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# FEASIBILITY OF CROSS-LAMINATED TIMBER CORES FOR THE UBC TALL WOOD BUILDING

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**ABSTRACT:** The innovation in tall mass-timber buildings is illustrated by the Brock Commons student residence at the University of British Columbia also known as the UBC Tall Wood Building. It is amongst the world's tallest timber hybrid building with 18 stories and 53 meters' height. The building has 17 stories of mass-timber superstructure resting on a concrete podium with two concrete cores that act as a lateral force resisting system for earthquake and wind forces. Construction of the mass-timber superstructure took ten weeks whereas the concrete cores were built in fourteen weeks. There could have been a substantial reduction in the project timeline leading to cost savings, as well as a further reduction of environmental footprint if mass-timber had been used for the cores. The objective of this work was to evaluate the possibility to design the UBC Tall Wood Building using mass-timber cores. A validated numerical model was used to study the feasibility of replacing the concrete cores by cores made of Cross Laminated Timber (CLT). The results presented herein show that, with adjustments in the configuration, the structure can meet the seismic performance criteria as per the Canadian code with CLT cores.

**KEYWORDS:** Brock Commons student residence, Cross-laminated timer, Inter-story drift, Hybrid systems

## 1 INTRODUCTION

### 1.1 Background

Wood, as important renewable structural material, has always been an important part of built environment [1]. The importance of using sustainable construction materials is increasing with growing global demand for housing in a time of accelerating climate change.

Several studies, e.g. [2,3] demonstrated the advantageous environmental balance when using wood or engineered wood products as building material when compared to concrete or steel. With the increasing push towards lower carbon footprint buildings, there is a worldwide effort to develop solutions for mid-rise buildings and preparing solutions for high-rise buildings.

Recent publications such as the 'Technical Guide for the Design and Construction of Tall Wood Buildings in Canada' [4] and 'Use of Timber in Tall Multi-Storey Buildings' [5] are testament to the current interest in tall timber buildings.

This development is also been labelled a 'Renaissance', as already the construction of up to nine-storey post and beam timber buildings with exterior walls made of un-reinforced brick was common in the early 20th century in North America [6].

Changes to building codes driven by large fire disasters limited the height of timber buildings for half a century and only after decades of research and policy discussions, were Building Codes changed to permit six-storey light-frame wood residential buildings [7] and more recently 12-storey mass-timber buildings [8].

During the last decades, several innovative materials, connectors, and systems were introduced that contributed to the tall-timber renaissance. Specifically at the system level, novel hybrid solutions, such as pre-stressed and self-centering systems [9], steel moment frames with mass-timber infill panels [10], mass-timber balloon-frames with steel links [11], the timber-concrete jointed-frame concept [12], or timber-steel hybrid systems [13] are attracting more attention. Construction typologies with prefabricated elements made of engineered wood products are preferred since the process can be quickly industrialized and the on-site mounting time can be reduced; prefabricated elements can be equipped with additional layers to provide the building with full envelope, services, and finishing.

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## 1.2 UBC TALL WOOD BUILDING

Brock Commons, shown in Figure 1, also known as the University of British Columbia's (UBC) Tall Wood Building (TWB), is a 53m high 18-storey student residence with 404 beds for upper-year undergraduate and graduate students. TWB is one of the structures chosen among three tall wood buildings in Canada as demonstration projects funded by Natural Resources Canada [14]. The design and pre-construction phase spanned nine months; construction started in November 2015 and was completed in August 2016 [15].

TWB is a hybrid building with a concrete podium and two concrete cores for elevator and staircases. The remainder of the superstructure is built of combustible engineered mass timber: Glulam columns and Cross-laminated Timber (CLT) floors [16,17]. The concrete cores in TWB act as a lateral force resisting system against seismic and wind forces. Both cores consist 450 mm thick concrete shear walls. Floors were completed by adding a 40 mm concrete topping cast-in-place to the 169 mm thick CLT panels. More in detail, concrete strength classes of 35, 32 and 30 MPa were used for columns and cores, floor topping, and other building components, respectively.

The decision to work with concrete cores was taken in the early stages of the design process to speed up the approval process [15].

*Figure 1:* *UBC Tall Wood Building*

Each CLT panel was directly supported at its corners by the columns. 5-ply CLT panels were used for floors, glued-laminated timber (GLT) and parallel strand lumber (PSL) were used for columns. Further details of the TWB project can be found in [15-17].

CLT floor panels were connected at their edges using single surface splines fastened with 4 mm diameter and 60 mm length nails, 100 mm spaced, and 8 mm and 120 mm length self-tapping screws, 600 mm spaced.

Along the perimeter of the concrete cores, the surrounding CLT panels were anchored horizontally by using steel drag straps (Figure 2a) and vertically with ledgers (Figure 2b). Both drag straps and ledgers were connected to CLT panels by using screws arranged with different orientations, and to the concrete walls by headed steel studs with different lengths.

Column-to-column vertical connections were developed to facilitate the installation of the elements and to accommodate for building settlement. Figure 2c shows the connection prototyped, which is characterized by the use of customized HSS flanged steel pipes fastened with glued rods and tightened using bolts.

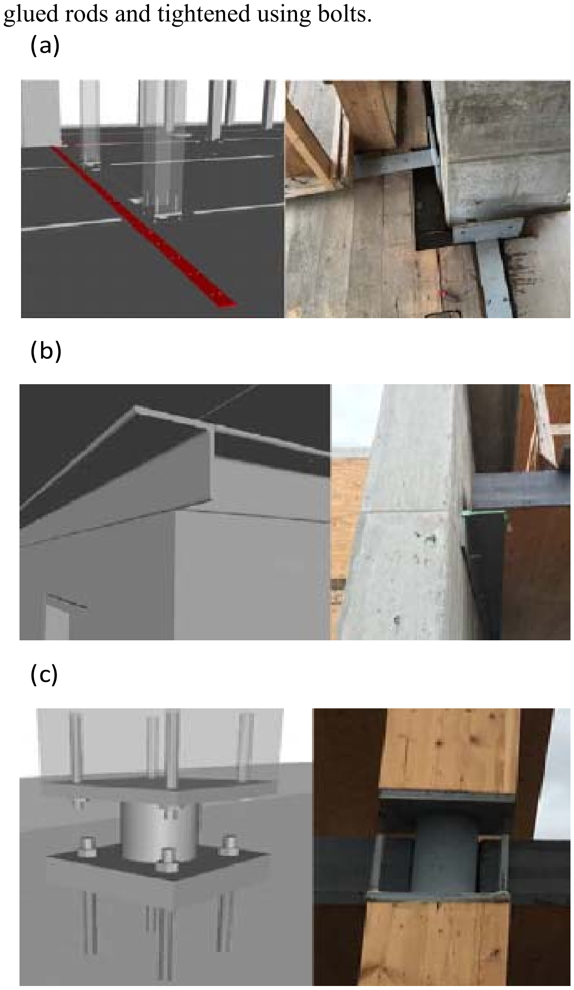


***Figure 2:*** *Joints: (a) steel drag strap connections for floors, (b) ledgers for floors and (c) vertical connections for columns; 3D model and picture from mock-up (Courtesy CADMakers)*

### 1.3 Objective

The construction process of TWB displayed a significant difference in duration for cores and superstructure. The construction of the cast-in-place reinforced concrete cores took more than 12 weeks whereas the mass-timber superstructure including the envelope took just ten weeks. The construction of TWB with mass-timber cores could have undoubtedly decreased the schedule leading to cost savings and environmental benefits.

This notion motivated to investigate the structural feasibility of TWB with CLT cores instead of concrete cores from a seismic perspective according to the National Building Code of Canada (NBCC) [18].

## 2 NUMERICAL INVESTIGATION

### 2.1 Methodology

The impact of changing the core material from concrete to CLT on the structural response was evaluated. The structural plan layout as shown in Figure 3a and the geometry of the original building were maintained. For comparison purposes, the original design solution with concrete cores has been included, labelled herein D1.

In the alterative design solutions 2 and (labelled D2 and D3), schematically shown in Figure 3b, 9-ply 315mm thick E1M5 CLT panels were considered for the cores with the material properties provided by the manufacturer [19]. Finally, in design solution 4 (D4), a modified CLT option with additional 'C-shaped' walls to provide additional lateral strength and stiffness was analyzed, see Figure 3c.

For D2, the core walls were assumed to be monolithic CLT panels, which from a practical point of view is unrealistic. For D3 and D4, a pragmatic and conservative approach, as often applied in practice for preliminary designs, was adopted where the stiffness properties of the core were reduced by 50% to account for the joints distributed along the borders of the timber panels and through the wall height.

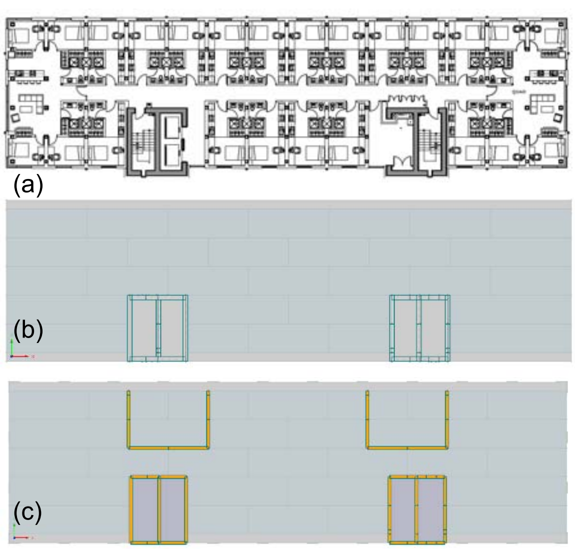


***Figure 3:*** *Floor plan of UBC TWB (a); cores for numerical models for designs D2 and D3 (b) and design D4 (c)*

### 2.2 Design requirements

Design options D1 to D4 were studied considering structural performance as prescribed by NBCC [18]. More in detail, inter-story drift limits of 2.5% and 0.2% (1/500) were considered as required for seismic and wind design, respectively.

For D1, which reflects the TWB design, each concrete core was considered as coupled ductile shear walls system in the X-direction and partially-coupled ductile shear walls system in the Y-direction, the latter to account for large openings. Therefore, ductility, $R_d$, and over-strength, $R_o$, factors were assumed equal to 3.5 and 1.6 in the X-direction and 3.5 and 1.7 in the Y-direction, respectively.

For the CLT design options, without having established details and behavior of the connections, conservative values of 1.0 and 1.3 were assumed for $R_d$ and $R_o$, respectively, as required by CSA 086-16 [20] for systems other than platform-framing with limited aspect ratios of the CLT panels and specific ductility requirements for the connections.

### 2.3 Numerical Model

A model was developed using the commercial software RFEM [21] reflecting the original geometry and assumptions adopted by the structural designers of the building. Appropriate boundary conditions and types of elements were used to describe the response of the structural members of the building. Fixed end-nodes restraints were adopted for concrete columns, slabs and cores, whereas timber elements assumed pinned at their ends. Columns were modelled using 1-D linear elements, while slabs and cores were modelled with 2-D plate elements, see Figure 4.

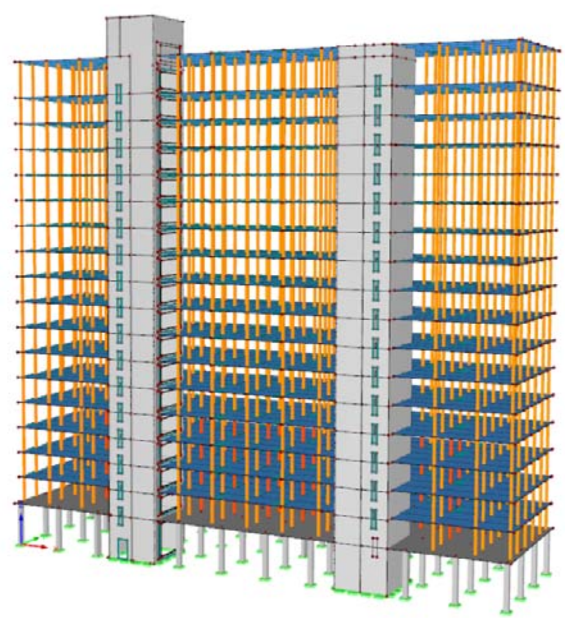

***Figure 4:*** *Numerical Model of UBC TWB*

## 2.4 Analyses

Linear dynamic response spectrum analyses (RSA) were performed using UHS response spectra determined as a function of the building location, and assuming site class C and an important factor of 1.0. Seismic mass adopted in all design options considered the load's combination of 1.0xDead + 0.5xLive + 0.25xSnow. The complete quadratic combination of the resulting modes of vibration with a 5% damping ratio has been used to assess the final lateral loading of the building. To limit computation times, only the first ten modes were considered. The resulting loads were applied to the diaphragms assuming a ± 0.1$D_{nx}$ eccentricity, as per NBCC [20]. Base shear and deflection expressed in terms drift values were derived from the RSA analyses for each of the case previously described.

Dynamic wind analyses were executed following the NBCC procedure [20]. An importance factor of 0.75 and exposure factor for open terrain areas were adopted and reference velocity pressure was set as recommended by the code. Windward and leeward pressure were applied in correspondence of each floor and analyses executed considering wind acting separately along each principal horizontal axis of the building.

## 2.5 Dynamic elastic behaviour

The results of the modal analyses are expressed in the form of periods and mass-participation ratios referred to the total seismic mass. Table 1 reports the 1$^{st}$ ($T_1$) 2$^{nd}$ ($T_2$) and 3$^{tr}$ ($T_3$) mode of vibration that corresponded to the translational modes along the X-direction and along the Y-direction and torsional mode, respectively, as well as the sum of the mass-participation ratios considering the first ten modes of vibration for both the X- ($\sum m_x$) and Y-direction ($\sum m_y$).

The as-built numerical model, as shown in Figure 4, was validated comparing dynamic characteristics as adopted by the structural designers of the TWB [4]. A good match between the buildings' first, second and third period of vibration declared by the designers and obtained from the modal analysis using the numerical model herein presented has been found, see Table 1.

***Table 1:*** *Results from modal analysis*

| Design | $T_1$ (s) | $T_2$ (s) | $T_3$ (s) | $\sum m_x$ (%) | $\sum m_y$ (%) |
|---|---|---|---|---|---|
| D1* | 2.0 | 1.7 | 1.4 | n/a | n/a |
| D1 | 2.0 | 1.5 | 1.3 | 77 | 61 |
| D2 | 4.3 | 3.0 | 2.8 | 87 | 84 |
| D3 | 6.1 | 4.3 | 4.3 | 89 | 89 |
| D4 | 3.9 | 3.5 | 3.1 | 83 | 83 |

* values provided by TWB design company

## 2.6 Lateral response

Results from the modal response spectrum analysis listed in Table 2 are expressed in terms of base shear ($V_{b,x}$ and $V_{b,y}$), elastic values and design values within brackets, and maximum inter-story drift values ($\delta_{max,X}$ and $\delta_{max,Y}$).

***Table 2:*** *Results from response spectrum analysis*

| Design | $V_{b,X}$ (kN) | $V_{b,Y}$ (kN) | $\delta_{max,X}$ (%) | $\delta_{max,Y}$ (%) |
|---|---|---|---|---|
| D1 | 34,285 (6,122) | 36,847 (7,184) | 1.1 | 1.3 |
| D2 | 20,882 (19,322) | 30,209 (23,238) | 1.9 | 2.7 |
| D3 | 22,662 (19,322) | 25,118 (19,322) | 3.5 | 3.3 |
| D4 | 25,056 (19,743) | 27,752 (21,348) | 1.6 | 3.0 |

The inter-story drift profiles for both the X- and Y-direction under seismic loads are shown in Figure 5 and Figure 6, respectively.

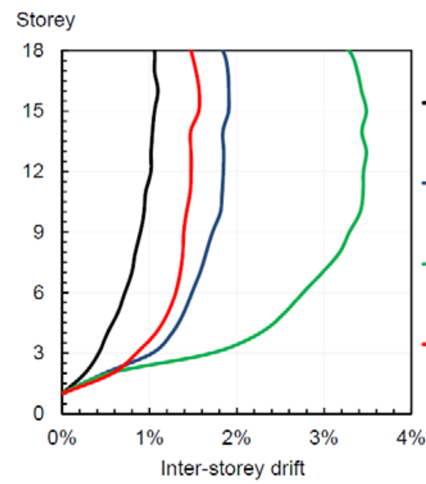


***Figure 5:*** *Seismic inter-story drift profile for the X-direction*

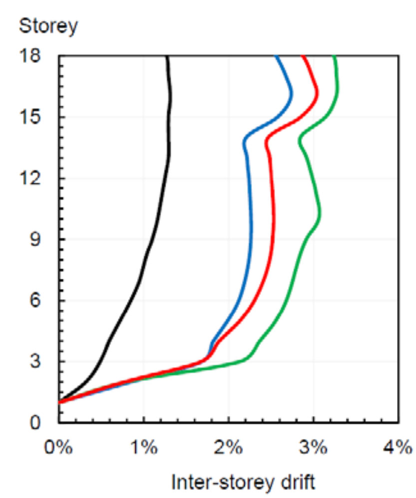


***Figure 6:*** *Seismic inter-story drift profile for the Y-direction*

The resulting base shear and inter-story drift under winds loads are included in Table 3, while Figures 7 and 8 show the matching inter-store drift profiles, for the X- and Y-direction, respectively.

***Table 3:*** *Results from dynamic wind analysis*

| Design | $V_{b,X}$ (kN) | $V_{b,Y}$ (kN) | $\delta_{max,X}$ (%) | $\delta_{max,Y}$ (%) |
|---|---|---|---|---|
| D1 | 615 | 2,319 | 0.02 | 0.04 |
| D2 | 804 | 2,856 | 0.13 | 0.23 |
| D3 | 925 | 3,286 | 0.27 | 0.52 |
| D4 | 747 | 2,633 | 0.07 | 0.32 |

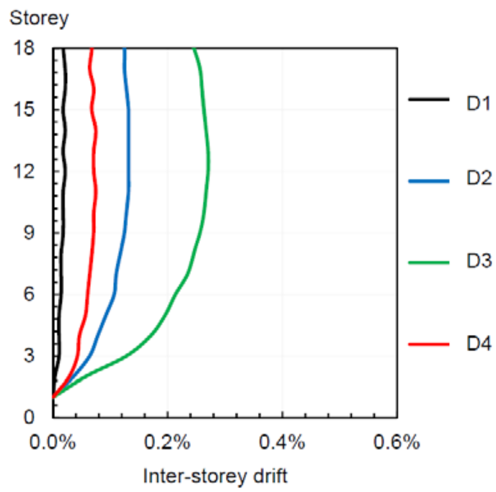


***Figure 7:*** *Wind inter-story drift profile for the X-direction*

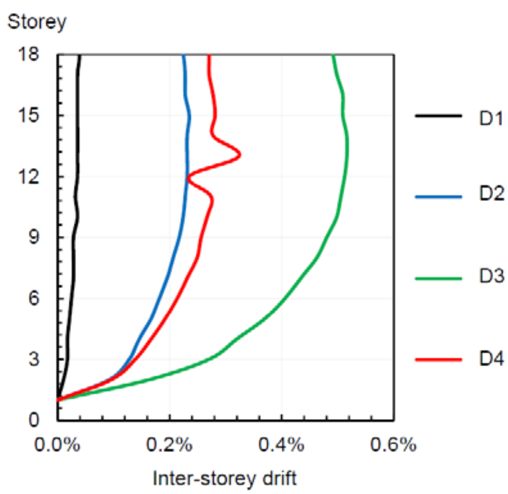


***Figure 8:*** *Wind inter-story drift profile for the Y-direction*

### 2.7 Discussion

None of the analyses for design options evaluated herein fully satisfy the 90% modal participation requirement of the NBCC [5]. Hence, not all significant modes of vibration are included in the results. Nevertheless, other than for the unrealistic design option D2, modal participation for all models was between 83% and 89%. Therefore, the results contained herein can be used for comparative purposes.

Periods of the building increased to values beyond 2 seconds in all CLT solutions and independently of the mode and direction of vibration considered. With respect to the original design (D1), D3 showed a 200% increase for the first three periods, whereas D4 with the additional shear-walls led to a 100% increase in the first three modes of vibration and periods still below 4 seconds.

The substitution of concrete cores with CLT decreased the lateral stiffness of the building, which together with the associated decreased of mass, reduced the acceleration induced by earthquakes. As a result, there has been a significant reduction of the elastic base shear.

Despite this advantage which is often cited as one of the main advantaged for the use of mass-timber products for tall buildings, due to the conservative approach adopted for the reduction factors $R_o$ and $R_d$, the design base shear was found on average three times higher than that of the concrete system.

For the elastic response of building under wind loading, the flexibility of timber-based solutions resulted in an increase of the base shear, approximately 50% for design option D3 and approximately 20% for design option D4.

The inter-story drift increased through the change from concrete to CLT, in particular considering the Y-direction where the large core openings significantly reduced their capacity and stiffness. Even though the X-direction of the building was less critical for both the seismic and wind load combinations, a significant increase of drift was observed. Examining the maximum seismic and wind drifts, it becomes clear that only D4 guaranteed code compliance.

## 3 Conclusions

This research investigated the feasibility of using CLT cores for the UBC TWB. It can be concluded that independently of the connections adopted, it seems not reasonable to pursue the solution with substituting the existing concrete cores with CLT cores considering the commercially available CLT panel thickness in Canada, currently limited to 315 mm. A feasible way to increase the building performance to meet code requirements is the addition of two 'C-shaped' additional cores. As an alternative un-bonded post-tensioned cables can be used to increased stiffness and capacity of buildings [9].

More research is required to investigate and quantify the actual stiffness of CLT balloon-framed cores and shearwalls walls based on different typologies of timber joints. Such research is necessary to provide the ductility and over-strength factors for tall-wood buildings. With further research, code provisions and design rules could be implemented, and that could increase the usage of mass-timber as lateral load-resisting systems.

## ACKNOWLEDGEMENTS

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