

INVESTIGATING THE MECHANICAL BEHAVIOUR OF MULTI-PANEL BALLON-TYPE CLT SHEARWALLS THROUGH FULL-SCALE TESTS

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ABSTRACT: Balloon-type Cross-Laminated Timber (CLT) shearwall systems have gained widespread application in modern construction due to their ability to reduce perpendicular-to-grain compression deformation and minimize vertical shrinkage compared with traditional platform-type shearwall constructions. However, despite the increasing adoption of balloon-type CLT shearwalls in construction practices, research on their mechanical performance remains limited, and existing timber design codes lack specific provisions and guidelines. This paper presents an experimental campaign aimed at investigating the structural performance of multi-panel balloon-type CLT shearwalls under lateral loading conditions. The study examines connection stiffness as the key influencing parameter and discusses the kinematic modes as well as structural resistance and lateral displacement. A comparison between experiments and numerical modelling analysis is also undertaken and discussed.

KEYWORDS: balloon-type, cross-laminated timber, deformation and displacement, kinematic mode

1 – INTRODUCTION

Balloon-type CLT shearwalls represent a widely utilized lateral load-resisting system (LLRS) in mid-rise timber buildings, offering several advantages, including reducing deformation due to perpendicular-to-grain compression, minimizing vertical shrinkage effects within the structural system, and streamlining both installation and construction processes, which can contribute to overall cost and time efficiency in construction [1,2]. Despite their growing application in modern timber construction, current building codes and design standards [3-5] have yet to establish the necessary design provisions and detailing guidelines, which present challenges for designers to ensure structural reliability while optimizing the performance of these systems.

Balloon-type CLT shearwalls typically extend across multiple stories and often exhibit panel aspect ratios exceeding 4:1. As a result, the analytical approaches and design methodologies developed for platform-type shearwalls [6] may not be directly applicable. A key distinction between the two systems lies in their deformation mechanisms. In platform-type shearwalls, panel deformation is generally considered negligible, while the primary contributors to system flexibility and resistance are the mechanical connections adopted to anchor the panels to the foundations (or the floor below) and to connect adjacent panels [7-9]. Conversely, for balloon-type shearwalls, deformations of panels can be significant [10], particularly in configurations where the aspect ratio is over 6:1 [2].

While analytical and numerical models have proven effective in predicting the horizontal deformation behaviours and emphasizing the critical role of boundary conditions in system performance [1,2], the availability of experimental data for validating these models remains limited. Investigations of balloon-type CLT shearwall tests have either focused on lower aspect ratio [11] or considered walls consisting of only a single panel [12]. Hence, the scarcity of full-scale experimental investigations poses a challenge in bridging the gap between theoretical predictions and real structural performance, highlighting the necessity for further studies.

The current research project aims to undertake an experimental campaign to investigate the mechanical behaviour of full-scale multi-panel balloon-type CLT shearwalls. This study seeks to provide a deeper understanding of the structural performance of such

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systems by examining critical parameters that influence their response. This paper focuses on the results of the first phase of the experimental campaign involving the impact of parameters such as the relative stiffness between hold-down and vertical joints. The ratio between the hold-down stiffness k_{hd} and that of vertical joints k_{vj} is of particular significance since it influences the kinematic modes of the shearwalls under the lateral loads. The paper also presents a comparison between results obtained from a numerical model and those from the experimental tests.

2 – MATERIALS AND METHODOLOGY

2.1 MATERIALS AND GEOMETRICAL PROPERTIES

The test specimens consisted of five-layer CLT panels with a total thickness of 175 mm with outer layers oriented along the vertical direction of the wall. The CLT panels used in this study conform to grade E1 as specified in the Canadian standard [4] and are supplied by Nordic Structures (Canada). Each panel measures 5,000 mm in height and 600 mm in width.

Proprietary hold-downs (WHT620 [13]) were connected to the CLT panel using 55 Φ 4 × L60 mm annular shanked nails and to the steel base beam using a Φ 20 × L1000 mm steel rod. The spline joint between panels consisted of 25.4 mm (1 inch) thick plywood, nailed with countersunk Φ 8 × L80 mm screws.

The materials and properties relevant to the CLT panels, mechanical connectors, and vertical joint fasteners are summarized in Table 1. The mechanical performance of the hold-down and vertical joints was obtained from tensile and shear tests [14].

Table 2 presents a summary of the testing matrix, detailing the different specimen configurations. In the

Tuble 1. Material and properties				
Properties				
Young's modulus of	11700			
longitudinal layers E_L (MPa)				
Young's modulus of	0000			
transverse layers E_T (MPa)	9000			
Maximum tensile resistance $T_{hd, max}(kN)$	93.4			
Displacement at the maximum tensile resistance point $\Delta_{T-hd, max}$ (mm)				
		Ultimate tensile resistance $T_{hd, u}$ (kN)	73.9	
Ultimate tensile displacement $\Delta_{T-hd, u}$ (mm)	17.2			
Maximum shear resistance $S_{vj, max}$ (kN)	7.5			
Displacement at the maximum shear				
resistance point $\Delta_{S-vj, max}$ (mm)	-3.2			
Ultimate shear resistance $S_{vj, u}$ (kN)				
Ultimate shear displacement $\Delta_{S-vj, u}$ (mm)	55.7			
	Properties Young's modulus of longitudinal layers E_L (MPa) Young's modulus of transverse layers E_T (MPa) Maximum tensile resistance $T_{hd, max}$ (kN) Displacement at the maximum tensile resistance point $\varDelta_{T-hd, max}$ (mm) Ultimate tensile resistance $T_{hd, u}$ (kN) Ultimate tensile displacement $\varDelta_{T-hd, u}$ (mm) Maximum shear resistance $S_{y_j, max}$ (kN) Displacement at the maximum shear resistance point $\varDelta_{S-y_j, max}$ (mm) Ultimate shear resistance $S_{y_j, u}$ (kN) Ultimate shear displacement $\varDelta_{S-y_j, u}$ (mm)			

Table 1: Material and properties

first phase of the study, two-panel shearwall systems with fixed panel aspect ratio of 8.3 were considered, while the number of hold-downs ranged from one to four and the number of connections in the vertical joints between adjacent panels was fixed at thirty-two pairs, thereby varying the relative stiffness. A standardized naming convention is adopted, where for instance, in specimen "S-5/0.6-P2-H2V32", the notation "S-5/0.6" specifies that each panel had dimensions of 5 m (height) \times 0.6 m (width), "P2" indicates that the specimen comprises two panels, while "H2V32" represents the inclusion of two hold-downs and thirty-two pairs of vertical joints between adjacent panels.

2.2 TEST SET-UP AND LOAD PROTOCOL

Fig. 1 provides an overview of the test setup. The CLT panels were positioned horizontally and braced securely out of plane at their base.

The experimental test setup represented full-scale, twostory balloon-type shearwalls, subjected to two equal horizontal point loads at the diaphragm level of each story. To maintain equal force distribution at each loading point, a single hydraulic actuator was centrally positioned at the midspan of a rigid I-shaped steel spreader beam, ensuring balanced load application.

A steel base beam with a length of 3505 mm was anchored to the ground, simulating the foundation of the system. Additionally, to prevent lateral sliding of the shearwalls during testing, a steel shear key was installed at the panel ends, providing necessary lateral confinement and maintaining the intended structural behaviour throughout the experiment.

A monotonic displacement-controlled loading protocol was employed, utilizing a hydraulic actuator with a loading rate of 25 mm/min. During the tests, the ambient temperature was maintained at $20 \pm 2^{\circ}$ C, with a relative humidity of $65 \pm 5\%$. Additionally, the equilibrium moisture content of the specimens was controlled at $12 \pm 2\%$.

The lateral displacement of the shearwall was monitored at the two diaphragm heights of 2.4 m and 4.8 m. These

Table 2: Testing matrix						
Specimen No.	Panel aspect ratios (h/b)	Number of panels	Number of connections in the vertical joints	Number of hold-downs		
S-5/0.6-P2- H1V32	8.3	2	32	1		
S-5/0.6-P2- H2V32	8.3	2	32	2		
S-5/0.6-P2- H4V32	8.3	2	32	4		



Figure 1. (a) Site layout and overview; (b) Testing setup; (c) Details about the monitored positions at Point 1 (P1) and Point 2 (P2)

measurements were obtained using linear variable differential transformers (LVDTs) placed at both shearwall edges. In addition to tracking the lateral displacement, other key performance parameters included panel uplift, relative slip between adjacent panels, and compressive deformation at the rotation centres of the CLT panels (Fig. 1c). The uplift of the panels was directly measured using LVDTs at Point 1 (P1) positioned at the central axis of the outermost hold-down and Point 2 (P2) located at the opposite bottom corner of the panel.

3 - TEST RESULT AND DISCUSSION

3.1 DEFORMATION AND KINEMATIC MODES

Fig. 2 illustrates a typical deformation progression observed during the loading of specimen S-5/0.6-P2-H1V32.

This shearwall exhibited rocking motion about its center of rotation, accompanied by a small uplift in panel 1 at point P2. The deformation in the vertical joints was also evident as loading increased.

The ultimate failure was governed by rupture in the holddown, marking a complete loss of the shearwall lateral resistance. Residual deformations were observed at the panel-to-panel interface.

When the number of hold-downs were increased in specimens S-5/0.6-P2-H2V32 and S-5/0.6-P2-H4V32, the specimens displayed rocking behaviour with two centre of rotations (one for each panel) and significant deformation was observed in the vertical joints, but ultimately the failure occurred in the hold-downs. This can be attributed to the mechanical properties of the connectors, with the hold-downs possessing significantly lower ultimate tensile displacement ($\Delta_{T-hd, u}$) compared to the ultimate shear displacement of the vertical joints ($\Delta_{S-vj, u}$), as shown in Table 1.





(b) The deformation development at the bottom of panels

Figure 2: The structural deformation development of S5/0.6-P2-H1V32 under the lateral load: (a) the entire structural deformation; (b) the deformation development at the bottom of panels

A close look at the panel uplift and deformation progression is presented in Fig. 3. The characteristics of the hold-down can be effectively quantified based on the displacement recorded at P1, whereas the kinematic behavior of the shearwall system can be deduced from the displacement measurements at P2. Analysis of the load-displacement curves at P1 reveals that the tensile displacement at the peak resistance of the hold-downs remained consistent across different specimens, generally ranging from 13 mm to 16 mm. This result aligns well with the connection testing results obtained from the literature [14]. Regarding the kinematic response, two distinct kinematic modes were identified based on the observed rotational behavior of the panels. These include coupled-panel (CP) mode, where each panel rocks independently around its respective center of rotation, and single-wall mode (SW), where the panels rock about a single center of rotation positioned at the end of the entire shearwall. It was observed, for the configurations tested, that SW mode was dominant when only one hold-down was used. However, as the number, and hence stiffness, of holddowns increased, the system transitioned from SW mode to CP mode. For example, when four hold-downs were



(c) S-5/0.6-P2-H4V32

Figure 3: Two monitored points for measuring the tensile displacement of hold-downs (P1) and tracing the kinematic modes of shearwalls (P2) for the specimens: (a) S-5/0.6-P2-H1V32, (b) S-5/0.6-P2-H2V32 and (c) S-5/0.6-P2-H4V32

utilized, the CLT panel at P2 was consistently in contact with the base support, as shown in Fig. 3b and 3c.

3.2 LATERAL RESISTANCE AND DISPLACEMENT

Fig. 4 illustrates the relationship between the total lateral loads applied and the corresponding lateral displacements measured at the edge of the second diaphragm level (4.8 m).

As summarized in Table 3, the specimen with a single hold-down (S-5/0.6-P2-H1V32) attained a maximum lateral resistance of 36.2 kN, corresponding to a displacement of 156 mm. When the number of hold-downs was doubled (S-5/0.6-P2-H2V32), the maximum lateral resistance increased by roughly 1.7 times, accompanied by a 40% increase in displacement. Another doubling of the number of hold-downs (S-5/0.6-P2-H4V32) resulted in only a 1.5 times higher maximum lateral resistance and a 26% larger displacement. This observation implies that increasing the number or stiffness of hold-downs is not linearly proportional to the overall horizontal resistance of the structure, which is likely caused by the shift in kinematic mode.

4 – COMPARISON BETWEEN TESTING AND NUMERICAL MODELLING

A comparison between the experimental results and the numerical simulation was undertaken using the

commercially available finite element (FE) software package SAP2000 [16]. A pushover analysis was performed to simulate the applied loading conditions, as illustrated in Fig. 5.

In the FE model, two-dimensional thick-shell elements were employed to represent the behaviour of the CLT shearwall panels. A mesh size of $10 \text{ mm} \times 10 \text{ mm}$ was adopted [6] and each mesh unit was modelled as a four-node quadrilateral element. The mechanical properties of the CLT panels were incorporated using the homogenized approach [17,18].

The behaviours of the hold-downs and vertical joints were modelled using link or spring elements. Holddowns were represented by two-node link elements connected to the steel base beam, accounting for both tensile and shear responses. Similarly, vertical joints were modelled as two-node links that connected adjacent panels, providing shear resistance in both in-plane orthogonal directions.

The results of the comparison are illustrated in Fig. 6. Overall, the pushover curves obtained from the numerical simulations show good agreement with the experimental results. The numerical curves demonstrated consistent behaviour, comparable peak lateral resistances, and close displacements at peak load when compared to the physical tests. A quantitative comparison is summarised in Table 4. The results show reasonable agreement for the peak lateral load and displacement with errors ε within



Figure 4: The relationship between actuator load and displacement at the second diaphragm level for the specimens: S-5/0.6-P2-H1V32, S-5/0.6-P2-H2V32, and S-5/0.6-P2-H4V32

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Specimen No.	Number of hold-down	Kinematic mode	Structural lateral stiffness ⁽¹⁾ k _l (kN/mm)	Peak lateral load Fmax (kN)	Displacement at peak load ⊿ _{max} (mm)	Ultimate lateral load ⁽²⁾ F _u (kN)	Ultimate lateral displacement Δ_u (mm)
S-5/0.6-P2- H1V32	1	SW	0.36	36.2	155.5	29.0	159.8
S-5/0.6-P2- H2V32	2	СР	0.40	61.3	212.6	49.1	214.5
S-5/0.6-P2- H4V32	4	СР	0.50	89.9	269.5	71.9	288.8

Note: (1) Total lateral stiffness k_l is calculated according to ASTM-E 2124 [14].

(2) Ultimate lateral load F_u is defined as 80% of the peak lateral load F_{max} after peak.



Figure 5: The numerical model for the specimen S5/0.6-P2-H2V32 (a) before loading and (b) after loading

Table 4: Numerical values of peak i	ateral load, displacement at pea	ak load and lateral stiffness with the	ir deviations from experimental value
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Model No.	Peak lateral load Fmax (kN)	E (%)	Displacement at peak load ⊿ _{max} (mm)	ε (%)	Lateral stiffness k _l (kN/mm)	E (%)
S-5/0.6-P2- H1V32	33.7	7	141.5	9	0.30	16
S-5/0.6-P2- H2V32	60.7	1	225.8	6	0.34	15
S-5/0.6-P2- H4V32	91.9	2	278.5	3	0.40	20



(c) S-5/0.6-P2-H4V32

Figure 6: The comparisons between FE numerical modelling and testing result for the specimens (a) S-5/0.6-P2-H1V32, (b) S-5/0.6-P2-H2V32 and (c) S-5/0.6-P2-H4V32

10%, while greater deviation (up to 20%) is found for the initial stiffness.

Such discrepancies (ε) between the numerical and experimental results may be attributed to variability in material properties, and the use of idealised multi-linear constitutive models to represent the non-linear behaviour of mechanical connectors. Despite these limitations, the overall correlation indicates that the numerical approach provides a reasonably accurate prediction of the structural performance of the tested CLT shear walls.

5 – CONCLUSION

The experimental investigation of the multi-panel balloon-type CLT shearwall systems provides significant insights into their performance, including the deformation characteristics, failure mechanisms, and kinematic behaviour. The test results revealed that the shearwall primarily underwent rocking motion, and the ultimate failure consistently occurred due to hold-downs rupture, which happened prior to the vertical joints achieving significant inelastic damage, due to the disparity in their ultimate displacement capacities.

Both coupled-panel (CP) and single-wall (SW) mode behaviours were observed, depending on the relative stiffness of the connectors. The increase in the number or stiffness of hold-downs was not linearly proportional to the overall horizontal resistance of the structure.

The numerical model effectively captured the key structural behaviours observed in the experiments, with strong agreement for the peak lateral load and displacement, and slightly greater deviation for the initial stiffness.

Future research will explore the impact of other factors, including aspect ratio and number of panels.

6 – REFERENCES

[1] Z. Chen and M. Popovski. "Mechanics-based analytical models for balloon-type cross-laminated timber (CLT) shear walls under lateral loads." Engineering Structures, vol. 208, 2020, p. 109916.

[2] D. Xing, D. Casagrande, and G. Doudak. "Investigating the deformation characteristics of balloontype CLT shearwall." Canadian Journal of Civil Engineering, 2023.

[3] Canadian Commission on Building and Fire Codes. National Building Code of Canada 2020. Ottawa: National Research Council Canada, 2020.

[4] CSA Group. CSA O86-19: Engineering Design in Wood. Toronto: Canadian Standards Association, 2019.

[5] European Committee for Standardization, Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings, EN 1995-1-1, Brussels, Belgium: CEN, 2004.

[6] D. Casagrande, G. Doudak, L. Mauro, and A. Polastri. "Analytical approach to establish the elastic behaviour of multipanel CLT shear-walls subjected to lateral loads." Journal of Structural Engineering, vol. 144, no. 2, 2018, p. 04017210.

[7] R. Brandner, G. Flatscher, A. Ringhofer, G. Schickhofer, and A. Thiel, "Cross laminated timber (CLT): overview and development," Eur. J. Wood Wood Prod., vol. 74, pp. 331–351, 2016, doi: 10.1007/s00107-015-0999-5.

[8] M. Izzi, D. Casagrande, S. Bezzi, D. Pasca, M. Follesa, and R. Tomasi, "Seismic behaviour of crosslaminated timber structures: a state-of-the-art review," Eng. Struct., vol. 170, pp. 42–52, 2018, doi: 10.1016/j.engstruct.2018.05.060.

[9] D. Casagrande, S. Bezzi, G. D'Arenzo, S. Schwendner, A. Polastri, W. Seim, and M. Piazza, "A methodology to determine the seismic low-cycle fatigue strength of timber connections," Constr. Build. Mater., vol. 231, p. 117026, 2020, doi: 10.1016/j.conbuildmat.2019.117026.

[10] E. Karacabeyli and S. Gagnon, Canadian CLT Handbook: 2019 Edition, Pointe-Claire, QC: FPInnovations, 2019.

[11] Z. Li, X. Wang, and M. He, "Experimental and analytical investigations into lateral performance of cross-laminated timber (CLT) shear walls with different construction methods," J. Earthq. Eng., vol. 26, no. 7, pp. 3724–3746, 2020, doi: 10.1080/13632469.2020.1815609.

[12] X. Zhang, H. Isoda, K. Sumida, Y. Araki, S. Nakashima, T. Nakagawa, and N. Akiyama, "Seismic performance of three-story cross-laminated timber structures in Japan," J. Struct. Eng., vol. 147, no. 2, p. 04020319, 2021.

[13] ETA-11/0086, RothoBlaas WHT Hold Downs and Angle Brackets, European Organization for Technical approval, Denmark, 2018.

[14] M. Mohammad, Y. Zhang, and E. Karacabeyli. "Experimental investigation of dissipative connections in cross-laminated timber shearwalls." Journal of Structural Engineering, vol. 146, no. 4, 2020, p. 04020031.

[15] ASTM International, Standard Test Method for Static Testing of Framed Vertical Shear Resisting Panels Using a Substitution Method, ASTM Standard E2124-20, 2020.

[16] CSA Group. "CSA O86-19: Engineering design in wood." In: Canadian Standards Association. Toronto (2019).

[17] H. J. Blass and P. Fellmoser, "Design of solid wood panels with cross-layers," in Proc. 8th World Conf. on Timber Engineering (WCTE), vol. 14, no. 17.6, 2004, p. 2004.

[18] R. Brandner, P. Dietsch, J. Dröscher, M. Schulte-Wrede, H. Kreuzinger, and M. Sieder, "Cross laminated timber (CLT) diaphragms under shear: test configuration, properties and design," Construction and Building Materials, pp. 312–327, 2017, doi: 10.1016/j.conbuildmat.2017.04.153.