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EXPERIMENTAL PARAMETER STUDY ON CLT SHEAR WALLS WITH HIGH-PERFORMANCE SELF-TAPPING SCREW CONNECTIONS

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ABSTRACT: This paper presents experimental investigations on the performance of Cross-laminated Timber (CLT) shear walls with high-performance self-tapping screw (STS) connections. Hold-down and panel-to-panel connections were tested under monotonic and quasi-static reversed cyclic loading. The derived design values of STS were significantly higher than those calculated using the current Canadian Standard for Engineering Design in Wood. Subsequently, a total of twenty-six CLT shear walls (in single-, double- and triple-panel configurations) were tested under monotonic and quasi-static reversed cyclic loading. The shear walls consisted of 5-ply, 139 mm thick and 3 m high CLT panels with aspect ratio of either 2:1 or 3:1, and different numbers of STS in the connections. The shear wall strength and corresponding displacements increased with the number of screws in hold-downs and spline joints, further it increased with number of connected panels and decreased as a function of aspect ratio.

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

1 INTRODUCTION

High-rise mass timber construction has been successfully re-introduced across North America [1]. The 2020 version of National Building Code of Canada [2] and the 2021 version of the International Building Code [3] have increased the building height limit of mass timber structures up to 12 stories and 18 stories, respectively.

Extensive experimental and analytical research on the seismic performance of Cross-laminated Timber (CLT) shear walls was conducted over the last decade, demonstrating that connections govern their strength, stiffness, ductility, and energy dissipation. A capacity design philosophy is normally employed, where the dissipative components are designed to be ductile and absorb energy while non-dissipative parts are over-designed [4].

However, previous work on self-tapping screws (STS) found connections to be between 2 and 6 times stronger than the design values derived using standard provisions [5]. In consequence, the energy-dissipative connections may be severely over-designed and remain elastic during seismic events. The resulting increased stiffness potentially increases the seismic force demands and put the ‘capacity-protected’ structural components at risk.

2 OBJECTIVE

In this research, CLT shear walls with high-performance STS connections were investigated to determine the strength, stiffness, yield point, failure point, and ductility of a rocking wall system as a function of the number of STS in the connection, and subsequently develop appropriate and economic design values for the STS.

3 EXPERIMENTAL INVESTIGATIONS

3.1 CONNECTION TESTS

The CLT panels were strength grade 191V2, 5-ply, 139 mm thick. Both HD and SB were made of custom steel plates of grade 44W/300W. The HD was 186 mm wide, 435 mm high, with a 70 mm long base plate, thickness 12.7 mm (vertical part) and 25 mm (base plate). 21 ø13 mm holes were prepared in the vertical part for installing STS; the base plate had 2 ø19 mm holes for installing bolts. The SB was 340 mm wide with a length of the vertical and horizontal plate equal to 127 mm and 89 mm, respectively, and a constant thickness of 6.35 mm. It had 8 ø 11 mm vertical slots, for installing the STS to eliminate any uplift resistance. STS ø12×120 mm were used as fastener for HDs and SBs. The panel-to-panel vertical splines was surface mounted 25×140 mm D-Fir plywood attached with ø8×100 mm STS. Figures 1, 2 and 3 show the drawings for each connection detail.

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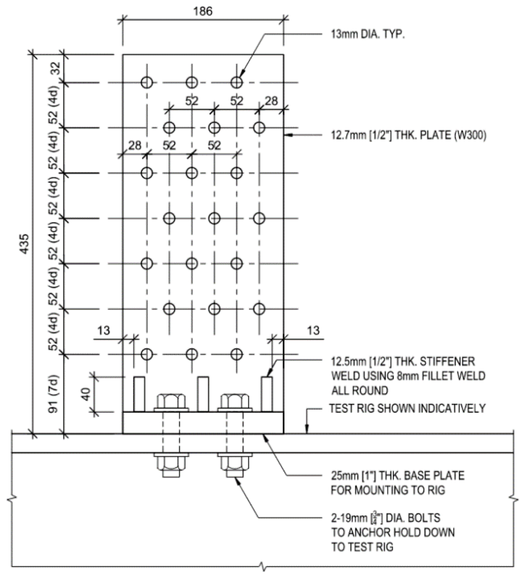

Figure 1: *Hold-down*

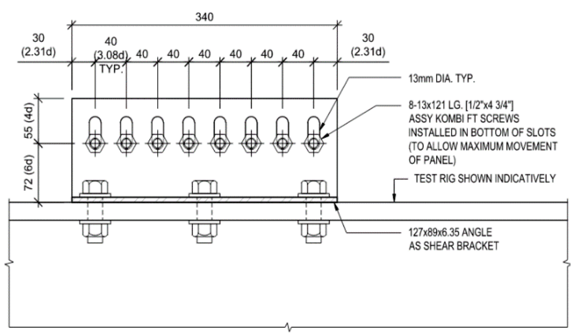

Figure 2: *Shear bracket*

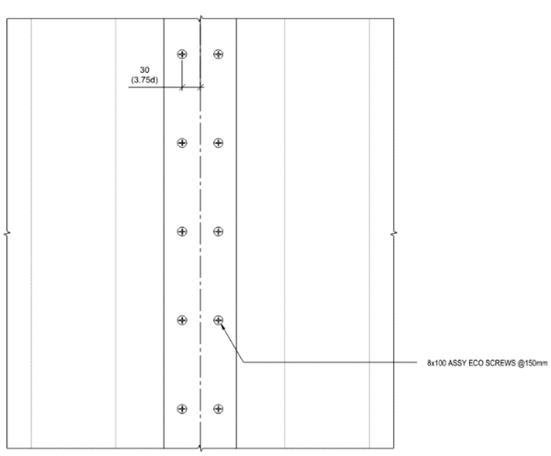

Figure 3: *Spline joint*

3.2 SHEAR WALL TESTS

A total of 26 CLT shear walls were assembled and tested, including 4 single panel (SP), 16 coupled panel (CP), and 6 triple panel (TP) walls. The shear walls consisted of 3 m tall CLT panels, either 1.5 m or 1.0 m wide, resulting in aspect ratios of 2:1 and 3:1, respectively, see Figure 4. All wall configurations were tested with two HDs, CP and TP shear walls with one HD on the outside edge of the outside panels. For all configurations, one SB was mounted at the midpoint of each CLT panel. The CP and TP shear walls were coupled vertically using a screwed plywood spline joint on one side. The splines were attached to the panels with either 13, 16, or 19 screws per panel edge side (total 26, 32, 38 screws per spline). All SBs were fully attached to the walls with 8 STS, while the HDs had 9, 11, 13, or 15 screws each. The screw types matched the connection tests.

The lateral loads were applied by a actuator at the top of the left wall panel through a steel plate connected to steel H-beam. The H-beam was placed on two wooden blocks at the center of each panel. A 10 kN/m superimposed vertical gravity load was applied representing a moderately loaded wall in a two-storey system. This gravity load system simultaneously prevented out-of-plane horizontal movements in addition to allowing for the lateral movement of the shear wall.

Figure 4: *Full-size triple-panel CLT shear wall tests*

3.3 ANALYSIS

For each connection and shear wall test, the strength, stiffness, displacement, and ductility were determined and analyzed. The maximum forces (load-carrying capacity) from both positive and negative phases (F_{max+} and F_{max-}) were recorded for the reversed cyclic tests. The displacements corresponding to these forces, d_{Fmax+} and d_{Fmax-} were the measurements at the top right corner of the shear walls.

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4 RESULTS

4.1 HOLD-DOWN RESULTS

The typical failure mode of the HD was fastener yielding. The corresponding hysteresis behavior of the HD connections combined with a representative monotonic curve are presented in Figure 5. Until reaching capacity, the HD showed little stiffness degradation with increasing displacement. Beyond the capacity, distinct nonlinear degradation and pinching behavior in cyclic re-loading was observed. Throughout the entire loading history, the HDs showed almost the same high unloading stiffness.

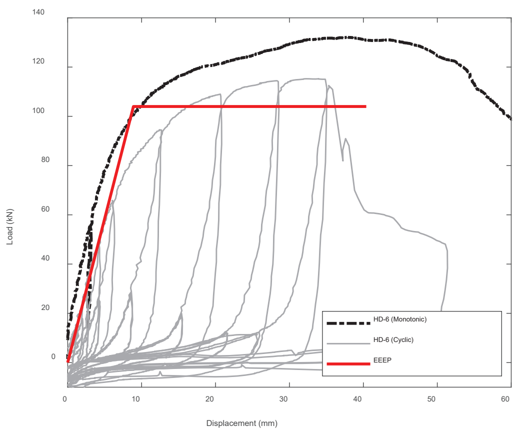

Figure 5: *Typical HD connection hysteresis curve*

4.2 SPLINE RESULTS

The hysteresis behavior of typical spline joints combined with a representative monotonic curve is shown in Figure 6. The curves showed high re-loading stiffness combined with degradation and pinching effects. A small amount of cyclic strength degradation was also observed in the tailing cycles with reduced displacement. Displacement above approximate 30 mm caused brittle failure. Overall, both the strength and displacement capacities under cyclic loading were reduced when compared to monotonic loading. Load-carrying capacity linearly increased with the number of STS. From 2 screws to 8 screws, all strength parameters increased about fourfold regardless of the loading type, e.g. F_{max} increased from 15 kN to 57 kN under monotonic loading and from 11 kN to 46 kN under cyclic loading.

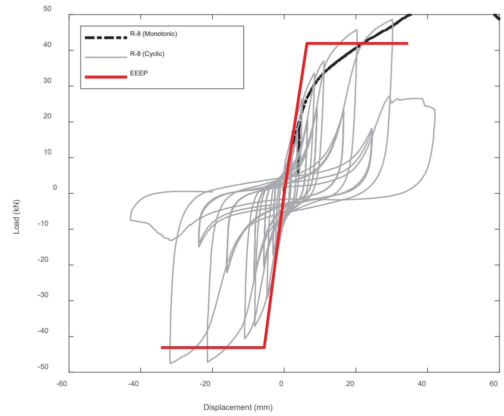

Figure 6: *Typical spline hysteresis curve*

4.3 CONNECTION DESIGN VALUES

Since CSA O86 does not provide any design guidance for STS, the design provisions for lag screws are commonly used by practitioners to calculate the design values. These values are dramatically smaller than the experimental results, see Table 1, confirming that STS in energy-dissipative connections may be severely over-designed and are likely to remain elastic during seismic events. As a consequence, the non-dissipative connections that require capacity protection are also over-designed; however the wood components may no longer be capacity protected.

As an alternative, the test results were used to derive expected performance values: $R_d = k_{mod} \cdot R_k / \gamma_{mod}$, where R_d is the design resistance, R_k is the characteristic value of load-carrying capacity of a single screw, k_{mod} is modification factor taking into account the duration of load and moisture content, and Υ_{mod} is the material property factor. Herein, k_{mod} = 0.9 (for short-term action and dry condition) and Υ_{mod} = 1.3 were applied. R_k was determined from the test data, assuming that the strength distribution follows a lognormal distribution. The expected performance design values for a single screw in HDs and spline joints were derived using Equation (1) based on $R_{k0.05}$, see Table 5. The design resistance for a single HD screw was 11.8 kN, which was still only 69% of the experimental values, but significantly better than that calculated based on CSA O86 provision (almost 3 times higher). Similarly, the derived resistance for spline joint screw was only 70% of the experimental value but more than 3 times higher than the values based on CSA O86 provisions.

Table 1: *Connection test results*

Connection	Screw Type	Characteristic strength (kN/screw)			Resistance (kN/screw)	
		5th percentile	Mean	95^{th}%	Derived	CSA O86
HD	12Ø×120 ASSY Kombi	17.1	19.4	22.1	11.8	4.1
Spline	8Ø×100 ASSY Eco	4.6	5.0	5.5	3.2	0.9

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4.4 SHEAR WALL TEST RESULTS

The effects of four parameters: i) the number of STS in HDs, ii) the number of STS in spline joints, iii) the wall aspect ratio, and iv) the number of panels (SP, CP, TP) on the CLT shear wall performance were investigated. The results are summarized in Table 2; the monotonic load-displacement curves are shown in Figure 7.

Under monotonic loading, the curves were initially linear up to 5 mm displacement when nonlinear behavior started due to plastic deformation in the spline joints connections. After reaching the peak, the curves exhibited a sudden drop because of the HD screws exceeding their capacity.

Figure 7a provides a comparison between SP and CP with different number of STS in HDs (9 vs 15). All shear walls exhibited similar performance at the beginning of loading but reached different load-carrying capacities, F_{max}, as a function of number of STS, following a similar linear relationship observed in HD connection tests. For example, for CP3 and CP1 which had 11 and 15 HD screws, respectively, F_{max} increased from 163 kN to 213 kN (31% increase). The displacement at maximum loads, d_{max}, as well as yield point and failure point were also a function of number of STS.

The impact of the number of STS in the spline joints is illustrated in Figure 7b for CP walls with 13, 16, and 19 STS in the spline and the same HD configuration (9 STS). While the performance of CP4 and CP5 with 19 and 16 screws was similar in terms of strength, displacement, and stiffness, when the number of spline screws was reduced to 13 in CP6, a distinct reduction in performance was observed: when compared with CP4 of 19 screws: F_{max} was reduced by 19%, d_{max} was reduced by 14%,

The significant impact of the panel aspect ratios (2:1 for CP3/5 and 3:1 for CP11/12) on the performance of shear walls with the same connector configuration is shown in Figure 7c. F_{max} for CP3 and CP5 were 23% and 39% higher than those from CP11 and CP 12, respectively.

Finally, the influence of the number of panels for panels with an aspect ratio of 3:1 is illustrated in Figure 7d. Going from CP to TP with the additional SB and spline joint, a higher capacity and stiffness was achieved. F_{max} for TP1 and TP2 were 24% and 42% higher when compared to those from CP11 and CP12

4.5 SHEAR WALL FAILURE MODES

The wall failure modes are presented in Figure 8. All coupled walls exhibited rocking behavior, as seen in Figure 8a. With increasing lateral displacement, some minor plastic deformations occurred in SB, see Figure 8b. Similar to the failure modes observed from connection tests, at the end of each test, the screws in HD were heavily damaged with a failure combination of yielding and withdrawal, and eventually developed plastic hinges, as seen in Figure 8c. Caused by the relative displacement between adjacent panels in the multi-panel tests (CP and TP walls), failure of spline wall-to-wall joints were observed, including screw yielding combined with fracture (TP3), as seen in Figure 8d.

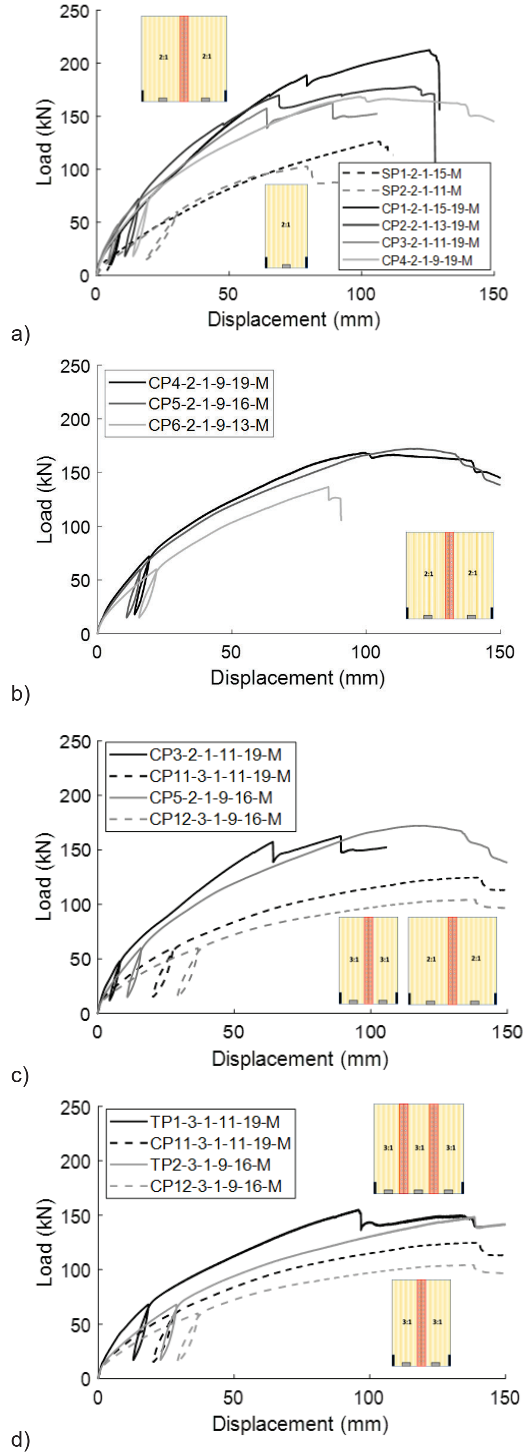

Figure 7: *Monotonic load-displacement curves with effects of: a) HD screws; b) spline screws in CP; c) aspect ratio effect; and d) panel number*

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Table 2: *Shear wall test results*

Test ID	F_{max+} [kN]	F_{max-} [kN]	d_{max+} [mm]	d_{max-} [mm]
SP1-2-1-15-M	126.2	-	106.5	-
SP2-2-1-11-M	102.6	-	79.6	-
SP3-2-1-15-C	119.8	-115.7	91.1	-91.1
SP4-2-1-11-C	125.0	-99.9	112.1	-70.1
CP1-2-1-15-19-M	212.6	-	125.6	-
CP2-2-1-13-19-M	177.8	-	119.8	-
CP3-2-1-11-19-M	162.7	-	89.0	-
CP4-2-1-9-19-M	168.4	-	99.3	-
CP5-2-1-9-16-M	172.1	-	116.6	-
CP6-2-1-9-13-M	136.6	-	86.0	-
CP7-2-1-11-19-C	142.4	-153.5	75.0	-75.1
CP8-2-1-11-19-C	174.4	-164.5	70.0	-70.0
CP9-2-1-9-16-C	149.6	-150.8	70.0	-70.1
CP10-2-1-9-16-C	161.1	-146.4	70.0	-70.0
CP11-3-1-11-19-M	124.9	-	140.0	-
CP12-3-1-9-16-M	104.3	-	137.0	-
CP13-3-1-11-19-C	109.7	-106.6	104.1	-104.1
CP14-3-1-11-19-C	102.8	-99.1	80.1	-104.1
CP15-3-1-9-16-C	89.8	-83.6	80.0	-80.1
CP16-3-1-9-16-C	93.6	-96.2	104.1	-104.1
TP1-3-1-11-19-M	155.2	-	95.9	-
TP2-3-1-9-16-M	148.6	-	138.6	-
TP3-3-1-11-19-C	147.5	-132.4	80.1	-80.1
TP4-3-1-11-19-C	150.6	-130.7	104.1	-104.1
TP5-3-1-9-16-C	124.4	-116.2	80.1	-80.1
TP6-3-1-9-16-C	138.0	-122.3	104.1	-104.1

5 CONCLUSIONS

The performance of CLT shear walls with high-performance self-tapping screw connections was investigated under monotonic and cyclic loading. Connection tests and full wall tests were carried out to derive design values of STS. The following conclusions can be drawn:

1) The strength and stiffness of connections were found to linearly increase with the number of screws. The derived design values of STS from the connection tests under short-term cyclic loading were about 3 times higher than those calculated in accordance with the current CSA O86 provisions for lag screws. This shows that STS in energy-dissipating connections are extremely over-designed when designed in accordance with CSA O86 and may not yield as intended.

2) The strength capacity and the corresponding displacement of the shear walls decreased linearly with the reduction of screws in HD. The same trend was also found for the spline joints connections.

3) The aspect ratio had a significant impact on the performance of the coupled shear walls. 2:1 aspect ratios increased load-carrying capacity by 40%, and decreased deformation capacity by 30%.

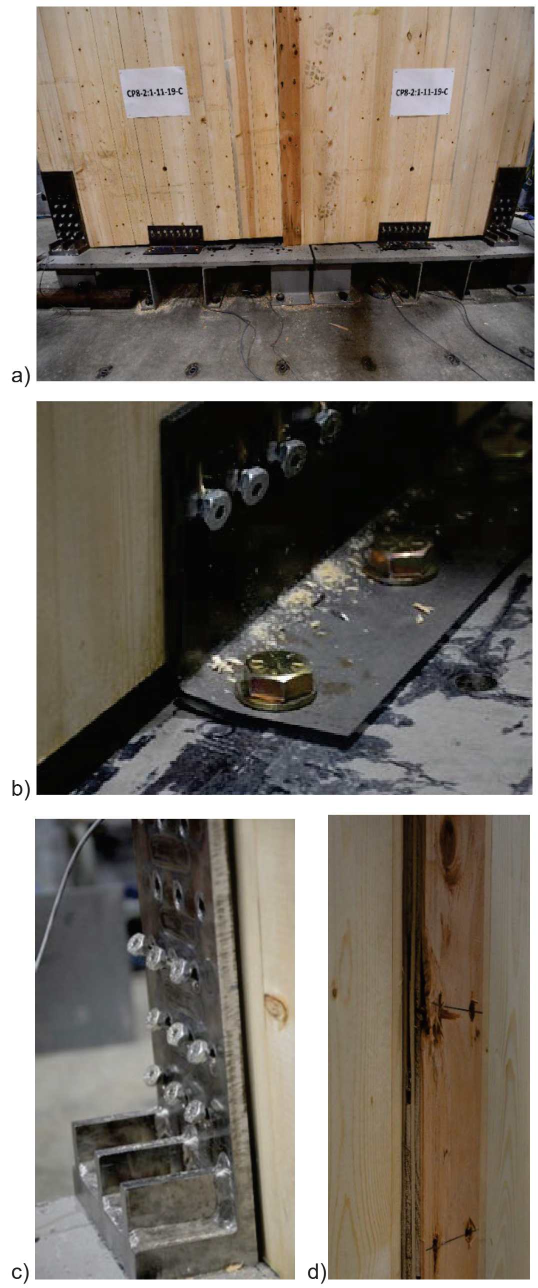

Figure 8: *Observed damages: a) wall rocking, b) deformation of SB, c) screw failure in HD, d) screw failure in spline*

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