EARTHQUAKE RESILIENT TIMBER STRUCTURES USING ROCKING CROSS LAMINATED TIMBER (CLT) WALLS COUPLED WITH RESILIENT SLIP FRICTION JOINTS (RSFJS)

A. Hashemi[1], P. Quenneville[1] & P. Zarnani[2]

[1]Department of Civil and Environmental Engineering, The University of Auckland

[2]Department of Built Environment Engineering, Auckland University of Technology

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ABSTRACT

There is an increasing public pressure to have damage avoidant structural systems in order to minimize the destruction after severe earthquakes with no post-event maintenance. This study presents and investigates a hybrid steel-timber damage avoidant Lateral Load Resisting System (LLRS) using Cross Laminated Timber (CLT) walls coupled with innovative Resilient Slip Friction (RSF) joints and boundary steel columns. RSF joints are used as ductile links between the adjacent walls or between the walls and the columns. These joints are capable to provide a self-centring behaviour (the main deficiency of conventional friction joints) in addition to a high rate of energy dissipation all in one compact device. One significant advantage of this system is that there are practically no bending stresses in the CLT panels which considerably increases the allowable capacity of the system. A numerical model for a four story prototype building containing the proposed concept is developed and subjected to time-history simulations. The results confirm that this system can be considered as the new generation of resilient LLRSs for different types of structures.

1 INTRODUCTION

Cross Laminated Timber (CLT) is a relatively new type of engineered wood product which was firstly developed in Europe in the 1990s and then globally expanded as a reliable construction material (Karacabeyli et al. 2013). It is a strong, sustainable and dimensionally stable product which offers different structural characteristics similar to that of a pre-cast concrete panel yet with a relatively higher strength to weight ratio. Additionally, CLT structures possess flexible planning and high level of prefabrication which considerably accelerate the construction process and reduce the overall cost. Hence, CLT has been notably gaining popularity among building owners and designers.

During the PRESSS (PREcast Seismic Structural Systems) program in the early 1990's, a new design approach for structural walls was introduced which was based on the application of simple joints between the prefabricated panels in order to localize the inelastic deformations in those joints (Priestley et al. 1999). In addition, unbounded post-tensioned steel members were employed to provide the self-centring behaviour. The dissipation capacity of such system is highly related to the type of the dissipaters implemented between the walls (also known as the sacrificial fuses). For timber structures, Palermo et al. adopted a similar approach and conducted preliminary experimental tests on Laminated Veneer Lumber (LVL) walls with different types of fuses (Palermo et al. 2005). The results confirmed that the enhanced performance of the system was attributed to the behaviour of the ductile joints. Smith et al. further extended the concept into coupled wall systems (Smith et al. 2007). They proved that the design flexibility of the hybrid coupled wall systems combined with the offered speed of construction creates a significant potential in multi-story buildings. Iqbal et al. studied the application of U-shaped Flexural Plates (UFPs) as supplementary damping devices in post-tensioned LVL timber coupled rocking walls (Iqbal et al. 2015). The test results exhibited an efficient energy dissipation mechanism through the inelastic deformation of the UFPs during the earthquakes. Sarti et al. experimentally investigated the seismic performance of the hybrid rocking walls with end columns (Sarti et al. 2015). Their experimental results confirmed a notable improvement in the seismic performance in terms of energy dissipation and stability of the hysteretic behaviour. Iqbal et al. tested coupled post-tensioned rocking LVL walls with sacrificial nailed plywood sheets as hysteretic dissipative elements (Iqbal et al. 2015). The experimental results affirmed the seismic performance of the system. Nevertheless, relatively lower hysteretic stability was observed compared to the similar systems with UFPs.

Friction based passive damping devices were originally introduced for steel structures. Popov et al. proposed symmetric slotted bolted connections which dissipates energy through friction during equilateral tension and compression cycles (Popov et al. 1995). Popov's comprehensive experiments exhibited stable rectangle-shaped hysteretic loops. Clifton et al. proposed the asymmetric sliding hinge joint for steel moment resisting frames which had non-rectangular yet stable force-deformation behaviour (Clifton et al. 2007).

Filiatrault used friction dampers at the four corners of a traditional timber sheathed shear wall (Filiatrault 1990). The results demonstrated a significant improvement in the hysteretic behaviour of the walls while large amount of seismic energy was absorbed through friction. Loo et al. investigated the application of slip friction connections as the replacement for traditional hold-downs in rocking LVL walls (Loo et al. 2014). The experimental results showed an excellent seismic performance in terms of hysteretic behaviour and the minimized residual deflections. Additionally, and most importantly, the timber wall remained in the elastic region after several non-linear dynamic numerical analyses. Furthermore, no substantial damage was observed in the timber members during the quasi-static experimental tests. Hashemi et al. introduced the application of slip friction connections in CLT coupled walls as the hold-down connections and also as ductile links between the wall panels (Hashemi et al. 2016a). The numerical results confirmed the efficiency of the introduced system in terms of seismic energy absorption and durability of the hysteretic behaviour. The proposed concept was further developed to hybrid rocking core walls in which the post-tensioned joints are used with supplementary devices in the beam-column connections to provide a self-centring behaviour (Hashemi et al. 2016b).

This study seeks to develop a damage avoidant rocking CLT wall system coupled with innovative Resilient Slip Friction (RSF) joints invented by Zarnani and Quenneville (Zarnani and Quenneville 2015) as the ductile links between the adjacent walls or columns. Owing to the characteristics offered by the RSF joints, this innovative lateral load resisting system is able to provide self-centring behaviour in addition to significant rate of seismic energy dissipation. The proposed system is used in the numerical model developed for a four story prototype building. Furthermore, a novel type of shear key invented by Hashemi and Quenneville (Hashemi and Quenneville 2017) is introduced to be used at the base of the rocking walls.

2 RESILIENT SLIP FRICTION (RSF) TECHNOLOGY

Conventional slip friction connections with flat plates sliding over each other have always been recognized as one of the most efficient energy dissipation devices. The provided hysteresis which is close to an elastic-perfectly-plastic one combined with the cost effectiveness characteristic of these devices has made them very favourable. Nevertheless, the lack of self-centring behaviour is a major disadvantage for the structures containing these dampers which may result in considerable residual displacement after an earthquake (Hashemi et al. 2016a). To compensate this issue, a novel energy dissipation device entitled Resilient Slip Friction (RSF) joint is developed. The components of the RSF joint are configured in a way that energy dissipation and self-centring can be achieved all in one compact device. Figure 1 shows the components and assembly of a RSF joint. The angle of the ridges is designed in a way that at the time of unloading, the reversing force caused by the elastically compacted Belleville springs is larger than the resisting friction force between the slipping surfaces. Thus, the slotted plates are re-centred to their original position while energy dissipates over sliding. RSF joint exhibits a flag-shaped hysteresis which is experimentally confirmed by the authors (Hashemi et al. 2017). Figure 1(d) displays the theoretical hysteretic loop of the RSF joint. The slip force and the residual force in the joint can respectively be determined by Equation 1 and Equation 2 where

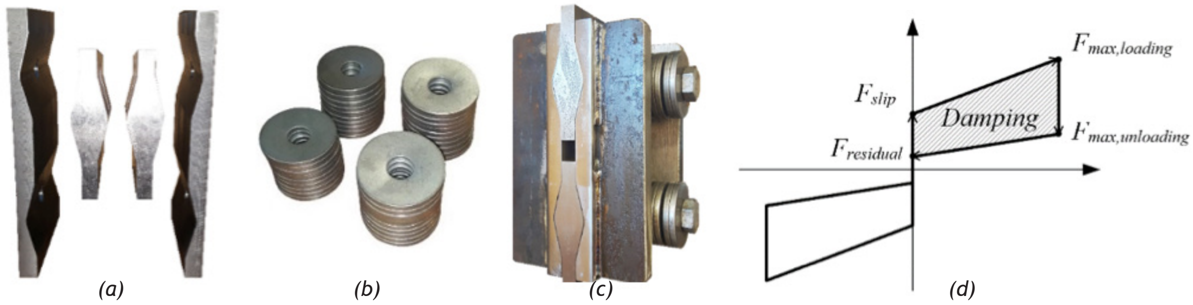

Figure 1: *RSF joint: (a) Cap plates and centre slotted plates (b) Belleville springs (c) Assembly (d) Hysteresis.*

$F_{b,pr}$ is the clamping force in the bolts originated from the pre-stressing of the Belleville springs, 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 replacing μ_s and $F_{b,pr}$ in Equation 1 and Equation 2 by μ_k and $F_{b,u}$, respectively. The reader is referred to (Zarnani et al. 2016) for more information about the design equations for the RSF joints.

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

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

Experimental tests were carried out by Hashemi et al. on a rocking CLT wall with RSF joints as the hold-down connectors (Hashemi et al. 2016). Two identical RSF joints were installed in the notches at the bottom corner of the CLT wall. Each joint consists of two centre slotted plate and two cap plates made with mild steel grade 350. The angle of the ridges was 15 degrees. Fig. 2 shows the general arrangement of the test setup. The RSF joints were designed and fabricated to be able to accommodate a maximum displacement of 65mm in tension and 15mm in compression. The tested CLT panel had five 40 mm thick layers made of MSG8 timber (200mm of thickness in total). Each RSF joint had two bolts and 11 Belleville springs per bolts per side. The springs had a maximum load capacity of 110 kN and a maximum displacement capacity of 1.5 mm at flat state. The loading protocol displayed in Figure 2(a) was applied to the wall at the height of 3350 mm with an approximate loading rate of 1.25 mm/sec.

A novel type of shear key is considered at the base of the wall to transfer the shear forces from the wall to the foundation (Hashemi and Quenneville 2017). This device is able to efficiently transfer the shear forces while accommodating the uplift caused by the rocking movement. As can be seen in Figure 2(d), the bolts are installed in the timber wall while the slotted plate is attached to the foundation. When the rocking motion initiates, the bolts are dragged by the wall while the special shape of the slotted holes allow the bolts to stay in contact with the slotted surfaces during the rocking movement. This allows the device

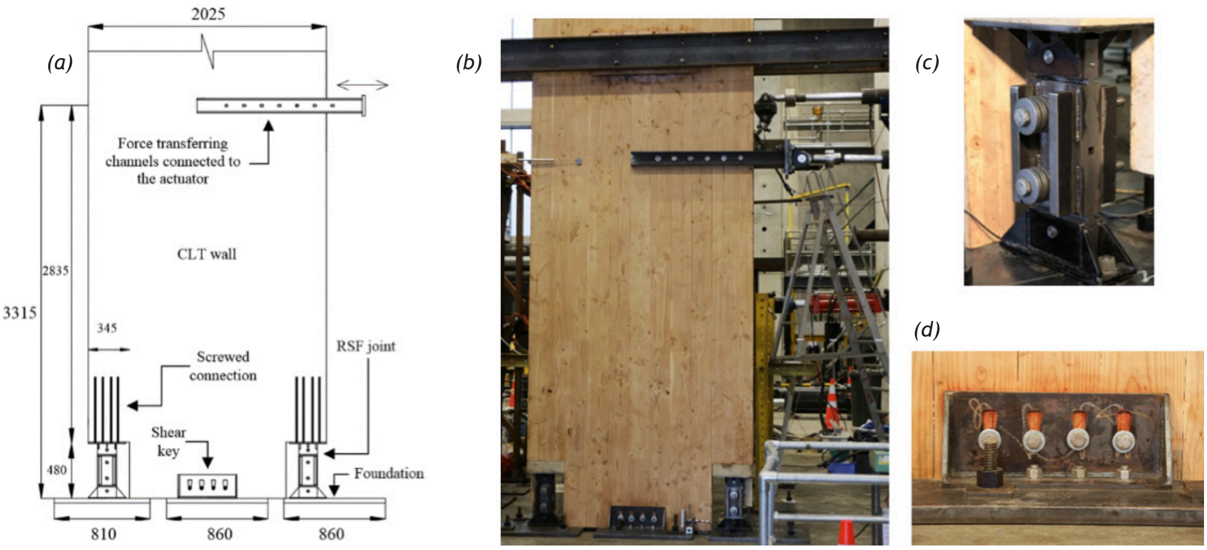

Figure 2: *Experimental test of rocking CLT wall with RSF joints: (a) General arrangement (b) Test setup (c) RSF joint (patent no. 7083) (d) Shear key (patent no. 728725)*

the transfer shear forces with a mechanism similar to conventional bolted connections.

The authors carried out eight tests in total on the wall. No damage was observed to the wall with no evidence of deterioration in strength or stiffness of the RSF joints. Figure 3(b) shows the typical hysteresis for the wall as a representative for the test results. As can be seen, the pre-stressing force was approximately 50% of the maximum force. The flag-shaped hysteresis in Figure 3(b) clearly highlights the self-centring behaviour of the tested wall. It should be pointed out that the self-centring behaviour was independent from the gravity loads considering the fact that the only applied vertical load was the self-weight of the CLT panel.

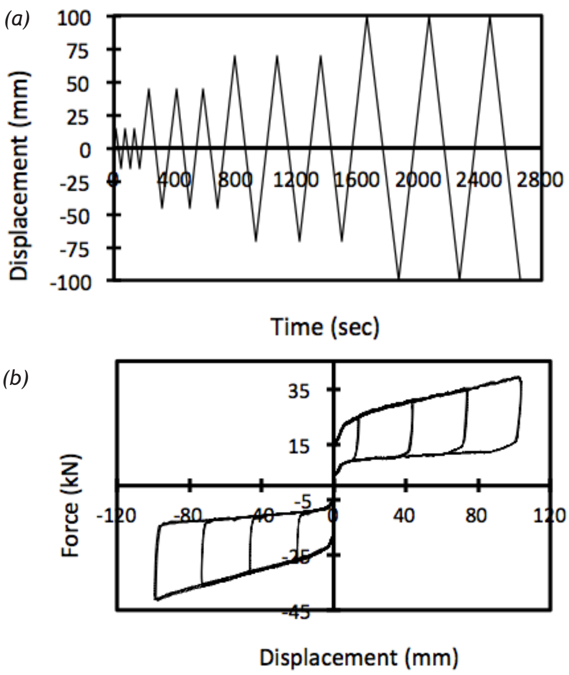

Figure 3: *Test results: (a) Applied load-schedule (b) Load-deformation behaviour.*

3 ROCKING CLT WALLS COUPLED WITH RSF JOINTS

This section describes the concept of rocking CLT walls coupled with RSF joints. The proposed system consists of coupled CLT walls, boundary steel columns and RSF joints as ductile links. The RSF joints connect the walls to the adjacent rocking CLT walls and/or the boundary columns. The steel columns are used to de-couple the perpendicular walls in bi-directional rocking motion. Moreover, they can be used as the gravity load resisting members. As can be seen in Figure 4, the RSF joints are positioned in the notches within the CLT panels. Self-tapping screws can be used to connect the RSF joints to the CLT panels. For the RSF joints connected to a boundary steel column, a welded connection between the flange and the column is considered.

On the brink of rocking, the acting forces on each wall include: the base moment which triggers the movement ($\Sigma F_i h_i$); the sum of RSF ductile link slip forces (ΣFj); and the self-weight of the CLT panel which is the only considered vertical load (W). Taking the moments about the rocking point of each wall, ΣFj can be determined by Equation 3 where n is the number of walls (providing that they are identical), m is the number of stories and F_i is the seismic force applied at each story level.

$$\sum F_j = \frac{1}{nb}\sum_{i=1}^{m} F_i h_i - \frac{W}{2} \qquad (3)$$

The slot length for the RSF ductile links connecting two adjacent walls has to be twice as it is for the RSF ductile links connecting a wall to a boundary column. This is because they are designed to slide in both tension and compression while the RSF joints connecting the walls to the steel columns are only supposed to work in one direction. Accordingly, the slot length can be determined by Equation 4.

$$s = \frac{\Delta b}{h} \qquad (4)$$

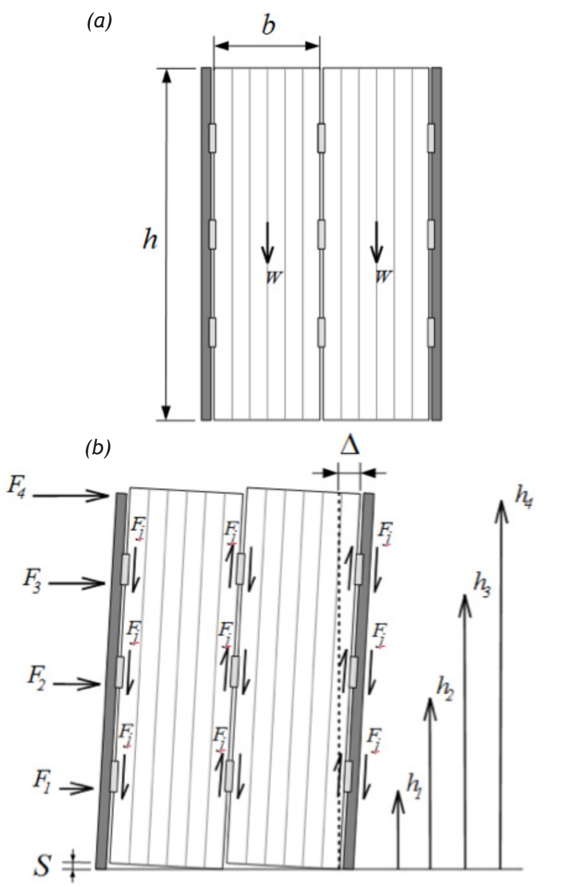

Figure 4: *Rocking CLT walls coupled with RSF joints: (a) Before rocking (c) After rocking.*

4 PROTOTYPE BUILDING

To demonstrate the behaviour of a structure containing the proposed concept, a prototype building is considered. This structures is a four story residential building designed for a shallow soil (type C). The overall height of the building is 12.5m with 3.5m of height for the first story and 3m of height for the other three stories. As can be seen in Figure 5, the building has five 6m long spans in each direction. The building is assumed to be a hybrid frame building with steel hollow section columns, LVL beams and light timber floors. The permanent loads including ceiling, services, light partitions, workstations, floor panels (made with plywood sheets and light plywood box joists), self-weight of the exterior walls, the weight of the beams and the weight of the columns are considered as 1.35 kPa for the first level, 1.25 kPa for levels two to three and 1.1 kPa for the roof. The imposed loads are assumed as 2 kPa for the first level, 1.5 kPa for levels two to three and 0.5 kPa for the roof. The associated seismic masses are 1.56 * 10^5 kg, 1.24 * 10^5 kg and 1* 10^5 kg for the first story, stories two to three and the roof, respectively. Two LLRS each comprised of two CLT walls coupled with four RSF ductile links along the height are considered for both major directions (see Figure 5). The CLT walls are assumed to be similar to the one described in section 2 with a density of 5.3 kN/m^3.

The ULS seismic forces (with a return period of 500 years) were calculated using the Displacement Based Design (DBD) method targeting 2% design drift and soil type C (shallow soil) located in Christchurch, New Zealand. The design resulted in a base shear of 537 kN with seismic forces of 72 kN, 106kN, 155 kN and 205 kN for stories one to four, respectively. These values are obtained based on the assumption that there is essentially a sinusoidal, first mode response at the peak displacement. Thus, the base shear can be distributed in proportion to mass and displacement.

Accordingly, the ULS seismic forces applied to each coupled wall system (including 10% of accidental eccentricity) are 39.5 kN, 58 kN, 85 kN and 111 kN for stories one to four, respectively.

The approach adopted in this study is that the walls should start to move and the RSF joints should start to slide under a ULS earthquake. By following Equation 3, the slip force for the RSF joints is determined is F_{slip} = 103kN. It is assumed that four bolts with 13 Belleville springs per bolt per side were used for each of the centre slotted plates. Note that since the RSF joints are assumed to be double acting joints, each one contains two centre plates (see Figure 1(a)). The slip force was assumed to be almost 30% of the ultimate force. Figure 5(b) presents the designed hysteresis with a maximum deflection of 108mm.

Note that in the structures similar to the one presented here, it should be checked that if the restoring moment produced by the RSF joints and self-weight of the walls is sufficient to overcome the overturning moment caused by the *P-Δ* effect. For this case, the overturning moment for each wall in the prototype building is 706 kNm while the restoring moment caused by the RSF joints and the self-weight of the wall is 1004 kNm. This shows that the RSF joints are capable of re-centring the structure upon unloading.

5 NUMERICAL MODELLING OF THE PROTOTYPE BUILDING

Figure 6 shows the general arrangement of the developed numerical model. The CLT walls are modelled as a 12.5m by 3m shell elements with the material properties related to MSG8 timber (E_L = 8000 MPa). The RSF joints were modelled by "Damper – Friction spring" link element which is available in

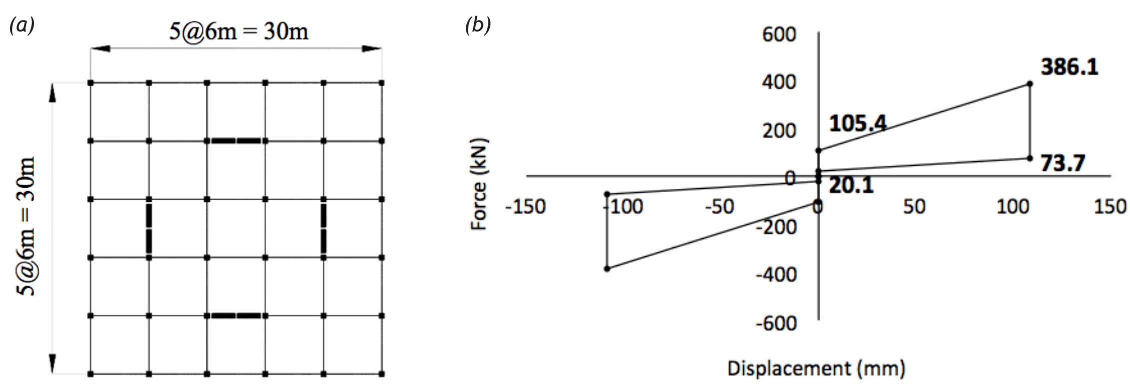

Figure 5: *The prototype building: (a) Plan view (b) RSF ductile link design.*

SAP2000 version 17 and above. Hashemi et al. showed that this link element is able to accurately predict the hysteretic behaviour of the RSF joint (Hashemi et al. 2017). This was confirmed by comparing the RSF joint component test results by the numerical data. The calibrated design parameters are shown in Table 1. Gap elements with the gap set to zero were used at the rocking pivots (at the base of the wall) to represent the foundation level. The boundary steel columns were modelled as steel box members with 400mm * 400mm * 12mm section for the first two stories and 350mm * 350mm * 10mm section for the two upper stories. The shear keys are modelled by restraining the horizontal degree of freedom of one node at the base of each wall. The beams were modelled as 63mm * 400mm timber beams made with LVL13.2. The floors were modelled as 150 mm thick shell members with the material properties specified to represent the assumed light timber flooring system with plywood box joists.

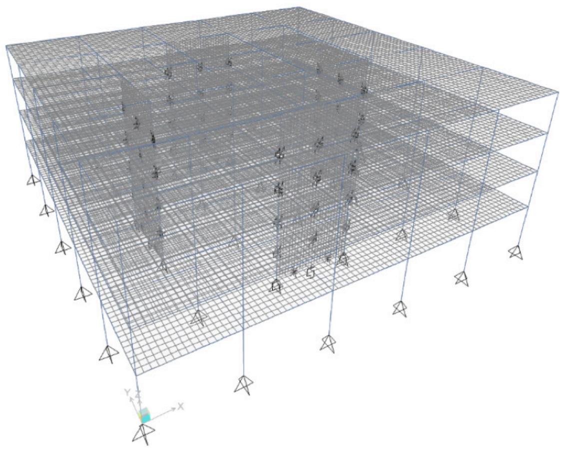

Figure 6: *Numerical model of the prototype building.*

A suite of five ground motions scaled to ULS and MCE (Maximum Credible Earthquake) were selected for non-linear dynamic time-history simulations (10 analysed cases in total). They are scaled to match the Christchurch 500 year return period for ULS and 2500 year return period for MCE. Table 2 tabulates the earthquakes and the scaled peak ground motions.

The records were applied to the model in the two major directions. Figure 7 shows the numerical results. Generally, it can be seen that the obtained peak roof drifts meet the criteria for the maximum lateral drift indicated by the New Zealand standard (2.5% for ULS and 3.75% for MCE). The maximum drift among the ULS events is for Christchurch which is slightly lower than 2.5%. For MCE events, the highest peak roof drift was close to 3.75% which is for Christchurch. From Figure 7(a), it is apparent that all maximum lateral drifts expect for the Christchurch event are under the target DBD design limit.

Figure 7(b) illustrates the peak roof accelerations. It can be seen that the accelerations for the MCE events are relatively higher than those for the ULS events. The highest recorded values are for the Landers event which are 0.7g and 1.1g for ULS and MCE, respectively. During the shake table tests within the SOFIE project, it was concluded that additional energy dissipation devices are required to decrease the earthquake induced accelerations (Ceccotti et al. 2013). The reason was the relatively high accelerations (as high as 3.8g for level 6) recorded for the seven story CLT building. Although it may not be realistic to compare the results of the time-history simulations in this study (as a four story building) with the experimental results from a seven story building, however, it can give a general idea about the efficiency of the proposed concept in terms seismic energy dissipation.

6 CONCLUSIONS

The paper explains the development of a new type lateral load resisting system which includes CLT rocking walls coupled with Resilient Slip Friction (RSF) ductile links and boundary steel columns. This

Table 1: Calibrated design parameters for the numerical model of the RSF joints.

Loading stiffness (N/mm)	Unloading stiffness (N/mm)	Pre-compression displacement (mm)	Stop displacement (mm)
2585	464	40.8	-108

Table 2: Selected seismic events.

Parameter	El Centro	Northridge	Kobe	Landers	Christchurch
PGA (g)	0.31	0.23	0.82	0.28	0.48
Scaled PGA for ULS (g)	0.28	0.27	0.25	0.31	0.28
Scaled PGA for MCE (g)	0.49	0.51	0.50	0.56	0.53

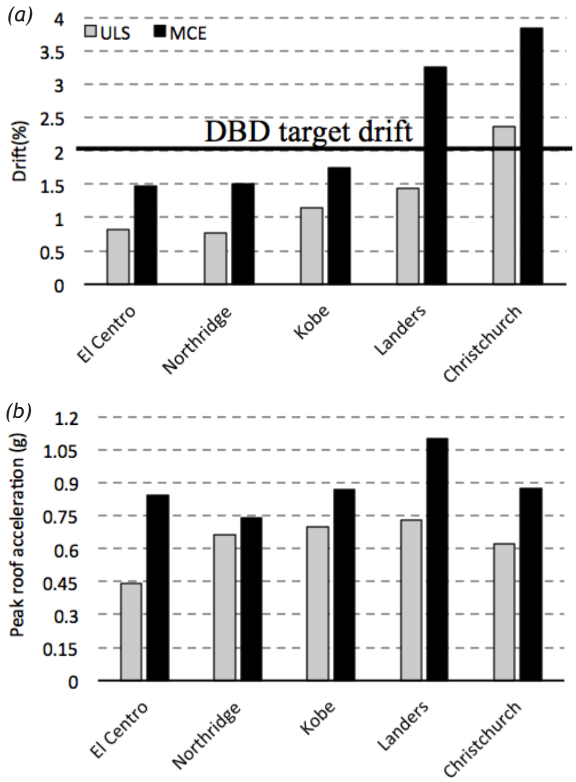

Figure 7: *Numerical results: (a) Maximum lateral drifts (b) Peak roof accelerations*

system provides self-centring in addition to energy dissipation. To evaluate the efficiency of the proposed concept, a four story prototype building was designed based on Displacement Based Design (DBD) procedure. The results of the time-history simulations highlighted the good seismic performance of the proposed concept in terms of maximum lateral drifts and peak roof accelerations. The maximum lateral drifts were well kept under the code-prescribed values. Overall, the results of this preliminary study confirmed that the proposed concept has the merit to be considered as a damage avoidant seismic solution. Further experimental tests may be required to confirm the findings of this study.

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