January 29, 2025

Ecora File No.: 2400834-22

Ministry of Transportation and Transit
Suite 321 – 1801 Princeton Kamloops Highway
Kamloops, BC V2E 2J7

Attention: **Michael George, P.Eng.**
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Reference: **Sackum Overhead No. 1491 Replacement – Hydrotechnical Design Report**

1. Introduction

1.1 Background

Ecora Engineering & Environmental Ltd. (Ecora) was retained by the British Columbia Ministry of Transportation and Transit (MoTT) to provide engineering services for the Sackum Overhead No. 1491 Bridge Replacement, Option A1. This project is situated along the Trans-Canada Highway #1 (TCH) at Sackum Creek (LKI0932- STA 22.35 km), located approximately 22 km northeast of the Village of Lytton, BC. The existing bridge spans the Canadian Pacific Kansas City (CPKC) single track railroad and Sackum Creek, approximately 125 m east of the Thompson River. Just upstream (east) of the Sackum Overhead crossing, Sackum Creek flows across a gravel access road. The crossing was historically used to access the CPKC single track railroad just north of the existing Sackum Overhead Bridge and runs north for approximately 300 m from the south side of the Sackum Overhead bridge. Following the 2021 Lytton wildfire, Sackum Creek experienced multiple debris flow events, which occurred between 2021 and 2023. These debris flows plugged and buried a 900 mm culvert that was used to convey streamflows through the access road embankment. The creek then continues to flow beneath the existing bridge and via a culvert through the CPKC railroad embankment before discharging into the Thomson River. This report was initially issued as an options analysis for remediation solutions for the access road crossing, then revised to reflect the design that was accepted by MoTT (ford crossing and creek armouring), which are discussed in this revision.

The focus of this hydrotechnical assignment was to determine the design flows for Sackum Creek at the project site and present solutions for the existing access road crossing of Sackum Creek as well as channel stability and erosion protection measures beneath the new Sackum Overhead structure.

1.2 Software

The following software was used for analysis and design of options:

- QGIS 3.23.8 (2024),
- HYFRAN Plus (2008),
- USACE HEC-HMS 4.12 (2024),
- FHA HY-8 7.80.0.2 (2022),
- AutoDesk Civil 3D (2023), and

- Microsoft Office Excel (2023).

2. Hydrologic Analysis

2.1 General

A hydrological analysis was completed with the purpose of estimating design flows for Sackum Creek at the project site. Multiple methods were explored in determination of these flows. The methods explored were the following:

- Rational Method,
- Regional Analysis Method,
- BC Streamflow Inventory Method,
- The BC Extreme Flood Project Approach, and
- Rainfall-Runoff Analysis Modelling with HEC-HMS.

The selection of the governing design flow return period was guided by the BC Supplement to TAC Geometric Design Guide (MoTT, 2021) [BC TAC], Table 1010.A. For a freeway bridge, the design flow is specified to be the 200-year return period maximum instantaneous discharge.

2.2 Watershed Characteristics

The proposed Sackum Overhead No 1497 Replacement works cross Sackum Creek, a tributary of the Thompson River, 22 km northeast of Lytton along the Trans-Canada Highway (TCH). The watershed area above the site was estimated to be approximately 13.6 km^2. The extents are shown below in Figure 2.1.

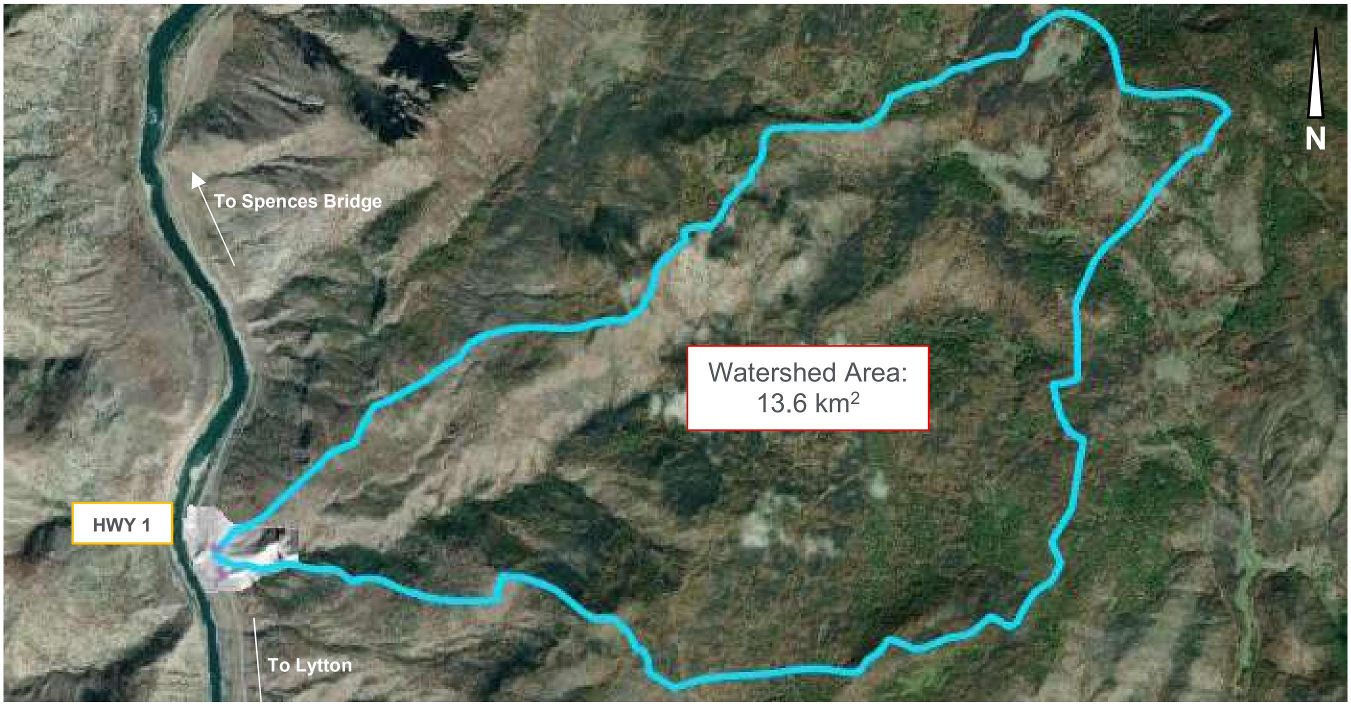


Figure 2.1 Sackum Creek Watershed Above Sackum Overhead

Over 50% of the Sackum Creek catchment is covered by forest with minimal clearings, forestry activity, or development within its boundaries. In 2021, much of the vegetation cover within the watershed was burnt as a result of the Lytton Creek wildfire (Figure 2.2). As a result, the Sackum Creek watershed is at risk of post-wildfire debris flows/debris floods. Since 2021, there have been at least three (3) debris flow events on Sackum Creek, with estimated maximum flow rates between 17 m^3/s and 38 m^3/s. This is discussed further in *Natural Hazard Assessment for the Sackum Overhead No. 1491 Replacement – Option A1* (Ecora, 2024).

The maximum elevation of the Sackum Creek catchment is estimated to be 1,459 m, with the minimum elevation being approximately 246 m, at the access road crossing. The mean elevation of the watershed was estimated to be 1041 m and it has an average slope of 14%. The values were calculated using QGIS Version 3.34.6 based on the Digital Elevation Model of British Columbia (BC DEM) last updated in 2016. BC DEM is roughly accurate to 10 m which was considered acceptable due to the size of the watershed.

A photo of the creek channel upstream of the project site is shown below in Figure 2.2. Currently, due to the historic culvert being blocked and buried, Sackum Creek flows across the access road on site, disrupting use of the road (Figures 2.3 and 2.4).

It is important to note that Sackum Creek is considered an ephemeral stream (i.e. intermittent flow), as can be seen in Figure 2.5. Most hydrological modelling and examination focus on perennial streams that flow year-round during typical hydrological years. Sackum Creek is characterized as an ephemeral stream as there is no flow for variable time periods. Because of this, the methodologies presented in this report focus on the more conservative measures given this is an atypical situation. Ephemeral streams flow in direct response to rainfall and snowmelt, therefore the hydrology for the site was carefully analyzed with multiple scenarios considered.

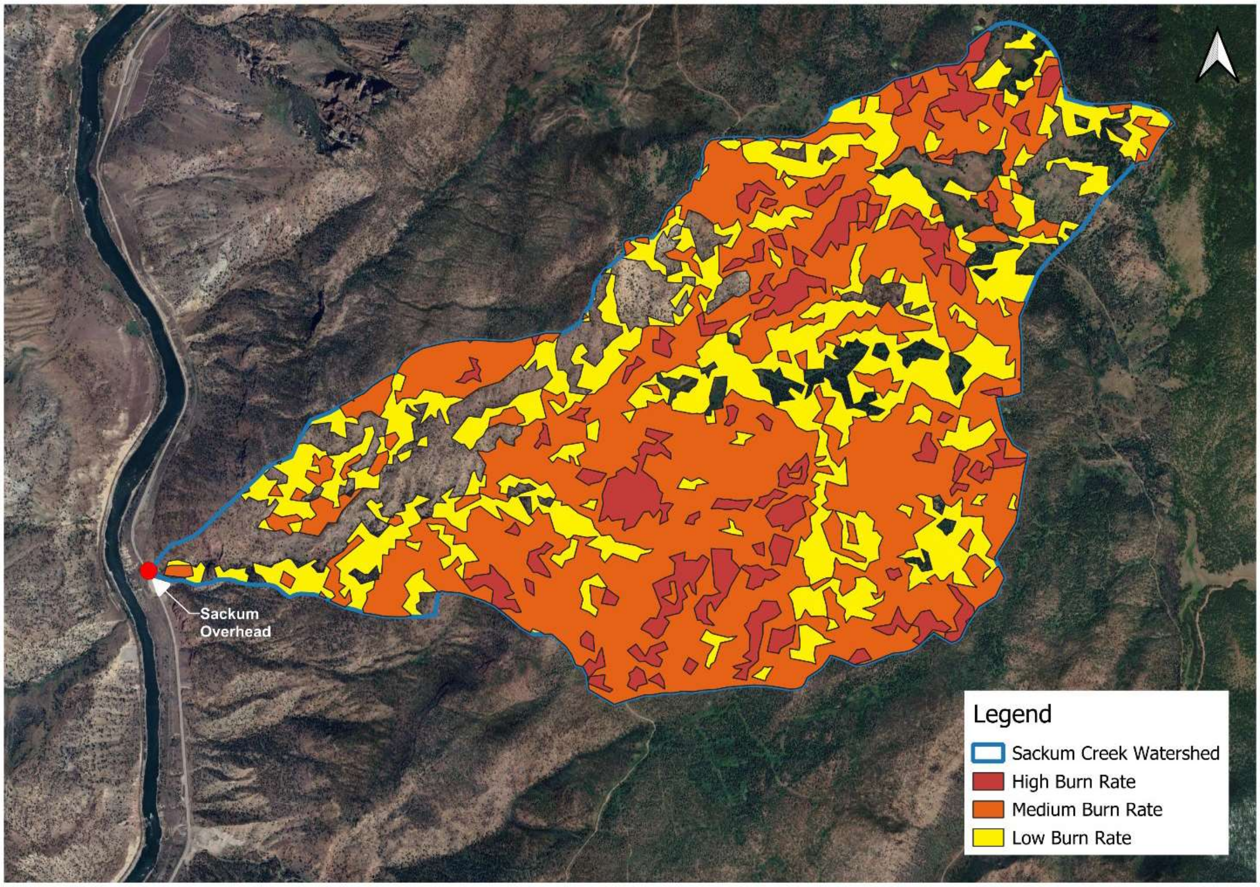


Figure 2.2 Sackum Creek Watershed - 2021 Burn Map (iMapBC)

Figure 2.3 Sackum Creek upstream of the access road, looking upstream

Figure 2.4 Approximate location of Sackum Creek crossing below the TCH, looking upstream

Figure 2.5 Sackum Creek eroding through access road, blue arrow denotes flow direction

Figure 2.6 Sackum Creek during a period of no flow

2.3 Climate Change

The BC TAC Section 1010 requires that engineering design work evaluate risk and include adaptation measures for impacts of future climate change. This is also in accordance with Engineers and Geoscientists British Columbia (EGBC) Professional Practice Guidelines on Legislated Flood Assessments in a Changing Climate in BC (2018), where design flows must include an allowance for the effects of climate change.

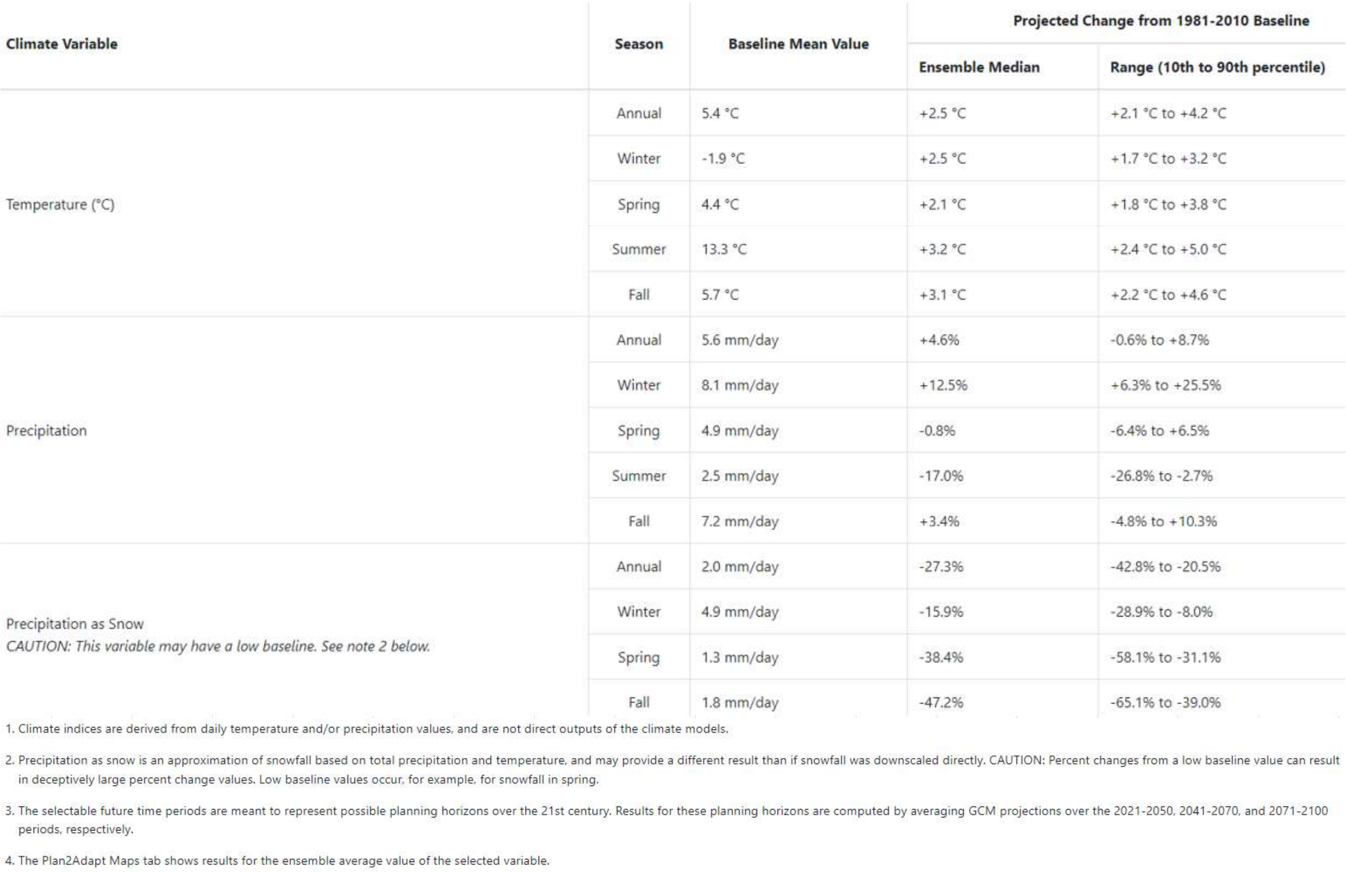
Climate adaptations to the intensity-duration-frequency (IDF) precipitation values were determined using the Western University IDF_CC Tool 7.0, which provided adjusted precipitation values for Sackum Creek using the PCIC (Pacific Climate Impacts Consortium) CMIP6 (Coupled Model Intercomparison Project Phase 6) climate adaptation values for the SSP (Shared Socio-economic Pathway) SSP1.26, SSP2.45, and SSP5.85. These pathways correspond to minor, moderate, and major climate change. The adjustments provide increases in precipitation of 11.7%, 12.3%, and 19.3%, respectively.

There is anticipated to be an increase in the amount of rainfall falling for short duration rainfall events, however, these events may not result in corresponding increases in total runoff as there is expected to be less snow available in future climate projections.

Climate change has the potential to affect peak flows through changes in temperature, precipitation, and snow depth. Climate projections for the Thompson-Nicola region are available through the PCIC Plan2Adapt online tool. These projections indicate that by the 2050s (2041 to 2070) the following changes could occur in comparison to the measured baseline from 1981 to 2010 in the spring season:

- Annual precipitation is projected to change between -0.6% and +8.7%,
- Annual precipitation as snow is projected to change between -42.8% and -20.5%,
- Temperatures are projected to change between +2.1°C and +4.2°C, and
- Future extreme one day precipitation events are projected to increase.

Figure 2.7 shows the PCIC projections for the region.



Climate Variable	Season	Baseline Mean Value	Projected Change from 1981-2010 Baseline	
			Ensemble Median	Range (10th to 90th percentile)
Temperature (°C)	Annual	5.4 °C	+2.5 °C	+2.1 °C to +4.2 °C
	Winter	-1.9 °C	+2.5 °C	+1.7 °C to +3.2 °C
	Spring	4.4 °C	+2.1 °C	+1.8 °C to +3.8 °C
	Summer	13.3 °C	+3.2 °C	+2.4 °C to +5.0 °C
	Fall	5.7 °C	+3.1 °C	+2.2 °C to +4.6 °C
Precipitation	Annual	5.6 mm/day	+4.6%	-0.6% to +8.7%
	Winter	8.1 mm/day	+12.5%	+6.3% to +25.5%
	Spring	4.9 mm/day	-0.8%	-6.4% to +6.5%
	Summer	2.5 mm/day	-17.0%	-26.8% to -2.7%
	Fall	7.2 mm/day	+3.4%	-4.8% to +10.3%
Precipitation as Snow *CAUTION: This variable may have a low baseline. See note 2 below.*	Annual	2.0 mm/day	-27.3%	-42.8% to -20.5%
	Winter	4.9 mm/day	-15.9%	-28.9% to -8.0%
	Spring	1.3 mm/day	-38.4%	-58.1% to -31.1%
	Fall	1.8 mm/day	-47.2%	-65.1% to -39.0%

1. Climate indices are derived from daily temperature and/or precipitation values, and are not direct outputs of the climate models.
2. Precipitation as snow is an approximation of snowfall based on total precipitation and temperature, and may provide a different result than if snowfall was downscaled directly. CAUTION: Percent changes from a low baseline value can result in deceptively large percent change values. Low baseline values occur, for example, for snowfall in spring.
3. The selectable future time periods are meant to represent possible planning horizons over the 21st century. Results for these planning horizons are computed by averaging GCM projections over the 2021-2050, 2041-2070, and 2071-2100 periods, respectively.
4. The Plan2Adapt Maps tab shows results for the ensemble average value of the selected variable.

Figure 2.7 PCIC Climate Change Projections

As part of the frequency analysis completed in the following Section 2.4.2, a Mann-Kendall test was completed on the data sets of the regional hydrometric gauge stations. A Mann-Kendall test is utilized to assess whether a data set has a monotonic downward or upward trend. It was found that for all hydrometric gauge stations used, no historical trends were present.

According to the Professional Practice Guidelines for Legislated Flood Assessments in a Changing Climate in BC (EGBC, 2018), for small basins where there is no historical trend evident, a 10% upward adjustment in design discharge is typically recommended, while if a statistically significant trend is evident, then an upward adjustment of 20% is typically applied.

Based on the results of the climate change analysis, a 10% adjustment is justified as there is no notable trend in regional streamflow data. This adjustment was applied to predicted design flow estimates in cases where climate change was not already factored in. Due to the nature of this report and use of PCIC values in the selected flows, a more extensive climate change analysis was not deemed necessary at this time.

2.4 Design Flow Determination

The following sections summarize the different methodologies explored for determining the design flow for the site.

2.4.1 Rational Method

According to Section 1020.06 of the BC TAC, the rational method can be considered for rural watersheds up to 10 km². The drainage area for the Sackum Overhead culvert is approximately 13.6 km² and is above the acceptable range for the rational method. As the drainage area is greater than the recommended watershed size, the rational method was only used as a rough check against other calculation methods. The rational method equation is as follows:

$$Q = \frac{CiA}{360} \qquad \text{(Equation 1)}$$

Where:

Q = Peak Flow (m³/s),

C = Runoff coefficient, variable with land use,

i = Intensity of rainfall of chosen frequency for a duration equal to time of concentration t_c (mm/hr)

t_c = Time for rainfall occurring at the most remote portion of the basin to contribute flow to the point of interest (hr)

A = Area of watershed (ha)

Applying the rational method to the Sackum Creek watershed produced a design flow of 16.3 m³/s for a return period of 200 years. The results of the rational method for multiple return periods can be seen in Table 2.4.a below. These offer a good estimation of what flows could be expected, however, are usually relatively conservative.

Table 2.4.a Rational Method – Peak Flow Estimates

Return Period (Years)	Peak Flow Estimate (m^3/s)
10	10.3
100	15.0
200	16.3

2.4.2 Regional Frequency Analysis

A regional hydrological analysis was carried out to provide an estimate of the design flows for Sackum Creek. Flood frequency analyses were conducted using the selected regional hydrometric stations detailed in Table 2.4.b below, using the HYFRAN software Version 2.2. Four different frequency distributions: Gumbel, Weibull, Three Parameter Lognormal, and Log Pearson Type III were applied to the data. The return period flows calculated for each station were plotted against the respective drainage areas, and a polynomial regression equation was fitted to obtain the design flows. The results of the flood frequency analysis for the four stations used in the regional analysis are shown in Table 2.4.b, and the locations of the stations in relation to the project site are shown in Figure 2.7.

Table 2.4.b Flood Frequency Analysis Results of Regional Hydrometric Stations

Station ID	Station Name	Drainage Area (km^2)	Peak Instantaneous Flow (m^3/s)			Years of Data
			10-year	100-year	200-year	
08LG055	Bethsaida Creek Above Highland Valley Road	15.5	0.95	1.37	1.48	19
08LF099	Arrowstone Creek near the Mouth	50.5	2.32	3.68	4.05	24
08LF100	Dairy Creek above Tsotin Lake	10.6	0.35	0.60	0.68	24
08LG056	Guichon Creek above Tunkwa Lake Diversion	78.2	2.55	3.82	4.15	58

Figure 2.8 Regional Hydrometric Gauge Stations used in Analysis with Hydrological Zones Overlain

The peak flow estimates for the 10-year (Q_{10}), 100-year (Q_{100}), and 200-year (Q_{200}) return periods at the project site are tabulated in Table 2.4.c. Using a regional frequency analysis, the 10-, 100-, and 200-year peak flows for the clear water flood were determined to be 0.77 m^3/s, 1.04 m^3/s, and 1.14 m^3/s, respectively. As there are a limited number of suitable hydrometric stations within the hydrological zone of Sackum Creek, peak flows for a catchment of this size may not accurately estimated, therefore, this methodology was discounted as being suitable for the project site.

Table 2.4.c Regional Frequency Analysis - Peak Flow Estimates

Return Period (Years)	Peak Flow Estimate (m^3/s)
10	0.77
100	1.04
200	1.14

2.4.3 BC Streamflow Inventory Method

The BC Streamflow Inventory Method, as detailed in British Columbia Streamflow Inventory (C.H. Coulson, W. Obedkoff, 1998) was applied to the Sackum Creek watershed above the TCH. The isolines that were generated by the report, and available online through iMapBC, are shown in Figure 2.7 below. The hydrometric gauge stations used in the regional analysis are included in the overlay to show the difference in expected runoff between the project site and the station locations. The equation used in the BC Streamflow Inventory Method is as follows:

$$Q_{watershed} = Q_{isoline}\left(\frac{A_{watershed}}{100\ km^2}\right) \qquad \text{(Equation 2)}$$

Where:

$Q_{watershed}$ = Peak flow corresponding to watershed (m^3/s)

$Q_{isoline}$ = Peak flow (m^3/s) per 100 km^2 as read from the isolines shown in Figure 2.7

$A_{watershed}$ = Area of the watershed (km^2)

The peak flow for the Q_{10} and Q_{100} events are calculated directly from the equation. Based on Figure 2.7 below, the $Q_{isoline}$ for the $Q_{10}$and Q_{100}events are 30 m^3/s per 100 km^2 and 50 m^3/s per 100 km^2, respectively.

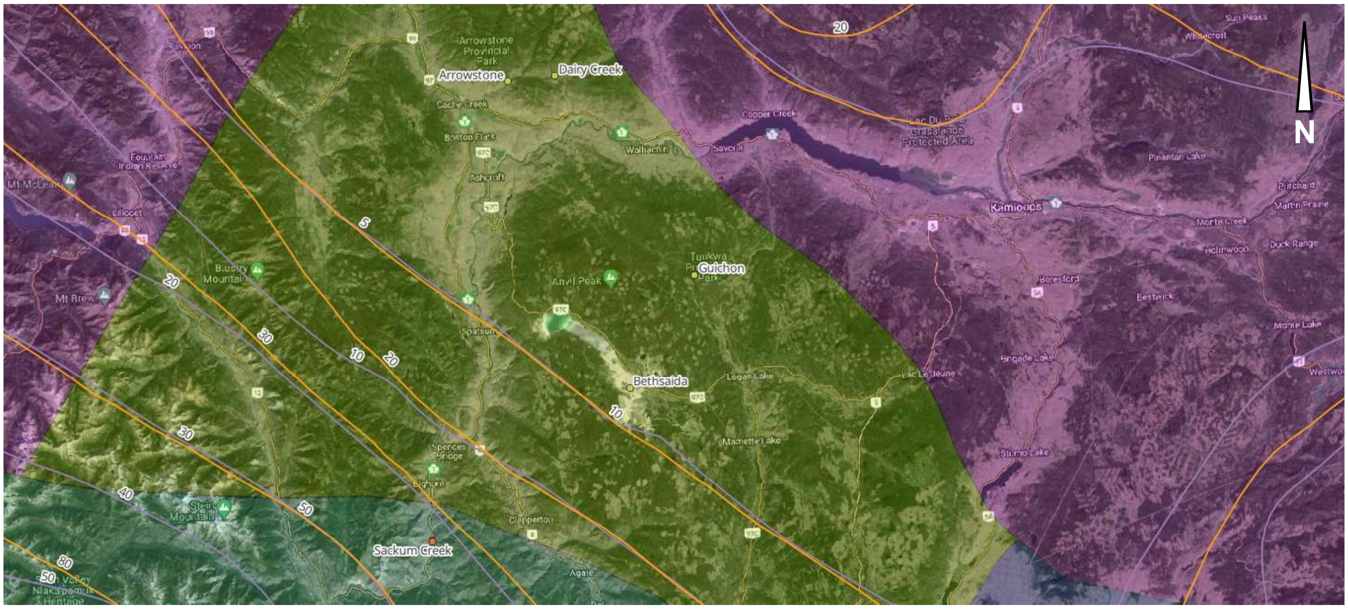


Figure 2.9 BC Streamflow Inventory Isolines. Orange lines correspond to the 100-year event while Purple lines correspond to the 10-year event.

Based on an area of 13.6 km^2, and the values of the isolines stated above, the Q_{10} and Q_{100} were estimated to be 2.7 m^3/s and 4.1 m^3/s, respectively. The peak flow estimates are summarized in Table 2.4 below.

Table 2.4.d BC Streamflow Inventory Method - Peak Flow Estimates

Return Period (Years)	Peak Flow Estimate (m^3/s)
10	2.7
100	4.1

Due to the broad nature of this analysis method, as well as its likely outdated methodology, it is often only relied upon as a check for comparison to other flow estimation methods. This method of peak flow estimation does not provide 200-year flows without extrapolating. Due to only having two reference points for extrapolation, Ecora did not deem the extrapolation of the values to a 200-year event likely to be adequately accurate.

2.4.4 BC Extreme Flood Project Method

In 2021, the BC Ministry of Forests, Lands, Natural Resource Operations & Rural Development (FLNRORD), released Bulletin 2020-1-RFFA, which is a publication concerning extreme flood determination in BC. The purpose of the publication is to assist in regional flood frequency analyses and to provide a simplified method for estimating peak flows. The bulletin devised a methodology to calculate peak flows for various return periods used in engineering design.

The BC Extreme Flood Project Method presented in Bulletin 2020-1-RFFA involves using a series of tables and equations to determine peak flows for flood return periods of 10-years, 100-years, and 200-years. The equation utilized for this method is provided below:

$$Q = 10^{A} Area^{B} MAP^{C} MedianZ^{D} \qquad \text{(Equation 3)}$$

Where:

Q = Peak Flow (m^3/s)

$Area$ = Effective Area of the Watershed (km^2)

MAP = Mean Annual Precipitation (mm)

$MedianZ$ = Mean Basin Elevation (m)

Coefficients A, B, C, D are based on Tables 2-19, 2-20, and 2-21 of Bulletin 2020-1-RFFA for the British Columbia Extreme Flood Project and vary depending on which designated “Eco-Province” a site is located within. Sackum Creek is located in the Southern Interior Zone which is numbered as 14.3 in the coefficient tables. Using the watershed drainage area of 13.6 km^2, a mean annual precipitation of 367.96 mm, and a median basin elevation of 1041 m, flows of 0.655 m^3/s for the Q_{10}, 1.457 m^3/s for the Q_{100}, and 1.735 m^3/s for the Q_{200} were estimated. The Mean Annual Precipitation was obtained from the Pacific Climate Impacts Consortium (PCIC) High-Resolution PRISM (Parameter-elevation Regressions on Independent Slopes Model) data. The peak flow results are summarized in Table 2.5 below.

Table 2.4.e BC Extreme Flood Project Method - Peak Flow Estimates

Return Period (Years)	Peak Flow Estimate (m^3/s)
10	0.66
100	1.46
200	1.74

This regional method was discounted as the peak flow estimates are unreasonably low. This could be because the catchment is too small to correlate well with the referenced data.

2.4.5 Rainfall-Runoff Method

A rainfall-runoff model was produced to simulate the behaviour of the watershed when a rainfall event occurs. Multiple IDF rainfall values, and three separate storm unit hyetographs, were generated prior to running simulations on the watershed. After these simulations were run, the resulting flows were compared, and the most appropriate flow was selected.

Watershed Characteristics

HEC-HMS (Hydrologic Engineering Center Hydrologic Modeling System) is a tool designed to simulate how rainfall turns into runoff in a watershed. It models processes such as surface runoff, soil absorption, and water movement through watersheds. This tool was used for the rainfall-runoff modelling for the site.

For the HEC-HMS simulation, the watershed was modelled using the area, SCS curve number for loss, and run through an SCS unit hyetograph transform method. The SCS Curve Number (CN) method is a simple and widely used approach to estimate how much rainfall will turn into runoff in a given watershed area. Developed by the Soil Conservation Service (now NRCS), it uses factors like soil type, land use, and surface conditions to determine how much water is infiltrated into the soils, and how much becomes surface runoff. The parameters used to model the watershed are summarized in the Table 2.4.f below.

Table 2.4.f HEC-HMS Watershed Characteristics

Watershed Characteristic	Value
SCS Curve Number	65
Drainage Area	13.6 km²
Initial Abstraction	27.4 mm
Impervious Area	0 %
Lag Time	76 min

IDF Rainfall Estimates

The BC Extreme Flood Project MetPortal is a practical tool that helps assess flood risks across British Columbia. It provides detailed rainfall data, focusing on extreme weather events, to support hydrological modelling. The IDF_CC Tool 7.0, developed by Western University, helps generate IDF curves to estimate rainfall extremes both historically and under future climate conditions. Using data from the latest climate models, potential changes in rainfall patterns are considered under various climate change scenarios - ranging from low-emission (SSP1.26) to high-emission (SSP5.85). The tool provides data that can be input into and compared between different statistical methods to best fit the data.

Three sources of IDF rainfall estimates were chosen. The first is a historical Lytton IDF from Environment and Climate Change Canada (2022), the second is the recommended precipitation from the BC Extreme Flood Project MetPortal, and the third is an ungauged location in the IDF_CC Tool 7.0. The historical Lytton IDF was curve-fit to a Gumbel distribution with a method of moments estimation, while the ungauged location produced an interpolated historical IDF fitted to both a Gumbel distribution and a Generalized Extreme Value (GEV) distribution. The IDF_CC Tool also provides IDFs under climate change scenarios, including the CMIP6 PCIC's Bias Corrected for multiple SSPs, specifically SSP1.26, SSP2.45, and SSP5.85. As all values from the IDF_CC Tool are approximations, there are two options for distributions to use generating the values: the Gumbel distribution or the GEV distribution. When looking at a comparison graph of the IDF_CC Gumbel and GEV fittings it was clear that the GEV fittings were more consistent with one another, as such the IDF_CC Gumbel fitting results were not considered further.

IDF rainfall values were selected for a 24-hour storm event with a 200-year return period, these values can be seen in Table 2.4.g below.

Table 2.4.g 200-year IDF Rainfall Values Summary Table

IDF Rainfall Estimate	200-year Return Period 24-hour Precipitation (mm)
Lytton Climate Station IDF	72.0 mm
MetPortal Recommended Precipitation	65.0 mm
IDF_CC Ungauged Historical GEV	66.0 mm
IDF_CC Ungauged CMIP6 SSP1.26 GEV	70.2 mm
IDF_CC Ungauged CMIP6 SSP2.45 GEV	74.6 mm
IDF_CC Ungauged CMIP6 SSP5.85 GEV	81.3 mm

Unit Hyetographs

Six unit hyetographs were checked to represent the temporal distributions of likely rainfall events in the subject watershed. The first is a soil conservation service (SCS) hyetograph, three were generated with the Hydrometric Report (HMR) 57 (1957) guideline, and two were BC Extreme Flood Project MetPortal unit hyetograph storms (2024).

The SCS unit hyetograph produces a high peak rainfall amount in 24 hours, the point at which the peak flow occurs varies based on the type of SCS storm. Based on the location of the site, a SCS Type IA hyetograph was selected for this analysis. Although the SCS unit distribution was included in our analysis, it was not included in the final results as we do not believe it to be a realistic rainfall scenario for this site.

HMR 57 provides guidance on the creation of 72-hour hyetograph storms for the Pacific Northwest region of the United States and portions of southern British Columbia. The first 24 hours achieve the unit hyetograph total rainfall of 1, while the remaining 48 hours achieve an additional 0.55. These HMR storm calculations were completed for the 100-year return period and results were found to be less conservative than the MetPortal storms. This combined with the older methodology eliminated the HMR distributions for further consideration in the generation of 200-year return period estimates.

The MetPortal 24-hour unit hyetograph storms that were assigned to the Sackum site were the 2006 168-hour storm and the 2009 144-hour storm. These storms are transposed historical storms that have been scaled to contain a maximum rainfall amount within a 24-hour segment of the unit hyetograph. The 24-hour period starts at hour 55 for the 2009 storm and at hour 99 for the 2006 storm. These storms were the recommended storms from the PMP generation page of MetPortal.

Figure 2.10 below shows the difference between the unit hyetographs.

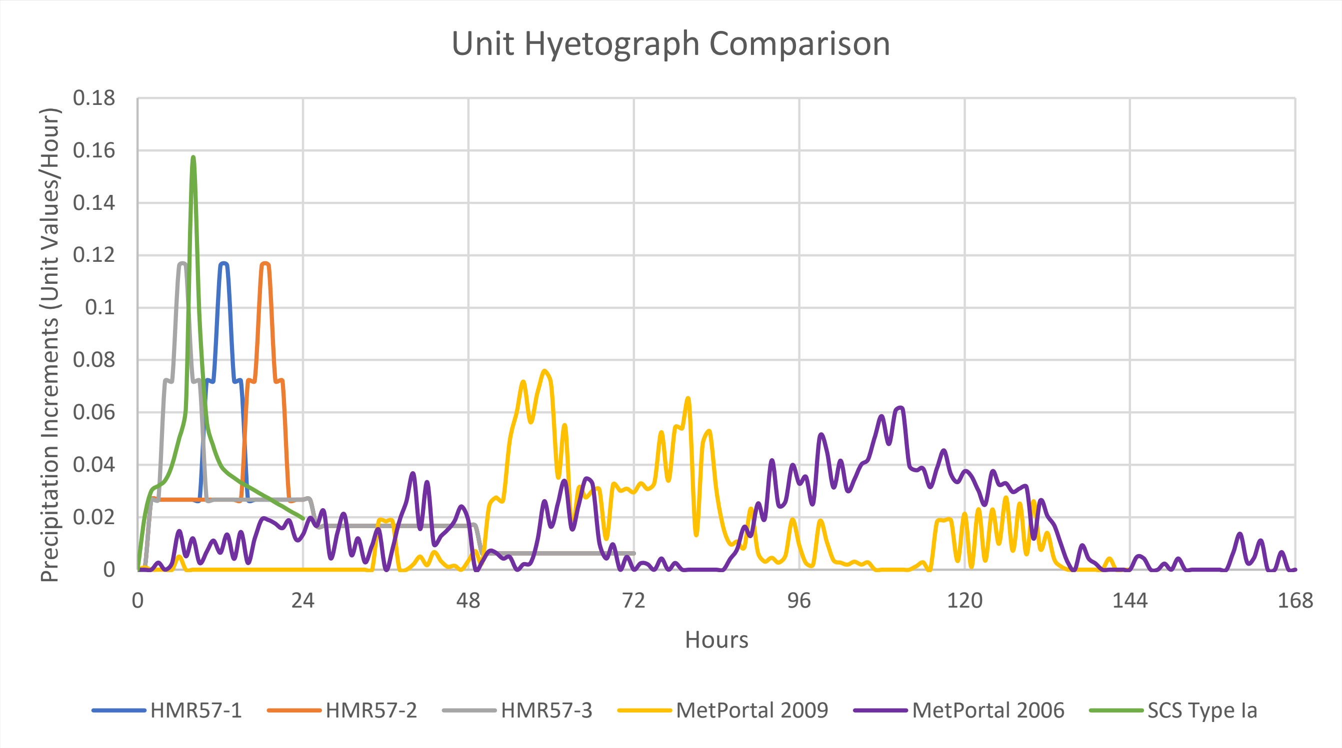


Figure 2.10 Unit Hyetograph Comparison Graph

Snowmelt

In addition to the simulation of precipitation, a spring rain-on-snow event was applied to the entire rainfall event. Historically, floods in the Fraser Canyon have been the result of spring rain-on-snow events, therefore it the most likely flood regime for the study area. The snowmelt is approximated using one of two empirical equations provided by Gray (1970). The equation for open and partly forested watersheds was selected due to percent of forest cover of the catchment and is provided below:

$$M = (0.029 + 0.0084ku_B + 0.007Pa)(Ta - 32) + 0.09 \quad \text{(Equation 4)}$$

Where:

M = Snowmelt during rainfall (inches)

Pa = Precipitation (inches)

k = Basin condensation-convection melt factor

u_B = Wind speed 50 ft above the snow surface (miles/hr)

Ta = Temperature (°F)

The above equation assumes uniform conditions across the watershed. Since this is not likely to be the case given the relief of the catchment, average values were used for windspeed and temperature.

The equation produced a result of 17.0 mm/day of snowmelt generated runoff for a Q_{100} event and 17.3 mm/day for a Q_{200} using the recommended 24-hour MetPortal precipitation values.

The average temperature was drawn from the mean temperature estimates for April and May for the site location based upon nearby climate station temperatures and interpolation based on elevation. The estimated average temperature for the site in April was 4.88 °C and 8.78 °C in May. Wind speed was estimated using an average of wind data from the Kamloops A climate normal station values from April and May; this value is 11.4 km/h.

Daily snowmelt values were determined for both April and May to be used in conjunction with storms under and over 24 hours. The April snowmelt values were used for the majority of the simulations while the May snowmelt values were used for the SCS Type IA scenario.

Table 2.4.h below details the results from the calculations.

Table 2.4.h 24-Hour Snowmelt Estimates

Scenario	2-year Event	10-year Event	100-year Event	200-year Event
Precipitation Value (mm)	37.0	54.5	68.9	74.6
April Daily Snowmelt (mm/day)	16.1	16.1	17.0	17.3
May Daily Snowmelt (mm/day)	27.1	27.1	28.7	29.3

Available snowpack values were studied from the Highland Valley (1C09A) snow survey station, the nearest station to Sackum Overhead. This snow survey station is considered to have a conservatively high amount of recorded snow as it is at a higher elevation than the site. This station recorded maximum snow water equivalent (SWE) values for any given day for each month from 1966 to 2024. The maximum values recorded in April and May are 249 mm and 101 mm respectively. This data was used to produce a frequency analysis which gave the most probable values for specific return periods. Because the Highland Valley station elevation is higher than the median elevation of the Sackum Creek watershed, Lytton's Environment Canada's (EC) snowpack data was utilized to linearly interpolate the SWE at Sackum Creek based on median elevation, for the months of April and May.

By looking at the interpolated maximum available SWE, it was determined that there wouldn't be enough snow available to melt at the rates shown in Table 2.4.h to contribute to runoff values for the longer storm events; therefore, this available SWE was set as the maximum limit of runoff produced from snowmelt. The calculated available SWE values are summarized below in Table 2.4.i.

Table 2.4.i SWE Estimates for Sackum Creek Watershed

Scenario	10-year Event	100-year Event	200-year Event
April Max Potential SWE (mm)	61.30	80.05	80.83
May Max Potential SWE (mm)	12.81	48.42	63.26

2.4.6 Results

Following the HEC-HMS simulation of the storms on the Sackum Creek watershed, the maximum flow rates were determined for each scenario including snowmelt. The results are shown in Table 2.4.j below.

Table 2.4.j HEC-HMS Q_{200} Results Summary Table

Precipitation Estimate Method 200-year Max Flow (m^3/s)						
Storm	Lytton Climate Station	MetPortal Precipitation Estimate	IDF_CC Historical Precipitation Estimate	IDF_CC SSP1.26 Precipitation Estimate	IDF_CC SSP2.45 Precipitation Estimate	IDF_CC SSP5.85 Precipitation Estimate
MetPortal 2006	16.6	15.1	15.3	17.5	17.2	18.7
MetPortal 2009	11.8	10.4	13.4	15.5	12.3	16.7

Although the Metportal 2006 storm produced higher flow rates, the Sackum Creek watershed is on the eastern most edge of the transposition limits for that storm and the entire watershed isn't within the transposition boundaries. Therefore, it was disregarded as the governing distribution at the subject location. The governing 200-year maximum clearwater design flow, including considerations for climate change (Q_{200+CC}), was determined to be 12.3 m^3/s.

2.5 Mass Movement Potential

The Sackum Creek watershed has been determined to the susceptible to debris flows/floods. Debris flows are generated by very rapid to extremely rapid flows that are fully saturated with non-plastic (i.e., plasticity index less than 5% in sand and finer fractions) debris in steep channels (Hungr et al., 2001) that have considerable momentum and high destructive potential. These flows can have magnitudes that are up to 40 times larger than the 1 in 200-year peak discharge.

Conditions that can lead to debris flows include the presence of an established channel or regular confined path and the presence of rough sorting that tends to bring the largest streambed particles close to the flow surface producing inversive grading. Geomorphological indicators of channels susceptible to debris flow generation include signs of scour along the watercourse, instability of the channel side walls, and the presence of a well-defined depositional cone or alluvial fan built up by a number of separate events along the same path.

Debris floods are characterized as sediment-charged flood events with sediment concentrations between 20% and 47% by volume (Hungr et al., 2001) and flow magnitudes of up to two times the 1 in 200-year peak discharge. Debris floods may be triggered by extreme precipitation events, or by the blockage and subsequent release of creek flows impounded by landslides or by debris blockages from debris flows in tributaries entering the main creek channel.

For detailed information, refer to the *Natural Hazard Assessment for the Sackum Overhead No. 1491 Replacement – Option A1* (Ecora, 2024).

2.5.1 Melton Ratio

The potential for debris flow, debris flood, and clear water flood were studied for Sackum Creek and assessed using the Melton ratio (Wilford et al., 2004). The Melton ratio is calculated using the equation below:

$$Melton\ Ratio = \frac{Watershed\ Relief}{\sqrt{Watershed\ Area}} \qquad \text{(Equation 5)}$$

Where:

$Watershed\ Relief$ = Elevation difference between the top and bottom of the watershed (km)

$Watershed\ Area$ = The area collecting runoff which contributes to design flows (km^2)

Sackum Creek has a watershed length of 8.5 km, a relief of 1.2 km, and an area of 13.6 km^2. Applying these parameters to Equation 5, the calculated Melton ratio for the Creek is 0.33. This result was compared to the Church and Jakob (2020) Melton Ratio distribution plot and the result is provided in Figure 3.1 below.

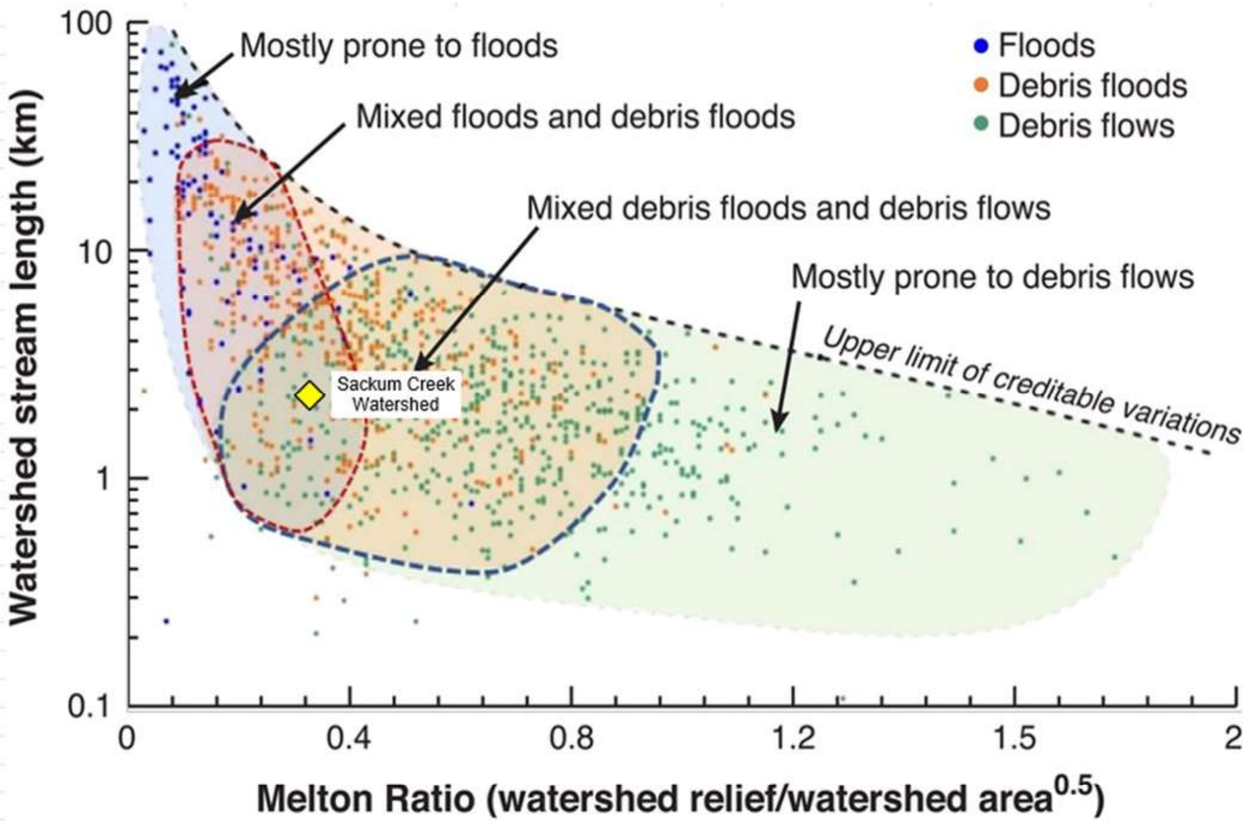


Figure 2.11 Mass Movement Potential of Sackum Creek

Based on the placement of Sackum Creek's Melton ratio in the figure above, there is the potential for the creek to generate a mixture of floods, debris floods, and debris flows. Areas scorched by wildfires typically result in elevated debris flow and debris flood risks in the years immediately after a fire event. As a result of much of the catchment burning in 2021, there is increased potential for debris floods and debris flows at the project site.

2.5.2 Bulking Factor

A bulking factor can be used to upscale the magnitude of flow to approximate the total debris flood or debris flow. It was assumed that floods or debris floods are the most likely conditions that the Sackum Creek watershed would experience. Using Figure 2.12 (Figure 58, *FLO-2D Software, Inc. (2021), FLO-2D Reference Manual*), it was assumed that these conditions would be best described by the point just past the transition zone between water flow and mud flood. This range has sediment concentrations by volume (C_v) values ranging from 0.2 to 0.5. These values were used to calculate a debris flow bulking factor in the range between 1.33 and 2.0 using the equation below. This equation is provided in the FLO-2D Reference Manual.

$$BF = \frac{1}{(1-Cv)} \qquad \textit{(Equation 6)}$$

Where:

BF = Bulking Factor

C_V = Volume of the sediment / volume of water plus sediment

Since no significant events were observed or recorded between 1948 and 2018, and that the watershed is now in recovery post-wildfire, it is reasonable to assume that typical conditions align with debris floods rather than high-intensity debris flows. The peak discharge from the 2021 event, estimated to be 17.35 m^3/s based on field measurements, was used to calibrate the design bulking factor; this equates to a bulking factor of 1.41. Using this calibrated factor and accounting for potential variability, we have estimated a design bulking factor of 1.45. This ensures a level of conservatism without overestimating flows beyond what is reasonably expected.

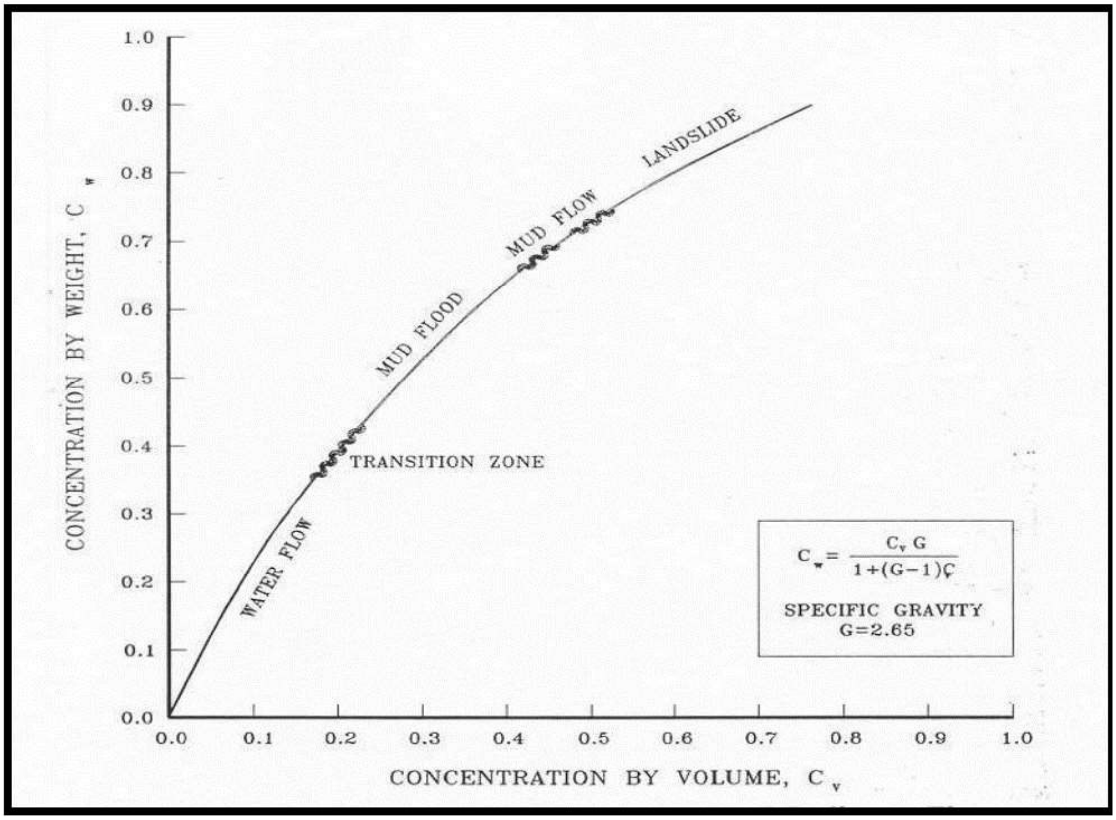


Figure 2.12 FLO-2D Concentration of Flow Types Graph (FLO-2D, 2021)

2.6 Design Flow Summary

Considering the results of the hydrologic methods reviewed in Section 2, the regional frequency analysis and isoline methods produce similar estimates of the peak flow as those from the BC Extreme Flood Project method. However, the rainfall-runoff hydrological modelling method was chosen as it accounts for more site-specific variables and gives more conservative results than the regional methods, which are more appropriately applied to larger catchments. It produces a similar peak flow to the rational method. A climate change factor of 10% was considered in estimating the design flow for most methods. As the rainfall-runoff hydrologic modelling includes a climate adjustment in the IDF_CC precipitation value, the 10% factor was not applied to the rainfall-runoff results. In addition to the climate change adjustment, a bulking factor of 1.45 was applied to the clearwater flow estimate to account for debris floods. The following table summarizes the results of the above methods to estimate 200-year flood flows.

Table 2.6.a Estimated 200-Year Design Flows for the Sackum Creek Crossing

Estimation Method	Clear Water Peak Flow Q_{200} (m³/s)	Peak Flow with Climate Change Q_{200+CC} (m³/s)	Peak Flow with Climate Change and Bulking $Q_{200+CC+BF}$ (m³/s)
Rational Method	16.3	17.9	26.0
Regional Frequency Analysis	1.3	1.4	2.0
BC Extreme Flood Project	1.7	1.9	2.8
Rainfall-Runoff Hydrologic Modelling with HEC-HMS[1]	10.4	12.3	17.8

[1] Climate Change values drawn from IDF_CC 7.0 SSP2.45, in all other cases a 10% increase in flow was assumed.

Based on the results of the flood frequency analysis and climate change impact analysis, it was determined that the design discharge would be the 200-year peak flow, plus an additional 10% to account for the effects of climate change (in cases where other methods were not used to account for climate change), plus a further 45% to account for debris bulking. The 200-year peak clear water flow without climate change was estimated to be **10.4 m³/s (Q_{200})**, which increases to **12.3 m³/s (Q_{200+CC})** with climate change projections. The design flow after accounting for climate change and bulking is **17.8 m³/s ($Q_{200+CC+BF}$).**

3. Scour Analysis

3.1 General

The channel downstream of the access road was considered for scour potential because it passes beneath the bridge structure and appurtenances and there is no existing armouring. Scour is a form of erosion within the channel. Potential types of scour considered include general, local (abutment & pier), and contraction scour. These in combination contribute to the total scour, and it is assumed that each component is independent of the others.

3.2 General Scour

General scour refers to the long-term degradation / processes of channel morphology in a permanently flowing channel. General scour was disregarded in this case due to the channel only having intermittent flows.

3.3 Contraction Scour

Contraction scour has different mechanisms that need to be considered, namely, clear-water or live-bed scour. Clear-water scour occurs when there is no transport of suspended material from the upstream reach into the crossing, and live-bed scour occurs when there is. Clear-water scour was initially considered in this analysis due to potential for backwatering in the event of a flood or debris flow (blockage in the downstream culvert). Backwatering can decrease the velocity, shear stress and the sediment transport upstream, which in turn will increase the scour at the contracted section. However, following a review of site conditions, contraction scour was disregarded due to evidence of deposition in the areas of interest.

3.4 Local Scour

Abutment scour was not considered after a thorough review of historical aerial imagery, LiDAR, site photos (2017-2024), and site reconnaissance showing no evidence of erosion at the left (south) embankment.

Pier scour is caused by the formation of vortices at the base of the structure (see Figure 3.1). Pier scour was considered as Sackum Creek has the potential to overtop the existing channel and flow around the pier during the peak design flow. Contraction scour occurs at the constriction of a bridge and can be a cyclic process (can also be related to the passing of a flood which is one of the considerations in the analysis).

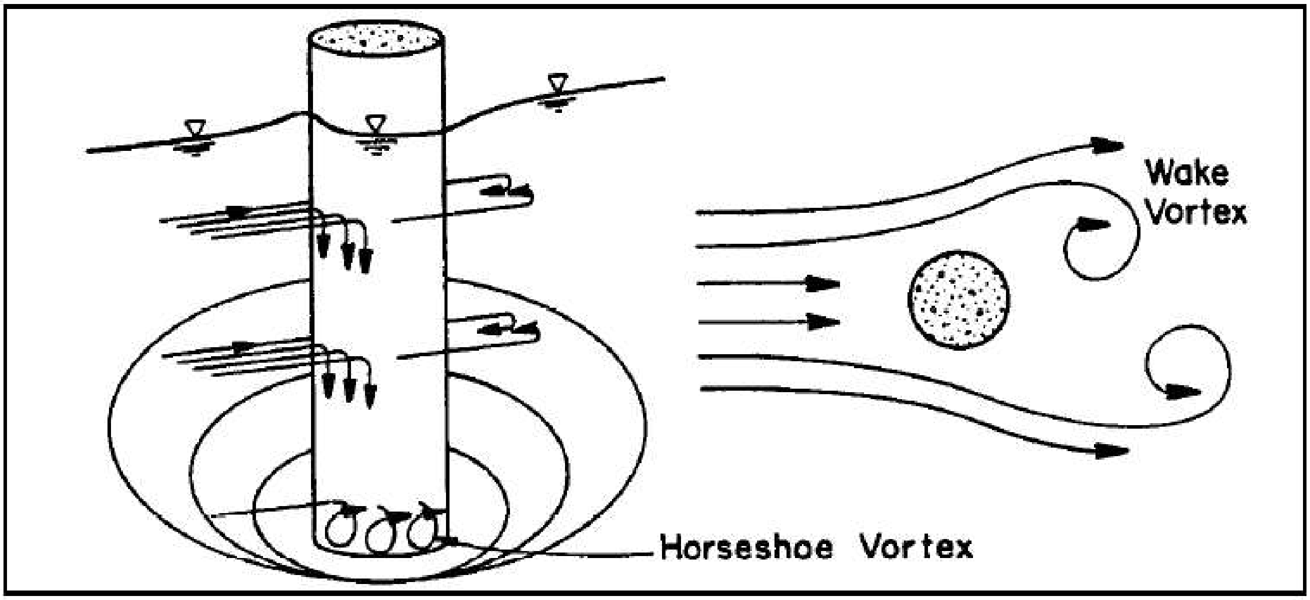


Figure 3.1 Simple schematic representation of scour at a cylindrical pier

3.5 Pier Scour

To estimate the effects of pier scour, flow velocities and depth need to be determined. The clear-water Q_{200} peak flow with climate change was calculated to be 12.3 m^3/s. A bulking factor of 1.45 was applied to the flow considered for the scour analysis due to the potential for debris flows, therefore Q_{scour} was taken as 17.8 m^3/s. This preliminary analysis utilized Manning's equation to obtain flow velocities and depths (see below), and later compared and optimized these values using hydraulic modelling software.

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \qquad \text{(Equation 7)}$$

Where:

Q = Discharge (m^3/s)

A = Flow Area (m^2)

R = Hydraulic Radius (m)

S = Channel Slope (m/m)

n = Manning's Roughness Coefficient

Pier scour depths were determined using the following equation from the HEC-18 Manual. Because the pier is within the overtopping section, outside of the main channel, the velocity at and around the pier will differ from the Mannings velocity result. In order to account for this, a HEC-RAS 2-Dimensional model was developed to confirm the velocity values and obtain a more representative value. Based on the following equation, the estimated pier scour depth was determined to be **1.95 m**.

$$\frac{y_s}{a} = 2.0\ K_1 K_2 K_3 \left(\frac{y_1}{a}\right)^{0.35} {Fr_1}^{0.43} \qquad \textit{(Equation 8)}$$

Where:

y_s = Scour depth (m)

y_1 = Flow depth directly upstream of the pier (m)

K_1 = Correction factor for pier nose shape from Figure 3.2 below.

K_2 = Correction factor for angle of attack of flow from Figure 3.2

K_3 = Correction factor for bed condition from Figure 3.2

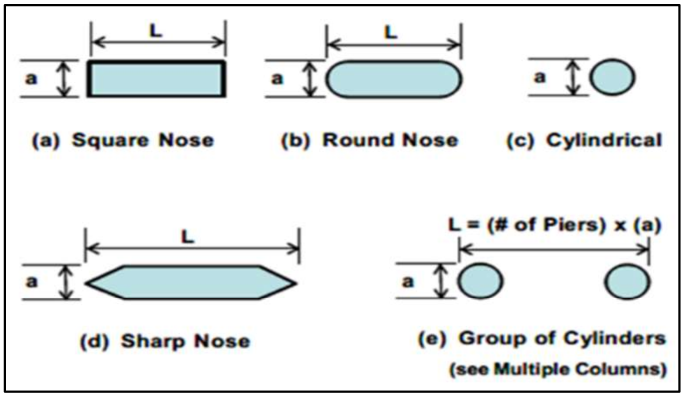
a = Pier width (m)

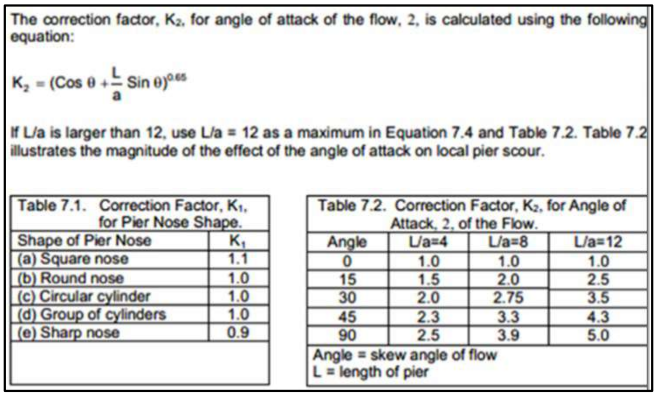
L = Length of pier (m)

Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$

V_1 = Mean velocity of flow directly upstream of the pier (m/s)

g = Acceleration of gravity (9.81 m/s^2)





The correction factor, K_2, for angle of attack of the flow, 2, is calculated using the following equation:

$$K_2 = (\mathrm{Cos}\ \theta + \frac{L}{a} \mathrm{Sin}\ \theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 7.4 and Table 7.2. Table 7.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

Table 7.1. Correction Factor, K_1, for Pier Nose Shape.

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.2. Correction Factor, K_2, for Angle of Attack, 2, of the Flow.

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier

Figure 3.2 Pier Scour Figures and Tables from HEC-18 Manual

4. Sackum Creek Crossings Design

4.1 General

Sackum Creek has three crossings that are of interest in this report: the access road, the TCH Sackum Overhead bridge, and the CPKC railway. The initial focus of this design assignment was the access road, however as this design impacts the downstream crossings of the overhead bridge and the railway crossing, the hydraulic capacity and scouring potential were investigated for the downstream channel. As the railway crossing is not within the scope of the project, Ecora recommends that its sizing, and adequacy be confirmed.

4.2 Access Road Crossing

The previous revisions of this report estimated replacement sizes of culverts for the crossing of the access road over Sackum Creek, along with a preliminary hydraulic analysis completed using HY-8 (V7.8) as part of an options analysis. Table 4.2.a below provides the results of the conceptual culvert sizing, adjusted for the updated clear-water design flow. The clear-water design flow was used in this exercise to determine the minimum possible culvert sizes.

Table 4.2.a Conceptual Access Road Culvert Sizing

Culvert Type	Culvert Size (mm)	Culvert Slope (%)	Design Flow (Q_{200+CC})
1 Round CSP	3200*	8.0	12.3
2 Round CSPs	2500	9.3	12.3
3 Round CSPs	2000	11.9	12.3
1 CSP Pipe Arch	3251x2108*†	11.0	12.3
1 Concrete Box	3600x2400*	11.0	12.3

* Structure
† H/D>1, largest pipe arch in HY-8

Due to the size of the culvert (s) that would be required to pass the design flow, these options were eliminated from consideration due to the extensive excavation that would be required. As this site has high archaeological potential, it is desirable that excavations be limited. The recent debris flows that have occurred since 2021 have filled in the creek channel upstream of the access road therefore to ensure proper coverage over the culvert under the access road, a portion of the upstream channel would need to be lowered by several metres. The culvert designs would also require a 25% embedment into the streambed due to the construction occurring on a natural watercourse, as per the BC MoTT Supplement to TAC Section 1040: Invert Elevations at Streams. This would require substantial excavation to tie the stream channel into the upstream inlet elevation.

A clear-span bridge option was also initially explored however a bridge for this crossing was determined not to be cost effective due to low traffic volumes.

A ford crossing was chosen to be the preferred crossing option for the access road at Sackum Creek. This alternative would be the most cost-effective and easily serviced option due to the ease of access and the ease of installation as the design does not require substantial excavation. The ford crossing would be constructed of articulated concrete block mats choked with gravels to create an even driving surface and prevent erosion of the crossing. One drawback to this design is that during high flow events the crossing would likely be impassable as there would be up to 0.6 m of water flowing across the crossing. This has been deemed acceptable by the design team. The ford crossing is sketched in Figure 4.1 below.

For further details on the recommended ford crossing, see drawing R2-1262-710.

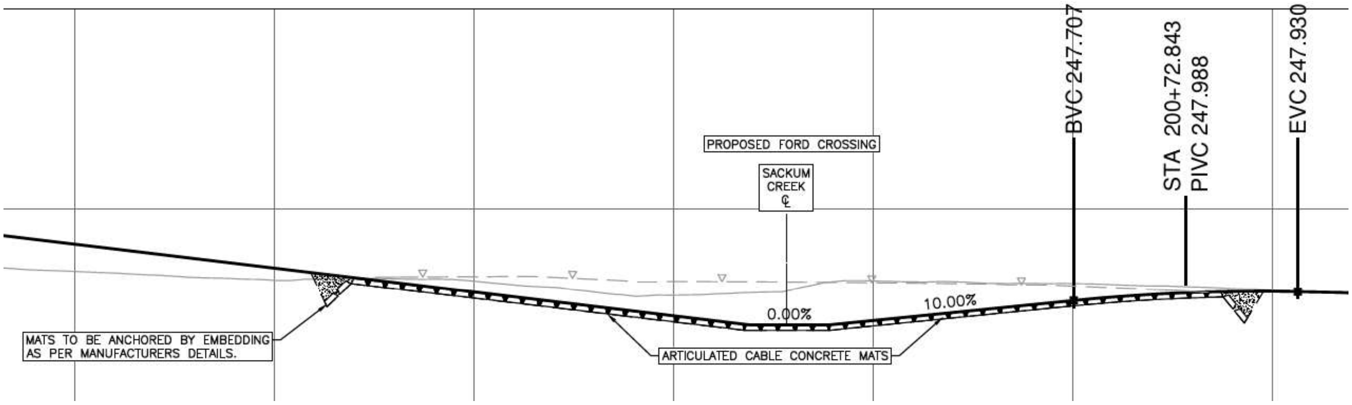


Figure 4.1 Ford Crossing - Centreline of Access Road Profile

Class 250 riprap armouring of the creek channel has been specified to be installed upstream and downstream of the ACB mats. This is to mitigate against undermining of the mats due to erosional forces. The riprap was sized using the US Army Corps of Engineering (USACE) methodology as outlined in Section 4.3.2 (Equation 8).

To ensure high-flow events are directed into the creek channel, a 1.0m high deflection berm has been specified to be installed at the northwest corner of this crossing (see Figure 4.2 below).

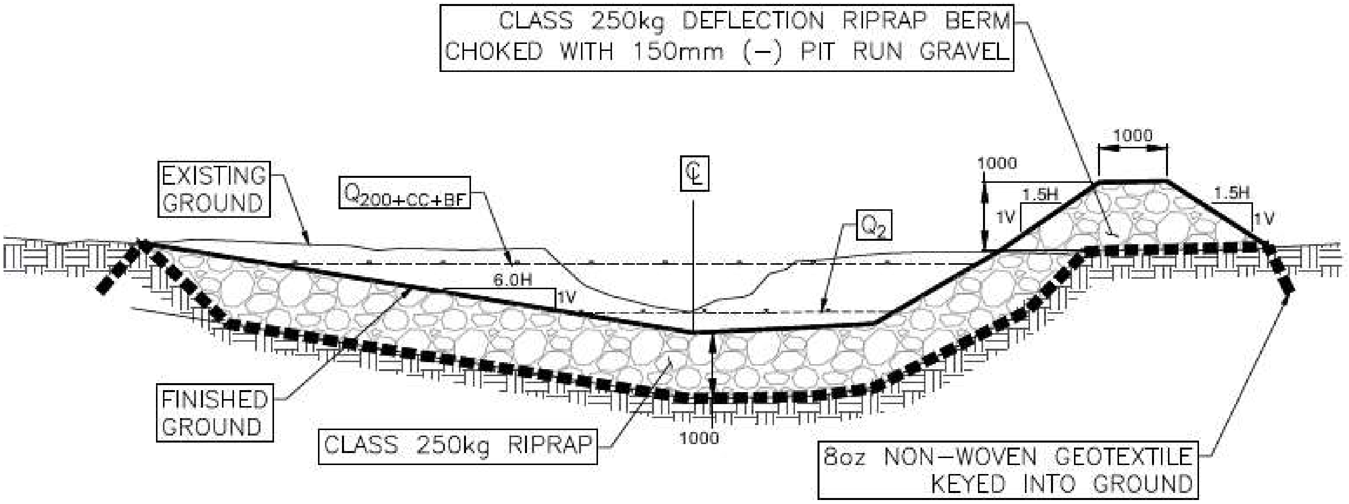


Figure 4.2 Sackum Creek Cross Section – Downstream of Access Road Crossing, Showing Deflection Berm

This berm is to be constructed of Class 250 void-filled riprap with the void filled with 150mm (-) pit-run gravel. The 1.0 m berm height is higher than what is required for the design flow plus freeboard however this dimension was chosen for constructability as the D_{100} grain size for Class 250 riprap is 1.0 m.

4.3 Erosion Protection Design

Areas of erosion concern exist around the proposed pier section, the approach to the access road crossing, and the chute downstream of the access road crossing. It is therefore proposed that a combination of class 250 kg and 500 kg riprap be placed in these areas. Riprap should conform to the MoTT Standard Specifications Section 205. For further details on layout, see Drawing 1491-127.

Detailed riprap sizing follows the National Cooperative Highway Research Program (NCHRC, 2006) guidelines to adequately size the riprap at the pier, referencing: Riprap at Bridge Piers. Also, guidance from the American Society of Agricultural Engineers (1998) was used to size the riprap at the chute section downstream of the access road ford crossing, referencing: Design of Rock Chutes. These riprap sizing methods were also compared with Figure 1030, Riprap Design Chart from the BC MoTT Supplement to TAC Geometric Design Guide.

4.3.1 Rock Chute

Directly downstream of the access road crossing, there is a sudden drop in elevation forming a chute (existing grade of 57%). It is recommended that a riprap rock chute be constructed with a proposed slope of 45%. During the design flow, the flow and velocity are relatively high, therefore, the riprap will be required to protect the soil surface in order to maintain a stable slope and prevent backwards erosion of the access road embankment. The riprap placed in the chute will also dissipate a portion of the flow energy. The riprap should be angular, interlocked, keyed in at a minimum thickness of 1.20 m (MoTT SS, Volume 1, Section 205, Table 205-D) as per the design drawings.

The equation used to estimate the D_{50} is:

$$D_{50} = \left[\frac{(qS^{0.58})}{8.07E{-}6}\right]^{1/1.89}$$ *(Equation 5)*

Where:

D_{50} = Median diameter of bed material (m)

q = Surface Flow Unit Discharge ($m^3/s/m$)

S = Slope of chute section (m/m)

The calculated D_{50} was 650 mm. Referencing MoTT SS Section 205, Table 205-C, this most closely corresponds to a riprap class of 500 kg.

4.3.2 Bridge Pier

Because the existing channel has the potential to overtop and flows can reach the proposed bridge pier, there is potential for pier scour (see Section 3.5).

The equation used to estimate the D_{50} of riprap required to mitigate the erosion potential from the design flow is:

$$D_{50} = \frac{0.692(\,V_{des})^2}{(S_g{-}1)2g}$$ *(Equation 7)*

Where:

D_{50} = Median diameter of bed material (m)

V_{des} = Design velocity for local conditions at the pier (m/s)

S_g = Specific gravity of riprap (2.5)

G = Acceleration due to gravity ($9.81\ m/s^s$)

The US Army Corps of Engineers riprap design equation was used as a cross-check:

$$D_{30} = S_f C_s C_v C_t d \left(\left(\frac{\gamma_W}{\gamma_S - \gamma_W} \right)^{0.5} \frac{V}{\sqrt{K_1 g d}} \right)^{2.5} \qquad \text{(Equation 8)}$$

Where:

D_{30} = riprap sizing of which 30 percent is finer by weight (m)

S_f = safety factor (minimum = 1.1)

C_s = stability coefficient for incipient failure, thickness = 1D_{100} (max) or 1.5D_{50} (max), whichever is greater (D_{85}/D_{15} = 1.7 to 5.2),
= 0.30 for angular rock, and
= 0.375 for rounded rock.

C_V = vertical velocity distribution coefficient,
= 1.0 for straight channels inside of bends,
= $1.238 - 0.2 \log\left(\frac{R}{W}\right)$ for outside of bends (1 for $\frac{R}{W} > 26$),
= 1.25 downstream of concrete channels, and
= 1.25 at end of dikes.

R = centreline radius of curvature of bend

W = water surface width at upstream end of bend

C_T = blanket thickness coefficient

d = local depth of flow

γ_W = unit weight of water

γ_S = unit weight of stone

V = local depth-averaged velocity

K_1 = side-slope correction factor

g = gravitational constant

The design velocity used in this equation should be representative of the conditions in the immediate vicinity of the bridge pier. As mentioned in Section 3.5 Pier Scour, a HEC-RAS 2-Dimensional model was developed to obtain a more representative velocity at the pier. In addition to this, the water flow around the pier will have localized high velocities. Therefore, the velocity should be multiplied by a factor of 1.5. This is a function of the shape of the pier (1.5 for round-nose piers or 1.7 for square-faced piers).

The estimated D_{50} was 445 mm. Referencing MoTT SS Section 205, Table 205-C, this most closely corresponds to a riprap class of 250 kg. Due to excavation limit constraints around the proposed pier footing as imposed by the CPKC rail line, the traditional method of burying riprap to the maximum scour depth around the pier was not feasible. Therefore, we are proposing that the riprap at the pier be in the form of a half-buried deflection berm. Equation 8 was used to cross-check the riprap class as the pier equation (equation 7) only applies to riprap when it is fully embedded below finished grade. The berm consists of void-filled Class 250 riprap 1.0 m high with 1.5H:1.0V side slopes and should be constructed along the south and east sides of the bridge pier. Along with the berm (above ground), the riprap will be keyed in below ground (1.0 m deep with 1.5H:1.0V side slopes) to ensure that the scour hole does not undermine the riprap berm and pile cap. If the soils were to scour to their maximum potential depth, the above ground riprap berm would be self-launching, falling into the scour hole preventing erosion around the pile cap.

4.3.3 Bridge Abutment

The left bank of Sackum Creek, which abuts the southern bridge abutment, underwent a thorough review of all available aerial imagery, LiDAR, and photos (2017-2024) of the project site. The focus of the review was to identify evidence of geomorphological changes (lateral stream migration) to the stream channel within the project limits during this time period. Due to the ephemeral nature of Sackum Creek and the observed stable condition of the stream channel through this section, no sudden erosional changes to the south abutment are expected. There may be some long-term migration in the future, however, this is expected to be slowly developing over several flood events and not pose an imminent risk to the new bridge. It was therefore determined that the risk of lateral stream migration due to erosion is low.

Note that the above analysis and conclusion is based on the current hydraulic state and known proposed modifications to the Sackum Creek channel. Any unknown future changes to the watershed or stream channel, upstream or downstream, could have impacts on the flow regime. In the event of future changes, it is recommended that the erosion potential be reassessed. MoTT should be aware that there is risk in leaving this unprotected in the event of a larger rainfall event or debris flow than the design flow used in this study. As such, the creek banks should be periodically inspected for signs of erosion and may require maintenance or reconstruction.

4.4 Existing Concrete Arch Railway Crossing

The existing crossing below the CPKC Siding Thompson Railway at Sackum Creek is a concrete arch culvert. According to 100% Detailed for Design Review Drawing No. 01491-101 by Stantec dated March 5, 2018, the existing culvert has a span of 2100 mm and a rise of 1900 mm. The drawings refer to a topographical survey performed by WSP and 3D Geomatics Inc. in October 2015. The inverts according to this survey were 234.49 m and 228.23 m with a total culvert length of 21.77 m. A detailed check of the existing culvert capacity should be performed to confirm the railway crossing is adequate for the $Q_{200+CC+BF}$ flows. There is potential for erosion of the railway embankment in the design scenario and it should be assessed for armouring to mitigate this.

5. Sensitivity Analysis

As presented in Natural Hazard Assessment for the Sackum Overhead No. 1491 Replacement – Option 1A (Ecora, 2024), the debris bulking factor (BF) for the Sackum Creek watershed was estimated to be between 1.33 and 2.0. For this design assignment, as discussed in Section 2.5, we have chosen a bulking factor of 1.45.

Below is a summary assessing the sensitivity of the BF and its effect on the riprap design. This analysis supports the decision to assign a BF of 1.45 to the design flow and shows that the design riprap class remains the same even if a relatively large (unlikely) debris flow is considered.

Table 5.0.a Bulking Factor Design Basis Sensitivity

Q_{200+CC} (m^3/s)	Bulking Factor (BF)	$Q_{200+CC+BF}$ (m^3/s)	Estimated Scour at Pier (m)	Riprap Class Pier (kg)	Riprap Class Chute (kg)
12.30	0	12.30	1.73	100	250
12.30	1.33	16.36	1.88	100	500
12.30	1.45	17.80	1.95	250	500
12.30	1.75	21.53	2.02	250	500
12.30	2.00	24.60	2.10	250	500

Basing the design flow on a BF of 1.45 provides resiliency to the design while not being overly conservative.

6. Summary and Recommendations

This report summarizes the hydrotechnical design elements for the Sackum Overhead project as well as the hydrologic and hydraulic analyses performed to inform the design. It is anticipated that Sackum Creek is subject to a mixture of floods and debris floods which increases the predicted design flow beyond a typical clear-water flood. The governing design flow for Sackum Creek is the 200-year clear-water flow, adjusted for both future climate change and debris bulking: **$Q_{200+CC+BF} = 17.8\ m^3/s$.**

Based on the results of the options analyses performed for the access road crossing, it is recommended a ford crossing be installed which comprises articulated concrete block mats to prevent erosion damage to the road structure. In addition, riprap armouring upstream and downstream of the access road is recommended to mitigate undermining and backwards erosion of the road embankment.

To guard against potential scour around the bridge pier, Ecora is recommending that a half-buried riprap deflection berm be constructed on the south and east sides of the pier.

For detailed design drawings, see series 1262-700 for the access road and ford crossing, and series 1491-100 for the riprap armouring downstream of the ford crossing.

7. Closure

We trust this report meets your present requirements. If you have any questions or comments, please contact the undersigned.

Sincerely,

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Attachments: Appendix A Calculation Summary Sheets

Version Control and Revision History

Version	Date	Prepared By	Reviewed By	Approved By	Notes/Revisions
0	2024-12-12	TWE	AJK/AGC	MJL	Issued For Review
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Appendix A

General Conditions

Calculation Sheet A - Pier Scour

Client	Ministry of Transportation & Infrastructure	**Project No.**	201740_05
Project Name	Sackum Overhead No. 1491 Replacement - Hydrotechnical	**Calculation Sheet No.**	A
Calculation Title	Pier Scour		
		Page	1 **of 1** pages

Calculation Objective

The areas downstream of the access road was considered for scour potential because no existing armouring exists, and it passes beneath the bridge structure and appurtenances. One of the requirements to design scour erosion protection, is the maximum potential scour hole depth. This calculation predicts the depth using the design discharge and channel geometry. This was used as a guideline in designing the pier protection.

Equations & Calculation Method

1. To determine if the flow upstream of the bridge is transporting bed material, the critical velocity of motion, Vc of the D50 size material must be compared with the mean velocity, V, of the flow in the overbank area of the bridge opening. This determines if the type of scour would be clear water or live bed. The equation is as follows:

$$V_c = K_u\, y^{1/6}\, D^{1/3}$$

where:

V_c = Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y = Average depth of flow upstream of the bridge, ft (m)
D = Particle size for V_c, ft (m)
D_{50} = Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u = 6.19 SI units
K_u = 11.17 English units

The calculation determined that Vc (0.92 m/s) < V (2.64 m/s), therefore live-bed scour governs.

2. Because the channel has intermittent flow, general scour was not considered.
3. Pier Scour was calculated as per HEC-18 equation 7.1 (figure 5.2 in the attached hydrotechnical memo detail the correction factors):

$$\frac{y_s}{y_1} = 2.0\, K_1\, K_2\, K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad K_2 = \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65}$$

where:

y_s = Scour depth, ft (m)
y_1 = Flow depth directly upstream of the pier, ft (m)
K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
K_3 = Correction factor for bed condition from Table 7.3
a = Pier width, ft (m)
L = Length of pier, ft (m)
Fr_1 = Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier, ft/s (m/s)
g = Acceleration of gravity (32.2 ft/s^2) (9.81 m/s^2)

Variables

The inputs to the above equations are detailed below:

1. Vc = Calculated to be 0.92 m/s ; y = 0.22 m (Mannings formula & confirmed with HEC-RAS model) ; D & D50 = 0.007 m (nearby test pit & borehole gradation samples), Ku = 6.19.

2. ys = Calculated to be 1.95 m ; y1 = 0.22 m (Mannings formula & confirmed with HEC-RAS model) ; K1 = 1 ; K2 = 1.25 (a = 0.9 m ; L = 0.9 m ; Theta = 48 deg (from civil3D)) ; K3 = 1.1 (clear-water scour) ; V1 = 2.64 m/s

Software Used

Title	Version	Validation (Y / N / N/A)
Civil3D 2024 (Geometry measurements)	*2024*	*N/A*
Excel (Calculations)	*Office 16*	
HEC-RAS (Hydraulic Modelling)	*6.5*	

References

National Engineering Handbook, August 2007 - Technical Supplement 14B
US Department of Transportation Federal Highway Administration (2012), Evaluating Scour at Bridges, Fifth Edition, Hydraulic Engineering Circular No. 18 (HEC 18), Publication No. FHWA-HIF-12-003

0	30-Oct-24	Pier Scour Calculations	TWE		
Rev	**Date**	**Description**	**By**	**Checked**	**Approved**

Calculation Sheet B - Rock Chute Riprap Sizing

Client	Ministry of Transportation & Infrastructure	**Project No.**	201740_05
Project Name	Sackum Overhead No. 1491 Replacement - Hydrotechnical	**Calculation Sheet No.**	B
Calculation Title	Rock Chute Riprap Sizing		
		Page	1 **of 1** pages

Calculation Objective

Directly downstream of the access road crossing, there is a sudden drop in elevation forming a chute (existing grade of 57%). It is proposed that a riprap rock chute be constructed, with a grade of 45%. During the predicted Q200+CC+BF, the flow and velocity are relatively high, therefore, the objective is to size riprap such that it is globally stable, considering resistance to the hydraulic load while dissipating a portion of the flow energy. The noted reference was used as a guideline in the design of the chute.

Equations & Calculation Method

1. *D50 estimation is based on equation 9 in the noted reference, the equation is as follows:*

$$D_{50} = \left[\frac{(qS^{0.58})}{8.07E-6}\right]^{1/1.89}$$

Where:

D_{50} = *Median diameter of bed material (m)*

q = Surface Flow Unit Discharge ($m^3/s/m$)

S = Slope of chute section (m/m)

The unit discharge is calculated as the discharge divided by the embankment overtopping length.

2. *The reference has also developed a relationship for angular riprap on steep slopes relating to the manning's n value. This was calculated and used in the HEC-RAS model. The formula is:*

$$n = 0.0292\,(D_{50}S_0)^{0.147}$$

Variables

1. *Discharge Q200+CC+BF = 18.45 cms (this is based on the design flow with a BF of 1.5 as specified in the hydrotechnical design memo) ; Embankment overtopping length = 7 m (average from Civil3D geometry) ; q calculated as 2.64 m^3/s/m ; Slope = 0.45 m/m ; D50 calculated as 650 mm, this corresponds to a riprap class of 500 kg.*

2. *Manning n value for use in HEC-RAS was calculated as 0.068. This uses the D50 (0.725 m) specified in section 205 volume 1 of MoTI Standard Specifications.*

3. *Riprap nominal thickness will be 1.20 m, as per MoTI SS Volume 1, Section 205, Table 205-D.*

Software Used

Title	Version	Validation (Y / N / N/A)
Civil3D 2024 (Geometry measurements)	*2024*	*N/A*
Excel (Calculations)	*Office 16*	
HEC-RAS (Hydraulic Modelling)	*6.5*	

References

K.M. Robinson, C.E. Rice, K.C. Kadavy. American Society of Agricultural Engineers (1998, Vol. 41(3):621-626), "Design of Rock Chutes".

0	30-Oct-24	Rock Chute Riprap Calculations	TWE		
Rev	**Date**	**Description**	**By**	**Checked**	**Approved**

Calculation Sheet C - Pier Riprap Berm Sizing

Client	Ministry of Transportation & Infrastructure	**Project No.**	201740_05
Project Name	Sackum Overhead No. 1491 Replacement - Hydrotechnical	**Calculation Sheet No.**	C
Calculation Title	Pier Riprap Berm Sizing		
		Page	1 **of 2** pages

Calculation Objective

Because the existing channel has the potential to overtop and flows can reach the proposed bridge pier, there is potential for scour erosion at the toe of the pier. Flow movement surrounding a pier can generate vortices and localized high velocities causing scour erosion. Initially, it was considered that embedded toe riprap be implemented surrounding the pier, however, due to the excavation limit constraints, it was decided that a deflection berm be constructed tying into the adjacent banks. The riprap pier sizing reference was still considered as a comparison in riprap class, however, the United States Army Corps of Engineering Riprap Sizing method was used to cross-check the class. Berm embedment depth was also guided by potential scour depth.

Equations & Calculation Method

1. *Estimating the riprap class for pier protection used the following formula:*

$$D_{50} = \frac{0.692(V_{des})^2}{(S_g-1)2g}$$

Where:

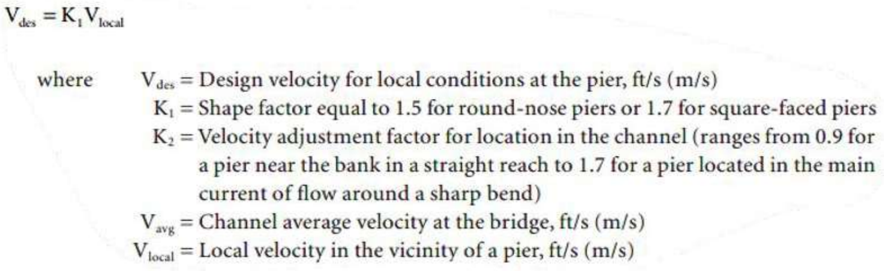
D_{50} = *Median diameter of bed material (m)*

V_{des} = *Design velocity for local conditions at the pier (m/s)*

S_g = *Specific gravity of riprap (2.5)*

g = *Acceleration due to gravity (9.81 m/s^s)*

If a local velocity is available from an hydraulic model, then only the pier shape coefficient should be used to calculate the design velocity.



$$V_{des} = K_1 V_{local}$$

where V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)

K_1 = Shape factor equal to 1.5 for round-nose piers or 1.7 for square-faced piers

K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier located in the main current of flow around a sharp bend)

V_{avg} = Channel average velocity at the bridge, ft/s (m/s)

V_{local} = Local velocity in the vicinity of a pier, ft/s (m/s)

2. *USACE Riprap design equation:*

$$D_{30} = S_f C_s C_v C_t d \left(\left(\frac{\gamma_W}{\gamma_S - \gamma_W} \right)^{0.5} \frac{V}{\sqrt{K_1 g d}} \right)^{2.5}$$

D_{30} = *riprap sizing of which 30 percent is finer by weight (m)*

S_f = *safety factor (minimum = 1.1)*

C_s= *stability coefficient for incipient failure, thickness = 1D_{100} (max) or 1.5D_{50} (max), whichever is greater (D_{85}/D_{15} = 1.7 to 5.2,*
= 0.30 for angular rock, and
= 0.375 for rounded rock.

C_v = *vertical velocity distribution coefficient,*
= 1.0 for straight channels inside of bends,
= 1.238 – 0.2 log $\left(\frac{R}{W}\right)$ for outside of bends (1 for $\frac{R}{W}$ > 26),
= 1.25 downstream of concrete channels, and
= 1.25 at end of dikes.

R = *centerline radius of curvature of bend*

W = *water-surface width at upstream end of bend*

C_T = *blanket thickness coefficient*

d = *local depth of flow*

γ_W = *unit weight of water*

γ_S = *unit weight of stone*

V = local depth-averaged velocity

K_1 = side-slope correction factor

g = gravitational constant

Variables

1. V local = 2.9 m/s (HEC RAS localized velocity) ; K1 = 1.5 (round nose piers) ; V design = 4.35 m/s ; Sg = 2.5 (Table 205-C MoTi) ; Calculated D50 value = 445 mm , this corresponds to a riprap class of 250 kg (lower limit, however, this only considers fully embedded riprap).

2. Sf = 1.1 ; Cs = 0.30 (angular rock) ; Cv = 1.35 (outside of bend where R = 5.9 & W (overtopping width) = 13 (from Civil3D geometry)) ; Ct = 1 ; d = 0.22 m ; Yw = 1000 kg/m3 ; Ys = 2500 kg/m3 ; V design = 2.9 m/s x 0.9 (Maynord side slope velocity factor) ; K1 = 0.707 (considering 1.5H:1V berm side slope with angle of repose of 40 deg) ; Calculated D30 = 383 mm, this corresponds to class 250 kg (upper limit)

Software Used

Title	Version	Validation (Y / N / N/A)
Civil3D 2024 (Geometry measurements)	*2024*	*N/A*
Excel (Calculations)	*Office 16*	
HEC-RAS (Hydraulic Modelling)	*6.5*	

References

EM 1110-2-1601 Hydraulic Design of Flood Control Channels (USACE,1994)
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0	30-Oct-24	Pier Riprap Berm Calculations	TWE		
Rev	**Date**	**Description**	**By**	**Checked**	**Approved**