22-13

DESIGN OF A CANTILEVERING WOOD CANOPY FOR A PARK IN SOUTH SURREY, BC

Prepared for:

Industry Projects Committee, BCIT Civil Engineering Department
Kian Karimi, Faculty Advisor, BCIT Civil Engineering Department
Jacquie Russell, BCIT Communications Department

Prepared by:

Submitted on:
April 20, 2022

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.

April 20, 2022

Industry Project Sponsor

Dear

Submission of Final Report on Design of a Cantilevering Wood Canopy For a Park in South Surrey, BC

Attached is my report for the canopy structure I designed. Located in South Surrey Athletic Park, this canopy would provide shelter to audiences watching sports as well as host events and informal gatherings. My objectives in this project were to determine an optimal canopy size, design member sizes and connections, and use 3D software to produce a layout drawing set.

I have listed the outcomes of my project as follows:

- I found it most efficient to use full length DLT panels to span the entire length. This also made it possible to have deflection in the center of the structure for drainage purposes.
- By cambering the beams, I was able to have a continuous beam at each support while still sloping the roof on either side.
- The beam at the center of the roof carried the most load, and therefore governed in size. However, for efficiency, I designed the outer beams to be shallower.
- I designed the columns, connections, and footings to take seismic load, which governed over wind or gravity load.

In total, I spent 127 hours on the project, the majority of which were spent on the design calculations. As discussed towards the end of the project, a finite element model was not needed as the structure was simple enough to require only hand calculations.

Designing this canopy has greatly broadened my knowledge of structural engineering. Not only has it given me insight into how to design simple gravity and lateral systems, but it has taught me the workflow and time management an engineer must have when designing.

, thank you for your willingness to sponsor me for this project. , thank you for the time you spent showing me the concepts in designing this canopy and the speed with which you responded to me. If you would like to reach out to me regarding this report, my number is .

Sincerely,

cc: Kian Karimi, Faculty Advisor
Jacquie Russell, Communication Instructor

Attachment: final report

SUMMARY

Having had previous experience with drafting wood canopies, felt it would be a useful experience for me to design a cantilevering wood park canopy. The location of the canopy structure is in South Surrey Athletic Park and has been designed as a multi-purpose structure to host sports audiences and informal events.

To design the structure, I began by analyzing the overall design requirements. Some of these requirements included the panel length being limited to 60 feet and the head height being sufficient for an audience on the top row of bleachers. The final canopy size I determined to be 60 feet long by 36 feet wide by over 16 feet tall. I utilized the deflection of the roof panels to meet the 1:50 slope required for the canopy roof as specified in NBCC 2015 (NRCC, 2018).

The next step in the design of the canopy was to find the gravity, lateral and seismic loads. I determined the dead load of the DLT (Dowl Laminated Timber) panels to be 1.03 kPa and the dead load of the beams to be 5.45 kN for the shallower beams and 6.21 for the deeper beam. The snow load was 1.8 kPa and the wind load was governed by the seismic load which was 0.47 kPa.

To find determine the member sizes of the DLT panels and the beams, I found the shear, bending and deflection demand in each. I then used the CSA 086-19 code to check assumed member sizes. I found that 2x8 DLT satisfied the demand for the panels, while 175x532 and 175x608 sizes satisfied the demand for the outer and center beams, respectively. In addition to sizing the roof members, I found the shear force in the roof diaphragm and specified 2 ½" nails at 150mm o.c. which more than satisfied the demand of 0.314 kN/m.

Once I had sized the roof members, I sized the columns for wind load demand and seismic demand. I found that the seismic load governed and checked the columns against cross-sectional strength, overall member strength, and lateral-torsional buckling using the CSA S16-14 code. From this, I determined that an HSS 203x203x9.5 member size satisfied the demand.

For connection design, I designed the connections from panel to beam and beam to column. For the panel to beam connection, I found that 13mm x 300mm lag screws at 300mm o.c. satisfied the demand of 5.25 kN. To design the beam to column connection, sized two side plates with two bolts running through them and the beam. I sized the plates for seismic load, finding that a 200x700x19mm plate more than satisfied the demand. For each bolt to resist a 7.5 kN force, I selected 1"ϕ A325 bolts.

To finish my design, I sized the footings required beneath each row of columns. Using a bearing pressure of 75 kPa in the soil (The Ontario Building Code, n.d.), I determined the eccentricity due to the moment and the resultant force of the structure on the soil below a 3759x3048x610mm footing. I found the overturn pressure to be lower on either side of the footing than the maximum of 75 kPa, therefore the footing size was acceptable.

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

TABLES

FIGURES

1.0 INTRODUCTION

The purpose of this project was to design a cantilevering wood park canopy for , with the site location in Surrey, BC. felt that this project would be a useful means of education for me within the coming year. Having had some experience in the drafting and design of wood canopies, this project was a unique chance for me to further explore the design of canopies and incorporate a cantilever as the primary design.

Many parks use canopies to provide shelter for gatherings and events. Canopies are also used to provide cover and seating for an audience or a sports team. An example of this kind of canopy is shown in Figure 1 below.

Figure 1: Canopy for Sports Audience (USA Shade)

I designed the canopy to have functionality for both seating of small audiences for sports as well as providing shelter for casual gatherings. The site I used for the design of this canopy is the sports field at South Surrey Athletic Park as shown in Figure 2 below.

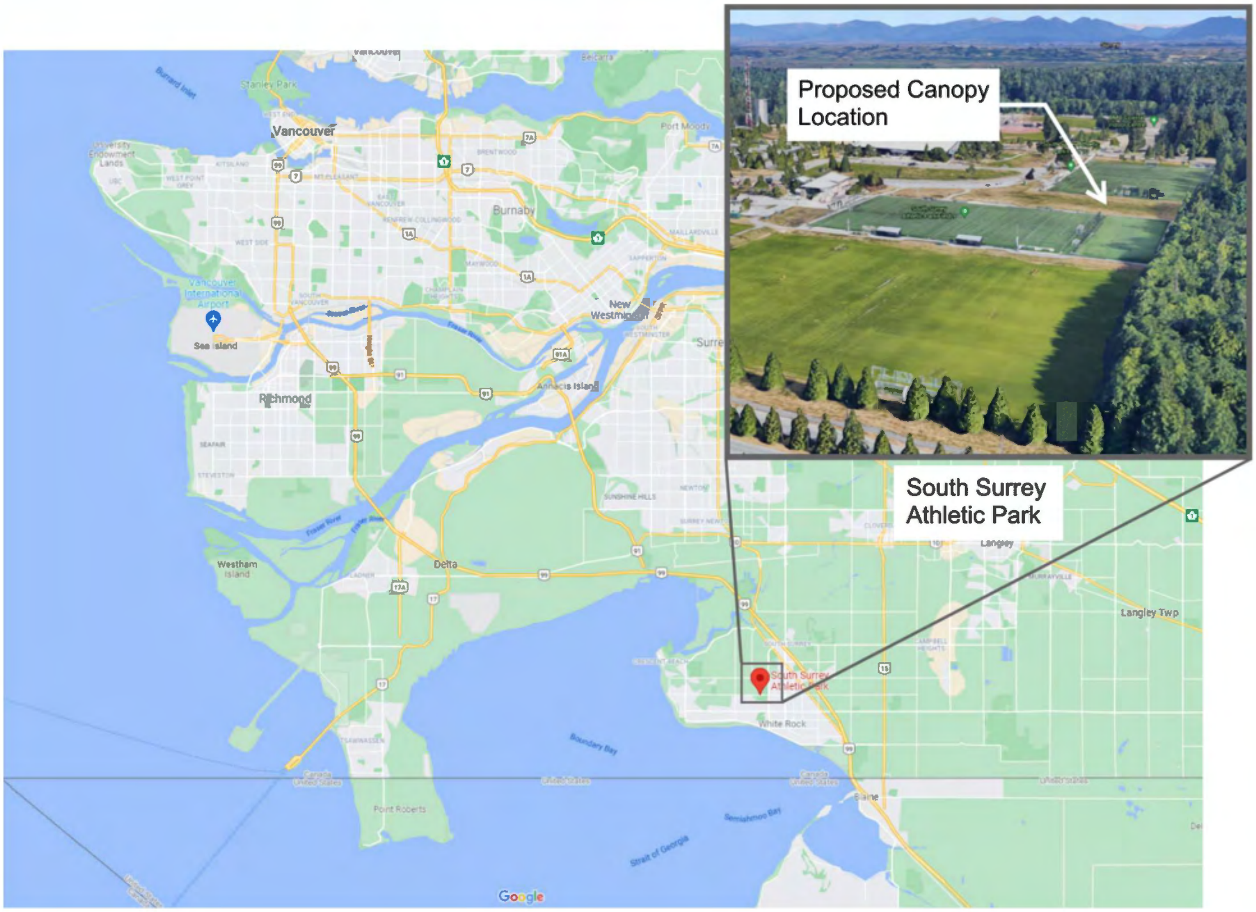


Figure 2: Map of Proposed Site Location (South Surrey Athletic Park, n.d.)

South Surrey has a rapidly growing population, with projections that approximately 10,000 new residents will move into the area by 2026 (City of Surrey, n.d.). The multi-purpose canopy I have designed will be useful to the residents, especially those with young families. It is also situated in an easily accessible part of the park that is close to multiple sports fields.

The objectives I have accomplished in the design of the cantilevering canopy are as follows:

- Determined the size of the structure considering functionality and aesthetics
- Performed engineering calculations to determine the design loads, member sizes and connections.
- Used 3D software to produce a model and drawings of the canopy

As discussed in my proposal, the project scope did not include the concrete base design. However, I did some preliminary design of a footing as an additional item in this project. I also included seismic loads in the scope of my project as it pertained to the columns, connections, and footings. Upon discussion with my sponsor and faculty advisor, I removed the creation of a finite element mode from the scope of my project because of its complex nature.

When designing the connections, I did not design certain elements that were too complicated or beyond the scope of my knowledge (e.g., welds). I also did not design the column base connection including the concrete pedestal design.

The remainder of this report will cover my design of the canopy including the loads and member sizing as well as the design of connections and the footings. It will also discuss the process by which I made my 3D model and include the structural layout drawings.

2.0 DESIGN OF THE CANOPY

To efficiently design the canopy, I had to consider initial design factors that would determine the size, aesthetics, and functionality of the structure. From there, I began calculating unfactored gravity, lateral and seismic loads which would determine the size and requirements of the members. To finish off my design, I calculated the connection types and sizes required at the roof and beam levels.

Throughout this process, I utilized software such as Rhino 3D and AutoCAD to aid me in finding measurements and in determining the functionality of the canopy space. I created spreadsheets in Excel that sped up the process of calculations, while maintaining the quality of work with sample hand calculations.

2.1 INITIAL DESIGN FACTORS

Because the canopy will be used for informal gatherings and sports audiences, a key design factor was the size and height of the structure. These dimensions were driven by constraints such as the manufacturer's limit on panel length, the head-height needed above bleachers, and passageway for people between columns.

The limit has on panel length is 60 feet. To meet this limitation, I decided to make the canopy roof 60 feet long and have a row of columns as supports in the middle. This would provide enough space for each bay to contain 2 bleachers for a total of 4 bleachers.

For the head height to the underside of the panels, I designed the canopy to be approximately 16 ½ feet high at the lowest point. A typical bleacher height is shown in Figure 3 below:

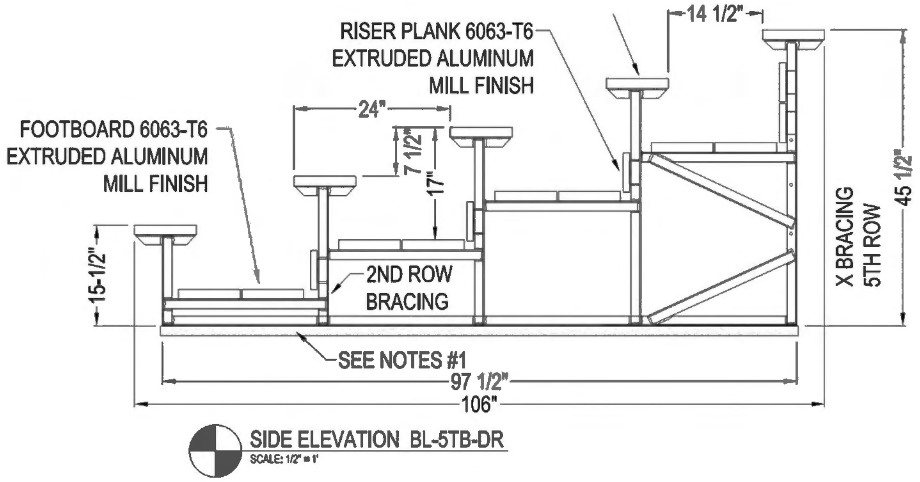


Figure 3: Typical Bleacher Dimensions (SportSystems Canada, 2009)

As shown above, a typical 5 seat bleacher has a height of 45 ½ inches, or almost 4 feet. A tall person that is 7 or 8 feet in height would feel no constraint in head height. With this height, the beams would also not be in the sight line of viewers who are sitting at the top of the bleachers.

To maintain easy passageway between the columns, I made them approximately 5 feet apart at the base. This will allow attendees of events to easily mingle between the tables set up underneath the structure.

I also designed the width of the canopy to have large cantilevers to enable audiences to have unrestricted views. To achieve this, the beams had to be cambered to maintain a continuous member while sloping the roof upwards. The taper also aided in keeping the beams from restricting the view.

Another key design factor I considered was the roof slope. In the N-S direction, I designed the beams to have a camber that created a horizontal slope of over 8° at the steepest part. This slope would allow water to flow from either side towards the center of the structure.

In the E-W direction, I designed the canopy roof to slope towards the middle from either side at a slope of 1:50. This slope comes from NBCC Table 9.26.3.1 for Modified Bituminous Membranes (NRCC, 2018), and the membrane used on the canopy will be TPO (Thermoplastic Polyolefin), a common commercial roofing material.

To create a 1:50 slope in the longitudinal direction of the canopy, I used the natural deflection of the 60-foot-long panels to deflect to the required elevation in the middle of the structure. This required deflection came to be approximately 180mm, while the deflection capacity of the panels is sufficiently more.

2.2 GRAVITY, LATERAL AND SEISMIC LOADS

After determining the dimensional and functional constraints of the structure, I determined the gravity, lateral and seismic loads. All these loads were calculated as unfactored and remained that way until the member calculations.

To determine the gravity loads, I calculated the dead load of the panels and beams and the snow load on the roof area. I also calculated the weight of columns later which were only used for the footing design. I used Douglas Fir, which has a density of approximately 33 lb/sf as the timber material for the glulam beams and DLT (Dowel Laminated Timber) panels.

The dead load due to DLT panels was 1.03 kPa, and the dead load due to beams was 5.45 kN for Beam A and 6.21 kN for Beam B. See section 2.3.2, Glulam Beam Design, for more details on the two depths of beams.

The snow load was calculated using the formula in NBCC 4.1.6.2 (NRCC, 2018) with the Snow Load (S_S) factor for White Rock being 2.0 kPa. After modification factors, I found the final snow load to be 1.8 kPa.

The lateral loads were calculated using more complex means. I used the formula in NBCC 4.1.7.3 (NRCC, 2018) but substituted the C_gC_p factors for factors given by the ASCE 7-16 code (ASCE/SEI, 2017). These modifications are standard for
to use in situations where the NBCC code does not directly address the requirements of the project. The modified formulas I derived are shown below.

	NBCC 4.1.7.3		ASCE 7-16 Modifications
Troughed Roof:	$\boxed{p = I_W q C_e C_t C_g C_p}$	$\longrightarrow$	$\boxed{p = I_w q C_e C_t G C_N}$
Beam or Panel Edge:	$\boxed{p = I_W q C_e C_t C_g C_p}$	$\longrightarrow$	$\boxed{p = I_w q C_e C_t G C_F}$

For troughed roofs, the wind load is modified by the surface and gust factors, given by C_N and G respectively. For wind load on beam and panel edges, the wind load is modified by the force and gust factors, given by C_F and G respectively.

I split the wind loads up between the E-W side of the canopy (the long side), and the N-S side (the ends). The pressures I calculated for each side are shown in the table below:

Table 1: Wind Pressures on the Canopy

Wind Pressure, E-W Side (kPa)		
	Windward	Leeward
Panel Surface, Case A	-0.341	0.093
Panel Surface, Case B	-0.062	0.372
Panel Edge	0.61	

Wind Pressure, N-S Side (kPa)	
Beam Face	0.595
Panel Edge	0.61

The highest pressure on both sides was along the edges of members. To make these pressures into useable numbers for my calculations, I created line loads from them for each side of the canopy.

For the E-W side, I summed the horizontal components of the surface pressures as well as the panel edge pressure. For the N-S side, I summed the pressures on the beams and the panel edges. I also took the height of snow into account for the panel edge pressures. The final line loads for each side were $w_{E\text{-}W} = 0.544$ kN/m and $w_{N\text{-}S} = 0.443$ kN/m. See my calculations in Appendix B for more detail.

To determine the seismic loads, I calculated the approximate values using a modified approach suggested to me by my industry sponsor. I factored the dead and snow loads from Case 5 in the NBCC (1D + 0.25S). I then multiplied this value by a factor of 0.3 to obtain the approximate seismic forces on the structure of 0.47 kPa.

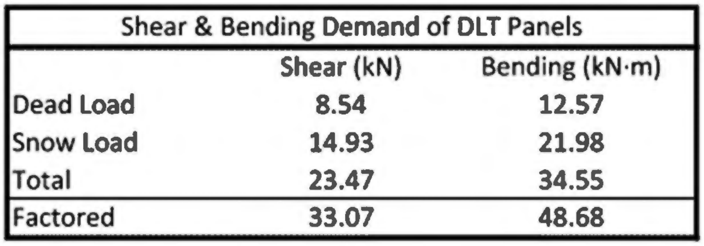
2.3 GRAVITY SYSTEM ANALYSIS

To design the gravity system, the major components included the DLT roof panels, Glulam beams, and axial loads in the columns. I only considered gravity loads when designing the DLT panels and Glulam beams.

2.3.1 DLT Panel Design

The first step I took in designing the roof panels was finding the unfactored and factored shear and bending demand. Using the shear-force and bending-moment diagrams in my calculations, I found the following results:

Table 2: Shear & Bending in DLT Panels



Shear & Bending Demand of DLT Panels		
	Shear (kN)	Bending (kN·m)
Dead Load	8.54	12.57
Snow Load	14.93	21.98
Total	23.47	34.55
Factored	33.07	48.68

Using the factored values above, I designed the panels using 2x8 DLT. treats DLT as sawn lumber in the code; therefore, I used the formulas for sawn lumber given in section 6.5 of the CSA 086 code (CSA Group, 2021). Using the 38x180mm lamination sizes and panel widths of 6 ft, I found the 60 ft long 2x8 DLT panels to be acceptable.

To find the deflection in the panels, I first found what the maximum deflection would be if a panel were only resting on the two outer beams of the canopy structure. This would create a span of 50 feet, and a maximum deflection in the middle of approximately 310mm. To meet the 180mm deflection requirement at the middle, I dropped the middle beam 180mm so that the partially deflected panel would rest on the center beam.

For serviceability requirements, the max deflection in panels with a center support is approximately 10mm, well within the allowable 85mm.

2.3.2 Glulam Beam Design

As shown in the layout drawings in Appendix A, there are two depths of beams used due to the higher loading in the center of the structure (labelled Beam A and B). These beams were cambered to create a roof slope towards the center of the structure on either side. I also tapered the cross section to be shallower at the ends of the beams for aesthetic and functionality purposes as shown below:

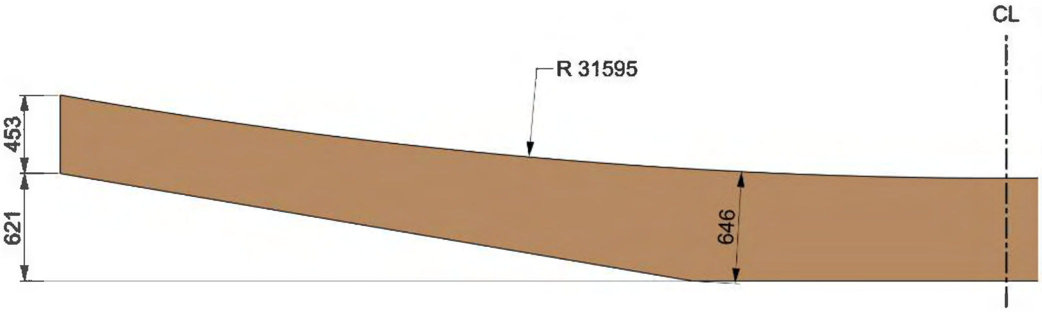


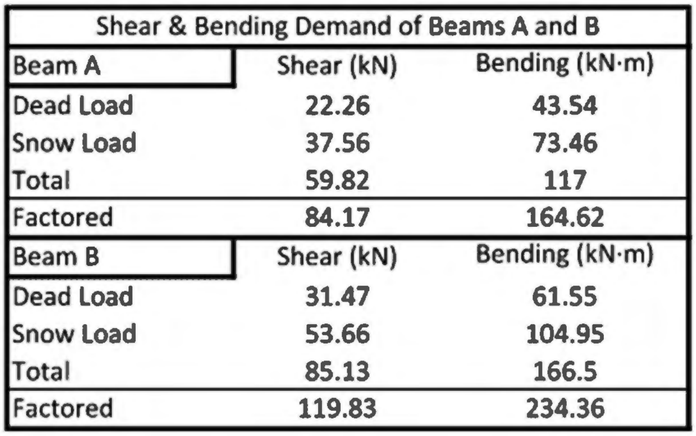
Figure 4: Taper and Camber of Beam A

Since these tapers will significantly affect the moment of inertia of the beams, I accounted for the decrease in capacity by increasing the uncut depth of the beam. Also, the demand in bending and shear is much lower than the maximum demand at the tapered locations of the beam.

When checking the beams for deflection, I also approximated the required moment of inertia. Considering that the beams are deeper than necessary at the supports where the deflection due to rotation occurs, I ensured the required depth occurred at 2/3 length of the cantilever in from each end. See my calculations in Appendix B for further explanation.

Using the factored dead and snow load, I found the shear and bending demand in both Beam A and Beam B as listed in the table below:

Table 3: Shear & Bending in Beams

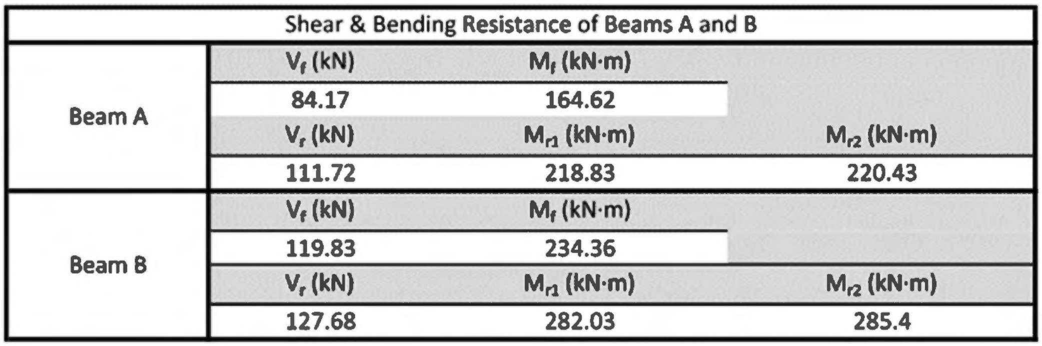


Shear & Bending Demand of Beams A and B		
Beam A	Shear (kN)	Bending (kN·m)
Dead Load	22.26	43.54
Snow Load	37.56	73.46
Total	59.82	117
Factored	84.17	164.62
Beam B	Shear (kN)	Bending (kN·m)
Dead Load	31.47	61.55
Snow Load	53.66	104.95
Total	85.13	166.5
Factored	119.83	234.36

Analyzing the values calculated above, the optimal cross section sizes I used for Beams A and B were 175 x 532 and 175 x 608, respectively.

Calculating the bending, shear, and deflection checks on the beam, I found the following results for each beam size:

Table 4: Shear & Bending Resistance in Beams



Shear & Bending Resistance of Beams A and B			
Beam A	V_f (kN)	M_f (kN·m)	
	84.17	164.62	
	V_r (kN)	M_{r1} (kN·m)	M_{r2} (kN·m)
	111.72	218.83	220.43
Beam B	V_f (kN)	M_f (kN·m)	
	119.83	234.36	
	V_r (kN)	M_{r1} (kN·m)	M_{r2} (kN·m)
	127.68	282.03	285.4

As shown in the table above, the shear most often governed over the bending. One of the factors that took some time to calculate was the lateral stability factor, K_L. Because of the beams being cantilevered with a uniform load, the effective length I had to use was 1.23 times the unsupported length for the cantilever and 1.92 times the unsupported length for the midspan. This resulted in my K_L factor being less than 1.0.

When finding the maximum deflection caused by load on the beams, I used superposition of deflection from rotation at the supports in addition to deflection caused by the cantilever itself. This more than doubled the deflection that would have otherwise just been caused by a simple cantilever.

The maximum deflection was approximately 40mm at the cantilever end, while the allowable deflection was approximately 44mm.

2.3.3 Column Axial Design

To find the axial load on each of the six HSS columns, I calculated the factored dead and snow loads for each tributary area above columns A and B. I also added on the weight of the beam into the dead load calculation for the columns.

Since the point load on each column was vertical and the column was at an angle, I resolved the factored point load into an axial force that aligned with the column. This force was slightly higher than my factored point load. I calculated P_{AC} to be 122.80 kN and P_{BC} to be 174.01 kN.

Using these values, I checked an HSS 152 x 152 x 6.35 for local buckling and flexural buckling. I found the local buckling to be acceptable within the range specified by Table 1, CSA S16-14 (CSA Group, 2015). The C_r for flexural buckling was also acceptable, with values of 285.91 kN and 308.12 for Columns A and B, respectively.

2.4 LATERAL SYSTEM ANALYSIS

Considering the lateral loads in my structure involved analyzing the wind load on the diaphragm of the roof and the subsequent moment resisted by the columns. Using these loads, I designed the nailing in the panel sheathing, and checked the columns for lateral loads as well as the P-Δ effect.

2.4.1 Roof Diaphragm Design

To find the shear induced in the panel and subsequently resisted by the beams below, I drew a shear-force diagram with the panel as a rigid body. I then found the reactions at each beam line which I used later for the column design. A sample of this calculation is shown in the figure below.

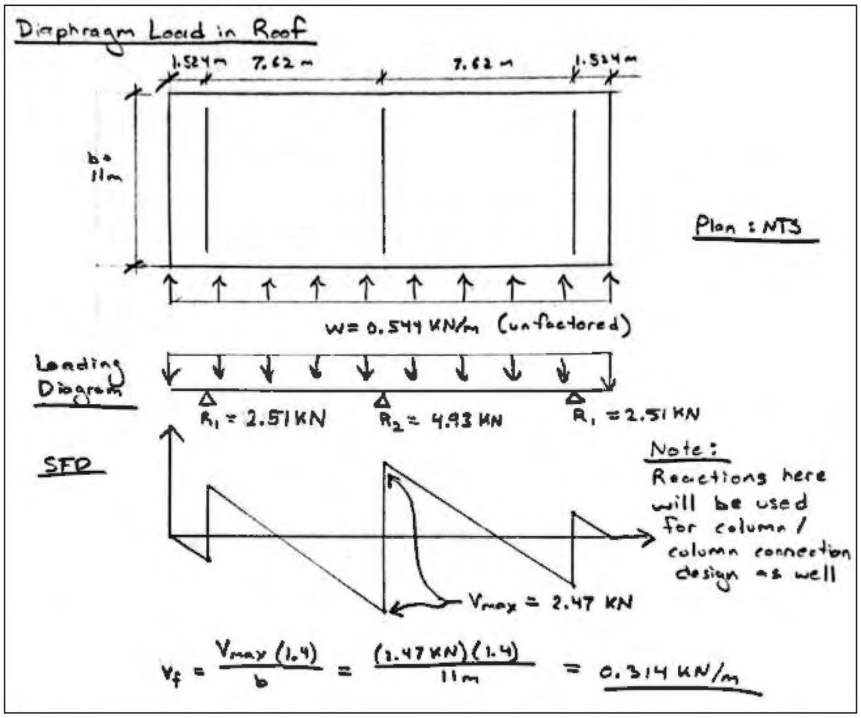


Figure 5: Calculation of Shear in Diaphragm

The maximum shear I found to be 2.47 kN, which I then factored and divided by the width of the structure. Therefore, I found the shear resisted by the diaphragm, v_f, to be 0.314 kN/m.

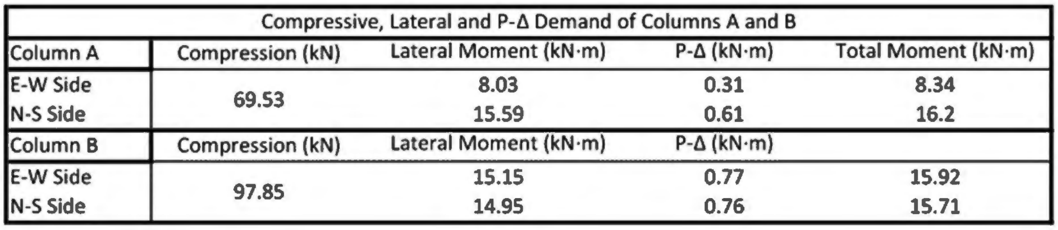
To select the nailing and the plywood sizes, I used the formula for v_{rd} found on pg. 572 of the Wood Design Manual, 2017 (CWC, 2017). The modification factors listed involved checking several cases of n_u, the unit lateral strength resistance per shear plane. From section 12.9.3.2 in the CSA 086-19 (CSA Group, 2021), I calculated items a, b, d, e, f and g with an assumption of 2-1/2" nails at 150mm o.c.

I then found v_{rd} to be 4.53 kN/m which was much greater than the 0.314 required. Therefore, the 2-1/2" nails at 150mm o.c. with ½" plywood was acceptable.

2.4.2 Column Bending Design

To design the columns for bending from wind loads, I found the factored compressive demand (kN), the factored lateral demand (kN·m), and the P-Δ demand (kN·m). These all were factored using Case 4 of the NBCC (1.25D + 0.5S, 1.4W). The demands are shown in the table below:

Table 5:Demand for Column in Bending

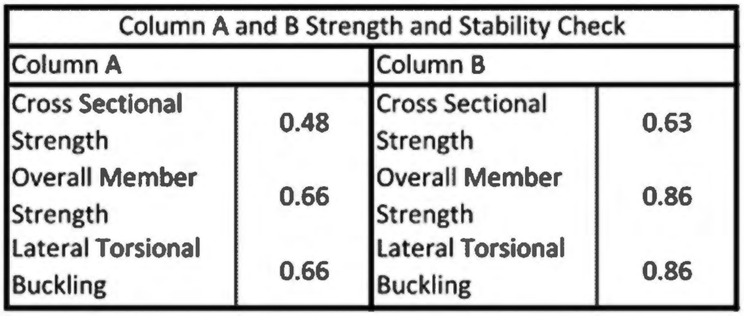


Compressive, Lateral and P-Δ Demand of Columns A and B				
Column A	Compression (kN)	Lateral Moment (kN·m)	P-Δ (kN·m)	Total Moment (kN·m)
E-W Side	69.53	8.03	0.31	8.34
N-S Side		15.59	0.61	16.2
Column B	Compression (kN)	Lateral Moment (kN·m)	P-Δ (kN·m)	
E-W Side	97.85	15.15	0.77	15.92
N-S Side		14.95	0.76	15.71

Using the compression and moments calculated above, I checked an HSS 152x152x6.35 column for the class of section, cross sectional strength, overall member strength, and lateral torsional buckling. I used the interaction formula from S16-14, 13.8.3 (CSA Group, 2015) where the M_{rx} and M_{ry} values were equal for a square HSS section.

For the class of section, the member I selected was a Class 1 section. Below is a table with the calculated values from the interaction equation, all less than 1.0.

Table 6: Column Strength and Stability Check



Column A and B Strength and Stability Check			
Column A		Column B	
Cross Sectional Strength	0.48	Cross Sectional Strength	0.63
Overall Member Strength	0.66	Overall Member Strength	0.86
Lateral Torsional Buckling	0.66	Lateral Torsional Buckling	0.86

With the results above being well under 1.0, the member size I chose was more than capable of carrying the loads. However, discussing with my industry sponsor, the wind load I calculated may have been on the low side. Therefore, I increased the member size to account for this.

The overall member strength and lateral torsional buckling also are the same value. Looking into it, the M_{rx} and M_{ry} for each are the same value since the square HSS has no tendency to buckle laterally. As a result, M_r is equal to ϕM_p.

2.5 SEISMIC ANALYSIS

As discussed previously, I did an approximate seismic analysis on the columns, connections, and footings to provide more accurate sizes for these components of my design. For the columns, I performed a strength and stability check, similar to the one done previously but with seismic loads. For the connections, I designed the lag screws in the roof and the bolts and plates for the beam to column connection. I then designed the footing size for overturn.

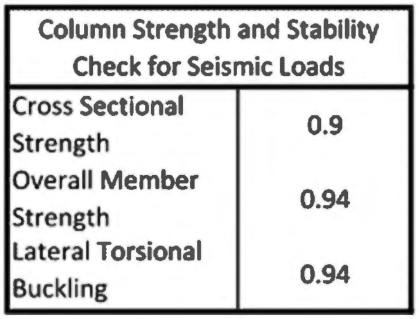
2.5.1 Column Seismic Design

Using the previously calculated value of 0.47 kPa for the total seismic load, I multiplied by the roof area and divided by six columns to find a factored lateral demand of 15.79 kN per column. I distributed this evenly among all columns since the roof is treated as rigid and all columns receive the same load.

To find the compressive demand of the column under seismic conditions, I used Case 5 of the NBCC to find P_C was 66.86 kN. I did not consider Column A and B separate from each other since they received the same loads. M_{fx} and M_{fy} also were each 69.25 kN·m.

I selected an HSS 203x203x9.5 column to check the seismic demand with. The class of section was Class 1, and the following table shows the results for the strength and stability checks.

Table 7: Column Strength and Stability Check for Seismic Loads



Column Strength and Stability Check for Seismic Loads	
Cross Sectional Strength	0.9
Overall Member Strength	0.94
Lateral Torsional Buckling	0.94

The strength and stability check returned values much closer to 1.0, emphasizing how the seismic load governs over the wind and gravity loads for the columns.

2.5.2 Connection Design

For designing the panel to beam connection, I found the factored seismic demand per panel. Using the previously calculated value of 0.47 kPa, I found the shear demand in each line of beams to be 31.52 kN. Dividing this among six panels resting on one beam, I found Q_f equal to 5.25 kN.

Using the equation from Wood Design Manual 2017, pg. 460 (CWC, 2017), I calculated the required diameter and length of lag screws. I used an n_R factor of 2 for two rows, and a n_{Fe} factor of 2.2 for the effective number of fasteners in a row. I also assume Q_r' to be 1.34 kN for a worst-case scenario.

The final value of Q_r that I calculated was 5.90 kN which was greater than the 5.25 kN required. The required length from penetration plus panel depth, I determined to be 294mm. Therefore, I specified two rows of 13mmϕ x 300mm long lag screws at 300mm o.c. Because the 6-foot-wide panels are wide enough to have 5 screws in each row of fasteners at this spacing, this will provide an extra factor of safety.

To design the beam to column connection, I used a double plate on either side of the beam with two bolts running through. The seismic load then is transferred from the beam into the plates as shown in the figure below.

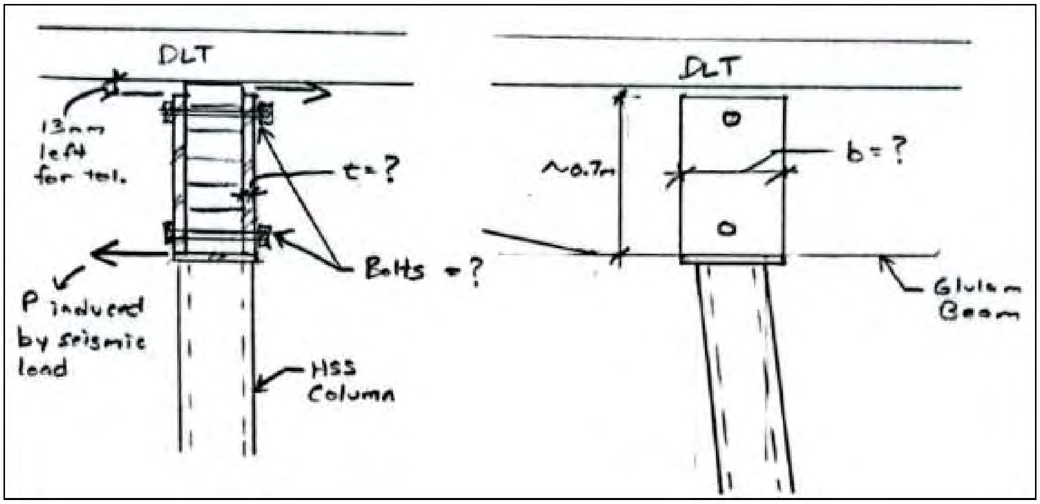


Figure 6: Design of Beam to Column Connection

The force induced by the seismic load in the plate can then be treated like a force on the end of a fixed column. To find the seismic demand, I calculated the P for each plate to be ½ of the 15.79 kN calculated earlier. With a conservative estimate of 0.7m for the length of the plate, the Mf was found to be 5.53 kN·m.

Using S16-14, 13.5 (CSA Group, 2015), I calculated Mr to be equal to ϕZF_y. As shown in Appendix B, I found the dimensions of the plate required to be a 200x700x19mm plate.

Since there are two bolts in each connection, I calculated that each bolt must resist 7.9 kN. Using CISC S16 Steel Handbook Table 3-4 and 3-6 (CISC, 2010), I selected a 1"ϕ A325 bolt with more than adequate bearing and double shear resistance.

2.5.3 Footing Design

Although not familiar with footing design yet, I researched and had guidance from my industry sponsor in calculating the required footing size. Below is a diagram of the footing I designed.

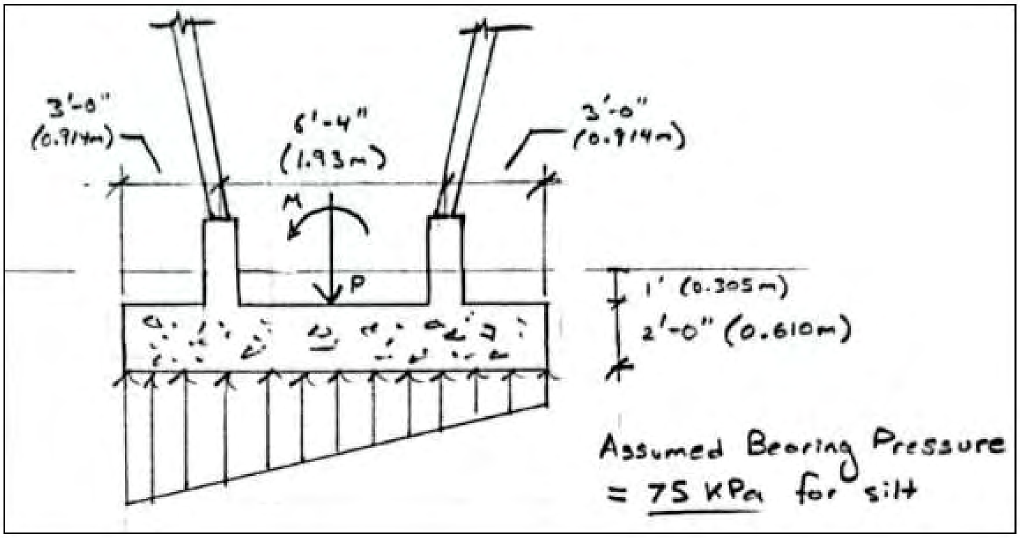


Figure 7: Design of Footings

As shown, the composition of the ground for White Rock is silt and clay, giving a bearing pressure of approximately 75 kPa (The Ontario Building Code, n.d.). I selected a footing size of 3759 x 3048 x 610mm to check against the bearing pressure of the soil.

I calculated the total weight of the structure including the footing to be 313.2 kN. I then found the moment induced by seismic loads to be 153.4 kN·m. Using this, I found an eccentricity of 0.51m from the center of footing for my resultant force.

Using D/6, I determined that both the E-W and N-S sides of the structure would remain in full compression over the respective width of the footing. To calculate the maximum and minimum pressure on this eccentrically loaded footing, I used the following formula.

$$q = \frac{P}{A} \pm \frac{M}{S}$$

This formula gave me the maximum and minimum bearing pressures on either side of the footing. I then found the maximum pressure at the toe of the footing to be 48.7 kPa for the E-W side, and 53.7 kPa for the N-S side. These values were both under 75 kPa, so the footing worked.

3.0 3D MODEL AND DRAWINGS

As mentioned earlier, I found the 3D model invaluable when determining member sizes and iterating solutions to find the best one geometrically. I used a program called Rhino 3D for much of my preliminary design because of its versatility and ease of use. As shown in the layout drawings, I also created a parabolic shape in my 3D model to show the deflection that the panels take on towards the center of the structure.

I created my layout drawings in AutoCAD as the software is well suited from a drafting point of view. The first page of my layout set (Appendix A) shows the plan views of the foundation, columns, beams, and panels. The second page shows elevation views, while on the third page I created several high-level details.

As I was not able to design all the elements of this structure (e.g., the pedestals), I used concepts from similar projects to still display these elements where necessary on the layout set of drawings.

4.0 CONCLUSION

Because of the growing population in South Surrey, this multi-use canopy will be well used by its residents. I decided on the location of it to be between two athletic fields, so that it would maximize its accessibility. In the design process, I first analyzed initial design factors such as functionality and limitations of the canopy's components. From this I determined the canopy to be 36 ft wide by 60 ft long by over 16 ft tall.

The next step in my design process was to analyze the gravity, lateral and seismic loads that would drive the member sizing and design within the structure. I used a rough estimate for seismic loading since my knowledge is limited in that area.

I found the dead load of the DLT panels to be 1.03 kPa, and the dead load of the beams to be 5.45 kN and 6.21 kN for Beams A and B, respectively. The dead load due to snow was calculated to be 1.8 kPa. The line loads due to wind were 0.544 kN/m for the E-W side, and 0.433 kN/m for the N-S side. The seismic load was determined to be 0.47 kPa, a factor of the dead and snow loads.

I then proceeded to design the members for gravity, lateral and seismic loads. The seismic loads governed in terms of column size, connections, and the footing design. The 2x8 DLT roof panels with ½" sheathing and 2 ½" nails were acceptable for the loads on the structure. I determined member sizes of 175x532 for Beam A, and 175x608 for Beam B. The columns selected were HSS 203x203x9.5, and the footings I found to be 3749 x 3048 x 610mm.

I also designed the panel to beam connections and the beam to column connection. For the panel to beam connection, I used two rows of 13mmϕ x 300mm long lag screws at 300mm o.c. For the beam to column connection, I designed found the two side plates to be 200x700x19mm, and the two bolts running through them to be 1" A325 bolts.

REFERENCES

ASCE/SEI. (2017). *ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* American Society of Civil Engineers.

CISC. (2010). *Handbook of Steel Construction.* Canadian Institute of Steel Construction.

City of Surrey. (n.d.). *Population Estimates & Projections*. Retrieved April 20, 2022, from https://www.surrey.ca/business-economy/business-data/population-estimates-projections

CSA Group. (2015). *CSA S16-14 Design of Steel Structures.* Canadian Standards Association.

CSA Group. (2021). *CSA 086-19 Engineering design in Wood.* Canadian Standards Association.

CWC. (2017). *Wood Design Manual.* Candian Wood Council.

Google. (n.d.). *[South Surrey Athletic Park]*. Retrieved November 13, 2021, from https://goo.gl/maps/ByAUSNUotvzvDs4X8

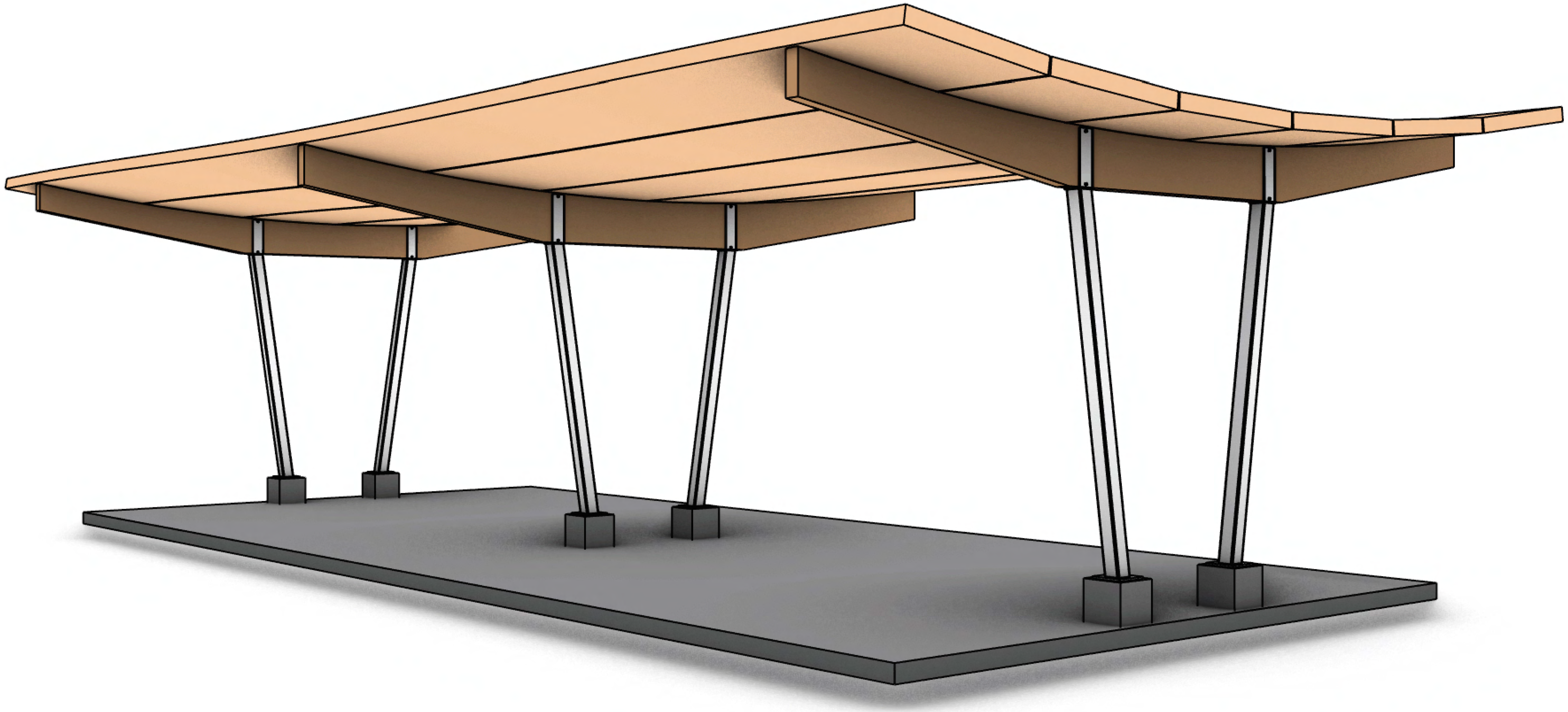
NRCC. (2018). *National Building Code of Canada 2015.* Retrieved from Government of Canada: https://doi.org/10.4224/40002005

SportSystems Canada. (2009). *5-Row Outdoor Bleachers*. Retrieved from SportSystems Canada: https://www.sportsystemscanada.com/collections/outdoor-bleachers/products/5-row-outdoor-bleachers

The Ontario Building Code. (n.d.). *Allowable Bearing Pressures*. Retrieved April 20, 2022, from BuidlingCode.Online: https://www.buildingcode.online/1253.html

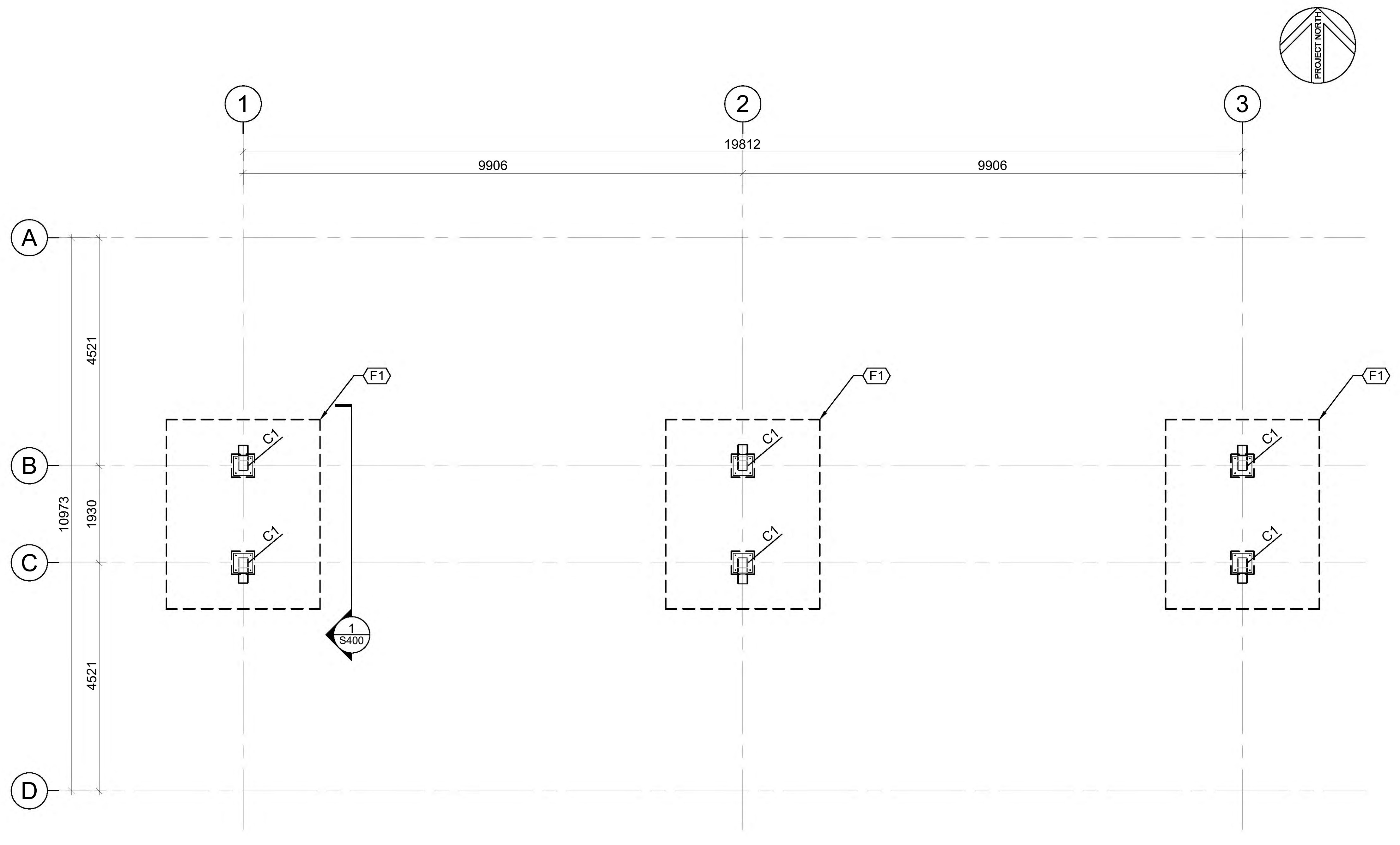
USA Shade. (n.d.). Ontario Soccer Park. Retrieved from https://www.usa-shade.com/projects/ontario-soccer-park

Appendix A: Structural Layout Drawings

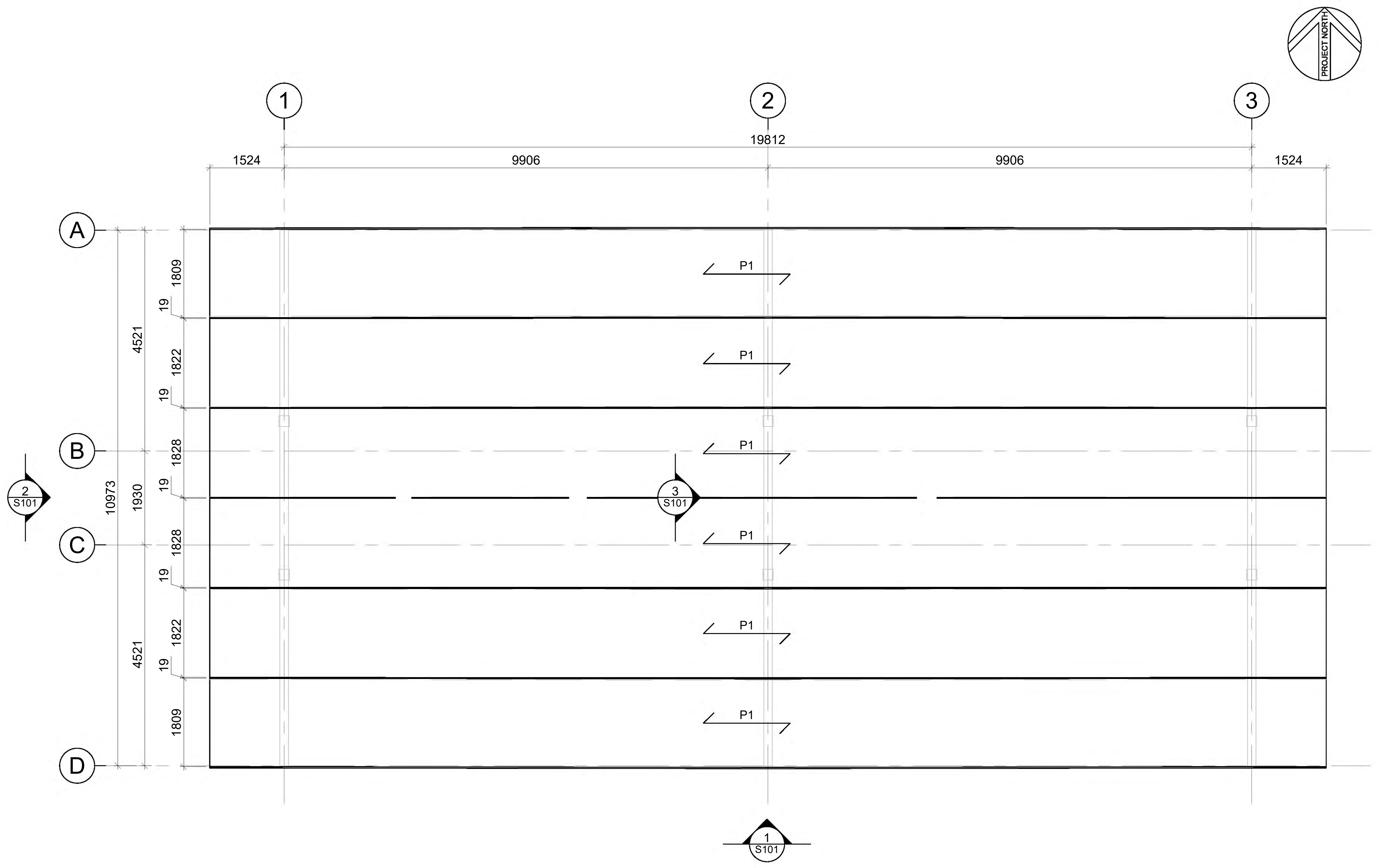


SHEET LIST TABLE				
SHEET #	SHEET TITLE	REV #	ISSUED FOR	DATE
S000	COVER PAGE	0	--	2022-04-20
S100	FOUNDATION & STRUCTURAL PLAN	0	--	2022-04-20
S101	ELEVATIONS	0	--	2022-04-20
S400	DETAILS	0	--	2022-04-20

22-13 - SSAP CANOPY



1 FOUNDATION & COLUMN LAYOUT
1:75



3 DLT LAYOUT - PLAN VIEW
1:50

2 BEAM LAYOUT
1:50

FOOTINGS		
TYPE	SIZE	REINFORCEMENT
F1	3759mm x 3048mm x 610mm DP. SPREAD FOOTING	NOT IN DESIGN

STEEL COLUMNS			
MARK	SIZE	MATERIAL	GRADE
C1	HSS8x8x0.375	STEEL	350W

GLULAM BEAMS					
MARK	WIDTH(mm)	DEPTH(mm)	GRADE	MATERIAL	CAMBER
B1	130	532	GL24f-EX	D.FIR	R = 31595mm
B2	130	608	GL24f-EX	D.FIR	R = 31595mm

DLT SCHEDULE					
TYPE	LAMINATION SIZE	GRADE	MATERIAL	SHEATHING	SHEATHING NAILING
P1	2x8	SS	D.FIR	1/2" THICK PLY	EDGE NAILING @ 150 O/C AND FIELD NAILING @ 300 O/C

NOT FOR CONSTRUCTION

REV.	DESCRIPTION	DATE
0	----	2022-04-20

ISSUED

ARCHITECT
N/A

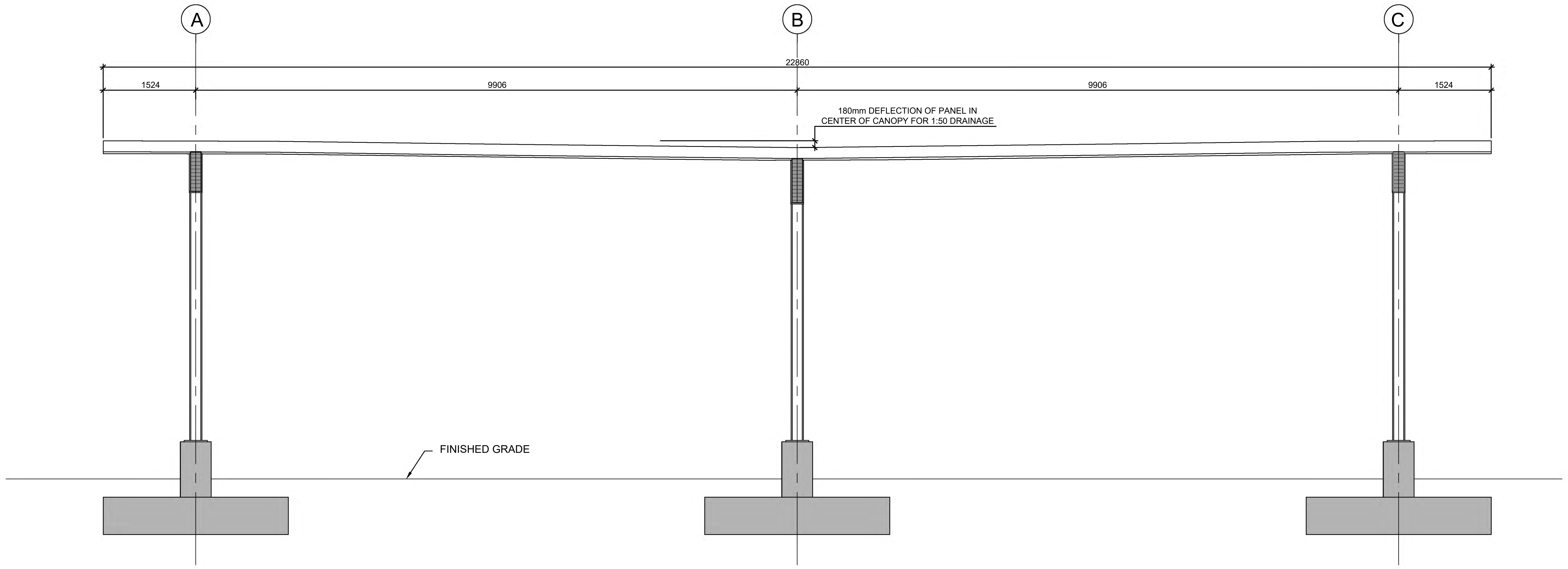
CLIENT
N/A

PROJECT
SSAP CANOPY

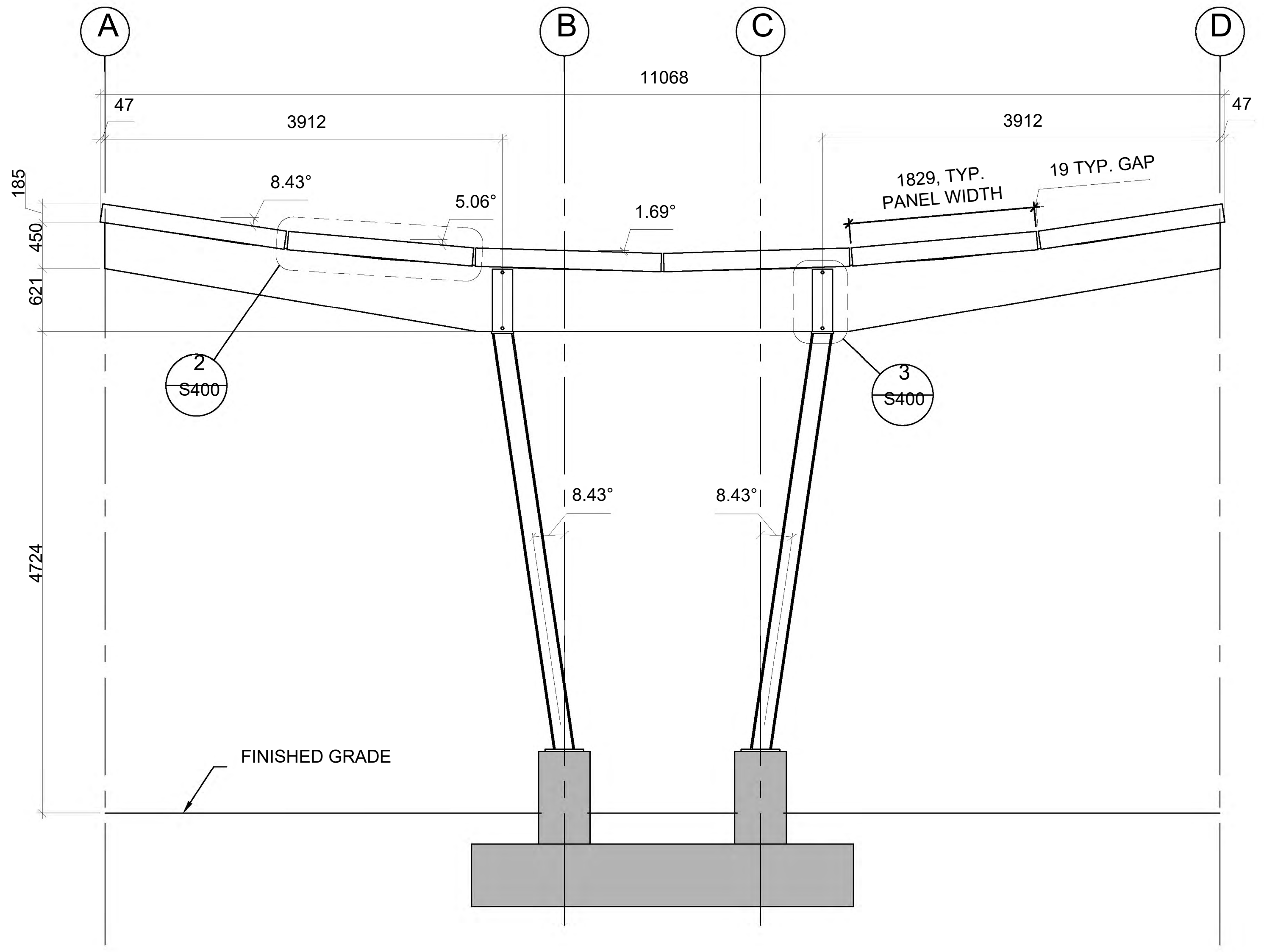
TITLE
FOUNDATION & STRUCTURAL PLAN

DRAWN:
CHECKED:
DATE:
JOB No. 22-13
REVISION 0

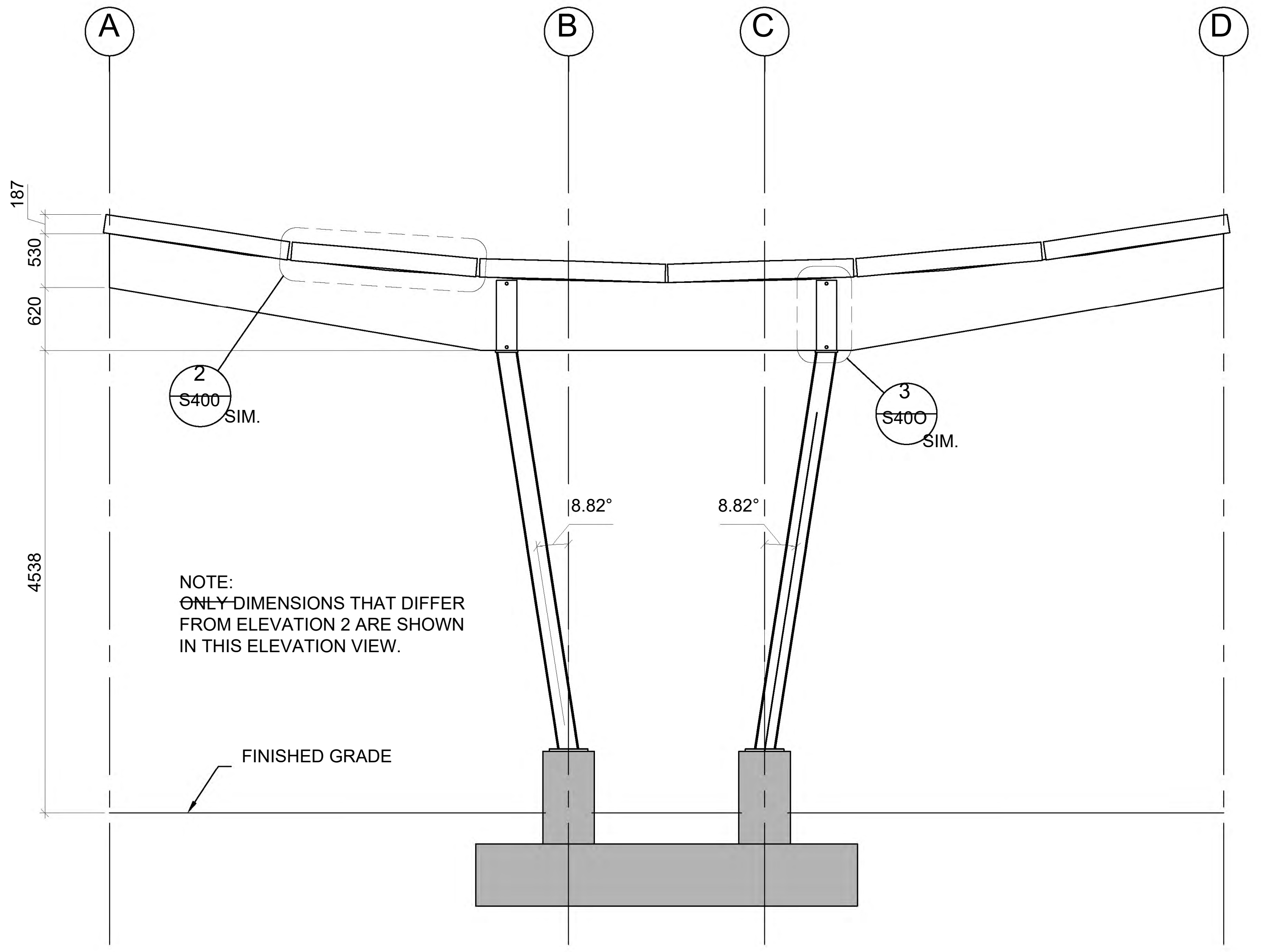
DWG. No.
S100



1 SOUTH ELEVATION
1:50



2 WEST ELEVATION - OUTER BEAM
1:50



NOTE:
~~ONLY~~ DIMENSIONS THAT DIFFER FROM ELEVATION 2 ARE SHOWN IN THIS ELEVATION VIEW.

3 WEST ELEVATION - CENTER BEAM
1:50

NOT FOR CONSTRUCTION

REV.	DESCRIPTION	DATE
0	----	2022-04-20

ISSUED

ARCHITECT
N/A

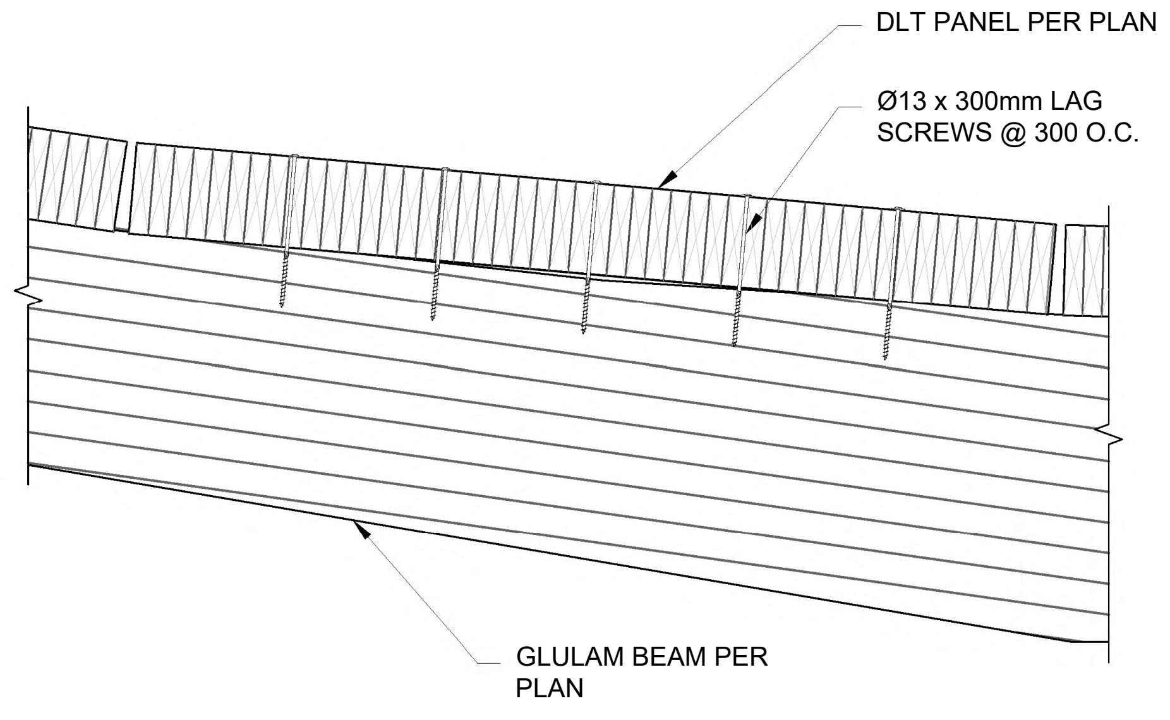
CLIENT
N/A

PROJECT
SSAP CANOPY

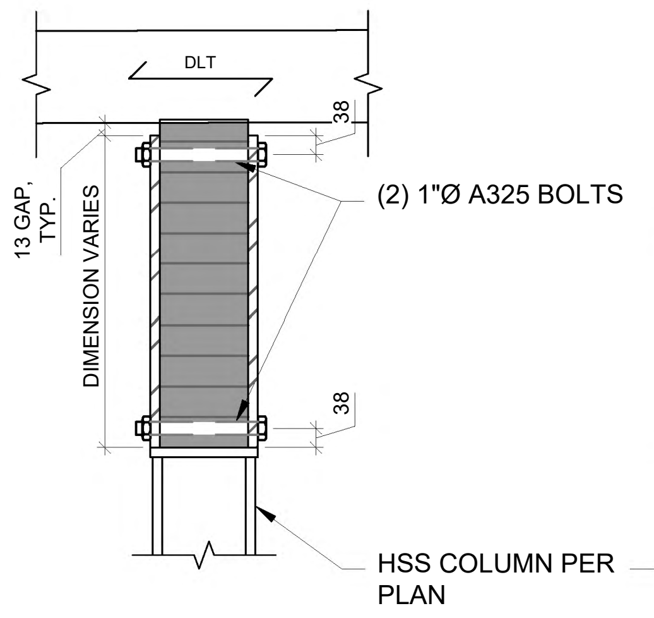
TITLE
ELEVATIONS

DRAWN:
CHECKED:
DATE:
JOB No. 22-13
REVISION 0

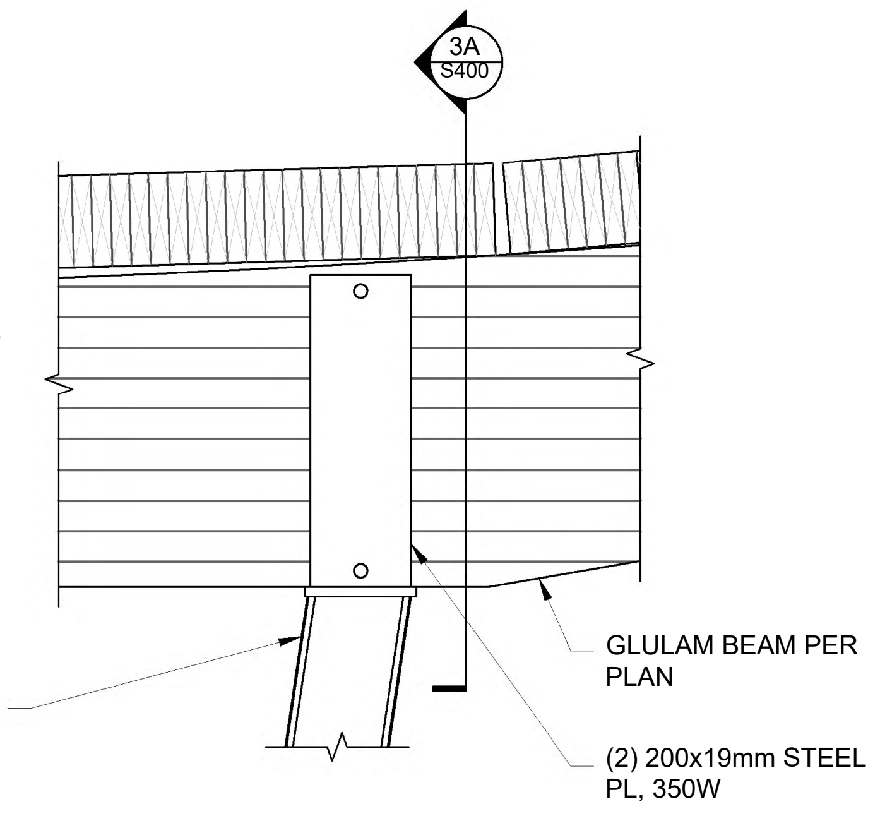
DWG. No.
S101



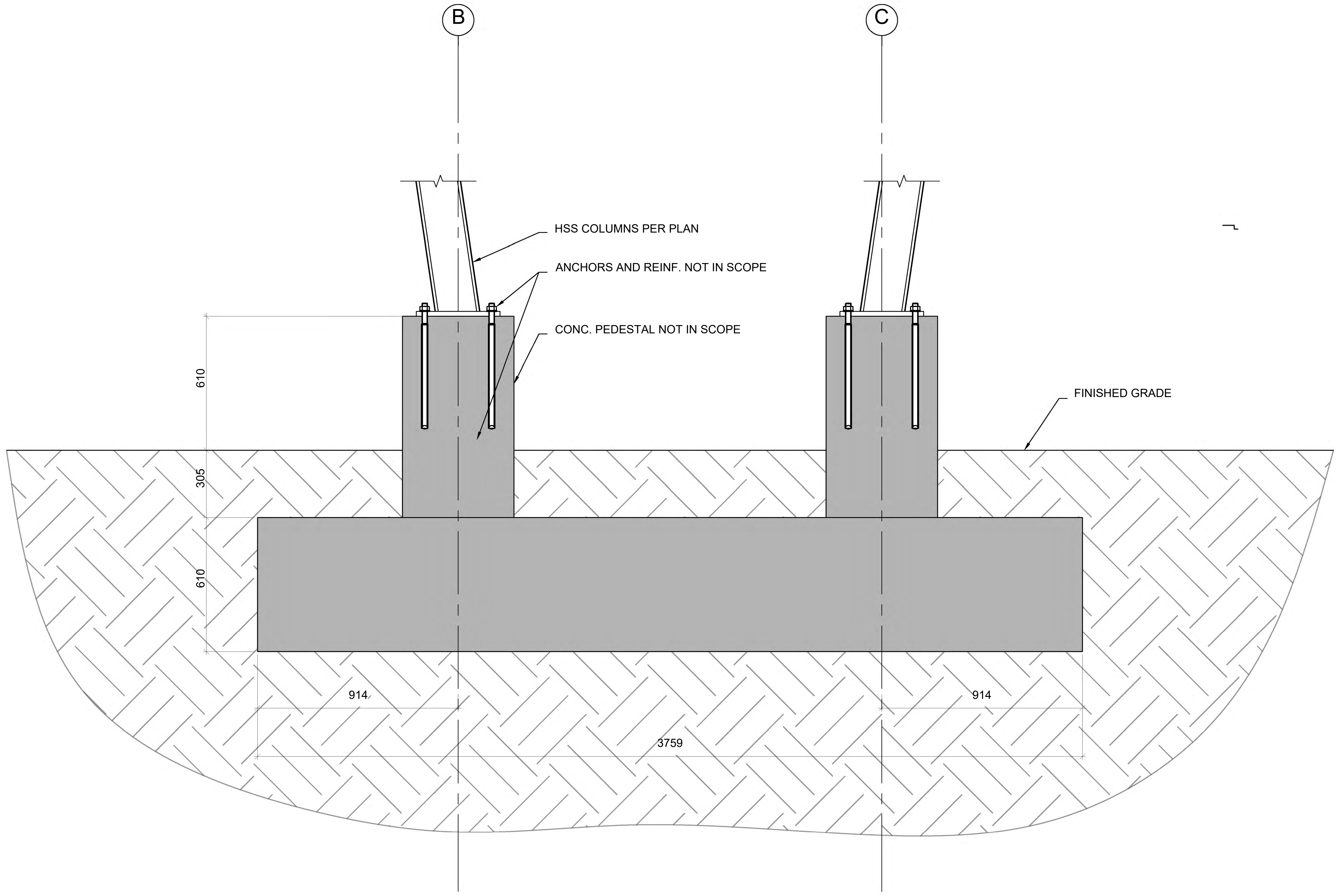
2 PANEL TO BEAM CONN.
1:15



SECTION 3A



3 BEAM TO COLUMN CONN.
1:15



1 TYPICAL FOOTING DETAIL
1:15

NOT FOR CONSTRUCTION

REV.	DESCRIPTION	DATE
0	----	2022-04-20

ISSUED

ARCHITECT
N/A

CLIENT
N/A

PROJECT
SSAP CANOPY

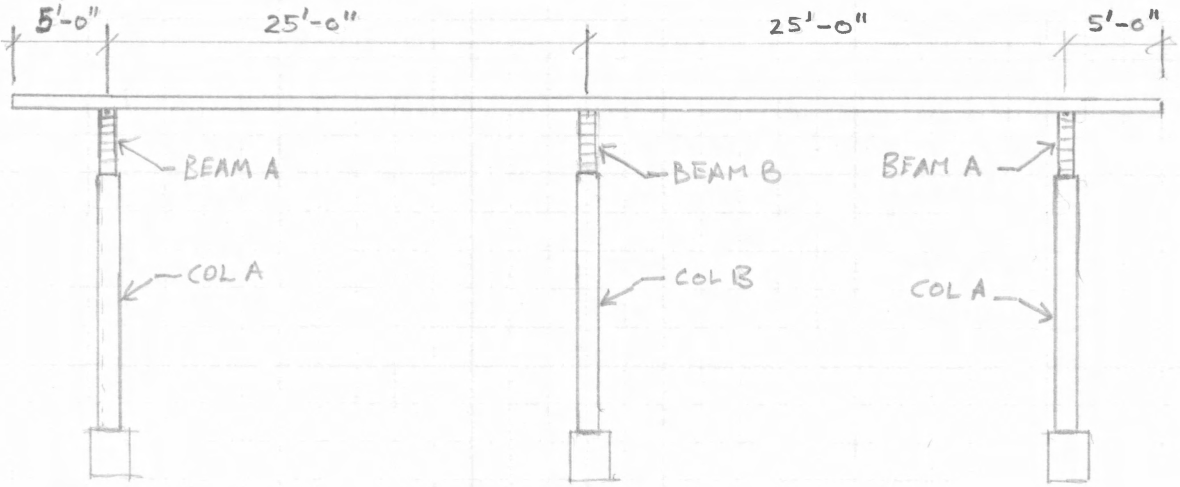
TITLE
DETAILS

DRAWN:
CHECKED:
DATE:
JOB No. 22-13
REVISION 0
DWG. No.
S400

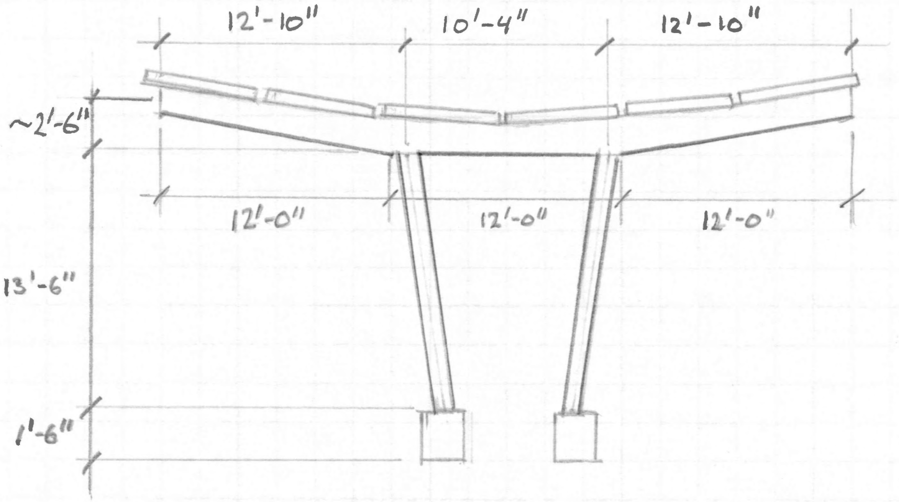
Appendix B: Hand Calculations

List of Contents:

Industry Project | Geometry Layout | Apr. 16/22 | A-1/1



ELEVATION E-W 1 sq = 2'-0"



ELEVATION N-S 1 sq = 2'-0"

Dead Load

DLT Roof Panels

- Density of D. Fir ≈ 33 lb/ft^3

Using 2x8 DLT, Volume of DLT $= 180\,mm \times \frac{1\,in}{25.4\,mm} \times \frac{1\,ft}{12\,in} \times 60\,ft \times 36\,ft$
$= 1276\ ft^3$

∴ Weight of DLT $= 33\ lb/ft^3 \times 1276\ ft^3$
$= \underline{42{,}108\ lb}$

- Density of Max 3/4" Ply = 2 lb/ft^2

Area of Ply $= 60\,ft \times 36\,ft$
$= 2160\ ft^2$

∴ Weight of Ply $= 2\ lb/ft^2 \times 2160\ ft^2$
$= \underline{4320\ lb}$

Combined Ply and DLT Load over Roof
$= (42{,}108\ lb + 4320\ lb) / (60\ ft \times 36\ ft)$
$= \underline{21.5\ lb/ft^2 \quad or \quad 1.03\ KPa}$

Beams

- Density of D. Fir Glulam ≈ 33 lb/ft^3

Beam A

Approx. Member Size : 175 × 532 mm

Volume of Member = 37.1 ft^3 (Measured in model)

∴ Weight of Member = $\underline{1225\ lb\ or\ 5.45\ KN}$

Beam B

Approx. Member Size : 175 × 608

Volume of Member = 42.25 ft^3

∴ Weight of Member = $\underline{1395\ lb\ or\ 6.21\ KN}$

Snow Load

$S = I_s [S_s(C_b C_w C_s C_a) + S_r]$ ⟶ NBCC, 4.1.6.2

where $I_s = 1.0$ for normal importance

$S_s = 2.0$, from Appendix C, Table C-2 for White Rock

$C_b = 0.8$, for $L_c \leq \frac{70}{C_w^2}$, where $L_c = 2w - \frac{w^2}{L}$

$= 2(36') - \frac{(36')^2}{(60')}$

$15.4 < 70$ ✓

$= 50.4$ ft. or 15.4m

$C_w = 1.0$

$C_s = 1.0$ for roof slope less than 30°

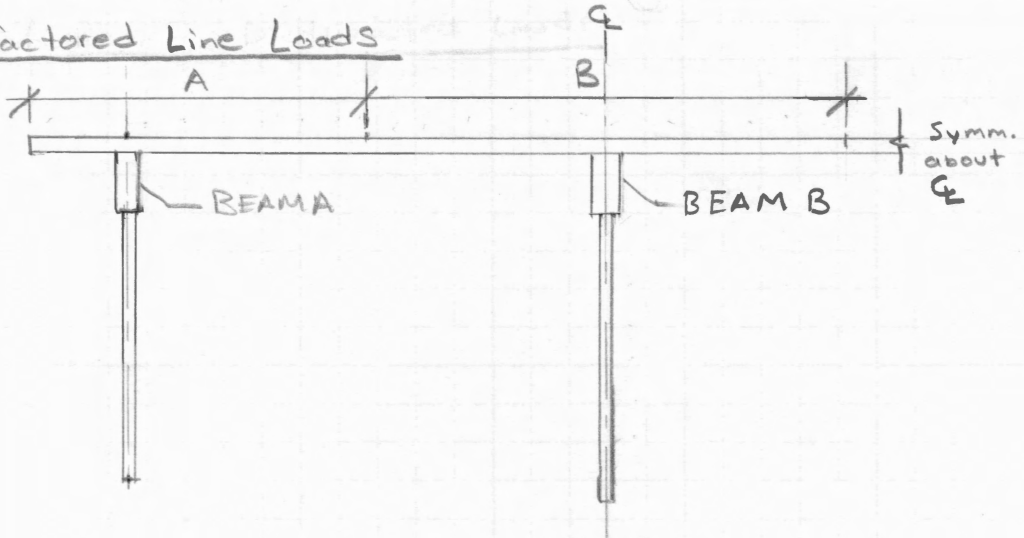
$C_a = 1.0$ for very shallow comber

$S_r = 0.2$, from Appendix C, Table C-2, White Rock

$S = 1.0[2.0(0.8 \times 1.0 \times 1.0 \times 1.0) + 0.2]$

$= \underline{1.8 \text{ kPa} \text{ or } 37.6 \text{ lb/ft}^2}$

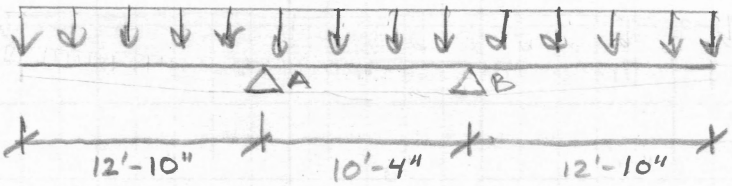
Unfactored Line Loads



Trib Area A

Width = 17'-6"

→ Load on Glulam Beam (Treating beam as straight element)



Dead & Snow

D: 21.5 lb/ft²(17'-6") + 495 lb/36'-0" = 390 lb/ft OR 5.69 kN/m

S: 37.6 lb/ft² (17'-6") = 658 lb/ft OR 9.60 KN/m

Trib Area B

Width = 25'-0"

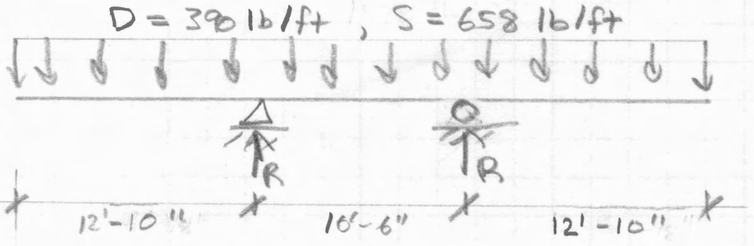
→ Load on Glulam Beam

Dead & Snow

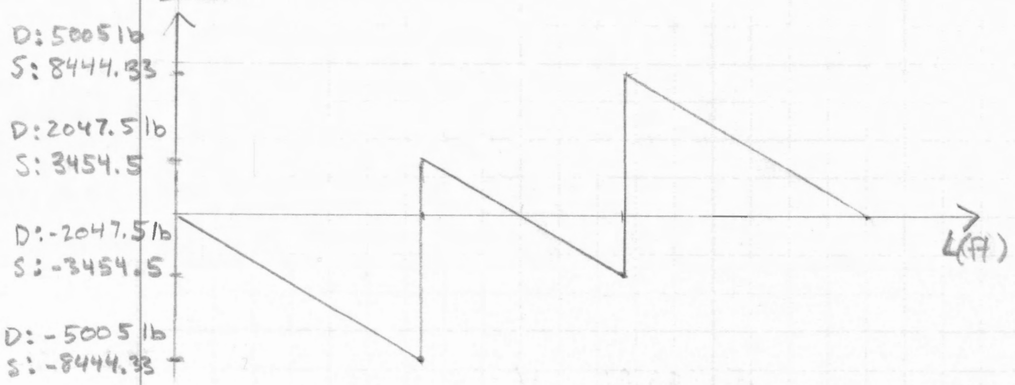
D: 21.5 lb/ft²(25'-0") + 495 lb/36'-0" = 551.3 lb/ft OR 8.05 KN/m

S: 37.6 lb/ft²(25'-0") = 940 lb/ft OR 13.72 KN/m

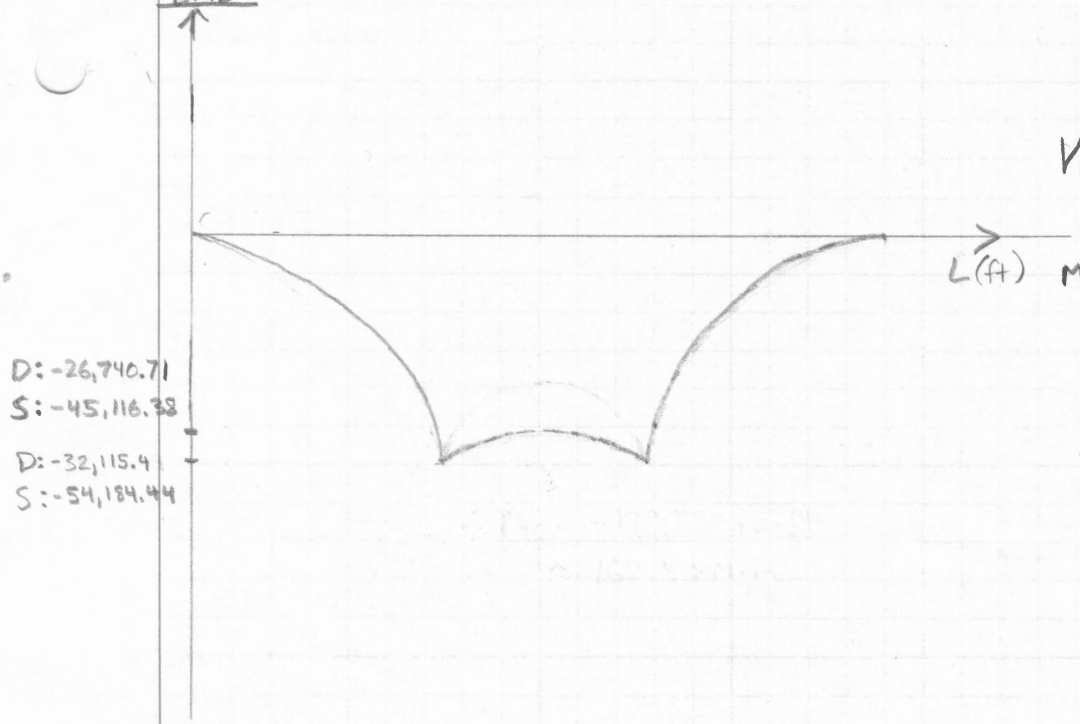
Beam A - Unfactored Demand



SFD



BMD



Dead Load

$$R = \frac{(36'-2" \times 390 \text{ lb/ft})}{2}$$

$= 7052.5$ lb

$= 31.37$ kN

$V_{max} = 5005$ lb

$= 22.26$ kN

$M_{max} = -32,115.4$ lb·ft

$= -43.54$ kN·m

Snow Load

$$R = \frac{(36'-2" \times 658 \text{ lb/ft})}{2}$$

$= 11898.83$ lb

$= 52.93$ kN

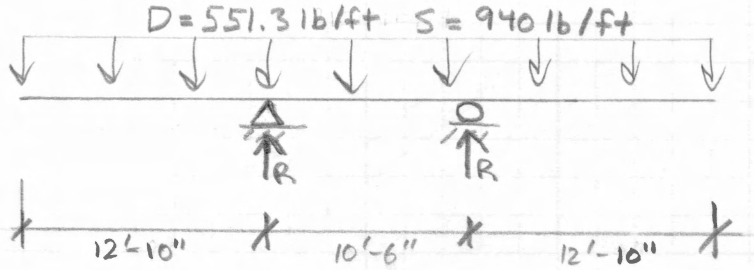
$V_{max} = 8444.33$ lb

$= 37.56$ kN

$M_{max} = -54,184.44$ lb·ft

$= -73.46$ kN·m

Beam B - V_f and M_f



Dead Load

$R = \frac{(551.3 \text{ lb/ft} \times 36'\text{-}2'')}{2}$

= 9969.3 lb

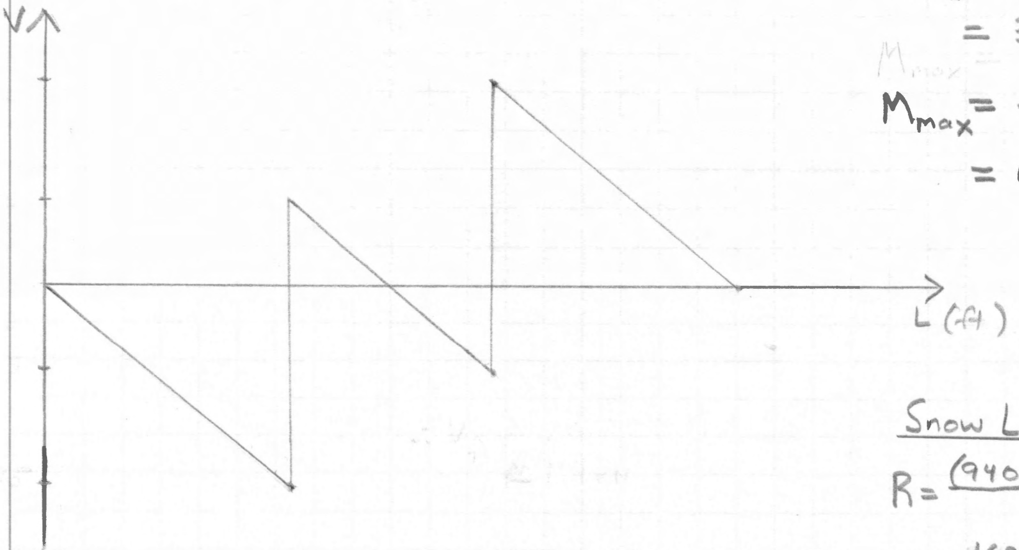
= 44.35 kN

V_{max} = 7075 lb

= 31.47 kN

M_{max} = -45,397.8 lb·ft

= 61.55 kN·m



Snow Load

$R = \frac{(940 \text{ lb/ft} \times 36'\text{-}2'')}{2}$

= 16998.3 lb

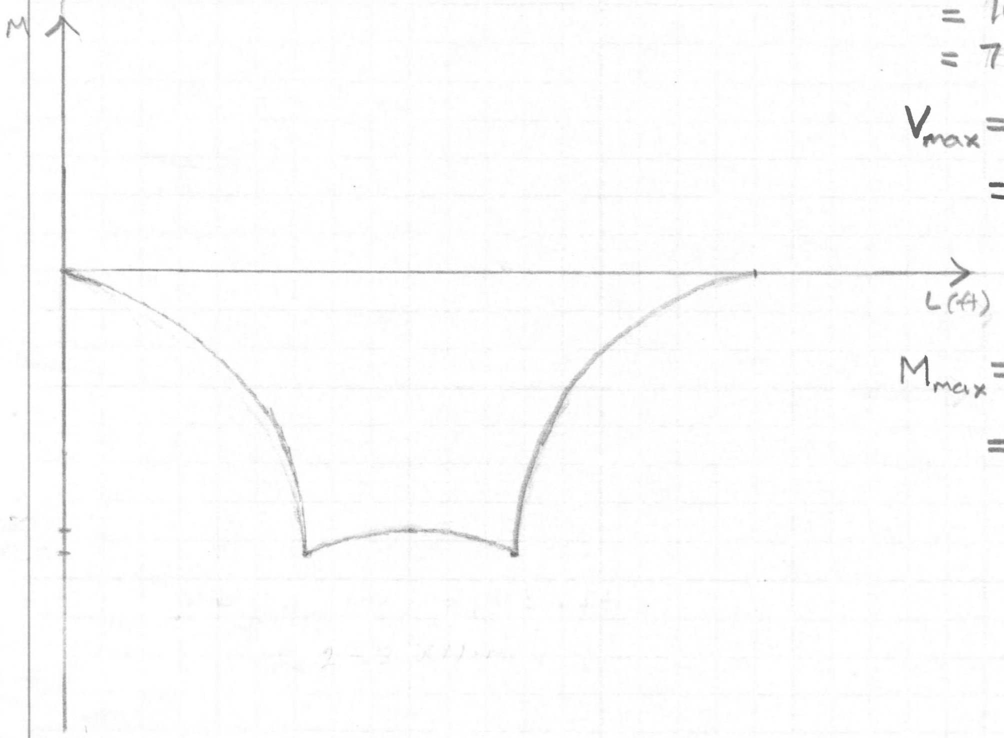
= 75.61 kN

V_{max} = 12063.3 lb

= 53.66 kN

M_{max} = -77406 lb·ft

= -104.95 kN·m



Beam A - M_r, V_r

- Factored Demand : $V_f = 1.25D + 1.5S = 1.25(22.26) + 1.5(37.56)$
  $= 84.17 \text{ kN}$
  $M_f = 1.25D + 1.5S = 1.25(43.54) + 1.5(73.46)$
  $= 164.62 \text{ kN·m}$

- Beam Modification Assumptions :

  1) The shallow camber of the beam will generally be ignored, and the beam will be treated as straight. I have still calculated K_x for M_{r1} and M_{r2}, but these have little effect.

  2) The beams are significantly cut on the bottom with bevel cuts. Since these will impact the Moment of Inertia of a selected size, I have increased the beam size to account for this decrease in capacity. The decrease in section height only occurs towards the ends, which demand less bending and shear resistance. As a result, the section size at any point will be capable to resist the shear and bending at that point.

- From WDM 2017, pg. 69 → Use GL D. Fir 24f-EX 175 × 532

Note: The actual section size will be a 175×646 cut down with a minimum 175×532 cross section at points of max shear and bending

Bending

(Tension) $M_{r1} = \varphi f_b (K_D K_H K_{Sb} K_T) S K_{zbg} K_x$

where: $\varphi = 0.9$

$f_b = 30.6 \text{ MPa}$

$K_D, K_H, K_{Sb}, K_T = 1.0$

$S = \frac{I}{y}$, where $I = \frac{bh^3}{12} = \frac{175(532)^3}{12} = 2.196 \times 10^9 \text{ mm}^4$

$y = \frac{532}{2} = 266 \text{mm}$

$= \frac{2.196 \times 10^9 \text{mm}^4}{266 \text{ mm}}$

$= 8.255 \times 10^6 \text{ mm}^3$ or 0.00825m^3

$K_{zbg} = \left(\frac{130}{175}\right)^{1/10} \left(\frac{610}{532}\right)^{1/10} \left(\frac{9100}{11028}\right)^{1/10} \leq 1.3$

$= 0.965 < 1.3$

Beam A (cont'd)

(Tension cont'd) $K_x = 1 - 2000\left(\frac{t}{R}\right)^2$

where: $t \approx 38\,mm$ lams

$R = 31595\,mm$

$= 1 - 2000\left(\frac{38}{31595}\right)$

$= 0.997$

$M_{r_1} = (0.9)(30{,}600\,kPa)(0.008255\,m^3)(0.965)(0.997)$

$= 218.73\ kN\cdot m > 164.62\ kN\cdot m$ ✓ Acceptable

(Compression) $M_{r_2} = \varphi f_b (K_D K_H K_{Sb} K_T) S K_L K_x$

where: $\varphi = 0.9$

$f_b = 30.6\ MPa$

$K_D, K_H, K_{Sb}, K_T = 1.0$

$S = 0.00825\,m^3$

$K_L \longrightarrow C_B = \sqrt{\frac{L_e(d)}{b^2}}$ where: $\underbrace{L_e = 1.23\,\ell_u}^{\text{cantilever}}$ or $\underbrace{L_e = 1.92\,\ell_u}^{\text{midspan}}$

(Table 7.4) $= 1.23(3912)$ $\quad = 1.92(3150)$

$= 4812\,mm$ $\quad = \underline{6048\,mm}$ governs

$= \sqrt{\frac{(6048)(532)}{175^2}}$

$= 10.25 > 10$

$K_L = 1 - \frac{1}{3}\left(\frac{C_B}{C_K}\right)^4 = 1 - \frac{1}{3}\left(\frac{10.25}{19.11}\right)^4$ where $C_K = \sqrt{\frac{0.97 E K_{SE} K_T}{F_b}}$

$= 0.972$

$= \sqrt{\frac{0.97(12800)(0.9)(1)}{30.6}}$

$K_x = 0.997$

$= 19.11$

$M_{r2} = (0.9)(30{,}600\,kPa)(0.008255\,m^3)(0.972)(0.997)$

$= 220.31\ kN\cdot m > 164.62\ kN\cdot m$ ✓ Acceptable

Shear

Volume of Beam $= 1.05\,m^3 < 2\,m^3$, Therefore V_r can be used

$V_r = \varphi f_v (K_D K_H K_{SV} K_T)\, ^2/_3\, A_g$

where: $\varphi = 0.9$

$f_v = 2\ MPa$

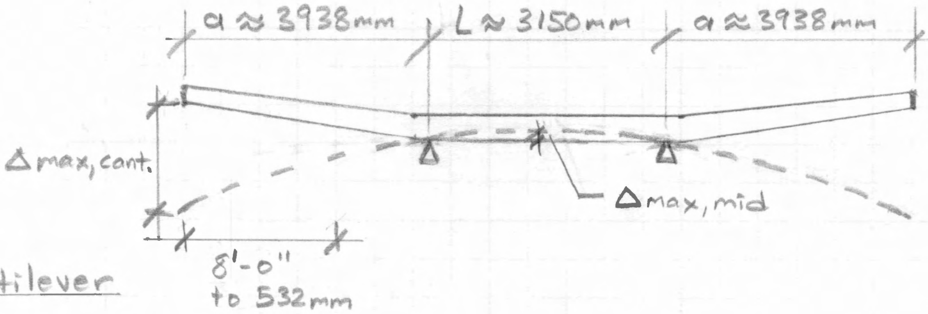
$K_D, K_H\ K_{SV}, K_T = 1.0$

$A_g = 0.175 \times 0.532 = 0.0931\,m^2$

$V_r = 0.9(2000\,kPa)(^2/_3)(0.0931\,m^2)$

$= 111.72\ kN > 84.17\ kN$ ✓ Acceptable

Beam A - Deflection



Cantilever

Effective $L = 3912\,mm$

$\Delta_{REQ} = \frac{L}{90}$ (for cantilever)

$= \frac{3938\,mm}{90} = \underline{\underline{43.47\,mm}}$

$\Delta_{cant.} = \frac{wL^4}{8E_sI}$ where $w = 15.29\,KN/m$ for total unfactored dead + live load

$E_sI = 2.81 \times 10^{13}\,N \cdot mm^2$

Note: As mentioned previously, the beam is modified in terms of its section depth. This affects the MoI that is used for deflection check as well. Considering that the beam is deeper than necessary at the supports where the $\Delta_{rotation}$ occurs, an average depth of 532mm will be used at 8'-0" (~2/3) in from the edge.

$\Delta_{cant.} = \frac{15.29\,N/mm\,(3938\,mm)^4}{8(2.81 \times 10^{13}\,N \cdot mm^2)}$

$= 16.36\,mm$

$\Delta_{rotation} = \frac{waL(6a^2 - L^2)}{24E_sI}$ → derived from equation 9-67, Goodno - Mechanics of Materials

$= \frac{15.29\,N/mm\,(3938\,mm)(3150\,mm)((6 \times 3938^2) - 3150^2)}{24(2.81 \times 10^{13}\,N \cdot mm^2)}$

$= 23.38\,mm$

Therefore $\Delta_{max,cant.} = 16.36 + 23.38$

$= \underline{39.74\,mm} < 43.47\,mm$ ✓ Acceptable

Beam A deflection cont'd...

Midspan

Effective L = 3150mm

$$\Delta_{req} = \frac{L}{180}$$
$$= \frac{3150}{180} = 17.50\text{mm}$$

$$\Delta_{mid} = \frac{5wL^4}{384 E_s I}$$
$$= \frac{5(15.29\text{N/mm})(3150\text{mm})^4}{384(2.81\times10^{13}\text{N}\cdot\text{mm}^2)}$$
$$= 0.698\text{mm}$$

$$\Delta_{rotation} = \frac{wa^2L^2}{16E_sI}$$ → derived from Table H-2, Case 10, Goodno–Mechanics of Materials

$$= \frac{(15.29\text{N/mm})(3938\text{mm})^2(3150\text{mm})^2}{16(2.81\times10^{13}\text{N}\cdot\text{mm}^2)}$$
$$= 5.23\text{mm}$$

Therefore $\Delta_{max,mid} = 5.23\text{mm} - 0.698\text{mm}$
$= 4.53\text{mm} < 17.50\text{mm}$ ✓ Acceptable

Beam B - M_r, V_r

- Factored Demand: $V_f = 1.25D + 1.5S = 1.25(31.47) + 1.5(53.66)$
  $= 119.83$ KN

  $M_f = 1.25D + 1.5S = 1.25(61.55) + 1.5(104.95)$
  $= 234.36$ KN•m

Note: For efficiency, I calculated the shear and bending resistance as well as deflection in Excel for Beam B. I will only list the results here.

- From WDM 2017, pg. 69 → Use GL D.Fir 24f-EX 175 × 608

Bending

M_{r1} (Tension) $= 282.03$ KN•m > 234.36 ✓ Acceptable

M_{r2} (Compression) $= 285.40$ KN•m > 234.36 ✓ Acceptable

Shear

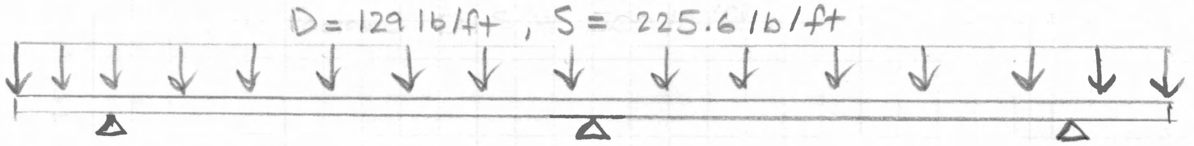
$V_r = 127.68$ KN > 119.83 KN ✓ Acceptable

Deflection

$\Delta_{max,\ cant.} = 37.83\ mm < \Delta_{all.} = 43.76\ mm$

$\Delta_{max,\ mid.} = 4.32\ mm < \Delta_{all.} = 17.50\ mm$

✓ Acceptable

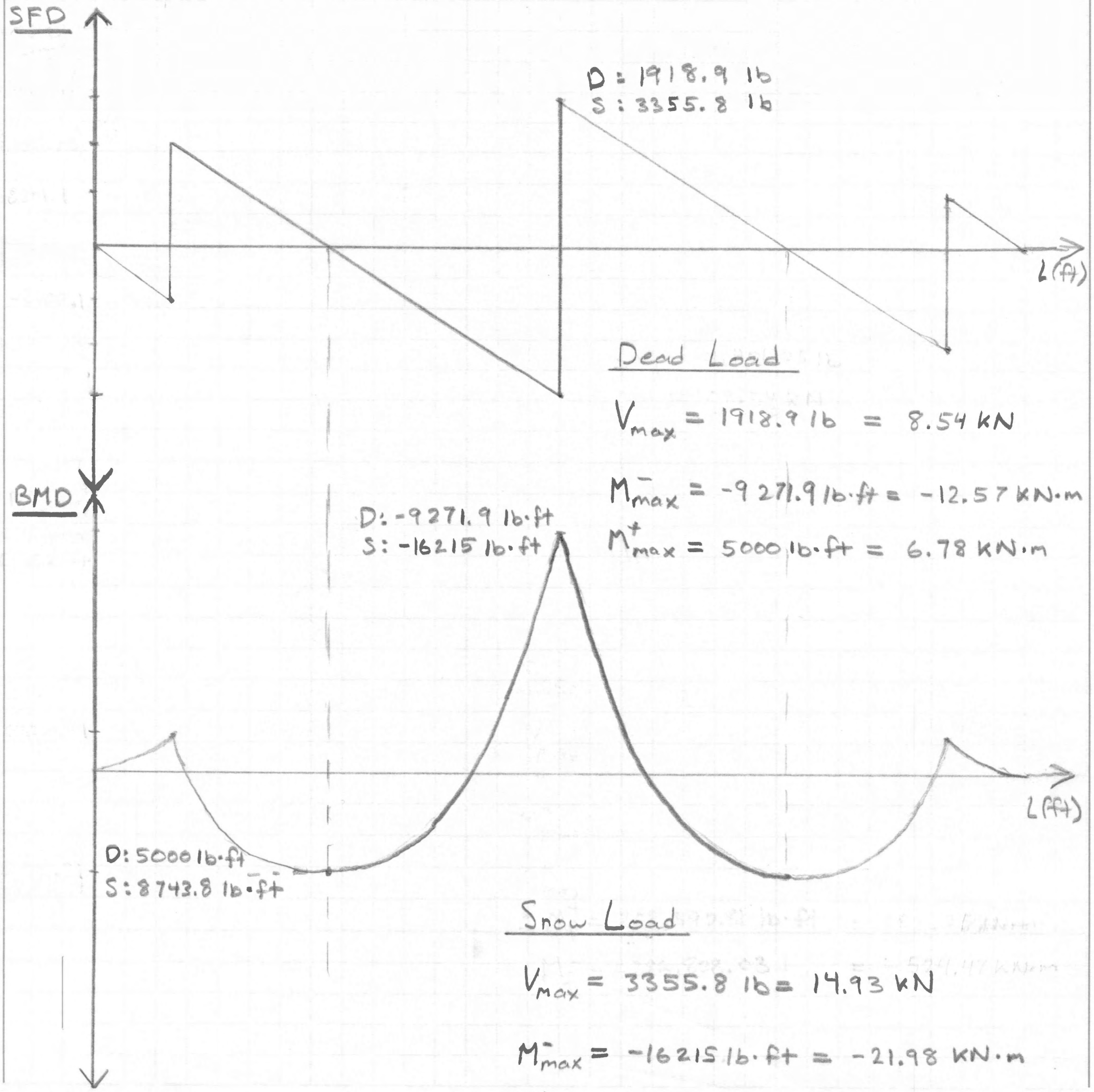


Width of Panel: 6'-0"

Dead Load (due to self weight): 21.5 lb/ft^2 x 6'-0" = 129 lb/ft

Snow Load: 37.6 lb/ft^2 x 6'-0" = 225.6 lb/ft

Unfactored Demand



Dead Load

V_{max} = 1918.9 lb = 8.54 kN

M^-_{max} = -9271.9 lb·ft = -12.57 kN·m

M^+_{max} = 5000 lb·ft = 6.78 kN·m

Snow Load

V_{max} = 3355.8 lb = 14.93 kN

M^-_{max} = -16215 lb·ft = -21.98 kN·m

M^+_{max} = 8743.8 lb·ft = 11.86 kN·m

Panels - M_r, V_r, Δ

Factored Demand: $V_f = 1.25D + 1.5S = 1.25(8.54) + 1.5(14.93)$
$= 33.07 \text{ KN}$

$M_f = 1.25D + 1.5S = 1.25(12.57) + 1.5(21.98)$
$= 48.68 \text{ KN·m}$

→ Using 2x8 DLT, DFir

Bending

$$M_r = \phi f_b (K_D K_H K_{Sb} K_T) S K_{Zb} K_L$$

where: $\phi = 0.9$

$f_b = 16.5 \text{ MPa}$

$K_D, K_T = 1.0$, $K_H = 1.10$ for Case 1, $K_{Sb} = 0.84$ for wet conditions

$S = \frac{I}{y}$ where $I = \frac{bh^3}{12} = \frac{1830(180^3)}{12} = 0.889 \times 10^9 \text{ mm}^4$

$S = 90\text{mm}$

$= 9.882 \times 10^6 \text{ mm}^3$

$= 0.009882 \text{ m}^3$

$K_{Zb} = 1.2$ for ~38×184 member

$K_L \rightarrow d/b = \frac{180}{1830} << 4 \therefore$ no support

$= 1.0$

$M_r = 0.9(16{,}500 \text{ KPa})(1.10)(0.84)(0.009882 \text{ m}^3)(1.2)$
$= 162.71 \text{ KN.m} > 48.68 \text{ KN·m}$ ✓ Acceptable

Shear

$$V_r = \phi f_v (K_D K_H K_{SV} K_T) \tfrac{2}{3} A_n K_{ZV}$$

where: $\phi = 0.9$

$f_v = 1.5 \text{ MPa}$

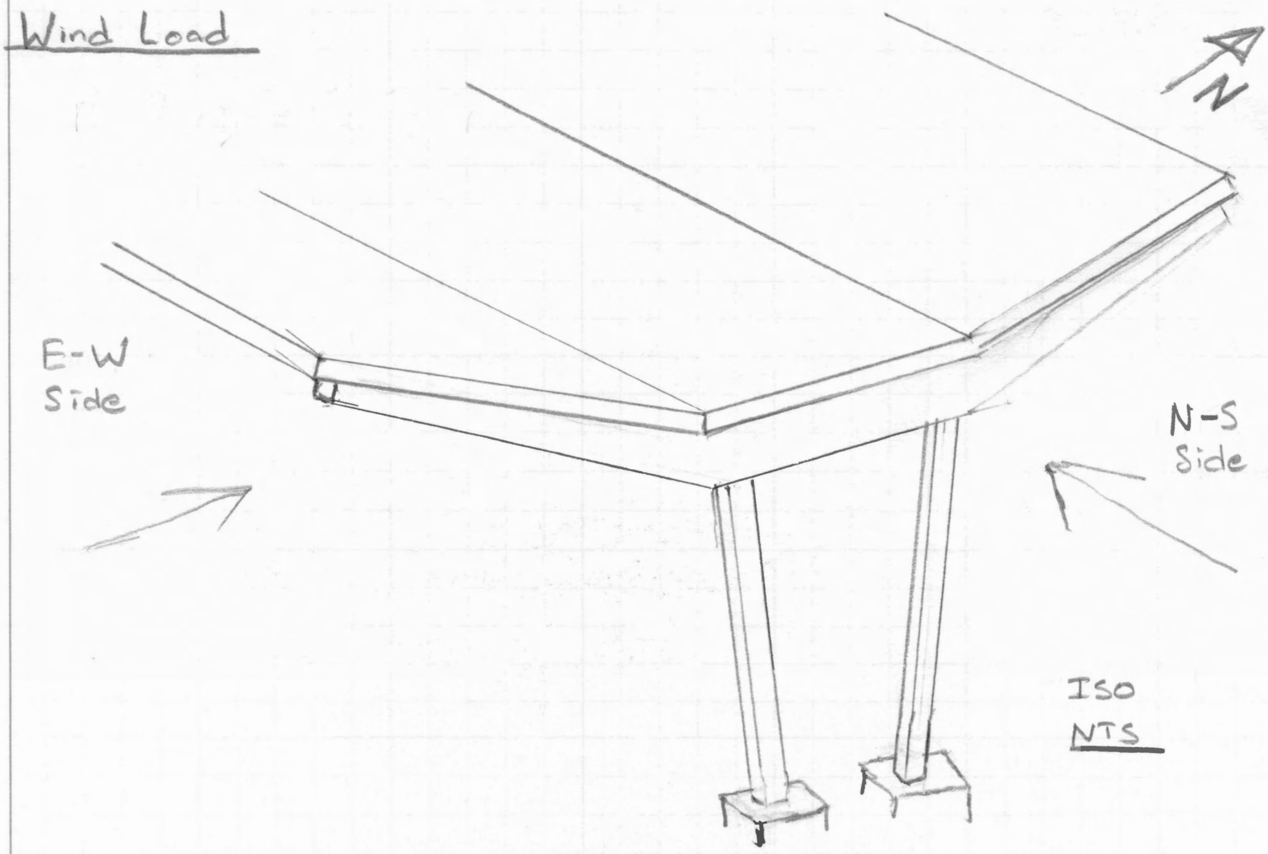
$K_D, K_T = 1.0$, $K_H = 1.10$ for Case 1, $K_{SV} = 0.96$ for wet conditions

$A_n = b \cdot h = 180 \times 1830 = 329{,}400 \text{ mm}^2$
$= 0.3294 \text{ m}^2$

$K_{ZV} = 1.2$

$V_r = 0.9(1500 \text{ KPa})(1.10)(0.96)(2/3)(0.3294 \text{ m}^2)(1.2)$
$= 375.67 \text{ KN} > 33.07 \text{ KN}$ ✓ Acceptable

Wind Load



$P = I_W \, q \, C_e C_t C_g C_p$

where: $I_W = 1.0$ for Normal Importance

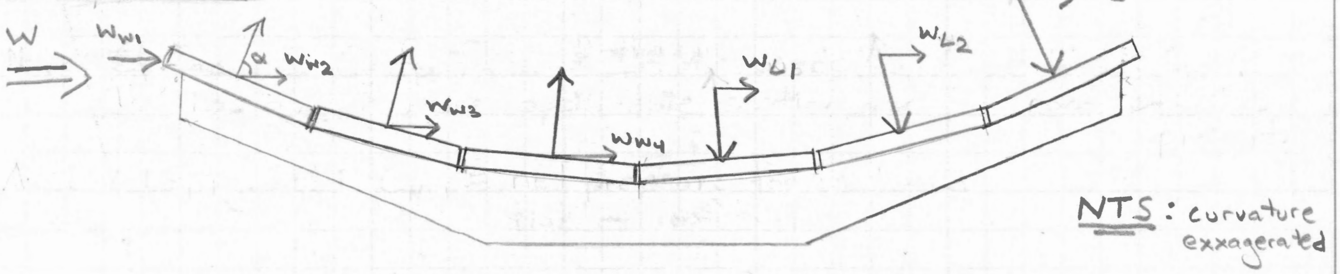
$q = 0.44$ KPa for 1/50

$C_e = (h/10)^{0.2} = (5.25/10)^{0.2} = 0.88 < 0.9$

$= 0.9$

$C_t = 1.0$ for relatively flat terrain

E-W Side



NTS: curvature exxagerated

Note: Because of the shape of the canopy, I have used the ASCE 7-16 to calculate the gust factor, G and the pressure coefficient (C_N or C_f). These will be used instead of $C_g C_p$ as applicable. I will show the modified formulas before the calculation of pressures.

ASCE 7-16, pg. 281 : C_{NW} = net pressure for windward roof surface

C_{NL} = net pressure for leeward roof surface

Canopy Roof Angle : Since the roof panels increase in angle along the curvature of the beam, I will use a median angle

Steepest Panel Angle ≈ 8.5°
Shallowest Panel Angle ≈ 1.7°

Average Angle (θ) ≈ 5.1°

To be conservative, will use $\underline{\theta = 7.5°}$

Case	C_{NW}	C_{NL}
A	-1.1	0.3
B	-0.2	1.2

(+) denotes wind toward,
(−) denotes wind away from top of roof surface

ASCE 7-16, Eq'n 26.11-6 : G = Gust Factor

$$G = 0.925\left(\frac{1+0.7 g_Q I_{\bar{z}} Q}{1+0.7 g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1+0.7(3.4)(0.305)(0.889)}{1+0.7(3.4)(0.305)}\right)$$

$= 0.88$

Note: To shorten the length of some calcs, I have completed them in Excel

ASCE 7-16, pg. 323 : C_f = Force coefficient for freestanding walls and signs.
(I will use this for the load on the panel edge)

Aspect Ratio $= B/s = \frac{18.288\,m}{0.18\,m}$, where B = panel length
$= 101.6$ s = panel depth

Clearance Ratio $= s/h = \frac{0.18\,m}{5.53\,m}$, where h = average height from ground
$= 0.033$

From Table, $C_f = 1.95$

For Pressure on the Panel Surfaces, $p = I_W q C_e C_t G C_N$

For Pressure on the Panel Edge, $p = I_W q C_e C_t G C_f$

Wind Pressure, E-W Side (kPa)

	Windward	Leeward
Case A	−0.341	0.093
Case B	−0.062	0.372
Panel Edge	0.610	

Line Load for E-W Side

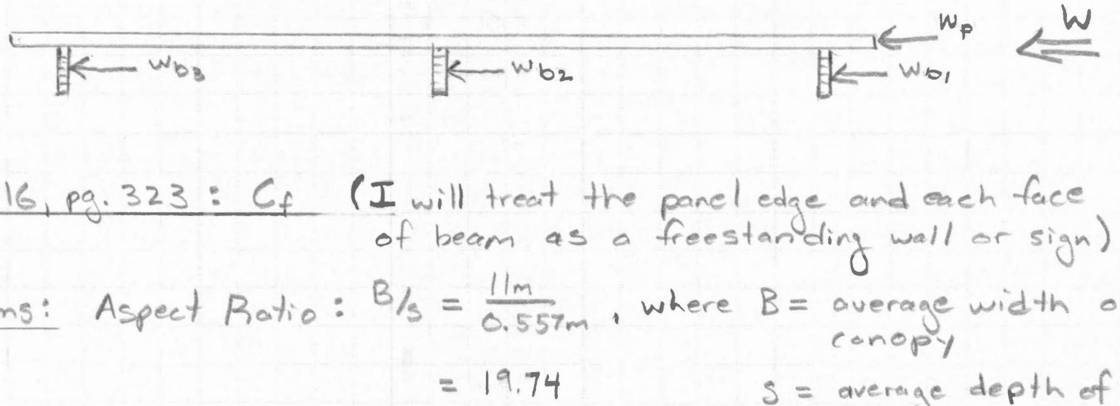
Using the figure at the start of the E-W Side calculations, I will find the line load on each panel and on the panel edge. Then, I will sum these to find the w_{res} on the E-W Side. The line loads on the panel surfaces have only an x-component of the pressure applied on them.

Load	Panel Angle	w, Case A	w, Case B
w_{W1}	--	0.334	0.334
w_{W2}	81.572°	0.092	0.017
w_{W3}	84.943°	0.055	0.010
w_{W4}	88.314°	0.018	0.003
w_{L1}	88.314°	0.005	0.020
w_{L2}	84.943°	0.015	0.060
w_{L3}	81.572°	0.025	0.100
w_{res}		0.544 KN/m	0.544 KN/m

Note: w_{W1} is found using the pressure × panel height. I have also added 0.37m of snow height on as surface area for the wind.

w_{W2} and on are found using pressure × panel width × cos α (shown in figure)

N-S Side



ASCE 7-16, pg. 323 : C_f (I will treat the panel edge and each face of beam as a freestanding wall or sign)

For Beams: Aspect Ratio : $B/s = \frac{11m}{0.557m}$, where B = average width of canopy

$= 19.74$

s = average depth of beams

Clearance Ratio : $s/h = \frac{0.557m}{5.3m}$, where h = average height from ground

$= 0.105$

From Table, $C_{f(beams)} = 1.9$

For Panels: Aspect Ratio: $B/s = \frac{11m}{0.18m}$, where B = width of canopy
s = depth of panel

$= 61.11$

Clearance Ratio: $s/h = \frac{0.18m}{5.53m}$, where h = average height from ground

$= 0.033$

From Table, $C_{f(panels)} = 1.95$

ASCE 7-16, Eq'n 26.11-6 : $G = 0.89$ Note: G is calculated the same way as for E-W side. A minor change for this side produces a slightly different result

For Pressure on all surfaces, $p = I_w q C_e C_t G C_f$

Wind Pressure, N-S Side (KPa)

Beams	0.595
Panels	0.610

Line Load for N-S Side

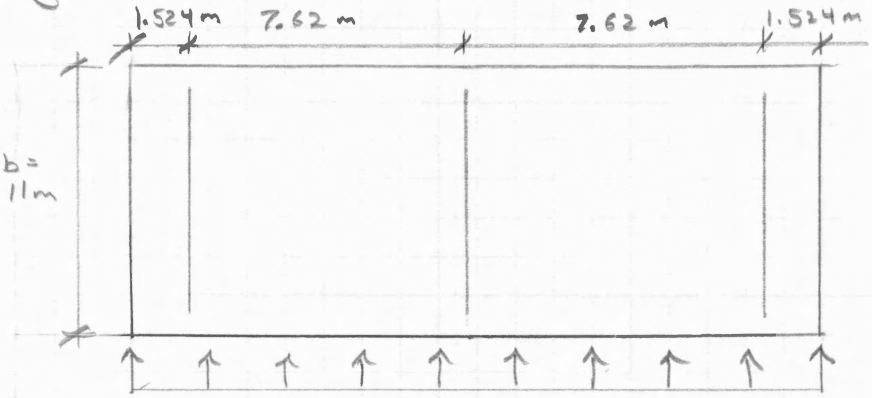
Using the figure at the start of N-S Side calculations, I will find the line load on each face using the pressure × member height. As done previously, I will calculate the panel height to include 0.37m of snow height.

Load	w, (kN/m)
w_p	0.334
w_{b1}	0.331
w_{b2}	0.331
w_{b3}	0.331
w_{res}	0.443 KN/m

$\Sigma w \times \frac{1}{3}$

where w_{res} is the line load along a single beam line. Therefore the Σw's was multiplied by $1/3$ to get w_{res}

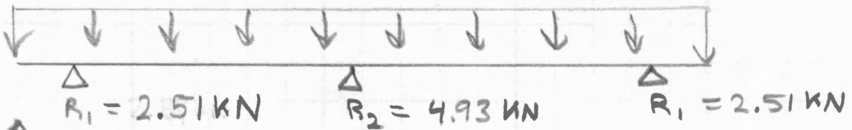
Diaphragm Load in Roof



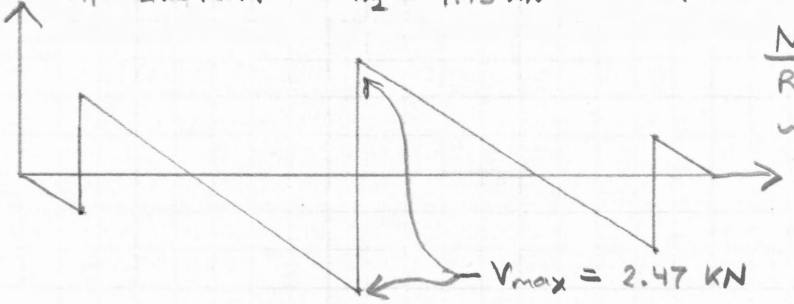
Plan : NTS

W = 0.544 KN/m (unfactored)

Loading Diagram



SFD



Note:
Reactions here will be used for column / column connection design as well

$$v_f = \frac{V_{max}(1.4)}{b} = \frac{(2.47\,KN)(1.4)}{11m} = \underline{0.314\ KN/m}$$

Nailing & Ply Selection

WDM 2017, pg. 572 : $v_{rd} = \phi v_d J_D J_s J_f J_{ud}$

where $\phi = 0.8$

$v_d = N_u/s$, where $N_u = n_u K_D K_{SF} K_T$

$J_D = 1.3$

$J_s = 1.0$

$J_f = 1.0$ for one row and members adeq. connected

$J_{ud} = 1.0$ for blocked (DLT)

where n_u is described further on

$K_D = 1.15$

$K_{SF} = 1.0$

$K_T = 1.0$

$s = 150$ mm (Assumed)

Calculation of n_u from CSA O86 12.9.3.2

Try 2-½" nails @ 150 o.c. ⟵ Diaphragm selection Table, WDM

a) $f_1 d_F t_1$ where $f_1 = 50G(1-0.01d_F)J_x$

$= 50(0.47)(1-0.01(3.25))(1)$

$= 22.74$ MPa

$d_F = 3.25$ mm (diam)

$t_1 = 12.5$ mm (Ply)

Case a) $= 22.74(3.25)(12.5)$

$= \underline{924 N}$

b) $f_2 d_F t_2$ where $f_2 = 22.74$ MPa

$t_2 = 180$ mm

Case b) $= \underline{13{,}303 N}$

d) $f_1 d_F^2\left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_1}{d_F}\right)$ where $f_y = 50(16-d_F)$

$= 637.5$

$f_3 = 110G^{1.8}(1-0.01d_F)J_x$

$= 27.34$ MPa

Case d) $= \underline{568.4 N}$

e) $f_1 d_F^2\left(\sqrt{\frac{1}{6}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}} + \frac{1}{5}\frac{t_2}{d_F}\right)$ where $t_2 = 51$ mm

Case e) $= \underline{1137.4 N}$

f) $f_1 d_F^2 \frac{1}{5}\left(\frac{t_1}{d_F} + \frac{f_2}{f_1}\frac{t_2}{d_F}\right)$ ⇒ Case f) $= \underline{938.6 N}$

g) $f_1 d_F^2\sqrt{\frac{2}{3}\frac{f_3}{(f_1+f_3)}\frac{f_y}{f_1}}$ ⇒ Case g) $= \underline{767.2 N}$

Smallest Result $= \underline{568.4 N} \leftarrow n_u$

So, $N_u = 568.4(1.15) = 653.7$ N

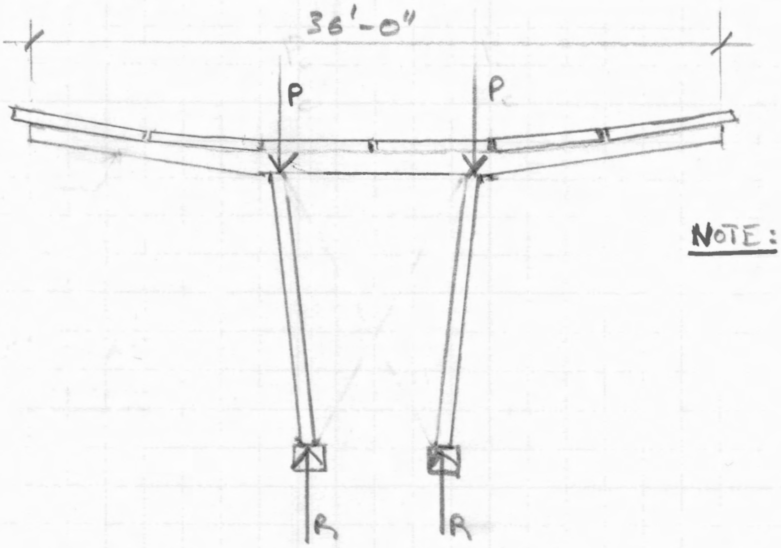
$v_d = N_u/s = 4.36$ N/mm

∴ $v_{rd} = 0.8(4.36\text{ N/mm})(1.3) = 4.53$ N/mm or $\underline{4.53 \text{ kN/m}}$

$v_{rd} = 4.53$ kN/m $>> v_f = 0.314$ kN/m ✓ Acceptable

So, will use $\underline{\text{½" Ply with 2-½ nails @ 150 o.c.}}$

Unfactored Point Load



NOTE: Cables will not carry any gravity load, so ignore

P is the point load due to the trib width and length for one column under the beam

Column under Beam A

P_A: Dead

$$390\,lb/ft \times 36ft/2 + \overbrace{1225\,lb/2}^{\text{beam weight}} = 7633\,lb$$
$$= 33.95\,KN$$

Snow

$$658\,lb/ft \times 36ft/2 = 11844\,lb$$
$$= 52.69\,KN$$

Column under Beam B

P_B: Dead

$$551.3\,lb/ft \times 36ft/2 + \overbrace{1395\,lb/2}^{\text{beam weight}} = 10{,}621\,lb$$
$$= 47.25\,kN$$

Snow

$$940\,lb/ft \times 36ft/2 = 16{,}920\,lb$$
$$= 75.26\,kN$$

Column A - Axial Resistance (see Column B for detailed calculation)

- Factored Demand : $P_{Af} = 1.25D + 1.5S = 1.25(33.95) + 1.5(52.69)$
$= 121.47$ KN

$$P_{Ac} = \frac{P_{Af}}{\cos(8.43°)} = 122.80 \text{ KN}$$

Note : Since I did the sample calculations for Column B, I will only outline my results here which I calculated in Excel.

- From CISC S16 Steel Handbook → Use HSS 152 x 152 x 6.4

Local Buckling

$$21.94 < \frac{670}{\sqrt{F_y}} = 35.8$$ ✓

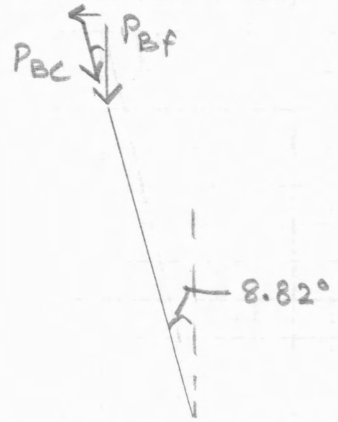
Flexural Buckling

$C_r = 285.91$ KN > 122.80 KN ✓ Acceptable

Column B - Axial Resistance

- Factored Demand : $P_{Bf} = 1.25D + 1.5S = 1.25(47.25\,kN) + 1.5(75.26)$
  $= 171.95\ kN$

FBD of Column B



Note: Column does not have bending from gravity load. The horizontal component is resolved as tension in the beam. The symmetry of the columns eliminates any moment.

$$P_{Bc} = \frac{P_{Bf}}{\cos(8.82°)} = 174.01\ kN$$

- From CISC S16 Steel Handbook → Use HSS 152×152×6.4

Local Buckling

CSA S16-14, Table 1 $\rightarrow \frac{bel}{t} \leq \frac{670}{\sqrt{F_y}} \Rightarrow \frac{w-2t}{t} \leq \frac{670}{\sqrt{F_y}}$

$$\rightarrow \frac{152-2(6.35)}{6.35} = 21.94 < \frac{670}{\sqrt{F_y}} = 35.8\ ✓$$

Flexural Buckling

$$C_r = \frac{\phi A F_y}{(1+\lambda^{2n})^{1/n}}$$

where : $\phi = 0.9$

$A = 3610\ mm^2$

$F_y = 350\ MPa$

$n = 1.34$

$\lambda = \sqrt{\frac{F_y}{F_e}}$ where: $F_e = \frac{\pi^2 E}{(\frac{KL}{r})^2}$ where:

$= \sqrt{\frac{350}{109.4}}$ $= 109.4$

$= 1.789$

$E = 200000\ MPa$

$K = 2.0$

$L = 3976\ mm$

$r = 59.2\ mm$

$$C_r = \frac{0.9(3610)(350)}{\left(1+1.789^{2(1.34)}\right)^{1/1.34}} \times 10^{-3}$$

$= 308.12\ kN > 174.01\ kN$ ✓ Acceptable

Column A - Axial Compression & Bending Resistance

(See Column B for detailed Calculations)

- Factored Compressive Demand : $P_{Af} = 1.25D + 0.5S$

$$= 1.25(33.95) + 0.5(52.69)$$
$$= 68.78 \text{ kN}$$
$$P_{Ac} = 69.53 \text{ kN}$$

- Factored Lateral Demand :

E-W Side	N-S Side
8.03 kN·m	15.59 kN·m

- Factored P-Δ Demand :

E-W Side	N-S Side
0.31 kN·m	0.61 kN·m

- Combined Moment Demand :

E-W Side	N-S Side
8.34 kN·m	16.20 kN·m

Note: Because I did the sample calculations for Column B, I will only outline my results here which I calculated in Excel.

Class of Section

$21.94 < \frac{420}{\sqrt{F_y}} = 22.45$ ∴ Class 1 Section

Cross - Sectional Strength

$0.48 < 1.0$ ✓ Acceptable

Overall Member Strength

$0.66 < 1.0$ ✓ Acceptable

Lateral Torsional Buckling

$0.66 < 1.0$ ✓ Acceptable

Column B – Axial Compression & Bending Resistance

- Factored Compressive Demand: $P_{Bf} = 1.25D + 0.5S$

$$= 1.25(47.25) + 0.5(75.26)$$

$$= 96.69 \text{ kN}$$

$$P_{BC} = \frac{96.69}{\cos(8.82°)} = \underset{\substack{\uparrow \\ C_f}}{97.85} \text{ kN}$$

- Factored Lateral & P-Δ Demand:

Lateral Demand	E-W Side	N-S Side
Reaction along Beam	4.93 kN	4.87 kN
Reaction for 1 column	2.47 kN	2.44 kN
Factored Load (1.4W)	3.46 kN	3.42 kN
Moment Arm (m)	4.386	4.386
Moment (kN·m)	15.18	15.00

← (Line load × Canopy width)

← (vertical height of column)

Note: The moment arm heights include the pedestal height, so my results are a bit conservative.

P-Δ Demand	E-W Side	N-S Side
Reaction along Beam	2.47 kN	2.44 kN
Deflection (m)	0.0205	0.0202
Factored Snow (0.5S)	37.36 kN	37.36 kN
Moment kN·m	0.77	0.76

Note: Deflection calculated using $\frac{P\ell^3}{3EI}$ for a point load on a cantilevering beam (column as beam)

	E-W Side	N-S Side
Combined Moment (kN·m)	$\underset{\substack{\uparrow \\ M_{fx}}}{15.95}$	$\underset{\substack{\uparrow \\ M_{fy}}}{15.76}$

Class of Section

$$\frac{b_{el}}{t} \le \frac{420}{\sqrt{F_y}} \text{ for Class 1} \Rightarrow \frac{w-2t}{t} \le \frac{420}{\sqrt{F_y}}$$

$$\frac{w-2t}{t} = \frac{152-2(6.35)}{6.35} \le \frac{420}{\sqrt{F_y}} = \frac{420}{\sqrt{350}} \Rightarrow 21.94 < 22.45 \checkmark$$

Therefore, Class 1

Cross Sectional Strength

$C_r = \phi A F_y = 0.9(3610)(350) \times 10^{-3} = \underline{1137.15 \text{ kN}}$

$M_{rx} = M_{ry} = \phi Z_x F_y = 0.9(196 \times 10^3)(350) \times 10^{-6} = \underline{61.74 \text{ kN}\cdot\text{m}}$

$U_{1x} = U_{1y} = \left(\frac{\omega_1}{\left(1 - \frac{C_f}{C_e}\right)}\right)$ where $\omega_1 = 0.6 - 0.4\kappa \geq 0.4$

where $\kappa = -\frac{\text{small moment}}{\text{large moment}} = -1$

$= 1$

$= \left(\frac{1}{\left(1 - \frac{97.85 \text{ kN}}{1573.29 \text{ kN}}\right)}\right)$

$= \underline{1.07}$

$C_e = \frac{\pi^2 EI}{L^2} = \frac{\pi^2(200000)(12.6 \times 10^6)}{3976^2} \times 10^{-3}$

$= 1573.29 \text{ kN}$

Using Interaction Formula:

$$\frac{C_f}{C_r} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{M_{ry}} \leq 1.0$$

$$\Rightarrow \frac{97.85}{1137.15} + \frac{1.07(15.95)}{61.74} + \frac{1.07(15.76)}{61.74} = 0.64 < 1.0 ✓$$

Overall Member Strength

$C_r = \frac{\phi A F_y}{(1+\lambda^{2n})^{1/n}} = \underline{308.12 \text{ kN}}$ (Already calculated in Flexural Buckling)

$M_{rx} = M_{ry} = \underline{61.74 \text{ kN}\cdot\text{m}}$ (Calc'd above)

$U_{1x} = U_{1y} = \underline{1.07}$ (Calc'd above)

$$\frac{97.85}{308.12} + \frac{1.07(15.95)}{61.74} + \frac{1.07(15.76)}{61.74} = 0.86 < 1.0 ✓$$

Lateral Torsional Buckling

$C_r = 308.12 \text{ kN}$ (Calc'd above, each axis is the same)

$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$ where $\omega_2 = 1.75 + 1.05\kappa + 0.3\kappa^2 \leq 2.5$

$= 1.75 - 1.05 + 0.3$

$= 1 < 2.5 ✓$

$= \frac{\pi}{3976}\sqrt{(200000)(12.6 \times 10^6)(77000)(20200 \times 10^3)}$

$= \underline{1564.32 \text{ kN}\cdot\text{m}}$

$G = 77000 \text{ MPa}$

$J = 20200 \times 10^3 \text{ mm}^4$

$I_y = 12.6 \times 10^6 \text{ mm}^4$

$E = 200\,000 \text{ MPa}$

$C_w = 0$ for rect. HSS, therefore second term is 0

$$\rightarrow 0.67M_p = 0.67(Z_x)(F_y) = 0.67(196000)(350)\times10^{-6}$$
$$= \underline{45.96 \text{ KN}\cdot\text{m}} < M_u$$

$$\rightarrow M_r = 1.15\phi M_p\left(1-\frac{0.28M_p}{M_u}\right)$$
$$= 1.15(0.9)(196000)(350)(10^{-6})\left(1-\frac{0.28(196000)(350)(10^{-6})}{1564.32}\right)$$
$$= 70.13 > \phi M_p = 61.74$$

$$\therefore M_{rx} = M_{ry} = \underline{61.74 \text{ KN}\cdot\text{m}}$$

$U_{1x} = U_{y} = \underline{1.07}$ (Calc'd earlier)

$$\rightarrow \frac{97.85}{1137.15} + \frac{1.07(15.95)}{61.74} + \frac{1.07(15.76)}{61.74} = 0.86 < 1.0 \checkmark$$

Note: → Although I have not learned seismic design yet, I will perform an approximate seismic analysis to more accurately size the columns, beam-column connections, and foundations.

Assumed Seismic Load

Dead (KPa)	1.12	← (includes averaged beam weight)
Snow (KPa)	1.80	
Factored (Case 5)	1.57	← (Case 5: 1D + 0.25S)
0.3W (KPa)	0.47	← (Using 0.3 of Factored load for an approximate seismic analysis)

Factored Lateral Demand at one support = (0.47 KPa)(18.288 m × 11 m) / 6 columns

$= \underline{15.79 \text{ KN}}$

Column Design - (Column B governing)

- From CISC S16 Steel Handbook → Use HSS 203 × 203 × 9.5
- Factored Compressive Demand :
$$P_{Bf} = 1D + 0.25S$$
$$= 47.25 + 0.25(75.26)$$
$$= \underline{66.07 \text{ KN}}$$
$$P_{Bc} = \frac{66.07}{\cos(8.82°)} = \underline{66.86 \text{ KN}}$$
- Factored Seismic Demand :
$$M_{fx} = M_{fy} = 15.79 \text{ KN}(4.386\text{m})$$
$$= \underline{69.25 \text{ KN·m}}$$

For efficiency, I did the calcs in Excel, and will outline the results here:

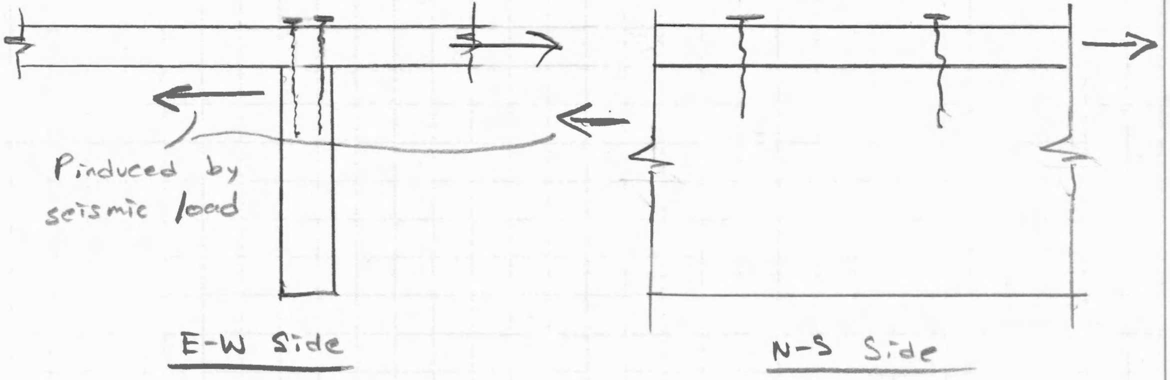
Class of Section: Class 1

Cross Sectional Strength: 0.90 < 1.0 ✓

Overall Member Strength: 0.94 < 1.0 ✓

Lateral Torsional Buckling: 0.94 < 1.0 ✓

Therefore, Seismic heavily governs over lateral for the columns. An HSS 203 × 203 × 9.5 member will be used.



Lag Screw Design

Seismic Load

Factored Shear Demand per Beam Line = (0.47 kPa)(18.288 m × 11 m) / 3 beams
= 31.52 kN per beam line

Factored Shear Demand per Panel = 31.52 kN / 6 panels
= 5.25 kN ← Q_f

WDM 2017, pg. 460 : $Q_r = Q'_r \, n_{Fe} \, n_R \, K'$ (worst case loading)

- Calculating n_{Fe} using Table 7.20:

A_s (panel) = 329 184 mm^2
A_m (beam) = 93,100 mm^2 > $\frac{A_m}{A_s} = 0.28$

Extrapolating, $n_{Fe} = 3.92\left(\frac{0.28}{0.5}\right)$ ← Assuming 4 fasteners in a row
$= 2.2$

- $K' = K_D K_{SF} K_T = 1.0$
- $Q'_r = 1.34$ kN for main member ⊥ and side member // to grain for 1/2" screws (worst case) ~ pg. 467
- $n_R = 2$ (2 rows)

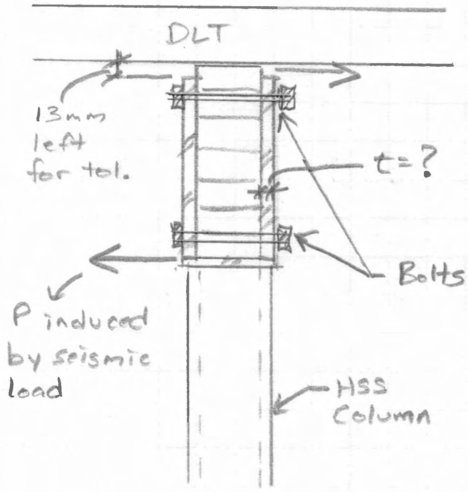
$Q_r = (1.34 \text{ kN})(2.2)(2)(1)$
$= 5.90 \text{ kN} > 5.25 \text{ kN}$ ✓

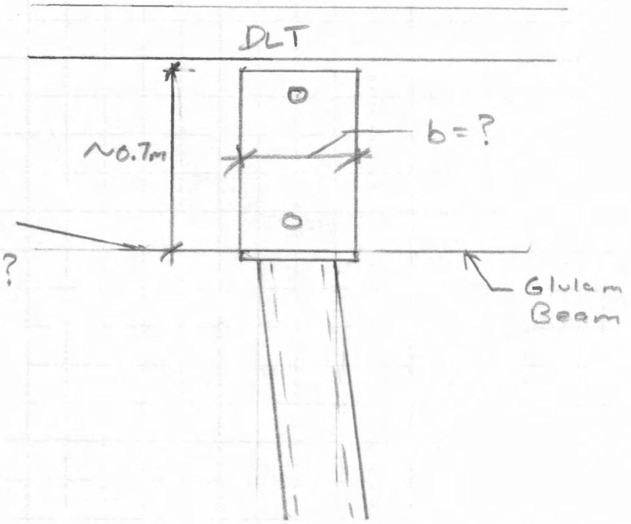
Finding length: L_p (penetration length) = 114 mm for D Fir, 1/2" Ø (Table 7.13)

$L = 180 \text{ mm} + 114 \text{ mm} = 294 \text{ mm}$ (180 mm: panel)

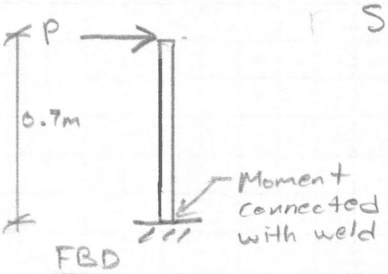
Therefore, use 2 rows of 13 Ø × 300 mm lag screws @ 300 o/c

Beam - Column Connection





Req'd Plate Size



Since Seismic Demand at 1 column = 15.79 kN,

$P = \frac{1}{2}(15.79\ kN) = \underline{7.90\ kN}$ (Because there are two plates)

$\therefore M_f = 7.90\ kN(0.7m)$

$= \underline{5.53\ kN\cdot m}$

CSA S16-14, 13.5 → Use $M_r = \phi Z F_y$, where $Z = \frac{bt^2}{4}$

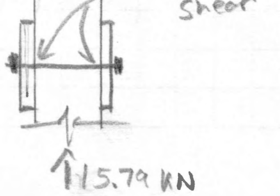
$\therefore t = \sqrt{\frac{4 M_f}{b \phi F_y}}$, try $b = 200mm$

$t = \sqrt{\frac{4(5.53)}{(0.20)(0.9)(350000)}} = 18.7mm$ req'd

∴ Use 200 × 700 × 19 mm PL

<u>Note:</u> 700 may vary with the outer beams, but this is worst case.

Shear & Bearing in Bolts



Seismic Demand = 15.79 kN/2 bolts

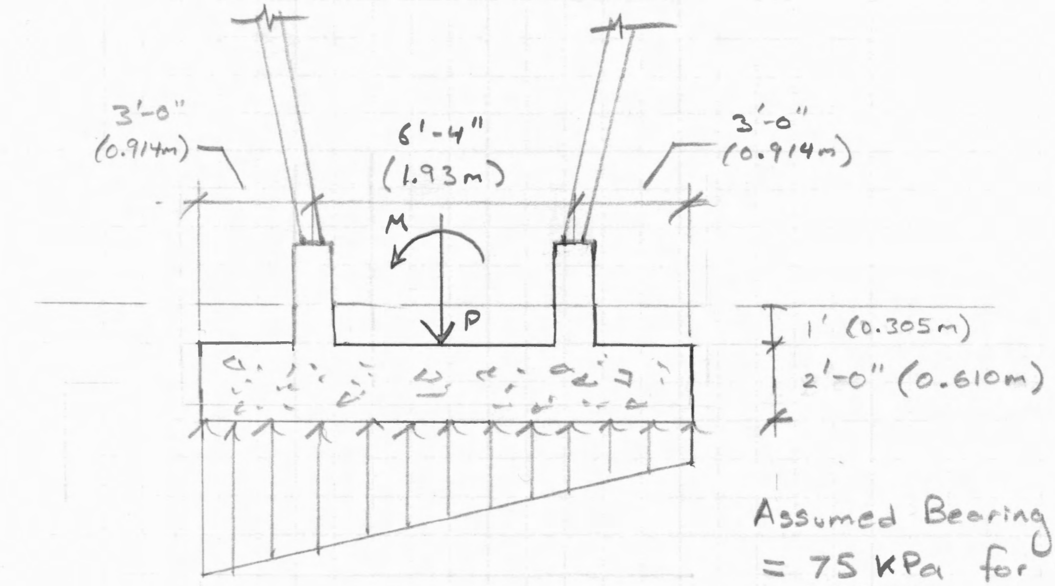
= 7.9 kN per bolt

CISC S16 Steel Handbook, Table 3-4 & 3-6

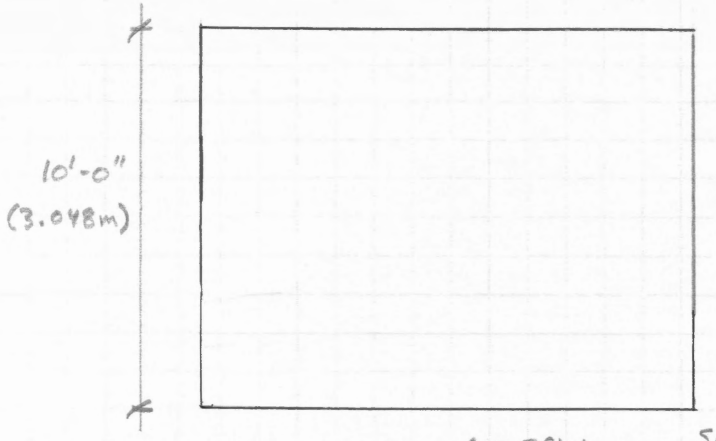
→ Use 1" A325 bolt:

Bearing: 521 kN >> 7.9 kN ✓

Double Shear: 201 ×2 = 402 kN >> 7.9 kN ✓



Assumed Bearing Pressure = <u>75 KPa</u> for silt



Area = 3.048 m × 3.759 m
= 11.457 m^2

$$S_{E-W} = \frac{bd^2}{6} = \frac{3.048(3.759)^2}{6} = 7.178\ m^3$$

$$S_{N-S} = \frac{3.759(3.048)^2}{6} = 5.820\ m^3$$

$P = 1.57\ KPa \times 18.288\ m \times 11 m \times \left(\frac{17.5 ft}{60 ft}\right)$ ← Seismic Gravity Load of Roof (worst case)
$+\ 0.551\ kN/m \times 4 m \times 2$ ← 2 columns weight
$+\ 24\ kN/m^3 \times 3.048\ m \times 0.610\ m \times 3.759\ m$ ← Concrete weight
$+\ 14\ kN/m^3 \times 3.048\ m \times 0.305\ m \times 3.759\ m$ ← Approx. Soil weight
$= 313.2\ KN$

$M = (15.76\ kN \times 4.87 m)$ 2 columns
$= 153.5\ kN \cdot m$ (seismic moment)

$$e = \frac{M}{P} = \frac{153.5}{313.2} = 0.49\ m$$

<u>E-W Side</u>

$$\frac{D}{6} = \frac{3.759}{6} = 0.63 m > e\ ✓$$

$$q = \frac{P}{A} \pm \frac{M}{S} = \frac{313.2}{11.457} \pm \frac{153.5}{7.178}$$

$$= 27.3 \pm 21.4$$

$\therefore q_{max} = \underline{48.7\ KPa < 75\ KPa}$ ✓

$q_{min} = 5.9\ KPa$

<u>N-S Side</u>

$$\frac{D}{6} = \frac{3.048}{6} = 0.51 m > e\ ✓$$

$$q = \frac{P}{A} \pm \frac{M}{S} = \frac{313.2}{11.457} \pm \frac{153.5}{5.820}$$

$$= 27.3 \pm 26.4$$

$\therefore q_{max} = \underline{53.7\ KPa < 75\ KPa}$ ✓

$q_{min} = 0.9\ KPa$