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Investigation of Post-Tensioned CLT Core-Wall with Steel Link Beams for a Lateral Load Resisting System

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ABSTRACT

Recently completed research on C-shaped post-tensioned (PT) timber core-walls, using the Pres-Lam system, has demonstrated their lateral load resisting capability and provided data for the calibration of numerical models. Additionally, coupled CLT shear walls with steel link beams have been cyclically tested and exhibited a ductile failure mode through yielding of its link beams between adjacent CLT wall piers. This study first provides a summary of the two high-capacity mass timber wall systems, then proposes their combination as a PT core-wall assembly for applications around elevator shafts or stair cores to resist earthquake loading. The proposed system uses two C-shaped core-walls which are coupled in their weak-axis by a series of steel link beams over the system's height.

A 6-storey sample building was designed following the equivalent static force procedure according to the National Building Code of Canada 2015 with assumed factors of $R_d = 3.0$ and $R_o = 1.5$. A nonlinear model of the system was created and calibrated based on the results of system-level experiments on C-shaped PT and beam-coupled walls. Nonlinear pushover analyses were completed in each orthogonal direction of the system and its seismic performance was assessed using the capacity demand diagram method. Based on this assessment method, global drifts of 0.67% and 1.0% were predicted for a ULS earthquake event in the C-shaped and beam-coupled wall directions, respectively. Although limited, the results of this study indicate the proposed core-wall system concept is feasible and with further study (such as nonlinear time history analyses and evaluation of increased building heights) may be a viable LLRS for mid-rise mass timber buildings in seismically active regions.

Keywords: Post-Tensioned Timber, steel link beams, mass timber, lateral load resisting systems

INTRODUCTION

Cross-laminated timber (CLT) is a mass timber panel product with significant growth forecasted throughout the 2020s [1]. Early adoption of CLT shear wall systems as lateral load resisting systems (LLRSs) has focused on the implementation of connection systems adapted from light timber frame (LTF) construction. Currently, the Canadian and American building codes allow the use CLT shear walls in LLRSs and provide seismic design factors for the equivalent static force procedure (ESFP) [2]–[4]. While the adoption of prescriptive code provisions is a significant step toward increased use of CLT shear walls, their codified configurations are unable to meet the strength and stiffness demands of mid-rise structures in seismically-active regions. Therefore, the LLRSs of tall mass timber buildings most commonly use steel braced frames or reinforced concrete (RC) core-walls.

Due to the high in-plane strength and stiffness of CLT panels, their connections significantly influence the system's strength and stiffness. High-strength hold-downs and panel-to-panel connection systems have been the focus of recent research [5], [6]. Additionally, alternative system configurations such as flanged core-walls or beam-coupled walls can provide greater LLRS capacities. The recently completed Catalyst building is believed to be the first example of a CLT core-wall in using self-tapping screw connections of with CLT panels and ductile hold-downs [7].

This study introduces a new type of CLT core-wall LLRS combining recent research in post-tensioned (PT) CLT C-shaped core-wall and coupled CLT shear walls with steel link beams. A brief background is first provided for the two system types, then a prototype building is designed using the ESFP and an assumed ductility factor. Finally, nonlinear pushover analyses are completed to predict the system's global drift ratio in each orthogonal direction for a ULS earthquake event.

BACKGROUND

Post-Tensioned CLT Core-Walls

Adopting concepts and principles originally developed for precast RC construction [8], PT timber systems, also called Pres-Lam technology (from prestressed laminated timber), have been developed and researched since 2005 at the University of Canterbury (UC) [9]. In PT timber shear walls, conventional hold-downs are not used and instead the moment capacity at the wall base is provided by the clamping action of the PT tendons and/or by special ductile "hold-downs", consisting of axially loaded internally epoxied or external and replaceable rebars/dissipaters [10]. One of the outputs from the extensive research at UC was the Pres-Lam design guide [11], which summarized the research and provided guidance on both force-based and displacement-based design methodologies. Within this work, the viability of PT timber core-wall systems was first investigated by Dunbar et al. [12] through experimental testing. Subsequently, there has been a growing interest in the Pres-Lam systems in North America [13], [14].

Most recently, PT C-shaped CLT core-walls have been investigated at UC [15]. Through this research, the contribution of the flange walls to the core-wall strength and stiffness was quantified both experimentally and analytically based on the connection details between the CLT wall panels [16]. A total of 17 wall tests were performed: 4 PT single wall tests, 5 PT coupled wall tests, 7 PT core-wall tests and one conventional core-wall test. This staged testing quantified the increased lateral strength and stiffness contribution due to each component (CLT wall and joint). The wall specimens replicated a four storey structure at 2/3 scale with a total height of 8.6m. Key test variations among the specimen types included: screwed connection type between the CLT panels, initial post-tensioning force, and others further described within [17]. The un-bonded PT bars provided strong and stiff elastic base connections with recentering capability while the screwed connections provided the primary source of energy dissipation as ductile yielding fasteners between the CLT wall panels at the orthogonal and in-plane panel-to-panel joints. The core-wall specimen test-up and screw yielding failure mode are shown in Figure 1.

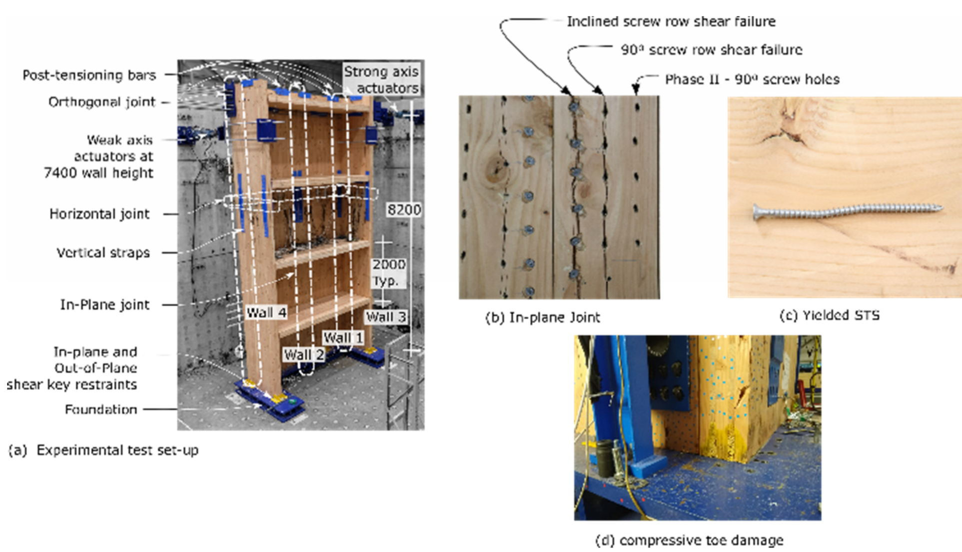


Figure 1: PT Core-wall system test set-up and failure photos adopted from [15], [16]

Brown et al. [18] extended the analytical MMBA method presented in the Pres-Lam design guide [11] to be applicable for in-plane PT CLT single, and coupled shear walls, which quantified a suitable and unique stiffness reduction factor for the compressive toe to account for the increased compressive toe strain variability due to the increased material inhomogeneity of

CLT with non-edge glued lamellas. To accurately capture the nonlinear behavior of the screwed connections, the model proposed by Foschi [19] was used to capture the behavior of the in-plane and orthogonal wall panel joints. The MMBA model was then further extended for PT timber core-walls connected primarily with self-tapping screws to capture the three unique kinematic rocking mechanisms which can occur [16]. Depending on the relative stiffness of the screwed connections to the PT and energy dissipating devices, the model considered a staged kinematic response.

Figure 2a shows the experimental result of the final phase of experimental PT core-wall testing (specimen CW-6) and analytical model. In this experimental test, the kinematic rocking response where flange and web wall uplift occur simultaneously, denoted as high composite action (HCA) in Figure 2a, and flange wall uplift only, denoted as medium composite action (MCA) in Figure 2a, was captured with the analytical model. During testing, the kinematic rocking mode changed as the in-plane screwed connection stiffness decreased due to its nonlinear behavior and cyclic degradation; the elastic post-tensioning bars did not sustain any damage. Figure 2b shows the curve fitting of the experimental screwed connection test (named 16X+16S) reported by Brown et al. [20] with a nonlinear curve fit to the model by Foschi [19]. The screwed connections between CLT panel joints were implemented into the extended iterative MMBA analysis as nonlinear springs to model the behavior of CLT core-walls.

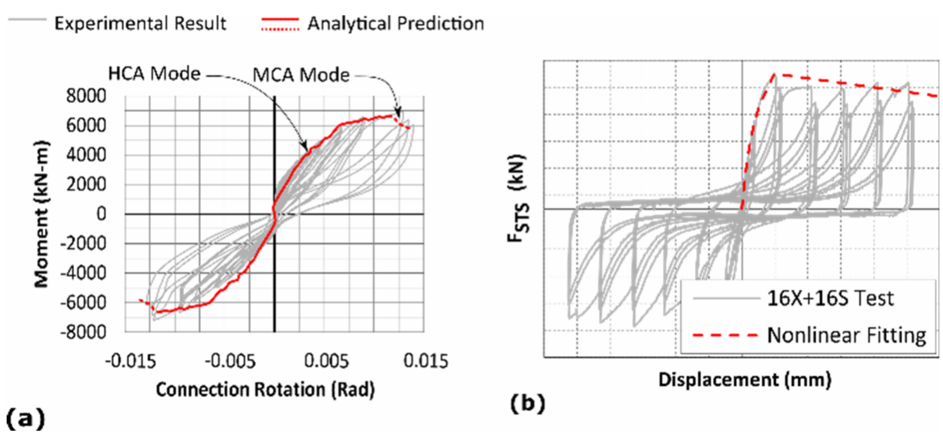


Figure 2:(a) PT Core-wall experimental test with analytical model, (b) calibration of non-linear screwed connection model adopted from Brown et al. [16]

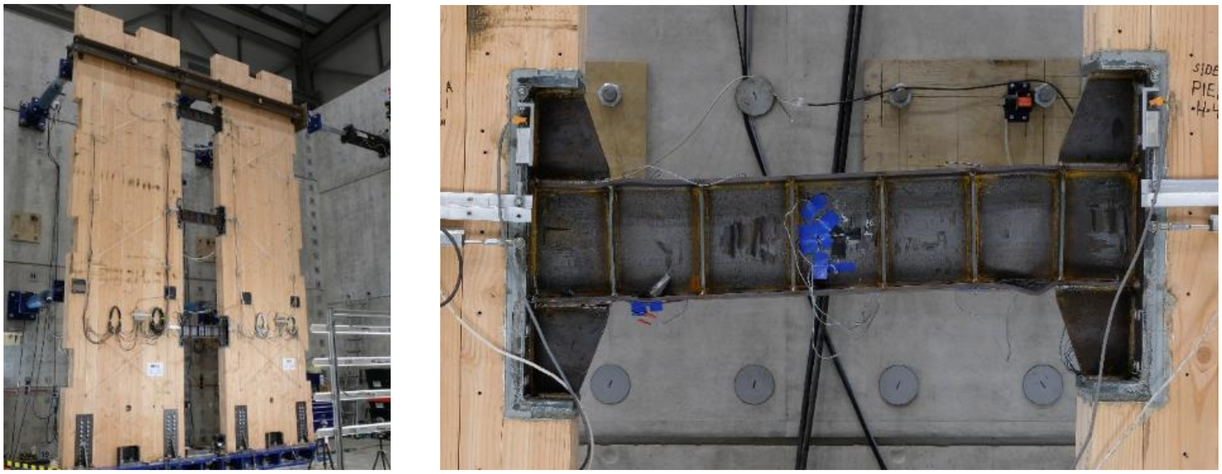
Coupled CLT Shear Walls with Steel Link Beams

Similar in concept to RC coupled walls, hybrid coupled CLT shear walls can be created with ductile steel link beams between adjacent CLT wall piers. This type of coupled wall configuration presents several advantages, including the ability to fit within architectural plans with regular doors or openings, enhanced energy dissipation, reduced foundation demands, and greater lateral stiffness.

The steel link beams between CLT wall piers can be detailed to ensure a ductile failure mode according to design provisions for the links in eccentrically braced frames, which are specified in most national steel design standards [21], [22]. The length and section properties of the steel link beams determine whether their failure mode is shear or flexure-controlled. Shear-controlled link beams are structurally advantageous due to their greater deformation capacities [23], however, they may be impractical for large openings between CLT wall piers.

The connection between the steel link beam and CLT wall pier is critical to ensuring adequate seismic performance of the system and many connection types are possible. Zhang et al. [24] experimentally investigated one type of connection using embedded steel beams in the side of a CLT panel and found that when adequate embedment was provided, failure of the steel link beam was achieved. Self-drilling dowel groups [25], bolted end plates [5], and screwed end plates were also cyclically tested and demonstrated their feasibility for providing connections with adequate strength and stiffness in a coupled CLT shear wall system.

Cyclic testing of a 2/3-scale, 3-storey coupled CLT shear wall (Figure 3a) validated the system concept and exhibited a ductile failure mode. 200mm-deep wide flange link beams were installed between 205mm-thick, 5-ply CLT wall piers. Mixed angle screw hold-downs and steel shear keys formed the base connections. The specimen was cyclically tested to failure and it was observed that damage in the system was concentrated in the steel link beams (Figure 3b), CLT compression toes, and hold-down screws. The hysteretic behavior of the specimen is shown in Figure 4.



(a) (b)

Figure 3: Large-scale testing of a coupled CLT shear wall specimen with steel link beams: (a) test setup and (b) representative damage to steel link beam.

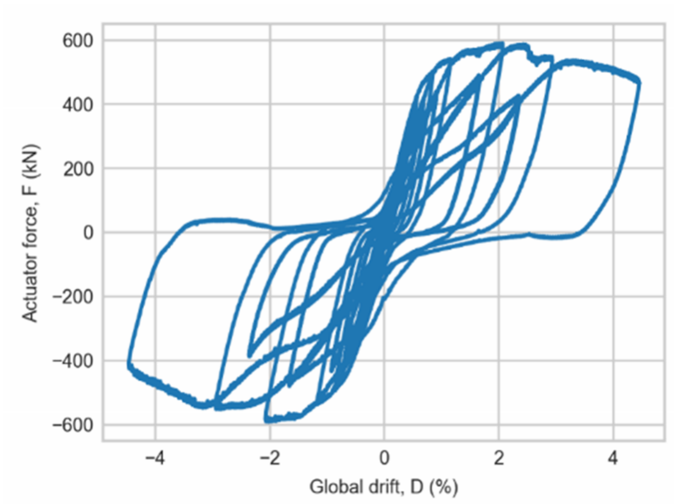


Figure 4: Hysteresis plot of cyclic test on coupled CLT shear wall.

SYSTEM CONCEPT AND DESIGN

System Concept and Sample Building

Based on the previously described research, a hybrid LLRS can be created with PT C-shaped CLT core-walls connected at their flange walls with steel links beams. The core-wall can dissipate energy through ductile fastener failure between adjacent CLT wall panels and in the steel link beams, which provide coupling action in the weak-axis of the two C-shaped assemblies. Furthermore, the vertical PT rods through the CLT panels provide hold-down forces and self-centering capability to the core-wall.

Figure 5a provides a floor plan for a sample building located in Vancouver, Canada with a typical storey height of 3.6m. A schematic of the proposed LLRS is shown in Figure 5b. To evaluate the feasibility of the proposed LLRS, the prototype building was designed using the ESFP according to the NBCC 2015 [3]. The horizontal design actions were determined assuming a normal importance factor (I_E = 1.0) and soil site class C (defined as *very dense soil and soft rock*). A flexible diaphragm approach was assumed to distribute the typical seismic weights of 3kPa and 2kPa assigned to the floor and roof, respectively.

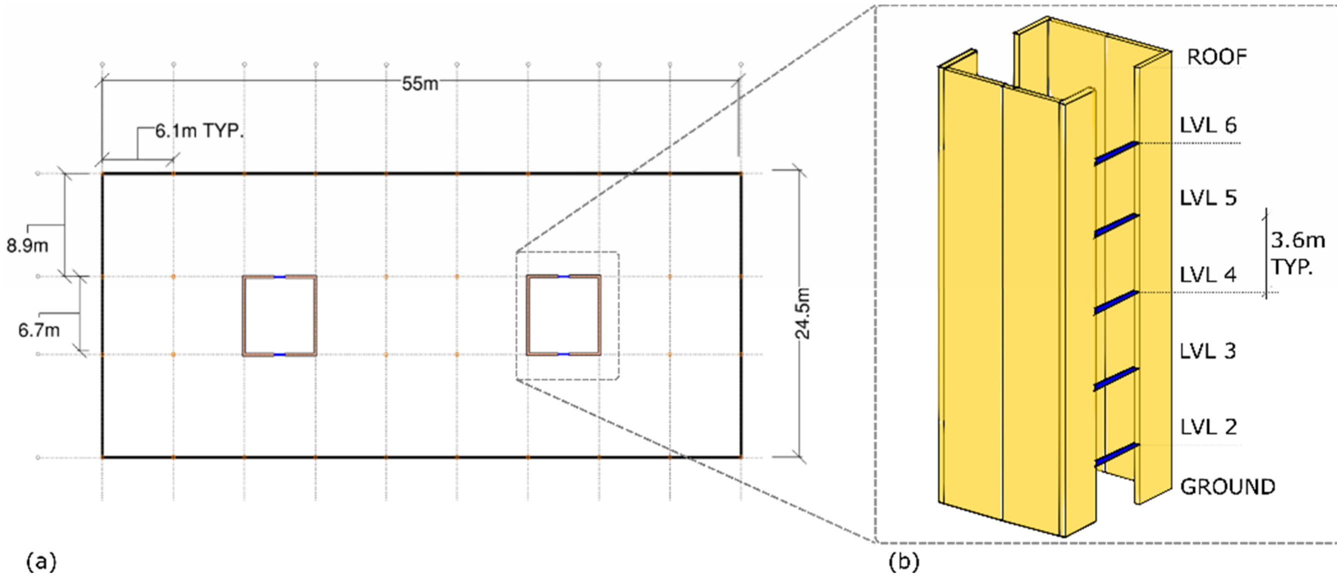


Figure 5: (a) Prototype building typical floor plan and (b) isometric of PT core-wall system with steel coupling beams

For design, a value of R_d = 3.0 was used, similar to nailed wood-panel sheathed shear wall systems. In the C-shaped wall direction, the choice of R_d was also based on work by Sarti et al. [26], which determined a seismic reduction factor, R = 7, in accordance with ASCE 7 [27] for PT timber shear wall systems. Nailed wood-panel sheathed shear wall systems also use R = 7 within the context of ASCE 7. Further, R_o = 1.5 was selected as it is the value currently used for platform CLT shear wall construction with CSA 086-19 [2].

In the beam-coupled wall direction, values of R_d = 3 and R_o = 1.5 were chosen on account of the system's resemblance to the failure mechanism of eccentrically braced frames, which has specified factors of R_d=4 and R_o=1.5 in the NBCC.

The fundamental period in the C-shaped wall direction was determined using the empirical equation $T_1 = 0.05{h_n}^{0.75}$, where h_n is the building height, according to the Pres-Lam design guide [11]. In the beam-coupled wall direction, the period approximation of T_1=$0.2N$ was used, where N is the number of storeys. A summary of parameters from the ESFP is provided in Table 1.

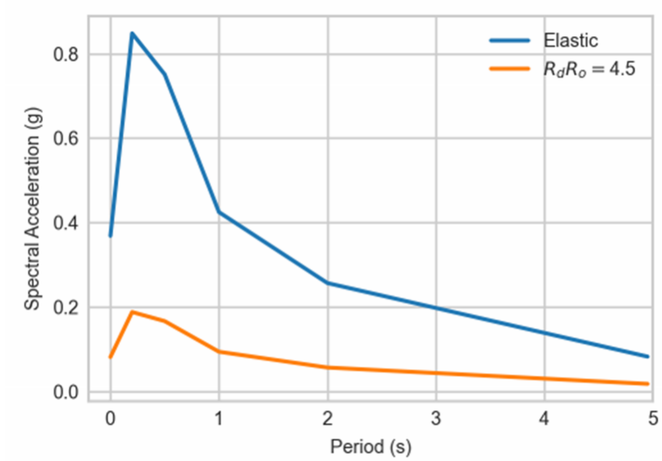


Figure 6: Seismic design hazard for Vancouver (City Hall) according to NBCC 2015 [3].

Table 1: Summary of parameters from equivalent static force procedure design of the protype building.

Wall Direction	T_1 (s)	Horizontal Design Action Coefficient V_{base}/W	W (kN)	V_{base} (kN)	M_{base} (kN-m)
C-Shape	0.5	0.17	22750	3800	57000
Beam-Coupled	1.2	0.12	22750	2650	38600

Design of Core-Wall LLRS

Figure 7 shows the detailed plan view of the PT core-wall LLRS with steel coupling beams. The design of the system required an iterative approach to optimize the CLT panel elements and joint fastening considering the application of forces in each orthogonal direction.

In the C-shaped core-wall direction, the system was designed following the Pres-Lam design guide [11], with extensions made by Brown et al. [16] for a PT core-wall connected with self-tapping screws. A theoretical rigid core-wall system was designed following the extended MMBA analysis and then a design core-wall base moment capacity equal to 2/3 the theoretical rigid core-wall system was assumed to account for screwed connection flexibility. This 2/3 value was determined based on experimental findings by Brown et al. [15] when mixed angled screwed connections were implemented as shown in Figure 7b.

In the beam coupled wall direction, a force-based design approach was followed, similar to the method used for RC coupled walls [28]. The degree of coupling factor, which is the ratio of overturning moment resisted by the link beam coupling action, was assumed to be β=0.6.

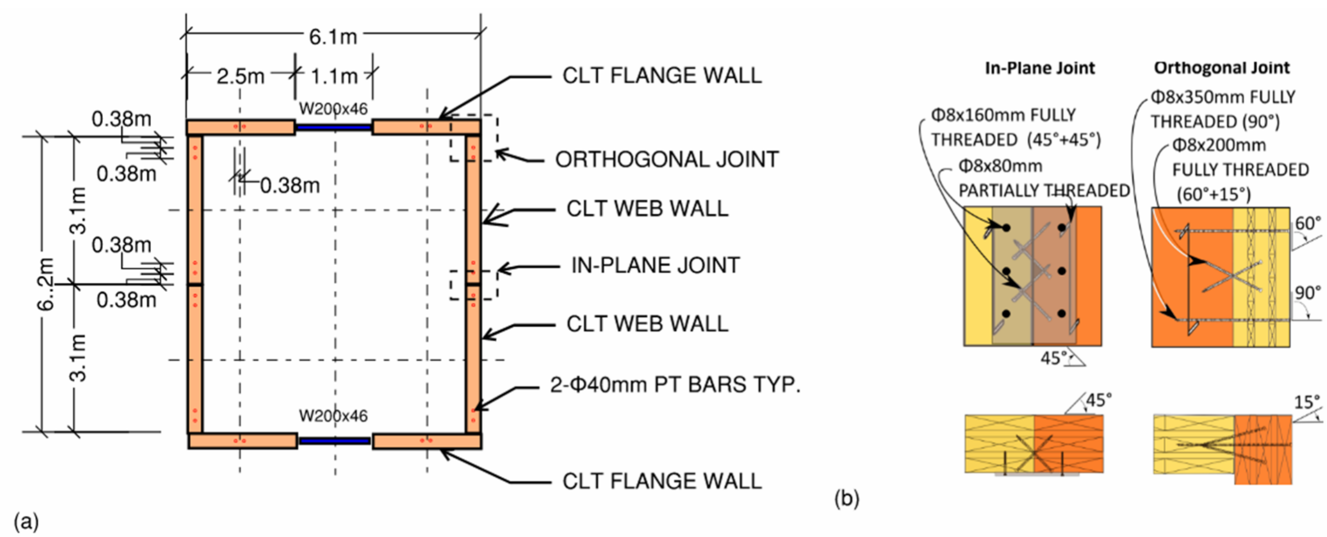


Figure 7: (a) Plan view of PT CLT Core-wall system with steel coupling beams, (b) screwed connection details at in-plane and orthogonal joint, adopted from Brown et al. [15]

The design of the core-wall system resulted in the use of 7-ply CLT panels with a grade of E1M5 per the ANSI/APA PRG 320 [29]. Pairs of Ø40mm Macalloy post-tensioning bars [30] separated by 380mm were placed at the corners of the CLT web walls and the center of the flange walls. The initial post-tensioning force was 150kN in each rod (approximately 15% of the yield force) and there was a total of 24 PT bars in the core-wall assembly. The CLT wall thickness was chosen to ensure adequate bending capacity, compressive strength, and local bearing strength for the PT bearing plates. The material properties are summarized in Table 2.

Table 2: Material properties of PT core-wall SFRS

Material	Property	Value
Cross-laminated timber	Modulus of elasticity, E_0 (MPa)	12,400
	Characteristic compression strength parallel to grain, $f_{c,0}$ (MPa)	19.9
Post-tensioning steel	Modulus of elasticity, E_p (GPa)	170
	Yield stress, f_{py} (MPa)	835
	Ultimate stress, f_{pu} (MPa)	1,030

The properties of the in-plane and orthogonal joints used mixed angle screwed connections (shown in Figure 7b) were derived from component screw connections experimentally tested by Hossain [31] and Brown et al. [20], respectively. The screwed connection configurations resembled those implemented on core-wall test CW-6 which was validated with the analytical nonlinear model proposed by Brown et al. [16].

W200x46 link beams were selected based on the calculated force demands. The pier-to-pier length was 1.1m and the link beam connections to the CLT wall pier were assumed to provide a rigid offset of 0.1m on each end which reduced their effective length to 0.9m to ensure a shear-controlled link beam failure mode.

MODELLING AND ANALYSIS

A single core-wall assembly was modelled with the finite element software ETABS[32], as shown in Figure 8. The CLT panels were represented by orthotropic shell elements. Compression-only contact springs were placed at the core-wall base and were calibrated to the previously completed PT wall experiments. The steel link beams used linear-elastic beam elements with lumped plasticity moment-rotation hinges at each end with properties determined according to ASCE 41 [33]. Backbone curves for the screwed in-plane and orthogonal joints were calibrated to experimental data using the SAWS model [19].

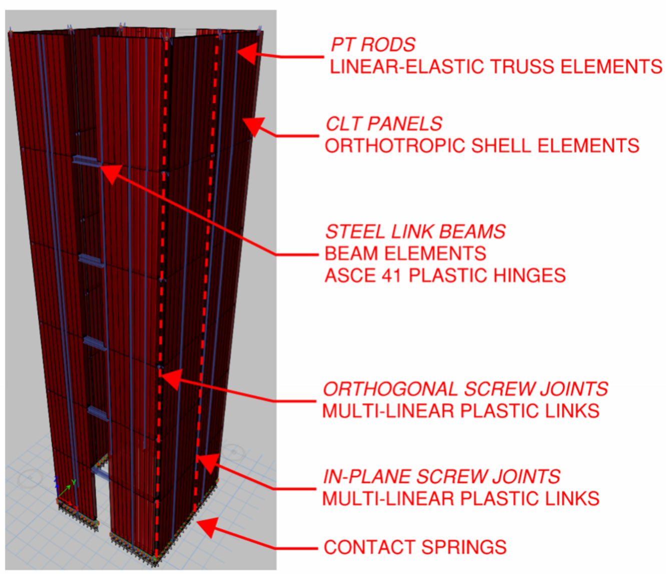
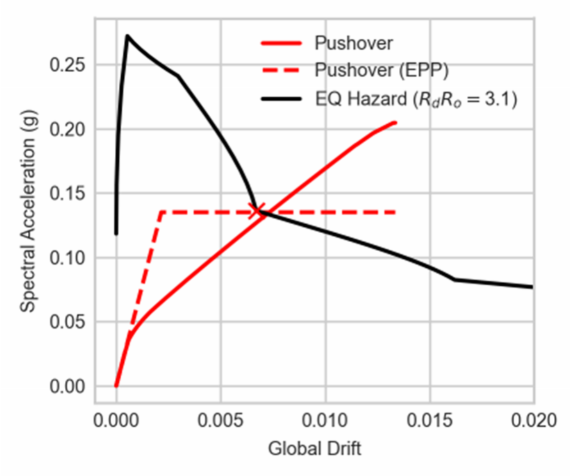


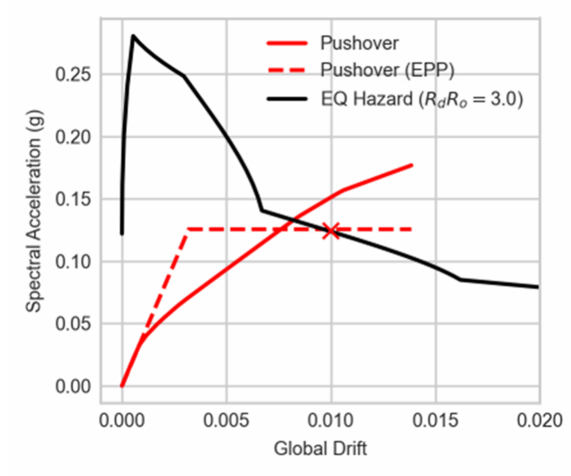
Figure 8: Summary of ETABS model configuration and element types.

A modal analysis of the structure revealed fundamental periods of T_1=1.3s and T_2=1.0s in the beam-coupled and C-shaped core-wall directions, respectively. Evidently, the empirical equation used previously to predict the fundamental period of the C-shaped core-wall direction significantly under-predicted the value. The error can be attributed to the fact that the empirical equation does not consider specific wall dimensions or timber properties.

Pushover analyses of the core-wall system in each orthogonal direction were completed using the load pattern specified in the ESFP in the NBCC 2015 [3]. The capacity demand diagram (CDD) method [34] was used to estimate the system's global drift ratio for a ULS earthquake event, as shown in Figure 9. The estimated global drifts and ductility demands are summarized in Table 3.



(a)



(b)

Figure 9: Capacity-Demand Diagrams for: (a) C-Shape cantilever wall direction and (b) beam-coupled wall direction.

Table 3: Estimated global drift and ductility demand determined by capacity-demand diagram method.

Wall Direction	**Estimated Global Drift Ratio**	**Ductility Demand**
C-Shaped	0.67%	3.1
Beam-Coupled	1.0%	3.0

The global drift ratios are significantly lower than the ULS limit of 2.5% specified for buildings of normal importance in the NBCC and ASCE 7 building codes [3], [27]. Therefore, the system concept appears to be feasible in terms of limiting global drift demands. In addition, the system's strength was adequate based on the performance point determined by the CDD method. However, it should be noted that the fundamental period of the C-shaped core-wall direction was underpredicted in the ESFP design process. A second iteration of the design based on a more accurate fundamental period prediction would likely produce greater drift and ductility demands in this direction of the system.

LIMITATIONS AND FUTURE WORK

This study was limited to a single configuration of CLT panels in a PT hybrid core-wall system based on previous research. However, the types of panel-to-panel connections and wide flange link beams are not necessarily the optimal elements for connecting adjacent panels. Therefore, future research should investigate alternative forms of high-capacity panel-to-panel connections that are capable of ductile failure modes and alternative coupling elements, such as viscous or active dampers, which could replace the steel link beams in the configuration proposed in this study.

Additionally, the design and nonlinear evaluation in this study was limited to a single 6-storey sample building. Future work should investigate a greater variety of building configurations and use nonlinear time history analyses to provide a more accurate evaluation of the proposed system's seismic performance.

CONCLUSIONS

This paper introduced a new type of LLRS for mid-rise mass timber buildings. A summary of recent research on PT CLT core-wall systems and beam-coupled CLT shear walls was presented and used to support the concept of a combined PT core-wall system. The proposed LLRS was designed using the ESFP for a 6-storey prototype building according to the NBCC 2015 and an assumed force reduction factor of $R_dR_o = 4.5$. Nonlinear static (pushover) analyses were completed and the CDD method was used to predict the systems global drifts in each orthogonal direction. Global drift ratios of 0.67% and 1.0% were found in the C-shaped and beam-coupled wall directions, respectively.

Based on the results of the study, the following conclusions are drawn:

1. Previous research in C-shaped PT CLT core-walls and beam-coupled CLT shear walls can be combined to create a high-capacity LLRS for mid-rise timber buildings.

2. The proposed PT CLT core-wall system is a feasible LLRS with the ability to rock about its base and provide energy dissipation through the ductile failure of screws between panels and yielding of steel link beams.
3. The results of a nonlinear pushover analyses on the proposed hybrid core-wall system indicates the system can adequately control global drifts for a 6-storey building and has the potential to be a feasible alternative LLRS for taller mass timber structures in regions with significant seismic hazard.

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