17 May 2021

Baxter Bridge No. 00539 Replacement

Final Hydrotechnical Design Report

McElhanney Consulting Services Ltd.
Suite 500, 3960 Quadra Street
Victoria BC, V8X 4A3

Contact: Ken Rebel, PEng
Project Manager
Phone: 778-746-7515
Email: krebel@mcelhanney.com

2241-02729-00

Final Hydrotechnical Design Report

for

Baxter Bridge No. 00539 Replacement Project

Prepared by:

PROFESSIONAL
PROVINCE
OF
J. H. MORLEY
BRITISH
COLUMBIA
ENGINEER

John Morley, PEng

Reviewed by: Ken Rebel, PEng

17 May 2021

Document Ref. No.: 2241-02729-00

Contents

Tables

Figures

Appendices

1. Introduction

The Ministry of Transportation and Infrastructure BC (the Ministry) retained McElhanney Consulting Services Ltd (McElhanney) to provide design services for the replacement of Baxter Bridge, spanning the Shuswap River. The project site is located approximately 11 km east of the intersection of Highway 97A and Cliff Road in Enderby. Baxter Bridge connects Enderby to rural communities, farms, and forest south of the river. See the red box in *Figure 1* for the project location.

The ~92 m long Baxter Bridge was built circa 1950, comprising two timber Howe truss main spans and five timber stringer approach spans. The substructure is made up of five timber pile bents and three timber pile piers. Along the existing alignment, the river channel is approximately 85 m across at normal water levels. The river gradient is relatively flat at about 0.0004 m/m and results in a slow flowing river that meanders along the valley bottom flanked by pasture lands.

During the larger summer freshets, the river overflows onto the adjacent flood plain and even during average years the depressions on the flood plain fill with water. At the bridge site the river channel is at the southern edge of the flood plain where the land rises steeply in a forested hill side whereas on the north side of the river there is pasture land. At the bridge site, where the river is flowing from the east, the drainage area is 4720 km^2; it is located along the eastern flank of the Monashee Mountains, that range up to an elevation of 2700 m in this area.

Figure 1: Project Location

2. Previous Studies at Bridge Site

2.1. Flood Plain Mapping

In 1986, the Ministry of Environment published Flood Plain mapping for the Shuswap River between Mara and Mable Lakes which includes the Baxter Bridge reach. This work designated the flood construction limits which is defined as the minimum elevation for habitable building space and includes a freeboard allowance 0.6 m above the 200-year flood level. At Baxter Bridge, the 200-year return period flood construction elevation was given as 357.8 m which is indicative of a 200-year return period water level of 357.2 m.

2.2. Ministry Bridge Sounding Programme

Periodically, the Ministry retains a contractor to take riverbed soundings around some of their structures and under this programme there have been at least three sets of soundings carried out at the Baxter Bridge since 1997. The sounding program results are summarised by Associated Engineering in their report dated February 2010. They indicate that the river bed through the bridge can aggrade and degrade by several metres between soundings and seems to be related to the flow magnitude at the time of the soundings; they do not recommend any measures to improve the stability of the existing structure but recommend continued monitoring.

2.3. Scour Evaluation Report

In 2016 Northwest Hydraulics Consultants (NHC) carried out a Scour and Erosion Assessment Report for the Baxter Bridge for the Ministry. The report estimates the 200-year return period flood flow at 700 m^3/s based on the records of Gauge 08LC002 and recommends a 10% flow increase to allow for climate change which results in an anticipated 200-year return period flood elevation of 358.2 m.

We have conducted an updated draft Scour and Erosion Assessment Report for the new Baxter Bridge with the information available which is attached as *Appendix A*.

3. Hydrology

Hydrological studies are required to determine the river flow volumes that should be used in the design of the proposed bridge. Design flows are usually calculated by statistical analysis of measured river flows. In the case of Baxter Bridge over the Shuswap River, it is fortunate that there is a Water Survey of Canada hydrometric station (08LC002) at the bridge site with flow records dating back to 1912. Although the flow record is not continuous, it has been possible to fill-in the missing records by reference to other gauging sites on the Shuswap River. The flow record of annual peak daily flows ranges from a low of 217 m^3/s in 1926 to a maximum of 626 m^3/s in 1928. It is possible that the 1948 flood was slightly larger than the 1926 flood, but the gauge was not active at that time, and there are also reports that an even larger flow of 654 m^3/s occurred in 1896.

Table 1 provides a summary of a statistical analysis of the records and shows the corresponding flow volumes for various return periods.

Table 1: Flows for Various Return Period Flow Events for WSC gauge 08LC002

Return period	River flow (daily flows, m^3/s)	Instantaneous flow (m^3/s) *
Mean Annual Flood	370.5	375
10-years	493.7	500
100-years	649.4	655
200-years	693.5	700

**Flows have been rounded-up*

The frequency analysis is based on peak annual daily flows; however, since 1991 the flow has been measured continuously thus it has been possible, since that time, to compare the mean daily flows with the instantaneous peak flows. For the ten largest flow years after 1991 the average difference, of 5 m3/s between mean daily and instantaneous peak flow was relatively small being only 1.3 % larger but was slightly greater (1.0 m3/s) than average difference for the full record. For the years where only mean daily figures were available 5 m3/s was added to the daily flows to give instantaneous values. *Appendix B* contains hydrometric data for Gauge 08LC002 together with the results of the frequency analyses. The analyses were calculated using a program called "Hydrosoft", developed at the University of Alberta. The software performs the analysis by the Gumbel Extreme Value, the Lognormal Type 3, the Pearson Type 3, and the Log Pearson Type 3. The presented values are the average of the four methods.

4. The Effect of Climate Change on River Flows

The impact of climate change has been studied in detail by the Pacific Climate Impacts Consortium who have related changes in precipitation patterns and temperate changes to specific regions of the province for various time horizons. For the Columbia—Shuswap area, it is anticipated that for the period to 2080 annual rainfall quantities will increase by between 2% to 14% while winter snowfall quantities will decline by an average of 7%, there may also be a 2.7% increase in temperatures. These statistics are given in Figure 4 and 5 in the Highway Infrastructure Climate Change Report prepared for this project. Study of the river flow records seem to indicate that peak flows have slightly decreased over the last few decades and that river freeze-up periods have decreased in both duration and frequency.

The most significant impact of climate change may have begun only in the recent years with the increasing prevalence of forest fires associated with hot dry summers. Snow falling on tree cover has increased evaporation rates, compared to snow falling on the ground, and results in decreased snowpack accumulation; it also has a dampening effect on the rate of snowmelt. If the treeless area in the drainage basin is increased, it may cause river flows to be more erratic with higher peak flows and longer low-flow periods. In a drainage basin of 4720 km^2, the largest burn area might represent only a minor proportion of the total area but successive fires over a period of years might eventually cover a significant portion of the drainage basin. Although it is not possible to give a precise estimate of how peak flows will be affected by climate change it is recommended that the flow estimates based on historical records be increased by 15% to accommodate this possibility.

More details of the effects of climate change are available in the report Highway Infrastructure Climate Resilience Report (21st October 2020). Table 2 lists flows of various return periods including the 15% adjustment for climate change.

5. Hydraulics

5.1. Water Levels

The hydraulic studies will determine the water surface levels at the bridge site, for various flood flows, together with flow velocities, scour depths and erosion potential. The surface water profiles and velocities are generally calculated by means of a flow model such as Hec-Ras published by the US Corp of Engineers. The modelling procedure with a one-dimensional model such as Hec-Ras involves successive calculations of the total flow energy at a series of surveyed river cross-sections; by deducting the various head losses between cross-sections the water surface elevations can be determined. The procedure requires a known stage / discharge or water surface gradient measurement at some point along the channel.

In the case of the Baxter Bridge, the Water Survey of Canada Hydrometric Station 08LC002 is located just a few metres downstream of the existing bridge; before it became a fully automated station, it was a staff gauge attached to the bridge. The stage / discharge relationship for the gauge has been developed by field measurements of the discharge at different water surface elevations and is constantly under revision. The stage / discharge relationship can be extrapolated to give the elevation of the estimated flood flows.

Figure 2 provides the stage / discharge graph for Gauge 08LC002 composed from a selection of the published flows and discharge data. The data can be closely represented by the following function which was fitted to the stage / discharge data using least squares in Microsoft Excel:

$$y = 0.4449 * x^{0.3895}$$

where: x = flow (m^3/s)

It is to be expected that this type of function would be appropriate as it is compatible with open channel flow equations such as Manning's equation (shown below) where it is evident that flow depth is a power function of discharge.

$$Q = \frac{1}{n} * A * R^{2/3} * \sqrt{S}$$

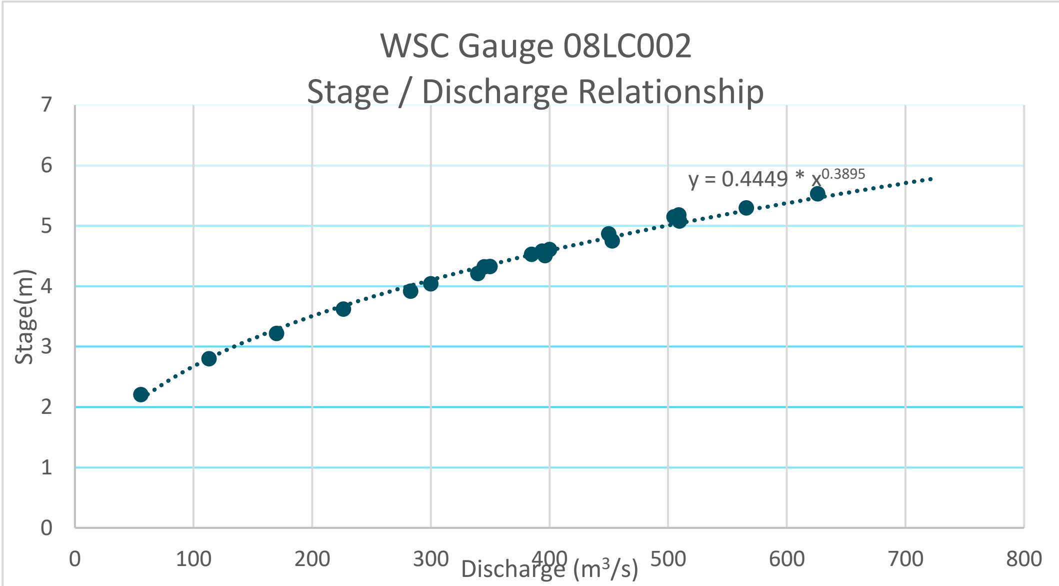
where: Q = flow rate (m^3/s)

A = flow area (m^2)

n = Manning's roughness coefficient

R = hydraulic radius (m)

Q = channel slope (m / m)



y = stage (m)

Figure 2: Stage / Discharge Relationship for WSC Gauge 08LC002

Table 2 shows the water elevation at the location of new bridge during the 10-, 100- and 200-year return period flood flows. The datum (i.e. gauge zero) for WSC Gauge 08LC002 was recently established as elevation 351.00 m Canadian Geodetic Vertical Datum (1928) (CGVD28); which is also the datum used for the project survey.

It is important to note that the most recent published datum for Hydrometric Station 008LC-002 is based on the latest 2011 GSC system, which is approximately 0.3 m higher (in the Enderby area) than the 1928 GSC datum upon which both the project survey and the official flood plain mapping is based.

Table 2: Water Elevation at New Baxter Bridge at Various Flow Events

Return period of flow	Design Flow (m^3/s) *	Gauge reading (m)	Water surface elevation (m)
Mean Annual Flood	431	4.72	355.72
10 years	575	5.28	356.28
100 years	753	5.87	356.87
200 years	805	6.02	357.02

** The design flows include a 15% increase to allow for climate change and were obtained from the stage/discharge relationship for WSC Gauge 08LC002.*

The calculated 200-year flood elevation is approximately 0.21 m lower than that given in the 1986 Flood Plain Mapping report (see *Section 2.1*). It is thought that this difference is due to an improved curve fitting of the stage / discharge data (i.e. a power function relationship versus a linear relationship). The data plot has a whale-back configuration and if a straight line is fitted, the linear equation will over-estimate the values at the upper and lower ends of the curve. This is even more pronounced when it is projected beyond the data range to determine the 200-year flood elevation.

It is possible for the stage / discharge relationship for a river gauge to change over time due to scour or aggradation of the riverbed, Water Survey staff periodically confirm the accuracy of the stage / discharge relationship by measuring the average river velocity at various stages.

The full flow record for WSC Gauge 08LC002 was obtained and the data for different time periods studied to establish whether the river levels might have changed; it was evident that at this location the river stage versus flow relationship has been remarkably stable for many years.

5.2. Sensitivity to Flow Increases

It can be seen from *Table 3* that at high flows the river surface elevation is relatively insensitive to quite large flow increases. For instance, increasing the Q_{200} flow of 700 m^3/s by 15% to 805 m^3/s only increases the water elevation by 300 mm and the average velocity 0.12 m/s. All water levels at the bridge were calculated from the stage/discharge relationship for WSC gauge 08LC002, but the corresponding river velocities were taken from the Hec Ras model.

Table 3: Flow Increases and Resulting Water Surface Elevations and Velocities

Flow magnitude (m^3/s)	% increase	Water surface elevation (m)	Average velocity (m/s)
700	0	356.71	1.44
735	5	356.81	1.47
770	10	356.92	1.51
805	15	357.02	1.54
840	20	357.12	1.57

5.3. Flow Modelling

The Hec-Ras open channel flow model was used to calculate river surface profiles through the bridge site for a range of river flows; the effect of the new structure on water levels and velocities could thus be assessed. The modeling also showed the extent of over-bank flows upstream of the bridge.

Calibration of the Hec-Ras model was achieved by trial and error using various combinations of river roughness and river gradient until the appropriate water survey gauge reading was matched; Manning's 'n' was found to be 0.035 in the channel and 0.04 on the overbank areas with a hydraulic gradient of approximately 0.0005 mm / mm at the bridge site.

The north bank of the river upstream of the bridge starts to be inundated when floods greater than the 10-year return period occur and during the 200-year event the overbank flow would be up to 150 m wide.

At the bridge site, the overbank flow would be forced back into the main channel by the northern approach fills which could cause increased erosion in the channel at the abutment. The modelling also facilitated a method of evaluating the scour potential at the bridge; by calculating the average velocity in the channel at assumed scour depths and comparing them with the recommended acceptable velocity for the particle size of the bed sediments. Output from some of the model runs are presented in *Appendix C*.

6. Scour

River soundings taken at the existing bridge over several years have shown that the bed level can vary by several metres with the bed level at any time, probably dependent on the magnitude of the recent flows. It is generally recognised that scour at bridge sites can be ascribed to several causes:

Natural Scour—is the scour that occurs due to periodic variation in the conditions in the river system, such as:- increasing or decreasing flows; changing bed forms and bed materials; natural obstructions; and changes to bed sediment transport loads

Contraction scour – can be a natural effect due to the bridge being located at a narrow point in the channel or it could be an artificial narrowing caused by the bridge approaches cutting off overbank flows.

Local scour – is due to turbulence created by bridge piers, abutments, and other local impingements into the flow.

The most recent survey of the river bed, taken in March of 2018, gives the lowest bed elevation at the existing bridge at approximately 350.0 m GSC with an average elevation of approximately 351.0 m.

Because the survey was taken at the end of the winter low-flow period it is assumed that any further scour would be below the 350.0 m elevation.

6.1. River Bed Materials

Several historic geotechnical investigations have been completed at the site of the existing bridge that give an indication of the river bed sediments, including boreholes drilled through the deck near the centre pier and the north pier supporting the main spans of the existing bridge. The historic data was supplemented with the results of the 2018 drilling program conducted by Thurber Engineering Ltd. as part of this project. The results of these investigations are summarised in *Table 4*.

Table 4: Borehole Information

Name	Location	Depth / Elevation (m)	Description
TH 18-1	Approximately 30m north of existing north abutment	0 – to –14.0 14.0 – to – 30.18 (end)	Loose to compact sand / gravel Firm to stiff silty clay
TH 18-5	Approximately 60m downstream of existing south abutment	0 – to – 1.52 1.52 – to – 3.05 3.05 – to – 5.49 5.49 – to – 6.1 (end)	Silty sand / clay (fill) Silty clay Clayey sand / sandy clay Very stiff silty clay

Name	Location	Depth / Elevation (m)	Description
TH 18-6	Approximately 40m south of existing south abutment	0 – to – 6.1 6.1 – to – 30.18 (end)	Very stiff silty clay Stiff silty clay
TH 95-01	Near existing south abutment	358.5 – to – 356.2 356.2 – to – 354.3 354.3 – to – 343.6 (end)	Loose gravel Firm clay (olive/brown) Firm clay (dark grey)
TH 95-02	Near north pier supporting a main truss span of existing bridge	353.5—to – 348.3 348.3—to – 344.7 344.7—to – 343.7 (end)	Loose gravel Compact sand Compact gravel & boulders
CPT 11-1	3m south of centre pier of existing bridge	351.5—to – 349.4 349.4 – to – 347.9 347.9 – to – 346.4 (end)	Sandy Gravel – sandy Silty clay

In the 2016 Scour and Erosion report for Baxter Bridge by NHC the river bed material at the bridge site was classified as loose gravel with a 50% size of 18 mm diameter, which could be the same as the granular material below elevation 349.4 m in borehole CPT 11-1 taken near the centre of the channel.

6.2. Scour Depth Estimates

Existing Bridge

One method of estimating potential contraction scour depth is to assume various depths of scour and use the Hec-Ras model to see how this affects the average flow velocity; comparing the scour potential of the river bed material with the calculated velocities can give a good indication of the extent of the scour before the velocity is too low to cause bed movement. A graph of bed material size versus the average river velocity that will cause the material to erode is presented in the TAC Guide to Bridge Hydraulics (December 2000), see *Figure 3*. The sediment size at the bridge site has been added to the graph.

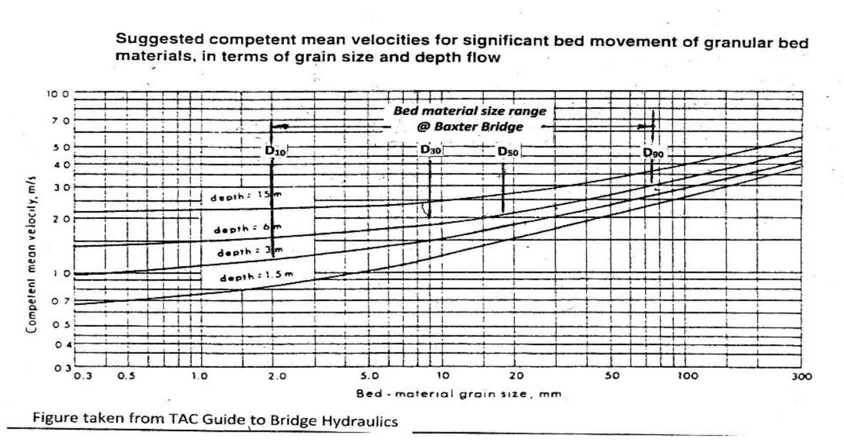


Figure 3: Suggested Competent Mean Velocities for Bed Movement (from TAC Guide to Bridge Hydraulics)

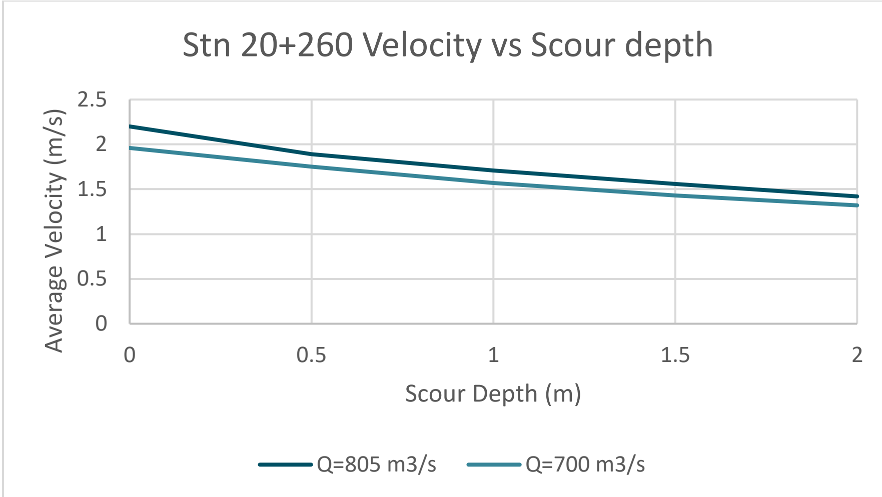


Figure 4: Existing Baxter Bridge Flow Velocities for Various Scour Depths

The average velocity at the existing Baxter Bridge during the 200-year flow (with and without the 15% climate change increase) for various depths of scour is plotted in *Figure 4*. The Hec-Ras model was run several times with the riverbed lowered in increments of 0.5 m to allow the effect of the lowered bed on the average velocity to be determined.

A comparison of *Figure 3* and *Figure 4* shows that there could be bed erosion until the average velocity is reduced to less than 1.5 m/s, it would probably require an average of one metre of erosion to achieve this velocity.

In the velocity calculations it was assumed that erosion would be evenly distributed across the channel, but experience shows that erosion is usually unevenly distributed with the deepest point being approximately twice the average depth. Since the deepest erosion could be located anywhere across the riverbed, an envelope of maximum contraction scour would be two metres below the normal riverbed elevation of 350.0 m (i.e. 348.0 m) at the existing bridge location.

In the 2016 Scour Evaluation Report for Baxter Bridge by NHC they adopted a different approach to estimating scour depth by using the river regime equations developed by Blench (1966). The NHC analysis also estimated the maximum contraction scour depth to be elevation 348.0 m.

New Bridge

The proposed new Baxter Bridge is located approximately 40 metres upstream of the existing bridge on an alignment that is skewed to the river center line. At this location, the channel is approximately 30% wider than at the existing bridge and the average river-bed elevation approximately 1.0 m higher at elevation 351.0. The new bridge will be longer than the existing structure with the south abutment outside the wetted perimeter and with the north abutment above the average flood level. Calculations of river velocity show that at the new location there will not be any contraction scour even during the extreme flood events but there is the potential for natural scour due to the varying bed load inputs from the steep tributary streams. being transported down river.

6.3 Natural Scour at new Bridge

Due to the length and height of the new bridge, its impact on riverbed levels, because of contraction scour, should be minimal. However, the Shuswap River experiences a wide range of flows and has many steep tributaries that provide a continuous but variable quantity of bed load supply. At the existing bridge periodic riverbed soundings have shown that the bed elevation can vary by several metres. For the new bridge Blench's regime equation shows that natural scour could cause bed lowering of approximately 0.5 m to elevation 350.5 m.

Natural scour estimated by Blench's Regime equation is as follows:

$$y_s = (q^2/F_{b\varnothing})^{0.333} * Z$$

Where: y_s = Scour depth below river surface,

q = River discharge divided by average flow width. (805 $_{m3/s}$ / 90 m = 8.94 $_{m2/s}$)

$F_{b\varnothing =}$ Silt factor; 1.1 for bed material of 18 mm diameter

Z = A factor that depends on character of the river. In the case of the Baxter bridge reach a relatively straight, single channel, assumed to be 1.5; this Z value was confirmed by assuming the current bed elevations were the result of the 10 year return period flood.

For the above conditions, the depth of scour would be approximately 6.5 m below the 200 year Return Period flood elevation giving a scoured bed elevation of 350.5 m. The values of the various factors in the Blench equation were taken from the TAC Guide to Bridge Hydraulics

6.4 Contraction Scour at New Bridge

The proposed new Baxter Bridge is located approximately 40 metres upstream of the existing bridge on an alignment that is skewed to the river center line. At the new location, the river channel is approximately 30% wider than at the existing bridge and the average river-bed elevation approximately 1.0 m higher, at elevation 351.0. The new bridge is longer than the existing structure with the south abutment outside the low flow wetted perimeter and with the toe of the north abutment fill above the low water elevation. Calculations with the Hec-Ras model show that the new bridge will not have a significant effect on river velocity and will not cause contraction scour.

6.5 Local Scour at New Bridge

Local scour, when it occurs, is in addition to any Natural scour and Contraction scour and in the case of the new bridge local scour would be measured below elevation 350.5 m

.

6.5.1 Scour at Bridge Piers

The depth of scour at bridge piers is a function of the pier width and shape plus how well the pier is aligned with the flow. At the new Baxter Bridge there will be two piers located in the river channel and they will be aligned with the river flow; each pier will consist of a single row of 914 mm diameter pipe piles. There will be a concrete diaphragm wall between the piles down to the low water elevation.

The TAC Guide to Bridge Hydraulics recommends using Laursen's equation for calculating local scour at bridge piers which relates scour depth (d_s) to the pier width (b) and depth of the approach flow (y). Laursen's equation, $ds = b \times 1.5 \times (y/b)^{0.3}$, is empirical and is based on scour measurements taken at many bridges. For the case of Baxter Bridge, where y/b is 6.56, the estimate of local scour adjacent to the piers is 2.4 m and would be below the riverbed level after any contraction or general scour. The minimum bed elevation at the bridge piers including natural scour and local scour would be 348.1 m.

6.5.2 Abutment Scour

South Abutment

The south bridge abutment and the toe of the approach fill is located above the 200-year flood level and therefore will not cause any local scour in the river channel. However, the river bank below the abutment could be susceptible to erosion from either runoff from the road surface or from the river if either ice or debris caused the flow to impinge upon the river bank; it should therefore be protected with riprap.

North Abutment

The north abutment is a spill-through type and will have the toe of the at, or slightly below, the river level during the low flow period, during high flows the bridge approach road will cut-off the flow along the north flood plain and direct it through the bridge opening. The flow retuning to the main channel could cause scour along the abutment fill but will be partially mitigated by construction of a spur dyke set back from the river edge, at approximately the 10 year return period river level, this will ensure that the excess flows re-enter the river upstream away from the bridge. There will still be a potential for scour several metres deep along the toe of the north abutment fill where it projects into the channel in front of the spur dyke, but this will be prevented by a riprap apron around the toe of the fill slope.

6.6. Erosion and Scour Protection

Although the river velocity through the new structure is relatively low, it is recommended that riprap protection be installed along both abutment fills. The river velocities suggest that a smaller size of riprap would provide sufficient protection, However; it is the Ministry's policy that in areas where there is a potential for vandalism of the riprap (i.e. theft) that the riprap size be a minimum of 100 kg class, with a thickness of 700 mm. Therefore, the proposed embankment protection recommended below may have to be upsized to comply with this policy.

At the south abutment, the riprap should extend 5.0 m, upstream and downstream, beyond the toe of the abutment fill (or abutment wing walls) and the edge of the adjacent pier. The riprap should be at least 50 kg class riprap 550 mm thick (see note above regarding the Ministry's policy on minimum riprap size) and should extend from elevation 352.0 m to elevation 357.62 m, which is 0.6 metres above the 200-year water level. The riprap slope should be a maximum of 2.5H:1V (based on geotechnical recommendations) and be laid on a 300 mm thick filter layer with the toe keyed into the bank a minimum of 3.0 m.

On the north bank the protection for the abutment fill should be at least 50 kg class riprap 550 mm thick (see note above regarding the Ministry's policy on minimum riprap size). The riprap slope should be a maximum of 2H : 1V (based on geotechnical recommendations) and be laid on a 300 mm thick filter layer. On the upstream side of the fill the riprap should extend 5.0 m beyond the bridge abutment wall, and on the upstream side of the fill it should extend 3.0 m beyond the north edge of the spur dyke along the roadway embankment. Around the toe of the abutment fill an apron of 10 kg riprap 3.0 m wide and 350 mm thick will be placed. The area between the 3.0 m wide apron and the low water river edge will be covered with a 300 mm thick layer of 150 mm (minus) layer of rounded gravel.

A spur dyke (or berm) should be constructed upstream of the road fill parallel to the river; it should extend approximately 50 m from the road centre line and should be located at the top of the bank near the fence line. The top of the dyke should be 4 m wide with an elevation of 357.6 m, the side slopes should be relatively flat at about 3:1 and the dyke should be grassed. Most spur dykes associated with bridges have the river-side face aligned with the face of the abutment fill with the purpose of preventing lateral movement of the river channel; however, at Baxter Bridge the spur dyke is to divert overbank flow away from the structure and will not be subjected to high velocity flows and, if well vegetated, does not require rock protection. The spur dyke details are shown in *Appendix D*.

The assumed scour depth of elevation 348.1 m at the bridge piers is not a critical factor in the stability of the proposed structure and as such riprap is not required at the piers.

7. Ice Forces

Over the past 25 years the occurrence of ice at the bridge site has decreased significantly both in number of incidents and the duration that the ice lasts, this can be seen in *Figure 5* below.

The magnitude of the ice forces on a bridge pier depend on the thickness of the ice and the hardness of the ice. A study of ice thickness in Canadian Rivers carried out by WTR Allen (1976) for the Department of Fisheries and Environment Canada gives the typical mean maximum ice thickness as 250 mm in this area of the province.

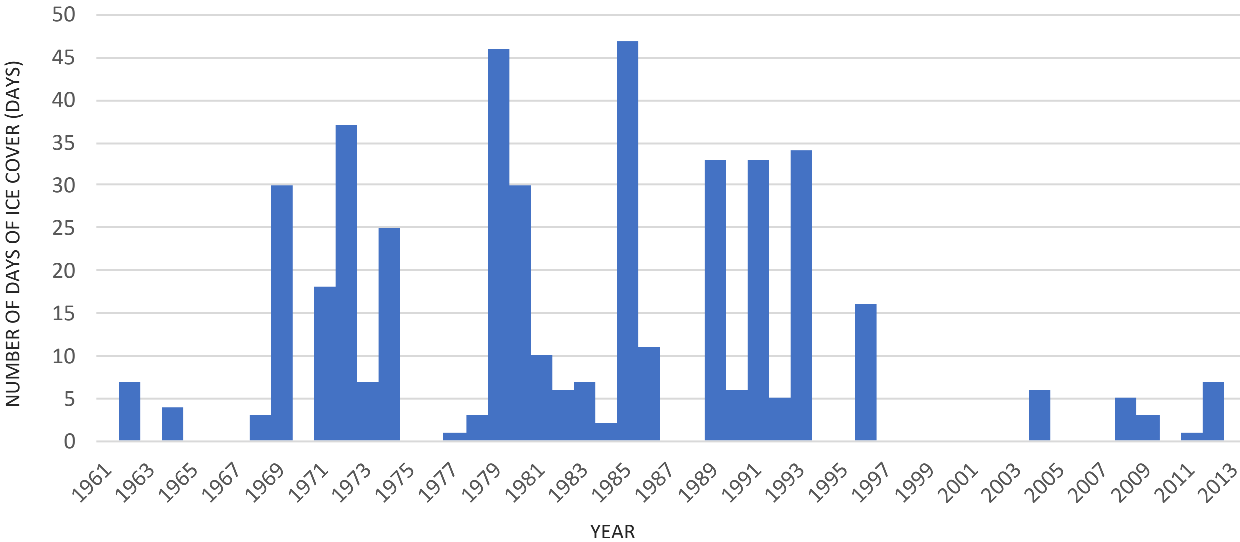


Figure 5: Number of Days of Ice Cover on the Shuswap River

According to Allen, the ice would typically be deteriorating by March 15th and completely gone by April 1st; this is well in advance of the spring / summer freshet and is when the river level is near its minimum flow. A review of water levels at the end of March indicates that at this time a conservative estimate of the elevation would be 353.0 m. As the ice is melting in-place it is most likely to have a low crushing strength but because it could be in the form of ice sheets, rather than broken into small pieces, it is recommended that the ice crushing strength be 1100 kPa (as per CAN/CSA S6-14 CHBDC).

Given the expected ice thickness and low crushing strength, ice jacking of the pier diaphragms is not anticipated to be an issue.

7.1. Ice Jams

Ice jams in rivers can cause rapid increases in upstream water levels that are not necessarily related to large flow volumes. The jams are caused when floating ice becomes hung-up at narrow sections in the channel; typically, at bridges with inadequate clear-span lengths. The existing Baxter Bridge, with a wide central pier, would be susceptible to ice jams. The proposed new bridge will have a long central span of 54 m with two shorter 40 m approach spans and will be much less prone to ice jams than the existing structure. Because there are no structures on the upstream flood plain short-term inundation due to ice jams is not perceived as a problem.

8. Floating Debris Forces

During the larger flood events when the flood plain becomes inundated and bank erosion is more active there will be considerable logging debris in the river that could become snagged on the bridge piers.

The new bridge, with wider spans and more slender piers than the existing bridge, should result in fewer problems with debris jams. However, since there will still be a possibility of debris jams it should be addressed in the structural design.

Debris jams are likely to occur during periods of extreme high water; it is recommended that the pier design accommodate a raft of logging debris 10 m wide by one metre thick with a velocity of 1.54 m/s (from the Hec-Ras model) applied at the 200-year flood elevation of 357.02 m. Major debris accumulations are most likely to occur during the annual freshet and are therefore predictable. Debris rafts wider than the recommended 10 m are possible but since they are predictable and the result as a steady accumulation, (rather than an instantaneous event) the debris can be easily managed from the road surface and it is anticipated that extremely large debris accumulations can be avoided.

9. Navigation Clearance

Navigational clearance for river traffic at bridges is determined by the Canadian Navigable Waters Department and is usually set above the 100-year return period flood elevation, which is 357.15 m at the proposed Baxter Bridge. For recreation traffic, the required minimum navigation clearance between the water surface and the bridge soffit is often set 1.7 m, which equates to elevation 358.85 m at the new bridge. Although this elevation is marginally higher than the Q_{200} elevation plus freeboard, the navigational clearance requirements did not govern as the navigation clearance would be required near the centre of the bridge where the soffit is approximately 2.0 m higher than at the north end of the structure.

10. Construction Period Water Levels

Construction of the bridge piers and other works will require working in or close to the river. To assist the contractors in planning the instream works, Table 5 and *Figure 6* are provided to show typical water levels at the bridge throughout the year. It should be remembered when using *Figure 6* that except for the year 2011 the curves represent data from separate annual events and are not actual flow hydrographs.

If temporary construction platforms are constructed in the river and are to be in place during the spring freshet, they should be high enough to be above an appropriately sized flood event. A flood flow often used for construction purposes is the 10-year return period event which has an elevation of 355.91 m at the bridge site (this is slightly higher than the monthly values given Table 5 and does not include an allowance for climate change). An appropriate clearance above the 10-year water level should be adopted, 1.0 m is often used, but the actual construction level should be the responsibility of the contractor building the bridge.

Table 5: Water levels for mean and 10 year return period flows for each month

Month	Monthly water level (m)	10 year RP water level (m)	Month	Monthly water level (m)	10 year RP water level (m)
January	352.9	353.1	July	354.8	355.5
February	352.9	353.1	August	353.6	354.2
March	353.1	353.5	September	352.2	352.6
April	352.9	354.3	October	353.1	353.6
May	355.0	355.5	November	353.2	353.6
June	355.3	355.8	December	353.0	353.3

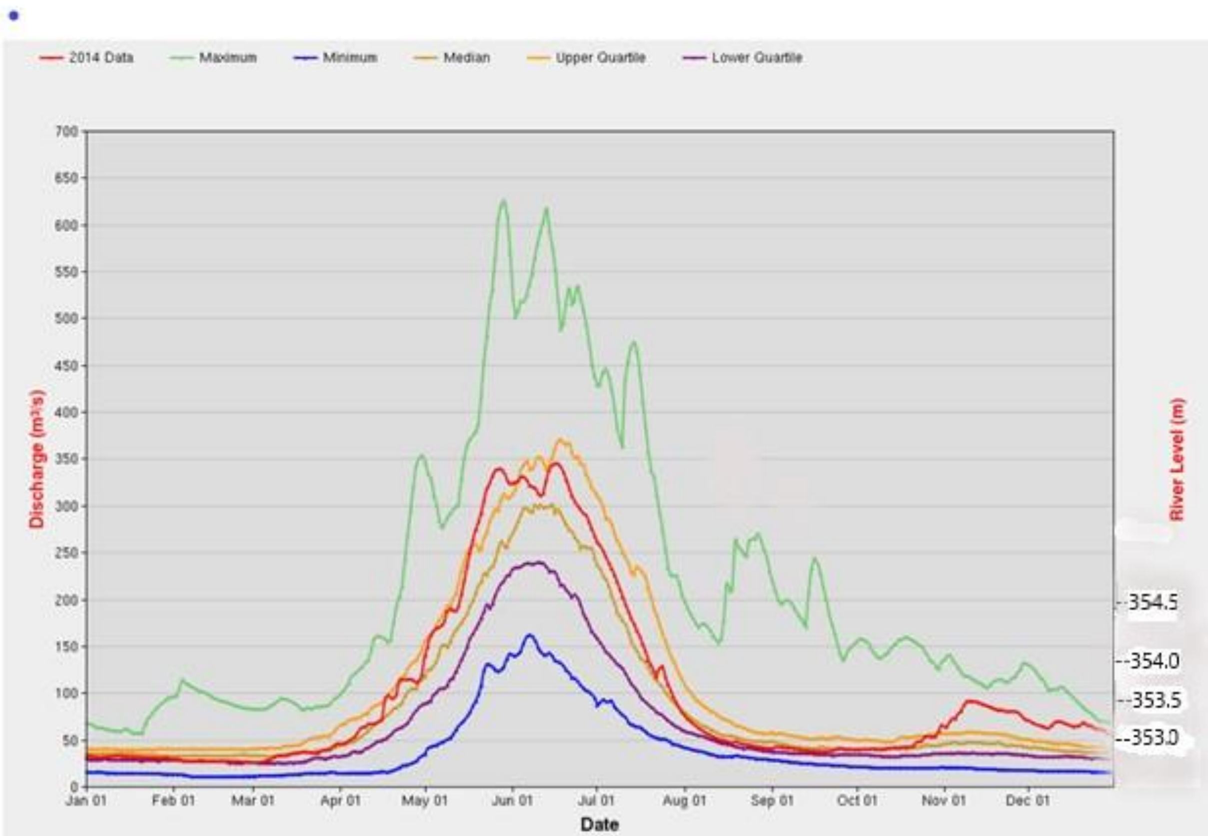


Figure 6: Daily Discharge Graph for Shuswap River near Enderby (08LC002)

11. Summary of Information for Bridge Design

- 200-year flood elevation at bridge (including 15% climate change) – 357.02 m.
- 100-year flood elevation at bridge (including 15% climate change) – 356.87m.
- Recommended minimum clearance between 200-year flood to bridge soffit – 1.5 m or El. 358.52 m.*
- Required navigational clearance between 100-year flood to bridge soffit – 1.7 m or El. 358.57 m.*
- Average river velocity through bridge during the 200-year flood – 1.54 m/s.
- Maximum scour depth elevation at bridge piers – 348.1 m.
- Maximum ice thickness 250 mm at elevation 353.0 m.
- Ice crushing strength 1100 KPa.
- Debris Loading – 10 m wide raft of debris, one metre thick with a velocity of 1.54 m/s at elevation 357.02 m.
- All riprap should be 100 kg class riprap 700 mm thick and should extend from elevation 352.0 m to elevation 357.6 m, which is 0.6 metres above the 200-year water level and be laid on a 300 mm thick filter layer.
- At the south abutment, the riprap should extend 5.0 m upstream of the edge of abutment and 5.0 m downstream beyond the projected line of the pier. The riprap slope should have a maximum slope of 2.5H: 1V (based on geotechnical recommendations) and have with a toe key 1.0 m deep.
  The north bank abutment fill, at a slope of 2:1, will be protected with 100 Kg riprap, 700 mm thick, laid on a 300 mm thick filter layer; it will have an apron of 10 kg riprap 3.0 m wide and 350 mm thick arounds its toe The area between the apron and the low water river edge will be protected with aa 300 mm thick layer of 150 mm (minus), rounded, gravel and cobbles..
- A spur dyke (SD) on the north riverbank will project upstream from the road fill for 50 m, from the road centreline. The SD has a top width of 4.0 m, 3:1 side slopes and top elevation of 357.6 m; the river-side face should be a continuation of the natural riverbank. No riprap protection is required on the spur dyke if it is seeded with appropriate vegetation.

* <u>Note:</u>
The recommended 1.5 m clearance above the 200-year flood elevation is measured at the lower, north end of the bridge whereas the navigation channel is at the centre of the bridge, where the soffit elevation is considerably higher. The 200-year flood elevation will therefore govern the bridge height.

Appendix A – Scour Evaluation Report

BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE
SCOUR/EROSION EVALUATION REPORT

Report By: McElhanney Consulting Services Ltd. J Morley P.Eng	**Report Date:** 2019-Jan-17

Str No:	**Str Type:**	**Str Name:**	**Watercourse:**	**Road Description:**
00539	Bridge	Baxter	Shuswap River	Trinity Valley
Road Type:	**Road Class:**	**Region:**	**District:**	**Contract Area:**
UCV	4	2-Southern Interior	5- Okanagan Shuswap	13- Okanagan Shuswap

1 STRUCTURE DESCRIPTION

Bridge 00539 will be a new steel and concrete structure 134 m long with 3 spans (2 x40m and 1 x 53 m) and two piers located in the river. It will be skewed at 30 degrees to the river channel and will be located approximately 40 m upstream of the existing wooden structure. The south bridge abutment will be founded on steel piles on the riverbank above the 200 year flood elevation; the north abutment will be founded on steel piles but the bridge end fill will spill through the abutment at 2:1 slope with its toe at about the normal highwater level. The bridge will have a gradient of 3.5% with the lower, northern end, beam soffits 1.5 m above the 200 year flood elevation. The final elevation of the pile tips have yet to be determined but are estimated to be at least 40 m below ground surface to provide the necessary levels of settlement. The piers will consist of a single row of 914 mm diameter of 4 steel piles with similar penetration as the abutment piles; there will be concrete diaphragms between the piles to low water elevation.

Year constructed: 2020?	**Length (m): 133**	**Spans (m): 2 x 30 & 1 x53**

Foundation type: 2 x piers of 4 x 914 mm dia steel piles, embedment Approx 40 m

Foundation Soils: ☐ **Known** ☐ **Unknown**

Ballast/wingwalls: **Known** **Unknown** concrete walls on steel piles, 40 m embedment

Counter Measures: ☐ **Yes** **No** riprap at both abutments

2. SCOUR/ EROSION SUSCEPTABILITY SUMMARY

The new bridge will be located on a relatively straight reach of the river where the lateral migration has been halted by the higher ground to the south, although the river surface width at normal flows is in the 70 to 90 m range, but at the new bridge the width is nearer to the higher value. Along the north bank, where there is flood plain several hundred metres wide, the bank material is gravel while on the south side steeply the steep rising bank is a cohesive soil which is erosion resistance. Because of the greater channel river width through the new structure contraction scour will not occur, however it is likely that the bed elevation will fluctuate as the sandy/gravel bed load is deposited and eroded with the varying river flows. Local scour at the new 914 mm diameter piers will be at maximum about 2.4 m below the prevailing bed elevation. With the new structure regularly scheduled hydrographic surveys will not be necessary.

<table>
<tr><td>Susceptibility Category:
Very low</td><td colspan="3">Screening Indices Scores:</td><td rowspan="2">Weighted Score:</td></tr>
<tr><td>Damage Risk Index: N/A</td><td>Watershed:</td><td>Reach:</td><td>Local:</td></tr>
</table>

Field Inspection: **Yes** **No**	**Detailed Evaluation:** **Yes** **No**	**Detailed Plan of Action:** **Yes** **No**

Plan of Action Summary:
No action as bridge yet to be built

Pictures of bridge when built

(See general arrangement for bridge)

3.MINISTRY BRIDGE MANAGEMENT INFORMATION SYSTEM UPDATES												
BMIS Items	**Most Recent Rating:** N/A						**Recommended Updates**					
	E	**G**	**F**	**P**	**V**	**X**	**E**	**G**	**F**	**P**	**V**	**X**
1. Debris Risk												
2. Channel												
3. Erosion Protection												
4. Substructure Scour												
ITEMS TO OBSERVE AT THIS SITE DURING MONITORING AND INSPECTION												
1. Possible erosion around spur dyke and adjacent river bank												

4. SCREENING INDICES ASSESSMENT (Scores: 0 = no susceptibility, 10 = high susceptibility)	score	out
A. Human Influences	3.0	10
Rationale: Moderate agricultural development along lower reaches of river, Logging higher in the watershed probably dampened by Mabel Lake		
B. Natural Geomorphic Hazards	1.0	10
Rationale: No significant hazards in lower river reaches except possibly debris torrents on the steeper tributaries may cause infusions of logging debris and bed load.		
C. Flood Severity	5.0	10
Rationale: Snowmelt flood regime with rare minor peaks in Fall due rainfall. Climate change and forest fires may increase flood peaks in the long term.		
D. Channel Type and Lateral Instability	2.0	10
Rationale: Flat river gradient keeps velocities low in the reach containing the bridge, gravelly flood plain soil on the north bank and cohesive soils on the south bank result in little lateral channel movement'		
E. Degradation Potential	1.0	10
Rationale: Review of the stage/discharge relationship for the nearby WSC gauge show no long-term changes over the 110 years of record; an indication that the bed/water surface elevation is relatively stable.		
F. Erosion Protection	3	10
Rationale: The river velocities are relatively low but the newly placed north abutment fill could be vulnerable to erosion. The river bank below the south abutment could also be vulnerable to erosion if debris should accumulate around the adjacent river pier. Nominal riprap protection is recommended at both abutments		
G. Natural Scour Potential	1.0	10
Rationale: Local scour at an upstream bend and scour through the narrows at the existing bridge indicate that scour of several metres is possible in the granular bed material. The wider than average section at the new bridge and the proposed extreme penetration depth (40 m) of the pilings indicate that degradation will not be an issue at the new bridge.		
H. Contraction Scour Potential	1.0	10
Rationale: At the new bridge location the river channel is some 20% wider than the average width through this reach; this, combined with the offset nature of the bridge piers due to the skewed bridge alignment, supported by Hec-Ras modelling results, indicate that contraction scour will not occur		
I. Local Scour Potential	2	10
Rationale: The depth of local scour at the proposed 914 mm diameter piers is estimated to be approximately 2.4 m below the prevailing general bed elevation; the extreme, 40 m, ground penetration of the piles makes the local scour insignificant. Because the toe of the south abutment fill is above the 200 year flood elevation it will have no impact on river flows; the north abutment fill will project into the flood-way with the toe located at about the mean annual flood elevation but the low flow velocities and an apron of nominal riprap protection around the toe of the fill will prevent scour. The potential for scour due to flood plain flows plunging into the main channel at the north abutment is reduced by a spur dyke parallel to the river channel at the top of the natural river bank.		

1. DETAILED EVALUATION SUMMARY									
OTHER DISCIPLINES INPUT TO THIS REPORT									
The anticipated pile penetration at the abutments and piers was provided by the geotechnical consultant (Thurber Consulting); previous scour evaluation reports for the existing bridge by NHC were also referred to.									
CHANNEL SURVEYS									
Hydrographic survey through the bridge reach was carried out in 2018 as part of this project. The results were compared with previous surveys in 2014 and periodic bed surveys between 1997 and 2002.									
FLOOD HYDROLOGY									
The drainage of the Shuswap River at the bridge site and the WSC Water Gauge 08LE008 which is located about 55 m downstream of the new bridge is approximately 4720 km2. The Gauge has records back to 1911, there are lengthy gaps in the record but the missing data was synthesised by correlation with the flow records at other locations on the river that covered the data gaps. The 200 year return period flood at the bridge was estimated to be 700 m3/s but this was increased by 15% (805 m3/2) to allow for climate change.									
Drainage area (km2)	4720	Q2 (m3/s)	431	Q10 (m3/s)	575	Q100 (m3/s)	753	Q200 (m3/s)	805
LATERAL STABILITY ASSESSMENT (AIRPHOTOS ETC)									
The bridge reach appears to be relatively straight and laterally stable, possibly due to its lateral southward migration being halted by the steeply rising hillside of more erosion resistant soils									
HYDRAULIC MODELLING AND ANALYSIS									
The HEC-RAS one dimensional river profile model was used to develop the water surface profiles and velocities for various flood situations. The topographic data was surveyed in in May 2018, an iterative procedure, having various values of channel roughness and downstream channel gradient, was used to reproduce the known stage/ discharge relationship for the WSC gauge near the bridge. The appropriate manning's n was found to be 0.035 for the channel and 0.04 for the overbank areas while the downstream channel gradient was assumed to be 0.0005 m/m									
SCOUR AND RIPRAP STABILITY CALCULATIONS									
The average channel velocity through the new bridge was approximately 1.54 m/s at a flow of 805 m3/s and only 1.34 m/s at the 10 year flood flow and it is surmised that the channel is relatively stable given the size of the gravel bed material. Because of the susceptibility to erosion of newly place fill, riprap protection will be provided around the north abutment fill and in front of the south abutment. A spur dyke will be placed upstream of the north the abutment fill to prevent overbank flow plunging back into the main channel at the abutment. Local scour at the two 914 mm wide river piers is estimated to be at maximum 2.4 m but this will not be significant given the anticipated 40 m of pile embedment									
COUNTERMEASURES DRAWINGS AND SKETCHES									
Since the bridge is yet to be constructed as- built drawing should be added at later date									

6. DETAILED EVALUATION – HYDRAULIC ANALYSIS AND CALCULATION RESULTS

SURFICIAL BED MATERIAL GRADATIONS (mm)

source	D10	2	D30	9	D50	18	D90	42	D100	74

GEOMETRY OF APPROACH CHANNEL (mm, or m/m)

CHANNELS	1	BOTTOM WIDTH	65	BANK WIDTH	93	FLOODPLAIN WIDTH	±100
BANK HEIGHT		GRADIENT	0.0004				

GEOMETRY OF CONTROLLED OPENING (m, deg or m,)

TYPE	BRIDGE	NO PIERS	2	PIERS IN CHANNEL	2	PIER WIDTH	0.914
BOTTOM WIDTH OR CULVERT SPAN				CULVEERT RISE		TOP WIDTH	
SUBTRUCTURE SKEW ANGLE			30	SOFFIT/OBVERT ELEV	358.52	MIN BED ELEV	350.

HYDRAULIC GEOMETRY AT 200 -year (m, m3/s or m/s, as applicable)

CHANNEL MANNING'S 'n'	0.35	APPROACH CHN FLOW	805	OPENING FLOW	805
APPROACH HYDRAULIC DEPTH	6.2	OPENING HYDR DEPTH	5.0	FLOOD LEVEL	356.92
APPROACH AV VELOCITY	1.51	OPENING HYDR VELOCITY	1.54	CLEARANCE	1.5 (min)

BED MATERIAL TRANSPORT CONDITION (HEC-18) (m or m/s as applicable)

LOCATION		FLOW DEPTH	6.2	AVG VELOCITY	1.54	CRIT VELOCITY	2.2
TRANSIT COND		Clear water condition		COMMENT			

BED MATERIAL STABILITY IN STEEP CHANNELS (AQUIRRE- PE, 2003)

LOCATION		FLOW DEPTH		FLOW VELOCITY		CHANNEL SLOPE	
STABLE D50		OBSERVED D50					
COMMENT							

BED MATERIAL STABILITY IN STEEP CHANNELS (AQUIRRE- PE, 2003)

LOCATION		FLOW DEPTH		FLOW VELOCITY		CHANNEL SLOPE	
STABLE D50		OBSERVED d50					
COMMENT							

PIER RIPRAP PROTECTION STABILITY (HEC-11 PROCEDURE)

LOCATON		SPEC GRAVITY		AVG VELOCITY		LOCAL VELOCITY	
STABLE D50		OBSERVED D50					
COMMENT							

PIER RIPRAP PROTECTION STABILITY (HEC-11 PROCEDURE)

LOCATION		SPEC GRAVITY		AVG VELOCITY		LOCAL VELOCITY	
STABLE D50		OBSERVED D50					
COMMENT							

BANK RIPRAP PROTECTION STABILITY (USACE 1991 PROCEDURE)

LOCATION		LOCAL DEPTH		AVG VELOCITY		LOCAL VELOCITY	
COEF Cs		COEFF Cv		COEFF Ct		SS FACTOR K	
SAFTEY Fs		SPEC GRAVITY		ESTIMATED D50		OBSERVED D50	
COMMENT							

BANK RIPRAP PROTECTION STABILITY (USACE 1991 PROCEDURE)

LOCATION		LOCAL DEPTH		AVG VELOCITY		LOCAL VELOCITY	
COEF Cs		COEFF Cv		COEFF Ct		SS FACTOR K	
SAFTEY Fs		SPEC GRAVITY		ESTIMATED D50		OBSERVED D50	
COMMENT							

6. DETAILED EVALUATION—SCOUR CALCULATION RESULTS (continued)

NATURAL SCOUR: BLENCH'S REGIME DEPTH METHOD (m3/s, m, m/m, mm as applicable)

LOCATION		FLOW		TOP WIDTH		SLOPE	
D50		REGIME DEPTH		Z-FACTOR		FLOOD LEVEL	
SCOUR LEVEL		COMMENT					

NATURAL SCOUR: BLENCH'S REGIME DEPTH METHOD (m3/s, m, m/m, mm as applicable)

LOCATION		FLOW		TOP WIDTH		SLOPE	
D50		REGIME DEPTH		Z-FACTOR		FLOOD LEVEL	
SCOUR LEVEL		COMMENT					

COMPETENT VELOCITY (m3/s, m, as applicable

SED TRANSPORT				APPL CHAN FLOW		FLD PLAIN FLOW	
BRIDGE FLOW		APL HYD DEPTH		BRG HYD DEPTH		APP CHN TOP WD	
TRANS COMP k		BRG FLD EL		BRG SCOUR EL			
COMMENT							

PIER SCOUR SUMMARY

Flow return period	200-year						
Location	P1	P2					
Effective pier width (m)	0.914	0.914					
Local upstream velocity (m/s)	1.54	1.54					
Local flow depth upstream (m)	6.3	6.3					
Critical velocity (m/s)							
Local upstream Froud No	0.22	0.22					
K1 shape factor							
K2 attack angle factor							
K3 Bed factor							
K4 armouring factor							
Kp Cap factor							
Ki inclination factor							
Kw Pier width factor							
ds scour depth							

TOTAL SCOUR SUMMARY

Location	Pier 1	Pier 2					
Top of footing (El. M)							
Bot of footing (El, m)							
Bottom of sheet wall (El, m)							
Pile penetration (El, m)	40	40					
Riprap protection	no	no					
B_{NS}, Natural scour (El, m)	351.5	351.5					
B_{CS}, Contraction scour level (El, m)	351.5	351.5					
B_{LS}, Local scour level (El, m)	2.4	2.4					
$B_{TS,}$ Total Scour Level (El, m)	348.1	348.1					

Appendix B – Hydrological Information

Appendix B

Hydrological Data

The flow in the Shuswap River at the bridge site has been measured at station 08LC002 since 1911 to the present year, unfortunately the there is a gap in the record period between 1936 and 1972. Fortunately, there are other recording stations on the Shuswap River (Stn 08LC019) and Thompson river (Stn 08LE031) that not only span the gap years but also provide sufficient matching data before and after the gap to provide a correlation the flows and therefore allow estimates of the gap year flows'

WSC Gauge 08LC019 has a drainage area 4040 km^2 only slightly smaller than the drainage area at Gauge 08LC002 of 4720 km^2. The flow records for each gauge are shown in Table B1 and shown plotted against each other in Chart B1. The best fit line through the data of $y= 0.9164X – 3.155$ shows close correspondence between the two stations. Unfortunately Gauge 08LC019 flow records only covered part of the gap and it was necessary to use correlation with the flows farther down the Shuswap River system on the South Thompson River at Chase to fill the remainder of the gap. The drainage area at Chase is 15800 km^2 and when the flows at gauges 08LC002 and 08LE031 are plotted against each other in Chart B2 there is a quite wide scatter of the data points about the best-fit trend line. The equation of the trend line $y= 2.2714X + 183.36$ was used to fill in the remainder of the flow record gap.

The full record is shown in Table B2. The river flow for the year 1963 was not covered by any of the three stations and this was provided by comparison with the record for Gauge 08LC003 on the Shuswap at Lumby, which has a smaller drainage area of 2000 km^2, by simply picking another year with a similar flow rate as 1963 and assuming the same relationship would also apply at Gauge 08LC002.

TABLE B1

data common to guages 08LC018 and 08LC 002

	08LC002	08LC019
1928	626	552
1929	311	267
1930	243	223
1931	289	257
1932	419	340
1933	464	405
1934	354	306
1935	391	328
1961	388	354
1962	274	252
1964	411	442
1965	334	309
1966	328	306
1967	436	413
1968	379	340
1969	368	343
1970	275	258
1971	377	345
1972	566	532
1973	266	235
1974	535	498
1975	357	323
1976	430	388
1977	255	229
1978	331	297
1979	268	242

data common to gauges 08LC002 & 08LE031

	08LC0022	08LE031
1912	391	985
1913	617	1370
1914	340	864
1915	303	759
1916	348	1020
1917	362	1030
1918	374	923
1919	323	779
1920	345	1030
1921	439	1280
1922	385	1050
1923	422	1220
1924	385	830
1925	385	1020
1926	217	566
1927	462	1070
1928	626	1310
1929	311	770
1930	243	648
1931	289	850
1932	419	1160
1933	464	1340
1934	354	844
1935	391	1160
1971	377	1170
1972	566	1480
1973	266	782
1974	535	1310
1975	357	1040
1976	430	1030
1977	255	665
1978	331	847
1979	268	694
1980	261	716
1981	377	960
1982	446	1160
1983	376	943
1984	400	1070
1985	465	1080
1986	429	991
1987	263	678

TABLE B1 (continued)

1988	263	770
1989	271	864
1990	450	1160
1991	307	827
1992	237	653
1993	386	893
1994	284	811
1995	280	800
1996	373	954
1997	508	1280
1998	301	782
1999	476	1420
2000	353	1030
2001	308	824
2002	387	1180
2003	316	898
2004	250	789
2005	261	753
2006	381	1050
2007	335	1010
2008	394	1060
2009	299	772
2010	256	770
2011	385	1120
2012	509	1450
2013	394	1050
2014	345	1060

CHART B1

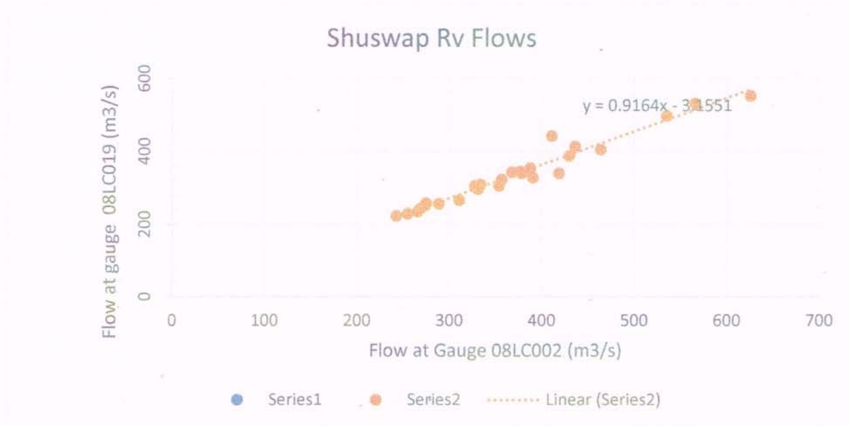


CHART B2

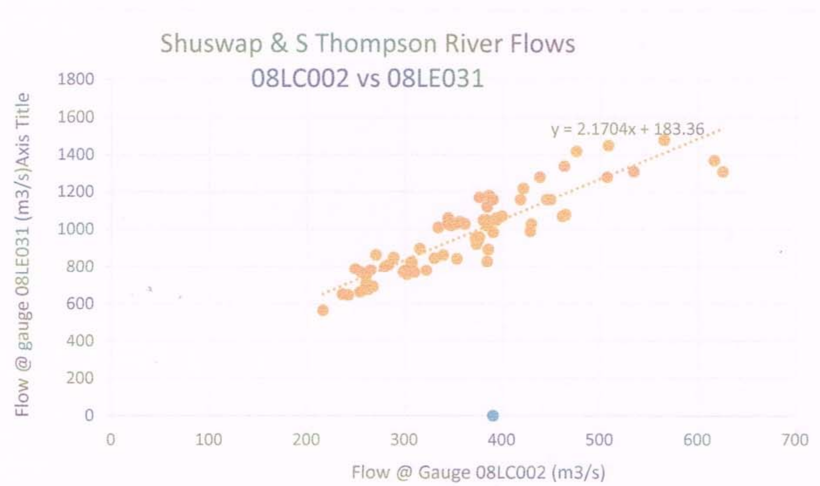


TABLE B2

Missing data in 08LC002 record generated from the records at 08LC019 & 08LEo31

	08LC002	08LC019	08LE031	08LC002
1912	391			391
1913	617			617
1914	340			340
1915	303			303
1916	348			348
1917	362			362
1918	374			374
1919	323			323
1920	345			345
1921	439			439
1922	385			385
1923	422			422
1924	385			385
1925	385			385
1926	217			217
1927	462			462
1928	626			626
1929	311			311
1930	243			243
1931	289			289
1932	419			419
1933	464			464
1934	354			354
1935	391			391
1936			1320	523.6979213
1937			903	331.5683237
1938			731	252.3206239
1939			869	315.9030807
1940			830	297.9341255
1941			634	207.6286072
1942			830	297.9341255
1943			773	271.6718064
1944			688	232.508699
1945			934	345.8513393
1946			1270	500.6607993
1947			971	362.8988096
1948			1610	657.313229
1949			968	361.5165823
1950			1120	431.5494332
1951			850	307.1489743

TABLE B2 (continued)

1952		314	346.0797
1953		326	359.1741
1954		331	364.6301
1955		447	491.2093
1956		331	364.6301
1957		365	401.7309
1958		337	371.1773
1959		385	423.5549
1960		267	294.7933
1961		354	389.7277
1962		252	278.4253
1963			243
1964		442	485.7533
1965		309	340.6237
1966		306	337.3501
1967		413	454.1085
1968		340	374.4509
1969		343	377.7245
1970		258	284.9725
1971	377		377
1972	566		566
1973	266		266
1974	535		535
1975	357		357
1976	430		430
1977	255		255
1978	331		331
1979	268		268
1980	261		261
1981	377		377
1982	446		446
1983	376		376
1984	400		400
1985	465		465
1986	429		429
1987	263		263
1988	263		263
1989	271		271
1990	450		450
1991	307		307
1992	237		237
1993	386		386
1994	284		284
1995	280		280
1996	373		373

TABLE B2 (continued)

1997	508	508
1998	301	301
1999	476	476
2000	353	353
2001	308	308
2002	387	387
2003	316	316
2004	250	250
2005	261	261
2006	381	381
2007	335	335
2008	394	394
2009	299	299
2010	256	256
2011	385	385
2012	509	509
2013	394	394
2014	345	345
2015	345	345
2016	272	272
2017	477	477
2018	505	505

HydroFreq v1.0 Output for Project : Shuswap RV daily 1894--2018 **Date : 18-09-16**

	Flows	Log of Flows	Hist Wt. Flows	
Mean	370.45	5.88	369.67	
St. Dev	94.66	0.25	93.53	
Skew	0.84	0.17	0.82	
RP	GEV	LN3	LP3	P3
2.00	357.27	355.31	356.32	357.17
5.00	441.32	440.03	440.11	442.19
10.00	493.87	494.50	493.35	494.36
20.00	542.09	545.74	543.19	541.73
25.00	556.95	561.83	558.82	556.31
50.00	601.47	611.10	606.60	599.85
100.00	643.82	659.74	653.74	641.55
200.00	684.28	708.20	700.72	681.81
500.00	735.11	772.42	763.06	733.23
Fit Method	Moments	Max. Like	Moments	Moments
Location	328.84	5.60	2.76	369.67
Scale	78.44	0.32	0.02	93.53
Shape	0.06	85.87	161.07	0.82

Flood frequency analysis used flow record for WSC gauge 08LC002 for the years 1912 to 2018 and in addition included the 1894 flow of 654 m3/s as an historical record.

Mean Annual Flow	=	370.89 m3/s
10 year return period flow (average)	=	493.7 m3/s
100 year return period flow (average)	=	649.35 m3/s
200 year return period flow (average)	=	693.45 m3/s

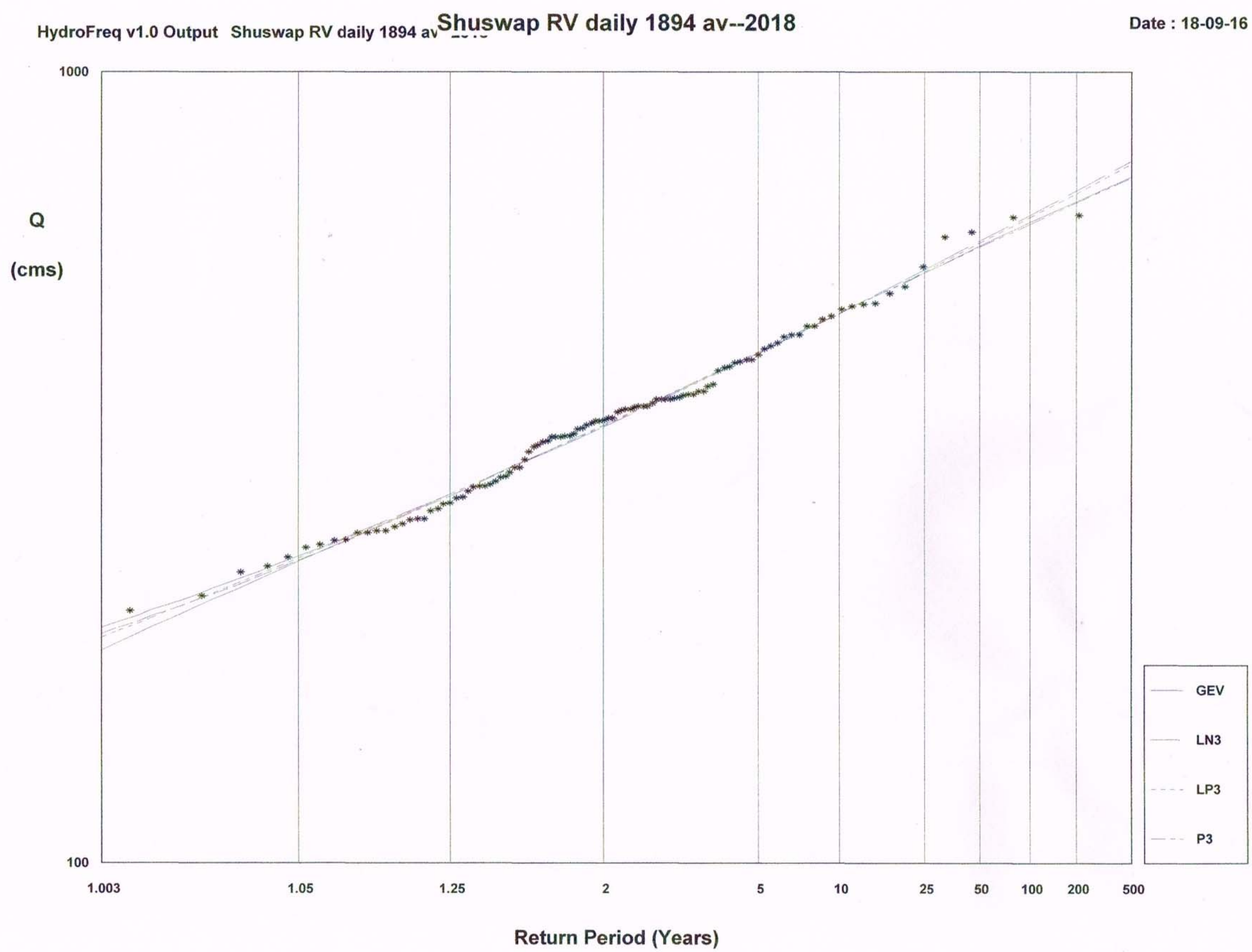
HydroFreq v1.0 Output for Project : Shuswap RV daily 1894--2018

Date : 18-09-16

Extended Length of Record (Years) : 125

Historical Threshold (cms) : 654

Rank	Flow	Year	R.P.
1	657	1948	208.67
2	654	1894	78.25
3	626	1928	45.18
4	617	1913	31.75
5	566	1972	24.48
6	535	1974	19.92
7	524	1936	16.79
8	509	2012	14.51
9	508	1997	12.78
10	505	2018	11.41
11	501	1946	10.31
12	491	1955	9.40
13	486	1964	8.64
14	477	2017	8.00
15	476	1999	7.44
16	465	1985	6.96
17	464	1933	6.53
18	462	1927	6.15
19	454	1967	5.82
20	450	1990	5.52
21	446	1982	5.25
22	439	1921	5.00
23	432	1951	4.78
24	432	1950	4.57
25	430	1976	4.39
26	429	1986	4.21
27	424	1959	4.05
28	422	1923	3.91
29	419	1932	3.77
30	402	1957	3.64
31	400	1984	3.52
32	394	2013	3.41
33	394	2008	3.30
34	391	1935	3.20
35	391	1912	3.11
36	390	1961	3.02
37	387	2002	2.94
38	386	1993	2.86
39	385	2011	2.79
40	385	1925	2.71
41	385	1924	2.65
42	385	1922	2.58
43	381	2006	2.52
44	378	1969	2.46
45	377	1981	2.41
46	377	1971	2.36
47	376	1983	2.31
48	374	1968	2.26
49	374	1918	2.21
50	373	1996	2.17
51	371	1958	2.12
52	365	1956	2.08
53	365	1954	2.04
54	363	1947	2.00
55	362	1949	1.97
56	362	1917	1.93
57	359	1953	1.90
58	357	1975	1.86
59	354	1934	1.83
60	353	2000	1.80
61	348	1916	1.77
62	346	1952	1.74
63	346	1945	1.71
64	345	2015	1.69
65	345	2014	1.66
66	345	1920	1.64
67	341	1965	1.61
68	340	1914	1.59
69	337	1966	1.56
70	335	2007	1.54
71	331	1937	1.52
72	323	1919	1.50
73	316	2003	1.48
74	316	1939	1.46
75	311	1929	1.44
76	308	2001	1.42
77	307	1991	1.40
78	303	1915	1.38
79	301	1998	1.36
80	299	2009	1.35
81	299	1942	1.33
82	298	1940	1.31
83	295	1960	1.30
84	289	1931	1.28
85	285	1970	1.27
86	284	1994	1.25
87	280	1995	1.24
88	278	1962	1.22
89	272	2016	1.21
90	272	1943	1.20
91	271	1989	1.18
92	268	1979	1.17
93	266	1973	1.16
94	263	1988	1.14
95	263	1987	1.13
96	261	2005	1.12
97	261	1980	1.11
98	256	2010	1.10
99	255	1977	1.09
100	252	1938	1.08
101	250	2004	1.06
102	243	1963	1.05
103	243	1930	1.04
104	237	1992	1.03
105	233	1944	1.02
106	217	1926	1.01
107	208	1941	1.00


HydroFreq v1.0 Output Shuswap RV daily 1894 av
Shuswap RV daily 1894 av--2018
Date : 18-09-16
1000
Q
(cms)
100
1.003
1.05
1.25
2
5
10
25
50
100
200
500
Return Period (Years)
GEV
LN3
LP3
P3

Appendix C – Flow Modelling Results

Appendix C

The flow modelling of the Shuswap River at Baxter Bridge was for the following purposes:

a) To establish the water surface profiles during flood conditions so that the extent of the overbank flow on the upstream flood plain.
b) To establish maximum velocities at the bridge to facilitate bank protection design.
c) To calculate the flow velocities at the bridge site for various degree of bed scour to allow an evaluation of potential scour.

Figure C1 is a plan of the river through the bridge site showing the location of the existing bridge and the centre line of the proposed new structure, it also shows the locations of the river cross-section location used in the Hec-Ras model used for the calculations.

Table C1 is the output from the modelling and gives the flow statistics for various river discharges at each of the cross-section stations.

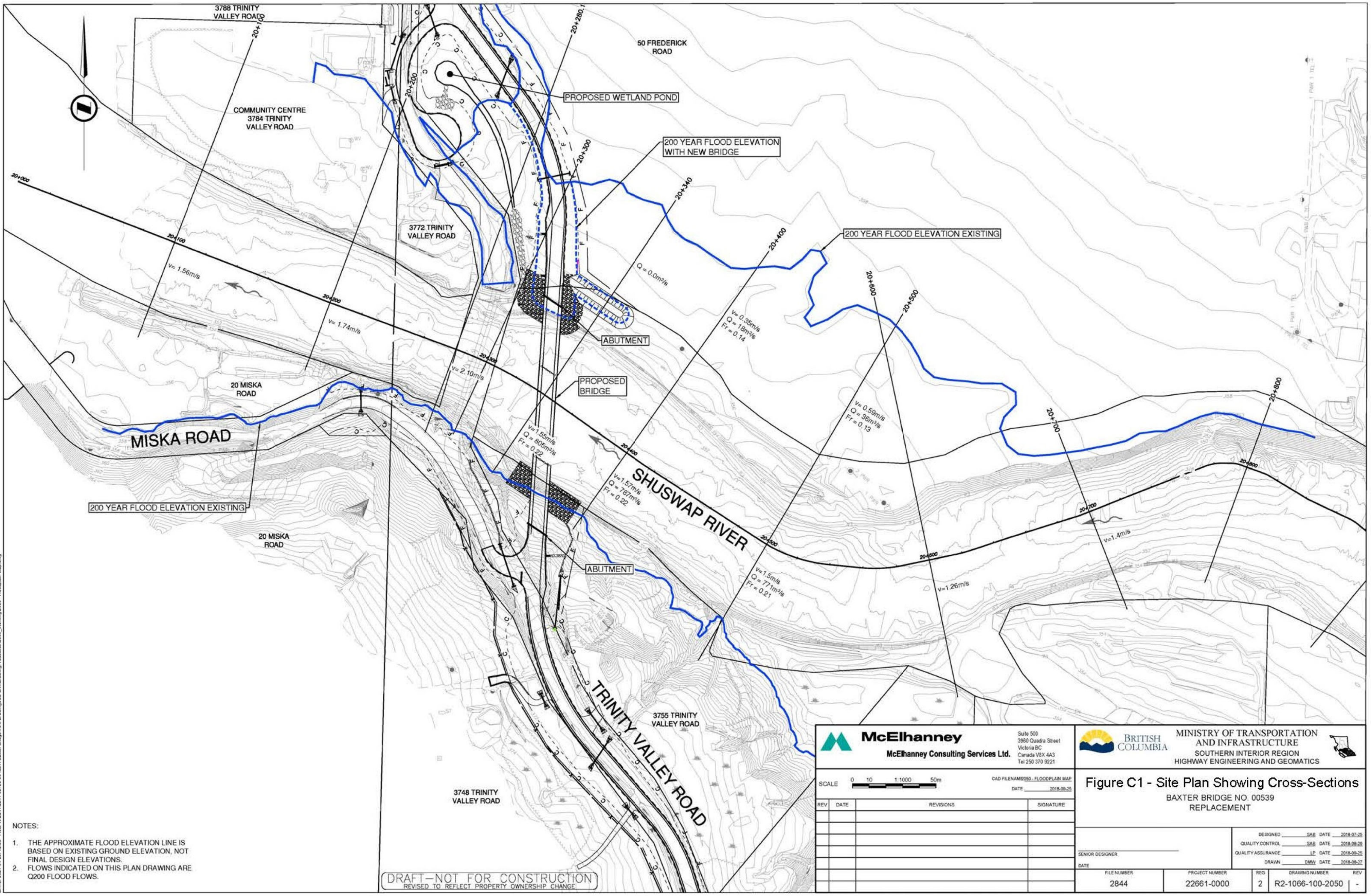
At Station 20+300, at the existing bridge, the water surface elevation during the 200 year return period flood (Q=805 m3/s) is calculated to be 356.87 m which is very close to the level estimated from the stage/discharge relationship; the average velocity at this flow would be 1.88 m/s.

Figure C2 is a plot of the flood profiles for the 10 year, 100 year, the 200 year return periods and the mean annual flood flows.

Tables C2a is the model output at Stn 20+260 for various river flows for a range of assumed scour depths.

Table C2b is a summary of the scour depths and the corresponding average velocities and is the basis for Figure 5 in the report.

Note: Since the modelling calculations were carried out the datum for the water gauge has been changed. The result of the change has been to increase height of all the flow profiles by 0.100 m. .



NOTES:

1. THE APPROXIMATE FLOOD ELEVATION LINE IS BASED ON EXISTING GROUND ELEVATION, NOT FINAL DESIGN ELEVATIONS.
2. FLOWS INDICATED ON THIS PLAN DRAWING ARE Q200 FLOOD FLOWS.

McElhanney
McElhanney Consulting Services Ltd.
Suite 500
3960 Quadra Street
Victoria BC
Canada V8X 4A3
Tel 250 370 9221

SCALE 0 10 1:1000 50m

CAD FILENAME 2050 - FLOODPLAIN MAP
DATE 2018-09-25

REV	DATE	REVISIONS	SIGNATURE

BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE
SOUTHERN INTERIOR REGION
HIGHWAY ENGINEERING AND GEOMATICS

Figure C1 - Site Plan Showing Cross-Sections

BAXTER BRIDGE NO. 00539
REPLACEMENT

SENIOR DESIGNER
DATE

DESIGNED	SAB	DATE	2018-07-25
QUALITY CONTROL	SAB	DATE	2018-09-29
QUALITY ASSURANCE	LP	DATE	2018-09-25
DRAWN	DMW	DATE	2018-09-27

FILE NUMBER	PROJECT NUMBER	REG	DRAWING NUMBER	REV
2844	22661-0000	2	R2-1066-100-2050	-

DATE: 2021-01-29, 12:58 FILE: X:\2241\2241-02729-00 MoTI-Baxter Bridge\10.0 Drawings\02 Civil\Drawing\Production\2000_Working\2050 - Floodplain Map.dwg

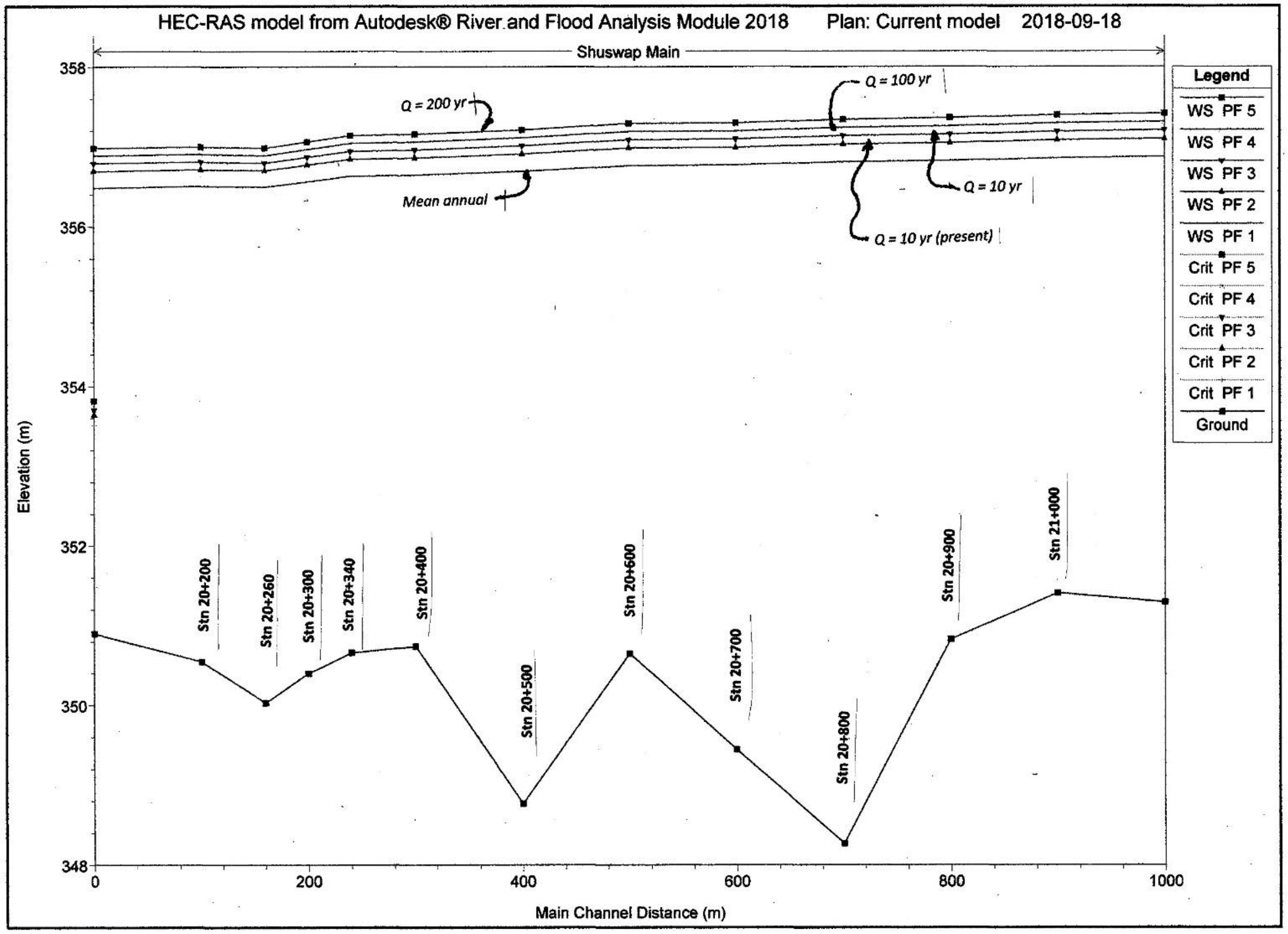
TABLE C1

HEC-RAS Plan: Current mode River: Shuswap Reach: Main

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Main	21100	PF 1	431.00	351.29	355.77		355.79	0.000120	0.77	711.31	300.20	0.13
Main	21100	PF 2	505.00	351.29	356.12		356.14	0.000112	0.79	815.44	300.20	0.12
Main	21100	PF 3	580.00	351.29	356.43		356.46	0.000107	0.81	910.74	300.20	0.12
Main	21100	PF 4	753.00	351.29	357.05		357.08	0.000104	0.87	1096.13	300.20	0.12
Main	21100	PF 5	805.00	351.29	357.21		357.24	0.000104	0.89	1144.48	300.20	0.12
Main	21000	PF 1	431.00	351.41	355.75		355.78	0.000133	0.80	666.74	272.64	0.13
Main	21000	PF 2	505.00	351.41	356.10		356.13	0.000125	0.82	761.56	273.56	0.13
Main	21000	PF 3	580.00	351.41	356.42		356.45	0.000120	0.84	848.54	274.51	0.13
Main	21000	PF 4	753.00	351.41	357.03		357.07	0.000123	0.93	1025.04	300.20	0.13
Main	21000	PF 5	805.00	351.41	357.19		357.23	0.000123	0.95	1073.40	300.20	0.13
Main	20900	PF 1	431.00	350.83	355.73		355.76	0.000153	0.90	549.81	210.26	0.14
Main	20900	PF 2	505.00	350.83	356.07		356.11	0.000150	0.94	622.59	210.81	0.14
Main	20900	PF 3	580.00	350.83	356.39		356.43	0.000150	0.98	689.26	211.32	0.14
Main	20900	PF 4	753.00	350.83	357.00		357.05	0.000154	1.08	818.92	212.29	0.15
Main	20900	PF 5	805.00	350.83	357.16		357.21	0.000156	1.11	852.78	212.55	0.15
Main	20800	PF 1	431.00	348.27	355.71		355.75	0.000111	0.92	576.48	197.60	0.13
Main	20800	PF 2	505.00	348.27	356.06		356.10	0.000115	0.97	644.66	198.27	0.13
Main	20800	PF 3	580.00	348.27	356.37		356.42	0.000120	1.03	707.16	198.88	0.13
Main	20800	PF 4	753.00	348.27	356.98		357.04	0.000131	1.15	828.76	200.06	0.14
Main	20800	PF 5	805.00	348.27	357.14		357.20	0.000135	1.18	860.53	200.36	0.14
Main	20700	PF 1	431.00	349.45	355.68		355.74	0.000198	1.15	472.95	179.88	0.17
Main	20700	PF 2	505.00	349.45	356.02		356.08	0.000199	1.20	534.95	180.46	0.17
Main	20700	PF 3	580.00	349.45	356.34		356.40	0.000201	1.25	591.71	180.98	0.17
Main	20700	PF 4	753.00	349.45	356.94		357.02	0.000211	1.38	701.80	181.96	0.18
Main	20700	PF 5	805.00	349.45	357.10		357.18	0.000215	1.41	730.48	182.21	0.18
Main	20600	PF 1	431.00	350.65	355.67		355.71	0.000199	1.04	517.36	185.94	0.16
Main	20600	PF 2	505.00	350.65	356.01		356.06	0.000194	1.08	581.72	187.26	0.16
Main	20600	PF 3	580.00	350.65	356.33		356.38	0.000192	1.13	642.13	210.52	0.16
Main	20600	PF 4	753.00	350.65	356.93		356.99	0.000197	1.23	796.02	281.78	0.17
Main	20600	PF 5	805.00	350.65	357.09		357.15	0.000197	1.26	841.22	288.21	0.17
Main	20500	PF 1	431.00	348.77	355.62		355.69	0.000253	1.14	381.20	97.46	0.18
Main	20500	PF 2	505.00	348.77	355.96		356.03	0.000275	1.23	416.14	121.55	0.19
Main	20500	PF 3	580.00	348.77	356.26		356.35	0.000287	1.31	461.57	160.40	0.20
Main	20500	PF 4	753.00	348.77	356.86		356.96	0.000306	1.46	569.39	194.61	0.21
Main	20500	PF 5	805.00	348.77	357.01		357.12	0.000311	1.50	599.99	201.07	0.21
Main	20400	PF 1	431.00	350.74	355.59		355.66	0.000308	1.18	363.77	98.14	0.20
Main	20400	PF 2	505.00	350.74	355.92		356.00	0.000321	1.27	397.09	101.63	0.20
Main	20400	PF 3	580.00	350.74	356.22		356.32	0.000333	1.36	430.02	126.71	0.21
Main	20400	PF 4	753.00	350.74	356.81		356.93	0.000362	1.53	523.04	175.81	0.22
Main	20400	PF 5	805.00	350.74	356.97		357.09	0.000370	1.57	550.29	180.30	0.23
Main	20340	PF 1	431.00	350.66	355.57	352.69	355.64	0.000275	1.15	379.62	109.12	0.19
Main	20340	PF 2	505.00	350.66	355.91	352.86	355.98	0.000294	1.23	416.41	113.82	0.19
Main	20340	PF 3	580.00	350.66	356.21	353.02	356.30	0.000306	1.31	450.28	122.25	0.20
Main	20340	PF 4	753.00	350.66	356.80	353.35	356.91	0.000341	1.49	516.02	170.66	0.22
Main	20340	PF 5	805.00	350.66	356.95	353.44	357.07	0.000352	1.55	533.15	182.25	0.22
Main	20300	PF 1	431.00	350.40	355.51		355.62	0.000468	1.45	297.56	82.14	0.24
Main	20300	PF 2	505.00	350.40	355.84		355.96	0.000493	1.56	324.47	83.40	0.25
Main	20300	PF 3	580.00	350.40	356.14		356.28	0.000519	1.67	349.30	84.47	0.26
Main	20300	PF 4	753.00	350.40	356.70		356.88	0.000587	1.91	397.37	86.03	0.28
Main	20300	PF 5	805.00	350.40	356.84		357.04	0.000608	1.98	409.75	86.07	0.29
Main	20280	PF 1	431.00	350.03	355.49		355.61	0.000496	1.54	281.01	82.20	0.25
Main	20280	PF 2	505.00	350.03	355.81		355.95	0.000526	1.67	308.35	87.90	0.26
Main	20280	PF 3	580.00	350.03	356.10		356.26	0.000555	1.78	334.92	93.50	0.27
Main	20280	PF 4	753.00	350.03	356.66		356.87	0.000627	2.03	390.95	106.83	0.29
Main	20280	PF 5	805.00	350.03	356.81		357.03	0.000650	2.10	406.52	110.20	0.30
Main	20200	PF 1	431.00	350.55	355.48		355.57	0.000342	1.29	334.48	89.15	0.21
Main	20200	PF 2	505.00	350.55	355.80		355.90	0.000387	1.40	364.32	97.34	0.22
Main	20200	PF 3	580.00	350.55	356.10		356.21	0.000402	1.49	395.25	112.51	0.23
Main	20200	PF 4	753.00	350.55	356.67		356.81	0.000435	1.69	479.50	172.19	0.24
Main	20200	PF 5	805.00	350.55	356.81		356.96	0.000443	1.74	504.98	174.55	0.25
Main	20100	PF 1	431.00	350.90	355.45	352.95	355.53	0.000400	1.24	348.60	112.48	0.22
Main	20100	PF 2	505.00	350.90	355.77	353.12	355.86	0.000400	1.32	387.26	130.46	0.22

Bridge location

FIGURE C2



HEC-RAS model from Autodesk® River and Flood Analysis Module 2018 Plan: Current model 2018-09-18
Shuswap Main
Legend
WS PF 5
WS PF 4
WS PF 3
WS PF 2
WS PF 1
Crit PF 5
Crit PF 4
Crit PF 3
Crit PF 2
Crit PF 1
Ground
Q = 200 yr
Q = 100 yr
Q = 10 yr
Q = 10 yr (present)
Mean annual
Stn 20+200
Stn 20+260
Stn 20+300
Stn 20+340
Stn 20+400
Stn 20+500
Stn 20+600
Stn 20+700
Stn 20+800
Stn 20+900
Stn 21+000
Elevation (m)
358
356
354
352
350
348
Main Channel Distance (m)
0
200
400
600
800
1000

TABLE C2a

Flow velocity at Stn 20+260 for various river discharges and flow depths

Average scour depth = 0.0 m

Main	20260	PF 1	431.00	350.03	355.48		355.60	0.000499	1.55	280.31	82.05	0.25
Main	20260	PF 2	505.00	350.03	355.80		355.94	0.000530	1.67	307.53	87.72	0.26
Main	20260	PF 3	580.00	350.03	356.09		356.25	0.000559	1.78	334.21	96.00	0.27
Main	20260	PF 4	753.00	350.03	356.65		356.86	0.000626	2.02	402.33	151.66	0.29
Main	20260	PF 5	805.00	350.03	356.80		357.01	0.000643	2.08	425.00	162.12	0.30

Average scour depth = 0.5 m

Main	20260	PF 1	431.00	349.53	355.50	352.37	355.59	0.000337	1.37	315.48	92.45	0.21
Main	20260	PF 2	505.00	349.53	355.82	352.58	355.93	0.000369	1.49	338.96	111.80	0.22
Main	20260	PF 3	580.00	349.53	356.11	352.77	356.24	0.000401	1.61	360.56	149.38	0.23
Main	20260	PF 4	753.00	349.53	356.67	353.17	356.85	0.000480	1.87	402.15	179.77	0.26
Main	20260	PF 5	805.00	349.53	356.81	353.29	357.00	0.000505	1.95	412.89	186.59	0.27

Average scour depth = 1.0 m

Main	20260	PF 1	431.00	349.03	355.51	351.87	355.58	0.000236	1.22	353.09	140.04	0.18
Main	20260	PF 2	505.00	349.03	355.83	352.08	355.92	0.000264	1.34	377.00	163.96	0.19
Main	20260	PF 3	580.00	349.03	356.13	352.27	356.23	0.000292	1.45	398.95	178.18	0.20
Main	20260	PF 4	753.00	349.03	356.69	352.67	356.83	0.000359	1.71	441.33	189.39	0.23
Main	20260	PF 5	805.00	349.03	356.83	352.79	356.99	0.000380	1.78	452.34	189.69	0.23

Average scour depth = 1.5 m

Main	20260	PF 1	431.00	348.53	355.52	351.36	355.58	0.000171	1.10	391.19	175.16	0.15
Main	20260	PF 2	505.00	348.53	355.85	351.57	355.92	0.000195	1.22	415.61	188.67	0.17
Main	20260	PF 3	580.00	348.53	356.14	351.77	356.23	0.000218	1.32	438.07	189.30	0.18
Main	20260	PF 4	753.00	348.53	356.71	352.17	356.83	0.000275	1.56	481.30	190.49	0.20
Main	20260	PF 5	805.00	348.53	356.85	352.29	356.99	0.000293	1.63	492.43	190.99	0.21

Average scour depth = 2.0 m

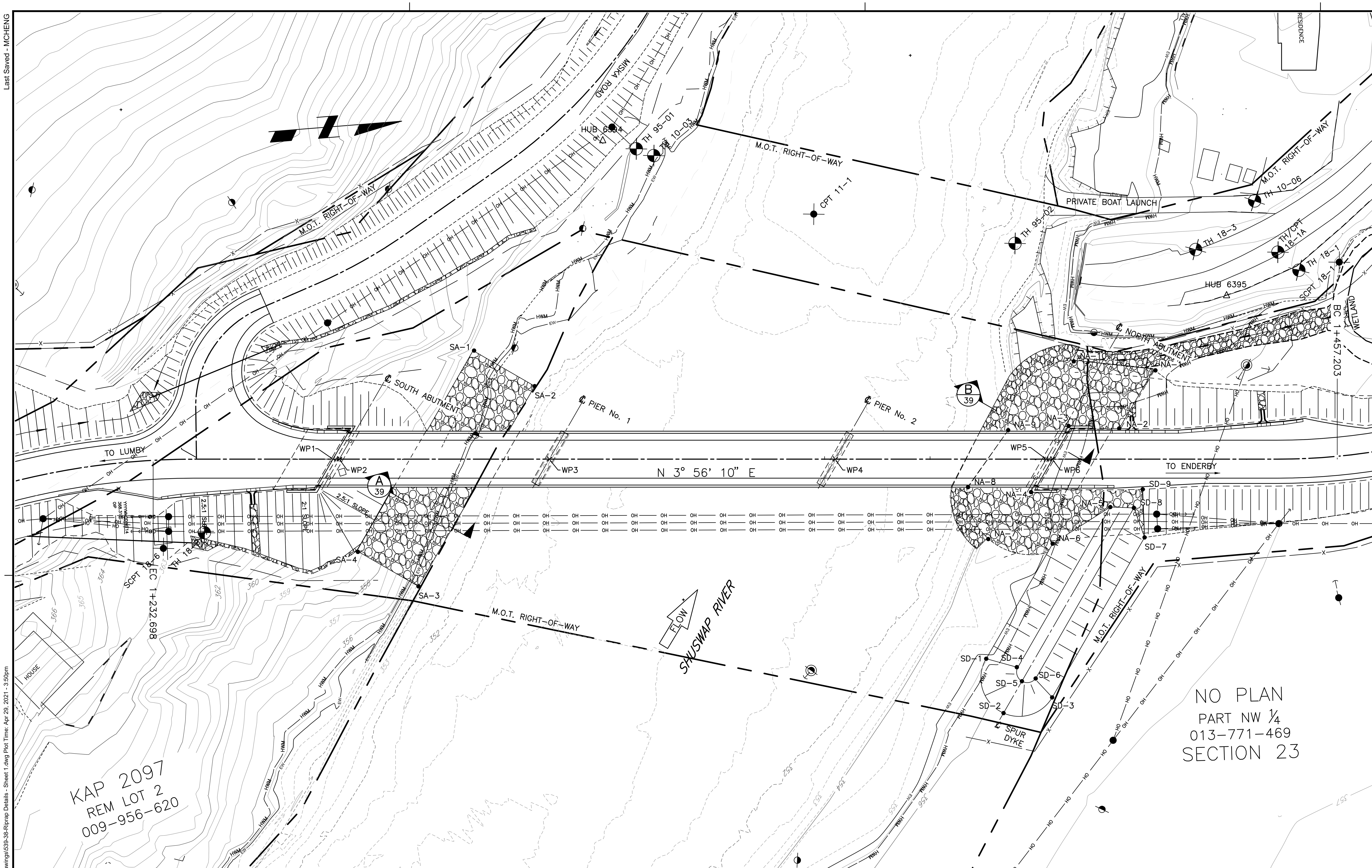
Main	20260	PF 1	431.00	348.03	355.53	350.86	355.58	0.000128	1.00	429.59	189.06	0.13
Main	20260	PF 2	505.00	348.03	355.86	351.08	355.92	0.000148	1.11	454.49	189.75	0.15
Main	20260	PF 3	580.00	348.03	356.16	351.27	356.23	0.000167	1.21	477.42	190.38	0.16
Main	20260	PF 4	753.00	348.03	356.72	351.67	356.83	0.000220	1.44	522.02	193.28	0.18
Main	20260	PF 5	805.00	348.03	356.87	351.79	356.99	0.000236	1.51	533.73	194.16	0.19

TABLE C2b

Flow velocity at Stn 20+260 for various river discharges and flow depths

River flow (m3/s)	431	505	580	753	805	
Scour depth (m)						
0.0	1.55	1.67	1.78	2.02	2.08	
0.5	1.37	1.49	1.61	1.87	1.95	
1.0	1.22	1.34	1.45	1.71	1.78	
1.5	1.10	1.22	1.32	1.56	1.63	
2.0	1.00	1.11	1.21	1.44	1.51	

Appendix D – Riprap and Spur Dyke Details



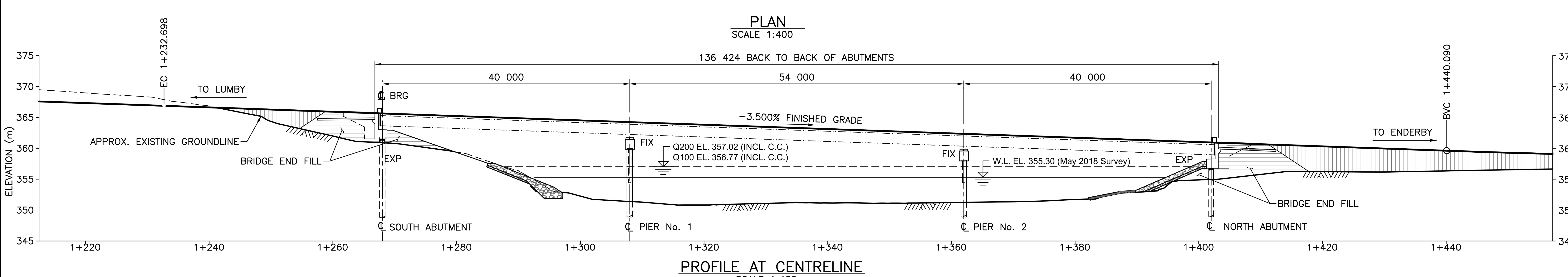


TABLE 38-1 RIP RAP WORK POINTS

WORK POINT	NORTHING	EASTING
SA-1	600,963.469	357,450.129
SA-2	600,974.660	357,457.659
SA-3	600,950.079	357,494.011
SA-4	600,938.888	357,486.481
NA-1	601,092.041	357,462.490
NA-2	601,084.676	357,473.256
NA-3	601,074.940	357,472.126
NA-4	601,066.835	357,484.181
NA-5	601,081.978	357,488.013
NA-6	601,070.591	357,494.191
NA-7	601,058.280	357,492.460
NA-8	601,055.081	357,482.580
NA-9	601,063.656	357,472.144
NA-10	601,076.784	357,459.770
SD-1	601,062.449	357,517.036
SD-2	601,059.081	357,525.579
SD-3	601,068.385	357,523.440
SD-4	601,062.449	357,517.036
SD-5	601,062.992	357,519.812
SD-6	601,065.768	357,519.269
SD-7	601,088.060	357,494.201
SD-8	601,086.421	357,488.480
SD-9	601,090.971	357,485.034
SD-10	601,090.309	357,495.691

NOTES:

1. RIPRAP GRADATION SHALL BE AS PER SS205.
2. RIPRAP CONSTRUCTION IN ACCORDANCE WITH SPECIAL PROVISIONS.

McElhanney
McElhanney Ltd.
SUITE 500, 3960 QUADRA STREET
VICTORIA BC, CANADA V8X 4A3
PH (250) 370 9221
FAX 1-855-407-3895

Rev	Date	Description	Init

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

OKANAGAN SHUSWAP DISTRICT
TRINITY VALLEY ROAD

BAXTER BRIDGE No. 539
RIPRAP DETAILS – SHEET 1

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

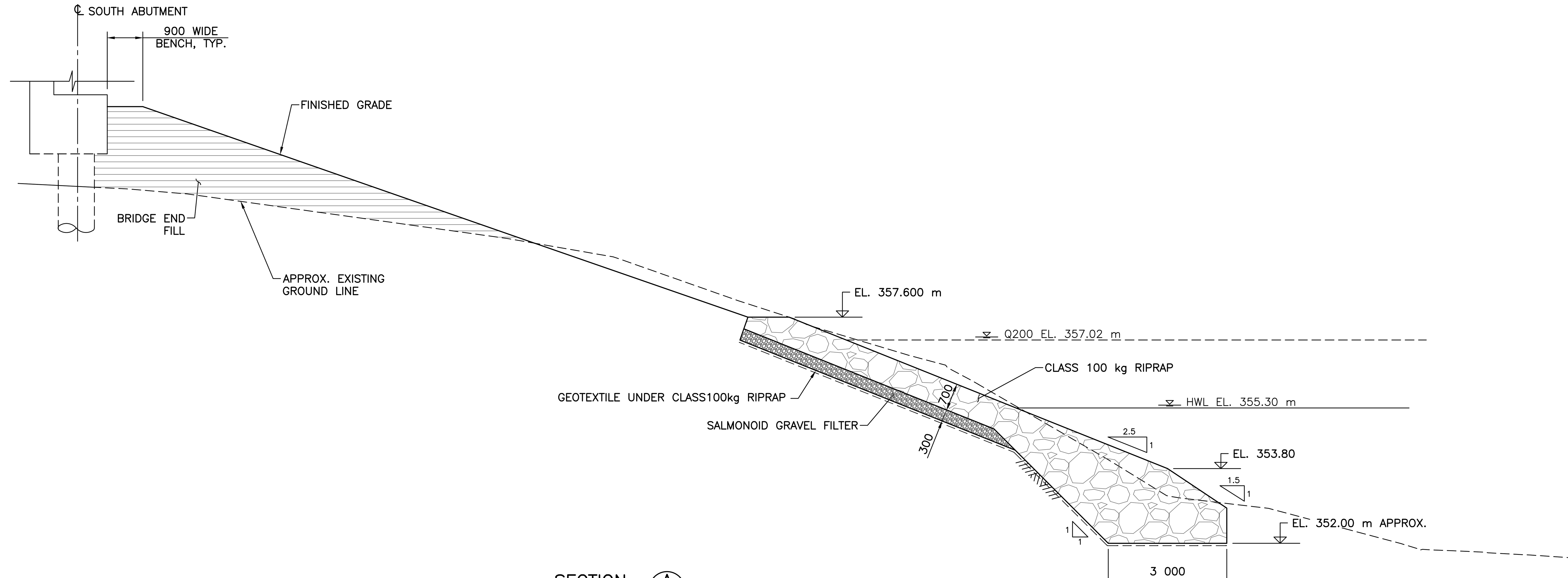
DATE

DESIGNED S.P.S. DATE Feb. 16, 2021
CHECKED E.Y.W. DATE Feb. 16, 2021
DRAWN D.R.M. DATE Feb. 16, 2021
SCALE AS NOTED
NEGATIVE No.

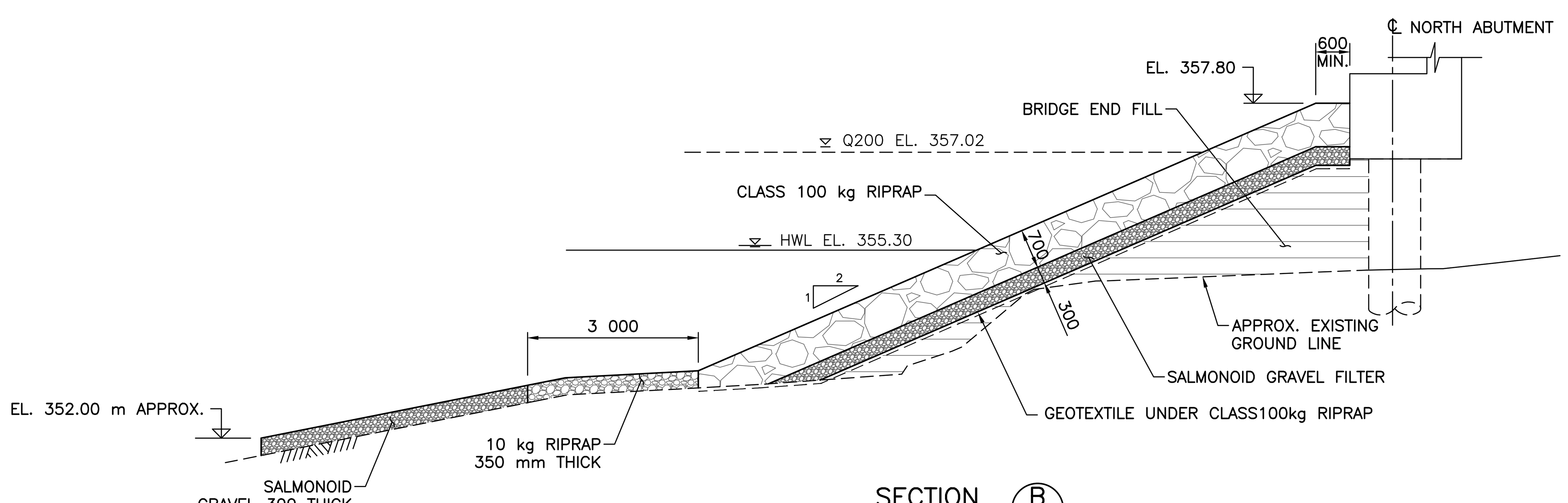
FILE No.	PROJECT No.	REG.	DRAWING No.
	22661-0000	2	539-38

CANCEL PRINTS BEARING PREVIOUS LETTER

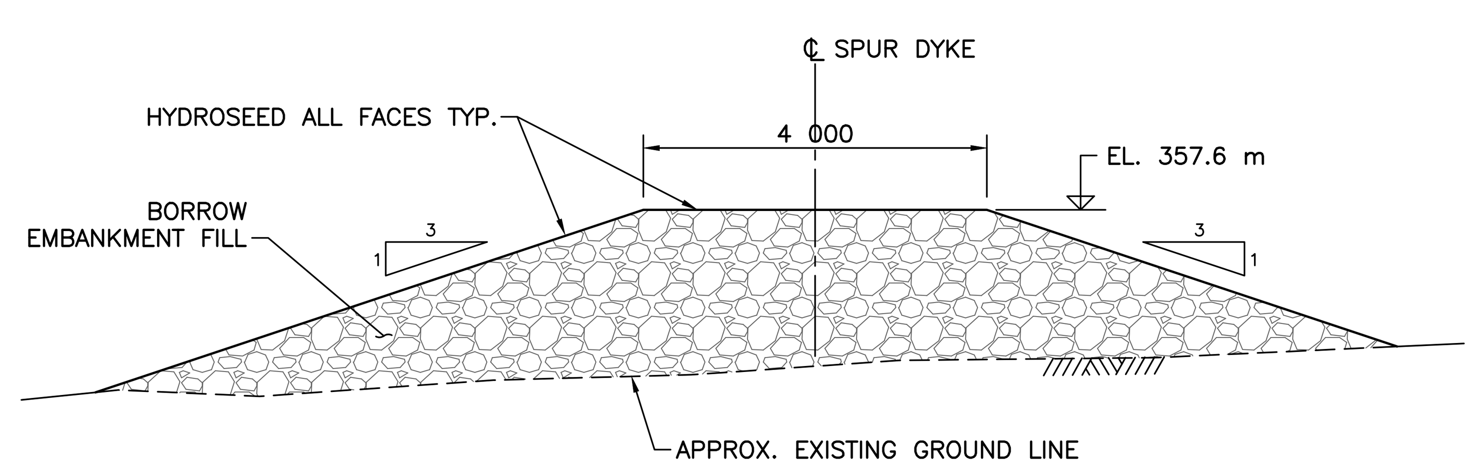
H-308-2-cs(07-08)



SECTION A
SCALE 1:75 38



SECTION B
SCALE 1:75 38



SECTION C
SCALE 1:75 38

NOTES:

1. FOR NOTES SEE DWG. 539-38
2. FOR SALMONOID GRAVEL GRADATION SEE SPECIAL PROVISIONS.

McElhanney
McElhanney Ltd.
SUITE 500, 3960 QUADRA STREET
VICTORIA BC, CANADA V8X 4A3
PH (250) 370 9221
FAX 1-855-407-3895

Rev	Date	Description	Init

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

OKANAGAN SHUSWAP DISTRICT
TRINITY VALLEY ROAD

BAXTER BRIDGE No. 539
RIPRAP DETAILS – SHEET 2

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

DATE

DESIGNED	J.H.M.	DATE	Feb. 16, 2021
CHECKED	D.R.E.	DATE	Feb. 16, 2021
DRAWN	M.P.D.	DATE	Feb. 16, 2021
SCALE	AS NOTED		
NEGATIVE No.			

FILE No.	PROJECT No.	REG.	DRAWING No.
	22661-0000	2	539-39

CANCEL PRINTS BEARING PREVIOUS LETTER

H-308j-r2-cs(07-08)