Fire-Resistance of Timber-Concrete Composite Floor using Laminated Veneer Lumber

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**PROJECT N°**: 301010618
Fire-Resistance of Timber-Concrete Composite Floor using Laminated Veneer Lumber

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# TABLE OF CONTENTS

1 INTRODUCTION ............................................................................................................................ 1
2 OBJECTIVES ................................................................................................................................. 1
3 TECHNICAL TEAM ........................................................................................................................ 1
4 BACKGROUND ............................................................................................................................... 2
  4.1 Previous FPInnovations Research ........................................................................................... 2
  4.2 Deflection ................................................................................................................................ 3
5 MATERIALS AND METHODS ........................................................................................................ 4
  5.1 TCC Floor Assembly Details for Fire-Resistance Test ............................................................. 4
  5.2 TCC Floor Assembly Details for Shear Tests ........................................................................ 9
6 RESULTS ..................................................................................................................................... 11
  6.1 Fire-Resistance Test .............................................................................................................. 11
    6.1.1 Observations .................................................................................................................. 11
    6.1.2 Deflection ....................................................................................................................... 14
    6.1.3 Temperatures ................................................................................................................. 15
    6.1.4 Charring rates ................................................................................................................. 16
    6.1.5 Heat Transfer into Concrete ............................................................................................ 17
  6.2 Shear Tests ........................................................................................................................... 18
7 DISCUSSION ............................................................................................................................... 19
  7.1 Structural Fire-Resistance ..................................................................................................... 19
8 CONCLUSION AND RECOMMENDATIONS ................................................................................ 23
9 REFERENCES ............................................................................................................................. 24
LIST OF FIGURES

Figure 1 – TCC floors under construction ........................................................... 2
Figure 2 – TCC floors after test completion .......................................................... 3
Figure 3 – Overall TCC floor construction details ............................................... 4
Figure 4 – Lag screw spacing across the width of a single LVL beam .................. 5
Figure 5 – Lag screw spacing along the span ...................................................... 5
Figure 6 – Lag screw installation into LVL beams ............................................... 5
Figure 7 – Connection between LVL beams ...................................................... 6
Figure 8 – Thermocouple locations ................................................................. 6
Figure 9 – Placement of thermocouples at various depths ................................. 7
Figure 10 – Thermocouple installation ............................................................... 7
Figure 11 – Assembly prior to concrete pouring ............................................... 8
Figure 12 – Completed assembly ................................................................. 8
Figure 13 – Exposed side of assembly in furnace ............................................. 8
Figure 14 – Assembly loaded on furnace prior to testing at NRCC ................. 8
Figure 15 – (a) Symmetric and (b) non-symmetric configurations for wood-concrete shear tests 9
Figure 16 – Symmetric configuration for wood-concrete shear test with the concrete inside 10
Figure 17 – Specimen shear test construction details (a) elevation view, (b) top view 10
Figure 18 – Shear test specimen ready to be tested ........................................... 11
Figure 19 – Furnace temperature during the fire test ......................................... 12
Figure 20 – Screws exposed at end of test ...................................................... 12
Figure 21 – Assembly falling into furnace ....................................................... 12
Figure 22 – Empty loading frame ................................................................. 13
Figure 23 – Assembly fallen into furnace ....................................................... 13
Figure 24 – Evidence of remaining LVL .......................................................... 13
Figure 25 – Remaining LVL after test and exposed lag screws ....................... 13
Figure 26 – Mid-span deflections measured during test ................................. 15
Figure 27 – Adjusted thermocouple readings .................................................. 16
Figure 28 – Temperature profiles at LVL-concrete interface and lag screws .... 17
Figure 29 – Load-slip curves for each specimen tested in shear ....................... 18
Figure 30 – Embedded thermocouple temperature comparison to the 2 x 8-concrete assembly 20
Figure 31 – Comparison between predictions and experimental deflection .... 21
Figure 32 – Connection behaviour in function of certain values of \( \theta \) ............... 22

LIST OF TABLES

Table 1 – Material properties used for the structural analysis (in MPa) ................. 9
Table 2 – Observations during the fire test .................................................. 14
Table 3 – Times at which charring temperatures were reached ....................... 16
Table 4 – Temperature rise ........................................................................... 17
Table 5 – Shear test results ............................................................................ 18
Table 6 – Calculated time resistance for the LVL-concrete floor with certain connection stiffness 22
1 INTRODUCTION

Mid-rise wood buildings have been approved for construction in British Columbia for several years, and more recently in the provinces of Quebec and Ontario. Designers and Authorities Having Jurisdiction (AHJs) have had the opportunity to become more familiar with this type of construction and realized the benefits of constructing these taller buildings with wood products.

There is a desire in the province of British Columbia and some other Canadian provinces, as well as internationally, to construct taller wood buildings, greater than 6 storeys. The University of British Columbia has begun constructing an ambitious 18-storey project. Tall wood buildings require more robust designs and are subject to more stringent requirements than those set forth in the National Building Code of Canada (NBCC) [1], such as an improved inherent fire-resistance. Among some reasons, and not only related to fire safety, the use of mass timber construction will most likely be required for these buildings.

For mass timber floor systems to span long distances and meet serviceability limit states, it will likely be necessary to incorporate some kind of topping to meet vibration and acoustic criteria; timber-concrete composite floors are a practical option for this application.

2 OBJECTIVES

There is a need to demonstrate how novel timber-concrete composite floors can span long distances and be a practical alternative to other traditional structural systems. Better understanding of the fire behaviour of these hybrid systems is essential. To achieve this, the fire-resistance of a timber-concrete composite floor assembly, using BC wood products, will be evaluated in accordance with CAN/ULC-S101 [2]. A 2 hr fire resistance rating will be targeted, as this is the current requirement in high-rise buildings for floor separations between occupancies.

The structural behaviour of this type of system will also be assessed from conducting pull-out tests of the shear connectors.

In conjunction with previous test data, the results of this test will be used to develop an analytical model to assess the structural and fire-resistance of timber-concrete composite floors.

3 TECHNICAL TEAM

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Most of the instrumentation and assembly of the LVL samples was done at FPInnovations’ laboratory in Québec City. The fire-resistance test was conducted at National Research Council Canada in Ottawa (Ont.).
4 BACKGROUND

In 2014/2015 The Forestry Innovation Investment of British Columbia funded early research into the fire performance of long span timber-concrete composite (TCC) floors, specifically investigating the overall structural behaviour and potential impact of heat transfer from the shear connectors in fire conditions. These tests demonstrated little to no heat transfer between the wood and the concrete due to the metal shear connectors.

FPInnovations has already done some preliminary tests related to the structural performance of these types of systems.

A review of various TCC systems was completed in conjunction with a project funded by Natural Resources Canada that is exploring structural considerations for long span wood systems [3].

4.1 Previous FPInnovations Research

FPInnovations has been investigating the structural behaviour of TCC floors as well as their fire performance for some time now. In 2015, FPInnovations tested two TCC assemblies simultaneously on a full-scale furnace at National Research Council Canada in Ottawa [3]. One floor assembly consisted of a series of nine screw laminated 2 x 8 “beams” (38 x 185 mm, on edge), where each beam combined five pieces of lumber boards. Conventional truss connector plates were pressed into either side of the “beams” to act as shear connectors into the 89-mm (3½”) concrete topping. The other assembly consisted of a 5-ply (175 mm) CLT and an 89-mm (3½”) concrete topping, which used self-tapping wood screws driven in at 45º as shear connectors. Under a 2.4 kPa superimposed load (representing a representative live load for office spaces and a greater load than that required for residential occupancies according to the NBCC), both floors performed very well when exposed to the CAN/ULC-S101 [2] time-temperature fire curve. The assemblies under construction are shown in Figure 1.

![a) CLT-concrete with screws at 45º](image1)

![b) 2x8 screw laminated wood-concrete](image2)

Figure 1 – TCC floors under construction
During the test, both floors deflected until the CLT-concrete assembly failed first, after 214 minutes of standard fire exposure. Failure of the screw laminated floor was not reached due to the testing configuration, which required the furnace to be turned off once the first assembly failed. Temperature profiles were relatively similar for the first hour of testing, after which the laminated-wood assembly began to demonstrate better performance. The difference is attributed, in part, to the ability of the 2 x 8 charred wood to remain in place for long periods and to its homogenous cross-section. The CLT-concrete assembly exhibited heat delamination as the glue lines approached 180-200°C (localized fall-off was also visually observed), resulting in increased charring rates of 0.89 and 0.74 mm/min for the 2nd and 3rd layers, respectively. Nevertheless, a global effective charring rate of 0.68 mm/min was measured from embedded thermocouple data (up to 105 mm penetration) for the CLT-concrete assembly, which accounts for heat delamination of plies due to adhesive failure. A global charring rate of 0.58 mm/min for the 2 x 8-concrete assembly was observed (based on a total char depth of up to 105 mm). A picture of the floor assembly being lifted off the furnace after the test is presented in Figure 2.

![Figure 2 – TCC floors after test completion](image)

The temperature increases were recorded as 20°C at the CLT-concrete interface and 93°C at the shear connector interface. Temperatures at the 2 x 8-concrete interface increased by only 16°C and by 64°C at the shear connector interface. Mid-depth into the concrete, temperature rise was minimal for both assemblies (<10°C), indicating that negligible heat was being transferred into the concrete itself. Overall, there was no significant heat transfer into the concrete through the shear connectors in either sample.

### 4.2 Deflection

According to ISO 834 [4], deflections are to be measured at the location where maximum deflections are expected to occur, in this case at mid-span. This standard defines the loadbearing capacity as the ability to support the test load determined by the amount and rate of deflection. Rate of deflection criteria are not deemed applicable until a deflection of \( L/30 \) is reached. It is noted that these deflection criteria are not stipulated in CAN/ULC S101.
For flexural elements, limiting deflection ($\Delta$) is found based on Equation (1), where $L$ is the clear span (in mm), and $d$ is the distance from the extreme fibre of the compression zone to the extreme fibre of the design tensile zone (in mm). The limiting rate of deflection ($\frac{\partial \Delta}{\partial t}$) is calculated using Equation (2).

\[
\Delta (mm) = \frac{L^2}{400d}
\]

\[
\frac{\partial \Delta (mm)}{\partial t (min)} = \frac{L^2}{9000d}
\]

5 MATERIALS AND METHODS

5.1 TCC Floor Assembly Details for Fire-Resistance Test

The design was chosen so as to limit duplication of any previous testing that has been conducted to date. The assemblies evaluated the year prior used a cross-laminated timber (CLT) and a nail-laminated-timber (NLT) wooden lower base. To be innovative for this specific project, laminated veneer lumber (LVL) was used. The latter meets ASTM Standard Specification D5456 [5] as evaluated by the Canadian Construction Materials Center (CCMC) and the International Code Council Evaluation Services (ICC-ES) for code compliance [6, 7]. The LVL beams were of 133 x 406 mm (5¼" x 16") oriented flatwise and manufactured from wood veneers bonded with a phenol-based exterior-type structural adhesive, oriented vertically and running parallel to the length of the member. It is assumed that the samples met the manufacturer’s LVL product specification. A schematic of the TCC floor assembly is shown in Figure 3.

Figure 3 – Overall TCC floor construction details
The shear connection between the LVL beams and the concrete topping is created using lag screws of 13 mm (½") diameter and 152 mm (6") long with a penetration depth of 89 mm (3½") into the LVL beams, meeting lag screw material requirements of CSA O86 [8]. A diameter of 13 mm was selected to investigate whether such diameter would result in greater heat transmission up through the assembly during the fire test when compared to the self-tapping screws used in previous tests.

As with typical placement of lag screws in connections, lead holes for the threaded shank portion were predrilled prior to insertion of the lag screws. They were placed perpendicularly (90°) to the LVL panels. Across the 406-mm (16") width of the LVL beams, two rows of screws were spaced at 4" from either edge, as shown in Figure 4. The spacing of the lag screws along the 4.8 m length of the assembly is shown in Figure 5, where spacing at the center of the span was of 610 mm (24") and gradually decreased towards the outer edges as the longitudinal shear flow became greater. Installation of the lag screws into the LVL beams during construction is shown in Figure 6.
The LVL beams were connected side-by-side together using $\phi 8$-160/100mm self-tapping screws spaced at 305 mm o.c. (12") and drilled at 45° angles. The connection between LVL beams is shown in Figure 7.

![Figure 7 – Connection between LVL beams](image)

The LVL beams were instrumented with thermocouples at five locations and at various depths to capture temperature profiles within the assembly during the fire test as shown in Figure 8. Within the LVL beam itself, thermocouples were installed at 35 mm, 70 mm, and 105 mm from the fire-exposed face (bottom face). At locations #1 through #4, two thermocouples were placed at the unexposed face of the LVL beams (i.e., the interface between the LVL and the concrete): one touching a lag screw and the other simply on the LVL surface. Thermocouples that were in contact with the lag screws were wrapped and glued with epoxy. At location #5, one thermocouple was placed mid-depth into the concrete topping, not in contact with any screws. Exposed wires were securely stapled to the LVL structure. Embedded thermocouple depths are shown in Figure 9. Thermocouple installation during construction is shown in Figure 10.

![Figure 8 – Thermocouple locations](image)
The individual LVL beam components were constructed at the FPInnovations' laboratory in Québec City, including the installation of lag screws and thermocouples. The remaining preparation, including connecting the beams side-by-side together, final thermocouple installation and concrete pouring were completed by the staff of the National Research Council (NRC) fire laboratory in Ottawa (Ontario).

A concrete topping of an 89-mm thickness and a 30 MPa strength was poured on top of the assembly. A 9/9g wire mesh with 305 mm (6") spacing was placed mid-depth into the concrete (i.e. on top of lag screws). The prepared assembly ready to receive concrete is shown in Figure 11. Concrete was poured on December 11, 2015, and was provided sufficient time to cure. The average 28-days strength of the concrete was measured to be 35.8 MPa from three cylinders. The finished assembly is shown in Figure 12. The exposed side of the assembly once installed on the furnace is shown in Figure 13, and the overall assembly on the furnace prior to testing is shown in Figure 14.
An applied load of 2.4 kPa was chosen. This was the same load that had been applied during the TCC tests conducted in 2015 [3] and allows for straightforward comparison of results, in particular because CLT and LVL products have similar strength values. 2.4 kPa is the live load that would be expected for office or residential buildings in accordance with the NBCC.
5.2 TCC Floor Assembly Details for Shear Tests

To minimise the assumptions during the structural analysis, the material properties and the connection properties are needed. The bending strength \( (F_b) \), the tension parallel to grain strength \( (F_t) \) and Young’s modulus \( (E) \) of the LVL are known by the CCMC evaluation report [6]. The strength of the concrete was determined by a standardized compression test [9] made on three cylinders as described in section 5.1. Young’s modulus \( (E_c) \) can be estimated from the CSA A23.3 [10] with the compressive strength \( (f_c') \) (Equation (3)). Table 1 gives the material properties used for the structural analysis.

\[
E_c = 4500\sqrt{f_c'}
\]

(3)

Table 1 – Material properties used for the structural analysis (in MPa)

<table>
<thead>
<tr>
<th>Material</th>
<th>( E )</th>
<th>( f_c' )</th>
<th>( F_b )</th>
<th>( F_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>26 940</td>
<td>35.85</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Wood</td>
<td>13 790</td>
<td>-</td>
<td>37.59</td>
<td>18.58</td>
</tr>
</tbody>
</table>

The connection properties were determined by a shear test. There is no standard method to evaluate the properties of a wood-concrete connector, but several authors have already conducted shear tests in several configurations. The configurations can be symmetric or non-symmetric as shown in Figure 15.

![Figure 15 – (a) Symmetric and (b) non-symmetric configurations for wood-concrete shear tests](image)

In this study, a symmetric configuration has been chosen. With this configuration, the concrete can be outside as shown in Figure 15 (a) or inside as shown in Figure 16. When the concrete is outside, the wood element has two shear planes, which means that the wood needs to be sufficiently thick to avoid any possible interference between the connectors on either side. For this reason, the configuration shown in Figure 16 was chosen, since the LVL beam used was of 133 mm (5¼”) thick and the penetration of the lag screw into the LVL beam was of 89 mm (3½”).
Three specimens were prepared to evaluate the connection behaviour of the LVL-concrete floor. The specimens were constructed at FPInnovations’ Québec City laboratory. The construction details of these specimens are shown in Figure 17. Two welded meshes per specimen (6 x 6in 9/9g) were placed at approximately 64 mm (2½") from the LVL-concrete interface to prevent cracking due to shrinkage. The dimensions of the specimens and the position of the connection were determined as to reproduce as closely as possible the connection of the LVL-concrete floor used for the fire-resistance test. The average 28-days strength of the concrete used for the shear test was measured to be 32.7 MPa from three cylinders.

ISO 6891: 1983 [11] was used as the loading protocol for the shear test except that the preload cycle was omitted. The tests were controlled in displacement by the crosshead with a speed of 0.5 mm/min up to a displacement of 4 mm and 1.5 mm/min afterward until failure or large displacement was reached (over 15 mm). Typically, the total testing time was of about 15 min. Two lasers were installed per specimen. The slip taken for the analysis is the average slip measure from the two lasers. The standard provides the method to calculate connection stiffness, which is the slope of the load-slip curve between 40% and 10% of the maximum load. Figure 18 shows a specimen ready to be tested. One laser is visible on the Figure and the other laser is placed on the diagonal opposite side of the specimen.
6 RESULTS

6.1 Fire-Resistance Test

The assembly was tested at the NRCC fire laboratory in Ottawa on January 14 2016, in accordance with CAN/ULC S101. Prior to testing, the moisture content of the LVL assembly was taken from its underside at three locations. The measured moisture contents were of 10.3%, 9.5%, and 7.8% (average 9.2% MC). The moisture content of the concrete was measured to be 78.7% through a small pilot hole.

6.1.1 Observations

When the fire test began, there was initial vigorous burning of the exposed LVL beams. During this initial period of time, propane was no longer being supplied to the furnace, because the burning wood was producing enough heat to generate the CAN/ULC-S101 curve. This was apparent by fluctuating furnace temperature values. A comparison between the furnace temperature and the standard fire curve is given in Figure 19. The furnace temperature was accurately following the standard time-temperature curve throughout the entire test duration, within a 0.1% difference.
Very little movement was apparent during most of the test. There was no evidence of LVL veneers or sections falling off as the char front progressed into the LVL beams. There was one odd small localized piece that fell off, but this was a rare occurrence and it happened very late into the test. Minimal deflections were apparent, which slowly increased to about 1 cm after 2 h of standard fire exposure. After 2 hours it became evident that several thermocouple temperatures were reading lower temperatures than the ones that had already been displayed. This trend occurred at various thermocouples during the test, which would suggest that it was not related to a simple thermocouple malfunction.

After 3 h into the fire test, deflections began to increase rapidly, eventually until failure, based on a very fast rate of deflection. As the assembly deflected, the wood screws attaching the beams together became more prominent, as shown in Figure 20. At 3 h 17 min, the load was removed from the assembly because excessive deflections had reached the maximum level of the loading pistons, and no load was able to be applied after this point. Upon removal from the furnace, the assembly was not able to support its own self-weight and it collapsed into the furnace (Figure 21 to Figure 23). This would most likely have not happened if the concrete topping was designed as a structurally reinforced slab.
The largest piece of the concrete slab was resting upright against the furnace after the assembly had fallen. This piece was pushed down into the furnace during post-testing clean up. The assembly was left to smolder overnight, which resulted in the nearly complete burning of any remaining LVL beams. The photographs shown in Figure 24 and Figure 25 were taken a few days after the test, when all smoldering had ceased. At this time, there was essentially no remaining LVL left in the furnace.

Hour by hour observations are provided in Table 2 below.
Table 2 – Observations during the fire test

<table>
<thead>
<tr>
<th>Time</th>
<th>Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0:00</td>
<td>Initial deflection of 1.5 mm; initial unexposed temperatures between 19-20°C</td>
</tr>
<tr>
<td>0:15</td>
<td>Wood starts to burn vigorously; no propane is being supplied to furnace</td>
</tr>
<tr>
<td>5:00</td>
<td>Lots of dark smoke is coming out from the exterior exhaust at the beginning of the test</td>
</tr>
<tr>
<td>22:00</td>
<td>Continuous burning of the LVL. Flames appear in strands similar to grass in the wind</td>
</tr>
<tr>
<td>33:00</td>
<td>3.5 mm deflection</td>
</tr>
<tr>
<td>40:00</td>
<td>35 mm embedded TCs around 100°C</td>
</tr>
<tr>
<td>54:00</td>
<td>TC51 at 230°C (35 mm)</td>
</tr>
<tr>
<td>55:00</td>
<td>Light smoke visible along east edge of assembly</td>
</tr>
<tr>
<td>1:05:00</td>
<td>TC51 at 300°C, indicating the char front location</td>
</tr>
<tr>
<td>1:08:00</td>
<td>TC41 at 300°C (35 mm)</td>
</tr>
<tr>
<td>1:10:00</td>
<td>TC31 at 300°C (35 mm)</td>
</tr>
<tr>
<td>1:15:00</td>
<td>No evidence of LVL layers falling off</td>
</tr>
<tr>
<td>1:20:00</td>
<td>Deflection of approximately 1 cm, reduction in temperature noted in TC31 (less than 300)</td>
</tr>
<tr>
<td>1:33:00</td>
<td>TC21 at 300°C (35 mm)</td>
</tr>
<tr>
<td>1:37:00</td>
<td>TC11 at 300°C (35 mm)</td>
</tr>
<tr>
<td>1:47:00</td>
<td>Flame movement appears more gradual and undulating</td>
</tr>
<tr>
<td>2:10:00</td>
<td>Deflection of approximately 2 cm</td>
</tr>
<tr>
<td>2:15:00</td>
<td>TC32, TC52, TC42 reach 300°C (70 mm)</td>
</tr>
<tr>
<td>2:24:00</td>
<td>TC12, TC22 reach 300°C (70 mm)</td>
</tr>
<tr>
<td>2:29:00</td>
<td>A very small piece falls off</td>
</tr>
<tr>
<td>2:54:00</td>
<td>A small piece of LVL falls off. Screws between the LVL beams become visible</td>
</tr>
<tr>
<td>3:09:00</td>
<td>Deflection rate increases significantly</td>
</tr>
<tr>
<td>3:10:00</td>
<td>Connecting screws become more visible as floor deflects</td>
</tr>
<tr>
<td>3:17:00</td>
<td>Load is removed from furnace, because the maximum length of pistons is reached</td>
</tr>
</tbody>
</table>

6.1.2 Deflection

Throughout the test, the rate of deflection stayed very low. The rate began to increase around 170 minutes into the test. At 191 minutes (3 h 11 min), the rate of deflection spiked, which suggests that the assembly had failed. Figure 26 illustrates the deflections measured at mid-span during the test.

The load was removed from the assembly at 197 min, but the assembly continued to deflect under its own self-weight, until the whole assembly ultimately collapsed into the furnace.
Based on the ISO 834 criteria for deflection and rate of deflection, the maximum deflection of $L/30$ can be taken as 160.5 mm, which occurred at 193 min at the centreline of the assembly. The limiting rate of deflection was taken as 11.6 mm/min, which was exceeded at 191 min. As such, based on observation and from the ISO 834 rate of deflection limit, the assembly is deemed to have failed at 191 min (3 h 11 min). The maximum deflection at this time was of 137.0 mm.

![Figure 26 – Mid-span deflections measured during test](image)

### 6.1.3 Temperatures

The fire-resistance of the TCC floor assembly was determined based on the load-carrying capacity of the assembly. Using deflection criteria from ISO 834, it is determined that the assembly failed at 191 min. Integrity and insulation criteria were satisfied throughout the test duration. At failure, the maximum unexposed temperature was of 42°C, resulting in a temperature rise of 23°C.

During the fire test, several thermocouples indicated decreases in temperature readings and/or delayed temperature increases, which was awkward and unexpected. Several of the thermocouples may have been faulty and may not have had a good connection, which resulted in brief temperature irregularities. Eventually the measured temperatures at these locations returned to levels that would be expected. Another possible explanation could be related to thermocouples within charred LVL veneers detaching from the assembly, while still being housed within the char. This phenomenon is sometimes apparent and results in thermocouples reading lower temperatures. However, since this has occurred at several locations, and in some cases was related to significant differences in temperature readings (as high as 400°C), this may not be the primary explanation for this behaviour.

Because of these irregularities, temperature values were adjusted to follow the expected trend of the data, to eliminate any significant decreases. The adjustment was done simply with the inclusion of a straight line between realistic data points. Plots of the original data, demonstrating how the data was adjusted is given in Appendix I. The average values of the embedded thermocouples, that have been adjusted, are shown in Figure 27.
6.1.4 Charring rates

The char front was assumed to have reached the various thermocouples depths once temperatures exceeding 300°C were recorded. Most of the temperature decrease inconsistencies did not have an effect on when charring was believed to have occurred. Table 3 summarizes the times at which 300°C was exceeded at different locations and the associated average charring rate. The charring rate at 105 mm was recorded from only 1 thermocouple location. Based on the depth of the thermocouples, charring rates were calculated. The charring rate between 0 and 35 mm of the LVL was of 0.47 mm/min and slightly increased to 0.52-0.55 mm/min afterwards, until a depth of 105 mm. Such charring rate is consistent with that claimed by LVL products’ manufacturers (0.59 mm/min) [12].

Table 3 – Times at which charring temperatures were reached

<table>
<thead>
<tr>
<th>Location Number</th>
<th>Average Time 300°C reached (min)</th>
<th>Average Charring Rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35 mm</td>
<td>73.9</td>
<td>0.47</td>
</tr>
<tr>
<td>70 mm</td>
<td>134.6</td>
<td>0.52</td>
</tr>
<tr>
<td>105 mm</td>
<td>186.9¹</td>
<td>0.56</td>
</tr>
<tr>
<td>133 mm – LVL</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>133 mm – lag screw¹</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

¹ Only reached at 1 thermocouple
² 133 mm represents the interface between the LVL and the concrete.

Thermocouples were either directly attached to the LVL, or touching a lag screw, as indicated.

No thermocouples at the LVL-concrete interface, either attached to the LVL structure or touching a lag screw reached 300°C, suggesting that the char front did not progress to this depth.
6.1.5 Heat Transfer into Concrete

The lag screw shear connectors extended 89 mm (3½”) into the LVL beams, and therefore were embedded at 64 mm (2½”) into the concrete. Table 4 summarizes the temperature rise at the various thermocouples embedded at the interface and into the concrete. At the interface, temperatures increased on average of 70°C at the LVL and 96°C at the locations of the lag screws. This indicates that the lag screws were experiencing a greater heat transfer through the assembly. Similarly, at mid-depth into the concrete, temperature rise was merely of 5°C, but the lag screws at this depth were measured at 47°C. This indicates that the shear connectors did transfer heat along their length, but this temperature increase did not have a significant impact on the surrounding assembly, nor their mechanical properties. Figure 28 shows the temperature profiles recorded at the various interfaces.

<table>
<thead>
<tr>
<th>Location</th>
<th>LVL – concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interface – at LVL</td>
<td>69°C</td>
</tr>
<tr>
<td>Interface – at lag screw</td>
<td>96°C</td>
</tr>
<tr>
<td>Mid-depth in concrete – at lag screw</td>
<td>47°C</td>
</tr>
<tr>
<td>Mid-depth in concrete</td>
<td>5°C</td>
</tr>
</tbody>
</table>

Figure 28 – Temperature profiles at LVL-concrete interface and lag screws
6.2 Shear Tests

The results of the three specimens are shown in Table 5 and their load-slip curves are shown in Figure 29. $k_i$ is the initial stiffness of the connector, $F_u$ is the measured peak load within a slip of 15 mm and $\delta_u$ is the slip at the peak load.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$k_i$ (N/mm)</th>
<th>$F_u$ (kN)</th>
<th>$\delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6850</td>
<td>22.1</td>
<td>15.0</td>
</tr>
<tr>
<td>2</td>
<td>6848</td>
<td>21.6</td>
<td>15.0</td>
</tr>
<tr>
<td>3</td>
<td>5970</td>
<td>21.1</td>
<td>15.0</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>6556</strong></td>
<td><strong>21.6</strong></td>
<td><strong>15.0</strong></td>
</tr>
</tbody>
</table>

Given that the wood, concrete and connection properties are known, the flexural stiffness and the mechanical resistance of the TCC floor assembly can be evaluated by existing methods such as the $\gamma$-method proposed in Annex B of the Eurocode 5 [13]. By applying this method, the flexural stiffness of the LVL-Concrete floor is of $6.401 \times 10^{12}$ N-mm$^2$ and the load capacity is of 62.7 kPa (186.3 kN·m/m). To evaluate the load capacity of the floor, the contribution of the concrete tensile strength is neglected and $k_i$ is multiplied by 2/3. However, several researchers doubt of the precision of the $\gamma$-method to evaluate the load capacity, especially with ductile connectors, which is characteristic to the lag screws used. To take into account the connection resistance and ductility, the elasto-plastic model proposed by Frangi and Fontana [14] is used. Following this method, the load capacity of the floor was assumed to be of 60.4 kPa (179.6 kN·m/m). In the case of the studied floor, the evaluated capacities from the two methods were found to be similar.

![Figure 29 – Load-slip curves for each specimen tested in shear](image-url)
7 DISCUSSION

7.1 Structural Fire-Resistance

The LVL-concrete floor assembly was able to withstand 3 h 11 min of standard fire exposure before structural collapse. In previous FPInnovations fire testing of TCC floor assemblies [3], a CLT-concrete assembly structurally failed after 3 h and 34 min under similar loading conditions. A screw laminated 2 x 8-concrete assembly that was tested simultaneously alongside the CLT-concrete assembly did not fail during the test, but was likely close to failure. Overall, these TCC floor assemblies have demonstrated ample fire-resistance to be used in tall wood building applications where 2 h fire resistance ratings are required.

The maximum deflections in the previous tests were of 7.5 cm for the CLT assembly at failure, and only 3 cm for the 2 x 8 assembly. The specimen in this test was determined to have failed with a deflection of 13.7 cm, based on a rate of deflection of 11.6 mm/min. All of the assemblies demonstrated a similar degree of minimal deflection for essentially 3 h into the test. The CLT assembly failed when cracks formed in the concrete, resulting in a fast increase of deflection after failure. The LVL assembly in this test was able to deflect significantly, up to nearly 40 cm until it ultimately collapsed completely, which is a significantly greater deflection that the CLT assembly was able to support before failure.

The temperatures measured at the embedded thermocouples increased in a gradual manner (after adjustment), similar to how the 2 x 8-concrete assembly performed, as is demonstrated in Figure 30. As layers fell off from the CLT-concrete assembly, temperatures would increase rapidly to be consistent with the furnace temperature. The more gradual temperature increase is attributed to the wood material staying in place once it had charred, which ultimately insulated and protected the wood beneath from thermal degradation.

The times at which charring is expected to have occurred at the various depths of 35 mm, 70 mm and 105 mm were also similar to those from the 2 x 8 assembly, resulting in a fairly constant charring rate throughout the test duration when compared to that of CLT where an increased charring rate is observed due to adhesive heat delamination occurring around 150-200ºC (Figure 30). The charring rates in the LVL assembly were slightly slower than that observed in the 2 x 8 assembly, potentially attributed to its greater relative density. Only one thermocouple was measured at above 300ºC in the LVL assembly at a depth of 105 mm, indicating that more of the LVL wood thickness was able to remain uncharred during the test.
Temperatures at the interface between the materials did rise to a higher level in the LVL assembly than with the previous two tests. The temperature at the interface touching a shear connector was consistent with what was observed in the CLT assembly, which similarly used self-tapping screws of 8 mm in diameter as shear connectors. However, the lag screws extended to 89 mm (3½”) into the wood as compared to 76 mm (3”) into the CLT assembly, resulting in a faster time to exposure to fire from underneath. The larger diameter of the lag screws did not seem to create an increase in heat transfer up through the assembly. Lastly, as was the case in the preceding two tests, temperature increase mid-depth into the concrete was negligible.

Figure 31 shows a comparison between the experimental deflection measured during the test and the predictions using a 1-D finite element method, as described in [15], in which the tension strength of the concrete is neglected. A constant charring rate of 0.55 mm/min is assumed in the model. The “Full composite action” curve represents the deflection when the connection between the wood and the concrete is assumed infinitely rigid, which is the upper bound estimate of the assembly’s stiffness. The “Composite action” curve represents the deflection of the wood-concrete floor by assuming the same connection law from the shear test. The “JPL Composite action” curve represents the deflection of the wood-concrete floor by assuming the same connection law from the shear but by multiplying the resistance by the $I_{PL}$ factor related to the lag screw penetration length, as given in article 12.6.6.1.1 of CSA O86-14 [16]. The “Wood only” curve represents the deflection when the concrete is assumed to be simply a dead load on the wooden floor, which is the lower bound estimate of the assembly’s stiffness (i.e. no structural contribution from the concrete).
When examining Figure 31, one can observe that the “Composite action” curve predicts well the deflection until approximately 125 min. This means that the connection starts losing its stiffness after that time, most likely due to the reduced lag screw penetration in the LVL and the increasing heat transfer in the connector. After 125 min of standard fire exposure, the lag screw is embedded in approximately 65 mm of LVL, while the initial penetration was of 88.9 mm. To validate the assumption of the loss in connection stiffness due to the reducing penetration length, an analysis was made by modifying the connection behaviour as a function of the wood’s thickness and by multiplying the connection strength by the $J_{PL}$ factor where a linear interpolation was assumed between 0 for 0 mm of wood penetration and 1 for 101.6 mm ($8d_F$) of wood. It should be noted that, normally, the CSA O86-14 [16] assumed a linear interpolation between 0.625 for $5d_F$ and 1 for $8d_F$. There is currently no guidance in CSA O86 for lag screw penetration lower than $5d_F$. With 88.9 mm of initial penetration, $J_{PL}$ is taken as 0.875 since the penetration is less than the upper limit of $8d_F$. Figure 32 show the connection behaviour as a function of selected $J_{PL}$ values. The $J_{PL}$ values represent 0.875, 0.219, 0.044 and 0.031, which translates to 100%, 25%, 5% and 3.5% of the initial stiffness, respectively. Table 6 give the calculated resistance time using these selected $J_{PL}$. It can be observed that the first 75% loss of connection stiffness does not influence the resistance of the floor assembly, probably due to the fact that the wood is so thin when it collapses (around 7.5 mm) that the connection does not need to be really stiff to transfer a good tension force to the wood. To obtain the same resistance time, the connection stiffness needs to be reduced to 3.5% of its initial stiffness.
Figure 32 – Connection behaviour in function of certain values of $J_{PL}$

Table 6 – Calculated time resistance for the LVL-concrete floor with certain connection stiffness

<table>
<thead>
<tr>
<th>Experimental</th>
<th>$k = 1 \cdot \frac{2}{3} k_i$</th>
<th>$k = 0.25 \cdot \frac{2}{3} k_i$</th>
<th>$k = 0.05 \cdot \frac{2}{3} k_i$</th>
<th>$k = 0.035 \cdot \frac{2}{3} k_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>191 min</td>
<td>229 min</td>
<td>229 min</td>
<td>221 min</td>
<td>191 min</td>
</tr>
</tbody>
</table>

Nevertheless, from the “JPL Composite action” curve shown in Figure 31, it can be observed that a better fitting is achieved until approximately 175 min of standard fire exposure, but requires an improvement most likely due to the $J_{PL}$ assumption for wood penetration between 0 and 5 $d_f$, which is not documented or determined.
8 CONCLUSION AND RECOMMENDATIONS

When examining the results from the three timber-concrete composite floor fire resistance tests completed by FPInnovations, it seems clear that the shear connectors have little to no impact on heat transfer into the assembly. Even the larger diameter of the lag screws did not seem to create a significant increase in heat transfer up through the assembly.

Moreover, these TCC floor systems reached significant fire-resistance, beyond 3 hours, when subjected to a 2.4 kPa loading condition. These performances are significantly greater than the 2 h fire-resistance rated assembly required in tall buildings.

During the fire-resistance test, the LVL material stayed in place throughout the duration of the test, which provided thermal protection of the wood beneath. This resulted in a fairly uniform charring rate of 0.56 mm/min throughout the test duration, which is consistent with that claimed by some LVL products manufacturers.

Lastly, when examining the results from the structural analysis, it seems obvious that the connection between the concrete and LVL beams have lost a lot of its initial stiffness (around 96.5% loss) towards the end of the fire-resistance test. It seems that this loss of stiffness is essentially due to the loss of penetration of the lag screws into the LVL beams due to charring. CSA O86-14 provides a factor to account for the penetration which only gives a value when the penetration is between 5 and 8 times the lag screw diameter. There is currently no provision for penetrations less than 5\(d_F\). However, during a fire test, the loss of wood due to charring may become so important, that the minimum penetration of 5\(d_F\) value is no longer fulfilled and the yield model may no longer provide accurate or reasonable values. Whether the yield model still applies at such low penetrations or how it should be applied will need to be investigated. Therefore, more experimental results for a wood-concrete connection system as a function of its penetration length are needed to better predict the fire-resistance of a wood-concrete floor, namely with respect to its strength and deflection towards the end, thus near structural failure.
9 REFERENCES


[16] CSA O86-14, Engineering design in wood, Canadian standard association.
APPENDIX I – Thermocouple Data
thermocouple location #5