CIVE 418: Airplane Hangar Design Project

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1.0 Introduction

1.1 Problem Conceptualization

As an innovative structural engineering design firm, Frost has completed the efficient design of an airplane hangar in Kuujjuaq, Quebec. This report outlines the design methodology and considerations when developing a structurally sufficient building. The design requirements comprised of three main aspects including the design of the airplane hangar, adjacent office, and foundation. The project design phase took place between September 8th and December 7th 2017, the deadline set by Frost's client.

The overall approach to this design consisted of using a rigorous structural engineering design process while constantly evaluating the constructability, cost, and time constraints affiliated with construction in northern Quebec. The entire structure covers an area of 4808 m^2. The hangar, as shown in Figure 1., is 77 m long, 54 m wide and 19 m high. The adjacent office, shown afterwards in Figure 2., is 50 m long, 13 m wide, and 10 m high. The deep pile foundation was designed using the bedrock level in Kuujjuaq of 9.7 m.

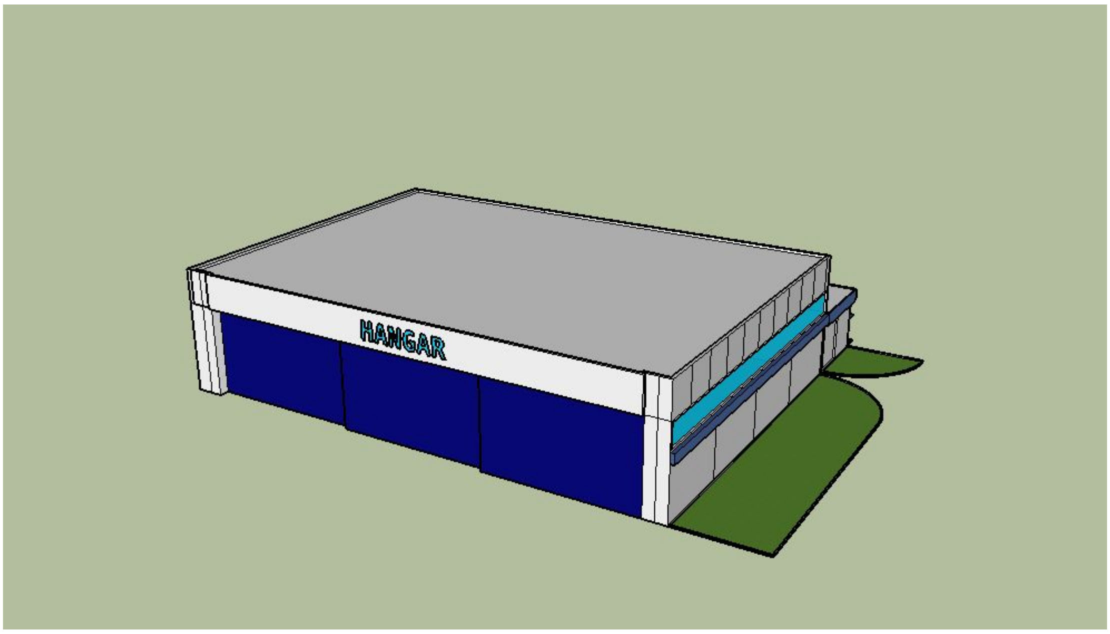


Figure 1. Isometric View of the Airplane Hangar

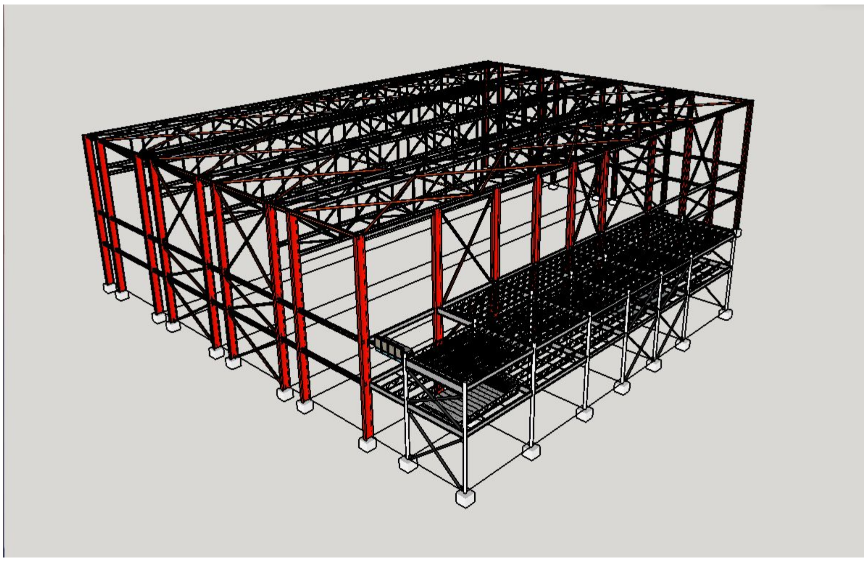


Figure 2. Isometric View of the Office Space

1.2 Literature Review of Similar Project

The design and the construction of the Beijing A380 hangar roof system at Capital Airport faced very similar problems as the hangar project in Kuujjuaq, Quebec. Both hangars had open spacing requirement and the opening on the door side would create a torque under wind load or seismic load. The A380 hangar had 362 meters span, and used steel space frame together with a truss at the door-side as its roof system. The roof was supported by the four-leg concrete-filled steel tube columns at three sides of the perimeter and a rectangular hollow reinforced concrete column at the middle of the door-side. The non-uniform column spacing adjusted the center of rigidity close to the center of mass of the whole structure to satisfy the drift limit (Zhu et al., 2008).

2.0 Project Objective

2.1 Project Requirements

The primary objective of this project was to design an airplane hangar that had the structural capacity to store a Boeing 737-800 and a DASH8 Q-400. Adjacent to the hangar, the client also specified that the design of a two storey office was required. The hangar is comprised of a long span truss systems including purlins. Design considerations for the megadoor on the long side of the hangar were considered to ensure that the foundation below could support its weight and that the structure's design was adequate for the large opening. The office space was specified to be a primarily steel structure. The ground floor of the entire structure required a slab on grade design as well as a deep foundation design for the poor soil conditions.

To optimize client satisfaction, the most economical and efficient components were considered throughout the structure. Throughout the design process, economic factors as well as the efficient constructability and transportation were accounted for. The design requirements included that the design had to satisfy the client's project specifications while also complying with the NBCC.

2.2 Constraints

2.2.1 Accessibility

The site location is in Kuujjuaq, Quebec, which is situated in northern Quebec approximately 15,000 km north of Montreal. Due to the remote location, all materials and equipment must be shipped by boat and then transported to site by truck. This construction constraint was taken into consideration throughout the entire design phase. It often impacted the types of components selected and the fabrication process. This had the largest impact when selecting the truss system for the hangar, as outlined in Section 8.3.2 Box-Truss System.

2.2.2 Time

Construction in Kuujjuaq only occurs for three months of the year. Due to extreme weather constraints, construction commences in August, for a three month period until late October. Thus the structure had to be easily constructable and leverage pre-fabrication techniques.

2.2.3 Hangar Open Space

Unlike typical structures, aircraft hangars only have exterior columns to allow for the storage of aircrafts inside. This imposed design constraint meant that the entire roof load had to be transferred through a truss system to the exterior columns. A box-truss system was designed to compensate for this constraint as outlined in Section 8.3.2.

2.2.4 Megadoor

To ensure airplanes could easily enter and exit the hangar, it was essential that a megadoor system be designed on the long side of the hangar. This large open area, 77 m long and 19 m high, stimulated a large torque on the structure. To compensate for this design constraint, a lateral bracing system was designed as outlined in Section 8.3.6 and 8.3.7.

2.2.5 Snow Load Accumulation

Due to the varying heights of the hangar and office, as specified by the architectural drawings, a large snow accumulation occurred on the office from the adjacent hangar. Thus, a stronger joist and deck system were designed to compensate for this accumulated load, which is discussed in Section 8.2.1.

2.2.6 Poor Soil Conditions

Due to the poor soil conditions, Frost designed a deep pile foundation to ensure the structural integrity of the hangar at the specified site location. This process is outlined in Section 8.4.2.

2.3 Design Approach

2.3.1 Gantt Chart

After receiving the project requirements set out by the client, the team developed a systematic approach to ensure the client's design requirements were met by the specified deadline. The team, along side advisor Dennis D'Aranco, outlined a logical and time efficient approach which was compiled into a Gantt chart as shown in Figure 3. The Gantt chart outlined the proposed time, and interconnectedness between all design components. Team member contribution can be found in Section 4.0 Division of Responsibility.

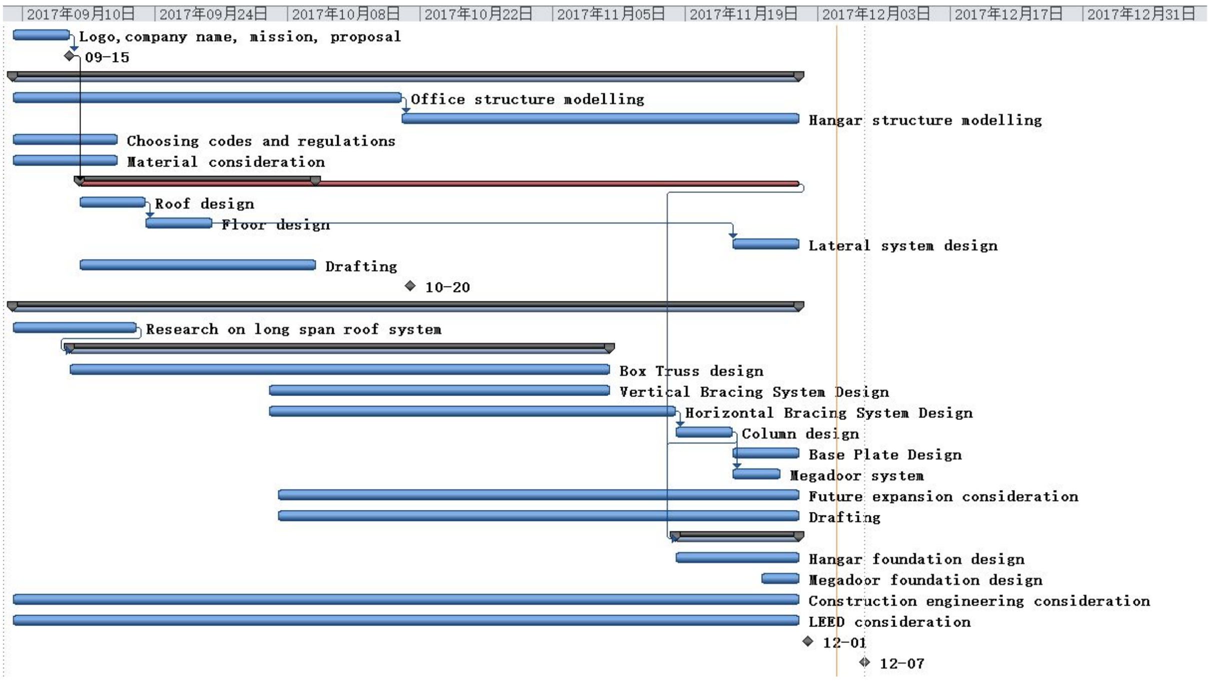


Figure 3. Project Gantt chart

2.3.2 Hangar Design Approach

The predominant feature of the overall structure was the hangar structure and therefore required the most intensive design process. The overall approach consisted of starting from the gravity systems on the roof and moving down towards the foundation design. Figure 4.

outlines the systematic approach taken to design all components. After initially calculating the gravity loads, a gravity system was developed to accommodate for both the open space interior and high snow load exterior through designing a long span box truss system. The preliminary column design was carried out using the reactions from the box-truss system. The hangar had 4 columns on each of the shorter sides, 10 columns along the long side adjacent to the office and no columns on the interior or megadoor side. Decking and purlin design for the hangar roof were completed. Following the box-truss system design, the lateral system was developed to accommodate for the lateral loads and specifically the torque induced by the megadoor. It was assumed that the megadoor would be opened when entrance was desired by the aircrafts, but it should be completely closed otherwise. Both a vertical and horizontal system were designed to effectively transfer the loads. The next portion was to finalize the design of the columns, and design the base plates. Following the base plate design, both the connections as well as deep pile foundation were designed. Finally, a slab on grade design was completed accounting for the gravity loads on the ground floor, governed by the load from the aircraft wheel. Figure 4 summarizes the hangar design procedure.

Figure 4. Hangar design approach

2.3.3 Office Design Approach

Directly adjacent to the hangar is two-storey office, which was designed as a predominantly steel structure. Firstly, the loads on both the roof and second floor were calculated. Next, the decks and joist systems were designed. The roof was comprised of a steel deck and joist systems, the joists were placed to complement the accumulated snow loading experienced on the office roof. The second level of the office was designed using a composite steel and concrete deck to accommodate the interior office load. Exterior beams were designed along the edge of the structure on both floors. Afterwards, the columns were designed based on the accumulated loads from all office levels and two-story columns were selected for feasible assembly purposes. Finally the connections were designed. Other components were designed using the governing cases from the adjacent hangar, including the lateral system, slab on grade and deep pile foundation design. Figure 5. outlines the design procedure for the office.

Figure 5. Office design approach

2.4 Final Deliverables

As specified by the client, three main final deliverables were provided by Frost Consulting Engineers. First, this report was compiled, which summarized the design methodology and considerations. Secondly, the attached appendices were provided to the client, which provided detailed calculations for the structure. Finally, multiple models had been built to carry out the design analysis and ensure efficient constructability of the structure. These included a 3D SAP2000 model, a 3D Sketchup model, and 2D Autocad drawing set.

3.0 Source of Data

3.1 Architectural Drawings

The hangar dimensions were compiled based on the architectural drawings provided by the client. For design purposes, the measurements were converted into metric units and modelled in AutoCAD. Figure 6, Figure 7, and Figure 8 below showed the building section, typical floor plan, and the roof plan respectively. As shown below, the hangar spans 77m in length, 54 m in width, and 19m in height. The office has a length of 50m, a width of 13m, and a height of 10m. The hangar and office constituted a significant part of the structural design.

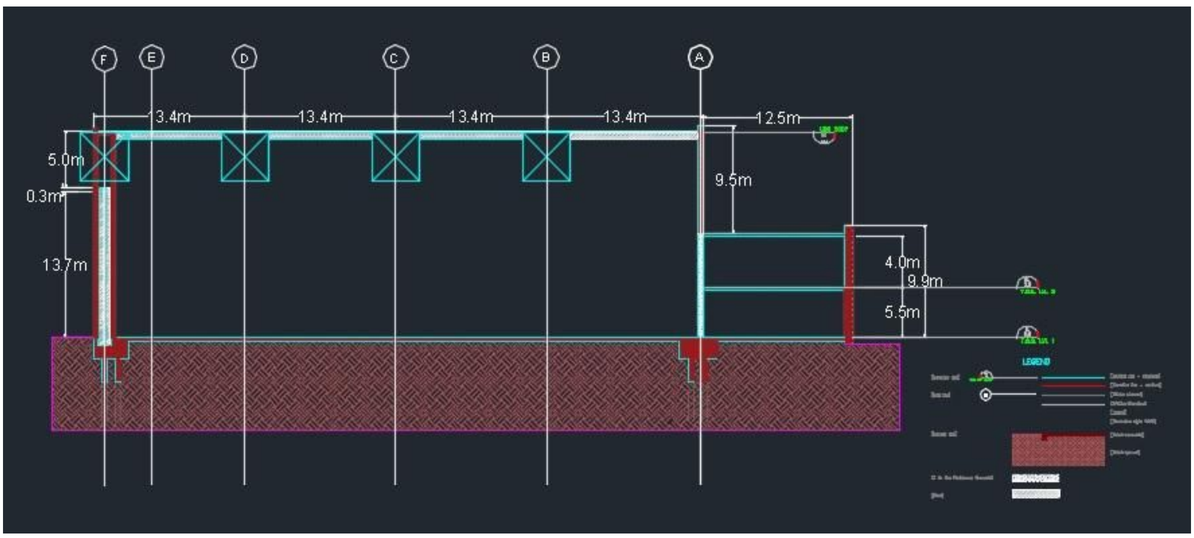


Figure 6. Building section

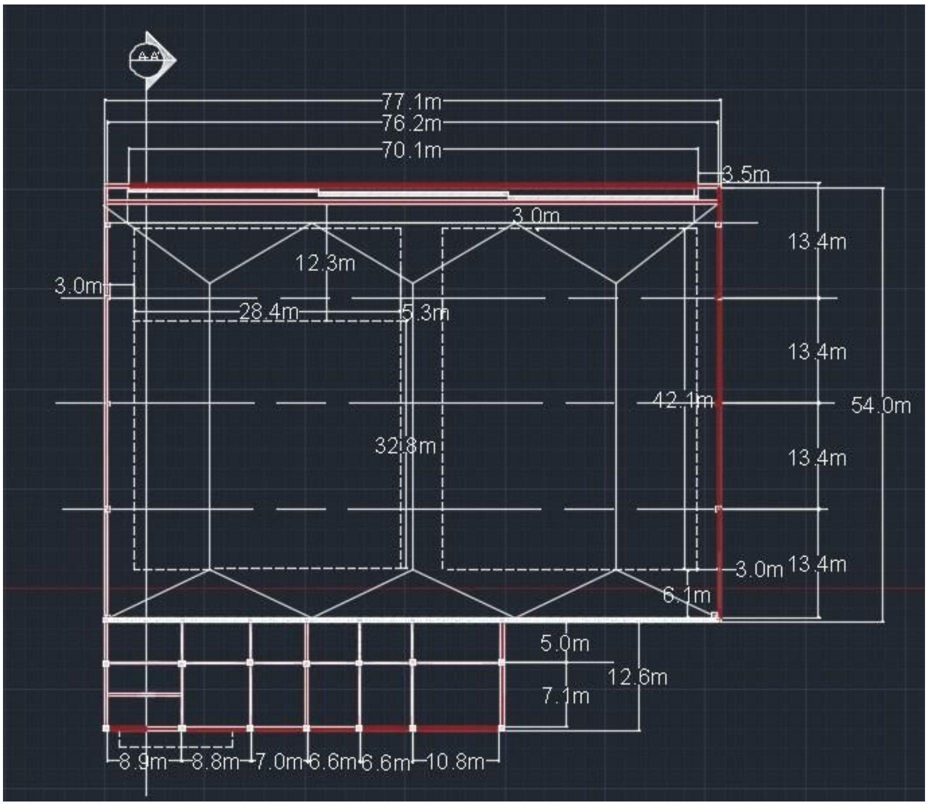


Figure 7. Building typical floor plans

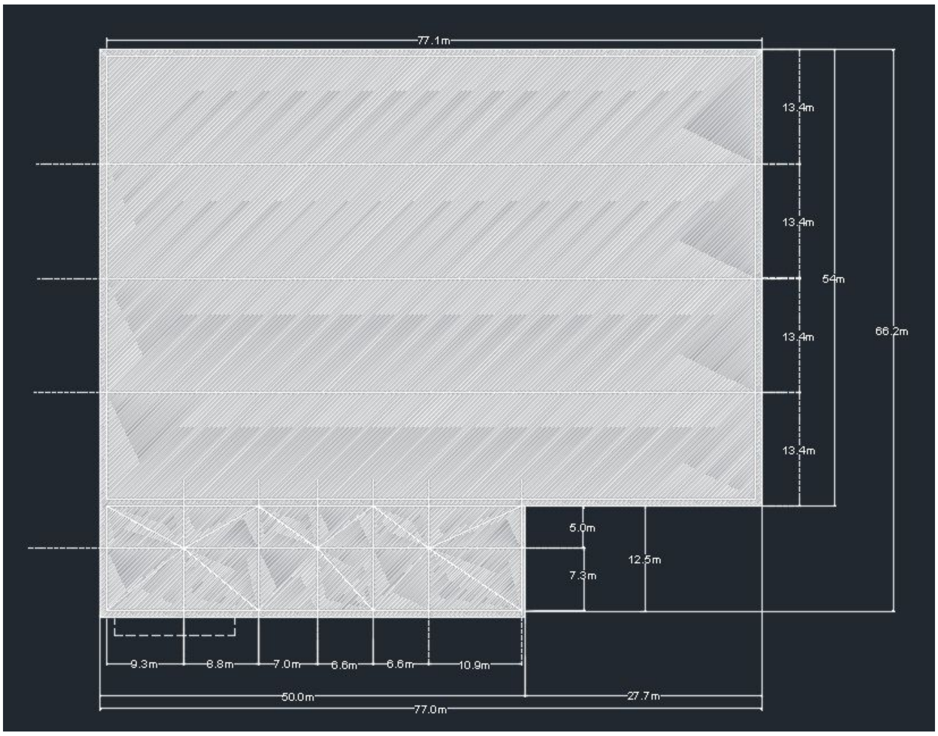


Figure 8. Building roof plan

3.2 Climate Conditions

Specific climate conditions were assessed for the site location in Kuujjuaq. The data collected included average temperature, snow, wind, and seismic data.

The temperature data was collected from S-11. Most notably, it was acknowledged that the average min temperature in Kuujjuaq ranges between -42°C to -44°C.

3.3 Geotechnical Data

For the design of the foundation, consultation was provided by QualiLab Inspection Inc. on the soil conditions in Kuujjuaq. Due to the specified weak soil, a deep foundation was designed.

4.0 Division of Responsibility

The project involved the collaboration and teamwork of all four members at Frost. Due to the nature of this project, a variety of skill sets were required, including the modelling techniques using AutoCAD, Sketchup, SAP2000; the hand calculations of structural loads; and most importantly, the design of all structural members.

Responsibilities were divided based on group member's expertise and capabilities. Frost's team was made up of individuals with experiences in research, industry, academic projects, and academic engineering courses. The chart below outlines the project's division of responsibilities between all group members.

Jacqueline Barbara
Overall
- Material considerations
- Constructability

Office Design
- Joist selection
- Column design
- Beam design

Hangar Design
- Pile design

Ziyi Gu
Overall
- Base plate
- Seismic loads

Hangar Design
- Box truss system
- Horizontal system
- Lateral system
- Column selection
- Hangar SAP2000 model

Kailing Qiao
Overall
- LEED considerations
- Cladding
- AutoCAD modelling
- Concrete mix
- Pedestal design
- Pile cap design

Hangar Design
- Purlin system-SAP2000 modelling

Omar Shemy
Overall
- Sketchup modelling
- Structural loads

Office Design
- Deck selections
- Wind columns

Hangar Design
- Connection design
- Slab on grade

The above chart outlines the team member responsible for each task. While each team member was assigned respective responsibilities, the team also collaborated on many design aspects throughout the project. There was also fluid and constant communication throughout the project to ensure all interconnected components were designed properly and cohesively. To ensure the quality and accuracy of the work, the review process was carried out such that for each member who completed a specific portion of the design, another team member reviewed and verified the calculations independently.

5.0 Codes, Standards and Regulations Objectives

The design of the airplane hangar required the compliance with several codes, standards and regulations. The most commonly used code was National Building Code of Canada (NBCC 2015). Its accompanying commentaries were also consulted for the design of steel structures, connections, and industrial building components. Other references were used to design for the steel deck, joists, and deep foundation.

Noticeably, climatic data was found from the Environment Canada website for references of our seismic loading design, as can be found in the Appendix. The below summarizes the code and standards, as well as some other source of references used in the design process.

Codes and standards:

- National Building Code of Canada 2015
- CSA S16-14 Design of Steel Structures
- CISC Connections For Design Engineers
- CISC Industrial Building Design

Other References:

- CANAM Steel Deck Catalogue
- CANAM joists and Joist-girders Catalogue
- Canadian Foundation Engineering Manual 3rd Ed.
- Environment Canada website
- AISC Steel Design Guide Base Plate and Anchor Rod Design 2nd Ed.

6.0 Materials

Material data came from multiple sources as outlined in Table 1 below.

Table 1. Material Data

Source	Material Location in Structure	Data Collected
Industrial Building Design Guidelines	Hangar Steel Members	Bolt specification, steel coating
S-11	Steel Members	CVN tested materials, anchor rods,
Summary of Surface Preparation Standards	Hangar Steel Members	Steel finish
Concrete properties and durabilities	Foundation, hangar, office concrete	Strength, concrete mix percentage by volume

The overall structure was comprised of steel, concrete, and composite materials to compliment the design.

6.1 Steel

Steel is a widely available construction material, which means that there is a large selection range to accommodate economical considerations and ease fabrication. In addition, steel generally has a high ductility and strength. Steel structures also have higher resistance to poor or harsh weather conditions. Considering all the significant factors, steel was selected to be predominantly used in our structure. The hangar and office areas were mostly designed using steel components. Steel grades of different components were selected based on accessibility to materials and economic factors, as indicated in Table 2. In particular the G40.21 and ASTM A992 were selected due to availability in Quebec. The HSS members were also selected as Class C as a more economic option compared to Class H.

Table 2. Steel component grade selection

Steel Component	Grade
HSS	G40.21 345MPa Class C
Plates	G40.21 300W
Channels	G40.21 300W
Angles	G40.21 300W
Wide Flange	ASTM A992 Grade 50
Anchor Rods	ASTM 1554 Grade 55

When designing the open hangar, special considerations for the steel members had to be considered including extra tests, coatings, and finishes. Firstly, it was concluded that the steel did not have to be CVN tested. CVN test materials are quite costly and required for structures under temperature dependent behaviour or high dynamic loading (S-11 1-215). Since the steel hangar members are only exposed to view and will not undergo these conditions outlined this material was not required.

However, the box truss system and columns are exposed to view and thus special considerations surrounding coating and finish were required. For the coating, a 2-coat system will be employed based on the Zone 1B:CISC CPMA. This standard specifies a 2-75mm Commercial blast cleaning system (Industrial Building Design). To complement the 2-coat system, a SSPC-SP3 Power-tool cleaning finish will be used to to remove rust, mill scale and foreign matter (Industrial Building Design). Finally, galvanized bolts will be used to ensure the bolts are properly treated and rust does not occur during construction. Overall the use of coatings, finishes and galvanized bolts will improve the durability of the exposed hangar system.

6.2 Concrete

Although steel governed most of our structure, the use of concrete was also essential since the slab on grade and foundation design utilized concrete as the primary design material. A geotechnical report was provided by the QualiLab Inspection Inc. to help us with the foundation design. However the presence of sulphate contents was not mentioned. To be conservative, it was assumed that sulfate contents were moderately present in the soil. Another consideration was the cold weather of the site. This meant that the concrete had to have properties to resist freeze-thaw issues. In addition, the slow curing of the cement is expected since the water content would freeze and not evaporate easily from the cement; therefore the setting process would be slow and bleeding would also start later than expected and more bleed water would be produced.

To resolve the issue of the sulphate attack, two measures shall be taken. Firstly, the intrinsic type of the cement was considered. In this case, as revealed by figure 9, since Type II had a moderate resistance against the sulphate attack and would not expand as much as Type I cement, it was chosen as the cement type. Furthermore, the w/c ratio was controlled as low as 0.45, in order to decrease the relative rate of deterioration, as revealed in Figure 10. This was accompanied by the addition of the superplasticizer to increase the workability of the concrete.

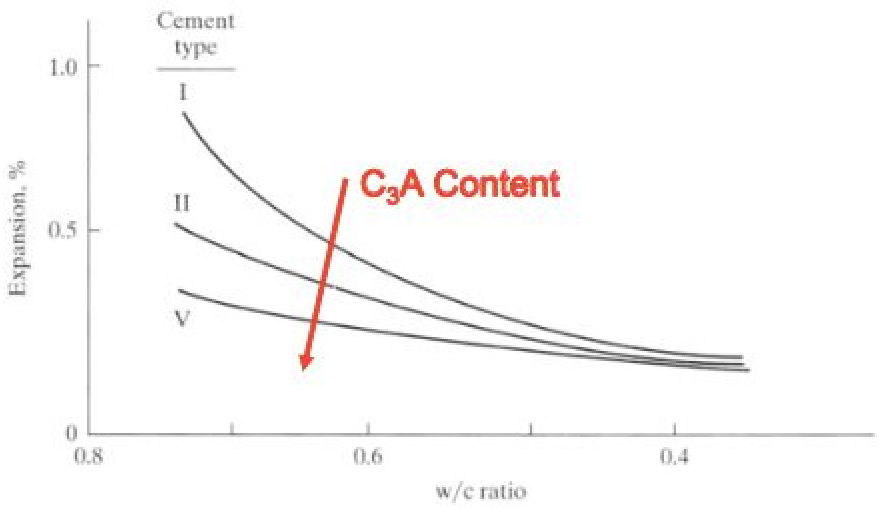


Figure 9. Expansion of cement due to the C_3A content (Andrew J. Boyd, 2016)

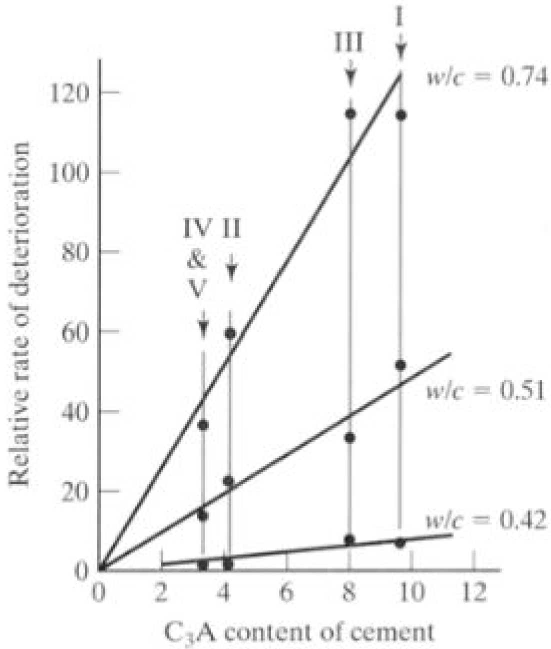


Figure 10. Deterioration rate with respect to C3A content (Andrew J. Boyd, 2016)

As mentioned above, the cold weather of the site required additional considerations. Entrained air was vital in the resistance of freeze-thaw cycles as well as the sulphate attacks. For this reason the entrained air pockets were small, non-interconnected, and dispersed but still close to each other. The proposed air entrainment content is 5% of the total volume. The minimum curing period for ASTM Type II concrete is 10 days. During the 10 day period, the concrete needs to be protected by insulating blankets to keep the curing concrete at a constant temperature. A minimum compressive strength of 30 MPa was obtained to complement the compressive strength chosen for all concrete members in the structure. Figure 11. Below is the concrete mix for our structure.

Concrete Mix

- Portland cement (Type II) 0.10000V
- Water 0.04500V
- Course aggregates 0.55600
- Fine Aggregates 0.28766
- Superplasticizer Admixture 0.01134

Figure 11. Concrete mix of structure

6.3 Deck

Two types of decking systems were designed including a steel deck on the roof and composite concrete steel deck on the second floor of the office. The roof deck was designed based on the snow loads on the hangar and accumulated snow load on the office roof. The 2nd floor of the office was designed using a composite concrete-steel deck to ensure simple construction and a flat surface for the office space above. Table 3. outlines the selected decks.

Table 3. Deck Selection for structure

Location	Material	Specifications
Hangar Roof	Steel	Canam P-2436 Type 22 0.76 mm
Office Roof	Steel	Canam P-2436 Type 20 0.91 mm
Office 2nd Floor	Composite Steel-Concrete	Canam P-3615 composite Type 22 115mm concrete, 0.91 mm steel

7.0 Loading

7.1 Dead Load

Dead loads have small variations over time but the consistent loading can result in member deflection. The maximum deflection should be smaller than the specified limits. Frost accounted for the weight of the partitions, concrete and steel deck self-weight in the dead load calculations. For the hangar roof, a uniform dead load of 1.2 kPa was specified for the roof of the office For the office, a uniform dead load of 1.34 kPa was specified for the roof and 3.77 kPa on the floor, that includes the self weight of the composite concrete on steel deck.

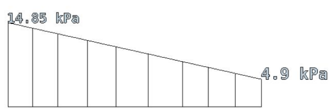
7.2 Live Load

The typical specified live loads were obtained from NBCC 2015 Clause 4.1.5, where the code specifies a minimum live load for the roof of the office and hangar of 1.0 kPa. For the office floor a minimum of 4.8 kPa was used as specified by the code for floors above the first storey. The live loads introduced to the hangar floor by the aircraft will be discussed in the design of a slab on grade in Section 8.4.1.

7.3 Snow Load

Snow loads on the roofs of the hangar and the office were obtained using the NBCC 2015 Clause 4.1.6. For the roof of the hangar, the snow load was calculated as a uniform 2.12 kPa, a low value compared to the snow load on the office. This low value for the hangar load is due to the fact that the structure is located north of the treeline and there is no drift from a higher roof, so a 50% reduction for the wind exposure factor could be applied as per clause 4.1.6.2 sentence (4). However, on the roof of the office there was a high snow accumulation on the interface of the hangar and the office due to snow drift from the hangar and so the resulting non uniform snow loads varied from 14.85 kPa to 4.9 kPa. Table 4. below summarizes the gravity loads on the structure.

Table 4. Summary of gravity loads

Load	**Office Specific Loading (kPa)**	**Hangar Specific Loading (kPa)**	**Considerations**
Dead	Roof: 1.34 Floor: 3.77	Roof: 1.2	• Partitions • Concrete & Steel Deck SW
Live	Roof: 1.0 Floor 4.8	Roof: 1.0	• Minimum roof live load • Floors above the first storey
Snow	 14.85 to 4.9	2.12	• Normal Importance • North of Treeline

7.4 Wind Load

The wind loads were determined using NBCC 2015 Clause 4.1.7. The wind pressures were obtained for Kuujjuaq Quebec from the Appendix of climatic data. The structure was located in an open terrain with maximum height H < 20 m.

7.4.1 Primary structural action

As the office and hangar are connected, a virtual box encompassing the dimensions of both combined was used in the analysis as shown below in Figure 12. Further, Figure 4.1.7.6 A was used for the peak values of the external pressure coefficients.

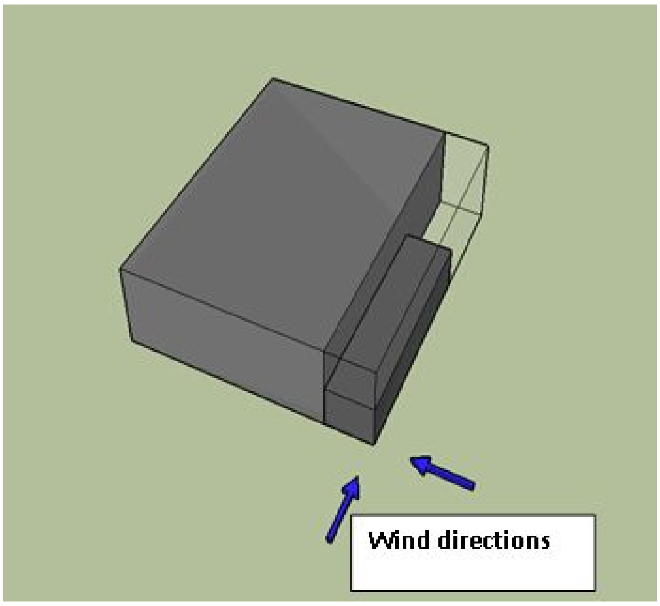


Figure 12. Shell encompassing Hangar and office

7.4.2 Walls and roofs

On the other hand, the pressures experienced by the individual walls and roofs were computed using Figure 4.1.7.6 - B and Figure 4.1.7.6 - C respectively by considering each structure individually. Moreover, the dimensions of the office building were used to compute pressure coefficients along with end zone widths and the same procedure was applied for the hangar. Both roofs were considered flat roofs but an additional check had to be made for the roof of the office using Figure 4.1.7.6 - D. The additional check was required due to the portion of the office connected to the hangar, which would made the office's roof a stepped roof. However, the height difference was not significant enough to generate additional wind pressures on the office building roof. All the individual pressure components are summarized in the Appendix for wind calculations.

7.4.3 Internal pressure

For both the office and hangar a category 2 internal pressure coefficient was used, (non uniformly distributed opening of which none is significant or significant opening that are wind-resistant and closed during storms) as per Table 4.1.7.7. Table 5. summarizes the wind loading for the structure.

Table 5. Wind loading summary

Load	Acting on	Pressure (kPa)	Considerations
Wind	Windward hangar and office	0.45 and 0.69	**Open terrain** **H < 20 m** **Internal Pressure coefficient category 2**
	Leeward hangar and office	0.33 and 0.48	
	Internal	0.36 to - 0.54	

7.5 Seismic Load

Seismic induced base shears were determined for the hangar and office structure by consulting the NBCC 2015 Volume 1 Division B 4.1.8 Earthquake Load and Effects. Seismic data was obtained from NBCC 2015 Volume 1 Division B Appendix C Table C-3. Kuujjuaq is located in a low relative hazard seismic zone, as shown in Figure 13. The height above grade of the structure is greater than 15 m for the hangar. Thus the equivalent static force procedure was used to calculate the minimum lateral earthquake force.

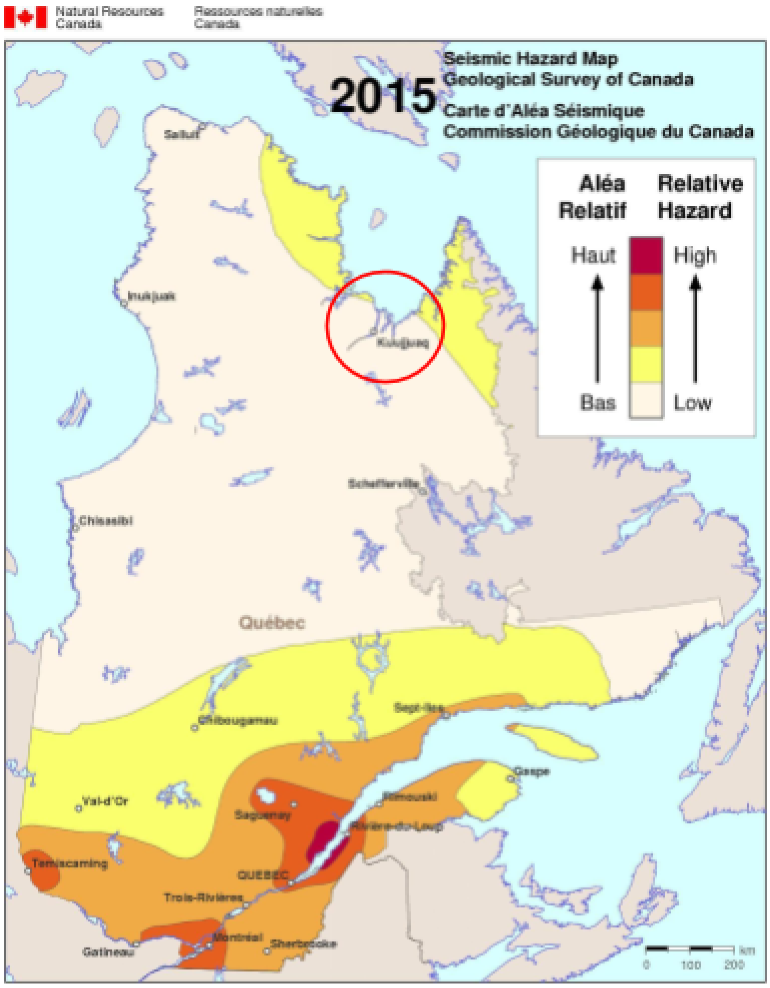


Figure 13. Seismic hazard map of Quebec. From "Seismic Hazard Map", by Natural Resources Canada, 2015.

Seismic base shear were determined for the whole structure and were compared with the wind base shear. The weight of the structure was determined based on the member selection of hangar and office. Both the hangar and office use conventional construction of braced frames, R_d and R_o, which were determined to be 1.5 and 1.3 respectively. The coefficient M_v, to account for higher modes of vibration, was determined to be 1.The minimum lateral earthquake force V was then be calculated using the clause 4.1.8.11. The results were shown below in Table 6. The wind base shear was higher than the seismic base shear in both long and short direction, thus the wind force governed the lateral system.

Table 6. Wind base shear and seismic base shear comparison

<table>
<tr><td rowspan="3">Wind Load</td><td>Acting on</td><td>Base Shear (kN)</td><td>Base Shear (kN)</td><td rowspan="3">Seismic Load</td></tr>
<tr><td>Long direction</td><td>1705</td><td rowspan="2">391.4</td></tr>
<tr><td>Short direction</td><td>1423</td></tr>
</table>

8.0 Analysis and Design

8.1 Computer Software

The computer software's used during the design of the overall structure included:

- SAP2000: utilized for hangar design for both 2D and 3D design
- AutoCAD: modelled 2D plan and section view of structure including dimensions in metric units
- SketchUP: used to show general visualization of structure in 3D
- Microsoft Project: utilized to build Gantt chart for scheduling purposes
- Microsoft Excel: column calculations and selections, lateral system forces
- MathCAD: used as design templates to carry out calculations

8.2 Office Design

An office space was designed adjacent to the hangar structure as shown in Figure 14. The office is a two-storey structure and primarily made of steel. The architectural drawings specified that the structure was designed to have 6 main bays, depicted in Figure 15 .

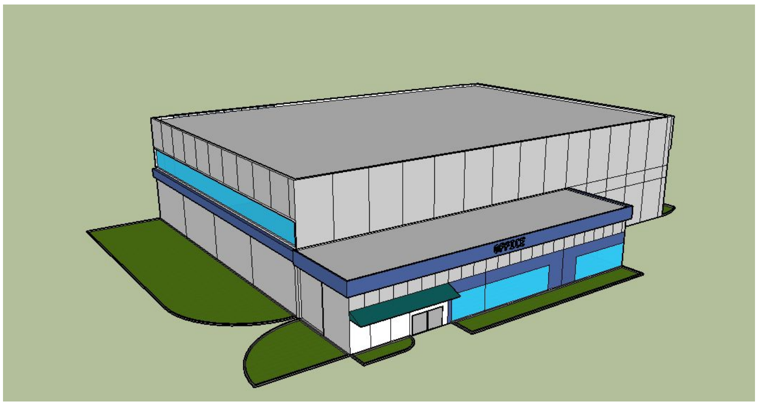


Figure 14. 2-Storey office structure

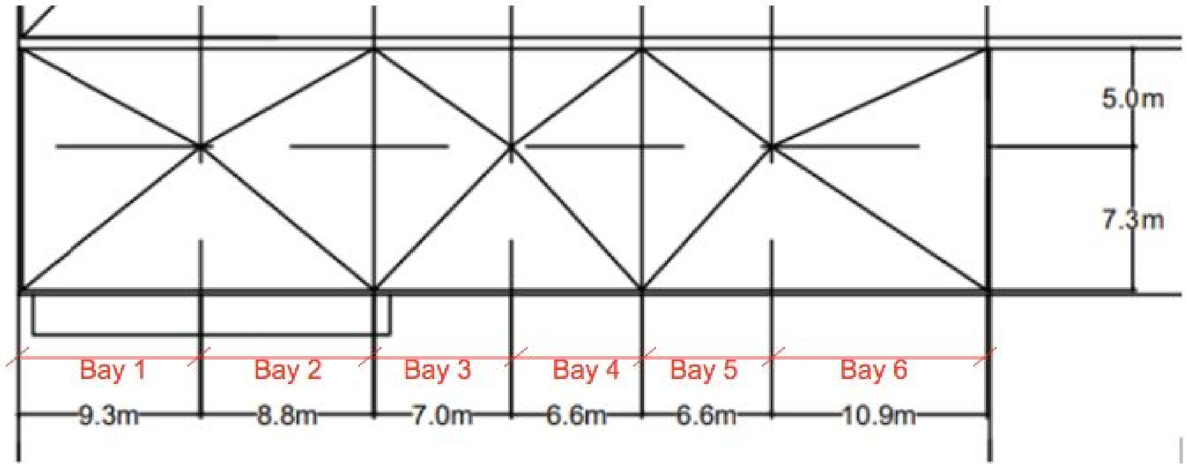


Figure 15. Office Plan View with 6 Bays

After calculating the office loads in section 7, most notably the accumulated snow load on the roof, the design of the structure was completed. The process included selecting the deck, determining the joist spacing, sizing the joists and beams, determining the column sizes, designing the connections, and designing the lateral system.

8.2.1 Deck

An adequate deck was selected for the office roof by comparing the accumulated loads on the roof with the Canam Steel Deck Catalogue. A steel deck was chosen over a concrete-steel composite deck because it could both support the roofs loads and was a more economic choice. The deck was designed to be adequate under the maximum moment and deflection.

8.2.1.1 Roof Deck

For the office roof, a P-2436 Type 20 steel deck was selected as it satisfied the necessary requirements. Figure 16. summarizes the physical properties of the deck.

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M^+	Section Modulus M^-	Moment of Inertia for Deflexion
	mm (in.)	mm (in.)	mm (in.)	kg/m^2 (lb/ft^2)	mm^3 (in^3)	mm^3 (in^3)	mm^4 (in^4)
22	0.76 (0.030)	0.762 (0.0300)	76.2 (3.00)	11.85 (2.43)	24 134 (0.4489)	25 690 (0.4778)	1 006 306 (0.7369)
20	0.91 (0.036)	0.909 (0.0358)	76.4 (3.01)	14.04 (2.88)	29 407 (0.5470)	31 169 (0.5797)	1 262 487 (0.9245)
18	1.21 (0.048)	1.217 (0.0479)	76.7 (3.02)	18.33 (3.75)	40 633 (0.7558)	41 655 (0.7748)	1 819 220 (1.3322)
16	1.52 (0.060)	1.511 (0.0595)	77.0 (3.03)	22.71 (4.65)	51 473 (0.9574)	51 681 (0.9613)	2 294 846 (1.6805)

Figure 16. Physical Properties of P-2436 Type 20 Office Steel Roof Deck (Canam, 2006).

The Canam deck catalogue was used to determine the minimum spacing based on the allowable deflection. Considering the P-2436 Deck Type 20 double span, and based on the deflection calculation, it was determined that a minimum of 1200 mm spacing was required. To satisfy the architectural drawings, the spacing was decreased to 833 mm and 811 mm, making the design conservative. Figure 17. shows how the P-2436 Deck satisfies the spacing selected.

FACTORED AND SERVICE LOADS TABLE (kPa) — METRIC

Type	Nominal Thickness (mm)		SPAN (mm)												
			1 200	1 350	1 500	1 650	1 800	1 950	2 100	2 250	2 400	2 550	2 700	2 850	3 000
			SINGLE SPAN												
22	0.76	F	10.69	8.49	6.90	5.72	4.82								
		D	7.60	5.34	3.89	2.92	2.25								
20	0.91	F	12.95	10.29	8.37	6.93	5.84	4.98							
		D	9.58	6.73	4.90	3.68	2.84	2.23							
18	1.21	F	17.70	14.06	11.44	9.48	7.98	6.82	5.89	5.13					
		D	13.66	9.60	7.00	5.26	4.05	3.18	2.55	2.07					
16	1.52	F	22.14	17.59	14.31	11.86	9.99	8.53	7.36	6.42	5.65				
		D	17.01	11.95	8.71	6.54	5.04	3.96	3.17	2.58	2.13				
			DOUBLE SPAN												
22	0.76	F	11.11	8.85	7.22	5.99	5.05	4.32	3.73						
		D	18.21	12.86	9.38	7.04	5.43	4.27	3.42						
20	0.91	F	13.23	10.54	8.59	7.14	6.02	5.14	4.44	3.88					
		D	20.67	16.20	11.81	8.87	6.84	5.38	4.30	3.50					
18	1.21	F	17.63	14.05	11.45	9.51	8.02	6.85	5.92	5.17	4.55	4.03	3.60		
		D	32.92	23.12	16.85	12.66	9.75	7.67	6.14	4.99	4.11	3.43	2.89		
16	1.52	F	21.82	17.39	14.17	11.77	9.92	8.48	7.33	6.39	5.63	4.99	4.46	4.00	3.62
		D	40.97	28.78	20.98	15.76	12.14	9.55	7.65	6.22	5.12	4.27	3.60	3.06	2.62
			TRIPLE SPAN												
22	0.76	F	(13.60)	10.88	8.90	7.40	6.25	5.35	4.63	4.04					
		D	14.35	10.08	7.35	5.52	4.25	3.34	2.68	2.18					
20	0.91	F	16.19	12.96	10.59	8.82	7.45	6.37	5.51	4.82	4.24	3.77			
		D	18.08	12.70	9.26	6.96	5.36	4.21	3.37	2.74	2.26	1.88			
18	1.21	F	21.59	17.27	14.12	11.75	9.93	8.49	7.35	6.42	5.65	5.02	4.48	4.03	
		D	25.80	18.12	13.21	9.92	7.64	6.01	4.81	3.91	3.22	2.69	2.26	1.93	
16	1.52	F	26.72	21.38	17.47	14.54	12.28	10.51	9.09	7.94	6.99	6.21	5.55	4.98	4.50
		D	32.11	22.56	16.44	12.35	9.52	7.48	5.99	4.87	4.01	3.35	2.82	2.40	2.06

Figure 17. P-2436 Type 20 Office Steel Roof Deck Deflection Check

8.2.1.2 2nd Floor Deck

A concrete-steel composite deck was selected for the second floor of the office. A composite deck was selected such that the concrete top provided the base of the second floor. The process to determine the deck selection was carried out similar to the roof procedure. Using the Canam Deck Catalogue, a P-3615 Composite Type 22 was selected to satisfy the design requirements. Figure 18. summarizes the physical properties of the 2nd floor deck.

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M+	Section Modulus M−	Moment of Inertia	Steel Area	Center of Gravity
	mm (in.)	mm (in.)	mm (in.)	kg/m² (lb/ft²)	mm³ (in³)	mm³ (in³)	mm⁴ (in⁴)	mm² (in²)	mm (in.)
22	0.76 (0.030)	0.762 (0.0300)	37.4 (1.47)	8.50 (1.74)	9 529 (0.1772)	10 081 (0.1875)	202 228 (0.1481)	1 016 (0.480)	22.50 (0.89)
20	0.91 (0.036)	0.909 (0.0358)	37.5 (1.48)	10.07 (2.06)	11 558 (0.2150)	12 005 (0.2233)	254 750 (0.1865)	1 212 (0.573)	22.58 (0.89)
18	1.21 (0.048)	1.217 (0.0479)	37.8 (1.49)	13.26 (2.72)	15 813 (0.2941)	15 994 (0.2975)	363 493 (0.2662)	1 622 (0.766)	22.73 (0.89)

Figure 18. Physical Properties of P-3615 Composite Type 20 Office 2nd Floor Deck (Canam, 2006).

The Canam deck catalogue was again used to determine the minimum spacing based on the allowable deflection for the office second floor. Based on the factored deflection, a spacing of 1670 mm and 1460 mm was adequate.

8.2.2 Joist

8.2.2.1 Roof Joists

Joists were designed on the office roof to support the gravity loads. The joists were placed horizontally, as depicted in Figure 19, to optimize in the transfer of loads from the accumulated snow load from the roof to the columns. Based on the calculations in Section 8.2.1 Deck, it was determined that the area of the office adjacent to the hangar, which received the highest amount of accumulated snow, would have a spacing of 833 mm while the portion adjacent to this area would have a spacing of 811mm. A summary of the roof spacing is depicted in Figure 19. It is worth noting that the spacing was actually larger in the areas where the accumulated snow load was higher due to geometry constraints of the building. However, both spacings are satisfactory for the required gravity loads.

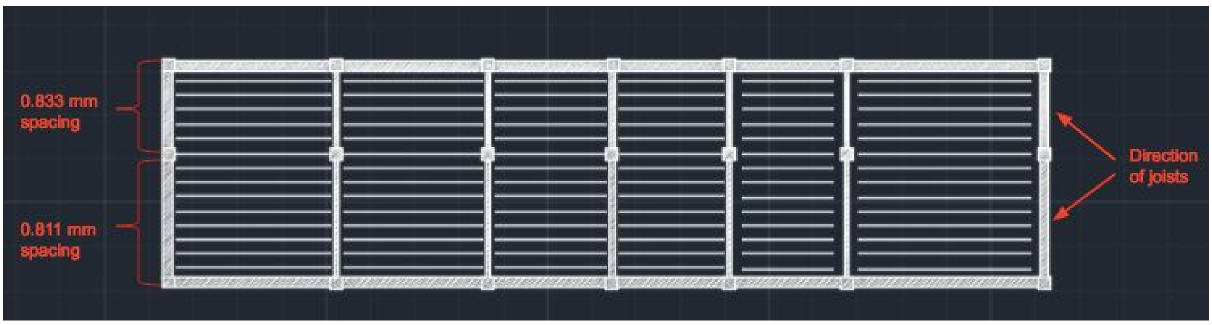


Figure 19. Direction and Spacing of Joists on Roof

The Office area was divided into 12 sections based on the span length and spacing requirements, as shown in Figure 20. The Canam Joist catalogue was used to select the adequate joists based on the loads and selected spacing. Figure 21. shows how the joist depth was selected from the joist catalogue. To optimize the design efficiency, the lightest section was selected. This selection process was carried out for the entire roof of the office and the joist specifications are summarized in Table 7. below.

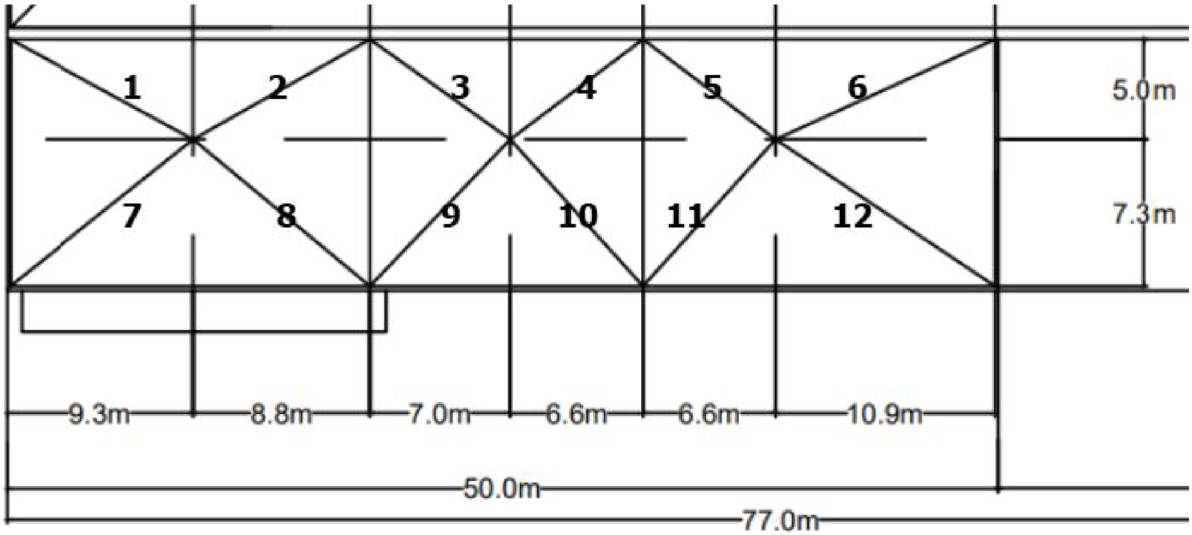


Figure 20. Office Area Divided Sections for Joist Selection

Span (m)	Joist depth (mm)	Factored load (kN/m) Service load (kN/m)												
		4.5 3.0	6.0 4.0	7.5 5.0	9.0 6.0	10.5 7.0	12.0 8.0	13.5 9.0	15.0 10.0	16.5 11.0	18.0 12.0	19.5 13.0	21.0 14.0	22.5 15.0
	500	11.6 66	13.5 65	16.8 64	18.2 65	21.8 65	24.7 64	31.5 64	33.1 64	33.6 64	37.0 65	42.0 68	45.5 69	45.5 64
	550	10.5 70	13.3 68	13.9 68	15.6 65	18.4 65	20.2 63	24.6 65	28.3 65	28.3 64	30.0 64	33.3 64	36.1 67	38.4 64
	600	11.1 83	13.2 77	13.6 76	14.4 70	17.2 71	18.8 69	21.8 67	23.9 65	24.8 67	26.4 65	28.6 64	31.7 65	35.2 68
10	650	11.8 132	13.4 112	13.7 89	14.2 83	16.0 78	17.8 76	20.7 74	22.7 72	23.2 72	25.3 73	27.0 69	28.9 70	31.8 72
	700	11.9 153	13.5 14	13.8 104	14.3 87	15.4 85	17.2 85	19.9 81	22.3 80	22.3 76	24.8 83	25.2 75	26.7 75	29.9 80
	750	12.1 177	13.6 133	14.0 120	14.4 100	15.7 95	16.8 98	18.3 90	19.9 90	21.6 87	23.[illegible] [illegible]	25.0 88	26.5 89	28.3 87
	800	12.3 200	13.7 172	14.1 137	14.5 114	16.0 98	17.1 95	19.3 100	21.9 96	21.9 93	22.9 95	24.1 93	26.0 94	27.4 93

Figure 21. Joist Catalogue Sample Joist Depth Selection for Section

Table 7. Office Roof Joist and Beam Selection

Area Section	Span length (m)	Spacing (m)	Joist depth (mm)	Joist mass (kg/m)
6	11	0.833	900	29.1
3,4,5	7	0.833	600	19.0
2	9	0.833	750	23.6
1	10	0.833	800	27.4
7	10	0.811	800	26.0
8	9	0.811	750	22.5
9,10,11	8	0.811	600	17.6
12	11	0.811	900	27.8
Single Joist between 1 & 7	10	0.822	750	21.6
Single Joist between 2 & 8	9	0.822	750	19.9
Single Joist between 3 & 9, 4 & 10, 5 & 11	7	0.822	600	15.8
Single Joist between 1 & 7	11	0.822	800	23.3

8.2.2.2 2nd Floor Joists

The same process for selecting the joists on the 2nd floor of the office was followed. However, it is worth noting that when designing the second floor, the joist size governed the design, so a stronger deck with a narrower spacing was selected to compliment the joist choice. Figure 22. shows the required spacing for the office 2nd Floor and Table 8. summarizes the joists selected for the office second floor.

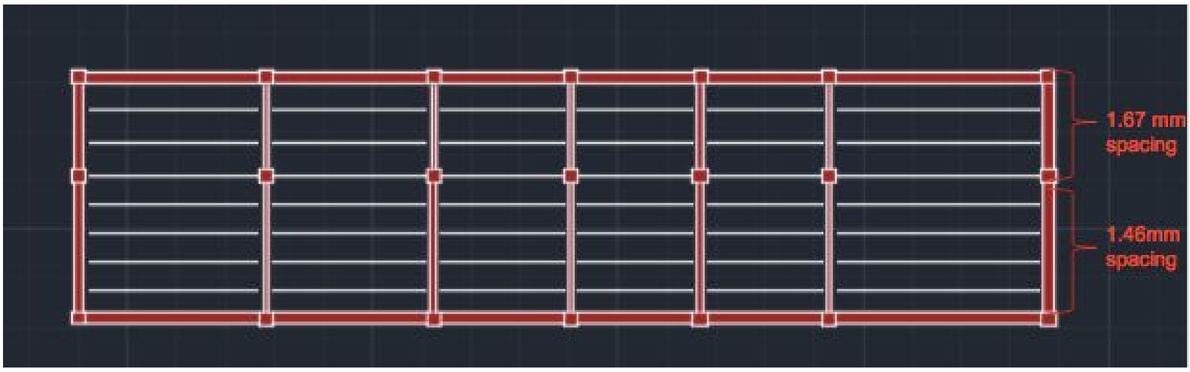


Figure 22. Direction and Spacing of Joists on Second Floor

Table 8. Office 2nd Floor Joist Selection

Area Section	**Span length (m)**	**Spacing (m)**	**Joist depth (mm)**	**Joist mass (kg/m)**
6	11	1.67	900	29.1
3,4,5	7	1.67	600	19.0
2	9	1.67	750	23.6
1	10	1.67	800	27.4
7	10	1.46	800	27.4
8	9	1.46	750	21.4
9,10,11	8	1.46	600	17.6

12	11	1.46	900	29.1
Single Joist between 1 & 7	10	1.57	800	26
Single Joist between 2 & 8	9	1.57	750	23.6
Single Joist between 3 & 9, 4 & 10, 5 & 11	7	1.57	600	17.6
Single Joist between 1 & 7	11	1.57	800	29.1

8.2.3 Beams

8.2.3.1 Exterior Beams

After determining the adequate joists sizes, the beams were designed along the exterior of the structure. Figure 23. shows where the exterior beams are situated on the office roof and Figure 24. shows the layout for the different types of edge beams. Table 9. summarizes all the exterior beams selected. The exterior beams were designed to satisfy the maximum shear, moment, and deflection.

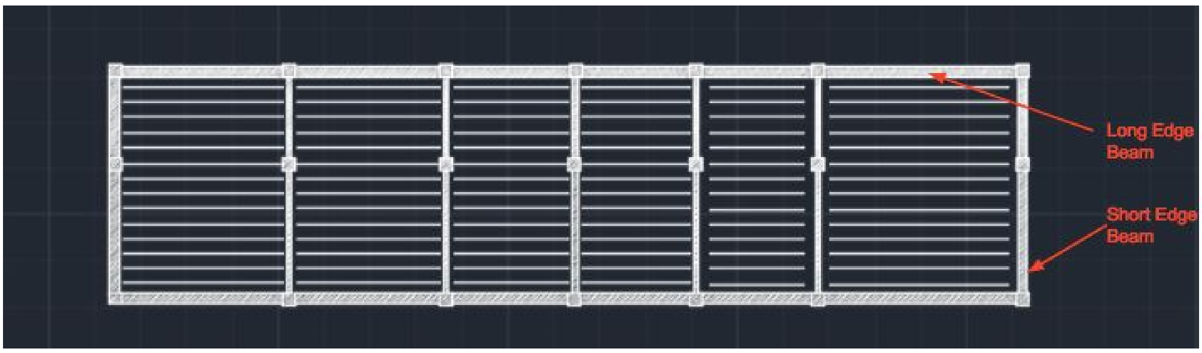


Figure 23. Exterior beams on office roof

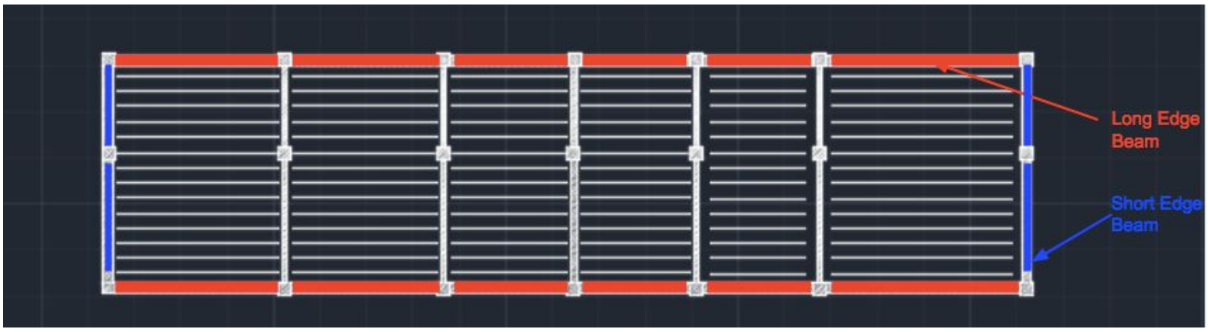


Figure 24. Exterior beams types

Table 9. Exterior beams table

Type	Location	Member
Roof	Short Edge, Joist Bear on Beam	W460x144
Roof	Long Edge	W310x79
2nd Floor	Short Edge, Joist Bear on Beam	W460x144
2nd Floor	Long Edge	W250x89

8.2.3.2 Interior Beams

Interior Beams were designed to carry the gravity loads from the joists and deck on both the roof and 2nd floor level. Figure 25. and Figure 26. outline where the beams were designed. For constructability purposes, the interior beams were designed based on the governing case and repeated vertically across the bays. The roof and 2nd floor were designed using W610x92 as the interior beams.

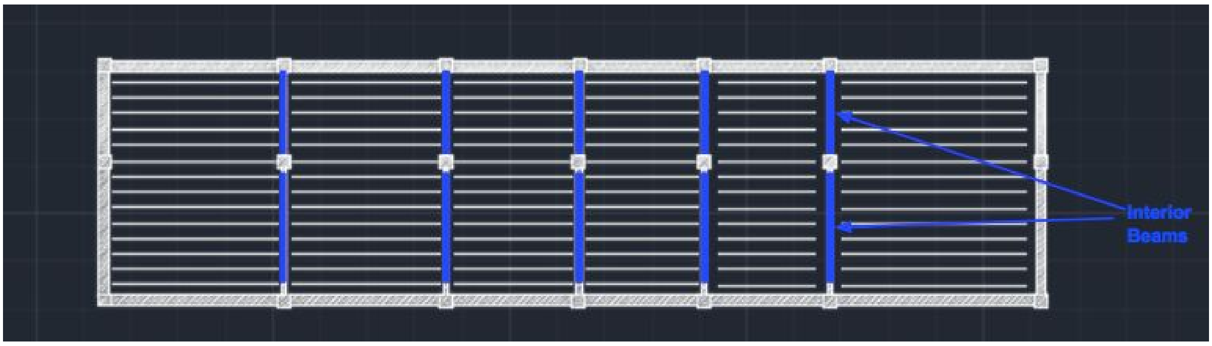


Figure 25. Interior beams office roof

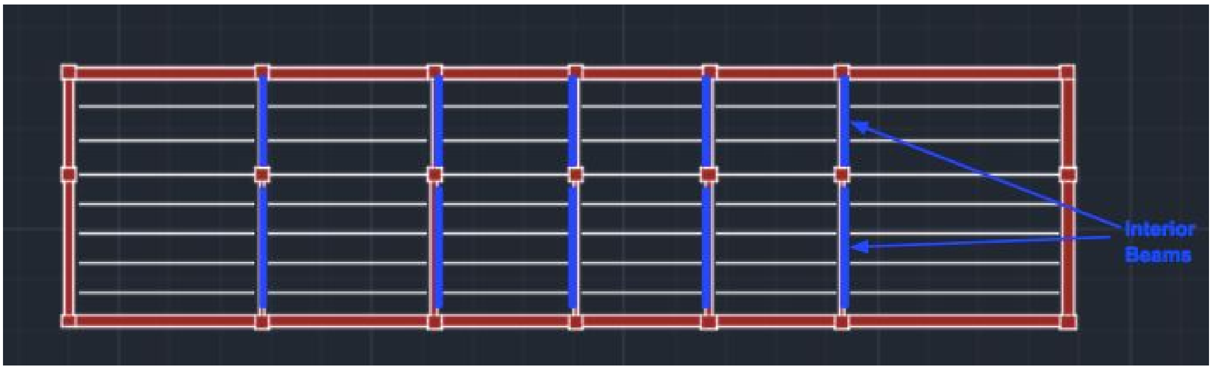


Figure 26. Interior beams office 2nd floor

8.2.4 Columns

Since the office structure wasn't analyzed using a 3D Model in SAP2000, two critical columns were chosen for design purposes. These columns were revisited from the initial column selections due to gravity loads and re-modelled as beam columns. The beam columns thus took accounted for the eccentric loading of the beams onto the columns due to the shear tab connections, as shown below in Figure 27, as well as the wind loading around the exterior of the building.

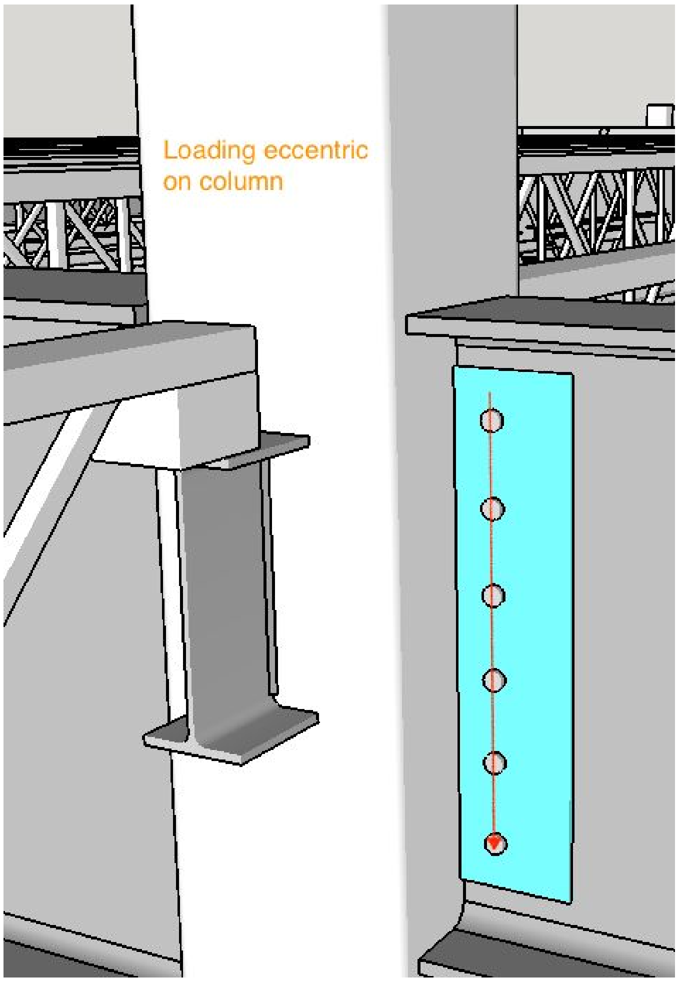


Figure 27. Bracket and shear tab eccentric on column

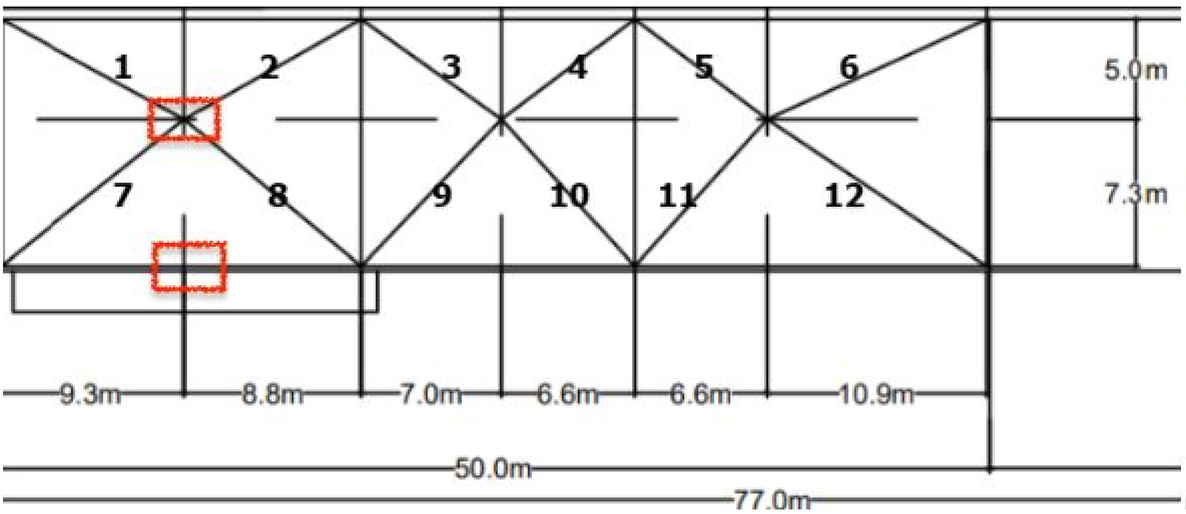


Figure 28. Beam columns selection

The central column between areas 1-2 and 7-8 was the most critical as shown in Figure 28., carrying the gravity load from the snow accumulation due to drift as well as having two different eccentric reactions from the beams in the y direction. This column was checked under 1.25 D + 1.5 S + 1.0 L and it was designed as an HSS 254x254x13. Another edge column between areas 7 and 8 was carrying the largest tributary area from gravity as well as wind loading in the N-S direction. This columns was checked under 1.25 D + 1.5 S +1.0 L (gravity only case) as well as 1.25 D + 1.4 W + 0.5 S (gravity + wind case) and it was designed as a HSS 254x254x16. These edge and center columns were repeated throughout the office.

8.2.5 Connection Design

8.2.5.1 Joist to Beam Connection

The joists in the hangar sit on the beams, as shown below in Figure 29. and Figure 30. To meet the design requirements, as stated by CANAM, the joist shoe must have a bearing spacing of at least 100 mm onto the beam.

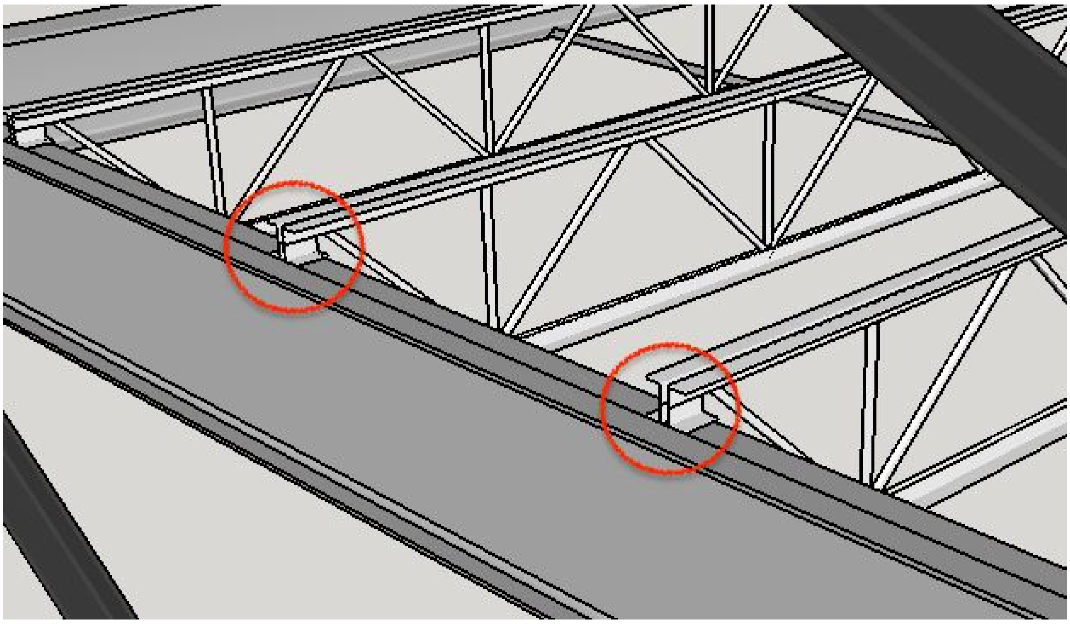


Figure 29. Joists resting on exterior beams

The joists are then bolted into place onto the flange of the beam using the standard connection detail shown below, using two ¾'' A325 bolts.

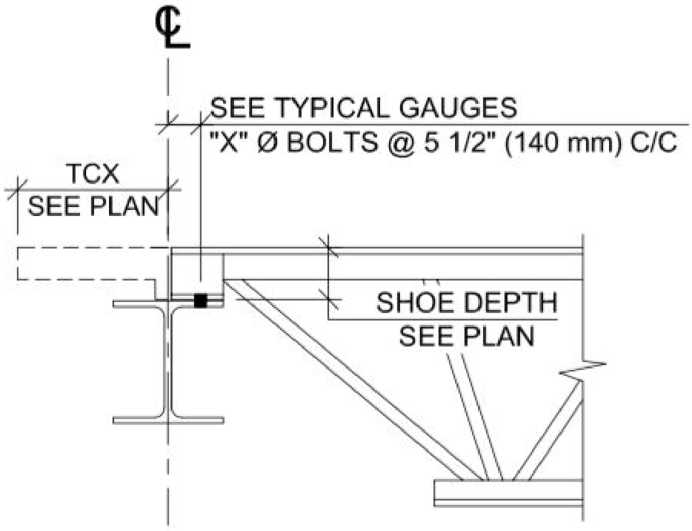


Figure 30. Standard joist detail

The beam selections made for the office space were also designed with this consideration and therefore they have sufficient flange width for the required bearing and bolt gauge.

8.2.5.2 Joist to HSS Column Connection

For the critical central and edge gravity columns on the office side, Frost had to ensure that their stability was not an issue through the use of brackets for the joists meeting the columns and standard shear tabs. For the bracket scenario, the most critical joist reaction was studied and a W250x89 bracket that was 110 mm long was chosen for the joist as shown below in Figure 31. The bearing and shear resistance of the bracket and the weld at the interface of the web and HSS, were checked to ensure they could carry the shear force from the joist. For a more detailed procedure, see the attached Appendix. Furthermore, the eccentricity from the shear reaction on the HSS column was considered in the modelling of the column as a beam column as discussed in the previous section.

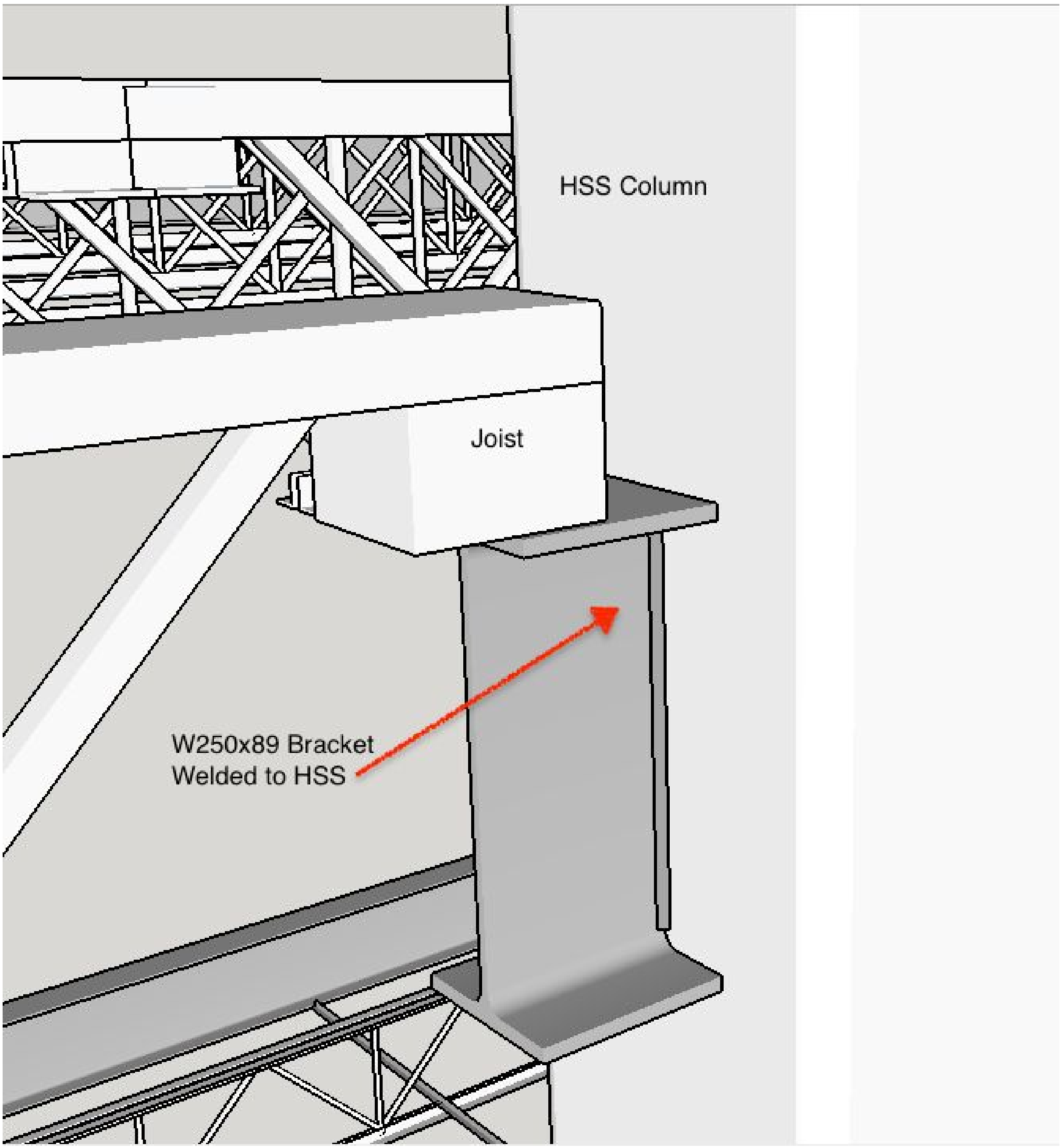


Figure 31. Bracket welded to column

8.2.5.3 Beam to HSS or W Shape Column Connection

The beams within the office building were analyzed and designed as simply supported so the ends of the beam only transfer shear forces to the columns. Thus, only a simple connection was needed to carry the shear forces. Standard shear tab beam connections specified in the steel handbook were used as per Table 3-41. The connections are bearing type connections. One inch A325 bolts were selected with G40.21-300 W plates and 490 MPa electrodes. For example, on the roof level a W610x92 had to transfer a factored shear force of 470 kN. Therefore, from Table 3-41, a 480 mm plate with 6, 1 inch bolts were selected. Table 10.

below summarizes the size of the plate, spacing and number of bolts varies depending on the required shear resistance.

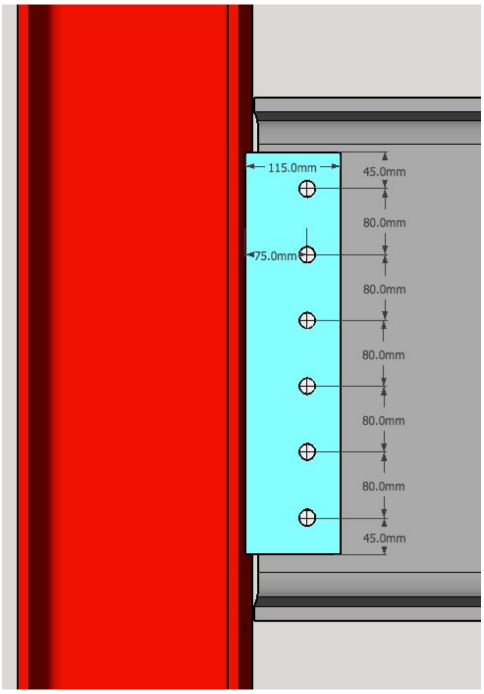


Figure 32. Shear tab connection

It was ensured that the web of the beam had more than the plate length requirement in order to allow for sufficient space for installing the bolts during construction.

Table 10. Shear tab connection summary

Number of bolts	Plate length (mm)	Resistance (kN)	Plate thickness (mm)	Weld size (mm)
6	480	607	12	10

8.2.6 Lateral System

The office and the hangar are connected, so the office could not be studied alone when considering the lateral loads. For example, when the wind is blowing in the N-S the full wind load was assumed to be carried by the hangar. As discussed later in the hangar design section, although the braces on the office side assist in carrying some of the lateral load, the braces on the hangar side were assumed to carry all the loads for simplicity of design. Braced bents were placed on the office side as shown below, but it was assumed that they would not assist the braces on the hangar side.

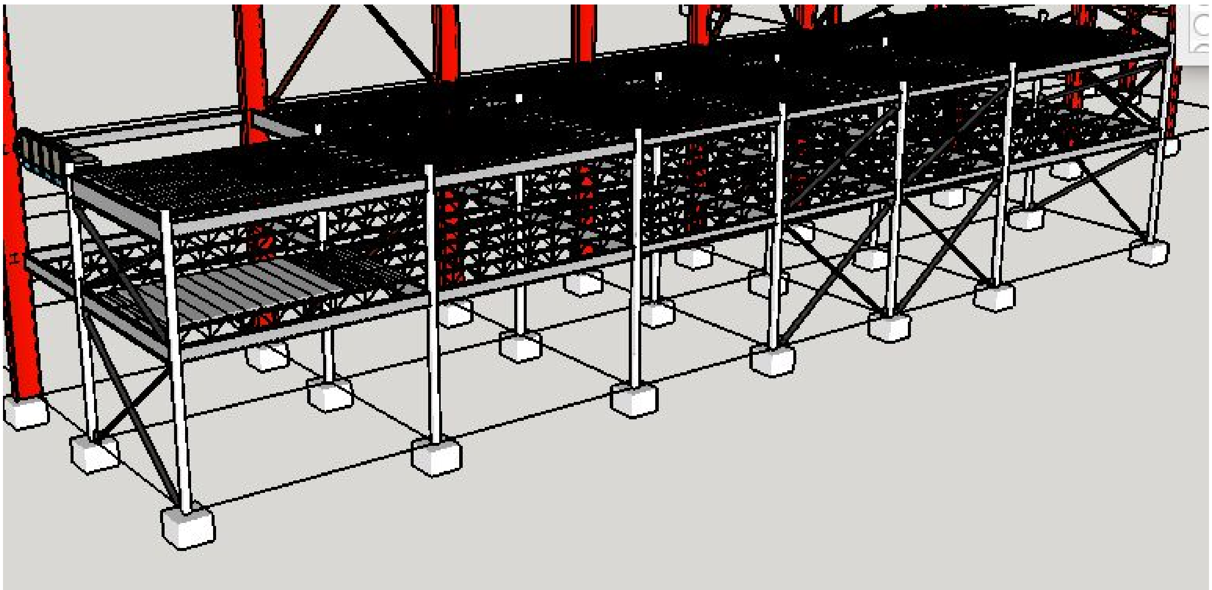


Figure 33. Office side vertical bracing

8.3 Hangar

8.3.1 Overview of Hangar

The hangar structure was 77 m in length, 54 m wide and 18.7 m high. Due to the spacing requirement, there were no columns in the middle of the hangar. Thus, a long span roof truss system was selected, specifically a box truss system, which will be explained in section 8.3.2. The hangar consisted of 4 box trusses, a lateral bracing system, a horizontal bracing system, 8 columns on each of the short side of the structure to support the box trusses, 7 columns along the edge of the office, 3 additional columns on the long side of the hangar, and wind columns on each side of hangar, expect on the side with megador, as shown in the Figure 34 and Figure 35. The sliding megadoor system was located on the opposite side of the office. Figure 36. and Figure 37. showed the spacing between the columns.

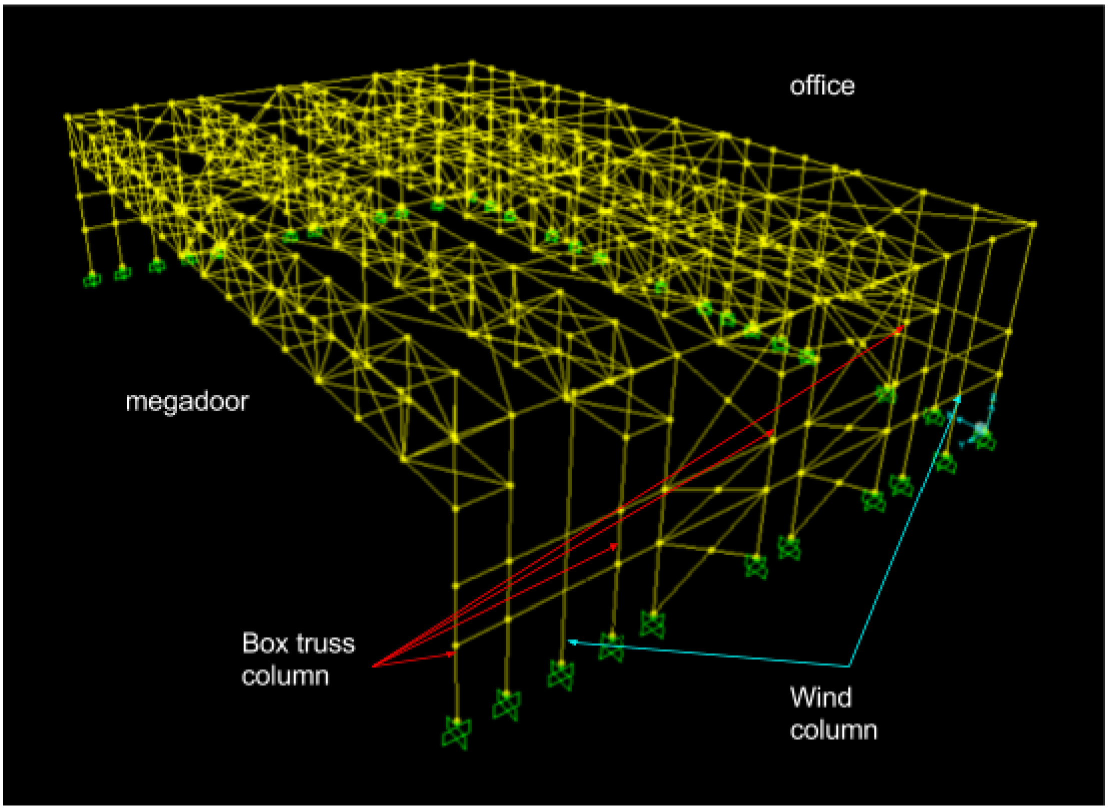


Figure 34. 3D hangar model showing megadoor side

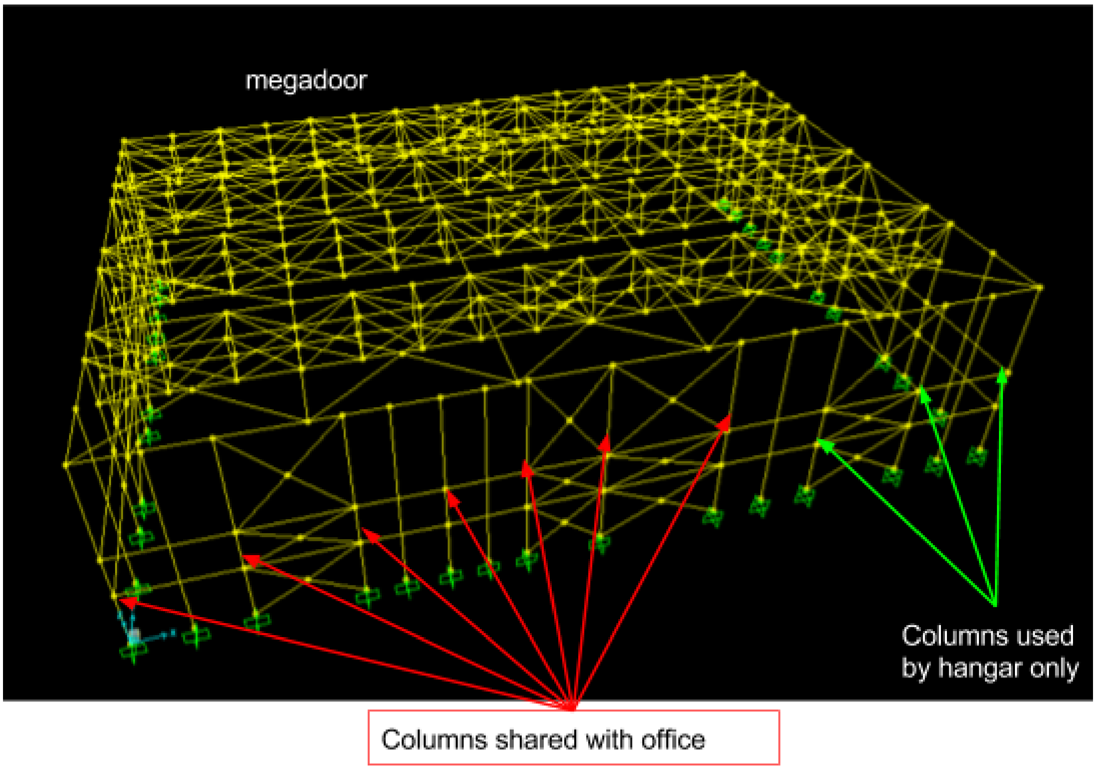


Figure 35. 3D hangar model showing the side shared with office

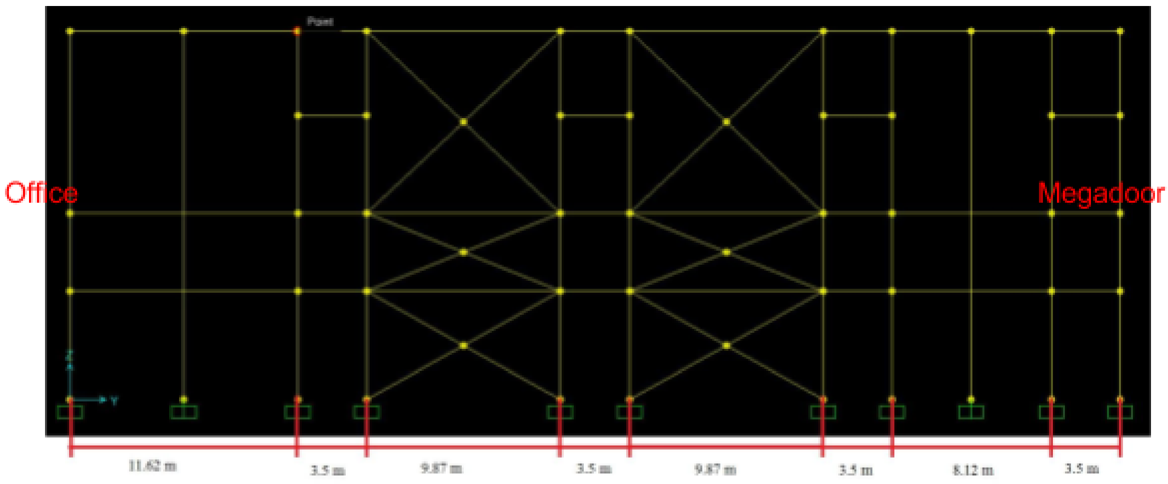


Figure 36. Column spacing on the short side of the hangar

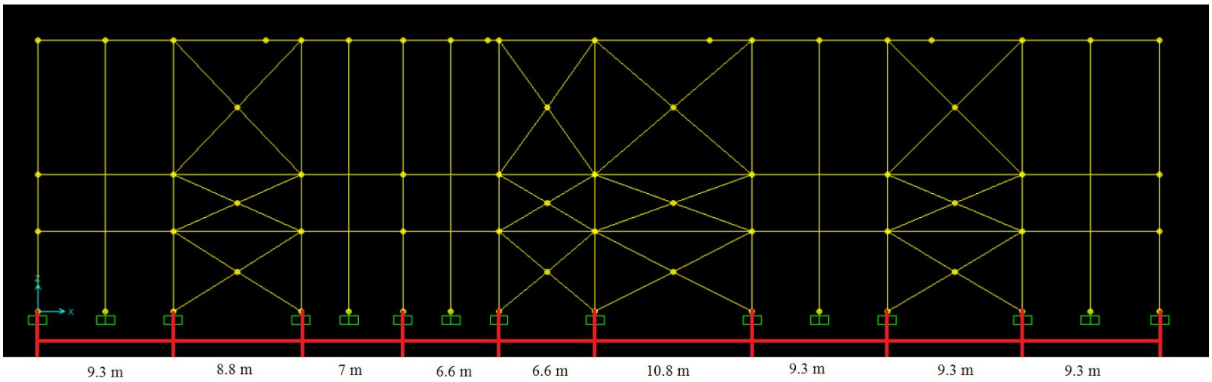


Figure 37. Column spacing on the long side of the hangar

8.3.2 Box-Truss System

Due to the open spacing requirement, the unsupported length of the truss would be 77.1 m. Long span trusses were selected to carry the load. The first criteria considered was the dimension of the truss system. Three different types of trusses were considered: planar truss, box truss, and space truss.

Although planar truss were easier to transport, its 2D shape, creating out of plane stability issue, would require more cranes during assembly when compared with box trusses. The additional crane would be used to support the lifted truss until braces or purlins between the trusses were installed. Additionally, more shoring was needed to put up the planar truss. The equipment cost and labour cost to put up planar truss would be significantly higher as well. Similarly, space truss would require more cranes to put up the structure and it took more time to assemble it on site. Considering the labour cost in Kuujjuaq was high, and construction time was limited, box truss system was selected. A Box truss could be pre-assembled in shop and shipped to site, and then connected to the columns on site. Furthermore, due to its 3D nature, the box truss was more stable, and no additional support was required during construction. Shoring can also be reused once the box truss was connected to the columns, which could reduce the overall shoring required. Another consideration was that the box truss would provide stability for the megadoor and enough width to inhibit the connection between the megadoor and the truss.

Next truss configurations were considered. Warren, the modified Warren, and the Pratt truss were considered. The different configuration were shown in Figure 38. The modified Warren truss was selected because it was the most economical configuration. For identical loading conditions, the Modified Warren uses only 80% of the members used for a Warren or Pratt configuration (Boyle, 2014).

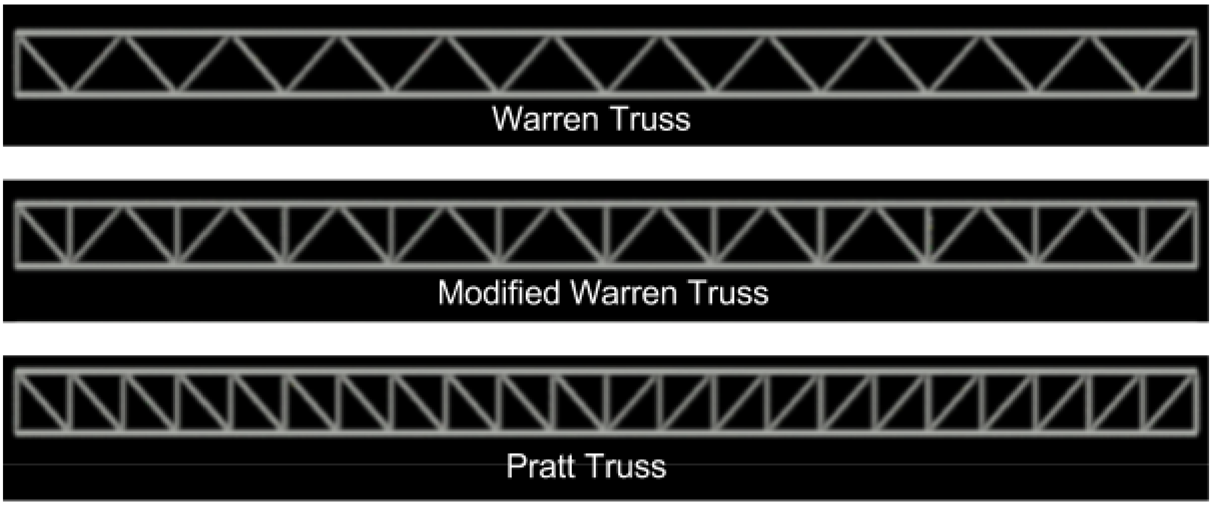


Figure 38. Warren, the modified Warren, and Pratt truss.

Furthermore, the span to depth ratio for the truss was selected. A design constraint was that the depth of the truss had to be within a L/12 to L/18 limit (University of Ljubljana, n.d.; Ioannides, S.A., Ruddy, J.L., 2000). The depth of the truss could range from 4.28 m to 6.42 meter. Considering transportation restrictions, the Oversize/Overweight Permit Manual of Quebec was consulted. Based on shipping concerns in Quebec, class 2 permit was applicable. The dimensions of the truss would have to fit within a total width of 4.30 meter, height of 4.30 meter, and a length less than 30 m (Societe de l'assurance Automobile, 2015). The depth of truss was then chosen using the limit L/18, which was 4.28 m, satisfying the shipping restrictions. The width of the box truss was chosen as 3.50 meter. Each box truss was also divided into 5 spans for the ease of transport. The dimension of the box truss are shown in Figure 39.

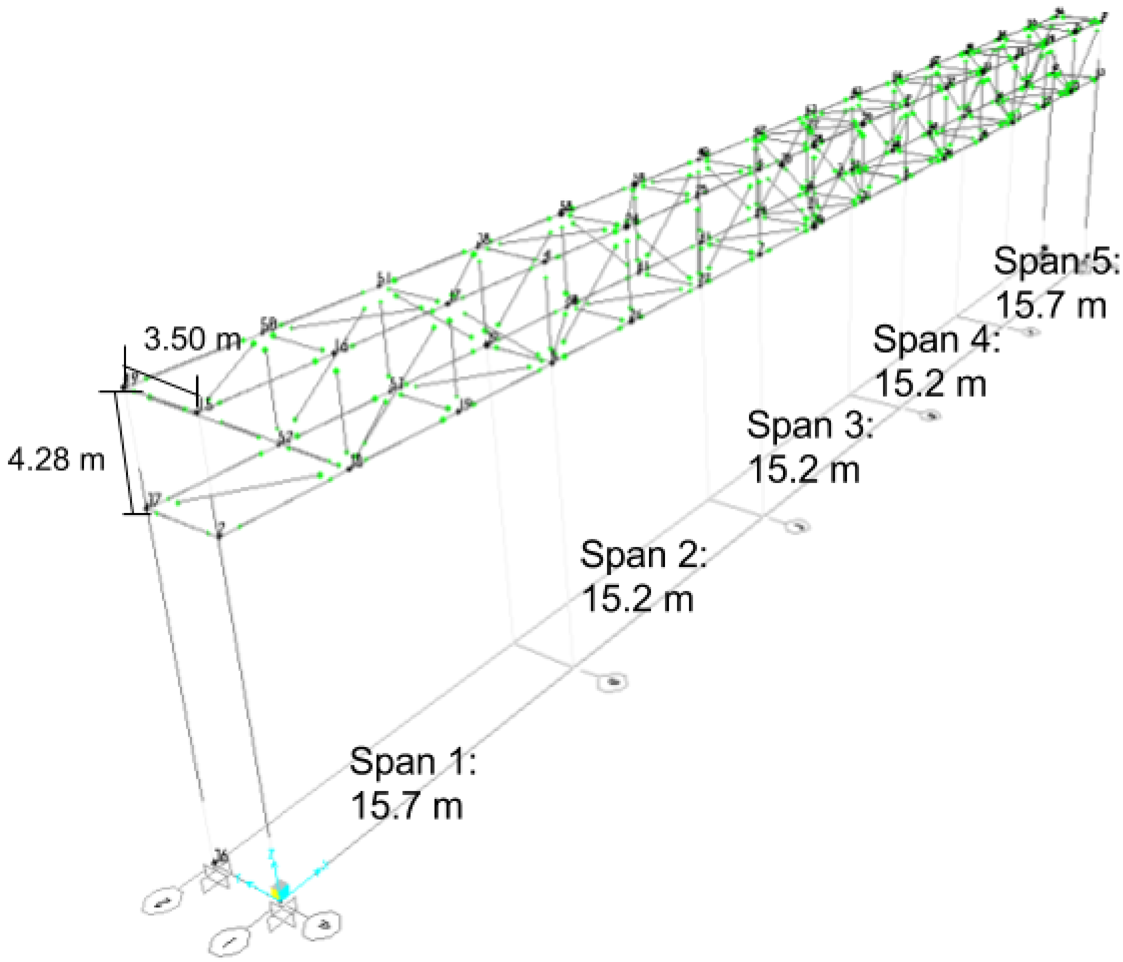


Figure 39. 3D box-truss dimension

8.3.3 Deck Selection

Similar to the office deck selection, an adequate deck was selected for the hangar roof by comparing the accumulated loads on the roof with the Canam Steel Deck Catalogue. The deck was designed to be sufficient under the maximum moment and deflection.

For the hangar roof, a P-2436 Type 22 steel deck was selected as it satisfied the necessary requirements. Figure 40. summarizes the physical properties of the deck.

Physical Properties

Type	Nominal Thickness	Design Thickness	Overall Depth	Weight	Section Modulus M+	Section Modulus M−	Moment of Inertia for Deflexion
	mm (in.)	mm (in.)	mm (in.)	kg/m² (lb/ft²)	mm³ (in³)	mm³ (in³)	mm⁴ (in⁴)
22	0.76 (0.030)	0.762 (0.0300)	76.2 (3.00)	11.85 (2.43)	24 134 (0.4489)	25 690 (0.4778)	1 006 306 (0.7369)
20	0.91 (0.036)	0.909 (0.0358)	76.4 (3.01)	14.04 (2.88)	29 407 (0.5470)	31 169 (0.5797)	1 262 487 (0.9245)
18	1.21 (0.048)	1.217 (0.0479)	76.7 (3.02)	18.33 (3.75)	40 633 (0.7558)	41 655 (0.7748)	1 819 220 (1.3322)
16	1.52 (0.060)	1.511 (0.0595)	77.0 (3.03)	22.71 (4.65)	51 473 (0.9574)	51 681 (0.9613)	2 294 846 (1.6805)

Figure 40. Physical properties of hangar roof deck

The Canam deck catalogue was used to determine the minimum spacing based on the maximum factored loads controlled by the bending capacity. The maximum factored load was calculated to be 5.68 kN. Considering the P-2436 Deck Type 22 and double span, based on the maximum factored loads, it was determined that a minimum of 2700 mm spacing was required. Figure 41. shows how the P-2436 Deck satisfies the spacing selected.

P-2436 & P-2404

Factored and Service Loads Table (kPa) **Metric**

Type	Nominal Thickness (mm)		SPAN (mm) 2 100	2 250	2 400	2 550	2 700	2 850	3 000	3 150	3 300	3 450	3 600	3 750	3 900
			SINGLE SPAN												
22	0.76	F	8.94	7.80	6.87	6.09	5.44	4.88	4.41	4.00					
		D	7.06	5.74	4.73	3.94	3.32	2.82	2.42	2.09					
20	0.91	F	10.93	9.54	8.39	7.44	6.64	5.96	5.38	4.89	4.45	4.08			
		D	8.86	7.20	5.93	4.95	4.17	3.54	3.04	2.62	2.28	2.00			
18	1.21	F	15.13	13.19	11.61	10.29	9.18	8.25	7.45	6.76	6.16	5.64	5.18	4.77	4.41
		D	12.76	10.37	8.55	7.13	6.00	5.11	4.38	3.78	3.29	2.88	2.53	2.24	1.99
16	1.52	F	19.16	16.71	14.70	13.03	11.63	10.44	9.43	8.56	7.80	7.14	6.56	6.04	5.59
		D	16.10	13.09	10.78	8.99	7.57	6.44	5.52	4.77	4.15	3.63	3.20	2.83	2.51
			DOUBLE SPAN												
22	0.76	F	9.42	8.23	7.25	6.44	5.75	[illegible]	[illegible]	4.24	3.87	3.54			
		D	17.00	13.82	11.39	9.50	8.00	6.80	5.83	5.04	4.38	3.83			
20	0.91	F	11.51	10.04	8.84	7.85	[illegible]	6.30	5.69	5.16	4.71	4.31	3.96	3.65	
		D	21.33	17.34	14.29	11.91	10.04	8.53	7.32	6.32	5.50	4.81	4.23	3.75	
18	1.21	F	15.43	13.46	11.85	10.51	9.38	8.43	7.61	6.91	6.30	5.77	5.30	4.88	4.52
		D	30.74	24.99	20.59	17.17	14.46	12.30	10.54	9.11	7.92	6.93	6.10	5.40	4.80
16	1.52	F	19.14	16.70	14.70	13.04	11.64	10.46	9.44	8.57	7.81	7.15	6.57	6.06	5.60
		D	38.78	31.53	25.98	21.66	18.24	15.51	13.30	11.49	9.99	8.74	7.70	6.81	6.05
			TRIPLE SPAN												
22	0.76	F	(11.11)	(10.18)	8.98	7.98	7.14	6.42	5.81	5.27	4.81	4.41	4.05	3.74	
		D	13.33	10.84	8.93	7.44	6.27	5.33	4.57	3.95	3.43	3.01	2.65	2.34	
20	0.91	F	14.26	12.46	10.98	9.75	8.71	7.83	7.08	6.43	5.86	5.37	4.93	4.55	4.21
		D	16.72	13.59	11.20	9.34	7.87	6.69	5.73	4.95	4.31	3.77	3.32	2.94	2.61
18	1.21	F	19.15	16.72	14.73	13.07	11.68	10.49	9.48	8.61	7.85	7.19	6.61	6.09	5.63
		D	24.09	19.59	16.14	13.46	11.34	9.64	8.26	7.14	6.21	5.43	4.78	4.23	3.76
16	1.52	F	23.76	20.75	18.28	16.22	14.49	13.02	11.76	10.68	9.74	8.92	8.20	7.56	6.99
		D	30.39	24.71	20.36	16.97	14.30	12.16	10.42	9.00	7.83	6.85	6.03	5.34	4.74

Figure 41: P-2436 Type 22 hangar steel roof deck maximum factored load check

To satisfy the span of the box truss, the spacing was decreased to 2616 mm for span 1 and span 5 of the box truss and 2540 mm for span 2, 3, and 4, making the design conservative as shown in Figure 42.

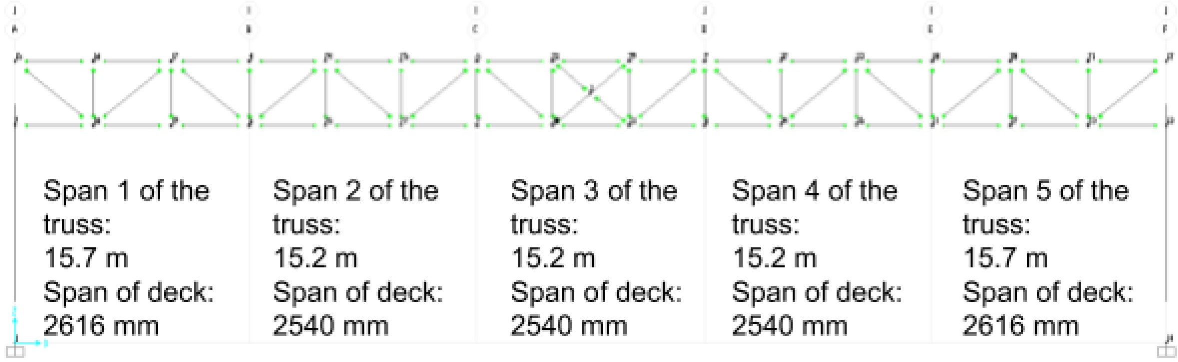


Figure 42. Span of the deck.

8.3.4 Purlin Design

The purlins were designed according to the gerber system, which meant that the bending moments dropped to zero at the end of each span. The box trusses sat below the retained spacings, between the hinged and roller support at each span. As shown in the purlin load calculations in the Appendix, there are 4 cases for the purlin design according to 4 different tributary widths. Table 11. below summarized the four different cases for the purlin spacing and loads. The factored distributed loads were based on the maximum load combination of 1.25D+1.5S+1.0L from NBCC 2015. Since the span 2 had shear connections on both sides, it exerted a concentrated load on span 1 and 3.

Table 11: Purlin spacing and loads

Case	Tributary Width(m)	Factored Distributed load (KN/m)	Factored Point load (KN)
1	2.580	14.640	44.070
2	3.920	22.280	67.068
3	2.540	14.427	43.429
4	2.615	14.853	44.711

The spacing and loading conditions were imputed and analyzed in SAP2000 to find the maximum shear and moment on each member. The below analysis was a demonstration of the processes for case 1.

Span1 loading

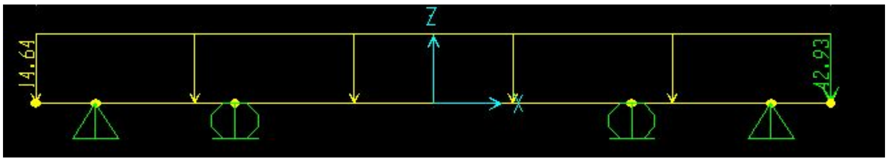


Figure 43. Span 1 loading

Span1 deformed shape

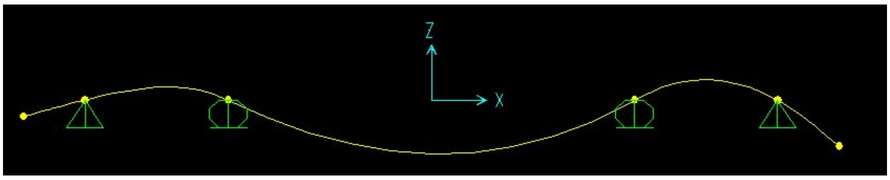


Figure 44. Span 1 deformed shape

Span1 shear diagram

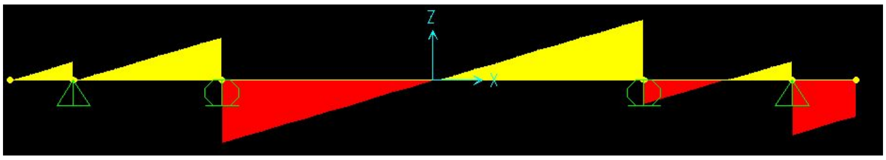


Figure 45. Span 1 shear diagram

V_{max}=-74.022KN at 5m

Span1 moment diagram

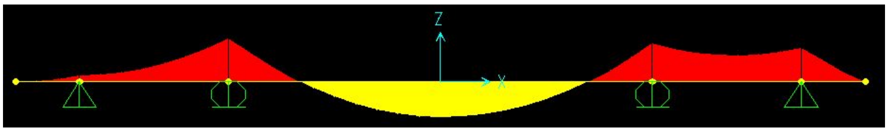


Figure 46. Span 1 moment diagram

M_{max}=-103.33KN-m at 5m

Span 2 loading

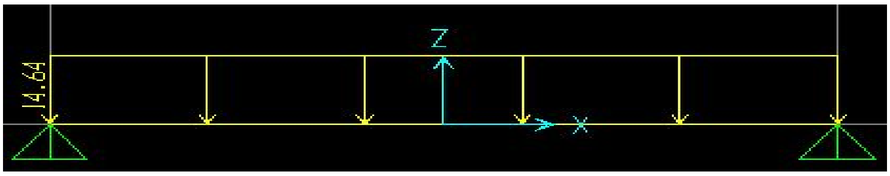


Figure 47. Span 2 loading

Span 2 deformed shape

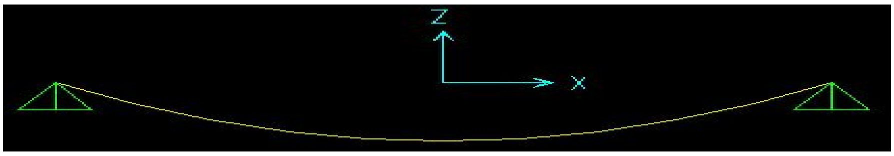


Figure 48. Span 2 deformed shape

Span 2 shear diagram

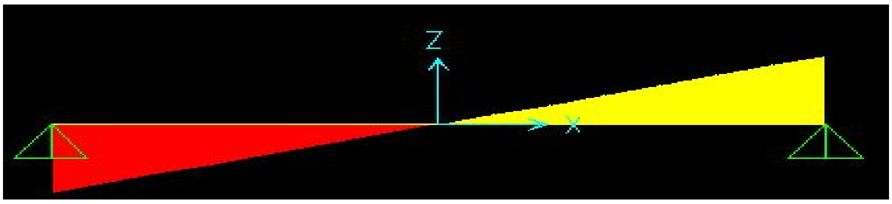


Figure 49. Span 2 shear diagram

V_{max}=-50.801KN at 0m

Span 2 moment diagram

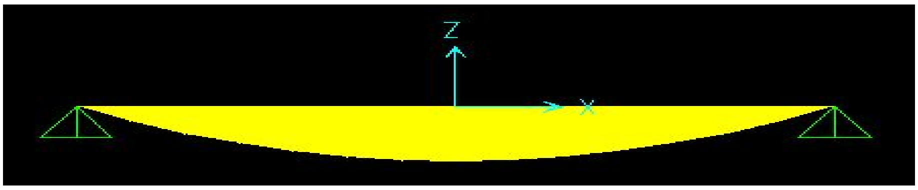


Figure 50. Span 2 moment diagram

M_{max}=88.1394KN-m at 3.47m

Span 3 loading:

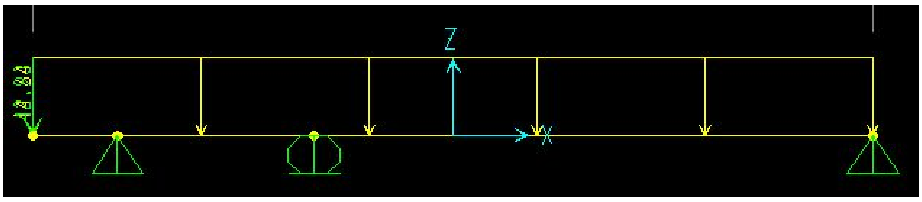


Figure 51. Span 3 loading

Span 3 deformed shape:

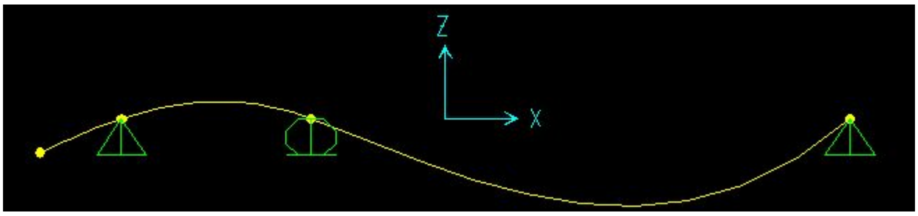


Figure 52. Span 3 deformed shape

Span 3 shear diagram:

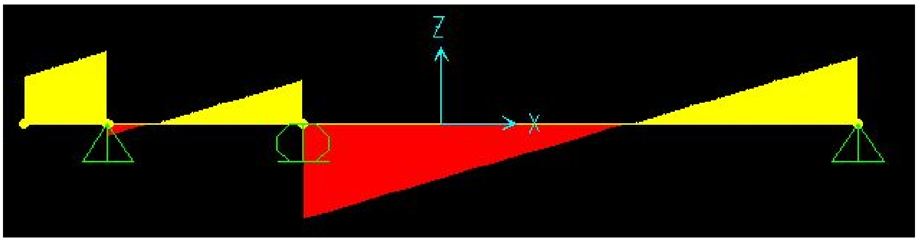


Figure 53. Span 3 shear diagram

V_{max}=-85.689KN at 5m

Span 3 moment diagram:

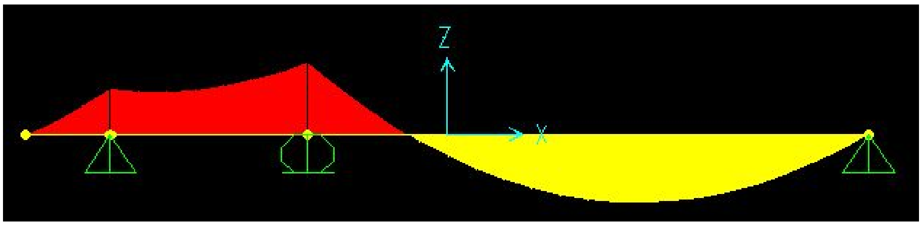


Figure 54. Span 3 moment diagram

M_{max}=-128.51KN-m at 5m

Span 4 loading:

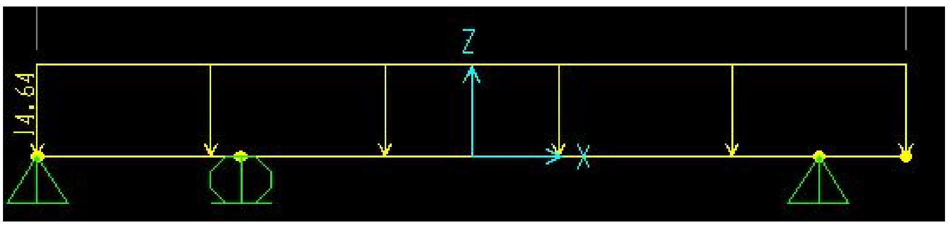


Figure 55. Span 4 loading

Span 4 deformed shape:

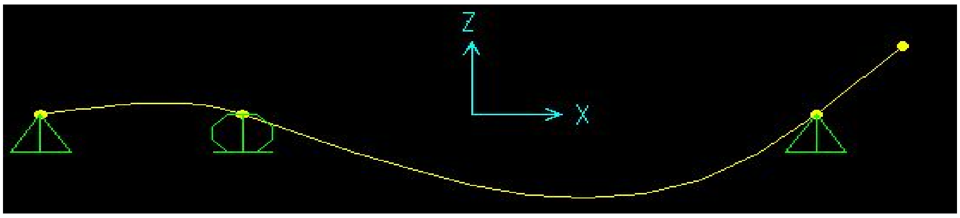


Figure 56. Span 4 deformed shape

Span 4 shear diagram:

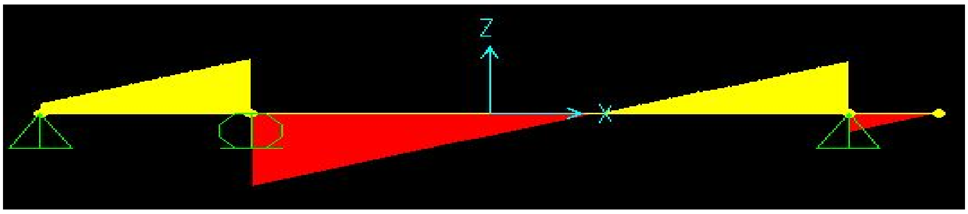


Figure 57. Span 4 shear diagram

V_{max}=-84.433KN at 3.5m

Span 4 moment diagram:

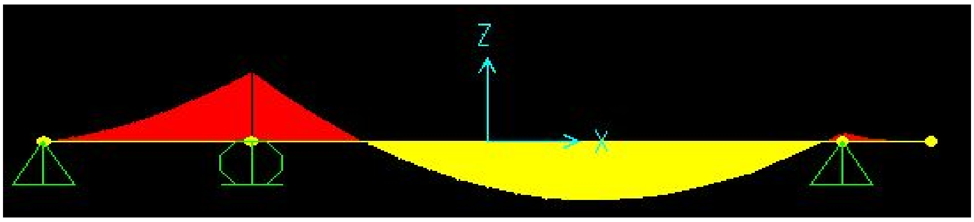


Figure 58. Span 4 moment diagram

M_{max}=-132.4958KN-m at 3.5m

The maximum shear and moment were used to design for the member sizes. Table 12 showed all of the selected purlin member sizes.

Table 12: Purlin members summary table

	Span 1			Span 2			Span 3			Span 4		
	V_{max} (kN)	M_{max} (kN*m)	Purlin Size	V_{max} (kN)	M_{max} (kN*m)	Purlin Size	V_{max} (kN)	M_{max} (kN*m)	Purlin Size	V_{max} (kN)	M_{max} (kN*m)	Purlin Size
1	75	104	W360*122	51	89	W200*59	86	129	W310*129	85	133	W310*129
2	113	158	W360*122	78	135	W200*59	131	196	W310*129	129	202	W310*129
3	73	102	W360*122	51	87	W200*59	85	127	W310*129	84	131	W310*129
4	76	105	W360*122	52	90	W200*59	87	131	W310*129	86	135	W310*129

8.3.5 Box-Truss Member

Vertical member of the box truss were designed based on the combination of dead load, including the self weight of the purlin, snow load, live load, and wind load.

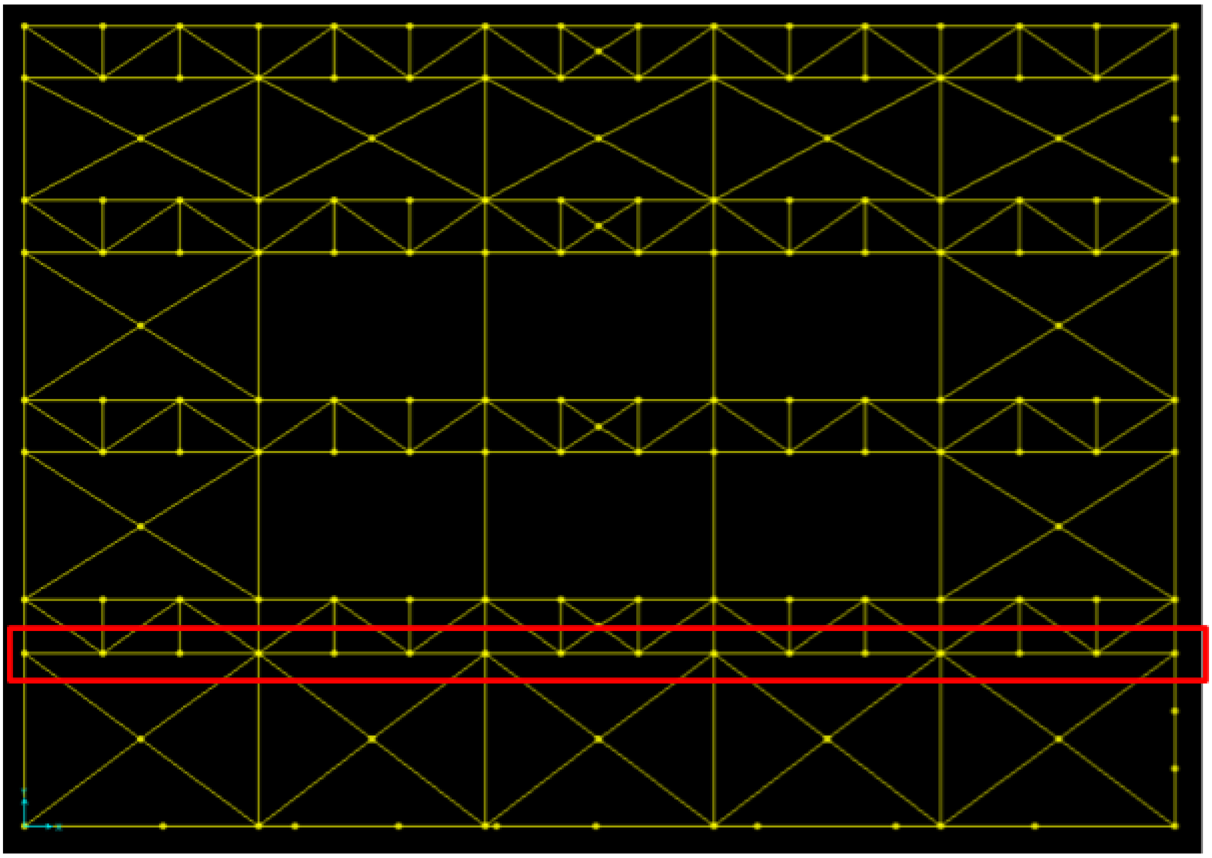


Figure 59. Critical case for truss vertical member

Figure 59. shows the critical case for the vertical member with the maximum tributary width of 5.9 meter. Due to the symmetrical shape of the box truss, the member selection for span 1 and span 5 were the same. The member selection for span 2 and span 4 were the same, as shown in Figure 60. Furthermore, all the top chords, bottom chords, diagonal webs, and vertical webs were grouped separately based on their location in the truss within each span and designed accordingly. The uniform selection within each span allowed for easier fabrication and connection on site.

In terms of the shape of the truss member, the top chords and bottom chords were W-shape due to their large axial force and bending moment carrying capacity. The web members were double angles with the minimum selection of 2L76x76 except in span 1 and 5. However, the axial force in the diagonal member in span 1 and 5 were significantly larger. Double angles were not adequate to carry the axial force, thus W-shape were chosen. Table 13 summarizes the critical member selection case.

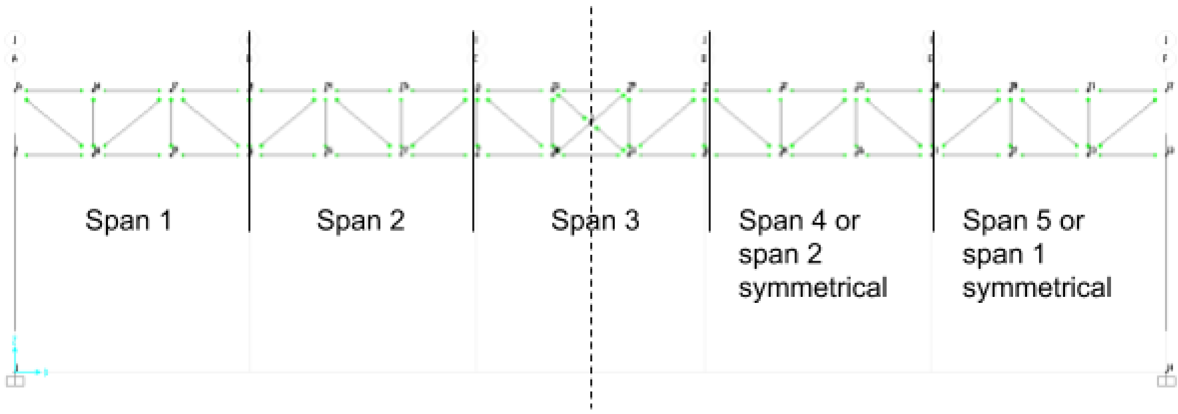


Figure 60. Symmetrical truss geometry and span assignment.

Table 13. Box truss vertical member selection

Span	Member	Member Selection
1	Top Chord	W360x147
1	Bottom Chord	W310x118
1	Diagonal Web	W310x129
1	Vertical Web	2L102x102x9.5x20
2	Top Chord	W360x216
2	Bottom Chord	W760x161
2	Diagonal Web	2L203x203x19x20
2	Vertical Web	2L102x102x9.5x20
3	Top Chord	W360x262
3	Bottom Chord	W760x185
3	Diagonal Web	2L76x76x4.8x20
3	Vertical Web	2L102x102x9.5x20

8.3.6 Vertical Bracing System

8.3.6.1 2D Considerations

When starting the analysis of the lateral system, the team was well aware of the fact that vertical bracing bents could not be placed on the Megadoor side to satisfy the requirement of having a clear entrance. Therefore, the team decided to begin a 2D analysis to understand the behaviour of the structure under that imposed lateral loads. The layout of the structure was drawn in plan and arbitrary vertical bracing bents were placed all around the structure except on the Megadoor side as shown in Figure 61. It was assumed that the massive weight of the Megadoor is beared by the foundation and is only guided by rails in the box truss. Therefore the weight of the Megadoor is not carried by the frame. For simplicity, the center of mass (CM) was placed at the center of the structure. Next, bracing bents were placed on both of the short sides and 4 bracing bents were placed on the long side opposite to the Megadoor. The center of rigidity was assumed to be centered on this side.

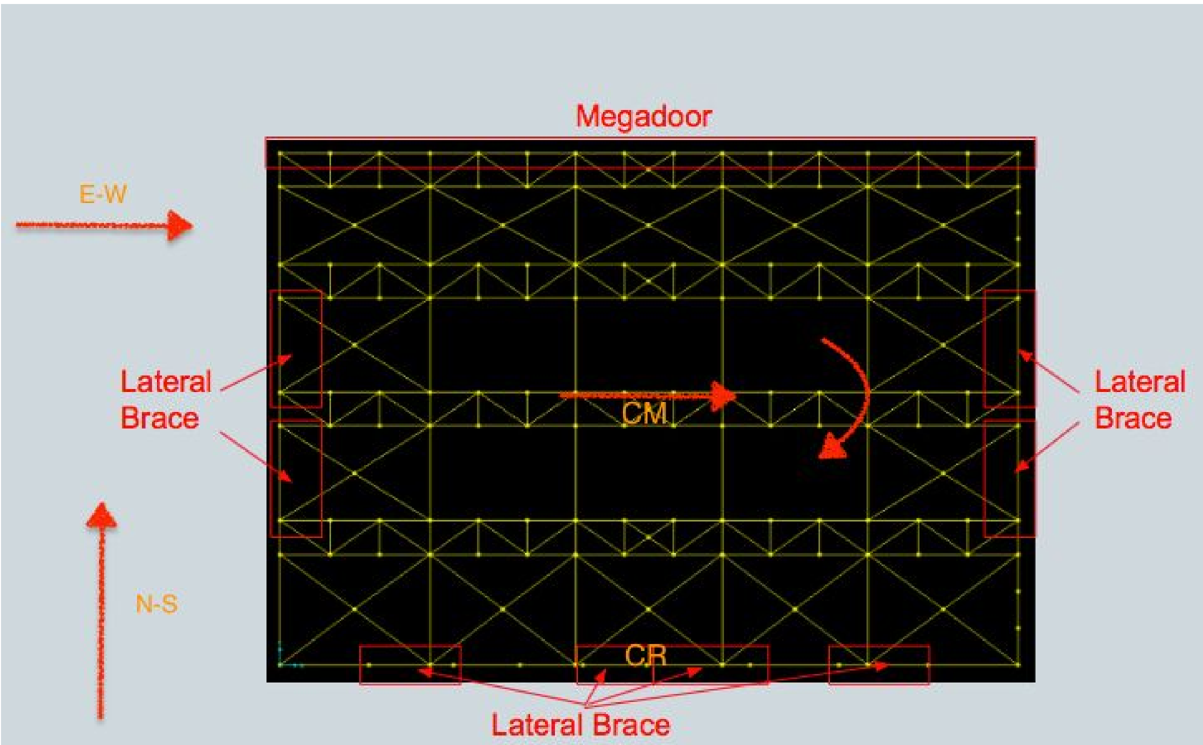


Figure 61. 2D analysis overview of lateral system

If the wind were blowing in the N-S direction, bracing bents on the short sides of the hangar would carry the lateral wind load. However, if the wind were blowing in the E-W direction while having vertical braces only on one side of the structure, the wind would create huge torsion on the members in the frame, a behaviour that could be visualized as a moment in 2-D, as the wind base shear is applied at the CM for simplification multiplied by the moment arm about the CR. So this moment had to be carried by the available bracing bents in order for the frame to be stable. Therefore, the analysis started and it was an iterative process. Each bracing bent was analyzed in SAP2000 by applying ¼ of the wind load (as an initial starting point) since we have 4 bents in each direction the wind is blowing. The bracing bent members were designed under 1.4 W only and then a unit 1000 kN load was applied and the deflection in the lateral direction was noted and a stiffness k (kN/mm) value was recorded. After repeating the steps for all bents the results were summarized in an excel spreadsheet. The bracing bent would be virtually seen as a resisting force that has a moment arm about the CR. Thus, when the wind is blowing in the E-W, there are two components a wind force that need to be resisted and a moment of the wind base shear about the CR. The wind force would be carried by the four braces on the long side parallel to this direction. The moment would be carried by the braces on the short side perpendicular to the direction of the wind as the brace bent would have a moment arm about the CR. The moment would be distributed based on how far the bracing bent were from the CR and the stiffness of the bracing bent. Moreover, the bracing bents on the short direction are carrying a force when the wind is blowing in the opposite direction, E-W, this force is multiplied by 0.75 and added to 0.75 of what the brace is carrying when the wind is blowing in the N-S direction, that is because the wind blowing at full capacity in both directions is unusual. Finally, the brace was designed to carry this lateral force and the analysis was taken to a 3D model using SAP2000 and the members designed were compared to the members selected by the software.

8.3.7 Horizontal Bracing System

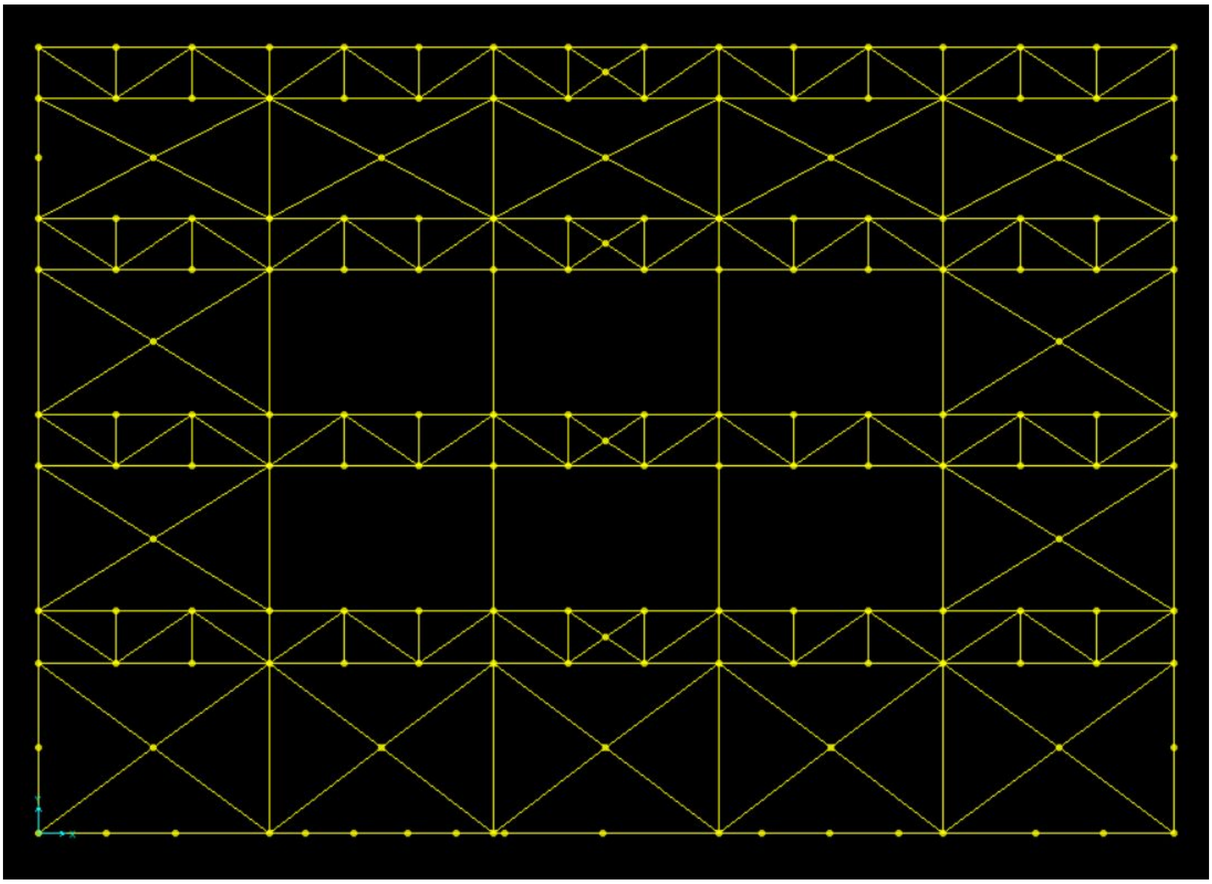


Figure 62. Layout of horizontal bracings on the roof of the hangar

After designing the vertical bracing system, the team had to ensure the lateral forces were transferring from the Megadoor side, with no vertical bracing bents backwards towards the braced side of the hangar. This would ensure stability of the frame. Using the 3D model of the hangar frame in SAP2000, a box configuration was chosen to be implemented on the roof of the hangar.

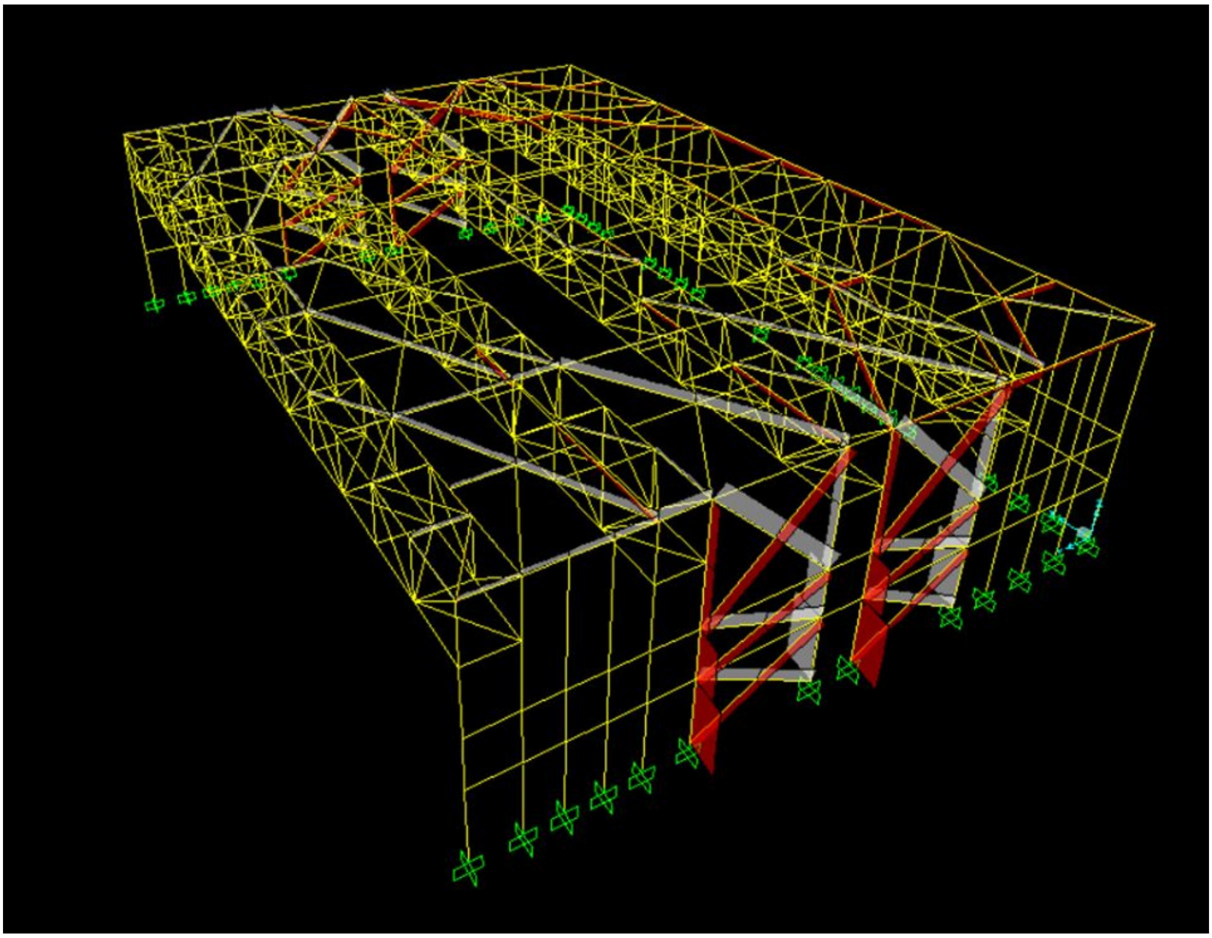


Figure 63. Lateral force transfer through horizontal braces to vertical bents

Since this was a 3D analysis and not 2D, the forces would transfer through the horizontal bracing members towards the vertical bracing bents and into foundations, as shown in Figure 63. The model was then analyzed under the different gravity load combinations and most importantly wind cases in the N-S, E-W and 75% of the N-S and 75% E-W directions simultaneously. The most critical members with the highest axial loads were selected to be verified. Some members were in tension and others were in compression, as the direction of the wind could change and each individual member would be carrying an opposite force. Overall, the members were designed based on the compression resistance, as the compression resistance is depending on the unbraced length. HSS members were chosen for the horizontal bracing due to the high compressions noted. A 203x203x16 HSS was designed for the brace with the highest compression. It is to be noted that an HSS would be placed between the purlin and the two HSS braces, crossing one another at midspan in order to stabilise the out of plane buckling and therefore half the unbraced length could be used in the calculations.

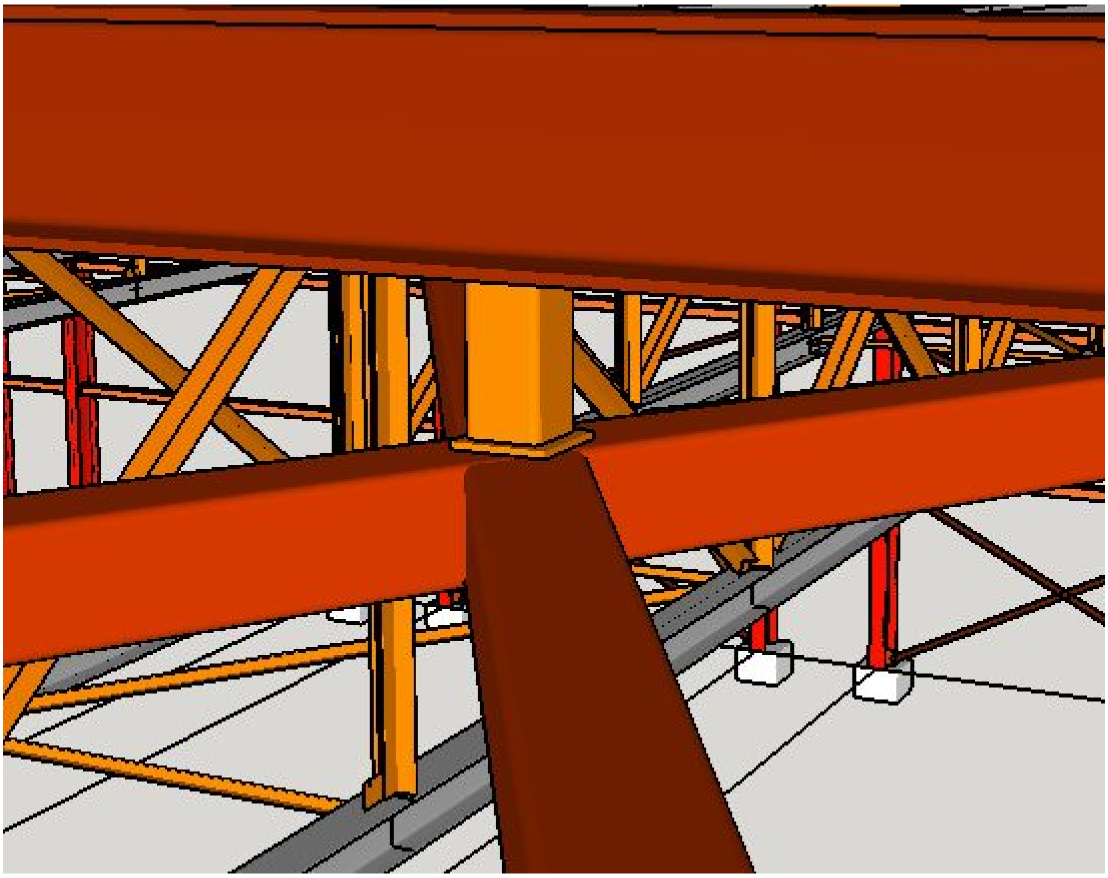


Figure 64. HSS member at midspan to control out of plane buckling

8.3.8 Column Design

With the purlins, vertical box truss members, vertical bracing members, and horizontal bracing members selected, the columns were sized based on the combined axial compression and biaxial bending, due to dead, live, snow, and wind load. The difference in member sizes were due to different tributary width. When designing for the shared columns between the office and the hangar, forces from the office were added into SAP2000 model as joint load. For all the structural columns, load combination 1.25D + 1.5S +1.0 L governed in design.

8.3.8.1 Shim Consideration/Stage Construction

The axial force and bending moment were provided by SAP2000 analysis. The initial selection from the 3D model is W1000x412, as shown in Figure X, for the axial force and bending moment.

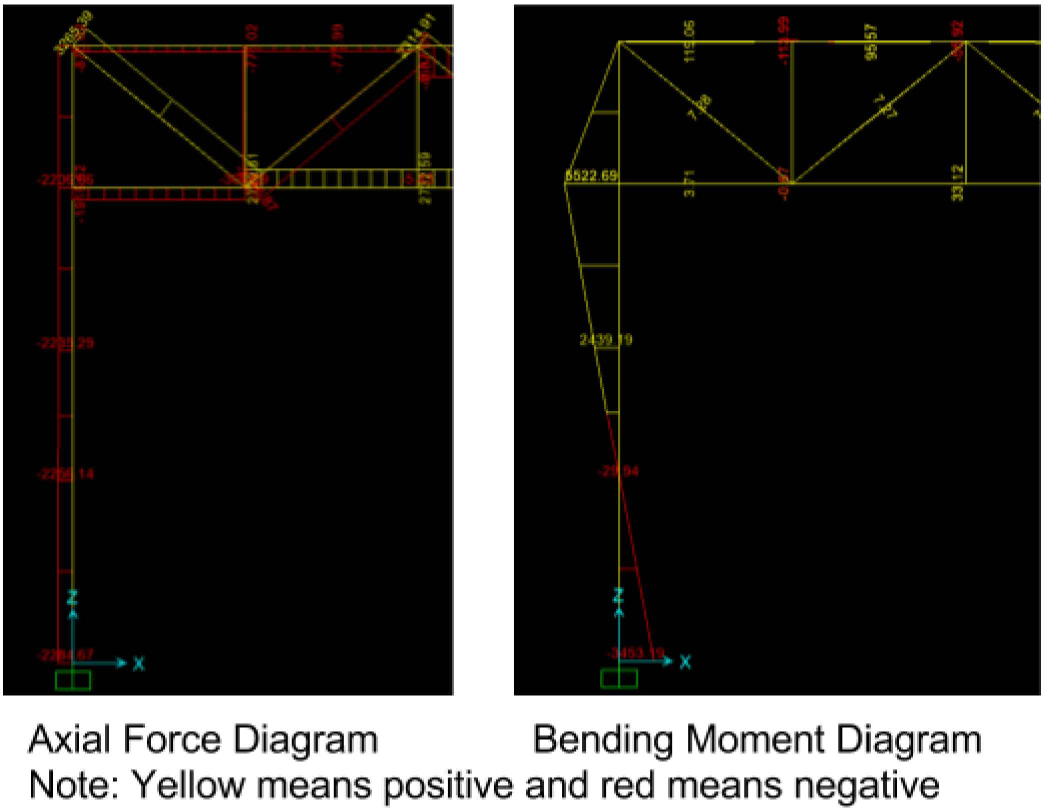
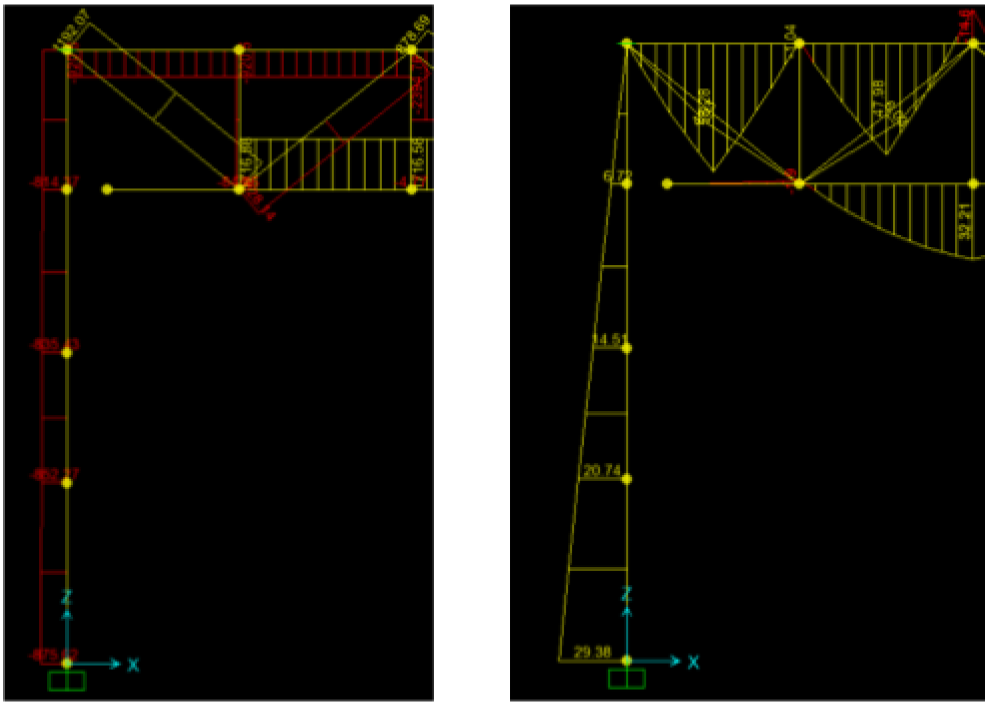


Figure 65: Axial force and bending moment diagram without shim consideration

The large member selection was due to the high moment transfer from the truss bottom chord to the column. To reduce the high moment transfer, shim was considered in the column design. The analysis for columns was divided into three stages.

The first stage was to disconnect the bottom chords in span 1 and span 5 from the columns. The structure was then run under factored dead loads in SAP2000. A small axial force F1, at the base of the column, and a small moment M1, at the node where bottom chord, used to connect to the column were generated as shown in Figure 65.



Axial Force Diagram Bending Moment Diagram
Note: Yellow means positive and red means negative

Figure 66: Axial force and bending moment diagram with shim plate consideration under factored dead load

The second step was to reconnect the bottom chords in span 1 and span 5 to the columns. The structure was then run under the combination of factored snow load and live load. Another axial force F2 at the base of the column and bending moment M2 at the connection between the bottom chord and column were generated, as shown in Figure 66.

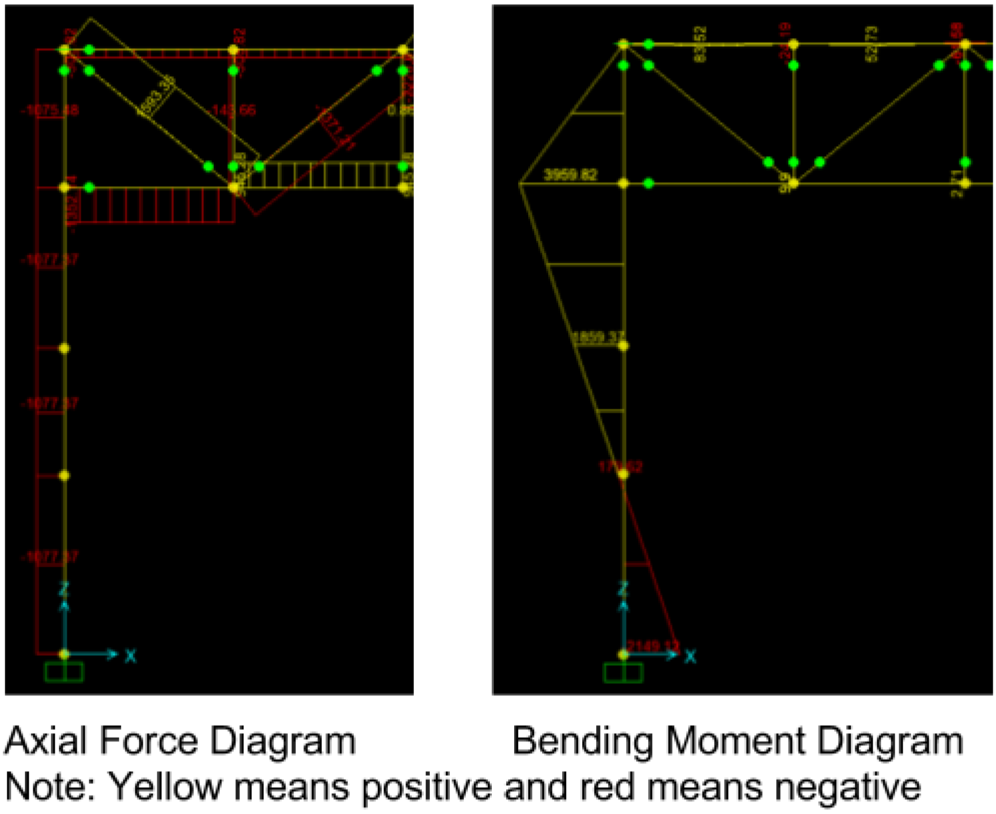


Figure 67: Axial force and bending moment diagram with shim plate consideration under factored snow and live.

The third step was to add up the force to generate F_{final} and moment M_{final}. The force and moment comparison between the no shim plate and shim plate scenario are shown in Table 14. The shim plate was able to reduce the moment transferred from the bottom chord to the column. The selection for the critical column was then determined to be W920x390.

Table 14. Effects of shim plate

Before Staged Construction (Shim)	After Staged Construction (Shim)
Maximum Factored Axial Force [kN]	Maximum Factored Axial Force [kN]
2285 [C]	1953 [C]
Maximum Moment on Major Axis [kN*m]	Maximum Moment on Major Axis [kN*m]
5523	3989

8.3.8.2 Critical Columns Selection

Another consideration in column design was the columns shared between the office and the hangar, as shown in Figure 68. Since the structure analysis for the hangar and office were done separately, as well as the hangar was higher than the office, biaxial bending moment from wind and force transferred from the office were considered in the design of the corner column where the hangar and office structure align. Similarly, the force transferred from the office was added into the axial force in the selection of the edge column in Figure 67. The final selection of the critical columns in the hangar was shown in Table 15.

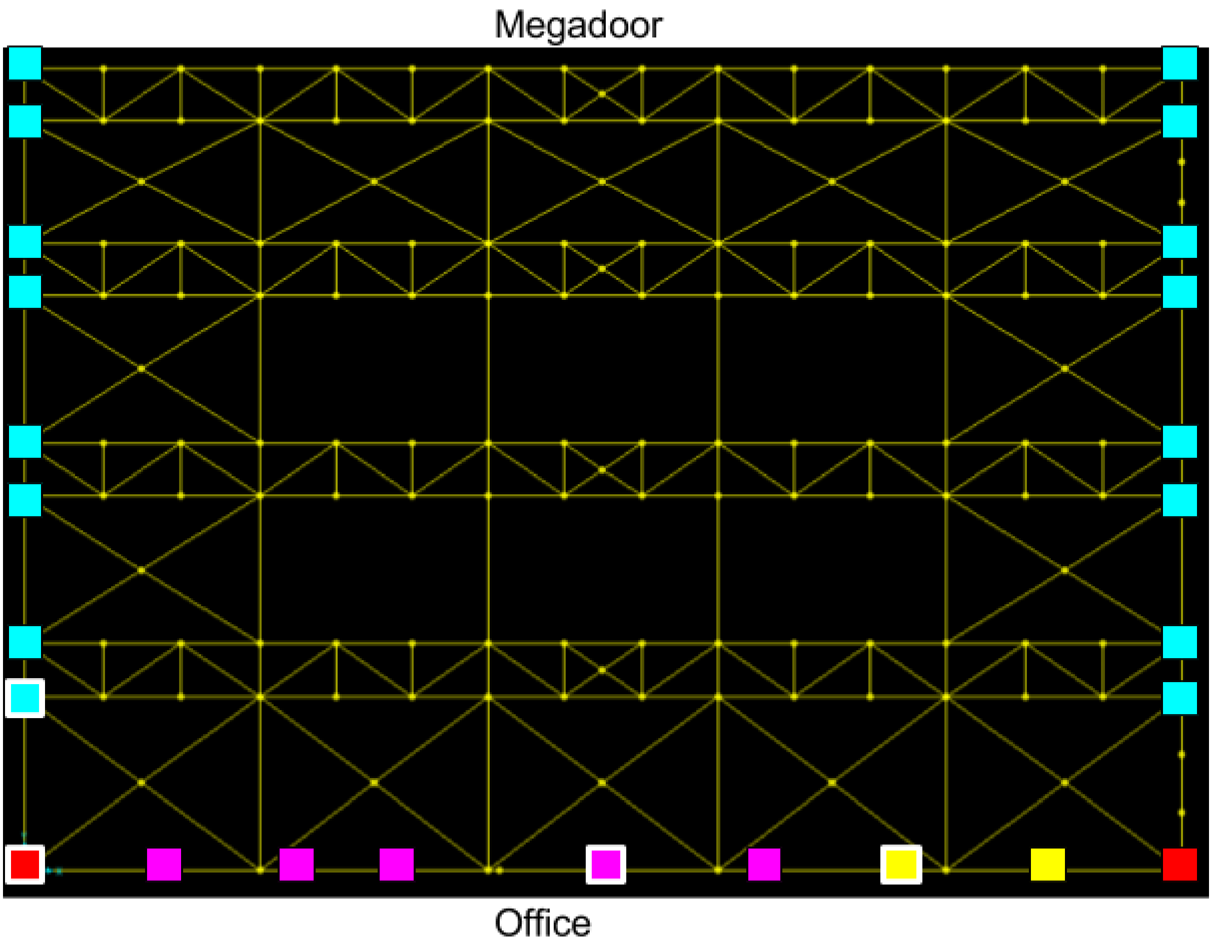


Figure 68. Columns in the hangar structure.

Note: The box with white border was the critical columns in each type.

Table 15: Column selection summary table

Type	Member Selection
	W920x390
	W360x134
	W360x179
	W360x134

8.3.8.3 Wind Column

Due to the large wind load applied to the structure, and the opening from the megadoor, wind columns were placed along the exterior building lines to assist in carrying the wind loads from the lateral bracing system. The use of wind columns could also reduce the the span for the flexural girt member. One wind column was introduced when there was no lateral bracing between columns. 2 wind columns were added on each of the short side of the hangar structure, as shown in Figure 69. And 5 wind columns were introduced on the long side of the hangar structure, as shown in Figure 70.

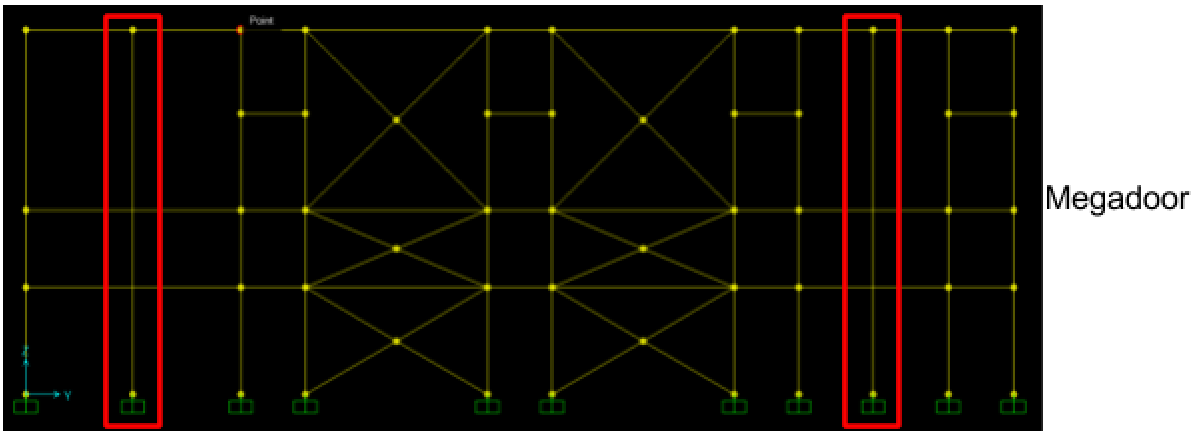


Figure 69. Wind column locations on the short side of the hangar.

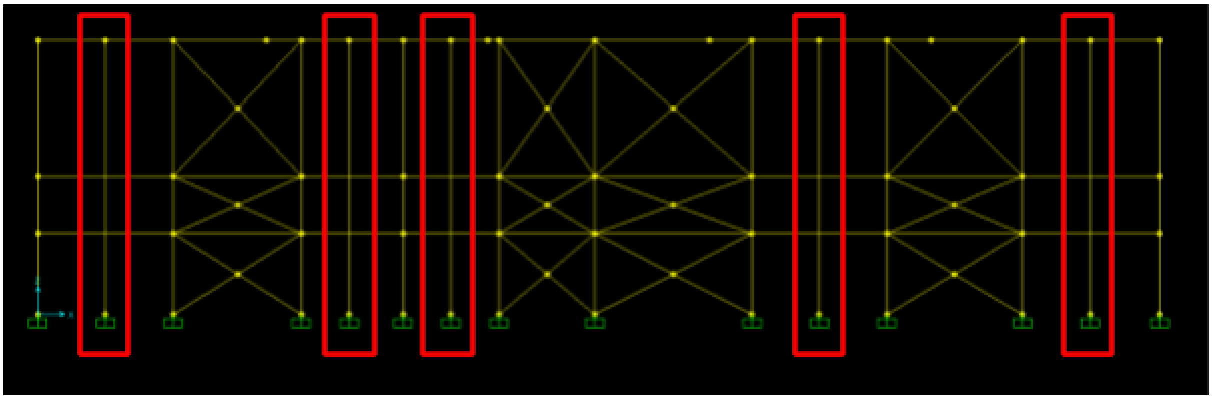


Figure 70. Wind column locations on the long side of the hangar.

Considering that wind columns withstood horizontal wind load, and small gravity loads due to the weight of siding or girts, wind columns were designed as beam column. The critical wind column member was W310x60.

8.3.9 Base Plate Design

The compression stress that can be resisted by concrete was significantly lower than the stress in the column, thus base plates were required to spread the load over an area to reduce the bearing stress. The hangar column was designed using fixed connections to the ground. The magnitude of the bending moment is large relative to the column axial load in the critical edge column. Thus, anchor rods are required to connect the base plate to the concrete foundation so that the base does not tip due to the uplift forces applied nor fail the concrete in bearing. Also, due to the high moment, stiffeners were used to transfer the high bending moment and reduce the plate thickness. In typical base plate situations, the compression force between the base plate and the concrete will usually develop shear resistance sufficient to resist the lateral forces. Shear forces would also be transferred in bearing by the use of shear key(s). With the factored axial load and bending moment at the base of the column taken from the SAP2000 model, the based plate was designed to be of size 1100 mm x 500 mm x 70 mm with eight 50.8 mm diameter anchor rod, as shown in Figure 71.

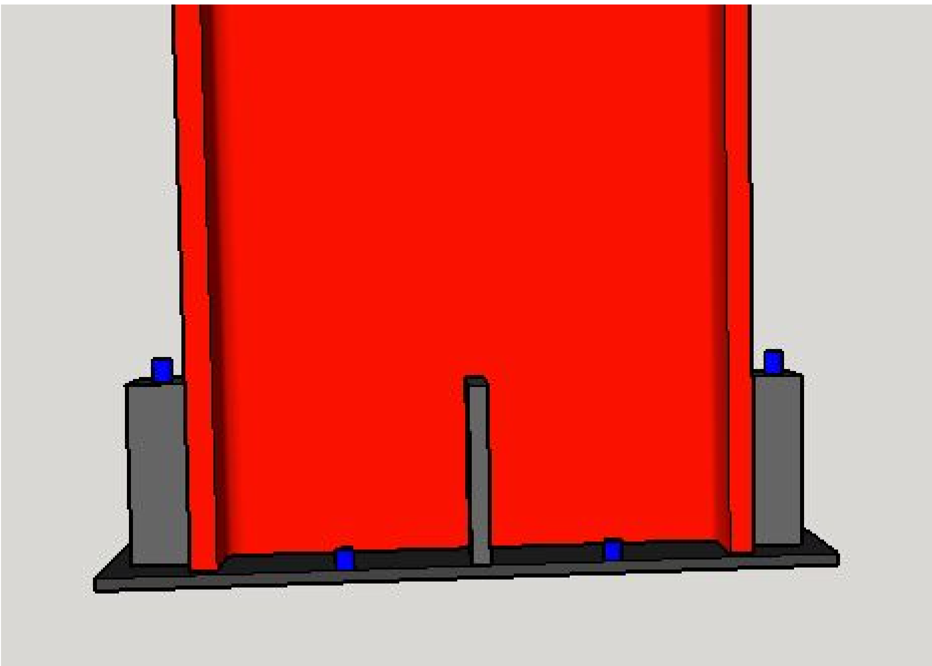


Figure 71. Base plate with stiffeners.

8.3.10 Connection Design

As explained earlier in the column design section, a staged construction would be done by shipping the box truss with shorter bottom cords to the site and then the box truss would be erected by connecting the top chord only. Then during construction, the gap between the shorter bottom chord and the column would be measured and standard size shim plates would be ready to be placed between the column flange and the bottom chord, as shown in Figure 72. below for demonstration.

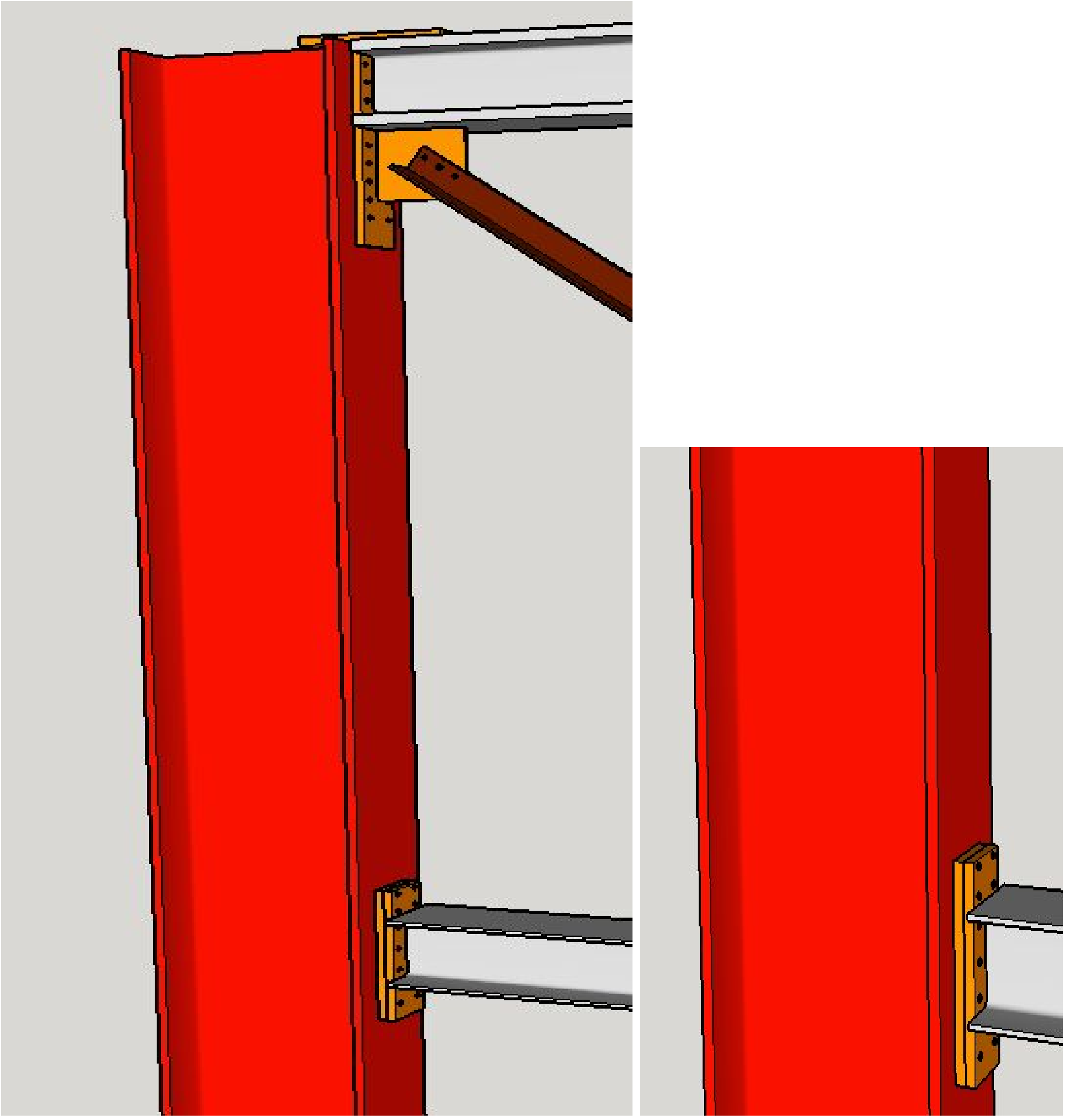


Figure 72. Bottom chord with shim consideration

The double angle braces, as shown in Figure 73., and the gusset plate to beam and column connection, as shown in Figure 74. were designed as follows. The double angle had three, ¾ inch A325 bolts configuration. The gusset plate was connected to the column using a 310 mm long angle. The angle is bolted to the column using four, ¾ inch A325 bolts and welded to the gusset plate using an all around 5 mm weld. The gusset plate is further welded to the top column using a 5 mm weld that is 300 mm long. The resistance of the gusset plate itself was checked and the 20 mm spacing of the double angle was adequate. For further reference, all the calculations are shown in the Appendix.

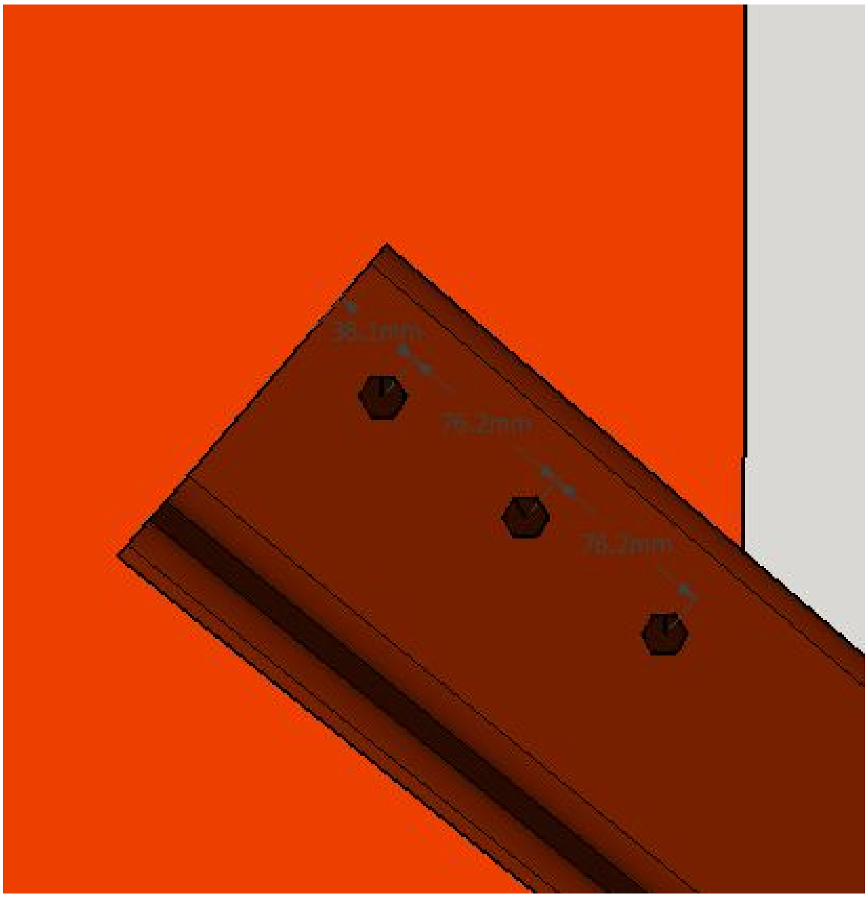


Figure 73. Angle to gusset plate connection

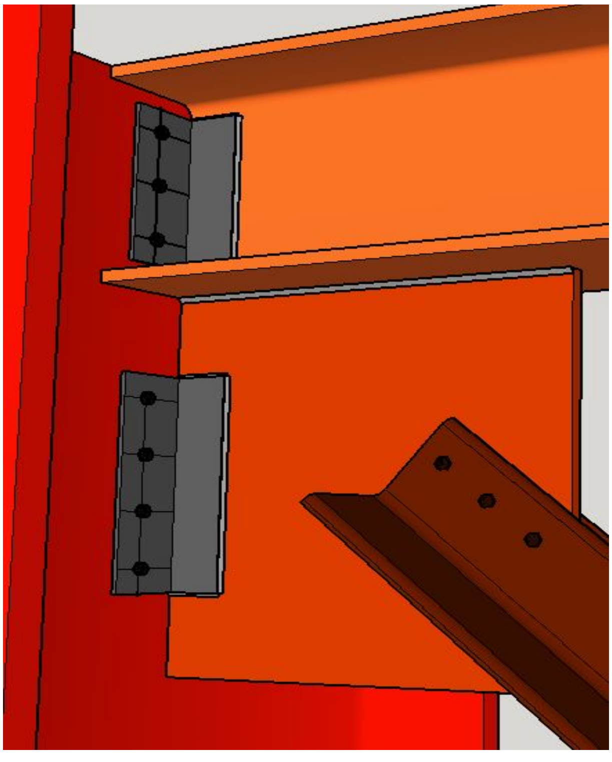


Figure 74. Gusset plate to beam and column connection

The three truss members in one of the box sections, designed as double angels, had to be connected to a gusset plate resting on the bottom chord of the truss as shown in Figure 75. below. A 20 mm fillet weld that is 400 mm long is specified for the gusset to beam interface and the angels in tension would be welded using a 15 mm fillet weld on both sides.

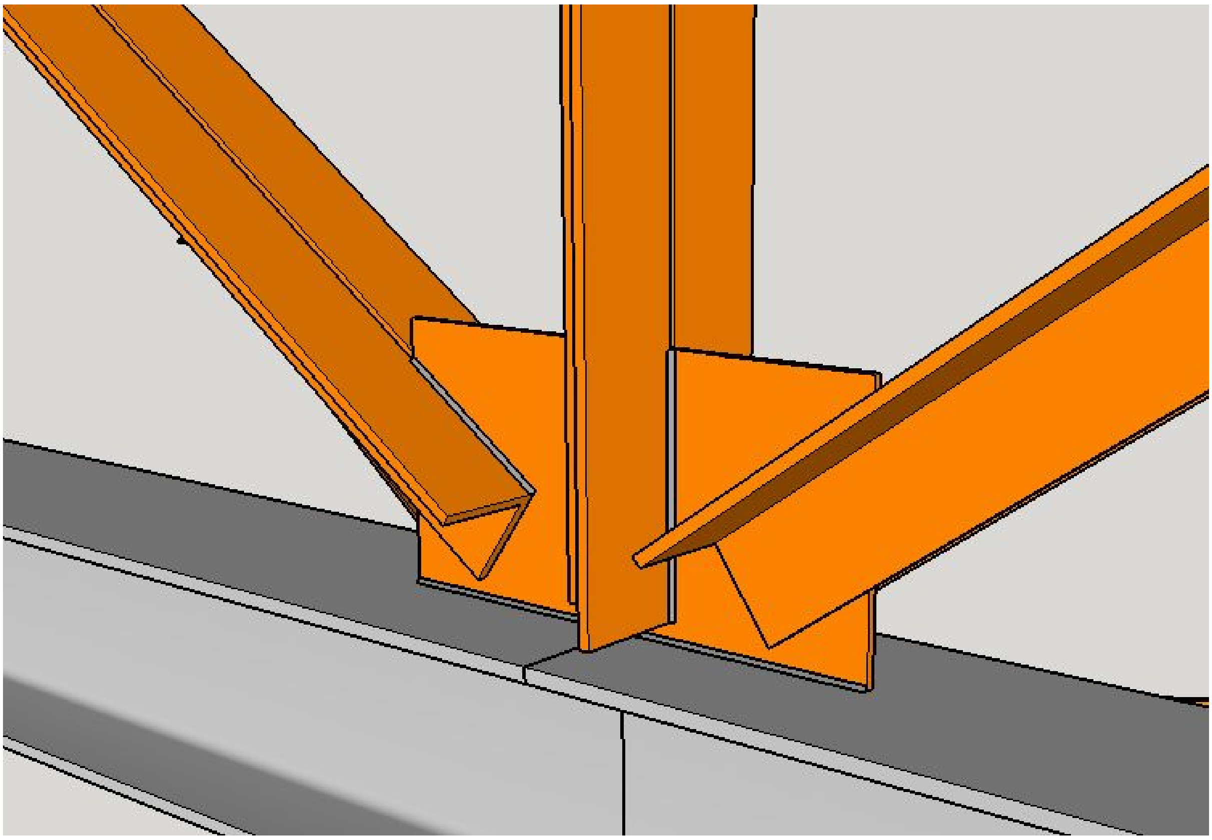


Figure 75. Diagonal and vertical truss members with gusset plate configuration

8.4 Foundation

8.4.1 Slab on Grade

8.4.1.1 Slab on grade design

Slab on grade was designed according to the 'Design of slab on ground' manual by the American Concrete Institute 360R-06. The slab was designed based on the thickness design methods which is based on an elastic behaviour between a rigid subbase and the slab. To ensure that this is the case, a uniform 12 in thick subbase of crushed gravel had to be specified to improve the modulus of subbase reaction. Then, to provide a starting point, the soil was assumed to be SC with modulus of reaction of 230 pci. The uniform subbase would further be improved to around 350 pci as shown in Figure 76.

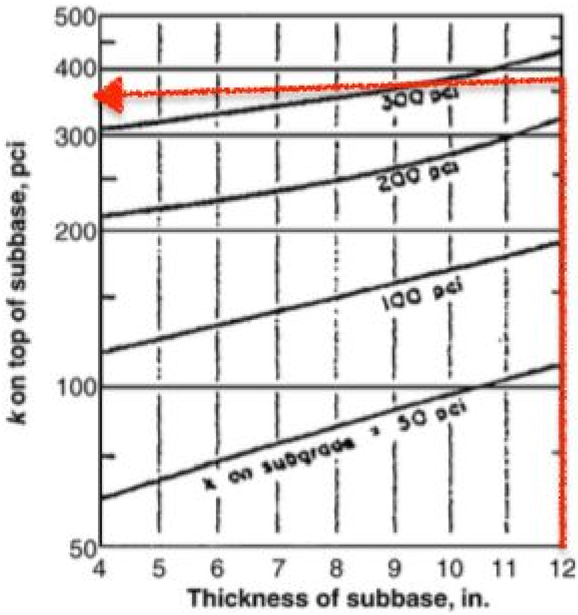
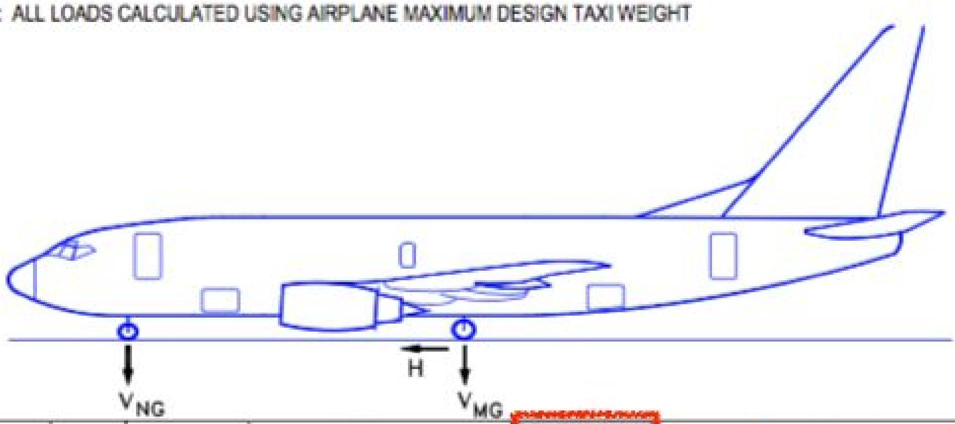


Figure 76. Modulus of subgrade reaction based on subbase thickness

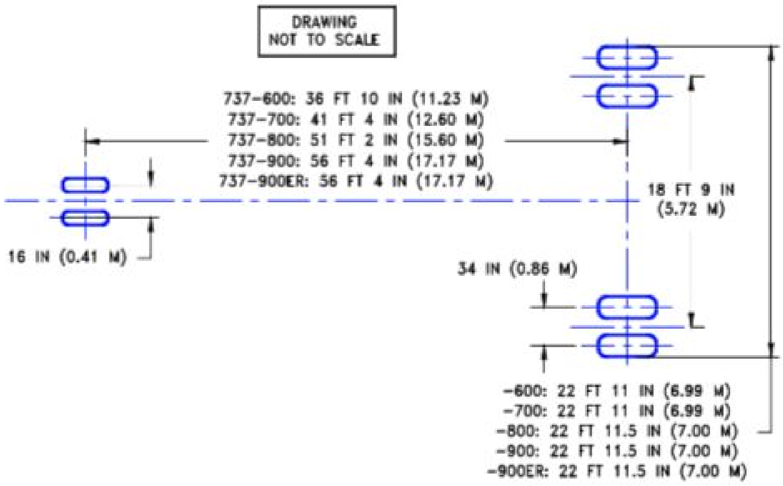
After that the loading on the slab was studied, it was the determined that aircraft wheel point loads present a more complex pattern of loading than loads from distributed live and partition components. Therefore, the aircraft wheel loads were chosen to govern the design, since the hangar was supposed to receive two different types of aircrafts a DASH Q-8 400 and a Boeing 737-800. Next, the slab was designed based on the weight of the heavier aircraft, the Boeing 737-800, as per the initial drawing plans. A procedure was followed to obtain the tire contact area and the tire pressure in order to be able to calculate the point load. To determine this, a boeing 737 manual was used in order to get the maximum load at the static center of gravity of this airplane. Since this airplane had 4 wheels per main gear, the load was divided by 4 to obtain the load per wheel and then divided by the tire pressure to get the contact area. Special attention was given while calculating the point load as it was very critical and governs the design of the slab on grade.

V_{NG} = MAXIMUM VERTICAL NOSE GEAR GROUND LOAD AT MOST FORWARD CENTER OF GRAVITY
V_{MG} = MAXIMUM VERTICAL MAIN GEAR GROUND LOAD AT MOST AFT CENTER OF GRAVITY
H = MAXIMUM HORIZONTAL GROUND LOAD FROM BRAKING
NOTE: ALL LOADS CALCULATED USING AIRPLANE MAXIMUM DESIGN TAXI WEIGHT



MODEL	UNITS	MAXIMUM DESIGN TAXI WEIGHT	V_{NG} STATIC AT MOST FWD C.G.	V_{NG} STATIC + BRAKING 10 FT/SEC^2 DECEL	V_{MG} PER STRUT AT MAX LOAD AT STATIC AFT C.G.	H PER STRUT STEADY BRAKING 10 FT/SEC^2 DECEL	H PER STRUT AT INSTANTANEOUS BRAKING (μ= 0.8)
737-600	LB	124,500	16,839	26,489	58,333	19,298	46,666
	KG	56,472	7,638	12,015	26,459	8,708	21,167
737-600	LB	144,000	19,020	30,180	66,708	22,320	53,366
	KG	65,317	8,627	13,689	30,258	10,124	24,206
737-600	LB	145,000	19,000	30,236	66,454	22,475	53,163
	KG	65,771	8,618	13,715	30,143	10,194	24,114
737-700	LB	133,500	17,558	26,711	63,000	20,692	50,400
	KG	60,554	7,963	12,116	28,576	9,386	22,861
737-700	LB	153,500	18,740	29,265	71,482	23,792	57,185
	KG	69,626	8,500	13,274	32,424	10,792	25,939
737-700	LB	155,000	16,925	27,552	71,060	24,025	56,847
	KG	70,307	7,677	12,497	32,232	10,898	25,785
737-800	LB	156,000	16,770	25,510	75,062	24,180	60,050
	KG	70,750	7,607	11,571	34,047	10,968	27,442
737-800	LB	173,000	17,059	26,752	82,143	26,815	65,715
	KG	78,471	7,738	12,134	37,259	12,163	29,808
737-800	LB	174,700	15,100	24,886	81,730	27,078	65,384
	KG	79,242	6,849	11,279	37,060	12,282	29,658
737-900	LB	164,500	14,998	23,369	78,962	25,498	63,169
	KG	74,616	6,803	10,600	35,817	11,566	28,653
737-900	LB	174,700	14,155	23,045	81,743	27,078	65,394
	KG	79,242	6,421	10,453	37,078	12,282	29,662
737-900ER	LB	188,200	15,206	24,810	88,993	29,227	71,194
	KG	85,366	6,897	11,254	40,367	13,257	32,293

Figure 77. Boeing 737-800 main gear weight Airplane. From “737 Characteristics for Airport Planning” by Boeing Commercial Airplanes (2013).



	UNITS	737-600	737-700	737-800	737-900	737-900ER
MAXIMUM DESIGN TAXI WEIGHT	LB	124,500 THRU 145,000	133,500 THRU 155,000	156,000 THRU 174,700	164,500 THRU 174,700	164,500 THRU 188,200
	KG	56,472 THRU 65,771	60,554 THRU 70,307	70,760 THRU 79,242	74,616 THRU 79,242	74,616 THRU 85,366
NOSE GEAR TIRE SIZE	IN.	27 x 7.7 - 15 12 PR			27 x 7.75 - 15 12 PR	27 x 7.75 - 15 12 PR
NOSE GEAR TIRE PRESSURE	PSI	206	205	185	185	185
	KG/CM2	14.50	14.44	13.03	13.03	13.03
MAIN GEAR TIRE SIZE	IN.	H43.5 x 16.0 - 21 24PR OR 26 PR	H43.5 x 16.0 - 21 26 PR	H44.5 x 16.5 - 21 28 PR	H44.5 x 16.5 - 21 28 PR	H44.5 x 16.5 - 21 30 PR
MAIN GEAR TIRE PRESSURE	PSI	182 THRU 205	197THRU 205	204 THRU 205	204 THRU 205	205 THRU 220
	KG/CM2	12.80 THRU 14.41	13.85 THRU 14.41	14.39 THRU 14.41	14.34 THRU 14.41	14.41 THRU 15.47
OPTIONAL TIRES						
MAIN GEAR TIRE SIZE	IN.	H44.5 x 16.5 - 21 28PR (1)	H44.5 x 16.5 - 21 28PR	NOT AVAILABLE	NOT AVAILABLE	NOT AVAILABLE
MAIN GEAR TIRE PRESSURE	PSI	168 THRU 205	179 THRU 205	NOT AVAILABLE	NOT AVAILABLE	NOT AVAILABLE
	KG/CM2	11.81THRU 14.41	12.59 THRU 14.41	NOT AVAILABLE	NOT AVAILABLE	NOT AVAILABLE

Figure 78. Boeing 737-800 tire pressure. From "737 Characteristics for Airport Planning" by Boeing Commercial Airplanes (2013).

Normal density, 30 MPa concrete was specified to be used for the slab with 3.28 MPa tensile strength. A factor of safety of 1.7 was applied to the modulus to obtain an allowable tensile stress of 1.93 MPa or 280 psi as the design manual formulas were in english units. Then, a 9 inch slab was hypothesized and then three different cases for the point load were investigated. The tensile stress obtained from these cases was compared to the allowable tensile stress of the slab as shown in the Appendix.

The 3 cases specified include:

- Case 1: wheel load close to corner of slab
- Case 2: wheel load at a considerate distance from edges of slab
- Case 3: wheel load at edge of slab but removed considerable distance from corner

After all cases were evaluated to have less tensile stress than the allowable, it was confirmed that a 9 inch slab would be sufficient. Moreover, the slab is unreinforced and the capacities obtained were dependent solely on the slab strength.

8.4.1.2 Slab on grade Considerations

The hangar above the slab is not humidity controlled and so the slab does not need a vapor barrier. The 12 in thick crushed gravel layer was chosen to provide good surface drainage. The water table was assumed to be much deeper than the slab subbase interface at 2.5m. Since the slab was unreinforced Frost had to specify joints for curling and crack width control. Moreover, sawcut contraction joints are used to limit random, out of joint, floor slab cracking. As a 9 in slab had been selected and normal density concrete specified, the manual suggests a maximum joint spacing of 5.5 m. Next, joints were specified every 4 m in each of the long and short directions of the hangar, in order to maintain an aspect ratio of 1:1 for the slab panels. The joints will be cut to about ¼ of the slab thickness, which is 60 mm from the top. Joints are usually located on column lines, so the major joint lines will be on the column lines with intermediate joints located at 4 m spacing between the column lines. Also, isolation joints should be used wherever complete freedom of vertical and horizontal movement is required between the floor and adjoining structural elements. Thus, isolation joints are specified at the column interfaces and where the slab meets the foundation elements like the pile cap in order to ensure independent movement of the slab.

8.4.2 Pile Design

8.4.2.1 Overview

For the foundation design, QualiLab Inspection Inc. was consulted on the procedure for designing foundations in Kuujjuaq, Quebec. It was specified that the soil conditions included silty clay with a varying firm to stiff consistency. Permafrost was another considerations when designing in Kuujuuaq and thus it was specified by QualiLab to design a deep foundation, bearing on bedrock as opposed to conventional insulated shallow foundations. Therefore, H shaped piles were selected to be driven down to the 9.2 m deep bedrock.

The pile design was carried out using the β method from the Canadian Foundation Engineering Manual. Design specifications were provided by QualiLab and are outlined in Table 16.

Table 16. Geotechnical parameters

Parameter	Value
Depth to bedrock	9.2 m
Groundwater level	2.5 m
Bearing capacity factor (Nt)	250
Firm Clay Coefficient (β)	0.25
Soil Saturated Unit Weight (ϒ)	17 kN/m^3
Angle of Friction (Φ)	25 deg
Factor of Safety	2

For constructability considerations, a working platform will be built to accommodate the pile driving process. The onsite contractor is responsible for building the platform, however it is recommended that a Texel Geo-9 geotexile membrane be placed down with 600mm of crushed MG-20 stone on top. Another onsite requirement includes carrying out dynamic load tests to verify the allowable loads of piles compared to the calculated pile resistance. While settlement was considered negligible because piles were designed to bedrock, dynamic tests will also verify this on site.

The foundation was designed as three attached components including a pedestal, pile cap and piles. A 2 m high pedestal was designed to ensure that if the ground floor slab experienced any excessive settlement, it would act independently of the foundation system and not bear on the pile cap. Each pedestal, pile cap and pile system was designed to accommodate the load from two columns above, as shown in Figure 79.

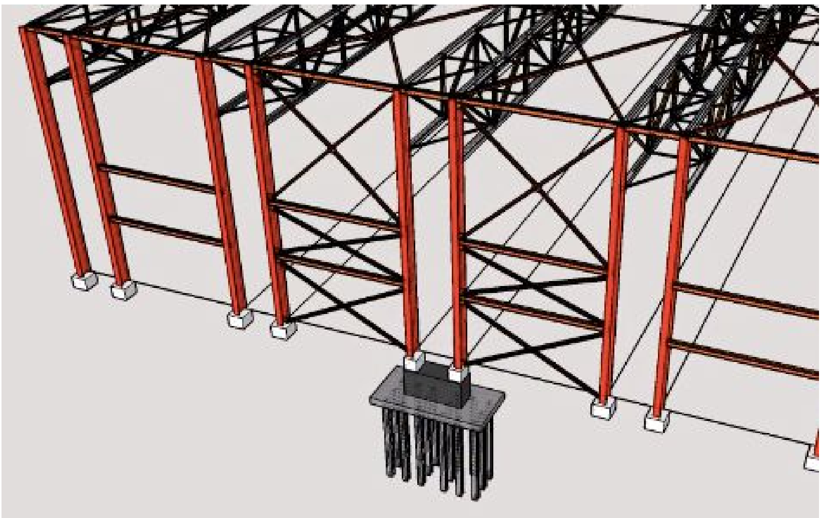


Figure 79. Foundation system outlining pedestal, pile cap and piles

8.4.2.2 Pile

As mentioned above, H piles were used for the deep piles. Using the β method outlined in the Engineering Foundation manual, the pile size and grouping were selected based on the respective maximum pile resistance, which surpassed the maximum column loads from the above structure. A HP 360x17 steel pile was selected. Due to constructability constraints outlined by the client, the pile cross-section height was constrained to a maximum height of 360 mm. Therefore, to satisfy the height constraint and the loads from the columns, 8-piles were designed to carry the loads from one column, for a total of 16-piles per pile cap. Figure 80. shows the foundation system with the 16-pile layout.

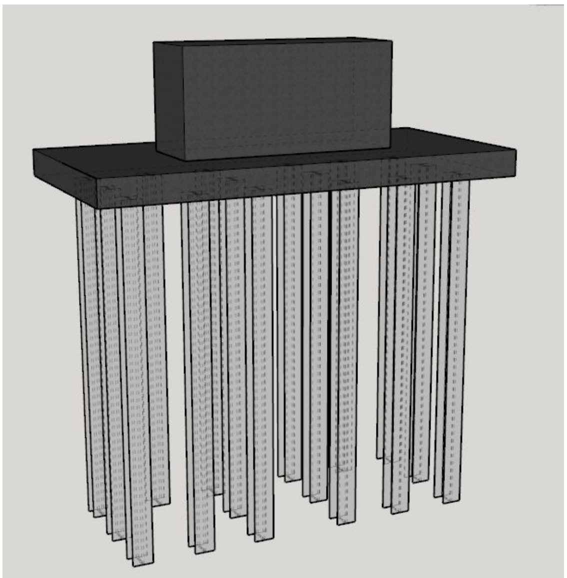


Figure 80. Pile foundation system

It was determined through load inspection that the two column pedestal, pile cap and pile system outlined above was the worst case scenario that the foundation had to be designed for. However, there are two other types of foundations that have been considered. Firstly, for the office the same system will be used however each foundation system will sit under one column thus having 8 W360x17 piles, a 8.15 m by 4 m by 0.5 m pile cap and a 5 m by .6 m by 2 m pedestal. This is a conservative design for the office foundation as the columns carry less loads than the hangar columns. Secondly, the megadoor foundation was considered. To decrease the strength requirements of the box-truss system, it was assumed that the megadoor loads bears on the foundation as opposed to being completely suspended from the box-truss. Thus a distinct foundation had to be considered for this side of the hangar. The megadoor foundation would vary in that it would not include a pedestal, but rather the pile cap, 4 m wide and 0.5 m deep, would sit at ground level and be continuous along the entire length of the hangar's long side. Then piles would be drilled continuously, along the length down to the bedrock. Figure 81. shows where the pile cap and pile system would be located.

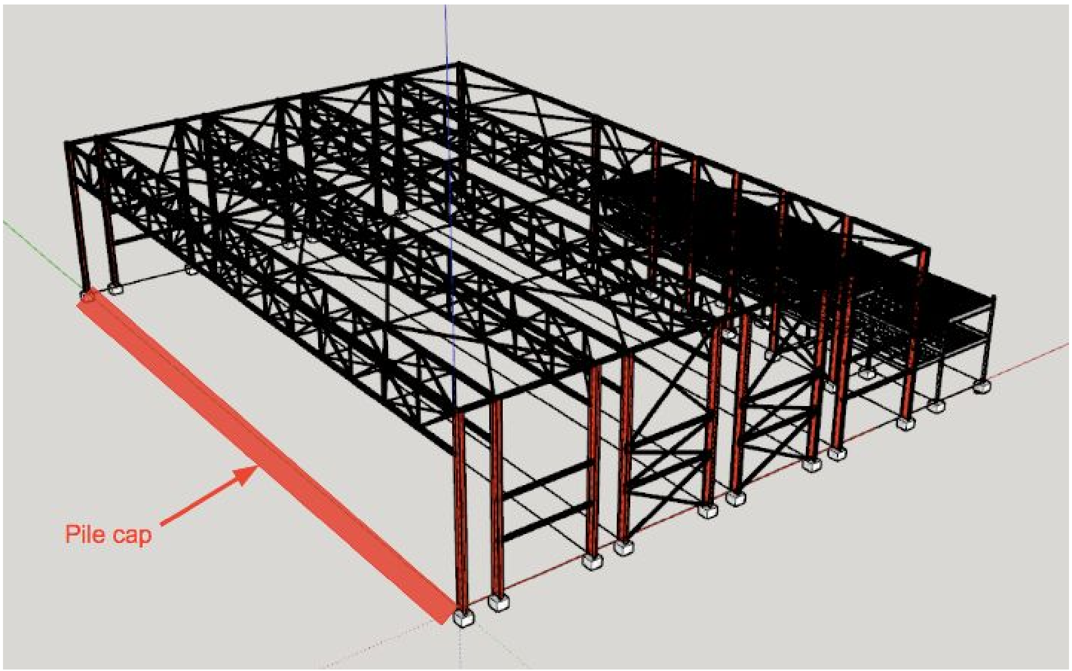


Figure 81. Pile cap foundation

8.4.2.3 Pedestal

There were 9 foundations to be designed. The most critical loading scenario appeared to happen mid-column joining the office space and the hangar, with a factor column loading of 1953 KN. All foundations were designed according to the most critical loading scenario.

As mentioned above, the layout for the deep foundation was that two base plates sat on one pedestal and the pedestal sat on the pile cap with piles embedded into the pile cap. The below figure gave the basic dimensions of the base plate.

Due to the fact that the two columns were close to each other, as well as the request of our client, one pedestal was designed to sustain the two column loads from the above. The overall dimension of the pedestal was 5 m x .6 m x 2 m. The pedestal was designed as a short column with reinforcement. The plain dimension of the pedestal had to be greater in length and width than the base plates for the plates to fit and sit on the pedestal. Since the column loads only acted on the baseplate areas of the pedestal, in order to increase the rebar efficiency, it was determined that the reinforcements would be placed under the baseplates, as shown in Figure 82. arrangement below.

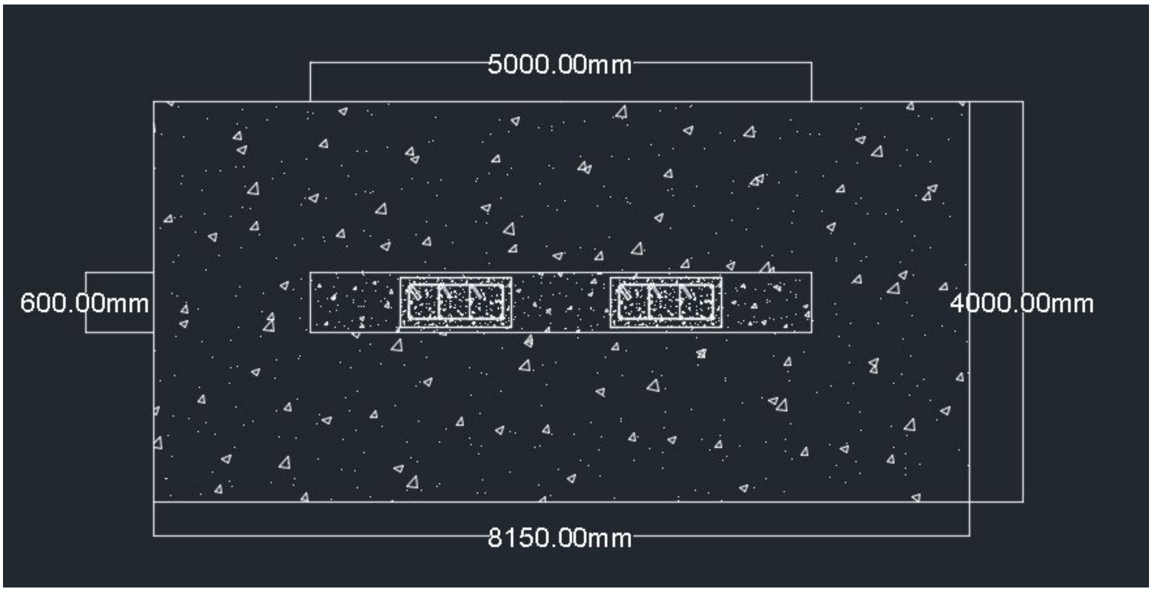


Figure 82. Plan view of the footing.

A detailed reinforced section under the base plates were shown in Figure 83 below. 12-25M bars were used to provide minimum reinforcement for the pedestal for one side.

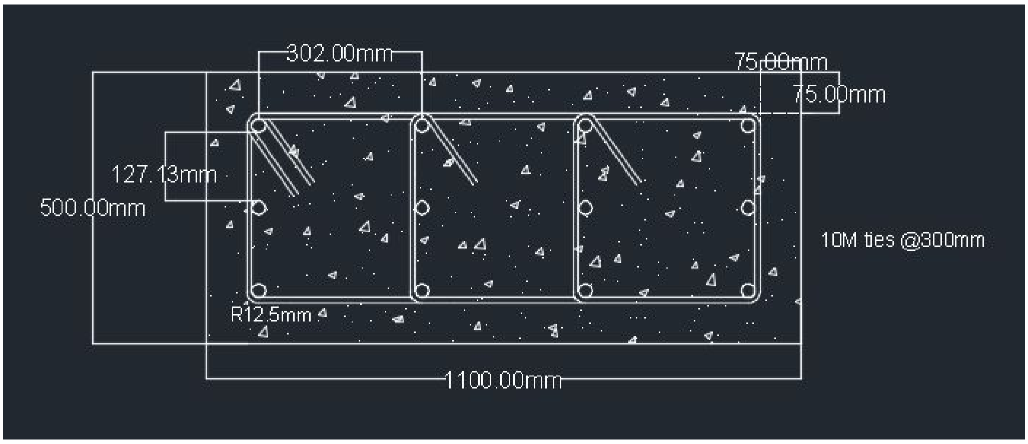


Figure 83. Pedestal reinforcement detail

8.4.2.4 Pile Cap

The minimum pile intrusion depth of 0.16 m was required and the minimum spacing between the top of the pile to the bottom of the rebar was 0.16 m. This meant that the pile cap must have a greater dimension than the sum of the spacing: 0.32 m. Another consideration was that the minimum pile cap dimension according to the layout of the H piles. It was determined to be 8150 mm x 4000 mm x 500 mm. Reinforcements were placed to satisfy and retain the minimum pile cap dimension.

First of all, the foundation depth was checked according to Rankine's formula. The assumed area of the pile cap was then checked with respect to the pile pressure. The minimum dimension of the pile was calculated by dividing the column loads by the pile pressure. The proposed dimension was larger than the minimum dimension of the pile cap according to the pile pressure. The reinforcement in the long direction was calculated to be 22-25M bars, using the similar design approach, the reinforcement in the short direction was calculated to be 114-25M bars.

As the calculation revealed, the dowels would not be needed to aid in the strength at the bottom of the column or pedestal; therefore a minimum dowel placement was designed. The dowel placements for connecting the pedestal with the pile cap were 4-10M bars.

The overall layout of the foundation was shown in Figure 84., and the basic dimensions were summarized in Table 17.

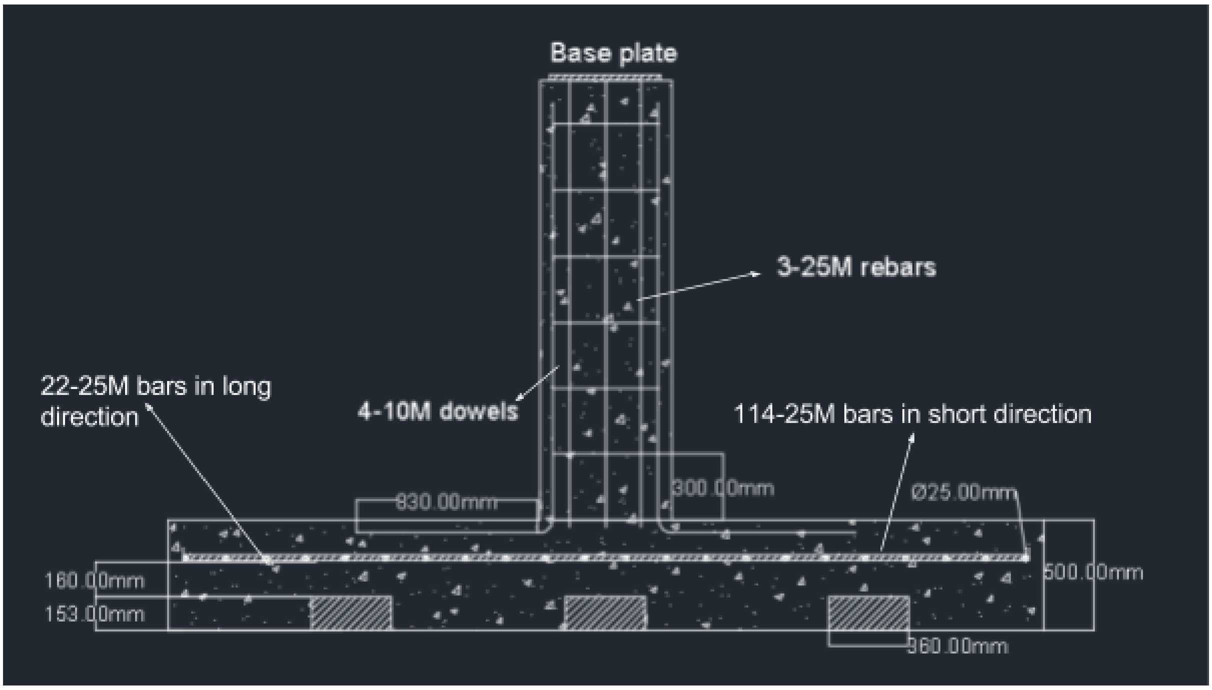


Figure 84. Footing layout.

Table 17. Footing dimensions

dimensions	L (mm)	W (mm)	H (mm)
Base plate	1100	500	25
Pedestal	5000	600	2000
Pile cap	8150	4000	500

8.5 Additional Considerations

8.5.1 Cladding

Cladding had the functions of providing thermal insulation and protection against the weather. It also helped to improve the appearance of the building exterior. Two types of cladding were considered for installation: aluminum panel cladding for the hangar and glass curtain wall system for the office.

First of all, aluminum composite panel cladding system had several benefits and was particularly suitable for the hangar exterior. The aluminum composite panels were lightweight, high strength, durable, and cost-effective. Insulation properties were also important particularly for such a structure in the permanent frost area. Aluminum had been well-known for its good insulation properties. Aluminum thermal insulation was based on the radiant barrier principle and worked in both hot and cold environments (Aerolam, 2012) . The installation of such panels utilized the rail and chip system. The rails were bolted to the concrete wall and the prefabricated cladding panels with cold-rolled C shaped HSS on the inner side of the panels acted like chips to allow the connection between the two pieces. The sections of contact would be fixed by the joints to ensure the rigidity of the connection. This scenario was shown in Figure 85. As shown by Figure 86, the detailing of office cladding was not very different from the detail of the hangar space, except that the the panel material was chosen to be glass for improving the window efficiency as well as the appearance.

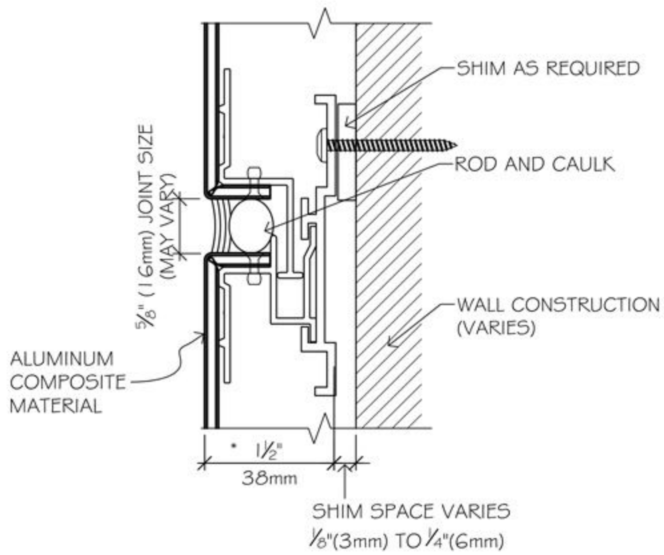


Figure 85. Hangar cladding detail. From "Composite Panel System" by CMC Systems, http://www.custommetal.ab.ca/Systems.html 2014

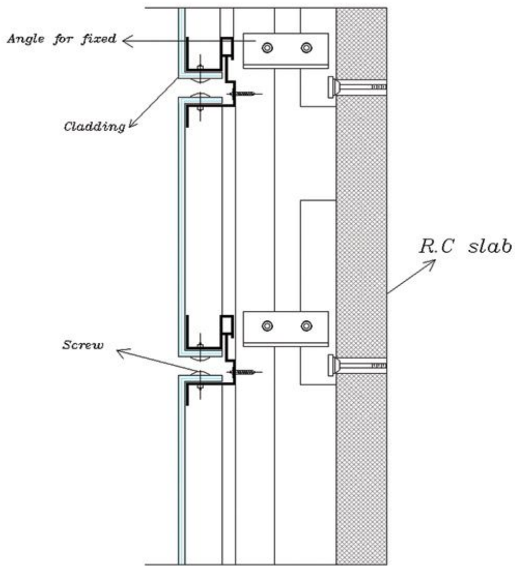
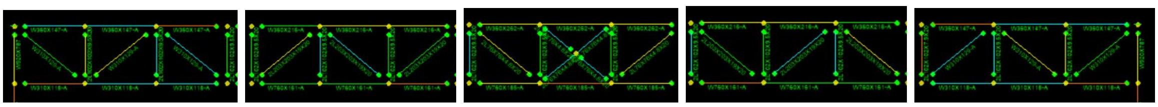


Figure 86: Glass curtain wall detail. From "Composite Panel System" by CMC Systems, http://www.custommetal.ab.ca/Systems.html 2014

8.5.2 Transportation

Special considerations needed to be accounted for with regards to transportation, due to the site location. First of all, all materials and equipment must be shipped by boat and then travel by truck to site. In addition, steel structural members such as the pre-cambered box trusses, joists, braces, shear pads, plates, and web members were prefabricated and shipped to the site. The boats can ship members up to 20 m in length. This meaned that the box trusses needed to be sliced for the convenience of shipping. The box trusses spanned 77 m in length and 4 cuts would be made to make 5 sections. Figure # shows the box trusses sections and their splice location.



Span	**1**	**2**	**3**	**4**	**5**
Length(m)	**15.69**	**15.24**	**15.24**	**15.24**	**15.69**

Throughout the design process, all members were selected to be as uniform as possible to simplify fabrication. For example, the purlin member selections had 4 unique spans for 4 different cases; however, in the end the members were only designed for the governing case.

For the fabrication of the concrete sections, the low temperature required additional considerations. As mentioned in the materials section, the cast in place concrete would take longer to settle and cure; therefore insulation blankets would be required for maintaining the concrete at a constant temperature.

8.5.3 Assembly

The assembly process happened simultaneously in the hangar as well as the office due to the limited construction period. Figure 87. gives the assembly sequence for the erection of the structure.

The very first step was the foundation drilling down to the bedrock. After the vertical erection of the 12 piles, a drilled prefabricated pile cap with the minimum intrusion depth and the required spacing to the rebar was placed on top of the piles. The baseplates were then placed on top of the pedestals for the erection of the columns. The excavated soil were then backfilled and compacted.

Once the foundation construction was completed, the overall shuttering was planned to allow the construction of overland parts. Baseplates were placed and bolted to the pedestal and reinforcements were fixed in place. The next step was to complete the slab on grade casting. As mentioned in the material section, the curing of concrete could be slow therefore it was suggested that the column casting happened at the same time with the slab on grade casting.

The box trusses would be placed onto the respective erected columns. The purlins were then placed on top of the box trusses to be supported by pins and rollers. Once the structure skeleton was erected, the subsequent detailed procedures could proceed.

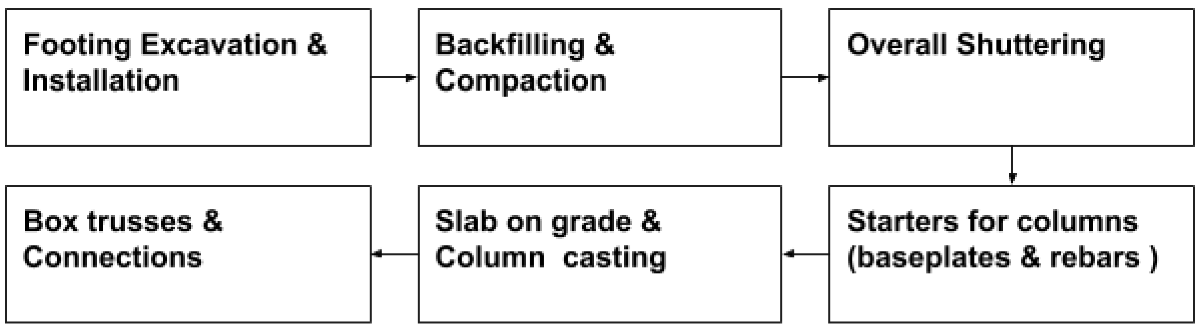


Figure 87. Assembly sequence

8.5.4 LEED considerations

LEED stands for Leadership in Energy and Environmental Design and acts as a green building rating system. This program encourages the global adoption of sustainable green building development practices. As a fast-growing engineering consulting firm, Frost always takes the initiative to incorporate green building construction and design into projects. Particularly, for this project, implementing LEED standards surrounding the type of steel used was considered. Due to the fact that steel constituted almost 70% of the structure, it was determined that using recycled steel would be an environmentally-friendly decision. Three other aspects that were considered for LEED in this project include the use of high performance windows, incorporating superplasticizer to increase the workability of the concrete, and using more gravel compared to concrete in the composite slab material.

9.0 Design Summary

Overall, Frost has completed the efficient design of an airplane hangar in Kuujjuaq, Quebec. The design requirements were comprised of three main components including the airplane hangar, the adjacent office, and the foundation design. The site location in Northern Quebec required extra constructability and transportation considerations, which have been outlined in the report.

Firstly, the client had specified that an open hangar be designed with the structural capacity to store a Boeing 737-800 and a DASH8 Q-400. The hangar structure was designed with a box-truss system roof with horizontal bracings and a complex lateral bracing system. Unlike typical structures, aircraft hangars only have exterior columns to allow for the storage of aircrafts inside. This imposed clear span design constraint meant that the entire roof load had to be transferred through a truss system to the exterior columns through the development of the box-truss system. The large megadoors on one of the long sides of the hangars, meant that vertical braces couldn't be placed on this side which in turn would create a large torsion on the frame if the lateral forces are not safely transferred to the foundations. To compensate for this design constraint, a lateral bracing system was designed with this issue in mind. Other significant structural members included the decking, purlins, columns, column bearing baseplates, and connections.

Secondly, a predominantly steel two-storey office building was designed adjacent to the airplane hangar. One significant design challenge was that a large snow accumulation occurred on the office as a result from snowdrift from the adjacent hangar. This resulted in the choice of a high strength decking for the roof system. The decks were designed to sit on steel joists, which are closely spaced to provide the required support to carry the high snow pressure. Moreover, the joist were placed on external beams that transfer their reaction to the square HSS columns. below. Other significant components of the office that were designed and considered included the deck and joist on the second floor, the lateral system and connection design.

Finally, due to the poor soil condition in Kuujjuaq, Frost designed a deep pile foundation to ensure the structural integrity of the hangar at the specified site location. The client required the use of H piles for the foundations, and they were designed accordingly. The deep pile foundation system was comprised of a pedestal, pile cap and piles. A slab on grade design was used for the ground floor of the structure.

10.0 Conclusions

After completing the design process for the airplane hangar, it is worth noting some of the main take-aways from this project. Firstly, while the project had many requirements when Frost initially took it on, it was crucial to break-down the project into smaller components in order to assign responsibilities, time constraints, and ultimately ensure the project requirements were met by the given deadline. One way to ensure this was done was to add buffer time to tasks that were anticipated to take longer or were more likely to face design challenges. For example, when completing the hangar, the SAP2000 model of the lateral system was a new challenge for Frost, and thus additional time had to be distributed to accommodate for the steep learning curve. Through splitting the tasks up into smaller components this also allowed group members to work with one another and on areas of interest or expertise.

Secondly, understanding how to communicate technical ideas while still providing a strong overall vision of the project had to be achieved in both the Midterm and Final presentation. To do so, the team had to adapt to focus less on the detailed calculations but rather build presentations that were visually appealing, including both videos, tables, and images, to allow the audience to understand the entire scope of the project. To further help the audience and our team understand the scope of the project and constructability, a SketchUp model was built to account for all design components, and portray all the design considerations to the audience.

Thirdly, while our team had a strong academic background through undergraduate level courses (i.e., steel, concrete, geotech, etc.), Frost had to learn how to design a conservative and adequate structure in a time sensitive situation. For example, while in academic courses the members or connections designed are often stated, for the design of the hangar the most critical situation had to be determined analytically and designed accordingly. This also meant that the team had to make trade-offs between designing all sections to complete accuracy, which would have taken an immense amount of time, and designing only for the most critical situation, which was less time consuming but meant some members may be oversized. In the end, Frost found a balance in which governing components were designed and when applicable, thorough designs were carried out to allow for more optimal and cost-effective components to be selected.

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