CSCE 2022 Annual Conference
Whistler, British Columbia
May 25th – 28th, 2022

EXPERIMENTAL INVESTIGATION OF SINGLE-STOREY CLT SHEAR WALLS

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Abstract: This paper presents experimental investigations on Cross-Laminated Timber (CLT) shear walls for application in platform-type construction. A total of 8 single shear walls and 19 vertically with plywood splines connected coupled shear walls were tested with nailed hold-downs and shear brackets. The CLT panels were 7-ply 191 mm thick, the hold-downs and shear brackets were installed in different numbers and nails as well as with and without plate washers to create different yielding mechanisms. The shear wall lateral performance was assessed as a function of panel aspect ratios, number, and spacing of hold-downs and brackets, the number of nails in the spline joints, and the level of superimposed gravity loading. The tests showed that the shear wall aspect ratio and the size of the hold-down had a significant influence on the wall performance, while the reduction in hold-down nails and the shear bracket size did not.

1 INTRODUCTION

CLT is a type of mass timber panel made of three or more layers of orthogonally glued lumbers (Karacabeyli and Gagnon 2019). With the inclusion of encapsulated mass timber structures into both in the National Building Code of Canada (NBCC 2020) for buildings up to 12 stories, and the International Building Code (IBC 2021) for buildings up to 18 stories, the use of CLT panels in tall building construction is becoming common across North America. The Canadian Standard for Engineering Design in Wood (CSA O86) outlines provisions for CLT as shear walls for platform-type construction where the floors act as a platform for the next storey. CLT walls are comprised of a series of CLT panels are commonly connected to adjacent panels vertically with splines using screws or nails, and connected at their base with shear brackets (SB) and hold-downs (HD) (Shahnewaz et al. 2017). Numerous research on the performance of CLT connections and shear walls CLT panels has shown that these can resist lateral loads from wind and seismic events (Popovski et al. 2010, Gavric et al. 2015, Hossain et al. 2019, Shahnewaz et al. 2019, Shahnewaz et al. 2021, Tannert et al. 2018).

The objective of the research presented herein was to investigate the seismic behaviour of single and coupled CLT shear wall systems considering various panel aspect ratios, superimposed dead loads, number of SBs, and vertical spline nail spacing.

2 EXPERIMENTAL INVESTIGATIONS

The CLT panels were 7-ply 191 mm thick (35+17+35+17+35+17+35), strength grade V2 (CSA O86), supplied by Structurlam Ltd. The shear walls were anchored at the base with WHT440 HDs at the outer edges of the wall with 75- ø4×60 mm anker nails (Figure 1a) and TCN200 SBs with 36- ø4×60 mm anker nails (Figure 1b). The panel-to-panel vertical spline connections for the coupled shear walls were provided with surface mounted 25×140 mm D.Fir plywood splines, spliced at one-third of the wall height, and attached to the panel with ø4×60 mm anker nails.

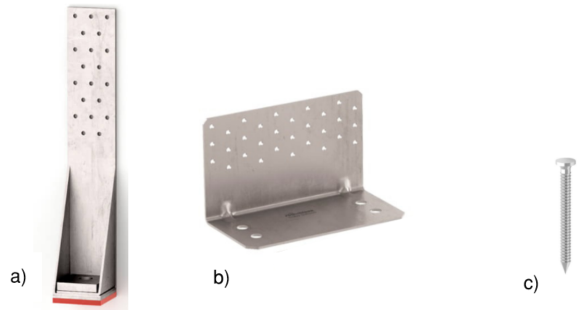

Figure 1: CLT wall connectors: a) HD, b) angle brackets, and c) nail

A total of 27 shear walls, 8 single-panel and 19 coupled panels, were assembled and subsequently tested in the UNBC Wood Innovation and Research Laboratory in Prince George, Canada. 4 monotonic and 4 reversed cyclic tests were conducted on single panel walls, and 4 monotonic and 13 reversed cyclic tests were conducted on coupled panels walls. The tests investigated a variety of panel aspect ratios, spline fastener spacing, number of brackets, and gravity loading. Two wall panel aspect ratios (height to length) were investigated: 2.5:1 (3000×1200 mm) and 3.5:1 (3000×850 mm) for both single and coupled panel shear walls, see Figure 2. Additionally, each specimen had either one or two SB per panel for both the single and coupled wall specimens. The coupled panels were vertically connected using a nailed plywood spline joint on one side with nails spacings that varied between 75 mm and 300 mm.

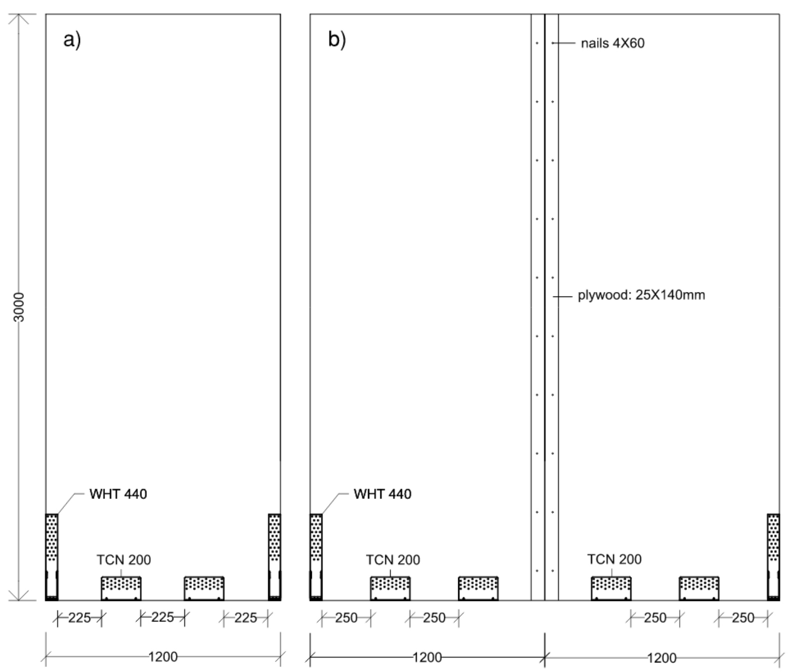

Figure 2: Shear wall schematic: a) single walls, b) coupled walls [dimensions in mm]

A schematic and a photo of the test setup are shown in Figure 3. Lateral loads were applied by a 250 kN actuator at the top of the wall through a steel side plate connected to a steel H-beam that loaded each panel at it's mid-point with large steel pins. Wooden blocking between the H-beam and the panel at the center of each panel accommodated the superimposed dead loads were applied onto the panels. Two levels of superimposed vertical gravity loads representing moderately loaded walls in typical platform-type timber building were applied: 20 kN/m and 30 kN/m. The load was applied using three steel beams connected to a nearby strong wall and cantilevered over the H-beam, with weights on hollow structural sections. This gravity load system also prevented out-of-plane horizontal movements and minimized impact to the lateral movement of the shear wall.

The shear wall configurations were tested under quasi-static monotonic loading at a rate of 10mm/min, to determine the displacement target for the subsequent quasi-static reversed cyclic tests. Tests were stopped at failure, defined as the point where the applied load dropped to 80% of the maximum. The reversed cyclic tests followed the abbreviated CUREE loading history, a displacement-controlled loading procedure per ASTM E2126 (2001). The 100% target displacement for the cyclic loading tests was set to 60% of the observed displacement at failure from the monotonic tests.

The horizontal, vertical, and relative panel displacements were recorded at twelve locations as shown in Figure 3. Sensors #1 was a string pot measuring the wall's horizontal displacements. Sensors #2 and #7 in coupled walls and #2 and # 4 in single walls are the LVDTs measuring the vertical displacement at the bottom corner of each panel recording the uplifts. Sensors #3 and #6 in coupled walls and #3 in single walls are LVDTs measuring the horizontal displacement between the testing apparatus and the bottom center of each panel to record the horizontal wall sliding, Sensors #8 measured the relative displacement between the panels at spline joints in coupled wall panels.

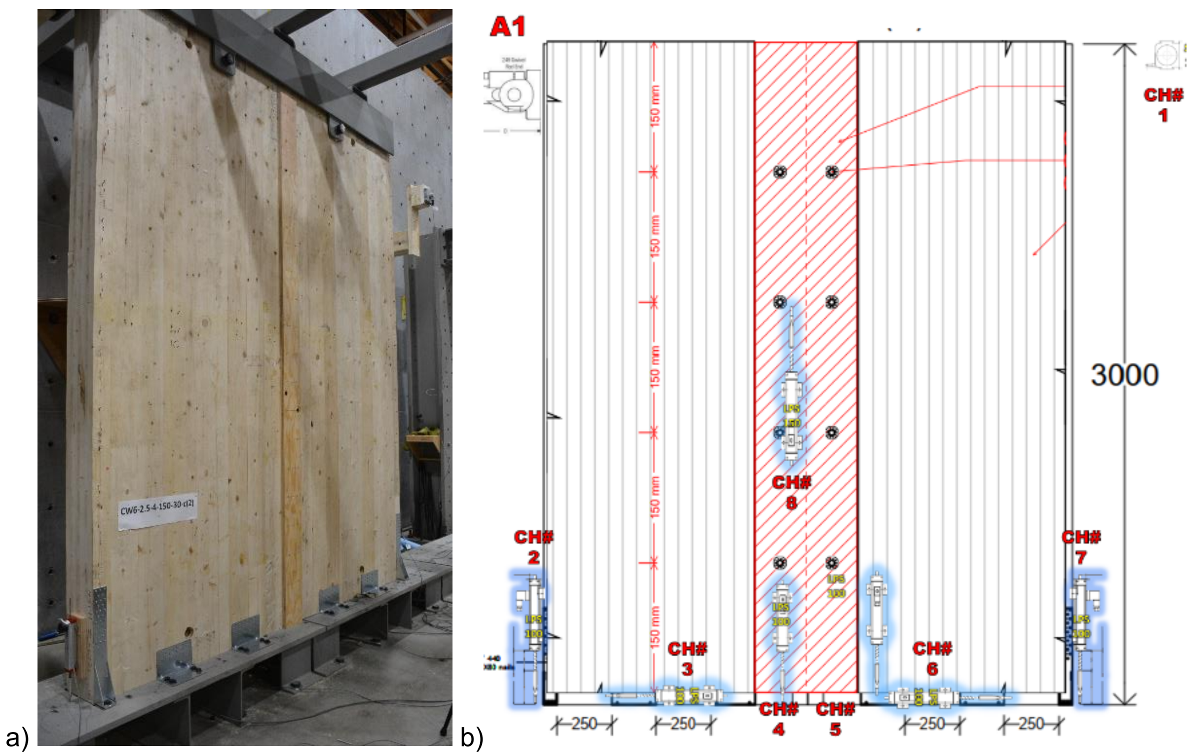

Figure 3: Coupled shear wall test setup: a) photo, b) instrumentation

3 RESULTS AND DISCUSSION

The load-deflection curves from monotonic and reversed cyclic tests are shown in Figure 4 and Figure 5, respectively. The key test results for the wall behaviour are listed in Table 1.

Under monotonic tests, load-deflection curves were initially linear up to 15% of peak loads (Figure 4) and then the non-linearity began due to deformations at the wall-to-floor HD and SB connectors and wall-to-wall spline joints. The peak loads under monotonic loading, F_{max} with panel aspect ratios of 2.5 and 3.5 ranged from 73-74 kN and 35-37 kN, respectively for the coupled shear walls and 28-41 kN and 25-33 kN, respectively for the single shear walls. The coupled walls reached their peak loads at a displacement of between 84-115 mm, whereas the peak displacement observed in single walls were observed quite dispersed ranging from, 59-140 mm. The coupled walls with an aspect ratio of 2.5 showed that the increase in gravity loads caused an increase in average capacity in all walls by 7-8%, except in CW6 where the capacity reduced by 9% when compared to CW5.

Table 1: CLT shear wall test results

Wall #	Aspect [-]	# of HD [-]	# of SB [-]	Nail space [-]	Ver. Loads [kN/m]	Test [-]	F_{max}[+] [kN]	F_{max}[-] [kN]	d_{Fmax}[+] [mm]	d_{Fmax}[-] [mm]
SW1	2.5	2	2	-	20	Mon	41	-	111	-
SW2	2.5	2	1	-	20	Mon	33	-	103	-
SW3	2.5	2	2	-	20	Cyc	39	42	91	91
SW4	2.5	2	1	-	20	Cyc	35	38	91	83
SW5	3.5	2	1	-	20	Mon	25	-	140	-
SW6	3.5	2	1	-	20	Mon	24	-	71	-
SW7	3.5	2	1	-	20	Cyc	22	24	120	122
SW8	3.5	2	1	-	20	Cyc	26	24	70	70
CW1	2.5	2	4	300	20	Mon	73	-	110	-
CW2	2.5	2	4	150	20	Mon	74	-	108	-
CW3	2.5	2	4	300	20	Cyc	65	67	94	103
CW4	2.5	2	4	300	30	Cyc	69	74	90	112
CW5	2.5	2	4	150	20	Cyc	75	75	117	117
CW6	2.5	2	4	150	30	Cyc	69	67	63	63
CW7	2.5	2	2	300	20	Cyc	51	50	60	85
CW8	2.5	2	2	300	30	Cyc	54	54	88	90
CW9	2.5	2	2	150	20	Cyc	58	58	63	90
CW10	2.5	2	2	150	30	Cyc	62	64	63	63
CW11	2.5	2	2	75	20	Cyc	73	74	63	63
CW12	3.5	2	2	300	20	Mon	35	-	115	-
CW13	3.5	2	2	300	20	Cyc	27	28	63	117
CW14	3.5	2	2	150	20	Cyc	35	35	63	63
CW15	3.5	2	2	75	20	Cyc	52	47	86	63
CW16	3.5	2	2	300	20	Mon	37	-	83	-
CW17	3.5	2	2	300	20	Cyc	40	38	86	63
CW18	3.5	2	2	150	20	Cyc	39	43	63	63
CW19	3.5	2	2	75	20	Cyc	60	54	90	90

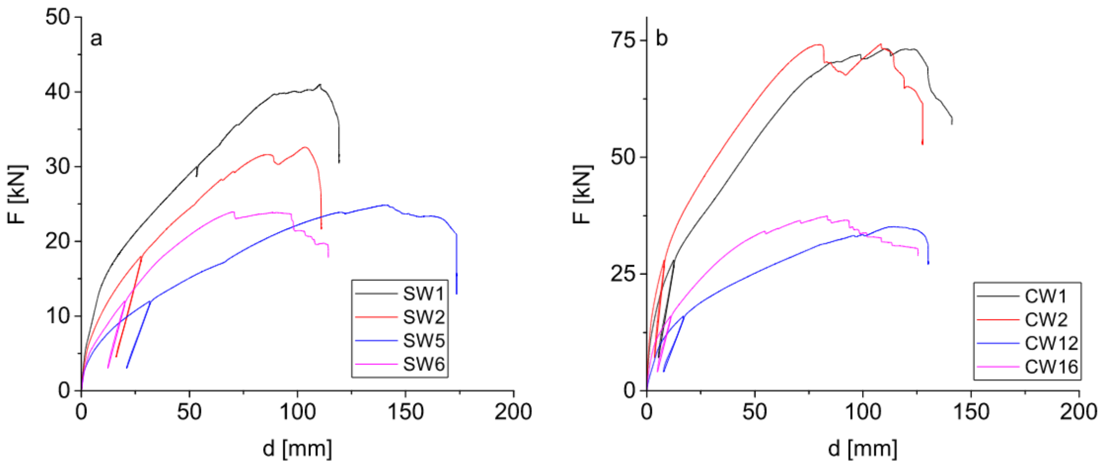

Figure 4: Load-deflection curves for monotonic tests: a) single walls, b) coupled walls

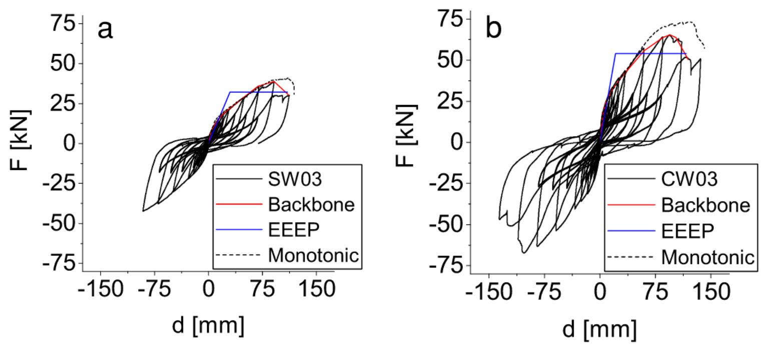

Figure 5: Typical load-deflection curves from reversed cyclic tests: a) single walls, b) coupled walls

The aspect ratios (2.5 and 3.5) had a significant influence on the peak loads under reversed cyclic loading. The average F_{max+} and F_{max-} for the coupled walls with an aspect ratio of 2.5 was 54% and 59% higher, respectively compared to walls with an aspect ratio of 3.5. Similarly, the average F_{max+} and F_{max-} for the single walls with an aspect ratio of 2.5 was 52% and 67% higher, respectively compared to walls with an aspect ratio of 3.5. Among the coupled walls CW5, aspect ratio of 2.5 with 4-SBs had the highest F_{max+} = 76 kN and displacement at peak loads d_{Fmax+} = 117 mm. For the cyclic tests, the average capacity F_{max} increased significantly as nail spacing was decreased in the splines, 15% and 46% in CW9 and CW11 when compared to CW7. However, the average d_{Fmax} was not consistent, the average d_{Fmax} was 6% higher in CW9 and 13% lower in CW11 when compared to CW7. The single wall SW3, with an aspect ratio of 2.5 and 2-SBs had the highest F_{max+} = 39 kN and displacement at peak loads d_{Fmax+} = 91 mm.

The number of HD nails was reduced by 50% in four coupled walls tests and by 50-75% in three single walls. The walls with an aspect ratio of 3.5 had only been tested for the reduced HD nails to facilitate the yielding in HDs nails (a desirable failure mechanism to allow rocking of the wall panels) and also to evaluate the variation in capacity and post-yield deformation. The coupled wall CW16, tested under monotonic loads, had the same configuration as CW12 except for 50% fewer HD nails; The F_{max} was similar for both walls, however, the deformation at peak loads d_{Fmax} for CW16 was reduced by 27%. The monotonic test results observed in single walls were inconsistent when the number of nails was reduced by 50 to 75%. The F_{max} for the SW6 (50% reduction in HD nails) was 24 kN, 4% less than SW5 (with 100% HD nails). Nevertheless, the displacement at peak loads d_{Fmax} reduced linearly with the reduction in nails, a 23% and 50% lower d_{Fmax} observed when the HD nails reduced by 75% and 50%, respectively.

4 CONCLUSIONS

In this study, the performance of single-storey platform-type CLT shear walls was tested under quasi-static monotonic and reversed cyclic loading. Shear walls with two aspect ratios tested with various types of hold-downs, and shear brackets, plywood surface splines, and gravity loading. Shear wall connections: hold-downs, shear brackets, and vertical spline joints were tested under monotonic and cyclic/reversed cyclic loading. The aspect ratio of the wall panel and the size of the hold-down had a significant influence on the capacity and displacement of the CLT shear walls. The results presented herein will be helpful for the design of low to mid-rise CLT platform construction.

Acknowledgements

The project was supported by Natural Resources Canada (NRCAN) through the Green Construction Wood (GCWood) program and by the Government of Canada through a Canada Research Chair.

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