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EVALUATION OF THE STRUCTURAL PERFORMANCE OF SHEAR WALLS BUILT WITH MULTI-LAYER COMPOSITE LAMINATED PANELS

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ABSTRACT: An innovative multi-layer composite laminated panel (CLP), comparable to CLT, has been developed by combining Laminated Strand Lumber (LSL) and dimension lumber to overcome the rolling shear failure while maintaining high mechanical performance and the aesthetic appearance of natural wood. Experimental investigations have been conducted to assess the lateral resistance of CLP connections using self-tapping screws, as well as full-scale CLP shearwalls with varying connection layouts to achieve the target kinematic wall behaviour under both monotonic and cyclic loading. The findings indicate that incorporating Laminated Strand Lumber (LSL) in the laminations has a substantial impact on the mechanical properties of the connections. Replacing lumber with Laminated Strand Lumber (LSL) in the core layer exhibited a remarkable increase in stiffness and strength, and tended to fail in a ductile manner, while the utilisation of LSL in face layers enhanced stiffness and strength, but reduced ductility. Furthermore, the shear wall layout and the number of self-tapping screws in each connection were found to dramatically affect the overall structural performance of the shear wall.
KEYWORDS: Composite Laminated Panel (CLP), Shear walls, Connection properties, Mechanical properties.

1 INTRODUCTION

Over the past decade, mass timber panels (MTPs) have been increasingly implemented in construction as a sustainable and cost-effective building material. The best-known MTP product is cross-laminated timber (CLT), made from sawn lumber planks orthogonally glued together. As a viable product in the mid- and high-rise construction market, CLT offers a promising solution for a diverse range of structural applications in both all-wood and wood-hybrid buildings, including roof, floor, and wall assemblies, and is increasingly replacing other traditional materials such as steel and concrete. The panels can be easily connected to other structural members or materials using fasteners and connectors. The invention of CLT has significantly advanced panelised building technologies.

Although CLT possesses both good dimensional stability and the ability to transfer forces in two-way directions due to its crosswise lamination, its transverse layers are prone to rolling shear failure under out-of-plane loading. This is why new generations of MTPs are being driven forward. The prototype of the next generation of MTP is a combination of lumber and Structural Composite lumber. An innovative multi-layer composite laminated panel has recently been developed by combining laminated strand lumber (LSL) and dimensional lumber to overcome the rolling shear failure while maintaining the high mechanical performance and aesthetic appearance of natural wood, which is referred to here as composite laminated panel (CLP). Test results [14] show that the shear strength, bending stiffness, and moment resistance of CLP were up to 143%, 43%, and 87% higher than their counterparts of regular CLT, respectively, and the use of LSL in transverse layers could eliminate the potential rolling shear failure in CLT. The above promising performance of CLP has prompted researchers to further explore the potential for using CLP as a means of providing lateral load resistance in building systems.

Currently, the most commonly used approach for MTP walls is to consider the MTP as a rigid body and the lateral resistance of the whole wall is mainly governed by connection systems. This theory is based on a set of experimental investigations on full-scale CLT shear walls subjected to lateral loads. [3-4, 7-8] As there are presently no standardized provisions for determining the lateral resistance of CLP shear walls, it is necessary to further investigate this innovative engineered wood product, CLP, used as a lateral load resisting system (LLRS). Additionally, two types of LLRS are typically considered in CLT buildings: monolithic shear walls and multi-panel shear wall configurations. However, in North America, the multi-panel wall configurations are favoured over monolithic shear walls, primarily due to its transportation convenience and potential to enhance energy dissipation in the panel joint connections. (Masroor et al. [13]). Hence, the structural performance of full-scale CLP coupled-panel shear walls were fully evaluated through both monotonic and cyclic loading tests. Accurate quantification of the CLP screw connections was also presented, as it is essential for understanding the shear wall behaviours.

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2 MATERIALS

2.1 SPECIMENS

In total, 21 three-ply MTPs were manufactured by InnoTech Alberta (Edmonton, AB), consisting of 19 three-ply CLP panels with 4 different layer arrangements, and 2 three-ply CLT panels. Figure 1 gives an illustration of the four CLP layups. All shear wall specimens were assembled by placing two panels of the same type side-by-side and connecting them to each other using splice connections. Four shear wall specimen types were fabricated with these four layups respectively. Two specimens were produced for each type with one subjected to a monotonic test and the other to a cyclic test. In addition, one CLT shear wall was also tested as a reference under cyclic loading. Afterwards, specimens for connection tests were obtained by cutting the CLP panels from the undamaged parts of the panels after the shear wall tests.

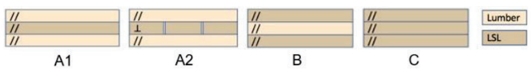

Figure 1: Illustration of CLP layups

2.2 FASTENERS

The study utilized SDS25300 self-tapping screws (STS) with a diameter of 6.35 mm and a length of 63.5 mm, as illustrated in Figure 2. They were made of SAE J403 low-carbon steel wire, possessing a bending yield strength of 1130 MPa (164000 psi), as stated by the manufacturer (SAE 2014). This type of screw has high strength in structural application with no-predrilling installation requirements, which is recommended for wood-to-wood and wood-to-steel connections.

Figure 2: SDS25300 wood screws (courtesy of Simpson Strong-Tie®)

2.3 STEEL PLATES

The steel side plate (HRS416Z) depicted in Figure 3 was manufactured from galvanized steel complying with ASTM A653, SS Grade 33, with a minimum yield strength of 227 MPa (33,000 psi) and a minimum ultimate tensile strength of 310 MPa (45,000 psi) based on the data provided by the supplier.

Figure 3: Steel side plate-HRS416Z (courtesy of Simpson Strong-Tie®)

3 EXPERIMENTAL PROGRAM

This section presents the experimental campaign conducted to investigate the mechanical properties of CLP screw connections as well as full-scale CLP shear walls. All tests were carried out at the Wood Science and Technology Centre, University of New Brunswick, in Fredericton, Canada.

3.1 CONNECTION TESTS

3.1.1 Test setup and procedure

The screw connection tests included 60 specimens with half being subjected to monotonic loading and the other half to reversed cyclic loading. CLP specimens with 4 different layups were tested, as well as a CLT specimen group used as a reference. Table 1 shows the connection specimen configurations. Three replicates were tested for each configuration.

The specimen dimensions were 200 mm × 120 mm × 107 mm for those with the load applied parallel to the face layer grain and 200 mm × 200 mm × 107 mm for those with the load applied perpendicular to the face layer grain. Each side of the CLP member had a 3mm thick steel plate attached using two STSs (SDS HEAVY-DUTY SDS25300), leading to a total of 4 STSs in 2 single-shear connections per specimen. The test setup is shown in Figure 4.

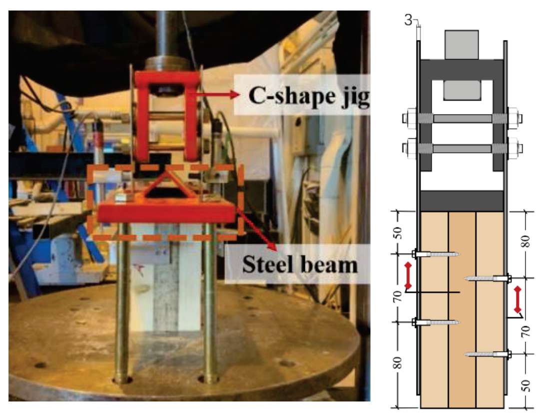

Figure 4: Connection test setup

The monotonic test was conducted at a loading rate of 5 mm/min prior to the cyclic test to obtain a reference displacement value for the cyclic test. The cyclic test was conducted using the reversed loading protocol based on the displacement-controlled loading procedure outlined in ASTM E2126 (Method B) [1].

Table 1: Test matrix for connection tests

Test series	CLP layup	Loading direction	Loading type
A1	L-LSL (par)-L	Par/Perp	m/c
A2	L-LSL (perp)-L	Par/Perp	m/c
B	LSL-L-LSL	Par/Perp	m/c
C	LSL-LSL-LSL	Par/Perp	m/c
CLT	L-L-L	Par/Perp	m/c

Note: "L" is lumber; "LSL" is laminated strand lumber. "Par" indicates the load applied parallel to grain, "Perp" indicates the load applied perpendicular to grain. "m" and "c" means monotonic and cyclic, respectively.

3.2 SHEAR WALL TESTS

3.2.1 Test setup

The CLP walls were anchored to a steel foundation using hold-downs and base shear connectors. As the rollers had horizontal translational and rotational degrees of freedom, four rollers (Ø 7 mm) were placed at the bottom of the panel in order to ensure that the movement of the panel was not restricted and also to prevent friction. A C-shaped channel was attached to the load cell and the actuator, transferring the lateral load through the pin joints to the CLP shear walls. The C-shaped channel was attached to the top of the CLP panel using two 25.4-mm bolts, evenly spaced. It should be noted that lateral supports were used in all the tests to prevent the out-of-plane movement of the wall by using parallel rigid bars and wooden blocks between the guides and specimen. Figure 5 illustrates the shear wall test setup. The shear wall test used Simpson Strong-Tie® steel brackets (Bracket AE116: 90 × 48 × 116 mm), hold-downs (HDU8: 422.5 × 90 × 35 mm). The steel plates used in the panel joint connections were identical to those used in the connection tests. Additionally, the same type of screw was used for all connections, as those tested in the connection tests. Table 2 describes all of the wall configurations tested.

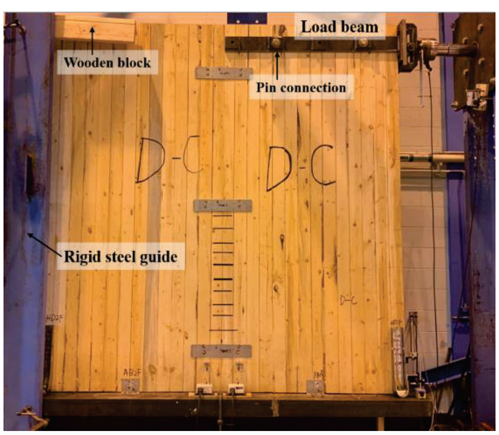

Figure 5: Shearwall test setup

Six linear variable differential transformers (LVDTs) were installed on the specimen as shown in Figure 6.

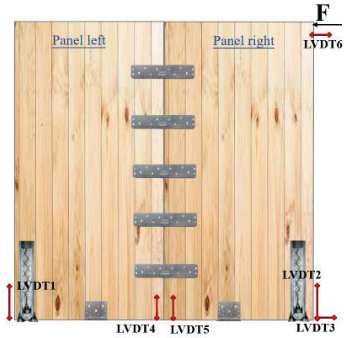

Figure 6: The shear wall sketch with measurement instrumentation

3.2.2 Test Configurations

In this study, three main connections in the shearwalls were panel-to-panel, hold-down, and base shear connections. All these connections share a common characteristic that they can be represented by a steel-to-wood screw connection.

To investigate the kinematic wall behaviour and aspect ratios of the wall panels, different connection layouts were designed by varying number of STS in these connections. Specifically, two target kinematic wall behaviours were considered according to [3]: (a) **Coupled wall** - the wall segments of the coupled wall are connected with a relatively weak panel joint (yielding of the panel joint occurs before yielding of the hold-down/base shear connections) and the wall segment can slide relative to each other under lateral loading; (b) **Single wall** - two panels are connected with a stiff and strong panel joint (yielding of the panel joint occurs after yielding of the hold-downs/base shear connections) and the wall would rotate as a single panel under lateral loading, as illustrated in Figure 7. Identical heights of 2400 mm and widths of 1200 mm were adopted for all CLP panel segments, resulting in an aspect ratio of 2:1 for coupled panel kinematic behaviour and 1:1 for single wall behaviour.

Table 2: Shearwall test matrix

Test Wall ID	No. of screws in brackets	No. of screws in Hold-down	No. of vertical joints	Kinematic behaviour
A1-m	6	9	10	Single
A1-c	6	9	10	Single
A2-m	6	8	6	Coupled
A2-c	6	8	6	Coupled
B-m	6	7	6	Coupled
B-c	6	7	6	Coupled
C-m	6	7	6	Coupled
C-c	6	6	6	Single
D-c	6	8	6	Coupled

A total of 9 shear wall tests were conducted, four of which were monotonic tests and the remaining five being cyclic tests. Table 2 provides a detailed overview of the wall

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configurations tested. All the tests were performed with no vertical load applied.

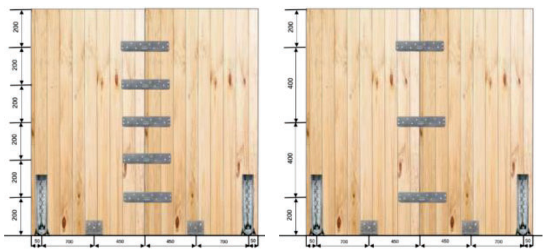

Figure 7: Kinematic wall behaviours: (a) Single wall behaviour (SW); (b) Coupled-panel behaviour (CP) (dimensions in mm)

4 RESULTS AND DISCUSSION

4.1 CONNECTION TESTS

4.1.1 Monotonic tests

Figure 8 and Figure 9 present the load-displacement curves (dashed lines) and their mean value curves (solid lines) of specimens subjected to monotonic loading parallel and perpendicular to the grain, respectively. The measured load values were divided by the total number of screws used to assess the capacity of an individual screw.

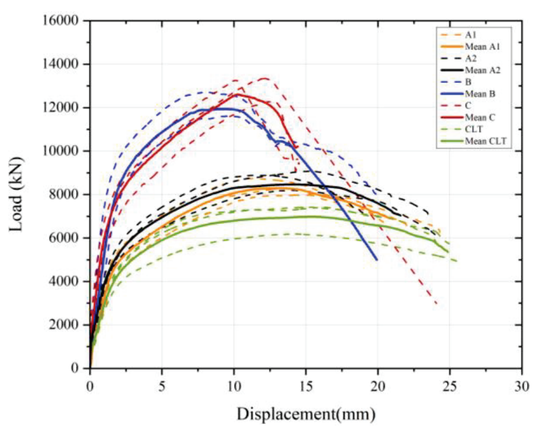

Figure 8: Load-displacement and mean value curves under monotonic load applied parallel to grain (per screw)

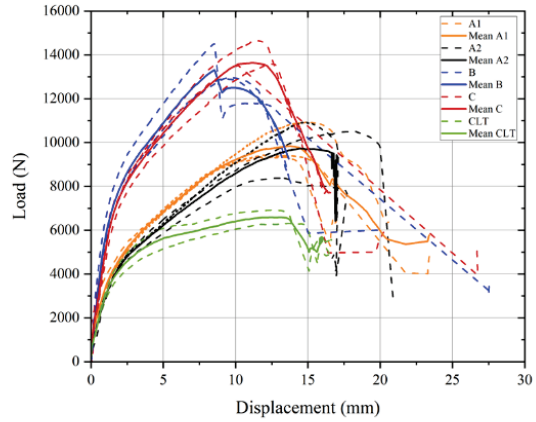

Figure 9: Load-displacement and mean value curves under monotonic load applied perpendicular to grain (per screw)

The presented figures highlight the distinctive behaviours of two test groups, categorized as layup A and traditional CLT in the first group, and layups B and C in the second. It can be observed that specimens A1, A2, and traditional CLT exhibited similar ductile behaviour, with comparable stiffness, yield strength, and ductility. Their load-displacement responses demonstrated that they could be loaded continuously beyond their yield point without significant loss of strength.

In contrast, the screws in specimens B and C demonstrated notably higher strength and stiffness but relatively low ductility. This phenomenon can be attributed to the use of LSL as the face layer of CLP. In other words, CLP with LSL on the face tends to have higher strength and stiffness than lumber on the face. This is because LSL had a higher density than sawn lumber. However, despite the higher strength of specimens B and C, the displacements at failure were extremely small. This observation becomes more apparent when the connection was loaded parallel to grain.

4.1.2 Cyclic tests

The cyclic test results are evaluated in terms of the peak load P_{peak} , yield strength P_{yield} , displacement at capacity Δ_u, displacement at yield load Δ_{yield}, stiffness K_e, ductility ratio μ and dissipated energy E_d according to the standard procedure ASTM E2126 [1]. The mechanical properties were analysed by considering both sides of the hysteresis loops. The mean values of the mechanical properties for each test group, together with the corresponding range of minimum and maximum values (values in brackets are the range of measured values), are presented in

Table 3. The cyclic mechanical properties of the testing group exhibit a trend similar to monotonic cases.

Table 3: Mechanical properties of CLP screw connections under cyclic loads

Specimen ID	K_e (kN/mm)	P_{yield} (kN)	Δ_{yield} (mm)	P_{peak} (kN)	Δ_u (mm)	E_d (kJ)	$\mu = \frac{\Delta_u}{\Delta_{yield}}$
A1-//-c	3.81	6.80	3.45	8.00	6.35	707.88	2.31
	(3.6 - 4.0)	(6.6 - 7.0)	(2.7 - 4.6)	(6.7 - 8.2)	(5.9 - 7.1)	(670.8 - 744.9)	(2.2 - 2.4)
A2-//-c	4.20	7.98	4.30	9.39	8.74	1048.66	2.08
	(3.8 - 4.4)	(7.6 - 8.5)	(3.4 - 5.0)	(9.0 - 10.3)	(8.3 - 9.0)	(852.2 - 1348.8)	(1.8 - 2.5)
B-//-c	4.11	8.63	3.98	10.22	7.44	1013.25	1.87
	(4.1 - 4.8)	(8.6 - 8.9)	(3.9 - 4.1)	(9.7 - 10.6)	(7.2 - 7.7)	(982.2 - 1030.6)	(1.8 - 1.9)
C-//-c	4.98	8.65	3.74	10.19	6.04	754.52	1.62
	(4.9 - 5.0)	(8.5 - 8.8)	(3.6 - 4.0)	(10.0 - 10.5)	(5.8 - 6.3)	(650.4 - 878.6)	(1.5 - 1.8)
CLT-//-c	3.57	6.25	3.54	7.36	10.44	910.00	2.42
	(2.7 - 4.6)	(5.4 - 6.9)	(3.0 - 4.0)	(6.3 - 8.1)	(9.2 - 11.5)	(724.6 - 1100.8)	(2.2 - 2.6)
A1-⊥-c	3.92	7.05	3.0	8.50	8.00	658.97	2.59
	(3.3 - 4.5)	(6.9 - 7.5)	(2.9 - 3.1)	(8.2 - 8.8)	(6.4 - 9.1)	(618.8 - 735.1)	(2.2 - 3.1)
A2-⊥-c	3.22	6.65	4.73	7.89	8.80	713.28	1.78
	(2.8 - 3.4)	(6.6 - 6.8)	(4.4 - 5.1)	(7.8 - 8.0)	(7.5 - 10.1)	(702.8 - 722.1)	(1.7 - 2.0)
B-⊥-c	4.56	8.99	3.83	10.58	6.88	762.28	1.67
	(3.9 - 6.1)	(7.8 - 9.6)	(3.3 - 4.3)	(9.1 - 11.4)	(6.2 - 7.9)	(714.3 - 822.2)	(1.6 - 1.8)
C-⊥-c	6.61	10.34	4.50	11.62	6.20	939.49	1.76
	(6.2 - 7.0)	(9.4 - 11.3)	(3.9 - 5.1)	(10.5 - 13.2)	(5.5 - 6.9)	(800.7 - 1120.9)	(1.7 - 1.8)
CLT-⊥-c	3.70	5.91	4.02	6.95	9.62	735.82	2.42
	(3.3 - 4.1)	(5.5 - 6.3)	(3.7 - 4.4)	(6.5 - 7.4)	(9.6 - 9.7)	(708.1 - 763.6)	(2.2 - 2.7)

4.2 SHEAR WALL TESTS

4.2.1 Failure modes

The CLP shear walls behaved almost as rigid bodies during the test. The failure modes occurred mainly at the connections under both monotonic loading and cyclic loading, as displayed in Figure 10.

For monotonic loading cases, CLPA1-m failed due to both excessive uplift and shear of screws in the hold-down, resulting in slight plastic deformation of the hold-down (right corner), along with wood crushing and wood splitting around the hold-down and angle bracket were observed, while panel joint connection behaved elastically with no visible deformation or failure.

CLPA2-m failed mainly due to brittle failures occurring at the base shear connections (wood splitting and panel edge tear-out) (see Figure 10(d)). In addition, CLPB-m showed an obvious deformation in the hold-down (right corner) with screws pull-out (see Figure 10(a)). No failure was observed on CLPC-m due to the capacity of the test frame. The test was stopped before CLPC-m could fail.

In light of these observations, although CLPA2 and CLPB were designed to have flexible panel connections, the failures were predominantly localized at the hold-downs and base shear connections under monotonic loading. Nevertheless, it is noteworthy that the panel joint connection behaved in an almost elastic manner for all cases tested.

On the other hand, the dominant failure mode observed in the cyclic loading test was the screw yielding followed by screw withdrawal at the panel joint connections. The steel plate also exhibited some degree of plastic deformation, as shown in Figure 10(e) and (f).

(a)

(b)

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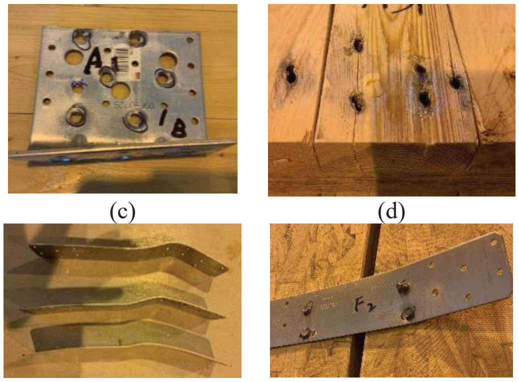


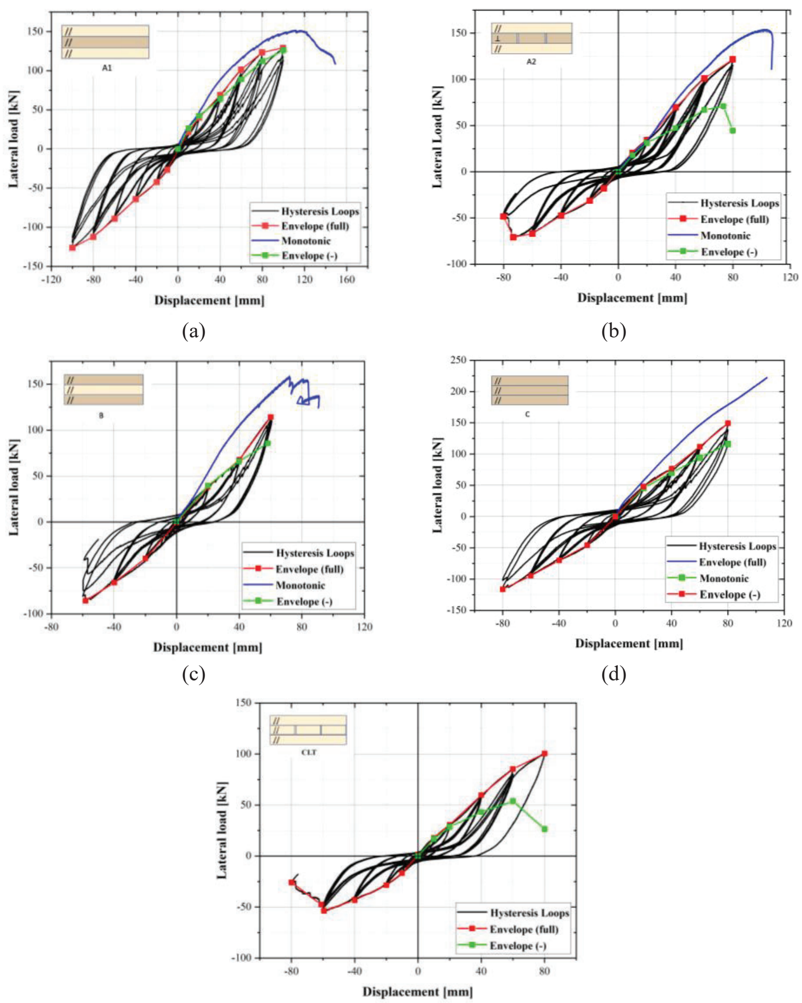
(c) (d)
(e) (f)

Figure 10: Typical failure modes of shear walls

4.2.2 Load-deformation response

The load-displacement response of each shear wall specimen under monotonic and cyclic tests was obtained. In addition, the envelope curve was generated from the cyclic load response by connecting the peak point of each load cycle. Figure 11 shows the monotonic and cyclic responses of the different wall layouts and the associated envelope curves. The EEEP curve analysis was performed in the same manner as the connection test data and the corresponding mechanical properties are given in

Table 4 and
Table 5.



(a) (b)
(c) (d)
(e)

Figure 11: Comparisons of A1-c and A1-m(a), A2-c and A2-m (b), B-c and B-m (c), C-c and C-m (d) and CLT-c

Table 4: Test results of CLP shear wall tests under monotonic loading

Wall ID	K_e (kN/mm)	P_{yield} (kN)	Δ_{yield} (mm)	P_{peak} (kN)	Δ_{peak} (mm)	Δ_u (mm)	E_d (kJ)	$\mu = \Delta_u/\Delta_{yield}$
CLPA1-m	2.32	98.62	52.32	151.73	111.51	142.53	11.96	2.72
CLPA2-m	1.78	104.71	69.21	154.00	118.06	122.19	10.97	1.76
CLPB-m	2.39	80.28	57.25	158.89	91.1	157.77	12.12	2.75
CLPC-m	2.89	-	-	222.48*	95.59*	-	-	-

Note: * testing of wall CLPC was stopped before failure occurred.

Table 5: Test results of CLP shear wall tests under cyclic loading

Wall ID	K_e(+) (kN/mm)	K_e(-) (kN/mm)	P_{peak}(+) (kN)	P_{peak}(-) (kN)	Δ_{ult}(+) (mm)	Δ_{ult}(-) (mm)
CLPA1-c	1.81	1.90	129.18	126.17	99.59	100.1
CLPA2-c	1.78	1.73	121.77	70.92	79.85	79.53
CLPB-c	1.92	1.98	114.31	85.65	60.14	58.19
CLPC-c	2.38	2.26	149.33	116.32	80.03	80.01
CLT-c	1.49	1.65	100.66	53.77	79.97	77.53

The load-displacement response figures above clearly show the differences between the monotonic and reversed cyclic results. The peak loads from the monotonic tests were greater than the corresponding values for the cyclic tests, which was also observed by Popovski [8]. The peak loads from cyclic tests were reduced by 17.4%, 26.5%, 38.9%, and 48.9%, respectively, compared to the corresponding monotonic values. Furthermore, the ultimate displacements obtained from monotonic tests were also greater than the corresponding values obtained from cyclic tests.

In terms of initial stiffness, there is a reduction (20%) when compared to the monotonic test results. The largest difference was for the CLPC wall, with a 35% loss in stiffness under cyclic loading, which can be explained by the different connection details between the monotonic and cyclic tests. It should be noted that to reach the failure limit state of the CLPC wall under cyclic loading, the total number of screws in connection systems of the wall was reduced in the cyclic tests. Furthermore, it can be observed that CLPA1, CLPA2, and CLT walls showed progressively decreasing stiffness values before reaching the ultimate load, whereas a slight increase can be observed for wall CLPB and CLPC walls at displacements between 40 mm and 60 mm. This phenomenon further proves that the use of LSL in the face layer instead of lumber has made a difference.

In addition, the load-displacement responses of all the specimens appeared to exhibit relatively symmetrical load-deflection behaviour. The CLPA2 and CLT walls appeared to have some asymmetric response characteristics. A closer examination of the load-deflection responses would suggest that these walls exhibit apparent asymmetric load-deflection loops, as they failed in one direction before being able to reach the same level of displacement in the opposite direction. It should be noted that several previous studies (Popovski [8]; Dires et al. [5]) also observed the unsymmetrical lateral force-deformation curves for cyclic tests, and low-cycle fatigue may be one of the reasons.

5 CONCLUSIONS

Connection tests and full-scale shearwall tests were carried out for an innovative mass timber panel, CLP, to better understand their performance under lateral loads. From the connection test results, the core layer of the CLP had a non-negligible effect on the lateral strength of the STS connections but the overall performance of connections is less affected by the orientation of the core layer made of the same type of material. However, the face layer of LSL in the CLP significantly increases the stiffness and strength as the density of the face layer increased. Overall, it can be concluded that CLP had higher strength and stiffness than CLT regardless of whether LSL was placed in the face or core layer. However, specimens with lumber as the face layer and LSL as the core, i.e., A1 and A2, exhibited higher strength, stiffness and comparable ductility compared with CLT. This made them more suitable for seismic applications than traditional CLT and CLP with LSL as face layers (B and C).

Furthermore, the shear wall tests revealed that the resistance of coupled shear walls is mainly determined

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by the panel-to-panel joints, as well as the hold-down and base shear connections reaching their respective yield strengths. The results indicated that the use of stiffer panel-to-panel connections may be difficult to realise the single wall behaviour in practice, but did make some difference to the entire wall behaviour. The panel-to-panel joint between adjacent panels is critical in determining the kinematic behaviour of the wall system. However, some unexpected brittle failures were observed in the connections during the shear wall tests. Therefore, it is necessary to implement a capacity-based design approach for CLP shear walls. The non-dissipative elements should be over-designed to remain elastic and prevent brittle failures.

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