Investigating CLT lateral load resisting systems for taller timber buildings

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ABSTRACT

Recent taller timber buildings worldwide have often utilized hybrid systems of mixed materials and have been built in regions of low seismicity. Designers often adopt the hybrid material solution of utilizing mass timber for the gravity system and reinforced concrete or steel for the lateral system as a viable building option. Most current timber building codes are behind state-of-the-art timber design possibilities. Besides, the inherent timber material properties such as relatively low stiffness and possible brittle behaviour make it more challenging for timber lateral systems to resist high lateral loads, especially for taller buildings in high seismic regions. This paper presents an overview of recent research of a cross-laminated timber (CLT) core-wall system and its connections as one new mass timber lateral system. First, high performance connections such as dowelled hold-downs with internal steel plates, self-tapping screws with mixed angles, and CLT to CLT castellated joints were tested and compared to commercial connection systems. Selected connections were then implemented in the experimental testing of a 2/3 scale 8.6m tall CLT core-wall structure, composed of multiple CLT wall panels connected with vertical and horizontal joints. Further, post-tensioning was implemented to replace traditional hold-down anchors to achieve increased system strength, stiffness and recentering capability. New Zealand CLT material property and component testing was conducted to determine connection overstrength values for design. Through this work, designers will have more options for timber lateral load resisting systems in timber buildings in high seismic regions.

1 INTRODUCTION

Recent technological advancements and socio-economic factors have generated a renewed interest in timber buildings globally (Dangel, 2016). Engineered wood products such as Cross-laminated Timber (CLT) are being used in building types of increasing size and complexity (Green, 2017). The early adoption of CLT shear walls as the primary lateral load resisting system (LLRS) implemented commercial connectors adopted from light timber frame (LTF) construction. It is well reported that these LTF connectors limit the potential of CLT LLRS and do not capitalize on the relatively higher CLT in-plane stiffness compared to LTF LLRS (Flatscher et al., 2015). Thus, high performance hold-downs with energy dissipating and recentering capability have been investigated (Hashemi et al., 2020). The inherent timber material properties such as relatively low stiffness (approximately ¼ reinforced concrete) and possible brittle behaviour make it more challenging for ductile lateral design of taller timber building, especially in higher seismic regions. Within this lies the displacement paradox, because while a designer needs to control lateral displacements to meet the code prescribed drift criteria, one also needs to provide displacement capacity to ensure the assumed system ductility factor is achieved (Smith et al., 2015). To overcome the inherent flexibility of timber structures, Buchanan (2016) stated transforming conventional planar shear walls to core-walls using efficient connections including post-tensioning could meet the increased strength and stiffness demands for taller timber buildings. Post-tensioned timber systems, also called Pres-Lam technology (from prestressed laminated timber), have been developed and researched since 2005 at the University of Canterbury (Palermo et al., 2005). Since then, numerous buildings and research have occurred globally and a state-of-the-art in Pres-Lam concept, testing, and implementation was presented by Granello et al. (2020). Recently, there has been significant interest in Pres-Lam systems in North America under a multiyear research project which will culminate with a shake table test on a 10-storey full-scale post-tensioned CLT building (Pei et al., 2017).

1.1 Research plan

The objective of the research was to evaluate the structural performance of a CLT core-wall (CW) LLRS using different connection details. The recently completed Catalyst building (2018) is one rare example of a CLT core-wall LLRS but more research is required to inform further application of this system type. In recognizing the importance of connection performance on CLT CW system performance, some key connections were explored before implementation in the CLT CW specimen. These connections included dowelled hold-down/tie-down connections for horizontal moment-resisting joints, self-tapping screw (STS) connections for orthogonal joints, and castellated connections for shear-resisting horizontal joints. Figure 1 shows a concept design building plan with a potential CW LLRS, to a CW isometric with key joints, and to connection details investigated for each joint. Finally, a three-phase wall testing programme included a post-tensioned (PT) single-wall (SW), PT coupled double-wall (DW), PT core-wall (CW), and then a conventional core-wall (CCW) system with hold-down connecters and without PT.

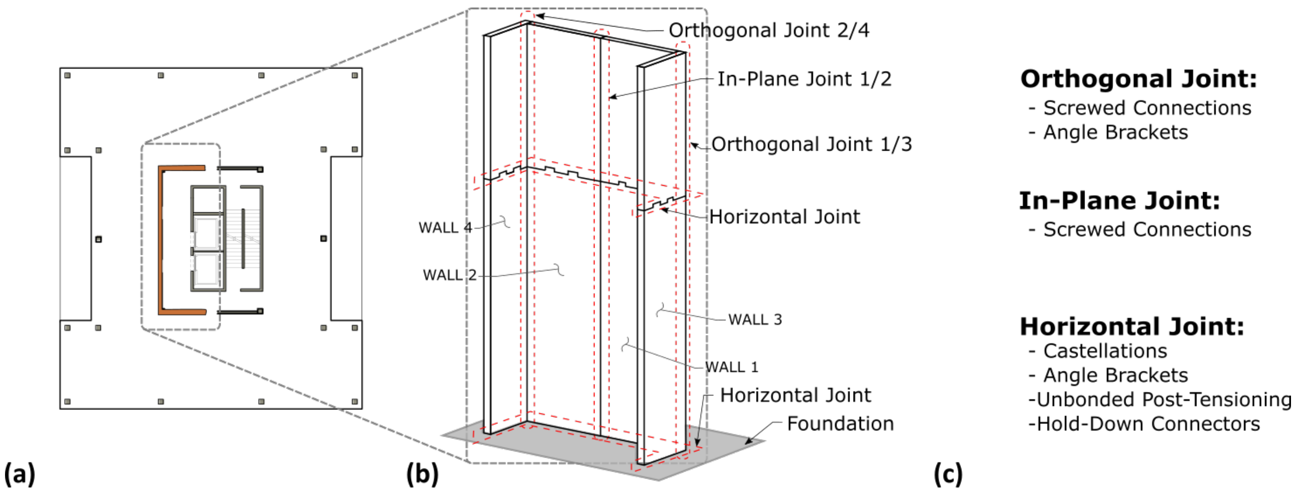

Figure 1: Core-wall system research plan: (a) plan view building adopted from (Green, 2017)*, (b) isometric of four wall core-wall system with key joints, (c) joint options investigated*

2 CONNECTIONS FOR TALLER TIMBER BUILDINGS

2.1 Dowelled hold-down with internal steel plates for horizontal joints

The structural performance of 47 dowelled CLT hold-down connections was investigated and results are summarized in Brown et al. (2021a). The study was a continuation of previous work by Ottenhaus et al. (2018) which found that, although minimum spacing requirements specified by current timber standards were followed, early and cross-over brittle timber failure modes could still occur. As such, the study focused on the influence of increased fastener row spacing (a_2) and end distance (a_3) on strength, stiffness, ductility and overstrength of the connections. Figure 2a-Figure 2d show the front and side view of the four dowel hold-down connection and then a commercial hold-down bracket (adopted from LTF) which was tested by Dong et al. (2020). While nailed hold-down brackets are commonly used in CLT buildings, past research has shown they have limited capacity usually less than 100kN or less if ductile behaviour characterized by yielding of the nails is required (Dong et al., 2020). Figure 2e compares the force-displacement curves for the 4-ϕ20mm Grade 300E dowelled hold-down connections tested by Ottenhaus et al. (2018) and Brown et al. (2021a) in comparison to a Rothoblaas WHT440 bracket (2019) using full and partial nail patterns with ϕ4x60mm LBA nails (2018) tested by Dong et al. (2020). The displacement capacity increased more than 3 times when comparing dowelled hold-down tests by Brown et al. (2021a) to nailed hold-down tests by Dong et al. (2020).

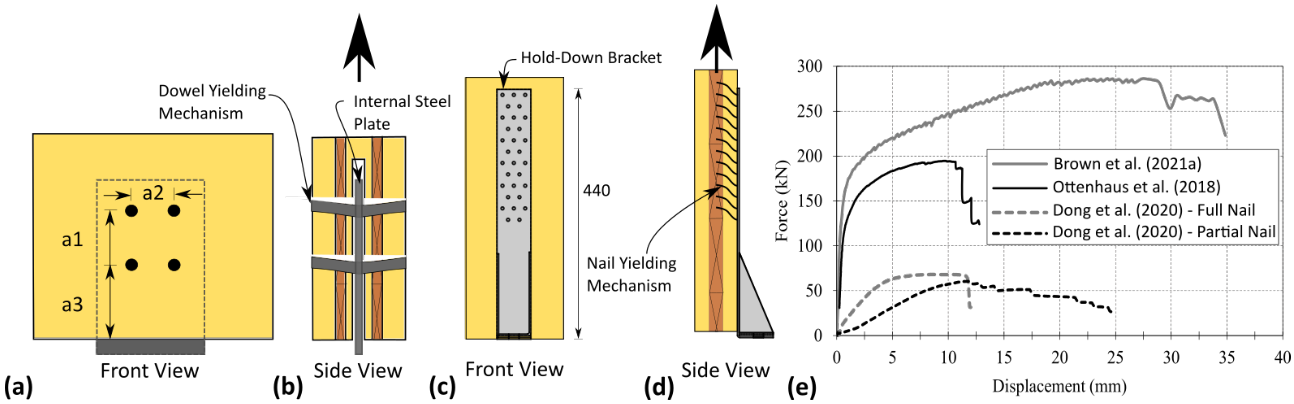

Figure 2: Hold-down connectors: (a) and (b) dowelled hold-down with internal steel plate, (c) and (d) commercial hold-down bracket, (e) comparison of hold-down load-slip curves

Table 1 lists the different fastener spacing in terms of dowel diameters, displacement capacity, and then experimental connection overstrength, γ_{Rd}. γ_{Rd} was reported in each study and the maximum value was 1.7. It was shown that the connection strength could be predicted well using the Eurocode 5 (2014) prediction equation and embedment strength equation from the CLT Handbook (2011) with steel yield strength of 300 MPa and timber characteristic density of 440kg/m^3 as provided in the upcoming DZ NZS AS 1720.1/V6.0. The results show that dowelled hold-down connectors, especially with increased fastener row and end spacing, can provide a suitable option as a ductile link for a CLT LLRS.

Table 1: Performance comparison of selected hold-down connectors

Series	Ottenhaus et al. (2018)	Brown et al. (2021)		Dong et al. (2020)	
a1	5d	5d	5d		
a2	3d	4d	6d	Partial Nail Pattern (Nail quantity - 15)	Full Nail Pattern (Nail quantity - 30)
a3	5d	7d	9d		
Δ_u (mm)	5.7	29.9	40.2	21.1	12.1
γ_{Rd}	1.52	1.7	1.66	1.62	-[1]

[1]Due to brittle tensile failure of bracket, overstrength is not provided as it was not recommended for use as ductile element in CLT LLRS

2.2 Self-tapping screws for vertical orthogonal joints

The structural performance of 59 CLT orthogonal joint specimens with self-tapping screws (STS) installed with mixed angled STS were tested and presented in Brown et al. (2021c). Past research has reported that STS installed inclined can provide high strength and stiffness but limited displacement capacity while STS installed at 90° to the CLT surface provide low strength and stiffness but high displacement capacity (Loss et al., 2018). To achieve joint design with high strength, stiffness and ductility, the study focussed on different ratios of STS installed inclined and STS installed at 90° to the CLT surface to determine an optimum ratio.

Figure 3 shows different STS installations and combinations and Figure 4 compares their force-displacement curves as a group of four STS. When comparing Long Inclined and Short Inclined tests, it was found that the thread embedment length should be limited to ensure gradual screw withdrawal failure instead of brittle screw tensile failure. A STS withdrawal study for one type of STS in NZ CLT found that the embedment length should be limited to 12d (12 times the screw diameter) to avoid screw tensile failure (Brown et al., 2020a). When comparing different STS combination ratios, it was found that a ratio of one 90° STS for every two inclined STS ensured significant increase in connection displacement capacity. Further, 90° STS contributed to both the strength and stiffness of the mixed angle STS joints. While 90° fully threaded STS experienced significant rope effect, Eurocode 5 (2014) limits the rope effect contribution to 100% of the lateral capacity, which is shown in blue in Figure 4.

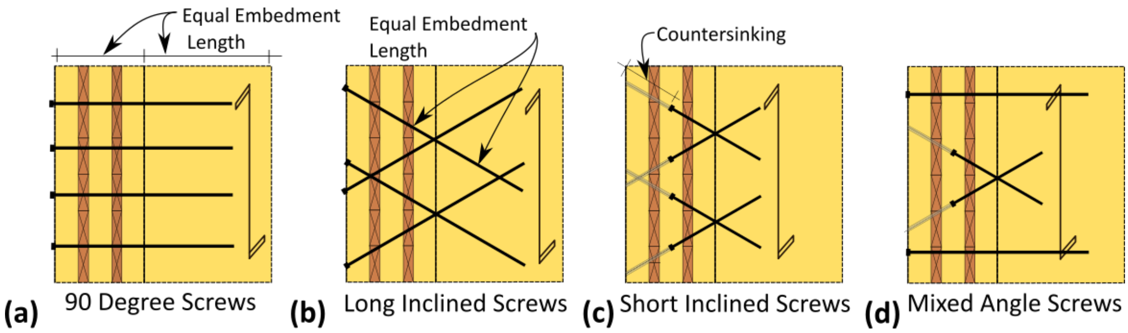

Figure 3: Elevation view of screwed orthogonal joint connection options

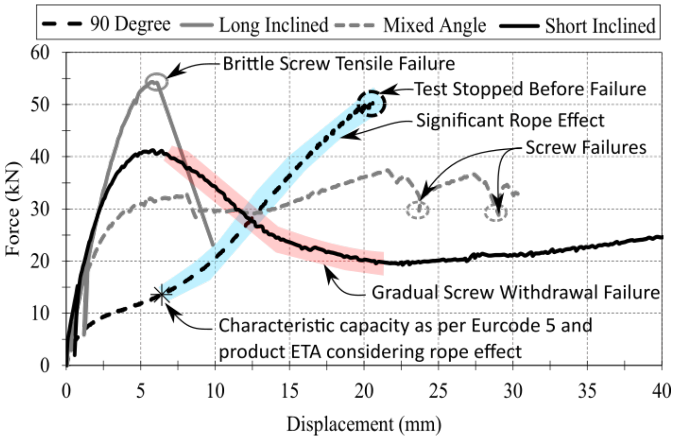

Figure 4: Selection of force-displacement curves

STS joint design is currently not covered by many design standards including NZS3603 (1993). Therefore, state-of-art STS connection strength models, originally developed by Bejtka and Blaß (2002) and extended by Jockwer et al. (2014) were used with design guidance within the SPAX ETA (2017). A detailed presentation of the analytical model can be found in Brown et al. (2021c). In brief, the strength model accounts not only for the embedment strength of the timber member and the bending capacity of the STS, but also the withdrawal capacity of the STS and the friction between the members as:

$$F_{STS} = R_a\,(\psi\ \sin\varphi + \cos\varphi) + R_v(\sin\varphi - \psi\ \cos\varphi) \tag{1}$$

where R_a is the withdrawal strength, R_v is the lateral strength, φ is the angle between the screw axis and timber grain, and ψ is the coefficient of friction, taken as 0.25 for wood-wood surfaces as per Eurocode 5 (2014). Within Brown et al. (2021), STS installed inclined and STS installed at 90° to the timber grain were combined by superposition to determine the connection strength and it was found that the average experimental overstrength, γ_{Rd}, was 1.7. Mixed angle STS connections could provide suitable ductile links for a CLT LLRS with displacement capacity greater than 20mm when gradual screw withdrawal was the primary failure mode.

2.3 CLT to CLT castellations for in-plane vertical and horizontal joints

Carpentry joints were historically employed based on tradition, skilled craftsmen, and proven good performance (Siem, 2017) but highly skilled and costly labour rendered them uncompetitive to modernized timber buildings with joints using mechanical fasteners. The increased use of computer numerically controlled (CNC) fabrication technologies however has led to some carpentry joints being revisited (Tannert, 2016). Schmidt and Blaß (2016) explored ten different interlocking CNC carpentry joints for in-plane CLT shear connections loaded parallel to the outer CLT layer and reported that while the joints were not suitable as energy dissipating ductile links, increased load-carrying capacities as a percentage of CLT panel shear capcaity were reported when compared to joints with mechanical fasteners. In the research introduced herein, the shear strength and stiffness of mortise and tenon castellated CLT joints were compared to connections with commerical angle brackets. Loading parallel and perpendicular to the outer CLT layer were investigated under varying tenon dimensions L and H to represent different loading scenarios as shown in Figure 5.

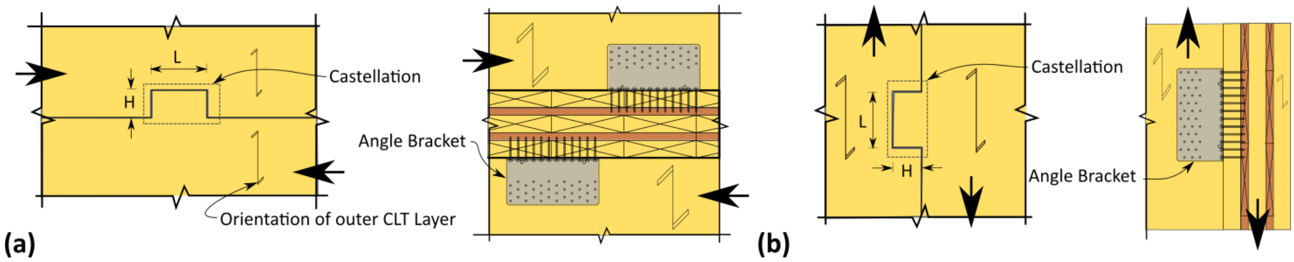

Figure 5: CLT to CLT Castellation and angle bracket shear connectors: (a) load perpendicular to the outer CLT layer, (b) load parallel to the outer CLT layer

Figure 6 shows the typical failure modes observed in the testing. Castellated joints were able to achieve high strength and stiffness and did not experience a sudden load drop once longitudinal or rolling shear failure occurred mainly due to the cross-layer reinforcement. The nailed steel angle brackets experienced significant in-plane and out-of-plane deformation (Figure 6c) which limited their stiffness. Their peak strength was also limited due to timber failure instead of a more favourable nail yielding mechanism shown in Figure 6d (Tomasi & Smith, 2015). As with the commercial hold-down brackets tested by Dong et al. (2020), partial nail patterns may be required if a more ductile mechanism characterized by fastener yielding is desired. Preliminary results indicate castellated joint strength and stiffness values were 2.5 and more than 7 times greater when compared to the joints using the angle brackets, neglecting construction tolerances. Castellated carpentry joints could provide capacity protected high performance shear connections for a CLT LLRS.

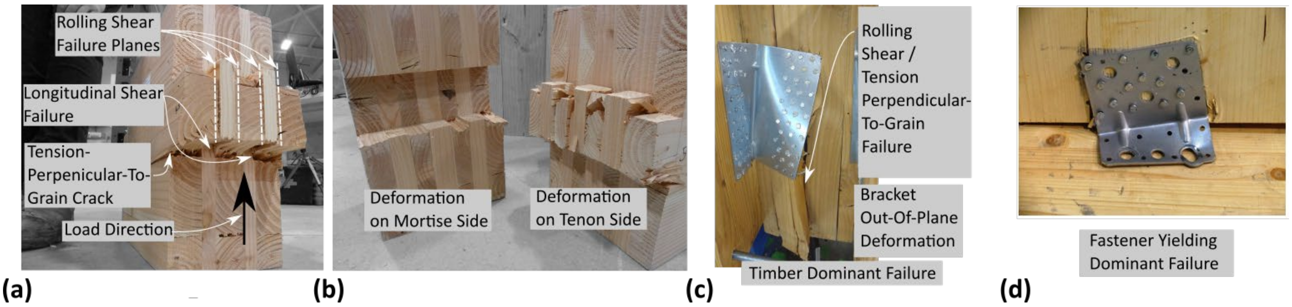

Figure 6: Failure photos: (a) castellation tenon failure modes, (b) castellation deformations, (c) angle bracket timber dominant failure, (d) angle bracket fastener yielding failure (Tomasi & Smith, 2015)

3 CLT SHEAR WALL TESTING

3.1 Overview of wall testing programme

A total of 17 wall tests were performed in the shear wall testing programme: 4 post-tensioned (PT) single wall (SW) tests, 5 PT coupled double wall (DW) tests, 7 PT core-wall (CW) tests and one conventional core-wall (CCW) test. In order to understand the increased strength and stiffness contribution due to each component (wall and joint), the proposed core-wall system was decoupled and PT SW and DW tests were investigated. The full post-tensioned wall test programme is introduced in Brown et al. (2020b) and shown in Figure 7. The five selected tests presented herein are described in Table 2. The tests considered variations in a) testing phase, b) loading protocol, c) initial PT force, d) STS connection detail, and e) use of U-shaped flexural plates (UFPs). In the CCW test, PT bars were not used and STS hold-downs provided anchorage to the foundation. A uni-directional (Uni) and bi-directional (Bi) cloverleaf loading protocol were followed. For Test CW-7, one cycle group of uni-directional loading was followed by cloverleaf bi-directional loading cycle.

The wall specimens were four storey with a 2/3 scale factor and 8.6m tall. The CLT wall panels were 5-ply and 175mm thick (45/20/45/20/45) with SG8 Douglas-Fir laminations as specified in NZS3603 (1993). In the CW testing, CLT floors at each of the four levels provided required lateral restraint. At the CLT wall base, EA 125x125x12 shear keys provided in-plane and out-of-plane restraint. For some tests, UFP dissipaters were installed at the corner and base of each CLT wall to provide additional energy dissipation. A total of 12 ø26.5mm Macalloy bars (2018) anchored the four CLT walls to the foundation. The Macalloy bars were located within 100 x 45mm voids in the middle layer of the CLT, and secured to the top of each CLT wall with a 50mm thick steel anchorage plate. A more detailed explanation on the experimental test set-up, programme and methodology can be found in Brown et al. (2021b).

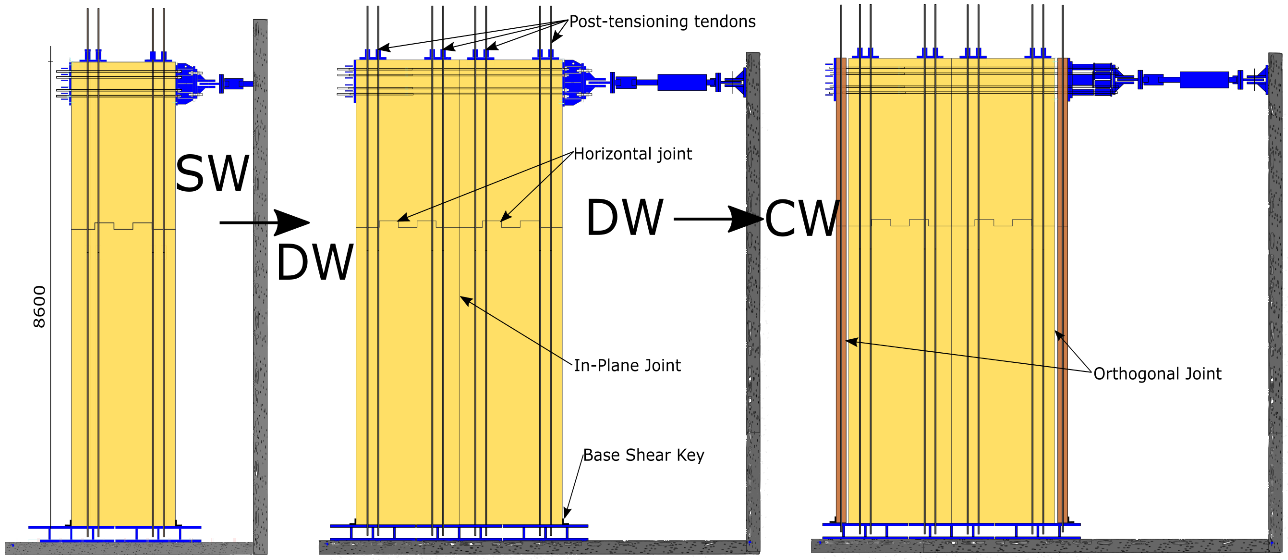

Figure 7: Wall Testing Plan Programme adopted from Brown et al. (2020b)

Figure 8 shows the core-wall strength hierarchy for the post-tensioned and conventional core-wall specimens. The prior connection testing knowledge ensured stable behaviour under cyclic loading occurred. For the PT wall specimens, the un-bonded PT bars provided strong and stiff elastic base connections with recentering capability while the STS connections provided the main source of energy dissipation as ductile links between the CLT wall panels at the orthogonal and in-plane joint. UFP devices also provided stable energy dissipation but they were primarily installed to investigate different connection details' performance under bi-directional loading. For the conventional core-wall specimen (CCW), ductile links with mixed angle STS hold-downs were detailed. In addition, the in-plane joint was detailed with mixed angle STS to ensure displacement capacity and energy dissipating capability. Figure 9 shows schematics of these ductile links. Commercial angle brackets (ETA-11/0496, 2018) were installed with a full nail pattern of ϕ4x60mm LBA nails (2018) at the orthogonal joint and designed to be capacity protected. At the horizontal joint, a capacity protected dowelled tie-down connection was designed as shown in Figure 8b to resist overturning moments. For both the PT CW and CCW specimens, capacity protected castellated joints at the horizontal joints provided shear transfer.

Table 2: Overview of selected wall test programme

Name	**Phase Test**	**SW 2**	**DW 3**	**CW 6**	**CW 7**	**CW 8**
Loading	Type	Uni	Uni	Uni	Bi	Uni
Initial PT/bar	kN	25	25	75	25	-
In-Plane Joint	Type	-	8x80 PTH (90°)	8x80 PTH (90°) 8x160 FTH (Inc.)	8x80 PTH (90°)	8x80 PTH (90°) 8x160 FTH (Inc.)
	Qty.	-	220 (90°)	206 (90°) 244 (Inc.)	220 (90°)	110 (90°) 110 (Inc.)
Orthogonal Joint	Type	-	-	8x350 PTH (90°) 8x200 FTH (Inc.)	90°	Angle Bracket[1]
	Qty.	-	-	Mix[2]	83	22
UFPs		No	No	Yes	Yes	No

Note: PTH = partially threaded STS; FTH = fully threaded STS; ply = plywood as per NZS 3603; Inc. = inclined STS installed in cross-wise pattern as per Figure 3c; Mix = mixed STS installation as per Figure 3d.

[1]Rothoblaas Titan TTN240 brackets (ETA-11/0496, 2018) with full LBA ϕ4x60 (ETA-13/0523, 2018) nailing pattern.

[2]78-8x350 PT (90°) per joint and 78 and 70-8x200 FT (Inc.) per wall 1/3 and wall 2/4 joint respectively.

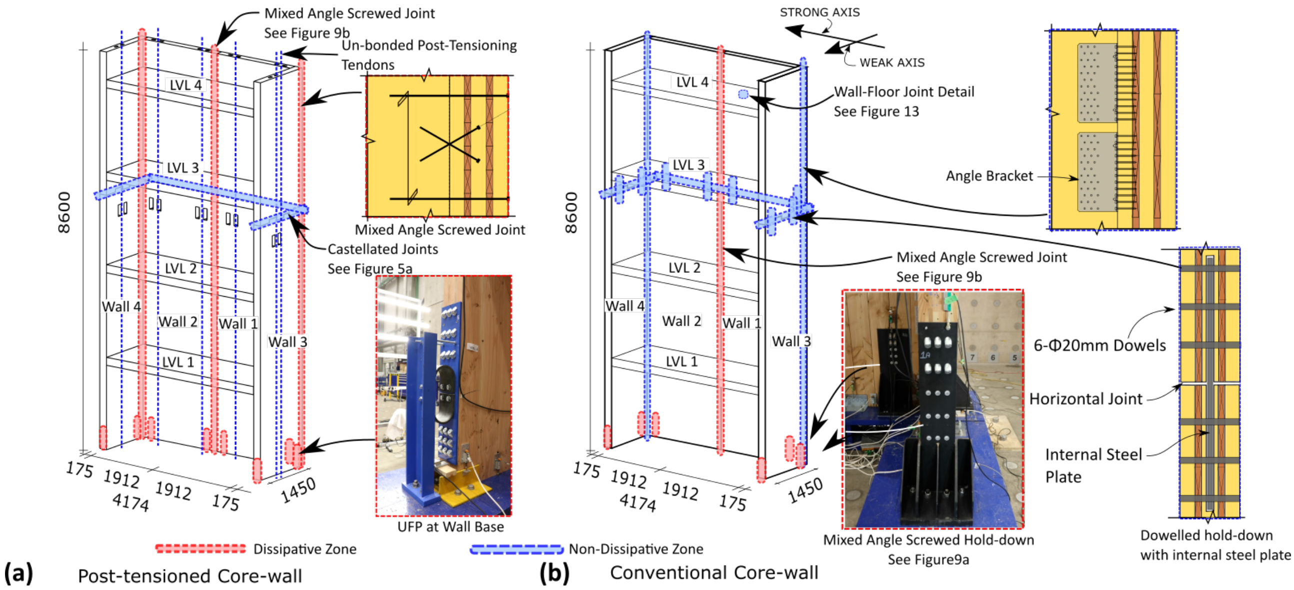

Figure 8: Hierarchy of strength highlighting dissipative (in red) and non-dissipative (in blue) zones for (a) post-tensioned core-wall system and (b) conventional core-wall system

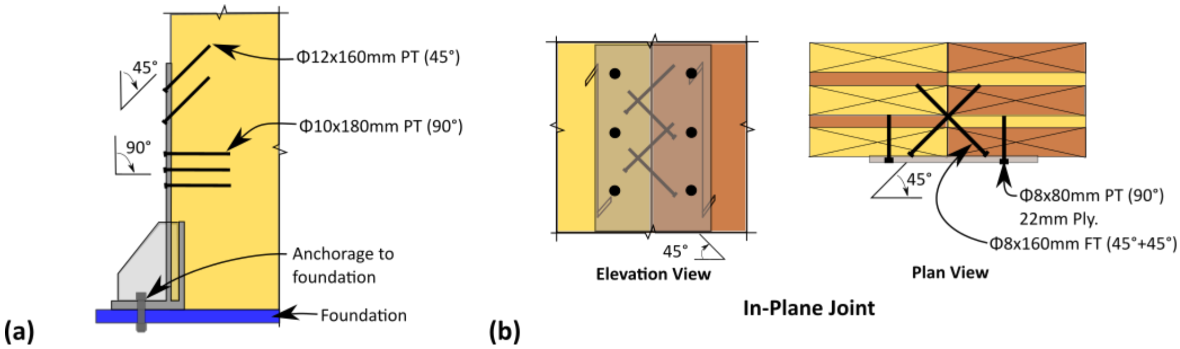

Figure 9: Conventional core-wall test key details: (a) mixed angle STS hold-down (b) in-plane joint

3.2 Selected of Key Results

The key test results along the strong axis loading are reported in Table 3. The table contains experimental results at the serviceability limit state (SLS) level, defined herein as 0.33% inter-story drift ratio, and peak drift (PD) level. In Tests SW-2 and DW-3 the PD was limited to avoid significant damage to the compression toe at the wall base for future tests. The secant stiffness values at SLS and PD levels in Table 3 include all possible slip and translation sliding due to tolerances between the CLT walls. The use of STS could provide high coupling action as observed in Tests DW-3 and CW-6. Comparing SW-2 to Test CW-6 showed that while the CLT wall area increased by 3.5 times, the stiffness at SLS increased by greater than 8 times. Similarly, Comparing SW-2 to Test DW-3 showed that the stiffness at SLS increased by greater than 3 times for a wall area increase by 2 times. This demonstrated the significant increase achievable when the composite action between CLT wall panels is considered.

Table 3: Selection of Key Results

Phase		SW	DW	CW	CW	CW
Test	**Unit**	**2**	**3**	**6**	**7**	**8**
$\mathbf{k_{SLS}}$	kN/mm	1.1	3.7	8.9	4.7	6.3
Peak Drift (PD)	%	0.9	1.2	2.3	2.3	1.8
$\mathbf{F_{PD}}$	kN	57	217	845	460	425
$\mathbf{k_{PD}}$	kN/mm	0.8	2.3	4.5	2.5	2.8

Note: k_{SLS} = stiffness at SLS;; F_{PD} = force at peak drift; k_{PD} = stiffness at peak drift

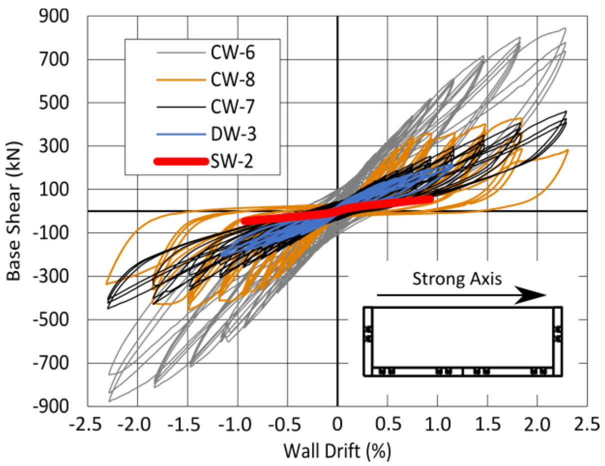

Figure 10: Strong axis hysteresis curves

Figure 10 shows the base shear-wall drift plots. The PT walls displayed a nonlinear geometric elastic behaviour. In Test CW-6 at 2.3% drift, a peak load of 845kN (≈7,000kNm overturning moment at the core-wall base) was achieved. Test CW-8 was unique as it did not use PT to resist overturning moments but used 8 mixed angle STS hold-down connectors as detailed in Figure 9a. It is interesting to note the CW-8 secant stiffness was within the range of CW-6 and CW-7. This indicated that the connection stiffness between CLT panels at the in-plane and orthogonal joint has significant influence on system stiffness in addition to anchorage to the foundation (by PT or hold-down connector).

Tests CW-6 and CW-7 peak drifts (PD) were limited by the actuator's stroke limit of 2.3% and in Test CW-8, PD was limited to 1.8% as the load drop in the 2.3% drift cycle exceeded 20%, typically defined as the ultimate drift in the testing standards. At the end of each test, total residual drifts, which included base sliding due to tolerances, relative joint slip, and timber crushing, were negligible and 0.1% in Tests SW-2 and DW-3 respectively. In Test CW-6, residual drift was negligible until the 1.8% drift cycle, in which it was 0.3% and then 0.5% in the 2.3% drift cycle. In contrast, residual drift occurred throughout the conventional core-wall test. In Test CW-8 residual drifts continually increased from 0.5%, to 1.0%, and finally 1.4% in the 1.5%, 1.8% and 2.3% drift cycles respectively. This shows one significant difference between post-tensioned and conventional shear wall systems in terms of recentering capability. Test CW-8 base shear-wall drift hysteresis curve shows the typical pinching behaviour of connectors with yielding mechanical fasteners and timber embedment crushing mechanisms. A more detailed discussion on the post-tensioned CLT core-wall experimental testing is presented in Brown et al. (2021b).

3.3 Connection Behaviour

The STS connection details for the in-plane and orthogonal joints had primary influence on the CLT shear wall system strength and stiffness. Figure 11 shows the varying joint slips for the in-plane and orthogonal joint of Tests DW-3 and CW-6. At 0.75% drift, the relative slips for the in-plane joint was 9.0 and 5.0mm for Tests DW-3 and CW-6. In Test CW-6, at +0.75% drift the slips of the tension flange orthogonal joint were 1.5mm whereas at -0.75% drift the slip for the compression flange orthogonal joint was -2.5mm and then -10.5mm at -2.3% drift. This indicated the significantly higher shear flow and stiffness demand to engage the compression flange when compared to the tension flange. Further, out-of-plane compression flange rotation occurred at the wall base in the core-wall testing which decreased composite action and this phenomena is discussed in more detail in Brown et al. (2020b). In all wall tests, the in-plane joint was the weakest link due to the high shear demands and indicated the importance of using an energy dissipative connection such as mixed angle STS.

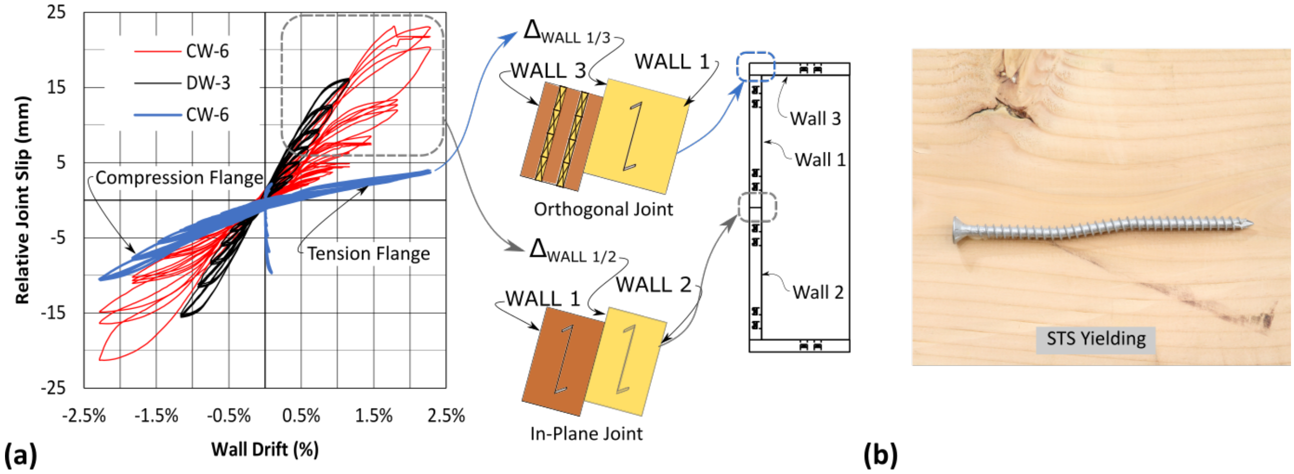

Figure 11: STS in-plane and orthogonal joint slip versus wall drift adopted from Brown et al. (2021)

The castellated joints provided a strong and stiff horizontal joint. Figure 12 shows the castellated joint slip during Test CW-6 where the highest shear demands occurred. At 2.3% drift, the local deformation of the joint was less than 0.4m. The global deformation was approximately 5mm which was primarily due to the installation construction tolerances as shown in Figure 12b. It should be noted that a large amount of friction provided by the PT clamped the panels together, and the vertical jumps in the global joint slip – drift curve indicate instances where instantaneous slips occurred. Future work will develop an analytical calculation method which accounts for the increased complexity of cross-wise layup of CLT carpentry joints.

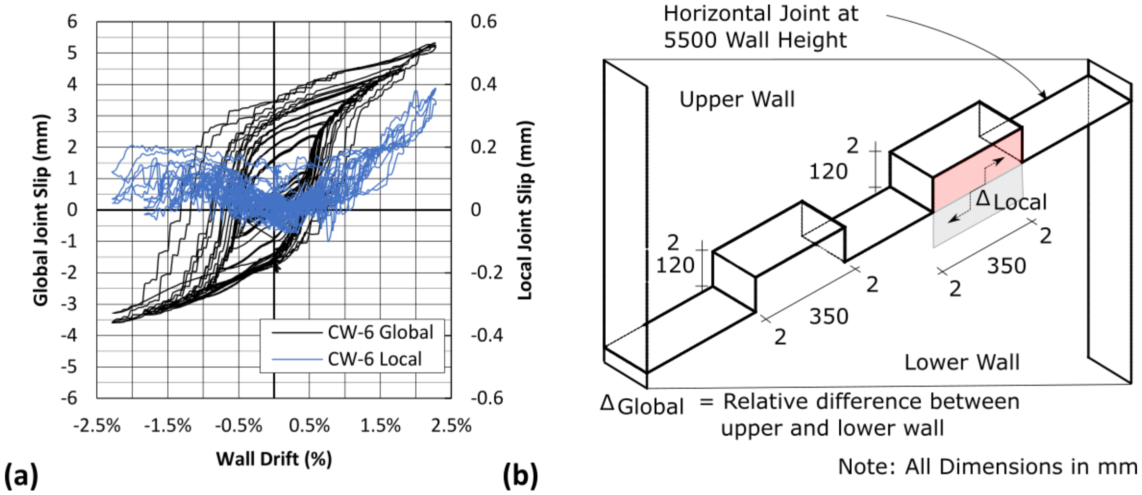

Figure 12: Castellation CLT-CLT joint connection: (a) Test CW-6, (b) castellation isometric

The CLT floors provided out-of-plane restraints in the core-wall testing. Figure 13b shows the L-shaped steel sections with slotted holes for partially threaded STS used which had been previously tested by Moroder et al. (2017). Though the loading was not applied through the CLT floors to represent real applications, Figure 13 shows that the connection detail performed well in decoupling floor rotations from wall rotations due to the rocking motion. At 2.3% wall drift, wall rotation was 0.9° while floor rotations were reduced to 0.3°.

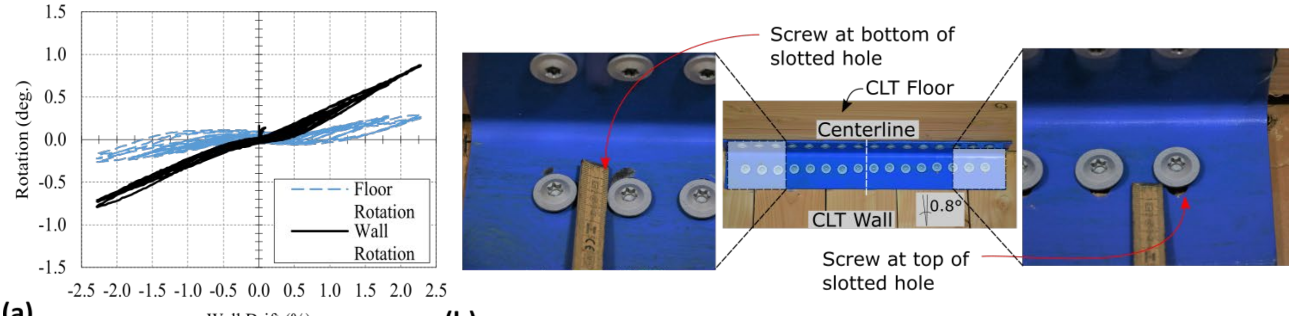

Figure 13: Diaphragm connection performance: (a) wall and floor rotation versus wall drift, (b) during testing photo at 1.8% wall drift ratio

3.4 Conventional Core-Wall Test Performance

The purpose of the conventional core-wall Test CW-8 was to provide a performance benchmark against the previous post-tensioned core-wall tests. As illustrated in Figure 8b, the in-plane joint and screwed hold-down anchorage to the foundation were designed to provide the main sources of system energy dissipation. The 1.8% peak drift (PD) of Test CW-8 was primarily limited by the hold-down connectors which had significant loss in load carrying capacity. At +/-1.8% drift, the average slip of the in-plane joint was 19mm and the extreme hold-down connector slip was greater than 35mm. In contrast, the orthogonal joint remained relatively elastic as designed and at +/- 1.8% drift the tension flange joint slip was approximately 3mm while the compression flange joint slip was approximately -5mm. This affirmed the observation of the increased shear demand for compression flange engagement as occurred in the PT core-wall testing. Figure 14 shows after test photos of the hold-down and in-plane joint. The desired ductile failure modes in the STS hold-down connector of STS yielding, timber embedment crushing and gradual STS withdrawal occurred. This hold-down type with mixed angle STS is a topic of current research (Wright et al., 2021). At the in-plane joint, timber embedment crushing and STS yielding occurred.

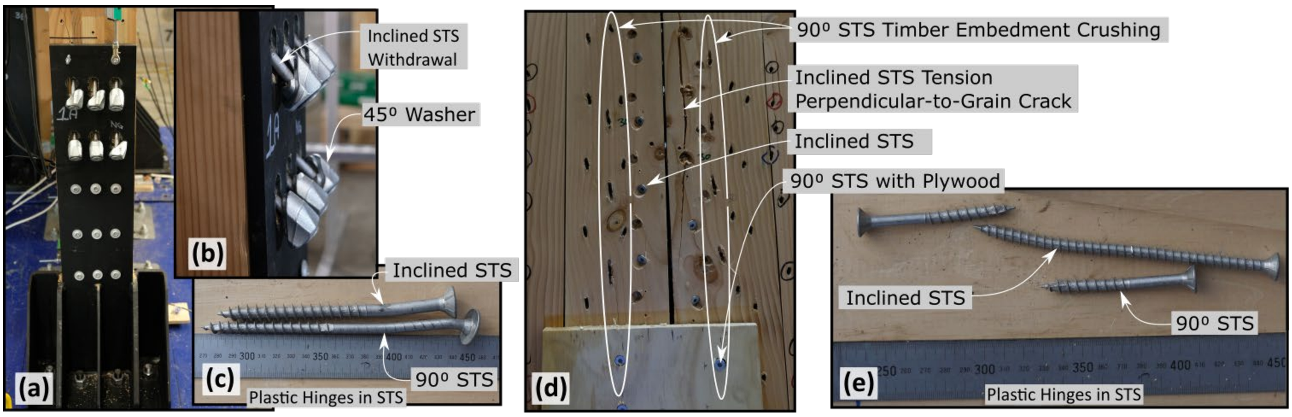

Figure 14: Test CW-8 after test photos of ductile links: (a) – (c) mixed angle hold-down connector, (b) withdrawal failure of inclined STS, (c) yielding of inclined and 90° STS from hold-down connector, (d) after test photo of in-plane vertical joint, (e) yielding of inclined and 90° STS from the in-plane joint

4 CONCLUSIONS

This paper presented an overview and key findings from recent research on CLT lateral load resisting systems (LLRS) for taller timber buildings. The inherent increased flexibility of CLT LLRS due to material properties will require designers to explore methods to increase system stiffness through both connection detailing and/or geometrical means such as core-wall systems. The key findings are summarized as follows:

- Implementing commercial hold-down and angle brackets adopted from LTF will limit the potential of CLT LLRS. Commercial hold-down brackets had reduced strength and displacement capacity when compared to dowelled hold-downs with internal steel plates. Castellated carpentry joints strength and stiffness values were 2.5 and more than 7 times greater when compared to commercial angle brackets
- Dowelled hold-downs with internal steel plates and mixed angle self-tapping screw connections can be designed as a ductile links for a CLT LLRS. Experimental results have shown significant energy dissipation, displacement capacity (greater than 30mm) and experimental overstrength factors for both connections were found to be 1.7.
- It has been demonstrated that a post-tensioned (PT) CLT core-wall system using mainly STS connections could provide significant increase in stiffness greater than 8 times a single post-tensioned CLT wall with a 3.5 times increase in CLT wall area.
- While the conventional CLT core-wall system provided good performance to 1.8% drift, the PT CLT core-wall proved superior in minimizing residual drift and having increased drift capacity.

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