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Enhanced seismic performance of buildings using tension-only Resilient Slip Friction Joint (RSFJ) braces

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ABSTRACT

The innovative Resilient Slip Friction Joint (RSFJ) has recently been introduced and used in the New Zealand construction industry. This technology not only aims to provide life safety, but also minimises the earthquake damage so that the building can be reoccupied immediately with minimal business disruption. The flag-shaped hysteresis of the RSFJs provides the required seismic performance including a self-centring behaviour.

One of the applications of this technology is the tension-only bracing system in which the diagonal cross braces are used with the RSFJs attached to one or both ends. Since the braces only work in tension, Reidbars, rods, rebars or any similar member that can resist tension can be used as the elastic elements resisting the tension forces. This system is well suited for new and existing buildings (specially with long spans) when compared with the conventional tension/compression braces.

This paper addresses the enhanced seismic performance of the structures using this system. A four-story case study steel building is designed using the conventional forced-based method and its seismic performance is evaluated by non-linear static pushover and nonlinear dynamic time-history simulations. The results showed that this system is able to provide a high level of structural ductility while providing a fully self-centring behaviour.

1 INTRODUCTION

Nowadays, there is an increased demand for seismic damage avoidance systems that can offer a high resistance level against the severe seismic events, allowing the buildings to be swiftly returned to service, with permissible or no residual displacement. Previous studies have shown the significance of the residual displacement on the post-functionality of the structures. Many researchers reported a residual drift ratio of 0.5% as the permissible limit that residual drifts more than that would jeopardize the post-event functionality of the structures (Erochko et al., 2010; McCormick et al., 2008).

The Resilient Slip Friction Joint (RSFJ) technology (Zarnani and Quenneville, 2017) is a recently developed damage avoidance technology that has already been implemented in two real projects in New Zealand (and currently is under study for more). This technology provides self-centring behaviour and seismic energy dissipation in one package. It also includes a built-in collapse prevention secondary fuse function that adds more resilience to the system in case of a seismic event larger than the design level. Hashemi et al. (2017) experimentally verified the flag-shaped hysteresis and the self-centring characteristic of the RSFJ.

Figure 1 shows the components and the assembly of the RSFJ. In this joint, the energy is dissipated by frictional sliding of the moving plates while the specific shape of the ridges combined with the use of disc springs provide the necessary self-centring behaviour. At the time of unloading, the restoring force induced by the elastically compacted disc springs is greater than the resisting frictional force between the sliding parts. Thus, the elastic force of the discs re-centres the middle plates to their original stationary position. Figure 1(c) shows the device at rest when the disc springs are partially compacted. When the force applied to the joint overcomes the resistance between the clamped plates, the middle plates start to move and the cap plates start to expand until the joint is at the maximum deflection and the discs are flat (see Figure 1(d)).

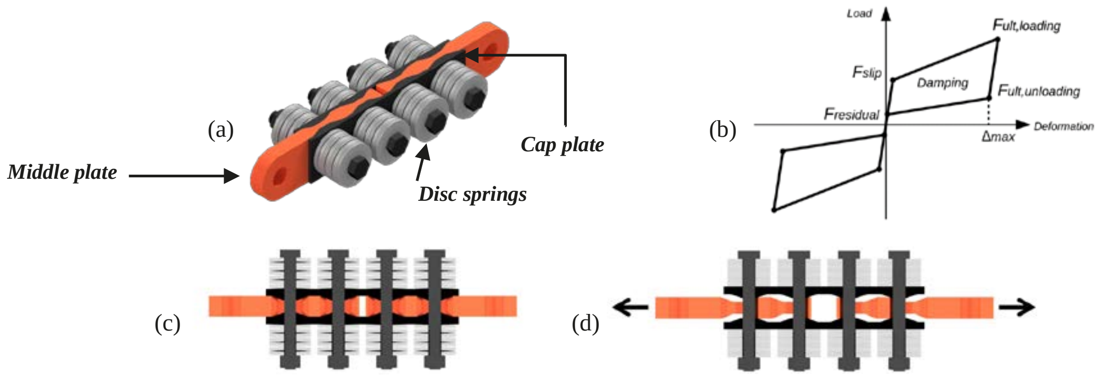

Figure 1: Resilient Slip Friction Joint (RSFJ): (a) assembly, (b) hysteresis, (c) the joint at rest, (d) the joint at the maximum deflection

Figure 1(b) displays the load-deformation behaviour for the RSFJ. The slip force (F_{slip}) and the residual force ($F_{residual}$) in the joint can respectively be determined by Equation (1) and Equation (2) where $F_{b,pr}$ is the clamping force in the bolts, n_b is the number of bolts, θ is the angle of the ridges, μ_s is the static coefficient of friction and μ_k is the kinetic coefficient of friction. The ultimate force in loading ($F_{ult,loading}$) and unloading ($F_{ult,unloading}$) can be calculated by substituting μ_s and $F_{b,pr}$ in Equation (1) and Equation (2) with μ_k and $F_{b,u}$, respectively. It should be noted that the initial stiffness of the RSFJ (the stiffness before F_{slip} in Figure 1(b)) is related to elastic stiffness of the sliding plates and of any other component connected to the RSFJ. The reader is referred to (Hashemi, 2017) for more information about the full-scale experimental tests that have been conducted on different applications of the RSFJs, including the test results and discussions.

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

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

2 DYNAMIC PERFORMANCE OF THE RSFJ

The performance of the RSFJ has previously been verified by quasi-static tests on the component level. Nevertheless, there has been little discussion about the behaviour of the joint under dynamic loading. Accordingly, a dynamic component testing was conducted to investigate the performance of the RSFJ subjected to rapid load schedule. Figure 2 shows the test specimen and the test setup. The testing is done using the dynamic 100 kN Instron machine in the Transport Lab at the University of Auckland.

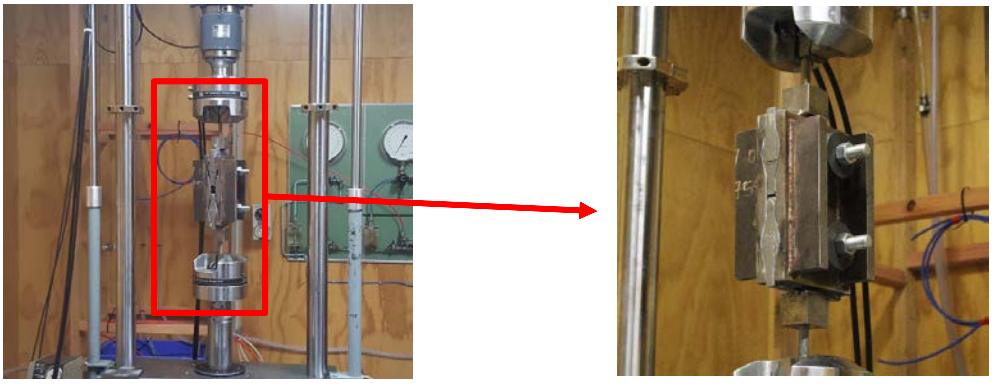

Figure 2: Dynamic component testing of the RSFJ

The RSFJ was designed to achieve 30 mm as the maximum displacement at an axial force of 90 kN. Figure 3 shows the experimental results. As can be seen, the specimen was subjected to three different loading rates. In the first test, 24 full displacement-control cycles (30 mm as the ultimate displacement) was applied with a frequency of 0.1 Hz to represent the quasi-static loading. It can be seen in Figure 3(a) that the load-slip curves of the RSFJ are repetitive and stable.

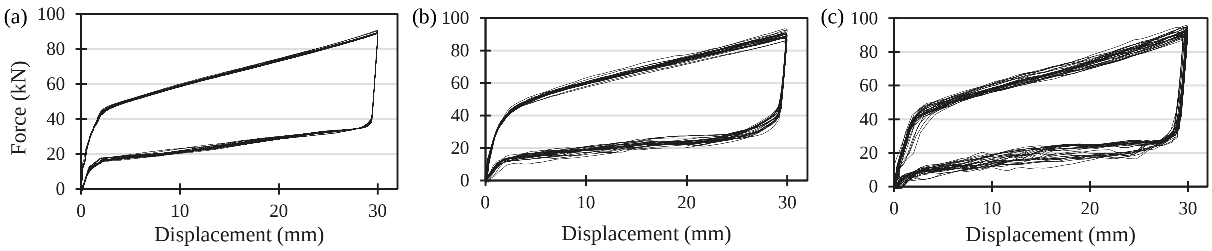

Figure 3: Results of the dynamic component testing of the RSFJ: (a) quasi-static loading (b) loading frequency of 1 Hz (c) loading frequency of 2 Hz

In the second test, the frequency of the loading was increased to 1 Hz and as can be seen in the response graphs (Figure 3(b)), the ultimate load in the RSFJ slightly increased to 94 kN (corresponding to a 4% increase). In the third test, the loading rate was further increased to 2 Hz and the peak force in the joint reached a maximum of 96.6 kN (corresponding to a 7% increase). These slight increases in the peak force ($F_{ult,loading}$) may be related to the change in the coefficient of friction between the sliding plates due to the higher sliding speed. Note that the applied load protocol was more demanding when compared with the load protocols recommended by the standards (AISC 341-10, 2010).

It can be seen that this rise in the ultimate forces has not affected the performance of the RSFJ (the behaviour is stable and self-centring is achieved). Nevertheless, the over-strength factors normally considered for the capacity design of structures are sufficient to cover this rise in $F_{ult,loading}$. For example, an over-strength factor of 1.25 is normally considered for the RSFJ because of the performance of the secondary collapse-prevention fuse (Hashemi et al., 2018).

3 THE RSFJ TENSION-ONLY BRACE

Figure 4 schematically shows the RSFJ tension-only concept. In this concept, the RSFJ device is in series with the diagonal tension members forming an x-braced system effective in tension only so there will be no global buckling in the system. Rebars, threaded rods, or any other type of tension-resistant element can be considered for the diagonal members resulting in an economical damage avoidance lateral load resisting system. Full-scale experimental tests by Bagheri et al. (2019) have indicated that the hysteretic performance of this system is similar to a system with tension/compression braces that have equal strength and stiffness in both directions of loading in the plane of the frame. The additional advantage of this system is that there no need for buckling-specific consideration the design of end connections and gusset plates resulting in a more economical design. This extra advantage is even more important for long spans where buckling of the braces usually governs the design.

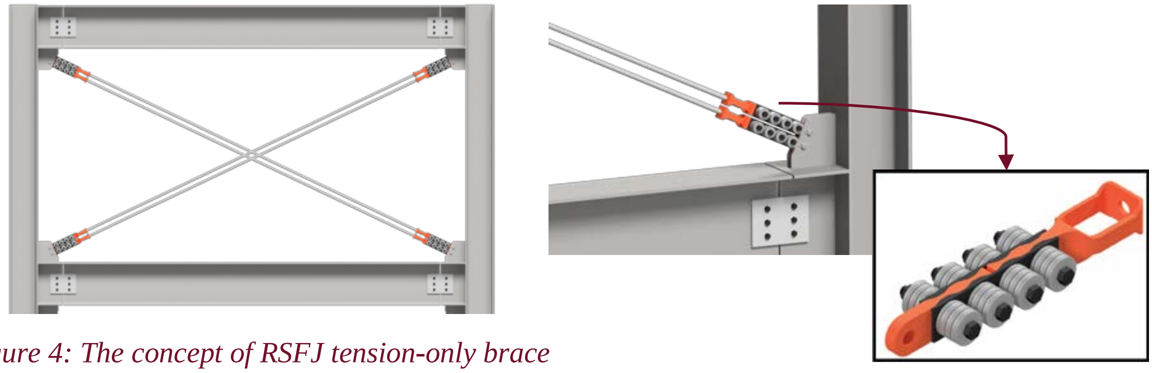

Figure 4: The concept of RSFJ tension-only brace

4 THE DESIGN OF THE CASE STUDY BUILDING

A multi-storey case study steel building with RSFJ tension-only braces as its primary seismic resisting system is considered in this section in order to evaluate the seismic performance of this system and the adopted design philosophy. The general configuration of this structure is shown in Figure 5. The philosophy of design for this structure is to satisfy the following performance criterions in addition to the requirements of (NZS1170.5, 2004) and (NZS3404, 2007): (a) the Ultimate Limit State (ULS) lateral drifts are kept under 1.4% to minimize the damage to structural and non-structural components (b) the structure remains elastic, the dampers are not activated and the lateral drift ratios are kept under 0.35% for the Serviceability Limit State (SLS) (c) the structure has zero or negligible residual displacement at the end of the seismic event.

The considered four-storey building is 13.2 m tall (with identical storey heights of 3.3 m) and is symmetrical about the two main axes. Along each axis, two braced frames equipped with RSFJ tension-only braces are used as the Lateral Load Resisting Systems (LLRSs). Note that four LLRSs are considered in each direction to satisfy the proposed changes to NZS1170.5 for redundancy that are being implemented as a Canterbury Earthquakes Royal Commission (CERC) recommendation (CERC recommends that there should be at least two LLRSs in each main direction). Considering the symmetrical configuration of the braced bays, it was assumed that each braced frame resists 50% of the lateral seismic loads, ignoring the accidental eccentricity for this design example but needs be considered in practice. The structure is composed of steel frames with composite floor slabs. It was assumed that this building is located at Christchurch with a Class C soil and a hazard factor of Z=0.3. The building is an office-type thus it has an importance level of 2 and is designed for a working life of 50 years.

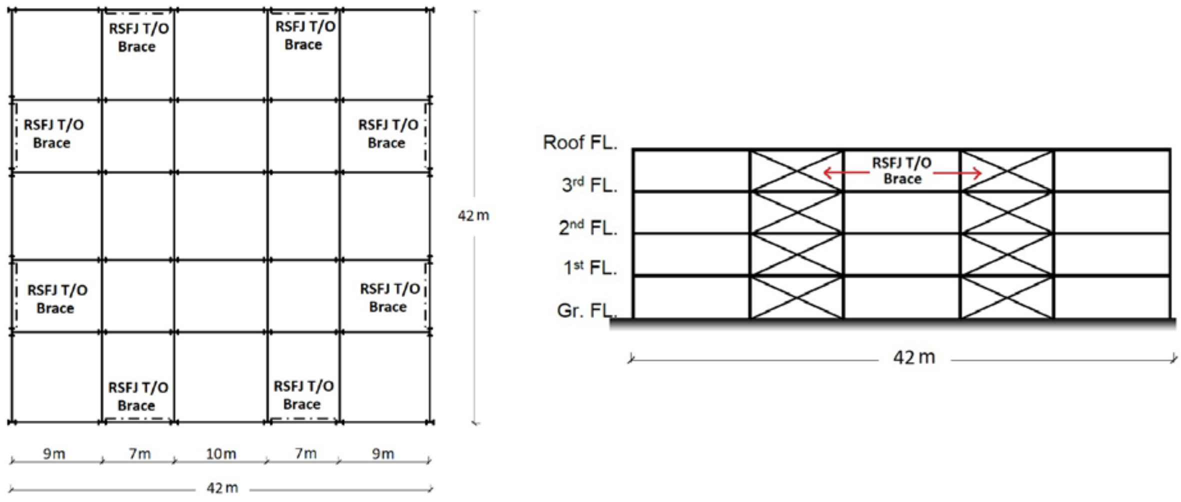

Figure 5: General arrangement of the case study building

The loads applied during the preliminary design are: A self-weight (for the frame) of 0.6 kPa, a super-imposed dead load of 0.5 kPa, a floor weight of 3 kPa, a cladding weight of 1 kPa, a floor live load of 3 kPa and a roof live load of 0.25 kPa. The calculated seismic weights for the structure are 1058 tonnes and 913 tonnes for floor one to four and the roof, respectively.

A ductility factor of μ=4.0 and a structural performance factor of S_p=0.7 are adopted for the design. The value of S_p is linked to the design ductility/level of detailing used and the value of 0.7 is appropriate for structures deigned in accordance with (NZS3404, 2007). However, for other structural systems that uses damage avoidance technologies, this value might be un-conservative and may need to be re-evaluated. The period of the structure is determined as T_1=0.43 seconds using the empirical equation provided in (NZS1170.5, 2004):

$$T = 1.25 * k_t * h_n^{0.75} \tag{3}$$

Accordingly, the spectral shape factor $C_h(T_1)$ =1.87, the hazard factor Z=0.3, the near fault factor N=1.0 and the return period factor of R_u=1.0 (for 1/500 annual probability of exceedance) are determined. The inelastic spectrum scaling factor k_μ=2.85 is calculated using the following equation (NZS1107.5, 2004) for fundamental periods less than 0.7 seconds:

$$k_\mu = \frac{(\mu-1)T_1}{0.7}+1 \tag{4}$$

Note that for periods equal or more than 0.7 seconds, $k_\mu=\mu$. The ULS base shear of the structure is specified as V_b = 7079 kN. This base shear is distributed (see table 1) in the structure respecting the storey weights and heights as per the recommendations in (NZS1170.5, 2004). Table 1 also shows the axial force demands in the RSFJs. Note that the factor Cs is taken as 1.0 for the preliminary design of the braces following the discussions provided in (Hashemi et al., 2019) for RSFJ tension-only systems.

The RSFJ has a built-in collapse-prevention secondary fuse which will be activated when the applied load to the brace is larger than the design load. The experimental studies of (Hashemi et al., 2018) demonstrated that the device can provide 50% displacement more than the design displacement, while the force in the device will increase by a factor of 1.25. Therefore, an over-strength factor of 1.25 on the force associated with full expansion of the RSFJs as is considered to design the braces. The same over-strength factor is considered to design the collector beams. For the columns, an over-strength factor of 1.5 is considered to develop the desired 'weak beam-strong column' mechanism. For this structure, non-threaded rods of grade 830 MPa were considered and designed for the braces. Note that any other type of tension-resisting member could be used. The diameters of the rods are therefore specified as 45 mm, 55 mm, 65 mm and 75 mm for level 1 to roof, respectively.

Table 1: Seismic load calculations summary

Level	Height (m)	Storey lateral force (kN)	Storey shear (kN)	Brace forces (kN)
Roof	13.2	2944	2944	818
3	9.9	2068	5011	1392
2	6.6	1378	6390	1775
1	3.3	689	7079	1966
Sum		7079		

A structural analysis using the SAP2000 program is performed in this step to evaluate the performance of the structure and to check the seismic lateral drifts. A 2D model of one of the braced frames was considered for modelling. The beam to column connections and the base connections are considered as pinned joints while the column sections are continuous as required by NZS 3404. The effect of the rotational stiffness of the columns bases has a minor effect on the performance of the braces in reality so it was not considered in the model for simplicity. A rigid diaphragm has been assigned to each floor to constrain the horizontal displacements of the beams. The gravity loads have been applied to the beams and the seismic weights have been assigned to the nodes in each elevation.

The RSFJs are modelled using the 'Damper – Friction Spring' link element with the 'tension-only' feature activated (Hashemi et al., 2017b). The devices are accordingly designed and calibrated (see Table 2). The F_{slip} was assumed as 75% of $F_{ult,loading}$ for all RSFJs. This ratio is determined to ensure that the structure remains elastic and RSFJs are not expanding at the SLS level loads (to satisfy the second performance target mentioned at the beginning of this section). The initial stiffness of the links is considered as the elastic stiffness of the diagonal members.

Table 2: RSFJ specifications

Level	F_{slip} (kN)	$F_{ult,loading}$ (kN)	$F_{ult,unloading}$ (kN)	$F_{residual}$ (kN)	Δ_{max} (mm)
Roof	13.2	2944	2944	818	42
3	9.9	2068	5011	1392	42
2	6.6	1378	6390	1775	42
1	3.3	689	7079	1966	42
Sum		7079			

Figure 6 shows the general arrangement of the modelled frame including the assigned sections.

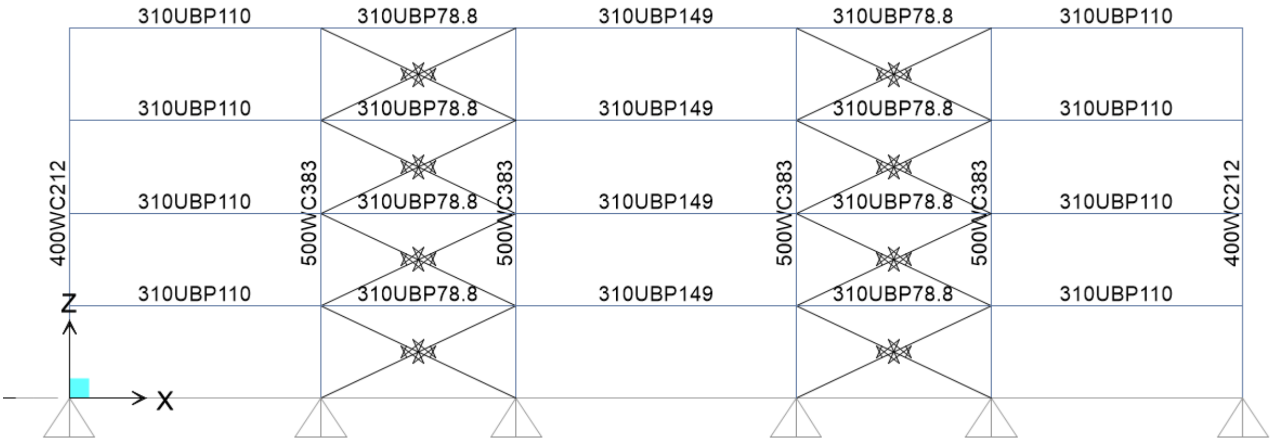

Figure 6: Numerical model of the frame and the sections assigned

From the non-linear static pushover analysis, it was found the over strength-factor for designing the diagonal tension members needs to be increased from 1.25 to 1.75 to satisfy the SLS requirements. Therefore, the new diameters for rods are specified as 50 mm, 65 mm, 75 mm and 80 mm for level 1 to roof, respectively.

Figure 5 shows the results of the cyclic pushover analysis after this optimization. As can be seen, the lateral drift of the structure is effectively limited to 1.4% at the ULS level loads 0.35% at the SLS level loads. The fundamental period of the structure from the modal analysis was very close to what was calculated (T_1=0.43 seconds) using the empirical equation provided in (NZS1170.5, 2004)

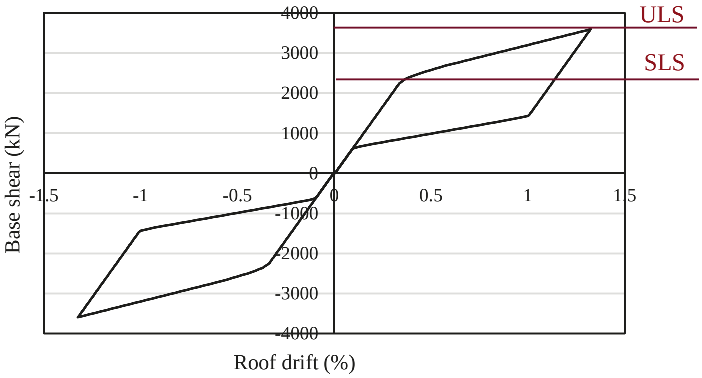

Figure 6: Results of the cyclic pushover analysis

Note that from the numerical model, it was found that the lateral displacement profile of the structure is very close to a linear one. The stability coefficients (θ) for all storeys (determined using the equation provided in clause 6.5.2(c) of (NZS1170.5, 2004)) were under 0.1 therefore there is no need to include the P-delta effects in the calculations.

5 NONLINEAR DYNAMICTIME-HISTORY SIMULATIONS

Non-linear dynamic time-history analyses are performed on the modelled frame to investigate the seismic response of the system and verify that the performance targets are achieved. Seven seismic events are selected for the simulations. The record are scaled for ULS based on the method described in (NZS1170.5, 2004) for the given location and soil type. Figure 7(a) shows the maximum recorded base shears for SLS and ULS. According to (Bradley, 2014), the ‘peak of three’ or the ‘average of seven’ records may be considered when designing using time-history analysis. It can be seen that the average base shear of the seven simulations is respectively 1365 kN and 2635 kN for SLS and ULS while the base shear determined from the ESM is 2526 kN and 3539 kN for SLS and ULS, respectively. The highest recorded base shear for SLS is related to the Kobe event (2397 kN) which is still under the base shear calculated using the ESM. For ULS events, the highest base shear is for the Christchurch event which is 3296 kN. Figure 7(b) illustrates the peak roof drifts for SLS and ULS events. As can be seen, the average peak roof drift is respectively 0.24% and 0.83% for SLS and ULS that shows the performance targets of the structure are satisfied. The highest recorded roof drifts for SLS and ULS is 0.39% (Kobe) and 1.29% (Christchurch), respectively.

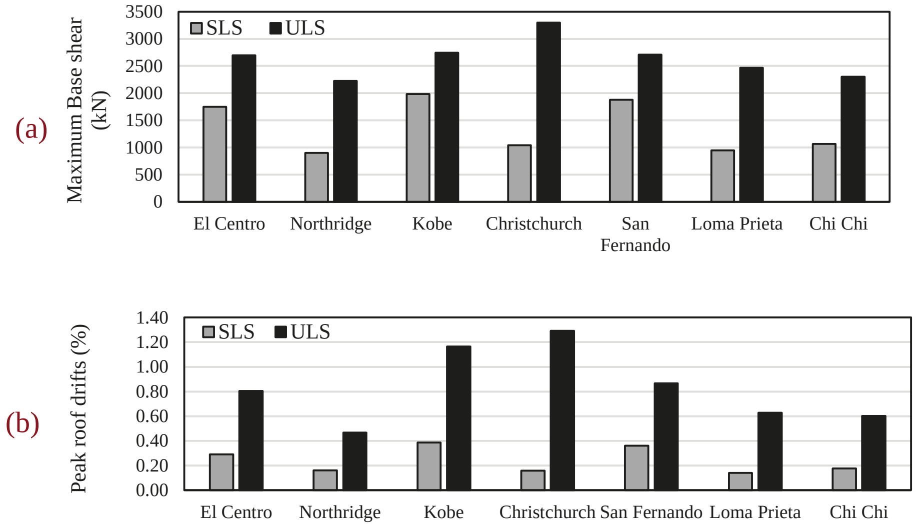

Figure 7: Results of the nonlinear dynamic time-history analyses: (a) base shears (b) peak roof drifts

Figure 8 shows the roof displacement histories for four of the simulations as representatives for all analysed cases. It can be seen that owing to the flag-shape response of the braces (and the structure), the structure returned to the original pre-earthquake position with no residual displacement at the end of all events. This shows a fully self-centring behaviour.

Overall, the results of the numerical simulations confirmed that the structure is properly designed and was able to satisfy the performance targets indicated for the design.

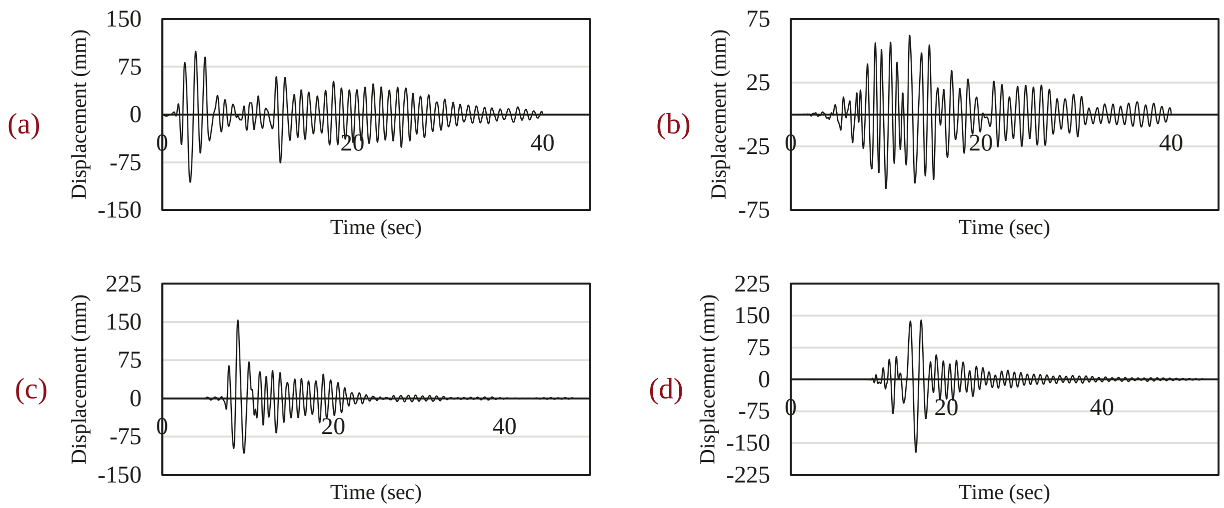

Figure 8: Roof displacement histories: (a) El Centro (b) Northridge (c) Kobe (d) Christchurch

6 CONCLUSIONS

This paper addresses the enhanced seismic performance of structures using the tension-only Resilient Slip Friction Joint (RSFJ) braces. In this system, RSFJ damping devices are attached to one or both ends of the tension-resisting diagonal braces. This concept whose performance has experimentally been verified, has a load-slip behaviour similar to a low damage tension/compression brace with additional advantages such as self-centring behaviour and easy installation.

To demonstrate the performance of this novel concept on the system level, a four-story case study steel building is designed and analysed using the conventional forced-based method. The design philosophy used was to limit the inter-storey drift ratios to 0.35% and 1.4% for serviceability limit state and ultimate limit state, respectively. The results of the nonlinear dynamic time-history simulations showed that the structure is well designed and was able the meet the target performance criteria. Moreover, a fully self-centring behaviour was observed for all analysed cases.

Overall, the findings of this research confirmed the good seismic performance of the structure with the novel lateral load resisting system. This concept can be an efficient solution when post-functionality criteria is considered for designing of the buildings.

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