

Advancing Timber for the Future Built Environment

STRUCTURAL MODELLING OF A NOVEL HYBRID TIMBER FLOOR SYSTEM

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ABSTRACT: To demonstrate the potential for carbon-sequestering mass timber to be used across various building types and scales, DIALOG and EllisDon have developed a Hybrid Timber Floor System (HTFS). This composite system consists of post-tensioned (PT) concrete beams, cross laminated timber (CLT) panels, and concrete topping connected to one another using self-tapping screws (STS) and kerf plates. This paper investigates the structural performance of this novel HTFS through a combination of physical testing and numerical modelling conducted at FPInnovations. The structural performance of STS and kerf plate connections was experimentally evaluated to develop the shear connector parameters for modelling the composite action. Refined and comprehensive three-dimensional (3D) finite element models were developed, calibrated, and verified against the deflection and vibration measurements obtained from non-destructive full-scale HTFS testing. The verified model was then used to investigate the structural demand on the HTFS' basic components (i.e., CLT, steel reinforcement, steel tendons, STS screws, kerf plates, and concrete) under out-of-plane loading. The results from connection testing and floor simulation provide valuable insights into the structural performance of this novel HTFS, supporting its potential for broader application in sustainable construction.

KEYWORDS: timber structures, hybrid systems, floor systems, post-tensioned mass timber, structural performance

1 – INTRODUCTION

The use of mass timber for tall and large-scale buildings is gaining momentum worldwide as a strategy to reduce the embodied carbon of the built environment. Sustainably harvested mass timber can sequester substantial amounts of carbon, capturing approximately 1.9 metric tons of CO₂-equivalent emissions per cubic meter of wood product. Despite this environmental benefit, mass timber remains a niche construction material. According to a global audit by CTBUH, approximately 150 mass timber buildings of eight stories or taller have been completed, are under construction, or are proposed worldwide.

DIALOG, a Canadian consulting firm, has designed a 105storey Hybrid Timber Tower prototype that efficiently combines carbon-sequestering mass timber with steel and concrete to demonstrate the potential for mass timber to be used across various building types and scales [1]. Life cycle analyses indicate that floor systems contribute nearly three-quarters of a tall building's environmental impact. In collaboration with EllisDon, a Canadian general contractor, DIALOG developed the Hybrid Timber Floor System (HTFS) (Figure 1), designed to achieve 12 m clear floor spans for office commercial space. When incorporated into the tower prototype, the HTFS enables a 46% reduction in structural embodied carbon while utilizing 36,649 m³ of mass timber – 14 times more than the current tallest wood structure. Recognizing its innovative potential, the HTFS received the Architecture award in Fast Company's World Changing Ideas for 2021.

The HTFS integrates post-tensioned (PT) concrete beams, cross-laminated timber (CLT) panels, and structural concrete topping, enabling mass timber-based floor systems for mixed-use, long-span construction — an application previously dominated by concrete or steel-concrete solutions. However, no research has yet been

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conducted on this innovative system, where PT cables and reinforcing bars are embedded within concrete beams with an optimized profile, and composite action is achieved through self-tapping screw (STS) and kerf plate connections.



Figure 1. Hybrid timber floor system

A collaboration among DIALOG, EllisDon and FPInnovations was established to investigate the fire resistance and structural performance of the HTFS. The findings on fire resistance were published in the ASCE Journal of Structural Engineering [2]. This paper focuses on the structural performance of the HTFS, reporting key HTFS connection tests and presenting an advanced modelling approach. The models are calibrated and verified against the deflection and vibration measurements from the full-scale HTFS testing and subsequently used to assess structural performance under out-of-plane loading.

2 – HYBRID TIMBER FLOOR SYSTEM

As shown in Figure 1, the HTFS, arranged from the bottom to the top, consists of PT concrete beams, CLT panel, and concrete topping. STS and kerf plate connections (Figure 2) facilitate the composite action by connecting the CLT panel to the concrete beams and concrete topping, respectively. Figure 2 illustrates (a) the longitudinal section at the concrete beam and (b) the cross-section at the midspan of the HTFS. This system is constructed using a 3-meter wide CLT panel with 12-meter spans, though shorter or longer spans are feasible. The HTFS can be supported by reinforced concrete walls or steel beams in the interior and steel or reinforced concrete beams at the building perimeter.



Figure 2. Hybrid timber floor system: (a) longitudinal section, and (b) cross section

CLT panels arrived at the modular fabrication shop inverted, with routed-out sections designed to accommodate the placement of two PT concrete beams. Figure 3a illustrates the arrangement of reinforcing bars and PT tendons within these routed strips prior to concrete pouring. Once the concrete is poured and allowed to cure, the panel is flipped over, and post-tensioning process, as shown in Figure 3b, is performed. After tensioning, the ducts are grouted to establish a bonded post-tensioned system, and the ends of the beams are dry packed with grout so that it is flush with the end of the CLT panel. Upon completion of the manufacturing process, the panel is transported to a building site, where it is erected and topped with additional concrete.



Figure 3. Manufacturing of HTFS in a modular fabrication plant: (a) upside down panel prior to casting concrete beams, and (b) panel flipped with post-tensioning jacks in place

The HTFS are designed as a composite system, consisting of CLT, post-tensioned concrete, and a concrete topping layer where the overall strength and stiffness of the HTFS will be greater than the simple sum of the performance of the individual layers. The section of the floor can be represented as a transformed section, where each part's area is replaced by a corresponding transformed area. The transformed area for any given part *i* is calculated as $(E_i/E_{ref})A_i$, where E_i is the modulus of elasticity of part *i*, E_{ref} is the reference modulus of elasticity (taken as the modulus of the CLT face and layers along the major strength direction), and A_i is the area of part *i*. The service load stresses in each part and the floor deflections under both prestressing forces and gravity loads are determined using established procedures for prestressed concrete

and modelling of the HTFS. By testing a small group of each type of connector, the data would include some group effects while also allowing the data to be adjusted for the varying connector spacing. Three STS connection specimens ($549 \times 560 \times 950$ mm, Figure 4a) and three kerf plate connection specimens ($450 \times 560 \times 850$ mm, Figure 4b) were tested under short-term loading.



Figure 4. Testing setup with specimen of (a) kerf plate connection and (b) self-tapping screw connection

To maintain symmetry, the specimens were fabricated and tested with two concrete-to-CLT shear planes. Separate groups of STSs or kerf-plates were located at each shear plane (designated as "left" and "right" when viewed as shown in Figure 4).

The CLT panels were 7-layer SPF E1 grade, conforming to PRG 320. The specified 28-day strength of the concrete topping was 25 MPa, and the modulus of elasticity for the mild reinforcing steel was 200,000 MPa. Each STS connection specimen consisted of four M10 \times 300 STSs on each shear plane. Screws were inclined 60° upward beams. The ultimate limit states capacities are verified by adapting provisions from design standards for timber [5] and prestressed concrete [6].

3 – CONNECTION TESTS

3.1 TESTING PROGRAM

STS connections, which connect the concrete beam to the CLT panel, and kerf plate connections, which connect the CLT panel to the concrete topping, are essential for enabling composite action within the HTFS. To examine how the connector's structural performance and spacing contribute to the composite action, information on a small group of each type of connector is required for the design

relative to a line normal to the CLT face, with a vertical spacing of 250 mm and a horizontal spacing of 180 mm. Each kerf plate connection specimen consisted of two steel kerf plates (PL $6 \times 279 \times 76$ mm) on each shear plane. The plates were inclined 5° downward relative to a line normal to the CLT face, with a vertical spacing of 300 mm.

The specimens were fabricated at EllisDon's modular fabrication plant, and instrumented and tested at FPInnovations' Vancouver laboratory (Figure 4). The two concrete topping layers of each specimen were spaced from the base of the testing frame. A loading block, through which load would be applied, was centred on the top of the CLT panel. Because the kerf plate connections were expected to be stiffer than the STS connections, they were ramp loaded at 0.05 in/min and 0.1 in/min, respectively, using an actuator with a total stroke range of 16 in (406 mm). A 100-kip (440 kN) load cell was installed between the actuator and the specimen to measure the applied force during testing. Because connector groups on each of the two shear planes are likely to behave differently, a swivel head was also used to ensure the applied load was divided equally between the two shear planes. Sixteen (16) displacement transducers, half at the front and half at the back, were installed at the mid-height of the specimens to measure the relative deflection between the concrete and the outer layer of the CLT, as well as between the CLT laminations. Two additional displacement transducers, one on the left and one on the right, were installed at the top to measure the relative movement between the concrete block and the CLT panel in case the measured slip on front and back faces are not consistent with the interior laminations that the STS or kerf plate connectors directly embed on.

3.2 TESTING RESULTS

The load-stroke curves of the STS and kerf plate connection specimens are shown in Figure 5. The load resisted by the STS connections increased linearly after a brief initial slack segment, then decreased linearly beyond the peak load. During the testing, a snapping sound was distinctly heard from the screw(s) on the right side of the S-107 specimen. Similarly, the load carried by the kerf plate connections increased linearly after a small initial slack segment attributed to crushing between the kerf plates and CLT and/or concrete. Due to the unexpectedly high capacity of the kerf plate connections, the testing was stopped when the load reached the maximum capacity of the loading frame (100 kip).



Figure 5. Load-stroke curves of specimens of (a) self-tapping screw connection and (b) kerf plate connection

Assuming the load was equally resisted by the connections at both sides of the CLT, the failure load for the left and right connections in the same specimen was equal, i.e., half of the peak load. However, the deflection values at a given load differed between the left and right connections due to the variation in stiffness, as listed in Table 1 for the STS and Table 2 for the kerf plate connections. The effective connector stiffness was calculated as the slope of the load-slip curve between 10% and 40% of the peak load, and the measured slip between the CLT and concrete. According to Table 1, the stiffness of an STS averaged 53.9 kN/mm with a COV of 18.3%, while its strength averaged 45.1 kN with a COV of 4.5%. Similarly, as shown in Table 2, the

stiffness of a kerf plate averaged 127.0 kN/mm with a COV of 9.8%, making it approximately 2.4 times stiffer than an STS. Due to the capacity of the testing frame, the strength of the kerf plate connections could not be measured except to conclude that each kerf plate can resist at least 25 kip (110 kN). However, the non-linearity at the end of the load-stroke curves suggests that these connections may be approaching their ultimate capacity.

Table 1 Testing results of self-tapping screw connections

Specimen	Left/Right	K [kN/mm]	F _{max} [kN]	Note
S 100	L	200.5	175	-
5-109	R	179.0	175	Failed
S 107	L	189.4	191	-
5-107	R	241.0	191	Failed
S 105	L	199.8	176	-
5-105	R	283.6	176	Failed
	Min	179.0	175	
	Max	283.6	191	
4 STS on	Average	215.5	181	
each side	Median	200.2	176	
	StdD	39.4	8.1	
	COV [%]	18.3	4.5	1
Average per STS		53.9	45.1	
Min per STS		44.7	43.7	

Kerf Plate Specimen	Left/Right	K [kN/mm]
K 101	L	260.4
K-101	R	255.9
K-105	L	251.2
11 100	R	241.4
K_102	L	295.8
K-102	R	219.8
	Min	219.8
	Max	295.8
2 Karf Diatas an asah sida	Average	254.1
2 Kerl Flates off each side	Median	253.5
	StdD	25.0
	COV [%]	9.8
Average per Kerf	127.0	

Table 2 Testing results of kerf plate connections

4 – ADVANCED 3D MODELLING

A detailed three-dimensional (3D) finite element (FE) model (Figure 6) was developed in ABAQUS to investigate the structural performance of the HTFS under out-of-plane loading.



Figure 6. 3D modelling of HTFS

4.1 GEOMETRICAL MODEL

A geometric model incorporating all the key components was developed based on DIALOG's drawings. To reduce computational cost, a quarter 3D model was created using symmetry. To better understand and include the effect of rolling shear in the cross laminations, each lumber piece in the CLT panel, as well as the concrete beams and topping, were meshed with 3D solid elements. The PT tendons and reinforcing bars were meshed with 3D beam and truss elements, respectively. properties of the longitudinal and transverse layers were derived from CSA O86 [5] (Table 3). The strength values (Table 4) were obtained by converting the specified CSA O86 values using the O86 committee method described in the Modelling Guide [4]. To capture the full series of the HTFS failure modes, the brittle failure mechanism (e.g., tensile and/or shear failure) was adjusted by shifting the zero-strength strain by a factor of 100. The wood density was assumed to be 4.49 kN/m³. A nominal wood moisture content of 7% was assumed based on the observed moisture meter reading.

for the Spruce-Pine-Fir (SPF) E1 grade. The elastic

4.2 MATERIAL MODELS

The advanced constitutive model Wood^s [3] (Figure 7) was used to simulate the complex orthotropic stress-strain behaviour of wood [4]. The CLT panel of the HTFS was

A damaged plasticity model [8] (Figure 8) was used to simulate the complex bi-modulus stress-strain behaviour of concrete.



Table 3 Elastic properties of lumber in CLT

Layer	E// [MPa]	E⊥ [MPa]	\mathbf{v}_{ij}	G// [MPa]	G⊥ [MPa]
Longitudinal	11700	390	0.4	731.3	73.1
Transverse	9000	300	0.4	562.5	56.3

Table 4 Strengths [MPa] of lumber in CLT

Layer	f _{t,//}	f _{c,//}	f _{t,⊥}	f _{c,⊥}	f _{s,//}	f _{s,⊥}
Longitudinal	28.6	38.5	2.8	5.3	2.4	0.8
Transverse	5.9	18.0	2.8	5.3	2.4	0.8

concrete using damaged plasticity model

The specified strengths of the concrete topping and concrete beams are 25 MPa and 45 MPa, respectively, while the measured compression strengths, $f'_{c,28}$, are 43 MPa and 58 MPa. The compressive strength ($f'_{c,t}$), modulus of rupture, and modulus of elasticity of concrete (E_c) vary over time (t), e.g., by days, and can be derived from [6]

$$f_{c,t}' = \frac{t}{a+tb} f_{c,28}'$$
(1)

$$f_r = 0.6\sqrt{f_{c,t}'} \tag{2}$$

$$E_c = \left(3300\sqrt{f'_{c,t}} + 6900\right) \left(\frac{\rho}{2300}\right)^{1.5}$$
(3)

where, t is the number of days the concrete has been cured; a and b are taken as 4 and 0.85, respectively, according to CSA A23.3 [6]; and ρ is the density of concrete (e.g., 2400 kg/m³). The Poisson's ratio of concrete was assumed to be 0.2. To capture the full series of the HTFS failure modes and avoid convergence issues, the descending portions of the stress-strain curve were replaced with horizontal lines.

An elasto-plastic model was used to simulate the isotropic behaviour of steel. The mechanical properties of reinforcing bar (rebar) steel (Grade 400 – CSA G30.18) and PT tendons (Grade 1860 – ASTM A416/A416M) are listed in Table 5. The elastic properties, i.e., modulus of elasticity and Poisson's ratio, were obtained from CSA S16 [7], while the yield strengths were based on the tested results provided by the manufacturers.

Table 5 Mechanical properties of steel

Steel	E [MPa]	v	f _y [MPa]
Rebar-10M	200,000	0.3	469.7
Rebar-15M	200,000	0.3	510.4
Rebar_20M	200,000	0.3	453.4
Tendons	198,569	0.3	1813.0

4.3 CONNECTION MODELS

The STS and kerf plate connections were modelled using cohesive elements (Figure 9) [8]. This approach was chosen to (a) eliminate and avoid the meshing complexity associated with 3D modelling of STSs and kerf plates, as well as the corresponding convergence issues; (b) incorporate directly the mechanical properties obtained from the connection tests; and (c) leverage the efficiency of the cohesive element modelling approach in subsequent design and analysis stages, such as optimizing the number and spacing of fasteners.



Figure 9. Illustration of cohesive element modelling

STSs were used to connect the CLT panel to the concrete beam with varying number and spacing across three zones,

from midspan to the support: Zone I, II, and III. Based on the number and spacing of the STS used in each zone, the properties of cohesive elements for STS connections (Table 6) were derived using the average stiffness and minimum strength per STS measured in the tests (Table 1). The minimum observed test strength was adopted for a conservative approach.

Table 6 Properties of cohesive elements for STS connections

Zone #	I (middle)	II	III (support)
#ofRows & Spacing [mm]	2 @ 500	5 @ 250	12 @ 250
Screw per row within beam width	2	2	4
Stiffness [kN/mm/mm ²]	0.653	1.306	2.613
Yield strength [kN/mm ²]	0.529	1.059	2.118

Kerf plates were used to connect the CLT panel to the concrete topping with varying spacing across two zones, from midspan to the support: Zone I and II. Based on the number and spacing of the kerf plates in each zone, the properties of cohesive elements for the kerf plate connections (Table 7) were derived using the average stiffness per kerf plate measured in the tests (Table 2). Additionally, it was assumed that the yield strength per kerf plate could be taken as 120% of the maximum measured load (e.g., 111.2 kN). This assumption was made based on the initial non-linearity observed at the end of the load-stroke curves, which implied that the kerf plate connections were nearing their strength very soon.

Table 7 Properties of cohesive elements for kerf plate connections

Zono #	I	II	
Zone #	(middle)	(support)	
#of rows & Spacing [mm]	4 @ 600	9@300	
Kerf plates per row within panel width	4	4	
Stiffness [kN/mm/mm ²]	0.282	0.565	
Yield strength [kN/mm ²]	0.297	0.593	

4.4 OTHER MODELS

The embedded element modelling approach (Figure 10) [8] was adopted for rebars (including stirrups) and the ducting system in concrete, as well as for PT tendons grouted within the ducting system. Before the PT tendons were grouted, they were able to slide within the ducting system; therefore, tube-to-tube interaction (Figure 11) [8] was used to model the PT tendons and ducting system prior to grouting. Symmetrical boundary conditions were applied at both the midspan end and one edge side. The support and loading conditions will be described in the following sections, as they varied during the model calibration, verification, and the final loading scenario.



Figure 10. Illustration of embedded element: solid and hollow dots are the nodes on the host and embedded elements, respectively; while solid and dotted lines are the host and embedded elements



Figure 11. Illustration of tube-to-tube interaction

5 – MODEL CALIBRATION AND VERIFICATION

The investigated HTFS is a novel hybrid floor system with composite actions, manufactured through eight main steps (as illustrated in Figure 12, corresponding to Stages *a* to *h* in Table 8), including CNC machining, concrete pouring, panel flipping, and post-tensioning. These steps are influenced by both physical-chemical effects. To verify the modelling approach and determine a set of calibration factors for the HTFS, the manufacturing process (except for Stage *g*, due to the lack of data), along with two stages of the deck before and after being loaded with 16 concrete blocks (Figure 13), each weighing about 3,900 lbs (1.95 ton), were simulated in this study. The deflection and vibration measurements from these stages were used as a benchmark for comparison with the modelling results. The

models were calibrated only in Stages a, d, e, and i, and verified in the remaining five stages.



Figure 12. Illustration of manufacturing process of HTFS: (a) CNC machining, (b) screws, cages, and PT ducts installed, (c) concrete cast, (d) panel flipped, (e) post-tensioned, (f) PT cables grouted, (g) kerfplates and rebars installed, and (h) topping poured



Figure 13. Deformation shape of HTFS

		а	b	С	d	е	f	g	h	i	j
	Stage	CLT	CLT, screws, cages,	CLT + concrete	Panel	Post-	PT cables	Installed at	Topping	CT 139	16
		only	and PT ducts	beams	flipped	tensioned	grouted	lab	poured	days	blocks
	E & G of	0.74	× ×		× 1		×		~	、 、	0.74
_	CLT	0.74	7	~	~	~	~	~	~	~	0.74
tion	E of CB				0.04(2)	0.51(3)	\rightarrow	\rightarrow	0.51	1.00	1.00
ora	STS				1.00	\rightarrow	\rightarrow	\rightarrow	\rightarrow	\rightarrow	1.00
alil	PT force					1.00	\rightarrow	\rightarrow	1.00	0.878	0.878
<u> </u>	E of CT									0.125	0.125
	Kerf-plate									1.00	1.00
	Exp [mm]	35.5	42.5	77.0	-9.5	52.3		N/A	19.8	8.0	-33.4
Def	Mod [mm]	35.5	43.3	76.0	-9.5	51.5			20.8	8.0	-23.1
<u> </u>	Diff [%]	0.0	2.0	-1.2	0.0	1.5			4.8	0.0	-24.8
Fre	Exp [Hz]						3.55	N/A		4.31	
	Mod [Hz]						3.56			4.02	
	Diff [%]						0.3			-6.8	

Table 8 Calibration and verification

Note: Def - Deflection, a positive value indicates a + Y(up) deflection in the model space (Figure 13), while loads (including gravity) are applied in a positive (+) or negative (-) Y direction depending on the manufacturing process; Fre - Frequency; E - Modulus of elasticity; G - Shear modulus; CB - Concrete beam; CT - Concrete topping; Exp - Experiment; Mod - Modelling; Diff - Difference.

5.1 STAGE A (CALIBRATION)

As mentioned in Section 2, the CLT panel arrived at the modular fabrication shop routed to accommodate the placement of PT concrete beams (Figure 12a) and with therouted face up. The panels were supported at each end. Under gravity loads, the CLT panel exhibited a midspan deflection of 35.5mm relative to the supports. An FE model of the routed CLT panel was built using the basic lumber design properties listed in Table 3. To account for the variations in material properties, size effects, and manufacturing factors (e.g., gaps, incomplete bonding, and stress concentrations arising from the routed profile), a calibration factor of 0.74 (Table 8) was applied to the moduli of elasticity and shear modulus, resulting in the same deflection under gravity loads (in the +Y direction).

5.2 STAGE B (MODEL A VERIFICATION)

After screwing STSs into the CLT panel and installing the rebar cages and PT ducts (Figure 12b), the midspan deflection increased to 42.5 mm. Therefore, in addition to the gravity loads, a uniform surface load equivalent to the total weight of the STSs, rebars, PT ducts, and formwork was added in the +Y direction to the FE model developed in Section 5.1, resulting in a deflection of 43.3 mm, which is 2.0% higher than the measured value.

5.3 STAGE C (MODEL A VERIFICATION)

The midspan deflection reached 77.0 mm on the 10th day after the concrete was poured into the beam slots (Figure 12c). Since the concrete beams had not yet cured, it was considered conservative to ignore their contribution to the bending stiffness of the system as most of the additional deflection would have been due to the dead weight of the concrete while still in a fluid state. Therefore, only an additional surface load, equivalent to the weight of the concrete beams, was added in the +Y direction to the FE model, without considering the structural influence of the un-composited concrete beams. A deflection of 76.0 mm was obtained, which is 1.2% lower than the measure value.

5.4 STAGE D (CALIBRATION)

After the concrete beams had cured for 14 days, the CLT panel with the concrete beams was flipped over (Figure 12d). End supported in this orientation, the now had a midspan 9.5 mm deflection. Although the concrete had partially cured, the reversal of the stresses on the beams from compression to tension was sufficient to crack the concrete during the flipping process.

A corresponding model was built by adding the concrete beams, rebars, ducts, and PT tendons following the plan outlined in Section 4. Only gravity loads were applied to the model. The flipping was simulated by reversing the direction of gravity loads (+Y vs -Y) and moving the supports from the ends of the CLT panel to those of the concrete beams. To account for cracking in the concrete beams during flipping and their low contribution to the system stiffness, the model was assigned a very low calibration factor of 0.04 (Table 8) was applied to the modulus of elasticity of concrete, yielding the same deflection (9.5 mm) as observed under gravity loads. No calibration (or a factor of 1.0) was applied to the STS connections (Table 8).

5.5 STAGE E (CALIBRATION)

One day after flipping, the HTFS was post-tensioned (Figure 12e) with 889.7 kN per beam (1271 MPa per tendon), resulting in the midspan deflection for a total camber of 52.3 mm. Since the PT force is expected to close the cracks in the concrete beams that occurred during the flipping, a modified calibration factor of 0.51 (Table 8) was applied to the elastic properties of the concrete beams. The same camber was observed in the model. As the PT force was just applied, prestress relaxation was ignored, and thus no calibration (or a factor of 1.0) was applied to the PT force (Table 8).

5.6 STAGE F (MODEL D/E VERIFICATION)

Sixteen (16) days after the concrete beams were poured, the PT tendons were grouted into the ducting system (Figure 12f). The effect of grouting was simulated by embedding the PT tendons into the concrete beams, as the ducts had already been embedded and were part of the concrete beams. The model predicted a fundamental frequency vibration of 3.56 Hz. This was only 0.3% higher than the average value measured from the two decks (3.55 Hz) received by FPInnovations for full-size testing.

5.7 STAGE H (MODEL E VERIFICATION)

One hundred and twenty-six (126) days after the concrete beams were poured, the kerf plates and rebars were installed and the concrete topping was poured (Figure 12h). The camber of the HTFS decreased to 19.8 mm. Since the concrete topping had just been poured and had not yet cured, it was considered conservative to ignore its contribution to the bending stiffness of the system. Therefore, only an additional uniform surface load, equivalent to the weight of the concrete topping, rebars, and kerf plates, was added to the FE model, without considering the structural influence of the un-composited concrete topping. A camber of 20.8 mm was obtained, which is 4.8% higher than the laboratory measured value.

5.8 STAGE I (CALIBRATION)

One hundred and thirty-nine (139) days after the concrete topping had been poured (and the concrete beams for 265 days), the camber in the HTFS decreased from 55.1 mm to 8.0 mm due to a combination of complex physicalchemical effects, such as shrinkage and creep in both the wood and concrete. The model from Section 5.5 was modified following the plan outline in Section 4 to account for the concrete topping and rebars. Since the concrete beams had been post-tensioned to close the cracks that formed during the flipping, the calibration factor for the modulus of elasticity of the concrete beams was increased to 1.0 (Table 8) according to the concrete handbook (9.8.4.2). A calibration factor of 0.125 (Table 8), which is 50% of the factor (i.e., 0.25) suggested by the concrete handbook (10.14.1.2), was applied to the modulus of elasticity of the concrete topping. A calibration factor of 0.878 (Table 8) was applied to the PT force to account for prestress relaxation. The same camber (8.0 mm) and a frequency of 4.0 Hz were obtained from the FE model with the frequency being 6.8% lower than the measured value.

5.9 STAGE J (MODEL I VERIFICATION)

On the same day the camber and frequency of the HTFS were measured for Stage I, 16 concrete blocks were loaded onto the deck (Figure 13), causing a deflection of 33.4 mm at midspan relative to the supports. The model developed in Section 5.8 was updated by adding 4 blocks to the onequarter FE model (i.e., 16 blocks on the full deck model). The updated model yielded a deflection of 23.1 mm. The relative difference in the total midspan movement between the modelling (23.1+8.0) and testing (33.4+8.0) results was -24.8%. This discrepancy was attributed to the formation of shrinkage cracks in the concrete topping; such cracks are bridged by the temperature steel in the topping (10m bars at 300 mm). Prior to loading the panel with blocks, shrinkage cracks across the width of the panels and through the thickness of the concrete topping were observed. These cracks closed when the panel was loaded by the concrete blocks. Because these cracks were not modelled, the contribution to the system's bending stiffness from the concrete topping will be overestimated until the cracks close.

5.10 COMPARISON

During the calibration process, only six key material properties were adjusted using calibration factors (Table 8). Most of these were fixed since they were derived, while the other two, i.e., the elastic properties of the concrete beams and PT forces, varied across different stages, as they were time- and phase-dependent. As listed in Table 8, the developed models show good agreement with the measured deflection and vibration results in the first eight stages (a to f and h to i), with difference less than 7%. The model underestimated the total midspan movement in the last stage (*j*), as it did not account for the shrinkage gap in the concrete topping closing. Once this occurred, however, and since it is unlikely to happen again under the same loading conditions/direction, a much smaller discrepancy (e.g., 10%) would be expected from the model. Therefore, the developed model can be used to further investigate the structural performance of the HTFS under out-of-plane loading above the specified live loading.

6 – STRUCTURAL PERFORMANCE IN OUT-OF-PLANE DIRECTION

The calibrated and verified 3D model was further used to estimate the response of the HTFS under out-of-plane loading. The floor was assumed to be simply supported at the beam ends and was loaded monotonically in the -Y direction by increasing the vertical loads on the 16 blocks (Figure 13). The load-deflection curve was obtained and is shown in Figure 14, along with indications of different failure modes. For comparison, some failure modes estimated by DIALOG are also included in Figure 14.



The model successfully captured all the failure modes estimated by DIALOG. Table 9 compares the first five failure modes obtained from the FE model with those by DIALOG. The sequence of the first five failure modes is identical, starting with STS yielding, followed by CLT rolling shear failure, kerf plate connection yielding, concrete topping (CT) crushing, and ending with concrete

beam (CB) shear failure. The relative difference of failure load (F) is listed in the last column in Table 9. The largest difference was 22.6%.

		DIAL	.OG	FE M	odel	Rolativo Diff	
#	Failure mode	D	F	D	F	of E 19/1	
		[mm]	[kN]	[mm]	[kN]	011 [/0]	
1	STS yielding	85	785	99	871	11.0	
2	CLT rolling shear	95	899	126	1020	13.5	
3	kerf-plate yielding	107	1055	154	1236	17.2	
4	CT crushing	143	1493	229	1727	15.7	
5	CB shear	148	1548	262	1898	22.6	

Table 9 Comparison between FE model and DIALOG estimation

7-CONCLUSION

A novel hybrid timber floor system (HTFS) was developed by DIALOG and EllisDon. In this study, the structural performance of the HTFS was investigated at FPInnovations through short-term loading tests on connection specimens, pushover analysis using an advanced 3D model, and verification of the model with non-destructive measurements of a full-size HTFS. The findings from this study are summarized as follows:

- The stiffness per screw in the STS connections was 53.9 kN/mm on average, with a COV of 18.3%, while the average strength per STS was 45.1 kN with a COV of 4.5%.
- The stiffness per plate in the kerf plate connections was 127.0 kN/mm on average, with a COV of 9.8%. A kerf plate is approximately 2.4 times stiffer than an STS.
- Due to the capacity of the test frame, the ultimate capacity of the kerf plate connections could not be measured other than to confirm that a single plate has a capacity of at least 110 kN; however, the non-linearity at the end of the load-stroke curves suggests that the kerf plate connections may be close to reaching their strength.
- The refined and comprehensive FE models, along with a set of calibration factors, were able to estimate the midspan deflections and frequencies at the eight manufacturing stages with difference less than 7%.
- The model underestimated the total midspan movement in the last stage with 16 concrete blocks, as it did not account for the shrinkage gap closing. Under the specified live loading, these cracks are expected to close, so the model is expected to exhibit a much smaller discrepancy (e.g., 10%) when the model is used to investigate the system's ultimate capacity.
- The developed model successfully captured all the failure modes estimated by DIALOG. The first five failure modes are STS yielding, CLT rolling shear

failure, kerf plate connection yielding, concrete topping crushing, and concrete beam shear failure.

• The developed model estimated the strengths for the first five failure modes with difference less than 23%, compared to the estimation by DIALOG.

In the next step, the verified 3D model will be used to investigate the structural performance of the HTFS exposed to both standard and non-standard fire scenarios under different load levels, through coupled fire-structure analysis.

8 – ACKNOWLEDGMENTS

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