RF 24-40

DESIGN OF A 2-STOREY INSTITUTIONAL SPACE FEATURING MASS TIMBER IN VICTORIA, BC

(Treeple Architects, n.d.)

Prepared for:

Christian Slotboom, EIT, MASc, Fast+Epp, Industry Project Sponsor
Ryan Taylor, PEng, MIStructE, Fast+Epp, Industry Project Sponsor
Industry Projects Committee, BCIT Civil Engineering Department
Qi Zhang, PEng, PE, PhD, Faculty Advisor, BCIT Civil Engineering Department
Jacquie Russell, Communication Instructor, BCIT Communications Department

Prepared by:

Alex Thormeyer

Submitted on:

April 12, 2024

Civil Engineering Department
British Columbia Institute of Technology
Burnaby, BC

DISCLAIMER

The work represented in this client report is the result of a student project at the British Columbia Institute of Technology. Any analysis or solution presented in this report must be reviewed by a professional engineer before implementation. While the student's performance in the completion of this report may have been reviewed by a faculty advisor, such review and any advice obtained therefrom does not constitute professional certification of the work. This report is made available without any representation as to its use in any particular situation and on the strict understanding that each reader accepts full liability for the application of its contents.

In addition,

- All design aspects of this project, including calculations, are based on the National Building Code of Canada 2020 (NBCC 2020), and Canadian Standards Association Engineering Design in Wood (CSA-o86-19)
- All initial drawings were provided by my sponsor, Christian Slotboom, EIT, MASc, Industry Sponsor from Fast+Epp

ACKNOWLEDGEMENTS

I would like to thank the following people who assisted me with this project:

- Jacquie Russell, who provided me with guidance and feedback in producing this report
- Christian Slotboom, who provided this excellent project and met with me regularly to give guidance regarding structural engineering processes, analysis, and design
- Ryan Taylor, who approved this project as the signing PEng
- Qi Zhang, who met with me upon request to answer questions and provided me with great resources with regards to wood design

Alexander Thormeyer
______ • ________ • ____________

April 12, 2024

Christian Slotboom, EIT, MASc
Engineer, Fast+Epp
Industry Project Sponsor
300 – 397 West 7th Ave
Vancouver, BC V5Y 1M2

Dear Christian

Submission of Final Report on Design of a 2-Story Institutional Space Featuring Mass Timber in Victoria, BC

Attached is my final report for the 2-story institutional space project which featured mass timber and is in Victoria, BC. This project consisted of a gravity design for glulam structural members, a 120-minute fire-resistance rating, the design and producing of detailed drawings for structural connections, and a markup of the initial plans including beam, column, and panel schedules.

The results of this project were beam, column, and panel schedules provided on marked up drawings of the initial plans provided, connection details, and a structural system that can withstand the anticipated gravity loads while also meeting a 120-minute fire-resistance rating. Overall, the entirety of the project took approximately 195 hours to complete, with the bulk of the work being invested into learning how connections function and creating detailed drawings of them in Bluebeam.

I would like to extend my gratitude and thank you for providing me with this project. The completion of this project could not have taken place without your generous dedication to giving me time and guidance on typical structural engineering practices. Working on this project provided me with a much better understanding of how connections work within timber structures, how exposed timber members are affected by fire, and how to produce detailed connection drawings in Bluebeam. In addition, I would like to thank Qi Zhang for providing me with additional guidance, references, and feedback throughout the course of the project.

If you have any questions, please contact me at ___________ or by email at ___________________.

Sincerely,

Alex Thormeyer

CC: Qi Zhang, PEng, PE, PhD, Faculty Advisor
Jacquie Russell, Communication Instructor

Attachment: Final Report

SUMMARY

The purpose of this project was to design a two-storey glulam structure that can handle the gravity loads imposed on it per the NBCC 2020 (National Building Code of Canada) as well as meet a specified 120-minute fire-resistance rating as defined in CSA-o86-19 (Canadian Standards Association – Engineering Design in Wood). A primary design feature of this project is the second level and roof level long-span mass timber panels. The project acted as an academic exercise and an introduction to the processes that practicing structural engineers implement regularly. I was provided with all necessary drawings and details by my sponsor, Christian Slotboom, EIT, MASc, engineer at Fast+Epp.

The initial stage of the design considered the project location and anticipated use to determine gravity loads the structural system will need to support. Following that, similar types of structural components were grouped to expedite the structural analyses and design aspects. Markups of the initial plans were produced and analysis of each type of structural component took place in accordance with NBCC's ultimate and serviceability limit states. Member sizes were obtained through calculations following CSA-o86, and then verified through hand calculations.

A fire-design was conducted to ensure that the system would not collapse if members were exposed to a 120-minute-long fire. Per CSA, member size reductions were determined and applied to existing members to check against anticipated loading under a fire event. Beam sizes were determined to be adequate without any adjustments needed. In contrast, all columns needed adjustment to meet the specified 120-minute fire-resistance rating. Initial calculations provided beams sizes that had larger cross sections than most columns. It was noted that as member sizes were smaller to start, the greater the decrease in capacity when exposed to fire.

Connections were designed using embedded steel plates, tight fit pins, and dowels. Due to steel being the choice of material, it was also critical to ensure that it would be protected from a fire event. Using the minimum distance calculated from the fire design, placement of steel plates was such that fire would not affect the structural integrity should a 2-hour fire event occur. Two separate styles of checks were required as the loading was perpendicular to the wood grain in beam scenarios but parallel to the wood grain in column scenarios. Within these two modes, connections at the beam ends were checked for both yielding and splitting failure modes. Connections at the column end only required a yielding check. Specialty connection plates were designed to satisfy junctions where multiple beams oriented at 90° to each other connected to the same column. Pre-engineered beam hangers were selected based on load demands and utilized at any beam-to-beam junction. In accordance with the requirements to utilize these pre-engineered products, certain roof level members required increased sizes.

With the project time totaling at 195 hours, all objectives were completed. Beams and columns were designed to ultimate and serviceability limit states as outlined in the NBCC. Fire design governed column member sizes and provided insulation distances for steel plates and pins utilized in the connection design. The deliverables produced were detailed drawings of the connections at each junction, a calculations package with supplementary hand calculations, and a full markup of the building schematic including structural component schedules.

Table of Contents

List of Illustrations

TABLES

FIGURES

1.0 INTRODUCTION

The purpose of this project was to design all structural members for the gravity system of a 2-storey institutional building located in the Mt Gelmer region of Victoria, BC. This project was given to me by my sponsor Christian Slotboom, EIT, MASc from the engineering firm Fast+Epp based in Vancouver, BC. The main reason for my sponsor providing this project was due to my heavy interest in mass timber. A key project feature is the use of mass timber, which will be used as the structural element supporting the second floor and roof levels. This project was chosen by my sponsor to act as an introduction into the type of work that structural engineers face daily.

This two-level structure consists of a lecture hall and atrium on the first floor, and a lab space on the second floor. The entirety of the structural system is composed of glulam columns, glulam beams, and CLT (cross-laminated timber) panels. The CLT panels, also known as mass timber, is supporting a layer of concrete directly under the second-floor lab space.

Figure 1 below displays what a 2-storey mass timber structure could look like.

Figure 1: Representation of a 2-storey mass timber building (The Riba Journal, n.d.)

This project utilized mass timber as a key structural component, while also using timber columns to create an aesthetically pleasing space for its occupants. There were three main objectives regarding this project:

- Develop structural engineering drawings using Bluebeam and CAD
- Design the structure in accordance with NBCC 2020 and CSA-o86-19
- Select appropriate CLT members through structural analysis

Aspects of this project that were not considered are stair design, wind load analysis, seismic loading, and foundation analysis. The scope of the project remained unchanged and the gravity design, fire design, and structural connections design, were all completed.

The following sections of this report will provide further details regarding key project components including processes and tools used in conjunction throughout the project, the outputs produced, the results of the structural analysis and design, and final recommendations.

2.0 STRUCTURAL ANALYSIS & DESIGN

This structure was analyzed and designed using engineering principles to identify critical design loads, and then selecting appropriate member sizes with sufficient capacity to safely transfer the load through the structure to the foundation. As this project excludes wind load analysis, seismic analysis, and foundation analysis, the location was only relevant in determining the appropriate snow load as provided by the NBCC. CSA-o86 was followed to ensure members met the appropriate strength, serviceability, and fire resistance requirements.

2.1 Design Loads

The first stage in the design process was to establish the design loading that the structure will need to support. This structure was only analyzed with respect to dead loads, live loads, and snow loads. A dead load is a permanent load due to the weight of the building components. Live loads are any variable load due to intended use and occupancy, and a snow load is a variable load due to snow based on the location of the structure. All definitions are provided in NBCC Section 4.1.2.1 and as required in the NBCC, the overall design will be governed by two limit states, ultimate and serviceability.

Ultimate limit states (ULS) deal with the strength capacity of the structure, whereas serviceability limit states (SLS) are more concerned with the maintenance of the structure and what feels safe deemed by public opinion. The key difference between the two is the differing factors that get applied to the loads and where they are used in the design calculations.

Values for dead loads were provided by my industry sponsor with a total of 2.60kPa for the roof portion and 3.50kPa for the second level. Live loads were gathered from NBCC Table 4.1.5.3, providing values of 1.00kPa for the roof, and 4.80kPa based on assembly use for the second level.

Determining the total snow load depends on various factors such as the importance class of the building, slope of the roof, exposure of the building, loads due to rain and snow, and an accumulation factor. The importance class of this building was determined to be normal, giving it an importance factor of 1.0. The snow and rain load values were obtained from NBCC Table 4.1.5.2.-A. The appropriate factors were determined and applied, and two snow load values were obtained at 1.98kPa and 1.78kPa for ultimate and serviceability limit states respectively.

As stated in NBCC Section 4.1.1.3 and using Table 4.1.3.2.-A and Table 4.1.3.4, governing load combinations and factors were applied and were verified through hand calculations as shown in Appendix A1. The governing design load for the roof level ULS will be 6.22kPa and 4.38kPa for SLS.

Table 1 below shows a summary of the various load cases, with an exclusion of the fifth case which deals with seismic loading, and the governing one highlighted in blue.

Table 1: Ultimate Limit State design load calculations for roof level

Case	Principal (kPa)	Companion (kPa)	Total (kPa)
1	3.64	0.00	3.64
2	4.75	0.00	4.75
3	6.22	0.00	6.22
4a	3.25	0.50	3.75
4b	3.25	0.99	4.24

Governing load values for the second level were determined with the same process and were calculated to be 11.58kPa for ULS and 8.30kPa for SLS.

2.2 Structural Member Grouping

To be efficient in the design timewise while also ensuring that structural members are not overdesigned, the second stage of this project entailed determining which beams and columns share similar load types and supported similar size areas. Ultimately it was determined that there were seven different beam types, four different column types, and two different CLT panel types, as shown by the schedule tables shown in Appendix B1. Locations of each component can be found in Appendices B2, B3, and B4.

2.2.1 Beams

A top-down approach was used to establish how to group different members. Starting at the roof level shown in Appendix B4 and combining this with the elevation view (Appendix B5), it is observed that certain beams will be angled and others flat. The flat beams all had a relatively similar length with only a 1530 mm variation, which is small in the grand scheme of the structure. Since they all support the same loading, are spaced equally apart, and have a similar length, they were all marked as beam B1.

Separating the remainder of the beams on the roof plan by length and loading style led to beams B2, B3, and B4. Beam B4 was deemed a special case because it acts as a girder that holds up the ends of beams B1 and B3 giving it a unique loading compared to other beams. The same process was used to establish beam types on the second level shown in Appendix B3. The results were three different beam types, B5-B7, with B7 matching the same loading style as B4 but only taking the load from one beam rather than two.

2.2.2 Columns

The column designation process was very similar with one architectural consideration provided by my industry sponsor which was that the lower-level column sizing will be utilized at the upper level as well. This consideration served two purposes, with the first being that it made the design process more efficient by reducing the number of checks and calculations that need to be completed. Lower-level columns always take the load from the above structure, and in this case the upper-level columns are placed directly in line with the lower-level ones. The second purpose is purely due to aesthetic considerations.

A key factor in grouping columns is the idea of tributary area, which is essentially the area above the column that will end up transferring load to that specific column. As loads will always want to transfer through the most direct path, tributary area can be determined by calculating the midpoint between the column being analyzed and the adjacent columns surrounding, then connecting these midpoints as shown in the red area in Figure 2 below.

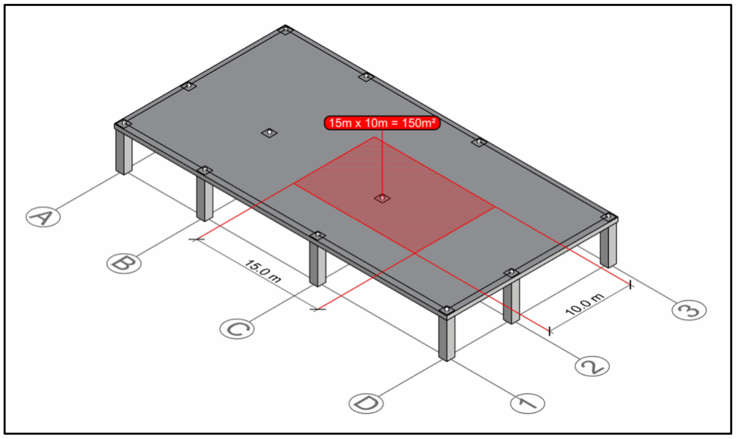


Figure 2: Example of tributary area for columns (Tribby3d, Feb 2021)

Judging from the elevation view shown in Appendix B5, there are two distinct column heights going from the ground level up, 9000mm and 6000mm. The 9000mm columns are all located at the exterior of the structure to the right of gridline C, are all supporting one end of beam B1, and therefore have been given the designation C1. Using the concept of tributary areas, the remaining three columns were marked on the three remaining plan views.

2.2.3 CLT Panels

From the design loads it is known that there will be different panel strength and serviceability requirements for the roof level and second level. An additional consideration for these panels is that they all must be the same thickness as they act together as a floor or roof. Therefore, there can only be two separate panel types to ensure uniformity across each level. Roof level panels were designated P1 and second level floor panels as P2.

2.3 Structural Member Analysis

As the structural members have been categorized into different types, and the design loads have been calculated, the next step was to determine how these loads will affect the individual members. Depending on the type of structural member, various critical values must be calculated and checked against to ensure that it meets both ULS and SLS requirements.

2.3.1 Beam Analysis

In this project, all the beams will be supported at either end, also known as simply supported, and will terminate at the first column it encounters. Per CSA, the critical values that must be checked when it comes to beams are maximum bending moment, maximum shear, and the maximum allowable deflection due to loading.

The bending moment indicates how much a beam will bend under an external load, if the beam bends too much, it will fail and break. Similarly, a shear value indicates how much force a member may experience on a given plane. Both values need to be accounted for to ensure stability and prevention of structural failure. Using beam B6 as an example, the maximum bending moment was calculated to be 678.22kNm and a corresponding maximum shear of 217.03kN. Deflection checks ensure that occupants within the structure feel safe. Per CSA 5.4.3 and again using beam B6 as an example, the maximum allowable deflection governed by long-term loading is calculated to be 34.72mm.

Beams B4 and B7 have different loading conditions and were therefore evaluated with different equations. All necessary values to check against were calculated and can be seen in Appendix C3. In addition, these values were confirmed through hand calculations as can be seen in Appendix A2.

2.3.2 Column Analysis

The analyses of columns are different from beams in that deflection is not checked, and the only critical value to obtain is the combined compressive load from everything above it. At the member selection stage, possible bending within the column due to the beams pulling down on the connection interface of the column will be accounted for through an equation provided by the Wood Design Manual 2020 (WDM).

Compressive loads were determined through multiplying the factored design load against the calculated tributary area. In the case of this structure, the highest compressive load was 592.53kN and occurs at the lower-level C4 column. Observing that the distance between C4 columns on Appendix B2 is 6000mm and not the typical 3000mm elsewhere, it is logical that the highest load would occur where the column supports the largest area.

2.3.3 CLT Analysis

CLT panels are an engineered wood product consisting of layers of wood oriented at right angles to each other and then glued together creating a structural panel as defined by naturally:wood and shown in Figure 3 below.

Figure 3: Visual of CLT panel layering (naturally:wood, n.d.)

They are typically manufactured at three, five, or seven layers thick. Due to this layering being an odd number, it produces a strong and weak direction within the panel. This structure will utilize the strong direction of the panel perpendicular to the directions of the beams, allowing for structural stability in both directions.

In terms of analysis, this meant that only the strong direction of the panel would need to be checked as the weak direction will be handled by the beams. Due to panels potentially having varying widths, a unit width of 1m was used in calculations for analysis and strength. Information from my industry sponsor indicated that it is good practice to have the panels bridge multiple spans to assist with deflection, as this is typically the governing factor in CLT designs.

This meant that with regards to load analysis, a fixed-pinned boundary condition was established and beyond my solving capabilities at this time. An online beam solver by SkyCiv was used to obtain maximum bending and shear values for these panels. CLT panel P2 had the highest bending and shear values at 13.02kNm per 1m panel width, and a corresponding shear value of 21.70kN. Graphs displaying how the bending moment and shear forces travel through these panels are available in Appendix A2.

2.4 Member Selection

The critical values obtained from the previous stage will now govern the member selection process. Members that are larger will typically provide higher resistances against these critical values. To be the most efficient and economical, the smallest member sizes that can satisfy all limit states have been chosen.

The choice of engineered wood for this project is glulam, short for glue laminated. Defined again by naturally:wood, this is a product that consists of several wood laminations glued together so that the wood grain is parallel among all pieces. Figure 4 below depicts what typical glulam products look like.

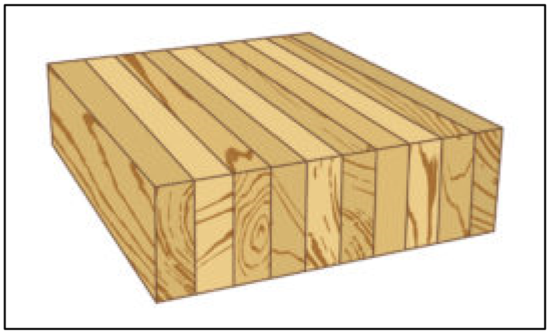


Figure 4: Typical glulam member (naturally:wood, n.d.)

Within the glulam product, there are various species of wood that can be chosen. As SPF is the most used wood species in British Columbia, it has been utilized in this design. CSA Section 7 provides more insight into how resistances can be calculated for glulam products against bending moments and shear.

2.4.1 Beam Selection

In general, the member selection process is an iterative process, a member must first be chosen, and its resistances calculated to see whether it is over or under designed. As mentioned previously, beam checks that need to be completed are bending, shear, and deflection. SPF comes in various grades each with their specific benefits. As all beams in this design are simply supported, the most economical and efficient grade to use is the 20f-E grade and has been chosen for all beam members.

Starting with bending, per CSA 7.5.6.5 there are various factors that affect the resistance of a member. With respect to this design, the external load duration, the member dimensions, the lateral support provided to the member, the grade of wood, and the species of wood, are what need to be considered.

The load duration factor is governed by CSA 5.3.2 and is a factor that will reduce calculated member capacities in anticipation that the member will need to be stronger to support any load for a much longer period. Beams for the roof end up with a calculated factor of 0.94 and second level beams default to the typical 0.65 for long-term loading.

Both the roof and second level beams are laterally supported along their top edge by the CLT panels, which indicate sufficient lateral stability and therefore will not reduce the member capacity. All other factors are calculations based on the member dimensions. The factors pertaining to the shear resistance are slightly different with some overlapping similarities. Differing factors are the uniformity of the external loading, and that the cross-sectional area of the member must be considered.

Once all governing factors have been calculated and tabulated, calculations for the initial beam estimate can be completed. A beam size of 365mm by 950mm is determined to be suitable for B6 against ULS conditions. Per SLS, expected deflection must also be calculated and be less than the previously mentioned 34.72mm. The calculated deflection of B6 under serviceability loads is expected to be 29.47mm, meaning that this member size is suitable for use.

The remaining beams went through the same design process and all results can be seen in Appendix C4, with supplementary hand calculations to verify shown in Appendix A3. For aesthetic reasons, special considerations were given to beams B4 and B7 which state that they must be of equivalent sizes or greater than B3 and B6 respectively.

2.4.2 Column Selection

The column selection process utilizes many of the same factors as mentioned in the bending section. Shear and deflection are not required to be checked, however column buckling becomes the primary concern. Loading that is not directly applied through the center of the column parallel with the member will greatly amplify this buckling concern, this is known as eccentric loading and has been considered in this design.

Both the CSA and WDM provide unique equations in accounting for the eccentric loading combined with the compression forces at the same time. While the CSA equation is more conservative, the WDM equation is more specific to the conditions of this structure and has been used to govern the column design. Eccentric loading present within columns can also be viewed as bending moments within beams, as such, these columns can now be treated as beam-columns and make use of the typical bending resistance equations when considering bending, as well as the standard compressive equations when evaluating axial compression.

Factors that affect compressive resistance are somewhat like that of shear in beams, with the major difference being the slenderness factor of the column. Tall and slender columns will be more prone to buckling and therefore cannot support larger external loads. Grade 12c-E, a grade exclusively used for compressive design, has been utilized for C1, while grades 20f-EX are utilized for C2-C4 due to the high bending forces generated from the connecting beams.

A critical section of this structure is the bi-axial bending most prominently occurring in the lower-level C4 column. Bi-axial bending is when the eccentric loading occurs on perpendicular faces of the column, causing bending in two directions. This is a particular concern because 20f-EX grades only provide specific bending resistances on the narrow face of the member. Per CSA 7.5.3, this column has been treated as a built-up sawn lumber column and has referenced CSA Section 6. This section was followed in the calculations for bending resistance in this weaker direction.

Calculations for columns that satisfy the ULS can be seen in Appendix A4. Using the interaction equation provided from the WDM, a size of 400mm by 456mm is suitable to handle the worst-case combined compression and bi-axial bending.

2.4.3 CLT Panel Selection

Per sponsor recommendation, only strength checks were conducted on the CLT panels using values from the Standard for Performance-Rated Cross-Laminated Timber by the APA. This document provides typical unfactored strength values for all grades of CLT. Maintaining the use of SPF, grades E1 and V2 were deemed viable options, with varying thicknesses ranging from 105mm to 245mm.

As the load conditions and factors have already been established for the roof and second level, straightforward strength calculations were conducted to determine the factored resistance of each panel. Ultimately it was determined that for panel P1, a V2 grade at 105mm is sufficient. The second level panel P2 required the stronger grade E1, and an even thicker panel at 175mm thick. Full results and hand calculations are available in Appendices C6 and A5.

2.5 Fire Design

A critical component of the structural design was to ensure that the structural system was able to meet a fire-resistance rating of 120-minutes. Per CSA Annex B, the fire design is achieved by ensuring that structural members can withstand expected loads at a reduced member size. This reduced size was calculated by using values correlated with the type of wood product per CSA and burn duration of 120-minutes. Deflection considerations are not required within a fire design. The specific design of each glulam member layer in accordance with this rating was beyond the scope of this project.

Figure 5 below illustrates the reduction of member sizing due to fire exposure.

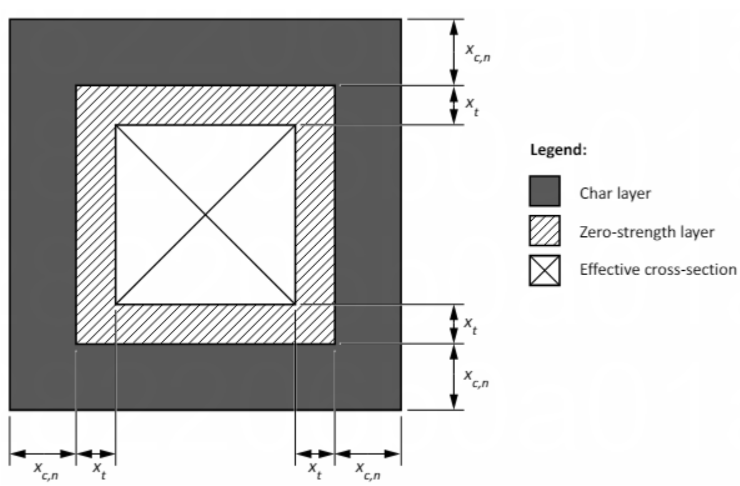


Figure 5: Cross-sectional reduction of member exposed to fire (CSA, 2019)

As shown in Figure 5 above, an additional penalty per CSA B.5 of 7mm is also applied, which assumes that this much of the member can no longer provide strength as it is too close to the surface that is being burned, deeming it a zero-strength layer. Per CSA B.4, the side of a member that is exposed to a 120-minute fire would be reduced by a calculated 84mm. Totalling at 91mm per side exposed to fire, it can already be seen that the initial C1 size of 215mm by 266mm would be reduced to 33mm by 84mm which is likely far too small to support the anticipated roof loading.

Specified in CSA B.1.3, specified or unfactored loads are used in the calculations for a fire design as this is strictly an emergency type scenario and the event is not expected to last a significant portion of time. CSA B.3 indicates what resistance factors are to be used, of which most notably is that the strength is increased by a factor of 1.35, and the load-duration is shifted to short-term giving it another increased factor of 1.15. The general member resistance reduction factor of 0.9 is also removed.

Further calculations are carried out using existing member sizes to determine which structural components need to be increased. Results indicated that the smaller the member was to begin with, the more impactful the size reduction was in terms of resistance capacity. As most beams were governed by the deflection portion, there was sufficient strength in all beams to pass the fire design and no beams required adjustment.

In contrast, all columns required changes and can be seen in Table 2 below.

Table 2: Comparison of column sizes before and after fire exposure

		C1	C2	C3	C4
Old sizes	Base (mm)	215	265	265	400
	Depth (mm)	266	304	304	456
New sizes	Base (mm)	365	400	400	490
	Depth (mm)	418	418	418	532

It is observed that C1 required the largest size increase at approximately 150mm in both directions. Moving to C4, only an increase of around 90mm was required in each direction. The results of the fire design can be viewed in Appendix C4 for beams and Appendix C5 for columns. Appendix A6 contains hand calculations that verified the values displayed in the calculations package.

2.6 Connection Design

The final stage of the design process considers the selected structural components and determines what is required to sufficiently connect them. At the instruction of my industry sponsor, steel knife plates in conjunction with steel pins and wood dowels were used to connect all beams to their adjacent columns. Where a beam needed to connect to another beam, a pre-engineered industry product made by MTC Solutions known as a MEGANT beam hanger was used.

The performance of steel as a building material significantly decreases when exposed to fire, as such it was critical that all steel components were insulated by the calculated 84mm distance from the fire design. Additional considerations were to maintain a single row of pins within the beam side of the connection to reduce the eccentricity loading on the columns as much as possible. Columns were required to have two rows of pins to provide better overall load transfer. Other sponsor provided information was that the pins had a yielding strength of 410MPa and a diameter of 19mm, that the steel plate would be between 10-20mm thick and conform to G40.21-350W specifications.

The connection design was carried out adhering to CSA Section 12.4. As wood is not a uniform material and is stronger in the direction of the grain, two separate modes of checks were conducted. The beam ends required checks where the load is considered perpendicular to the wood grain, and the column ends needed to be checked for loading parallel to the wood grain. Within that, beam ends were checked against two types of failure modes, yielding, and splitting. Column ends were only susceptible to yielding and thus only had one failure mode to evaluate.

The yielding failure mode considers all possible scenarios where a component within the connection has yielded. As instructed in CSA 12.4.4.3.2, items A, C, D, and G were checked which evaluated yielding by the wood member, plate, and pin. The splitting failure mode check ensured that the wooden glulam beams do not encounter a splitting of the wood material anywhere internally due to the loading that the pins are transferring.

A 15mm thick plate was used in the design of all connections, with the highest number of bolts at the beam end being 11 for B6. Columns C4 had the highest number of bolts at eight being divided into two rows of four when connecting to B6, and an additional two rows of two bolts in the perpendicular direction to connect to B7. The designs for connections involving columns C4 proved most challenging as the beams connecting were at 90° angles to each other. A plus style connection was utilized with bolts spacing being kept in mind so there were no interferences.

MEGANT beam hangers were selected purely based on the load demand requirements. To insulate these connections from fire, beams B1 and B3 required a further increase in size to ensure that they would not be compromised by a 120-minute fire exposure. Detailed drawings of all unique connections can be seen in Appendix B6. Full results are displayed in Appendices C7 and C8, with supporting hand calculations shown in Appendix A7.

3.0 CONCLUSION

This project was designed to meet the gravity load design and 120-minute fire resistance rating. Gravity loads were determined in accordance with NBCC and governed the initial design of the beams, columns, and CLT panels. Efficient grouping of similar structural components allowed for a swifter analysis on the structure. Appropriate member sizes were then selected to ensure that the load would be safely transferred to the foundation of the structure following ULS and SLS requirements.

A fire design was conducted to ensure that the structure would not collapse even if exposed to a 2-hour fire. In this stage, a member size reduction was established and applied to existing members to confirm their sufficiency. It was determined that beams were acceptable, but columns required an increase to be able to sustain the necessary loads to meet a 2-hour fire-resistance rating.

The final design portion of the project was to perform a connection design on the structural system. Connections were designed with steel plates and pins, while maintaining an insulation distance appropriate in accordance with the fire design. Connection details were produced in AutoCAD and marked up using Bluebeam. The initial building plan provided by my sponsor was marked up to show schedules summarizing the beams, columns, and CLT panels.

4.0 RECOMMENDATIONS

Recommendations for this project should it be evaluated further are as follows:

1. Further check into the C4 connection at the column end by ensuring that the wood member can handle being cut into quarters to support the existing connection
2. Conduct serviceability checks for the CLT panels
3. Determine CLT panel member widths and lengths such that a more detailed schedule could be produced

REFERENCES

APA (2020) *Standard for Performance-Rated Cross-Laminated Timber*
https://www.apawood.org/publication-search?q=PRG+320&tid=1

CSA-o86-19 (2019) *Canadian Standards Association: Engineering Design in Wood*
https://www-csagroup-org.eu1.proxy.openathens.net/store/product/CSA%20O86%3A19/

Treeple Architects (n.d.) *University of Victoria National Centre for Indigenous Laws*
http://www.teeplearch.com/portfolio/university-of-victoria-national-centre-for-indigenous-laws/

MTC Solutions (n.d.) *MEGANT beam Hanger catalogue*
https://mtcsolutions.com/products/megant/

naturally:wood (n.d.) *Cross-laminated timber (CLT)*
https://www.naturallywood.com/products/cross-laminated-timber/

naturally:wood (n.d.) *Glue-laminated timber (glulam)*
https://www.naturallywood.com/products/glulam/

NBCC (2020) *National Building Code of Canada*
https://nrc.canada.ca/en/certifications-evaluations-standards/codes-canada/codes-canada-publications/national-building-code-canada-2020

Riba Journal, The (n.d.) *Exposed timber roof sets theatrical scene*
https://www.ribaj.com/products/peter-hall-performing-arts-centre-cambridge-interiors-haworth-tompkins-stephen-cousins

Tribby3d (Feb 2021) *Tributary Areas of Columns and how to Best Calculate them*
https://tribby3d.com/blog/tributary-areas-of-columns/

WDM (2020) *Wood Design Manual*
https://webstore.cwc.ca/product/wood-design-manual-2020/

APPENDIX A: HAND CALCULATIONS

Table of Contents:

Appendix A1: Design Loads

This appendix displays sample calculations for design loads due to gravity loading on the roof and second levels.
Loads due to wind and seismic are excluded from this project.

Roof level:

Dead Load:

$DL_{1\_roof} := 1.00\ kPa$ Assumed dead load from self-member weight.

$DL_{2\_roof} := 1.60\ kPa$ Superimposed roof load, provided by sponsor.

$DL_{roof} := DL_{1\_roof} + DL_{2\_roof} = 2.6\ kPa$

Live Load:

$LL_{roof} := 1.0\ kPa$ Per **NBCC Table 4.1.5.3**; (Roof)

Snow Load:

Importance Factors:

$I_{s\_u} := 1.0$ Per **NBCC Table 4.1.5.2.-A**

$I_{s\_s} := 0.9$ Per **NBCC Table 4.1.5.2.-A**

$S_s := 2.10\ kPa$ Per **NBCC Table C-2:** 1-in-50 snow load for Victoria, BC (Mt Tolmie)

$S_r := 0.30\ kPa$ Per **NBCC Table C-2:** 1-in-50 rain load for Victoria, BC (Mt Tolmie)

$C_w := 1.0$ Per **NBCC 4.1.6.2.3**

Check Cb factor:

$w := 11\ (3000\ mm) = 33\ m$

$l := 250\ mm + 7000\ mm + 5000\ mm + 6560\ mm + 500\ mm = 19.31\ m$

$l_c := 2 \cdot w - \frac{w^2}{l} = 9.6\ m$ $\frac{70}{C_w^{\ 2}} = 70$ $l_c < 70$

$C_b := 0.8$ Per **NBCC 4.1.6.2.2**

Check Cs factor:

$Roof_{height} := 1500\ mm$

$Roof_{length} := 7500\ mm$

$\alpha := \tan\left(\frac{Roof_{height}}{Roof_{length}}\right) = 11.61\ °$ At steepest portion, angle is still below threshold.

$C_s := 1.0$ Per **NBCC 4.1.6.2.5**

$C_a := 1.0$ Per **NBCC 4.1.6.2.8**

$SL_{roof\_u} := I_{s\_u} \cdot \left(\left(S_s \cdot C_b \cdot C_w \cdot C_s \cdot C_a\right) + S_r\right) = 1.98\ kPa$

$SL_{roof\_s} := \left(I_{s\_s} \cdot \left(\left(S_s \cdot C_b \cdot C_w \cdot C_s \cdot C_a\right) + S_r\right)\right) = 1.78\ kPa$

Limit States:

Per **NBCC Table 4.1.3.2.-A:**

Governing load combinations:

$w_{f\_roof} := 1.25 \cdot DL_{roof} + 1.5 \cdot SL_{roof\_u} = 6.22\ kPa$ Ultimate

$w_{s\_roof} := DL_{roof} + SL_{roof\_s} = 4.38\ kPa$ Serviceability

Additional note:
Per **NBCC 4.1.5.5.2**, live loads and snow loads will not be used together when calculating load combinations for roofs, therefore LL was excluded.

Second level:

Dead Load:

$DL_{1\_L2} := 1.00\ kPa$	Assumed dead load from self-member weight.
$DL_{2\_L2} := 1.00\ kPa$	Weight from partitions, value provided by industry sponsor.
$DL_{3\_L2} := 0.30\ kPa$	Mechanical weight, value provided by industry sponsor.
$DL_{4\_L2} := 1.20\ kPa$	50mm THK concrete topping, value provided by industry sponsor.

$DL_{L2} := DL_{1\_L2} + DL_{2\_L2} + DL_{3\_L2} + DL_{4\_L2} = 3.5\ kPa$

Live Load:

$LL_{L2} := 4.80\ kPa$ Per **NBCC Table 4.1.5.3**; (Assembly area)

Limit States:

Per **NBCC Table 4.1.3.2.-A:**

Governing load combinations:

$w_{f\_L2} := 1.25 \cdot DL_{L2} + 1.5 \cdot LL_{L2} = 11.58\ kPa$ Ultimate

$w_{s\_L2} := DL_{L2} + LL_{L2} = 8.30\ kPa$ Serviceability

Appendix A2: Structural Analysis

This appendix displays sample calculations for the structural analysis on the columns, beams, and CLT panels.
All beams are simply supported.

Beam B1:

$Trib.W_{B1} := 3000\ mm$

$L_{B1} := 6530\ mm$

$w_{f\_B1} := Trib.W_{B1} \cdot w_{f\_roof} = 18.66\ \frac{kN}{m}$ Factored design load

$w_{s\_B1} := Trib.W_{B1} \cdot w_{s\_roof} = 13.15\ \frac{kN}{m}$ Specified design load

$$M_{f\_B1} := \frac{w_{f\_B1} \cdot L_{B1}^{2}}{8} = 99.46\ kN \cdot m$$

$$V_{f\_B1} := \frac{w_{f\_B1} \cdot L_{B1}}{2} = 60.92\ kN$$

$$V_{s\_B1} := \frac{w_{s\_B1} \cdot L_{B1}}{2} = 42.92\ kN$$

Beam B3:

Due to B3 being angled, the horizontal projection method was used to recalculate the appropriate factored dead load.

$Trib.W_{B3} := 3000\ mm$

$L_{B3\_H} := 12500\ mm$

$H_{B3} := 1500\ mm$

$L_{B3} := \sqrt{L_{B3\_H}^{2} + H_{B3}^{2}} = 12589.68\ mm$

$SL_{roof\_u} = 1.98\ kPa$

$$DL_{B3} := \frac{1.25 \cdot DL_{roof} \cdot Trib.W_{B3} \cdot L_{B3}}{L_{B3\_H}} = 9.82\ \frac{kN}{m}$$

$w_{f\_B3} := DL_{B3} + 1.5 \cdot SL_{roof\_u} \cdot Trib.W_{B3} = 18.73\ \frac{kN}{m}$ Factored design load

$w_{s\_B3} := \left(\frac{DL_{roof} \cdot L_{B3}}{L_{B3\_H}} + SL_{roof\_s}\right) \cdot Trib.W_{B3} = 13.2\ \frac{kN}{m}$ Specified design load

$$M_{f\_B3} := \frac{w_{f\_B3} \cdot L_{B3\_H}^{2}}{8} = 365.82\ kN \cdot m$$

$$V_{f\_B3} := \frac{w_{f\_B3} \cdot L_{B3\_H}}{2} = 117.06\ kN$$

$$V_{s\_B3} := \frac{w_{s\_B3} \cdot L_{B3}}{2} = 83.1\ kN$$

Beam B4:

$L_{B4} := 6000\ mm$

Beam B4 acts as a girder that carries point loads from B1 & B3 acting at midspan of B4.

$$V_{f\_B4} := \frac{V_{f\_B1} + V_{f\_B3}}{2} = 88.99\ kN$$

$$M_{f\_B4} := \frac{V_{f\_B4} \cdot L_{B4}}{2} = 266.98\ kN \cdot m$$

$$V_{s\_B4} := \frac{V_{s\_B1} + V_{s\_B3}}{2} = 63.01\ kN$$

Beam B6:

$Trib.W_{B6} := 3000\ mm$

$L_{B6} := 12500\ mm$

$w_{f\_L2} = 11.58\ kPa$

$w_{s\_L2} = 8.3\ kPa$

Per **NBCC 4.1.5.8:** No reduction in tributary area for live loads.

$$w_{f\_B6} := Trib.W_{B6} \cdot w_{f\_L2} = 34.73\ \frac{kN}{m}$$

$$w_{s\_B6} := Trib.W_{B6} \cdot w_{s\_L2} = 24.9\ \frac{kN}{m}$$

$$M_{f\_B6} := \frac{w_{f\_B6} \cdot L_{B6}^{\ 2}}{8} = 678.22\ kN \cdot m \qquad M_{s\_B6} := \frac{w_{s\_B6} \cdot L_{B6}^{\ 2}}{8} = 486.33\ kN \cdot m$$

$$V_{f\_B6} := \frac{w_{f\_B6} \cdot L_{B6}}{2} = 217.03\ kN \qquad V_{s\_B6} := \frac{w_{s\_B6} \cdot L_{B6}}{2} = 155.63\ kN$$

Beam B7:

$L_{B7} := 6000\ mm$

Beam B7 acts as a girder that carries load from B6 acting at the midspan of B6.
Per **NBCC 4.1.5.8:** No reduction in tributary area for live loads.

$$V_{f\_B7} := \frac{V_{f\_B6}}{2} = 108.52\ kN$$

$$M_{f\_B7} := \frac{V_{f\_B6} \cdot L_{B7}}{2} = 651.09\ kN \cdot m$$

$$V_{s\_B7} := \frac{V_{s\_B6}}{2} = 77.81\ kN$$

Column C1:

$Trib.W_{B1} = 3000\ mm$

$L_{B1} = 6530\ mm$

$w_{f\_roof} = 6.22\ kPa$

$w_{s\_roof} = 4.38\ kPa$

$$Trib.A_{C1} := \left(\frac{L_{B1}}{2} + 500\ mm\right) \cdot Trib.W_{B1} = 11.3\ m^2$$

$$P_{f\_C1} := w_{f\_roof} \cdot Trib.A_{C1} = 70.25\ kN$$

$$P_{s\_C1} := w_{s\_roof} \cdot Trib.A_{C1} = 49.49\ kN$$

Column C4:

Load from upper level:

$Trib.W_{C4} := 4500\ mm$ $\quad L_{B3\_H} = 12500\ mm$

$L_{B1} = 6530\ mm$ $\quad SL_{roof\_u} = 1.98\ kPa$

$L_{B3} = 12589.68\ mm$

$$Trib.A_{C4\_DL\_Upper} := Trib.W_{C4} \cdot \frac{L_{B1} + L_{B3}}{2} = 43.02\ m^2$$

$$P_{f\_C4\_DL\_Upper} := 1.25 \cdot DL_{roof} \cdot Trib.A_{C4\_DL\_Upper} = 139.81\ kN$$

$$Trib.A_{C4\_SL} := Trib.W_{C4} \cdot \frac{L_{B1} + L_{B3\_H}}{2} = 42.82\ m^2$$

$$P_{f\_C4\_SL} := 1.5 \cdot SL_{roof\_u} \cdot Trib.A_{C4\_SL} = 127.17\ kN$$

$$P_{f\_C4\_Upper} := P_{f\_C4\_DL\_Upper} + P_{f\_C4\_SL} = 266.98\ kN$$

$$P_{s\_C4\_Upper} := \frac{P_{f\_C4\_DL\_Upper}}{1.25} + Trib.A_{C4\_SL} \cdot SL_{roof\_s} = 188.15\ kN$$

Load from lower level:

$Trib.W_{C4} = 4500\ mm$

$L_{B6} = 12500\ mm$

$w_{f\_L2} = 11.58\ kPa$

$w_{s\_L2} = 8.3\ kPa$

Per **NBCC 4.1.5.8:** No reduction in tributary area for live loads.

$$Trib.A_{C4\_Lower} := Trib.W_{C4} \cdot \frac{L_{B6}}{2} = 28.13\ m^2$$

$$P_{f\_C4\_Lower} := w_{f\_L2} \cdot Trib.A_{C4\_Lower} = 325.55\ kN$$
$$P_{s\_C4\_Lower} := w_{s\_L2} \cdot Trib.A_{C4\_Lower} = 233.44\ kN$$
$$P_{f\_C4} := P_{f\_C4\_Upper} + P_{f\_C4\_Lower} = 592.53\ kN$$
$$P_{s\_C4} := P_{s\_C4\_Upper} + P_{s\_C4\_Lower} = 421.59\ kN$$

CLT Panel P2:

$DL_{L2} = 3.5\ kPa$

$LL_{L2} = 4.8\ kPa$

$w_{L2} := 1.25 \cdot DL_{L2} + 1.5 \cdot LL_{L2} = 11.58\ kPa$

Panel calculations based on a 1m unit width per panel.
Support conditions are fixed-pinned.

$Span_{P2} := 3000\ mm$

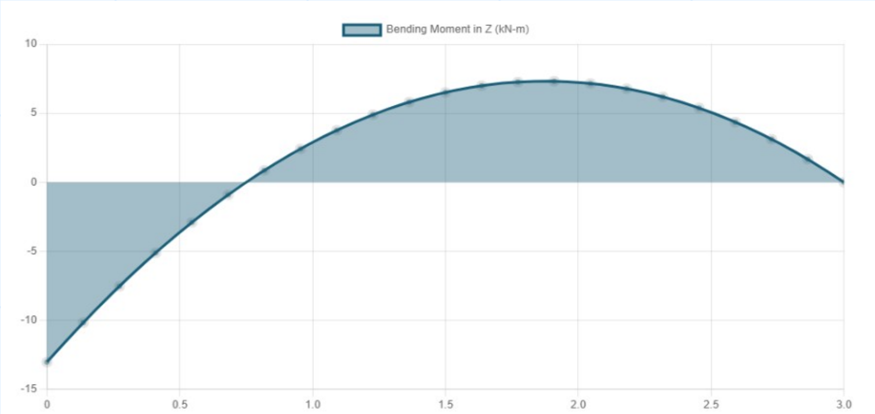
$Width_{P2} := 1000\ mm$

$$w_{P2} := w_{L2} \cdot Width_{P2} = 11.58\ \frac{kN}{m}$$

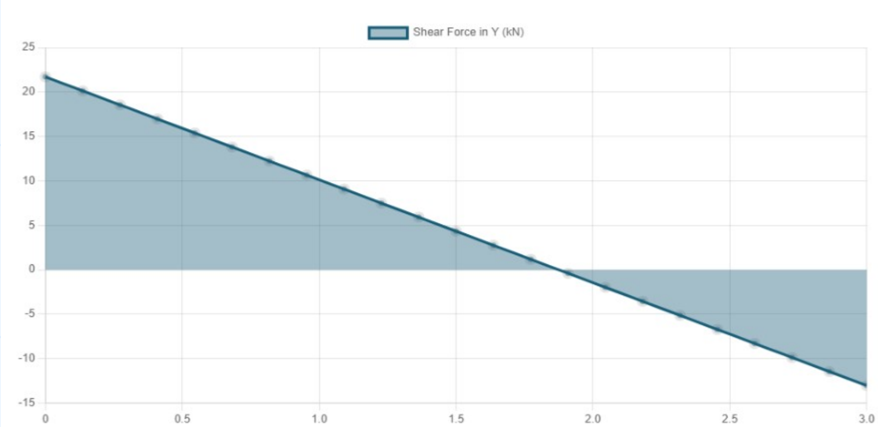
Unable to solve with current knowledge and therefore deferred to online beam calculators to obtain Mmax and Vmax.

See diagrams below from SkyCiv:

Bending moment diagram:



Shear force diagram:



$M_{f\_P2} := 13.02\ kN \cdot m$

$V_{f\_P2} := 21.7\ kN$

Appendix A3: Beam Selection

This appendix displays calculations regarding beam selection following CSA-o86-19 and WDM 2020.
All beams are simply supported.

Beam B3:

Species: SPF
Grade: 20f-E

$V_{f\_B3} = 117.06\ kN$

$M_{f\_B3} = 365.82\ kN \cdot m$

$w_{f\_B3} \cdot L_{B3} = 235.8\ kN$

$w_{s\_B3} = 13.2\ \frac{kN}{m}$

Try size 315mm x 798mm :

$base_{B3} := 315\ mm$

$depth_{B3} := 798\ mm$

$L_{B3} = 12589.68\ mm$

Check bending resistance per **CSA 7.5.3**:

Find load duration, factor KD per **CSA 5.3.2:**

$P_{L\_B3} := DL_{roof} \cdot Trib.W_{B3} \cdot L_{B3} = 98.2\ kN$

$P_{S\_B3} := SL_{roof\_u} \cdot Trib.W_{B3} \cdot L_{B3} = 74.78\ kN$

$P_{L\_B3} < P_{S\_B3}$ **Clause 5.3.2.2** applicable

$$K_{D\_B3} := 1.0 - 0.5 \cdot \log\left(\frac{P_{L\_B3}}{P_{S\_B3}}\right) = 0.94$$

Find size factor, KZbg per **CSA 7.5.6.5.1:**

$$K_{Zbg\_B3} := \left(\frac{130\ mm}{base_{B3}}\right)^{0.1} \cdot \left(\frac{610\ mm}{depth_{B3}}\right)^{0.1} \cdot \left(\frac{9100\ mm}{L_{B3}}\right)^{0.1} = 0.86$$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

Beam is laterally supported across the entire length via CLT panels

$K_{L\_B3} := 1.00$

$K_{Zbg\_B3} < K_{L\_B3}$ KZbg factor governs.

Find section modulus, S:

$$S_{B3} := \frac{base_{B3} \cdot depth_{B3}^{\ 2}}{6} = 33432210\ mm^3$$

Find factored bending strength, Fb:

$f_{b\_B3} := 25.6\ MPa$ Per **CSA Table 7.2**

$F_{b\_B3} := f_{b\_B3} \cdot K_{D\_B3} = 24.09\ MPa$

Calculate factored bending resistance for B3:

$\phi_{Beam} := 0.9$

$M_{r\_B3} := \phi_{Beam} \cdot F_{b\_B3} \cdot S_{B3} \cdot K_{Zbg\_B3} = 625.12\ kN \cdot m$

$M_{r\_B3} > M_{f\_B3}$ (Acceptable)

Check shear resistance per **CSA 7.5.7.3**:

Find shear load coefficient Cv per **CSA 7.5.7.6**:

$C_{v\_B3} := 3.69$ Per **CSA Table 7.8**, loading equal across section

Find factored shear strength, Fv:

$f_{v\_B3} := 1.75\ MPa$ Per **CSA Table 7.2**

$F_{v\_B3} := f_{v\_B3} \cdot K_{D\_B3} = 1.65\ MPa$

Calculate factored shear resistance for B3:

$Z_{B3} := base_{B3} \cdot depth_{B3} \cdot L_{B3} = 3.16\ m^3$

$A_{g\_B3} := base_{B3} \cdot depth_{B3} = 251370\ mm^2$

$W_{r\_B3} := \phi_{Beam} \cdot F_{v\_B3} \cdot 0.48 \cdot A_{g\_B3} \cdot C_{v\_B3} \cdot Z_{B3}^{-0.18} \cdot m^{3 \cdot 0.18} = 536.19\ kN$

$W_{r\_B3} > w_{f\_B3} \cdot L_{B3}$ (Acceptable)

Check deflection per **CSA 5.4**:

Beam is subject to long-term loads, **Clause 5.4.3** will govern.
Maximum allowable deflection:

$$\Delta_{Max\_B3} := \frac{L_{B3}}{360} = 34.97\ mm$$

Calculated deflection from loading:

$E_{B3} := 10300\ MPa$ Per **CSA Table 7.2**

$$I_{x\_B3} := \frac{base_{B3} \cdot depth_{B3}^{\ 3}}{12} = 13339451790\ mm^4$$

$$\Delta_{B3} := \frac{5 \cdot w_{s\_B3} \cdot L_{B3}^{\ 4}}{384 \cdot E_{B3} \cdot I_{x\_B3}} = 31.43\ mm$$

$\Delta_{Max\_B3} > \Delta_{B3}$ (Acceptable)

Beam size 315mm x 798mm is acceptable.

Beam B4:

Species: SPF
Grade: 20f-E

$V_{f\_B4} = 88.99\ kN$

$M_{f\_B4} = 266.98\ kN \cdot m$

$w_{f\_B4} := V_{f\_B1} + V_{f\_B3} = 177.99\ kN$

$w_{s\_B4} := V_{s\_B1} + V_{s\_B3} = 126.03\ kN$

Architectural consideration provided by sponsor, beam B4 is to be equal size or greater than B3.

Try size 315mm x 798mm :

$base_{B4} := 315\ mm$

$depth_{B4} := 798\ mm$

$L_{B4} = 6000\ mm$

Check bending resistance per **CSA 7.5.3**:

Find load duration factor, KD per **CSA 5.3.2:**

$$P_{L\_B4} := \frac{DL_{B3} \cdot L_{B3\_H}}{1.25 \cdot 2} + \frac{DL_{roof} \cdot Trib.W_{B1} \cdot L_{B1}}{2} = 74.57\ kN$$

$$P_{S\_B4} := \frac{SL_{roof\_u} \cdot Trib.W_{B3} \cdot L_{B3\_H}}{2} + \frac{SL_{roof\_u} \cdot Trib.W_{B1} \cdot L_{B1}}{2} = 56.52\ kN$$

$P_{L\_B4} < P_{S\_B4}$ **Clause 5.3.2.2** applicable

$$K_{D\_B4} := 1.0 - 0.5 \cdot \log\left(\frac{P_{L\_B4}}{P_{S\_B4}}\right) = 0.94$$

Find size factor, KZbg per **CSA 7.5.6.5.1:**

$$K_{Zbg\_B4} := \left(\frac{130\ mm}{base_{B4}}\right)^{0.1} \cdot \left(\frac{610\ mm}{depth_{B4}}\right)^{0.1} \cdot \left(\frac{9100\ mm}{L_{B4}}\right)^{0.1} = 0.93$$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

Beam is laterally supported across the entire length via CLT panels

$K_{L\_B4} := 1.00$

$K_{Zbg\_B4} < K_{L\_B4}$ KZbg factor governs.

Find section modulus, S:

$$S_{B4} := \frac{base_{B4} \cdot depth_{B4}^{\ 2}}{6} = 33432210\ mm^3$$

Find factored bending strength, Fb:

$f_{b\_B4} := 25.6\ MPa$ Per **CSA Table 7.2**

$F_{b\_B4} := f_{b\_B4} \cdot K_{D\_B4} = 24.06\ MPa$

Calculate factored bending resistance for B3:

$\phi_{Beam} = 0.9$

$M_{r\_B4} := \phi_{Beam} \cdot F_{b\_B4} \cdot S_{B4} \cdot K_{Zbg\_B4} = 672.48\ kN \cdot m$

$M_{r\_B4} > M_{f\_B4}$ (Acceptable)

Check shear resistance per **CSA 7.5.7.3**:

Find shear load coefficient Cv per **CSA 7.5.7.6**:

$C_{v\_B4} := 3.69$ Per **CSA Table 7.8**, loading equal across section

Find factored shear strength, Fv:

$f_{v\_B4} := 1.75\ MPa$ Per **CSA Table 7.2**

$F_{v\_B4} := f_{v\_B4} \cdot K_{D\_B4} = 1.64\ MPa$

Calculate factored shear resistance for B3:

$Z_{B4} := base_{B4} \cdot depth_{B4} \cdot L_{B4} = 1.51\ m^3$

$A_{g\_B4} := base_{B4} \cdot depth_{B4} = 251370\ mm^2$

$W_{r\_B4} := \phi_{Beam} \cdot F_{v\_B4} \cdot 0.48 \cdot A_{g\_B4} \cdot C_{v\_B4} \cdot Z_{B4}^{-0.18} \cdot m^{3 \cdot 0.18} = 612.05\ kN$

$W_{r\_B4} > w_{f\_B4}$ (Acceptable)

Check deflection per **CSA 5.4**:

Beam is subject to long-term loads, **Clause 5.4.3** will govern.
Maximum allowable deflection:

$$\Delta_{Max\_B4} := \frac{L_{B3}}{360} = 34.97\ mm$$

Calculated deflection from loading:

$E_{B4} := 10300\ MPa$ Per **CSA Table 7.2**

$$I_{x\_B4} := \frac{base_{B4} \cdot depth_{B4}^{3}}{12} = 13339451790\ mm^4$$

$$\Delta_{B4} := \frac{w_{s\_B4} \cdot L_{B4}^{3}}{48 \cdot E_{B4} \cdot I_{x\_B4}} = 4.13\ mm$$

$\Delta_{Max\_B4} > \Delta_{B4}$ (Acceptable)

Beam size 315mm x 798mm is acceptable.

Beam B6:

Species: SPF
Grade: 20f-E

$V_{f\_B6} = 217.03\ kN$

$M_{f\_B6} = 678.22\ kN \cdot m$

$w_{f\_B6} \cdot L_{B6} = 434.06\ kN$

$w_{s\_B6} = 24.9\ \frac{kN}{m}$

Try size 365mm x 950mm :

$base_{B6} := 365\ mm$

$depth_{B6} := 950\ mm$

$L_{B6} = 12500\ mm$

Check bending resistance per **CSA 7.5.3**:

Find load duration factor, KD per **CSA 5.3.2:**

$P_{L\_B6} := DL_{L2} \cdot Trib.W_{B6} \cdot L_{B6} = 131.25\ kN$

$P_{S\_B6} := LL_{L2} \cdot Trib.W_{B6} \cdot L_{B6} = 180\ kN$

$P_{S\_B6} > P_{L\_B6}$ **Clause 5.3.2.2** not applicable, defer to **Table 5.1**

$K_{D\_B6} := 0.65$

Find size factor, KZbg per **CSA 7.5.6.5.1:**

$$K_{Zbg\_B6} := \left(\frac{130\ mm}{base_{B6}}\right)^{0.1} \cdot \left(\frac{610\ mm}{depth_{B6}}\right)^{0.1} \cdot \left(\frac{9100\ mm}{L_{B6}}\right)^{0.1} = 0.84$$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

Beam is laterally supported across the entire length via CLT panels

$K_{L\_B6} := 1.00$

$K_{Zbg\_B6} < K_{L\_B6}$ KZbg factor governs.

Find section modulus, S:

$$S_{B6} := \frac{base_{B6} \cdot depth_{B6}^{\ 2}}{6} = 54902083.33\ mm^3$$

Find factored bending strength, Fb:

$f_{b\_B6} := 25.6\ MPa$ Per **CSA Table 7.2**

$F_{b\_B6} := f_{b\_B6} \cdot K_{D\_B6} = 16.64\ MPa$

Calculate factored bending resistance for B6:

$\phi_{Beam} = 0.9$

$M_{r\_B6} := \phi_{Beam} \cdot F_{b\_B6} \cdot S_{B6} \cdot K_{Zbg\_B6} = 687.26\ kN \cdot m$

$M_{r\_B6} > M_{f\_B6}$ (Acceptable)

Check shear resistance per **CSA 7.5.7.3**:

Find shear load coefficient Cv per **CSA 7.5.7.6**:

$C_{v\_B6} := 3.69$ Per **CSA Table 7.8**, loading equal across section

Find factored shear strength, Fv:

$f_{v\_B6} := 1.75\ MPa$ Per **CSA Table 7.2**

$F_{v\_B6} := f_{v\_B6} \cdot K_{D\_B6} = 1.14\ MPa$

Calculate factored shear resistance for B3:

$Z_{B6} := base_{B6} \cdot depth_{B6} \cdot L_{B6} = 4.33\ m^3$

$A_{g\_B6} := base_{B6} \cdot depth_{B6} = 346750\ mm^2$

$W_{r\_B6} := \phi_{Beam} \cdot F_{v\_B6} \cdot 0.48 \cdot A_{g\_B6} \cdot C_{v\_B6} \cdot Z_{B6}^{-0.18} \cdot m^{3 \cdot 0.18} = 482.87\ kN$

$W_{r\_B6} > w_{f\_B6} \cdot L_{B6}$ (Acceptable)

Check deflection per **CSA 5.4**:

Beam is subject to long-term loads, **Clause 5.4.3** will govern.

Maximum allowable deflection:

$$\Delta_{Max\_B6} := \frac{L_{B6}}{360} = 34.72\ mm$$

Calculated deflection from loading:

$E_{B6} := 10300\ MPa$ Per **CSA Table 7.2**

$$I_{x\_B6} := \frac{base_{B6} \cdot depth_{B6}^{\ 3}}{12} = 26078489583.33\ mm^4$$

$$\Delta_{B6} := \frac{5 \cdot w_{s\_B6} \cdot L_{B6}^{\ 4}}{384 \cdot E_{B6} \cdot I_{x\_B6}} = 29.47\ mm$$

$\Delta_{Max\_B6} > \Delta_{B6}$ (Acceptable)

Beam size 315mm x 950mm is acceptable.

Appendix A4: Column Selection

This appendix displays calculations regarding column selection following CSA-o86-19 and WDM 2020.
All columns are pinned-pinned.

Column C1:

Species: SPF
Grade: 12c-E

$P_{f\_C1} = 70.25\ kN$

Try size 215mm x 266mm :

$base_{C1} := 215\ mm$
$depth_{C1} := 266\ mm$
$L_{C1} := 9000\ mm$
$E_{C1} := 9700\ MPa$

Eccentricity on C1:

$V_{f\_B1} = 60.92\ kN$ Reaction from B1
$e := 100\ mm$ Eccentricity distance provided by sponsor

$$M_{f\_C1\_B1} := V_{f\_B1} \cdot \left(e + \frac{depth_{C1}}{2}\right) = 14.2\ kN \cdot m$$

Check interaction equation per **WDM 2020 5.1**:

Find compression resistance per **CSA 7.5.8**:

Find load duration factor, KD per **CSA 5.3.2:**

$P_{L\_C1} := DL_{roof} \cdot Trib.A_{C1} = 29.37\ kN$
$P_{S\_C1} := SL_{roof\_u} \cdot Trib.A_{C1} = 22.36\ kN$
$P_{L\_C1} > P_{S\_C1}$ **Clause 5.3.2.2** applicable

$$K_{D\_C1} := 1.0 - 0.5 \cdot \log\left(\frac{P_{L\_C1}}{P_{S\_C1}}\right) = 0.94$$

Find factored compression combined with bending strength, Fcb:

$f_{cb\_C1} := 25.2\ MPa$ Per **CSA Table 7.2**
$F_{cb\_C1} := f_{cb\_C1} \cdot K_{D\_C1} = 23.71\ MPa$

Find size factor, KZcg per **CSA 7.5.8.5:**

$Z_{C1} := base_{C1} \cdot depth_{C1} \cdot L_{C1} = 0.51\ m^3$
$K_{Zcg\_C1} := 0.68 \cdot Z_{C1}^{-0.13} \cdot m^{3 \cdot 0.13} = 0.74$

Find slenderness factor, KC per **CSA 7.5.8.6:**

Find slenderness ratio, Cc per **CSA 7.5.8.2:**

$L_{e\_c\_C1} := L_{C1} = 9000\ mm$ Column is pinned-pinned, Ke = 1.0

$$C_{c\_C1\_Width} := \frac{L_{e\_c\_C1}}{base_{C1}} = 41.86 \qquad C_{c\_C1\_Depth} := \frac{L_{e\_c\_C1}}{depth_{C1}} = 33.83$$

$C_{c\_C1\_Width} > C_{c\_C1\_Depth}$ Use width slenderness ratio

$$K_{C\_C1} := \left(1 + \frac{F_{cb\_C1} \cdot K_{Zcg\_C1} \cdot {C_{c\_C1\_Width}}^3}{35 \cdot 0.87 \cdot E_{C1}}\right)^{-1} = 0.19$$

Calculate factored compression combined with bending for C1:

$\phi_{Column} := 0.8$

$A_{C1} := base_{C1} \cdot depth_{C1} = 57190\ mm^2$

$P_{r\_C1} := \phi_{Column} \cdot F_{cb\_C1} \cdot A_{C1} \cdot K_{Zcg\_C1} \cdot K_{C\_C1} = 149.89\ kN$

$P_{r\_C1} > P_{f\_C1}$ (Acceptable)

Find bending resistance per **CSA 7.5.3**:

Find factored bending strength, Fb:

$f_{b\_C1} := 9.8\ MPa$ Per **CSA Table 7.2**

$F_{b\_C1} := f_{b\_C1} \cdot K_{D\_C1} = 9.22\ MPa$

Find size factor, KZbg per **CSA 7.5.6.5.1:**

$$K_{Zbg\_C1} := \left(\frac{130\ mm}{base_{C1}}\right)^{0.1} \cdot \left(\frac{610\ mm}{depth_{C1}}\right)^{0.1} \cdot \left(\frac{9100\ mm}{L_{C1}}\right)^{0.1} = 1.03$$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

$L_{e\_b\_C1} := 1.92 \cdot L_{C1} = 17280\ mm$

$$C_{B\_C1} := \sqrt{\frac{L_{e\_b\_C1} \cdot depth_{C1}}{{base_{C1}}^2}} = 9.97$$

$K_{L\_C1} := 1.00$ Per **CSA 7.5.6.4.4**

$K_{Zbg\_C1} > K_{L\_C1}$ KL factor governs.

Find section modulus, S:

$$S_{C1} := \frac{base_{C1} \cdot {depth_{C1}}^2}{6} = 2535423.33\ mm^3$$

Calculate factored bending resistance for C1:

$\phi_{Beam} = 0.9$

$M_{r\_C1} := \phi_{Beam} \cdot F_{b\_C1} \cdot S_{C1} \cdot K_{L\_C1} = 21.04\ kN \cdot m$

$M_{r\_C1} > M_{f\_C1}$ (Acceptable)

Calculate interaction equation:

$$C1_{Capacity} := \left(\frac{P_{f\_C1}}{P_{r\_C1}}\right)^2 + \left(\frac{M_{f\_C1\_B1}}{M_{r\_C1}}\right) = 0.89$$

$C1_{Capacity} < 1$ (Acceptable)

Column size 265mm x 266mm is acceptable.

Column C4:

Species: SPF
Grade: 20f-EX

$P_{f\_C4} = 592.53\ kN$

Try size 400mm x 456mm :

$base_{C4} := 400\ mm$
$depth_{C4} := 456\ mm$
$L_{C4} := 6000\ mm$
$E_{C4} := 10300\ MPa$

Eccentricity on C4:

$V_{f\_B6} = 217.03\ kN$ Reaction from B6
$V_{f\_B7} = 108.52\ kN$ Reaction from B7
$e = 100\ mm$ Eccentricity distance provided by sponsor

$$M_{f\_C4\_B6} := V_{f\_B6} \cdot \left(e + \frac{depth_{C4}}{2}\right) = 71.19\ kN \cdot m$$

$$M_{f\_C4\_B7} := V_{f\_B7} \cdot \left(e + \frac{base_{C4}}{2}\right) = 32.55\ kN \cdot m$$

Check interaction equation per **WDM 2020 5.1**:

Find compression resistance per **CSA 7.5.8**:

Find load duration factor, KD per **CSA 5.3.2:**

$P_{L\_C4} := DL_{roof} \cdot Trib.A_{C4\_DL\_Upper} + DL_{L2} \cdot Trib.A_{C4\_Lower} = 210.29\ kN$
$P_{S\_C4\_SL} := SL_{roof\_u} \cdot Trib.A_{C4\_SL} = 84.78\ kN$
$P_{S\_C4\_LL} := LL_{L2} \cdot Trib.A_{C4\_Lower} = 135\ kN$
$P_{S\_C4} := P_{S\_C4\_LL} + 0.5 \cdot P_{S\_C4\_SL} = 177.39\ kN$
$P_{L\_C4} > P_{S\_C4}$ **Clause 5.3.2.2** applicable

$$K_{D\_C4} := 1.0 - 0.5 \cdot \log\left(\frac{P_{L\_C4}}{P_{S\_C4}}\right) = 0.96$$

Find factored compression combined with bending strength, Fcb:

$f_{cb\_C4} := 25.2\ MPa$ Per **CSA Table 7.2**
$F_{cb\_C4} := f_{cb\_C4} \cdot K_{D\_C4} = 24.27\ MPa$

Find size factor, KZcg per **CSA 7.5.8.5:**

$Z_{C4} := base_{C4} \cdot depth_{C4} \cdot L_{C4} = 1.09\ m^3$
$K_{Zcg\_C4} := 0.68 \cdot Z_{C4}^{-0.13} \cdot m^{3 \cdot 0.13} = 0.67$

Find slenderness factor, KC per **CSA 7.5.8.6:**

Find slenderness ratio, Cc per **CSA 7.5.8.2:**

$L_{e\_c\_C4} := L_{C4} = 6000\ mm$ Column is pinned-pinned, Ke = 1.0

$$C_{c\_C4\_Width} := \frac{L_{e\_c\_C4}}{base_{C4}} = 15 \qquad C_{c\_C4\_Depth} := \frac{L_{e\_c\_C4}}{depth_{C4}} = 13.16$$

$C_{c\_C1\_Width} > C_{c\_C1\_Depth}$ Use width slenderness ratio

$$K_{C\_C4} := \left(1 + \frac{F_{cb\_C4} \cdot K_{Zcg\_C4} \cdot C_{c\_C4\_Width}^{3}}{35 \cdot 0.87 \cdot E_{C4}}\right)^{-1} = 0.85$$

Calculate factored compression combined with bending for C1:

$\phi_{Column} = 0.8$

$A_{C4} := base_{C4} \cdot depth_{C4} = 182400\ mm^2$

$P_{r\_C4} := \phi_{Column} \cdot F_{cb\_C4} \cdot A_{C4} \cdot K_{Zcg\_C4} \cdot K_{C\_C4} = 2024.67\ kN$

$P_{r\_C4} > P_{f\_C4}$ (Acceptable)

Find bending resistance per **CSA 7.5.3**:

Bi-axial bending is present, check strong direction first:

Find factored bending strength in strong axis, Fb:

$f_{b\_C4\_x} := 25.6\ MPa$ Per **CSA Table 7.2**

$F_{b\_C4\_x} := f_{b\_C4\_x} \cdot K_{D\_C4} = 24.65\ MPa$

Find size factor, KZbg per **CSA 7.5.6.5.1:**

$$K_{Zbg\_C4} := \left(\frac{130\ mm}{base_{C4}}\right)^{0.1} \cdot \left(\frac{610\ mm}{depth_{C4}}\right)^{0.1} \cdot \left(\frac{9100\ mm}{L_{C4}}\right)^{0.1} = 0.96$$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

$L_{e\_b\_C4} := 1.92 \cdot L_{C4} = 11520\ mm$

$$C_{B\_C4} := \sqrt{\frac{L_{e\_b\_C4} \cdot depth_{C4}}{base_{C4}^{2}}} = 5.73$$

$K_{L\_C4} := 1.00$ Per **CSA 7.5.6.4.4**

$K_{Zbg\_C4} < K_{L\_C4}$ KZbg factor governs.

Find section modulus, S:

$$S_{C4\_x} := \frac{base_{C4} \cdot depth_{C4}^{2}}{6} = 13862400\ mm^3$$

Calculate factored bending resistance for C4 against B6:

$\phi_{Beam} = 0.9$

$M_{r\_C4\_x} := \phi_{Beam} \cdot F_{b\_C4\_x} \cdot S_{C4\_x} \cdot K_{Zbg\_C4} = 295.04\ kN \cdot m$

$M_{r\_C4\_x} > M_{f\_C4\_B6}$ (Acceptable)

Bi-axial bending present, **CSA 7.5.3** applies in weak direction:

Find factored bending strength in strong axis, Fb:

$f_{b\_C4\_y} := 0.77 \cdot 6.3\ MPa = 4.85\ MPa$ Per **CSA Table 7.2**

$K_{H\_C4} := 1.10$ Per **CSA 6.4.4.3** and **Table 6.12**

$F_{b\_C4\_y} := f_{b\_C4\_y} \cdot K_{D\_C4} \cdot K_{H\_C4} = 5.14\ MPa$

$K_{Zb\_C4} := 0.9$ Per **Table 6.13:**

Find section modulus, S:

$$S_{C4\_y} := \frac{depth_{C4} \cdot base_{C4}^{2}}{6} = 12160000\ mm^3$$

Calculate factored bending resistance for C4 against B7:

$\phi_{Beam} = 0.9$

$M_{r\_C4\_y} := \phi_{Beam} \cdot F_{b\_C4\_y} \cdot S_{C4\_y} \cdot K_{Zb\_C4} = 50.62\ kN \cdot m$

$M_{r\_C4\_y} > M_{f\_C4\_B7}$ (Acceptable)

Calculate interaction equation:

$$C4_{Capacity} := \left(\frac{P_{f\_C4}}{P_{r\_C4}}\right)^2 + \left(\frac{M_{f\_C4\_B6}}{M_{r\_C4\_x}}\right) + \left(\frac{M_{f\_C4\_B7}}{M_{r\_C4\_y}}\right) = 0.97$$

$C4_{Capacity} < 1$ (Acceptable)

Column size 400mm x 456mm is acceptable.

Appendix A5: CLT Panel Selection

This appendix displays calculations regarding CLT panel selection following CSA-o86-19.
Per sponsor guidance, calculations were based on the assumption that panels must cover at least 2 spans.

CLT Panel P2:

Species: SPF

$M_{f\_P2} = 13.02\ kN \cdot m$

$V_{f\_P2} = 21.7\ kN$

Try grade E1, 175mm thick:

Check bending capacity of panel per **CSA 8.4.3.1:**

Find load duration factor, KD per **CSA 5.3.2:**

$P_{L\_P2} := DL_{L2} \cdot Span_{P2} \cdot Width_{P2} = 10.5\ kN$

$P_{S\_P2} := LL_{L2} \cdot Span_{P2} \cdot Width_{P2} = 14.4\ kN$

$P_{S\_P2} > P_{L\_P2}$ **Clause 5.3.2.2** not applicable, defer to **Table 5.1**

$K_{D\_P2} := 0.65$

Find flatwise bending strength in major direction, Fb per **CSA 8.4.3.1:**

Values from American Notional Standard for Performance-Rated CLT:

$f_{b\_P2}S_{eff\_f\_0} := 98 \cdot 10^6\ N \cdot mm$

$F_{b\_P2}S_{eff\_f\_0} := f_{b\_P2}S_{eff\_f\_0} \cdot K_{D\_P2} = 63.7\ 10^6\ N \cdot mm$

Calculate flatwise bending resistance, Mrf0 per **CSA 8.4.3.1:**

$\phi_{CLT} := 0.9$

$K_{rb\_0} := 0.85$

$M_{r\_f\_0\_P2} := \phi_{CLT} \cdot F_{b\_P2}S_{eff\_f\_0} \cdot K_{rb\_0} = 48.73\ kN \cdot m$

$M_{r\_f\_0\_P2} > M_{f\_P2}$ (Acceptable)

Check shear capacity of panel per **CSA 8.4.4.2:**

Find flatwise shear strength in major direction, Vrf0 per **CSA 8.4.4.2:**

Values from American Notional Standard for Performance-Rated CLT:

$v_{s\_0\_P2} := 58\ \frac{kN}{m} \cdot 1\ m = 58\ kN$

$V_{s\_0\_P2} := v_{s\_0\_P2} \cdot K_{D\_P2} = 37.7\ kN$

Calculate flatwise shear resistance, Vrf0 per **CSA 8.4.4.2:**

$\phi_{CLT} = 0.9$

$V_{r\_f\_0\_P2} := \phi_{CLT} \cdot V_{s\_0\_P2} = 33.93\ kN$

$V_{r\_f\_0\_P2} > V_{f\_P2}$ (Acceptable)

Use CLT panel E1 at 175mm thickness.

Appendix A6: Fire Design

This appendix displays calculations regarding the fire design portion of CSA-o86 Annex B.
A value of 120-minutes was used as the fire-resistance rating.

Calculate reduction in members due to fire exposure per **CSA B.4:**

Per **Clause B.4.1**, use **B.4.4** to calculate reduction:

$t_{fire} := 120\ min$

$\beta_n := 0.70\ \frac{mm}{min}$ — Per **Table B.2**

$x_{c\_n} := \beta_n \cdot t_{fire} = 84\ mm$

Per **CSA B.5**, use account for zero-strength layer depth:

$x_t := 7\ mm$ — t > 20 minutes, therefore xt defaults to 7mm

Beam B6:

Species: SPF
Grade: 20f-E

$V_{s\_B6} = 155.63\ kN$
$M_{s\_B6} = 486.33\ kN \cdot m$
$w_{s\_B6} \cdot L_{B6} = 311.25\ kN$

Per **Clause B.1.3**, specified loads are used to check fire resistance

Fire exposure only occurs on sides and bottoms of beams, therefore the depth is only reduced on one side

$base_{B6\_reduced} := base_{B6} - 2 \cdot (x_{c\_n} + x_t) = 183\ mm$

$depth_{B6\_reduced} := depth_{B6} - (x_{c\_n} + x_t) = 859\ mm$

$L_{B6} = 12500\ mm$

Confirm that initial size of 365mm x 950mm is acceptable per **Annex B**:

Check bending resistance per **CSA 7.5.8** in accordance with **Annex B:**

Modified K factors:

$K_{D\_fire} := 1.15$	Per **CSA B.3.3**
$K_{Zbg\_B6} = 0.84$	Per **CSA B.3.5**
$K_{L\_B6} = 1$	Per **CSA B.3.6**
$K_{fi} := 1.35$	Per **CSA B.3.9**
$K_{Zbg\_B6} < K_{L\_B6}$	KZbg factor governs.
$\phi_{fire} := 1.0$	Per **CSA B.3.2**

Calculate new section modulus:

$$S_{B6\_reduced} := \frac{base_{B6\_reduced} \cdot depth_{B6\_reduced}^{\ 2}}{6} = 22505370.5\ mm^3$$

Find factored bending strength, Fb:

$F_{b\_B6\_reduced} := f_{b\_B6} \cdot K_{D\_fire} \cdot K_{fi} = 39.74\ MPa$

Calculate factored bending resistance for B6:

$M_{r\_B6\_reduced} := \phi_{fire} \cdot F_{b\_B6\_reduced} \cdot S_{B6\_reduced} \cdot K_{Zbg\_B6} = 747.65\ kN \cdot m$

$M_{r\_B6\_reduced} > M_{s\_B6}$ (Acceptable)

Check shear resistance per **CSA 7.5.3** in accordance with **Annex B:**

Find factored shear strength, Fb:

$F_{v\_B6\_reduced} := f_{v\_B6} \cdot K_{D\_fire} \cdot K_{fi} = 2.72\ MPa$

Calculate shear resistance for B6:

$Z_{B6\_reduced} := base_{B6\_reduced} \cdot depth_{B6\_reduced} \cdot L_{B6} = 1.96\ m^3$

$A_{B6\_reduced} := base_{B6\_reduced} \cdot depth_{B6\_reduced} = 157197\ mm^2$

$W_{r\_B6\_reduced} := \phi_{fire} \cdot F_{v\_B6\_reduced} \cdot 0.48 \cdot A_{B6\_reduced} \cdot C_{v\_B6} \cdot Z_{B6\_reduced}^{-0.18} \cdot m^{3 \cdot 0.18} = 669.85\ kN$

$W_{r\_B6\_reduced} > w_{s\_B6} \cdot L_{B6}$ (Acceptable)

No deflection checks required when performing fire design.

Initial beam size of 365mm x 950mm is acceptable.

Column C1:

Species: SPF

Grade: 20f-E

$P_{s\_C1} = 49.49\ kN$

$V_{s\_B1} = 42.92\ kN$

Per **Clause B.1.3**, specified loads are used to check fire resistance

Fire exposure only occurs on all sides of column

$base_{C1\_reduced} := base_{C1} - 2 \cdot (x_{c\_n} + x_t) = 33\ mm$

$depth_{C1\_reduced} := depth_{C1} - 2 \cdot (x_{c\_n} + x_t) = 84\ mm$

$L_{C1} = 9000\ mm$

Confirm that initial size of 265mm x 266mm is acceptable per **Annex B**:

Check compressive resistance per **CSA 7.5.3** in accordance with **Annex B:**

K factors per Annex B:

$K_{D\_fire} = 1.15$ Per **CSA B.3.3**

$K_{Zcg\_C1} = 0.74$ Per **CSA B.3.5**

Find factored compressive strength, Fcb:

$F_{cb\_C1\_reduced} := f_{cb\_C1} \cdot K_{D\_fire} \cdot K_{fi} = 39.12\ MPa$

Find modified slenderness factor, KC:

Find slenderness ratio, Cc per **CSA 7.5.8.2:**

$L_{e\_c\_C1} = 9000\ mm$ Column is pinned-pinned, Ke = 1.0

$C_{c\_C1\_Red.Width} := \frac{L_{e\_c\_C1}}{base_{C1\_reduced}} = 272.73$ $C_{c\_C1\_Red.Depth} := \frac{L_{e\_c\_C1}}{depth_{C1\_reduced}} = 107.14$

$C_{c\_C1\_Red.Width} > C_{c\_C1\_Red.Depth}$ Use width slenderness ratio

$$K_{C\_C1\_reduced} := \left(1 + \frac{F_{cb\_C1\_reduced} \cdot K_{Zcg\_C1} \cdot {C_{c\_C1\_Red.Width}}^3}{35 \cdot 0.87 \cdot E_{C1}}\right)^{-1} = 0.0005$$

Calculate new compressive strength for C1:

$A_{C1\_reduced} := base_{C1\_reduced} \cdot depth_{C1\_reduced} = 2772\ mm^2$

$P_{r\_C1\_fire} := \phi_{fire} \cdot F_{cb\_C1\_reduced} \cdot A_{C1\_reduced} \cdot K_{Zcg\_C1} \cdot K_{C\_C1\_reduced} = 0.04\ kN$

$P_{r\_C1\_fire} < P_{s\_C1}$ (Unacceptable)

Initial column size of 265mm x 266mm is not sufficient.

Try new size of 365mm x 418mm:

$base_{C1.1} := 365\ mm$

$depth_{C1.1} := 418\ mm$

$base_{C1.1\_reduced} := base_{C1.1} - 2 \cdot (x_{c\_n} + x_t) = 183\ mm$

$depth_{C1.1\_reduced} := depth_{C1.1} - 2 \cdot (x_{c\_n} + x_t) = 236\ mm$

$$M_{s\_C1.1} := V_{s\_B1} \cdot \left(\frac{depth_{C1.1}}{2} + e\right) = 13.26\ kN \cdot m$$

Check compressive resistance per **CSA 7.5.3** in accordance with **Annex B:**

Find modified slenderness factor, KC:

Find slenderness ratio, Cc per **CSA 7.5.8.2:**

$L_{e\_c\_C1} = 9000\ mm$ Column is pinned-pinned, Ke = 1.0

$C_{c\_C1.1\_Red.Width} := \frac{L_{e\_c\_C1}}{base_{C1.1\_reduced}} = 49.18$ $C_{c\_C1.1\_Red.Depth} := \frac{L_{e\_c\_C1}}{depth_{C1.1\_reduced}} = 38.14$

$C_{c\_C1.1\_Red.Width} > C_{c\_C1.1\_Red.Depth}$ Use width slenderness ratio

$$K_{C\_C1.1\_reduced} := \left(1 + \frac{F_{cb\_C1\_reduced} \cdot K_{Zcg\_C1} \cdot {C_{c\_C1.1\_Red.Width}}^3}{35 \cdot 0.87 \cdot E_{C1}}\right)^{-1} = 0.08$$

Calculate new compressive strength for C1:

$A_{C1.1\_reduced} := base_{C1.1\_reduced} \cdot depth_{C1.1\_reduced} = 43188\ mm^2$

$P_{r\_C1.1\_fire} := \phi_{fire} \cdot F_{cb\_C1\_reduced} \cdot A_{C1.1\_reduced} \cdot K_{Zcg\_C1} \cdot K_{C\_C1.1\_reduced} = 98.78\ kN$

$P_{r\_C1.1\_fire} > P_{s\_C1}$ (Acceptable)

Check bending resistance per **CSA 7.5.3**:

K factors per Annex B:

$K_{Zbg\_C1} = 1.03$ Per **CSA B.3.5**

Find factored bending strength, Fb:

$F_{b\_C1.1} := f_{b\_C1} \cdot K_{D\_fire} \cdot K_{fi} = 15.21\ MPa$

Find lateral stability factor, KL per **CSA 7.5.6.4:**

$L_{e\_b\_C1} = 17280\ mm$

$$C_{B\_C1.1} := \sqrt{\frac{L_{e\_b\_C1} \cdot depth_{C1.1\_reduced}}{base_{C1.1\_reduced}^{\,2}}} = 11.04$$

$$C_{K\_C1.1} := \sqrt{\frac{0.97 \cdot E_{C1}}{F_{b\_C1.1}}} = 24.87$$

$10 < C_{B\_C1.1} < C_{K\_C1.1}$

$$K_{L\_C1.1\_reduced} := 1 - \frac{1}{3}\left(\frac{C_{B\_C1.1}}{C_{K\_C1.1}}\right)^4 = 0.99$$

$K_{Zbg\_C1} > K_{L\_C1.1\_reduced}$ KL factor governs.

Find reduced section modulus, S:

$$S_{C1.1\_reduced} := \frac{base_{C1.1\_reduced} \cdot depth_{C1.1\_reduced}^{\,2}}{6} = 1698728\ mm^3$$

Calculate factored bending resistance for C1:

$M_{r\_C1.1\_reduced} := \phi_{fire} \cdot F_{b\_C1.1} \cdot S_{C1.1\_reduced} \cdot K_{L\_C1.1\_reduced} = 25.51\ kN \cdot m$

$M_{r\_C1.1\_reduced} > M_{s\_C1}$ (Acceptable)

Check interaction equation:

$$C1.1_{Reduced\_capacity} := \left(\frac{P_{s\_C1}}{P_{r\_C1.1\_fire}}\right)^2 + \left(\frac{M_{s\_C1.1}}{M_{r\_C1.1\_reduced}}\right) = 0.77$$

$C1.1_{Reduced\_capacity} < 1$ (Acceptable)

Use new size of 365mm x 418mm.

Appendix A7: Connection Design

This appendix displays calculations regarding the connection designs between beams and columns following **CSA-o86-19 12.4**.

Due to the orientation of beams and columns under gravity loading, all beams will need to be checked when loaded perpendicular to grain. All columns will be checked for parallel to grain loading.

Information provided from industry sponsor:

- G40.21 350W steel plates are used
- 19mm dia. pins are used
- Maximum of 1 row of pins allowed in beams
- Minimum of 2 rows of pins in columns
- Ignore angle loading for this project
- Plates and pins must be protected from fire
- Beam-to-beam connections will not be checked via calculations (use MEGANT catalogue)

Check connection at beam end:

Perpendicular loading to grain configuration per **CSA Figure 12.6:**

$F_y := 310\ MPa$

$d_F := 19\ mm$

$S_{P\_Beams} := 0\ mm$

$S_{Q\_Beams} := 3 \cdot d_F = 57\ mm$

$a_{Beams} := 4 \cdot d_F = 76\ mm$

$e_{P\_Beams} := 1.5 \cdot d_F = 28.5\ mm$

$e_{Q\_Beams} := 4 \cdot d_F = 76\ mm$

Beam B6:

Check 1 - Yielding failure mode per **CSA 12.4.4.3:**

$N_{f\_B6} := V_{f\_B6} = 217.03\ kN$

Find Unit lateral yielding resistance, nu, per **CSA 12.4.4.3.2:**

Required to check Items A, C, D, and G:

$G_{SPF} := 0.42$ Per **Table A.11**

$J_x := 1.0$ Per **CSA 12.4.4.3.3.1**

$\theta_{bearing} := 0\ °$

$K_{D\_B6} = 0.65$

$K_{SF} := 1.0$

$K_T := 1.0$

$$f_{iP} := 50 \cdot G_{SPF} \cdot \left(1 - \frac{0.01 \cdot d_F}{mm}\right) \cdot J_x = 17.01$$

$$f_{iQ} := 22 \cdot G_{SPF} \cdot \left(1 - \frac{0.01 \cdot d_F}{mm}\right) \cdot J_x = 7.48$$

$$f_{1\_B6} := \frac{f_{iP} \cdot f_{iQ}}{f_{iP} \cdot \sin\left(\theta_{bearing}\right)^2 + f_{iQ} \cdot \cos\left(\theta_{bearing}\right)^2} \cdot K_{D\_B6} \cdot K_{SF} \cdot K_T = 11.06$$

$t_2 := 15\ mm$

$t_{1\_B6} := \dfrac{(base_{B6} - 2 \cdot x_{c\_n} - t_2)}{2} = 91\ mm$ Per **CSA Figure 12.4**

$K_{SP} := 3.0$ Per **CSA 12.4.4.3.3.2**

$f_u := 450\ MPa$ Value from industry sponsor

$\phi_{Steel} := 0.8$ Per **CSA 12.4.4.3.3.2**

$\phi_{y\_steel} := 0.8$ Per **CSA 12.4.4.3.3.2**

$$f_2 := K_{SP} \cdot \left(\frac{\phi_{Steel}}{\phi_{y\_steel}}\right) \cdot f_u = 1350\ MPa$$

$$Item_{A\_B6} := \frac{d_F \cdot f_{1\_B6} \cdot t_{1\_B6}}{mm^2} = 19116.69$$

$$Item_{C\_B6} := \frac{0.5 \cdot f_2 \cdot d_F \cdot t_2}{N} = 192375$$

$$Item_{D\_B6} := f_{1\_B6} \cdot d_F^{\,2} \cdot \frac{\left(\sqrt{\frac{1}{6} \cdot \frac{f_2}{f_{1\_B6} \cdot MPa + f_2} \cdot \frac{F_y}{f_{1\_B6}}} \cdot MPa^{0.5} + \frac{t_{1\_B6}}{5 \cdot d_F} \cdot MPa\right)}{N} = 12416.44$$

$$Item_{G\_B6} := f_{1\_B6} \cdot d_F^{\,2} \cdot \frac{\left(\sqrt{\frac{2}{3} \cdot \frac{f_2}{f_{1\_B6} \cdot MPa + f_2} \cdot \frac{F_y}{f_{1\_B6}}} \cdot MPa^{0.5}\right)}{N} = 17186.21$$

Item D is the lowest value and therefore governs

Try 1 row with 11 pins:

Calculate yielding resistance, Nr, per **CSA 12.4.4.3.1**:

$\phi_y := 0.8$

$n_s := 2$ 2 shear planes in connection

$n_{F\_B6} := 11$ 11 fasteners in connection

$n_{u\_B6} := min\left(Item_{A\_B6}, Item_{C\_B6}, Item_{D\_B6}, Item_{G\_B6}\right) = 12416.44$

$N_{r\_B6} := \phi_y \cdot n_s \cdot n_{F\_B6} \cdot n_{u\_B6} \cdot N = 218.53\ kN$

$N_{r\_B6} > N_{f\_B6}$ (Acceptable)

Check 2 - Splitting failure mode per **CSA 12.4.4.7:**

$Q_{f\_B6} := V_{f\_B6} = 217.03\ kN$

Calculate factored splitting resistance, QSri:

$\phi_w := 0.7$

$d_{e\_B6} := depth_{B6} - e_{P\_Beams} = 921.5\ mm$

$$QS_{i\_B6} := 14 \cdot base_{B6} \cdot \sqrt{\frac{d_{e\_B6}}{1 - \frac{d_{e\_B6}}{depth_{B6}}}} \cdot \frac{1}{mm^{\frac{3}{2}}} = 895587.33$$

$QS_{ri\_B6} := \phi_w \cdot QS_{i\_B6} \cdot K_{D\_B6} \cdot N = 407.49\ kN$

$QS_{ri\_B6} > Q_{f\_B6}$ (Acceptable)

As beam is not loaded in tension, no other checks are required.

Use G40.21 350W steel plate at 15mm thickness with 11-19mm dia. pins at beam B6 end.

Check connection at column end:

Parallel loading to grain configuration per **CSA Figure 12.5.b:**

$F_y = 310\ MPa$

$d_F = 19\ mm$

$S_{P\_Columns} := 4 \cdot d_F = 76\ mm$

$S_{Q\_Columns} := 3 \cdot d_F = 57\ mm$

$a_{Columns} := 4 \cdot d_F = 76\ mm$

$a_{L\_Columns} := 5 \cdot d_F = 95\ mm$

$e_{P\_Columns} := 1.5 \cdot d_F = 28.5\ mm$

Column C4 from beams B1, B3, B4:

Loading is purely in compression, only check required is yielding.

Check 1 - Yielding failure mode per **CSA 12.4.4.3:**

Part 1 - loads from B1 and B3:

$N_{f\_C4\_B1\_B3} := V_{f\_B1} + V_{f\_B3} = 177.99\ kN$

Find Unit lateral yielding resistance, nu, per **CSA 12.4.4.3.2:**

Required to check Items A, C, D, and G:

$G_{SPF} = 0.42$ Per **Table A.11**

$J_x = 1$ Per **CSA 12.4.4.3.3.1**

$\theta_{bearing} = 0$

$K_{D\_B6} = 0.65$

$K_{SF} = 1$

$K_T = 1$

$f_{iP} = 17.01$

$f_{iQ} = 7.48$

$$f_{1\_C4} := \frac{f_{iP} \cdot f_{iQ}}{f_{iP} \cdot \sin\left(\theta_{bearing}\right)^2 + f_{iQ} \cdot \cos\left(\theta_{bearing}\right)^2} \cdot K_{D\_C4} \cdot K_{SF} \cdot K_T = 16.38$$

$t_2 = 15\ mm$

$base_{C4\_fire\_size} := 490\ mm$

$$t_{1\_C4} := \frac{\left(base_{C4\_fire\_size} - 2 \cdot x_{c\_n} - t_2\right)}{2} = 153.5\ mm$$ Per **CSA Figure 12.4**

$f_2 = 1350\ MPa$

$$Item_{A\_C4} := \frac{d_F \cdot f_{1\_C4} \cdot t_{1\_C4}}{mm^2} = 47776.93$$

$$Item_{C\_C4} := \frac{0.5 \cdot f_2 \cdot d_F \cdot t_2}{N} = 192375$$

$$Item_{D\_C4} := f_{1\_C4} \cdot d_F^{\ 2} \cdot \frac{\left(\sqrt{\frac{1}{6} \cdot \frac{f_2}{f_{1\_C4} \cdot MPa + f_2} \cdot \frac{F_y}{f_{1\_C4}}} \cdot MPa^{0.5} + \frac{t_{1\_C4}}{5 \cdot d_F} \cdot MPa\right)}{N} = 19994.69$$

$$Item_{G\_C4} := f_{1\_C4} \cdot d_F^{\ 2} \cdot \frac{\left(\sqrt{\frac{2}{3} \cdot \frac{f_2}{f_{1\_C4} \cdot MPa + f_2} \cdot \frac{F_y}{f_{1\_C4}}} \cdot MPa^{0.5}\right)}{N} = 20878.6$$

Item D is the lowest value and therefore governs

Try 2 rows with 3 pins each:

Calculate yielding resistance, Nr, per **CSA 12.4.4.3.1**:

$\phi_y = 0.8$

$n_s = 2$ 2 shear planes in connection

$n_{F\_C4\_base} := 2 \cdot 3 = 6$ 6 fasteners in connection

$n_{u\_C4} := min\left(Item_{A\_C4}, Item_{C\_C4}, Item_{D\_C4}, Item_{G\_C4}\right) = 19994.69$

$N_{r\_C4} := \phi_y \cdot n_s \cdot n_{F\_C4\_base} \cdot n_{u\_C4} \cdot N = 191.95\ kN$

$N_{r\_C4\_B1\_B3} > N_{f\_C4\_B1\_B3}$ (Acceptable)

Use 2 rows with 3 pins each to connect B1 and B3.

Part 2 - load from B4:

$N_{f\_C4\_B4} := V_{f\_B4} = 88.99\ kN$

Due to B4 mounting on the wider side of the column, t1 would be the only value changing and would make all other Items involving that value larger. Use part 1's t1 value to be conservative.

Try 2 rows with 2 pins each:

Calculate yielding resistance, Nr, per **CSA 12.4.4.3.1**:

$\phi_y = 0.8$

$n_s = 2$ 2 shear planes in connection

$n_{F\_C4\_depth} := 2 \cdot 2 = 4$ 4 fasteners in connection

$n_{u\_C4} = 19994.69$

$N_{r\_C4\_B4} := \phi_y \cdot n_s \cdot n_{F\_C4\_depth} \cdot n_{u\_C4} \cdot N = 127.97\ kN$

$N_{r\_C4\_B4} > N_{f\_C4\_B4}$ (Acceptable)

Use 2 rows with 2 pins each to connect B4.

APPENDIX B: DRAWING PACKAGE

List of Drawings:

GLULAM BEAM SCHEDULE

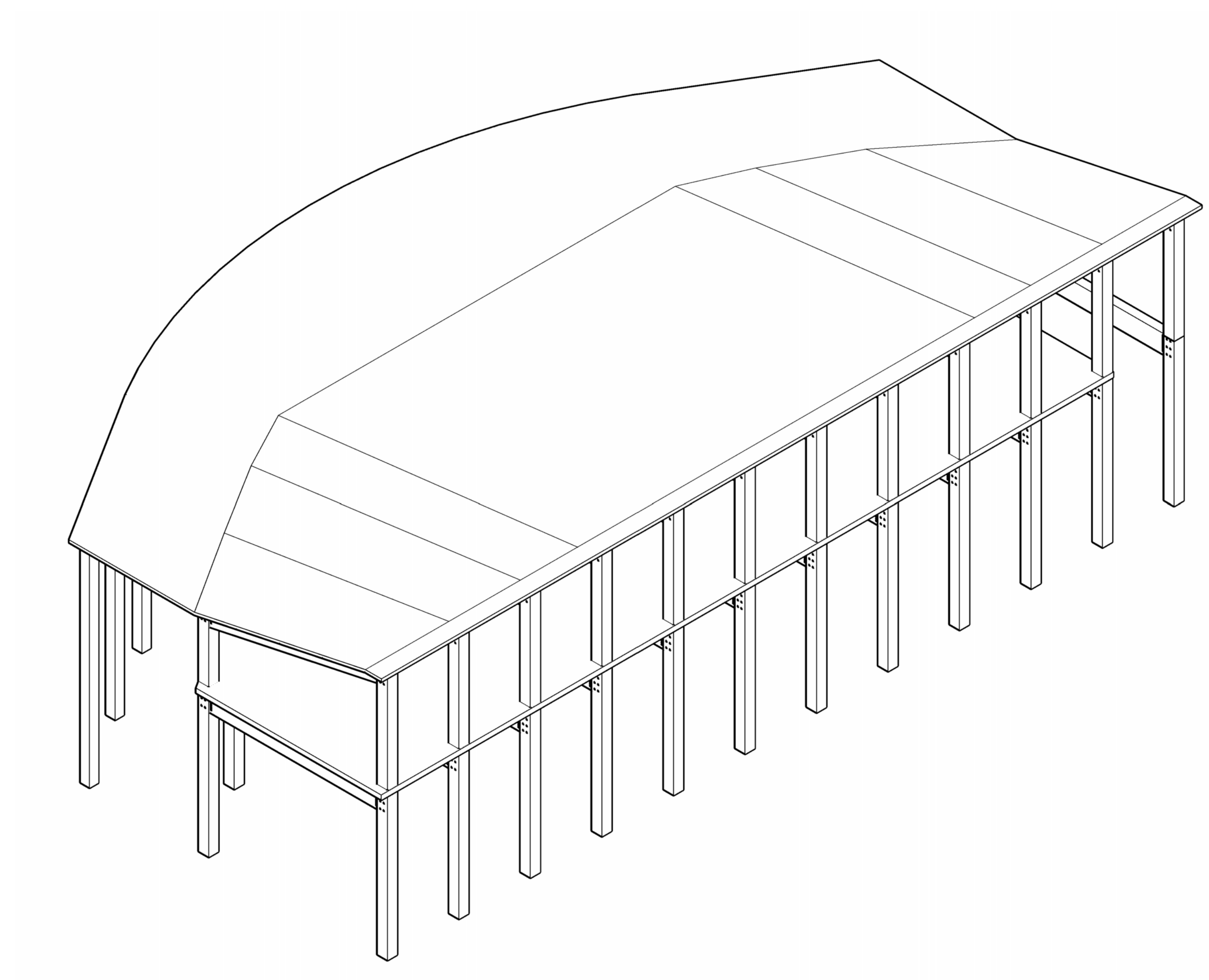
MARK	SIZE	REMARKS
B1	315mm x 418mm 315mm x 608mm	SPF, 20f-E Larger size req. only when connecting to B4
B2	315mm x 646mm	SPF, 20f-E
B3	315mm x 798mm 365mm x 798mm	SPF, 20f-E Larger size req. only when connecting to B4
B4	315mm x 798mm	SPF, 20f-E
B5	315mm x 760mm	SPF, 20f-E
B6	365mm x 950mm	SPF, 20f-E
B7	365mm x 950mm	SPF, 20f-E

GLULAM COLUMN SCHEDULE

MARK	SIZE	REMARKS
C1	365mm x 418mm	SPF, 12c-E, MAX HEIGHT 9000mm
C2	400mm x 418mm	SPF, 20f-EX, MAX HEIGHT 6000mm
C3	400mm x 418mm	SPF, 20f-EX, MAX HEIGHT 6000mm
C4	490mm x 532mm	SPF, 20f-EX, MAX HEIGHT 6000mm

CLT PANEL SCHEDULE

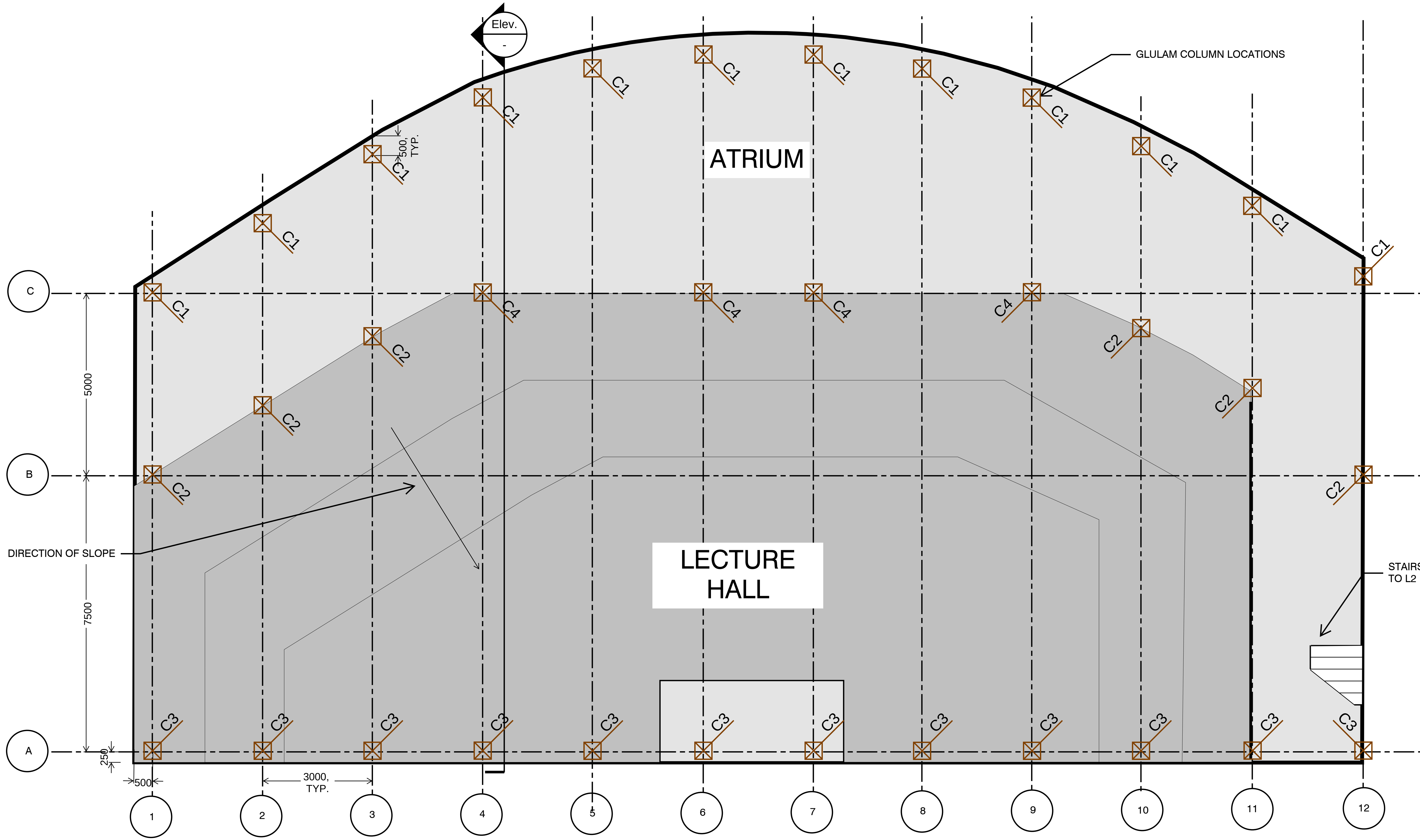
MARK	SIZE	REMARKS
P1	-	V2, 105mm THK
P2	-	E1, 175mm THK



Alex Thormeyer

athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40
	Drawn: A.T.	Ref. Doc.:
Title: COVER SHEET	Checked: -	Dwg. No.: 000
	Date: 04/11/2024	
Ref. Dwg.:		

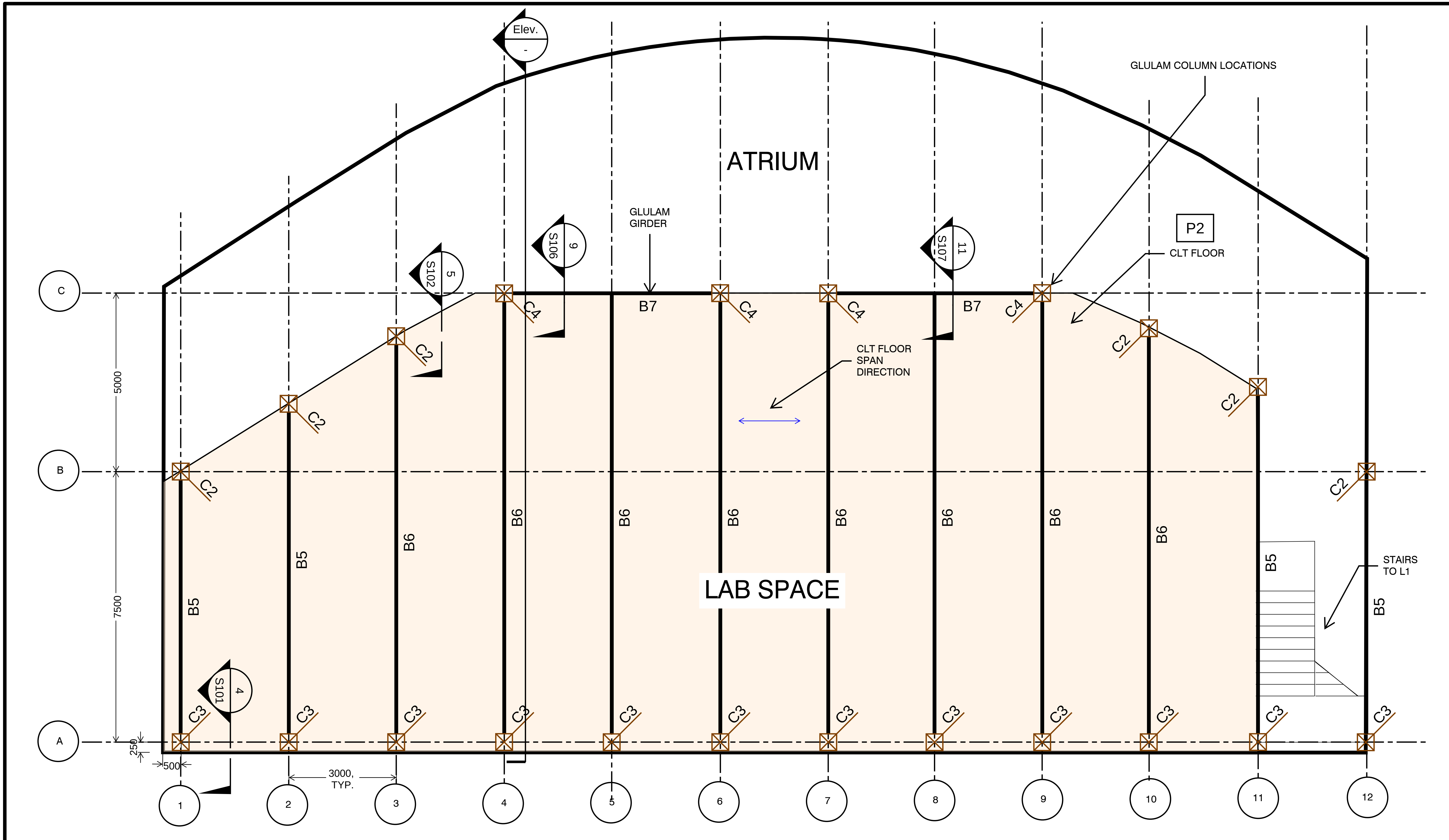


Fast + Epp

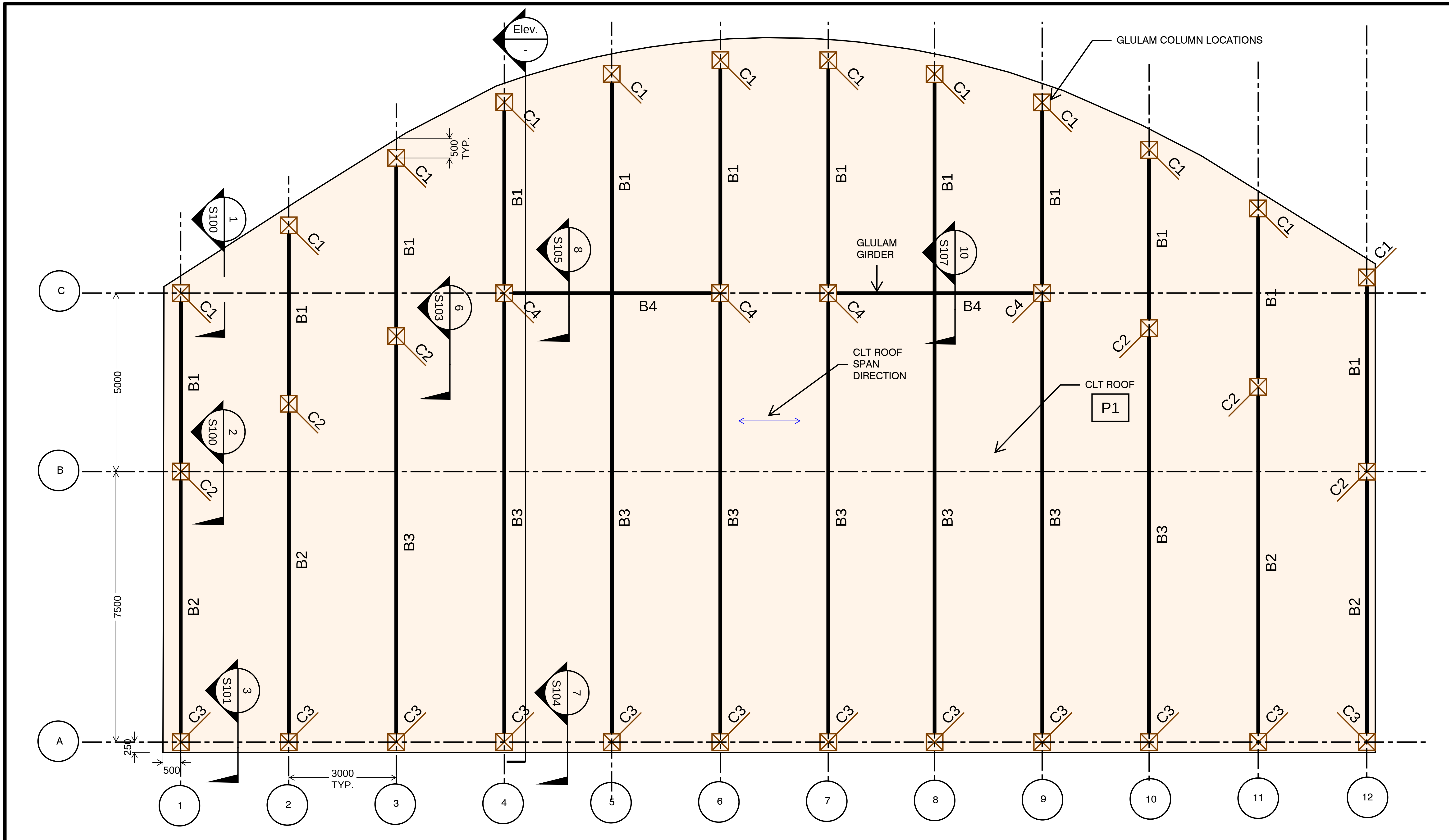
Suite 201
1672 West 1st Ave
Vancouver BC
Canada V6J 1G1

T 604 731 7412
F 604 731 7620
mail@fastepp.com

Project:	BCIT STUDENT PROJECT
Title:	GROUND FLOOR PLAN
Ref. Dwg.:	
Scale:	1:100
Drawn:	CS / AT
Checked:	-
Date:	11/08/2023
Project No.:	N.A.
Ref. Doc.:	
Dwg. No.:	SSK - 001



Fast + Epp Suite 201 1672 West 1st Ave Vancouver BC Canada V6J 1G1	T 604 731 7412 F 604 731 7620 mail@fastepp.com	Project: BCIT STUDENT PROJECT	Scale: 1:100	Project No.: N.A.
			Drawn: CS / AT	Ref. Doc.:
		Title: LEVEL 2 PLAN	Checked: -	Dwg. No.: SSK - 002
			Date: 11/08/2023	
		Ref. Dwg.:		

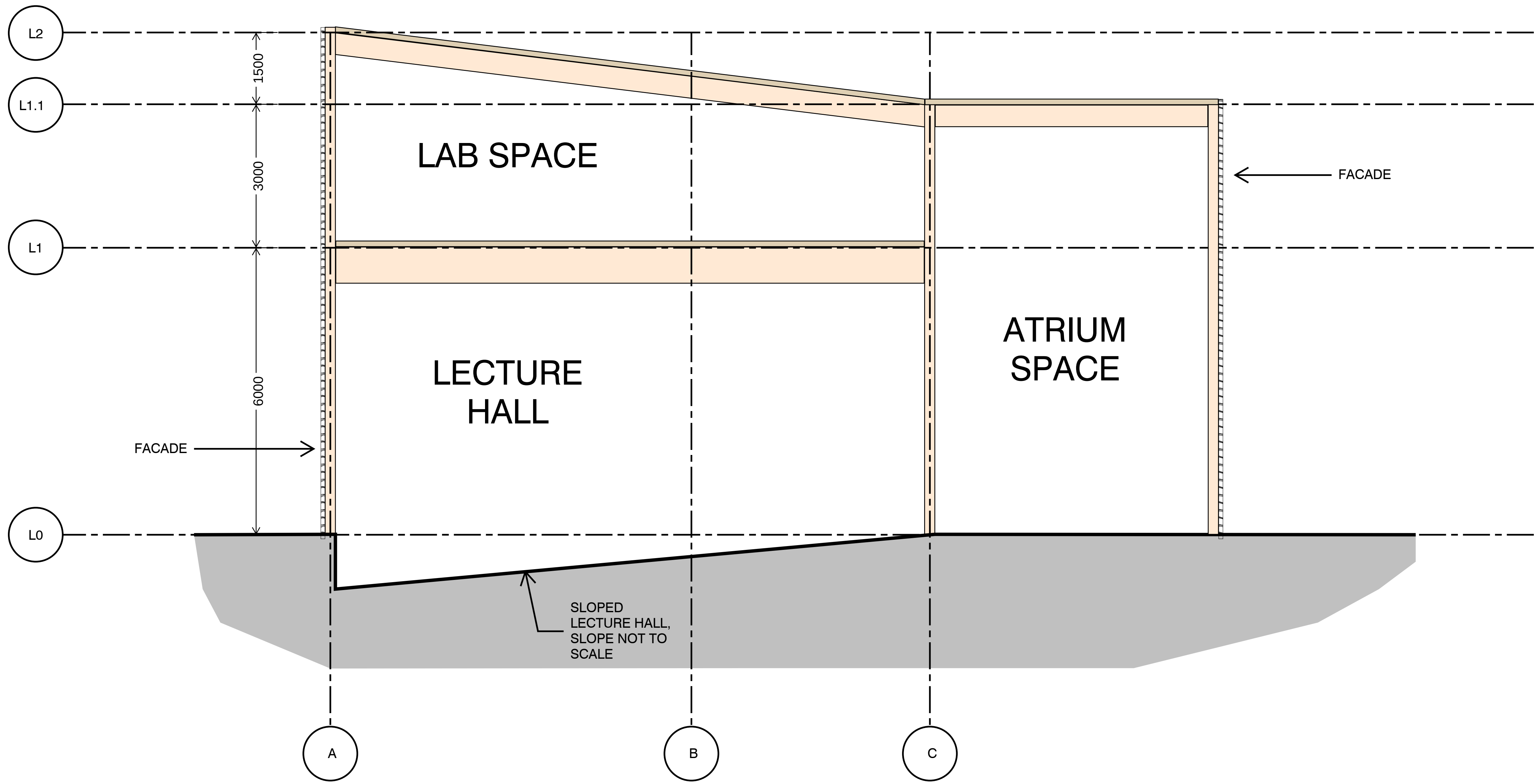


Fast + Epp

Suite 201
1672 West 1st Ave
Vancouver BC
Canada V6J 1G1

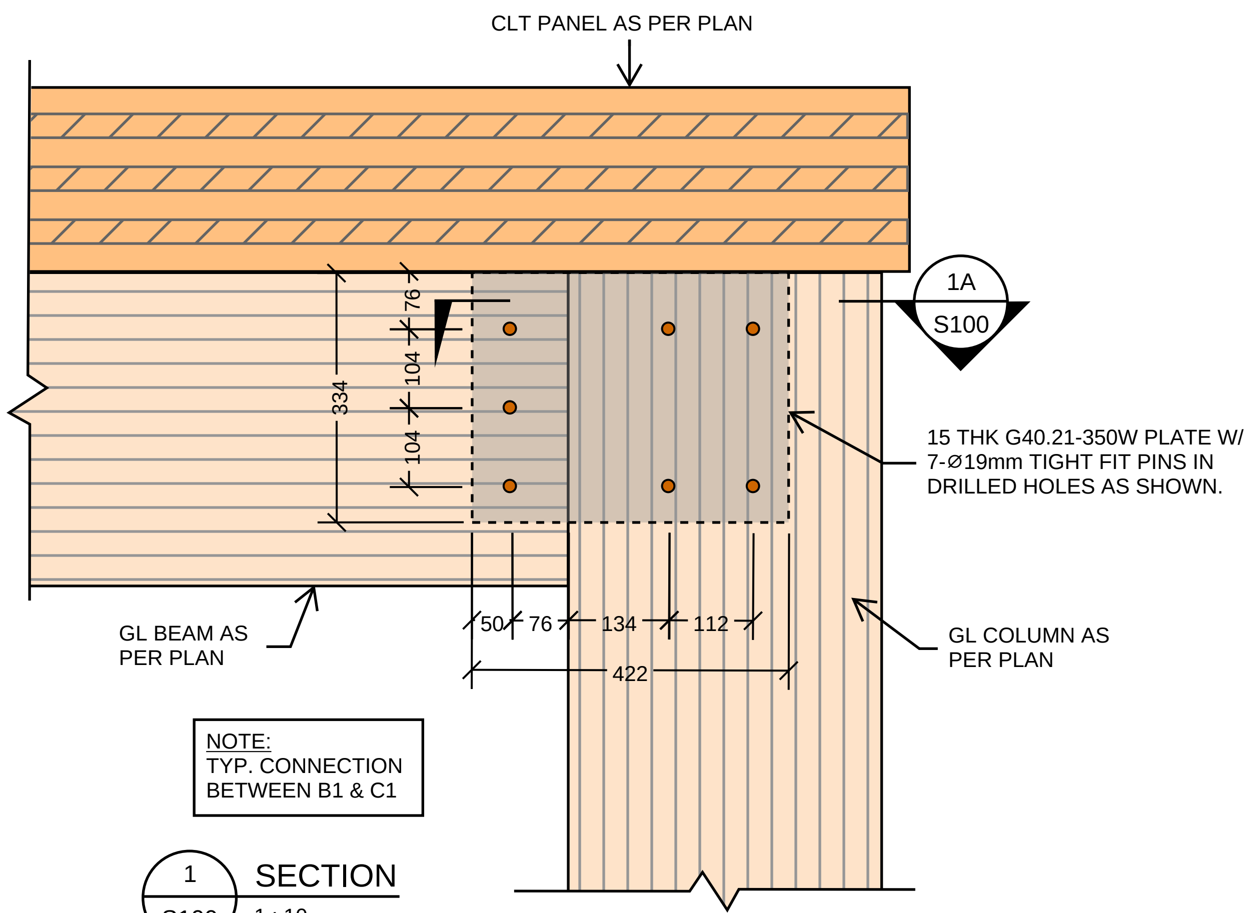
T 604 731 7412
F 604 731 7620
mail@fastepp.com

Project: BCIT STUDENT PROJECT	Scale: 1:100	Project No.: N.A.
	Drawn: CS / AT	Ref. Doc.:
Title: ROOF PLAN	Checked: -	Dwg. No.: SSK - 003
	Date: 11/08/2023	
Ref. Dwg.:		

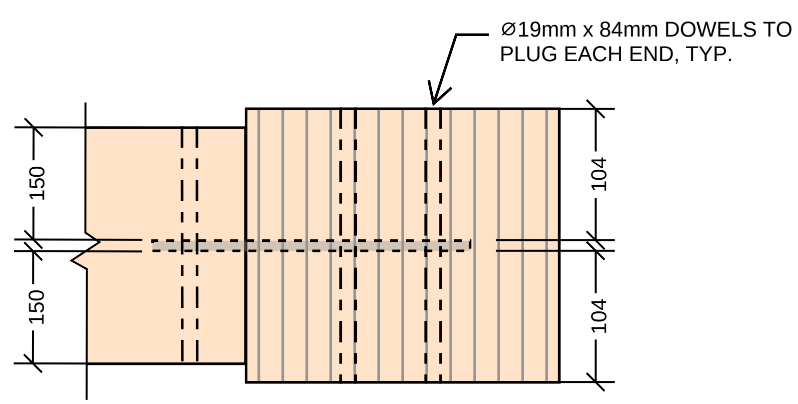


Fast + Epp
Suite 201
1672 West 1st Ave
Vancouver BC
Canada V6J 1G1
T 604 731 7412
F 604 731 7620
mail@fastepp.com

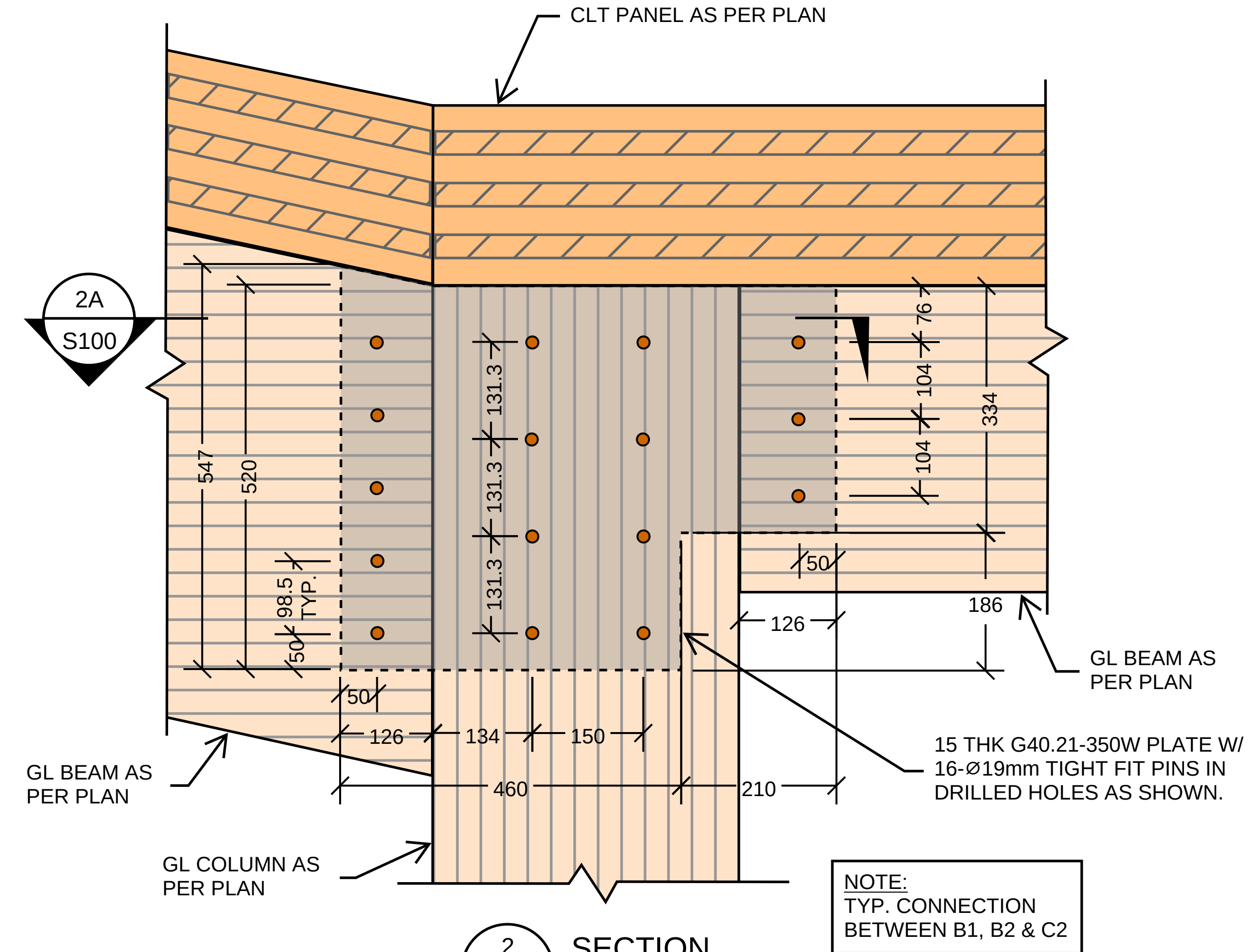
Project: BCIT STUDENT PROJECT	Scale: 1:100	Project No.: N.A.
	Drawn: CS	Ref. Doc.:
Title: ELEVATION	Checked: -	Dwg. No.: SSK - 004
	Date: 11/08/2023	
Ref. Dwg.:		



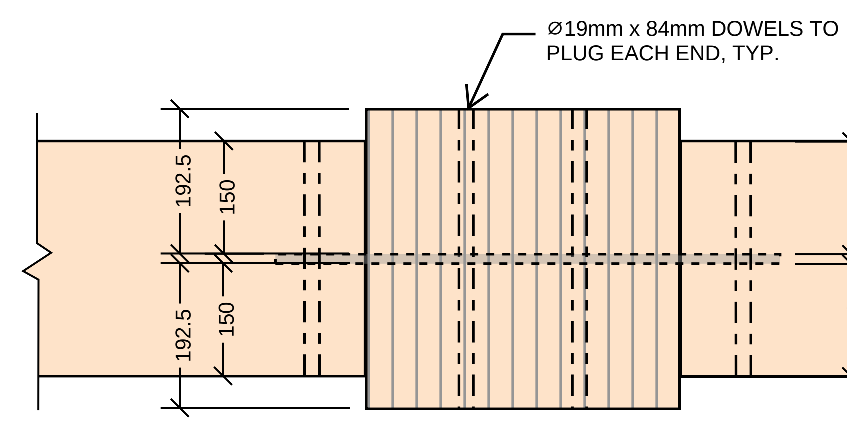
1 S100 SECTION 1 : 10



1A S100 SECTION 1 : 10



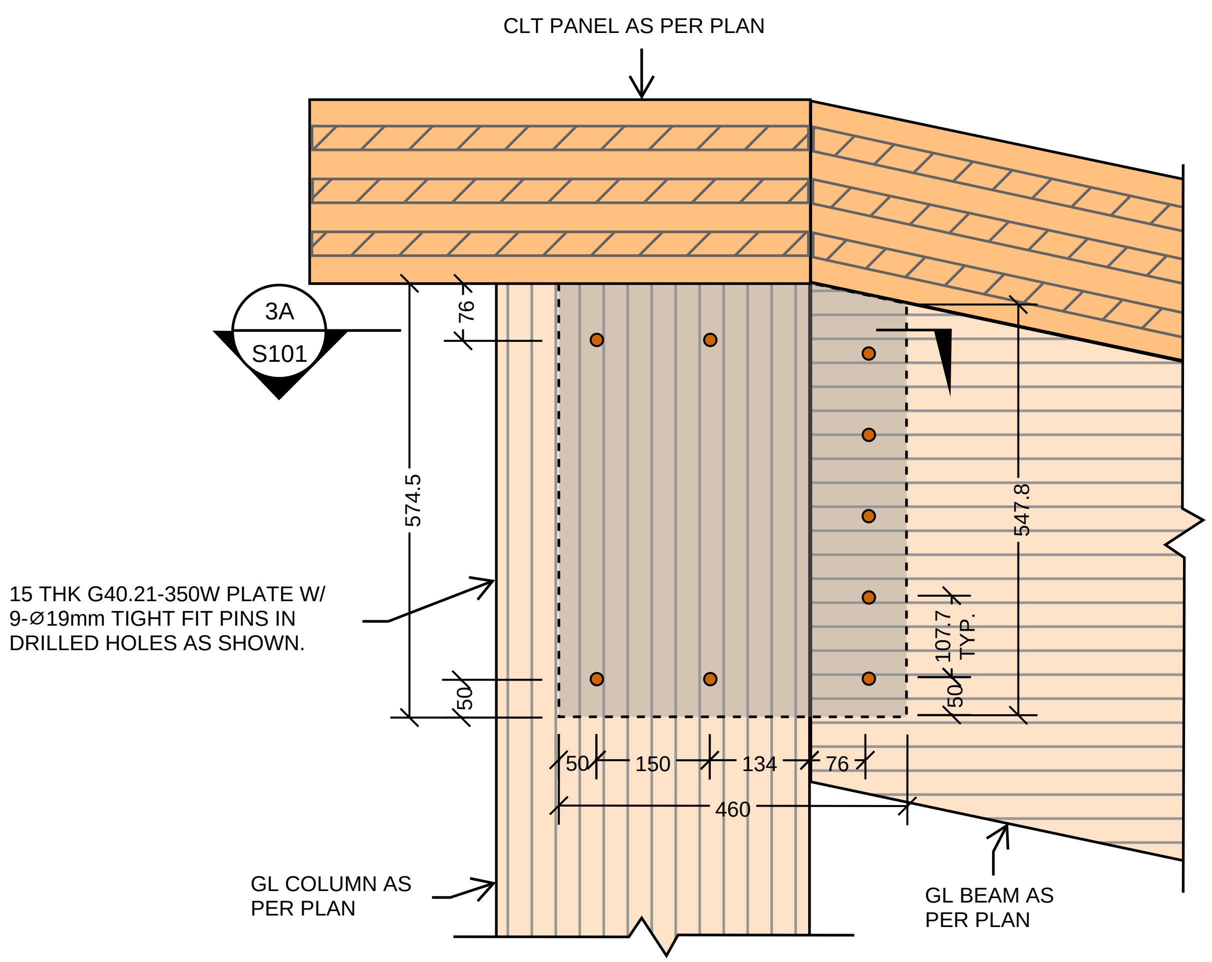
2 S100 SECTION 1 : 10



2A S100 SECTION 1 : 10

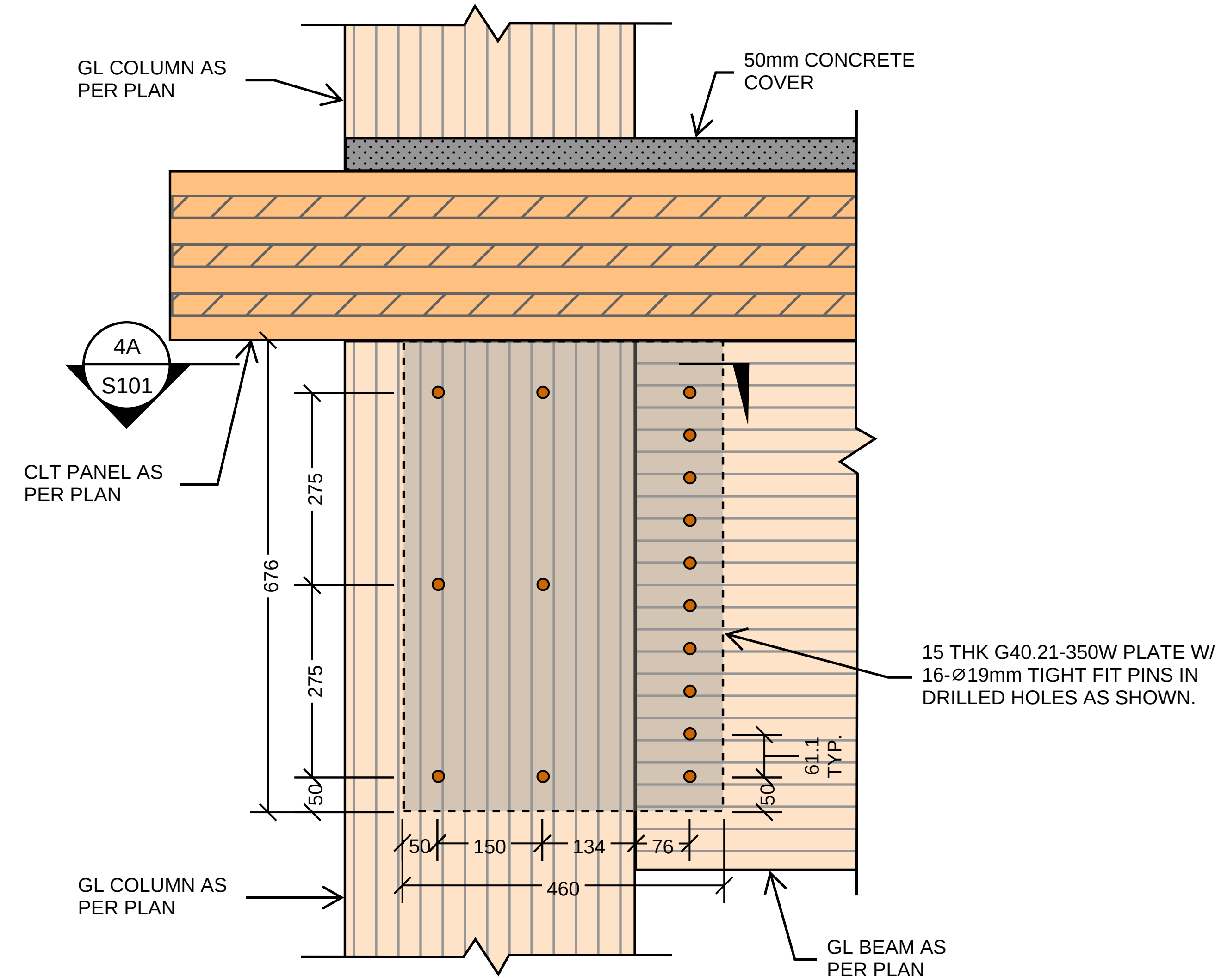
Alex Thormeyer
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40	
	Drawn: A.T.	Ref. Doc.:	
Title: CONNECTION DETAIL	Checked: -	Dwg. No.: S100	
	Date: 04/11/2024		
Ref. Dwg.:			



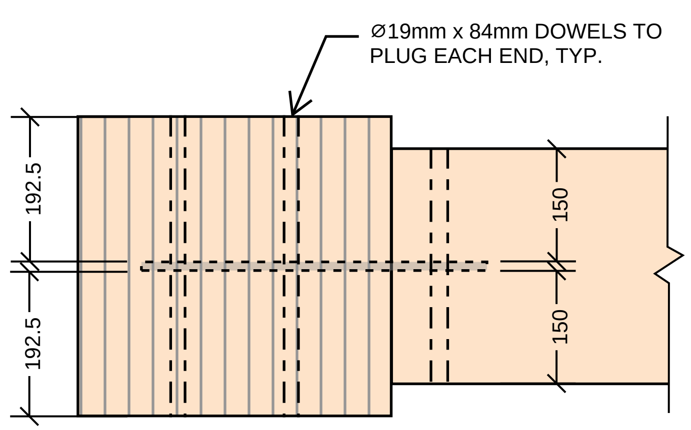
3 S101 SECTION
1 : 10

NOTE:
TYP. CONNECTION
BETWEEN B2 & C3

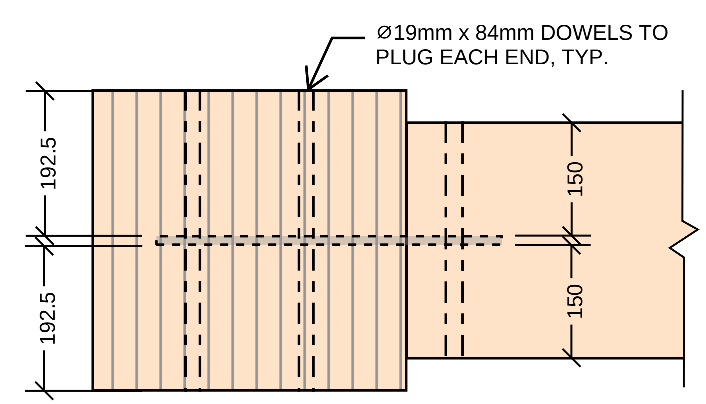


4 S101 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN
B5 & C3, AND B5 & C2



3A S101 SECTION
1 : 10

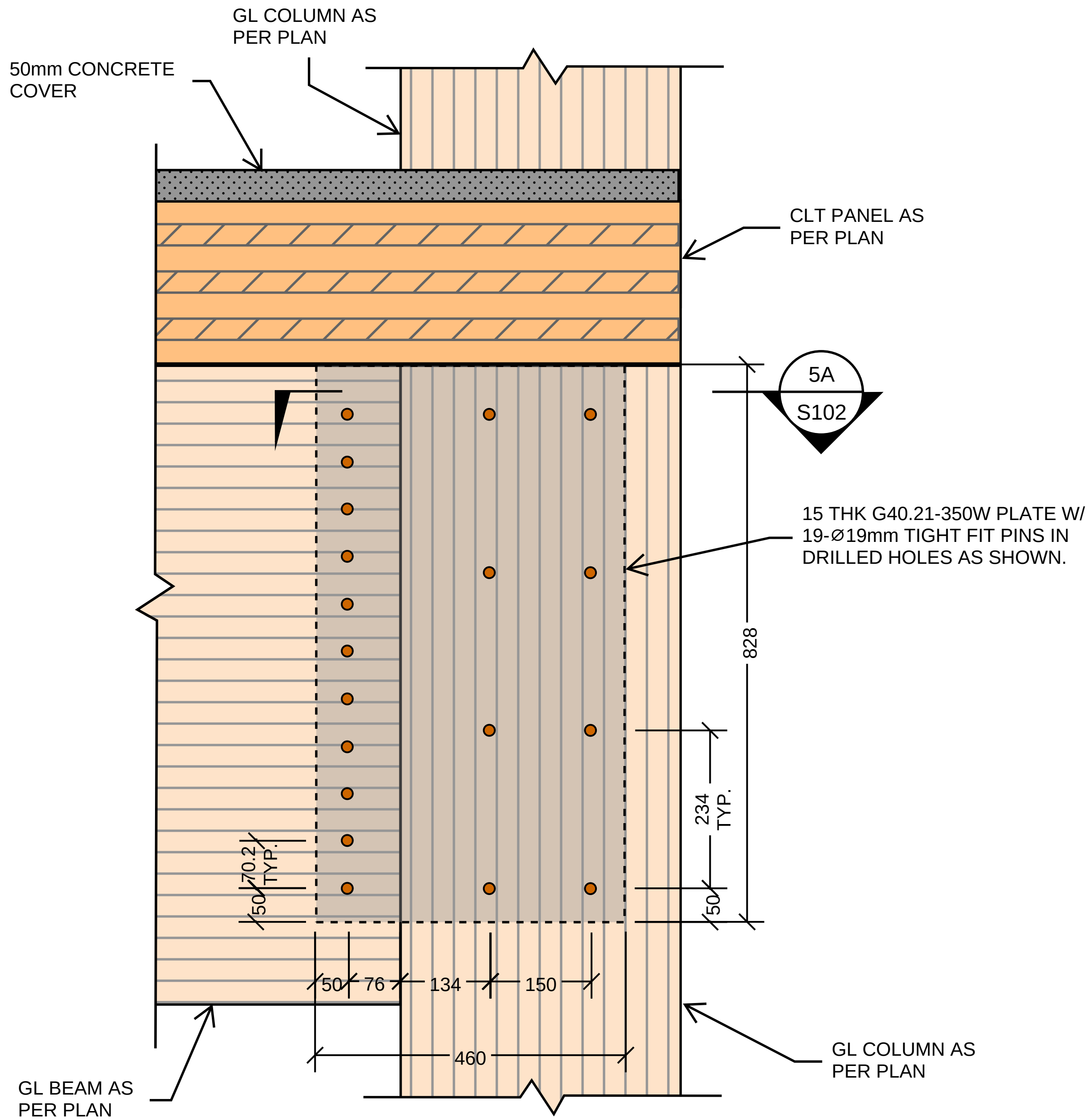


4A S101 SECTION
1 : 10

Alex Thormeyer

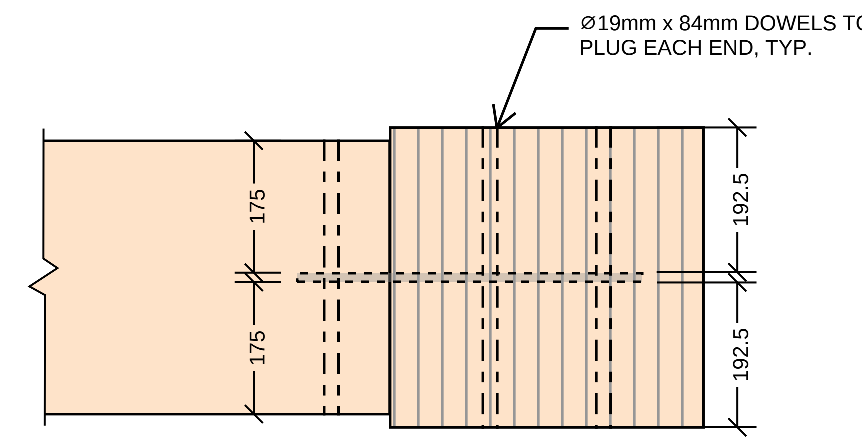
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40
	Drawn: A.T.	Ref. Doc.:
Title: CONNECTION DETAIL	Checked: -	Dwg. No.: S101
	Date: 04/11/2024	
Ref. Dwg.:		



5 SECTION
S102 1 : 10

NOTE:
TYP. CONNECTION BETWEEN
B6 & C3 AND B6 & C2



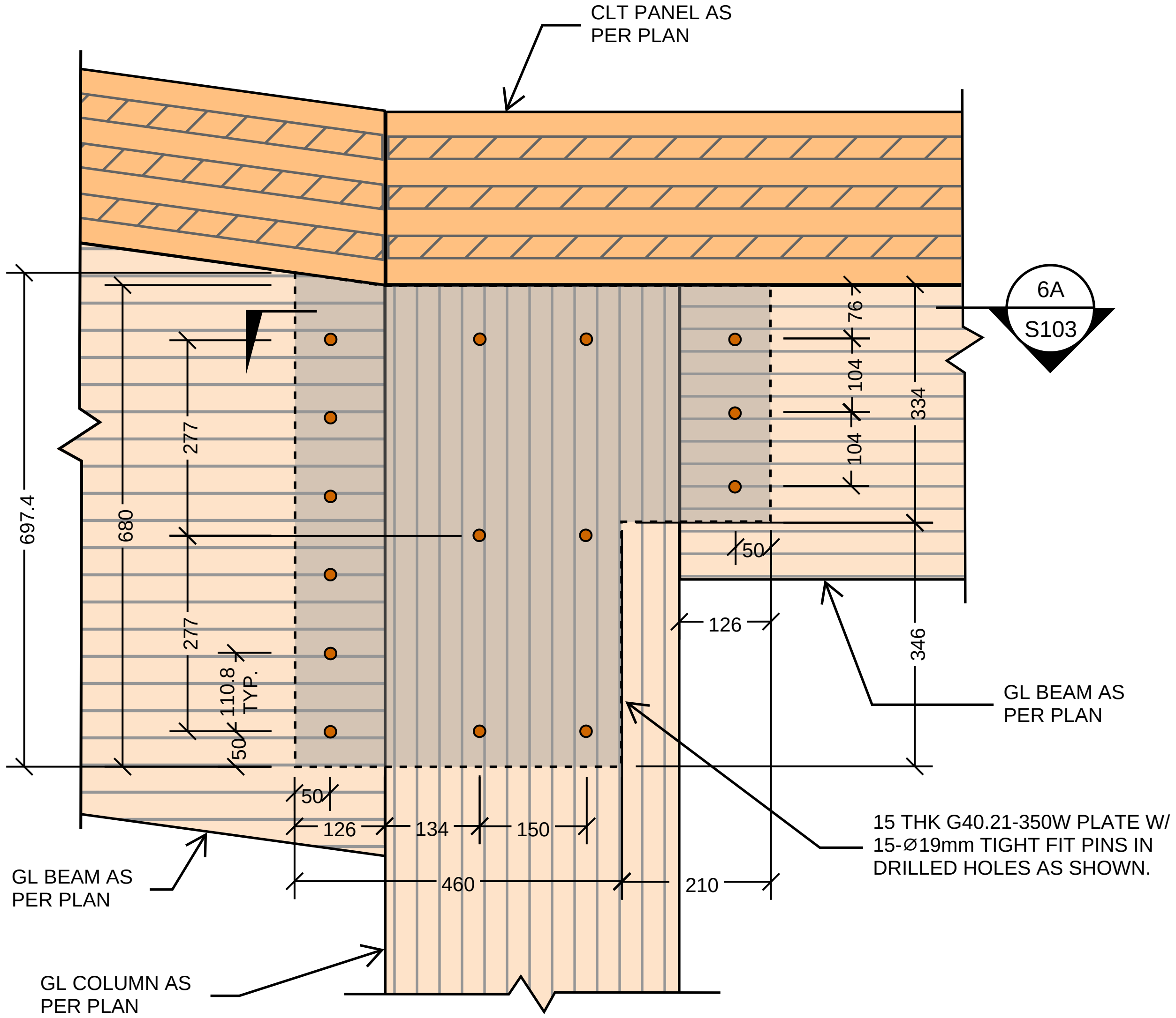
5A SECTION
S102 1 : 10

Alex Thormeyer
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT
Title: CONNECTION DETAIL
Ref. Dwg.:

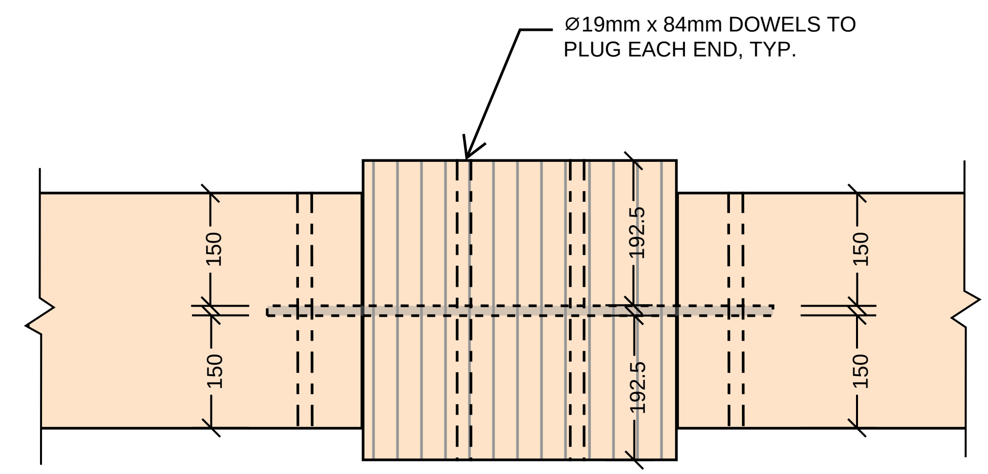
Scale: 1:10
Drawn: A.T.
Checked: -
Date: 04/11/2024

Project No.: 24-40
Ref. Doc.:
Dwg. No.: S102



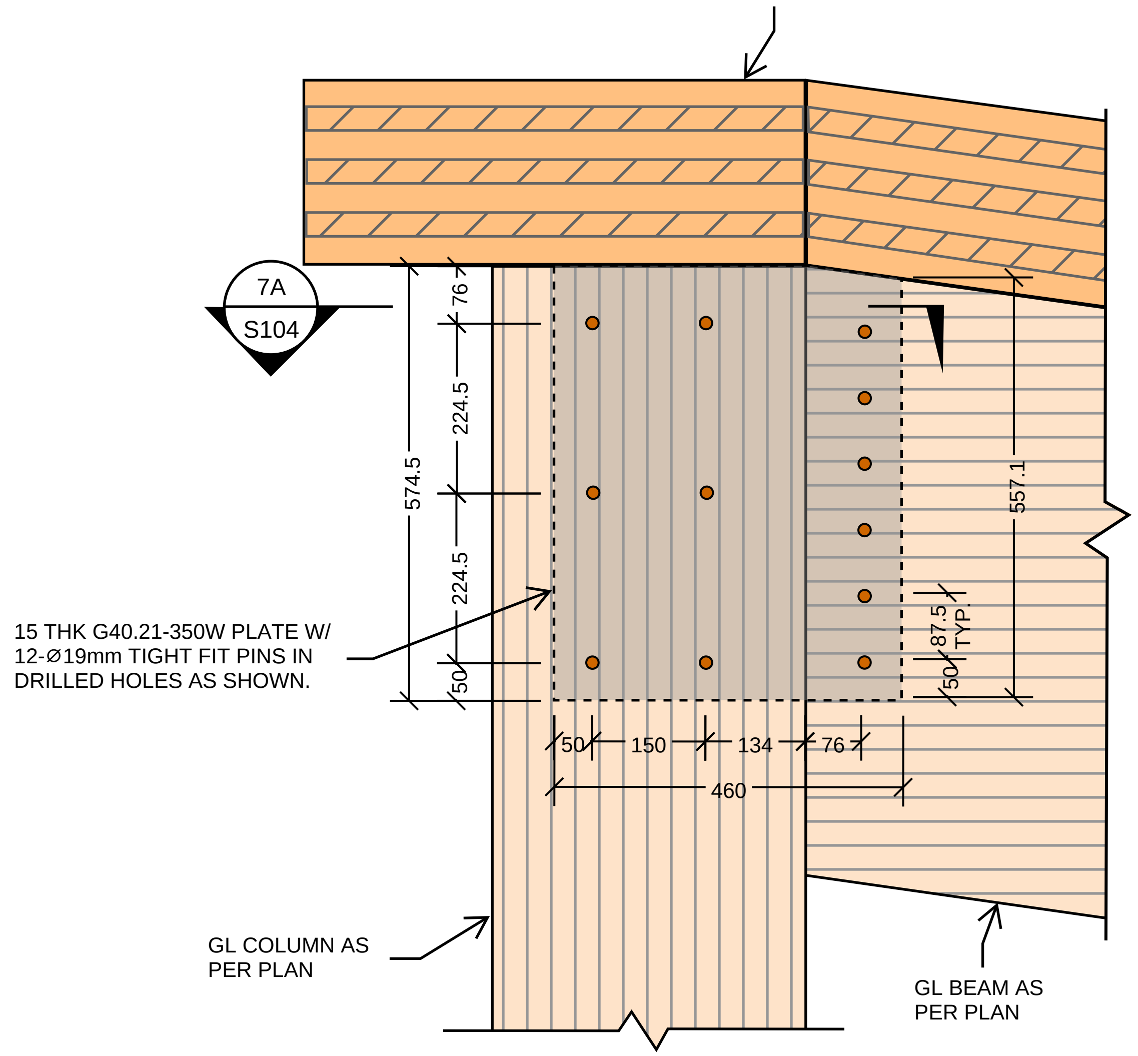
6 / S103 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN
B1, B3 & C2



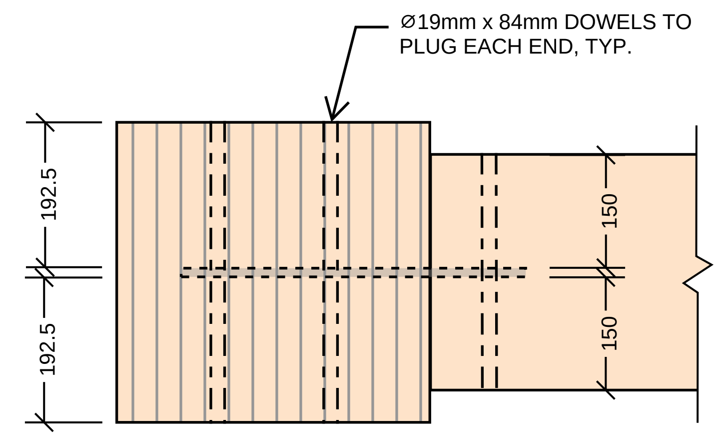
6A / S103 SECTION
1 : 10

Alex Thormeyer athormeyer@gmail.com 778-887-8618 linkedin.com/in/alex-thormeyer	Project:	BCIT STUDENT PROJECT	Scale: 1:10 Drawn: A.T.	Project No.: 24-40	Ref. Doc.:
	Title:	CONNECTION DETAIL	Checked: - Date: 04/11/2024	Dwg. No.: S103	
	Ref. Dwg.:				



7 S104 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN
B3 & C3

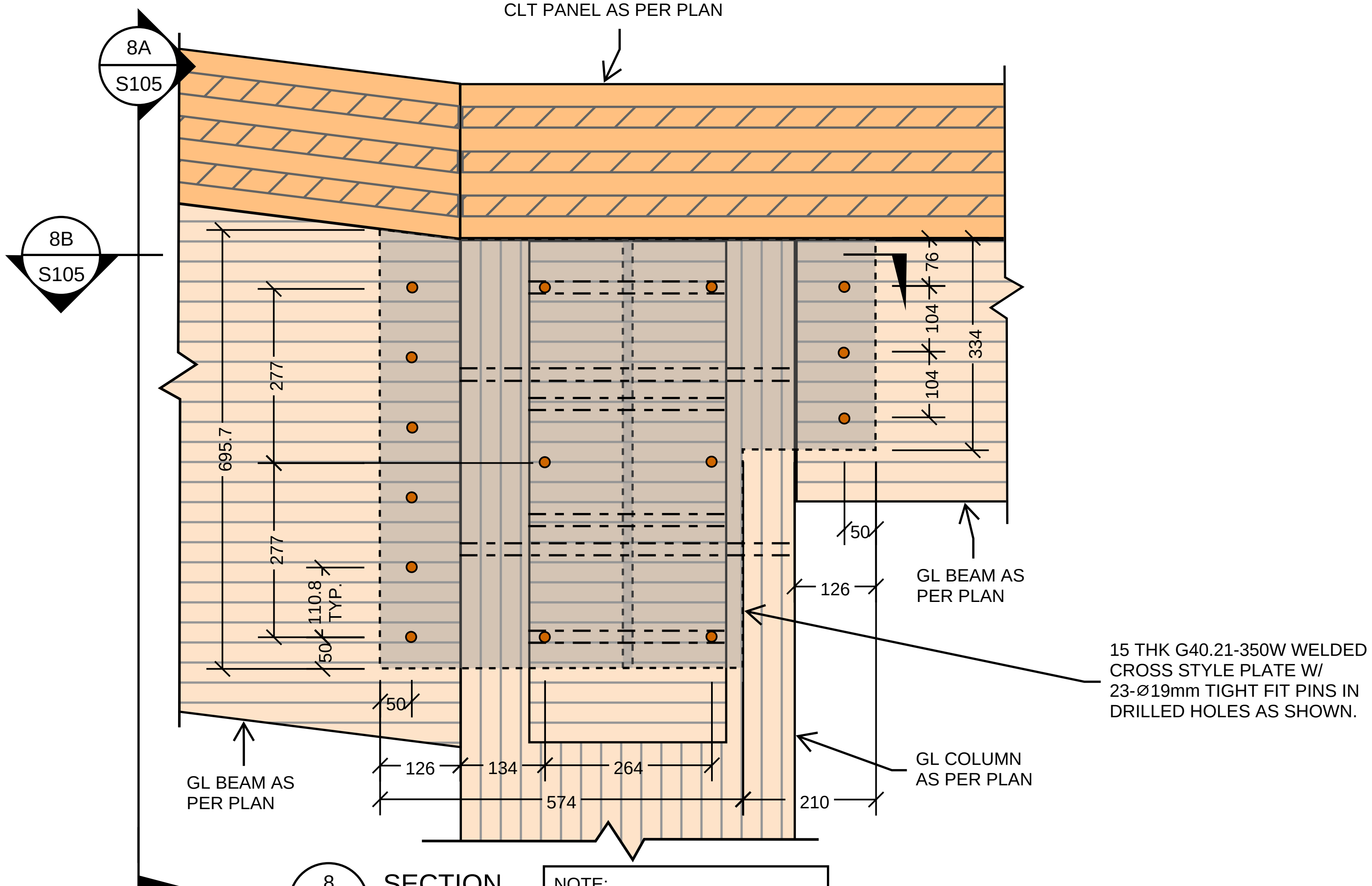


7A S104 SECTION
1 : 10

Alex Thormeyer

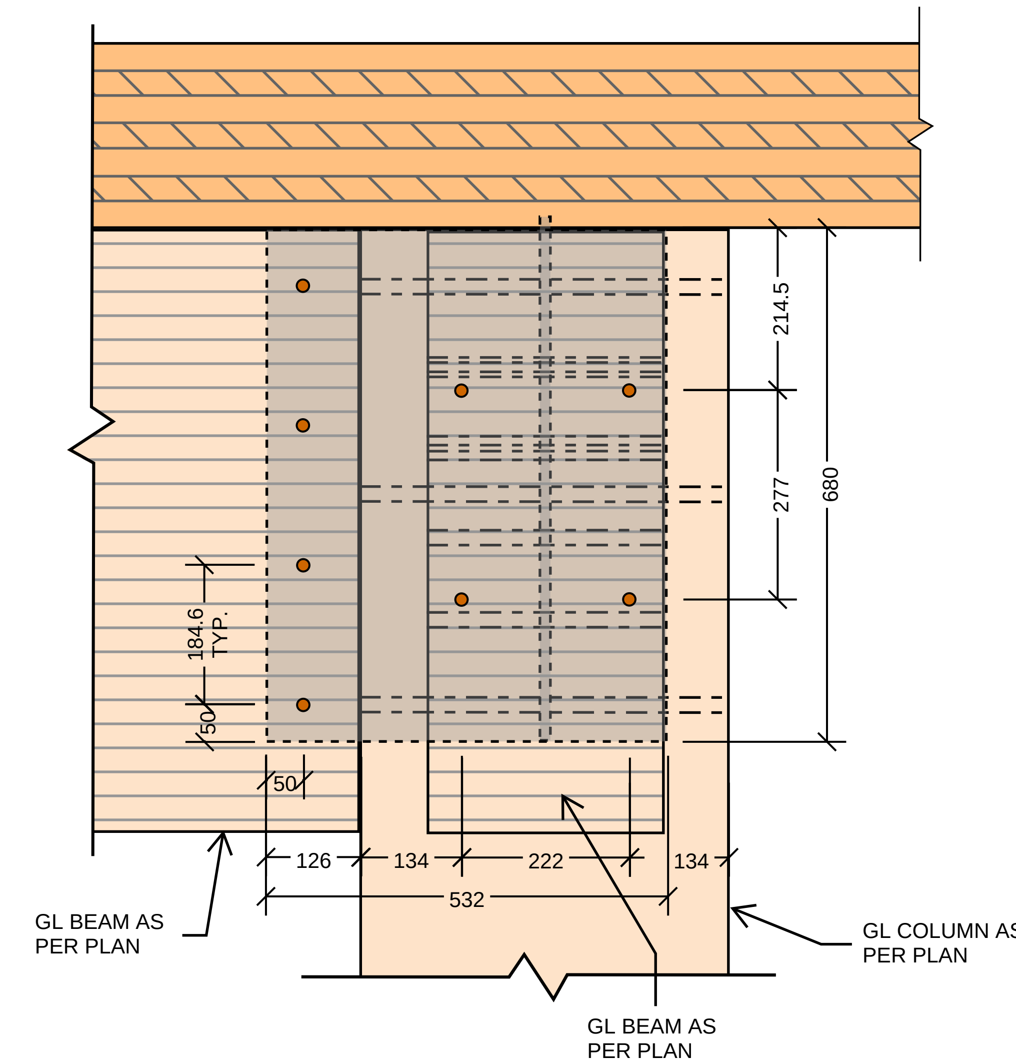
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40
	Drawn: A.T.	Ref. Doc.:
Title: CONNECTION DETAIL	Checked: -	Dwg. No.: S104
	Date: 04/11/2024	
Ref. Dwg.:		

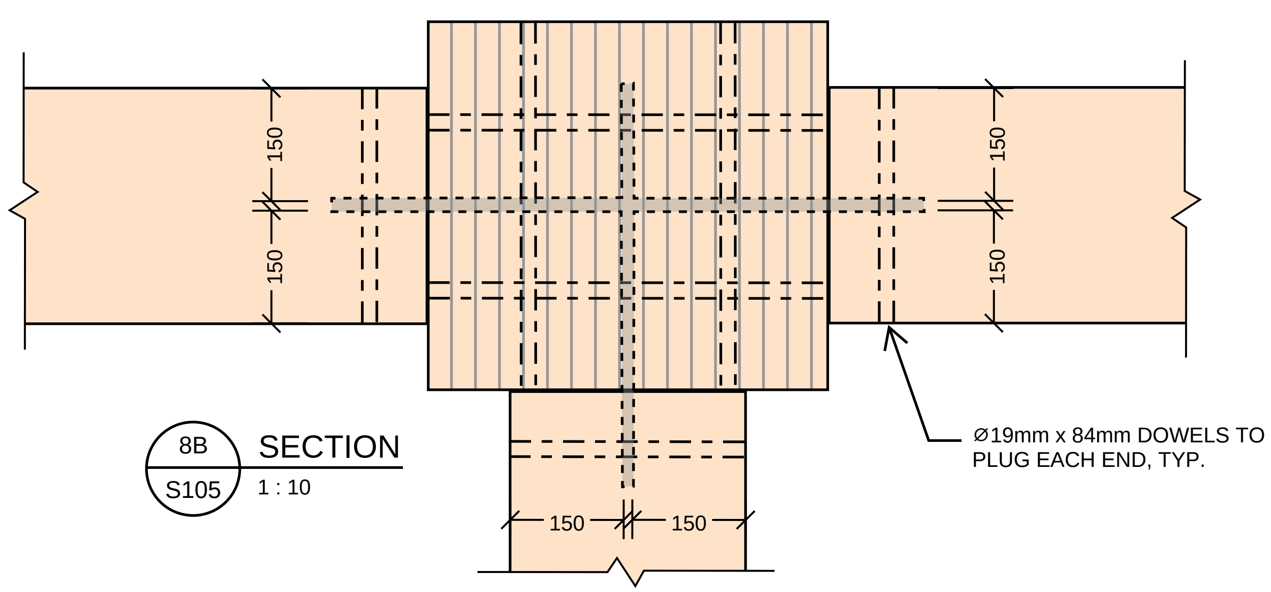


8 S105 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN B1, B3, B4 & C4



8A S105 SECTION
1 : 10

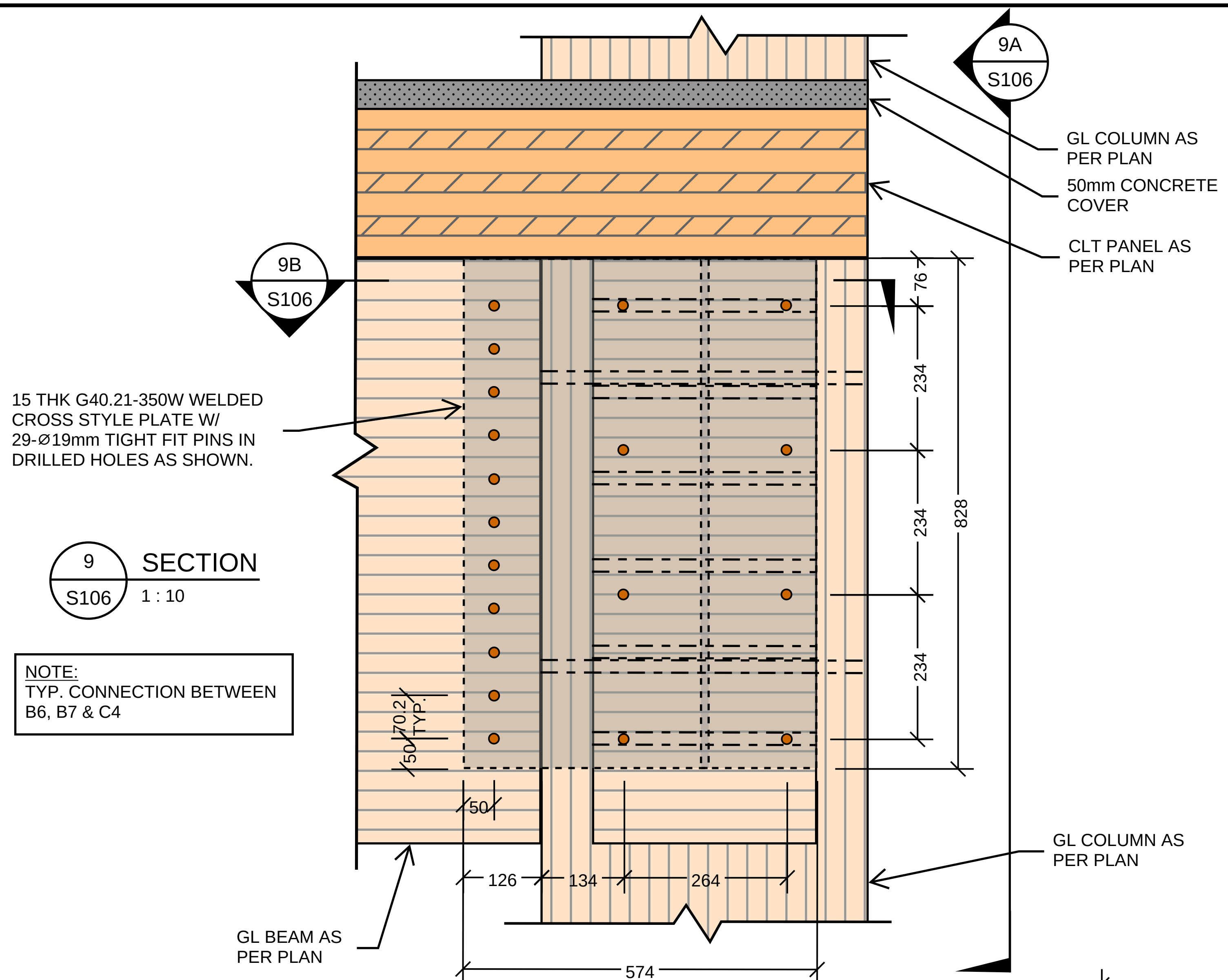


8B S105 SECTION
1 : 10

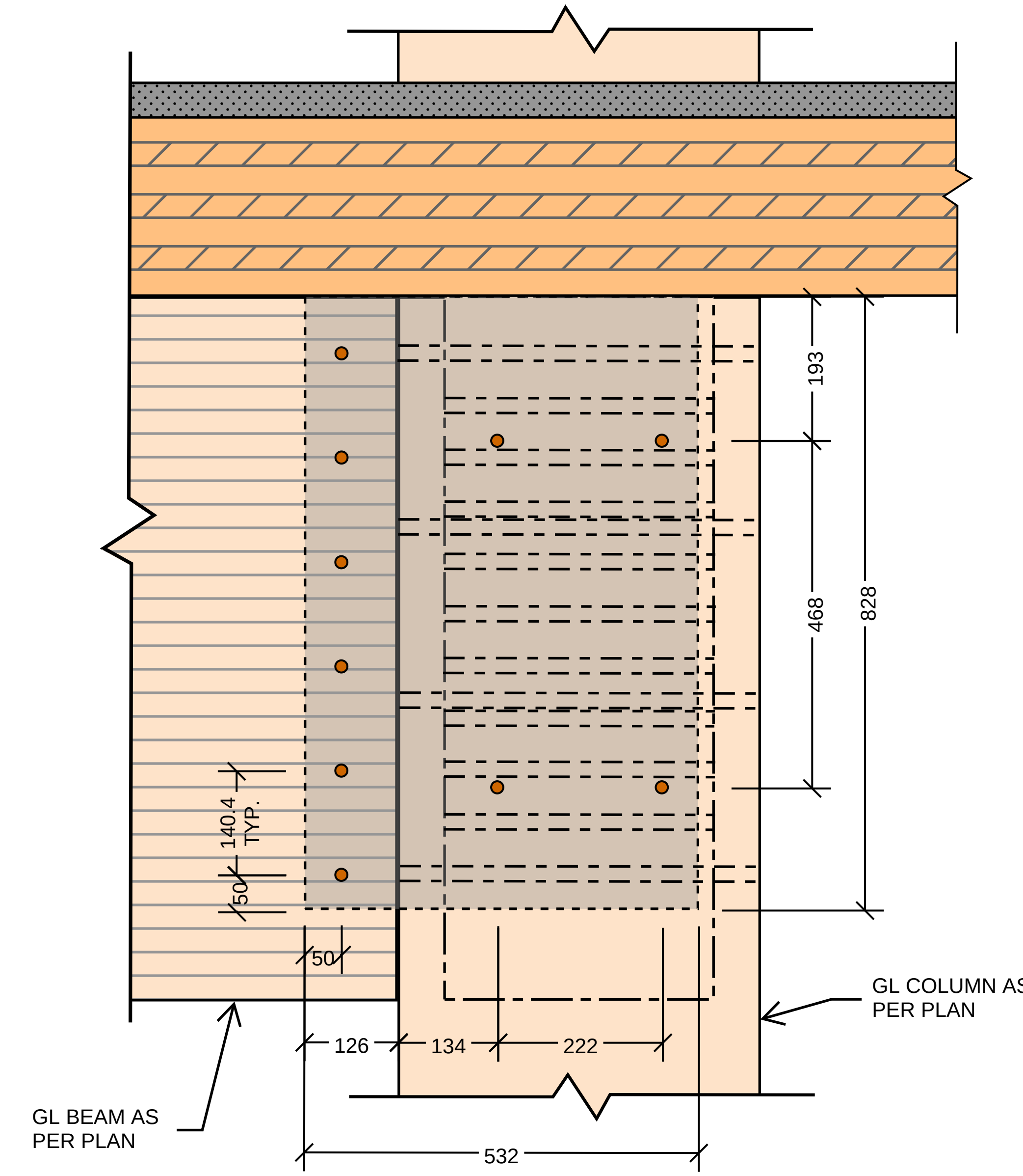
Alex Thormeyer

athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

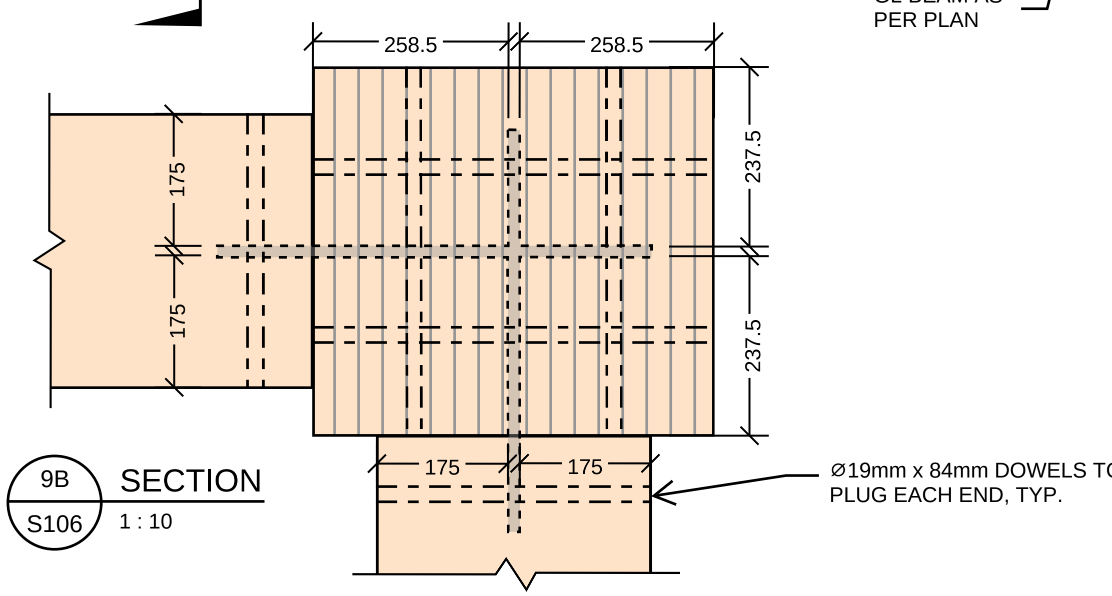
Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40
	Drawn: A.T.	Ref. Doc.:
Title: CONNECTION DETAIL	Checked: -	Dwg. No.: S105
	Date: 04/11/2024	
Ref. Dwg.:		



9 SECTION
S106 1 : 10



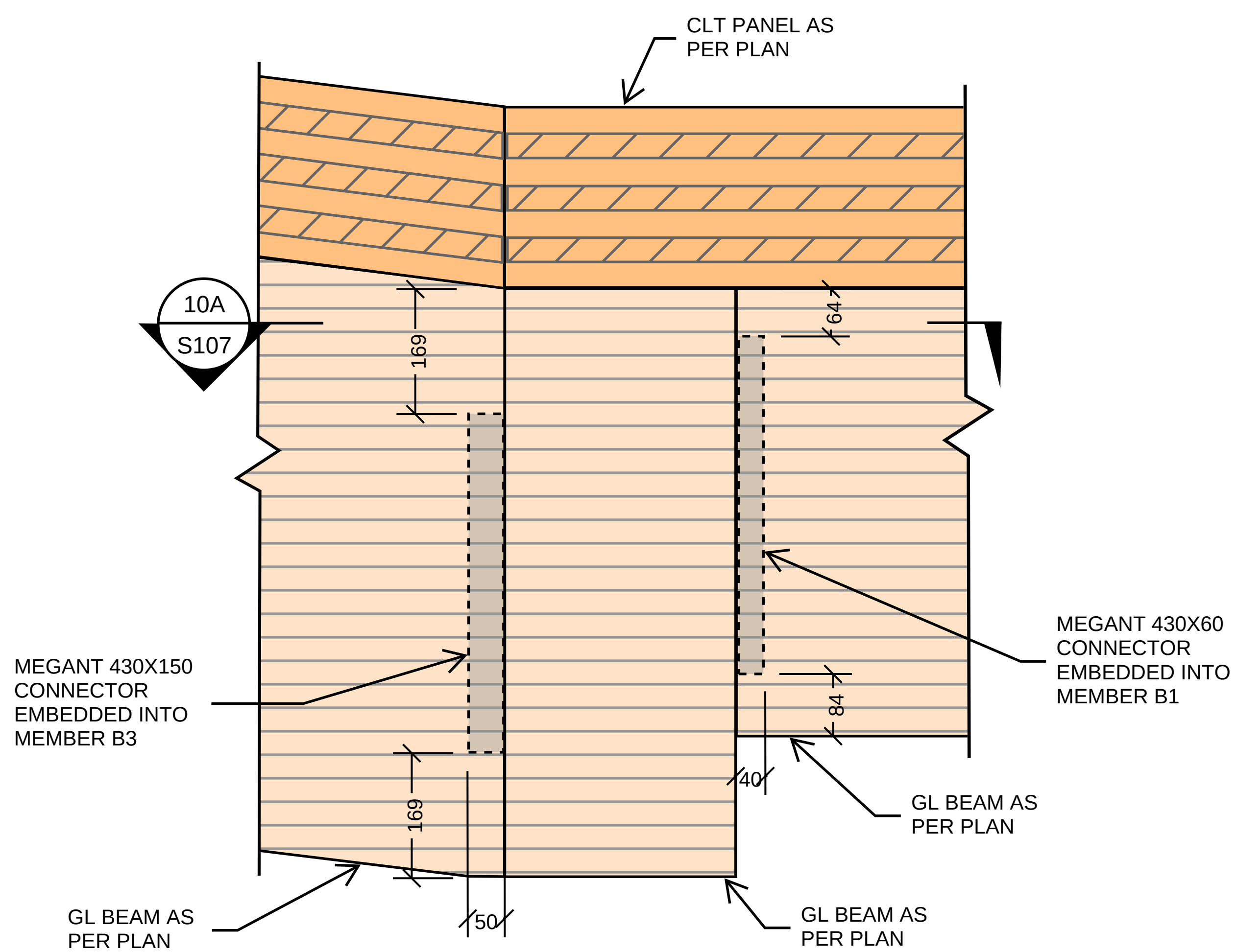
9A SECTION
S106 1 : 10



9B SECTION
S106 1 : 10

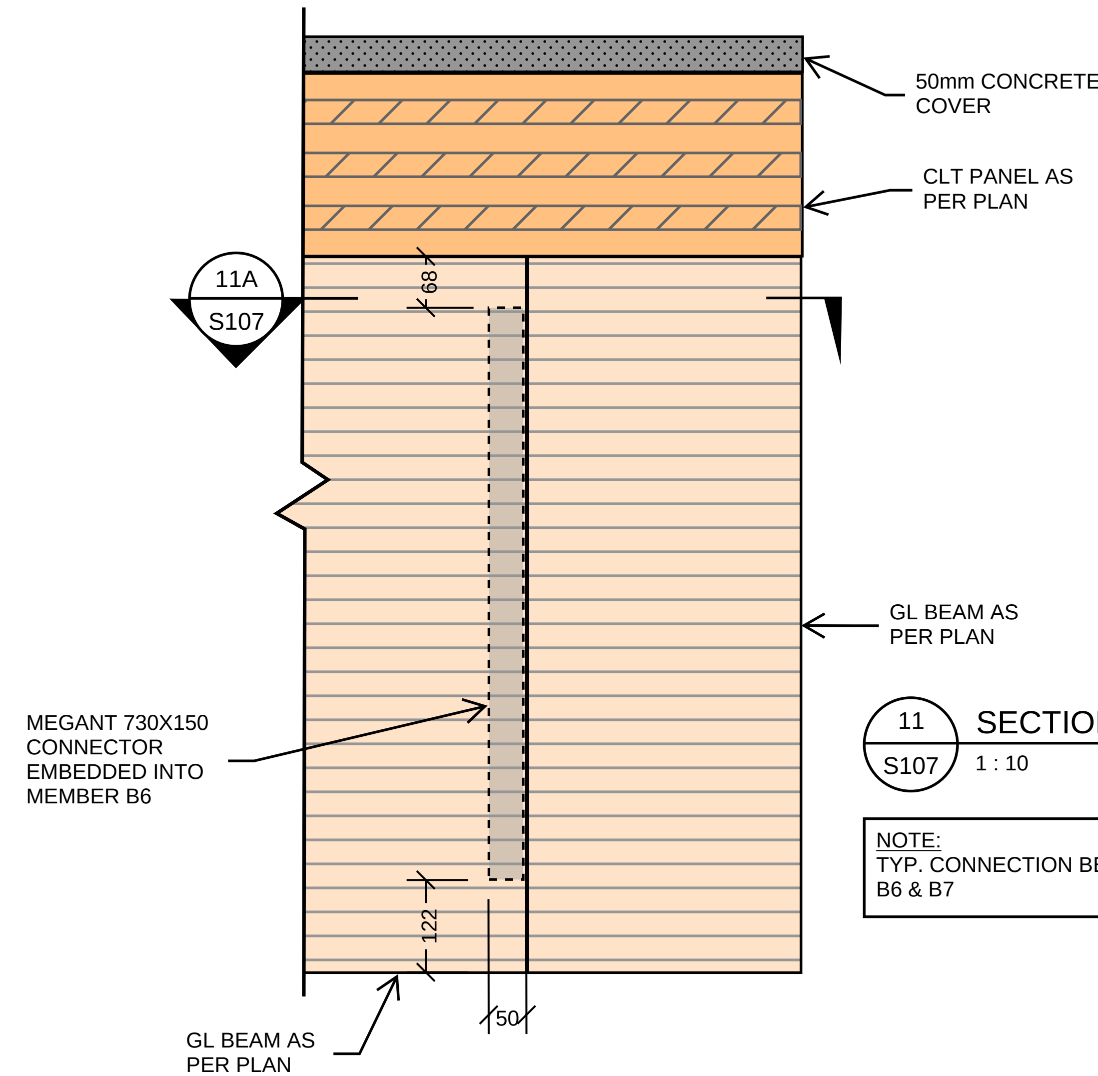
Alex Thormeyer
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Project No.: 24-40
	Drawn: A.T.	Ref. Doc.:
Title: CONNECTION DETAIL	Checked: -	Dwg. No.: S106
	Date: 04/11/2024	
Ref. Dwg.:		



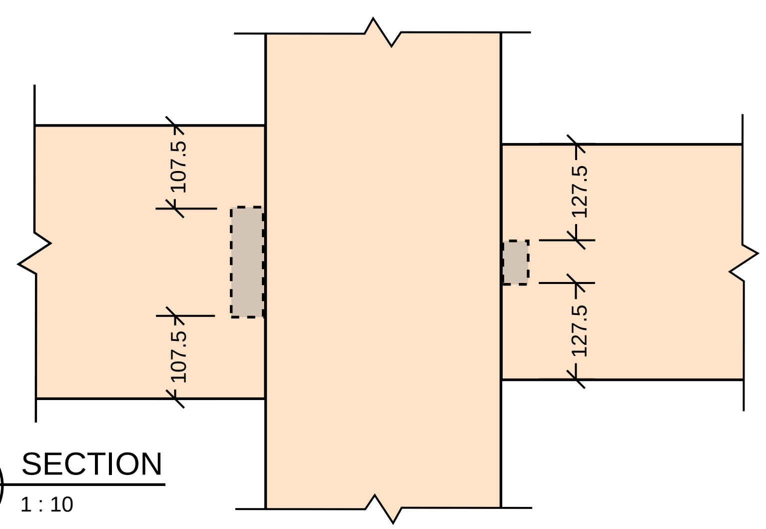
10 / S107 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN B1, B3 & B4

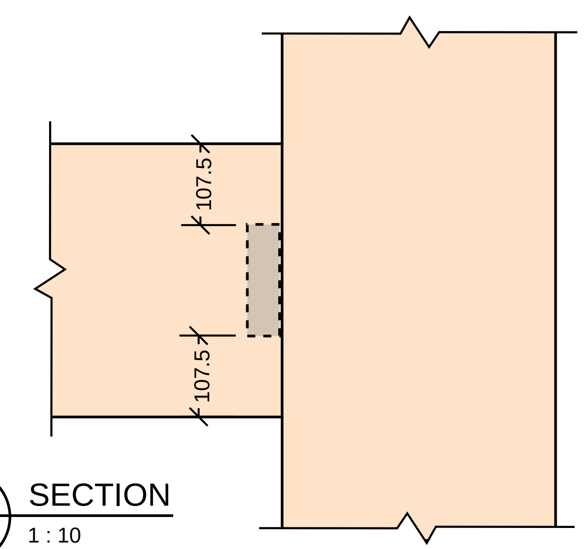


11 / S107 SECTION
1 : 10

NOTE:
TYP. CONNECTION BETWEEN B6 & B7



10A / S107 SECTION
1 : 10



11A / S107 SECTION
1 : 10

Alex Thormeyer
athormeyer@gmail.com
778-887-8618
linkedin.com/in/alex-thormeyer

Project: BCIT STUDENT PROJECT	Scale: 1:10	Drawn: A.T.	Project No.: 24-40
Title: CONNECTION DETAIL	Checked: -	Date: 04/11/2024	Ref. Doc.:
Ref. Dwg.:			Dwg. No.: S107

APPENDIX C: CALCULATIONS PACKAGE

Table of Contents:

Summary Sheet

Prepared By: Alex Thormeyer
Industry Sponsor: Christian Slotboom, EIT, MASc, Fast+Epp
Ryan Taylor, PEng, MIStructE, Fast+Epp
Project Location: Mt. Gelmer region of Victoria, BC
Ref. #: 24-40
Date: 2024-04-12

Mark	Species	Grade	Member Size (mm)	Description	Remarks
B1	SPF	20f-E	315x418	Roof beam	Use larger beam size 315x608 when connecting to B4 per Megant req.
B2	SPF	20f-E	315x646	Roof beam	-
B3	SPF	20f-E	315x798	Roof beam	Use larger beams size 365x798 when connecting to B4 per Megant req.
B4	SPF	20f-E	315x798	Roof girder	-
B5	SPF	20f-E	315x760	2nd level beam	-
B6	SPF	20f-E	365x950	2nd level beam	-
B7	SPF	20f-E	365x950	2nd level girder	-
C1	SPF	12c-E	365x418	Exterior column	-
C2	SPF	20f-EX	400x418	Interior column	Lower level column to be used at upper level
C3	SPF	20f-EX	400x418	Exterior column	Lower level column to be used at upper level
C4	SPF	20f-EX	490x532	Interior column	Lower level column to be used at upper level
P1	SPF	V2	105 THK	Roof panel	-
P2	SPF	E1	175 THK	2nd level panel	-

Design Loads

Notes

No wind load or negative values, no need to check 0.9 multiplier

NBCC Clause 4.1.5.5.2, do not include live and snow loads at the same time

Importance Factors

Ultimate	1.0
Serviceability	0.9

Roof Level

Dead Loads		
Assumed dead load from member self-weight	1.00	kPa
Superimposed roof dead load from sponsor	1.60	kPa
D, total dead load	2.60	kPa
Live Loads		
Per NBCC Clause 4.1.5.5.3	1.00	kPa
L, total live load	1.00	kPa
Snow Loads		
Ss, 1-in-50 snow load for Victoria, BC	2.10	kPa
Sr, 1-in-50 rain load for Victoria, BC	0.30	kPa
Cb, basic snow load factor	0.80	
Cw, wind exposure factor	1.00	
Cs, slope factor (steepest roof angle = 11deg)	1.00	
Ca, accumulated factor	1.00	
S, total snow load (ultimate)	1.98	kPa
S, total snow load (serviceability)	1.78	kPa
Wind Loads		
None for this project	0.00	kPa
W, total wind load	0.00	kPa

Ultimate Limit States

Case	Principal	Companion	Total
1	3.64	0.00	3.64
2	4.75	0.00	4.75
3	6.22	0.00	6.22
4a	3.25	0.50	3.75
4b	3.25	0.99	4.24

Serviceability Limit States

Case	Principal	Companion	Total
1	3.60	0.00	3.60
2a	2.60	0.35	2.95
2b	2.60	0.62	3.22
3	4.38	0	4.38

Second Level

Dead Loads		
Assumed dead load from member self-weight	1.00	kPa
Superimposed dead load from sponsor	2.50	kPa
Mechanical weight (0.3 kPa)		
50mm thk concrete layer (1.2 kPa)		
Partitions (1.0 kPa)		
D, total dead load	3.50	kPa
Live Loads		
Per NBCC Table 4.1.5.3 (assembly area)	4.80	kPa
L, total live load	4.80	kPa
Snow Loads		
None for second level	0.00	kPa
S, total snow load	0.00	kPa
Wind Loads		
None for this project	0.00	kPa
W, total wind load	0.00	kPa

Ultimate Limit States

Case	Principal	Companion	Total
1	4.90	0.00	4.90
2	11.58	0.00	11.58
3	4.38	0.00	4.38
4a	4.38	2.40	6.78
4b	4.38	0.00	4.38

Serviceability Limit States

Case	Principal	Companion	Total
1	8.30	0.00	8.30
2a	3.50	1.68	5.18
2b	3.50	0.00	3.50
3	3.50	1.68	5.18

Beam & Column Load Calculations

Unfactored Loads

Roof Level

Dead Load	2.60	kPa
Live Load	1.00	kPa
Snow Load (U)	1.98	kPa
Snow Load (S)	1.78	kPa

Ultimate Limit States

Case	Principal	Companion	Total
1	3.64	0.00	3.64
2	4.75	0.00	4.75
3	6.22	0.00	6.22
4a	3.25	0.50	3.75
4b	3.25	0.99	4.24

Serviceability Limit States

Case	Principal	Companion	Total
1	3.60	0.00	3.60
2a	2.60	0.35	2.95
2b	2.60	0.62	3.22
3	4.38	0.00	4.38

Unfactored Loads

Second Level

Dead Load	3.50	kPa
Live Load	4.80	kPa

Ultimate Limit States

Case	Principal	Companion	Total
1	4.90	0.00	4.90
2	11.58	0.00	11.58
3	4.38	0.00	4.38
4a	4.38	2.40	6.78
4b	4.38	0.00	4.38

Serviceability Limit States

Case	Principal	Companion	Total
1	8.30	0.00	8.30
2a	3.50	1.68	5.18
2b	3.50	0.00	3.50
3	3.50	1.68	5.18

Columns

						Ultimate						*Serviceability*				
Mark	Height (mm)	Flat Trib. Area (mm^2)	Sloped Trib. Area (mm^2)	Proj. Trib. Area (mm^2)	Total Trib. Area (mm^2)	P_{DL} (kN)	P_{SL} (kN)	P_{LL} (kN)	$P_{f\text{-}Ult}$ (kN)	*Check*		P_{DL} (kPa)	P_{SL} (kPa)	P_{LL} (kPa)	P_S (kN)	*Check*
Column C1	9000	11295000	0	0	11295000	36.71	33.55	0.00	70.25	70.25		29.37	20.13	0.00	49.49	49.49
Column C2 (Upper)	3000	7500000	16967071	16843875	24343875	79.52	72.30	0.00	151.82	151.42		63.61	43.38	0.00	107.00	106.67
Column C2 (Ground)	6000	16950000	0	0	16950000	74.16	0.00	122.04	196.20	196.20		59.33	0.00	81.36	140.69	140.69
C2 combined						153.67	72.30	122.04	348.02	347.62		122.94	43.38	81.36	247.68	247.36
Column C3 (Upper)	4500	0	29451776	29250000	29250000	95.72	86.87	0.00	182.59	181.94		76.57	52.12	0.00	128.70	128.17
Column C3 (Ground)	6000	19500000	0	0	19500000	85.31	0.00	140.40	225.71	225.71		68.25	0.00	93.60	161.85	161.85
C3 combined						181.03	86.87	140.40	408.30	407.65		144.82	52.12	93.60	290.55	290.02
Column C4 (Upper)	3000	14692500	28326776	28125000	42817500	139.81	127.17	0.00	266.98	266.32		111.85	76.30	0.00	188.15	187.63
Column C4 (Ground)	6000	28125000	0	0	28125000	123.05	0.00	202.50	325.55	325.55		98.44	0.00	135.00	233.44	233.44
C4 combined						262.86	127.17	202.50	592.53	591.87		210.29	76.30	135.00	421.59	421.06

**member selections for ground level columns will be used as upper level columns*

Load Duration Factors

DL	SL	LL	K_D factor
29.37	22.36		0.941
63.29	48.20		
59.33		81.36	
122.62		105.46	0.967
76.05	57.92		
68.25		93.60	
144.30		122.56	0.965
111.85	84.78		
98.44		135.00	
210.29		177.39	0.963

Beams

						Ultimate						*Serviceability*				
Mark	Spacing (mm)	Horizontal Length (mm)	Vertical Length (mm)	Length (mm)		w_{DL} (kN/m)	w_{SL} (kN/m)	w_{LL} (kN/m)	M_f (kNm)	V_f (kN)		w_{DL} (kN/m)	w_{SL} (kN/m)	w_{LL} (kN/m)	M_s (kNm)	V_s (kN)
Beam B1	3000	6530	0	6530		9.75	8.91	0.00	99.46	60.92		7.80	5.35	0.00	70.07	42.92
Beam B2	3000	9900	1500	10013		9.86	8.91	0.00	229.97	92.92		7.89	5.35	0.00	162.15	66.26
Beam B3	3000	12500	1500	12590		9.82	8.91	0.00	365.82	117.06		7.86	5.35	0.00	257.85	83.10
Beam B4	N/A	6000	0	6000		*N/A*	*N/A*	*N/A*	266.98	88.99		N/A	N/A	N/A	189.04	63.01
Beam B5	3000	9480	0	9480		13.125	0.00	21.60	390.09	164.60		10.50	0.00	14.40	279.72	118.03
Beam B6	3000	12500	0	12500		13.125	0.00	21.60	678.22	217.03		10.50	0.00	14.40	486.33	155.63
Beam B7	N/A	6000	0	6000		*N/A*	*N/A*	*N/A*	325.55	108.52		N/A	N/A	N/A	233.44	77.81

DL	SL	LL	K_D factor
50.93	38.79	0.00	0.941
78.10	59.48	0.00	0.941
98.20	74.78	0.00	0.941
74.57	56.52	0.00	0.940

CLT

						Ultimate			*Major direction*		
Mark	Span (mm)	Unit Length of panel (mm)				w_{DL} (kN/m)	w_{SL} (kN/m)	w_{LL} (kN/m)	$M_{f,0}$ (kNm)	$V_{f,0}$ (kN)	
CLT Panel P1	3000	1000				3.25	2.97	0.00	7.00	11.66	*used Skyciv to calculate max moment and shear within the pane*
CLT Panel P2	3000	1000				4.38	0.00	7.20	13.02	21.70	*used Skyciv to calculate max moment and shear within the pane*

Beam Member Selection

Mark	B1	B2	B3	B4	B5	B6	B7
Member Info							
Species:	Spruce-Pine	Spruce-Pine	Spruce-Pine	Spruce-Pine	Spruce-Pine	Spruce-Pine	Spruce-Pine
Grade:	20f-E	20f-E	20f-E	20f-E	20f-E	20f-E	20f-E
Length (mm):	6530	10013	12590	6000	9480	12500	6000
Ultimate State							
V_f (kN):	60.92	92.92	117.06	88.99	164.60	217.03	108.52
M_f (kNm):	99.46	229.97	365.82	266.98	390.09	678.22	325.55
w_fL (kN):	121.85	187.96	235.80	177.99	329.19	434.06	217.03
Serviceability State							
V_s (kN):	42.92	66.26	83.10	63.01	118.03	155.63	77.81
M_s (kNm):	70.07	162.15	257.85	189.04	279.72	486.33	233.44
w_sL (kN):	85.84	132.52	166.21	126.03	236.05	311.25	155.63
w_{DL} (kN/m):	7.80	7.89	7.86	*N/A*	10.50	10.50	*N/A*
w_{SL} (kN/m):	5.35	5.35	5.35	*N/A*	0.00	0.00	*N/A*
w_{LL} (kN/m):	0.00	0.00	0.00	*N/A*	14.40	14.40	*N/A*
P_{DL} (kN):	*N/A*	*N/A*	*N/A*	74.92	*N/A*	*N/A*	65.63
P_{SL} (kN):	*N/A*	*N/A*	*N/A*	51.11	*N/A*	*N/A*	0.00
P_{LL} (kN):	*N/A*	*N/A*	*N/A*	0.00	*N/A*	*N/A*	90.00
Beam Dimensions							
base, b (mm):	315	315	315	315	315	365	365
depth, d (mm):	418	646	798	798	760	950	950
Strength check (CSA)							
Volume (m^3):	0.860	2.038	3.165	1.508	2.270	4.334	2.081
K_D:	0.94	0.94	0.94	0.94	0.65	0.65	0.65
K_{zbg}:	0.98	0.90	0.86	0.93	0.89	0.84	0.90
K_L:	1.00	1.00	1.00	1.00	1.00	1.00	1.00
f_b (MPa):	25.6	25.6	25.6	25.6	25.6	25.6	25.6
F_b (MPa):	24.09	24.09	24.09	24.06	16.64	16.64	16.64
E (MPa):	10300	10300	10300	10300	10300	10300	10300
I_x (mm^4):	1917159090	7076636070	13339451790	13339451790	11523120000	26078489583	26078489583
S (mm^3):	9173010	21909090	33432210	33432210	30324000	54902083	54902083
M_r (kNm):	195.39	428.10	625.12	672.48	404.97	687.26	739.60
f_v (MPa):	1.75	1.75	1.75	1.75	1.75	1.75	1.75
F_v (MPa):	1.65	1.65	1.65	1.64	1.14	1.14	1.14
Area (mm^2):	131670	203490	251370	251370	239400	346750	346750
C_v:	3.69	3.69	3.69	3.69	3.69	3.69	3.69
V_r (kN):	130.08	201.03	248.33	248.06	163.39	236.66	236.66
W_r (kN):	355.11	469.86	536.19	612.05	374.56	482.87	551.07
Deflection check							
L/360 (mm):	18.14	27.81	34.97	16.67	26.33	34.72	16.67
DL def. (mm):	9.35	14.17	18.70	2.45	9.30	12.43	1.10

Notes:

B4 and B7 increased sizes to match B3 and B6 respectively

All beams are simply supported

Initial sizes of beams governed by deflection (serviceability)

Fire design only reduces beams on the sides and bottoms

Fire design calls for specified loading rather than factored

Beam depth not reduced at top face due to CLT

CSA B.4.5, do not need to take into account corner rounding when using notional reduction

Use 315x608 for B1 only when connecting to B4

Use 365x798 for B3 only when connecting to B4

SL def. (mm):	6.41	9.60	12.73	1.67	0.00	0.00	0.00
LL def. (mm):	0.00	0.00	0.00	0.00	12.76	17.04	1.51
Total def. (mm):	15.76	23.77	31.43	4.13	22.06	29.47	2.61
Capacity Summary							
% extra (Moment):	96.45	86.15	70.88	151.88	3.81	1.33	127.19
% extra (Shear):	113.50	149.98	127.39	178.73	13.78	11.24	153.91
% extra (Deflection):	13.11	14.55	10.12	75.23	16.22	15.13	84.36

Fire Design	**B1**	**B2**	**B3**	**B4**	**B5**	**B6**	**B7**
New Dimensions							
base, b (mm):	315	315	315	315	315	365	365
depth, d (mm):	418	646	798	798	760	950	950
Fire resistance rating, t (mins):	120	120	120	120	120	120	120
Notional, $x_{c,n}$ (mm):	84	84	84	84	84	84	84
Zero-strength layer, x_t (mm):	7	7	7	7	7	7	7
Reduced base, b_{red} (mm):	133	133	133	133	133	183	183
Reduced depth, d_{red} (mm):	327	555	707	707	669	859	859
Strength check (fire clauses)							
Reduced area (mm^2):	43491	73815	94031	94031	88977	157197	157197
Reduced Vol. (m^3):	0.28	0.74	1.18	0.56	0.84	1.96	0.94
K_D:	1.15	1.15	1.15	1.15	1.15	1.15	1.15
K_{zbg}:	0.98	0.90	0.86	0.93	0.89	0.84	0.90
K_{fi}:	1.35	1.35	1.35	1.35	1.35	1.35	1.35
S (mm^3):	2370260	6827888	11079986	11079986	9920936	22505371	22505371
f_b (MPa):	25.6	25.6	25.6	25.6	25.6	25.6	25.6
F_b (MPa):	29.44	29.44	29.44	29.44	29.44	29.44	29.44
M_r (kNm):	92.57	244.61	379.85	409.07	351.61	747.65	804.59
f_v (MPa):	1.75	1.75	1.75	1.75	1.75	1.75	1.75
F_v (MPa):	2.72	2.72	2.72	2.72	2.72	2.72	2.72
C_v:	3.69	3.69	3.69	3.69	3.69	3.69	3.69
V_r (kN):	78.77	133.70	170.31	170.31	161.16	284.72	284.72
W_r (kN):	262.51	375.07	438.95	501.59	441.49	669.85	764.46
Capacity Summary							
% extra (Moment):	32.11	50.86	47.31	116.39	25.70	53.73	244.67
% extra (Shear):	205.80	183.03	164.10	298.01	87.03	115.21	391.22

Column Member Selection

Mark	C1	C2	C3	C4
Member Info				
Species:	Spruce-Pine	Spruce-Pine	Spruce-Pine	Spruce-Pine
Grade:	12c-E	20f-EX	20f-EX	20f-EX
Length (mm):	9000	6000	6000	6000
Ultimate State				
P_f from column (kN):	70.25	348.02	408.30	592.53
P_f from beam 1 (kN):	60.92	217.03	217.03	217.03
P_f from beam 2 (kN):	*N/A*	*N/A*	*N/A*	108.52
M_f from beam 1 (kNm):	14.20	54.69	54.69	71.19
M_f from beam 2 (kNm):	*N/A*	*N/A*	*N/A*	32.55
Serviceability State				
P_{DL} (kN):	29.37	122.94	144.82	210.29
P_{SL} (kN):	20.13	43.38	52.12	76.30
P_{LL} (kN):	0.00	81.36	93.60	135.00
P_s from column (kN):	49.49	247.68	290.55	421.59
P_s from beam 1 (kN):	42.92	155.63	155.63	155.63
P_s from beam 2 (kN):	*N/A*	*N/A*	*N/A*	77.81
Column Dimensions				
base, b (mm):	215	265	265	400
depth, d (mm):	266	304	304	456
Strength check				
Volume (m^3):	0.515	0.483	0.483	1.094
K_D:	0.94	0.97	0.96	0.96
K_{zcg}:	0.74	0.75	0.75	0.67
f_{cb} (MPa):	25.2	25.2	25.2	25.2
F_{cb} (MPa):	23.71	24.38	24.31	24.27
Area (mm^2):	57190	80560	80560	182400
K_e:	1.00	1.00	1.00	1.00
$C_{c\text{-}b}$:	41.86	22.64	22.64	15.00
$C_{c\text{-}d}$:	33.83	19.74	19.74	13.16
E (MPa):	9700	10300	10300	10300
K_c:	0.19	0.60	0.60	0.85
P_r (kN):	149.89	701.30	700.11	2024.67
f_b (MPa):	9.8	25.6	25.6	25.6
F_b (MPa):	9.22	24.76	24.69	24.65
K_{zbg}:	1.03	1.04	1.04	0.96
K_L:	1.00	1.00	1.00	1.00
S (mm^3):	2535423	4081707	4081707	13862400
M_r (kNm):	21.04	90.96	90.71	295.04
P component:	0.22	0.25	0.34	0.09

Notes:

Columns assumed to be pinned-pinned

Design for lower level columns will be copied to above level

CSA 7.5.3 for bi-axial bending, utilize No.2 grade strength for weak direction

Upper C4 column check not required:

- Moments from B1 and B3 are in opposite directions resulting in a smaller overall moment
- Smaller compression forces and smaller moments due to eccentricity from joining beams

Due to specific conditions of column, WDM interaction equation was used

M_r per CSA 7.5.3:	
f_b (MPa):	4.85
K_H:	1.10
F_b (MPa):	5.14
K_{Zb}:	0.90
S (mm^3):	12160000
M_r (kNm):	50.62

M component 1:	0.67	0.60	0.60	0.24
M component 2:	*N/A*	*N/A*	*N/A*	0.64
Interaction Eqn (WDM):	0.89	0.85	0.94	0.97

Fire Design	**C1**	**C2**	**C3**	**C4**
New sizes				
base, b (mm):	365	400	400	490
depth, d (mm):	418	418	418	532
Fire resistance rating, t (mins):	120	120	120	120
Notional, $x_{c,n}$ (mm):	84	84	84	84
Zero-strength layer, x_t (mm):	7	7	7	7
Reduced base, b_{red} (mm):	183	218	218	308
Reduced depth, d_{red} (mm):	236	236	236	350
Strength check (fire clauses)				
M_f from beam 1 (kNm):	13.26	48.09	48.09	56.96
M_f from beam 2 (kNm):	*N/A*	*N/A*	*N/A*	26.85
Reduced area (mm^2):	43188	51448	51448	107800
K_D:	1.15	1.15	1.15	1.15
K_{fi}:	1.35	1.35	1.35	1.35
K_{zcg}:	0.74	0.75	0.75	0.67
f_{cb} (MPa):	25.2	25.2	25.2	25.2
F_{cb} (MPa):	39.12	39.12	39.12	39.12
K_e:	1.00	1.00	1.00	1.00
$C_{c\text{-}b}$:	49.18	27.52	27.52	19.48
$C_{c\text{-}d}$:	38.14	25.42	25.42	17.14
E (MPa):	9700	10300	10300	10300
K_c:	0.08	0.34	0.34	0.62
P_r (kN):	98.78	511.03	511.03	2362.38
f_b (MPa):	9.8	25.6	25.6	25.6
F_b (MPa):	15.21	39.74	39.74	39.74
K_{zbg}:	1.03	1.04	1.04	0.96
K_L:	0.99	1.00	1.00	1.00
S (mm^3):	1698728	2023621	2023621	6288333
M_r (kNm):	25.51	80.43	80.43	239.73
P component:	0.25	0.23	0.32	0.03
M component 1:	0.52	0.60	0.60	0.24
M component 2:	*N/A*	*N/A*	*N/A*	0.65
Interaction Eqn (WDM):	0.77	0.83	0.92	0.92

M_r per CSA 7.5.3:	
f_b (MPa):	4.85
K_H:	1.10
F_b (MPa):	6.14
K_{Zb}:	0.90
S (mm^3):	5533733
M_r (kNm):	41.26

CLT Panel Selection

Mark	**P1**	**P2**
Member Info		
Species:	Spruce-Pine	Spruce-Pine
Grade:	V2	E1
Length in weak direction(mm):	1000	1000
Length (mm):	3000	3000
Ultimate State		
$V_{f,0}$ (kN):	11.66	21.70
$M_{f,0}$ (kNm):	7.00	13.02
Panel Dimensions		
height, h (mm):	105	175
Strength check (CSA)		
K_D:	0.94	0.65
$K_{rb,0}$:	0.85	0.85
$f_b(S_{eff,f,0})$ (10^6 N-mm/m):	18	98
$F_b(S_{eff,f,0})$ (N-mm/m):	16.94	63.70
$M_{r,f,0}$ (kNm):	12.96	48.73
$V_{r,f,0}$ (kN):	29.64	33.93
Capacity Summary		
% extra (Moment):	85.14	274.22
% extra (Shear):	154.12	56.34

Notes:
Calculations based on 1m width of CLT and 1 span length of 3000mm
One end fixed and other pinned, cantilever system due to CLT bridging 2 spans
Used values from provided catalogue
Minor direction not checked due to beams handling load

Find suitable CLT panel based on maximum values from catalogue below
CLT APA Publication Search - APA – The Engineered Wood Association (apawood.org)
E1 & V2, strength check only

Connections at Beam end

Beam	**B1**	**B2**	**B3**	**B4**	**B5**	**B6**	**B7**
Width (mm):	315	315	315	315	315	365	365
Depth (mm):	418	646	798	798	760	950	950
K_D:	0.94	0.94	0.94	0.94	0.65	0.65	0.65
Fire Calculations							
Insulated Distance (mm):	84	84	84	84	84	84	84
Reduced Width (mm):	147	147	147	147	147	197	197
Reduced Depth (mm):	334	562	714	714	676	866	866
Plate							
Thickness (mm):	15	15	15	15	15	15	15
Max width (mm):	334	562	714	714	676	866	866
Pin Dimensions and Spacing							
Diameter (mm):	19	19	19	19	19	19	19
F_y (MPa):	310	310	310	310	310	310	310
S_P (mm):	0	0	0	0	0	0	0
S_Q (mm):	57	57	57	57	57	57	57
a (mm):	76	76	76	76	76	76	76
e_P (mm):	28.5	28.5	28.5	28.5	28.5	28.5	28.5
e_Q (mm):	76	76	76	76	76	76	76
Max # of pins per row:	4	8	10	10	10	13	13
Trial # of pins per row:	3	5	6	4	10	11	6
# of rows:	1	1	1	1	1	1	1
Connection checks							
Check 1 - Yielding							
N_f (kN):	60.92	92.92	117.06	88.99	164.60	217.03	108.52
N_r (kN):	68.80	114.67	137.60	91.67	181.86	218.53	119.20
n_s:	2	2	2	2	2	2	2
n_F:	3	5	6	4	10	11	6
n_u:	14333	14333	14333	14324	11366	12416	12416
Unit Lateral Yielding Resistance							
f_1 (MPa):	16.00	16.00	16.00	15.99	11.06	11.06	11.06
f_{iP}:	17.01	17.01	17.01	17.01	17.01	17.01	17.01
f_{iQ}:	7.48	7.48	7.48	7.48	7.48	7.48	7.48
f_2 (MPa):	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0
t_1 (mm):	66	66	66	66	66	91	91
t_2 (mm):	15	15	15	15	15	15	15
θ (deg):	0	0	0	0	0	0	0
Item A	20069	20069	20069	20047	13865	19117	19117
Item C	192375	192375	192375	192375	192375	192375	192375
Item D	14333	14333	14333	14324	11366	12416	12416
Item G	20639	20639	20639	20628	17186	17186	17186
Check 2 - Splitting							
Q_f:	60.92	92.92	117.06	88.99	164.60	217.03	108.52
QS_i:	333317	521736	647324	647324	615928	895587	895587
d_e:	389.5	617.5	769.5	769.5	731.5	921.5	921.5
Q_r:	219.52	343.61	426.32	425.86	280.25	407.49	407.49
Check Summary							
% extra (Yielding):	12.93	23.41	17.54	3.01	10.49	0.69	9.84
% extra (Splitting):	260.31	269.80	264.18	378.53	70.26	87.76	275.51

Notes:

Do not need to check for group tear out or net tension

For B1 to B4, use MEGANT 430x60

For B3 to B4, use MEGANT 430x150

For B6 to B7, use MEGANT 730x150

Megant connectors required to be insulated from fire

Connections at Column end

Column	C1	C2	C2	C2	C2	C3	C3	C3	C3	C4	C4	C4	C4
Width (mm):	365	400	400	400	400	400	400	400	400	490	490	490	490
Depth (mm):	418	418	418	418	418	418	418	418	418	532	532	532	532
K_D:	0.94	0.97	0.97	0.97	0.97	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Connecting Beam(s)	**B1**	**B1,B2**	**B1,B3**	**B5**	**B6**	**B2**	**B3**	**B5**	**B6**	**B1,B3**	**B4**	**B6**	**B7**
Plate Dimensions													
Thickness (mm):	15	15	15	15	15	15	15	15	15	15	15	15	15
Max width (mm):	334	334	334	676	866	562	714	676	866	334	714	866	866
Fire Assumptions													
Insulated Distance (mm):	84	84	84	84	84	84	84	84	84	84	84	84	84
Reduced Width (mm):	197	232	232	232	232	232	232	232	232	322	322	322	322
Reduced Depth (mm):	250	250	250	250	250	250	250	250	250	364	364	364	364
Pin Dimensions and Spacing													
Diameter (mm):	19	19	19	19	19	19	19	19	19	19	19	19	19
F_y (MPa):	310	310	310	310	310	310	310	310	310	310	310	310	310
S_P (mm):	76	76	76	76	76	76	76	76	76	76	76	76	76
S_Q (mm):	57	57	57	57	57	57	57	57	57	57	57	57	57
a (mm):	76	76	76	76	76	76	76	76	76	76	76	76	76
e_P (mm):	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5	28.5
Max # of pins per row:	4	4	4	10	13	8	10	10	13	4	10	13	13
Trial # of pins per row:	2	3	4	3	4	2	3	3	4	3	2	4	2
Max # of rows:	6	6	6	6	6	6	6	6	6	8	8	8	8
Trial # of rows:	2	2	2	2	2	2	2	2	2	2	2	2	2
Connection checks													
Check 1 - Yielding													
N_f (kN):	60.92	153.84	177.99	164.60	217.03	92.92	117.06	164.60	217.03	177.99	88.99	217.03	108.52
N_r (kN):	101.46	165.56	220.74	165.56	220.74	110.16	165.23	165.23	220.31	191.95	133.62	255.93	133.62
n_s:	2	2	2	2	2	2	2	2	2	2	2	2	2
n_F:	4	6	8	6	8	4	6	6	8	6	4	8	4
n_u:	15854	17245	17245	17245	17245	17212	17212	17212	17212	19995	20879	19995	20879
Unit Lateral Yielding Resistance													
f_1 (MPa):	16.00	16.45	16.45	16.45	16.45	16.41	16.41	16.41	16.41	16.38	16.38	16.38	16.38
f_{iP}:	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01	17.01
f_{iQ}:	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48	7.48
f_2 (MPa):	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0	1350.0
t_1 (mm):	91	108.5	108.5	108.5	108.5	108.5	108.5	108.5	108.5	153.5	174.5	153.5	174.5
t_2 (mm):	15	15	15	15	15	15	15	15	15	15	15	15	15
θ (deg):	0	0	0	0	0	0	0	0	0	0	0	0	0
Item A	27671	33918	33918	33918	33918	33823	33823	33823	33823	47777	54313	47777	54313
Item C	192375	192375	192375	192375	192375	192375	192375	192375	192375	192375	192375	192375	192375
Item D	15854	17245	17245	17245	17245	17212	17212	17212	17212	19995	21302	19995	21302
Item G	20639	20924	20924	20924	20924	20894	20894	20894	20894	20879	20879	20879	20879
Check Summary													
% extra (Yielding):	66.54	7.61	24.02	0.58	1.71	18.55	41.15	0.39	1.51	7.84	50.15	17.92	23.14

Notes:

Do not need to check for group tear out or net tension

Use 2 rows of bolts