World Conference on
Timber Engineering
Oslo 2023

EXPERIMENTAL TESTING OF MIXED ANGLE SCREWED HOLD-DOWN CONNECTIONS FOR CLT SHEAR WALLS

Thomas Wright[1] **and Minghao Li**[2]

ABSTRACT: Cross laminated timber (CLT) shear walls are an efficient lateral load resisting system for mass timber buildings. Ductility and energy dissipation of timber buildings is mostly provided by well detailed connections. Therefore, to achieve good seismic performance of CLT shear walls, hold-down connections must be not only strong and stiff but also ductile with sufficient displacement capacity to meet the drift demands. A novel hold-down connection is proposed using a mixture of screws installed at an inclined angle and 90° to the grain. The respective benefits of inclined and 90° screws can be combined to create a strong, stiff, and ductile connection. Two stages of experimental testing were undertaken with target connection capacities of 600 kN and 1200 kN respectively. The experimental results confirmed that mixed angle screws can provide a strong, stiff, and ductile hold-down solution for CLT shear walls. The optimal ratio of φ12mm inclined screws to φ10mm 90° screws was 1:2, and the optimal ratio of φ12 mm inclined screws to φ12 mm 90° screws was 1:1.5. The primary failure modes were screw withdrawal and wood embedment crushing. Only localised damage of timber around the screw holes was observed and this was repaired with epoxy. New screws were then reinstated with a small offset to the original screw locations and the repaired hold-down connections were found to have the same or even slightly better performance.

KEYWORDS: Cross Laminated Timber, CLT, Hold-down Connections, Self-Tapping Screws, Mixed Angle, Cyclic Loading, Shear Walls, Ductility

1 INTRODUCTION

Cross-laminated timber (CLT) shear walls are commonly used as lateral load resisting systems in mass timber buildings. It is well understood that timber exhibits brittle behaviour when it reaches failure loads, particularly under tension. Therefore, when designing a building to resist seismic loading, any ductility or energy dissipation has to be provided by connections between timber elements. Therefore, in seismic countries like New Zealand, CLT shear walls are often designed with capacity design principles so that the CLT wall panels remain elastic while specified connections have ductile behaviour under severe ground shaking. To achieve optimal seismic performance, these ductile connections must be well detailed with sufficient strength, stiffness, and ductility / displacement capacity to allow the building to yield and dissipate earthquake energy.

Self-tapping screws are widely used for mass timber construction including CLT buildings. They are made out high strength steel and are easy to implement on site with hand tools. Previous research on self-tapping screws has highlighted the benefits of installing screws at an inclined angle to the grain in terms of strength and stiffness [1]–[3]. However, it is well recognized that with this increase in strength and stiffness, ductility / displacement capacity is compromised as the inclined screws primarily transfer load in an axial tension action. Screws installed perpendicular to the grain (or 90° screws) have relatively low strength and stiffness compared with inclined screws, but as they are relatively slender and can deform well and have good ductility / displacement capacity when loaded laterally. Previous research on timber to timber connections by Tomasi [4] showed that connections comprised of mixed angle screw installations, i.e. a combination of inclined screws and 90° screws, can lead to optimised connection behaviour with high strength, high stiffness and reasonably good ductility. When both inclined and 90° screws are used in a connection, their contributions are superimposed with the inclined screws providing high strength and stiffness, and the 90° screws providing high ductility and displacement capacity.
Further research has adopted this concept for in-plane joints between CLT shear walls [5] and orthogonal joints for C-shaped post-tensioned CLT shear walls [6]. Given the clear performance benefits of mixed angle screws, over inclined or 90° only installations, it is proposed to extend on previous research and develop mixed angle screwed hold-down connections for CLT shear walls.

Typical timber connections designed with ductility are susceptible to damage during a major seismic event. If these connections can be repaired quickly and economically, seismic resilience of timber buildings can be improved. It is well understood that the timber material

---

[1] Thomas Wright, University of Canterbury, New Zealand, thomas.wright@pg.canterbury.ac.nz
[2] Minghao Li, University of Canterbury, New Zealand / University of British Columbia, Canada, minghao.li@ubc.ca

https://doi.org/10.52202/069179-0268

is relatively easy to work with on site and likely to be repaired.

Projects such as the SOFIE project have highlighted the feasibility of repairing CLT shear wall buildings [7], [8]. Methods that have been used include removing damaged fasteners and reinstalling new ones at an offset and installing new hold-down brackets. To enact a successful repair, it is important that damage is concentrated to the connections, brittle failure of the timber element itself is avoided, and there is an easy access to conduct the repair.

This paper provides an overview of an experimental research program at the University of Canterbury to develop a robust hold-down connection solution for high capacity CLT shear walls that uses mixed angle self-tapping screws. The key research objectives are listed as follows:

- Assess the performance of mixed angle screws in CLT hold-down connections;
- Determine the optimal ratios of inclined to 90° screws for optimal seismic performance;
- Investigate connection damage under cyclic loading and compare to other connection systems with similar capacity; and
- Determine options for post-earthquake repair and evaluate performance of repaired connections

2 EXPERIMENTAL PROGRAMME

A total of 55 connection tests of mixed angle screwed hold-downs were conducted over two testing stages. Stage 1 consisted of 46 hold-down specimens with approximately 600 kN target capacity. Stage 2 consisted of 9 larger hold-down specimens with approximately 1200 kN target capacity. Of the 55 tests, 36 were conducted on original connections and 19 were conducted on repaired connections.

2.1 TESTING PROGRAMME

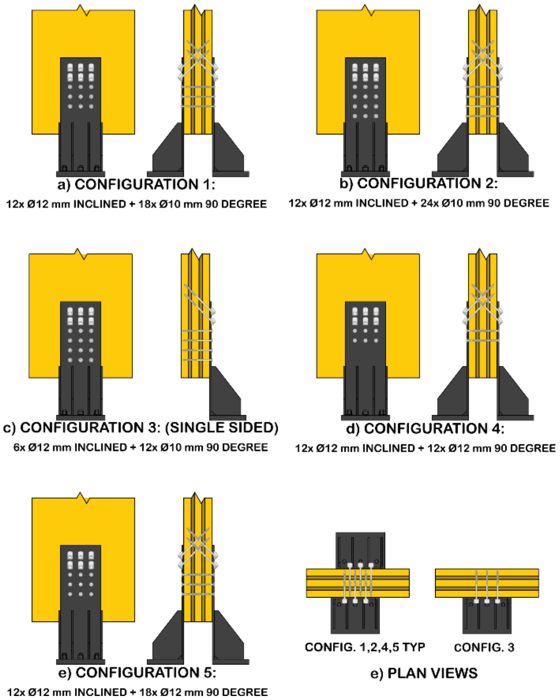
Table 1 and Table 2 show the test matrix of Stage 1 (labelled as S1) and Stage 2 (labelled as S2) on the original and repaired connection specimens, respectively. Repairs were not undertaken for all the original specimens due to time constraints. The 36 original connection specimens were grouped into 9 test sets with 5 distinct connection layouts. The 5 connection layouts tested in Stage 1 are shown in Figure 1. Stage 2 only tested one connection layout shown in Figure 2. This layout is a scaled-up version of Configuration 5 from Stage 1. In the test matrices test sets are labelled (stage)-(material species)-(screw configuration). a1 spacings were 60 mm for both Stage 1 and Stage 2. a2 spacings were 65 mm for Stage 1 (32 mm considering fasteners from both sides) and 60 mm (30 mm) for Stage 2. These spacings are shown for a typical connection layout in Figure 3.

2.2 TESTING APPARATUS

The testing apparatus for Stage 1 is shown in Figure 4. A steel reaction frame is used in conjunction with a single 1000 kN hydraulic actuator. Load was transferred to the CLT specimen using an inclined screw connection for Configurations 1-2, 4-5, and a dowelled connection for lower capacity single sided tests of Configuration 3.

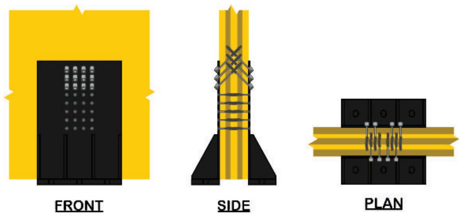
Displacement was measured by 4 potentiometers measuring between the steel hold-down and timber surface as shown in Figure 6.

The testing apparatus for Stage 2 is show in Figure 5. The test specimen is laid horizontally compared to the vertical configuration in Stage 1, with two 1000 kN actuators anchored to the strong wall. The reaction force was provided by a steel frame with 1500 kN design capacity. Similar to Stage 1, load is transferred into the CLT specimen using two large inclined screw connections. As the test specimen is laid horizontally, rollers are used to support its gravity load, and also to restrain it from out of plane buckling under compressive loading. Displacement is measured by 6 string potentiometers placed between the timber specimen and the steel hold-down bracket.


a) CONFIGURATION 1:
12x Ø12 mm INCLINED + 18x Ø10 mm 90 DEGREE
b) CONFIGURATION 2:
12x Ø12 mm INCLINED + 24x Ø10 mm 90 DEGREE
c) CONFIGURATION 3: (SINGLE SIDED)
6x Ø12 mm INCLINED + 12x Ø10 mm 90 DEGREE
d) CONFIGURATION 4:
12x Ø12 mm INCLINED + 12x Ø12 mm 90 DEGREE
e) CONFIGURATION 5:
12x Ø12 mm INCLINED + 18x Ø12 mm 90 DEGREE

e) PLAN VIEWS

Figure 1: *Stage 1 connection configurations*



STAGE 2: CONFIGUATION 5
24x Ø12 mm INCLINED + 36x Ø12 mm 90 DEGREE

Figure 2: *Stage 2 connection configuration*

Table 1: *Test matrix for original connection tests*

	Test Set	Species	Qty (inclined)	Size (inclined)	Qty (90°)	Size (90°)	Ratio	Mono	Cyclic
S1	S1-DF-1	D. fir	12	12x260 PT	18	10x180 PT	1:1.5	2	3
	S1-DF-2	D. fir	12	12x260 PT	24	10x180 PT	1:2	1	3
	S1-RP-2	Pine	12	12x260 PT	24	10x180 PT	1:2	1	3
	S1-DF-3	D. fir	6	12x260 PT	12	10x180 PT	1:2	2	3
	S1-DF-4	D. fir	12	12x260 PT	12	12x180 PT	1:1	1	3
	S1-RP-4	Pine	12	12x260 PT	12	12x180 PT	1:1	1	3
	S1-RP-5	Pine	12	12x260 PT	18	12x180 PT	1:1.5	1	3
S2	S2-DF-5	D. fir	24	12x260 PT	36	12x180 PT	1:1.5	1	2
	S2-DF-5	Pine	24	12x260 PT	36	12x180 PT	1:1.5	1	2

Table 2: *Test matrix for repaired connection tests*

	Test Set	Species	Qty (inclined)	Size (inclined)	Qty (90°)	Size (90°)	Ratio	Mono	Cyclic
S1	S1-DF-2-R	D. fir	12	12x260 PT	24	10x180 PT	1:2	1	3
	S1-RP-2-R	Pine	12	12x260 PT	24	10x180 PT	1:2	1	3
	S1-DF-4-R	D. fir	12	12x260 PT	12	12x180 PT	1:1	1	3
	S1-RP-4-R	Pine	12	12x260 PT	12	12x180 PT	1:1	1	3
S2	S2-DF-5-R	D. fir	24	12x260 PT	36	12x180 PT	1:1.5	1	2

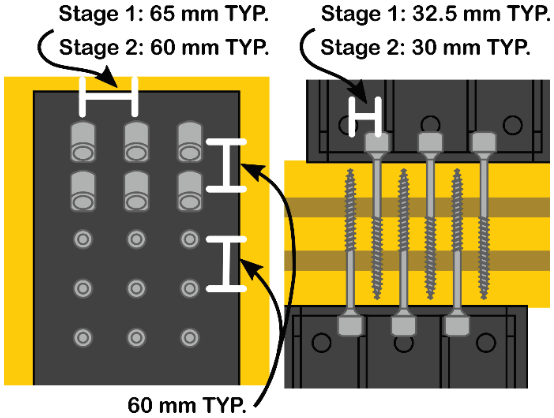


Figure 3: *Typical dimensions for spacings between screws*

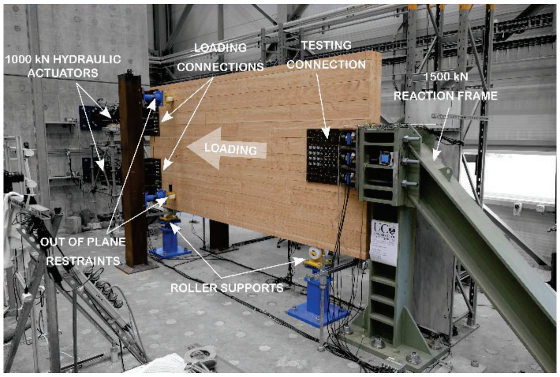


Figure 5: *Annotated photograph of Stage 2 test setup*

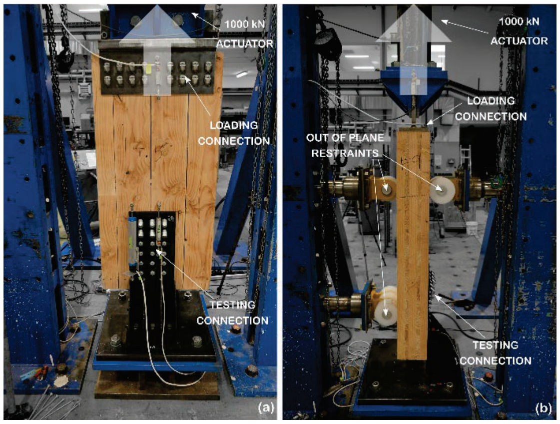
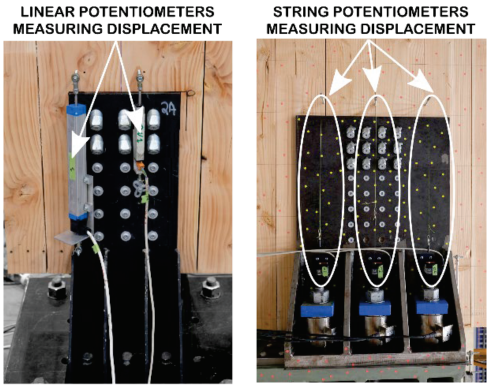


Figure 4: *Annotated photograph of Stage 1 test setup. a) double sided tests b) single sided tests*



a) STAGE 1 a) STAGE 2

Figure 6: *Instrumentation setup for displacement measurements*

2.3 MATERIALS

All CLT material was manufactured by XLAM New Zealand. Lamella were graded to meet NZS3603:1993 SG8 grade (8 GPa mean elastic modulus), with Douglas fir specimens using New Zealand grown Douglas fir, and radiata pine specimens using New Zealand grown radiata pine [9]. For Stage 1 testing Douglas fir specimens were 5-layer 175 mm thick with a 45/20/45/20/45 layup, and for Stage 2 Douglas fir specimens were 5-layer 205 mm thick with a 45/30/45/30/45 layup. For both stages, radiata pine specimens were 205 mm thick with a 45/30/45/30/45 layup. All moisture contents were found to be approximately 11% following the standard [10]. Density was calculated in accordance with the standard [11], with mean and characteristic densities of both the Douglas fir and radiata pine CLT specimens being presented in Table 3.

Inclined screws were SPAX ϕ12x260 mm partially threaded (PT) screws with countersunk heads. 90° screws were either SPAX ϕ10x180 mm PT screws with washer heads, or SPAX ϕ12x180 mm PT screws with countersunk heads. Inclined angles were achieved using 12 mm Rothoblaas VGU inclined washers.

***Table 3** - Mean and characteristic densities of CLT specimens*

Species	Douglas Fir		Radiata pine
Stage	Stage 1	Stage 2	Stages 1 & 2
Mean Density (kg/m^3)	496	472	460
Characteristic Density (kg/m^3)	440	429	417

2.4 LOADING PROTOCOL

The loading protocol used was from ISO 16670 as shown in Figure 7 [12]. The loading protocol is a half cycle protocol loading only in the tension direction, similar to what would be expected in the hold-down connection of a CLT shear wall. For both stages of testing, the targeted loading rate was 0.2 mm/sec.

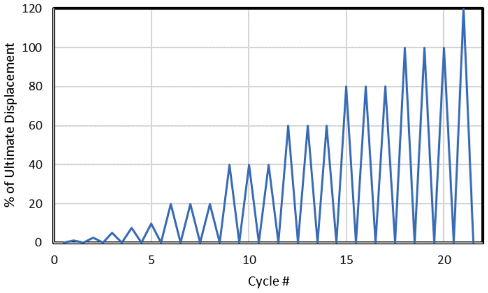


***Figure 7:** Cyclic loading protocol based on ISO16670*

2.5 REPAIR METHODOLOGY

The repair process presented in this paper is made up of two steps. The first step is to enact a repair on the timber that has been damaged by screw withdrawal and embedment crushing. The second step is to install new screws at locations with a small horizontal offset to the original locations such that the screws are installed into fresh timber.

2.5.1 Step 1: Repair holes with epoxy

As shown in Figure 10, the failure modes of mixed angle screw connections are primarily withdrawal for inclined fasteners and withdrawal and embedment crushing for 90° fasteners. These embedment failures leave large vertical holes in the timber with loose and torn fibres around the edges. First the holes were cleaned of loose timber. For inclined screws this was done by drilling out the screw hole with a long series drill bit. This drill bit was sized to suit the Hilti Epoxy injection tube such that epoxy could be injected under a small amount of pressure to force out air and fill all the voids. For 90° screws, the holes were first reamed with a drill bit, then the edges where significant embedment failure had occurred were tidied up by removing loose or damaged timber fibres either with pliers and chisel, or with a handheld electric router. Holes were then vacuumed and blown out with compressed air to clear them of dust. Epoxy was then injected into the holes from the inside out, using the pressure of the epoxy to force the nozzle out of the hole. In this study Hilti HIT RE-500 was used with the standard nozzles and extension tubes used for concrete anchors. Excess epoxy was wiped off, and once cured the area sanded to remove any excess epoxy from the timber surface.

2.5.2 Step 2: Reinstate fasteners with an offset

The second step is to reinstate the connection with new fasteners. Small cold-formed hold-downs such as the Rothoblaas WHT with small nails or screws could be reinstalled beside the original hold-down position. However, the mixed angle screw connections tested have fasteners with larger diameter and hence larger row spacing. Due to the much larger dimensions and capacity of these mixed angle screw hold-downs, the change in level arm required to install beside the damaged connection would significantly impact the strength of the wall. A large shift would also require any anchors into the foundations to be moved which may not be practicable if cast in place. Therefore, a small horizontal shift of half row spacing is used. For the connections tested the shift was 16.25 mm for Stage 1, and 15 mm for Stage 2. This shift places the replacement screws half-way in between the repaired holes from the original fasteners.

This horizontal shift may require new brackets to be constructed or can be achieved using well planned details. In Stage 1 testing, the offset was achieved simply using a large tolerance on the hold-down bolt holes. In Stage 2 it was done by creating two sets of brackets with holes offset by 15 mm. These two sets can be used on either end of a shear wall and swapped post-earthquake when repair is required.

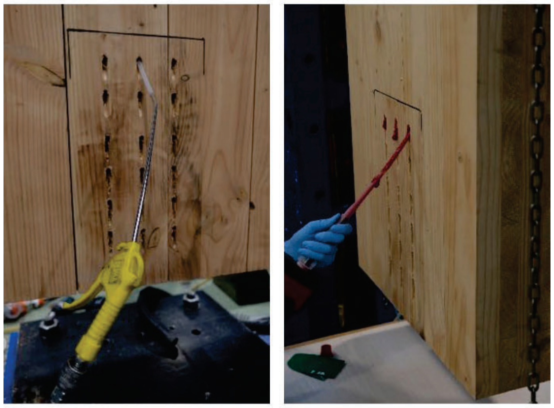
3 RESULTS

Experimental results are shown in Table 4 in terms of key performance parameters including strength, stiffness, ductility, and ultimate displacement. For the definition of yield point and ductility, two approaches are presented.

https://doi.org/10.52202/069179-0268

The first approach is described by EN12512, where the yield point is found at the intersection between the initial stiffness and a second line tangent to the curve at 1/6 the slope of the initial stiffness [13]. Due to the high stiffness of these mixed angle screw connections, this approach seems to significantly underestimate the yield force and displacement estimated by visual inspection. The second approach is the equivalent energy elastic plastic (EEEP) approach which derives an equivalent elastic perfectly plastic curve based on equal energy or area under the force displacement curve [14].


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) Finshed repair ready for new fasteners

Figure 8: Repair process using epoxy

The EEEP method was found to provide a good representation of the yield point compared to what would be visually identified, except in cases where the force increased significantly post-yield. In these cases, the EEEP method was found to slightly overestimate the yield point. Both EN12512 and EEEP results are presented for comparison in Table 4 and Table 5. Due to some initial non-linearity in the elastic portion of the force displacement curve, stiffness values were determined using a similar approach to [15], whereby a regression analysis was undertaken on multiple sections of the initial elastic portion of the force displacement curve.

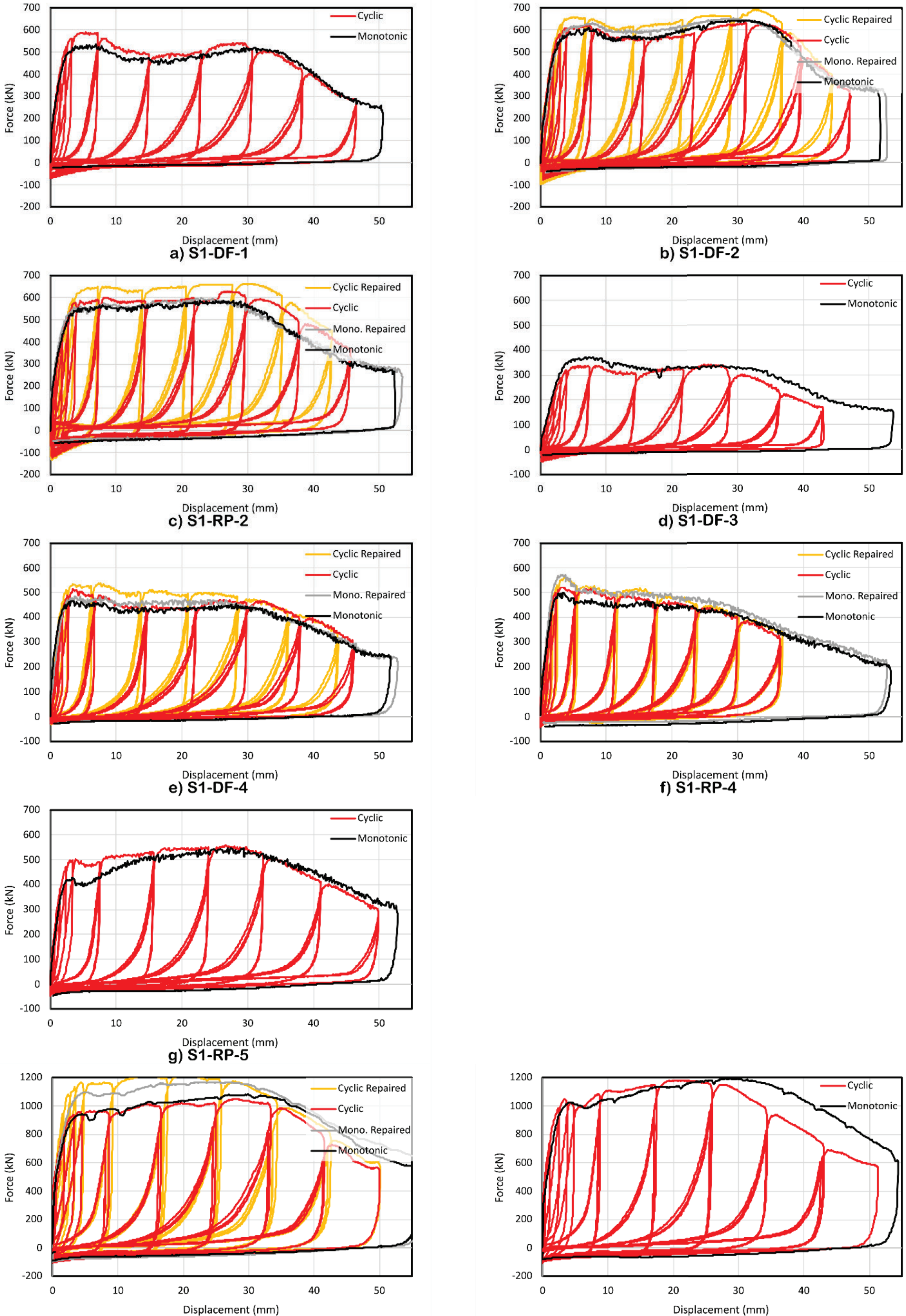
Representative hysteric force-displacement curves of the mixed angle screw connections are shown in Figure 9. These plots overlay the monotonic and a representative cyclic curve from testing. For the connections where repair was undertaken, the corresponding repaired tests are also shown on the same plot.

4 DISCUSSION

4.1 CONNECTION PERFORMANCE

Overall, satisfactory performance of the mixed angle screw hold-down connections was observed with high strength, stiffness, and ductility. Peak loads F_{max} in Table 4 show the mean value was 500 to 650 kN for Stage 1 double sided connections, 350 kN for Stage 1 single sided connections, and 1081 to 1286 kN for Stage 2 double sided connections. Stiffness values varied between 212 and 269 kN/mm for Stage 1 tests and 361 and 370 kN/mm for Stage 2 tests. Despite using the regression approach described above, the stiffness was found to be quite variable and the expected correlation between the connection tests with more 90° fasteners and higher stiffness was not observed. The variability of stiffness might be due to the initial conditions and friction between the steel side plate and the timber surface. Therefore, more research is required to better understand this. It is also noted that no initial slips were observed or corrected for, with the initial non-linearity in the elastic portion of the force displacement curve being of higher stiffness than that reported.

All of the tested connections exhibited high connection ductility. Ductility is commonly defined as the ratio of ultimate displacement to yield displacement [13]. As discussed, the method of determining yield displacement can have a large impact on the value obtained. Therefore, the ductility and yield displacement values from both the EN12512 approach and the EEEP approach are presented. As the ultimate displacement was relatively large, and the yield displacement was small (between 1 and 3 mm), relatively small changes of the yield displacement can make large changes to the ductility value. For example, if yield displacement changes from 1 to 2 mm, the ductility will be the half. Therefore, for the mixed angle screw connections, it is also recommended to consider the ultimate displacement or displacement capacity of the connection as well. A comparison between S1-RP 4 and S1-RP-2 explains why the ultimate displacement should be considered. The mean ductility of S1-RP-4 is 17.6 with the mean ultimate displacement of 31.4 mm while the mean ductility of S1-RP-2 is 13.2 with the mean ultimate



a) S1-DF-1

b) S1-DF-2

c) S1-RP-2

d) S1-DF-3

e) S1-DF-4

f) S1-RP-4

g) S1-RP-5

h) S2-DF-5

i) S2-RP-5

Figure 9: *Representative curves of all tests sets*

https://doi.org/10.52202/069179-0268

Table 4: *Testing results for original tests*

								EN12512			EEEP		
	Test Set	Test #	F_{max} (kN)	F_u (kN)	Δ_{Fmax} (mm)	Δ_u (mm)	K (kN/mm)	F_y (kN)	Δ_y (mm)	μ	F_y (kN)	Δ_y (mm)	μ
Stage 1	S1-DF-1	M1	546	437	6.10	35.8	214	475	1.84	19.5	507	2.37	15.1
		M2	535	428	6.92	38.5	250	448	1.46	26.3	520	2.08	18.5
		C1	614	491	4.38	39.5	284	532	1.55	25.4	601	2.12	18.6
		C2	590	472	5.31	35.4	248	505	1.68	21.1	546	2.20	16.1
		C3	535	428	4.04	36.9	251	459	1.52	24.3	500	1.99	18.6
		Mean	**564**	**451**	**5.35**	**37.2**	**249**	**484**	**1.61**	**23.3**	**535**	**2.15**	**17.4**
	S1-DF-2	M1	643	515	30.0	38.7	233	520	1.91	20.3	615	2.64	14.7
		C1	622	498	35.2	39.8	205	521	2.18	18.3	597	2.91	13.7
		C2	609	487	30.7	39.1	228	507	1.88	20.8	587	2.58	15.1
		C3	633	506	31.2	39.6	236	544	1.93	20.5	621	2.63	15.0
		Mean	**627**	**502**	**31.8**	**39.3**	**226**	**523**	**1.98**	**20.0**	**605**	**2.69**	**14.6**
	S1-RP-2	M1	587	470	25.2	35.3	247	464	1.51	23.3	565	2.29	15.4
		C1	608	487	26.1	36.6	230	484	1.54	23.7	580	2.52	14.5
		C2	628	502	26.8	37.5	204	528	2.17	17.3	594	2.91	12.9
		C3	643	515	26.7	36.8	168	535	2.77	13.3	612	3.64	10.1
		Mean	**617**	**494**	**26.2**	**36.6**	**212**	**503**	**2.00**	**19.4**	**588**	**2.84**	**13.2**
	S1-DF-3	M1	372	297	7.97	35.5	104	333	2.98	11.9	341	3.3	10.8
		M2	314	251	28.0	35.7	97.9	248	2.2	16.2	287	2.93	12.2
		C1	372	298	31.5	39.3	100	319	2.76	14.3	357	3.56	11.1
		C2	342	274	25.5	33.9	85.0	313	3.34	10.2	334	3.93	8.63
		C3	328	262	25.1	34.6	93.9	282	2.63	13.2	315	3.36	10.3
		Mean	**346**	**276**	**23.6**	**35.8**	**96.0**	**299**	**2.78**	**13.2**	**327**	**3.42**	**10.6**
	S1-DF-4	M1	466	373	3.49	38.9	308	390	1.02	38.0	447	1.45	26.8
		C1	515	412	4.72	40.7	208	457	1.92	21.2	492	2.37	17.2
		C2	494	395	4.51	38.1	261	420	1.38	27.6	461	1.77	21.5
		C3	512	410	3.36	36.9	228	455	1.77	20.9	473	2.07	17.8
		Mean	**497**	**398**	**4.02**	**38.7**	**251**	**431**	**1.52**	**26.9**	**468**	**1.92**	**20.8**
	S1-RP-4	M1	500	400	3.41	31.5	240	451	1.59	19.8	456	1.9	16.6
		C1	509	407	3.11	33.4	289	435	1.22	27.4	473	1.64	20.4
		C2	522	418	3.39	27.8	254	468	1.58	17.5	481	1.9	14.6
		C3	540	432	14.9	33.0	291	454	1.37	24.1	510	1.76	18.8
		Mean	**518**	**414**	**6.20**	**31.4**	**269**	**452**	**1.44**	**22.2**	**480**	**1.80**	**17.6**
	S1-RP-5	M1	548	439	27.4	41.8	202	393	1.68	24.9	499	2.46	17.0
		C1	560	448	26.7	38.5	238	444	1.6	24.1	525	2.21	17.4
		C2	599	479	25.8	37.8	238	429	1.46	25.9	548	2.3	16.5
		C3	592	474	26.7	36.9	280	396	1.11	33.3	538	1.92	19.2
		Mean	**575**	**460**	**26.7**	**38.8**	**240**	**416**	**1.46**	**27.1**	**528**	**2.22**	**17.5**
Stage 2	S2-DF-5	M1	1082	865	29.9	42.1	342	828	1.66	25.4	1024	2.99	14.1
		C1	1108	886	28.9	39.6	394	808	1.32	29.9	1039	2.64	15.0
		C2	1052	842	27.7	39.9	385	833	1.28	31.1	1009	2.62	15.2
		Mean	**1081**	**864**	**28.8**	**40.5**	**374**	**823**	**1.42**	**28.8**	**1024**	**2.75**	**14.8**
	S2-RP-5	M1	1194	955	28.7	43.5	309	943	2.16	20.2	1119	3.63	12.0
		C1	1300	1040	24.3	37.8	353	1024	2.18	17.3	1241	3.52	10.7
		C2	1183	946	20.3	34.3	421	922	1.44	23.9	1127	2.68	12.8
		Mean	**1226**	**980**	**24.4**	**38.5**	**361**	**963**	**1.92**	**20.5**	**1162**	**3.27**	**11.9**

https://doi.org/10.52202/069179-0268

Table 5: *Testing results for repaired tests*

								EN12512			EEEP		
	Test Set	Test #	F_{max} (kN)	F_u (kN)	Δ_{Fmax} (mm)	Δ_u (mm)	K (kN/mm)	F_y (kN)	Δ_y (mm)	μ	F_y (kN)	Δ_y (mm)	μ
Stage 1	S1-DF-2-R	M1	653	522	29.4	37.9	207	557	2.41	15.7	625	3.02	12.6
		C1	658	527	6.06	38.4	223	562	2.17	17.7	632	2.83	13.6
		C2	621	497	7.13	36.9	195	561	2.69	13.7	610	3.13	11.8
		C3	692	554	32.6	39.5	233	589	2.1	18.8	660	2.84	13.9
		Mean	**656**	**525**	**18.8**	**38.2**	**215**	**567**	**2.34**	**16.5**	**632**	**2.96**	**13.0**
	S1-RP-2-R	M1	601	481	22.8	34.6	264	470	1.41	24.6	571	2.16	16.0
		C1	630	504	22.1	33.8	250	532	1.78	18.9	611	2.45	13.8
		C2	664	531	29.9	38.8	195	577	2.77	14	643	3.29	11.8
		C3	650	520	24.1	36.8	182	577	2.91	12.7	633	3.47	10.6
		Mean	**636**	**509**	**24.7**	**36.0**	**223**	**539**	**2.22**	**17.6**	**615**	**2.84**	**13.1**
	S1-DF-4-R	M1	484	387	4.46	36.7	226	437	1.72	21.4	464	2.06	17.9
		C1	521	417	8.35	40.3	211	443	1.76	22.8	499	2.37	17.0
		C2	507	405	4.49	36.7	276	428	1.33	27.6	482	1.75	21.0
		C3	542	433	7.64	34.8	263	474	1.54	22.6	504	1.92	18.2
		Mean	**514**	**411**	**6.24**	**37.1**	**244**	**446**	**1.59**	**23.6**	**487**	**2.03**	**18.5**
	S1-RP-4-R	M1	575	460	3.05	27.5	288	514	1.59	17.3	508	1.76	15.6
		C1	526	421	3.09	31.7	291	453	1.31	24.2	498	1.71	18.6
		C2	561	448	3.63	23.7	241	501	1.78	13.4	515	2.14	11.1
		C3	536	429	3.28	33.7	278	476	1.5	22.5	514	1.85	18.2
		Mean	**550**	**440**	**3.26**	**29.2**	**275**	**486**	**1.55**	**19.4**	**509**	**1.87**	**15.9**
Stage 2	S2-DF-5-R	M1	1170	936	24.4	40.6	370	951	1.81	22.4	1126	3.04	13.4
		C1	1199	960	20.5	36.8	470	891	1.21	30.4	1122	2.39	15.4
		C2	1233	987	20.4	36.0	466	1017	1.45	24.9	1172	2.52	14.3
		Mean	**1201**	**961**	**21.8**	**37.8**	**435**	**953**	**1.49**	**25.9**	**1140**	**2.65**	**14.4**

displacement of 36.6 mm. Due to its lower yield displacements, S1-RP-4 turned out to have a higher ductility value even though it had 5 mm less ultimate displacement.

4.2 CONNECTION DAMAGE/FAILURE MODES

As mentioned in Section 2, all the screws used in this study were partially threaded. The inclined screws were designed to fail in the withdrawal mode as the threaded length was controlled to avoid the tensile failure of the screw shank. It is critical to avoid tensile failure of the screws in the mixed angle screw connections as it allows the gradual transfer of load from inclined screws to 90° screws when the deformation increases.

The 90° screws were loaded laterally and caused localised embedment failure. At large displacements they began to withdraw from the timber. In Figure 10a, the elongated holes were caused by embedment crushing, and ***Figure 10***b shows a section cut through the fastener holes, clearly showing the embedment crushing and the fastener plastic hinge location. The hinges formed in the screws are shown in Figure 11. The 90° screws form two hinges, one right under the steel plate, the other at a depth into the timber. The inclined fasteners, as shown in Figure 11 can also form hinges as they undergo bending actions at high displacements as well, but to a lesser extent.

Throughout the testing, no block tear out or other brittle failure modes were observed.

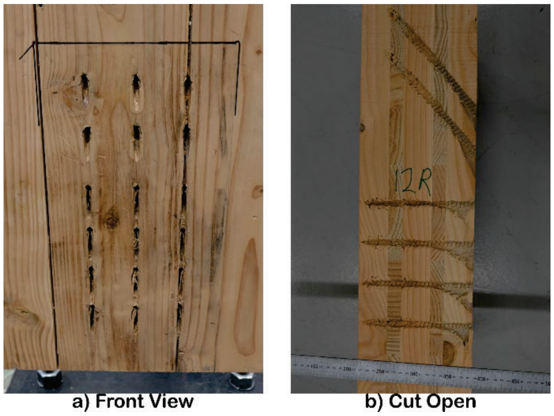

a) Front View **b) Cut Open**

Figure 10: *S1-DF-2 connection with damage*

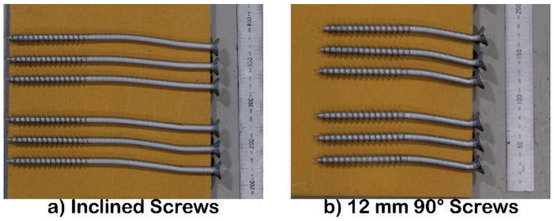

a) Inclined Screws **b) 12 mm 90° Screws**

Figure 11: *Pictures of screws after testing the plastic hinges formed*

4.3 OPTIMAL RATIO BETWEEN INCLINED AND 90° SCREWS

For the mixed angle screw connections tested, all the configurations showed high strength, stiffness and ductility. The designer can choose the ratio of inclined to 90° screws based on the performance required. In general, a connection with more 90° screws will have a higher post yield force increase (or post yield stiffness) due to significant rope effect of the 90° screws at high displacements.

To determine a recommended ratio, further criteria would need to be defined. Efficiency, post yield force increase, and energy dissipation are all compared at length in the literature [16]. The conclusions of [16] can be replicated by visual inspection of the force displacement plots in Figure 9. In general, a connection that can sustain the yield force till a large displacement is desired. Literature also suggested that an increase in force post yield is desirable in terms of limiting drift [17]. From Figure 9 it can be determined the connections using Configuration 2 and Configuration 5 meet these criteria better. Therefore, the optimal ratio of ϕ12 mm inclined screws to ϕ10 mm 90° screws was 1:2; and it was 1:1.5 for ϕ12 mm inclined screws and ϕ12 mm 90°.

4.4 FEASIBILITY OF POST-EARTHQUAKE REPAIR

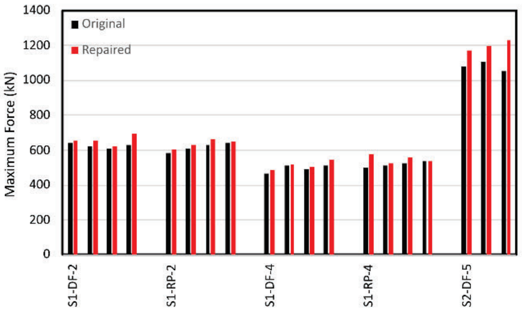
A key observation from this study was the localised and confined damage to the CLT in comparison to extensive damage in the dowelled CLT hold-down connections tested in a previous study [18]. With the localised damage, post-earthquake repair of these connections becomes viable, and tests were undertaken to evaluate the performance of the repair by comparing original and repaired tests of the same experimental specimen.

To evaluate the effectiveness of the repair, strength, stiffness, and ultimate displacement are compared in Figure 12. Figure 12a shows a comparison of maximum force for each original and repaired test. From Figure 12 it can be seen that all repaired tests reached a higher maximum force than the original tests. This increase averaged 5.5% over all tests with a maximum of 15%.

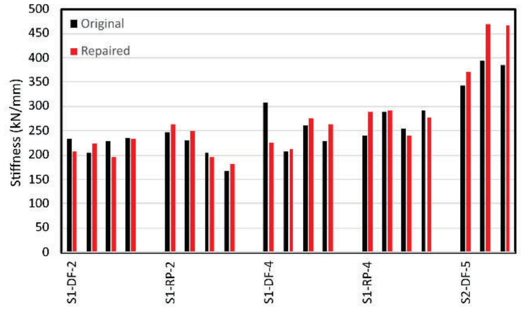
Figure 12b shows a comparison of stiffness, with the original and repaired stiffnesses being similar for most tests.

Figure 12c shows a comparison of ultimate displacement, with repaired tests having lower ultimate displacement for all but two results. This may be in part due to the definition of ultimate displacement being dependent on the maximum force which was higher for the repaired tests. Given the increase in strength, and similar stiffness values, the epoxy repair and shift method is deemed to be effective in repairing this type of connection post-earthquake.

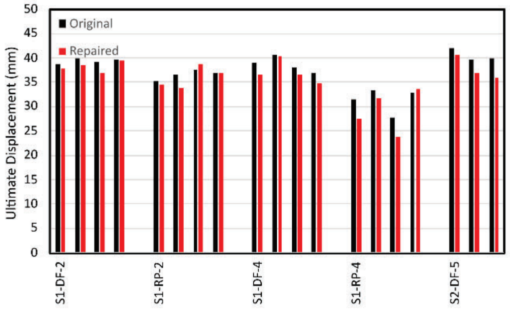
Figure 13 shows the epoxy repair of the connection after the repaired test. Overall the epoxy has filled the damaged holes well, with no large trapped pockets of air. There are some small air bubbles in the epoxy, but given the difference in strength between the epoxy and the surrounding timber material, these are unlikely to cause adverse effects.



a) Maximum Force

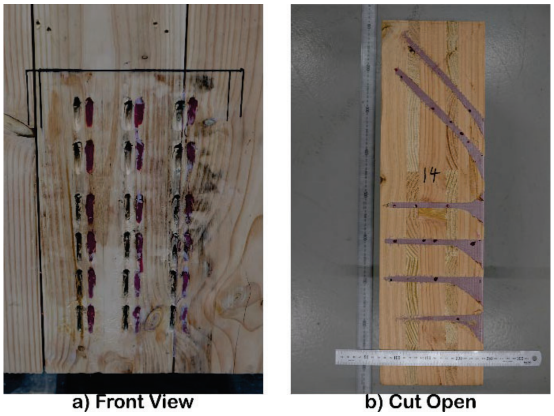


b) Stiffness



c) Ultimate Displacement

Figure 12: *Bar graphs showing pairwise comparisons of maximum force, initial stiffness, and ultimate displacement between original and repaired tests of the same specimen*



a) Front View b) Cut Open

Figure 13: *Picture of repaired S1-DF-2-R connection post testing a) View from front of connection after steel bracket is removed b) Section cut through the epoxy repair*

5 CONCLUSIONS

The paper presents an experimental study on large-capacity CLT hold-downs using mixed angle self-tapping screws. Monotonic and cyclic tests were undertaken to assess the hold-down performance. The key conclusions are:

- Mixed angle screws can provide a strong, stiff, and ductile hold-down connection system for CLT shear walls;
- The optimal ratio of inclined screws to 90° screws was 1:2 for ϕ12 mm inclined and ϕ10 mm 90° screws, and 1:1.5 for ϕ12 mm inclined and ϕ12 mm 90° screws. These ratios led to connections with high strength, high stiffness, and high ductility;
- Under cyclic loading, mixed angle screws can be detailed such that the primary failure modes are screw withdrawal and wood embedment crushing. This leaves only localised damage and results in repairable damage in timber; and
- Localised damage observed in mixed angle screw hold-down connections can be repaired with epoxy and the screws reinstated with a small offset to original fastener holes leading to the same or even slightly better performance.

REFERENCES

[1] H. J. Blaß and I. Bejtka, “Screws with continuous threads in timber connections,” in *In: Joints in timber structures. Proceedings of the International RILEM Symposium, Stuttgart, Germany 2001. Ed.: S. Aicher*, 2001, vol. 22, pp. 193–201.

[2] Bejtka and Blaß, “Joints with inclined screws,” in *Working Commission W18 - Timber Structures*, Kyoto, Japan, 2002, vol. 35-7–4, p. 141.

[3] Kevarinmaki, “Joints with inclined screws,” in *Working Commission W18 - Timber Structures*, Kyoto, Japan, 2002, vol. 35-7–4.

[4] R. Tomasi, M. Piazza, A. Angeli, and M. Mores, “A new ductile approach design of joints assembled with screw connectors,” in *WCTE 2006*, Portland, USA, 2006.

[5] A. Hossain, M. Popovski, and T. Tannert, “Cross-laminated timber connections assembled with a combination of screws in withdrawal and screws in shear,” *Eng. Struct.*, vol. 168, pp. 1–11, Aug. 2018, doi: 10.1016/j.engstruct.2018.04.052.

[6] J. R. Brown, M. Li, T. Tannert, and D. Moroder, “Experimental study on orthogonal joints in cross-laminated timber with self-tapping screws installed with mixed angles,” *Eng. Struct.*, p. 111560, Dec. 2020, doi: 10.1016/j.engstruct.2020.111560.

[7] A. Ceccotti and M. Follesa, “Seismic Behaviour of Multi-Storey XLam Buildings,” p. 15, 2006.

[8] A. Ceccotti, C. Sandhaas, M. Okabe, M. Yasumura, C. Minowa, and N. Kawai, “SOFIE project - 3D shaking table test on a seven-storey full-scale cross-laminated timber building: 3D SHAKING TABLE TEST ON A SEVEN-STOREY FULL-SCALE X-LAM BUILDING,” *Earthq. Eng. Struct. Dyn.*, vol. 42, no. 13, pp. 2003–2021, Oct. 2013, doi: 10.1002/eqe.2309.

[9] Standards New Zealand, “NZS 3603:1993 Timber Structures Standard,” Standards New Zealand, 1993.

[10] Standards New Zealand, *Timber: Moisture content. methods of test*, Second 2012. Sydney, NSW;Wellington; Jointly published by Standards Australia Limited Limited, 2012.

[11] Standards New Zealand, “Timber: methods of test. Method 3, Density.” Standards New Zealand, 2000.

[12] International Organization for Standardization, “Timber structures - Joints made with mechanical fasteners - Quasi-static reversed-cyclic test method (ISO 16670:2003).” 2003.

[13] British Standards Institution, “BS EN 12512:2001: Timber structures. Test methods. Cyclic testing of joints made with mechanical fasteners.” British Standards Institute, 2001.

[14] ASTM, “E2126 − 11: Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings,” ASTM International, 2011. doi: 10.1520/E2126-11.

[15] R. Brandner, “Cross laminated timber (CLT) in compression perpendicular to plane: Testing, properties, design and recommendations for harmonizing design provisions for structural timber products,” *Eng. Struct.*, vol. 171, pp. 944–960, Sep. 2018, doi: 10.1016/j.engstruct.2018.02.076.

[16] Wright, M. Li, D. Moroder, and D. Carradine, “DRAFT: Cyclic Behaviour of CLT Shear Wall Hold-Down Connections Using Mixed Angle Self-Tapping Screws,” *Eng. Struct.*, vol. (Under Review), 2023.

[17] G. A. MACRAE and K. KAWASHIMA, “POST-EARTHQUAKE RESIDUAL DISPLACEMENTS OF BILINEAR OSCILLATORS,” *Earthq. Eng. Struct. Dyn.*, vol. 26, no. 7, pp. 701–716, 1997, doi: https://doi.org/10.1002/(SICI)1096-9845(199707)26:7<701::AID-EQE671>3.0.CO;2-I.

[18] L.-M. Ottenhaus, M. Li, and T. Smith, “Structural performance of large-scale dowelled CLT connections under monotonic and cyclic loading,” *Eng. Struct.*, vol. 176, pp. 41–48, Dec. 2018, doi: 10.1016/j.engstruct.2018.09.002.