The Equivalent Ductility approach for designing the structures using Resilient Slip Friction Joints (RSFJs)

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ABSTRACT

The innovative Resilient Slip Friction Joint (RSFJ) technology has recently been developed and introduced to the New Zealand construction industry. This damage avoidance technology not only aims to provide life safety, but also to minimize the earthquake-induced damage so the building can be reoccupied after the event with minimal downtime. The RSFJ is a friction-based energy dissipation device that provides the required seismic performance regardless of the material used for the main structural components. It can be used in various lateral load resisting systems including (but are not limited to) shear walls, rocking columns, tension-compression braces, tension-only braces and moment resisting frames. The performance of the RSFJ technology has previously been verified by joint component testing and full-scale experimental tests.

Different design codes around the world have different approaches to determine the design seismic loads yet most of them recommend to reduce the elastic base shear by a factor that is related to the ductility. Most of the codes (including the National Building Code of Canada (NBCC)) recommend ductility-related values for different types of conventional structures based on the type of lateral load resisting system and the material used. Nevertheless, there is still lack of information about the seismic design of buildings with more advanced technologies such as RSFJ.

This paper aims to provide a simple analysis and design procedure for the structural engineers when designing a seismic resilient building with RSFJs. A step-by-step forced-based design procedure is provided that generally requires the use of the Equivalent Static Method (ESM) to specify the structural design actions followed by non-linear static pushover and non-linear dynamic time-history simulations to verify the performance. In this procedure, the designer adopts a force reduction factor at the start and verifies it at the end. A case-study structure that uses RSFJ braces as the lateral load resisting members is considered to explain and follow the proposed design procedure. Additionally, the paper discusses the use of the proposed procedure with the respect to the NBCC.

Overall, the findings of this paper confirms that the proposed approach can be confidently used when a seismic resilient design with the RSFJ technology is targeted.

Keywords: ductility, seismic-proofing, design procedure, self-centering, residual drift.

INTRODUCTION

Friction-based energy dissipation devices have already been proven to be one of the most efficient solutions amongst the passive damping systems to control the seismic performance of the structure and decrease the damage during and after earthquakes. These devices that were originally developed by Pall et al. for the steel braced frames and concrete panels [1-2] are also known to be cost efficient respecting the simplicity of installation and the configuration used for the assembly. The concept was further developed later by Popov et al. (Slotted Bolted Connections (SBC) [3]) and Clifton et al. (Sliding Hinge Joint (SHJ) [4]) for steel moment resisting frames. Regardless of the good performance of these devices and their large energy dissipation ratio, the residual displacements after the seismic event has always been a concern for the structures equipped with friction devices. This is due to the large force required to bring the device back to the pre-earthquake configuration. According to Erochko el al. [5] the residual drifts more than 0.5% can be considered as the total loss threshold where replacing the building is more economical than repairing it. To compensate for this issue, self-centering structural solutions have been developed by researchers and engineers to minimize the post-event residual displacement. Tremblay et al. [6] and Christopoulos et al. [7] introduced and developed the Self-Centering Energy Dissipative (SCED) brace that consists of steel bracing elements and friction energy absorption members coupled with a simple self-centering mechanism using pre-stressed steel tendons. The PRESSS system for reinforced concrete structures [8] and Pres-Lam system [9] for timber structures are other examples of previously developed self-centering structural forms.

The innovative Resilient Slip Friction Joint (RSFJ) [10] technology has recently been developed and introduced to the New Zealand construction industry. This damage avoidance technology that already has been implemented in two real projects (and is under study for more), provides self-centering behavior and seismic energy dissipation in one compact package. It also includes a built-in collapse prevention secondary fuse function that adds more resiliency to the system in case of a seismic event larger than the design level. Hashemi et al. [11] experimentally verified the flag-shaped hysteresis and the self-centering 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-centering behavior. The angle of the ridges is specified in a way that 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-centers the slotted sliding 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 disc springs are flat (see Figure 1(d)).

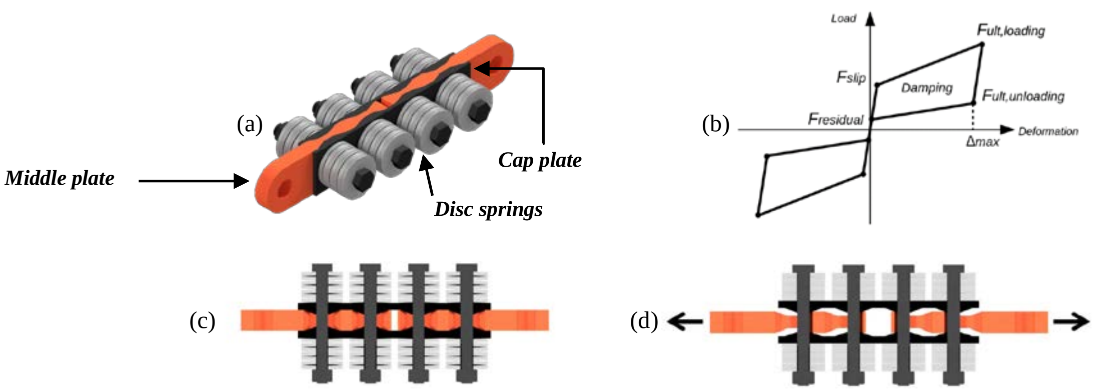

Figure 1. Resilient Slip Friction Joint (RSFJ): (a) assembly, (b) hysteresis, (c) the joint at rest, (d) the joint at the maximum deflection

Figure 1(b) displays the load-deformation behavior for the RSFJ. The slip force (F_{slip}) and the residual force ($F_{residual}$) in the joint can respectively be determined by Eq. (1) and Eq. (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 Eq. (1) and Eq. (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 [12]–[14] for more information about the conducted full-scale experimental tests on different applications of the RSFJs including the test results and discussions.

THE ANALYSIS AND DESIGN PROCEDURE RECOMMENDED FOR STRUCTURES WITH RSFJS

The Equivalent Static Method (ESM) is the most favorable way to calculate the seismic forces worldwide. The reason is firstly the simplicity of this method and secondly the fairly accurate (yet conservative [15]) results. In this method, the earthquake excitations are represented as horizontal static loads applied to story levels in which the amount of loads usually depend on soil type, period of the structure, importance level, location and the type of the lateral load resisting system. When a ductile behavior is expected from the structure, the calculated elastic seismic loads are reduced by a factor which is related to the level of ductility. In the New Zealand standard for structural design actions [15], this factor is defined as the "inelastic spectrum scaling factor (k_μ)" which is related to the structural ductility factor (μ) and the period of the structure (T_1). k_μ can be calculated using the following equations:

$$k_\mu = \mu \qquad \text{(for } T_1 \geq 0.7 \text{ sec)} \tag{3}$$

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \qquad \text{(for } T_1 < 0.7 \text{ sec)} \tag{4}$$

The previous studies of Hashemi et al. [16], [17] showed that the specifications of the RSFJs can be tuned based on the design requirements and this technology is quite flexible in terms of the wide range of specifications (target force and displacement) it can offer. Therefore, there should be a method or a procedure available for the designers so they can efficiently specify required specifications for the RSFJs to meet the design demands.

Following the discussion above, the step-by-step structural analysis and design procedure shown in Figure 2 is developed and recommended to be used when designing seismic resistant structures with RSFJs. The overall aim of the procedure is to determine the force/displacement capacity of the RSFJs based on a given structural performance and accordingly size the devices. This procedure which is based on the ESM, requires non-linear static push-over simulations to tune the RSFJs and non-linear dynamic time-history simulations to verify the target performance. This procedure generally is compatible with most of the building codes around the world. The last section of this paper discusses the compatibility of this procedure with NBCC.

As can be seen in Figure 2, an equivalent ductility factor of μ = 3 is adopted (a start point based on the previous parametric studies) at the start and will be verified and optimized by the time-history analyses at the end. When adopting this procedure, iterations may be required to achieve the accurate equivalent ductility factor and the optimized design. Note that as part of the procedure, a numerical model for the structure is required to be developed (including the RSFJs). The next section describes each step of the procedure outlined in Figure 2 together with an example case where the procedure is used to design the RSFJs for a CLT structure.

NUMERICAL MODELLING OF A FIVE-STORY STRUCTURE WITH RSFJ BARCES

A design example case study building is provided here following the proposed analysis and design procedure. The considered prototype building uses Cross Laminated Timber (CLT) floors, CLT load-bearing walls as the gravity loads resisting members and balloon type CLT shear walls with RSFJ hold-downs at the bottom corners as the lateral load resisting system. Different solutions can be considered for connecting the RSFJs to the CLT walls. Note that since the seismic performance of the system is provided by the geometrically non-linear behavior of the RSFJs and rest of the structure remains elastic, the performance of the system is independent from the material used for the main structural members. The only concern is how to attached the RSFJs to the structural members which for this case, as shown in [18], long self-tapping screws or bolted connections can be used. It was assumed that the CLT floors and the CLT walls are 200 mm thick panels with five layers of MSG8 timber. The load-bearing CLT panels were assumed to be 150 mm thick with three layers.

The building is designed for soil type C in Christchurch, New Zealand. The total height of the structure is 15 m with 5 m wide spans. Figure 3(a) shows the typical plan view of the structure where each wall uses two RSFJs at the base level. The design dead loads including the CLT panels, services, ceiling, cladding and self-weight of the structure were specified as 3 kPa and 1.6 kPa for the first four floors and the roof, respectively. The design live loads were assumed 2.0 kPa and 0.5 kPa for the first four floors and the roof, respectively. The abovementioned design loads correspond to seismic masses of $1.9 * 10^5$ kg and $1.0 * 10^5$ kg for the first four floors the roof, respectively. The target design drift is considered as 1.5% (0.015h).

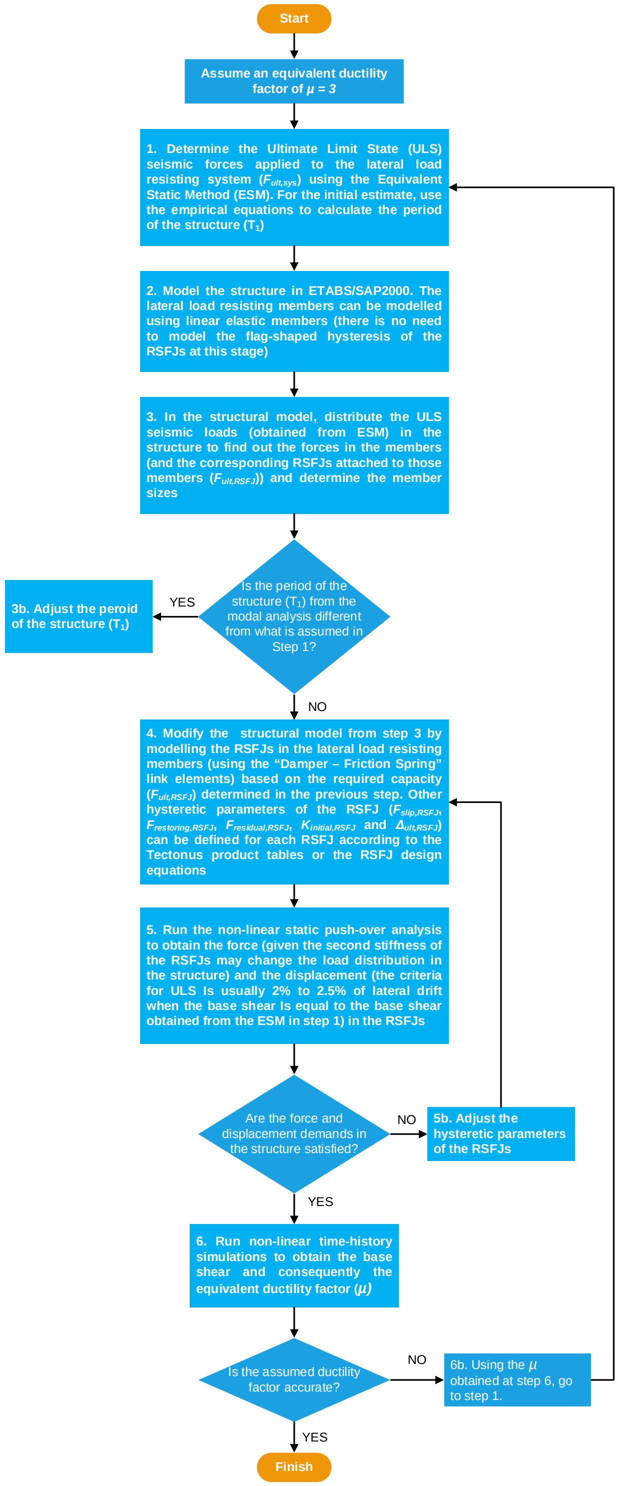

Figure 2. The proposed analysis and design procedure

The step-by-step flowchart procedure provided id figure is followed:

-Assume an equivalent ductility factor of μ = 3.

1. Determine the design seismic forces applied to the lateral load resisting system ($F_{ult,sys}$) using the Equivalent Static Method (ESM). For the initial estimate, use the empirical equations to calculate the period of the structure (T_1):

The structural performance factor (S_p) is considered as 0.7 (this factor is comparable to the R_0 in the NBCC as discussed in the last section of this paper). Note that the New Zealand standard requires to increase the base shear of the structure when including the P-delta effect in the design. This would be done by multiplying the calculated base shear by a factor that is related to the ductility of the system and the seismic weight. For this preliminary example, the P-Delta effect is not considered but for a more detailed design, it is recommended to increase the base shear based on the chosen building standard to account for this effect.

2. Develop a numerical model for the structure. The lateral load resisting members can be modelled using linear elastic members (there is no need to model the flag-shaped hysteresis of the RSFJs at this stage):

The structure is modelled in SAP2000 version 19.0 [19] . Figure 3(b) shows the general arrangement of the numerical model at this stage.

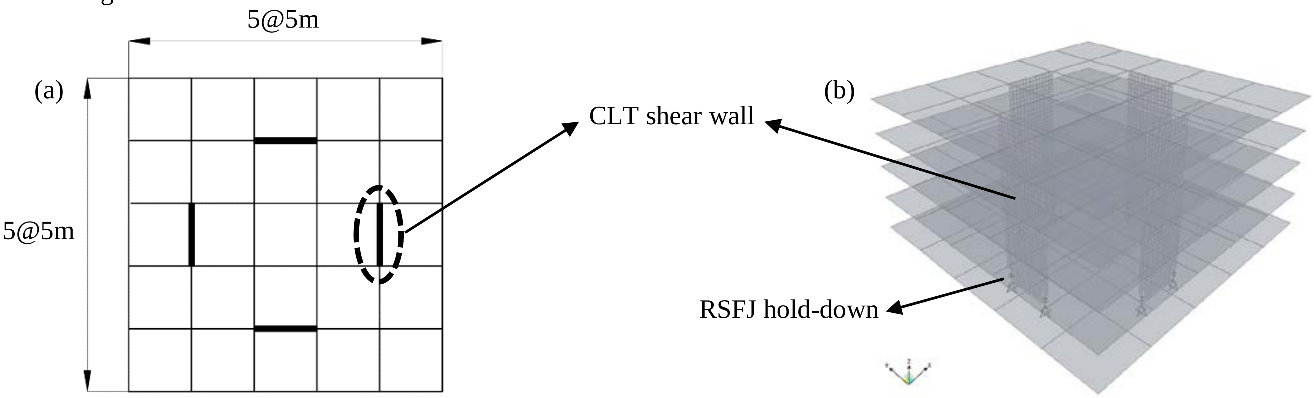

Figure 3. Numerical model: (a) plan view of the structure, (b) general arrangement

3. In the structural model, distribute the design seismic loads (obtained from ESM) in the structure to find out the forces in the members (and the corresponding RSFJs attached to those members ($F_{ult,RSFJ}$)):

- Is the period of the structure (T_1) from the modal analysis different from what is assumed in Step 1?

The period of the structure (T_1) at this stage is determined as 0.56 seconds from the modal analysis which is higher than what was assumed in step 1. Therefore, steps 1 to 3 should be repeated with this new value. Following the ESM with the new period, the base shear of the structure is reduced from 1150 kN to 956 kN. Then, the force demand in the RSFJs at this stage is 1160 kN. The lever arm for the RSFJ hold-down is assumed as 4.8 m given the width of the RSFJ devices is considered as 200 mm.

4. Modify the structural model from step 3 by modelling the RSFJs in the lateral load resisting members (using the "Damper – Friction Spring" link elements) based on the required capacity ($F_{ult,RSFJ}$) determined in the previous step. Other hysteretic parameters of the RSFJs ($F_{slip,RSFJ}$, $F_{restoring,RSFJ}$, $F_{residual,RSFJ}$, $K_{initial,RSFJ}$ and $\Delta_{ult,RSFJ}$) can be defined for each RSFJ according to the manufacturer's product tables or from the RSFJ design equations

At this stage, an elastic drift (the drift of the structure at the slip force of the RSFJs ($F_{slip,sys}$) before they start to open) of 0.2% is assumed for the structure. This is consistent with the previous numerical models developed for CLT shear walls [20]. Based on this and the target design drift (1.5%), the displacement demand of the RSFJ hold-downs was determined as 62 mm. Table 1 shows the specifications for the required RSFJs that are determined with respect to the force demands (specified in the previous step) and the design equations provided in [18].

Table 1. Initial configuration of the RSFJ hold-downs

Initial stiffness (kN/mm)	F_{slip} (kN)	$F_{ult,loading}$ (kN)	$F_{ult,unloading}$ (kN)	$F_{residual}$ (kN)	Δ_{max} (mm)
600	580	1160	435	235	62

Following step 4, the RSFJs were modelled using the "Damper – Friction Spring" link element (available in SAP2000 and ETABS). The accuracy of using this link element for numerical modelling of the RSFJs have previously been verified by comparing the experimental data with numerical results [16]. For each RSFJ, the numerical parameters of the corresponded link element were calibrated using the definitions described in [14].

5. Run the non-linear static pushover analysis to obtain the force and the displacement in the RSFJs.

The results of the non-linear static pushover analysis are displayed in Figure 4. The structure is pushed to 1.5% of lateral drift corresponding to 225 mm of deflection at the roof. Please note that the terminology "non-linear static pushover analysis" is used here but the non-linearity is in fact provided by the non-linear geometrical behavior of the RSFJs. All structural components up to the design drift (1.5%) still behave within their elastic limit.

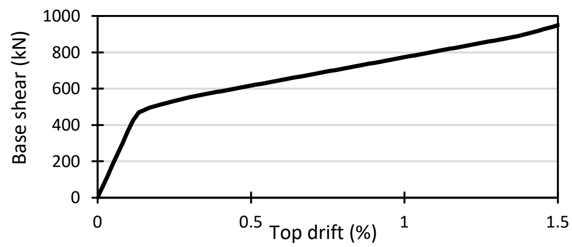

Figure 4. Results of the first non-linear static pushover analysis

- Are the force and displacement demands in the structure satisfied?

It can be seen in Figure 4 that the maximum force in the system at the given drift (1.5%) is 948 kN which is consistent with the base shear from the ESM (956 kN).

6. Run non-linear time-history simulations to obtain the base shear and consequently the equivalent ductility factor (μ)

Non-linear dynamic time-history simulations were carried out to investigate the behavior of the structure. 10 records were chosen for the analysis [21] which were scaled based on the New Zealand standard for the design level (with 1/500 probability of exceedance) for the given soil type and location.

Figure 5 shows the maximum base shears recorded during the time-history simulations. As can be seen, the average is 840 kN. Furthermore, the results showed that the average top roof drift was 1.28% that is lower than the target drift (1.5%). Thus, the new equivalent ductility factor is calculated as $\mu = 3.4$ which is back calculated using Eq. (4) (Note that the period of the structure is less than 0.7 seconds) and the k_μ derived from the records. Given that this ductility factor is higher than the first assumption in Step 1 ($\mu = 3$), the procedure needs to be repeated from the start with the new equivalent ductility factor of $\mu = 3.4$. Note that in this study, the average response from 10 ground motions is considered for calculations. However, the majority of the international building standards accepts the results of the time history simulations if 'the peak of 3' or 'the average of seven records (or more)' is considered in the analyses [22]. Either way, the approach used should be consistent with the chosen building code.

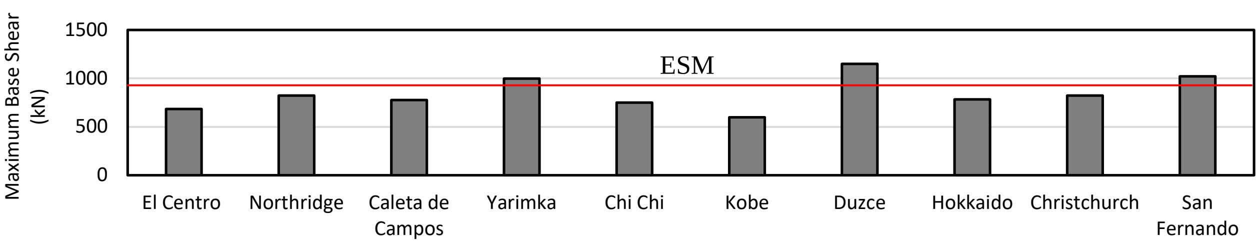

Figure 5. Results of the first non-linear dynamic time-history analysis

Following the design procedure in Figure 2 with $\mu = 3.4$, the new base shear of the structure is determined as 830 kN. Note that the period of the structure (T_1) at this stage is increased to 0.58 seconds from the modal analysis which is close enough to what was considered in the first iteration (0.56 seconds). The force demand in the RSFJs is then reduced to 1020 kN and the specifications are revised accordingly (see Table 2).

Table 2. Revised configuration of the RSFJ hold-downs

Initial stiffness (kN/mm)	F_{slip} (kN)	$F_{ult,loading}$ (kN)	$F_{ult,unloading}$ (kN)	$F_{residual}$ (kN)	Δ_{max} (mm)
500	505	1020	370	200	62

Following step 4, the revised RSFJ hold-downs are modelled in SAP2000 using the "Damper – Friction Spring" link element. Following step 5, the results of the revised non-linear static pushover simulation are displayed in Figure 6. As can be seen, the structure performs as expected by reaching the calculated base-shear at the 1.5% drift.

Following step 6 of the procedure, Non-Linear dynamic Time-History (NLTH) simulations were re-conducted on the system. The new average inter-story drift is increased to 1.47% which is consistent with the target drift (1.5%). Moreover, the new average base shear of the structure is 814 kN which is reasonably close to what was specified following the ESM (830 kN). This shows that the adopted ductility factor in the second iteration (μ = *3.4*) is consistent with the reality given that the difference between the calculated base shears is under 2%.

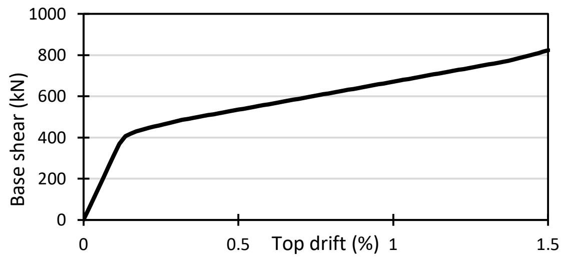

Figure 6. Results of the revised non-linear static push-over analysis

This design example shows the efficiency of the proposed procedure and also demonstrates that a structural ductility factor as high as 3.4 is achievable for structures when using the RSFJ technology. Nevertheless, it should be noted that the base shear resulted from the ESM tends to be higher than the one from time-history simulations. So, it is in fact the designer's choice to consider a safety margin between the base shear from the ESM and the base shear from the NLTH simulations. The it is recommended that in the last step of the procedure, the base shear from the NLTH is equal or less than the one from the ESM.

COMPATIBILITY OF THE PROPOSED PROCEDURE WITH THE NATIONAL BUILDING CODE OF CANADA

The procedure proposed in this paper is developed in a way to be compatible with the international building standards. As mentioned, according to the New Zealand standard for seismic actions, the elastic base shear of the structure is reduced by the inelastic spectrum scaling factor (k_μ) and the structural performance factor (*Sp*). The NZ code suggests S_p=0.7 for the design level earthquakes. So in the example provided in this paper S_p=0.7 and k_μ was calculated at the end. Accordingly, the equivalent structural ductility factor (μ) was back calculated respecting the period of the structure. When considering the National Building code of Canada (NBCC), the ductility-related force modification factor (R_d) can be instead optimized and determined at the end of the procedure. Note that the indicated upper limits should also be considered when following this procedure (despite the fact that the upper limits are suggested for conventional structural systems and in some cases, the systems with RSFJs may offer a higher level of ductility).

Note that the structural performance factor (S_p) in the New Zealand Standard is a force-reduction factor applicable to the elastic base shear to account for the unpredicted higher than considered capacity of the structural and non-structural members and components, the higher than expected energy dissipation in the structure and the 'effective acceleration' which in reality is smaller than the maximum acceleration which the design is based on. Similarly, the NBCC suggests the over-strength-related force modification factor (R_0) to account for the difference between the nominal and the factored resistances, restricted choices of members sizes, less the real minimum yield strength of materials, strain hardening and the additional resistance that can be developed before a collapse mechanism forms in the structure [23]. Both S_p factor and R_0 serves for a similar purpose which is the to cover any unpredicted behavior in the structure that may not be directly measured.

When using the RSFJs in the structure, there would be less uncertainty in the behavior given the structural members are designed to remain elastic (so no additional strength as a result of strain hardening) and the structure will follow the flag-shaped load-deformation relationship that comes from the RSJFs. Therefore, there is room to discuss if the recommended values for R_0 is still appropriate (or safe) to adopt. Ultimately, it is the designer's choice to adopt the appropriate R_0 (or S_p). However, with a damage avoidance philosophy of design, a lower R_0 (or higher S_p) factor may be more appropriate to keep the structure on the safe side.

CONCLUSIONS

The Resilient Slip Friction Joint (RSFJ) technology is a novel seismic energy dissipation system that has recently been introduced to the construction industry. This joints not only provides energy damping but also a fully self-centering behavior meaning that the structure will return to its initial position at the end of the seismic event. The flag-shaped hysteresis of the RSFJ has been previously verified by full-scale experimental tests.

This paper provides a step-by-step analysis and design procedure for the structures that uses RSFJs in their lateral load resisting system. This procedure which is based on the Equivalent Static Method (ESM) involves numerical modelling of the structure, performing non-linear static pushover analysis and non-linear dynamic time-history simulations. The proposed design procedure was implemented for a five-story structure with shear walls RSFJ hold-downs. The findings of this study showed that an equivalent ductility factor of $\mu = 3.4$ was achievable for the example case study structure. Overall, the findings of this investigation showed that the proposed analysis and design procedure could be efficiently used when RSFJs are employed in seismic resilient structures. Respecting the fact that the ESM and NLTH are internationally accepted and widely being used, the proposed procedure is compatible to the different international building codes.

ACKNOWLEDGMENTS

The authors would like to thank the Ministry of Business, Innovation and Employment of New Zealand (MBIE) and the Earthquake Commission (EQC) of New Zealand for the financial support of this research.

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