TALLWOOD 1: LESSONS LEARNED ON COMPLETION OF CANADA'S FIRST 12 STOREY TIMBER-STEEL HYBRID BUILDING

Ornagh Higgins[1], Ilana Danzig[2], Brendan Fitzgerald[3], Jackson Pelling[4], Mehrdad Jahangiri[5]

ABSTRACT: This paper presents the design considerations and construction challenges for the District 56 Tallwood 1 project; a 12 storey mixed-use steel-timber hybrid tower constructed in the South-West region of British Columbia, Canada. The project was completed in 2022 and is located in a region with some of the highest seismic demands in the country. The structural systems made use of pre-fabricated timber and steel components with the goal of decreasing on-site assembly time. This paper describes the structural systems chosen to resist both gravity and lateral loads, design for fire resistance, robustness, and lessons learned during construction.

KEYWORDS: Tall Timber Buildings, Glulam, CLT, Mass Timber, Hybrid, Ductile

1 INTRODUCTION

Tallwood 1 at District 56 is a 12-storey mixed-use residential and commercial mass timber tower located in Langford, BC, Canada. The project is the first of its kind to incorporate the newly adopted British Columbia Building Code (BCBC) [1] provisions for Encapsulated Mass Timber Construction. At the time of completion in 2022, it was also the tallest steel-timber hybrid structure in Canada.

The design takes advantage of prefabricated mass timber panels and pre-assembled steel braced frame components to increase the speed of assembly on site.

Figure 1: *Tallwood 1, completed construction*

This paper will present the solutions chosen for the gravity and lateral systems and discuss the advantages of each. The approach towards fire protection and robustness will be explored as well as challenges faced during construction and lessons learned.

Key Project Info

Location: Langford, Victoria, British Columbia

Height: 41m (12 storeys above ground + 2 levels below ground)

Typical storey height: 3.0 m

Typical floor Area: 1,175 m^2

Total Area: 18,300 m^2

Gravity Super Structure: SPF CLT & Douglas Fir glulam

Lateral structure: Steel Eccentrically Braced Frame (EBF)

Main level & below ground structure: Concrete

Figure 2: *Key project information*

2 STRUCTURAL DESIGN

2.1 GRAVITY FRAMING

The building consists of 11 storeys of mass timber construction with residential occupancy over a one-storey concrete podium with commercial occupancy. There are two levels of underground parking. A concrete transfer structure at L2 accommodates the different column grids between the different structural systems and occupancies.

[1] Ornagh Higgins, ASPECT Structural Engineers, Vancouver, British Columbia, Ornagh@aspectengineers.com
[2] Ilana Danzig, ASPECT Structural Engineers, Vancouver, British Columbia, Ilana@aspectengineers.com
[3] Brendan Fitzgerald, ASPECT Structural Engineers, Vancouver, British Columbia, Brendan@aspectengineers.com
[4] Jackson Pelling, ASPECT Structural Engineers, Vancouver, British Columbia, Jackson@aspectengineers.com
[5] Mehrdad Jahangiri, ASPECT Structural Engineers, Vancouver, British Columbia, Mehrdad@aspectengineers.com

https://doi.org/10.52202/069179-0551

Gravity framing for the typical residential levels consists of point supported cross-laminated timber panels, spanning in two directions, supported on glue-laminated timber columns. The flat-slab structural system was chosen to minimize the overall floor-to-floor height and to reduce the total number of structural elements to be installed.

The typical residential column grid is 2.95m by 3.60m to accommodate maximum panel widths available from local CLT suppliers. The architectural size and layout of the residential units generally allow for columns to be located within partition walls. Additional glulam beams are used in select locations at larger living rooms or around the elevator openings, where the typical column spacing could not be maintained. The floor plates are identical at all levels with the exception of the penthouse, where longer spans were required to meet the architectural intent.

The typical CLT panel used is a 5 ply panel, 175mm in depth, with two way spanning capabilities. The critical panels are located in the corridor areas where the live load design requirement is 4.8kpa, the design is governed by the minor axis bending moment capacity and deflection. In the major axis the panels are either 2 or 3 span continuous; in the minor axis the panels are single span due to the limit on panel width. Design for bending and one way shear was carried out as per the Canadian standard for Engineering Design in Wood, CSA O86:19 [2]. There is currently no guidance in the Canadian standard on how to calculate punching shear capacity of a CLT panel, however one source of relevant research on this topic can be found in the doctorial thesis by Mestek [3]. The research developed equations to calculate the rolling shear analysis which we then used in our design.

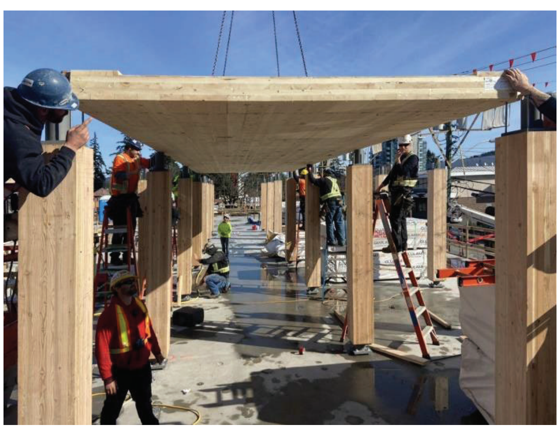

Figure 3: *Installation of CLT on Glulam Columns*

The balconies are also designed for 4.8kpa live load and these panels are exposed from the underside. They are designed for 2hr fire rating without encapsulation and thus are 7 ply panels, 245mm thick.

The penthouse structure consists of 3 residential units, each with custom-designed areas, open spans, and impressive vaulted ceilings. The grid spacing of the columns was increased at this level to achieve the architectural intent. It is constructed from glulam post and beam structure forming a gable roof, with CLT panels primarily spanning in one direction. A combination of custom steel hangers with bearing plates, and pre-engineered concealed connectors are used for the glulam beam connections.

Figure 4: *Penthouse Structural Framing*

2.2 COLUMN TO COLUMN CONNECTION

Due to the standardized panel sizes and grid layout, the glulam columns are generally evenly loaded across the floor plate. This regularity allows standardization of columns sizes and connections. The column sizes are uniform across each floorplate and decrease only 3 times moving up the building. Each of the 3 column sizes uses a variant of the standard column-to-column connection detail.

The column connection contains two elements, see Figure 3; the upper element is a horizontal steel plate screwed to the underside of the column welded to a vertical plate with a slotted hole. The lower element is a horizontal plate screwed to the top of the column welded to a HSS stub. The upper and lower elements are each shop-installed into the glulam columns. During install on-site the vertical steel plate is slotted into the HSS, and a single 19mm dia. pin passes through the HSS and plate to secure the connection. The vertical gravity load is transferred through bearing on the horizontal steel plates and the HSS. The connection is also designed for a tensile force to allow for robustness. This tensile force is resisted through screws in withdrawal installed at an angle into the end grain of the column. The load is then transferred

https://doi.org/10.52202/069179-0551

through the steel components of the connection, and the pin acts in shear and bearing to resist the load.

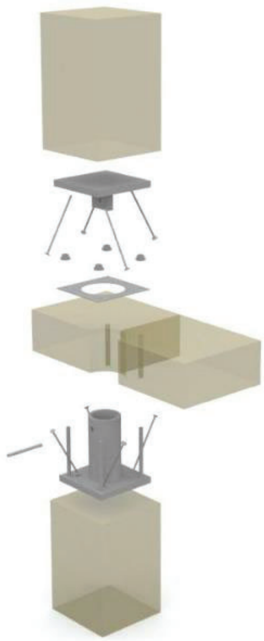

Figure 5: *Column connection detail components*

An important consideration in the detailing of a hybrid mass timber and steel structure is the way in which the different materials behave and playing to the strength of each. One of the benefits of this connection is that it avoids any perpendicular to grain loading of the mass timber elements, which can result in cumulative shrinkage and crushing in excess of what would be manageable from a serviceability limit state. While axial shortening will occur in the glulam columns, with the stress oriented parallel to grain the calculated cumulative deformations are not significant even for 12 storeys. To account for the potential axial shortening, multiple 2mm shim plates were introduced in the glulam column connection. The level of each column could be checked on site and shimmed as appropriate. The result of this approach was an essentially flat floor plate at each level. These shims also helped to reduce the impact of deflections in the concrete transfer slab at Level 2 as it was loaded during erection of the timber structure.

The CLT panels bear directly onto the plates on top of the glulam columns. Threaded anchors provide locating points for the panels during erection and provide temporary lateral support.

Due to the repetitive nature of the floor plate, and the simplicity of the point supported CLT slabs, this column-to-column detail is the primary connector throughout the entire project, with a very limited number of other detail types required. The details that were provided were shop installed where possible, to allow for quick and simple erection on site.

Figure 6: *Glulam column connection – mock up*

2.3 LATERAL FORCE RESISTING SYSTEM

Langford, BC has some of the highest seismic demands in Canada, therefore choosing an appropriate lateral system was one of the key considerations in the design process. Typically for structures of this height in this region we would see ductile concrete shear walls commonly used. These employ a design using a ductility force modification factor, $R_d = 3.5$, and an overstrength force modification factor $R_o = 1.6$. The concern with using this system in combination with mass timber was:

1) Concrete construction tolerances are much higher than with mass timber, which could lead to issues with the detailing of connections.
2) Speed of erection on site would be impacted by the time required to construct the concrete core in advance of the timber erection.
3) A single central core would not have been sufficient for the lateral demands, and additional cores or shear walls would be required.

As such, the design team sought out a lateral force resisting system with similar or improved ductility and overstrength, along with a material which allowed for tighter construction tolerances and improved erection times. The system chosen was a highly ductile eccentrically braced steel frame (EBF). This system has similar elastic stiffness when compared to concentrically braced frames, but does not rely on brace buckling to provide ductility. Rather, the stable inelastic response is closer to that of a ductile steel moment frame. The EBF ductility force modification factor, $R_d = 4.0$, and the overstrength force modification factor $R_o = 1.5$.

https://doi.org/10.52202/069179-0551

The building's rectangular floor plan necessitated a distribution of frames throughout the building, rather than a single central core, to avoid excessive torsional displacements. This distribution was easily achieved by positioning the frames within walls between units as well around the core.

The steel frames were partially pre-fabricated in multi-storey half-width segments. Each half of the frame was installed and then connected together with a link beam at each level, critical to the performance of the EBF. The goal of this approach to prefabrication was to decrease overall construction time by simplifying on-site assembly, and eliminating most of the site welding.

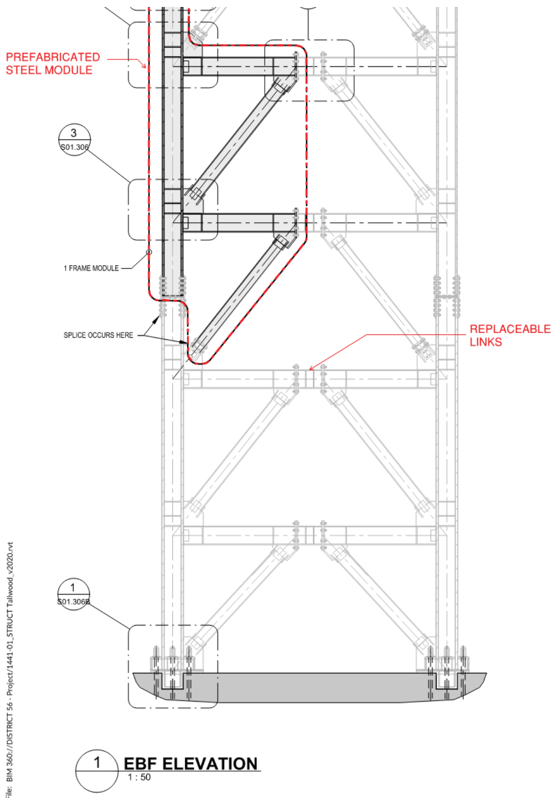

Figure 7: *Lateral force resisting system – EBF*

Capacity design principles in EBF design ensures that inelastic deformations are concentrated in yielding steel links that are capable of repeated cycles of large displacement demand, while remaining stable, and without significant strength degradation. The remaining structural elements are designed to remain elastic at load levels associated with all yielding links having reached their upper limit on capacity.

After a seismic event, yielded links could be replaced to repair the brace frames, increasing the building's resiliency. This repairability has an advantage over more common construction methods such as concrete cores, where the yielded core would likely have to be replaced in its entirety after a major seismic event.

Figure 8: *Installation of EBF modules on site*

Diaphragms consist of CLT panels connected with nailed plywood splines with steel plates and light gauge steel straps providing the chord and collector elements. All timber panels, joint connections, chords, and collectors are capacity protected to ensure they remain elastic. The roof diaphragm's increased complexity due to the geometry resulted in a lengthy design process with a number of bespoke details. A high level of coordination was required between with the CLT supplier and architect during design and construction.

Figure 9: *Roof diaphragm drag strap*

https://doi.org/10.52202/069179-0551

2.4 LATERAL ANALYSIS

Internal forces due to seismic loads were determined from an elastic response spectrum analysis. Three-dimensional dynamic analysis is a requirement of the 2018 BC Building Code due to the building height, seismic hazard, and mass irregularity present with the much heavier L2 level.

Diaphragm storey forces capture higher mode effects and are based on the greatest of: (1) code minimum equivalent static forces based on a percentage of the building's base shear, and (2) forces from the response spectrum analysis scaled up to exceed the probable capacity of the steel braced frames. At all levels, the scaled response spectrum value governs the storey force for this building, and results in an almost constant storey force, and a consistent diaphragm design, over the building height.

Serviceability limit states for drift and accelerations are in accordance with the requirements of the 2018 BC Building Code and 2015 Structural Commentaries [4]. Careful design consideration was given to wind induced accelerations due to the lightweight nature of the building structure relative to concrete buildings of a similar size. Canadian design standards do not provide specific damping values to be used for service level wind analysis in hybrid mass timber buildings, so a lower bound critical damping ratio of 0.01 was assumed, which is consistent with steel framed buildings. Along wind and across wind accelerations are highly influenced by the building's mass, stiffness, and fundamental frequencies in the two principle directions. Tallwood 1, designed for the governing seismic loads, did not require any additional damping or lateral stiffness to minimize response to wind loads.

2.5 FIRE PROTECTION

Changes in the 2020 National Building Code of Canada (NBCC) [5] allow for the construction of timber buildings up to 12 stories tall by introducing a new construction type called Encapsulated Mass Timber Construction (EMTC).

The NBCC is a model code, which is then adopted by the provinces. An amendment to the 2018 BCBC adopted the EMTC provisions in 2020. However, only a select number of local jurisdictions within the province adopted these code provisions. The city of Langford where Tallwood 1 is located was one of 13 local jurisdictions that did.

The EMTC code provisions include details on fire protection requirements during construction, fire rating, minimum member size, allowable occupancies, and square footage limitations. There are also limits on the amount of exposed (unencapsulated) mass timber permitted. Prior to this new construction type, limits on combustible construction of up to 6 stories were permitted, unless an application was made to the Authority Having Jurisdiction (AHJ) proposing an alternative solution demonstrating that the design complies with the objectives of the code, a potentially long and costly process.

Tallwood 1 was the first building in BC to be built to the newly adopted code and the first instance of an EMTC building.

Load bearing columns and floor assemblies are designed for a 2hr fire resistance rating achieved through a combination of encapsulation with 2 layers of 16mm type X gypsum board, which provides 1 hour of fire resistance, and allowance for 1 hour of charring. Char rates, effective cross-sectional properties, and residual member capacities are in accordance with the CSA-O86 standard.

Figure 10: *Mass timber encapsulation progress*

At the penthouse level a significant percentage of the roof structure is exposed so that the timber making up the vaulted ceilings can be visible and celebrated. The design team prepared and submitted an alternative solution application to the AHJ, and the AHJ agreed that a 1-hour fire resistance rating and a higher percentage of exposed wood were acceptable. These exposed elements were designed for 1 hour char as described above.

The steel braces are fire protected through a combination of encapsulation where braces are located within walls, and intumescent paint where braces are exposed across window locations.

2.6 ROBUSTNESS

Structural integrity is a requirement of CSA-O86, stating: *"the general arrangement of the structural system and the interconnection of its members shall provide positive resistance to widespread collapse of the system due to local failure"*

It is important to address integrity when designing high rise mass timber structures since traditional detailing practises do not necessarily provide inherently robust structures. Tallwood 1's integrity design is based on the Eurocode 1 2006 [6]. The tying method is used for the timber portion of the structure and key element design for the concrete transfer slab. The splines provide the horizontal ties for the CLT floor panels with additional ties provided by the steel diaphragm straps. The columns act as vertical ties with the column-to-column connections being designed for the required tensile force, as previously discussed.

3 CONSTRUCTION

3.1 SHOP DRAWING PROCESS

The shop drawing process for the mass timber elements of the structure was highly detailed and included modelling the slab service penetrations to minimize site drilling. This high level of coordination also allowed the design team to review and comment on all penetrations prior to fabrication.

The timber shop drawings were produced ahead of the steel shop drawings, each using a separate model, preventing proper integration between the timber and steel shop drawings and associated models. Having one complete structural model including both the timber and steel would have helped flag some, if not all, of the tolerance issues we encountered on site between the steel and mass timber components.

3.2 ON SITE ASSEMBLY

Construction of the mass timber structure began in March of 2021 and the structure was topped off in November 2021. The mass timber and structural steel generally came together quite smoothly, in part because steel tolerances tend to be similar to mass timber tolerances. The structure was framed in blocks of 2 to 3 stories, matching the EBF prefabricated module sections. Once the EBF sections were erected and connected together with the links, the mass timber portion of the structure was installed to the same level with no need for temporary lateral support. This sequencing simplified the construction and maximised speed on site, however, due to supply chain challenges related to the wall panels, speed of construction was not as fast as initially anticipated. A code requirement for EMTC states that a maximum of four stories can be constructed without encapsulation, and inability to ship and receive the planned prefabricated external wall panels in time impeded progress on site. Furthermore, there were tolerance issues between the mass timber and steel components, likely due to insufficiently coordinated fabrication models, which required remediation on site and lead to delays. Finally, a highly intricate penthouse structure had a lengthy build, but with stunning results.

Moisture management in timber structures is critical to avoid issues associated with degradation, rot, mould, and visual issues like staining. The bulk of the mass timber construction took place during the drier months, however the climate on Vancouver Island is renowned for its rain and therefore additional measures were required to mitigate the risks associated with moisture. Steps included:

1) Providing protective waterproof covering to glulam beams during construction
2) Sealing joints between the CLT panels once the splines were installed.
3) Removing pooling water from the CLT slabs during heavy rain events
4) Allowing panels that were subject to rain to fully dry before being covered to avoid issues with mould forming.
5) At level 9 the CLT panels had SIGA wetguard applied creating a rainproof layer. An exterior sill plate created a tub effect and 6 drains were installed to manage any surface water. It has become more common to see all CLT floor panels complete with a shop applied membrane. This functions to protect the CLT and provides a dry interior space below so that fit out can progress.
6) At level 12 there were extended delays while the final design of the penthouse was being completed. A torch on waterproof membrane was applied at this level to try mitigate water ingress. There were still localised issues at the column locations where there were openings in the CLT panels. At drag strap locations, where the strap and fasteners stood proud of the slab, repeated impact from walking and moving materials caused the membrane to wear down and allowed moisture to pass through.

3.3 LESSONS LEARNED

Overall, one of mass timber's primary advantages to the cost of a project can be speed of construction, so realizing that speed is critical on a tall wood building. Lessons learned were generally related to items that slowed down construction:

- Although the tolerances of a steel frame are far closer to mass timber than a concrete core, field welds were required at some locations due to misalignments. Introducing more tolerance in the steel to timber details can reduce the need for field welds, along with more stringent coordination between suppliers.
- The importance of simple, repetitive detailing was evident on this project. While the typical panel to column detail was efficient and highly repetitive, the diaphragm connectors were numerous and in the future could be simplified and streamlined to improve fabrication and site installation.
- Due to supply-chain challenges, the non-load bearing external walls were fabricated on-site instead of off-site as originally planned, which slowed down the pace of construction. Offsite prefabricated wall panels are much more ideally

https://doi.org/10.52202/069179-0551

suited to a largely prefabricated mass timber building like this one.

4 CONCLUSIONS

This paper has outlined the main structural design aspects of a completed 12-storey steel-timber hybrid tower. Lessons learned during the design and construction of the project are valuable in advancing more efficient, safe, and cost-effective tall timber buildings in Canada.

ACKNOWLEDGEMENT

The authors would like to acknowledge the consultants and mass timber supplier that worked on Tallwood 1: Design Build Services, Jack James Architect, and Structurlam Mass Timber Corporation.

REFERENCES

[1] British Columbia Building Code, Crown Publications, 2018.
[2] CSA O86:19 Engineering Design in Wood, CSA Group, 2019.
[3] Punktgestützte Flächentragwerke aus Brettsperrholz (BSP) – Schubbemessung unter Berücksichtigung von Schubverstärkungen; Mestek, 2011
[4] Structural Commentaries (User's Guide – NBCC 2015: Part 4 of Division B), Fourth Edition, National Research Council of Canada, 2017.
[5] NBCC 2020: Part 3 of Division B, First Print, National Research Council of Canada, 2020.
[6] BS EN 1991-1-7:2006: Eurocode 1 – Actions on structures, Part 1-7 General actions – Accidental actions

https://doi.org/10.52202/069179-0551