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CASE STUDY: AN 18 STOREY TALL MASS TIMBER HYBRID STUDENT RESIDENCE AT THE UNIVERSITY OF BRITISH COLUMBIA, VANCOUVER

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ABSTRACT: This article outlines the structural design approach used for the Brock Commons Student Residence project, an 18-storey wood building at the University of British Columbia in Vancouver, Canada. When completed in summer 2017, it will be the tallest mass timber hybrid building in the world at 53 meters high. Fast + Epp are the structural engineers, working in conjunction with Acton Ostry Architects and Hermann Kaufmann Architekten. Total project costs, inclusive of fees, permits etc. are $51.5M CAD.

KEYWORDS: Tall Wood Buildings, CLT, Mass Timber, Rolling Shear, Prefabrication, Tolerances, Damping

1 INTRODUCTION

Brock Commons is an 18-storey mass timber student residence currently under construction at the University of British Columbia in Vancouver, Canada. The 53-meter-tall structure is comprised of 16 floors of five-ply cross laminated timber (CLT) floor panels, point supported by glulam columns on a 2.85m x 4.0m grid. Beams were eliminated from the design by utilizing CLT's two-way spanning capabilities. A single-storey concrete podium at the base, a prefabricated steel roof, and two full-height concrete cores for lateral stability complete the structure. See Figure 1 and 2 for building renderings.

Figure 1: *Building Rendering*

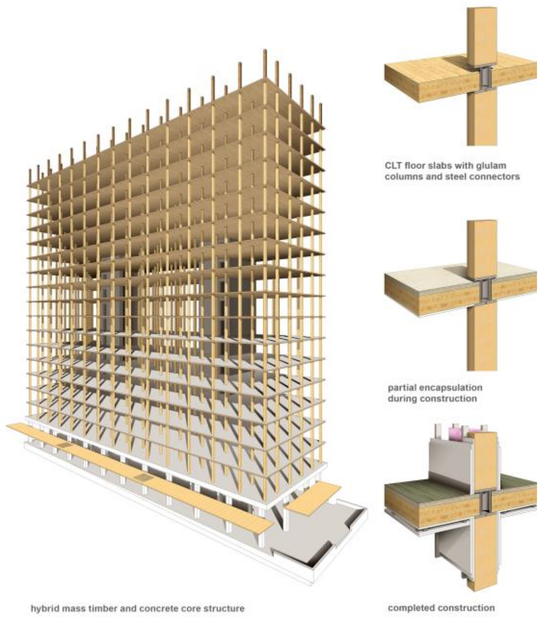

Figure 2: *Building Component Rendering*

Building design began in December 2014, and construction commenced in October 2015, just 11 months later. In addition to the construction capital funding put forward by the University of British Columbia for the project, four external agencies also contributed funds for research and design, promoting the mass timber building and engineering industry in Canada.

The intent of the project was to create a mass timber building whose construction cost would be on par with the unit cost of a concrete tower in the local construction market. This building demonstrates that mass timber is a viable option in the high-rise market.

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2 CODES & STANDARDS

The current British Columbia Building Code (BCBC 2012) limits the height of wood buildings to six storeys[1]. As such, a special approvals process was required for this project. The design is based on a Site Specific Regulation (SSR), administered by the Building Safety and Standards Branch of the BC Provincial Government and applicable solely to this project and site.

One specific requirement of the SSR was that the building be designed according to the not-yet-adopted 2015 National Building Code of Canada (NBCC) rather than the prevailing BCBC 2012. The main impact of this requirement was an increase in the applicable seismic acceleration values, which are approximately 50% higher than those associated with the older code at a 2-second period.

Two independent structural engineering peer reviews were completed for this project, as mandated by the SSR. The first independent review was completed by Merz Kley Partner ZT GmbH in Dornbirn, Austria. The second was completed by Read Jones Christoffersen Consulting Engineers in Vancouver, Canada. An in-house structural engineering peer review was also undertaken at Fast+Epp.

The SSR also specifies the fire rating requirements of this unique building system. Allowing the columns and CLT panels to char in a fire event is indeed a feasible and accepted engineering solution to achieve the required fire rating of two hours; however, type X drywall cladding was selected as the fire protection strategy to expedite the approvals process, and to raise the acoustic performance of the units.

This project received the second SSR issued in the province, following in the footsteps of the 6-storey Wood Innovation and Design Center project at the University of Northern British Columbia in 2014.

3 STRUCTURAL SYSTEM

3.1 DESIGN APROACH

In order to meet the cost objectives for the project, a disciplined team design approach had to be employed. The intent was to keep the structure simple and sensible: develop a prefabricated “kit-of-parts” that could be installed quickly and easily, with minimal labour on site. Materials were used where they made sense.

Early on in the design phase, the construction manager was appointed, and the timber installers/supplier and concrete trades joined the team in a design-assist role. They provided real-time feedback on the evolving structural design, gave schedule projections, and offered valuable constructability advice.

To help achieve a high level of prefabrication for all design disciplines, a third-party consultant modelled the building and helped coordinate design documents prior to and during construction. This 3D model, created with CATIA software, includes fully detailed structural elements and connections, as well as mechanical/electrical systems and architectural fit-outs. The model allowed all CLT penetrations for mechanical and electrical sleeves to be fully coordinated during the design process and to be converted into fabrication files (CAD/CAM) needed to for CNC machining. Component renderings from this model can be seen in Figure 3 and 4 below.

Figure 3: *Concrete Elements*

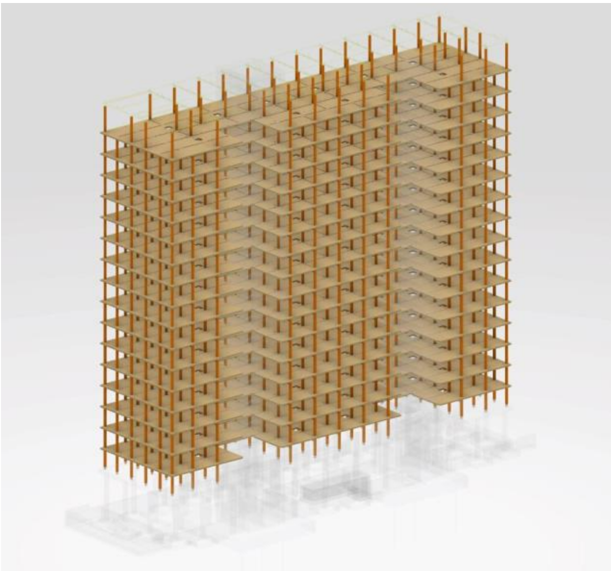

Figure 4: *Timber Elements*

3.2 GRAVITY SYSTEM

CLT is often used as a one-way decking system, largely ignoring the two-way spanning capability afforded by its cross laminations. By using CLT to span in both directions, the design team was able to eliminate beams, significantly reducing the structural depth, and thereby creating a clean, flat, point-supported surface allowing for unobstructed service distribution as in flat-plate concrete construction. Further, by adjusting the column grid and architectural program to suit the maximum available panel size, the team was able to both minimize the overall number of panels (and therefore the number of crane picks) and maximize structural efficiency.

Finite-element analysis of the CLT panels was carried out with Dlubal RFEM software with an add-on module to analyze laminated shell elements. Approximations and hand calculations using first principles were used to validate results from the software. Based on an initial analysis, the moment and deflection demands exceeded the capacities of standard 5-ply panels available in western Canada. To achieve the required performance, a custom layup was designed using machine stress-rated spruce laminations on the outer layers with an elastic modulus of E = 10 300 MPa and a specified bending stress capacity of f_b = 23.9 MPa. The inner layers are based on typical No.1/No.2 spruce-pine-fir (SPF) lumber with E = 9500 MPa and f_b = 11.8 MPa[2].

In addition to stiffness and bending requirements, rolling shear stresses at supports are typically a controlling factor in two-way, point-supported CLT floor plates. A rolling shear failure is one in which the fibers "roll over" each other due to shear forces perpendicular to grain. The only North American code or material standard reference that speaks to this issue is the CLT fabrication standard ANSI/APA/PRG 320, which specifies rolling shear resistance values, f_s, for CLT panels at approximately one-third of longitudinal shear resistance values, f_v[3]. However, recent research by Mestek, et.al indicates that rolling shear capacities are higher in point-supported panels due to added restraint from the high compressive forces near the supports[4].

A simple method for calculating the rolling shear stresses in point-supported panels is to approximate the shears based on tributary area and panel geometry (Figure 5), similar to the procedure outlined by Mestek, et.al[4].

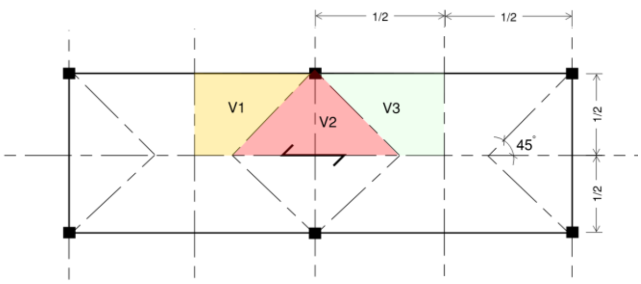

Figure 5: *Plan View of CLT Panel – Shear Approximation*

The rolling shear stresses can be found based on the directions of load transfer and the layup orientation, at each section of the critical perimeter. See Equation (1).

$$\tau_1 = \frac{(1.5)(V_1)}{(B_1)(D_1)} \tag{1}$$

Where τ = rolling shear stress (MPa), V_1 = load based on tributary Area (N), B_1 = distance to critical section (mm), D_1 = Panel depth (mm). See Figure 6 below for clarification. Note that Equation (1) applies to V_3/τ_3 as well, by way of symmetry.

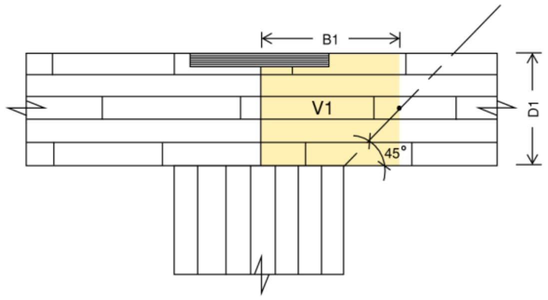

Figure 6: *V1 or V3 Shear Area*

The same principles can be applied for τ_2, as shown in Equation (1) below:

$$\tau_2 = \frac{(1.5)(V_2)}{(B_2)(D_2)} \tag{2}$$

Where τ = rolling shear stress (MPa), V_2 = load based on tributary Area (N), B_2 = distance to critical section (mm), D_2 = Depth to the furthest perpendicular lamination fibre (mm). See Figure 7.

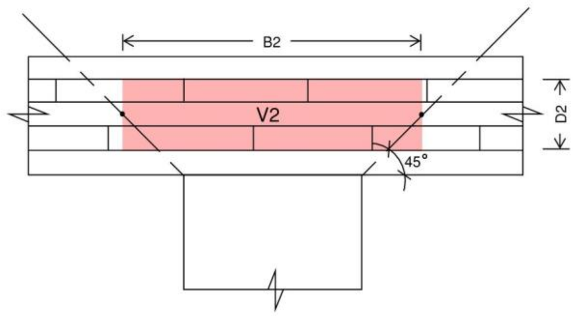

Figure 7: *V2 Shear Area*

There are 78 columns per floor, resulting in 1248 CLT panel nodes in the building where rolling shear stresses could be the controlling design factor. Reinforcing the panel points with inclined fully threaded screws was not a desired solution due to the significant increase in material cost and labour.

After designing the custom layup to suit the rolling shear and flexural demands, the design team completed 18 full-scale load tests on panels from three prospective CLT suppliers, at FPInnovations laboratory in Vancouver, BC[5]. The testing apparatus can be seen in Figure 8, where two-span continuous panels on six point supports were loaded along their centrelines, at quarter points.

Figure 8: *CLT Testing Apparatus*

Based on the test results and failure modes observed, rolling shear capacities are significantly higher than the values published in ANSI/APA/PRG 320[3]. In addition, there appeared to be some capability for the CLT to redistribute forces as internal shear cracks propagated through the panel before the critical failure mode occurred. Multiple types of shear/bending failures were observed near the supports (Figure 9).

Figure 9: *CLT Shear Failure at Support*

Two types of panels were tested, one with an 800mm x 800mm mechanical service penetration, and one solid panel, both of which were two span continuous. Loading results can be seen in Table (1 & 2) below[5].

Table 1: *Testing Results For Panels Without Service Holes*

Testing Results For Panels Without Service Holes			
	Average Failure Load	Factored Design Load	Ratio
Supplier #1	374 kN	210 kN	1.8
Supplier #2	358 kN	210 kN	1.7
Supplier #3	402 kN	210 kN	1.9

Table 2: *Testing Results For Panels With Service Holes*

Testing Results For Panels With Service Holes			
	Average Failure Load	Factored Design Load	Ratio
Supplier #1	362 kN	143 kN	2.5
Supplier #2	340 kN	143 kN	2.4
Supplier #3	388 kN	143 kN	2.7

The solid panels without service holes controlled the panel design, as they were subjected to heavier corridor loading. By taking off the load amplification factors, as well as the material strength reduction factors (ϕ), it is desired to see testing results at least 2.0 times the calculated capacities.

Additional information on the CLT testing can be found in the WCTE 2016 “Structural Behaviour of Point-Supported CLT Floor Systems” paper by Popovski, et.al.

3.3 LATERAL SYSTEM

The primary lateral support for earthquake and wind loading consists of two concrete cores. Although timber-based lateral force-resisting systems such as CLT walls/cores, timber braced frames, or post-tensioned/self-centering systems were feasible design options for this project, the testing, time, and costs required to obtain regulatory approvals would have negatively impacted the client’s budget and completion date.

The cores were designed as ductile concrete shear walls in the shorter, north/south direction and partially coupled ductile concrete shear walls in the longer, east/west direction. Building fundamental periods are T=2.0s east-west and T=1.65s north/south; the design base shear correlates to 4.5% of the building weight.

In addition to the cores, the floor diaphragms are a critical part of the lateral system and must be designed to remain elastic when the cores yield in flexure[2]. This requirement results in diaphragm design forces of up to V_f = 16kN/m in the CLT panels and connections for this structure. The CLT spline connection consists of continuous Douglas Fir plywood splines, nailed into CLT dadoes with ring shank nails to transfer in-plane diaphragm shear forces. Additional partially threaded screws transfer vertical shear across the joint and ensure a flush panel-to-panel fit (Figure 10).

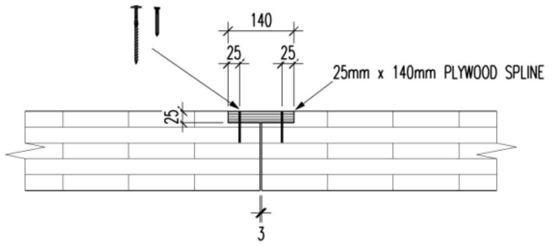

Figure 10: *CLT Spline Detail*

To drag diaphragm forces into the cores, steel straps were fastened to the CLT floor plates with partially threaded screws and bolted to cast-in embed plates (Figure 11).

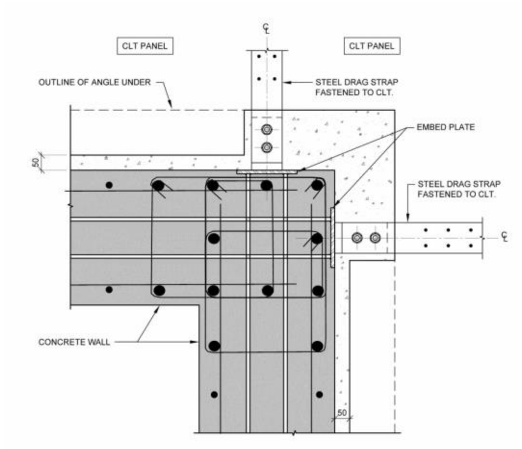

Figure 11: *Drag Strap Detail at Core*

Similar to the diaphragms, the raft slab foundation was designed as a "capacity-protected" element, in this case to resist overturning moments equal to the probable flexural capacity of the cores[6]. The probable capacity of the cores is calculated by removing material safety factors and increasing the yield strength of the reinforcing by 25%, resulting in a calculated capacity of approximately two times the design value.

4 SPECIAL CONSIDERATIONS

4.1 PRE-FABRICATION

Pre-fabrication needs to be an essential consideration when designing tall wood structures. Well-planned erection and shop drawings are key to ensuring smooth production and installation of timber elements. This results in less errors on site, less remedial work, and a shorter overall construction schedule. All CLT and glulam elements are CNC machined with quality control protocols to better ensure a seamless erection of the timber superstructure.

4.2 COLUMN SHORTENING & SHRINKAGE

In tall wood buildings, axial column shortening needs to be considered during design. When properly accounted for, the shortening should not negatively affect the construction, use, or long-term performance of the building.

Several factors affect glulam column shortening:

- Dead load elastic axial shortening ($\Delta = PL/AE$)
- Live load elastic axial shortening ($\Delta = PL/AE$)
- Shrinkage parallel to grain
- Joint settlement
- Column length tolerances
- Wood creep

The main concern surrounding these shortening effects is the impact the deformations can have on the vertical mechanical services and the elevation tolerances between the wood superstructure and the stiff concrete cores. The effects of these factors culminate at the roof level, where all columns below contribute to the shortening. Figure 12 shows the nearly 50mm of estimated deflection at the roof if left unmitigated.

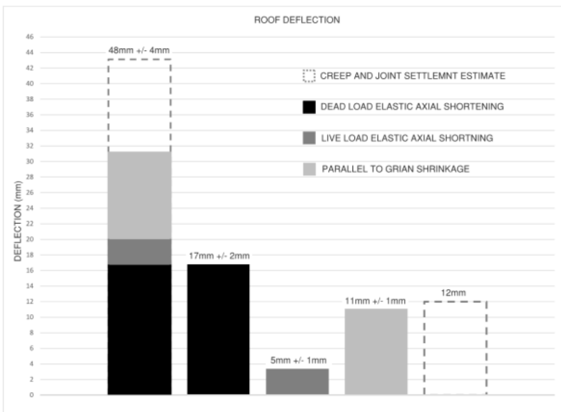

Figure 12: *Cumulative Axial Shortening at Roof*

The strategy for mitigating these effects is to add a series of 1.6mm-thick steel shim plates at the column-to-column connections at three strategic levels: L7, L11, and L15. The total shim package thickness varies with assumed stresses to correct for differential shortening. Due to expected variations in elastic modulus and a degree of uncertainty surrounding the anticipated shortening values, only 50% of the calculated deformations are being shimmed in order to avoid overcompensation. This approach results in approximately five 1.6mm-thick shims at each of the three reset levels (24mm total). Continuous mechanical stacks and HVAC services have been designed to accommodate up to 32mm of deflection.

A detailed design of the connection can be seen in Figure 13: hollow structural section (HSS) spigots with base plates slide into one another to provide steel-to-steel bearing. At the strategic levels, the shim plates are installed at this bearing interface.

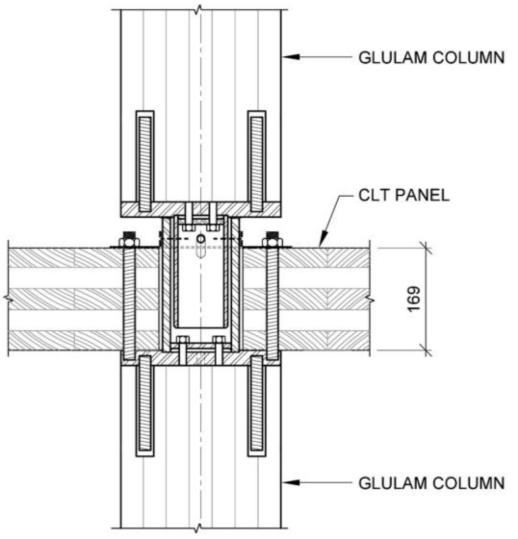

Figure 13: *Column to Column Connection*

As the practical vertical tolerance of the cast-in-place concrete door and elevator sills is fairly significant at +/- 19mm, details had to be created to accommodate the axial shortening of the adjacent timber structure against the cast sill elevations (Figure 14). The concrete cores are installed prior to the timber superstructure and do not support significant gravity load other than their self-weight. Therefore, it is likely that the small amount of creep and elastic axial shortening will have taken place before the timber is installed, and could therefore be ignored.

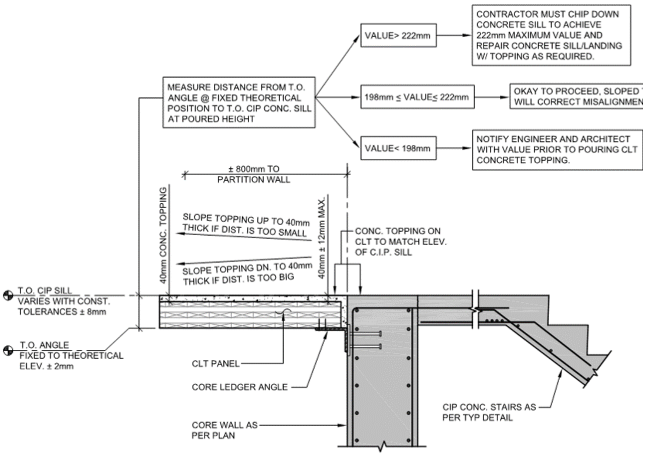

Figure 14: *Detail at Core Sill*

The core ledger angle picking up the CLT is welded to concrete wall embeds and is installed at its theoretical design datum. The dimension between the angle and the cast-in-place concrete door or elevator sill is being measured at each level to determine if the in-situ discrepancy from theoretical can be made up by sloping concrete topping up or down within allowable dead load limitations. In cases where the angle is too low, and thereby topping dead load would too high, the cast-in-place sills and stair landings have to be chipped down to accommodate.

4.3 PROGRESSIVE COLLAPSE

Progressive collapse is the sequential spread of local damage from an initial event from element to element, resulting in the ultimate collapse of the building or a portion thereof. The mitigation strategy for this project is the column tie method as outlined in EN 1991-1-7[7]. By installing a bolt and cotter pin through the steel tube sections and epoxying the threaded vertical rods from the plates into the column ends, an effective tension tie was created. The floor above will essentially suspend the floor below in the event of a column failure.

4.4 DYNAMIC WIND-INDUCED VIBRATIONS

The NBCC 2015 requires a dynamic wind load analysis when structures are greater than 120m tall, have a height to width ratio of greater than 4.0, or have natural frequencies of less than 1.0 Hz. As this structure's first mode has a frequency of 0.5 Hz, a dynamic wind load analysis was undertaken. The building's vibration behaviour was studied using a finite-element model with a 10-year return period for wind loading. To ensure occupant comfort, the building was designed to limit wind-induced accelerations at level 18 to 1.5% of gravity (15 milli-g).

As part of this analysis, a value for damping needed to be assumed, although limited research has been completed on the topic of damping in tall timber buildings. A recent FPInnovations paper reports in-situ damping ratios for three mid-rise mass timber buildings with varying lateral force-resisting systems[8]. The values, determined with ambient vibration testing, range from 1.0% to 4.0%. The report also suggests that damping increases significantly when non-structural components such as finishes and partitions are included. Other reports from various sources confirm these damping values. Based on this information, a sensitivity analysis was performed with various damping values between 1% and 3%. Accelerations at level 18 fell within the design limits with a damping value of at least 1.5%. For a student residence building with multiple full-height partitions on every floor, this damping value seems reasonable. These damping assumptions will be monitored with building sensors, which is further discussed in Section 4.8.

4.5 PROOF OF CONCEPT MOCK-UP

To help improve the constructability of the proposed design, the construction team completed a full-scale mock-up of a portion of the building, 8 m x 12 m in plan and two storeys tall (Figure 15 & 16). The mock-up included several connection types to help determine which to use in the final design and how to optimize them. In addition to the structural lessons learned, the mock-up was also used for the development and evaluation of various building envelope systems considered for the project.

Figure 15: *Proof of Concept Mock-up (1)*

Figure 16: *Proof of Concept Mock-up (2)*

Three different column-to-column connections were used in the mock-up. The first connection was very simple with a wood pedestal/tennon machined into the top, two locator pins, and a thin steel bearing plate to allow for uniform end-grain bearing. The second connection was similar to the first but with glued-in rods and a thicker bearing plate in an effort to provide some base fixity to the column.

The steel HSS column-to-column connection described above was selected over the two different wood-to-wood connections, as it proved the easiest to erect and shim while also providing the tightest tolerances and smallest required column size, thus reducing the overall volume of glulam. The most notable improvement from the mock-up prototype to the final version was the revision from welding the threaded rods and HSS to the steel plate to drilling and tapping them with a CNC machine. The welding of the rods and HSS caused warping and cupping of the steel plate, which adversely affected the bearing connection. By switching to a CNC-machined detail, the overall length tolerances could be reduced to +/- 0.5mm, better matching the dimensional tolerances of the CNC-machined timber elements.

4.6 CONSTRUCTION TOLERANCES & SEQUENCING

A carefully detailed structural system that accounts for construction tolerances can greatly reduce the risk of on-site issues causing delays and undue costs to the owner. All material interfaces were identified and evaluated in regards to their material standard tolerance compatibility. Where required, the details were adjusted to overcome the discrepancies, where in other cases, the specifications required a project-specific adjustment. Additionally, quality control (QC) requirements have been outlined in the specifications for all CLT and glulam elements.

In order to facilitate the use of one crane and provide sufficient time for manufacturing and shipping of the heavy timber elements, the construction team is erecting all 18 stories of the concrete cores before the wood arrives on site. Once the cores are fully erected, the crane will switch over and begin to install the timber and envelope, which is scheduled to begin in May 2016. The prefabricated curtain wall system is clad with Trespa® panels, consisting of 70% wood-based fibres and resin. The envelope panels will be installed in 8m-long segments, in parallel with the timber erection. This sequence will provide weather protection at each floor and allow building fit-outs to begin immediately.

The timber and envelope installation sequence will be completed in four phases. The first will be erecting all columns on one level, diagonally bracing them, and using horizontal spreader braces at the column caps to set the grid. The columns will be installed by hand from bundles on the active deck, freeing up the crane for envelope panel installation. The second phase is the installation of the CLT panels, stitching adjacent panels as the active deck moves away from the cores. The third is the installation of the steel drag plates and perimeter angles supporting the curtain wall system. The fourth is the installation of the envelope panels on the floor below the active deck. This installation sequence will repeat itself. Currently, the anticipated erection time of the timber and envelope elements is one week per floor, but efficiencies are expected. Concrete construction progress as of March 2016 can be seen in Figure 17.

Figure 17: *Current Progress.*

4.7 WEATHER PROTECTION STRATEGIES

A considerable effort was made to complete the design and all concrete work to level 18 during the winter and spring months, in order to have a summer installation window for the timber and envelope elements. The timber is scheduled to arrive on site in late May 2016. However, with Vancouver experiencing significant rainfall throughout the year, sometimes even in the summer months, a weather protection strategy had to be created.

The first strategy, as previously noted, is to bring up the timber elements with speed, in tandem with the envelope panels. The second strategy is to provide a temporary coating on the exposed face of the CLT panels to repel moisture when it rains. Testing was completed on multiple sealers by RDH Building Engineering Ltd. to select the product. The third strategy is to install peel-and-stick tape over all machined mechanical penetrations and along the splines to stop water from penetrating to the floor below. Lastly, a sink-style drain will be created with plywood at each of the ten 800mm x 800mm mechanical shafts per floor. This will allow the water to be effectively managed on site.

Temporary cover solutions were considered for the project but were found to be too costly and restrictive to the erection sequencing.

4.8 MONITORING

In an effort to better understand the unique behaviours of this building, the structure will be fitted with accelerometers, moisture meters, and vertical shortening string pots. Research teams at the University of British Columbia and SMT Research Ltd are undertaking this work.

The accelerometers will allow the research teams to determine in-situ damping values from ambient vibration testing (wind). These values will help to determine a baseline damping ratio for future hybrid buildings of this type, specifically useful for dynamic wind acceleration calculations discussed in Section 4.3. Additionally, sensors will be placed on the concrete cores to record the building's angle of inclination during a seismic event. The data collected from the accelerometers and inclination gauges will help to verify the building's performance in a significant seismic event.

The string pots will measure the floor-to-floor axial column shortening at strategic levels, which will provide more insight into axial column shortening in highly loaded glulam columns.

Lastly, moisture meters (Figure 18) and data loggers will be installed in the CLT panels, collecting data from the manufacturing plant to the final installed condition. The meters will continue to measure moisture content throughout the service life of the building. In a few years' time, this will give an effective moisture content timeline from fabrication to moisture equilibrium.

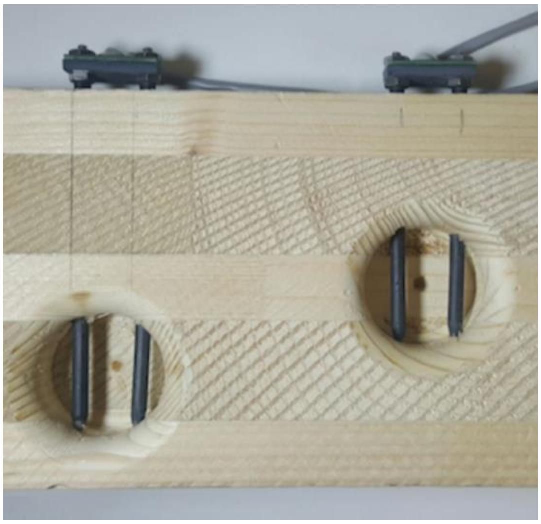

Figure 18: *Moisture Meter Probes*

4.9 FURTHER INFORMATION

Table 3: *Project Data*

Project Data	
Owner/Client	UBC Student and Hospitality Services & UBC Properties Trust
Construction Manager	Urban One Builders
3D Coordination Consultant	CadMakers Virtual Construction
Timber Manufacturer	Structurlam Products
Timber Installer	Seagate Structures
Concrete/Rebar	Whitewater/LMS/Lafarge
Wood (m^3)	2120m^3
Concrete (m^3)	2740m^3
Total Cost ($ CAD)	51.5 M (Including Soft Costs)
Projected Completion Date	Summer 2017

5 CONCLUSIONS

This paper outlines some of the unique engineering challenges associated with this mass timber building, and the design strategies used to overcome them. The project has proven to be cost-competitive with concrete towers in the local marketplace, which was largely achieved by an integrated design team, real-time input from trades, and structural discipline.

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