# The Arbour: An Innovative Composite Floor System

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## ABSTRACT

The $130M Arbour, a 10-storey, 175,000 ft$^2$, exposed tall wood structure located on George Brown College's waterfront campus in Toronto, Canada, will serve as an educational hub for the College, housing the Tall Wood Research Institute, a childcare centre, and teaching and social spaces. The innovative large-span beamless structural floor system is comprised of cross-laminated timber (CLT) concrete composite slab bands with perpendicular CLT infill panels, all supported on glulam columns. This long-span flat plate system allows for flexibility in architectural programming and unobstructed mechanical distribution; its performance was validated by full-scale laboratory tests.

The project, currently under construction, received external funding through the National Resources Canada GC Wood Program, alongside other partners, to facilitate mass timber innovation specific to this project. This catalyzed a full-scale structural testing regime at the University of Northern British Columbia, investigating timber concrete composite 'slab bands', to determine their performance and develop a low-cost composite connector.

## BUILDING DESCRIPTION

### Introduction

The 'Arbour' building is a 10-storey educational building located for George Brown College in Toronto, Ontario, Canada, that will host classrooms, lecture halls, and the Tall Wood Institute (Figure 1). The project is targeting LEED Gold and Toronto Green Standard v3, Tier 4. The building is 52.5 m high with a footprint of 62×37 m and was designed for the following load conditions as required by the 2017 edition of the Ontario Building Code (OBC) and 2015 edition of the National Building Code of Canada (NBC): a floor superimposed dead load of 2.0 kPa, a floor live load of 4.8 kPa across all interior spaces, stairs, corridors, terrace, etc.; a roof superimposed dead load of 1.0 kPa and roof live load of 1.0 kPa; and a snow load of 1.12 kPa.

To reflect the purpose of the building and to develop sustainable structural solutions, timber was chosen as the primary structural material. Each mass timber floor will be exposed from underneath, and structural concrete topping will be added to achieve the performance conducive to institutional programming. These floors will be supported on glulam columns. From level 2 to 9, CLT panels are used as the primary floor system. To eliminate the use of beams and create more head clearance as well as the space for mechanical and electrical components, panels span 9.2 m in north-south direction to act as slab bands on which thinner CLT panels bear in the transverse direction.

**Figure 1. Rendering of ‘The Arbour’ building (left) and Mass Timber Assembly (right)**

The structure has been designed for a two-hour fire event, with all structural timber fully exposed. A char analysis has been undertaken with the provisions given in Annex B of CSA O86. In addition to this char analysis, supplemental calculations were also undertaken using the MMAH Supplementary Standard, SB-2 Fire Performance Ratings, in subsection 2.11 of the OBC for glue-laminated timber beams and columns. The structural steel in the project will also be required to achieve a two-hour fire-resistance rating and will be achieved through drywall encapsulation. The fire rating and associated alternative solutions were a significant part of this project, undertaken by GHL Code Consultants, alongside CHM Fire.

## Superstructure

The ground floor suspended slab consists of a 300 mm thick reinforced concrete slab supported on concrete columns and walls below. A 100 mm concrete layer will be added on top of the concrete slab over 50 mm of rigid insulation. The transition from concrete columns to glulam columns is made at 500 mm above the ground level slab, as shown in Figure 2a. From Level 2 to Level 8, a slab-band system using CLT panels serves as the primary floor structure and eliminates the need for deep beams, providing more headroom clearance and space for mechanical and electrical components. Seven-ply CLT panels acting compositely with 150 mm concrete topping act as slab bands spanning 9.2 m in the north-south direction and support the non-composite 7-ply CLT panels between them. The slab-bands are typically supported by 430 x 1178 mm glue-laminated columns, or “Wallumns”, that are designed and positioned beneath the slab-bands to resist the effects of unbalanced loading.

While CLT panels offer advantages compared to traditional light-frame wood construction, e.g. improved dimensional stability (Karacabeyli and Gagnon 2019), long-span CLT floors suffer from poor vibration performance. Adding a structural concrete topping and connecting the components with shear connectors results in Timber-Concrete Composite (TCC) floor systems which can overcome some of the inefficiencies associated with conventional reinforced concrete or light wood frame floors (Yeoh et al. 2011, Dias et. al. 2016). Figure 2b shows the 50 mm non-structural and 150 mm structural concrete topping that is being added to the floor structure.

The column-to-column connection, shown in Figure 3, is configured to provide direct load transfer between the vertical elements rather than transmitting forces through the TCC floor panels. Glulam columns will arrive on-site with a steel connecting plate and Hollow Square Sections (HSS) stubs fastened to the end-grain with glued-in rods. The glulam column above will have a similar connection with smaller diameter HSS stubs. Stubs are connected using bolts, which allow for simple installation and can act as a tension connection in the extreme event where a column below is eliminated, according to progressive collapse principles. CLT floor panels will be notched around the HSS tubes and bear directly on the column below.

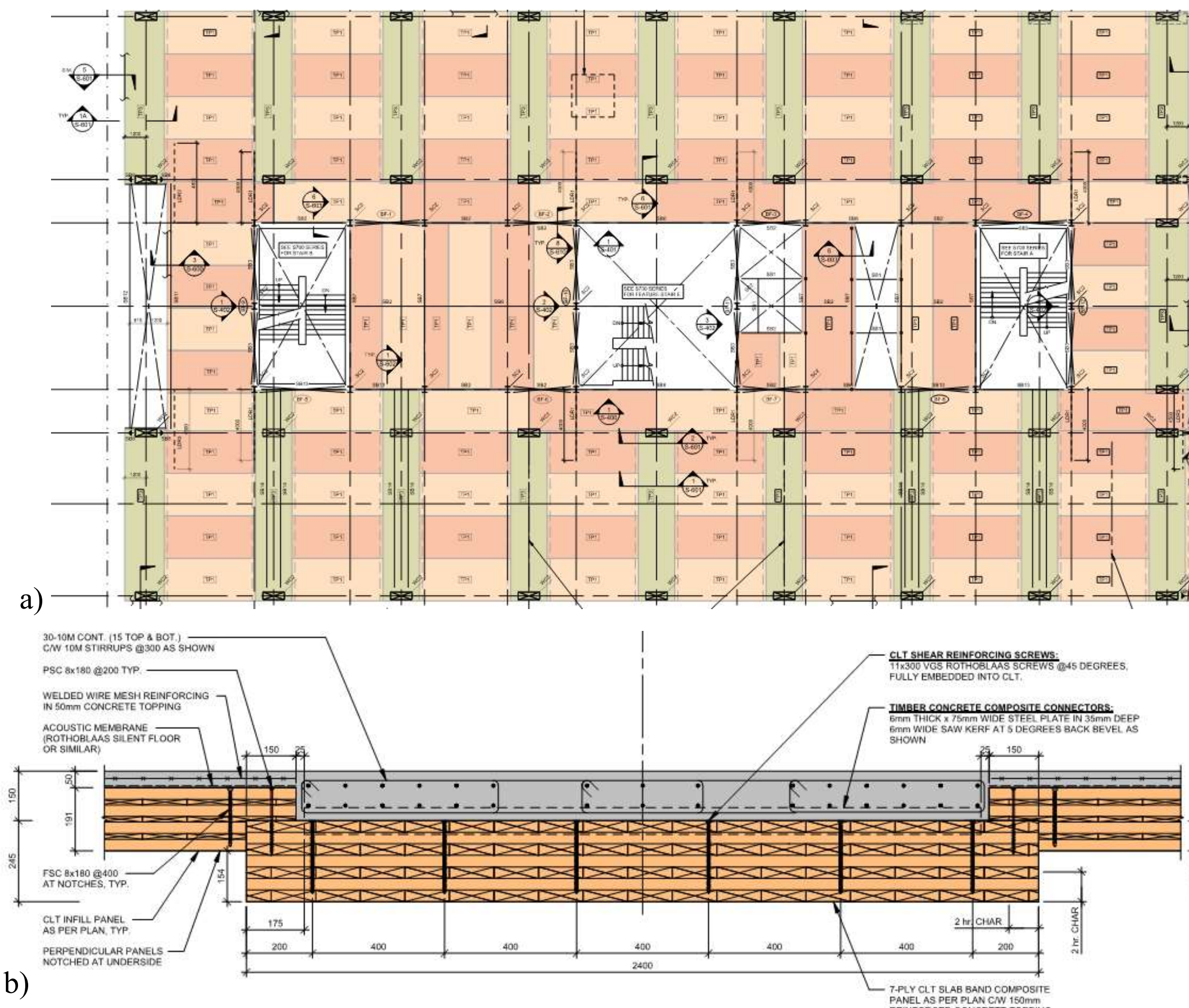



**Figure 2. Typical floor plan (a) and typical slab band section (b)**

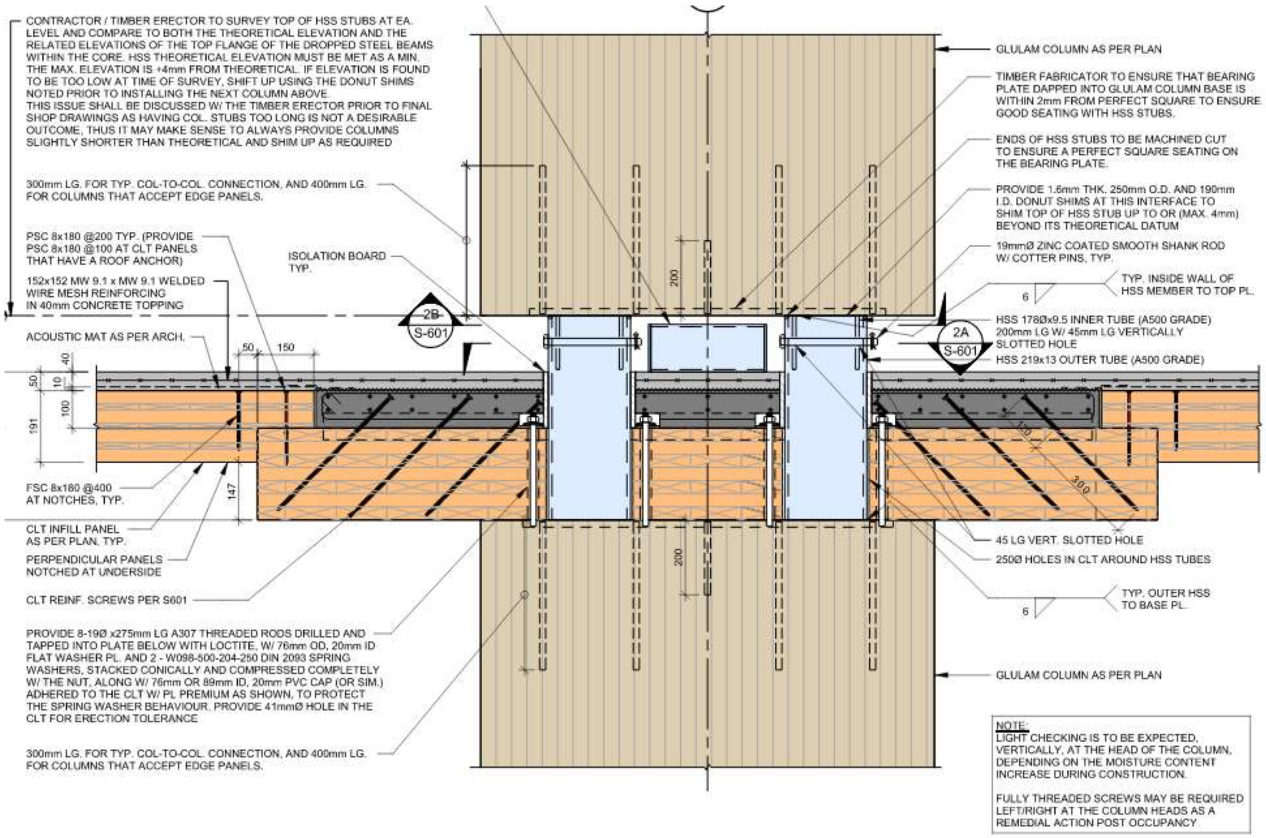



**Figure 3. Floor to Column Connection**

The lower roof at Level 9 will consist of the same structural floor system as described in section 2.4. The CLT floor 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 steel beams underneath each TCC band to support the column loads above. The columns above, which partially support Level 10 and the roof panels, will be anchored to the top of the TCC bands at the mid-span. The typical “wallumns” will be terminated at this level. Where the building continues up to Level 10 and the upper roof, the “wallumns” will transition into smaller 430 x 456 mm glulam columns.

A 244 mm thick 7-ply CLT deck with 50 mm non-structural concrete topping will span north-south between glulam purlins supported by the glulam columns below. The structural steel members in the core area are to remain the same as the floor below. All structural components within this space have been designed for 2hr fire rating. The same glulam columns supporting Level 10 will continue up to support the glulam roof purlins which are 244 mm thick 7-ply CLT roof panels spanning between them, following the slope of the roof. No concrete topping is present at this level at the high roof for envelope reasons. 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, big louvred openings are to be created to serve as solar chimneys.

**Lateral force resisting system**

The primary means of lateral stability will consist of a long central core of steel braced frames coordinated with the stairs, elevators, and services spaces in the center of the building (Figure 4). Several timber options, including CLT shear walls and glulam braces, were explored, as was a concrete core system. Ultimately a steel braced system was found to offer more benefits than the timber and concrete options. Firstly, the steel braces are much more slender than a glulam equivalent, allowing for greater flexibility for wall openings for architecture and required services. Secondly, the steel braced cores add more ductility and overstrength to the overall structure, meaning the design is more efficient as a seismic force-resisting system compared to a timber alternative. They are also a better fit for the project when compared to CLT shearwalls or concrete shearwalls, as the de-centralized mechanical rooms mean a significant number of services are running into the 'core' space, which would otherwise mean many holes in a wall system. Furthermore, the steel components can be erected simultaneously with the timber components, possibly even being pre-assembled as larger units resulting in a fast erection period. The steel braces are designed using HSS, while the beams and columns within the cores will be various wide-flange sections. These steel components will be 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.

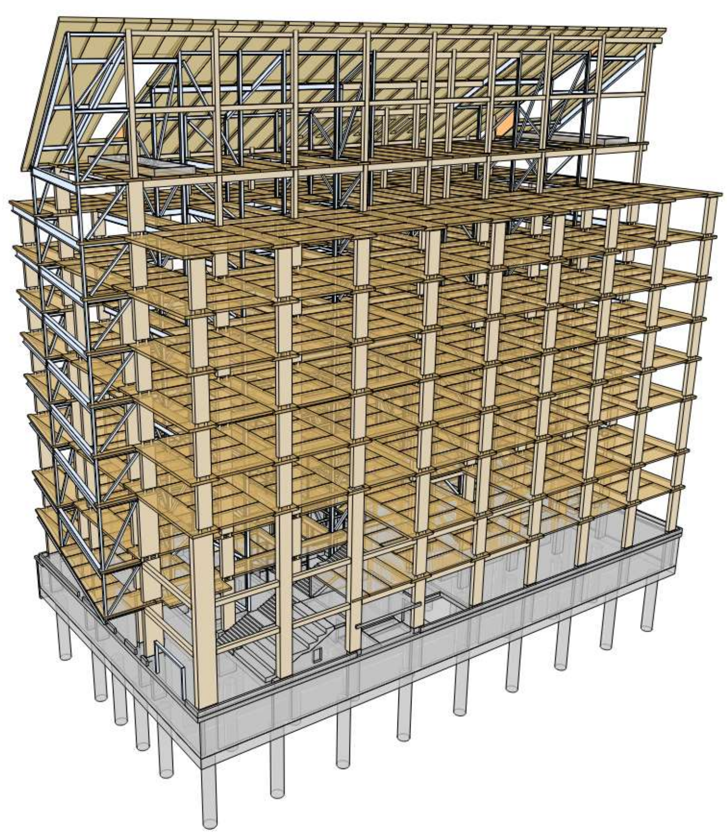


**Figure 4. Schematic of Building Gravity and Lateral Systems**

## Progressive collapse analyses

The National Building Code of Canada (NBCC) is addressed in Commentary B of NBCC 2015 under "Structural Integrity". The hazard and probability of occurrence for accidental events should be identified, and five measures to prevent progressive collapse are recommended. ASCE 7-16 (2016) illustrated the philosophy and design requirements for the progressive collapse in Section 2.5 and commentary C1.4; the design philosophy is based on by ensuring structural integrity which is outlined that "*members of a structure shall be effectively tied together to improve integrity of the overall structure.*" The "England and Wales Regulations" published guidelines for designing building against progressive collapse which included structural robustness requirements (Arup 2011), specific to material types e.g. reinforced concrete, steel and timber, define the tie-force requirements for buildings lower than four storeys to ensure effective horizontal ties. For tall buildings between four to fifteen storeys, the guidelines recommended both horizontal and vertical ties for structural integrity. Also, alternate load paths approach is recommended to ensure practical and economical solutions to bridge over the initial damage. The European code EN1991-1-7 (CEN, 2006) provides provisions for designing against accidental situations.

In "the Arbour" project, progressive collapse has been explicitly considered by following specific local resistance methods. The method suggests that all structural elements and connections which are the essential part of the stability must be designed to resist abnormal loads. Four columns i.e., two corner columns, one edge column, and one interior column were been selected for the analysis (Figure 5). The columns are analyzed for the blast loading of 34 kPa recommended by Eurocode (EN1991-1-7). According to EN1991-1-7, accidental loads shall be applied in both horizontal and vertical directions of the building in addition to service loads. The load combination of 1.2 D + 0.5 L + $A_k$ has been considered (ASCE 7-16, 2016), where D= dead load, L= live load, and $A_k$= accidental load. The maximum utilization under combined loading (axial + bending) was 81% which indicated that the columns have sufficient capacity to absorb the recommended blast loading of 34 kPa. It should be noted that the present analysis did not consider the tie-force and alternate load path method which can be followed for designing a more robust system to prevent progressive collapse.

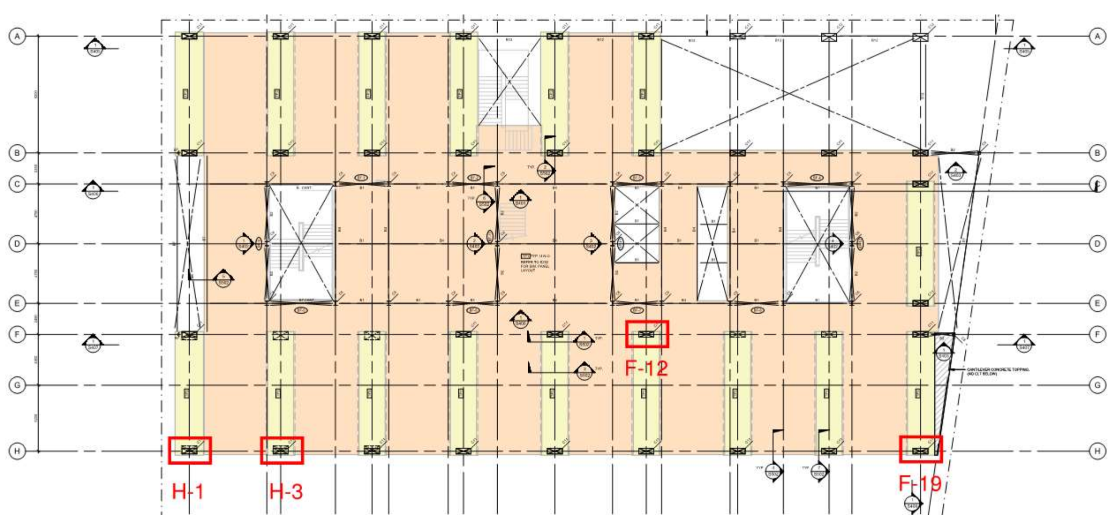
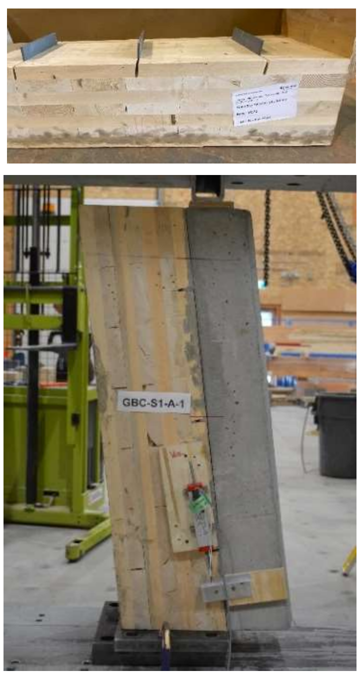



**Figure 5. Columns considered for progressive collapse: corner columns H-1 and F-19, edge column H-3 and interior column F-12.**

## TESTING PROGRAM

### Connector tests

Small-scale tests were conducted to investigate the capacity, stiffness, and failure mechanisms of steel kerf plates as TCC shear connectors. 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 6a. Three varying embedment depths into - 35 mm (series 1A), 70 mm (series 1B), and 90 mm (series 1C) were chosen to investigate the efficiency and failure pattern of steel plates embedment depth into the CLT. A total of 18 specimens were manufactured and subsequently tested at the University of Northern British Columbia Wood Innovation and Research Lab (WIRL) in Prince George. Test results showed that the assumptions of 35 mm CLT embedment for the composite action of the TCC system is sufficient and by increasing the kerf plate embedment depth from 35 mm to 90 mm does not increase the capacity and does change some of the failure mechanisms to rolling shear instead of concrete crushing.

a) 

b) 


**Figure 6. (a) Connector tests; (b) Full-Scale TCC Floor**

### Full-scale TCC floor bending tests

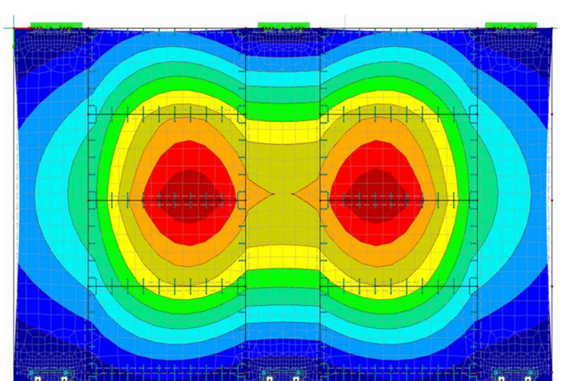
Half- and full-scale specimens were tested under four-point bending. Half-scale TCC floor systems were tested as unreinforced, half-reinforced, and full-reinforced CLT panels for a total of 18 specimens. Full-scale TCC floors were 9.6 m long, with Type 1, 2, and 3 connectors, and 150 mm concrete topping were tested under four-point bending, see Figure 6b. Results showed that CLT reinforcement with self-tapping screws greatly increased the shear capacity of the panels, moving the first mode of failure to bending in some heavy reinforcing instances. The 150 mm concrete topping compositely connected to the CLT panel significantly improves out-of-plane shear capacity by up to 167% when compared to raw CLT panels. TCC with steel kerf plates exhibited high capacity and stiffness and were priced by multiple suppliers to be the most economical option. Findings from this test program have been implemented into the base building design for The Arbour. The steel kerf plate TCC connector solution was deployed for the Arbour with this innovative slab band solution.

## Vibration Performance

The modal frequency of the full-scale TCC slab bands was measured by vibration-based non-destructive impact hammer tests. The accelerations were measured at different locations of the floors e.g., at corners and mid-span to determine the natural frequencies. The result showed that the natural frequency of TCC specimens was very similar varied from 7.8 to 8.0 Hz. Additionally, the TCC floor vibration tests were conducted on each floor according to ISO 10137 and ISO 18324 to determine the floors' natural frequencies and acceleration levels. The walking tests were performed by a 75 kg male evaluator by walking from end to end with a step frequency of approx. 2Hz. The results showed that the natural frequencies varied from 7.5 to 7.9 Hz for the TCC floors.

Finite element models (Figure 7a)were developed to analyze and evaluate the vibration performance of the floors and to compare the experimental results. Three RFEM models of single slab bands were developed: model 1 with no edge loads, model 2 with one sided edge loads and model 3 with edge loads on both sides. A fourth model was developed to evaluate the vibration performance on multiple slab bands with infill CLT panels. The mode shapes from the FE analysis are plotted in Figure 7a. It was found that the natural frequencies reduced with the increase in loads, as expected and the natural frequencies between tests and models were very well aligned.

A footfall analysis was performed to evaluate the vertical response of the TCC slab bands subject to human footfall. The acceleration response was estimated and compared with the acceptable limits. The acceleration response of the RFEM models is shown in Figure 7b. The graph clearly shows that with the increase in line loads along the edge, the acceleration decreases. By modeling multiple slab bands, similar to the actual building case, the acceleration level reduced significantly at the first natural frequency level.

a) 

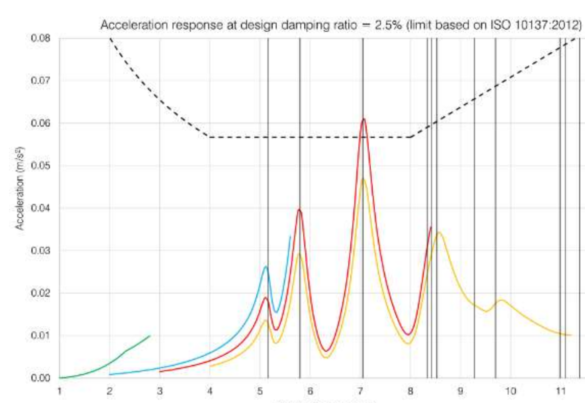
b) 


**Figure 7. a) Modal shape from finite element analysis and b) acceleration response from the footfall analysis**

## CONCLUSIONS

The innovative 'slab banded' gravity system proposed for this project 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 pre-fabricated 'kit of parts', alongside the envelope system. The structural testing program catalyzed by the NRCAN GC Wood program and completed at the University of Northern British Columbia resulted in the selection of a low-cost shear flow connector for the timber concrete composite system, and will provide invaluable design information on TCC systems back to the design community. The building will no doubt become a landmark timber project within North America and will bolster the client's commitment to sustainability. It is currently under construction, with completion scheduled for 2024.

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