Seismic upgrading of friction-damped steel frames integrated with self-centring devices

A. Hashemi, H. Bagheri & P. Quenneville
The University of Auckland, Auckland.

P. Zarnani
Auckland University of Technology, Auckland.

ABSTRACT

Conventional friction-based damping devices are known for their large energy dissipation capacity and economic benefits compared to other seismic mitigation systems. These devices have been implemented in many practical projects around the world since their introduction to the construction industry during the 1980s. From that time, not only did the building standards have evolved and become more demanding, but also stricter controls are proposed for the design of low damage seismic-resistant structures. This paper presents the integration of self-centring friction-based tension-only braces with friction-damped braced frames for the application in high importance steel structures. With the proposed concept, the structure can resist intense earthquakes whilst the inter-storey drifts are controlled. Furthermore, the self-centring capability of the structure is significantly improved compared to systems with only conventional friction-damped braces. A step-by-step procedure for this integration is provided and applied to a case study structure. The numerical results showed that the designed structure could satisfy the criteria related to high importance buildings with significantly improved performance characteristics. The findings of this paper confirm the potential of the proposed concept as an alternative solution for seismic resilient steel structures.

1 INTRODUCTION

The application of sliding friction devices in steel braced frames was originally proposed by Pall et al. (Pall and Marsh 1982) as an effective way to control and mitigate the seismic damage. Their analyses showed that a large portion of introduced seismic energy can be absorbed by these devices resulting in better performance under earthquakes. Later on, Popov et al. (Popov et al. 1995) and Clifton et al. (2007) introduced new versions of friction connections for the use in steel moment frames.

Owing to the unique characteristics of these devices, including a large energy dissipation ratio, they have been implemented in many practical projects around the world. Although a recent study showed that the repair cost of friction-damped steel frames is lower compared to common steel systems (Yeow et al. 2018), however, one potential drawback of these connectors is the possible residual deformations at the end of the earthquakes (Hashemi et al. 2016). As the energy absorption capacity of the system increases, a larger restoring force is required to re-centre the structure (Wang et al. 2020). Thus, damping devices with large damping ratios (such as friction devices) are less likely to demonstrate a self-centring behaviour. There have been several attempts to combine conventional friction dampers with supplementary elastic (or elastoplastic) elements to create the restoring force required to self-centre the system. Tremblay et al. (Tremblay et al. 2008) proposed a self-centring steel brace by integrating friction dampers and pre-tensioned tendons. Kim et al. (2008) introduced the combination of friction dampers and post-tensioned bars for the use in self-centring steel moment-resisting frames. Since the application of friction devices has begun, not only did the building standards evolved (mostly affected by significant seismic events that occurred in different parts of the world) but also the performance objectives for seismic-resistant structures have changed. With the introduction of low damage seismic design, controlling the inter-story drifts under different design limit states has become more crucial. Therefore, it is possible that the design philosophy adopted for conventional friction-damped braced frames need to be revised so the resulted structure could meet stricter performance objectives.

This investigates the seismic performance of friction-damped braced frames integrated and upgraded with resilient tension-only braces in a way that the strengthened structure can satisfy the low damage design requirements related to a higher importance level compared to the original design. The results give an insight to researchers and engineers about how a dual lateral load resisting system performs under intense shakings.

2 SEISMIC UPGRADING OF FRICTION-DAMPED STEEL FRAMES

The criteria considered for strengthening a friction-damped steel braced frame is to upgrade its lateral load resisting system in a way that the structure which originally was designed as a normal building with importance level of 2 can resist seismic demands related to importance level 4. The New Zealand standard for earthquake actions categorises buildings into four groups based on the importance level considered. It starts from Importance Level 1 (IL1) for low hazard structures to Importance Level 4 (IL4) for high importance structures.

The serviceability requirements for these two categories are different. For normal buildings (IL2), the minimum Serviceability Limit State (SLS1) is required. However, for high importance buildings (IL4), in addition to the SLS1 requirement, the operational continuity criteria (SLS2) should also be considered and checked for the design (see Table 1).

Table 1: Summary of the significant test properties.

Annual probability of exceedance	Return period factor (R_s or R_u)	Normal buildings (IL2)	High importance Buildings (IL4)
1/2500	1.8	MCE	ULS
1/500	1.0	ULS	SLS2
1/25	0.25	SLS1	SLS1

Note that The Maximum Considered Event (MCE) is not explicitly defined in the New Zealand Standard. However, for normal buildings, it is stated that the actions can be specified by the return period of R_{MCE}=1.8 corresponding to a 1/2500 annual probability of exceedance. For high importance buildings, (Bruneau and MacRae 2017) recommend is to use a 7500-year return period factor of R_{MCE}=2.25. Figure. 1 shows the schematic pushover plot of the structure with the maximum allowable drift values indicated for different limit states. It was demonstrated that permanent residual displacements could compromise the performance of steel structures (Clifton et al. 2011). This is more critical for high importance structures, given that most of these structures are required to rapidly return to service following a major earthquake. Considering the information in Table 1 and the literature, the following performance objectives are proposed for the high importance structures considered in this study:

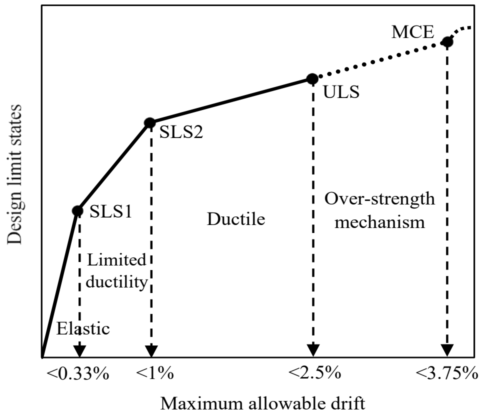

Figure 1: Design limit sates and maximum allowable inter-story drifts

1. The structure to remain linear and elastic during SLS1 level shakings with the lateral drifts kept under 0.33%.
2. The structure to behave slightly ductile under the SLS2 level shakings and the lateral drifts kept under 1%.
3. The structure to demonstrate a ductile behaviour with appropriate ductility factor considered for the design level actions (ULS). The lateral drifts should be kept under 1.5% for this limit state.
4. Appropriate over-strength mechanism is considered for the structure to be able to tolerate MCE level shakings with the lateral drifts kept under 3.75%.
5. The structure to demonstrate a self-centring behaviour with no or negligible residual deformation following a design level (ULS) shaking.

3 CASE STUDY STRUCTURE CONSIDERED FOR THE UPGRADE

A prototype building with steel friction-damped braced frames is designed, modelled and considered for strengthening. Figure 2 illustrates a schematic plan view of the building. It was assumed that the story height is 3.8 m for the first floor and 3.3 m for the other four floors. The permanent loads considered for the preliminary design of the building are a self-weight of 0.8 kPa for the frame, a 3.3 kPa floor and a cladding wright of 0.8 kPa. The imposed loads considered were 3.0 kPa for all floors and 0.5 kPa for the roof. These loads result in seismic weights of 373 tonnes for all floors and 325 tonnes for the roof. The building was assumed to be located in Nelson, New Zealand with a site hazard factor of Z=0.27 with Soil type D.

Friction-damped braces frames are designed as the lateral load resisting system. The demand in the braces are determined following a forced-based design approach with a ductility factor of μ=4.0. The building is designed as a normal building (IL2) with return period factors of R_s=0.25 and R_u=1.0 for SLS1 and ULS, respectively. Using the information mentioned, a design base shear of 3042 kN is found. Note that the structure should remain linear elastic for SLS1 limit state so the friction dampers should not be activated at this stage. Therefore, the minimum the slip load in the braces are found and then increased by 25% to achieve the optimum slip load recommended for friction-damped braced frames (Chandra et al. 2008; Pasquin et al. 2004). (see Table 2). Note that for each brace, the friction damper is attached to the brace body at one end.

A numerical model is developed in the SAP2000 package for the structure using the provided data. The general arrangement of this model is displayed in Figure 4. The friction-damped braces are modelled using a "multi-linear plastic" link element with "Kinetic" hysteresis type. The brace sections are specified using the capacity design principle (Priestley and Calvi 1991). For friction-damped braced frames, a design over-strength factor of 1.3 is suggested based on previous experience (Chandra et al. 2008).

4 RESILIENT TENSION-ONLY BRACES CONSIDERED FOR STRENGTHENING

The Resilient Slip Friction Joint (RSFJ) is introduced to the construction industry in 2015 (Zarnani and Quenneville 2015). This device is an innovative variation of conventional friction dampers where the sliding plates are profiled and form sliding "grooves". Stacks of conical disc springs in series are used to clamp and sandwich the sliding plates. Figure 5 depicts the arrangement of the RSFJ and the expected hysteresis. The hysteretic behaviour of the RSFJ can be calculated using Eqs. (1) and (2).

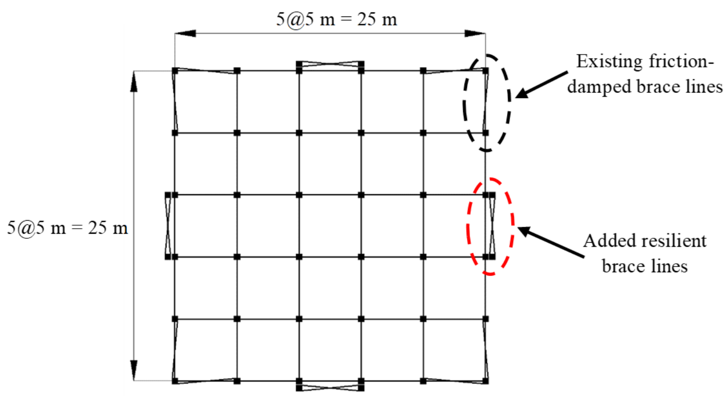

Figure 2: Case study structure

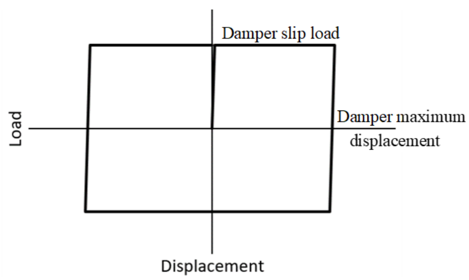

Figure 3: Hysteresis of the friction dampers

Table 2: Specifications of the conventional friction braces

Story	Brace section	Initial stiffness (kN/mm)	Slip load (kN)
5	200UC52.2	222	450
4	200UC52.2	222	800
3	250UC72.9	311	975
2	310UC118	478	1125
1	310UC118	478	1250

$$F_{slip} = 2n_b F_b \left(\frac{\sin\theta + \mu_s \cos\theta}{\cos\theta - \mu_s \sin\theta}\right) \tag{1}$$

$$F_{residual} = 2n_b F_b \left(\frac{\sin\theta - \mu_k \cos\theta}{\cos\theta + \mu_k \sin\theta}\right) \tag{2}$$

Where n_b is the number of bolts, F_b is the clamping force in each bolt (or rod), θ is the angle of the profiled grooves, μ_s is the coefficient of friction (static) and μ_k is the coefficient of friction (kinetic). $F_{ult,loading}$ and $F_{ult,unloading}$ can be respectively computed using Eq. (1) and Eq. (2) with μ_s substituted with μ_k and F_b substituted with F_u. Note that F_u is the ultimate capacity (flat load) of the disc springs. For this study, a tension-only braced frame with RSFJs is considered for seismic strengthening of friction-damped braced frames. Figure 6 shows a schematic view of this system. In this concept, RSFJ units are attached to tension-only members such as rods, cables or bars to form an x-shaped cross-bracing system effective in tension only (Bagheri Mehdi Abadi et al. 2019).

5 UPGRADING OF THE BRACED FRAME.

In this section, the procedure adopted for seismic upgrading of the structure presented in section 3, is described. The step-by-step procedure described below is followed to perform the upgrade:

Step 1: Specify the seismic performance objectives of the structure

The five seismic performance objectives detailed in section 2 are considered for this structure.

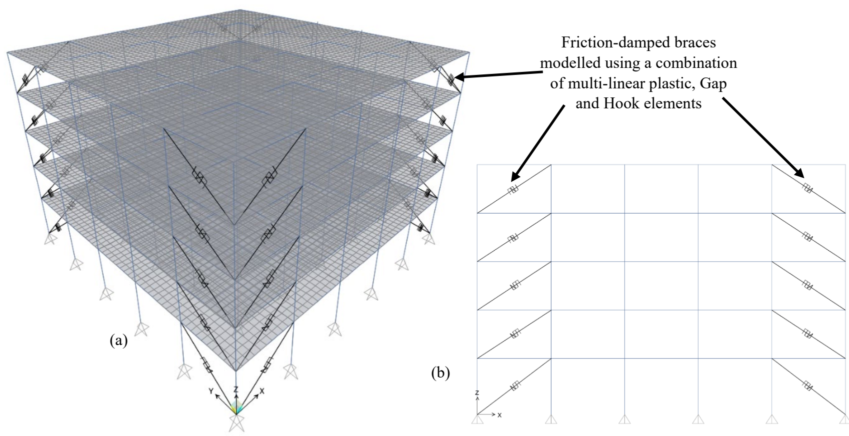

Figure 4: Numerical model for the case study structure: (a) three-dimensional view (b) a friction-damped braced frame

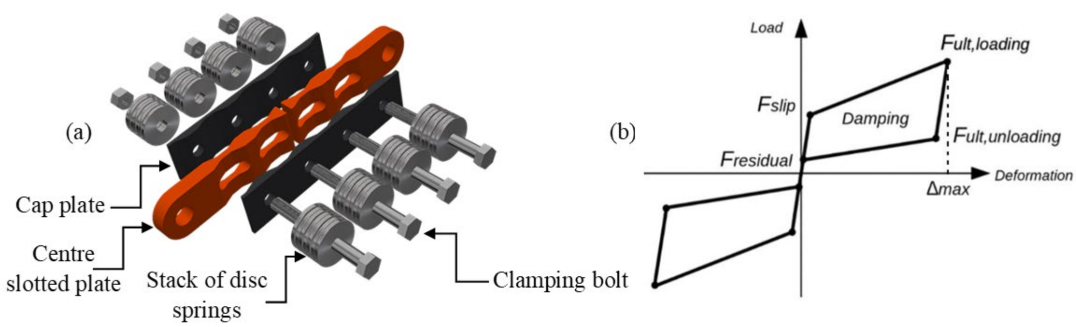

Figure 5: The Resilient Slip Friction Joint (RSFJ): (a) parts (b) load-deformation curve

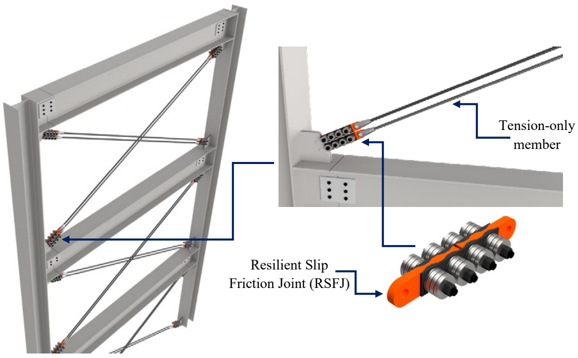

Figure 6: Tension-only braces with RSFJs

Step 2: Evaluate the seismic capacity of the designed structure
The designed friction-damped steel structure outlined in section 3 is considered for the seismic upgrading. The new brace lines are added to the perimeter of the building. It was assumed that new steel frames braced with RSFJ tension-only braces are added to the centre bays at the outer frames (see Figure 2). A displacement-control approach is taken to plot the pushover curve where the displacement at the top is monitored to reach the 2.5% drift for ULS (Figure 7). A bi-linear pushover performance with insignificant post-slip stiffness can be observed. The lateral drift ratio is about 0.24% which is less than determined for SLS1 level drifts (0.33%). The response base shear of the system at 2.5% drift is approximately 4500 kN which is higher than the design level calculated base shear (3042 kN).

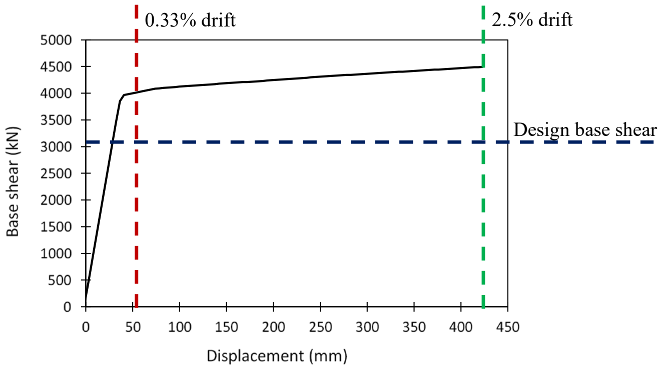

Figure 7: Pushover performance of the friction-damped braced structure

Step3: Determine the slip threshold of the RSFJs based on the performance criteria
The aim is to upgrade the structure to resist ground motions corresponding to a high importance building (IL4). Respecting the information provided in Table 1, the ULS demand on the designed structure corresponds to the SLS2 demands for the target structure. The added braced frames are designed in a way that the tension only RSFJs does not slip before the SLS2 level loads. A ductility factor of μ=1.25 is considered for computing the SLS2 actions. Accordingly, the SLS2 design base shear is determined as 9733 kN. Note that the return period factor used for the calculations was R_s=1.0 (see Table 1 and Table 3).

Figure 8 illustrates the numerical model following the addition of the RSFJ tension-only braced frames. The RSFJ braced frames are added to the central bays on the perimeter of the structure. Stiff springs are defined at the floor levels to transfer the loads from the diaphragms to the added frames. The RSFJ tension-only braces are modelled using "Damper – Friction spring" link elements with their active direction set to "tension".

Table 3: SLS2 lateral loads

Story	Lateral force (kN)	Story shears (kN)
5	3443	3443
4	2462	5905
3	1869	7774
2	1276	9050
1	683	9733

Step 4: Determine the post-slip stiffness of the RSFJ braces based on the adopted ductility and target drifts

The post-slip stiffness of the system, and consequently, the post-slip stiffness of the RSFJ braces can be determined based on the ULS design ductility factor. A design ductility factor of μ=1.75 (Oliver and Pettinga 2015) is used to determine the ULS base shear for the IL4 building. Then, the RSFJ force demands were specified in accordance with the computed base shear. Moreover, the maximum displacements of the RSFJs are specified based on the target ULS drift (1.5%) plus the over-strength mechanism which is discussed in Step 5 of the procedure. Table 4 summarises the design characteristics of the RSFJ tension-only braces. Threaded rods grade 12.9 are used as tension members. Table 5 shows the numerical inputs for the "Damper-Friction Spring" link representing the RSFJ tension-only braces. These inputs are calibrated based on the characteristics provided in Table 3.

Step 5: Determine the over-strength mechanism considered for the structure

For this structure, the characteristics of the RSFJs are defined in a manner that they could continue displacing (with the same loading stiffness) up to 1.5 times the ULS design displacement to cater for MCE level shakings. the ULS displacement and force demands in the brace are 5650 kN and 46 mm, respectively.

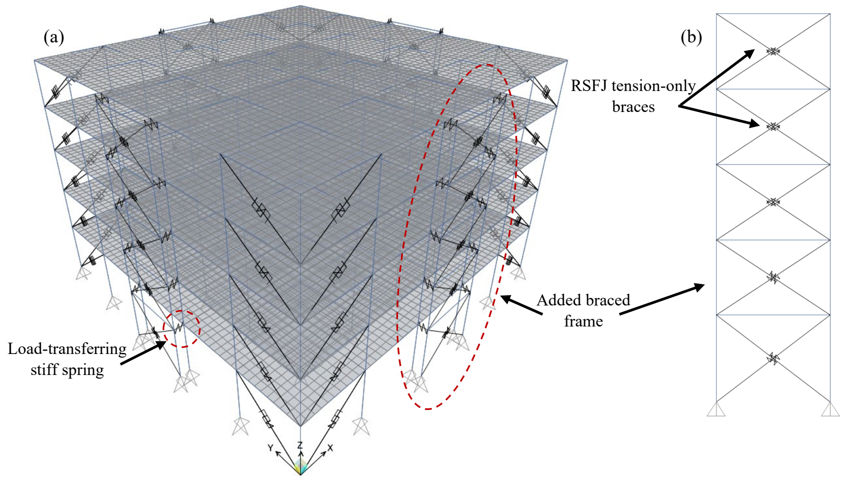

Figure 8: Numerical model after the addition of the RSFJ braced frames: (a) three-dimensional view (b) RSFJ tension-only braced frame

Table 4: Characteristics of the RSFJ braces

Story	Brace indicator	Rod specification	Initial stiffness (kN/mm)	F_{slip} (kN)	$F_{ult,loading}$ (kN)	$F_{ult,unloading}$ (kN)	$F_{residual}$ (kN)	Δ_{max} (mm)
5	TO-5	2M48	90	1260	1675	474	357	63
4	TO-4	2M62	151	2150	2850	796	600	63
3	TO-3	2M70	192	3100	4125	1333	1000	63
2	TO-2	3M70	289	3500	4675	1780	1332	63
1	TO-1	3M70	289	4200	5650	2070	1540	69

Table 5: Numerical inputs for the links

Story	Brace indicator	Initial stiffness (kN/mm)	Slipping Stiffness (Loading) (kN/mm)	Slipping Stiffness (Unoading) (kN/mm)	Pre-compression Displacement (mm)	Stop Displacement (mm)	Direction
5	TO-5	90	15.4	4.9	-82.93	63	Tension
4	TO-4	151	25.9	8.3	-82.93	63	Tension
3	TO-3	192	41.0	15.5	-75.61	63	Tension
2	TO-2	192	51.1	23.3	-68.5	63	Tension
1	TO-1	289	46.0	18.6	-91.2	69	Tension

If the devices continue displacing for 50% more displacement after ULS, the force and displacement demands in the brace increase to 6725 kN and 69 mm, respectively. The ratio of the over-strength action (6725 kN) over the ULS demand (5650 kN) is approximately 1.2.

Step 6: Perform nonlinear pushover analysis to evaluate the performance of the structure respecting the design performance objectives

Figure 9 displays the pushover performance of the structure after the RSFJ tension-only braces are added. It also indicates different limit states and the associated lateral drifts. It can be seen in the figure that for SLS1 level base shear, the structure is linear, elastic and relatively stiff. The lateral drift at this stage is 0.12%. Zone A is where the conventional friction-damped braces start to slip. Given that not all of them will reach their slip resistance at the same time, a curved transition zone is observable. For the SLS2 base shear demand, the lateral drift is 0.59% which is considerably lower than the maximum allowable value (1%). Zone B is where the RSFJ braces start to slip. Similar to Zone A, not all of the RSFJ braces start to slip at the same time. Therefore, a similar curved region is noticeable for Zone B.

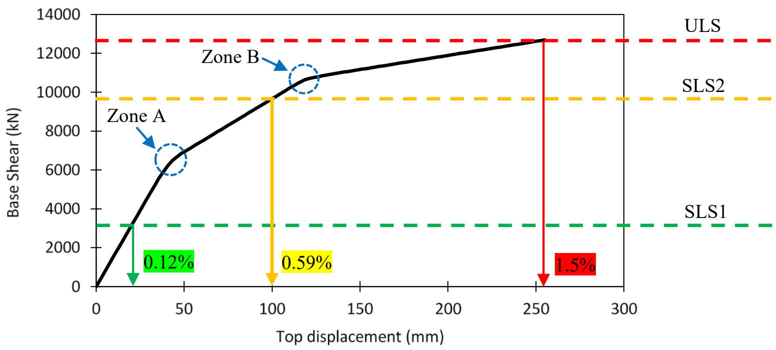

Figure 9: Pushover performance of the structure

6 TIME-HISTORY ANALYSES

dynamic time-history simulations are performed on the structure and the results are presented. In addition to the structure that was designed in the last section, another two numerical models were also developed where instead of RSFJ tension-only braces in the added frames, conventional friction dampers are used. The intention for creating the second and third models was to compare the seismic performance of three different concepts for the application in high importance structures. For the second model, the slip loads of the friction dampers are determined based on the SLS2 demands on the braces magnified by 1.25. In the third model, a backup MRF is designed for 25% of the demand is added to the peripheral friction-damped braces. Table 6 shows the slip loads and sections of the added braces for the FD-TC-FD and FD-TC-FD%25 models. Figure 10 illustrates the numerical assembly for the FD-TC-FD and FD-TC-FD%25 models.

Table 6: Properties of the added frame for FD-TC-FD and FD-TC-FD%25

Story	Considered brace section	Slip load (kN)
5	310UC118	1360
4	310UC137	2250
3	310UC158	3360
2	350UC197	3700
1	350WC230	4550

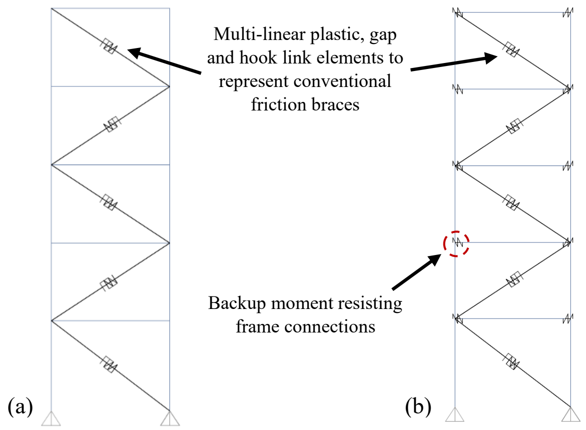

Figure 10: Peripheral upgrading frame for: (a) the FD-TC-FD (b) FD-TC-FD%25

The three developed models were subjected to nonlinear time-history analyses. For this study both mean and peak values from the NLTH results are presented and discussed. The records are scaled using the method outlined in (New Zealand Standards 2004) based on the assumed soil type and location. Table 7 shows the ground motions, and the scaling factors for the four considered limit states.

Table 7: Ground motions and scaling

Seimsic event	Year	Station	Epicentral distance (km)	PGA(g)	Sclaing factor ([27])			
					SLS1	SLS2	ULS	MCE
El Centro	1979	El Centro Array #9	5.35	0.30	0.31	1.21	2.10	2.60
Northridge	1994	Hollywood Storage	0.1	0.23	0.40	1.83	3.21	4.31
Kobe	1995	KJM	1.46	0.82	0.12	0.41	0.73	0.81
Landers	1992	Joshua Tree	19.7	0.28	0.33	1.20	2.15	2.62
San Fernando	1971	LA Hollywood Store	2.19	0.21	0.64	2.32	4.12	5.26
Chi Chi	1999	CHY074	0.57	0.23	0.41	1.64	2.94	3.60
Christchurch	2011	Cathedral	5.5	0.26	0.20	0.76	1.37	1.62
Loma Prieta	1989	Capitola	2.23	0.45	0.24	0.74	1.30	1.73

Figure 11 shows the recoded inter-story drifts for the three models. It can be seen that for all three models, the SLS1 level drifts are below the recommended value (0.33%).

For SLS2 events, the inter-story drift values recorded for all three models satisfied the defined limit (1%) considered as the second performance objective. The highest recorded value is related to the FD-TO-RSFJ model for the Chi Chi event that was 0.75% which is well below the 1% limit.

For ULS scaled events, the FD-TO-RSFJ model exhibited a better performance compared to the other two models. While the mean value for all analysed cases was under the target ULS drift (1.5%), the limit is slightly exceeded for one event (Chi Chi). For this event, the inter-story drift was 1.6% and 1.7% for the first and second floor, respectively. For the FD-TC-FD model, the ULS inter-story drift limit was exceeded for multiple events at different floor levels (Christchurch, Chi Chi and Northridge). For Christchurch and Chi Chi events, the value went up to 3% and 2.9%, respectively. For the FD-TC-FD%25 model, a better ULS response is observable compared to the FD-TC-FD model. As can be seen, the mean displacement profile shows that the structure was able to satisfy the ULS drift limits.

For MCE, the FD-TO-RSFJ model provided the best performance amongst the three modes. For this model, the mean displacement profile of the building was well under the target over-strength mechanism considered (2.25%). The inter-story drift at the two bottom stories was slightly higher than the considered over-strength drift capacity. However, it means that for those two events, the RSFJ secondary fuse function will be activated to cover for the slightly extra drift demands. For the FD-TC-FD model, the drift response data seem to be far more scattered. While the mean displacement profile satisfies the over-strength drift capacity, the limit is exceeded for four of the events at multiple story levels. Furthermore, the maximum values recorded are 3.4% and 3.5% for the two bottom stories subjected to the Chi Chi event. These values are significantly larger the assumed over-strength drift capacity (2.25%) for the design. For the FD-TC-FD%25 model, the drift response is slightly improved compared to the FD-TC-FD model.

It can be concluded that for the FD-TO-RSFJ model with RSFJ braces, the considered over-strength mechanism (1.5 times the design ULS displacements) was efficient in controlling the inter-story drifts at the ULS and MCE events. However, for the systems with conventional friction-damped braces, an over-strength drift capacity increased to a larger value in the range of at least two times of the ULS drift is recommended.

Figure 12 shows the observed residual displacement at the roof level. Furthermore, two residual deformation limits are indicated in the figure. The bottom dashed line represents the 0.3% drift which is indicated as the low damage threshold (Clifton et al. 2011). Moreover, McCormick et al. (McCormick et al. 2008) indicated that 0.5% is an index level for permissible residual displacement for which values beyond that could extensively affect the functionality of the building. For SLS2 level events, only minor residual displacements were observed for the FD-TC-FD and FD-TC-FD%25 models that were in the range of 0.14%.

The FD-TO-RSFJ model did not exhibit any significant residual displacement for the ULS level. The highest recorded values were related to the Chi Chi and Christchurch events with 0.07% and 0.08% drifts, respectively. This observed re-centring behaviour could be attributed to the internal restoring force in the RSFJ units.

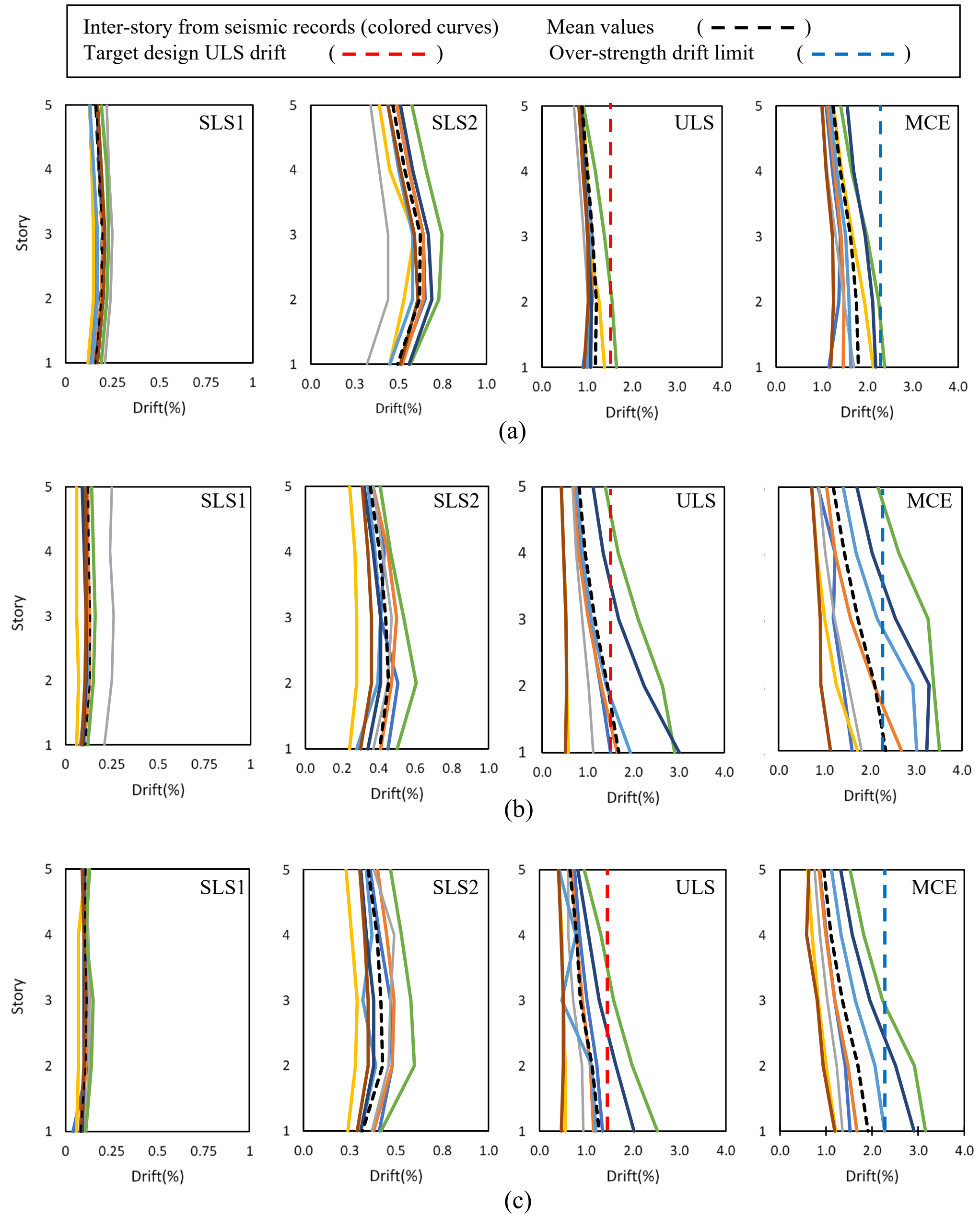

Figure 11: Inter-story drift responses for: (a) FD-TO-RSFJ (b) FD-TC-FD (b) FD-TC-FD%25

On the contrary, the FD-TC-FD model demonstrated significant residual displacement for the ULS level shakings. As can be seen in the figure, the index for low damage system (0.3%) is exceeded for five of the events and the maximum permissible limit (0.5%) is surpassed for three of the events. For the FD-TC-FD%25 model, the low damage residual drift limit and the maximum permissible limit are exceeded for three and one the analysed cases, respectively. This shows that the implementation of the backup MRF can potentially decrease the residual deformations for friction-damped braced frames but cannot eliminate them.

For MCE events, both FD-TC-FD and FD-TC-FD%25 models demonstrated considerable residual displacement values. The highest inter-story drifts for these two models were 1.32% and 0.81%, respectively.

For the FD-TO-RSFJ model, the largest documented residual drift for MCE events was 0.18% which was for the Christchurch event. The time-history results showed that the FD-TO-RSFJ system could outperform the other two systems in terms of minimising the residual displacements at large rare earthquakes. Therefore, the use of RSFJ braces is suggested as the preferred solution when a self-centring behaviour is one of the design performance objectives.

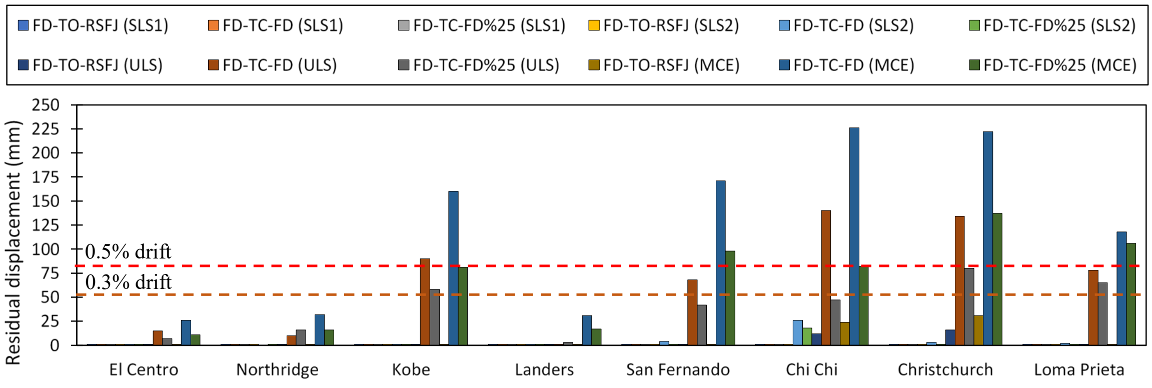

Figure 12: Residual Displacements

7 CONCLUSIONS

This paper proposed a solution for seismic integration of steel friction-damped braced frames with resilient tension only braces. In this concept, peripheral frames with tension only Resilient Slip Friction Joint (RSFJ) braces are added to the structure to increase its lateral resistance while controlling the responses drift within the designated range. From the results of the study, the followings concluding points can be drawn:

- The efficiency of the provided design procedure is confirmed, considering that the system was able to satisfy the design performance objectives defined for the structure.
- The designed structure with RSFJ braces was able to meet the drift limits defined for the four considered design limit states. These limit states were related to Serviceability Limit States (SLS1 and SLS2), Ultimate Limit State (ULS) and Maximum considered Event (MCE).
- The over-strength mechanism considered for the RSFJs is proven to be able to mitigate the drift demands related to the MCE level shakings.
- The systems with alternative solutions were not able to meet the defined drift limits on multiple occasion. Nevertheless, it has been shown that implementing a backup moment-resisting frame designed for 25% of the seismic demand was effective in reducing and controlling the drift demands. The over-strength drift capacity for conventional friction dampers is suggested to be at least twice the design drift demand to meet the MCE level demands.
- The internal restoring force of the RSFJ braces provided a re-centring capability for the structure. This characteristic was indicated as one of the design performance objectives. For alternative systems, significant levels of residual deformations were observed. Nevertheless, the friction-damped system with a backup frame demonstrated relatively lower residual drifts.
- It has been demonstrated that the proposed concept has the potential to be considered for the application in high importance structures.

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