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Proposed design procedure for damage-avoidance braced frames with braces effective in tension only

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ABSTRACT

While conventional seismic resistant structural systems, such as ductile moment resisting frames (MRFs), eccentrically braced frames (EBFs) and concentrically-braced frames (CBFs), may provide sufficient safety for the occupants, generally they depend on damage to selected structural members to resist severe earthquakes. Therefore, more than likely, after such an event there may be considerable economic loss and significant repair followed by business downtime. In addition, the post-event residual displacement of the building can also play a significant role in the post-disaster functionality of the structure. Previous studies have shown that a residual displacement more than 0.5% means the building should be demolished and replaced.

Recent severe earthquakes of 2010/2011 and 2016 have highlighted the problems with assessment and repair or replacement of buildings designed for controlled damage and have provided a strong motivation for engineers to start developing damage avoidance self-centring systems. One of the recent developments is the Resilient Slip Friction Joint (RSFJ), a friction-based energy dissipating device that provides damping and self-centring behaviour in one package with a flag-shaped load-deformation behaviour. One of the applications of this technology is the RSFJ tension-only brace. Although this technology has already been used in the New Zealand construction industry, the lack of a reliable design procedure may be a barrier for the adopters of the technology.

This paper proposes a step-by-step design procedure for steel braced frames with diagonal tension-only RSFJ braces that is in compliance with New Zealand Standards and relative provisions. The proposed step-by-step procedure is used to design a 5-story steel structure with RSFJ tension-only x-

braces to verify its efficiency and to demonstrate that the system can meet the ductility demand recommended by the Standard. The findings indicate that this procedure can also be used for other low damage bracing systems that have a flag-shaped hysteresis.

1 INTRODUCTION

Following the Canterbury earthquake sequence (2010 to 2012), it was observed that the steel structures have performed very well, considering the severity of the seismic events, most particularly structures with seismic resisting systems such as Eccentrically Braced Frames (EBFs) or Concentrically Braced Frames (CBFs) (Bruneau & MacRae, 2017). However, the structures were designed for the 'life-safety' criteria so post-disaster repair costs (if the structure is repairable) and the associate business downtime have significantly affected the economy of the recovering city. Moreover, the previous studies have demonstrated that residual drifts more than 0.3% can impact on the structural functionality and more than 0.5% require realignment which is difficult and would probably result in building replacement. Even residual drifts of 0.15% will require realignment of lift shaft guide rails, involving significant cost and disruption (Bruneau et al., 2011).

With the growing acknowledgement of the post-event economic impacts on society has come the increased demand for damage avoidance systems that can deliver a high resistance level against the severe earthquakes, allowing buildings to be rapidly returned to service, with negligible or no residual displacement and either requiring no maintenance or maintenance which can be delayed and undertaken at a time to suit the client.

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 (and 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)).

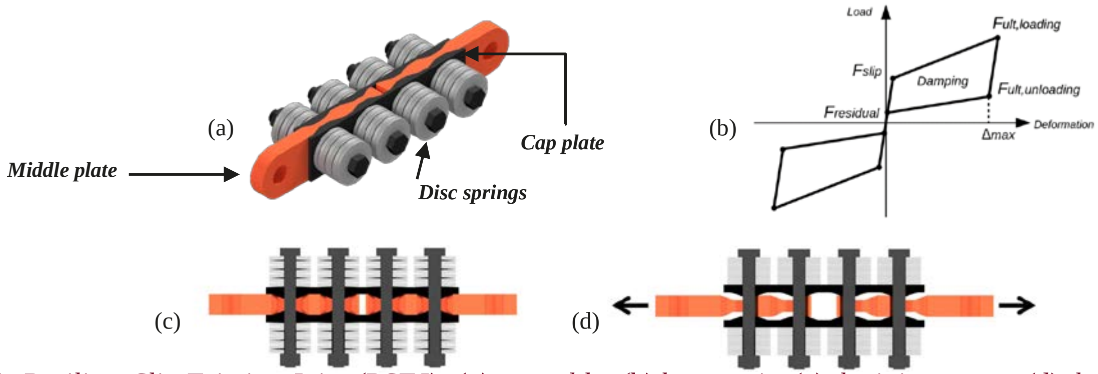

Figure 1: Resilient Slip Friction Joint (RSFJ): (a) assembly, (b) hysteresis, (c) the joint at rest, (d) the joint at the maximum deflection

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.

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

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

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.

2 THE RSFJ TENSION-ONLY BRACE

Figure 2 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-only 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.

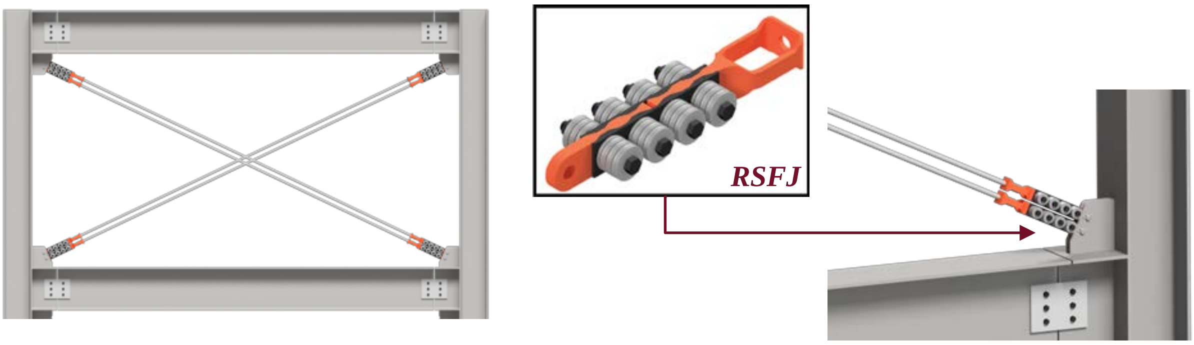

Figure 2: The RSFJ tension-only brace concept

3 THE PROPOSED DESIGN PROCEDURE

In this section, a design procedure is proposed and then implemented for the design of a multi-storey case study steel building with RSFJ tension-only braces as its primary seismic resisting system. The proposed design procedure is based on the New Zealand standard for steel structures (NZS3404, 2007) and the procedure contained in the HERA Report R4-76 (Feeney & Clifton, 2001) for CBFs with modifications to account for the characteristics of the RSFJ tension-only brace. This report forms the basis of current steel seismic design practice in New Zealand meaning that designers are familiar with the procedures involved. It has also required no modifications to the steel specific design procedures following the Canterbury earthquake series. The only recommended change has been to make the column splice provisions of NZS 3404 Clause 12.9.2.2 applicable to all columns in the structure, not just the seismic resisting system ones, to ensure that the gravity system columns are continuous over the height of the structure. This is to meet the additional focus on structural system ductility recommended by the Canterbury Earthquakes Royal Commission (CERC).

The design force for all structural members in this example is determined using the Equivalent Static Method (ESM) described in (NZS1170.5, 2004) which is a Forced-Based Design (FBD) method. The Modal Response Spectrum Analysis Method (RSA) would have been equally applicable.

Section 3.1 describes the general arrangement of the example structure that the proposed procedure is applied to. The first part of Section 3.2 describes the preliminary design procedure and second part represents the design procedure used to specify the member sizes.

3.1 The example case study structure

The general configuration of the considered case study structure is shown in Figure 3. The plan layout of this structure is similar to the one used by (Wijanto, 2012) for investigating the design requirements of structural systems with Buckling Restrained Braces (BRBs). The five-storey building is 18 m tall and is symmetrical about the two main axes. Along each axis, RSFJ tension-only braces are used as the Lateral Load Resisting Systems (LLRSs). The brace arrangement is symmetrical to fulfil the condition required by clause 12.12.2.2 of (NZS3404, 2007), where the difference in the braces forces should not exceed 20%. Respecting the symmetrical configuration of the braced bays, it can be concluded that each braced frame resists half of the lateral seismic loads, ignoring accidental eccentricity which is appropriate for this design example but must 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 D soil type and a Hazard factor of Z=0.3. The building is an office-type building thus it has an importance level of 2 and is designed for a working life of 50 years. This building has only two seismic resisting systems in each direction and so won't meet the proposed changes to NZS1170.5 for redundancy that are being implemented as a CERC recommendation, however this doesn't detract from its usefulness as a design example. Keeping this layout and making the gravity columns continuous would meet the intent of these provisions rather than adding a third seismic resisting system in each direction.

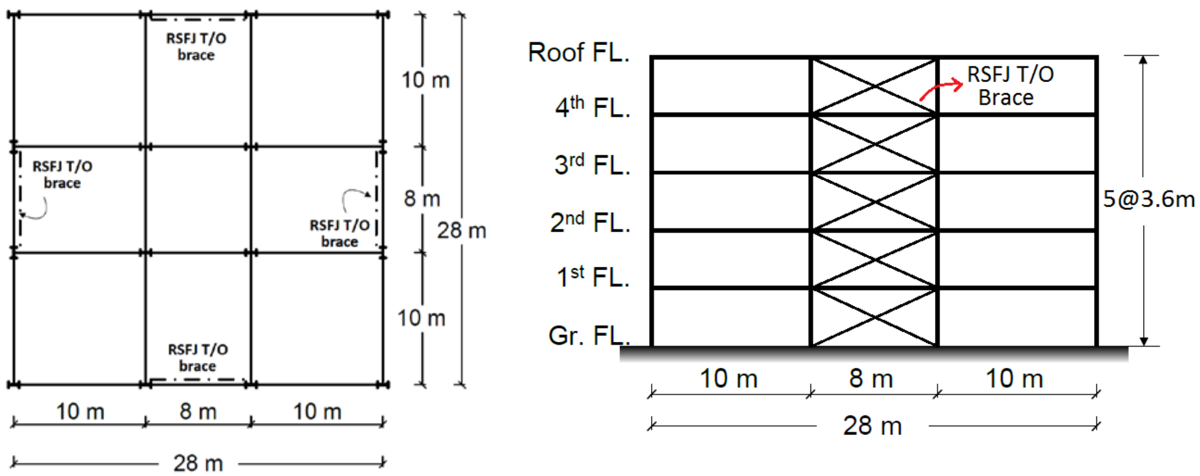

Figure 3: General arrangement of the case study structure

3.2 The design procedure

The proposed design procedure includes two parts. The first part presents the preliminary design of the structure, while the second part focuses on the design of the members. The RSFJ tension-only brace system can be classified as a fully ductile (category 1) or limited ductile system (category 2) depending on the adopted ductility factor and detailing of the structure. The design procedure here will follow the methodology proposed in (Feeney & Clifton, 2001) for designing category 1 and category 2 x-braced CBFs with braces effective in tension only, with modifications to account for the non-yielding, actively self-centring nature of the RSFJ system.

3.2.1 Preliminary design

Step 1: Check that the maximum height limitation is satisfied

The first step to design a braced frame is to check whether a maximum height limitation is required and, if so, whether it is satisfied. According to Clause 12.12.6.5.1 in (NZS3404, 2007), the tension-braced CBF systems are permitted only in x-braced systems up to two storeys. However, the experimental test of Bagheri et al. (2019) on full-scale RSFJ tension-only braced frames showed that the performance of this system (load-deformation behaviour) in terms of retention of strength and stiffness under reversing loading is as good as that of a system with braces effective in tension and compression and with a brace slenderness ratio ≤ 30. Thus for a category 2 system (similar to the design example here) a height limit of 12 storeys applies from NZS 3404 Table 12.12.4(2). This is similar to the recommendations of Wijanto (2012) in adapting the CBF procedure to design of a CBF system with buckling restrained braces. However, the RSFJ tension only system has the additional advantage of active self-centring.

Step 2: Determine the design seismic load incorporating the factor C_S

Clause 12.12.3. of NZS 3404 requires the application of a C_s factor to account for the less desirable inelastic response of capacity designed CBF systems in general compared with that of capacity designed MRF or EBF systems. However, the C_s factors in this clause are developed for conventional braced systems, using the procedure given in the commentary clause C12.12.3. To determine the appropriate C_s factor for the RSFJ tension only system, the commentary procedure needs to be applied to this system. This results in the following:

1. The variable **A** accounts for the deterioration in inelastic performance of CBF with increasing brace slenderness. The expression for the variable is shown by Equation (3) where "α'_c" is the post-buckling compression capacity of the brace.

For the case of the RSFJ tension-only brace, because there is no buckling in the braces and the capacity of the braces is only related to the devices (which are deigned based on the demand), the α'_c can be taken as 1.0. Therefore, the variable **A** will be taken as 1.0 when designing the RSFJ tension-only braces.

$$A = 1/[0.5\left(1+\alpha_c^{/}\right)] \quad (3)$$

2. The variable **B** accounts for the departure of the CBF system from the optimum weak beam-strong column (overall) mechanism towards the less desirable strong beam-weak column (shear) mechanism. This can be due to less than ideal hysteretic behaviour of the CBF system (Feeney & Clifton, 1995). In NZS3404, this variable is storey-dependent and is expressed as a function of the height of the building compared with the maximum height limit from Table 12.12.4. It ranges from 1.1 for structures under 1/3 of this height limit to 1.3 for structures over 2/3 of the limit. On that basis, the value of for the design example here would be 1.1. However, it can be argued that a well-designed and detailed RSFJ tension-only brace (that has been experimentally proven to have a stable hysteretic) with capacity designed beams and columns will develop a reliable overall mechanism comparable to an eccentrically braced frame that in fact does not require this magnification. Thus, for this case, the variable **B** is taken as 1.0.

3. The variable **C** accounts for the influence of inelastic demand on the system by modifying values of products taken from variables A and B to specify the C_s factor. For category 1 systems, **C** =**A**.**B** which for the case of RSFJ tension-only brace, C_s=1.0.

Step 3: Analyse the frame for the required load cases and load combinations

In this step, the frame is analysed considering different load cases and combinations. The loads applied during the preliminary design are: a self-weight (for the frame) of 0.8 kPa, a super-imposed dead load of 0.5 kPa, a floor weight of 3 kPa, a cladding weight of 0.8 kPa, a floor live load of 3 kPa and a roof live load of 0.25 kPa. The seismic weight applicable to each braced frame is accordingly calculated as 11437 kN.

Note that, in order to avoid the design actions on the secondary elements of the seismic resisting system (i.e. those elements suppressed from inelastic demand through the capacity design process; in this case the collector beams and the columns), the design procedure includes calculation of upper limit actions for member and connection strength, stipulated in Clause 12.9.1.3 of NZS 3404 and denoted by E_{max}. In principle, these actions are just as applicable to this tension only RSFJ system as they are to every other capacity designed seismic resisting system. However, given that the strength of the RSFJ can be designed to accurately match the seismic demand on each level, and to a much closer extent than is possible in a conventional CBF system, it is very unlikely that the upper limit actions will be less than the capacity design derived design actions. It means they are unlikely to be needed and it is reasonable to ignore them.

The ESM method or RSA method is used (NZS1170.5, 2004) to calculated the base shear of the structure; in the design example presented herein the ESM method is used. For preliminary design, the fundamental period of the structure is calculated using the empirical formula given in the NZS1170.5 commentary (and is equal to 0.54 seconds). A ductility factor of μ=3.0 and a structural performance factor of S_p=0.7 are adopted for the design. Note that the ductility selected here could be conservative and it is recommended to verify this factor by nonlinear dynamic time-history simulations later to efficiently size the RSFJs and the rest of the structure, although for a typical design that would be much more design effort than is routinely used or needed. The value of S_p is linked to the design ductility/level of detailing used and the value of 0.7 is appropriate. This is effectively a building system over-strength. There is no reason to use a higher numerical value of S_p at least for the design of steel structures covered by NZS3404; field studies of building responses in Christchurch to the Canterbury earthquakes show that this value is numerically higher than the actual system over-strength of these buildings.

On this basis, the Storey shears have been determined as 2739 kN, 2563 kN, 2211 kN, 1683 kN and 978 kN for the level 1 to roof, respectively. Note that 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 device can provide 50% displacement more than the design displacement, while the force in the device will increase by a factor of 1.25 (Hashemi et al., 2019). This factor actually comes from the ratio of the maximum inelastic capacity of the clamping rods over the capacity of the discs used. Thus, an over-strength factor of 1.25 is used to design the brace body and the collector beams while an over-strength factor of 1.5 is used to capacity design the columns.

Step 4: Assess P-delta effects and check the seismic lateral deflections

A structural analysis in the SAP2000 program is performed in this step to evaluate the P-delta effects 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. It might have been more accurate to model the base as nominally fixed with a rotational stiffness of 1.67EI/L, where E, I and L relate to the column connecting into the base, however for the braced frame the effect will be relatively minor. A rigid diaphragm has been assigned to each floor to represent the effect of floor slabs and to ensure that each node in a same floor experience a same lateral deflection. The gravity loads have been assigned to the beams. The seismic weights have also assigned to the nodes in each elevation. Figure 4 shows the general arrangement of the model.

The RSFJs are modelled using the ‘Damper – Friction Spring’ link element with the ‘tension-only’ feature activated (Hashemi et al., 2017b). The ULS deflection limit for the devices was considered as 2% to calibrate and design the flag-shaped response of the devices (on the basis of the frame undergoing an elastic lateral drift of 0.3% before opening the joints). The F_{slip} was assumed as 70% of $F_{ult,loading}$ for all RSFJs. (see Figure 1(b)). The initial stiffness of the links is considered as the elastic stiffness of the brace body after running the analysis and optimising the model. Table 1 shows the hysteretic properties of the RSFJs.

Table 1: Hysteretic properties of the RSFJ braces

Level	Code	F_{slip} (kN)	$F_{ult,loading}$ (kN)	$F_{ult,lunoading}$ (kN)	$F_{residual}$ (kN)	Δ_{max} (mm)
Roof	RSFJ5	753	1075	375	108	56
4	RSFJ4	1294	1849	647	185	56
3	RSFJ3	1700	2429	850	243	56
2	RSFJ2	1971	2816	985	281	56
1	RSFJ1	2107	3010	1055	301	56

Figure 4 shows the configuration of the model and the deformed shape of the structure. The fundamental period of the structure from the model is 0.55 seconds that is consistent with the previous assumption (0.54 second). The earthquake-induced deflections of the structure are majorly controlled by the RSFJs and for all storeys, the inter-storey drift was reasonably close to 2% which is in fact the design target. In addition to the displacement found from the analysis, the standard also considers the increase in displacements due to P-delta effects. These effects, however, are not required for this case study respecting the clause 6.5.2(c) in section 6 of (NZS1170.5, 2004). The analyses have shown that the stability coefficients (θ) calculated using Equation (4) are less than 0.1 for all the storeys, which is expected for a CBF system. In Equation (4), W_i is the seismic weight of the storey, (h_i-h_{i-1}) is the storey height, δ_{ui} is the ULS storey displacement and V_i is the storey shear strength (conservatively can be taken as the design storey seismic force).

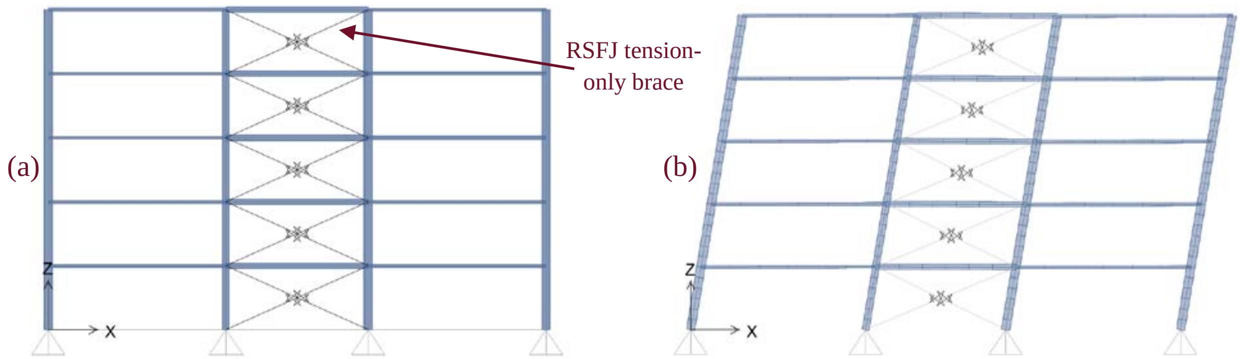

Figure 4: The model developed for the braced frame: (a) general arrangement (b) lateral response

$$\theta = \frac{W_i \delta_{ui}}{V_i (h_i - h_{i-1})} \qquad (4)$$

3.2.2 Structural member design

Step 5: Determine the required brace member sizes

In this step, the brace member seizes are designed. For the tension-only x-braces, any element that can resist tension loads can be used. For this structure, non-threaded rods of grade 830 MPa were considered and designed with an over-strength factor of 1.25 on the force associated with full expansion of the RSFJs as described in Step 3 of the procedure. As per (Feeney & Clifton, 2001), for tensioned braced CBFs, the forces due to gravity loads are usually negligible and the braces are designed only for seismic loads.

The diameter of the rods is therefore specified as 80 mm, 75 mm, 70 mm, 60 mm and 50 mm for level 1 to roof, respectively. Note that instead of one rod, using even number of smaller in size rods (similar to the detail shown in Figure 2) may be more practical in real cases.

Step 6: Calculate the brace over-strength tension capacities

This step implements the capacity design procedures where all other structural members are designed for the capacity-design derived actions from the RSFJs to ensure that the chosen ductile failure mechanism develops. The HERA report R4-76 suggests the following equation:

$$N^{ot}_{brace} = \emptyset_{oms} A_g f_y \tag{5}$$

Where N^{ot}_{brace} is the brace over-strength tension capacity, $\emptyset_{oms}$ is the appropriate over-strength factor used to size the non-yielding element (for the case of the RSFJ tension-only brace, the brace body and the attachments) and A_g is gross area of the cross section. As mentioned in Step 3 and Step 5, an over-strength factor of $\emptyset_{oms}$ =1.25 is suggested for this case respecting the experimental tests described in (Hashemi et al., 2018). Note that it is very unlikely for the RSFJ to develop a more than expected resistance. Nevertheless, it is the designer's choice to make allowances to account for greater than specified strength of the material.

Step 7: Determine the collector beam capacity design derived actions

The collector beams at each level are subjected to axial compression forces (due to the transfer of horizontal component of the brace tension force from the floor above to the floor below) and the shear/bending actions from the gravity loads. The collector beam capacity design derived seismic compression force can be determined using Equation (6):

$$N^{c}_{beam,i} = N^{ot}_{brace,i} \cos\theta_i \tag{6}$$

Where N^{ot}_{brace} is calculated using Equation (5) and θ_i is the angle between the brace at level i and the beam. The shear and bending action are calculated assuming simply supported boundary conditions for the beams. The design of the beams was governed by the combined actions of axial forces and bending moments.

Step 8: Determine the column capacity design derived actions and design actions

The capacity design seismic compression and tension forces are determined using Equation (7) and (8):

$$N^{c}_{col,i,com} = \sum_i^n N^{ot}_{brace,i} \sin\theta_i \tag{7}$$

$$N^{c}_{col,i,ten} = \sum_{i+1}^n (N^{ot}_{brace} \sin\theta_i)_{i+1} \tag{8}$$

Where 'n' is the level at the top of the CBF system. The column seismic axial forces are combined with the compression forces for the gravity load combination considering the appropriate sign. When column seismic tension force exceeds the column gravity compression force, the column design tension force is negative indicating that the column is subjected to net uplift. This was not the case for this structure.

The column design bending moments usually results from the eccentric beam shear force (eccentric beam shear force relative to the column centreline) and the nodding eccentricity (the eccentricity between the axial force in the brace and the column centreline). (Wijanto, 2012) suggest the moment resulted from these eccentricities are not considerable thus they are neglected for this case study structure. Nevertheless, the numerical model can be modified properly to catch the column bending moment from these two types of eccentricities.

Steps 9 and Step 10: Design the collector beams and columns

The beams and columns are designed based on the capacity design actions derives from previous steps. Standard Universal Beams (UB) and Universal Column (UC) sections are considered for the collector beams and columns, respectively. As mentioned earlier, the capacity design over-strength factor applied to the columns is 1.5. A column splice is considered at level 3 of the structure. The column size changes from a 500WC to a 350 WC at level 3, however it might be more economical to keep the 500WC range full height and change from the 440 kg/m length weight below the splice to say a 500WC228 column above the splice. This is because splicing between two columns of the same designation is much more cost effective than splicing between two columns of different designation. The specified sections were then assigned to the members in the numerical model. Figure 5 shows the assigned sections. Note that given the pinned connections for the beams, braces and column bases, the system level seismic performance is mostly governed by the characteristics of the braces.

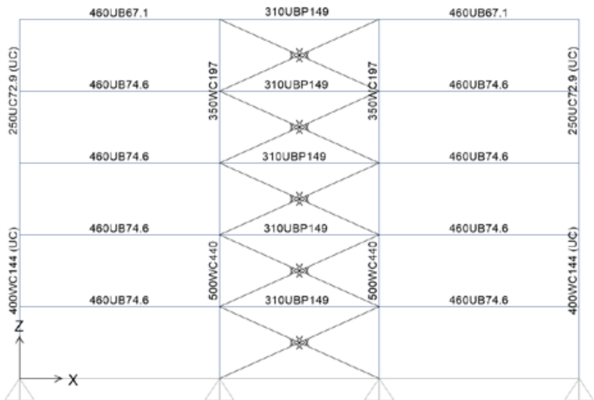

Figure 5: The assigned sections for the structural members

Step 11: Design and detail the connections

Designing and detailing of the connections should be properly done using the appropriate over-strength factors. For the RSFJ tension-only braces, an example of connection detailing concept is illustrated in Figure 2. As mentioned, based on the performance of the RSFJ secondary fuse (Hashemi et al., 2018), a minimum over-strength factor of 1.25 is recommended for designing the pins, connections and other attachments. For space limitations, the detailed design of the connections is not covered for this example.

4 NONLINEAR STATIC PUSHOVER AND NONLINEAR DYNAMICTIME-HISTORY ANALYSIS

In this section, non-linear static pushover and non-linear dynamic time-history analyses are performed on the designed case study structure to investigate the global seismic performance of the system and verify the proposed design procedure. Seven earthquake records are chosen for the simulations. The record are scaled for ULS based on the method described in (1170.5, 2004) for the given location and soil type. In addition to the time-history analysis, a nonlinear static pushover analysis was performed to verify the intended performance of the system that is originating from the RSFJ tension-only braces. Figure 6(c) shows the pushover curve. As can be seen, the base shear of the structure at 2% of roof lateral drift matches the base shear calculated from using ESM with the slip starting at approximately 0.3%. The structure shows a bi-directional performance resulting from the flag-shape hysteresis of the RSFJ tension-only braces. Note that for time-history simulations, the RSFJs are allowed to continue expanding up to 1.5 times of the ULS design deflection to account for the effect of the secondary fuse.

Figure 6(a) shows the maximum recorded base shears. According to (Bradley, 2014), the ‘peak of three’ or the ‘average of seven’ records may be considered for design purposes. It can be seen that the average base shear of the seven simulations is 2167 kN which is lower than the base shear from the ESM. The highest recorded base shear is related to the Christchurch event (2958 kN) which is slightly higher than the target base shear. Figure 6(b) shows the maximum recorded roof drifts. As can be seen, the average top lateral drift is 0.99% and the largest drift is for the Christchurch event (2.24%). These observations show that the structure was able to meet the force and displacement demands. More to the point, it is demonstrated that the structure is properly designed and the RSFJ braces are properly sized. It should be emphasized that the residual displacement was zero at the end of all simulations.

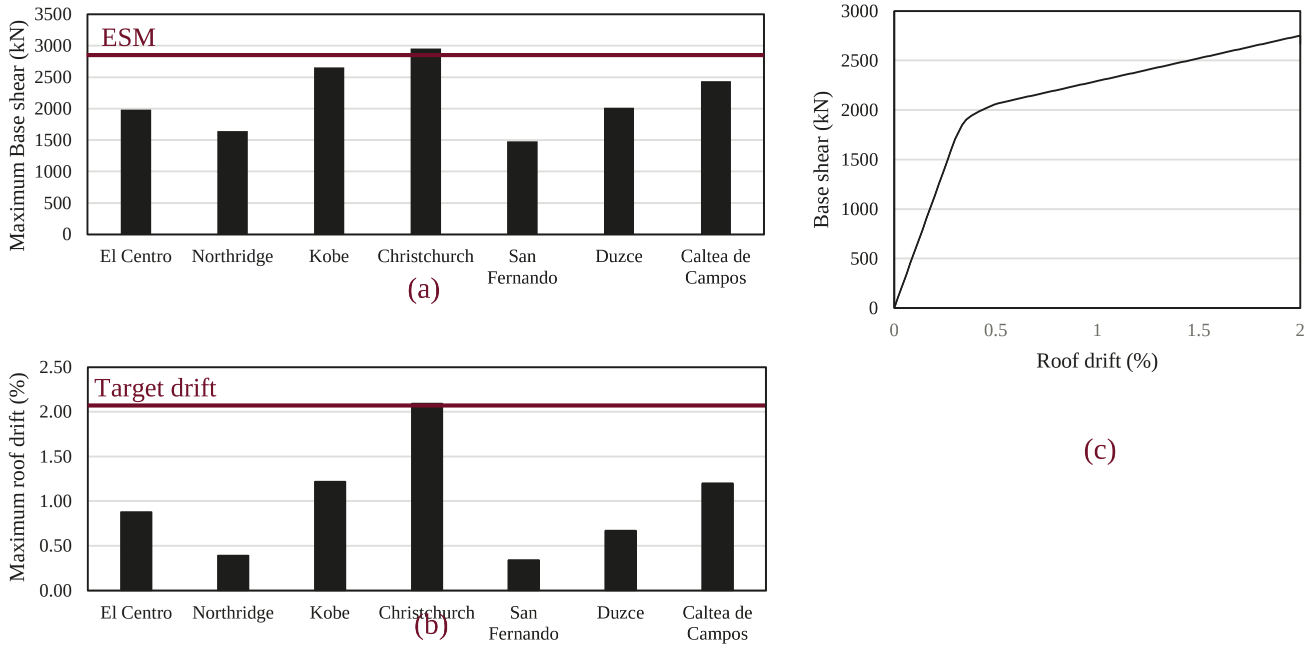

Figure 6: Results of the non-linear dynamic time-history simulations: (a) maximum base shears (b) maximum top lateral drifts (c) pushover curve

Note that for this case study, the numerical results seem to suggest that the adopted ductility factor μ=3.0 was correct and the structure could meet the demand. Nevertheless, for some designs, there may be a need to revise and adopt a higher ductility factor providing the serviceability limit state conditions are satisfied. Note the local displacement ductility demand on the devices is likely to be higher to achieve the desired system ductility.

Overall, the results of the numerical simulations confirmed the efficiency of the proposed procedure for designing the steel frames with RSFJ braces effective in tension only.

5 CONCLUSIONS

This paper proposes a step-by-step design procedure for steel structures with diagonal Resilient Slip Friction Joint (RSFJ) braces effective in tension only. The RSFJ is a friction-based energy dissipation device that can provide damping and self-centring behaviour in one package resulting in a flag-shaped hysteresis. The procedure is based on the New Zealand standard for steel structures and the HERA Report R4-76 originally proposed for steel Concentric Braces Frames (CBFs). The paper also discussed that the code-prescribed height limitations and *Cs* factors for conventional tension-only braces may not apply for the RSFJ tension only brace.

The proposed procedure is used to design a 5-storey steel structure with RSFJ tension-only braces followed by non-linear static pushover analysis and non-linear dynamic time-history simulations. The results confirmed that efficiency of the proposed design procedure for designing the structure and sizing the devices. The findings indicate that this procedure can also be used for other low damage bracing systems that have a flag-shaped hysteresis.

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