Seismic resistant cross laminated timber structures using an innovative resilient friction damping system

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ABSTRACT: Multi-storey timber structures are becoming progressively desirable owing to their aesthetic and environmental benefits and to the high strength to weight ratio of timber. A recent trend in timber building industry is toward cross laminated timber (CLT) panelized structures. The shake table tests within the SOFIE project have shown that the CLT buildings constructed with traditional methods can experience high damage especially at the connections which generally consist of hold-down brackets and shear connectors with mechanical fasteners such as nails or bolts. Thus, current construction methods are not recognised as reliable in seismic prone areas. The main objective of this project is to develop a new low damage structural concept using innovative resilient slip friction (RSF) damping devices. The component test results demonstrate the capacity of this novel joint for dissipating earthquake energy as well as self-centring to minimize the damage and the residual drift after a severe event. The application of RSF joints as hold-down connectors for walls were investigated through numerical studies. Moreover, a core wall system comprised of cross laminated timber and RSF connectors is subjected to time-history earthquake simulations. The numerical results exhibit no residual displacement alongside a significant reduction in peak acceleration which can be attributed to significant amount of dissipated seismic energy over the RSF joints within the system.

1 INTRODUCTION

Structural walls offer outstanding seismic resistance compared to other structural systems and are commonly employed as the main lateral resisting system. Structural walls constructed with prefabricated components have additional advantages including offsite fabrication, improved quality and speed of construction. Recently, there has been a tremendous interest towards design and construction of multi-storey timber structures with engineered wood products such as Cross Laminated Timber (CLT) and Laminated Veneer Lumber (LVL) panels. Nevertheless, the application of these products in seismic regions as the primary lateral resisting system was limited by codes firstly because of insufficient technical information about their dynamic behaviour and secondly the non-ductile nature of them which leads to high response accelerations especially in high-rise buildings. During the large-scale experiments of a seven story building made of CLT panels within the SOFIE project, high accelerations with a maximum of 3.8 g in higher floors were recorded (Ceccotti et al. 2013). Despite the fact that these accelerations may be acceptable for human health, they are uncomfortable and displeasing for the habitants. As an important outcome of the SOFIE project, other research studies were started with the aim of developing new seismic solutions for timber industry to absorb the seismic energy, decrease the subsequent response accelerations and minimize the residual damage.

During the PRESS (PREcast Seismic Structural Systems) program in the early 1990's, hybrid wall systems were recognized as relatively efficient lateral load resisting members (Priestley et al. 1999). The tested hybrid system comprised of unbonded post-tensioned tendons with dissipative devices such as grouted longitudinal post-tensioned bars. While the dissipating devices absorb the seismic energy, the post-tensioned tendons provide self-centring behaviour (Palermo et al. 2005). The concept had later been extended to reinforced concrete jointed walls. In such system, two or more single prefabricated walls are connected to each other with special energy dissipative connectors as ductile links along the vertical joints between the adjoining walls. Similar assembly is employed for timber coupled walls. The experimental investigation on coupled post-tensioned LVL walls with internal mild steel bars as the energy dissipater devices demonstrated promising results in terms of ductility and residual displacements (Palermo et al. 2005). Similar systems were developed and tested with external fuse type longitudinal threaded steel rods as the connection between the wall and the base (Smith et al. 2007).

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. 2007). The system had later been experimentally investigated and a design procedure was proposed (Iqbal et al. 2015). The test results confirmed an efficient energy dissipation mechanism over yielding the UFPs during the earthquakes. Sarti et al. tested coupled LVL walls and UFP connectors with fuse type damping devices at the base of the walls (Sarti et al. 2014). Despite the fact that all mentioned wall systems represented relatively superior seismic behaviour compared to traditional systems, however, because the energy absorption is through yielding of metal members, large amount of stiffness degradation occurs in them during and after a sever seismic event. Thus, such system cannot be accredited as a low damage solution.

Loo et al. introduced the application of symmetric slip friction hold-downs for LVL rocking timber shear walls. The symmetric slip friction hold-down offers a rectangular load-deflection curve which means efficient energy dissipation. Furthermore, it provides constant resistance force against overturning moment. The proposed configuration later had been experimentally tested and demonstrated a stable hysteresis with minimum stiffness degradation which is the key characteristic of a low damage system (Loo et al. 2014). Although the energy dissipation mechanism of slip friction connections is one of the most efficient amongst passive damping devices, however, the lack of self-centring in these joints requires the use of an additional system (such as post-tensioned tendons) to bring back the structure to its initial position after an earthquake if the self-weight is not sufficient.

In this paper, a new structural wall system based on a novel friction joint is presented. The components of this joint are formed and arranged so that the self-centring capacity as well as energy absorption is achieved all in one connection system. This new Resilient Slip Friction (RSF) joint invented by Zarnani and Quenneville (Zarnani et al. 2015) could have a considerable impact on the building industry as it is exactly aligned with the high demand for cost-efficient low damage systems.

2 RESILIENT SLIP FRICTION (RSF) JOINT

Figure 1 shows the components and assemblage of the RSF joint. The grooved plates are bolted and clamped in a manner that the centre slotted plates are sandwiched by the cap plates. When the imposed force to the joint overcomes the frictional resistance between the surfaces, the centre plate starts to slide and energy will be dissipated through cycles of sliding. The frictional resistance is a function of the pre-stressed force of the bolts, the coefficient of friction between the surfaces and the angle of the grooves. The specific shape of the grooves along with the use of Belleville washers (or equivalent die springs) and high strength bolts provide the desirable self-centring characteristic.

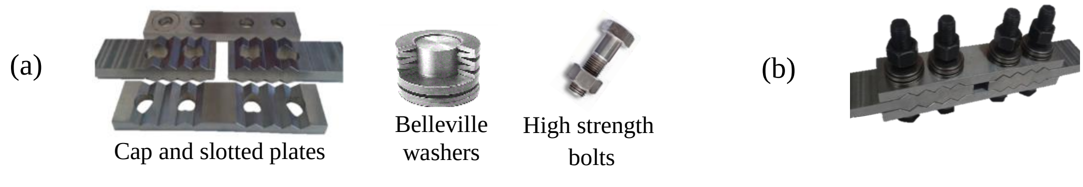
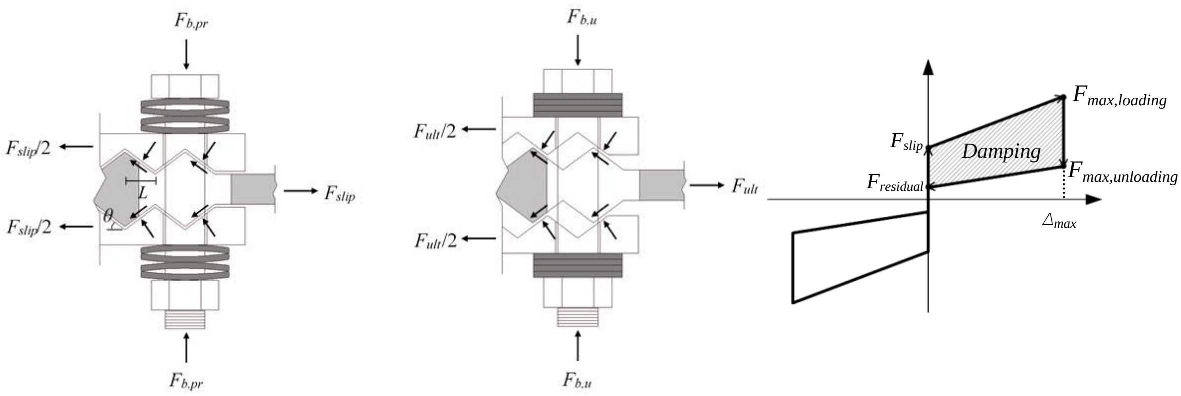


Figure 1 - RSF joint: a) Components; b) Assemblage

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 Belleville washers is larger than the resisting friction force acting between the plates surfaces. Hence, the system is re-centred by the reversing force upon unloading. In addition, the lateral resistance of this new joint is relatively higher than the conventional friction joints for a similar clamping force provided by the high strength bolts. It should be pointed out that Figure 1(a) displays a double acting RSF joint which two centre slotted plates are employed within the connection.

3 RSF JOINT DESIGN PROCEDURE

A design procedure has been developed for the capacity prediction of this new joint based on the free body diagrams shown in Figure 2 (Zarnani et al. 2015). The slip force (F_{slip}) for a symmetric configuration can be determined by Equation 1.



(a) (b) (c)

Figure 2 - Schematic illustration of the symmetric RSF joint: a) Friction plates before slip; b) Friction plates at ultimate deflection; c) Schematic hysteretic loop

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

Where $F_{b,pr}$ is the bolt clamping force as a result of being pre-stressed, n_b is the number of bolts, θ is the angle of the grooves and μ_s is the coefficient of static friction. The residual force in the device at the end of unloading is calculated by Equation 2 where μ_k is the coefficient of kinetic friction which can be assumed as $0.85\mu s$

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

The ultimate capacity at loading ($F_{ult,loading}$) and unloading ($F_{ult,loading}$) can be determined by replacing the μs and $F_{b,pr}$ in Equation 1 and Equation 2 with μ_k and $F_{b,u}$, respectively. The ultimate force on the bolt ($F_{b,u}$) is given by Equation 3 in which k_s and Δ_s are the stiffness of the stack of washers on the bolt and the maximum deflection of them in the fully compressed state, respectively.

$$F_{b,u} = F_{b,pr} + k_s \Delta_s \tag{3}$$

It should be emphasized that in an asymmetric configuration, the friction bolts in the connection need to transfer the applied force through shear and tension. Therefore, they should be designed for both. Refer to (Loo et al. 2014) for more details about the differences between symmetric and asymmetric friction connections.

$$\delta_{\max} = n_j \frac{\Delta_s}{\tan\theta} \tag{4}$$

Where n_j is the number of joints acting in a series (e.g. n_j equals to 1 for a single acting joint and equals to 2 for a double acting connector). The connection exhibits self-centring behaviour providing that the following considerations are taken into account.

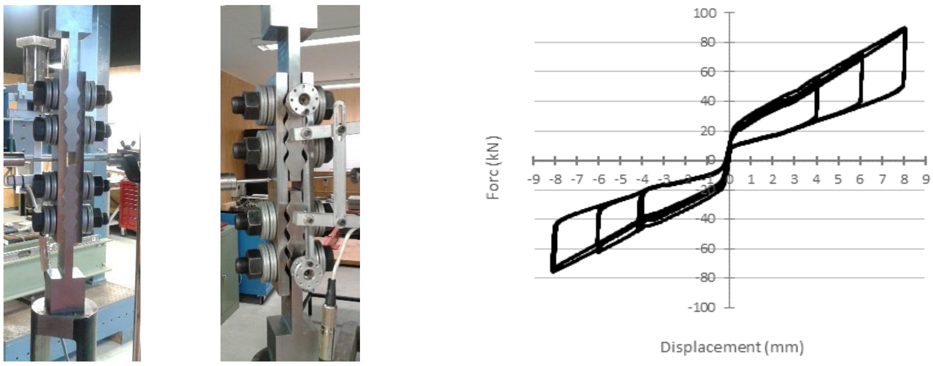
$$\mu_s < \tan\theta \tag{5}$$

$$L > \frac{\Delta_s}{\sin\theta} \tag{6}$$

Where L is the horizontal distance between the top and bottom of the groove.

4 EXPERIMENTAL INVESTIGATION OF THE RSF JOINT CYCLIC BEHAVIOUR

In order to experimentally investigate the hysteretic behaviour of the RSF joint, a double acting symmetric RSF joint was fabricated and tested with a 100 kN Instron Machine. Cap plates were manufactured with mild steel grade 300 and slotted centre plates with bisalloy grade 400. Four high strength bolts with minimum of 830 MPa tensile strength were employed (Two bolts in each side). The stack of Belleville washers on each side of each bolt comprised of four series springs with 38 kN of load at flat position. The pre-stressed force of each bolt ($F_{b,pr}$) was 5 kN. It should be noted that for a double acting RSF joint, the number of bolts in Equation 1 (n_b) refers to the number of bolts in one of the slotted plates which equals to 2 in this case. The stacks of Belleville washers were placed under the nuts of the bolts and the nuts were tightened until the desired pre-stressed force (5 kN) per bolt is achieved. A specific lubricant is used between the cap plates and centre plates to increase the durability of the friction surfaces by controlling the possible galling and rusting. Figure 3 shows the test setup and the experimentally obtained hysteresis for the RSF joint prototype.



(a) (b)

Figure 3 - Experimental test of a double acting RSF joint a) Test setup; b) Hysteresis loops

The flag shape hysteresis in Figure 3(b) demonstrates the fully self-centring behaviour of the system. It shows that the system returned to its initial position after cycles of loading and unloading. The coefficient of friction was obtained as 0.18 for the specified surfaces with the special lubrication. The maximum displacement for this preliminary experiment was 8 mm. Further experimental tests and in-depth numerical simulations are being carried out by the authors at The University of Auckland to further investigate the dynamic behaviour and different applications of the proposed concept.

5 NUMERICAL MODELLING OF CLT WALLS WITH RSF CONNECTIONS

A numerical analysis is carried out in SAP2000 (CSI 2011) to investigate the cyclic behaviour of the CLT rocking walls with double acting symmetric RSF hold-downs. A 6 m by 1.5 m CLT wall with five layers of 40 mm thick MSG8 (Machine Graded Timber – Grade 8) boards is considered (Buchanan 1999). The width of the boards is assumed as 200 mm. The CLT is modelled using layered shell elements to represent the longitudinal and transverse layers. To optimize the efficiency of the wall, the three longitudinal layers have been placed perpendicular to the applied lateral load. Defining a reasonable approach to determine the slip threshold for the rocking timber walls is extremely imperative. In all low damage structural concepts in timber structures, the wooden elements should remain elastic and the elastoplastic behaviour is provided by the connections. In this case, the wall needs to rock before the stress in any of the timber boards within the CLT wall exceeds the allowable elastic tension or compression stresses. CLT is a highly non-uniform engineered wood product which makes is extremely hard to accurately define its lateral resistance by using conventional analytical methods. In order to specify the maximum tolerable force applied to the top of the wall (F_E) before the timber goes beyond the elastic region, a numerical model is developed in ABAQUS (Hibbitt et al. 2014) software package (see Figure 4(b)). The numerical assembly and the normal stress distribution are shown in Figure 4. The density of the timber is considered as 540 kg/m^2. The F_E of 26.5 kN was found from the numerical analysis. In Figure 4(a), the wall is shown flipped over horizontally for better clarity.

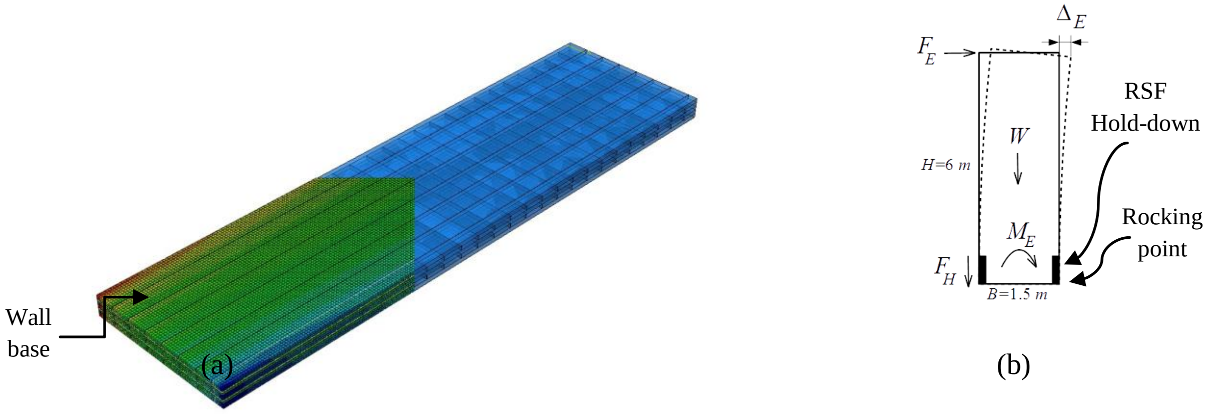


Figure 4 - Numerical modelling of a CLT wall a) Assembly and stress distribution; b) Elastic deflection of the wall

Taking the moments about the rocking point (Figure 5(c)), the force of the hold-down connector (F_H) can be calculated by Equation 7 for a determined F_E.

$$(F_H + W/2)B = F_E H \tag{7}$$

With reference to Figure 2(c) and F_H = 100kN from Equation 7, the maximum load within the RSF connection ($F_{max,loading}$) is determined as 100 kN. The hold-down is designed by following the proposed design procedure in section 3. The calculated design parameters for the Belleville washers (or equivalent die springs) and the sliding plates are presented in Table 1. The slot length should be determined in accordance with the 2.5% of the lateral drift at the top of the wall as the target limit. This limit is recommended by The New Zealand Standard as the reference upper bound limit for ultimate limit state considerations for all buildings (NZS1170.5 2004).

Table 1. Design parameters for the RSF hold-down

n_b	θ	μ_s	Washer Thickness (mm)	Washer Height (mm)	Washer Capacity (kN)	Number of Washers Per bolt	$F_{b,pr}$ (kN)	Slip Distance (mm)
2	15	0.18	6.47	1.75	55	6	27	39.9

From Equation 1 and substituting the μ_s and $F_{b,pr}$ with μ_k and $F_{b,u}$, the RSF joint has a slip threshold of F_{slip} = 50.8 kN and the maximum force of $F_{max,loading}$ = 96.5 kN which is within the acceptable range (less than 100 kN). The clamping force of $F_{b,pr}$ = 27 kN for each bolt is determined to achieve the mentioned sliding and maximum forces. From Equation 2, the residual force at the end of unloading $F_{residual}$ = 9.7 kN is specified. The RSF hold-down is modelled in SAP2000 using a Damper-Friction

Spring link element to represent the proposed hysteretic behaviour (see Figure 2(c)). A gap element is used as well to simulate the foundation level which the wall cannot move below. The displacement loading regime of Figure 5(b) is applied at the top of the wall. A 40 mm slot length is considered for the joint with respect to the target lateral drift and the designed slip distance for the connection. The cyclic behaviour of the system (the applied load at the top vs. base shear) displayed in Figure 5(c) demonstrates that RSF connectors can effectively retain the forces on shear walls below the destined level, thereby protecting them from inelastic damage. The reason is that the force in each of the connections does not exceed $F_{max,loading}$ which is the force that the joint is designed for it. Furthermore, the flag shape hysteretic loops represent the self-centring behaviour as an important performance criterion for RSF joints.

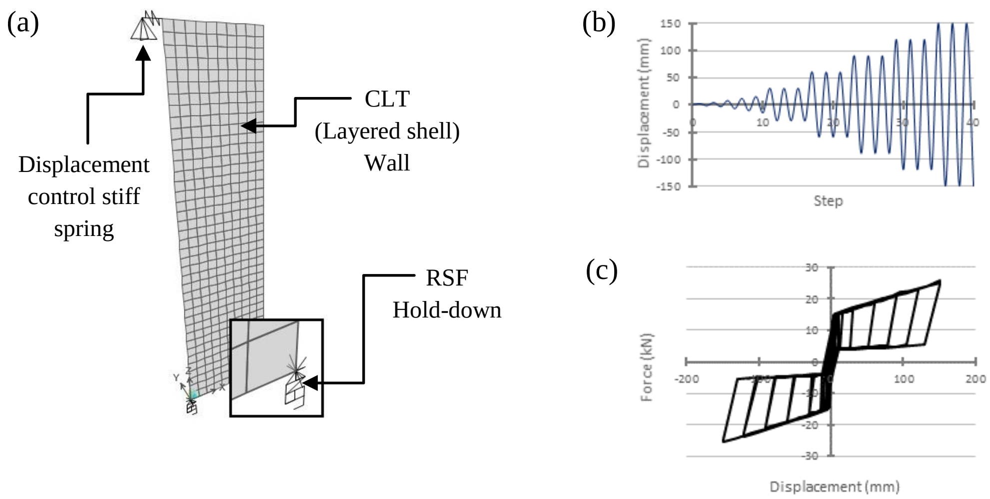


Figure 5 - Numerical analysis of CLT wall with RSF joints a) Numerical model; b) Loading regime; c) Force-displacement loops

6 DYNAMIC TIME-HISTORY ANALYSIS OF CLT CORE SYSTEM WITH RSF JOINTS

Rocking CLT walls with RSF connections can efficiently be extended to core wall applications. Core walls are typically known as the primary lateral load resisting element in the multi-residential buildings. High ductility of the proposed system in conjunction with significant energy absorption rate and self-centring characteristic of the RSF joints make the core capable of effectively mitigating the potential seismic loss and post-earthquake residual damage.

Nonlinear dynamic time-history analyses were carried out on a rocking CLT core wall with RSF connections in SAP2000. The core included two identical wall systems in each direction with each system comprised of two 15 m by 2.5 m CLT walls. The arrangement of the layers and the material properties of the timber boards within the CLT walls are as defined in section 5. RSF hold-downs are used to anchor the walls to the foundation while each wall is connected to the adjacent walls or columns by slotted plate ductile links. These links are meant to transfer the horizontal forces while de-coupling the vertical movement of the walls during rocking. Steel columns are placed at the corners of the core to de-couple the perpendicular walls in bi-directional rocking. Nonlinear layered shell element and Damper-Friction Spring link element are used to model the CLT wall and the RSF joints, respectively. Ductile vertical links are modelled as linear stiff springs with zero stiffness in the vertical direction.

A numerical analysis of a five layer 15 m by 2.5 m CLT wall in ABAQUS shows that the maximum overturning moment providing that all timber boards in the CLT panels stay elastic is M_E = 215 kNm and as a result, $F_{max,loading}$ = 36 kN is found for the RSF joints using Equation 7. Consequently, the design parameters of the RSF joints within the proposed system are determined by following the design procedure introduced in section 3 (see Table 2).

Table 2. Design parameters for the RSF hold-downs in the core system

n_b	θ	μ_s	Washer Thickness (mm)	Washer Height (mm)	Washer Capacity (kN)	Number of Washers Per bolt	$F_{b,pr}$ (kN)	Slip Distance (mm)
2	15	0.18	6.47	1.75	38	8	10	94.6

The slot length for the RSF hold-downs is designed with a 3.75% maximum target drift at the top. It should be noted that the slot length for the vertical ductile links is twice as the hold-down as the connection has to accommodate the corresponding displacements in both upward and downward directions. Four earthquake acceleration records were selected for dynamic time-history loading (PEER 2006). The records were scaled for type D (deep soil) in Christchurch with 500 year return period for Ultimate Limit State (ULS) and 2500 year return period for Maximum Credible Earthquake (MCE). The scale factors were calculated based on numerically determined fundamental period and fundamental frequency of 0.66 seconds and 1.51 Hz, respectively. The scaled factors are presented in Table 3.

Table 3. Considered earthquakes for time-history analysis

Event	El Centro (1940)	Northridge (1994)	Kobe (1995)	Landers (1992)
PGA (g)	0.31	0.23	0.82	0.28
Scale factor (ULS)	1.0	1.6	0.4	1.1
Scale factor (MCE)	1.7	2.9	0.6	2.0

Seismic masses of 7500 kg for the roof and 14000 kg for other four stories were assigned to the structure to represent a five story panelized timber structure. It should be pointed out that in the proposed system, the core is the main lateral load resisting system and the gravity loads are carried by other perimeter CLT walls. The elastic viscous damping of 2% is adopted for all modes of vibration.

Figure 6(a) shows the numerical model of the proposed core system. In Figure 6(b) a possible solution for vertical ductile links is exhibited as the connection can transfer horizontal loads while is free to move upward or downward during the wall's rocking.

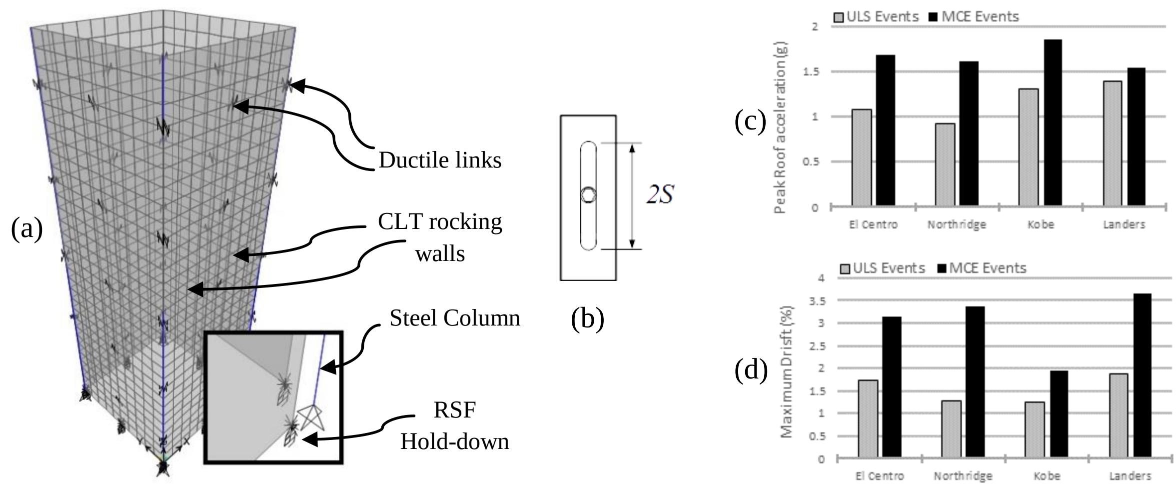


Figure 6 - Numerical analysis of the CLT core with RSF joints a) Numerical model; b) slotted plate ductile link; c) Maximum lateral drifts; d) Peak roof accelerations

The numerical results showed that the structure returned to its initial position at the end of earthquake for all ULS and MCE events. This exhibits the self-centring behaviour which can be attributed to the RSF connections within the system. Figure 6(c) compares the maximum horizontal drift at the roof level for the two limit states. For ULS events, the peak drift is for the Landers event which is less than 2%. It should be noticed that none of the maximum ULS drifts exceeds 2.5% limit indicated by most of the building codes around the world. NZS1170.5 suggests that the deflection limit for 1/2500 annual period of exceedance (MCE events) can be increased to 3.75% (NZS1170.5 2004). For the MCE events, the most significant drift is for the Landers event (3.6%) which is still within the acceptable range.

The peak response accelerations at the roof level are displayed in Figure 6(d). The recorded accelerations are between 0.9 g to 1.8 g for the ULS and MCE events. This should be compared to observed accelerations as high as 3.8 g in the shake table tests of a 7-story CLT building within the SOFIE project (Ceccotti et al. 2013). This significant reduction can be attributed to the large amount of dissipated seismic energy through RSF joints.

7 CONCLUSIONS

Latest research findings have shown that CLT buildings constructed with traditional connections such as nailplates or bolts can experience high damage during and after a severe earthquake. In this paper a new type of low damage CLT wall system with innovative resilient slip friction (RSF) joints is introduced. The component test results in conjunction with numerical cyclic analyses demonstrate the capacity of this system for dissipating seismic energy as well as self-centring behaviour. The numerical time-history seismic analyses on a core system with RSF connections demonstrates the efficiency of the proposed system in terms of ductility, energy dissipation and self-centring behaviour as key factors for providing a low damage seismic design.

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