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Steel truss moment frames with self-centring energy dissipation mechanisms for enhanced seismic performance

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ABSTRACT

This article introduces truss moment frame (TMF) systems that are detailed with three different energy dissipation mechanisms to achieve enhanced seismic response. The TMF systems studied are F-TMFs equipped with friction dampers, FT-TMF with friction dampers and elastic tendons and RS-TMFs with ring spring friction dampers. The latter two systems offer re-centring capabilities to mitigate P-delta effects and control residual displacements. In all three systems, the energy dissipation (ED) devices are inserted between the extensions of the truss bottom chord and the columns. Truss members and columns are designed to resist gravity load effects together with the maximum forces developed in the ED mechanisms. The seismic response of the three TMF systems is examined and compared for a single-storey industrial building located in Vancouver, BC. The TMFs were designed in accordance with the 2015 National Building Code of Canada using a value of 4.0 for the R_d factor. The response of the structures is investigated by means of nonlinear response history analysis using ground motion records scaled for a hazard level corresponding to a probability of exceedance of 2% in 50 years. Structural collapse was not observed in none of the structures studied. The F-TMF building sustained the largest peak and residual displacements due to progressive drifting occurring in one direction. This undesirable behaviour was mitigated when using the two other systems with re-centring capacity. The RS-TMF exhibited the most stable nonlinear response with no residual displacements. The FT-TMF system also displayed excellent response with reduced force demands compared to the TMF with the ring spring dampers.

Keywords: Friction, residual displacements, ring spring, tendon, truss moment frames,

INTRODUCTION

The truss moment frame (TMF) system has been proposed for the seismic resistance of building structures with long clear span conditions [1-3]. The conventional configuration for which seismic energy dissipation is expected to occur within a special segment of the truss is illustrated in Figure 1a. The special segment may be a Vierendeel panel with plastic hinges forming in the top and bottom chords at the corners and, when used, near the ends of intermediate web members. Special segments can also be formed of X-braced panels such energy is also dissipated through yielding of the diagonal members in tension and compression. Design guidelines have been developed for this system [4] and code provisions have been included in the AISC Seismic Provisions [5]. Under severe earthquakes, it is anticipated that the system will sustain permanent residual lateral deformations and damage to the floor and roof structures. The possibility of excessive plastic deformations and premature fracture of truss members is also a concern for trusses with short special segments [6-9]. Performance-based design approaches as well as special segments incorporating double HSS chord members, diagonal viscous dampers or diagonal buckling restrained braces have been proposed to improve the seismic behaviour of the system [10-12].

An alternative truss moment frame configuration shown in Figure 1b has been created by introducing buckling restrained knee-braces between the columns and the bottom chord extensions of regular floor and roof trusses [13-15]. In this promising system, referred to as BRKB-TMF, seismic induced energy is dissipated in the BRB members acting as structural fuses such that the gravity load carrying function of the trusses is not altered during earthquakes. The BRB members can also be easily replaced after a severe earthquake. The system is however prone to residual lateral displacements and its seismic performance of the system however relies on the capacity of developing the required cyclic ductility in short BRB members, which still represents a challenge today. This article explores three other structural fuse designs that could be used in lieu of the BRB elements: friction dampers, friction dampers coupled with steel tendons and ring spring dampers. The latter two exhibit self-centering capabilities that can be exploited to minimize residual displacements. The three systems are first briefly described. They are then applied for a simple, single-span, single-storey building located in Vancouver, British Columbia. Different options are investigated in the design and nonlinear response history analyses are performed to evaluate and compare the seismic response that can be achieved with each system.

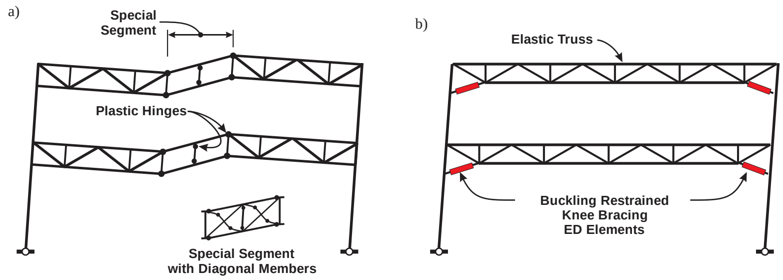

Figure 1. Truss moment frame systems: a) Conventional configuration with a yielding segment; b) Newly proposed configuration with buckling restrained knee-bracing energy dissipation elements.

PROPOSED TMF SYSTEMS

Systems studied

The three different systems are illustrated in Figure 2 for the prototype structure that is examined later in the article. In the first system, friction damper assemblies are introduced between the columns and the tips of the truss bottom chord extensions. Friction dampers can be designed and detailed to offer a predictable and uniform slip resistance over the anticipated range of deformations. Their ability to dissipate seismic input energy has been verified in several past experimental programs [16-17]. One main advantage of the system, defined herein as F-TMF, is its limited lateral overstrength as the forces they impose to the columns and adjacent truss members when slipping are well controlled. However, framing systems with friction dampers exhibit no post-slip lateral stiffness and are therefore prone to progressive drifting due to P-delta effects and residual displacements.

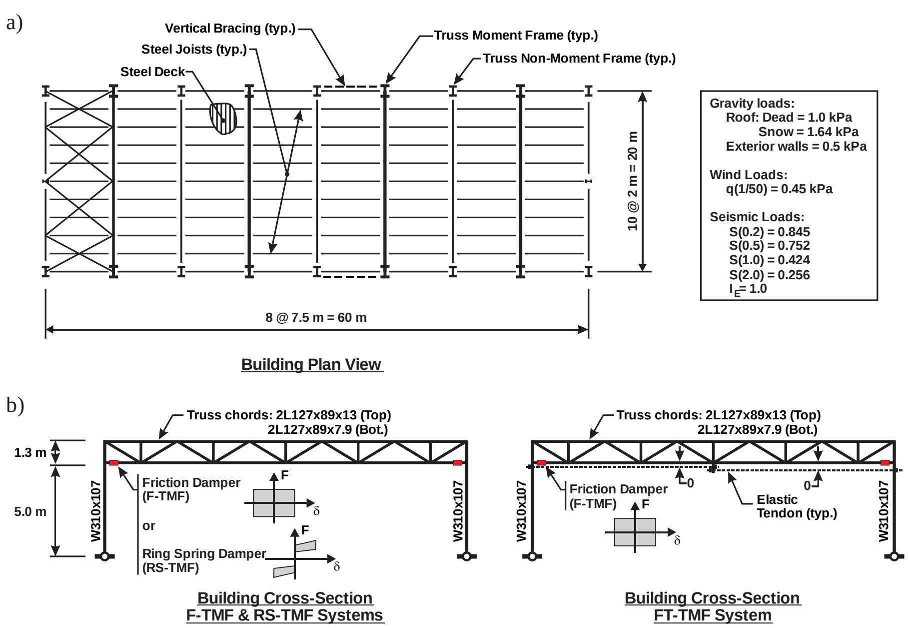

Figure 2. Building studied: a) Building plan view and design loads; b) Building cross-sections illustrating the F-TMF, RS-TMF and FT-TMF systems.

This drawback can be eliminated by using dampers that display self-centering behaviour characterized by a flag shape hysteretic load-deformation response. This behaviour can be obtained using different mechanisms. In this study, dampers that are built with pre-compressed ring springs that can be sized and arranged to develop the desired activation load and post-activation stiffness properties in both directions [18] were considered. In the article, the TMF system with the resulting self-centering capacity is referred to as RS-TMF. An alternative strategy to obtain TMFs with re-centring capabilities is to add to the F-TMF system horizontal tendons that connect the columns to given points along the truss bottom chord. When the frame laterally deforms in one direction and slip is triggered in the friction dampers, the tendon acting in parallel with the friction damper under tension is activated and develops a positive force that tends to return the structure to its original position. In this FT-TMF (friction dampers with tendons) system, the length of the tendons can be extended up to the full length of the truss such that strains in the tendons remain in the elastic range under the maximum anticipated storey drifts. In figure 2, tendons spanning over half the building width are illustrated. The cross-section area of the tendons is then adjusted to achieve the required lateral stiffness when the dampers are slipping. This concept has already been investigated for developing self-centering capabilities for steel plate shear walls [19-21].

Prototype building

The application of the three alternative systems is examined for a simple single-storey industrial building assumed to be constructed on a class C site in Vancouver, British Columbia. The structure is illustrated in Figure 2. The roof trusses span over the full 20 m width of the building and are spaced 7.5 m apart along the length of the structure. The trusses support open web steel joists that are placed 2 m o/c. The building total height is 6.3 m and the depth of the trusses is 1.3 m. Design gravity loads are given in the figure, together with the wind and seismic data for the site. As shown on the building plan view, truss moment frames resisting lateral loads in the N-S direction were only formed at every 15 m. The other truss-column assemblies were assumed to be pin-connected and laterally stabilized by the truss moment frames through the roof diaphragm. All columns were considered as pinned at their bases.

Design of the TMF configurations

The design of the truss moment frames was performed in accordance with the 2015 National Building Code of Canada (NBCC) and the CSA S16 standard for the design of steel structures in Canada [22-23]. The structure was assumed to be of the normal importance category. The truss members were designed to resist combined factored dead and snow loads assuming simply supported conditions at the truss ends. Trusses part of the TMFs were also verified assuming rigid connections with the columns for the applicable combinations of gravity load, gravity plus wind loads, and gravity plus earthquakes loads. The TMF columns were also selected for these load combinations and control drifts under wind and seismic loads. The selected shapes for the truss chords and columns are indicated in Figure 2b.

For a 15 m long building segment tributary of each TMF, the seismic weight was 495 kN and the TMFs had a fundamental period of 1.17 s. With this period, the anticipated seismic drift including nonlinear deformations, was estimated to be 134 mm, i.e. 2.1% h. The design seismic load was established using the static force procedure using a period T_a = 0.51 s corresponding to the upper limit of 1.5 times the period from the NBCC empirical formula for steel moment frames (0.34 s). The factor for multi-mode effects and the importance factor were both set equal to 1.0. Force modification factors R_d and R_o are not specified in the NBCC for this system. For this exploratory study, the following values were adopted: R_d = 4.0 and R_o = 1.0. This resulted in a design base shear V = 92 kN for each TMF, which induces axial loads of 223 kN in the truss top and bottom chords. When considering combinations of factored gravity, gravity plus wind and gravity plus seismic loads, the axial load in the truss bottom chord extension, where the dampers are located reach 234, 314, and 312 kN, respectively.

According to the NBCC design approach, activation of the dampers should not occur under these loads and an activation (slip) load of 315 kN was therefore selected for all dampers. When comparing this value to the code specified seismic force demand alone, this resulted in an effective seismic overstrength of 315 kN / 223 kN = 1.4. Reducing this overstrength was the main motivation for using TMFs at every other bay. For the same loading conditions, dampers of TMFs spaced at 7.5 m o/c would have had to resist a seismic induced axial load of 111 kN only while being designed not to slip under a load of approximately 200 kN to resist factored load effects, which would have resulted in an overstrength of 1.8 and less effective design.

Under specified gravity and wind loads, the storey drift is equal to 26 mm = h/242, which was deemed acceptable for an industrial application. This completed the design of the F-TMF system. For the RS-TMF with ring spring dampers, a damper assembly was designed having an activation load of 315 kN and a deformation capacity of 28 mm. This deformation corresponds to 2.0 times the deformation induced by the difference between maximum expected storey drift and storey drift at damper activation (= 1300 mm / 6300 mm × [134 mm – 66 mm] = 14 mm); the factor 2.0 being applied to account for expected differences between actual and predicted seismic displacements. Ring springs by Ringfeder [24] were considered to obtain the realistic properties for the damper assembly. A 510 kN force capacity assembly was selected with a pre-load of 315 kN. By adjusting the number of spring elements to achieve the required deformation capacity, the damper was found to exhibit post-activation stiffness values of 4.09 kN/mm upon loading and 1.36 kN/mm upon returning to original position. For the FT-TMF system with friction dampers and tendons, the tendons are sized to obtain a net positive lateral stiffness for the frame when including P-delta effects. Negative stiffness from P-delta effects is equal to -P/h, where P is the concomitant gravity loads (dead load + 0.25 snow load) supported by the 15 m building length segments, i.e. 300 m^2 × (1.0 kPa + 0.25 × 1.64 kPa) = 423 kN, and h is the building height (6300 mm). Figure 3 shows a free-body diagram of the system illustrating the additional loads and deformations that develop after slip has initiated in the dampers. The force T is the force in one tendon as only one tendon at a time is active in anyone direction, whereas forces V, T, C and P are for the entire TMF. Moment equilibrium gives:

$$V\,h\ +\ P\,\Delta = T\,d \qquad => V = T(d/h) + (P/h)\Delta \tag{1}$$

and the tension force in the tendon is given by:

$$T = k_T\delta \quad \text{, where: } k_T = EA_T/L_T \text{ and } \delta = \Delta(d/h) \tag{2}$$

, where k_T, A_T and L_t are respectively the axial stiffness, cross-section area and length of the tendon and δ is the elongation of the tendon. From these two equations, the net lateral stiffness of the frame when slips occurs in the dampers is:

$$V/\Delta = k_T(d/h)^2 - P/h \tag{3}$$

In this equation, $-P/h$ is the negative stiffness from P-delta effects (= -0.0671 kN/mm), and the required minimum stiffness k_t for net positive frame stiffness is therefore equal to 1.58 kN/mm. This tendon stiffness can be achieved with one 10 m long tendons, as shown in Figure 2b, having a cross-sectional area A_t = 79 mm^2. For such a configuration, a design 28 mm slip in the damper, as determined above, would cause 0.28% strain in the tendons, which represents 30% of the strain at ultimate for 7-wire prestressing strands made of steel having an ultimate tensile stress of 1860 MPa. For this study, two tendon designs were considered to evaluate the effect of k_t on the seismic response: A_t = 141 mm^2 (one 15 mm strand) and A_t = 282 mm^2 (two 15 mm strands). These two tendon designs result in a frame lateral stiffness of 0.12 and 0.24 kN/mm, i.e. 1.79 and 3.58 times the P-delta stiffness. The second FT-TMF with two pairs of tendons is referred to hereafter as FT2-TMF. Eq. 3 can also be used to determine the lateral stiffness of the RS-TMF when the ring spring dampers are in the post-activation range. In this case, dampers at both ends of the truss bottom chord contribute, which gives a frame lateral stiffness of 0.348 kN/mm, a value higher than that provided by the tendons of the FT-TMFs.

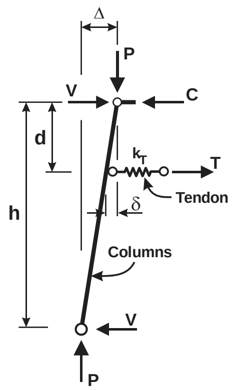

Figure 3. Equilibrium of the FT-TMF system upon activation of the dampers.

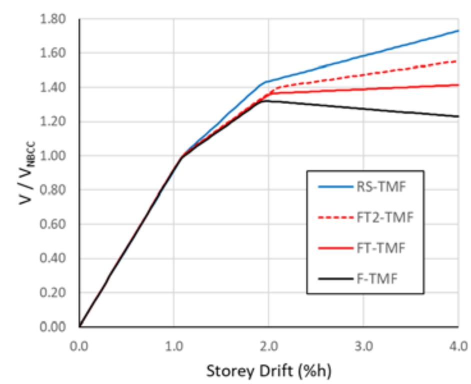

Figure 4. Load-deformation responses of the TMF systems studied

Figure 4 shows the lateral load-deformation response of the structures studied from nonlinear static analysis performed up to a storey drift of 4%*h*. Before applying the gradually increasing lateral load, concomitant gravity loads (D + 0.25S) were applied and maintained during the lateral analysis. The dampers were therefore initially subjected to compression from gravity loads, which resulted in a sequential activation of the two dampers upon lateral loading. P-delta effects were considered in the analyses. As shown, activation of the first damper occurred at approximately 1%*h* when lateral load reached the code design base shear, as predicted in design. Activation of the second damper occurred at a lateral load approximately equal to 1.4 V_{NBCC}, at drifts close to 2%*h*. This was followed by a linear branch exhibiting negative stiffness for the F-TMF system and positive stiffness for the FT- and RS-TMF systems, as was also planned in design.

NONLINEAR RRESPONSE HISTORY ANALYSES (NLRHA)

Analysis procedure

The structures were modelled using the SAP2000 structural analysis software [25]. The models included the truss moment frames studied and one leaning column representing the companion non-moment trussed frame. The upper end of the leaning column was linked to the trussed moment frame by means of a rigid horizontal diaphragm constraint. Nonlinear link elements were used to reproduce the expected hysteretic behaviour of the friction and ring spring dampers. Elastic frame elements were employed for the remaining structural elements. The frame elements for the tendons were assigned a zero-compression load limit to reproduce tension-only response with negligible compressive strength. Rayleigh damping corresponding to 3% of critical in the first two vibrational modes of the structures was assigned to the model. Stiffness proportional damping was assigned to the frame material to avoid excessive forces developing in the link elements. NLRHA was performed under an ensemble comprising three suites of 5 ground motion records, one suite for each sources of earthquakes contributing to the seismic hazard at the site: crustal earthquakes, subduction in-slab earthquakes and subduction interface earthquakes. The ground motion records were selected and scaled in accordance with the NBCC 2015 guidelines. The resulting 5% damped acceleration spectra of the scaled records of each suite are presented in Figure 5. Prior to initiating the time history analyses, concomitant gravity loads D + 0.25S were applied to the structure and maintained for the duration of the ground motion records. P-delta effects were included in the analyses.

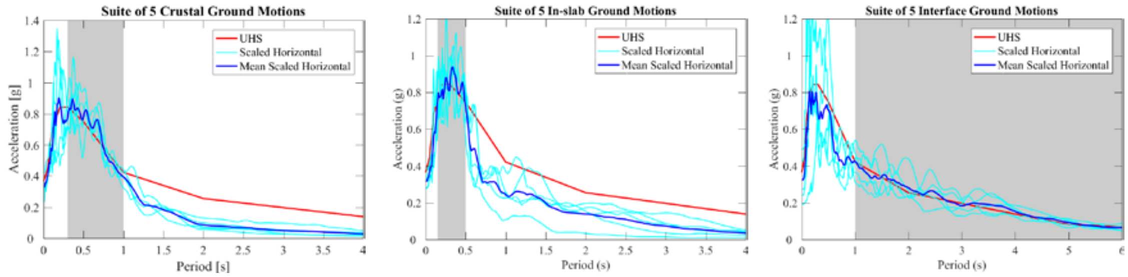

Figure 5. 5% damped spectra of the ground motion records of each suite.

History responses

The response of the four systems under one crustal earthquake ground motion (CR1) and one ground motion record from a subduction interface earthquake (SI5) are compared in Figure 6. Under the CR1 ground motion, all systems behave similarly, exhibiting nearly symmetrical responses. The RS-TMF returned to its initial position after the earthquake whereas the other three systems experienced some, but small residual drifts. Under the long duration subduction SI-5 signal, the RS-TMF exhibited the same response, without any permanent deformations due to its superior re-centering capacity. The same response was achieved with the FT2-TMF system designed with two pairs of tendons. The F-TMF displays moderate drifting with a small residual drift of 0.2%*h* at the end of the earthquake. Contrary to the other systems, the F-TMF with friction dampers only sustained significant progressive drifting under this ground motion, reaching a peak value of 4.1%*h*, in excess of the NBCC limit, and a permanent offset of 3.1%*h* after the event. The response under this motion, which is the worst among all ground motions, shows that such a system with no re-centering capacity is more prone to P-delta effects.

Storey shear vs storey drift hysteretic responses of the four systems under the SI5 motion are presented in Figure 7a. The one-sided response with negative stiffness in the nonlinear range of the F-TMF can be clearly seen. As expected, the RS-TMF shows symmetrical response with re-centering in every loading cycle, despite its limited energy dissipation capacity. The two systems including friction dampers acting with tendons have very similar response characterized by high energy dissipation and drifts oscillating about the zero-deformation position. Clearly, the restraining effects from the tendons improves the seismic stability of these frames in the nonlinear range. For the FT2-TMF system, the positive lateral stiffness provided by the tendons can be readily observed on the hysteretic loops. The load-deformation response of the friction dampers of all systems are plotted in Figure 7b. In all cases, the response is non-symmetrical with bias towards negative values due to the compression force induced by the gravity loads in the end segments of the truss bottom chord. The dampers behaved as expected with a constant slip load of 315 kN for the F and FT frames and a flag shaped hysteresis for the damper with friction ring springs.

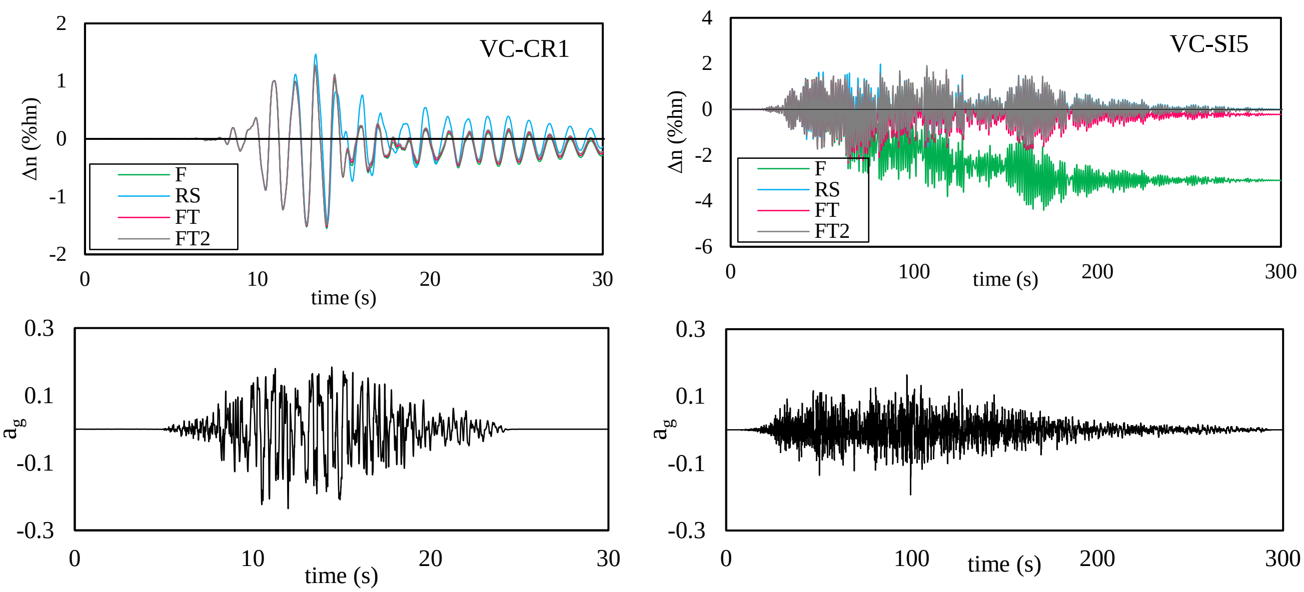

Figure 6. Time history response of the roof displacements under crustal and subduction interface ground motion records.

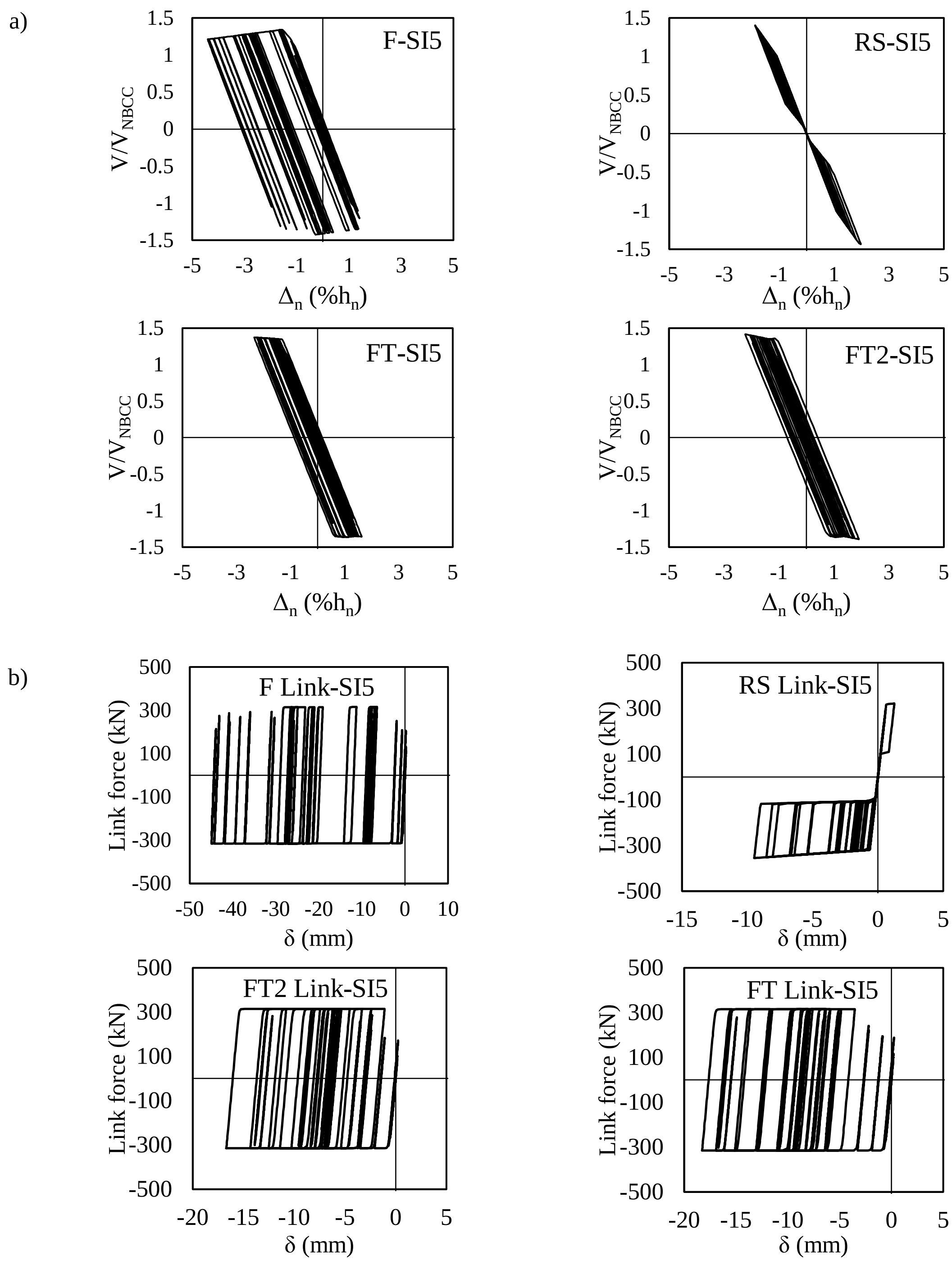

Figure 7. a). Storey shear vs storey drift hysteretic responses of the systems *b).* Load vs. deformation response of the friction dampers (links) of the systems

Response parameters

Seismic design values of the peak storey drifts, residual storey drifts, peak negative and positive slip distances in the dampers, and base shears are presented in Table 1. According to the NBCC 2015 guidelines for nonlinear dynamic seismic analysis, the design values for each response parameters was taken as the mean value of the 5 largest results obtained for the 15 ground motion records. As expected, the F-TMF sustained the largest peak and residual drift values. The FT2- and RS-TMF systems showed the best performances regarding peak and residual drifts, respectively. For The RS-TMF, the self-centering capacity could not entirely compensate the lower energy dissipation, which resulted in slightly larger drifts compared to the systems equipped with friction dampers and tendons. In all systems, the dampers generally experienced larger negative deformations due to gravity loading effects. This unsymmetrical behaviour results in limited tensile strains in the tendons, indicating that tendons can be effectively utilized to control the structure response. The systems experienced similar base shear forces that are consistent with their drift responses, as could be predicted from the pushover curves in Figure 4. Because the larger drifts and higher post-activation stiffness, the RS-TMF system developed higher lateral resistance.

Table 1. Response Parameters.

System	**Peak Δ (%h)**	**Residual Δ (%h)**	**Peak δ^- (mm)**	**Peak δ^+ (mm)**	**Peak V/V_{NBCC} ( )**
F-TMF	2.96	1.46	19.8	4.5	1.40
RS-TMF	2.52	0.0	13.6	6.4	1.51
FT-TMF	2.37	0.60	13.7	2.1	1.37
FT2-TMF	2.27	0.49	14.0	1.9	1.42

CONCLUSIONS

A study was performed to examine the seismic response of three alternative steel truss moment frame systems with energy dissipation mechanisms place between the column and the truss bottom chord ends: TMF with friction dampers (F-TMF), TMFs with ring spring dampers (RS-TMFs), and TMFs with friction dampers and steel tendons (FT-TMFs). The RS- and FT-FTM systems can exhibit re-centring capabilities to mitigate P-delta effects and reduce residual displacements. A prototype structure located in Vancouver, BC, was designed for each system and their seismic response was examined through nonlinear response history analysis. For the FT-TMF system, two tendon designs were considered: one and two 15 mm tendons on either side of the truss. The conclusions of this study can be summarized as follows:

- Although it exhibits a stable hysteretic response, the F-TMF system with friction dampers only was found to be prone to large peak and residual seismic displacements due to P-delta effects.
- Systems designed with ring spring friction dampers exhibited a symmetrical flag-shaped hysteretic response that provided sufficient re-centering capacity that entirely eliminated residual lateral displacements.
- The re-centering capacity combined with high energy dissipation of the FT-TMF system also resulted in excellent seismic response with no structural damage nor permanent deformations. Increasing the number of tendons improved further the performance of the structure.

The study showed that simple enhancements to the TMF system can result in significantly improved seismic resiliency for single-storey buildings with large span roof trusses. Further studies should be conducted to investigate the applicability of the proposed concepts to multi-storey buildings.

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