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# Enhancing the Seismic Performance of Multi-storey Buildings with a Modular Tied Braced Frame System with Added Energy Dissipating Devices

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**Abstract**

The tied braced frame (TBF) system was developed to achieve uniform seismic inelastic demand along the height of multi-storey eccentrically braced steel frames. A modular tied braced frame (M-TBF) configuration has been recently proposed to reach the same objective while reducing the large axial force demand imposed on the vertical tie members connecting the link beams together in TBFs. M-TBFs may however experience variations in storey drifts at levels where the ties have been removed to form the modules. In this paper, the possibility of reducing the discontinuity in displacement response of a 16-storey M-TBF structure by introducing energy dissipating (ED) devices between the modules is examined. Two M-TBF configurations are investigated: an M-TBF with two 8-storey modules and an M-TBF with four 4-storey modules. Three types of ED devices are studied: friction dampers (FD), buckling restrained bracing (BRB) members and self-centering energy dissipative (SCED) members. The ED devices were sized such that no additional force demand was imposed on the discontinuous tie members. Nonlinear response history analysis showed that all three ED systems can be used to reduce discontinuities in storey drifts of M-TBFs. The BRB members experienced the smallest peak deformations whereas minimum residual deformations were obtained with the SCED devices.

**Keywords:** Buckling restrained member, Building, Eccentrically braced frame, Energy dissipation device, Friction damper, Self-centering member

## 1. Introduction

Steel braced frames are very popular to resist lateral loads acting on low- and mid-rise building structures. However, when subjected to seismic loading, taller braced frames are prone to concentration of lateral deformations along the structure height due to their limited capacity to distribute vertically the inelastic demand. The resulting large storey drifts may impose excessive ductility demand on key components of the seismic force resisting system or affect the stability of the structure. Concentration of inelastic demand is illustrated in Fig. 1(a) for an eccentrically braced frame (EBF). For this framing system, uneven distribution of the plastic deformations can be accentuated when the link beams exhibit non uniform seismic demand-to-capacity or overstrength ratios over the structure height (Popov et al., 1992; Rossi and Lombardo, 2007). This is the case when design criteria or limit states other than seismic strength requirements govern the selection of the ductile link beams. A damage distribution capacity factor was introduced by Bosco and Rossi (2009) to better predict the inelastic demand over the height of EBFs.

Several structural systems have been proposed to mitigate the concentration of inelastic demand in steel braced frames. Those include zipper braced frames (Khatib et al., 1988; Tremblay and Tirca, 2003; Yang et al., 2008, 2010; Tirca and Chen, 2012), braced frames with elastic trusses (Tremblay et al., 1997; Tremblay, 2003; Tremblay and Merzouq, 2005; Merzouq and Tremblay, 2006; Tremblay and Poncet, 2007; Mar, 2010) and tied eccentrically braced steel frames (TBFs) (Martini et al., 1990; Ghersi et al., 2000, 2003; Rossi, 2007). The latter is illustrated in Fig. 1(b). Vertical tie members are added to connect the ends of the ductile link beams between floors. Two vertical elastic trusses are then formed which force simultaneous yielding of the link beams and prevent concentration of inelastic demand. Past studies of tied braced frames have shown, however, that tie members attract large axial forces under seismic ground motions, which reduces the cost-efficiency of the system. To overcome this drawback, Chen et al. (2012, 2014) proposed to interrupt the tie members at specific locations along the building height to form truss

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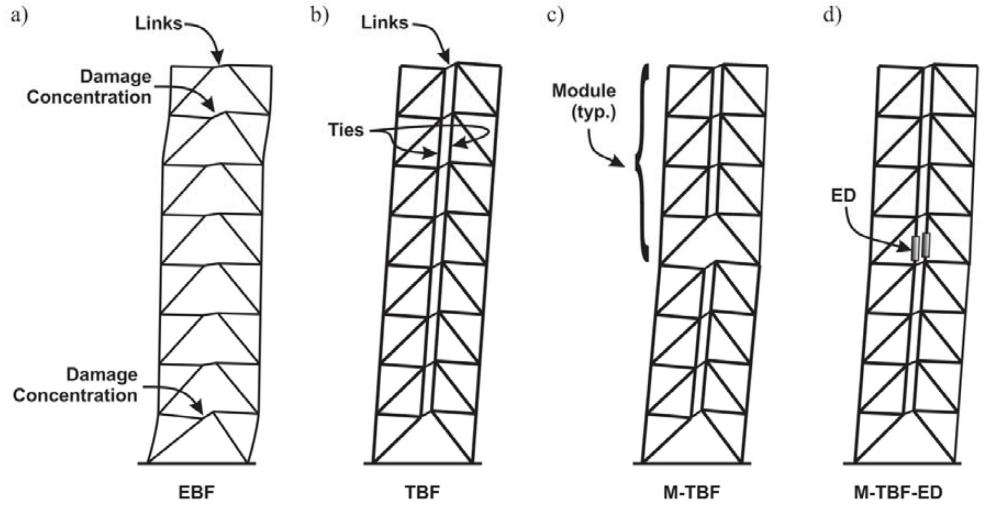



**Figure 1.** Eccentrically braced frame systems: (a) EBF, (b) TBF, (c) M-TBF, and (d) M-TBF-ED.

modules (Fig. 1(c)). Seismic analysis of this modular tied braced frame (M-TBF) system revealed that the force demand on the ties can be reduced considerably, leading to more economical designs. Larger storey drifts may develop, however, when increasing the number of modules along the frame height due to the discontinuity of the vertical trusses between modules. To mitigate this behaviour, it is proposed to add energy dissipation (ED) devices between the modules, as illustrated in Fig. 1(d). Compared to the reference M-TBF system, continuity between the modules is partially restored in this M-TBF-ED configuration as the activation loads of the ED devices are adjusted such that no additional forces are induced in the tie members.

This paper presents a comparative study where the performance of the TBF, M-TBF and M-TBF-ED systems are compared for a prototype 16-storey office building located in Victoria, British Columbia, Canada. For the modular systems, two configurations are studied: one with two 8-storey modules and one with four 4-storey modules. For the M-TBF-ED structures, three different energy dissipation systems are evaluated (Fig. 2): friction, yielding, and self-centering. As shown, the friction and yielding ED mechanisms exhibit high energy dissipation capacity but both systems may lead to undesirable residual (permanent) structural deformations. For a similar peak axial force, the third ED device has reduced energy dissipation capacity but this limitation is compensated by the re-centering capability of the system. All three ED devices can be easily implemented in axially loaded members. The first two sections of the paper respectively describe the design and numerical modelling of the different framing systems. Thereafter, the results of nonlinear response history analysis are presented to examine the deformation and force demands on the three braced frame configurations. For the M-TBF-ED system, the performances obtained with the three different ED-devices are compared.

## 2. Braced Frames Studied

### 2.1. Prototype building

The prototype building is a regular 16-storey office building located on a firm (class C) site in Victoria, British Columbia. This city is located along the Pacific coast of Canada, one of the most seismically active regions of the country. The structure plan view and the gravity loads considered in design are given in Fig. 3(a). The structure has two identical braced frames in each orthogonal direction. One of the two frames along the E-W direction is studied herein. The braced frame configurations examined

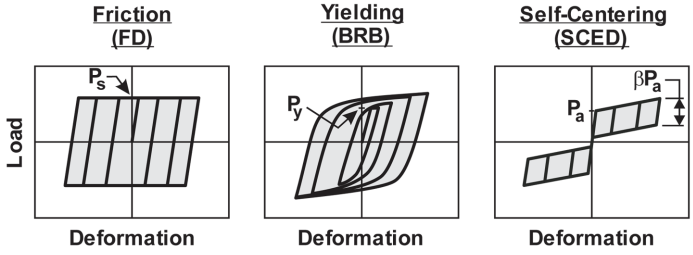



**Figure 2.** Hysteretic axial load-deformation response of tie members incorporating friction, yielding and self-centering ED devices.

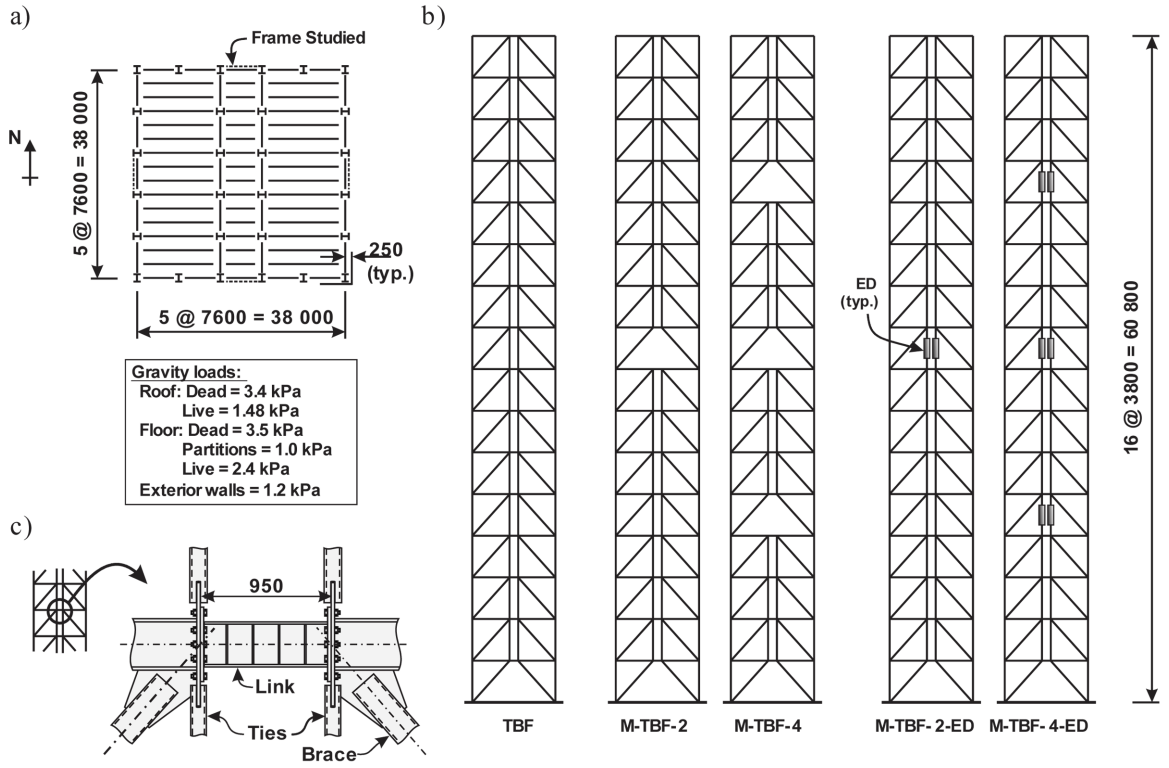



**Figure 3.** Building studied: (a) Structure plan view and design gravity loads; (b) Braced frame configurations investigated; (c) Replaceable links with tie connections (dimensions in mm).

in the paper are illustrated in Fig. 3(b). As discussed, the TBF has pair of continuous vertical tie members that connect all link beams together. For the M-TBF configurations, the vertical ties are removed at the 9th level to form two 8-storey modules (M-TBF-2) and at the 5th, 9th and 13th storeys to form an M-TBF system with four 4-storey modules (M-TBF-4). For each M-TBF configuration, ED devices are added between the modules to form the M-TBF-ED systems. In all braced frames, replaceable link beams with bolted end plate connections as proposed by Mansour et al. (2011) are used (Fig. 3(c)). This technique allows for a tighter selection of the link sizes and more uniform link capacity-to-demand ratios. All links are designed and detailed to yield in shear. Link members, beams outside the links and columns are I-shaped members whereas square tubing (HSS) is used for the bracing and tie members. As shown in Fig. 3(c), the tie members are directly connected to the plates used to connect the link beams. All members are made of steel with specified minimum yield strength $F_y$ of 345 MPa.

## 2.2. Braced frame design

The design was performed in accordance with the current Canadian seismic design provisions. In the 2010 NBCC (NRCC, 2010), the design spectrum, S(T), is based on uniform hazard spectrum (UHS) ordinates established for a probability of exceedance of 2% in 50 years. The design spectrum for the Victoria site is shown in Fig. 4, together with the response spectra of the ground motions used later in the response history analyses. This design spectrum is used as input for the modal response spectrum analysis carried out to determine seismic effects. For link design, the shear forces from analysis are reduced by a ductility-related force modification factor, $R_d = 4.0$, and an overstrength-related force modification factor, $R_o = 1.5$. According to the NBCC, the analysis results are also adjusted such that the base shear from analysis is not less than 80% of the static base shear prescribed in the code. For this 16-storey structure, the fundamental period is 4.5 s and the static base shear is equal to 0.024 W, where W is the structure seismic weight. According to the capacity

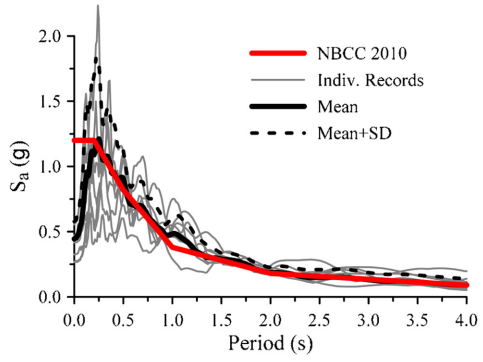



**Figure 4.** NBCC design spectrum and 5% damped absolute acceleration spectra of the scaled ground motion records.

design procedure implemented in the Canadian steel design standard (CSA 2009), the remaining frame members must be designed to resist gravity loads plus the seismic induced forces that develop when the links reach their strain hardened probable shear resistance, i.e., 1.3 times their shear resistance calculated with a probable steel yield strength $R_yF_y = 385$ MPa. This general design approach was applied to all structures, except that specific adjustments were considered for each framing system, as discussed in the next paragraphs.

For the TBF system, the recommendations by Rossi (2007) were incorporated in the design process. Because of the continuity of the ties in TBFs, inelastic response is constrained to develop essentially in the structure first vibration mode, with simultaneous yielding of all links over the frame height. Hence, the design link shear forces were obtained from static analysis of the frame subjected to a set of lateral loads that were vertically distributed following an inverted triangular shape and scaled to develop the same base overturning moment as the one obtained from response spectrum analysis. Link beams were selected individually at every level to closely match the link shear force demand. Once the links were sized, the design forces for the remaining frame members forming the two vertical elastic trusses on either side of the links were obtained from statics assuming that all links reach their strain hardened probable resistance, consistent with the hypothesis that inelastic response mainly develops in the first mode. However, since higher mode response may also induce flexural and horizontal shear demands in the two continuous elastic vertical trusses, inertia lateral loads due to second mode response were also applied to the structure when determining forces in the elastic vertical truss members. These loads were computed using the design spectrum and second mode properties. As proposed by Rossi (2007), a correction vector and a reduction factor were applied to these second mode loads to account for the effect of the yielding links on the second mode response. For the structure studied herein, that reduction factor was equal to 0.26.

Contrary to TBFs, inelastic deformation patterns that mimic elastic second and higher mode shapes are expected to develop in modular tied braced frames. The design link shears in both M-TBF structures without ED devices were therefore determined from response spectrum analysis including the contribution from higher modes. Since yielding in each module is expected to occur concurrently in all links, the links in a given module were sized for the average link shear forces over that module. After sizing the link beams, the forces in the other frame members were obtained from nonlinear response history analyses conducted with the same set of ground motion records that was used later to assess the structure seismic performance. The steel tonnage per bracing bent are 67.1 t and 63.4 t for the M-TBF-2 and M-TBF-4 configurations, respectively, which is significantly less than the 82.2 t of steel needed for one TBF. The difference is mainly attributed to the smaller forces that must be resisted by the elastic frame members (vertical ties, columns, braces and beams outside links) when adopting the modular concept. This is discussed later when evaluating the response of the systems. Further detail on the design of the TBF and M-TBF systems can be found in Chen et al. (2014).

The member sizes used for the two M-TBF-ED systems are the same as in the corresponding M-TBF systems except for the additional tie members incorporating the ED devices that were inserted between the modules. For the friction energy dissipaters, the same tie member as in the storey above was used except that a friction damper (FD) designed to slip at a predetermined load was inserted at one of the member ends. Readily available friction dampers such as those proposed by Pall and Marsh (1982) can be used for this application. In Fig. 2, the slip load $P_s$ of the FD device was set equal to 80% of the compression load used for the design of the tie member located in the next storey of the corresponding M-TBF system. The 20% margin was introduced to accommodate possible variations in the slip resistance of the dampers and, thereby, prevent overloading of the adjacent tie members. Buckling restraining bracing (BRB) members were used to obtain energy dissipation through yielding between the modules. BRB members include a steel core that yields in both compression and tension to develop stable hysteretic response under cyclic inelastic loading (e.g., Black et al., 2004). In this project, the BRB core plates were cut from steel plates with $F_y = 345$ MPa to yield at a load $P_y$ equal to 70% of the axial load capacity of the tie located in the level above. So doing, the BRBs could develop their probable axial resistances including strain hardening and frictional responses without causing failure in adjacent ties.

The self-centering energy-dissipation (SCED) members proposed by Christopoulos et al. (2008) were adopted to form the ties with self-centering ED response. These members comprise two embedded structural steel shapes that are initially pre-stressed using pre-tensioned aramid tendons. The steel shapes are also longitudinally connected by means of friction bolted connections. The activation load $P_a$ is the sum of the tendon pre-tension and the slip resistance of the friction connections. In the post-activation range, re-centering is obtained by elongation of the tendons, while energy dissipation is achieved by a friction mechanism between the two steel profiles. For this application and frame geometry, the SCED members were assumed to have an elastic initial stiffness, $K_{el}$, equal to 1.0 $P_a$ (in kN/mm) and a post-activation stiffness equal to 3.5% of their initial stiffness. In view of the relatively high post-activation stiffness, the load $P_a$ was set equal to 50% of the design loads adopted for the ties located in the storeys above them. The SCED members were also designed with $\beta = 0.95$ to ensure full re-centering behaviour (the factor $\beta$ is shown in Fig. 2). The properties of the three ED devices are summarized in Table 1.

**Table 1.** Properties of the energy dissipative devices

| Frame | at Level | Energy Dissipative Devices | | | | | |
|---|---|---|---|---|---|---|---|
| | | FD | | BRB | | SCED | |
| | | $P_s$ (kN) | $K_{el}$ (kN/mm) | $P_y$ (kN) | $K_{el}$ (kN/mm) | $P_a$ (kN) | $K_{el}$ (kN/mm) |
| M-TBF-2 | 9 | 525 | 301 | 459 | 125 | 328 | 328 |
| | 13 | 458 | 257 | 417 | 113 | 298 | 298 |
| M-TBF-4 | 9 | 393 | 257 | 358 | 97 | 256 | 256 |
| | 5 | 424 | 257 | 386 | 105 | 276 | 276 |

**Table 2.** Selected ground motions

| No. | Event | Station & Direction | R (km) | pga (g) | pgv (m/s) | $t_d$ (s) | SF |
|---|---|---|---|---|---|---|---|
| 963 | **M**6.7 Jan. 17, 1994 Northridge | Castaic, Old Ridge Route, 90° | 44 | 0.57 | 0.52 | 9.1 | 0.80 |
| 1039 | **M**6.7 Jan. 17, 1994 Northridge | Moorpark Fire Station,180° | 36 | 0.52 | 0.20 | 14.2 | 2.00 |
| 1049 | **M**6.7 Jan. 17, 1994 Northridge | Pacific Palisades-Sunset, 280° | 25 | 0.20 | 0.15 | 10.5 | 2.40 |
| 1077 | **M**6.7 Jan. 17, 1994 Northridge | Santa Monica City Hall, 360° | 28 | 0.59 | 0.31 | 10.7 | 1.50 |
| 736 | **M**6.9 Oct. 18, 1989 Loma Prieta | Apeel9-Crystal springs resort, 227° | 41 | 0.11 | 0.18 | 16.2 | 2.60 |
| 767 | **M**6.9 Oct. 18, 1989 Loma Prieta | Gilroy Array #3, 0° | 36 | 0.56 | 0.36 | 6.4 | 1.20 |
| 776 | **M**6.9 Oct. 18, 1989 Loma Prieta | Hollister - South & Pine, 90° | 51 | 0.18 | 0.29 | 28.8 | 1.50 |
| 787 | **M**6.9 Oct. 18, 1989 Loma Prieta | Palo Alto - SLAC Lab, 360° | 54 | 0.28 | 0.29 | 11.6 | 1.50 |
| 838 | **M**7.3 June 28, 1992 Landers | Barstow, 90° | 95 | 0.14 | 0.26 | 18.2 | 2.40 |
| 15 | **M**7.4 July 21, 1952 Kern County | Taft Lincoln School, 21° | 46 | 0.16 | 0.15 | 30.3 | 2.80 |

## 3. Analysis

### 3.1. Numerical model

Nonlinear response history analysis was performed using the OpenSees platform (McKenna and Fenves, 2004). The numerical models included one of the two braced frames acting in the E-W direction plus the structure leaning gravity columns that are laterally supported by the braced frame studied. The shear links were modelled using the Steel02 material that accounts for both kinematic and isotropic strain hardening responses (Koboevic et al., 2012). Beams outside the links and columns were modelled with elastic beams with concentrated plastic hinges at their ends. Probable yield strength values of 385 and 460 MPa were assigned to the steel materials used for the I-shaped and tubular members, respectively. Rayleigh damping was specified with 3% of critical damping in the first and third modes of vibration. P-delta effects were considered in the analyses, with gravity loads from dead load plus 50% of the live load and 25% of the roof snow load.

For the M-TBF-ED systems, the ED ties between the modules were modelled using spring elements with applicable uniaxial material properties. Bilinear elastic-plastic response was chosen for the friction ED devices. For the buckling restrained members, the Steel02 material with isotropic and kinematic strain hardening properties was adopted. An equivalent axial elastic stiffness equal to 1.6 times the axial stiffness associated to the bare steel core cross-sectional area was used to account for the stiffer end connection regions. The member strain hardening properties were based on test data by Tremblay et al. (2006). The SelfCentering uniaxial material available in OpenSees was used for the SCED tie members. The stiffness and energy dissipation properties were described in the section on frame design. The computed periods in the first three modes of the TBF system are respectively 4.5, 1.4 and 0.70 s. For the M-TBFs, the periods slightly elongate to 4.7, 1.44 and 0.76 s for the more flexible M-TBF-4 structure. The addition of the ED devices did not affect the frame periods.

### 3.2. Seismic ground motions

The structures were subjected to the suite of ten historical ground motion records presented in Table 2. The records were selected from the PEER database (PEER 2010) to reflect the magnitude-distance (M-R) scenarios that dominate the hazard at the site studied. The peak ground acceleration (pga) and peak ground velocity (pgv) of the unscaled ground motions are given in Table 2, together with the Trifunac duration, $t_d$. The ground motions were linearly scaled to match the design spectrum in the period range of interest (Fig. 4). The resulting scaling factors, SF, are given in Table 2.

## 4. Braced Frame Response

### 4.1. General

All braced frame systems performed as intended in design, i.e., the inelastic deformations concentrated in the link beams while all other frame members remained elastic. Peak storey drifts reached at every level under each ground motion are given for all framing systems in Fig. 5. Mean and mean plus one standard deviation (mean+ SD) results are also plotted in the graphs. The peak axial

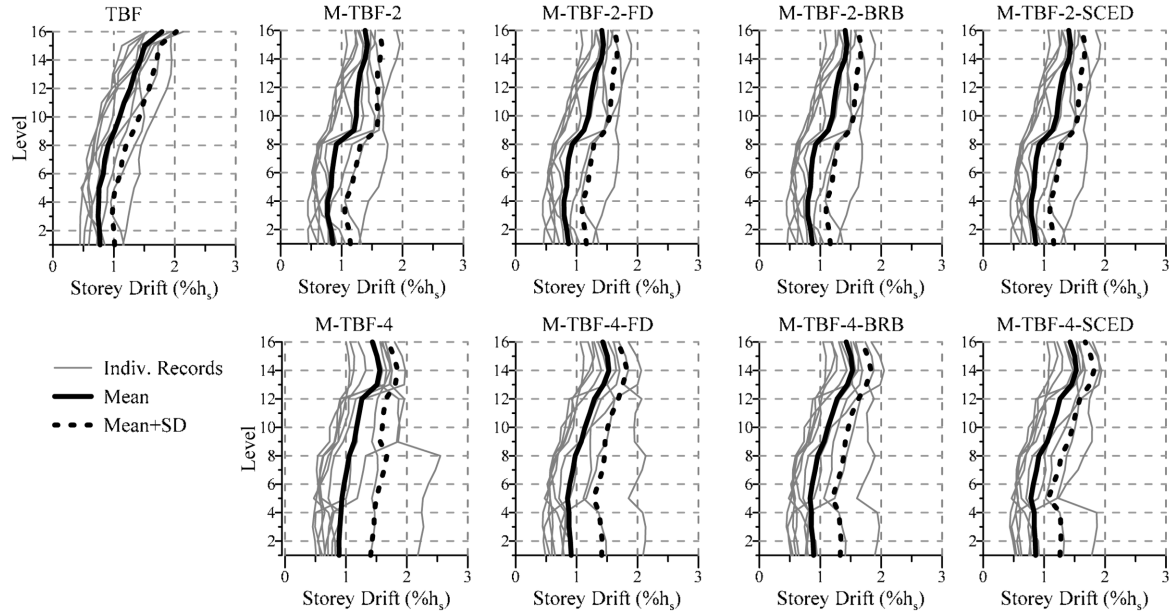



**Figure 5.** Peak storey drift response.

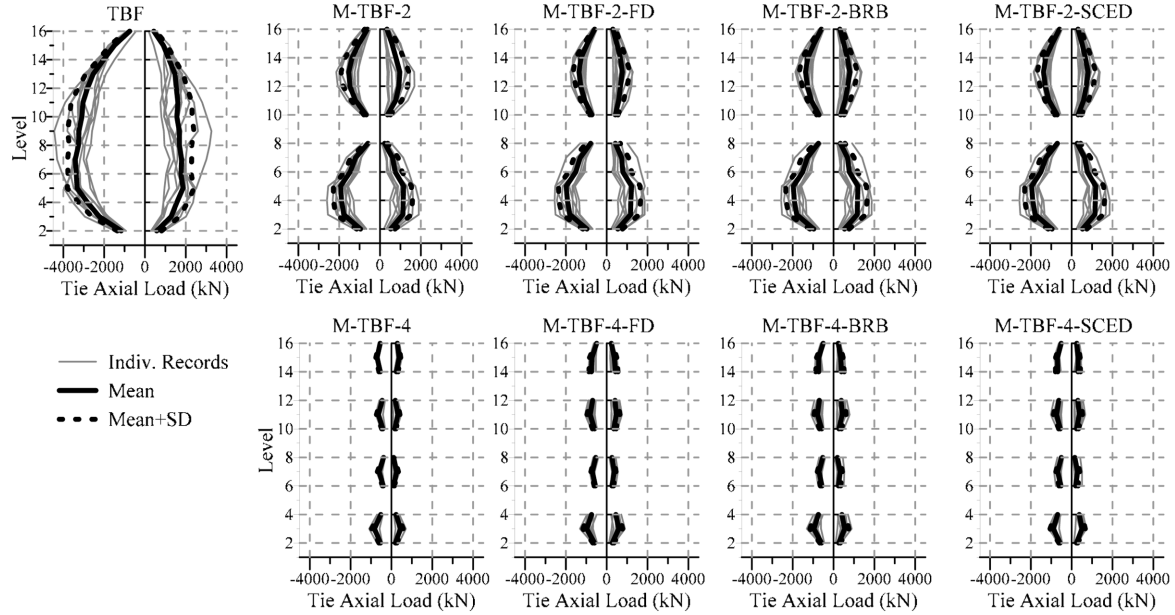



**Figure 6.** Peak tie force response.

force demands in the vertical tie members are presented in Fig. 6.

In addition, the following two response parameters are used to assess and compare the performance of the studied framing systems: the maximum peak storey drift along the structure height and the ratio of the maximum to average peak storey drift along the structure height. The former reflects the capacity of the framing systems to prevent the development of large storey drifts. The latter, which is referred to as the drift concentration factor (DCF), is used to evaluate the capacity of the systems to achieve uniform storey drift demand over the building height. These parameters are evaluated individually for each ground motion record and the mean and mean plus one standard deviation (mean+SD) values are then calculated for the 10 seismic ground motions. The results are presented in Tables 3 and 4. Link plastic rotations are not reported in this study but they can be estimated from the storey drift values using the expression proposed by Koboevic et al. (2012).

**Table 3.** Statistics of the maximum peak storey drifts (% $h_s$)

| System | Energy Dissipative Devices | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | - | | FD | | BRB | | SCED | |
| | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD |
| TBF | 1.79 | 2.03 | - | - | - | - | - | - |
| M-TBF-2 | 1.47 | 1.72 | 1.47 | 1.71 | 1.47 | 1.71 | 1.47 | 1.72 |
| M-TBF-4 | 1.67 | 2.10 | 1.58 | 1.91 | 1.56 | 1.87 | 1.54 | 1.82 |

**Table 4.** Statistics of DCFs

| System | Energy Dissipative Devices | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | - | | FD | | BRB | | SCED | |
| | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD |
| TBF | 1.73 | 2.04 | - | - | - | - | - | - |
| M-TBF-2 | 1.40 | 1.57 | 1.41 | 1.61 | 1.40 | 1.60 | 1.40 | 1.60 |
| M-TBF-4 | 1.49 | 1.74 | 1.46 | 1.68 | 1.46 | 1.68 | 1.46 | 1.68 |

### 4.2. Seismic response of TBF and M-TBF systems

For the TBF system, Fig. 5 shows that in general the peak storey drifts from the individual records vary gradually along the frame height, without much discontinuity. This behaviour was expected in view of the presence of the ties forming two elastic vertical trusses that are continuous over the entire frame height. The mean and mean +SD drift values generally increase when moving towards the structure top, with a maximum mean value reaching 1.79% $h_s$ at the uppermost level. This is less than the limit of 2.5% $h_s$ specified in the NBCC. The larger drift demand in the upper floors is attributed to higher mode response. For the M-TBF-2 and M-TBF-4 systems, discontinuity in individual peak storey drifts can be observed at levels where the ties have been removed to create the truss modules. For the M-TBF-2, the mean and mean+SD drift values show a substantial increase at the 9$^{th}$ level, indicating that a kink formed at the junction of the two modules, which led to larger drifts at all levels of the upper module. For the 4-module configuration, variations in mean drifts occurred only at the 13$^{th}$ level while mean+ SD values show changes at the 9$^{th}$ and 13$^{th}$ levels. Compared to the M-TBF-4 system, the storey drifts in the M-TBF-2 are more uniform within each module.

For both M-TBFs, the peak storey drifts at the roof level is less than in the TBF. This is because the same link beams were used over the height of each of the modules of the M-TBF-2 and M-TBF-4 structures, which resulted in relatively stronger links at the roof level of the modular frames compared to the TBF. For the MTBF-2, the discontinuity at mid-height of the vertical elastic trusses probably reduced the higher mode response that induces the large drift demand in the upper floors. For the M-TBF-4 system, however, the additional discontinuity at the 13$^{th}$ level and the relatively weaker links at the bottom of the fourth tier allowed for the development of larger displacements at levels 13 and 14. From the data in Table 3, the M-TBF-2 system was therefore more effective than the TBF and M-TBF-4 systems for mitigating the development of large storey drifts. The M-TBF-4 exhibits the largest mean+SD value of the maximum storey drifts, indicating a greater sensitivity to ground motion characteristics. Similar trends are observed when examining the DCF values in Table 4 as more uniform response over the building height is obtained with the M-TBF-2 configuration. The main benefit of using the modular tied braced frame concept can be readily seen by examining the peak axial forces on the vertical ties that are shown in Fig. 6: the tie forces are much lower in both M-TBF structures compared to the TBF. As expected, the reduction is more pronounced for the four module scenario because it introduces greater relaxation to the constraints imposed by the vertical ties on the system response. Equally important, peak tie forces are more uniform over the structure height and their scatter for the 10 ground motions is reduced when the frame contains shorter, more numerous vertical truss modules. Hence, contrary to lateral displacements, better control of the elastic member forces can be achieved by increasing the number of modules.

### 4.3. Seismic response of the M-TBF-ED systems

The results in Fig. 5 show that the discontinuity in peak storey drifts at the 9$^{th}$ level of the M-TBF-2 system could be reduced by the addition of the ED devices. For that structure, all three types of added devices produced similar effects which were concentrated within the storeys above and below their location in the structure. The drift responses in the bottom and upper parts of the building structure remained nearly unchanged compared to the M-TBF-2 structure. This behaviour is confirmed in Tables 3 and 4: the presence of the ED devices had no influence on the maximum peak storey drifts and the distribution of the peak storey drifts along the structure height, the main reason being that the maximum response of the M-TBF-2 generally occurred at the roof level, away from the position of the ED systems. This localized impact of the ED devices can be observed in Fig. 7. The figure shows the response of the various two-module systems under

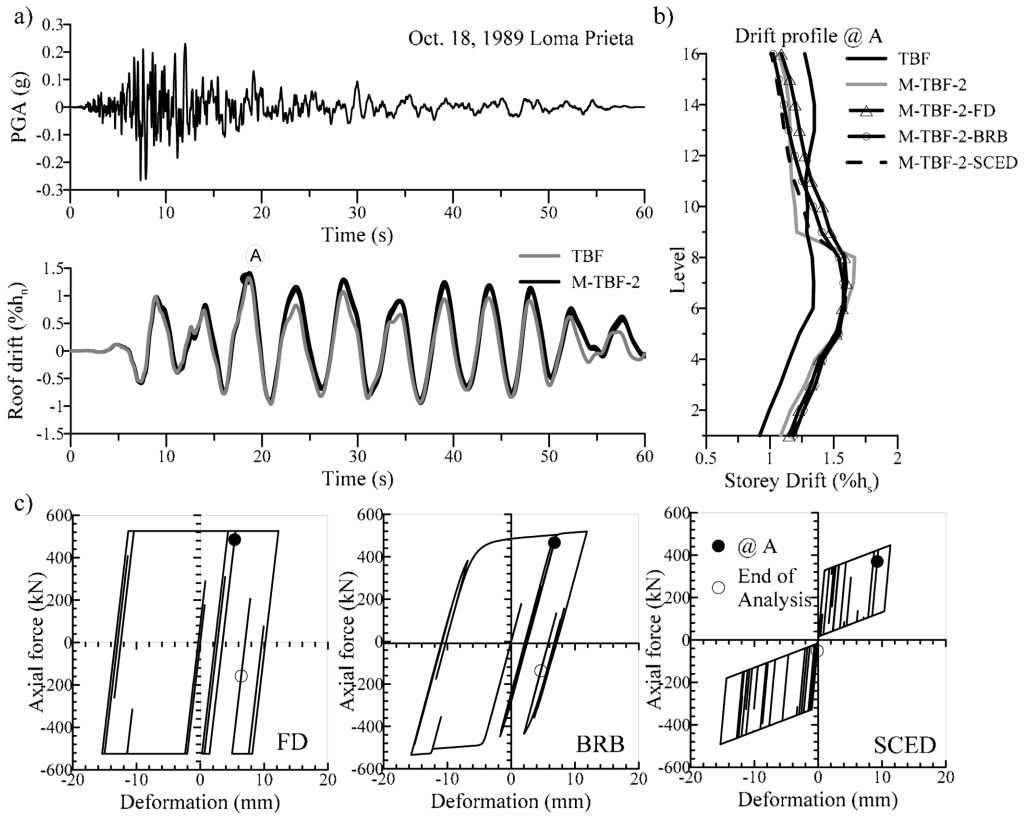



**Figure 7.** Response of the M-TBF-2-ED systems to the 1989 Loma Prieta earthquake (Hollister - South & Pine 90° record): (a) Time histories of the ground acceleration and roof drifts; (b) Storey drift profiles at point A; (c) Hysteresis of the ED devices at the 9th storey.

record No. 776 from the 1989 Loma Prieta earthquake. This record has high energy in the long period range and induced the largest demand on the studied structures. The response of the TBF is also included for comparison purposes. In Fig. 7(b), the storey drift profile is plotted at the time when the difference in storey drifts reaches a maximum between the two modules of the M-TBF-2 system without ED devices (point A in Fig. 7(a)). As shown, all three ED devices resulted in a smoother drift response in the vicinity of the junction between the two modules. For this particular example, the FD device is slightly more effective whereas the SCED system has the least effect. However, when compared to the TBF, all three M-TBF-2-ED structures still exhibit more pronounced drift variations near the structure mid-height. This behaviour was expected as M-TBFs have smaller, more axially flexible tie members than TBFs. Moreover, contrary to TBFs in which the ties are designed to resist and remain elastic under the large induced forces, the ED devices between the elastic truss modules are sized to activate at much lower loads in order to control the tie forces. They must then undergo nonlinear deformations before they can dissipate energy and positively affect the structure response, which led to the greater localized deformations that were observed.

In Fig. 7(c), the hysteresis responses of the three different ED systems at level 9 are plotted for the ground motion shown in Fig. 7(a). In that particular case, all three devices experienced the same peak axial deformation and developed similar peak forces. Under all ground motion records, the force demand in the ED system remained below or close to the maximum permissible force adopted in design. Consequently, as shown in Fig. 6, the addition of the ED devices in the M-TBF-2 frame had nearly no effect on the peak tie force demand, as was intended in design.

As illustrated in Fig. 5, the use of ED devices in the M-TBF-4 system resulted in smoother variations of storey drifts in the upper levels, and the mean and mean+SD values of the maximum peak storey drift were therefore reduced (Table 3). In the lower levels, the profile of the storey drift demand of the three M-TBF-4 systems approaches that observed for the TBF system. As a result, the drift concentration factors in Table 4 were also reduced. The response to the 1989 Loma Prieta earthquake record is examined in Fig. 8 for the M-TBF-4-ED systems. For this ground motion, the largest demand was imposed to the devices located at the 5th level, between the first two modules. Point A in this figure corresponds to the time when the change in storey drifts is largest at this location.

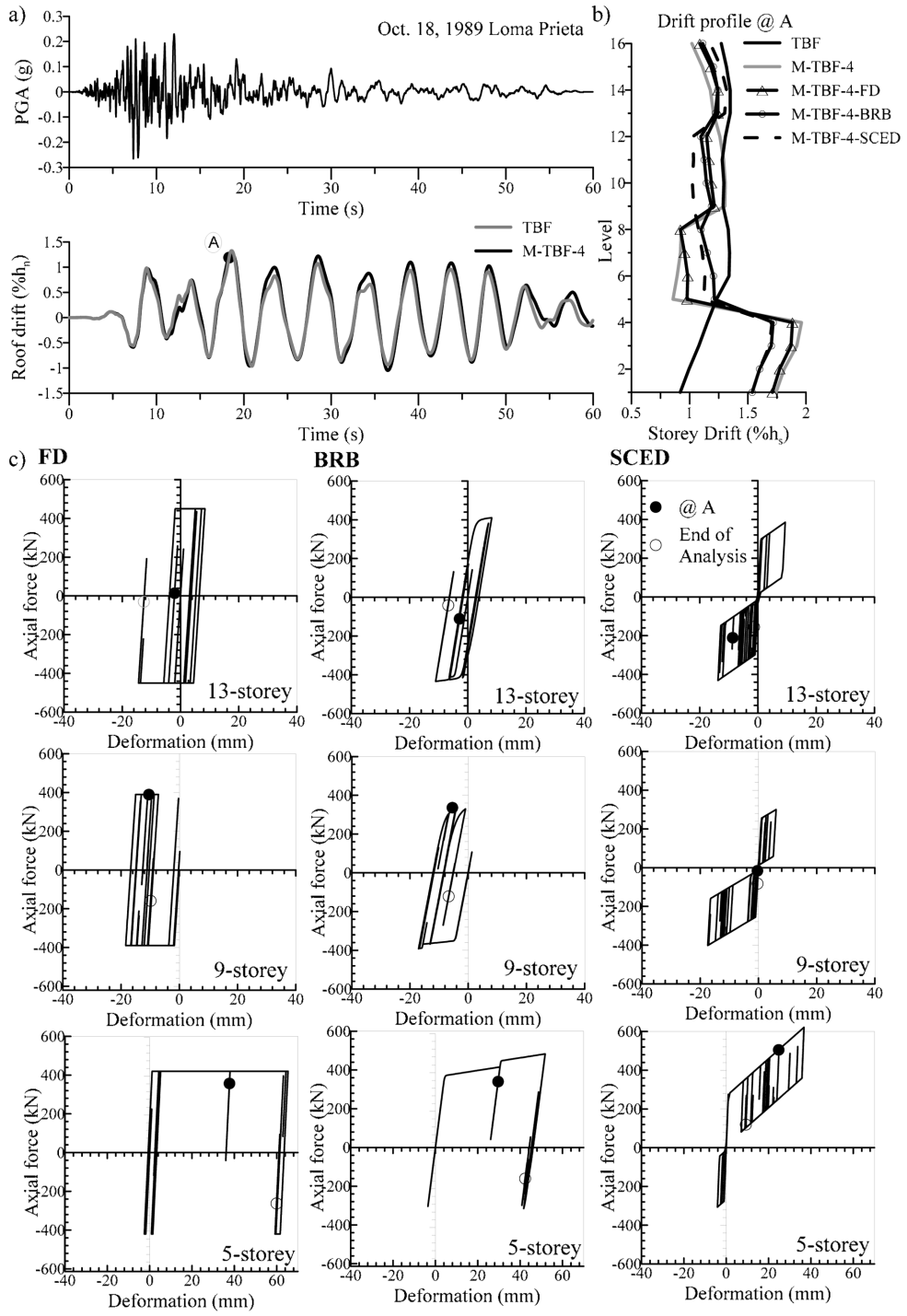



**Figure 8.** Response of the M-TBF-4-ED systems to the 1989 Loma Prieta earthquake (Hollister - South & Pine 90° record): a) Time histories of the ground acceleration and roof drifts; b) Storey drift profiles at point A; c) Hysteresis of the ED devices at the 5th, 9th and 13th storeys.

As shown, the ED devices could minimize the large difference in drifts experienced by the M-TBF-4 system between these 2 modules. Similar improvement can be observed between the upper modules, where storey drifts became more uniform along the frame height. The demand imposed on each of the added ED devices during the ground motion is shown in Fig. 8(c). As was the case for the M-TBF-2 system, the force demand in the tie members was very well controlled by adopting the design strategy proposed for the ED devices (Fig. 6). The above observations indicate that the addition of the ED devices was more beneficial for the M-TBF-4 configuration than for the M-

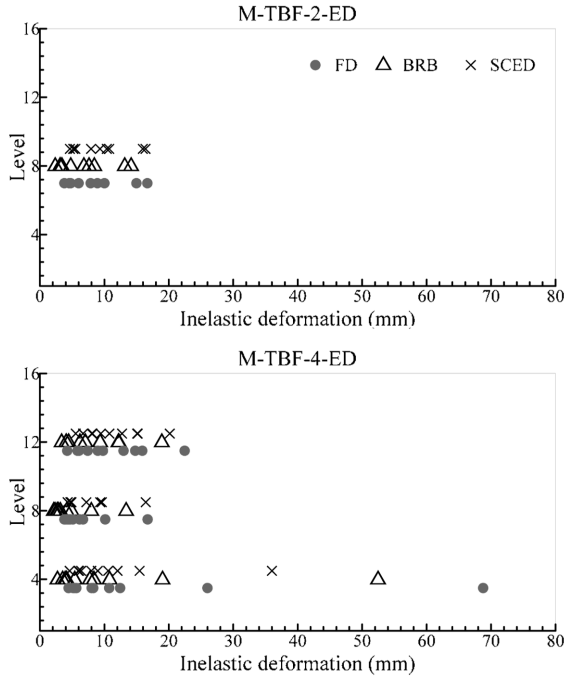



**Figure 9.** Peak inelastic deformations in energy dissipative devices in M-TBF-ED systems.

TBF-2: the response approached that of the TBF system with a reduction of the peak storey drifts at the roof level and, more importantly, much lower axial forces imposed to the tie members. The variations of the demand on the ED devices along the frame height suggests that the efficiency of energy dissipating systems could probably be enhanced by varying their activation loads or their position along the height of the structure.

As depicted in Fig. 8, the yielding ED mechanism is found to be more effective in correcting the drift profile and therefore sustained less axial deformations (Fig. 8(c)). In general for the M-TBF-4 structure, the analysis results presented in Fig. 5 show that the friction and yielding mechanisms were slightly more effective in mitigating the sudden changes in storey drifts between modules. Differences were also observed between the three types of ED systems used in the M-TBF-2-ED frames. Peak axial deformations experienced by each ED device under individual ground motions are presented in Fig. 9 and statistics of these results are given in Table 5. As shown, mean values of the peak deformations are typically higher for the SCED system, except at the 5$^{th}$ level of the M-TBF-4, which can be attributed to its reduced activation load and smaller energy dissipation capacity. In all cases, the use of yielding BRB members resulted in the lowest axial deformation demand, meaning smaller drift variations between modules. The BRB members also generally exhibited the smallest mean+SD deformation values, indicating greater consistency in the response. Under the large demand at the 5$^{th}$ level of the M-TBF-4-ED structures, the self-centering ED shows lower mean+SD value than the BRB, likely because the higher post-activation stiffness and self-centering capacity were more effectively mobilized under large earthquakes. Conversely, the FD system does not offer strain hardening response and stiffness upon sliding, which likely contributed to the higher deformations and greater scatter in the results, as illustrated in Fig. 9.

### 4.4. Residual deformation response

Profiles of residual storey drifts are presented for all systems in Fig. 10. The TBF system experienced smaller residual deformations with mean permanent storey drifts varying between 0.11 and 0.17% $h_s$ and a maximum value of 0.36% $h_s$, which is lower than the 0.5% $h_s$ permissible residual drift value proposed by McCormick et al. (2008). This suggests that the structure could be repaired and reused after a major earthquake. More pronounced residual drifts are observed in the M-TBF structures with mean values reaching 0.16% $h_s$ in the upper half of the M-TBF-2 structure and 0.31% $h_s$ in the lower levels of the M-TBF-4 system. As shown in the figure, the residual drifts in the M-TBFs are uniform within each module, but the permanent deformations vary between modules.

In this context, the use of self-centering ED devices is found to be the most effective in reducing these permanent rotations, especially for the four module configuration. In Figs. 7(c) and 8(c), the SCED devices are capable of returning the frame close to its original position at the junction of two adjacent modules, which is not the case for the FD and BRB devices. This behaviour is confirmed when examining the statistics of the permanent axial deformations in the devices as presented in Table 6. The best response is offered by the SCED system, followed by the BRB and FD systems. It is noted that full centering response would require that the centering force capacity of

**Table 5.** Statistics of the peak inelastic axial deformations of the ED devices in the M-TBF-ED systems (mm)

| Frame | at Level | Energy Dissipative Devices | | | | | |
|---|---|---|---|---|---|---|---|
| | | FD | | BRB | | SCED | |
| | | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD |
| M-TBF-2 | 9 | 8.21 | 12.73 | 6.70 | 10.9 | 9.12 | 13.5 |
| | 13 | 10.84 | 16.50 | 8.72 | 13.51 | 11.18 | 15.75 |
| M-TBF-4 | 9 | 6.61 | 10.62 | 4.63 | 8.15 | 7.04 | 10.91 |
| | 5 | 15.49 | 35.26 | 11.81 | 26.90 | 11.84 | 22.39 |

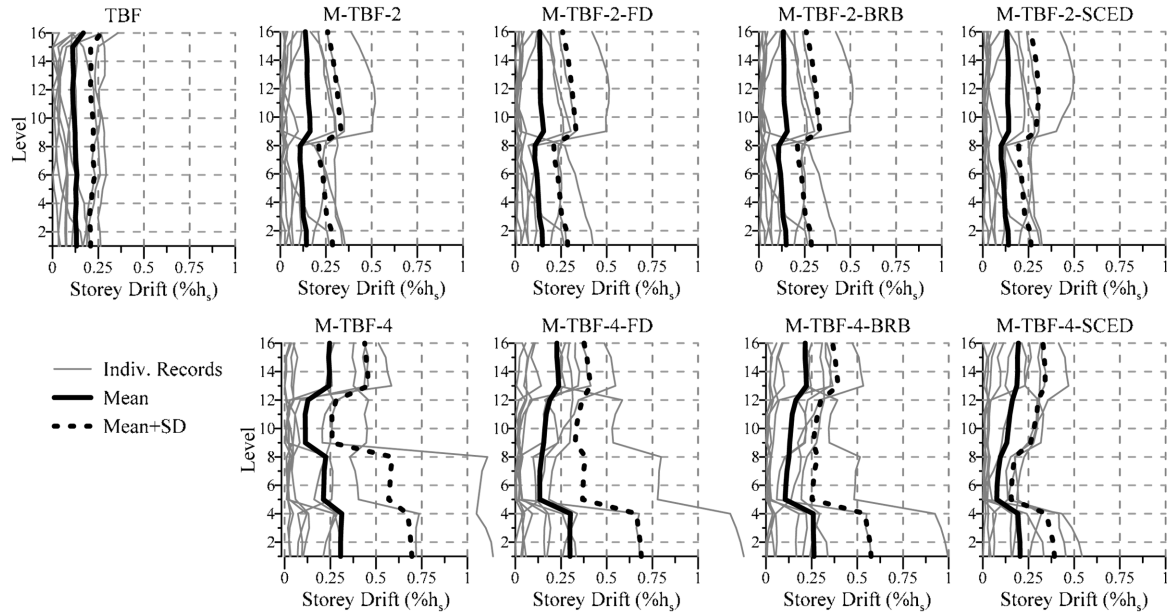



**Figure 10.** Residual storey drift response.

**Table 6.** Statistics of the residual axial deformations of the ED devices in the MT-BF-ED systems (mm)

| Frame | at Level | Energy Dissipative Devices | | | | | |
|---|---|---|---|---|---|---|---|
| | | FD | | BRB | | SCED | |
| | | Mean | Mean+SD | Mean | Mean+SD | Mean | Mean+SD |
| M-TBF-2 | 9 | 6.47 | 10.51 | 5.56 | 9.01 | 2.94 | 5.53 |
| | 13 | 7.87 | 12.39 | 6.09 | 9.98 | 2.93 | 5.56 |
| M-TBF-4 | 9 | 4.42 | 6.61 | 3.60 | 5.08 | 2.59 | 4.06 |
| | 5 | 14.14 | 33.55 | 10.91 | 24.44 | 5.53 | 9.99 |

the SCED unit be specified with consideration of the total yield shear strength of the link beams located in the module above. This criteria was not considered in the design of the sample frames studied herein and should be included if residual drift response has to be improved in future designs.

## 5. Conclusions

The seismic response of a 16-storey steel building was examined to compare the seismic performance of three different tied braced frame systems: continuous tied braced frame (TBF), modular tied braced frame (M-TBF) and modular tied braced frame equipped with added energy dissipation devices (M-TBF-ED). For the modular M-TBF and M-TBF-ED systems, two- and four-module configurations were examined. Friction, yielding and self-centering energy dissipation mechanisms were examined for the M-TBF-ED systems. The response parameters of interest were the peak and residual storey drifts and peak axial forces in the vertical ties. The following conclusions can be drawn from the study:

The TBF system is effective in achieving uniform storey drift demand and, thereby, inelastic link rotations, along the frame height. However, this is at the expense of large axial forces developing in the members of the two elastic vertical trusses formed on either side of the link beams. This force demand can be considerably reduced when using an M-TBF system and this benefit is more pronounced when the number of modules is increased.

The use of an M-TBF system results in variations in storey drift demands between adjacent modules. These variations are more pronounced when increasing the number of modules within a structure, which may lead to large storey drifts from ground motions imposing larger demand. For the 16-storey structure studied herein, the four-module configuration led to significant force reduction without significant increase in storey drift response.

The addition of ED devices at the junction of the modules of M-TBF structures improved the storey drift response at levels just below and above to the location of the ED devices. The influence of the ED devices was found to be more significant when the number of modules was increased. The use of ED devices in M-TBFs has no impact on the force demands imposed on the tie members provided that the ED devices are proportioned such that

their resistances is less than the design loads considered for the tie members in the original M-TBFs.

Yielding (BRB) ED devices were found to experience smaller peak axial deformations and, thereby, allowed better control of the global peak rotations between adjacent modules. Conversely, the SCED devices experienced larger peak axial deformations. However, ED devices exhibiting strain hardening response such as the SCED and BRB systems developed smaller deformations when subjected to ground motions imposing larger demand. When more than two modules are used, the deformation demand on the ED devices was found to vary along the structure height, suggesting that the ED properties could be optimized to further improve their efficiency.

The use of self-centering ED devices can reduce the residual drift response of the modular frames.

Further research is needed to examine the possible benefits of using ED systems in buildings taller than the one investigated in this study. Optimization of the properties of the ED devices along the frame height should be considered in this future work.

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