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INNOVATIVE HIGH PERFORMANCE SEISMIC RESILIENT TIMBER WALL STRUCTURES WITH LOW DAMAGE FLOOR CONNECTIONS

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ABSTRACT: Rocking timber walls provide superior seismic performance in comparison with conventional light timber structures. Nevertheless, there is an uplift movement at the base of the wall that is translated as vertical displacement and rotation demands at the floor levels. With current conventional approach, not only floors and connections are prone to damage, but also the rocking movement is compromised. Presented in this paper is a new wall-to-floor and beam-to-floor solution for mass timber wall structures that not only transfer the lateral loads but also provide full self-centering while dissipating seismic energy without damage. A new shear key like system including friction dampers is proposed, to both safely allow wall uplift relative to floor and at the same time dissipate energy. Among the advantages of the new system are increased damping capacity of the system, elimination of bulky and expensive fastener connections, mitigation of displacement demands on the structure, reduction in the size and capacity of the hold-downs, and the possibility of reducing the size and number of walls. As a result, the design is more economical and cost-effective, while delivering a high-performance, competitive solution compared to conventional timber structures.

KEYWORDS: Low damage, Rocking wall, Cross Laminated Timber, Damage avoidance, Self-centring, Energy dissipation.

1 INTRODUCTION

There has been an increase in the use of engineered (mass) timber products in structures in the past decade due to the advantages they present over conventional light timber and concrete, such as construction speed, efficiency, sustainability, and reduced seismic loads due to their light weight. Shear walls (including rocking walls) are one of the most economical and efficient Lateral Load Resisting Systems (LLRS) to construct. They have been around for decades and have earned an excellent reputation. It has become increasingly popular among engineers and researchers to employ Cross Laminated Timber (CLT) rocking walls due to their reliability and efficiency in terms of seismic performance. In rocking walls, there is an uplift movement at the base of the wall that is translated as vertical displacement and rotation demands at the floor levels. So far, two general approaches have been taken to deal with these demands. Firstly, the coupled approach where the timber floors (or beams) are rigidly or semi-rigidly connected to the timber walls which are typically plywood shear walls. With this approach, not only floors and connections are prone to damage, but also the rocking movement is compromised. Secondly, the decoupled approach, where the floors are isolated from the walls. The extent of damage is relatively less with the decoupled approach; however, the gravity system is separated, and the wall capacity is not fully utilized. Conventionally, rigid or semi-rigid bracket plates and fasteners are utilized for either of these approaches, where not only the floors and connections are susceptible to damage, but also the overall seismic performance of the system is compromised. To date, there has been no solution to address the current shortcomings. This paper introduces an innovative system that addresses current shortcomings while taking advantage of the wall uplift at the floors to dissipate energy and increase the seismic resistance efficiency of the structure.

1.1 INADEQUACIES OF THE CURRENT WALL-TO-FLOOR DESIGN

To understand the performance of CLT walls in a variety of configurations, numerous numerical and experimental studies have been conducted [1-6]. Furthermore, extensive research and experimental tests have been conducted to assess the performance and failure modes of the conventional wall-to-floor connections [7-9]. Based on all of these studies, it can be concluded that CLT walls exhibit a reliable behavior, remaining intact with minimal damage, while plasticization and non-linearity occurring locally at the point of connections and fasteners (see Figure 1). Additionally, boundary conditions have been found to have a significant impact on the lateral resistance capacity of CLT walls. This includes bottom wall connections, i.e., hold-downs and shear keys, as well as the connections between walls and floors (diaphragms). In experimental tests, there were repeated instances of damage to CLT floor panels as a result of displacement incompatibility between the rocking motion and the

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floor's swaying motion. Experimental and numerical studies using conventional rigid and semi-rigid connections with conventional hold-downs demonstrate pinching hysteresis and stiffness degradation after each cycle. The pinching hysteresis indicates yield and permanent damage to the rigid or semi-rigid connections and hold-downs, resulting in considerable residual drift and leaving the structure vulnerable to aftershocks. Thus, it is not desirable, nor is it consistent with mass timber structure's objective of seismic resilience and stainability. Presented in Figure 2 are the common failure modes associated with these conventional connections.

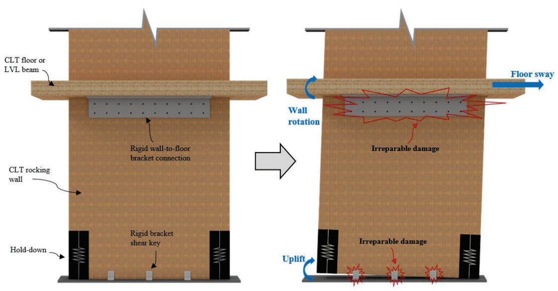

Figure 1. *Rocking wall motion and its interaction with the floor and the rigid connection, leading to irreversible and irreparable damage.*

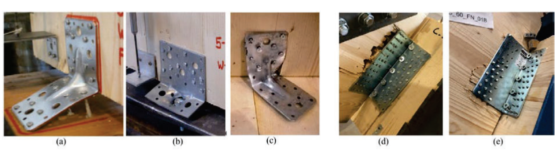

Figure 2. *Connection failure modes: (a) fasteners withdrawal, (b) fasteners head pull-through, (c) metal bracket buckling, (d & e) timber panel brittle failure (tearing).*

2 CONCEPT OF THE PROPOSED SYSTEM

The new concept utilizes the established Resilient Slip Friction Joint (RSFJ) [10] as hold-downs to provide energy dissipation, self-centering, and allow safe rocking movement of the wall. It not only provides the necessary energy dissipation and complete self-centring behaviour, but also additional mechanism such as secondary fuse activation (collapse prevention) to ensure life safety during a major earthquake event. RSFJ consists of grooved cap plates, grooved middle plates, disk springs and pre-stressed bolts or rods. RSFJ is activated when force demand exceeds the slip force (resisting friction force between clamped plates). It is the friction between the grooved plates that dissipates energy as the cap plates slide onto the middle plates, while the pre-stressed bolts and disk springs are compressed together increasing the friction force required to slide the plates, and thus forcing the grooved cap plates to return to their original position, which leads to self-centering behavior. A more detailed description of the RSFJ mechanism can be found in [3, 11].

Experimental testing of a rocking CLT wall with RSFJ hold-downs demonstrated that these joints provide outstanding seismic performance while allowing damage-free deformation and ductility [3]. The RSFJ assembly and parameters of its flag-shaped load-deformation behavior are displayed in Figure 3.

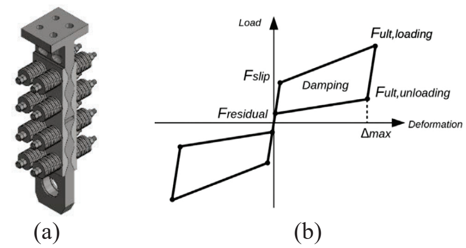

Figure 3: *Resilient Slip Friction Joint (RSFJ): (a) hold-down assembly (b) flagged shaped hysteresis.*

Additionally, slip friction dampers are employed at floor levels to take advantage of the uplift and enhance the damping capacity of the structure. Slip friction dampers are derivatives of symmetric slip-friction dampers introduced by Loo et al. [12]. The symmetric slip friction joint is composed of sliding steel plates clamped together with bolts, Belleville disks, and nuts (no shims are used). Friction dampers dissipate energy through friction between clamped plates, with the inner steel plate incorporating a slot to facilitate free movement of the outer plates without damage while providing Isotropic loop hysteresis behaviour (see Figure 4). The joint was successfully tested in an individual configuration as well as a hold-down for a LVL rocking wall, providing repeatable hysteresis while retaining strength and stiffness [12, 13].

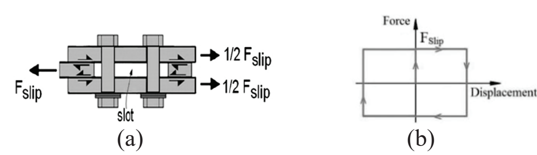

Figure 4: *Symmetric slip friction joint: (a) joint assembly (b) idealized joint isotropic hysteresis*

As an important component of this concept, the innovative shear key developed by Hashemi [3] is incorporated to ensure adequate shear transfer while accommodating the rocking of the CLT wall (Figure 5). It is further proposed to provide a similar shear key at the locations of wall-to-floor to subsequently transfer lateral demands from floor panel (essentially diaphragm) to wall and allow safe and damage free rocking motion. An illustration of the proposed concept assembly is shown in Figure 6. The proposed concept, in summary, utilizes RSFJs as hold-downs of the rocking CLT wall, while friction dampers are utilized at the locations of wall-to-floor, and the innovative shear key is employed at the base of the rocking wall as well as the locations of wall-to-floor.

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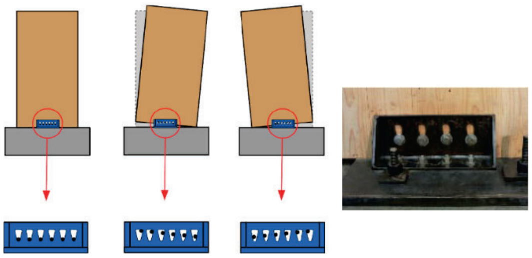


Figure 5: Innovative shear key

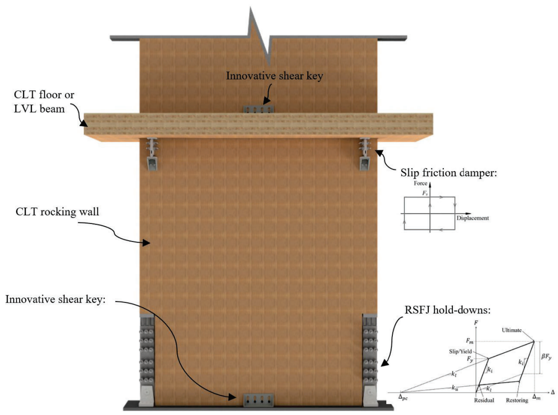
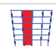


Figure 6: Proposed concept configuration with RSFJ hold-downs and friction dampers at floor connection.

3 METHODOLOGY

Through numerical analyses of case study structures with various number of storeys and configurations, this study provides a comprehensive assessment of the proposed CLT rocking wall system. Table 1 presents a summary of the case study structures and Figure 7 illustrates the assembly of the largest case study structure (seven storey K7-FD) as an example. As a means of highlighting the benefits of friction dampers, numerical analysis of each case study structure was conducted in two configurations, one with friction dampers at floor levels, the other without. It should be noted that the loading considerations are assumed to be for a commercial structure. While the first-floor measures 4.8 meters in height, all subsequent floors measure an equal 3.8 meters in height. Gravity is completely taken by the LVL gravity frame, hence the rocking CLT wall is decoupled from the gravity load resisting frame for the purposes of this study. CLT rocking walls are composed of Machine Stress Graded sawn timber with a modulus of elasticity of 8 GPa (MSG8) in the longitudinal direction and MSG6 in the transverse direction. ETABS software [14] is used for the numerical analysis. Non-linear pushover analyses are carried out to highlight the key performance characteristics of the proposed systems, followed by dynamic time history analyses that provide a wide range of variables suitable for comprehensive analytical study. Dynamic time history analyses are conducted by scaling seven ground motion records in accordance with NZS1170.5 guidelines [15].

Table 1: Summary of the case study structures.

Case study wall structure:	Configuration, Ki	Number of storeys	Structure height (m)
K2 	K2 - FD K2 – R	2	8.6
K3 	K3 - FD K3 – R	3	12.4
K4 	K4 - FD K4 – R	4	16.2
K5 	K5 - FD K5 – R	5	20.0
K6 	K6 - FD K6 – R	6	23.8
K7 	K7 - FD K7 – R	7	27.6
Configuration, Ki	**Number of hold-downs**	**Number of shear keys**	**Number of friction dampers**
K2 - FD K2 – R	2	2	4 -
K3 - FD K3 – R	2	3	6 -
K4 - FD K4 – R	2	4	8 -
K5 - FD K5 – R	2	5	10 -
K6 - FD K6 – R	2	6	12 -
K7 - FD K7 – R	2	7	14 -

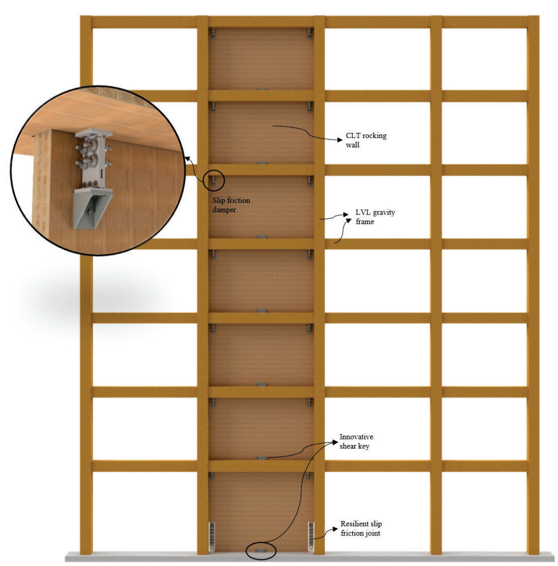


Figure 7: Assembly of the case study structure (seven story K7-FD)

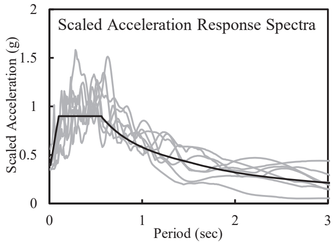


Figure 8: *Scaled ground motion accelerations*

The design envisages that there to be notches at both the bottom corners of the walls to fit the RSFJ hold downs, therefore, the rocking wall lever arm is from the rocking toe to the opposite RSFJ hold down. In practice, the RSFJ hold downs will be equipped with spherical swivel bearings at their base to allow rotation in line with the rocking wall. Thus, a pinned boundary condition is assigned to the bottom of the link elements representing the RSFJs in the numerical model. RFSJ hold downs are modelled using "Damper-friction spring" and slip friction dampers are modelled using "multilinear plastic" link elements in ETABS. The validity of the modelling techniques has already been confirmed by several studies [6, 16, 17].

4 PROPOSED SYSTEM PERFORMANCE

It is appropriate to use non-linear pushover analysis for this study since the case study structures are regular low- to mid-rise structures and the first mode of vibration is the governing mode (fundamental mode). In order to determine the optimal friction damper slip force for each case study structure, nonlinear pushover analysis iterations (tuning) were conducted. Each system was optimized by minimizing the residual force-displacement at the end of every quadrant cycle while maximising energy dissipation (damping). When the friction damper slip force is low, even though complete self-centering is achieved without any residual force, the flag-shaped hysteresis is narrow, and it is possible to increase the damping capacity of the system by increasing the slip force (capacity) of the friction damper in order to enlarge the hysteresis area. When the slip force of the friction damper is high, considerable residual displacement and force are experienced, which most likely result in significant residual deformation after an earthquake.

Hysteretic damping ($\xi_{hysteresis}$) is calculated via Jacobsen's simplified method (hysteresis area) [18] with the provided backbone curves from cyclic pushover analyses. In Figure 9, the load-deformation curve (hysteresis) of the system is shown for cases with friction dampers (Ki-FD series) versus cases without friction dampers (Ki-R series). Non-linear pushover results show that hysteretic damping capacity ($\xi_{hysteresis}$) has increased by 7% on average with the implementation of tuned friction dampers. Increases in damping capacity is correlated with an increase in stories, or in other words, an increase in friction dampers.

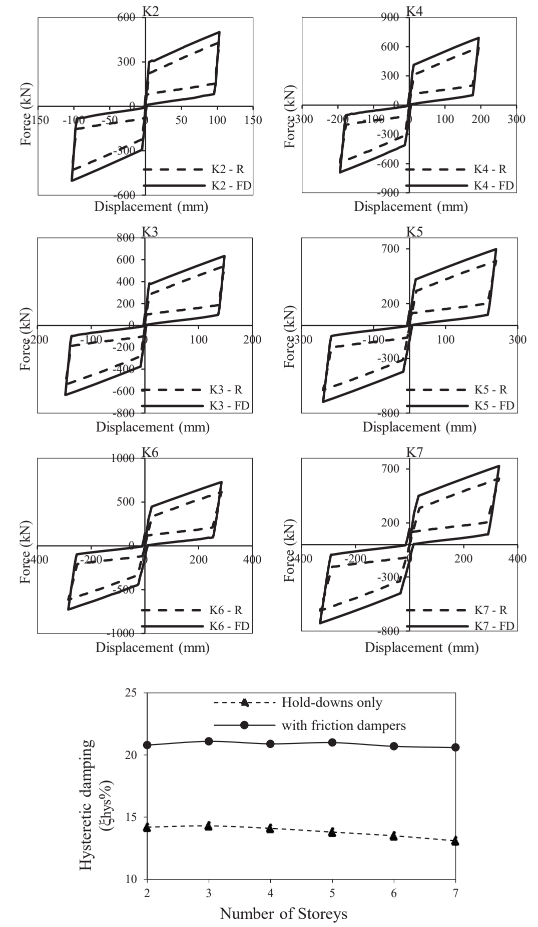


Figure 9: *Comparison of system load-deformation curves of cases with and without friction dampers.*

Dynamic time history offers not only a wide array of variables for this study, but also incorporates all modes of vibration, thereby capturing any dynamic effects that may have been overlooked in nonlinear pushover analysis. Furthermore, dynamic time history appropriately accommodates the non-linearity of members, links (dampers), and boundary conditions such as gaps with their respective inherent and hysteretic damping. For each case study "mean of seven" approach [19] is used as a means of interpreting the results. One of the most noticeable advantages of the proposed system is the reduction in displacement demands on the structure (as an important index); where roof drifts have been significantly reduced by an average of 35% (see Figure 10). It is observed that the greatest reduction in roof drift is about 65%, while the least reduction is about 10%. Based on these results, it is evident that the proposed system is effective in curtailing lateral displacement demands under a variety of cases.

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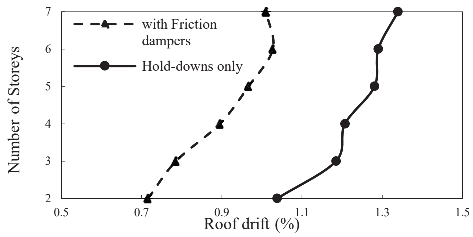

Figure 10: *Performance of the case study structures via dynamic time history analyses, roof drift comparison.*

Seismic force demands (base shear) have been reduced by about 15% on average. This decrease becomes more significant with taller structures, demonstrating an indication of the potential benefit of the proposed system on mitigating higher mode effects which have been of concern with tall wall structures [20, 21]. While complete self-centering behavior is observed in all cases, the proposed system appeared to be more effective in controlling residual drifts, especially for taller walls (see Figure 11). This provides further confidence in its likely performance during a major event in which ground motions exceeding the design level are experienced, developing force and displacement demands exceeding the design levels due to dynamic amplification caused by higher mode effects. All residual drifts fall well under the permissible residual drifts levels suggested by McCormick et al. [22] where 0.5% residual drift is a suitable threshold, after which the structure requires repairs to ensure structural soundness. Combined with RSFJ hold-downs, the concerns regarding self-centring of friction dampers, discussed in the previous section, are rendered irrelevant.

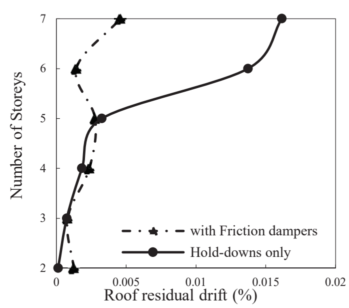

Figure 11: *Performance of the case study structures via dynamic time history analyses, roof residual drift comparison.*

Consequently, there is a reduction in both the force and displacement demands of the hold-downs, resulting in smaller hold-downs required. The reduction in force and displacement demands of the hold-downs is substantial, where force demand have been significantly reduced by an average of 40%, and displacement demand have been reduced by an average of 25%. Similar to the system, the hold-down force demand reduction becomes more evident with taller structures, validating that the presence of friction dampers helps control lateral demand, enhances lateral force distribution, and mitigates higher mode effects. These reductions entail a considerable decrease in capacity, size, and cost of the hold-downs. Figure 12 presents comparison of the hold-down load-deformation (hysteresis) obtained for El Centro (1940) ground motion. Furthermore, it is perceived that by utilizing smaller hold-downs, the stresses induced to critical points of CLT rocking walls can be reduced by approximately 20%. It is therefore possible to reduce the number of layers in the CLT wall or reduce the manufacturing grade of the CLT.

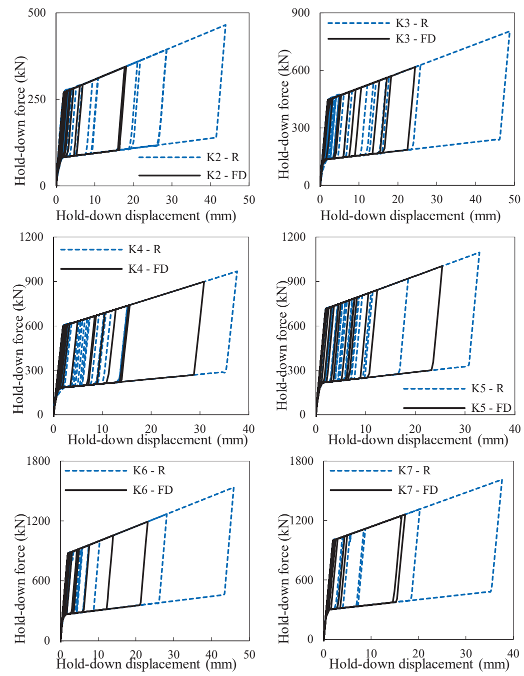

Figure 12: *Comparison of critical hold-down load-deformation (hysteresis) of the case study structures for El Centro (1940) ground motion.*

5 CONCLUSIONS

A new wall-to-floor and wall-to-beam system is presented in this paper that eliminates bulky rigid connections and addresses the shortcomings of existing conventional methods. A new shear key-like system, incorporating friction dampers, has been proposed to allow safe wall uplift, while dissipating energy at the same time. A numerical investigation of the proposed concept was conducted by selecting seven case study structures. By implementing friction dampers, the hysteretic damping capacity ($\xi_{hysteresis}$) has increased by 7% on average. According to the results of dynamic time history analyses, the most notable advantage of the proposed system is the reduction in displacement and force demand by about 35% and 15%, respectively. Furthermore, the force and displacement demand of the hold-downs has reduced noticeably by 40% and 25% respectively, leading to smaller capacity, reduced size, and more affordable hold-downs. The result is a more economical and cost-effective design as a whole, while delivering a high-performance, competitive solution compared to conventional timber structures. The cost of friction dampers can be offset by eliminating the conventional bracket connections and reducing the number of fasteners.

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