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EXPERIMENTAL INVESTIGATIONS ON BALLOON FRAME CLT SHEARWALLS

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ABSTRACT: This paper presents experimental investigations on the use of Cross Laminated Timber (CLT) shear walls in a balloon frame configuration for use in low-rise construction. The larger objective of this research was to compare the behaviour of the balloon frame configuration to previously tested platform-type CLT shearwalls and determine the differences in ductility. The tested system consisted of two 7-ply 191 mm thick CLT panels with typical generic hold-downs, steel angle brackets, and plywood surface splines nailed to the CLT panels. A 2-storey system was tested at half scale with a panel aspect ratio of 3:1 with different steel and wood ledgers under monotonic and quasi-static reversed cyclic loading. The ledgers were subsequently tested under vertical quasi static monotonic loading to determine their remaining load-carrying capacity. The shearwall displacement was due to the rocking of the wall panels which themselves behaved as rigid bodies with negligible in-plane deformations. The results also showed that the ledger does not impede the desired rocking behaviour of the wall, nor does the rocking of the wall reduce the remaining load carrying capacity of the ledgers.

KEYWORDS: Mass-timber buildings, cross-laminated timber, balloon frame, ledger connections, seismic design

1 INTRODUCTION

1.1 BACKGROUND

Mass timber construction is becoming more common across North America and encapsulated tall wood construction is being incorporated in the 2020 version of the National Building Code of Canada (NBCC) up to 12 storeys [1] and the 2021 International Building Code (IBC) up to 18 storeys.

One form of mass timber commonly used in mass timber construction is Cross Laminated Timber (CLT). CLT consists of sawn lumber elements laid-up on-flat in alternating directions and glued together. The resulting panels have high in-plane strength and stiffness [2] making them suitable for Lateral Forces Resisting Systems (LRFS) like diaphragms or shearwalls [3].

CLT shearwalls will be incorporated into the upcoming NBCC 2020 alongside the encapsulated mass timber construction provisions. The Canadian Standard for Engineering Design in Wood, CSA O86-19 [4], provides guidance on the design of CLT shearwalls not exceeding to achieve the ductility and overstrength values outlined in the NBCC 2020.

CSA O86-19 [4] includes design guidance ad limitations for CLT shearwalls. The standard states that all the contents of the "Design of CLT shearwalls and diaphragms" applies to platform-type CLT construction, with the commentary elaborating that "balloon-type construction applications are beyond the scope of these guidelines". It states that CLT shearwalls shall not exceed 30m, or 20m in height in high seismic zones. Additionally, CSA O86-19 [4] provides guidance on achieving the code ductility and overstrength factors through rocking behaviour. Direction is provided for dissipative and non-dissipative connections to help ensure rocking behaviour governs. Strict height-to-width limitations are required for individual panels; longer walls are required to be made from multiple panels stitched together with dissipative splines.

Previous studies reported on the in-plane performance and design guidance to estimate the resistance and deflection for platform-type CLT buildings [7-9]. Connections between the CLT shearwalls and the foundation, and connections between panels-to-panel were consistently found to be the primary contributors to ductility. Additionally, a rocking mechanism has consistently shown better performance than sliding [7].

1.2 PLATFORM FRAMED CLT SHEAR WALLS

In a platform-type building, each floor acts a platform for the floor above where the walls are connected to floor diaphragm or foundation below by hold-downs (HDs) and brackets. A major drawback of platform framed systems is the accumulation of perpendicular-to-grain compression with each additional storey. In some cases, this may exceed the perp-to-grain bearing capacity

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of the panels for gravity conditions, or for the rocking occurring from the rocking of the panel.

There are several additional requirements for CLT platform frame walls, including the requirement for panels sizes are required to be within a 2:1 and 4:1 vertical-to-horizontal ratio; generally requiring multiple panels along the length of a single wall. The CLT panels are connected with vertical splines, typical either screwed or nailed plywood, or screwed half-lap joints [5-6]. These limitations significantly impact construction considerations as they increase both the number of panels that must be handled on site, and the number of fasteners required between panels.

1.3 BALLOON FRAME CLT SHEARWALLS

Balloon frame CLT shearwalls offer several advantages including: a) eliminating perp-to-grain bearing between floors, b) eliminating cumulative perp-to-grain shrinkage over the building height, c) fewer panels required to meet panel aspect ratio requirements, and d) fewer connections are required for cumulative HD forces and shear forces over the height of a building [10].

To date limited research is available on balloon-type construction. A mechanics-based analytical model to predict the lateral behaviour of CLT balloon-type shearwalls was proposed [11] that accounts for the contribution of shear and bending of CLT panel as well as sliding, rocking, and slip of vertical joints to estimate the resistance and deflection of balloon shearwalls.

1.4 OBJECTIVE

The NBCC [1] defines ductility (R_d) and overstrength (R_o) for the reduction of seismic design forces; the NBCC 2021 [1] and CSA O86-19 [4] includes ductility (R_d) of 2.0 and overstrength (R_o) of 1.5 for platform-type CLT shearwalls, where energy is dissipated through connection yielding due to rocking of the panels, while all other elements and connections are capacity protected. The standard provides several requirements intended to ensure that the rocking mechanism is maintained, any configuration outside the specified requirements is recommended to be designed with an R_dR_o = 1.3; although not specifically noted in the standard, this would apply to balloon-type construction as well.

The objective of the research presented herein investigates balloon frame CLT shearwalls with typical HDs and brackets base connectors and panel-to-panel spline connection to establish if the intended behaviour is achieved. Various ledger assemblies connected at mid-height are studied to determine the influence of these elements to the desired behaviour. Additionally, this research also investigated the remaining gravity load carrying capacity of the ledgers after a seismic event.

2 BALLOON FRAME CLT SHEARWALL TESTS

2.1 SPECIMEN DESCRIPTION

A total of twelve CLT balloon shearwalls and six ledgers were tested in the UNBC Wood Innovation and Research Laboratory in Prince George, BC.

The tests consisted of two CLT panels of 1219 mm wide and 3658 mm tall with an aspect ratio of 3:1representing a half scale two-storey shearwall, see Figure 1. Each wall connected to foundation/floor steel beams using 2-HDs and 4-brackets in the front face of the panels, whereas, the coupled panels are connected vertically using nailed plywood spline joint on the back.

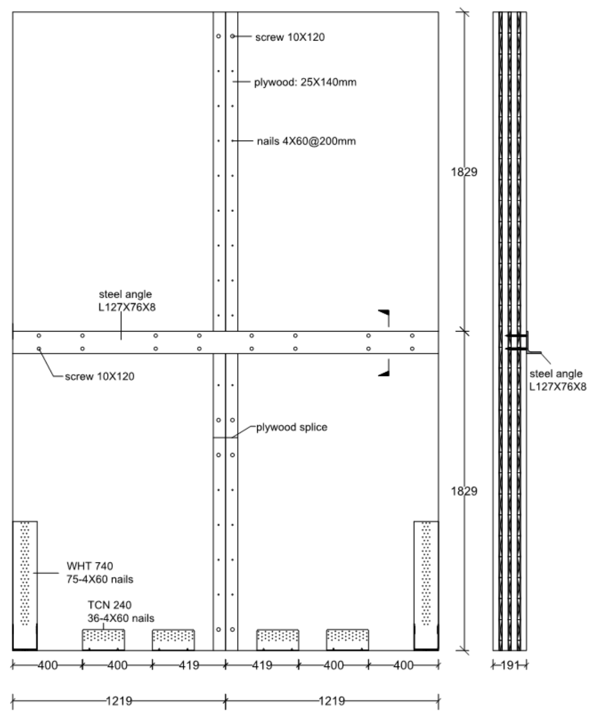

__Figure 1:__ Balloon-frame shearwall with Ledger Type A

2.2 MATERIALS

The CLT panels used we 1.219m x 3.658m 191V 7-ply CLT. The panel layup composed of 35mm primary lams, and 17mm cross alms [35+17+35+17+35+17+35].

The panel-to-panel vertical connections were surface mounted 25×140 mm plywood spline fastened to the CLT with 4Ø×60 mm anker nails at 200 mm space with an additional one row of ASSYS Kombi 10Ø×120 mm PT screws on top and bottom of the plywood piece The plywood was spliced at one-third height of the wall (Figure 1).

The shearwalls were anchored to steel beam foundation with two HDs on the outer edges and four angle brackets- two on each panel. The HDs were generic steel plate nailed Rothoblaas WHT740 hold-downs with 75-4Ø×60 mm anker nails and the base shear connectors were generic nailed Rothoblaas TCN240 angle brackets with 36-4Ø×60 mm anker nails (Figure 2).

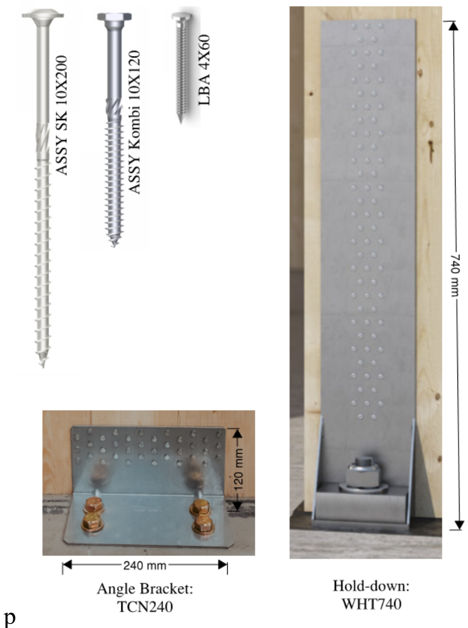

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Figure 2: *Connectors and fasteners [not to scale]*

Table 1 shows the specification of various materials and Figure 2 shows the photos of various connectors and fasteners used for the testing program.

Table 1: *Materials for Balloon Walls*

Material	Description
CLT	191V 7ply SPF CLT
Spline	CSP 25mm plywood
Hold-down	Rothoblaas WHT740 w/ 75 nails
Shear Bracket	Rothoblaas TCN240 w/ 36 nails
Spline Nails	Rothoblaas LBA 460 – 4Øx60 nails
Spline Screws	ASSY Kombi 10Øx120 screws

2.3 LEDGER TYPES

Three types of ledger connections, as schematically shown in Figure 3 and summarized in Table 2, were attached a mid-height of the CLT shearwall.

Table 2: *Ledger Types Specimens*

Ledger Type	Ledger Material	Section	Fasteners
Type A	Steel	L127x76x8	16 - 10Ø×120 screws in 2-rows evenly spaced along ledger length
Type B	Steel	L178x102x8	8 - 10Ø×120 screws concentrated at the centre of each panel (16 total)
Type C	Steel	L127x76x8	1 – 38Ø thru-bolt w/ nut and washer at centre of each panel
Type D	Wood	GL 75x190	16 - 10Ø×200 screws in 2-rows evenly spaced along ledger length

All the steel ledgers used ASSY Kombi screws (hex head), but the wood ledger used ASSY SK screws (washer head).

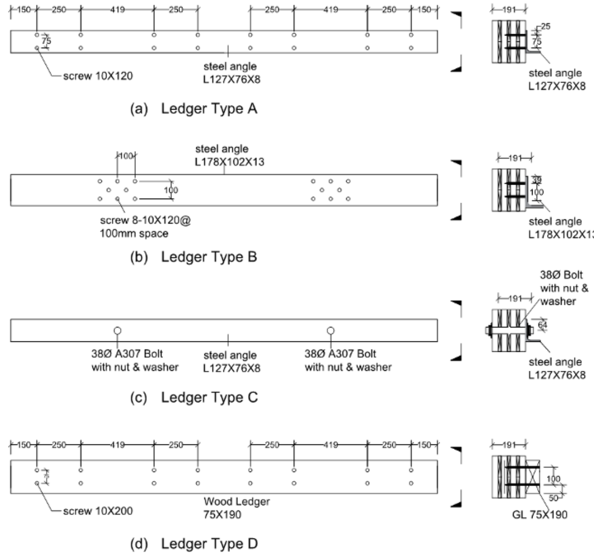


Figure 3: *Different Ledger Assemblies for Balloon Walls*

2.4 SHEARWALL TEST SETUP AND METHODS

CLT balloon shearwalls were tested to investigate how the ledger types affect their rocking behaviour. Each wall configuration as described in Table 2 was tested three times: once under quasi-static monotonic loading to determine the displacement target for the subsequent two quasi-static reversed cyclic tests as outlined in the CUREE loading protocol.

Lateral loads were applied by two 250 kN actuators at the top of the wall panel through a steel side plate connected to steel I-beam that sits on two wooden blocks at the centre of each panel and at mid height directly to the ledger (Figure 4).

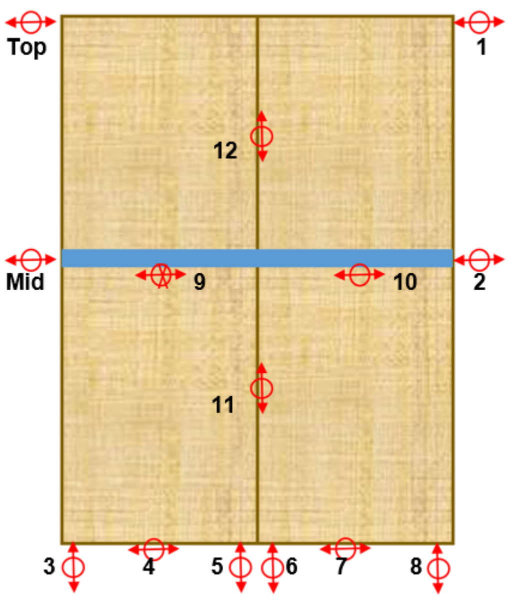
The load was applied in such that the top actuator was the 'master' and the mid-height actuator the 'slave' trailing the top actuator's loads. For all specimens, tests were stopped when the loads drop to 80% below of maximum loads.

A 20 kN/m typical vertical gravity load was applied using three cantilever steel beams, bolted to three rectangular hollow structural sections (HSS) which were pin connected to the I-beam to allow for lateral movement of the shearwall. This gravity load system simultaneously prevented out-of-plane horizontal movements.

The quasi-static monotonic pushover tests were conducted at a rate of loading of 10mm/min for the actuator applying the load to the top of the shear wall and the second actuator applying the load to the ledger always applying the same force. The ultimate displacement from the monotonic tests was used to establish he target displacement for the subsequent reversed cyclic tests following the abbreviated displacement controlled CUREE loading procedure.

Figure 4: *Balloon shearwall test setup*

The horizontal, vertical, and relative panel displacements were recorded with at twelve locations i.e., at top and mid height of the wall, base, ledger and spline locations using LVDTs and string pots as shown in Figure 5.



Top, Mid:	Integrated actuator LDVTs
1,2:	String pots wall-mounted for horizontal displacements
3,5,6,8:	LDVTs for wall uplifts
4,7:	LDVTs for horizontal displacement at wall bottom
9,10:	LDVTs for displacement between panels and ledger
11,12:	LDVTs for displacement between panels

Figure 5: *Instrumentation for shearwall testing*

2.5 LEDGER TESTS

For shearwalls with ledger types A, B and D, the ledgers were subsequently tested under vertical quasi-static monotonic loading, c.f. Figure 6, to determine the ledgers' remaining load-carrying capacity. Each ledger type was tested twice as discussed in section 2.4. A summary of the tests completed is provided in Table 3:

Table 3: *Test Summary*

Test ID	Ledger Type	Loading Type	Ledger Gravity Test Completed
A-M1		Monotonic	N
A-C1	Type A	Cyclic	N
A-C2		Cyclic	Y
B-M1		Monotonic	N
B-C1	Type B	Cyclic	N
B-C2		Cyclic	Y
C-M1		Monotonic	N
C-C1	Type C	Cyclic	N
C-C2		Cyclic	Y
D-M1		Monotonic	N
D-C1	Type D	Cyclic	N
D-C2		Cyclic	Y

For each ledger type, two monotonic tests were completed on the ledger (Figure 6). A baseline test of each ledger type was completed to establish the strength of the ledger prior to any lateral loading on the system. Then an additional test was completed for each ledger type after the final cyclic shearwall tests. The intent of these tests is to establish if that rocking behaviour of the shearwalls resulted in any strength degradation of the capacity protected ledger that would provide gravity support for a floor.

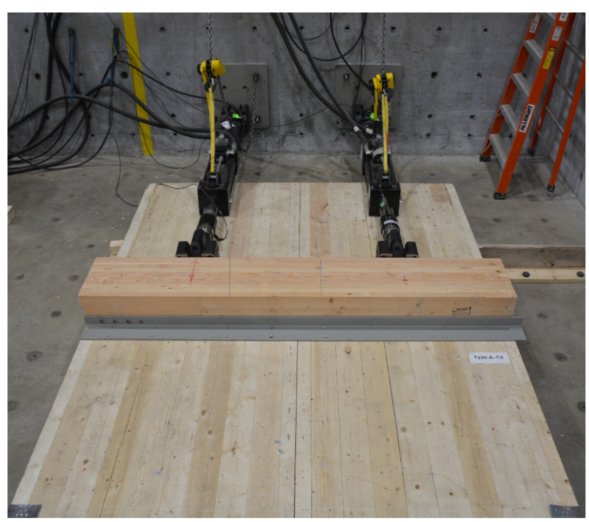


Figure 6: *Setup for Ledger Tests*

3 RESULTS

3.1 LATERAL TEST RESULTS

The load-deflection curves from the monotonic tests are shown in Figure 7 and the hysteresis behaviour at the top of the walls from first cyclic test from each group in Figure 8. Each monotonic shearwalls test and the envelope of the cyclic tests follow an idealized bi-linear behaviour with a higher initial stiffness a secondary stiffness sustained up to the peak strength of the system. The deformation at the change from initial to secondary stiffness is less than 10 mm and the deformation at ultimate strength was between 90 and 120 mm. After the ultimate deformation was reached a sudden decrease in load carrying capacity in the system was observed with the strength flattening out to 80% of ultimate (i.e. failure).

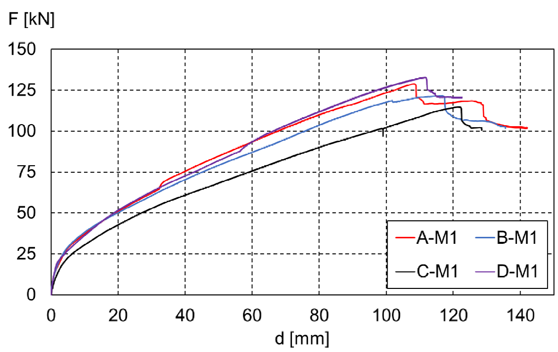


Figure 7: *Load-deflection curves for monotonic tests*

The shearwalls test results were assessed in terms strength, stiffness, deformation, and energy dissipation capacity, c.f. Table 4. Type A, B and D had similar capacity and stiffness. Type C shearwalls had 11%, 6%, 14% lower capacity compared to shearwalls type A, B, and D, respectively.

Table 4: *Balloon Shearwall Ultimate Strength and Deformation*

Test ID	F_{ult} [+]	F_{ult} [-]	Avg F_{ult} Change	Δ_{ult} [+]	Δ_{ult} [-]	Avg Δ_{ult} Change
	[kN]		[%]	[mm]		[%]
A-M1	128.8			108.3		
A-C1	121.2	-101.1	-12%	111.7	-91.4	-11%
A-C2	117.4	-111.9		88.2	-94.6	
B-M1	121.7			116.7		
B-C1	99.9	-99.4	-17%	111.1	-114.7	-13%
B-C2	105.6	-96.9		89.5	-90	
C-M1	114.7			121.8		
C-C1	85.7	-94.8	-20%	89.1	-120.9	-14%
C-C2	96.5	-89.8		116.8	-92.7	
D-M1	133			111.6		
D-C1	115.3	-102.9	-21%	115.9	-92.6	-13%
D-C2	104.4	-96.8		88.4	-93.7	

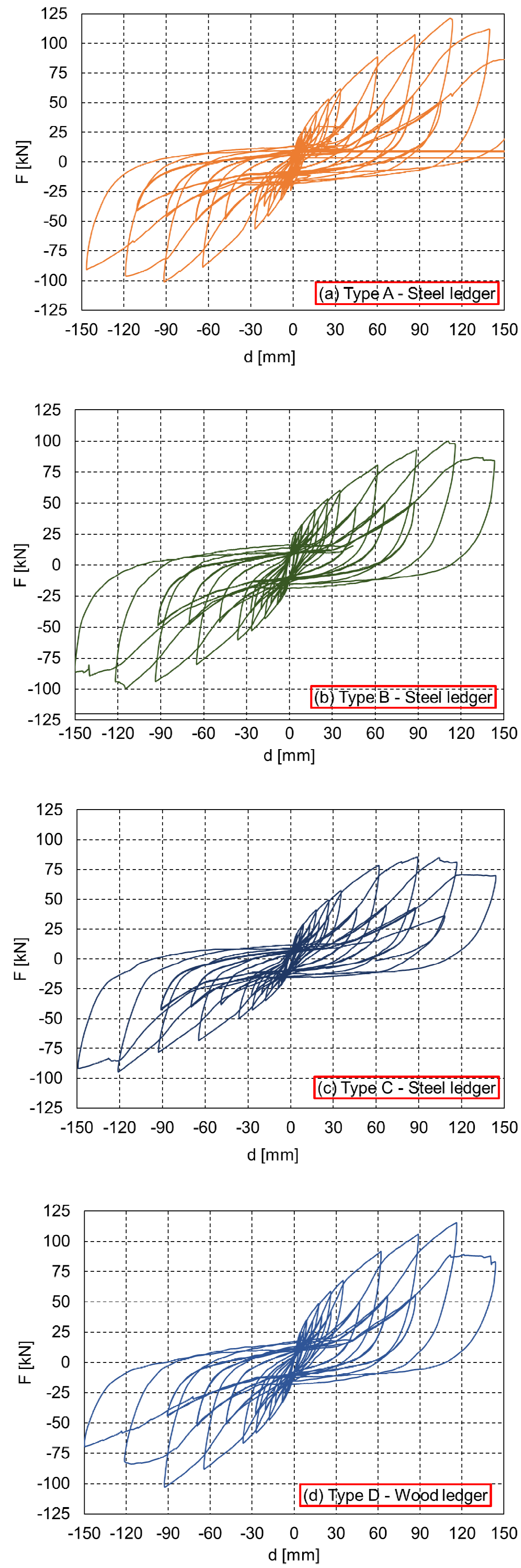


Figure 8: *Hysteresis Curves- displacement measured at top of the wall: (a) steel ledger Type A, (b) steel ledger Type B, (c) steel ledger Type C, (d) wood ledger Type D*

Under reversed cyclic loading, the load-carrying capacities and displacements were reduced by up to 21% and 14%, respectively, as a function of ledger type. The average strength reductions of the shearwalls for Types A, B, C and D were 12%, 17%, 20% and 21%, respectively, compared to monotonic tests of the same series, whereas, the average deformation reductions were 11%, 13%, 14% and 13%, respectively compared to monotonic tests. From the monotonic tests, the interstorey drifts at failure at the top of the shearwalls were calculated as 3.2%, 3.4%, 3.5%, and 3.2%, respectively for Type A, B, C, and D walls which were reduced to 2.8%, 2.8%, 3%, and 2.8%, respectively in the cyclic tests.

The CLT panels behaved as rigid bodies, therefore, the in-plane deformations of the panels were negligible. The horizontal displacement of the shearwalls was due to the rocking of the wall panels. No Apparent damage or deformation was observed in the CLT panels away from the spline fastener connections. The concentration of the forces due to rocking of the walls were observed at the base connectors and vertical spline joints (Figure 9). The failure in the vertical spline connections trigger the subsequent failure of the walls.

Figure 9: *Failure at hold-down (left) and spline joint (right)*

3.2 LEDGER TEST RESULTS

Each ledger types A, B and D were tested twice: once with newly connected ledgers under simulated horizontal loads and once after completing the reversed cyclic tests to investigate the ledgers' remaining load-carrying capacity. Ledger type C was not tested because it is designed to isolate the ledger and ledger connection from deformation resulting from rocking; no fastener deformation was intended or observed during the lateral testing. The results i.e., peak loads (F_{max}), displacement at peak loads (d_{Fmax}) and elastic stiffness (K_e), calculated as the ratio of $0.4F_{max}/\Delta_e$, where Δ_e is the displacement at $0.4F_{max}$, are shown in Table 5.

Table 5: *Ledgers Gravity Test Results*

Ledger Type	Test	Fmax		dFmas		Stffiness (K)	
		[kN]	[%]	[mm]	[%]	[kN/mm]	[%]
Type A	Initial	253.2		30.9		30.3	
	Post-cyclic	246.5	-6%	22.1	-29%	15.4	-49%
Type B	Initial	270.8		48.6		11.6	
	Post-cyclic	252.1	-7%	38.2	-21%	10.2	-12%
Type D	Initial	253.5		64.6		10.3	
	Post-cyclic	250.5	-1%	52.4	-19%	8.0	-22%

The load-deflection curves from the ledger tests are plotted in Figure 10. Test results showed that the applied reversed cyclic loading only led to small reductions of up to 7% in the ledger capacity. Type A ledger was found as the stiffest ledger, with 62% and 66% higher stiffness compared to Type B and D, respectively. When the ledgers were tested subsequently after cyclic tests, Type A ledger experience the maximum drop in elastic stiffness of 49%, however, the peak load dropped only 6%. The failure modes of all ledgers were similar due to shearing of screws as seen in Figure 11.

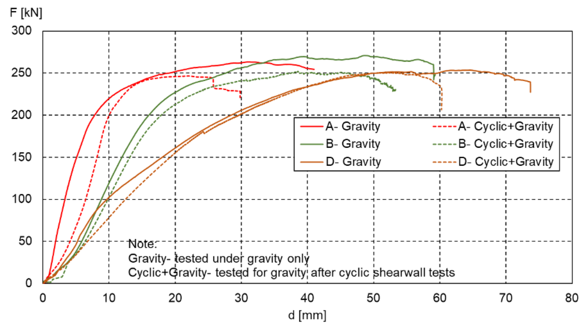


Figure 10: *Load-deflection curves for ledger's gravity tests*

Figure 11: *Screw failures in Type A shearwalls (left) Type D shearwalls (right)*

4 CONCLUSIONS

This study describes the lateral performance of balloon frame CLT shearwalls. Twelve two-storey balloon CLT shearwalls with four types of ledger assembly were tested under monotonic and reversed cyclic testing. In addition, six ledgers tests were conducted to investigate the ledgers' remaining load-carrying capacity after an earthquake event. The influence of steel and wood ledgers and its connectors on the rocking behaviour of the walls was evaluated. The results show that none of the ledgers impede the desired rocking behaviour of the wall, nor does the rocking of the wall significantly impact the ledger's gravity load carrying capacity.

Results obtained from this testing program will be utilized for designing a balloon frame school building in Vancouver, BC. The ability to use balloon frame CLT shearwalls will allow for taller and more efficient CLT shearwall construction in Canada and beyond.

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