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EXPERIMENTAL INVESTIGATIONS OF FULL-SCALE CROSS-LAMINATED TIMBER CONCRETE COMPOSITE SYSTEMS

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Abstract: Experimental investigations on full-scale Cross-Laminated Timber (CLT)-concrete composite floor systems with various composite connectors are presented in this paper. The stiffness, strength, and failure modes of different TCC systems were experimentally evaluated. The 9.2 m long and 2.4 m wide TCC floor segments were comprised of 245 mm thick, 7-ply CLT panels with 150 mm concrete topping connected with three types of shear connectors: i) self-tapping screws, ii) steel kerf plates, and iii) proprietary glued-in Holz-Beton-Verbund (HBV) plates. In addition to the shear flow connectors, the CLT panels were reinforced using self-tapping screws. Six TCC floor segments were tested to failure under symmetric four-point bending and three TCC floor segments were tested under torsional bending by applying eccentric loading near the edge. The floor deformations at nine locations and connector slips at CLT-concrete interfaces at eight locations along the length of the floor were measured. The full-scale tests showed that the steel kerf plates exhibited the highest capacity and stiffness, and were the most economical solution compared to TCC systems with screws and proprietary HBV plates.

1 INTRODUCTION

Timber-Concrete Composite (TCC) can be an attractive alternative solution to replace conventional timber or concrete floors. TCC systems are generally comprised of timber elements (beam or panel) connected to a concrete layer by means of a shear connector. Deflection and vibration often govern the design of TCC floor systems. Previous studies have shown that TCC can overcome some of the inefficiencies associated with traditional reinforced concrete or light wood frame floors regarding strength, section depth, stiffness, and vibration performance [1,2]. The availability of panel-type engineered wood products such as CLT offers designers greater versatility in terms of architectural expression and structural performance. The efficiency of TCCs depends largely on the cross-sectional properties of timber and concrete as well as the shear connections between them. Currently, connectors ranging from low to high stiffness' are available. Some examples are a) self-tapping screws, b) dowelled shear keys, c) proprietary HBV, d) steel kerf plates, e) transverse notch connection. It should be noted that typical reinforced concrete floors really don't harness the full section efficiently; 60% of the gross cracked concrete section does not contribute to the total load resistance [3].

In this paper experimental investigations on a Cross-Laminated Timber (CLT)-concrete composite system with three different types of shear connectors: i) self-tapping screws, ii) steel kerf plates, and iii) proprietary glued-in Holz-Beton-Verbund (HBV) plates. The TCC floor systems were designed for "The Arbour" project, a 10-storey educational building for George Brown College, located in the East Bayfront district of Toronto, Canada. This building will facilitate classrooms, lecture halls, and host The Tall Wood Institute. To reflect the purpose of the building and to develop economic and environmental structural solutions, timber was chosen as the primary structural material. Each mass timber floor will be exposed from the underside and structural concrete topping will be added to achieve 9.2 m span conducive to institutional programming. These floors will be supported on glulam columns from the ground floor all the way to the upper roof.

The main goal from this research was the successful implementation of a low-cost and efficient TCC system for “The Arbour”. Therefore, the objectives of the experimental programs were set to investigate i) the contribution of shear between wood and concrete and ii) failure mechanisms and capacity of TCC floor systems.

2 EXPERIMENTAL INVESTIGATIONS

A comprehensive test program as shown in Table 1 was designed to investigate the parameters for the successful implementation of a low-cost and efficient TCC system in the Arbour.

Table 1: Description of Test Series

Series	ID	Description	Test Type	#of tests	L [mm]	b_c [mm]	b_t [mm]
S6	S6-HR-A	Full-scale, Type A TCC	Bending	2	9630	2200	2400
	S6-HR-B	Full-scale, Type B TCC	Bending	2	9630	2200	2400
	S6-HR-C	Full-scale, Type C TCC	Bending	2	9630	2200	2400
S7	S7-HR-A	Full-scale, Type A TCC	Torsion	1	9630	2200	2400
	S7-HR-B	Full-scale, Type B TCC	Torsion	1	9630	2200	2400
	S7-HR-C	Full-scale, Type C TCC	Torsion	1	9630	2200	2400

2.1 Materials

The TCC floors were comprised of 7-ply, 245 mm CLT with 150 mm concrete topping. The CLT panels were made from SPF (Spruce Pine Fir) lumber, with 2100 Fb-1.8E MSR grade stock for strong axis laminations and SPF No.3/Stud grade stock for weak axis laminations (Table 2). The floors are supported on 430×1178 mm glulam columns which are made of Douglas Fir and are 16c-E grade. The CLT panels were partial or half-reinforced (HR) with self-tapping screws (STS) Rothoblaas VGS FT 11×300 mm screws at 450 (Table 3). All mass timber products for the testing were supplied by Structurlam. The minimum specified strength of concrete was 35 MPa. Concrete cylinder compression tests were conducted on 7th day and 34th day (the day of testing) and the average strengths were recorded as 34 MPa and 50 MPa, respectively. Type I ordinary Portland cement was used with the maximum aggregate size of 10 mm with superplasticizer as an additive to achieve a high flow, 80 mm slump associated with topping mixes. The concrete properties are shown in Table 4.

Table 2: Properties of Mass Timber Products

Material	Grade	Species	Direction	Thickness [mm]	MOE [MPa]	F_t [MPa]	F_c [MPa]	F_b [MPa]	F_s, F_v [MPa]	F_{cp} [MPa]
CLT	245E	SPF	Strong	245	12400	17.7	19.9	30.5	0.5	6.5
			Weak		9500	5.5	11.5	11.8	0.5	5.3
Glulam	16c-E	SPF		1178	12,400	20.4	30.2	14.0	2.0	7.0

Three types of connectors were used for the test program: a) Type A - fully threaded STS: (Rothoblaas VGS FT 11x250), b) Type B – 6 mm thick and 75 mm deep steel kerf plates; and c) Type C - 90 mm wide proprietary glued-in Holz-Beton-Verbund (HBV) mesh, c.f. Table 3.

Table 3: Shear Connector Description

Type	Connector	Description
CLT Shear Reinforcement	Rothoblaas VGS FT 11x300	Install at 45^0 to grain
TCC Type A	Rothoblaas VGS FT 11x250	Install at 45^0 to grain
TCC Type B	75X2100 mm steel plates	Installed at 7 mm wide saw kerf with polyurethane adhesive
TCC Type C	90 mm deep mesh	Installed at 3 mm wide saw kerf with polyurethane adhesive

Table 4: Concrete Properties

Material	*f'_c*	*Max Agg.*	*Weight*	*Admixture*	*Additive*	*Air Content*	w/c ratio	Slump
	[Mpa]	[mm]	[-]	[-]	[-]	[%]	[-]	[mm]
Ready-mix	35	10	Normal	Sika Air 260, Viscocrete 2100	Super-plasticizers	5	0.42	80±20

2.2 Specimen Description

A comprehensive test program as shown in Table 1 was designed to investigate the parameters for the successful implementation of a low-cost and efficient TCC system. Two full-scale test series were conducted at the University of Northern British Columbia Wood Innovation and Research Laboratory in Prince George.

The full-scale TCC specimens were tested with all three types of TCC connectors described in Table 3. The schematic cross section of the testing TCC floor panels are shown in Figure 7. Series S6 was tested under four-point bending, c.f. Figure 2a, whereas, series S7 was tested using an eccentric load, applied on the slab edge, to assess the capacity and failure mechanism under torsion, c.f. Figure 2b. The full-scale TCC floors in series S6 and S7 were comprised of half-reinforced CLT panels. A total of 6 tests were conducted in series S6 under four-point bending and 3 tests were conducted in series S7 under torsion.

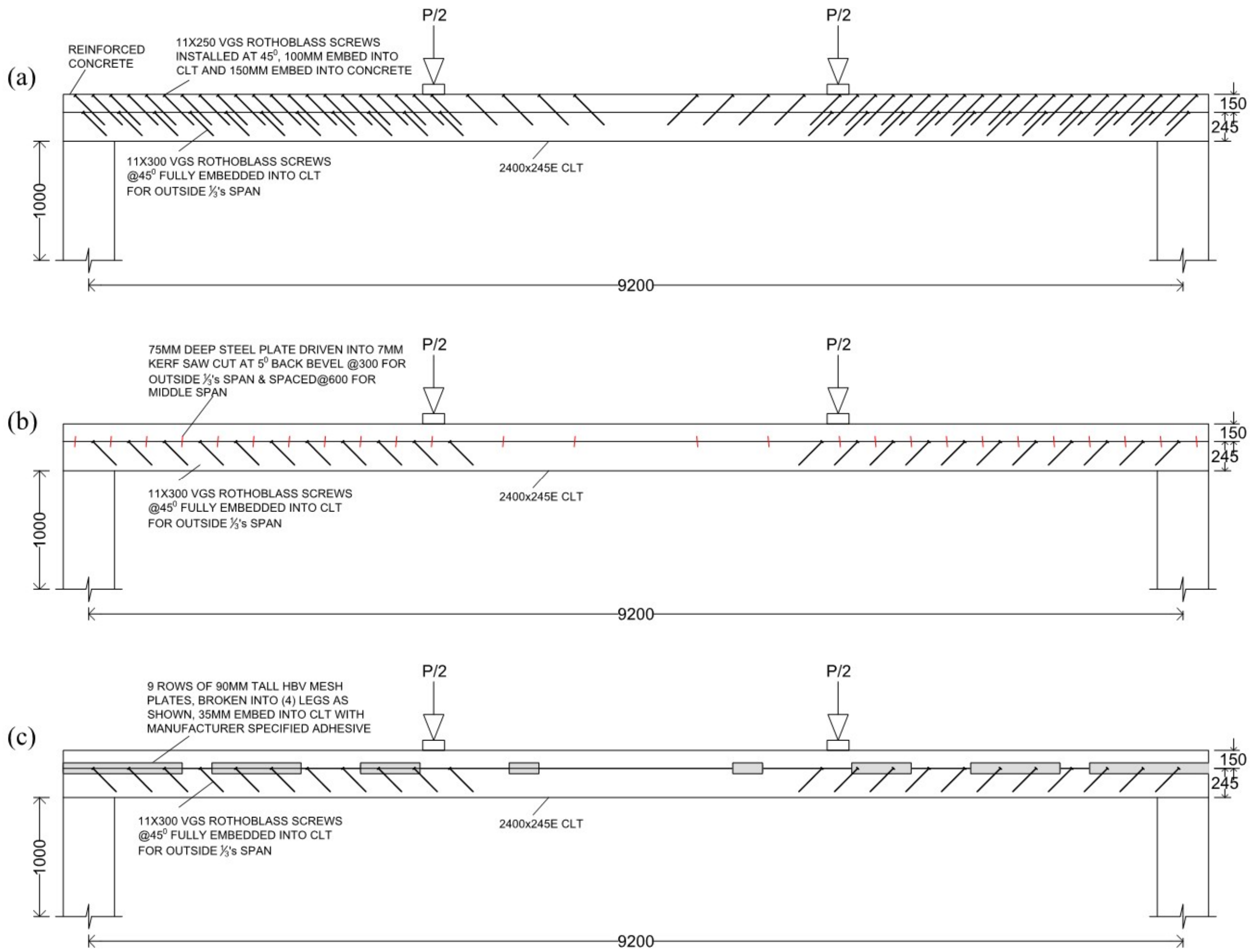


Figure 1: Schamatic of test series: a) TCC with Type A connector, b) TCC with Type B connector, and c) TCC with Type C connector

2.3 Test Setup

The schematic testing setup for the TCC floors are shown in Figure 2. Series S6 floor specimens were tested under 4-point bending as shown in Figure 2a. Series 7 was tested under torsional loads as shown in Figure 2b. A photo of 4-point bending and torsion tests for the full-scale tests are shown in Figure 3.

The floors are connected to 430×1178 mm glulam columns by 12-16 mm Ø, 250 mm long glued-in threaded rods. The glulam columns are connected to the concrete strong floor by angle brackets with anchor bolts. Loads were applied to the floors at approximately one-third points using two 500 kN and two 250 kN maximum capacity actuators for a total of 1500 kN maximum capacity from the steel load frame. The actuator loads are distributed equally to one-third points using steel beams and a timber spreader beam. The edge torsional line loading was applied using three actuators- two 500 kN and one 250 kN maximum capacity, for a total of 1250 kN as shown in Figure 2b.

2.4 Instrumentation

The actuator loads were recorded from load cells. A total of 20 sensors were attached to the TCC specimens to record the vertical deflection of the floor specimens and relative slip at the concrete to CLT interface, c.f. Figure 2. A total of 8-LVDTs (L1-L8) were attached to each specimen. LVDTs L1 to L4 measure interface slips at the four corners of the slab and L5-L8 measure slips at the one-third loading points. A total of 9 string pots were installed to record the vertical deflections: 2 at mid-span (sensors D2/D5), 4 at one-third points (sensors D1/D6; D3/D4), and 3 on the right side of the slab end (sensors D7-D9) to record the weak axis warping of the panels. Three additional 3 string pots were installed on the left side of the slab end (sensors D10-D12) to record the weak axis warping of the panels (Figure 2).

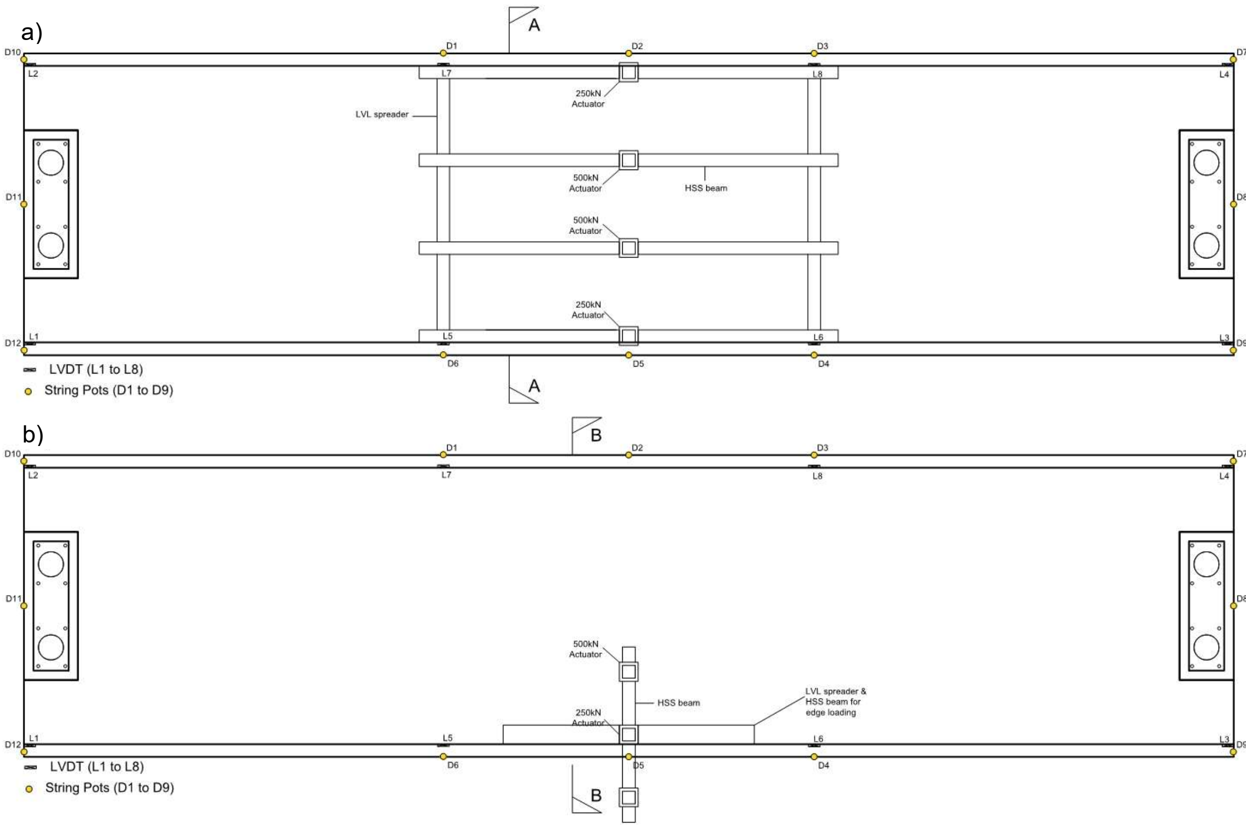


Figure 2: Plan view of full-scale TCC speciemns and location of sensors for: a) 4-point bending test, and b) torsion test

2.5 Results Analysis Methods

The performance of TCC specimens with Type A, B, C composite connectors were assessed in terms maximum loads, F_{max}, stiffness, k, deformation at maximum loads, d_{Fmax}, moment capacity, $M_{capacity}$, and shear capacity, $V_{capacity}$. The forces were presented as the sum of the four and three applied actuator loads for the bending and torsion tests, respectively. The stiffness was calculated for the range of 10% to 40% of capacity according to EN 26891 [5].

(a) (b)

Figure 3: Photo of a full-scale TCC test- (a) under 4-point bending, b) under torsion loading

3 RESULTS AND DISCUSSION

3.1 Series S6: Full-Scale Bending Tests

The test results are summarized in Table 5. The mid-span load-deflection curves of all six full-scale TCC specimens in series S6 tested with all 3 types of composite connectors are illustrated in Figure 4a. The panels with composite connector Type A, screws, (S6-HR-A) failed in bending at an average load of 706 kN (creating a maximum moment at failure of 1,100 kNm) and a displacement at failure of on average 124 mm. The panels with composite connector Type B, steel kerf plates (S6-HR-B) failed in bending at an average load of 1,040 kN (creating a maximum moment at failure of 1,619 kNm) and a displacement at failure of on average 124 mm. The panels with composite connector Type C, HBV mesh, (S6-HR-C) failed in connector shear at an average load of 670 kN (associated with a maximum bending moment of 1,043 kNm) and a displacement at failure of on average 76 mm. Initial stiffness the panels with screws and the HBV mesh was similar, the stiffness of the panels with steel kerf plates was approximately twice as high.

3.2 Series S7: Full-Scale Torsion Tests

The mid-span load-deflection curves of all specimens in series S7, tested under torsion by applying line loads on one edge of the slabs, are illustrated in Figure 4b. The panel with composite connector Type A, screws, (S7-HR-A) reached the first failure at 285 kN and 80 mm, and reached a maximum load and displacement of 442 kN and 255 mm. The panel with composite connector Type B, steel kerf plates, (S7-HR-B) reached the first failure at 234 kN and 70 mm, and reached a maximum load and displacement of 489 kN and 261 mm. The panel with composite connector Type C, HBV mesh, (S7-HR-C) reached the first failure at 257 kN and 61 mm and reached a maximum load and displacement of only 413 kN and 159 mm. The initial stiffness of all three panels was again very similar.

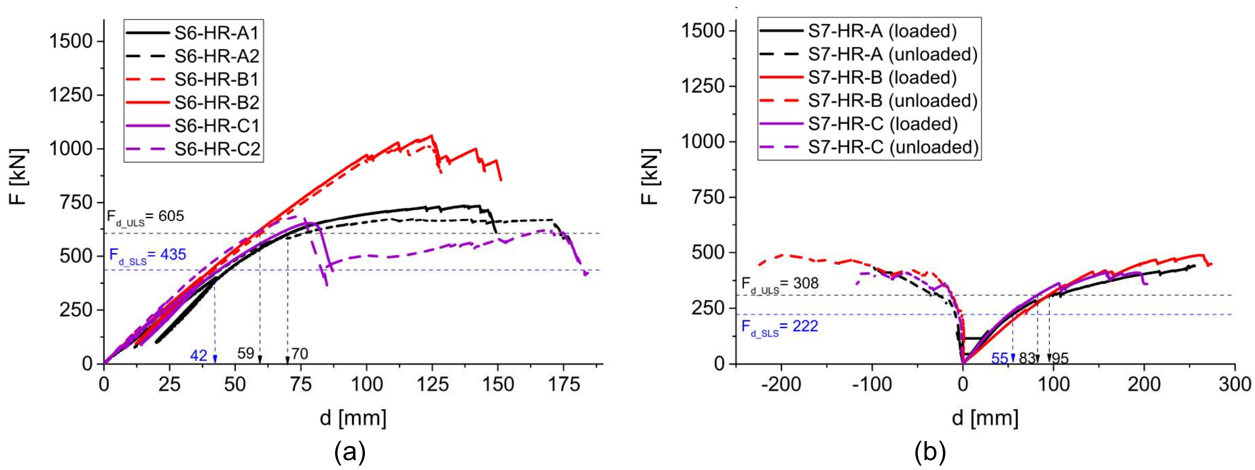


Figure 4: Load-deflection curves for: a) Series S6 and b) Series S7

Table 5: Results from TCC tests

Series	ID	F_{max} [kN]	$M_{capcity}$ [kN.m]	$V_{capcity}$ [kN]	d_{Fmax} [mm]	k_1 [kN/mm]	Failure [-]
S6	S6-HR-A1	735	1145	368	137	16	B
	S6-HR-A2	677	1054	339	110	11	B
	S6-HR-B1	1018	1586	509	123	25	B
	S6-HR-B2	1061	1653	531	125	26	B
	S6-HR-C1	653	1017	327	78	11	B
	S6-HR-C2	687	1070	344	74	12	C
S7	S7-HR-A	442	887	221	255	4	S+ B
	S7-HR-B	489	981	244	261	4	S+ B
	S7-HR-C	413	830	207	159	5	S+ B

3.3 Capacity versus Demand

The full-scale slab test performance and demands are plotted in Figure 5. At service loads, the average capacity/demand ratio for the half-reinforced TCC specimens with screws, kerf plates and HBV mesh when tested in a four-point bending configuration were 1.6, 2.4, and 1.5, respectively. At service loads, the average mid-span deflection observed in the TCC specimens in series S6 was 42 mm. Whereas, under torsional tests in series S7, the capacity/demand ratio at service loads for the half-reinforced TCC specimens with screws, kerf plates and HBV mesh were 2.0, 2.2, and 1.9, respectively. The mid-span deflection at service loads was 55 mm, where again, a portion of this will be cambered out with the mid-span shoring frame. Capacity to demand ratios at ultimate and service loads were 1.7 and 2.4, respectively in series 6 and were 1.6 and 2.2, respectively in series S7.

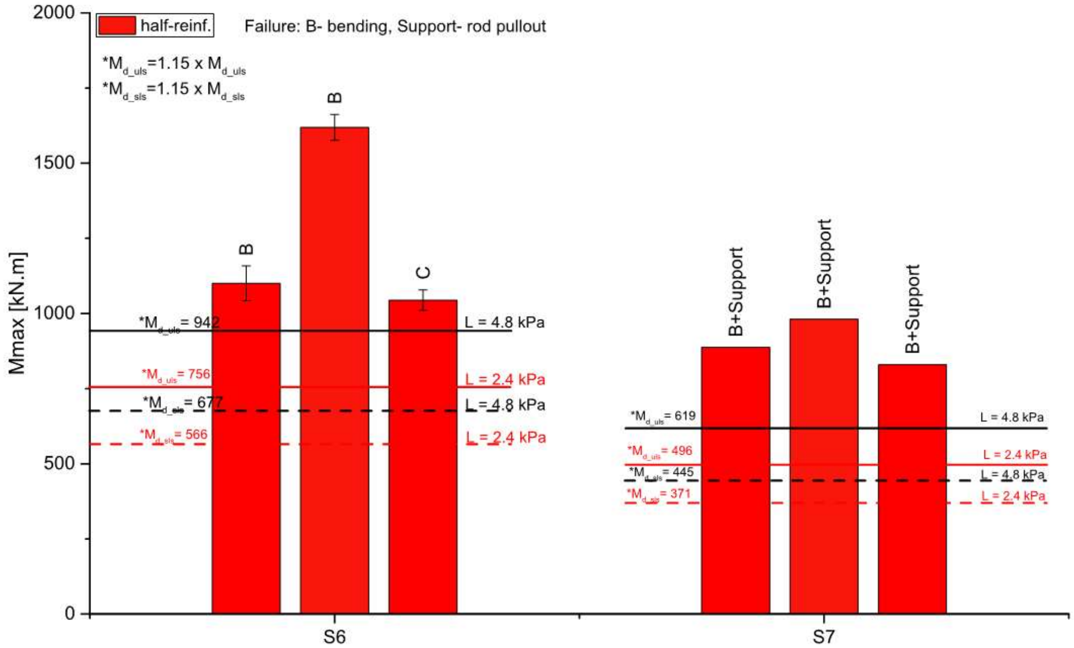


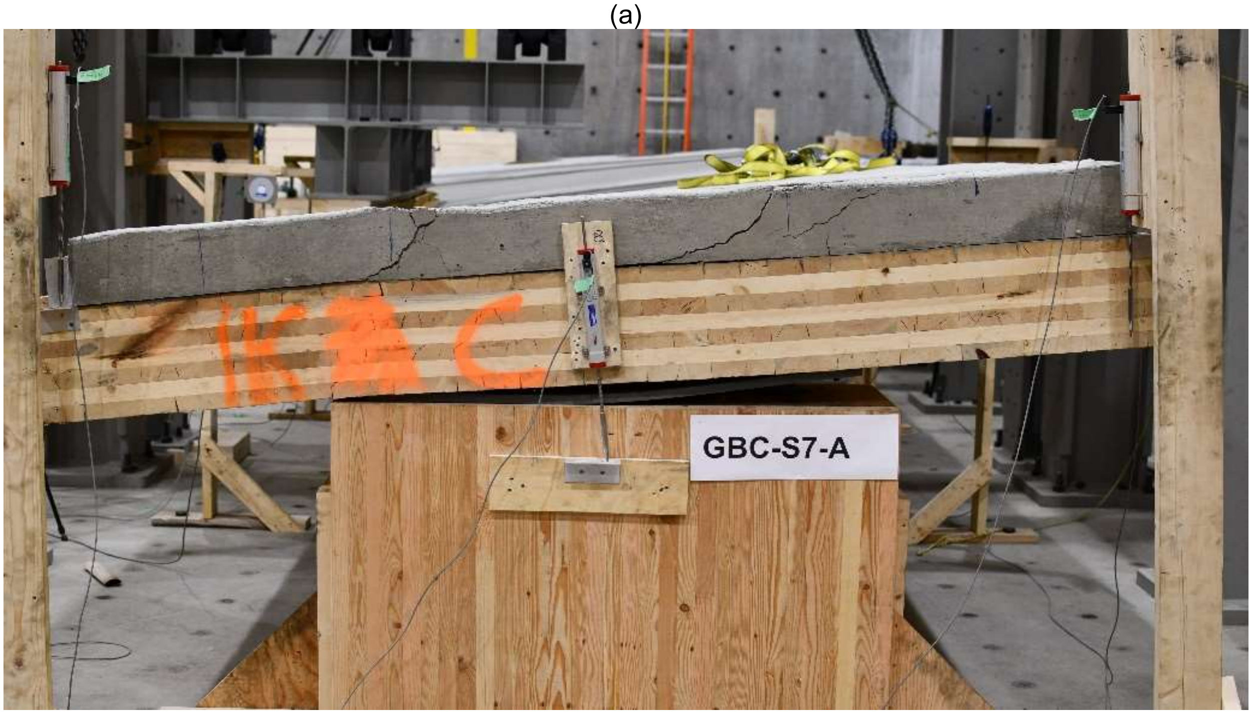
Figure 5: Capacity vs demand for the TCC floor speciemns

3.4 Failure Modes

The full-scale specimens with Type A and Type B connectors failed in bending at mid-span (Figure 6a). The initial failure in the TCC specimens with Type C connectors, HBV mesh, happened in the connector with the final subsequent failure occurred at mid span due to bending.

When the TCC specimens were tested under torsion in series S7, the first failure was always characterized by the pull-out of the glued-in rod from the wallumn as shown in Figure 6b. At ultimate loads, the specimens were failed in bending at mid-span in the loading side.

(a)



(b)

Figure 6: Typical failure pattern: a) Series S6, and b) Series S7

4 CONCLUSIONS

This research evaluated the performance and efficiency of TCC systems with three different shear connectors. There has been a relatively small amount of research on shear capacity of TCC panels as this seldom controls design. In addition to this being one of the main points of investigation, finding a low cost, simple to install shear connector was also of interest. The following key conclusions can be drawn from this testing program:

- TCC with steel kerf plates exhibited the highest capacity and stiffness and were priced by multiple suppliers to be the most economical option.
- In order to move forward with screws or HBV solutions, the TCC slab band design would be revised slightly, increasing the number of connectors.
- 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.

Results and final findings from this test program have been implemented into the base building design for The Arbour, a 10-storey educational building for George Brown College, located in the East Bayfront district of Toronto, Canada. Upon completion of this testing, the steel kerf plate TCC connector solution with 35 mm CLT embed was deployed with this innovative slab band solution.

5 ACKNOWLEDGEMENTS

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