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SELF-CENTRING PRE-CAST CONCRETE SHEAR WALLS WITH RESILIENT SLIP-FRICTION JOINTS: A NEW SEISMIC DAMAGE AVOIDANCE STRUCTURAL SYSTEM

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ABSTRACT

Ductile concrete shear walls are suitable for protecting buildings from collapse. However, high level of damage is expected after moderate to severe earthquakes. Low damage design concepts can be considered as an efficient alternative to traditional high damage design to minimise damage so that buildings could be reoccupied quickly with minimal business interruption and repair costs. Rocking wall structures absorb and dissipate seismic input energy during their rocking motion. However, this rocking motion should be controlled by a set of additional systems in which high initial stiffness, damping and self-centering are provided. The newly developed Resilient Slip Friction Joint (RSFJ) is a damage-avoidance structural connection which satisfies all these requirements. In this research, an experimental study is carried out at the joint component level to verify the developed analytical model to predict the load-displacement relationship of the RSFJ. Also, the performance of rocking wall systems with RSFJ hold-downs is explained. An anti-locking mechanism within the joint is also introduced. A structural configuration is then proposed for rocking pre-cast concrete shear walls with RSFJs as hold-downs using post tensioning rods or tendons to minimize the effect of cracks in wall's behavior. The performance of the proposed system is then assessed via detailed numerical modeling developed for designing the system. The seismic response the proposed system to earthquake ground motions is then assessed using nonlinear time-history analysis. Results demonstrate the satisfactory seismic performance of the rocking wall system with the RSFJs as well as the concrete panel itself.

INTRODUCTION

Ductile concrete shear walls have widely been used in earthquake resisting systems. High stiffness and high strength as well as acceptable ductility levels can be named as positive points of such lateral load-resisting walls (Paulay and Priestley, 1992). Yielding of the longitudinal bars is the main source for dissipating seismic energy in these walls to provide the required ductility. To achieve this level of ductility, many details should be taken into account which makes the design procedure of conventional concrete shear walls complicated, in addition to the permanent damage. For example, concrete confinement, reinforcement of the potential plastic hinge regions and avoiding brittle failure modes, could be named as the design and practical challenges (Paulay and Priestley, 1992). Shear failure, wall toe crushing and local buckling of the wall segments are some of the brittle failure modes which are observed during the past earthquakes (Kam et al., 2011) and should be avoided (Figure 1). Furthermore, the traditional design which is based on ductile concepts, is costly to be repaired or reconstructed after moderate to severe earthquakes. It should be noted that the cost of business interruption should be also added to the above mentioned costs. Following 2010 and 2011 Canterbury earthquakes and the consequent economic and social impacts, researchers and engineers got more inclined to move from ductile design concepts to low damage ones given the advantages involved. The low damage design concept is based on the fact that the earthquake energy should be dissipated in certain structural elements as sacrificial fuses which can be easily repairable or replaceable after moderate to severe earthquakes.

(a) **(b)** **(c)** **(d)**

Figure 1: Observed Failure modes of structural walls during past earthquakes: (a) Bar yielding followed by bar rupture (b) Toe crushing (c) Diagonal shear (d) Compression failure and buckling (Kam et al., 2011)

To minimise the earthquake damage in addition to satisfying the life-safety of the occupants, researchers and engineers' focus has been on the development of low damage concepts for concrete structures where they could still have the benefits of concrete shear walls. The initial solution introduced was the rocking wall systems in which the rocking motion is controlled using post-tensioned cables (Priestley et al., 1999). In earthquake resistant rocking systems, the earthquake input energy will be balanced by the required energy to swing the structure (Housner, 1963). Even though such systems have shown acceptable seismic performance in comparison to the traditional high damage design, the lack of damping leads to high acceleration and ductility demands. This lack of damping makes the design for deformation compatibility, occupants comfort, and non-structural components more challenging (Sritharan et al., 2015). Adding supplemental damping devices is proposed by researchers to improve the seismic response of the rocking wall systems. Yielding, friction and viscous damping are proposed, tested and used in rocking shear walls (Sritharan et al., 2015).

While these rocking walls with additional damping have become more reliable given the improved performance, there are still some challenges with these systems. For example, yielding elements providing energy dissipation should be inspected and repaired/replaced after each intense event which is costly and causes operational problems for the building. In addition, as the residual capacity of the damaged sacrificial elements will not be sufficient to resist against severe aftershocks, the structure will still be vulnerable until getting fully repaired by replacing the damaged fuses. Furthermore, the creep in the post-tensioning steel strand cables and their supporting members, has always been an issue requiring regular checks and adjustments of the pre-tensioning force. Therefore, there is still a remaining step to achieve an ideal maintenance-free rocking wall system which provides the desired level of performance during the earthquake and aftershocks.

Resilient Slip Friction joint (RSFJ) is a new technology introduced by Zarnani and Quenneville (2015) which provides self-centring and energy dissipation in one compact damage-free connection. This new seismic technology minimizes the earthquake damage and helps the building to be quickly operational after earthquakes. The objective of this research is to assess the seismic performance of rocking pre-cast concrete shear walls with RSFJ hold-downs. In the following sections, the basic equations and hysteretic curve of the RSFJ is introduced and the analytical equations of rocking shear walls with RSFJ hold-downs are developed and experimentally verified. Furthermore, design challenges for implementing the joint in rocking concrete shear walls will be discussed and a configuration for this type of lateral load resisting system is suggested. Finally, a design procedure is presented and verified using numerical modelling.

RESILIENT SLIP FRICTION JOINT (RSFJ)

The components and assemblage of the RSFJ are shown in Figure 2. The outer cap plates and the middle plates are grooved and clamped together by using the high strength bolts. When the imposed force to the joint overcomes the frictional resistance between the surfaces, the centre slotted plates start to slide and energy will be dissipated through cycles of sliding. The specific shape of the grooved plates along with the use of disc springs and high strength bolts provide the desirable self-centring characteristic for this slip-friction joint. The angle of the grooves is designed in such a way that at the time of unloading, the reversing force induced by the elastically compacted disc springs is larger than the resisting friction force acting between the plates' surfaces. Therefore, the system is re-centred by the reversing force upon unloading (Zarnani et al., 2016). The flag-shaped response of RSFJ and its characteristics are shown in Figure 2d.

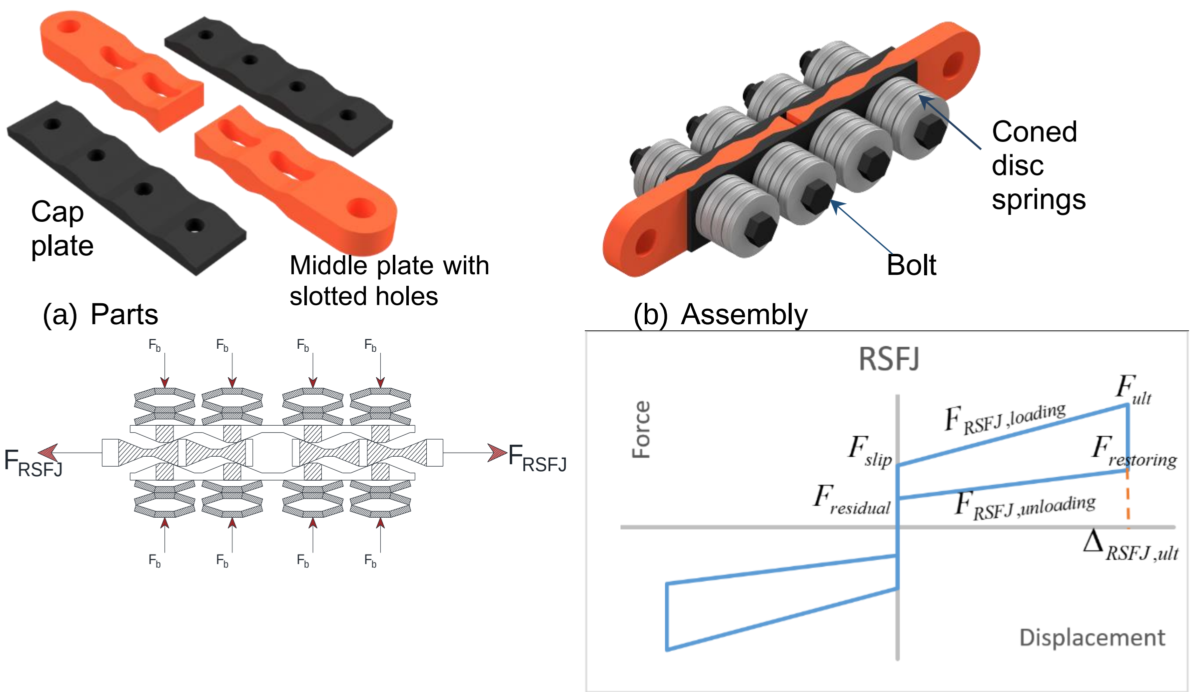


Figure 2: Resilient Slip Friction Joint

Considering the free body diagram of RSFJ (Figure 2c), The slip force of the connection can be determined by

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

in which, θ is the grooves angle; μ is the coefficient of static friction and can be considered equal to 0.18; $F_{b,pr}$ is the bolt clamping force as a result of being pre-stressed; and n_b is the number of bolts on each splice. The residual force at the end of unloading is calculated by Equation 2.

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

The joint ultimate capacity at loading (F_{ult}) and unloading ($F_{restoring}$) can be determined by replacing the $F_{b,pr}$ in Equations 1 and 2 with $F_{b,u}$.
The ultimate force on the bolt ($F_{b,u}$) is given by $F_{b,u} = F_{b,pr} + k_w \Delta_w$ in which, k_w is the stiffness of the stack of disc springs on the bolt and Δ_w is the maximum possible deflection of disc springs after initial pre-stressing.

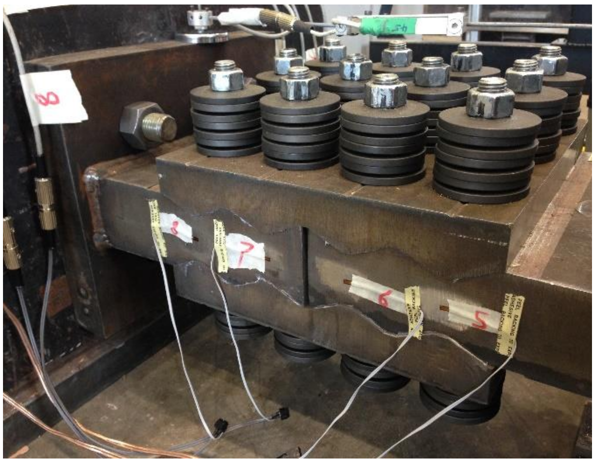
The maximum lateral deflection of the joint can be expressed by $\delta_{ult} = n_j \frac{\Delta_w}{\tan\theta}$ in which, n_j is the number of joints in serial arrangement. It should be noted that for achieving the joint self-centring behaviour, it is necessary to satisfy $\tan\theta > \mu$, and $L > \frac{\Delta_w}{\tan\theta}$ where L is the horizontal distance between the top and bottom of the groove.

In this section, the experimentally achieved load-displacement response of a RSFJ is compared to the analytical models. The properties and parameters of the tested RSFJ are summarised in Table 1.

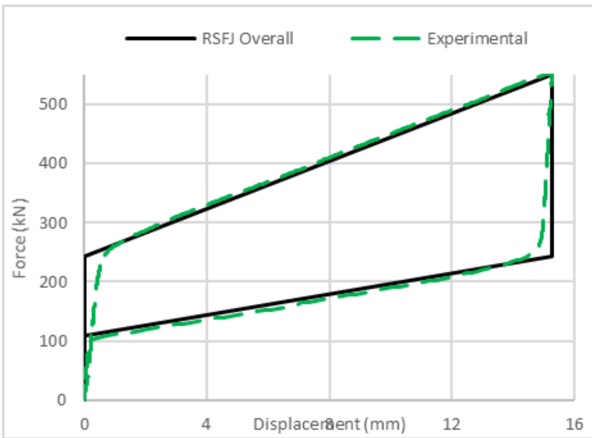
The test setup is shown in Figure 3a. The experimental load-displacement response is compared to the analytical predictions in Figure 3b. As it can be seen from the graphs, the response is well predicted using the analytical model. Considering the flag shape response of the connection, the input energy is dissipated by friction and the joint is finally self-centred by the partially pre-stressed disc springs.

Table 1: the properties of the tested RSFJ

Parameter	Value
θ	25°
μ	0.18
n_j	2
Number of bolts per joint (n_b)	6
Number of discs in one stack	7
Discs internal height	1.55 mm
Discs ultimate capacity	110 kN
Bolts Pre-stressing force ($F_{b,pr}$)	30 kN



(a) Test setup



(b) Load-displacement responses

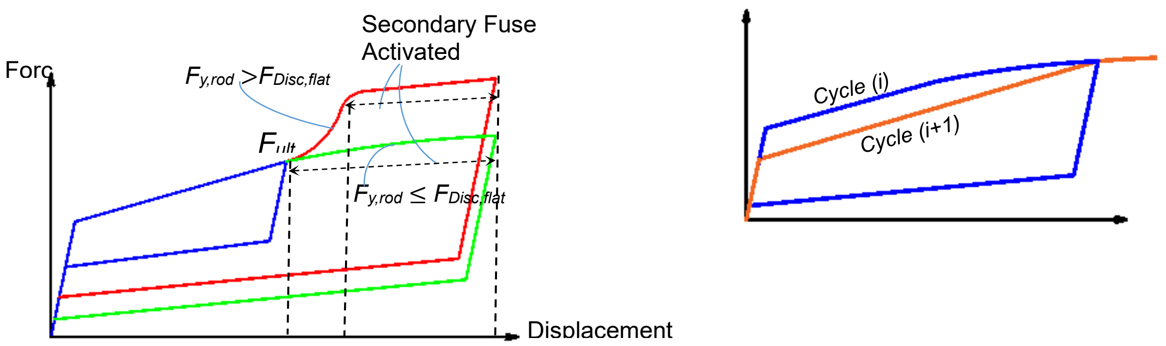
Figure 3: experimental testing on a RSFJ

RSFJ ANTI-LOCKING MECHANISM

When the connection reaches its maximum displacement capacity, the discs are fully flattened and the joint is basically at locked position. At this point, the force demand in the joint supporting elements increases rapidly. This could happen when the earthquake input loading is higher than the joint design capacity. A possible-One solution could be designing the joint for displacement capacities higher than the design requirements. However, the problem of joint locking will remain with less likelihood of occurrence, therefore, not an efficient design. Implementing an internal yielding mechanism as a secondary fuse inside the joint can be considered as a more efficient yet cost-effective solution. In Figure 4, the performance of the RSFJ secondary fuse is shown when the clamping bolts (or alternatively rods) are selected as the sacrificial elements to be yielded in tension when discs get flattened. The main advantage of considering rods or bolts as the yielding elements is that they can be easily replaced after the earthquakes if the events are stronger than the design expectations.

As shown in Figure 4a, based on the relative yielding strength of the rods to the disc springs capacity, two different load-displacement responses are expected for joints with the secondary fuse mechanism. In the case that the rod yield strength is less than or equal to the disc capacity (the green line in Figure 4a), the rods start to yield before the discs are fully compressed. The nonlinear step starts with a gradually decreasing slope behaviour, when the discs are close to be completely flat. In case that the yielding strength of the rods is higher than the disc capacity, the joint is expected to follow an increasing response after F_{ult} up to the force where rods start to yield in tension as shown with the red line in Figure 4a.

After each cycle that the secondary fuse is activated, the load-displacement response of the joint will be changed for the next cycles. This is because of the decrease in the disc springs per-stressing force due to the residual plastic elongation in the elongated rods. The load-displacement response of the RSFJ in cycle (i+1) after the cycle (i) in which the secondary fuse is activated is shown in Figure 4b. Figure 4b shows the case that the yielding force of the clamping rods, $F_{y,rod}$, is less than the capacity of disc springs $F_{Disc,flat}$.



(a) Theoretical load-displacement response

(b) Cyclic response of RSFJ when the secondary fuse is activated

Figure 4: RSFJ with the secondary fuse

RSFJ HOLD-DOWNS IN ROCKING SHEAR WALLS

Figure 5a shows the proposed rocking wall system. The free body diagram of a rocking shear wall with RSFJs and the hysteretic behaviour of the system are shown in Figure 5b and Figure 5c. The deflection of the joints is related to their lever arm, which is the horizontal distance between the RSFJ centre to the rotating edge of the shear wall. Therefore, $\Delta_{RSFJ,1}/\Delta_{RSFJ,2}=L_1/L_2$, where Δ_{RSFJ} and L are, respectively, the expansion of the RSFJ*s* and the associated lever arms (Figure 5b).

By taking the moments about the centre of rotation, which is the wall's toe, the horizontal force applied at the top ($F_{top,slip}$) can be determined by Eq. (3). In this equation, H is the height of the wall, W is the vertical loads, L_W is the horizontal distance from the vertical load to the centre of rotation, and $F_{RSFJ,slip}$ is the slip force of the RSFJ. It is assumed that the employed RSFJs are identical.

$$F_{top,slip} = \frac{1}{H}\left[WL_W + F_{RSFJ,slip}(L_1 + L_2)\right] \tag{3}$$

After the slip stage, the force within the RSFJ corresponds to the deflection within them. Therefore, the lateral strength of the wall can be specified by Eq. (4), where $F_{RSFJ,1}$ and $F_{RSFJ,2}$ are the forces within the tensioned and compressed RSFJ, respectively. The relationship between $F_{RSFJ,1}$ and $F_{RSFJ,2}$ can be determined by Eq. (5) during the loading of the wall. Eq. (5) can also be used for calculations related to unloading stage, with $F_{RSFJ,slip}$ replaced with $F_{RSFJ,residual}$. The value of $F_{RSFJ,residual}$ is equal for the two RSFJs. For the lateral direction of movement indicated in Figure 5b, the ascending RSFJ is in tension and the descending one is in compression. The hysteretic curves are shown in Figure 5c. By employing Eqs. (4) and (5), the overall load-deformation behavior of the wall can be determined.

$$F_{top} = \frac{1}{H}\left[WL_W + F_{RSFJ,1}L_1 + F_{RSFJ,2}L_2)\right] \tag{4}$$

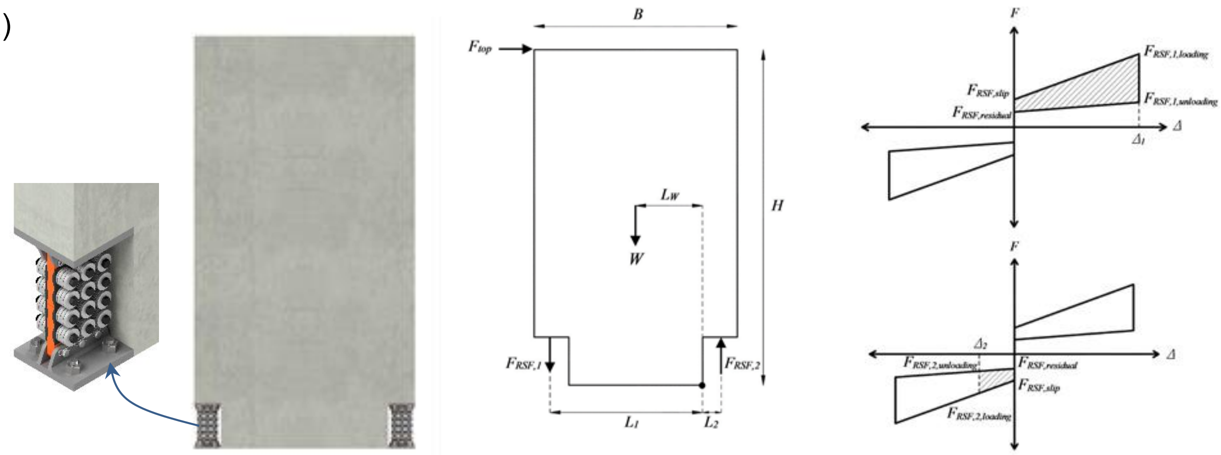
and during the loading of the RSFJs

$$F_{RSFJ,2} = \frac{L_2}{L_1}\left(F_{RSFJ,1} - F_{RSFJ,slip}\right) + F_{RSFJ,slip} \tag{5}$$

The residual force in the wall ($F_{top,residual}$) can be determined by Eq. (3) by replacing $F_{RSFJ,slip}$ with $F_{RSFJ,residual}$. The described analytical approach is based on the assumption that the wall panel represents a rigid body during the rocking motion.

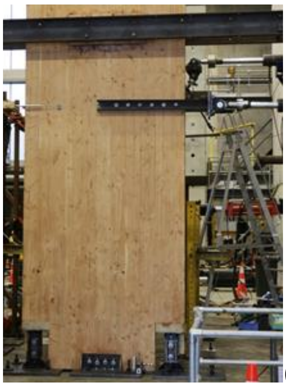
To verify the performance of the developed model, a large scale timber CLT wall was tested by (Hashemi et al., (2017). The dimensions of the wall were designed in a way that the deformation of the wall due to lateral loading can be neglected. Figure 6a shows the rocking wall test setup. A special kind of shear key was used to transfer shear forces to the foundation (Hashemi and Quenneville, 2017). As it can be seen in Figure 6b, the experimental results was predicted very well by the proposed analytical model. It was presumed that the screwed connection (connecting the RSFJs to the wall) was sufficiently rigid and the bending and shear deformation of the panel could be neglected when compared with the overall displacement of the wall. Note that for the slender concrete shear walls more challenges are expected in both analysis and design given the cracking of the concrete significantly changes the initial stiffness of the system and concentrated forces in the hold-downs may cause local failure in concrete. These challenges are discussed more in the following section.

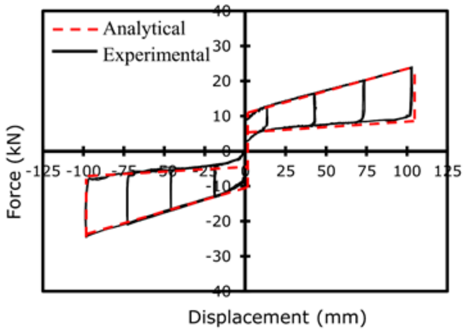
(a)



(b) (c)

Figure 5: Proposed rocking shear wall: a) RSFJs adopted as hold-downs, b) Free body diagram of the wall


(a)



(b)

Figure 6: Tested large-scale timber CLT shear wall (Hashemi et al., 2017b): (a) Test specimen (b) Comparison of analytical and experimental results

POST-TENSIONED PRE-CAST CONCRETE PANELS WITH RSFJ HOLD-DOWNS

In this section, the concept of post tensioned pre-cast concrete walls with RSFJs as hold-downs is introduced. Since the RSFJ will be used as hold downs, bending in the wall due to lateral loading will produce axial compression and tension stresses in the concrete. Considering the weakness of concrete in tension, tension cracks will be formed in the first phases of loading which significantly reduces the initial stiffness of the system before the joint initial slip stage. An efficient way to compensate this is applying pre-stressing forces on the wall.

In order to eliminate the tension cracks, a pre-stressing concept is developed. In this concept, the pre-cast concrete panel is connected to the foundation using RSFJ hold-downs. This precast concrete panel is post-tensioned using unbonded cables or rods. There is no need for additional connections to the foundation such as post-tensioning tendons (Figure 7). After manufacturing and considering proper timing for the concrete curing, the pre-cast concrete panel will be compressed using the unbonded tensioning elements. It should be noted that the post-tensioning force should be higher than the joint slip force in order to prevent the cables from elongating before slipping of the joint. Post-tensioning can be done either at the factory or the construction site when the wall is laid-down on the floor. This decreases the construction time and cost in comparison to the current post-tensioning concepts.
The wall can then be mounted vertically and get connected to the foundation using the RSFJs and end pins. In addition to providing a resilient damage avoidance solution, this concept makes the construction process of the earthquake resisting structures with pre-cast concrete elements easier and more efficient when it is compared to current approaches.

Another advantage of this system in comparison with the current post-tensioned rocking walls is that there is no need to design the post-tensioning elements for high displacement demands as the flexibility of the system comes from the RSFJs.

The RSFJ end connections at the foundation side can be used as the shear load transferring mechanism (shear key) of the system, in order not to allow the wall to slide laterally.

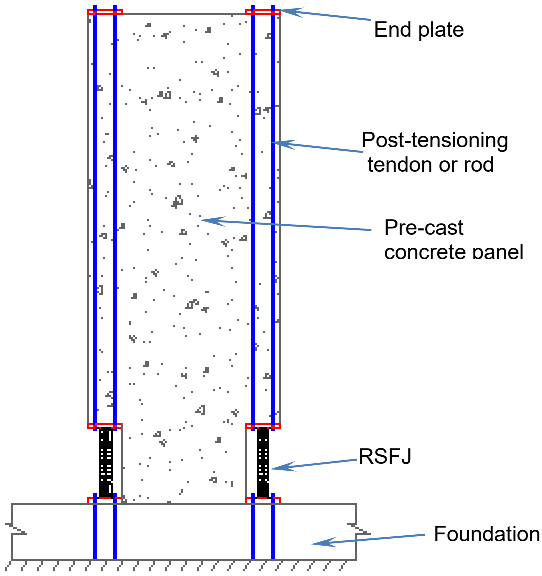


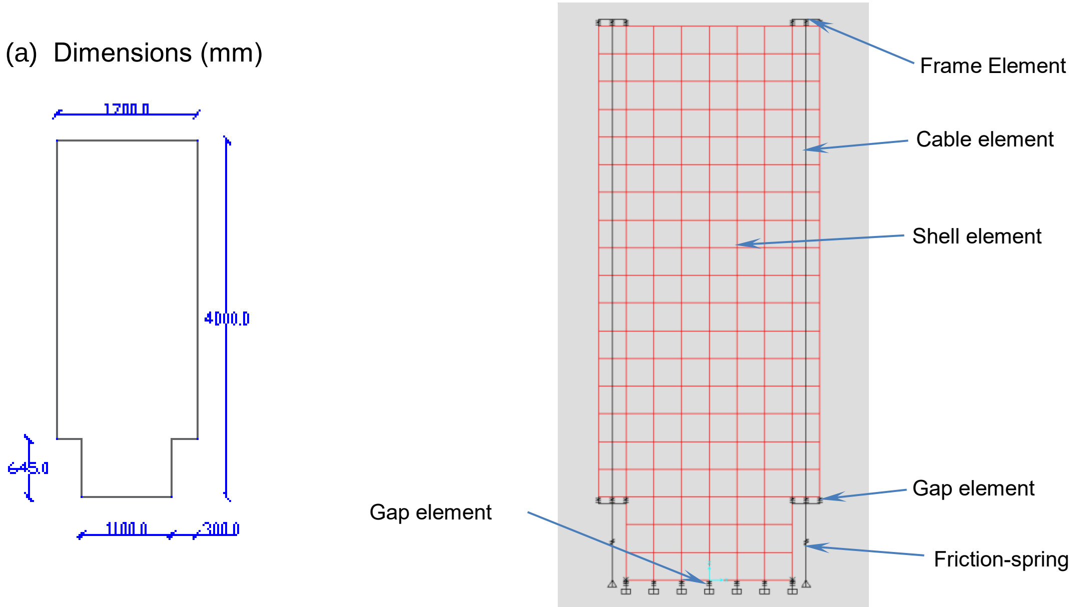
Figure 7: different components of the proposed rocking pre-cast concrete wall system

NUMERICAL MODELLING

In order to verify the performance of the proposed system, a numerical model of a rocking wall system is developed and analysed using the SAP2000 software. This wall is to be manufactured and tested under lateral cyclic loading as the next step of this research.

The dimensions of the wall are mentioned in Figure 8a where the thickness of the wall is 200 mm. Non-linear shell elements are used for modelling the concrete and the reinforcements. The compressive strength of 35 MPa is considered for the concrete. Longitudinal reinforcement ratio of 1% and shear transverse reinforcement ratio of 0.5% are used for modelling the reinforcing steel with a yielding stress of 400 MPa. Additional stirrups are considered for confining the concrete surrounding the post-tensioning elements and at the rocking toes of the wall.

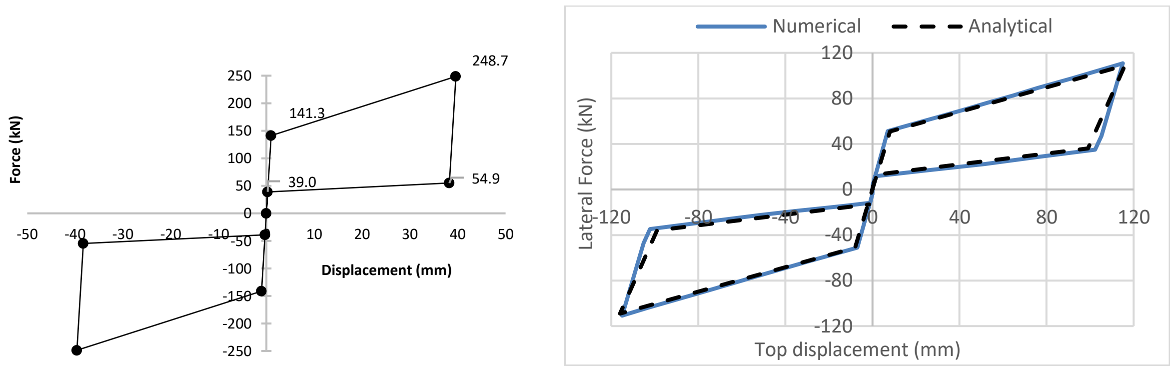
As shown in Figure 8b, Damper – Friction Spring link element is used for modelling the RSFJs. This method of modelling has already been verified by comparing the experimental results with the numerical data. The flag-shaped behaviour of the designed RSFJ is shown in Figure 9a. Cable elements are used to model the post-tensioning elements and the pre-stressing force is applied to them as initial deformation. End plates are modelled as frame elements and they are connected to the wall using the gap elements to accurately model the proposed pre-stressing method. Gap elements are also used for modelling the wall/foundation interface allowing the wall to rock at its toes.



(b) Model characteristics

Figure 8: Sample model of the rocking pre-cast concrete wall

A cyclic analysis is performed to capture the lateral load-displacement response of the model. As shown in Figure 9b, the numerical results are in agreement with the analytical model proposed in the previous sections. This confirms the accuracy of the model for designing different elements of the proposed structure based on the corresponding internal actions. In order to assess the seismic response of the proposed structural system, a nonlinear time history analysis is performed and discussed in the next section.



(a) (b)

Figure 9: a) flag-shaped behaviour of the modelled RSFJs and b) the response of the proposed rocking pre-cast concrete wall system under cyclic loading

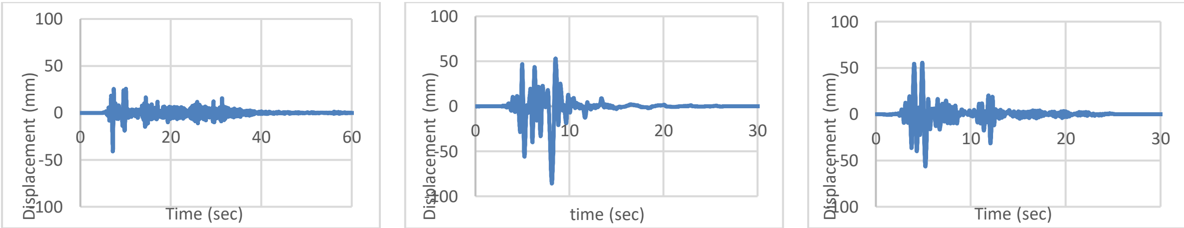
NON-LINEAR TIME HISTORY ANALYSIS

Non-linear dynamic time-history analysis is carried out on a single degree of freedom numerical model with performance similar to the structure described in the last section (see Figure 8). It was assumed that the structure is located on a soil type D zone (deep or soft soil) with a 1000 year return period in Wellington, New Zealand. The hazard factor (Z), the near fault factor (N) and the return period factor (R) are respectively determined as 0.4, 1.0 and 1.3 according to NZS1170.5 (New Zealand standard for earthquake actions). A seismic weight of 120kN is assumed and assigned to the top of the wall. A suite of three conventional earthquake records were selected and scaled in accordance with NZS1170.5 to match the Wellington 1000 year return period for Ultimate Limit State (ULS). Table 2 presents the selected seismic events and the associated scaled peak ground motions. The scaled acceleration records were applied at the base of the model.

Table 2: the selected ground motions and scaling

Event	Year	PGA	Scaled PGA
El Centro	1940	0.32	0.61
Christchurch	2011	0.36	0.72
Duzce	1999	0.53	0.74

The displacement responses for the three simulations are shown in Figure 10. It should be emphasized that the self-centring behaviour is achieved without relying on any additional vertical loads such as the force from the post-tensioned cables (Sritharan et al, 2015) or the gravity loads.



(a) El Centro 1940 (b) Christchurch 2011 (c) Duzce 1999

Figure 10: Roof displacement demand for selected ground motions

The New Zealand standard recommends a drift of 2.5% as the upper bound limit for the structures subjected to the ULS earthquakes. This limitation is indicated in Figure 12. The uppermost recorded displacement among the events is for Christchurch one which is approximately 2.15% of lateral drift. It can be seen that all of the recorded displacements are below the defined maximum allowable limits which indicates the efficiency of the proposed system in terms of satisfying the drift demands.

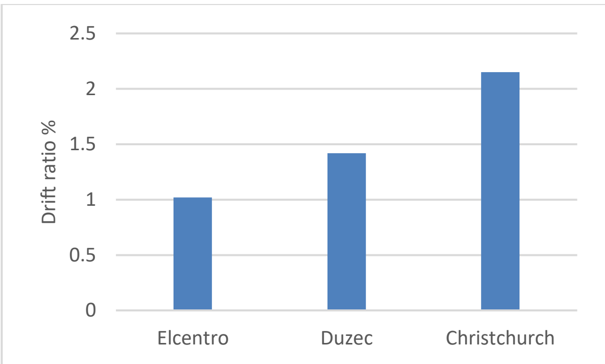


Figure 12: Maximum floor drift ratio for the selected earthquake events

CONCLUSION

This paper presents a new low-damage seismic resisting system in which Resilient Slip Friction Joints (RSFJs) have been adopted for a rocking pre-cast concrete shear wall. This new system provides self-centring as well as energy dissipation through incorporating compact damage-free connections. The results of the preliminary study have shown that this new rocking pre-cast concrete shear wall can be considered as an efficient alternative to traditionally ductile designed walls to minimise damage so that buildings could be reoccupied quickly with minimal business interruption and repair costs. The seismic performance of the proposed system has been evaluated using nonlinear time history analysis and to be further verified by large-scale experimental testing.

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