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Seismic upgrading of structures with different retrofitting methods

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Abstract. This paper presents an analytical study aimed at evaluating the seismic performance of steel moment resisting frames (MRFs) retrofitted with different approaches. For this, 3, 6 and 12 storey MRFs having four equal bays of 5 m were selected as the case study models. The models were designed with lateral stiffness insufficient to satisfy code drift and hinge limitations in zones with high seismic hazard. Three different retrofit strategies including traditional diagonal bracing system and energy dissipation devices such as buckling restrained braces and viscoelastic dampers were used for seismic upgrading of the existing structures. In the nonlinear time history analysis, a set of ground motions representative of the design earthquake with 10% exceedance probability in fifty years was taken into consideration. Considering the local and global deformations, the results in terms of inter-storey drift index, global damage index, plastic hinge formations, base shear demand and roof drift time history were compared. It was observed that both buckling-restrained braces and viscoelastic dampers allowed for an efficient reduction in the demands of the upgraded frames as compared to traditional braces.

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

1. Introduction

During the lifetime of a structure, many severe events such as earthquakes and winds that might influence to a structural system may impact the structural performance and cause potential damage (Bitaraf *et al*. 2012). To mitigate the structural vibration and damages during these extreme events, new and innovative concepts of energy dissipation devices for use as part of structural protection systems have been employed at various design stages (Yang *et al*. 2010). These energy dissipation systems can be classified into two main parts, namely, active and passive. Passive dissipation devices use displacement (rate-independent) or velocity (velocity dependent) between attachment points to generate control forces or energy dissipation. Rate-independent devices including metallic yielding and friction components provide initial stiffness but energy does not dissipate until yielding or slip. Velocity-dependent devices include viscous fluid and viscoelastic solid devices. These devices can provide high levels of motion, but do not possess the same energy

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dissipation capacity as metallic or friction devices (Marshall and Charney 2009).

Conventional braces (CBs) exhibit buckling deformation when loaded with large compression forces (Martinelli *et al.* 1998, FEMA-450 2003) and show unsymmetrical hysteresis behaviour in tension and compression, and typically the load resisting capacities are reduced when loaded cyclically or monotonically in compression (Qiang 2005, Asgarian and Amirhesari 2008). In order to overcome this problem, a relatively new lateral force resisting system in United States such as buckling-restrained brace (BRB) system has been researched since 1994 Northridge and California earthquakes and has been quickly accepted for practical applications in high seismic regions (Kiggins and Uang 2006). The BRB systems differ from CB systems in that the braces are capable of yielding in both tension and compression instead of buckling, as shown in Fig. 1 (Qiang 2005).

The idea behind BRBs is to fabricate a structural element that is able to work in a stable manner when subject to compressive deformations. Fig. 2 shows schematically the concept of a BRB as well as its different components such as: a) a ductile steel core that dissipates energy through axial deformation (only the core of the brace should yield); b) mortar, concrete or grout fill that restrict buckling of the core; and c) a steel jacket that confines the mortar, concrete or grout fill and provides further restriction to buckling. Usually the steel core is isolated from the mortar, concrete or grout fill in an attempt to minimize or eliminate the transfer of axial stresses between both materials; in order to keep the compression strength of the brace same as tension strength (Teran-Gilmore and Ruiz-Garcia 2011). In the literature, various theoretical and experimental studies have been available to show the response of different BRBs under monotonic and cycling loading (Zhao *et al.* 2014, Gu *et al.* 2014).

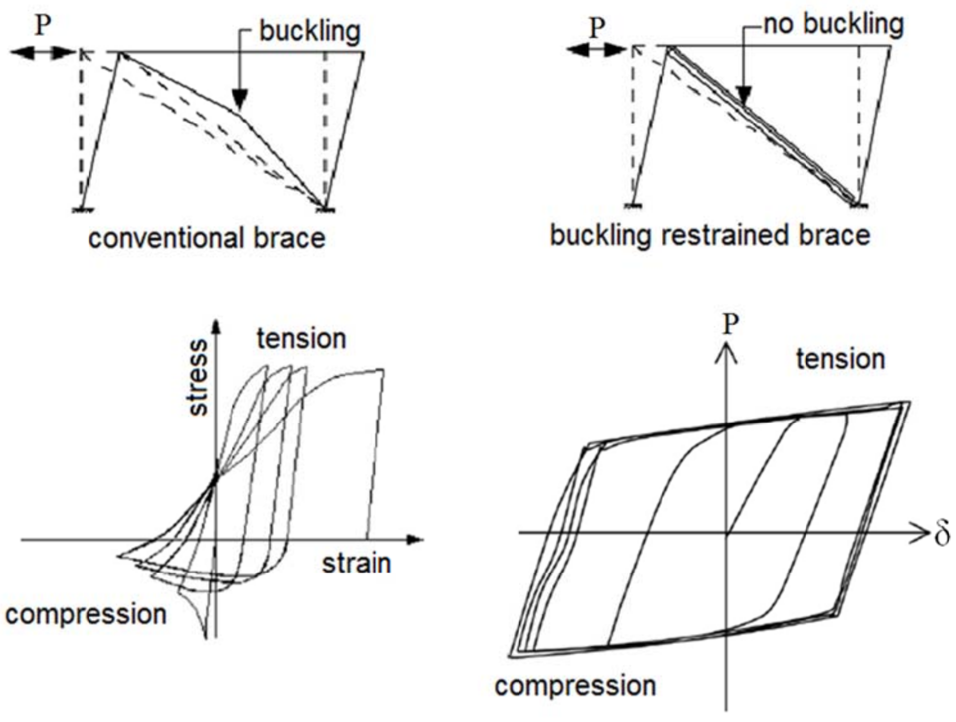


Fig. 1 Behaviour of a) conventional brace and b) buckling restrained brace (Qiang 2005)

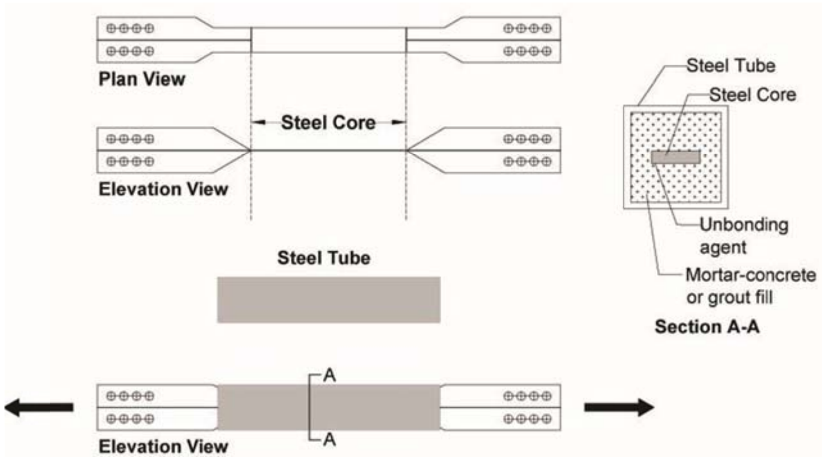


Fig. 2 Schematic configuration of BRB (Tremblay *et al.* 2006)

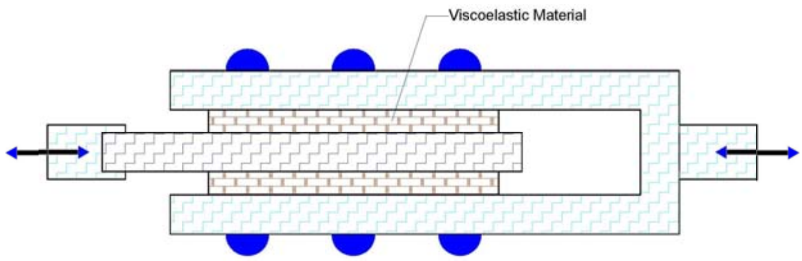


Fig. 3 Schematic configuration of viscoelastic damper (Langenbach and Kelley 1991)

Another innovative technique used to improve energy-absorption capacity of the structures is the utilization of viscoelastic damper in which a highly dissipative polymeric material is incorporated. The helical springs and the accompanying viscoelastic dampers have been employed by Gerb (1978) for energy dissipation purpose under machine foundation (Tezcan and Uluca 2003). These devices contain viscoelastic material with certain viscosity characteristics. The most commonly used materials are acrylic copolymers. The materials are known to be very stable with good aging properties, are chemically inert and are also resistant to environmental pollutants. When used as the energy-absorbing components in dampers, they are normally used in the form of shear layers and the exposed surface area is very small relative to the volume of material. Thus, any chemical process that depends on diffusion, for example, moisture absorption or penetration, would be very slow, as shown in Fig. 3 (Langenbach and Kelley 1991). In this regard, viscoelastic dampers are known to be used in new buildings to alleviate the harmful effect of the earthquakes (Nagarajaiah and Narasimhan 2007), in bridges (Madhekar and Jangid 2009), and also in retrofit of existing structures (Malhorta *et al.* 2004, Potty and Nambissan 2008). Dampers have proven to be effective systems for reducing earthquake forces in structures (Chandra *et al.* 2000). Moreover, the performance of dampers near collapse during large seismic events has proven feasible by analytical and experimental investigation (Armouti 2013).

As highlighted above, the use of bracing systems is a very effective upgrading strategy to enhance the global stiffness and strength of the structures. They have been successfully employed in order to minimize the effect of earthquake and wind forces. However, traditional braces may exhibit buckling deformation when loaded under large axial force and typically exhibit substantial strength deterioration and transmit very high forces to connections and foundations. Hence, connections and foundations require frequent strengthening. On the other hand, the application of the energy dissipation devices (i.e., BRBs and VDs) can be considered as a viable alternative with respect to the traditional systems. In this regard, especially for the seismic retrofitting purposes, further analytical researches are still needed to better understand the performance and effectiveness of using different retrofitting strategies.

The main goal of this research is to investigate the structural performance of the existing moment resisting frames before or after retrofitting with three different techniques, namely, traditional braces, buckling restrained braces, and viscoelastic dampers. To achieve this objective, the influence of the retrofitting methods on 3, 6, and 12 storey steel framed residential buildings was assessed. Nonlinear time history analyses were carried out with three different earthquakes to examine the dynamic responses of the structures. Seismic demands on the bare and upgraded frames were estimated in terms of plastification, maximum inter-storey drift indices, height-wise distribution of lateral drifts, and base shear. The case study provided in this paper allows for a better understanding of the rational retrofitting situations in terms of structural performance and efficiency for the steel moment resisting frames.

2. Description of structural models

The structural models selected for this study were 3, 6 and 12 storey steel moment resisting frames. The steel moment resisting frames were designed according to UBC (1997) with lateral stiffness insufficient to satisfy code drift and hinge limitations in zones with high seismic hazard. They have the same floor plan (4×4 bays) with 5 m bay spacing, whereas the height is 4.2 m at the ground storey and 3.2 m at the remaining floors. The building height is 10.6, 20.2, and 39.4 m for 3, 6, 12 storey structures, respectively. The design dead load and the live load for the storey slabs was taken as 4.2 kN/m^2 and 2.0 kN/m^2; whereas for the roof slab they were taken as 3.6 kN/m^2 and 0.5 kN/m^2, respectively. The frames were assumed to have uniform mass distributions. Steel having nominal yield strength equal to 345 MPa and modulus of elasticity 200 GPa were used for the structural steel members. Fig. 4 shows the elevation views of all structures. The fundamental period of vibration obtained by eigen value analysis for 3, 6 and 12 storey frames were obtained as, 0.74, 1.25, 2.15 s, respectively.

For upgrading the seismic behavior, three types of the retrofitting approaches including conventional braces (CBs), buckling-restrained braces (BRBs), and viscoelastic dampers (VDs) were considered by inserting such devices diagonally in two middle bays of unbraced frame (UBF) as seen in Fig. 5. For conventional braces, square hollow sections of 6”×6”×3/16”, 8”×8”×3/16” and 10”×10”×3/16” were used in 3, 6 and 12 storey frames, respectively. In order to compare the efficiency of conventional and buckling restrained braces, the BRBs were designed for the same capacity with the conventional braces in tension. Two dimensional analytical models of the frames were developed by using SAP 2000 Nonlinear version 14.0, which is a general purpose structural analysis program (CSI 2009). In the analytical models, the columns and beams were modeled with frame elements whereas the braces were modeled as truss elements. For modeling the material nonlinearities of the structural members, the lumped plasticity approach, which is characterized by addition of discrete nonlinear hinges at predetermined locations, was followed. Accordingly the nonlinear behavior of the beam and column members was defined at concentrated plastic hinges.

Properties of plastic hinges were defined in accordance with FEMA 356 (FEMA-356 2000). P-Δ effects were considered by modeling a fictitious column which is connected to the moment resisting frame using rigid links with pin ends. For each storey level, this column was hinged both at its top and bottom; and was considered to carry the gravity loads in the building not directly acting on the moment frame.

For nonlinear behavior of the braces, two types of elements were used: one with an elasto-plastic load deformation curve (Fig. 6(a)) for modeling of BRBs (Kumar *et al.* 2007) and the other with the phenomenological model (Fig. 6(b)) for modeling of CBs (Jain *et al.* 1980, Rai and Goel 2003, Kim and Chio 2005). As in the study of Kumar *et al.* (2007) for damage evaluation of BRBs, being on the conservative side, hinge specifications in FEMA 273 (1997) for modeling braces in tension were used. The limit strains of the BRB for the calculation of the performance level, was taken as $12\varepsilon_y$ for ultimate failure, $10\varepsilon_y$ for collapse prevention, $8\varepsilon_y$ for life safety and ε_y for immediate occupancy performance levels.

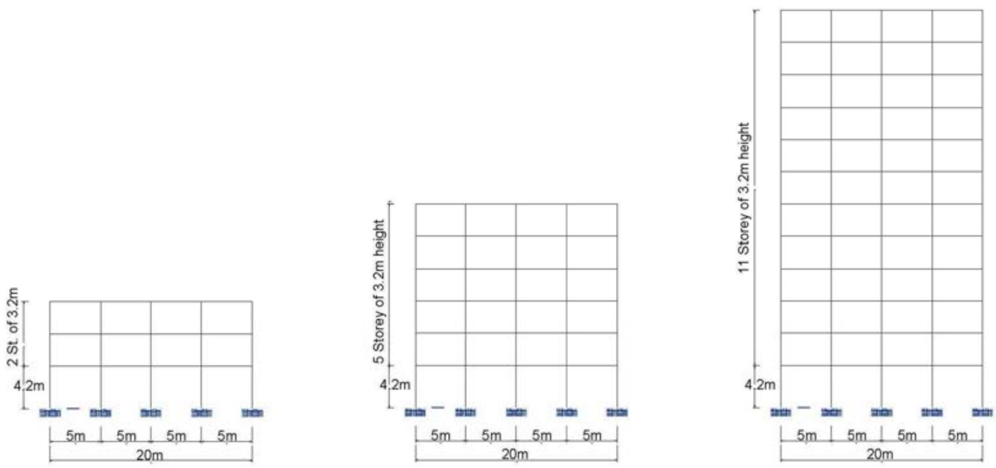


Fig. 4 Elevation view of the structural models under consideration

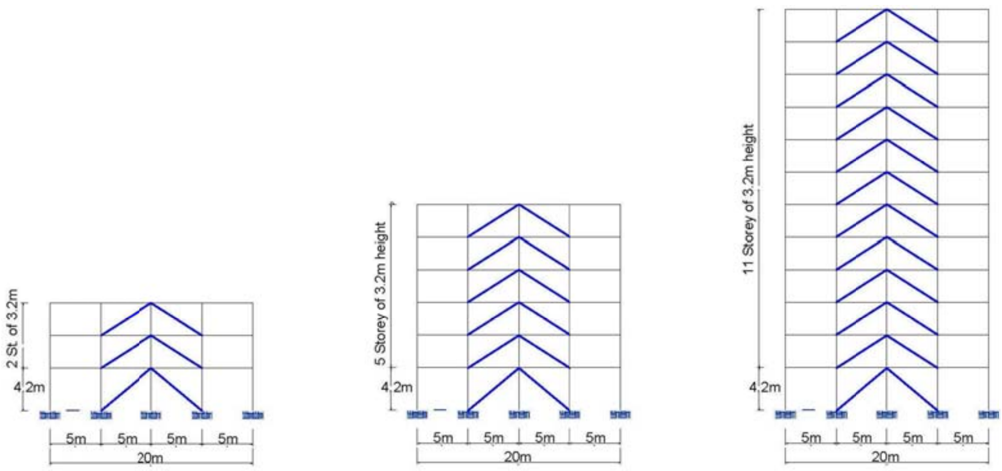


Fig. 5 Bracing configurations for the retrofitted frames

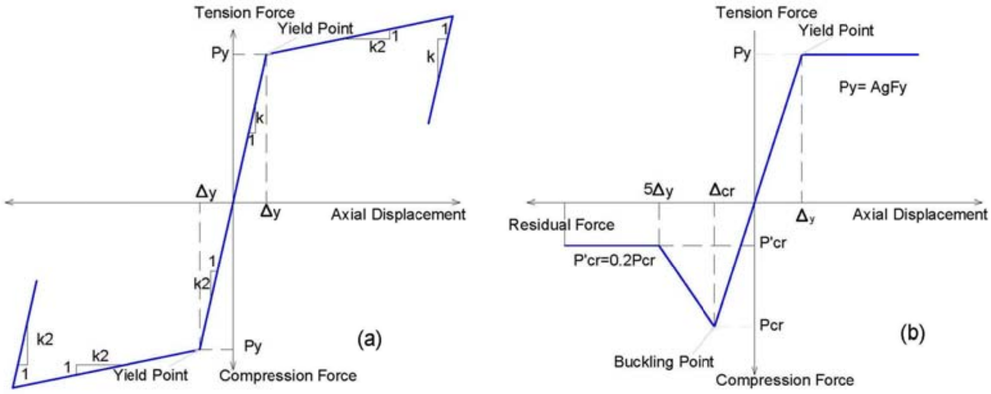


Fig. 6 (a) Constitutive model for BRBs (Kumar *et al.* 2007) and (b) phenomenological model for CBs (Kim and Chio 2005)

Table 1 Damping coefficient (c) and corresponding effective damping ratio (ζ)

Frame no	Three-storey frame		Six-storey frame		Twelve-storey frame	
	c (kNs/m)	ζ (%)	c (kNs/m)	ζ (%)	c (kNs/m)	ζ (%)
VDF1000	1000	15.77	1000	10.9	1000	11.9
VDF2500	2500	25.53	2500	17.5	2500	17.8
VDF10000	10000	35.43	10000	33.6	10000	37.1

The viscoelastic dampers (VDs) were modeled by Link members. In these members, it is possible to identify independent damping properties for each deformational degree of freedom. Viscoelasticity is defined with the Maxwell model that consists of linear or nonlinear damper in series with a spring. The nonlinear behaviour of the VDs is defined according to the following equation

$$f = kd_k = cv^{cexp} \tag{1}$$

In this equation, f is the nonlinear force, k is the spring constant, d_k is the deformation across the spring, c is the damping coefficient, v is the velocity across the damper, and $cexp$ is the damping exponent. The sum of the spring deformation (d_k) and the damper deformation (d_c) gives the deformation of the link member. The rational selection of the damping exponent is in between 0.2 and 2.0. (CSI 2009, Teran-Gilmore and Ruiz-Garcia 2011, Tezcan and Uluca 2003) In this study, the damping exponent was assumed as 1.0. For seismic upgrading, three different VDs with damping coefficients of 1000, 2500 and 10000 kNs/m were utilized. Equal structural bracings having the same stiffness coefficient with the CBs were used for the viscoelastically damped frames with the intention of focusing on the effect of VDs.

The damping ratio provided by the VDs is achieved by following an analogy of the logarithmic decrement method for single degree of freedom (SDOF) system. In the computation of the damping ratio, a decreasing sine curve of SDOF system which defined as displacement versus time was utilized as given in the following equation (Tezcan and Uluca 2003)

$$\mathrm{Ln}\left(\frac{u_1}{u_2}\right) = 2\pi\zeta/\sqrt{(1-\zeta^2)} \tag{2}$$

In this equation, u_1 and u_2 are the peak displacements at two consecutive time periods, ζ is the effective damping ratio. In the computer program, by altering the damping coefficient c of VDs, the effective damping ratio ζ is altered likewise. The damping coefficients and the corresponding effective damping ratios obtained for each frame are summarized in Table 1.

3. Ground motion records

For the nonlinear dynamic analysis of the frames, a set of ground motions (1984 Morgan Hill, 1992 Erzincan, and 1999 Hector Mine) with different intensities and characteristics were

Table 2 Properties of the earthquake ground motion records

Earthquake Record	Station	Magnitude (M_w)	R_{rup}* (km)	V_{s30}** (m/s)	PGA (g)	PGV (m/s)	PGD (m)	Scale Factor
Erzincan	Erzincan	6.7	4.4	274.5	0.48	0.52	0.19	1.5
Morgan Hill	Agnews State Hospital	6.2	24.5	239.7	0.38	0.58	0.26	1.9
Hector Mine	Beverly Hills Pac Bell Bsmt	7.1	191.4	301	0.51	0.92	0.78	2.0

*Restrict range of closest distance to rupture plane
**Average shear wave velocity of top 30 m of the site

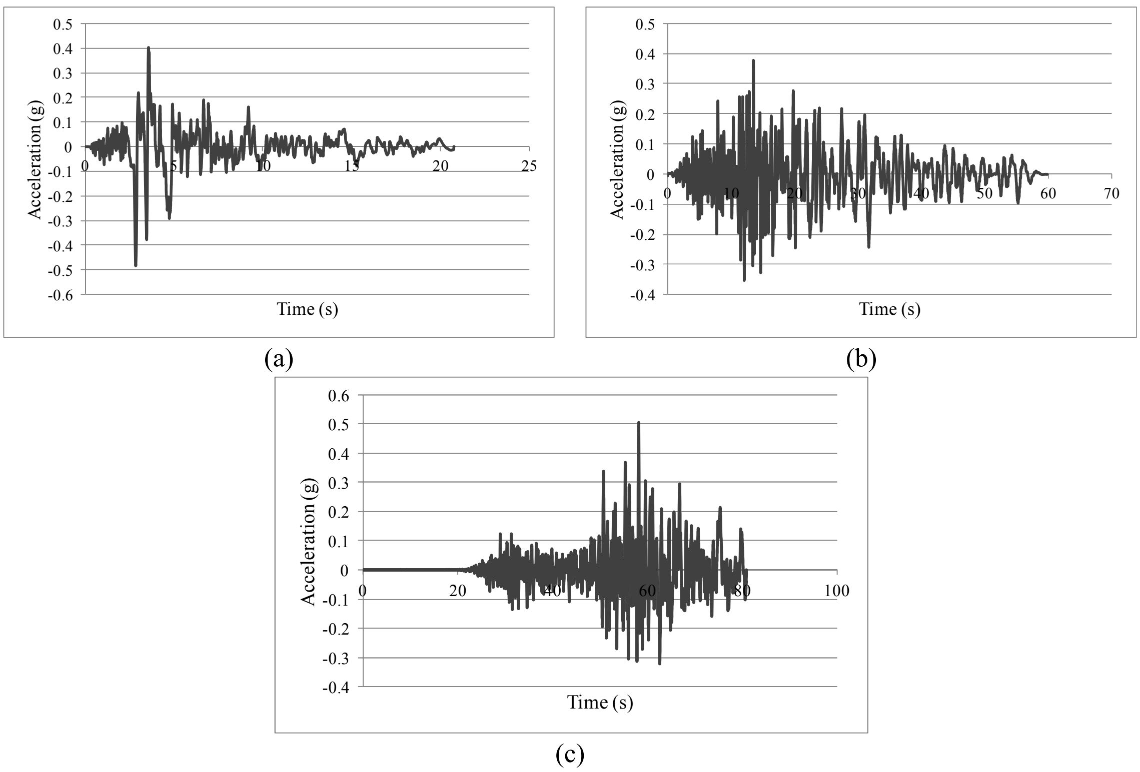


Fig. 7 Earthquake ground motion records used for the analysis: (a) Erzincan, (b) Morgan Hill, and (c) Hector Mine

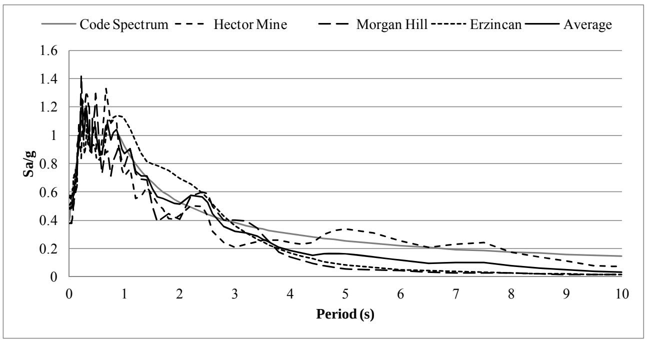


Fig. 8 Code spectrum and average acceleration spectrum of the scaled earthquake ground motions

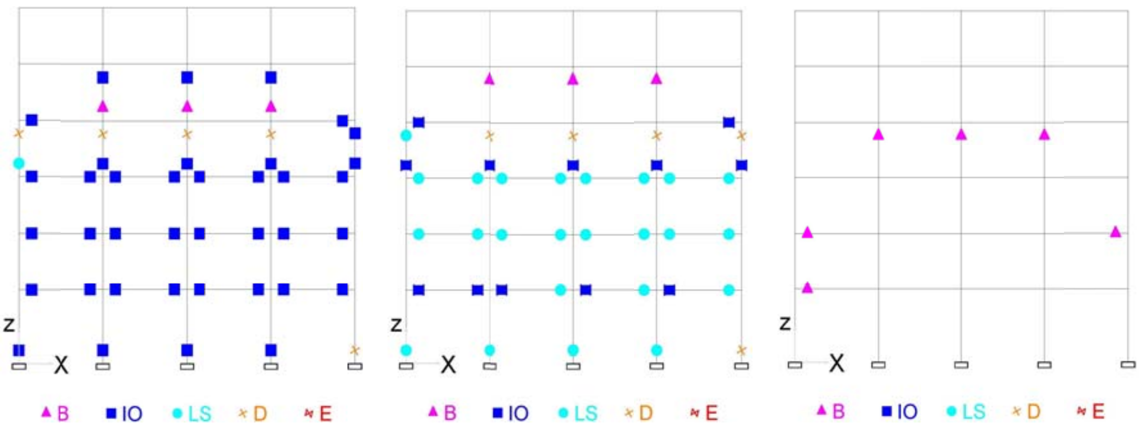
considered. As seismic hazard level, the acceleration spectrum defined in Turkish Earthquake Code (TEC 2007) for 10% probability of exceedance in 50 yrs was taken into account to assess the earthquake performance of the existing and retrofitted structures. Scaled earthquake records in order to match this target spectrum mainly in a range of periods from 0.2T to 1.5T (where T is the first natural period of the building) were obtained from PEER ground motion database (PEER 2011). Fig. 7 shows the earthquake acceleration time histories used for the analysis. The 5% damped average acceleration spectrum of the ground motions used, and the elastic code spectrum is given in Fig. 8. Furthermore, Table 2 lists the scale factor and characteristic of the earthquake ground motion records considered in this study.

4. Results and discussion

Considering the structural models and excitations described in this study, a comparison between different retrofitting strategies was made. The primary response parameters considered are plastic hinge formations, maximum inter-storey drift ratio, maximum roof drift, base shear demand, and roof drift time history. For this, a series of nonlinear dynamic analysis was performed using three different ground motion records for 18 different cases. These cases cover three storey, six storey, and twelve storey unbraced frames (UBFs), conventional braced frames (CBFs), buckling-restrained braced frames (BRBFs), and viscoelastically damped frames (VDFs) with three different damping coefficients of 1000, 2500, and 10000 kNs/m.

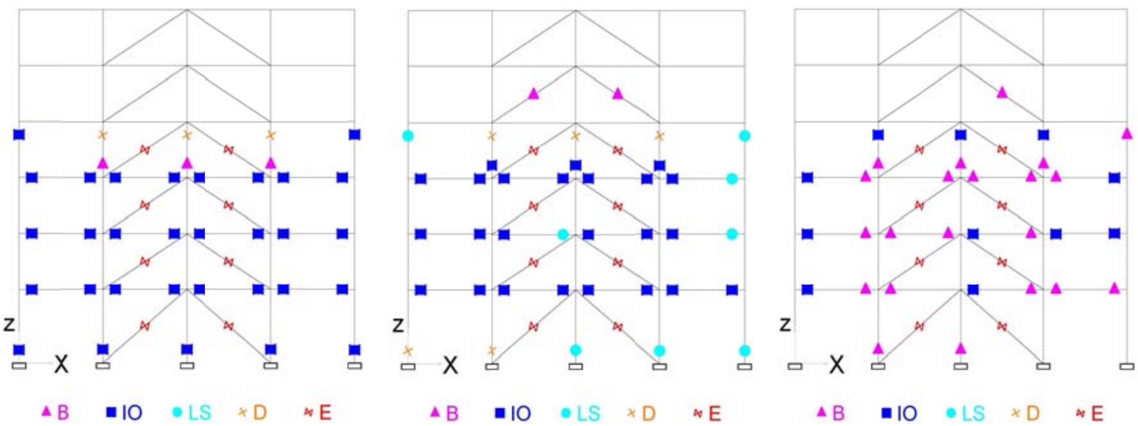
In order to address the local deformations of the structural members, the plastifications of the structural members were investigated. Plastic hinge formations and their performance levels for the 6 storey frames considered in this study are shown in Figs. 9-14. The notations IO, LS, and CP stand for immediate occupancy, life safety, and collapse prevention performance levels, respectively. The results revealed that especially the buckling restrained braces and the viscoelastic dampers with high damping coefficients are effective in decreasing the plastification demand of the unbraced frames (UBFs). It was also observed that, depending on the earthquake acceleration, the plastification demand of the UBFs changed. For instance, for the 6 storey UBF, Morgan Hill and Hector Mine earthquake accelerations impose larger plastic hinge formation demand compared to Erzincan earthquake acceleration. As seen in Fig. 9, under the effect of Morgan Hill and Hector Mine earthquake accelerations, four columns at the fourth storey reached the collapse stage whereas under the effect of Erzincan earthquake acceleration, the frame was in the elastic range of deformation.

Comparing the performance of different seismic resisting systems considered in this study, it was observed that viscoelastic dampers (VDs) with damping coefficient 10000 kNs/m (c=10000) offers the best reduction of the plastic hinge rotations in beams and columns for 3, 6 and 12 storey structures. Furthermore, the BRBs and VDs with damping coefficient of 2500 kNs/m provided considerable improvement in seismic response of the UBFs by decreasing the plastification demand in critical structural members. Generally, this fact holds true independent of the frame height and the exposed earthquake acceleration. For example, the frames upgraded with BRBs remained mainly in the elastic range of deformation except the BRBs. In the VDF with c=2500 kNs/m, under Hector Mine and Morgan Hill earthquakes some plastifications mainly in the beams; under Erzincan earthquake, complete elastic behaviour was observed as seen in Fig. 13. On the other hand, the VDF with c=1000 kNs/m was not effective for upgrading the seismic behaviour of



(a) (b) (c)

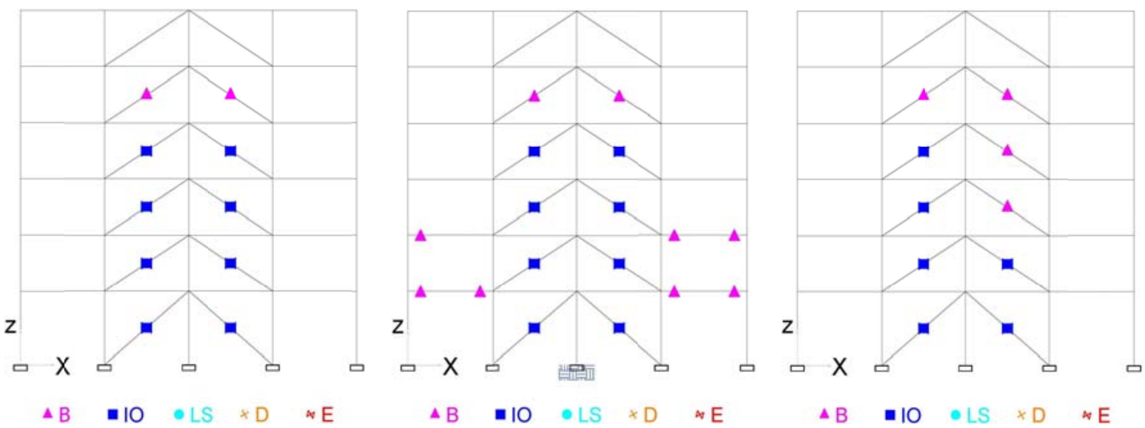
Fig. 9 Plastic hinge formation for 6 storey UBFs subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes



(a) (b) (c)

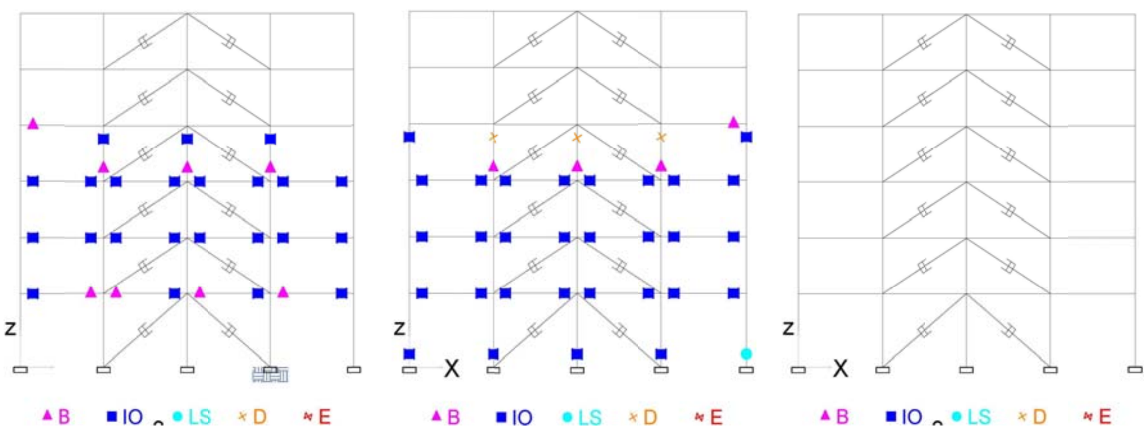
Fig. 10 Plastic hinge formation for 6 storey CBFs subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes the UBFs resisting system as shown in Fig. 12. Addition of CBs to the unbraced frames was found to be the least effective in reducing the plastification. The poor behaviour of conventional braced frames (CBFs) is due to buckling of the CBs. Conventional brace systems that have different load carrying capacity in compression and tension, exhibit buckling deformation in compression. The buckling of the braces resulted in a sudden transferring of the input energy to the beams and columns and caused formation of plastic hinges in the other structural members that might also cause the failure of the building. The significantly improved behaviour of the BRBF comes from the enhanced nonlinear hysteretic performance and energy absorption capacity of the BRBs when loaded in excess of compression yield capacity.

A wide consensus exists in the earthquake engineering community that for most of the structures, the displacement demand can be an indicator of expected damage. As an indication of displacement demand, the ratio of the roof displacement to the building height (D/H) called as



(a) (b) (c)

Fig. 11 Plastic hinge formation for 6 storey BRBFs subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes



(a) (b) (c)

Fig. 12 Plastic hinge formation for 6 storey VDF with c=1000 kNs/m subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

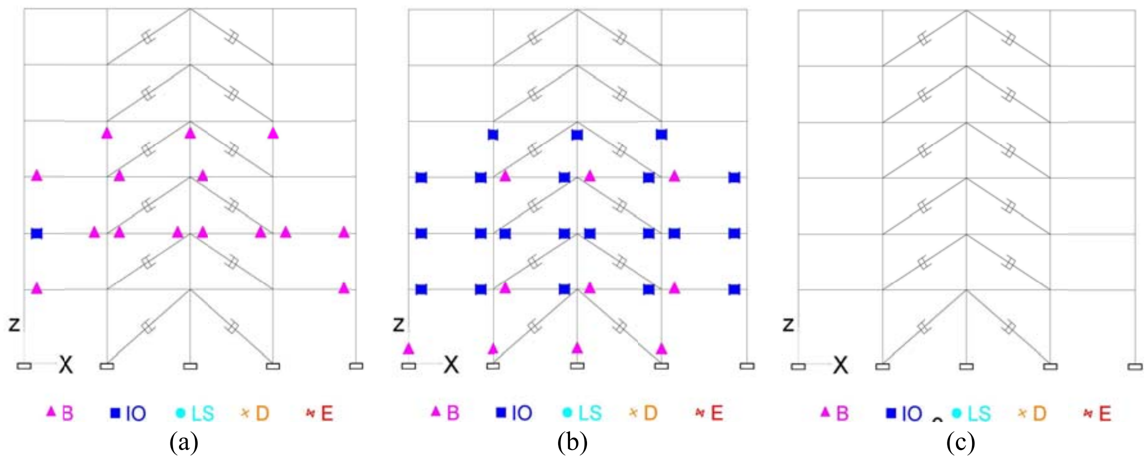


Fig. 13 Plastic hinge formation for 6 storey VDF with c=2500 kNs/m subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

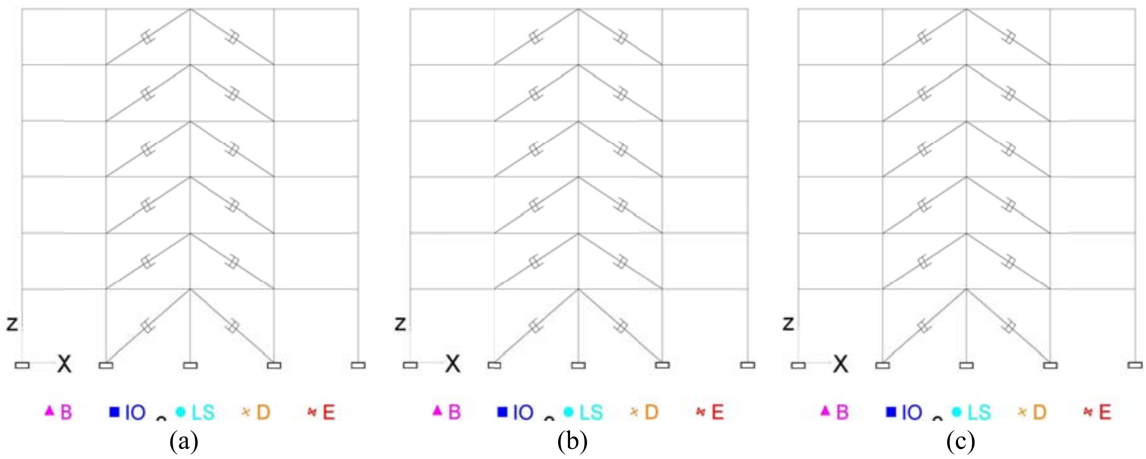


Fig. 14 Plastic hinge formation for 6 storey VDF with c=10000 kNs/m subjected to (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

global damage index and the maximum inter-storey drift ratio (d/h) also called as inter-storey drift index has been generally evaluated. Even though both of them have been used as a global damage parameter, the inter-storey drift index has been much more preferred than the global damage index, since in an individual storey it may exceed the latter by a factor of 2 or more (Krawinkler and Gupta 1988, Barroso *et al.* 2002). In this study, both the inter-storey drift index and the global damage index have been utilized and compared for evaluation of the seismic response of the frames.

As expected, the dynamic responses of the frames can vary depending on the earthquake acceleration used. In this study, generally under Morgan Hill earthquake acceleration, the greater inter-storey drift index and global damage index was observed for the frames with and without passive energy dissipation systems. The inter-storey index and the global damage index obtained for 3, 6 and 12 storey frames are presented in Fig. 15. As seen from Fig. 15, generally the frames with passive energy control devices had inter-storey index and damage index values less than those of the unbraced frames. However, the decrease in the displacement demand in terms of inter-storey drift index and/or global damage index could not be seen for 3 and 6 storey CBFs. Such increment in the displacement demand of these CBFs particularly refers to the poor nonlinear behavior of conventional braces under large compressive stresses.

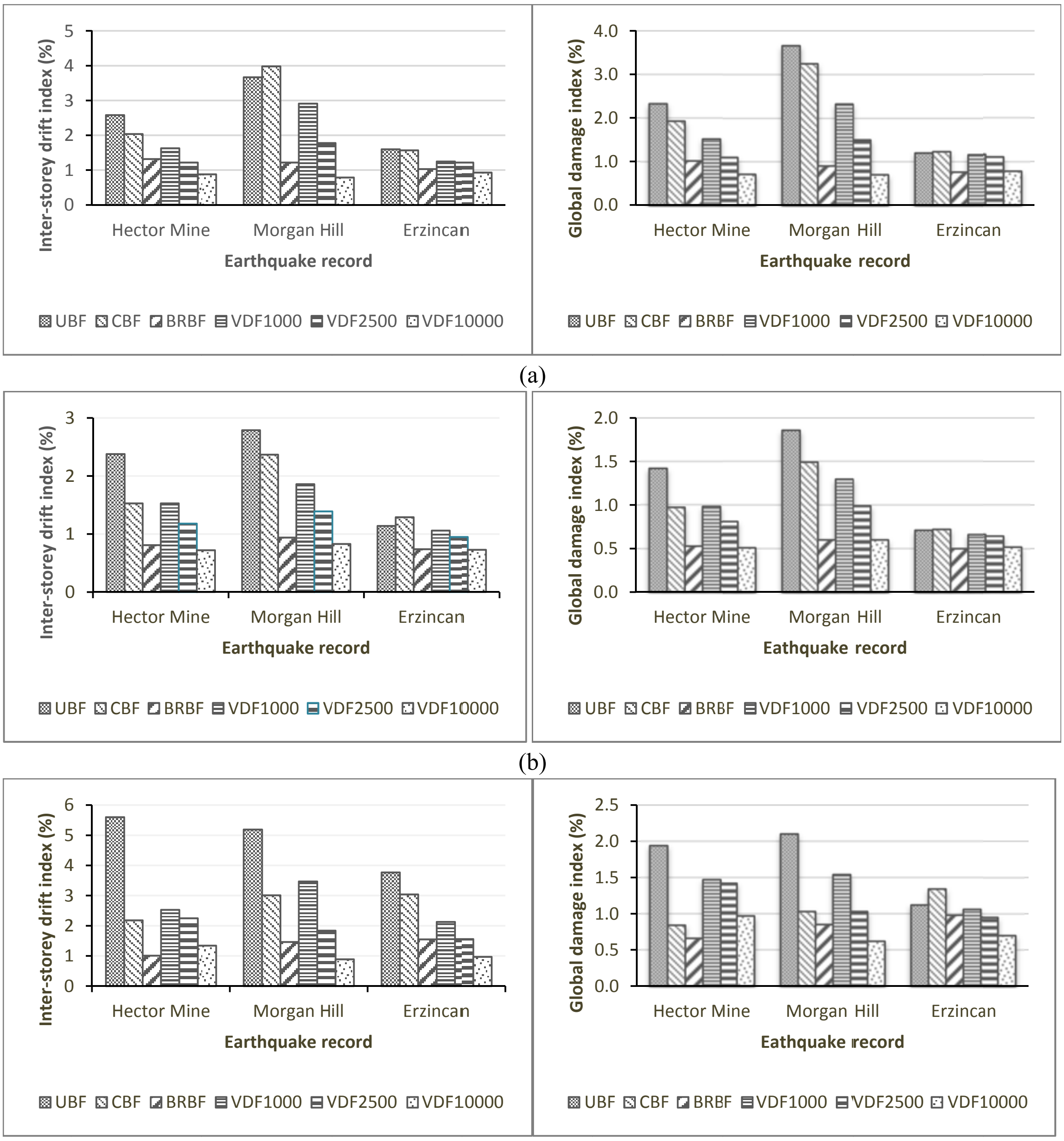


Fig. 15 Inter-storey drift index (*left*) and global damage index (*right*) for (a) 3, (b) 6 and (c) 12 storey frames

Among the passive energy dissipation systems utilized, the BRBs and VDs with damping coefficient of c=10000 kNs/m appear to be very effective in reducing the displacement demand of the UBFs. For example, with the addition of BRBs, a significant reduction in inter-storey drift index values was obtained especially when subjected to Hector Mine and Morgan Hill earthquakes. The average reduction obtained in the inter-storey drift index with respect to the original frame is 60.7% and 66.4% by the use of BRBs and VDs with damping coefficient of c=10000 kNs/m, respectively. While the reduction of global damage index with respect to the original frame on average equal to 52.4% and 55.8% for BRBFs and VDFs with damping coefficient of c=10000 kNs/m, respectively.

Figs. 16 to 18 provide roof drift time-history response of the 3, 6 and 12 storey frames before and after addition of traditional and innovative systems when subjected to Hector Mine, Morgan Hill, and Erzincan earthquake accelerations. The results of the drift response histories confirm the beneficial effects of using innovative energy dissipation systems especially BRBs and VDs with high damping coefficient, in reducing the lateral displacements. On contrary, the traditional bracing systems and the VDs with low damping coefficient (c=1000 kNs/m) were not as efficient as the other systems. For instance, under Morgan Hill earthquake acceleration, the residual roof drift obtained for 6 storey CBF and VDF with damping coefficient of c=1000 kNs/m is 0.693% and 0.391%, respectively. While the corresponding residual drifts for BRBFs and VDFs with damping coefficient of c=10000 kNs/m are 0.01% and 0%, respectively. On the other hand, when the 6 storey VDFs were compared with each other, the residual drifts are 0.386%, 0.129%, and 0% for VDFs with damping coefficient of 1000, 2500 and 10000 kNs/m, respectively. Consequently, it can be said that as the damping coefficient of the VDs and the total effective damping ratio of the frame increases, the roof residual roof drift decreases. It should be noted that each record was fictitiously lengthened by 5 seconds at zero acceleration in order to consider 5 seconds of free vibrations after the final post-quake configuration, which are essential to determine the residual drifts from the nonlinear time history analyses. Furthermore, when the time of the maximum response of the frames were compared, it was observed that the maximum response of the frames did not occur at the same time. The maximum roof drift for 6 storey frames under Morgan Hill record occurred at 37.75 s and 24.1 s for CBF and BRBF, respectively; while the maximum roof drift occurred at 32.1 s, 32.05 s, and 31.95 s for VDFs with damping coefficients of 1000 kNs/m, 2500 kNs/m, 10000 kNs/m, respectively. The performed comparative analysis of the roof drift demand exhibited that both BRBs and VD with damping coefficient of c=10000 kNs/m are the most effective controlling systems. They result in average reduction of the maximum roof drift by 51% and 52%, respectively.

Only the inter-storey drift index is not sufficient to fully understand the demands on the structural systems. One must also evaluate the distribution of the drift demands over the height of the structure (Barroso *et al.* 2002). Figs. 19 to 21 show the height-wise distribution of the maximum inter-storey drift demands of the original and upgraded frames when subjected to ground excitations of Hector Mine, Morgan Hill, and Erzincan. Moreover, in the present study, four structural performance levels (i.e., operational (OP), occupiable (OC), life safety (LS) and near collapse (NC) limit states) were considered for the structural assessment carried out. These limit states were determined in compliance with the SEAOC provision (SEAOC 1999); and the relationship between seismic performance limit states and maximum transient drift ratios according to this provision is summarized in Table 3. The limit inter-storey drift ratios for the OC, LS and CP performance levels have been also included in these plots as benchmarks.

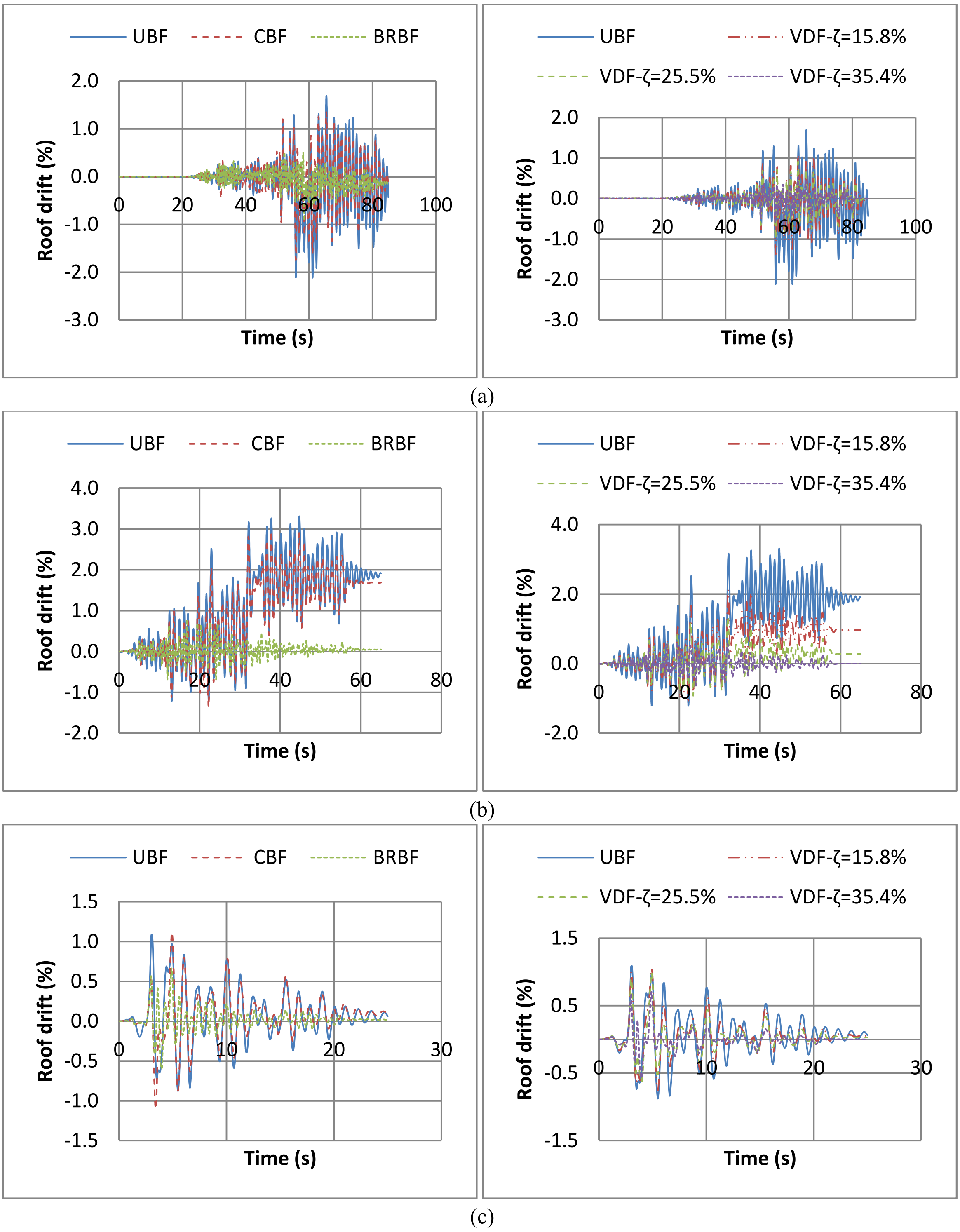


Fig. 16 Roof drift history for 3 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

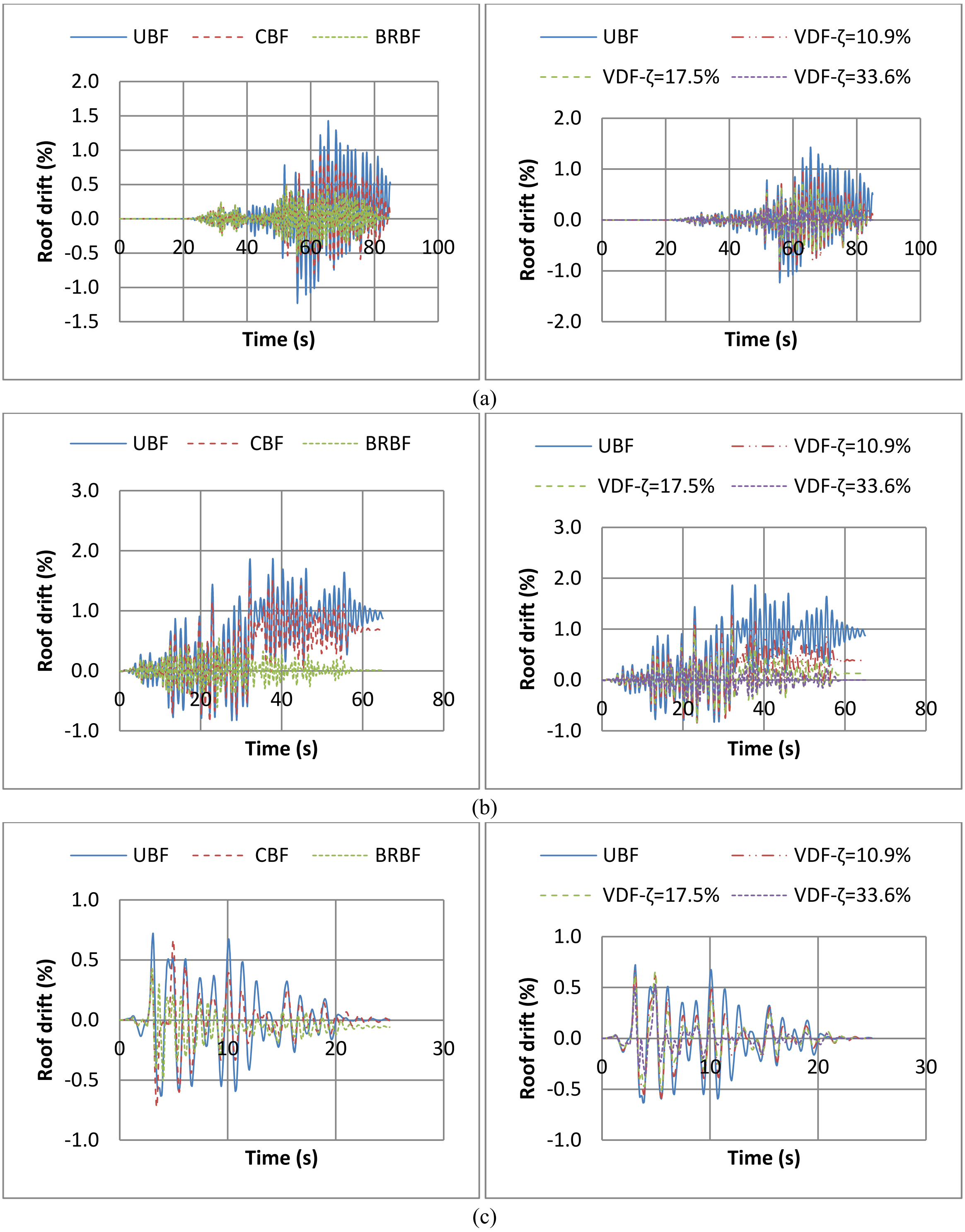
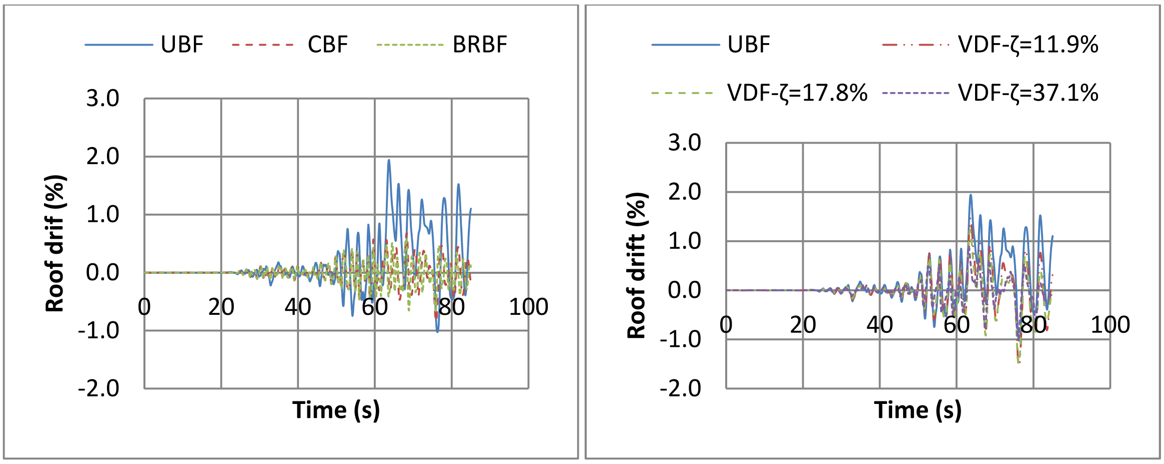
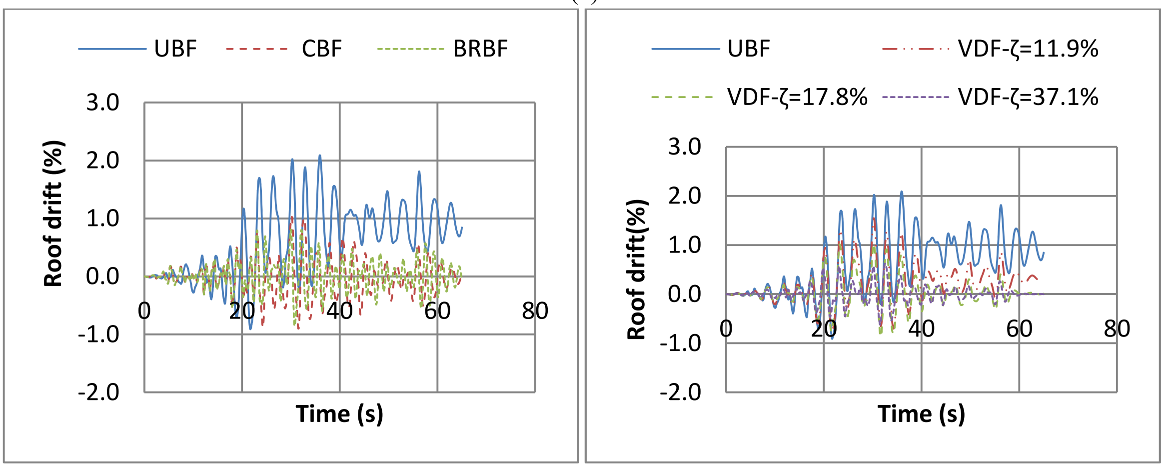


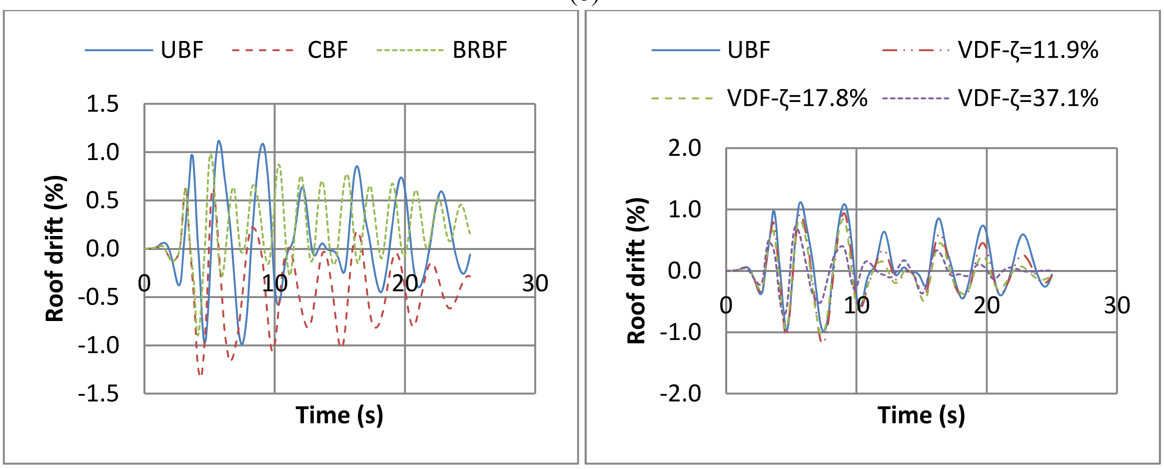
Fig. 17 Roof drift history for 6 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes



(a)

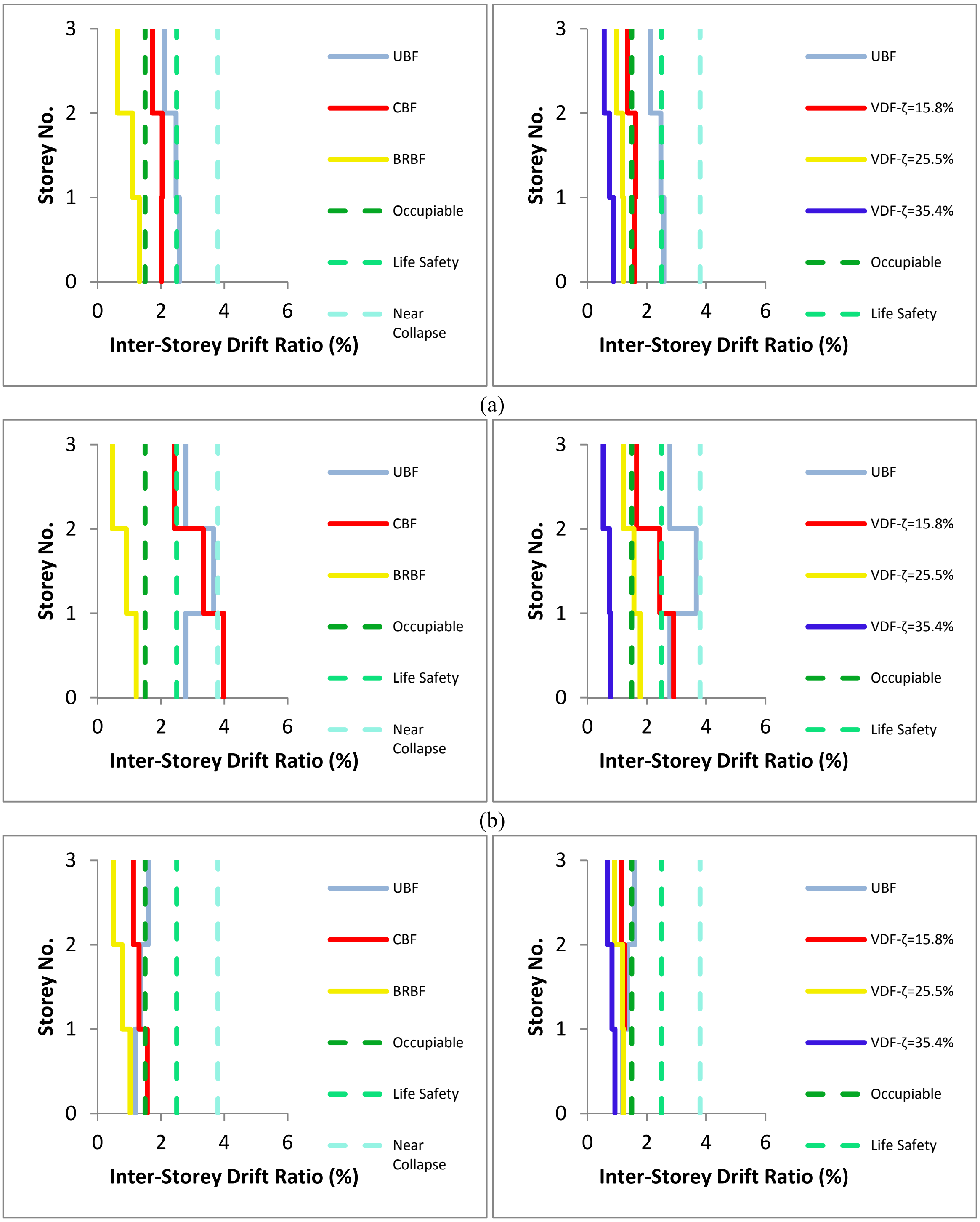


(b)



(c)

Fig. 18 Roof drift history for 12 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes



(c)

Fig. 19 Height-wise distribution of peak storey drifts for 3 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

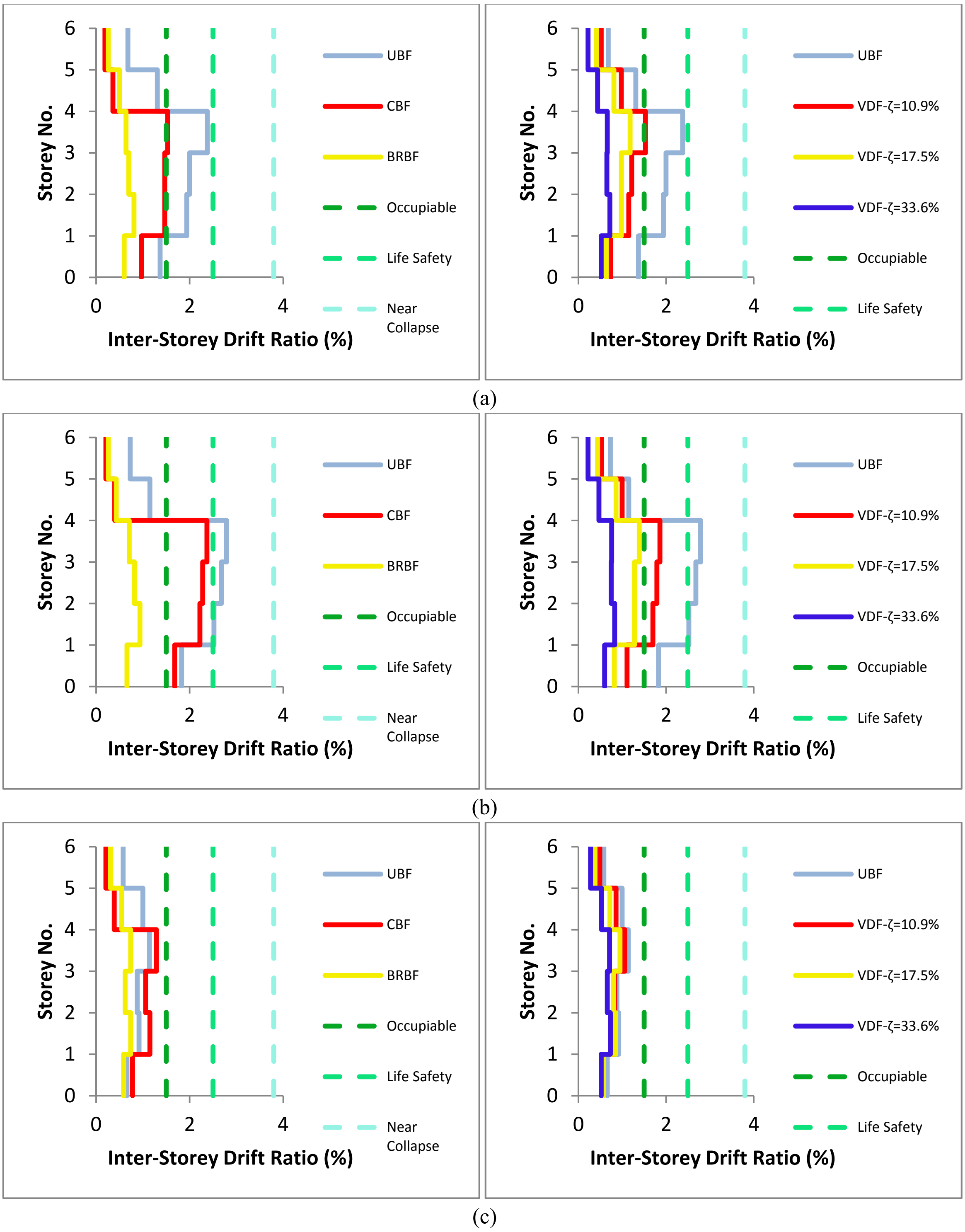
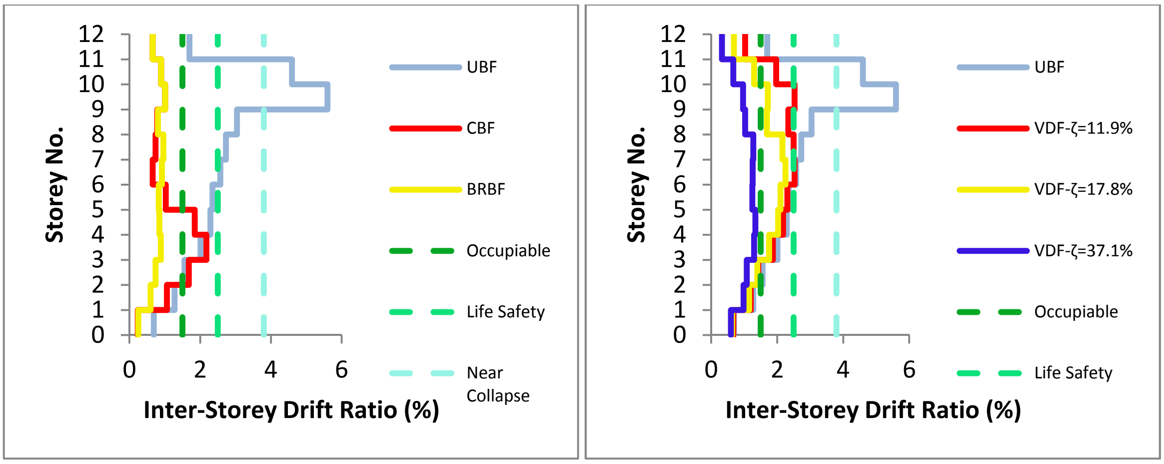
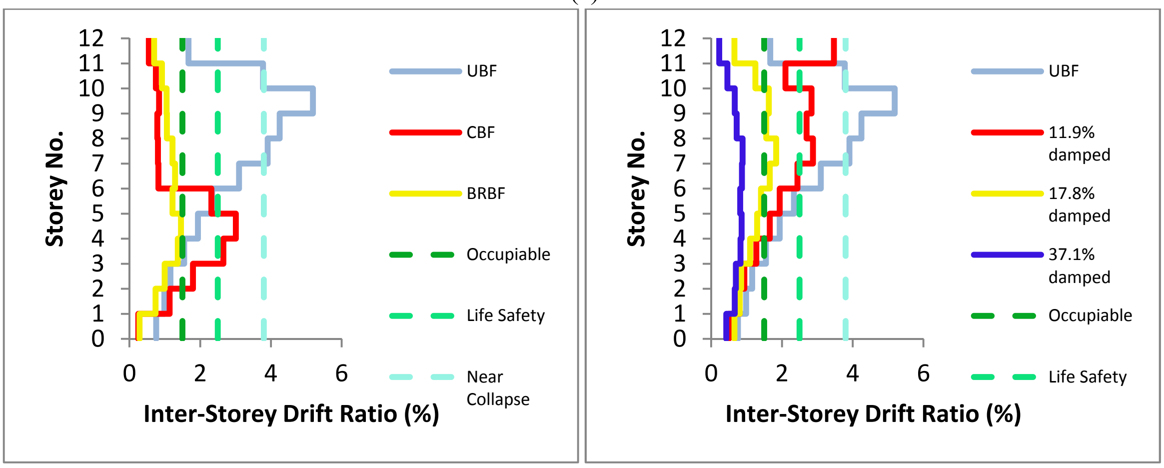


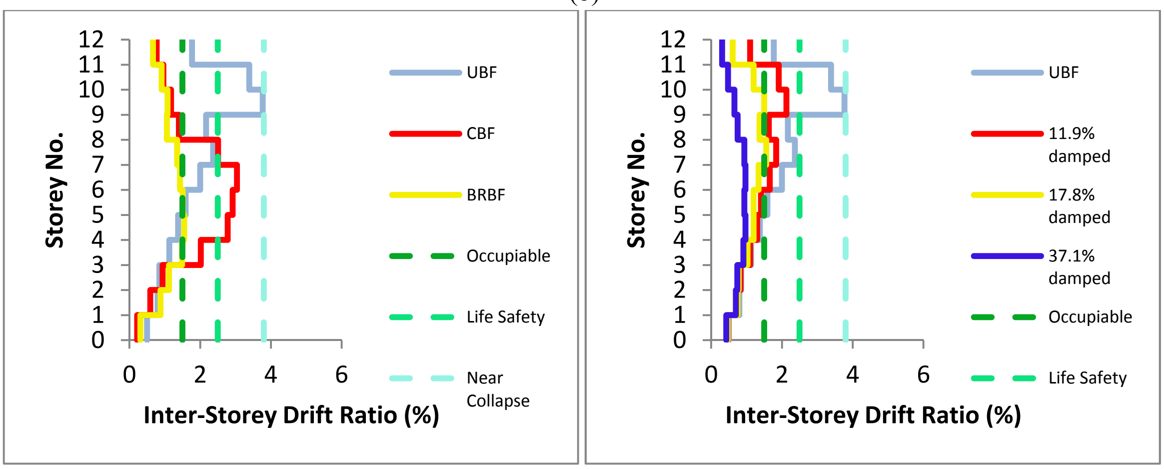
Fig. 20 Height-wise distribution of peak storey drifts for 6 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes



(a)



(b)



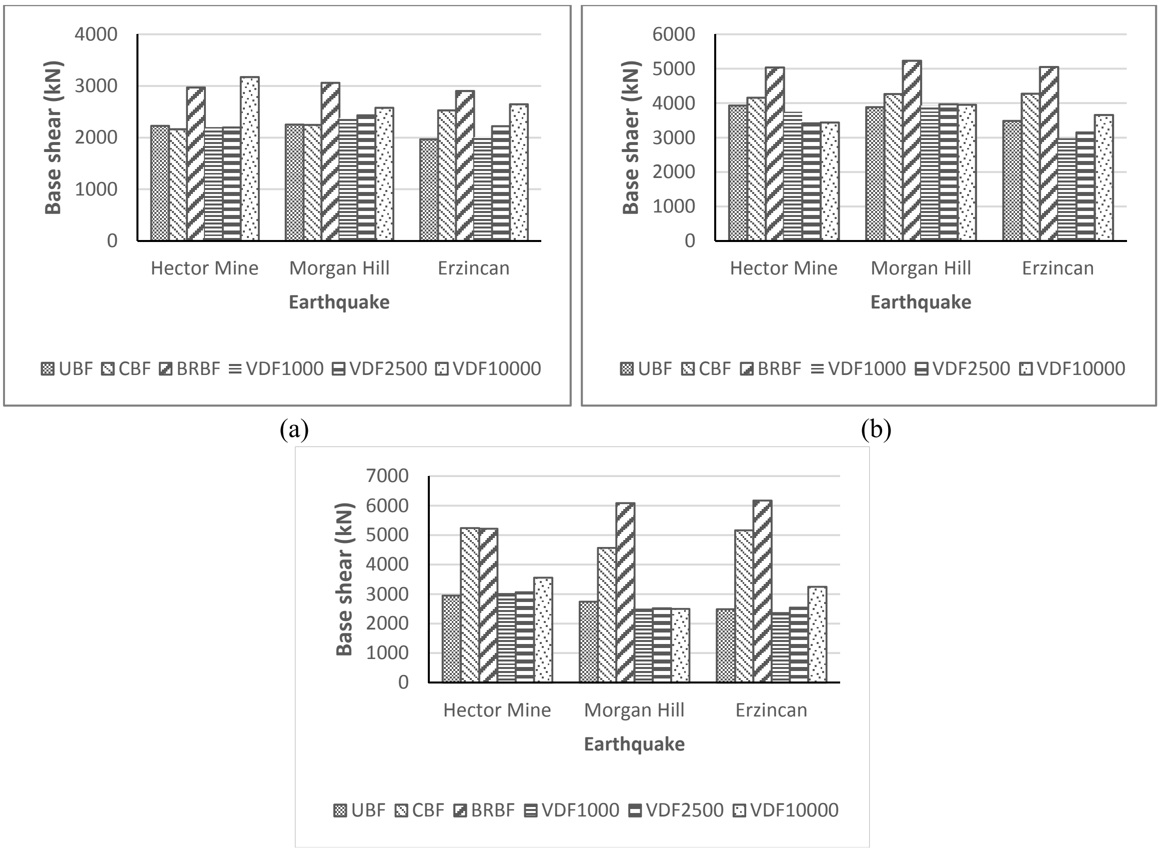
(c)

Fig. 21 Height-wise distribution of peak storey drifts for 12 storey frames under (a) Hector Mine, (b) Morgan Hill, and (c) Erzincan earthquakes

In particular, BRBs provide a uniform distribution of the inter-storey drift demand throughout the height of the structure and lead to inter-storey drift demands that are below OC limit state except the 12 storey frame of which inter-storey drift ratio reached to 1.55%. In the CBFs, there is a concentration of large deformation in one or more storeys of the frame such that there were abrupt changes in the first and second floors of 3 storey CBF under Morgan Hill record as seen from Fig. 19(b), and in the first and fourth floors of the 6 storey CBF under Morgan Hill ground motion record as shown in Fig. 20(b). Similarly, in the 12 storey CBFs, as seen from Figs. 21(a) and 21(b) that inter-storey drift demand has exceeded the LS limit state and there is a

Table 3 Structural performance levels (SEAOC 1999)

Performance level	Qualitative	Damage type	Recommended value
SP-1	Operational	Negligible	0.5%
SP-2	Occupiable	Light	1.5%
SP-3	Life safety	Moderate	2.5%
SP-4	Near collapse	Severe	3.8%



(a) (b)

(c)

Fig. 22 Comparison of the maximum base shear for (a) 3 storey, (b) 6 storey, and (c) 12 storey frames retrofitted with different methods concentration of large deformation in the third and eight floors under Erzincan and in the sixth floor under Morgan Hill earthquakes.

Examination of the maximum inter-storey drift throughout the height of the viscoelastic damped frames shows that the drift demand in all storeys is controlled effectively for the case with a damping coefficient of c=10000 kNs/m. In which, a uniform distribution of inter-storey drift over the height are provided and inter-storey drift demands are below OC limit state. However, it was found that there is a concentration of large deformation for the VDFs with damping coefficients of c=1000 kNs/m and c=2500 kNs/m. As it was the case for 6-storey VDF with damping coefficient of c=1000 kNs/m under Morgan Hill record, the inter-storey drift demand in the second, third, and fourth floors is appreciably higher (by a factor of about 2) than first, fifth and sixth floors. Similarly, for 6 storey VDF with a damping coefficient of c=2500 kNs/m subjected to Morgan Hill ground motion, the peak inter-storey drift in the mid floors is 1.3 and 2.1 times higher than the lower and upper floors, respectively. Thus, with the increase in the damping coefficient of VDs, more uniform distribution of the drift was obtained.

The base shear demand of the UBFs, CBFs, BRBFs and VDFs subjected to this set of ground motion is given in Fig. 22. The outcome of the analysis demonstrates that the use of viscoelastic dampers does not have considerable effect on the base shear demand. On contrary, the additions of the conventional and buckling restrained braces resulted in a significant increase in the base shear demand of the original frame.

5. Conclusions

This study aimed to investigate the seismic performance of steel moment resisting frame buildings equipped with diagonal conventional braces (CBs), buckling restrained braces (BRBs), and viscoelastic dampers (VDs) subjected to a set of spectrum compatible natural earthquake accelerations. Depending on analysis results, the following conclusions are be drawn:

- Use of VDs with appropriate damping coefficient and BRBs resulted in decrease in displacement demands and enhanced the performance state of the structure and/or avoid the interruption of its functionality.
- As the damping coefficient of the viscoelastic damper increased, the performance of the building structures was superior in terms of local and global deformation demands.
- Additions of VDs reduced the global deformations with providing base shear demand close to the original frame. On contrary, the results of the performed inelastic analyses demonstrate that the use of conventional or buckling restrained bracing systems increased the base shear demand with respect to original frame.
- The behaviour of the frames with BRBs is comparable and often better than that associated with conventional concentric braced frames. The average reductions in the inter-storey drift index were 60.7% and 21.6% for BRBs and CBs, respectively. The reduction of global damage index with respect to the original frames was on average equal to 52.4% and 25.1% for BRBs and CBs, respectively.
- In general, it was evident that frames equipped with VDs with a damping coefficient of c=10000 kNs/m and BRBs kept it in the elastic range of deformation. In the case of the frames with BRBs, the plastification was concentrated in the braces which might be replaced easily after the earthquake.
- Both VDs and BRBs minimize the roof residual drifts significantly, and that is due to the fact

that addition of such controlling systems reduces the plastic hinge formations in the structural members.

• During large and damaging earthquakes, the use of VDs and BRBs as a retrofitting strategy is expected to be a viable solution. They provide a uniform distribution of the storey drifts throughout the height of the structure. However, addition of CBs may result in sudden change in drift pattern and concentration of large deformation in one storey due to the buckling deformation of CBs in that storey prior to others.

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