World Conference on Timber Engineering
Oslo 2023

# ROBUSTNESS OF ADHESIVELY BONDED PANEL-TO-PANEL CONNECTIONS IN CLT FLOORS

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**ABSTRACT:** Cross-laminated timber (CLT) panels are increasingly used in floor construction with the individual panels often connected with self-tapping screws (STS) through surface spline, half-lap, and butt joints. An alternative solution is provided by using the Timber Structure 3.0 (TS3) technology that connects butt joints through high-performance adhesives and creates a near-rigid connection; therefore, two-way resistance of CLT panels can be utilized. However, TS3 joints fail in a brittle manner, and floor diaphragms lose integrity after crack develops. In this study, measures to improve the robustness and prevent floor collapse after TS3 joint failure were investigated. Seven panel-to-panel connections with TS3 and a secondary connection system were fabricated and tested under bending to evaluate the joint behaviour and observe crack propagation. After TS3 failed, bending or shear tests were conducted to test the residual strength of the secondary connections. The tests showed that it is possible to increase the robustness of TS3 joints by using additional mechanical connectors; i.e., the secondary connections exhibited residual strength after the TS3 joints failed. More tests are planned to further investigate measures to improve the robustness of TS3 joints.

**KEYWORDS:** Cross-laminated timber, TS3, Self-tapping screw, X-fix, Robustness

## 1. INTRODUCTION

### 1.1 BACKGROUND

Cross-laminated timber (CLT), a plate-like material made from layers of dimension lumber glued crosswise to one another, is commonly used in walls, floors, and roofs [1]. Due to prefabrication limitations, CLT panels are manufactured with width up to approximately 3.4 metres and assembled with panel-to-panel connections on-site. When used as floor diaphragm, the panels resist out-of-plane gravity loads and in-plane lateral (wind and seismic) loads. To successfully build CLT floors, connections between individual panels need to be designed for appropriate strength, stiffness, and ductility.

Traditional panel-to-panel connections employ dowel-type fasteners such as self-tapping screws (STS) to transfer in-plane shear forces. Tests have shown that screw connections display desirable structural performance under in-plane loads [2,3]. An alternative to STS in connecting CLT panels is the X-fix connector [4], which is a double dovetail-shaped wood wedge made of birch or beech veneer plywood.

The desire for open space and clear storey height favours the use of flat-plate system where CLT panels are point supported by columns and span two ways. In such systems, the out-of-plane performance of connections is critical to engage two-way behaviour. However, screw connections have limited moment capacity under out-of-plane loads [5]. As a result, CLT panels are often designed as one-way systems without considering the load-sharing action in the transverse direction.

### 1.2 TIMBER STRUCTURE 3.0 TECHNOLOGY

The Timber Structure 3.0 (TS3) technology provides a viable alternative to connect butt joints on-site using two-component Polyurethane adhesive [6]. TS3 butt-joints provide near-rigid connection performance under bending moments and shear forces and can realize the biaxial load-carrying timber flat slabs with a column grid up to 8 m × 8 m and a live load of 5 kN/m$^2$. This construction technology contributes to CLT replacing concrete in constructing large-scale flexible floor slabs.

However, under seismic loads, the ductility of CLT diaphragms relies on the connections, and engineers are reluctant to use adhesive connections due to their brittle failure. The combination of TS3 and mechanical fasteners such as STS or X-fix provides a possible solution for achieving both high stiffness and high ductility in the joints. In this hybrid system, TS3 provides stiffness for the connections under the service loads while STS or X-fix acts as a backup system and can be designed only for the failure loads. The combination of TS3 and STS joints has been successfully utilized in the floor construction of ON5 building in Vancouver, Canada [7].

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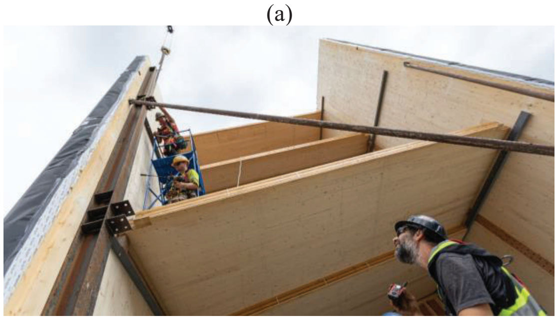
https://doi.org/10.52202/069179-0159

## 1.3 ON5 MASS TIMBER BUILDING

ON5 is an 840 $m^2$ innovative four-storey mixed-use commercial and office building located on a 7.6 m wide infill lot, as shown in Figure 1. The building was designed to Passive House principles by Hemsworth Architecture, engineered by Timber Engineering Inc., and constructed by Naikoon Contracting Ltd.

(a)



(b)

***Figure 1**: ON5 (a) Completed building (b) During construction (credit KK Law courtesy naturally:wood)*

The building was constructed with three-storey mass timber on top of a concrete and masonry podium. The gravity load-resisting systems of the mass timber portion are the CLT floors, roof, and walls, while the lateral load-resisting system relies on a CLT core, shear walls, and diaphragms. The CLT core uses innovative resilient slip friction hold-downs by Tectonus [8] at four corners to dissipate energy and reduce lateral drift. The CLT floor, roof, and wall panels are 5-ply (175 mm) E1 grade [9] while CLT core has both 7-ply (245 mm) and 5-ply (175 mm) running in East-West (short) and North-South (long) directions, respectively.

CLT floor panels spanning 7 m along East-West direction were sitting on balloon-type CLT walls along North-South direction. Under gravity loads, the design of CLT floor panels is governed by deflection and vibration requirements, while the strength requirements can be met when designed as one-way panel in the major strength direction. To achieve the required span with 5-ply E1 grade, the stiffness of CLT in the secondary direction was engaged by employing TS3 connection between panels. This way, the flat slab system was achieved without additional beams. As shown in Figure 2, the adhesive was injected into the gap between panels, creating a seamless connection. In addition, 220 mm FT/CSK screws [10] with a diameter of 10 mm were installed in 45° at a maximum spacing of 500 mm as the backup to TS3 joints in the event of a failed TS3 connection.

Along East-West direction (short direction), the CLT diaphragm cantilevered about 15 m on both sides of the core. Under seismic loads, CLT diaphragm can deflect due to slips between panels. The TS3 joints between panels eliminate slip of diaphragm between panels and control lateral deflection of floors and roof. With the rigid connection solution, a rigid diaphragm is achieved, and forces are transferred to stiffer CLT walls and core.

***Figure 2**: Application of TS3*

## 1.4 FLOOR ROBUSTNESS

In addition to the lack of ductility, another challenge towards the widespread adoption of TS3 is the loss of floor integrity after initial failure. At the ultimate strength level, cracks of TS3 appear at locations of imperfect bonding and wood defects. Initial cracks from extreme or abnormal loadings not considered in the design stage have a risk of propagation, which can lead to floor collapse. This is especially important for mass timber floors as existing studies on disproportionate collapse prevention on mass timber buildings are scarce [11-13].

Implementing structural robustness as a floor property is considered the best-suited method for disproportionate collapse prevention. With structural robustness, the floor can withstand initial damage and stop propagation by developing alternative load paths. A floor is considered robust if it can develop collapse-resistance mechanisms to absorb the initial local damage without collapse [14].

For a continuous TS3 joint, it is beneficial to install crack stoppers that block the crack propagation. The goal of crack stoppers is to physically divide the TS3 joints into different sections and allow stress redistribution after some sections fail. The floor should be able to resist dead and live loads without collapse. The simplest crack stoppers could be gaps or dowels that isolate TS3 into different sections. These simple measures can be combined with mechanical connectors that are often used as secondary connections to TS3, such as STS or X-fix, to provide additional strength.

 https://doi.org/10.52202/069179-0159

## 2. EFFECTIVENESS OF CRACK STOPPERS

### 2.1 OBJECTIVES

For the acceptance of TS3 in CLT panel connections, it is necessary to demonstrate the robustness and ductility of the joints and provide adequate technical information for designers and engineers to use. This study focuses on the effectiveness of crack stoppers in preventing the progressive collapse of TS3. Tests were conducted on seven panel-to-panel connections with combined TS3 and a secondary connection system. The bending stiffness, strength, and shear strength of the joints were determined and compared with each other.

### 2.2 MATERIALS

The panel-to-panel connections with combined TS3 and a secondary connection system were fabricated and tested at ETH Zurich. The CLT panels were 5-ply with a thickness of 150 mm and a lamella grade of C24 [15]. The panels were 1.3 m long and 1.5 m wide, and connected by butt joints in the major strength direction. The TS3 joints had a thickness of 4 mm. The cross-section of the tested joints is shown in Figure 3. Crack stoppers were spaced 750 mm and divided the joint into three sections. To initiate the crack, two external sections were weakened by drilling holes with a diameter of 5 mm at the bottom after TS3 was cured.

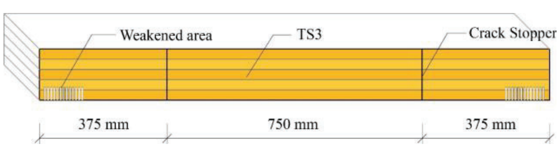



***Figure 3***: *Cross-section of tested TS3 joints*

The specimens used the following crack stoppers:

1) Specimen 1 (SP1) was connected with two pairs of STS.
2) Specimen 2 (SP2) was connected with two X-fix connectors, glued to the bottom of the specimen with PU adhesive, as shown in Figure 4(a).
3) Specimen 3 (SP3) was connected with two pairs of STS. In addition, a 20 mm hole was drilled next to each pair of screws, as shown in Figure 4(c).
4) Specimen 4 (SP4) was connected with two mechanically inserted X-fix at the bottom, as shown in Figure 4(b).
5) Specimen 5 (SP5) and specimen 6 (SP6) comprised two pairs of STS. In addition, vertical beech dowels were installed next to screws to separate TS3, as shown in Figure 4(d).
6) Specimen 7 (SP7) was connected with two mechanically inserted X-fix at the top. At the bottom of each connector, a 20 mm hole was drilled.

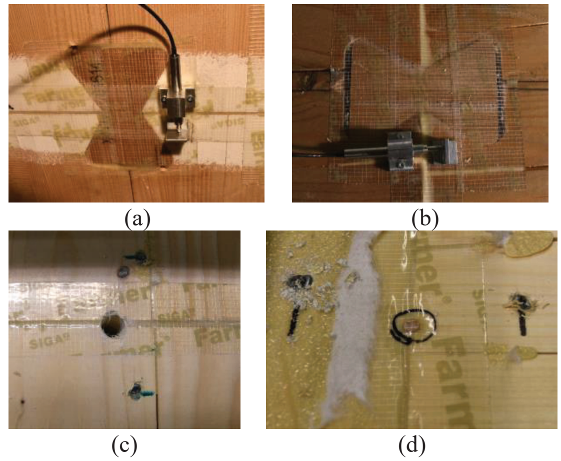



***Figure 4***: *Crack stoppers used in the specimens (a) Glued X-fix in SP2; (b) X-fix in SP4 (c) Self-tapping screws and 20mm hole in SP3 (d) Self-tapping screws and beech dowel in SP5.*

Pairs of double-thread SFS WT-T screws [16] with a diameter of 8.2 mm and a length of 160 mm were installed at 45°. The X-fix connectors were 90 mm high, 130 mm long, and 96 mm wide. In SP4 and SP7 where the X-fix was not glued to CLT, the connector transferred tensile forces through interlocking and shear forces through friction. The 20 mm holes in SP3 and SP7 were drilled after TS3 adhesive was cured. The beech dowels in SP5 and SP6 were installed before the application of TS3. The holes and vertical wood dowels only serve as crack stoppers while STS and X-fix can provide additional tensile and shear strengths to the joints. The effectiveness of the crack stoppers is evaluated based on the following criteria: 1) A crack stopper should stop the continuous crack propagation from one section to another; 2) Load increase should be observed between individual cracks.

### 2.3 METHODS

The specimens were tested under four-point bending and Iosipescu shear as shown in Figure 5 and Figure 6, respectively. Instead of applying loads over the whole width of the panels, loads were only applied at edges of the panels with steel plates under the steel beams to provoke the failure of the glue line in the weakened region. The loading protocol was following the procedure in EN 26891 [17]. The load was applied to 40% of the estimated peak load and maintained at this level for 30 s. The load was then reduced to 10% of the estimated peak load and maintained for another 30 s. Thereafter, the load was increased monotonically until failure. To observe the crack propagation, servo-hydraulic with a displacement control was adopted. The loading rate varied in the range of 1-2 mm/min and was adapted to each specimen so that the specimens failed within 10 min.

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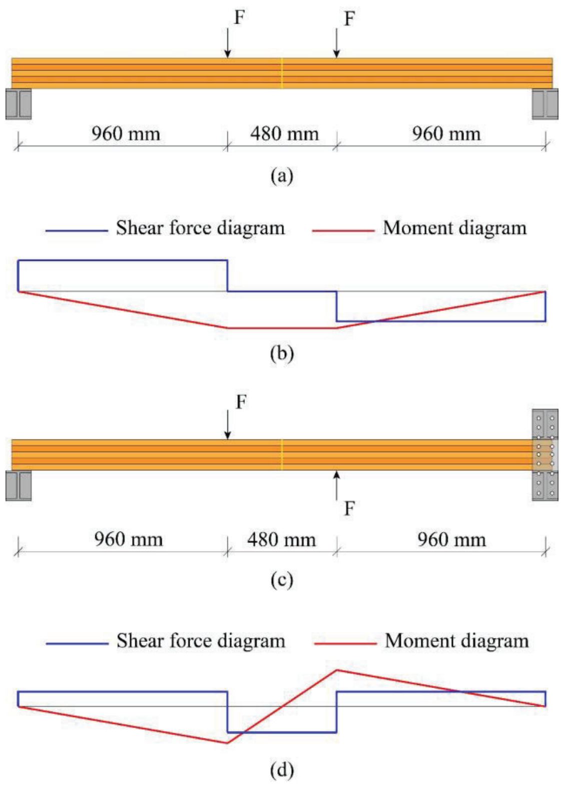


***Figure 5***: *Bending and shear tests on panel-to-panel connections (a) Out-of-plane bending tests; (b) Moment and shear diagrams in bending tests; (c) Out-of-plane shear tests; (d) Moment and shear diagrams in the shear tests*

Four Linear Variable Differential Transformers (LVDTs) were mounted at the bottom of the specimens along the glue line to monitor the crack opening. LVDTs were mounted at two edges and locations of crack stoppers, as shown in Figure 7. The vertical deflection of the joint was measured by two LVDTs mounted on the top two edges.

After TS3 joints failed under bending, specimens SP1, SP2, SP5, SP6, and SP7 were re-tested under shear to determine the residual capacities of the secondary connections. The right hydraulic cylinder was kept in position by mounting the steel plates to the panel while the left hydraulic cylinder applied a downward force. The rightmost support was designed in such a manner that it can resist uplift forces. Under the new loading protocol, the joint in the middle was subjected to pure shear forces without any moment, as shown in Figure 5(d). For specimens SP3 and SP4, second bending tests on the failed TS3 joints were conducted to test the residual bending capacities of the secondary connections.

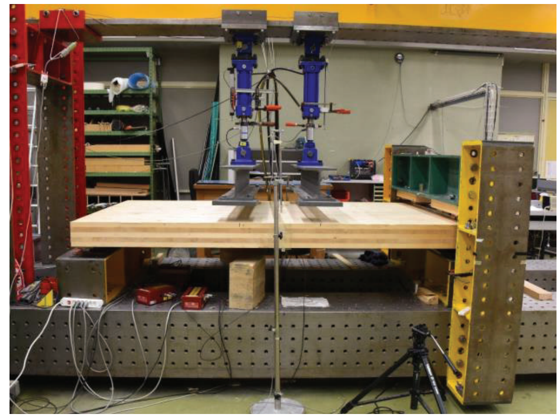

***Figure 6***: *Laboratory bending test setup on panel-to-panel connections*

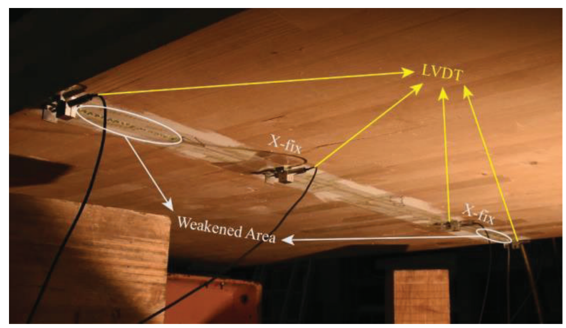


***Figure 7***: *LVDTs mounted at bottom of specimens to measure crack opening*

### 2.4 RESULTS OF FIRST BENDING TESTS

Under bending, the failure of the TS3 joints started from one end at the location of the weakened area. The crack then appeared at the other end and finally in the middle of the joints. An example of crack opening at the bottom of the joint is shown in Figure 8 for SP2. It can be seen that there were three major cracks happening one after another in SP2 before the failure of the entire TS3.

The initial crack of TS3 did not cause the failure of the joint, but resulted in a drop of the applied load, as shown in the load-deflection relation of SP2 in Figure 9 where the vertical deflection was taken as the average of two deflections measured at two edges of the specimen. Following the first crack, the load rose again to the peak load. For most specimens, the second crack happened at the peak load. With more cracks appearing, the TS3 joints failed completely. The failure of a typical TS3 joint under bending is shown in Figure 10. No complete collapse of the specimens was observed due to the secondary connections.


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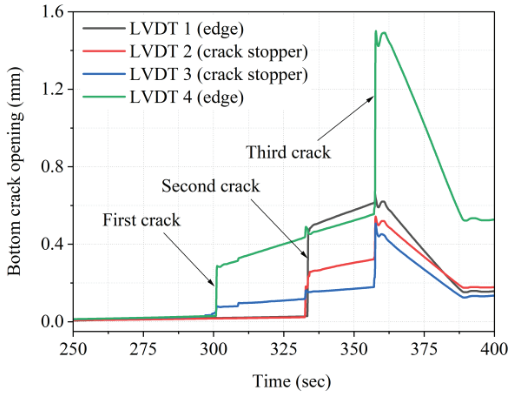


***Figure 8**: Bottom crack opening in SP2*

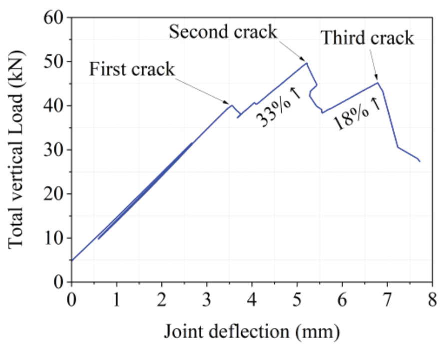


***Figure 9**: Load-deflection relation of SP2*

***Figure 10**: Failure of SP5 under bending*

Based on the crack opening and load-deflection relations of specimens, the number of cracks, the load level of each crack, and the load increase after each crack are listed in Table 1. The load level is the ratio between the load when the crack happened and the peak load. The load recovery represents the maximum increase of load following the load drop due to crack. Most specimens had 3-4 major cracks before TS3 failure. SP6 exhibited only two cracks while SP4 had five. It can also be observed that the peak load usually happened at the second crack except for SP5. The load recovery after each load drop indicates stress redistribution in the joints. On the contrary, a continuously decreasing load after the final crack suggests the complete failure of TS3. The most effective crack stopper was found in SP4 with two X-fix at the bottom, while the least effective crack stopper was found in SP6 which contained STS and vertical beech dowels.

***Table 1**: Effectiveness of crack stoppers*

| | Crack openings | Load level | Load recovery after crack |
|---|---|---|---|
| SP1 | 1st | 100% | 1% |
| | 2nd | 100% | 16% |
| | 3rd | 71% | / |
| SP2 | 1st | 81% | 33% |
| | 2nd | 100% | 18% |
| | 3rd | 91% | / |
| SP3 | 1st | 99% | 1% |
| | 2nd | 100% | 5% |
| | 3rd | 95% | 6% |
| | 4th | 78% | / |
| SP4 | 1st | 68% | 52% |
| | 2nd | 100% | 7% |
| | 3rd | 97% | 17% |
| | 4th | 87% | 19% |
| | 5th | 42% | / |
| SP5 | 1st | 78% | 16% |
| | 2nd | 82% | 11% |
| | 3rd | 77% | 34% |
| | 4th | 100% | / |
| SP6 | 1st | 66% | 63% |
| | 2nd | 100% | / |
| SP7 | 1st | 85% | 23% |
| | 2nd | 100% | 10% |
| | 3rd | 73% | 8% |
| | 4th | 69% | / |

The mechanical properties of the panel-to-panel joints are listed in Table 2. $P_{max}$ is the maximum total vertical load the specimens sustained during bending tests. Based on the maximum load, the moment capacity of TS3 joints can be determined as

$$M_{max} = \frac{P_{max} L_1}{2} \times \frac{2}{3} \qquad (1)$$

where $L_1$ is the distance between loading point and the nearest support, which was 960 mm. The moment resistance has been normalized to 1 m width in Eq. (1). As shown in Table 2, the moment resistance of the joints was in the range of 11.3-20.6 kNm. As a reference, the design strength of C24 CLT is 72 kNm [15]. The lowest bending strength was found in SP5, which was attributed to the defects of TS3 joint. The failed section of TS3 in SP5 showed there was trapped air in the joint during TS3 application process.

The stiffness $K$ of the joints was determined by calculating the slope of load-deflection relations of the specimens in the initial pre-loading phase. From Table 2, it can be observed that the stiffness of the joints showed less variation (COV=13.3%) than the moment resistance of the joints (COV=20.9%). Based on the joint stiffness ($K$ in kN/mm), the apparent bending stiffness $EI$ of the tested cross-section can be determined as:

$$EI = \frac{K}{48}(3L^2 - 4L_1^2)L_1 \qquad (2)$$

where $L$ is span which was 2.4 m. The apparent bending stiffness $(EI)_{ap}$ of C24 CLT under four-point bending load can be determined from shear analogy method as

$$(EI)_{ap} = \frac{(EI)_{eff}}{1 + \frac{24(EI)_{eff}}{(3L^2 - 4L_1^2)GA}} \tag{3}$$

where $(EI)_{eff}$ is the pure bending stiffness and $GA$ is the term accounting for shear deformation. The determined bending stiffness of the TS3 joints according to Eq. (2) is in the range of 1122-1818 kNm$^2$ with an average value of 1560 kNm$^2$. The determined bending stiffness of C24 CLT according to Eq. (3) is 1916 kNm$^2$. It can be concluded that TS3 joints can reach more than 80% of the continuous CLT bending stiffness.

***Table 2****: Summary of test results*

| | $P_{max}$ kN | $M_{max}$ kNm | $K$ kN/mm | $EI$ kNm$^2$ | $M_{re}$ kNm | $V_{re}$ kN |
|---|---|---|---|---|---|---|
| SP1 | 52.4 | 16.8 | 9.6 | 1744 | / | 15.6 |
| SP2 | 49.7 | 15.9 | 10.0 | 1818 | / | 58.5 |
| SP3 | 42.2 | 13.5 | 8.1 | 1469 | 2.2 | / |
| SP4 | 38.6 | 12.4 | 6.2 | 1122 | 3.7 | / |
| SP5 | 35.4 | 11.3 | 8.6 | 1566 | / | 45.5 |
| SP6 | 64.4 | 20.6 | 8.6 | 1558 | / | 46.8 |
| SP7 | 37.7 | 12.1 | 9.1 | 1644 | / | 53.8 |

Note: $P_{max}$ is the maximum total vertical load the specimens sustained during bending; $M_{max}$ is the moment capacity of TS3 joints normalized to 1 m width; $K$ is the stiffness of TS3 joints based on total vertical load and deflection; $EI$ is the bending stiffness of TS3 joints normalized to 1 m width; $M_{re}$ is the residual moment resistance of the secondary joints normalized to 1 m width; and $V_{re}$ is the residual shear capacity of the secondary joints.

## 2.5 RESULTS OF SECONDARY TESTS

For specimens SP3 and SP4, bending tests were conducted again on the failed TS3 joints to test the residual moment capacities of STS and X-fix. The failure of X-fix under bending is shown in Figure 11(a) which shows the shear failure of plywood in the wedge and local shear failure of CLT. For the rest of the specimens, shear tests were conducted on the secondary connections and the shear failure of STS and X-fix under out-of-plane shear are demonstrated in Figure 11(b) and (c), respectively, from which the bending of screws, embedment of screws into wood, and sliding of X-fix can be observed.

The load-deflection relations of SP3 and SP4 under secondary bending are plotted in Figure 12. The moment resistances of two secondary connections are listed in Table 2 as $M_{re}$. While the moment resistances of STS and X-fix were both significantly lower than TS3, X-fix had a higher moment resistance than STS. More importantly, the failure of X-fix joint under bending was in a more ductile manner, as can be seen from the large deflection of the joint in Figure 12(b). The ductility came from bearing of wood fibres between CLT and X-fix.

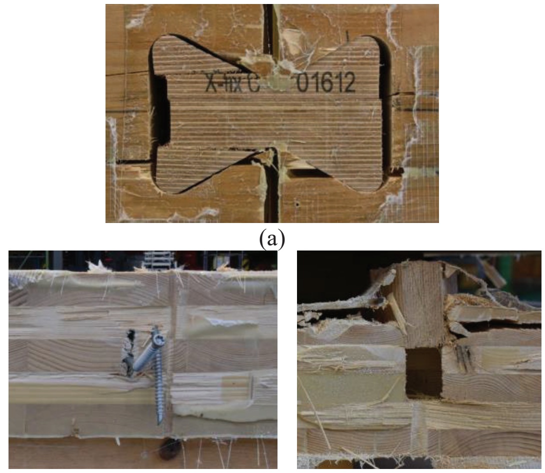


***Figure 11****: Failure of secondary connections (a) X-fix at the bottom of SP4 under bending (b) STS in SP5 under shear (c) X-fix in SP7 under shear*

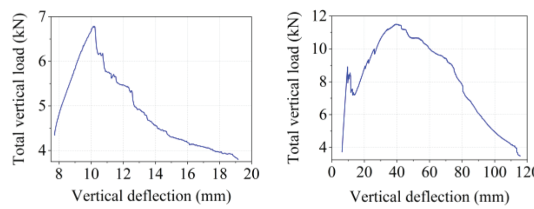


***Figure 12****: Load-deflection relations in secondary bending tests on (a) STS in SP3 (b) X-fix in SP4*

The residual shear strengths of STS in SP1, SP5, and SP6, and X-fix in SP2 and SP7 are listed in Table 2 as $V_{re}$. The shear capacities of X-fix in SP2 and SP7 were similar, while the shear capacities of STS in SP5 and SP6 were close to each other. According to [16], the characteristic shear capacity of two pairs of WT-T screws is 19.3 kN which is less than half of the tested values of SP5 and SP6. It should be noted that X-fix connectors have no technical approval for out-of-plane shear loading. Nonetheless, the tests showed that the X-fix connectors had higher shear strength than STS. It is not obvious why the STS in SP1 had a much lower shear strength than the rest of the specimens. As the shear tests were conducted on the failed TS3 joints, it is possible that STS were already loaded in the first bending test.

# 3. ONGOING EXPERIMENTAL WORK

The experimental study discussed in Section 2 focused on the potential of using crack stoppers to prevent the complete collapse of floors with TS3 joints. The tests were limited to joints in the major strength direction under out-of-plane loading, while TS3 performances in minor strength direction of CLT and under in-plane loads were not investigated. Another critical property of the panel-to-panel connections is the ductility under in-plane and out-


https://doi.org/10.52202/069179-0159

of-plane loads, especially under seismic loads. However, the failure mode of the tested joints was brittle in general.

To address the brittle failure pattern of TS3, metal fasteners such as STS can be installed in a closer spacing than the tested specimens in this study, thus STS is not only used as crack stoppers after TS3 fails, but also as additional fasteners to provide ductility. A research program is underway at University of Northern British Columbia (UNBC) to study the performance of combined TS3-STS joints in connecting CLT panels. The testing program includes combined TS3-STS in both major and minor strength directions under in-plane and out-of-plane loads, as well as two test series under long-term bending load. Figure 13 depicts the proposed testing plan on CLT panel-to-panel connections. The complete testing matrix is shown in Table 3.

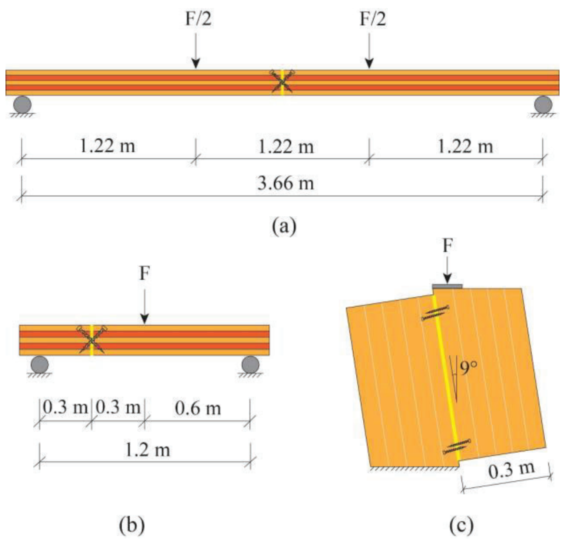


***Figure 13***: *Proposed testing on CLT panel-to-panel connections (a) Out-of-plane bending test (b) Out-of-plane shear test (c) In-plane shear test*

Out-of-plane bending tests will be conducted on the panel-to-panel joints, see Figure 13(a). Four-point bending tests will create a pure bending zone for the joints. The variations of the specimens are the CLT layup (5-ply and 7-ply), joint type (TS3, STS, combined TS3 and STS), and panel orientation (major direction and minor direction). Most specimens will be tested under short-term loading while two series will be tested under long-term loading with load levels of 25% and 50% of the short-term peak load, respectively.

The shear tests on the joints will be conducted on a reduced variation of joints. The main variations in the shear tests are the CLT layup and orientation, while only the joints with combined TS3 and STS will be tested. The out-of-plane shear tests on the connections are shown in Figure 13(b). The load applies in the mid-span and the joint is located at a quarter span from the nearest support. This way, the joint is subjected to the highest shear force and reduced bending moment. The combined shear and bending moment mimic the loading case for the joints in real structures. The in-plane shear test is illustrated in Figure 13(c) where the specimen contains a single shear plane, and the vertical load aligns with the centre of the shear plane.

***Table 3*** *Test matrix of ongoing work*

| Series | CLT ply | CLT direction | Load duration | Joint |
|---|---|---|---|---|
| B5PSA | 5 | // | ST | TS3 |
| B5PSS1 | 5 | // | ST | STS |
| B5PSAS1 | 5 | // | ST | TS3+STS |
| B5PLAS1-1 | 5 | // | LT | TS3+STS |
| B5PLAS1-2 | 5 | // | LT | TS3+STS |
| B5TSA | 5 | ⊥ | ST | TS3 |
| B5TSS1 | 5 | ⊥ | ST | STS |
| B5TSAS1 | 5 | ⊥ | ST | TS3+STS |
| B7PSA | 7 | // | ST | TS3 |
| B7PSS2 | 7 | // | ST | STS |
| B7PSAS2 | 7 | // | ST | TS3+STS |
| B7TSA | 7 | ⊥ | ST | TS3 |
| B7TSS2 | 7 | ⊥ | ST | STS |
| B7TSAS2 | 7 | ⊥ | ST | TS3+STS |
| S5PSAS1 | 5 | // | ST | TS3+STS |
| S5TSAS1 | 5 | ⊥ | ST | TS3+STS |
| S7PSAS2 | 7 | // | ST | TS3+STS |
| S7TSAS2 | 7 | ⊥ | ST | TS3+STS |
| I5PSAS1 | 5 | // | ST | TS3+STS |
| I5TSAS1 | 5 | ⊥ | ST | TS3+STS |
| I7PSAS2 | 7 | // | ST | TS3+STS |
| I7TSAS2 | 7 | ⊥ | ST | TS3+STS |

Note: // and ⊥ represent the major and minor strength directions of CLT, respectively; ST stands for short-term loading; and LT stands for long-term loading.

In all the specimens, the width of CLT is 600 mm. STS are installed in pairs and inclined 45°. For each test series, six replicates will be tested. The connection stiffness, strength, and failure pattern will be evaluated, and then the characteristic values of connections will be provided for the engineering design of connections. The goal of the proposed research is to provide adequate technical guide for designers and engineers to use, thus promoting a growing application of mass timber panels in two-way flat slab floor systems.

## 4. CONCLUSIONS

The effectiveness of crack stoppers in preventing the collapse of TS3 joints was examined by testing seven panel-to-panel CLT connections under bending and shear. Test results showed promising performance of STS and X-fix as secondary connections in transmitting shear forces and bending moments after TS3 failure. Therefore, in floor construction, X-fix or STS can be used as a

https://doi.org/10.52202/069179-0159

temporary measure to connect CLT panels and hold the panels in position for TS3 applications. In the event of TS3 failure, these connections serve a second purpose as the backup system to maintain the integrity of the joint. Overall, X-fix showed better mechanical performance than pairs of STS as secondary connections. However, due to the limited number of specimens tested, no recommendation concerning the design of secondary connections can be provided. Ongoing experimental work will provide data for the robust design of TS3 joints.

## ACKNOWLEDGEMENT

The research project on the crack stoppers was funded by Innosuisse and supported by Timber Structures 3.0 AG, Schilliger Holz AG, and Henkel & Cie AG. The research project was a collaboration between ETH Zurich and the Bern University of Applied Sciences. Funding for the research project on the hybrid joints has been sought from the Natural Sciences and Engineering Research Council of Canada for an Alliance project in collaboration between Timber Engineering Inc. and the University of Northern British Columbia and supported by Timber Structures 3.0 AG.

## REFERENCES

[1] FPInnovations. Canadian CLT Handbook. Pointe-Claire, QC, Canada, 2019.
[2] A. Hossain, M. Popovski, T. Tannert. Cross-laminated timber connections assembled with a combination of screws in withdrawal and screws in shear. *Engineering Structures*, 168: 1–11, 2018.
[3] B. Taylor, A. R. Barbosa, A. Sinha. Cyclic performance of in-plane shear cross-laminated timber panel-to-panel surface spline connections. *Engineering Structures*, 218: 110726, 2020.
[4] ETA-18/0254 of 01.09.2020. European Technical Assessment for X-fix C. Austrian Institute of Construction Engineering, 2020.
[5] S. Zhang, Y. H. Chui. Characterizing flexural behaviour of panel-to-panel connections in cross-laminated timber floor systems. *Structures*, 28: 2047-2055, 2020.
[6] S. Franke, S. Zöllig. TS3 – A New Technology for Efficient Timber Structures. *Current Trends in Civil & Structural Engineering*, 4(4): 1-4, 2020.
[7] A. Hashemi, P. Quenneville, H. M. Bita, R. Malczyk, S. Varier, P. Zarnani. Application of resilient rocking cores in low damage mass timber structures: A case study. NZSEE Annual Conference, 2022.
[8] A. Hashemi, T. Fast, C. Dickof, R. Jackson, P. Fast, A. Dunbar, P. Zarnani, P. Quenneville. Seismic Resilient Mass Timber Structures with Damage-Avoidance Connections: Design Methods and Case Studies. SESOC conference, Hamilton, New Zealand, 2021.
[9] ANSI/APA PRG 320: Standard for Performance-Rated Cross-Laminated Timber. APA-The Engineered Wood Association, 2019.
[10] ETA-12/0373 of 03.11.2017. European Technical Assessment for Schmid screws. Austrian Institute of Construction Engineering, 2017.
[11] H. M. Bita, T. Tannert. Experimental Study of Disproportionate Collapse Prevention Mechanisms for Mass-Timber Floor Systems. Journal of Structural Engineering 146(2): 04019199, 2020.
[12] K. Voulpiotis, J. Köhler, R. Jockwer, A. Frangi. A Holistic Framework for Designing for Structural Robustness in Tall Timber Buildings. *Engineering Structures*, 227: 111432, 2021.
[13] K. Voulpiotis, S. Schär, A. Frangi. Quantifying Robustness in Tall Timber Buildings: A Case Study. *Engineering Structures*, 265: 114427, 2022.
[14] H. M. Bita. Disproportionate Collapse Prevention Analysis for Mid-rise Cross-Laminated Timber Platform-type Buildings. Thesis, University of British Columbia, 2019.
[15] EN 338: Structural timber - strength classes. European Committee for Standardization, 2016.
[16] ETA-12/0063 of 15 April 2019. European Technical Assessment for SFS Self-tapping Screws WT. DIBT, 2019.
[17] EN 26891. Timber structures - Joints made with mechanical fasteners - General principles for the determination of strength and deformation characteristics. European Committee for Standardization, 1991.


https://doi.org/10.52202/069179-0159