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DRAINAGE REPORT

Ministry of Transportation and Infrastructure

Highway No. 16 – Guest Road and Shallow Bay Road Intersection Improvements
Issue for Tender

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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 Guest Road and Shallow Bay Road Intersection Improvements along Highway 16. The intersection improvements at both locations are intended to improve user safety by including left turn bays, right turn deceleration lanes, and improving the size of the intersection to account for large vehicle turning movements. The project is located north of Cluculz Lake, approximately 33km east of Vanderhoof, B.C. as shown in **Figure 1-1**.

Figure 1-1: Project Location - Source: Google Maps

As part of the project, existing drainage infrastructure within the project footprint will be replaced and new infrastructure added to meet current flood design standards that account for the impacts of climate change. Proposed drainage infrastructure improvements include the installation of new culvert crossings to convey flows across the highway and smaller roads, excavation of roadside ditching to capture and divert upstream runoff, and placement of concrete spillway barriers on the road shoulder for pavement drainage.

This report summarizes the drainage assessment carried out for the project including hydrologic assessment of upstream watersheds to estimate design flood flows and hydraulic analyses to size culvert crossings, ditches, and riprap erosion protection, as well as design details and considerations that were accounted for based on site-specific limitations and restrictions. A similar hydrologic assessment was carried out by DWB Consulting Services Ltd. (DWB) in 2015 to support their culvert design; the results of the DWB assessment were compared with those determined as part of this assessment. The drainage design conforms to the BC Supplement to TAC Geometric Design Guide – Section 1000 (British Columbia Ministry of Transportation, 2019) and Technical Circular T-04/19 Climate Change and Extreme Weather Event Preparedness and Resilience in Engineering Infrastructure Design.

2 SITE DESCRIPTION

2.1 Watershed Characteristics

Highway 16 (Yellowhead Highway) runs parallel to Cluculz Lake through rolling to moderately flat topography. The overall watershed is comprised of rural and forested terrain that eventually discharge into Cluculz Lake through a system of culverts and ditches.

Existing watershed sub-catchments within the project footprint were delineated using ground survey data collected and processed by Binnie in August 2020, LiDAR data collected in 2010 as provided by MoTI, and DEM data from Natural Resources Canada (NRCan). Based on the assessment, the watersheds for the project are divided into nine sub-catchments, labelled A1 through C2, and encompasses a total area of 1132.8 ha. Sub-catchment areas are defined based on the locations of existing culverts (Section 2.2) and presented in Table 2-1. The watersheds have slopes ranging from nearly 0% to 4%, and elevations ranging from approximately 940m at the upstream point to 770m at the Highway 16 crossing. Catchment maps for the project site are presented in **Figure 1** and **Figure 2** of Appendix A.

Table 2-1: Existing Watershed Sub-Catchment Areas

Sub-Catchments ID	Total Area (ha)
A1	1124.5
A2	1.2
A3	0.5
A4	4.3
A5	0.7
A6	0.6
B	0.4
C1	0.3
C2	0.3

Note:

1- Catchment areas are presented in plan in Figure 1 and Figure 2 of Appendix A.

It is noted that the alignments of the proposed culvert improvements are unchanged from those of existing culverts. Therefore, following project implementation, the watershed sub-catchments and flow will also unchanged from that of the existing network.

2.1.1 Sub-catchment A1

Sub-catchment A1 has an area of 1124.5 ha and is comprised of relatively rolling to moderately sloped, mixed rural and sparsely forested terrain. Sub-catchments A2, A3, A4 and A5 feed into sub-catchment A1 through the culvert crossing at Sta. 107+48.

Sub-catchment A1 flows through sparsely forested terrain and thereafter through flat rural land. The primary drainage paths (unnamed creeks) in sub-catchment A1 flow in a southwest direction prior to crossing Highway 16 in a southerly direction towards Cluculz Lake. Two of the paths on the north side of the watershed converge approximate 350 m north of the upstream pond at the highway crossing at Sta. 104+91, while the remaining path drains directly to this pond. Survey indicates that the unnamed creeks is a tributary to Cluculz Lake and is conveyed across Highway 16 and the Frontage Road by an existing steel culvert installed in 2015.

2.1.2 Sub-catchment A2

Sub-catchment A2 is approximately 1.2 ha in size and drains via a moderately-sloped highway ditch, flowing in an east to west direction. It is bounded by Highway 16 to the north, Frontage Road to the south, Tapping Road to the east, and Shallow Bay Road to the west. Runoff from Highway 16 or the Frontage Road flows into the roadside ditch network and is then conveyed across Shallow Bay Road into sub-catchment A5 through an existing 450mm CSP culvert at Sta. 700+17.

2.1.3 Sub-catchment A3

Sub-catchment A3 is approximately 0.5 ha in size and drains via a rolling to moderately-sloped road ditch, flowing in an east to west direction. It is bounded by the Frontage Road to the south and Shallow Bay Road to the west. Runoff is conveyed across Shallow Bay Road into sub-catchment A4 through an existing 300mm CSP culvert at Sta. 700+33.

2.1.4 Sub-catchment A4

Sub-catchment A4 has an area of 4.3 ha and is comprised of predominately rolling to moderately-sloped sparsely forested terrain. Runoff from the sub-catchment flows in a northernly direction into a roadside ditch system that discharges into sub-catchment A5 via an existing 450mm CSP culvert at Sta. 801+74.

2.1.5 Sub-catchment A5

Sub-catchment A5 has an area of 0.7 ha and is in a section of the roadside ditch system between Highway 16 and Frontage Road that collects stormwater from sub-catchments A2, A3, and A4. The total combined catchment area, including contribution from Sub-catchments A2 to A4, is 6.7 ha. Flows are conveyed across Highway 16 in a northernly direction into sub-catchment A1 through an existing 700mm CSP culvert at Sta. 107+48.

2.1.6 Sub-catchment A6

Sub-catchment A6 has an area of 0.6 ha and is located in the northwest quadrant of Highway 16 and Guest Road. Runoff along the relatively flat roadside ditch flows in a west to east direction across Guest Road into sub-catchment A1 through an existing 450mm CSP culvert at Sta. 300+30.

2.1.7 Sub-catchment B

Sub-catchment B has an area of 0.4 ha and is located along a section of the roadside ditch between Highway 16 and Frontage Road. Runoff flows in an east to west direction and discharges in a southerly direction across Frontage Road through an existing 450mm CSP culvert at Sta. 601+50.

2.1.8 Sub-catchment C1

Sub-catchment C1 has an area of 0.3 ha and is located at the southwest quadrant of Highway 16 and Guest Road. Runoff from the roadside ditch flows across Guest Road into sub-catchment C2 through an existing 300mm CSP culvert at Sta. 400+23.

2.1.9 Sub-catchment C2

Sub-catchment C2 has an area of 0.3 ha, collecting and discharging runoff from sub-catchment C1 into the creek system south of Frontage Road through an existing 450mm steel culvert at Sta. 600+95.

2.2 Culvert Conditions

Table 2-2 summarises the existing locations, sizes, conditions, and total contributary drainage area for each existing culvert within the project scope based on information collected during field survey by Binnie in 2020. There are currently nine (9) existing culvert crossings, two (2) of which cross Highway 16. Culvert materials identified are Smooth Wall Steel Pipes (STEEL) and Corrugated Steep Pipes (CSP). As indicated in Table 2-2, the majority of runoff within the project watershed is conveyed through the 2400 mm STEEL culvert crossing Highway 16 (herein referred to as the *Main Culvert Crossing*).

Table 2-2: Existing Culvert Crossing Details

Station (m)	Existing Culvert Diameter(1)	Existing Condition	Sub-Catchment ID	Contributing Area (ha)
104+91	2400mm STEEL	Good	A (unnamed creek)	1131.2
700+17	450mm CSP	Good	A2	1.2
700+33	300mm CSP	Poor	A3	0.5
801+74	450mm CSP	Good	A3+A4	4.8
107+48	700mm CSP	Plugged	A2+A3+A4+A5	6.7
300+30	450mm CSP	Good	A6	0.6
601+50	450mm CSP	Good	B	0.4
400+23	300mm CSP	Plugged	C1	0.4
600+95	450mm STEEL	Good	C1+C2	0.6

Binnie carried out site visits to assess existing culvert conditions in March 2021 and August 2022. During these visits, the Main Culvert Crossing at Sta. 104+91 appeared to be in good condition; however, it was observed that the bottom half of the culvert inlet was tapered at an approximately 60% angle, as shown in **Figure 2-1**. Reasons for this modification are unknown. Based on the post-construction report, the culvert is embedded to 20% of its diameter (i.e., 480 mm) and includes welded baffles for fish passage (DWB Consulting Services Ltd., 2015). By August 2022, embedment materials placed during construction had been significantly eroded and were noticeably below the crest of the baffles **Figure** 2-4). A considerable debris blockage had also formed at the culvert inlet by the time of this visit. It is uncertain whether the debris had been transported there by creek flows or by human intervention (see **Figure 2-3**) as an existing debris barrier, located approximately 5 m upstream of the inlet, did not appear to retain any debris. The thickness of the steel pipe walls was measured at approximately 21 mm.

Photos of the Main Culvert Crossing and other culvert crossings from the site visits are shown below.

Figure 2-1: Existing 2400mm Steel Culvert Inlet in March 2021 (Main Culvert Crossing, Sta. 104+91)

Figure 2-2: Pond upstream of the Main Culvert Crossing in March 2021 (Sta. 104+91)

Figure 2-3: Main Culvert Crossing debris blockage at inlet in August 2022

Figure 2-4: Main Culvert Crossing facing upstream from the outlet; note the lack of embedment (August 2022)

Figure 2-5: Existing 450mm Culvert Outlet at Sta. 601+42

Figure 2-6: Existing 450mm Culvert Inlet at Sta. 601+42

Figure 2-7: Existing 450mm Culvert Inlet at Sta. 700+10 (Shallow Bay Road)

Figure 2-8: Shallow Bay Culvert Inlet at Sta.700+10

Figure 2-9: Existing 750mm CSP Culvert Inlet at Sta. 107+48

Figure 2-10: Existing 450mm Culvert Outlet at Sta. 600+95

Figure 2-11: Existing 450mm Culvert Inlet at Sta. 600+95

2.3 Existing Culvert Assessments

2.3.1 Aquatic Assessment of the Main Culvert Crossing at Sta. 104+91 (2006)

DWB Forestry Services Ltd. completed an Aquatic Resource Assessment in 2006 to identify and classify streams and aquatic habitat adjacent to Highway 16, from Guest Road to Hillcrest Way, for consideration in future highway improvement projects (DWB Forestry Services Ltd., 2006). At Guest Road, three juvenile trout were captured, and several others were observed in the stagnant pool upstream of the crossing. The channel upstream of the pool was poorly define and runs through a wetland area. First inspection showed that the culvert was a 1000mm diameter Corrugated Metal Pipe (CMP) at the inlet and 1200mm diameter CMP at the outlet. Further inspection determined that the 1000mm diameter CMP was inserted into the 1200mm diameter CMP. During periods of high flow, water would flow through the gap between the two culverts and erode the substrate in-between. Although it was determined that the 1000mm CMP was not a barrier to fish passage, it was in very poor condition with rusting at the bottom. Note that in subsequent 2011 and 2015 documents (*Hydrotechnical Summary, Guest Road Crossing Site Plan Drawings* and *Post Construction Report*), the 1000mm CMP is noted as an 800mm culvert. The former 1000mm and 1200mm CMP culverts outlined in the DWB report were replaced with a new steel culvert in 2011 as discussed below.

2.3.2 Hydrotechnical Assessment for the existing culvert at Sta. 104+91 (2011)

In 2011, Yellowhead Road & Bridge (YRB) identified the Main Culvert Crossing as a maintenance concern and approached DWB to perform a hydrotechnical summary of the culvert (DWB Consulting Services Ltd., 2011). The hydrotechnical summary determined a watershed drainage area of 10.5 km^2 and a 1-in-

200 year flood flows of 5.43 m^3/s based on regional hydrologic analysis. The Rational Method was used to predict the 1 in 2-year flow for fish passage design. No mention of climate change was made in this assessment. A comparison between the design flows from the DWB report and findings from this assessment is summarized later in this report in **Section 5.1**.

YRB presented three options for the proposed culvert replacement:

1. A 2400 mm embedded culvert and an 1800 mm diameter overflow culvert;
2. Twin 2000 mm embedded culverts; and
3. A 3400 mm embedded culvert.

Options were proposed based on hydraulics, fish passage, as well as construction feasibility due to the high fill height above the culvert. Based on the 2015 culvert post-construction report (DWB Consulting Services Ltd., 2015), it was documented that YRB installed a new 2200 mm embedded steel pipe, infilled the old culvert with concrete slurry, and re-armored the embankment and stream channel. Binnie notes that the installed culvert size is not in line with the sizing options presented in the 2011 report, nor does it match the topographical survey conducted by Binnie in August 2020 which show that the existing culvert is a 2400mm diameter steel pipe. Therefore, in the subsequent sections of this report, the culvert is described as a 2400mm diameter CSP culvert. The actual culvert size should be confirmed by the contractor prior to construction.

2.4 Geotechnical Assessment

Ecora Engineering & Resource Group Ltd. (Ecora) was retained by the Ministry to carry out a geotechnical investigation of the site. Ecora advanced a total 18 test holes (comprised of drill holes, test pits, and hand auger holes) along the existing highway section to characterise soil conditions and completed various geotechnical assessments including seismic analysis, slope stability analysis, and pavement design in support of the highway design. Ecora also provided recommendations for construction (e.g., materials, methodology). Refer to the Geotechnical Assessment Report (Ecora, 2022) completed in June 2022 for more information.

2.5 Design Criteria References

The following references have been used to develop design criteria for the Project:

- BC MOTI Supplement to TAC Geometric Design Guide – Section 1000 (2019)
- Technical Circular T-04/19: Resilient Infrastructure Engineering Design – Adaptation to the Impacts of Climate Change and Weather Extremes (2019)
- RTAC Drainage Manual – Volume 1 (1982)
- Corrugated Steel Pipe Institute (CSPI) Handbook of Steel Drainage and Highway Construction Products (2007)
- U.S. Department of Transportation – FHWA HEC 14 – Hydraulic Design of Energy Dissipators for Culverts and Channels (2006).
- California Department of Transportation - Highway Design Manual – 6th Edition. (2017)

2.6 Design Criteria

From the Section 1000 of the Supplement to TAC Geometric Design Guide (British Columbia Ministry of Transportation, 2019):

Culverts

- Culverts are to be designed with a 75-year design life.
- Culverts < 3m span on a natural watercourse (i.e., Main Culvert Crossing) are designed to accommodate a 200-year return period peak instantaneous flow with climate change allowance. [1]
- Culverts to meet 1 in 200-year return period for maximum daily flow (MDF) without the water level at the inlet exceeding the top of the culvert (Water Sustainability Act)[1]
- Culverts < 3m span for ditch drainage networks are designed to accommodate a 100-year return period peak instantaneous flow with a climate change allowance.
- Culverts under inlet control were designed based on a maximum headwater to diameter ratio (HW/D) $\leq$ 1.
- Culverts under outlet control were designed with a maximum 0.3m head loss through the culvert[1]
- Design flows were calculated using the Rational Method. Refer to **Section 4.2** of this report for hydrologic parameters used for the calculations.
- Minimum 400 mm diameter culvert for driveways.
- Minimum 500 mm diameter culvert for frontage roads.

[1] The existing Main Culvert Crossing at Sta. 104+91.120 does not meet the above design criteria. The existing culvert (2400mm embedded culvert) does not align with the proposed sizing recommendations listed in the YRB report. The existing culvert is considered to be undersized and cannot accommodate the 200-year storm with a HW/D < 1.0. Refer to **Section 6.1.1** for further information.

- Minimum 900 mm diameter culvert for under a highway.

Highway Ditches

- Ditches to be designed to convey the 25-year return period design flow
- Ditches shall have a minimum of -0.5% sustained grade, with -0.3% as an absolute minimum
- Ditch invert to be minimum 0.3m below the SGSB layer
- Maximum allowable depth of flow in minor ditches is 0.6 m
- Minimum bottom width of 1.0 m
- Minimum recommended freeboard of 0.3m for highway ditches

Pavement Drainage

- Pavement drainage infrastructure to have a 20-year design life
- Maximum ponding widths equal to 65% of the paved shoulder width, with a minimum of 1.2 m
- Double catch basins or spillways required at low points
- Minimum 20 m spacing, maximum 150 m spacing
- Paved or riprap outfalls to be installed on short drops to ditch bottom
- Piped or riprap outfalls to be installed on high fills with riprap preferred for maintenance
- Piped outfalls are to be a minimum 200 mm diameter buried HDPE pipe with fused joints

3 CLIMATE

3.1 Historical Precipitation Data

Historical precipitation data were obtained from Environment Canada's Climate Normals. The closest weather station with a lengthy period of climate record is the Prince George Airport Climate Station (Climate ID: 1096450), located approximately 56km east of the project site at an elevation of 691 m. Climate Normals for the Prince George Airport Climate Station are presented in Figure 3-1.

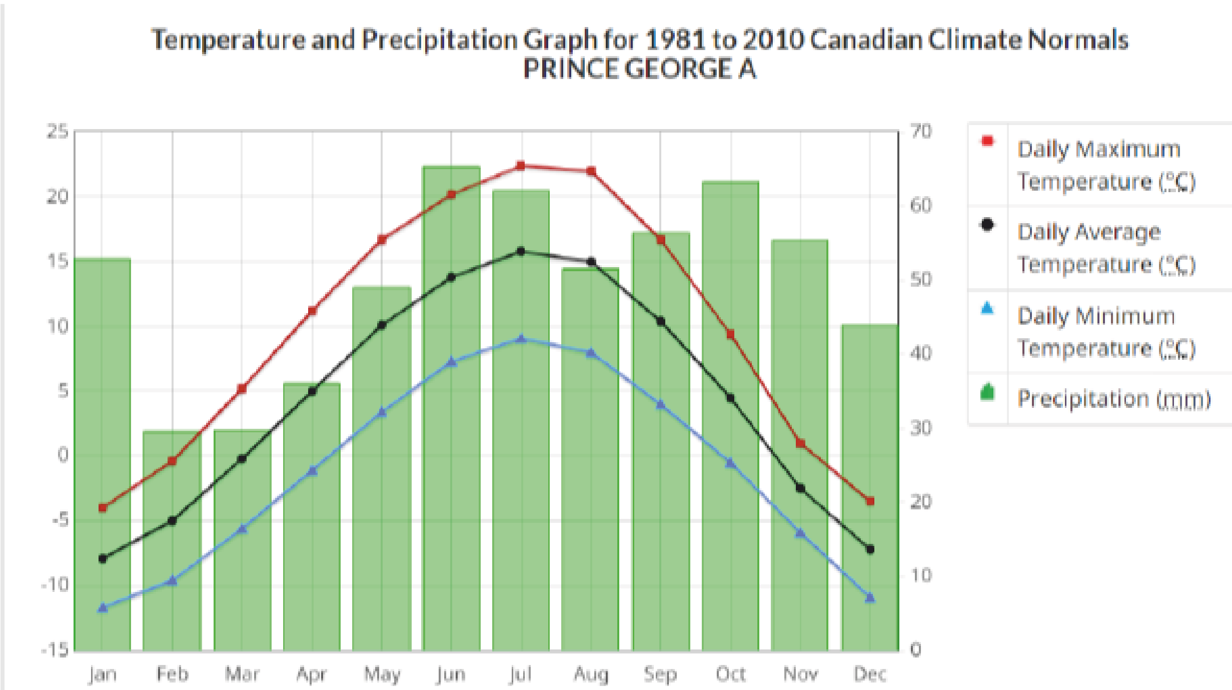

Figure 3-1: Prince George Airport Climate Station Climate Normals 1980 to 2010 (Government of Canada, n.d.)

The average annual precipitation at Prince George Airport Climate Station between 1981 and 2010 was 594.9 mm. The average monthly precipitation ranges from 65.3 mm of rain in June to 6.7 mm of rain with 28.1 cm of snow in February. Maximum rainfall events typically occur in the summer months of June to August, with heavy snowfall common from November to March. It is noted that this climate station has since been relocated elsewhere at the airport and is now identified as the Prince George Airport Auto Climate Station (Climate ID: 1096453) with an elevation of 680 m.

For the selection of the design rainfall storm event at the project site, we compared the IDF curve for the ungauged location (Lat: 53.89177°, Lon: -123.53510°) extracted from IDF-CC Tool v6.0, to those of nearby gauged weather stations at Prince George Airport Auto (ID: 1096453), and Fort St James (ID: 1092970). 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. While stations located farther from the project site such as Southbank (ID: 1087600), Burns Lake (ID: 1091169), Kersley (ID: 1094125), were not directly assessed for selection of the design IDF curve, they were reviewed to determine their influence on the IDF curve of the ungauged station.

Both the Prince George Airport Climate Station and Fort St. James Climate Station are situated at an elevation of 691 m, similar to that of the project site (road elevation at the project site is approximately 775 m); however, the former has 49 years of data and is closer in proximity to the project site than the Fort St. James Climate Station which only has 30 years of data and is a distant 80 km away from the project site. The IDF curve derived for the ungauged station was also considered but a review of data from the Southbank Climate Station (ID: 1087600), which identified significantly higher rainfall intensities than at other nearby stations, suggested that it would have a disproportionate skew on the ungauged IDF curve. Based on this assessment, data from the Prince George Airport Climate Station was considered most representative of climate at the project site and therefore adopted for design.

3.2 Climate Change Projections

The technical circular, T04-19 (British Columbia Ministry of Transportation, 2019) states that all MoTI projections must consider climate change when taking weather projections into account. We conducted a climate change assessment to determine the applicable effect on the project by assessing two climate change assessment tools to evaluate the projected climate change.

Plan2Adapt Tool

The Plan2Adapt tool developed by the Pacific Climate Impacts Consortium (PCIC) provides maps and data for projected climate change conditions for locations throughout British Columbia. We used this tool to assess the projected climate change conditions at the project site (Bulkley-Nechako region). A summary of the projections is shown in **Table 3-1.** The projected climate change conditions for the Fraser-Fort George region were also examined as the project locations exists within the Bulkley-Nechako region, but is just outside the Fraser-Fort George region. It was determined that the results from the IDF-CC tool for Prince George Airport Climate Station is better aligned with the projected increases indicated in the Bulkley-Nechako region.

The Plan2Adapt projected climate suggests slightly less rainfall during the summer months, but higher precipitation during the winter months, with more of the winter precipitation falling as rain than the current climate conditions. Based on the PCIC projections, summer precipitation is expected to increase by an average of only 5.9%, but with an upward range of 29% at the 90th percentile.

Table 3-1: PCIC Summary of Climate Change for Bulkley-Nechako in the 2080s

Climate Variable	Season	Projected Change from 1961-1990 Baseline	
		Ensemble Median	Range (10th to 90th percentile)
Mean Temperature (°C)	Annual	+5.3 °C	+3.9 °C to +6.8 °C
Precipitation (%)	Annual	+12%	+9.1 % to +27 %
	Summer	-4.9%	-20% to +29%
	Winter	+13%	+1.1% to +25%
Snowfall (%)	Winter	-27%	-36% to -21%
	Spring	-67%	-78% to -58%

Notes:

1. The table above shows projected changes in average (mean) temperature, precipitation, and several derived climate variables from the baseline historic period (1961-1990) to the **2080s** for the **Bulkley-Nechako** region. 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.

IDF-CC Tool

We also used modified rainfall intensity duration frequency curves simulated for the project area using the IDF-CC Tool version 6.0 (accessed 22 February 2022), an online application developed at Western University in partnership with the Institute for Catastrophic Loss Reduction (ICLR) and the Insurance Bureau of Canada (IBC).

Various Global Climate Models are available in the IDF-CC tool for projecting future climate change. We selected the PCIC – Bias Corrected version 2 model with a projection period of 2022 to 2100. The IDF-CC Tool uses historical observed data combined with data from global climate models to predict future precipitation patterns. The IDF-CC Tool analyzes three Shared Socioeconomic Pathways (SSPs) representing different climate change policies being implemented. We used SSP5-8.5 for the project as it represents a "business-as-usual" scenario.

We used the IDF-CC Tool version 6.0, to obtain the IDF curve for Prince George Airport Auto Climate Station (ID: 1096453), see **Figure 3-2** below.

IDF for: PRINCE GEORGE AIRPORT AUTO ID:1096453

Station Info | IDF historical data | IDF under climate change

Climate Model Selection | SSP1.26 | SSP2.45 | SSP5.85 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

Total precipitation amounts presented in mm and precipitation intensity rates presented in mm/h for different return periods (T) presented in years

○ Total PPT (mm) ◉ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	64.60	94.48	121.16	153.98	165.63	206.29	256.94
10 min	43.85	62.63	78.79	97.89	104.96	127.74	157.01
15 min	33.72	48.22	61.22	77.16	82.88	102.63	127.66
30 min	19.72	28.50	35.88	44.45	47.64	57.77	70.65
1 h	11.82	16.63	20.69	25.34	27.08	32.62	39.58
2 h	7.03	9.41	11.63	14.43	15.45	18.94	23.37
6 h	3.36	4.37	5.27	6.33	6.73	8.02	9.67
12 h	2.09	2.65	3.15	3.74	3.95	4.67	5.57
24 h	1.39	1.74	2.02	2.28	2.37	2.67	3.03

Figure 3-2: IDF data under Climate Change Condition (RCP8.5) for a Prince George Airport Auto (ID:1096453)

Using this ensemble of climate models resulted in an average increase in peak precipitation of 45% for the 100-year design storm at the project location for the year 2100. We note that the anticipated increase in peak precipitation using the PCIC and IDF-CC methods are 29% and 45%, respectively. Since the PCIC tool produces average rainfall increases rather than extreme event predictions, we used the IDF-CC tool projection for the design of drainage infrastructure for the project.

For the 200-year return period flood event, we carried out flood frequency analysis on the annual maximum short-duration (≤ 24 hours) rainfall data to derive interpolation equation coefficients for historic 200-year rainfall intensities. Using this method, we estimated a 37% increase for climate change during the 200-year flood event based on the regression relationship between return period and percentage increase in rainfall intensity due to climate change.

4 DESIGN FLOWS

Peak flows for the project drainage basin are estimated using the Rational Method for the design 100-year and 200-year return period storm events. The Rational Method is recommended for use in the Supplement to TAC Guidelines (MoTI 2019a) for small drainage areas up to 1 km² for urban basins and up to 10 km² for rural basins. Peak flow is estimated based on parameters including the Runoff Coefficient and Time of Concentration (t_c), Rainfall Intensity (Section 3.2), and catchment area. Details on the application of the Rational Method can be found in the Supplement to TAC Guidelines (British Columbia Ministry of Transportation, 2019), RTAC Drainage Manual Volume 1 (RTAC, 1982), and other commonly used hydrologic assessment guidelines and reference manuals.

4.1 Flow Generation Mechanism

Flow regimes can be categorized as either rainfall-driven or snowmelt-driven based on climatic and topographic conditions within the watershed. The project site is located in the interior plateau region of British Columbia where snowmelt-dominated regimes are common (Eaton & Moore 2007). These regimes often exhibit low flows through the winter and peak flows in late spring to early summer. While peak flows in smaller watersheds in the B.C. interior are often generated by summer convective rainstorms, assessment of daily flows on smaller gauged watercourses near the project site indicate that peak annual flows are likely generated by a snowmelt event between May and June (i.e., freshet). This is evident in the timings of annual maximum daily flows at the nearby *Twain Creek Tributary Near Babine Lake* (ID: 08EC014) and *North Beach Creek Above Allin Creek* (ID: 08JB013), as shown in Figure 4-1 and Figure 4-2, which have drainage areas of approximately 10 km² and 9 km², respectively. Therefore, a spring snowmelt event is adopted as the critical peak flow generation event.

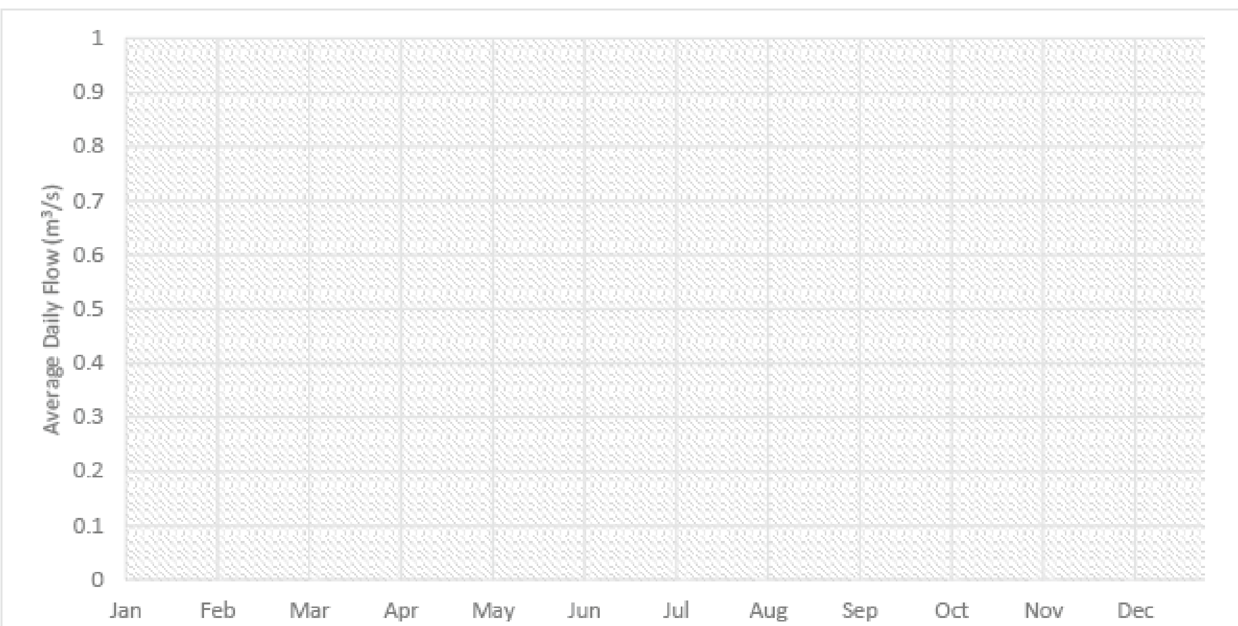

Figure 4-1: Average Daily Flow Hydrographs – Twain Creek Tributary Near Babine Lake Hydrometric Station

Note: Each line represents one year of flow data.

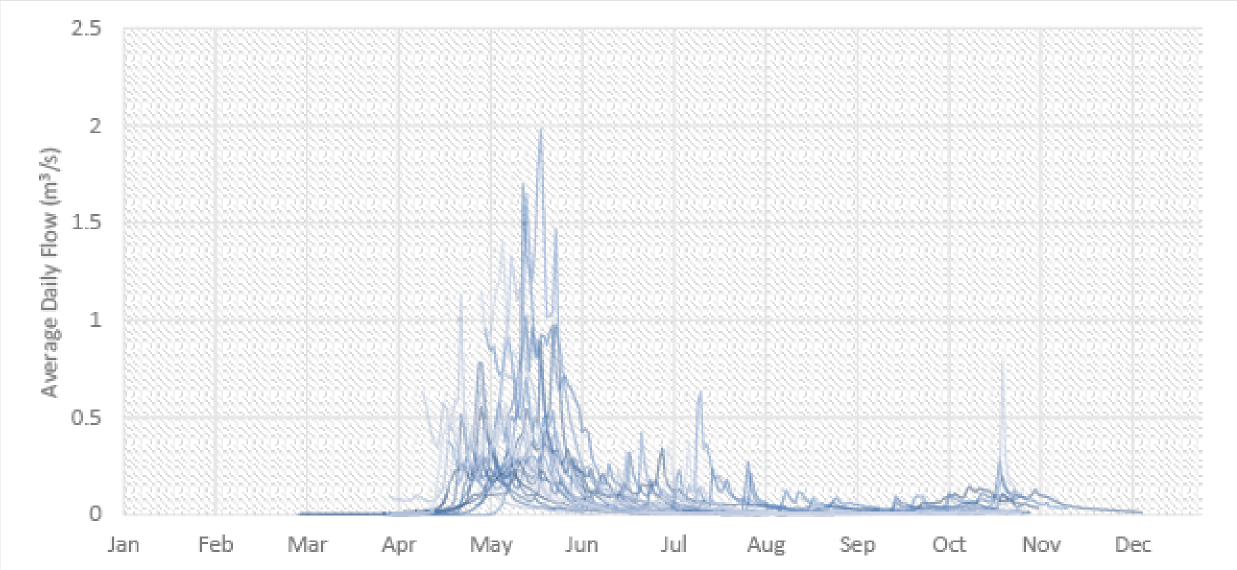

Figure 4-2: Average Daily Flow Hydrographs – North Beach Creek Above Allin Creek Hydrometric Station

Note: Each line represents one year of flow data.

4.2 Design Parameters

The following parameters were used to calculate design flows using the Rational Method:

4.2.1 Runoff Coefficient (C)

For the Main Culvert Crossing, California Department of Transportation (Caltrans) coefficients from Figure 819.2A in the Highway Design Manual (California Department of Transportation, 2017) were used in preference over runoff coefficients from Table 1020.A from Supplement to TAC (British Columbia Ministry of Transportation, 2019). MoTI runoff coefficients are suited for coastal type regions where fall and winter rainfalls are dominant and would therefore be over-conservative for the project location. Caltrans runoff coefficients are comparable to those presented in the RTAC Drainage Manual for interior regions while allowing for direct characterization of surface depressions (e.g., ponds, marshes), several of which are observed in Sub-Catchment A from aerial imagery, that provide peak flow attenuation. Binnie estimated the following runoff coefficient values for each of the four Caltrans categories:

- Relief: 0.10
  - Figure 819.2A in the Highway Design Manual provides a range from 0.08 to 0.14 for relatively flat land with average slopes of 0 to 5%.
- Soil Infiltration: 0.08

  BC Soil Information Finder Tool (BC SIFT) was used to identify the soil in the area as sandy loam. Figure 819.2A in the Highway Design Manual provides a range from 0.06 to 0.08 for normal, well drained light or medium textured soils, sandy loams and silt and silt loams.

- Vegetal Cover: 0.10

  Figure 819.2A in the Highway Design Manual provides a range from 0.08 to 0.12 for poor to fair natural cover. This range was selected based on logging trends off aerial imagery (Google Maps, Bing Maps) overtime.

- Surface Storage: 0.04

  Figure 819.2A in the Highway Design Manual provides a range from 0.04 to 0.06 for high surface storage. From aerial imagery, there is considerable surface depression and several ponds or marshes, including the pond upstream of the Main Culvert Crossing inlet.

Runoff coefficients from all four categories were summed and then increased by a factor of 1.25 to account for the 200-year return period and by 0.1 for potential snowmelt (based on Table 1020.A from Supplement to TAC). The resulting runoff coefficient for design is 0.50.

MoTI runoff coefficients were used for the remaining smaller catchments (A2-A6, B, C1-C2), which are comprised of ditches adjacent to the roadway and nearby rural/forested areas, as they exhibit similar characteristics of surface cover for Coastal Type Basins. Runoff coefficients were area-weighted between impermeable, forested, and rural for flat (<5%) land based on the Supplement to TAC.

- Impermeable (C = 1.0): 0.8 + 0.10 (>25-year return period) + 0.10 (snowmelt)
- Forested (C = 0.55): 0.40 + 0.05 (>25-year return period) + 0.10 (snowmelt)
- Rural (C = 0.75): 0.55 + 0.10 (>25-year return period) + 0.10 (snowmelt)

4.2.2 Time of Concentration

Time of concentration for the Main Culvert Crossing watershed (Sub-catchment A) was calculated using an average value from the Water Management Method indicated in Supplement to TAC guideline (British Columbia Ministry of Transportation, 2019) and the SCS Curve Number method from Drainage Manual (RTAC, 1982). Time of concentration for the smaller catchments was calculating using an average value from the Water Management Method, Kerby-Kirpich method from the Hydraulic Design Manual (Texas Department of Transportation, 2019), and Bransby-Williams method from the Supplement to TAC guideline (British Columbia Ministry of Transportation, 2019). A minimum time of concentration of 10 minutes was used for ditch drainage networks.

4.3 Peak Flows Estimates

Peak design flows estimated for the 100-year return period flood event using the Rational Method are listed in **Table 4-1**.

Table 4-1: 100-Year Return Period Peak Flows

Catchments	Total Area	Slope	Time of Concentration	Q100 – No Climate Change	Q100 - Climate Change
	(ha)	(%)	(hr.)	(m^3/s)	(m^3/s)
A (creek) includes A1 through A6	1131.2	2.1	7.70	9.03	12.29
A2	1.2	3.6	0.30	0.23	0.34
A3	0.5	1.8	0.36	0.08	0.12
A3+A4	4.8	2.0	0.47	0.44	0.59
A2+A3+A4+A5	6.7	2.9	0.47	0.73	0.98
A6	0.6	0.1	0.61	0.06	0.09
B	0.4	1.2	0.34	0.08	0.11
C1	0.4	0.6	0.41	0.06	0.08
C1+C2	0.6	1.7	0.33	0.12	0.17

Peak design flows estimated for the 200-year return period flood event using the Rational Method are listed in **Table 4-2**. The design peak flow adopted for the Main Culvert Crossing, using the site-specific runoff coefficient derived from the Caltrans methodology, was compared with those estimated using the high range and low range runoff coefficients as well as that presented in the 2015 DWB report. The comparison shows that the 200-year peak flow used to design the existing culvert size is significantly lower than that estimated in this hydrologic assessment.

Table 4-2: 200-year Return Period Peak Flow Comparison

Method	Runoff Coefficient	Q200 – No Climate Change	Q200 - Climate Change
		m^3/s	m^3/s
Caltrans (Site-Specific)	0.500	10.41	**14.29**
Caltrans (High Range)	0.625	13.01	17.87
Caltrans (Low Range)	0.425	8.85	12.15
DWB Design	Not Available	5.43	Not Available

Note: **Bolded** value is selected as the 200-year flood peak flow for design of the Main Culvert Crossing.

5 DEBRIS FLOW POTENTIAL

Binnie used the Melton Ratio (Bergerud et al., 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 5-1**, the boundary between floods and debris floods/flows is defined by a Melton Ratio of 0.3.

We estimated the Melton Ratio for Sub-Catchment A using available topographic data (**Table** 5-1). Based on the analysis, Sub-Catchment A is unlikely to be susceptible to debris flows or debris floods.

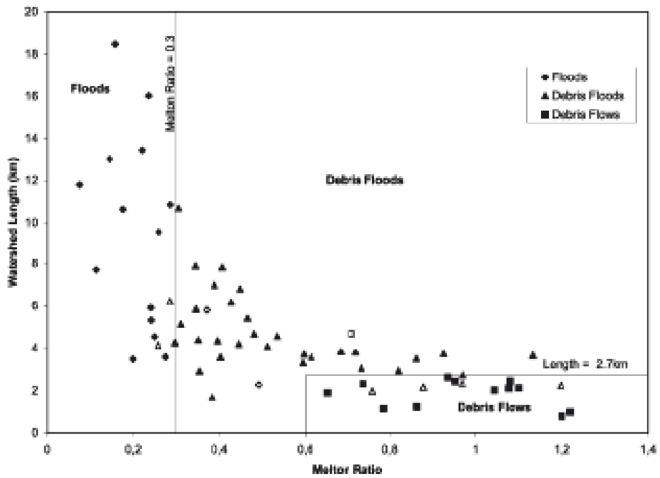

Fig. 5 Scattergram using Melton ratio and watershed length with class limits for the hydrogeomorphic processes. *Symbols without fill colour* are watersheds that do not fall within the appropriate process class limits

Figure 5-1: Proposed Boundaries Between Floods, Debris Floods, and Debris Flows

Table 5-1: Estimated Watershed Melton Ratio

Parameter	Catchment A
Catchment Area (km^2)	11.32
Watershed Relief (km)	0.165
Watershed Length$^{(1)}$ (km)	6.19
Melton Ratio	0.05

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.

6 PROPOSED DRAINAGE INFRASTRUCTURE

6.1 Culverts

There are currently nine (9) existing culverts located within the project limits. Based on discussion presented in **Section 6.1.1**, the Main Culvert Crossing at Sta. 104+91 will not be replaced but will be extended beyond the toe of the design road fill. We propose to replace the other eight (8) culverts: two (2) highway crossings, four (4) local road crossings, and three (3) frontage road crossings to increase hydraulic capacity and meet climate change criteria. To note, two (2) culverts (one highway crossing and one frontage road crossing) are located just outside of the western project limits and have not been considered in the design.

Culverts operating under inlet control were sized to convey the 100-year storm event while maintaining a HW/D $\leq$ 1.0. Culverts were sized using equations for circular culverts presented in Hydraulic Design of Highway Culverts HDS-5 (Federal Highway Administration, 2012).

Culverts operating under outlet control were checked using HY-8 software (version 7.6) to convey the 100-year return period peak storm event with a maximum head loss through the culvert of 0.3 m. For the culvert at Sta. 107+48, the head loss criterion could not be achieved; however, we deemed the proposed culvert size sufficient as the estimated HW/D is less than 1.0 and would not surcharge.

A summary of the proposed culvert infrastructure improvements is listed in **Table 6-1**.

Table 6-1: Proposed Culvert Sizes

Catchment	Culvert Station	Q_{100} Design Flow – Climate Change	Q_{200} Design Flow – Climate Change	Required Culvert Size[1]	HW/D[2]	Headloss[3]
		(m^3/s)	(m^3/s)	(mm)		
A (creek) includes A1 through A6	104+91	-	14.29	Retain existing 2400mm ST	1.96	-
A2	700+17	0.34	-	800	0.65	-
A3	700+33	0.12	-	600	0.54	-
A3+A4	801+74	0.59	-	900	-	0.12
A2+A3+A4+A5	107+48	0.98	-	1000	-	0.54[(4)]
A6	300+30	0.09	-	600	-	0.12
B	601+51	0.11	-	600	0.52	-
C1	400+23	0.08	-	600		0.23
C1+C2	600+95	0.17	-	600	0.64	-

Notes:

1. Minimum size of the culvert under main roads 600mm in diameter.
2. HW/D ratio is calculated for culverts under inlet control conditions only.
3. Head loss is calculated for culverts in outlet control conditions only.
4. Head loss for culvert at Sta. 107+48 exceeds the maximum allowable head loss criterion ($\leq$ 0.3 m). This is considered acceptable as the maximum headwater is below the pipe crown and will not surcharge the culvert.

End treatments (e.g., headwall) were included to mitigate flotation risks for culverts with minimal cover (Sta. 801+74) or on steep slopes (Sta. 600+95). Anti-seepage collars were also included in the design of the culvert at Sta. 600+95 to mitigate piping potential due to steep grade based on the Minnesota Supplement to the Engineering Field Handbook (US NRCS, 2011).

6.1.1 Main Culvert Crossing at Sta. 104+91

Design Considerations

During Conceptual Design, the existing embedded 2400 mm diameter culvert was determined to be undersized for the 100-year and 200-year return period peak flows for both historic and climate change conditions without surcharging. The maximum capacity of the existing 2400mm culvert is approximately 6.5 m^3/s. Binnie presented three options to MoTI; all three options considered fish passage criteria:

1. **Retain the existing 2400 mm diameter embedded culvert and allow surcharging at the inlet**. Option 1 would result in additional risk to the highway embankment, and as such would require additional armoring along the embankment to protect against scour.
2. **Install a 2200mm diameter overflow culvert**. Option 2 requires the installation of a large culvert close enough to the existing culvert to serve as an overflow. A high-level cost of this option is $1,000,000 assuming trenchless installation. The maximum HW/D for this overflow culvert would be approximately 0.87.
3. **Replace the existing culvert with a single pipe.** The minimum culvert size required would be a 3300mm embedded culvert with a HW/D < 0.88 to meet MoTI design criteria and fish passage criteria.

Following the conceptual design submission (March 18, 2021), MoTI elected to proceed with Option 1 due to the significant cost implications of installing a secondary culvert or replacing the existing culvert.

Conceptual design drawings (March 18, 2021) for the Guest Road intersection project showed infilling at the inlet to achieve clear zone. This would result in a required extension of the existing culvert by approximately 12 m. Possible alternatives to minimize the extent of infill and minimize environmental impacts have been considered, including:

1. Providing a concrete roadside barrier along the highway encompassing the culvert and wetland. This would reduce the fill slope to 2:1 and reduce the culvert extension to approximately 6m as well as limiting the environmental impact of the large fill.
2. Installing a small lock block retaining wall adjacent to the highway. This may eliminate or reduce the environmental impacts and may remove the need for a culvert extension entirely. For this option high level geotechnical input was requested to assess the slope stability and provide input for the lock bock retaining wall. This option was considered infeasible and does completely negate the environmental impact during the construction process. The cost implications of this options are currently unknown.

Since the 90% detailed design submission, we have proposed a concrete roadside barrier along the highway to reduce the culvert extension length and reduce the environmental impacts.

As widening of the highway would infill over the Main Culvert Crossing inlet, we assessed two options for the culvert inlet:

1. Extending the existing culvert, and armoring the inlet with riprap, or
2. Installing a headwall to retain some of the above embankment and provide protection at the culvert inlet.

Option 2 to retain the new fill was not preferred as the largest headwall would not be able to adequately retain the fill, mitigate the risks to the remaining embankment, or substantially reduce the environmental impact. Therefore, we propose to extend the existing culvert upstream by approximately 7.1 m using a matching 2400 mm diameter steel pipe segment (Option 1). The culvert inlet will be armoured with riprap and plugged with a low-permeability end seal to cut-off any preferential flow paths along the pipe. The proposed culvert extension is designed at 0% grade to maintain the existing culvert invert and minimum pond depths, and to reduce its impact to fish passage. Accordingly, the extended Main Culvert Crossing will function as a broken-back culvert.

Water Sustainability Act Criteria for Fish Passage

The existing culvert capacity was also assessed against the Water Sustainability Act criteria, where the culvert should be "*capable of passing the 1 in 200-year Maximum Daily Flow (MDF) without the water level at the culvert inlet exceeding the top of the culvert*." The ratio between the peak instantaneous flow and MDF is called the "peaking factor." For rainfall floods, which are the typical driver of culverts on small, steep streams, the typical "peaking factor" can be greater than 5. For larger freshet driven rivers, the "peaking factor" can be as low as 1.1 (Transportation Association of Canada, 2001). To calculate an MDF for the Main Culvert Crossing, we used a peaking factor of 1.5, which resulted in a flow of 9.53 m^3/s. Therefore, the existing culvert is also unable to meet the 200-year MDF without surcharging.

6.2 Ditches

The project will include standard and special roadside ditches, sized to accommodate the 25-year return period flows in accordance with the BC Supplement to TAC guidelines. The existing ditches will be relocated to accommodate new road toes, but the overall ditch network and flow patterns remain unchanged. Ditch slopes will typically follow the longitudinal highway slope except where special ditching is required for culvert grading purposes, namely for the flow path between culverts at Sta. 300+29.585 and Sta. 104+91.120, along north side of Highway 16 from approximately Sta. 107+00 to the channel downstream of the culvert at Sta. 107+47, and upstream of the culvert inlet at Sta. 801+74.

6.3 Spillways

A concrete barrier is proposed in a section along the northern side of the Highway 16 (approximately Sta. 102+00 to Sta. 106+00) alignment near Guest Road. Spillways are proposed to drain pavement runoff along this segment, where the pavement width is also greater than 15 m. Additional drainage curbs and spillways are proposed where fill heights exceed 3 m, longitudinal grades are greater than 4%, which occurs at approximately Sta. 111+60 to the eastern limit of construction at Sta. 113+31.

Asphalt curbs are proposed along certain turning alignments to guide runoff towards the low points at the intersections. Typical spacing ranges between 20 m – 30 m as the longitudinal grade is relatively flat along most of the project. Spillways were utilized instead of catch basins to reduce maintenance requirements.

6.4 Scour Protection

6.4.1 Riprap Aprons

To protect against scour, riprap aprons are to be installed at the inlet and outlet of all culverts, except at the outlet of culvert Sta. 104+91 where the existing outlet has not been impacted by the project and no changes will be made.

Where culverts outlet onto steeper slopes or where significant flows are expected, such as on natural watercourses, the hydraulic characteristics of flow was checked based on the approach indicated in HEC 14 (FWHA, 2006). Riprap aprons will be constructed using, at a minimum, Class 25 kg riprap. The riprap apron at the outlet of the Sta. 600+95 culvert was checked based on HEC-14 riprap stability check for steep slopes. The riprap apron at the inlet of the Main Culvert Crossing (Sta. 104+91) does not require a riprap size larger than Class 25kg as it is downstream of a large wetland where velocities are expected to be minimal. However, as the inlet is anticipated to surcharge in a 200-year return period event, the riprap armouring was extended 0.3 m above the surcharged Q_{200} elevation to protect against erosion.

Where roadside ditch slopes exceed 1%, the erodibility potential was checked against flow velocity and depth. Additional scour protection is proposed along the Highway between Sta. 109+80 and 112+30 to minimize risk of erosion (as the longitudinal slope of the section exceeds 1%) following construction before sufficient time has passed to allow vegetation to mature.

Refer to drawings **R3-375-309, R3-375-310 and R3-375-701 to 704** for riprap apron locations and details.

6.4.2 Debris Risk

There is an existing steel debris barrier (**Figure 6-1**) upstream of the inlet of the Main Culvert Crossing across an incised channel within the pond. Section 5 indicated that debris flood and debris flow risks are generally minimal in the upstream watershed. Available environmental studies indicated that the existing debris structure does not prevent fish passage.

Figure 6-1 : Existing Debris Barrier at Main Culvert Crossing Inlet (Sta. 104+91)

We presented three options for consideration:

1. **Remove the existing debris barrier without replacement.** Given that there is a sizeable pond upstream of the culvert inlet, the risk of sediment loading at the culvert is lower than in typical conditions since (1) the peak velocities from the watershed will be reduced, therefore reducing the size of debris that can be transported and, (2) potential debris has a wide area to deposit prior to entering the culvert. There is still debris risk to the culvert during larger storms.
2. **Remove and replace the existing debris barrier with an engineered debris barrier/deflector.** An engineered triangular-shaped debris deflector is proposed to manage debris risk at the culvert inlet. This type of structure will reduce the likelihood of culvert clogging and is configured to prevent blockage near the culvert inlet. However, design input will be required from a geotechnical engineer and/or structural engineer.
3. **Remove and replace the existing debris barrier with a similar debris barrier design.** Alternatively, a less robust structure comprised of steel posts or W-beams can be provided to re-instate the existing structure. The steel posts would be driven into the native ground to a suitable depth for stability rather than anchored to a structurally designed foundation.

Following discussion with MoTI, an alternative option to install a beaver-guard or -cone type structure was proposed and preferred by MoTI as it anchors directly to the culvert end and would eliminate any geotechnical or structural design requirements. The structure can be manufactured to provide an enlarged opening for fish passage. An example of such a product in use is presented in **Figure 6-2**.

Figure 6-2: Beavercone Product Installed at Culvert Inlet to Prevent Debris Clogging (Fleming, 2022)

7 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.

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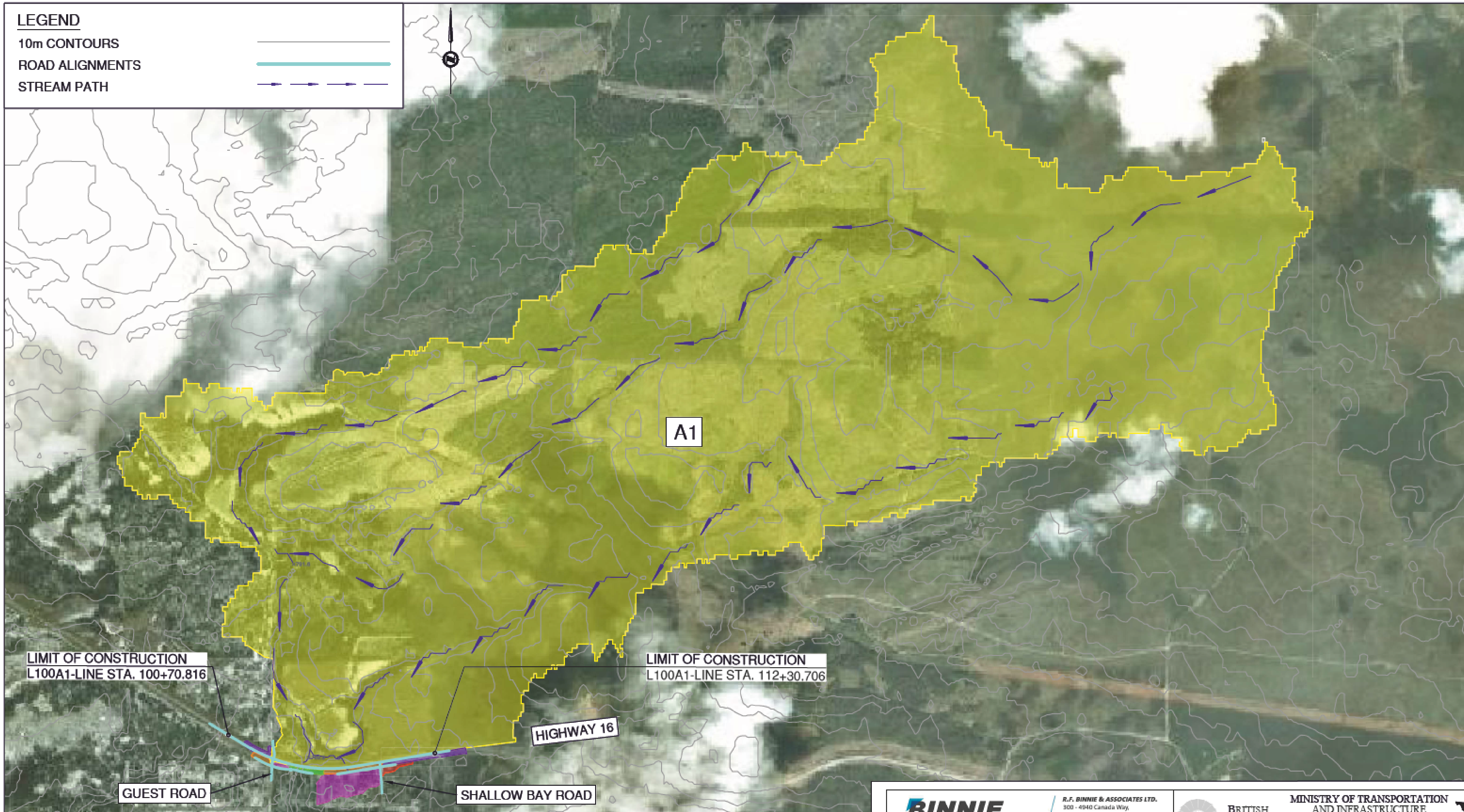
APPENDIX A

WATERSHED MAPS



NOTES:

1. REFER TO FIGURE 2 FOR SUBCATCHMENT AREAS AND EXISTING AND PROPOSED DRAINAGE FEATURES

APPENDIX A - FIGURE 1 - DECEMBER 2, 2022

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NORTHERN REGION
HIGHWAY ENGINEERING AND GEOMATICS

SCALE 0 100 1:10,000 500m

CAD FILE: CATCHMENT MAP 02DEC2022.DWG
DATE 2022-12-02

REV	DATE	REVISIONS	SIGNATURE

DRAINAGE CATCHMENTS

HIGHWAY 16

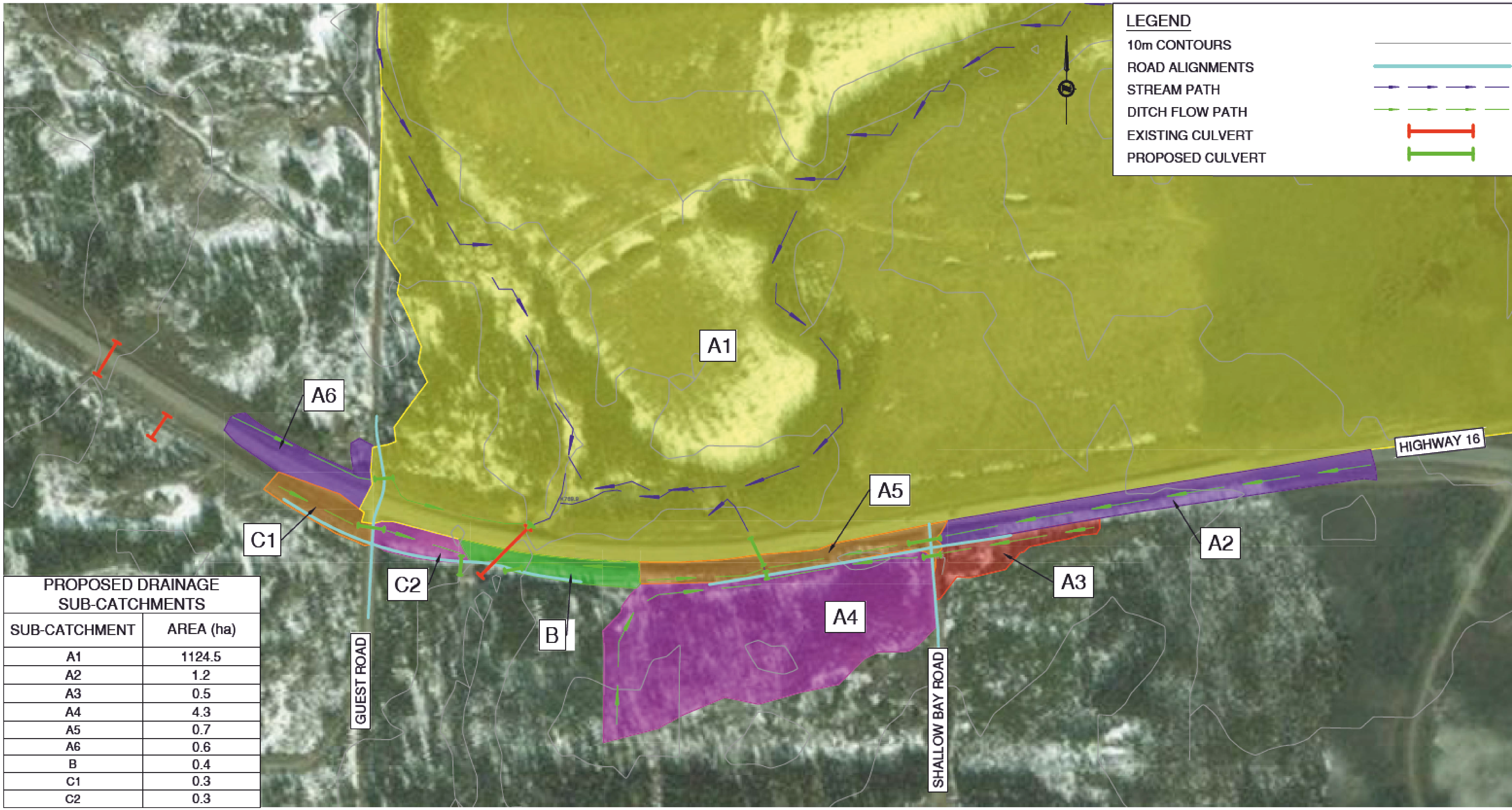
GUEST ROAD AND SHALLOW BAY ROAD INTERSECTIONS

SENIOR DESIGNER

DATE

DESIGNED H.M. DATE FEB. 2022
QUALITY CONTROL J.T. DATE FEB. 2022
QUALITY ASSURANCE J.T. DATE FEB. 2022
DRAWN H.M. DATE FEB. 2022

FILE NUMBER	PROJECT NUMBER	REG	DRAWING NUMBER	REV
21-0125-00	5537851-0000	3	FIGURE 1	



PROPOSED DRAINAGE SUB-CATCHMENTS

SUB-CATCHMENT	AREA (ha)
A1	1124.5
A2	1.2
A3	0.5
A4	4.3
A5	0.7
A6	0.6
B	0.4
C1	0.3
C2	0.3

APPENDIX A - FIGURE 2 - DECEMBER 2, 2022

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NORTHERN REGION
HIGHWAY ENGINEERING AND GEOMATICS

SCALE 0 20 1:2000 100m

CAD FILENAME CATCHMENT MAP 02DEC2022.DWG
DATE 2022-12-02

REV	DATE	REVISIONS	SIGNATURE

DRAINAGE CATCHMENTS
HIGHWAY 16
GUEST ROAD AND SHALLOW BAY ROAD INTERSECTIONS

SENIOR DESIGNER
DATE

DESIGNED H.M. DATE FEB. 2022
QUALITY CONTROL J.T. DATE FEB. 2022
QUALITY ASSURANCE J.T. DATE FEB. 2022
DRAWN H.M. DATE FEB. 2022

FILE NUMBER	PROJECT NUMBER	REG	DRAWING NUMBER	REV
21-0125-00	5537851-0000	3	FIGURE 2	

APPENDIX B

DESIGN FLOW CALCULATIONS

Equations & Charts:

Rainfall Intensity Equation:

$$\text{Rainfall Intensity (mm/hr.)} = A * (t + t_0)^B$$

For the above equation, *A*, t_0, and *B* are coefficients of best fit, and *t* represents the time of concentration in hours. The equation was developed by IDF_CC Tool V5.0 and is a best-fit curve for the GEV distribution for rainfall intensity, modified for climate change.

Bransby-Williams: $t_c = \frac{0.605*L}{A^{0.1}*S^{0.2}}$ [hrs]

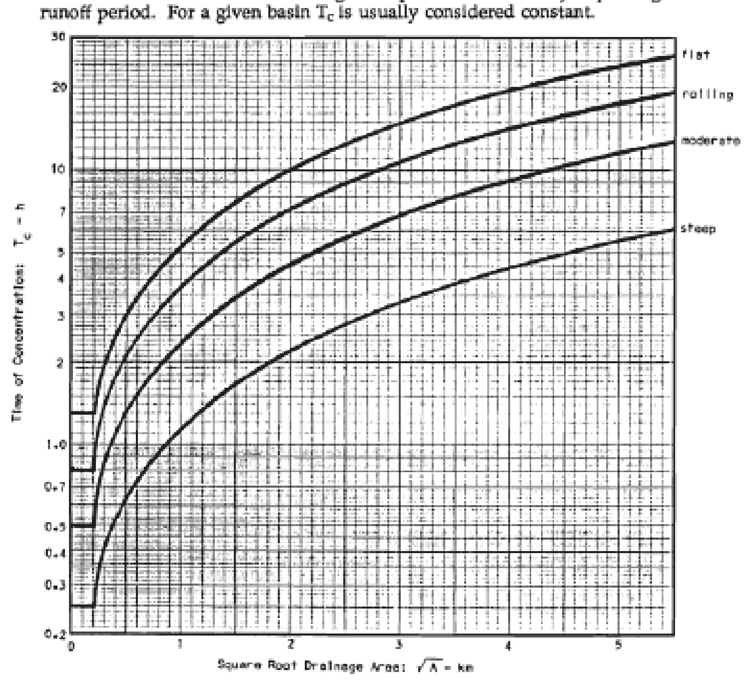
Kerby-Kirpich: $t_c = t_{ov} + t_{ch} = 1.44\frac{(L_{ov}*N)^{0.467}}{S^{0.235}} + 0.0195\frac{(L_{main})^{0.770}}{S^{0.385}}$ [hrs]

SCS Curve Number Method: $t_c = 1.7\left[\frac{L^{0.8}\left(0.039\left(254\left(\frac{100}{CN}-1\right)\right)+1\right)^{0.7}}{735*Y^{0.5}}\right]$ [hrs]

BC Water Management Method:

7.5.3 Time of Concentration

Time of concentration (T_c) is the time required for surface runoff from the most remote part of the drainage basin to reach the point of consideration on the stream. For uniform rainfall intensity, this would be the time of equilibrium when all parts of the basin are contributing to streamflow and additional rainfall will not cause a greater peak flow but will just prolong the runoff period. For a given basin T_c is usually considered constant.



Rational Method: $Q = \frac{ciA}{360}$

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Map Version/Date:

Q200 Design Flow Calculation

(input colour)	= Input
(calculated colour)	= Calculated

RAINFALL DATA: Historic IDF Curve from Prince George Airport Auto (ID: 1096453) and scaled for Climate Change

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

Return Period IDF-CC		A	B	t0
Climate Change	200	46.834	-0.803	0
Historic	200	34.115	-0.803	0

Climate Change

Catchment Area	L (km)	L (km)	A_B	Sqrt(Area)	S (m/m)	Land		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*	*Average*	*Average*	*Average*
A		7.85	1131.2	3.36	2.1%	0.50	63.0	7.40	7.99	7.70	9.10	14.29

No Climate Change

Catchment Area	L (km)	L (km)	A_B	Sqrt(Area)	S (m/m)	Land		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*	*Average*	*Average*	*Average*
A		7.85	1131.2	3.36	2.1%	0.50	63.0	7.40	7.99	7.70	6.63	10.41

Notes: culverts <3m span on a natural watercourse

Map Version/Date: ***Q100 Design Flow Calculation***

	= Input
	= Calculated

RAINFALL DATA: IDF Curve from Prince George Airport Auto (ID: 1096453) under climate change (RCP 8.5) for time frame 2022-2100

Return Period IDF-CC		A	B	t0
Climate Change	100	41.8	-0.820	0.026
Historic	100	30.9	-0.823	0.027

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 Characteristics			Tc (hr)					I (mm/hr)	Qp (m^3/s)
	Overland Flow	*Main Flow Path*	*(ha)*	*(km)*	*Main Channel Ave Slope*	*C*	*CN*	*N*	*BC Method (Figure 1020.B)*	*SCS Curve Number Method*	*Kerby-Kirpich*	*Bransby-Williams*	*Average*	*Average*	*Average*
A		7.85	1131.2	3.36	2.1%	0.50	63.0	0.5	7.40	7.99			7.70	7.82	12.29
A2	0.01	0.46	1.2	0.11	3.6%	1.00		0.1	0.40		0.17	0.33	0.30	104.84	0.34
A3	0.03	0.17	0.5	0.07	1.8%	1.00		0.6	0.63		0.30	0.15	0.36	90.91	0.12
A3+A4	0.03	0.35	4.8	0.22	2.0%	0.59		0.6	0.80		0.36	0.25	0.47	74.37	0.59
A2+A3+A4+A5	0.03	0.65	6.7	0.26	2.9%	0.71		0.5	0.61		0.37	0.41	0.46	74.94	0.98
A6	0.01	0.16	0.6	0.08	0.1%	1.00		0.1	1.30		0.29	0.25	0.61	60.35	0.09
B	0.01	0.14	0.4	0.07	1.2%	1.00		0.1	0.75		0.13	0.14	0.34	95.49	0.11
C1	0.01	0.11	0.4	0.06	0.6%	1.00		0.1	0.96		0.15	0.13	0.41	82.30	0.08
C1+C2	0.01	0.22	0.6	0.08	1.7%	1.00		0.1	0.66		0.14	0.20	0.33	96.93	0.17

No Climate Change

Catchment Area	L (km)	L (km)	A_B	Sqrt(Area)	S (m/m)	Land Characteristics			Tc (hr)					I (mm/hr)	Qp (m3/s)
	Overland Flow	*Main Flow Path*	*(ha)*	*(km)*	*Main Channel Ave Slope*	*C*	*CN*	*N*	*BC Method (Figure 1020.B)*	*SCS Curve Number Method*	*Kerby-Kirpich*	*Bransby-Williams*	*Average*	*Average*	*Average*
A		7.85	1131.2	3.36	2.1%	0.50	63.0	0.5	7.40	7.99			7.70	5.75	9.03
A2	0.01	0.46	1.16	0.11	3.6%	0.90		0.1	0.40		0.17	0.33	0.30	77.57	0.23
A3	0.03	0.17	0.47	0.07	1.8%	0.90		0.6	0.63		0.30	0.15	0.36	67.25	0.08
A3+A4	0.03	0.35	4.79	0.22	2.0%	0.59		0.6	0.80		0.36	0.25	0.47	55.00	0.44
A2+A3+A4+A5	0.03	0.65	6.67	0.26	2.9%	0.71		0.5	0.61		0.37	0.41	0.46	55.42	0.73
A6	0.01	0.16	0.57	0.08	0.1%	0.90		0.1	1.30		0.29	0.25	0.61	44.61	0.06
B	0.01	0.14	0.43	0.07	1.2%	0.90		0.1	0.75		0.13	0.14	0.34	70.65	0.08
C1	0.01	0.11	0.37	0.06	0.6%	0.90		0.1	0.96		0.15	0.13	0.41	60.88	0.06
C1+C2	0.01	0.22	0.65	0.08	1.7%	0.90		0.1	0.66		0.14	0.20	0.33	71.71	0.12

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

Basin/Station	Design Flow m3/s	Existing Culverts (mm)	Proposed Box? (Y/N)	Proposed Embedded? (Y/N)	Proposed Material	Proposed Inlet Configuration	Location	New Culverts Diameter (mm)	New Culverts Height (Box only) (mm)	HW/D @ 2%	General Notes
A (Creek)	14.29	2400mm Steel	N	Y	CSP	Projecting	104+91	2400	2400	1.96	200-year peak (Caltrans avg)
A (Creek)	9.53	2400mm Steel	N	Y	CSP	Projecting	104+91	2400	2400	1.24	200-year MDF
A2	0.34	450mm CSP	N	N	CSP	Projecting	700+17	800	800	0.65	100-year (MoTI) 700mm Hw/D < 1 but upsized to 800mm for standard stock CSP sizing
A3	0.12	300mm CSP	N	N	CSP	Projecting	700+33	600	600	0.54	100-year (MoTI) 500mm Hw/D < 1 but upsized to 600mm due to min size for main roads
A3+A4	0.59	450mm CSP	N	N	CSP	Headwall	801+74	900	900	0.72	100-year (MoTI)
A2+A3+A4+A5	0.98	700mm CSP	N	N	CSP	Projecting	107+48	1000	1000	0.92	100-year (MoTI)
A6	0.09	450mm CSP	N	N	CSP	Projecting	300+30	600	600	0.47	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm due to min size for main roads
B	0.11	450mm CSP	N	N	CSP	Projecting	601+50	600	600	0.52	100-year (MoTI) 500mm Hw/D < 1 but upsized to 600mm to minimize size variation
C1	0.08	300mm CSP	N	N	CSP	Projecting	400+23	600	600	0.44	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm due to min size for main roads
C1+C2	0.17	450mm Steel	N	N	CSP	Headwall	600+95	600	600	0.64	100-year (MoTI) 500mm Hw/D < 1 but upsized to 600mm to match U/S sizing

No Climate Change

Basin/Station	Design Flow m3/s	Existing Culverts (mm)	Box? (Y/N)	Embedded?	Material	Inlet Configuration	Location	New Culverts Diameter (mm)	New Culverts Height (mm)	HW/D @ 2%	General Notes
A (Creek)	10.41	2400mm CSP	N	Y	CSP	Projecting	104+91	2400	2400	1.35	200-year peak (Caltrans, avg)
A (Creek)	6.94	2400mm Steel	N	Y	CSP	Projecting	104+90	2400	2400	0.94	200-year MDF
A2	0.23	450mm CSP	N	N	CSP	Projecting	700+17	600	600	0.76	100-year (MoTI)
A3	0.08	300mm CSP	N	N	CSP	Projecting	700+33	600	600	0.42	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm due to min size for main roads
A3+A4	0.44	450mm CSP	N	N	CSP	Headwall	801+74	800	800	0.73	100-year (MoTI) 700mm Hw/D < 1 but upsized to 800mm for standard stock CSP sizing
A2+A3+A4+A5	0.73	700mm CSP	N	N	CSP	Projecting	107+50	900	900	0.84	100-year (MoTI)
A6	0.06	450mm CSP	N	N	CSP	Projecting	300+30	600	600	0.37	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm due to min size for main roads
B	0.08	450mm CSP	N	N	CSP	Projecting	601+50	600	600	0.41	100-year (MoTI) 400mm Hw/D < 1 but upsized to 500mm due to min size for frontage roads; but upsized to 600mm to minimize size variation
C1	0.06	300mm CSP	N	N	CSP	Projecting	400+23	600	600	0.35	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm due to min size for main roads
C1+C2	0.12	450mm CSP	N	N	CSP	Headwall	600+95	600	600	0.53	100-year (MoTI) 400mm Hw/D < 1 but upsized to 600mm to match U/S sizing

Table 4 - RIPRAP BASIN FOR CULVERT OUTLETS

Manning's Flow	Culvert Location									Units
Update	Sta. 300+29 A6 catchment	Sta. 400+23 C1	Sta. 107+48 A2+A3+A4+A5 Highway Crossing	Sta. 104+91 Main Creek A	Sta. 700+17 A2	Sta.700+33 A3	Sta. 600+95 C1+C2	Sta. 601+50 B	Sta 801+74 A3+A4	
Length =	20.2	29.6	33.4	77.3	37.9	20.5	20.3	21.2	7.7	m
Inlet Elev =	776.27	777.64	774.31	770.23	777.16	777.32	774.06	774.09	774.35	m
Outlet Elev =	776.16	777.01	774.14	768.49	776.21	776.94	771.96	773.45	774.31	m
Depth of Flow =	**0.26**	**0.17**	**0.89**	**3.1**	**0.35**	**0.21**	**0.11**	**0.18**	**0.63**	m
Culvert Type =	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	
Area of Flow =	0.117	0.066	0.738	4.992	0.171	0.088	0.036	0.071	0.476	cu.m
Angle (water) =	1.437	1.123	2.465	2.786	1.738	1.266	0.885	1.159	1.982	radians
Embedded (Y/N) =	N	N	N	Y	N	N	N	N	N	
Wetted Perimeter =	0.862	0.674	2.465	7.636	1.043	0.760	0.531	0.696	1.784	m
Top width of water =	0.595	0.541	0.626	1.114	0.592	0.572	0.464	0.550	0.825	m
Culvert Diameter =	0.6	0.6	1.00	**3.2**	0.6	0.6	0.6	0.6	0.9	m
Hydraulic Radius, R_h =	0.136	0.098	0.299	0.654	0.164	0.116	0.067	0.103	0.267	m
Culvert Discharge, Q =	**0.09**	**0.08**	**0.98**	**20.12**	**0.34**	**0.12**	**0.18**	**0.11**	**0.59**	cu.m/s
1:100 year Design Flow 1:200 year for main creek =	0.09	0.08	0.98	17.87	0.34	0.12	0.17	0.11	0.59	cu.m/s
Culvert Outlet Velocity, V =	0.81	1.28	1.33	4.03	1.97	1.34	5.00	1.57	1.24	m/s
Culvert Slope, S =	0.54%	2.14%	0.51%	2.25%	2.51%	1.85%	10.34%	3.02%	0.52%	m/m
Mannings Coefficient =	0.024	0.024	0.024	0.028	0.024	0.024	0.024	0.024	0.024	
Equivalent Depth =	0.24	0.18	0.61	1.58	0.29	0.21	0.13	0.19	0.49	m
Froude Number =	0.5	1.0	0.5	0.7	1.1	0.9	4.8	1.2	0.5	
Final Outlet Scour Method =	Class 25kg apron	Class 25kg apron	Class 25kg apron	Culvert surcharges at the inlet and is downstream of a pond. High velocities not anticipated. Culvert size upsized to estimate a conservative velocity to determine riprap sizing. Class 25kg apron at culvert inlet	Class 25kg apron	Class 25kg apron	Class 25kg apron	Class 25kg apron	Class 25kg apron	

HEC 14 - Rip Rap Apron

	Tailwater	=	0.26	0.24	0.89	3.10	0.35	0.24	0.24	0.24	0.63	m
(Use if supercritical)	D'	=	0.43	0.39	0.95	3.15	0.48	0.41	0.36	0.39	0.77	m
	D50	=	0.01	0.01	0.05	0.16	0.08	0.02	0.07	0.03	0.04	m
Min 25KG	Class (KG)	=	Class 10kg	Class 10kg	Class 10kg	Class 10kg	Class 10kg	Class 10kg	Class 10kg	Class 10kg	Class 10kg	
	La	=	2.4	2.4	4	12.8	2.4	2.4	2.4	2.4	3.6	m
	RR Depth	=	0.034425185	0.031988049	0.115064515	0.392123083	0.189109377	0.050034069	0.173418309	0.082995083	0.094494091	m

Ditch Capacity using Blodgett and bathurst

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

only valid where $1.5 \le d_a/d_{50} \le 185$

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

where $f(Fr) = (0.28 \cdot Fr/b)^{(\log(0.755/b))}$

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 =	Sta. 600+95 C1+C2
Depth of Flow =	0.260
Average Depth of Flow =	0.130
Area of Flow =	0.203
Slope of left bank (1:z) =	3.000
Slope of right bank (1:z) =	3.000
Wetted Perimeter =	1.644
Top width of water, T =	1.560
Bottom width of water =	0.000
Hydraulic Radius, R_h =	0.123
Avg. Channel Velocity, V =	0.857
Channel Slope (max), S =	10.40%
Mannings Coefficient, n =eqn 6.1	0.385
Mannings Coefficient, n =eqn 6.2	0.093
Eq. 5.7.2.1-1, n	0.039
Froude Number =	0.5
f(Fr) =	0.763
b =	0.284
f(REG) =	6.516
f(CG) =	0.494
Ditch Capacity, Q =	0.17
1:100 year Design Flow =	0.17
Trial size d50 (m) =	0.270
Calculated d50	0.218
Class size of rip rap	Class 25kg
CHECK Froude Number Subcritical < 1 Critical = 1 Supercritical > 1	0.5
CHECK 1.5<da/d50<185	Method not valid- use equation 6.2
CHECK 0.3<da/d50<1.5	equation 6.2 = OKAY
HEC 15 results	Class 25kg
HEC 23 results	Class 25kg
USACE Results	Class 10kg
Robinson, Rice, Kadavy	Class 25kg
Final Scour Method	Class 25kg riprap
NOTES:	Side slopes should not be steeper than 1:3 to avoid instability. Where side slopes steeper than 3:1 are required then HEC eq 6.11 and Eq 6.8 (in conjunction with Eq 6.15) should be assessed.

End Treatment Considerations

Culvert Station	Culvert Diameter	Culvert Material	Wall Thickness (CSP or HDPE)	Cover	Culvert OD	Headwall Requirements				Inlet Treatment					Outlet Treatment				
						Floatation	Fill Heights	Velocities/Steep Slopes/Erosion	Major watercourse	Special consideration	Fence required (based on TAC - 660)	*Fence required if culvert >2.0m diameter*	*Traversable grates as per TAC 7.4.2.3 Or security grates or debris racks*	Selected treatment	Special consideration	Fence required (based on TAC - 660)	*Fence required if culvert >2.0m diameter*	*Traversable grates as per TAC 7.4.2.3 Or security grates or debris racks*	Selected treatment
104+90	2400	CSP	3.5	5.2	2407	NO	Y	Y	Y	5.2m fill height, main watercourse channel flow path	Y	Y	N	-There is an existing debris rack (refer to photos) at the inlet to prevent debris entering the culvert and it doesn't prevent fish passage, beavercone is proposed based on discussions with the ministry - Max height of concrete headwall is unable to contain the embankment, riprap armouring is used instead of a concrete headwall and apron. - Flotation calculations assumes pipe is CSP with lower WT, which is conservative calculation. actual pipe is STEEL which is provides higher resistance to flotation	6m fill height, main channel flow path	N	Y	N	The downstream end of the large culvert is in forested area, fence not required. No work to be done near outlet
107+48	1000	CSP	2	1.4	1004	NO	Y	N	N	Height of fill is 1.5m but the slopes are 4:1 so the chance of slopes sloughing is minor; projecting is feasible	N	N	N	Projecting, inlet just outside of clearzone 9m from through travel lane		N	N	N	Projecting, outlet just out of clearzone
300+30	600	CSP	2	1.3	604	NO	N	N	N		N	N	N	Projecting		N	N	N	Projecting
400+25	600	CSP	2	1.3	604	NO	N	N	N		N	N	N	Projecting		N	N	N	Projecting
600+95	600	CSP	2	1.0	604	NO	N	Y	N	10.4% slope	N	N	N	Add headwall at inlet with antiseepage collar	Outlet in forested area	N	N	N	Projecting
601+50	600	CSP	2	0.7	604	NO	N	N	N		N	N	N	Projecting		N	N	N	Projecting
801+70	800	CSP	2	0.5	804	YES	N	N	N		N	N	N	Add headwall at inlet to resist flotation		N	N	N	Add headwall at outlet to resist flotation
700+15	700	CSP	2	1.0	704	NO	N	N	N		N	N	N	Projecting		N	N	N	Projecting
700+30	600	CSP	2	1.2	604	NO	N	N	N		N	N	N	Projecting		N	N	N	Projecting