Attachment 13.7

North Railway Arch Bridges Hydraulic Assessment

(53 Pages)

Baffinland Iron Mines Corporation
Mary River Expansion Project

Rail Bridge Design - Hydraulic Report

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Table of Contents

1. Introduction

The proposed alignment for the new railway for the Mary River Project has four stream crossings, which will require bridges. Various hydraulic analyses have been undertaken in order to assess the bridge hydraulics for bridge design, calculate expected local bridge scour, and design abutment and pier scour protection countermeasures.

2. Site Description

The four crossing locations of the new railway for the Mary River Project are:

- BR #15.913 (referred to as Bridge 1)
- BR #70.363 (referred to as Bridge 2)
- BR #85.640 (referred to as Bridge 3)
- BR #101.842 (referred to as Bridge 4)

The site descriptions for all four crossing locations are based on the data available, as listed in Section 3. The lack of historical information and historical satellite imagery makes it difficult to comment on the long term morphology of the streams. However, some observations on the stream morphology are made based on the river hydraulics and site photos in Section 9.1.

Figure 2-1 illustrates the location of all four crossing locations relative to each other, as well as their total drainage areas. More details regarding the drainage areas and hydrology is described in Section 6.

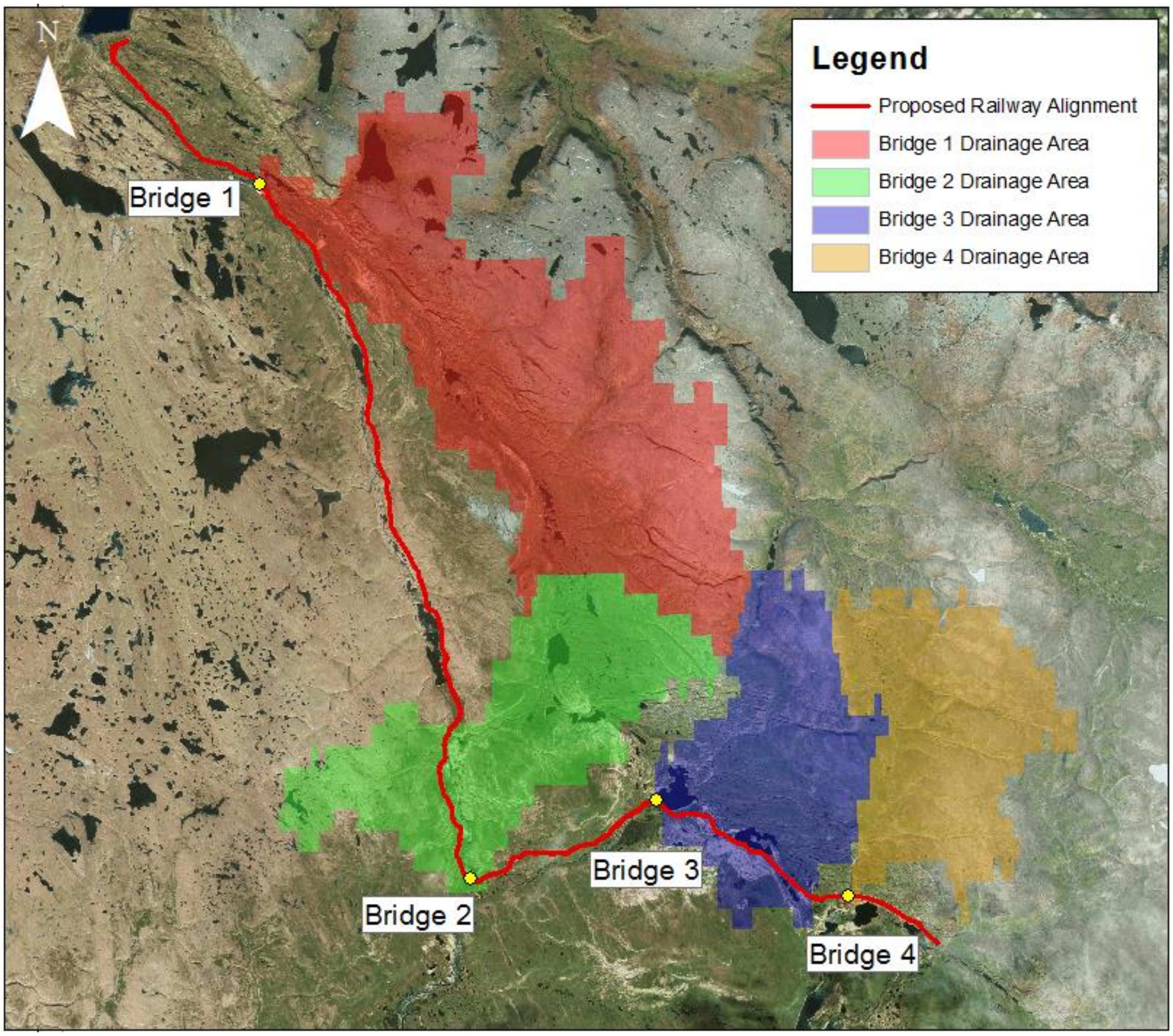

Figure 2-1: Bridge Crossing Locations and Drainage Areas

2.1 Bridge 1

The proposed location of Bridge 1 and the available bathymetric data is shown in Figure 2-2, and a site photo of the crossing area is shown in Figure 2-3. Bathymetric cross-sections were collected for a distance of approximately 550 m upstream, and 350 m downstream of the crossing location. The surveyed cross-section spacings are around 15 m to 20 m. As shown in Figure 2-2, there is a gap in bathymetry near the center of the river and it is believed that river depths were too great for survey crews to obtain this data safely at the time the data was retrieved. Satellite imagery suggests that this is the location of the deepest thalweg. Engineering judgement was used to fabricate the elevations in this area for the hydraulic model (Section 7.1.1).

The stream width between the steeper banks near the proposed crossing are approximately 100 m before flattening out over the floodplain, and the bank depth is between 1.0 m to 1.5 m. The Geotechnical Investigation Factual Data Report [Ref. 1] reports that investigations for the Bridge 1 abutments encountered primarily sand and silt. Boulders or cobbles were intersected in all investigations. Boulders and cobbles are also observed at the riverbed surface in the site photos. The site photos also show that the floodplain is vegetated (grass) and therefore experience reduced sediment erosion or deposition during flood events. Details regarding the hydraulics of Bridge 1 are described later in Section 1.

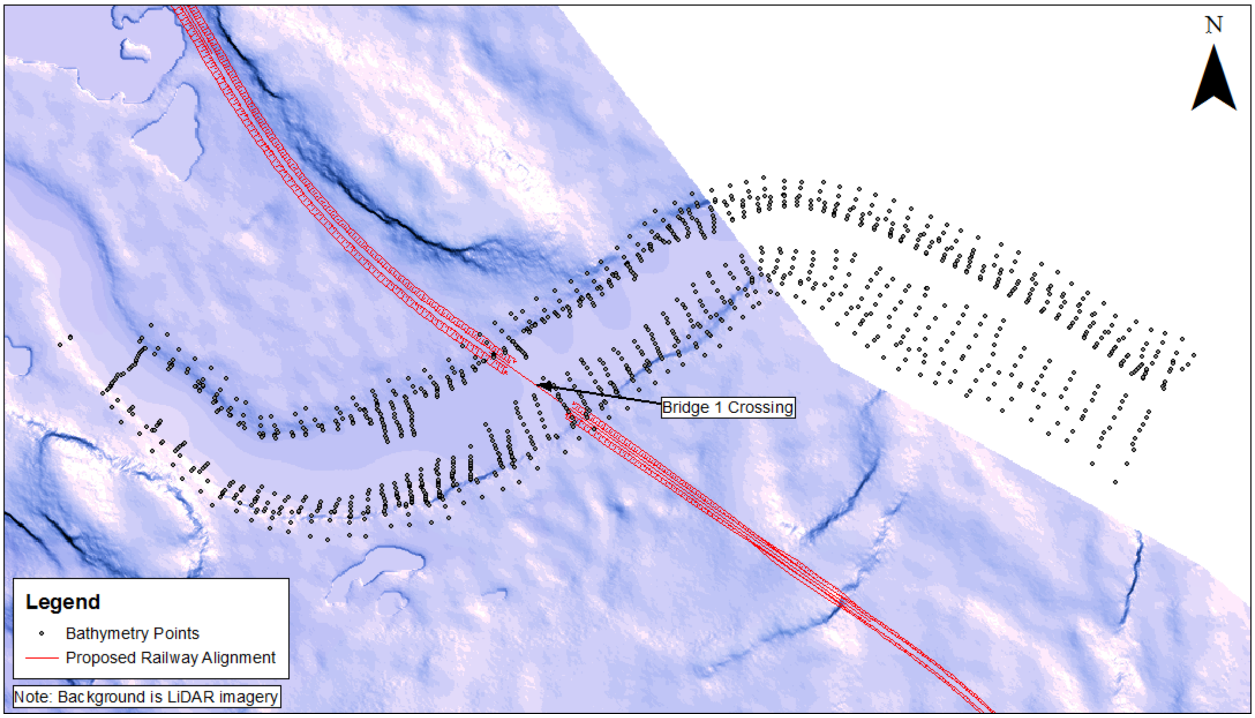

Figure 2-2: Location of Proposed Bridge 1 Crossing

Figure 2-3: Site Photo of Proposed Bridge 1 Crossing Area

2.2 Bridge 2

The proposed location of Bridge 2 and the available bathymetric data is shown in Figure 2-4, and a site photo of the crossing area is shown in Figure 2-5 and Figure 2-6. Bathymetric cross-sections were collected for a distance of approximately 350 m upstream, and 475 m downstream of the crossing location. The surveyed cross-section spacings are around 15 m to 25 m.

As shown in Figure 2-5, the stream was fairly shallow at the time of the bathymetric survey. However, it is anticipated that the stream inundates the wide sandy floodplain during the spring freshet. The stream width between the banks at the proposed crossing location is approximately 60 m before flattening out over the floodplain, and the depth between these banks is only 0.6 m. The Geotechnical Investigation Factual Data Report [Ref. 1] reports that investigations for the Bridge 2 abutments generally encountered silt, silty sand or sand. This fine sand and silt material is observed in the site photos (Figure 2-5 and Figure 2-6), and these photos indicate that the fine sand and silt material extends further upstream. This could indicate an active bed with bed scour occurring during spring floods, but this material is likely replenished by bed movement from further upstream. Details regarding the hydraulics of Bridge 2 are described later in Section 1.

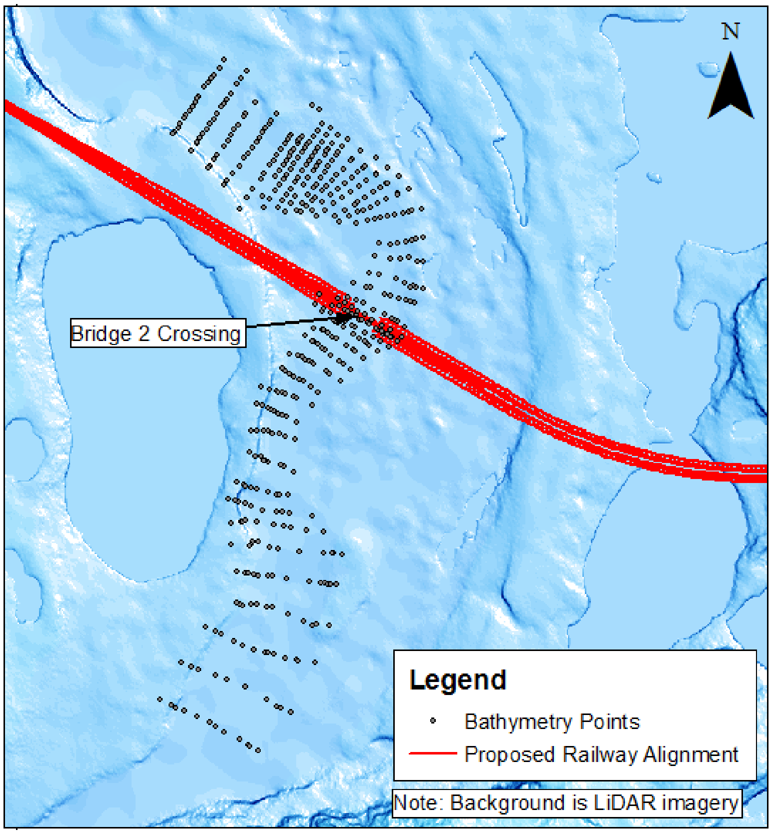

Figure 2-4: Location of Proposed Bridge 2 Crossing

Figure 2-5: Site Photo #1 of Proposed Bridge 2 Crossing Area

Figure 2-6: Site Photo #2 of Proposed Bridge 2 Crossing Area

2.3 Bridge 3

The proposed location of Bridge 3 and the available bathymetric data is shown in Figure 2-7, a site photo of the crossing area is shown in Figure 2-8, and a site photo of the existing tote bridge upstream of the Bridge 3 crossing area is shown in Figure 2-9. Bathymetric cross-sections were collected for a distance of approximately 190 m upstream, and 450 m downstream of the crossing location. The surveyed cross-section spacings are around 10 m to 20 m. The stream width between the steeper banks near the proposed crossing are approximately 100 m. The vertical depth between the top of the banks and the river bottom is approximately 2.5 m, although the water surface may not often reach the bank full elevation. The Geotechnical Investigation Factual Data Report [Ref. 1] reports that the overburden materials encountered at the Bridge 3 abutments generally consisted of well bonded frozen sand. Sand, gravel and silty sand layers were observed in some of the boreholes. Details regarding the hydraulics and potential erosion at Bridge 3 are described later in Section 1.

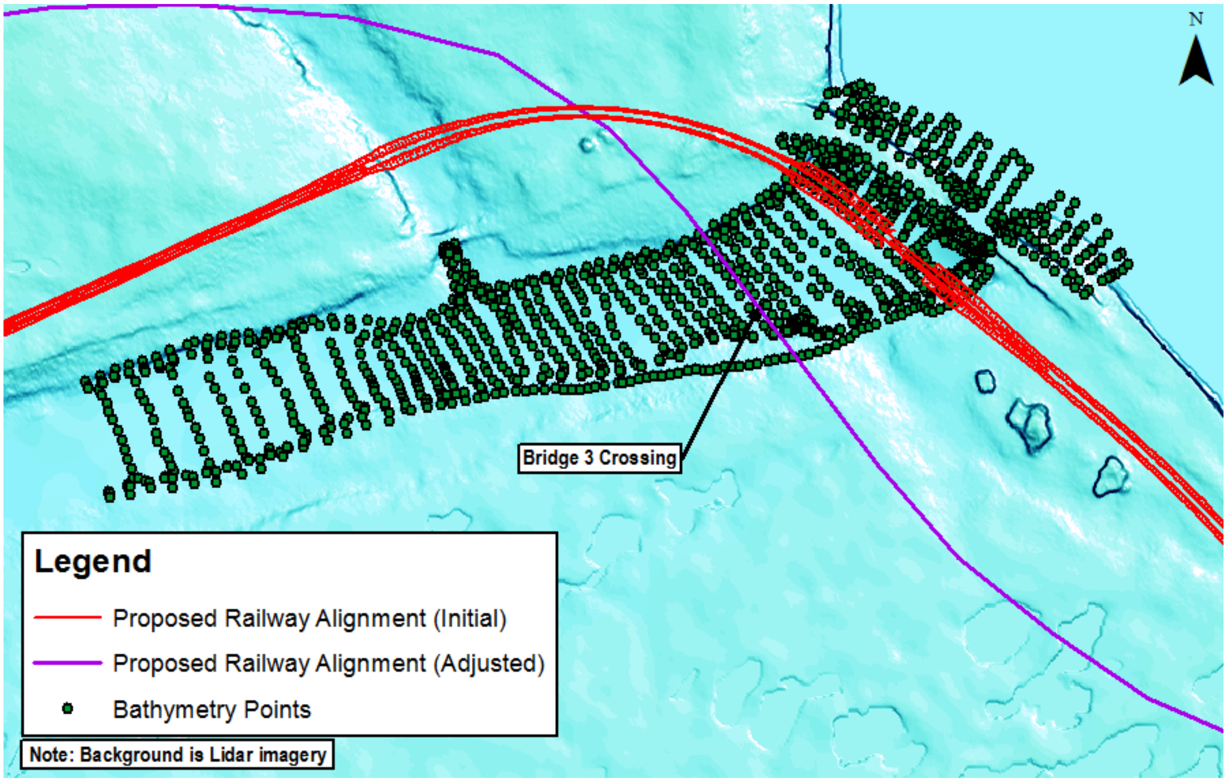

Figure 2-7: Location of Proposed Bridge 3 Crossing

Figure 2-8: Site Photo of Proposed Bridge 3 Crossing Area

Figure 2-9: Site Photo of Tote Bridge upstream of Bridge 3 Crossing Area

2.4 Bridge 4

The proposed location of Bridge 4 and the available bathymetric data is shown in Figure 2-10, and site photos of the crossing area is shown in Figure 2-11 and Figure 2-12. Bathymetric cross-sections were collected for a distance of approximately 290 m upstream, and 310 m downstream of the crossing location. The surveyed cross-section spacings are around 10 m.

The stream width between the steeper banks near the proposed crossing are approximately 65 m before flattening out on a higher shelf. As shown in Figure 2-11, the stream near the crossing consists of a steep and shallow rapid water section. The Geotechnical Investigation Factual Data Report [Ref. 1] reports that gravel, and gravel/sand are generally encountered at both abutment locations, and that the bedrock was outcropped at the surface of both edges of the river, but dips below the surface moving away from the abutments. As shown in the site photos, the riverbed consists of coarser rock such as cobbles and boulders. Details regarding the hydraulics and potential erosion of Bridge 4 are described later in Section 1.

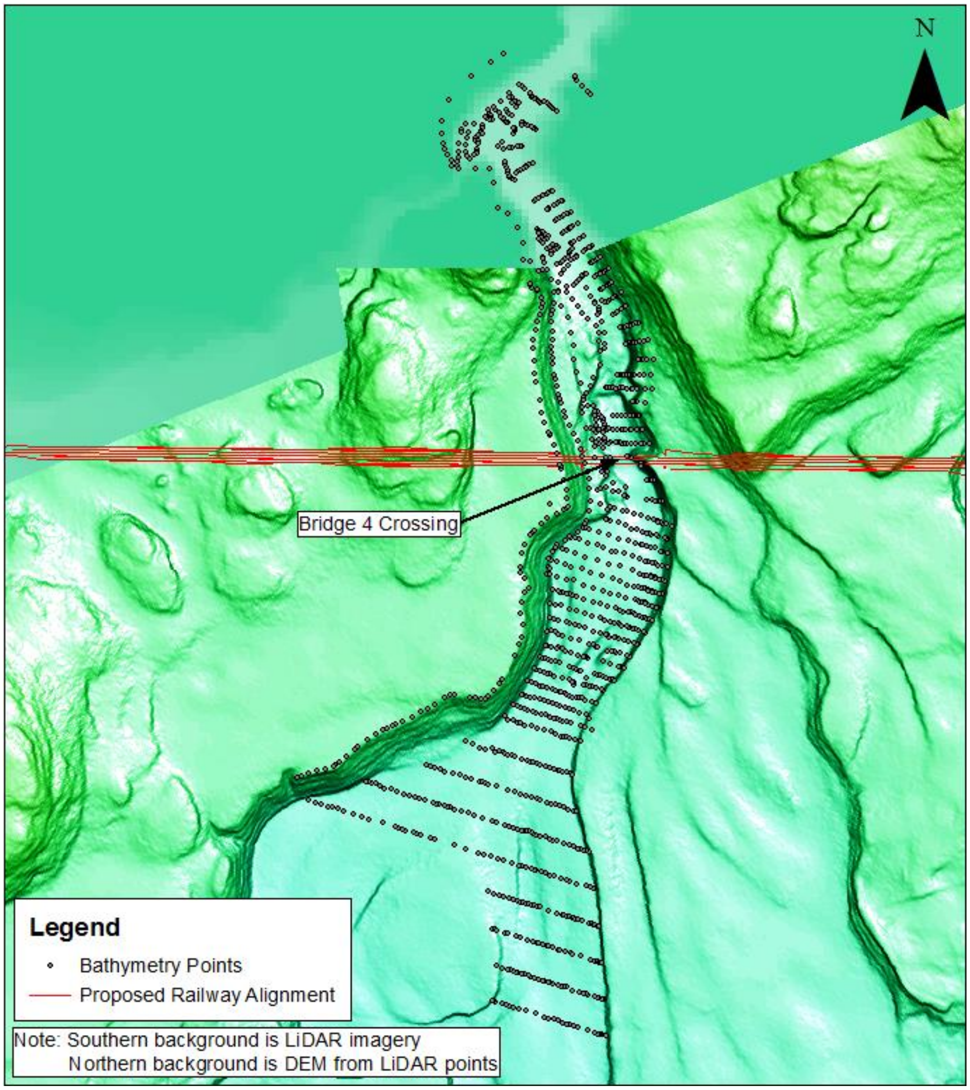

Figure 2-10: Location of Proposed Bridge 4 Crossing

Figure 2-11: Site Photo of Proposed Bridge 4 Crossing Area

Figure 2-12: Site Photo of Area Downstream of Proposed Bridge 4 Crossing

3. Available Data

During the execution of this hydraulic study, various data sources were collected and available, which included:

- Bathymetric cross-sections at all four crossing locations.
  - Bridge 1 bathymetry collected 550 m upstream and 350 m downstream of the proposed crossing location, with cross-section spacings of around 15 m to 20 m.
  - Bridge 2 bathymetry collected 350 m upstream and 475 m downstream of the proposed crossing location, with cross-section spacings of around 15 m to 25 m.
  - Bridge 3 bathymetry collected 190 m upstream and 450 m downstream of the proposed crossing location, with cross-section spacings of around 10 m to 20 m.
  - Bridge 4 bathymetry collected 290 m upstream and 310 m downstream of the proposed crossing location, with cross-section spacings of around 10 m.
- LiDAR points of the streams and floodplains.
- A coarse Canadian Digital Elevation Model (DEM) of the entire project area (Geogratis).
- Depth-discharge rating curve at gauge located approximately 70 m upstream of the proposed Bridge 4 crossing location.
- Satellite imagery of the Bridge 1 crossing location (PDF format).
- Baseline hydrology report from Knight Piesold Consulting [Ref. 2].
- Updated design peak flow assessment from Knight Piesold Consulting [Ref. 3].
- Environment Canada historic weather gauge information.
- CAD drawings of the proposed railway alignment and crossing locations.

In addition to the data above, ongoing communication with the bridge structures and geotechnical groups provided information and details regarding feasible options for the bridge design and scour countermeasures.

4. Crossing Types

The development of the four bridge crossings is an iterative effort between the various design groups. In general, the preference was for multi-span bridges with 15 m spans, piers consisting of groups of steel pipes, and spill through abutments protected by riprap.

Based on the conceptual designs of the bridges, pier widths of 0.6 m were used in the numerical hydraulic modeling. In addition, the abutments were set as a vertical wall between the bridge deck and 0.6 m above the 200-year design water level, followed by a 2H:1V slope extending from 0.6 m above the 200-year water level to the riverbed to represent a riprap slope protecting the abutments (following recommendations described in Section 9.3.2).

The number of spans were selected based on stream geometry and hydraulic analysis to prevent excessive scour and morphologic changes. Details regarding the span lengths evaluated in the hydraulic model, and the hydraulic recommendations are described in Section 8.1. The scour analysis and recommended scour countermeasures are described in Section 9.

5. Design Guidelines and Criteria

Various design guides and project reports were used to develop criteria regarding the hydraulics, scour assessment and scour protection countermeasures. These include:

- Federal Highways Administration (FHWA) HEC-20 publication “Stream Stability at Highway Structures – Fourth Edition” [Ref. 4].
- FHWA HEC-18 publication “Evaluating Scour at Bridges – Fifth Edition” [Ref. 5].
- FHWA HEC-23 publication “Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition (Volumes 1 and 2)” [Ref. 6].
- Transportation Association of Canada (TAC) publication “Guide to Bridge Hydraulics – Second Edition” [Ref. 7].
- Ontario Ministry of Transportation (MTO) publication “Highway Drainage Design Standards (WC -2 Freeboard and Clearance at Bridge Crossings)”, January 2008 [Ref. 8].
- Canadian Highway Bridge Design Code [Ref. 9].
- Various Mary River Project reports (referenced throughout report).

5.1 Design Flows

As described later in Section 8.7, the 200-Year design flood event was used for all bridge design purposes (open-water high water mark, riprap rock size) as a mitigation measure to climate change.

For winter conditions, both the 2-Year and 5-Year design flood events were considered for ice considerations.

5.2 Freeboard

The selected freeboard for this project is 1.5 m above the open water 200-Year flood event. This design flood was chosen to account for increases in future peak flows during floods due to climate change.

The selected freeboard for the winter conditions is 1.0 m above the winter 5-Year flood event. This is for adequate clearance to ensure ice passing through the bridge opening, and for various ice processes such as ice ride-up on the piers and abutments.

5.3 Ice Loading

For design, it is recommended that an effective ice strength value of 700 kPa be used as per the Canadian Highway Bridge Design Code (CSA 2006).

6. Site Hydrology

6.1 Design Flows

Knight Piésold Consulting (KP) completed an updated design peak flow assessment in December 2016 [Ref. 3]. This updated report incorporated recent streamflow data collected since the previous report was completed in 2012 [Ref. 2]. KP used information from six seasonal hydrometric stations operated by Baffinland to determine return period flows and scaling equations for determining design flows to be used in the vicinity of the Mary River project. These six hydrometric stations were operational for periods ranging from 6 to 10 years between 2006 and 2016. The recommended design curve used to determine the scaling equations was an intermediate curve bordered by an envelope curve based on the conservative regional assessment in 2006 on the upper end and an envelope curve based on site data on the lower end. These envelope curves provide some contingency to account for potential increases in peak flows that may be influenced by climate change. The scaling equations derived by KP and used to determine return period flows in this study are as follows:

$$Q_2 = 0.72 \times A^{0.86}$$

$$Q_5 = 1.10 \times A^{0.84}$$

$$Q_{10} = 1.32 \times A^{0.83}$$

$$Q_{25} = 1.70 \times A^{0.82}$$

$$Q_{100} = 2.27 \times A^{0.80}$$

$$Q_{200} = 2.53 \times A^{0.80}$$

Where:

Q = peak instantaneous flow in m^3/s

A = drainage area in km^2

The upstream drainage area for each bridge was determined using a digital elevation model (DEM) from GeoGratis. The drainage areas for bridges can be found in Table 6-1.

Table 6-1: Drainage Areas Upstream of Each Bridge

Structure	Drainage Area (km^2)
Bridge 1	542
Bridge 2	289
Bridge 3	237
Bridge 4	232

Using these drainage areas along with the scaling equations determined by KP, return period flows were determined for each watershed. These flows were used in subsequent HEC-RAS Models. Flows calculated can be seen in Table 6-2.

Table 6-2: Return Period Flows at Each Bridge

Return Period	Flow at Bridge 1 (m^3/s)	Flow at Bridge 2 (m^3/s)	Flow at Bridge 3 (m^3/s)	Flow at Bridge 4 (m^3/s)
Q_2	162	94	79	78
Q_5	218	128	108	107
Q_{10}	246	146	123	121
Q_{25}	297	177	151	148
Q_{100}	350	211	180	177
Q_{200}	390	235	201	197

6.2 Ice Cover

Ice thickness calculations were completed to determine possible ice loading on the piers of the bridges, and whether it could lead to higher water levels impacting the bridge deck. An average and maximum ice thickness was determined for the study area based on the Stefan equation for thermal ice thickness in rivers and lakes:

$$hi = ai \times DF^{0.5}$$

Where:

hi = solid ice thickness (cm)

DF = accumulated degree-days of freezing

ai = empirical coefficient (assumed to be 2.0 to represent the worst-case scenario of a slow moving river without snow).

To determine DF, climate data was obtained from Environment Canada for the gauge closest to the project site with the longest record and similar weather characteristics. Although a seasonal gauge exists on the Mary River, records span only three years from 1963 to 1965. Therefore, the gauge chosen for this ice analysis was Dewar Lakes which is 430 km south of the project site. Although this gauge is some distance from the project site it has over 60 years of climate data and is one of few inland gauges on Baffin Island, making it an appropriate choice to use for these ice calculations since the project site is also inland. A comparison of the temperatures gathered by the gauges during periods in which they were both operational showed that temperatures were similar. Dewar Lakes showed cooler temperatures than Mary River which can be attributed to it's higher elevation although average monthly temperatures were generally within 5°C. A DF value was determined for each year using minimum temperatures at Dewar Lake, starting the fall of one year and continuing into the spring of the next year, capturing the longest period of time in which temperatures were consistently below 0°C. The DF values were then ranked to determine the percent exceedance. The average ice thickness was determined using the 50th percentile DF value and maximum ice thickness using the 95th percentile DF value. The values are summarized in Table 6-3.

Table 6-3: Average and Maximum Ice Thickness at Project Site

Scenario	Ice Thickness (m)
Average	1.50
Maximum	1.64

The calculated values for the average and maximum ice thickness at the project site may be unrealistic as there may not be sufficient depth of water for these thicknesses to form. Considering the water depths at all four bridge crossing locations, it was assessed that a 1.0 m ice cover thickness should be used for the ice condition modeling in Section 8.2.

7. HEC-RAS Modeling

Hatch performed the hydraulic modeling of the bridge crossings using the United States Army Corps of Engineers Hydraulic Engineering Center's River Analysis System (HEC-RAS) software. HEC-RAS was developed by the U.S. Army Corps of Engineers with the purpose of modeling the flow of water through systems of open channels and computing water surface profiles in both steady state and hydrodynamically. Additionally, the HEC-GeoRAS tool for ArcGIS was used, which allows the model to be set up readily from digital terrain data.

7.1 Model Setup

One-dimensional HEC-RAS models for each of the four crossings were developed by drawing cross-sections along the reaches where bathymetric sections were surveyed (Figure 2-2, Figure 2-4, Figure 2-7 and Figure 2-10). The cross-section elevations were extracted from a DEM which was based on the bathymetry over the extent it was surveyed, and LiDAR on the extents outside of the bathymetric survey.

As there is no calibration data available for Bridge 1, Bridge 2 and Bridge 3, several sensitivity analyses were performed to determine the model's sensitivity to the required inputs and boundary conditions, such as:

- Manning's 'n' bed roughness (n = 0.03, 0.035 and 0.04)
- River's slope at the downstream boundary (0.03 m/m, 0.003 m/m and 0.0003 m/m).

As expected, a higher bed roughness results in higher modeled water levels. However, the difference in modeled water levels for the Bridge 1, Bridge 2 and Bridge 3 models were in the order of 10 cm or less for each increase of Manning's 'n' of 0.005 (e.g. from 0.03 to 0.035). Therefore, the Manning's 'n' was set based on features observed in the site photos and geotechnical investigations (i.e., substrate material, vegetation).

The stream's slope boundary condition has a fairly large influence on the water levels near the downstream boundary of the model, but the differences in modeled water levels decrease with distance upstream the river. The model simulations with 0.03 m/m and 0.003 m/m slopes result in the same water levels at the bridge crossing locations, and the flatter 0.0003 m/m slope results in water levels only 5 to 10 cm higher at the bridge crossing locations. Therefore, the slope for each model was based on a combination of the river bed slope near the downstream boundary from the surveyed bathymetry, and the water surface elevation captured by the LiDAR.

A depth-discharge rating curve is available from a gauge located approximately 70 m upstream of the proposed Bridge 4 crossing location. Therefore, the Manning's 'n' and slope boundary condition were adjusted to fit the stage-discharge rating curve.

Specific details of each HEC-RAS model is provided in the sections below.

7.1.1 Bridge 1

Figure 7-1 illustrates the Bridge 1 HEC-RAS model layout, along with the DEM of the bathymetry and LiDAR. Below are a few notes regarding the model setup:

- As mentioned earlier in Section 2.1 and shown in Figure 2-2, there is a gap in bathymetry near the center of the river. Satellite imagery suggests that this is the location of the deeper thalweg. Engineering judgement was used to fabricate the elevations in this area for the hydraulic model based on the bathymetry near the gap, and visual observations from the satellite imagery.

- There is an area downstream of the bridge crossing which has a lower ground elevation, which may be prone to having water spill over to the adjacent river. A lateral weir was added, based on the LiDAR elevations, to capture water spilling over this area and account for the effects on the bridge crossing.
- The LiDAR does not extend to the upstream area of the HEC-RAS model. However, the hydraulics at the bridge crossing are based on the backwater hydraulics from the downstream reach. Therefore, this gap in LiDAR does not impact the hydraulic modeling of the bridge.

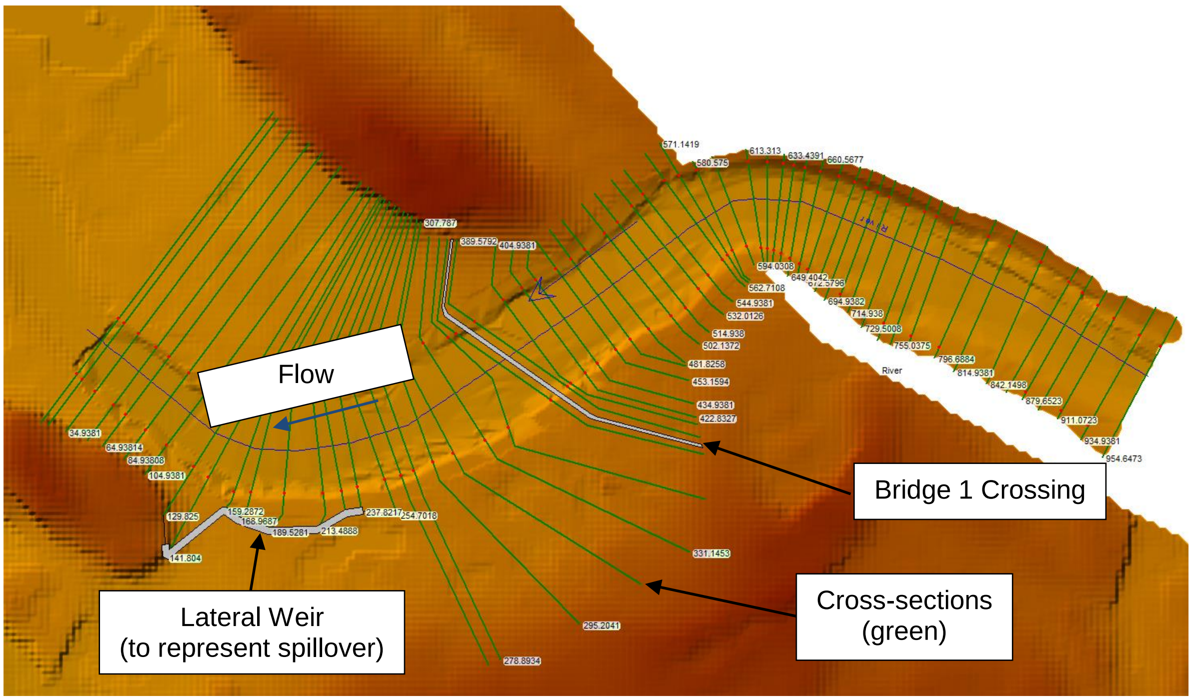

Figure 7-1: Bridge 1 HEC-RAS Model Layout

The Manning's 'n' roughness was set to 0.038 for the stream bed, and 0.033 for the overbank/floodplain area. Site photos illustrate cobbles and boulders protruding from the riverbed, which results in a higher hydraulic roughness, while the floodplain has a fairly smooth vegetative cover. A downstream boundary slope of 0.0015 m/m was set for all model simulations based on a combination of the stream bed slope near the downstream boundary from the surveyed bathymetry, and the water surface elevation captured by the LiDAR.

7.1.2 Bridge 2

Figure 7-2 illustrates the Bridge 2 HEC-RAS model layout, along with the DEM of the bathymetry and LiDAR. Below are a few notes regarding the model setup:

- The cross-sections were extended far past the surveyed bathymetric sections as this reach has a wide floodplain.

- Some additional cross-sections were added downstream of the surveyed bathymetric sections, as this area funnels from a wide floodplain to a narrower valley. These cross-sections were extracted from the LiDAR, except for the stream portion which was set to 0.5 m lower than the LiDAR, as the LiDAR elevations reflect the water surface and not the riverbed. The 0.5 m depth was based on the water depth of the surveyed sections.

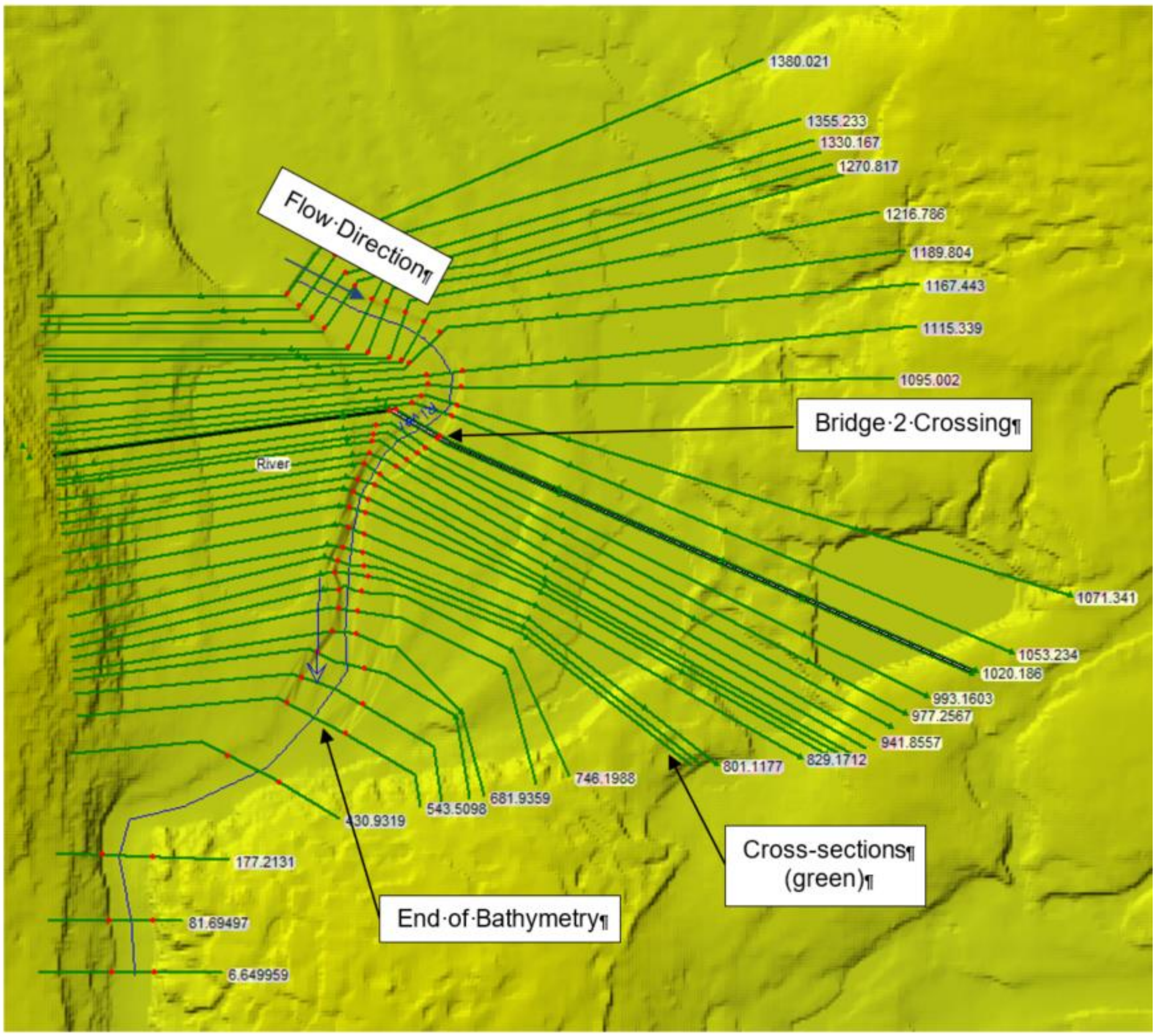

Figure 7-2: Bridge 2 HEC-RAS Model Layout

The Manning's 'n' roughness was set to 0.035 for the stream bed and the overbank/floodplain areas. Site photos illustrate a similar sand substrate in the river and in the overbank/floodplain. A downstream boundary slope of 0.0038 m/m was set for all model simulations based on a combination of the stream bed slope near the downstream boundary from the surveyed bathymetry, and the water surface elevation captured by the LiDAR.

7.1.3 *Bridge 3*

Figure 7-3 illustrates the Bridge 3 HEC-RAS model layout, along with the DEM of the bathymetry and LiDAR. Below are a few notes regarding the model setup:

- There is an existing Tote road bridge upstream of the proposed Bridge 3 crossing. Drawings were available with the required information to input this bridge in the model.
- The Bridge 3 location shown in Figure 7-3 reflects the latest location of the crossing. An earlier iteration of this hydraulic model reflected a previous alignment that was found to be hydraulically unsuitable due to it being located too close downstream of the existing tote bridge.

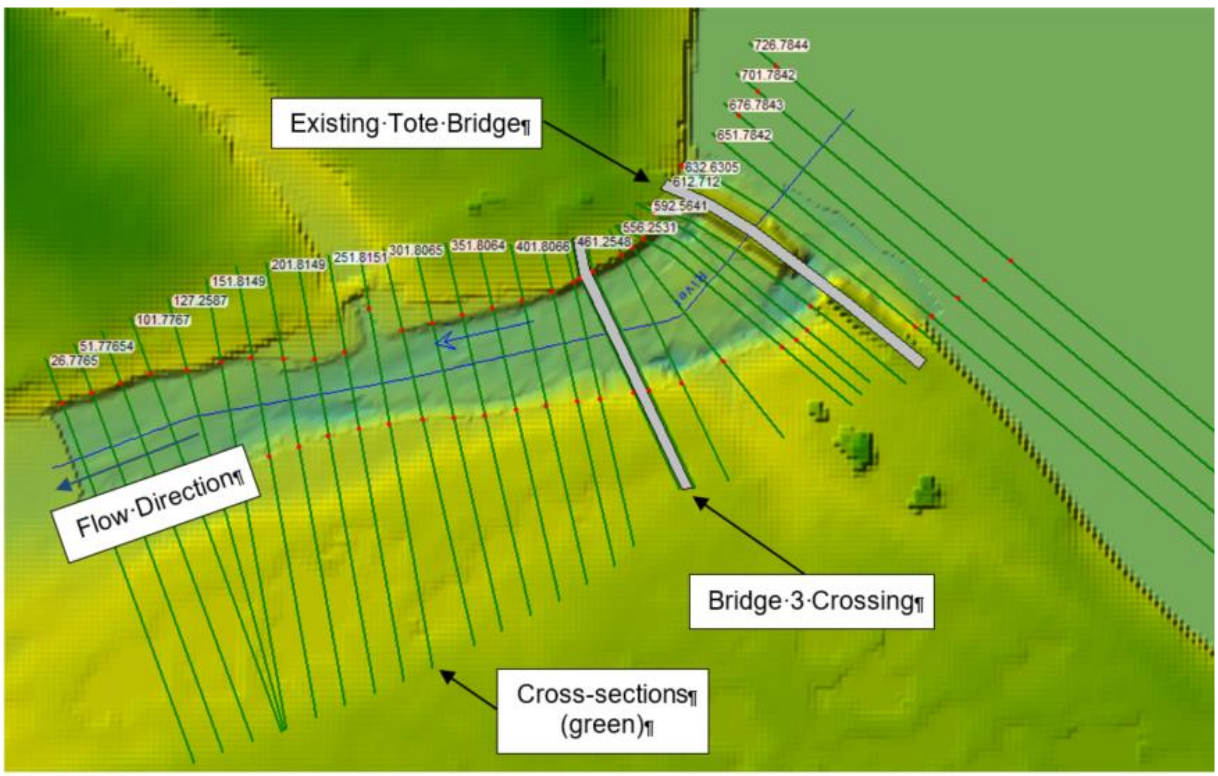

Figure 7-3: Bridge 3 HEC-RAS Model Layout

The Manning's 'n' roughness was set to 0.035 for the stream bed, and 0.04 for the overbank/floodplain area. Site photos illustrate a sand substrate in the river, while the overbank/floodplain consists of a mix of sand and coarser granular material. A downstream boundary slope of 0.001 m/m was set for all model simulations based on a combination of the stream bed slope near the downstream boundary from the surveyed bathymetry, and the water surface elevation captured by the LiDAR.

7.1.4 Bridge 4

Figure 7-4 illustrates the Bridge 4 HEC-RAS model layout, along with the DEM of the bathymetry and LiDAR. As mentioned earlier, a depth-discharge rating curve is available from a gauge located approximately 70 m upstream of the proposed Bridge 4 crossing location.

The model was calibrated to three flows, 32.6 m^3/s, 53.3 $m^{3/}s$ and 77.9 m^3/s. The roughness and the downstream boundary slope were adjusted to provide a close match to the three calibration points from the depth-discharge rating curve, with a higher priority on the high flow (77.9 m^3/s) as it is a better representation of the higher flows needed for bridge design. The model calibration results are summarised in Table 7-1. The -0.8 m discrepancy at the lower flow may be due to the model's bathymetry not capturing a higher shelf which is controlling the water at the gauge location. However, the discrepancy is much smaller at the high flow condition, and it may also be due to a difference between the river invert elevation used to develop the depth-discharge curve, and the invert elevation in the HEC-RAS model from the surveyed bathymetry. Overall, with the limited calibration data available, the model was deemed acceptable for the hydraulic analysis of Bridge 4.

Table 7-1: Bridge 4 Model Calibration Summary

Flow (m^3/s)	Rating Curve Depth (m)	Modeled Depth (m)	Difference (m)
32.6	3.0	2.2	-0.8
53.3	3.2	2.9	-0.3
77.9	3.6	3.7	0.1

The calibrated model has a Manning's 'n' to 0.06 for the stream bed, and 0.045 for the overbank/floodplain area. This is consistent with the site photos, which illustrates a rough rapid water stream. The downstream boundary slope was set to 0.003 m/m.

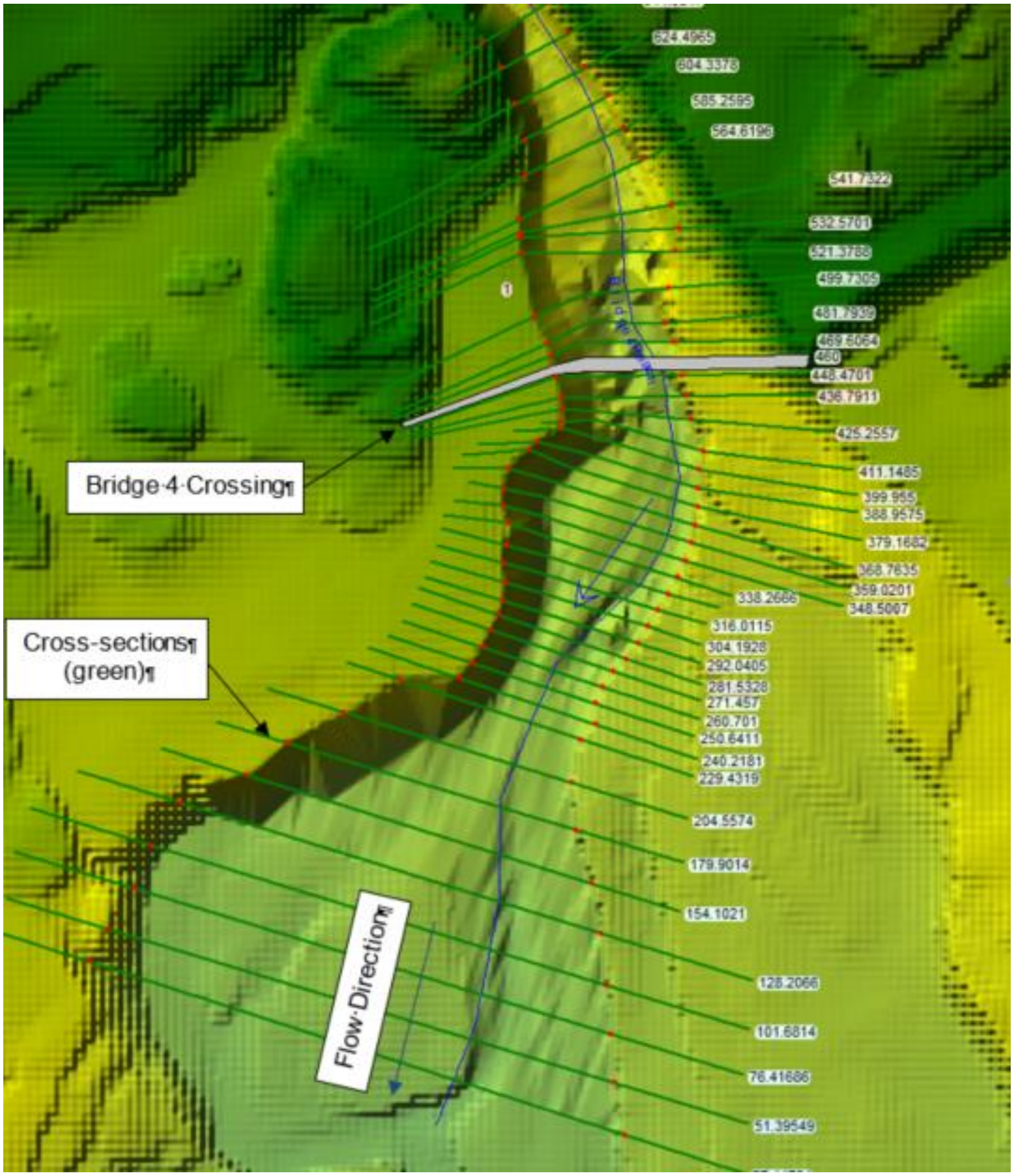

Figure 7-4: Bridge 4 HEC-RAS Model Layout

8. Hydraulic Analysis and Results

8.1 Water Levels and Velocity

The models of all four bridge crossings were used to simulate the passage of all flood events listed in Section 6.1 under the existing conditions (without the proposed bridges) to define the baseline site conditions. Following this, two and three bridge spans were modeled as part of an iterative process to determine the recommended total spans to be considered for the preliminary design. The considerations for determining the recommended bridge spans, based on engineering judgement, were:

- Do not cause a significant increase in channel velocity at the bridge crossing compared to the baseline condition.
- Place the abutments such that they do not slope too deeply into the stream (i.e. abutment toe ends near the top of the steep banks).
- Scour conditions (varies by site).

As described in Section 4, the preference was for multi-span bridges with 15 m spans, piers consisting of groups of steel pipes, and abutments protected by riprap. Based on the conceptual designs of the bridges, pier widths of 0.6 m was used in the numerical hydraulic modeling. In addition, the abutments were set as a vertical wall between the bridge deck and the 200-year design water level, followed by a 2H:1V slope extending from the 200-year water level to the riverbed to represent a riprap slope protecting the abutments.

A recommended total span length for each of the four bridge crossings were chosen by assessing the hydraulic results, and through discussions with the structural and geotechnical teams for constructability. The hydraulic results and descriptions of the recommended bridges are described below. Details regarding scour analysis and recommended scour countermeasures are discussed in Section 9.

8.1.1 Bridge 1

A total bridge span of 105 m (seven 15 m spans) was chosen for Bridge 1. This was mostly due to the crossing consisting of fairly steep banks forming the stream channel with a shallow floodplain. Therefore the abutments were set to not encroach into the deeper stream section.

Table 8-1 summarizes the hydraulic model results at Bridge 1, and Figure 8-1 illustrates a sketch of the bridge crossing representation in HEC-RAS with the modeled 200-year flood event water level.

Table 8-1: HEC-RAS Numerical Model Results for Bridge 1 Crossing Location

Flood Return Period	Flow Magnitude (m^3/s)	Natural Condition (Baseline)		105m Total Span (Bridge 1)	
		U/S Water Level (m)	Velocity (m/s)	U/S Water Level (m)	Velocity (m/s)
2-Year	161.8	77.96	1.27	77.97	1.33
5-Year	217.9	78.13	1.49	78.14	1.58
10-Year	245.5	78.19	1.59	78.20	1.70
25-Year	296.9	78.30	1.76	78.31	1.92
100-Year	349.5	78.40	1.92	78.40	2.14
200-Year	389.6	78.46	2.04	78.45	2.30

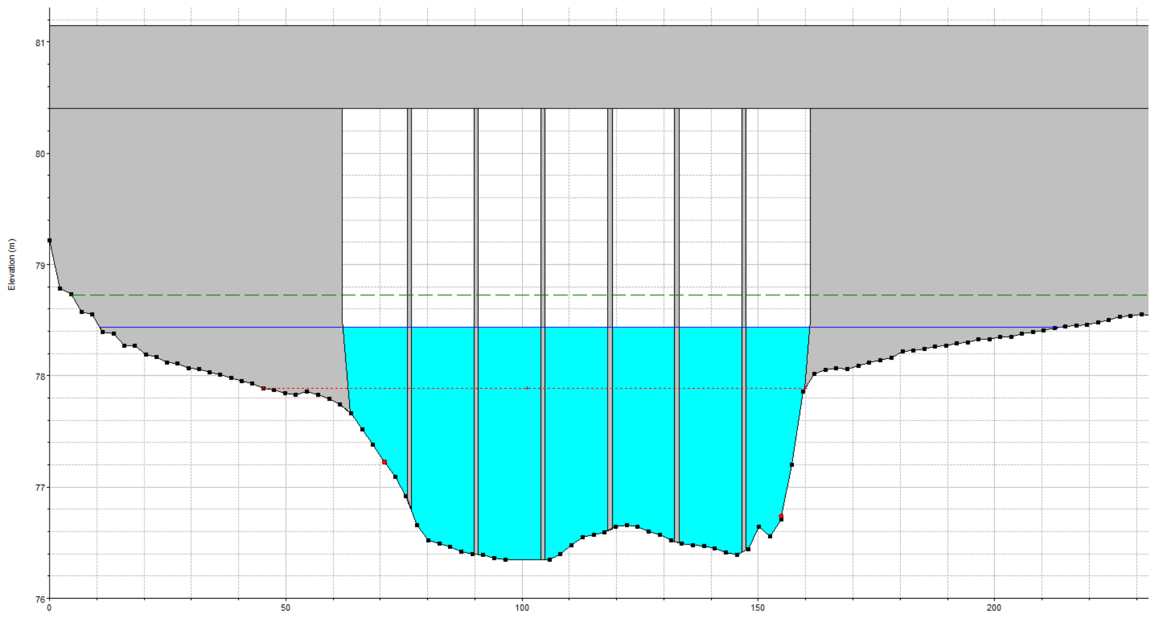

Figure 8-1: Bridge 1 Representation in HEC-RAS (200-year Flood Event)

As shown in

Table 8-1, the channel velocities are only marginally greater with the bridge compared to the baseline condition, and there is no significant head losses to raise the upstream water levels. This is due to the proposed bridge only blocking flow from the shallow floodplain area, and the slight reduction in flow area in the channel from the piers.

Due to the alignment of the Bridge 1 crossing, a 20° skew angle is recommended for the piers to align them parallel with the river's flow direction.

8.1.2 Bridge 2

A total bridge span of 120 m (eight 15 m spans) was chosen for Bridge 2. The Bridge 2 crossing location appears to be along an area with a fairly wide floodplain at 350 m wide. However, due to the area experiencing fairly low velocities (see Table 8-2), it would be unreasonable to have the bridge span over the entire 350 m floodplain area. The 120 m total span was chosen as it resulted in channel velocities below 1 m/s.

Table 8-2 summarizes the hydraulic model results at Bridge 2, and Figure 8-2 illustrates a sketch of the bridge crossing representation in HEC-RAS with the modeled 200-year flood event water level.

Table 8-2: HEC-RAS Numerical Model Results for Bridge 2 Crossing Location

Flood Return Period	Flow Magnitude (m^3/s)	Natural Condition (Baseline)		120m Total Span (Bridge 2)	
		U/S Water Level (m)	Velocity (m/s)	U/S Water Level (m)	Velocity (m/s)
2-Year	94.1	125.14	0.33	125.15	0.64
5-Year	128.4	125.39	0.36	125.38	0.73
10-Year	145.6	125.52	0.38	125.50	0.77
25-Year	177.2	125.73	0.52	125.71	0.83
100-Year	211.3	125.92	0.47	125.90	0.90
200-Year	235.5	126.04	0.46	126.01	0.95

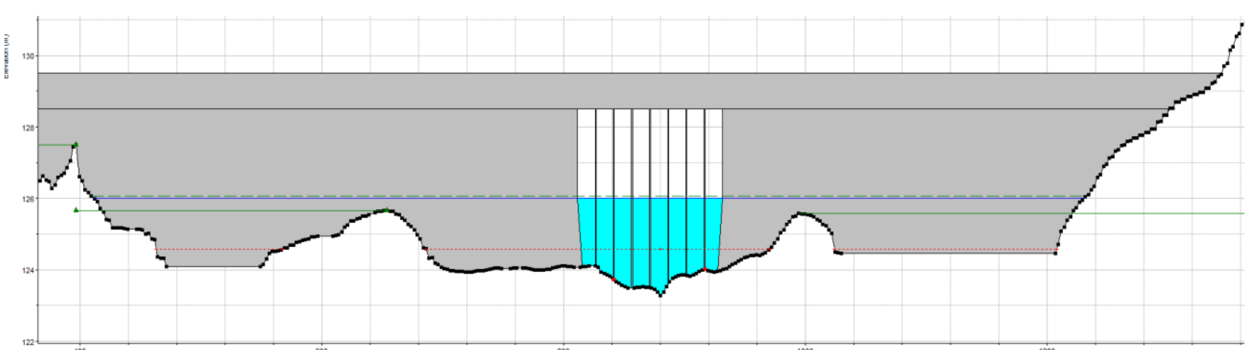

Figure 8-2: Bridge 2 Representation in HEC-RAS (200-year Flood Event)

As shown in Table 8-2, the channel velocities are almost twice as high with the bridge compared to the baseline condition, although there is no significant head losses to raise the upstream water levels. This is due to the proposed bridge blocking a significant portion of the wide floodplain, resulting in higher velocities due to the constriction (compared to the baseline condition). However, these velocities are still relatively low, at less than 1 m/s.

The alignment of the Bridge 2 crossing does not require a skew angle for the piers.

8.1.3 Bridge 3

A total bridge span of 60 m (four 15 m spans) was chosen for Bridge 3. A 45 m bridge span was originally considered to mimic the existing upstream Tote road bridge, which has a 40 m span. However, this resulted in fairly high channel velocities, and the surveyed bathymetry and site photos indicate that significant scour may have occurred at the existing Tote road bridge. In addition, a 45 m span for the Bridge 3 crossing would likely require culverts through the north railway embankment to allow drainage flow and fish passage within the deeper stream bed on the north side of the river (similar to the existing tote bridge).

Table 8-3 summarizes the hydraulic model results at Bridge 3, and Figure 8-3 illustrates a sketch of the bridge crossing representation in HEC-RAS with the modeled 200-year flood event water level.

Table 8-3: HEC-RAS Numerical Model Results for Bridge 3 Crossing Location

Flood Return Period	Flow Magnitude (m^3/s)	Natural Condition (Baseline)		60m Total Span (Bridge 3)	
		U/S Water Level (m)	Velocity (m/s)	U/S Water Level (m)	Velocity (m/s)
2-Year	79.3	143.82	0.71	143.79	1.26
5-Year	108.6	144.05	0.81	144.01	1.63
10-Year	123.4	144.15	0.86	144.10	1.55
25-Year	150.5	144.32	0.94	144.26	1.71
100-Year	180.1	144.48	1.02	144.41	1.88
200-Year	200.8	144.58	1.08	144.50	1.99

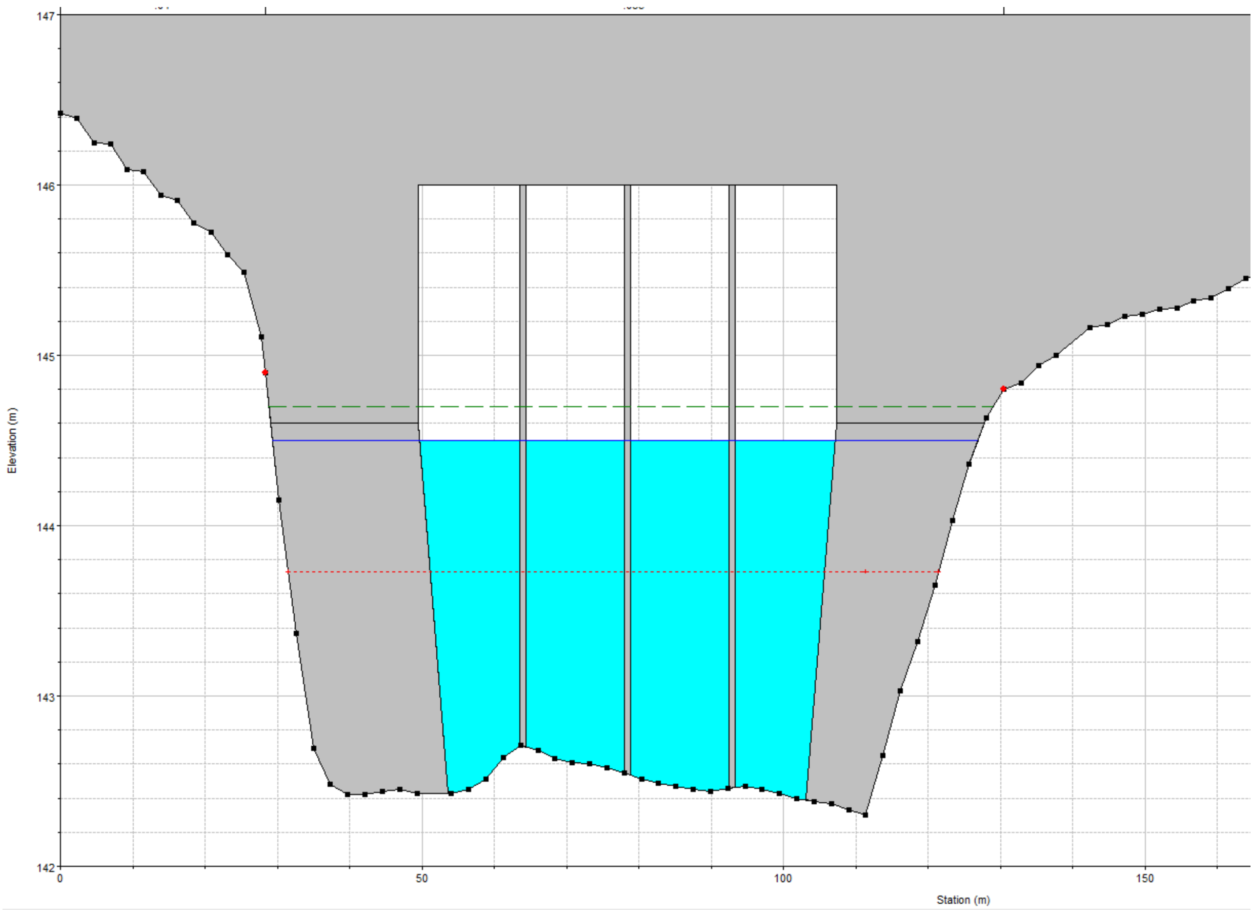

Figure 8-3: Bridge 3 Representation in HEC-RAS (200-year Flood Event)

As shown in Table 8-3, the channel velocities are almost twice as high with the bridge compared to the baseline condition, although there is no significant head losses to raise the upstream water levels. This is due to the proposed bridge blocking a significant portion of the river, resulting in higher velocities due to the constriction (compared to the baseline condition). However, these velocities are approximately 0.3 m/s less than those modeled for the existing tote bridge.

Due to the alignment of the Bridge 3 crossing, a 20° skew angle is recommended for the piers to align them parallel with the river's flow direction.

8.1.4 Bridge 4

A total bridge span of 60 m (four 15 m spans) was chosen for Bridge 4. Due to the steep rapid water nature of the Bridge 4 crossing location, the design flood levels are significantly below the bridge structure. Therefore, the total bridge span is based on positioning the abutments so that they sit on the higher rock shelf of the steep banks, instead of having to extend an additional 5 m lower (approximately) and sit on fractured, or less competent rock.

Table 8-4 summarizes the hydraulic model results at Bridge 4, and Figure 8-4 illustrates a sketch of the bridge crossing representation in HEC-RAS with the modeled 200-year flood event water level.

Table 8-4: HEC-RAS Numerical Model Results for Bridge 4 Crossing Location

Design Flood	Flow Magnitude (m^3/s)	Natural Condition (Baseline)			60m Span Bridge (Bridge 4)		
		U/S Water Level (m)	U/S Velocity (m/s)	D/S Velocity (m/s)	U/S Water Level (m)	U/S Velocity (m/s)	D/S Velocity (m/s)
2-Year	77.9	162.32	1.81	1.87	162.26	2.01	2.04
5-Year	106.7	162.60	2.01	2.27	162.56	2.20	2.48
10-Year	121.2	162.73	2.10	2.46	162.70	2.28	2.68
25-Year	147.9	162.97	2.22	2.77	162.95	2.39	3.02
100-Year	177.1	163.20	2.33	3.10	163.23	2.47	3.38
200-Year	197.3	163.36	2.40	3.31	163.44	2.51	3.66

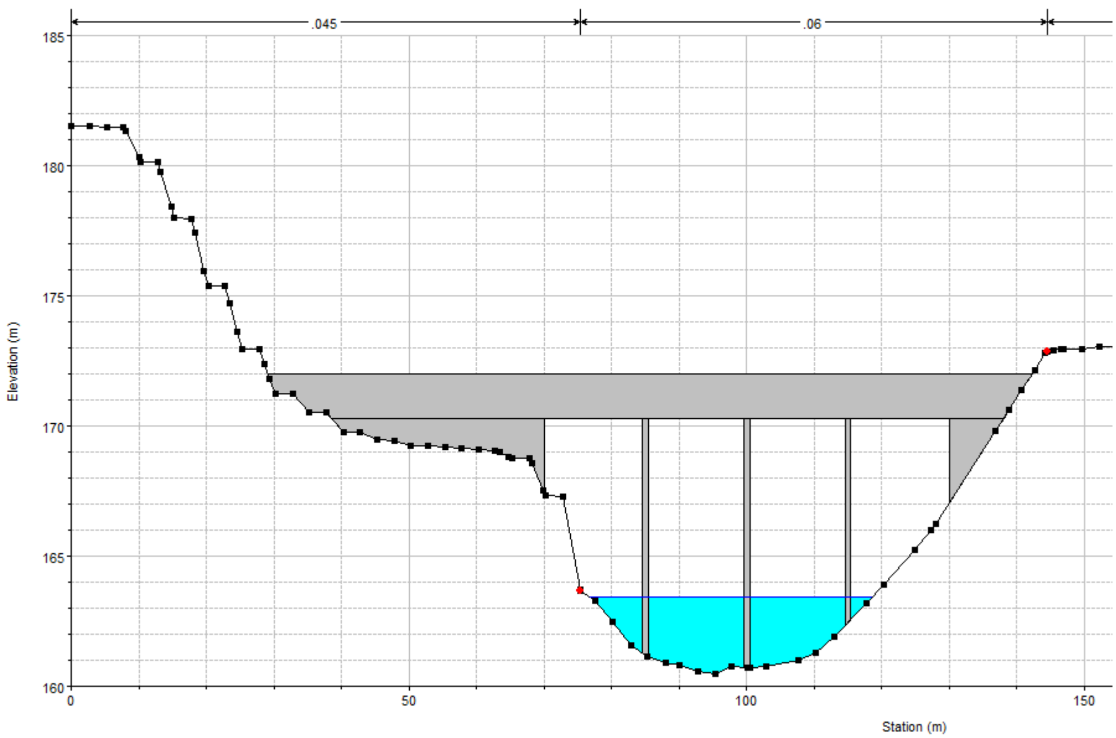

Figure 8-4: Bridge 4 Representation in HEC-RAS (200-year Flood Event)

As shown in Table 8-4, the channel velocities are marginally greater with the bridge compared to the baseline condition, and there is no significant head losses to raise the upstream water levels. This is due to the piers of the proposed bridge slightly reducing the flow area at the crossing.

The alignment of the Bridge 4 crossing does not require a skew angle for the piers.

8.2 Ice

Ice conditions could impact the four proposed bridge crossing by resulting in additional loadings on the piers, and ice affected flooding could result in higher water levels. To better understand the nature of the ice that could develop at the bridge locations, the HEC-RAS models were modified to simulate the hydraulics at the bridge crossings under the influence of a stable ice cover, and also during spring ice jam events.

As discussed in Section 6.2, a 1 m ice cover was used as inputs to the HEC-RAS models. The 2-year and 5-year flood magnitudes summarized in Section 6.1 were used to represent possible flood magnitudes that could occur in the early spring when there is still a full stable ice cover, or during a spring ice jam event.

Both the stable ice cover, and spring ice jam events were modeled in HEC-RAS, which allows user inputs for ice cover thickness, ice roughness (in the form of Manning's 'n'), the ice's specific gravity, friction angle, and whether the ice is allowed to jam. A roughness of 0.015 was used for the stable ice cover simulations, while a roughness of 0.06 was used for the ice jam simulations to represent individual ice blocks being jammed and creating a rough surface.

The HEC-RAS model results for the various ice conditions of all four bridge crossing are summarized in Table 8-5,

Table 8-6, Table 8-7, and Table 8-8. However, as noted in the result tables, the velocities at the Bridge 1, Bridge 2 and Bridge 3 locations are such that rough ice jam conditions are unlikely to form, and are therefore not used for freeboard considerations. As shown in the tables, all ice conditions modeled maintain the minimum 1 m freeboard suggested under the 5-Year design flood condition (when excluding the ice jam condition results for Bridge 1, Bridge 2 and Bridge 3).

Table 8-5: HEC-RAS Ice Conditions Numerical Model Results for Bridge 1 Crossing

Design Flood	Ice Cover Type	Ice Thickness (m)	U/S Modeled Water Level (m)	Difference from Base of Rail (El. 81.689 m)
2-Year	Stable	1.0	79.02	1.38
	Ice Jam	1.0	79.40	*1.00
5-Year	Stable	1.0	79.08	1.32
	Ice Jam	1.0	79.43	*0.97
*: Velocities are such that rough ice jam conditions are unlikely. Not used for freeboard considerations.				

Table 8-6: HEC-RAS Ice Conditions Numerical Model Results for Bridge 2 Crossing

Design Flood	Ice Cover Type	Ice Thickness (m)	U/S Modeled Water Level (m)	Difference from Base of Rail (El. 129.00 m)
2-Year	Stable	1.0	126.15	1.56
	Ice Jam	1.0	126.69	*1.02
5-Year	Stable	1.0	126.43	1.28
	Ice Jam	1.0	127.00	*0.71
*: Velocities are such that rough ice jam conditions are unlikely. Not used for freeboard considerations.				

Table 8-7: HEC-RAS Ice Conditions Numerical Model Results for Bridge 3 Crossing

Design Flood	Ice Cover Type	Ice Thickness (m)	U/S Modeled Water Level (m)	Difference from Base of Rail (El. 148.535 m)
2-Year	Stable	1.0	144.84	2.41
	Ice Jam	1.0	145.33	*1.92
5-Year	Stable	1.0	145.08	2.17
	Ice Jam	1.0	145.63	*1.62
*: Velocities are such that rough ice jam conditions are unlikely. Not used for freeboard considerations.				

Table 8-8: HEC-RAS Ice Conditions Numerical Model Results for Bridge 4 Crossing

Design Flood	Ice Cover Type	Ice Thickness (m)	U/S Modeled Water Level (m)	Difference from Base of Rail (El. 171.574 m)
2-Year	Stable	1.0	163.26	7.03
	Ice Jam	1.0	163.56	6.72
5-Year	Stable	1.0	163.51	6.78
	Ice Jam	1.0	163.84	6.44

As indicated in Section 5, for design, it is recommended that an effective ice strength value of 700 kPa be used as per the Canadian Highway Bridge Design Code (CSA 2006).

Studies and past experience have indicated that riprap placed in environments experiencing ice processes should have a stone weight of 100 kg, which reflects an approximate 0.4 m diameter stone. Therefore, a minimum D_{50} rock size of 0.4 m should be considered for scour protection countermeasures (Section 9.3).

8.3 Debris

No detailed history of debris is available within the area. However, satellite imagery of the local watershed areas (Figure 2-1) do not suggest a significant amount of trees or vegetation available to contribute to debris. Therefore, it is not anticipated that debris will pose a threat to the bridge crossings, and that the clearance provided under the design flow events will be sufficient to pass any debris experienced at the proposed crossing locations.

8.4 High Water Mark and Freeboard Considerations

Table 8-9 lists the open-water high water marks for the four bridge crossings, which are based on the modeled water levels under the 200-Year design flood. As shown in the table, each bridge provides freeboard well above the recommended 1.5 m under the open-water design condition.

Table 8-9: Modeled Open-Water High Water Mark and Freeboard

Bridge	Modeled 200-Year Open-Water Water Level (m)	Bridge Base Elevation (m)	Difference (m)
Bridge 1	78.45	80.40	1.95
Bridge 2	126.01	127.711	1.701
Bridge 3	144.50	147.246	2.746
Bridge 4	163.44	170.285	7.845

8.5 Abutment Locations

The recommended abutment locations for the four bridge crossings, based on the hydraulic analysis, are summarized in Table 8-10. The abutment locations for Bridge 1 and Bridge 4 are based on placing the abutment at the river banks so that the abutments do not spill over too much into the river. The abutment locations for Bridge 2 are based on centering the bridge over the deeper portion of the river and floodplain. The abutment locations for Bridge 3 are based on centering the bridge within the river, but also ensuring that the deep channel on the north side is not blocked to prevent fish stranding.

These locations should be further defined during final design, and verified by a field survey.

Table 8-10: Recommended Abutment Locations

Bridge 1	Easting	Northing	Total Bridge Span (m)
North Abutment	514191.795	7965658.841	105
South Abutment	514277.581	7965598.297	
Bridge 2	**Easting**	**Northing**	**Total Bridge Span (m)**
North Abutment	529077.920	7916717.205	120
South Abutment	529181.225	7916656.148	
Bridge 3	**Easting**	**Northing**	**Total Bridge Span (m)**
North Abutment	542145.375	7922160.931	60
South Abutment	542184.097	7922115.098	
Bridge 4	**Easting**	**Northing**	**Total Bridge Span (m)**
East Abutment	555750.635	7915441.450	60
West Abutment	555690.640	7915442.241	

8.6 Aquatic Considerations

A summary of the proposed engineering design mitigation measures for the Mary River project has been prepared by Knight Piesold Consulting [Ref. 10], and the environmental design criteria was prepared by Hatch [Ref. 11]. As outlined in the KP report summary, each of the four bridges are located in Arctic char streams.

A review of the modeled flow velocities for each design flow condition summarized in Section 8.1 should be reviewed by the aquatics group to determine whether mitigation measures are required to ensure fish passage requirements at all four bridges.

As per the proposed engineering design mitigation measures [Ref. 10], the following mitigation measures, specific to bridges along the rail corridor, will be applied if flow velocities are found to restrict fish passage:

- Support piers will be placed on concrete pads or steel pile caps covered in riprap to stabilize the streambed.
- Wherever feasible, riprap material will be selected to match existing streambed material to provide potential habitat for lower trophic biota and fish and to minimize alteration to fish habitat.
- The channels will be lined with boulders to alter the velocity profile. If necessary, large boulders will be placed in a staggered formation to provide velocity refugia for fish as they move through the bridge structures.

8.7 Climate Change Considerations

A summary of the proposed engineering design mitigation measures for the Mary River project has been prepared by Knight Piesold Consulting [Ref. 10]. As per the suggested engineering design mitigation measures, the 200-Year design flood events were used for all bridge design purposes (open-water high water mark, riprap rock size) as a mitigation measure to climate change. A 1.5 m freeboard above this design event was selected as a conservative value to further ensure the protection of the rail bridges into the future.

9. Scour Assessment and Scour Countermeasures

9.1 Stream Stability (Baseline Condition)

The lack of historical information and historical satellite imagery makes it difficult to comment on the long term morphology of the streams. However, general observations can be made from site photos presented in Section 2 and the hydraulic numerical results presented in Section 8.1. These observations will be with respect to the bank stability, and the bed substrate.

As indicated in the TAC guidelines [Ref. 7], rivers generally form their channels to accommodate large but not extreme flow events. Therefore, the 2-Year flow events will be used when commenting on the stream stability.

9.1.1 Bridge 1

The proposed Bridge 1 crossing is located in a meandering section of a relatively flat stream, which appears to be in a generally hilly area with moderately sloped tributaries. No historic imagery is available to comment on the bank's historic movement.

Based on the surveyed sections, the channel banks are only 1 m to 1.2 m deep, and have slopes in the order of 7H:1V to 10H:1V at the crossing location. In addition, the site photos show vegetation (grass, no trees) near the stream, with a mix of sand with cobbles and boulders throughout the stream. Therefore, the banks appear to be stable with no significant signs of erosion or slope failure.

The modeled 2-Year design flow velocities under the natural conditions are approximately 1.3 m/s, which, based on shear stress analysis, can potentially erode coarse gravel (grain size 16 mm to 32 mm). When compared to the site photo observations, it is likely that there is sand and gravel movement along the river during high flow events, which is re-supplied by material originating from upstream (as sand is present in the site photos). The observed cobbles and boulders within the stream should offer some protection from scour significantly impacting the stream bed.

9.1.2 Bridge 2

The proposed Bridge 2 crossing is located in a wide but shallow stream, which appears to be in a generally hilly area with moderately sloped tributaries. No historic imagery is available to comment on the bank's historic movement.

Based on the surveyed sections, the channel within the floodplain is only approximately 0.8 m deep, and has slopes greater than 20H:1V at the crossing location. The site photos show mostly sand along the floodplain, but the edges have vegetation (grass, no trees) with no visible signs of slope failures. Therefore, the banks appear to be stable with no significant signs of erosion or slope failure.

The modeled 2-Year design flow velocities under the natural conditions are approximately 0.35 m/s, which, based on shear stress analysis, can potentially erode very coarse sand (grain size 1 mm to 2 mm). When compared to the site photo observations, it is likely that there is sand movement along the river during high flow events, which is re-supplied by material originating from upstream. However, these velocities are still fairly low, which should not result in any significant scour or mass erosion.

9.1.3 Bridge 3

The proposed Bridge 3 crossing is located in a wide but shallow stream located downstream of a lake. There is an existing tote bridge upstream of the proposed Bridge 3 crossing. No historic imagery is available to comment on the bank's historic movement.

Based on the surveyed sections, the channel banks are around 2.5 m deep, and have an approximate 3H:1V slope on the north bank, and approximate 5H:1V slope on the south bank at the crossing location. The site photos show mostly sand along the river and floodplain, with some gravel, which is either natural to the river, or may be from the construction of the upstream tote bridge. It is difficult to determine from the site photos whether the river banks are stable or show signs of bank failure.

The modeled 2-Year design flow velocities under the natural conditions are approximately 0.7 m/s, which, based on shear stress analysis, can potentially erode very fine to medium gravel (grain size 4 mm to 8 mm). When compared to the site photo observations, it is likely that there is sand movement along the river during high flow events, which may be re-supplied by material originating from upstream or from the river banks.

9.1.4 Bridge 4

The proposed Bridge 4 crossing is located in steep rapid flowing stream within a deep valley which consists of gravel, cobbles and boulders. No historic imagery is available to comment on the bank's historic movement.

Based on the surveyed sections and site photos, the channel banks are fairly steep at the crossing location, with some areas consisting of a nearly vertical rock faces. The site photos show mostly gravel, cobbles and boulders along the river, and the top of the valley is around 10 m above the river bed.

The modeled 2-Year design flow velocities under the natural conditions are approximately 1.8 m/s, which, based on shear stress analysis, can potentially erode very coarse gravel (grain size 32 mm to 64 mm). When compared to the site photo observations, the river bed appears stable, although there may be some gravel movement during high flow events.

9.2 Scour Assessment

Three types of scour can occur at a bridge crossing; general contraction scour, localized pier scour, and abutment scour. As summarized in the FHWA HEC-18 document [Ref. 5], Contraction scour occurs when the flow area of a stream at its flood stage is reduced, either by a natural contraction (or constriction) of the stream channel, or by a bridge. Local scour at bridge piers and abutments are due to the horse shoe vortices that form at their base due to the obstructions to flow.

All scour calculations were completed using FHWA methodologies from HEC-18 [Ref. 5], and verified using the scour calculations built into the HEC-RAS model. The HEC-RAS scour calculations were generally in agreement with the FHWA calculations. The scour calculations are based on the 200-year flood events (Section 5).

The total scour at the piers is a combination of the contraction scour, which occurs over the entire crossing, and the local scour calculated for the piers. Similarly, the total scour at the abutments is a combination of the contraction scour and the local scour calculated for the abutments.

The scour calculations for each bridge crossing are summarized below, and are based on preliminary bridge drawings provided by the design team [Ref. 12].

A factor of safety should be set for the depth of the pier footings (or pile depths) over the calculated total pier scour depths. Recommendations from the Ministry of Transportation of Ontario (MTO) include either a minimum pier footing elevation of 2.0 m below the natural riverbed, or at 1.7 times the calculated scour depth, whichever is greatest [Ref. 8]. However, footings may be founded on scour-resistant, durable bedrock at a higher elevation, provided that the depth is sufficient to ensure that they remain unaffected by scour, freezing, weathering, degradation, or artificial deepening.

9.2.1 Bridge 1

The riverbed grain sizes used for the scour assessment are D_{50} = 0.18 mm, and D_{95} = 5 mm. These are based on borehole BH16-B001 (summarized in [Ref. 1]).

The Bridge 1 scour calculations are summarized in

Table 9-1. As shown in the table, the predicted scour depths are fairly deep, and may require some scour protection countermeasures. These are summarized in Section 9.3.

Table 9-1: Bridge 1 Scour Calculations

Individual Calculations	
Contraction Scour	0.4 m
Pier Scour (Typical)	2.1 m
Pier Scour (Super Bent)	2.7 m
Left Abutment Scour	3.6 m
Right Abutment Scour	2.8 m
Combined Results	
Total Pier Scour (Typical)	2.5 m
Total Pier Scour (Super Bent)	3.1 m
Total Left Abutment Scour	4.0 m
Total Right Abutment Scour	3.2 m
Elevations	
Minimum Surveyed Natural Bed (Existing)	76.3 m
Pier Minimum Post-Scour Ground El. (Typical)	73.8 m
Pier Minimum Post-Scour Ground El. (Super Bent)	73.2 m
Pier Footing Elevation – 1.7 x Scour Depth (Typical)	72.0 m
Pier Footing Elevation – 1.7 x Scour Depth (Super Bent)	71.0 m

9.2.2 Bridge 2

The riverbed grain sizes used for the scour assessment are D_{50} = 0.16 mm, and D_{95} = 0.37 mm. These are based on borehole BH17-B002 (summarized in [Ref. 1]).

The Bridge 2 scour calculations are summarized in

Table 9-2. As shown in the table, the predicted scour depths are fairly deep, and may require some scour protection countermeasures. These are summarized in Section 9.3.

Table 9-2: Bridge 2 Scour Calculations

Individual Calculations	
Contraction Scour	2.2 m
Pier Scour (Typical)	1.2 m
Pier Scour (Super Bent)	1.5 m
Left Abutment Scour	4.8 m
Right Abutment Scour	4.9 m
Combined Results	
Total Pier Scour (Typical)	3.4 m
Total Pier Scour (Super Bent)	3.7 m
Total Left Abutment Scour	7.0 m
Total Right Abutment Scour	7.1 m
Elevations	
Minimum Surveyed Natural Bed (Existing)	123.3 m
Pier Minimum Post-Scour Ground El. (Typical)	119.9 m
Pier Minimum Post-Scour Ground El. (Super Bent)	119.6 m
Pier Footing Elevation – 1.7 x Scour Depth (Typical)	117.5 m
Pier Footing Elevation – 1.7 x Scour Depth (Super Bent)	117.0 m

9.2.3 Bridge 3

The riverbed grain sizes used for the scour assessment are D_{50} = 1.0 mm, and D_{95} = 14 mm. These are based on borehole BH17-BR86-2 (summarized in [Ref. 1]).

The Bridge 3 scour calculations are summarized in

Table 9-3. As shown in the table, the predicted scour depths are fairly deep, and may require some scour protection countermeasures. These are summarized in Section 9.3.

Table 9-3: Bridge 3 Scour Calculations

Individual Calculations	
Contraction Scour	1.5 m
Pier Scour (Typical)	1.8 m
Left Abutment Scour	5.6 m
Right Abutment Scour	5.6 m
Combined Results	
Total Pier Scour (Typical)	3.3 m
Total Left Abutment Scour	7.1 m
Total Right Abutment Scour	7.1 m
Elevations	
Minimum Surveyed Natural Bed (Existing)	142.5 m
Pier Minimum Post-Scour Ground El. (Typical)	139.2 m
Pier Footing Elevation – 1.7 x Scour Depth (Typical)	136.9 m

9.2.4 Bridge 4

The riverbed grain sizes used for the scour assessment are D_{50} = 100 mm, and D_{95} = 500 mm. These are based on borehole BH18-BR102-1 (summarized in [Ref. 1]) and site photos.

The Bridge scour calculations are summarized in

Table 9-4. As shown in the table, there is no abutment scour as they are outside of the flow under the design water level. Therefore, the abutments do not require any scour protection. The piers, however, may require some scour protection countermeasures. These are summarized in Section 9.3.

Table 9-4: Bridge 4 Scour Calculations

Individual Calculations	
Contraction Scour	0 m
Pier Scour (Typical)	2.3 m
Left Abutment Scour	0 m
Right Abutment Scour	0 m
Combined Results	
Total Pier Scour (Typical)	2.3 m
Total Left Abutment Scour	0 m
Total Right Abutment Scour	0 m
Elevations	
Minimum Surveyed Natural Bed (Existing)	160.0 m
Pier Minimum Post-Scour Ground El. (Typical)	157.7 m
Pier Footing Elevation – 1.7 x Scour Depth (Typical)	156.1 m

9.3 Scour Protection Countermeasures

The scour protection countermeasures are based on both the FHWA methodologies in the HEC-23 manual [Ref. 6], and the TAC guidelines [Ref. 7]. The chosen scour protection countermeasures for this study is to use rock riprap aprons for both the piers and abutments. The methodologies consist of:

i. Sizing of rock riprap (separate calculations for piers and abutments)

ii. Extent of riprap aprons (installation, extent, depth).

It should be noted that other alternatives could be considered if the riprap apron recommendations are not feasible.

9.3.1 Sizing and Extent of Riprap for Piers

The riprap aprons for the piers should be installed following Design Guideline 11 from the FHWA HEC-23 manual [Ref. 6], and as shown in Figure 9-1. Recommendations include:

- The riprap apron should be installed with its top flush with the riverbed, as opposed to mounded around the pier.
    - Due to the relatively high contraction scour estimates at Bridge 2 and Bridge 3, the pier riprap may not be flush with the riverbed if contraction scour were to occur. Therefore, a 25% factor of safety was applied to the minimum riprap calculations.
- The minimum riprap thickness should be either three (3) times the riprap D_{50}, or 0.3 m below the calculated contraction scour, whichever is greater.

- The riprap extent should be at a minimum of two (2) times the pier width (based on FHWA guidelines [Ref. 6], as shown in Figure 9-1).
    - As this is a group of piles, the pier width could be set to the width of an individual pile. However, the riprap should be placed along the entire width of the pile group, and extended to 2.0 times the width of the outer pile, and measured from the pile location at the bottom of the riprap apron (for angled, or flared piles).
- As indicated in the TAC guidelines [Ref. 7], filters should be placed between the riprap and the bed material when placed over beds of sand or fine gravel to prevent loss of underlying fines. This filter should be 4/3 the pier width (as shown in Figure 9-1).
- Minimum riprap size for ice considerations is 0.4 m (Section 8.2).

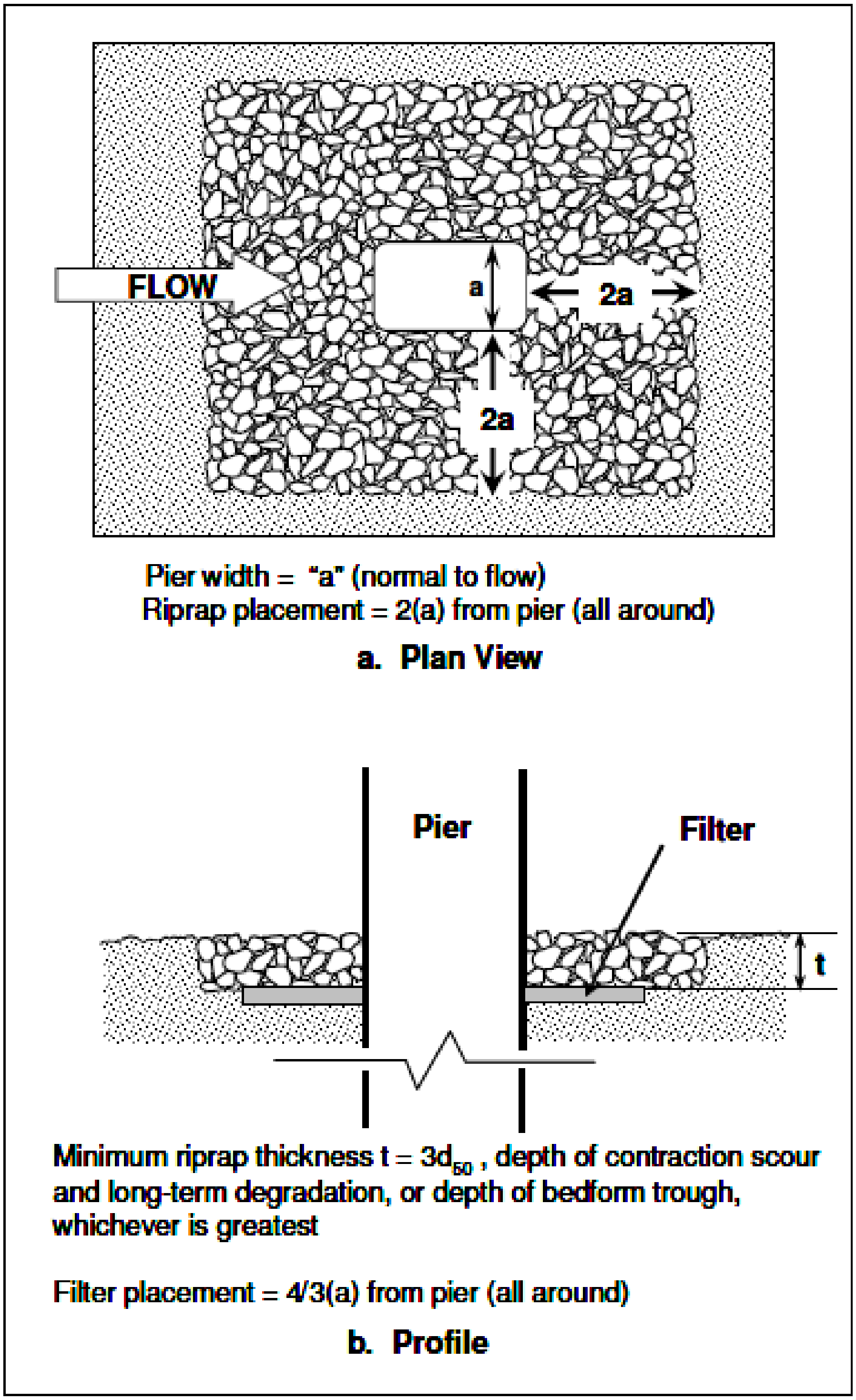

Figure 9-1: Riprap Layout Diagram for Pier Scour Protection (Figure 11.15 from Ref. 6)

The sizing of rock riprap for the pier aprons are based on the methodologies outlined in Design Guideline 11 from the FHWA HEC-23 manual [Ref. 6]. Table 9-5 lists the minimum calculated D_{50} grain sizes for each of the four bridge crossings. These D_{50} grain sizes are the minimum requirements, where larger riprap would be acceptable and would provide greater scour protection. The calculations in Table 9-5 are based on setting the riprap apron flush with the riverbed (see Figure 9-1), with a 25% factor of safety for the post-contraction scour conditions.

Table 9-5: Pier Riprap Apron – Minimum Rock Size and Thickness

Crossing	Minimum D50 Grain Size Requirement (m)	Minimum Riprap Apron Thickness (m)
Bridge 1	0.6	3 x D_{50}, or *0.7 m (whichever is greater)
Bridge 2	0.4	3 x D_{50}, or *2.5 m (whichever is greater)
Bridge 3	0.6	3 x D_{50}, or *1.8 m (whichever is greater)
Bridge 4	1.2	3 x D_{50}

*: Contraction scour (Section 9.2) +0.3 m.

It has been learned that the riprap material available for construction for this project include a Type A (Main Armour Stone) and Type C (Sub-armour Stone). These correspond to a D50 of 1.31 m and 0.61 m respectively. In lieu of the restrictions of available material, the pier riprap aprons for Bridges 1, 2, and 3 should be constructed with a Type C stone riprap and Bridge 4 should use the Type A stone riprap.

9.3.2 Sizing and Extent of Riprap for Abutments

The riprap aprons for the abutments should be installed following Design Guideline 14 from the FHWA HEC-23 manual [Ref. 6], and as shown in Figure 9-2, and also following recommendations from the TAC guidelines [Ref. 7].

The apron should extent should follow at a minimum two (2) times the flow depth, or 25 ft (7.6 m), whichever is less (Figure 9-2). However, since the calculated abutment scour depths are fairly large (Section 9.2), considerations should be to prevent undermining of the apron which can cause it to fail. Options ‘c’ (launching apron) or ‘d’ (toe trench) shown in Figure 9-3 are possible undermining mitigative measures from the TAC guidelines [Ref. 7]. The extent of the launching apron, or the volume of material included in the toe trench should be sufficient to cover the total abutment scour depths presented in Section 9.2.

As shown in Figure 9-4, the riprap should begin at least 0.6 m above the design high water level, the thickness should be at a minimum 1.5 times the D_{50}, or the D_{100} (whichever is thickest). In addition, the riprap slope should be 2H:1V or flatter, and the apron should be underlined with geotextile or a granular filter.

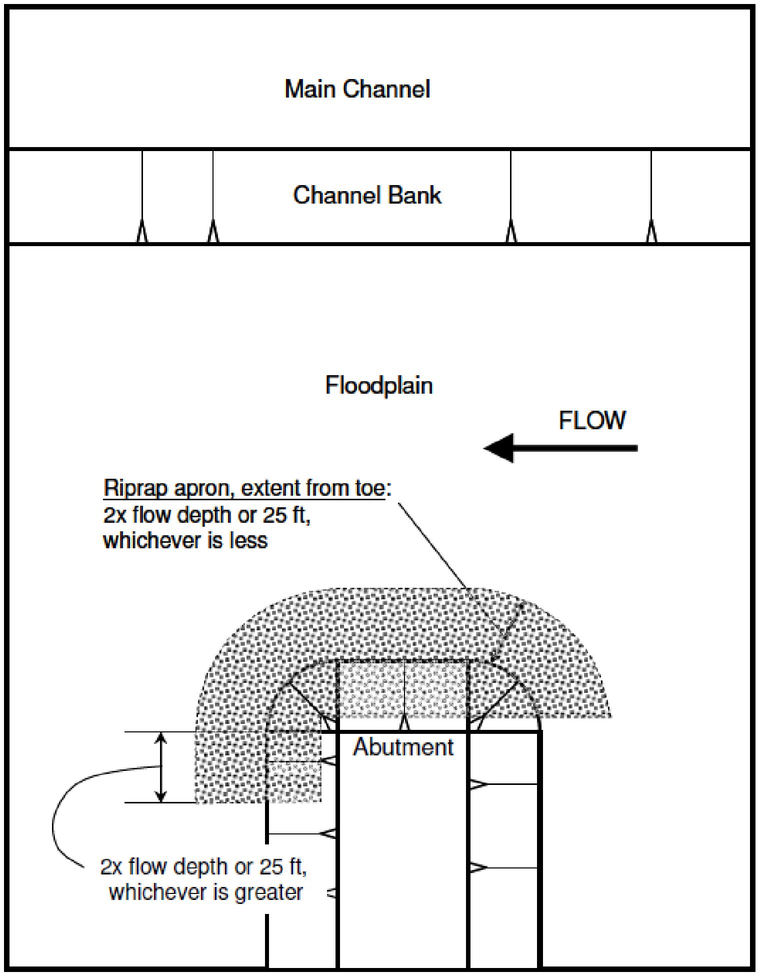

Figure 9-2: Plan View of the Extent of Rock Riprap Apron for Abutments (Figure 14.7 from Ref. 6)

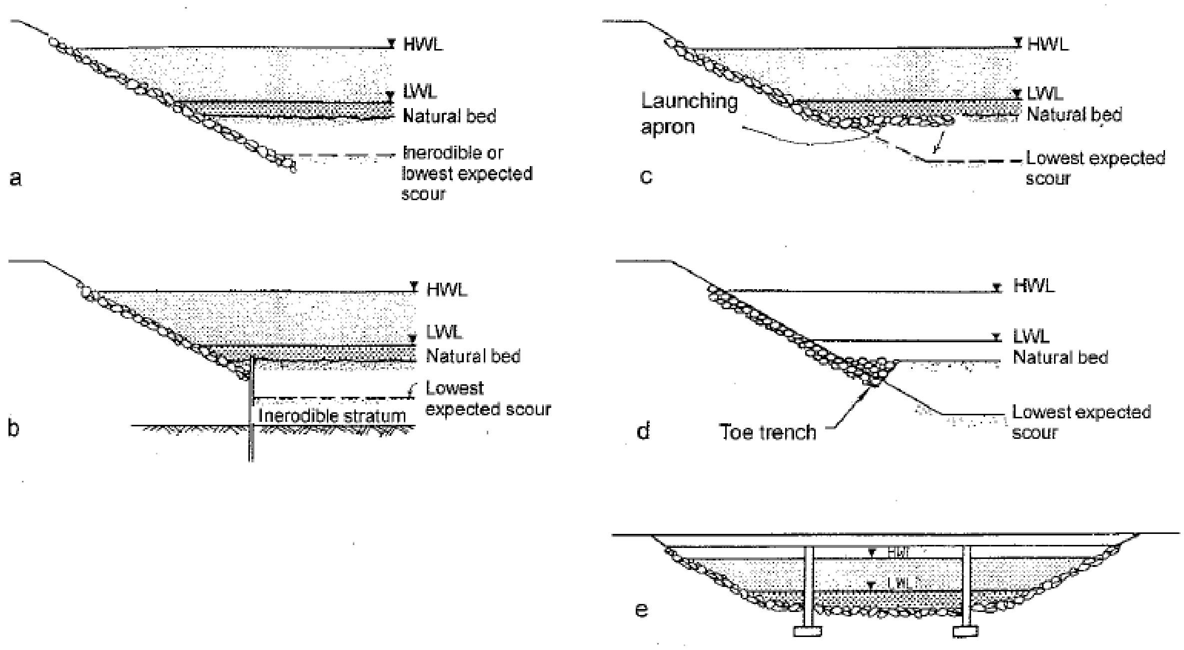

Figure 9-3: Alternative Methods of Protecting Against Undermining (Figure 5.7 from Ref. 7)

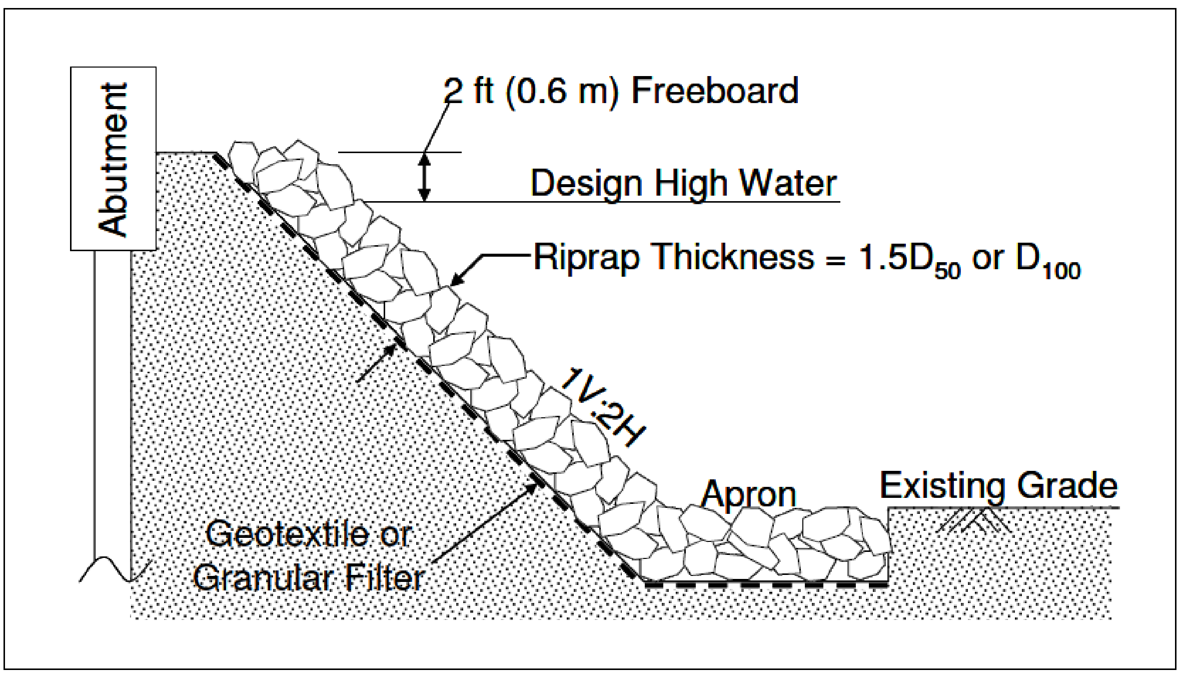

Figure 9-4: Typical Cross-Section for Abutment Riprap (Figure 14.8 from Ref. 6)

The riprap The sizing of rock riprap for the abutment aprons are based on the methodologies outlined in Design Guideline 14 from the FHWA HEC-23 manual [Ref. 6]. In addition, the minimum riprap size for ice considerations is 0.4 m (Section 8.2). Table 9-6 lists the minimum calculated D_{50} grain sizes for each of the four bridge crossings, which are governed by the ice considerations criteria. These D_{50} grain sizes are the minimum requirements, where larger riprap would be acceptable and would provide greater scour protection.

Table 9-6: Abutment Riprap Apron – Minimum Rock Size

Crossing	Minimum D50 Grain Size Requirement (m)
Bridge 1	0.4
Bridge 2	0.4
Bridge 3	0.4
Bridge 4	Not Required

Similar for the recommendations made for pier riprap, in lieu of the restrictions of available material, the riprap abutment protection for Bridges 1, 2, and 3 should be constructed with a Type C stone riprap (D50 = 0.61 m).

9.3.3 Riprap Monitoring

Periodic inspections of the riprap scour protections countermeasures should be performed to monitor the performance of the riprap, and identify if any maintenance is required. Guidance on such inspections can be found in the Federal Highways Administration (FHWA) – HEC 23 publication "Bridge Scour and Stream Instability Countermeasures", Section 5.21 (Riprap Inspection Guidance).

10. References

1) Hatch Ltd., "Baffinland Iron Mined Corporation – Mary River Project – Geotechnical Investigation Factual Data Report", Rev. 1, September 2018 (Hatch document # H353004-10000-229-230-0005).

2) Knight Piesold Consulting, "Baffinland Iron Mined Corporation – Mary River Project – Baseline Hydrology Report", Rev. 1, January 2012.

3) Knight Piesold Consulting, project memo "Updated Design Peak Flow Assessment", December 13, 2016.

4) Federal Highways Administration (FHWA) HEC-20 publication "Stream Stability at Highway Structures – Fourth Edition", publication No. FHWA-HIF-12-004, April 2012.

5) Federal Highways Administration (FHWA) HEC-18 publication “Evaluating Scour at Bridges – Fifth Edition”, publication No. FHWA-HIF-12-003, April 2012.

6) Federal Highways Administration (FHWA) HEC-23 publication “Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition (Volumes 1 and 2)” publications No. FHWA-HIF-09-111 and No. FHWA-HIF-09-112, September 2009.

7) Transportation Association of Canada (TAC) publication “Guide to Bridge Hydraulics – Second Edition”, June 2011

8) Ontario Ministry of Transportation (MTO) publication “Highway Drainage Design Standards (WC -2 Freeboard and Clearance at Bridge Crossings)”, January 2008.

9) Canadian Standards Association (CSA) publication “Canadian Highway Bridge Design Code”, 2006.

10) Knight Piesold Consulting, project memo “Updated Proposed Engineering Design Mitigation Measures for the Phase 2 Proposal”, February, 2018.

11) Hatch Ltd., “Environmental Design Criteria”, Rev. 1, July 2018 (Hatch document # H353004-00000-121-210-0001).

12) Project drawings H353004-35000-232_MARY RIVER (2018-11-15).

13) AREMA “Manual for Railway Engineering, Part 3 – Natural Waterways”, 2016.

14) AREMA “Manual for Railway Engineering, Part 5 – Retaining Walls, Abutments and Piers”, 2016.

15) AREMA “Manual for Railway Engineering, Part 23 – Pier Protection Systems”, 2016.