Project Report

Experimental and Parametric Studies on Douglas-fir CLT Shear Walls with High Capacity Connections

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EXECUTIVE SUMMARY

As an engineered timber product, cross-laminated timber (CLT) is currently gaining popularity around the world including New Zealand and Australia. Compared with Radiata pine, Douglas-fir is underutilised in the New Zealand construction market. However, Douglas-fir has a great potential to be manufactured into high value added CLT products for mass timber construction. To help engineers specify Douglas-fir CLT in building construction, a series of research projects funded by SWP research partnership are conducted to establish critical engineering design properties of Douglas-fir CLT and develop high performance Douglas-fir CLT connections and structural systems for resilient seismic design. This two-year project aims to develop multi-storey Douglas-fir CLT shear wall structures that have high strength and stiffness and are suitable for high seismic countries like New Zealand. This report mainly presents preliminary results of two connection types that are critical for the Douglas-fir CLT shear walls. The first connection type is the steel-CLT connections to connect steel link beams to CLT wall panels in a coupled CLT shear wall system. The second connection type is the screwed hold-down connections that can provide wall anchoring under earthquake loads.

The experimental results demonstrated excellent connection behaviour and the research findings have been used to optimize the connection details in the Douglas-fir CLT wall tests in the following project phase.

1. INTRODUCTION

Cross-laminated timber (CLT) is becoming more and more popular for mass timber construction around the world including New Zealand and Australia. For CLT buildings built in high seismic countries like New Zealand, shear walls are the critical elements to resist earthquake loads. Often due to the lack of high performance connections, the inherent high strength and stiffness of CLT products cannot be fully utilised and the structural efficiency of CLT shear walls is thus compromised. Therefore, high-capacity connections are critical components to achieve structurally efficient CLT shear walls with great seismic performance.

Due to the difficulties of sourcing Douglas-fir CLT in New Zealand, this project has been delayed for six months and this annual report mainly covers Phase 1 experimental testing to evaluate the structural performance of two types of critical connections in the high-capacity Douglas-fir CLT shear walls. The first type is the steel-CLT interface connections to connect steel link beams to CLT wall panels to form a coupled CLT shear wall system with high strength, high stiffness and high energy dissipation capacity. The second connection type is the screwed hold-down connections that aim to provide strong wall anchoring and prevent wall uplifting under earthquake loads. Meanwhile, reparability of these screwed hold-down connections was assessed by comparing the performance of the original and the repaired connections. Lastly, the CLT wall test plan and the large-scale connection test plan will be briefly introduced.

It should be noted that the connection specimens in Phase 1 testing had a mixed use of Radiata pine CLT and Douglas-fir CLT due to the shortage of Douglas-fir CLT materials. As the Radiata pine laminations and the Douglas-fir laminations had the same grade, it is believed that the research findings from the Radiata pine CLT connection specimens can be extrapolated and provide useful information for Phase 2 experimental testing on the Douglas-fir CLT walls.

2. STEEL-CLT CONNECTIONS

This section presents a part of the experimental results of the steel-CLT connections. A coupled CLT shear wall consisting of CLT wall panels and steel link beams (Figure 1) is structurally more efficient than a single shear wall. In such a system, the connections between the steel link beams and the CLT panels are the critical components to provide the coupling effect. Following capacity design approach, these steel-CLT connections need to be strong and stiff enough to transfer the forces between the CLT wall panels and facilitate the yielding of the steel beams.

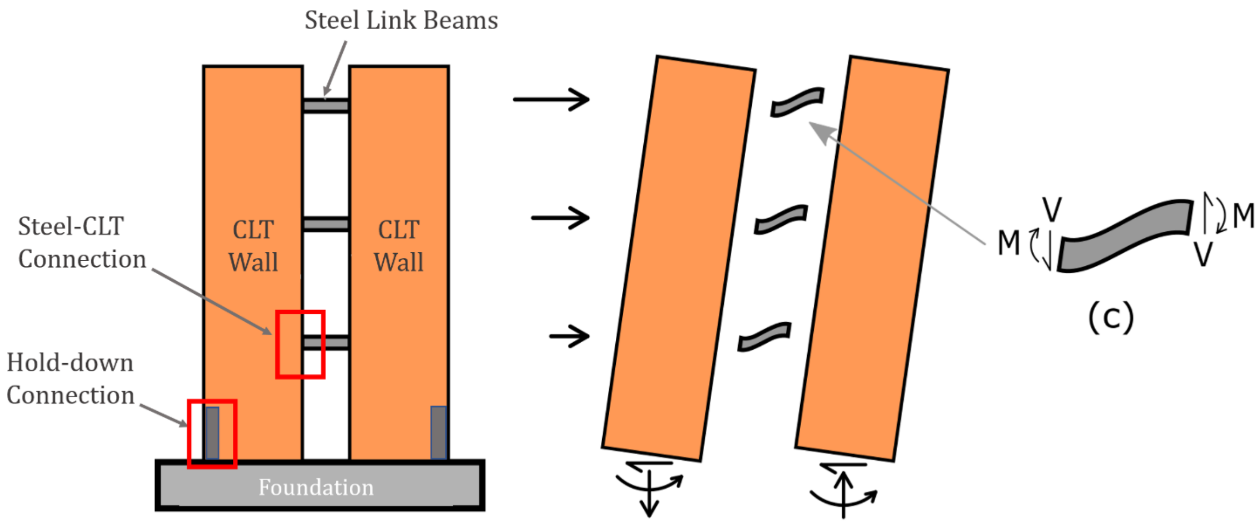


Figure 1 A coupled CLT shear wall with steel link beams under earthquake loads.

2.1 Steel-CLT Connection Test matrix

Table 1 lists the test matrix for the three types of connections as well as the number of replicates. Three types of connection details for the steel-CLT connections were tested: 1) self-drilling dowels with an inserted steel plate (DP specimens); 2) threaded rods with notches (RN); and 3) fully threaded self-tapping screws with a notch (SN specimens).

Table 1 – Test matrix for steel-CLT connections

Connection Type	Connection Description	No. of Specimens
DP	Steel knife plate inserted in mid-thickness of CLT and fastened with 50 – 7.5mm self-drilling dowels.	3
RN	End of steel beam placed in notched CLT edge, grouted, and fastened with 8-M24 threaded rods.	7
SN	End of steel beam placed in notched CLT edge, grouted, and fastened with a group of fully threaded self-tapping screws.	15

Figure 2 to Figure 4 show the test photos of three connection types in which the loads were applied to the mid-span of the steel beams so that the steel-CLT connections were subjected to shear loads and bending moment to resemble the actual loading scenarios in a coupled CLT shear wall under seismic loads, as indicated in Figure 1.

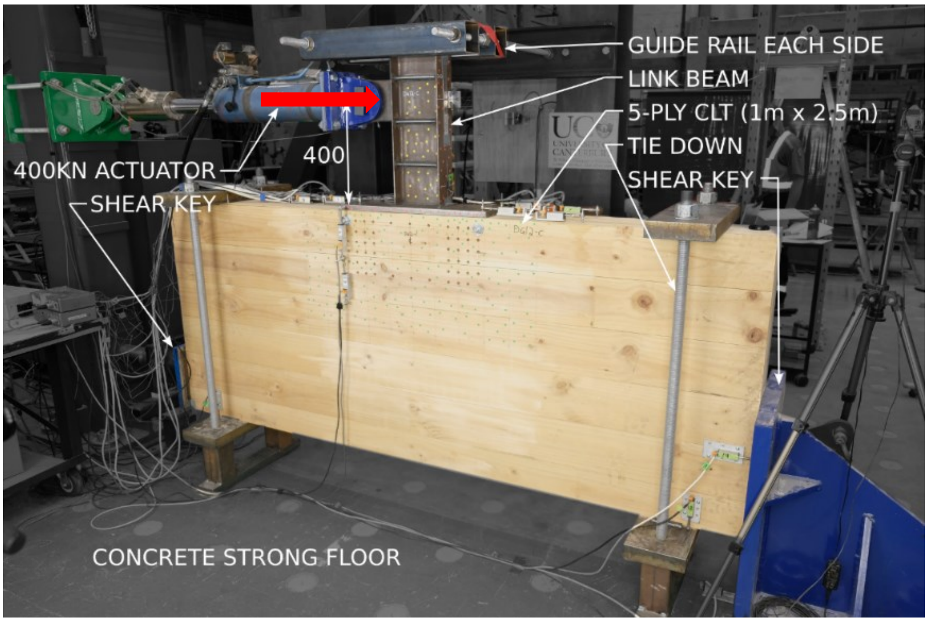


Figure 2 Test photo for the DP connections

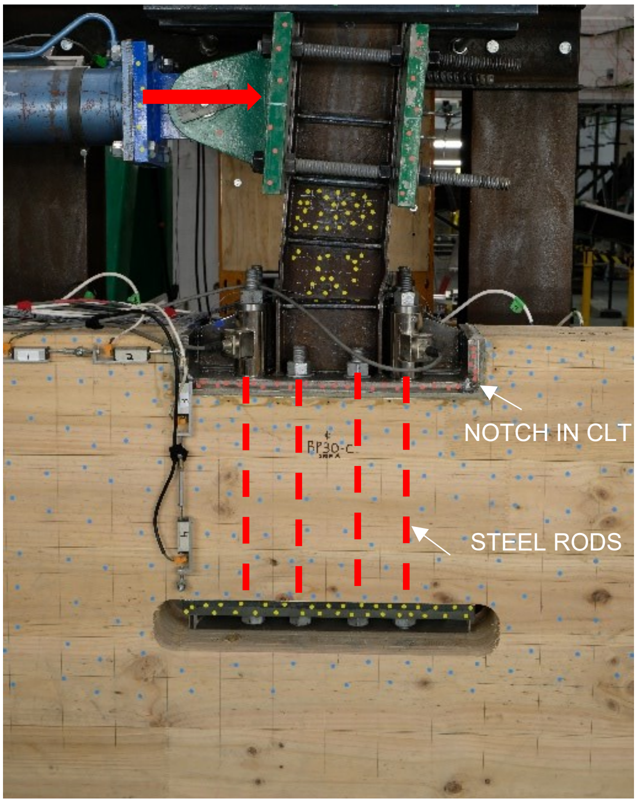


Figure 3 Test photo for a RN connection

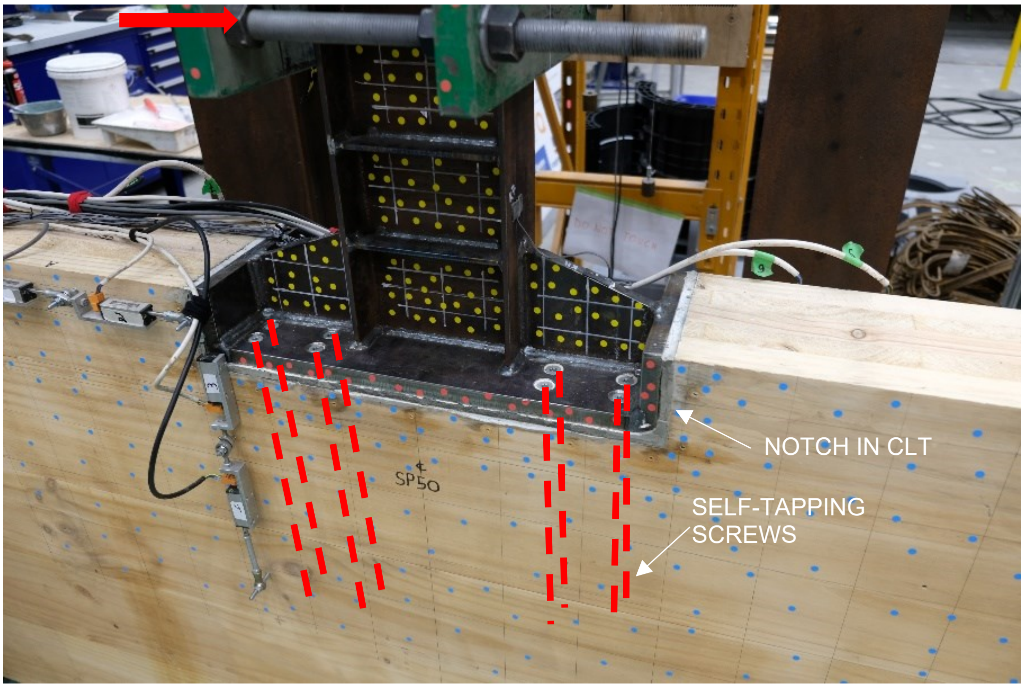


Figure 4 Test setup for a SN connection

2.2 Test Results

These steel-CLT connections are designed to transfer loads between the CLT shear walls and protect CLT panels from damage in an earthquake event. Therefore, the steel beams were intentionally designed as the weak links to allow them to yield and dissipate energy. Figure 5 shows the typical failure mode of the specimens in which the lower portion of the steel beam was extensively damaged and the flanges and web were yielded and buckled. However, the CLT panel and the connection were well protected without any significant damage. Figure 6 to Figure 8 show the typical load-displacement curves of three connection types, indicating the high energy dissipation by yielding of the steel beams.

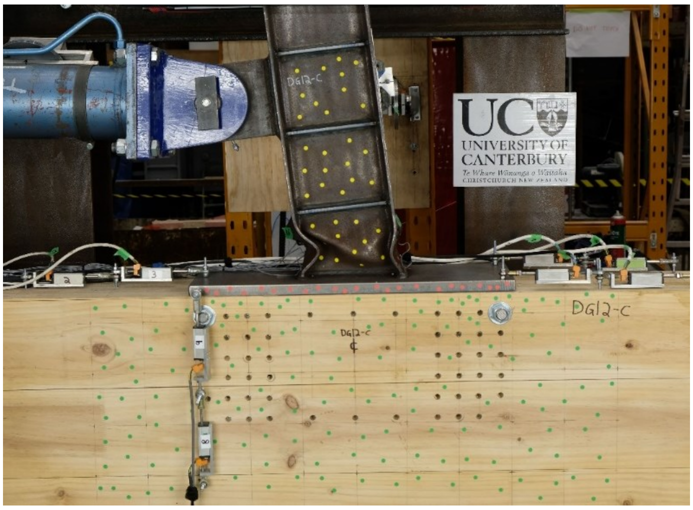


Figure 5 Typical steel beam yielding failure mode with flange and web buckling

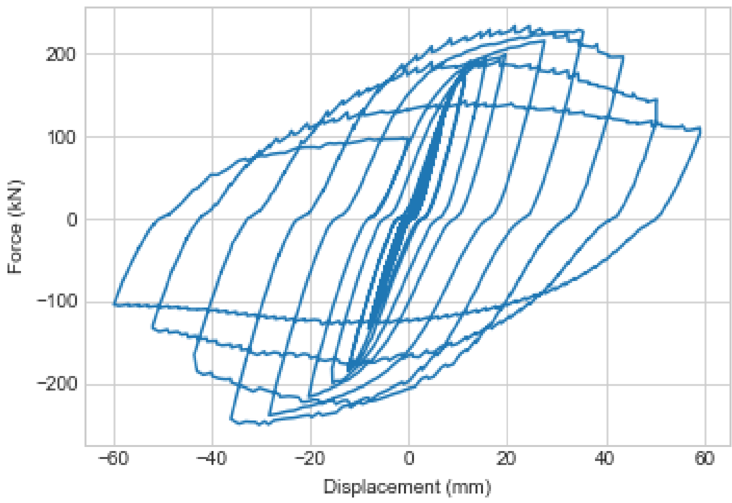


Figure 6 Typical force-displacement curves of a DP specimen

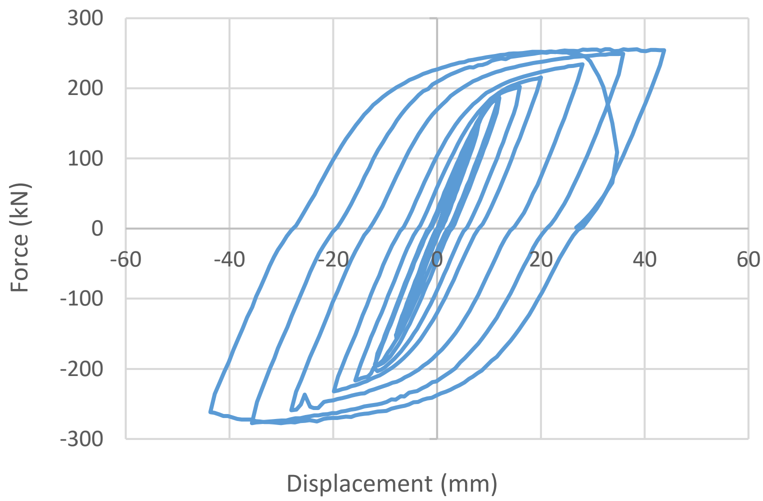


Figure 7 Typical force-displacement curves of a RN specimen

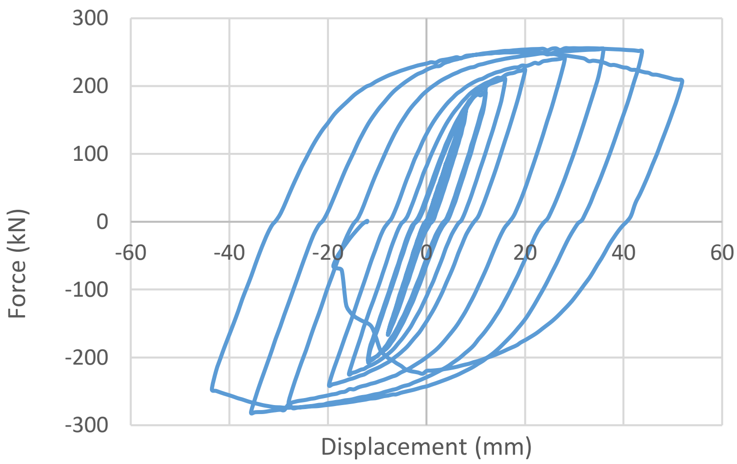


Figure 8 Typical force-displacement curves of a SN specimen

A portion of the experimental results on the DP connections has been published at the 2021 New Zealand Society for Earthquake Engineering Annual Conference (Moerman et al., 2021). The processing and analysis of all experimental data is still ongoing and the relevant design methods are being developed for design engineers. These results will be included in the final project report.

3. HIGH-CAPACITY HOLD DOWN CONNECTIONS WITH SCREWS

This section presents the progress on the development of mixed angle screw hold-downs connections for Douglas-fir CLT. The overall experimental testing consists of three key stages: 1) the first stage conducted small scale testing of mixed angle screw hold-downs to study the performance of screws installed at an inclined angle of 45 degree only, 90 degree only, and a combination of the two. The results have been presented in the previous annual report; 2) the second stage conducted medium scale testing of mixed angle screws to study the influence of the different ratios of inclined and 90 degree screws on the connection performance. The results have been partially presented in the previous annual report. In this report, additional test results of the performance of connections using countersunk head fasteners, and the repair of mixed angle will be presented; and 3) the third is the large scale connection testing to be completed from July to Dec 2021. The last stage will investigate the performance of high-capacity (~1000kN) mixed angle screw hold-downs, and the performance of mixed angle screw hold-downs in the CLT wall testing.

3.1 Medium-scale Screwed Hold-down Testing

The previous testing on the mixed angle screw connections has been limited to using 12 mm diameter inclined screws and 10 mm diameter washer head 90 degree screws. To broaden the applicability of these connections, additional testing has been undertaken using Φ12 mm inclined screws and Φ12 mm counter-sunk 90 degree screws. Figure 9 shows the test setup and the schematic of mixed angle screw installations in a hold-down connection specimen.

As an example, Figure 10 and Figure 11 show the force-displacement curves of the connection using 12 Φ12 mm 90 degree screws and a connection using 24 Φ10 mm 90 degree screws, respectively. From these plots it can be seen that the Φ10 mm screws provide a greater maximum force (expected given the 12 extra fasteners required). It can also be seen that a high level of ductility and displacement capacity can be achieved using Φ12 mm 90 degree screws with significantly less fasteners (12 vs 24 for 90 degree fasteners). Tabulated results for both Φ12 mm 90 degree screws and Φ10 mm 90 degree screws are shown in Table 2 and Table 3 respectively.

Some experimental results have also been published at the 2021 New Zealand Society for Earthquake Engineering Annual Conference (Wright et al., 2021).

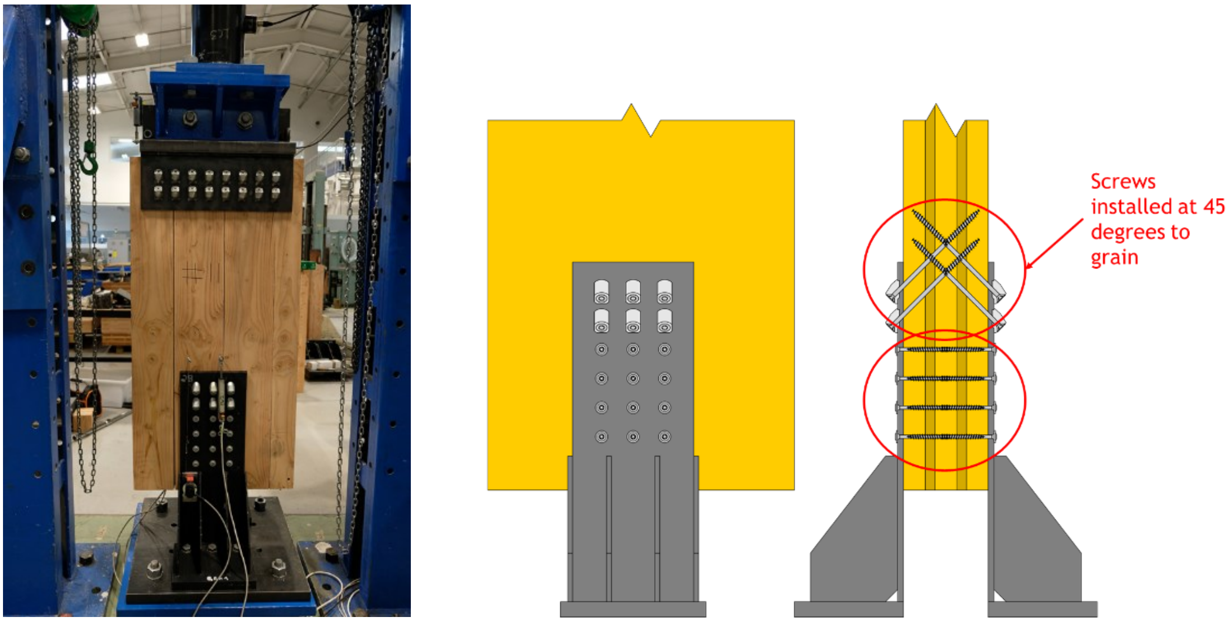


Figure 9 Test setup of Douglas-fir CLT hold-downs with mixed angle screws

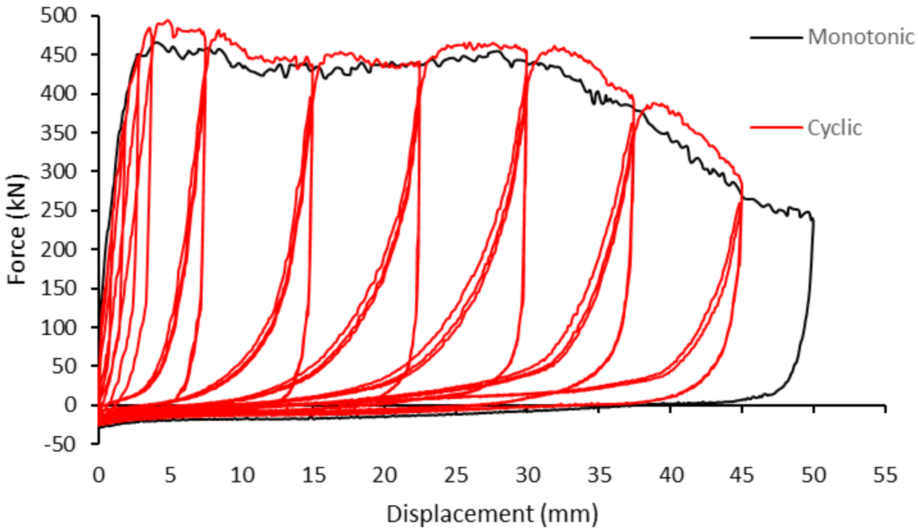


Figure 10 Plot of test results for a connection with 12 inclined screws and 12 Φ12x180 mm 90 degree screws with countersunk head

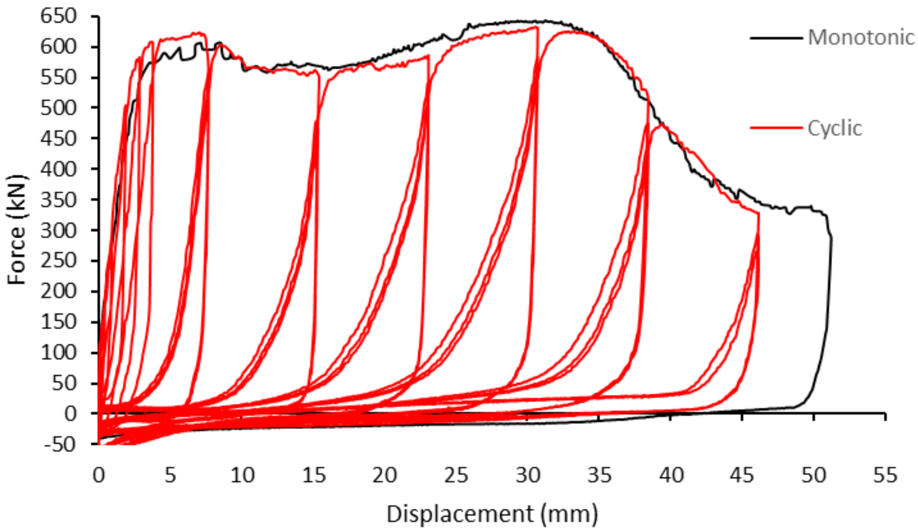


Figure 11 Load-displacement curves for a connection with 12 inclined screws and 24 Φ10x180 mm 90 degree screws with washer head

Table 2 - Results summary for tests of a connection with 12 inclined screws and 12 Φ12x180 mm countersunk head 90 degree screws

Test	Yield Force (kN)	Max Force (kN)	Ultimate Force (kN)	Yield Displacement (mm)	Ultimate Displacement (mm)	Stiffness (kN/mm	Ductility
Monotonic	394	466	373	1.31	38	238	29
Cyclic 1	497	515	412	2.65	38.8	151	14.6
Cyclic 2	424	494	395	1.83	37.3	212	20.4
Cyclic 3	472	512	410	2.85	36.5	156	12.8
Cyclic Mean	464	507	406	2.44	37.5	173	15.9

Table 3 – Results summary for tests of a connection with 12 inclined screws and 24 Φ10x180 mm washer head 90 degree screws

Test	Yield Force (kN)	Max Force (kN)	Ultimate Force (kN)	Yield Displacement (mm)	Ultimate Displacement (mm)	Stiffness (kN/mm	Ductility
Monotonic	533	643	515	2.28	38.1	189	16.7
Cyclic 1	536	622	498	2.43	38.7	180	15.9
Cyclic 2	502	609	487	2.06	38.5	204	18.7
Cyclic 3	563	633	506	2.25	38.4	208	17.1
Cyclic Mean	533	621	497	2.25	38.6	197	17.2

3.2 Repaired Connection Testing

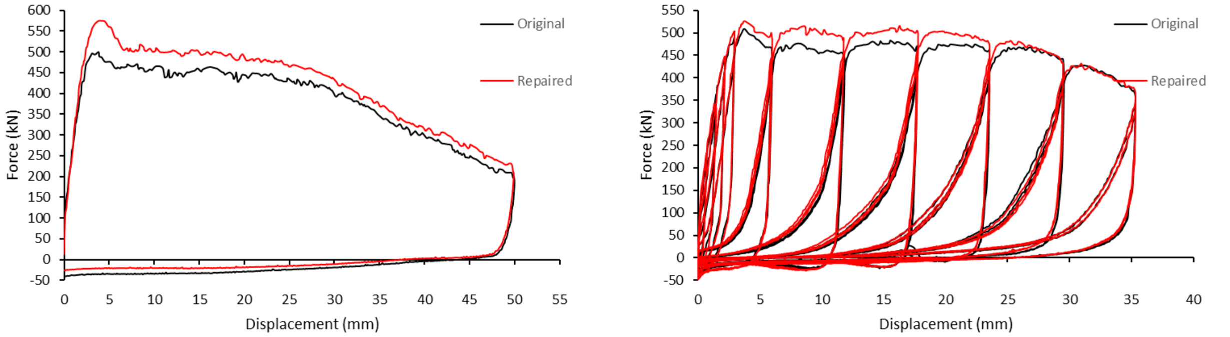
The reparability of damaged mixed angle screw connections was briefly mentioned in the previous annual report. Since then, an additional 16 tests have been undertaken on the repair of damaged connections. Two repair typologies have been investigated. The first involves cleaning up the damaged holes from previously installed fasteners, filling these holes with epoxy, shifting the connection over by half screw spacing, and then inserting new fasteners. The second simply shifts the connection over by half screw spacing. These repair methodologies are shown in Figure 12a and 12b respectively.

Figure 13 and Figure 14 show the force-displacement curves for both monotonic and cyclic tests utilising the epoxy + shift and shift only repair typology respectively. From Figure 13 it can be seen that the epoxy + shift repair typology slightly increased the strength of the repaired connection when compared to the original. From Figure 14 it can be seen that the shift only typology led to reduced connection strength. It is worth noting that this reduction is quite low at low displacements where the primary load resisting mechanism is axial withdrawal of the inclined fasteners and the strength reduction gets larger as the displacement increases and the load resisting mechanism switches to 90 degree fasteners in shear.

These findings are confirmed by the tabulated results shown in Table 4 and Table 5. Comparing the maximum force observed in both the original and repaired tests using the epoxy + shift typology it can be seen that for 15 of the 16 tests presented the maximum force obtained increased between original and repaired tests with an average increase of 4.4%. For the shift only typology, it can be seen that all 9 of the presented tests show a decrease in maximum force between original and repaired tests with an average decrease of 13.0%.

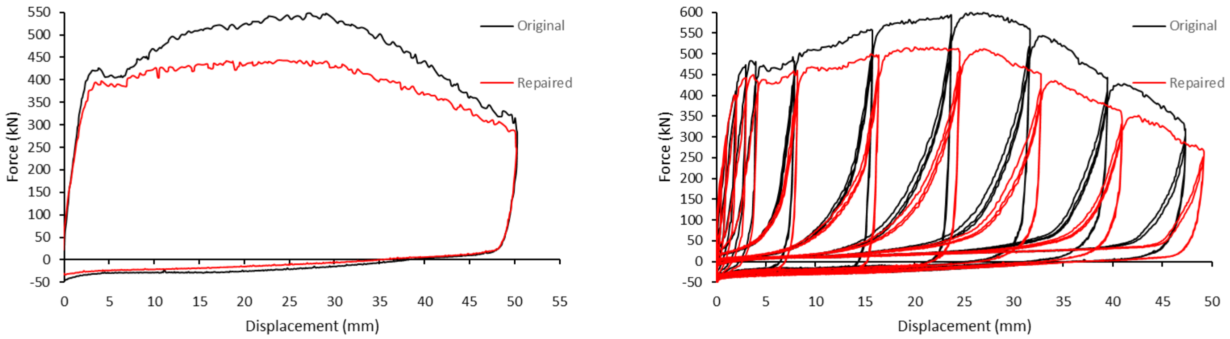
a – Epoxy + shift *b – Shift only*

Figure 12 Connection areas after two tests (original testing and additional testing after repair)



a – Monotonic loading *b – Cyclic loading*

Figure 13 Plots of test results for connections repaired with the epoxy + shift typology



a – Monotonic loading *b – Cyclic loading*

Figure 14 Plots of test results for connections repaired with the shift only typology

Table 4 - Comparison of maximum force for original tests and repaired tests on the same specimen using the shift only repair typology

Test Set	# Inclined	# 90 Degree	90 Degree Type	Loading	Original F_{max} (kN)	Repaired F_{max} (kN)	% Diff
1	12	24	Φ10x180 WH	Monotonic	643	653	1.56
				Cyclic 1	622	658	5.79
				Cyclic 2	609	621	1.97
				Cyclic 3	633	692	9.32
2	12	12	Φ12x180 CSK	Monotonic	466	484	3.86
				Cyclic 1	515	521	1.17
				Cyclic 2	494	507	2.63
				Cyclic 3	512	542	5.86
3	12	24	Φ10x180 WH	Monotonic	587	601	2.39
				Cyclic 1	608	630	3.62
				Cyclic 2	628	664	5.73
				Cyclic 3	643	650	1.09
4	12	12	Φ12x180 CSK	Monotonic	500	575	15.00
				Cyclic 1	509	526	3.34
				Cyclic 2	522	561	7.47
				Cyclic 3	540	536	-0.74

Table 5 – Comparison of maximum force for original tests and repaired tests on the same specimen using the epoxy + shift repair typology

Test Set	# Inclined	# 90 Degree	90 Degree Type	Loading	Original F_{max} (kN)	Repaired F_{max} (kN)	% Diff
1	12	18	Φ10x180 WH	Monotonic 1	546	509	-6.78
				Monotonic 2	535	489	-8.60
				Cyclic 1	614	542	-11.73
				Cyclic 2	590	516	-12.54
				Cyclic 3	535	447	-16.45
2	12	18	Φ12x180 CSK	Monotonic	548	444	-18.98
				Cyclic 1	560	486	-13.21
				Cyclic 2	599	517	-13.69
				Cyclic 3	592	501	-15.37

4. UPCOMING DOUGLAS-FIR CLT WALL AND CONNECTION TESTING

4.1 CLT Wall Testing Plan

Large-scale CLT wall testing is the primary focus of this research project and will begin in late June 2021. Design and analysis of these experiments have been the focus of work completed over the past 6 months.

Table 6 list the test matrix of all Douglas-fir CLT shear walls. The first tests (W1x) include 1-storey CLT walls with varying high capacity hold-down connections. The second set of tests (W2x) include 3-storey CLT walls at 2/3-scale which will investigate walls with half-lap spline joints between two panels. Finally, as shown in Figure 15, the third set of tests (W3x) will test a hybrid coupled timber wall system using the beam-to-wall connections tested previously.

Table 6 – Test matrix for planned CLT wall experiments.

Test ID(s)	**Description**	**Approximate Test Date**
W11/W12	3m x 5m wall with dowel hold-downs (W11) and screw hold-downs (W12).	July, 2021
W13	3m x 2m wall with one-sided screw hold-downs.	August, 2021
W14/W15	3m x 2m wall with dowel hold-downs (W14) and screw hold-downs (W15).	September, 2021
W21/W22	7m x 2m wall with dowel hold-downs (W21) and screw hold-downs (W22).	October, 2021
W23/W24	7m x 4m wall with screwed half-lap joint between panels and screw hold-downs. Test repeated with different half-lap screws.	November, 2021
W31/W32	Two 7m x 2m walls connected with steel link beams. Test repeated and repaired.	January – March, 2021

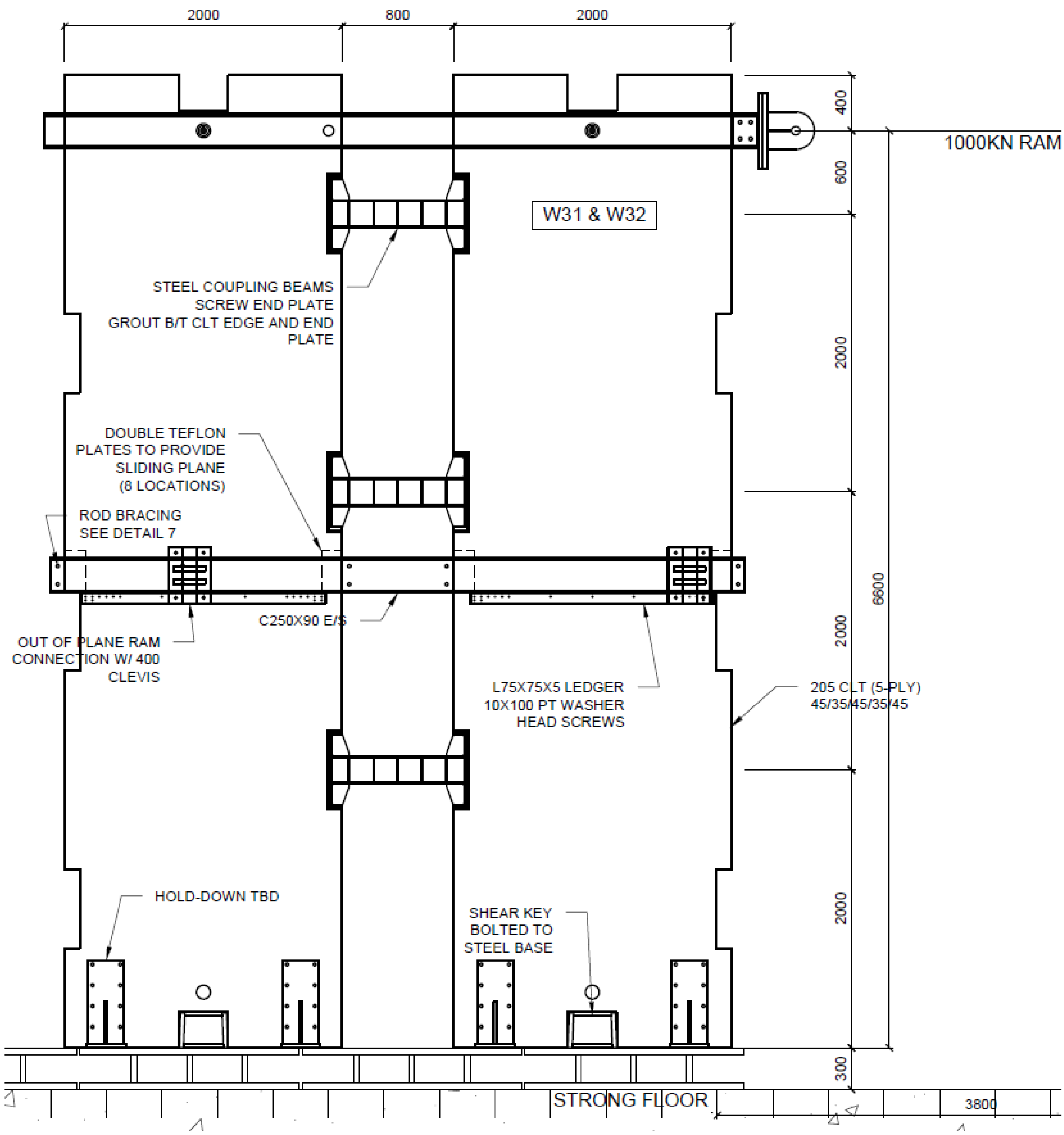


Figure 15 – Test setup for CLT wall tests W31 and W32.

The Douglas-fir CLT for these experiments was made with New Zealand grown Douglas Fir and manufactured by XLAM Australia. The panels recently arrived in Christchurch and are currently being stored in the Structural Engineering Laboratory at University of Canterbury (Figure 16).

Figure 16 – Packaged Douglas-fir CLT for wall testing arriving in SEL lab at University of Canterbury

4.2 Large-scale Screwed CLT Hold-down Test Plan

Building on the results of the second medium scale stage of testing for mixed angle screw hold-downs, the third stage aims to investigate the performance of very high capacity hold-downs with 1000+ kN capacity.

The key goals of this large scale/high capacity testing are to determine:

- The impact of friction on high capacity mixed angle screw hold-downs
- The impact of spacing on the performance of high capacity mixed angle screw hold-downs.
- The impact of spacing on the repaired performance of high capacity mixed angle screw hold-downs.

A test plan and timeline for these high capacity tests is shown below in Table 7. Figure 17 shows a drawing of the high capacity mixed angle screw hold-down testing rig currently under construction at the University of Canterbury Structural Engineering Lab.

Table 7 – Test plan and timeline for high capacity mixed angle screw hold-down tests

Test Set	Description	Replicates	Approximate Test Timeline
1	Baseline Performance	3	July 2021
2	Low Friction	3	August 2021
3	Large a2 spacing	3	September 2021
4	Large a1 spacing	3	October 2021
5	Unique lateral restraint	3	November 2021

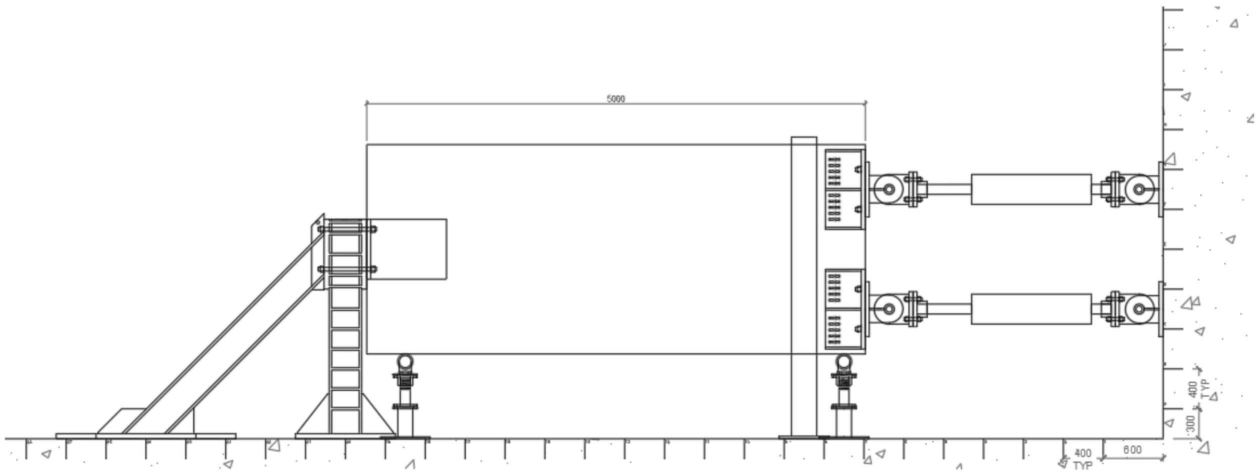


Figure 17 – Figure showing a side view of the high capacity mixed angle screw hold-down rig

5. CONCLUSIONS

Douglas-fir is currently an underutilised species in the New Zealand construction market compared with Radiata pine. However, Douglas-fir has a great potential to be manufactured into high value added CLT products for mass timber construction. To help engineers to specify Douglas-fir CLT in building construction, this multi-year research project aims to develop critical engineering design properties of Douglas-fir CLT products, connections and the critical lateral loading resisting systems for resilient seismic design. This annual report provides a summary of the progress towards the development of high performance Douglas-fir CLT shear walls with high capacity connection systems. A series of experimental tests on two types of critical connections were conducted in order to identify the suitable connection details for the shear wall design. It was found that the screwed steel-CLT connections with notches may provide structurally efficiency and cost effective solutions between the steel link beams and the CLT wall panels in a coupled wall systems. For the hold-down connection design, steel hold-down brackets with mixed angle self-tapping screws can also provide an effective solution which is able to provide high strength, high stiffness and sufficiently ductile responses. The research project is currently ongoing and a complete summary of research findings will be provide in the next annual report.

6. REFERENCES

- Moerman, B., Li, M., Palermo, A., Liu, A. 2021 Self-drilling dowel connections for ductile link beams in coupled CLT walls, New Zealand Society for Earthquake Engineering Annual Conference, Christchurch, New Zeland 14-16 April 2021.
- Wright T., Li, M., Moroder, D., Carradine, D.2021 Cyclic behaviour of hold-downs using mixed angle self-tapping screws in Douglas-fir CLT, New Zealand Society for Earthquake Engineering Annual Conference, Christchurch, New Zeland 14-16 April 2021.