Project No. 24-29

DESIGN OF STEEL STRUCTURE AND PUMP STATION ESTABLISHED BY SECRET LAKE IN OKANAGAN SIMILKAMEEN, BC

(Romtec, 2021)

Prepared for:

Industry Projects Committee, BCIT Civil Engineering Department

Kian Karimi, Faculty Advisor, BCIT Civil Engineering Department

Jacquie Russell, Instructor, BCIT Communications Department

Prepared by:

Benjamin Mendoza

Submitted on:

April 12, 2024

Civil Engineering Department

British Columbia Institute of Technology

Burnaby, BC

DISCLAIMER

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

ACKNOWLEDGMENTS

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

- Kian Karimi for helpful insight on the structural part of my project, ensuring that it seems realistic.
- Jacquie Russell for contributing to the technical writing aspect of this report, providing constructive feedback, and locating any errors.
- ________and _____________ for sponsoring this project and providing helpful advice for the SAP2000 model as well as how the industry functions.
- Zaki Abdullah for the pump and water aspects of my project, confirming the layout seems reasonable.

Benjamin Mendoza

Burnaby, BC – _____________ – (___)-___-____

April 12th, 2024

____
____
____
Industry Project Sponsors
____
____

Dear ________________,

Submission of Final Report on the Design of Steel Structure and Pump Station Established by Secret Lake in Okanagan Similkameen, BC

This is the submission for the final report of the steel structure and pump station constructed in the Okanagan, BC. The task of this project was to supply drinkable water to a nearby community in a remote location by designing a pump station consisting of a pipe network, pump, and protective steel structure. This project also consists of various cost analysis, a model of the pipe network in AutoCAD as well as a model of the structure in SAP2000 with hand calculations.

Using the information and details you have given to me, I created a comprehensive and conceptual structural design of a pump station while accommodating the difficulties of having the location in a remote area. A total of 128 hours was spent on the entire project.

This complex project allowed me to gain a comprehensive understanding of the amount of effort and detail that goes on in the industry. I gained experience with SAP2000, a structural software that is well-known in the industry, by implementing my steel structure into SAP2000. Finally, I also gained more of an understanding of the components of a pump station such as effluent and influent pipes, pipe supports, and a chemical storage for water treatment.

Thank you ______________________, for sponsoring my project by giving helpful advice consistently throughout the duration of the project, despite being busy with work.

If you have any questions, please contact me at ____________ or ___-___-____.

Sincerely,

Benjamin Mendoza

Benjamin Mendoza

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

Attachment: Project Report

SUMMARY

The purpose of this project was to design a functional pump station and protective steel shelter near Secret Lake in Okanagan Similkameen, BC, aimed at supplying drinkable water to a nearby remote community. This comprehensive project involved the creation of a pipe network, pump installation, and logistical planning for material and equipment transportation led by ________________________and ______________________. The project also utilized SAP2000 and AutoCAD for structural modeling and layout design, respectively. The project addressed the technical requirements of both steel and concrete structures to ensure durability and efficiency.

Using the City of Kelowna Design Standards, an inside pipe diameter of 200mm (ductile iron) was determined from the required flow of 70L/s. Utilizing the ground profile view from Google Earth Pro, the pump head from this system came to 75.3m. The pump required for this application was determined to be MPC-E 4 CRE 95-2 running for 45 years for 6 hours a day bringing the total cost for the pump aspect to be $845,225.57. The pipe support consists of an HSS 127x6.4 with a saddle support having a spacing of 5.7m.

Utilizing the National Building Code of Canada (2020) and the Handbook of Steel Construction (11th edition) for steel design, along with the CSA A23.3-04: Design of Concrete Structures for Concrete Specifications, the project delivered a framework capable of enduring the necessary loads. The steel structure, characterized by purlins (C200x17), side girts (C180x15), end wall rafters (W200x42), front girts (C180x15), columns (W200x52), and middle frames (W200x52), was optimized for strength and weight, ensuring a resilient shelter for the pump station. The total cost for all steel members came to $135,675 including cladding.

The concrete components, including the pump pad (200mm thick), base slab (400mm thick), and strip footing foundation (250mm thick), were calculated to support the operational needs of the pump station, using 25 MPa concrete and 350 MPa rebar, confirming the infrastructure's stability. The concrete cover for the strip footing foundation and base slab came to 75mm while the pump pad concrete cover came to 40mm. The strip footing had a base of 1m by 1m with a total depth under the soil of 850mm. The size of the rebar calculated came to 15M for all temperature reinforcement, main reinforcement in the strip footing and pump pad as well as 25M for main reinforcement in the base slab.

Logistical planning addressed the challenges of the project's remote location, detailing the clearing of a 219m pathway (5m wide) for construction and maintenance access, thereby aligning with the project's environmental considerations and operational requirements.

Results from SAP2000 simulations affirmed the structural integrity under various loading conditions, showcasing minimal deflections and stress levels within acceptable limits, thus highlighting the effectiveness of the selected materials and design approaches.

The project culminated in a series of recommendations aimed at ensuring code compliance, validating structural calculations, and proposing further investigations for cost analysis and soil inspection, ensuring the project's adaptability and safety. Additionally, a detailed cost analysis provided insight into the financial implications of the steel structure, cladding, and logistical operations, facilitating informed decision-making for future implementations.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

Tables

Figures

1.0 INTRODUCTION

The purpose of this project was to design a functional pump station near a water reservoir in a difficult-to-access, remote location. The project consisted of designing a protective steel structure for the pump station as well as logistics planning for material/equipment transportation. The pump will be supplying water to a nearby community from Secret Lake Okanagan-Similkameen, BC. The lake is approximately 3.45 hectares and 1.4 kilometers away from the community. Figure 1 below shows the location of the pump relative to the nearby community.

Figure 1: Secret Lake Access to Designated Community (Google, 2023-a.), (B. Mendoza)

Figure 1 also shows a path with 219 meters of forest (5m wide) that will need to be cleared for access to and from the pump for construction and maintenance.

This project was guided by _______________and ______________________________. They chose this project because they believed it would be fun but sufficiently complex. They also believed it would effectively demonstrate what it is like to work in the water group at ______.

The objectives of this project were the following:

- Design the pump station and its requirements.
- Design and calculate a simple steel structure for the pump station and determine what types of beams to use.
- Design the concrete foundation for the pump station and structure.
- Design the structural aspects in SAP2000 for a structure analysis.

Several areas fell outside the project's scope. Some aspects include designing drainage areas for the surrounding pump station site, assessing seismic loading, and conducting an environmental site assessment. Additionally, the project's scope does not cover bolt connections or the design of members around windows and doors. It also excludes details on how electricity will be supplied to the pump station.

The remainder of this report describes the pump design and pipe footing design as well as the design of the steel structure and concrete foundation. It will also discuss the methods used, logistics planning, results, and recommendations.

2.0 PUMP DESIGN

To design the pump station for this application, a required flow had to be determined to acquire a minimum diameter for the pipe to compute friction losses. The friction losses were then used to determine pump head to select the proper size of a pump.

After determining all the water requirements of the pump station, a layout of the station was created using AutoCAD. The layout consisted of the location of the pump, chemical storage, effluent and influent pipes, and the overall size of the structure with pipe supports being added later on. Refer to Volume II for drawings.

It is important to note that any information referencing to hand calculations, equations or pages are referencing to Volume III unless otherwise stated.

2.1 Water Requirements

Firstly, the per capita demand was calculated using the given information in the City of Kelowna Design Standards, as shown in Figure 2 below.

1.3 Per Capita Demand

Use the following per capita demands for future residential requirements:

- Average annual daily demand (ADD): 900 litres per capita per day (L/c/d)
- Maximum day demand (MDD): 1800 litres per capita per day
- Peak hour demand (PHD): 4000 litres per capita per day

Design population density:

Single Family	3.0 people/dwelling
Multi-Family	2.0 people/dwelling

Figure 2: Water Distribution - 1.3 Per Capita Demand (City of Kelowna, 2020)

Figure 2 illustrates the projected residential requirements and the population density to be used for design purposes. The population density used was 4.0 people/dwelling to supply a more realistic flow for 100 houses. Using Equation 1 on page 1, it was determined that a total of 400 people would be supplied water from this pump.

Additionally, the Design Standards outline the required fire flow in Table 1.5, Section 1.5, on page 3 of 18. For Single Family & Two Dwelling Residential, the fire flow is 60 L/s. In section 1.6, the standards show the minimum required flow is MMD (Maximum Day Demand) plus FF

(Fire Flow) or PHD (Peak Hour Demand), whichever flow is larger. Using Equation 2 on page 1, the largest required flow calculated was 70L/s from MMD plus FF.

2.2 Pump Requirements

To determine the type of pump to use, head loss had to be calculated. To determine head loss for the pipe system, Equation 3 on page 2 was used as well as Figure 3 below, showing the existing ground level and proposed pipe.

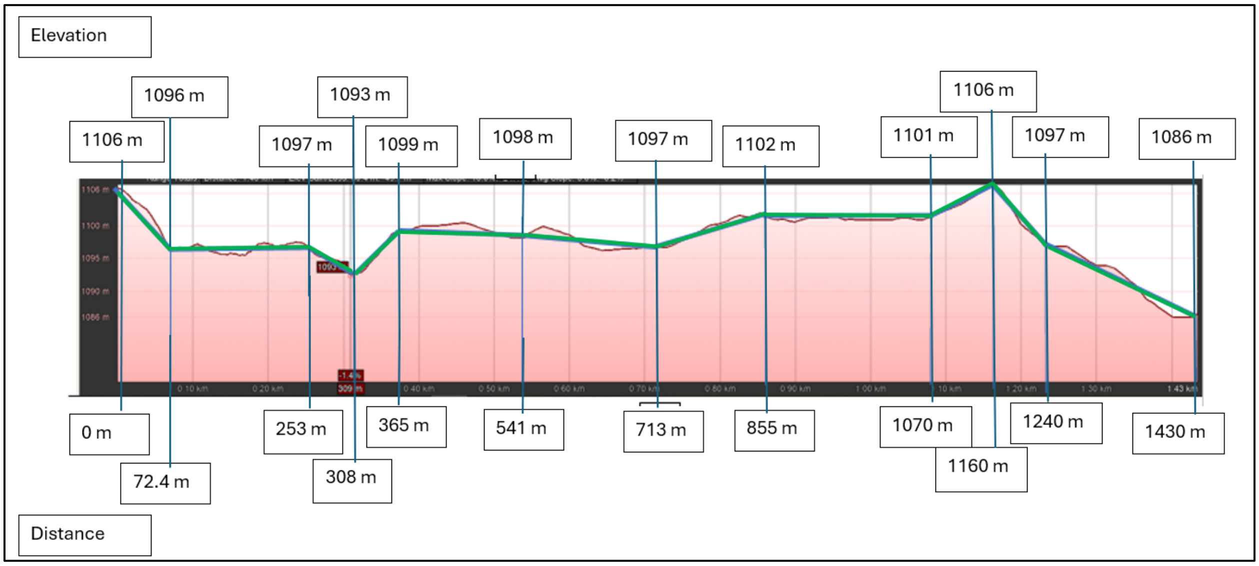


Figure 3: Proposed Pipe System (Google, 2023-b.), (B. Mendoza)

Figure 3 shows the elevation points on top and distance from the pump station on the bottom of the profile view. The green lines represent the proposed pipe, and the red shaded area represents the existing ground.

The selected 'C' value, representing the roughness coefficient for ductile iron, was 140. The minimum allowable pressure for MMD+FF is 140 kPa, so a pressure of 250 kPa was selected. The initial elevation of the pump station was recorded as 1086 meters, while the final elevation reached 1106 meters. Using the elevation points, pressure, head loss as well as keeping a constant flow of 70 l/s with a minimum diameter of 200mm, the pump head was calculated using Equation 4 (Bernoulli's equation) on page 3. The head pump calculated was 75.3 meters.

Page 4 consists of calculating the x, y, z coordinates of the pipe system for AutoCAD modeling. Appendix A Figure A1 shows the top view as well as the ground profile view of the 200mm pipe modeled in AutoCAD. The ground profile view in Figure 3 is exaggerated when compared to the profile view of the AutoCAD drawing.

Initially, four pumps were selected based on a pump head of 43 meters, using information from both the Pump Stop LLC (2024) and Grundfos (n.d.) websites, as detailed on pages 5 and 6. These pumps were later discarded in favor of selecting another four pumps, as detailed on page 9, based on a pump head of 75.3 meters.

Appendix B Table B1 shows the cost breakdown of the four pumps selected by comparing the pump cost and power consumption. According to the British Columbia Utilities Commission (2024), the cost of power can be assumed to be $0.0975/kWh. Assuming the pump runs for 6 hours a day and for 45 years before being replaced, the best pump to select would be the Hydro MPC-E 4 CRE 95-2 at $845,225.57.

2.3 Pipe Supports

Pipe supports needed to be accounted for within the station, and these were divided into two parts. The first part consists of elements that cradle and secure the pipe in place. The second part includes components that elevate the pipe 600 mm off the ground.

2.3.1 Saddle Support

The initial step involved selecting the appropriate type of support for securing the pipe in position. To this end, the 'Piping Handbook, 7th edition' by Nayyer, M.L. (2000), was consulted to determine both the spacing of the pipe supports and the specific type of support required. According to Part B on page 223, the saddle support (type 36) was identified as the optimal choice.

Furthermore, Table B5.1 on page 219 (Table 1 below) provides the recommended maximum span for pipes of various diameters and intended services.

Table 1: Suggested Pipe Spacing (Nayyar, 2000)

	Suggested maximum span, ft (m)	
NPS (DN)	Water service	Steam, gas, or air service
1 (25)	7 (2.13)	9 (2.74)
2 (50)	10 (3.05)	13 (3.96)
3 (80)	12 (3.66)	15 (4.57)
4 (100)	14 (4.27)	17 (5.18)
6 (150)	17 (5.18)	21 (6.40)
8 (200)	19 (5.79)	24 (7.32)
12 (300)	23 (7.01)	30 (9.14)
16 (400)	27 (8.23)	35 (10.7)
20 (500)	30 (9.14)	39 (11.9)
24 (600)	32 (9.75)	42 (12.8)

For the watermain with a 200mm diameter, a pipe spacing of 5.79m was selected, subsequently rounded down to 5.7m. The calculations for the forces acting on the saddle support are detailed on page 7. ____________ provided the initial values: 0.24kN for the x-direction, 0.14kN for the y-direction, and 0.10kN for the direction of gravity. Upon calculating the combined mass of the water and the pipe, this figure was added to the force in the direction of gravity, totaling 4.54kN.

To determine the appropriate size for the saddle support, the Grainger (2024) website was consulted. A pipe saddle with a 1 1/2 in – 10 in pipe size and a 3,800 lb load capacity was chosen, as it met all the necessary criteria when comparing the resistance to the load, as documented on page 8.

2.3.2 HSS Pipe Support

The HSS pipe support is designed to be mounted on a plate, which in turn will be securely bolted to the ground. This configuration is intended to suspend the pipe 600mm above the ground. The computations for determining the compressive resistance, as well as ensuring the structure's resilience against local buckling under applied loads, are documented on pages 11 and 12.

To calculate these critical resistance values, including the diameter, thickness, and gross area of the support, the 'Handbook of Steel Construction, 11th edition' published by the Canadian Institute of Steel Construction (2017) was utilized. This resource provided the essential parameters needed for the calculations.

The selected HSS pipe support was the HSS 127x6.4 to meet the necessary requirements.

3.0 STEEL DESIGN

When designing the steel members, starting with the roof was prioritized. This approach ensured that the loads from the roof were effectively transferred to the middle frames and columns, maintaining that the resistance was always greater than the factored load.

Utilizing the National Building Code of Canada (2020) by the National Research Council of Canada, commonly referred to as the NBCC, the snow load on the roof was calculated using equation 11, detailed on page 13. The calculation resulted in a load of 3.25kPa, which was applied to the roof, in addition to the weight of 10mm thick steel cladding. Additionally, a wind load was assessed, revealing varying values across different surfaces of the structure.

The selection of load cases for analysis was guided by the NBCC. The worst-case scenario for the loads, identified as case three from NBCC Table 4.1.3.2A, was applied to the roof. Conversely, case four was applied to the walls to determine the factored loads that the steel structure must withstand. These calculations are documented on pages 19 and 27.

For the roof's design, no ceiling system was included in the load calculations. _______did emphasize the importance of a load takedown in designing members subject to gravity loads. He later clarified the exclusion of a ceiling system or ducting for the roof, stating, “Assume no ceiling system or ducting...because the client (me) says we don’t want a suspended ceiling system or ducting on the roof.”

In calculating the loads for the sides of the structure, elements such as windows, doors, and other relatively small gaps were deemed negligible. _______ concurred with this assessment, noting, “I agree… that it is negligible. There is a very small effect on the frame and the overall building.”

It's important to note that all references to hand calculations, equations, or pages mentioned here are found in Volume III unless stated otherwise. Additionally, all steel sections selected for this project are of 350MPa grade steel. The source for all equations, unless specified differently, is the Handbook of Steel Construction by the CISC.

Figure 4 below shows the 3D image of the steel structure produced in SAP2000 and how each member will be orientated.

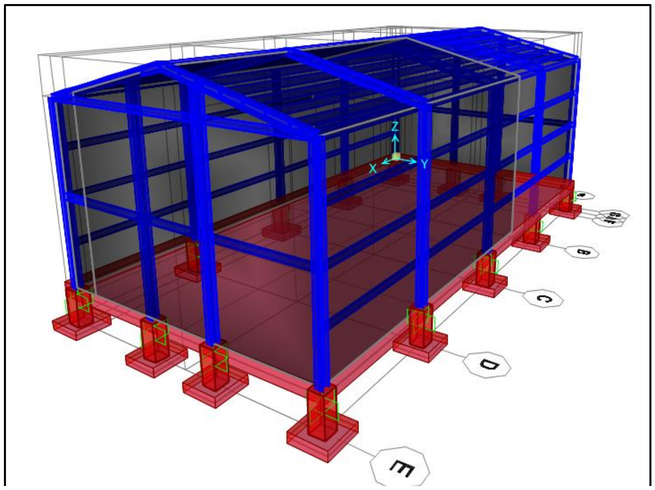


Figure 4: SAP2000 Model of Steel Structure (B. Mendoza)

Figure 4 shows the steel members highlighted in blue with concrete being highlighted in red.

3.1 Purlins

In the process of calculating the purlins for the roof, an initial cross-section for the purlin was assumed. This assumed section was then incorporated into the total weight calculation for the purlin, resulting in a comprehensive weight, including the purlin's own weight, of 3.2 kN/m.

Given the symmetry of the purlin's design, the analysis necessitated dividing the purlin into two distinct sections. Figure 5 below illustrates the resulting factored shear and bending moment diagrams.

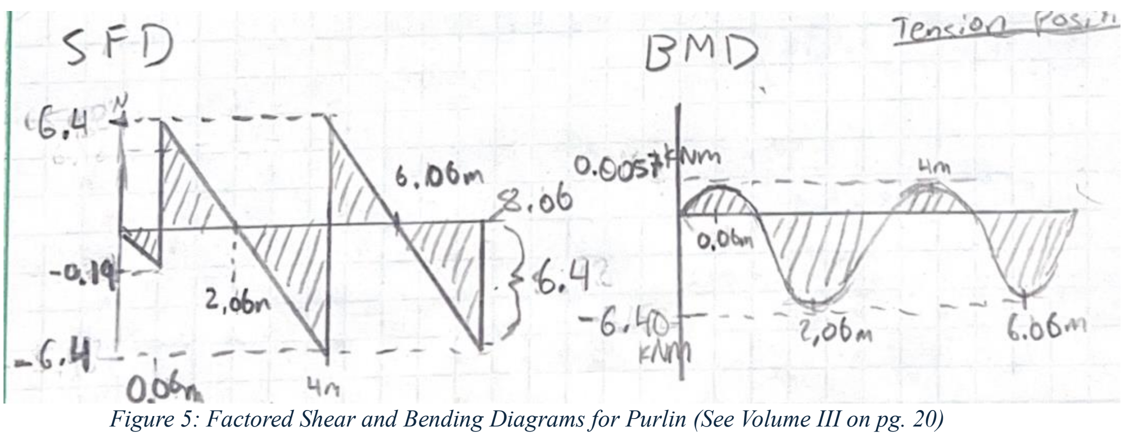


Figure 5: Factored Shear and Bending Diagrams for Purlin (See Volume III on pg. 20)

The analysis resulted in a maximum factored shear and bending moment of 6.4kN and 6.4kNm, respectively. These critical values were then compared against the purlin's resistance capacities to confirm its adequacy for the intended structural demands.

Selection of the purlin's section dimensions was guided by data sourced from the Handbook of Steel Construction. This comprehensive resource also facilitated the calculation of important structural parameters, including compactness, moment resistance, shear resistance, and bearing resistance. The detailed calculations supporting these determinations can be found on pages 19-24.

The chosen section, C200x17, met all necessary criteria. For these 16m long purlins, a spacing of 0.590m was adopted across the board, with an exception for the first purlin, which was set at 0.580m apart. Detailed specifications regarding this configuration are documented in Volume II, Note 3. In total, 13 purlins were utilized to support the roof, which is inclined at an angle of 14 degrees and rises 1m above the wall height.

3.2 Side Girts

In the calculation of factored shear and moments for the initial section, the weight of the section itself was not considered in the load calculations due to wind loads predominantly influencing the y-direction. The girt experienced varying loads: 0.893kN/m for the initial 6m and 0.591kN/m over the remaining 10m.

Subsequent analysis employed shear force diagrams (SFD) and bending moment diagrams (BMD) to ascertain the factored moment and shear. Figure 6 provides a visual representation of the unfactored bending and shear forces.

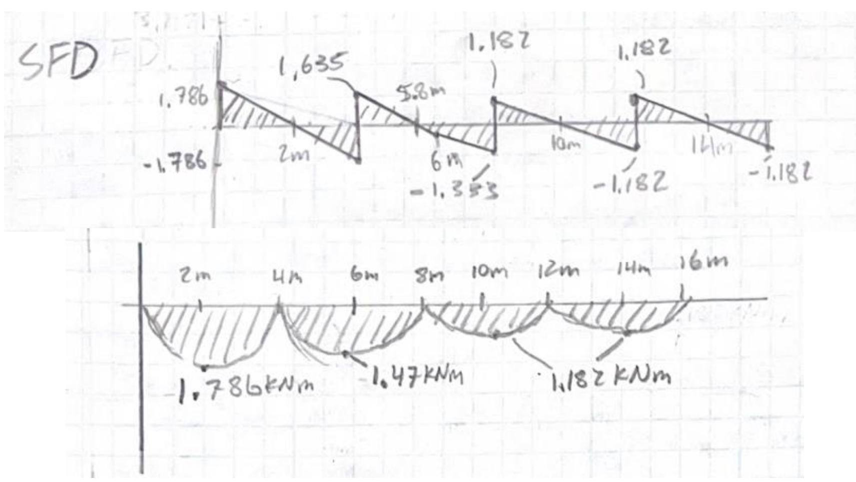


Figure 6: Unfactored Shear and Bending Diagrams for Side Girt (See Volume III on pg. 26, 27)

The outcomes revealed maximum unfactored shear and bending moments at 1.786kN and 1.786kNm, respectively. Factored shear and bending moments for the side girts were determined by amplifying the wind force by a factor of 1.25, resulting in values of 2.23kN and 2.23kNm. The analysis extended across the full length of the 16m side girt to accurately account for its uneven loading.

Utilizing the steel handbook, the resistance for the side girts were calculated, mirroring the approach used for the purlins. The detailed calculations for these determinations span pages 25-30.

Meeting all specified requirements, the chosen section for the 16m long side girt was C180x15, set at a spacing of 1.25m. Given a total wall height of 5m, the structure required four girts on each side, bringing the overall building height to 6m.

3.3 End Wall Rafters

Rafters were placed at the ends of the steel structure, functioning as separate beams to support the purlins. To ensure the accuracy of the structural analysis, the self-weight of these rafters was incorporated into the calculations of the factored moment and shear from the outset. Figure 7 illustrates the derived factored shear and bending moment diagrams for the rafters located at the structure's end walls.

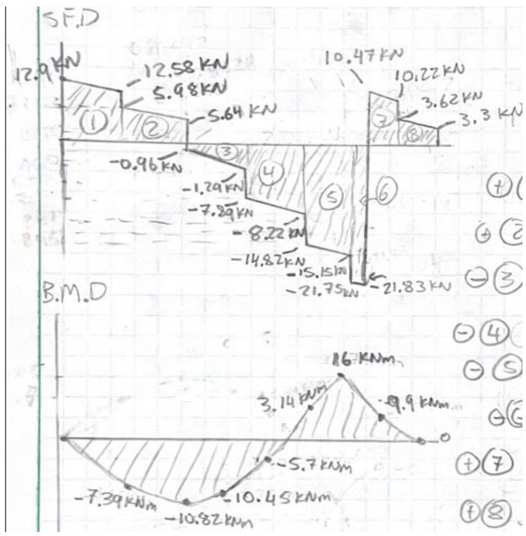


Figure 7: Factored Shear and Bending Diagrams for End Wall Rafter (See Volume III on pg. 32)

The analysis yielded a maximum factored shear of 21.83kN and a bending moment of 16kNm. The rafters, positioned at a 14-degree angle, span from a starting height of 5m to an ending height of 6m, reflecting the slope of the roof.

Adhering to the methodology employed in previous structural calculations, a W200x42 beam was determined to be adequately robust to support the specified loads. Each rafter, measuring 4.12m in length, is supported by two columns, providing a stable framework for the roof structure.

3.4 Front Girts

The design approach for the front girts mirrored that of the side girts. The initial front girt was positioned 2.5m above the ground, while the subsequent girt was placed at a height of 5m. Both girts, extending 8m in length, required analysis to determine the forces acting perpendicular to the columns. Figure 8 below provides insight into the calculated factored bending and shear forces for these girts.

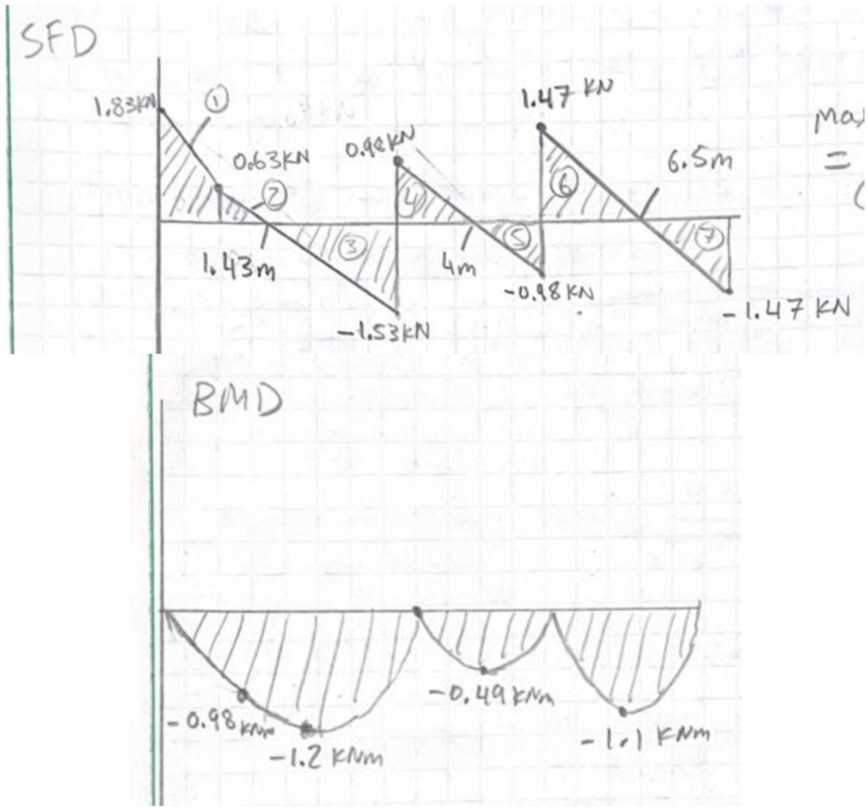


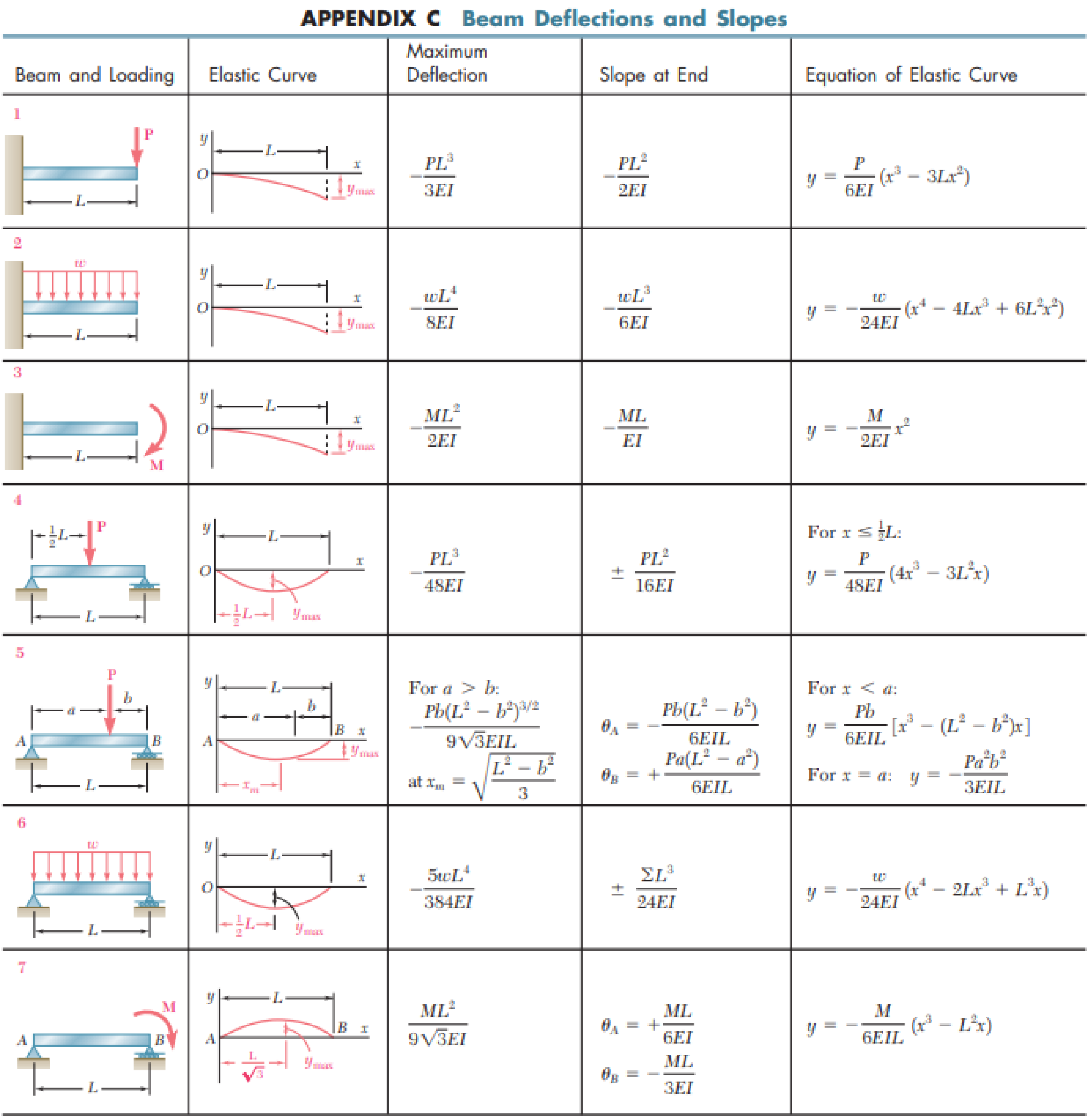
Figure 8: Factored Shear and Bending Diagrams for Front Girt (See Volume III on pg. 38)

In the analysis, wind loads on the girts were considered, whereas the self-weight of the beam was omitted. This calculation led to a maximum factored moment of 1.2kNm and a shear of 1.83kN. These values underwent a comparison with the resistances previously documented. The calculations for the front girt are shown on pages 36-41. Consequently, the beam selected for the front girts was identified to be the same as that used for the side girts, specifically, C180x15.

3.5 Columns

In this structural analysis, two distinct columns were scrutinized for their roles within the structure. The first column, positioned at the structure's corner and stretching 5m in height, supports both the front and side girts and an end wall rafter. The second column, situated 3m away from the first and extending to a height of 5.75m to reach the inclining roof, is tasked with supporting the front girts and end wall rafters.

Designated as beam columns, these structural elements are subject to both longitudinal and axial loads. They are securely fixed to the ground at their base and are connected to the end wall rafters with pin connections. The calculations account for the self-weight of the girts and the columns themselves, rendering the beam column system statically indeterminate. To navigate this complexity, deflection calculations from "Statics and Mechanics of Materials" by Beer, F. et al., (2011) in Appendix C, page 692, were employed. Figure 9 below delineates the various scenarios and equations necessary to resolve the indeterminacy.



APPENDIX C Beam Deflections and Slopes

Beam and Loading	Elastic Curve	Maximum Deflection	Slope at End	Equation of Elastic Curve
1		$-\frac{PL^3}{3EI}$	$-\frac{PL^2}{2EI}$	$y = \frac{P}{6EI}(x^3 - 3Lx^2)$
2		$-\frac{wL^4}{8EI}$	$-\frac{wL^3}{6EI}$	$y = -\frac{w}{24EI}(x^4 - 4Lx^3 + 6L^2x^2)$
3		$-\frac{ML^2}{2EI}$	$-\frac{ML}{EI}$	$y = -\frac{M}{2EI}x^2$
4		$-\frac{PL^3}{48EI}$	$\pm\frac{PL^2}{16EI}$	For $x \leq \frac{1}{2}L$: $y = \frac{P}{48EI}(4x^3 - 3L^2x)$
5		For $a > b$: $-\frac{Pb(L^2 - b^2)^{3/2}}{9\sqrt{3}EIL}$ at $x_m = \sqrt{\frac{L^2 - b^2}{3}}$	$\theta_A = -\frac{Pb(L^2 - b^2)}{6EIL}$ $\theta_B = +\frac{Pa(L^2 - a^2)}{6EIL}$	For $x < a$: $y = \frac{Pb}{6EIL}[x^3 - (L^2 - b^2)x]$ For $x = a$: $y = -\frac{Pa^2b^2}{3EIL}$
6		$-\frac{5wL^4}{384EI}$	$\pm\frac{wL^3}{24EI}$	$y = -\frac{w}{24EI}(x^4 - 2Lx^3 + L^3x)$
7		$\frac{ML^2}{9\sqrt{3}EI}$	$\theta_A = +\frac{ML}{6EI}$ $\theta_B = -\frac{ML}{3EI}$	$y = -\frac{M}{6EIL}(x^3 - L^2x)$

Figure 9: Beam Deflections and Slopes (Beer et al., 2011)

This analysis is ensuring that the deflection at the beam's pinned end sums to zero, allowing for the isolation and solution of the supporting force, as detailed on pages 43-44.

The analysis presented in Figure 10 below includes the factored shear force diagrams (SFD) and bending moment diagrams (BMD) for the 5m long column, identifying the worst-case scenario as case one. Both cases were still analyzed to account for directional differences, with case one aligned with the x-direction and case two with the y-direction.

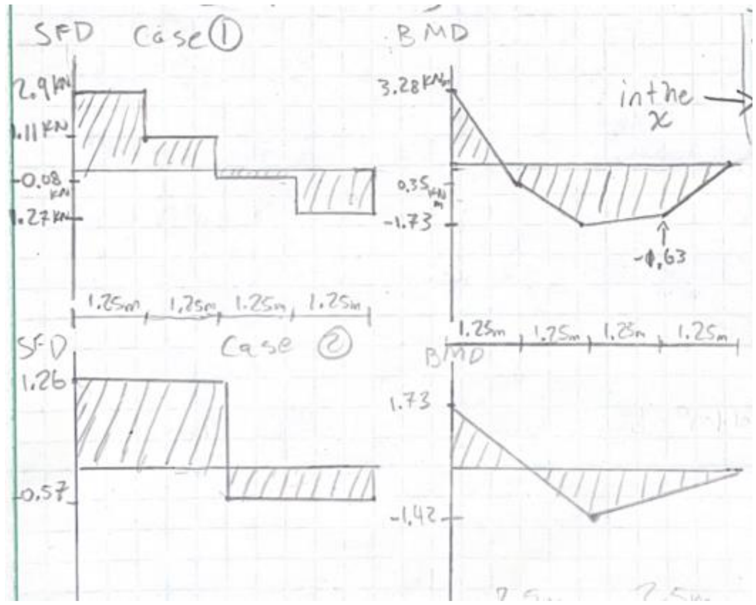


Figure 10: Factored Shear and Bending Diagrams for 5m Long Column (See Volume III on pg. 45)

The worst case was case one with a max shear of 2.9kN and a max bending of 3.28kNm.

The longer column was also analyzed as shown in Figure 11 below.

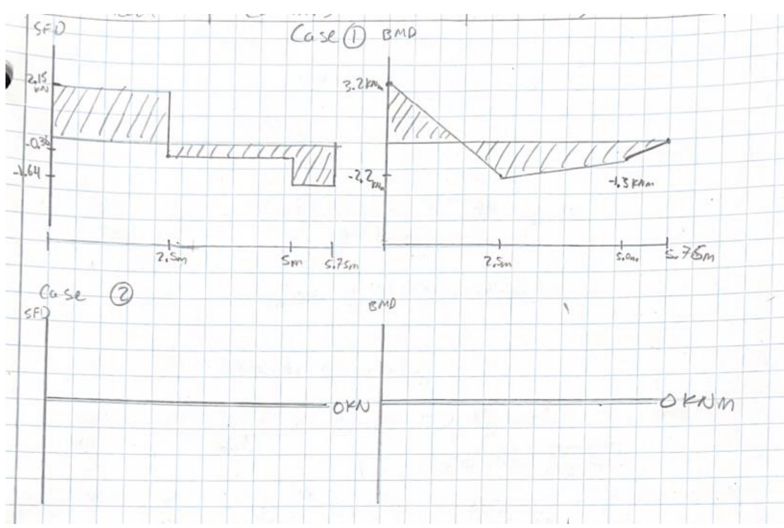


Figure 11: Factored Shear and Bending Diagram for 5.75m Long Column (See Volume III on pg. 53)

In Figure 11, the maximum factored shear and bending for case one was calculated to be 2.15kN and 3.2kNm, respectively. For case two, both the factored shear and bending were determined to be zero.

The ultimate selection of the W200x52 section for both columns was informed by comparing the maximum factored shear and bending moments against the resistances pertaining to cross-sectional strength, overall member strength, and lateral torsional buckling. These comparisons utilized equations outlined on pages 46-51 and 53-56.

3.6 Middle Frames

The middle frame was analyzed using SAP2000 to address its third-degree indeterminacy. It supports the purlins and side girts, is anchored to the ground, and functions as one continuous member. Shear and bending forces are shown in Figure 12 below.

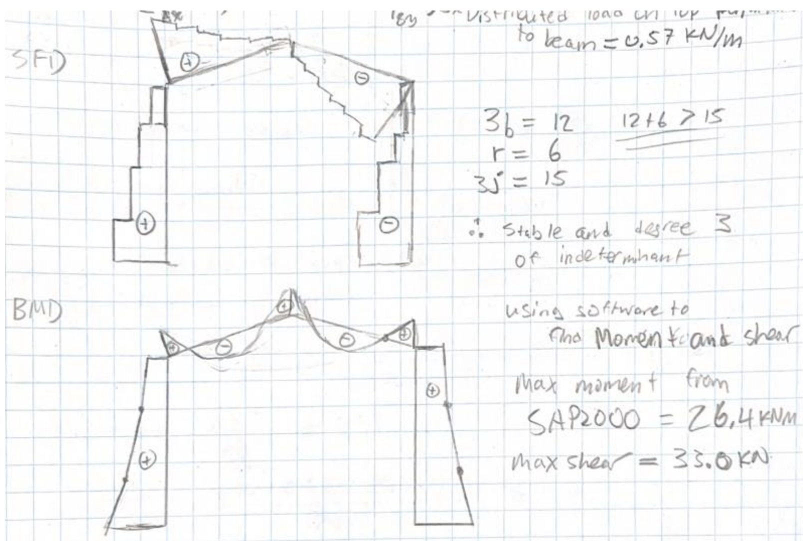


Figure 12: Shear and Bending Diagram for Middle Frame (See Volume III on pg. 57)

Figure 12 confirms the internal stability of the frame and its degree of indeterminacy, indicating a stable structure. To efficiently determine the maximum bending and shear, SAP2000 was utilized. The software revealed that the maximum factored shear and moment were 33.0 kN and 26.4 kNm, respectively.

The compressive force within the member was calculated by dividing the stress, as determined by SAP2000, by the cross-sectional area. This approach, along with combined loading calculations previously discussed in the Columns section and other necessary resistance calculations, ensured a thorough analysis. The detailed calculations are available on pages 57-60, with the section chosen being W200x52, consistent with the columns.

After the analysis of all structural members was completed, deflection calculations were performed. Maximum deflections, typically occurring at the midspan of each section, were determined using Figure 9. Adhering to the CSA S16-09: Design of Steel Structures by the Canadian Standards Association (2009), the maximum allowable deflections were referenced from Table D.1 on page 146. The detailed deflection calculations were documented on pages 85-87.

Uplift force was also calculated on page 61 but was not the governing case when factored. The uplift force came to 0.869kPa which was lower than the governing case of 1.25 dead load and 1.5 snow load.

A rough cost analysis was preformed in excel as shown in Appendix B Table B2. The lengths and weight of each member was taken and multiplied together to get the total weight. The total weight of the steel (not including the cladding) comes to 11,736kg. Cladding is needed on the top of the roof as well as the sides of the structure, coming to a total coverage area of 380.8m^2.

According to Scrap Metal Pricer (2024), the cost of a steel beam per kg is $3.87 in Canada. For the cladding the total cost $22 per square foot which comes to approximately $237 per square meter from Nortem (n.d.).

This brings the total cost to $135,675 for all steel including the members and cladding.

4.0 CONCRETE DESIGN

In the design of concrete structural components, the formulae used were sourced from 'Reinforced Concrete Design: A Practical Approach' (Brzev & Pao, 2009). Calculations were standardized on the premise that the concrete has a compressive strength of 25 MPa, and the steel reinforcement bars employed have a yield strength of 350 MPa.

According to 'Table 4.1.5.3' from 'Division B' of the National Building Code of Canada (NBCC) 2020 version, a live load factor of 3.6 kPa was applied to the concrete. Additionally, the self-weight of the concrete was considered by assuming a density of 24 kN/m³, and this was included in the computation of the factored moments and shear forces.

Once again, it's important to note that all references to hand calculations, equations, or pages mentioned here are found in Volume III unless stated otherwise.

4.1 Pump Pad

The concrete cover thickness required for the pump pad was specified as 40mm, as per Table 17 in 'CSA A23.3-04: Design of Concrete Structures' (Canadian Standards Association, 2004). The pad, whose dimensions are detailed as 4 meters by 4 meters in the 'Pump Layout Plan View' found in Volume II, was assumed to have a total thickness of 200 millimeters. This assumption, alongside the designated density, was used to calculate a total factored load of 13.2 kN/m, which includes the live load.

The calculation of the total factored bending moment, which resulted in 26.4 kNm, was performed using Equation 25 obtained from the 'Reinforced Concrete Design.' The selection of 15M rebar, spaced at 250mm intervals and temperature reinforcement (15M spaced at 450mm), was adequate for the moment resistance requirements, indicating a steel-controlled failure, as documented on page 66 of the referenced material.

In addition, to ensure the longevity of the structure, the crack control parameter ensured that the maximum of 30,000N/mm was not exceeded as per the CSA A23.3-04 Clause 10.6.1. The factored shear force, calculated at 26.4 kN, is recorded on page 70. The shear resistance was determined by the aggregate of concrete shear resistance only, as this slab does not incorporate stirrups or prestressed resistance, rendering the concrete resistance as the sole contributor to the shear resistance capacity of the member.

Figure 13 below shows the pad that will be supporting the pump.

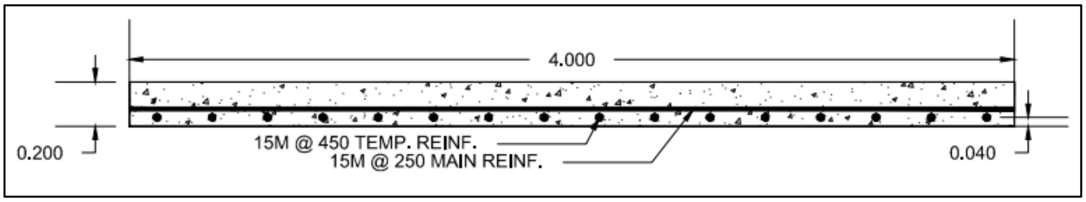


Figure 13: Concrete Pump Pad (See Volume II, 'Pump Pad Plan - Section A)

Figure 13 shows the dimensions of the concrete cover, height, and overall width of member in meters. Calculations for the pump pad are shown on pages 62-72.

4.2 Base Slab

The thickness of the concrete cover for the base slab, as per Table 17 in the CSA A23.3-04, is specified to be 75mm. For subsequent calculations, the slab's thickness was presumed to be 400mm. The dimensions of the section under analysis were defined as 1m in depth and 8m in length.

The slab was designed to bear several types of loads. These include the same live load that was applied to the pump pad, amounting to 3.6kPa. In addition to this, the self-weight of the concrete and the total weight of the structure, which, when spread over its entire surface area (16m x 8m), results in a load of 1.0kPa. Therefore, the total factored load for the concrete slab was calculated to be 18.7kN/m.

For the design parameters, the same ones used for the pump pad were applied. It was determined that the slab could sustain a maximum factored moment of 150kNm and a shear of 74.8kN, meeting the required resistance levels.

Regarding the control of concrete cracking, the cracking control parameter indicated that the concrete would avoid cracking as long as the stress did not exceed the maximum limit of 25,000N/mm. This criterion is in line with the CSA A23.2-04, Clause 10.6.1.

Figure 14 below shows the base slab with the necessary main reinforcement and temperature reinforcement with the dimensions showing in meters.

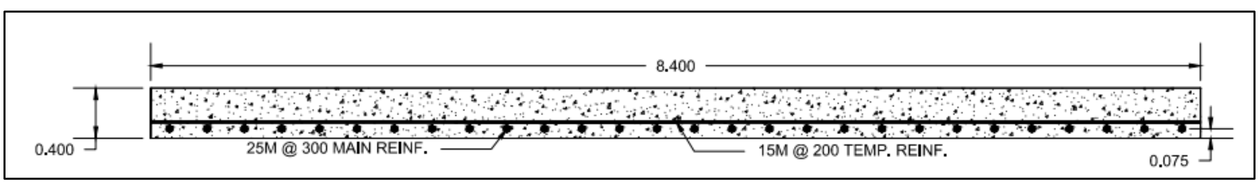


Figure 14: Concrete Base Slab (See Volume II, 'Base Slab Plan - Section C')

Figure 14 specifies the reinforcement details for the structure, with the main reinforcement designated as 25M bars, set apart at 300mm intervals. The temperature reinforcement is identified as 15M bars, spaced 200mm apart. The actual length of the member utilized in the design measures 8.4m. It's important to note that, for calculation purposes, a length of 8.0m was used. This minor difference is not expected to significantly impact the overall design. The relevant calculations are detailed across pages 71-77.

The immediate deflection calculated for the slab was found to be 27.1mm, as documented on pages 87-88, and this value was deemed sufficient. The calculation for long-term deflection was not performed because it necessitates the inclusion of compression reinforcement at the top of the slab. This aspect of the design was not addressed due to time constraints.

4.3 Strip Footing Foundation

The loads acting on the strip footing mirrored those applied to the base slab, albeit with a variation in the tributary width. The total live load was calculated to be 14.4kN/m, and the dead load amounted to 42.4kN/m. It was assumed that the maximum allowable bearing pressure for the foundation would be 100kPa. The dimensions of the foundation were presumed to be 1m in both length and width.

Two different scenarios were analyzed for the footings: one scenario for a footing located at the corner and another for a footing positioned at the edge of the slab. For both scenarios, the total concrete cover specified is 75mm, with an assumed footing height of 250mm.

The factored shear force was calculated to be 17.7kN, satisfying both the one-way shear and two-way shear (punching shear) requirements. Additionally, the factored moment was calculated to be 6kNm. The cracking control parameter was examined as well, ensuring that its value did not surpass the threshold of 25,000N/mm. These calculations and verifications are documented on pages 78-84. Figure 15 below shows the dimensions of the strip footing in meters.

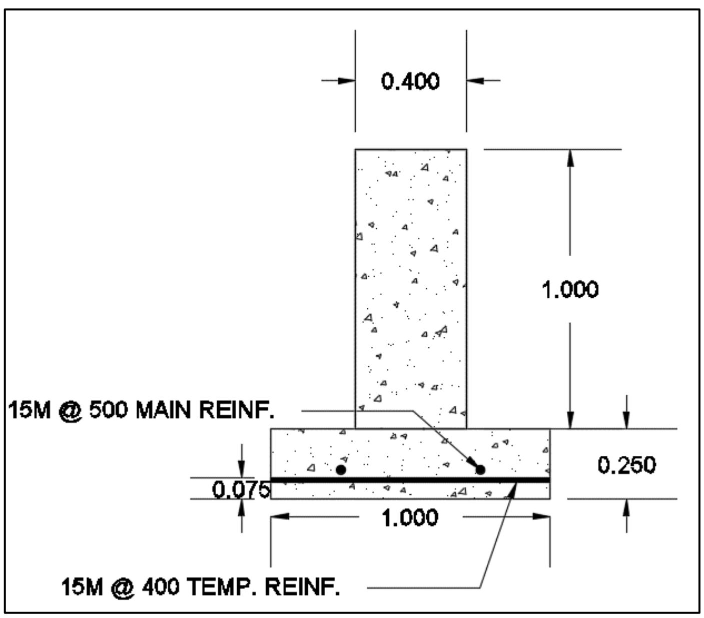


Figure 15: Strip Footing Foundation (See Volume II, 'Footing Plan - Section B')

Figure 15 details the reinforcement requirements for the strip footing, specifying 15M bars spaced 500mm apart for the main reinforcement. Additionally, for temperature reinforcement, 15M bars are to be spaced 400mm apart. The dimensions for the stump are given as 400mm both in width and length. Furthermore, the total depth beneath the base slab for the footing is stated to be 850mm.

No cost analysis was done for concrete as it is too varying and requires outside experience.

5.0 LOGISTICS PLANNING

A 219m long pathway, as depicted in Figure 1, is set for clearance. This pathway will have a width of 5m to ensure ample access and maintenance space for the pump station. Citing NHS Forest (n.d.), a hectare of forest typically houses between 1000 to 2500 trees. Using Google Earth Pro for visual inspection, the forest appeared to be on the less dense side, thus an estimate of 1200 trees per hectare was used for calculations.

Given the specified dimensions of the path, the area to be cleared was calculated to be 0.1095 hectares, which equates to roughly 130 trees needing removal. For the cost analysis, referenced from Koncewicz's 2024 guide on tree removal costs, pine trees with a height of 50ft were considered. The price range for removing pine trees of 5ft to 90ft in height was $300 to $3000. Through interpolation, the cost per tree for removal was estimated at $1600.

Additional prices included were permits, debris removal, stump removal, and remote location cost. The hand calculations can be seen on page 89. Table 2 below shows the breakdown for all of these costs.

Table 2: Cost Analysis of Clearing Path

Total Trees	130		Total
Tree removal	$1,600.00	/tree	$208,000.00
Permits	$ 100.00		$ 100.00
Debris removal	$ 75.00	/tree	$ 9,750.00
Stump removal	$ 350.00	/tree	$ 45,500.00
Remote location	$ 200.00		$ 200.00
		Total	$263,550.00

Table 2 details the costs assessed per tree, excluding permit and remote location expenses which do not vary with the tree count. The total expense for clearing a 5m wide and 219m long path was estimated at approximately $263,550.

6.0 RESULTS

The model of the structure was created in SAP2000 including all steel members, the concrete base slab, and the strip footing foundation. The pump pad was omitted from the model. SAP2000 is able to show the deflection of the model as well as the stresses within each member under various cases set by the user. This provides a visual aid as to what member may need to be resized.

The first case analyzed was 1.25D + 1.5S. Figure 16 below shows the deflection of this loading case in the gravity direction (Uz). The deflection is shown using a scaling of 10. This figure also shows the stress in each steel member.

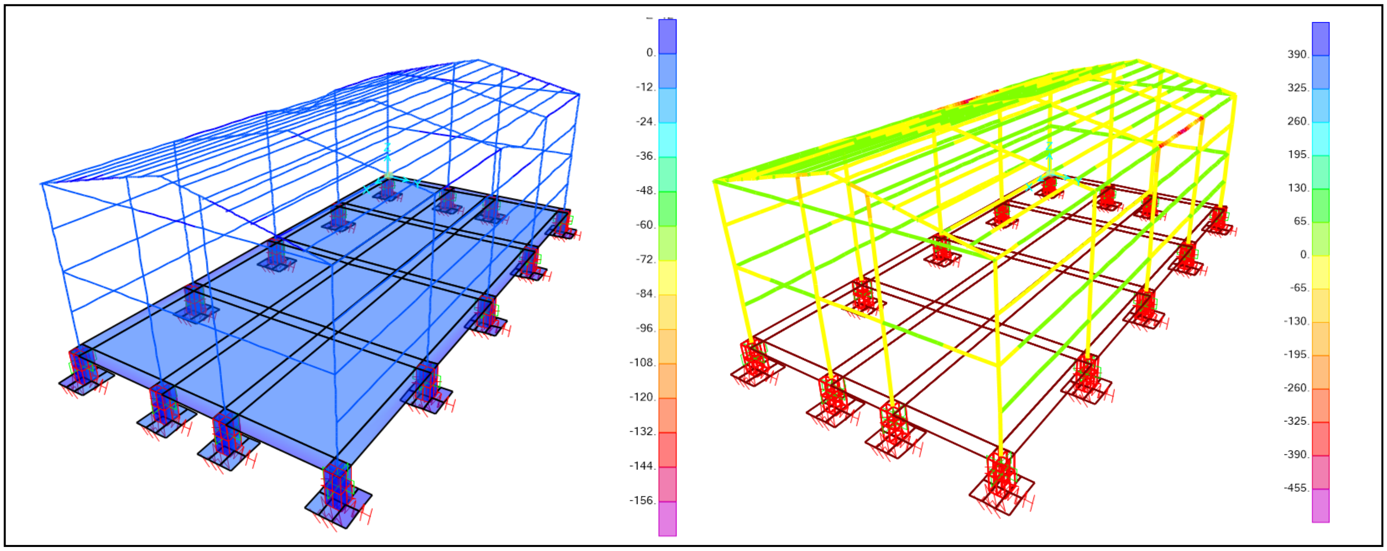


Figure 16: Deflection and Stress for 1.25D + 1.5S in SAP2000.

The left picture in Figure 16 shows the deflection. The colourful bar represents the deflection in millimetres. This shows that the model only deflects a maximum of 12mm and below, under this loading case.

The right picture of Figure 16 shows the stress of each steel member under this loading case. The colourful bar represents the stress in kPa. Since the worst case is the red colour on this model, showing approximately -325kPa which is -0.325MPa. This shows that the steel selected does not yield due to its yield strength of 350MPa.

The next case analyzed was 0.9D + 1.4W in the x and y direction. Figure 17 below shows the deflection from the wind loading in the x direction and the y direction. It shows the deflection using a scaling of 10.

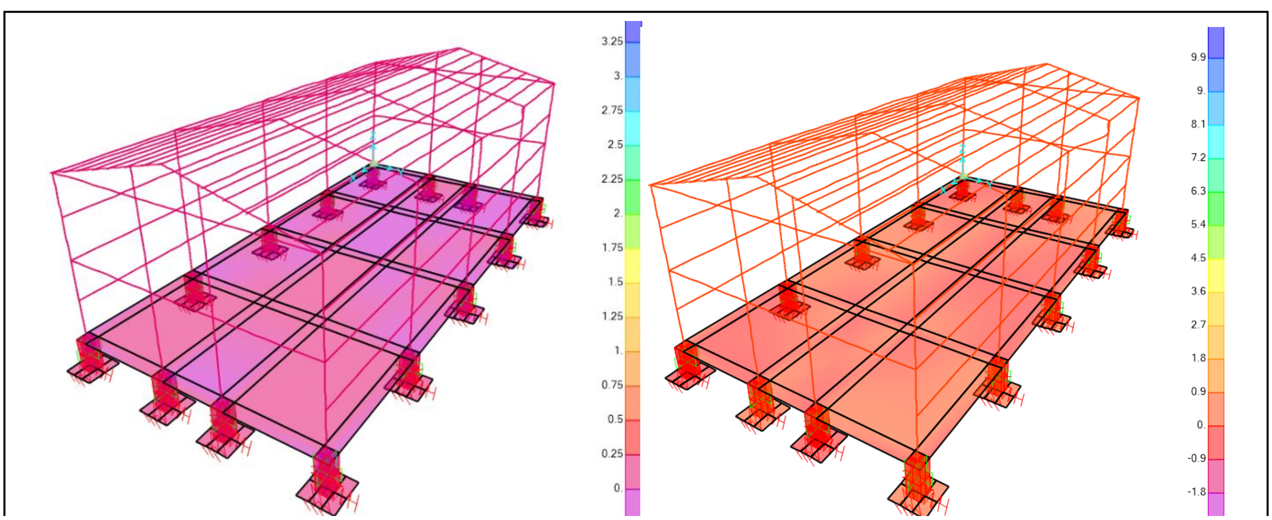


Figure 17: Deflection in the x and y Direction for 0.9D + 1.4W in SAP2000.

The left picture in Figure 17 shows the deflection in the x direction while the right picture shows the deflection in the y direction. Both of the coloured bars represent the deflection in millimeters. Both pictures show a deflection that is fairly close to zero which is favourable.

Figure 18 below shows the stresses in the steel members under the 0.9D + 1.4W case.

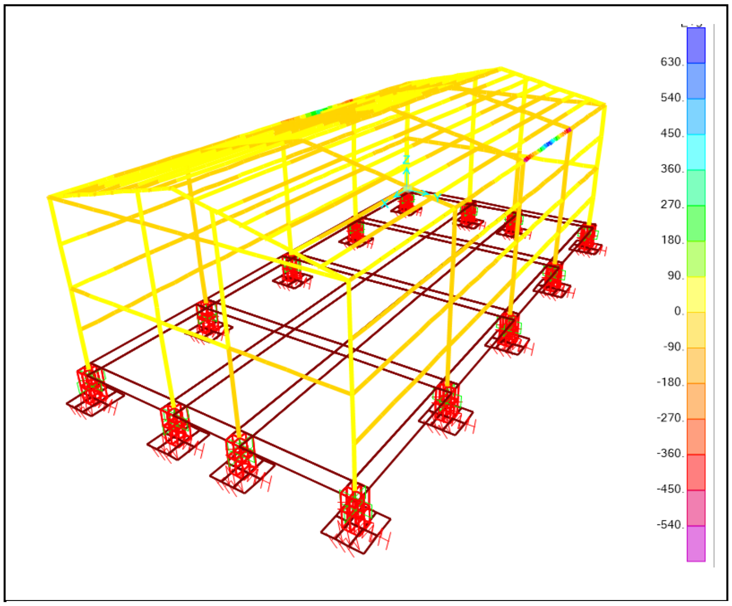


Figure 18: Stress in Steel Members for 0.9D + 1.4W in SAP2000.

Figure 18 shows the max stress (blue) to be roughly 630kPa or 0.630MPa. This again does not exceed the yield strength of the selected steel of 350MPa.

The next case analyzed was 1.25D + 1.5L for the concrete. The live load was only applied to the concrete while the dead load was applied to both the steel and the concrete. Figure 19 below shows the deflection in the gravity direction (Uz) using a scale of 10. The figure also shows the stress of the concrete from the live load.

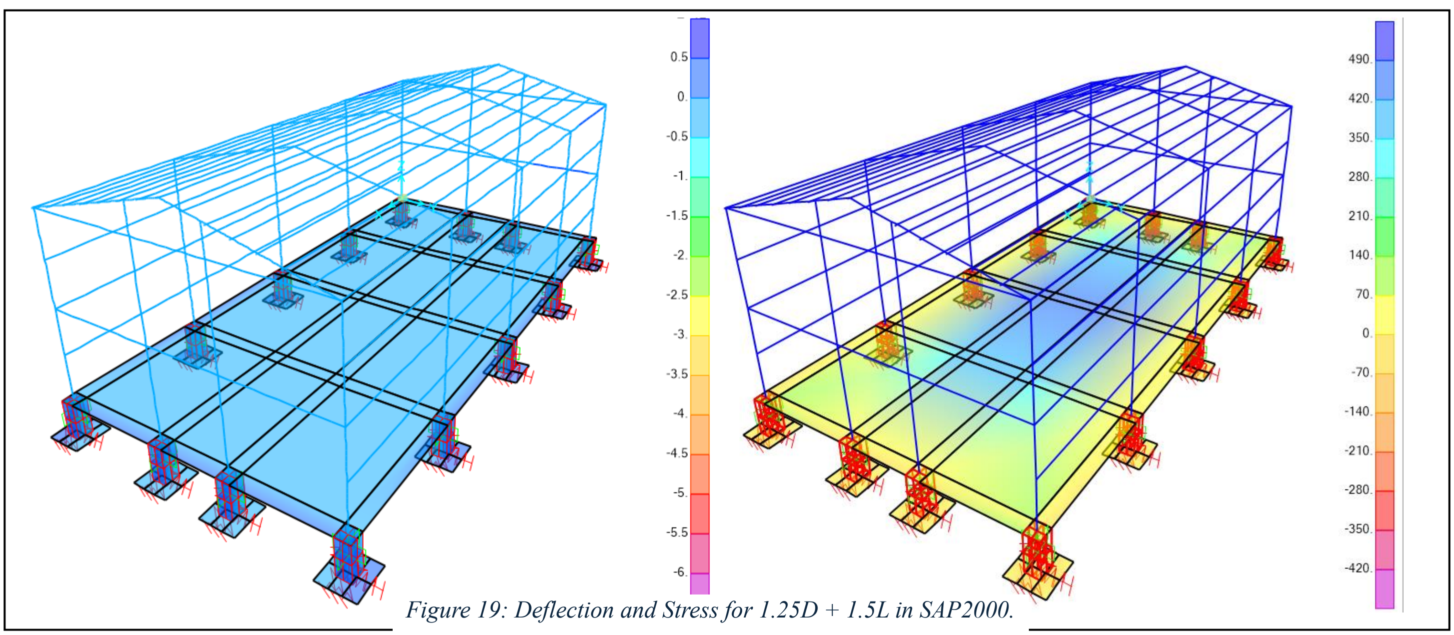


Figure 19: Deflection and Stress for 1.25D + 1.5L in SAP2000.

The left picture shows the maximum deflection to be around 0.5mm which is desirable. The right picture shows the stress on the concrete under this loading case. The maximum stress comes to approximately 450Pa which is relatively low.

For full drawings on the dimensions of the structure, please see Volume II.

7.0 CONCLUSION

The project entailed the design and implementation of a pump station and protective steel structure to facilitate water supply from Secret Lake, Okanagan Similkameen, BC, to a nearby community. Central to the design was a Hydro MPC-E 4 CRE 95-2 pump, chosen for its operational efficiency, capable of delivering a flow rate of 70L/s against a required head of 75.3 meters. The total cost for purchasing and maintaining the pump came to $845,225.57.

Structural integrity was ensured through the selection of steel members specific to each component's load-bearing requirements. The design utilized W200x52 sections for both the primary columns and middle frames and W200x42 for end wall rafters, subjected to analysis for gravity, lateral, and environmental loads. Roof support was provided by C200x17 purlins, designed to carry imposed loads including dead, live, and snow loads. Lateral stability for the structure was enhanced by C180x15 side and front girts, calculated for wind loading conditions. The total cost for all steel members including the 10mm thick cladding came to $135,675.

Concrete components were comprised of a pump pad (200mm thick), base slab (400mm thick), and strip footing foundation (250mm thick) designed with 25 MPa concrete. Reinforcement was provided by rebar (either 15M or 25M), spaced according to the structural demand imposed by operational and environmental conditions. The strip footing had a base of 1m by 1m with a total depth under the soil of 850mm. The design accounted for the dead load of the structure and the live load during operation, ensuring durability and stability.

Logistics planning addressed the challenge of accessing the remote site, necessitating the clearance of a 5m wide path spanning 219m long through forestry. The analysis determined the necessity for tree removal and the associated costs, contributing significantly to the project's logistical considerations. This would bring the total cost for clearing this path to $263,550.

In summary, the project's design was anchored in a detailed technical analysis of each component's structural and operational requirements but should be reviewed by a Professional Engineer as well as double checking the SAP2000 file to ensure everything was meshed properly together.

8.0 RECOMMENDATIONS

Further actions should be taken to ensure the entirety of the project is compliant with all codes. The first recommendation is to conduct a deeper investigation for the cost analysis for the pump station, steel members, and logistics planning. Another cost analysis should also be done for the concrete aspect of the project.

It is also recommended that the hand calculations for all steel members should be checked, specifically the middle frame capacity and any concrete sections.

As mentioned previously in section 4.2, compressive rebar shall be considered when designing the base slab to compute long-term deflection, to ensure necessary longevity of the structure. A Professional

Engineer should also check the rebar selection of 350MPa for the yield strength, as it is not commonly used compared to 400MPa.

An in-depth soil inspection should also be considered as the soil was assumed to have a bearing pressure of 100kPa which may be over assumed.

Finally, the structure in SAP2000 should be reviewed carefully as any members (steel and concrete) may not be meshed properly together. This could cause to show smaller (or larger) deflections than what was designed for. It is also recommended that the values used for the loads are to be reviewed and possibly increased to ensure safety of the structure.

REFERENCES

Beer, F. P., Johnston, E. R., DeWolf, J. T., & Mazurek, D. F. (2011). Statics and Mechanics of Materials. McGraw-Hill.

British Columbia Utilities Commission. (2024). Residential energy rates. Retrieved from https://www.bcuc.com/WhatWeDo/ResidentialEnergyRates

Brzev, S., & Pao, J. (2009). Reinforced concrete design: A Practical Approach (Updated ed.)

Canadian Institute of Steel Construction. (2017). Handbook of Steel Construction (11th ed., 3rd rev. printing). Canadian Institute of Steel Construction.

Canadian Standards Association. (2004). CSA A23.3-04: Design of Concrete Structures.

Canadian Standards Association. (2009). CSA S16-09: Design of Steel Structures. CSA Group.

City of Kelowna. (2020). Schedule 4: Design standards of Bylaw 7900. Retrieved from https://apps.kelowna.ca/CityPage/Docs/PDFs/Bylaws/Subdivision,%20Development%20and%20Servicing%20Bylaw%20No.%207900/Schedule%204%20-%20Design%20Standards.pdf

Google. (2023-a). [Secret Lake, Okanagan, BC]. Retrieved October 10, 2023, from https://earth.google.com/web/@49.70622502,-120.24239705,1120.41540031a,1465.15247434d,35y,5.26210701h,26.71519068t,0r/data=OgMKATA

Google. (2023-b). [Secret Lake, Okanagan, BC]. Retrieved December 21, 2023, from https://earth.google.com/web/@49.70622502,-120.24239705,1120.41540031a,1465.15247434d,35y,5.26210701h,26.71519068t,0r/data=OgMKATA

Grainger. (2024). ANVIL Pipe Saddle: For 1 1/2 in – 10 in Pipe, 3,800 lb Load Capacity. Retrieved from https://www.grainger.com/product/ANVIL-Pipe-Saddle-For-1-1-2-in-10-5LWT7

Grundfos. (n.d.). Hydro MPC, North America. Retrieved from https://product-selection.grundfos.com/us/products/hydro-mpc-north-america?tab=models

Koncewicz, S. (2024). How much does tree removal cost? (2024 Guide). This Old House. Retrieved from https://www.thisoldhouse.com/gardening/reviews/average-cost-of-tree-removal

National Research Council of Canada. (2020). National Building Code of Canada 2020. Retrieved from https://nrc-publications.canada.ca/eng/view/object/?id=515340b5-f4e0-4798-be69-692e4ec423e8

Nayyer, M.L. (2000). Piping handbook (7th ed.) Retrieved from https://engineeringdocu.wordpress.com/wp-content/uploads/2011/12/piping-handbook.pdf

NHS Forest. (n.d.). How many trees per hectare? Retrieved from https://nhsforest.org/how-many-trees-can-be-planted-hectare/

Nortem. (n.d.). How much do different types of metal cladding cost? Retrieved from https://nortem.ca/how-much-do-different-types-of-metal-cladding-cost/

Pump Stop, LLC. (2024). Grundfos 99689147 - Hydro MPC-E 4 CRE 95-2. Retrieved from https://pump-stop.com/products/hydro-mpc-e-4-cre-95-2-99689147?_pos=1&_sid=59bc5d183&_ss=r

Romtec. (2021). *Booster Pump Stations*. Romtec Utilities. Retrieved from https://romtecutilities.com/booster-pump-stations2/

Scrap Metal Pricer. (2024). Retrieved from https://www.scrapmetalpricer.ca/

APPENDIX A: AUTOCAD DRAWING OF PIPE

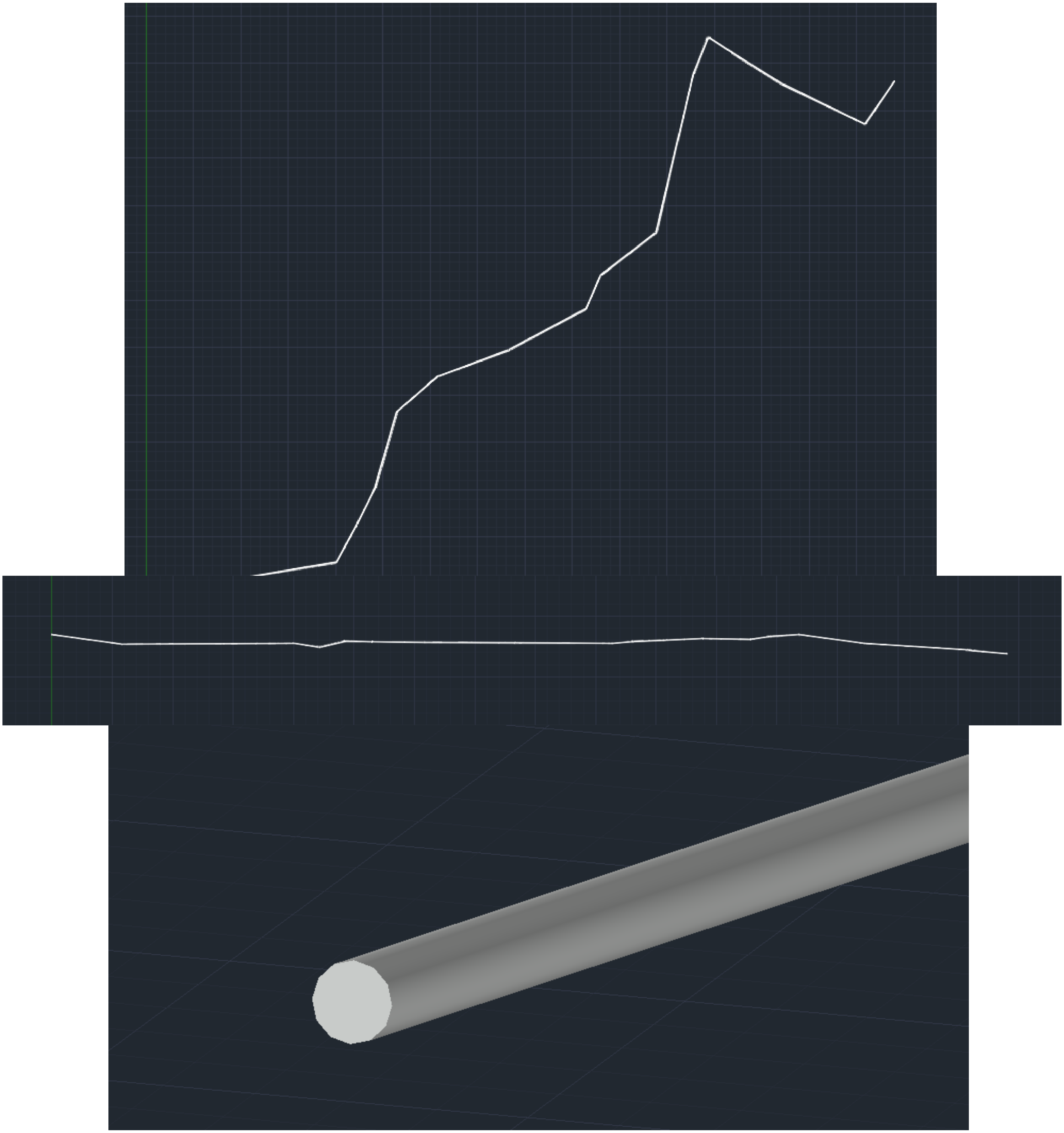


Figure A1: Plan View, Profile View, and Closeup View of 200mm Pipe in AutoCAD.

APPENDIX B: COST ANALYSES

Table B1: Cost Analysis in Excel of Purchasing and Maintaining Pump.

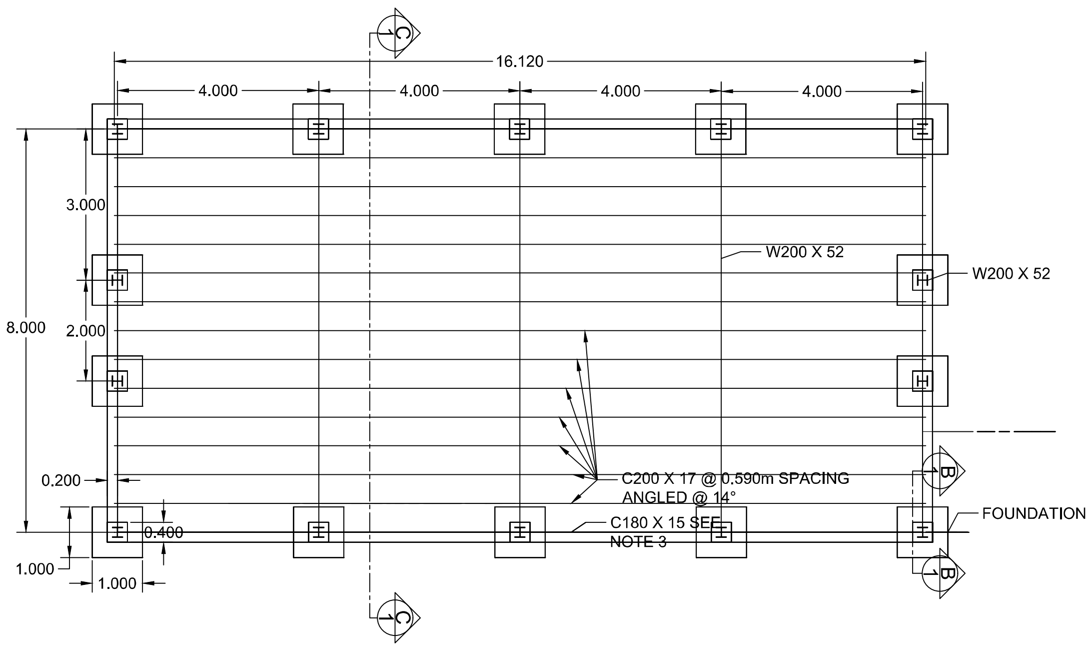
Pumps											
Hydro MPC-E 4 CRE 95-2			Hydro MPC-E 5 CRE 45-3			Hydro MPC-E 6 CRE 45-3			Hydro MPC-E 5 CRE 64-3-2		
Pump Cost	$ 115,066.16		Pump Cost	$ 112,277.44		Pump Cost	$ 157,233.12		Pump Cost	$ 142,786.16	
Power Consumption	75.99	kW	Power Consumption	77.93	kW	Power Consumption	75.03	kW	Power Consumption	78.45	kW
Power Cost	$ 0.0975	/kWh	Power Cost	$ 0.0975	/kWh	Power Cost	$ 0.0975	/kWh	Power Cost	$ 0.0975	/kWh
Assuming pump will run for 45 years 6 hours a day											
Total Cost	$ 845,225.57		Total Cost	$ 861,077.59		Total Cost	$ 878,168.25		Total Cost	$ 896,582.79	
				Best pump option is the Hydro MPC-E 4 CRE 95-2 at $845,225.57							

Table B2: Cost Analysis in Excel of All Steel Members.

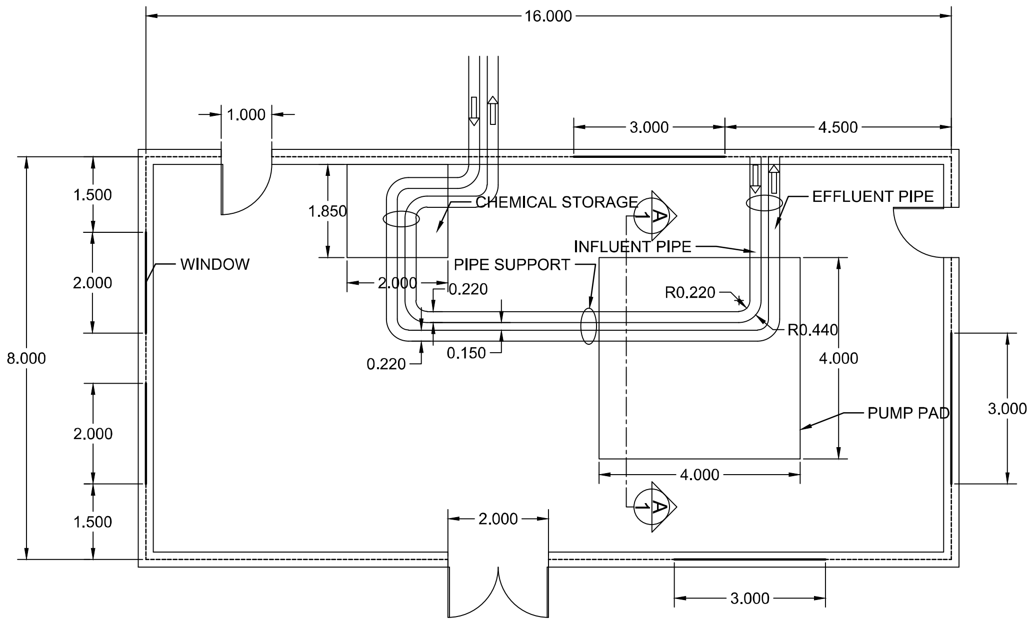
Steel members	3.87	$/kg	Cladding	237	$/m^2
Item	Type	Length (m)	Amount	Total Weight (kg)	Cost
Purlins	C200x17	16.12	13	3563	$ 13,786.95
Endwall Rafters	W200x42	4.12	4	692	$ 2,678.66
Short Colums	W200x52	5.00	4	1040	$ 4,024.80
Long Columns	W200x52	5.75	4	1196	$ 4,628.52
Side Girts	C180x15	16.00	8	1920	$ 7,430.40
Front Girts	C180x15	8.00	4	480	$ 1,857.60
Middle Frames	W200x52	18.24	3	2845	$ 11,011.85
				11736	
		Area			
Cladding	10mm thick	380.8	m^2	1023	$ 90,256.43
				TOTAL	$ 135,675.21

VOLUME II – DESIGN DRAWINGS

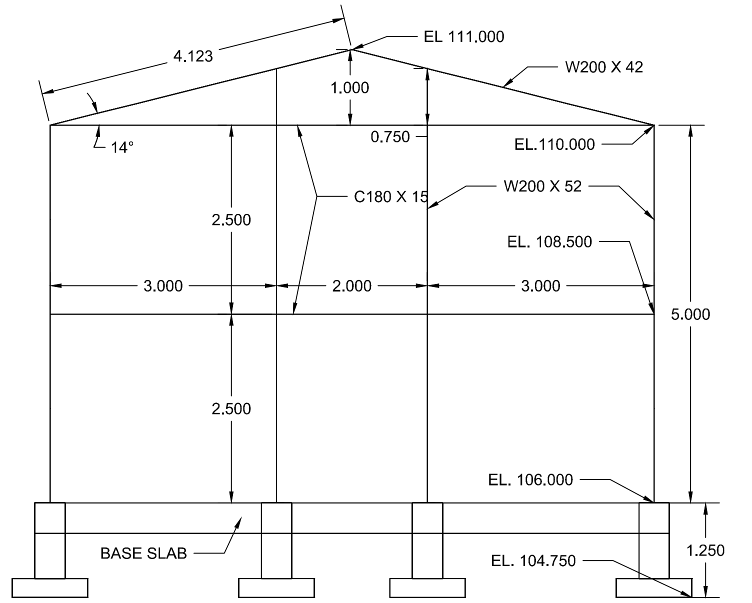
N



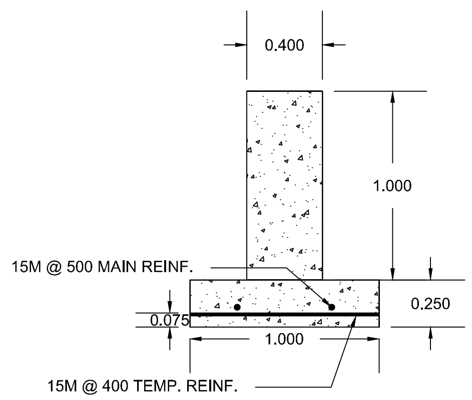
STRUCTURAL PLAN VIEW
SCALE 1:75



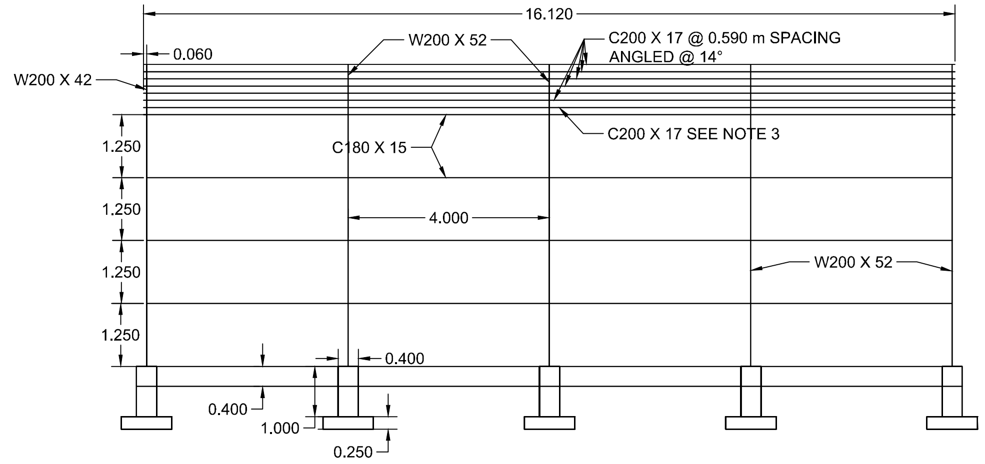
PUMP LAYOUT PLAN VIEW
SCALE 1:75



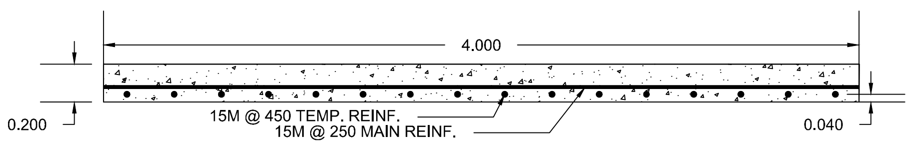
FRONT ELEVATION VIEW
SCALE 1:50



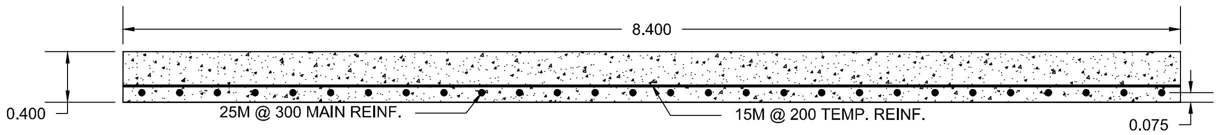
FOOTING PLAN – SECTION B/1
SCALE 1:20



SIDE ELEVATION VIEW
SCALE 1:75



PUMP PAD PLAN – SECTION A/1
SCALE 1:20



BASE SLAB PLAN – SECTION C/1
SCALE 1:30

NOTES

1. ALL DIMENSIONS ARE IN METERS.
2. ALL SPECIFIED CONCRETE IS TO HAVE A COMPRESSIVE STRENGTH OF 25MPa WITH 20mm AGGREGATE SIZE
3. THE TOP SIDE GIRT (EAVE STRUT) AND THE FIRST PURLIN ON THE ROOF IS TO BE SPACED AT 0.580m. THE REMAINING PURLINS ARE TO BE SPACED AT 0.590m WITH THE ROOF BEING ANGLED AT 14 DEGREES.
4. ALL STEEL INCLUDING REBAR IS TO HAVE A SPECIFIED STRENGTH OF 350MPa.
5. ALL DRAWING ARE POINTING IN THE DIRECTION OF THE NORTH ARROW EXCEPT FOR FRONT ELEVATION VIEW AND ANY SECTION CUTS.

By:	Benjamin Mendoza	
	N/A	
No.	Revision/Issue	Date

Firm Name and Address
[No Company]
123 Street, Burnaby
British Columbia

Project Name and Address
Pump Station Layout
Okanagan, BC

Project Number
24-29
Date
Mar. 27, 2024
Scale
VARIES
Sheet
1