Design of a Pipeline Bridge in Northern BC

Report No. CECDP – 2015/04

April 2015

Civil Engineering Capstone
Design Project

Moja Inženjer Ltd.
Myles Cape
Amos Kim
Omnirey Lacson

Department of Civil Engineering

School and Construction and the Environment
British Columbia Institute of Technology
Burnaby, BC, Canada, V5G 3H2

Design of a Pipeline Bridge in Northern BC

Report No. CECDP – 2015/04

April 2015

Design Report

Moja Inženjer Ltd.

Myles Cape
Amos Kim
Omnirey Lacson

Department of Civil Engineering
School of Construction and the Environment
British Columbia Institute of Technology
Burnaby, BC, Canada, V5G 3H2

March 16, 2015

Capstone Faculty Committee
Department of Civil Engineering
British Columbia Institute of Technology
3700 Willingdon Avenue
Burnaby, BC V5G 3H2

Dear Capstone Faculty Committee,

Submission of the Final Report on the Design of a Pipeline Bridge in Northern BC

Team Moja is pleased to provide you the Final Report on the Design of a Pipeline Bridge in Northern BC. This project was to design a 70 m aerial pipeline crossing over a creek within an active floodplain in Northern BC. The work consisted of the design of the superstructure and substructure to support the pipeline, a recommended construction plan, and an erosion protection plan.

The final design is a modified Warren truss composed of a 40 m and 30 m span. The truss is made up of HSS rectangular and square sections with welded connections. Three concrete piers support the bridge superstructure with driven piles. The substructure is protected by launching riprap from the migrating watercourse. BMPs were implemented for the recommended construction plan to ensure the effects of work activities are minimized in the protection of fish and wildlife habitats.

We hereby declare that the work represented in this submission is original and entirely completed by Team Moja. We would like to thank all of those who have helped us along our journey, including David Bajic of Allnorth Consultants for his guidance. We would also like to thank the Capstone Faculty Committee for their help and advice. If there are any questions regarding this project, feel free to contact us at mac5_8@hotmail.com.

Sincerely,
Team Moja

Myles Cape Amos Kim Omnirey Lacson

cc: David Bajic

Enclosed: Final Report

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 students' 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.

ACKNOWLEDGEMENTS

Team Moja would like to thank the following people who helped us throughout the completion of this project:

David Bajic, who acted as our sponsor and made this project possible. He has guided us throughout the term and promptly answered all of our questions regarding the project.

Allnorth Consultants, who provided a challenging project to us and the necessary information to complete the project.

Martin Bollo, who provided guidance on team management and aided us following the loss of a team member.

The Capstone Faculty Committee, who provided regular feedback and addressed our questions and concerns for the project.

EXECUTIVE SUMMARY

Pipeline crossings are common in British Columbia and are required along pipeline routes when a pipeline intersects with a watercourse. These watercourse crossings are often environmentally sensitive and regulatory requirements need to be met for approval of construction.

The pipeline bridge is to be located near Hazelton, BC. The aerial pipeline crossing supports a natural gas pipeline with a diameter of 1219 mm and a thickness of 26 mm. In addition, the pipe elevation was pre-defined and no other construction techniques were considered outside of a self-supporting clear span bridge over the watercourse. At the project site, a third party carried out a preliminary hydrotechnical and geotechnical assessment. This project was provided by Allnorth Consultants.

In the design of the superstructure, trusses were selected as the most suitable option amongst the possible bridge systems. These bridge systems were evaluated at the conceptual stage. CSA S6-06 and CSA S16-09 were the codes used in the design of the superstructure.

At the preliminary design phase, design loads were calculated to determine the governing load combination and comprised of dead snow. An analysis of alternatives was performed between Warren and Pratt configurations and Excel spreadsheets were used to determine the optimal panel widths, depths and to initially size the preliminary members. It was found the Warren provided the most economical solution while meeting the deflection requirements specified by the pipeline designer.

The truss was comprised of square and rectangular HSS sections to efficiently resist the factored loads. Connection resistances were calculated for HSS members using methods outlined by CIDECT and IIW. Gap K connections were typical in the design of the truss, and it was determined that these connections resisted the factored loads. As well, spliced connection and welds were designed using CSA S16. Lateral members were designed using computer modelling software to resist lateral torsional buckling and to stabilize the structure under ultimate loading conditions. Bearings were designed to transfer the loads from the superstructure to the substructure. The bearing's required thickness and deflection limits were designed according to CSA S6-06.

To support the bridge superstructure, three piers were designed within the 70 m span. Two pier options were assessed for design. During the conceptual phase, design loads were determined for these piers. Two pier options were analysed and sized for flexural stress during the preliminary phase. Using column interaction diagrams, approximate component sizes and flexural reinforcement requirements were determined. In the detailed design phase pile caps were designed. In addition, additional checks per CSA S6-06 and CSA A23.3 were conducted in the detailed design phase to finalize the pier design.

Recommended pile specifications were provided by others and Team Moja was tasked to confirm the viability of this design. During the conceptual design phase, the compressive bearing capacity of the pile was confirmed, and the provided bearing capacity was assumed in the design. During the detailed design phase, downdrag, pile settlement, lateral deflection, and frost heave effects were proposed to be checked by Team Moja. Of these, checks for downdrag and frost heave effects were unfinished due to time constraints.

Scour is a major concern at the project site. Team Moja considered various methods for protecting the embankments and the substructure from soil erosion. Ultimately, launching riprap was selected and designed according to the *TAC Guide to Bridge Hydraulics*. As the creek is environmentally sensitive, Team Moja recommends to follow the regulatory requirements outlined in the *Water Act*. In addition, BMPs should be implemented to minimize adverse effects to fish and wildlife habitats such as using regional timing windows for work activities.

A construction plan that recommends work activities and sequencing for the construction the pipeline bridge was created Team Moja. In this plan, excavation extents, the proposed stockpile location, sedimentation plan, a lifting plan, and the construction sequence were outlined. It was determined that additional excavation was needed to reduce the required riprap volume. Excavation volumes and extents were determined by modelling in Civil 3D.These areas are aimed to reduce cost to the client by providing a feasible design with the critical items already planned.

A profile view of the final design is illustrated below.

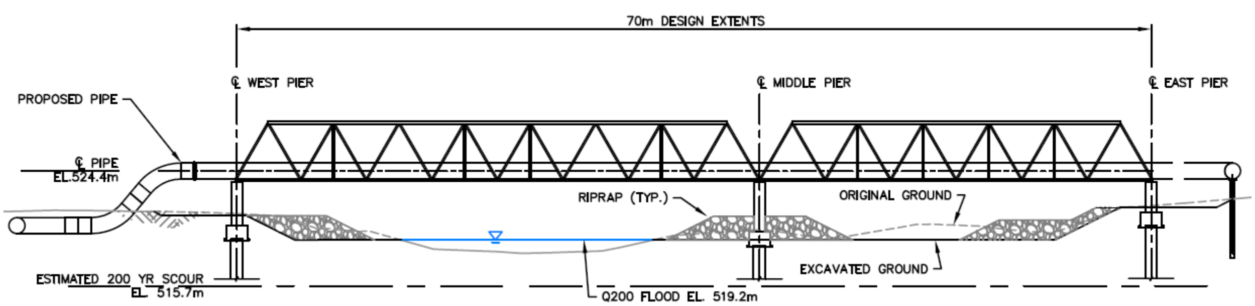


TABLE OF CONTENTS

LIST OF APPENDICES

LIST OF FIGURES AND TABLES

List of Figures

List of Tables

1.0 INTRODUCTION

This report outlines the design of a 70 m aerial pipeline crossing in Northern British Columbia. The designed structure is to support a 1219 mm diameter natural gas pipeline over a potentially migrating creek within an active floodplain. The project is located near Hazelton, British Columbia, however, its exact location is undisclosed due to project confidentiality. This project incorporates the design of the bridge superstructure, the bridge substructure, an erosion protection plan, and a recommended construction plan.

Moja Inženjer Ltd. (Team Moja) was tasked to design the pipeline bridge by the project sponsor and the Capstone Faculty Committee of the Civil Engineering Department at BCIT. This report serves as our submission as a part of CIVL 7090: Capstone Design Project course.

This project was sponsored by Allnorth Consultants, a multidisciplinary engineering and technical services consulting company with focus in the mining, oil and gas, pulp and paper, infrastructure, chemical, and power sectors. The company representative who served as our industry contact was David Bajic, P.Eng., a senior structural engineer at the Allnorth Prince George Office. This Capstone project was provided as an academic exercise while this was an ongoing project by Allnorth Consultants.

Pipeline crossings are common in British Columbia and are required along pipeline routes when a pipeline intersects with a watercourse. These watercourse crossings are of particular importance due to environmental sensitivities and regulatory requirements for the approval and construction of these projects. One of Team Moja's main objectives was to produce a sustainable solution for our design problem by minimizing the construction effects near the watercourse and ensuring no deleterious materials are deposited into the watercourse. By achieving these objectives, we aim to reduce the effects on the natural environment.

This report covers the background, superstructure, substructure, environmental, and construction design sections.

2.0 BACKGROUND

This project was to design a 70 m aerial crossing over an active floodplain near Hazelton in Northern British Columbia. Figure 1 shows the approximate location of the project site.

Figure 1. Location of Hazelton, BC (Wikipedia, 2015)

The aerial pipeline crossing supports a specified natural gas pipeline with a diameter of 1219 mm and a thickness of 26 mm. In addition, the pipe elevation was pre-defined and no other construction techniques were considered outside of a self-supporting clear span bridge over the watercourse. At the project site, a third party carried out a preliminary hydrotechnical and geotechnical assessment and is shown in Appendix A. The site description, design objectives, scope revisions, and design criteria will further be discussed in the following sections with information provided by Allnorth in Appendix A.

2.1 Site Description

The floodplain is confined by moderate to steep gradient slopes on either side of the crossing location and is described to have two hazard zones, the zone of active influence and the zone of river influence. The zone of active influence is 15 m wide at the crossing location and represents the possible extents where the creek is likely migrate based on topographic data and site observations. Figure 2 shows the upstream view of the watercourse at the proposed aerial crossing.

Figure 2. Upstream View of the Proposed Aerial Crossing (Allnorth, 2015)

The zone of river influence represents the entire width of the floodplain and is approximately 175 m wide. It is assumed that the creek can occupy any position within the floodplain in the future due to hydrological conditions in the area. However, the proposed aerial crossing does not span the entire width of the floodplain, but spans over a distance that is considered to be most likely occupied by the creek in the pipeline design life of 50 years. Figure 3 shows the project site general arrangement with the 70 m span within the floodplain.

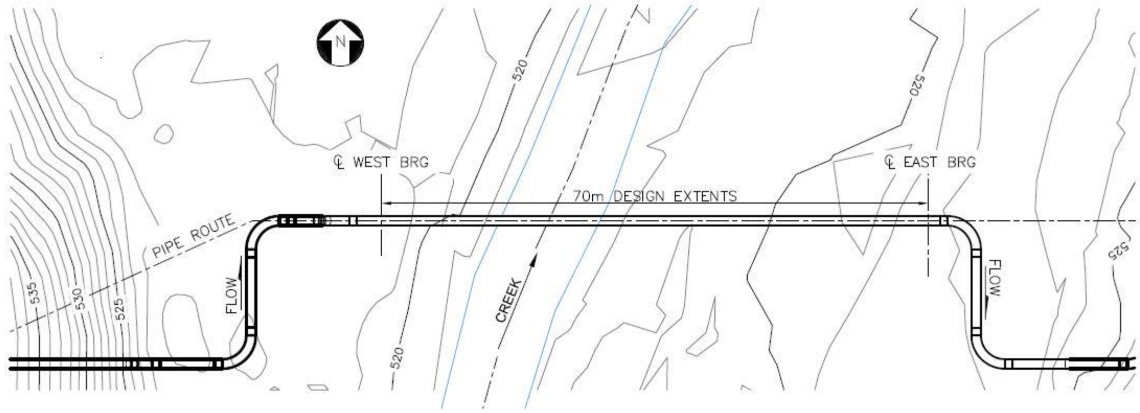


Figure 3. Project Site General Arrangement (Allnorth, 2015)

As seen in the figure above, the creek flows in the south-north direction along the floodplain, and the pipeline flows in the west-east direction. Downstream of the proposed aerial crossing is an existing forestry service road that is assumed to provide adequate access to the project site.

A total of five hand-dug test pits were excavated to characterize the site surficial geology to a depth of 0.3 m. Three tests indicated the overburden soil comprised of fluvial sand and gravel with traces of clay. The remaining two tests encountered till comprising of gravelly silt. Bedrock outcrops were observed near the area, however, the depth of the bedrock stratum is uncertain at the proposed aerial crossing.

2.2 Design Objectives

The objectives of this project are to

- design the bridge superstructure
- design the bridge substructure
- create a recommended construction plan
- create an erosion protection plan
- apply sustainable practices to the above objectives.

However, some design components will not be included in this report as they were provided by Allnorth Consultants. These include the

- assessment of the hydrotechnical and geotechnical conditions of the site
- design of the pipeline itself, where its geometry and material properties were specified.

2.3 Scope Revisions

On January 9, 2015, Team Moja was informed that one of our team members has withdrawn from the project, resulting in a group consisting of the three remaining members. This has caused a reduction in scope to our original proposal that was submitted to the Capstone Faculty Committee on November 12, 2014.

The scope was revised through the following actions by

- removing detailed seismic design of the bridge structure
- reducing analysis of alternatives from three preliminary designs to two preliminary designs
- removing the detailed consideration of alternatives for foundation design
- reducing connection design to the design of only major connections.

The above scope revisions have been discussed with the Capstone Faculty Committee and Team Moja completed the project based on these revisions.

2.4 Design Criteria

In our design of the pipeline bridge, our main goal was to comply with criteria outlined by the *Canadian Standards Association* and the *Ministry of Transportation and Infrastructure* while meeting the requirements of regulatory bodies associated with watercourse crossings. Team Moja designed the bridge according to additional criteria provided by the sponsor, including the

- design life is 50 years and the service life is 25 years
- structure must be hydro-tested before going into service
- bridge must have an access along its length for maintenance workers
- construction loading must include the allowance of 20 workers plus tool carts.

Furthermore, Team Moja designed the bridge according to the following criteria:

- economic sustainability
- durability
- environmental sustainability
- constructability

2.4.1 Economic Sustainability

Economic sustainability has many aspects, such as upfront cost of permanent materials, maintenance costs, and potential long-term benefits of the structure. Resources must be used in such a way to achieve profit. This is a key criterion in project evaluation as capital is most often the driving force behind projects.

2.4.2 Durability

According to *CSA S6-06 Canadian Highway Bridge Design Code*, durability refers to the long-term capability of a structure to perform its function throughout its life. Maintaining the structural integrity of the pipeline bridge by ensuring the bridge's maximum deflection remains within its prescribed limits throughout its service life was important for the design of this project.

2.4.3 Environmental Sustainability

The effects of the structure on its surrounding environment also presents design constraints. In particular, the creek within the bridge crossing is fish-bearing, so there were additional environmental considerations for this project. In accordance with provisions from the *Water Act* and the *Fish Protection Act*, Team Moja designed the bridge such that the creek's velocity, shape and natural sedimentation will not significantly change due to bridge construction and due to the bridge itself. This requires that all construction works are outside the 200-year flow (Q200), and that, preferably, no piers are located within the 70 m extents. In addition, care must be taken to ensure that no sediments due to construction fall within the creek.

2.4.4 Constructability

Constructability refers to the ease of construction of the bridge structure. Construction time, labour, and cost comprises constructability. The amount of formwork, construction equipment, delivery of materials and falsework are examples of factors that affect this criterion. As construction and material costs are directly related to economic sustainability, this was another key design criterion for this project.

3.0 SUPERSTRUCTURE DESIGN

The design of the superstructure was completed in accordance with CSA S6-06 and *CSA S16-09 Design of Steel Structures*. The sections below summarize the design phases of the bridge superstructure, the consideration of alternatives for different bridge systems, and the detailed design of the chosen alternative. See Appendix L for the final design drawing package including the superstructure design

3.1 Conceptual Design

For several structural systems, the design requirements and their applicability for the 70 m span were determined. Various factors such as span, material, placement of the deck, and the configuration of the bridge were considered at this stage. Bridge configurations were chosen and evaluated based on the their relative constructability, environmental impact, and cost. This evaluation process and an expansion on each structural configuration is outlined below.

3.1.1 Cable-stayed Bridge

In a cable-stayed bridge, pylons are erected from which cables stretch down diagonally to support the deck. Team Moja considered a layout with a single pylon at mid-span. This layout is illustrated in Figure 4.

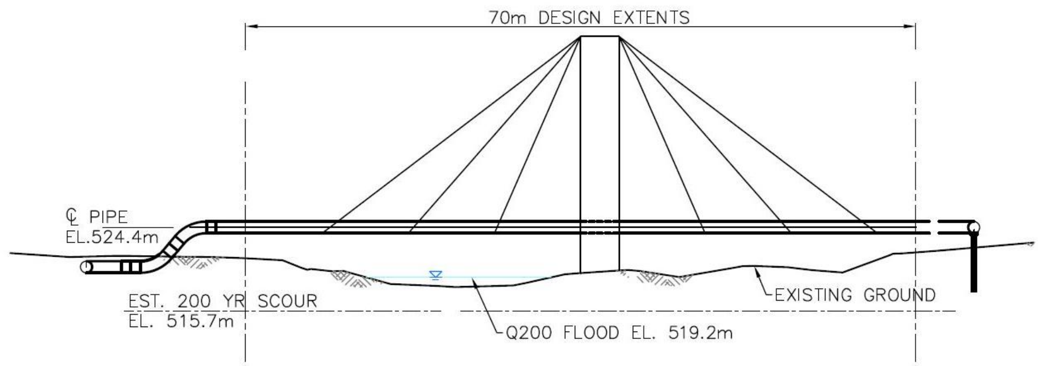


Figure 4. Cable-Stayed Concept 1

The cables are in a fan design, where all the cables connect to the top of the pylon. A second layout for a cable-stayed bridge was also considered with two pylons at the ends of the 70 m span with tiebacks anchoring each end. This layout is shown in Figure 5.

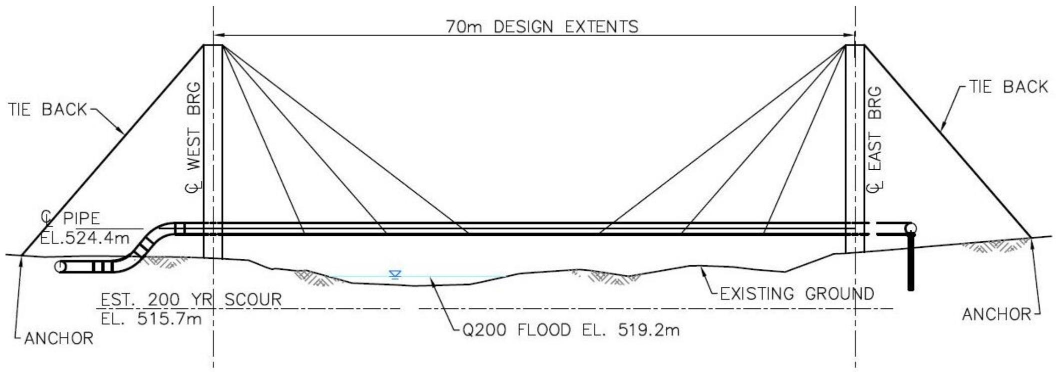


Figure 5. Cable-Stayed Concept 2

The load on the deck is transferred from the cables, which are in tension to the pylons, putting the pylons in compression. A cable-stayed bridge is seen in medium to long spans and are typically more expensive than other bridge types. This is due to the fabrication of cables being complex and expensive. From these reasons and literature review of bridges of similar spans, Team Moja deemed cable-stayed bridges to not be the optimum system.

3.1.2 Suspension Bridge

Like cable-stayed bridges, suspensions bridges use cables. However, the deck of a suspension bridge is supported by vertical suspenders where the main cables are suspended between pylons. Team Moja has considered a layout where there are two pylons at the ends of the span with tiebacks anchoring each end and is illustrated in Figure 6.

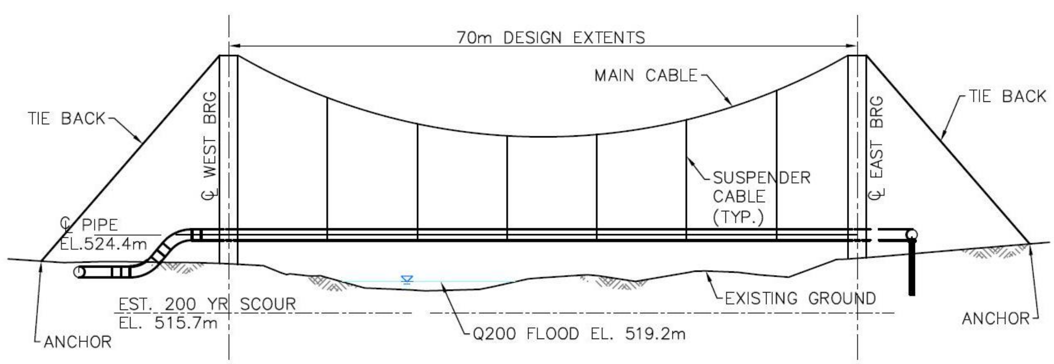


Figure 6. Suspension Concept

Anchors involve the use of bedrock or buried concrete blocks to achieve the required tensile resistance in the tiebacks. For this reason, anchors add additional cost and require solid material or competent bedrock. Long spans can be obtained with a suspension bridge; however, these bridges have relatively low deck stiffness. This lower stiffness is not ideal for a pipeline bridge with strict deflection requirements. For these reasons, Team Moja deemed suspension bridges not feasible.

3.1.3 Truss Bridge

Truss bridges are structures with connected members forming triangular frames. These connected members act either in compression or tension under load. Out of the many configurations of truss bridges available, we considered Warren and Pratt trusses due to their wide use. The Warren configuration is shown in Figure 7.

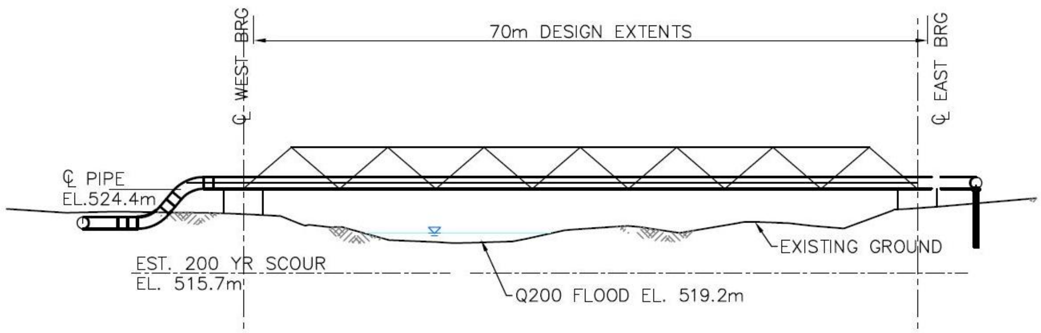


Figure 7. Warren Truss Concept

The Warren truss forms either equilateral or isosceles triangles and the diagonal web members alternate between tension and compression. Pratt trusses, on the other hand, have both vertical and diagonal members. The diagonals are angled towards the centre of the span. Figure 8 shows a Pratt configuration.

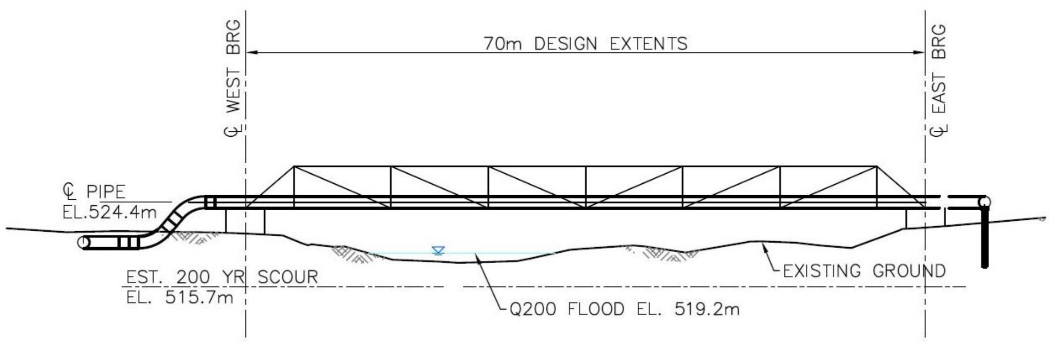


Figure 8. Pratt Truss Concept

The diagonals are in tension while the vertical members are in compression. Truss bridges make efficient use of their material; however, they require a high level of maintenance.

3.1.4 Beam Bridge

Beam bridges are the simplest bridge type consisting of horizontal members supported by piers. We have considered both steel and concrete beam bridges. Figure 9 shows a steel girder bridge.

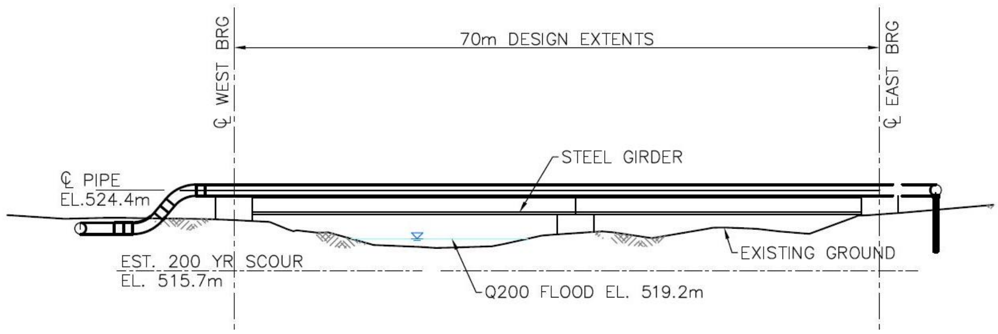


Figure 9. Steel Girder Concept

This concept involves wide flange steel girders supporting the deck and pipeline and is relatively easy to construct. These types of bridges require additional piers for larger spans and are heavier structures compared to non-beam bridge structures. Due to the increased material cost and requirement for additional piers Team Moja deemed this system not feasible.

Continuous concrete bridges were also considered. Like the steel girder concept, continuous concrete bridges require additional piers for longer spans. This structure would require formwork and falsework unless precast sections are used which would increase costs and the complexity of construction. Figure 10 shows the continuous concrete bridge concept.

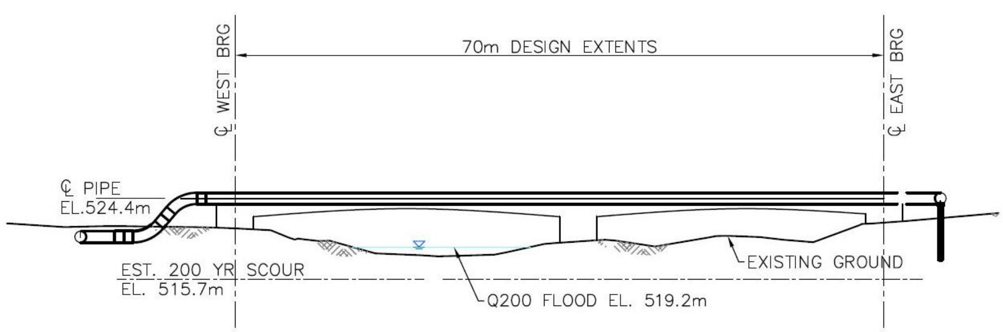


Figure 10. Continuous Concrete Concept

To increase the beam capacity while minimizing the depth of the beam over its entire length, haunches can be designed. Since concrete bridges require large amounts of formwork and falsework, construction costs and impacts to the floodplain are increased. As such, Team Moja deemed this bridge structure not feasible.

3.1.5 Preliminary Bridge System

Through research and comparing the advantages and disadvantages of each of the bridge systems above, we came to the conclusion truss bridges were the most suitable bridge. The truss bridge ranked overall the highest in terms of constructability, cost, and environmental impact.

This initial evaluation was based on if the truss could span the entire 70 m without the need of an additional pier.

Table 1. Conceptual Design Summary

	ADVANTAGES	DISADVANTAGES
Cable-Stayed Bridge	• Lightweight structure • Span large distances • Low material usage	• Low stiffness • Difficult to fabricate cables • Expensive
Suspension Bridge	• Lightweight structure • Span large distances • Little to no access required below the bridge	• Low stiffness • Difficult to fabricate cables • Expensive
Truss Bridge	• Efficient use of materials • Only axial loads	• High level of maintenance
Beam Bridge	• Simple construction • Low labour costs for steel	• Limited span • High material weights

3.2 Preliminary Design

During the preliminary design stage, alternative studies were carried out by comparing the performance of the Warren and Pratt configurations. For each of the configurations, ultimate limit states were investigated such that the factored resistance exceeded the total factored loads. The design loads, hollow structural sections (HSS), alternatives analysis, and deflection sections are discussed in the following sections.

3.2.1 Design Loads

Design loads were first determined during the preliminary design stage using CSA S6-06. Load factors and load combinations were investigated for the bridge superstructure using Table 3.1 from the code and is shown in Table 2.

Table 2. Load Factors and Load Combinations (CSA S6-06, 2006)

	Permanent loads			Transitory loads					Exceptional loads			
Loads	*D*	*E*	*P*	*L**	*K*	*W*	*V*	*S*	*EQ*	*F*	*A*	*H*
Fatigue limit state												
FLS ombCination	11.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	1.70††	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.60	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.40	1.00	0.45§	0.45	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.50§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.80§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

**For the construction live load factor, see Clause 3.16.3.*
†For superstructure vibration only.
‡For ultimate limit states, the maximum or minimum values of α_D, α_E, and α_P specified in Table 3.2 shall be used.
§For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.
***For long spans, it is possible that a combination of ice load F and wind load W will require investigation.*
††Also to be applied to the barrier loads.

Where,

A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels
K = all strains, deformations, and displacements
L = live load
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic

W = wind load on structure
a = maximum and minimum values of load factors for ULS

Through review of the possible load factors and load combinations, it was found that snow load was absent. CSA S6-06 does not account for snow load for bridges because it is typically assumed that significant snow will not accumulate and combine with the design live loads. As stated by CSA S6-06, "in normal circumstances the occurrence of a significant snow load will cause a compensating reduction in traffic load" (2006, p. 41). As Team Moja is designing a pipeline bridge, snow may accumulate over certain portions of the structure. Hence, snow load was accounted for using the *British Columbia Building Code* (BCBC) requirements. Climatic design data from the BCBC was used to determine snow pressures near the project site. As data from Hazelton was not available, snow load data from Smithers was used as Smithers is the nearest city to Hazelton out of the available locations. Climatic data for design was taken from Table C2 of the BCBC. By supplementing the CSA S6-06 with the BCBC, four possible load combinations were determined to govern the superstructure design. These load combinations are highlighted in Table 3.

Table 3. Governing Load Combinations

1.	**PRESSURE TESTING COMBINATION**		
	Self-weight	Water Weight	
2.	**CONSTRUCTION LOADING COMBINATION**		
	Self-weight	20 workers	2 kN tool cart
3.	**DEADLOAD COMBINATION (BCBC)**		
	1.4D	0.4S	0.4A
4.	**SNOW LOAD COMBINATION (BCBC)**		
	1.25D	1.5S	1.4A

Where,

D = dead load
S = snow load
A = ice accretion load

In the consideration of dead loads, a conservative estimate was used to determine the governing load combination. The dead loads consisted of the pipe self-weight, grating self-weight, and the weight of the members themselves. Design loads are further explained in Appendix B.

Ice accretion loads were determined using CSA S6-06 as these loads are expected to occur on all exposed surfaces of superstructure members. The design ice thickness was specified in Figure A3.1.4 of CSA S6-06 and is shown as Figure 11 in this report.

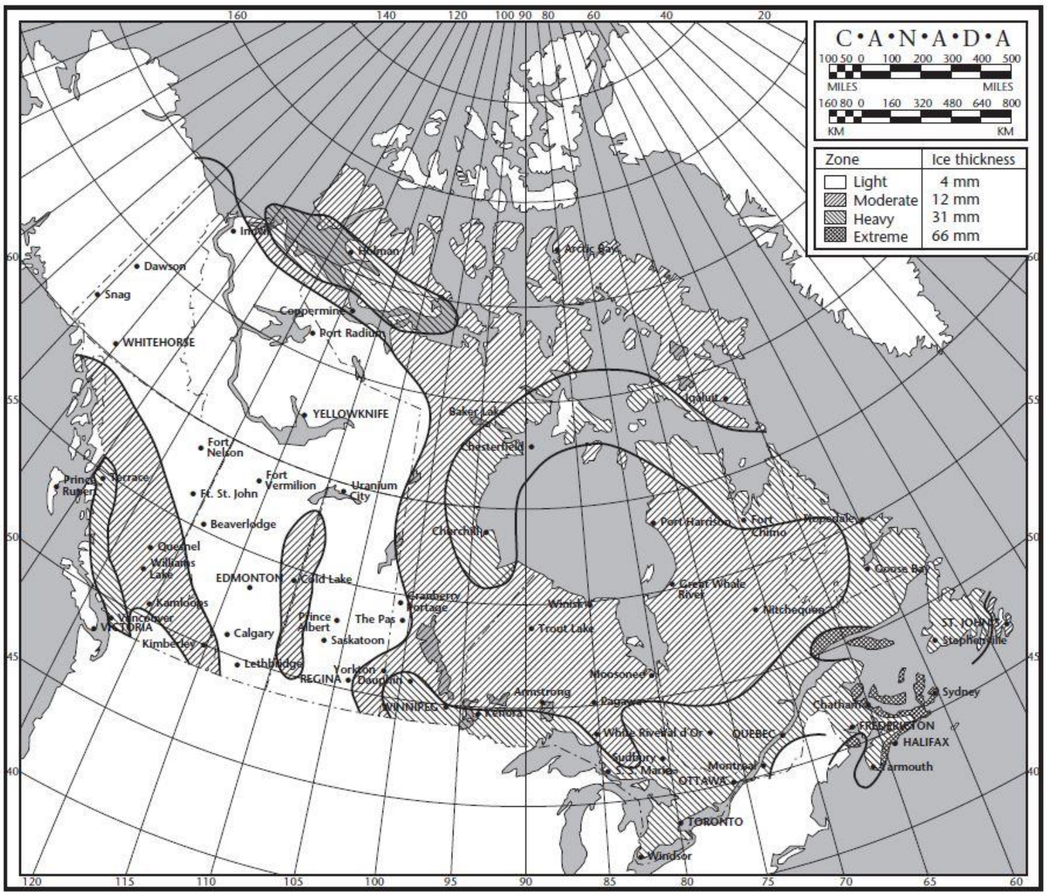


Figure 11. Ice Accretion Design Thickness (CSA S6-06, 2006)

It was identified that the design thickness for ice accretion is 12 mm. A unit weight of 9.8 kN/m^3 was used to calculate the ice accretion loads.

By determining the factored loads imposed on the superstructure, it was found that the governing load combination was Load Combination 4, involving snow and ice accretion loads. By inspection Load combination 2 was deemed to not be a governing case for the designs performed. This case may, however, govern in the steel grating design.

3.2.2 Hollow Structural Sections

HSS were chosen in the design of the pipeline bridge. In Canada, HSS have been frequently used in trusses as some of the most common applications of HSS include the chord and web members of steel trusses (Krentz, 1996). Furthermore, HSS appeal to architects for aesthetic purposes. The standard steel for HSS in Canada is in accordance with CSA G40.20/G40.21 grade 350W which has a yield strength of 350 MPa and is used for general structural purposes. Due to their strength in compression, there can be economical advantages in using HSS over other steel sections. These savings come from reduced weight and can be extended into savings in transportation and erection even though the unit material costs are generally higher.

Square and rectangular HSS were selected over round HSS as square and rectangular members are easier to fabricate and more economical. When compared to similar sized I-shaped members, square and rectangular HSS have lower surface areas, thus reducing the cost of painting and corrosion protection. Material costs for HSS may be up to 25% higher than open rolled sections; however, the extra costs is often negated as HSS have greater efficiency in resisting compression and torsion (Packer & Henderson, 1997).

G40.21 sections for HSS come in both Class H and Class C. Class H sections are either hot-formed or cold-formed to final shape and then stress-relieved, whereas Class C sections are cold-formed and are not stress-relieved. Since the manufacturing methods differ for Class H and Class C sections, residual stresses in Class H sections are small relative to Class C sections (CISC, 1995). As such, Class H sections have greater axial compressive resistances than Class C sections. However, Class C sections were chosen in the design as these sections are normally available in stock by major manufacturers. Class H sections often require a special mill order and are more expensive than Class C sections.

3.2.3 Alternatives Analysis

To design the geometric layout of the Warren and Pratt configurations, an initial depth was used using guidelines published by the Comité International pour le Développement et l'Etude de la Construction Tubulaire (CIDECT). CIDECT is an international association of HSS manufacturers and cite the ideal span to depth ratio is usually found to be between 10 and 15. A set of sample trusses were designed and analysed using the factored design loads using this span to depth ratio as a starting point. Structural analysis of the trusses were determined using the simplified method. This method assumes that all members are pin-connected and loads are only applied at panel points (CSA S16-09, 2009). As a result, only axial tensile and compressive forces are induced in the members. See Appendix C for the calculations on both the Warren and Pratt configurations.

Following, Excel spreadsheets were developed for both Warren and Pratt configurations to determine the optimum depths and panel widths for the truss design. Multiple options were made with the number of panels and depth being the varying parameters. All member axial forces were calculated for each of the options for the 70 m span. Appropriate HSS sizes were then chosen from the Handbook of Steel Construction to resist the factored axial forces.

Deflection of each of the options were determined at this stage of the design. However, none of the options met the required deflection limits and the overall design of the pipeline bridge needed to be re-evaluated. As a result, an additional pier was introduced, segmenting the original 70 m span into a 40 m span and a 30 m span. The additional pier is illustrated in Figure 12 and is denoted as the middle pier.

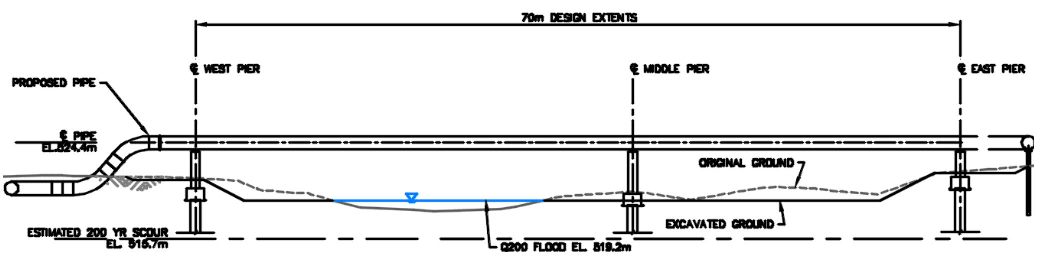


Figure 12. Additional Pier in the 70 m Span

The preliminary analysis of the structure was completed using the new pier location as a support and the Excel spreadsheets in Appendix C reflect this change. These Excel spreadsheets represent only the 40 m span as this will be the governing case with the with largest forces in these members.

For the Warren configuration, 12, 10, 8, 6, and 4 panels were considered for the 40 m span. Similarly, the Pratt truss was analysed using 10, 8, 6, and 4 panels. When determining HSS sizes for the chord and web members, the number the different sizes were limited for optimum economy.

3.2.4 Deflection

The deflection of the truss was calculated simultaneously with the member force calculations to determine preliminary member sizes. The deflection requirements were specified by the pipeline designer and used the following criteria:

- Maximum support span is restricted to 11.6m for a 1.22m diameter pipeline
- Maximum pipe deflection is the smaller of 5% of the pipe diameter or L/500 (where L is the span between main vertical supports)

From these deflection criteria, the pipeline was supported at panel points at a maximum distance of 11.6 m and the maximum overall deflection of the truss was determined to be 61 mm. The deflection of each of the truss options was determined through the principle of superposition at mid-span using specified loads and the equation below:

$$\Delta_{max} = \frac{Pa}{24EI}(3l^2 - 4a^2)$$

Where,

Δ_{max} = maximum deflection at mid-span
P = load
a = distance from end of span to load
l = overall span length
E = modulus of elasticity
I = moment of inertia

This equation calculates the deflection based on a loading pattern where two equal concentrated loads are symmetrically placed. See Appendix D for the Excel spreadsheet calculations for deflection.

3.2.5 Preliminary Design Summary

The weight for a single planar truss was calculated based on the member sizes for each of the options. Both the truss weight and deflection were used as evaluating criteria to determine the optimal number of panels for the Warren and Pratt configurations. A summary of the Warren and Pratt configurations is highlighted in Table 4.

Table 4. Summary of Preliminary Warren and Pratt Weights and Deflections

No. OF PANELS	WARREN		PRATT	
	Weight (kN)	Deflection (mm)	Weight (kN)	Deflection (mm)
12	41.5	91	n/a	n/a
10	40.9	61	39.1	60
8	37.1	43	37.4	59
6	38.0	22	44.3	49
4	45.1	9	53.5	56

The results indicate the eight panel Warren and Pratt configurations satisfies the deflection limits of 61 mm while having the least weight relative to the other options. Literature was referenced in determining which alternative to continue with detailed design. Packer and Henderson (1997) express Warren trusses generally provide the most economical solution as they have approximately one half the number of connections compared to Pratt trusses. Consequently, there are labour and cost savings associated with Warren trusses. Taking this into consideration, the Warren configuration was chosen to continue on with detailed design. A summary of the selected HSS sizes is shown in Table 5.

Table 5. Eight Panel Warren Truss HSS Sizes

	SECTION
Top Chord	178x178x6.4
Bottom Chord	178x178x6.4
Web Diagonals	114x114x4.8

As previously mentioned, both chord and web members were limited to a single size each to lower fabrication costs and optimize economy.

3.3 Detailed Design

As the alternatives analysis concluded with the preliminary design, a detailed design was completed based on the Warren configuration. Detailed analysis was completed using CSA S16-09 and CSA S6-06. Detailed design involved the design of the lateral members, connection, and bearing design.

3.3.1 Lateral Members

A bracing system was designed to resist lateral loads, maintain the stability of the structure, and provide restraint to the chord member to prevent lateral torsional buckling. The bracing system lateral sections that are perpendicular to the chords, cross-bracing, and sway-bracing.

For the geometric layout of the bracing sections, lateral members were placed every 5 m for both the top and bottom chords. Five metres was chosen as the compression chord was designed to resist a compressive load of an unsupported length of up to 5 m. Hand calculations were done to size these members and were aided using computer modeling. See Appendix E for supporting hand calculations.

SAP2000 was used to create a 3D model to simulate the stability of the structure under the applied loads. Using SAP2000, cross-bracing members were designed and placed at every second panel for the top chord and at every panel for the bottom chord. This is due to the additional loads imposed at the bottom of the superstructure coming from factored pipe loads and wind loads acting on the pipe surface area. To meet pressure testing loads, a thicker section was chosen (HSS 178x178x9.5) for the bottom lateral sections compared to the top chord (HSS 178x178x6.4). See Appendix E for supporting calculations.

A similar procedure was carried out to design the sway-bracing to prevent overturning of the entire structure. Vertical members were added at sway-braced points to provide a location to connect the sway-bracing from the underside of the top lateral members to the sides of the truss. See Drawing D04 in Appendix **X** for lateral sections. As vertical members were introduced to the truss, it became a modified Warren truss. Table 6 summarizes the lateral sections used for this design.

Table 6. Summary of Lateral Sections

	SECTION
Top Chord Struts	114x114x4.8
Pipe Support Beams	178x178x9.5
Cross-Bracing	114x114x4.8
Sway-Bracing	102x102x4.8
Verticals	114x114x4.8

3.3.2 Gap Connections

The predominant HSS connection for our truss design is a K arrangement. KT connections are also present in the design, however, due to time constraints only K connections were designed. K connections has a branching member in compression while another branching member is in tension. These members are attached to the chord through welds and are oriented so that either a gap exists or the members overlap. Both a gap connection and a partial overlap connection are illustrated in Figure 13.

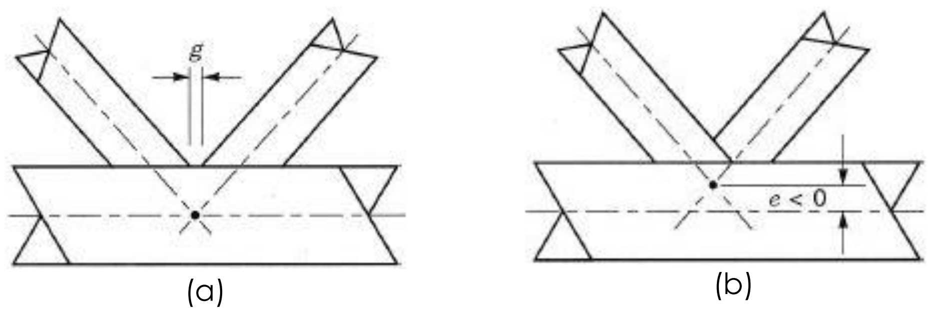


Figure 13. Joint Configurations: (a) Gap Connection; (b) Partial Overlap Connection (Packer & Henderson, 1997)

It is generally advised to design for gap connections as the fabrication costs are lower when compared to other connection types. The branch members are easier to cut and weld if gap connections are used.

When designing HSS connections, different failure modes need to be considered to identify the governing case. Plastic failure of the chord face is the most common failure mode for K gap connections and was the governing case for this design. Figure 14 illustrates the chord face plastification failure mode.

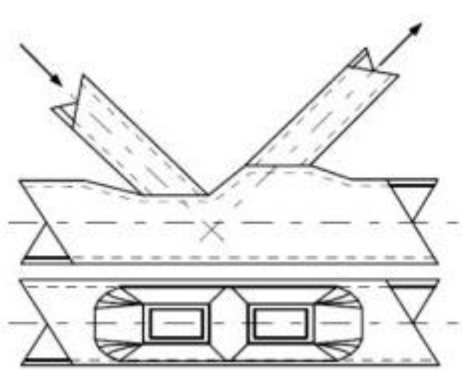


Figure 14. Chord Face Plastification Failure (Shimkus, 2011)

In this mode of failure, one branch member pushes the chord face in while the other member pulls the chord face out.

Guidelines and design formulae created by the International Institute of Welding (IIW) were used to determine typical connection resistances for the truss. The design procedure began by checking the preliminary member sizes at critical panel points. For compression members, compressive resistances were interpolated from the Handbook of Steel Construction and compared with the axial forces in compression members. Tensile resistances were calculated by finding the minimum cross-sectional area of steel required to resist the axial forces in tension members. See Appendix F for the supporting calculations.

Once the preliminary members were checked, it was required to investigate whether gap connections were feasible for the selected configuration and member sizes. These validity limits included to have an appropriate web to chord width ratio shown in the equation below:

$$\beta = \frac{b_1 + b_2}{2b_0}$$

Where,

β = web to chord width ratio
b_1, b_2 = web width
b_0 = chord width

Packer and Henderson presented charts evaluating whether the proposed configuration with square web members would meet allowable eccentricity limits. Figure 15 shows the evaluation of maximum β.

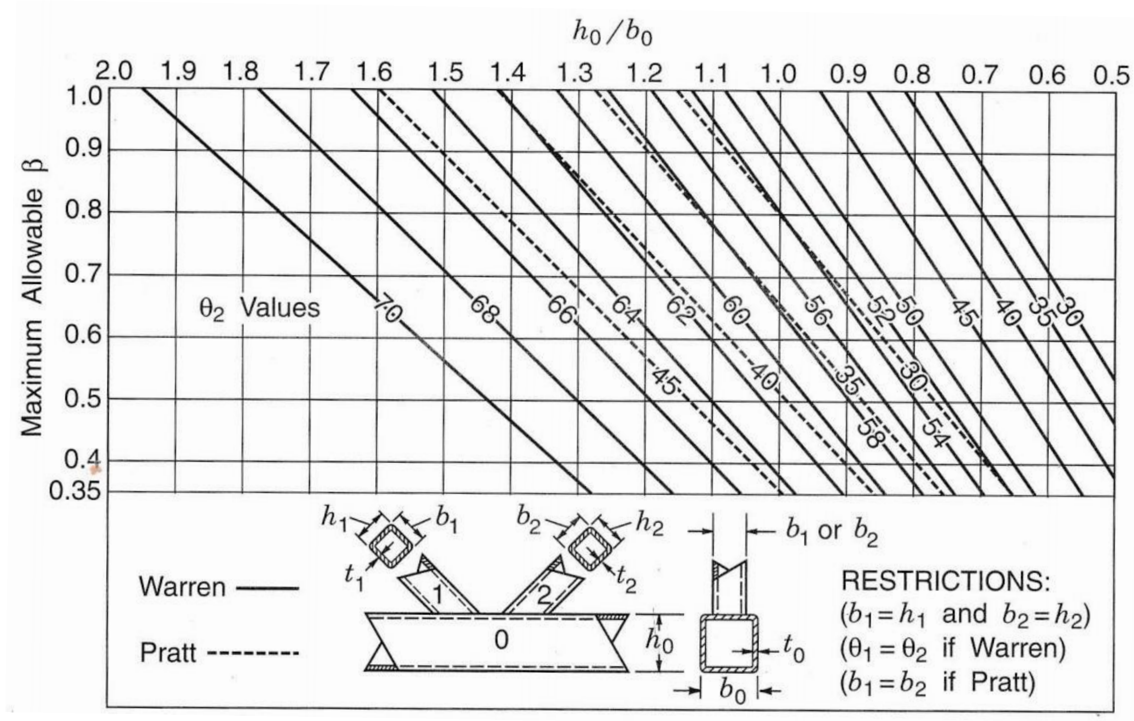


Figure 15. Max Allowable β Based on Allowable Eccentricity Limits, Chord Aspect Ratio, and Inclination of Web Members (Packer & Henderson, 1997)

From determining β, it was found using the selected HSS from the preliminary analysis yielded a β value greater than the maximum β based on the chord aspect ratio and inclination of web members. Consequently, HSS sizes for the chords were changed to have a satisfactory β value. A list of HSS changes are shown in Table 7.

Table 7. Preliminary and Final Sections for Design

	PRELIMINARY SECTION	**FINAL SECTION**
Top Chord	HSS 178x178x6.4	HSS 203x152x6.4
Bottom Chord	HSS 178x178x6.4	HSS 203x152x6.4
Web Diagonals	HSS 114x114x4.8	HSS 114x114x4.8

The rest of the detailed design continued with the final sections. See Appendix F for the remaining validity checks for the web members.

Once it was determined that gap connections were feasible for the design of K connections, the connection resistances were analysed. Connection resistances were found using design charts produced by Packer and Henderson. Figure 16 shows a design chart.

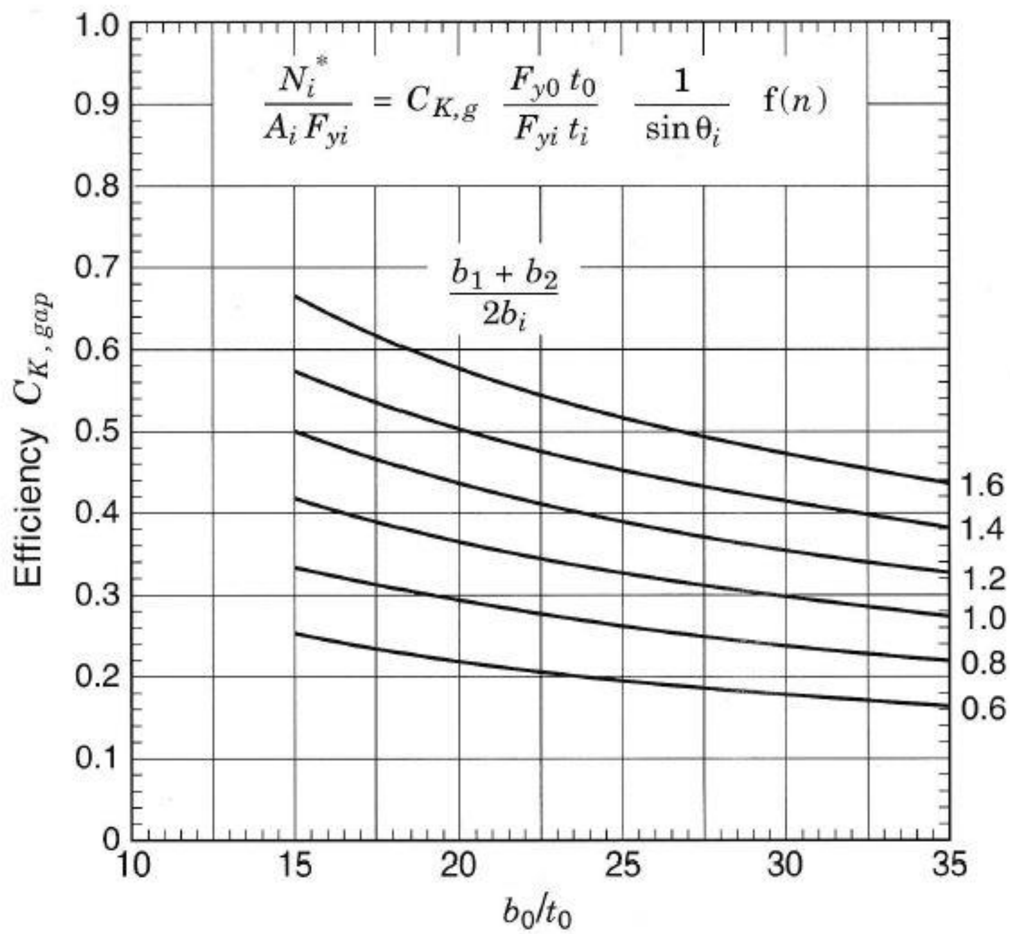


Figure 16. Web Member Efficiency for Square HSS K and N Gap Connections (Packer & Henderson, 1997)

The design chart on the previous page was used in conjunction with the equation below:

$$N_1^* = C_{K,gap}\frac{t_0}{t_1}\frac{1}{sin\theta_1}f(n)A_1F_{y1}$$

Where,

N_1^* = connection resistance normal to the chord
$C_{K,gap}$ = efficiency coefficient
t_0, t_1 = section thickness
θ_1 = web inclination angle
f(n) = function of chord longitudinal stresses

Using Figure 16, an efficiency coefficient for the connection was obtained to calculate the connection resistance normal to the chord. From these calculations, it was determined critical panel points were acceptable in resisting the factored loads. The designed gap detail is shown in Drawing D04. The designed gap connection has a gap of 20 mm while meeting the specified limits of eccentricity.

3.3.3 Splice Connection

As the larger span of the bridge is 40 m, it was assumed that the truss was long enough to require chord splices. Bolted flange-plate splice connections are preferred over field welding as bolting allows the fabricator more room to accommodate for tolerances when compared to field welding. Two 22 mm plates with six M22 bolts at the splice location were designed. See Appendix F for supporting calculations and Drawing D04 for connection details.

3.3.4 Welds

E49XX electrodes are normally used for HSS and has a minimum ultimate stress of 450 MPa. Effective weld lengths were calculated for a typical K connection for the truss. In addition, the default method for sizing welds was used and it was calculated that an 8 mm fillet weld was sufficient for the design. See Appendix F for the welds design.

The size of the welds can further be optimized; however, due to time constraints Team Moja did not consider other methods for sizing welds.

3.3.5 Bearings

Bridge bearings provide sufficient loading area for stress distribution of concentrated loads. For the pipeline bridge, the truss loads are transferred onto the pier through the bearings.

Plain elastomeric bearings were selected by Team Moja for design and was done following the provisions under CSA S6-06 11.6.6.3. See Appendix G for bearing calculations. A summary of the bearing requirements are outlined in Table 8, below.

Table 8. Bearing Requirements

Bearing Type	Elastomeric Bearing
Effective Thickness	40 mm
Compressive Deformation Limit	3 mm
Rotational Deformation Limit	6 mm
Shear Deflection Limit	20 mm
Effective Area	250 mm x 250 mm

4.0 SUBSTRUCTURE DESIGN

The bridge substructure consists of all the bridge components below the bearings. These structural components are the piers, pile caps, and piles. The transfer of loads from the superstructure to the soil is done through the substructure. Both lateral and vertical loads were considered in the design of the substructure. However, due to time constraints, seismic loading was not considered in the substructure design. See Appendix L for the final design drawing package including the substructure design.

Provisions from *CSA A23.3-14 Design of Concrete Structures* and CSA S6-06 specify material properties and material resistance factors relating to concrete and reinforcing steel. These parameters were used in the design of the bridge substructure. The values used for design and their corresponding sources are outlined in Table 9 on the following page.

Table 9. Parameters used in Substructure Design

	PARAMETER	VALUE		DESCRIPTION	SOURCE
Steel	φ_s	0.9		Resistance of steel reinforcement	CSA S6 8.4.6
	f_y	400	MPa	Yield strength of steel reinforcement	CSA S6 8.4.2.1.3
	E_s	200	GPa	Modulus of elasticity of steel reinforcement	CSA S6 8.4.2.1.4
Concrete	φ_c	0.75		Resistance factor for concrete	CSA S6 8.4.6
	f'c	400	MPa	Specified compressive strength of concrete	CSA S6 8.4.1.2
	α_1	0.81		Ratio of average stress in a rectangular compression block to the specified concrete strength	CSA A23.3 10.1.7
	β_1	0.90		Ratio of neutral axis stress distribution of a section to the neutral axis of the section	CSA A23.3 10.1.7
	ε_{cmax}	0.0035		Maximum strain at extreme compression fibre of a concrete member	CSA A23.10.1.3
	λ	1.0		Factor accounting for concrete density	CSA A23.3 8.6.5
	F_{cr}	2.2	MPa	Cracking strength of concrete	CSA S6 8.4.1.8

Material resistance factors φ_s and φ_c correspond to the ultimate limit states design used for steel and concrete, respectively. These factors account for uncertainties and variations of the structural materials used. It was noted that the material resistance factors pertaining to CSA S6-06 were less conservative than material resistance factors used in the BCBC.

According to CSA S6 8.4.2.1.3, the specified strength of steel reinforcement, F_y, shall be between 300 and 500 MPa. For the bridge substructure, an average value of 400 MPa was selected as the yield strength of reinforcing bars. The specified yield strength, along with the specified modulus of elasticity of 200 GPa, imply an elastic strain of 0.002 for reinforcing steel.

A concrete compressive strength (f'c) of 30 MPa was used in structural concrete members in accordance with CSA S6 8.4.1.2. This was stated as the minimum strength for non-prestressed concrete bridge components. In addition, normal density concrete was selected for design, yielding a λ factor of 1.0. These two parameters resulted in the calculated cracking strength of 2.2 MPa for all concrete structural components.

The ratios α_1 and β_1 are used to simplify the stress distribution in sections of concrete members subjected to flexure. The flexural stress distribution of concrete members have a parabolic shape, and using the aforementioned ratios simplifies the stress distribution into an equivalent rectangular stress block. Brzev and Pao (2009) recommended α_1 and β_1 values of 0.8 and 0.9, respectively, for concrete with compressive strength that varies between 25 and 40 MPa. Following the provisions outlined in CSA A23.3 10.1.7 yielded comparable results for 30 MPa concrete.

4.1 Conceptual Design

The approximate geometry and structural member dimensions of the piers and piles were determined during the conceptual design phase. Two pier options were considered while the design of the pile caps and piles were assumed to be the same for both pier options.

Once the preliminary dimensions of both pier options were determined, the most economical option was selected at the end of the preliminary design phase. The selection of the pier option was based on their material volume requirements, formwork requirements, and factored loads.

4.1.1 Piers

Two pier options were considered for the substructure design. Both pier options were assumed to be 3 m high from the top of the pile cap to underneath the bearing. Using provisions from CSA A23.3-14 and CSA S6-06, the factored axial, lateral and flexural loads were determined in the conceptual phase.

The compressive resistance for axially loaded members was determined as per Equation 10.11 from CSA A23.3-14. The analysis for concrete components subjected to flexure have been done in accordance with CSA A23.3-14. The beams and columns of the piers have been analyzed using provisions from this code. However, the axial and flexural resistance of the pier columns were expressed using column interaction diagrams. The points on the column interaction diagram represent the combinations of axial and flexural stresses that develop within a column (Brzev & Pao, 2009). This approach offers a quick method of assessing the preliminary column sizes required of each pier option.

During the conceptual design of the piers, the size and location of both the pile cap and the piles were assumed to be the same for both pier options. This assumption was made to simplify the evaluation process and allow the pier options to be assessed solely on their relative material volumes and factored loads.

4.1.1.1 Pier Option 1

Pier option 1 is a rigid frame. It is composed of two square columns and is laterally braced by a beam at the top. These columns are supported by pile caps which then transfers loads onto the pile. Figure 17 below shows Pier Option 1.

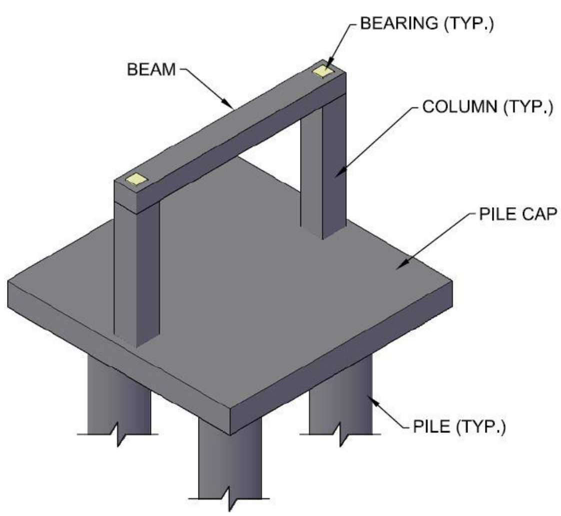


Figure 17: Pier Option 1 Representation Model

The columns are loaded concentrically by the trusses of the bridge superstructure through bearing pads. The concrete beam helps resist lateral loads induced by wind on the superstructure. These columns then transfer all their loads to the pile cap.

For design purposes, Pier Option 1 is modelled as a 3° statically indeterminate rigid frame. By inspection, lateral wind load imposed on the bridge superstructure governs the design of the pier columns since the concrete pier has high compressive resistance against gravity loads.

Load Combination 4 as outlined in Table 3 was the governing load case considered in the conceptual design phase. By inspection, wind load perpendicular to bridge governs over wind load parallel to the bridge because the contact surface area of the first load configuration is much greater. Figure 18 shows the external forces acting on the pier due to the governing load case.

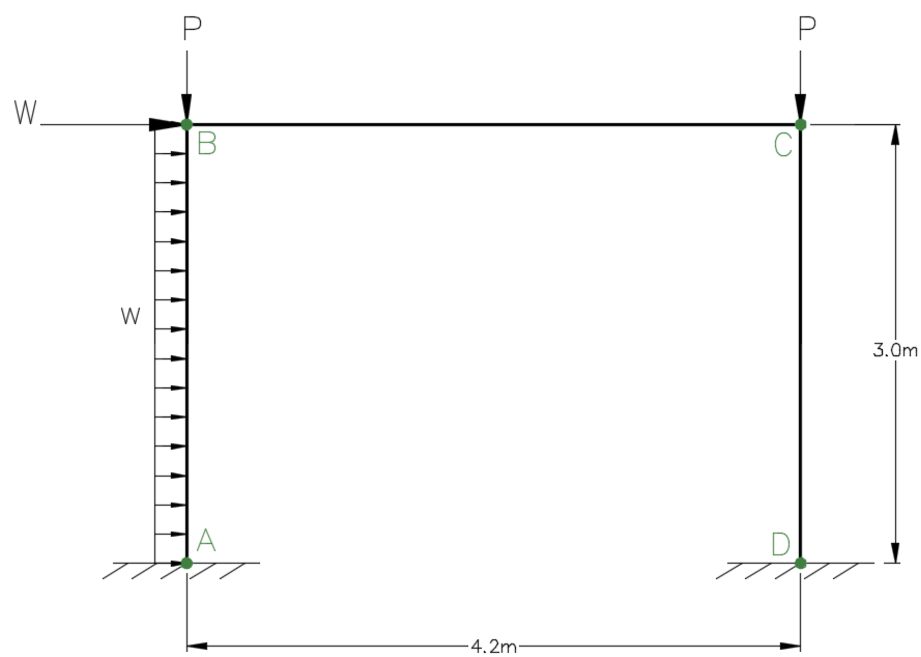


Figure 18: External Design Loads on Pier Option 1

On the figure, P represents the forces exerted by the trusses on the pier which was 583 kN given this load case. W represents the lateral wind force exerted on both the windward and the leeward trusses which gets transferred to the bearings on top of the pier. The lateral load was then calculated to be about 224 kN. Finally, w represents the wind load acting directly on the pier which is negligible compared to the wind force, W acting on the superstructure.

Because Pier Option 1 is statically indeterminate, an indeterminate analysis was performed using MASTAN2 and SAP2000 to determine the support reactions of the frame and its internal moments. Both structural software were used to confirm the values obtained from one another. Internal shear is not evaluated in this phase since this was to be completed during the detailed design phase.

Table 10 shows the comparison of the internal moments obtained from both MASTAN2 and SAP2000.

Table 10. Internal Moments obtained from SAP2000 and MASTAN2 for Pier Option 1

INTERNAL MOMENT	**RESULTS**		**RELATIVE ERROR (%)**
	SAP2000 (kNm)	MASTAN2 (kNm)	
M_a	206	291	29.2
M_b	136	133	2.2
M_c	135	86	37.8
M_d	202	152	25.2

The relative error between the MASTAN2 and SAP2000 calculations were significant since MASTAN2 bases its calculations on given section properties and material properties of the members and the deflections that amount from the given loading conditions. Since the member sizes of the pier are not yet known, Team Moja added arbitrary sectional and material properties for each member in the MASTAN2 model. As such, differences in the results were expected and the greater of the calculated moments were considered in the conceptual design phase.

From the above, the subscripts of the internal moments in the table above denote their appropriate location in Figure 18. The reaction moments M_a and M_d are in the counter clockwise direction while the internal moments M_b and M_c are on opposite directions. The orientation and direction of the internal moment does not matter because the

- direction of wind load can occur in both directions
- internal moments on the left hand side of the pier are similar to those on the right hand side.

Therefore, Pier Option 1 will have to be designed to be able to resist symmetrical moments acting at opposite directions.

To be conservative, the governing moment from Pier Option 1 was then taken to be the maximum of the moments shown in

Table 10. The governing moment is therefore about 300 kNm taken from Support A in Figure 18 using the results from MASTAN2.

4.1.1.2 Pier Option 2

Pier Option 2 is composed of a non-prismatic cross-beam supported at mid-span by a square column. The square column is located at the centre of the pile cap which is supported by the foundation. Figure 19 shows a schematic of Pier Option 2.

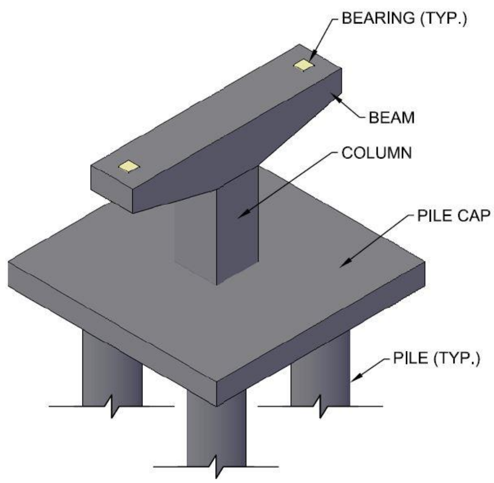


Figure 19: Pier Option 2 Model Representation

The two bearing pads at each end of the cross-beam take the reaction forces from the trusses of the superstructure. The cross-beam is non-prismatic to reduce its self-weight. Negative bending is induced at the beam's mid-span where it is supported by the column. As well, the column will take compressive loads and lateral wind pressure from the bridge superstructure.

Gravity loads govern the design of Pier Option 2's cross-beam. The load case governing the cross-beam is Load Combination 4 from the BCBC outlined in Table 3. The factored load of the trusses under this load case was 711 kN. The cross beam has to resist negative bending at the column location due to the cantilevered ends of the cross-beam carrying the trusses. As well, internal shear at the column-beam interface was recognized to be a potential design concern.

The governing load case for Pier Option 2 is ULS Combination 4 from CSA S6-06 as outlined in Table 2. Similar to Pier Option 1, this is due to lateral wind forces acting perpendicular to the bridge superstructure. The external forces exerted on the pier are shown in Figure 20.

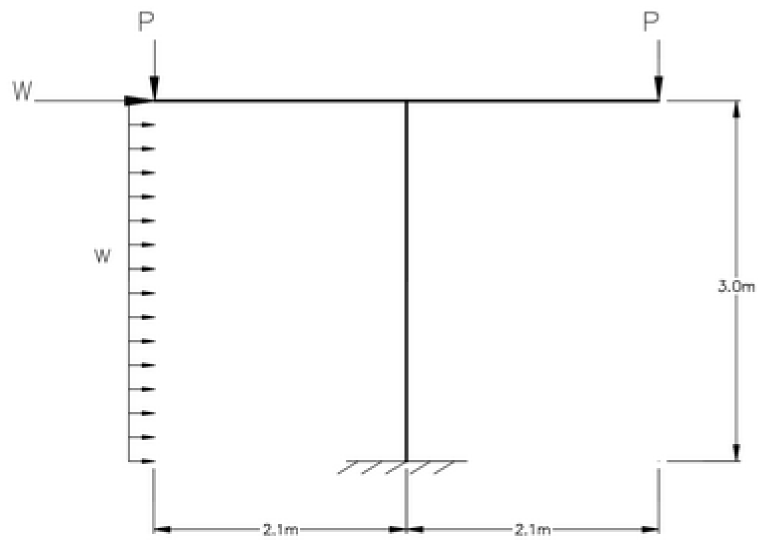


Figure 20. External Design Loads of Pier Option 2

Figure 18 exhibits the same forces indicated on Figure 20, above. The factored moment required to be resisted by the cross-beam is therefore simply P multiplied by the length of the cantilever, 2.1 m. The column, on the other hand, has to resist the moment produced by the wind force W multiplied by the column height of 3 m. The resulting factored moments for the beam and the column are about 1500 kNm and 690 kNm, respectively.

4.1.1.3 Comparison of Alternatives

The load analysis indicated that the flexural requirements for both the beam and the column of Pier Option 2 was much greater than Pier Option 1. The computer analysis for Pier Option 1 yielded a maximum moment of 300 kNm on one column base while Pier Option 2 yielded 1360 kNm at its column base. In addition, the internal moments for the beam in Pier Option 2 was much greater than the internal moments in the beam of Pier Option 1. While the difference in factored moments are substantially large, the more economical option can only be determined once the two pier options are sized. This step is outlined in the preliminary design section.

4.1.2 Piles

Piles were recommended by Allnorth since the bridge foundations needed to be protected from creek scour. From the geotechnical analysis, two cases on the soil conditions were presented in the design of piles. These two cases are summarized in Table 11.

Table 11. Cases for Geotechnical Conditions for Pile Design

Case A	Competent bedrock was assumed to be at a depth of approximately 8.4 m below the existing ground. A minimum 4.2 m pile length embedded into bedrock is required.
Case B	Piles would be embedded in overburden only and not reach bedrock. The pile length is such that a minimum geotechnical pile resistance is attained from embedment 18 m below scour level.

The overburden was identified to comprise of sandy gravel with traces of clay.

A pile bearing capacity of 1300 kN was recommended by others using the β-method and the use of 914 mm diameter piles. The soil below the total scour level was assumed to be primarily made of sand and gravel as recommended by others. Hence, Team Moja used the design procedure for cohesionless soils in the *Canadian Foundation Engineering Manual* to design the piles. This bearing capacity was calculated by others using the following parameters in Table 12:

Table 12: Parameters Used by Others to Determine Pile Bearing Capacity

PARAMETER	VALUE	SOURCE
Geotechnical resistance factor for compression, ϕ_c	0.4	Canadian Foundation Engineering Manual 18.2.1
Geotechnical resistance factor for compression, ϕ_t	0.3	
β	0.6	
N_t	50	
Pile Dimensions	914 mm x 12.7 mm (diameter x thickness)	Recommended from the geotechnical and hydrotechnical assessment provided by Allnorth
Pile Spacing	2.5 x Pile Diameter	CSA S6 6.8.9.2

The geotechnical resistances are ultimate limit state factors that are to be multiplied to the total ultimate axial capacity of the pile. ϕ_c accounts for compression and ϕ_t for uplift (tension). For design purposes, the piles were assumed by others to be open-toed driven steel pipe piles, and the soil is assumed to be cohesionless, medium sand, which yields a β value of 0.6 and an N_t value of 50.

As a rough estimate, a preliminary pile size and spacing were assumed by others to determine a preliminary bearing capacity for the bridge superstructure. The preliminary piles were 914 mm in diameter and 12.7 mm thick. They were to be spaced 2.5 times their diameter to ignore pile group effects as per CSA S6-06. These simplified design assumptions were re-evaluated by Team Moja in the preliminary and the detailed design phases to determine the most cost-effective pile distribution and size.

For the conceptual design phase, Team Moja will only consider compression loads on the piles. Pile bearing capacities were obtained from others using the K-tan method and the β-method. A comparison of the pile bearing capacity for single piles obtained by others and Team Moja are as follows:

Table 13. Calculated Bearing Capacity due to Pile Skin Friction

PROCEDURE	CALCULATED BEARING CAPACITY
β-method by others	1300 kN
K-tan method by Team Moja	1200 kN-1500 kN

4.2 Preliminary Design

The preliminary design phase of the substructure only encompassed developments in the design of the piers. Additional design checks were done on the piles once the preliminary dimensions of the piers and pile cap are determined. Other structural capacities of the piles, aside from compressive bearing capacity, such as settlement, downdrag, lateral deflection and maximum moment will be discussed in the detailed design phase.

The goal of the preliminary phase is to obtain the approximate sizes of the structural members. During this phase, the flexural reinforcement, and the dimensions of the pier required to provide sufficient flexural resistance to the internal moments determined on the previous section. Other requirements as specified by CSA S6-06 and CSA A23.3-14 such as shear, slenderness effects, and cracking requirements were done in the detailed design phase. Once the required dimensions for flexure are determined for both pier options, the most economical option will be evaluated in the detailed design phase. The estimated required concrete volumes and the required area of formwork were used to evaluate the economic viability of each option.

4.2.1 Pier Option 1

The beam and the column of Pier Option 1 were designed according to Load Combination 4 in Table 3. The internal moments determined in Section 4.1 were used in the preliminary design.

4.2.1.1 Beam Design

The beam was designed to resist the internal moments induced by lateral wind load on the bridge. These moments are M_b and M_c indicated in

Table 10. Since both of these moments have relatively similar magnitudes, it is necessary to design the beam as a symmetrical doubly-reinforced beam to resist moments acting in both directions.

The flexural resistance of doubly-reinforced concrete beams are usually done through iteration, checking if strains within concrete and reinforcing steel exceed their maximum strains. However, since the beam is symmetrically reinforced both on the tension side and the compression side, the moment is assumed to be resisted solely by the tension and compression reinforcement of the beam. This is conservative because if and when the concrete fails before the reinforcement due to the compressive bending stress induced on the beam, that compressive load will be taken by compression reinforcement.

Even though it is ideal to design for a steel-controlled failure (having the reinforcing steel yield before the concrete fails), this is not a practical design requirement for the beam of Pier Option 1 since having symmetrical reinforcement is a necessity. The preliminary design amounted to a beam with dimensions 400 mm x 500 mm and 4-25M Bars spaced 150 mm apart. The beam is rectangular to simply to be flush with the column width for simplicity in construction.

4.2.1.2 Column Design

As determined in Section 4.1, the factored moment governing the size of the columns was about 300 kNm. The approximate required column size were determined by using a column-interaction diagram. Checks for compressive resistance and buckling resistance were also done but was found not to govern the design.

The column interaction diagram for one column of Pier Option 1 is illustrated in Figure 21.

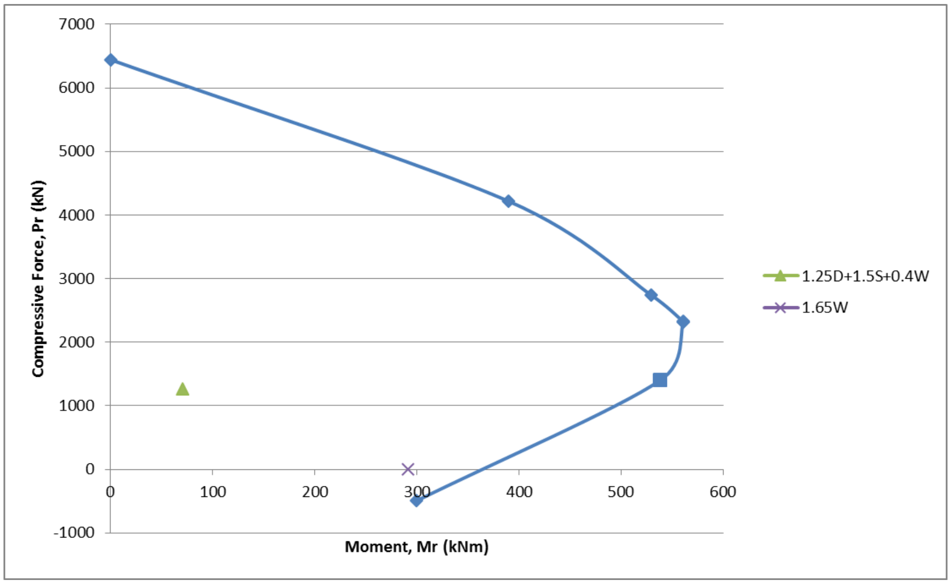


Figure 21. Preliminary Phase: Column Interaction Diagram for Pier Option 1

The blue line is the column interaction curve which represents the combinations of axial forces and moments that the column is able to resist given its size and reinforcement design. The horizontal axis of the diagram denote a steel controlled failure mechanism induced by pure bending. The vertical axis denotes a concrete controlled failure due to failure by compression (Brzev & Pao, 2009).

The green triangle on the graph represents the combined compressive load and bending induced by Load Combination 4 from the BCBC, while the purple cross mark represents ULS Combination 4 from CSA S6-06. Points within the column interaction curve indicate a safe loading condition where the compressive resistance and the moment resistance of the column exceed the compressive and flexural loads.

As seen from the interaction diagram, the governing failure mechanism is a steel-controlled failure induced by pure bending. This closely corresponds to the load case due to lateral wind load on the bridge. The columns were assumed to have a square cross-section and eight reinforcing bars evenly spaced. Using the column interaction diagram, various column sizes and reinforcement designs were implemented in this configuration while trying to maintain a reasonable safety factor. The resulting preliminary column size had a cross-section of 500 mm x 500 mm and has 8-30M bars for reinforcement.

4.2.2 Pier Option 2

Pier Option 2 is a statically determinate structure, and the factored loads and moments for each member were analyzed using conventional methods. The preliminary design phase uses the same methods of analysis as the ones performed on Pier Option 1.

4.2.2.1 Beam Design

The column-beam connection was modelled to be fixed. The beam was designed for flexure based on the moment at the column-beam interface. As such, the bending resistance requirements of the beam decreases further away from the column.

Thus, the beam does not require a consistent depth to resist the decreasing moment along its length. The beam was then designed to be non-prismatic to reduce concrete volume requirements and to minimize material cost and self-weight. The beam requires 9-25M bars spaced 100 mm. The top beam was sized to be 5 m long and 1 m wide. Its depth varies linearly from 0.5 m to 1 m. It is also important to note that shear reinforcement was found to be required for the beam.

4.2.2.2 Column Design

The compressive, flexural, and buckling resistance were evaluated to check the assumed size of the columns for Pier Option 2. The column was assumed to have a square cross-section and is assumed to be a 3 m high fixed cantilever. As with Pier Option 1, lateral wind loads imposed on the bridge was found to govern the flexural design of the column.

A column interaction diagram is used to determine the required column size and reinforcement to resist the column’s factored loads. The column interaction diagram for Pier Option 2 is on the next page.

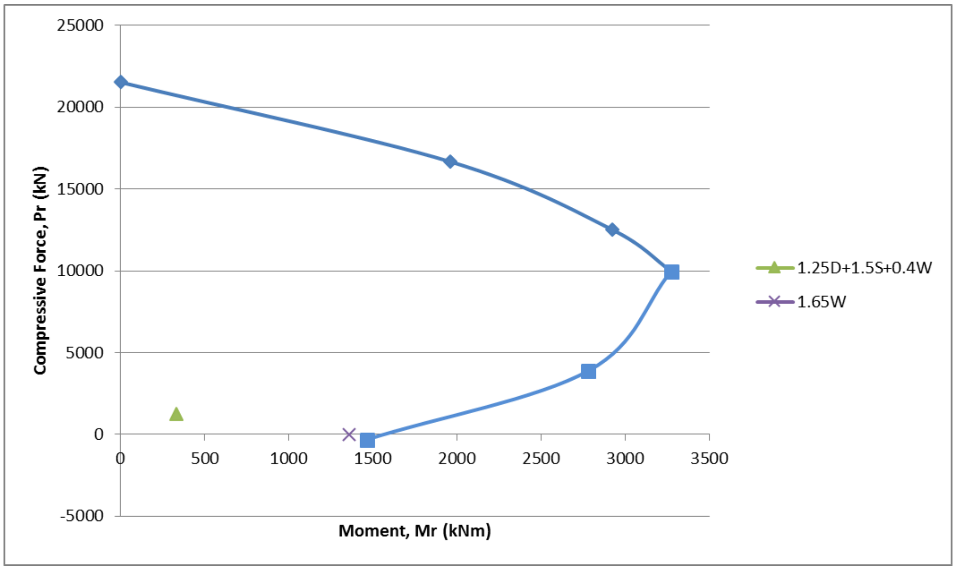


Figure 22. Preliminary Phase: Column Interaction Diagram for Pier Option 2

The symbols and lines used for this diagram are similar to those used in Figure 21. The resulting column had a cross-section of 1000 x 1000 mm with 10-35M reinforcing bars. The result obtained from the column interaction diagram above makes sense since the column was required to resist a moment greater than the previous option into one column whereas the previous option had two columns sharing the load.

4.2.3 Comparison of Alternatives

After the determination of the approximate sizes of the structural components of both pier options, the concrete volume requirements and the formwork requirements were calculated. These quantities are outlined in Table 14.

Table 14. Material Quantities of Pier Options after Preliminary Design Phase

	Pier Option 1	**Pier Option 2**
Volume of Concrete (m^3)		
Columns	0.75 (Single column)	2.0
Beam	0.74	4.0
Total (m^3)	2.24	6.0
Required Formwork (m^2)		
Columns	5.2	21.2
Beam	6.0	8.0
Total (m^2)	16.4	50.4

As shown in Table 14, Pier Option 1 is significantly more economical since the required volume of concrete and the required formwork is much less than Pier Option 2. Hence, Pier Option 1 was chosen as the optimal option in going forward to the detailed design phase.

4.3 Detailed Design

The detailed design phase of the substructure began after the selection of the pier option. Other load conditions and requirements were to be checked and designed for during this phase.

With regards to the piers, detailed design encompasses several checks required by the CSA S6-06 and CSA A23.3-14 codes. Shear reinforcement requirements, crack control, and slenderness effects are among the checks that were done in detailed design. Provisions for these design requirements are outlined in the following sections.

The proper concrete cover requirement for CSA S6-06, in particular, was not appropriately incorporated during the conceptual and preliminary phases of pier design. This requirement was followed during the detailed design phase in accordance with CSA S6-06 8.11.2.2. The pier was assumed to be exposed to earth and fresh water and thus required a minimum concrete cover of 70 mm. The pile cap was assumed to be considered a footing and permanently exposed to earth. This requires a minimum concrete cover of 100 mm.

For the foundations, provisions from several sources were used. The pile cap was designed with adherence to CSA A23.3-14 and CSA S6-14. Checks including settlement, deflection, pile settlement, and downdrag from the *Canadian Foundation Engineering Manual* (CFEM) were completed by Team Moja in this Phase.

4.3.1 Substructure Re-evaluation

During the detailed design phase, an additional pier was deemed necessary to reduce the truss deflection to meet the design criteria for the pipeline. The new pier divides the original 70 m bridge span into two spans, 40 m and 30 m long. As a result, the pier design was re-evaluated. The new pier, referred to in this report as 'middle pier' was the governing pier design since it was taller than the other two piers and carries loads from both truss spans. The pier height was estimated to be about 5 m and an analysis was done similar to the analysis done in preliminary design.

At first, the east and west piers were designed separately from the middle pier. However, due to development length requirements of CSA S6-06, the columns of these piers had to be upsized. As a result, all three piers were designed to have members of the same size with the same reinforcement requirements for simplicity. Section 4.3.4 provides more details on the development length requirements of CSA S6-06.

SAP2000 and MASTAN2 were used to compute the factored internal moments using the updated lateral wind loads from the superstructure. The internal moments received from the computer analyses for the middle pier are shown on Table 15.

Table 15. Results from SAP2000 and MASTAN2 for Middle Pier

INTERNAL MOMENT	RESULTS		RELATIVE ERROR (%)
	SAP2000 (kNm)	MASTAN2 (kNm)	
M_a	401	391	2.5
M_b	303	303	0.0
M_c	303	302	0.3
M_d	395	381	3.5

The internal moments indicated in Table 15 corresponds to locations shown in Figure 18. The governing moment for the columns of the middle pier is about 400 kNm while the governing moment for the beam was about 300 kNm. The relative error between the results from SAP2000 and MASTAN2 decreased since the preliminary section properties were used in the MASTAN2 model. The governing factored internal moment increased for the beam and column. Thus, the original pier design had to be upsized. In addition, the lateral deflection of the pier was about 2 mm.

A column interaction diagram was developed for the middle pier. The pier was sized such that it satisfied flexural stress requirements and reinforcement development requirements as discussed in Section 4.3.4. Figure 23 shows the column interaction diagram for the designed pier.

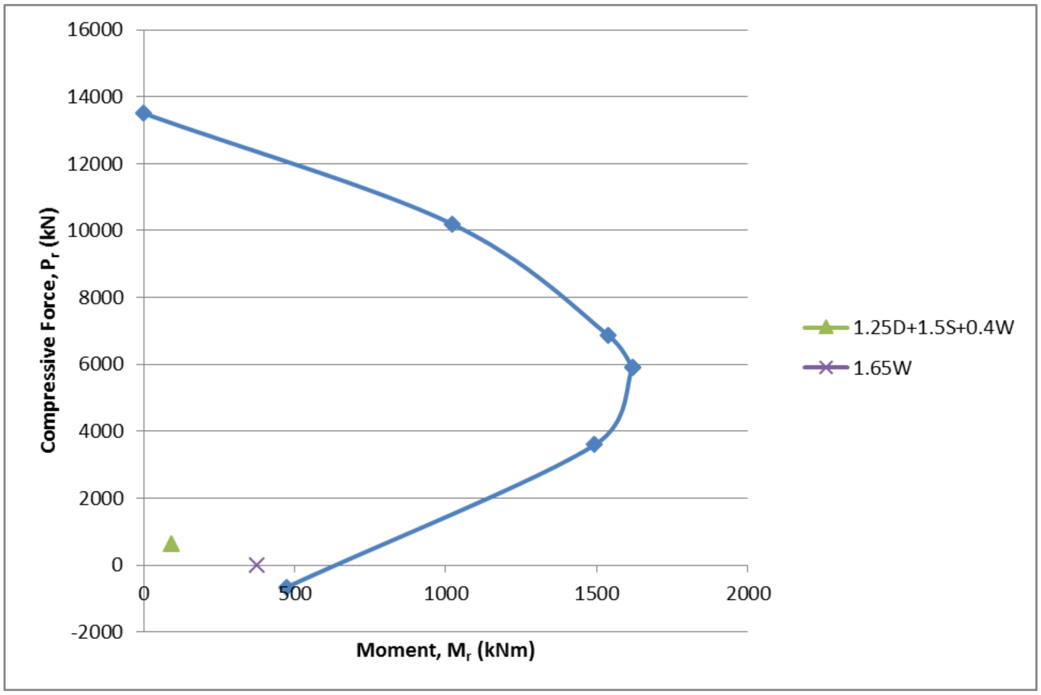


Figure 23. Column Interaction Diagram for Final Pier Design

The pier design has a cross-section of 800 mm x 800 mm and was estimated to have a height of 5 m. The column has 8-30M bars for flexural reinforcement. See Appendix H for details on this section.

4.3.2 Shear Reinforcement

The factored shear on the beam and column is governed by the lateral wind load on the bridge. The factored shear forces within these components were determined using SAP2000 and is shown in Figure 24.

Figure 24. Factored Shear in the Beam and Column under Wind Load

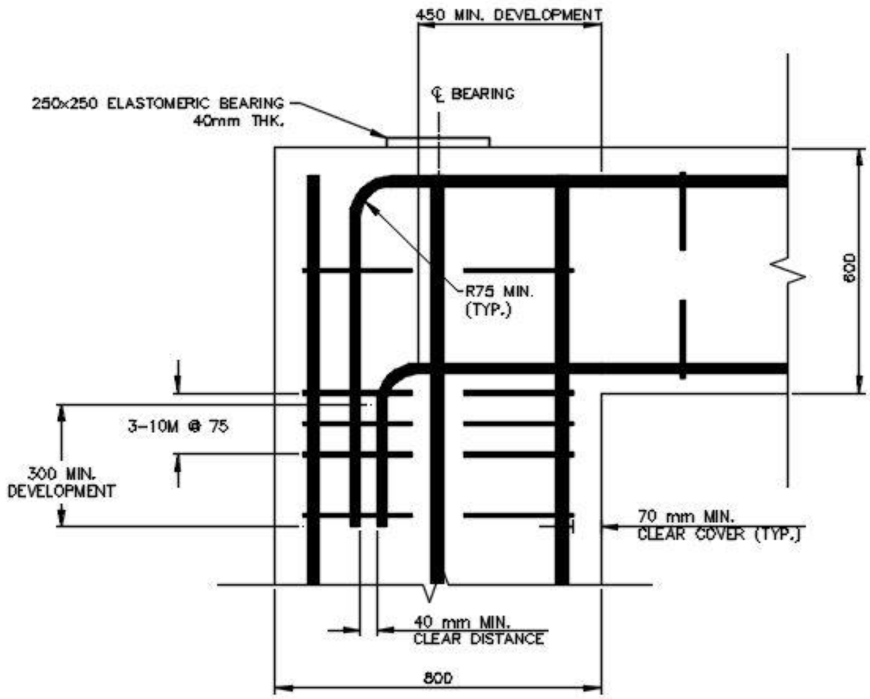
PIER COMPONENT	FACTORED SHEAR	CONCRETE SHEAR RESISTANCE, V_C
Beam	92 kN	240 kN
Column	145 kN	381 kN

The factored shear resistance of both these components were evaluated as per CSA S6-06 8.9.3.4. Since the concrete shear resistance, V_c, for both the beam and column was greater than their respective factored shear, theoretically no shear reinforcement is required for these components. Nevertheless, CSA S6-06 requires ties for flexural reinforcement. Ties for flexural reinforcement for the pier are discussed in Section 4.3.3.

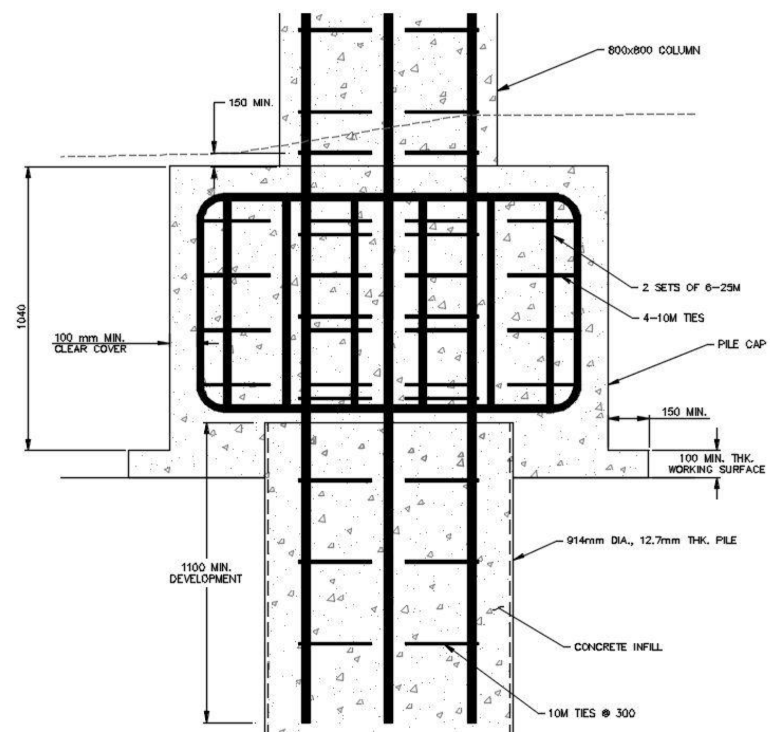
4.3.3 Transverse Reinforcement Requirements

Under the provisions of CSA S6-06 8.14.3 and 8.14.4.3, concrete components under compression and tension need to have flexural reinforcement to be enclosed by ties or an equivalent material for lateral support. According to the aforementioned clauses, the beam and concrete for Pier Option 1 need to have 10M ties with a spacing of 300 mm.

Near footings or other supports, however, the tie spacing needs to be further reduced. The ties near these locations need to be within half a tie spacing (150 mm) from the face of the footing or support. However, the tie spacing near the column-beam connection is further reduced to decrease the development length requirements for hooks as outlined in Section 4.3.4. The ties near the beam to column interface and column to pile cap interface are shown in Figure 25.



(a)



(b)

Figure 25. Connection Details: (a) Beam-to-Column; (b) Pilecap

4.3.4 Development Length

Meeting the development length requirements of reinforcing bars is important to ensure that the full capacity of the reinforcing bars are realized near the interface of other structural components. For the pier, the beam to column interface, column to pile cap interface, and the pile cap to pile interface are locations where sufficient development length was required.

Different development length requirements are outlined in CSA S6-06 8.15 for reinforcement under tension and under compression. Required development lengths for compression are typically smaller than those for tension. However, reinforcement under tension may develop by the use of hooks, which reduce the amount of development required of the bar.

Since the beam and column of the pier resist flexural loads, both tension and compression forces are resisted by the components' reinforcement. Thus, their development lengths have to be designed for both cases. CSA S6-06 8.15.6 states that the development length may consist of a hook (for tension) and combined with additional embedment length measured from the point of tangency of the hook.

Thus, in the case of the beam, both normal embedment for compression and a hook for tension was used to reduce the required development of the beam's flexural reinforcement. Even still, this design required a larger column dimension, so the column was upsized up from 600 mm x 600 mm to 800 mm x 800 mm. The formula specified in CSA S6-06 for compression development length, l_d is as follows:

$$l_d = \frac{0.1 * f_y * d_b}{f_{cr}}$$

Where,

d_b = diameter of reinforcing bar

The resulting development length is about 450 mm. The length of required development for hooks under tension is as follows:

$$l_d = \frac{0.7 * 0.8 * 40 * d_b}{f_{cr}}$$

The development length is multiplied by factors 0.7 and 0.8 to account for sufficient concrete cover and increased tie spacing, respectively. The use of the latter reduction factor is the reason for the 75 mm spacing of ties shown in Figure 25(a). The resulting minimum development length is about 300 mm.

The pile cap connection only uses the standard tension development length outlined in CSA S6-06 8.15.2.3:

$$l_d = \frac{0.18 * 1.3 * f_y * d_b}{f_{cr}}$$

The factor 1.3 corresponds to the bar location placed on top of more than 300 mm of cast concrete in the component below. The resulting development length is about 1100 mm as shown in Drawing D06.

4.3.5 Slenderness

Compression components of concrete bridge members have to be evaluated for slenderness effects. The effect of slenderness amplifies the factored moment of a given component. As such, slenderness effects were checked using approximate methods outlined in CSA-06 S6 8.8.5.3.

During this evaluation, the pier was assumed to be fixed-free, with an effective length twice its unsupported length. The unsupported length is assumed to be measured from the surface of the pile cap, and buckling was longitudinal to the bridge was assessed. The middle pier is the governing case and it was determined that slenderness effects can be ignored. See Appendix H for calculations on slenderness checks.

4.3.6 Pile Caps

The designed pile caps provide the connection between the column and the pile. Since only one pile is required to provide sufficient bearing resistance to the pier, square piles underneath the columns were assumed for design. In sizing the pile caps, the following guidelines from *Structural Engineering Forum of India* (n.d.) were used are were modified to meet requirements from CSA S6-06:

- The overhang from the pile cap to the edge of the pile is a minimum of 150 mm
- The depth of the pile cap should be sufficient to allow reinforcement development
- The clear cover of the pile cap is sufficient
- The embedment of the piles should be sufficient
- The thickness of the piles should not be less than 500 mm
- An appropriate working surface should be used.

The pile caps were designed under the assumption that the column reinforcement that runs through its depth provides no flexural resistance. This is to ensure a fixed connection between the pile cap and the column. The pile cap reinforcement were designed for minimum flexural reinforcement requirements from CSA S6-06 and CSA A23.3-14. See Appendix H for the pile cap spreadsheet. See Drawing D06 for the full pile cap dimensions and reinforcement details.

4.3.7 Piles

In the detail design phase, several design checks were done to ensure that the current pile design meets design requirements. The checks planned by Moja are the deflection due to compression, lateral deflection, downdrag, and frost heave. However, downdrag, frost heave, and lateral deflection were not evaluated thoroughly due to time constraints.

4.3.7.1 Downdrag

Downdrag is the downward force exerted by the soil on the piles due to loads on the soil. Since the soil around the bridge structure was not to be loaded, it was assumed that downdrag forces are insignificant. However, we were not able to check downdrag on the crane load during the construction phase as the crane comes in close proximity to the middle pier.

4.3.7.2 Pile Settlement

Settlement of the foundation is comprised of settlement of the soil itself and elastic deformation of the foundation. Pile settlement checks were done for both Soil Case A and Soil Case B as outlined in Table 11.

For soil Case A, settlement is assumed to comprise solely from elastic deformation of the pile since according to the CFEM (2006) the bedrock deflection is typically small in comparison. The resulting pile settlement for this soil case was only 2 mm. See Appendix I for pile settlement calculation for piles in soil Case A and B.

For piles soil Case B, pile settlement was calculated using the empirical method specified in the CFEM:

$$S = S_p + S_s$$

Where,

S_p = elastic deformation of the pile cap
S_s = settlement on the ground

The assumption is that all the load is transmitted by the pile shaft since the pile has not reached full bearing capacity due to factored loads. The resulting settlement is about 12 mm.

4.3.7.3 Lateral Deflection and Moment Capacity

Lateral deflection and Moment Capacity of the piles for soil Case A were done using Evans and Duncan's charts. These charts were derived using the p-y method using a computer program running numerous analyses (CFEM, 2006). These charts offer a quick, direct method of determining the lateral deflection and moment for a given confidence interval.

It should be noted, however, that Evans and Duncan's charts only apply to steel and concrete piles embedded into the soil at least 35 times their diameter. The piles assumed for our design does not meet this criteria. This method was used by Team Moja as an approximation since lateral analyses of piles are complex and often require specialized computer software.

A free-head condition is assumed since only single piles are used under each pier column. The calculated deflection due to lateral load and moment was done based on the governing load conditions of the middle pier from lateral wind loads on the bridge. To be conservative, the calculated deflection was multiplied by a safety factor such that the result falls within the upper limit of the 90% confidence interval specified in their charts. The results obtained from this analysis is summarized below:

Table 16. Governing Lateral Deflection and Maximum Moment of Piles

Total lateral deflection using upper limit of 90% confidence interval	25 mm
Total lateral deflection using median of 90% confidence interval	14 mm
Maximum internal moment using upper limit of 90% confidence interval	760 kNm
Associated maximum bending stress due to the maximum moment	93 MPa

The maximum bending stress exerted on the pile due to the calculated conservative deflection of 25 mm is only 93 MPa which is less than the assumed yielding strength of the pile of 400 MPa. Since the CSA S6-06 does not specify serviceability requirements for wind loads, it is assumed that the deflection calculated is acceptable.

5.0 ENVIRONMENTAL CONSIDERATIONS

As the proposed bridge alignment crosses a watercourse, it is important to identify the environmental effects imposed on the structure. Research was done on the potential hydrotechnical hazards at the project site as these hazards can compromise the structure's durability and longevity. Several hydrological hazards at the bridge crossing location impacted the design of the pipeline bridge. These include

- creek migration
- erosion
- scour.

It is also important to identify the resulting impacts of work activities to the watercourse. General Best Management Practices (BMPs) should be implemented to ensure the impacts to fish and wildlife habitats are minimized. In addition, watercourse crossings are subjected to regulatory requirements on both the federal and provincial level and the project must comply with these requirements.

5.1 Scour

Scour refers to the erosion of channel beds and/or the surrounding bank by flowing water. Scour typically occurs when bridges are built over water surrounded by erodible material. In the context of the designed pipeline bridge, the overburden of the floodplain within the 70 m design extents was assumed to be composed of fluvial sand and gravel, traces of clay and gravelly silt. As determined by a hydrotechnical and geotechnical assessment by others, scour is a major design consideration for the pipeline bridge.

According to the *Transportation Association of Canada (TAC) Guide to Bridge Hydraulics* (2001), there are four categories of scour:

- General scour
- Local scour
- Natural scour
- Channel profile degradation

General scour refers to the scour that occurs through the constriction of flood flows by the bridge structure, approach embankments, forced overbank flow, or disturbed land (Transportation Association of Canada, 2001). Local scour refers to the scour produced by water flow and turbulence around piers, abutments, or other structures. Natural scour occurs even in the absence of the bridge due to sediment transport, natural erosion, or creek migration. Finally, channel profile degradation may occur from natural geological processes or urbanization.

In the case of the pipeline bridge's design, general scour, local scour and natural scour are considered. General scour and natural scour is used interchangeably. Their combined effect is referred to in this report as 'general scour'. Channel profile degradation is assumed to have no effect given the limited geographic information of the project site.

The geotechnical and hydrotechnical assessment done by others recommended a final scour elevation of 515.7 mASL. This included a general scour depth of 0.8 m and a local scour depth of 1 m. Figure 26 shows the pipeline profile and the estimated total scour elevation.

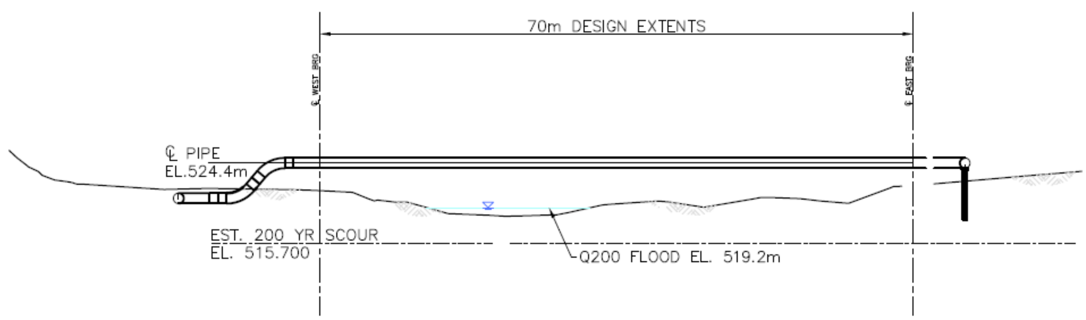


Figure 26: Pipeline Profile Showing Scour Elevation

The determination of general scour using the Blench Regime equation which, according to others, provides a conservative result. General scour and local scour are typically combined additively in design. As a conservative approximation, however, the final recommended scour depth is 2 m. As such, Team Moja implemented erosion protection measures based on this assumption. Checking the scour values provided to us was not done due to time constraints.

5.2 Erosion Protection

Various erosion protection measures were considered by Team Moja. Because of the deep scour level within the floodplain as shown in Figure 26, undermining of the designed erosion protection works is a major concern. Undermining occurs when the soil underneath the erosion protection structure erodes due to scour. Various erosion protection measures are ineffective due to undermining such as the use of grouted riprap, shallow riprap aprons, and rigid pavements.

The TAC Guide to Bridge Hydraulics offers suggested methods for protection against undermining. Table 17, below, outlines the possible solutions for undermining and our assessment of each method.

Table 17: Undermining Protection Methods

METHOD NO.	DESCRIPTION	ASSESSMENT BY TEAM MOJA
1.	Excavation and revetment of a slope down to an inerodible material or to below expected scour levels.	• Ineffective since doing so will require large excavations • Will also require excavating within the Q200 extents which Moja is not permitted to do
2.	Installation of a sheet pile cutoff wall from the revetment toe down to inerodable material or down to below expected scour levels.	• Would require tiebacks due to soil pressures once material erodes on one side of the sheetpile wall • Will typically increase local scour in that area, so a sheet pile higher than the estimated scour depth would be required.
3.	Placement of a flexible, horizontal launching apron is horizontally at the revetment toe. The apron is designed to settle on a natural slope (2H:1V) when undermining occurs preventing further erosion.	• Feasible option • Aprons should be wide enough such that the riprap extent covers up to or beyond the deepest scour limits
4.	A variant of the launching apron where a rock-filled toe trench is installed at the revetment toe.	• Similar to the launching apron but would require deeper excavations.
5.	Paving of the entire streambed with riprap or other materials. Paving should not be done above the normal stream bed levels	• would require paving within the Q200 extents • Due to the potential of creek migration, this implies paving the entire flood plain

Methods 1 and 5 were ruled out since these methods require working within the 200-year flow extents. Implementing Method 2 would imply increasing the effects of local scour and supplying sufficient anchorage across the sheet pile's entire length. Method 2 was deemed unfeasible since determining the increased effect of local scour is outside of our scope and providing sufficient anchorage along the sheet pile was uneconomical.

Method 3 and method 4 are similar in concept, but Method 3 requires less excavation. Thus, Method 3 was selected by Team Moja. The launching riprap will be designed to be placed outside the existing Q200 flood extents around the piers to account for the possibility of channel migration and undermining due to scour.

5.2.1 Riprap Sizing

The TAC Guide to Bridge Hydraulics recommends sizing riprap for the launching apron using a study done by the U.S. Army Corps of Engineers in 1991. The following equation was used to determine the required riprap size:

$$\frac{D}{y} = S_f C_s C_v C_r \left[\frac{V^2}{(s-1)K_1(gy)}\right]^{1.25}$$

Where D is the nominal rock size, considered to be D_{30} of the riprap gradation curve. This size represents the size in which 30% of the riprap mixture's mass is finer. The other variables in the aforementioned formula are shown in Table XXX along with a description of each variable and the corresponding value Moja used.

Table 18. Variables for Riprap Sizing Equation

VARIABLE	DESCRIPTION	VALUE USED
V	Local flow velocity	3.4 m/s
Y	Local flow depth corresponding to V	0.5 m
S_f	Safety factor	1.2
C_s	Stability coefficient	0.3
C_v	Velocity distribution coefficient	1.0
C_t	Thickness coefficient	1.0
K_1	Side slope factor	0.9
S	Rock specific gravity	2.65
g	Gravitational acceleration	9.81 m/s^2

Local flow velocity is related to the average velocity to the design flow. The TAC Guide to Bridge Hydraulics recommends a value equal to 80% of the average flow for straight channels, and was thus used for the calculating local flow velocity. We calculated the average flow using the following equation:

$$V_{avg} = \frac{Q}{A}$$

Where Q is the 200-year design flow of 59 m^3/s and A is the cross-sectional area of the channel of 13.9 m^2. The Q200 flow was provided from the geotechnical and hydrotechnical assessment done by others, while the cross-sectional area was roughly measured from the pipeline profile provided by Allnorth. The resulting average velocity of the creek is about 4.2 m/s. See Appendix A for the geotechnical and hydrotechnical report provided to us.

The local flow depth, y, does not greatly influence the required nominal size of riprap, but lower values should be assumed to be conservative (TAC, 2001). The local flow depth represents the design depth from which water flows against the riprap. Since the geometry of the creek is difficult to determine when creek migration occurs and the existing total depth of the channel is about 1 m, a local depth of 0.5 m was arbitrarily chosen by Team Moja for the computation of the required nominal riprap size.

The safety factor of 1.2 was used since it was recommended by TAC. The stability coefficient of 0.3 corresponds to angular riprap. A vertical velocity distribution coefficient of 1.0 was used since it is recommended for relatively straight channels. According to the Corps of Engineers, a thickness coefficient of 1.0 is recommended for standard thicknesses that equal 1.5 x D_{50}. However, the TAC guide recommends a minimum thickness of 1.75 x D_{50}. Nevertheless, the reduction in the thickness coefficient associated with the increased thickness is very little so a conservative value of 1.0 was used in design. A specific gravity of 2.65 is standard for riprap, and, lastly, a side slope factor of 0.9 was used for slopes of 2H:1V since this is the assumed natural slope of the riprap revetment upon launching.

Using Equation the riprap sizing equation, the required D_{30} size of the riprap was calculated to be about 310 mm. According to the TAC Guide to Bridge Hydraulics, the D_{50} size is approximately larger than the D_{30} size by about 25% which results in a size of 400 mm. The D_{50} size is important because the TAC guide recommends a revetment thickness 75% larger than D_{50}. The resulting required revetment thickness was then calculated to be 700 mm.

5.2.2 Riprap Gradation

The gradation of the launching revetment is also an important design consideration for erosion protection. Riprap mixtures that are too uniform often leave gaps and result in the washing away of the finer material underneath (TAC, 2001). Often, geotextiles, rock filters, or carefully placed riprap are used to prevent this from happening. However, these methods are impractical for use in launching riprap, since inadequate geotextile coverage or improper layering of riprap can result from the undermining of the launching apron.

It is recommended, therefore, that well graded to quarry-run material be used for the riprap revetment and launching apron. According to the Corps of Engineers, these mixtures are characterized by having a D_{85}/D_{15} of between 3 and 7 and more than 7, respectively. Using these riprap gradations will ensure that the finer material beneath the riprap revetment is well protected from erosion.

5.2.3 Launching Riprap Volume

The required volume of riprap is dependent on the distance of the toe of the launching apron from the lowest expected scour. Equation XXX below describes this relationship:

$$V = 1.5 * 2.24TD$$

Where V is the required volume of riprap per unit length along the launching apron, T is the required thickness of 0.7m of the launched revetment, and D is the distance from the original toe of the launching apron to the projected scour depth. The factor 2.24 can be derived using similar triangles assuming the launched slope of 2H:1V, and the factor 1.5 is a safety factor employed in the design (Ministry of Environment, 2000).

As observed from the above equation, the greater the distance from the toe of the apron to the total scour elevation, the more volume required for the launching apron. As such, Team Moja found that it was necessary to excavate to elevation 519.2 mASL for the launching aprons surrounding all the piers to reduce the amount of riprap required for each revetment. This results in the distance between toe of revetment and total scour level, D, to be 3.5m. The required volume for the launched revetment was calculated to be about 8.2 m^3 per meter width of launching apron.

5.2.4 Riprap Layout Around Piers.

The riprap revetments protecting the piers and foundations of the pipeline bridge were designed using the design parameters established in the previous sections. Figure 27 shows the layout of the revetment close to the west pier which is also typical of the east pier.

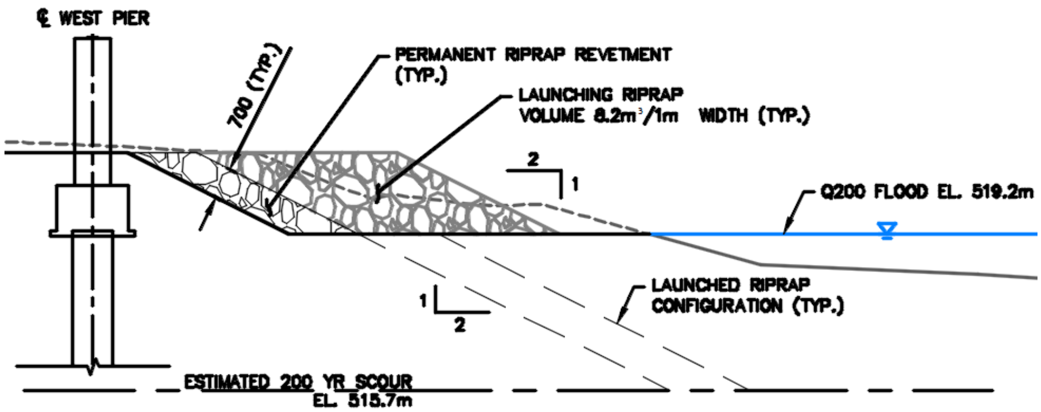


Figure 27: West Revetment Looking North

The design launching apron is outlined in 50 shades of grey, while the permanent riprap revetment is outlined in black. The elevation of the launching riprap volume may vary so long as the volume requirements are met. The dashed line shows the intended configuration of the launched riprap. The permanent riprap layer serves as additional protection in case the launching riprap layer partially launches and the creek manages to contact the permanent layer.

It is also important to note that the excavation elevation is roughly the same as the Q200 extents. Care must be taken to ensure that the Q200 extents is not disturbed during construction of the bridge or during the placement of riprap. This was done through the use of silt fences around the applicable areas on the site.

A similar launching riprap design is to be implemented around the middle pier. The launching apron layout surrounding the middle pier is shown on Figure 28 below.

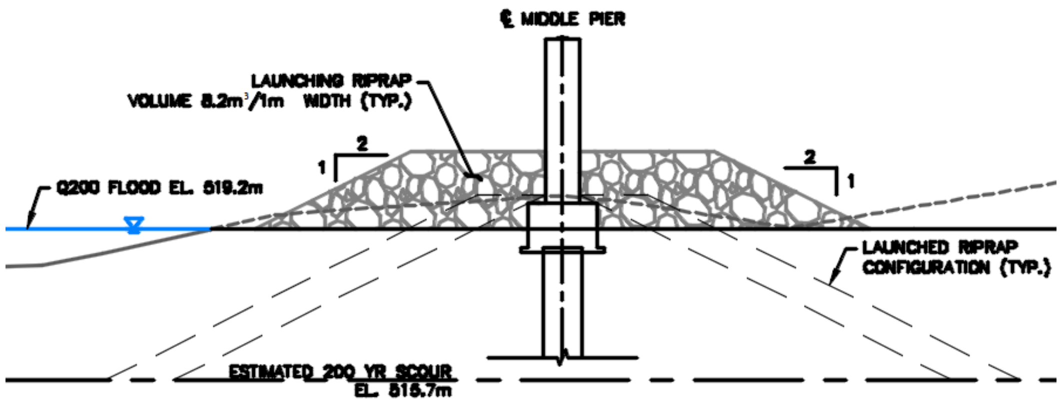


Figure 28: Middle Launching Apron Looking North

Similar to Figure 27, Figure 28 shows the launch riprap volume and the launched configuration for the middle pier. Due to close proximity to the Q200 extents, no permanent riprap revetment can be utilized near the proximity of the middle pier. Additional monitoring of the bridge scour and creek is required to ensure that sufficient riprap is present for substructure protection when undermining occurs on the middle pier location.

To summarize, Table 19 outlines Team Moja's recommendations regarding the riprap launching apron and revetment.

Table 19. Recommended Specifications for Riprap Design

SPECIFICATION	RECOMMENDATIONS
Riprap slope	2H:1V
Riprap size, D_{50}	400 mm
Riprap gradation (D_{85}/D_{15})	3-7 or above 7
Permanent riprap revetment thickness	700 mm
Elevation of launched riprap toe upon placement	519.2 mASL
Launched riprap volume	8.2 m^3/m

See Appendix L for the final design drawing package including the riprap design.

5.3 Water Act

The Water Act is the main provincial statute regulating water resources in BC.

Aerial pipeline crossing must be constructed in accordance with the requirements prescribed in paragraph 44(b) of the *Water Act* (MoE 2015). This includes

- the bridge and its approach roads do not produce a backwater effect or increase the head in the stream,
- the equipment used for construction, including site preparation, maintenance or removal of the bridge, is situated in a dry stream channel or is operated from the top of the bank,
- the hydraulic capacity of the bridge is equivalent to the hydraulic capacity of the stream channel, or is capable of passing the 1 in 200 year maximum daily flow, and the height of the underside of the bridge is also adequate to provide free passage of flood debris and ice flows, and
- the bridge material meets the standards of the Canadian Standards Association, as applicable.

Since the existing forestry road lies outside the given topography that was provided, it is assumed that these roads do not induce a backwater effect on the stream. As well, none of the bridge components and any construction works are to be done within the Q200 flow. Therefore, no immediate backwater effects can be produced by the bridge.

All work activities were planned to situate construction equipment outside of the Q200 flow and the pipeline elevation was specified at 524.4 mASL and is capable of passing the 200-year flow.

5.4 Regional Timing Windows

General Best Management Practices (BMPs) were followed to minimize the potential negative impacts to fish and fish habitats. Regional timing windows is a practice to protect fish from the impacts of construction activities during critical life history stages of fish such as spawning migration seasons. Thus, works in and around watercourses must be accomplished during times of the year when the possible negative impacts to fish are low, minimizing the risk from work activities.

The Ministry of Water, Land and Air Protection (MWLAP) has produced documents regarding the implementation of timing windows. Timing windows were identified from the Skeena-Stikine Forest District in the Skeena Region.

Four fish species were identified to be present at the project watercourse and are listed below.

- Rainbow trout
- Northern pikeminnow
- Prickly sculpin
- Peamouth chub

Although four species were identified to be present, rainbow trout is the only species with information available on timing windows for the Skeena Region. Thus, it was assumed using the reduced risk work window for rainbow trout will be sufficient in the consideration of the remaining species in the watercourse. The reduced risk work window for rainbow trout is from September 1 to January 31 and it is recommended that the work be carried out in this time frame.

6.0 CONSTRUCTION PLAN

Construction work activities must be determined to ensure a feasible construction plan at the project site. As work activities are planned, any problem areas can be identified before the construction phase proceeds. As the project site is an environmentally sensitive watercourse, actions must be taken during the planning, design, and construction of works to minimize adverse effects to the watercourse and the surrounding natural environment. Team Moja has created a recommended construction plan to address these problems through a construction sequencing and sedimentation plan. In addition, an excavation and lifting plan were designed for the design project. Each of these plans are further discussed in the following sections. See Appendix M for the construction drawing package.

6.1 Construction Sequence

To begin construction planning, Team Moja determined the necessary construction steps and the recommended sequence for the work activities. The construction tasks and the steps to completion are outlined on the following page in Table 20.

Table 20. Sequence of Construction Work Activities

TASKS	STEPS
Earthworks	1. Clearing and grubbing 2. Flag excavation extents and stockpile location 3. Sediment control measures surrounding excavation and stockpile 4. Excavate to designed depth for installation of piles
Truss construction	1. Material delivery 2. Truss construction
Pile installation	1. Layout pile location 2. Drive pile 3. Fresh head pile (as required) 4. Splice pile (as required) 5. Drive pile to depth 6. Cut-off pile to design elevation
Pile cap construction	1. Construct formwork 2. Construct rebar 3. Pour concrete 4. Allow curing to working strength (~3 days) 5. Remove formwork 6. Allow curing to sufficient strength prior to pier concrete pour
Pier construction	1. Construct formwork and falsework 2. Construct rebar 3. Pour concrete 4. Allow curing to working strength (~3-4 days) 5. Remove formwork 6. Allow curing to sufficient strength prior to bridge installation
Riprap installation	1. Place rip-rap berm 2. Place rip-rap revetment
Construct crane pads	1. Sedimentation control surrounding crane pad 2. Grade area flat 3. Build up material to flatten (if required) 4. Compact material
Hoist truss into place	1. Position crane onto crane pad 2. Hook onto truss 3. Hoist truss 4. Swing truss into position 5. Lower truss unto bearings
Walkway grating installation	1. Hoist grating into place
Pipeline installation	1. Launch pipe segments into position
Pressure testing	To be done by others

Table 20 outlines the major bridge construction tasks. Moja recommends the following sequence of these construction steps.

Table 21. Construction Sequencing Tasks

TASKS	CONCURRENT TASKS
1. Excavation	
2. Pile installation	Truss construction
3. Pile cap installation	
4. Pier installation	
5. Rip-rap launching ramp installation	Crane pad construction
6. Truss installation	
7. Walkway grating installation	Rip-rap installation
8. Pipeline installation	
9. Pressure testing	

After reviewing the above construction steps and sequence, Team Moja identified the construction lift and the excavation as key construction steps and outlined them in construction sequence drawings shown in Appendix M.

6.2 Construction Excavation

Excavation within the floodplain on the project site has to be done because of riprap placement requirements, substructure construction and crane pad construction. Silt fences shall be installed at key locations to ensure no sediments are washed into the creek. Refer to Drawings C01 and C02 for work-points for excavation, stockpile, silt fences, and pile installation.

Initial excavation shall be done to el. 519.2 mASL for riprap placement and for the crane pad. All excavation slopes are assumed to be 2H:1V. Team Moja modelled the excavation in Civil3D to obtain approximate material quantities and are outlined in Table 22.

Table 22. Excavation Volumes to 519.2 mASL

West Excavation	150 m^3
East Excavation	700 m^3

The east excavation is much greater than the west excavation since the east excavation accounts for the middle pier, east pier, and the crane pad. Excavated material will have to be hauled into the stockpile location through the existing forestry road and into the stockpile location located in a flat area northwest of the site. The stockpile partially lies outside the given topography, but is assumed to be relatively flat and unobstructed based on nearby topographic information.

The stockpile capacity is about 1300 m^3, which is accounts for 1.5 times the estimated excavation volume. This increase in volume accounts for the unconsolidation of disturbed soil after excavation.

After the initial excavation has been done, excavations for the foundations can begin. Volumes for foundation excavation were obtained using Civil3D. The excavation elevations for the bottom of the foundations can be found in Drawing C01 and D05. Table 23 shows the excavation volumes for the foundations of the piers.

Table 23. Excavation Volumes for the Pier Foundations

West Pier	130 m^3
Middle Pier	30 m^3
East Pier	140 m^3

Construction of the piers can begin after the aforementioned excavations are finished. These excavations will have to be backfilled with native material up to el. 519.2 mASL before the construction lifting phase can begin.

6.3 Construction Lifting

Team Moja's construction plan for the truss design was to construct the truss on the ground and lift the constructed truss into place. This construction method was chosen to limit the required falsework and minimize impact to the floodplain. This construction technique does not require falsework to support sections of the partially constructed bridge which would impact the flood plain.

The first step to determining whether or not this construction step would be feasible and to determine the level of site access and the largest size crane that can be transported to site. From speaking with the sponsor, a 150 Ton crane is the largest size crane that can be used without additional site access upgrades. A Kobelco SKS1350 was chosen.

The second step in determining the feasibility of this construction technique was to determine the lifting capacity of the above crane at the required radius. The lifting capacity of this crane at 18 m was 26 400 kg. The long span of our bridge design weighs 18 900 kg. This weight accounts for a factor of 1.5 which was recommended by our sponsor. Once it was determined that a feasible size crane could lift the long span of the truss from outside the Q200 flow area, the truss was examined to determine whether or not it could handle the stresses acted upon it from lifting. This is further explained in Appendix K.

To determine the stresses imposed onto the truss from lifting, SAP2000 was used. To determine the stresses imposed using SAP2000, the SAP2000 model from the structural design was modified. The model was modified such that the supports were on the top and that the truss was resisting its self-weight multiplied by a factor of 1.5. Using this analysis, it was determined that the stresses imposed due to lifting were minimal compared to the ultimate allowable stresses.

6.3.1 Lifting Lug Design

The final step to construction lifting was the design of a lifting lug to lift the truss from the ground. No literature governing this design was readily available and basic principles and the S16-09 were used to perform this design. However, this design yielded an unrealistically small lifting lug which raised concerns of its validity. Team Moja then began searching for prefabricated lifting lugs.

Several prefabricated lifting lugs were compared against the design completed by Team Moja, which confirmed our concerns with the validity in our design. Team Moja's design was approximately one third the size of the prefabricated lifting lug. Due to time constraints, Team Moja modified the truss to suit the prefabricated lifting lug. This is shown in Appendix K.

The truss wall thickness was not sufficient for the prefabricated lifting lug so a plate must be welded to the truss to reduce the stress in the truss chord members. The lifting lug manufacturer requires a minimum thickness. For this reason, a plate of the minimum thickness that is over double the area of the lug was used to halve the stress imposed upon the chord member walls.

6.3.2 Crane Mat Design

A major concern when performing critical crane lifts is that the crane has a stable and level lifting surface. Cranes have a very small allowable tolerance from plane when performing lifts to ensure safety and prevent damage to the equipment from the increased stress on the rotation motors. For these reasons, Team Moja designed crane mats to ensure the stability of the crane during the lifting of the trusses. This process is outlined further in Appendix K.

To determine the required size of the crane pads the bearing capacity of the in-situ soil must be determined. Site soil investigation performed by others were minimal. Through this investigation, the site soil was shown to be fluvial sand and gravel. For the design of the crane lifting pad, it was assumed that the soil is homogenous throughout the excavation depth. Using this assumption, a conservative bearing capacity of the soil was determined from *Reinforced Concrete Design a Practical Approach* as well as past experience of 100 kPa. This bearing capacity was taken from the silty gravel or clayey gravel bearing capacity in Table 14.1. This bearing capacity was taken because fluvial sand is small grain sand from a river or water course, which would have similar physical behavior to sandy gravel and silty gravel.

The second piece of required information to determining the size of crane pads was the pressure exerted on the soil during the critical lift. To determine this the non-lifting pressure was first determined from the manufacturer's specifications. This was then added to the additional pressure caused by the weight of the hoisted object. This pressure was determined to be 136 kPa.

Since soil bearing capacity is less than the pressure exerted by the crane during the critical lifts, crane pads will be required to be used. A required area of crane mat and typical sizes are specified in Appendix K and summarized below in Table 24.

Table 24. Crane Mat Design Summary

Soil Bearing Capacity	100 kPa
Critical Lift Pressure	136 kPa
Required Crane Mat Area	14 m^2

To achieve the required crane mat area shown in the previous table, four crane mats of 15' x 5' can be used. This is a typical crane mat size constructed by manufacturers.

7.0 CONCLUSION

Team Moja has completed the design of the pipeline bridge meeting structural, environmental, and construction requirements.

In the design of the superstructure, trusses were deemed the most suitable option amongst the possible bridge systems. An analysis of alternatives was performed and it was found that the Warren configuration provided the most economical solution while meeting the deflection requirements specified by the pipeline designer. The truss was comprised of square and rectangular HSS sections to efficiently resist the factored loads. In addition, lateral members were designed to resist lateral torsional buckling and to stabilize the structure under ultimate loading conditions. Team Moja recommends the superstructure to be further analysed to account for seismic conditions.

To support the bridge superstructure, three piers were designed within the 70 m span. Two pier options were analysed and sized for flexural stress during the conceptual and preliminary phase. It was found that Pier Option 1 was more economically viable due to lesser concrete volume requirements. Reinforcement details for the piers and pile caps were designed according to CSA A23.3-14 and CSA S6-06. Furthermore, it was determined that slenderness effects can be ignored.

The pile size and spacing was recommended by others, and Team Moja checked the compressive bearing resistance that was provided. The compressive bearing capacity of 1300 kN was confirmed and used for the design of the piles. In the detailed design phase, the pile settlement and lateral deflections were determined to be minimal. In addition, downdrag was assumed to have negligible effects on the piles. As the project site is susceptible to freeze-thaw cycles, frost heave effects on piles should be further investigated.

To mitigate scour effects such as erosion and undermining, Team Moja considered various methods for erosion protection. Launching riprap was determined to be the most feasible option due to potential creek migration and undermining. As the creek is environmentally sensitive, Team Moja recommends to follow the regulatory requirements outlined in the *Water Act*. In addition, BMPs should be implemented to minimize adverse effects to fish and wildlife habitats such as using regional timing windows for work activities.

A construction plan that outlined key work activities for constructing the pipeline bridge was recommended by Team Moja. Excavation extents, the proposed stockpile location, sedimentation plan, a lifting plan, and the recommended construction sequence were laid out. These areas are aimed to reduce cost to the client by providing a feasible design with the critical items already planned.

8.0 EPILOGUE

The following section offers our thoughts with regards to CIVL 7090: Capstone Design Project.

During this course we were tasked with the job of finding an industry sponsor, similar to the industry project in second year, who could provide us with a multi-disciplinary civil engineering project that contained aspects of sustainability and large enough for "140 hours" per team member. Like many of you at the beginning of this project, we were bright-eyed and bushy-tailed, ready to take on the task at hand. However, once we set off into the task of creating a proposal for our newly acquired project and then designing this project in second semester we realized that we had no idea how to complete this project. But don't worry with some helping hands and many words of encouragements you will complete your preliminary scope and proposal.

Now that the true task is upon you to complete what you have so eagerly proposed to complete the real challenge begins. At this point you are probably already behind because you did not put in the hours you eagerly allotted yourself over the Christmas Break. But don't worry there will be lots of time to make up those hours in the last few days when your efficiency is at its highest. Now back to the real challenge that I mentioned earlier. You may think now that the largest challenge of this project is the design itself. However, it is not. The biggest challenges of this project are

- Keeping on task and on schedule
- Unforeseen time sucks such as report writing, editing, and formulation.

Not to scare the next batch of Capstone targets but let this come to you as a warning of the next few months to come. These months will be busy, challenging, rewarding, frightening, and intimidating but you will succeed and complete your Capstone project. So, in conclusion this project provided us with many challenging tasks in and outside of the designing of the project and the opportunity to test what we have learned through the past four years at BCIT.

REFERENCES

Brzev, S., & Pao, J. (2009). *Reinforced Concrete Design A Practical Approach.* Toronto: Pearson Prentice Hall.

Canadian Institute of Steel Construction. (2010). *Handbook of Steel Construction.* Canada: Canadian Institute of Steel Construction.

Canadian Standards Association. (2006). *CSA S6-06: Canadian Highway Bridge Design Code.* Canadian Standards Association.

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

Canadian Standards Association. (2014). *CSA A23.3: Design of Concrete Structures.* CSA Group.

CISC. (1995). *Hollow Structural Sections Questions*. Retrieved February 2015, from CISC: http://www.cisc-icca.ca/solutions-centre/technical-resources_list/frequently-asked-questions/hollow-structural-sections-questions/?lang=fr-CA

Coduto, D. P. (2001). *Foundation Design Principles and Practices.* Saddle River: Prentice-Hall.

CPP; CEPA; CGA. (2005, October). *Pipeline Associated Watercourse Crossings 3rd Edition.* Retrieved from CEPA: http://www.cepa.com/wp-content/uploads/2014/01/Pipelines-Associated-Watercourse-Crossings.pdf

Fletcher, B. (n.d.). *Truss Connections.* Retrieved February 2015, from Atlas Tube: http://www.atlastube.com/video/hss-connections/truss-connections

Krentz, H. A. (1996, November). *Substituting HSS A500 for G40.21.* Retrieved from CISC: http://www.cisc-icca.ca/solutions-centre/technical-resources/technical-resources/steel-supply/substituting-hss-a500-for-g40-21

Ministry of Environment, Lands and Parks. (2000). *Riprap Design and Construction Guide.* Province of British of Columbia.

Ministry of Forests, Lands and Natural Resource Operations. (n.d.). *Resource Stewartship*. Retrieved January 2015, from Ministry of Forests, Lands and Natural Resource Operations: http://www.env.gov.bc.ca/wsd/water_rights/licence_application/section9/

Ministry of Water Land and Air Pollution. (2005). *Reduced Risk In-stream Work Windows and Measures.* Ministry of Water Land and Air Pollution. Retrieved from http://www.env.gov.bc.ca/wsd/regions/ske/wateract/work_windows_measures_030205.pdf

Office of Housing and Construction Standards. (2012). *British Columbia Building Code .*

Packer, J. A., & Henderson, J. E. (1997). *Hollow Structural Section Connection and Trusses.* Alliston: Universal Offset Limited.

Shimkus, M. R. (2011). *Effects of Chord Face Penetration by Branch/Web Member in Rectangular Hollow Section (RHS) Truss Joint.* Ames: Iowa State University of Science and Technology.

Structural Engineering Forum of India. (n.d.). Pile Cap Design. Retrieved February 2015

The Canadian Geotechnical Society. (2006). *Canadian Foundation Engineering Manual.* Canada: The Canadian Geotechnical Society.

Transportation Association of Canada. (2001). *Guide to Bridge Hydraulics.* Ottowa: Transportation Association of Canada.

Wikipedia. (2015, March). *Hazelton, British Columbia.* Retrieved from Wikipedia: http://en.wikipedia.org/wiki/Hazelton,_British_Columbia