Alloy Consulting

Doing it Right

March 2015

Civil Engineering Capstone
Design Project

Report No. CECDP – 2015/01 – 01

Volume I – Client Report

Clearbrook interchange Improvement Project

Abbotsford BC

By: Alloy Consulting Ltd.

Department of Civil Engineering

School of Construction and the Environment

British Columbia Institute of Technology

Burnaby, BC, Canada V5G 3H2

COVER PHOTO:

HTTP://WWW.FOTOLIA.COM/CONTENT/COMP/41425187

Alloy Consulting
Doing it Right

March 16, 2015

Civil Engineering Capstone Design Project

Submitted To:
Mr. Martin Bollo MEng, PEng, PE, SE, LEED (r) AP
Faculty – Department of Civil Engineering
British Columbia Institute of Technology
3700 Willingdon Avenue
Burnaby, BC   V0N 3A2
e. Martin_Bollo@bcit.ca

Copied To:
Mr. Paul Deol PEng
Division Manager, Design Build P3
McElhanney Consulting Services Ltd
Suite 2300 Central City Tower,
13450 - 102nd Avenue
Surrey, BC   V3T 5X3
e. pdeol@mcelhanney.com

REPORT NO. CECDP – 2015/01 – 01

CLEARBROOK INTERCHANGE IMPROVEMENT PROJECT

ABBOTSFORD BC

Submitted By:
Alloy Consulting Ltd.
3700 Willingdon Avenue
Burnaby, BC V0N 3A2

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

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.

Alloy Consulting Ltd.
3700 Willingdon Avenue
Burnaby, BC V0N 3A2

March 16, 2015

BCIT Civil Engineering Department
3700 Willingdon Avenue
Burnaby, BC   V0N 3A2

Attention: Capstone Committee

Subject: Clearbrook Interchange Improvement Project

We are pleased to submit *"Report No. CECDP – 2015/01 Clearbrook Interchange Improvement Project, Abbotsford, BC"* in response to fulfilling the requirements of the CIVL 7089/CIVL 7090 – Capstone Design course.

The project described in the attached design report consists of the design for two new roundabouts, an overpass, the geotechnical analysis of the site, the drainage plan, and the construction sequencing plans. We, the undersigned, declare that the work presented in this draft submission is entirely original, and has been done by us, independently as well as together, so each of us has a complete understanding of the calculations and results presented.

Sincerely,

Alloy Consultants Ltd.

Logan Brown	Jacob DeVos	Keith Dodgshon	Jeff Ferraby	Matt Hackett

March 16, 2015

Date

ACKNOWLEDGEMENTS

Alloy Consulting Ltd. would like to thank the following people for their help and guidance throughout the completion of this project.

Paul Deol	For providing our team with the opportunity to work on this project.
Martin Bollo	For providing guidance through the proposal and design phases of the project. Additional thanks for taking our team out for drinks next Friday; you're a class act.
Paul Thurston	For providing guidance in the decision-making process of the transportation design and for providing general transportation guidance throughout the project.
Colleen Chan	For informing us about and providing us with access to the CES Edupack 2014 software which was very helpful in the environmental assessment of the bridge girder selection process.
Renata Wood	For providing assistance and guidance in the geotechnical aspects of the project.
Steve Boon	For providing amazing guidance in the use of Civil 3D. Our drawings never could have come together without your help.
Our Significant Others	For being so understanding when we disappeared for days at a time to work on this project. We are sure that you are all very happy that this is finally finished.

Executive Summary

We, Alloy Consulting Ltd., have prepared the following client report in response to fulfilling the requirements of the CIVL 7089/CIVL 7090 – Capstone Design course. Additional requirements for this project were provided by our sponsor Paul Deol, PEng, of McElhanney Consulting Services Ltd., and by City of Abbotsford and the BC Ministry of Transportation.

The Clearbrook Interchange, in Abbotsford, BC, is in major need of improvements to increase the capacity of the interchange and to improve safety for its users. The existing interchange features a two-lane overpass with conventional intersections on each end. This system is insufficient to satisfy traffic flow requirements for the future. In addition to the overpass and intersections, the existing deceleration lanes and freeway off-ramps are too short and tight, respectively, and must be improved for the safety of the interchange's users.

Our work for this project consists of five primary area of work relating to the improvement of the Clearbrook Interchange in Abbotsford, BC: geotechnical analysis, transportation design, bridge structural design, construction management, and environmental design.

The geotechnical analysis of the Clearbrook Interchange site included consideration for surface and subsurface conditions. We conducted an analysis of the surface conditions of the site based on site photos provided by McElhanney Consulting Services Ltd. Subsurface conditions for the site of the Clearbrook Interchange were determined from analysis of borehole, test-pit, and percolation test data.

Recommendations based on the analysis of surface and subsurface conditions were prepared and summarized in a complete geotechnical report. The report provides recommendations and design parameters for the following: seismicity, excavations and temporary drainage, infiltration ponds, site preparation and backfilling, foundation design, and retaining walls.

The transportation design of the Clearbrook Interchange consists of two design stages: preliminary and detailed. In the preliminary design stage of the Clearbrook Interchange, we completed three different design options to be compared against each other so that we could determine the option the best satisfied our selection criteria.

Before we could perform the three preliminary designs, several other tasks related to the transportation design were required. The first task was the import of survey data into Civil 3D. Using the eastings, northings, and elevations provided to us by McElhanney Consulting Services Ltd., a 3-dimensional surface was created from which we could base our transportation model.

After we prepared the 3-dimensional model in Civil 3D, we relocated the Trans-Canada Highway to allow for longer deceleration lanes and larger off-ramps. We designed the freeway realignment to satisfy requirements for horizontal alignment, vertical alignment, and superelevation, as specified in *AASHTO 2001 Standards*, *1999 TAC Geometric Design Guide*, and the *2007 Supplement to the TAC Geometric Design Guide*.

The first transportation option (Option A) that we designed includes a conventional intersection on the north side of the interchange and a roundabout on the south side of the interchange. This design option is similar to the design produced by McElhanney Consulting Services Ltd. when the project was initially completed. As required by the City of Abbotsford and MoTI, the bridge was designed with 6 lanes – 3 northbound and 3 southbound. From this, and the projected traffic volumes for 2040, each intersection was designed. The roundabout intersection is two lanes in size, with right-hand slip lanes for three of the four approaches. The conventional intersection was designed to be as efficient as possible, but we could not create an arrangement that met 2040 traffic volume requirements.

The second preliminary transportation design (Option B) that we prepared for the Clearbrook Interchange includes roundabouts at the north and south sides of the interchange. The south intersection in this design option is identical to the roundabout intersection design in the first option. To create a roundabout intersection on the north side of the interchange, land must be acquired from the adjacent businesses. After we assumed that the required could be acquired, we designed the intersection. The intersection is nearly symmetrical in all directions, with a two-lane roundabout, similar to the south intersection.

The final preliminary transportation design (Option C) that we considered for the Clearbrook Interchange is a diverging diamond interchange. In this type of interchange, traffic approaching the overpass switches to the opposite side of the road before entering the bridge. By changing sides of the road like this, all turns across oncoming traffic are eliminated. We created the horizontal alignment of the diverging diamond interchange in Civil 3D. From this alignment, it was observed that replacing the existing interchange with a diverging diamond interchange would require the acquisition of a large amount of land. Additional concerns of the diverging diamond interchange include construction of the intersection on either side of the overpass and needing to eliminate access from the interchange to Marshall Road.

To select the best interchange design, we compared the three design options for cost, traffic capacity, construction time/ sequencing, and drivability. Our comparison of the three design options showed that Option B was the best alternative, performing the best in all 4 comparison categories. Option C had the next best score, though it had a failing cost criterion. We scored Option A the lowest, primarily because it did not satisfy the 2040 traffic volume requirements required by the City of Abbotsford and MoTI.

Having compared the three design options in the four categories described above, we determined that the double roundabout interchange (Option B) was the best design solution. Having selected Option B as the premier design, we proceeded to developing a detailed transportation design of the selected interchange. The detailed design of the interchange consists of horizontal and vertical alignments, superelevations, intersections, and other design components.

The purpose of the detailed design was to prepare a 3D model and drawing set that displays all relevant design information. We created the design drawings in accordance with requirements specified by MoTI. Included in the detailed design are the following components: roadway alignments (horizontal and vertical), roundabout layouts, pedestrian and cyclist access, and road barriers.

Horizontal alignments for the interchange were prepared by following requirements specified in Transportation Association of Canada (TAC) guidelines. We referred to these guidelines for determining curve radii of the Clearbrook overpass and on-ramps and off-ramps. Merging lanes and tapers were also determined by referring to guidelines provided by the TAC.

The vertical alignment of the interchange is most critical for the Clearbrook Road overpass. In designing the vertical alignment of the overpass, we considered the following:

- Maintaining a minimum clearance of 5.5 m above the freeway
- Ensuring a maximum grade of 6% as required by the TAC

Using the above considerations, the vertical alignment of the overpass was developed. In addition to the overpass vertical alignment, we also designed the vertical alignments of the on-ramps and off-ramps.

The roundabout intersections were designed according to *NCHRP Report 672 – Roundabouts: An Informational Guide Second Edition*. We designed the roundabouts with inscribed diameters of 60 m. To maximize the efficiency of the intersections, right-turn slip-lanes were provided. The roundabout pavement is sloped towards the centre of the intersection at a slope of 2%.

Pedestrian access was provided with crosswalks at each roundabout intersection. We also included a sidewalk on one side of the bridge to allow pedestrians to safely cross the bridge. We considered cyclists in the design by providing 4.3 m wide curb lanes for the overpass. The widened lane will allow for shared use between cyclists and motorists. Signs will be provided to inform road users of the shared lane. Incorporated into our design are bicycle ramps at the entrance to the roundabouts. These ramps will allow cyclists to dismount and use the sidewalk instead of using the roundabout.

To provide the client with the best possible solution for the Clearbrook Interchange Improvement Project, we conducted a preliminary structural design of three different bridge girder options. The three designs that we considered were steel plate girders, precast concrete I-beams, and precast concrete box girders. The girders were designed in accordance with *CSA S6 – 06,* considering flexural, shear, bearing, and serviceability requirements.

The first element of the bridge structure that we designed was the cast-in-place concrete bridge deck. The bridge deck was designed using the empirical design method specified in *CSA S6 – 06*. By following the design process, we determined that a 215 mm thick deck reinforced with 10M @ 300 mm top and bottom, each way, is required.

The designed steel plate girder consists of a 1300 mm x 20 mm web with 500 mm x 38 mm flanges. This design satisfies requirements for flexure and shear. For bearing, we determined that 200 mm x 19 mm bearing stiffeners are required at each support, on each side of the web. To allow for the consideration of composite action between the steel plate girder and the concrete deck, we designed shears studs to transfer the interface shear. The resulting design, however, did not quite satisfy live load deflection requirements specified by *CSA S6 – 06*.

The precast concrete I-beam was designed to be 1350 mm deep. Using this design, we determined that 18 – 25M reinforcing bars plus 60 – size 13 post-tension tendons are required in the bottom flange. Additionally, 27 – 25M reinforcing bars are required in the top flange of the beam. We designed the beam to resist shear forces in the web and at the interface with the cast-in-place bridge deck. The deflection experienced by the designed concrete I-beam was approximately three times the limit specified by *CSA S6 – 06.*

For the precast concrete box girder, we only designed the outside girder since it experiences that largest level of loading. Similar to the precast concrete I-beam, the section is 1350 mm deep. The bottom flange of the girder is reinforced with 100 – 25M reinforcing bars. Prestressing of the concrete was not considered in this design. Shear reinforcing was designed for the girder and located in each of the three webs. Like the precast concrete I-beam design, the deflection of the precast concrete box girder design was more than the permitted limit specified by *CSA S6 – 06*.

The three preliminary structural designs were compared to determine the best option. Four different design criteria were compared: function, cost, constructability, and sustainability. From the comparison of the structural design options, we determined that the steel plate girder design was the best alternative. The precast concrete box girder was scored very closely behind the steel plate girder option.

A cast-in-place concrete, full cantilevered bridge abutment was designed to support the loads associated with the Clearbrook bridge overpass. The abutment was designed to meet the criteria set out in the *Canadian Highway Bridge Design Code* (S6-06). The design ensured that the abutment satisfied bearing capacity, settlement, sliding, and overturning resistance requirements.

Since the existing Clearbrook Interchange does not have any storm water management system, we developed a storm water management design to control runoff at the Clearbrook Interchange. Our drainage design consists of two main elements: storm mains and infiltration ponds. Storm mains were designed to meet the 1 in 25-year storm, according to the *BC Supplement to TAC Geometric Design Guide*.

The infiltration ponds were designed using infiltration rates determined from the geotechnical analysis. Two ponds were designed, one within each off-ramp of the interchange. The infiltration ponds were designed to drain within 24 hours of the design storm event. To meet this requirement, we determined that 1.5 m of existing material would need to be removed from the surface of each pond and replaced with 1.5 m of clear sand.

The final portion of work that we completed was a construction sequencing plan for the construction of the new interchange. In our sequencing of the interchange construction, we determined that there would be nine major construction phases, followed by the removal of the existing overpass: (1) prepare Highway 1 relocation, (2) build bridge pier, (3) complete Highway 1 relocation, (4) construct abutments and approaches, (5) construct bridge deck, (6) construct on and off-ramps, (7) construct infiltration ponds, (8) build intersections, and (9) install storm mains. A preliminary schedule for each of these construction phases was also prepared.

TABLE OF CONTENTS

Logan Brown Jacob DeVos Keith Dodgshon Jeff Ferraby Matt Hackett

Logan Brown Jacob DeVos Keith Dodgshon Jeff Ferraby Matt Hackett

LIST OF ILLUSTRATIONS

FIGURES

TABLES

1.0 INTRODUCTION

Alloy Consulting is pleased to present the following design report for the Clearbrook Interchange Improvement Project. Throughout the completion of the work described in this report, our team of experienced staff maintained the goal of providing the City of Abbotsford and the BC Ministry of Transportation and Infrastructure (MOTI) with the most efficient and sustainable design.

The following report provides a detailed summary of our work methodology, results, and recommendations. Further to the information provided within this report, printouts of the complete design drawings, calculations, and additional reports have been included in *Volume II – Appendices* and *Volume III – Drawings*. Digital files for design spreadsheets can be found on the USB drive attached to the physical copy of this report.

We believe that the work contained within this report will satisfy the requirements set out by the City of Abbotsford, MOTI, and the Capstone Project Committee.

1.1 Project Background

The existing Clearbrook Interchange (Figure 1) is in major need of improvements to increase both the capacity and safety of the interchange.

Figure 1 - Existing Clearbrook Interchange (McElhanney Consulting Services Ltd., 2008)

As shown in the above figure, the existing interchange consists of a two-lane overpass with conventional intersections on either end. The existing westbound off-ramp's turning radius is small and only adequate for a 30 km/h design speed. Furthermore, the eastbound deceleration lane is insufficient in length. These deficiencies contribute to an interchange that is both inefficient and unsafe.

The City of Abbotsford and MOTI have prepared the *Document of Requirements for the Clearbrook Interchange Improvement Project* to which Alloy Consulting Ltd.'s design will conform. This document is available upon request.

1.2 Scope of Work

The scope for this project consists of five primary areas of work:

1. Geotechnical analysis
2. Transportation design
3. Bridge structural design
4. Construction management
5. Environmental design

The **geotechnical analysis** for this project includes evaluation of the following parameters based on borehole and test-pit data, aerial photographs, and site investigations:

- Seismicity
- Excavation and temporary drainage
- Site preparation and backfilling
- Foundation design parameters
- Lateral earth pressures
- Permanent drainage

The above-mentioned parameters are summarized in a geotechnical report (see Appendix A). Using the information from the prepared geotechnical report, design of the bridge abutments have also been prepared.

The **transportation design** contains two main phases: preliminary design and detailed design. The preliminary design phase consists of preparing horizontal alignments for three design alternatives. These design alternatives consisted of the following interchange arrangements:

- **Option A** – conventional intersection (north) and roundabout (south)
- **Option B** – double roundabout
- **Option C** – diverging diamond interchange

We compared the three design alternatives using the design selection criteria described in Section 3.0 of this report and a detailed design of the preferred option was prepared.

The **bridge structural design** consists of the preliminary design of three bridge girder alternatives: steel I-girders, prestressed concrete I-girders, and prestressed concrete box-girders. Drawings of each design alternative were prepared and a preferred option was selected based on function, cost, constructability, and sustainability.

The **construction management** portion of the project consists of construction staging for each major section of the chosen design option. Each stage has an associated drawing made to outline the location of the work and comments about the work to be completed. Additionally, the roundabout construction is broken down into smaller stages since this is the most complex part of the project.

The **environmental design** consists of the design of infiltration ponds on either side of the freeway to account for road runoff. Additional environmental considerations have also been included in the various decision-based designs incorporated in this project.

1.3 Codes and Assumptions

The analysis and design of the Clearbrook Interchange have been completed in accordance with the following codes and standards:

Geotechnical Analysis and Abutment Design:

- *CSA S6 – 06 - Canadian Highway Bridge Design Code*
- *2010 National Building Code of Canada*
- *Canadian Foundation Engineering Manual, 4th Edition*
- Ministry of Transportation and Highways technical bulletin *GM9801: Guidelines for Geotechnical Reports*.
- The City of Abbotsford's *Development Bylaw No. 2070-2011*
- *BC Supplement to TAC Geometric Design Guide 2007 Edition*

Transportation Design:

- *Transportation Association of Canada Geometric Design Guide for Canadian Roads 1999*
- *BC Supplement to TAC Geometric Design Guide 2007 Edition*
- *NCHRP Report 672 – Roundabouts: An Informational Guide Second Edition*

Preliminary Design Comparisons:

- *Canadian Capacity Guide for Signalized Intersections 2008 Edition*
- *Roundabouts: An Informational Guide*

Bridge Structural Design:

- *CSA S6 – 06 - Canadian Highway Bridge Design Code*
- *CSA S16 – 14 - Design of Steel Structures*
- *CSA A23.3 – 14 - Design of Concrete Structures*

In addition to our work being guided by the above-mentioned codes and standards, Alloy Consulting Ltd. was required to make the following design assumption:

- Alloy Consulting assumed that land could be expropriated from the neighbouring properties and business at a reasonable cost and without an extension of the project construction schedule. This was done so that design alternatives other than what was previously designed by McElhanney could be considered.

Additional assumptions pertaining to specific portions of this project's work may be found in the appropriate section of this design report.

1.4 Modifications to Proposal

To account for unforeseen challenges, to improve clarity, and to accommodate recommendations made by the Capstone Project Committee, several modifications were made from Alloy Consulting Ltd's proposal dated November 12, 2014. These changes are summarized in the table below:

Table 1 - Modifications to Proposal

Modifications to suite challenges or to improve clarity:
1. **Bridge Girder Drawings** – To provide a more complete summary of all structural design alternatives, design drawings will be prepared for each girder type in lieu of solely for the preferred alternative as originally proposed. 2. **Traffic Management and Construction Signage** – Due to time limitations, the traffic management could not be completed as intended. A construction sequencing plan was completed, however. In lieu of construction signage layouts, we prepared a layout for post-construction signage.
Modifications to suit recommendations made by the Capstone Project Committee:
1. **Sustainability** – Alloy's consideration of sustainability in the completion of this project was not made sufficiently clear in our proposal. To ensure that the consideration of sustainability is made apparent, Alloy Consulting has provided specific subsections within each area of the project scope to discuss the impacts of sustainability on the decision making process. 2. **Design Option Selection** – The Capstone Project Committee requested clarification regarding the design selection methodology. Alloy Consulting has developed a selection criteria method based on several important factors. This clarified selection criteria can be found in Section 3.0 for the transportation design and in Section 6.5 for the structural design.

The above-mentioned modifications have been accounted for within this report, without the requirement to revise the previously proposed fee of $93,000.

2.0 GEOTECHNICAL DESIGN

The primary purpose of the geotechnical aspect of the project was to produce a complete geotechnical report. The goal of the report was to provide all of Alloy Consulting's engineering disciplines with the information necessary for design and construction.

2.1 Geotechnical Report

A preliminary geotechnical report was completed by mid-January. Report additions were made when other disciplines indicated that they required information that was not included in the preliminary report. The content of the report meets the standards set out in Ministry of Transportation and Highways technical bulletin *GM9801: Guidelines for Geotechnical Reports*.

The complete geotechnical report has been attached as Appendix A of this report. Geotechnical design calculations and reference materials can be found in Appendix B and Appendix C, respectively.

2.1.1 Background

McElhanney provided borehole, test-pit, percolation test, and Benkelman beam data that had originally been acquired by Braun Geotechnical Ltd. and Levelton Consultants Ltd. Table 2 presents a summary of the types and quantities of data provided:

Table 2 - Geotechnical Data Types and Quantities Provided

Data Type	Quantity
Boreholes logs	6 Boreholes
Test-pit logs	20 Test-pits
Percolation test logs	3 Percolation tests
Benkelman beam tests	2 Benkelman beam tests

In addition to the test data listed above, McElhanney provided a number of photographs of the site, taken prior to construction.

2.1.2 Surface Conditions

Existing surface conditions were detailed in the geotechnical report. Since the project had actually been completed in 2010, a site visit to observe the pre-project conditions was not possible. A pseudo-site visit was conducted by studying the pre-project photographs provided by McElhannney. When used in conjunction with aerial photographs and a provided topographic drawing, the photographs provided enough information to accurately describe the pre-project surface conditions.

Results of Benkelman beam tests, conducted along Marshall Road, indicated that the road did not require upgrading. The road was found to have a most probable spring rebound (MPSR) of 0.75 inches, which is below the maximum MPSR of 1.0 inches permitted for an arterial road, as outlined in Schedule F of the City of Abbottsford's *Development Bylaw No. 2070-2011*.

2.1.3 Subsurface Conditions

The subsurface conditions provide the foundation of the geotechnical report. To obtain a general idea of the soils underlying the site, *GeoMap Vancouver,* a geological map provided by the Geological Survey of Canada, was examined. The map indicated that the site was underlain by deposits of sand and gravel. The sand and gravel deposits were confirmed by the test-hole data (boreholes, test-pits, and percolation tests). However, the boreholes drilled to depths greater than 8 m encountered thick layers of very stiff clay underlying the sand and gravel.

The six boreholes were strategically drilled adjacent to the Clearbrook overpass. They were drilled to depths exceeding 20 m and provided the data required for abutment foundation and pier design.

Fifteen test-pits were excavated to depths of up to 2.0 m along the proposed Highway 1 realignment. For the most part, the fifteen test-pits contained sand and gravel and, on occasion, some fill material. It was determined that the sand and gravel provided a suitable road base for the realignment. The fill material, however, was deemed to be unsuitable as a road base and recommendations were made to have it excavated prior to road construction.

The remaining test-pits and 3 percolation tests were located within the grass covered areas enclosed by the east-bound and west-bound off-ramps from Highway 1. These areas were within the proposed location of infiltration ponds. These 5 test-pits were excavated to depths of up to 4.0 m. The test-pit data, along with the percolation rates, provided the data necessary for the infiltration pond design.

2.1.4 Geohazard Assessment

Using a combination of aerial and pre-project site photographs, a geohazard assessment was conducted for the site. The assessment was based on the 1993 paper *Hazard Acceptability Thresholds for Development Approvals by Local Government* by Dr. Peter Cave (Cave Report).

The Cave Report details eight different geotechnical hazards that may pose a risk to a site. The eight hazards are as follows:

- Inundation by flood waters
- Mountain stream erosion and avulsion
- Debris flows and debris torrents
- Debris floods
- Small-scale and local landslides
- Snow avalanche
- Rock fall
- Large-scale landslides

Each hazard was analyzed and, as part of a risk analysis, the annual return frequency of each hazard was estimated. Due to the significant distance of the Clearbrook Interchange from any large water courses or mountain ranges, as well as the lack of any significant rock faces or steep slopes, it was determined that the site was not susceptible to any of the hazards.

2.1.5 Recommendations

The recommendations section of the geotechnical report contains the majority of the information required by other engineering disciplines. The information acquired and summarized in the surface and subsurface condition sections, as well as that of the geohazard assessment, was used to provide recommendations and design parameters for the following:

- Seismicity
- Excavation and temporary drainage
- Infiltration ponds
- Site preparation and backfilling
- Foundation design
- Retaining walls

Seismic coefficients were determined based on both of the *2006 Canadian Highway Bridge Design Code* (*CSA S6-06*) and the *2010 National Building Code of Canada*. The peak ground acceleration was determined using the online 2010 National Building Code Seismic Hazard Calculator.

Recommendations concerning excavation and temporary drainage are detailed in Worksafe BC guidelines. Additionally, based on the soil conditions encountered at the site, it was recommended that unbraced cut slopes deeper than 1.2 m be cut with a side slope ratio of 3 horizontal to 4 vertical, to a maximum depth of 2 m. While groundwater is not expected to be of concern at the site, recommendations were made regarding temporary dewatering in the case that a localized or perched groundwater table is encountered.

For the infiltration ponds, suggestions include the removal of all impermeable materials to expose permeable sand and gravel layers. Similarly, site preparation recommendations suggest the stripping of all vegetation, topsoil, and organic material prior to construction of the bridge abutments and pier.

In an effort to reduce project costs and improve sustainability, recommendations were made regarding the reuse of excavated material as backfill. The sand and gravel materials found throughout the site were deemed satisfactory for reuse as abutment backfill as well as for fill material for the bridge approaches.

Where new roads are to be constructed, the geotechnical report provides minimum pavement structure thicknesses. The pavement sections follow the requirements as detailed in the *BC Supplement to TAC Geometric Design Guide 2007 Edition*.

Finally, the recommendations section of the geotechnical report contains a number of recommendations regarding foundation design. To provide coefficients of lateral earth pressures, educated assumptions about the existing soil properties were made. Table 3 summarizes the soil properties used to determine lateral earth pressures, based on the assumption that the existing on-site sand and gravel will be used as backfill material.

Table 3 - Soil Properties Used to Determine Lateral Earth Pressures

Property	Value
Weight (γ_S)	$\gamma_S = 18.6$ kN/m^3
Angle of Internal Friction (ϕ)	$\phi = 28°$
Cohesion (c)	c = 0 MPa

Especially important in the design of the bridge abutments, Table 4 summarizes the static and dynamic earth pressure coefficients.

Table 4 - Coefficients of Lateral Earth Pressures

Coefficient Pressure Description	Symbol	Coefficient
Static Active Earth Pressure	K_A	0.361
Static Passive Earth Pressure	K_P	2.770
Dynamic Active Earth Pressure	K_{AE}	0.551
Dynamic Passive Earth Pressure	K_{PE}	0.551

Static coefficients are based on Rankine theory, and assume a backfill slope of 0° ($\beta = 0$). Dynamic coefficients were calculated using the Mononobe-Okabe method (as suggested in CSA S6-06), and assume a horizontal seismic coefficient of one-half of the peak ground acceleration, a negligible vertical seismic coefficient (Au-Yeung, 1995), a backfill slope of 0° (β = 0), and a backfill slope with the horizontal of 0° (Θ = 0). Sample calculations for the earth pressures are included in Appendix B.

The geotechnical report was produced prior to the structural design of the bridge abutments. In order to provide an allowable bearing capacity for the abutment, a bearing capacity chart based on potential footing width was produced. Also included on the chart are discrete settlements associated with the allowable bearing capacity and footing width. The bearing capacity and settlement chart is included in Figure 2.

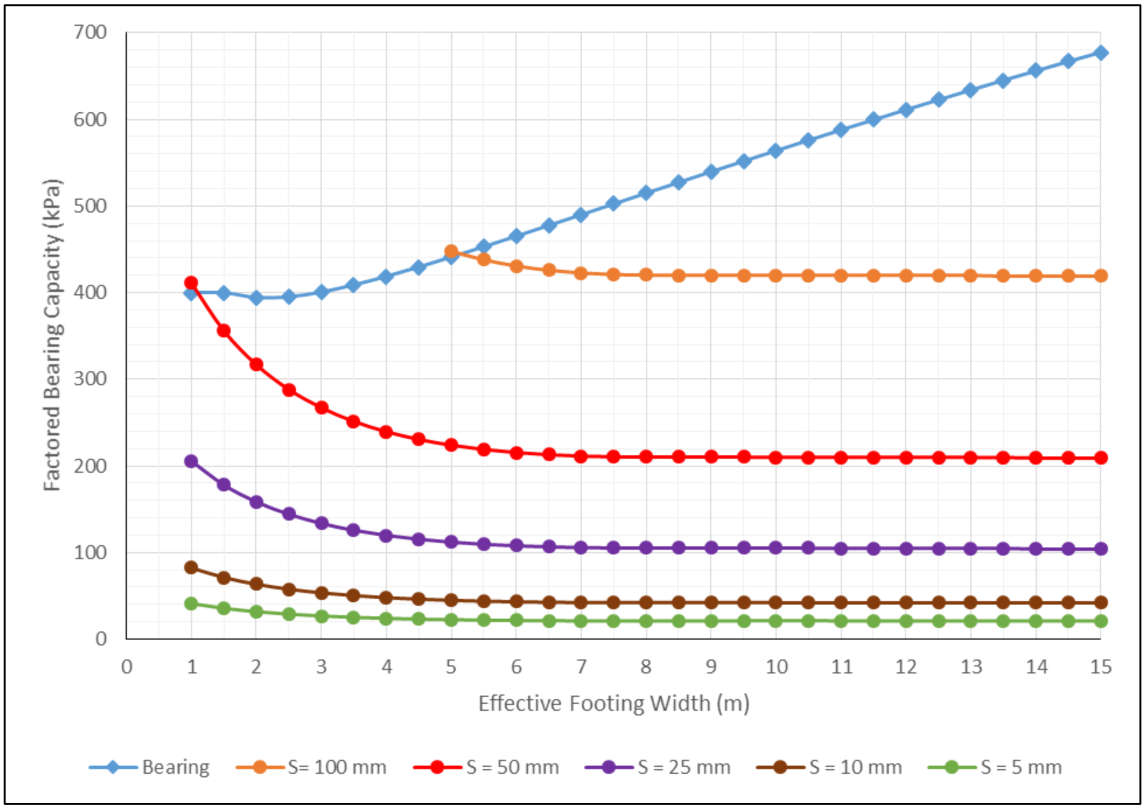


Figure 2 - Bearing capacity and settlements for various footing widths (Alloy Consulting Ltd., 2015)

The format of the bearing capacity chart is based on the Arizona Department of Transportation's requirements for geotechnical reports. To calculate bearing capacity, Terzaghi's ultimate bearing capacity equation was used:

$$q_u = cN_cS_c + q_sN_qS_q + \frac{1}{2}\gamma BN_\gamma S_\gamma$$

where:	q_u	=	ultimate bearing capacity
	N_c, N_q, N_γ	=	dimensionless bearing capacity factors
	S_c, S_q, S_γ	=	dimensionless modification factors
	q_s	=	vertical stress acting at the base of the foundation
	B	=	width of the foundation
	c	=	soil cohesion
	γ	=	soil unit weight

The factored bearing capacity is calculated by applying a geotechnical resistance factor (ϕ) of 0.5, as per *CSA S6-06*, to the ultimate bearing capacity of the soil.

Settlements for multiple soil layers were calculated using three-dimensional elastic strain integration as follows,

$$S_{TF} = \sum_{i=1}^{n}[\Delta\varepsilon_z \delta h]_i = \sum_{i=1}^{n}\left[\frac{1}{E}\left(\Delta\sigma_z - \nu\left(\Delta\sigma_x + \Delta\sigma_y\right)\right)\delta h\right]_i$$

where:	$\Delta\varepsilon_z$	=	increment in vertical strain in sublayer *i*
	$\Delta\sigma_x$, $\Delta\sigma_y$, $\Delta\sigma_z$,	=	increment in effective stress in x, y, and z directions of sublayer *i*
	E	=	Young's Modulus for the soil in layer *i*
	ν	=	Poisson's ratio for the soil in layer *i*
	n	=	number of sublayers
	δh	=	thickness of sublayer *i*

All calculations used in the recommendations section are based on those outlined in the 2006 edition of the *Canadian Foundation Engineering Manual*.

3.0 PRELIMINARY TRANSPORTATION DESIGN

The purpose of the preliminary design is to configure the layout for three options to evaluate the validity of the options based on the weighted criteria of cost, traffic capacity, drivability, and construction sequencing and feasibility. To prepare for the comparison, each option's layout was prepared using Autodesk Civil 3D. The drawn components include all major roadways and intersections. Complete Civil 3D corridors were not prepared for the preliminary design phase, however, since only the preferred option would require these. The preliminary layouts can be found in Appendix D.

3.1 Survey Data Import

Before any work could be done on the Autodesk Civil 3D model, we needed to convert existing site survey data provided by McElhanney into a Civil 3D TIN surface. The survey data was provided as a series of coordinates with accompanying elevation text. The coordinates and the accompanying text were exported into Excel as a table of values. The eastings, northings, and elevations were saved as comma-separated variables and imported into Autodesk Civil 3D. With the provided data now imported in Civil 3D as 3-dimensional coordinates with easting, northing, and elevations, a TIN surface was created for use in future transportation models.

3.2 Trans-Canada Highway Relocation

One of the major problems with the existing interchange is the westbound off-ramp's small turning radius. To maximize this turning radius, the westbound lanes of the Trans-Canada Highway (TCH) needed to be relocated southward. A secondary advantage of performing this realignment is that the span of the new overpass is decreased, making it more economical.

According to "Section 400 Cross Sections Chapter" of the 2007 *Supplement to TAC Geometric Design Guide*, the required space between the outside lane markings of divided rural highways is 13 m. This 13 m gap between the opposing traffic lanes was provided at the crossing of the new overpass above the TCH. The new highway alignment was located to reconnect with the existing highway about 800 m past each side of the overpass, allowing for smooth spiral curves. The vertical alignment of the TCH was then revised to suit the revised horizontal alignment and the created ground surface. The horizontal alignment, vertical alignment, and superelevation were designed with checks built into Autodesk Civil 3D. The freeway realignment satisfies the requirements of the American Association of State Highway and Transportation Officials (AASHTO) 2001 Standards as well as the 1999 *Transportation Association of Canada Geometric Design Guide for Canadian Roads* (TAC) manual.

The typical cross-section assembly was then created according to "Section 400 Cross Sections Chapter" of the 2007 *Supplement to TAC Geometric Design Guide*, using the rural divided freeway specifications. The assembly was combined with the prepared horizontal and vertical alignments to create a corridor. This corridor surface will be used during our full design to match the on and off ramps to the TCH.

Although the eastbound TCH lanes did not require realignment, we did require a working corridor surface for the final interchange design. Because of this, a corridor was created for the eastbound traffic lanes that overlays the existing eastbound lanes of the TCH.

3.3 Option A – Conventional Intersection and Roundabout (McElhanney Design)

The lane layout was designed based on three main components: the overpass, intersections, and on/off ramps. For each of these components, mandatory or logical arrangements of each were used to combine the three components into a working interchange.

The following paragraph will use the road naming convention for the south roundabout shown in Figure 3 below:

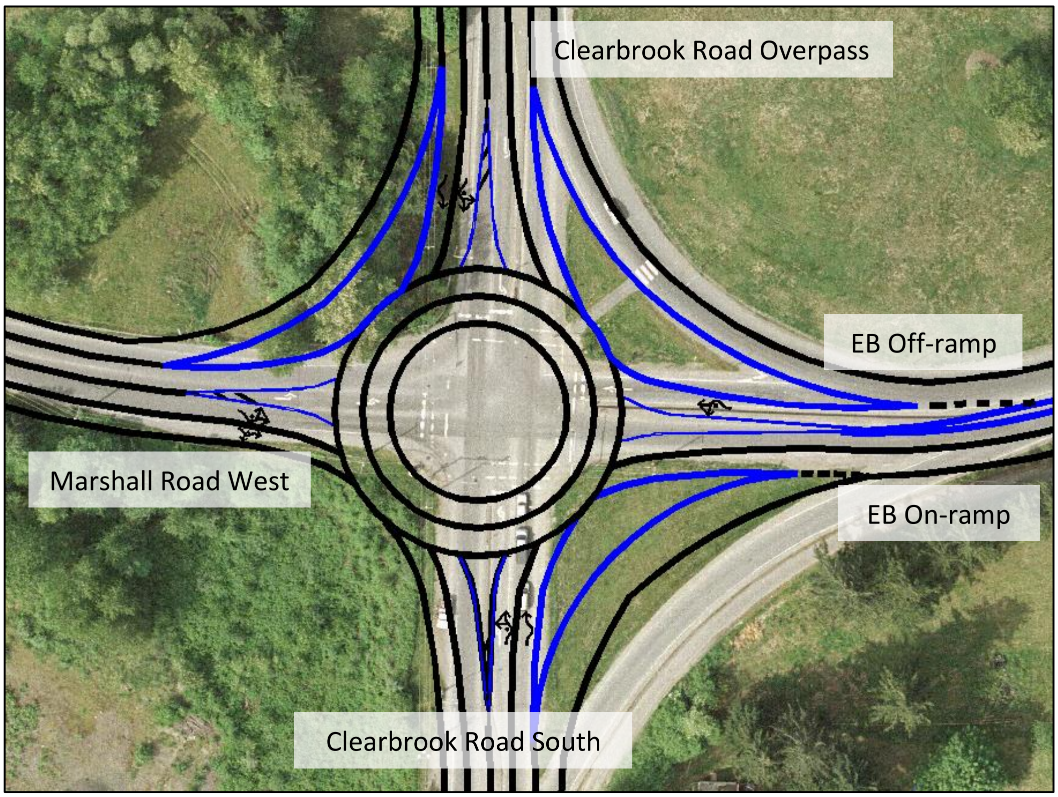


Figure 3 - South Roundabout Naming Convention (Alloy Consulting Ltd., 2015)

Considering the south intersection, according to the *Document of Requirements for Clearbrook Interchange Improvement Project*, the overpass must be six lanes wide (City of Abbotsford, Ministry of Transportation and Infrastructure, 2010). Based on the number of existing lanes coming into the south intersection, the roundabout must consist of two lanes. Therefore, to connect with Clearbrook Road Overpass, there must be at least six lanes leaving the intersection to the north. Since there are four existing lanes running north-south along Clearbrook Road, the six lane overpass allows for two dedicated right hand turning lanes; one coming from the EB Off-ramp and the other from Clearbrook Road Overpass onto Marshall Road West. Given the dedicated right lane onto Marshall Road West and the requirement for two straight-through lanes from Marshall Road West (based on the traffic projections), Marshall Road West will be expanded to four lanes. Based on the high traffic volumes turning right from the EB Off-ramp and relatively low number of cars going left or straight, there will be a dedicated right turn slip lane with its own lane on Clearbrook Road Overpass and a single lane allowing drivers to enter the roundabout from the off-ramp. Due to the low traffic volume turning right from Marshall Road West, a dedicated right-turn lane is not required.

The following paragraph will use the road naming convention for the north intersection shown in Figure 4 below:

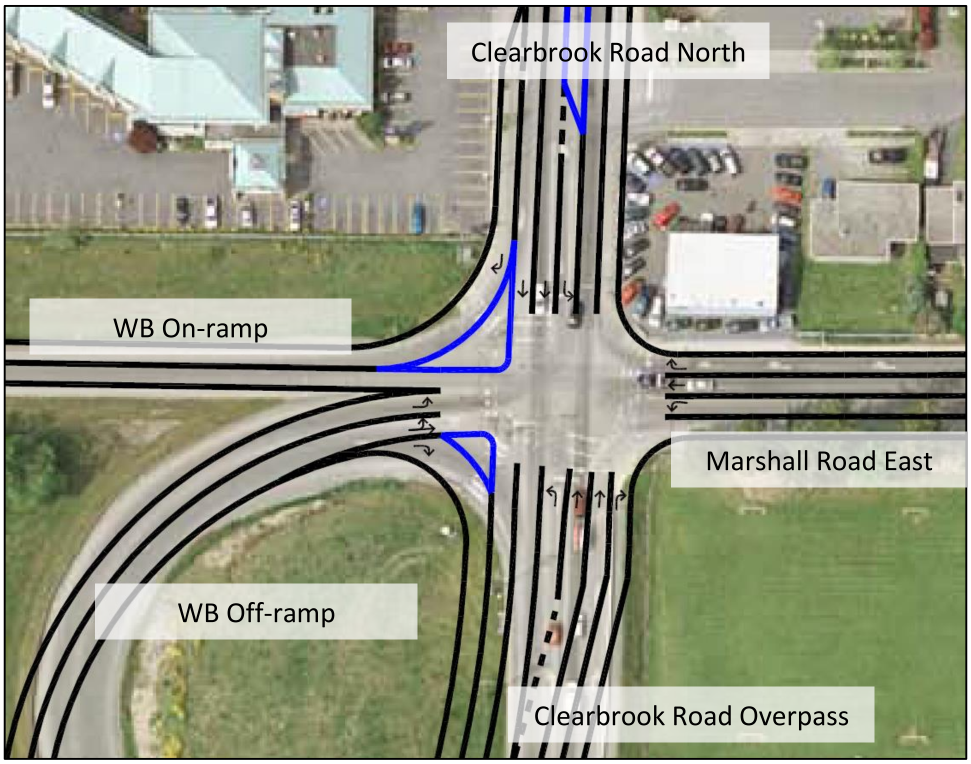


Figure 4 - North Intersection Naming Convention (Alloy Consulting Ltd., 2015)

For the north intersection, due to property constraints on the east side of the intersection, there is no room for slip lanes with island separation so a right turn lane along the curb line is required. To maximize the traffic flow through the intersection there are two straight through lanes for each direction. From the westbound loop ramp, there are two lanes approaching the intersection. The outside lane will be turning left only and the inside lane will split into a dedicated right slip lane and a combined straight-left lane to accommodate the large volume of traffic turning left onto Clearbrook Road North. The other three directions require a single left turning lane. Therefore, seven lanes are required from Clearbrook Road Overpass. To add the extra lane, the road will widen and the lane turning left onto the westbound onramp will merge from the inside through lane.

For ease of transitioning on and off the Trans-Canada Highway, there will be one lane (instead of two) merging on and off the highway, each expanding into two lanes when entering the intersections. Although it will be calculated in the chosen detailed design, the merge lengths will conform to the TAC guidelines and allow plenty of time for merging and room for site lines.

3.4 Option B – Double Roundabout

Option B is similar to option A except the north intersection will be replaced with a roundabout. The upgrade to a roundabout should increase the traffic capacity but requires some land acquisition. The following features in Option B are identical to Option A:

- South roundabout
- Marshall Road West
- Clearbrook Road South
- Clearbrook Road Overpass
- Eastbound TCH On and Off-ramps
- Westbound Off-ramp.

To improve the north roundabout's traffic capacity compared to Option A's intersection, a dedicated right turn slip lane (outside of the roundabout) is required in all four directions. To accomplish this, we will require the purchase of a portion of the Abbotsford School of Integrated Arts' field (southeast), the Abbotsford Auto World car dealership (northeast), and a portion of the Comfort Inn parking lot (northwest). The required land acquisitions are shown below outlined in red in Figure 5:

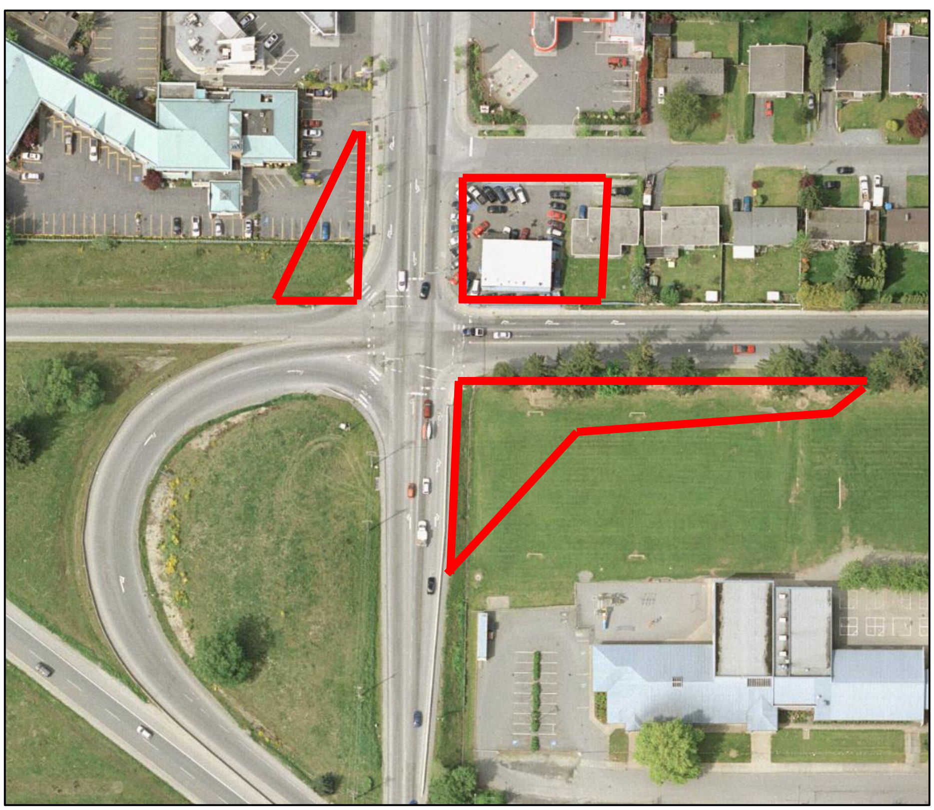


Figure 5 - Land Acquisition Requirements (McElhanney Consulting Services Ltd., 2008)

The cost of the land acquisitions shown in Figure 5 will be explained in Section 3.0 – Preliminary Option Decision Process.

The roundabout itself is nearly symmetrical in all directions, with two lanes entering the roundabout from each direction and two lanes exiting. Entering from the WB Off-ramp, both lanes offer drivers the options of proceeding straight and left through the roundabout (due to the large volume of vehicles turning left from the off-ramp). All other directions have the outside lanes only continuing straight through the roundabout, while vehicles can drive left and straight from the inside lanes. The reason the outside lanes are not designed for left-turning traffic is to allow less yielding time for vehicles attempting to pass straight through the roundabout on the outside lanes.

The north roundabout brings additional changes to its surrounding roads. Since there are only six lanes entering or exiting the roundabout on the south, there is no need for an additional expansion of Clearbrook Road Overpass as there was in Option A. Marshall Road East will require a total of five lanes entering and exiting the roundabout, the southern-most lane (right turn from Clearbrook Road Overpass) will merge with the adjacent lane within about 120 m of the roundabout. The lane turning right onto Clearbrook Road North from Marshall Road East doesn't have its own approach lane to the turn as in option A, but the dedicated right turn merges from the northern-most lane immediately before the intersection and passes through the existing car dealership. The WB On-ramp now has three lanes leaving the roundabout. These three lanes will be merged into one progressively over a 130 m length. The final alteration is the blocking of the eastbound lane on Joyce Avenue (north of Marshall Road).

The dedicated right turn from Marshall Road East turning onto Clearbrook Road North does not allow enough room for drivers to safely turn right onto Joyce Avenue, especially if they are exiting through either of the roundabout lanes. With this considered, an island barrier will be built at the entrance to the existing eastbound Joyce Avenue lane. This barrier will still allow users to turn right from westbound Joyce Avenue onto Clearbrook Road North. This barrier will affect about 30 homes in the enclosed loop. These residents will be forced to use nearby Sherwood Crescent to access their properties. The new route is illustrated in Figure 6 on the following page.

Figure 6 - Diversion to Local Traffic Resulting from Option B (Google Inc., 2015)

As seen above, the required diversion to local traffic will affect a relatively small number of residences.

3.5 Option C – Diverging Diamond Interchange

Option C consists of a Diverging Diamond Interchange (DDI). This interchange alignment varies greatly from Option A and Option B. The DDI, illustrated in Figure 7 on the following page, consists of two intersections on either side of a highway overpass. At these intersections (or crossover nodes), traffic is directed to the opposing side of the roadway as it exits the intersection and begins to cross the bridge. This switching action allows the commuters to proceed on the opposite side of the arterial road, eliminating the need for left hand turns across traffic when trying to access the Trans-Canada Highway. The DDI was pioneered to improve interchanges by reducing the number of conflict points within the overpass and intersection areas and by providing a solution to deal with exceptionally high critical lane volumes.

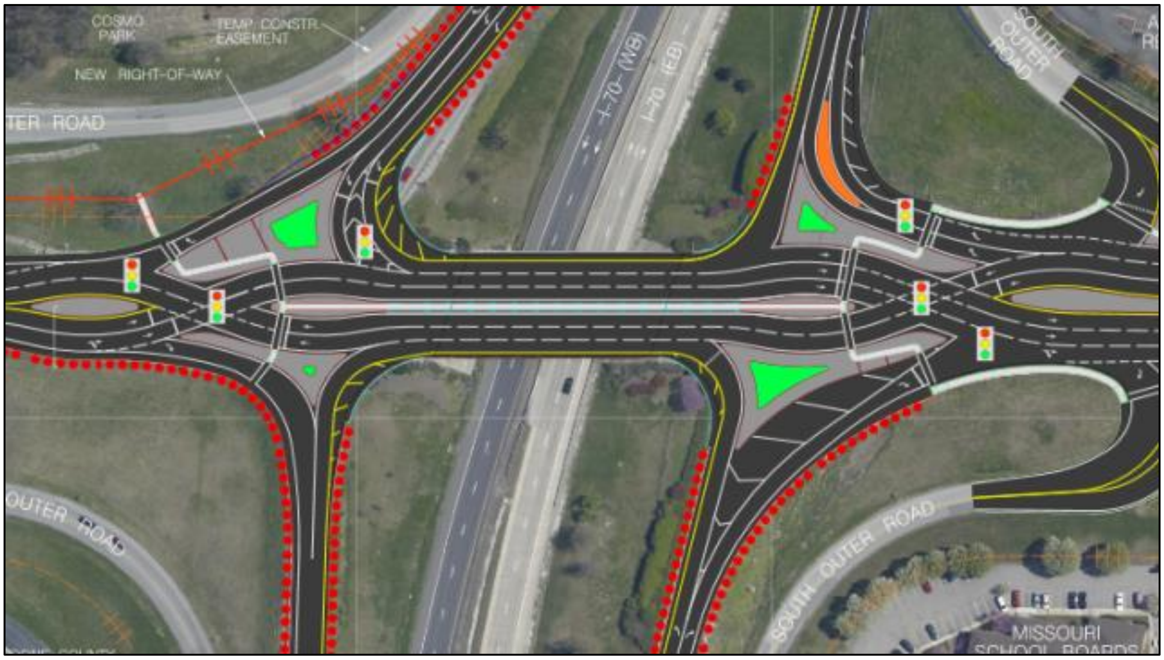


Figure 7 - Diverging Diamond Interchange Concept (Fasbinder, 2013)

As with the other preliminary design options, only the horizontal alignment with the necessary on and off ramps was designed in Civil 3D. When constructing the DDI, we provided a dedicated merge lane for the commuters exiting the TCH onto Clearbrook Road Overpass, which could also be used as a U-turn route. After completing the horizontal alignment, the aerial map and property lines were re-established, and we found that there were large issues related to property infringement with the industrial area to the southwest as well as with the school parking to the northeast. After adding lane lines to the drawing, as shown in Figure 8 on the following page, we also determined that both intersections would pose great difficulties.

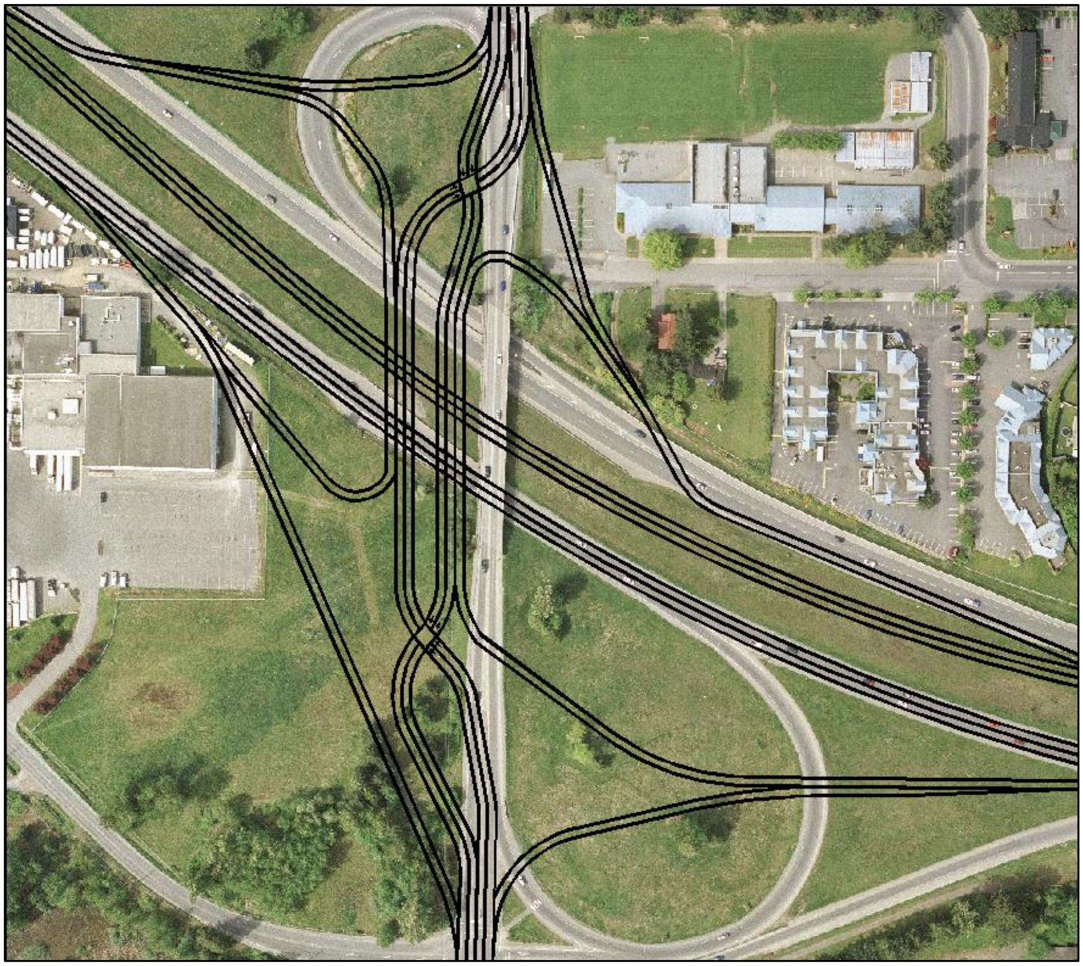


Figure 8 - Option C Preliminary Design (McElhanney Consulting Services Ltd., 2008)

Further review suggests that the intersection to the south could be relocated further south with the purchase of additional land, but the intersection would still require a large amount of land to function. A large amount of land would need to be purchased from the nearby businesses and school based on the location of the north intersection. It can also be observed that the Marshall Road East will no longer be accessible and it will need to be rerouted, causing a large disruption to existing users.

4.0 PRELIMINARY OPTION DECISION PROCESS

Alloy Consulting set out a rating system to select the suitable option for the detailed design. The categories in this system include cost, traffic capacity, construction time/sequencing, and drivability. The rating process and score for each option in each category is outlined in the sections below.

4.1 Cost

Cost for all three options were calculated using the Ministry of Transportation and Infrastructures conceptual level project cost estimating tool, which averages the unit costs for bridges, road structures, and project level costs such as design, project supervision, and inspections. The City of Barrier also has produced a similar document for the average cost to remove and replace a roadway and the cost to build a new roadway. Using these documents, the preliminary costs for each of the options was determined. Table 5 contains the estimated costs, including a combined 25% of the construction cost for the project level costs and the estimated cost to purchase the necessary land.

Table 5 - Summary of Cost Ratings

	Construction Cost	Total Project Cost	Cost Rating
Option A	$ 18,740,500	$ 23,425,625	Pass
Option B	$ 19,590,500	$ 24,488,125	Pass
Option C	$ 21,478,500	$ 26,848,125	Fail

*Cost Rating based on a total project cost maximum of $ 25,000,000

Appendix E contains a more detailed breakdown of the cost and quantities for the three options and shows the breakdown for each side of the interchange.

4.2 Traffic Projections

Traffic projections were done using the data provided by McElhanney. They provided the amount of traffic in the intersections today and the rate of growth for each traffic direction. There was also data on how many additional large vehicles use the intersection. The traffic projections assume that each year the traffic increases by the percent of inflation multiplied by the initial traffic flow in the specified direction. This inflation is only applied to the passenger vehicles and the large vehicles are added as a separate quantity that does not change. The traffic projections were created for the years 2020, 2034 and 2040 to see which year the intersections will pass when analyzed for traffic capacity.

4.3 Traffic Capacity

The traffic capacity was calculated for each of the three preliminary design options. Traffic capacity calculations for all transportation design options are included in Appendix F.

4.3.1 Option A

The north intersection for option A is a standard signalized intersection. The capacity for this intersection was calculated using the *Canadian Capacity Guide for Signalized Intersections*, which was developed by TAC. To analyze the intersection capacity, a saturation flow value is required. The guide provided saturation values for different locations and the value of 1700 was chosen, which represents values in downtown Victoria. To complete the analysis, there is also a requirement to have green light times. Initial green light times were assumed to start the analysis and they were then refined using solver to optimize the times. To simplify the analysis, it was assumed that on red lights two vehicles would turn right every cycle. With all of the assumptions in place, the worst case flow to capacity was calculated. Furthermore, the ratio of flow to saturation was calculated. The critical ratios from each phase of green lights are summed together. The sum is then compared to tabulated level of service values. According to the table, the level of service just exceeds an “E” rating in the year 2034, which means it does not achieve the desired rating of “E” in the year 2040.

The south intersection for Option A is a two-lane roundabout and is identical to the roundabout used in option B. To analyze the roundabout, we calculated circulating and entry traffic flows. The circulating values were used in conjunction with a capacity equation provided in *Roundabouts: An Informational Guide*, which was developed by the Federal Highway Administration from studies. Comparing the actual capacity of the entry flow to the flow at the entry gives the degree of saturation. Using this method, we calculated the worst-case degree of saturation as 65%. Using the degree of saturation and a delay equation provided in the guide, we calculated delays for each entry and found that the longest delay occurs in the evening and is 7.7 seconds long. Finally, the 95 percentile queue length was calculated for each entry and we found that the worst case would have a length of 6 vehicles at the south entrance. According to the guide, roundabouts level of service are evaluated using the same criteria as unsignalized intersections, which are based on delay time. An unsignalized intersection with a delay time less than 10 seconds has a category “A” level of service. Therefore, the south intersection is satisfactory for the 2040 category “E” level of service.

Overall, option A does not meet the level of service demand for the year 2040 due to the north intersection not meeting the desired category “E” level of service.

4.3.2 Option B

The north and south roundabouts for option B were analyzed following the same procedure outlined for the south intersection in option A. Since the south intersection in option B is the same as the roundabout used in option A, it will satisfy the 2040 level of service required. However, the north intersection deals with more traffic than the south intersection. After analyzing this intersection, we found that it will have a worst case delay of 14.9 seconds. It also has a worst case degree of saturation of 88% with a 95 percentile queue length of 19 cars during the morning in the northbound lanes. The 14.9 second delay and the unsignalized level of service table show that the north intersection achieves a category "B".

Overall, this option is able to achieve the required level of service category "E" required for the design of this interchange.

4.3.3 Option C

Due to the complex nature of the DDI option, traffic capacity analysis during the planning phase of the project could be very expensive and time consuming. For this reason, the analysis was simplified using the method of critical lane volume (CLV). This method for the DDI was presented in a paper prepared by Avijit Maji, Sabyasachee Mishra, and Manoj K. Jha, titled *Diverging Diamond Interchange Analysis: Planning Tool* (Appendix G) and was tested against existing diverging diamond interchanges to test its validity. When using this tool we determined it has a 2040 v/c rating of 0.89 with a corresponding level of service rating of "D".

After calculating the level of service for each interchange, they were comparatively rated based on meeting the level of service "E" rating, which was outlined in the project guideline documents. The following table summarizes each option's traffic capacity assigned score.

Table 6 - Summary of Traffic Capacity Ratings

Option	Traffic Capacity Rating
Option A	25
Option B	50
Option C	40

4.4 Construction Time / Sequencing

To determine which design option will have the easiest construction, three different categories are considered: construction time, ease of staging, and traffic impact. The best option will get 40 points and the other two options will score less than 40 based on their deficiencies compared to the best option. Since the only major differences between the options are the intersection arrangements, this is the only aspect being considered. The constructability of the relocation of Highway 1 is going to be ignored since it occurs for each option and will not impact the evaluation.

4.4.1 Option A

Option A will require the installation of a new roundabout at the south intersection and an upgrade of the north intersection lanes. The construction of the roundabout will cause some significant issues with staging along with traffic impacts. Ideally, a roundabout should be constructed with the roads closed but we are aiming to have no closures. This requires the construction of the roundabout to be done in quarters and making the traffic travel around the construction, possibly implementing a four-way stop intersection. This means that the intersection construction will cause some impact to traffic flow until the intersection is complete.

The north intersection will have less of an impact, but will still have some conflict. The intersection is getting additional left turn lanes added and some right turn lanes. Therefore, the road will need to be widened and may affect the lane alongside the widening construction. The other major impact to traffic will be upgrading the current intersection signals. To minimize this impact, the change could be done at night and over a weekend so that a temporary four-way stop can be implemented. Overall, this intersection construction will have less of an impact than the south roundabout construction.

Both intersections should take approximately 4-5 months according to other construction schedules of previous intersection projects.

4.4.2 Option B

Option B requires the construction of two roundabouts to replace the existing north and south intersections. Each roundabout is going to require the same construction method as used for the roundabout in option A. This will lead to major traffic impacts at both intersections. However, with proper management of the construction it will not cause worse delays than Option A.

4.4.3 Option C

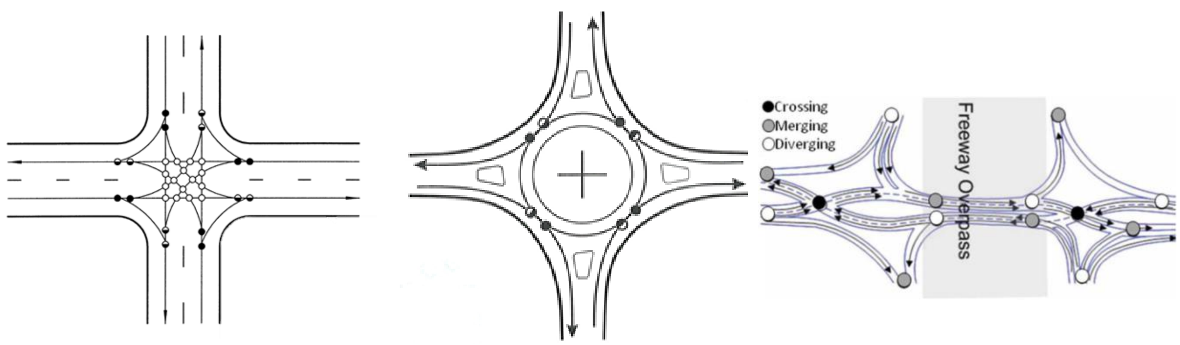
The diverging diamond interchange is more difficult to analyze since there are few interchanges that have been done, and not a lot of available construction information. However, there are some benefits to this option over the other two. Firstly, the on and off ramps do not interfere with the existing ramps. Most of them are perpendicular to the existing ones and can remain open while the new ramps are constructed. The crossover points are also not where the current lanes travel so the construction should not interfere with traffic. On the other hand, to construct the full diverging diamond a lot of land is required and with that comes greater amounts of time and more areas where problems can arise. With these facts in mind, the diverging diamond should be easier to construct with less impact on traffic than the other two options. However, the construction time should be longer due to more land being required for this option. This additional time is considered a higher factor since time is crucial to this project and therefore this option is worse with regards to construction time.

Table 7 - Summary of Construction Ratings

Option	Construction Rating
Option A	30
Option B	30
Option C	20

4.5 Drivability

The drivability of the three options is a difficult task to determine what constitutes as improving or decreasing a solutions drivability rating. One of the most studied and important statistics that are often reported is the number of injuries and deaths that occur at a location. If we analyze the number of conflict points for each of the proposed solutions it can be seen that the number of conflict points significantly decrease when the intersection is not used. Figure 9 illustrates where these conflict points occur in the intersections.



Typical Intersection

32 Conflict Points

Typical Roundabout

8 Conflict Points

Typical Diverging Diamond

14 Conflict Points

Figure 9 - Conflict Points for Different Intersection Arrangements (US Department of Transportation, 2004)

From Figure 9, we see that a total number of conflict points for Option A is 64, Option B is 16, and Option C is 14. When comparing accident statistics for roundabouts compared to intersections, there is a 37 percent reduction in overall collisions, 75 percent reduction in injury collisions, 90 percent reduction in fatality collisions, and a 40 percent reduction in pedestrian collisions (Washington State Department of Transportation, 2015). These large reductions can be attributed to the lower travel speed through the roundabout as well as reducing the number of conflicts in the intersection by four times. When implanting the DDI it, there was a 46% reduction in crashes, 80% of the drivers felt that traffic flow improved, and 91% of drivers felt it was easy to understand how to drive through the intersection (Nevada Department of Transportation, 2015).

The roundabout and the DDI option both provide excellent reduction in traffic accidents due to removing excess conflict points and both systems seem to be easily followed by road users. Since the roundabout is a more common traffic structure in Canada and the lower mainland, it gets a slightly higher drivability rating. The following table summarizes Alloy Consulting's drivability ratings for the three options.

Table 8 - Summary of Drivability Ratings

Option	Drivability Rating
Option A	5
Option B	10
Option C	8

4.6 Summary and Recommendations

The following table summarizes the resulting scores from the above sections.

Table 9 - Summary of All Intersection Ratings

	Cost	Traffic Capacity	Construction Time/Sequencing	Drivability	Total
Option A	Pass	25	30	5	60
Option B	Pass	50	30	10	90
Option C	Fail	40	20	8	68*

*Has a failing cost criterion

From these results, Alloy Consulting recommends proceeding with Option B for the detailed design. With this option, we suggest creating a right-turn only out of Joyce Avenue, located on the north side, and we will be required to purchase a small amount of land. This land comprises of a small portion of the Abbotsford School of Integrated Arts field, the car dealership located at the northeast corner of the northern intersection, as well as the corner of the Comfort Inns' parking lot. The entrance off Joyce Avenue into Petro-Canada will be required to be removed and the entrance along Clearbrook Road. will be moved to the north corner along Clearbrook Road and another entrance will be add to the south corner along Clearbrook Road to allow the gas station to maintain duel entrances and maintain filling operations.

5.0 DETAILED TRANSPORTATION DESIGN OF OPTION B

The purpose of the detailed design is to create a partial 3D model that displays all relevant design information including geometric layout, pavement markings, barriers, roundabouts, cross-sections, and profiles. All of the alignments (horizontal and vertical), cross-sections, and typical sub-bases are designed according to the 1999 *Transportation Association of Canada Geometric Design Guide for Canadian Roads* (TAC) manual.

Design checks were performed for all critical design parameters. These design checks are included in Appendix H.

5.1 Alignment Design

Since our team designed the alignment layouts for option B during the preliminary project stage, the detailed alignment design involved identifying design speeds for each road and creating spiral-curves accordingly. All roads besides the overpass and on/off-ramps were designed according to TAC guidelines, and did not include any specific challenges. This section will describe areas of the interchange that required special design attention in order to meet the design specifications.

5.1.1 Clearbrook Overpass

Three main requirements controlled the design for the overpass section of Clearbrook Road:

- Design speed of 50 km/hr
- 10 m offset at abutment from previous overpass
- Planar cross-section required through bridge section

The 10 m offset at the abutment was identified as a necessity to keep the existing bridge open during construction. The planar cross-section in the bridge allows for the same design regardless of the selected bridge structure. With these constraints, the minimum radius with reverse crown for a design speed of 50 km/hr is 400 m (TAC Table 2.1.2.5), which allowed enough curvature to avoid the existing bridge and allow drivers to safely navigate between roundabouts.

5.1.2 On-Ramps

In order to design the on-ramps, the main requirement was to introduce a spiral curve that meets tangent with the through lanes of the highway. By setting up tangent lines with this in mind, spiral curves were added that met the TAC requirements for roads with a design speed of 50 km/hr. Lane merging and highway lane tapers will be discussed further in the Section 5.2.

5.1.3 Off-Ramps

Because the off-ramps (or loop-ramps) were a main issue with the existing overpass, a key design component was to increase the design speed beyond the original 30 km/hr. To do this, a larger radius curve is required, but there also needs to be a safe distance to allow drivers to enter the roundabout after the outgoing spiral. After the relocation of the westbound highway lanes, both loop ramps had about the same space from the highway to the roundabout to insert a spiral curve. Our goal design speed for the loop ramps was 50 km/hr, but according to TAC Table 2.1.2.5, the minimum radius required was 100 m for this design speed, which does not fit with our space limitations. Both loop ramps were brought down to a design speed of 40 km/hr with a curve radius of 65 m and spiral parameters of 50 m in and out of the curves, which affords drivers the ability to enter the roundabouts after the spiral reach tangency and increases the original design speed.

5.2 Interchange Lane Layout

In general, the lane layout designed for Option B in the preliminary project stage was maintained.

According to TAC Section 2.4.6.2, single lane highway ramps are to be 4.8 m - 5.0 m wide and double lane ramps should have 3.7 m wide lanes. Both of our project's entrance ramps involve merging from two lanes into one before reaching the highway. There are two types of lane merging styles for entrance ramps as shown in below in Figure 10.

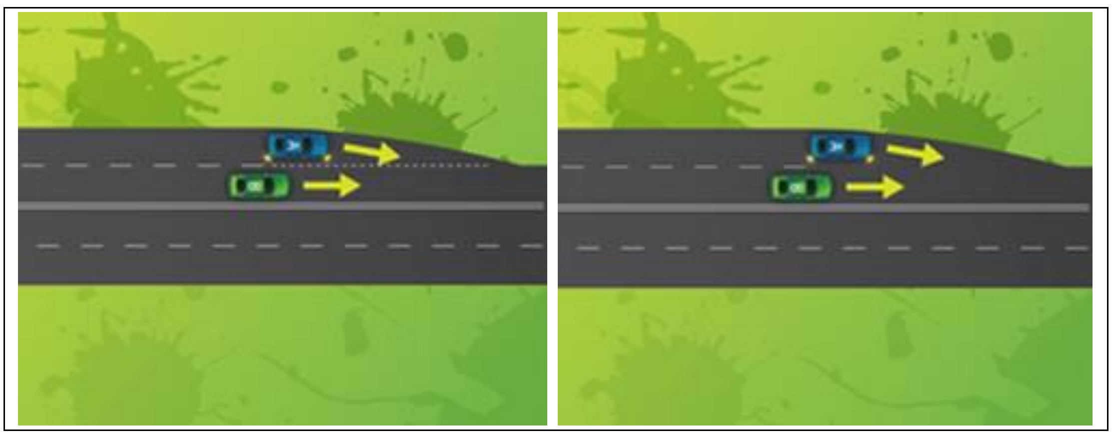


Figure 10 - Entrance Ramp Merging Options (Queensland Goverment, 2015)

The left figure, above, involves extending the dotted merging line to the end of lane giving the green car, in this case, the right of way. In the right figure, above, the dotted merging lane ends abruptly, giving first car in either lane the right of way to proceed first. We have designed our exit ramps according to option B to allow a smoother merging process in both low and high traffic scenarios.

Travelling either direction on Highway 1, off-ramps involved a parallel deceleration lane on the outside of the road allowing drivers leaving the highway to slow down and safely exit the highway. Similarly, the on-ramps included an acceleration lane that runs parallel to the highway lanes for an adequate distance for drivers to gain speed and merge with through-traffic. As expanded on in our interchange ramp design calculation checks, these lengths were designed according to TAC chapter 2.4.6. The values for each acceleration/deceleration lane and taper lengths are summarized in Table 10 on the following page.

Table 10 - On/Off-Ramp Transition Lengths

Road	Deceleration/Acceleration Lane Length (m)	Taper Length (m)
Westbound On-ramp	330	90
Westbound Off-ramp	230	90
Eastbound On-ramp	300	90
Eastbound Off-ramp	210	90

5.3 Roundabout Design

Both roundabouts were designed with the same criteria and methodology, so unless specified, the methodology presented in this section of the report was employed for both roundabouts. The reference guide used for the roundabout design was *NCHRP Report 672 – Roundabouts: An Informational Guide Second Edition.* The first component of roundabout design is the inscribed circle diameter (outermost part of the roundabout). For two-lane roundabouts, the NCHRP guide recommends an inscribed circle diameter ranging from 46 to 67 m (NCHRP Exhibit 6-9). We chose an inscribed circle diameter of 60 m because without a vehicle turn modeling software available, we were unable to confirm the ability for large trucks to maneuver our roundabout and chose a diameter on the upper end of the recommended range.

According to NCHRP 6.5.2, entry widths for two lane roundabouts range from 7.3 to 9.1 m; since most of our incoming lanes are at 3.7 m, this width was maintained into the roundabout for a combined width of 7.4 m. NCHRP 6.5.3 recommends lane widths in the roundabout to be between 4.2 and 4.9 m, so designed according to the middle of this range at 4.5 m.

To design the controlling curve for the roundabout slip-lanes, Alloy Consulting used NCHRP Exhibit 6-46 (Figure 11) to find the fastest path through the roundabout.

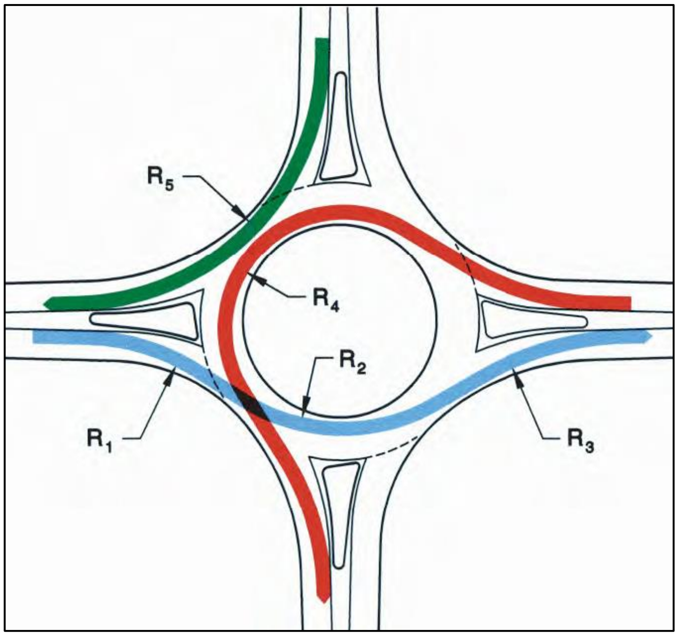


Figure 11 - Controlling Roundabout Curves (NCHRP, 2010)

Our minimum R_1 was measured to be 55 m, which fits within the recommended range of 53-84 m (NCHRP 6.5.4) to control speed within the roundabout. NCHRP 6.8.6 recommends that the turning radius of right-turn bypass lanes should not greatly exceed the minimum R_1 radius to control speed of bypass lane users. On the other hand, raised islands within the roundabout need a minimum width of 1.8 m (NCHRP 6.8.1.2) to protect pedestrians. So to accommodate both of these constraints, radii were minimized but maintained a 1.8 m island width where pedestrian access was required (to be expanded on in pedestrian access section). All bypass lane widths were designed to be 4.5 m wide.

For the cross-section within the roundabouts, the lanes slopes were constant at 2% sloping down to the outside of the roundabout. Another required feature is a mountable concrete apron on the inside on the roundabout in the case where a large truck has trouble staying within the inside lane. An example of this is shown below in Figure 12 taken from Section 6.8.7.4 in the NCHRP guide:

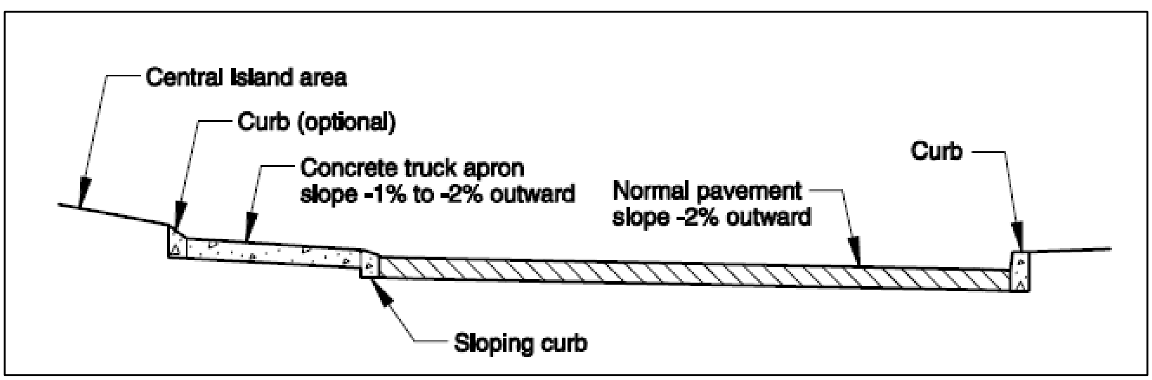


Figure 12 - Mountable Concrete Truck Apron (NCHRP, 2010)

All of the slip-lanes have dedicated entry lanes except for the southeast slip-lane at the south roundabout and the northeast slip lane at the north roundabout. Without a dedicated entry lane, it is ideal to have a tapering of the nearest lane to allow right-turning vehicles to bypass the cue of cars at the intersection. The southeast slip-lane at the south roundabout allowed for tapered widening of 4.5 m at a taper ratio of 15:1 according to TAC Table 2.3.5.1. Space limitations at the north roundabout do not allow for a tapering, but traffic is generally low through Marshall Road East, so the lack of a taper lane should not have adverse effects on traffic flow.

5.4 Vertical Profile Design

For most of the roads in our interchange, vertical profiles were not a large concern in terms of rough terrain or steep slopes. These roads were designed according to their curvature (K) in concurrence with the TAC requirements for sight distances and comfort for both sag and crest curves, depending on their design speed.

This section will expand upon roads where special attention was required to meet TAC requirements or to match connecting elevations and grades. These areas include on/off-ramps and the overpass.

5.4.1 Clearbrook Overpass

There were a number of constraints faced when designing the vertical profile for the Clearbrook overpass. These include the following:

- Maintaining a minimum of 5.5 m clearance from the bottom of the bridge girder to highway lane elevations
- Ensuring the maximum grade of 6% (TAC Table 2.1.3.1)
- Matching elevation at the entrance to both roundabouts
- Allowing for at least 0.5% grade entering the roundabout (in the direction of the roundabout cross-fall)

According to the BC Bridge design guide, the minimum clearance for the bridge overpass is 5 m. Our design used a minimum of 5.5 m because the abutment required additional space. According to Section 6.8.7.1 of the NCHRP guide, grades meeting the roundabout do not need to match the 2% cross fall of the roundabout, but should arrive sloping in the same direction at a minimum of 0.5% to allow a smooth transition into the roundabout.

With all these constraints considered, we were required to design the overpass with minimum K values and near-minimum grades when approaching and leaving the bridge. In addition, in order to meet the TAC's requirements, the design speed was assumed to drop to the roundabout speed of 30 km/hr within 40 m of the roundabout.

5.4.2 On/Off-Ramps

There were no critical areas in the on or off-ramps in terms of curvature or grade. All ramp curves were generally designed well above the minimum values. Similar to the overpass, though, the on and off-ramps had to meet the roundabout at a specific elevation with a grade over 0.50%. In addition, where the ramps met the highway, the elevation and grade were designed to exactly match the through lanes in order to create a flawless entrance or exit with the highway.

5.5 Pedestrian Access

Pedestrian access was a major design consideration, especially within the roundabout areas. For the south roundabout, sidewalks were designed except for on the on/off-ramps where there was no curb/gutter and no need for pedestrian access. A diagram of pedestrian routes for the north and south roundabouts are shown in red in Figure 13 and Figure 15 (green lines represent sidewalks/greenways, orange represents islands). We designed the sidewalks surrounding both roundabouts according to the NCHRP pedestrian section (6.8.1). Included in this was a 1.5 m landscape strip between the sidewalks and roundabout lanes/slip lanes to provide comfort to nearby pedestrians. Also, pedestrian sidewalks were designed to 1.8 m wide, and combined bike and pedestrian walkways were designed to be 3 m wide.

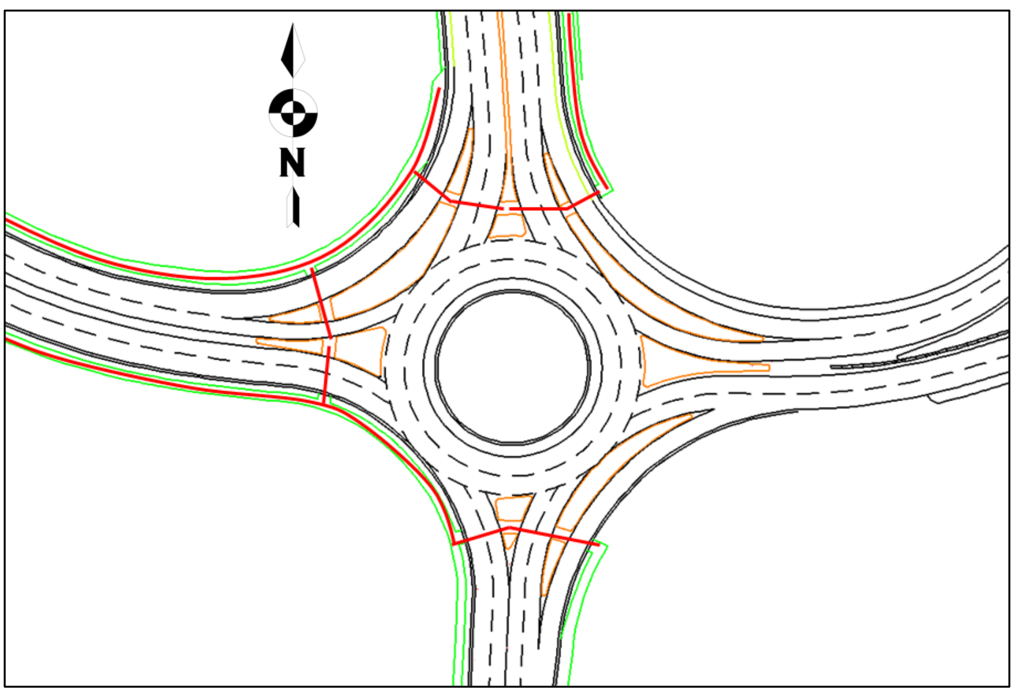


Figure 13 - South Roundabout Pedestrian Access (Alloy Consulting Ltd., 2015)

Alloy designed splitter islands according to NCHRP Exhibit 6-12 (see below) and 6-13 to accommodate both driver and pedestrian safety. A 30 m-long splitter island is recommended and a 15 m-long splitter required as a minimum, which is reflected in our design.

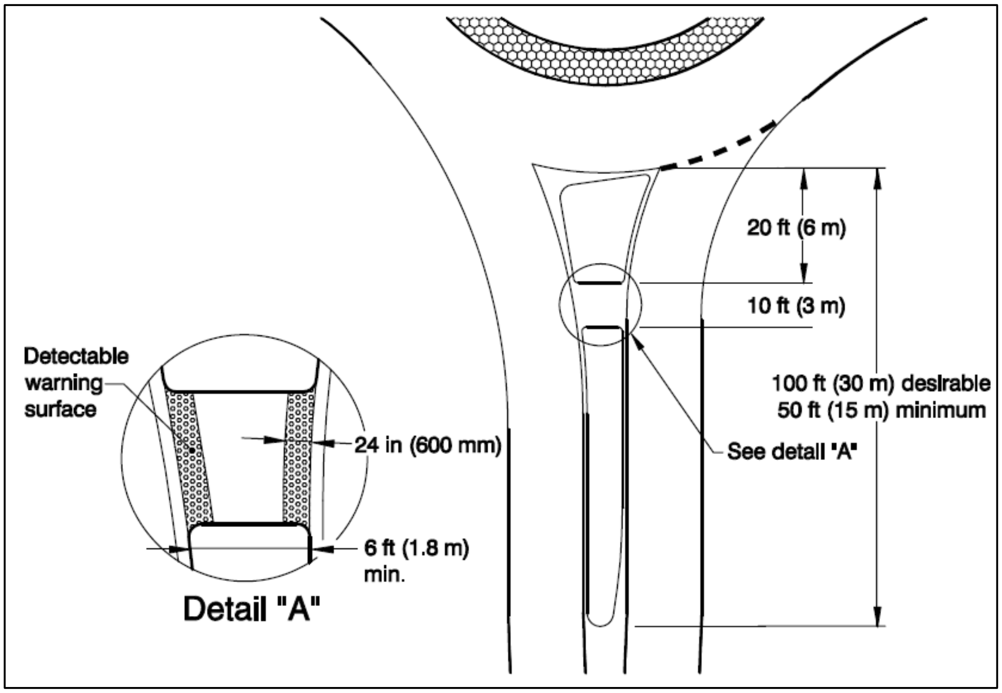


Figure 14 - Splitter Island Example (NCHRP, 2010)

The section of the splitter nearest to the roundabout was designed to a width of 6 m to allow a single car to yield at the roundabout while pedestrians cross behind them. Gaps in splitter and pedestrian islands are required to be 1.8 m long to adequately protect crossers, and 3 m wide to allow strollers or cyclist to pass through without constriction.

The north roundabout includes similar features to the south roundabout. Main differences include the omission of a landscaping strip on the north sidewalks to reduce land taken from local businesses. As shown below, pedestrian access is extended along the northwest side of the roundabout in order to allow access to the nearby hotel.

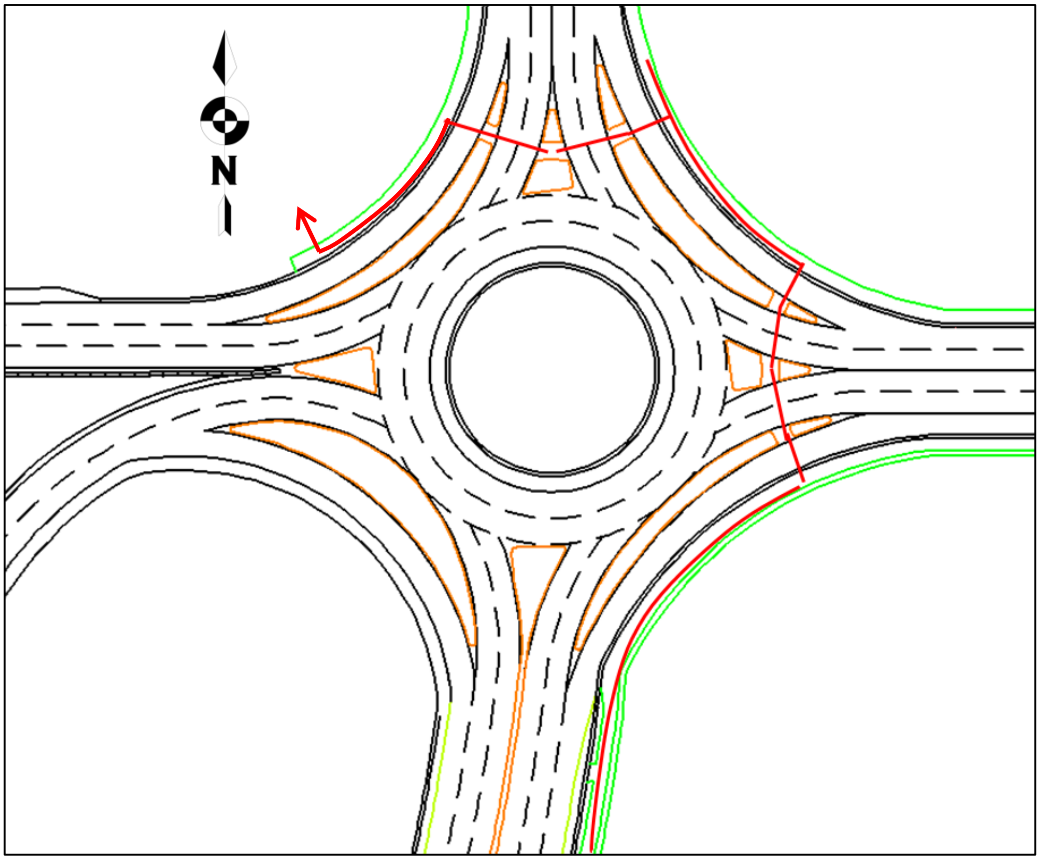


Figure 15 - North Roundabout Pedestrian Access (Alloy Consulting Ltd., 2015)

The roundabout crosswalk markings (zebra style) were designed according to the NCHRP Exhibit 7-7 that is shown below.

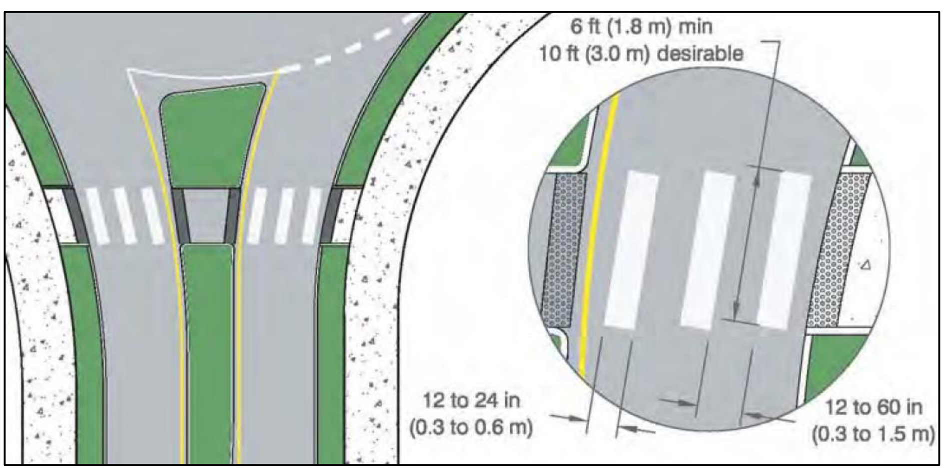


Figure 16 - Pedestrian Crossing Markings (NCHRP, 2010)

Alloy designed these markings according to the desired 3.0 m length, with a marker width of 0.5 m and a gap between markers of 1 m. Access to and from the roundabouts is available on the east side of the overpass with a 2.4 m wide walkway protected by concrete roadside barriers.

5.6 Bicycle Access

Overall, there were more options to include cycling access to users at the south roundabout due to property constraints at the north roundabout. The rightmost lanes heading to and from the roundabout on Clearbrook Road South and Marshall Road East were extended to 4.3 m to allow comfortable shared use by cyclists and drivers (Jacobson, 2009). To encourage cyclists along Marshall Road East and Clearbrook Road South to use the 3 m wide sidewalk along the roundabout (instead of entering the roundabout), Alloy designed bulged bicycle ramps. Exhibit 6-67 from the NCHRP guide was used for the ramp design and can be seen below in Figure 17.

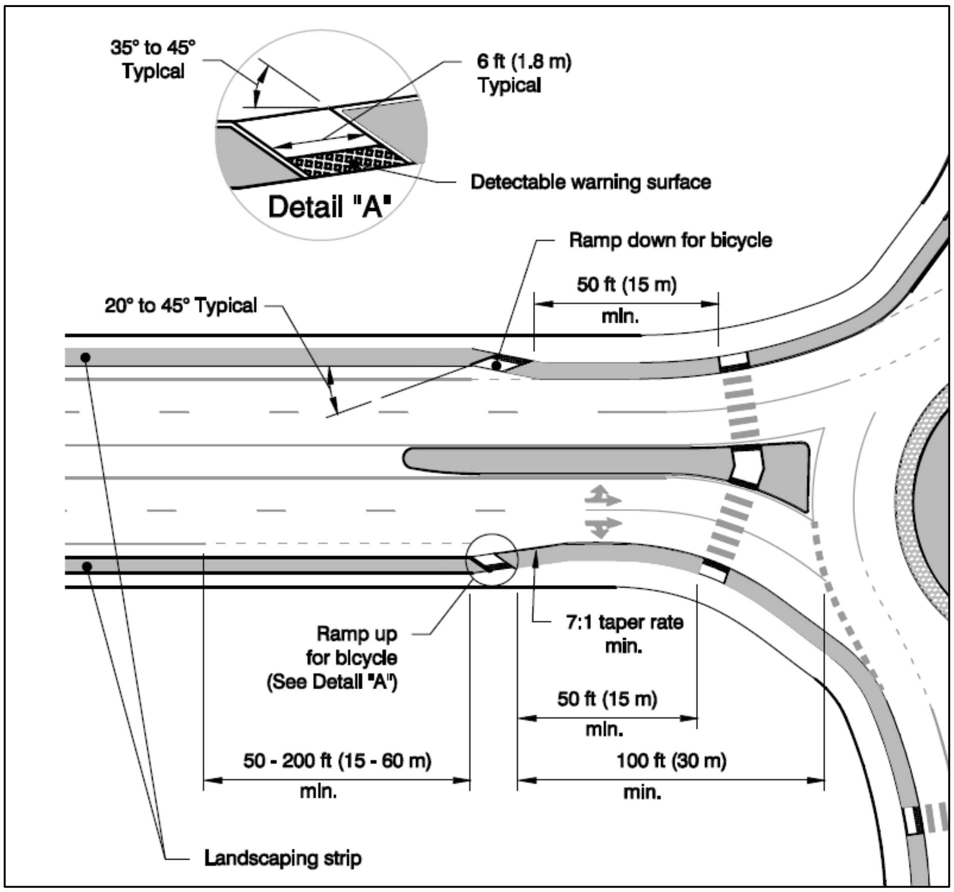


Figure 17 - Bicycle Ramp Design Guide (NCHRP, 2010)

We designed our ramps at least 20 m down from the pedestrian crosswalk (15 m minimum), with angles 30-35˚ from the entering road. The road was tapered at 10:1 to ensure driver safety and the ramp width was expanded to 3 m to ensure an easy transition for cyclists.

Leaving the south roundabout, there is a dedicated cycling lane that is 1.5 m on the east side of the overpass. The bike lane terminates on both the south and north before reaching each roundabout with ramps to the sidewalks.

5.7 Barrier Design

In general, concrete roadside or median barriers are designed where users are at higher risk to in the event of a crash or to protect (or protect from) a structure or hazard. Detailed calculations regarding barrier design can be found in Appendix H, but this section will summarize key decisions or methodology in the barrier design. The connections or transitions between types of barriers were not designed for this project, only the location where concrete barriers will be required. The following are sections of the interchange where barriers are required:

- Across the overpass on both ends
- Below the overpass on the highway to protect against the abutments and pier
- Entrance ramps where lanes where lanes merge

Most barriers require an entrance offset flare to properly introduce the barrier to oncoming traffic without surprise. Most barriers in our project did not have an exit flare except for under the overpass near the abutments. The exit flare dimensions should be compared with the values according to the bridge design manual to confirm their validity.

6.0 BRIDGE STRUCTURAL DESIGN

To provide the client with the best overall solution for the Clearbrook Interchange, Alloy Consulting performed a preliminary structural design for the bridge overpass. The preliminary design of the bridge overpass considered three design alternatives: steel plate girders, precast concrete I-beams, and precast concrete box girders. These three designs were done using approximate bridge dimensions from the early transportation design. To determine the best alternative, Alloy Consulting compared them using the following selection criteria:

- Cost of materials and fabrication
- Ease of installation
- Durability
- Sustainability

Alloy will use the selected bridge girder design in the construction sequencing plan detailed in Section 9.0.

Preliminary structural design calculations have been included in Appendix I. Drawings for the preliminary design have been included in *Volume III – Drawings* of this report.

6.1 Preliminary Design Assumptions

The initial design of the bridge structure was completed using approximate dimensions taken from the transportation model. From the model, it was determined that the overpass would span approximately 50 - 60 m across the relocated freeway. A center pier was added so that the bridge would comprise of two 30 m spans.

The preliminary design width of the bridge was calculated using six 3.7 m wide lanes with additional allowances being required for sidewalks and a center median. From this, it was assumed that the final bridge would be approximately 30 m wide.

The bridge assembly was assumed to be 1.7 m thick in total. This assembly thickness accounts for the asphalt wear surface, concrete structural deck, and the bridge girders. For the preliminary design, the asphalt wear surface was assumed to be 0.1 m thick.

6.2 Design Loads

Alloy Consulting determined the design loads for the bridge in accordance with CSA S6 – 06. Since we did not consider lateral effects, the applicable design loads were either dead or live loads.

Dead loads for bridge include the reinforced concrete deck, an asphalt wear surface or sidewalks where applicable, utilities, and as required by CSA S6 – 06, the weight of water. To determine the load caused by the weight of water, a volume of water equal to the 24 hour rainfall over the bridge area was distributed to suit the bridge cross slope. This distribution is shown in Figure 18 on the following page.

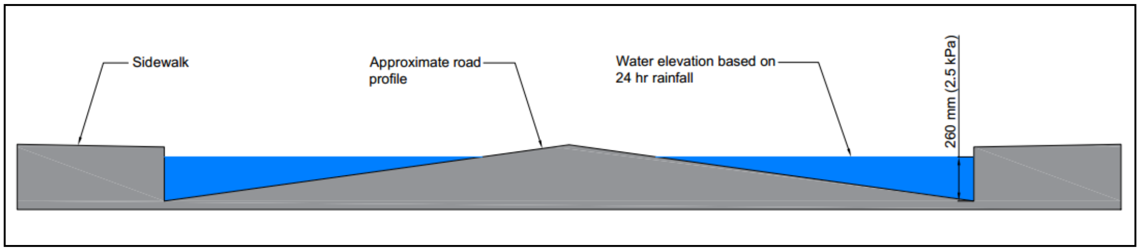


Figure 18 - Water Distribution Across Bridge Section (Alloy Consulting Ltd., 2015)

The maximum additional load from the rain water, as shown above, was determined to be 2.5 kPa where the roadway meets the sidewalks. Though conservative, it was assumed that the water load could occur concurrently with the full traffic loads. Additional dead loads to be considered are the weight of concrete sidewalks.

Live loads acting on the bridge will include all vehicular traffic. For these loads, the Canadian Highway Bridge Design Code specifies the use of CL-W Truck Loading. For this loading, a uniform load of 9 kN/m is applied over a 3 m width with additional point loads from the design truck's tires. This arrangement of additional point loads are fixed relative to each other but can be located anywhere along the bridge span. To determine the location of these loads, a spreadsheet was developed to determine the location that generated the most significant effect. This spreadsheet is discussed further in the following section.

The Canadian Highway Bridge Design Code specifies a sidewalk live load according to the following equation:

$$p = 5.0 - \frac{s}{30}$$

p = Design live load (kPa)
s = Loaded length of sidewalk (m)

The above equation, from section 3.8.3 of CSA S6 – 06, produces a live load of 4.0 kPa.

Since there are several levels of loading over the bridge cross section, the entire cross section was sketched such that the most critical beam could be identified. For the critical beam of each alternative, the critical Ultimate Limits State and Serviceability Limits State load combination was considered.

6.3 Structural Analysis Spreadsheet

As discussed in the previous section, an Excel spreadsheet was developed to determine the location of moving point loads that produces the most significant effect. The spreadsheet was programed using Visual Basics for Application in Excel and it follows the solution process in Table 11 on the following page.

Table 11 - Structural Analysis Spreadsheet Solution Process

Solution Step	Description
1. User Inputs:	The user inputs values for the uniform load on the beam, magnitudes and relative spacing of moving point loads, and the total span of the beam.
2. Initial Calculation:	Using the input values, the spreadsheet calculates the maximum moment that occurs under the assumption that the first point load occurs at the leftmost support. The determination of the maximum moment is done using the Golden Search Method. This maximum moment is stored in the program.
3. Iterative Process:	The program then moves the arrangement of point loads to the right by a small fraction of the span. The maximum moment is calculated for this case and compared to the stored maximum. If the new maximum moment is larger than the previous, this new value is stored to be compared with future iterations. Calculations are performed for the entire span of the beam.
4. Solution Outputs:	When the iterative progress is completed, the condition that yields the maximum moment is outputted to the spreadsheet. With this information, a bending moment diagram is generated.

Printouts of the Excel workbook and VBA code can be found in Appendix I.

6.4 Preliminary Design

Preliminary design of the bridge structure was completed for the concrete deck and three design alternatives: steel plate girder, precast concrete I-beam, and precast concrete box girder. Using results from the preliminary design detailed in Section 6.4.1 to Section 6.4.4, we performed the design comparison and selection detail in Section 6.5.

6.4.1 Concrete Deck

The first element of the bridge structure designed by Alloy Consulting Ltd. was the concrete deck. Instead of using the concrete deck as a wear surface, which would require additional concrete thickness, an asphalt wear surface will be installed on top of the concrete deck. This allows for easier repairs to the driving surface over the course of the bridge's lifespan.

Alloy considered two alternative designs for the bridge deck: precast concrete panels and cast-in-place concrete. The precast concrete panels were initially thought to be the ideal structural solution because they would allow for rapid construction of the bridge deck and would not require concrete to be placed over the freeway. According to CSA S6 – 06 Section 8.18.4.4. (a), the precast panels must span the entire width of the bridge. In this case, 30 m long panels would be required. Such concrete panels are too large to be transported and installed or formed on site and installed. Because of these reasons, a cast-in-place bridge deck was selected.

The cast-in-place bridge deck was designed according to the empirical design method specified in Section 8.18 of CSA S6 – 06. This method provides a list of requirements that must be satisfied by the bridge deck. Such requirements include maximum spans, span to thickness ratios, reinforcing, and more. The design resulting from the CSA S6 – 06 empirical design method is shown in Figure 19 below.

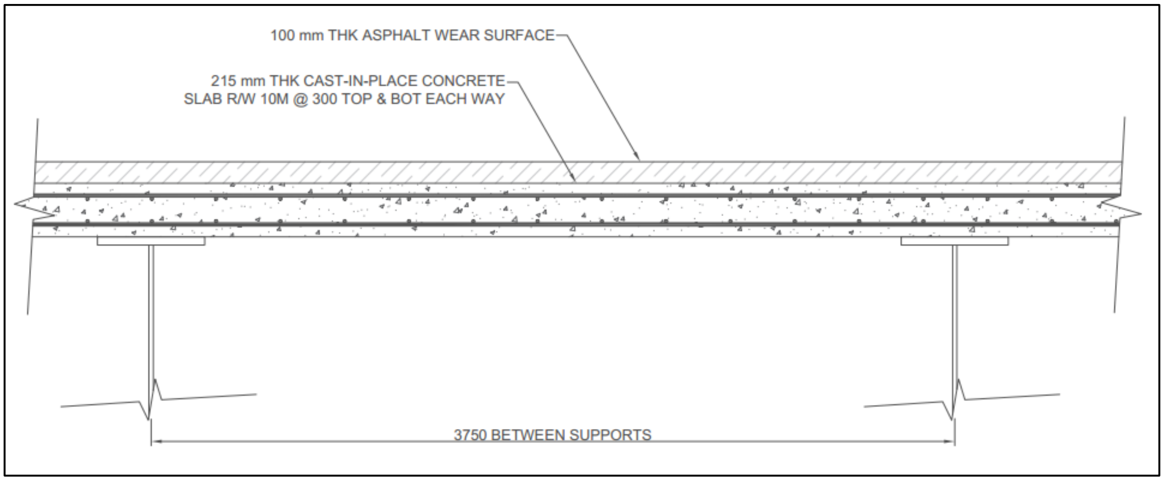


Figure 19 - Bridge Deck Design (Alloy Consulting Ltd., 2015)

As seen above, the cast-in-place bridge deck is 215 mm thick with 10M @ 300 mm top and bottom each way. This deck will be cast on top of the steel plate girder and precast concrete I-beams and will be incorporated into the precast concrete box girder section. For the designed deck, the maximum spacing between support elements is 3750 mm.

6.4.2 Steel Plate Girder

Preliminary design for the steel plate girders was performed in accordance with CSA S6-06 Section 10 and CSA S16-09 using 300 MPa steel.

With the concrete deck set at 215 mm thick, the steel plate girder was limited to 1385 mm in order keep the bridge assembly within the 1700 mm limit. Using this limit, Alloy selected a girder size to satisfy the bending moment effect of the ULS loading. As a composite section is required by the empirical design method of CSA S6 – 06, a composite section design spreadsheet, developed in CIVL 7073, was used for the initial girder sizing (Figure 20). The full bridge will consist of 9 girders spaced at 3750 mm.

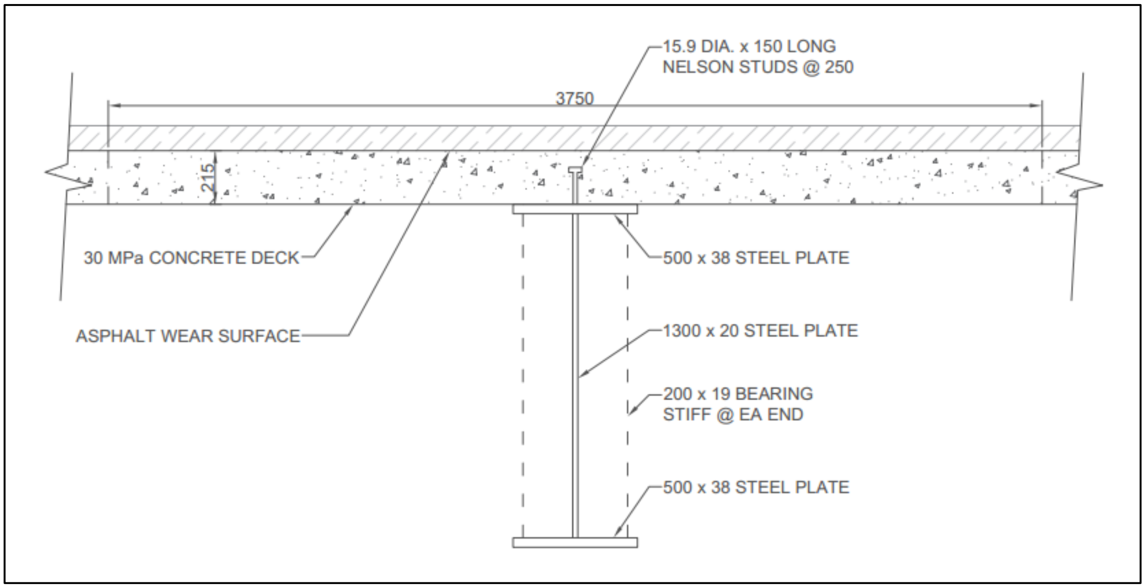


Figure 20 - Steel Plate Girder Preliminary Design Section (Alloy Consulting Ltd., 2015)

The girder size shown in the above figure was checked for each of the following design requirements: bending, shear, bearing, deflection, and lateral bracing.

Following the initial section design using ULS loading and the capacity of the composite section, two additional bending checks were performed. The first check was to verify that the steel plate girder could support loads placed on it prior to the introduction of composite action. For the non-composite load case, the following loads were applied:

- Dead Loads - Self-weight, wet concrete
- Live Loads - 2.0 kPa construction load as specified by Worksafe BC

The selected plate girder was checked against the factored effects of these loads and it was determined that the girder could support these loads while staying fully elastic.

The second additional bending case checked was the effect of service loads in addition to the effects of pre-service loading. For this case, the bending stress in the girder due to loading applied prior to service is 90 MPa and the additional effect of service loading is 120 MPa. The effects of service loading was determined using the transformed moment of inertia and the following bending stress equation:

$\sigma = \frac{Mc}{I_t}$	*σ = Bending Stress* *M = Maximum moment in beam* *c = Distance from neutral axis to extreme fibre* *I_t = Transformed moment of inertia of composite section*

Since the combined effects of bending stress sum to less than the yield stress of the steel plate girder, the girder is sufficient for bending in this case.

Following the flexural design, Alloy compared the shear capacity of the selected girder to the effects of the bridge loading. Since it would prove to be financially advantageous, the girder was first checked with its unstiffened shear capacity. An unstiffened web is permitted according to CSA S6-06 Clause 10 since the web depth-to-width ratio is less than 150. The resulting design check revealed that the shear capacity of the girder is nearly two times the required capacity.

A bearing design check was performed to determine whether or not bearing stiffeners were required at supports. The *Highway Bridge Design Code* provides two checks for determining the need for bearing stiffeners on steel girders. According to the first design check, a bearing length of 270 mm would be required to forego bearing stiffeners. The second bearing design check revealed that the web bearing capacity was insufficient despite a possible increase in bearing length. As such, Alloy designed the required bearing stiffeners. To satisfy the bearing capacity and dimensional requirements in CSA S6-06, 200 mm x 19 mm bearing stiffeners are required on each side of the girder web.

The *Highway Bridge Design Code* does not provide limitations on deflections for dead loads; rather, they only provide limitations of live load deflections not exceeding 0.1 % of the span length. In our design of the steel plate girder, we checked both dead and live load deflections. The dead load deflection, though not a design criterion, is required for determining that amount of camber to be placed in the girder during fabrication. Dead load deflection analysis was done in two steps: deflection prior to composite action and deflection after composite action. Combining the effects of both types of deflection resulted in a total dead load deflection of 75 mm.

The live load deflection was checked using the composite section's transformed moment of inertia. Since the live load case composes of several unequally spaced point loads, we used SAP 2000 to analyze the deflection. The resulting deflection was 37 mm, or 0.12 % of the bridge span. This deflection is 20 % greater than the permitted deflection, but there was insufficient time to revise the girder such that it satisfied the CSA S6 – 06 requirements. To revise the section to meet the deflection requirements of CSA S6-06, the girder flanges should be enlarged. Increasing the flanges would be more efficient than increasing the web since the moment of inertia is increased at a greater rate when mass is added further away from the neutral axis.

The final consideration in the design of the steel plate girder is the requirement for lateral bracing. For the sake of this project, we will not design the lateral bracing; it will, however, be considered in the comparison of the bridge design alternatives. The steel plate girder requires lateral bracing for the bottom flanges of all external girders to help resist wind loads. Additional lateral bracing is required between all girders in order to satisfy requirements of the empirical concrete deck design. Clause 8.18.5 of the *Highway Bridge Design Code* specifies that cross-frames are required at spacing no greater than 8.0 m. This results in 48 cross-frames that must be installed following the girder installation.

A complete summary of the design results are listed in Table 12 below. Drawings of the completed preliminary design can be found in *Volume III – Drawings* of this report. All calculations for the preliminary steel plate girder design can be found in Appendix I.

Table 12 - Steel Plate Girder Design Summary

Item	Design Value / Capacity	Required	Comments
Flexure - ULS	11700 kN.m	11700 kN.m	Pass
Flexure - Construction	8200 kN.m	4550 kN.m	Pass
Flexure - Const. + Service	300 MPa	210 MPa	Pass
Shear	3300 kN	1500 kN	Pass
Bearing	3100 kN	1500 kN	Pass
Deflection - Dead	75 mm	---	To be accounted for via cambering
Deflection - Live	37 mm	30 mm	Fail - Requires increased girder size

As seen above, the steel plate girder design satisfies the ULS design requirements, however, the selected girder must be revised to meet SLS design requirements.

6.4.3 Precast Concrete I-Beams

Alloy Consulting Ltd. completed the preliminary design of the precast concrete I-Beams in accordance with CSA S6 – 06 Section 8 and CSA A23.3 – 14.

To improve the effectiveness of the concrete I-beam design and to improve the learning outcomes of this project, we decided that pre-stressed concrete should be used in the design. Though clause 8.4.1.2 of *CSA S6 – 06* specifies minimum concrete compressive strength of 35 MPa, our analysis of the beam determined that the concrete should be 40 MPa.

Similar to the steel plate girder design, the maximum depth of the concrete I-beam is limited by the total depth of the bridge assembly. So that dimensions were rounded, we limited the concrete I-beam depth to 1350 mm. With the height of the girder set, we needed to determine the required flange dimensions to resist the compressive force induced by the applied moment. To do this, we approximated the distance between to the tensile and compressive forces of the member and chose flange dimensions that could resist the resulting force. From this analysis, we determined that the flange would need to be approximately 750 mm wide and 500 mm deep.

After we had a general idea for the dimensions of the precast concrete I-beams, a spreadsheet was developed to calculate the moment resistance of a concrete I-beam with pre-stressing tendons and tension and compression reinforcing. The calculations for the capacity of the prestressed beam are in accordance with CSA A23.3 – 14 Section 18. For the calculation procedure, we referenced Example 10.1 of the *Concrete Design Handbook*. A printout of the spreadsheet results can be found in Appendix I and the spreadsheet file can be found on the USB drive included with this report.

Using the design spreadsheet, the following section was developed with a non-composite moment capacity of 9700 kN.m (Figure 21).

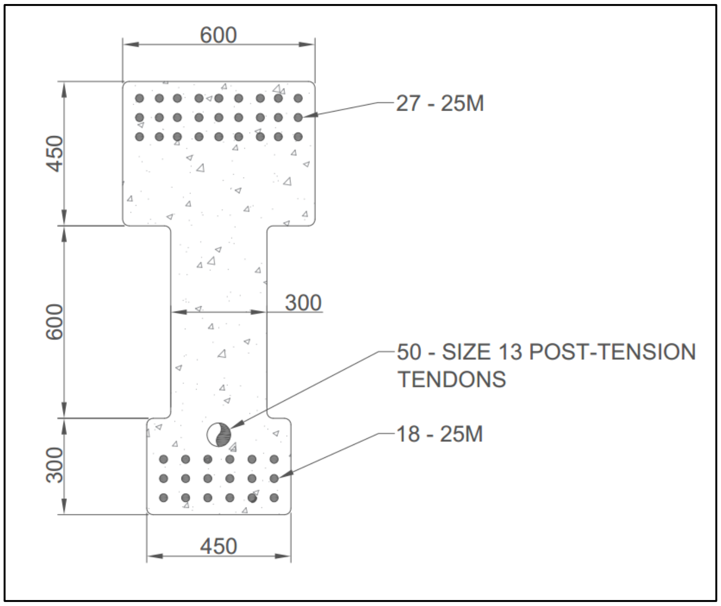


Figure 21 - Initial Sizing of Precast Concrete I-beam (Alloy Consulting Ltd., 2015)

Initial analysis of the above section suggest that it is not efficient given the large amount of compressive reinforcing required to resist the applied bending moment. An increased girder depth would improve the efficiency of the member, but it is not permitted because of clearance limitations.

Next, we needed to confirm that the girder could function compositely with the cast-in-place concrete deck to resist the service loading case. Using the same spreadsheet as was used for the non-composite section, inputs were revised to treat the effective slab width as the beam flange. Compressive reinforcing in the girder was ignored for this calculation since they we located very near the neutral axis of the composite section. We used the spreadsheets for the non-composite and composite sections in parallel to develop a section the satisfied the flexural requirements of both cases. Doing this, we generated the following final section for the precast concrete I-beam.

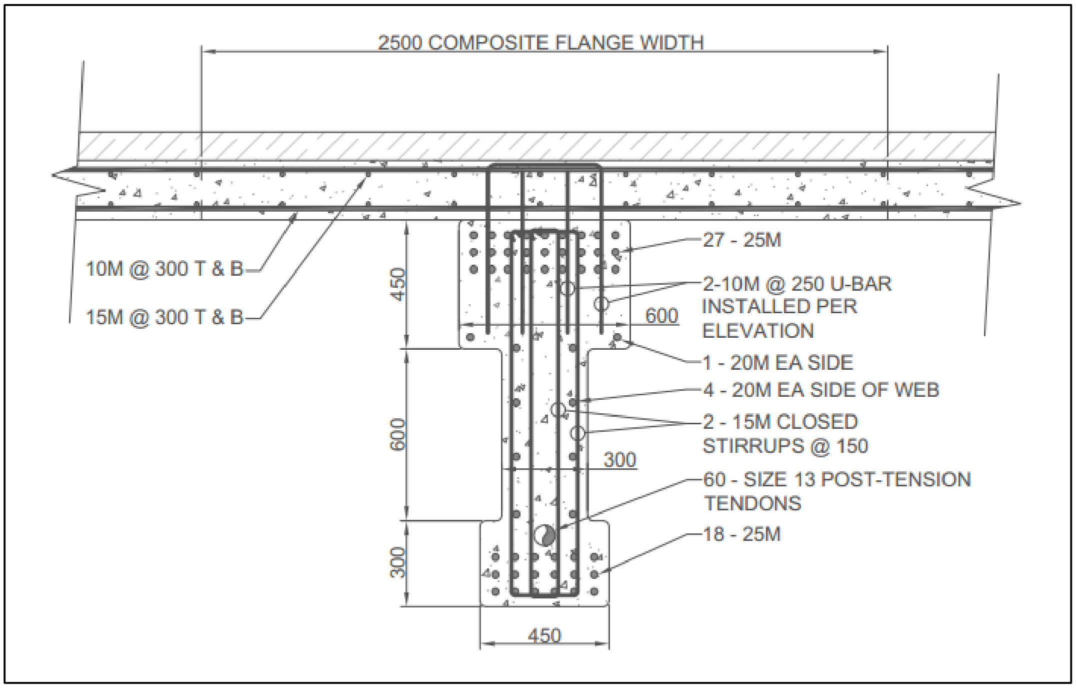


Figure 22 - Precast Concrete I-beam Preliminary Design Section (Alloy Consulting Ltd., 2015)

Following the design of the flexural reinforcing, we needed to design additional reinforcing at the slab and beam interface to ensure composite action of the section. Since the compressions block of the composite section is below the underside of the concrete slab, the transfer force is equal to the compressive force in the slab. This force, 12,900 kN, must be resisted by interface shear and diagonal shear reinforcement. We designed the diagonal shear reinforcement according the CSA A23.3 – 14 Clause 11.5. The resulting design was to have two 15M U-bars @ 250 mm angled at 45˚ towards the center of the beam. The installation of the diagonal reinforcing will likely be quite challenging. Additionally, it will make the installation of slab reinforcing near the precast girders more difficult.

Shear design of the section was performed in accordance with CSA S6 – 06 Clause 8.9. Using the 300 mm wide flange to resist shear, we determined that a large amount of additional shear reinforcing was required to satisfy load requirements. The resulting design includes 2 – 15M closed stirrups spaced at 150 mm for the entire span of the girder. With the shear design completed, we checked it against the maximum permitted shear capacity of a section as specified by CSA S6 – 06; our girder satisfied the requirement by 17%.

We performed a bearing check of the beam according to CSA S6 – 06 Clause 8.8.7 and determined that 175 mm of bear length is required at each support.

The final checks performed for the precast concrete I-beam were for serviceability. To determine the deflection of the precast I-beams under different loading, the gross and cracked moment of inertia were determined. For the sake of this project, we did not consider the effects of composite action in determining the moment of inertia of the section. The gross moment of inertia was calculated by ignoring the contribution of the steel reinforcing and only considering the gross area of the concrete. To determine the cracked moment of inertia of the girder section, we first determined the location of the neutral axis using Goal Seek in Microsoft Excel. Using the determined location of the neutral axis, the cracked moment of inertia was found accounting for reinforcing and prestress tendons. When determining the effective moment of inertia, we chose to use the value of the cracked moment of inertia since the flexural effects were much larger than the uncracked moment resistance of the section. The following are the determined deflections:

Instantaneous dead load deflection: 270 mm
Long term dead load deflection: 540 mm
Instantaneous live load deflection: 89 mm

The final item that we checked in the design of the precast concrete I-beams is the requirement for crack control reinforcing in the web. According to CSA S6 – 06 Clause 8.12.4, 5 – 20M bars are required on each side of the web. To prevent cracking in the corners of the top flange, one of the crack control bars from each face of web were placed in the bottom corners of the top flange.

A complete summary of the design results are listed in Table 13 below. Drawings of the completed preliminary design can be found in *Volume III – Drawings* of this report. All calculations for the preliminary precast concrete I-beam design can be found in Appendix I.

Table 13 - Precast Concrete I-beam Design Summary

Item	Design Value / Capacity	Required	Comments
Flexure - Non-composite	10900 kN.m	9000 kN.m	Pass
Flexure - Composite	14200 kN.m	13700 kN.m	Pass
Interface Shear	1.37 MPa	1.14 MPa	Pass
Shear	1772 kN	1675 kN	Pass
Bearing (Bearing Area)	78750 mm^2	76000 mm^2	Pass
Deflection – Inst. Dead	270 mm	---	Significant
Deflection – Long-term Dead	540 mm	---	Significant amount to try to camber – too high
Deflection – Inst. Live	89 mm	30 mm	Fail by factor of 3

In considering all aspects of the precast concrete I-beam design, this girder type does not appear to be the optimal selection.

6.4.4 Precast Concrete Box Girder

The final design alternative that we considered for the bridge girders is precast concrete box girders. The first step in the design process for the precast concrete box girders was to determine the arrangement of the girders. In determining the arrangement of the girders, we considered the location of concentrated loads and the number of required girders. Figure #, below, shows one half of the final arrangement of the precast concrete box girders. The arrangement includes six box girders in total and is symmetrical about the centerline of the bridge.

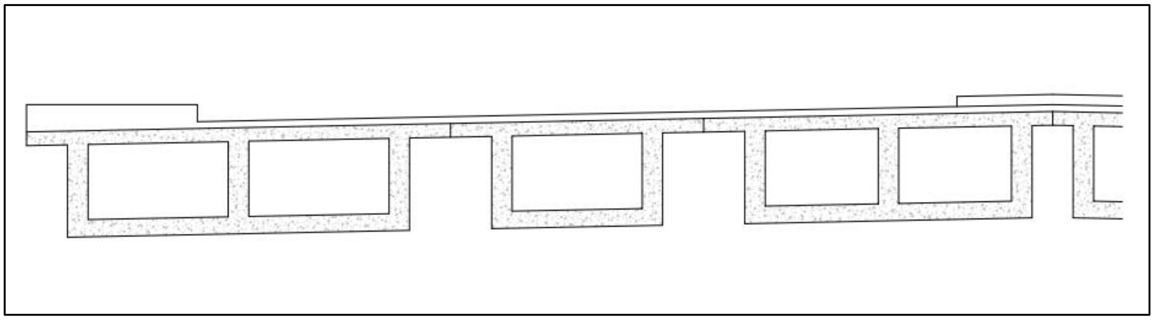


Figure 23 - Half of Precast Concrete Box Girder Arrangement (Alloy Consulting Ltd., 2015)

From the arrangement shown above, we chose to prepare a design for the exterior girder member to compare with the other design alternatives. Dimensions of the member were selected to create a symmetric beam. The symmetric nature of the beam will make construction of the member easier since similar formwork can be used in both openings.

Since the box girder layout changed the previously designed deck slab, we redesigned the slab deck with the revised dimensions. The deck was designed using the empirical design method as described in Section 6.4.1. From this design, we determined that a 200 mm thick deck reinforced with 10M @ 300 mm top and bottom was sufficient.

Analysis of the box girder required a revision to the load analysis described in Section 6.2 of this report. We recalculated the applied loads and determined that the self-weight of the girder was equal to the uniform loading that would be placed on the beam in service. This effect was not too surprising considering the large size of the beam.

We started the flexural design of the precast concrete box girder by performing preliminary calculations regarding the depth of the compression block and the amount of reinforcement required in the bottom flange. To determine the approximate depth of the compression block, we assumed that the tensile reinforcing would be centered in the middle of the bottom flange. With the approximate depth of the compressive block, we could calculate the depth of the tensile reinforcing and determine the amount of tension reinforcing required. Using the 100 – 25M that we determined to be required, the precise moment resistance of the girder was calculated and checked against the factored load effects.

For the shear design of the section, we assumed that the shear stress is shared equally between the three webs of the girder. Using this assumption, we determined that each web requires 15M stirrups at 150 mm near the supports. Since this concentration of stirrups is not required in the middle portion of the beam, we increased the spacing of the stirrups to 600 mm.

Since we could not rely of the entire length of the bottom flange to resist bearing, we assumed that shear transferred through the beam webs were spread out at a 2V:1H ratio. From this assumption, we determined that a bearing length of 150 mm should be provided at each support.

Deflection for the precast box girder design was done in the same way as the precast concrete I-beam design. We calculated the gross and cracked moments of inertia by hand and used them to determine the deflection under different loading conditions. Since the concrete deck is incorporated into the construction of the box girder design, we did not need to calculate separate deflections for construction and service conditions. We calculated the deflection of the beam under the effects of dead loads, dead and live loads, and finally live loads on their own. The calculated deflections are as follows:

Instantaneous dead load deflection:	80 mm
Long term dead load deflection:	240 mm
Instantaneous live load deflection:	56 mm

The deflections listed above are less than those calculated for the precast concrete I-beams but are still larger than what should be permitted. Had we used a prestressed concrete design, the deflections could have been further reduced since the effect of prestressing increases the size of the compressive concrete area which increases the cracked moment of inertia.

Similar to the design of the precast concrete I-beams, additional crack control reinforcing is required in the webs of the girder. Using the requirements specified in CSA S6 – 06 Clause 8.12.4, we determined that 6 – 20M reinforcement bars are required on each face of the beam webs.

Figure 24, below, shows the final preliminary design of the precast concrete box girder.

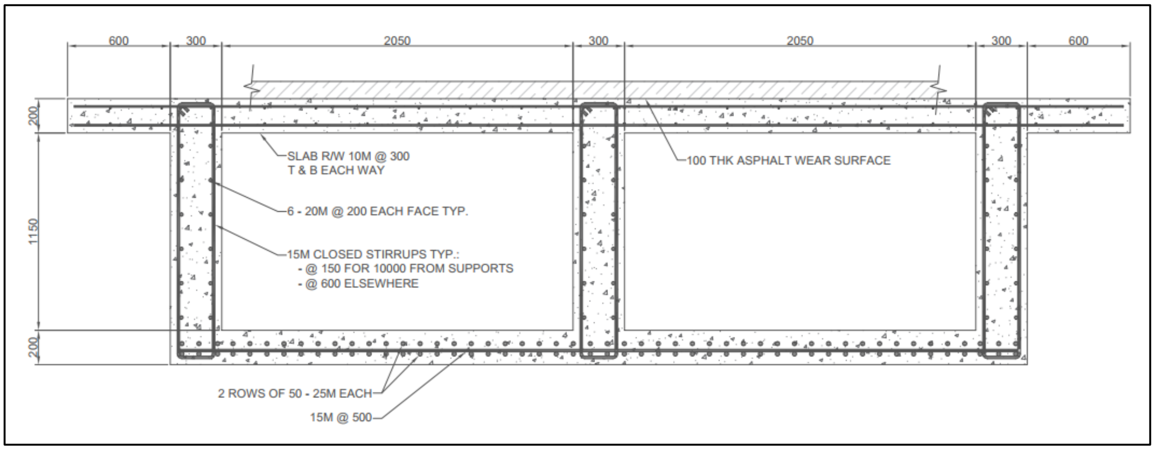


Figure 24 - Precast Concrete Box Girder Preliminary Design Section (Alloy Consulting Ltd., 2015)

A summary of the design results can be found in Table 14 below. Drawings of the completed preliminary design can be found in *Volume III – Drawings* of this report. All calculations for the preliminary precast concrete box girder design can be found in Appendix I.

Table 14 - Precast Concrete Box Girder Design Summary

Item	Design Value / Capacity	Required	Comments
Flexure	23900 kN.m	22500 kN.m	Pass
Shear	3300 kN	2950 kN	Pass
Bearing (Bearing Area)	195000 mm^2	133000 mm^2	Pass
Deflection – Inst. Dead	80 mm	---	Significant
Deflection – Long-term Dead	240 mm	---	Significant
Deflection – Inst. Live	56 mm	30 mm	Fail by factor of about 2

As seen above, the precast concrete box girder design satisfies the ULS design requirements but, without considering the effects of prestressing, the box girder design does not satisfy SLS design requirements.

6.5 Design Selection

Selection of the preferred design option was done by comparing their performance in four categories: function, cost, constructability, and sustainability. We gave each selection criteria a weight such that the maximum overall score for an option would be 100.

Printouts for the environmental assessment of the structural design options and a summary of the design comparison can be found in Appendix J.

6.5.1 Function

We scored the function of each alternative based on the structural performance of the option. We considered the efficiency of the design as well as how much additional work we felt would be required in order to make the option structurally viable. The additional work factors for improving the serviceability of the members and for the inclusion of prestressing in the designs.

The steel plate girder design performed the best in terms of the functionality of the member. The preliminary design of the steel plate girder satisfies the ultimate limit states design requirements of *CSA S6 – 06* and could be modified to satisfy the serviceability requirements quite easily.

The precast concrete I-beam design was the least effective structural alternative. For the dimensional limitation of the project, we needed to put in a lot of effort to create a member that satisfied the ULS requirements of the girder. The design of the member required a very large amount of both tension and compression reinforcing in addition to prestressing tendons. Deflections of the designed member were three times larger than the required values, which we felt was too large to be fixed through a secondary design or by considering the effects of prestressing.

We graded the function of the precast concrete box girders only slightly below that of the steel plate girder design. The precast concrete box girder design works efficiently to satisfy the ULS requirements of *CSA S6 – 06*. As for serviceability, we felt that modifying the design to incorporate prestressing would make the box girder design satisfy the SLS requirements of *CSA S6 – 06.*

6.5.2 Cost

To compare the relative costs of each structural alternative, we referred to articles and case studies on the topic. According to a case study of two similar short span bridges in Montana, a steel girder design is more cost-effective (U.S. Bridge). However, according to a separate report by a supporter of concrete construction, concrete bridges are more cost-effective (Hurd, 1985).

Since it appeared as though the results of each report were strongly influenced by the personal view of each writer, we determined that the cost for steel and concrete bridge options were quite similar. As such, we scored each option according to the potential difference in construction costs. Because the precast concrete box girder design does not require a deck to be installed following the girder installation, we gave the highest score to this design option.

6.5.3 Constructability

The score of each option for constructability was determined by considering two separate stages of construction: fabrication and installation. From our comparison of the options, all three options had similar results, with precast concrete I-beams being slightly worse than steel plate girders and precast concrete box girders.

The fabrication process for steel plate girders will be a quick process provided the material is locally available at required time. Installation of the girders will be possible within a single night, however, additional work is required after the girder installation. The placement of the concrete deck, with its required formwork, will be difficult above an operating freeway and will be costly and schedule intensive.

Similar to the installation of the steel plate girders, the installation of the precast concrete I-beams can be done within one night, with additional work being required for the concrete deck. Fabrication of the concrete I-beams, however, is more difficult given the large amount of reinforcing and prestressing cables. Fabrication will also take longer because the concrete must cure before it is installed.

We scored the constructability of the precast concrete box girder the same as the steel plate girder. Fabrication of the members will take longer than the steel, though the amount of reinforcing is reasonable compared to the precast concrete I-beams. Installation of the concrete box girders will be challenging given their large size, but the incorporation of the concrete bridge deck into the design saves the contractor the additional work required to install it afterwards.

6.5.4 Sustainability

To compare the relative sustainability of each structural option, we used the CES Edupack 2014 software. Using the Eco Audit function of the software, we compared the sustainability of the alternatives based on several different characteristics: material type and amount, transportation, and disposal/ reuse.

Using the CES Edupack 2014 software, we determined that all three structural alternatives had relatively similar environmental impacts. The comparison of each alternative's CO_2 footprint is shown in Figure 25 on the following page.

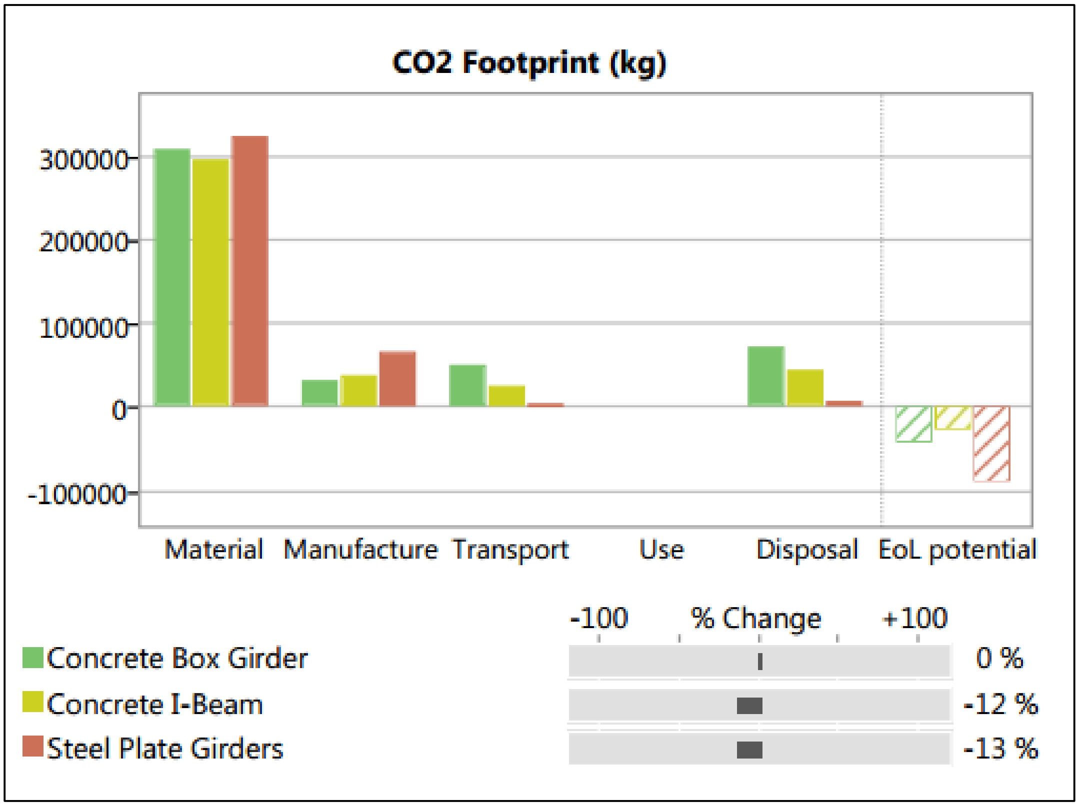


Figure 25 - Comparison of CO2 Footprint for Structural Options

As seen in Figure 25, the CO_2 footprints of the different structural design options vary by less than 15%. Additionally, the relative energy consumptions of the different options also vary by less than 15%. The plot of the energy consumption comparison, as well as summaries of the environmental impact for each option, can be found in Appendix J.

Despite the similar consumption of energy and emission of CO_2, we chose to score the steel design option higher because a larger portion of the material can be recycled. The concrete in the precast I-beams and box girders can be crushed and reused for aggregates and fill material but the reinforcing is more difficult to recycle. The steel design option, however, can be recycled more easily.

6.5.5 Recommendations

Based on the design comparison described in the sections above, we determined that the steel plate girder design is the best design option. The results from the comparison are summarized in Table 15 below.

Table 15 - Preliminary Structural Design Option Comparison

Option	Function	Cost	Constructability	Sustainability	Total Score
Steel Plate Girder	16	18	18	12	64
Precast Concrete I-Beam	5	18	15	10	49
Precast Concrete Box Girder	14	21	18	10	63

As seen in Table 15, the precast concrete box girder scored only slightly below the steel plate girder design. Because of this, we recommend that analysis is done to compare the structure efficiency of prestressed box girders and steel plate girders. If this analysis shows that the prestressed concrete box girder is a viable structural option, then a second, more detailed, comparison should be completed to determine the ideal structural option. This analysis, unfortunately, could not be completed as a part of this project.

Logan Brown Jacob DeVos Keith Dodgshon Jeff Ferraby Matt Hackett

7.0 ABUTMENT DESIGN

Two bridge abutments are required to support the Clearbrook overpass at both the north and south ends. As detailed in the *Clearbrook Interchange Improvement Project – Document of Requirements,* the bridge abutments are to be of reinforced concrete construction. The document also suggests that integral abutments be constructed. An integral abutment is supported on a row of piles cast monolithically with the bridge deck. This encases the bridge girders and allows for rotational movement of the abutment. Integral abutments are preferred because they do not require expansion joints, eliminating water penetration through the bridge deck, and reducing maintenance costs.

Due to their design complexity, integral abutments were not designed for this project. Instead, a simpler, full cantilevered abutment was designed. The cantilevered abutment consists of a vertical arm (stem) rigidly fixed to a horizontal footing. Similar to a cantilevered retaining wall, the abutment uses the opposing cantilever action of the stem and footing to balance horizontal and vertical earth pressures, as well as the abutment self-weight and bridge loading. A typical cantilevered abutment is shown in Figure 26.

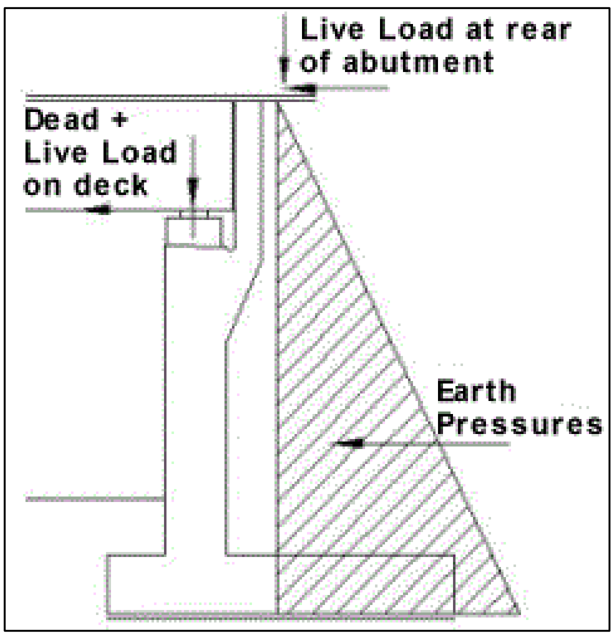


Figure 26 - Typical Cantilevered Abutment (Childs, 2009)

Only one abutment was designed, based on the assumption that both north and south abutments would be nearly identical. Of note, abutment wingwalls are required to retain embankment backfill; however, their design was not completed as part of this project. A full set of calculations is included in Appendix K, while reference documents are included in Appendix L.

7.1 Preliminary Sizing

The preliminary sizing of the abutment was completed using guidelines outlined in the *National Cooperative Highway Research Program Report 343: Manuals for the Design of Bridge Foundations*

(NCHRP 343). The NHCRP is an American national academy that promotes and supports research in the various transportation fields. The preliminary dimensions are included in Figure 27.

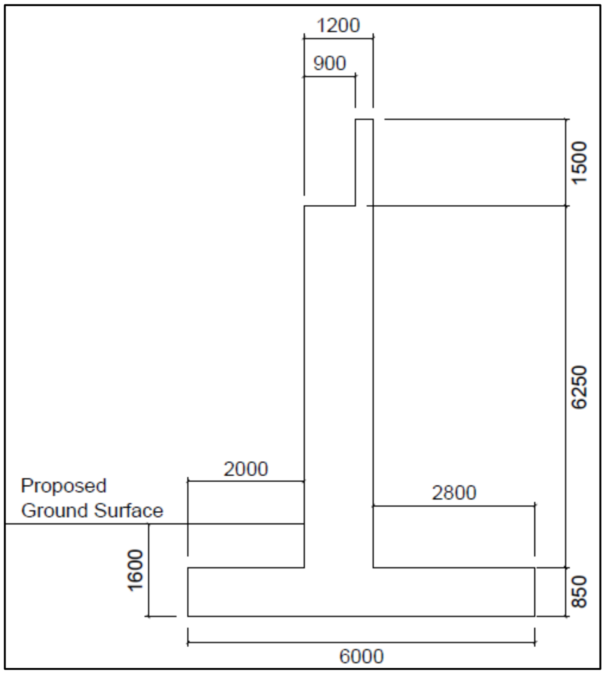


Figure 27 - Preliminary Dimensions of the Cantilevered Abutment (Alloy Consulting Ltd., 2015)

Based on similar overpasses, an assumption of 5.5 m clearance between the proposed ground surface and the bottom of the bridge girders was made.

7.2 Abutment Design

In designing the abutment a number of design values were assumed. It is assumed that the 30 m bridge will consist of two pier-to-abutment lengths of 15 m. The bridge width is assumed to be 30 m, while the total lane width is assumed to consist of 6 lanes of 3.7 m, for a total bridge lane width of 22.2 m. Finally, the bridge is assumed to be skewed at 15° to Highway 1.

A design example included in the Arizona Department of Transportion's *ADOT Bridge Design Guidelines* was used as a basis for design. The design example, *LRFD Substructure Example 1: Full Height Abutment on Spread Footing,* evaluates the abutment on the following criteria:

- Bearing Resistance
- Settlement
- Sliding Resistance
- Overturning
- Structural Resistance

These criteria are the same as those outlined in the *2006 Canadian Highway Bridge Design Code* (S6-06). While the methods of the design example were followed, calculations were adjusted to meet the standards (load combinations, load factors, resistance factors) of S6-06. The abutment was designed to resist both lateral and transverse loads.

7.3 Loads and Load Combinations

The load types considered in the abutment design are summarized in Table 16.

Table 16 - Summary of Abutment Design Values

Load Type	Symbol	Notes
Dead	D	includes bridge superstructure, abutment self-weight, and vertical earth pressures
Earth Pressure	E	includes horizontal earth pressures
Live	L	includes vehicular live load, vehicular braking forces, and live load surcharge
Wind	W	includes wind load applied to the superstructure and transferred to the substructure
Strain	K	includes the frictional forces generated between the superstructure and bearing pads
Earthquake	EQ	includes dynamic earth pressures

Of note, the wind load applied directly to the substructure was not considered.

Load combinations were derived from S6-06. For all design criteria, the load combinations producing the maximum, or minimum when applicable, effects were considered. A summary of load combinations used in the abutment design is included in Table 17.

Table 17 - Load Combinations Considered in the Abutment Design

Limit State	S6-06 #	Combination
Ultimate	ULS #1	1.25D + 1.25E + 1.7L
Ultimate	ULS #2	1.25D + 1.25E + 1.6L +1.15K
Ultimate	ULS #3	1.25D + 1.25E + 1.4L +1.0K + 0.5W
Ultimate	ULS #4	1.25D + 1.25E + 1.25K + 1.65W
Ultimate	ULS #5	1.0D + 1.0 E + 1.0EQ*
Ultimate	ULS #9	1.35D + 1.25E
Serviceability	SLS #1	1.0D + 1.0E + 0.9L + 0.8K

*Earthquakes loads were calculated using Mononobe Okabe analysis resulting in a combined static and dynamic earth pressure value (as such, 1.0E is not used in ULS #5).

Load combinations not included in the analysis (ULS #6, ULS #7, ULS #8) contain exceptional loads that are not applicable to the abutment design.

7.4 Bearing Capacity

In calculating the maximum applied bearing pressure, the Ultimate Limit State load combinations were calculated. The maximum applied pressure, from ULS #2, was found to be 317 kPa with a reduced footing width for load eccentricities of 4.83 m. From the bearing resistance chart included in the geotechnical report, the factored allowable bearing capacity for a foundation with a width of 4.83 is roughly 440 kPa. Since the maximum applied pressure is less than the maximum allowable bearing capacity, the abutment satisfies the bearing requirements.

7.5 Settlement

Settlements, based on the Serviceability Limit State, were calculated using the method of three dimensional elastic integration (see section 2.1.5 of this report). The method, outlined in the *2006 Canadian Foundation Engineering Manual,* requires further assumptions to be made regarding soil properties. The abutment foundation soils were modelled as sand and gravel underlain by clay. A summary of the soil parameter assumptions used in the settlement calculations is included in Table 18.

Table 18 - Assumed Soil Parameters for Settlement Calculations

Layer	Depth (δh)	Young's Modulus (E)	Poisson's Ratio (ν)
Sand and Gravel	6.5 m	100 MPa	0.4
Clay	23.5 m	50 MPa	0.5

Values of Young's modulus and Poisson's ratio were selected based on accepted ranges for each soil type (Bowles, 1997). However, the most conservative values from those ranges were selected for the calculations in order to maximize settlement, and insure the design remained within acceptable settlement limits. Based on the preceding values, the abutment is predicted to settle roughly 40 mm.

Differential settlement is of greatest concern when designing a bridge. S6-06 does not specify tolerable limits for differential settlement; however, the American Federal Highway Administration (FHWA) recommends that the ratio differential settlement to span length be limited to less than 0.005 for single span bridges (Federal Highway Administration, 1985). Taking one 15 m section of the Clearbrook overpass as a single span, and conservatively assuming that no settlement occurs at the pier, the ratio of differential settlement to span length is 0.027 (0.040/15), well-within tolerable limits.

To confirm the three dimensional elastic integration calculations, a model to measure settlement was developed using Sigma/W computer software. The same soil parameters as those used in the calculation were used in the model. The model however included was designed to include the built up soil required for the bridge approaches. The completed model is included in Figure 28.

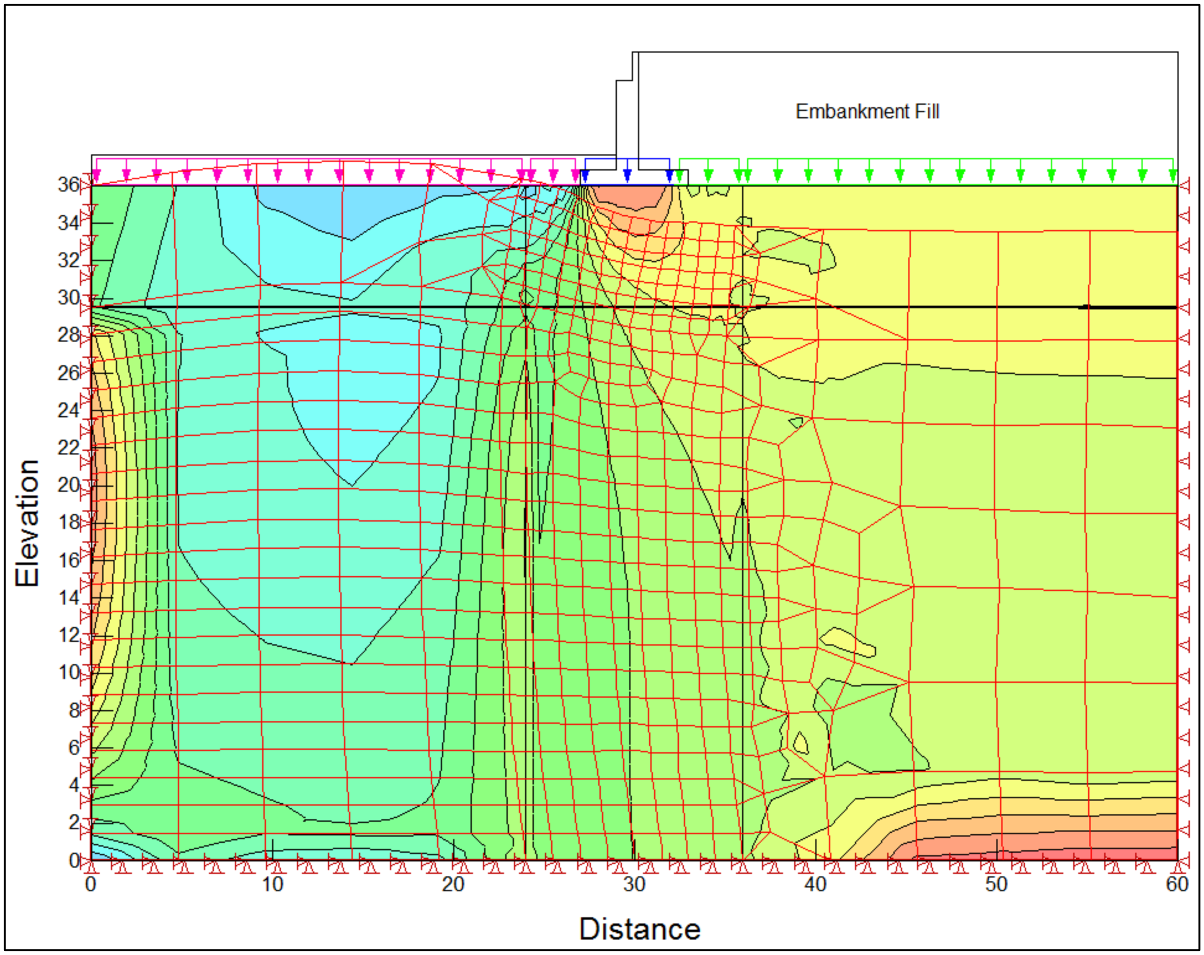


Figure 28 - Sigma/W Model for Abutment Settlement (Alloy Consulting Ltd., 2015)

The Sigma/W model resulted in a maximum settlement of 25 mm, confirming that the differential settlements will be within tolerable limits.

7.6 Sliding

To insure that the abutment can resist lateral loading, its' resistance to sliding was checked. The sliding resistance is a function of the normal force and the interface friction between the sand and gravel soil and the cast in place concrete footing. ULS load combinations were used to calculate a minimum normal force, while the coefficient of friction (μ) between the soil and concrete was calculated based on the angle of friction (δ = 30°) between the soil and concrete. Values and equations were obtained from the *Canadian Foundation Engineering Manual*. The coefficient of friction was calculated as follows,

$$\mu = \tan\delta = \tan 30° \quad \longrightarrow \quad \mu = 0.58$$

and the resistance to sliding (V_{res}) was calculated using the following equation,

$$V_{res} = \emptyset \mu P_{min}$$

where P_{min} is the minimum normal force and ϕ is a geotechnical resistance factor specified in S6-06.

Based on all ULS load combinations, it was determined that the abutment had adequate sliding resistance to resist all lateral loading.

7.7 Overturning

To insure that the abutment is not susceptible to overturning, eccentricities based on ULS loading must be limited. A summary of the maximum allowable and calculated eccentricities is included in Table 19.

Table 19 - Summary of Maximum allowable eccentricities

Dimension	True Dimension	Maximum Allowable Eccentricity		Maximum Eccentricity
Width (B)	6.0 m	e_{Bmax} = B/4	e_{Bmax} = 1.5 m	1.09 m (ULS #5)
Length (L)	30.0 m	e_{Lmax} = L/4	e_{Lmax} = 7.5 m	0.65 m (ULS #2)

Since the maximum eccentricities for both the abutment width and length are less than the allowable eccentricities, the abutment meets the criteria to resist overturning.

7.8 Structural Resistance

While it is recognized that the abutment requires structural steel to satisfy strength and stability requirements, the abutment's steel detailing was outside of the scope of this project.

8.0 DRAINAGE DESIGN

A major component of this design includes the drainage design. The Clearbrook Interchange does not currently have storm mains or any form of waterway around the area, which makes the design more challenging since there is no location for the water to runoff to. Therefore, we have decided to go with a ponding method that will store the water and allow it to infiltrate into the ground. The two major components to the design are the pond design and the storm works in the area. We designed both components using the rational method and IDF curves for Abbotsford, which is shown in Appendix M.

Drainage design calculations are also included in Appendix M.

8.1 Storm Works

The storm works design consists of two components, the inlet locations as well as the storm main sizes and slopes. The outlets were not designed due to them being outside of our scope, however their locations were shown on the drainage drawings in Volume III. To design the inlets and storm mains we followed the *BC Supplement to TAC Geometric Design Guide*.

8.1.1 Inlet Locations

We first designed the inlet spacing. According to the *BC Supplement to TAC Geometric Design Guide*, they are designed for the 5-minute 1 in 5 year storm. To design the inlet locations we followed an excel spreadsheet that is shown in the design guide. The spreadsheet uses the longitudinal slope, the cross fall, the rainfall intensity and some other factors to calculate peak flows in the gutter and also what flow the storm causes is in order to develop the required spacing. Due to the slope on the bridge deck being 0.5%, the spacing of the inlets needed to be approximately 9 meters. North of the bridge, they are spaced at approximately 40 meters and south of the bridge, they are spaced at 60 meters. The spreadsheets used to find these values are shown in Appendix M. Due to the road being superelevated to the north of the bridge and along the bridge deck, the inlets were only located on one side of the road. However, the road does change superelevation, so we placed one final inlet at the location where the crossfall becomes 0.0% to catch any water flowing to that point.

8.1.2 Storm Mains

After we located the inlets, we designed the storm mains for a 1 in 25-year storm, which is the design storm that the design guide recommends. According to the design guide, the pipes need to have a minimum of 1.5 meters of cover from the crown of the pipe. To begin the design we needed to locate the manholes. The first manhole is located perpendicular to the first catch basins and each subsequent manhole is located 100 meters or less from the previous one. According to the guide, 100 meters is the maximum spacing for pipe diameters less than 250mm. After locating the manholes we developed an excel spreadsheet that can calculate the diameters of the storm mains based on slope, contributing area and storm intensity. The formulas used and the excel spreadsheet can be found in Appendix M. After this first iteration, we checked to ensure that the minimum cover of 1.5 meters is met and that the outlet is at the location of the pond bottom. In the second iteration of pipe sizing, we were able to meet both requirements for the south storm mains, but for the north mains, we had to lower the pond elevation in order to meet both requirements. The pipe sizes, lengths, and slopes are summarized in Table 20.

Table 20 - Summary of Storm Mains

Pipe	Size	Length	Slope
P1	250 mm	92 m	4.7%
P2	450 mm	65 m	0.28%
P3	525 mm	54 m	0.28%
P4	200 mm	45 m	1.6%
P5	200 mm	81 m	6.2%
P6	450 mm	50 m	0.2%
P7	525 mm	45 m	0.33%

The locations of the pipes listed above can be found on the drainage drawings included in Volume III.

8.2 Pond Design

Once the storm works were completed, the next phase was to design the ponds. This design consisted of finding the pond type to use, the size of the ponds, the depth and designing a culvert to connect the ponds. The types of ponds available are retention ponds, detention ponds, and infiltration ponds. However, we reduced these down to infiltration only since the other two designs assume the water will eventually outfall into another waterway, which is not applicable to this design. We designed the infiltration ponds according to *A Design Manual for Sizing Infiltration Ponds*, which Joel W. Massman developed for the Washington State Transportation Commission. Additionally, we also used information about infiltration pond design from a document by Michigan called *Infiltration Basin.* We designed the ponds for two different scenarios, a 25-year storm that we used for the storm main design and a worst-case 100-year 24-hour storm event. The reason for the worst-case event is to ensure that flooding will not occur since the storm water cannot flow out of the area and it is adjacent to a major highway.

The basic principle behind the pond design is to have two major ponds that can handle the 25-year storm, which will then overflow to other smaller ponds through a culvert to handle the 100-year storm. A possible layout is shown in Figure 29. The ponds are located between the on-ramps, off-ramps and Highway No. 1, which add constraints to the location of the top of the ponds. According to *Infiltration Basin*, the ponds should have a grass buffer strip of at least 25 feet. We decided to use a 9-meter buffer strip from all the pavement edges to locate the top of the ponds. Additionally, the document also suggests a maximum of 3H:1V for the side slopes of the ponds to allow the pond to be mowed and maintained. We used this maximum slope in order to maximize the pond bottom area, which will increase the maximum amount of water the ponds can infiltrate.

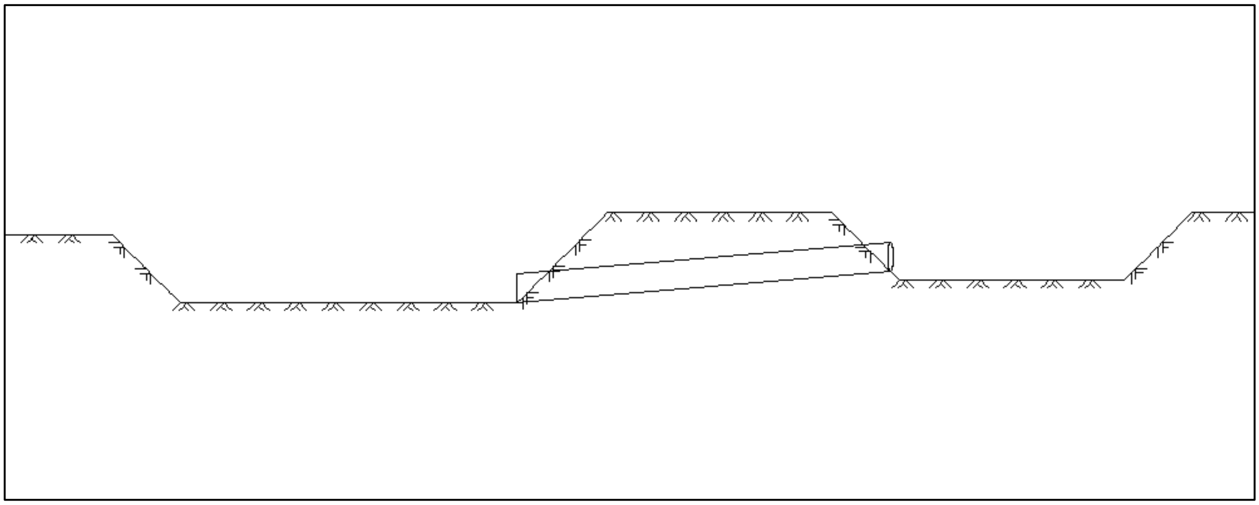


Figure 29 - Pond Cross-section (Alloy Consulting Ltd., 2015)

The next step in the design is to find the amount of water the ponds can infiltrate. To do this we used the geotechnical report to find the infiltration rates and types of layers beneath the pond bottom. In the geotechnical report, it was found that there is a 1.5-meter layer of organics on top of a 400mm layer of silty sand, which is all above a 5.6-meter layer of sand and gravel. At the bottom of this sand and gravel layer is the water table, which is the maximum depth you can go for analyzing the infiltration rate of the soils according to A Design Manual for Sizing Infiltration Ponds. Using the layer depths and each layers' associated hydraulic conductivity we were able to find an equivalent conductivity for all the layers combined. The equation used to calculate the equivalent conductivity is shown below.

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}}$$

Where *d* is the sum of all the layer depths, d_i is the depth of an individual layer and K_i is the hydraulic conductivity of the layer. After we calculated the hydraulic conductivity of the soil, we used it in conjunction with the hydraulic gradient to find an infiltration rate for the soil. We then applied additional factors to the infiltration rate to add in a factor of safety. The factors correct for the aspect ratios of the ponds, the amount of silt that will accumulate and for the size of the pond. After applying the correction factors to the infiltration rate, we applied this rate to the entire pond area to find the total infiltration flow for the pond. The formulas that we used and the entire procedure that we followed to find the infiltration flow are shown in Appendix M.

After finding the area and the flows for each pond, we then tested the flows against the 100-year storm. The main criteria that needs to be met for an infiltration pond is that it needs to drain completely within 24 hours from the end of the storm, which means that the ponds have a total of 48 hours to drain completely for the 24 hour 100-year storm. To find out if this is met we calculated the total volume that will fall during the storm and then compared that amount to the volume of water that will be infiltrated in 48 hours. After this comparison, we found that there would still be 7400 m^3 left in the ponds of the 8400 m^3 that accumulated over the 24 hours. Therefore, we needed to make the soil more permeable to allow more water to infiltrate. We found that by removing the 1.5-meter layer of organics from the surface and replacing it with a 1.5-meter layer of clean sand we were able to increase the conductivity enough to have both sets of ponds drain within 48 hours from the start of the storm. Finally, using these adjusted soil conditions we found the depth of water in the ponds after 24 hours, since this will be when the ponds reach a maximum depth. We found that ponds 1 and 2, the north ponds, would reach a depth of 285mm and ponds 3 and 4, the south ponds, would reach a depth of 138mm. We also considered the 25-year storm, since the design we are using only allows the major ponds to infiltrate the 25-year storm that we used to design the storm mains. We found that after the 8-minute storm, pond 2 will have 45mm of water and pond 3 will have 35mm of water.

With the water depths now known, we began designing the culverts since we can use the water depths to determine invert elevations and the pipe size needed. We followed the *BC Supplement to TAC Geometric Design Guide* to design the culverts. From the guide we found that culverts located under highways need to have a minimum diameter of 600mm and require at least 450mm of cover. We used the minimum diameter culvert to connect the ponds since its diameter is larger than the maximum pond depth and will not be full during the 100-year storm. The culvert will also have a slope of 1.0% to ensure it will drain after the ponds infiltrate all the water. Finally, since the ponds should not have water flowing between them during the 25-year storm, we decided to make the inlet invert 50mm above the major pond bottom elevation. This will ensure that the major pond will contain the entire 25-year storm and will be able to overflow to the secondary pond during more extreme storm events. Since the culvert invert will need to be 1050mm below the road due to the cover requirements, we decided to locate the culverts at the point where the road is highest. The reason for this is that we do not want to have the ponds be too deep, as it will require large amounts of excavation.

The depth of the ponds could now be determined using two criteria: the pond has to be deep enough to allow the culvert invert to be 1050mm below the road and the pond needs to be able to fill to the maximum depth and still be below the lowest elevation around the pond. Since the ponds fill to a maximum of 300mm, we decided to add an additional 100mm to allow some freeboard to the top of the pond. Therefore, we compared the elevation values of the culvert inlets to 400mm below the lowest elevation around the pond. We then chose the lesser value as the pond bottom elevation. This procedure applied to the major ponds, 2 and 3, and the other ponds had their elevations calculated by using the major pond elevations and the difference between the outlet invert elevation of the culvert and the elevation of the major pond. Then we compared these calculated elevations to the lowest point elevations and confirmed that the ponds are 400mm below the lowest points.

The final consideration that had to be made is the outlet invert elevations for the storm mains that lead into the ponds. As discussed earlier, we preferred that the pond elevations remain at the same location since that will prevent excavation of more material than necessary. However, due to minimum cover constraints on the storm main it was not feasible to get the invert to match the north pond elevations. Therefore, we had to lower the north ponds by 0.14 meters in order to accommodate the storm main. The final elevations of the north and south ponds as well as their maximum water depth are summarized in Table 21.

Table 21 - Pond Elevations and Depths

Pond	Elevation	Water Depth
1	62.49 m	530 mm
2	62.86 m	160 mm
3	61.90 m	70 mm
4	61.61 m	440 mm

The locations of the ponds listed above are shown in the drainage drawings included in Volume III.

9.0 CONSTRUCTION SEQUENCING

In order to ensure this project is completed on time, we came up with a construction sequencing plan. This plan is meant to be used as a guide for contractors but is by no means the only way to complete this project. The construction plan is broken down into the following nine major phases:

- Prepare the TCH relocation
- Build the bridge pier
- Complete the TCH relocation
- Construct the abutments and approaches
- Construct the bridge deck
- Construct the on and off ramps
- Construct the ponds
- Build the roundabouts
- Install the storm mains

Each of these major phases have multiple components associated with them and each component can be broken down into separate minor tasks. However, for the construction plan we did not consider each small task, instead compiled them together, and only considered the component they are a part of for analyzing each phase. For example, the abutment construction requires the installation of rebar, the pouring of concrete, setting formwork and removing the formwork after the concrete sets. However, we compiled all these smaller components into one larger component and set a number of days to complete that larger component.

In order to calculate the number of days required to build each phase of the project we used a materials based approach, and associated a productivity rate to each quantity. The calculations for these rates and the calculations for the number of days in each phase is outlined in detail in Appendix N. The goal for this construction sequencing plan is to have the project completed within 2 years from the start of the project. A complete preliminary design schedule has been prepared to account for the above-mentioned construction phases and has been included in Appendix N.

9.1 Prepare Trans-Canada Highway Relocation

In order to complete this project, we need to relocate 1.5 km of the TCH westbound lanes south of their current location in order to accommodate the new bridge that is to be constructed. This first phase will only get the gravel structure in place so that the contractor can construct the bridge pier without interfering with traffic and potentially damaging the new asphalt. This phase is composed of two components: removing organics from the new alignment location and installing new road structure material.

With the two components identified, we broke them down into the amounts of material associated with each component. The excavation rates we used were according to Methvin, we assumed a 0.4 m^3 bucket, which means it will excavate 22.2 m^3/hr (). Assuming the contractor uses two excavators working 8 hours per day, we found that they could excavate 355 cubic meters of material each day. For hauling in material, we decided to use trucks with trailers, which have a 30 t capacity according to *Cadman Trucking*. We decided on using 10 trucks every 2 hours, which equates to 1200 t of material per day. Using these production rates, we found that clearing organics will take 38 days to complete and the gravel structure will take 21 days. In total, there are 59 hours of work to be done, but by starting the gravel structure 18 days after excavation begins, the entire phase can be completed in 39 days. This will allow the excavation to be completed 1 day earlier than the structural fill.

9.2 Build Bridge Pier

Once the gravel structure is in place for TCH, the bridge pier is the next phase to be constructed. This phase requires driving piles for the footing, the pouring concrete to form the, placing columns on the footing and a installing a girder on the columns to support the eventual bridge girders. We suggest this phase be completed before finalizing the TCH relocation so that the crews can take advantage of an open gravel road where they can place pile-driving equipment to drive the piles. Additionally, it gives them space to lift and place the new columns and girder.

We looked at a couple sources to find the time required to drive piles and they ranged from 3-8 piles per day. We also found another source called *Construction planning of single-span bridges using precast, prestressed concrete beams*, which is a study done on multiple single span bridges that laid out average construction times, which is shown in Figure 30**.** The study suggests that pile driving takes an average of 3 days to complete. We went with the average of 3 days since there should not be more than 12 piles, and assuming they can drive 4 piles in a day, 3 days would allow them to complete the job. Once the piles are in place, they can begin the footing construction. The footing is a cast in place footing. Therefore, to place all the formwork, install the rebar, and pour the concrete we decided that it would take 5 days to complete this footing. Finally, for the columns and beams we decided to go with precast construction. It should be noted that this pier was not designed, however if it were to be designed we would have designed it for precast construction. We found two sources about precast construction; one was a video about a precast construction project in Georgia where they could construct four columns in a day. The other source is a case study about the San Juan bridges that were completely constructed using precast and only took 72 hours to complete both 720-foot bridges. Therefore, we decided that to construct the columns and place the girder should take only three days, two days for the columns and one day for the girder. Since each piece needs to be done in sequence, this gives phase 2 a total duration of 11 days.

TABLE 1. Project activities: their duration and cost

Activity	Description	Preceding activity	D_N (days)	C_N (10^3 \$)	C_D (days)	C_C (10^3 \$)
A	Clear site, move in	None	5	14.4	4	15.3
B	Excavate footing, side I	A	2	0.2	2	0.2
C	Drive pile, side I	B	3	5.7	3	5.7
D	Excavate footing II	A	2	0.2	2	0.2
E	Drive pile II	C, D	3	5.7	3	5.7
F	Footing I	C	5	3.3	4	3.6
G	Footing II	E	5	3.3	4	3.6
H	Abutment I to beam E1	F	9	9.4	7	13.3
I	Abutment II to beam E1	G	9	9.4	7	13.3
J	Roadwork part I	A	5	10.4	4	11.3
K	Prestressed beams	H, I, J	1	28.4	1	—
L	Form and pour deck	K	14	18.7	10	20.5
M	Remaining part of abutment I	K	9	6.1	7	7.0
N	Remaining part of abutment II	K	9	6.1	7	7.0
O	Deck curing	L	10	—	10	—
P	Barrier walls over deck	L	6	3.3	4	4.5
Q	Barrier walls over abutment I	M	5	2.6	4	3.2
R	Barrier walls over abutment II	N	5	2.6	4	3.2
S	Finish roadwork I	M	6	8.2	5	9.1
T	Finish roadwork II	N	6	8.2	5	9.1
U	Remove deck form	O	5	4.2	3	4.8
V	Steel railing, mulch seeding, riprap	P, Q, R, S, T	8	14.5	5	15.8
W	Clear up	U, V	4	0.6	2	1.2
	Total			165.5		

NOTES: D_N = normal duration; C_N = normal cost; C_D = crash duration; C_C = crash cost.

Figure 30 - Average Bridge Construction Durations (Sami, 1982)

9.3 Complete Trans-Canada Highway Relocation

After the pier is constructed, they can fully relocate TCH. In order to complete the relocation, the new westbound off ramp will also need to be constructed. This is due to the next phase having the abutments installed, which will cut off the old highway one, including the existing off ramp. Therefore, for this phase to be complete, the new off ramp needs to have the organics excavated out and replaced with gravel structure. Additionally, the base and surface asphalt needs to be paved on both TCH and the off ramp. The excavated trench area also needs to be filled with sand.

Using the production rates we had calculated it was found that the trench fill will take 3 days to complete, excavating the off ramp will take 4 days and placing the gravel structure for the off ramp will take 3 days. Once these structure is in place paving can begin. The paving for the off ramp is more difficult to complete because the new off ramp goes through the old TCH. Therefore, we recommend that the new TCH have the base asphalt paved as well as the new off ramp, but only up to the old TCH. After they pave the base asphalt, they can pave the new TCH surface, which they will complete prior to the new off ramp surface. This will allow traffic to be shifted off the old highway and onto the new highway while leaving the old off ramp in operation. Finally, they can pave the new off ramp surface through the old TCH. The old highway should be close to the new off ramp elevation, but if it is different by only 50 mm or less, they will need to mill the old highway down to 50mm below the new surface asphalt final elevation to ensure they can pave the new off ramp.

Using out production rates, we found that the base asphalt will take 5 days to complete and the new surface asphalt will only take one day, but will require a 10 hour pave. The first 8 hours will be done with 14 trucks every 2 hours and the last 2 hour round will only have six trucks to pave the off ramp.

This phase will take a total of 17 days to complete. To achieve this, the gravel structure will begin 2 days after the excavation of the off ramp. Once the gravel structure is complete then the base asphalt will be paved along TCH and along the new off ramp on either side of the old TCH. After the base asphalt is complete, they can pave the surface asphalt along TCH. Finally, they can pave the surface asphalt along the off ramp on both sides of the old TCH.

9.4 Construct Abutments and Approaches

The fourth phase in the interchange construction plan is building the abutments and the bridge approaches. According to Figure 30, Abutments take 23 days to construct. However, the abutments they analyzed were only around 10 m wide and our abutment is 30 m wide. In addition, the study was done in 1982 so construction techniques are much better now. Thus, we decided to use a duration of 35 days, which is approximately 1.5 times the duration in the study. After they complete the abutments, they can fill the approaches with fill material. A large amount of fill needs to be placed for the approaches. This is due to the bridge being 8 m above the existing ground and 30 m wide. Therefore, by using the hauling rates we have the bridge approaches should take 50 days to fill. Overall, this phase in the project should take 85 days to complete.

9.5 Construct Bridge Deck

Following the abutment construction is the bridge deck construction. This phase has four components to it: placing the girders, forming and pouring the deck, paving the bridge surface and approaches, and adding miscellaneous items like railings, sidewalks, curbs, etc. Since the girders will create a hazard for any drivers on the highway, they will need to shut down the highway to install the girders. Therefore, they will need to place the girders in 1 day to reduce the traffic impact. After the girders have been placed the formwork can be set up for the concrete deck and then the deck can be poured. According to Figure 30 the deck should take 24 days to form and cure, but we have decided that this is not accurate for today’s construction methods so we reduced it to 10 days. Once the deck cures and is usable, the can install the miscellaneous items including the railings, curb, and sidewalk. We have decided that this component should only take 10 days to complete. Finally, the bridge deck and both approaches can be paved. We found that with the production rates we are using it would take 3 days to pave the whole piece. Each of these components need to be done in order; therefore, this phase will take 24 days to complete.

9.6 Construct On and Off Ramps

Phase six in this project is constructing the westbound off ramp and the eastbound on and off ramps. During the construction of the ramps, they will also install new culverts to connect the ponds. The first component to this phase is to excavate organics, which will take 15 days to complete for all three ramps. After the excavation is complete, they can begin to add the gravel structure. The structure should take 7 days to complete, approximately 2 days per ramp. While the gravel is being placed, the part of the culvert under the new ramp will be installed. After both of those tasks are complete, they will pave the new on and off ramps, which will take 3 days or 1 day per ramp. Finally, after they shift traffic to the new ramp they can install the remaining culvert length up to its outlet location. Since the culverts are not very long and were partly constructed when the gravel was being placed, we only added 1 day to this phase to finish the culverts. Overall, this phase should take 26 days to complete.

9.7 Construct Ponds

The pond construction only has two components to it, excavating the ponds and then filling in the area that requires sand to improve the ponds hydraulic conductivity. The major time for this phase comes from the excavation of the ponds. They need to excavate the ponds 1 m down from the existing ground with an additional 0.5 meter removed from half of the pond areas. This requires the removal of 20,700 m^3 of material that will take 59 days to complete. Once this is complete, it will take an additional 7 days to fill in the areas excavated for sand. In total, this phase will take 66 days to complete.

9.8 Build Roundabouts

The roundabouts are the last major road phase. The reason they are last is that we need to utilize the new bridge and ramps to shift traffic and allow the roundabouts to be constructed without any major impacts. The roundabout construction has six components to it:

1. Paving to the west roundabout limits
2. Paving the northeast and southeast corners
3. Finishing the east side
4. Paving the northwest and southwest corners
5. Finishing the west side
6. Adding additional items (sidewalk, islands, etc.)

The limits for the components listed above are shown in Appendix N. The first component is done to allow the old intersection to be shifted to the west side. This will open up the northeast and southeast corners and allow them to pave to the design elevations, which is the second component to this phase of the project. After both corners are paved they will shift the intersection in order to pave the remaining east side piece that has not been paved due to it being used by traffic to access the on and off ramps. Finally, they will shift the intersection to the east side allowing them to pave the west using the same traffic-shifting scheme. Once they pave the entire intersection, they will be able to pour the curbs and island infills as well as paint all the traffic lines for the intersection. Each of these components has both excavation and hauling incorporated into their times.

9.9 Install Storm Mains

The final phase in this project is installing all of the storm mains manholes and inlets. This phase occurs throughout the project during the construction of the bridge approaches and during the intersection construction phase. For the duration of this phase, we used an installation rate of 80 meters of pipe per day, pipe sections are typically 4 m long and installing 20 lengths of pipe per day is only five lengths of pipe every 2 hours. This is not a very high number and could be higher, but this production rate includes inlets that need to be constructed and manholes. Thus, we found that it would be an accurate number but may be slightly conservative. We calculated that with this production rate it would take 10 days to complete all the storm mains including catch basin leads, inlets and manholes.

9.10 Phase Sequencing

The phases listed above do not all occur in order. Although this is possible to do, we decided to look at each phase and choose when it should start so that the project can be done as quickly as possible. Additionally, by condensing the project, it allows a greater amount of time at the end just in case some phases take longer than expected. The order we chose for this project is as follows:

- Phase 2 and 3 begin after phase 1 ends
- Phase 4 begins after phase 3 is completed
- Phase 5 begins after phase 4 is completed
- Phase 6 and 7 both begin once the deck is poured in phase 5
- Phase 8 begins after both 6 and 7 are completed
- Phase 9 begins once the approaches are being filled in phase 4

Additionally to the above sequencing of major phases, each phase's components are done in a sequential order unless mentioned otherwise in the phase descriptions. We developed a Gantt chart, which is in Appendix N to show the full project duration with each phase identified.

9.11 Removal of Existing Overpass

Following the completion of the new Clearbrook Interchange, the existing overpass (Figure 31) will be removed from the site.

Figure 31 - Existing Clearbrook Overpass (Google Inc., 2015)

The first step in the process of removing the existing Clearbrook overpass will be to remove all nonstructural components from the bridge. These components include guardrails, sidewalks, the asphalt wear surface, lighting, and other nonstructural components. The demolition team will be able to remove these components without affecting traffic on Highway 1 below. The removed components will be recycled wherever possible. The removed asphalt can be ground and used as part of new asphalt mixes or used as aggregates. Steel guardrails will also be recycled after they are removed from the structure.

With the nonstructural components removed from the bridge structure, demolition can begin on the bridge super structure. In this phase of demolition, special care must be taken to prevent the work from affecting traffic on the freeway. To do this, demolition work will begin on portions of the overpass that are no longer above the relocated freeway. At these locations, crews can remove the concrete bridge deck and concrete girder without impeding traffic on the freeway. Removal of the large members should be done from the center span moving outwards so that equipment for removing the components can be placed on the remaining bridge spans.

Removal of the south spans of the existing Clearbrook overpass will require the diversion of traffic on Highway 1 to be completed safely. Before diverting traffic, preliminary work will need to be completed so that the structural components can be removed quickly. This work will include preliminary removal of concrete are the supports. The concrete girders will be removed from the affected spans at night while the freeway is closed. Cranes and trucks will remove the girders while parked on the closed freeway.

The final portion of the existing overpass to be removed are the concrete support structures. The concrete support structures can be removed while the freeway remains open. Demolition crews will cut the supports into smaller pieces that can be taken to processing facilities off-site.

All reinforced concrete removed from the site can be recycled into aggregates or engineered fill material. Reinforcement from the bridge can be separated from the concrete and recycled as well. Local facilities are able to provide both of the above-mentioned services. (Richvan Holdings Ltd., 2013)

10.0 TRAFFIC SIGN LAYOUT

An integral part to road design is laying out signs to warn motorists of upcoming hazards or guide them in situations that may be confusing. For this project, we laid out all the signs that we deemed are necessary to keep drivers, cyclists and pedestrians safe while using any component of the interchange. We used the *Ministry of Transportation and Infrastructure*'s specifications for signs to find which signs are required for our interchange. We went through each category and determined that we require warning signs, regulatory signs, roundabout signs, bicycle signs, and guide signs. A sample of the signs we used in this project are shown in Figure 32**.**

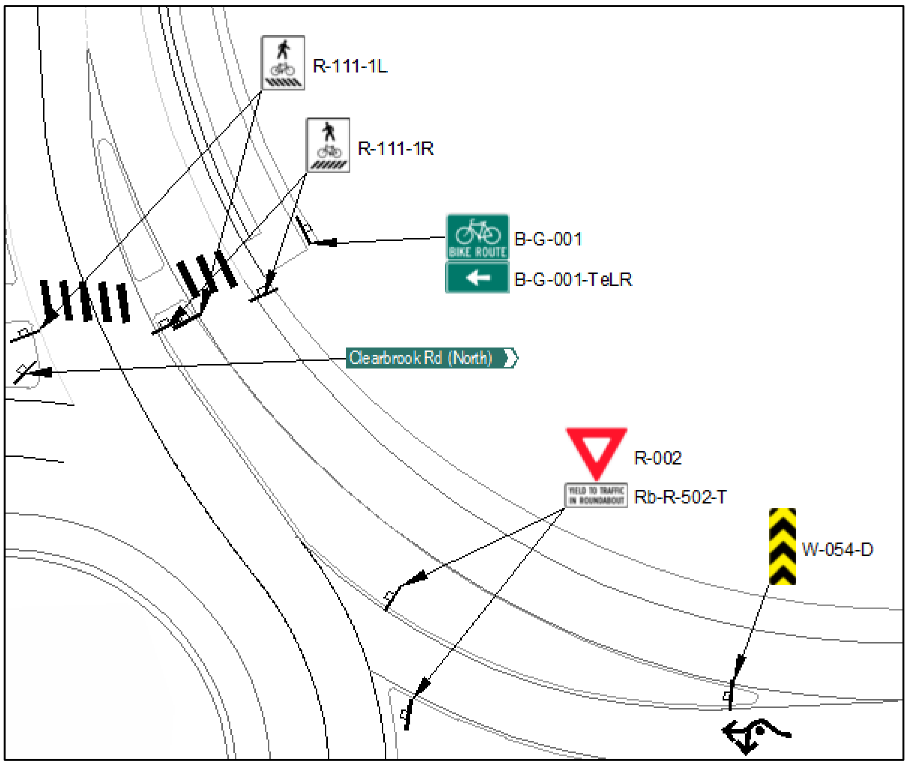


Figure 32 - Signs for NE Corner of South Roundabout (Alloy Consulting Ltd., 2015)

As shown in the figure, we used warning signs where obstructions endanger motorists. We also have bike route signs to guide cyclists in the direction they need to go. We also have yield signs to prevent car collisions in the roundabout and pedestrian crossing signs to prevent pedestrian accidents. Volume III contains all of the signage drawings, which show every sign that has been used and where they are located throughout the project.

11.0 CONCLUSION

After a thorough evaluation of three different transportation alternatives (Section 3.0), Option B was identified as the alternative that best addressed the project purpose and need. This alternative was refined into the preferred option described in Section 5 of this report. The preferred option includes these main geometric elements:

- Relocation of the EB Trans-Canada Highway
- Longer deceleration lanes prior to entering the TCH off-ramps
- New 6-lane overpass
- Two double lane roundabouts on each side
- Improved horizontal alignment and vertical profiles throughout the interchange

Along with selecting a new interchange system, a geotechnical report was produced and settlement and abutment designs were carried out to ensure that the new overpass was stable. A series of preliminary overpass structural options were considered (Section 6.0), with prestressed box girders and steel plate girders both scoring very close ratings. Due to these close ratings, Alloy Consulting recommends that the prestressed box girders be further investigated to determine if they might be a better option. To ensure the project is feasible, a project-phasing plan was prepared. Finally, a drainage and infiltration pond design was completed ensuring that the project is sustainable with limited impacts to the surrounding areas.

12.0 EPILOGUE

Dear future CIVL 7090 victims,

Having finally made it through the CIVL 7090 Capstone Design Project, there are a few pieces of advice that we wish to offer you. We hope that this information makes it to you before it is too late.

1. Choose a project that interests you. We were lucky enough to find a project in which each person could work on a component that interested them.
2. Choose a project that allows you to expand your knowledge but, at the same time, isn't too far beyond your capabilities. In our project we got to work with a lot of new concepts but had to stop certain portions of the work because we were simply unable to complete the work we initially set out to accomplish. This was the case with incorporating prestressed concrete into the structural design.
3. Choose a good team - one in which you don't have a team member leaving the continent for the second semester, completely abandoning the rest of the team. Our team was great at working together and we did not have any major communication issues throughout the execution of the project. Throughout the course of the project, we held weekly meeting to update each other on our progress and schedule for the next week. This practice is strongly recommended.
4. Civil 3D can be extremely difficult to work with. YOU HAVE BEEN WARNED! While simple alignments may be manageable, we reached a point in our project where he had to abandon completing our 3D model and begin drawing production. A significant portion of our time was eaten up by trying to make the Civil 3D alignments work.
5. Speaking of which, be aware that drawings and modeling take a lot of time. Because of our troubles with Civil 3D, laying out the drawings was started quite late and it took a lot of time to do (which was not anticipated when we prepared the proposal). Decide early if you want to include MoT standard drawings in your project, not 3 weeks before it is due.
6. Write your report as you complete tasks and take advantage of the draft report submission. We did most of our report writing as tasks were completed and it saved us a lot of time at the end of the project. And even having done this, the report writing took significantly more time than we initially anticipated. So do not hesitate to be generous with your time estimates for report writing.

Had we be required to repeat the project, however, there is one major component that we would have changed:

- We would have allotted much more time to the actual preparation of drawings. Since we were not fully aware of the detail required to produce MoTI standard drawings, we did not anticipate the 100+ man hours that it took.

We feel that, had more guidance been provided in the allotment of our time into specific tasks, we would have been able to more accurately predict the amount of time that certain tasks would take. This could be done in future years by way of panel meetings prior to the proposal submission and by providing access to the final client reports prior to the proposal submission.

With the conclusion of our project, we completed all the tasks that we had initially set out to accomplish, as outlined in our project proposal. To do this, however, we took much more time than anticipated. The extra time came exclusively within the final two week before the project submission. Our final 3-line graph is shown in the following figure.

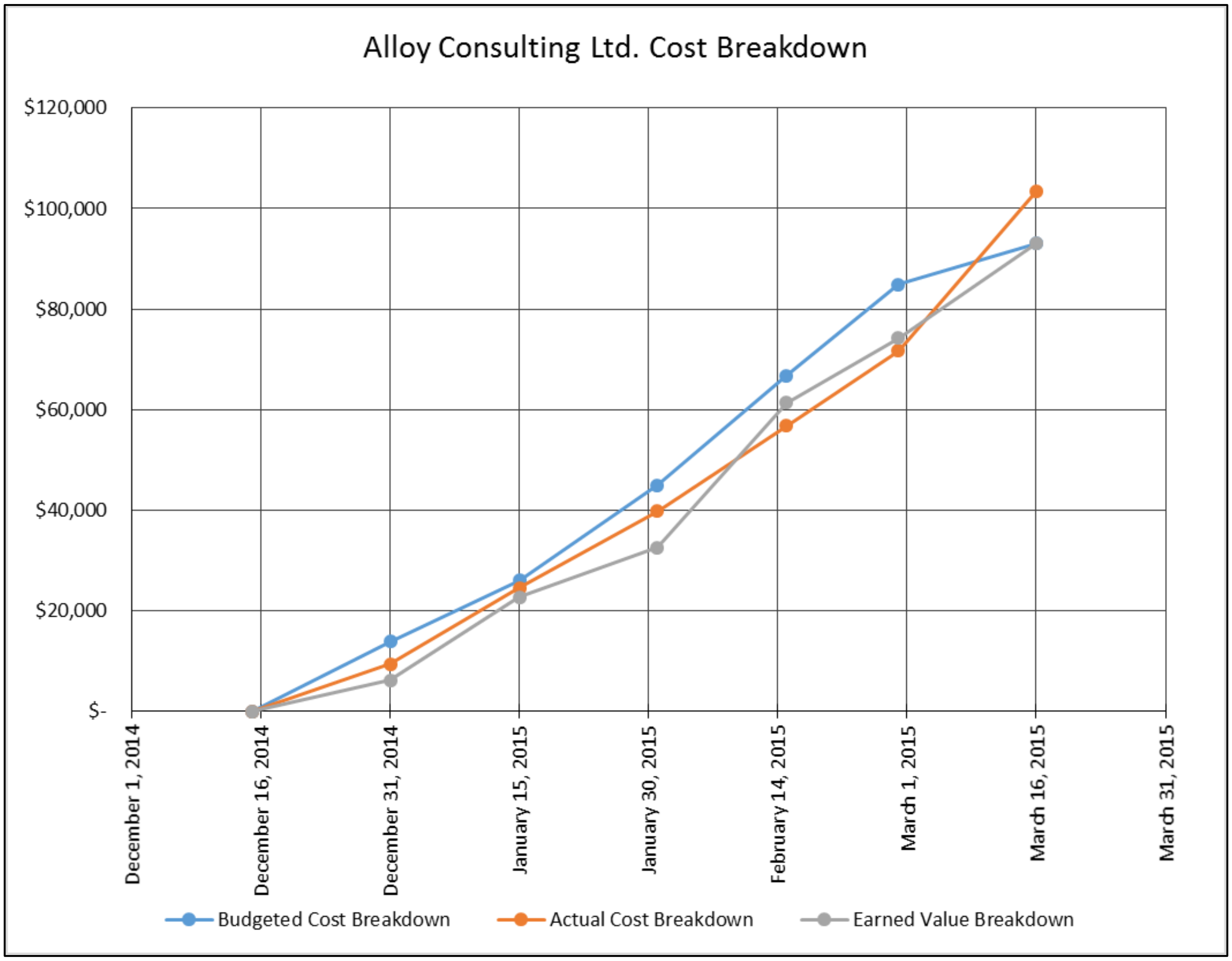


Figure 33 - Alloy Consulting's 3-line Graph (Alloy Consulting Ltd., 2015)

Overall, we had a positive project experience. We are, however, very pleased to have it handed in.

13.0 CLOSURE

Alloy Consulting would like to thank you for taking the opportunity to read our design report for the Clearbrook Interchange Improvement Project in Abbotsford, BC. We are happy submit this report to the Projects Committee and look forward to answering any questions or concerns that may arise.

Sincerely,

Alloy Consulting Ltd.

Logan Brown	Jacob DeVos	Keith Dodgshon	Jeff Ferraby	Matt Hackett

REFERENCES

Alloy Consulting Ltd. (2015). British Columbia, Canada.

Au-Yeung, Y. H. (1995). *Gravity Retaining Walls Subject to Seismic Loading.* Hong Kong: Geotechnical Engineering Office - Hong Kong Government.

Bowles, J. E. (1997). *Foundation Analysis and Design* (5th ed.). Singapore: McGraw-Hill Book Co.

Bulk Excavation. (2015). Retrieved from Methvin Web site: http://www.methvin.org/construction-production-rates/excavation/bulk-excavation

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

Canadian Standards Association. (2006). *Canadian Highway Bridge Design Code.* Standards Council of Canada.

Childs, D. (2009, 01 06). *Abutment Design to BD 30*. Retrieved from Bridge Design & Assessment: http://www.childs-ceng.demon.co.uk/tutorial/abutmentu.html

City of Abbotsford. (2011). *Development Bylaw No. 2070-2011.* Abbottsford: City of Abbottsford.

City of Abbotsford, Ministry of Transportation and Infrastructure. (2010). *Document of Requirements for the Clearbrook Interchange Improvement Project.*

Fasbinder, D. (2013, May 1). *A Diverging Diamond Interchange on Stadium and I-70 Explained*. Retrieved from Komu News: http://www.komu.com/news/a-diverging-diamond-interchange-on-stadium-and-i-70-explained/

Federal Highway Administration. (1985). *Tolerable Movement Criteria for Highway Bridges.* Springfield: US Department of Transportation.

Fowler, J. R. (2006). *Accelerated Bridge Construction.* Charlottetown.

Giffels Westpro. (2014). *Giffels Westpro Home*. Retrieved November 9, 2014, from http://www.giffelswestpro.com/En/Home.aspx

Google Inc. (2015). Retrieved from Google Maps: https://www.google.ca/maps

Hurd, M. K. (1985). Economical short span concrete bridges. The Aberdeen Group. Retrieved from http://www.concreteconstruction.net/Images/Economical%20Short%20Span%20Concrete%20Bridges_tcm45-341149.pdf

Jacobson, S. D. (2009). *An Overview of Shared Use Lane Pavement Markings for Cyclists.* Retrieved from http://conf.tac-atc.ca/english/resourcecentre/readingroom/conference/conf2009/pdf/jacobson.pdf

Massman, J. W. (2003). *A Design Manual for Sizing Infiltration Ponds.* Mercer Island.

McElhanney Consulting Services Ltd. (2008). Abby_2008_10cm. British Columbia, Canada.

Ministry of Transportation and Highways. (1998, May 1). Technical Bulletin. *Guidelines for Geotechnical Reports*. BC.

National Cooperative Hghway Research Program. (1991). *Report 343: Manuals for the Design of Bridge Foundations.* Washington: National Research Council.

National Cooperative Highway Research Program (NCHRP). (2010). *Report 672 - Roundabouts: An Informational Guide.* US Department of Transportation - Federal Highway Administration. Washington: Transportation Research Board of the National Academies.

Nevada Department of Transportation. (2015). *Diverging Diamond Interchanges*. Retrieved from Nevada Dot: https://nevadadot.com/Traveler_Info/Safety/Diverging_Diamond_Interchange.aspx

Queensland Goverment. (2015). *Merging*. Retrieved from Department of Transport and Main Roads: http://www.tmr.qld.gov.au/safety/queensland-road-rules/road-rules-refresher/merging.aspx

Richvan Holdings Ltd. (2013). *Recycling Services*. Retrieved from Richvan Holdings Web site: http://www.richvanholdings.com/our-products-services/materials-recycling-services/

Sami. (1982). *Construction planning of single-span bridges using precast, prestressed concrete beams.* Willowdale.

Trucking. (2015). Retrieved from Cadman: http://www.cadman.com/trucking.php

U.S. Bridge. (n.d.). *Audrain County Case Study.* Retrieved from U.S. Bridge Web site: https://www.usbridge.com/downloads/brochure/Audrain_County_Case_Study.ppt

US Department of Transportation. (2004). *Signlized Intersections: Informational Guide.* Federal Highway Administration. FHWA.

Washington State Department of Transportation. (2015). *Roundabout Benefits*. Retrieved from http://www.wsdot.wa.gov/Safety/roundabouts/benefits.htm