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DUCTILE CROSS LAMINATED TIMBER (CLT) PLATFORM STRUCTURES WITH PASSIVE DAMPING

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ABSTRACT: Multi-storey platform cross laminated timber (CLT) structures are becoming progressively desirable for engineers and owners. This is because they offer many significant advantages such as speed of fabrication, ease of construction, and excellent strength to weight ratio. With platform construction, stories are fixed together in a way that each floor bears into load bearing walls, therewith creating a platform for the next level. The latest research findings have shown that CLT platform buildings constructed with traditional fasteners can experience a high level of damage especially in those cases where the walls have adopted hold-down brackets and shear connectors with nails, rivets or screws. Thus, the current construction method for platform CLT structures is less than ideal in terms of damage avoidance. The main objective of this study is to develop a low damage platform timber panelised structural system using a new configuration of slip friction devices in lieu of traditional connectors. A numerical model of such a system is developed for a low rise CLT building and then is subjected to reversed cyclic load simulations in order to investigate its seismic performance. The result of these quasi-static simulations demonstrated that the system maintained the strength through numerous cycles of loading and unloading. In addition to this, the system is capable of absorbing significant amount of energy. The findings of this study demonstrate the proposed concept has the potential to be developed as a low damage seismic solution for CLT platform buildings.

KEYWORDS: Cross Laminated Timer, Low damage, Slip friction, Rocking walls, Platform construction.

1 INTRODUCTION

In recent years, Cross Laminated Timber (CLT) has been widely used for different types of buildings such as offices, commercial buildings, public buildings and multi-story residential complexes. In most cases, the platform method of construction is adopted. This method is perfectly suited for low rise to medium rise structures. The term “platform method” derives from the method of construction where the stories are like stacked shoe boxes joined together in a manner that each floor bears into load bearing walls, thereby creating a platform for the next level. The platform method is especially suited to structures which have a cellular plan. Internal wall panels can then be used to contribute to the cellular form and are used as load bearing components in addition to resisting the lateral loads. Typically, vertical loads from the walls and floors are supported by CLT wall panels which are connected to each other and to the floor panels by mechanical fasteners such as nailplates, rivets or screws. Since these panels are also the main lateral load resisting members, extensive research on the seismic behaviour of these structures are being conducted by many research groups around the world.

The most comprehensive experimental research about the seismic performance of CLT platform structures has been conducted within the SOFIE project [1] . That project included quasi-static tests on a single story building with different layouts, shake table tests on a three story CLT building and a series of full scale shake table tests on a seven story CLT building. The results showed that the CLT platform buildings with traditional connections are relatively stiff and can survive destructive seismic events with minimum damage. However, a number of connections (such as nailed hold-downs and nailed shear brackets) failed in bending and some withdrew from the timber elements. Additionally, high response accelerations particularly in the upper levels with a maximum acceleration of 3.8 g were recorded. Accelerations this high obviously have the potential to cause serious injury among the occupants, and it is desirable that a method to reduce them is considered.

Popovski et al. investigated the seismic response of the CLT wall panels of various arrangements and connection layouts [2,3]. It was concluded that these walls have adequate lateral resistance when nails or slender screws are used together with steel brackets. Moreover, the use of hold-downs with nails at the corners of the walls were

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proven to further improve the resistance to overturning from the lateral forces.

Garvic et al. experimentally investigated the cyclic behaviour of single and coupled CLT walls with different connections [4]. The test results confirmed that the layout and design of the connections govern the overall behaviour of the wall. While in-plane deformations of the panels were almost negligible, the observed plastification in the connection parts lead to local failure in the system.

Popovski et al. conducted a series of full scale quasi-static tests on a two story CLT house [5]. No global instability was observed even when the maximum force was reached. Regardless of the rigid connection between the floors and walls, rocking movement of the wall panels was not totally restricted by the floor above.

Yasumura et al. studied the mechanical performance of low-rise CLT structures with large and small panels subjected to reversed cyclic lateral loads [6]. They concluded that in the buildings with small panels, rotation of the panels was the major cause of the total deformation of the building. They also proposed a numerical model to predict the seismic behaviour of such structures.

Regardless of the adequate seismic resistance of the abovementioned CLT platform buildings, in almost all cases, the connections suffered from large inelastic deformation by the end of the earthquake or by the end of the cyclic test [7],[8]. This means that many of these connectors should be repaired or replaced after a major seismic event. This study introduces a new low damage seismic solution for CLT platform structures where traditional connectors are replaced with slip friction devices. This system is capable of dissipating large amount of seismic energy while avoiding inelastic damage to its elements through numerous cycles of loading and unloading. The performance of this system is investigated by quasi-static simulations with reversed cyclic load regimes.

2 ROCKING TIMBER WALLS WITH SLIP FRICTION CONNECTIONS

Passive sliding friction dampers were originally utilized for steel structures. Popov et al. introduced the symmetric slotted bolted connection which dissipates energy through friction while producing equilateral load-deformation loops in tension and compression [9].

Clifton et al. proposed the Asymmetric Sliding Hinge joint for steel moment resisting frames which had non-rectangular yet stable hysteretic behaviour [10]. For the first time in timber structures, Filiatrault utilized the sliding friction devices for timber sheathed shear walls [11]. His studies demonstrated a noticeable improvement in the hysteretic behaviour of the walls compared to traditional timber shear walls. Large amounts of dissipated energy was also observed at various lateral drifts up to a maximum of 1.5%.

Loo et al. investigated the application of slip friction connections as a replacement of traditional hold-downs for timber Laminated Veneer Lumber (LVL) walls [12,13]. Their experiments showed significantly improved seismic performance compared to traditional systems in terms of stability of hysteretic behaviour and residual deflections [16]. Additionally, and most importantly, the timber wall remained in the elastic region after several quasi static and dynamic numerical analyses.

Figure 1 shows a rocking CLT wall with slip friction hold-downs. F_H represents the applied lateral load at the top of the wall and W represents the applied vertical load to the wall including the self-weight of the wall and the gravity loads. Taking the moments about the rocking point of the wall, the slip force of the hold-down (F_{slip}) can be calculated by Equation 1.

$$F_{slip} = F_H \frac{h}{b} - \frac{W}{2} \tag{1}$$

When the applied horizontal load at the top reaches the threshold that the force in the hold-down exceeds F_H, the sliding is commenced in the device and the wall starts to rock. It should be pointed out that the in-plane elastic deflection of a CLT panel is negligible compared to the displacement due to rocking movement. Therefore, the elastic deflection of the wall panels are neglected in this study. In other words, the walls are assumed as rigid bodies during the rocking movement.

Note that since this study seeks to develop a low damage concept, all timber members (CLT panels) and their associated connections must remain in the elastic region. Therefore, the slip threshold for the friction devices (F_{slip}) should be specified in a way that the wall start to rock before any compression or tension failure occurs in the timber boards within the CLT panel.

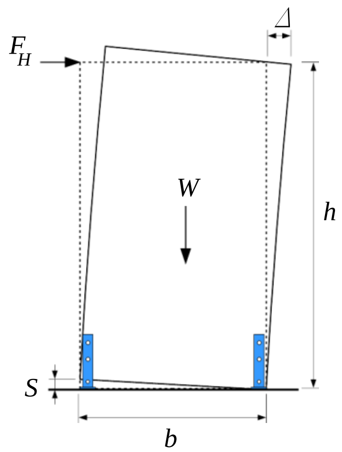

Figure 1: *Rocking wall with slip friction hold-downs*

As shown in Figure 2, slip friction hold-downs are comprised of several components. The centre plate (the wall embedded plate) is rigidly connected to the timber wall with mechanical fasteners. The outer plates clamp the centre slotted plate in a manner that the centre plate is sandwiched by them. When the imposed vertical force to the device overcomes the frictional resistance between the two surfaces, the centre plate starts to slide and energy will be dissipated through cycles of sliding. The slip threshold for a slip friction hold-down can be determined by Equation 2 where μ is the coefficient of friction between the two surfaces, n_b is the number of bolts and T_b is the tension force in each bolt [12].

$$F_{slip} = 2\mu n_b T_b \tag{2}$$

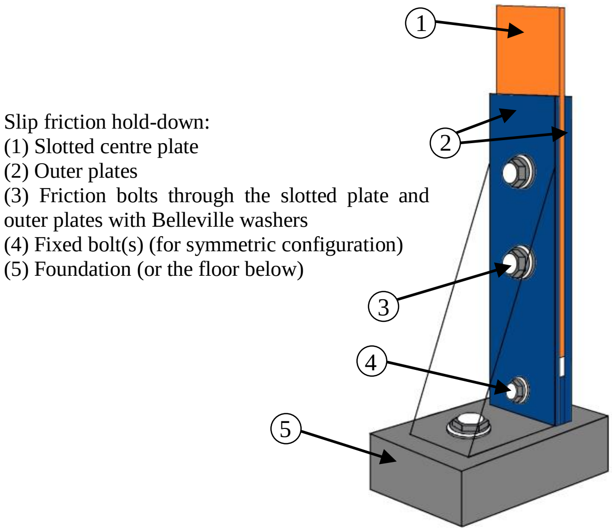

***Figure 2**: Slip friction hold-down connector*

3 PROPOSED SYSTEM FOR CLT PLATFORM CONSTRUCTION

Figure 3 schematically shows the introduced configuration for CLT platform structures. The wall panels are designed to resist both gravity and lateral loads. The proposed concept includes rocking CLT panels with relatively high height to weight ratio to ensure that the dominating deformation mechanism is the rotation of the walls.

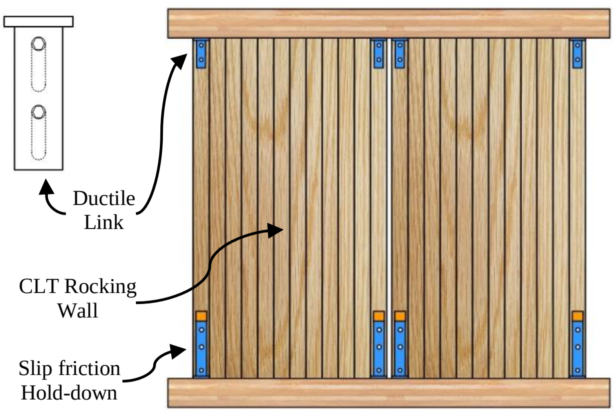

***Figure 3**: CLT panels with slip friction connections*

This configuration allows the individual walls to rock and provides a ductile response. Additionally, pre-defined gaps between the adjacent panels are considered to further increase the ductility of the system as the walls are free to rotate to a certain level before pounding on each other.
Each panel within the system is connected to the floor below (or the foundation in the base level) by slip friction hold-downs. These hold-downs are designed to slide when the induced lateral load at the top (upper floor diaphragm) reached a certain amount. Therefore, the walls are allowed to rock and energy will be dissipated at the joints.
Moreover, a slotted bolted connection is considered at the top of the walls that connects the wall to the upper floor. This connection, which is referred to as a "ductile link", is designed to accommodate the relative vertical displacement between the wall and the floor above while effectively transferring the lateral forces from the upper diaphragm to the wall. Figure 3 shows one possible solution for the ductile link.
During the full scale cyclic test of a two story CLT house with traditional metal connections, Popovski et al. reported that the sliding movement between the walls and floors has the highest contribution to the overall deformation of the structure [5]. This study focuses on the rocking movement of the walls (rather than sliding) and considers it as the dominating deformation mechanism. With this as the objective, a special low damage shear connector should be used between the walls and floors to efficiently transfer the shear forces while it is capable of accommodating the possible uplift caused by the rocking movement. Loo et al. proposed a solution for the shear key and verified its efficiency by experimental tests [13]. Nevertheless, the authors are currently working on different concepts for shear connectors which will be incorporated into the proposed structural system.
Figure 4 displays the deformed shape of the system. Out of plane bending of the floor diaphragm and the embedment of wall panels into the above and below floors allows the wall panels to rock about their corners. This is in agreement with the experimental findings of Popovski et al. [5] and Yasumura et al. [6].

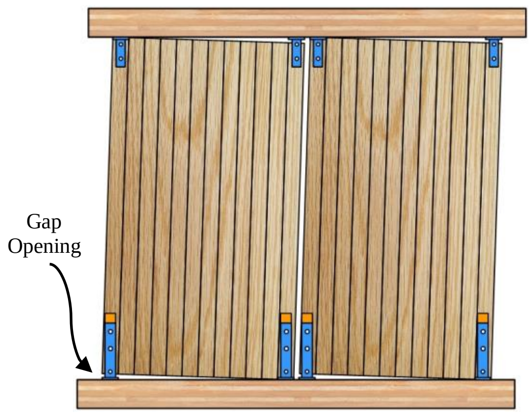

***Figure 4**: Deformed shape of the proposed concept for CLT platform structure with slip friction connections*

In the proposed concept, steel columns are considered at the corners and intersections to de-couple the relative movement of the perpendicular panels due to rocking. This is necessary to avoid the walls bearing on each other in bi-directional rocking (see Figure 5). The column are pin jointed at each floor level.

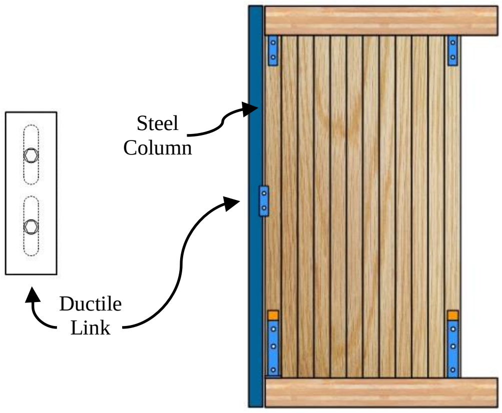

Figure 5: Steel columns at the corners to de-couple the relative movement of the perpendicular rocking walls

The configuration of the connection between the CLT panel and the steel column are similar to the ductile link shown in Figure 3. However, in this case the slotted bolted connection is designed to be able to accommodate the upward and downward displacements caused by the relative rocking movement of the panels in both directions.

4 NUMERICAL MODELING OF A LOW RISE CLT BUILDING WITH SLIP FRICTION CONNECTIONS

4.1 DESCRIPTION OF THE MODEL

The work presented in this section targeted the overall performance of the CLT platform structures with slip friction connections under lateral loads. A numerical model of a two story CLT building similar to the one that has been tested by Popovski et al. [5] is developed in SAP2000 [14]. The model is subjected to quasi-static cyclic horizontal loads in both directions.

The prototype was 6.0 m long and 4.8 m tall with heights of 2.3 m for both stories. A five layer CLT section with 100 mm thickness (20 mm for each layer) is considered for both floors and walls. It should be pointed out that for real CLT structures, the thickness of the floor panels are normally greater than that of the walls to meet the serviceability criteria.

Along the East side of the first story, three rocking walls with 1.6, 1.5 and 0.9 meters width were considered while along the West side, three 1.3 m wide rocking walls were modelled. For the North and South sides of first story, four rocking walls with 2, 1.5, 1 and 1.5 m width were used (from West to East direction in Figure 6).

For the second story, three 2 m wide walls were considered on the North and South sides while three 1.6 m wide walls were modelled for the East and the West sides. In addition to the exterior walls, there were also two 1.6 m width partition walls in the North-South direction in both stories.

Four 0.8 m by 0.8 m window openings were modelled on both the North and South sides at both levels while two window openings with same dimensions were considered on the East and West sides of the second story. Furthermore, the first level had a 2.2 m wide door opening on the West side. Figure 6 shows the plan view of the modelled CLT building.

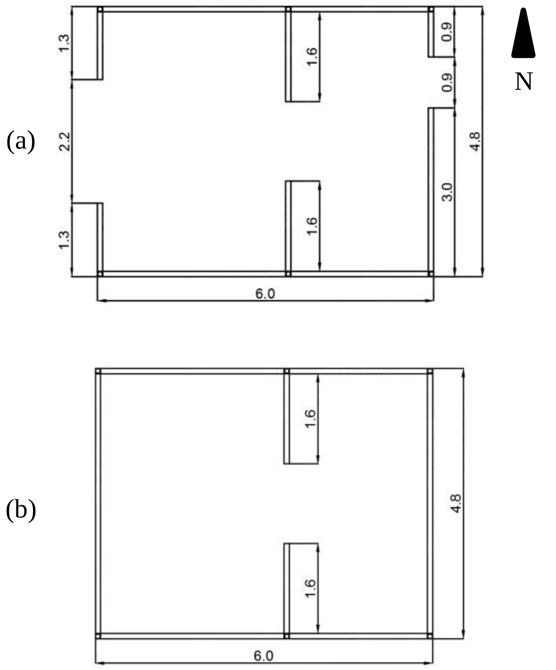

Figure 6: Plan view of the modelled CLT structure [5]*: a) First story b) Second story*

4.2 SLIP THRESHOLD FOR THE SLIP FRICTION HOLD-DOWNS

In this paper, all of the mentioned rocking walls are assumed to be connected to the foundation or the floor below by slip friction hold-downs. Generally, in all low damage timber structural systems, the key point is that timber elements must remain in the "elastic" region and ductile behaviour of the of the system will be provided by the steel connections. These connections can be traditional connections with mechanical fasteners such as nails, rivets and screws or can be more advanced connectors such as slip friction devices (which are highly elasto-plastic). Accordingly, the first step in the design and modelling of CLT rocking walls with slip friction connections is to determine the maximum tolerable lateral force at the top (F_H) which will allow the wall panel to remain in the elastic region both before and after the friction device is activated and the wall starts to rock (see Figure 1).

While there are numerous analytical methods or numerical models for analysing LVL walls (and consequently specifying the lateral load carrying capacity F_H), there is lack of research studies about CLT walls under lateral forces with the focus on the timber boards. This can be mainly attributed to the highly non-uniform composition of CLT.

In this study, a series of finite element analyses were carried out using ABAQUS software package [15] to determine the maximum tolerable F_H for the seven different wall configurations in the CLT house prototype

(0.9 m to 2 m width and 2.3 m height). A five layer CLT panel with three 20 mm thick longitudinal layers and two 20 mm thick transverse layers with 183 mm width for all boards within the panel is assumed for all models. This arrangement of layers represents a conventional configuration for CLT in the New Zealand market. Table 1 shows the assigned mechanical properties to all timber borards within the models.

Table 1: *Material properties of a CLT board (MSG8*)*

E_L** (MPa)	E_R (MPa)	E_T (MPa)	f_c (MPa)	f_t (MPa)
8000	363	363	18	6

* Grade 8 machine stress graded sawn timber [16,17]
** Principal axes of a timber board [16]

A 3.5 kN/m^2 uniform load is assigned to both stories to represent the permanent loads, imposed loads and the self-weight of the CLT members. Owing to the reason that all walls within the CLT building have the same height and thickness, it is assumed that the applied vertical loads in each story is shared between the walls in accordance with their tributary area. Table 2 tabulates the calculated vertical loads for seven different widths for the walls.

Table 2: *Calculated vertical loads for rocking CLT walls*

Story	Wall	Height (m)	Width (m)	W (kN)
Second Story	W0.9	2.3	0.9	5.14
	W1.0	2.3	1.0	5.82
	W1.3	2.3	1.3	7.56
	W1.4	2.3	1.4	8.15
	W1.5	2.3	1.5	8.73
	W1.6	2.3	1.6	9.31
	W2.0	2.3	2.0	11.64
First Story	W0.9	2.3	0.9	11.37
	W1.0	2.3	1.0	11.63
	W1.3	2.3	1.3	15.12
	W1.4	2.3	1.4	16.29
	W1.5	2.3	1.5	17.45
	W1.6	2.3	1.6	18.61
	W2.0	2.3	2.0	23.27

To include the effect of axial loads on the elastic lateral strength of CLT panels, each wall model was analysed with zero axial load in addition to the two calculated axial load limits indicated in Table 1. In each model, the applied lateral force at the top is increased until the normal stress in one of the timber boards exceeds its permissible characteristic stress (f_c and f_t in Table 1).
To optimize the efficiency of CLT wall applications, wall panels have been placed with their outer layers parallel to the gravity loads [18]. Figure 7 illustrates the general arrangement of the developed numerical model for CLT walls. This numerical approach has previously been used by the authors and demonstrated promising results [19],[20]. Note that in Figure 7, the wall is flipped horizontally for better clarity.

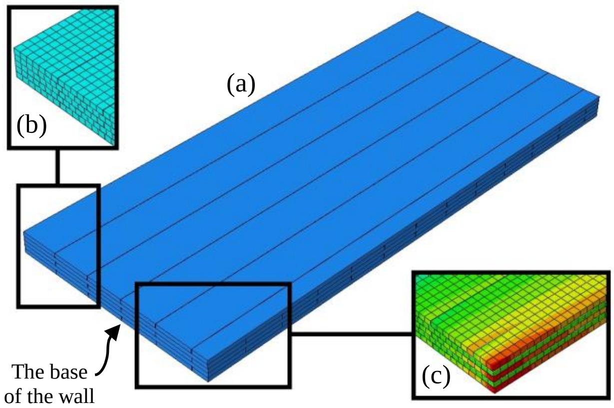

Figure 7: *Numerical model of a CLT wall panel: a) Assembly b) Mesh c) Stress distribution*

The details of the analysed models and the numerically obtained values for maximum tolerable F_H for each model are presented in Table 3.

Table 3: *F_H for CLT wall panels*

Model	Axial Load (kN)	F_H (kN)
W0.9-1	0.00	24.75
W0.9-2	5.14	24.95
W0.9-3	11.37	25.11
W1.0-1	0.00	27.51
W1.0-2	5.82	27.72
W1.0-3	11.63	27.90
W1.3-1	0.00	36.48
W1.3-2	7.56	36.82
W1.3-3	15.12	37.05
W1.4-1	0.00	39.76
W1.4-2	8.15	40.15
W1.4-3	16.29	40.42
W1.5-1	0.00	43.13
W1.5-2	8.73	43.46
W1.5-3	17.45	43.82
W1.6-1	0.00	46.34
W1.6-2	9.31	46.72
W1.6-3	18.61	47.08
W2.0-1	0.00	58.55
W2.0-2	11.64	59.03
W2.0-3	23.27	59.48

The relationship between the width to height ratio of the wall models with different levels of axial loads and F_H is displayed in Figure 8. It can be seen that the effect of the axial load is approximately 2 percent which can be conservatively neglected. Moreover, the results demonstrates a linear relationship between the aspect ratio and the lateral stiffness of the walls.
It should be pointed out that in many of the proposed low damage solutions for CLT construction, the gravity load resisting system is separated from the lateral load resisting system. The results of the presented study shows that the lateral strength of the CLT walls is not much affected by the applied axial loads at least in the investigated range of

the applied load. Furthermore, Figure 8 readily shows that a CLT panel can be replaced with an equivalent material providing that it stays in the elastic region.

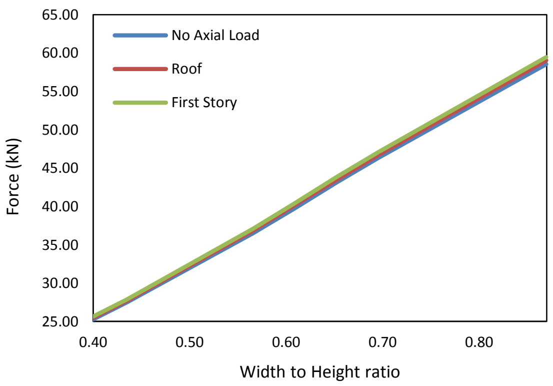

Figure 8: *Width to height ratio against F_H*

Based on the numerically obtained maximum F_H values for each wall (Table 3), the slip threshold for slip friction hold-downs (F_{slip}) is specified by Equation 1 (see Table 4). Note that the resultant forces are multiplied by 0.8 as the coefficient of safety.

Table 4: *Calculated slip thresholds (F_{slip})*

Story	Wall	F_{slip} (kN)
Second Story	W0.9	5.65
	W1.0	7.24
	W1.3	13.47
	W1.4	16.10
	W1.5	19.01
	W1.6	22.07
	W2.0	36.07
First Story	W0.9	3.56
	W1.0	4.91
	W1.3	10.45
	W1.4	12.85
	W1.5	15.52
	W1.6	18.35
	W2.0	31.42

4.3 NON-LINEAR ANALYSIS OF THE CLT BUILDING UNDER CYCLIC LATERAL LOADS

Figure 9 shows the developed numerical model in SAP2000 for the CLT building with slip friction devices. To model the hysteretic behaviour of the slip friction connections, three types of link elements are used. The multilinear plastic link with kinematic hysteretic behaviour is used to represent a bi-directional force-displacement loop without stiffness degradation through cycles of sliding. The gap element is used to restrict the negative vertical displacement in the hold-downs and the hook element is considered to specify the maximum displacement, or slot length, in the connector. The slot length was specified according to the geometry of the corresponding walls and the targeted lateral drift. For the walls with 0.9, 1.0, 1.3, 1.4, 1.5, 1.6 and 2.0 width, slot lengths of 34, 38, 49, 53, 57, 60 and 75 mm were respectively specified to make the walls capable of 3.75% rotation which is recommended by the New Zealand standard as the upper bound limit for a maximum credible earthquake (MCE) [21]. This numerical technique is experimentally validated by Loo et al. [13].

To model the ductile links that connect the rocking CLT wall to the upper floor, an elastic spring link element in addition to a gap element is used. The elastic spring element exhibits the characteristics of a rigid connection in both directions perpendicular to the element except for the vertical (or longitudinal) translational degree of freedom which allows the connections to freely accommodate the vertical movements. The gap element is considered to restrict the movement in the link to the upper floor level. Similar configuration is adopted for the ductile links that connect the CLT walls to the steel columns. The only difference is that the gap is set to the slot length because the link has to be able to accommodate both upward and downward vertical displacements (see Figure 5).

CLT panels are modelled by layered shell element with the indicated material properties in Table 1 that is assigned to each layer according to its angle.

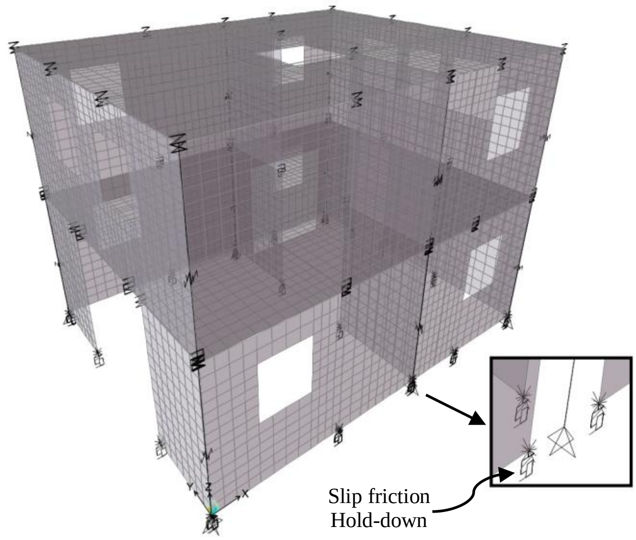

Figure 9*: Numerical model of the two story CLT house with slip friction connections*

It was decided to apply reversed cyclic lateral loads instead of displacements to ensure the inverted triangular distribution of earthquake loads were in conformance with the equivalent static method in NZS1170.5 [22]. From the non-linear pushover analysis, it is found that applying a 555 kN to the second story and half of it (277.5 kN) to the first story in the E-W direction, induces 115 mm deflection at the top of the building which corresponds to 2.5% of lateral drift. Therefore, the reversed cyclic load regime of Figure 9(a) is applied to the top floor in a manner that in each cycle, 50% of the force is applied to the first level. Note that the maximum force was limited to 555 kN for the roof and 277.5 kN for the first floor.

Following a similar procedure, the load regime in Figure 9(b) is separately applied to the building in the N-S direction where the maximum load for the roof and the first level is 608 kN and 304 kN, respectively.

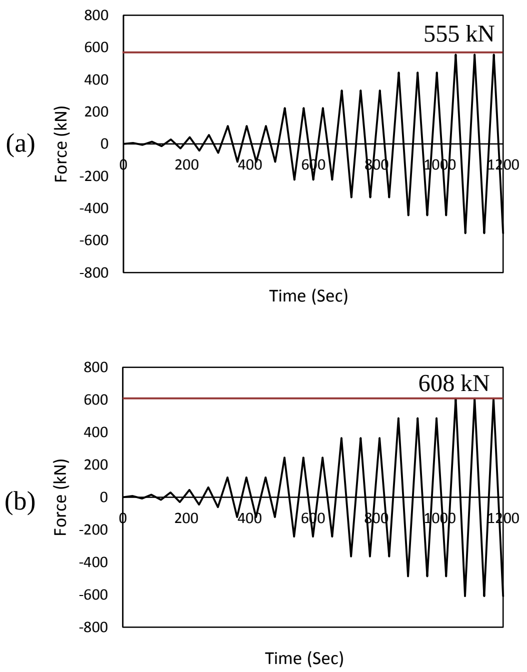

Figure 9*: Load protocol for reversed cyclic loading: a) E-W direction b) N-S direction*

In Figure 11, the displacement at the top of the building is plotted against the base shear in both directions. It can be seen that the initial lateral stiffness of the structure remained almost intact through a large number of load cycles. This can be mainly attributed to the low damage nature imparted by the slip friction devices. The story drift of the roof level was larger than that of the first level which is in agreement with findings of Yasumura et al. during the full scale test of a low rise CLT platform building [6]. It should be emphasized that the system is designed in a way that the rocking movement of the walls is the main source of the horizontal movement of the floors and elastic deformation of the CLT panels is a secondary consideration. The slip between the wall panels and the floors is not considered in this model. However, in order to have a low damage design, a specifically designed type of shear connection should be used to transfer the shear forces from the walls to the floors which tolerates the gap opening due to the rocking motion.

Because of the low damage characteristic of the proposed system, the overall strength is maintained throughout the cyclic tests. This should be compared to the systems with traditional connectors where the total strength is drastically decreased after a few cycles and accordingly the rate of energy dissipation is correspondingly reduced [12].

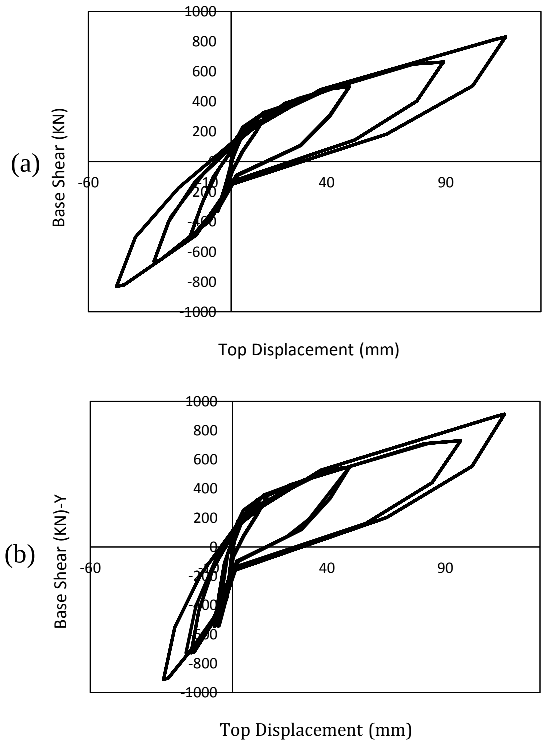

Figure 11*: Cyclic behaviour of the structure: a) E-W direction b) N-S direction*

It can be seen that the force-deformation behaviour of the building is close to a flag-shape one. This means that the system inclines towards self-centring behaviour which is highly influenced by the vertical loads. In other words, to achieve a self-centring system, a balanced relationship between the slip force of the friction devices and the vertical loads is required. If the damper forces considerably exceed the gravity loads, self-centring is less likely to be achieved. On the contrary, much higher vertical loads will restrict the amplitude of sliding in the devices, therefore reducing the absorbed energy.

In this study, the slip forces are determined based on the maximum elastic lateral strength of the CLT panels in order to address the global behaviour of this new structural system. However, for real structures, the slip forces should be designed in a way that the slippage is triggered by Ultimate Limit State (ULS) earthquake loads. This means that the slip friction devices represent rigid connections against wind loads and Serviceability Limit State (SLS) earthquake loads. Consequently, when the building is subjected to ULS seismic forces, they start to slide and energy will be dissipated over the joints while the rocking movement of the panels provides the required ductility for the system.

Although the hysteretic loops in Figure 11 represent the potential for low damage behaviour, further experimental tests are required to confirm the global behaviour of the structure and to investigate the possible failure modes of the system. Despite the fact that the inelastic behaviour is localized in the slip friction devices, other failure modes especially in timber members should be accurately monitored.

5 CONCLUSIONS

Results of the quasi-static analyses on a numerical model of a two story CLT building with slip friction connections are presented in this paper. The objective was to investigate the cyclic behaviour of the model under lateral loads. To determine the slip threshold for the slip friction devices, series of rigorous finite element models were analysed in ABAQUS. The results showed that effect of axial loads on the lateral strength of the CLT panels is less than 2 percent for the investigated range of axial loads.

When a reversed cyclic lateral load regime is applied to the structure instead of displacements to ensure the inverted triangular distribution of earthquake loads in accordance with the New Zealand standard, the results showed that the system maintained its initial lateral strength well through numerous cycles of loading and unloading. Consequently, the system is capable of absorbing significant amount of seismic energy. Further experimental investigations are required to confirm the outcomes of this project.

Overall, the findings of this preliminary numerical study proved that the introduced system has the potential to be developed as a low damage seismic solution for CLT platform structures.

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