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SEISMIC PERFORMANCE EVALUATION OF PLATFORM-TYPE COUPLED-PANEL CLT SHEAR WALL SYSTEMS

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ABSTRACT: This paper presents a case study to assess the seismic performance of platform-type coupled-panel CLT shear wall systems. A novel procedure to consider the strength and yielding hierarchy between various components are proposed. First, numerical models of components were developed and calibrated using experimental or simulation results. Subsequently, an archetype building with platform-type coupled-panel CLT shear walls was designed following the capacity design principles in CSA O86-24. Then, Finite Element (FE) models of the shear wall system incorporating the tested connections will also be developed. The performance of a representative coupled-panel CLT shear wall in platform-type construction composed of two CLT panels was evaluated through quasi-static analysis under cyclic loading, demonstrating that structures designed according to capacity design principles exhibit excellent ductility and energy dissipation under lateral loading, thereby improving the safety and resilience of timber structures.

KEYWORDS: Cross-laminated timber (CLT), coupled-panel shear walls, platform construction, seismic performance, capacity design principle

1 – INTRODUCTION

Mass timber buildings using CLT is gaining popularities around the world. CLT shear walls provide one common lateral load resisting system for mass timber construction and are able to provide higher lateral capacity than conventional light wood frame shear walls. In Seismic Category (SC) 4 regions with relively high seismicity in Canada, such as Vancouver, the use of platform-type CLT shear walls (Fig. 1) as seismic force resisting systems (SFRSs) is limited to a maximum height of 20 meters. In contrast, for regions with lower seismicity, the height limit for these structures increases to 30 meters. However, as the platform-type mass timber construction method for mid- and tall wood buildings is relatively new with limited performance history subjected to significant seismic events, the seismic performance of archetypes designed to meet NBCC [1] requirements needs to be conservatively and carefully benchmarked.

Within coupled-panel CLT shear wall systems, as shown in Fig. 2, a rational strength and yielding hierarchy along the load path can be established by following capacity design principles. Relevant equations for capacity protection factors are proposed in this study. In the Engineering Design in Wood CSA O86-24 [2], hold-downs are categorized as energy-dissipation elements and are designed to yield after the vertical panel-to-panel connections. Nevertheless, these modifications have seen limited practical application, and the seismic performance of buildings designed under the new version of the standard should be reassessed.

2 – BACKGROUND

The performance and hysteretic behaviour of the CLT-related connections and shear wall systems have been the focus of several research projects. For example, Popovski et al. [3] tested coupled-panel CLT shear walls, revealing that the connections sustained significant damage under lateral loading and the CP mode was observed.

An increasing number of studies have also concentrated on the seismic performance of CLT structures. The concept of capacity design [4] is now incorporated into different design codes for timber structures, such as Eurocode 5: Design of timber structures [5], CSA O86:24 Engineering design in wood [2], NZS AS 1720.1:2022 Timber structures [6], to ensure that SFRS has designated ductile behavior [7], [8]. Casagrande et al. [9], [10] proposed a simplified linear approach at CLT wall- and building - level using capacity design, considering the connection overstrength.

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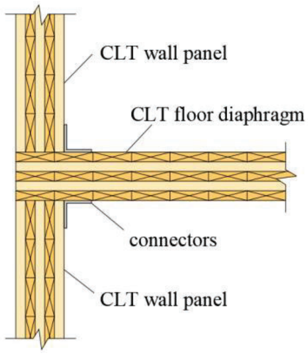

Figure 1. Platform conatruction of CLT structures

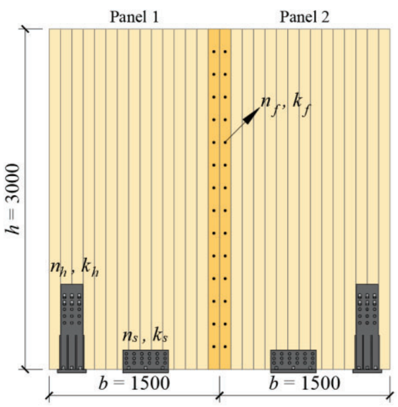

Figure 2. Configuration of the representative coupled-panel CLT shear wall

3 – A CASE STUDY

As the transition towards performance-based codes progresses, it is essential to research and address existing gaps to clarify seismic performance criteria. This will enable innovative solutions to demonstrate code compliance. This study will focus on the cyclic performance of the coupled-panel CLT shear walls in platform construction. A platform-construction building archetypes using coupled-panel CLT shear walls will be designed in Vancouver (6-storey, limited to 20 m) in accordance with CSA O86 and NBC 2020 for case study.

3.1 EXPERIMENTAL TESTS

The CLT connections, including vertical spline joints (VJ), hold-downs (HD), and angle brackets (AB), tested by the TEAM lab at the University of British Columbia were used for this study. All connections were tested under both monotonic and reversed-cyclic loadings. The loading rate for monotonic tests was set to 4.5 mm/min with failure occurring at 5-10 min, while for reversed-cyclic tests, it was set to 24 mm/min, with failure occurring approximately 5 minutes into the final cycle. The results from the monotonic and cyclic tests are provided in Fig. 3.

Representative load-deformation curves is shown in Fig. 4. The results of the reversed-cyclic loading tests will be used for further analysis. The yield resistances obtained from the Equivalent Energy Elastic-Plastic (EEEP) method are 14.2 kN, 23.0 kN, and 17.4 kN, respectively for spline joints, hold-downs, and angle brackets. The yield resistances are taken as the characteristic strength of connections in this study.

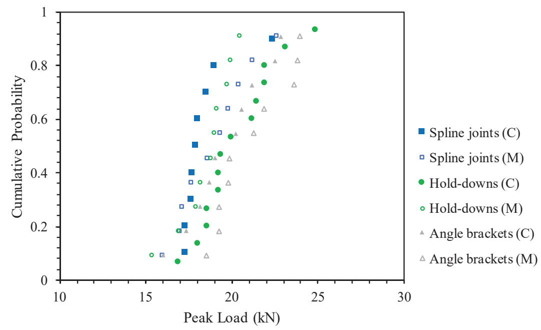

Figure 3. Cumulative probability of test results (M: Monotonic tests, C: Reversed-cyclic tests)

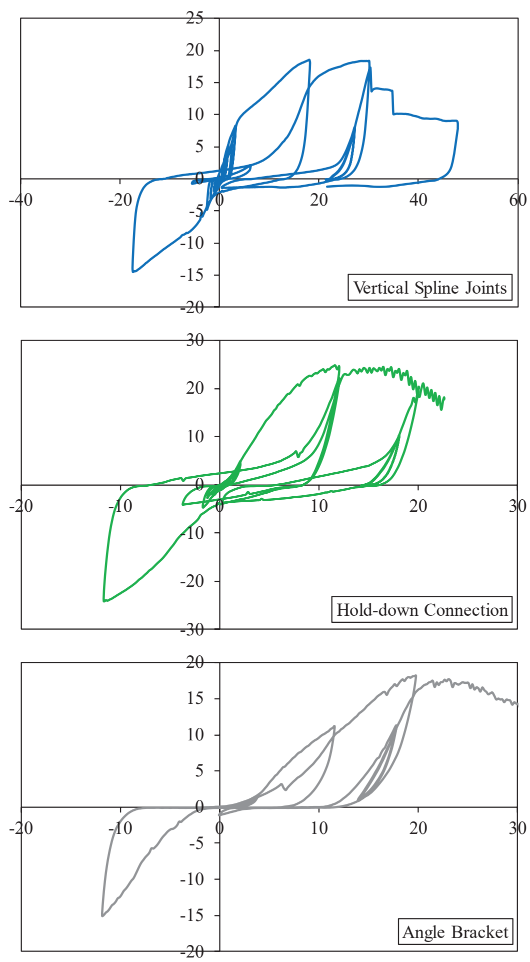

Figure 4. Representative load-deformation curves

3.2 CAPACITY DESIGN PRINCIPLE

A representative 3-meter-long shear wall consisting of two 1.5-meter-long panels, located on the ground floor, was chosen for the case study.

Estimation of force demands

The design base shear force was calculated using Equivalent Static Force Procedure (ESFP) as per NBC 2020 [1]. A simplified approach for the loading and design of CLT shear walls has been adopted using free body diaphragms (Fig. 5).

Capacity design principles

In the capacity design of timber structures, ductile behaviour is ensured under seismic loading by detailing ductile components as the weakest links along the load path and protecting all other elements from the capacity of the ductile components [11]. In coupled-panel CLT shear wall systems, panel-to-panel connections are designed as the primary energy-dissipative components, and hold-down connections serve as the secondary energy-dissipative components while other components are capacity protected.

Capacity protection factors

The Canadian Standard for Engineering Design in Wood, CSA O86:24 [2] also applies the capacity design principles by linking the design strength of other components to distribution of the peak resistance of the primary energy-dissipative connections. For the coupled-panel CLT shear walls, three capacity protection factors can be employed (Table 1), which is similar to the idea of connection overstrength factors in Eurocodes. The commonly recognized equation for γ_r is provided in (1).

$$\gamma_r = \gamma_{sc} \cdot \gamma_{an} \cdot \gamma_M \tag{1}$$

where, $\gamma_{sc} = \frac{r_{f,x^{th}}}{r_{f,5^{th}}}$, attributed to the variability of the connection strength properties. $\gamma_{an} = \frac{r_{f,5^{th}}}{r_{f,an}}$ is the conservatism in models [12], denoting the ratio between the 5th percentile of the test results $r_{f,5^{th}}$ and the characteristic strength $r_{f,an}$, assumed to be 1.0 for dissipative timber structures [13], [14]. $\gamma_M = \frac{1}{\phi}$, denotes the partial material factor, typically represented by the resistance factor (ϕ = 0.8) in CSA O86.

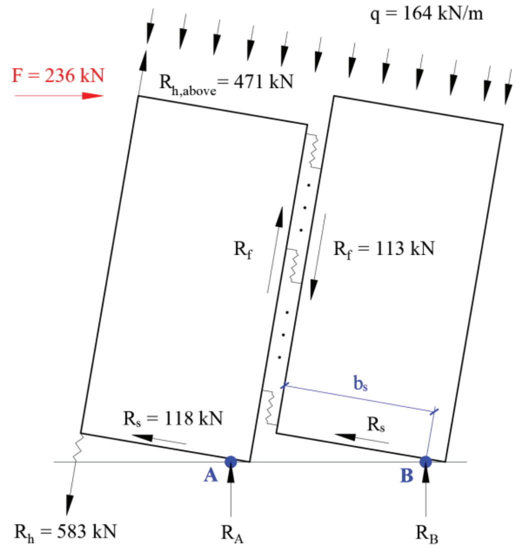

Figure 5. Schematic of adjacent CLT wall panels with CP behavior subjected to vertical and horizontal loads (bottom storey)

Table 1. Calculation of capacity protection factors

Components	**Vertical joints**	**Hold-downs**	**Angle brackets**	**CLT panels**
Energy Dissipative Category	Primary	Other	Limited	Non-
Behaviour	Yield	Yield	Elastic	Elastic
Related to x-percentile of VJ's peak resistance	/	15th	30th	95th
γ_r	/	$\gamma_{r,h}$	$\gamma_{r,s}$	$\gamma_{r,ND}$
	/	1.33	1.40	1.68

Design approach

A detailed step-by-step capacity design using the experimental data can be conducted for the coupled-panel CLT shear wall system. Fig. 6 illustrates the detailed capacity design approach, providing the relevant equations for designing vertical spline joints and hold-down connections.

Because the tested connections did not fully match the conditions of the case study building, corresponding scale factors were applied based on the force demand in the archetype building. The peak load was increased while maintaining the original deformation capacity, thereby creating artificial hysteresis loops for use in this archetype. The resulting hysteresis loops for the vertical joints and hold-downs, as integrated into the numerical model, are illustrated in Fig. 7.

In summary, the input parameters utilized in the models following the capacity design procedure are presented in Table 2.

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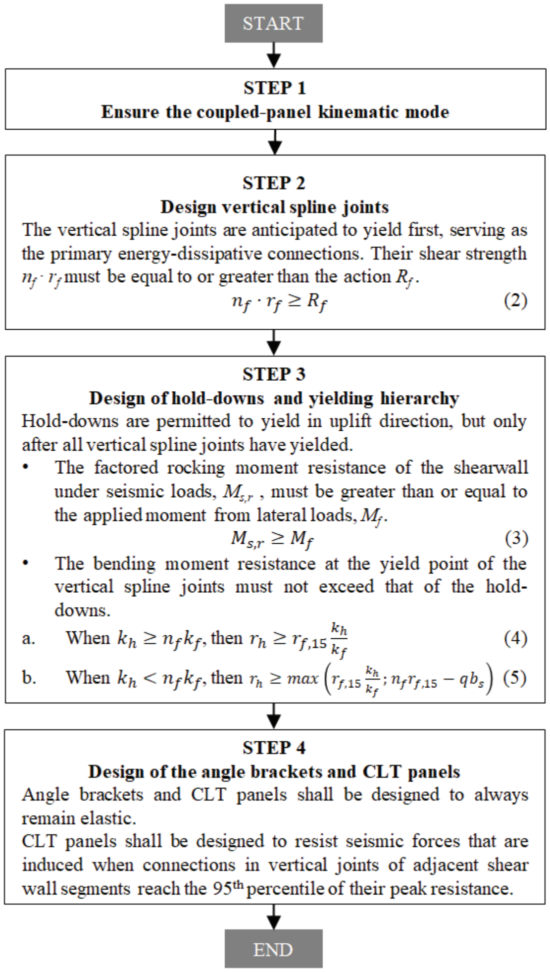
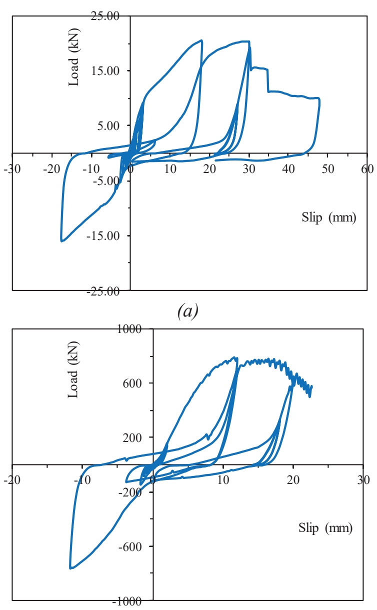


Figure 6. Design approach



(a)

(b)

Figure 7. Scaled hysteresis loops of: (a) vertical joints and (b) hold-downs, used for the numerical model

Table 2. Input model parameters

Components	Properties and units	Values
Vertical joints	k_f (kN/m)	3472
	n_f	9
	r_f (kN)	12.5
Traditional hold-downs	k_h (kN/m)	87506
	n_h	1
	r_h (kN)	583
Angle brackets	k_s (kN/m)	12886
	n_s	1
	r_s (kN)	132
Length of a CLT panel within shearwall	b_s (m)	1.5
Number of CLT panels in a shearwall	m	2
Total factored dead load applied at the top of shearwall	q (kN/m)	164
Design shear load due to lateral load	V_f (kN)	236
Design bending moment due to lateral load	M_f (kN·m)	1413

Notes:

a) n_f, n_h, and n_s are number of vertical joints, hold-downs, and angle brackets for each panel, respectively.

b) r_f, r_h, and r_s are factored resistance of vertical joints, hold-downs, and angle brackets, respectively.

3.3 NUMERIAL MODELS

Connetion models

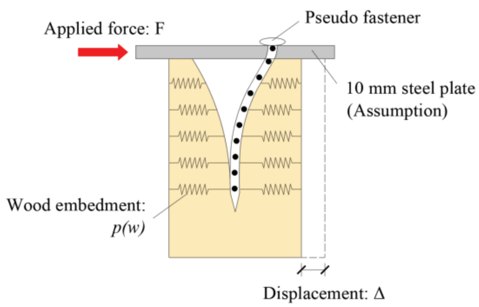
Numerical models for the tested connections are subsequently developed based on experimental results using a protocol-independent and mechanics-based procedure called HYST algorithm [15], [16], [17]. The fastener is modelled as elastoplastic steel beam element and the wood embedment is modelled as a series of nonlinear compression-only springs (Fig. 8 (a)). The force-displacement properties *p(w)* of the wood embedment springs is shown in Fig. 8 (b).

Representative backbone curves from reversed-cyclic tests were used for the HYST calibration and validation. A random search-based procedure was incorporated into the HYST subroutine to calibrate the optimal parameters, as shown in Table 3.

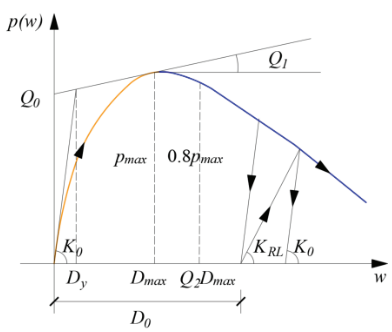
CLT shear wall model

The calibrated and validated HYST models for connections will subsequently be integrated into the CLT shear wall models. The cyclic analysis was conducted using CLTWALL2D FE program [15] (Fig. 9). CLT panels are modelled as plate elements. Zero-length HYST springs are integrated into the wall models to represent the behaviour of various connections.

The cyclic performance criteria of the coupled-panel CLT shear walls in platform construction is set as life safety, corresponding to drift ratios of 2.5% according to NBC 2020 [1]. To study the cyclic performance of coupled-panel CLT shear wall, the displacementcontrolled loading protocols based on the building drift ratios were developed and applied at the top corner of the panels. The reversed cyclic loading protocol follows ASTM E2126 Method B [18], consisting of a series of cycle groups, each containing three identical cycles.



(a)



(b)

$$\begin{cases} p(w) = (Q_0 + Q_1 w)\left(1 - e^{-\frac{K_0 w}{Q_0}}\right) & if\ w \le D_{max} \\ p(w) = P_{max} e^{Q_3 (w - D_{max})^2} & if\ w > D_{max} \end{cases}$$

Where $P_{max} = (Q_0 + Q_1 D_{max})\left(1 - e^{-\frac{K_0 D_{max}}{Q_0}}\right)$

and $Q_3 = \frac{log(0.8)}{[(Q_2 - 1.0) D_{max}]^2}$

$$\begin{cases} K_{RL} = \mathrm{K}_0 & if\ D_0 < D_y \\ K_{RL} = \left(\frac{D_y}{D_0}\right)^{\alpha} \mathrm{K}_0 & if\ D_0 \ge D_y \end{cases}$$

Figure 8. Schematics of: (a) HYST subroutine; (b) Force-displacement relationship for wood embedment springs [16]

Table 3. HYST parameters calibrated by connection test data

HYST parameters		VJ shear	VJ separation	T-HD uplift	AB shear
Equivalent Fastener	*L* (mm)	80	80	80	80
	D (mm)	6	6	4	4
Embedment Properties *p(w)*	Q_0 (kN/mm)	10	10	97	100
	Q_1 (kN/mm^2)	0.001	0.001	0.007	0.012
	Q_2	2	2	1.5	1.5
	K_0 (kN/mm^2)	1	1	5	2
	D_{max} (mm)	11	11	4.6	7
	α	0.2	0.2	0.2	0.2
Coefficient of determination R^2		0.93	0.93	0.97	0.96

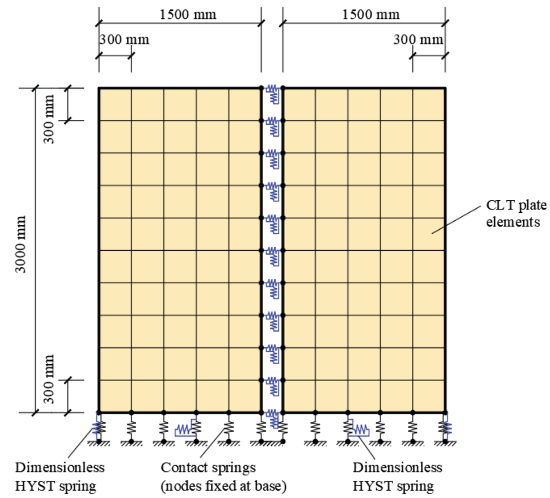


Figure 9. Schematics of CLTWALL2D FE program (modified from [15]*)*

3.4 SIMULATION RESULTS

Cyclic analyses were conducted on the developed archetype to evaluate the cyclic performance. The simulated monotonic and cyclic behaviour obtained using the CLTWALL2D FE program is presented in Fig. 10.

The backbone curve of the cyclic loop closely aligns with the monotonic curve, particularly within the elastic region, reflecting consistent stiffness characteristics during initial loading. Under cyclic loading, the walls exhibited a slightly lower peak strength but higher ductility, indicating a more ductile failure mechanism compared to monotonic loading. This enhanced ductility suggests improved energy dissipation capacity, which is critical for structural resilience under seismic loading scenarios.

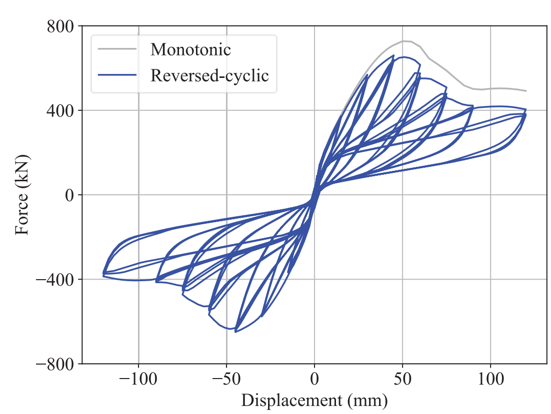


Figure 10. Simulated hysteretic behavior for the coupled-panel CLT shear wall

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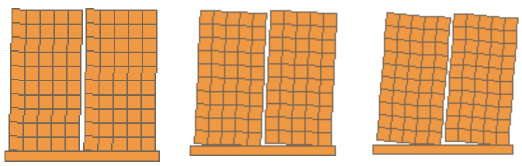
From the cyclic loading results, the initial stiffness is 30810 kN/m, maximum lateral load capacity is 660 kN, with a ductility of 4.36 and dissipated energy of 350 KJ, as calculated using the EEEP method. A moderate level of ductility and sufficient energy dissipation capacity are achieved. As expected, the simulation process reveals the CP mode (Fig. 11), which validates the capacity design procedure.



Drift = 1.0% *Drift = 2.5%* *Drift = 4.5%*

Figure 11. Wall deformation schematics at different drifts (monotonic)

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