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Experimental Testing of an In-Situ Strengthening Process for CLT Panels

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ABSTRACT:

This study investigates an in-situ strengthening process for cross-laminated timber (CLT) panels, focusing on restoring the structural capacity of floor panels. Experimental testing was conducted to evaluate the effectiveness of the strengthening technique, measuring key parameters such as load-bearing capacity, stiffness, and failure mechanisms. The research methodology included material characterization, mechanical testing, and structural analysis, providing insights into the feasibility of the proposed approach for practical applications. Results indicate that the strengthening process has the potential to restore the mechanical properties of CLT panels, offering a potential solution for strengthening CLT panels in the event that damage occurs. The development of robust strengthening methodologies can expand the use of CLT in construction by enhancing serviceability and resilience against damage. Development of robust panel strengthening methodologies may assist building owners/managers and structural engineers to expand the use of CLT in projects.

KEYWORDS: timber, CLT, repair, damage, testing

1 – INTRODUCTION

Cross-laminated timber (CLT) panels have the potential to be damaged during construction or throughout a building's service life. Damage may result from mechanical impacts during transportation or installation, prolonged moisture exposure due to waterproofing or envelope failures, or due to fire. Such damage can reduce the strength and stiffness of the panel, necessitating a strengthening method that can restore the structural capacity.

This study addresses a practical problem encountered on a project where panel strengthening was required. An incorrectly oriented CL3/130 panel, intended to span its major axis, was instead specified and ultimately installed with its minor axis aligned across a 2.2 m wide corridor. In service conditions this misorientation could have led to excessive deflections and the panels were determined to have inadequate strength to support the required project loading. In collaboration with the project team, a strengthening procedure was developed to strengthen the floor panel on site by installing an additional lamella layer to the floor panel soffit.

The proposed method is based on a composite section analysis and is validated through experimental testing. The goal is to demonstrate that the in-situ strengthening process can restore the key structural parameters—stiffness and strength—of the damaged CLT panels.

2 – METHODOLOGY

2.1 PANEL DESCRIPTION AND PROBLEM IDENTIFICATION

The strengthening process was developed for an XLam CL3/130 panel, with lamellas of 42.5 mm (external) and 45 mm (internal) layers. The panel was incorrectly oriented, causing it to span in its minor axis across the corridor. Upon discovery during a site inspection by the

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project team, XLam was engaged to assess the structural performance and propose corrective measures.

2.1.1 Geometric and Loading Details

- **Span:** 2200 mm; simply supported panel in minor axis
- **Panel Composition: CL3/130**
  - External layers: 42.5 mm thick (XGP1)
  - Internal layer: 45 mm thick (XGP2)
- **Loads:**
  - Self-weight (SW): 0.65 kPa
  - Superimposed dead load (SDL): 1.0 kPa
  - Combined dead load (G): 1.65 kPa
  - Live load (Q): 4 kPa

An initial analysis—assuming properties from a single 45 mm lamella—indicated that both the ultimate bending strength and serviceability limits would not be met under the required loads.

Note: All design actions and strengths presented in this paper are based on a 450mm wide strip of CLT to allow comparison with the tested samples

2.2 STRENGTHENING PROCESS

To address the structural deficiency, an additional 42.5 mm lamella was installed on the soffit side of the CL3/130 panel (see Figure 1). The replacement lamellas, from SG10 radiata pine to match the original XGP1 layer, was installed using a dual-mechanism approach:

- **Adhesive Bonding:** A polyurethane (PUR) adhesive was applied to bond the new lamellas and the existing CLT panel.
- **Mechanical Fastening:** Fully threaded, staggered screws were installed at 150 mm centres and inclined at 45° backwards towards the nearside support.

This combined strategy is intended to give a fully composite section under service level and ultimate limit state loads. If the adhesive bond failed, the screw connections are intended to provide a semi-composite action, ensuring continued performance under ultimate limit state (ULS) conditions.

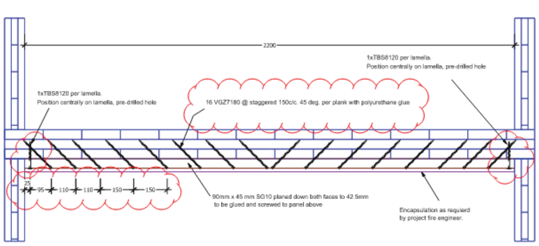

Figure 1 – Strengthened CL3/130 panel

Glued and screwed connections have the potential to offer high strength and high stiffness connections however there can also be challenges with the use of site adhesives in timber construction including issues with environmental control, surface preparation and achieving necessary clamping pressures. Since the strengthening process is required to be completed on site, a robust quality control system is required.

Where the additional lamella layers are installed on the panel soffit, it will generally be required that the lamellas terminate at the face of the supporting walls and do not bear onto these supports since this is restricted by the supporting wall or beam. This means that the strengthened panel at the supports acts a notched panel and this needs to be considered in determining the effectiveness of the strengthening process.

A summary of the calculated strength and deflection of as-built and proposed strengthening corridor panels is provided in **Error! Reference source not found.** below.

Table 1

Configuration	EI (Nmm²)	E_{AVG} (MPa)	ΦM_u (kNm)	ΦV_u (kN)	Predicted failure mode
AS-BUILT: CL3/130 minor axis	2.05×10^{10}	6000	1.1	42.0	Bending
TYPE A STRENGTHENING: <u>Screws Only</u> Analyse as semi-composite section	3.61×10^{11}	4380	15.3	23.4 NOTE 1	Splitting at notch
TYPE B STRENGTHENING: <u>Screw + Glue</u> Analyse as composite section	5.89×10^{11}	7150	21.5	23.4 NOTE 1	Splitting at notch

Note 1: For the strengthened panels, the shear capacity is estimated on a reduced strength section based on experimental testing carried out by Goodarzi et al. A notch strength reduction factor of 0.5 was used to estimate the strength of the notched panel.

3 – STRUCTURAL TESTING

Testing was performed in accordance with AS/NZS 4063.1 Section 2.4 (four-point bending). Three groups of specimens were evaluated:

1. **Control Samples:** Original CLT panels prior to strengthening with minor axis properties.
2. **Screw-Only Strengthened Samples (TYPE A):** Panels strengthened using inclined screw fixings only.

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3. **Screw + Glue Strengthened Samples (TYPE B):** Panels strengthened with PUR adhesive and inclined screws.

Details of the panel strengthening are shown below in Figure 2.

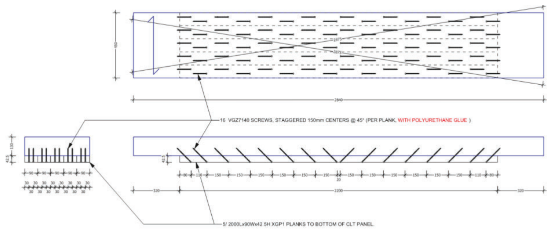

Figure 2 – Panel strengthening detail

3.1 TEST PROCEDURE

- **Control Samples:**
  - Three specimens were tested to failure to establish baseline stiffness and strength.
- **Strengthened Samples:**
  - Four specimens were strengthened using screws only **(Type A).**
  - Three specimens were strengthened using the combined screw + glue **(Type B)**

Load versus displacement curves were recorded for all specimens. The observed failure loads and corresponding modes were then compared to theoretical predictions derived from composite section analysis.

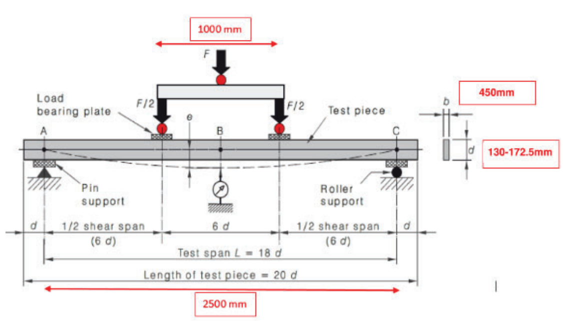

Figure 3 – Panel test set up

3.2 RESULTS

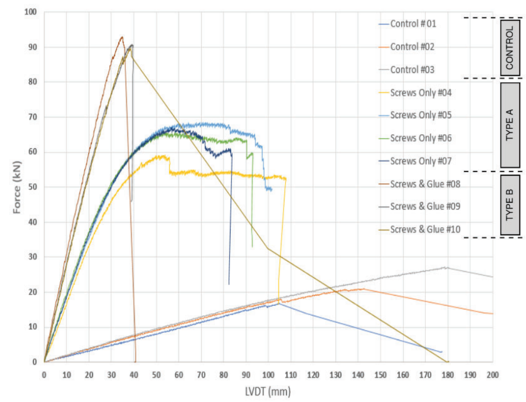

Figure 4 – Load-deflection results (control, Type A, Type B)

Figure 5 – Typical failure modes: Control samples (top centre), Type A screw only (bottom left) and Type B screw + glue (bottom right)

3.2.1 Stiffness

AS/NZS 4063.1-2010 Section 2.4.2 was used to calculate the apparent modulus of elasticity for the control and strengthened test samples. The calculated values were then used to check the deflections of the corridor panel under the as-built condition and under Type A and Type B strengthening methodologies (see Table 2).

$$\boldsymbol{E} = \frac{\mathbf{23}}{\mathbf{108}}\left(\frac{\boldsymbol{L}}{\boldsymbol{d}}\right)^{\mathbf{3}}\left(\frac{\Delta \boldsymbol{F}}{\Delta \boldsymbol{e}}\right)\frac{\mathbf{1}}{\boldsymbol{b}}$$

Table 2

Sample #	Apparent Modulus of Elasticity (AS 4063.1:2010 – Section 2.4.2)		
	Control Samples (MPa)	Screw Only (MPa)	Screw + Glue (MPa)
1	9010	3610	6480
2	8920	3920	6380
3	9640	3970	7030
4	-	3950	-
Mean	9190	3860	6630
Std Dev.	320	150	290
Deflection check	Estimated long term deflection (mm)		
(G+0.4Q) × j2 NOTE 2	28.9	3.0	1.7
Limit (Span/300)	7.3		

Note 2: j_2 = 2 for long-term bending to AS1720.1

3.2.2 Bending and Shear

AS/NZS 4063.1-2010 was used to calculate the bending (section 2.4.3) and shear strength (section 2.7) for the control samples, Type A and Type B strengthening methods methodologies (see Table 3 - Table 5).

$$\boldsymbol{f_b} = \frac{\boldsymbol{F_{ULT}\,L}}{\boldsymbol{bd^2}} \qquad f_v = \frac{\mathbf{0.75}\,\boldsymbol{F_{ULT}}}{\boldsymbol{b\,d}}$$

Table 3

Sample	Control samples					
	F_{ult} (kN)	Failure Mode	Moment at failure (kN/m)	Bending stress at failure (MPa) (AS 4063.1:2010 Section 2.4.3)	Shear at failure (kN)	Shear stress at failure (MPa) (AS 4063.1:2010 Section 2.7)
1	15.9	Bending	4.8	38.4	8.0	0.20
2	17.7	Bending	5.3	42.7	8.9	0.23
3	26.7	Bending	8.0	64.5	13.4	0.34
Mean	20.1	-	6.0	48.5	10.1	0.26
Std Dev.	4.7	-	1.4	11.4	2.4	0.06

Table 4

Sample	TYPE A (screw Only)					
	F_{ult} (kN)	Failure Mode	Moment (kN/m)	Bending stress (MPa) (AS 4063.1:2010 Section 2.4.3)	Shear (kN)	Shear stress (MPa) (AS 4063.1:2010 Section 2.7)
1	58.2	Supp. notch split	17.5	16.8	29.1	0.75
2	64.4	Supp. notch split	19.4	18.7	32.4	0.83
3	66.0	Supp. notch split	19.8	19.1	33.0	0.84
4	67.2	Supp. notch split	19.5	20.2	33.8	0.87
Mean	64.0	-	19.1	18.5	32.1	0.82
Std Dev.	3.5	-	0.91	1.0	1.8	0.05

Table 5

Sample	TYPE B (screw + glue)					
	F_{ult} (kN)	Failure Mode	Moment (kN/m)	Bending stress (MPa) (AS 4063.1:2010 Section 2.4.3)	Shear (kN)	Shear stress (MPa) (AS 4063.1:2010 Section 2.7)
1	89.1	Supp. notch split	26.7	25.8	44.6	1.14
2	90.5	Supp. notch split	27.2	26.2	45.3	1.16
3	92.7	Supp. notch split	27.8	26.8	46.4	1.19
Mean	90.8	-	27.2	26.3	45.4	1.16
Std Dev.	1.48	-	0.45	0.4	0.74	0.02

3.2.3 Target for Corridor Strengthening Application

Appendix D5 of AS 1720.1 was used to assess the adequacy of the strengthening methods using the *"prototype testing methodology"*. Prototype testing determines the structural characteristics of structures or elements that are nominally identical to the units tested. For the specific case of the corridor floor panel strengthening, the following parameters are applicable (see

Table 6). The equivalent test load ($Q_{E,SPECIFIC}$) is required to be 3.1× the maximum design actions in the application considered ($Q^{*}_{SPECIFIC}$) for moment and shear (see Table 7).

Table 6

k_1 (AS1720.1 Table 2.3)	k_2 (AS1720.1 D5.4)	k_{26} (AS1720.1 Table D1)	k_{27} (AS1720.1 Table D2)	k_{28} (AS1720.1 Table D3)
0.8	1.0	1.0	1.0	2.48
Load duration 5 months	Cases other than domestic construction	Failure occurs in timber initially dry	Time to reach test load < 15 mins	3 samples tested. Failure in fasteners.

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$$Q_{E,SPECIFIC} = \frac{k_2 k_{26} k_{27} k_{28}}{k_1} Q^*_{SPECIFIC}$$

$$\frac{Q_{E,SPECIFIC}}{Q^*_{SPECIFIC}} = 3.1$$

Table 7

Parameter	Bending (Specific application – corridor panel)			Shear Check (Specific application – corridor panel)		
	Control Samples	Screw Only	Screw + Glue	Control Samples	Screw Only	Screw + Glue
Specific design action ($Q^*_{SPECIFIC}$)	2.2			3.9		
Min fail. load action from tests ($Q_{E,SPECIFIC}$)	4.8	17.5	26.7	8.0	29.1	44.6
Target load ratio $\left(\frac{Q_{E,SPECIFIC}}{Q^*_{SPECIFIC}}\right)$	3.1			3.1		
Achieved load ratio $\left(\frac{Q_{E,SPECIFIC}}{Q^*_{SPECIFIC}}\right)$	**2.2**	**8.0**	**12.1**	**2.1**	**7.5**	**11.4**

3.3 KEY OBSERVATIONS

3.3.1 Stiffness performance:

- The control samples exhibited an average apparent modulus of elasticity of 9190 MPa.
- For the strengthened panels, the Type A (screw-only) strengthening method produced an average modulus of 3860 MPa, whereas the Type B method (screw + glue) yielded 6630 MPa. Theoretical estimates exceeded the test results by 13.4% and 7.8% for Type A and Type B respectively. Contribution of shear deformation, the screw axial stiffness calculation (in the screw only method), and the topmost transverse layer may be contributing to the calculation discrepancy.
- As expected, the screw + glue configuration provided significantly enhanced stiffness over the screw only method.

3.3.2 Bending and shear performance:

- Control specimens failed predominantly in bending at significantly lower load levels as expected with the minor axis panel orientation.
- The Type A (screw-only) samples demonstrated improved performance. In all cases the failure mode was characterized by splitting at the notched region. The more curved shape of the load-deflection curves may indicate that screw withdrawal initiated the failure at the notch.
- The Type B (screw + glue) specimens reached the highest bending moments with failure still initiated by notch splitting. The load-deflection curves were noticeably more linear under loading preceding a relatively more abrupt failure at peak loading.

3.3.3 Prototype Testing Target:

- Based on AS 1720.1 Appendix D5, the equivalent test load ($Q_{E,SPECIFIC}$) required was 3.1 times the maximum design action ($Q^*_{SPECIFIC}$).
- Both strengthening methods exceeded the shear and bending performance targets; however, only the screw + glue approach restored the stiffness to levels comparable with the original panel

3.4 RESULTS DISCUSSION

3.4.1 Control Samples

In the control sample tests the CL3/130 CLT panels were loaded in their minor axis to the point of failure. The load-deflection responses of the control samples were noted to be consistent across the 3 samples in terms of stiffness. At 9190 MPa, the test calculated apparent modulus of elasticity was significantly higher than the XGP2 feedstock average MOE of 6000 MPa. One of the samples showed a significantly higher failure load than the other two samples and all samples failed at a load significantly higher than that predicted by theory. It is possible the following factors may be influencing this:

- The feedstock in the panels was of higher grade than the minimum required by the manufacturing specification for the XGP2 internal layer.
- The outer lamellas in the transverse direction made a significant contribution the strength stiffness of the panel in its minor axis

While the higher strengths observed in the control samples indicated bending strengths of the corridor panels may have sufficient strength in the as-constructed condition, the stiffness values (even though higher than predicted) still indicated that strengthening of the corridor panels would be needed.

In all control samples, shear utilisation against the theoretical strength was noted to be low at the time of failure and not be an influencing factor in the minor axis bending condition. While not investigated in this study, it is interesting to note the Goodarzi study showed significant differences for the shear strength of un-notched 5-layer CLT when loaded in its major or minor axes. While in practice, significant shear loading in the minor axis direction is relatively rare and would typically be avoided in the design of CLT buildings, it is an interesting and complex behaviour of CLT which may require more detailed design consideration in some instances where clear guidance isn't currently available in accepted design methods.

3.4.2 Strengthened Samples

The apparent modulus of elasticity of the test calculated values for the Type A (screw) and Type B (screw + glue) were average values of 3860 MPa and 6630 MPa respectively. These values were calculated based on a 130 thick panel, in this case ignoring just the topmost lamella oriented in the transverse direction to the span.

Estimated stiffness of 4380 MPa based on a semi-composite, mechanically jointed section for the screw only situation was higher than the tested value of 3860 MPa but could be considered a reasonable approximation in this application. For the screw + glue method the estimated stiffness of 7150 MPa was also higher than the tested figure of 6630 MPa but also a reasonable estimate. As expected the screw + glue method was significantly stiffer than the screw fixed method. Both estimates of the strengthened methods were based on effective section properties which did not consider shear deformation which is a potential cause of the lower stiffness values observed in the testing. Overall, the testing indicated that reasonable estimates of stiffness can be calculated using simple calculation methods for semi-composite sections (gamma method) and fully composite sections. In the cases considered, ignoring the stiffness contribution of the top-most transverse layer seems reasonable to simplify the calculations.

Test results showed that for all of the strengthened panels, failure initiated from the notches. Therefore, on the strengthened samples, there is potential reserve in bending strength which was not realised in the tests due to the initial failure at the notched end. An analysis of the screw fixing loads in the TYPE A (screw only) method indicated that at the point of failure, the withdrawal loads of the screws were close to their limiting values or may have initiated failure at the notch as indicated by the more curved load-deflection curve.

3.4.3 Project Specific Strengthening

As mentioned, the prototype testing approach of AS1720.1 Appendix D5 was applied to check whether the control and strengthened panels had a strength (in shear and bending) that is ***at least equal to the critical design actions encountered in the project specific application multiplied by the calculated equivalent loading factor***. For the specific application of the corridor floor and with 3 or 4 test samples, the calculated loading factor was 3.1 (taking the lower 3 samples). The control panels did not meet the loading criteria however both strengthening strategies did by a significant margin.

For the strengthened panels, both Type A and Type B methodologies demonstrated minimum design actions for shear and bending above the required target levels. MOE values determined from the testing showed that the deflection of the strengthened panels would meet the required deflection limit. On this basis, the Type A and Type B strengthening methods can be accepted in this application.

3.4.4 Target for a General Strengthening Methodology

Beyond the specific project geometry and loading conditions, it is desirable to develop a general strengthening approach that could be applied in many different floor applications. The goal is to demonstrate:

The experimentally-tested structural parameters (stiffness, shear and bending strength) of the strengthened panels are equal to or greater than the theoretically-calculated performance of the original panel specification.

Since we are aiming to demonstrate that the panel strengthening can return the panel to its original structural design parameters, full utilisation in bending and shear is assumed under a permanent loading condition. It is also assumed that a greater number of samples would be tested (20) to reduce the sampling factor (k_{28}).

Table 8

k_1 (AS1720.1 Table2.3)	k_2 (AS1720.1 D5.4)	k_{26} (AS1720.1 Table D1)	k_{27} (AS1720.1 Table D2)	k_{28} (AS1720.1 Table D3)
0.57	1.0	1.0	1.0	1.6
Load duration 50 years	Cases other than domestic construction	Failure occurs in timber initially dry	Time to reach test load < 15 mins	Assume 20 samples tested. Failure in fasteners.

$$\boldsymbol{Q_{E,GEN} = \frac{k_2 k_{26} k_{27} k_{28}}{k_1} Q^*_{MAX}}$$

$$\boldsymbol{\frac{Q_{E,GEN}}{Q^*_{MAX}} = 2.8}$$

The following minimum actions from the target test load ($Q_{E,GEN}$) is then determined for the CL3/130 panel.

Table 9

Parameter	Bending Moment (kNm)	Shear		
		Full Panel (kN)	Notch top surface (kN)	Notch bottom surface (kN)
Full utilisation action for CL3/130 (Q^*_{MAX})	10.0	28.8	19.4	14.4
Target load ratio $\left(\frac{Q_{E,GEN}}{Q^*_{MAX}}\right)$	2.8			
Target test min. failure load from tests ($Q_{E,GEN}$)	28.0	80.6	54.3	40.3

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The test results indicate that only the screw + glue method is expected achieve a stiffness and bending strength equivalent to the original panel specification, as verified through AS1720.1 Appendix D5 prototype testing.

The notch effect caused by the lamella layer is a key factor in shear strength when it is installed on the soffit side. The study by Brandon et al. showed similar findings. This effect may be more significant in 3-layer panels compared to 5- or 7-layer panels, however only 3 layer panels have been investigated in the current study. Testing indicates that a screw + glue method is likely necessary to meet shear requirements. The effectiveness of different notch reinforcement details that can be practically installed on site could be investigated.

The current tests had the strengthening layers terminating at the face of the support. Further testing should be conducted with the strengthening layers extending through the bearing support to better understand how notches contribute to shear strength reduction. However, in many cases, the strengthening layers would still likely need to terminate at the face of the support due to the common platform construction used in CLT buildings.

4 – CONCLUSIONS

The experimental evaluation confirms that simple, on-site strengthening methods can restore the structural performance of damaged CLT panels.

The development of robust strengthening methodologies can expand the use of CLT in construction by enhancing serviceability and resilience against damage. Effective methods of strengthening that may be applied cost-effectively in the event that damage occurs may alleviate concerns developers and insurers of considering buildings with a CLT structure.

Experience gained through applying the strengthening method in the current project and structural testing showed that with a basic surface preparation on site and moisture control, it should be possible to achieve an effective adhesive connection between the replacement lamellas and the CLT substrate.

With an effective adhesive joint, the ULS and SLS conditions may be assessed as a fully composite section. If there are concerns around reliance on a site adhesive connection, the strengthened section could be calculated as a semi-composite section for the ULS condition and a fully composite section for the SLS condition. In this case the strengthened panel may not achieve the full structural properties of the original panel strength but there may be an efficient solution balancing structural performance and on-site works in undertaking the strengthening. The study by Chorlton et al. similarly investigated a screw only repair procedure for strengthening glulam beams.

Additional glue line integrity testing should be carried out to investigate different adhesive options and sensitivity to different conditions such as surface preparation, moisture conditions and clamping pressure to determine an optimum method. Different adhesives with better gap filling potential and/or lower clamping pressure requirements should be investigated may be more suitable where larger areas need to be strengthened or to achieve a reliable adhesive bond in a wider range of situations.

For floors, it is important to consider whether the strengthening is in place on the top or bottom surface of the CLT panel. When the strengthening is implemented on the bottom surface of the CLT, it is likely that the replaced lamella may terminate at the face of the supporting wall or beam and will not extend onto the bearing support. This means that a potential trigger for splitting is introduced on the bearing side of the panel. When the strengthening is implemented on the top side of the panel, the replaced lamella may also terminate at the face of the wall. This also results in a reduced shear area compared to the original panel, however further reduction for notch splitting is not required.

A general process for achieving full replacement of the external lamellas over a large area of floor or wall should be feasible however further work to determine the most efficient site process needs to be completed. The current study has indicated that in some applications a screwed only lamella replacement may be suitable, for example when a partial restoration of panel stiffness/strength is acceptable or where the required control over the adhesive application is not feasible. It should be noted that the installed lamellas in this study were 42.5mm thick. The effectiveness of screw only strengthening should be confirmed for thinner lamellas.

Fire performance of strengthened panels should be investigated be as part of a generalised strengthening process for CLT elements so it can be applied to elements that are required to be fire resistant.

Development of robust panel strengthening methodologies may assist building owners/managers and structural engineers to expand the use of CLT in projects.

Key conclusions include:

- **Strengthening Methods:**
  - For low-span or lower-utilization scenarios, a screw-only method may be adequate.

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- For higher demands and longer spans, the combined screw + glue method will generally be more suitable due to its superior stiffness and bending strength.

- **Structural Performance:**
  - The strengthening approach aims to restore stiffness and bending capacity to meet the original design specifications.
  - Reduced shear capacity and notch splitting are important failure modes to be considered, particularly when strengthening lamellas do not extend into the support bearing zone.
- **Future Work:**
  - Additional glue line integrity testing under varied environmental conditions, surface preparation, adhesive type and clamping pressures.
  - Extend methods to cover applications on 5 and 7 layer CLT floor panels and CLT walls
  - Investigation of fire performance is necessary to ensure comprehensive safety.

5 – ACKNOWLEDGEMENTS

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