DRAINAGE REPORT

Ministry of Transportation and Infrastructure

Aurum Road Washout
Detailed Design

August 9, 2023

Reviewed by:
Amanda Rust, P.Eng.
Water Resources Engineer

Prepared by:
Trevor Friedmann, P.Eng.
Water Resources Engineer

R.F. BINNIE & ASSOCIATES LTD.
300 - 4940 Canada Way,
Burnaby, BC V5G 4K6
Main: 604-420-1721

TABLE OF CONTENTS

TABLES

FIGURES

APPENDICES

REFERENCES

BC MoTI. (2019a). *BC Supplement to TAC Geometric Design Guide, 2019 3rd Edition.*

BC MoTI. (2019b). *Technical Circular T-04/19.*

BC MoTI. (2020). *2020 Standard Specifications for Highway Construction.*

Bergerud, D. J. (2004). Recognition of debris flow, debris flood and flood hazard through watershed morphometrics. *Landslides.*

C.H. Coulson, W. O. (1998). *British Columbia Streamflow Inventory.*

California Department of Transportation (Caltrans). (2020). *Highway Design Manual Chapter 810 - Hydrology.*

Eaton, B., Moore, R.D. (2010). *Compendium of forest hydrology and geomorphology in British Columbia - Regional Hydrology.*

FWHA. (2005). *HEC No. 9 - Debris Control Structures Evaluation and Countermeasures 3rd Edition.*

Government of British Columbia. (2016, February 29). *Water Sustainability Act - Water Sustainability Regulations.* Retrieved from BC Laws: https://www.bclaws.gov.bc.ca/civix/document/id/complete/statreg/36_2016#section39

Government of Canada. (2022). *Canadian Climate Normals 1982-1992 Station Data - Ladner Creek,*

and *Canadian Climate Normals 1934-1995 Station Data - Hope A.*

Retrieved from Historical Climate Data: http://climate.weather.gc.ca/climate_normals/index_e.html

Ministry of Transportation and Infrastructure. (2022). *Review of Roadway Washout along Siwash Creek Road North of Hwy 5.*

RTAC. (1982). *Drainage Manual Volume 1.*

U.S. Army Corps of Engineers. (1994). *Hydraulic Design of Flood Control Channel.*

U.S. FHWA. (2006). *HEC-14 Hydraulic Design of Energy Dissipators for Culverts and Channels.*

1 INTRODUCTION

R.F. Binnie & Associates Ltd. (Binnie) was retained by the Ministry of Transportation and Infrastructure (MoTI) to provide engineering design services for the remediation of the road washout (DFAA Site Number 14137) located at Siwash Creek Road (referred to as Aurum Road) approximately 1.28 km north of its intersection with Highway 5 (Coquihalla Highway) and approximately 18 km northeast of Hope, B.C. See Figure 1-1 for the location of the washout.

The Aurum Road washout was triggered by the November 2021 rainfall event. It is believed to have been caused by erosion due to concentrated surface runoff and groundwater seepage acting on already vulnerable cohesionless fill materials forming the road embankment (Ministry of Transportation and Infrastructure, 2022). The roadway failure generated a channelized debris flow, which ultimately created an alluvial fan across Highway 5 and Carolin Mine Road. The deposition has since been cleared to restore Highway 5 traffic.

Aurum Road is gravel surfaced and operates as an access road for the recreational use of Ladner Creek Mine, an inactive mine located at 49.504440, -121.285922. The project boundaries include two defined watercourses crossing Aurum Road, located 340 m north and 425 m south of the washout location, as well as several ditch crossings. Scope of work on Highway 5 is not included in this report. Binnie completed a desktop review of the project site to assess site conditions such as drainage infrastructure, drainage paths, and watershed characteristics based on available information. Binnie also conducted a site investigation to confirm review findings. This report summarizes the assessment, including the assumptions and methods used in the preliminary hydrologic analysis of the watersheds, estimation of design flows, and calculation of culvert sizes for the drainage paths within the project boundaries.

Figure 1-1: Washout Location

2 EXISTING DRAINAGE AND SITE CHARACTERISTICS

Watershed boundaries and streams through the project area were assessed using a combination of topographic data from the Canadian Digital Elevation Model (CDEM) dataset provided by Natural Resources Canada (NRCan), LiDAR data completed by a third-party for a nearby site and collected by MoTI, and site observations from Binnie's site inspection in Spring 2023. As the accuracy of topographic data varies between sources, preference was given to use of site observations and LiDAR where available. The CDEM dataset was used as necessary to fill in gaps in the data, notably in the upper reaches of the watershed.

2.1 Existing Site Conditions

As seen in Figure 1-1, there are two major defined drainage channels crossing Aurum Road, 340 m north and 425 m south of the washout location, referred to herein as the North Channel and the South Channel. These two channels serve as the outer boundaries of the project scope; Binnie located 2 additional crossings, between the North and South channels, appearing to convey roadway ditch flows. The identified ditch crossings are referred to in this report as the Ditch 1 and 2 Crossings.

While the available databases do not contain an inventory of culverts along Aurum Road, the presence of identified culverts was confirmed by Binnie during the site inspection. Additionally, it was previously noted in the Geotechnical Memorandum (Ministry of Transportation and Infrastructure, 2022) that an existing CSP culvert was identified within Aurum Road, approximately 80 m north of the washout site, now referred to as the Ditch 1 Crossing. The culvert condition was noted to be very poor with severe corrosion and perforations along the culvert bottom. In addition, the Geotechnical Memorandum notes the South Channel culvert crossing Aurum Road. Binnie's site visit findings are discussed further in Section 2.1.1.

Binnie did locate additional crossings further north of the North Channel Crossing. However, the additional crossings were determined to be outside of the project boundaries and are noted in the Aurum Road Washout – Additional Drainage Design scope estimate provided by Binnie June 12th, 2023.

2.1.1 Site Investigation

On May 11th, 2023, Binnie conducted a site assessment of Aurum Road, beginning approximately 500 metres north of the washout location, and walking south along the road to the South Channel crossing. Binnie's drainage engineer focused on identifying and assessing the condition of existing culverts along Aurum Road within the project boundaries.

The existing North Channel Crossing is located at Station 107+73.215 and is a CSP culvert measuring approximately 1400 mm in diameter. The culvert conveys flow from the North Channel, below Aurum Road, and downstream to a Highway 5 crossing. Upstream of the Aurum Road crossing, the channel is steep and cascades towards the culvert (Photo 2-1). There is large woody debris in the channel upstream of the crossing (Photo 2-1); additionally, some vegetation and debris is partially blocking the flow into the culvert inlet (Photo 2-3 and Photo 2-4). Binnie observed some minor bank erosion at the culvert inlet (Photo 2-3). However, the overall embankment appeared intact and heavily vegetated. Minor scour is visible on the North side of the inlet, slightly undermining the channel bank and sparsely placed riprap (Photo 2-3). At the invert of the inlet, 3-4 large rocks are partially obstructing channel flow, and additional small rocks and branches are visible along the first metre of the culvert inlet (Photo 2-4). Within the culvert, corrosion is visible on the bottom half of the culvert barrel (Photo 2-4).

A shallow flow path has formed along the east side of the roadway, flowing south and abruptly turning east to flow down the embankment, cascading over top of the culvert outlet (Photo 2-5 and Photo 2-6). Downstream of the outlet there are large branches in the channel (Photo 2-7). Large boulders downstream of the outlet form a steep cascading channel (Photo 2-8). Overgrown embankment vegetation significantly obstructs the view of the outlet, but barrel corrosion is still visible (Photo 2-7).

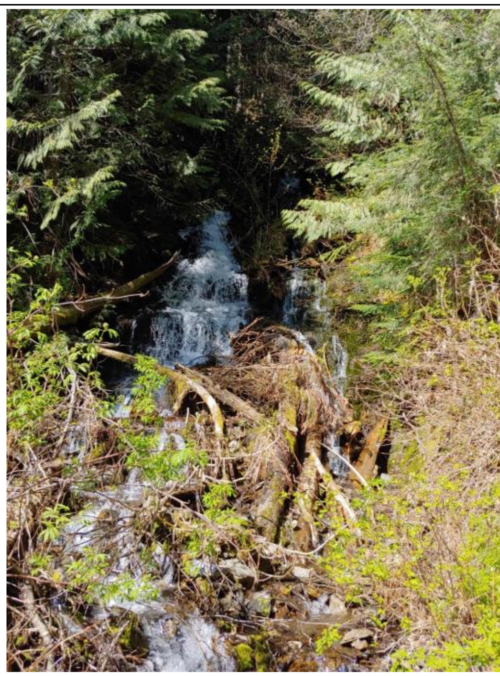
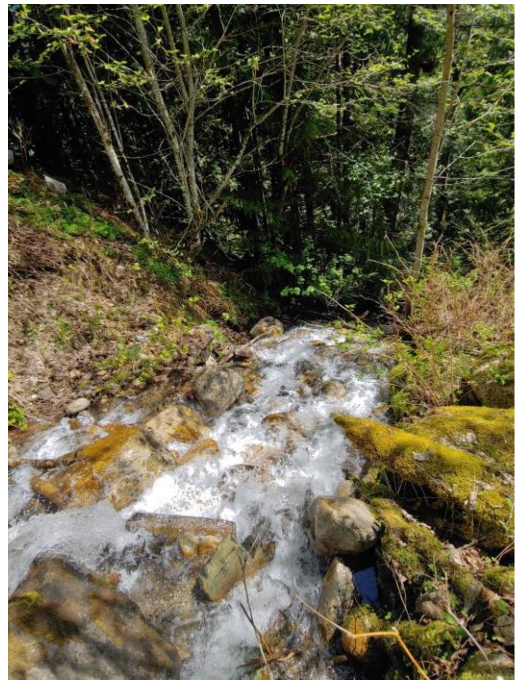
 Photo 2-1: Upstream of North Channel Crossing, West View	 Photo 2-2: Ditch Near North Channel Crossing Inlet, North View

Photo 2-3: North Channel Crossing Inlet, East View

Photo 2-4: North Channel Crossing Inlet, East View

Photo 2-5: Water Flowing South on Road Towards North Crossing Outlet, South View

Photo 2-6: Water Flow on Road Turning East and Flowing Overtop of the North Crossing Outlet, East View

 Photo 2-7: North Crossing Outlet, West View	 Photo 2-8: Channel Downstream of North Crossing, East View

The Ditch 1 crossing is located at approximately 105+11.500 and the existing culvert is approximately 900 mm in diameter, CSP pipe. The culvert conveys flow from Ditch 1, below Aurum Road and downstream to cross Highway 5. No streamflow was observed in the culvert at the time of Binnie's site inspection. Approximately 0.5 m of vertical embankment erosion was observed around the inlet (Photo 2-9). Vegetation from the embankment was also growing into the pipe's inlet (Photo 2-9). The inlet barrel is slightly deformed with visible surface damage (Photo 2-9 and Photo 2-10). Severe corrosion was observed with large invert segments, approximately 20% of the culvert barrel, missing (Photo 2-9, Photo 2-11 and Photo 2-14). Some scour along the underlying ground was visible (Photo 2-11). The barrel is also significantly deformed; the top of the culvert is collapsing inwardly (Photo 2-11). Binnie also observed erosion of the embankment surrounding the culvert outlet, as well as apparent embankment material loss immediately below the outlet (Photo 2-14 and Photo 2-15).

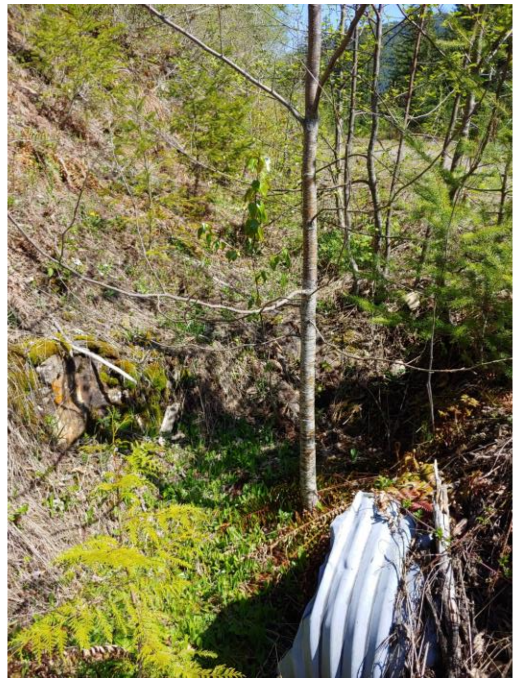
 Photo 2-9: Ditch 1 Crossing Inlet, Northeast View	 Photo 2-10: Ditch to the North of the Ditch 1 Crossing Inlet, North View
 Photo 2-11: Ditch 1 Culvert, East View from Inlet	

Photo 2-12: Roadway and Shoulder at Ditch 1 Culvert Outlet, North View

Photo 2-13: Channel Downstream from Ditch 1 Culvert Outlet, Northeast View

Photo 2-14: Ditch 1 Outlet, Northwest View

Photo 2-15: Channel Below Ditch 1 Outlet, Northwest View

The crossing at approximately 102+64.951 is a 900 mm diameter CSP pipe. The culvert conveys flow from Ditch 2, across Aurum Road and downstream to cross Highway 5. Moderate corrosion with pitting was observed along the invert of the culvert, and the barrel is slightly vertically deformed (Photo 2-16).

Sediments carried by the ditch channel, such as small rocks and leaves, were observed in the pipe upstream of the outlet (Photo 2-16). The culvert outlet projects approximately 1 m from the embankment (Photo 2-17).

Photo 2-16: Ditch 2 Culvert, West View from Outlet

Photo 2-17: Ditch 2 Culvert Outlet, East View

Approximately 89 m northeast from the South Channel crossing, Binnie observed a ditch flow path overtop the road with moderate scour in the upstream and downstream ditch; the downstream ditch joins the South Channel downstream of the South Channel crossing. Although no flow was observed in the upstream ditch or over top of the road at the time of the inspection, the roadway and the downstream ditch showed evidence of flow during precipitation events. The observed flow path is depicted in Photo 2-18. This ditch channel is referred to as Ditch 3 in this report.

 Photo 2-18: Ditch 3 Flow Path over Aurum Road, Northeast View	 Photo 2-19: Flow Path on Aurum Road to the Northeast of the South Channel Crossing Outlet

The South Channel crossing is located at approximately 100+21.085 and the existing CSP culvert is approximately 1200 mm in diameter. The culvert conveys flow from two stream channels that connect at the inlet of the culvert. Immediately upstream of the culvert is a waterfall (Photo 2-20). Northwest of the culvert inlet a second stream channel is conveyed along the roadside ditch to the inlet of the culvert (Photo 2-20). At the inlet, the riprap headwall is visible and there is little to no cover above the culvert. The inlet end of the pipe is projecting slightly, and water is flowing through 5-10% of the pipe (Photo 2-22). Binnie notes that the pipe is vertically misaligned, sloping down from the inlet to a low point approximately two thirds of the way into the culvert, before sloping slightly up towards the outlet. Water can be seen pooling in the outlet portion of the pipe, due to the misalignment (Photo 2-23). Binnie took note of a few rocks in the culvert, which appear to have been carried towards the outlet of the pipe, as well as offsetting of the joint nearest to the outlet, and corrosion on the bottom of the pipe (Photo 2-23).

Binnie was unable to access the outlet due to the steep embankment and the rocky channel shoulders. Minor scour of the left channel bank was noted at a bed in the downstream channel (Photo 2-25).

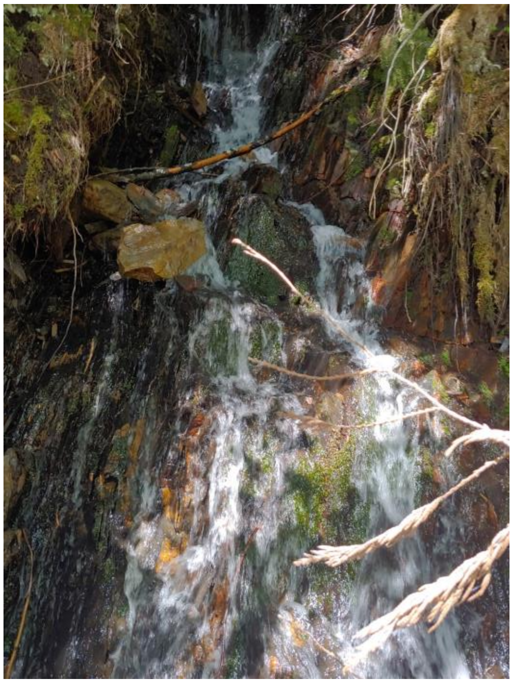


Photo 2-20: Waterfall feature in Channel Upstream of South Crossing, North View

Photo 2-21: Channel Upstream of South Crossing, Northwest View

Photo 2-22: South Crossing Culvert Inlet and Embankment, Southeast View

Photo 2-23: South Crossing Inside of Culvert, Southeast View from Inlet

 Photo 2-24: Outlet Embankment on the Southeast Side of the South Channel Crossing	 Photo 2-25: Outlet and Downstream of South Channel Crossing, Southeast View

2.2 Watershed Description

Watersheds were delineated for the existing two major drainage channels crossing Aurum Road, the North and South Channel, as well as for the Ditch 1 and 2 crossings at Aurum Road. In addition to the existing crossings, a watershed was delineated for Ditch 3, where the channel flow was observed to have overtopped the road. A catchment map with each culvert crossing is shown below. Catchment areas and drainage path information are listed in Table 2-1.

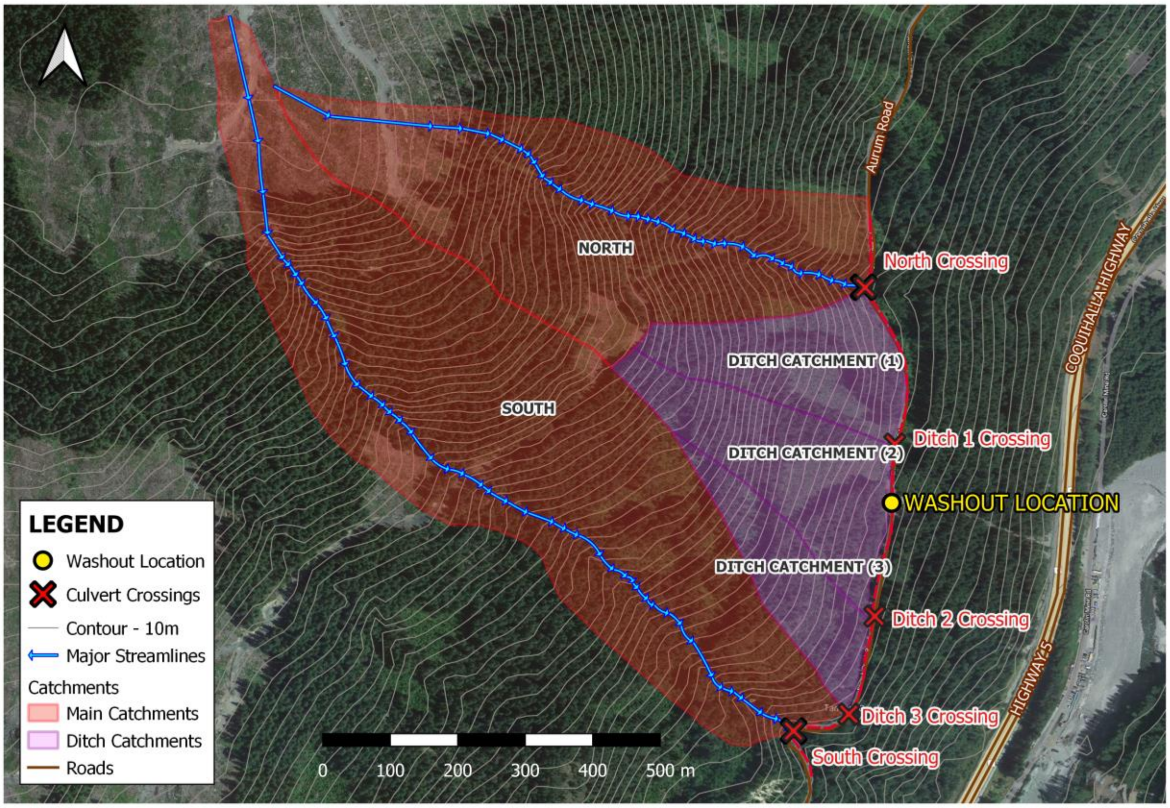


Figure 2-1: Catchment Areas and Main Drainage Paths

Table 2-1: Culvert Crossing Watershed Catchment Details

Catchment Name	Catchment Area	Average Channel Slope[(1)]	Flow Length[(1)]
	(ha)	(%)	(m)
North	18.2	57%	1110
Ditch 1	4.82	69%	512
Ditch 2	5.91	56%	645
Ditch 3	3.34	50%	567
South	30.9	44%	1579

Note:

1. The average catchment slope and flow length are calculated along the primary drainage path.

All the catchments within the project boundaries are comprised of mountainous terrain characterized by dense coniferous forest. Soils within the North and South catchments are assumed to be sand and gravel with cobbles and boulders over bedrock based on the soils that were exposed by the failure noted in the Geotechnical Memorandum (Ministry of Transportation and Infrastructure, 2022). Soils within the Ditch 1, 2 and 3 catchments are expected to be similar to the soils within the North and South catchments.

2.2.1 North Catchment

The North Catchment has a total area of 18.2 ha. Elevations in the watershed range from 1112 m to 563 m at the culvert crossing. The upper reaches of the watershed are at a slope of approximately 34 % while the remaining majority of the watershed drains at a consistently steep slope of 70 % along the 1110 m long defined channel.

2.2.2 Ditch 1 Catchment

The Ditch 1 Catchment has a total area of 4.82 ha. Elevations in the watershed range from 831 m to 541 m at the culvert crossing. Both the upper and lower reaches of the watershed have a consistently steep slope of approximately 70% along the main flow pathway, which measured 512 m in length.

2.2.3 Ditch 2 Catchment

The Ditch 2 Catchment has a total area of 5.91 ha. Elevations in the watershed range from 831 m to 515 m at the culvert crossing. The upper and middle reaches of the watershed have a consistently steep slope of 64%, with the lower reaches flattening out to approximately a 40% grade along the 645 m long undefined channel.

2.2.4 Ditch 3 Catchment

The Ditch 3 Catchment has a total area of 3.34 ha. Elevations in the watershed range from 748 m to 496 m at the crossing of the ditch with Aurum Road. The slope along the 567 m long main flow path remains consistent through the upper to lower reaches of the watershed at an average grade of 50%.

2.2.5 South Catchment

The South Catchment has a total area of 30.9 ha. Elevations in the watershed range from 1120 to 485 m at the culvert crossing. Upper reaches of the watershed are at a slope of approximately 15% while remaining majority of the watershed drains at a consistently steep slope of 50% along the defined channel. The South channel measures approximately 1579 m in length.

3 DESIGN CRITERIA

The following guidelines and references were used to develop the design criteria for the project:

- BC Supplement to TAC Geometric Design Guide – Section 1000 (BC MoTI, 2019)
- Technical Circular T-04/19: Resilient Infrastructure Engineering Design – Adaptation to the Impacts of Climate Change and Weather Extremes (BC MoTI, 2019b)
- Water Sustainability Act (WSA) Part 3 - Changes in and about a Stream, Section 39 (Government of British Columbia, 2016)

3.1 Culverts

The following design criteria to assess the hydraulic performance of anticipated culverts for the project:

- Design flows were calculated using the Rational Method for rural watersheds up to 10 km^2 (1,000 ha)
- Culverts less than 3m span on low volume roads are designed to accommodate the 100-year[1] storm peak instantaneous flow (Q100) including an allowance for climate change.
- Culverts shall be designed for a minimum 75-year design life.
- Culverts shall convey the 200-year maximum daily flow per the WSA.
- The following design criteria are recommended for typical culverts with span less than 3000mm:
    - Culverts under inlet control are designed for a headwater depth to diameter ratio (HW/D) of no greater than 1.0.
    - Culverts downstream from a natural watercourse with high debris and bedload are designed for a HW/D ratio no greater than 0.7 to address potential debris concern.
- For culverts less than 3000 mm diameter, a minimum cover of 450 mm (measured from the finished shoulder grade) over the crown of the pipe is required.
- For natural drainage channels, riprap is typically placed 0.3 to 0.6 m above the design depth of water.

[1] The 100-year flood refers to a flood event with a recurrence interval (i.e., return period) of 100 years, or a 1% annual exceedance probably (AEP) in any one year.

4 HYDROLOGIC ASSESSMENT

4.1 Climate

The project site is located 18 km northeast of Hope, BC. Binnie reviewed the availability of Environment Canada climate station data nearby (within 25 km) to better understand the climatic conditions at the project location. Details for selected climate stations from Figure 4-1 are listed in Table 4-1. Various Hope climate stations (i.e. Dog Mountain, Little Mountain and Kawkawa Lake) were eliminated due to short year of record or last year of record more than 50 years ago.

Figure 4-1: Locations of Environment Canada Climate Stations Near the Project Site

Note that none of the five stations below are active as of 2023 and the periods and lengths of record for each station varies considerably.

Table 4-1: Details of Environment Canada Climate Stations Near the Project Site

Climate Station Name	Station ID	Elevation	Years of Record	Distance from Project Site
		(m)	(# of years)	(km)
Ladner Creek	1114474	807	1982 – 1992 (11 years)	1.5
Hope A	1113540	39	1934 – 1995 (62 years)	22.1
Hope Airport	1113543	39	2012 – 2022 (11 years)	22.5
Yale	1119002	76	1984 –1994 (11 years)	15.8
Treasure Mountain	1118235	1430	1989 – 1990 (2 years)	16.5

The nearest station with climate data, Ladner Creek Climate Station (ID: 1114474), is located approximately 1.5 km north of the project site at an elevation of 807 m above sea level. Data from this station was reviewed to understand climatic conditions at the project site given its proximity and comparable elevation to the project site, despite only having 11 years of data (1982 to 1992). In addition, another climate station, Hope A (ID: 1113540), is located approximately 24 km southwest of the project site which is farther away but has a significantly longer period of record (62 years of data (1934 to 1995)) and therefore provides more reliable long-term climate normals. This station was also selected for hydrologic analysis of the washout site.

Monthly climate normals for Ladner Creek are shown in Figure 4-2. For the years between 1982 and 1992, the average annual precipitation at Ladner Creek was 226 mm. The region is rainfall dominated and received an average annual rainfall of 165 mm. In comparison, the average annual snowfall is 70.5 cm. Average monthly precipitation ranged from a maximum of 521 mm in November to a minimum of 87.5 mm in August. Maximum rainfall events typically occur from fall to winter, between October and January. Maximum snowfall events also typically occur from late fall to early spring between November and March.

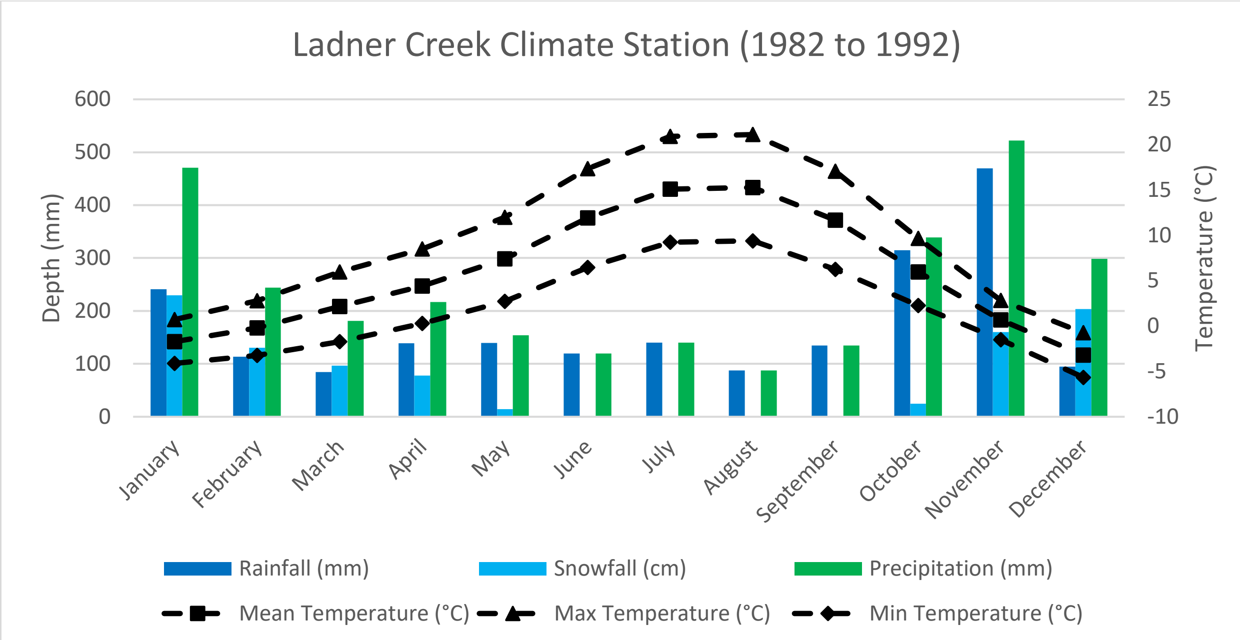


Figure 4-2: Climate Normals for Ladner Creek Climate Station (Government of Canada, 2022)

Monthly climate normals for the Hope A Climate Station are shown in Figure 4-3. For years between 1934 to 1995, the average annual precipitation at Hope A is 148 mm and average annual rainfall is 136 mm. The average annual snowfall is 13 cm. Average monthly precipitation ranged from a maximum of 267 mm in December to a minimum of 55 mm in August. Snowfall events also typically occur from late fall to winter between November and February.

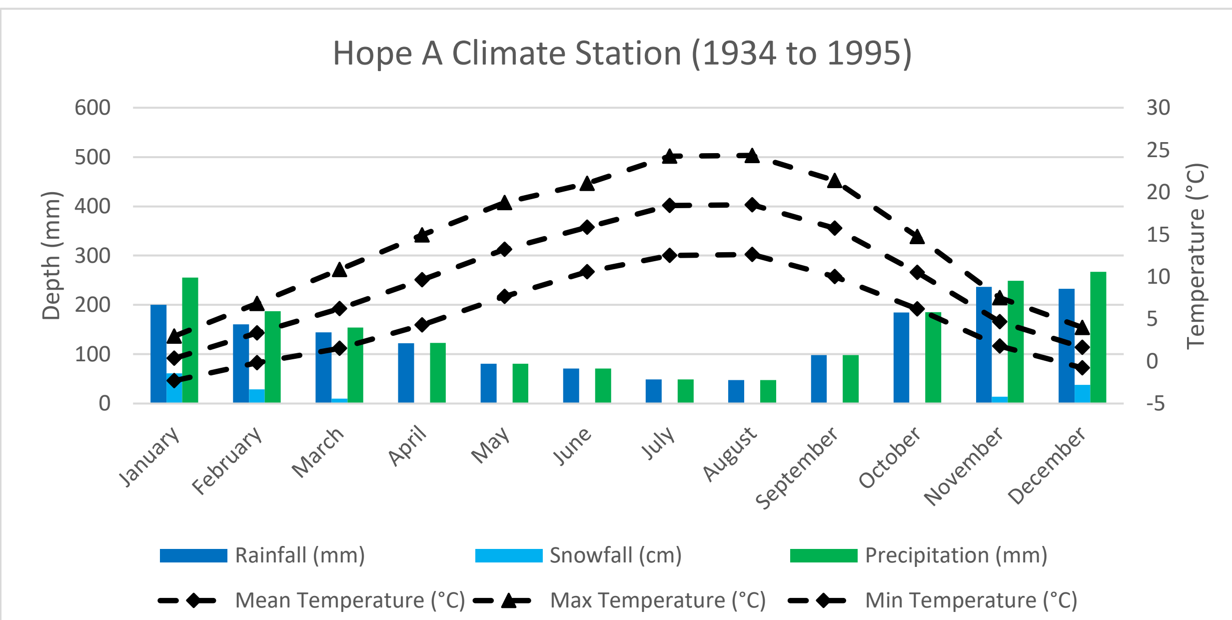


Figure 4-3: Climate Normals for Hope A Climate Station (Government of Canada, 2022)

It is expected that Ladner Creek is more representative of the climate at the site due to proximity and similarities in elevation. Although the Hope A climate station has stronger rainfall influences than

Ladner Creek due to a lower elevation, the lengthy years of record provide a more reliable long-term climate normal that shows similar percentages in precipitation as rainfall events as Ladner Creek. Based on this assessment, the project site is likely a mixed-streamflow regime (mix of rainfall and snowmelt) with a more prominent rainfall influence.

4.1.1 Precipitation Events

Intensity-Duration-Frequency (IDF) relationships for the project site and the nearby climate stations were reviewed to determine suitable return period rainfall intensities for design. Information on the nearby climate stations and their locations relative to the project site are presented in Table 4-1 and Figure 4-1.

IDF values are not available for the project site through Environment Canada as it is in an ungauged location (Lat: 49.48640794°, Lon: -121.25302722°). To extract a representative IDF curve for the project site, the IDF-CC Tool (Version 6.5), as discussed further in Section 4.1.2, was used. Given the variation in elevation, location, and length of data available for proximate Climate Stations, we used an ungauged IDF curve to predict the precipitation increase due to climate change. The IDF-CC Tool estimates IDF curves for ungauged stations based on a gridded IDF dataset of the entire Canadian landmass, developed using recorded precipitation data from nearby gauged locations (e.g., Environment Canada climate stations) and an inverse-distance weighting system.

4.1.2 Climate Change

Technical Circular T-04/19 (MoTI 2019b) states that engineering designs must incorporate information, analyses, and projections of the impacts of future climate change and weather extremes on all MoTI projects. For this project, a climate change assessment was conducted to assess the projected impacts of climate change on temperatures and precipitation in the project area. Three online climate assessment methods were used:

- Plan2Adapt Tool, developed by the Pacific Climate Impacts Consortium (PCIC), was used to assess average changes in temperature and precipitation in the Vancouver Island region over the next 50 to 80 years.
- Climate Data Method based on scaling of historic IDF values using projected future annual average temperatures.
- IDF-CC Tool Version 6.0, developed at Western University in partnership with the Institute for Catastrophic Loss Reduction (ICLR), was used to assess projected future IDF curves with climate change at nearby at the project site (ungauged).

Plan2Adapt

Climate change projections for the Fraser Valley region provided by the Plan2Adapt Tool are listed in Table 4-2. In general, precipitation is anticipated to increase moderately in the winter and decrease significantly in the summer from baseline values recorded between 1961 and 1990. As major precipitation events at the project site typically occur in the winter, the magnitude of critical storms is expected to increase by an average of 3.8%, and by upwards of 14% for the 90th percentile.

Table 4-2: Projected Climate Change for Fraser Valley in the 2080s (2070 to 2099) (PCI, 2023)

Climate Variable	Season	Projected Change from 1961 to 1990 Baseline	
		Ensemble Median(1)	Range(1) (10th to 90th Percentile)
Average Temperature (°C)	Annual	+5.1°C	+3.7°C to +6.8°C
Precipitation	Annual	+3.1%	-5.5% to 9.0%
	Summer	-22%	-60% to -2.0%
	Winter	+3.8%	-4.5% to +14%
Snowfall(2)	Annual	-69%	-75% to -55%
	Winter	-64%	-70% to -51%
	Spring	-82%	-89% to -64%

Notes:

1. The ensemble median is a mid-point value, chosen from a PCIC standard set of Global Climate Model (GCM) projections. The range values represent the lowest and highest results within the set.
2. Snowfall values are derived from temperature and precipitation.

Climate Data Method

The Climate Data method prescribed on climatedata.ca (accessed June 22, 2023) was used to estimate future extreme rainfall intensities using a temperature scaling approach. The generally methodology is as follows:

1. Estimate or obtain the rainfall intensity for the design flood return period and duration at the location of interest (e.g., the 1-hour duration 100-year flood). Data can be obtained using Environment Canada short-duration IDF relationships or another data source.

2. Estimate the change in annual average temperatures from historic to future conditions for the location of interest using climateatlas.ca, which projects future temperatures based on an ensemble of climate models. The periods used for historic and future temperature conditions are from 1981 to 2010 and from 2061 to 2090, respectively. The future 30-year period is chosen to represent a temperature for the year 2080, near the end of the life span for the project.
    a. Temperature projections are available for two Representative Concentration Pathway (RCP) scenarios, including RCP4.5 and RCP8.5. RCP8.5 was selected for the assessment as the "business-as-usual" climate scenario and represents a radiative force increase to 8.5 W/m^2 by year 2100.

3. Estimate the change in extreme rainfall at the location of interest, between historic and future conditions. The following equation utilizes the Clausius Clapeyron relation, which describes the relationship between temperate and the moisture retention capacity of the atmosphere:

$$R_p = R_C \times 1.07^{\Delta T}$$

Where,

R_p is the estimated future rainfall intensity,
R_c is the historic rainfall intensity,
ΔT is the temperature change between historic and future conditions.

Based on this methodology, Binnie estimated the percentage increase in rainfall intensity at the ungauged project site. The change in mean annual temperature between the aforementioned future and historic time periods was found to be 4.1 °C. Using the historic rainfall intensity of 25.16 mm/h for a 1-hour duration 100-year flood, the method estimates a percentage increase in rainfall intensity of 32% for the project location.

IDF-CC Tool

The IDF-CC Tool (version 6.5) was used to project future IDF curves with climate change for the project site based on the ensemble of CMIP6 Global Climate Models (GCMs) for the years from 2070 to 2100 (accessed June 22, 2023). This date range was selected as it would yield average precipitation values projected to the year 2085, close to the end of the design life of this project. The IDF-CC Tool uses historical observed data combined with data from global climate models to predict future precipitation patterns. The Shared Socio-Economic Pathway (SSP) 5-8.5 scenario was selected for the assessment as it also represents the "business-as-usual" condition, consistent with the RCP 8.5 scenario, and is the most conservative forecast of future climate change. The estimated future IDF values for the ungauged project site are presented in Table 4-3. In comparison to historical IDF values, the future IDF values correspond to a 34% increase in peak precipitation for the 100-year storm event due to climate change.

Table 4-3: IDF Table with Climate Change (SSP 5-8.5) at the Project Site (Ungauged; 49.48641°, -121.25299°)

Return Period	Rainfall Intensity (mm/hr)				
	1 hr	2 hr	6 hr	12 hr	24 hr
2-Year	13.58	10.96	7.1	5.24	3.68
5-Year	18.02	13.89	8.44	6.5	5.13
10-Year	21.1	15.82	9.25	7.26	6.17
25-Year	25.4	18.38	10.27	8.21	7.68
50-Year	29.38	20.69	11.22	9.05	9.1
100-Year	33.59	22.99	12.12	9.85	10.65

Note:

1. Results were obtained from IDF-CC Tool Version 6.5 (accessed June 22, 2023) using the ensemble CMIP6 Global Climate Models (GCMs) with PCIC-bias correction.

Based on the assessment of the three methods, the results obtained from the IDF-CC tool projections were selected for the design of drainage infrastructure for the project on the basis that the climate change data used is more site-specific than those used in the Plan2Adapt Tool and Climate Data method. The IDF-CC Tool and Climate Data methods also yielded similar percentage increases in rainfall intensity which provides a level of confidence in the projected changes. Accordingly, we have utilized a 34% increase to rainfall intensity for the project site due to climate change impacts.

4.2 Flow Generation Mechanism

Flow regimes can be categorized as either rainfall-driven (nival) or snowmelt-driven (freshet) based on climatic and topographic conditions within the watershed.

- Rainfall-driven regimes are commonly encountered in the coastal lowlands and at lower lying areas of the coastal mountains, where the project site is located (Eaton, B., Moore, R.D., 2010). These regimes often exhibit the highest annual flows in November and December.
- Snowmelt-driven regimes occur in the interior plateau and mountain regions, and in higher-elevation zones of the Coast Mountains. These regimes typically exhibit high flows in the summer months of May, June and July.

Two plots of annual daily discharge (ADD) hydrographs using data from the nearby hydrometric stations to analyze the flow generation mechanism are shown below: Coquihalla River Below Needle Creek (08MF062) (Figure 4-4), and Coquihalla River Above Alexander Creek (08MF068) (Figure 4-5). Although the watershed areas for the Hydrometric Stations (89-721 km^2) are magnitudes above the catchment areas for our projects (0.15-0.31 km^2), and the median elevation for the watershed areas for the hydrometric stations (1200-1500 m) are higher than at our project site (~800 m); the streamflow data can be representative of the smaller micro-climate at Aurum Road within the Central South Coast Mountain Range.

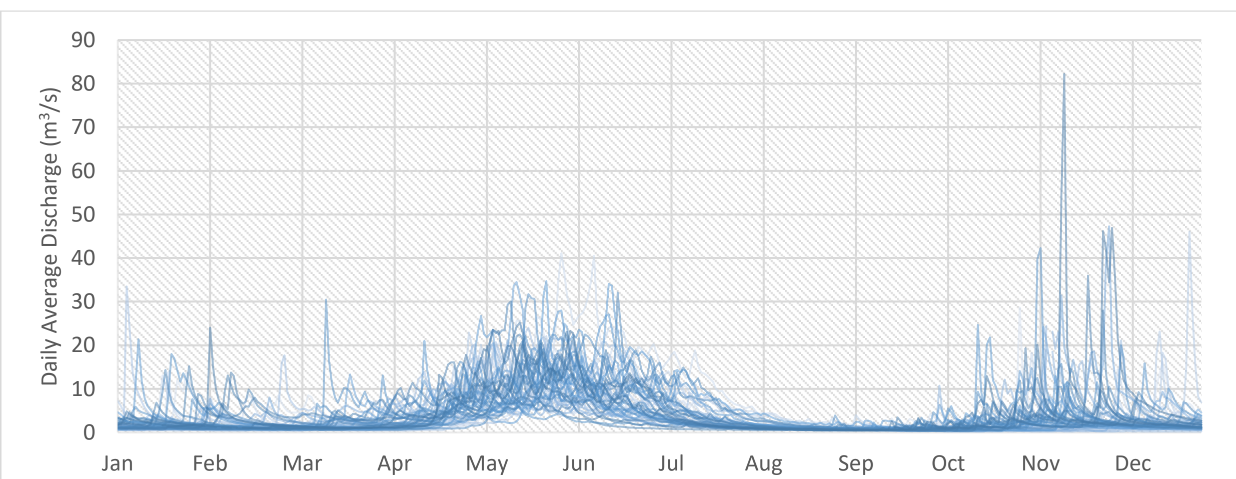


Figure 4-4: Coquihalla River Below Needle Creek Hydrometric Station – Average Daily Discharge Hydrograph

Note: Each line represents one year of flow data (1915-1987, 1989-2021).

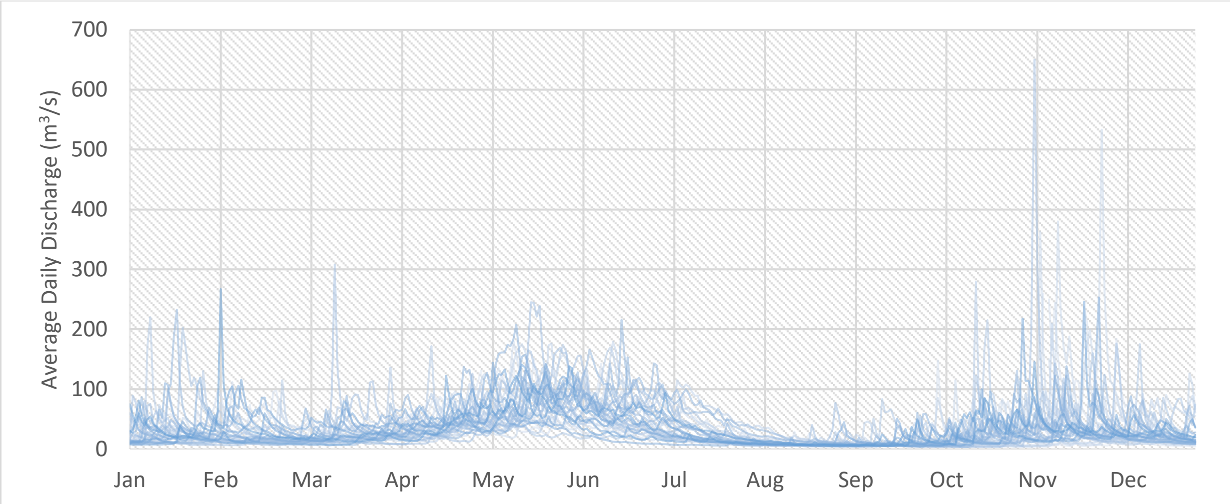


Figure 4-5: Coquihalla River Above Alexander Creek Hydrometric Station – Average Daily Discharge Hydrograph

Note: Each line represents one year of flow data (1987, 1989-2020).

Based on the assessment of annual streamflow patterns from the hydrometric stations, peak flows in the region often occur in late spring/early summer (likely due to freshet) but can also occur in the later fall/early winter. The regime for the Coquihalla River is mostly nival, but the drainage basin has sufficient low-elevation areas and proximity to coastal influences that strengthen the rainfall regime (Eaton, B., Moore, R.D., 2010) as represented by the peak ADD flows in November and December.

At Aurum Road, the rainfall influence is likely to be more prominent than the Coquihalla River due to increased mean temperatures from a lower median watershed elevation. While it is expected that the rainfall is the cause of peak runoff at the project site, there can often be a light snowpack present during major storms that is melted off by the rainfall (C.H. Coulson, 1998). The combination of rainfall and snowmelt generates a higher magnitude of runoff than would be incurred by rainfall only. Therefore, to estimate the largest peak flow for design, we assumed that the critical flood event is generated by a rain-on-snow (i.e., snowmelt) event.

4.3 Debris Flow Potential

Binnie used the Melton Ratio (Bergerud, 2004), defined as the ratio of watershed relief (i.e., elevation difference between highest and lowest points in watershed) to the square root of the catchment area, to assess the potential of debris flow and debris flood in the sub-catchment upstream of Highway 18. Bergerud et al. (2004) developed relationships between the Melton Ratio and other watershed characteristics using historic case studies to determine classification limits for floods, debris floods, and debris flows. As shown in Figure 4-6, the anticipated boundary separating clearwater flood and debris flood/flow susceptibility is defined by a Melton Ratio of 0.3.

We estimated the Melton Ratio of both the North and South catchments using available topographic data (Table 4-4). Based on the analysis, the North and South catchments are likely susceptible to debris flows or debris floods. The ditch catchments are not likely to have any significant debris mobilization as they do not have a defined upstream drainage channel.

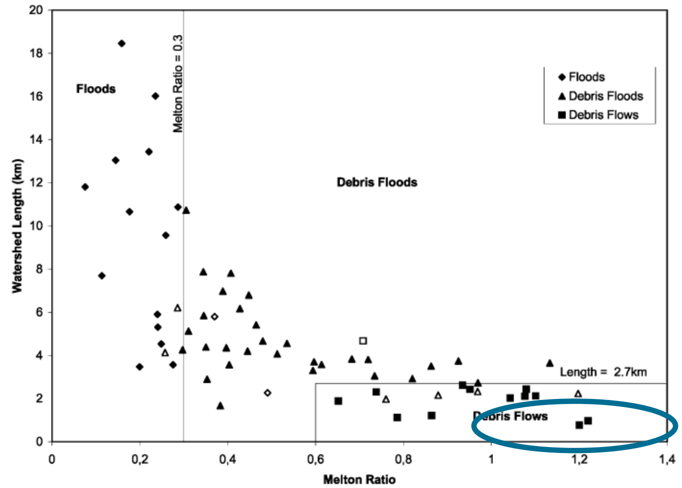


Fig. 5 Scattergram using Melton ratio and watershed length with class limits for the hydrogeomorphic processes. *Symbols without fill colour* are watersheds that do not fall within the appropriate process class limits

Figure 4-6: Proposed Boundaries Between Floods, Debris Floods, and Debris Flows

Table 4-4: Estimated Watershed Melton Ratio

Parameter	North Catchment	South Catchment
Catchment Area (km^2)	0.18	0.31
Watershed Relief (km)	0.55	0.64
Watershed Length[1] (km)	1.11	1.58
Melton Ratio	**1.29**	**1.14**

Note:

1. The watershed length is calculated as the straight-line length from the catchment outfall to the most distant point on the watershed boundary.

4.4 Design Flow

The Rational Method is recommended for use in the Supplement to TAC Guidelines (BC MoTI, 2019) for drainage areas up to 10 km^2 for rural basins. Peak flows for the project watersheds are estimated using the Rational Method for the 100-year design flood event. The peak flow is estimated based on parameters including the Runoff Coefficient (C), Time of Concentration (t_c), Rainfall Intensity (i), and Catchment Area (A) as discussed below:

- A runoff coefficient of 1.05 is adopted for design based on guidance presented in Table 1020.A in the MoTI Supplement to TAC. This value is selected using a baseline runoff coefficient of 0.90 for a forested mountainous (>30%) terrain, increased by 0.05 for a 100-year return period storm and by 0.1 for snowmelt contribution expected during rain-on-snow events.
- t_c values are adopted for design based on an average of suitable methods prescribed in the Supplement to TAC Guideline (BC MoTI, 2019) and Drainage Manual Volume 1 (RTAC, 1982) including the BC Water Management Method, Bransby-Williams Method and the SCS Curve

Number Method. The predicted time of concentration for each catchment is listed in Table 4-5 below.

Table 4-5: Time of Concentration used in the Rational Method per Catchment

Catchment Name	BC Method (Figure 1020.B)	SCS Curve Number Method	Bransby-Williams Method	Average
	(hr)	(hr)	(hr)	(hr)
North	0.58	0.23	0.35	0.39
Ditch 1	0.30	0.11	0.18	0.20
Ditch 2	0.35	0.15	0.23	0.24
Ditch 3	0.25	0.14	0.22	0.20
South	0.69	0.34	0.50	0.51

- A Curve Number of 76 is adopted for design using Table 2.2.3 and Table 2.2.6 in Drainage Manual Volume 1 (RTAC, 1982). This value is selected based on hydrologic soil group B for well-draining to moderately draining soil, a land use of woodlots and forest (10%), and antecedent moisture condition (AMC) III for an initially wet soil condition (e.g., heavy snowpack limiting soil infiltration).
- Rainfall intensities used in the design are discussed in Section 4.1and Section 4.2.

Additional details on the application of the Rational Method can be found in the Supplement to TAC Guidelines (MoTI 2019a), RTAC Drainage Manual Volume 1 (RTAC 1982), and other widely used hydrologic analysis guidelines and reference manuals.

Estimated design flows can be found in Table 4-6 below.

4.5 Estimated Peak Flows

Estimated 100-year peak flows for the catchments with and without climate change, using the Rational Method , are as follows:

Table 4-6: 100-Year Return Period Peak Flow Estimates

Catchment Name	Q100 - Historic	Q100 – with Climate Change
	(m^3/s)	(m^3/s)
North	2.28	3.04
Ditch 1	0.85	1.14
Ditch 2	0.94	1.25
Ditch 3	0.58	0.77
South	3.35	4.48

Notes:

1. Peak flows estimated using the Rational Method are an average of peak flows estimated using the three t_c methods.

100-year peak flows including climate change shown in the above table are adopted for design of proposed drainage infrastructure.

5 PROPOSED DESIGN

5.1 Culverts

Proposed culvert sizing options for the North, Ditch 1, Ditch 2, Ditch 3, and South catchment culverts crossing Aurum Road to meet the current design criteria is presented in Table 5-1.

Table 5-1: Proposed Culvert Upgrades

Design Criteria	Culvert Crossing[1]	Catchment Area	Existing Culvert Size[4]	Design Flow (Q_{100})	Proposed Culvert Size[2,3]	HW/D Ratio
		(ha)	(mm)	(m^3/s)	(mm)	
HW/D ≤ 0.7	North	18.2	1 x 1400 (circular CSP)	3.04	1 x 2000 (circular CSP)	0.62
	South	30.9	1 x 1200 (circular CSP)	4.48	1 x 2200 (circular CSP)	0.68
HW/D ≤ 1.0	Ditch 1	4.82	1 x 900 (circular CSP)	1.14	1 x 1200 (circular CSP)	0.74
	Ditch 2	5.91	1 x 900 (circular CSP)	1.25	1 x 1200 (circular CSP)	0.79
	Ditch 3	3.34	N/A	0.77	1 x 900 (circular CSP)	0.93

Notes:

1. Culvert upgrade locations are shown on Figure 2-1.
2. Proposed culvert sizes are at least the minimum typical culvert sizes required to meet the inlet- or outlet-control criteria (i.e., HW/D criterion and headloss criterion).
3. HW/D ratios are calculated based on the proposed inlet configuration and pipe sizes as shown in the Drawings and culvert sizing calculations presented in Appendix A.
4. Existing culvert sizes are approximate and are estimated from Binnie's visit to site.

For the North culvert crossing, a 2000 mm diameter circular CSP projecting culvert is proposed to meet the design criteria. The size of the proposed culvert will be able to adequately convey the predicted flow of 3.04 m^3/s in a 100-year storm event with climate change considerations. A headwater depth to diameter (HW/D) ratio of 0.62 will help to convey hydraulic flows during debris mobilization events within the natural watercourse.

We recommend a 2200 mm diameter projecting circular CSP culvert for the South Channel crossing. The South catchment is greater in size than the North catchment and has a larger predicted flow for the 100-year storm event with climate change, estimated to be 4.48 m^3/s. The HW/D ratio is 0.68 to also suit the Ministry's design criteria for debris mobilization.

For the sizing of the Ditch 1, 2 and 3 crossings, a HW/D ratio of 1.0 was utilized as these crossings do not have upstream natural watercourses and therefore are not expected to convey significant mobilized debris. Binnie recommends upsizing the Ditch 1 and 2 crossings to 1200 mm circular CSP projecting pipes, bringing the HW/D ratios to 0.74 and 0.79 respectively. In a 100-year storm event Binnie estimates a flow rate of 1.14 m^3/s at the Ditch 1 crossing and 1.25 m^3/s at the Ditch 2 crossing with climate change considered.

For the Ditch 3 crossing, Binnie proposes installation of a culvert where the ditch flows were observed to have overtopped the road, and scour was observed both upstream and downstream of Aurum Road. A 900 mm diameter circular CSP projecting pipe is proposed to be placed at approximately 49.483582°, -121.253838°, based on the predicted 100-year design flow of 0.77 m^3/s at the crossing with climate change considerations. The HW/D ratio for the crossing is approximately 0.93.

5.2 Erosion and Scour Protection

Riprap protection is proposed at the inlet and outlet of each culvert crossing to mitigate undermining of the culvert ends, lateral erosion, and scour beneath the culverts during the 100-year design flood event.

Riprap bank protection is sized based on engineering experience and guidance provided by various design manuals including HEC-15 (FWHA 2005) and HEC-23 (FWHA 2009). The recommended riprap sizes are as follows:

Table 5-2: Recommended Riprap Size for 100-Year Design Flow with Climate Change

Design Element	Riprap Class (kg)
North Culvert Crossing	250
Ditch Crossing Inlets	25
Ditch Crossing 1 Outlet	250
Ditch Crossing 2 & 3 Outlet	100
Roadway Riprap Ditch[1, 2]	25
South Culvert Crossing	100

1: For 25-year Design Flow with Climate Change.
2: Roadway Riprap Ditch located approximately Sta. 102+66 to 105+10.

5.3 Ditch Sizing

The existing ditches along Aurum Road vary in size and contain significant bedrock. The design proposes enlargement of existing ditches between Station 100+35 and 102+81, where the material allows for machine excavation, such that no blasting or drilling of bedrock is required, and the existing ditch dimensions are insufficient. The proposed minimum ditch dimensions are suited to convey the 25-year design flows, and are as follows: 300 mm depth, 1 metre base width, and 1.75H:1V side slopes. Hydraulic calculations can be observed in Appendix A.

5.4 Additional Drainage Considerations

The proposed ditch between Station 103+89 and 104+90 contains a geomembrane liner to prevent infiltration of ditch flows into the road embankment. Given the washout was exacerbated by saturated ground conditions, the impermeable geomembrane liner will help mitigate saturation of the slope, thereby helping to prevent future slope failures. Class 25 kg riprap will also be placed in the ditch to prevent erosion; to avoid significant damage and puncture of the geomembrane during riprap installation, a 150 mm granular layer is proposed between the riprap and geomembrane.

To prevent concentration of runoff and roadway drainage within the previous washout channel, concrete roadside barriers with gravel spillways and an accompanying drainage ditch are proposed on the east side of Aurum Road, between Stations 103+60 and 104+15. The proposed ditch will collect and convey roadway drainage south of the washout location, discharging to a stable location on the embankment slope. Proposed spillways include a precast concrete drainage barrier, a 50 mm gravel road surface recession adjacent to the drainage slot, and a Class 10 kg riprap outfall, similar to the Drainage Barrier Spillway with Riprap Outfall configuration depicted in the *Ministry of Transportation Standard Specifications for Highway Construction Volume 1*, *SP504-05*.

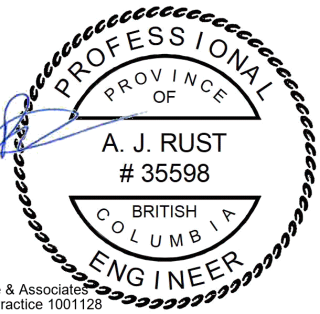
6 CLOSING

We trust you find the above suitable for your needs. Should you have any questions or comments on the information contained herein, please do not hesitate to contact the Project Manager.

Prepared by:

Trevor Friedmann, P.Eng.
Water Resources Engineer

Reviewed by:



R.F. Binnie & Associates
Permit to Practice 1001128

Amanda Rust, P.Eng.
Water Resources Engineer

APPENDIX A

DESIGN CALCULATIONS

Map Version/Date:

Design Flow Calculation

	= Input
	= Calculated

RAINFALL DATA:

Return Period IDF-CC		A	B	t0
Climate Change	100	35.3	-0.509	0
Historic	100	26.4	-0.509	0

Flat = 0%
Rolling = 1%
Moderate = 2.5%
Steep = 10%

Climate Change

Catchment Area	L (km)	L (km)	A_B	Sqrt(Area)	S (m/m)	Land Charactristics		Tc (hr)				I (mm/hr)	Qp (m3/s)
	Overland Flow	*Main Flow Path*	*(ha)*	*(km)*	*Main Channel Ave Slope*	*C*	*CN*	*BC Method (Figure 1020.B)*	*SCS Curve Number Method*	*Bransby-Williams*	*Average*	*Average*	*Average*
North Crossing		1.11	18.2	0.43	57%	1.05	76.0	0.58	0.23	0.35	0.39	57.24	3.04
Ditch 1 Crossing		0.51	4.8	0.22	69%	1.05	76.0	0.30	0.11	0.18	0.20	80.75	1.14
Ditch 2 Crossing		0.65	5.9	0.24	56%	1.05	76.0	0.35	0.15	0.23	0.24	72.55	1.25
Ditch 3 Crossing		0.57	3.3	0.18	50%	1.05	76.0	0.25	0.14	0.22	0.20	79.26	0.77
South Crossing		1.58	30.9	0.56	44%	1.05	76.0	0.69	0.34	0.50	0.51	49.65	4.48

No Climate Change

Catchment Area	L (km)	L (km)	A_B	Sqrt(Area)	S (m/m)	Land Charactristics		Tc (hr)					
	Overland Flow	*Main Flow Path*	*(ha)*	*(km)*	*Main Channel Ave Slope*	*C*	*CN*	*BC Method (Figure 1020.B)*	*SCS Curve Number Method*	*Bransby-Williams*	*Average*	*Average*	*Average*
North Crossing		1.11	18.24	0.43	57%	1.05	76.0	0.58	0.23	0.35	0.39	42.81	2.28
Ditch 1 Crossing		0.51	4.82	0.22	69%	1.05	76.0	0.30	0.11	0.18	0.20	60.39	0.85
Ditch 2 Crossing		0.65	5.91	0.24	56%	1.05	76.0	0.35	0.15	0.23	0.24	54.26	0.94
Ditch 3 Crossing		0.57	3.34	0.18	50%	1.05	76.0	0.25	0.14	0.22	0.20	59.27	0.58
South Crossing		1.58	30.95	0.56	44%	1.05	76.0	0.69	0.34	0.50	0.51	37.13	3.35

Notes:

Airport Drainage Method (RTAC pg. 2.23)

Suggested maximum area is one square kilometre.

Bransby Williams Formula (RTAC pg. 2.26)

Underestimates times of concentrations on permeable or wooded basins.

SCS Curve Number Method (RTAC pg. 2.24)

Suitable for basins up to 10 square kilometres.

Hathaway Formula (MoTI TAC Supplement pg. 1020-4)

Suitable for small urban, agricultural catchments, or small interior basins with light forest.

Kirpich Formula (MoTI TAC Supplement pg. 1020-4)

Suitable for well defined channels, for overland flow, grassed surfaces, multiply tc by 2; for over land flow concrete or asphal surfaces, multiply tc by 0.4)

SCS Upland Method (RTAC Fig. 2.4.2)

Limited to basins up to 10 square kilometres, and applies to overland flow and flow in gullyies and grassed waterways

Water Management Method (MoTI TAC Supplement Fig. 1020.B)

Limited to 25 square kilometres when used with the BC Rational Formular

Culvert Sizing

**Box dimensions must be entered manually if not square
**Embedded box culverts assumed 300 mm embedment (functional depth will be 300 mm less than shown height)

Climate Change

							New Culverts					
Basin/Station	**Design Flow**	**Existing Culverts**	**Box? (Y/N)**	**Embedded? (Y/N)**	**Material**	**Inlet Configuration**	**Location**	**Diameter**	**Height (Box only)**	**HW/D @ 2%**	**General Notes**	**Flow Condition**
	m3/s	**(mm)**						**(mm)**	**(mm)**			
North Crossing	3.04		N	N	CSP	Projecting		2000	2000	0.62	Q100, Hw/D ≤ 0.7	UNSUBMERGED
Ditch 1 Crossing	1.14		N	N	CSP	Projecting		1200	1200	0.74	Q100, Hw/D ≤1	UNSUBMERGED
Ditch 2 Crossing	1.25		N	N	CSP	Projecting		1200	1200	0.79	Q100, Hw/D ≤1	UNSUBMERGED
Ditch 3 Crossing	0.77		N	N	CSP	Projecting		900	900	0.93	Q100, Hw/D ≤1	UNSUBMERGED
South Crossing	4.48		N	N	CSP	Projecting		2200	2200	0.68	Q100, Hw/D ≤0.7	UNSUBMERGED

No Climate Change

							New Culverts					
Basin/Station	**Design Flow**	**Existing Culverts**	**Box? (Y/N)**	**Embedded? (Y/N)**	**Material**	**Inlet Configuration**	**Location**	**Diameter**	**Height (Box only)**	**HW/D @ 2%**	**General Notes**	**Flow Condition**
	m3/s	**(mm)**						**(mm)**	**(mm)**			
North Crossing	2.28		N	N	CSP	Projecting		1800	1800	0.61	Q100, Hw/D ≤ 0.7	UNSUBMERGED
Ditch 1 Crossing	0.85		N	N	CSP	Projecting		1000	1000	0.83	Q100, Hw/D ≤1	UNSUBMERGED
Ditch 2 Crossing	0.94		N	N	CSP	Projecting		1000	1000	0.89	Q100, Hw/D ≤1	UNSUBMERGED
Ditch 3 Crossing	0.58		N	N	CSP	Projecting		800	800	0.93	Q100, Hw/D <1	UNSUBMERGED
South Crossing	3.35		N	N	CSP	Projecting		2000	2000	0.66	Q100, Hw/D ≤0.7	UNSUBMERGED

Riprap Sizing - Ditches (25-Year with CC)

Mannings equation 6.1: $n = \dfrac{0.319 * d_a\text{\^{}}(1/6)}{(2.25+5.23\log(d_a/d_{50}))}$

only valid where $1.5 \le d_a/d_{50} \le$

Equation 6.2 $n = \dfrac{da\text{\^{}}(1/6)}{\sqrt{g}*f(Fr)*f(REG)*fCG)}$

where f(Fr)= (0.28*Fr/b)^(log(0.755/b))
f(REG) = 13.434*(T/D50)^0.492*b^(1.025*(T/D50)^0.118)
f(CG) = (T/da)^-b
T= Channel top width (m)
b = effective roughness concentration = 1.14*(D50/T)^0.453*(da/D50)^0.814
only valid where $0.3 < d_a/d_{50} < 1.5$

**MILD < 5%, STEEP > 10%, IN BETWEEN = LARGER OF MILD/STEEP METHOD

Station =	North Crossing	Ditch 1 Crossing	Ditch 2 Crossing	Ditch 3 Crossing	South Crossing 1
Depth of Flow =	0.134	0.241	0.245	0.180	0.142
Average Depth of Flow =	0.129	0.184	0.186	0.144	0.138
Area of Flow =	0.697	0.350	0.358	0.241	1.166
Slope of left bank (1:z) =	**1.500**	**1.750**	**1.750**	**1.750**	**1.500**
Slope of right bank (1:z) =	**1.500**	**2.000**	**2.000**	**2.000**	**1.500**
Wetted Perimeter =	5.483	2.025	2.042	1.765	8.512
Top width of water, T =	5.402	1.904	1.919	1.675	8.426
Bottom width of water =	**5.000**	**1.000**	**1.000**	**1.000**	**8.000**
Hydraulic Radius, R_h =	0.127	0.173	0.175	0.136	0.137
Avg. Channel Velocity, V =	4.355	2.388	2.604	2.411	3.847
Channel Slope (max), S =	**58%**	**7%**	**9%**	**10%**	**37%**
Mannings Coefficient, n =eqn 6.1	-0.198	0.175	0.171	0.154	-0.769
Mannings Coefficient, n =eqn 6.2	0.044	0.035	0.035	0.035	0.042
Eq. 5.7.2.1-1, n	**0.044**	0.039	0.039	0.037	0.042
Froude Number =	3.9	1.8	1.9	2.0	3.3
f(Fr) =	5.601	1.134	1.161	1.210	5.239
b =	0.122	0.344	0.347	0.333	0.118
f(REG) =	2.449	8.863	8.974	8.968	2.596
f(CG) =	0.633	0.447	0.446	0.442	0.615
Ditch Capacity, Q =	**3.04**	**0.84**	**0.93**	**0.58**	**4.49**
1:100 year Design Flow =	**3.04**	**0.84**	**0.93**	**0.57**	**4.48**
Trial size d50 (m) =	0.575	0.270	0.270	0.200	0.425
Calculated d50 (HEC-15)	7.998	0.179	0.360	0.275	24.957
Calculated d50 (Rock Chutes)	0.473	0.197	0.223	0.174	0.397
Class size of rip rap	FALSE	Class 10kg	Class 100kg	Class 50kg	FALSE
CHECK Froude Number Subcritical < 1 Critical = 1 Supercritical > 1	3.9	1.8	1.9	2.0	3.3
CHECK 1.5<da/d50<185	Method not valid- use e	Method not valid- use equati	Method not valid- use equatio	Method not valid- use equation 6.	Method not valid- use equation 6.2
CHECK 0.3<da/d50<1.5	Method not valid Use Eq. 5.7.2.1-1-	equation 6.2 = OKAY	equation 6.2 = OKAY	equation 6.2 = OKAY	equation 6.2 = OKAY
HEC 15 results (Not applicable to steep slope)	Class 250kg	Class 25kg	Class 25kg	Class 10kg	Class 100kg
Robinson, Rice, Kadavy (Rock Chutes)	Class 250kg	Class 25kg	Class 25kg	Class 10kg	Class 100kg
NOTES:					
SELECTED SIZE	**Class 250kg**	**Class 25kg**	**Class 25kg**	**Class 10kg**	**Class 100kg**

Riprap Sizing - Watercourses & Ditch Crossing Inlets (100 -Year with CC)

Mannings equation 6.1: $n = \frac{0.319 d_a^{(1/6)}}{(2.25+5.23\log(d_a/d_{50}))}$
only valid where $1.5 \le d_a$

Equation 6.2 $n = \frac{da^{(1/6)}}{\sqrt{g}*f(Fr)*f(REG)*fCG)}$

where f(Fr)= (0.28*Fr/b)^(log(0.755/b))
f(REG) = 13.434*(T/D50)^0.492*b^(1.025*(T/D50)^0.118)
f(CG) = (T/da)^-b
T= Channel top width (m)
b = effective roughness concentration = 1.14*(D50/T)^0.453*(da/D50)^0.814
only valid where $0.3<d_a/d_{50}<1.5$

**MILD < 5%, STEEP > 10%, IN BETWEEN = LARGER OF MILD/STEEP METHOD

Station =	North Crossing	Ditch 1 Crossing	Ditch 2 Crossing	Ditch 3 Crossing	South Crossing 1
Depth of Flow =	0.134	0.305	0.315	0.230	0.142
Average Depth of Flow =	0.129	0.224	0.230	0.177	0.138
Area of Flow =	0.697	0.479	0.501	0.329	1.166
Slope of left bank (1:z) =	**1.500**	**1.750**	**1.750**	**1.750**	**1.500**
Slope of right bank (1:z) =	**1.500**	**2.000**	**2.000**	**2.000**	**1.500**
Wetted Perimeter =	5.483	2.297	2.339	1.978	8.512
Top width of water, T =	5.402	2.144	2.181	1.863	8.426
Bottom width of water =	**5.000**	**1.000**	**1.000**	**1.000**	**8.000**
Hydraulic Radius, R_h =	0.127	0.209	0.214	0.166	0.137
Avg. Channel Velocity, V =	4.355	2.358	2.498	2.397	3.847
Channel Slope (max), S =	**58%**	**7%**	**9%**	**10%**	**37%**
Mannings Coefficient, n =eqn 6.1	-0.198	0.191	0.293	0.313	-0.769
Mannings Coefficient, n =eqn 6.2	0.044	0.040	0.042	0.040	0.042
Eq. 5.7.2.1-1, n	**0.044**	0.040	0.042	0.040	0.042
Froude Number =	3.9	1.6	1.7	1.8	3.3
f(Fr) =	5.601	1.081	1.133	1.213	5.239
b =	0.122	0.352	0.329	0.310	0.118
f(REG) =	2.449	8.792	7.551	7.144	2.596
f(CG) =	0.633	0.451	0.477	0.482	0.615
Ditch Capacity, Q =	**3.04**	**1.13**	**1.25**	**0.79**	**4.49**
1:100 year Design Flow =	**3.04**	**1.14**	**1.25**	**0.77**	**4.48**
Trial size d50 (m) =	0.575	0.340	0.425	0.340	0.425
Calculated d50 (HEC-15)	7.998	0.317	0.312	0.333	24.957
Calculated d50 (Rock Chutes)	0.473	0.225	0.246	0.213	0.397
Class size of rip rap	FALSE	Class 50kg	Class 50kg	Class 50kg	FALSE
CHECK Froude Number Subcritical < 1 Critical = 1 Supercritical > 1	3.9	1.6	1.7	1.8	3.3
CHECK 1.5<da/d50<185	Method not valid- use e	Method not valid- use equati	Method not valid- use equatic	Method not valid- use equation 6.	Method not valid- use equati
CHECK 0.3<da/d50<1.5	Method not valid Use Eq. 5.7.2.1-1-	equation 6.2 = OKAY	equation 6.2 = OKAY	equation 6.2 = OKAY	equation 6.2 = OKAY
HEC 15 results (Not applicable to steep slope)	Class 250kg	Class 50kg	Class 100kg	Class 50kg	Class 100kg
Robinson, Rice, Kadavy (Rock Chutes)	Class 250kg	Class 25kg	Class 25kg	Class 25kg	Class 100kg
NOTES:					
SELECTED SIZE	**Class 250kg**	**Class 25kg**	**Class 25kg**	**Class 25kg**	**Class 100kg**

Riprap Sizing - Ditch Crossing Outlets (100-Year with CC)

Mannings equation 6.1: n = 0.319*d_a^(1/6) / (2.25+5.23log(d_a/d_{50}))
only valid where 1.5≤d_a/d_{50}≤

Equation 6.2 n = da^(1/6) / √g*f(Fr)*f(REG)*fCG)
where f(Fr)= (0.28*Fr/b)^(log(0.755/b))
f(REG) = 13.434*(T/D50)^0.492*b^(1.025*(T/D50)^0.118)
f(CG) = (T/da)^-b
T= Channel top width (m)
b = effective roughness concentration = 1.14*(D50/T)^0.453*(da/D50)^0.814
only valid where 0.3<d_a/d_{50}<1.5

**MILD < 5%, STEEP > 10%, IN BETWEEN = LARGER OF MILD/STEEP METHOD

Station =	North Crossing	Ditch 1 Crossing	Ditch 2 Crossing	Ditch 3 Crossing	South Crossing 1
Depth of Flow =	0.134	0.125	0.140	0.091	0.142
Average Depth of Flow =	0.129	0.115	0.128	0.086	0.138
Area of Flow =	0.697	0.273	0.309	0.194	1.166
Slope of left bank (1:z) =	**1.500**	**1.500**	**1.500**	**1.500**	**1.500**
Slope of right bank (1:z) =	**1.500**	**1.500**	**1.500**	**1.500**	**1.500**
Wetted Perimeter =	5.483	2.451	2.505	2.328	8.512
Top width of water, T =	5.402	2.375	2.420	2.273	8.426
Bottom width of water =	**5.000**	**2.000**	**2.000**	**2.000**	**8.000**
Hydraulic Radius, R_h =	0.127	0.112	0.124	0.084	0.137
Avg. Channel Velocity, V =	4.355	4.159	4.045	3.983	3.847
Channel Slope (max), S =	**58%**	**63%**	**52%**	**77%**	**37%**
Mannings Coefficient, n =eqn 6.1	-0.198	-0.159	-0.194	-0.152	-0.769
Mannings Coefficient, n =eqn 6.2	0.044	**0.044**	0.044	0.042	0.042
Eq. 5.7.2.1-1, n	**0.044**	0.044	0.044	0.042	0.042
Froude Number =	3.9	3.9	3.6	4.3	3.3
f(Fr) =	5.601	3.591	3.050	4.613	5.239
b =	0.122	0.162	0.175	0.145	0.118
f(REG) =	2.449	2.973	3.277	2.738	2.596
f(CG) =	0.633	0.613	0.598	0.622	0.615
Ditch Capacity, Q =	**3.04**	**1.14**	**1.25**	**0.77**	**4.49**
1:100 year Design Flow =	**3.04**	**1.14**	**1.25**	**0.77**	**4.48**
Trial size d50 (m) =	0.575	0.575	0.575	0.425	0.425
Calculated d50 (HEC-15)	7.998	7.432	10.136	4.539	24.957
Calculated d50 (Rock Chutes)	0.473	0.456	0.450	0.400	0.397
Class size of rip rap	FALSE	FALSE	FALSE	FALSE	FALSE
CHECK Froude Number Subcritical < 1 Critical = 1 Supercritical > 1	3.9	3.9	3.6	4.3	3.3
CHECK 1.5<da/d50<185	Method not valid- use e	Method not valid- use equati	Method not valid- use equatio	Method not valid- use equation 6.	Method not valid- use equation 6.2
CHECK 0.3<da/d50<1.5	Method not valid	Method not valid	Method not valid	Method not valid	equation 6.2 = OKAY
	Use Eq. 5.7.2.1-1-	Use Eq. 5.7.2.1-1-	Use Eq. 5.7.2.1-1-	Use Eq. 5.7.2.1-1-	
HEC 15 results (Not applicable to steep slope)	Class 250kg	Class 250kg	Class 250kg	Class 100kg	Class 100kg
Robinson, Rice, Kadavy (Rock Chutes)	Class 250kg	Class 250kg	Class 250kg	Class 100kg	Class 100kg
NOTES:					
SELECTED SIZE	**Class 250kg**	**Class 250kg**	**Class 250kg**	**Class 100kg**	**Class 100kg**