REVISION #6

100% Detailed Design Hydrotechnical Design Report

Tappen and White Creek Crossings

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1. INTRODUCTION

McElhanney has been retained by the Ministry of Transportation and Infrastructure (MoTI) to provide structural and hydrotechnical engineering services for major structures in the Highway #1 widening project from Ford Road to Tappen Valley Road, located between Sorrento and Salmon Arm, BC.

This report provides the hydrotechnical assessment and design for the 100% detailed design phase of the project. The report focuses on the two major watercourse crossings of Tappen Creek and White Creek, and the minor drainage infrastructure.

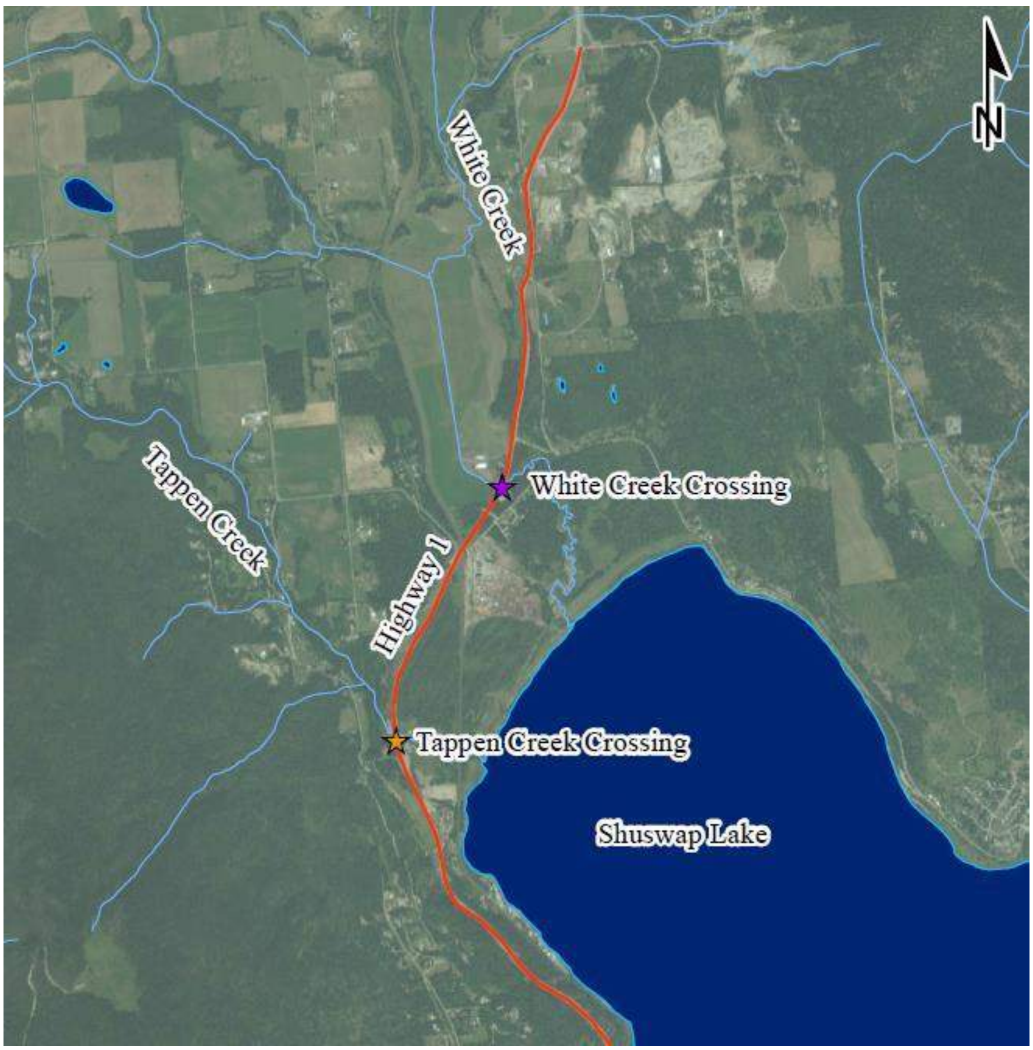


Figure 1-1: Project Area Site Map

2. BACKGROUND

2.1. BACKGROUND INFORMATION

2.1.1. LiDAR & Mapping Data

LiDAR information, provided by MOTI, was used to confirm existing overland drainage patterns in the project area. Provincial 1:20k digital mapping, available through the Freshwater Atlas (FWA), combined with provincial digital elevation models (DEMs), formed the GIS base that was used to determine the catchment areas and physical characteristics of the watersheds. Provincial 1:250k biogeoclimatic ecosystem (BEC) mapping was also employed to compare Water Survey of Canada (WSC) watershed characteristics with those of Tappen Creek and White Creek.

2.1.2. Topographic Survey

Topographic survey was completed for the White Creek and Tappen Creek crossings. It formed the basis for the design of options and construction drawings for each crossing.

2.2. WATERSHED DESCRIPTIONS

The Tappen Creek and White Creek watersheds are presented in Figure 2-1 and Figure 2-2, respectively. Both watersheds have unique elements that significantly affect their effective watershed area and the subsequent hydrologic analysis. The physical characteristics of the watersheds are presented in Table 2-1 and discussed below.

2.2.1. White Creek

White Creek drains a catchment area measured at 138.5 km^2 that is located between the two southern arms of Shuswap Lake. This creek eventually flows south into Shuswap Lake opposite the City of Salmon Arm. We identified two distinct areas within the catchment: Upper White Creek, comprising 64.9 km^2, and draining into White Lake and Little White Lake; and Lower White Creek, that drains the remaining 74.3 km^2 downstream of the lake (see Figure 2-2).

There is a flow control structure at the outlet of White Lake that regulates the flow from the upper watershed. A routing analysis was completed for the Upper White Creek watershed to confirm the amount of attenuation and contribution of runoff to the crossing location, which is summarized in Section 3.1.1.

Table 2-1 summarizes the physical characteristics of the White Creek watershed within the study area.

Table 2-1: Physical Characteristics of the White Creek Watersheds.

Watershed	Area (km^2)	Elevation (m) Minimum	Elevation (m) Mean	Elevation (m) Maximum	Mean Overland Slope (%)
Lower White Creek	74.3	351	598.9	1442	20.9
Upper White Creek	64.9	469	763.5	1635	23.2

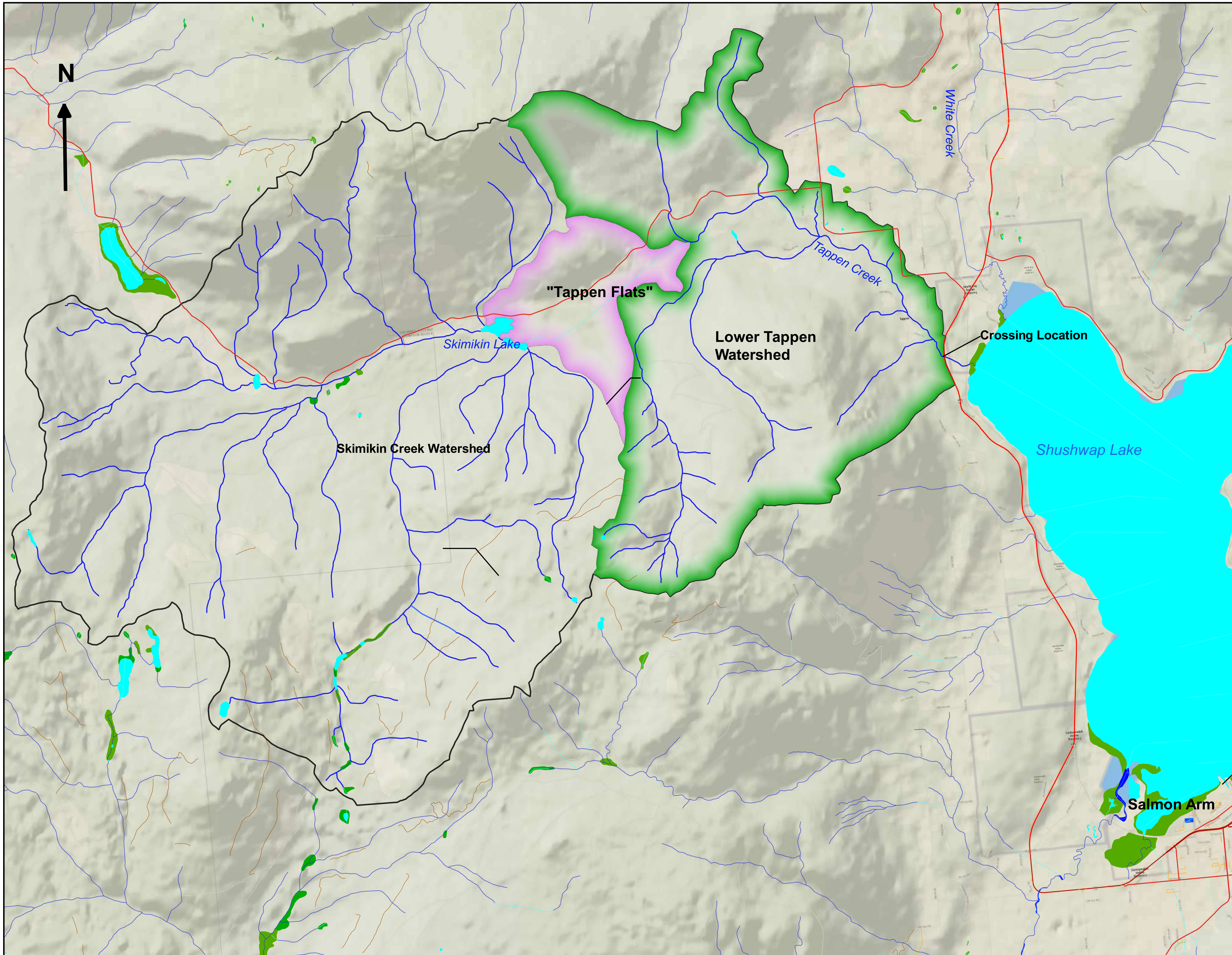
Coarse fish species are present in White Creek, as outlined in numerous environmental reports produced for this project. As such, the crossing must be fish passable.

2.2.2. Tappen Creek

Tappen Creek drains an area of 114.7 km^2. The watershed, however, is divided into three distinct parts. The upper watershed (Skimikin Creek) drains east into Skimikin Lake and comprises an area of 75.5 km^2. There is a 5.5 km^2 lower gradient area immediately east of Skimikin Lake (identified for this study as Tappen Flats). The Lower Tappen Creek watershed, draining 33.8 km^2, is situated east of Tappen Flats (see Figure 2-1). Examining topography and aerial imagery reveals that there is no overland drainage conveyance connection between the Skimikin Creek and Tappen Flats areas, and Lower Tappen Creek. We suspect that there is groundwater flow from these areas to the lower watershed.

With respect to the evaluation of extreme flows for the purpose of this design, we will only consider the Lower Tappen Creek watershed and its direct flow contribution. Subsurface or groundwater flow from the Skimikin Lake watershed is severely attenuated and thus, would not have a significant effect on the extreme peak flow anticipated for the design.

Numerous fish species have been documented in Tappen Creek, both upstream and downstream of the existing crossing. Therefore, crossing must be designed to allow fish passage.



BRITISH COLUMBIA
Ministry of Transportation

Ford Road to Tappen Valley Road

McElhanney

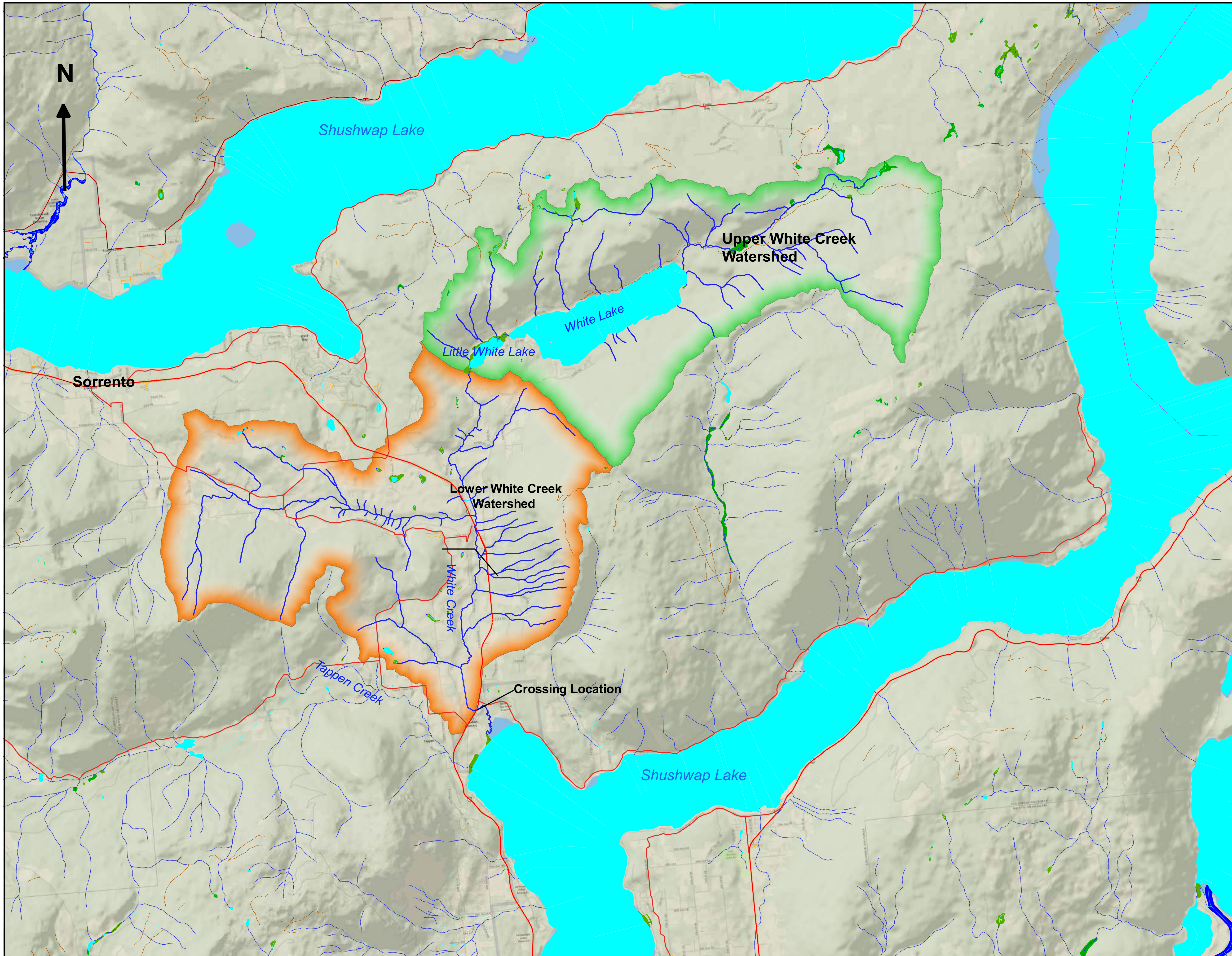
0 1 2 km

Scale= 1:60,000

Projection: BC Albers
Datum: NAD83

Project No.	Date
2121-00113-00	Dec 7, 2018

Figure 2-1
Tappen Creek Watershed



BRITISH COLUMBIA
Ministry of Transportation

Ford Road to Tappen Valley Road

McElhanney

0 2 4 km

Scale=1:100,000

Projection: BC Albers
Datum: NAD83

Project No.	Date
2121-00113-00	Dec 7, 2018

Figure 2-2

White Creek Watershed

3. HYDROLOGIC ANALYSIS

3.1. REGIONAL HYDROLOGIC ANALYSIS

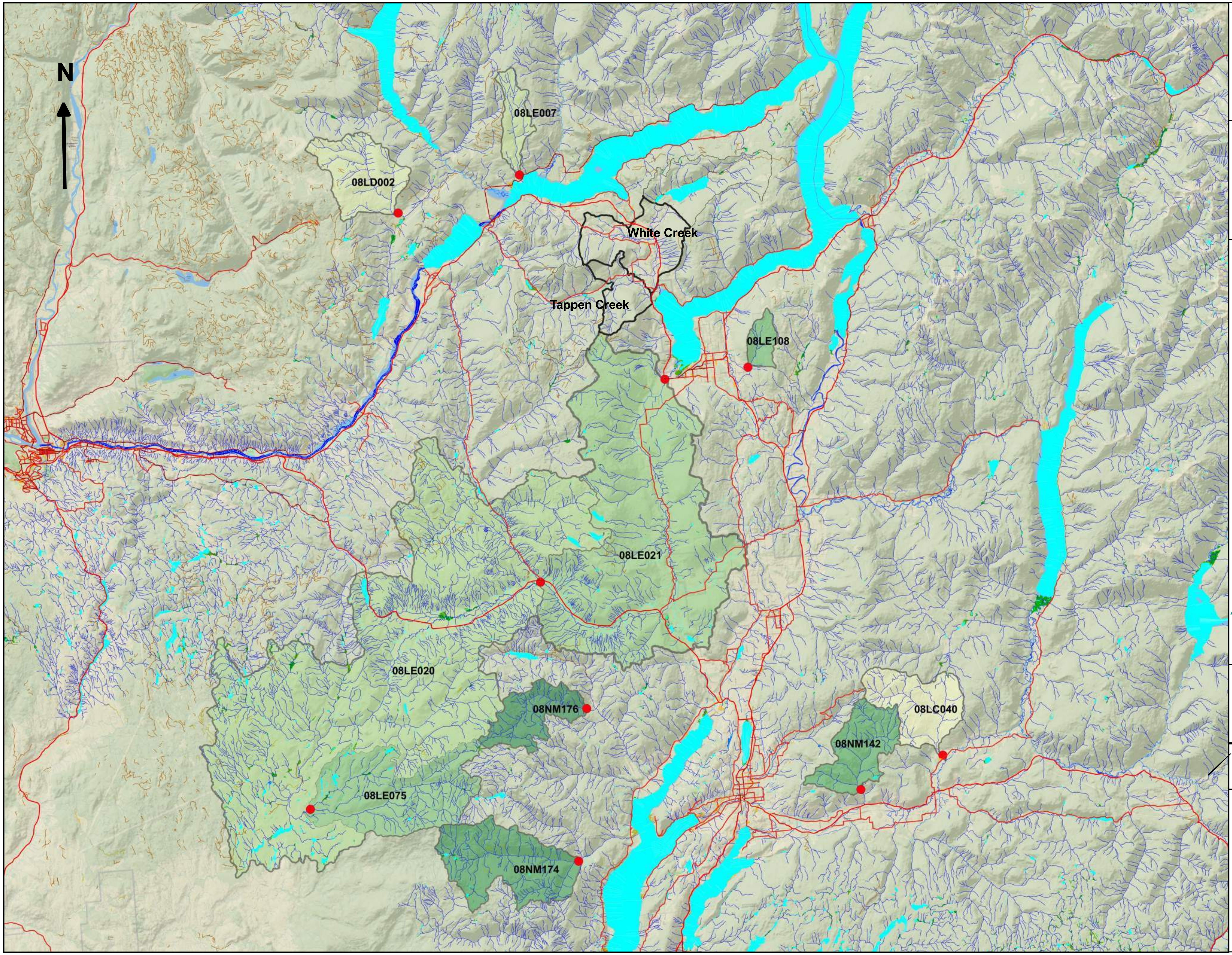
To estimate the flows for various return periods in Tappen and White Creeks, a regional hydrologic analysis was performed. A regional hydrologic analysis relies on the analysis of flow data from the Water Survey of Canada (WSC) stream gauging stations near or in the study area and applies that analysis to the watersheds of interest. The process for conducting a regional hydrologic analysis is as follows:

- Identify WSC stations near or in the study area, with similar physiographic characteristics and with sufficient period of record;
- Delineate the WSC stations' catchments and determine physiographic properties (GIS analysis);
- Perform a statistical frequency analysis, on the annual peak instantaneous flow data and annual maximum average daily flow data, to predict flows for various return periods for each WSC station;
- Plot flow versus station watershed area and perform a regression analysis to derive a regional curve for each return period; and
- Apply the regression curve equation to each watershed (Tappen Creek, White Creek) to predict flows for specified return periods for those catchments.

We identified seven WSC stations that are near the project area and have a sufficient period of record while displaying similar physical characteristics to the two watersheds of interest. Table 3-1 highlights the seven WSC stations and presents information relating to their status and available data. Figure 3-1 shows the location of each WSC station and its upstream watershed area. The physical characteristics of the watersheds are presented in Table 3-2. The WSC watershed area reported in the table and used in the analysis is based on GIS watershed delineation using 1:20k provincial freshwater atlas (FWA) watersheds that have been accurately delineated down to *1st Order* watercourses.

Table 3-1: WSC Stations Identified for Regional Analysis.

WSC Station		Area	Data Record		Regulated	Station Status
Number	*Name*	*(km²)*	*Period*	*# of Years*		
08LE108	East Canoe Creek above Dam	16.4	*1983-2014*	32	No	Active
08NM176	Ewer Creek near Mouth	51.5	*1971-1986*	16	No	Discontinued
08LC040	Vance Creek below Deafies Creek	68.3	*1970-2014*	45	No	Active
08NM174	Whiteman Creek above Boeleau Creek	107.6	*1971-2014*	44	No	Active
08LE075	Salmon River above Salmon Lake	144.1	*1965-2002*	38	No	Discontinued
08LE020	Salmon River at Falkland	1049.3	*1911-2015*	74	Yes	Active
08LE021	Salmon River near Salmon Arm	1556.3	*1911-2015*	57	Yes	Active



BRITISH COLUMBIA
Ministry of Transportation

Ford Road to Tappen Valley Road

LEGEND

WSC Station

WSC Station Watersheds

08LC040
08LD002
08LE007
08LE020
08LE021
08LE075
08LE108
08NM142
08NM174
08NM176

McElhanney

0 5 10 15 km

Scale= 1:400,000

Projection: BC Albers
Datum: NAD83

Project No.	Date
2121-00113-00	Dec 7, 2018

Figure 3-1

WSC Gauging Station and Watershed Location Used in Regional Hydrologic Analysis

Table 3-2: WSC Station Watersheds - Physical Characteristics

WSC Station		**Area**	**Elevation (m)**			**Ave. Overland Slope**
Number	*Name*	*(km^2)*	*Minimum*	*Mean*	*Maximum*	*(%)*
08LE108	East Canoe Creek above Dam	16.4	579	1041.6	1359	19.9
08NM176	Ewer Creek near Mouth	51.5	657	1398.6	1760	28.1
08LC040	Vance Creek below Deafies Creek	68.3	489	1079.4	1889	21.7
08NM174	Whiteman Creek above Boeleau Creek	107.6	614	1400.1	2039	29.9
08LE075	Salmon River above Salmon Lake	144.1	951	1388.2	2035	21.9
08LE020	Salmon River at Falkland	1049.3	584	1191.8	2035	21.1
08LE021	Salmon River near Salmon Arm	1556.3	351	1075.2	2035	20.8

Flood frequency analysis was performed on the annual peak instantaneous flow and annual peak average daily flow data for each station. For years where only average daily flow data is available, the corresponding peak instantaneous flow was estimated using the average peak instantaneous flow to average daily flow (I:D) ratio for years where both data were recorded for that station. The statistical analysis was completed using a dedicated script written in the *R* programming language. Publicly available algorithms and software packages, written to perform specific tasks like data analysis, statistical distribution fitting, and graphing, were used within the program script(*lmomco, ggplot2*). General methods employed in the flood frequency analysis included the following steps:

- Determining the L-moments for each data set;
- Fitting up to six statistical distributions to each data, including:
    - General Extreme Value (GEV);
    - Three Parameter Log-normal (3LN);
    - Log-Pearson Type III (LP3);
    - Wakeby (WAK);
    - Gumbell (EV1); and
    - Generalized Logistic
- Visually assessing the goodness-of-fit for each distribution against the empirical probability distribution of the data.

The Generalized Extreme Value (GEV) distribution was selected as the best-fit distribution across all stations. The predicted flow for each return period is presented in Table 3-3.

Table 3-3: WSC Stations Flood Frequency Analysis - Estimated Peak Instantaneous Flows

WSC Station		**Area**	**Flow (m^3/s)**			
Number	*Name*	*(km^2)*	*200-yr*	*100-yr*	*10-yr*	*2-yr*
08LE108	East Canoe Creek above Dam	16.4	4.16	3.6	2.02	1.01
08NM176	Ewer Creek near Mouth	51.5	12.72	11.36	7.13	4.11
08LC040	Vance Creek below Deafies Creek	68.3	8.81	8.31	6.17	3.82
08NM174	Whiteman Creek above Boeleau Creek	107.6	24.27	21.97	14.12	7.74
08LE075	Salmon River above Salmon Lake	144.1	18.23	17.24	13.01	8.39
08LE020	Salmon River at Falkland	1049.3	67.84	59.81	35.73	19.43
08LE021	Salmon River near Salmon Arm	1556.3	70.21	65.89	48.77	31.68

Plotting flow against watershed area for each return period and performing a power regression analysis derived a regional curve. Figures 3-2a to 3-2d show the regional regression curves for the 200-year, 100-year, 10-year, and 2-year peak instantaneous flows.

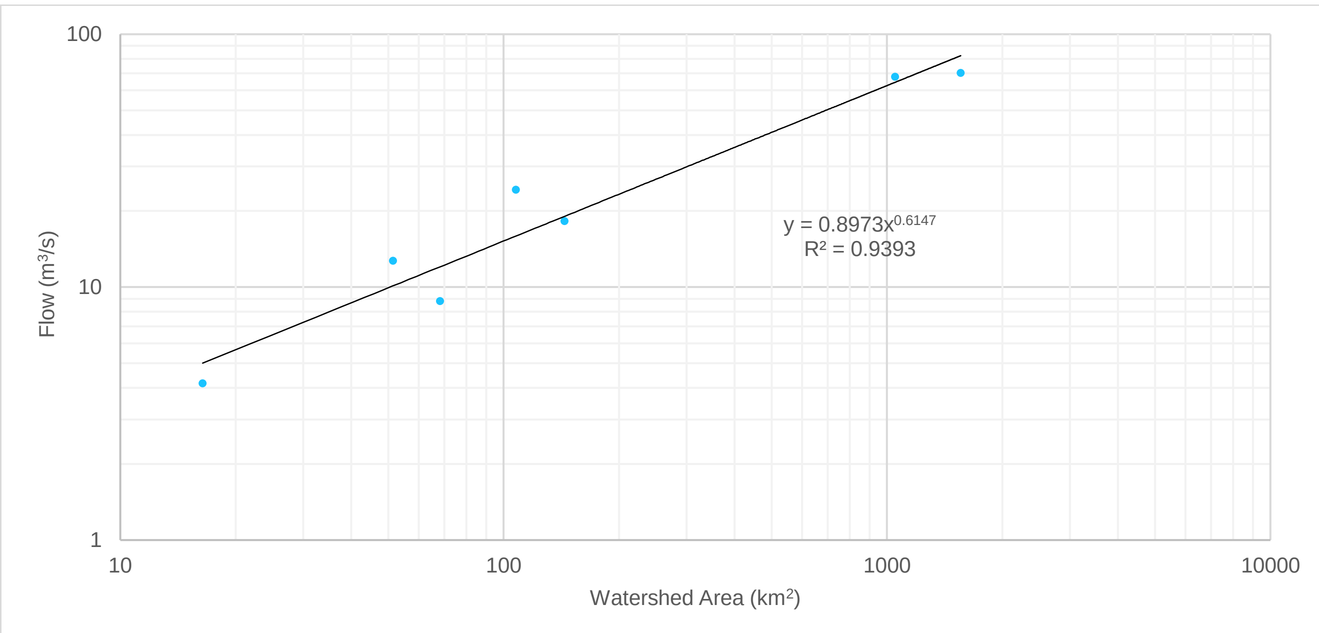


Figure 3-2a: Regional Hydrologic Analysis - 200-year Peak Instantaneous Flow

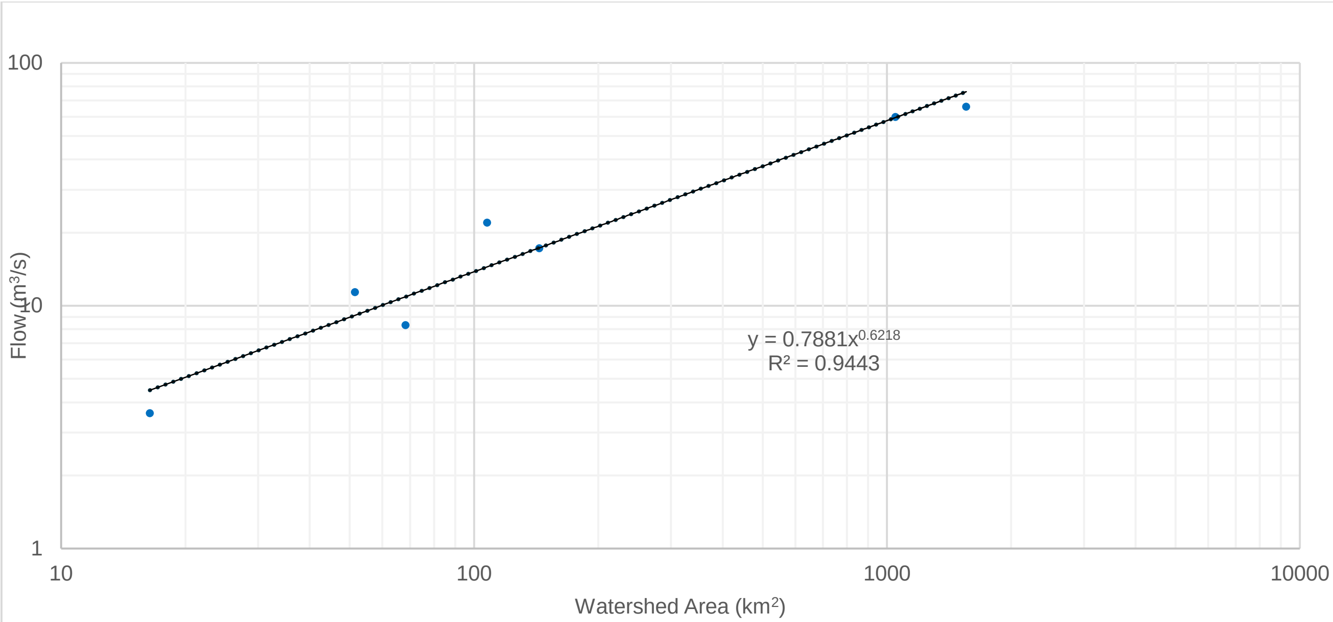


Figure 3-2b: Regional Hydrologic Analysis - 100-year Peak Instantaneous Flow

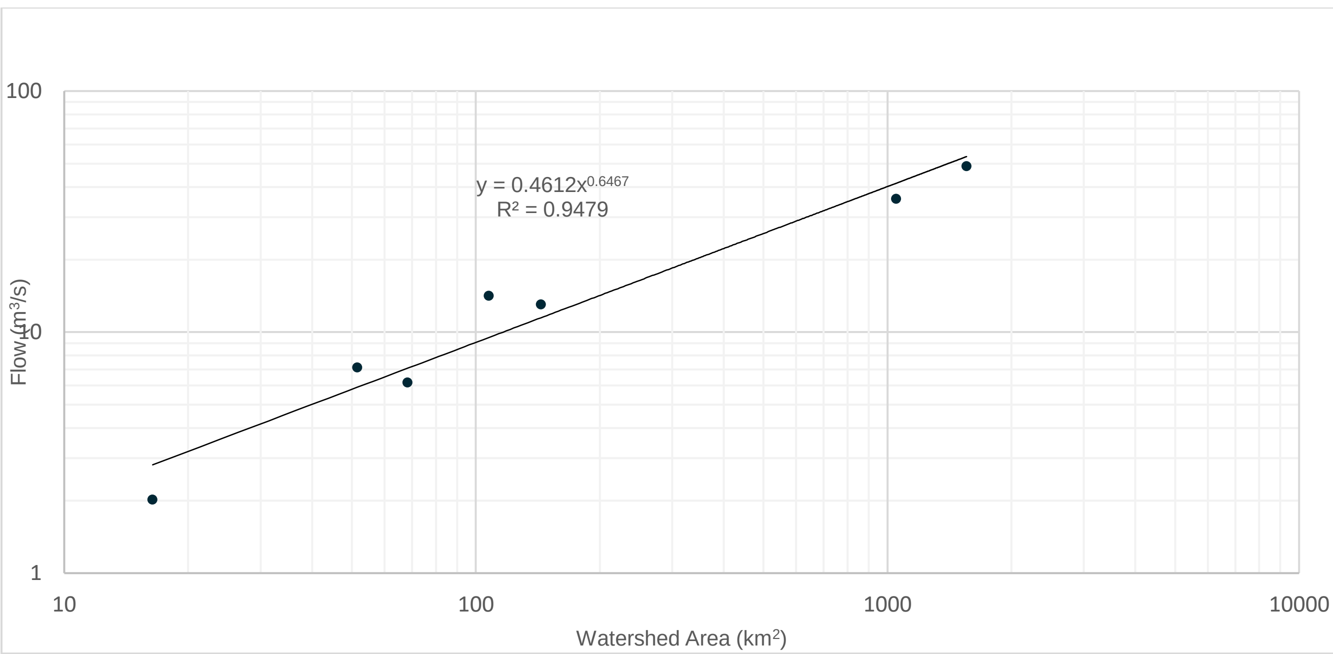


Figure 3-2c: Regional Hydrologic Analysis - 10-year Peak Instantaneous Flow

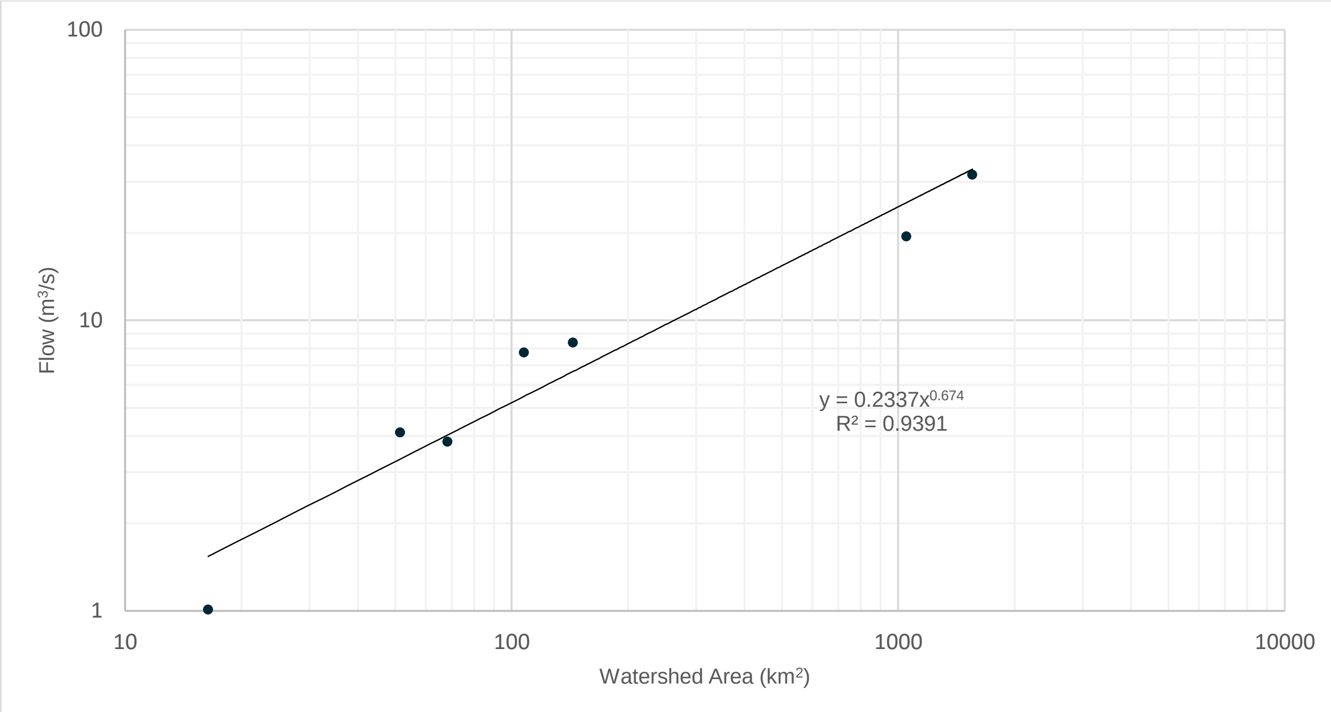


Figure 3-2d: Regional Hydrologic Analysis - 2-year Peak Instantaneous Flow

3.1.1. Application of Regional Analysis to Tappen Creek and White Creek

The application of the regional hydrologic analysis to our project watersheds is dependent on the *effective* watershed area for each watercourse.

Tappen Creek

For Tappen Creek, there is no obvious overland hydraulic connectivity between Skimikin Creek, Tappen Flats, and Lower Tappen Creek. We suspect that there is groundwater connectivity from the upper watersheds to lower Tappen Creek, however this will not significantly influence the peak flows experienced during extreme events. As a result, we have adopted an effective watershed area of **33.8 km^2** for Tappen Creek.

Local knowledge gathered through public meetings with stakeholders indicate that there are numerous springs within Tappen Creek, which suggests sub-surface and groundwater flow is prevalent in the area.

White Creek

White Creek is comprised of two distinct watershed areas, Upper White Creek and Lower White Creek. The upper watershed flows into White Lake, then into Little White Lake. White Lake has a weir at its outlet. Shown in Figure 3-3 and Figure 3-4, the weir is approximately 15 m long with a crest elevation that is 0.8 to 1.0 m higher than the downstream channel bed. It is a registered dam with the Province and is owned and operated by the White Lake Users Committee. We note that there is an access structure and scientific sampling structures downstream of the weir.

White Lake and Little White Lake represents approximately 10% of the total Upper White Creek watershed area and provides significant attenuation of runoff from both freshet snowmelt and storm events. The lower watershed, conversely, experiences little attenuation of runoff.

Figure 3-3: White Lake Outlet Control Structure

Figure 3-4: White Lake Outlet Control Structure

White Creek Watershed - Routing Analysis

To estimate an effective watershed area that will be used in the regional hydrologic analysis, we developed a hydraulic model that predicts the hydraulic response of the watershed to an extreme storm event. A PCSWMM model was developed to predict the time of concentration for the lower watershed, the upper watershed, and detail the attenuation provided by White Lake and Little White Lake. Initially developed for urban watershed simulations, PCSWMM can also be used to simulate hydrologic response(s) for small to median sized rural watersheds.

PCSWMM Model Development

The Digital Elevation Model (DEM) was sourced from the BC Data Catalogue. This DEM is available in 25 m resolution. To properly delineate sub catchments in the watersheds, a conditioned DEM was created using the raw DEM and the pre-defined White Creek stream network. A conditioned DEM is a DEM with the channel locations incorporated for the purpose of identifying drainage networks. The conditioned DEM was then used to create the sub catchments and the drainage network. The watershed discretization was completed using the built-in tool in PCSWMM. For modeling purposes, a minimum watershed area of 500 ha was established.

Design Storm

An SCS Type 1A 24-hour design storm was selected based on the site location. The projected rainfall depth for the 100-year, 24-hour duration storm due to climate change is 63.17 mm, based on the IDF information for the Salmon Arm Airport climate station (#1166R45). The rainfall was distributed uniformly across the watersheds for the purpose of this analysis.

Infiltration Losses

The land use and soil type data were also sourced from BC Data Catalogue. Upper White Creek is a forested watershed. Agriculture dominates the Lower White Creek watershed. Soil type was identified as sandy clay to silty loam. The hydrologic parameters were estimated based on the land use and soil type information. They are summarized in Table 3-4. The Modified Green-Ampt method was selected for estimating infiltration losses.

Table 3-4: Hydrologic Parameters Summary

DESIGN PARAMETERS	Value
Slope	various
Impervious %	3
Manning’s N Impervious	0.015
Manning’s N Pervious	0.15
Depression Storage Impervious (mm)	2
Depression Storage Pervious (mm)	5
Suction Head (mm)	240 (Sandy Clay) / 169 (Silty Loam)
Hydraulic Conductivity (mm/hr)	0.5 (Sandy Clay) / 6 (Silty Loam)
Initial Deficit (fraction)	0.2

Hydraulic Routing

The lake bathymetry was sourced from the BC Data Catalogue Lake Bathymetric Maps. The depth-area relationship of White Lake is illustrated in Figure 3-5.

The creek channel was simplified using a trapezoidal shape with 3m bottom width, 3:1 side slope, and a Manning’s n-value of 0.035. The channel length and slope were calculated automatically based on the DEM.

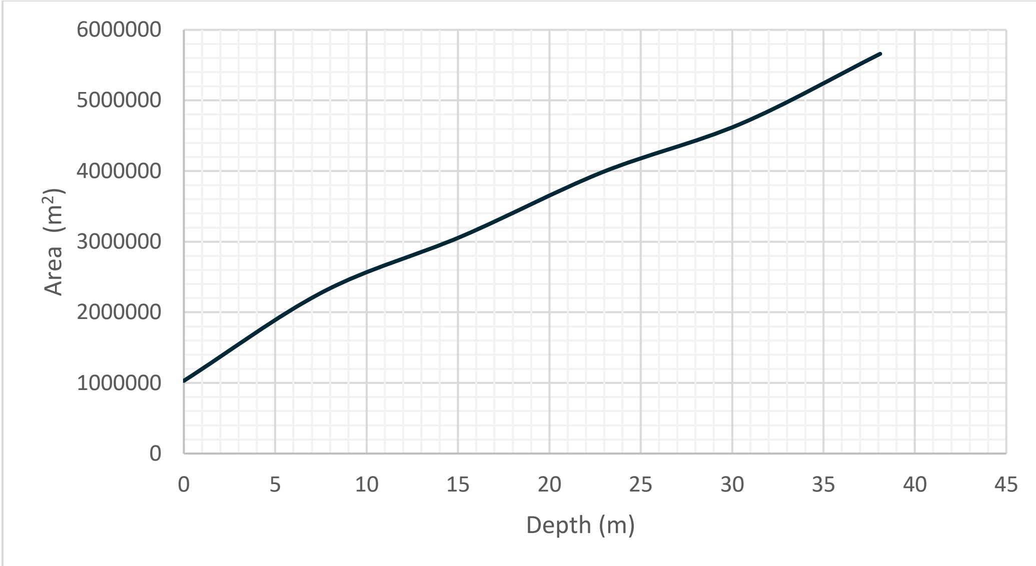


Figure 3-5: White Lake Depth-area Curve

Lake Outlets

A 15 m long sharp crested weir was modeled at the White Lake outlet. A trapezoidal channel was assumed at the Little White Lake outlet. The initial depth in the lake was set at the crest elevation of the weir, so that the outflow will gradually increase as the water surface elevation in the lake rises as a result of storm runoff.

PCSWMM Model Results

The 100-year, 24-hour design storm was simulated in the PCSWMM model. The simulation shows that the total time of concentration for the Upper White Creek watershed and the attenuation time of White Lake is longer than the time of concentration of the Lower White Creek watershed: the peak flow generated by the lower White Creek watershed will reach the crossing before the attenuated flow from the upper White Creek watershed. The inflow and outflow hydrographs for White Lake, as well as the simulated flow at the highway crossing under the 100-year, 24-hour design storm are illustrated in Figure 3-6.

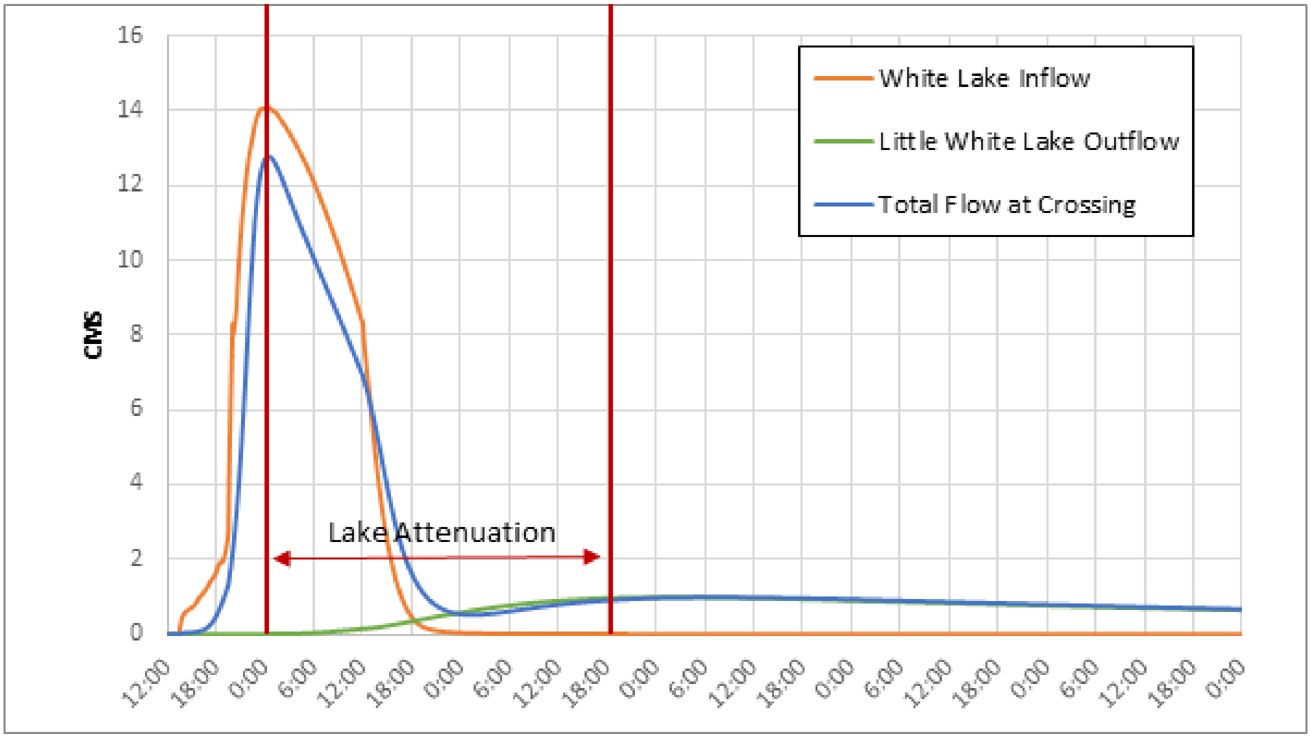


Figure 3-6: White Lake Inflow and Outflow Hydrograph

During the simulation, lake water levels increased by approximately 15 cm, thus indicating that the runoff from the Upper White Creek watershed is relatively small compared to the total storage volume of the lakes. The lakes provide approximately 18 hours of attenuation for the 100-year, 24-hour event.

White Creek Effective Drainage Area

Based on the results of the routing analysis, runoff from the Upper White Creek watershed does not contribute to the peak instantaneous flow at the proposed Highway 1 crossing. Flow from the Upper White Creek watershed reaches the crossing after the peak runoff flow from the Lower White Creek watershed. To estimate the design flow for the proposed crossing, we have adopted an effective watershed area of **74.3 km^2**, which is the watershed area of Lower White Creek.

Design Flow for Tappen Creek and White Creek

Applying the regression curve equations to the watersheds in our study area predicts the flow for various return periods for each watershed. These data are presented in Table 3-5.

Table 3-5: Estimated Peak Instantaneous Flows for Tappen and White Creeks

	Area	Flow (m^3/s)			
Watershed	*(km^2)*	*200-yr*	*100-yr*	*10-yr*	*2-yr*
Tappen Creek	33.8	7.8	7.0	4.5	2.5
White Creek	74.3	12.7	11.5	7.5	4.3

3.2. SHUSWAP LAKE

Under extreme conditions, the water level in Shuswap Lake will affect the hydraulic capacity of the White Creek crossing. The water level in Shuswap Lake, for the 2-year and 200-year return periods, was estimated by completing a flood frequency analysis on the historic WSC gauge at Sorrento (Station 08LE047) and comparing this to the estimates provided in the consultant's report *Floodplain Mapping Program – Salmon River – Shuswap Lake to Spa Creek (*Crippen Consultants, 1990). The values from the 1990 study were used in our analysis and are presented in Table 3-6.

Table 3-6: Shuswap Lake Water Levels (from Crippen, 1990)

WSC Station		**Data Record**			
Number	*Name*	*Period*	*# of Years*	**200-year Water Level**	**2-year Water Level**
08LE047	Shuswap Lake near Sorrento	*1924-1979*	54	350.1	348.3

3.3. CLIMATE CHANGE

MOTI Technical Circular T04/19 and Engineers and Geoscientists of British Columbia (EGBC / APEGBC) require that the potential effects of climate change be considered in a design. For this project, a wholistic *Highway Infrastructure Climate Change – Resilient Report* was produced and submitted.

As outlined in the memo, we have increased the design flows by 10% and believe that this is a conservative approach. The revised design flow (Q200+Climate Change (CC)) for Tappen Creek is **8.58 m^3/s**. The revised design flow for White Creek is **14.80 m^3/s**.

3.4. DESIGN FLOWS DURING CONSTRUCTION

Temporary diversion of Tappen Creek and White Creek will be required to facilitate construction of the new crossings and highway improvements. MOTI has adopted a 10-year return period peak flow as the standard for temporary diversion works.

For diversions that will be in place for a duration longer that the approved fisheries construction window, temporary diversions structures must accommodate the *annual* 10-year return period peak instantaneous flow. These values are provided in *Table 3-5*.

For short-term diversion within the fisheries construction window, a *seasonal* 10-year return period peak instantaneous flow. A flood frequency analysis was performed on a subset of flow data for three select stations used in the regional hydrologic analysis and a relationship between watershed area and seasonal flow was developed. The subset of data was limited to annual peak flows recorded between July 1st and September 30th. The results of the season peak flow analysis are presented in *Table 3.7*.

Table 3-7: Seasonal (July-Sept) 10-year Return Period Peak Flows

	Area	**Flow (m^3/s)**
Watershed	*(km^2)*	*10-yr*
Tappen Creek	33.8	0.57
White Creek	74.3	1.23

4. HYDRAULIC ANALYSIS

A 1-dimensional (1-D) hydraulic model was developed using HEC-RAS (v5.0.7). LiDAR and site survey information were combined into a DEM to represent the existing ground surface. The proposed channel alignment and culvert location were incorporated into the surface. This surface served as the basis for the hydraulic model. Proposed road alignments, crossing locations, and channel re-alignments were provided by the design team.

4.1. WHITE CREEK

The proposed White Creek culvert and realigned channel are presented in drawings 8513-4 and 8513-5.

4.1.1. Design Criteria

Design criteria for the Highway #1 crossing of White Creek includes the following:

- Design flood: 200-year Peak Instantaneous Flow (BC Supplement to CHBDC S6-14).
- Inlet control headwater to diameter [depth] ratio (HW/D) shall not exceed 1.0
- Outlet control head loss shall be less than 0.3 m
- White Creek is watercourse is not considered navigable under the *Navigable Waters Act*.
- Cut-off walls shall be used at both ends for closed-bottom type soil metal structure where there is a possibility of uplift, piping or undermining (BC Supplement to CHBDC S6-14).

4.1.2. Crossing Design

The proposed White Creek crossing has a single culvert, conveying the watercourse beneath Kirkpatrick Road, Highway 1, and Tappen Station Road. The proposed design specifies a 4300 mm round Structural Plate Corrugated Steel Pipe (SPCSP) embedded at 40% of its diameter to accommodate fish passage. The pipe will be infilled channel substrate and fisheries gravels similar to the reconstructed channel. Habitat boulders, embedded to 50% of their diameter, will be placed throughout the length of the pipe to provide channel diversity. A design options analysis was part of a previous assignment, and validation of the proposed design option was confirmed in this assignment.

Cast-in-place concrete headwalls are proposed at the inlet and outlet. The headwalls do not extend to the bottom of the embedded culvert.

Cut-off Walls

Cut-off walls are not proposed for the structure. Uplift is not a concern since substantial headwalls are proposed at both ends of the pipe. Piping is not a concern given the length of the culvert, the minor gradient of the culvert, and the clay and silts lining that is specified at the base of the channel bottom up to the culvert. Undermining is also mitigated by a 50 kg rip rap apron at the culvert outlet. A rip rap apron is not specified at the culvert inlet. Low water velocities at the inlet results in predicted scour depth above the culvert invert elevation.

White Creek Re-alignment

The proposed culvert is approximate 25 m north of the existing watercourse/crossing. The channel will be re-aligned to suit the proposed culvert.

A 120 m channel will be constructed upstream of the proposed structure. The channel geometry and features include:

- A channel bottom with of 4.2m, excavated side slopes of 2H:1V up to an elevation of 350 m (approximately 1 m above channel bottom) at a constant gradient of 0.25%

- A 0.5m wide, 0.5m deep, low-flow channel, situated in the center of the channel bottom, will provide a direct flow path under low flow conditions.

- Habitat boulders (600-800 mm average dimension), will be interspersed in the channel at a density ofi.e.,1 boulder per 4 linear meters of channel.

- Channel bottom substrate includes a 50/50 mix of 25 kg rip rap and semi-angular small boulders, cobbles and gravels. This substrate will be top-dressed with rounded coho gravel.

- The 2:1 channel bank side slopes will be lined with 25 kg rip rap. This is larger rock than required based on predicted water velocities, however the purpose of the rock is to establish a stable channel bank. The voids in the rip rap will be filled with native (excavated) material and planted with willow stakes. The top elevation for the rip rap of 350 m was established because it represents the approximate Q2 water surface elevation and is the approximate elevation of the existing ground (and channel bank). Above this elevation, the water currently inundates the flood plain and will continue to occur with the proposed crossing.

- Any slopes above the rip rap will be revegetated as per the revegetation plan.

- The 2:1 embankments and channel bottom will be lined with a 300 mm layer of clays and silts. This layer will be compacted to achieve an impermeable lining and prevent water from percolating into the water table. Should inspection of the excavated channel reveal soils with low hydraulic conductivity (i.e. silts and clays), this layer will not be installed.

- Where excavation is deeper (i.e. within 30 m of the culvert inlet), slopes above the 350 m elevation will be established at 4H:1V. These slopes, along with all other disturbed soils, will be revegetated as per the approved revegetation plan.

Downstream of the proposed crossing, the watercourse meanders through an established wetland. A 10 m long rip rap apron (50 kg) will line the culvert outlet channel. Unmitigated, approximately 1 m of scour is predicted at this location. The apron provides mitigation from scour. The outlet channel will mimic the design of the upstream channel, transitioning into the wetland.

Hydraulics

The design flow condition for the proposed structure is the 200-year peak instantaneous flow, including climate change (Q200 + CC) and a downstream boundary condition of the predicted 200-year lake water surface elevation.

The 200-year, 100-year, 10-year, and 2-year peak instantaneous flows were also evaluated without consideration of the downstream lake level (i.e. with a *normal depth* boundary condition) to estimate hydraulic capacity and water velocity under these conditions. The highest water velocities at the crossing will be experienced without backwater effects from the lake.

The surface, the proposed culvert, the approximate road location, and channel cross-sections are shown in Figure 4-1.

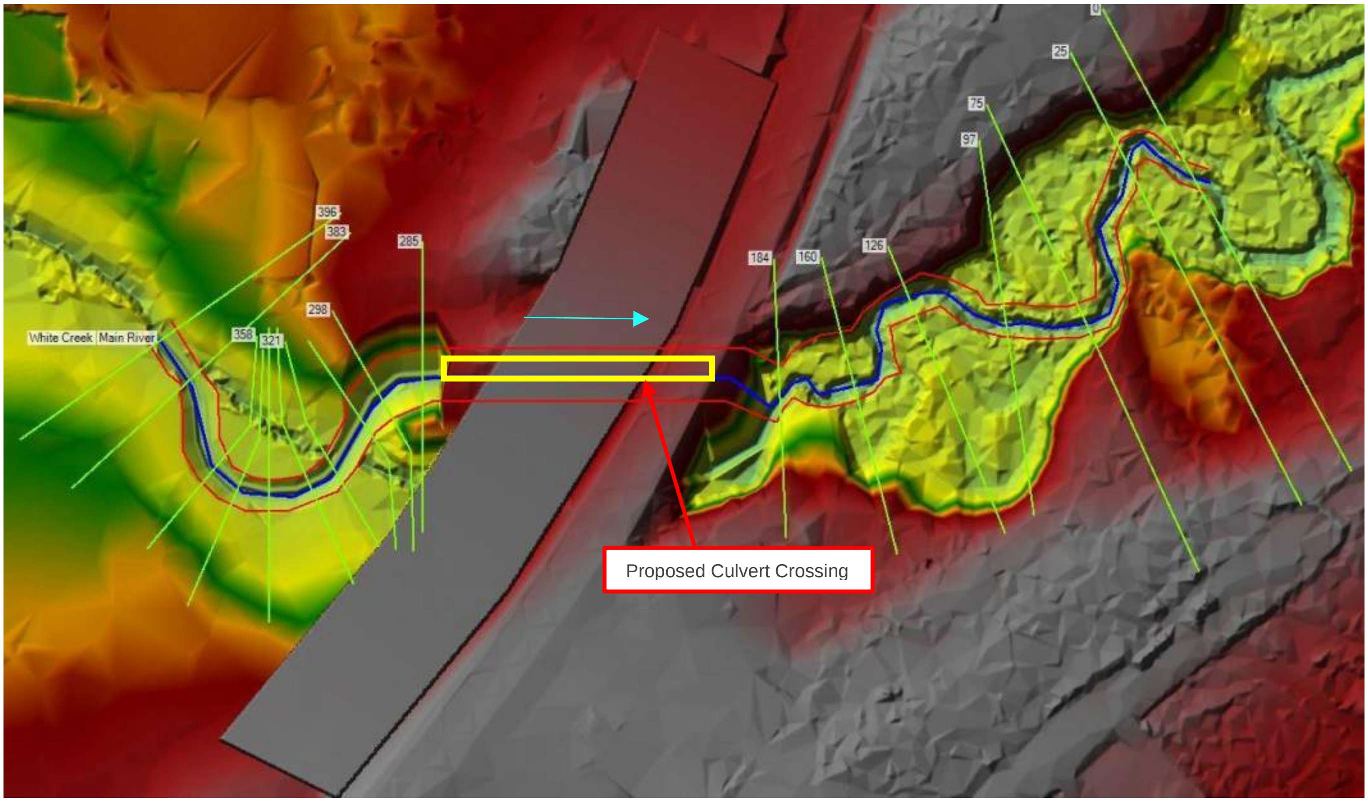


Figure 4-1: White Creek HEC-RAS Model Plan and Cross-sections

4.1.3. Results

Design Flow (Q200) with 200-year Lake Level

The results show that under Q200+CC design flow condition, the proposed culvert has the hydraulic capacity to convey the required flow. The culvert is 96.0 m long with a gradient of 0.25%.

The predicted water surface elevation upstream of the culvert is **351.0 m**. *Figure 4-2* shows the predicted water profile along White Creek at the culvert crossing.

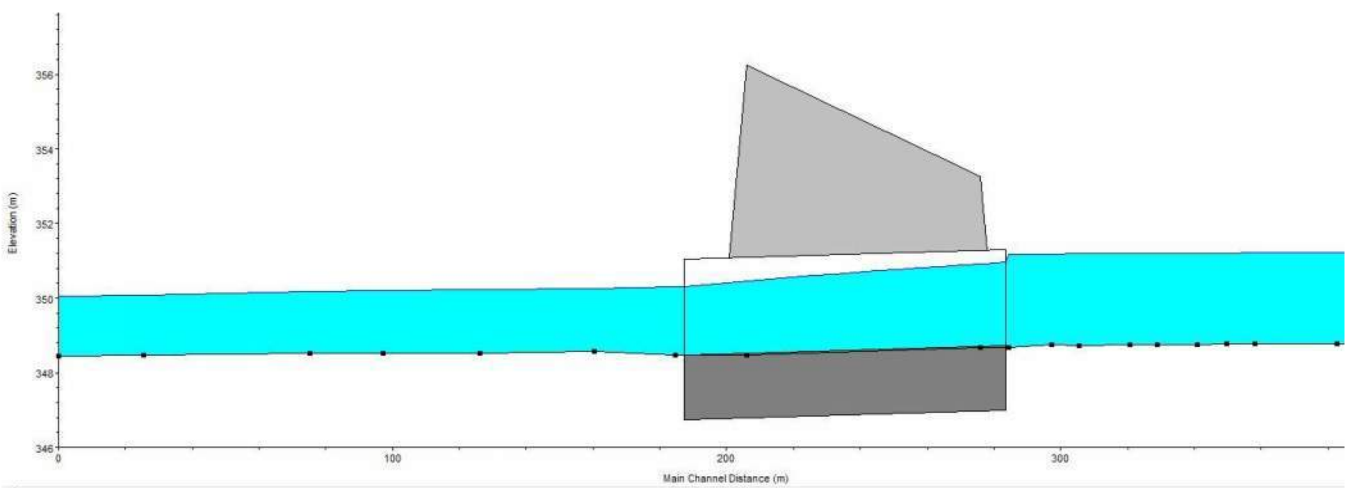


Figure 4-2: White Creek Water Surface Elevation Profile - Q200+CC with 200-yr Lake Level

Flow without Lake Influence

To estimate the maximum water velocities in the project area for various flows, a downstream boundary condition of normal depth was assumed. This suggests that the lake level will not influence the hydraulic conditions at the crossings. *Figure 4-3* shows the water surface profile of White Creek through the proposed crossing, under the design flow (Q200+CC) and without the influence of the lake.

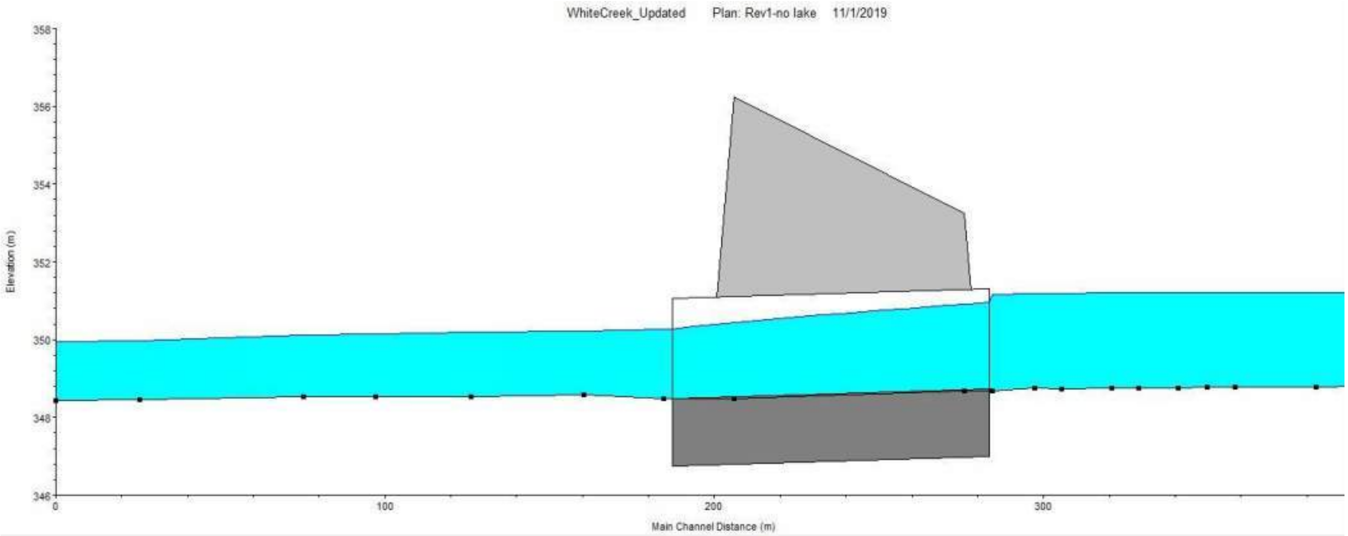


Figure 4-3: White Creek Water Surface Elevation Profile - Q200+CC with no lake influence

The water surface elevation at the culvert inlet is 350.95 m, resulting in an HW/D of 0.82. This value is similar to the water surface elevation with the lake's influence, albeit slightly lower.

The water surface elevations and water velocities, for various flows without lake influence are presented in *Table 4-1*.

Table 4-1: White Creek - Water Surface Elevations and Water Velocities for various Flows

		Return Period			
Crossing	**Location**	**2-year**	**10-year**	**100-year**	**200-year**
White Creek	*Inlet – WSE (m)*	350.17	350.38	350.74	350.86
	Outlet – WSE (m)	350.07	350.10	350.16	350.17
	Inlet –Velocity (m/s)	0.11	0.16	0.18	0.19
	Outlet–Velocity (m/s)	0.38	0.65	0.98	1.07

4.1.4. Scour Analysis

Scour at White Creek was assessed using two methodologies. The first used the Hydraulic Engineering Circular No. 18 (HEC-18) contraction scour method to analyze scour through the culvert. The second used Blench's Regime Formula as outlined in the TAC Guide to Bridge Hydraulics. Detailed calculations for each method are provided in *Appendix A*. A D_{50} grain size of 10 mm was assumed based on the gravels being used as the channel substrate to infill the newly constructed channel and to embed the culvert.

4.1.2.1. HEC-18 Contraction Scour

Following the steps outlined in HEC-18, the type of contraction scour occurring at the culvert would be clear-water contraction scour. The resulting scour depth is estimated at 0.1 m.

4.1.2.2. Blench's Regime Formula

Blench's regime formula for natural scour was calculated at six locations that ranged from 20 m upstream of the culvert to 40 m downstream of the culvert. The formula estimated that the largest amount of scour would occur at the downstream end of the culvert at **1.1 m**. The cross section 20 m downstream of the culvert had an estimated scour of 0.9 m, while the cross section 40 m downstream of the culvert had an estimated 0.6 m of scour.

4.1.2.3. Final Scour Estimate

The proposed channel slope is similar to the existing channel; therefore, we anticipate fines will settle in the channel, as was evident in the existing channel. These fines will eventually migrate through the channel and culvert. The final scour is estimated at **1.1 m**.

4.2. TAPPEN CREEK

Hydrotechnical drawings for the proposed Tappen Creek crossing are presented in Drawings 10254-11, 10254-12, 10254-13 and 10254-14 of the design package.

4.2.1. Design Criteria

The following design criteria were employed for the Tappen Creek crossing:

- Design flood: 200-year Peak Instantaneous Flow (*BC Supplement to CHBDC S6-14*).
- Clearance (freeboard) must be a minimum of 1.5 m (*BC Supplement to CHBDC S6-14*)
- Minimum elevation of wildlife path must be above the water surface elevation under the 10-year peak instantaneous flow condition (*established by project team*).
- Watercourse must be fish passable (per *Fish Stream Crossing Guidebook*)
- Tappen Creek in not considered navigable under the *Navigable Waters Act*.

4.2.2. Bridge

A 26.6 m long, 20.4 wide single span bridge founded on piles is proposed for the Tappen Creek crossing. It will accommodate both Tappen Creek and an elevated wildlife path that parallels the creek through the bridge opening.

Requirements for the wildlife passage (width, clearance, etc.) are described in the *Structural Design Report*. From a hydrotechnical perspective, it was determined that the minimum elevation of the path adjacent to Tappan Creek would be above the water surface elevation under the 10-yr peak instantaneous flow condition. During events larger than a 10-yr flow, maintenance may have to occur to re-establish vegetation, woody debris, and the walking surface on the wildlife path. The width of the wildlife path is 3.0 m.

Re-aligned Watercourse

Tappen Creek is currently conveyed beneath Highway #1 via a 2400 x 1800 concrete box culvert. In order to accommodate the new bridge, Tappen Creek will be re-aligned approximately 5 m north of the existing crossing.

The surveyed wetted channel width varies between 2.2 m and 3.7 m upstream and downstream of the proposed re-aligned watercourse. It has an average channel gradient of 1.5%.

The re-aligned watercourse has the following characteristics:

- A total channel length of approximately 110 m;
- Four distinct reaches based on channel gradient of (upstream to downstream) 1.8% (natural), 4.7%, 0.5% and 1.0%. The channel was steepened to 4.7% in order to achieve required bridge clearances and elevations based on design parameters.
- A pool-riffle morphology was created to provide quality aquatic habitat by constructing 5 weir-riffle structures in the re-aligned channel.
- The weir crests are approximately 300 mm above the average channel bottom elevation. The downstream riffles are 5-10 m long (depending on channel gradient) and composed of 50 kg rip rap, with large habitat boulders interspersed.
- Pools, up to 500 mm below the average channel bottom elevation, will be shaped (excavated between the riffle and the downstream crest). Figure 4.4a shows the shaped pools during construction. Figure 4.4b shows the same reach immediately post-construction, Figure 4.4c shows same reach 3-months post-construction.
- Habitat boulders (600-800 mm average diameter) will be placed along the length of the channel
- Large wood debris (LWD) structures, consisting of two 5-7 m long logs (cedar, hemlock, spruce or pine) with bark relatively intact and root wads attached, connected to a total of six anchor boulders with stainless steel cables, will be place in all the pools as per the design drawings. Anchoring specifications were derived from Watershed Restoration Technical Circular 8.
- The minimum channel width is established by the width of the weir crests (2.5 m). Average wetted widths of the channel will be 2.7 - 3.2 m depending on location and flow. A 1.5 m habitat bench that is set an elevation of 700m above the average channel bottom was included in Reach 2 and 3. Its purpose is to provide addition 1.5 m of channel width under specific storm events.

Figure 4-4a: Shaped pool during construction. Habitat boulders and weir crest rocks visible.

Figure 4.4b: Pool-riffle within one week of activation (water introduced).

Figure 4-4c: Riffle-pool 3 months post construction with LWD, root wads and habitat boulders.

Channel Substrate and Erosion Protection

- The channel side slopes will be 1.5H:1, a minimum excavated channel bottom with is 1.6 m
- The channel bottom will be lined 550 mm thick mixture of 50% 50kg rip rap and 50% semi-angular small boulders, cobbles and gravels. The substrate will be top dressed with a 150mm thick layer of fish (“coho”) gravel
- The side slopes will be lined with 50kg rip rap. Interstitial voids in the rip rap will be filled with excavated materials that will by hydraulically washed into the rock. This will occur up to 700 mm above the minimum weir crest elevation. Beneath the rip rap and channel substrate, the channel will be lined with a 300 m layer of clays and silts. This layer will provide an impermeable layer preventing water from seeping into the ground water. Should the excavated channel conditions reveal impermeable soil conditions, this layer will not be installed.
- Additional rip rap is specified in the immediate vicinity of the proposed bridge. A thicker layer near the channel bottom, extending up to the pile cap, is required to maintain slope stability as designed and specified by the geotechnical engineer. All rip rap placed below the Q10 water surface elevation (i.e. 700mm above the crest elevation) must be infilled with native fill to ensure the water flows on the surface, and not subsurface through the voids in the rip rap.

Backwater Channel

Downstream of the proposed bridge, a short back channel is proposed. It incorporates the existing channel located downstream of the culvert outlet. The back channel will be connected to the re-aligned channel. LWD will be added to the back channel. The purpose of the back channel is to provide additional aquatic habitat. Under normal flow conditions, we do not anticipate water from the channel inundating the back channel. Any water in the back channel will be from ground water seepage. If no water is in the back channel, under certain flow conditions, the risk of fish stranding is low since not pools will not be excavated.

Hydraulic Analysis

The hydraulic conditions for the proposed channel configuration were assessed for the 200-year peak instantaneous flow plus consideration for climate change (Q200+CC). The 100-year, 10-year, and 2-year peak instantaneous flows were also evaluated to estimate the water surface elevation and corresponding water velocities. The water surface elevation of Shushwap Lake does not influence the Tappen Creek crossing.

The proposed bridge, with the wildlife path, and channel cross-sections are shown in *Figure 4-5*.

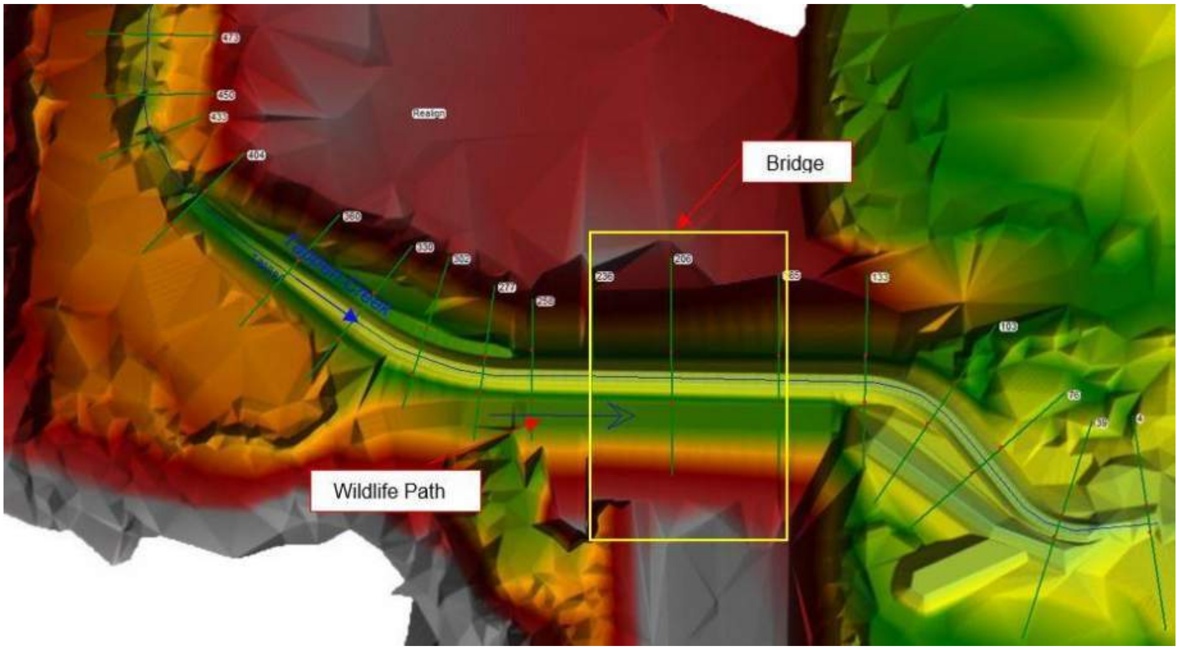


Figure 4-5: Proposed Bridge and Wildlife path at Tappen Creek HEC-RAS Model Layout

4.2.3. Results

The predicted water surface elevation upstream of the bridge is 354.0 m. *Figure 4-6* shows the predicted water profile of the re-aligned Tappen Creek at the upstream face of the bridge. Table 4.6 shows the detailed results of the hydraulic model, under design flow conditions. A minimum freeboard of 1.5 m is required. The minimum soffit elevation based on hydraulic parameters is 355.5 m. **We note that minimum soffit elevation and size of the bridge is not solely governed by hydraulic conditions**; the required clearance for wildlife passage was the governing factor in establishing the minimum soffit elevation of the structure.

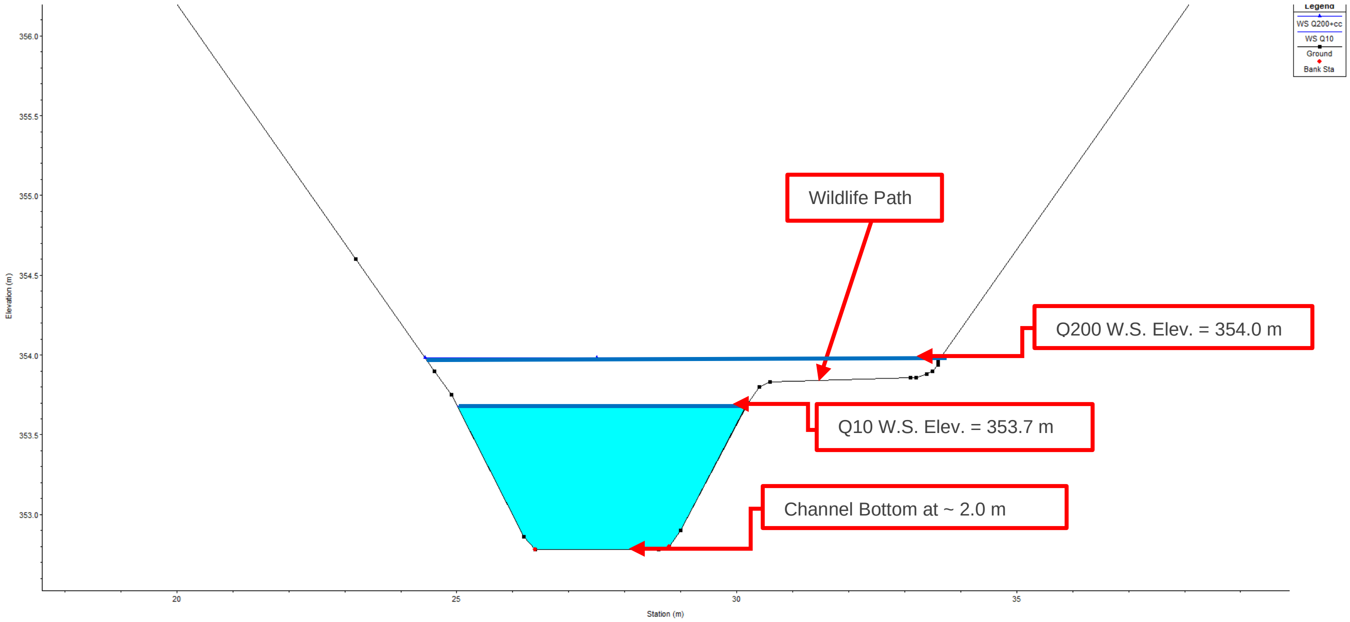


Figure 4-6: Tappen Creek Water Surface Elevation Profile at the Upstream Face of the Bridge - Q200+CC

Table 4-2: Tappen Creek - Hydraulic Modeling Results

Channel Station	Location	Flow (m^3/s)	Min Channel Elevation (m)	Water Surface Elevation (m)	Water Velocity (m/s)	Froude #
450		8.58	354.14	355.51	1.63	0.47
433		8.58	354.04	355.2	2.76	0.94
404	*Weir #1*	8.58	353.9	354.66	3.01	1.27
360		8.58	353.36	354.09	2.75	1.14
330	*Weir #2*	8.58	353.08	354.19	1.3	0.43
302		8.58	352.79	354.17	1.08	0.32
258	*Weir #3*	8.58	352.83	354.01	1.67	0.58
236	Bridge	8.58	352.49	353.97	1.48	0.47
206	Bridge	8.58	352.44	353.87	1.55	0.51
165	Bridge	8.58	352.38	353.7	1.77	0.61
133	*Weir #4*	8.58	352.58	353.21	2.92	1.33
103		8.58	352.23	353.27	1.19	0.45
76	*Weir #5*	8.58	352.45	353.15	1.55	0.63
39		8.58	352.03	353.05	1.31	0.43
4		8.58	352.12	352.92	1.7	0.65

Rip Rap Design

The design of rip rap is based on the predicted water velocities under design flow conditions. The highest water velocities will be experienced on the riffle immediately downstream of Weir Crest #1. This corresponds to the steep part of the channel. Table 4-2 shows the predicted water velocities in the channel. Using the predicted water velocity as the "velocity against stone", 50 kg rip rap is sufficient at the side slope of 1.5H:1V based on Table 1030.A in the *BC Supplement to TAC*.

4.2.4. Channel Scour

Contraction scour should not be an issue with the realigned channel since the basic channel geometry (channel width etc.) is being respected. The realigned channel provides a uniform channel geometry along its length. To confirm this, the HEC-18 methodology for contraction scour was applied to the Tappen Creek crossing as a check. Blench's Regime Formula was used to as it provides a more representative method for determining the scour depth in the channel. The detailed calculations for each method can be found in *Appendix A*. A D_{50} grain size of 10 mm was assumed based on the conservative assumption the channel is composed of gravels. The constructed channel will be composed of sands, gravels, cobbles and boulder mixed with 50 kg rip rap.

HEC-18 Contraction Scour

Following the steps outlined in HEC-18, the type of contraction scour occurring at the bridge would be live-bed contraction scour. The resulting scour depth is estimated at **0.05 m**.

Blench's Regime Formula

Blench's regime formula for natural scour was calculated at a cross section located within the middle of the bridge. The estimated scour level was **0.9 m**.

Total Scour Estimate

The estimated scour depth is **0.9 m**

4.2.5. Summary of Hydrotechnical Design Parameters

Table 4-3 presents a summary of the hydrotechnical design parameters.

Table 4-3: Tappen Creek Bridge - Summary of Design Parameters

Parameter	Value
Design Flow	8.58. m^3/s
Water surface elevation at Upstream Face of Bridge	354.0
Minimum Freeboard	1.5 m
Recommended Minimum Soffit Elevation (Hydraulics Only)	355.5 m
Max. Water Velocity (upstream channel)	3.0 m/s
Max Water Velocity (through bridge)	1.7 m/s
Max Water Velocity (downstream channel)	2.9 m/s
Required Rip Rap Class	50 kg
Max Scour Depth at bridge	0.9 m
Min Channel Width (at Weir Crest)	2.5 m

5. MINOR DRAINAGE

The *Hydrotechnical Design Report (Revised), Highway 1 – Ford Road to Tappen Valley Road (2019)* completed by McElhanney during the revised functional design is herein revised, specifically the minor drainage section.

Based on the project goals, the scope of work for the revised drainage analysis included:

1. Investigate potential changes to surface drainage patterns as a result of the modifications to the road geometry;
2. Confirm the increase in design storm rainfall volumes due to climate change;
3. Revise the detailed delineation of drainage subcatchments for the updated road design; and
4. Refine the hydrologic / hydraulic computational model for the proposed conditions;
5. Analyze the results of the computational model;
6. Confirm the proposed locations and sizes of culverts, catch basins, and ditches to prevent flooding; and
7. Present the results of the analysis.

5.1. METHODOLOGY

5.1.1. Design Criteria

The goal of the previous and current drainage design is to develop a conveyance system that can effectively drain the widening of the highway without flooding based on guidelines presented in Section 1000 of the *Supplement to TAC Geometric Design Guide* (2019). Culverts were sized to ensure that the HW/D ratio does not exceed 1.0 under either inlet or outlet control. Table 5-1 below outlines the design return periods used in the drainage design for various hydraulic structures. Climate change considerations were included for all return periods and a summary of the Design Criteria Sheet for Climate Change Resilience is included in the Climate Change Technical Memorandum.

Table 5-1: Design Return Periods for Hydraulic Structures (based on Table 1010.A - Supplement to TAC Geometric Design Guide (2019))

Hydraulic Structure	Return Period (years)
Stormwater Inlets	5
Highway Ditches	25
Culverts (<3m, ditches)	100

The 100-year event (with climate change considerations) was adopted as the design storm for all culverts (<3m), including along local roads, to ensure that all access points to properties and businesses would not be compromised during such an event. Highway ditch capacities were also assessed for the 100-year event to provide continuity with the capacity of the connecting culverts.

Runoff coefficients for the subcatchments were developed with reference to Table 1020.A - *Supplement to TAC Geometric Design Guideline* (2019). The relevant values that were accounted for in the assessment of the site hydrology are shown in Table 5-2.

Table 5-2: Runoff Coefficients

Surface Cover Physiography	Impermeable	Forested	Rural
Mountain (>30%)	1.00	0.90	-
Steep Slope (20-30%)	0.95	0.80	-
Moderate Slope (10-20%)	0.90	0.65	0.5
Rolling Terrain	0.85	0.50	0.4
Flat (<5%)	0.80	0.40	0.3

5.1.2. Computational Model

A single event hydrologic and hydraulic computational model was developed to ascertain the hydraulic performance of the proposed drainage system under peak flow conditions (based on the 25-year event with climate change consideration). This model was used to simulate rainfall on the various catchments and thus generate a resulting runoff through the drainage system. PCSWMM version 7.1.2480 software was used to create the hydrologic / hydraulic model. PCSWMM is an adaptation and enhancement of the well-known and widely used United States Environmental Protection Agencies (USEPA) Stormwater Management Model (SWMM) version 5.1. PCSWMM was developed by Computational Hydraulics International (CHI) as a combination hydrology-hydraulic modelling software. The hydrological component addresses the rainfall to runoff conversion by applying a user selected loss method which considers both pervious and impervious areas. A runoff hydrograph is then generated for each of the specified catchments based on the catchment's characteristic width and roughness. This hydrograph is then routed to a junction that either represents the upstream end of a ditch or the inlet of a culvert.

The input parameters used in the hydrologic simulation of the catchment areas were derived by assessing the available aerial imagery and geotechnical reports in order to develop an understanding of the surface and subsurface characteristics. Using this information, the input parameters were then selected from recommended values in the SWMM Manual. These values are presented in Table 5-3 for the both the hillside and roadway catchments.

Table 5-3: PCSWMM Input Parameters

Parameter	Value
Impervious Manning n	0.011
Pervious Manning n (hillside)	0.50
Pervious Manning n (roadway)	0.13
Impervious Depression Storage (mm)	1.80
Pervious Depression Storage (mm) (hillside)	7.60
Pervious Depression Storage (mm) (roadway)	3.00
Infiltration Loss Method	Green-Ampt
Soil Description	Silty Loam
Suction Head (mm)	170
Hydraulic Conductivity (mm/hr)	6.6
Initial Deficit	0.366

5.1.3. Rainfall Data & Design Storm Generation

The site hydrology assessment was completed with reference to Section 1020.05 in the *Supplement to TAC Geometric Design Guide* (2019). In order to develop the synthetic design storm for the PCSWMM hydrologic model, Environment Canada rain gauge locations were identified to obtain historical rainfall data for the project area. The 'Salmon Arm A' rain gauge is located at the Salmon Arm Airport and was selected as the representative gauge due to its proximity to the project area. Table 5-4 summarizes the design information for the 'Salmon Arm A' rain gauge.

Table 5-4: Rain Gauge Data Summary

Rain Gauge Name	I.D.	Period of Record	Years of Data	Return Period Rainfall (100-year 24-hour) (mm)
Salmon Arm A	1166R45	1964-2011	39	52.6

An SCS Type 1A 24-hour synthetic design storm was generated in PCSWMM since it best represents the rainfall distribution for coastal BC. The recorded rainfall volume for the 100-year return period was increased by 22% to account for the projected annual increase in precipitation derived from the Pacific Climate Impacts Consortium (PCIC) Plan2Adapt tool. The projected rainfall volume for the 100-year, 24-hour design storm is 64.17 mm and this was used to generate the synthetic hyetograph in the hydrologic model.

5.2. EXISTING MINOR DRAINAGE SYSTEM

The existing minor drainage system was assessed in the *Hydrotechnical Design Report – Final, Highway 1 – Ford Road to Kault Hill Four Laning, McElhanney (2016)* and was therefore not reassessed as part of this project.

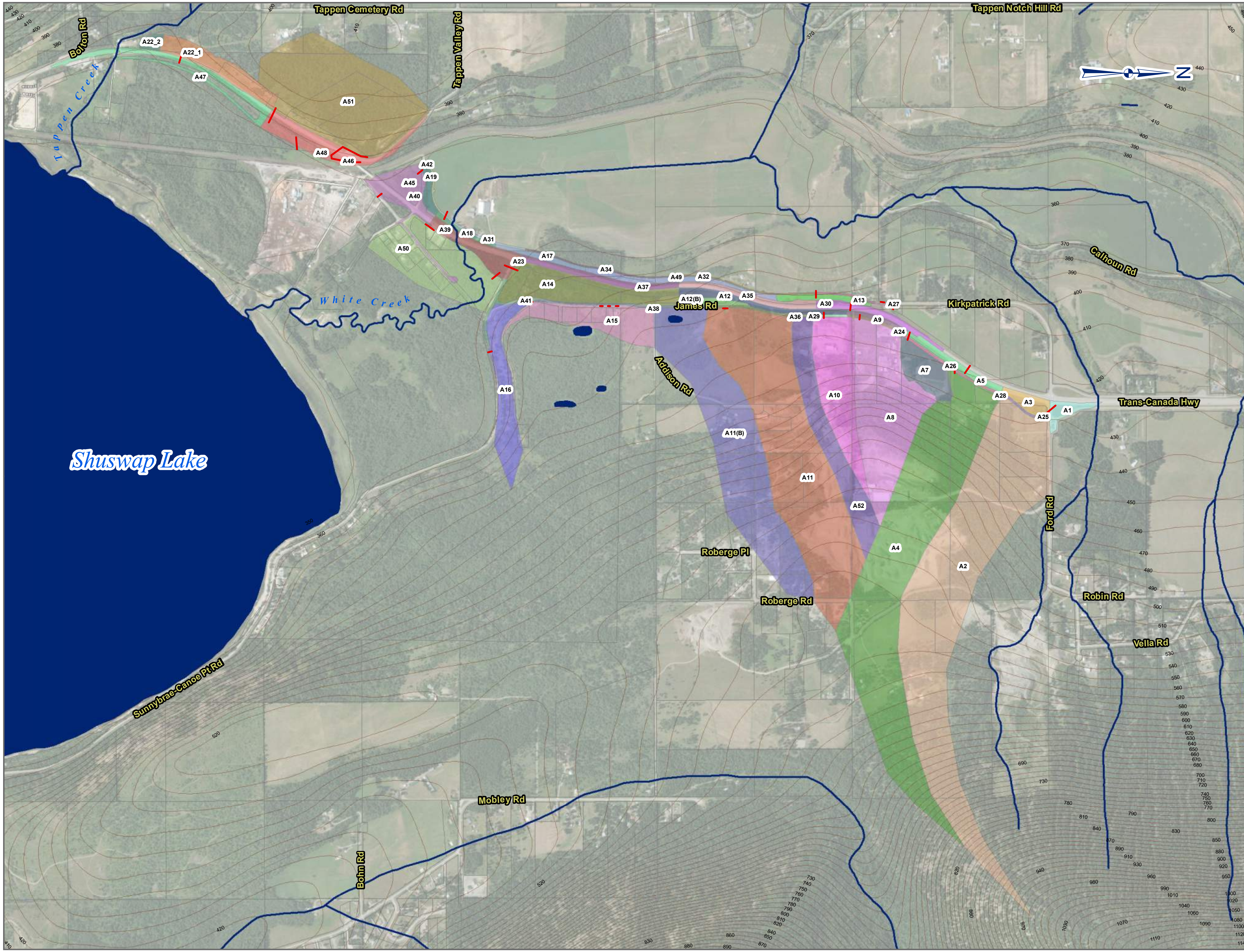
5.3. PROPOSED DRAINAGE SYSTEM

5.3.4. Drainage Subcatchments

In order to assess the hydrologic response of the project area to the design storm, the surrounding contributing areas and roads were delineated into subcatchments to determine their respective peak runoff flows. The existing LiDAR topography and proposed design surface of the roadway corridor were combined into a single surface in AutoCAD Civil 3D and the subcatchments were delineated based on proposed culvert locations and drainage patterns. Based on the latest revisions to the proposed design surface of the roadway, the subcatchments were updated. Figure 5-1 shows the previously delineated drainage subcatchments and proposed culvert locations while Figure 5-2 shows the revised drainage subcatchments and proposed culvert locations for comparison purposes.

Runoff from the hillside subcatchments to the east of Highway 1 will be captured in a network of local ditches that direct the flows to a series of cross-culverts beneath James Road, Highway 1, and Kirkpatrick Road. The hillside catchments within the project area generate significant runoff volumes due to their size and steep slopes. The runoff from these subcatchments is conveyed to the western side of Highway 1 where flows are eventually conveyed to White Creek through an existing channel and ultimately discharge into Shuswap Lake.

PCSWMM differs from the process involved in the Rational Method for determining peak runoff flows in that it does not directly apply runoff coefficients to determine peak runoff flows. PCSWMM instead uses the percent of impervious area, catchment slope, Manning's n-values, and depression storage volumes to calculate the peak runoff flow, with the runoff coefficient being back-calculated. In order to account for the slope of the hillside catchments and the impact it will have on runoff, the percentage of impervious areas was increased. Table 5-5 summarizes the subcatchment design parameters and peak runoff flows from the hydrologic model with the percentage of impervious areas increased for the projected climate change scenario.



Ministry of Transportation and Infrastructure

BRITISH COLUMBIA

Legend

- Proposed Culverts
- Streams
- Property
- Subcatchment

McElhanney

CANADA'S BEST MANAGED COMPANIES

0 125 250 500 750 (m)

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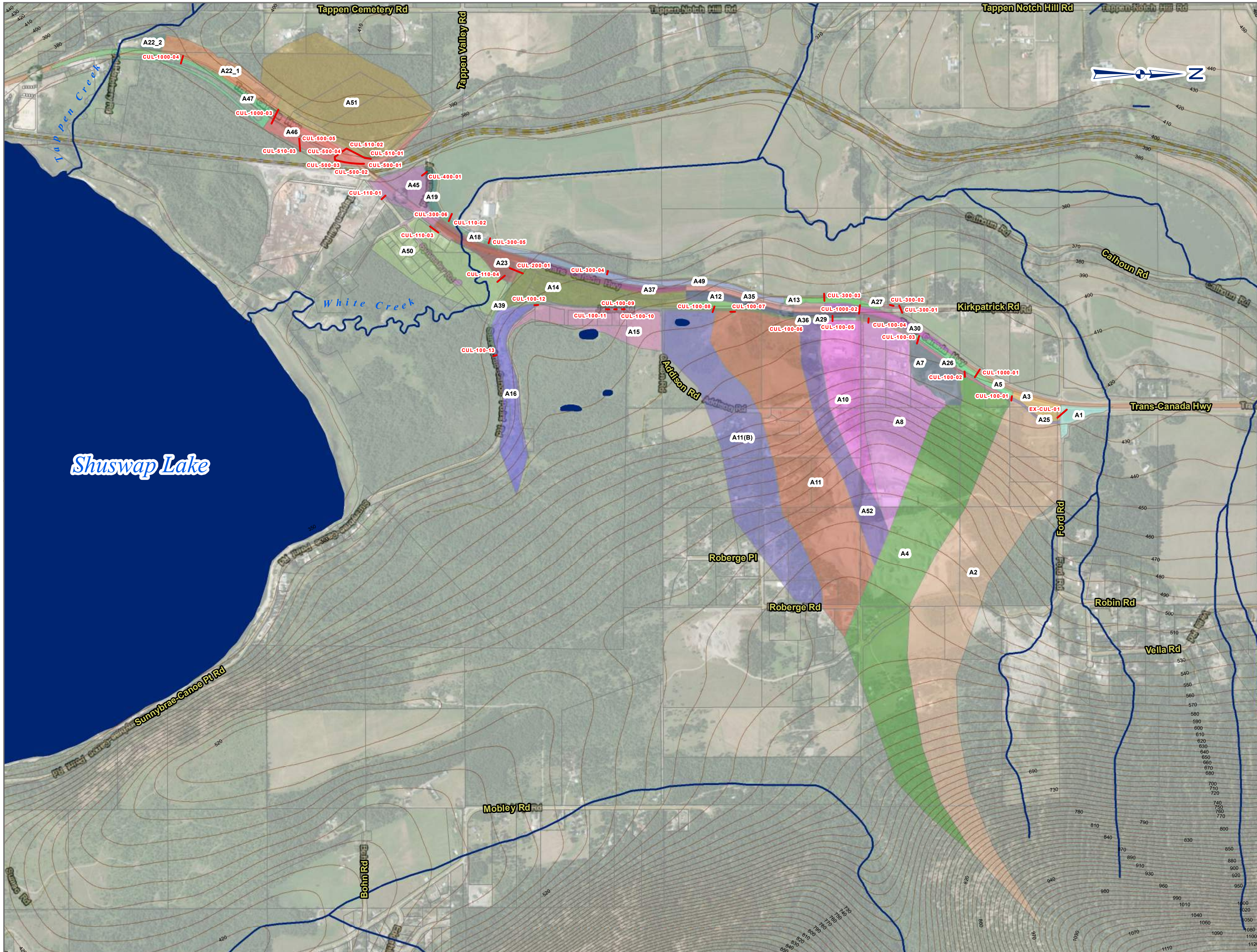
Coordinate System: NAD 1983 UTM Zone 11N

Project No.	Date
2121-00113-00	January 2020

Figure 5-1

Drainage Subcatchments & Proposed Culvert Locations



Legend

- Property
- Proposed Culverts
- Streams
- Subcatchment

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Coordinate System: NAD 1983 UTM Zone 11N

Project No.	Date
2121-00113-00	January 2020

Figure 5-2

Updated Drainage Subcatchments & Proposed Culvert Locations

British Columbia — Ministry of Transportation and Infrastructure

McElhanney

Best Managed Companies

Document Path: O:\Proj\2121-00113-00 MoTI - Hwy 1 Ford Road to Kault Hill\4.0 ENGINEERING DESIGN\4.6 Drainage\4.6.3 Sketches & Designs\50% Detailed Design\Figure\Figure 5-2 - Updated Drainage Subcatchments & Proposed Culvert Locations.mxd
Author: AOYEFIADE

Table 5-5: Subcatchment Design Parameters and PCSWMM Model Results considering Climate Change

Catchment I.D.	Area (ha)	Slope (%)	Infiltration (mm)	Peak Runoff (m^3/s)	Calculated Runoff Coefficient
A1	1.08	9.11	6.29	0.04	0.88
A2	42.84	11.00	11.55	1.42	0.80
A3	1.19	9.29	6.29	0.05	0.88
A4	38.86	24.79	20.53	1.11	0.66
A5	1.33	8.45	6.29	0.05	0.88
A7	3.19	10.21	38.50	0.05	0.39
A8	15.53	22.19	20.31	0.45	0.66
A9	0.49	12.33	6.28	0.02	0.88
A10	8.74	14.39	20.30	0.25	0.66
A11	32.63	12.62	20.53	0.92	0.66
A11(B)	23.42	15.56	20.53	0.67	0.66
A12	1.86	17.17	6.42	0.07	0.88
A12(B)	0.71	9.95	6.42	0.03	0.88
A13	0.93	18.56	6.42	0.04	0.88
A14	7.10	20.26	43.64	0.10	0.31
A15	5.85	16.51	6.42	0.22	0.87
A16	5.73	47.66	22.46	0.16	0.63
A17	0.74	13.68	6.42	0.03	0.88
A18	0.42	12.91	6.42	0.02	0.88
A19	0.71	10.39	6.42	0.03	0.88
A22_1	3.70	58.48	6.42	0.14	0.88
A22_2	0.54	38.62	6.42	0.02	0.88
A23	0.10	11.09	6.28	0.00	0.88
A24	0.24	7.33	6.28	0.01	0.88
A25	0.22	7.10	6.28	0.01	0.88
A26	0.45	8.22	6.28	0.02	0.88
A27	0.24	16.25	6.28	0.01	0.88
A29	0.17	18.29	6.28	0.01	0.88
A30	1.47	15.92	6.29	0.06	0.88
A31	0.31	7.79	6.28	0.01	0.88
A32	0.41	15.00	6.28	0.02	0.88
A34	0.46	17.08	6.28	0.02	0.88
A35	1.61	16.54	6.29	0.06	0.88
A36	0.56	13.29	6.28	0.02	0.88
A37	1.78	15.88	6.29	0.07	0.88
A38	0.59	20.44	6.28	0.02	0.88
A39	2.77	14.83	6.29	0.11	0.88
A40	3.80	14.95	6.30	0.15	0.88
A41	0.49	19.34	6.28	0.02	0.88

Catchment I.D.	Area (ha)	Slope (%)	Infiltration (mm)	Peak Runoff (m^3/s)	Calculated Runoff Coefficient
A42	0.18	3.98	6.28	0.01	0.88
A45	2.72	12.33	6.30	0.10	0.88
A46	3.88	21.99	6.29	0.15	0.88
A47	3.05	15.26	6.29	0.12	0.88
A48	1.36	13.10	6.42	0.05	0.88
A49	1.60	22.76	6.42	0.06	0.88
A50	10.56	12.67	20.53	0.30	0.66
A51	21.54	15.05	32.08	0.46	0.49
A52	8.12	11.02	20.53	0.23	0.67

5.3.5. Pavement Drainage

Pavement drains (catch-basins and embankment drains) are located along a continuous grade, or flow-by conditions or at trap-lows or based on ponding conditions. Pavement drains are designed to intercept runoff from gutters and low points and have been spaced as per Section 1000 - Supplement to TAC Geometric Design Guide (2019).

5.3.6. Culverts & Drainage Ditches

As per Section 1000 of the *Supplement to TAC Geometric Design Guide* (2019), the culverts were sized based on the 100-year design storm and the target of a HW/D less than one. The 100-year SCS Type 1A 24-hour hyetograph (including climate change) was created in PCSWMM to generate the runoff design flows from the sub-catchments. The capacity of the ditches to convey the 100-year design flows was revised along with the capacity of the culverts. As a result, several ditches and culverts were proposed to ensure continuity and sufficiently convey roadway runoff. The initial culverts are shown in Figure 5-1 while the revised culverts are displayed by Figure 5-2.

The flow within a culvert can either be inlet or outlet controlled depending on upstream conditions, culvert slope, culvert material, culvert length, culvert end treatment, and downstream conditions. Both inlet and outlet control scenarios were assessed to determine which condition governs the sizing of the culvert in question.

A minimum base width of 1.0 m, 2(H):1(V) side slopes, and Manning’s n equal to 0.034 were adopted for all ditches in the PCSWMM hydrologic model. This is a conservative simplification since it will result in higher water surface elevations in these ditches compared to ditches with shallower slopes and greater conveyance capacities. Based on the latest revisions to the 100% detailed design, all false graded ditches have similar base widths and side slopes with depths adjusted while ditches with rip rap are assigned a Manning’s n of 0.040.

Subcatchments A1-A8 all convey flows from the eastern side of Highway 1 to the infield area between Highway 1 and Kirkpatrick Road through a series of proposed ditches and cross culverts. Flows from these sub-catchments discharge through culvert *CUL-300-03* to an existing channel that conveys all runoff to White Creek, which is approximately 450 m to the west of the culvert outlet.

The outfalls for *CUL-140-04, CUL-140-05, CUL-200-02, CUL-300-05, CUL-300-06,* and *CUL-300-09,* which all discharge to White Creek, were modelled in PCSWMM as fixed boundary conditions representing the 100-year water surface elevation at the various locations in the creek. These water surface elevations were obtained from the HEC-RAS model for White Creek as discussed in Section 4.1. Using this approach enables potential tail-water conditions to be accounted for when sizing the culverts for the 100-year design storm. The inlets and outlets of each of the culverts should be protected by rip rap to ensure undermining does not occur.

Based on the latest revisions to the 100% detailed design, the culvert number, sizes, and associated design parameters were updated and are summarized in Table 5-6.

Table 5-6: Culvert Table

Culvert	Slope (%)	Diameter (mm)	Length (m)	Inlet Elevation (m)	Outlet Elevation (m)	Peak Flow (m^3/s)
CUL-1000-01	0.03	1200	43.35	412.50	411.20	0.30
CUL-1000-02	0.01	1400	40.60	405.75	405.51	0.63
CUL-1000-03	0.13	800	56.58	361.15	353.65	0.00
CUL-1000-04	0.10	800	33.76	355.92	352.41	0.02
CUL-100-01	0.01	1200	19.96	415.45	415.35	0.15
CUL-100-02	0.00	1000	22.55	413.00	412.89	0.22
CUL-100-03_1	0.01	600	21.27	409.79	409.62	0.03
CUL-100-04	0.01	1000	19.69	407.40	407.25	0.40
CUL-100-05	0.02	800	21.56	408.20	407.84	0.26
CUL-100-06	0.01	800	23.50	407.20	406.92	0.10
CUL-100-07	0.06	800	18.86	394.00	392.87	0.10
CUL-100-08	0.01	600	19.02	390.50	390.41	0.03
CUL-100-09	0.09	300	11.09	388.90	387.90	0.00
CUL-100-10	0.09	300	10.52	385.70	384.80	0.00
CUL-100-11	0.06	300	12.31	382.90	382.20	0.00
CUL-100-12	0.05	600	35.20	374.30	372.55	0.01
CUL-100-13	0.04	600	36.04	372.55	371.10	0.04
CUL-140-01	0.01	600	18.78	355.04	354.80	0.04
CUL-140-02	0.02	600	16.18	354.00	353.75	0.04
CUL-140-03	0.01	600	42.15	354.80	354.20	0.04
CUL-140-04	0.01	600	22.26	354.20	354.00	0.04
CUL-140-05	0.06	1000	35.57	351.90	349.87	0.36
CUL-140-06	0.01	600	17.71	356.00	355.80	0.04
CUL-200-01	0.01	800	31.81	352.46	352.03	0.25
CUL-200-02	0.04	800	22.53	356.85	355.93	0.09
CUL-300-01	0.03	800	52.42	403.08	401.45	0.30
CUL-300-02	0.02	800	49.80	401.45	400.34	0.31
CUL-300-03	0.02	1400	18.67	392.69	392.31	1.00
CUL-300-04	0.01	600	14.82	359.77	359.62	0.10
CUL-300-05	0.02	600	16.07	353.78	353.46	0.00
CUL-300-06	0.02	600	15.36	351.60	351.29	0.04
CUL-300-07	0.01	600	14.84	351.35	351.15	0.14
CUL-300-08	0.00	600	14.86	350.97	350.90	0.13
CUL-300-09	0.09	600	22.96	355.70	353.70	0.00
CUL-400-01	0.01	600	28.61	352.50	352.20	0.14
CUL-500-01	0.03	800	30.49	354.23	353.22	0.19
CUL-500-02_1	0.05	300	23.42	358.39	357.23	0.00
CUL-500-03	0.03	300	35.30	357.23	356.07	0.00
CUL-500-04	0.03	300	18.50	356.56	356.07	0.05

CUL-500-05	0.01	600	40.25	356.07	355.85	0.05
CUL-500-06	0.01	600	39.63	355.85	355.60	0.05
CUL-500-07	0.01	600	40.24	355.60	355.22	0.06
CUL-500-08	0.03	600	8.04	355.22	355.00	0.05
CUL-500-09	0.01	1000	24.92	351.39	351.26	0.03
CUL-510-01	0.02	1000	48.63	363.00	362.02	0.00
CUL-510-02	0.02	1000	45.73	362.00	361.08	0.00
CUL-510-03	0.04	1000	50.35	361.08	359.04	0.05
EX-CUL-01	0.00	600	50.56	417.47	417.25	0.03
Pipe-(24)	0.38	1000	49.85	373.65	355.84	1.00
Pipe-(40)	0.12	1000	22.11	355.79	353.26	1.00
Pipe-(40)-01	0.03	1000	40.47	353.02	351.89	1.00
Pipe-(48)	0.01	1000	27.36	351.84	351.57	1.00
Pipe-(52)	0.15	1000	27.30	386.03	381.91	1.00
Pipe-(53)	0.17	1000	47.21	381.79	373.72	1.00

6. CONCLUSION & RECOMMENDATIONS

Based on the hydrologic and hydraulic analysis presented in this report, the following summarizes the design for the Tappen Creek and White Creek crossings.

1. Culvert crossings at White Creek for Kirkpatrick Road, Highway 1, and Tappen Station Road should consist of 4300mm diameter Structural Plate Corrugated Steel Pipe (SPCSP) and embedded to a depth of 40% to accommodate fish passage.

    a. This culvert will be 96.0 m in length. The culvert is embedded to a depth of 1.72 m and it will be infilled with streambed material along the length of the culvert. The inlet and outlet of the culvert will have concrete headwalls. The channel bottom will be lined with 50 kg rip rap at the inlet and outlet. The culvert will be infilled with salmonid gravel, with large habitat boulders (600mm to 800mm diameter) placed along the length of the culvert channel.

    b. A new 6 m long channel will be excavated upstream or the culvert to transition the existing watercourse into the new culvert. It will have a low flow channel in the channel bottom. A class 50kg riprap with semi-angular large boulders will line the channel bottom. A 150 mm layer of salmonid gravels will be placed on top of the channel bottom material. The channel bottom material will mimic a natural channel bottom throughout.

2. A new single span, 26.6 m long, 20.4 m wide bridge will be built across Tappen Creek. This bridge will include an elevated wildlife pathway. A 110 m re-aligned channel will be constructed. The channel will incorporate rock weirs/riffles at specific locations to create a more natural pool/riffle channel morphology and improve aquatic habitat. The excavated channel bed will be lined with a mix (50/50) of 50kg riprap with semi-angular cobble and boulders. A 150 mm layer of salmonid gravels will be placed on top of the channel bottom material. The 1.5H:1.0V channel banks will be lined with 50 kg rip rap. Voids in the rip rap will be filled with gravels.

3. A design for the minor drainage system, including required ditches and culverts, is summarized in this report. Location of culvert and ditches are included in the overall design package.

7. CLOSING

We trust that this hydrotechnical design report meets the requirements for this 100% detailed design stage.

Yours truly,

McELHANNEY LTD.

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APPENDIX A
SCOUR CALCULATIONS

Appendix A: Scour Calculations

The details of the scour calculations for White Creek and Tappen Creek can be found below.

A.1 WHITE CREEK SCOUR CALCULATIONS

Scour at White Creek was assessed using both the HEC-18 methodology for contraction scour and Blench's Regime Method presented in the TAC Guide to Bridge Hydraulics.

A.1.1 HEC-18 Contraction Scour

Determination of Critical Velocity

The calculation of critical velocity is the first step in performing scour calculations. Critical velocity determines whether the contraction scour analysis will be for live-bed ($V>V_c$) or clear-water scour ($V<V_c$). Critical velocity is calculated for the channel actively transporting material upstream of the structure (Cross section 298 located 20 m upstream of the culvert).

The D_{50} material size was estimated at 0.01 m based on the gravels that will form the channel.

Equation 5.1 from HEC-18 for critical velocity is given as $V_c = K_{vy}{}^{1/6}D^{1/3}$

Where:

y = the average depth upstream of the bridge at the return period flood (m)

K_v = 6.19 for metric units

D = D_{50} (m)

V_c = the critical velocity above which the bed material of size D and smaller will be transported (m/s)

V = the velocity in the channel upstream of the structure at the return period flood (m/s)

The values used are summarized in Table 1.

Table 1 White Creek critical velocity parameters

y (m)	V (m/s)	D_{50} (m)	V_c (m/s)	Scour Type
1.56	1.25	0.01	1.44	Clear-water Scour

Clear-water Contraction Scour

The clear-water contraction scour equation given in HEC-18 is as follows:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3}\ W^2}\right]^{3/7} \qquad (6.4)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \qquad (6.5)$$

where:

- y_2 = Average equilibrium depth in the contracted section after contraction scour, ft (m)
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, ft^3/s (m^3/s)
- D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, ft (m)
- D_{50} = Median diameter of bed material, ft (m)
- W = Bottom width of the contracted section less pier widths, ft (m)
- y_o = Average existing depth in the contracted section, ft (m)
- K_u = 0.0077 English units
- K_u = 0.025 SI units

The values used to calculate the scour depth are summarized in Table 2.

Table 2 HEC-18 clear-water scour parameters for White Creek

Parameter	Value
Q	8.58 m^3/s
W	2.5 m
D_{50}	0.01 m
D_m	0.0125 m
K_u	0.025 m
y_o	2.03 m
y_2	2.12 m
y_s	0.09 m

The final clear-water contraction scour depth through the structure is estimated to be 0.1 m.

A.1.2 Blench's Regime Formula for Natural Scour

The scour analysis looked at locations upstream, downstream, and through the structure using Blench's Regime formula for natural scour. The formula is shown below. The D_{50} material size was estimated at 0.01 m based on the gravels that will form the channel.

$\mathbf{y_r=(q^2/F_{b0})^{1/3}}$

Where:

$\mathbf{y_r}$ is the "regime" or mean depth in m

A multiplier is applied to the $\mathbf{y_r}$ value. For a straight reach (through the culvert) the value is 1.25, for a moderate bend, as seen upstream and downstream of the culvert the value is 1.5.

q is the discharge intensity (**Q/W**, where **W** is the width at half depth in the channel) for a given cross section in m^2/s

$\mathbf{F_{b0}}$ is a bed material based on the median grain size in mm and is taken from a chart provided in Figure 4.24 of TAC (2001), given an estimated D_{50} of 10 mm the value is 1.

The mean (y_s) is determined by:

$\mathbf{y_s = y_o - y_r}$

Where $\mathbf{y_o}$ is the water depth from the crossing section.

The values used are listed in Table 3.

Table 3 Blench's regime formula parameters for White Creek

Location	Q (m3/s)	W (m)	q (m2/s)	q^2	F_{bo}	y_r (m)	multiplier	y_r with multipler (m)	y_o (m)	y_s (m)
XS 298 (20 m US of culvert)	14.8	12.4	1.19	1.42	1	1.13	1.5	1.69	2.43	0
XS 285 (10 m US of culvert)	14.8	11.4	1.3	1.69	1	1.19	1.5	1.79	2.43	0
US Culvert Section	14.8	4.2	3.52	12.42	1	2.32	1.25	2.89	2.65	0.24
DS Culvert Section	14.8	4.2	3.52	12.42	1	2.32	1.25	2.89	1.8	1.09
XS 184 (20 m DS of culvert)	14.8	6.3	2.35	5.52	1	1.77	1.50	2.65	1.79	0.86
XS 160 (40 m downstream of culvert)	14.8	5.8	2.55	6.51	1	1.87	1.25	2.33	1.69	0.64

The scour depth estimated through the channel is 0.9 m, and 1.1 m at the culvert outlet.

A.2 TAPPEN CREEK SCOUR CALCULATIONS

Scour at Tappen Creek was assessed using both the HEC-18 methodology for contraction scour and Blench's Regime Method presented in the TAC Guide to Bridge Hydraulics.

A.2.1 HEC-18 Contraction Scour

Determination of Critical Velocity

The calculation of critical velocity is the first step in performing scour calculations. Critical velocity determines whether the contraction scour analysis will be for live-bed ($V>V_c$) or clear-water scour ($V<V_c$). Critical velocity is calculated for the channel actively transporting material upstream of the structure (Cross section 236 located 5 m upstream of the bridge opening).

The D_{50} material size was estimated at 0.01 m based on the gravels that will form the channel.

Equation 5.1 from HEC-18 for critical velocity is given as $V_c = K_{vy}{}^{1/6}D^{1/3}$

Where:

> y = the average depth upstream of the bridge at the return period flood (m)
>
> K_v = 6.19 for metric units
>
> $D = D_{50}$ (m)
>
> V_c = the critical velocity above which the bed material of size D and smaller will be transported (m/s)
>
> V = the velocity in the channel upstream of the structure at the return period flood (m/s)

The values used are summarized in Table 4.

Table 4 Tappen Creek critical velocity parameters

y (m)	V (m/s)	D_{50} (m)	V_c (m/s)	Scour Type
2.43	1.48	0.01	1.42	Live-bed Scour

Live-bed Contraction Scour

The live-bed contraction scour equation given in HEC-18 is as follows:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \quad (5.2)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \quad (5.3)$$

where:

y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section, m (ft)
y_o = Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)

W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)
ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 5.8)
For fall velocity in English units (ft/s) multiply ω in m/s by 3.28
g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

The values used to calculate the scour depth are summarized in Table 5.

Table 5 HEC-18 clear-water scour parameters for Tappen Creek

Parameter	Value
Q_1	8.58 m^3/s
Q_2	8.58 m^3/s
W_1	1.5 m
W_2	1.5 m

Parameter	Value
y_1	1.48 m
y_2	1.48 m
y_o	1.43 m
y_s	0.05 m

The final scour depth is estimated to be 0.1 m.

A.2.2 Blench's Regime Formula for Natural Scour

The scour analysis looked at scour through the structure using Blench's Regime formula for natural scour. The formula is shown below. The D_{50} material size was estimated at 0.01 m based on the gravels that will form the channel.

$$y_r=(q^2/F_{b0})^{1/3}$$

Where:

y_r is the "regime" or mean depth in m

A multiplier is applied to the y_r value. For a straight reach the value is 1.25.

q is the discharge intensity (**Q/W**, where **W** is the width at half depth in the channel) for a given cross section in m^2/s

F_{b0} is a bed material based on the median grain size in mm and is taken from a chart provided in Figure 4.24 of TAC (2001), given an estimated D_{50} of 10 mm the value is 1.

The mean (y_s) is determined by:

$$y_s = y_o - y_r$$

Where y_o is the water depth from the crossing section.

The values used are listed in Table 6.

Table 6 Blench's regime formula parameters for Tappen Creek

Location	Q (m3/s)	W (m)	q (m2/s)	q^2	F_{bo}	y_r (m)	multiplier	y_r with multipler (m)	y_o (m)	y_s (m)
XS 206 (mid span of bridge)	8.58	4	2.15	4.60	1	1.66	1.25	2.08	1.2	0.88

The natural scour depth estimated through the bridge is 0.9 m.