Limberlost Place: A 10 Storey Slab Banded Structure

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1. Executive Summary

Limberlost Place (previously known as «The Arbour») is a groundbreaking mass timber project, situated on George Brown College's waterfront campus in Toronto, Canada. Rising 10 storeys and covering 16,000 square meters, the building will serve as a pivotal educational center, housing the Tall Wood Research Institute, a childcare facility, and various teaching and social spaces. At its heart lies an innovative beamless structural floor system, which integrates cross-laminated timber (CLT) concrete composite «slab bands» supported by glulam columns. This innovative structural design not only provides flexibility in architectural planning but also enables the efficient distribution of mechanical systems within the building. The project's realization was made possible through external funding, primarily from the National Resources Canada Green Construction Wood Program (GCWood), in conjunction with strategic partnerships aimed at fostering mass timber innovation tailored to project unique requirements. This collaboration has prompted an extensive structural testing program at the University of Northern British Columbia, focused on evaluating timber concrete composite «slab bands» and developing cost-effective composite connectors. The research phase involved comprehensive investigations of full-scale and half-scale cross-laminated timber (CLT) timber-concrete-composite (TCC) slab bands, with a keen focus on assessing their out-of-plane shear and ultimate flexural capacities utilizing a range of shear connectors. The testing program included evaluations of various shear flow connectors, including self-tapping screws, steel kerf plates, and proprietary glued-in Holz-Beton-Verbund (HBV).

2. Project Description

2.1. Introduction

Limberlost Place is the first of its kind in Canada, a 10 storey, assembly occupancy 2 hr rated, net zero carbon building (Figure 1). The project's environmental goals include achieving LEED Gold certification and meeting Toronto's Tier 4 Green Standard. The building has a footprint of 62x37 m and complies with load requirements outlined in the 2017 edition of the Ontario Building Code and the 2015 edition of the National Building Code of Canada, including wind and seismic loads as well as floor and roof dead, live, and snow loads. The structural system predominantly utilizes timber, with exposed mass timber floors and glulam columns. Cross-laminated timber (CLT) panels act as the primary floor system from levels 2 to 9. The structure is designed to withstand a two-hour fire event, with a comprehensive char analysis performed according to CSA O86 Annex B standards, along with supplementary calculations based on the Ministry of Municipal Affairs and Housing Supplementary Standard, SB-2 Fire Performance Ratings, in subsection 2.11 of the OBC for glue-laminated timber beams and columns. The structural steel components also require a two-hour fire-resistance rating.

Figure 1: Rendering of Limberlost Place

2.2. Superstructure

The superstructure of Limberlost Place incorporates all main structural materiasl to create a dynamic and efficient building system. The ground floor suspended slab consists of a 300 mm thick reinforced concrete slab supported by concrete columns and walls beneath it (Figure 2). The transition from concrete columns to glulam columns occurs at a height of 500 mm above the ground-level slab. From levels 2 to 8, the TCC slab bands serve as the primary floor structure. These bands eliminate the need for deep glulam beams, offering increased headroom and space for mechanical and electrical components. The slab band is comprised of 244 mm thick 7-ply CLT panels with a 150 mm concrete topping, spanning 9.2 m in the north-south direction. These panels also support non-composite 191 mm thick 7-ply CLT infill panels (Figure 3). Figures 4a and 4b show the cross-section and photo of a panel before concrete pouring, respectively, Figure 4c and 4d present photos of the slab band, before and after the installation of the infill panels.

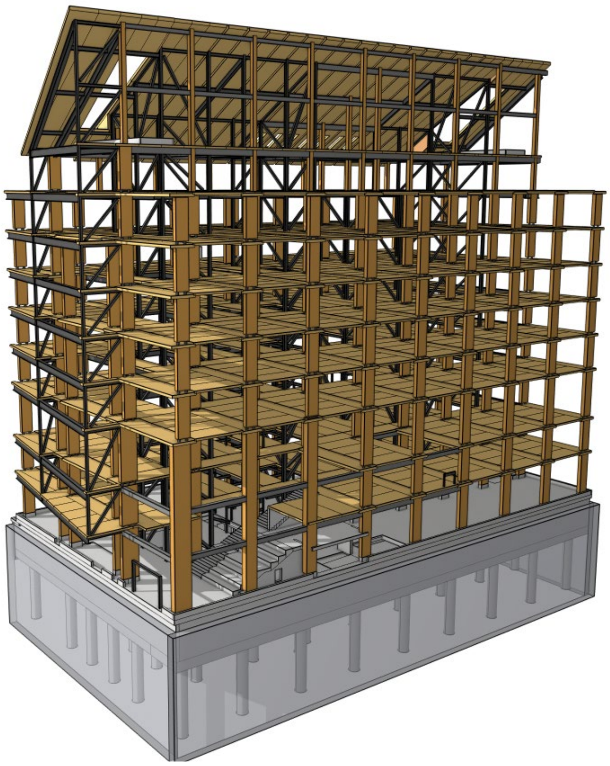


Figure 2: Schematic of Building Gravity and Lateral Systems

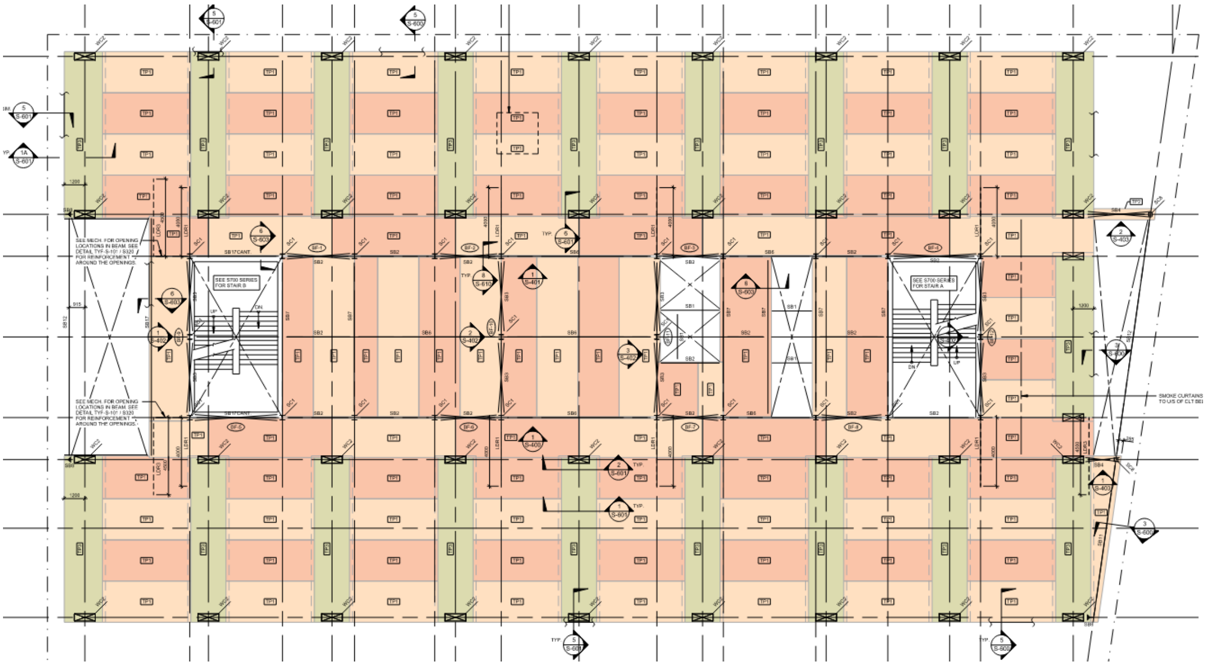
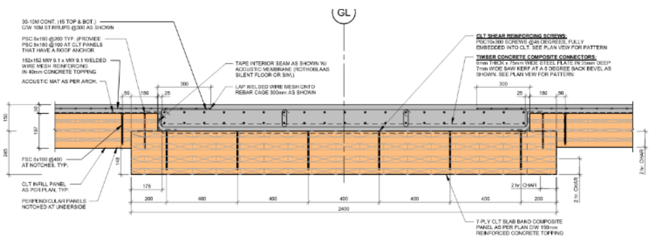
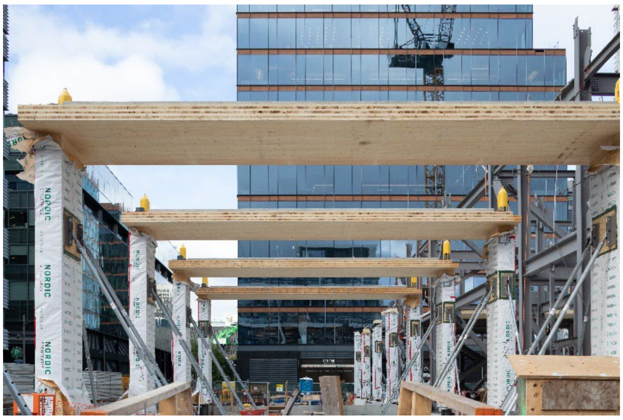


Figure 3: Typical floor plan



(a)

(b)

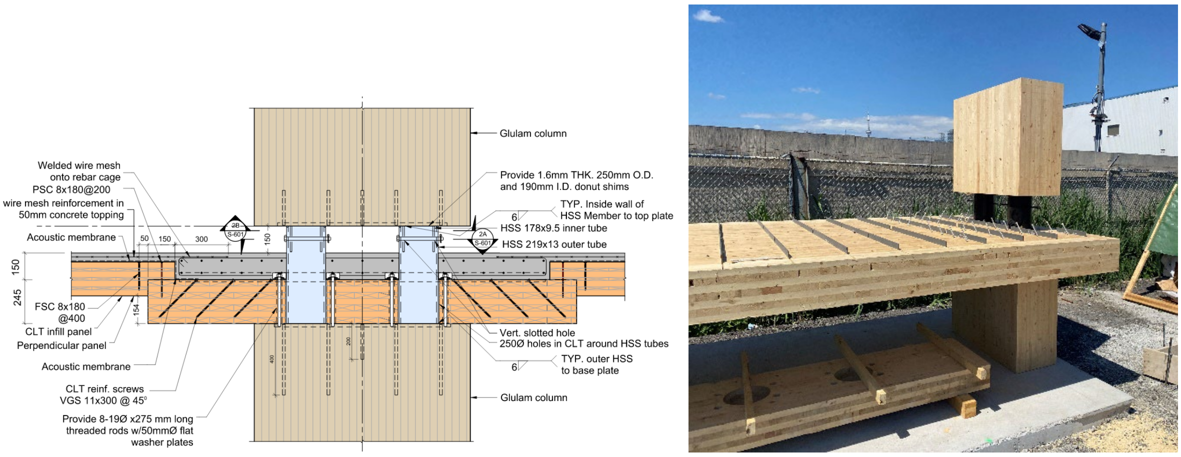


(c)

(d)

Figure 4: a) TCC slab band cross-section, b) slab band before concrete pouring, c) photo of a TCC slab band, d) slab bands with infill slab

The TCC slabs are supported by 422 × 1210 mm glulam columns which are meticulously detailed and positioned beneath the slab bands to withstand unbalanced loading. The column connection, as illustrated in Figure 5, ensures direct load transfer between vertical elements, bypassing the need to transmit forces through the TCC floor panels. Each glulam column is equipped with a steel connecting plate and Hollow Square Sections (HSS) stubs secured to the end grain via glued-in rods. These stubs are connected using bolts, facilitating straightforward installation and serving as a tension connection in rare cases where a column below might need to be removed, satisfying to progressive collapse principles. CLT floor panels are notched to accommodate the HSS tubes and bear directly on the column below.



(a) (b)

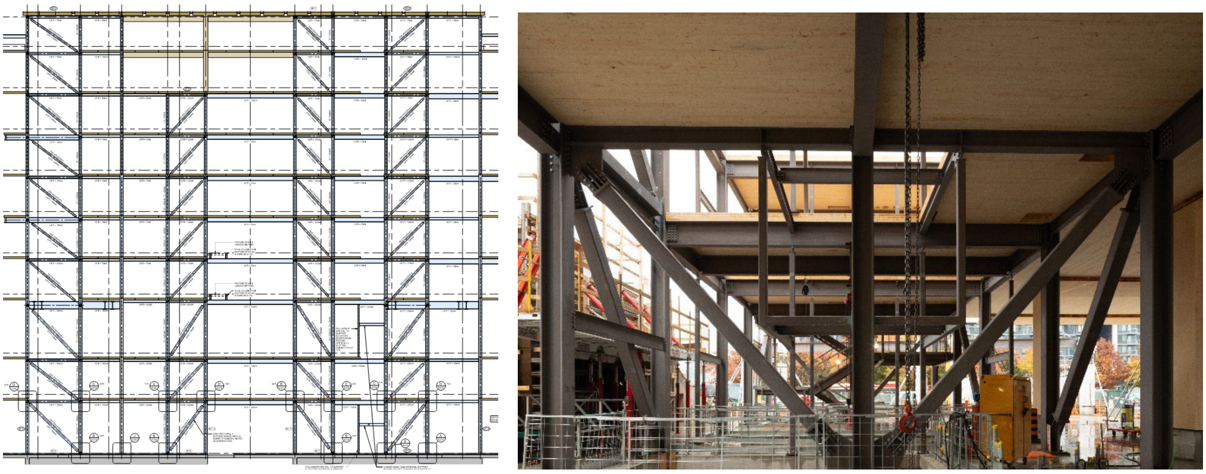
Figure 5: Typical floor to column connection- schematic (a), photo from mock-up (b)

The CLT floor panels on Level 9 support additional loads, including those from the green roof and snow. However, public access to the green roofs on the north and south sides of Level 9 is restricted. For the south green roof, additional double glulam beams are placed under each TCC band to support the column loads above. The columns above, which partially support Level 10 and the roof panels, are anchored to the top of the TCC bands at the mid-span, leading to the termination of the typical «wallumns» at this level. As the building extends to Level 10 and the upper roof, the «wallumns» transition into smaller 430 x 456 mm glulam columns.

On Level 10, a 7-ply CLT deck with a 50 mm non-structural concrete topping spans north-south between glulam purlins supported by the glulam columns below. The structural steel elements in the core area remain consistent with the floor below, and all components within this space are designed for a 2-hour fire rating. The same glulam columns continue upward to support the glulam roof purlins, which consist of 7-ply CLT roof panels spanning between them, following the roof's slope. At this level, there is no concrete topping, primarily for envelope considerations. The west and east sides of the upper roof feature side roofs made of 7-ply CLT panels supported by a series of steel purlins. In the center of these side roofs, sizable louvered openings are planned to function as solar chimneys.

2.3. Lateral System

The primary strategy for achieving lateral stability consists of a long central core of steel-braced frames coordinated with stairs, elevators, and service spaces (Figure 6). While various timber options, such as CLT shear walls and glulam braces, as well as a concrete core system, were considered, a steel-braced system emerged as the preferred choice for several reasons. Firstly, steel braces offer a slimmer profile than glulam equivalents, providing greater flexibility for architectural design and accommodating essential services. Additionally, the steel-braced cores add ductility and overstrength to the structure, improving its efficiency as a seismic force-resisting system compared to timber alternatives. This choice suits the project well due to decentralized mechanical rooms that minimize the need for wall penetrations. Moreover, steel components can be installed alongside timber components and possibly pre-assembled into larger units for swift erection. These steel braces are designed using Hollow Structural Sections (HSS), with various wide-flange sections for beams and columns within the cores, requiring no additional fire protection. The design accounts for both wind and seismic forces in east-west and north-south directions.



(a) (b)

Figure 6: a) Steel bracing for lateral system, b) construction photo of steel bracing

3. Testing Program

3.1. TCC Slab Band Materials

The TCC slab bands incorporate 245 mm thick E1M5 grade 7-ply CLT panels from Nordic, manufactured in accordance with PRG320. The slab bands were reinforced at two levels (high level: 9 rows of screws are spaced at 75 to 225 mm and low level: 6 rows screws are spaced at 300 mm). The 35 MPa concrete topping is 150 mm thick and was reinforced with 10M longitudinal rebar spaced at 150 mm. Three types of composite connectors were investigated: Type A –ø11×250 mm fully threaded self tapping screws, installed at an angle of 45º; Type B –2100×75×6.4 mm steel plates hammered into 7mm saw kerfs; and Type C – HBV- 2 mm thick and 90 mm deep (45 mm into CLT) perforated plates were glued into a 3 mm kerf using a polyurethane-based adhesive (Figure 7).

a) b) c)

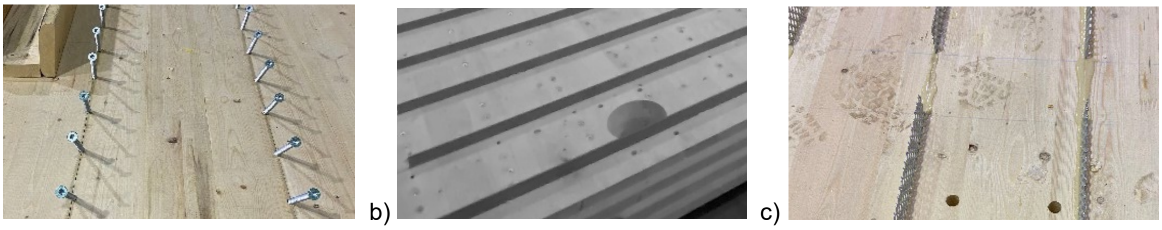


Figure 7: Slab band connectors a) Type A– fully threaded STS, b) Type B–steel kerf plates, and c) Type C– HBV

3.2. Connector Testing

A total of 18 small-scale shear tests were conducted. These tests investigated the capacity, stiffness, and failure mechanisms of steel kerf plates as TCC shear connectors. These steel kerf plates, measuring 6.4 mm thick and 200 mm in width, were installed within the CLT panel, precisely fitted into a 7 mm wide saw kerf with a 5^0-degree back bevel, as shown in Figure 8. The plate embedment depths into CLT were varied- i) 35 mm deep into the first layer of CLT, ii) 70 mm deep into the second layer of CLT, and iii) 90 mm deep partially into the third layer of CLT. The specimens were tested under in-plane shear by rotating the specimens at 12° similar as per EN-408.

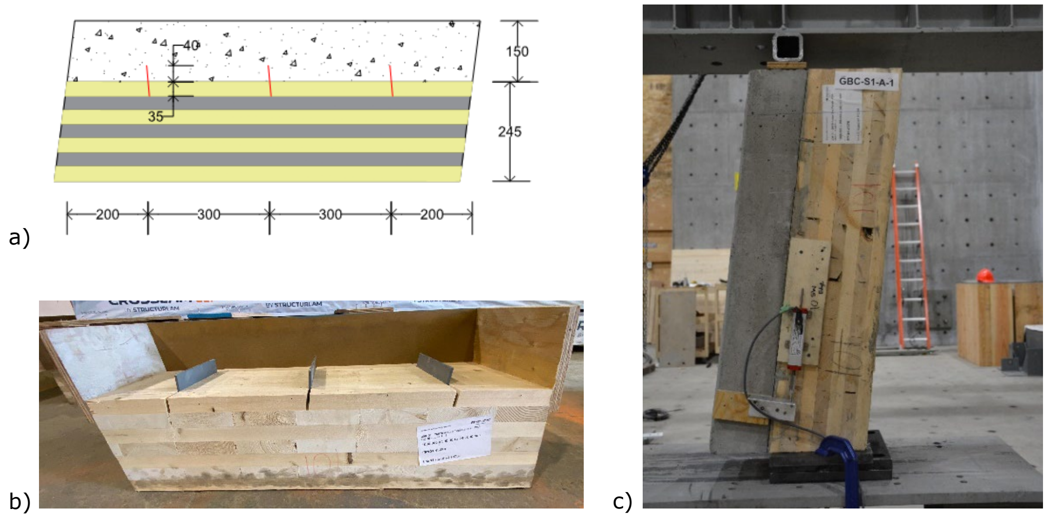


Figure 8: small scale tests with steel plates (a) & (b), photo of a small-scale test (c)

3.3. Half-Scale Slab Band Testing

The shear capacity and failure patterns in high-shear zones, as well as the performance of TCC floors with various shear connectors were evaluated through half-scale testing of CLT panels and TCC slab bands. Test series S2 encompassed 5 m long CLT panels, tested for two replicates each of unreinforced, low-reinforced, and high-reinforced CLT panels, as shown in Figure 9. Series S3-S5 were 5 m long CLT panels with a 150 mm concrete topping.

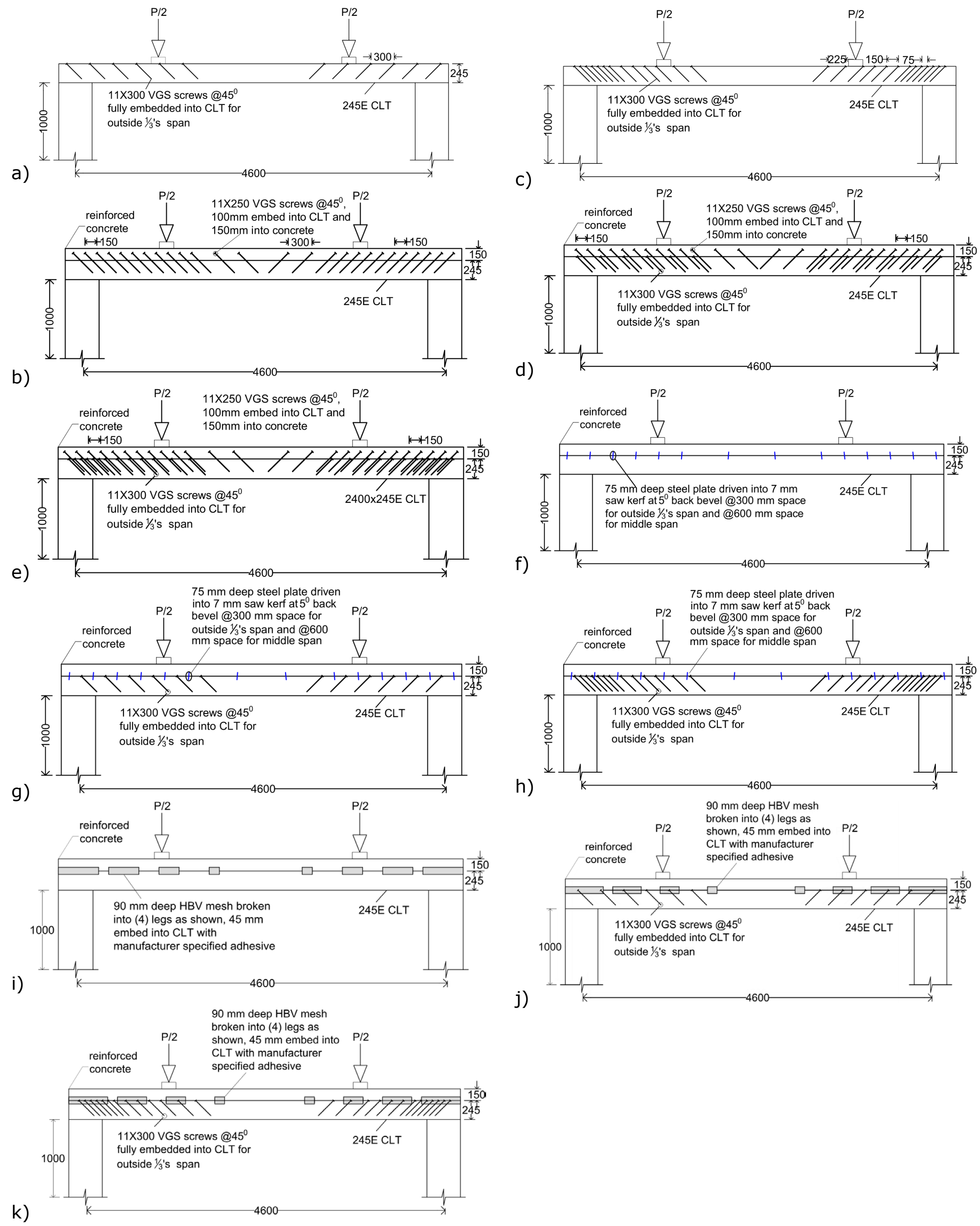


Figure 9: Half-span: series S2 (a) low- and (b) high-reinforced panels; series S3 (c) unreinforced, (d) low- and (e) high-reinforced panels; series S4 (f) unreinforced, (g) low- and (h) high-reinforced panels; series S5 (i) unreinforced, (j) low- and (k) high-reinforced panels

Three connector layouts were deployed in the half-span TCC configuration, illustrated in Figure 7: Type A utilized ø11×250 mm STS connectors installed at a 45º angle; Type B incorporated 2100 mm long and 75 mm deep steel kerf plates, positioned at 300 mm intervals for the outer one-third spans and 1,000 mm for the middle span; and Type C featured HBV mesh connectors with a depth of 90 mm. Each of the half-span TCC floor test series comprised two unreinforced, two low-reinforced, and two high-reinforced specimens, totaling 18 test specimens in all.

3.4. Full-Scale Slab Band Testing

Similar to half-scale testing, all full-scale specimens consisted of 245 mm thick, 7-ply CLT panels with 150 mm concrete topping. These specimens were connected using self-tapping screws, steel kerf plates, and HBV shear connectors (classified as Type A to C) and tested under four-point bending (series S6) and torsional loading (series S7), as depicted in Figure 10. Series S6-S7 comprised 9.6 m long low-reinforced CLT panels with 150 mm concrete topping, featuring three specimens for each connector type, and the connector installation layout can be seen in Figure 9. As with the half-scale specimens, the loads in series S6 were applied at one-third points employing two 500 kN and two 250 kN actuators and distributed along the third-point lines using spreader beams. The torsional specimens were tested under edge loading conditions, as shown in Figure 11, utilizing one 500 kN and two 250 kN actuators. It is worth noting that all full-scale slab band specimens in S6 and S7 were low-reinforced. A total of 6 and 3 TCC slab bands from series S6 and S7, respectively, were tested, with two replicates of S6 and one replicate of S7 for each connector type.

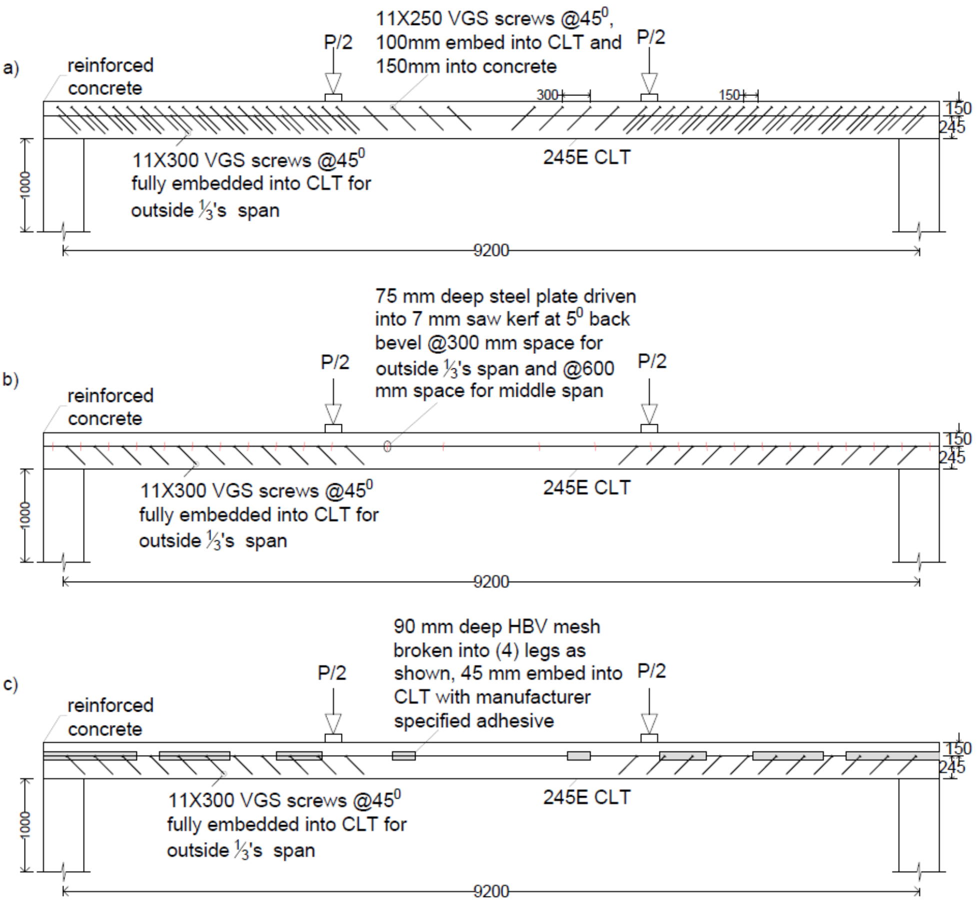


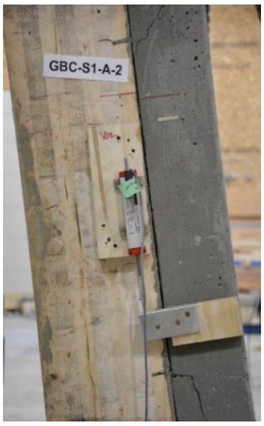
Figure 10: Full-span CLT panels: series S6 and S7 cross-section with: a) Type A, b) Type B and c) Type C TCC connectors

a) 
b) 

Figure 11: Photo of full-span 4-point bending test (a) and torsional test (b)

3.5. Test Results & Failure Mechanisms

Connector test results:
The connector performance was evaluated from the small-scale tests. As shown in Figure 12, the typical failure patterns revealed distinct post-yield behaviours for connectors with varying embedment depths. Specifically, Connector Type S1-A, featuring a 35 mm embedment depth, displayed the highest deformation capacity, while Connector Type S1-C, with a 90 mm embedment, exhibited the lowest deformation capacity, accompanied by a sharp reduction in load after reaching its peak. The results indicated that embedding kerf plates beyond the first layer of CLT resulted in reduced deformation capacity. Notably, the stiffness characteristics observed at both serviceability and ultimate loads were similar for Type S1-A, Type S1-B, and Type S1-C connectors. In conclusion, deeper embedment beyond the first CLT layer did not yield any enhancement in connection performance. Consequently, the full-scale floor specimens for testing were fabricated using Type S1-A steel plate connectors.

a) 

b) 

c) 

Figure 12: Photo of full-span 4-point bending test (a) and torsional test (b)

Half-Scale Test Results:
Figures 13a-c illustrate common failure patterns observed in series S2, which involves bare CLT panels. In unreinforced panels, failure occurred suddenly and in a brittle manner due to rolling shear near the support, with an average load of 885 kN. In contrast, the low-reinforced panels also experienced shear failure, but at a higher average load of 968 kN.

Figures 14a-c show the typical failure patterns in slab bands with screws in series S3. Unreinforced specimens failed in shear, at an average load of 1,445 kN. One of the low-reinforced panels experienced shear failure at an average load of 1,487 kN and the other did not fail during 4-point bending tests but reached actuator capacities, therefore, after retesting under 3-point loading, it ultimately failed in bending at mid-span. Similarly, both high-reinforced slabs only failed after being retested in a three-point bending configuration with an average load of 1,968 kN. Figures 15a-c show the failure patterns in series S4 for the slab bands with kerf plates. The unreinforced specimens failed in shear, at an average load of 1,482 kN. Both low-reinforced and high-reinforced slabs failed in mid-span bending after being retested in a three-point loading setup, with average loads of 2,586 kN and 2,441 kN, respectively. Lastly, Figures 16a-c show the failure patterns in series S5, which includes slab bands with HBV. All the S5 TCC slab bands with HBV failed in shear near the support, with average loads of 1,261 kN, 1,316 kN, and 1,493 kN, respectively.

a) 

b) 

c) 

Figure 13: Typical failure in series S2 (bare CLT):
a) unreinforced, b) low-reinforced, and c) high-reinforced specimens

a) b) c)

Figure 14: Typical failure in series S3 with screws connectors:
a) unreinforced, b) low-reinforced, and c) high-reinforced specimens

a) b) c)

Figure 15: Typical failure in series S4 with steel plate connectors:
a) unreinforced, b) low-reinforced, and c) high-reinforced specimens

a) b) c)

Figure 16: Typical failure in series S5 with HBV connectors:
a) unreinforced, b) low-reinforced, and c) high-reinforced specimens

Full-Scale Test Results:

Typical failure patterns of the full-scale TCC slab bands in series S6 are illustrated in Figure 17. The slab bands with screws composite connector failed in bending at mid-span at an average load of 706 kN and an estimated bending moment capacity of M_{max} = 1,100 kNm. Similarly, slab bands with Type B kerf plate connectors also failed in mid-span bending, however, at 47% higher loads compared to Type A screw connectors. The average failure load was observed at 1,040 kN and the estimated bending capacity of M_{max} = 1,619 kNm. The TCC slab bands with Type C composite connector HBV failed in connector shear at an average load of 670 kN which was the lowest among the three connector types and the estimated bending capacity was M_{max} = 1,043 kNm. Although HBV connectors in TCC slab bands failed initially, the final subsequent failure occurred at mid-span due to bending.

Typical failure patterns in series S7 under torsional loading are illustrated in Figure 18. The TCC slab bands with Type A screws composite connector reached first failure at 285 kN, and a maximum load of 442 kN at a deflection of 255 mm. The TCC slab bands with Type B kerf plates composite connector reached first failure at 234 kN and a maximum load of 489 kN at a deflection of 261 mm. The TCC slab bands with Type C HBV reached first failure at 257 kN, and a maximum load of only 413 kN at a deflection of 159 mm. Under torsional loading in series S7, the first failure was always triggered by the pull-out of the glued-in rod from the column support followed by failure in bending at mid-span on the loaded side at ultimate loads (Figure 18a). Therefore, in the final project design phase, all glued-in rods are capacity protected using 15 ø16×400 mm glued-in threaded rods instead of 12 ø 16×250 mm rods used in tests.

Figure 17: Typical series S6 failures under 4-point bending on full-scale TCC slab bands with:
a) screw connectors, b) kerf plates connectors, and c) HBV connectors

Figure 18: Typical series S7 failures under torsional tests:
a) bending failure on edge, b) panel uplift, and c) glued-in rod failure

4. Conclusion

Fast + Epp has undertaken a comprehensive and integrated design approach for one of Toronto's tallest timber buildings. A notable feature of this innovative design is the «slab banded» gravity frame, which not only offers architectural flexibility but also facilitates unobstructed mechanical distribution. This timber gravity system is seamlessly integrated with a steel-braced frame core, enabling the erection of a prefabricated «kit of parts» in conjunction with the envelope system.

Extensive testing was carried out to assess the performance and efficiency of TCC slab bands, utilizing various composite connectors. The incorporation of screw reinforcement within CLT panels significantly enhanced their shear capacity. Furthermore, some heavily reinforced specimens exhibited a shift in failure modes from shear to bending failure. The addition of a 150 mm concrete topping, connected in a composite fashion to the CLT panel, increased the out-of-plane shear capacity by as much as 167% compared to bare CLT panels. In some TCC specimens, weak-axis shear failure was observed due to warping over the glulam columns. To mitigate this issue, diagonal reinforcing screws were introduced in the transverse direction at the end of the slab bands, a crucial design modification was implemented. Among various options, TCC with steel kerf plates emerged as the most cost-effective choice, with the highest capacity and stiffness, as validated by pricing from multiple suppliers. Consequently, this solution was adopted for the final design.

The structural testing program has not only paved the way for a cost-effective timber composite system but has also provided invaluable insights to the engineering team. The project is close to the end of its construction phase, with a targeted occupancy in 2024.

Figure 19: Construction photo of a typical floor plate