Seismic performance of multi-storey cross laminated timber shear walls with high-capacity anchoring systems

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Executive Summary

New Zealand is a high seismic country and it is critical to design building structures with good seismic performance. Engineered timber products, such as cross laminated timber (CLT), laminated veneer lumber (LVL) and glue-laminated timber (glulam), are becoming popular for building construction in New Zealand and globally as they are considered as sustainable building products and offer cost competitive building solutions. This project aims to enhance seismic performance of multi-storey CLT buildings by developing robust and resilient CLT shear walls systems with high capacity anchoring connection systems. A total of 16 CLT hold-down connection specimens using self-tapping screws and steel brackets were tested to investigate their cyclic performance and the repair strategy for retrofitting. A total of 13 CLT shear wall specimens, including cantilever, multipanel, and hybrid coupled walls, were tested to investigate the overall cyclic performance with high capacity anchoring connections. Three walls were selected and repaired and their performance after repair was experimentally assessed.

Hold-down connections in CLT shear walls are critical components to provide overturning resistance under seismic loading. Meanwhile, they are often specified as ductile links to provide system ductility and energy dissipation. The CLT hold-down specimens used a mixture of inclined self-tapping screws and 90-degree self-tapping screws in order to achieve better seismic performance. Meanwhile, a step by step connection repair method was also proposed and the repaired connections were also tested following the same loading protocol. The results showed that the original and the repaired connections had comparable performance and they all achieved high strength, high stiffness and ductile behaviour.

The shear wall tests considered three types of wall configurations. The cantilever CLT wall tests examined three different height-to-length aspect ratios and two types of high capacity hold-downs (bolts and mixed angle self-tapping screws). It was observed that damage was typically concentrated in the hold-down connections and minor crushing was observed at the toes of the wall bases. The bolted hold-downs were not able to reach their ultimate failure mode due to a limited displacement capacity for the test setup. It was found that taller walls tended to have highest ductility.

Multipanel CLT walls with vertical half-lap joints were tested as they are also frequently used in buildings. Different vertical lap joint details with varying screw types and spacings were considered in three wall specimens. One wall used pairs of inclined fully threaded screws in the half-lap joint and exhibited the highest stiffness as the vertical joint did not deform significantly. The other two walls used partially threaded screws installed at 90 degree to the surface and exhibited lower stiffness but dissipated more energy due to the screw bending and wood embedment crushing in the vertical lap joint.

Finally, a hybrid coupled wall with steel link beams between two adjacent CLT panels was tested. The hybrid wall achieved very high strength and excellent ductile performance and did not exhibit severe pinching behaviour that was observed in the previous cantilever or multipanel wall tests. Initial yielding of the system occurred due to shear yielding of the link beam's web panels. Additionally, the mixed angle screw hold-downs exhibited screw withdrawal and bending while the wall base toes crushed and delamination was observed in some CLT local regions.

One wall from each of the three wall test series was repaired and re-tested to simulate a post-earthquake repair scenario. The repair work was simple and cost effective. Each repaired wall specimen demonstrated similar behaviour and slightly higher peak strength when compared to the original wall.

Keywords: Cross laminated timber, shear walls, seismic design, hold-downs, self-tapping screws, repair method

Contents

1 Introduction

CLT is an engineered mass timber panel product that is becoming increasingly popular for building construction in New Zealand and globally. CLT panels are manufactured by gluing layers of timber boards in a crosswise manner thereby creating solid panels with timber grain oriented in both orthogonal directions. Seismic performance of CLT shear walls is known to be governed by the connection system which also provides system ductility and energy dissipation (Mahammad, et al. 2018). In CLT shear wall construction, commercial anchoring products such as hold-downs and shear brackets are often used. However, these commercial products have limited load-carrying capacity (30~100 kN) because they use thin steel brackets (3~4 mm) and small fasteners such as ø4mm nails or screws. Thus, for multi-storey CLT buildings with high seismic demand, CLT shear walls using the present commercial hold-downs and shear brackets often do not have adequate capacity.

This project aims to develop CLT shear walls with high capacity anchoring connections using long self-tapping screws (STS) and bolts. Bolted connections are commonly used to form strong connections in heavy timber structures. STS are now most commonly used in CLT construction due to their high capacity, fast and easy installation, less strict tolerance requirements, and wide availability in the market. Typical STS vary in diameter from 5 mm to 13 mm and their maximum length may exceed 1000 mm. As these long screws are manufactured using hardened steel with high yield strength up to 1000 MPa, their load carrying capacity is much higher than common nails and screws especially when they are engaged in axial tension (withdrawal) while transferring loads. By having appropriate screw embedment length and balanced ratios between the number of the inclined screws and the 90-dgree screws, the STS connections can achieve better seismic performance than the connections with screws only installed either at an inclined angle or at 90-degree (Tomasi, et al. 2006).

This report presents an experimental study on CLT hold-down connections and shear wall systems with different configurations. The connection study was to investigate the cyclic behaviour of high-capacity CLT hold-downs with steel brackets and mixed angle STS installations. The shear wall study was to investigate the cyclic behaviour of large-scale CLT shear walls utilizing the high-capacity hold-downs with STS and bolts. Meanwhile, a repair strategy of the damaged connections and the shear walls was proposed so that these CLT shear walls with screwed connections could be quickly retrofitted after a major earthquake.

2 CLT hold-down tests

The concept of applying mixed angle self-tapping screw installations in timber connections was originally developed in Europe and further developed in CLT wall systems (Brown et al., 2020; Hossain et al., 2018). Screws are installed at two different angles of installation relative to the direction of loading. Screws installed at an inclined angle relative to loading, for example, 45 degree, have high initial stiffness and strength, but lack ductility or displacement capacity. Screws installed at 90 degree to loading have low initial stiffness, but high ductility and displacement capacity. By combining screws of both installation angles in a joint, it is possible to enhance connection seismic behaviour with high strength, high initial stiffness and high ductility / displacement capacity.

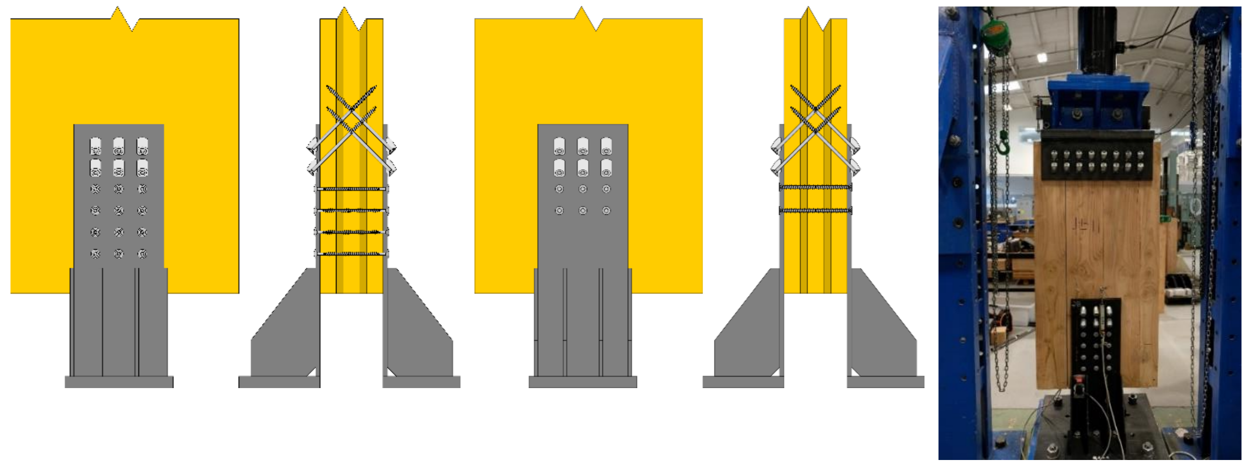
The hold-down connection tests aimed to determine the suitability of mixed angle screw connections for repair, and evaluate one possible method of repair and reinstatement in CLT wall systems that have experienced inelastic deformation due to earthquake events. Key properties to determine were strength and stiffness, and displacement capacity of both original and repaired connection specimens, so these properties could be used to evaluate the proposed repair methodology.

2.1 Test matrix

Table 1 lists the test matrix of the hold-down connections including the size and amount of the screws, number of replicates and loading types (monotonic and cyclic). A total of 16 tests were undertaken with two different mixed angle screw arrangements. These screw layouts are shown in Figure 1a and Figure 1b, respectively. Test Set 1 used 12 ϕ12x260 mm partially threaded screws installed on a 45 degree angle to grain, and 24 ϕ10x180 mm partially threaded screws installed on a 90 degree to grain. Similarly Test Set 2 used 12 ϕ12x260 mm inclined screws, but used 12 ϕ 12 mm 180 mm 90 degree screws instead. The steel brackets were made out of 12 mm thick Grade 300 or Grade 350 steel plates. The brackets were used either side of a 175 mm thick Douglas-fir CLT with a 45/20/45/20/45 layup. All screws were supplied by SPAX Pacific and inclined angle washers by Rothoblaas were used for inclined screw installations. The load was applied by a 1000 kN hydraulic actuator mounted onto a steel reaction frame. A loading rate of approximately 0.2 mm/s was used. The cyclic loading protocol followed the ISO16670 test standard (International Organization for Standardization, 2003)

Table 1 – Summary of hold-down test programme

Test Set	Inclined Screws		90 Degree Screws		Ratio	Original Replicates		Repaired Replicates	
	Qty	*Size*	*Qty*	*Size*		*Monotonic*	*Cyclic*	*Monotonic*	*Cyclic*
1	12	12x260 PT	24	10x180 PT	1:2	1	3	1	3
2	12	12x260 PT	12	12x180 PT	1:1	1	3	1	3



a – Test Set 1 fastener layout *b – Test Set 2 fastener layout* *c – Test Setup*

Figure 1 – Fastener layouts and test setup

2.2 Repair methodology

When detailed correctly, mixed angle screwed connections may have very localised damage around the fasteners. This localised damage leaves timber members intact and the damaged screws are able to be withdrawn and removed. It is also recognised that timber connections utilising external steel side plates have a key advantage that fasteners can be easily removed after a major earthquake. For inclined screws, local withdrawal failure occurs around the fastener, leaving internal damage around the screw hole. For 90 degree screws local timber crushing failure primarily occurs around the screw shank. Based on the localised damage observed, a two-step repair and reinstatement process is proposed.

Step 1: Repair with epoxy

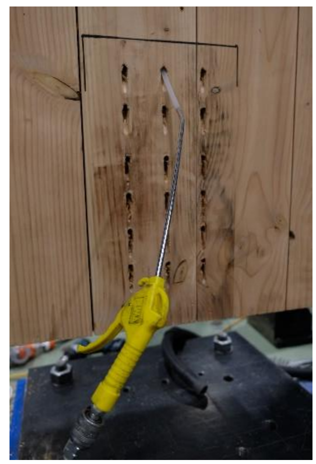
Figure 2 shows the repair process with epoxy. The first step is to repair the damage to timber caused by the inclined screws withdrawing, and the 90 degree screws bearing under the fastener. First the holes are cleaned of loose wood fibres. For the inclined screw holes, this is simply done by reaming the hole with a drill bit sized to suit the epoxy injection tube. For the 90 degree screw holes, the bearing results in lots of loose timber on the edges of the hole. This can be removed by hand with a chisel and pliers, or by using a handheld router to cut out the loose timber. These holes are then blown out with compressed air to remove any remaining timber fibres or dust. Epoxy is then injected into the holes making sure that the epoxy can penetrate and fill the entire hole. In this study Hilti HIT RE-500 was used, with an extension tube being used to inject epoxy into the base of the hole similar to conventional use in concrete construction. Excess epoxy was cleaned off, and once cured the area sanded to remove any excess epoxy from the timber surface.

Step 2: Shift and reinstate fasteners

The second step is to shift and reinstate new fasteners. Due to the localised damage, reinstalling fasteners at even a small offset from the original damaged holes are likely to achieve comparable performance to the original ones. When repairing with epoxy it is also important to install new fasteners at an offset to avoid driving them into the much stiffer and stronger epoxy. In this testing, the new fasteners were installed half way between the damaged holes. i.e. half a spacing either side to epoxy.

A – Damage post original test

B – Removal of loose fibres with pliers



C – Blowing out of dust

D – Injecting of epoxy

E – Epoxy post smoothing

F – Finished repair ready for new fasteners

Figure 2 – Repair Process

2.3 Results and discussion

Table 2 provides a summary of experimental testing results of all 16 tests. Yield strength, Fy, yield displacement, Δy, ultimate strength, Fu, ultimate displacement, Δu, stiffness, K, and ductility, μ, were calculated in accordance with EN12512 (British Standards Institution, 2001). The 30 mm limit on ultimate displacements stipulated in EN12512 was deemed not appropriate for these connections due to their highly ductile response.

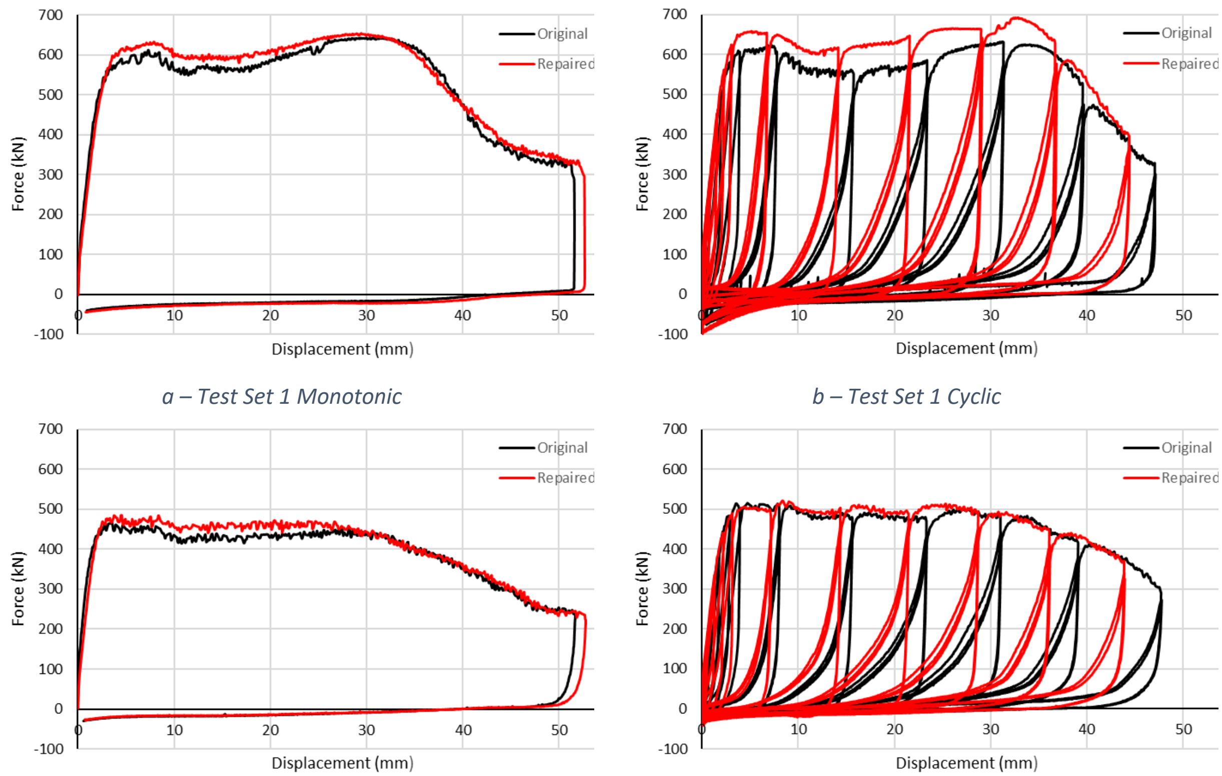
Table 2 – Results summary for connection tests

Test		F_y (kN)	F_{max} (kN)	F_u (kN)	K (kN/mm)	Δ_y (mm)	Δ_u (mm)	μ
1.1 Monotonic	Original	524	643	515	223	1.97	38.7	19.7
	Repaired	555	653	522	211	2.37	37.9	16
1.2 Cyclic	Original	506	622	498	230	1.95	39.8	20.4
	Repaired	561	658	527	226	2.17	38.4	17.7
1.3 Cyclic	Original	492	609	487	256	1.67	39.1	23.4
	Repaired	562	621	497	192	2.71	36.9	13.6
1.4 Cyclic	Original	537	633	506	249	1.82	39.6	21.7
	Repaired	579	692	554	253	1.96	39.5	20.1
2.1 Monotonic	Original	354	466	373	411	0.733	38.9	53.1
	Repaired	424	484	387	260	1.51	36.7	24.4
2.2 Cyclic	Original	447	515	412	233	1.74	40.7	23.4
	Repaired	431	521	417	237	1.59	40.3	25.3
2.3 Cyclic	Original	403	494	395	301	1.2	38.1	31.7
	Repaired	410	507	405	330	1.13	36.7	32.4
2.4 Cyclic	Original	451	512	410	241	1.71	36.9	21.6
	Repaired	458	542	433	303	1.34	34.8	25.9

2.3.1 Performance of mixed angle screws

The results presented in Table 2 confirmed the high strength and high initial stiffness of the mixed angle screw connections. Test Set 1 had average yield strength of 515 kN and average initial stiffness of 240 kN/mm. Test set 2 had average yield strength of 414 kN and average initial stiffness of 297 kN/mm. From Figure 3a and b, it can be seen that Test Set 1 using 12 inclined screws and 24 90 degree screws had considerably higher ultimate and yield strength than Test Set 2. Test set 1 also showed a clear initial peak at approximately 5 mm displacement where the inclined screws reached their maximum capacity, followed by a drop as the inclined screws started to withdraw from the timber. The second peak at approximately 30 mm displacement corresponded to the 90 degree screws reaching their maximum capacity.

From Figure 3c and d, it can be seen that Test Set 2 followed a similar pattern initially with a first peak at approximately 5 mm displacement followed by a drop, but in contrast to Test Set 1 has less of at 90 mm due to the reduced number of 90 degree screws. Covered in more detail by Wright et al. (2021), this shows how the inclined to 90 degree screw ratio can be used to adjust the performance of the overall connection.



a – Test Set 1 Monotonic

b – Test Set 1 Cyclic

c – Test Set 2 Monotonic

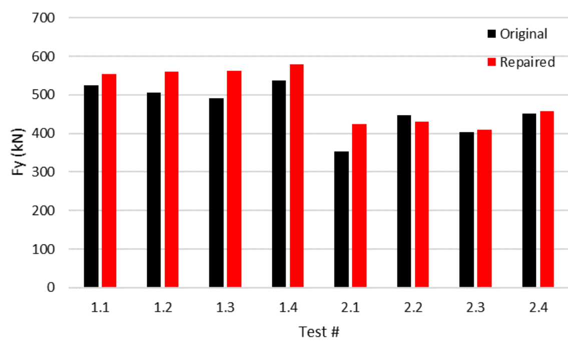
d – Test Set 2 Cyclic

Figure 3 – Load displacement curves showing original vs repaired connections

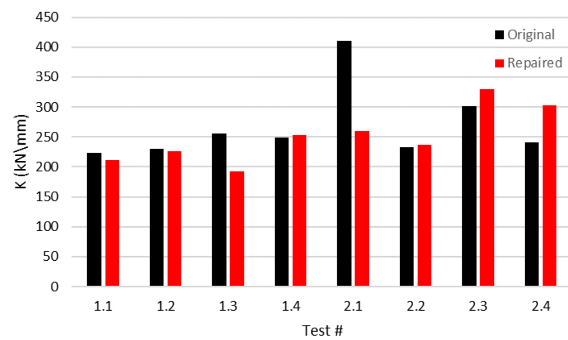
2.3.2 Strength and stiffness

A comparison of yield strength between original and repaired connections is shown in Figure 4a. It can be seen that yield strength after repair increased slightly among all tests. A statistical hypothesis t-test further confirmed that yield strength of the repaired connections was greater than the original test at a 5% level of significance. This increase in strength could be due to two factors: 1) some of the inclined fasteners may hit layers of epoxy on installation. This would lead to an increase in withdrawal strength in the inclined fasteners, and be evident on the initial peak seen in Figure 3; and 2) the epoxy may provide a local reinforcement of the timber and increase the bearing strength of the nearby 90 degree fasteners. This would lead to an increase in shear strength of the 90 degree fasteners and would be seen as a slight increase across the full displacement range, partially apparent where the 90 degree screws hit their max capacity at approximately 30 mm displacement. Based on the curves presented in Figure 3, it was most likely that the increase was due to the inclined screws hitting epoxy as the greatest increase in strength occurred at the initial peak.

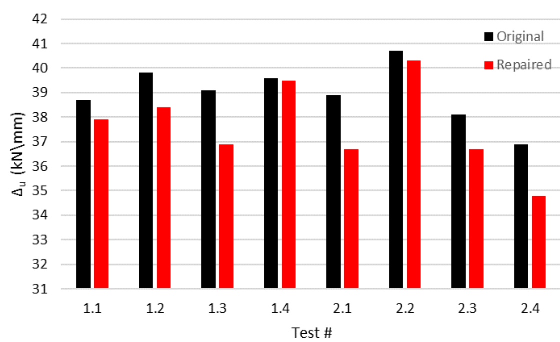
Figure 4b shows the stiffness comparison between the original connections and the repaired connections. It can be seen that there is no clear trend on stiffness between original and repaired tests. A statistical t-test further confirmed that there was no significant difference between the original and the repaired connections at a 5% level of significance. The inherent variability of timber materials and the high dependence of stiffness on the initial conditions of the connection led to some uncertainty in the stiffness values reported. In this case, all screws were torqued to the same setting, but differences were observed in how well the steel hold-downs were sitting against the timber surface, and how the screws were sitting in their holes despite the best efforts to be consistent. Given the increase of yield strength, it is unlikely that stiffness in the repaired connections would decrease significantly.



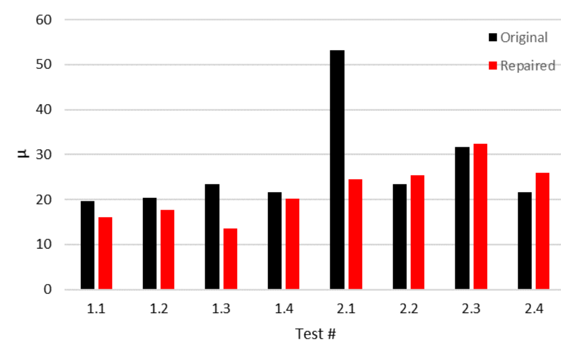
a – Yield Strength



b – Initial Stiffness



c – Ultimate Displacement



d - Ductility

Figure 4 – Bar charts comparing original and repaired test results for key properties

2.3.3 Displacement capacity and ductility

Displacement capacity or ultimate displacement is shown in Figure 4c, and ductility results are shown in Figure 4d. The displacement capacity of most of the repaired specimens was still quite high although slightly lower than that of the original ones. From Figure 3, it can be seen that the load-displacement curves of the original or repaired specimens are very similar. According to EN12512, ultimate displacement is defined as the displacement where the load drops to 80% of the peak. Due to the slight increase in strength discussed previously, the definition of ultimate displacement caused the ultimate displacement to be reported as slightly smaller than the original although their performance was similar. Comparing all original and repaired tests, there is no significant change in performance at large displacements. Similarly, ductility is highly dependent on the yield point of the connection. In stiff but highly ductile connections such as the connections using mixed angle screws, small changes in yield displacement can result in large changes in ductility. When considering the previous discussion about variability in stiffness, the differences in ductility observed should be interpreted with caution.

2.3.4 Failure modes

As previously discussed, mixed angle screw connections may have highly localised damage when detailed correctly. In all the tests, partially threaded screws were used to encourage a gradual withdrawal failure of the screws from the timber, rather than brittle screw tensile failure. In all original tests, the failure mode observed for the inclined fasteners was screw withdrawal. In Test Set 1 using ϕ10 mm 90 degree screws some screw head failures were observed in the 90 degree screws, but it was found that there was a strong correlation between incidents of over-torquing during installation and head failure. In Test Set 2 all 90 degree screws failed due to bending and timber crushing.

No significant changes in the damage to timber were observed between original and repaired tests, as illustrated by the original and repaired specimen 1.2 in Figure 5. In both original and repaired tests there were instances where cracks formed along a row of fasteners, but this row shear was restrained by the internal CLT cross-layers. There was no evidence to suggest any differences in failure mode due to epoxy repair, although it should be noted that in some cases epoxy was hit by the new screws. As discussed before, this may lead to the increase of the withdrawal strength, but it did not cause screw tensile failure because the inclined and 90 degree screws were all partially threaded.

a - Original

b - Repaired

Figure 5 – Comparison of timber surface after original test and after repaired test

3 CLT shear wall tests

CLT shear walls provide critical lateral load resisting systems for most of CLT buildings constructed in seismic regions. A series of single-storey and multi-storey continuous CLT shear walls with high capacity anchoring connections at the wall base were tested to assess their cyclic performance.

3.1 Test matrix

Figure 6 summarizes the test matrix of different wall configurations including cantilever walls, multipanel walls, and hybrid coupled walls. The cantilever walls (W11, W12, W13, W14, W21, W22) evaluated three different height-to-length aspect ratios and two types of high-capacity hold-downs: bolts and mixed angle screws and the hold-downs tests with mixed angle screws have been reported in Chapter 2. The multipanel walls (W31, W32, W33) used a vertical half-lap joint at the mid-length of the wall and varied the details of this joint. Finally, the hybrid coupled wall adopted steel link beams installed between adjacent CLT wall panels to provide coupling action and act as dissipative elements to increase system ductility and energy dissipation capacity. One wall from each test series was repaired and re-tested to evaluate the feasibility of repairing the damaged CLT wall structures after a severe earthquake event.

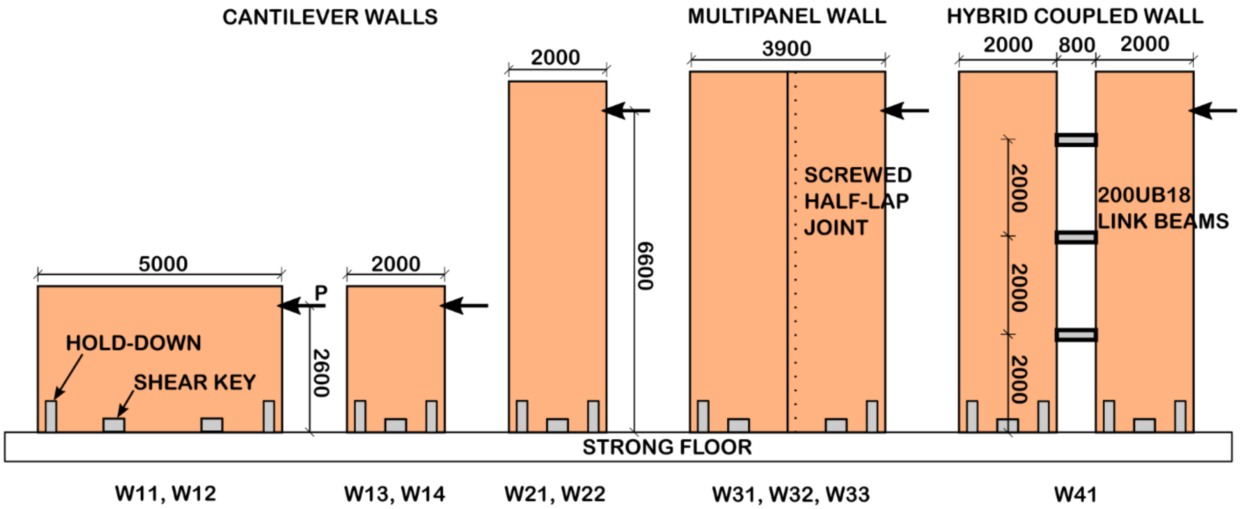


Figure 6: Overview of CLT wall tests.

To simulate a cyclic event, each wall specimen was subjected to the CUREE loading protocol (ASTM E2126-11), shown in Figure 7. A reference displacement was required to define the displacement series and was determined based on the results of nonlinear pushover models created prior to testing.

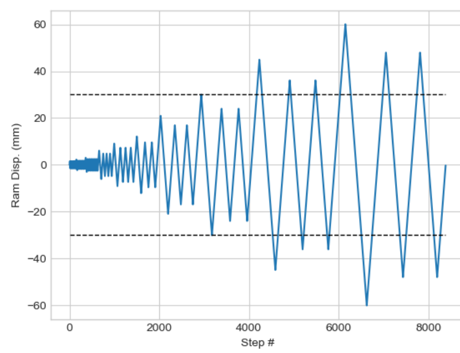


Figure 7: Example of the CUREE loading protocol for cyclic CLT wall testing with D_{ref}=30mm.

3.2 Cantilever wall tests

3.2.1 Overview

A total of six cantilever walls were tested and the test matrix is provided in Table 3. Three height-to-length aspect ratios (0.52, 1.3, and 3.3) were considered. Different hold-down details with bolts and mixed angle screws were considered in these tests.

Table 3: Test matrix for cantilever CLT walls.

Test ID	Length x Load Height (mm)	Hold-down Detail	D_{ref} for CUREE protocol (mm) / (Interstorey drift %)
W11	5000 x 2600	4 - Ø16mm bolts	30 / 1.2%
W12	5000 x 2600	Mixed angle screws 4 - Ø12X260 PT CSK @ 45° 6 - Ø12X180 PT CSK @ 90°	30 / 1.2%

		Each side	
W13	2000 x 2600	4 - Ø20mm bolts	65 / 2.5%
W14	2000 x 2600	Mixed angle screws 8 - Ø12X260 PT CSK @ 45° 12 - Ø12X180 PT CSK @ 90° Each side	65 / 2.5%
W21	2000 x 6600	4 - Ø20mm bolts	198 / 3.0%
W22 W22-R	2000 x 6600	Mixed angle screws 8 - Ø12X260 PT CSK @ 45° 12 - Ø12X180 PT CSK @ 90° Each side	198 / 3.0%

The typical test setup is shown in Figure 8a. The load was applied by a 1000kN hydraulic actuator through a steel clamp at the top of the wall. To provide out of restraint, additional actuators were connected to the specimen by pinned connections with spherical bearings. The base of the CLT walls was connected by hold-downs and shear keys bolted to steel base elements which were again bolted to the concrete strong floor. As shown in Figure 8b, the shear keys consisted of 250x350 mm notches cut at the bottom of the CLT panels and the shear forces were transferred by the contacts between the notches and the thick steel brackets bolted to the steel base elements.

An example of a bolted hold-down is shown in Figure 8b. The bolts were made from round mild steel bar (fy=300MPa) and the ends were threaded to receive a nut on each side. A typical 25mm side plate was used to impose a fixed condition at the ends of the bolt which increased the number of plastic hinges the bolts could form. Figure 8c shows one side of a typical mixed angle screw hold-down with a combination of self-tapping screws installed at 45-degree and 90-degree angles. This type of high capacity hold-down was designed and installed based on the experimental findings from Chapter 2. The combination of the inclined and 90 degree screws provides relatively high stiffness to limit interstorey drifts under service-level earthquake or wind events while also being able to exhibit ductile behavior and dissipate energy in severe earthquakes.

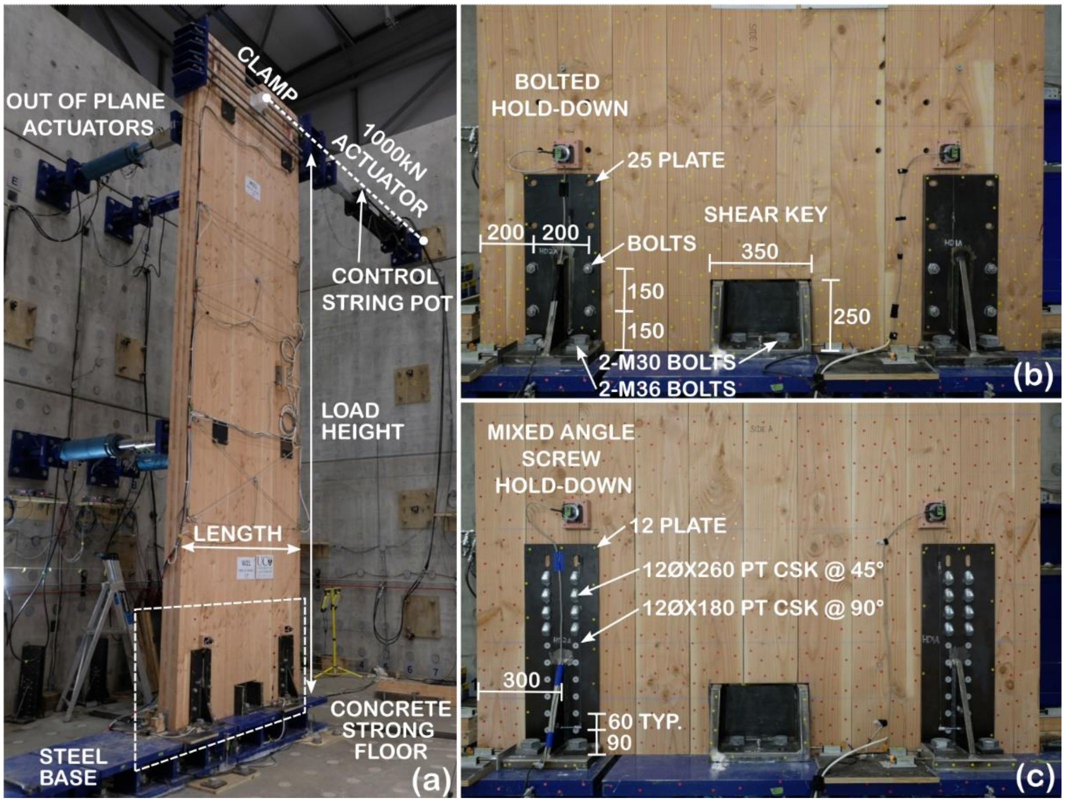


Figure 8: Typical cantilever wall test setup: (a) overall view, (b) bolted hold-down, and (c) mixed angle screw hold-down option.

3.2.2 Test results

Table 4 provides a summary of the key test results including stiffness K, yield strength and displacement (Fy and Dy), maximum strength and displacement (Fmax and Dmax), ultimate strength and displacement (Fu and Du, if applicable), ductility μ, and ultimate failure mode.

Table 4: Test results for cantilever CLT walls

Test ID	Stiffness K (kN/mm)		F_y (kN)		F_{max} (kN)		F_u (kN)		D_y (mm)		D_{max} (mm)		D_u (mm)		Ductility, μ (D_u/D_y)		Ultimate Failure Mode
	+	-	+	-	+	-	+	-	+	-	+	-	+	-	+	-	
W11	40.1	-37.6	374	-316	857[a]	-750[b]	-	-	8.9	-8.0	30.0	-25.0	-	-	-	-	Out-of-plane wall buckling
W12	46.0	-46.4	675	-647	721[a]	-748	-	-592	13.6	-12.9	19.9	-28.0	-	-35.8	-	2.8	(pos.) Out-of-plane wall buckling (neg.) Screw bending and withdrawal
W13	11.4	-15.3	182	-170	450[b]	-536[b]	-	-	15.2	-10.2	129	-152	-	-	-	-	Max drift
W14	14.7	-18.4	398	-423	490	-503	388	-396	24.8	-21.8	71.7	-37.9	97.4	-91.4	3.9	4.2	Screw bending and withdrawal
W21	2.4	-1.9	73	-76	190	-200	-	-	35.0	-30.9	265	-271	-	-	-	-	Max drift
W22	3.3	-3.3	161	-138	185	-170	140	-132	45.4	-43.2	153	-157	197	-209	4.3	4.8	Screw bending and withdrawal

a) Wall buckling occurred and the test was terminated.
b) Maximum displacement in test setup reached.

The wall specimens with bolted hold-downs (W11, W13, W21) were not able to achieve their ultimate strengths. The bolted hold-downs yielded when the load increased and they progressively gained strength due to an increasing contribution from the "rope effect" as these connections had two steel side plates and formed a steel-timber-steel joint. With adequate edge distances and spacings, brittle failure modes will not occur until significant slips cause a mode cross-over, as observed in comparable dowelled CLT connections tested by Ottenhaus et al. (2018). Walls W13 and W21 reached interstorey drifts of 4.5% and 6% (maximum uplift of 88mm at hold-down), respectively, which was the limit for the test setup used in this study and the ultimate failure of the bolted hold-down connection did not occur. The long wall (W11) exhibited buckling out-of-plane prior to reaching the displacement limit of the test setup and the test was terminated.

Ductility, defined as the ratio between the ultimate displacement divided by the yield displacement, was not obtained for these tests due to limited ram capacity or out-of-plane wall buckling. However, for the test specimens that achieved ultimate displacements, the walls with greater height-to-length aspect ratios had greater ductility values.

The global load-drift and moment-rotation curves of each wall is shown in Figure 9a with global drift plotted on the X axis (actuator displacement/load height). It is apparent that the hysteresis loops were severely pinched and therefore had a reduction in energy dissipation and effective stiffness. A significant portion of the global drift was due to rotation of the wall base, shown in Figure 9b.

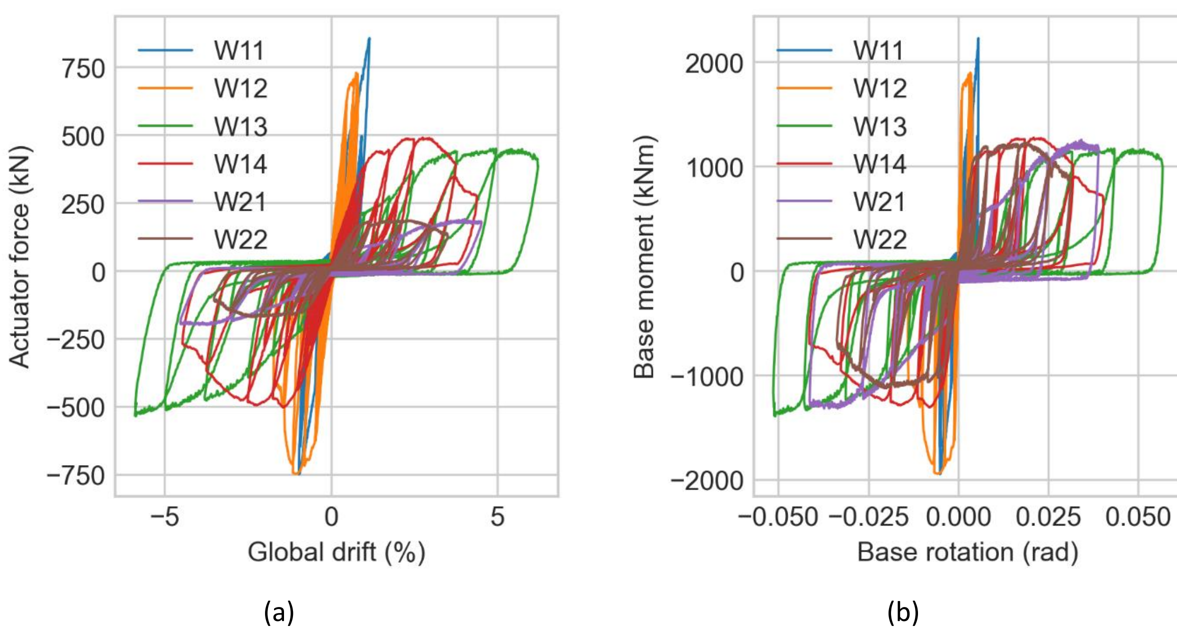


Figure 9: Cantilever wall test results: (a) global drift vs. shear force and (b) base rotation vs. base moment.

The testing of two long walls, W11 and W12, was affected by their tendency to buckle and therefore did not always reach their peak or ultimate loads. Only wall W12 reached its peak and ultimate load in the negative loading direction. Figure 10 shows the applied forces and deformed shape observed as the long wall buckled out of plane. This failure mode was unexpected and has not been considered in previous analytical or experimental research. In a real application, this buckling mode may be mitigated by joining an adjacent floor diaphragm to the wall with an adequately stiff, strong connection. However, the findings of two these tests warrant further investigation to assess such failure modes.

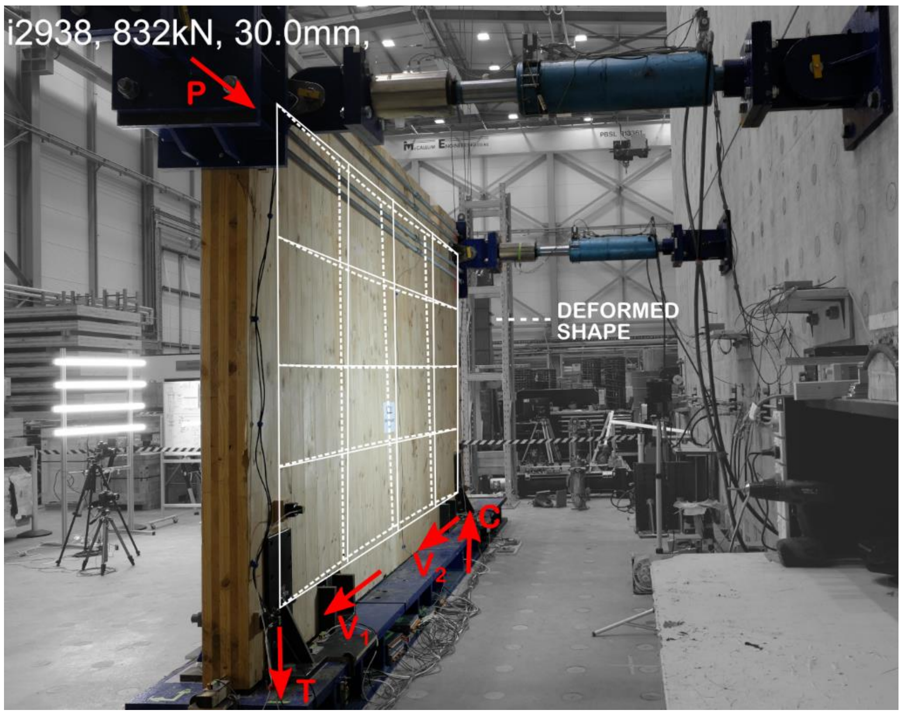


Figure 10: Deformed shape of long wall due to buckling during test.

Typical damage to the bolted hold-down is shown in Figure 11a and a section of one bolt is shown in Figure 11b. Local crushing of the CLT around the bolt was evident and four plastic hinges were formed as the bolt was engaged in bending. Some layers of the CLT panel around the bolted connection showed local brittle failure modes but no ultimate brittle failure mode (i.e. row shear, group tear-out, or net tension fracture) was observed in any of the tests with bolted hold-downs.

Typical damage to the mixed angle screw hold-down is shown in Figure 11c and Figure 11d. The inclined screws progressively pulled out from the timber, rather than fracturing, as intended by restricting the amount of threaded length on the screw shank. The 90° screws developed plastic hinges as they were bent and pulled out of the timber under large displacement demands as they developed an axial force component.

In addition to the damaged observed at the hold-down connections, the toes at each wall base corner showed moderate levels of crushing, shown in Figure 12, due to a concentration of compression force.

Figure 11: Damage observed for (a) bolted hold-down and (b) mixed angle screw hold-down.

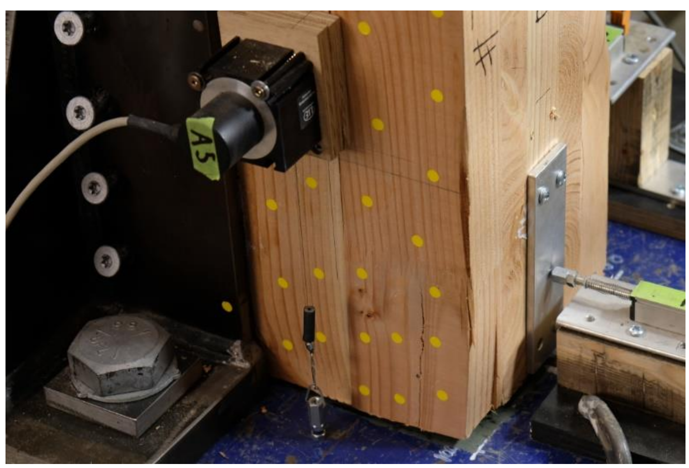


Figure 12: Toe crushing observed in test W22.

3.2.3 Repaired cantilever wall (W22-R)

The tallest cantilever wall with mixed angle screw hold-downs (W22) was repaired following the repair strategy presented in Chapter 2. As shown in Figure 13, damage to the wall specimen was concentrated at hold-down connection area, therefore only a small portion of the wall needed to be accessed for the repair. Crushing at the toes of the wall base was not severe and therefore was not addressed by the repair work.

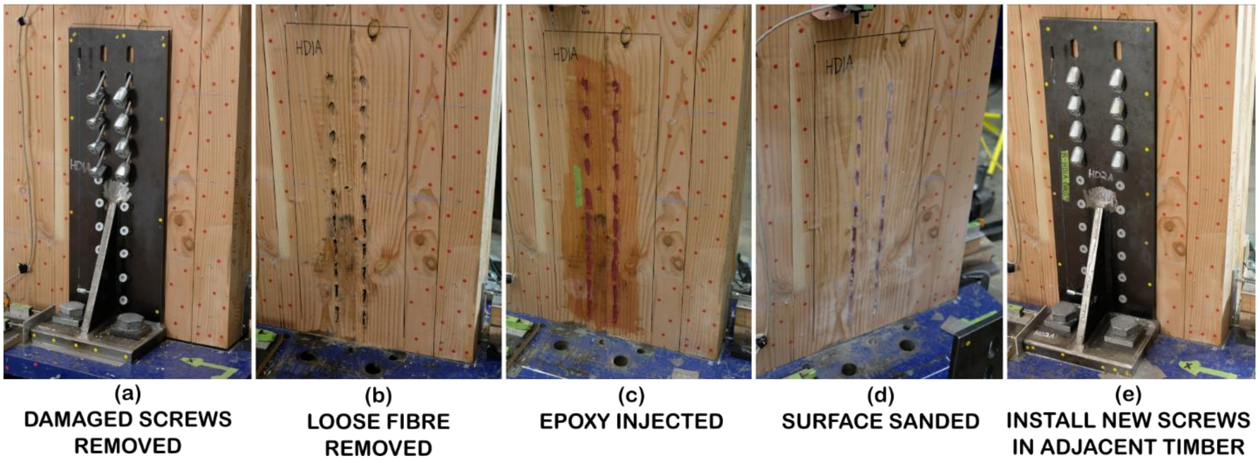


Figure 13: Repair methodology for mixed angle screw hold-down.

The global behaviour of the wall before (W22) and after the repair (W22-R) is shown in Figure 14. It can be observed that the repaired specimen behaved similar to the original wall and exhibited a moderate amount of increased peak strength (+9% positive, +18% negative). The yield strength was also greater than the original wall, particularly in the negative direction (+5% positive, +23% negative). The repaired wall initial stiffness was within 1% of the original wall's value. Therefore, the repair method was successful and provides a feasible option to consider in a post-earthquake scenario.

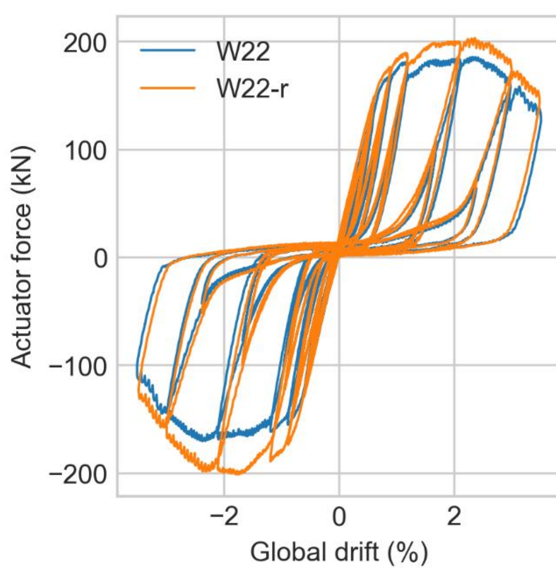


Figure 14: Comparison of global CLT wall behaviour with mixed angle screw hold-downs before (W22) and after repair (W22-r).

3.3 Multipanel wall tests

3.3.1 Overview

In CLT buildings, multipanel CLT walls are formed by joining adjacent CLT walls along their vertical edges, typically with screws. This is often done in buildings because the maximum width of CLT panels is typically limited to 3m due to transportation and manufacturing constraints.

Table 5 provides the test matrix of the multipanel walls that had a typical 100mm-deep half-lap joint with screws to form the vertical joint. The vertical joint details including screw sizes, orientations, and spacings are shown in Figure 15. Figure 16a shows the typical setup for the multipanel wall test which was similar to the cantilever wall setup except that the loading mechanism was different. The actuator load was transferred to two channel struts and further transferred to the wall specimen via pinned connections and steel drag plates, as shown in Figure 16b. The base of the wall (Figure 16c) used the same mixed angle screw hold-downs and notched shear keys as the cantilever walls W14 and W22.

Table 5: Test matrix for multipanel CLT wall tests.

Test ID	Length x Load Height (mm)	Hold-down Detail	Half-lap Joint Screws	D_{ref} for CUREE protocol (mm) / (Interstorey drift %)
W31 W31-R	3900 x 2600	Mixed angle screws 8 -Ø12X260 PT CSK (45°) 12 – Ø12X180 PT CSK (90°) Each side	Ø10x200 PT WH (90°) @ 200 spacing	30 / 1.2%
W32			Pairs of Ø8x260 FT CYL (45°) @ 150 spacing	30 / 1.2%
W33			Ø10x200 PT WH (90°) @ 100 spacing	65 / 2.5%

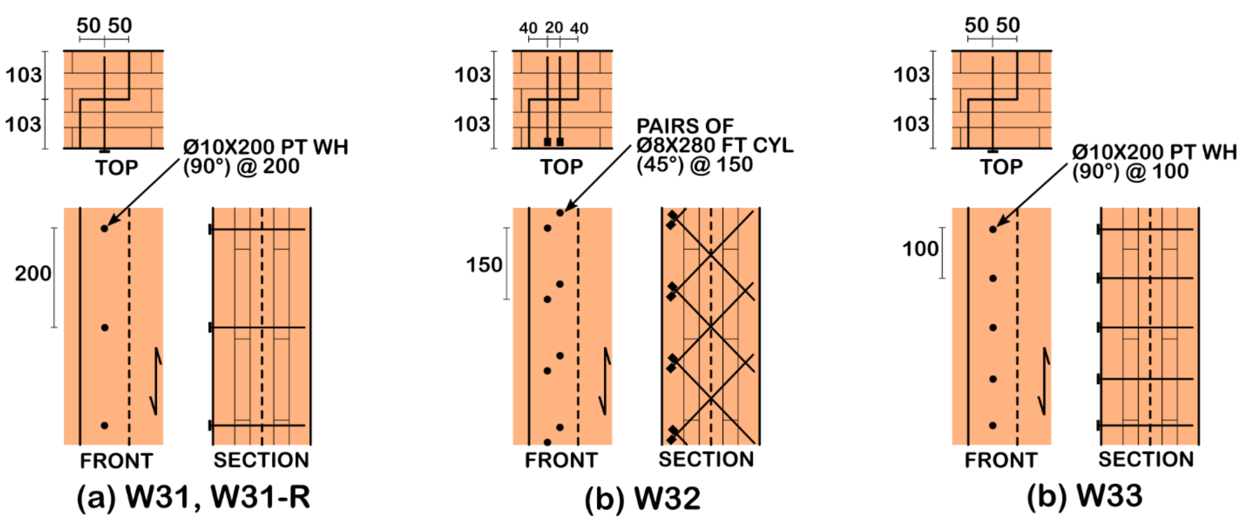


Figure 15: Screw patterns used in vertical half-lap wall joint: (a) W31, W31-R, (b) W32, and (c) W33.

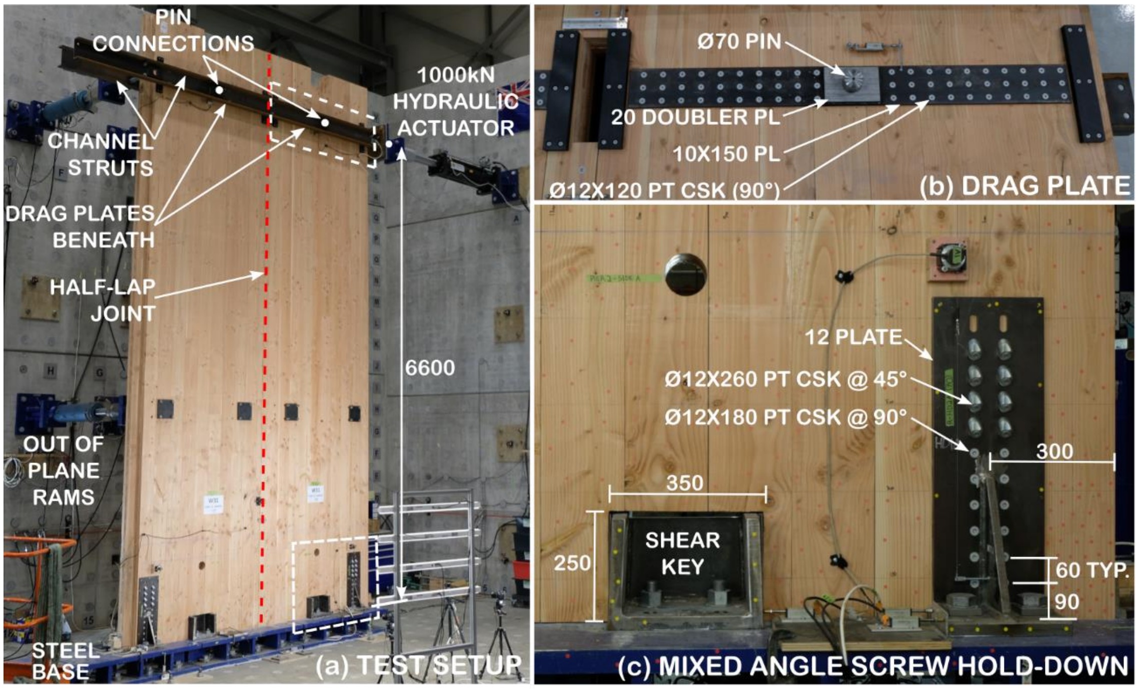


Figure 16: Multipanel CLT wall test setup: (a) overall, (b) drag plate screwed to wall, and (c) wall base connections.

3.3.2 Test results

Table 6 summarizes the key values from the multipanel CLT wall tests. Greater ductility values were generally observed in these walls when compared to the cantilever wall series mainly due to the additional contributions from the vertical lap joints with screws.

Table 6: Test results for multipanel CLT walls.

Test ID	Stiffness (kN/mm)		F_y (kN)		F_{max} (kN)		F_u (kN)		D_y (mm)		D_{max} (mm)		D_u (mm)		Ductility, μ (D_u/D_y)		Ultimate Failure Mode
	+	-	+	-	+	-	+	-	+	-	+	-	+	-	+	-	
W31	8.7	8.3	220	-181	300	-278	236	-200	25.7	-20.0	84.7	-91.9	90.8	-92.5	3.5	4.6	Lap joint screw fracture
W32	14.0	13.5	328	-305	388	-364	308	-280	23.6	-21.9	56.8	-55.4	96.2	-99.2	4.1	4.5	Hold-down screw bending and withdrawal
W33	11.9	10.0	276	-299	409	-418	317	-315	22.8	-28.5	112	-118	130	-133	5.7	4.7	Lap joint screw fracture

The initial stiffness of the wall was significantly influenced by the lap joint details. Wall W32 used pairs of inclined fully threaded screws and achieved the highest stiffness. The walls with 90-degree screws had lower stiffnesses and the lowest was observed in W31, with the least amount of screws (largest screw spacing). When compared to the similar cantilever wall (W22) with a length of 2m and the same screwed hold-down details, the multipanel walls stiffness increased by at least 150%.

The global load-drift curves of the multipanel walls are shown in Figure 16a. Wall W32 was evidently the stiffest and reached its yield, peak, and ultimate points at the lowest displacements. While W33 achieved nearly the same peak strength as W32, it dissipated significantly more energy, as shown by the larger hysteresis loops.

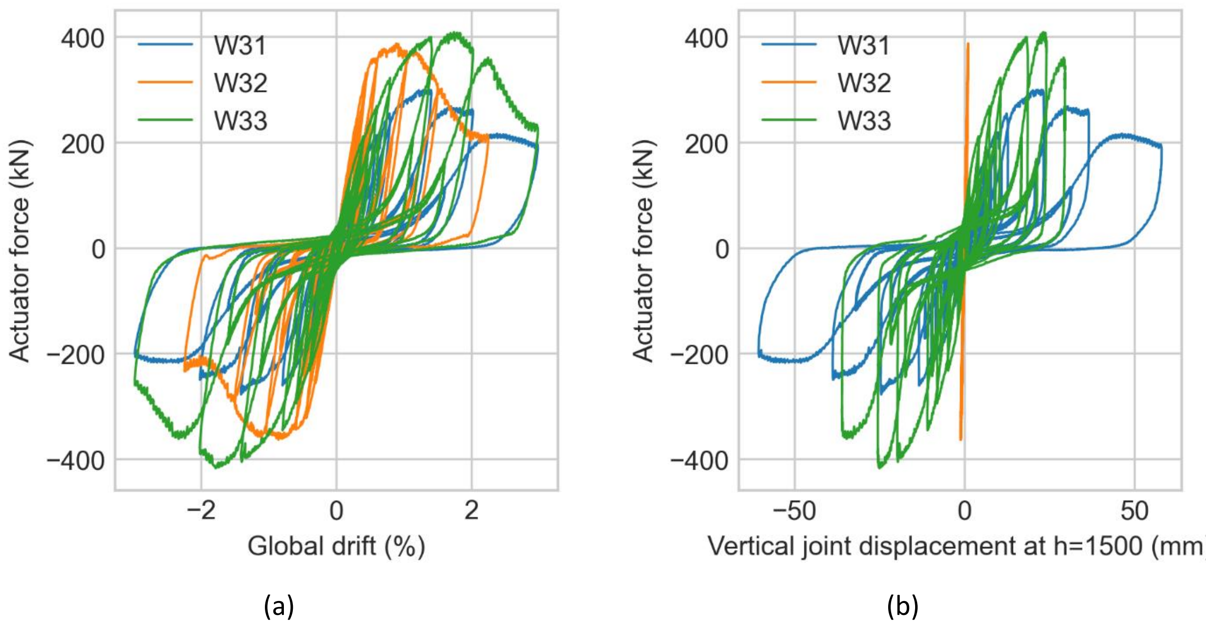


Figure 17: Multipanel CLT wall test results: (a) global drift vs. shear force and (b) vertical joint displacement vs. base shear force.

The kinematic mode of a multipanel wall beyond its yield point is governed by the strength hierarchy between the vertical lap joint and the hold-downs. If the vertical joint is intended to be nearly rigid for high wall coupling effect, then it must be significantly stronger than the hold-down. In contrast, if the designer specifies the vertical joint to be dissipative elements., then the hold-downs must be stronger than the vertical joint.

The nominal lap joint strengths, as governed by the screws, are shown for each specimen in Table 7. The mixed angle screw hold-downs in these tests had an overstrength of approximately 800kN. Therefore, the lap joint strengths of walls W31 and W33 were significantly lower than the hold-down's overstrength and therefore the vertical joint would deform and dissipate energy. Wall W32 had a nominal strength greater than the hold-down's overstrength and therefore was effectively rigid with minimal movement. Wall W32 did not exhibit any damage in the vertical lap joint and the vertical displacement in the lap joint reached a maximum of 1.7mm when the wall was subjected to a lateral force of 388kN. However, walls W31 and W32 did show evidence of damage in the vertical lap joint. The first sign of damage was observed during the test when the screw heads pulled out of the timber (Figure

18a). Upon deconstructing the specimen, local crushing was observed inside the half-lap joint around each screw (Figure 18b). Most of the screws were fractured during the test or fractured when torque was applied to withdraw them from the CLT (Figure 18c). The lap joint screws were bent in double curvature which caused local crushing around the screw and the formation of two plastic hinges, as shown in Figure 18d. In addition, the threaded part of the screw progressively pulled out of the timber. The combination of bending and axial stress on the screw shank with repeated cyclic action caused the fracture of the lap joint screws.

Table 7: Nominal strength of half-lap screws in multipanel walls.

Test ID	Vertical lap joint screws	Nominal lap joint strength (kN)
W31	Ø10x200 PT WH (90°) @ 200 spacing	234
W32	Pairs of Ø8x260 FT CYL (45°) @ 150 spacing	963
W33	Ø10x200 PT WH (90°) @ 100 spacing	475

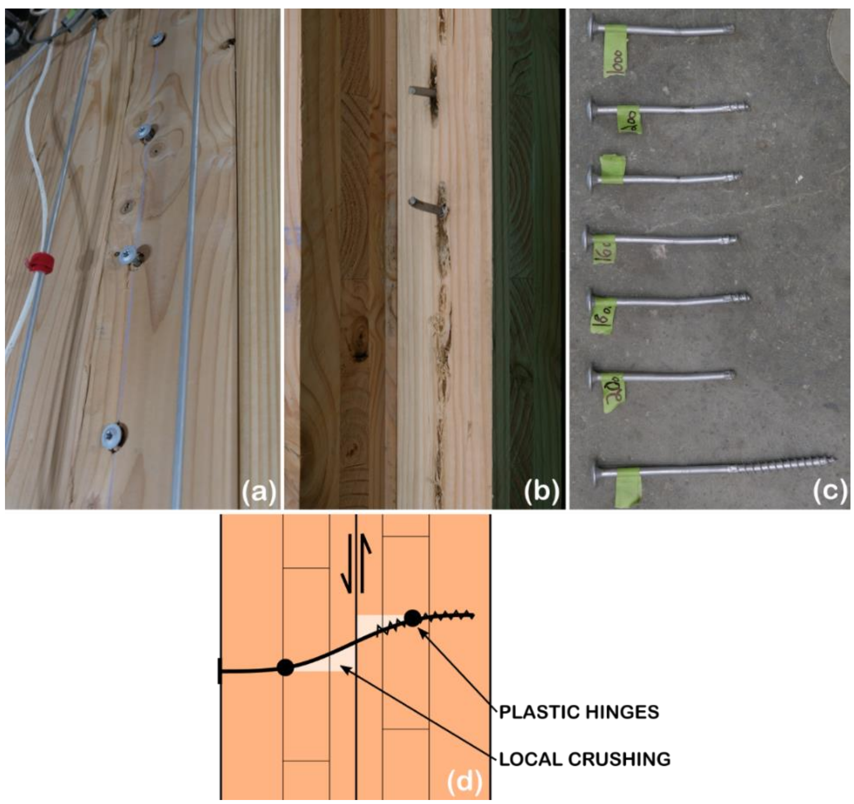


Figure 18: Damaged observed in half-lap joints with screws installed at 90 degrees: (a) head pull-out, (b) internal fracturing and local crushing, (c) fractured screws, and (d) screw failure mechanism.

3.3.3 Repaired multipanel wall (W31-R)

After completion of test W31, the wall was moved back to a vertical position and the screws were removed from the hold-downs and the lap joint, where the damage was concentrated. W31 was repaired and tested again as W31-R. Figure 19a summarizes the extent of repair work. Each hold-down was repaired using the same methodology as described in Section 3.2.3. The lap joint was reconnected using the same type and amount of screws in the previous test and were installed halfway between the previous screw holes, as shown in Figure 19b. Figure 20 compares the global hysteretic behaviour of wall W31 and the repaired specimen W31-R. The hysteretic behaviour of the two tests was very similar but the repaired specimen was slightly stronger, similar to the repaired cantilever wall W22-R. This test further demonstrated the efficiency of the repair methods.

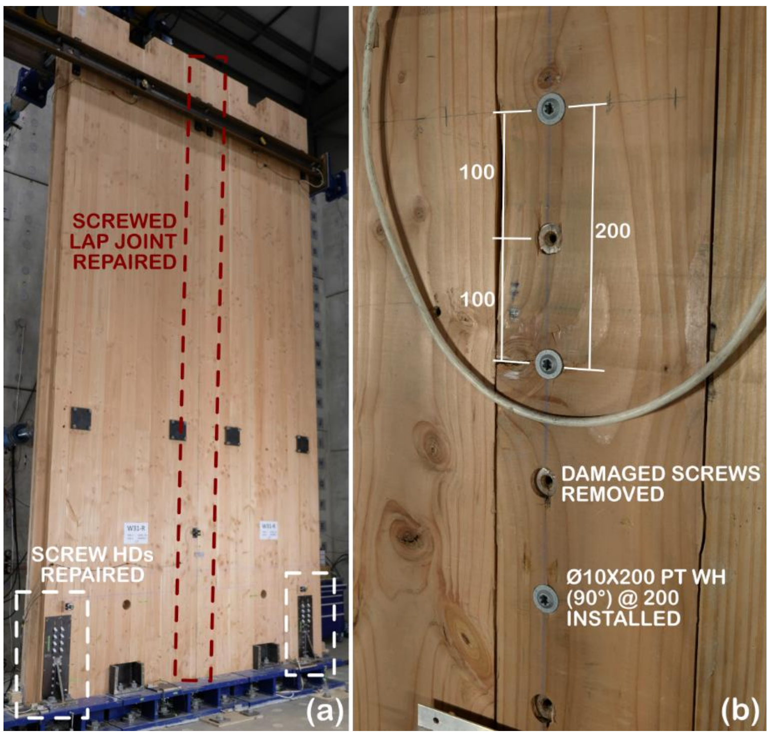


Figure 19: Repair methodology for multipanel wall: (a) overall regions repaired and (b) lap joint screws installed between previous fasteners.

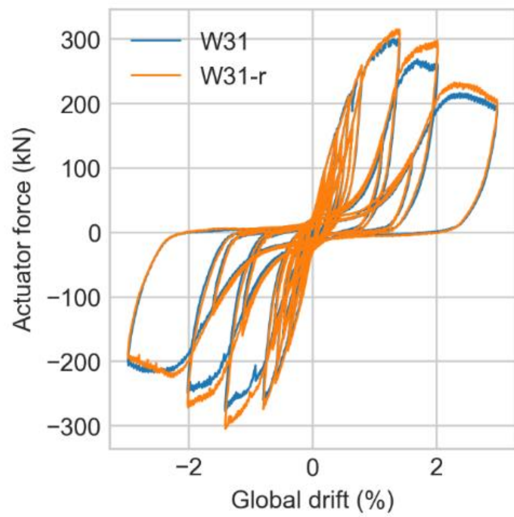


Figure 20: Comparison of multipanel CLT wall behaviour before (W31) and after repair (W31-r).

3.4 Hybrid coupled wall tests

3.4.1 Overview

A hybrid coupled CLT wall system can be created by using steel link beams connected to adjacent CLT wall panels, as shown in Figure 21. This system can be more versatile than the multipanel walls because the link beams can be placed around regular openings like doors or windows over the height of the CLT wall. It also has superior ductility because the steel link beams do not exhibit pinched behaviour like typical timber connections and can meet significant ductility demands, as established by the extensive research on steel EBF link beams.

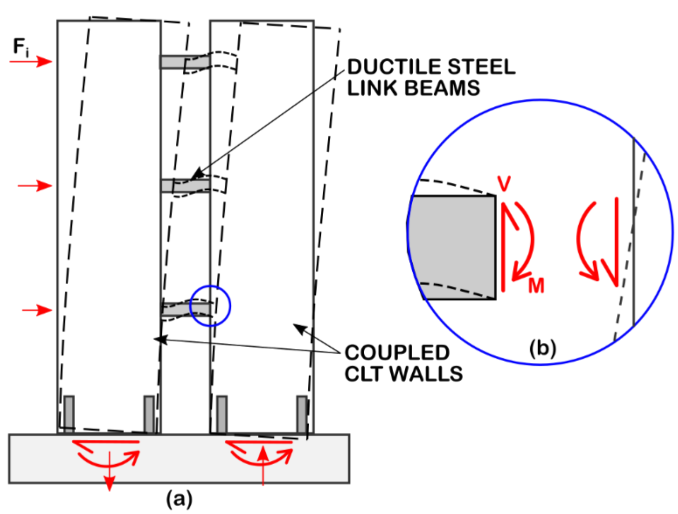


Figure 21: Simplified schematic of a coupled CLT wall system: (a) free body diagram with deformed shape and (b) connection forces.

The hybrid CLT wall test setup is shown in Figure 22. The specimen represented a 3-storey structure scaled by a scale factor of 2/3. The wall panels were 2m-wide 5-ply CLT elements with the same base connections as the cantilever wall specimen W22.

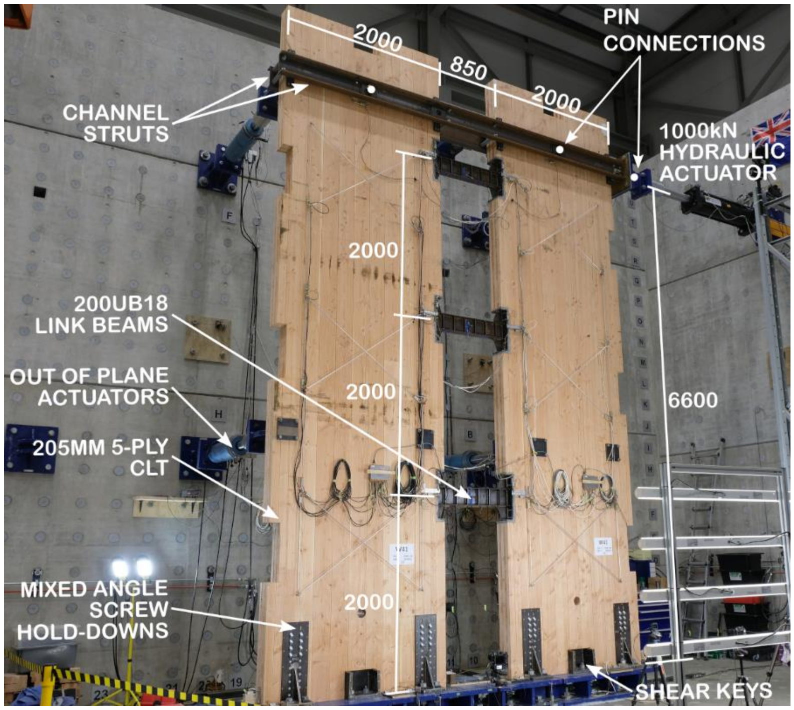


Figure 22: Hybrid coupled CLT wall test setup.

The typical 200UB18 link beams were connected to adjacent CLT wall piers with screwed end plates embedded in a rectangular notch, as shown in Figure 23. A 10mm layer of grout was poured between the elements to create a tight connection. A group of 16-Ø12x350 mm fully threaded screws with countersunk heads were installed into the 20mm steel end plates. The shear force component in the connection was resisted by bearing at the upper or lower notch faces while the moment force component was resisted by a combination of end plate bearing and tension in the screws.

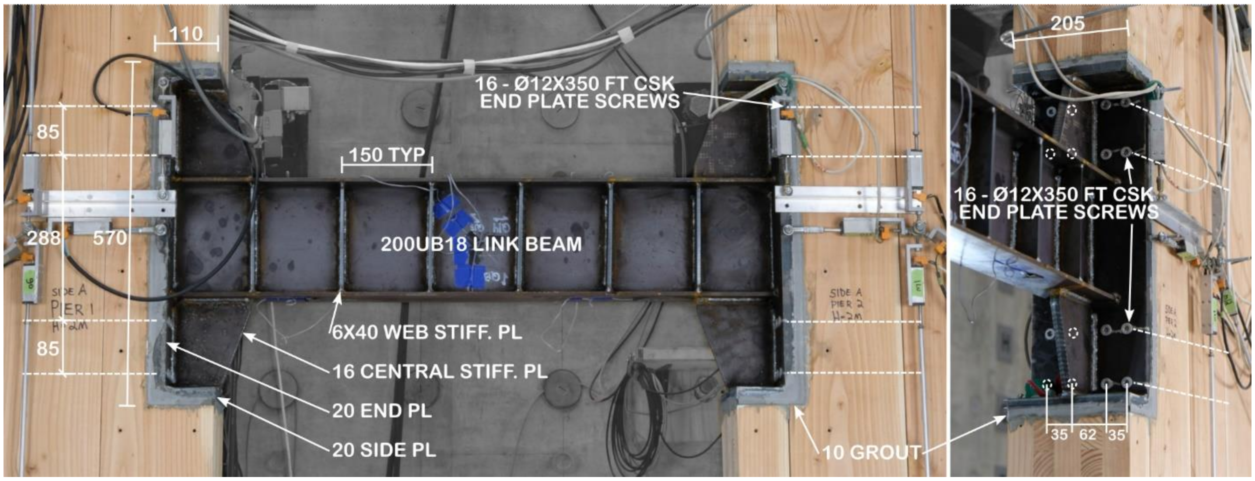


Figure 23: Typical link beam and connection to adjacent CLT panels.

3.4.2 Test results

Table 8 lists the key test results. This wall was significantly stronger than the corresponding multipanel wall due to the contribution of the steel link beams. The peak strength increased by 41% compared with the strongest multipanel wall W33 (590 kN vs. 418 kN). The ductility of this wall was significantly greater than those of the cantilever or multipanel wall series. Additionally, the hysteresis loops, shown in Figure 24, were much fatter than those of the cantilever or multipanel walls because the yielding of steel link beams did not create the pinching effect that is typically observed in timber dowel-type connections.

Table 8: Test results for hybrid coupled CLT wall tests.

Test ID	Stiffness (kN/mm)		F_y (kN)		F_{max} (kN)		F_u (kN)		D_y (mm)		D_{max} (mm)		D_u (mm)		Ductility, μ (D_u/D_y)		Ultimate Failure Mode(s)
	+	-	+	-	+	-	+	-	+	-	+	-	+	-	+	-	
W41	11.9	12.1	431	-409	590	-592	470	-465	34.8	-32.7	137	-133	288	-258	8.3	7.9	Link beam fracture and Hold-down screw bending and withdrawal

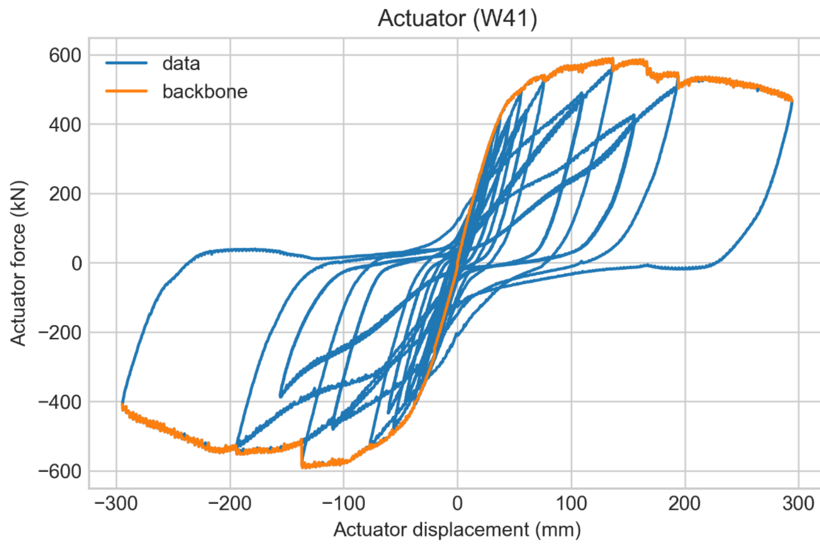


Figure 24: Global behaviour of hybrid coupled CLT wall.

Damage in the hybrid CLT wall first occurred through web yielding in the steel link beams and was observable by the mill scale flaking from the beams' web surfaces. As larger deformations were imposed on the specimen, the link beam flanges and some web panel zones began to buckle inelastically, as shown in Figure 25a. Similar to the other test series, the hold-down screws also withdrew and bent due to the uplift of the wall base (Figure 25b). Additionally, crushing occurred at the toes of the CLT wall piers and some outer boards delaminated due to rolling shear failure in the cross layers near the gluelines.

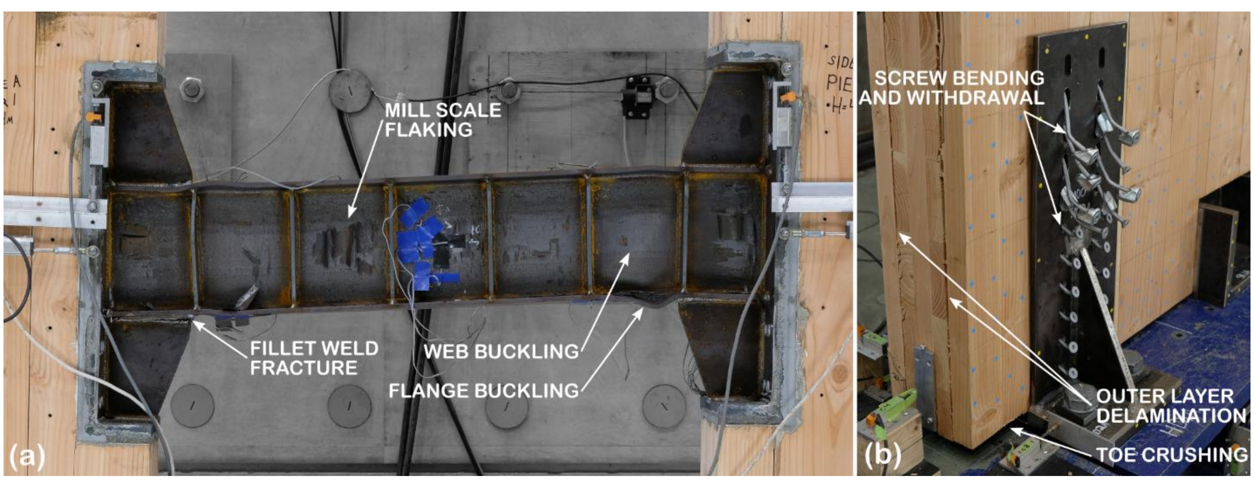


Figure 25: Damage observed after completing the hybrid coupled CLT wall test: (a) link beam and (b) wall base.

3.4.3 Repaired hybrid coupled wall (W41-R)

The coupled CLT wall damage was concentrated in the link beams, hold-downs, and wall toes. Therefore, these regions were repaired to bring the wall system back to a functional state as it would be in a post-earthquake scenario. To repair the beams, the screws and grout were first removed (Figure 26a). Second, the beams were lifted out from between the wall piers (Figure 26b). Finally, three new beams were installed, and grout was pumped between the steel end plate and notch surfaces (Figure 26c). The delaminated boards at the CLT wall base toes (Figure 27a) were re-attached to the CLT panels with screws (Figure 27b and Figure 27c). First, Ø8x80 partially threaded washer head screws with plate washers were installed to pull the board tight to the remaining CLT boards. Then a series of Ø8x260 fully threaded cylinder head screws were installed at an angle of 30 degrees to provide the shear force transfer between board layers.

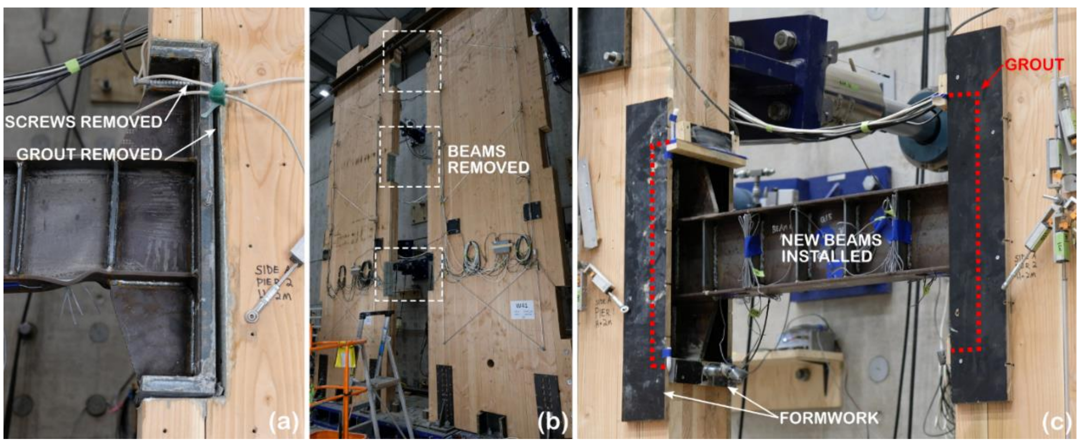


Figure 26: Link beam replacement methodology for hybrid coupled wall: (a) grout and screws removed, (b) beams removed, and (c) new beams installed with new grout poured between elements.

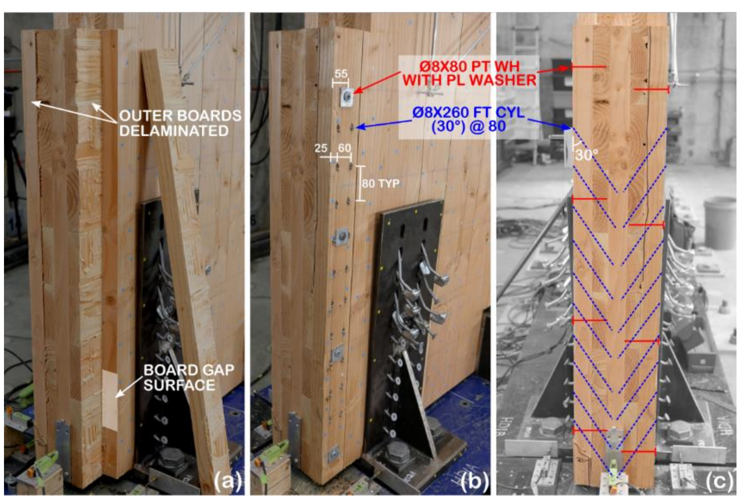


Figure 27: Wall base repair methodology: (a) delaminated boards near toe, (b) screw installation isometric view, and (c) screw installation side view.

The repaired coupled wall (W41-R) performed similarly to the original wall (W41) Figure 28 shows the load-drift curves from two cyclic tests were comparable, which demonstrated the feasibility of the repair method in post-earthquake scenarios. The repaired wall exhibited slightly greater peak strength in both the positive and negative loading directions except for more immediate post-peak strength degradation. Furthermore, the speed and low cost of the required materials make an economic repair solution.

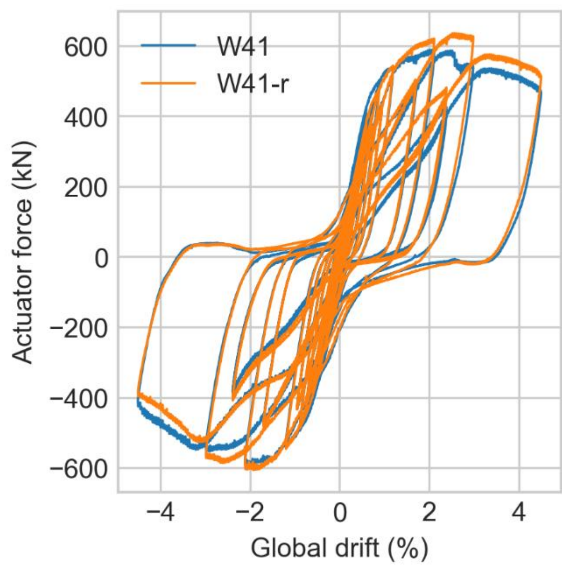


Figure 28: Comparison of hybrid coupled CLT wall behaviour before (W41) and after repair (W41-r).

4 Conclusions

In this project, a total of 16 mixed angle screwed hold-down connections were tested to assess the cyclic performance of high-capacity hold-solutions for CLT shear walls. And a total of 13 large-scale CLT shear walls were tested to assess their cyclic performance. The key findings are listed as follows:

- Mixed angle screw hold-down connections can provide a high performance connection solution for CLT shear walls. Balanced connection behavior with high strength, high initial stiffness and high ductility was observed from the connection tests.
- Mixed angle screw hold-down connections exhibited localised damage in timber that can be repaired and reinstated using epoxy and a small amount of shift of new screw locations. The test results showed a significant increase in strength of the repaired connections using the proposed repair method. And the repair did not significantly affect the connection stiffness and ductility.

- The cantilever CLT walls with high-capacity bolted and screwed hold-downs can provide much higher strength and stiffness when compared with conventional CLT walls using commercial hold-downs and shear keys.
- Half-lap joints in the multipanel CLT walls can either be dissipative or effectively rigid, depending on the strength and stiffness of the screws in the joint in comparison with the hold-downs.
- The Hybrid CLT wall systems with steel link beams can create high-capacity versatile wall systems with greater ductility capacities than the cantilever or multipanel CLT walls.
- The experimental study also indicated that CLT shear walls can be effectively repaired following a devastating earthquake event when capacity design is followed to protect the main timber members and the damage is concentrated in the ductile elements (hold-downs, lap joints, and/or link beams).

Acknowledgement

This research project was funded by Earthquake Commission Biennial Grants, WIDE Trust, and SWP Research Partnership. The authors are grateful for their support and for the technical support by Alan Thirlwell, David Carney, and Michael Weavers from Department of Civil and Natural Resources Engineering at University of Canterbury.

References

- ASTM E2126. (2011). Standard test methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings. ASTM International, West Conshohocken, PA, USA.
- British Standards Institution. (2001). BS EN 12512:2001: Timber structures. Test methods. Cyclic testing of joints made with mechanical fasteners. British Standards Institute.
- Brown, J. R., Li, M., Tannert, T., & Moroder, D. (2020). Experimental study on orthogonal joints in cross-laminated timber with self-tapping screws installed with mixed angles. Engineering Structures, 111560. https://doi.org/10.1016/j.engstruct.2020.111560
- Hossain, A., Popovski, M., & Tannert, T. (2018). Cross-laminated timber connections assembled with a combination of screws in withdrawal and screws in shear. Engineering Structures, 168, 1–11. https://doi.org/10.1016/j.engstruct.2018.04.052
- International Organization for Standardization. (2003). Timber structures—Joints made with mechanical fasteners—Quasi-static reversed-cyclic test method (ISO 16670:2003).
- Mohammad M., Blass H., Salenikovich A., Ringhofer A., Line P., Rammer D., Smith T. & Li M. (2018). Design approaches for CLT connections. Wood and Fiber Science 50: 27-47.
- Ottenhaus, L., Li, M., Smith, T., & Quenneville, P. (2018). Mode Cross-Over and Ductility of Dowelled LVL and CLT Connections under Monotonic and Cyclic Loading, J. Struct. Eng. 144 (2018) 04018074. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002074.
- Tomasi, R., Piazza, M., Angeli, A., & Mores, M. (2006). A new ductile approach design of joints assembled with screw connectors. WCTE 2006.
- Wright, T., Li, M., Moroder, D., & Carradine, D. (2021). Cyclic behaviour of hold-downs using mixed angle self-tapping screws in Douglas-fir CLT, Proceedings of the 2021 New Zealand Society for Earthquake Engineering Annual Technical Conference, Christchurch, New Zealand.