WORLD CONFERENCE ON TIMBER ENGINEERING 2025
BRISBANE, AUSTRALIA
Advancing Timber for the Future Built Environment

PERFORMANCE OF TWO-STOREY PLATFORM-TYPE CLT SHEAR WALLS

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ABSTRACT: Although many experimental tests have been performed on single-story cross-laminated timber (CLT) shear walls, studies on the drift performance of multi-story CLT structures remain scarce. This paper presents an experimental parameter study on the lateral performance of two-storey platform-type CLT shear wall structures using self-tapping screw (STS) connections. The test program included five reversed cyclic tests to investigate the effects of three key factors: additional floor mass, different angle bracket connections between floors, and different tension strap connections between floors. The results indicated that an increased dead load enhanced lateral resistance, while designing angle brackets to remain elastic minimized the sliding contribution to total lateral deformations to less than 10%. Tension straps played a crucial role in rocking performance, highlighting their significance in achieving uniform storey drifts in multi-storey CLT shear wall structures. Lastly, although prior single-storey shear wall tests informed structural design, hold-down uplifts observed in single-storey tests were found not to accurately represent tension strap uplifts in multi-storey tests.

KEYWORDS: Mass timber; cross laminated timber; self-tapping screws, seismic design

1 – INTRODUCTION

Mass timber construction is increasingly popular for mid- to high-rise buildings, offering lightweight, strong, and fire-resistant engineered wood systems. Its sustainability and performance have driven recent building code updates, allowing greater flexibility. Notably, the 2020 National Building Code of Canada [1] and the 2021 International Building Code [2] now permit mass timber buildings up to 12 and 18 stories, respectively

Previous studies highlight the rigid-body behavior of cross-laminated timber (CLT) panels and the critical role of connections in providing ductility [3]. Popovski et al. [4] demonstrated that vertical joints (VJ) with self-tapping screws (STS) enhance energy dissipation and flexibility. However, traditional anchors like hold-downs (HD) and angle brackets (AB) from light-frame timber construction have lower resistance and stiffness, limiting CLT shear wall performance [5].

Extensive testing on CLT shear walls has examined various connection types. Gavric et al. [6] and Shahnewaz et al. [7] found that flexible VJ increased ductility and improved seismic performance, influencing wall kinematics. Polastri & Casagrande [8] tested three-panel shear walls, showing that stiff VJ and flexible HD led to a single-wall response, effective for wind loads but unsuitable for seismic resistance.

In one study on multi-storey CLT shear walls, Popovski and Gavric [9] conducted cyclic loading tests on a two-storey building. The nailed HD and AB, with few nails, showed low stiffness and resistance, resulting in significant lateral deformations, including substantial sliding on the first floor. Each level experienced different inter-story drifts (ISD), and the role of tension straps (TS) in distributing deformation and resistance along the building height was not fully explored.

Despite extensive testing on CLT shear walls, research on multi-storey platform-type CLT buildings' drift performance remains limited. This study aims to experimentally evaluate their seismic performance, focusing on the role of different elements and connections. Providing this data will help designers improve multi-storey platform construction.

To achieve this, full-scale two-storey structures were tested at the UNBC Wood Innovation and Research Laboratory. The program included two preliminary monotonic tests to validate the setup and six reversed cyclic tests to assess structural response and connection performance. Key parameters evaluated included floor mass, AB connection behavior, and TS effectiveness between floors.

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https://doi.org/10.52202/080513-0537

2 – EXPERIMENTAL INVESTIGATIONS

2.1 OVERVIEW

The two-storey test structures featured a platform-framed coupled panel shear wall system (Figure 1) with a 1.5 × 2.0 m plan and a total height of 5.3 m. The system incorporated 5-ply CLT panels measuring 2.5 × 1.0 m, giving a panel aspect ratio of 2.5:1, in accordance with CSA O86 [10] to enable wall rocking. A target inter-storey drift of 3% was set, ensuring a 200 kN load could be sustained without significant strength degradation in either storey.

The findings from a previous testing campaign [11] were utilized in designing the coupled two-storey platform-type CLT shear walls of this study. The experimental program consisted of six reversed cyclic tests. The investigation sought to assess the influence of the following parameters: 1) superimposed dead load on floors; 2) different AB connections; and 3) different TS connections. For more details, including on addition test using and acoustic interlayer, the reader is kindly referred to Masroor et al. [12].

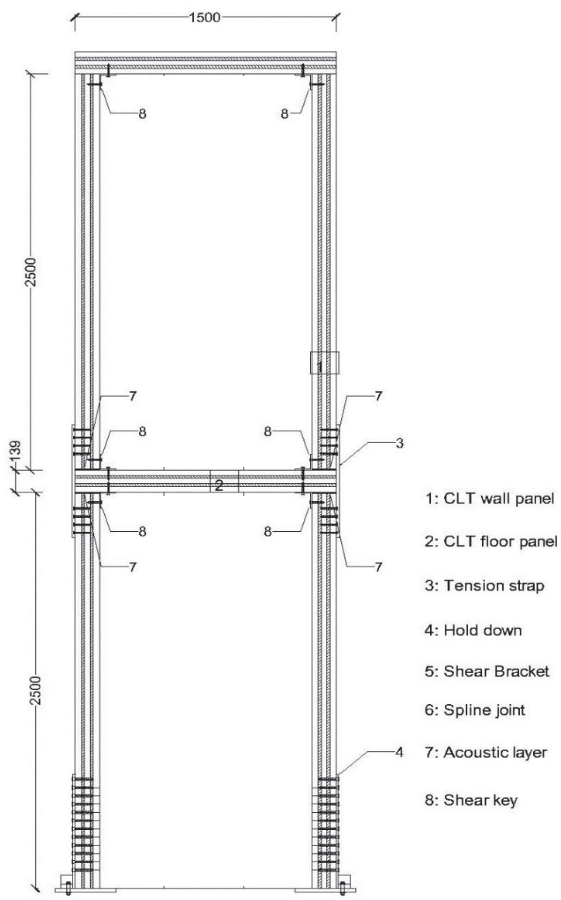

Figure 1. Two-storey CLT shear wall tests

2.2 MATERIALS

The CLT panels were classified as strength grade V2 [10], consisting of five plies with a thickness of 139 mm. The VJ were designed to yield simultaneously at both the first and second levels under a specified horizontal load, while the HD and TS were also intended to exhibit similar deformation and yielding behavior. The AB were capacity-protected to remain elastic throughout testing.

The panel-to-panel VJ incorporated surface-mounted D-Fir plywood splines (25 × 140 × 2500 mm), secured with partially threaded ø8×100 mm STS. Screws were spaced 150 mm apart with a 30 mm edge distance. To achieve the desired CP rocking kinematics, the number of screws was selected so that the VJ would yield first. Each panel edge on the first floor was secured with 16 STS, while the second floor used 11 STS per edge, totalling 32 and 22 screws per spline, respectively.

The HD were fabricated from custom 44W/300W steel plates, measuring 212 mm wide and 719 mm long, with a 70 mm horizontal base plate. Each HD was anchored using 21 fully threaded ø12×120 mm STS. The base plates contained three ø19 mm holes for bolted attachment to the test base.

Vertical TS, made from 4.8 mm thick, 186 mm wide, and 929 mm long custom 44W/300W steel plates, connected the shear walls across two stories. Three TS fastener configurations were tested: (1) 24 ø12×120 mm fully threaded STS at a 90° angle at each end, (2) 18 ø12×160 mm STS at a 45° angle with 45° washers, and (3) a combination of both, with ø12×160 mm STS at 45° on the bottom and ø12×120 mm STS at 90° on top. The number and orientation of screws were optimized to achieve similar drifts between stories.

The AB, measuring 6.35 mm thick, 127 mm long, and 340 mm wide, were also made from custom 44W/300W steel plates. They featured eight vertical slots (11 mm wide) to install STS, preventing uplift resistance and allowing only horizontal shear resistance. Fully threaded ø12×120 mm STS were used as fasteners to secure the AB.

In Test #H1c, the system behavior of the test setup and a baseline configuration of all connections were assessed. The AB configuration for the first-floor bottom (AB) in this and all subsequent structures included 3 bolts for attachment to the strong floor plus 6 STS. The first-floor top AB used 3 bolts and 4 STS. For the second-floor, both the bottom and top AB had 3 bolts and 4 STS. The TS were secured with 8 ø12 × 120 mm STS installed at a 90° angle. No additional dead load was applied.

All parameter combinations tested in the subsequent four tests are summarized in Table 1.

https://doi.org/10.52202/080513-0537

Table 1: Test parameters for reversed cyclic tests

Structure	Test	1st level bot AB	1st level top AB	2nd level bot AB	2nd level top AB	TS bottom	TS top	Dead load
1	#H1c	3 anchors + 6 STS	3 Bolts + 6 STS	3 Bolts + 4 STS	3 Bolts + 4 STS	8x STS 12×120 @90º	8x STS 12×120 @90º	-
2	#H2a	3 anchors + 6 STS	3 Bolts + 6 STS	3 Bolts + 4 STS	3 Bolts + 4 STS	12x STS 12×120 @90º	12x STS 12×120 @90º	2+2 tons
	#H2b	3 anchors + 6 STS	3 Bolts + 6 STS	3 Bolts + 6 STS	3 Bolts + 4 STS	9x STS 12×160 @45º	9x STS 10×160 @45º	2+2 tons
3	#H3a	3 anchors + 6 STS	3 Bolts + 6 STS	3 Bolts + 6 STS	3 Bolts + 8 STS	9x STS 12×160 @45º	9x STS 12×120 @90º	2+2 tons
	#H3b	3 anchors + 6 STS	6 STS + 6 STS	6 STS + 6 STS	3 Bolts + 8 STS	9x STS 12×160 @45º	9x STS 12×120 @90º	2+2 tons

2.3 TEST SETUP AND LOADING

The walls were constructed in a platform style, with the first-storey ceiling panels serving as the platform for the second-storey walls. To simulate dead and live loads in structures #2 and #3, 2-ton masses made of steel plates were placed on each floor. The reversed cyclic tests followed the CUREE loading history [13], with a target displacement of 150 mm, followed by additional cycles at 130% and 160% of the target displacement. Two actuators were used: the roof-level actuator (lead actuator) was displacement-controlled, while the first-storey upper floor actuator (slave actuator) was force-controlled. During each loading step, the force (F_{lead}) at the second-level actuator was recorded, and half of that force was applied by the first-storey upper floor actuator (F_{slave}), creating an inverted triangular load distribution. The total applied lateral load was $F = F_{lead} + F_{slave}$.

A total of 32 sensors were installed to allow detailed analysis of horizontal floor displacements and uplift deformations in the HD and TS. Panel distortion was also examined to assess the contribution of bending and shear deformation of CLT panels to the overall lateral displacement.

3 – RESULTS AND DISCUSSION

3.1 FLOOR DISPLACEMENT

The load-displacement curves for the total lateral load (F) and the inter-storey displacements of both floors are shown in Figure 2. In general, the curves displayed quasi-linear behavior up to 70% of the target displacement (105 mm top-level displacement), after which the system stiffness decreased due to connection yielding. The load continued to increase up to 130% or 160% of the target displacement; beyond these points, the system experienced local failures, resulting in a reduction in sustained loads. Additionally, pinching behavior, typical of connections using dowel-type fasteners, was observed. The positive and negative envelopes of the load-displacement curves showed very similar and symmetric characteristics in most cases.

From these curves, the ISD of the first and second floors at various positive and negative target displacements were evaluated. Notably, the ISD revealed discrepancies between the floors, with the second floor experiencing significantly higher displacement. This can be attributed to the varying connection parameters and stiffnesses, which contributed to rocking and sliding deformations at each storey and cumulative rotation along the height of the shear walls. The greatest discrepancy in ISD was observed in test #H1c, with a 220% difference. Tests #H2a and #H2b showed an approximately 70% ISD discrepancy, which was lower than that in test #H1a, likely due to the inclusion of additional dead load and the application of stiffer TS. Tests #H3a and #H3b exhibited the smallest discrepancy. The results underscore the significant role of TS in achieving more uniform displacement along the height of the structure.

3.2 LATERAL LOAD RESISTANCE

The lateral load resisted by the structures at various positive and negative target displacements is presented in Table 2. The load increased linearly up to 70% of the target displacement, after which it increased gradually, indicating connection yielding. The positive and negative values were similar. The highest loads were observed in tests #H2a and #H3a, reaching 336 kN (F_{max+}) and 337 kN (F_{max+}), respectively. This can be attributed to the use of stiff TS along with additional dead loads. In contrast, test #H1c, without dead loads, resisted the lowest lateral load, with F_{max+} = 263 kN and F_{max-} = -264 kN, approximately 20% lower than tests #H2a and #H3a. It is also worth noting that the resistance was very consistent for walls with additional gravity load, due to the consistent use of the same HD and VJ configurations across all tests.

https://doi.org/10.52202/080513-0537

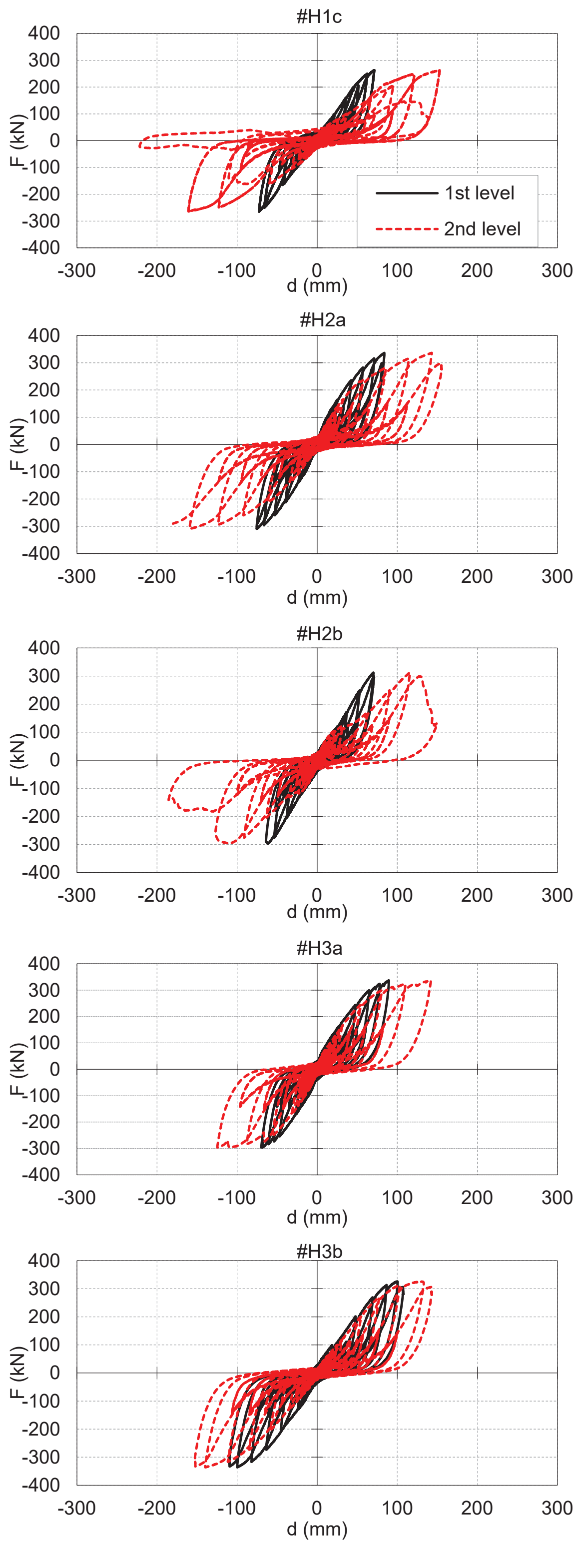

Figure 2. Lateral floor displacement in two-storey CLT shear wall tests

Table 2. Total lateral loads at target displacements [kN]

Target	H1c	H2a	H2b	H3a	H3b
40%	100	164	121	163	119
-40%	-104	-139	-132	-171	-130
70%	159	236	172	244	201
-70%	-163	-212	-205	-254	-205
100%	205	283	249	299	267
-100%	-143	-260	-276	-284	-273
130%	249	316	313	323	313
-130%	-250	-295	-264	-296	-317
160%	261	336	130	336	321
-160%	-262	-309	-142	-	-336

3.3 HOLD-DOWN / TENSION STRAP UPLIFT

The load-uplift curves closely matched the load-displacement curves in Figure 2, emphasizing the impact of HD and TS behavior on the seismic response of the shear walls. Symmetric behavior was observed between positive and negative cycles, with linear segments extending up to 70% of the target displacement. Uplift values for HD and TS from all tests are summarized in Tables 3 and 4, and exemplarily shown for test #H3b in Figure 3.

Since HD configurations remained unchanged, variations in HD uplifts were due to TS changes. Test #H1c, with the most flexible TS, showed the lowest HD uplift (6-9 mm at 100% target displacement) due to increased rocking at the second level. Other tests had similar HD uplifts (17-21 mm).

At 100% target displacement, test #H1c showed the largest TS uplift of 24 mm, particularly on the left-side TS. Comparing tests #H2a and #H2b, #H2a had higher TS uplift displacements of 14 mm, compared to 3.6 mm in #H2b. This discrepancy is due to the perpendicular STS penetration in #H2a, while #H2b used a 45° angle penetration. Tests #H3a and #H3b showed nearly identical TS uplifts, as both had the same STS inclination and details.

Table 3. Uplifts in hold-downs [mm]

Target	H1c	H2a	H2b	H3a	H3b
40%	5.1	7.0	4.5	6.4	6.3
-40%	-2.4	-2.1	-1.3	-0.9	-1.2
70%	8.9	11.5	7.3	11.5	11.4
-70%	-2.8	-2.6	-1.8	-1.4	-1.7
100%	5.8	15.7	12.2	16.6	16.0
-100%	-1.2	-3.0	-2.2	-1.8	-2.4
130%	14.3	20.3	18.2	21.2	21.1
-130%	-4.4	-3.4	-2.5	-2.1	-2.8
160%	8.8	24.9	17.5	25.5	25.0
-160%	-2.4	-3.7	-2.2	-	-3.3

https://doi.org/10.52202/080513-0537

Table 4. Uplifts in tension straps [mm]

Target	H1c	H2a	H2b	H3a	H3b
40%	2.4	2.5	0.1	2.8	2.5
-40%	-0.9	-1.2	-0.9	-1.6	-2.2
70%	6.7	6.0	0.7	6.4	5.8
-70%	-1.3	-2.4	-1.4	-1.8	-2.5
100%	10.3	9.7	1.8	11.2	9.7
-100%	-3.1	-3.1	-1.5	-2.4	-3.1
130%	23.8	13.7	3.6	17.3	16.4
-130%	-4.3	-3.5	-1.8	-3.5	-3.0
160%	33.6	17.8	4.7	24.1	26.0
-160%	-5.4	-4.3	-2.8	-4.7	-3.4

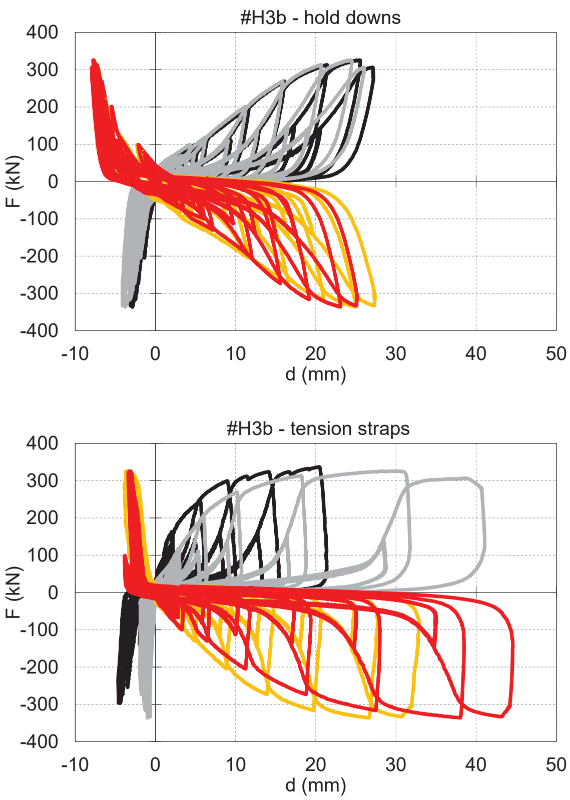

Figure 3. Uplift in hold-downs and the tension straps test #H3B

3.4 DISPLACEMENT COMPONENTS

At the 1st level, the total lateral displacement was primarily influenced by rocking, with distortion and sliding contributing only 2%-4% each at 100% target displacement. Rocking behavior accounted for 94%-96%. The TS had the most significant impact on the rocking performance of both storeys. Tests with flexible TS (e.g., #H1c and #H2a) showed greater rocking deformation in the 2nd storey, increasing the ISD discrepancy between levels.

In contrast, stiffer TS led to more uniform rocking deformations and a smaller ISD discrepancy. These findings highlight the importance of designing TS for consistent seismic performance. Tests #H3a, #H3b, and #H4a demonstrated that optimal HD and VJ configurations in the 1st storey contributed to uniform lateral resistance and energy dissipation across the structure. While single-storey shear wall tests informed design [11], HD uplift results from those tests don't fully represent TS uplift in multi-storey tests.

3.5 FAILURE MODES

Throughout all tests, no global structural instabilities were observed, see Figure 4. Second-floor wall rocking was primarily driven by STS deformations in the TS. Even under maximum load (160% of target displacement or 4.8% ISD), only localized failures occurred. The HD remained intact, as they did not exceed their displacement capacity. Screw yielding in VJ, TS, and AB, combined with some local crushing in splines and CLT, initiated energy dissipation, dominated rocking behavior, and determined lateral load resistance. STS withdrawal and head-pull through followed.

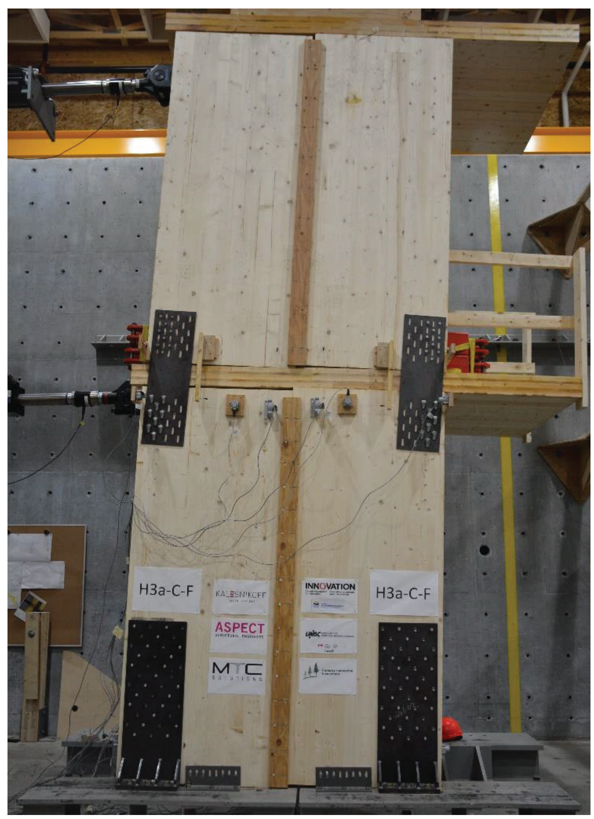

Figure 4. Test #H3a at 160% target displacement

4 – CONCLUSIONS

This research investigated the performance of two-storey platform-type structures with coupled panel CLT shear walls under quasi-static reversed cyclic loads. The following conclusions were drawn:

• The inclusion of dead load improved the lateral resistance of CLT shear walls by approximately 20%. However, given its impact on lateral deformations and the significant contribution of TS, further investigation is needed to fully understand this effect and accurately determine how dead load influences lateral deformations.

• The TS had a significant impact on the rocking performance of both storeys. More flexible TS led to greater rocking deformation in the second storey and a higher discrepancy in ISD between the storeys, while stiffer TS resulted in more uniform rocking deformations. These findings highlight the importance of selecting the appropriate TS to achieve the desired seismic performance in multi-storey CLT shear wall structures.

• While the preceding single-storey shear wall tests were useful in designing structures to meet the desired performance, it has been shown that the results from HD uplifts in single-storey tests are not representative of the TS uplift observed in multi-storey tests.

• The sliding contribution to total lateral deformations was found to be less than 4%, indicating that the design goal of keeping the fasteners in the AB within the elastic limit was successful. Additionally, various AB configurations did not significantly affect the overall lateral behavior of the structures. Therefore, using STS for both legs of the AB can be considered adequate for maintaining elastic behavior.

• Panel distortion contributed to less than 4% of the total top lateral displacement, confirming the assumption of rigid behavior for CLT panels with aspect ratios of less than 4:1.

• Consistent in all tests, no global instabilities were observed. Localized failures in connections, particularly STS used in VJ, TS, and HD—resulted in reduced lateral load resistance or plateaus in the load curves.

The findings from this study will be used to develop and validate numerical models for multi-storey CLT shear wall structures. Additionally, the insights gained will help refine existing analytical expressions, initially developed for single-storey applications, to improve their applicability in predicting the lateral resistance and deflection of multi-panel CLT shear walls.

ACKNOWLEDGEMENTS

This project received funding from the government of British Columbia through a Forest Innovation Investment grant. Special thanks go to UNBC technicians Michael Billups, James Andal, and Ryan Stern, as well as student research assistant Juan Jaimes Pico, for their valuable support. The CLT panels were supplied by Kalesnikoff, while the screws were provided by MTC Solutions.

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