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March 16, 2015

Martin Bollo, P.Eng.
Department of Civil Engineering
British Columbia Institute of Technology
3700 Willingdon Avenue
Burnaby, BC   V5G 3H2

Dear Martin Bollo:

RE: Report Submission for the Design of Hat Lake Forestry Service Road

Clear Cut Consulting (CCC) Ltd. is pleased to submit this design report, drawings and appendices for the design of the Hat Lake Forestry Service Road located at Fort St. James, BC. This submission has been completed to meet the requirements of the CIVL 7090, Capstone Design Project Course. The following design report includes:

- hydrologic analysis of stream crossing Site 7a and Site 10,
- survey data processing of Hat Lake Road,
- road alignment and forest road crossing design,
- alternative alignment evaluation,
- superstructure design at Site 10,
- substructure analysis and design at Site 10,
- environmental and erosion analysis,
- sustainability considerations,
- construction scheduling and cost.

We trust that the following design and report meets your expectations. If any questions arise, please contact Clear Cut Consulting at clearcutconsulting@gmx.com.

Sincerely,

Angus Milne	Jay Kim	Monica Ip	Nick Arena

CC: Capstone Committee
Attached: Report

Executive Summary

Canfor Corporation is a leading supplier of forestry products in Canada. They operate sawmills across BC and manage and harvest forestry resources in compliance with the Canadian Standards Association sustainable forest management practices. Canfor contracts DWB Consulting Services Ltd. to design, maintain, and construct their central and northern BC forestry service roads. Due to DWB's excellent performance on previous work, Canfor and DWB have a good working relationship.

Canfor requires a new forestry road 55 km north of the District of Fort St. James, BC, to connect Germansen Hat Road, a main logging route, to the highway connector Leo Stream Road. Canfor issued the design and construction of Hat Lake Road to DWB; DWB subdivided the work into three phases. DWB has subcontracted Clear Cut Consulting Ltd. to design and prepare a construction plan for the first phase of the Hat Lake Road, which spans 9.5 km east to west.

Phase 1 of the Hat Lake Road project involves the new road construction. CCC has designed an optimum alignment of the new connector road, performed a hydrological analysis on the surrounding topography, located and designed the stream crossing structures including a culvert and bridge, created an environmental protection and sedimentation plan, and produced a construction plan for Hat Lake Road. This report outlines the methods and procedures that CCC has used to complete the project.

The Hat Lake Road (Phase 1) project consists of multiple stream crossings, for which CCC designed one bridge and one culvert. After examination of the survey data, CCC determined that a culvert would be installed at Site 7a and a bridge crossing at Site 10. A culvert design was selected for Site 7a due to the small stream bed width. CCC designed a bridge crossing for Site 10 since the site has a larger stream running through it.

A detailed hydrologic assessment has been performed at Site 7a and Site 10. The assessment at Site 7a determined the 100-year peak flow and was used to size the culvert diameter. The analysis at Site 10 determined the 100-year flow water level. The water level was required to

determine the design height of the bridge crossing. Empirical formulas and historical stream evidence were the primary methods used to determine the peak flow at both the sites.

In preparing a crossing plan for Site 10, CCC developed three alternative crossing arrangements. These options consisted of:

- a 30 m bridge span crossing the stream at a 50º angle with no soil retaining structures,
- a 25 m bridge span crossing the stream at a 50º angle with soil retaining structures, and
- a 18 m bridge span crossing the stream at an 80º angle with no soil retaining structures.

CCC analyzed these options and decided that the shortest span would result in an unsafe design due to the aggressive horizontal alignment required to approach the stream at a near perpendicular angle. Of the two remaining options, the longest span presented more value because the cost of the retaining structures required for the 25 m span is larger than the cost of the 30 m bridge structure.

An environmental assessment was performed by CCC to highlight sensitive areas in the vicinity of a water course for Hat Lake Road project. Minimizing the environmental impact of the Hat Lake Road's design and construction is of upmost importance to CCC. The important regulations and necessary approvals have been noted to aid the contractor on the correct path. An erosion and sediment plan was developed to prevent any fine material from entering the water course, which could have negative effects on aquatic species.

Sustainable development has been considered throughout this design to minimize the effects on the natural environment. The purpose of constructing Hat Lake Road is to allow Canfor access to log the forests in the area. Although the harvesting of timber can be detrimental to the environment in the short term, timber is an important sustainable resource. Timber is one of the only renewable resources that can be used for construction of buildings and various other structures. The process of harvesting and processing timber creates numerous employment opportunities throughout the region of Fort St. James.

CCC prepared a preliminary construction phasing plan and cost estimate to ensure that the design of the Hat Lake Road is constructible, feasible and economical. CCC's construction plan resulted in a duration of 78 working days with work beginning in early April, and substantial completion occurring in late July. The total cost of labour, equipment rental, and material is estimated at $3.66 million.

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List of Abbreviations

A	Area
AASHTO	American Association of State Hwy & Transportation Officials
C3D	Civil 3D (A product of Autodesk)
CCC	Clear Cut Consulting Ltd.
CFEM	Canadian Foundation Engineering Manual
CSA	Canadian Standards Association
D	Diameter
DFO	Department of Fisheries and Oceans
ESCP	Erosion and Sediment Control Plan
FREG	Forest Road Engineering Guidebook
GPS	Global Positioning System
IDF	Intensity-Duration-Frequency
K	Curvature
L-100	Design Vehicle Weight (GVW of 890 kN)
MFLNRO	BC Ministry or Forests, Lands and Natural Resource Operations
MoTI	BC Ministry of Transportation and Infrastructure
Q100	100-year Peak Flow
Q	Flow in m^3/s
PMF	Peak Maximum Flow
PNEZD	Point Northing Easting Z (elevation) Description
PWL	Present Water Lever
TIN	Triangulated Irregular Network
UTM	Universal Traverse Mercator
WWF	Welded Wide Flange

Acknowledgements

Clear Cut Consulting would like to thank the following parties for their valuable input and contributions to our project:

DWB Consulting Services Ltd.

We would like to thank our sponsor, DWB Consulting for their technical contributions towards our project.

Emily Cheung, MASc, PEng, FEC

James Patterson, Survey Manager

BCIT Faculty

We would like to thank the Capstone Committee for their continued input throughout the project.

Martin Bollo, M.Eng, P.Eng.

Colleen Chan, Ph.D, P.Eng.

Paul Thurston, BASc., P.Eng.

Renata Wood, BASc., P.Eng.

Cipa Lumber Co. Ltd.

Jay McGeachan, RPF

1.0 INTRODUCTION

In the Bulkley-Nechako region of BC, the existing Germansen Hat Road begins 50 km north of Fort St. James and spans 40 km to the west. Germansen Hat Road hugs the south side of Hat Lake and is thus given its name to differentiate it from Germansen Road. Figure 1 displays the location of Germansen Hat Road.

Figure 1: Hat Lake Road location (Google Maps, 2015)

Germansen Hat Road has substantial commercial value as it provides a means of access and transportation for mining companies, logging companies, private businesses, recreational sites and our sponsor company's client Canfor. The existing Germansen Hat Road does not currently connect to any major road at the west end, which limits access to potential logging sites. Clear Cut Consulting Ltd. (CCC) was retained by our sponsor company, DWB Consulting Services Ltd., to develop a design to extend the west end of the existing Germansen Hat Road for DWB's client Canfor so that it is connected to Leo Stream Road, the nearest major road.

2.0 PROJECT DESCRIPTION

DWB retained Clear Cut Consulting Ltd. to provide engineering services related to extending the existing Germansen Hat Road west to tie-in to Leo Stream Road for Canfor. When construction of the Hat Lake Road (Phase 1) is completed, it will provide easy access to Canfor's northern sawmill operations, such as Pierre Sawmill, Plateau Sawmill, Polar Sawmill, Houston Sawmill and many more. Phase 2 and 3 is out of CCC's project scope and will be completed by DWB. CCC completed the forestry road design for Hat Lake Road (shown in red in Figure 2), which is located 55 km north of Fort St. James.

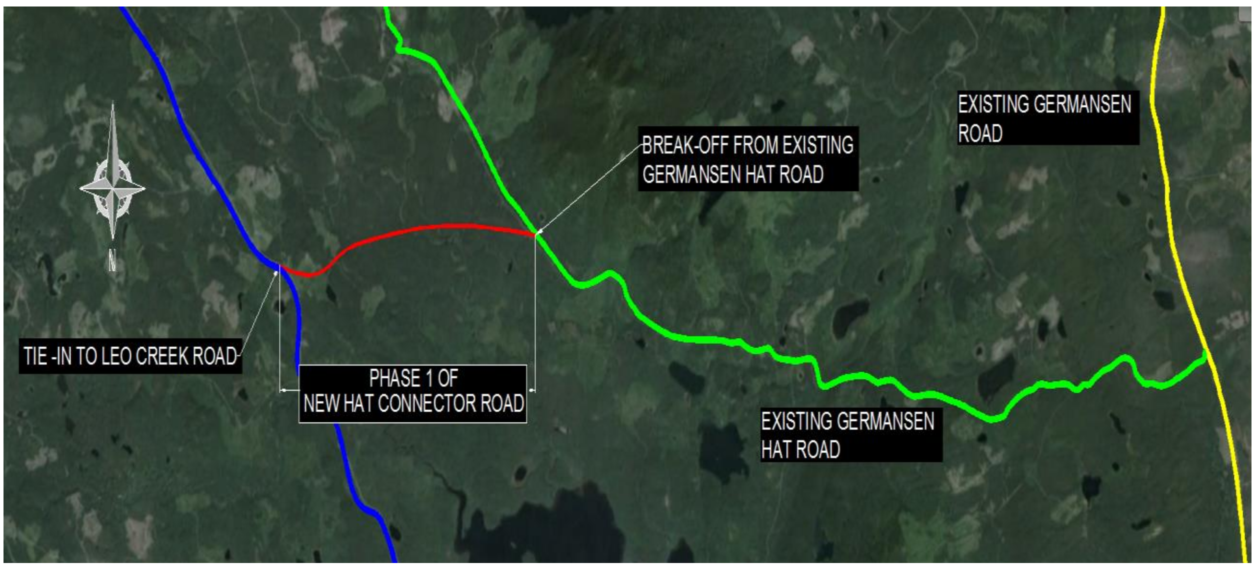

Figure 2: Arrangement of Hat Lake Road (Google Maps, 2014)

Beginning at Leo Stream Road, Hat Lake Road spans west to east for a distance of over 9.5 kilometers. CCC completed a detailed road design, bridge and culvert crossing design, environmental and geotechnical assessment of the crossings, and construction management plan.

The new Hat Lake Road connector involves four stream crossings, two of which were designed by CCC. We selected the largest crossing for the bridge location, and the second largest crossing for the location of a culvert. A plan view of the planned Hat Lake Road alignment in Appendix A shows the bridge crossing at Site 10, and the culvert crossing at Site 7a.

3.0 DATA ANALYSIS

Two types of surveys are used for forestry applications: field traverse and location survey. The purpose of the field traverse is to survey a field traverse for road layout and design to collect data and measurements for the road location. This type of survey is commonly referred to as a Level 1 or Level 2 survey and is appropriate only for roads with no geometric complication, such that the road layout is tied down and can be mapped and reproduced in the field (Ministry of Forests, Lands and Natural Resource Operations, 2013). The general procedure of field traverse surveys can be found in the MFLNRO *Engineering Manual*.

The location survey focuses on obtaining information and measurements necessary for a detailed design. Unlike the field traverse, the location survey is conducted at a high level of accuracy to capture the site specific features such as banks, stream, and slopes.

3.1 Field Traverse

For the Hat Lake Road project, the survey crews were instructed to establish a 30 meter width survey corridor. First, the surveyors pick up the centre line shot. Then, the side slope shots are taken at 15 meters from the center to the left and from the center to the right. However, due to the nature of forestry terrain, a single side shot to the left and right at 15 meters does not provide an accurate existing surface. The surveyors pick up multiple shots to capture the existing condition as precisely as possible when presented with existing surface slopes at various grades. The cross-section shown in Figure 3 is an example of a surveyor's interpretation of the existing site condition.

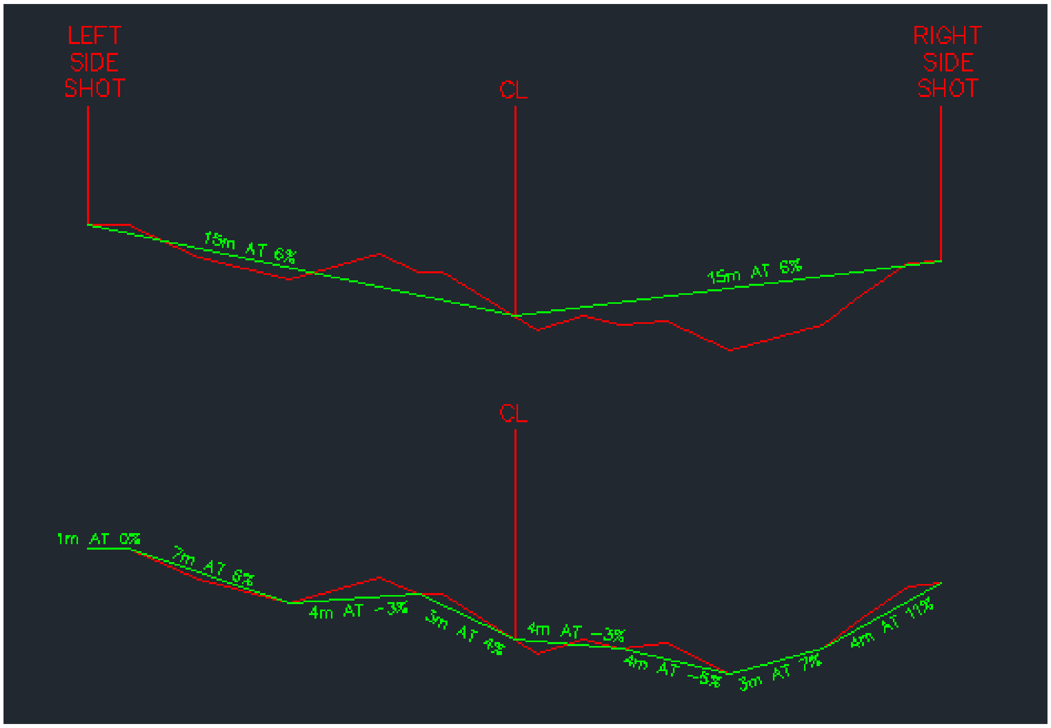

Figure 3: Surveyor's interpretation of existing surface (Kim, 2015)

The red jagged line in Figure 3 describes the true surface, whereas the green line describes the surveyor’s interpretation of the surface. The cross-section shown at the top Figure 3 is a crude interpretation of the existing ground using one shot to the left and one to the right. Note the large discrepancy between the true surface and model surface. To improve the accuracy, multiple intermediate shots were taken in between and results in a finer estimation as shown at the bottom cross-section of Figure 3.

Furthermore, field traverse surveying is relatively precise but not absolutely accurate. When establishing the initial control point of the survey segment, the surveyors tag the control point with an arbitrary universal transverse mercator (UTM) coordinate. For the purpose of field traverse, most control points occupy a Northing, N: 5000; Easting, E: 5000; and elevation, Z: 0 instead of its actual UTM coordinates. This is because the equipment the surveyors use for field traverse uses an unsophisticated global positioning system (GPS). Since the equipment provides unreliable UTM coordinates, surveyors assign easy numbers to work with, such as 5000, 5000, 0. This simplification does not undermine the overall accuracy of the field traverse since the relative position difference between two points is still accurate. In Civil 3D (C3D), the absolute position can be corrected by moving the entire survey to the correct UTM with the control point used as a reference point.

Hat Lake Road consists of ten (10) survey segments. One of these segments is surveyed with a "Level 3 RTK" survey method. Compared to the other nine Level 2 segments, the Level 3 segment is considered the most accurate method available for field traverse. The importance of the Level 3 segment will be covered in more detail in the Data Merging section of this report.

3.1.1 Location Survey

At crossing locations, DWB provided detailed information of the existing topography required to design a detailed forest road crossing. It is essential that the survey at this location be as accurate as possible to capture the existing conditions. Inaccurate surveys may result in road spills into the stream, fill volume over-run, cut volume under-run, stream flow underestimation, and bridge span discrepancies. The MFLNRO *Engineering Manual* recommends a location survey be conducted at a Level 2 survey or above.

The control station is arranged at high ground with clear line-of-sight to the survey perimeter. A minimum of three benchmarks are established at stable tree trunks. Note that the benchmarks are excluded from surface modeling since their elevations are 30 cm to a meter above ground. Upon referencing these benchmarks, the control station's Northing, Easting, and elevation are locked in.

The most critical information to be gathered during a location survey is the stream information. Typically, surveyors shoot top of bank, bottom of channel, and present water level in a stream area. Unless the stream is too deep, forbidding safe entry, the center of stream information is also picked up for the stream profile. Special features such as a fallen log, beaver dam, and gravel bar may be surveyed at the surveyor's discretion. Figure 4 displays the stream at Site 10 with a surveyor obtaining data.

Figure 4: Site 10 stream width (DWB, 2013)

Outside of the stream area, ground and spot elevations are picked up for triangulating a surface model. In the presence of a distinctive bank or slope, the top of slope and bottom of slope are surveyed as well. Unlike field traverse, the location survey picks up ground points at random intervals within 10 meters. Hence, the survey points are scattered with no distinctive pattern. The random survey points prevent any artificial influence to the surface model.

At the end of the survey day, the surveyors shoot benchmarks again to estimate their accuracy. The benchmark shot at the beginning and at the end of the survey day usually only differs by approximately one millimeter.

3.1.2 Data Processing

As mentioned before, field traverse points are relative to each other and their information is given in reference to the last point. Table 1 below presents typical field data entry for a Level 2 field traverse.

Table 1: Typical field traverse data entry (DWB, 2013)

STA	DIST	FS	BS	Slope	LSS	RSS
STA 2+763.8						
	17.3	57	237	-9.2	-7/10 15/3 5/2	8/13 -2/-
STA 2+746.5						

At STA 2+746.5, the surveyor measures the foresight and backsight angles to the next target STA 2+763.8. The next station is at 57 degrees in reference to the previous point. The distance between these two points is 17.3 m at -9.2%. Then, to the left side, the existing ground is sloped at -7% for 10 m, 15% for 10 m, and 5% for 2 m. To the right side, the existing ground slopes at 8% for 13 m and -2% for next 2 m.

CCC processed the first set of traverse manually and found it to be very time consuming. Having 10 segments with over 3200 points to process, we consulted DWB's guidance and were provided with software specifically developed to handle field traverse data. This field traverse processor produces XML, TXT, and XSC files, which were used to import survey points into C3D. Within C3D, a survey network was created to manage 10 survey segments.

Unfortunately, BCIT's computers run C3D on a cloud network, and therefore, CCC could not save the survey network. To view the entire survey data, refer to Appendix B.

To correct for the discrepancy between magnetic north and true north, all survey segments were adjusted with magnetic declination. These magnetic declination angles were calculated using a Magnetic Declination Calculator provided by Natural Resources Canada. Refer to Appendix C for the full list of magnetic declination angles used for each survey segment. After the magnetic declination adjustment, survey segments were merged with each other at the end and the beginning.

These segments' elevations are correct relative to each other but incorrect in absolute UTM at this point. To adjust the field traverse to the correct UTM location, Level 3 RTK points were used. The RTK points were provided in PNEZD format with correct absolute UTM tagged to it. This RTK segment acts as the reference point to field traverse data. Refer to Appendix D for a complete list of RTK points.

Separate surface models for Site 10 and Site 7a were prepared using a location survey. The location survey was provided in RW5 format. The RW5 were converted to FBK and imported into C3D as points. When a surface is first created using these points, there are unusual spikes, dips, and irregularities due to the nature of triangulated irregular network (TIN) method used by C3D. Simply put, the TIN method creates a 3D face by creating a triangle between three points. Unless explicitly told otherwise, these triangles cannot distinguish ridge lines and will give incorrect interpretation of banks. Thus, 3D polylines and feature lines were added along ridge lines such as top of bank (TOB) and bottom of channel (BC) to create banks. Figure 5 displays the processed and unprocessed C3D model for Site 10.

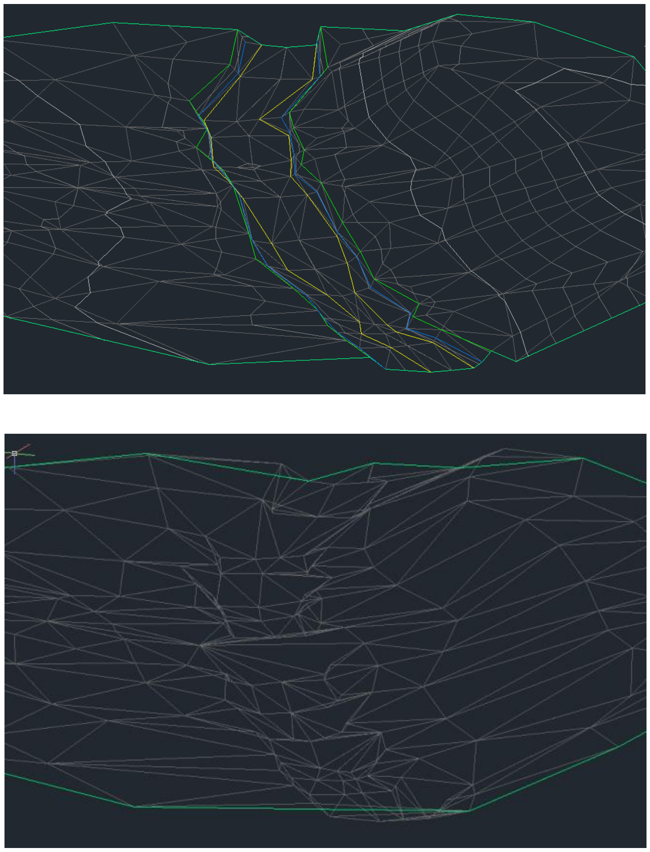

Figure 5: Site 10 Processed surface model (above) unprocessed (below) (Kim, 2015)

Figure 5 compares the surface model using ridge lines as a surface definition (above) whereas the second picture shows C3D's initial interpretation of the surface (below). The difference in surface model between these two is significant.

Site 7a is treated similarly by adding feature lines as definition to mark ridge lines. Figure 6 shows the processed surface model.

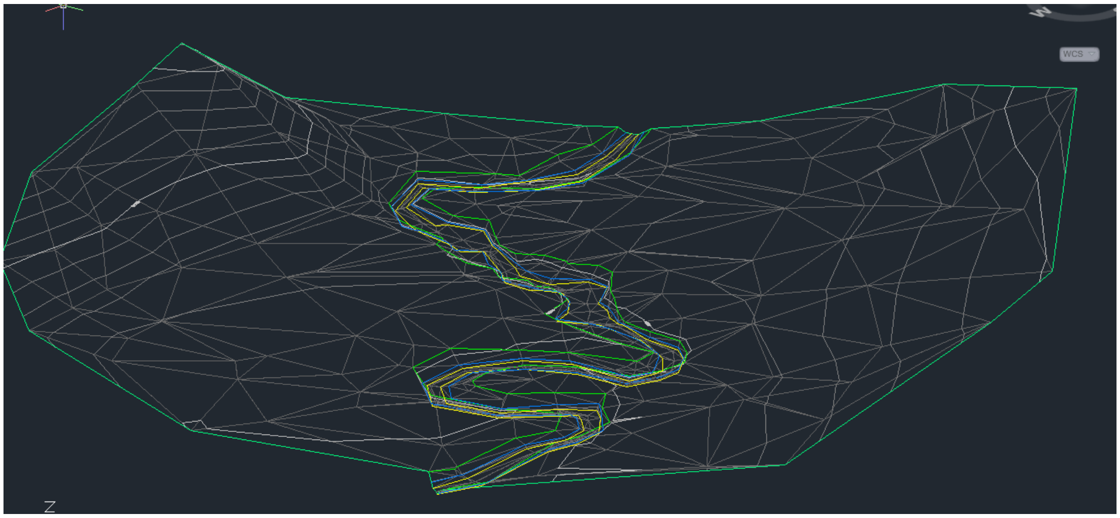

Figure 6: Site 7a processed surface model (Kim, 2015)

After processing the survey data for the two stream locations into a 3D surface model, CCC performed a hydrologic analysis of the surrounding areas to find the design flow through the modeled channels.

4.0 HYDROLOGICAL ANALYSIS

CCC performed a hydrological analysis at the stream crossings of Site 10 and Site 7a. Site 10 requires a bridge crossing because of the large flows of the stream during peak times. Site 7a requires only a culvert because the flows are significantly smaller than Site 10. The maximum water flow was used to determine the minimum clearance of the bridge span over the stream and the necessary culvert size.

Stream peak flows are governed by snowmelt in the spring to summer months. Peak discharge occurs during the month of July, according to the available data from nearby streams in the same region. Figure 7 below shows the peak discharge for Stuart Stream, which is over 50 km away. A rainstorm that occurs during this time in July would typically cause the largest flow in the streams. Warmer air temperatures and rain lead to an increased melt rate.

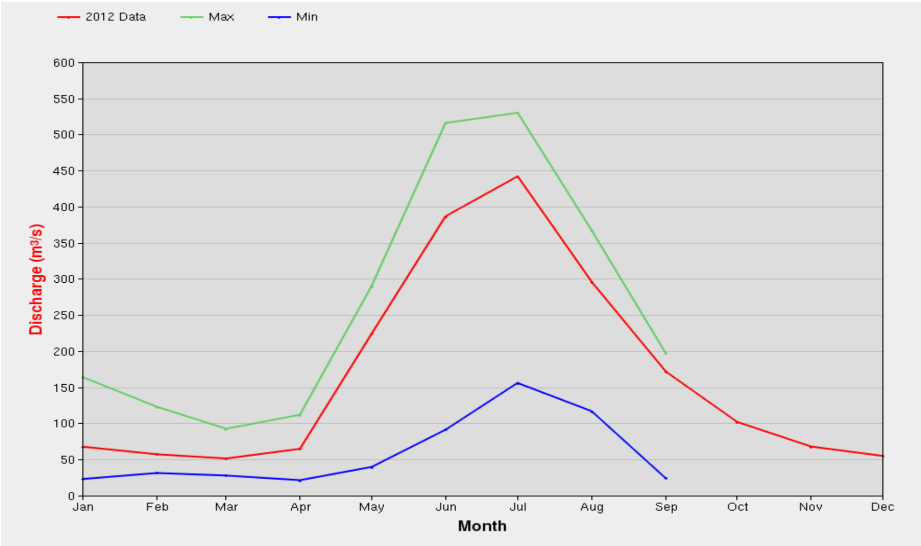

Figure 7: Stuart Stream historical flows (Government of Canada, n.d.)

Figure 7 above shows that from 1929 to 2012, the highest recorded flow for the Stuart Stream (green line) occurred in July. Although Stuart Stream is not the stream of interest for this project, it acts as a comparison of the times of peak flow in this region.

The *Forest Planning and Practices Regulations* require the design of forest service roads (FSR) in BC to safely withstand a recurrence interval of a 1 in 100 year storm event (Forest Planning and Practices Regulations, 2004). The recurrence interval represents one storm every 100 years on average. Consequently, bridge and culvert designs must manage the flow from a 100-year storm event, which is referred to as the Q100. This section describes the process obtaining the peak flow for Site 7a and Site 10.

4.1 Catchment Area

The catchment area is the region that contributes runoff to the location of interest. To determine the size of the catchment area, CCC used topographic maps. The contours are used to find the direction of flow, which determines the size of the area that contributes to the flow in the streams.

CCC analyzed several different topographic map resources to determine the most extensive contours for the Fort St. James region. Natural Resources Canada provides an online mapping program called the *Toporama* for public use; which we used to examine the terrain. A shortcoming of *Toporama* is its inability to draw lines and measure areas within the program. To remedy this issue, we took several screenshots of the maps and combined them onto an AutoCAD drawing. The AutoCAD drawing was scaled according to the *Toporama* scale. In addition, Google Earth was used to check that the AutoCAD scale produced accurate results.

Next, AutoCAD was used to trace the drainage boundaries for Site 7a and Site 10. Figure 8 shows the drainage boundary for Site 7a.

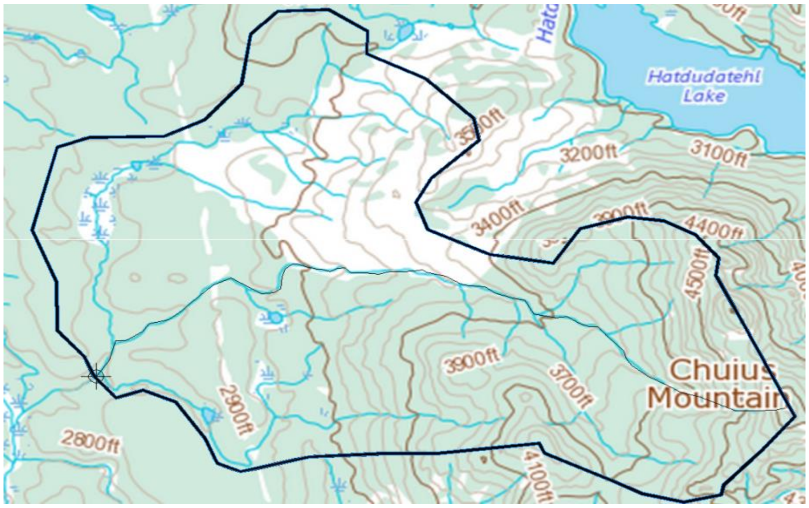

Figure 8: Catchment area for Site 7a (Natural Resources of Canada, n.d.)

The drainage area of Site 7a is 20 km2. Site 10 has a larger drainage area of 140 km2. Appendix E shows the drainage areas of Site 7a and Site 10.

4.2 Peak Flow Analysis

Significant research time was allocated for finding reasonable methods to determine the peak flow in the streams. The stream crossings of interest are located in Fort St. James, which is in central BC. Detailed climatic data and stream flow data for this location is very sparse and presents problems with using the traditional runoff approaches.

Empirical formulas were the primary methods used for determining the peak flows at both Site 10, and Site 7a. Three methods were used to calculate reasonable peak flow values. These methods typically resulted from in-depth studies that present a general, over-estimation of the flow expected at the location of interest. Empirical formulas do not directly use any historical stream flow measurements or precipitation data. All the peak flow calculations are show in Appendix CF.

4.2.1 Peak Flow Isolines Method

The BC Ministry of Environment produced a set of isolines representing a 100-year peak flow in m^3/s per 100 km^2 of catchment area. The isolines were developed by taking stream-flow data from the period 1960-1995 and preforming a frequency (Coulson, 1998). The frequency analysis primarily used a Log-Pearson Type 3 distribution to estimate runoff values for ungauged drainage basins (Coulson, 1998). The 100-year flow is representative of all runoff including rain and snowmelt. The isoline closest to Hat Lake Road has a magnitude of 20 m^3/s per 100 km^2 as shown in Figure 9 below. To determine the peak flows, the fraction of site drainage area was multiplied by the isoline magnitudes.

Figure 9: Peak flow isoline (Ministry of Environment, n.d.)

The flow isoline in Figure 9 represents the maximum predicted runoff for a 100-year storm event. Site 10 and Site 7a are roughly located between Trembleur Lake and Tezzeron Lake, lying almost directly on the isoline.

DWB endorses this peak flow method. From their experience, this method over estimates the 100 year flow.

4.2.2 Talbot's Equation

The Talbot's Equation is a modified version of the Rational Method that includes a built-in rainfall intensity and return period. The coefficients that Talbot's Equation typically use are based on a rainfall intensity of 2.5 inches or 50 mm per hour and a flow velocity of 10 ft/s. The velocity of 10 ft/s because that is the typical speed of water in a culvert (Tolland, Jaime, & Russell, n.d.).

This method does not provide an accurate representation of the peak flows but acts as a check to compare against the other methods used. Talbot's Formula is shown below:

$$\boldsymbol{a = CA^{\frac{3}{4}}}$$

Where: a = X-sectional area required (ft^2)

C = Talbot's coefficient

A = Drainage area (acres)

Appendix G displays the chart of Talbot's coefficients for various slopes and land types. This particular version of the Talbot's Equation produces the cross-sectional area required for culverts. Different versions of this equation are available to find the cross-sectional flow in an open channel.

Talbot's Equation is not used by our sponsor to calculate the Q100 flow. For this reason, we have decided to not use the results to calculate the maximum stream height and culvert size for our design.

4.2.3 FREG Method

The *Forest Road Engineering Guidebook* (FREG) specifies a simplified method for determining the 100-year flow area in a culvert based on the characteristics of the stream. Also known as the California Method, this method assumes that the high water width of the stream represents the mean annual flood cross-sectional flow. The Q100 is then represented by three times the cross-sectional flow (Ministry of Forests, Lands and Natural Resource Operations, 2013). Figure 10 shows the dimensions needed to estimate the Q100 flow.

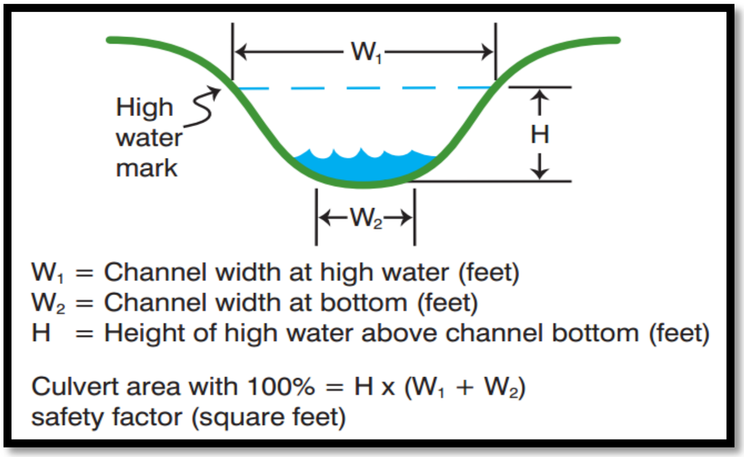

Figure 10: Flow cross-section (Colorado State Forest Service Manual, 2011)

As presented in Figure 10, the FREG method requires the high water width, stream depth and the bottom width be estimated. These dimensions are used to find the cross-sectional flow area which is multiplied by 3 to estimate the Q100 flow area in a culvert as shown in equation below.

$$a_{req} = \frac{(W_1 + W_2)}{2} * D * 3$$

CCC processed field survey data into C3D and determined the bottom channel width. The top channel width was obtained in a similar manner with a combination of site pictures. A thorough site inspection would allow for a more accurate estimate of yearly high water level. Appendix H shows the exert from the Forest Road Engineering Guidebook that covers peak flow estimation.

Figure 11 shows the current stream at Site 10.

Figure 11: Site 10 stream vegetation (DWB, 2012)

Figure 11 shows that the young deciduous vegetation growing at the sides of the bank have been growing for a short period of time. The younger vegetation indicates large flows occurring periodically that wipe out the vegetation growing on the stream banks.

The FREG method should only be used for a flow of less than 6 m^3/s and a culvert diameter of less than 2 meters (Ministry of Forests, Lands and Natural Resource Operations, 2013). This method was still used at Site 10 (a flow of more than 6 m^3/s) to act as a comparison against the other methods used. DWB noted that this method overestimates the peak flow.

4.3 Alternative Peak Flow Analysis

This section discusses other common methods of obtaining peak flows and why they were not applicable for CCC's design. There are three main types of methods for estimating peak flows in a forestry setting are:

- Frequency analysis
- Empirical formulas & historical evidence (covered in Section 3.2)
- Mathematical equations.

4.3.1 Frequency Analysis

There are two types of frequency analysis that estimate the peak flow in a stream: frequency analysis and regional frequency analysis. When snowmelt presents a significant concern, these methods are commonly used to evaluate peak flow. The complexity of mathematically modeling snowmelt leads to modelling historical flows to determine a sufficient design flow.

The first type, frequency analysis, is used when sufficient stream flow monitoring is available for the stream of interest. The historical flows are then used to predict a peak flow for a given recurrence interval. Frequency analysis does not apply to our crossing locations since stream monitoring data is unavailable. If our sites had a higher importance, then a stream monitory device should be installed to determine flow history before the crossing is constructed.

The second type of analysis, regional frequency analysis, examines nearby stream gauging data to obtain an estimated flow in the stream of interest. The historical flows are used to predict a peak flow for a given recurrence interval using frequency distributions. This method was not used because it requires significant time to accurately develop relationships to nearby streams. Moreover, it requires several years of hourly flow data which can be difficult to obtain in BC. The regional frequency analysis is likely the method the BC Ministry of Environment used to develop the 100-year flow isolines.

3.3.1.1 PMF

B. T. Abrahamson, P.Eng., prepared a report for the Minister of Agriculture on the probable maximum flood (PMF) estimation in BC. This study uses a frequency analysis of similar regions throughout BC to estimate the PMF for regions throughout BC. The PMF estimates the maximum flood that would probably occur in a given basin (Abrahamson, 2010). The PMF typically represents a return period of over 500 years. The equation that represents the BC interior, where Hat Lake Road is located, is shown in equation below.

$$\boldsymbol{Q = 19.933A^{0.6351}}$$

Where: $Q = Flow\ (m^3/s)$

$A = Area\ (km^2)$

The method yields a flow that 10 times higher than the other methods that were used in Section 4.2 above.

3.3.1.2 Comparison to Stuart Stream (Gauged Site)

The Stuart Stream catchment area is located within the Hat Lake Road boundaries and encompasses an area of 15,000 km^2. (Ministry of the Envirnoment and Parks, 1981). The catchment areas of Site 7a and Site 10 are sub-catchment areas for Stuart Stream.

Hay & Company performed a frequency analysis on Stuart Stream that was based on 59 years of recorded data. Hay & Company used 3 distribution techniques that results in a maximum 100-year peak flow of 590 m^3/s for Stuart Stream (Hay & Company Consultants , 1990).

If the 100-year peak flow is divided by the drainage area, a flow per km^2 can be established. For Stuart Stream, the flow per km^2 is 0.03933 m^3/s. If this value is then multiplied by the drainage area for Site 10 (140 km^2), we obtain a 100-year flow of 5.5 m^3/s. These 2 areas cannot be accurately compared linearly because of the vast size difference and uncertainty in the precipitation. The 100-year flow value reassures us that the design value of 30 m^3/s for Site 10 is more than reasonable to use for design.

3.3.1.3 Frequency Analysis Drawbacks

The primary reason for not using the regional frequency analysis was that there was considerable logging in the regions within the last few decades. The regional frequency analysis method is inaccurate and should not be used when the land has changed (US Army Corp of Engineers, 1979). Figure 12 displays the current areas that have been clear cut in the vicinity of Site 10 and Site 7a, which is very similar to nearby drainage basins.

Figure 12: Clear cut areas around Site 10 and Site 7a (Google Maps, 2015)

Figure 12 shows the extent of logging near the streams that are crossed over. The green marker represents Site 10, the red marker represents Site 7a, and the blue marker represents Site 7c which we have not designed.

After an area has been logged, considerable changes to the runoff and peak flow rates occur. Another concern of using regional frequency analysis was the distance to the closest gauging stations and how that would affect the flow at our location. The nearest gauging station at Stuart Stream is over 50 km away.

4.3.2 Mathematical Equations

Mathematical equations use relationships between historical data collected and site location characteristics. The Rational Method is one common method that uses Intensity-Duration-Frequency (IDF) curves to estimate the runoff based on the drainage area. The IDF curves are developed from rainfall data collected at a nearby region. This method is based on a rainfall event and does not take into account snowmelt. IDF curves should not be used in areas of high elevation, mountainous areas, or areas of significant snowmelt (Government of BC, 2007).

The snowmelt can be modelled individually but produces inaccurate results because there are too many unknowns that need to be estimated. Some of the elements that contribute to the snow melt rate include

- solar radiation,
- thermal radiation,
- sensible heat transfer from air,
- latent heat of vaporization from condensation,
- conduction from the ground, and
- advected heat from precipitation.

The variables above are all factors used when conducting an energy balance approach when trying to determine the change in snowpack volume. The Degree-Day-Method is a simpler way to determine the snowmelt rate. This method uses daily temperatures with an empirical coefficient to determine the melt rate. Both of these methods accurately model snowmelt but cannot be used to model the runoff from snowmelt (United States Department of Agriculture, 2004).

Other methods, such as the Burkli-Ziegler Equation and Soil Conservation (SCS) method, are also viable options when snowmelt is of insignificant concern.

4.4 Stream Depth

The stream dimensions were surveyed, which was processed in C3D to generate a cross-section. The cross-sectional flow area was overlaid onto the stream cross-section to determine the maximum height the water will be during a Q100 flood event.

5.0 ROAD ALIGNMENT

From July to October 2013, DWB conducted field surveys to acquire survey information. The survey team consists of three surveyors with ranging experience. The crew leader is given a set of instructions from the project engineer before starting the survey. Typically, the surveyors record the date of survey, weather, temperature, type of survey, and the given instructions on the first page of the survey notes.

5.1. Hat Lake Road Conceptual Design

CCC was provided with a topographic survey that has a 30 m width strip for the 9.5 km stretch of Hat Lake Road. The existing soil conditions are only known at the Site 10 and Site 7a crossing locations. Preliminary soil analyses reveal the sites consist of more than 80 percent silty sand.

CCC reduced the scope of road design to Site 10 and Site 7a crossings. Although the conceptual design of Hat Lake Road is outside of our scope, we included a conceptual design for completeness. To complete a comprehensive road design, CCC requires a detailed topographic survey of the area, thorough soil data, and time. The conceptual road design included in our report is CCC's best interpretation of the existing data and may not be an optimized design in terms of cut and fill balance. However, we believe our conceptual design provides valuable insight to overall road arrangement and earthwork volume.

5.1.1 Horizontal Alignment

DWB completed the route selection and layout of Hat Lake Road. The objective of our conceptual design is to develop a double-lane forestry road and to maximize the travel speed.

The majority of the existing surface slopes are relatively flat, ranging between 0 to 20 percent except for the Site 7a approach. The *Forest Road Engineering Guidebook* summarizes the relationship between stabilized road width and the corresponding suggested maximum road gradient; Figure 13 summaries the alignment controls for forest roads.

Stabilized Road Width (m)	Design Speed (km/h)	Minimum Stopping Sight Distance[a] (m)	Minimum Passing Sight Distance for 2-Lane Roads (m)	Minimum Radius of Curve (m)	Suggested Maximum Road Gradient[b,c]				
					Favourable		Adverse		
					S	P[d]	S	P[e]	Switchbacks
4	20	40		15	16%	18% for distance <150 m	9%	12% for distance <100 m	8%
5–6	30	65		35	12%	14% for distance <150 m	8%	10% for distance <100 m	8%
	40	95		65					
8+	50	135	340	100	8%	10% for distance <200m	6%	8% for distance <100m	6%
	60	175	420	140					
	70	220	480	190					
	80	270	560	250					

Figure 13: Summary of alignment controls for forest roads (Ministry of Forests, 2002)

The typical width of a L-100 logging truck is approximately 2.8 m. A road width greater than 6 meters is considered a double-lane forestry road. CCC's initial design consisted of a combination of 6 meter and 8 meter road widths. However, for the 6 meter road width design, the traffic path simulation identified conflicts in sweep path. Therefore, CCC considered 8 meters to be the minimum width for a double-lane road. Hat Lake Road is designed as a double-lane road except at specific locations. We found that the horizontal alignment of the road is relatively straight. The curve radius required at each curve is greater than 1000 m.

In terms of horizontal alignment, the most critical location is the Site 10 approach. At Site 10, the existing stream runs parallel to the proposed road alignment. To cross the stream, CCC designed a series of sharp curves at a 35 m radius as shown in Figure 14.

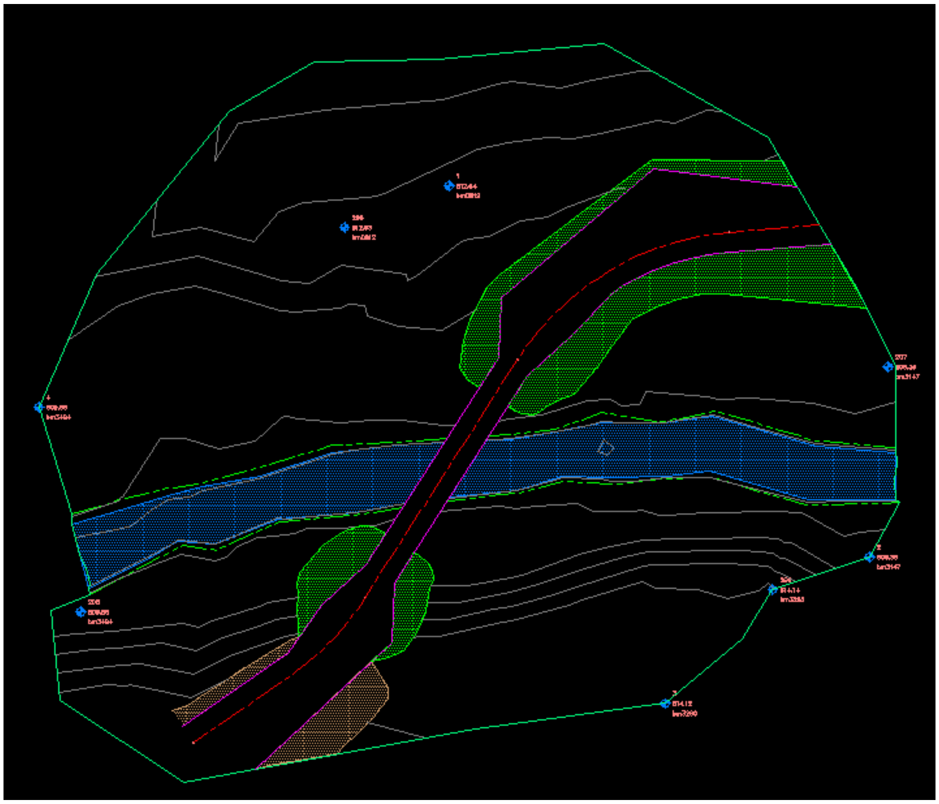

Figure 14: Site 10 stream crossing (Kim, 2015)

Figure 14 shows the 35 m radius curve that corresponds to a road width of 6 meters. As a result, the road width tapers down to 5 meters from 8 meters and becomes a single-lane at this location. The road widens to 8 meters and returns to a double-lane road when the road exits the crossing. Note that curve widening and turnouts are placed at the approach curves to permit pull out. Refer to Appendix I for the Site 10 vehicle sweep path simulation.

5.1.2 Vertical Alignment

The Hat Lake Road vertical profile is gentle except at the Site 7a approach. In most cases, the design vertical grade did not exceed 8 percent. The existing surface sloped considerably and required a 12 to 16 percent road grade at some locations. As per Figure 15, our standard 8 meter road width does not permit extreme vertical grades such as 12 to 16 percent.

Design speed (km/h)	Minimum SSD[a] (m)	Crest One-lane	Minimum SSD (m)	Crest Two-lane	Sag One-lane
20	40	1.7	20	1.0	2.1
30	65	4.5	35	3.1	5.1
40	95	9.6	50	6.3	8.5
50	135	19.4	70	12.3	13.4
60	175	32.7	90	20.3	18.7
70	220	51.6	110	30.3	24.0
80	270	77.8	135	45.7	30.8

Figure 15: Minimum K values (Ministry of Forests, 2002)

Hence, there are locations where the road width narrows down to 4 or 5 meters to accommodate a steeper vertical grade. By changing the road width and road grade, the design speed also changes. The 8 meter road width corresponds to a design speed of 20 km/h. The 4 and 6 meter road width corresponds to 20 km/h and 30 km/h respectively. We followed the minimum K values specified in Figure 15 for each design speed.

The horizontal and vertical alignment of Hat Lake Road is shown in Appendix J. At STA 0+360, 0+480, 1+120, and 2+820, sudden dips and valleys are visible along the profile. CCC suspects that these depressions are additional stream crossing sites which are not part of our scope. More thorough investigation is required at these crossing locations.

Due to the nature of the existing flat surface, full benching or self-balancing design is not feasible. Figure 16 presents the types of road cross-sections: full-benching, self-balancing, and over-landing.

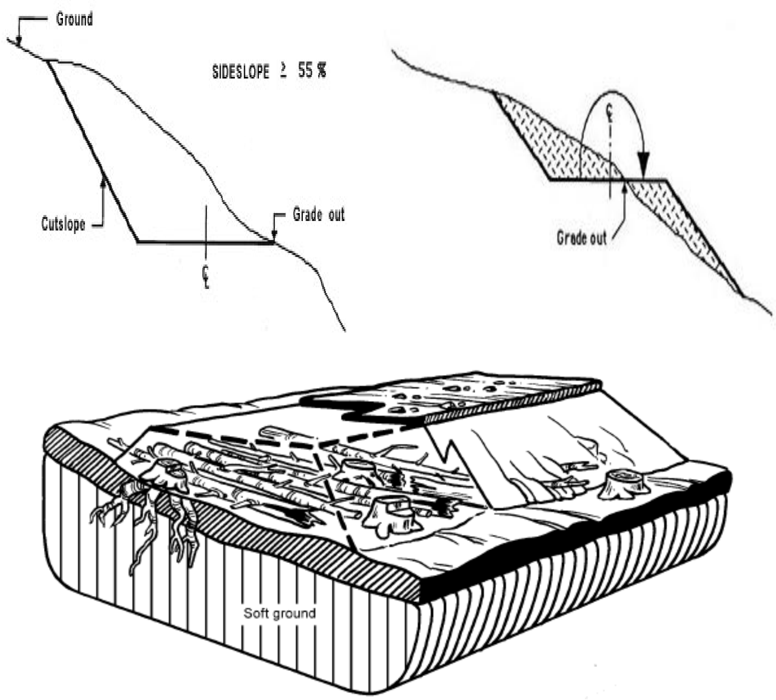

Figure 16: Full-benching (top left), self-balancing (top right), over-landing (bottom) design (Ministry of Forests, Lands and Natural Resource Operations, 2013)

Most of our road sections are the over-landing type. Refer to Appendix J for a typical road structure and cross section at a 20 m interval.

5.1.3 Typical Road Structure

CCC was provided with soil analysis data at Site 7a and Site 10 only. We only have soil data for 1 km of the 9.5 km Hat Lake Road. The soil analyses at Site 7a and Site 10 indicate the existing earth consists of fine sands. CCC explored iMapBC in an attempt to identify the soil type for the unknown 8.5 km of the road. Thus, we assumed the entire site is made of silty sand.

Figure 17 shows the general guidelines for cut and fill slope angles for forestry roads.

	Cut Slopes		Fill Slopes	
	Examples of Material Types[a]	Suggested Cut Slope Angles[b] for Cut Bank Height < 6 m[c]	Examples of Material Types[a]	Suggested Fill Slope Angles[b]
Coarse-grained Soils[d]	Road cuts in loose to compact SANDS or SANDS and GRAVELS (not cemented and non-cohesive)	1½ H : 1 V	Road fills composed predominantly of SANDS, or SANDS and GRAVELS, or drained mixtures of coarse-grained and fine-grained soils	1½ H : 1 V
Fine-grained Soils	Road cuts in loose SILTS, or soft cohesive soils such as SILTY CLAYS or CLAYS (not consolidated and not cemented)	1½H : 1V for lower cuts to 2H : 1V for higher cuts	Road fills composed predominantly of SILTS or CLAYS[e]	2H : 1V
	Road cuts in hard cohesive soils such as SILTY CLAYS or CLAYS (consolidated)	1H : 1V or flatter		

Figure 17: General guidelines for cut and fill slope angles for use in forest road design (Ministry of Forests, Lands and Natural Resource Operations, 2013)

Figure 17 recommends cut and fill slopes of 1.5H to 1V for our soil type. Based on our discussions with DWB, 1.5H to 1V cut and fill slopes is consistent with other similar forestry road projects.

The MFLNRO *Engineering Manual* does not go into detail regarding the gravel layer thickness. However, the American Association of State Highway and Transportation Officials (AASHTO) developed a comprehensive design method for low-volume roads. The AASHTO guidelines consider seasonal roadbed soil resilient modulus, effective roadbed soil resilient modulus, and season length to determine the thickness of the gravel layer.

Figure 18 shows the different climatic regions throughout the United States.

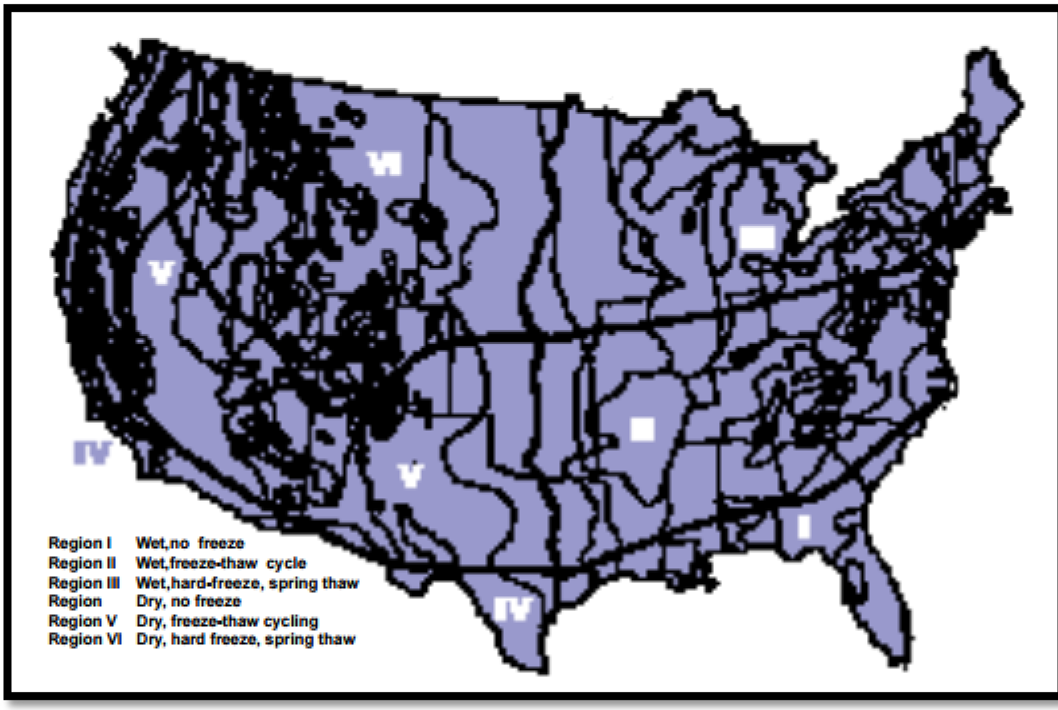

Figure 18: Climatic regions in the United States (AASHTO, 1993)

Hat Lake Road is in a dry region with repeated freeze-thaw cycling which shares the same characteristics as Region V (shown in Figure 18). Under this climatic condition, the road will be subject to the following roadbed soil moisture conditions shown in Figure 19.

	Season (Roadbed Soil Moisture Condition)			
U.S.CLIMATIC REGION	Winter (Roadbed Frozen)	Spring/Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
I	0.0[1]	0.0	7.5	4.5
II	1.0	0.5	7.0	3.5
III	2.5	1.5	4.0	4.0
IV	0.0	0.0	4.0	8.0
V	1.0	0.5	3.0	7.5
VI	3.0	1.5	3.0	4.5

[1] Number of month for the season

Figure 19: Suggested season length (months) for the US climatic regions (AASHTO, 1993)

The roadbed is frozen during the winter season, saturated during the spring-thaw season, wet during the spring and fall season for 3 months, and dry during the summer season for 7.5 months.

Assuming Hat Lake Road's roadbed soil quality is acceptable, the roadbed soil resilient moduli will be 30,000 psi during winter, 2,000 psi during spring-thaw, 6,000 psi during spring and fall, and 10,000 psi during summer (AASHTO, 1993). AASHTO states that low-volume forestry roads are subjected to a maximum of 100,000 18-kip ESAL and a practical minimum of 10,000 18-kip ESAL load (1993). CCC averaged the maximum and the practical minimum 18-kip ESAL load and used a 55,000 kip 18-kip ESAL load in our design (AASHTO, 1993).

The design serviceability loss parameter, ΔPSI, ranges from 0 to 5. When the road's ΔPSI reaches 0, the road is considered to have reached the terminal condition impossible for driving. AASHTO suggests using a range between 2 to 3 ΔPSI. CCC used 2.5 ΔPSI, at which 55 percent of the motorists find the road unacceptable to drive on.

Considering Hat Lake Road will be surfaced with a gravel layer, it is likely that rutting will happen over time. Continued rutting forms channels along the wheel paths and makes driving difficult. CCC considers 2 inches to be an acceptable tolerance for rutting in the surface layer before re-grading must be done.

AASHTO's design chart, shown in Figure 20, was used to determine the overall total damage to the surface layer considering serviceability loss.

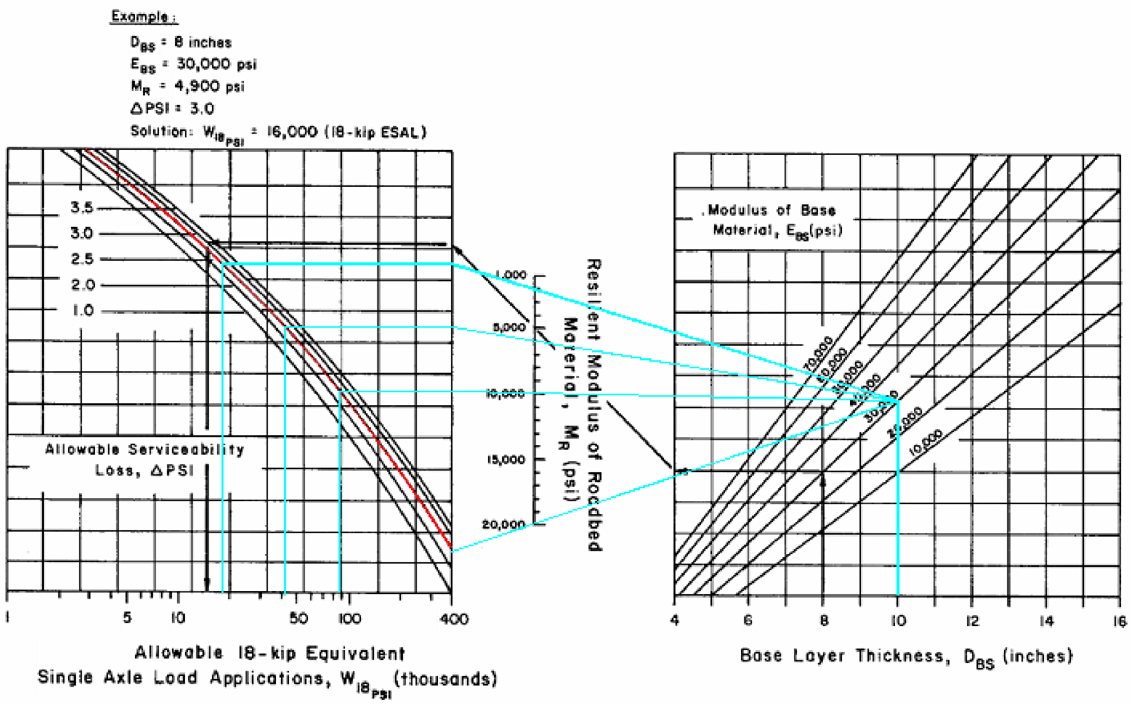

Figure 20: Design chart considering allowable serviceability loss (AASHTO, 1993)

In addition, the overall total damage to the surface layer considering allowable rutting depth is computed using the chart in Figure 21.

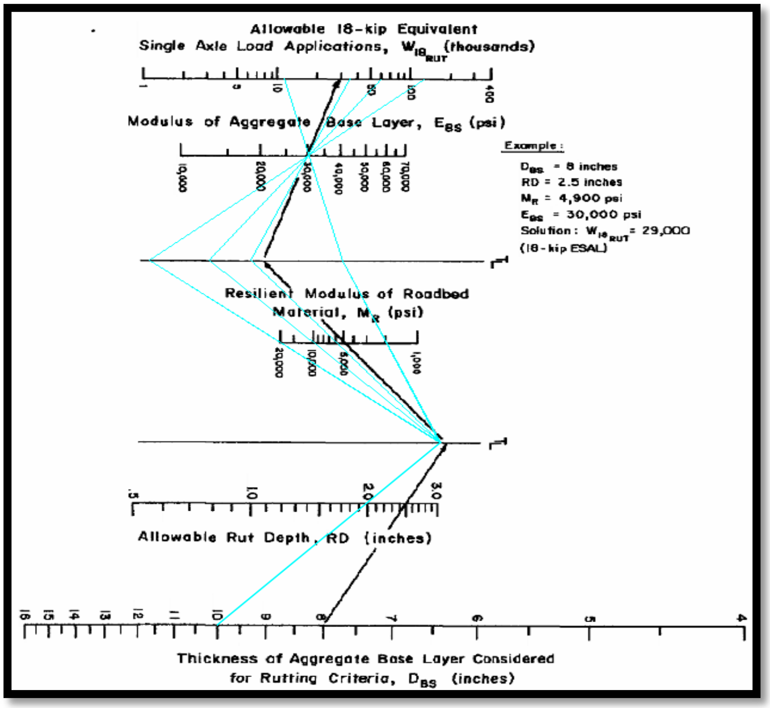

Figure 21: Design chart considering allowable rutting (AASHTO, 1993)

Using the chart in Figure 21, the trial base thicknesses of 10, 11, 12, and 13 inches are used to produce a graph that relates the total damage and pavement thickness. Figure 22 shows a plot of total damage versus pavement thickness.

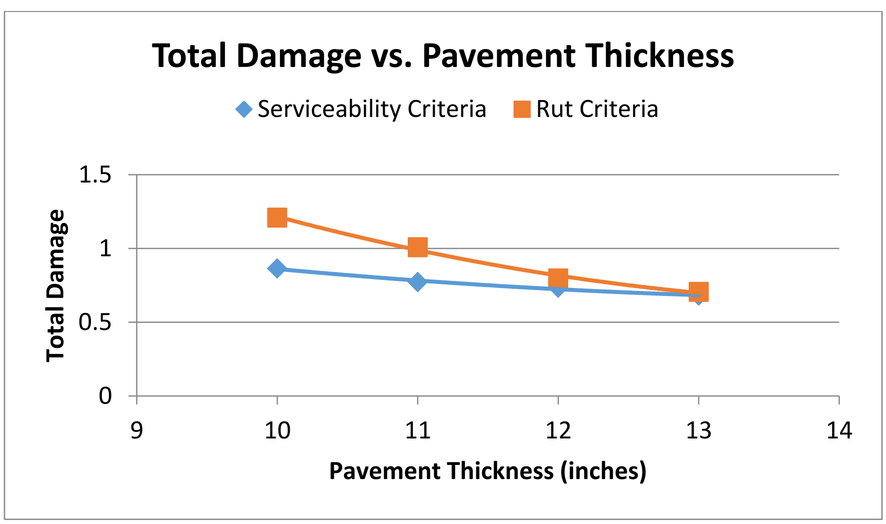

Figure 22: Total damage vs. pavement thickness (Kim, 2015)

For a total damage value of 1, the rutting criterion governs and requires 11 inches of gravel thickness. Appendix K shows the full pavement design chart.

Although 11 inches (275 mm) of gravel layer is far thicker than the recommended depth of 100 mm from DWB, CCC believes prescribing 11 inches will make up for the uncertainty in underlying sub-base. Therefore, the road shall be constructed using a minimum of 300 mm of native sub-base and 275mm of well-graded, 25 mm diameter gravel layer.

Since the road will be surfaced with permeable gravel, specifying a 2 percent cross-fall is not required for drainage purposes; it would also be difficult to obtain such precision on site. CCC designed the road with a normal crown of 2 percent for the purpose of this project.

Road width varies along the alignment and ranges 4, 6, and 8 meters. Table 2 summarizes the varying road width.

Table 2: Road width along Hat Lake Road (Arena, 2015)

Stationing	Road Width (m)
0+000 to 2+420	8
2+420 to 2+600	4
2+600 to 3+220	8
3+220 to 3+560	5
3+560 to 7+860	8
7+860 to 8+180	5
8+180 to 10+450	8

In addition to the road widths shown in Table 2, a forest clearing width must be established to clear vegetation from the road right of way. In the MFLNRO *Forest Road Engineering Guidebook*, the ministry recommends a clearing width of 11 m from the road centerline be used on the cut side, and 7 m be used on the fill side for an 8 m wide road as per Appendix 5, Table A5.5 (2002). In certain locations, this clearing width was not sufficient to accommodate the fill or cut; thus an additional 3 m clearing width was offset from the toe of the cut or fill where necessary. This clearing width involves clearing 20.9 Ha of forest to construct the road surface for the Hat Lake Road project. Refer to Appendix J for conceptual Hat Lake Road design. Note that only the first three kilometers of the cross-sections are provided in Appendix J.

5.2 Site 7a Crossing Design

Site 7a starts at 7+940 and ends at 8+020. The stream is 5.5 m along the top of bank and 1.0 m along the bottom of channel. During the surveying of Site 7a, the stream was 1.5 m wide and 200 mm deep. Figure 24 shows the existing stream at Site 7a.

Figure 25: Site 7a existing stream (DWB, 2013)

The soil at Site 7a consists of 83 percent fines with limited amounts of sand. Refer to Appendix L for a full particle size distribution report for Site 7a.

The stream at Site 7a has a unique feature: the stream loops and creates a distinct oxbow about 25 m downstream of the crossing location. For our project, no geotechnical analysis was carried out for the oxbow loop. CCC believes the bulb portion of the oxbow will eventually erode enough to separate from the stream. Erosion of the bulb will take several years and will not happen during the design life of the culvert. Re-vegetation will occur over time and the bulb portion will be no different than any other existing banks. Figure 23 shows the layout of the stream at Site 7a.

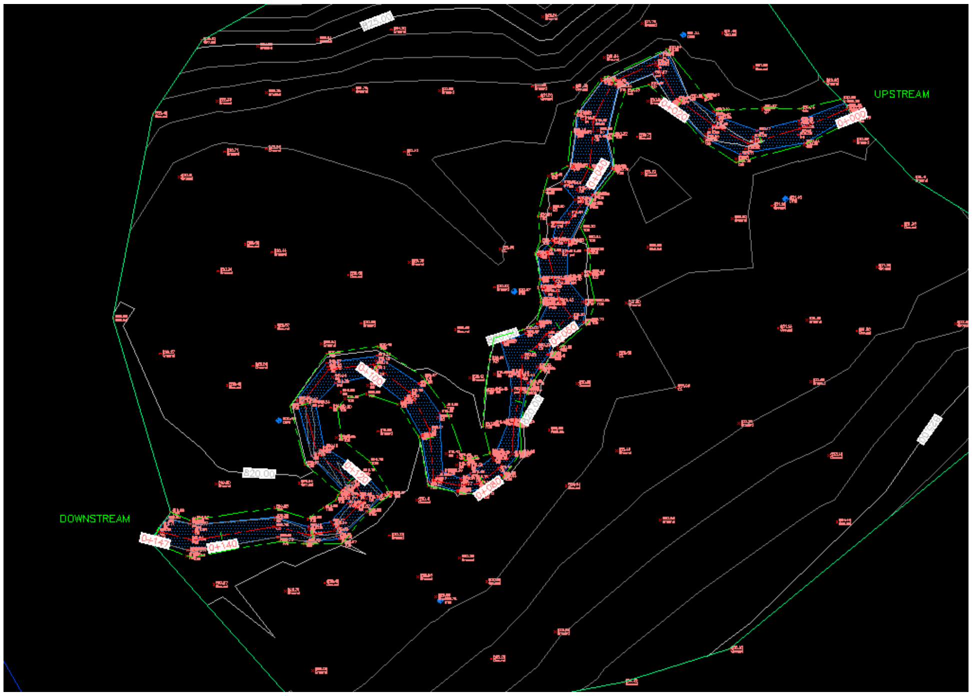

Figure 23: Site 7a existing topography (Kim, 2015)

In addition to the oxbow downstream, there is a steep incline on the north side of the site. Due to the steep slope at the north side and the presence of the oxbow on the south side, CCC was not able to apply much variation to the horizontal alignment.

Furthermore, the vertical alignment design had many constraints. The culvert required for the Site 7a crossing is 2.8 m in diameter in order to satisfy the cross sectional area requirement. With 40 percent embedment, the required culvert diameter is 3.6 m. On top of the culvert, 1000 mm of compacted fill is required and discussed in Section 9.2.2 of this report. As a result, the road elevation at the stream centerline needs to be at least 2.16 m higher than the bottom of the stream. To minimize the impact on the existing stream, CCC focused on keeping the road elevation as low as possible at the crossing location.

Minimizing the road elevation at the crossing led to the next challenge. Figure 24 below shows the crossing profile at Site 7a.

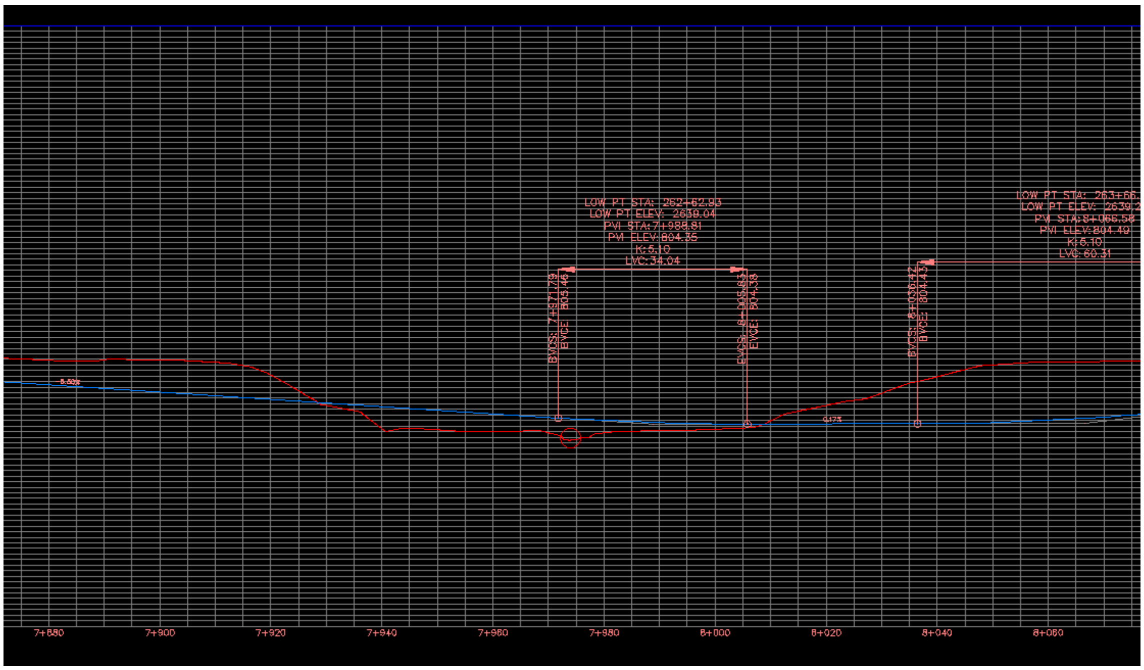

Figure 24: Site 7a crossing profile (Kim, 2015)

Figure 24 shows a 3.6 m diameter culvert with 40 percent embedment and 1300 mm cover at STA 7+970. The existing ground is shown in red and the proposed road profile is shown in blue. Note that the existing surface profile is relatively flat near the stream. However, at STA7+940 and 8+005, the existing ground slopes up at 40 percent over 20 meters. The existing ground elevation at the top of the 40 percent grade is about 13 m higher than the stream area. The road profile grade is limited to a maximum of 8 percent. Thus, the road profile cannot catch the existing surface at our desired location and creates an excessive cut problem. From STA 7+800 to 8+200, the approximate cut volume for the culvert design is 30,000 cubic meters.

One solution to resolve the cut problem is shown in Figure 25.

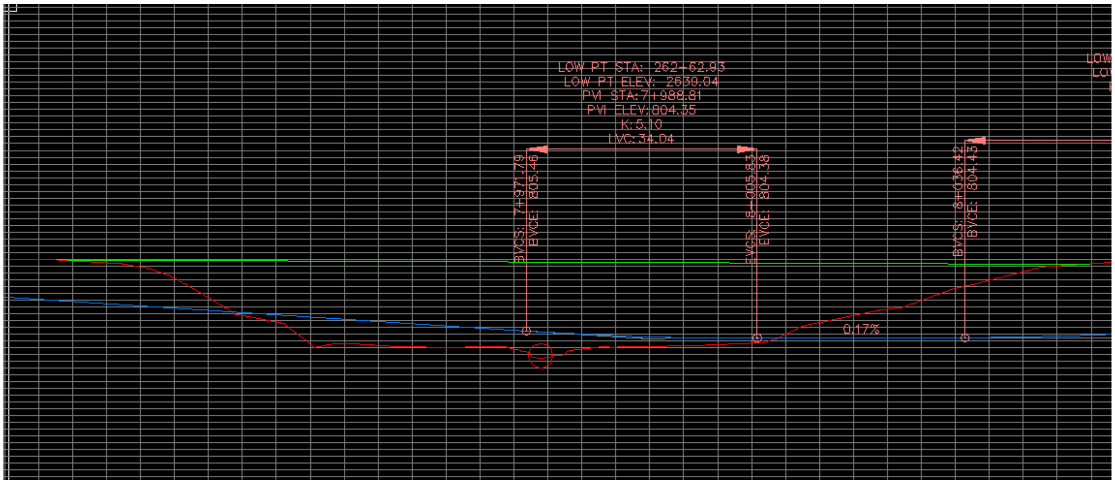

Figure 25: Site 7a crossing profile (2) (Kim, 2015)

The crossing could be made at the elevation shown in green. This option eliminates the cut issue but creates a massive fill problem. For this option to work, a 30 m span bridge is required for Site 7a.

However, based on our calculations for Site 10, the cost of building a 30 m bridge is more expensive than installing a culvert and excavating 30,000 cubic meters of soil. As per Appendix M, the supply and construction cost of a 30 meter bridge is $530 thousand. In contrast, excavating 30,000 cubic meters of earth is $110 thousand and the culvert installation is an additional $110 thousand. To summarize, the bridge option costs $310 thousand more than the culvert option.

We compared the bridge option and culvert option in terms of cost, construction duration, and environmental impact. Although the culvert option requires disturbing the existing stream, the culvert option is $310 thousand cheaper. Furthermore, installing a culvert is faster and easier than building a bridge. Moreover, with proper riprap installation, silt fencing, and culvert embedment, the environmental impact on the existing stream can be minimized. Therefore, CCC selected a culvert crossing for Site 7a. Refer to Appendix N for full Site 7a designs.

5.3 Site 10 Crossing Design

The stream at Site 10 is 7.5 m wide along the top of the bank and 5.5 m wide along the bottom of the channel. During the surveying of Site 10, the water was 6.5 m wide and 300 mm deep. DWB's photograph of the stream at Site 10 is shown in Figure 26.

Figure 26: Site 10 crossing (DWB, 2013)

Figure 26 shows the magnitude of flow in the Site 10 stream. The soil consists of 47.3 percent silty sand at this location. Refer to Appendix L for a full particle size distribution report for Site 10. To achieve a crossing at this location, three alignment and span options were explored. Refer to Appendix O for Site 10 designs.

6.0 SITE 10 - ALTERNATIVE EVALUATION

The road alignment and stream alignment are roughly parallel, so curved approaches are required to cross this stream at an appropriate angle. Three different options were developed based on the angle of approach and length of bridge span. Option A uses a 24.4 m long bridge and a design speed of 30 km/h. Option B uses a 30.5 m bridge and a design speed of 30 km/h. Lastly, Option C uses a 18.3 m bridge and a design speed of 20 km/h. A preliminary design of each option was developed so that the merits of each could be judged and the best design selected.

6.1 Option A (Medium Span)

Option A was the first crossing configuration we designed. Having completed the existing surface model of the entire Hat Lake Road area, we realized crossing the stream at a perpendicular angle is not feasible due to the steepness of the banks and the angle of the surveyed corridor. Figure 27 shows the surface model topography of Site 10.

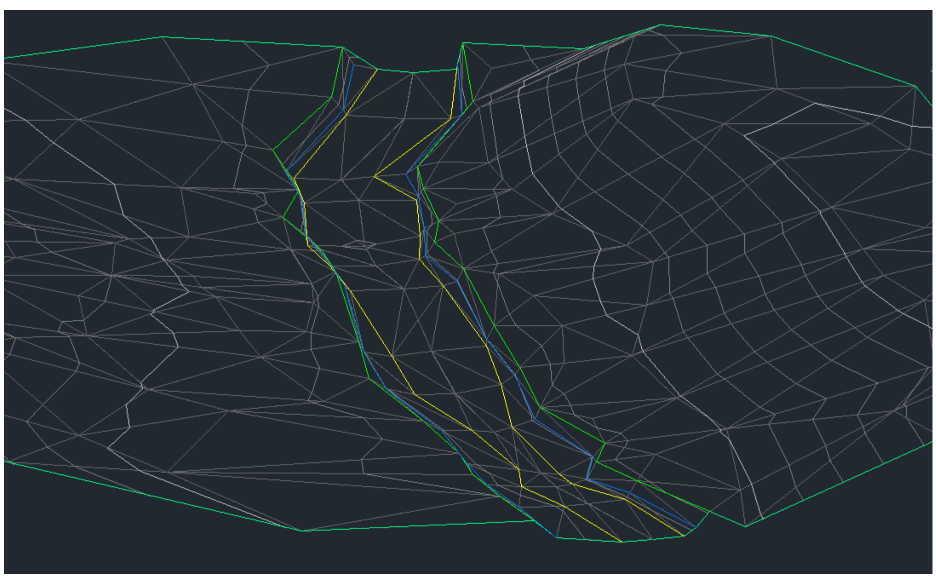

Figure 27: Site 10 existing topography (Kim, 2015)

As shown in Figure 27, the banks slope down drastically toward the stream, thus crossing at a perpendicular angle would require a large amount of earth fill. Since it is not ideal to slope the approach embankment, the bridge or the exit embankment, a 0 percent road grade would worsen the fill issue. Also, a perpendicular crossing requires a sharp truck sweep which jeopardizes safety. Finally, the existing topographic survey for this area is not large enough to be used for a perpendicular crossing.

Having considered these constraints, Option A crosses the stream with a 24.4 m long bridge at a 50 degree angle to the stream centerline. Figure 28 shows the proposed arrangement of Option A.

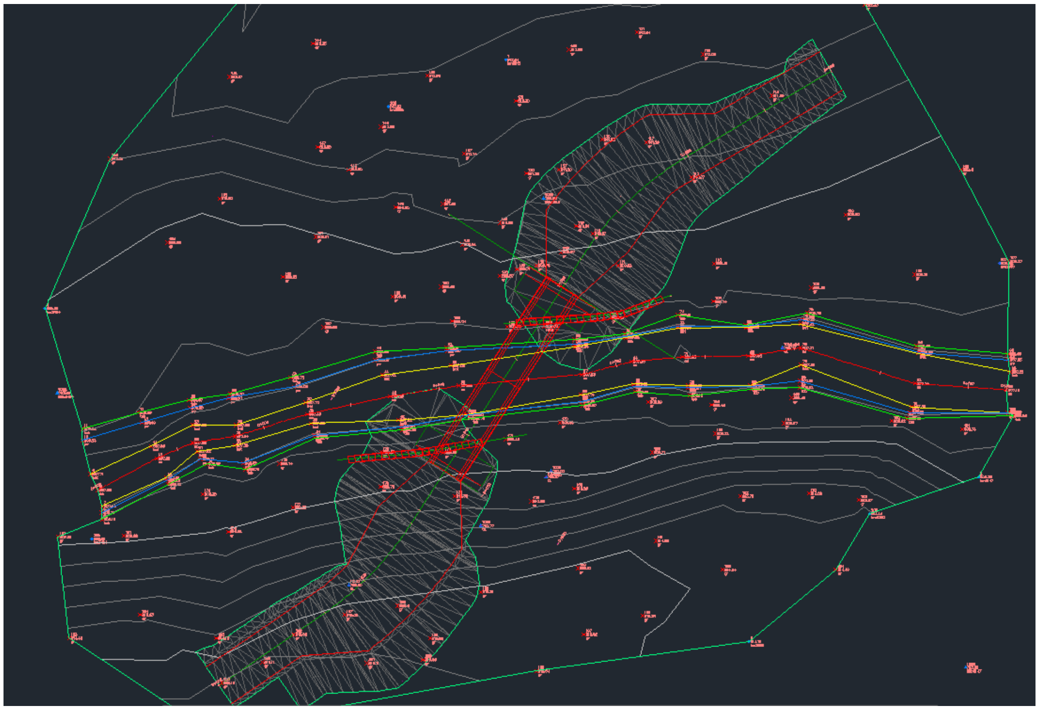

Figure 28: Site 10 Option A (Kim, 2015)

The red lines in Figure 28 denote the edge of the road and the dark green line denotes the centerline of the road. To obtain a crossing angle of 50 degrees, the road sweeps in at a 35 m radius approaching the bridge. After the crossing, the road curves at a radius of 35 meters on the high chain side to resume the original bearing. Note the light green lines around the road embankment represent the extent of road fill and spill.

In order to minimize material requirements and cost for the bridge structure, the start and end stations of the crossing were chosen to minimize the bridge's span. By locating the approach embankment close to the stream, the embankment fill encroaches the stream which necessitates soil retention structures.

To compare the performance of Option A to the other crossing options, two candidate retaining structures were analysed to retain the embankment soil. The retaining structure option that maximized CCC's design evaluation criteria was then selected for use. Each of the designs are discussed and compared.

6.1.1 Log Crib Retaining Structure

The first retaining wall structure considered was a lashed log-crib system. A log crib is built-up from limbed trees that are felled to clear the roadway. The log's ends are cut by chainsaw into Lincoln or A-V joint connections, then craned in place and lashed together with steel binding wire. Figure 29 demonstrates the details of Lincoln and A-V connections.

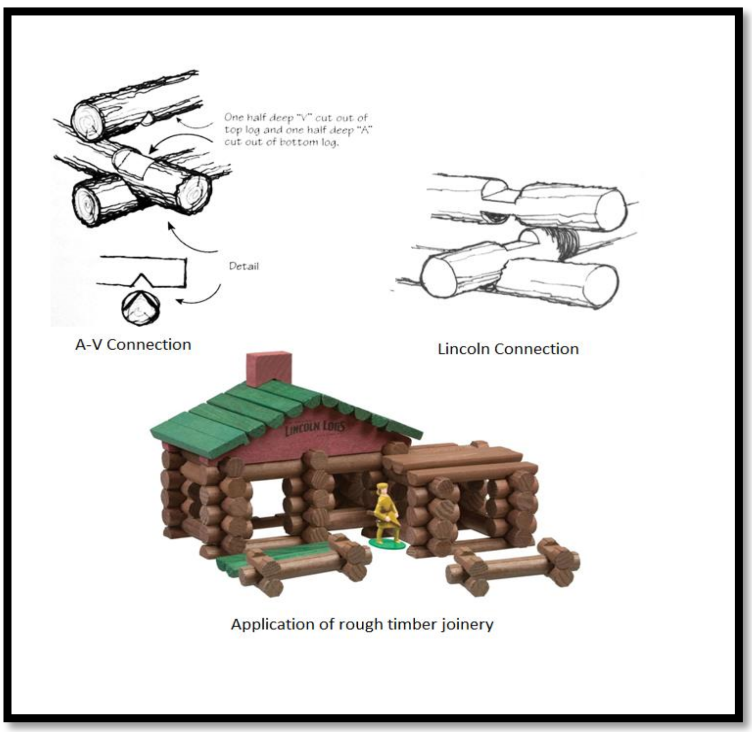

Figure 29: Lincoln and A-V connection detail and application (woodlandia, 2006)

As the majority of construction material is found onsite, the log crib option would result in low material purchase, transportation cost, and excellent environmental compatibility. This system requires the use of a crane for installation, intensive labour to construct, and frequent inspection to ensure proper functioning.

6.1.2 Precast Concrete Retaining Structure

The second retaining structure considered is composed of a combination of a precast reinforced concrete ballast wall to contain the embankment fill behind the bearing pads, with precast concrete lock-block wing walls to contain the embankment fill on the stream side. This system exclusively uses precast concrete elements which must be purchased from a production yard and transported to site resulting in additional material purchasing and transportation cost. The system also requires the use of a crane to install, but is less labour intensive as the elements are predesigned to fit together. This system involves little maintenance and inspection and has a long service life.

Although the environmental compatibility and cost savings associated with using existing material to create a log crib retaining structure presented an attractive option, CCC felt that the potential for loss of structural stability due to rot, and the resulting damage to the bridge structure and stream ecosystem was too high to pursue this option. Hence, we chose to use precast concrete elements for Option A. This would be comprised of two precast reinforced concrete ballast walls behind the bridge girders and concrete lock-block wing walls to retain the fill abutment side spill. A conceptual rendering of Option A as seen from the low chain approach is shown in Figure 30.

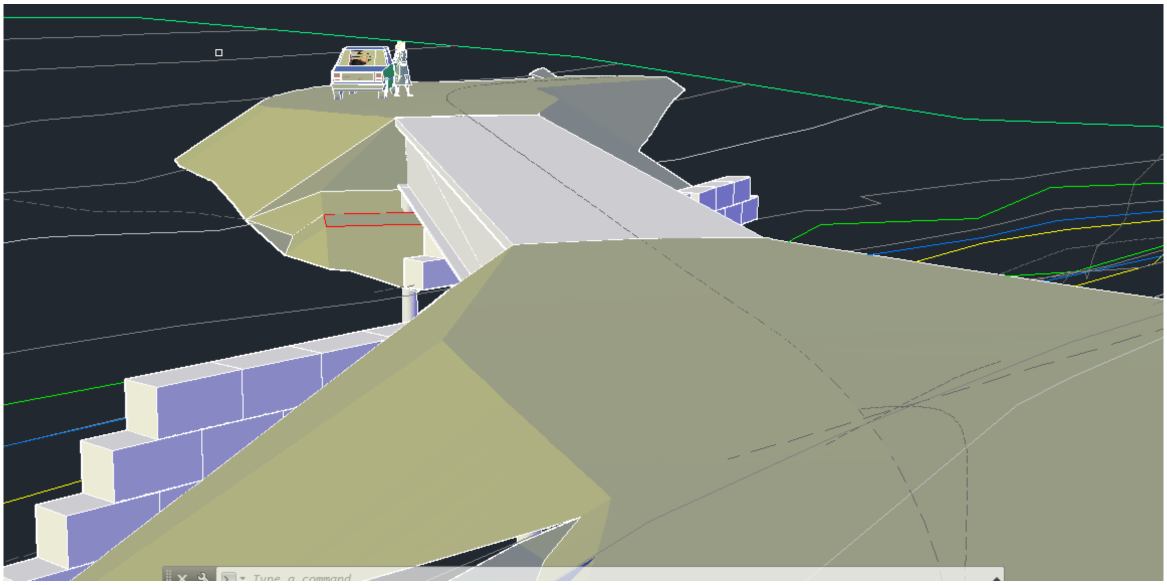

Figure 30: Site 10 Option A fill retainment (Kim, 2015)

Figure 30 shows the lock-block retaining the road spill. Refer to Appendix P for the site plan and lock-block configuration of Option A. The main problem with this arrangement is the extra work associated with containing the embankment fill. Since the road embankment was the issue, our next two options, Option B and C, are designed to minimize the impact of the embankment fill at the stream.

6.2 Option B (Long Span)

To improve upon Option A, Option B crosses the stream with a 30.5 m bridge at the same horizontal alignment angle as Option A. With the centerlines of the bridge at the same locations as Option A, the additional 6 meters in span moves the foundation location further away from the stream. By locating the approach embankment further away from the stream, the fill will not encroach on the stream which means no soil retention structures were necessary for Option B. Figure 31 shows CCC's arrangement of Option B without soil retention structures.

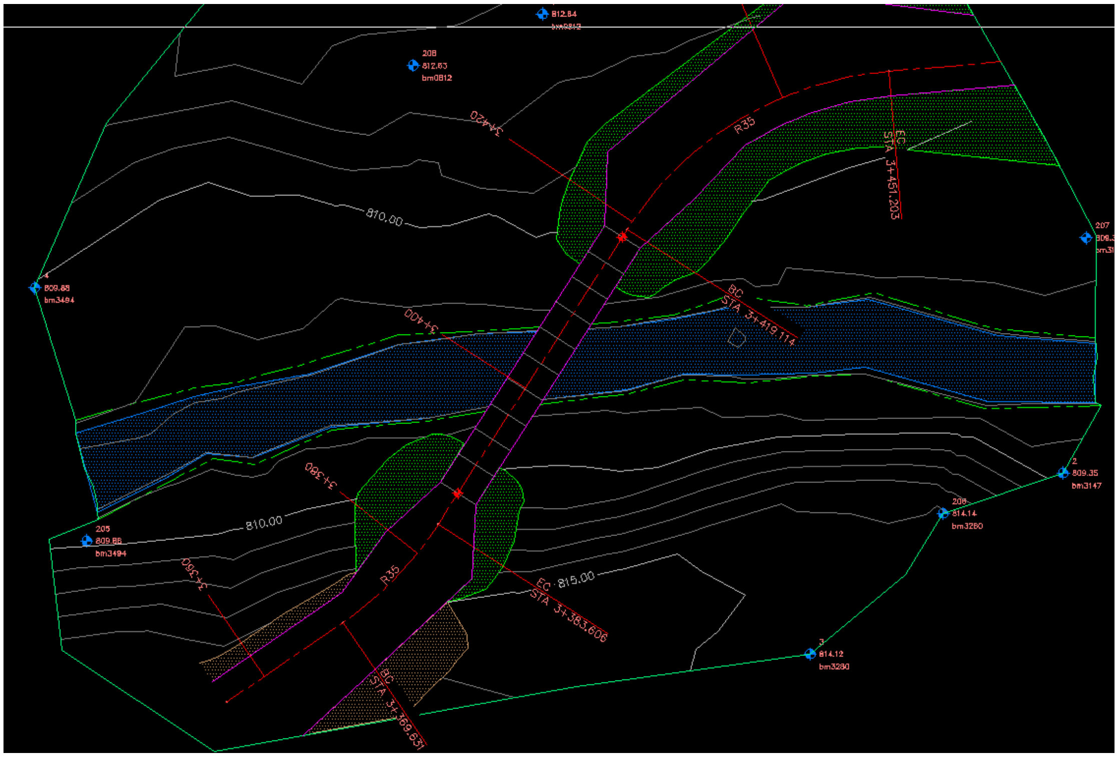

Figure 31: Site 10 Option B (Kim, 2015)

Option B (shown in Figure 31) solves the road embankment fill issue; however, the longer curves at the bridge approach and exit make it difficult for two (2) design trucks approaching the bridge span at the same time to pass one another. The L-100 design truck was modeled in *VehicleTracking* and used in a swept path analysis of the Option B layout to determine acceptable turnout geometry. The dimensions of the design truck body were taken from the MFLNRO standard design drawings of a L-100 design vehicle.

We originally designed a 15 m turnout width. From a *VehicleTracking* simulation, it was revealed the turnouts are not wide enough for a truck to safely pull out and yield to oncoming traffic. The turnouts were therefore increased in width and widened to accommodate truck turning. Figure 32 shows the arrangement of the truck pullouts for Option B.

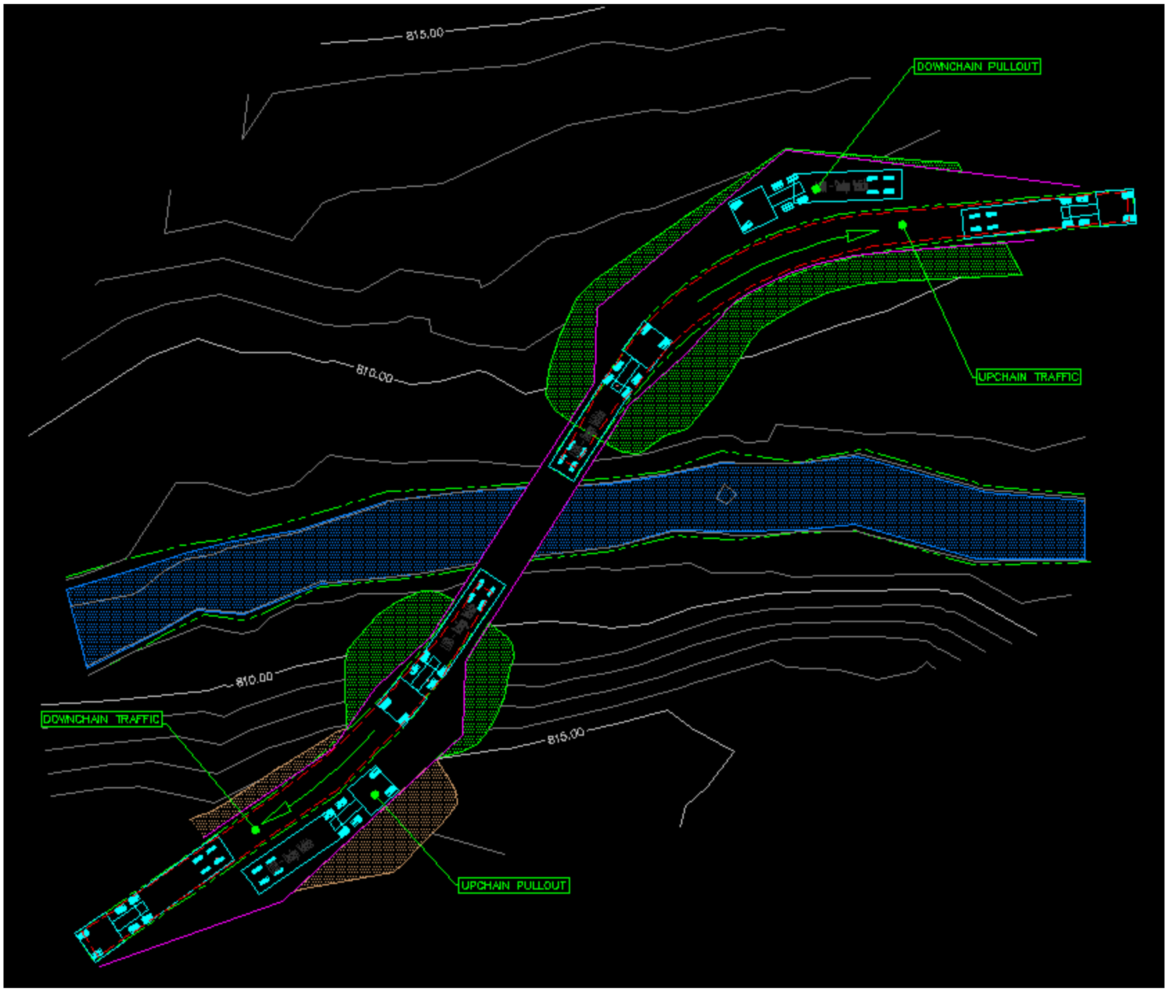

Figure 32: Site 10 Option B sweep path (Kim, 2015)

Figure 32 displays how a design vehicle crossing the bridge is able to pass an oncoming truck in the turnout zone. The green arrows denote the direction of traffic. Two trucks shown at the turnout location simulate the trucks yielding to the oncoming traffic. Refer to Appendix Q for the truck turning simulation, truck profile, and truck sweep path.

6.3 Option C (Short Span)

A higher design speed is desirable to facilitate faster transportation for the people who will use Hat Lake Road. Option A and B cross the stream at a 50 degree angle because the curve radius required for a 30 km/h road is 35 m.

CCC investigated lowering the design speed to 20 km/h which permits using a 15 m radius curve. With a smaller curve radius, the crossing can be designed at an angle close to 80 degrees. Figure 33 shows the proposed arrangement for Option C of the Site 10 bridge crossing.

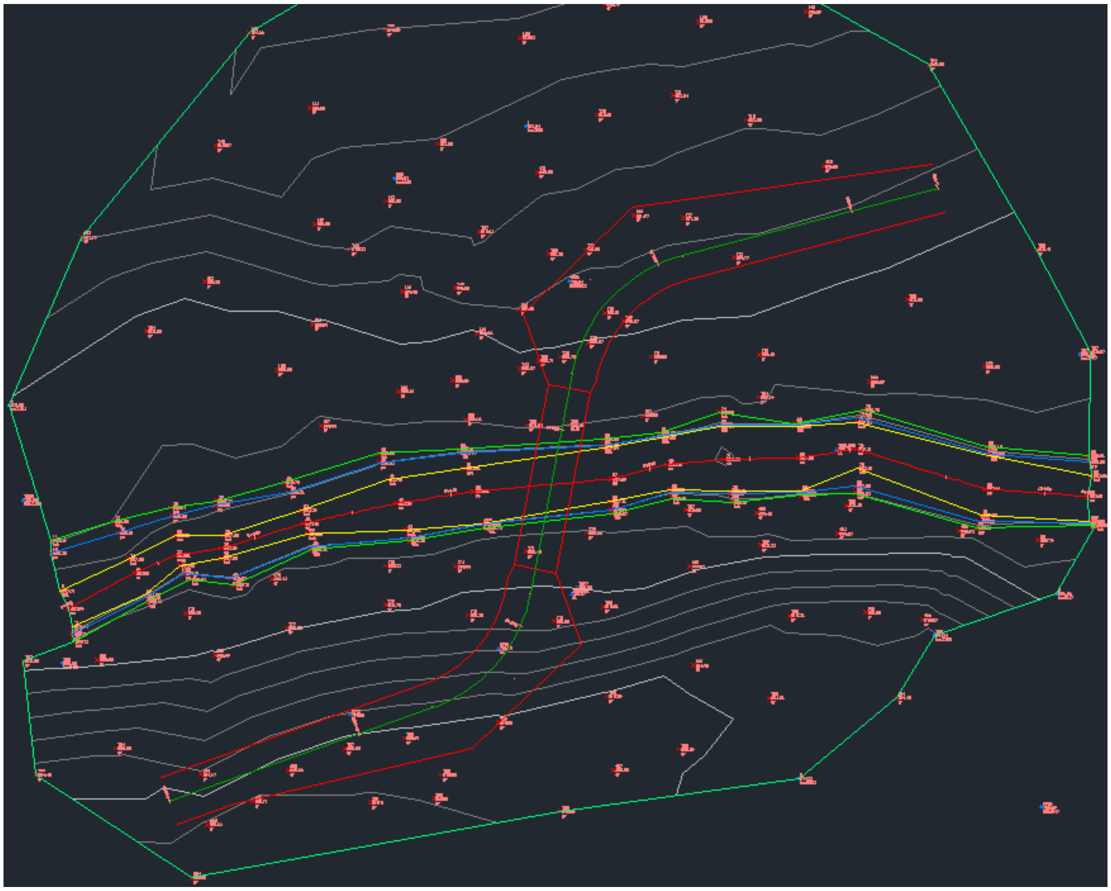

Figure 33: Option C crossing (Kim, 2015)

The aggressive approach angle shown in Figure 33 requires a narrow 4 m road width to accommodate the sharp turn. This is problematic since a substantially big turnout is needed, similar to the current 5 m road with for Option A and B. Furthermore, even though the design speed is 20 km/h, it is possible that the dstreams may cross the bridge at speeds greater than 20 km/h. At such sharp turns, vehicles exceeding the design speed could become unstable. Hence, Option C is less favorable than Option B in terms of horizontal geometry and design speed. Option C does not require retaining structures (as per Option A) and would require less girder material to support the load than Option B, due to its shorter span.

6.4 Alternative Selection

To determine the most feasible crossing option, CCC constructed a grading rubric considering safety and ease of truck maneuvering, sustainable use of materials, cost of material and labor, maintenance requirements, and constructability. Each of these criterion were assigned weighting factors ranging from 1 to 10 to reflect the relative importance that CCC deemed was appropriate based on our engineering judgement. The evaluation criteria and corresponding weighting factors are summarized in Table 3.

Table 3: Evaluation criteria and relative importance

Evaluation Criterion	Weighting Factor
1) Safety and comfort of use	10/10
2) Sustainable use of material	7/10
3) Cost of material purchase and installation	8/10
4) Maintenance requirement and frequency	5/10
5) Ease and speed of construction	3/10

As shown in Table 3, CCC feels that the safety of users is of utmost importance for our design. For this reason, the maximum weighting of 10 was applied to the safety criterion. Since the Site 10 bridge construction schedule is not on the critical path for completion of the Hat Lake Road construction schedule, we felt that the relative importance of the speed of construction was a considerably less important factor driving our design, so it was assigned a weighting factor of 3.

The three design options were then scored out of 10 in each of the evaluation criterion, and the score for each criterion was multiplied by the corresponding weighting factor. The summary of each of the three design option scores is summarized in Table 4.

Table 4: Evaluation of Site 10 crossing options (Arena, 2015)

Site 10 Option Evaluation Criteria	Score		
	Option A 25m span	Option B 30m span	Option C 20m span
1) Safety and comfort of use	8 * (10)	10 * (10)	5 * (10)
2) Sustainable use of material	6 * (7)	10 * (7)	9 * (7)
3) Cost of material purchase and installation	5 * (8)	9 * (8)	10 * (8)
4) Maintenance requirement and frequency	5 * (5)	10 * (5)	8 * (5)
5) Ease and speed of construction	3 * (3)	6 * (3)	10 * (3)
Total score	196	310	263

Based on the results from Table 4, CCC performed a detailed design of Option B as it scored the highest in our selection rubric. Option B is superior to Option A and C based on ease of construction, no fill issue, and comfortable truck turning path. Therefore, Option B is the option of choice. Since Option B is superior to both Option A and C, we did not further develop Option C beyond what is shown in Figure 33.

7.0 SITE 10 - SUPERSTRUCTURE DESIGN

The design of the 30.5 m bridge at the Site 10 stream crossing was done in two sections: superstructure and substructure. The superstructure design covers the bridge elements including decking, and girders, while the substructure design covers the approach embankment design and bridge abutment design.

A single-lane bridge was designed for this project. The bridge crossing design encompassed three alternative spans of 18.3 m, 24.4 m, and 30.5 m lengths which corresponded to the road alignment options. The sustainability and economics of each design was ranked and the best alternative was chosen for Hat Lake Road. CCC designed a typical bridge crossing for Site 10; the design acts as a template for the other forest road crossings needed.

We assessed the Hat Lake Road alignment at Site 10 and recommend an optimal route that includes a 30.5 m span bridge crossing. This section covers the bridge design, superstructure analysis, and resulting bridge structure.

7.1 Superstructure Design

Canfor requires a 75 year design life for Hat Lake Road. The *Engineering Manual* classifies bridges with more than 45 years of design life as permanent structures (Ministry of Forests, Lands and Natural Resource Operations, 2013). Permanent structures must have structural components of steel, concrete or treated wood. CCC's superstructure design comprises of steel girders with timber deck modules.

To determine the governing load combination for the superstructure, the design loads were calculated.

7.1.1 Design Loads

The superstructure design loads were divided into 5 categories: dead load, live load, snow load, wind load and ice accretion load. Seismic loading is not considered for forestry bridge crossings.

7.1.1.1 Dead Load

The dead load encompasses the timber decking components and the steel girders.

MFLNRO released standard drawings for forestry bridges. The standard drawings from set STD-EC-020 show the assembly of a typical timber deck module. All timber is treated to prolong the lifespan of the decking module. The decking layers are as follows:

- deck,
- sub-deck,
- timber guardrail,
- timber ties for the guardrails,
- stringer, and
- stiffening timbers.

Refer to Appendix U for a breakdown of dead loads for each decking component. The decking is prefabricated and comes in a module length of 3048 mm (10 ft). The total load of the timber deck was 6.5 kN per meter.

From CCC's experience in forestry roads, we estimated a preliminary girder depth of 1800 mm, which correlates to a dead load of 7.5 kN per meter. The *Handbook of Steel Construction* was referenced to obtain the girder weight (Canadian Institute of Steel Construction, 2010). We included 0.5 kN per meter of miscellaneous steel to be conservative. A total dead load of 8.0 kN per meters of structural steel was used for the bridge load analysis.

7.1.1.2 Live Load

Hat Lake Road is designed for L-100 trucks traversing its route. The load for a L-100 truck is 100 imperial tons or 890 kN. In addition, the truck load also accounts for the carrying capacity (typically logs), which is included in its distributed load of 25 kN per meter and point loads for the L-100's axels as shown in Figure 34.

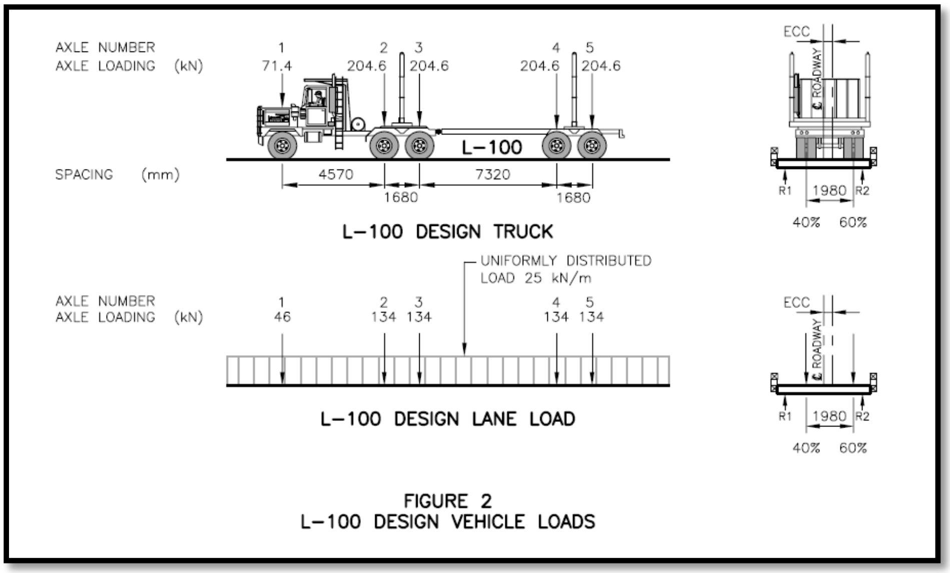

Figure 34: L-100 design vehicle loads (MFLNRO, n.d.)

MFLNRO requires the design of the bridge crossing to consider the truck load as per Figure 34. From reviewing the CSA S6-06 *Canadian Highway Bridge Design Code*, Figure 35 shows the axel loads for trucks classified as CL-W.

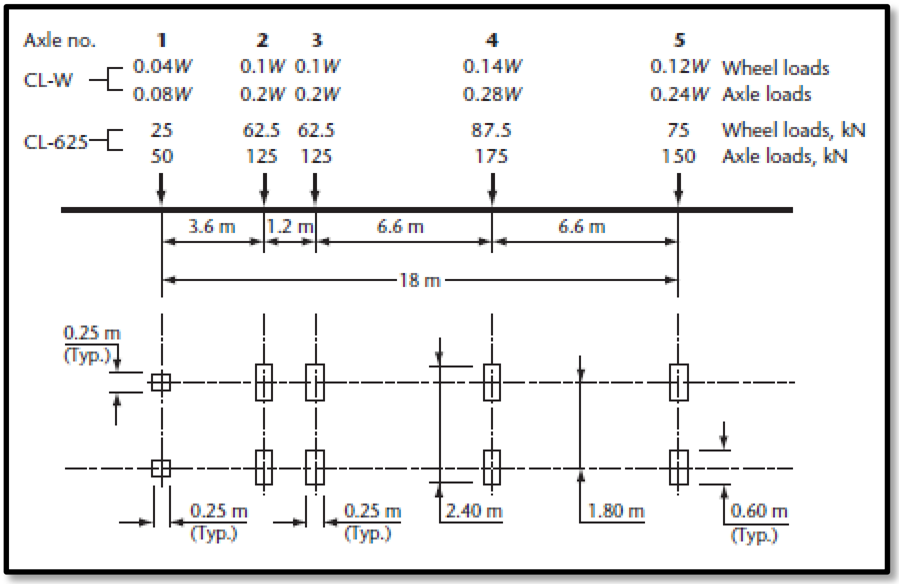

Figure 35: CL-W & CL-625 axel loads (Standards Council of Canada, 2013)

CSA S6-06 specified axel loads for 5 axel CL-W trucks are greater than those from the MFLNRO (Standards Council of Canada, 2013, p. 53). Therefore, CCC's design considers CSA CL-W axel loads shown in Figure 35 and the distributed load of 25 kN per meters required by the MFLNRO.

Since all road crossings for this project are a single-lane, only one L-100 truck is evaluated for in design calculations. However, MFLNRO and CSA S6-06 specify a dynamic load allowance to account for the "dynamic and vibratory effects of the interaction of the moving vehicle and the bridge, including the vehicle response to irregularity in the riding surface" (Standards Council of Canada, 2013, p. 41). The live load is exaggerated by 1.33, or 33 percent, to account for the dynamic load allowance. Note that CCC used a slightly higher dynamic load allowance than the 30 percent specified in CSA S6-06 to be conservative.

When assembling the crossing, construction live load on the bridge must also be considered. MFLNRO recommends a conservative construction load of 40 tons of equipment and an extra 10 tons of load, or 445 kN together, over a length of 4 meters.

The governing live load includes the L-100 with the dynamic load allowance. Live load calculations do not sum the live load, dynamic load allowance and construction live load because the construction of the bridge will not have L-100 trucks travel over the bridge until it is completed.

7.1.1.3 Snow Load & Wind Load

Table C-2 in Appendix C of the *British Columbia Building Code* (BCBC) provides the snow load for 50 year return period for various locations in BC. Fort St. James is not included in the table. As Fort St. James is located between Mackenzie and Prince George, CCC interpolated the data for a snow load of 9.5 kN per meter as per Part 4 of the BCBC. Refer to Appendix U for the snow load calculation.

The 10 year and 100 year return period wind load is also included for in Table C-2 of the BCBC. Again, data from Mackenzie and Prince George was interpolated to obtain the approximate values for Fort St. James. The 10 year hourly wind pressure is 0.27 kPa while the 100 year hourly wind pressure is 0.38 kPa (Standards Council of Canada, 2013, p. 74)

Furthermore, CSA S6-06 divides the wind load calculations into a horizontal drag load and a vertical load.

The horizontal drag load equation from CSA S6-06, Cl. 3.10.2.2 is

$$W_h = q * C_e * C_g * C_h$$

Where:

q = wind pressure for 1/100 years (for bridges)

C_e = wind exposure coefficient taken from CSA S6-06 Table 3.8

C_g = gust effect coefficient (taken as 2.5 for slender structural elements)

C_h = horizontal coefficient (taken as 2.0 or 1.2 for wind on live load)

The vertical wind load equation from CSA S6-06, Cl. 3.10.2.3 is

$$W_v = q * C_e * C_g * C_v$$

Where:

C_v = vertical coefficient (taken as 1.0)

Refer to Appendix U for wind load calculations.

7.1.1.4 Ice Accretion Load

Ice accretion is defined as the "buildup of an ice layer on the exposed surfaces of a body due to freezing rain or in-cloud icing" (Standards Council of Canada, 2013, p. 42). CSA S6-06 requires ultimate limit states load combination 7 include the evaluation of ice accretion load. Figure 36 shows different zones in Canada and the approximate thicknesses of ice buildup.

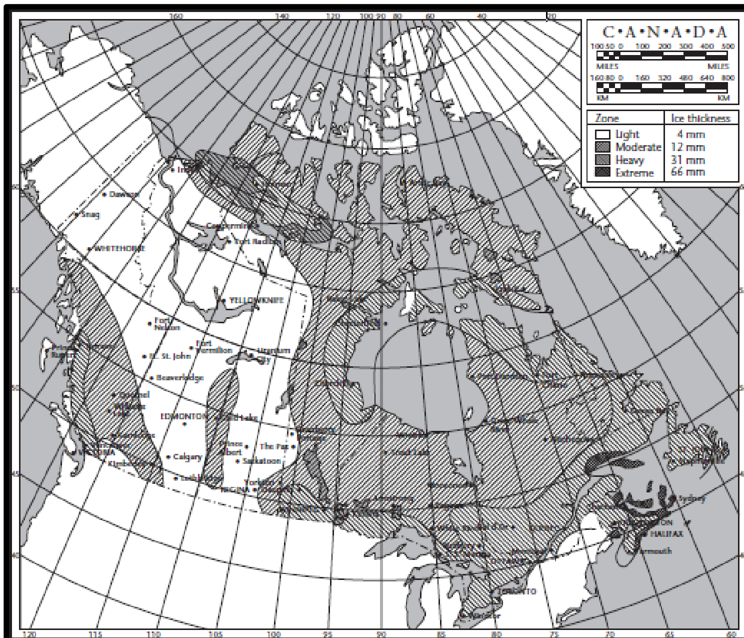

Figure A3.1.4
Ice accretion
(See Clause 3.12.6.2.)

Figure 36: Ice accretion (Standards Council of Canada, 2013, p. 90)

Based on Figure 36, Hat Lake Road falls in the moderate region which has an ice accumulation of 12 mm.

7.1.2 Design Parameters

MFLNRO standards state that forestry bridges less than or at 30.48 m in length has two girders spaced 3 meters apart, each 1.5 meters away from the centerline. For an L-100 truck, the standard bridge deck has a width of 4268 mm (14 ft). Ideally, each girder carries half the forces caused from the governing load combination. However, due to the way logs are piled onto logging trucks, sometimes more load is amassed on one side of the truck. MFLNRO recommends a 40 percent allocation on one side and 60 percent distribution on the other. The girder carrying 60 percent of the forces governs. CCC accounted for the unbalanced forces, in a simplification, by increasing the calculated factored shear and moment values by 10 percent. CCC analyzed the bridge as a simply-supported structure with two reaction forces at the end of each girder.

7.1.3 Changes to Scope

In CCC's proposal, the initial superstructure alternative compared steel girders to concrete girders. CCC would evaluate fabrication, transportation, and installation costs and also review of the amount of greenhouse gases emitted to maximize sustainability.

However, during CCC's literature research, we found that the *Forest Service Bridge & Design Manual* lists ideal girder materials for different span lengths (Ministry of Forestry Lands and Natural Resource Operations, 1999). A particular material type is associated with bridge spans and presented in Table 5.

Table 5: Economic lengths for bridge girders considering material properties (Ministry of Forestry Lands and Natural Resource Operations, 1999)

Bridge Span, L	**Material**
L < 12 m	Concrete (composite decking)
12 m < L < 18 m	Concrete (with non-composite decking)
18 m < L	Steel

From Clear Cut Consulting's road alignment options, the bridge length spanned 18.3 meters, 24.4 meters, or 30.5 meters. Referencing Table 5, CCC decided to alter the superstructure scope and eliminate the concrete girder alternative due to the crossing spans specified.

A cost comparison of the three options assessing the materials and installation for the bridge girders and timber deck is shown in Section 7.3.1.

7.2 Superstructure Analysis

The superstructure analysis comprised of the following to determine the bridge girder members:

1) ultimate limit states calculations to determine the governing load combination,
2) factored load calculations to find the maximum bridge reactions and moments,

3) a deflection check to select the girder member,
4) factored resistance calculations to confirm the selected girder member has the capacity to handle the factored load
5) steel I-girder calculations to validate flange strength
6) fatigue calculations to determine cyclic loading sufficiency, and
7) preliminary welding design required by the girder to check for adequate shear resistance.

There were three (3) options proposed for the bridge at Site 10. The three options differ by their bridge span lengths: Option A has a 24.4 m bridge span, Option B has a 30.5 m span, and Option C has a 18.3 m span. From the above list, CCC went through steps 1 to 4 above for all three options to determine a preliminary girder size. Upon completion of the road alignment, CCC compared the three alternatives in a rubric shown in report Section 6.4 and selected Option B as the best option for the project. Steps 5 to 7 were then calculated for the 30.5 m bridge.

7.2.1 ULS Load Combinations

The ultimate limit states (ULS) load combinations from the National Building Code of Canada (NBCC) and CSA S6-06 *Canadian Highway Bridge Design Code* were checked to determine the governing load case. NBCC considers five load combination cases while CSA S6-06 assesses nine cases. Refer to Appendix V for the ULS load combinations.

From ULS calculations for all options, the governing load combination is Case 1 from CSA S6-06:

$$1.1DL_{steel} + 1.2DL_{wood} + 1.7(LL + Dynamic\ Load\ Allowance)$$

Dead load for the bridge components is separated into prefabricated structural steel elements (DL_{steel}) and other dead load members (DL_{wood}). The live load (LL) comprises of one (1) L-100 truck which has been enhanced by 10 percent for unbalanced truck loading mentioned in Section 7.1.2 and 33 percent for the dynamic load allowance.

In the transverse direction, perpendicular to the bridge span, the governing load case is CSA S6-06 ULS combination 4. A factor of 1.5 applies to the wind load.

Some load factors specified by the ULS load combinations were not applicable to our project. Outside of the active seismic region, earthquake loading is not considered for forestry bridges. In addition, loads due to earth pressure, hydrostatic pressure, stream pressure, ice forces, debris torrents, and collision from highway vehicles or vessels were ignored. Furthermore, to simplify CCC's bridge analysis, effects due to strains, deformations, displacement, and secondary pre-stress effects and load due to differential settlement were disregarded.

7.2.2 Load Calculations

The superstructure was analyzed as a simply-supported structure. The dead load was taken as a uniformly distributed load longitudinally. To determine where the maximum moment occurred, CCC designed an excel spreadsheet that moved the L-100 truck in 500 mm increments across the span of the bridge. Figure 37 shows the position of the L-100 truck where it produces the maximum moment for Option A.

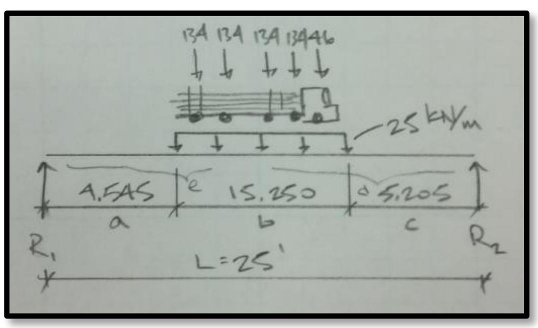

Figure 37: Option A L-100 truck location where max moment induced (Ip, 2015)

For all options, the maximum moment occurred slightly off center towards the end of the bridge. We assumed the maximum moment occurred at the center of the superstructure to be conservative. Refer to Appendix W for the excel spreadsheet; note that the spreadsheet only considers the L-100 load. The axel loads of the L-100 truck were taken as point loads and included in the spreadsheet. Moreover, the maximum shear occurred at the start of the bridge, at the support just as the truck passes over the column support. Again, the axel loads were taken as point loads referenced from the beginning of the bridge. Load calculations are found in Appendix X. CCC's calculations for Option B produced a moment of 13,120 kNm at the center of the bridge and a shear of 1,500 kN.

For the transverse direction, only wind load was considered. The dead load of the structure was neglected because each girder handles the load longitudinally as a simply-supported structure. The calculations in Appendix V show the applicable ULS load combinations. Horizontal wind load on the structure is treated as a uniformly distributed load and the live load, L-100 truck, was only considered as point loads at the axel locations. The resulting forces generated by the wind load for Option B has a moment of 1,820 kNm and a shear of 255 kN at the beginning support reaction.

To verify the loads calculated, CCC created a simple SAP 2000 model for option 2. Figure 38 shows the profile of the model.

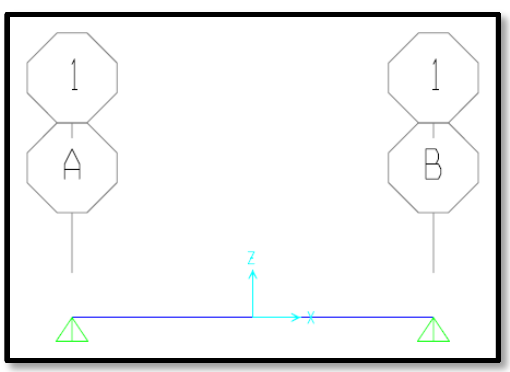

Figure 38: Option B SAP2000 model (Ip, 2015)

Gridline 1A is the beginning of the superstructure and gridline 1B is the end. The governing load cases were entered into SAP 2000 and run. Figure 39 shows the resulting deflection and moments in one girder.

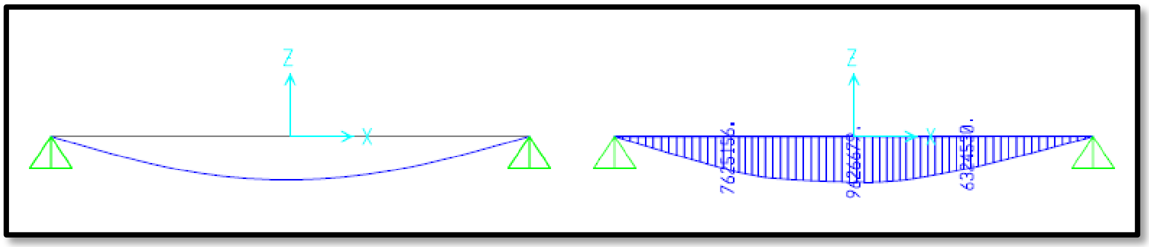

Figure 39: SAP2000 longitudinal model for Option B (Ip, 2015)

The SAP model in Figure 39 produces a maximum shear reaction of 1,030 kN and moment of 9,630 kNm. These values do not correspond to our calculated forces. A possible source of error includes user input. However, for the transverse wind load, the moment 1,820 kNm generated by SAP 2000 correlates with our calculations as shown in Figure 40.

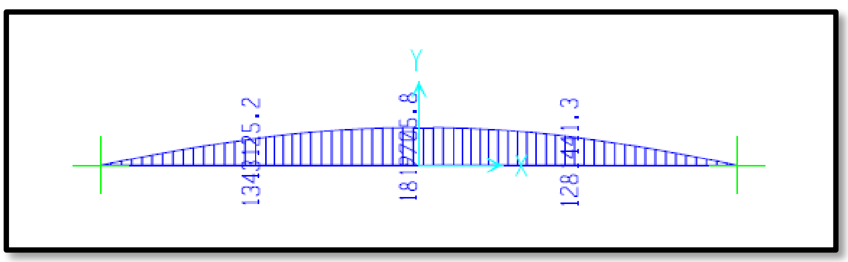

Figure 40: SAP 2000 transverse model for Option B (Ip, 2015)

Since the values from SAP 2000 were lower than those of CCC's calculations, we used our factored load values to be conservative.

7.2.3 Deflection

The deflection of girder members is limited by the MFLNRO. MFLNRO specifies a maximum live load deflection of:

$$\Delta = \frac{crossing\ span}{450}$$

The maximum deflection allowed for Option B was 67 mm. However, the deflection did not govern bridge member sizes. Further analysis showed that the section modulus governed girder size.

7.2.4 Girder Resistance Capacity

Appendix X shows the resistance capacity calculations for girder members for all options. The required moment of inertia was found from the deflection allowed. Using the WWF properties and dimensions tables in the *Handbook of Steel Construction*, a girder member was selected. WWF girders of Class 1 or 2 were preferential over those of Class 3.

Since the spans of Option A and C were below that of Option B, the required WWF girders were smaller than those for Option B. During the first try of selecting a girder for Option B, CCC chose a WWF 2000 girder which falls into bending Class 3. The elastic section moduli of the WWF 2000 girders noted in the Handbook of Steel Construction were still inadequate. In order to make one of the girders sufficient for our capacities, plates would need to be welded on the top and bottom flanges of the girder to increase the girder's moment resistance capacities.

Instead, CCC designed a welded plate girder to resist the factored loads calculated and provide sufficient resistance capacity. The designed girder's moment resistance is 21,930 kNm in the longitudinal direction and 3,080 kNm in the transverse direction while the shear resistance is 7,090 kN. These capacities surpass the required resistances.

7.2.5 Girder Flange Strength

Clause 10.13.6 from CSA S6-06 specifies steel I-girder flanges meet certain requirements (Standards Council of Canada, 2013, p. 478). There are three (3) requirements for the limits of applicability. The first requirement that the "absolute value of the ratio of the torsional warping normal stress to the normal flexural stress shall not exceed 0.5 at any point in the girder" was neglected as a simplification (Standards Council of Canada, 2013, p. 478). The next requirement that unbraced length not exceed 25 times the width of the flange is met with a diaphragm spacing of 7.62 m. The final requirement that flanges be Class 3 or better is met since our flange is a Class 1 section.

In addition, CSA S6-06 Clause 10.13.6.1.2 *Flanges* state that "flanges shall be proportioned to satisfy the strength of either flange and the stability of the compression flange" (Standards Council of Canada, 2013, p. 478). For the strength of a flange, we ignored the factored bending moment in the flange due to torsional warping as another simplification. Our flange dimension of 650 mm base and 45 mm thickness meets both requirements. The stability of the compression flange requirement governed the resulting size of the girder for option 2. Refer to Appendix X for Option B's calculations.

7.2.6 Fatigue Adequacy

The required cyclic loading for a forestry bridge over 12 meters is 500,000 cycles as per MFLNRO standards. The connection design for the bridge will comprise of plates and bolts to attach girder splices. Welding will not occur on site. The connection design yields a fatigue detail category B. Appendix X (for Option B) shows the fatigue resistance provided by the detail connection exceeds the fatigue generated by the cyclic loading; therefore, fatigue resistance is good.

Note that the CSA S6-06 fatigue limit state load combination was omitted from our design.

7.2.7 Preliminary Weld Design

Since CCC designed the girder used for Option B, we did a preliminary welding design for the girder. Referencing the *Handbook of Steel Construction*, 8 mm fillet welds are located at the interface between the top and bottom flanges and the web plate (Canadian Institute of Steel Construction, 2010).

The transverse shear force was calculated at 2,300 kN. From our preliminary calculations for Option B in Appendix X, our welding length of 15 m along the top and bottom flange to web interfaces on both sides of the girder result in more than enough shear resistance. The design engineer obtained to complete the connection design will provide the final weld and plate girder connection design.

7.3 Resulting Superstructure

The components of the superstructure comprises of a plate girder, timber deck, intermediate diaphragms, abutment diaphragms, and splice connections. This section covers the comparison of bridge span options for Site 10 and the resulting superstructure design for a 30.5 m bridge span.

7.3.1 Alternative Cost Comparison

CCC compared the material supply and installation cost for the different bridge spans.

Table 6 shows the total superstructure cost for the three options.

Table 6: Alternative options cost comparison (Ip, 2015)

DESCRIPTION		UNIT	QUANTITY	RATE	COST	SUBTOTAL
Option 1	Supply Bridge (24.4m)	ft	80	$ 3,125.00	$ 250,000.00	**$ 517,850.00**
	Girder coating	ea	1	$ 1,250.00	$ 1,250.00	
	Timber decking	ft	80	$ 1,200.00	$ 96,000.00	
	Bridge installation (Labour & Equipment)	ft	80	$ 2,120.00	$ 169,600.00	
	Inspection	ea	1	$ 1,000.00	$ 1,000.00	
Option 2	Supply Bridge (30.5m)	ft	100	$ 3,125.00	$ 312,500.00	**$ 645,772.00**
	Girder coating	ea	1	$ 1,250.00	$ 1,250.00	
	Timber decking	ft	100	$ 1,190.22	$ 119,022.00	
	Bridge installation (Labour & Equipment)	ft	100	$ 2,120.00	$ 212,000.00	
	Inspection	ea	1	$ 1,000.00	$ 1,000.00	
Option 3	Supply Bridge (18.3m)	ft	60	$ 3,125.00	$ 187,500.00	**$ 388,363.20**
	Girder coating	ea	1	$ 1,250.00	$ 1,250.00	
	Timber decking	ft	60	$ 1,190.22	$ 71,413.20	
	Bridge installation (Labour & Equipment)	ft	60	$ 2,120.00	$ 127,200.00	
	Inspection	ea	1	$ 1,000.00	$ 1,000.00	

As shown in Table 6, shorter bridge spans are associated with less cost. Due to our criteria for the Hat Lake Road project, CCC's finalized bridge design for 30.5 m has a superstructure cost of $646,000.

7.3.2 Option 2 Design

From our superstructure analysis, the resulting plate girder was designed by CCC. The dimensions of the top and bottom flange plates are 650 mm wide by 45 mm thick, and the web plate is 1710 mm by 25 mm. At the interface of the top and bottom flanges and the web plates, there is an 8 mm fillet weld on both sides. Figure 41 shows the top flange and web plate interface.

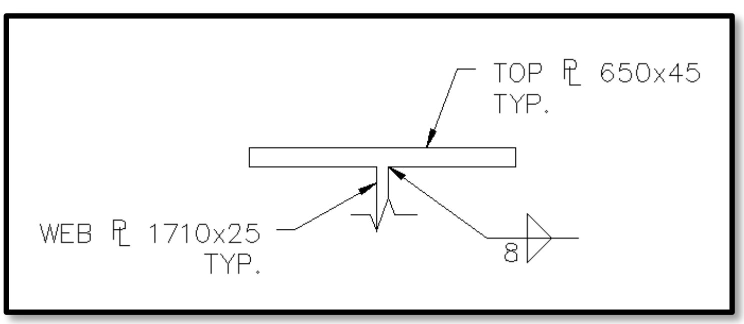

Figure 41: Upper portion of plate girder cross section (Ip, 2015)

The bottom flange and web plate interface is similar to the top flange and web plate interface. Final weld design will be completed by the connection detail engineer retained by CCC.

Other superstructure components include the bridge deck, diaphragms and splice connections. The timber deck and guardrail for the bridge follows MFLNRO standards as per STD-EC-020 set drawings. Abutment diaphragms will be placed at both ends of the superstructure and three (3) intermediate diaphragms will be placed along the span. MFLNRO standards use L127x127x8 members for the diaphragm. The bottom diaphragm member is doubled to enhance stiffness between the girders. Furthermore, to accommodate the transportation of the plate girders to Site 10, the girder span will be spliced into three members of 10.2 m. The connection detail engineer will provide splice connections using plates and bolt connections and check the diaphragm design meets code. Refer to CCC drawing 10-S-01 in Appendix O for the profile and cross-section of the superstructure.

8.0 SITE 10 - SUBSTRUCTURE DESIGN

The substructure of the Site 10 bridge crossing consists of structural elements designed to safely transmit forces from the bridge girders into the soil. To design the foundation structure of the bridge crossing, two structural systems were analyzed and compared; the best system that met CCC's design philosophy was selected. The two structural systems that were analyzed were

- a friction pile design, consisting of two capped piles on each abutment, and
- a stilted bearing pad design, consisting of two reinforced concrete spread footings beneath steel pile stilts on each abutment.

The design of structural foundations is dependent on the type and the properties of the soil that bears the weight of the structure. DWB was unable to provide CCC with a detailed soil investigation report, but we obtained a sieve analysis of a near-surface sample taken from a test pit near Site 10. Survey notes found in Appendix B and the sieve analysis in Appendix L identified the soil as sand with some fines, and included a particle size distribution. Since no other information on the soil was available, we assumed that the soil was a semi-infinite single layer of a cohesionless silty-sand. Using this assumption, the two foundation options were designed and compared, and the option that scored the highest on the selection criteria was selected.

8.1 Friction Pile Foundation Option

The first structural foundation option considered was composed of displacement friction piles beneath a cast-in-place reinforced concrete pile-cap with elastomeric bearing pads. To create a pinned-roller connection condition, the bearing pads on one of the pile caps will be left free to rotate and translate through elastic deformation of the elastomeric material, while the pads on the other pile cap will feature vertical through-pins to restrict horizontal translation but still allow for rotation. The bearing pads then transmit load to the concrete pile-cap. The pile-cap serves to distribute load to the friction piles as well as to create a moment-resisting frame to maintain perpendicularity between the plane of the deck and the piles. The friction piles then distribute the load into the earth by the skin friction at the pile/soil interface as well as the capped tip bearing on the soil. Figure 42 shows the forces experienced by a friction pile.

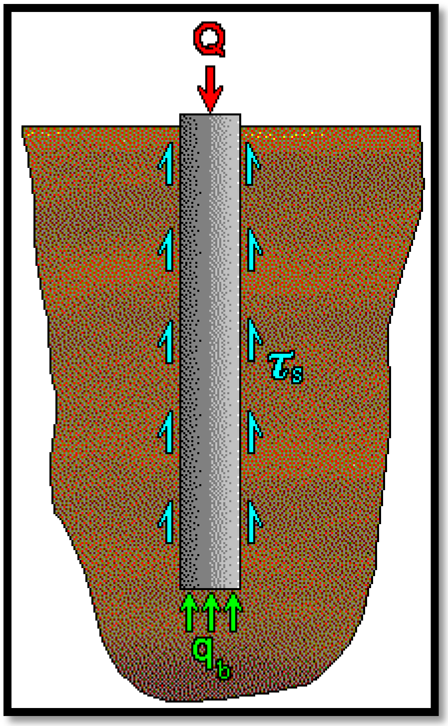

Figure 42: Forces on a friction pile (environment.uwe)

The MFLNRO *Engineering Manual* states that the design of pile foundations should be completed in accordance with the *Canadian Foundation Engineering Manual* (CFEM). Section 18.2.1 of the CFEM states that the Beta Method should be used for designing single friction piles (Canadian Geotechnical Society, 2006). The equation for determining the pile resistance is shown below:

$$R = \sum_{Z=0}^{L} (q_s \ \Delta Z \ C) + \ A_t \, q_t - \ w_p$$

Each of the parameters in the above equation is described and calculated in the hand calculations found in Appendix Y. The Engineering *Manual* suggests that a compression load resistance factor be applied to the pile resistance to reduce the calculated bearing capacity by 60 percent (Ministry of Forestry Lands and Natural Resource Operations, 1999).

CCC calculated that four 508 mm diameter capped-piles that extend 35 m deep would be sufficient to resist the maximum factored loads from the bridge girders. Since the pile foundations would be located near a stream that has been eroding the nearby soil for some time, it is likely that the depth of bedrock is less than the calculated pile depth. It is also likely that the soil does not consist of a 35 m layer of silty sand, and instead is comprised of several layers of soil with varying properties and strengths. Since the depth and type of bedrock and subsoil at Site 10 is unknown, the piles would have to be driven until sufficient resistance develops. This depth would likely be significantly smaller than the calculated 35 m.

The friction pile foundation structure option is resistant to stream bank washout failure as the piles extend deep into the earth; however, bringing pile driving equipment to the site would be difficult and expensive due to the rough terrain and remoteness of the site. Also, the amount of concrete necessary to form the pile caps would likely be larger than can be reasonably mixed by hand onsite. Thus, the concrete would need to be mixed in a temporary batch plant set up specifically for Hat Lake Road, which would result in considerable cost to the owner.

8.2 Bearing Pad Foundation Option

The other alternative system that was analyzed was a bearing pad system on pile stilts. Since the bridge supports are located close to the stream bank, Section 2.1.7 of the *Forest Service Bridge Design and Construction Manual* demands that the foundation structure must be resistant to embankment washout failure due to the erosion produced by the design flood event (Ministry of Forestry Lands and Natural Resource Operations, 1999). This requirement limits the use of shallow foundations, such as at-grade spread footings, as these foundation types are vulnerable to failure from soil erosion during flood events. The *Forest Service Bridge Design and Construction Manual* only limits foundations above the channel scour depth; by placing a reinforced concrete spread footing in a sufficiently deep excavation, connecting the spread footing to the bridge girders by steel pile-stilts and backfilling the excavation, a submerged spread footing configuration can still be used.

Industry standard drawings from the MFLNRO for this design details typical available sizes for elements based on the dimensions of the bridge (Ministry of Forests Lands and Natural Resource Operations, 2001). The arrangement of members for the stilted bearing pad design is shown in Figure 43.

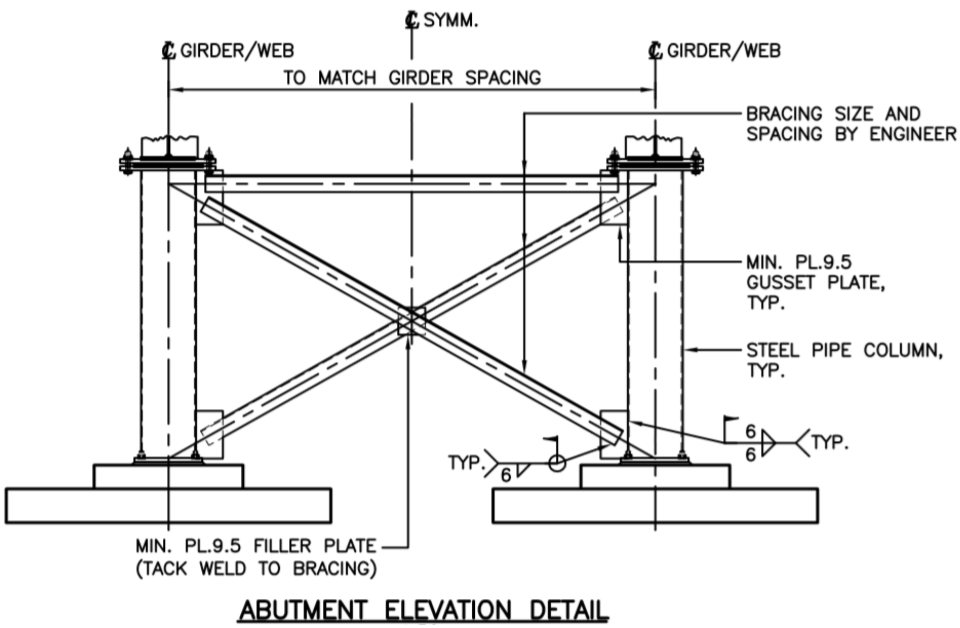

Figure 43: Bearing pad foundation with pile stilts (MFLNRO, n.d.)

This system is constructed from two square cast-in-place concrete bearing pads on each end of the bridge placed in excavated trenches below the elevation of the stream depth. A short steel pile stilt sits on each of these bearing pads and connects the pad to the underside of the bridge girder. The pads are set underneath the stream bed so that flood water is less able to scour and erode the soil underneath the bearing pads. Elastomeric bearing pads are bolted to the top of each pile stilt and the underside of the girders are seated on the bearing pads.

The MFLNRO *Forest Service Bridge Design and Construction Manual* states that the design of spread footing foundations should be completed in accordance with the *Canadian Foundation Engineering Manual* (CFEM). Section 10.2.1 of the CFEM states that the maximum bearing capacity of a spread footing can be calculated using the following formula:

$$q_u = \left(C\ N_c S_c + \ q_s\ N_q\ S_q + \frac{1}{2}\gamma\ B\ N_\gamma S_\gamma\right)\frac{1}{FS}$$

Each of the parameters in the above equation is described and calculated in the hand calculations found in Appendix Z. In order to find the minimum acceptable dimensions of the concrete bearing pad, a bearing capacity design was performed using the bearing capacity equation defined in Clause 10.2.1 of the CFEM using a safety factor of 2 (Canadian Geotechnical Society, 2006). Using this method, CCC calculated that the maximum allowable bearing capacity of the soil near Site 10 was 144 kPa at the surface, and 530 kPa when laterally supported by 1.5 m of well-compacted native backfill. We then determined that a reinforced concrete spread footing of dimension 1.68 m by 1.68 m would transmit the ultimate factored load to the ground at a bearing pressure less than the maximum allowable bearing pressure of the soil.

The 2.4 m by 2.4 m footing dimensions recommended by the MFLNRO standard drawings are substantially larger than the minimum dimensions calculated from the analysis. This is likely due to the large factor of safety assigned to the standard drawings so that they can be applied to projects without being individually designed for project specific conditions. Although the standard drawings would result in a more than satisfactory design, reducing the footing size results in a more easily constructed and more economical design.

This foundation option also requires the use of formed concrete, but in smaller quantities for the pile cap of the friction pile system. This volume of concrete is feasible to mix by hand onsite. It also uses a significantly smaller amount of steel, but requires excavation near the environmentally sensitive stream bank to reach a depth below the stream bed. Moreover, this system is prone to washout failure if a real flood event produces more erosion than is anticipated by the design flood.

These two foundation systems were then compared using CCC's evaluation criteria. Each option was judged out of a possible score of 10 on its cost effectiveness, environmental compatibility, reliance on uncertain soil conditions, and resistance to likely failure mechanisms. The scores of the options are summarized in Table 7.

Table 7: Evaluation of foundation structure options (Arena, 2015)

Foundation Option Evaluation Criteria	Score	
	Option 1 Friction piles	Option 2 Stilted bearing pad
1) cost effectiveness	4	10
2) environmental compatibility	6	8
3) reliance on uncertain soil conditions	2	7
4) resistance to washout failure	10	6
Total score	22	31

Since the bearing pad system scored higher on the evaluation criteria, it was chosen for use at the Site 10 crossing and a detailed design was prepared.

8.3 Design of Foundation Structure

After deciding to pursue a stilted bearing pad as the foundation structure for the Site 10 crossing, we completed a detailed design of the reinforced concrete elements. The completed design is similar to the standard S1 footing from the MFLNRO standard design drawings, but the total length of the slab portion of the footing was reduced from 2.4 m to 1.68 m and the height of the slab portion was increased from 200 mm to 250 mm. The reinforcement was also custom designed to suit our particular crossing structure. The design was completed using CSA A23.3 *Reinforced Concrete Design* and consisted of concrete shear resistance checks to determine the necessity of tie reinforcement and moment resistance calculations to determine the required flexural reinforcement.

CCC's design of the spread footings mimicked the design intent of the MFLNRO standard drawings; hence, they included a raised pedestal close to the column connection to aid in shear resistance within the critical section. Both one-way and two-way shear resistance was checked at the critical section within the pedestal and at the pedestal-slab interface. In both cases, no tie rebar was required because the concrete provided sufficient shear strength to resist the shear forces. Though it was not required for shear resistance, confining tie reinforcement was included in the pedestal to resist cracking due to thermal loading.

To find the amount of flexural reinforcement required to resist bending moment, a section of the footing at the column face and a section at the pedestal-slab interface was analyzed. Nine (9) 15M bars were used each way to resist the governing moment. Since the concrete is exposed to earth on all sides, a concrete cover of at least 75 mm was used for all faces of the footing. Four (4) hooked anchor bolts are to be embedded in the pedestal to allow for a bolted connection to the pile stilt with welded plate.

9.0 CULVERT DESIGN

Site 7a was selected to be the location of a culvert crossing during the preliminary design stage. It was chosen because Site 7a was the second largest crossing of the Hat Lake Road project. The results of our culvert design show that we are nearing the maximum size of culvert that would be desirable in a forest road setting.

9.1 Culvert Sizing

CCC used the average of two methods to size the culvert. The first method is the FREG method that has been discussed in detail in Section 4.2.3 of this report. The FREG method uses the stream characteristics to estimate the culvert area required for a 100-year flow. The FREG calculations are shown in Appendix F.

The second method used took the peak from the isoline method (found in Section 4.2.1 of the report) and determined the size of the culvert required. Hydraflow, a C3D extension was used to design the culvert. The results of the Hydraflow analysis are shown in Appendix AA. CCC designed for outlet control to ensure that the head water does not reach a level near the top of the culvert. Headwater control culverts also have the advantage of being a subcritical flow with a low velocity.
The results of the 2 methods are shown below in Table 8.

Table 8: Culvert diameter required (Milne, 2015)

Method	Diameter (m)
FREG	3.25
Hydraflow	2.286
Average	2.8

Table 8 shows the diameters calculated by the FREG method and by using *Hydraflow*. These diameters do not account for the 40 percent of the culvert diameter that is to be embedded into the stream bed (referred to in Section 9.2).

The required diameter of the culvert is 3.6 m to account for the embedment depth. CCC assumed that a larger culvert embedded into the stream will not significantly affect the culvert hydraulics. A culvert with a diameter of 3.6 m and 40 percent embedment represents a slightly larger cross sectional area than a 2.8 m diameter culvert to account for any additional steam material that may build-up. A corrugated steel culvert with dimensions 152x51x3 mm by 3.6 m shall be used for Site 7a or an approved alternative.

9.2 Culvert Installation

The culvert is to be installed so that it follows the natural gradient of the stream. The stream bed will be excavated to allow for the embedment depth.

9.2.1 Culvert Embedment Depth

The culvert should be installed with an embedment depth of 40 percent of the culvert diameter (Ministry of Environment, 2012). The embedment allows for the stream to retain its natural streambed profile which encourages fish passage. Figure 44 displays the typical embedment depth for a culvert in BC.

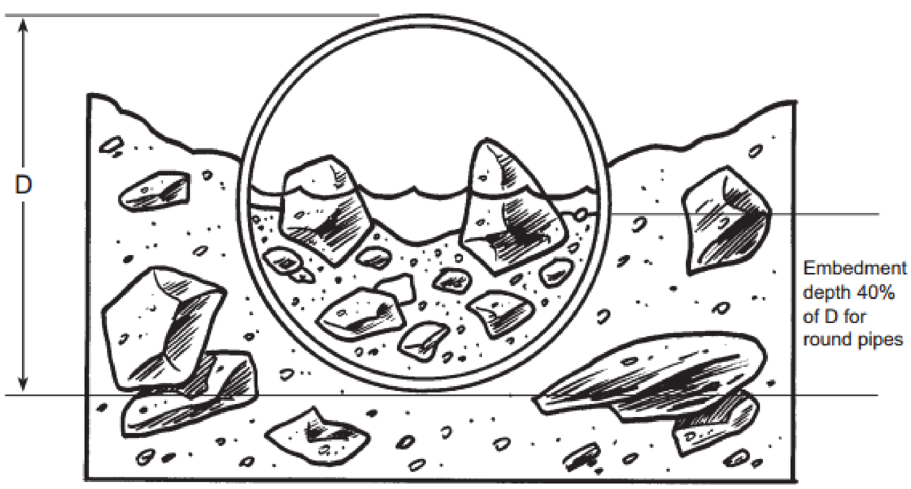

Figure 44: Culvert embedment depth (Ministry of Environment, 2012)

Figure 44 shows how culverts are to be installed to facilitate fish passage. CCC recommends an embedment depth of 20 percent of the culvert diameter. The reason for our recommendation is because during times of low flow, the depth of flow will be lower with an increased embedment depth. We believe that the 40 percent recommendation is tailed towards smaller diameter culverts, where it will have less of an effect during low flow times. The other drawback to having a large embedment depth is that the stream bed will have to be further excavated to facilitate the large embedment.

9.2.2 Depth of Cover

In order to satisfy the strength requirements, the corrugated steel pipe must be covered by a minimum of 1000 mm of specified fill material for a152x51x3 mm by 3.6 m diameter culvert. The maximum fill allowed overtop is 14 m which is not a concern for our current layout. Table HC-7 from the *Handbook of Steel Drainage & Highway Construction* was used to determine the minimum and maximum amounts of cover. The minimum depth of cover is based on E-80 railway loading requirements which have a higher strength requirement than the L-100 loading that is specified by the MFLNRO. Table HC-7 is show in Appendix BB.

The fill used must be well-graded gravel or sandy gravel with a compaction of 90 percent Standard Proctor Density (Corrugated Steel Pipe Institute, 2007)

9.3 Fish Passage

The culvert has been designed to retain the natural flow of the stream, limiting the effects on the passage of fishes. CCC determined that no baffles are needed to slow down the flow of water because of the low velocity and limited gradient.

The inlet and outlet of the culvert will be submerged in the streambed to allow fish to easily navigate through the culvert. The culvert will be embedded into the streambed to imitate the natural bed to allow for increased fish passage.

10.0 ENVIRONMENTAL ANALYSIS

CCC performed an environmental assessment of the crossings at Site 7a and Site 10 to minimize the construction impact on the environment and aquatic species. Sensitive species and other aquatic life typically live in the regions close to streams because of the abundance of food and water. All relevant design and construction regulations will be followed for implementation of crossings at Site 7a and Site 10.

10.1 Relevant Regulations

CCC strives to produce forestry roads that exceed the environmental regulations and guidelines applicable to construction in a forestry corridor. The design and construction of a forestry road in BC is required to follow provincial and federal regulations. The provincial statutes that apply to the Hat Lake Road include

- *Forest and Range Practices Act*
- *Water Act.*

The federal statutes that apply the Hat Lake Road include

- *Fisheries Act*
- *Species at Risk Act.*

The Fish-stream Crossing Guidebook by the BC Ministry of Environment lays out the regulations that must be followed depending on the type of project being constructed.

10.1.1 Forest and Range Practices Act

The Forest and Range Practices Act is the primary statute that regulates the construction, maintenance, and deactivation of stream crossings on crown land (Ministry of Environment, 2012). This statute does not require direct approval from any regulating body, but puts forth regulations that must be followed.

10.1.2 Water Act

The *Water Act* regulates changes in the stream and sets provisions to protect the water quality (Ministry of Environment, 2012). A notification of the work to be completed is required to be submitted to the MFLNRO to determine if an approval is necessary or not. Approval by the MFLNRO is, typically, necessary when a diversion of the stream is required for installing a new crossing. CCC does not plan to divert the stream; therefore, we plan on notifying the MFLNRO and do not expect to need approval.

10.1.3 Fisheries Act

The *Fisheries Act* covers the protection of fish, fish habitat and provides fish passage requirements at stream crossings. The *Fisheries Act* specifies activities that are not allowed unless authorized by the Minister of Fisheries and Oceans. Paragraphs 20, 35, and 36 represent the relevant sections from the *Fisheries Act* (Ministry of Environment, 2012):

> **20 (1), (2)** *Requires the maintenance of fish passage and sufficient flows for fish.*
>
> **35 (1)** *No person shall carry on any work, undertaking or activity that results in serious harm to fish that are part of a commercial, recreational or Aboriginal fishery, or to fish that support such a fishery.*
>
> **36 (3)** *No person shall deposit or permit the deposit of a deleterious substance of any type in water frequented by fish or in any place under any condition where the deleterious substance or any other deleterious substance that results from the deposit of the deleterious substance may enter any such water.*

The 3 paragraphs above do not have to be followed if written permission is obtained from the Minister of Fisheries and Oceans.

10.1.4 Species at Risk Act

The *Species at Risk Act* covers wildlife species from becoming endangered or threatened. The Minister of the Department of Fisheries is responsible for any aquatic life while the Minister of Environment Canada is responsible non-aquatic species.

The first step to determine if this statute is applicable is to conduct an ecological study of the area to find out if there are any species at risk located within the construction area of the project. If the study determines that there are species at risk located onsite, an assessment has to be submitted to the minister that includes the adverse effects of the project construction and the measures used to minimize those effects (Ministry of Environment, 2012).

10.2 Review Process

The Fish-stream Crossing Guidebook lays out a required review process for constructing new projects in the vicinity of a fish bearing water course. The following steps outline the general procedure that should be followed for determining necessary approvals.

Step 1*: *Determine Habitat Type

Determining the type of habitat for the site requires an ecological study to find out if there are any species at risk. Table 9 lays out the general procedure for determining what type of habitat is present.

Table 9: Definition of fish habitat (Ministry of Environment, 2012)

	Habitat at crossing site		
	Critical	**Important**	**Marginal**
Definition	Habitat that is critical in sustaining a subsistence, commercial, or recreational fishery, or any species at risk (i.e., terrestrial or aquatic red- and blue-listed species, those designated by the Committee on the Status of Endangered Wildlife in Canada, or those *SARA*-listed species), or because of its relative rareness, productivity, and (or) sensitivity.[a]	Habitat that is used by fish for feeding, growth, and migration but is not deemed to be critical. This category of habitat usually contains a large amount of similar habitat that is readily available to the stock.	Habitat that has low productive capacity and contributes marginally to fish production.
Indicators[b]	The presence of high-value spawning or rearing habitat (e.g., locations with an abundance of suitably sized spawning gravels, deep pools, undercut banks, or stable debris, which are critical to the population present), or the presence of any *SARA*-listed species, its residence, or critical habitat.[c]	Important migration corridors. The presence of suitable spawning habitat. Habitat with moderate rearing potential for the fish species present.	The absence of suitable spawning habitat, and habitat with low rearing potential (e.g., locations with a distinct absence of deep pools, undercut banks, or stable debris, and with little or no suitably sized spawning gravels for the fish species present).

Table 9 outlines the types of habitat based on common indicators. A thorough site investigation should be conducted to determine what type of habitat Site 7a and Site 10 fall under.

Step 2*: *Determine Necessary Approvals

The *Water Act* requires a notification to be submitted before construction begins at Site 7a and 10. The notifications must be submitted at least 45 days prior to the start of excavation near the stream. The notifications are subject to review by a habitat officer who will reply within 45 days if any changes need to be made to the construction plan. If no reply is received after 45 days from the habitat officer, the work may begin. (Ministry of Forests, Lands and Natural Resource Operations, 2015).

The *Fisheries Act* may require approval depending on the planned construction and how the construction may negatively affect the stream. The *Fish-stream*

Crossing Guidebook contains a simplified chart that determines if specific approvals are needed or not. Figure 45 determines whether approvals are needed.

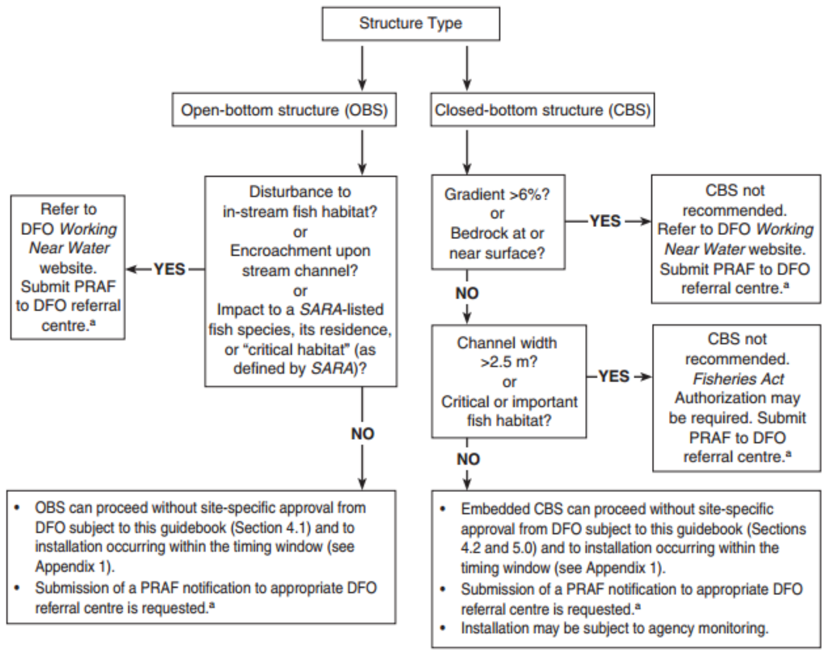

Figure 45: Department of Fisheries and Oceans review process (Ministry of Environment, 2012)

Figure 45 aids in the determination of whether an approval is needed from the Department of Fisheries and Oceans (DFO).

Site 10 may or may not proceed without approval depending on whether the ecological study finds species at risk. The construction of the bridge will have very little impact on the stream and channel. If no approval is required, DFO still requires a notification of at least 10 days before the work begins (Ministry of Environment, 2012).

Site 7a does require authorization from the Department of Fisheries and Oceans because the stream is wider than 2.5 m. The crossing at Site 7a has a stream width of 3.5 m. Applications must be submitted on the DFO website: *Working Near Water in BC and the Yukon.*

10.3 Instream Works

Performing construction related activities within the stream is a major environmental concern. The DFO defines in-stream work as any work taking place within the high water level of the stream (Department of Fisheries and Oceans, 1993). The construction at Site 7a and Site 10 will not require any de-watering which would require further environmental measures to be taken.

10.3.1 In-stream Construction Window

It is important to conduct the in-stream work when it will have as little impact on the aquatic species as possible. Typically, the summer months have the least impact on fish species because there is no reproduction going on during that time. Figure 46 below divides BC into 9 regions that illustrate when the fish are least sensitive to changes in their habitat.

Figure 46: Fisheries sensitive zone areas of BC (Department of Fisheries and Oceans, 1993)

Hat Lake Road is located within Zone 4 as shown in Figure 46. The fish located in Zone 4 are least sensitive to changes in the stream from mid-June to late July. Appendix CCC shows the chart of all the possible fish species in Zone 4 and at what times they are least sensitive to changes in the stream.

10.4 Sediment Control

CCC developed a Sediment Control Plan to minimize the amount of sediment that can be delivered into the stream at Sites 7a and 10. Reducing the amount of silts that enter a water course is important because silts have negative impacts on aquatic species and their habitat.

The amount of sediment that enters a stream can largely be reduced by leaving riparian vegetation on the stream banks. Natural vegetation acts as a filter for sedimentation, so that it does not drain into the stream.

It may be necessary to halt construction during times of heavy rainfall. The excavation of material during heavy rainfall events can cause excessive runoff and sedimentation that the barriers in place may not be able to handle. CCC recommends that no earthwork construction should proceed if heavy rain is expected to continue throughout the day. It also may be appropriate to adopt the use of watering trucks to wet the roads during the summer months to keep the dust down.

10.4.1 Silt Fencing

Silt fence is to be installed around the perimeter of the construction zone before excavation of material begins. The silt fence is required to be left in place for a period of 1 year after completion of the construction. The *Fish-Stream Crossing Guidebook* states that most sedimentation occurs in the first year because of the lack of vegetation and minimal compaction of the soil. Figure 47 shows the typical layout of silt fencing in the vicinity of a water course.

Figure 47: Typical silt fencing (Erosion and Sediment Control, 2015)

Figure 47 shows how silt fence is used to reduce the amount of sediment that can enter a stream.

Silt fencing is constructed of wood or steel posts that are used to support a permeable geotextile fabric (Ministry of Environment, 2012). The fences are used to retain sediment as runoff flows towards the fence.

10.4.2 Riprap

To protect the road embankment from erosion and washout during high flows, CCC designed riprap protection for Site 10 and Site 7a. MFLNRO *Engineering Manual* defines riprap as a filter that resists movement of underling soil. The manual briefly covers general construction and guides placement of riprap but lacks a design guide. Similarly, the Ministry of Transportation and Infrastructure (MoTI) riprap guide only outlines rock gradation for different riprap classes. Hence, CCC used *Riprap Design and Construction Guide*, MFLNRO *Engineering Manual*, and MoTI riprap guide to complete our design. According to *Riprap Design and Construction Guide*, the effectiveness of riprap depends on stone size, shape, weight, durability, gradation and thickness.

10.4.2.1 Riprap Dimensions and Shapes

Rock size is determined from the *Riprap Design and Construction Guide*'s design chart that relates mean stream velocity against the stone to the equivalent mean diameter of the stone. Figure 48 plots the required rock sizes for a range of velocities and depths.

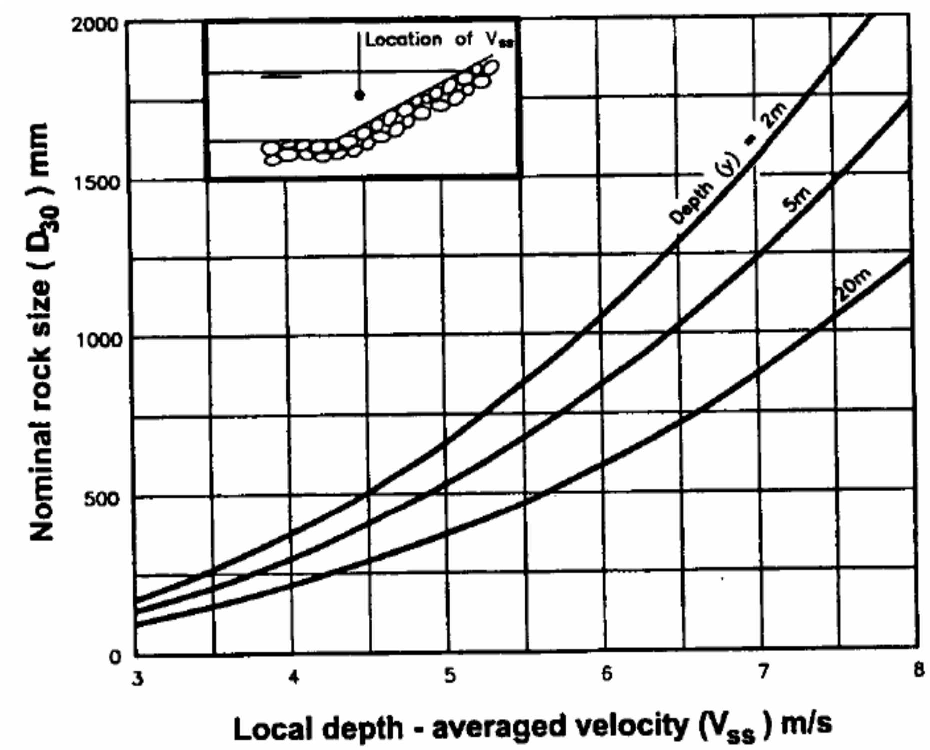

Figure 48: Required rock sizes for a range of velocities and depths (Ministry of Tranportationn and Highways, 1999)

The minimum average velocity available in Figure 48 is 3 m/s which is well above our design velocity of 1.3 m/s. Using a stream depth of 2 m and a design velocity of 3 m/s, the required nominal rock size is 300 mm. However, the *Engineering Manual* specifies the minimum factor of safety against global stability and sliding to be 1.5 (Ministry of Forests, Lands and Natural Resource Operations, 2013). Taking into consideration that the graphical approach used in Figure 48 uses a factor of safety of 1.2, CCC decided to use a nominal rock size of 375 mm for Site 7a and Site 10. Assuming a specific weight of 2.65 for riprap, the nominal rock size was converted to a stone mass using Figure 49 shown below.

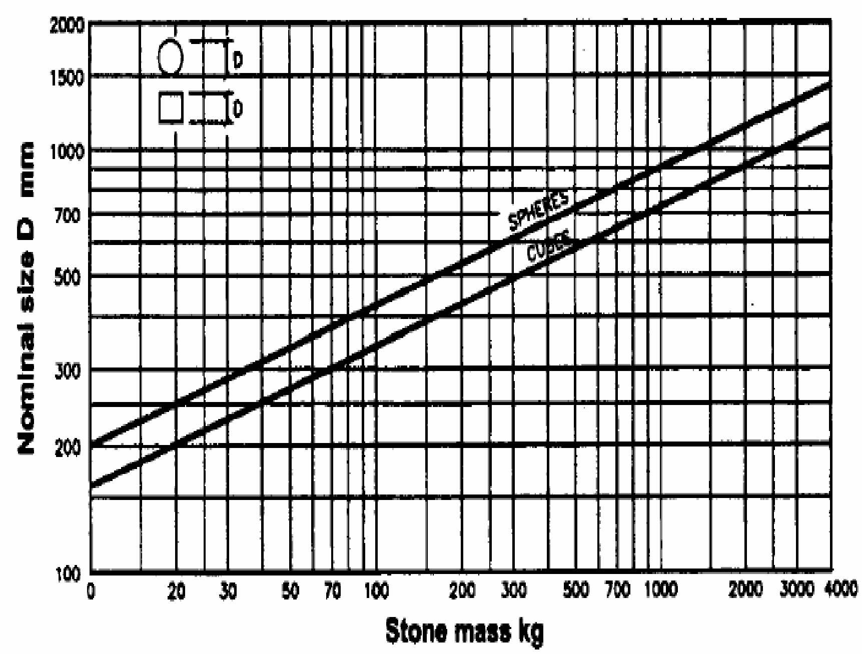

Figure 49: Riprap size and weight for a specific weight of 2.65 (Ministry of Tranportationn and Highways, 1999)

For spherical stones, the stone mass corresponding to a nominal size of 375 mm is 50 kg. For cubic stones, the stone mass corresponding to a nominal size of 375 mm is 125 kg. Typically, riprap are not spherical and its shape lies between a sphere and a cube. Thus, an average value of 100 kg is used for our design.

The following are recommended specifications for rock shapes as per the *Riprap Design and Construction Guide:*

- Less than 30% of the stones with a/c > 2.5
- Less than 15% of the stones with a/c > 3
- No stones with a/c >3.5.

The a/c ration refers to the average ratio of the long axis and thickness. Adhering to the guide’s recommendations, the riprap should be blocky and angular or sub-angular, with sharp clean edges and relatively flat faces.

10.4.2.2 Rock Gradations and Thickness

For maximum safety and effectiveness, the riprap must be of right thickness and have a well-graded matrix as specified in Table 10.

Table 10: Gradation of rock sizes in each class of riprap (MoTi, 2013)

Class of Riprap (kg)	Nominal Riprap Thickness (mm)	Rock Gradation Percentage Smaller Than Given Rock Mass (kg)		
		15%	50%	85%
10	350	1	10	30
25	450	2.5	25	75
50	550	5	50	150
100	700	10	100	300
250	1000	25	250	750
500	1200	50	500	1500
1000	1500	100	1000	3000
2000	2000	200	2000	6000
4000	2500	400	4000	12000

Class 100 kg riprap requires a 700 mm horizontal riprap thickness. Also, 15 percent of the matrix must be smaller than the 10 kg class, 50 percent less than 100 kg, 85 percent less than the 300 kg class. The complete riprap dimension guidelines can be found in Appendix DD.

10.4.2.3 Geotextile filter

During a 100-year storm event, the water elevation will rise up to 809 m and may saturate the road embankment. Since the proposed site is composed mostly of silty sand, the stream flow may washout some of the embankment fills. The riprap matrix serves as a filter that stops the underlying soils from moving as whole but does not stop the migration of surface soils.

To prevent fine sand particles from migrating, geotextile layer is proposed at following locations:

- At the interface of the riprap and road embankment
- Underneath the concrete footing’s gravel layer
- Behind timber ballast walls.

Propex Global’s Geotex series are commonly used geotextile layers in industry and conforms to AASHTO M-288 Standards. For subsurface drainage application, where geotextile is used for retaining the in-situ soil, Propex recommends Geotex 401, 601, or 801.

Table 11: Geotex class rating system (Propex Global, n.d.)

Application	Description	Class[1]		Nonwoven	Woven
Subsurface Drainage	Placing a geotextile against a soil to allow for long-term passage of water into a subsurface drain system retaining the in-situ soil.	Class 1	≤15% *in Situ* Soil passing 0.075 mm	Geotex 801	Geotex 2x2H F
			15 to 50% *in Situ* Soil passing 0.075 mm	Geotex 801	-
			>50% *in Situ* Soil passing 0.075 mm	Geotex 801	-
		Class 2	≤15% *in Situ* Soil passing 0.075 mm	Geotex 601	-
			15 to 50% *in Situ* Soil passing 0.075 mm	Geotex 601	Geotex 104F
			>50% *in Situ* Soil passing 0.075 mm	Geotex 601	Geotex 104F
		Class 3	≤15% *in Situ* Soil passing 0.075 mm	Geotex 401	-
			15 to 50% *in Situ* Soil passing 0.075 mm	Geotex 401	-
			>50% *in Situ* Soil passing 0.075 mm	Geotex 401	-

Note that class number refers to installation conditions and Class 1 is specified for harsh installation conditions with a potential for geotextile damage. We selected Class 1 filters to be used since riprap is heavy and is placed on top of the filter which may cause damage to the filter.

10.4.2.4 Toe protection

The *Riprap Design and Construction Guide* reveals that toe scour along the foot of the revetment is the most common cause of failure. Because our riprap structure is not directly within the scour, toe protection is not required. However, the cost of constructing a toe key is insignificant and provides added factor of safety. Therefore, we included a riprap toe key in our design.

10.4.3 Re-Vegetation

Replanting the lost vegetation onsite is one of the most effective ways at reducing erosion concerns at the site. Re-vegetation will also increase the soil stabilization of embankments. Typically, seeds and mulch are used to start the re-vegetation process. The mulch is used to increase the growth rate and to decrease the amount of seeds that are lost to rainfall runoff. It is important to select seeds that will thrive in the Hat Lake Road environment and will not be eaten by the wildlife in the area (Ministry of Environment, 2012).

10.5 Construction Best Practices

There are some simple steps that can be taken to minimize the construction effects on the environment. The following measures should be taken whenever possible (Department of Fisheries and Oceans, 1993):

- fall trees away from stream crossings,
- lift fallen trees rather than pulling them out when in sensitive areas,
- remove all debris that enters the stream,
- minimize vehicle idling,
- avoid working on rainy days,
- minimize duration of in-stream work, and
- use biodegradable hydraulic fluids for machines.

CCC recommends that, at a minimum, all contractors onsite follow these best practice measures.

11.0 SUSTAINABILITY CONSIDERATIONS

CCC values the importance of sustainable solutions to allow future generations to continue to live in a healthy environment, free from excess pollution. The purpose of the Hat Lake Road project is to increase the accessibility of forested land to the north of Fort St. James. The surrounding areas will be logged in the future to obtain the timber. Timber represents a sustainable material that can be used for constructing various structures.

CCC broke down the sustainability considerations into two categories: socio-economic benefits and sustainable logging practices.

11.1 Socio-Economic Benefits

The construction of Hat Lake Road has numerous direct and indirect advantages to the local population as well to the general population. The road and surrounding logging will employ local workers which will stimulate the local economy.

11.1.1 Local Employment

The forests sector in the Fort St. James region is the primary type of employment. The construction of Hat Lake Road will require a significant amount of labourers and other administrative related staff for the duration of the project.
The completion of the Hat Lake Road project will provide access to forested lots which will be logged by Canfor. The new road will allow access to over 3,000 hectares of forested land. This area will represent a significant employment zone for the area of Fort St. James.

11.1.2 Land Use

The expansion of logging roads in the backcountry gives access for people to enjoy the area for recreational activities. Mountain biking, hiking, and other various outdoor activities are prevalent in previously logged areas. These activities promote the community to others by increasing the activities that are available in the area.

11.1.3 Renewable Resources

Timber is one of the only renewable resources available for construction around the world. Recent trends show that local governments and industry experts push to use timber construction whenever possible. This is further emphasized by the fact that the BC Government has paid to start a Masters of Engineering in Integrated Wood Design at the University of Northern BC (UNBC). This shows the BC Government's commitment to incorporating more timber building throughout BC.

Technological advances further developed the use of timber for larger structures. The combination of engineered wood products and high strength connections has aided the adoption of timber for higher strength applications used in high rises and bridges. Figure 50 illustrates a 6-storey wood structure in Prince George, BC.

Figure 50: 6-Storey wood building (Cape, 2015)

Figure 50 shows the Wood Innovation and Design Center at UNBC. The Center represents the practical design of wood buildings and is the future of construction in BC.

With the population of the world surpassing 7 billion people, it is important to use materials that are sustainable and have a limited impact on the environment. Timber has the ability to be regrown within a century unlike any other building material earth has to offer.

11.2 Sustainable Logging Practices

The process of harvesting timber can only be sustainable if the wooded areas produce a steady growth rate that meets or exceeds the harvested amount. If forested areas are cut down faster than they can regrow, then the practice cannot be sustainable.

11.2.1 Re-forestation

CCC realizes that the replanting of trees after logging is one of the most important aspects of keeping the harvesting of trees a sustainable practice. It is out of CCC's control, but we strongly recommend to Canfor that all logged sites are replanted as soon as possible.

In 2012, a total area of 594 thousand hectares in Canada was logged and a total of 347 thousand hectares were replanted (Natural Resources Canada, 2012). Canada is on the right track to replanting logged areas, but still needs to improve to a 100 percent reforestation rate. For comparison purposes, the area of forest lost from forest fires in 2013 is over 7 times greater than the area lost from logging in Canada (Natural Resources Canada, 2012). This emphasizes that logging is not the primary cause of deforestation in Canada.

11.2.2 Low Impact Logging

Low impact logging is one solution to minimize the impacts on the environment and increase the sustainability of using the timber products. Low impact logging is a set of best practices that is used to cause the least impact on the ecology of the forest.

Creating a written forest management plan that outlines all the skid roads in an area before it is logged is the first step in minimizing the impact (Pearson, 2015). Designated skid roads will reduce the compaction of the site to the smallest area possible; thus having the least disturbance on the soil. The soil disturbance can also be minimized by constructing roads to the minimum width possible for machine access.

12.0 CONSTRUCTION STEPS

As part of the design of Hat Lake Road and its crossing structures, CCC prepared a cost estimate covering material and labour required to implement our design, as well as a tentative construction schedule. The schedule is mainly focused on the road construction and the Site 10 crossing. The cost breakdown of the project is found in Appendix M and the construction phasing plan is found in Appendix EE. The construction work for the entire Hat Lake Road project is expected to take 78 working days between April 6, 2015 and July 22, 2015, and is expected to cost $3.66 million.

Construction steps for the forest road are outlined below:

1. Beginning at the Leo Creek Road tie-in, a surveyor and arborist team plots the road centerline at 20 m intervals. A clearing width of 9 m on either side of the centerline is roped off and mercantable trees of a diameter greater than 6 inches are marked with fluorescent paint.

2. A logging crew directionally fells and de-limbs all trees in the clearing width by chainsaw, and bucks marked logs to appropriate stocking lengths. Stumps should be left at least 2 feet high to facilitate removal by bulldozer.

3. A tracked bulldozer with a multi-shank ripper attachment clears organic topsoil from the 8 m road width into the remainder of the clearing zone. An excavator with a claw attachment stacks trees outside of the road width and the trees are removed and sold later.

4. The excavator with a bucket attachment and the survey team begin the cut and fill process in areas where the difference between the design road elevation and existing ground elevation cannot be matched by grading.

5. A rubber-tire grader matches native material elevation to the design elevation of the road sub-base, and applies the design cross-fall to the road section for drainage purposes.

6. A steel-drum/rubber-tire combo roller compacts the road sub-base. A soil tester with a nuclear densometer ensures that the road is compacted to 95 percent modified proctor to meet the design specs of the road structure. If required, a water truck will apply water to the road surface so that the optimum water content of the soil is reached to aid in compaction.

7. When construction vehicles are obstructed by streams at the four crossing locations, a crawler crane and construction crew will set up temporary crossing structures of sufficient strength that the largest construction vehicle is able to cross.

Once the road construction has proceeded to a point where material can be trucked in to the Site 10 crossing, construction of the bridge structure will begin. Construction steps for the bridge structure are outlined below:

1. The crawler crane and construction crew set up temporary crossing structures so construction machinery and personnel can access both banks without disrupting aquatic species.

2. The construction crew sets up a silt fence about the stream bank to prevent sediment from entering the stream and to comply with the Erosion and Sediment Control Plan (ESCP).

3. The excavator excavates two 75 cubic meter trenches on either bank for concrete footings. The survey team ensures proper excavation location, depth, and grade. The horizontal grade is to be leveled to ± 2º. An environmental quality supervisor should oversee the work to ensure compliance with the ESCP.

4. Construction material for the foundation structures are brought to site via flatbed trucks.

5. The construction crew forms the four concrete bearing pads, places reinforcement and embedded objects, mixes the premeasured concrete material in an electric concrete mixer,

and places the concrete by wheelbarrow. The fresh concrete is vibrated, and sample cylinders for strength testing are cast and cured on site.

6. The concrete is allowed to cure for a minimum of 48 hours, and then the forms are stripped. Pile stilts with bearing pads are craned on to the concrete footings and grouted and bolted into place.

7. The six prefabricated girder pieces are brought to site via flatbed truck and are spliced to form the bridge girders on the low chain bank by the erection crew.

8. Each girder is craned in place over the bearing pads and is bolted in place.

9. The erection crew then carries the angle iron cross bracing between the girders and bolts them into place to form a diaphragm between the two girders.

10. The excavator then buries the bearing pad with excavation sidecast and tamps the fill with the bucket to compact the soil.

11. Timber ballast walls are constructed to prevent abutment fill soil from covering the bearing pads.

12. Abutment fill is trucked in and graded by the rubber tire grader. Each 1 ft lift is compacted with the combo roller until the specified compaction is achieved.

13. Timber decking and railings are installed and fixed to the bridge girders.

12.1 Cost Estimate

CCC completed a cost estimate for the construction of Hat Lake Road as per our design. The cost estimate was broken down into MasterFormat divisions managed by the Construction Specifications Institute and Construction Specifications Canada. The applicable divisions for our project include

- Division 1: General Requirements
- Division 2: Site Construction
- Division 3: Concrete
- Division 5: Metals, and
- Division 6: Wood and Plastics.

These divisions have sections associated with our construction costs. Refer to Appendix M for a cost breakdown. We referred to the *2013 RSMeans Site Work & Landscape* and *2013 National Construction Estimator* catalogues to price work out. Table 12 summarizes the construction cost for the entire project.

Table 12: Construction costs according to MasterFormat divisions (Ip, 2015)

DESCRIPTION	SUBTOTAL	COMMENTS
Division 1 - General Requirements	**$ 221,024.00**	
Division 2 - Site Construction	**$ 2,096,712.62**	
Division 3 - Concrete	**$ 3,339.00**	
Division 5 - Metals	**$ 1,104,546.76**	
Division 6 - Wood & Plastics	**$ 237,360.00**	
Grand Total	**$ 3,662,982.38**	
	$ 2,827,955.18	without gravelling

The construction estimate is $3.66 million with the cost to spread gravel along Hat Lake Road included.

CCC has assumed a 19.8 m (65 ft) culvert crossing at Site 7c and a 21.3 m (70 ft) bridge crossing at Site 8 which was outside of our project scope. In addition, CCC’s Hat Lake Road profile drawings show 4 crossings outside of CCC’s scope. Changes to the crossings at Site 7c and/or Site 8 and to the scope by adding extra crossings will produce additional costs. The cost to add a bridge crossing is approximately $570 thousand for a 26 m span; whereas the cost to add a culvert crossing is $100 thousand for a 19 m length about 2 m in diameter.

13.0 CONCLUSION

Clear Cut Consulting Ltd. has completed the design, construction phasing and cost estimate for the Hat Lake Road (Phase 1) project. The Hat Lake Road project consists of 4 major stream crossings: Site 10, Site 8, Site 7c and Site 7a. CCC's scope of work encompassed the design of a bridge for Site 10, culvert for Site 7a, and conception road design.

The Hat Lake Road project is located in the region of Fort St. James, BC. DWB provided all surveying data along Hat Lake Road and at the site crossings to CCC. CCC, then, translated the data to C3D and merged the data to create a surface model. The surface model was used for all designs.

Site 10 presented some challenges for our design. The road alignment crosses the stream at a nearly parallel angle, which creates fill embankment side spill issues. We addressed the issues by coming up with three different options: Option A is a 24.4 m bridge span, Option B is a 30.5 m bridge span, and Option C is a 18.3 m bridge span. CCC evaluated each option's advantages and disadvantages before deciding to proceed with Option B. After the final road design, a vehicle sweeping path analysis was carried out to ensure serviceability requirements could be met.

Site 7a also posed challenges for CCC. The issues were the oxbow stream and the steep side slope. Through hydraulic analysis, the culvert diameter and the minimum backfill was determined.

The conceptual road design is not a part of CCC's scope of work. We provided the conceptual road design because provides general understanding of the amount of earthwork required for Hat Lake Road. This conceptual design also determined the tree clearing width helped estimate the construction phasing schedule and cost.

CCC designed a 30.5 m bridge for Site 10. Referenced codes include CSA S16 and S6-06; MFLNRO references used are forestry bridge standard drawings, *Forest Service Bridge Design and Construction Manual*, *Forest Road Engineering Guidebook*, and *Engineering Manual*. The superstructure consists of 2 plate girders with a top and bottom flange dimension of 650 mm by 45 mm and a web plate of 1710 mm by 25 mm. The timber deck is as per MFLNRO standards. Diaphragms and connection design will be done by a retained detail engineer. The foundation structure of the Site 10 crossing is composed of a reinforced concrete bearing pad on pile stilts which are buried beneath the elevation of the stream scour depth to avoid creek washout failure during a design flood event.

Isoline and the FREG method were used to determine the 100-year peak flow. The FREG method also estimates the culvert size. *Hydraflow* took the peak flow from the isoline method to determine the culvert diameter. The resulting culvert diameter at Site 7a with a 40 percent embedment is 3.6 meters.

The estimated construction cost for the Hat Lake Road project is $3.66 million. This budget assumes Site 8 is a bridge and Site 7c is a culvert. Since gravelling is done annually, the cost of gravelling has been separated in our cost estimate for client information.

14.0 EPILOGUE

Clear Cut Consulting Ltd. was formed for the CIVL 7090 Capstone Project course at BCIT. The course requirement is to complete a civil design project with fellow classmates. CCC's team members have different sets of skills that worked well in combination for the Hat Lake Road project. We outline our final thoughts on the Capstone Project in four sections: team CCC, time management, research & design, and presentations, committee meetings & report writing.

14.1 Team CCC

The members of CCC collaborated in a well-organized and effective manner. All members of the team were present during group working sessions. When progress milestones were delayed by a certain task, all members were informed of the delay. The CCC team was also provided weekly updates on tasks completed, and ongoing. These updates aided in the coordination of future work.

14.2 Time Management

CCC's project proposal schedule indicated a start date during BCIT's winter break. Instead, the team took the entire break period off to rest. As a result, our entire schedule for the Hat Lake Road project was pushed back two weeks, corresponding to the break period. CCC completed this project on schedule on March 14, 2015.

CCC recommends future Capstone Project teams use their time wisely.

14.3 Research & Design

The research required for the Hat Lake Road was extensive due to CCC's lack of knowledge in forestry road design. We often started the design of a certain aspect, only to back-track upon gaining more information about the design of that aspect. Moreover, during the design of this project, we realized that our proposed scope was too extensive; we appreciate the Capstone Committee's suggestion to reduce our scope of work to two site crossings.

CCC also appreciates Jay Kim's commitment to this project. Mr. Kim enabled team CCC to produce the design and drawings of Hat Lake Road.

14.4 Presentations, Committee Meetings & Report Writing

The presentations required for CIVL 7090 helped improve our oral presentation skills. One of team CCC's goals for this project was to improve our public speaking skills, which we feel was accomplished. In addition to the presentations, the Capstone Committee progress meetings kept us aware of the project schedule and also provided insight on how actual meetings are held.

CCC wrote a lot for this project and editing caused temporary vision loss. We believe the submission of the draft report was very useful since our draft was 40 percent completed.

Finally, CCC would like to thank BCIT's Civil Engineering Department for all their hard work and support.

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