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LIMBERLOST PLACE: A 10-STOREY SLAB-BANDED STRUCTURE

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ABSTRACT: The $150M Limberlost Place (previously known as "The Arbour") is a 10-storey, 16,250 m^2, exposed tall wood building located on George Brown College's waterfront campus in Toronto, Canada. This project will serve as an educational hub for George Brown College, housing the Tall Wood Research Institute, a childcare centre in addition to a significant amount of teaching and social spaces for their architectural program. Fast + Epp developed an innovative large span beamless structural system that is comprised of CLT timber-concrete composite (TCC) slab bands with perpendicular CLT infill panels supported on glulam columns. This long-span flat plate system allows for flexibility in architectural programming and unobstructed mechanical distribution. An extensive testing program on the TCC slab bands was completed at the University of Northern British Columbia in 2020, which has provided valuable engineering information to the design community for future tall wood projects and composite systems. To facilitate innovation, the project received funding through the National Resources Canada Green Constriction Wood Program, alongside other partners.

KEYWORDS: Timber concrete composite, cross-laminated timber, connections, tall timber, slab band

1 INTRODUCTION

The 10-storey 52.5m tall Limberlost Place is poised to transform Toronto's skyline with the construction of a mass timber tall wood, net-zero carbon emission building (Figure 1). The building name was inspired by a nearby forest the "Limberlost Forest and Wildlife Reserve", Huntsville, ON, Canada. The building is located on George Brown College's waterfront campus in the East Bayfront district of Toronto, Canada. The building will facilitate classrooms and lecture halls, and host The Tall Wood Institute.

Figure 1: Rendering of Limberpost Place

To reflect the purpose of the building and to develop economic and environmental structural solutions, a holistic design approach with mass timber as the primary structural material was chosen. The building has a footprint of 62m×37m and was designed for load conditions as required by the 2017 edition of the Ontario Building Code (OBC) [1] and the 2015 edition of the National Building Code of Canada (NBCC) [2]. The 9.2m long timber concrete composite (TCC) slab bands will be exposed from the underside in some locations, for architectural expression. These floors will be supported on glulam columns from the ground floor to the upper roof (Figure 2). The typical glulam columns were designed to provide additional distribution areas for shear stresses and reduce the weak-axis bending in the panels. This project is one of the first tall timber buildings to proceed with "assembly" occupancy in Canada – where many of the existing tall timber buildings are for commercial or residential occupancies.

The structure was designed for a two-hour fire event, with all structural timber designed for full exposure. A char analysis was completed with the provisions given in Annex B of CSA O86 [3]. In addition to this char analysis, supplemental calculations were also undertaken using 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 also met the requirements to achieve a two-hour fire-resistance rating and through detailing with drywall encapsulation. The fire rating and associated alternative solutions were a

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significant part of this project, undertaken by GHL Code Consultants Ltd, alongside CHM Fire Consultants Ltd.

2 SUPERSTRUCTURE

The ground floor structure consists of a 300 mm thick reinforced concrete slab supported on concrete columns and walls below. The transition from concrete columns to glulam columns is made 500 mm above the ground level slab, jacketing the steel column bases in concrete. From level 2 to 8, the TCC slab bands serve as the primary floor structure and eliminate the need for deep glulam beams, providing more headroom and space for mechanical and electrical components. The 244 mm thick 7-ply CLT panels act compositely with 150 mm concrete topping spanning 9.2 m in the north-south direction and support the non-composite 191 mm thick 7-ply CLT infill panels between them, as shown in Figure 2a. A photo of the slab bands and infill panels can be seen in Figure 2b.

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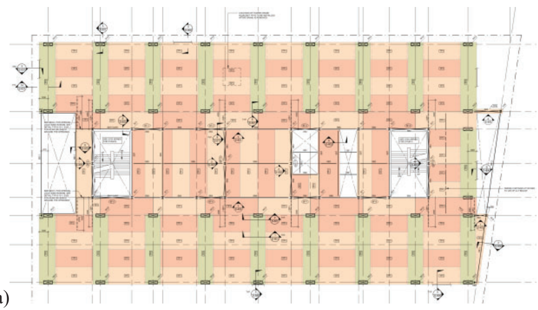

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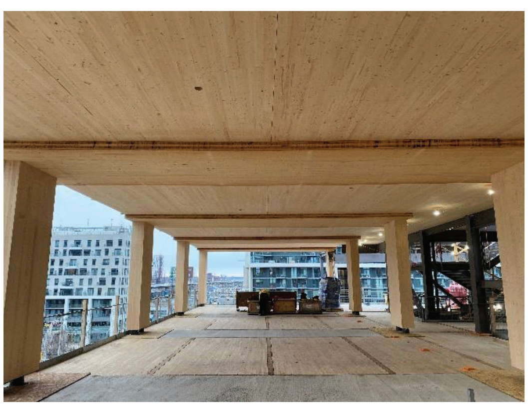
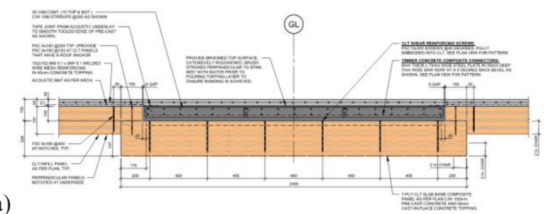


Figure 2: *a) Typical floor plan, b) Typical slab bands.*

The TCC slab bands are supported by 422 × 1210 mm glulam columns, designed and positioned beneath the slab-bands to resist the effects of unbalanced loading. The column connection (Figure 3) is configured to provide direct load transfer between the vertical elements rather than transmitting forces through the TCC floor panels. Glulam columns arrived on-site with a steel connecting plate and Hollow Structural Section (HSS) stubs, secured to the end-grain of the column with glued-in rods. The glulam column above had a a similar connection with smaller diameter HSS stubs allowing a ‘sleeve in’ connection. Stubs are connected using bolts, which allow for simple installation and act as a tension connection in the extreme event where a column below is eliminated, according to progressive collapse principles. CLT floor panels were notched around the HSS tubes and bear directly on the column below.

The CLT roof panels support additional loading from the weight of the green roof and the snow, however public access is restricted at those green roofs on the north and south side of Level 9, in order to match the design loading with that of the floors below. At the south green roof, however, there will be additional double glulam beams underneath each TCC band to support the column loads above. The columns above, which partially support the roof, are anchored to the top of the TCC bands at the mid-span. The typical “wallumns” are terminated at this level. Where the building continues up to Level 10 and the upper roof, the “wallumns” transition into smaller 430 x 456 mm glulam columns.

At Level 10, a 245 mm thick 7-ply CLT deck spans north-south between glulam purlins abd steek beams, supported by columns below. The structural steel members in the core area are the same as the floor below. All structural components within this space have been designed for a 2hr fire rating. The same glulam columns supporting Level 10 will continue up to support the roof. There, 245 mm thick 7-ply CLT panels span between purlins, along the slope of the roof. On the west and east sides of the upper roof, side roofs will also be made of 7-ply CLT panels supported on a series of steel purlins. In the middle of these side roofs, large louvred openings are to be created to serve as solar chimneys.


a)


b)

Figure 3: *a) Typical slab band section, b) Mock-Up*

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3 LATERAL SYSTEM

The primary means of lateral stability consists of a long central core of concentric limited ductility steel braced frames coordinated with the stairs, elevators, and mechanical spaces in the center of the building (Figure 4). Several timber options, including CLT shear walls and glulam braced frames were explored, as was a conventional concrete core system. Ultimately, a steel braced frame system (Figure 4) was found to be the best fit for the project due to the following reasons. Firstly, the steel braces are much more slender than a glulam equivalent, allowing for greater flexibility for wall openings for architectural and required services. Secondly, the steel braced frame cores add more ductility and overstrength to the overall structure in comparison to a timber braced frame system, allowing for a more efficient seismic force-resisting system. Thirdly, the de-centralized mechanical rooms require a significant number of services into the 'core' space, meaning many holes in the walls which would be practically feasible. Fourthly, the steel components can be erected simultaneously with the timber components, resulting in a faster erection period sharing the hook of one crane. The steel braces HSS, while the beams and columns within the cores are various sizes of wide-flange sections. These steel components are concealed, thus requiring no further fire protection. The design of the steel core members is governed by wind versus seismic forces in both E-W and N-S directions.

a)

b)

Figure 4: *a) Steel braced frames, b) Steel braced frame erection sequence in 2-storey lifts.*

4 STRUCTURAL TESTING

A comprehensive experimental testing program was conceived and executed on the TCC slab bands to inform the project design. The CLT specimens were prepared by Structurlam, in Penticton, BC and the concrete for slab bands was poured by Datoff Bros Construction Ltd, Prince George. All tests were conducted at the University of Northern British Columbia's Wood Innovation and Research Laboratory in Prince George, BC.

4.1 MATERIALS

The tested CLT panels for the slab bands were 245 mm thick 7-ply, grade E1M5 [3]. The panels were manufactured as per PRG320 [4] with Spruce-Pine-Fir (SPF) 2100 Fb-1.8E machine-stress rated and No.3 grade for the major and minor strength axis laminations, respectively. Two-thirds of the slab bands were reinforced at two intensity levels (high level: 9 rows of screws spaced at 75 to 225 mm and low level: 6 rows screws spaced at 300 mm). The concrete topping on the slab bands is 150 mm with a minimum specified strength of 35 MPa. The concrete was reinforced with 10M longitudinal rebar spaced at 150 mm top and bottom, and stirrups were 10M spaced at 300 mm.

Three types of composite connectors were used. Type A: Fully threaded self-tapping screws (ø11×250 mm) installed at an angle of 45º; Type B: perpendicular steel plates 2100mm x 75mm x 6.4 mm (length x depth x thickness) hammered into a 7mm saw kerf, and Type C: parallel 90mm x 2mm HBV plates glued into saw kerfs using a polyurethane-based adhesive.

4.2 CONNECTOR TESTS

A total of 18 small-scale shear tests were conducted to investigate the capacity, stiffness, and failure mechanisms of steel kerf plates as TCC shear connectors with varying embedment depths. Steel kerf plates of 6 mm thick and 200 mm wide were installed in the CLT in a 7 mm wide saw kerf at 5^0 back bevel as shown in Figure 5a-c. The steel plate embedment depths into the CLT were varied: i) 35 mm deep, ii) 70 mm deep, and iii) 90 mm deep. The specimens were tested under in-plane shear by rotating 12°, similar to the procedure suggested in EN-408 [5]. The loads were applied according to EN-26891 [6] with a displacement-controlled protocol at a rate of 5 mm/min as shown in Figure 5d.

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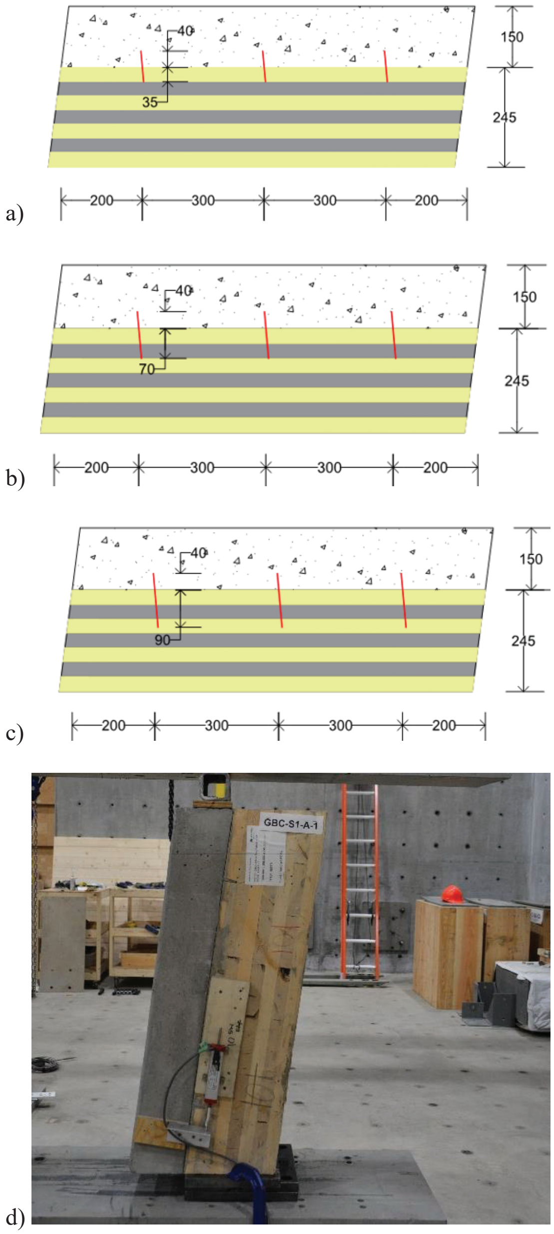

Figure 5: *a-c) Specimens including steel kerf plate embedment variations, d) Small-scale test setup.*

4.3 HALF-SCALE TESTS

The half-scale TCC slab bands were tested to assess their shear capacity and failure pattern at high shear zones with different levels of CLT shear reinforcements, to evaluate the performance of the TCC floors with different shear connectors and shear reinforcing. The test series are summarized in Table 1. The floors were connected to 430×1,178 mm glulam columns by 12-ø16×250 mm glued-in rods.

Table 1: Overview of Test Series

Series		ID	Reinf.	Connector	Test Type	#of tests
half-scale	S2	S2-UR	-	NA	B	2 each
		S2-HR	low			
		S2-FR	high			
	S3	S3-UR-A	-	Type A	B	2 each
		S3-HR-A	low			
		S3-FR-A	high			
	S4	S4-UR-B	-	Type B	B	2 each
		S4-HR-B	low			
		S4-FR-B	high			
	S5	S5-UR-C	-	Type C	B	2 each
		S5-HR-C	low			
		S5-FR-C	high			
full-scale	S6	S6-HR-A	low	Type A	B	2 each
		S6-HR-B		Type B		
		S6-HR-C		Type C		
	S7	S7-HR-A	low	Type A	T	1 each
		S7-HR-B		Type B		
		S7-HR-C		Type C		

Note: B – bending, T-torsion

Test series S2 is the control series, consisting of 5030 mm long raw CLT panels (no concrete topping) (Figure 6). Test series S3 to S5 (Figures 7 to 9) are TCC slab bands consisting of 5030 mm long CLT panels with 150 mm concrete topping. The three different connector layouts (Type A to C) are shown in Figure 7 to Figure 9. Two replicates from each unreinforced, low-reinforced, and high-reinforced CLT panels and TCC slab bands were provided, for a total of 6 raw CLT panels on the control side, and 18 TCC slab bands.

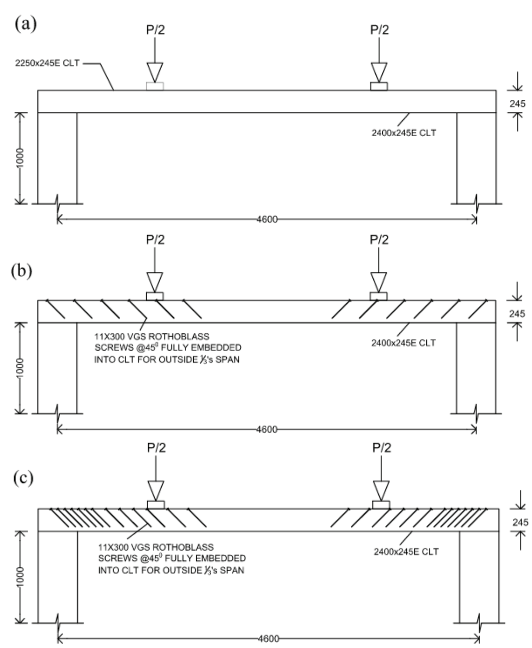

Figure 6: *Test series S2- half-scale raw CLT panels: a) unreinforced, b) low, and c) high reinforced CLT panels.*

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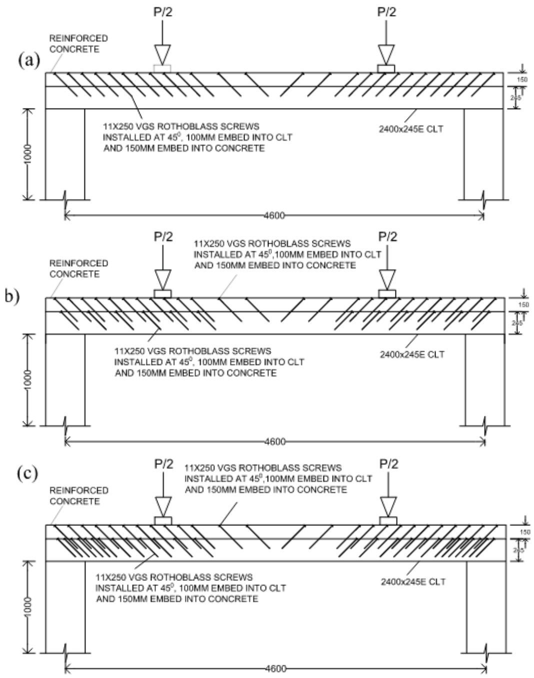

Figure 7: *Test series S3: half-scale TCC with screw connectors: a) unreinforced, b) low reinforced, and c) high reinforced CLT panels.*

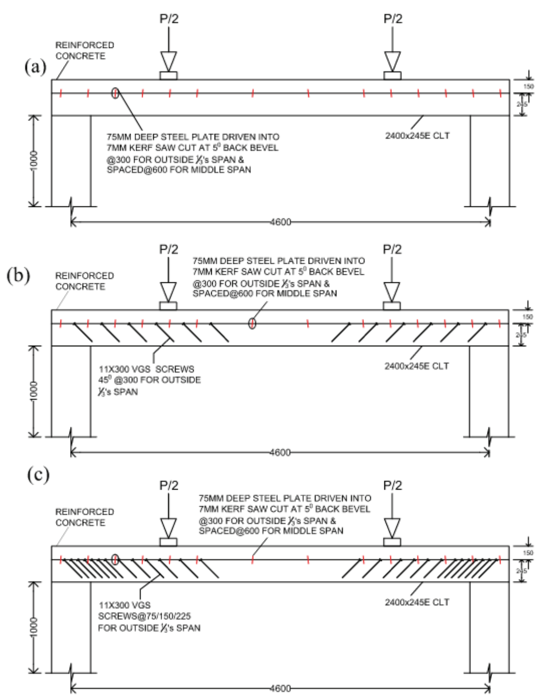

Figure 8: *Test series S4: half-scale TCC with kerf plate connectors: a) unreinforced, b) low, and c) high reinforced CLT panels.*

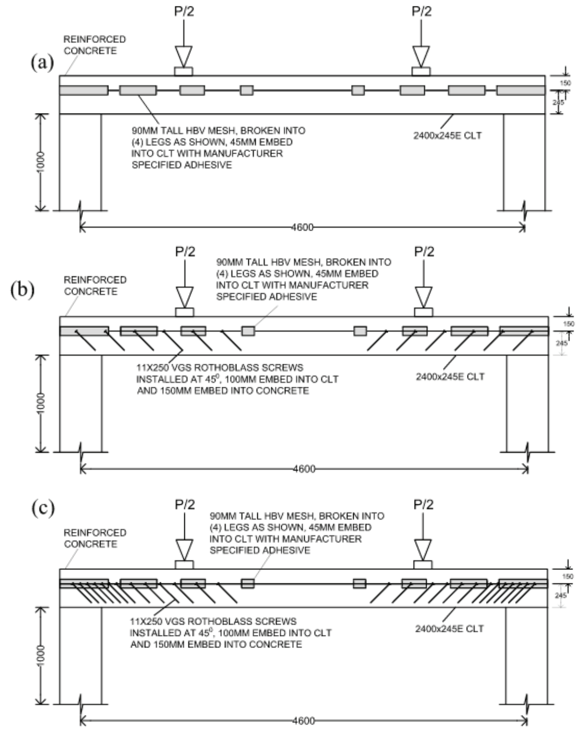

Figure 9: *Test series S5: half-scale TCC with HBV connectors: a) unreinforced, b) low, and c) high reinforced CLT panels.*

The tests on half-scale specimens were conducted under 4-point bending as shown in Figures 10a and 10b on raw CLT panels (Series S2) and TCC slab bands (Series S3-S5), respectively. The loads were applied at one-third points using two 500 kN and two 250 kN actuators for a total maximum load of 1,500 kN, and were distributed at the third-point lines using steel spreader beams. When the specimens did not fail after reaching the 1500 kN maximum actuator load, the spreader beam was moved to mid-span to re-test the specimen under three-point loading. All specimens were tested under displacement-controlled loading at a constant rate of 15 mm/min.

Figure 10: *Schematic of 4-point bending tests.*

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4.4 FULL-SCALE TESTS

Similar to the half-scale, all full-scale specimens were comprised of 245 mm thick, 7-ply CLT panels with 150 mm concrete topping connected with self-tapping screws, steel kerf plates, and HBV shear connectors (Type A, B, C respectively, as laid out in Table 1). These specimens were tested under four-point bending (Series S6) and torsional loading (Series S7). Series S6-S7 consisted of 9.6 m long, low-reinforced CLT panels with 150 mm concrete topping, with three specimens of each connector type, as shown in Figure 11.

Similar to half-scale specimens, the loads in series S6 were applied at one-third points using two 500 kN and two 250 kN actuators, for a total of 1500kN maximum, and were distributed at the third-point lines using spreader beams. The torsional specimens were tested with edge loading using one 500 kN and two 250 kN actuators, for a total of 1500kN maximum. All specimens were tested under displacement-controlled loading at a constant rate of 15 mm/min. The CLT panels of all full-scale slab bands specimens in S6 and S7 were low-reinforced. Two replicates of S6 and one replicate of S7 from each connector types were tested, a total of 6 and 3 TCC slab bands from series S6 and S7, respectively.

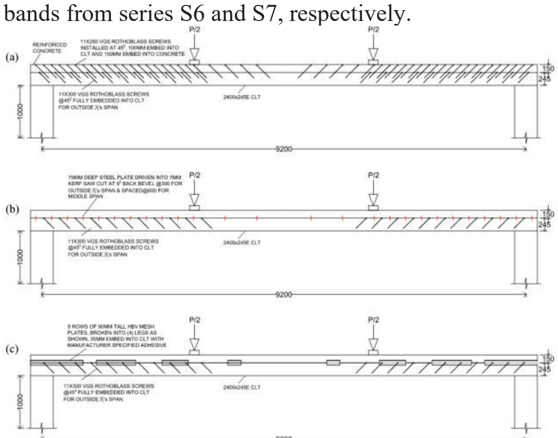
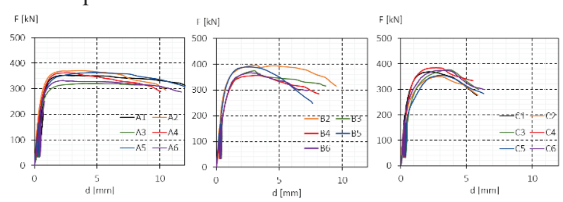


Figure 11: *Test series S6 and S7: full-scale TCC self tapping screw connections, kerf plates, and HBV.*

5 TEST RESULTS AND DISCUSSION

5.1 CONNECTOR TESTS RESULTS

From the small-scale tests, the connector performance was evaluated at the maximum load F_{max}, displacement at maximum load d_{Fmax}, serviceability stiffness K_{ser}, and ultimate stiffness K_u. The mean values of various metrics from the in-plane tests are listed in Table 2. The detailed analysis results were reported in a previous publication [8]. The load-deflection from small-scale series S1 tests are shown in Figure 12. Typical failure patterns are shown in Figure 13. The post-yield behaviours of the connectors with three different embedment depth were different. Connector type S1-A with 35 mm embedment resulted the highest deformation capacity, while connector Type S1-C with 90 mm embedment resulted the lowest deformation capacity with a steep decrease in load after reaching its maximum. Type S1-A connectors had the lowest load-carrying capacity, yield, and ultimate strengths (F_{max} = 350 kN), while Type S1-B and Type S1-C connectors' capacities were slightly higher, i.e. on average 7% and 5%, respectively higher compared to Type S1-A. All connector types had started yielding at similar range deformations of between 0.8 to 0.9 mm. Results showed that there are some reductions in deformation capacity when the kerf plates were embedded beyond the first layer of CLT. The stiffnesses observed both at serviceability and ultimate loads for Type S1-A, Type S1-B connectors, and Type S1-C connectors were similar (Table 2). In sum, with deeper the embedment depth beyond first layer of the CLT panel did not provide any improvement in the connection performance, therefore, the floor specimens for the full-scale testing were manufactured with Type S1-A steel plate connectors.



(a) (b) (c)

Figure 12: *Typical failure pattern in small-scale tests: a) S1-A, b) S1-B, and c) S1-C*

a) b) c)

Figure 13: *Typical failure pattern in small-scale tests: a) S1-A, b) S1-B, and c) S1-C*

Table 2: Results from series S1-small-scale

Series	CLT embedment	#of tests	F_{max} [kN]	d_{Fmax} [mm]	K_{ser} [kN/mm]	K_u [kN/mm]
S1-A	35 mm deep	6	350	3.3	416	444
S1-B	70 mm deep	6	376	2.9	395	354
S1-C	90 mm deep	6	368	3.0	438	453

5.2 HALF-SCALE TESTS RESULTS

Half-Scale CLT Panels

The load-deflection curves from series S2 on CLT panels shows three segments until failure, ascending elastic and elastic-plastic segments, and descending segment from peak loads to failure (Figure 14). Typical failure patterns from series S2 in unreinforced, low and high-reinforced panels are shown in Figure 15. The unreinforced CLT panels failed in a sudden brittle manner due to rolling shear failure near the support (Figure 15a) at an average load of 885 kN. The low-reinforced panels also failed in shear (Figure 15b), at a higher load of on average 968 kN. The failure mechanism in the high-reinforced panels pushed from shear to bending (Figure 15c).

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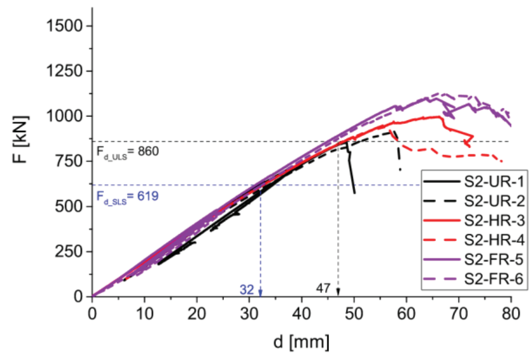

Figure 14: *Load-deflection curves for series S2- CLT panels*

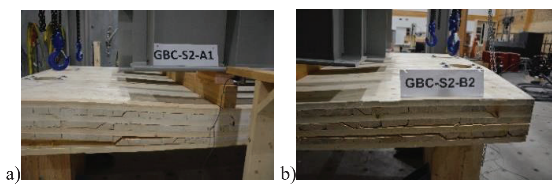

Figure 15: *Typical series S2 failures on CLT: a) unreinforced, b) low-reinforced specimens,*

Half-Scale TCC Slab Bands

Typical failure pattern in slab bands with screws are shown in Figure 16. The unreinforced specimens S3-UR failed in shear as seen in Figure 15a at an average load of 1,445 kN. One of the low-reinforced panel S3-HR slab band failed in shear (Figure 17b) at an average load of 1,487 kN. The other low-reinforced slab did not fail in 4-point bending tests as it reached actuators capacities, therefore, after retest under 3-point loading, it failed in bending at mid-span (Figure 17c). Similarly, both high reinforced slab S3-FR only failed after being re-tested in a three-point bending (Figure 17d) at an average load of 1,968 kN.

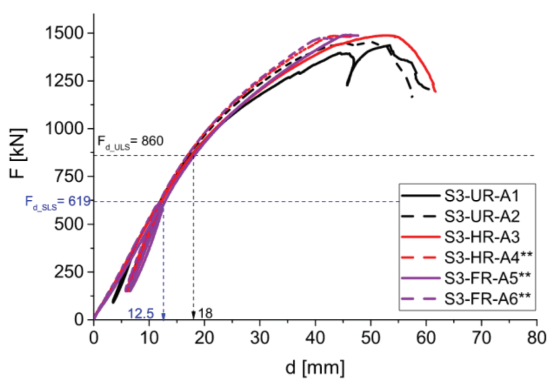

Figure 16: *Load-deflection curves for series S3- half-scale TCC bands with screw connectors*

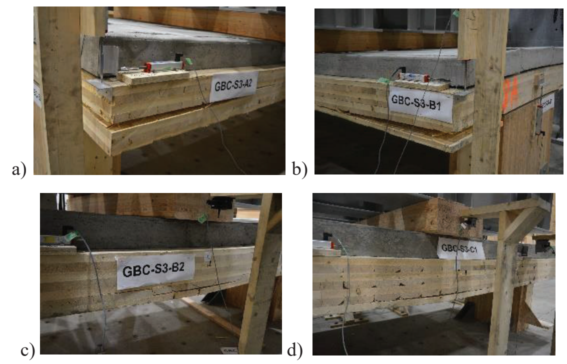

Figure 17: *Typical series S3 failures on TCC slab bands with screws connectors: a) unreinforced, b-c) low-reinforced specimens, and d) high-reinforced specimen*

Failure patterns in series S4 slab bands with kerf plates are shown in Figure 18. The unreinforced specimens S4-UR failed in shear as seen in Figure 19a at an average load of 1,482 kN. Both low S4-HR and high reinforced S4-FR slabs were failed at mid-span bending after being re-tested in a three-point loading (Figure 19 b-c) at an average load of 2,586 kN and 2,441 kN, respectively.

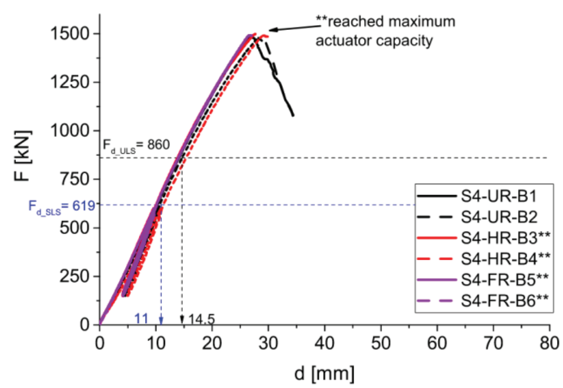

Figure 18: *Load-deflection curves for series S4- half-scale TCC bands with kerf plates connectors*

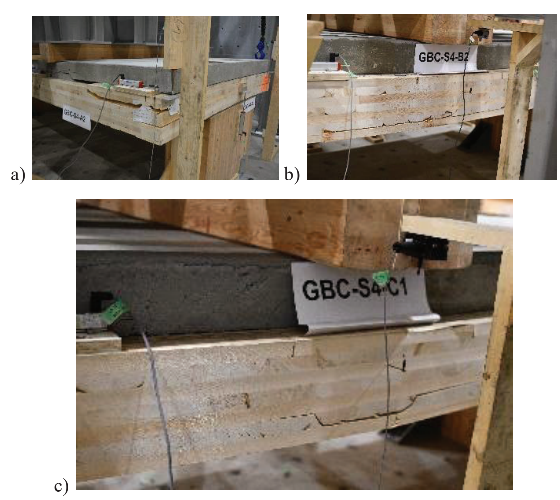

Figure 19: *Typical series S5 failures on TCC slab bands with HBV connectors: a) unreinforced, b) low-reinforced specimens, and c) high-reinforced specimens*

Failure patterns in series S5 slab bands with HBV are shown in Figure 20. All series S5 TCC slab bands with HBV failed in shear (Figure 21 a-c) at support at an average load of 1,261 kN, 1,316 kN and 1,493 kN, in S5-UR, S5-HR, and S5-FR, respectively.

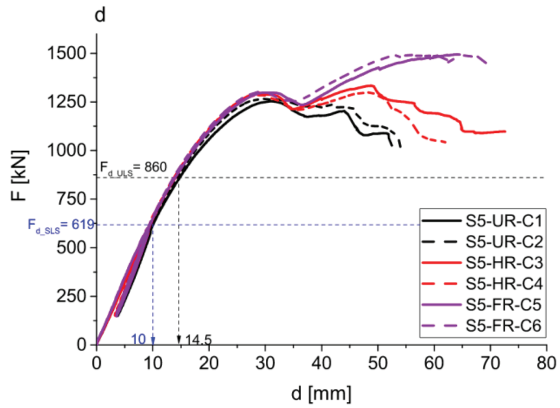

Figure 20: *Load-deflection curves for series S5- half-scale TCC bands with HBV connectors*

a) b)

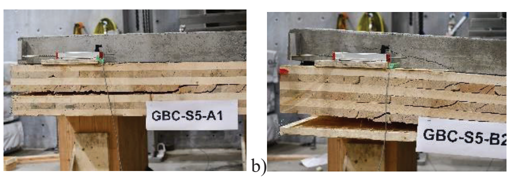

Figure 21: *Typical series S5 failures on TCC slab bands with HBV connectors: a) unreinforced, b) low-reinforced specimens*

5.3 FULL-SCALE TESTS RESULTS

The mid-span load-deflection curves of the full-scale TCC slab bands in series S6 are shown in Figure 22 and typical failure patterns are illustrated in Figure 23. The slabs bands with screws composite connector failed in bending at mid-span at an average load of 706 kN and estimated bending moment capacity of M_{max} = 1,100 kNm. Similarly, slab bands with Type B kerf plates connector also failed in mid-span bending, however, at 47% higher loads compared to Type A screw connectors. The average failure load was observed 1,040 kN and 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.

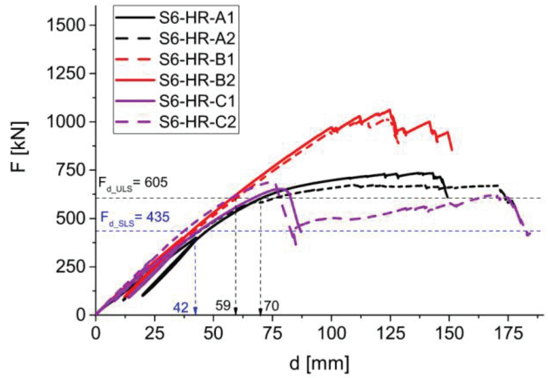

Figure 22: *Load-deflection curves for series S6- full-scale TCC bands -under 4-point bending tests*

a) b) c)

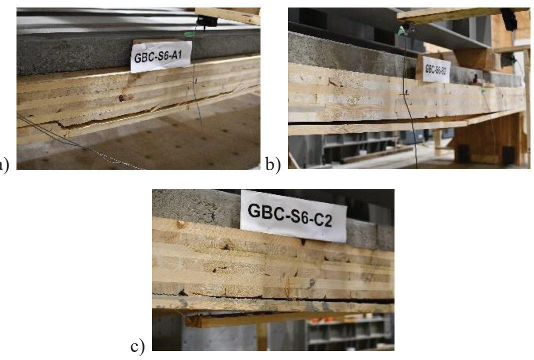

Figure 23: *Typical series S6 failures under 4-point bending on full-scale TCC slab bands with: a) screw connectors, b) kerf plates connectors, and c) HBV connectors*

The mid-span load-deflection curves of the full-scale TCC slab bands in series S7 under torsional loading are shown in Figure 24 and typical failure patterns are illustrated in Figure 25. 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 as shown in Figure 25c followed by failure in bending at mid-span on the loaded side at ultimate loads (Figure 25a). Therefore, in the final project design phase, all glued-in rods are capacity protected using 15 ø16×400 mm glued-in threaded rods were used instead of 12 ø 16×250 mm rods used in tests, see Figure 25c.

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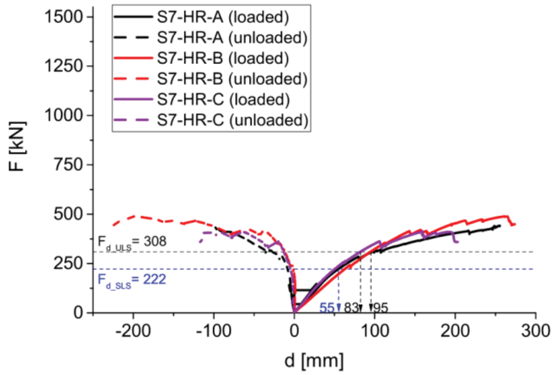


Figure 24: *Load-deflection curves for series S7- full-scale TCC bands -under torsional tests*

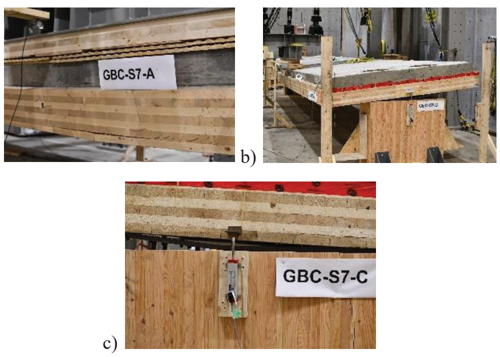
a) b) c)


Figure 25: *Typical series S7 failures under torsion on full-scale TCC slab bands with: a) screw connectors, b) kerf plates connectors, and c) HBV connectors*

6 CONCLUSIONS

Fast + Epp initiated an ambitious design approach to one of Toronto's tallest timber buildings. The innovative 'slab banded' gravity system allows for flexibility in architectural programming and unobstructed mechanical distribution. By coupling this timber gravity system to a steel braced frame core, the superstructure can be erected as a prefabricated 'kit of parts', alongside the envelope system.

The extensive testing program evaluated the performance and efficiency of TCC slab bands with various composite connectors. The screw reinforcement in the CLT significantly increased the shear capacity of the panels. The failure modes also shifted shear to bending failure in some heavy reinforcing specimens. The 150 mm concrete topping compositely connected to the CLT panel improved the out-of-plane shear capacity by up to 167% when compared to bare CLT panels. Weak-axis shear failure was observed in some TCC specimens due to warping over the glulam columns and this can be avoided by adding diagonal reinforcing screws in transverse direction at the end of slab bands, which was incorporated in the final design phase (as seen in Figure 26a). TCC with steel kerf plates exhibited the highest capacity and stiffness and were priced by multiple suppliers to be the most economical option, therefore, final design was moved forward with this solution (Figure 26b). Torsion tests showed pull-out failure of glued-in rods, which must be avoided, so lengthening of the rods and increasing the number of rods was deployed in final design.

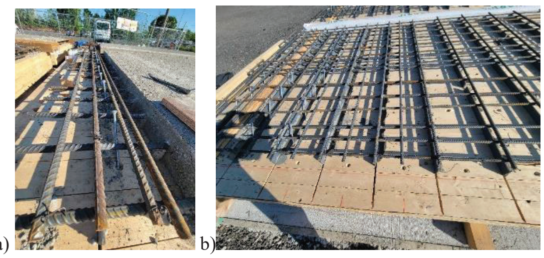
a) b)


Figure 26: *a) Diagonal screws at 45^0 in weak axis, b) typical slab band reinforcement before concrete pour*

The structural testing program allows a low-cost timber composite system, provide invaluable design information to the engineers. It is currently under construction, with structural completion targeted for late 2023.

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