Seismic Performance of Modular Braced Frames for Multi-Storey Building Applications

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SUMMARY
The seismic performance of a modular tied eccentrically braced system is examined to assess the ability of the system of mitigating soft-storey response in multi-storey building applications. Compared to conventional tied braced frames (TBF), the tie members in the new modular framing configuration (M-TBF) are only continuous over modules over the building height, the objective being to reduce seismic induced member forces and increase the structural efficiency. The seismic performance of both systems is assessed for an 8-storey building located in an active seismic zone of western North America through nonlinear time history analyses performed using the OpenSees software platform. The results show that seismic induced forces in tie members are reduced considerably at the expense of limited increases in storey drifts and link plastic rotations when using the M-TBF system instead of the conventional TBF configuration.

Keywords: Soft-storey; steel braced frame; tied eccentrically braced frame; modular braced frame.

1. INTRODUCTION

Eccentrically braced frame (EBF) is a lateral load resisting system for steel buildings that has been proposed in the late seventies for seismic applications in view of its high ductility and favourable combination of lateral stiffness and strength. In EBFs, ductile link beam segments act as structural fuses to dissipate energy through yielding. The system can be designed with either short links yielding in shear or long links yielding primary in flexure. It has been recognized that short links can provide excellent energy dissipation under severe cyclic loading (Kasai and Popov, 1986). Popov et al. (1992) highlighted the significant role of the link strength: a desired seismic behaviour can only be achieved when the link strength is properly distributed throughout the height of the EBF. As illustrated in Fig. 1.1a, ill proportioning of links in an EBF may result in concentration of inelastic demand in a few storeys which may eventually lead to collapse of structure (Whittaker et al., 1987). In AISC (1997), it was noted that links in EBFs may undergo very large deformation at a single floor as a result of lack of force redistribution mechanism such that "*in extreme cases this may result in a tendency to develop a soft story*". Bosco and Rossi (2009) underlined that just controlling link overstrength along the building height may not be sufficient to achieve uniform inelastic demand because link overstrength is defined on the basis of the frame elastic behaviour. They recommended to also examine the inelastic structural behaviour.

In order to improve the seismic behaviour of EBFs, tied EBFs with vertical members connecting link end joints as shown in Fig. 1.1b was explored by Martini et al. (1990). The vertical members were introduced with the intent of distributing the inelastic shear deformations of the link elements over the height of the structure and prevent severe and rapid reduction of the storey stiffness upon yielding of links. This Tied Braced Frame (TBF) structural system was further studied by Ghersi et al. (2000). A design method based on modal superposition technique was developed and validated by Ghersi et al. (2003; 2006) and Rossi (2007).

TBFs can also be conceived as constituted by two elastic braced frame sub-structures coupled by ductile shear links. The concentration of inelastic demand is prevented as long as both sub-structures remain elastic. However, in order for the trusses to remain elastic, first and higher mode effects have to be overcome, which may result in large member forces in the braces, ties and columns forming the trusses. Tremblay et al. (2004) introduced an additional global hinge at the building mid-height to reduce such higher mode effects in steel braced frames. Recent shake table testing conducted by Wiebe et al. (2012) showed that combining multiple rocking interfaces with a base shear ductile mechanism could significantly reduce the force demand arising from higher vibration modes for an 8-storey steel braced frame.

In view of these developments and findings, a new the modular tied braced frame (M-TBF) system is proposed and studied in this paper (Fig. 1.1c). The force transfer path created by the vertical tie elements in TBF is interrupted between multi-storey modules in order to produce a flexural hinge in each of the elastic braced frame sub-structures to reduce the forces induced by higher mode effects. Figure 1.1c shows an 8-storey M-TBF example with two 4-storey modules. In this study, the seismic performance of the TBF and M-TBF systems are compared for an 8-storey building to investigate the behavioral potentiality and effectiveness of the proposed M-TBF system. The comparison is based on nonlinear time history response analysis results with focus on storey drift, link rotation, member force demands. In order to assess the robustness of the modularized frame configuration, the structure response is also studied under ground motions with amplitudes exceeding the design level.

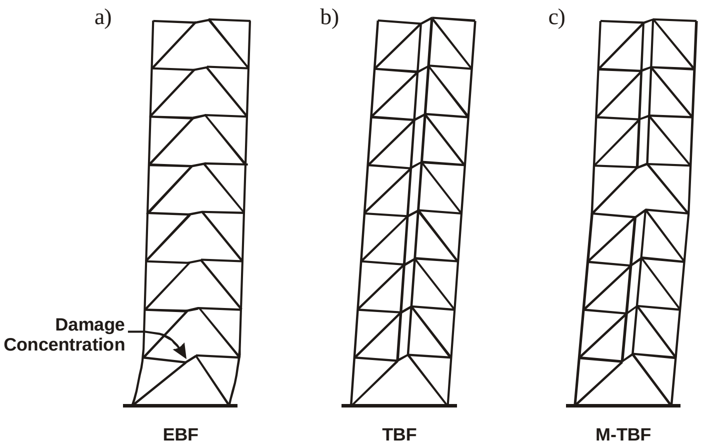

Figure 1.1 EBF systems: a) EBF; b) TBF, and c) proposed M-TBF.

2. FRAMING SYSTEMS STUDIED

2.1. Building studied

The plan view of the structures examined herein is illustrated in Figure 2.1a. The TBF and M-TBF elevations are shown in Fig. 2.1b. The building is a regular office building located on a firm ground site in Victoria, B.C., Canada. Occupancy live load was considered as 2.4 kPa and the snow load was calculated as 1.48 kPa following 2010 National Building Code of Canada (NBCC) (NRCC, 2010) requirement. Dead loads at roof and floor levels were considered as 3.4 kPa and 4.5 kPa, respectively. The weight of the exterior cladding was taken as 1.2 kPa.

2.2. Design Procedure

In the tied braced frame systems studied, beams, ties and columns form identical braced frame sub-structures that are assumed to be pinned at their bases. The two elastic trusses are connected together by the links which provide vertical shear resistance between the trusses and, thereby, resistance to lateral loads to the whole system. The links are the first elements to be designed. They are sized to resist the vertical shear forces V_f induced by the code specified seismic loads, as illustrated in Fig. 2.2a for the TBF system and in Fig. 2.2b for the M-TBF system. In view of the higher energy dissipation capacity associated to shear yielding, links are proportioned and detailed to yield in shear, according to the requirements of the applicable steel design standard.

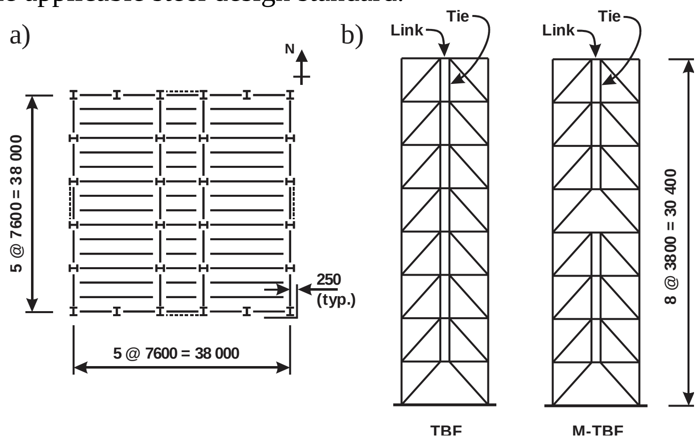

Figure 2.1 Building structure studied: a) Floor plan; b) Elevation of the 8-storey TBF and M-TBF frames.

The presence of the vertical tie members forces the inelastic demand to be uniformly distributed in adjacent storeys. Hence, variations in link overstrength along the structure height is not as critical as in conventional EBFs, and the same link beam sizes can be used over several consecutive levels, as suggested in the design procedure proposed by Ghersi et al. (2006) and Rossi (2007). In this study, the TBF links were sized on an individual basis to achieve minimum weight design and control capacity design forces. For the M-TBF system, it was expected that each module would develop simultaneous link yielding and the same section was used for all links in a given module, the links being designed for the average shear force V_f determined for the module.

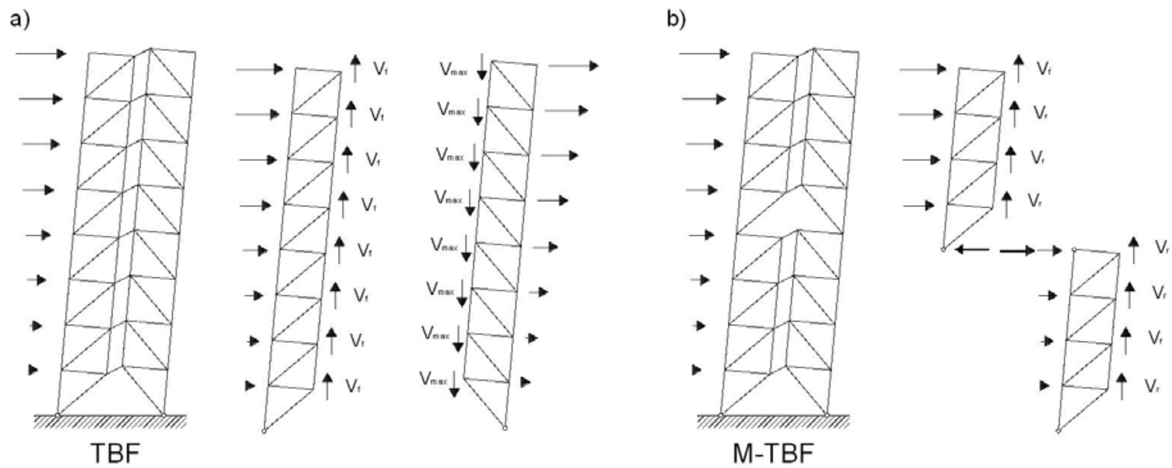

Figure 2.2. Design of the links in: a) TBF system; b) M-TBF system.

The method proposed by Rossi (2007) was used for the design of all other members of the TBF system, including the beams outside the links, the braces, the ties and the columns. These members are designed using horizontal equivalent forces F^{SD}, that include the response resulting from the first two modes of vibration. The amplitude of the forces from first mode response contributing to F^{SD} is adjusted on the basis of the probable strain hardened resistance of the links, V_{max}. The forces from the second vibration mode of the structure are based on elastic response in order to ensure that all members of the braced sub-structures remain elastic. However, the force vector from the second mode response is multiplied by a force reduction factor to more accurately represent the actual force demand observed in nonlinear time history analyses.

In absence of a defined design method for the new M-TBF system, the same approach was used to select the beams, braces, ties and columns. For this system, however, the discontinuity in the ties at the structure mid-height is expected to result in a greater reduction of member forces induced by higher vibration modes. To account for this, the choice of the sections was refined through an iterative process. Nonlinear time-history analyses were conducted in that process to determine member force envelopes that were used to adjust the member sizes.

2.3. Seismic Design

Seismic design was performed according to the provisions of the 2010 NBCC, including Uniform Hazard Spectrum with a probability of exceedance of 2% in 50 years. In NBCC, the elastic seismic forces are reduced by the overstrength- and ductility-related force reduction factors, R_o and R_d, respectively, which are equal to 1.5 and 4.0, respectively, for steel eccentrically braced frames. In NBCC, the fundamental period of vibration is calculated as 0.76 s, following the code empirical equation for steel braced frames. The computed periods in the first two modes of vibration, as obtained from dynamic analysis, are given in Table 2.1. Given that both the TBF and M-TBF structures had computed fundamental periods greater than twice the code empirical value, the spectrum acceleration at a period equal to two times the empirical period, i.e., T_a = 1.52 s, was used to determine the design base shear and seismic loads. The design base shear, V, is given in Table 2.1.

All members were selected based on the requirements given by CSA-S16 standard (CSA, 2009). Beams and columns were I-shapes made from ASTM A992 steel (F_y = 345 MPa). Square tubular sections conforming to ASTM A500, grade C (F_y = 345 MPa) were used for the braces and the ties. The length of the links was set equal to 600 mm and the primary yielding mode was in shear. The steel tonnage required for each frame is shown in Table 2.1. The modularized structure saved 15% steel compared to the original TBF design.

Table 2.1. Properties of the 8-storey frames and 84th percentile values of response parameters**.**

Structure Type	T_1 (s)	T_2 (s)	V (kN)	Steel Tonnage (t)	Base Shear (kN) (84th percentile)	Storey Drift (%h_s) (84th percentile)	Roof Drift (%h_s) (84th percentile)
TBF	1.73	0.57	1320	27.8	4070	1.1	0.74
M-TBF	1.79	0.63	1320	23.8	3010	1.1	0.74

3. ANALYSIS

3.1. Numerical models

Nonlinear time history dynamic analysis was performed using the OpenSees analysis platform (McKenna and Fenves, 2004). The model included one of the two bracing bents acting in the E-W direction of the building. The leaning gravity columns laterally supported by the braced frame studied were also included in the model. Braces and ties were modelled with forced-based beam-column elements. Initial out-of-straightness with sinusoidal shape and maximum amplitude of 1/500 of the total member length was assigned to these elements. Shear links were modelled as elastic beam column elements with concentrated end flexural and shear hinges (Rozon et al., 2008). Nonlinear material response for these elements was reproduced with the uniaxial Giuffré-Menegotto-Pinto (Steel02) steel material exhibiting both kinematic and isotropic strain hardening properties.

Rayleigh damping was specified with 3% of critical damping in the first and third modes of vibration. P-Δ effects were considered in the analyses, with gravity loads consisting of the dead load plus 50% of the live load and 25% of the roof snow load.

3.2. Selection and scaling of ground motions

Each building was subjected to a suite of 18 historical ground motion records selected from the PEER database (PEER 2011) to reflect the magnitude-distance scenarios that dominate the hazard at the site studied. The records were linearly scaled to match, on average, the design spectrum within the range of 0.2-2.0 second (Fig. 3.1).

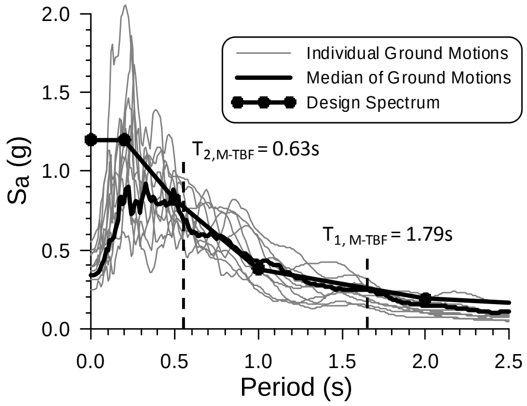

Figure 3.1 Design spectrum and 5% damped absolute acceleration spectra of the ground motion records.

4. ANALYSIS RESULTS

4.1. Comparison between TBF and M-TBF systems

The responses of TBF and M-TBF are compared in terms of storey drifts, link plastic rotations, and member forces. Table 2.1 gives the 84th percentile of the base shear, maximum storey drifts and roof drifts for each system.

4.1.1. Storey drifts

Depicted from Fig. 4.1, storey drifts for both framing systems are comparable and well controlled, below 1.5% for both systems. For the TBF system, storey drift demand is uniformly distributed in the lower storeys, but storey drifts increase in upper storeys due to higher mode effects. For the M-TBF, storey drift demand is almost identical within each module; however, a marked variation in storey drifts is observed between the two modules under some ground motions at the location where the vertical links are interrupted. The upper module shows slightly larger demand in most cases.

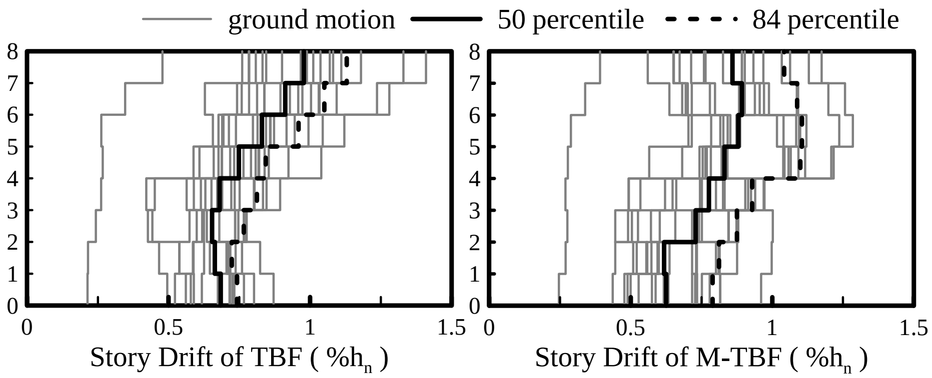

Figure 4.1. Comparison of storey drift demand between the TBF and M-TBF systems.

4.1.2. Link plastic rotations

Plastic rotations in the links are illustrated in Fig. 4.2. The TBF system shows a better capacity to distribute uniformly the plasticity among individual storeys. For the M-TBF configuration, all links within each module experienced similar plastic deformation demand, as expected in design. However, the plastic rotation in the links of the two modules can be different, generally with larger plastic rotation angles in the upper module, consistent with the storey drift demand. As shown, the code prescribed 0.08 radians limit on plastic rotation was satisfied in all ground motions for the TBF system. For the M-TBF, the limit was met, on average, but was exceeded at some levels of the upper module for some ground motions.

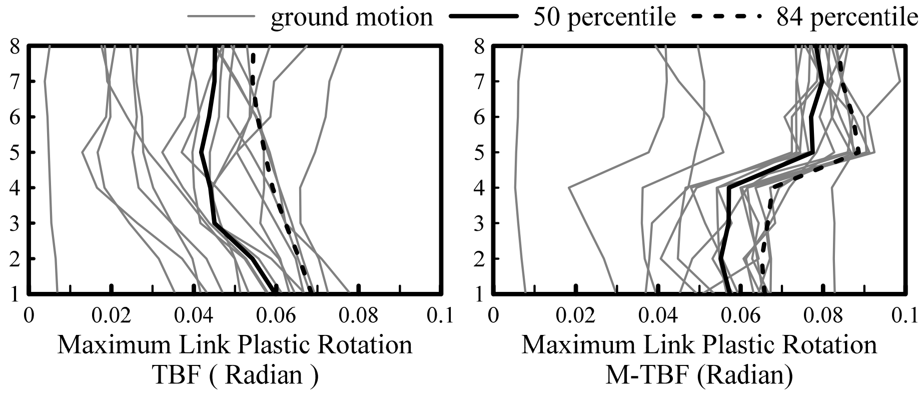

Figure 4.2. Comparison of link plastic rotation between the TBF and M-TBF systems.

4.1.3. Member forces

In Table 2.1, it is shown that using the M-TBF system resulted, on average, in a base shear reduction of 25% compared to the TBF system. In Fig. 4.3, storey shear forces are also reduced by 10-15% in all storeys in the M-TBF. In the TBF system, the relatively larger shear forces at 1^{st}, 2^{nd} and 6^{th} storeys likely result from second mode response. These shears are reduced by approximately 20-25% in the M-TBF system; however, the storey shear forces in other floors are comparable. In Fig. 4.3, axial forces in the ties are significantly reduced, by as much as 70%, in the M-TBF configuration. This reduction is attributed to smaller overall bending moments developing in each sub-structure of the M-TBF system as a result of the moments being released at mid-height due to the interruption of the tie members. Axial forces in the exterior columns are almost identical for both systems because they are essentially contributed by yielding of the link beams. However, up to 15% reduction is found at intermediate levels due to the reduction in the bending moments in the separate sub-structures.

4.1.4 Modal contributions to response

Structural response for both systems is investigated in detail in Figs. 4.4 and 4.5. In Fig. 4.4, the axial force response in one of the first-storey braces and in one of the tie members located at the seventh floor of the TBF and M-TBF systems are selected to demonstrate the differences between the two systems. Time history response and frequency content of the member force responses are given in the figure for the strong motion part of one of the earthquake records. The first-storey brace axial force reflects the base shear demand in the structure while the force in the 7^{th} level tie member represents the bending moment in the individual sub-structures in the upper part of the structures.

For the TBF system, the forces in the tie are essentially contributed by the first two modes of vibration, with marked second mode contribution. For the M-TBF system, the time history response is more complex and the frequency analysis shows a shift in the second mode frequency response. The response of the tie member in the M-TBF system is dominated by first and third mode, without any significant second mode contribution. For this structure, interrupting the individual braced frames at mid-height introduced nonlinearity in the second mode response of the structure and reduced considerably that response in the upper module.

Figure 4.5 shows the frequency content for the roof horizontal displacement under the same ground motion studied in Fig.4.4. For the M-TBF, the contribution from second vibration mode to roof displacements is also reduced and is affected by a period shit due to nonlinear response.

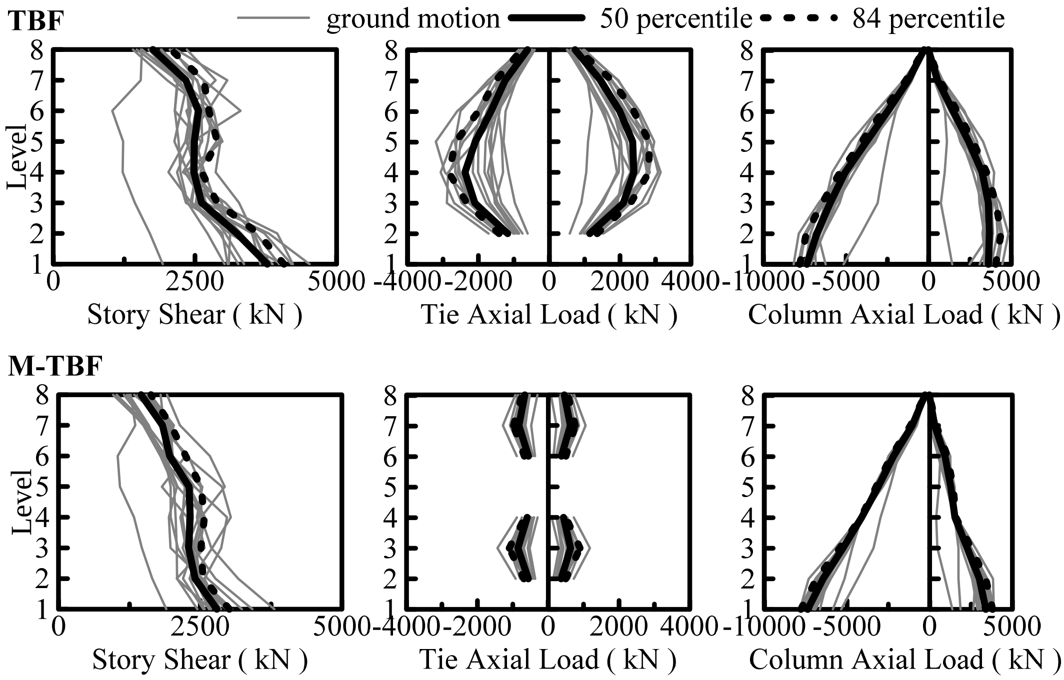

Figure 4.3. Comparison of storey shear, tie and column axial forces between the TBF and M-TBF systems.

4.2 M-TBF response under higher amplitude ground motion excitations

In a second series of analyses, the M-TBF structure was excited with the same ground motions except that the amplitude of the signals was doubled. In Fig. 4.6, the 50 percentile values of the storey drifts, storey shears, tie axial forces and column axial forces obtained from the two levels of motions are compared. The mean storey drifts approximately doubled when doubling the ground motion amplitude, indicating larger plastic deformations in the links, but the vertical distribution of the storey drifts remained nearly unchanged, showing that the system was capable of preventing soft-storey response under ground motions exceeding the design level amplitude. Storey shear forces increased by 15-20% in all storeys. Axial forces in the ties also increased in a similar proportion. Axial forces in the exterior columns were not sensitive to ground motion amplitude as they are mainly governed by the shear capacity of the links.

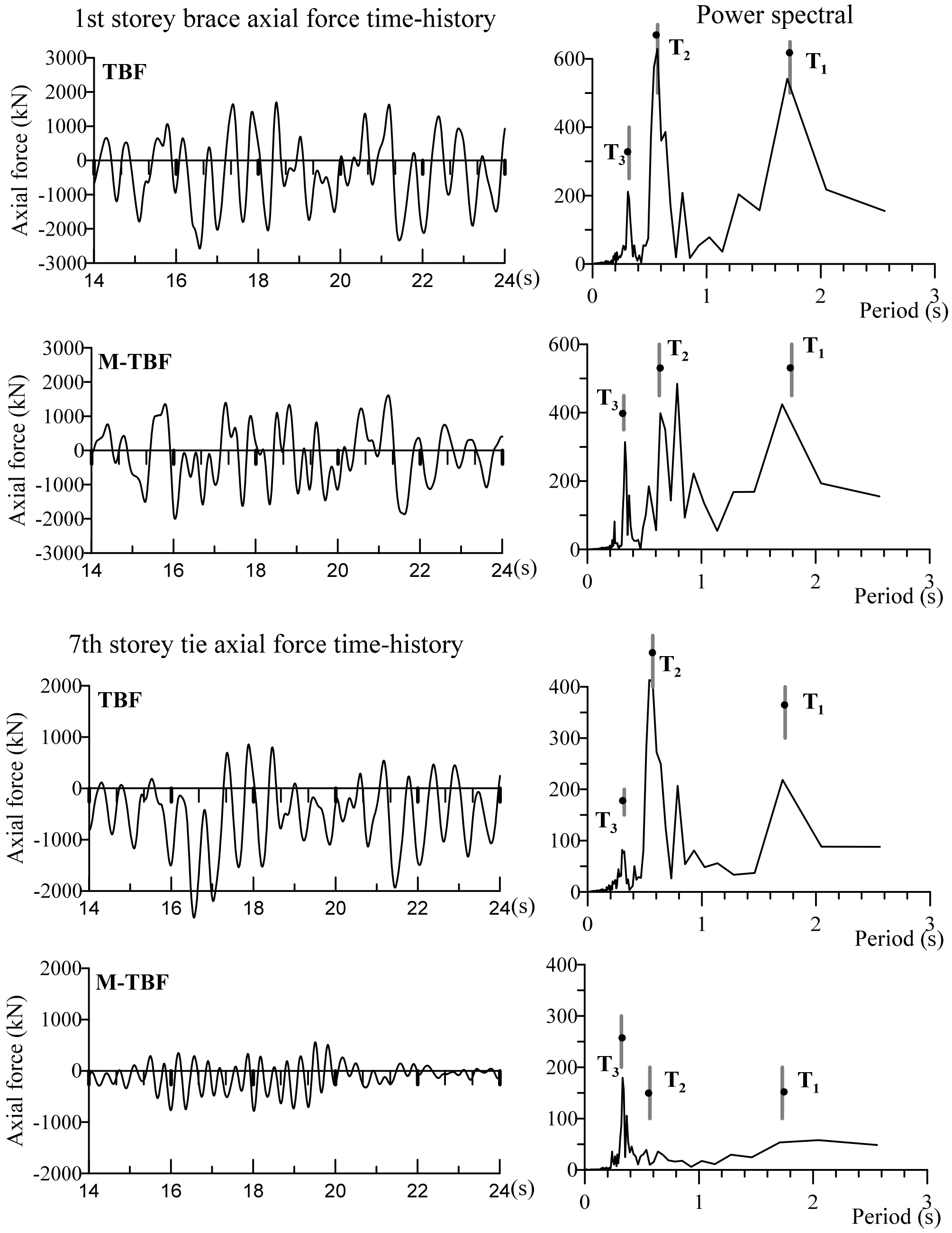

Figure 4.4. Comparison of brace and tie responses in time domain and frequency domain under Hector Mines, Joshua Tree, 1999

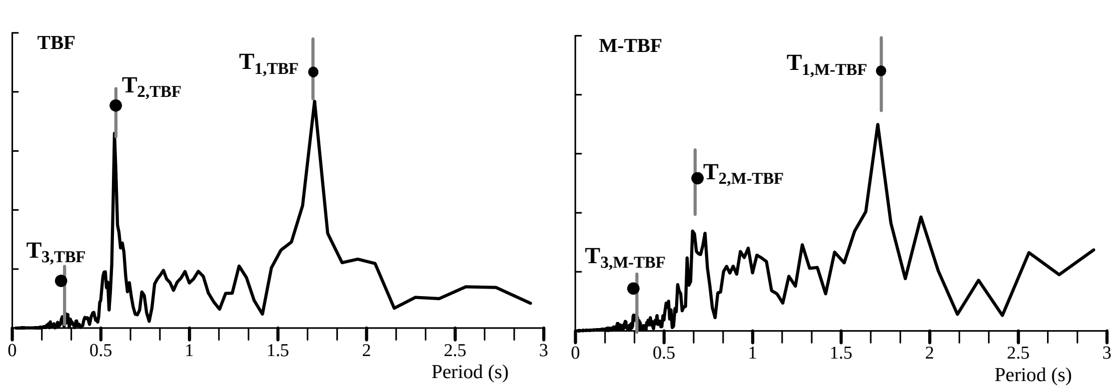

Figure 4.5. Power spectral of roof drift under Hector Mines Joshua Tree, 1999

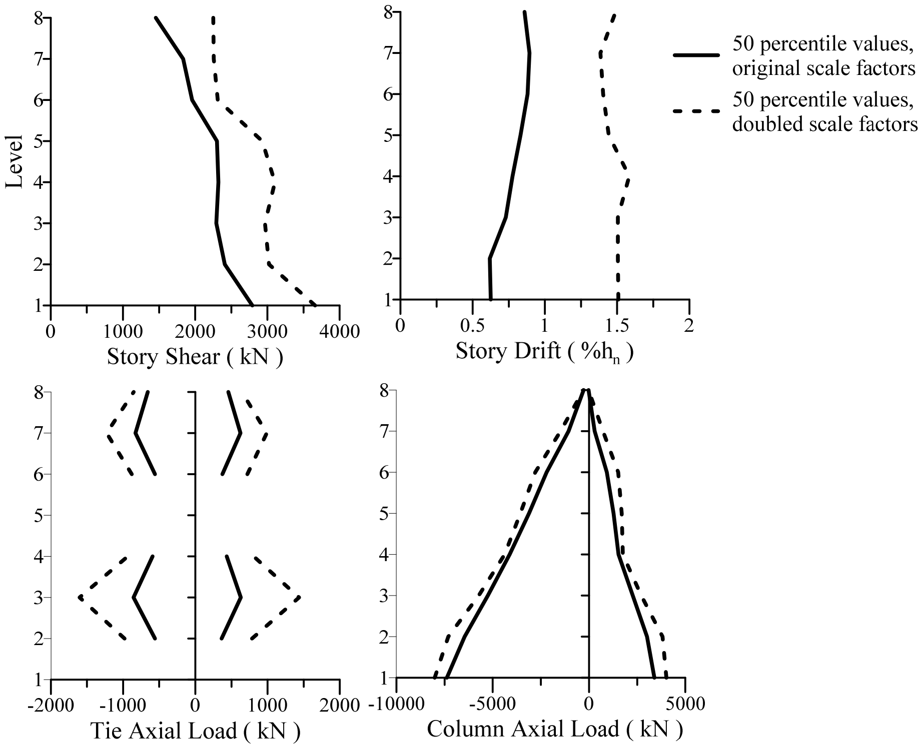

Figure 4.6. Comparison of the 50 percentile values of storey drifts, storey shears, tie axial forces and column axial forces under ground motion set with different scale factors

5. CONCLUSIONS

An exploratory study was performed to compare the seismic nonlinear response of two different tied eccentrically braced frame systems, the TBF and the M-TBF systems, designed to mitigate soft-storey response in multi-storey building structures. Nonlinear time history analyses were conducted on an 8-storey building to assess the storey drift, link plastic rotation and member force demands on these framing systems. Both frames studied exhibited satisfactorily performance in terms of storey drift response; however, peak storey drifts, link plastic rotations and member forces varied between the systems. The TBF system exhibited smaller and more uniform storey drift demand and link plastic rotation angle along the building height at the expense of significantly higher force demand in the tie members. The modular frame resulted in reductions in storey shears and brace forces in some levels. Detailed investigation showed that the second mode response for the M-TBF was reduced by the interruption in the tie members at the building mid-height. The study also revealed that storey shears and axial forces in ties in the M-TBF system increase when the amplitude of the ground motions is increased.

This study shows that the modular concept has potential for cost-effective and robust steel braced frame solutions for multi-storey buildings. This comparative study was however limited to a single building (8-storey) and the conclusions must be validated for additional building cases. Larger storey drifts and link plastic rotations were observed in the upper module due to uncoupling of the modules. This localized higher demand is likely to be mitigated by introducing vertical ductile links connecting the tie members between adjacent modules. The benefit of this alternative should also be investigated in future studies.

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