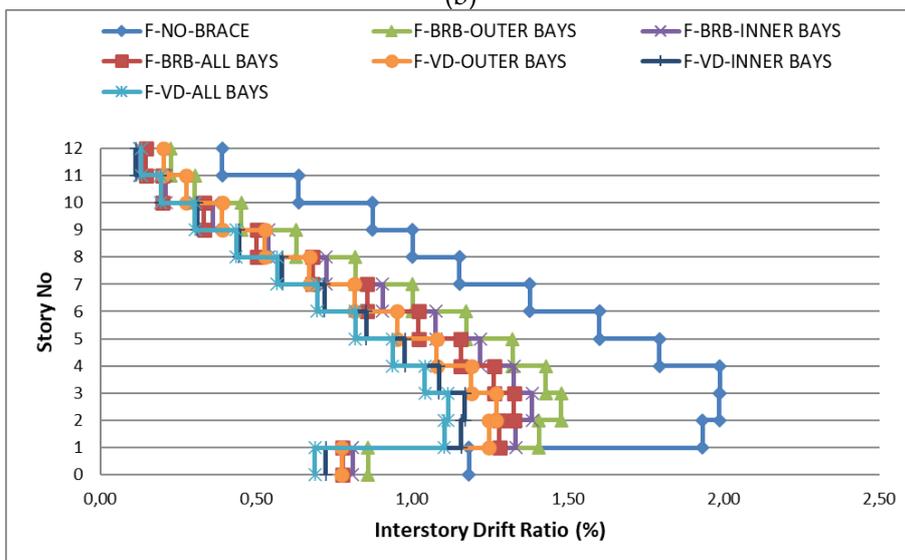
Figure 11. Variation of inter story drift ratio against story level of 8- story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



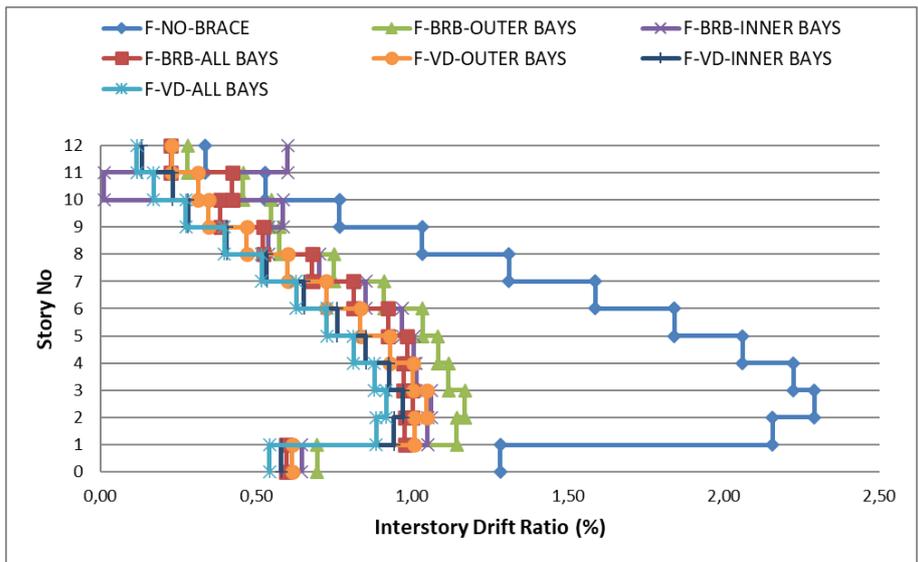
(a)



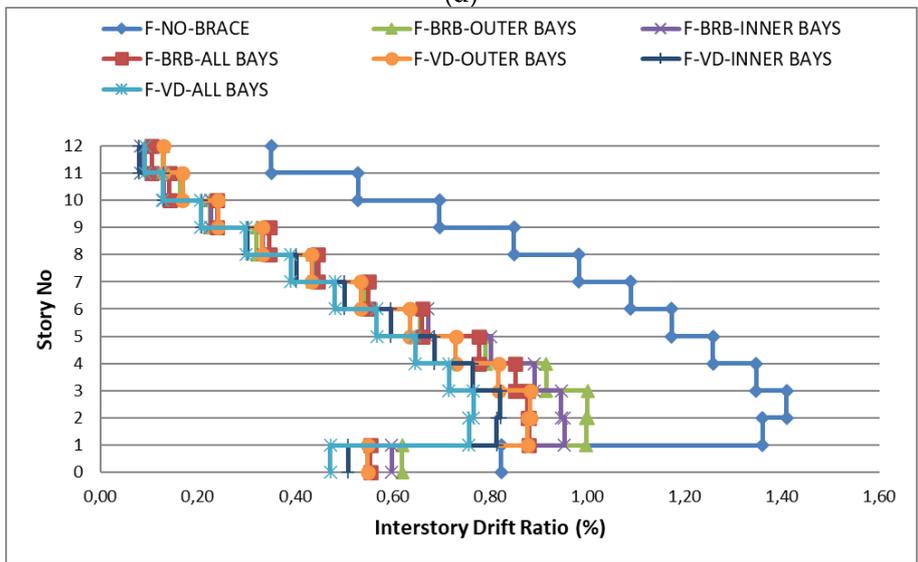
(b)



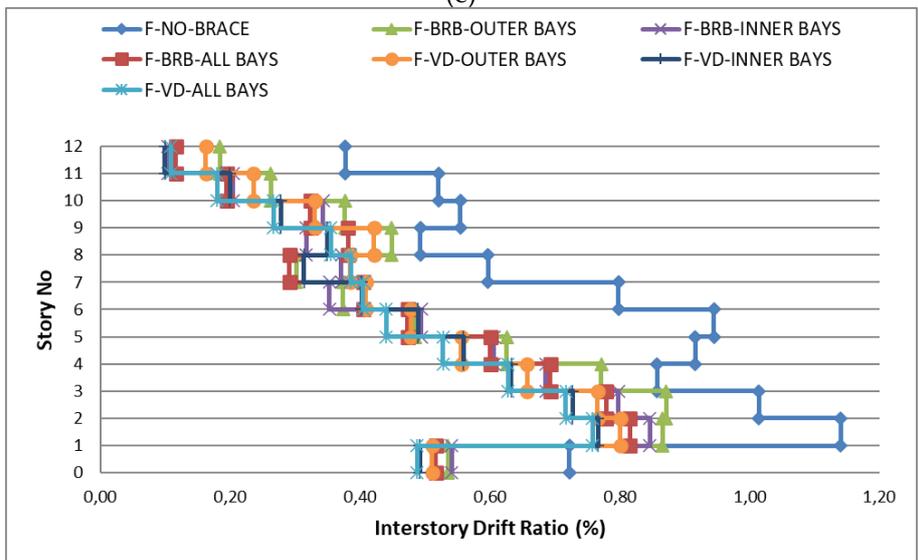
(c)



(d)



(e)



(f)

Figure 12. Variation of inter story drift ratio against story level of 12- story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.

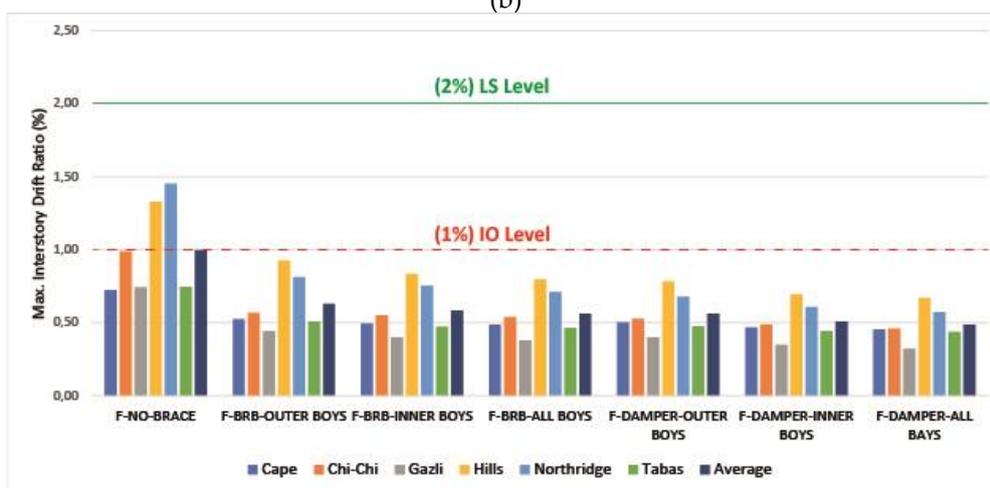
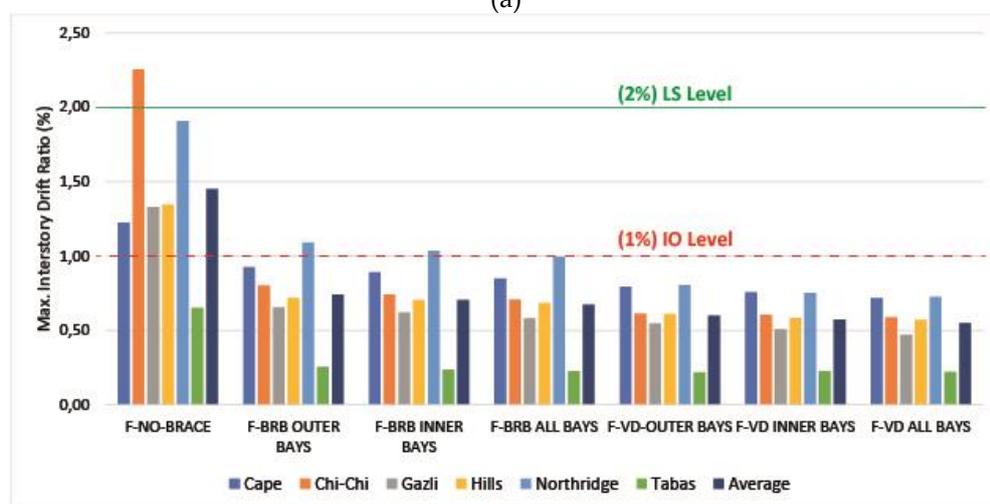
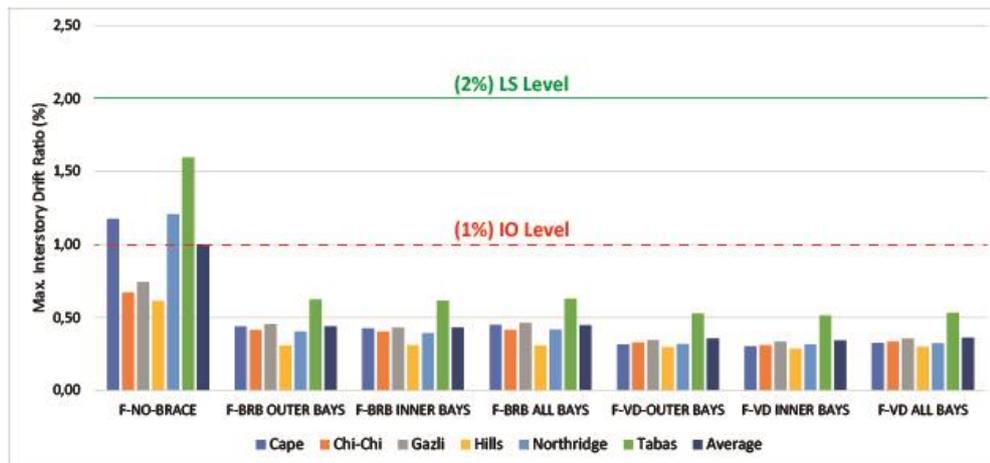
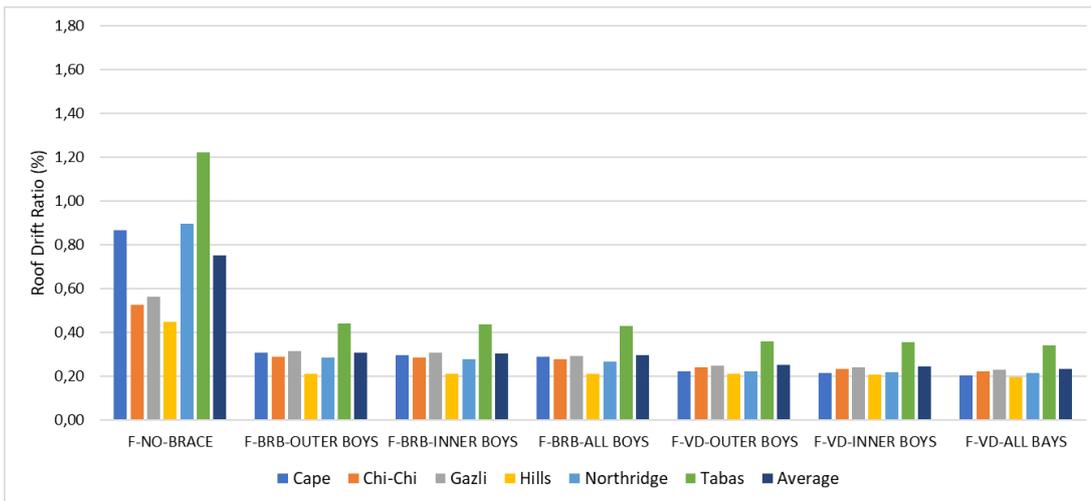
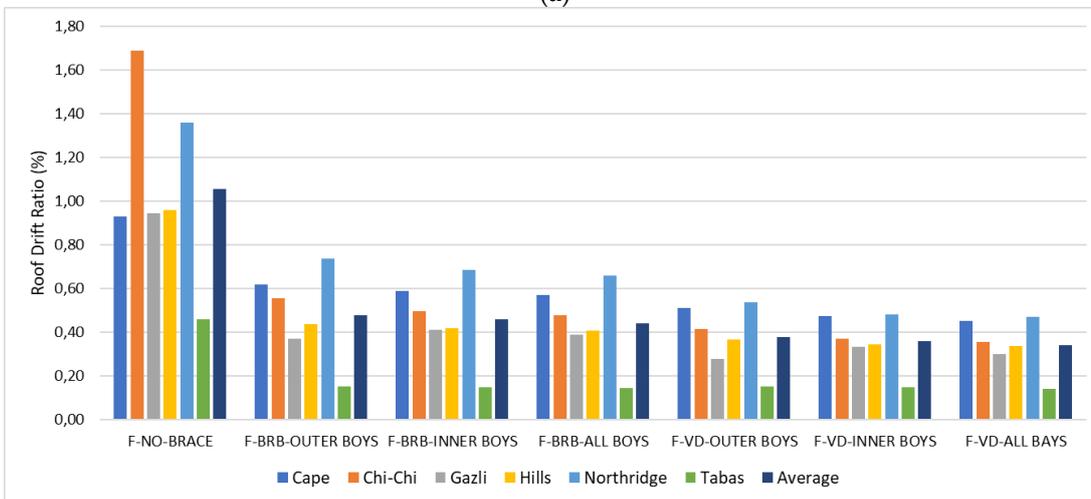


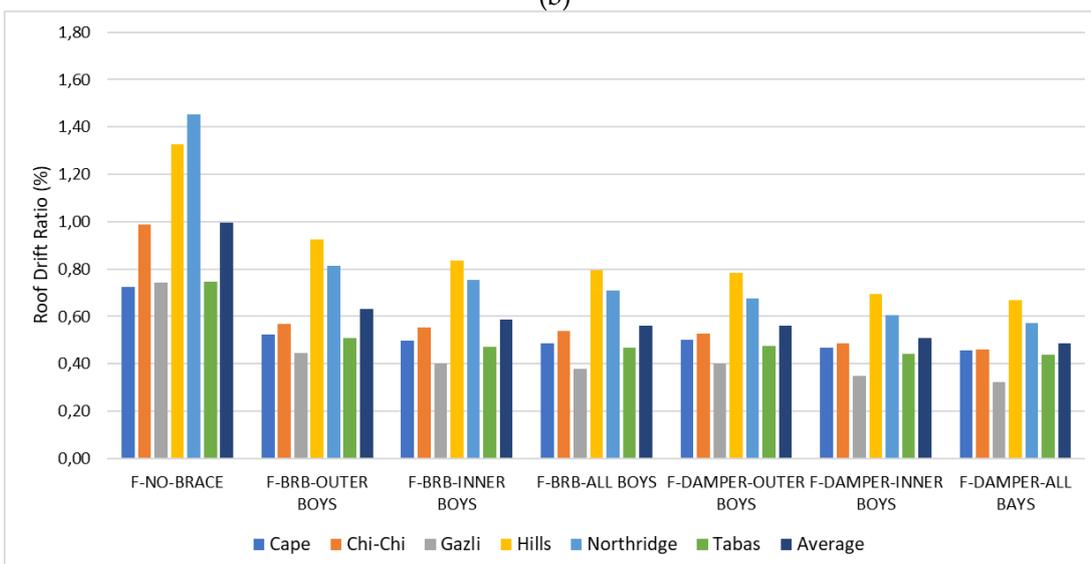
Figure 13. The maximum interstory drift ratio of; (a) 4, (b) 8 and, (c) 12-story frames.



(a)



(b)



(c)

Figure 14. The maximum roof drift ratio of; (a) 4, (b) 8 and, (c) 12-story frames.

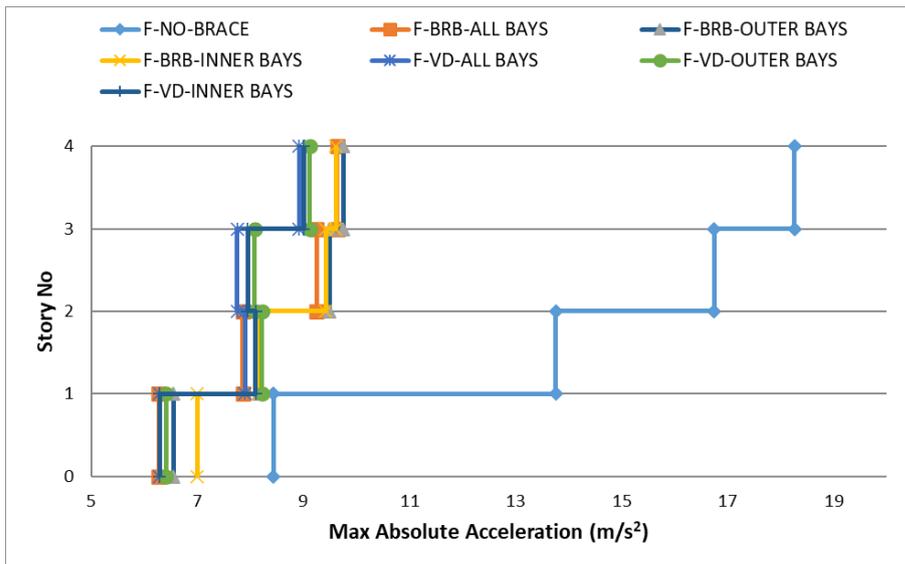
#### 4.4. Variation of maximum absolute acceleration with storey level

During an earthquake the structures suffer the damage because mainly of the relative Variation of maximum absolute acceleration with storey level displacement of building stories to each other and acceleration developed at the building floors. The role of the seismic protection, therefore, is to decrease the absolute acceleration. The maximum absolute acceleration versus storey level in the 4-storey structure is given in Fig. 15. The highest absolute acceleration took place at the fourth storey of the frames regardless of the ground motion as well as the seismic protection devices provided. There was a sharp decrease after the frames had been modified with either BRBs or VDs. The effect of the latter seemed to be more pronounced. Among the impacts exerted, the Gazlı earthquake resulted in the highest while the Chi-Chi earthquake gave the least accelerations. When the seismic protection installment pattern was considered, the all-bay pattern provided higher decrease in the absolute acceleration. Even though using VDs decreased the absolute acceleration more than the BRBs, the latter used in all-bay system was more influential particularly at the fourth storey of the frames under the Hill and the Northridge earthquakes. The decrease in the absolute acceleration with the use of BRBs and VDs increased with the storey level.

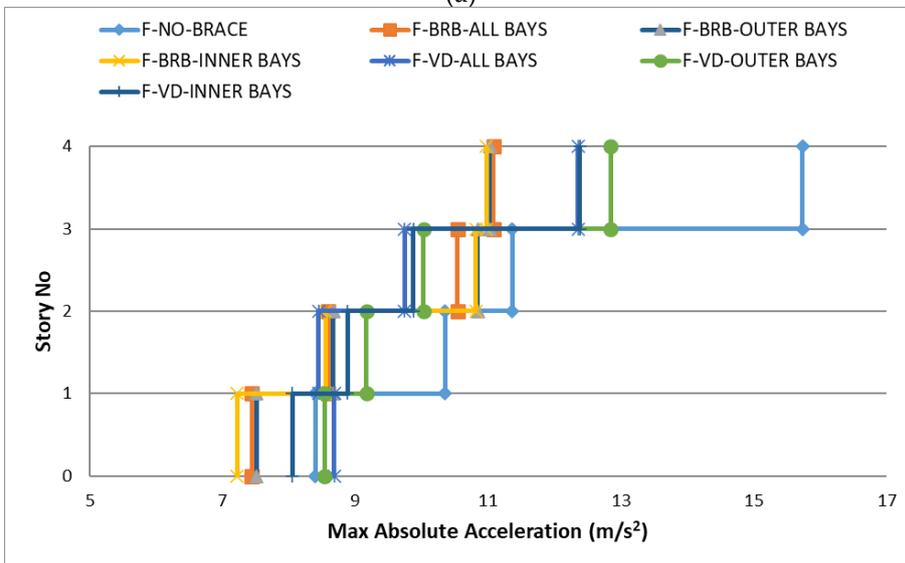
Figures 16 and 17 show the variation in the maximum absolute accelerations with storey level for 8 and 12-storey buildings, respectively. Unlike the 4-storey frames, the highest absolute acceleration was not monitored on their top floors. Moreover, the storey having the highest acceleration altered under the different ground motions. When the Cape earthquake was considered, the storey acceleration was higher at the lower storey of the frames with the BRBs and VDs, compared to the bare frames. It was seen in both Fig. 16 and 17 that the response of the building under varying earthquakes differed substantially. Nonetheless, after BRBs and VDs were introduced, the absolute acceleration between two subsequent floors had much less variation. The upgrading of the frames lessened the absolute acceleration by as much as 65% such that the higher the storey numbers, the greater reduction in this parameter. In line with the other seismic responses, the effect of using VDs was more noticeable compared to the BRBs. The absolute accelerations of the buildings varied with the configuration of the seismic protection. As seen in Fig.17 that the top storey acceleration under the Cape earthquake decreased steadily with the use BRBs or VDs in outer-bay, inner-bay, and all-bay respectively.

#### 4.5. Maximum base shear

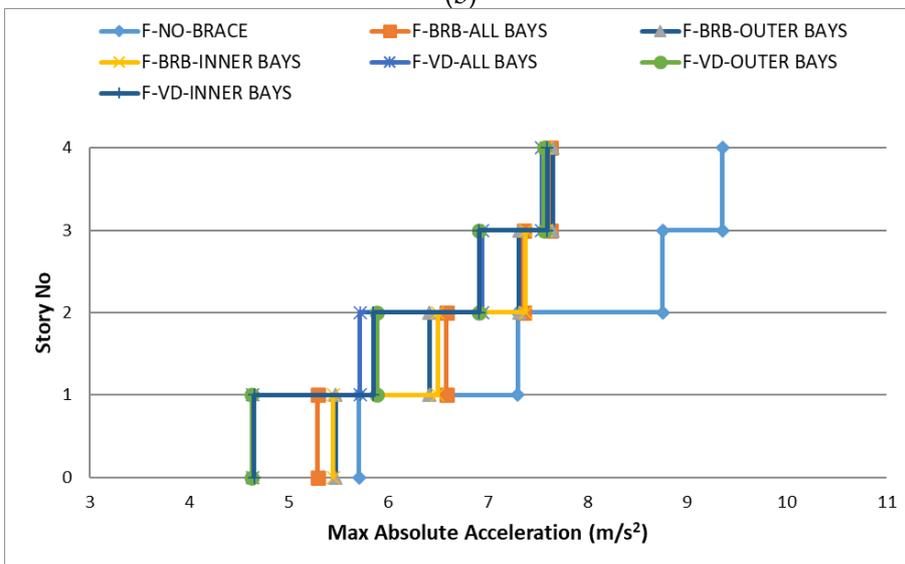
The base shear of the structural systems with and without seismic protection devices under the effects of the earthquakes is presented in Fig. 18. Basically, in each bare structure, different earthquake records give different maximum base shear values. Indeed, the Tabas, Chi-Chi, and Northridge earthquakes dominate the base shear of the 4, 8, and 12-storey buildings, respectively. After the BRBs and VDs are used, especially for the 4 and 12-storey buildings, the maximum base shear is observed under the Tabas and Hills earthquakes, respectively. However, all of the earthquakes have similar effects on the maximum base shear of the 8-storey building with the BRBs or VDs. Depend on the seismic protection type and configuration, their effects on the maximum base shear varies.



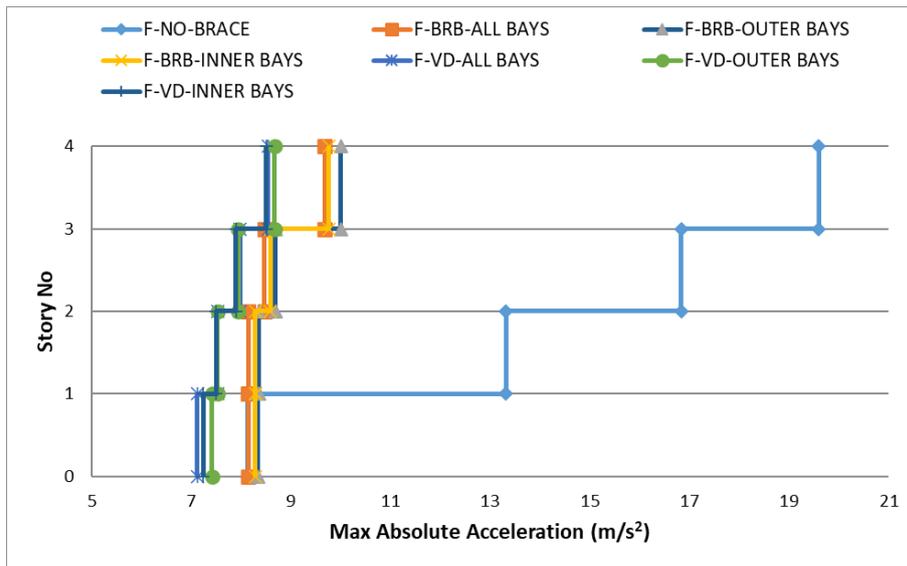
(a)



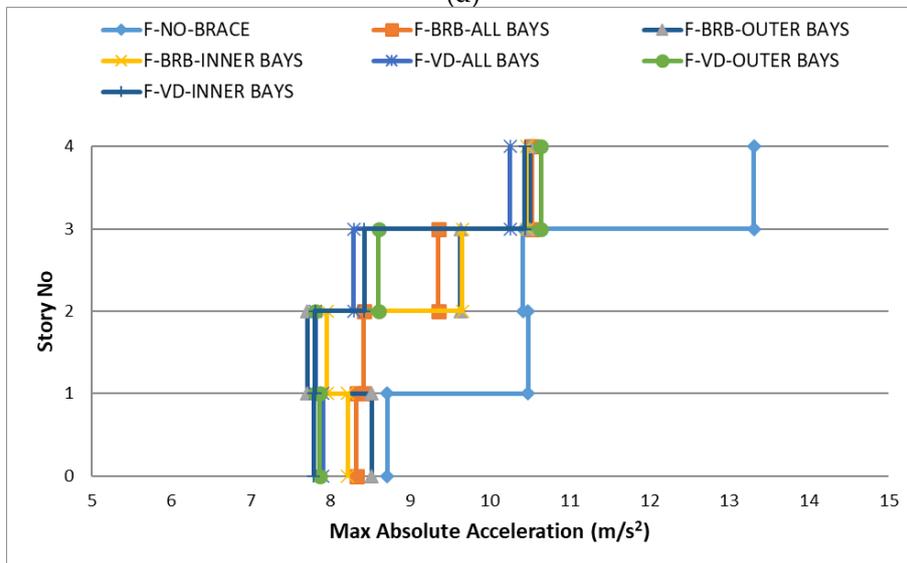
(b)



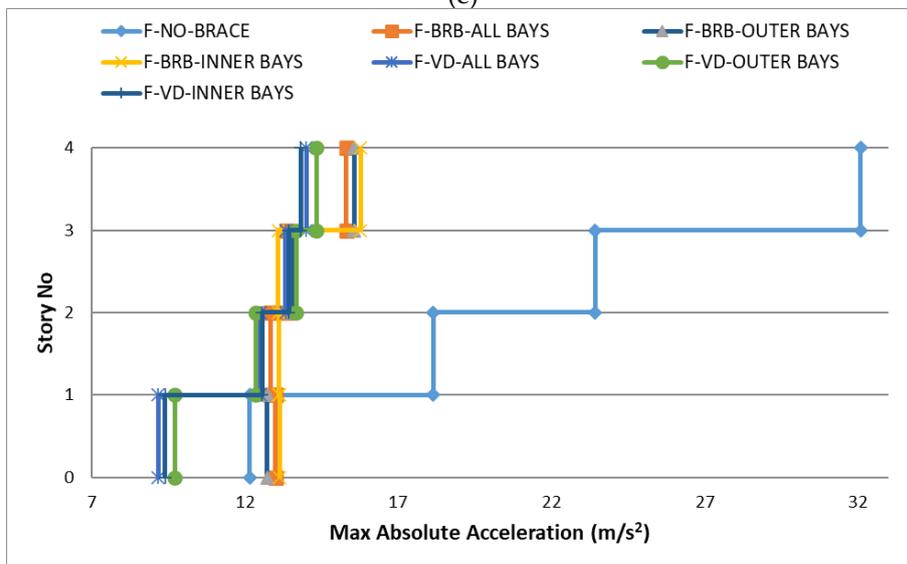
(c)



(d)

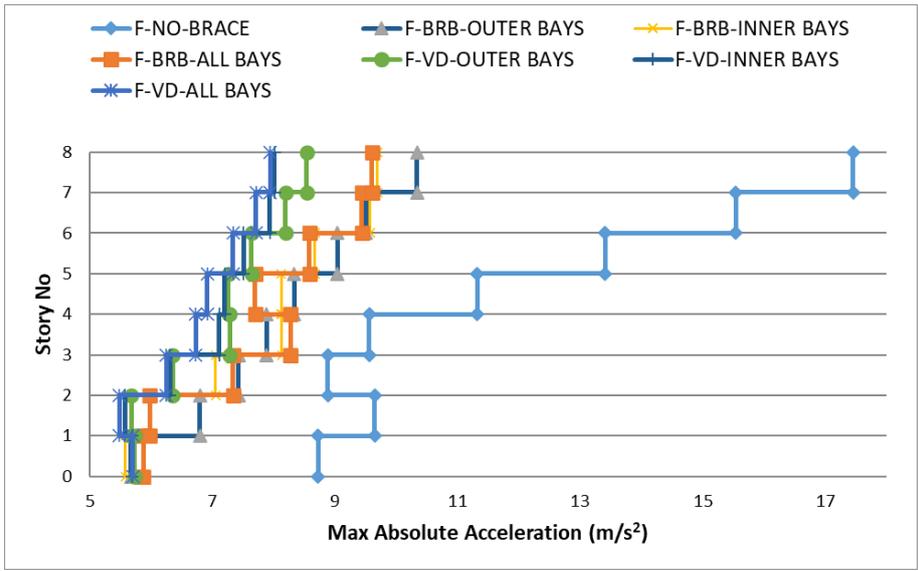


(e)

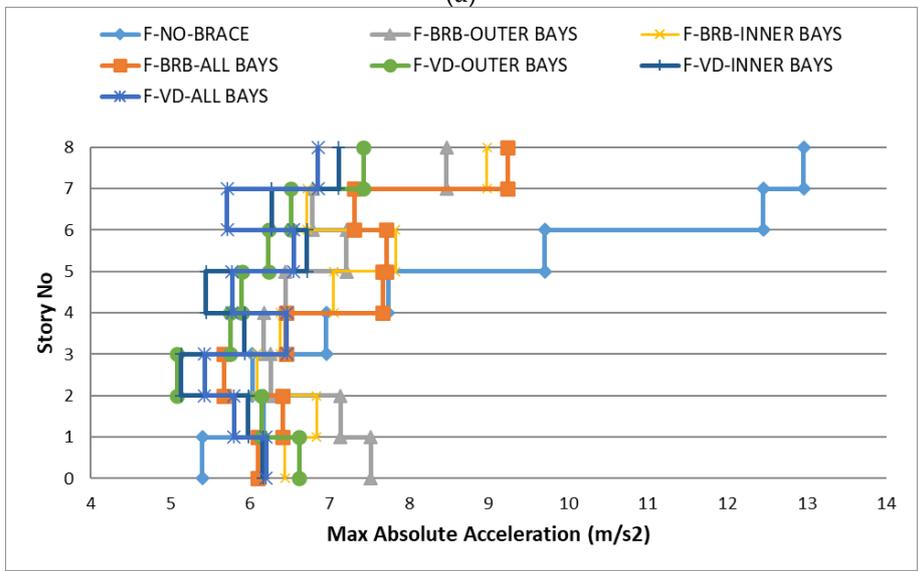


(f)

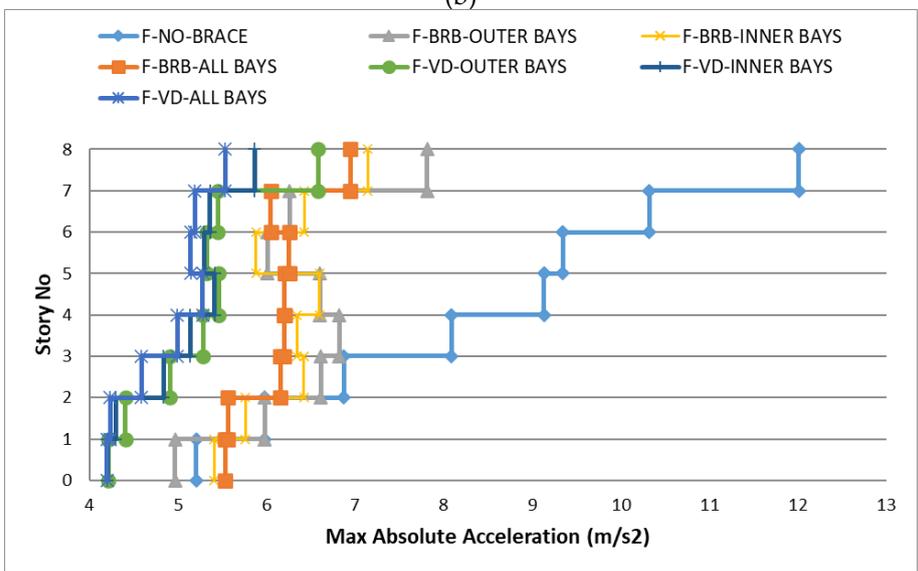
**Figure 15.** Variation of the maximum absolute accelerations against story level of 4-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



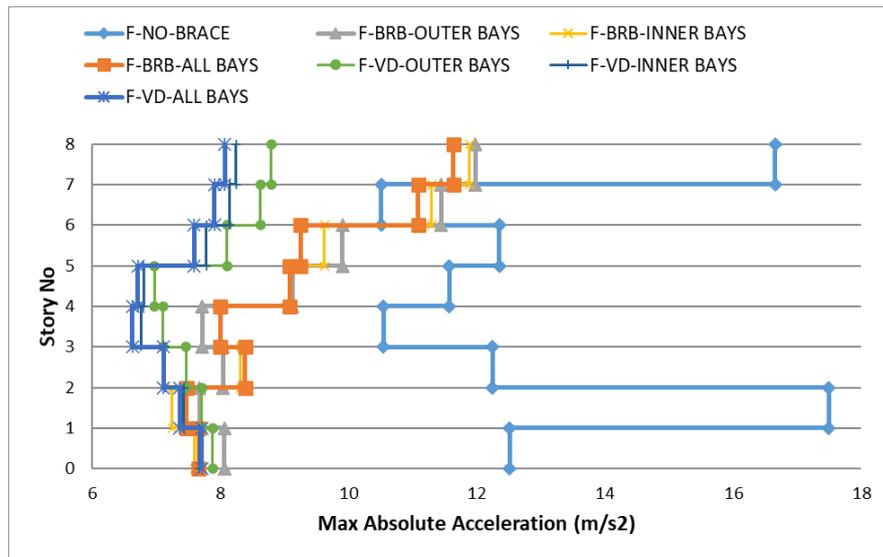
(a)



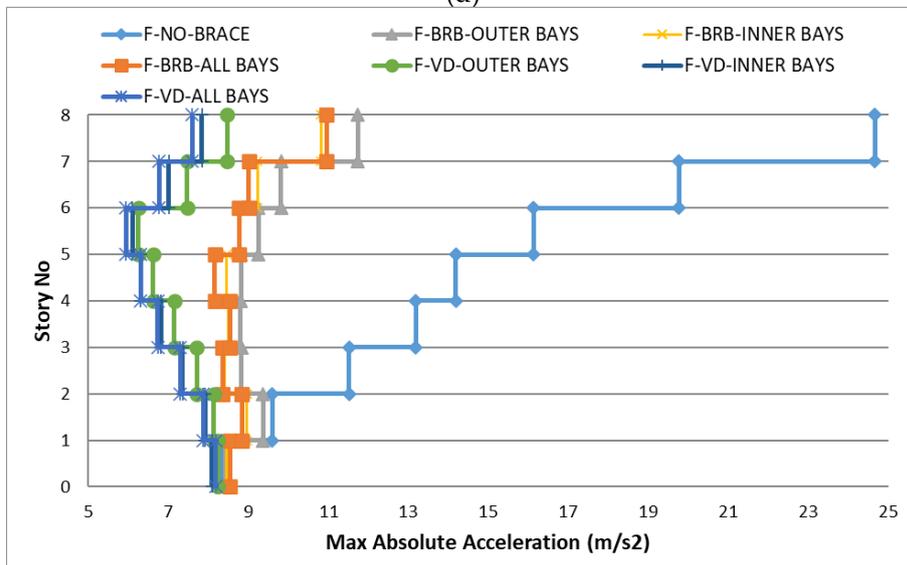
(b)



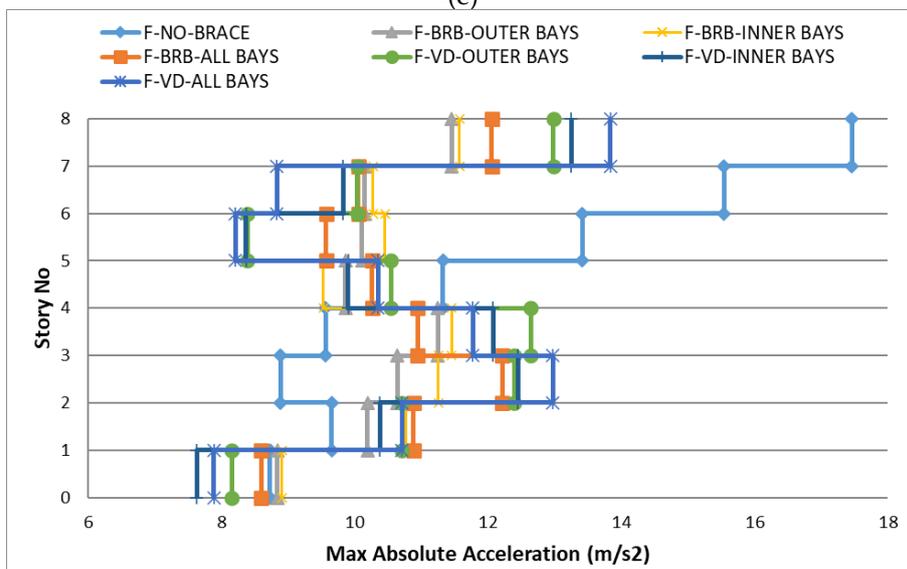
(c)



(d)

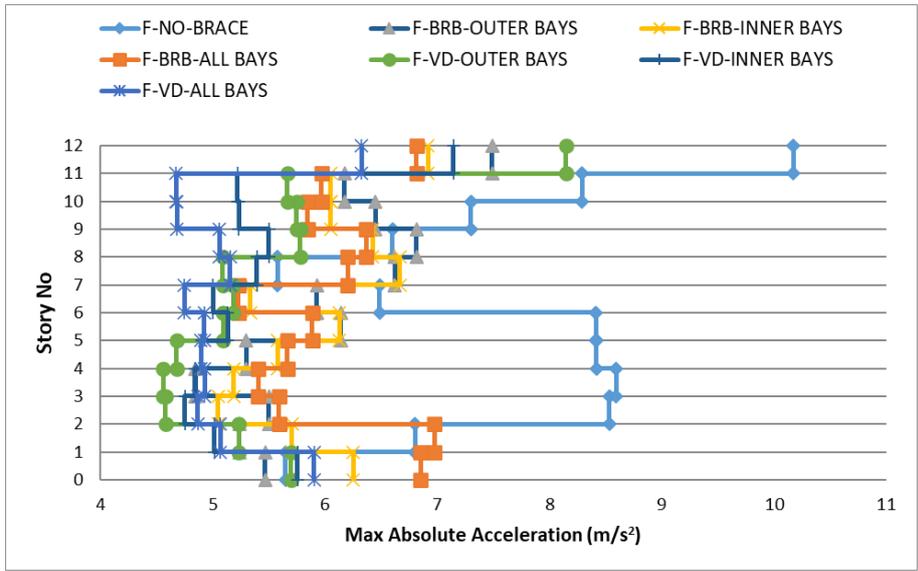


(e)

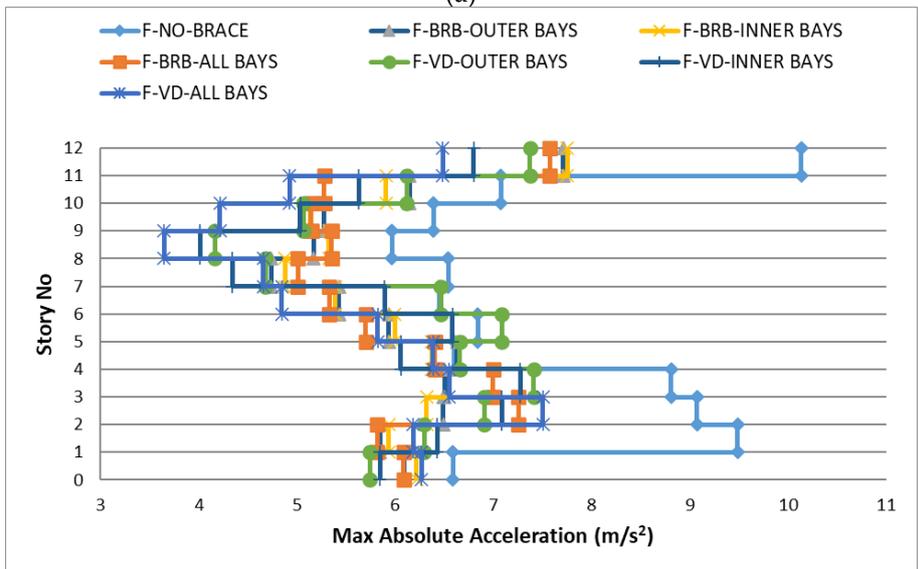


(f)

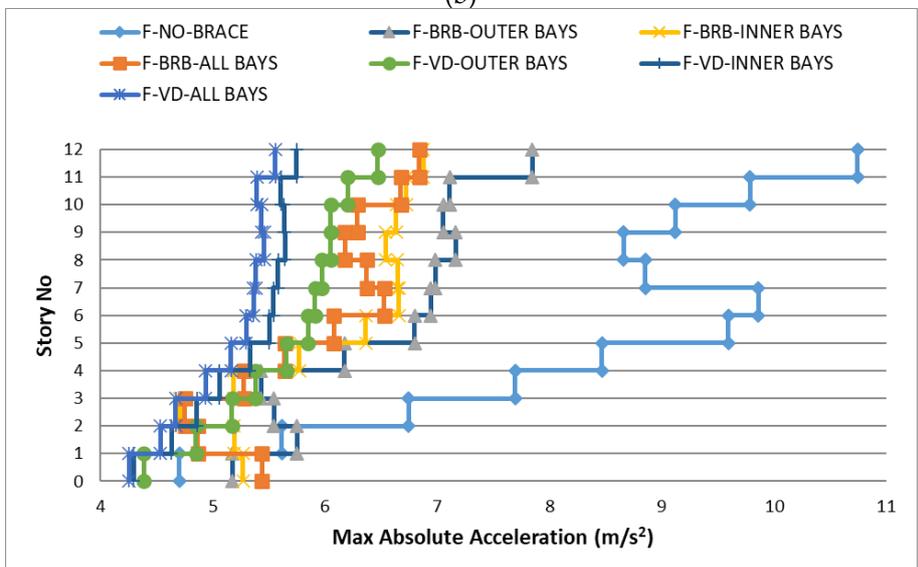
**Figure 16.** Variation of the maximum absolute accelerations against story level of 8-story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



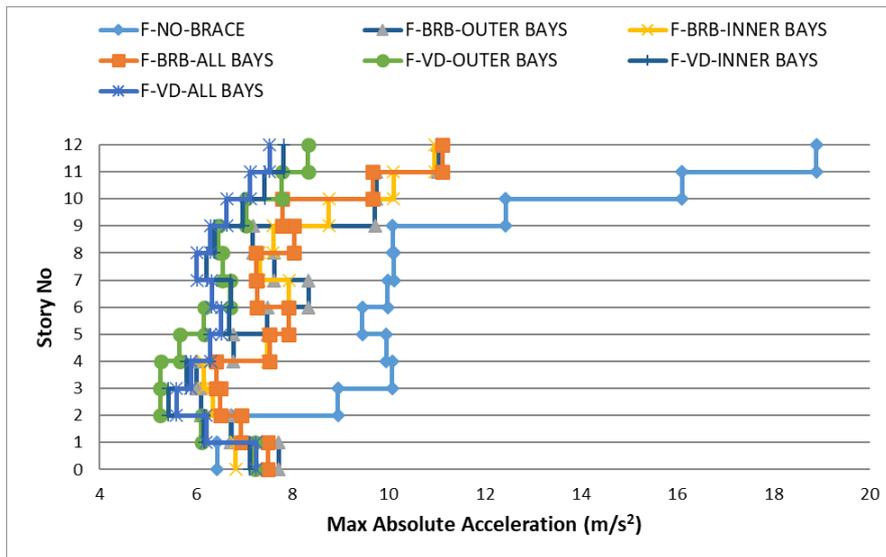
(a)



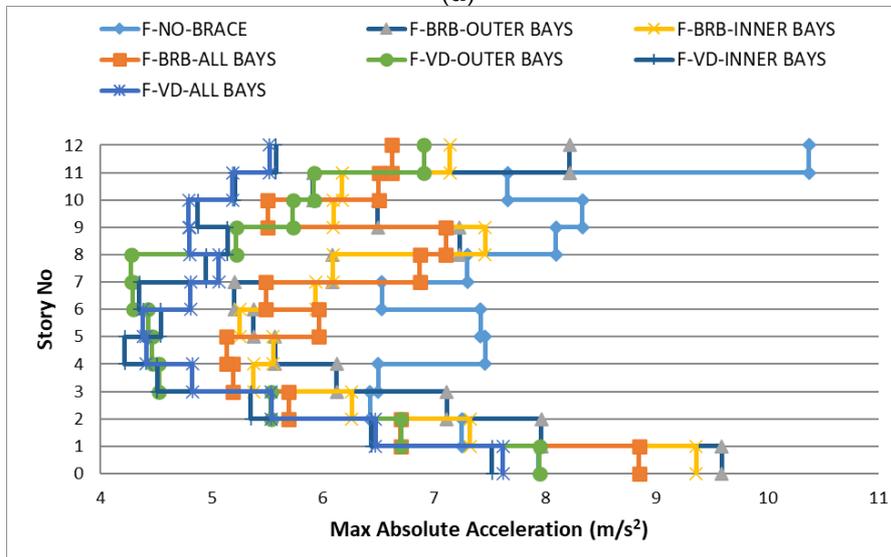
(b)



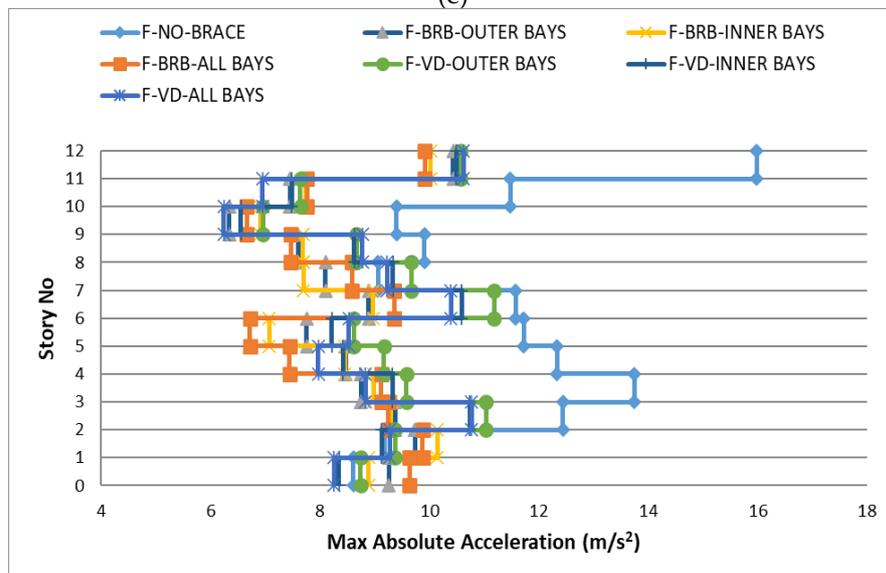
(c)



(d)

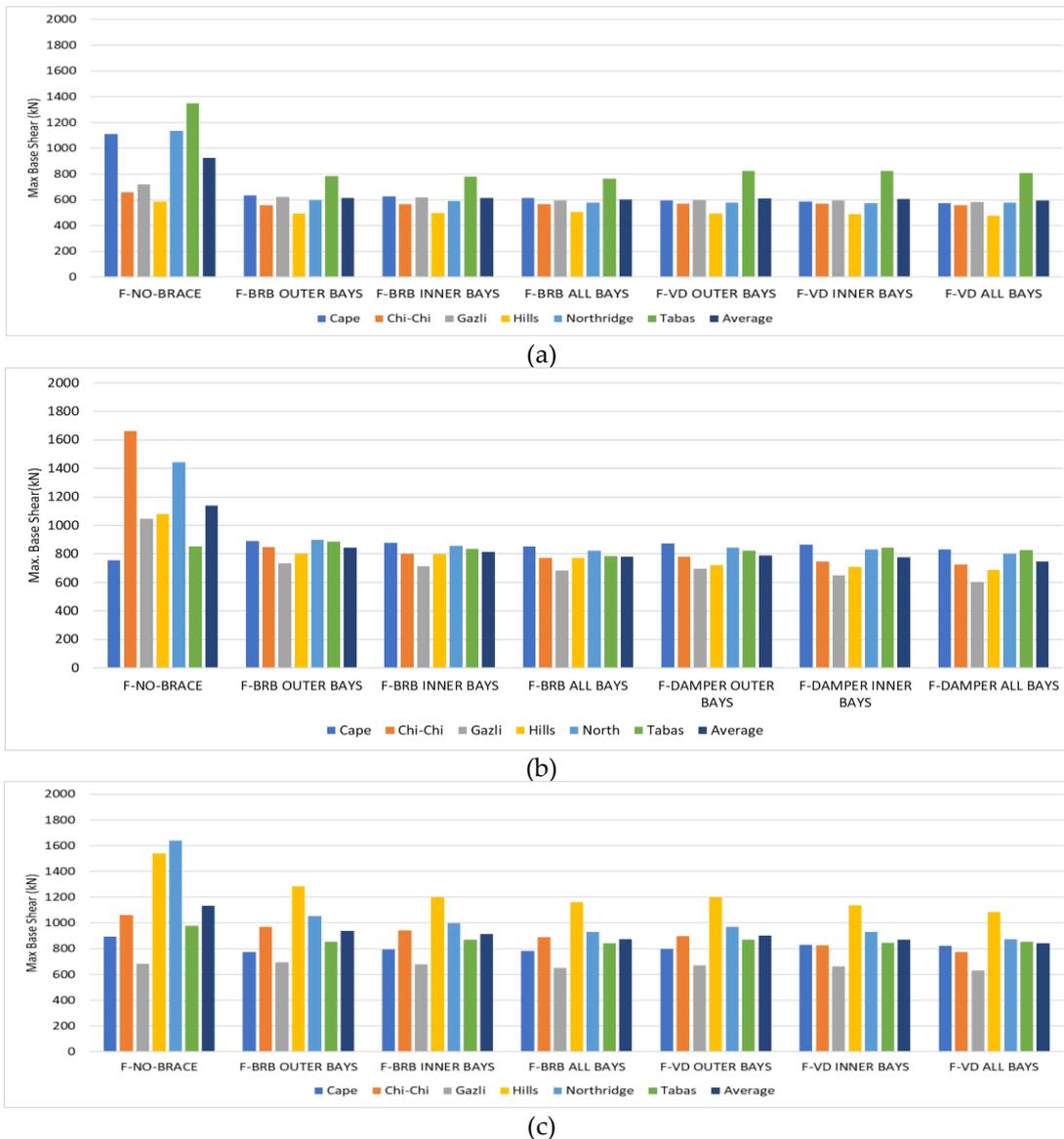


(e)



(f)

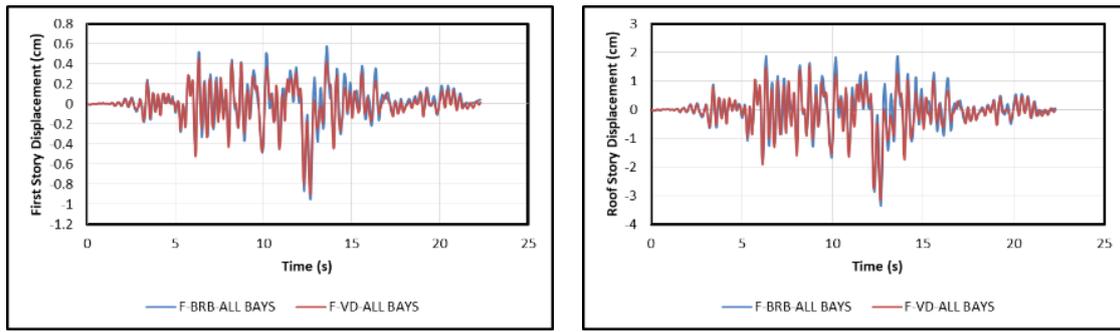
**Figure 17.** Variation of the maximum absolute accelerations against story level of 12- story frames under; (a) Cape, (b) Gazli, (c) Hills, (d) Northridge, (e) Chi-Chi, and (f) Tabas earthquakes.



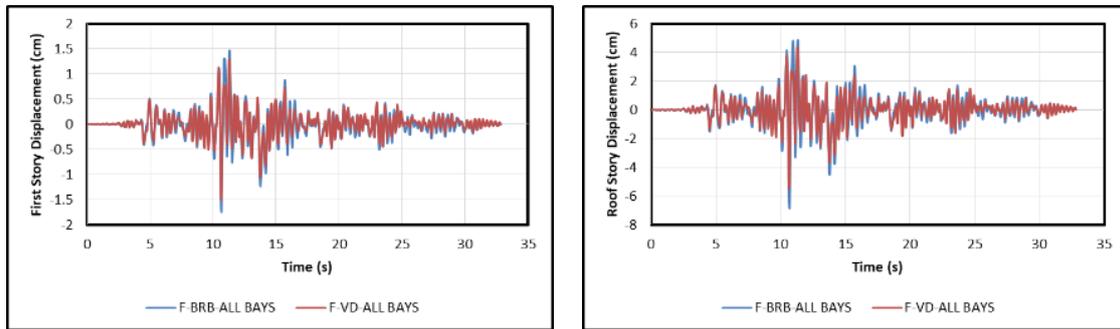
**Figure 18.** The maximum base shear of, (a) 4, (b) 8 and, (c) 12-story frames under earthquakes.

#### 4.6. Time history of the first storey and roof displacements

Time history of the first and the roof story displacements for the 4, 8, and 12-storey structures with BRBs and VDs placed in all-bays configurations under earthquakes showing the minimum and the maximum values are illustrated in Figs. 19 to 21, respectively. It was evident the use of VDs appears to be more effective than BRB in reducing the displacement especially at the roof level, irrespective of the storey numbers in the buildings. As seen in Fig. 19, the lowest displacements at the first and the roof stories of 4 story building are reported to be 1.0 and 3.3 cm as measured under the Hills earthquake. However, the highest displacements at the first and the roof levels are 1.8 and 7.0 cm, respectively as given by the Tabas earthquake. In the case of 8-storey buildings as seen in Fig. 20, the maximum displacement for the first and the roof story reach to 2.5 and 21.0 cm, respectively as being provided by the Northridge earthquake. Moreover, as seen in Fig. 21, Hills earthquake yields the highest first and roof displacements of 3.0 and 39.0 cm, respectively in the case of 12-storey buildings.

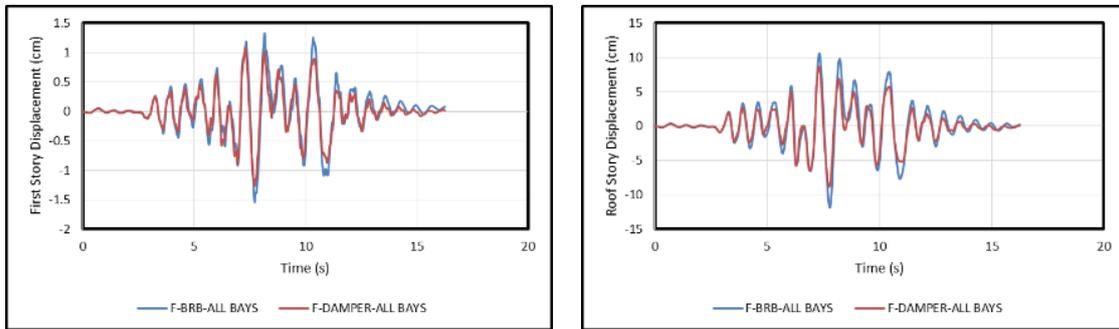


(a)

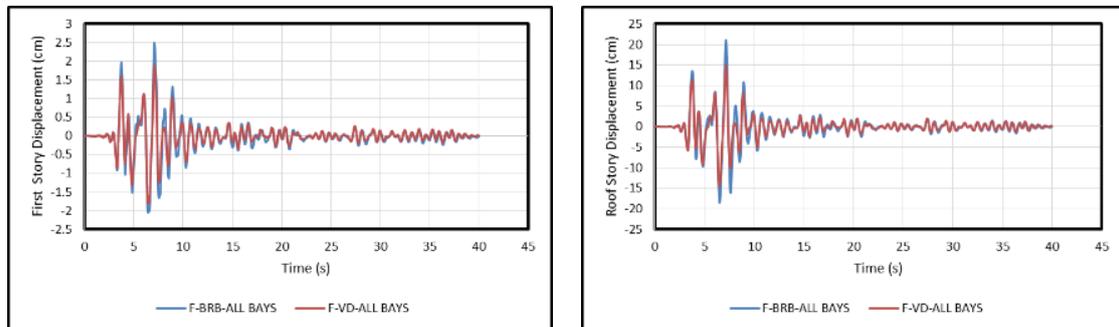


(b)

**Figure 19.** Displacement time history under a) Hills and b) Tabas earthquakes showing the minimum and maximum values for the 4-storey frames with BRBs and VDs

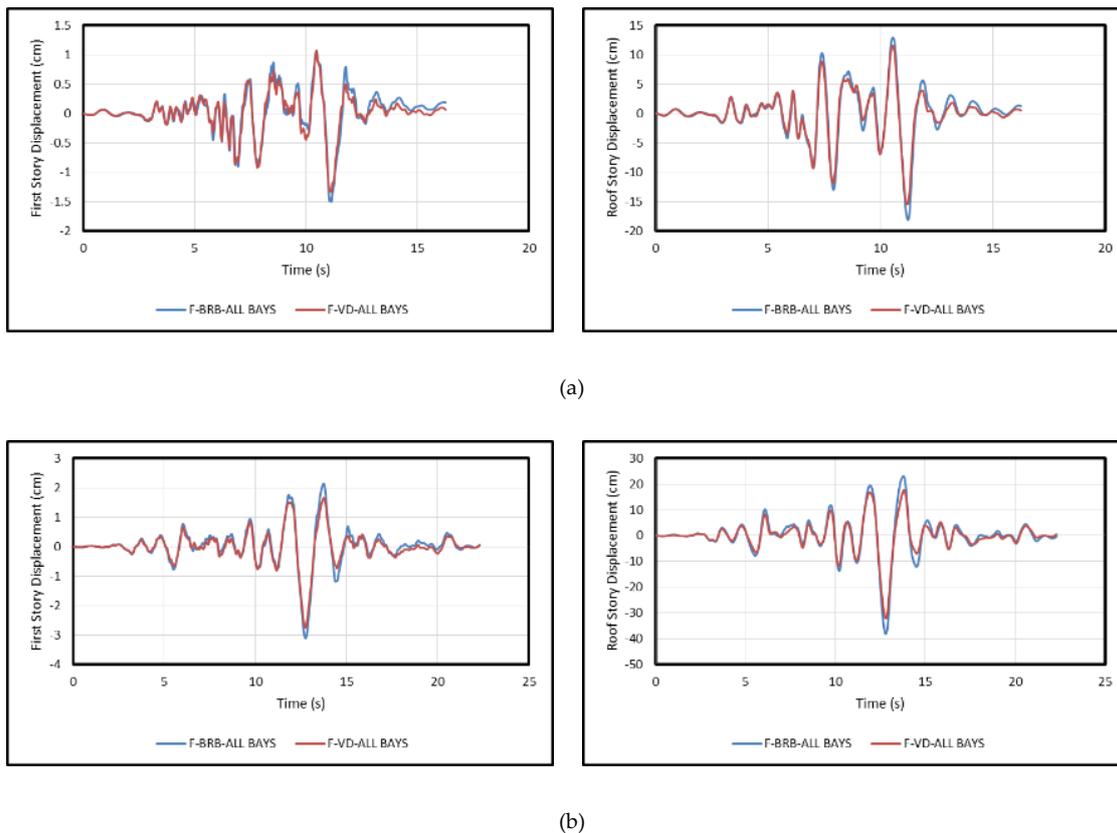


(a)



(b)

**Figure 20.** Displacement time history under a) Gazlı and b) Northridge earthquakes showing the minimum and maximum values for the 8-storey frames with BRBs and VDs



**Figure 21.** Displacement time history under a) Gazlı and b) Hills earthquakes showing the minimum and maximum values for the 12-storey frames with BRBs and VDs

## 5. Conclusions

Based on the analysis of the results, the following conclusions are drawn:

- The bare frames with 4, 8, and 12 stories have much higher lateral displacement demand which reduces with using the BRBs and VDs. This reduction is remarkably affected by the characteristics of the earthquakes, type and pattern of the seismic protection devices. Moreover, the highest contribution of the seismic device is observed in the 12-storey building. The upgraded frames with BRBs and VDs placed in all-bays have less roof displacement in comparison to those in the outer bays and inner bays.
- The frames with BRBs and VDs appear to have much less interstorey drift ratio compared to the bare frames, irrespective of the ground motions. It is observed that the use of VDs or BRBs not only reduces the storey drifts but also provides more uniform distribution over the frames. Similarly, the variation of the storey acceleration decreases remarkably in the case of upgraded frames.
- Using BRBs and VDs in the frame system have influence on the variation of the maximum base shear, depending on the type and configuration of them and building properties.
- In the assessment of structural performance, it is observed that all of the bare frames fail to satisfy the immediate occupancy limit state in all cases of the earthquakes. However, the upgraded frames satisfy the immediate occupancy, particularly for 4 and 12 story structures.

- Among the configurations examined in the study, BRBs or VDs in the case of the all-bays are found to be more influential in diminishing the seismic response of the examined structures.

**Author Contributions:** “Conceptualization, methodology, and validation, O.H.; writing—original draft preparation, O.H.; writing—review and editing, E.M.G.; visualization, O.H.; supervision and project administration, E.M.G. All authors have read and agreed to the published version of the manuscript.”

**Funding:** This research received no external funding.

**Conflicts of Interest:** The authors declare no conflict of interest.

## References

1. Xie Q. State of the art of buckling-restrained braces in Asia. *J Constr Steel Res* 2005;61:727–48. <https://doi.org/10.1016/j.jcsr.2004.11.005>.
2. Aiken ID, Kimura I. *The Use of Buckling-restrained Braces in the United States* 2001.
3. Sahoo DR, Chao SH. Performance-based plastic design method for buckling-restrained braced frames. *Eng Struct* 2010;32:2950–8. <https://doi.org/10.1016/j.engstruct.2010.05.014>.
4. Kumar RG, Satish Kumar SR, Kalyanaraman V. Behaviour of frames with Non-Buckling bracings under earthquake loading. *J Constr Steel Res* 2007;63:254–62. <https://doi.org/10.1016/j.jcsr.2006.04.012>.
5. Kim J, Choi H. Behavior and design of structures with buckling-restrained braces. *Eng Struct* 2004;26:693–706. <https://doi.org/10.1016/j.engstruct.2003.09.010>.
6. AISC (2005). seismic provisions for structural steel buildings, American institute of steel construction, Inc., Chicago, IL. AISC (2005b).
7. Asgarian, B. and Amirhesari, N. (2008). Comparison of dynamic nonlinear behavior of ordinary and buckling restrained braced frames subjected to strong ground motion. *The Structural Design of Tall and Special Buildings*, 17(2), 367- 386.
8. Apostolakis, G.; Dargush, G.F. Optimal seismic design of moment-resisting steel frames with hysteretic passive devices. *Earthquake Engng Struct. Dyn* 2010, 39, 355-376.
9. César, M.B.; Coelho, J.P.; Gonçalves, J. (2019) Semi-active vibration control of a non-collocated civil structure using evolutionary-based belbic. *Actuators*, 8, 1-19.
10. Ghanchi, N.; Kewate, S. Dynamic analysis of 25 storey RCC building with and without viscous dampers. *Int J of Sci Eng Res* 2015, 6, 63-68.
11. Celik OC, Berman JW, Bruneau M. Cyclic testing of braces laterally restrained by steel Studs. *J Struct Eng* 2005;131:1114–24. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2005\)131:7\(1114\)](https://doi.org/10.1061/(ASCE)0733-9445(2005)131:7(1114)).
12. Lin PC, Takeuchi T, Matsui R. Seismic performance evaluation of single damped-outrigger system incorporating buckling-restrained braces. *Earthq Eng Struct Dyn* 2018;47:2343–65. <https://doi.org/10.1002/eqe.3072>.
13. Dicleli M, Mehta A. Seismic performance of chevron braced steel frames with and without viscous fluid dampers as a function of ground motion and damper characteristics. *J Constr Steel Res* 2007;63:1102–15. <https://doi.org/10.1016/j.jcsr.2006.09.005>.
14. Wang H, Feng Y, Wu J, Jiang Q, Chong X. Damage Concentration Effect of Multistory Buckling-Restrained Braced Frames. *Adv Civ Eng* 2019;2019. <https://doi.org/10.1155/2019/7164373>.
15. Sabelli R, Mahin S, Chang C. Seismic demands on steel braced frame buildings with buckling-restrained braces. *Eng Struct* 2003;25:655–66. [https://doi.org/10.1016/S0141-0296\(02\)00175-X](https://doi.org/10.1016/S0141-0296(02)00175-X).
16. Tremblay R, Bolduc P, Neville R, DeVall R. Seismic testing and performance of buckling-restrained bracing systems. *Can J Civ Eng* 2006;33:183–98. <https://doi.org/10.1139/105-103>.
17. Hao H. Performance of Non-Buckling Segmented Brace Members for Mitigating Seismic Responses of Frame Structures 2014.
18. S.Karimi, F.Arbabi. Seismic Evaluation and Cyclic Testing Of Buckling Restrained Braces Manufactured In Iran. 14th World Conf Earthq Eng 2008.
19. Tremblay R, Filiatrault A. Seismic impact loading in inelastic tension-only concentrically braced steel frames: Myth or reality? *Earthq Eng Struct Dyn* 1996;25:1373–89. [https://doi.org/10.1002/\(SICI\)1096-9845\(199612\)25:12<1373::AID-EQE615>3.0.CO;2-Y](https://doi.org/10.1002/(SICI)1096-9845(199612)25:12<1373::AID-EQE615>3.0.CO;2-Y).

20. Hussain S, Van Benschoten P, Al Satari M, Lin S. Buckling Restrained Braced Frame (BRBF) Structures: Analysis, Design and Approvals Issues. 75th SEAOC Annu Conv Sept 13-16 2006.
21. Lin KC, Lin CCJ, Chen JY, Chang HY. (2010). Seismic reliability of steel framed buildings, *Structural Safety*; 32(3):174–182.
22. Deulkar W. N., Modhera C. D. and Patil H S (2010). Buckling restrained braces for vibration control of building structure. *International Journal of Research and Reviews in Applied Sciences*, 4, 363-372.
23. Di Sarno, L. and Manfredi, G. (2010). Seismic Response of Reinforced Concrete Buildings Retrofitted with Dissipative Steel Braces. *Journal of Civil Engineering and Architecture*, ISSN 1934-7359, USA, 4, No.2 (Serial No.27).
24. Symans MD, Charney FA, Whittaker AS, Constantinou MC, Kircher CA, Johnson MW, et al. Energy dissipation systems for seismic applications: Current practice and recent developments. *J Struct Eng* 2008;134:3–21. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2008\)134:1\(3\)](https://doi.org/10.1061/(ASCE)0733-9445(2008)134:1(3)).
25. Crewe A. Passive energy dissipation systems in structural engineering. *Struct Saf* 1998;20:197–8. [https://doi.org/10.1016/s0167-4730\(97\)00034-9](https://doi.org/10.1016/s0167-4730(97)00034-9).
26. Chang KC, Chen SJ, Lai ML. Inelastic behavior of steel frames with added viscoelastic dampers. *J Struct Eng* 1996;122:1178–86. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1996\)122:10\(1178\)](https://doi.org/10.1061/(ASCE)0733-9445(1996)122:10(1178)).
27. Xu ZD. Earthquake mitigation study on viscoelastic dampers for reinforced concrete structures. *JVC/Journal Vib Control* 2007;13:29–43. <https://doi.org/10.1177/1077546306068058>.
28. Xu ZD, Wang DX, Shi CF. Model, tests and application design for viscoelastic dampers. *JVC/Journal Vib Control* 2011;17:1359–70. <https://doi.org/10.1177/1077546310373617>.
29. SaiChethan, K.; Sirinivas, K.S.; Ranjitha, K.P. Seismic performance evaluation of fluid viscous dampers. *Int J of Res in Eng Tech* 2017, 06, 27-32.
30. Prasad, M.L.A. and Mazumder, E.A. Use of viscous damper as an energy dissipative device in steel structures. *Int J of Mechanical and Prod Eng* 2016;4:54-66.
31. Balkanlou, V.S.; Karimi, M.R.B.; Azar, B.B.; (2013) Behraves, A. Evaluating effects of viscous dampers on optimizing seismic behavior of structures. *Int J Current Eng Tec*, 3, 1150-1157.
32. Eurocode 3. Design of steel structures. Part 1.1: General rules and rules for buildings. Brussels (Belgium): European Communities for Standardisation; 2004.
33. Eurocode 8. Design provisions for earthquake resistance of structures. Part 1.3: General rules. Specific rules for various materials and elements. Brussels (Belgium): European Communities for Standardisation; 2004.
34. Karavasilis TL, Bazeos N, Beskos DE. Maximum displacement profiles for the performance based seismic design of plane steel moment resisting frames. *Eng Struct* 2006;28(1):9–22.
35. Deringol AH, Güneş EM. Effect of friction pendulum bearing properties on behaviour of buildings subjected to seismic loads. *Soil Dynamics and Earthquake Eng* 2019; 125:1-20
36. FEMA-356, ASCE- American Society of Civil Engineers. Prestandard and Commentary for the Seismic Rehabilitation of Buildings. *Rehabil Requir* 2000:1–518.
37. SAP 2000 Advanced Structural Analysis Program, Version 12 2010.
38. PEERC. The pacific earthquake engineering research center. User's manual for the PEER ground motion database application. Berkeley: University of California; 2011.
39. American Society of Civil Engineers, ASCE. Minimum design loads for buildings and other structures. ASCE/SEI 7-10, Reston, VA. 2010.
40. Palmer KD, Christopoulos AS, Lehman DE, Roeder CW. Experimental evaluation of cyclically loaded, large-scale, planar and 3-d buckling-restrained braced frames. *J Constr Steel Res* 2014;101:415-425.
41. Christopoulos AS. Improved Seismic Performance of Buckling Restrained Braced Frames. MSc thesis, University of Washington, DC (2005)
42. FEMA 273, N.Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency Washington, DC (1997)
43. Vision 2000, Conceptual Framework for Performance Based Seismic Engineering of Buildings. Structural Engineers Association of California. 2 (1995)