



BEAVER CREEK FLOOD RISK **ASSESSMENT**

November 20, 2025

URBAN
S Y S T E M S

204 - 625 Front Street, Nelson, BC V1L 4B6 | T: 250.352.9774

CONTACT: Jan Korinek
E: jkorinek@urbansystems.ca

PREPARED FOR:

The Village of Fruitvale
1947 Beaver Street
Fruitvale, BC V0G 1L0

204 - 625 Front Street, Nelson, BC V1L 4B6 | T: 250.352.9774

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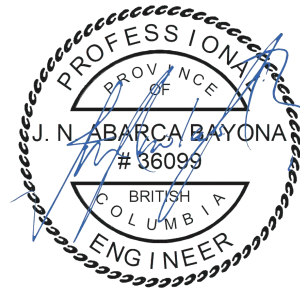
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Prepared by:

Breanne Roszko

Breanne Roszko, E.I. T
Water Resources E.I.T.

Reviewed by:



2025-11-20

Nicolas Abarca, M.Sc., P.Eng.
Water Resources Engineer

Permit to Practice:

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1.0 BACKGROUND

The Village of Fruitvale (The Village, Fruitvale) is located along the downstream reach of Beaver Creek, approximately 5 km upstream of its confluence with the Columbia River. Beaver Creek poses a routine flood risk to the community during spring freshet. Many of the residents along the creek banks, as well as community infrastructure such as parks and the wastewater lagoons, are often exposed to the hazards of the flooding creek. The most recent significant high-water event occurred in 2023, which resulted in damage to critical infrastructure and private property along the Creek. The event required temporary flood protection measures to be implemented, and evacuation alerts were also issued.

There is some existing flood protection infrastructure along Beaver Creek. However, this infrastructure was designed based on flood mapping conducted in 1989 and may be outdated and no longer provide an adequate level of protection. Since 1990, the Creek has been experiencing increased flows due to climate change affecting the Kootenay Boundary Area. Additionally, the Village has experienced population growth from historical populations, which has resulted in new infrastructure that could be affected by floodwaters.

To better understand the flood risk that Beaver Creek poses to the community and provide suitable recommendations for flood mitigation, planning, and capital infrastructure upgrades such as dike improvements, a hydrologic and hydraulic analysis of Beaver Creek and its tributaries along the Village was completed. The extents of the study area are shown in **Figure 1**.

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- DataBC (PMBC, Wetlands, Watercourses, Regional and Municipal District, Roads, Railway, Wetland), Esri (2024 Imagery), Harrier Aerial Surveys, Urban Systems Ltd.

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Study Area

Beaver Creek

Watercourse

Wetlands

Highway 3B

Railway

Building

Parcel

Municipal Boundary

Regional District Boundary

General Project Location in the Region

The Corporation of the Village of Fruitvale

Flood Risk Assessment

Study Area for Beaver Creek Flood Assessment

Figure 1-1

\\usl.urban-systems.com\projects\Projects_NEL\1011\0058\01\D-Design\GIS\Projects\Pro_Projects\1011.0058.01_FruitvaleFloodAsses.aprx\Figure Study Area for Beaver Creek Flood Assessment

1.1 SITE VISIT

A site visit to Beaver Creek and the Village of Fruitvale was conducted on November 13th, 2024. The creek levels were relatively low at the time of the inspection, which allowed for channel and bed material to be observed. In addition, site-specific conditions for the overbanks were investigated in order to establish roughness coefficients for the different land use types (further discussed in **Section 5.1.1 Manning's 'n' Coefficients**) and determine where water might flow under possible flooding conditions in the Creek.

There are eight roadway and pedestrian crossings along Beaver Creek within the study area; these were inspected during the site visit, and information such as configuration of the abutments, piers, low chord of bridges, and culvert inverts and obverts was obtained. **Photos 1-1** through **1-7** were taken during the site visit and reflect the conditions observed at that time.



Photo 1-1: Highway 3B/ Mainstreet Bridge Crossing Beaver Creek, Facing North (Upstream)



Photo 1-2: Beaver Creek Near Property 1996 Main Street, Facing Upstream



Photo 1-3: Beaver Creek Near Creekside Park, Facing Upstream



Photo 1-4: Beaver Creek Near Laurier Ave Dead-End, Facing Upstream



Photo 1-5: Beaver Creek With Beaver Dam, Near Aspen Drive Dead End



Photo 1-6: Beaver Creek Near Camp Tweedsmuir, Facing Upstream



Photo 1-7: Beaver Creek at Downstream End of Study Area, Facing Downstream

2.0 CHARACTERISTICS OF BEAVER CREEK AND TRIBUTARIES

2.1 BEAVER CREEK

Beaver Creek originates in the forested hills of the Selkirk Mountains north of Fruitvale at the peak of Mount Kelly (elevation 1954 m). The creek is approximately 25km long, and the land use of the watershed is mainly treed and vegetated, covered with some cleared forestry cut block areas. The urbanized land and areas suitable for future development are located within the valley bases at the downstream portion of the watershed. There are agricultural areas throughout the valley bottom.

The Beaver Creek watershed is part of the Lower Columbian Basin hydrologic zone. There are no sizeable lakes within the watershed; however, there are many named and unnamed creeks upstream of the study area, including Archibald Creek, Marsh Creek, Tracey Creek, Hudu Creek, Beaverville Creek, Benton Creek, Quarry Creek, Hammond Creek, and Bell Creek. At the upstream end of the study area, the watershed is approximately 186 km². Beaver Creek flows west, then southwest, before entering the study area.

After the study area, the creek continues to flow southwest for approximately 6km before discharging into the Columbia River (elevation 412m). Downstream of the study area, forested areas, and cleared farmland and forestry blocks become the predominant land use within the watershed. Fruitvale is the main populated center within the watershed, with the smaller communities of Montrose and Beaver Falls being further downstream. There are rural residential and agricultural properties scattered throughout the valley bottom. The Beaver Creek watershed and the watersheds of the tributaries are presented in **Figure 2-1**.

2.2 TRIBUTARIES

Within the study area, there are four (4) major creeks that flow into the main Beaver Creek channel. They are Unnamed Creek, Fruitvale Creek, Wyngill Creek, and Kelly Creek; further information on them is presented below. The watershed delineation for each tributary is shown in **Figure 2-1**.

2.2.1 UNNAMED CREEK

Unnamed Creek flows west to east, originating in forested areas at an elevation of 1007m, and discharging into Beaver Creek at an elevation of 602m. The creek has a length of 2.35km and an average slope of 17% for the flow path. Unnamed Creek crosses under Caughlin Road before discharging into Beaver Creek through a confined channel that opens into a small alluvial fan approximately 60m in width at the mouth. The watershed for Unnamed Creek has an area of 2.25km², and 7.4% of the catchment has disturbed land. Aside from the mouth of the creek within Fruitvale, it does not pass through any developed areas.

2.2.2 FRUITVALE CREEK

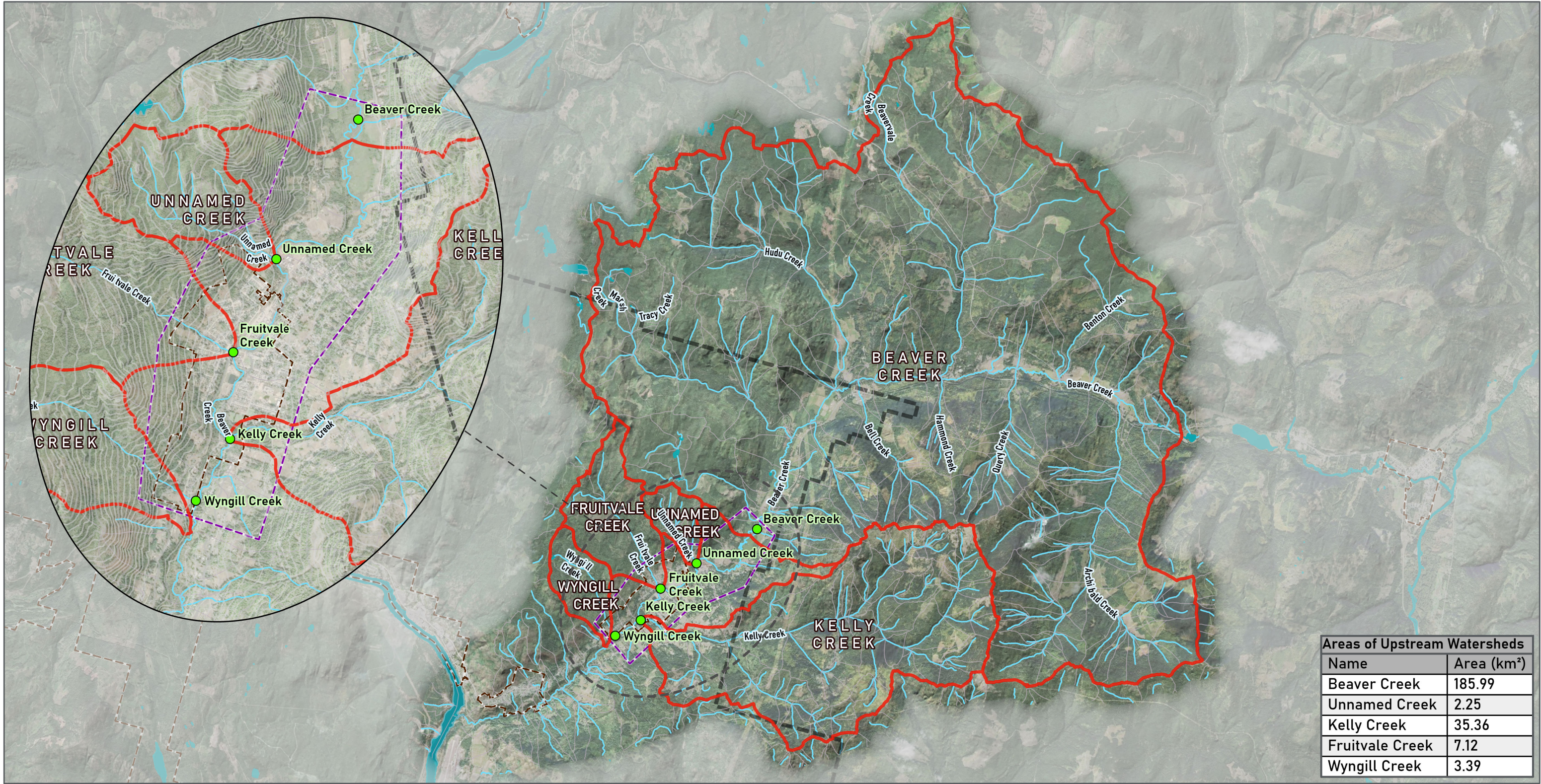
Fruitvale Creek also flows west to east, with a headwater elevation of 1163m and a discharge elevation of 594m. Fruitvale Creek has a length of 4.49km and flows through forested, undeveloped area; the watershed has a size of 7.12km². The average slope of the creek is 13% throughout the catchment. The disturbed land area within the catchment is 18.5%. It flows through the residential area north of Highway 3B, crossing four roadways and the railway before discharging into Beaver Creek via a culvert.

2.2.3 KELLY CREEK

Kelly Creek is the tributary with the largest catchment, which is 35.35 km²; it is also the only tributary that flows from east to west. The creek has a length of 12.18km and an average slope of 10%. The watershed originates near Mount Kelly and has a maximum elevation of 1850. It discharges into Beaver Creek through a constrained alluvial fan that is approximately 60m in width at the mouth. The lowest elevation of the watershed is 578m. The land use of the watershed is largely undisturbed, with some forestry cut block areas throughout. 19.7% of the land use within the watershed has been disturbed. Kelly Creek has five roadway crossings within the Fruitvale townsite.

2.2.4 WYNGILL CREEK

Wygill Creek flows from west to east and has a maximum elevation of 1060m. There is a lake ~5270 m² in area at the headwaters of the Creek. The watershed of Wyngill Creek is 3.38km² and is composed of 93% undisturbed land. Wyngill crosses Highway 3B, a private roadway, and the railway before discharging into Beaver Creek through a constrained channel and culvert crossing. The lowest elevation is 563m. Wyngill Creek is 3.39km in length and has an average slope of 15%.



Areas of Upstream Watersheds	
Name	Area (km²)
Beaver Creek	185.99
Unnamed Creek	2.25
Kelly Creek	35.36
Fruitvale Creek	7.12
Wyngill Creek	3.39

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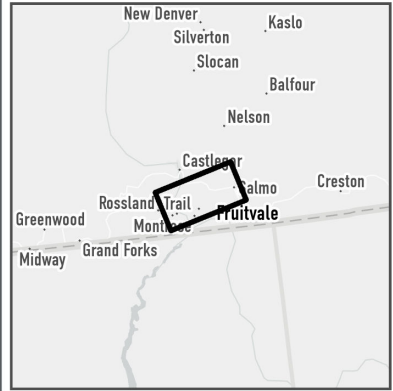
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Data Sources:
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- Legend
- Modelled Inflow Location
 - ▭ Upstream Watershed Area
 - ▭ Study Area
 - Watercourse
 - ▭ Watershed
 - ▭ Municipal Boundary
 - ▭ Regional District Boundary



The Corporation of the
Village of Fruitvale

Flood Risk Assessment
Figure 2-1
Beaver Creek and
Tributaries Watershed

3.0 DATA COLLECTION AND PROCESSING

3.1 LIDAR DATA

LiDAR data was obtained specifically for the project on October 25th, 2024. The data was corrected to NRCAN PPP and collected in NAD 83 (CSRS) / UTM Zone 11N and has an accuracy of 0.5m; the vertical datum was CGVD 2013. A bare earth file was used for the hydraulic analysis, which strips out trees and other non-ground-level features (including buildings). The LiDAR was obtained during a period of relatively low flow, which allowed for channel definition to be captured directly within the surface.

3.2 BATHYMETRIC SURVEY

Bathymetric survey was captured for the section of the study area from the Highway 3B roadway crossing to the Columbian Gardens crossing; since this area was identified during the site visit as having deeper standing water due to beaver dams. The bathymetric survey was used to compare channel elevations to the LiDAR surface. The depth of water during the LiDAR flight was not as deep as expected; since the difference was minimal, no channel was burned into the surface. The LiDAR file was the sole source of data for the base surface used within the hydraulic modelling.

3.3 TOPOGRAPHICAL SURVEY

In addition to the bathymetric survey, topographic survey was conducted at all road and pedestrian crossings. The low chord elevation of bridges, the positioning of abutments and piers, the elevations of the deck centerlines, and culvert invert and obvert elevations were obtained. This information was used to represent the crossings within the hydraulic model.

4.0 HYDROLOGIC ANALYSIS

4.1 REGIONAL ANALYSIS OF BEAVER CREEK

A Water Survey of Canada (WSC) hydrometric station exists on Beaver Creek (Station 08NE042 Beaver Creek at Fruitvale); however, the station only includes seasonal records from June to September of 1930. The National Research Council Canada recommends that interpolation of return events be no larger than four times the length of the available years of record. Flood Construction Levels in British Columbia are typically determined for a flooding event that has a 0.5% probability of occurring in any given year (200-year event or Q200). Based on this, a station with ~50 years of records would be required for proper estimation of design flows based on Frequency Analyses.

Given the lack of available historical hydrometric data for Beaver Creek, a regional analysis was conducted to establish design flows based on historical records from proxy hydrometric stations. This is a technique for estimating design flows in catchments where streamflow data is either inadequate or absent. Streamflow data is considered inadequate if its record length is less than one quarter (Watt et al., 1989) of the desired return period, or if the records are incomplete, sporadic, or inaccurate. Additionally, streamflow data may be deemed unsatisfactory if the river has been dammed or if there are extensive data gaps during periods when the river has changed.

Regional analysis supplements hydrologic information by utilizing data from similar gauged catchments. It assumes similarity based on measurable catchment parameters and confirms this similarity through statistical methods. A suitable station would mimic Beaver Creek, and it would follow similar rainfall and runoff patterns. This is not necessarily a watershed that is geographically close, but that has similar hydrologic characteristics. A pooling group was determined for Beaver Creek based on the following criteria:

- Located within the central region of British Columbia, west of the mountainous area of the Rockies and within the south-central region (south of Kamloops);
- At least 50 years of available records;
- A drainage area less than 300 km².

A total of 23 stations met the above criteria, which were further filtered based on a qualitative analysis of their respective catchments. Open data sources, aerial imagery, and contour information were used to determine if the watershed was glacier fed; these stations were removed as Beaver Creek originates in the forested hills of the Selkirk Mountains rather than the Rocky Mountain glaciers. Any hydrometric stations on regulated waterbodies were also removed, as Beaver Creek is a natural, unregulated stream. Additionally, any catchments that were less than 30 km² were removed. The upstream watershed of Beaver Creek has an area of 186 km² and very small catchments (less than 20% of the source) will likely have more rapid responses to rainfall and snowmelt. The pooling group selected for analysis included 9 hydrometric stations, summarized in **Table 4-1**.

Table 4-1: Stations within Pooling Group of Beaver Creek Regional Analysis

Station Number	Station Name	Drainage Area	Years of Record	Record length	Slope
08LC040	VANCE CREEK BELOW DEAFIES CREEK	71	1970-2022	53	7.2%
08NE077	BARNES CREEK NEAR NEEDLES	204	1950-200	71	3.8%
08NE087	DEER CREEK AT DEER PARK	82	1958-2023	66	8.8%

Station Number	Station Name	Drainage Area	Years of Record	Record length	Slope
08NE114	HIDDEN CREEK NEAR THE MOUTH	57	1973-2022	50	6.5%
08NG076	MATHER CREEK BELOW HOULE CREEK	135	1972-2022	51	0.9%
08NH016	DUCK CREEK NEAR WYNNDEL	57	1921-2023	59	9.1%
08NM142	COLDSTREAM CREEK ABOVE MUNICIPAL INTAKE	61	1967-2020	54	6.0%
08NM171	VASEUX CREEK ABOVE SOLCO CREEK	117	1970-2022	53	4.3%
08NM173	GREATA CREEK NEAR THE MOUTH	41	1970-20	53	2.6%

These nine stations were further reviewed to ensure each year of record contained a complete set of data (greater than 300 data points). They were also analyzed to confirm the watersheds were freshet driven and peaked in late spring/early summer rather than late summer, as would be expected with rainfall driven watersheds. To confirm the driving mechanism of the watershed, the annual peaks were graphed based on the day they occurred, regardless of the specific year. For example, for station 08LC040, all the peaks occurred before June 15th (**Figure 4-1**), a representative date selected to possibly divide freshet and rainfall flows. However, for Station 08NG076, some of the peaks occurred in late June (**Figure 4-2**). On further review of the specific historical events, they were shown to be freshet driven as they began to rise in spring with a limb that lasted for several weeks. Rainfall driven events are shorter and peakier. All nine stations were determined to be freshet driven and included for further analysis.

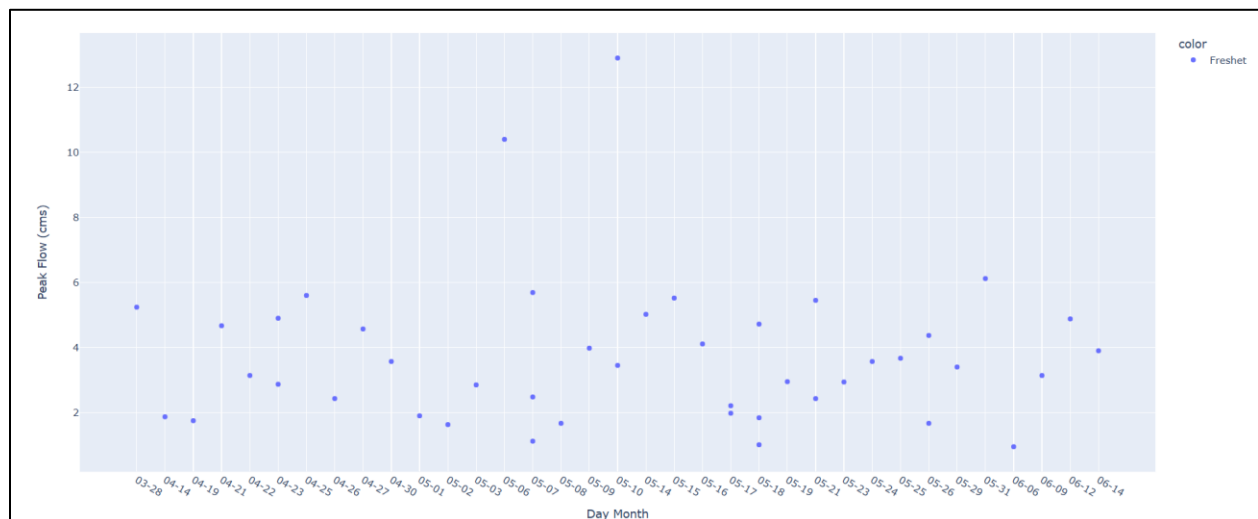


Figure 4-1: Annual Peaks of Station 08LC040 to Confirm Freshet Floods

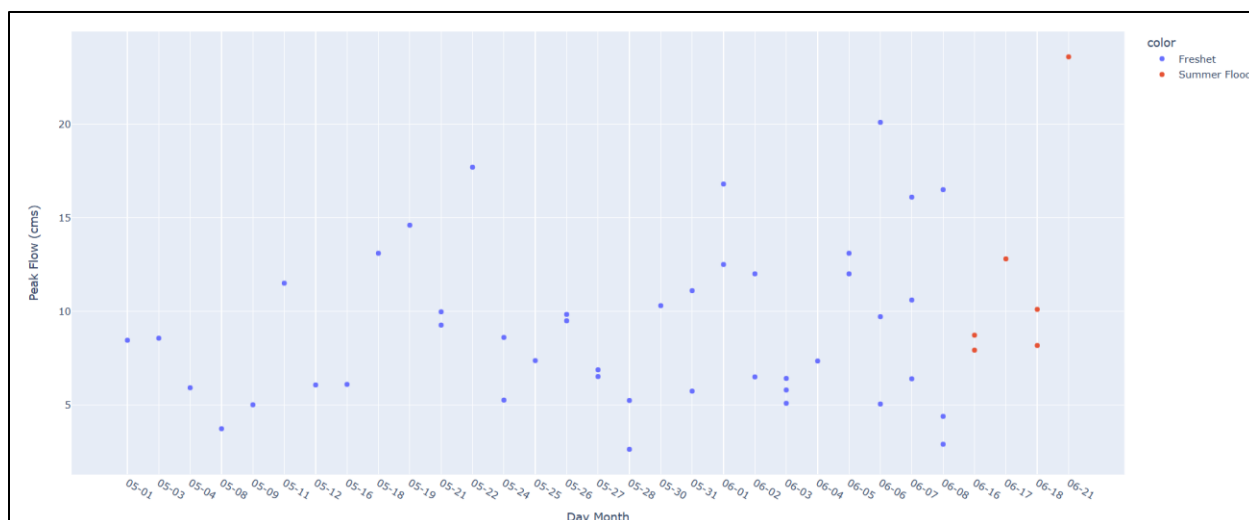


Figure 4-2: Annual Peaks of Station 08NG076 to Confirm Freshet Floods

4.2 FREQUENCY ANALYSIS OF BEAVER CREEK

Following the selection of the pooling group, a frequency analysis of peak instantaneous flow records was conducted for Water Survey of Canada hydrometric stations within the pooling group. As mentioned, these stations had already undergone a quality review for completeness of record and to ensure likely peaks were captured. Station 08NH016 had 2 years of data removed due to an incomplete record. Otherwise, all other stations had adequate datasets.

There were instances where only the annual maximum flow was recorded and not peak instantaneous flow. An average ratio of peak instantaneous flows/annual maximum flows was calculated based on the years for which these pairs of values were available; this ratio was used to extrapolate missing peak instantaneous flows from available annual maximum flows. The maximum peak instantaneous flow for each station is provided in **Table 4-2**.

Table 4-2: Maximum Peak Instantaneous Flows for Stations Within the Frequency Analysis Pooling Group

Station	Maximum Peak Instantaneous Flow (cms)	Date of Record
08LC040	19.9	2018-05-10
08NE077	67.7	2018-05-16
08NE087	21.5	2006-05-20
08NE114	39.6	1980-04-29
08NG076	28.9	2013-06-21
08NH016	14.8	1948-05-27
08NM142	8.98	1996-05-31
08NM171	27.7	1998-05-27
08NM173	2.53	1997-05-16

The frequency analysis on each station was completed using HyFRAN, a statistical analysis software that evaluates the data against a range of distributions to find the best fit. Some of the selected distributions for analysis of the data set included:

- Inverse Gamma;
- Gumbel;
- Lognormal;
- GEV;
- Generalized Gamma;
- Gamma;
- Log Pearson Type 3;
- 3- Parameter Lognormal;
- Pearson type 3; and
- Normal.

For three of the stations (08NE087, 08NG076, and 08NH016), lognormal was the distribution that provided the best fit, and it was within the top three distributions of an additional 4 stations (08LC040, 08NE077, 08NM142, and 08NM173). Results of the statistical analysis for the nine stations within the pooling group are presented in **Appendix A**. A summary of the peak discharges for each station, based on the individual statistical best fit, is provided in **Table 4-3**.

Table 4-3 - Frequency Analysis Results of Peak Discharge in m³/s.

Station	Best Fit Distribution	Q2	Q3	Q5	Q10	Q20	Q50	Q100	Q200
08LC040	Inverse Gamma	3.8	4.9	6.3	8.3	10.8	14.7	18.4	22.9
08NE077	Gumbel	39.1	43.7	48.7	55.1	61.2	69.0	74.9	80.8
08NE087	Lognormal	9.2	10.6	12.2	14.2	16.0	18.4	20.2	21.9
08NE114	Inverse Gamma	15.3	17.7	20.5	24.2	27.8	32.9	36.8	41.0
08NG076	Lognormal	9.3	11.5	14.0	17.3	20.7	25.2	28.7	32.4
08NH016	Lognormal	5.2	6.3	7.6	9.2	10.8	13.0	14.6	16.3
08NM142	Exponential	1.7	2.4	3.3	4.5	5.7	7.2	8.4	9.6
08NM171	Weibull	13.8	16.8	19.8	23.1	25.9	29.1	31.2	33.2
08NM173	Exponential	0.5	0.8	1.1	1.5	2.0	2.6	3.0	3.5

4.3 HOMOGENEITY TEST

The likelihood that these stations represent similar hydrologic regimes (homogeneity) was tested using an index flood method developed by Tate Dalrymple at the U.S. Geological Survey¹, modified to use Q25 (25-year design flow) from each station as well as the Q10 (10-year design flow) values as described in the paper. **Figures 4-3** and **4-4** show the results of the test; according to the method, values plotted within the tureen-shaped region of the graph exhibit homogeneity. This homogeneity test suggests that six out of the nine stations exhibit good homogeneity. Stations 08NE077 (Barnes Creek near Needles), 08NE087 (Deer Creek at Deer Park), and 08NE114 (Hidden Creek near the Mouth) were not homogeneous and thus removed from the pooling group.

¹ Dalrymple, T. (1960). Flood-Frequency Analysis. U.S. Geological Survey Water Supply Paper 1543-A.

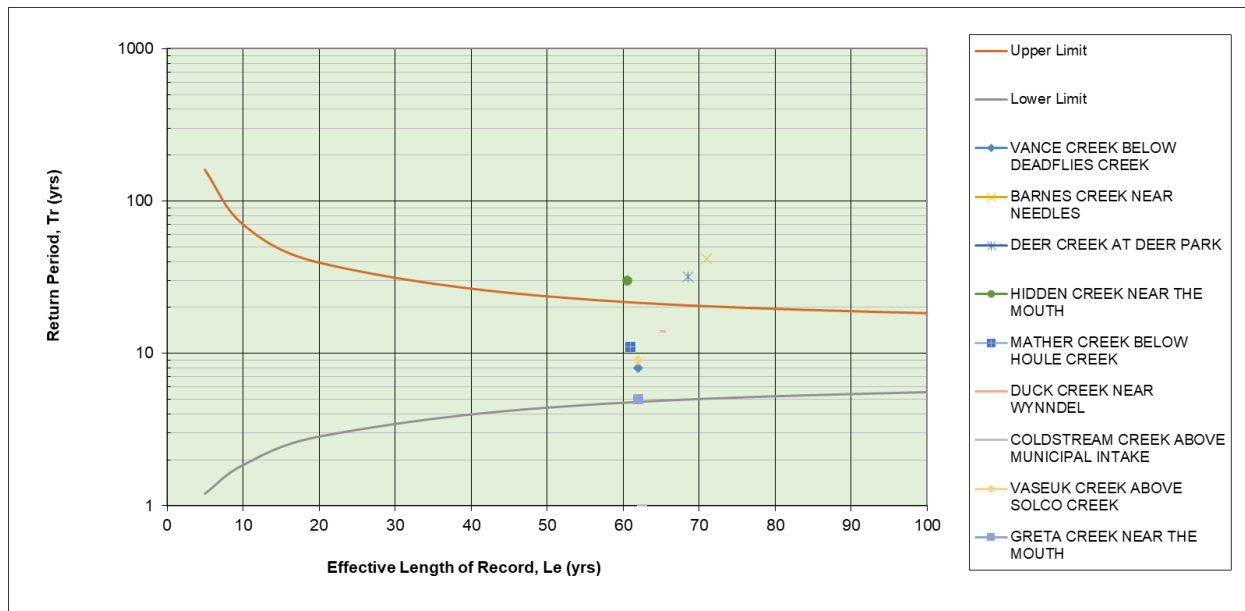


Figure 4-3: Q10 Homogeneity Test Results (Dalrymple, 1960)

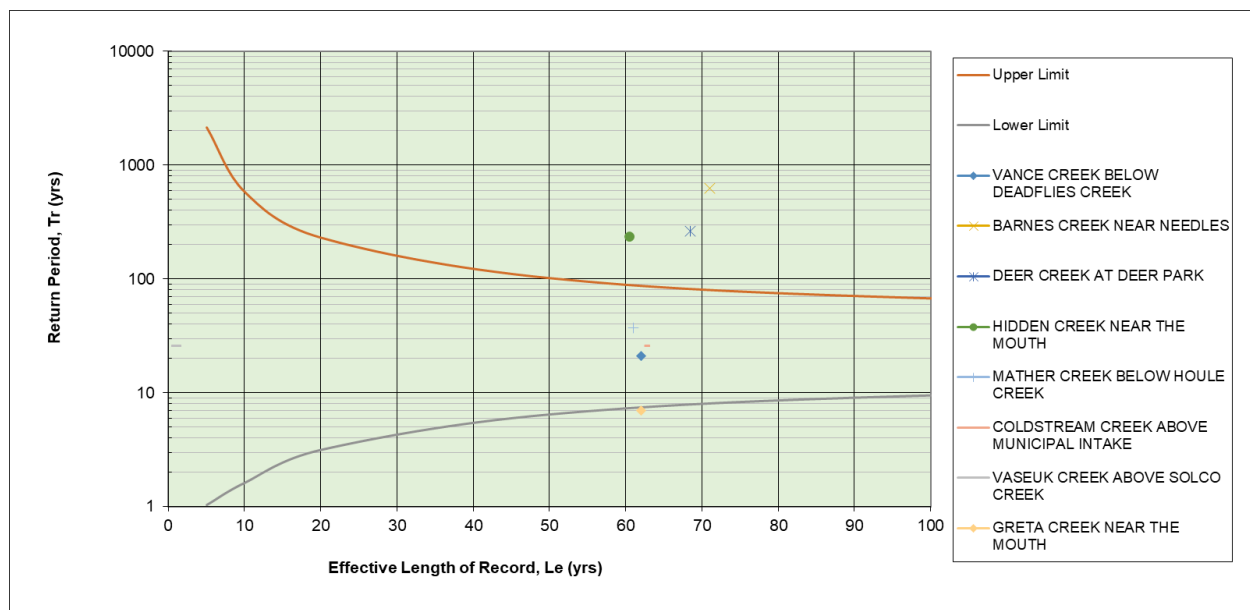


Figure 4-4: Q25 Homogeneity Test Results (Dalrymple, 1960)

4.4 DEVELOPMENT OF DISCHARGE-AREA RELATIONSHIPS

Once statistical analysis had been applied to the individual stations, plots of discharge (in m³/s) versus catchment area (in km²) for the 200- and 20-year events were made, and curves were fit to each data series. The form of the equation used for estimating daily peak flows for any return period is:

$$\text{Daily } Q_{pk} = aA^b$$

Coefficients for the 5% and 0.5% annual exceedance probabilities (20-year and 200-year events, respectively) using this equation are shown in **Table 4-4** below; plots are shown on **Figure 4-5**.

Table 4-4 - Coefficients for Estimating Design Discharges based on Basin Area

Annual Exceedance Probability (Frequency)	Coefficient 'a'	Coefficient 'b'
5% (20-year)	0.0027	1.9001
0.5% (200-year)	0.0114	1.6899

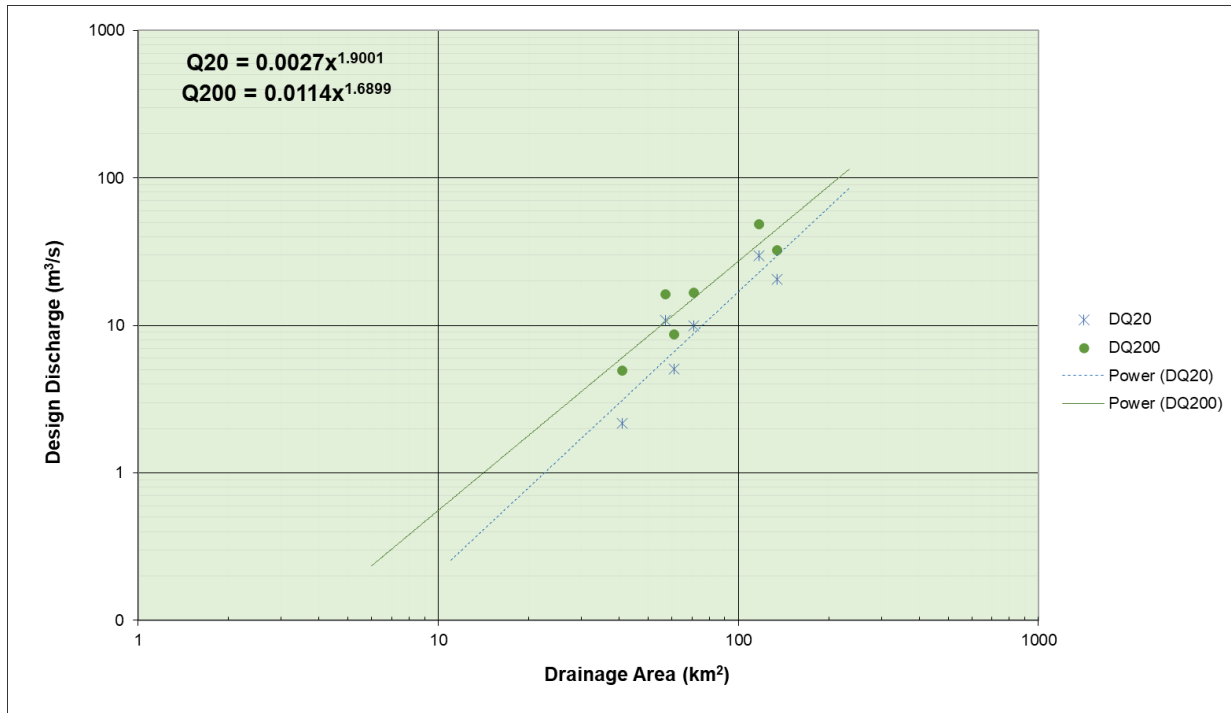


Figure 4-5 - Design Discharge-Area Relationships

The relationship was used to estimate the design discharge (not accounting for the possible effects of climate change) for Beaver Creek and its tributaries based on their respective catchment area. The resulting design discharges for the 200- and 20-year events are summarized in **Table 4-5**.

Table 4-5: Design Inflows for Beaver Creek and its Tributaries (not including climate change)

Watercourse	Catchment Area (km²)	Q200 (m³/s)	Q20 (m³/s)
Beaver Creek	186	78.00	55.41
Unnamed Creek	2.25	0.045	0.013
Kelly Creek	35.36	4.72	2.36
Fruitvale Creek	7.12	0.315	0.113
Wynhill Creek	3.39	0.090	0.027

4.5 CLIMATE CHANGE ANALYSIS

A climate change analysis was completed for the study area using publicly available open-source data such as the *Computerized Tool for the Development of the Intensity-Duration-Frequency Curves under Climate Change* (IDF_CC), the *Plan2Adapt Tool* from the Pacific Climate Impacts Consortium (PCIC), and the *Climate Atlas of Canada Map*.

The *IDF_CC Tool* provides historical data from gauged locations as well as future predictions under climate change scenarios. The *Warfield RCS Station* is located on Stoney Creek just upstream of the confluence with the Columbia River. The station is south of Rivervale, north of Warfield, and west of both Fruitvale and Trail. This gauged station collected data from 2009-2021. The tool provides predictions up to a 100-year return period for storms of different durations, a variety of climate models (emission scenarios) for a set time period, from current day up to 2100, that can be used for returning predictions.

The *Plan2Adapt Tool* is based on specific regions and provides projected future climate conditions; the study area is located within the Kootenay Boundary Region. The historical data period is from 1981-2010, and the projected effects of climate change are based on a high emissions scenario, which represents the most severe impacts (considered the business-as-usual scenario) (SSP5.85). An ensemble of 12 different models that have been downscaled over Canada (a subset of CanDCS-M6) were used to determine the projected values.

The *Climate Atlas of Canada Map* is an interactive tool that is based on the BCCAQv2 climate model data. The nearest applicable station is for Trail. Historical baseline data is from 1976 to 2005, and projections are available for both a near (2021-2050) and far (2051-2080) timeline. The map provides projections for a range of parameters, such as precipitation (maximum 1-day precipitation, annual precipitation), and temperature (mean annual temperature, maximum annual temperature, minimum annual temperature).

To compare data between the *IDF_CC Tool* and the *Plan2Adapt Tool*, the same scenarios and datasets were used in the analysis. For both the GEV and Gumbel distributions, the *IDF_CC Tool* showed a strong correlation between increases in precipitation as the return period increased for longer duration storms (**Figures 4-6** and **4-7**). For the 100-year return period for the GEV and Gumbel distributions respectively of a 24-hour duration storm, a 19.3% and a 17.4% increase in total precipitation was predicted.

Storm Duration	Return Period (Years)						
	2	5	10	20	25	50	100
5 min	15.7%	15.8%	16.1%	16.8%	17.5%	18.2%	18.2%
10 min	15.6%	15.9%	16.0%	16.9%	17.5%	18.6%	18.0%
15 min	15.5%	15.9%	15.6%	16.0%	17.0%	18.2%	20.2%
30 min	15.3%	15.9%	15.5%	16.1%	17.0%	18.2%	20.2%
1 h	15.5%	15.9%	15.5%	17.0%	16.9%	18.8%	19.7%
2 h	15.4%	15.9%	15.6%	16.5%	17.2%	17.5%	19.0%
6 h	15.4%	16.1%	15.8%	16.5%	17.1%	17.5%	19.2%
12 h	15.5%	16.0%	15.9%	17.3%	17.6%	18.5%	19.4%
24 h	15.4%	16.0%	15.6%	16.6%	17.2%	17.5%	19.3%

Figure 4-6: Percent Change in Total Precipitation Projections from Historical to Scenario SSP5.85 for Warfield RCS Station for GEV Distribution

Storm Duration	Return Period (Years)						
	2	5	10	20	25	50	100
5 min	15.6%	15.5%	15.7%	17.0%	17.3%	17.5%	17.5%
10 min	15.5%	15.5%	15.9%	17.1%	17.4%	17.5%	17.3%
15 min	15.6%	15.3%	15.9%	17.0%	17.3%	17.6%	17.4%
30 min	15.7%	15.4%	15.8%	17.0%	17.3%	17.6%	17.4%
1 h	15.8%	15.4%	15.9%	17.0%	17.2%	17.6%	17.5%
2 h	15.6%	15.4%	15.9%	17.0%	17.3%	17.6%	17.4%
6 h	15.6%	15.3%	15.9%	17.1%	17.5%	17.5%	17.4%
12 h	15.6%	15.4%	15.9%	17.1%	17.4%	17.5%	17.4%
24 h	15.6%	15.4%	15.9%	17.1%	17.4%	17.5%	17.4%

Figure 4-7: Percent Change in Total Precipitation Projections from Historical to Scenario SSP5.85 for Warfield RCS Station for Gumbel Distribution

The *Plan2Adapt Tool* for 2071-2100 predictions indicated that summer rainfall may decrease up to 37.0% while winter precipitation may increase up to 34.0% for the 90th percentile. It also predicted that temperatures in the spring may increase, up to 5.9 °C. Taken together, these indicate that freshet floods will occur sooner, rise and fall faster, and have larger peaks resulting in overall larger and more extreme floods. Given that historically, the largest open water floods in Beaver Creek occur during spring freshet, it is expected that the Q200 will increase because of climate change. Overall, the tool predicts a 15.1% increase in annual precipitation for the Kootenay Boundary Region. **Table 4-6** provides the full projected changes in temperature and precipitation. The PCIC tool provides a more holistic picture of the possible effects that climate change could have on the region's hydrology.

Table 4-6: Plan2Adapt Tool Climate Change Projections for the Kootenay Boundary Region out to 2100

Climate Variable	Season	Baseline Mean Value	Projected Change	Range (10th to 90th percentile)
Temperature (°C)	Annual	4.5 °C	+5.3 °C	+4.0 °C to +7.2 °C
	Winter	-5.0 °C	+4.5 °C	+3.6 °C to +6.5 °C
	Spring	4.1 °C	+4.1 °C	+3.7 °C to +5.8 °C
	Summer	11.9 °C	+6.6 °C	+4.7 °C to +9.2 °C
	Fall	4.4 °C	+5.5 °C	+4.0 °C to +7.6 °C
Precipitation	Annual	2.1 mm/day	+8.1 %	+2.2 % to +15.1 %
	Winter	2.5 mm/day	+21.3 %	+16.0 % to +34.0 %
	Spring	2.0 mm/day	+6.2 %	-5.6 % to +21.1 %
	Summer	1.7 mm/day	-20.1 %	-37.0 % to -2.3 %
	Fall	2.1 mm/day	+11.3 %	+4.0 % to +23.5 %
Precipitation as Snow	Annual	0.9 mm/day	-32.0 %	-47.5 % to -22.7 %
	Winter	2.3 mm/day	-14.4 %	-30.8 % to -4.5 %
	Spring	0.6 mm/day	-66.2 %	-82.5 % to -50.8%
	Fall	0.8 mm/day	-56.2 %	-77.7 % to -47.1 %
Growing Degree-Days	Annual	1226 degree-days	+1188 degree-days	+811 to +1670 degree-days
Frost-Free Days	Annual	243 days	+69 days	+54 to +92 days
Heating Degree-Days	Annual	4963 degree-days	-1533 degree-days	-2004 to -1265 degree-days

Climate Variable	Season	Baseline Mean Value	Projected Change	Range (10th to 90th percentile)
Cooling Degree-Days	Annual	33 degree-days	+363 degree-days	+198 to +612 degree-days

The *Climate Atlas Map* provides predictions that show the same baseline trends, despite being based on a different climate model. **Table 4-7** provides a summary of the projected changes in precipitation and temperature as per the Climate Atlas model.

Table 4-7: Climate Atlas Map Climate Change Projections for the Trail Region

Climate variable	Time Period	Baseline Mean Value	Percent Change Compared to Historical Time Period
Maximum 1 Day Precipitation (mm)	1976-2005	25	-
	2021-2050	27	8%
	2051-2080	29	16%
Maximum 3 Day Precipitation (mm)	1976-2005	37	-
	2021-2050	39	5%
	2051-2080	42	14%
Maximum 5 Day Precipitation (mm)	1976-2005	51	-
	2021-2050	55	8%
	2051-2080	58	14%
Annual Precipitation (mm)	1976-2005	736	-
	2021-2050	768	4%
	2051-2080	808	10%
Annual Mean Temperature	1976-2005	7.5	-
	2021-2050	9.4	25%
	2051-2080	11.4	52%
Annual Maximum Temperature	1976-2005	12.8	-
	2021-2050	14.7	15%
	2051-2080	16.6	30%
Annual Minimum Temperature	1976-2005	2.1	-
	2021-2050	4.2	100%
	2051-2080	6.2	195%

The summarized information in **Table 4-7** shows a general increase in both maximum precipitation and temperatures, which aligns with the information shown in both *Plan2Adapt* and *IDF_CC Tool*. That said, it is difficult to predict how the increase in precipitation may affect flows in the region without developing a detailed hydrologic model of the watershed, which is outside the scope of this work. All of the data indicated that an increase in precipitation will occur, along with higher annual temperatures, which may result in spring freshet occurring sooner and with larger peaks. Since the climate change analysis indicated that an increase in peak flows on Beaver Creek is likely, a climate change factor of 15% was applied to Beaver Creek and tributary inflows.

It is worth noting that as per EGBC's Professional Practice Guidelines for *Legislated Flood Assessments in a Changing Climate in BC* (2018)², the effects of climate change need to be considered, and if there is no statistically significant trend in flood magnitude or frequency, the application of a "10% upward adjustment in design discharge to account for likely future change in water input from precipitation" is recommended. Given that climate tools all predict an increase in peak rainfall, and thus flood magnitude, a higher climate change increase factor of 15% is justified.

4.6 UNIT HYDROGRAPH CREATION

From the analysis of catchment characteristics for all proxy hydrometric stations, it was determined that Station 08NG076 (Mather Creek below Houle Creek) had the closest average slope to the Beaver Creek catchment (0.5%), as well as a similar catchment size. Due to these similarities, Station 08NG076 was used as the proxy for the development of the unit hydrograph for the main channel. For the tributaries, Station 08NE113 (Kelly Creek at 850m Contour) was used as a proxy, given its proximity to the study area and the length of its record, which included data from 1971-1982.

The design unit hydrographs were developed based on the Archer Method, which assumes that the basic parameter in the development of the hydrographs is time. The Archer Method is for nonparametric data, so there are minimal underlying assumptions about the relationship of the data. It is not assumed to follow a specific distribution. The data is organized based on different standardized flow data groupings, or flow percentiles.

For each station, the entire record of daily maximum data was reviewed, and the 10 largest flow events were selected. The events captured the freshet flow, typically beginning in late spring and ending in mid-summer. **Table 4-8** provides a summary of the start and end dates for the selected events for the Beaver Creek proxy, Station 08NG076 (Mather Creek below Houle Creek), and **Table 4-9** provides a summary of the events selected for the tributaries proxy, Station 08NE113 (Kelly Creek at 850m Contour).

Table 4-8: Selected Event Start and End Dates for Station 08NG076

Event Start Date	Event End Date	Peak Flow
02-04-1974	10-08-1974	12.8
07-04-1996	31-08-1996	16.1
15-04-1997	23-07-1997	13.1
02-04-2002	03-08-2002	17.7
31-03-2006	10-08-2006	14.6
22-04-2011	26-07-2011	16.5
29-03-2012	20-08-2012	20.1
31-03-2013	30-07-2013	23.6
30-03-2017	13-08-2017	16.8
09-04-2018	02-07-2018	13.1

² From *Flood Mapping in BC – APEGBC Professional Practice Guidelines V1.0* (2012)

Table 4-9: Selected Event Start and End Dates for Station 08NE113

Event Start Date	Event End Date	Peak Flow
04-04-1971	10-06-1971	6.2
09-03-1972	12-07-1972	6.7
09-04-1973	19-07-1973	4.8
24-03-1974	08-07-1974	4.4
10-04-1975	06-07-1975	4.0
04-04-1976	30-06-1976	4.2
19-03-1978	30-06-1978	3.2
19-04-1979	30-06-1979	2.4
11-04-1980	01-06-1980	4.8
20-04-1982	29-06-1982	3.7

The time it took to reach 10%, 20%, 30% and so forth of the peak for each event was determined. To create the unit hydrograph for the station, the times to peak were then averaged across each event and smoothed using a spline function. **Figure 4-8** shows the unit hydrographs for the 10 events that were selected for Station 08NG076; **Figure 4-9** illustrates the corresponding events chosen for Station 08NE114.

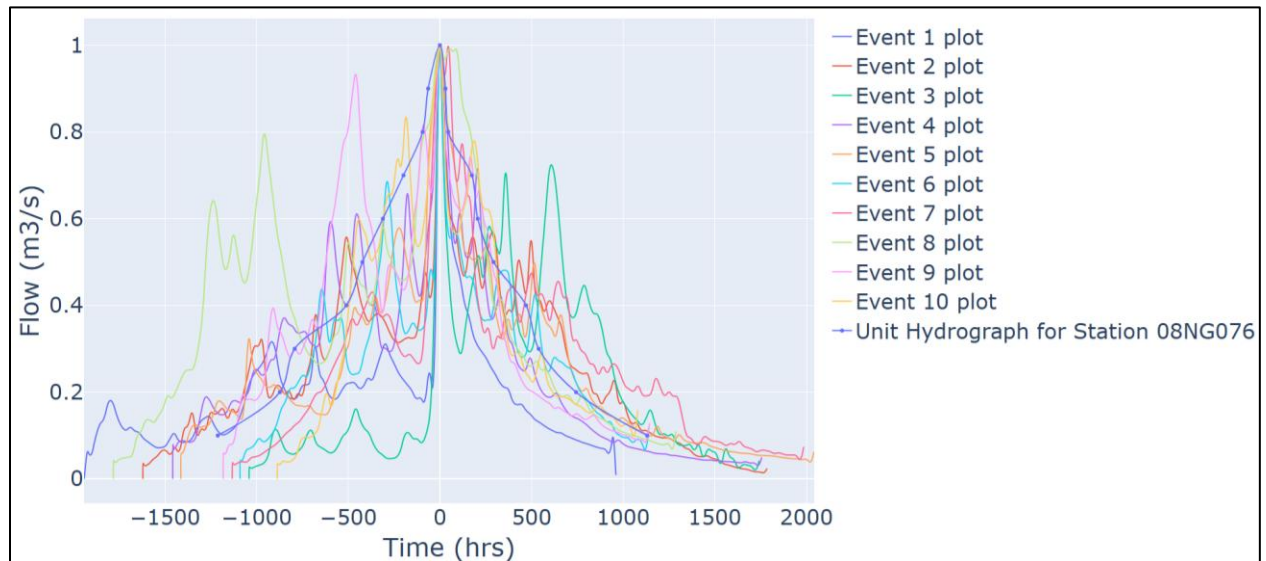


Figure 4-8: Hydrographs for the 10 Selected Events for Station 08NG076

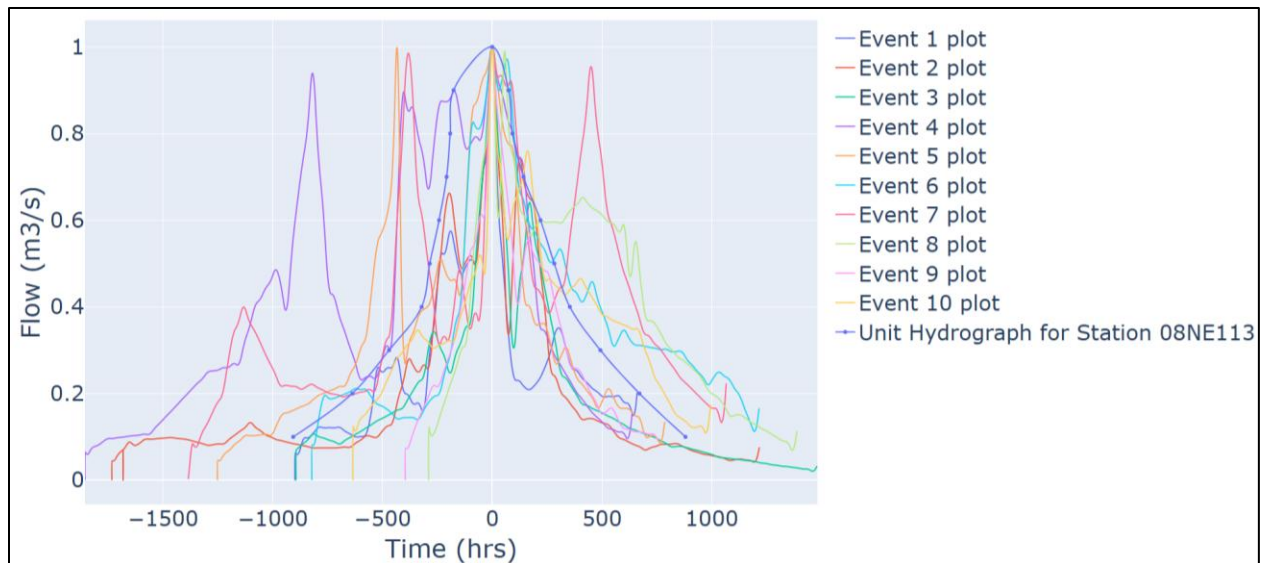


Figure 4-9: Hydrographs for the 10 Selected Events for Station 08NE113

Figures 4-10 and 4-11 show the final compiled unit hydrographs for Station 08NG076 and 08NE113, respectively. For each of the unit hydrographs, an interpolation function was applied so that timesteps of 1-hour intervals were available. These timesteps were required for use in the hydraulic modelling.

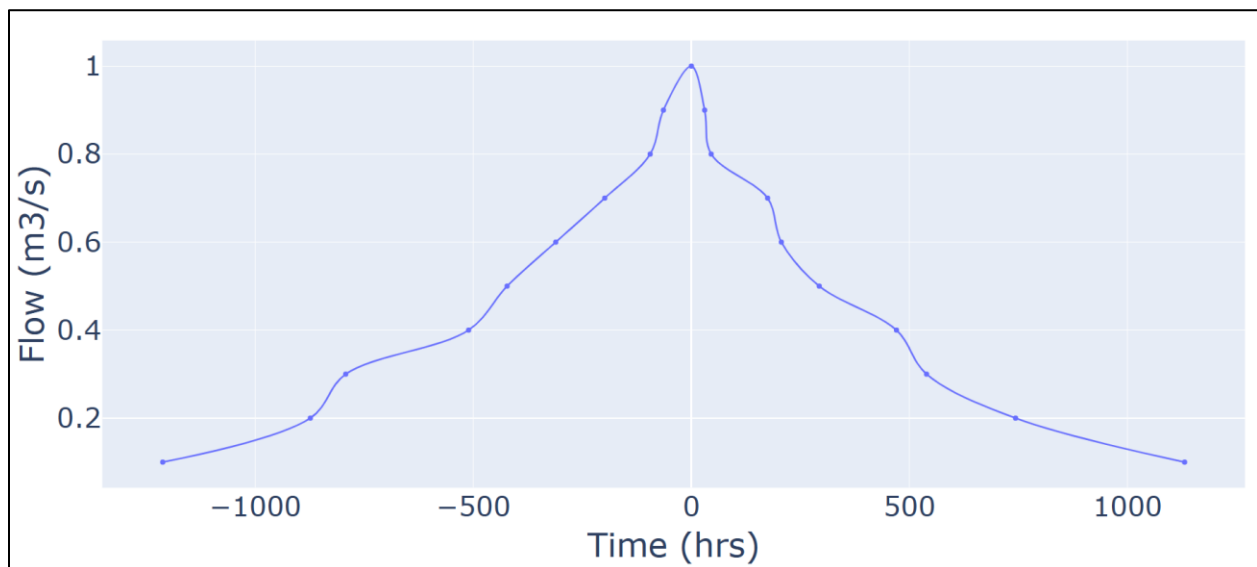


Figure 4-10: Unit Hydrograph for Station 08NG076 (Mather Creek below Houle Creek)

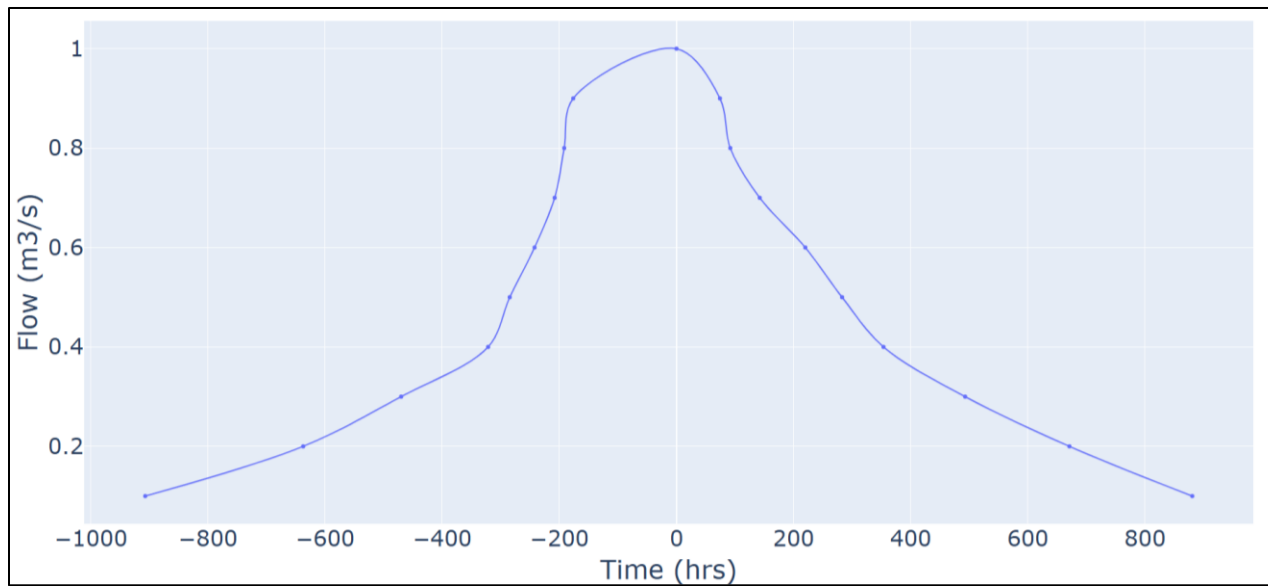


Figure 4-11: Unit Hydrograph for Station 08NE113 (Kelly Creek at 850m Contour)

As mentioned, there was very limited data available for Beaver Creek. Without overlapping information for both Beaver Creek and the tributaries, the lag time between the peak of the events could not be inferred. Lacking available data, the peaks for the main channel and its tributaries were aligned, which is a conservative approach.

5.0 HYDRAULIC ANALYSIS

5.1 MODEL CREATION

The hydraulic analysis was completed using GeoHECRAS Version 5.0.0, which is a proprietary software that uses the HEC-RAS computational engine developed by the U.S. Army Corps of Engineers. The software performs 1D, 2D, or combined 1D and 2D hydraulic computations for a full network of channels, which allows for modelling of not just the main channel, but also historical channels and tributaries. It is important to note that the tributaries were not modelled within the study area, as this was outside of the scope of the analysis. Instead, only the point inflow for the tributaries at the main channel was accounted for.

A 2D model has many advantages that produce higher accuracy of results through computing flow directions, transverse velocities, transverse momentum, and transverse variations in water surface. Due to these advantages and the production of higher accuracy results, a 2D model was selected for the analysis. The HEC-RAS Computational Engine Version 6.4.1. was released in 2025 and used for the analysis. The basic model inputs that are required for the hydraulic model are described in subsequent sections.

5.1.1 MANNING'S 'N' COEFFICIENTS

The Manning's 'n' coefficients represent the surface roughness of different land coverage throughout the study area. The 'n' value for the different land use types was determined using the Modified Channel Method³, which involves evaluating different components for each land use type based on standardized values. The sum of each component is combined to create the specific Manning's 'n' value for each land use type. Information collected during the site visit, including identification of changing land use types, type of vegetation in the area, and channel bed material, was used for this purpose. **Table 5-1** presents the respective Manning's 'n' value assigned to each land classification. Since analysis of the tributaries was not within the scope of the project, specific values for those watercourses were not calculated. The dominant land use of the surrounding area was used. Detailed calculations of Manning's 'n' values for each land classification are included in **Appendix B**. **Figure 5-1** shows the spatial distribution of Manning's 'n' values throughout the study area.

Table 5-1: Manning's 'n' Values

Land Use	Manning's 'n' Coefficient
Beaver Creek Main Channel	0.049
Forested Area Floodplain	0.128
Willow Area Floodplain	0.091
Grass Area Floodplain	0.036
Gravel Area Floodplain	0.023
Paved Areas	0.011

³ United States Geological Survey, Water Supply Paper 2339, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains, 1992.



Figure 5-1: Land Cover Map Representing the Different Manning's 'n' Values

5.1.2 2D MODEL CREATION

Hydraulic modeling of Beaver Creek was developed as a fully 2D hydrodynamic model in GeoHECRAS, which is based on the numerical solution of depth-averaged equations describing the conservation of mass and momentum. HEC-RAS 2D uses structured and unstructured computational meshes, which means the cells in a 2D area mesh can be a mixture of shapes and sizes. For Beaver Creek, an adaptive mesh of 20m cell spacing was created. The model solves depth-averaged equations for each cell in the 2D mesh at each time step. HEC-RAS uses an implicit finite volume solution algorithm, which not only allows 2D cells to be wet or dry at any given time (with the ability to handle a sudden rush of water into them) but can also handle flow regimes that change with time (*i.e.*, sub-critical flow, super-critical flow, or mixed flow).

For the HEC-RAS 2D computational methodology, the following modeling assumptions and limitations need to be taken into consideration:

- Vertical fluid motion is negligible;
- Velocity is vertically averaged at each cell center (depth-averaged flow);
- Energy head is computed at each cell center;
- Manning's roughness is assigned on the cell face using the roughness value at the cell face center;
- Manning's roughness is assumed constant across each cell face, although each cell face can have its own value;
- At least one external boundary condition must exist on the 2D mesh.

Breaklines

To provide further definition to the 2D computational mesh, breaklines were created at geographical features that represent a sudden elevation change. The breakline creates a boundary forcing new cells

to form along specific topographic features and allowing the model to conduct appropriate depth-averaged calculations individually for each cell while accounting for these changing features.

Breaklines were created along channel left and right overbanks, as well as major topographical features, to provide additional definition of these critical elements. The LiDAR elevation information was used when determining the location and alignment of breaklines. The breaklines have an increased mesh resolution of 2m along their profile. The breaklines increase the accuracy of the results through a finer mesh, and the additional cell boundaries prevent hydraulic connectivity between areas that may not necessarily flow to one another. **Figure 5-2** shows an example of breaklines applied to the computational mesh.

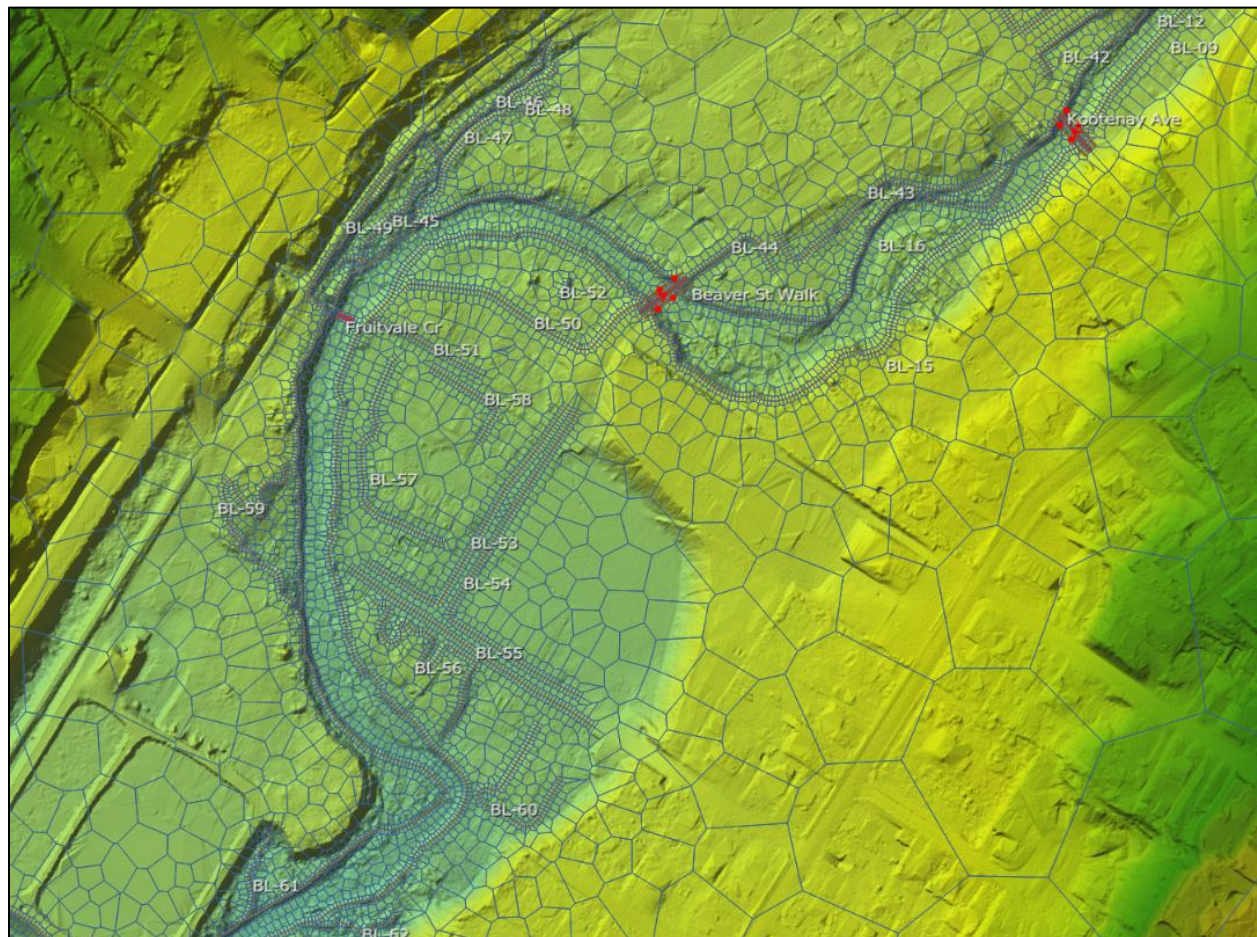


Figure 5-2: Sample of Breaklines (orange lines) in the Computational Mesh Defining Topographic Features

5.1.3 STRUCTURE MODELLING METHODOLOGY

There are eight roadway and pedestrian crossings for Beaver Creek within the study area; these were represented in the hydraulic analysis as 2D roadway crossings. Topographical survey was used to obtain spatial information for the elements of the crossings, including low and high chord elevations, pier, or abutment locations for bridges, as well as invert elevations for culverts. Visuals obtained during the site visit were used to supplement the modelling of the structures. **Table 5-2** provides a summary of the crossing input parameters for creating the structures in HEC-RAS. **Appendix C** contains photos from the site visit for each roadway crossing. At the Highway 3B roadway crossing, both the current structure and

the outdated structure exist within the channel; Both were represented within the model as they create independent obstructions to the flow. **Figure 5-3** shows the location of each crossing.

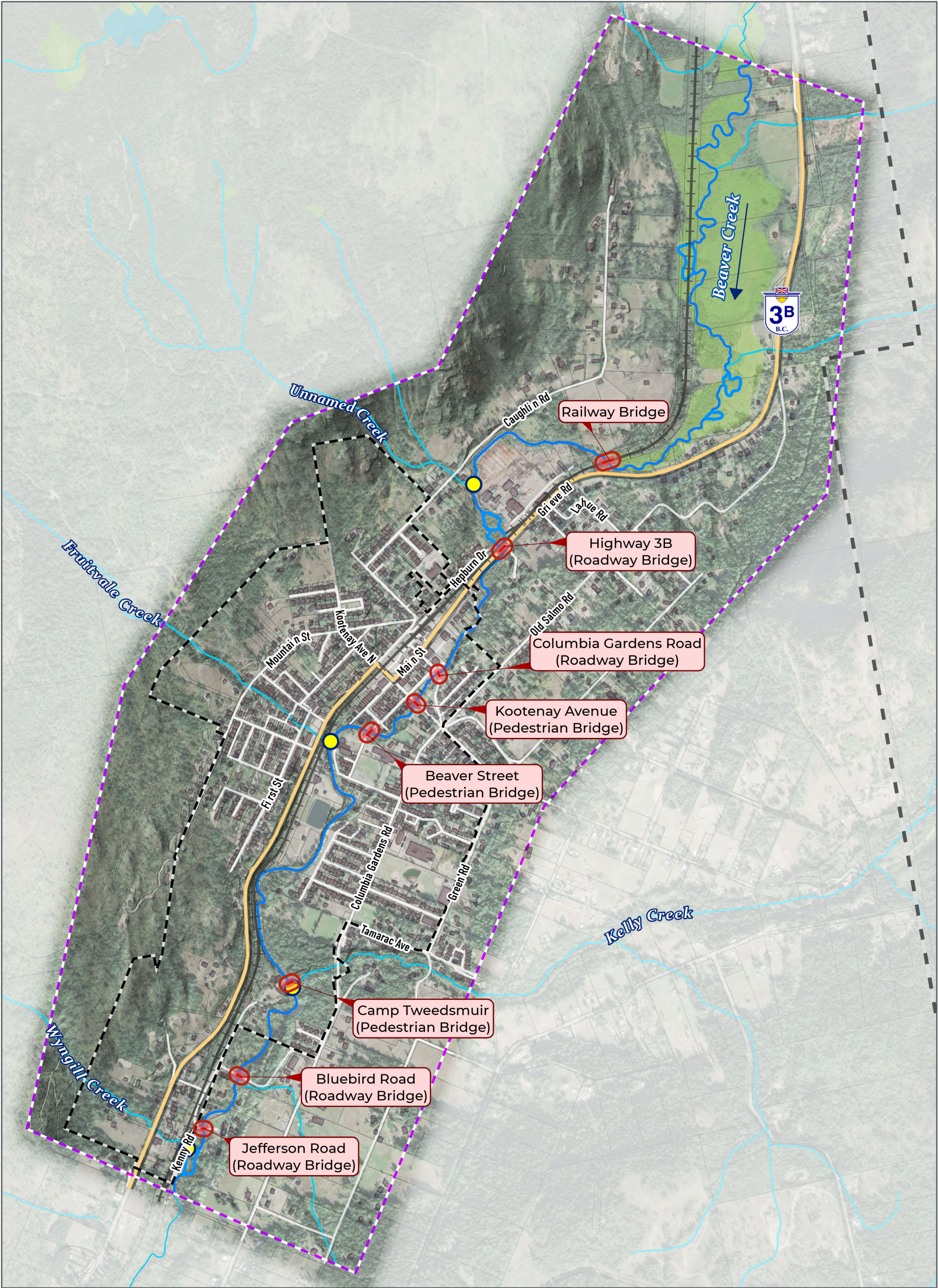
Table 5-2: Crossing Structure Summary

Crossing Location	Crossing Type	Bridge or Culvert	Deck & Low Chord Elevation (m)	Number of Piers	Diameter (m) of Culvert
Railway Bridge	Railway	Bridge	608.19 & 607.56	13	NA
Highway 3B-Outdated	Roadway - Abandoned	Bridge	606.26 & 605.66	15	NA
Highway 3B-Current	Roadway	Bridge	604.68 & 603.68	3	NA
Columbia Gardens	Roadway	Arch Culvert	600.98 (obvert) & 597.68 (invert)	NA	3.3m (Height), 10.5m (Span)
Kootenay Avenue	Pedestrian	Bridge	600.60 & 600.39	2	NA
Beaver Street	Pedestrian	Bridge	599.644 & 599.036	NA	NA
East of Camp Tweedsmuir	Pedestrian	Bridge	580.51 & 580.11	NA	NA
Bluebird Road	Roadway	Bridge	576.24 & 575.35	NA	NA
Jefferson Road	Roadway	Bridge	571.38 & 570.64	NA	NA

5.1.4 BOUNDARY CONDITIONS

The unit hydrographs developed through the hydrologic analysis, as described in **Section 4.0**, were peaked to the indicated design flows for the 200-year and 20-year return periods, accounting for the possible effects of climate change, for both Beaver Creek and the tributaries within the study area. The inflow hydrographs for the tributaries were applied directly within the main Beaver Creek channel at each confluence.

The study area ends southwest of Fruitvale, but Beaver Creek continues flowing until its eventual discharge into the Columbia River. The confluence is located far enough from the downstream end of the model that a normal boundary condition is considered appropriate. A normal boundary condition assumes the slope of the energy grade line equals the slope of the channel at that location. The location of each tributary inflow along Beaver Creek is also shown in **Figure 5-3**.



URBAN
SYSTEMS

Project #: 1011.0058.01

Author: BA

Checked: BR

Status:

Revision: A

Date: 2025 / 9 / 3

0 200 400 600 800
Meters

Scale: 1:15,000
(When plotted at 11"x17")

Coordinate System:
NAD 1983 UTM Zone 11N

Data Sources:
- DataBC (PMBC, Wetlands, Watercourses, Regional and Municipal District, Roads, Railway, Wetland), Esri (2024 Imagery), Harrier Aerial Surveys, Urban Systems Ltd.

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

Crossing Infrastructure

- Crossing Infrastructure
- Lateral Inflow
- Study Area
- Beaver Creek
- Watercourse
- Wetlands
- Highway 3B

- Railway
- Building
- Parcel
- Municipal Boundary
- Regional District Boundary



The Corporation of the Village of Fruitvale

Flood Risk Assessment

Crossing Infrastructure

Figure 5-3

6.0 COMMUNITY ENGAGEMENT SESSION

A community engagement session was held at the Village on April 15th, 2025. Residents, along with Council, attended the engagement to learn about the findings and provide valuable input. Online forms for collecting historical observations were also published, which allowed anyone unable to attend to provide input.

The session involved a brief presentation to share the methodology and initial findings of the hydraulic modelling. Limitations of the analysis (e.g., the inability to perform calibration due to the lack of existing historical information, the lack of hydrometric information within the region) and methods for mitigating the risks through the use of applicable data were shared. Draft results of the 200-year floodplain were presented. Engagement boards that summarized key inputs and findings of the project were displayed around the room. Following the presentation, attendees were encouraged to speak with Urban Systems' representatives to share their experiences surrounding Beaver Creek and flooding throughout the village.

Information collected from residents included areas where floodwaters had inundated with approximate depths of flooding. Residents were also given the opportunity to voice their concerns, including identifying areas where erosion was occurring or where debris flows had been experienced. Residents shared stories of sandbagging efforts to protect infrastructure and times of evacuation notices. Stories were shared, which indicated specific properties or areas that had experienced flooding, how the creek has been eroding over time, and where property fronting the banks of the Creek has been lost.

In addition to providing anecdotal information on past flooding events, a discussion was held with residents regarding possible mitigation measures. A range of possible options that included both structural and non-structural mitigation measures were discussed.

The anecdotal information was used to validate the results of the model, which is further discussed in subsequent sections. An overall summary of the information collected during the community engagement session is included in **Appendix D**.

7.0 MODEL VALIDATION AND SENSITIVITY ANALYSIS

Open channel flow models are typically calibrated to observed high-water marks and involve modifying Manning's 'n' coefficients until there is a good correlation. Unfortunately, there is no available information on high water marks corresponding to known historical flows within the study reach, and as such, calibration of Manning's 'n' coefficients was not possible.

However, historical anecdotal information collected during the community engagement session was used to validate the model results. Areas where residents experienced flooding were reviewed, and the extents of inundation in these areas were compared to ensure model results met or exceeded the experiences. Based on a discussion of past flooding events with long-term residents, and a review of gauged data near the region, it is unlikely that a 200-year flood event has been experienced in recent memory. Thus, model results for depth and extent are expected to be larger than historical flooding. **Appendix D** includes a summary of the information gained during the engagement session. All the historical flooding that was reported by residents has corresponding flooding indicated through the hydraulic analysis, to a lesser degree. This validates the results as it confirms likely areas of inundation.

In addition to the validation process, a sensitivity analysis on Manning's 'n' values was conducted, where values were increased and decreased by 20%. The analysis shows the model is only moderately sensitive to changes in Manning's 'n'. **Table 7-1** shows the differences in Water Surface Elevations (WSE) at various locations for the different Manning's 'n' scenarios. **Figure 7-1** shows the difference in the extents of flooding for each scenario.

Table 7-1 - Sensitivity Analysis on Manning's 'n' Values

Scenario	Modelled Water Surface Elevation(m)	Difference in WSE from Base Scenario (m)
Downstream of Highway 3B Bridge		
20% decrease in Manning's 'n' Values	601.90	0.18
Base Scenario	602.08	
20% Increase in Manning's 'n' Values	602.17	0.09
Upstream of Columbia Gardens (North Floodplain)		
20% decrease in Manning's 'n' Values	601.66	0.14
Base Scenario	601.80	
20% Increase in Manning's 'n' Values	601.82	0.02
Upstream of Fruitvale Creek Inlet (Within Channel)		
20% decrease in Manning's 'n' Values	597.87	0.20
Base Scenario	598.07	
20% Increase in Manning's 'n' Values	598.20	0.13

As seen in **Table 7-1** and **Figure 7-1**, a 20% change in Manning's 'n' values may represent an average of 0.13m change in the water surface elevation (both increase and decrease in equal measure), the smallest change is 0.02m, while the largest change is 0.18m. The location of the smallest change is near the

Columbia Gardens crossing. This area is a constriction in the channel, and the flood depths are likely being attenuated from the overall ponding in the area. Larger changes are experienced directly in the channel where the depths of water are largest.

The Modified Channel Method is a thorough process based on field observations that helps minimize uncertainties related to the estimation of roughness coefficients and, in lieu of being able to perform model calibration, provides confidence in the identified values. Additionally, freeboard will be applied to Flood Construction Levels to account for any uncertainties related to Manning's 'n' values.

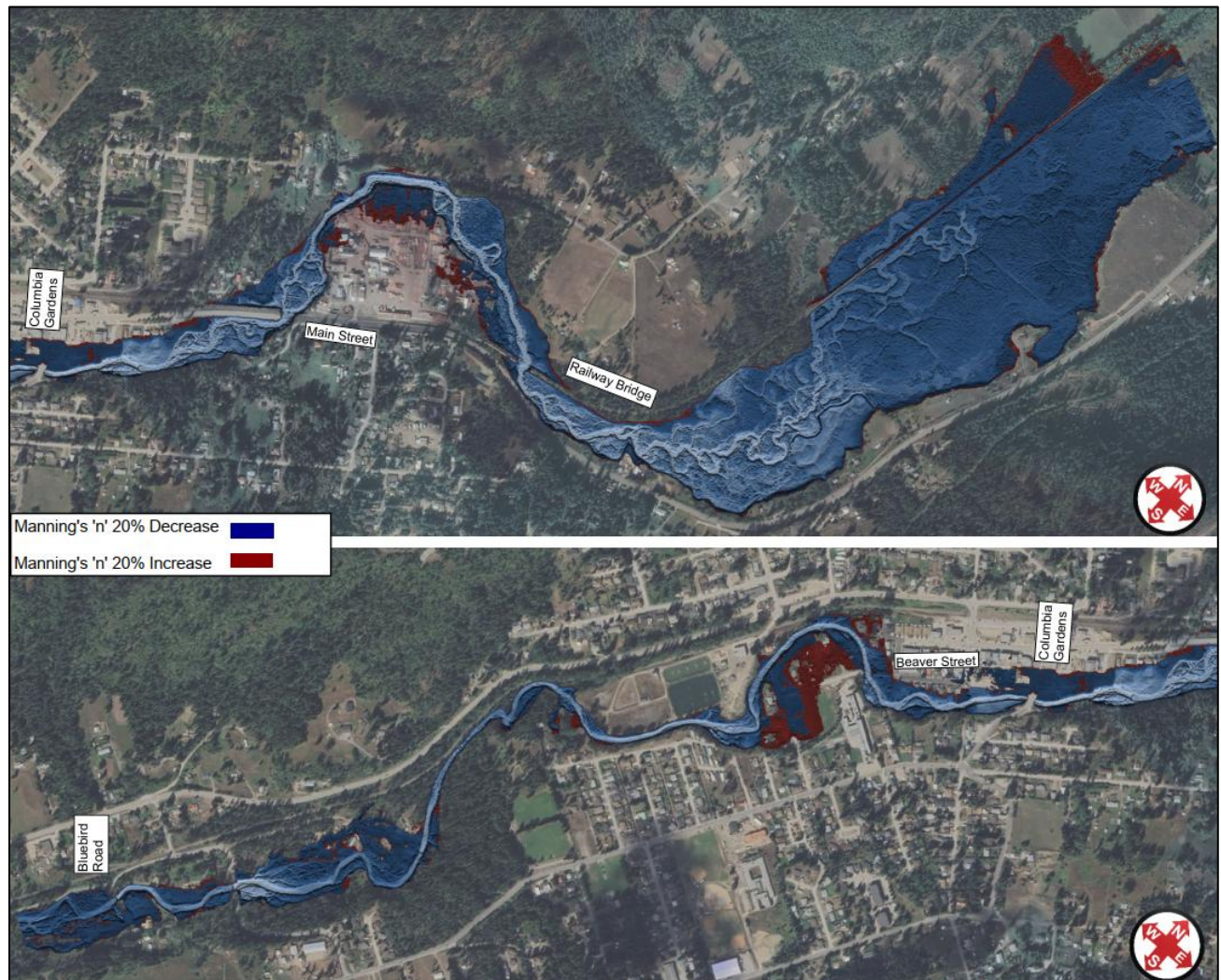


Figure 7-1: Difference in Flooding Extents for Manning's 'n' Sensitivity Analysis

8.0 CHANNEL STABILITY ASSESSMENT

8.1 AIR PHOTO ANALYSIS

A high-level channel stability assessment involves reviewing historical air photos of the study area at the same locations across varying time periods to identify lateral migration over time. For this process, the channel banks would be delineated, and when compared, the relative location of the banks are identified, and any lateral migration would be quantified. Ideally, air photos that have a high quality and resolution and were taken during times of the year when there was no snow cover and minimal vegetation cover are available for the analysis.

Unfortunately, the channel stability assessment for Beaver Creek was not able to be conducted due to the poor quality of historical air photos available for the area. Images were available for a range of years, including 1974, 1975, 1978, 1981, 1982, 1987, 1990, 1999, 2000, 2004, and 2018. Unfortunately, once the images were reviewed, it was determined they were either taken during poor times of the year for channel definition to be seen or did not have the creek within their extents, making them unsuitable for the analysis. The banks of the creek need to be visible within the images for proper delineation. Often, the vegetation cover was too heavy to allow for visuals of the ground. **Figure 8-1** is a sample of the air photo from 1974, showcasing the poor resolution.



Figure 8-1: Sample of 1974 Air Photo of Beaver Creek Available from GeoBC

8.2 CHANNEL MORPHOLOGY

Channel morphology and channel stability are governed by three main factors:

- Supply of water and sediment to the channel;
- The nature of the materials the river flows through; and
- The geological setting and geological history of the region.

It is accepted practice that channel morphology and channel stability should be assessed systematically within homogeneous reaches. The factors controlling the morphological characteristics and the “style” of channel instability within these reaches are expected to remain approximately unchanged.

The main parameters used to define reaches, based solely on aerial photography and LiDAR, included:

- Relation of the channel to the valley flat;
- Relation of the channel to the valley walls; and
- Channel planform pattern.

The relation of the channel to the valley flat describes the interaction between the main channel and the flatter portions of the valley that are typically part of the immediate floodplain. The relation of the channel to the valley walls describes the degree of confinement along a river. Channels that are frequently confined by terraces or non-erodible bedrock outcrops have only very limited potential for channel shifting or avulsions. The channel planform description includes characterizing the channel pattern as well as describing the occurrence of islands and bars. The purpose of this classification is to be able to diagnose the style of channel instability that is associated with each channel type.

For Beaver Creek within the Fruitvale area, the channel is confined due to the surrounding urbanization. As a result of bank protection measures installed throughout urbanized areas, there appears to have been minimal shifting of the banks over the years. It should be reiterated, however, that without adequate historical aerial imagery, rates of lateral migration (if any exist) cannot be quantified at this time.

9.0 FLOOD INUNDATION MAPPING

Flood inundation maps were produced for the 20- and 200-year events for Beaver Creek (**Appendix E**). The map provides the modelled water surface elevations based on the results of the GeoHECRAS hydraulic analysis with freeboard added. Based on the EGBBC *Professional Practice Guidelines for Flood Mapping in BC (Version 1.0, 2017)*, typical freeboard values for “water” floods that have been adopted in BC are 0.3m above the maximum instantaneous design flood level or 0.6m above the mean daily design flood level. In this analysis, historic peak instantaneous flows were used to establish the design flows, so a freeboard of 0.3m added onto the modelled water surface elevation would be appropriate to determine Flood Construction Levels. Freeboard accounts for uncertainties in the modelling process, and captures the intricacies of a real-world system that are unable to be replicated through numerical modelling.

10.0 FLOOD HAZARD MAPPING

Hazard mapping is created through combining flood depth with flow velocity. A debris factor is then applied to account for debris and solids that may be carried by the flood flows, which adds an increased danger.

The Flood Mapping in BC guidelines recommended using the hazard rating methodology outlined by the United Kingdom's Department of Environment, Food & Rural Affairs (DEFRA). The hazard rating for the maps is determined through the following equation:

$$HR = d * (v+0.5) + DF$$

Where HR is the hazard rating, d is the maximum instantaneous depth of the flood flow, v is the maximum instantaneous velocity of the flood flow, and DF is the debris factor. For rural woodland areas, the debris factor is 0.5 if the depth is between 0.25m to 0.75m. For areas that have a depth greater than 0.75m, the debris factor is 1. The debris factor is 1 for any areas that have a velocity greater than 2 m/s.

The methodology for determining the debris factor is outlined as a best practice in EGBC *Flood Mapping in BC- Professional Practice Guidelines V1.0 (2017)*⁴.

The hazard rating is then split into four different categories, as indicated on the legend of the map. **Table 10-1** below describes the different ranges of the categories and the associated definition.

Table 10-1: Hazard Rating Categories

Hazard Rating	Definition
<0.75	Very low hazard (caution)
0.75-1.25	Danger for some (includes children, the elderly, and the infirm)
1.25-2.00	Danger for most (includes the general public)
>2.00	Danger for All (includes emergency services)

The hazard rating is highest within the main channel, as this is where depths and velocities are the greatest. The lowest areas are typically on the fringes of the floodplain. In surrounding areas (forested, farmland) where floodwaters fill the area, the hazard is at danger for most and danger for some. In residential areas throughout the study area where flooding is scattered and only sparsely concentrated, there is only a very low hazard. The flood hazard maps for both the 200-year and the 20-year flooding events are presented in **Appendix F**.

⁴ See Flood Mapping in BC APEGBC Professional Practice Guidelines V1.0 by Professional Engineers and Geoscientists of BC (pg. 22, 23)

11.0 FLOOD RISK ASSESSMENT

11.1 ROADWAY CROSSING ASSESSMENT

The performance of the roadway crossings was evaluated under the 200-year scenario to understand if existing infrastructure is posing a significant constraint to flow in the Creek. Undersized infrastructure prevents full conveyance of flow and creates backwater that may result in surrounding areas being inundated. As part of this assessment, upgrades to existing road crossings were evaluated based on (1) whether they provide value in terms of effectively reducing the risk of flooding in the community, and (2) whether they are feasible from a constructability perspective.

The key performance indicators to evaluate the capacity of each crossing are the water surface elevation relative to the low chord of the bridge, or the headwater over diameter ratio when the crossing consists of a culvert instead of a bridge. Head losses through the crossings were also evaluated. These are affected by several factors, including flow constriction, contraction and expansion through the crossing, and the presence of piers and abutments. **Table 11-1** summarizes the performance of the roadway crossings and includes commentary on whether further actions are recommended.

Table 11-1: Crossing Performance During the 200-year Design Scenario

Crossing Location	Capacity Indicator*	Head loss (m)	Comment
Railway Bridge	Low Chord = 607.56m Headwater = 605.38m	0.20	Crossing is not significantly restricting the flow; no upgrades recommended.
Highway 3B- Outdated	Low Chord = 605.66m Headwater = 602.81m	0.03	Crossing is not significantly restricting the flow; no upgrades recommended.
Highway 3B - Current	Low Chord = 603.68m Headwater = 602.78m	0.09	Crossing is not significantly restricting the flow; no upgrades recommended.
Columbia Gardens	HW/D = 1.2	0.64	The culvert crossing is as large as the geometry of the channel allows. High headwater is due to a constriction of the channel, more so than the culvert crossing. Upgrades will not have a significant enough impact to be justified.
Kootenay Avenue	Low Chord = 600.39m Headwater = 600.87m	0.55	Head loss from piers is minimal. Raising the bridge above headwater elevation would have a negligible impact on mitigating flooding, and the benefit would not justify the cost.
Beaver Street	Low Chord = 599.04m Headwater = 599.22m	0.13	Although headwater is above the low chord of the bridge, head loss is minimal, and no upgrades are recommended as impacts to flood extents are negligible.
East of Camp Tweedsmuir	Low Chord = 580.11m Headwater = 580.15m	0.25	Although headwater is marginally above the low chord of the bridge, head loss is minimal, and no upgrades are recommended as impacts to flood extents are negligible.
Bluebird Road	Low Chord = 575.35m Headwater = 575.94m	2.09	Crossing is not restricting the capacity, natural constriction of the channel at this location.

Crossing Location	Capacity Indicator*	Head loss (m)	Comment
			Upgrades to infrastructure would only have minor alleviation to flooding.
Jefferson Road	Low Chord = 570.64m Headwater = 569.68m	0.035	Crossing is not significantly restricting the flow; no upgrades recommended.

*A crossing was considered to be at capacity if the headwater/diameter ratio > 1 for culverts, and if the headwater has reached the low chord of the bridge

The Bluebird Road crossing is experiencing significant head loss (2.09m) as a result of the water surface reaching the low chord of the bridge. To understand the effects of the crossing on the surrounding floodplain, a scenario was analyzed with the bridge removed, which assumed that a new single-span structure would be built with sufficient freeboard. The backwater effects of the bridge on the inundation of the Creek extend approximately 230m upstream (**Figure 11-1**). The modelled water surface elevation (WSE) at the upstream side of the bridge with the bridge in place is 575.94m. When the bridge has been removed, the future modelled WSE is 573.97 m. Even though this is a significant reduction in backwater, the area that would be alleviated from flooding is entirely forested and free of existing infrastructure. The effects of backwater on existing infrastructure are negligible and thus, upgrades to this crossing may not be warranted at this time.

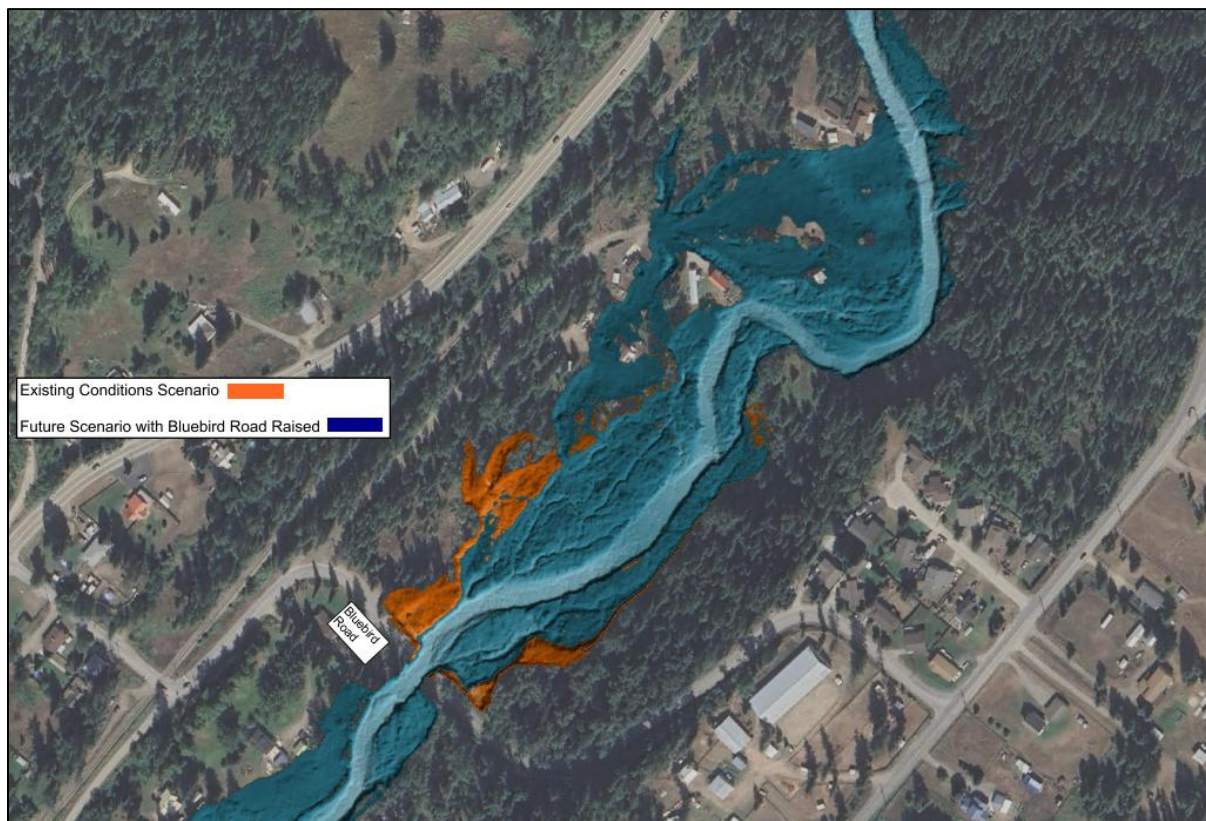


Figure 11-1: Comparison of Flood Inundation Extents if Bluebird Road is Raised

The remaining roadway crossings are not posing a significant constriction to flow along the channel. Most of the crossings are single-span bridges that don't constrain the flow or alter the cross-sectional geometry of the channel. A future scenario was modelled as well, where existing crossings with piers or

abutments were replaced with single-span crossings free of flow restrictions. The results showed a negligible reduction in WSE.

Upgrades to infrastructure are not recommended, as the surrounding impact to the extent of flooding would be minimal compared to the upgrade costs and its environmental implications. The leading cause of flooding of Beaver Creek is the limited capacity of the main channel to convey the 200-year flows without overtopping its banks, rather than flow restrictions at roadway crossings.

11.2 FLOOD RISK INVENTORY

An inventory of land parcels, buildings, infrastructure, and population that may be at risk of flooding, based on both open water flood inundation and flood hazard maps, was compiled to help quantify the potential flood vulnerabilities within and around the study reach for the 200-year event.

11.2.1 BUILDINGS

The Village has approximately 1,594 buildings within its jurisdiction, of which 75 are inundated during the 200-year event. To quantify the damage to these buildings, all the inundated buildings were assigned one of the following classifications:

1. Residential
2. Commercial
3. Industrial
4. Institutional/Other

Building footprints and elevations were extracted from LiDAR point cloud data, and building types were derived from Zoning Data sourced from the Regional District of Kootenay Boundary geographic data. The depth of flooding was obtained for each building from the water surface elevation and flood depth rasters of the 200-year flooding event exported from the GeoHECRAS model. Additionally, the area of each building impacted from flooding was obtained using the building footprints extracted from the LiDAR data.

Each building classification was further divided into structural damage and content classifications, which were obtained from the *Provincial Flood Damage Assessment Study* (2015) prepared by IBI Group for the Province of Alberta (referred to as “the study”). This study is one of the two methodologies recommended by Natural Resources Canada’s *Federal Flood Damage Estimation Guidelines for Buildings and Infrastructure* (2021) (referred to as “the guidelines”). The study provides depth-damage values for each sub-classification, estimating structural and content damages per square metre of flooded building footprint, based on the maximum water depth within the footprint. The depth damage values have been increased by a location factor of 20% as suggested by the study, to account for the additional costs of construction in more remote communities. All building damage estimates were calculated as one-story buildings, as the maximum water depth present in a building was 1.53m.

Residential buildings were classified into 5 different structural and content classes: A1, B1, C1, D1, and MA1. The structural and content classes of residential buildings were determined by their construction techniques, size and quality, and number of storeys. The residential depth damage values used in this analysis are shown below in Table 15. Note that negative water elevations are shown in **Table 11-2**, as the estimates consider basement flood damage as well.

Table 11-2: Residential Depth Damage Values (Incl. Location Factor)

Flood Depth (m)	Structural Damage (\$2015/m ²)					Contents Damage (2015\$/m ²)				
	A1	B1	C1	D1	MA1	A1	B1	C1	D1	MA1
-2.7	\$0	\$0	\$0	\$0	\$0	\$0	\$0	\$0	\$0	\$0
-2.6	\$277	\$278	\$284	\$0	\$0	\$480	\$271	\$353	\$0	\$0
-2.4	\$325	\$338	\$371	\$0	\$0	\$665	\$407	\$420	\$0	\$0
-2.1	\$359	\$374	\$427	\$0	\$0	\$858	\$450	\$462	\$0	\$0
-1.8	\$359	\$374	\$427	\$0	\$0	\$934	\$481	\$502	\$0	\$0
-1.5	\$366	\$386	\$449	\$0	\$0	\$941	\$492	\$506	\$0	\$0
-1.2	\$402	\$401	\$460	\$0	\$0	\$943	\$493	\$506	\$0	\$0
-0.9	\$402	\$401	\$460	\$0	\$0	\$946	\$494	\$508	\$0	\$0
-0.6	\$427	\$434	\$509	\$0	\$0	\$972	\$511	\$584	\$0	\$0
-0.3	\$428	\$436	\$512	\$32	\$0	\$1,003	\$605	\$710	\$0	\$0
0	\$438	\$449	\$527	\$32	\$0	\$1,003	\$605	\$710	\$0	\$0
0.1	\$706	\$514	\$1,087	\$434	\$539	\$1,451	\$870	\$1,007	\$292	\$265
0.3	\$713	\$522	\$1,102	\$486	\$539	\$1,752	\$1,066	\$1,164	\$455	\$461
0.6	\$809	\$582	\$1,195	\$486	\$816	\$1,913	\$1,121	\$1,231	\$511	\$522
0.9	\$1,018	\$726	\$1,333	\$564	\$950	\$1,974	\$1,195	\$1,289	\$577	\$617
1.3	\$1,018	\$726	\$1,333	\$564	\$1,124	\$1,982	\$1,198	\$1,301	\$580	\$632
1.5	\$1,018	\$726	\$1,333	\$564	\$1,124	\$1,982	\$1,198	\$1,301	\$580	\$634
1.8	\$1,018	\$726	\$1,333	\$564	\$1,124	\$2,010	\$1,199	\$1,302	\$580	\$634
2.1	\$1,018	\$726	\$1,333	\$564	\$1,124	\$2,010	\$1,199	\$1,302	\$580	\$634
2.4	\$1,018	\$726	\$1,333	\$564	\$1,124	\$2,010	\$1,199	\$1,302	\$580	\$646

All commercial buildings were classified into the S1 structural class (office/retail) and the C7 contents class (generalized retail), consistent with the study. The depth damage values for commercial buildings are shown below in **Table 11-3**.

Table 11-3: Commercial Depth Damage Values (Incl. Location Factor)

Flood Depth (m)	Structural Damage (2015\$/m ²) -S1	Contents Damage (2015\$/m ²) - C7
0	\$0	\$0
0.2	\$76	\$218
0.3	\$152	\$419
0.6	\$158	\$614
0.9	\$162	\$938
1.2	\$166	\$1,103
1.5	\$186	\$1,231
1.8	\$197	\$1,324
2.1	\$209	\$1,338
2.4	\$209	\$1,361
2.7	\$222	\$1,361
3	\$222	\$1,361
3.3	\$222	\$1,361
3.7	\$222	\$1,361
3.9	\$222	\$1,361

All industrial buildings were classified into the S2 structural class (industrial/warehouse) and the L1 contents class (industrial/warehouse), consistent with the study. The depth damage values for industrial buildings are shown below in **Table 11-4**.

Table 11-4: Industrial Depth Damage Values (Incl. Location Factor)

Flood Depth (m)	Structural Damage (2015\$/m ²) - S2	Contents Damage (2015\$/m ²) - L1
0	\$0	\$0
0.2	\$13	\$208
0.3	\$25	\$520
0.6	\$28	\$762
0.9	\$28	\$1,213
1.2	\$29	\$1,386
1.5	\$36	\$1,421
1.8	\$37	\$1,490
2.1	\$41	\$1,542
2.4	\$41	\$1,594
2.7	\$46	\$1,628
3	\$50	\$1,663
3.3	\$50	\$1,663
3.7	\$50	\$1,663
3.9	\$50	\$1,663

All institutional or other buildings were classified into the S5 structural class (institutional) and the N1 contents class (institutional/other), consistent with the study. The depth damage values for institutional or other buildings are shown below in **Table 11-5**.

Table 11-5: Institutional/Other Depth Damage Values (Incl. Location Factor)

Flood Depth (m)	Structural Damage (2015\$/m ²) - S5	Contents Damage (2015\$/m ²) - N1
0	\$12	\$0
0.2	\$70	\$71
0.3	\$128	\$143
0.6	\$130	\$374
0.9	\$131	\$535
1.2	\$132	\$570
1.5	\$138	\$570
1.8	\$140	\$570
2.1	\$148	\$570
2.4	\$148	\$570
2.7	\$156	\$570
3	\$156	\$570
3.3	\$156	\$570
3.7	\$156	\$570
3.9	\$156	\$570

High-level building damage estimates were prepared by calculating the damage per building based on its classification, maximum water depth, flooded area, and the corresponding price per square metre of flooded area. The cost estimates were then adjusted to present-day values based on Statistics Canada's

Building Construction Price Index (BCPI) for the structural damage and Consumer Price Index (CPI) for the contents damage, as shown below in **Tables 11-6 and 11-7**, respectively.

Table 11-6: Building Construction Price Index (BCPI)

Time Period	Residential	Non-Residential
Q1 2015	54.9*	81.7
Q2 2025	112.5	107.5

*Residential BCPI is only available from Q1 2017

Table 11-7: Consumer Price Index (CPI)

Time Period	Consumer Price Index (CPI)
Q1 2015	125.5
Q2 2025	164.3

The total high-level estimate for building damages based on the above methodology is summarized in **Table 11-8**:

Table 11-8: High Level Estimate for Building Damages

Class	Structural Damage (\$2025)	Contents Damage (\$2025)	Total Building Damage (\$2025)
Residential	\$6,639,508	\$6,557,067	\$13,196,574
Commercial	\$140,033	\$393,469	\$533,501
Industrial	\$1,312	\$21,509	\$22,821
Institutional/Other	\$163,107	\$353,190	\$516,297
Total Building Damage - All Classes (\$2025)			\$14,269,193

11.2.2 ROADWAYS

The guideline includes methodology for estimating damage to roads and transportation infrastructure. Specifically, it recommends a value between 10% and 25% of direct structural damages to buildings. For this analysis, roadway and transportation infrastructure were assumed to be 20% of the direct building structural damage costs. The estimate was chosen at the higher end of the recommended range as the Village has several critical roadways that are shown in GeoHECRAS to be inundated during a 200-year event. The total high-level estimate for total roadway and transportation infrastructure damaged based on the above methodology is **\$1,041,594**.

11.2.3 INDIRECT DAMAGE

The guidelines define indirect damages such as the costs of evacuation, employment losses, administrative costs, net loss of normal profit and earnings to capital, management, and labour, and general inconveniences. Given the complexity of estimating these items, indirect damage estimates are typically derived as a percentage of the direct damage estimates based on historical case studies and expert opinion. The guidelines find that indirect damage estimates typically range from 10% to 45% of direct damages, with 20% being the most commonly used in estimates. This assessment considered indirect damages to be 20% of direct damages, which totals **\$3,062,157**.

11.2.4 TOTAL ESTIMATED FLOOD DAMAGES

Based on the results of the analysis in Sections 11.2.1 to 11.2.3, the total damages from potential flood vulnerabilities within and around the study reach for the 200-year event are summarized in **Table 11-9** below:

Table 11-9 - Summary of Costs Related to Potential Flood Damages for a 200-year Event

Damage Type	Total Costs
Buildings	\$14,269,193
Roadways	\$1,041,594
Indirect Damages	\$3,062,157
TOTAL DAMAGES	\$18,372,944

11.2.5 LOSS OF LIFE

A loss of life was completed for two scenarios in order to provide the Village with a sense of the magnitude of the flood risks to humans. This calculation was performed based on the methodology presented in the *RCEM – Reclamation Consequence Estimating Methodology* (U.S. Department of the Interior Bureau of Reclamation, 2015). The following figures taken from the RCEM were used to develop loss of life estimates:

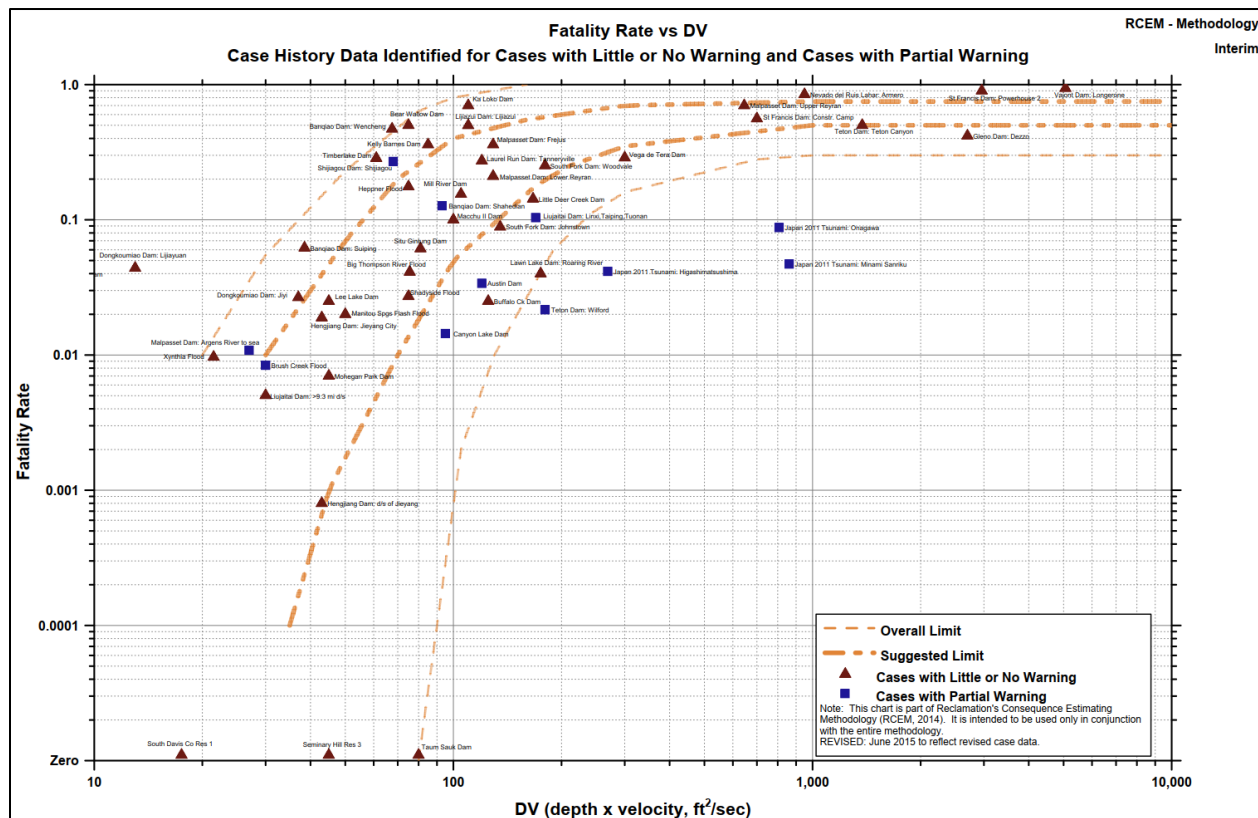


Figure 11-2: Fatality Rate vs. DV for Little or No Warning

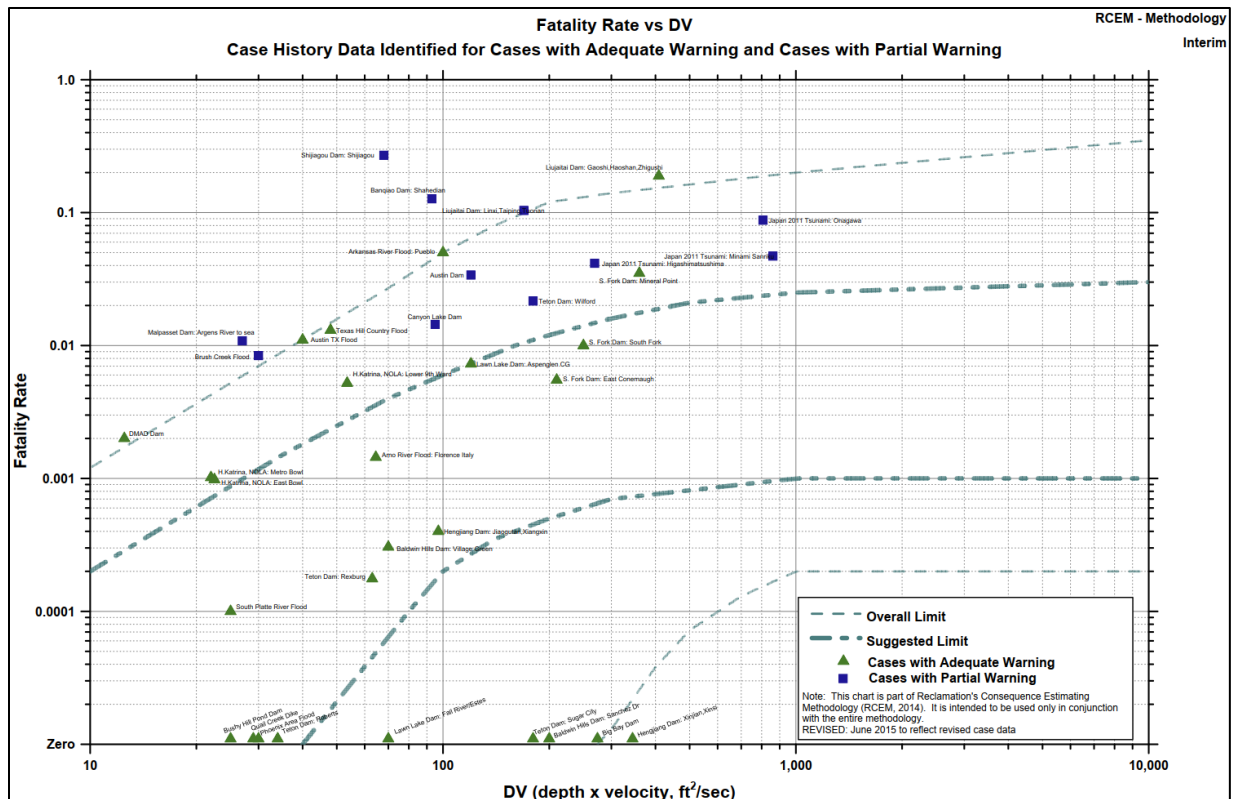


Figure 11-3: Fatality Rate vs. DV for Adequate or Partial Warning

Figures 11-2 and 11-3 show the overall and suggested limits for fatality estimation based on the depth x velocity (DV) values present at a given location. For the purposes of this assessment, an estimate was developed based on the upper suggested limits to be conservative. Figure 11-2 depicts the fatality rates where residents are given adequate warning prior to the flood events, and Figure 11-3 depicts fatality rates in the worst-case scenario where residents receive little or no warning. The maximum recorded DV within a building in the Village is 58.72 ft²/s (5.46 m²/s), and this value was used to estimate the fatality rate in both scenarios. An assumed population of 1,958 people was considered based on the 2021 census for the Village, as the entire population is affected by flooding due to loss of buildings, utilities, emergency services, transportation networks, and regular employment. Table 11-10 presents the estimates for each scenario based on the upper suggested fatality rate limits shown in Figures 11-2 and 11-3.

Table 11-10: Estimated Fatality Rates and Number of Fatalities

Scenario	Estimated Fatality Rate	Estimated Number of Fatalities
Little or No Warning	0.1	196
Adequate Warning	0.003	6

12.0 FLOOD MITIGATION PLAN

As mentioned in Section 11, there are no culvert/bridge crossing upgrades or other capital infrastructure upgrades that would provide a significant flood mitigation benefit at a justifiable cost to the Village. Structural mitigation efforts come with many important considerations, and from conversations with the Village, it is understood at this time that these measures may not be feasible to implement.

Structural mitigation options such as berms or flood walls have a significant impact on the surrounding land. The most straightforward case for implementing these measures is if the land surrounding the creek is owned by the Village and there is no other existing infrastructure in the vicinity. For berms to be efficient, their design height must be above the expected flood elevation, which often results in large footprints given the required side slopes for structural stability. The area surrounding Beaver Creek is, for the most part, privately owned, which would require a significant amount of land acquisition, in addition to the demolition or relocation of many pieces of publicly and privately owned infrastructure.

If there is sufficient space to construct these structural mitigation flood measures, proper access for operations and maintenance must also be provided. Within B.C, dike operations are closely monitored under the Dike Maintenance Act, which lays out strict requirements for operating and maintaining dike structures. This would be a significant undertaking for Village staff.

Structural mitigation measures are effective when implemented in their entirety. When done piecemeal, flood waters may flow around any gaps in the structures and continue to inundate the surrounding land. The construction process for implementing these measures would likely occur in a phased approach to allow for information collection, seasonal construction, and alignment with funding cycles. Until the structural measures are fully constructed along the entirety of Beaver Creek within the boundaries of the Village, the community would not be protected from flooding events, and risk will not be mitigated.

Finally, implementing structural mitigation may have significant effects on downstream areas. Under current flooding conditions, there is a considerable volume of storage in the overbanks. With structural flood mitigation in place, floodwaters would be forced to stay within the main channel, and volume previously stored in the overbanks would flow downstream, which could result in increased water levels downstream. Detailed analyses to evaluate the impacts of implementing structural mitigation measures would be required.

These concepts were discussed with the Village, and their appetite for flood mitigation leaned towards non-structural options. Flooding during a 200-year event would pose a considerable risk to residents, and these options can be effective in providing protection to the community, depending on what those specific goals for protection are. It is recommended that efforts be put into long-term monitoring of Beaver Creek to develop adequate warning systems that can lead to timely evacuations and prevent the potential loss of life. Additionally, the analysis presented in this report identified specific areas where a breach of floodwaters could occur. If temporary measures were enacted to prevent a breach in these areas, flooding of critical infrastructure could be avoided. The following recommendations are made to support these.

1. Wetland Level Sensor: The purpose of this would be to remotely indicate the level of water upstream of the Village in the event of a rise in water. Urban proposes that a reasonable setpoint is selected as a trigger to respond in some way. That could be preparation, communication, or otherwise mobilizing a response plan. The equipment would generally include a power source,

level transducer, data logger, and communication equipment. Basic access would be needed to check and maintain the device.

2. Stream Flow Monitor: The purpose of this would be to measure the stream flow in Beaver Creek in a stable portion of the channel (near Columbia Gardens Rd), to validate and record stream flow over time. The equipment would be similar to the level sensor but might not need communication, allowing data to be downloaded periodically.
3. Temporary Dam Equipment: There are areas identified in the flood inundation mapping where temporary flood protection installed along the road would significantly reduce the extent of flooding (i.e., Kootenay Avenue S. and Bluebird Road). The purpose of using temporary flood equipment would be to guard against road overtopping at these locations. The equipment would include an inflatable, reusable temporary dam that would be deployed as needed.

13.0 CONCLUSIONS

The Village of Fruitvale is enhancing their flood resiliency by assessing the risks that Beaver Creek poses to the community and the surrounding area. A hydrologic assessment was conducted to identify the 20- and 200-year peak flows on Beaver Creek and the surrounding tributaries, accounting for the possible effects of climate change. Water Surface Elevations for 20- and 200-year events were established through a hydraulic analysis using a 2D numerical model developed in GeoHECRAS. The resulting flood inundation maps show the extents of flooding for a 20-year event and a 200-year event, including 0.3 m of freeboard. The latter forms the basis for recommended Flood Construction Levels.

Flood hazard mapping was also developed as part of this study. Hazard mapping was created through combining flood depth with flow velocity and applying a debris factor to account for debris and solids that may be carried by the flood flows, which add an increased danger. The hazard rating is shown to be highest within the main channel and lowest on the fringes of the floodplain. In residential areas where flooding is scattered and only sparsely concentrated, hazards were classified as “very low”.

Based on the results of the flood inundation mapping and hazard ratings, a flood risk assessment was conducted. Several conclusions were drawn from this assessment:

1. There are no culvert/bridge crossing upgrades or other capital infrastructure upgrades that would provide a significant flood mitigation benefit at a justifiable cost to the Village. Structural mitigation measures are not feasible at this time;
2. Total estimated damages resulting from a 200-year flood in Beaver Creek are expected to be upwards of \$18 million; and
3. The total estimated number of fatalities with little to no warning during a 200-year flood is expected to be in the order of 196 people, but this number would be in the single digits if adequate warning systems are in place.

Recommended flood mitigation includes long-term monitoring of Beaver Creek through the installation of a water level sensor (with communication equipment) at the upstream end of the Village and flow monitoring near Columbia Gardens Rd. This will allow the Village to develop adequate warning systems that can lead to timely evacuations and prevent the potential loss of life. The use of temporary flood dams at specific breach locations where a larger extent of flooding can be prevented was also recommended.

APPENDIX A: FREQUENCY ANALYSIS



APPENDIX A: Frequency Analysis

The frequency analysis completed for Beaver Creek was used to determine the 200-year return flow based on the statistical distributions applied to the data set for station 08NG076. This station was selected as the proxy for Beaver Creek, as it was determined as the best fit during the regional analysis. The results are presented below.

The Lognormal distribution had the best fit with the dataset; the distributions within **Table 1** are listed in order of best fit to worst fit. **Figures 1, 2, and 3** are graphs of the top three distributions.

Table 1: Results of Frequency Analysis on Dataset for Station 08NG076

Distribution	Q200	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Lognormal (Maximum Likelihood)	32.406	8.33	31.28	299.295	295.471
Gumbel (Maximum Likelihood)	28.25	8.33	20.67	300.123	296.299
Gamma (Maximum Likelihood)	27.213	8.33	15.83	300.657	296.833
Inverse Gamma (Maximum Likelihood)	43.138	8.33	9.12	301.759	297.935
GEV (Maximum Likelihood)	32.79	8.33	4.54	303.155	297.419
3-parameter lognormal (Maximum Likelihood)	31.272	8.33	4.54	303.156	297.42
Generalized Gamma (Maximum Likelihood)	31.13	8.33	4.52	303.165	297.428
Log-Pearson type 3 (Méthode SAM)	31.115	8.33	4.52	303.165	297.429
Pearson type 3 (Maximum Likelihood)	28.979	8.33	3.91	303.452	297.715
Weibull (Maximum Likelihood)	25.816	8.33	1.03	306.118	302.294
Exponential (Maximum Likelihood)	44.052	8.33	0.03	313.506	309.682
Normal (Maximum Likelihood)	23.948	8.33	0.02	314.456	310.632

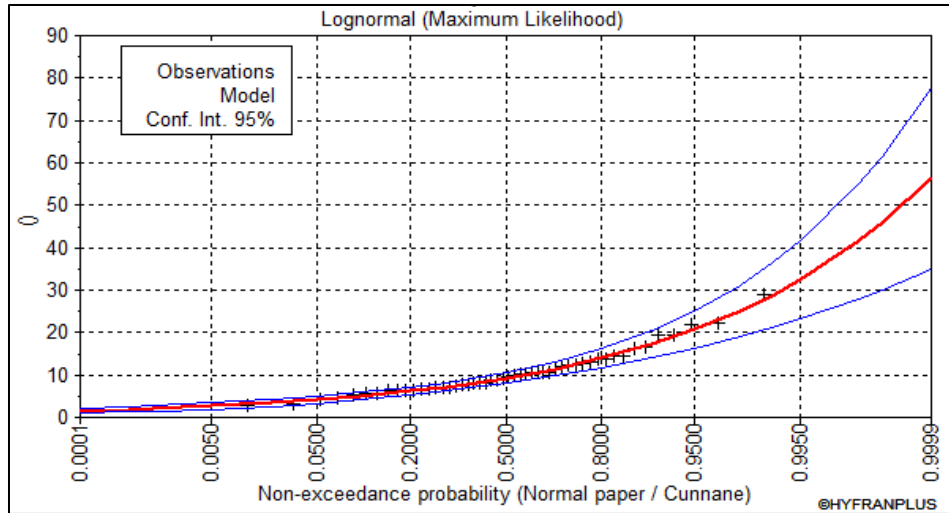


Figure 1: Graph Showing Lognormal Distribution

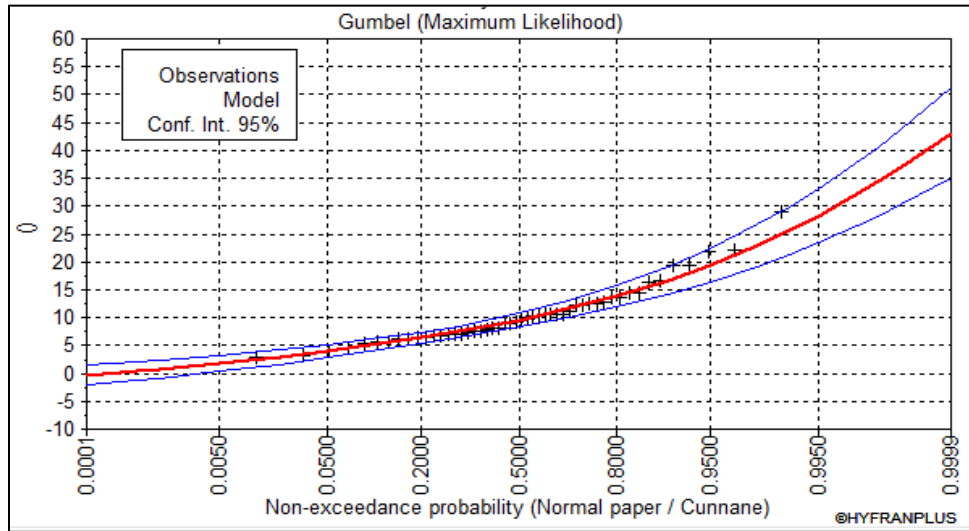


Figure 2: Graph Showing Gumbel Distribution

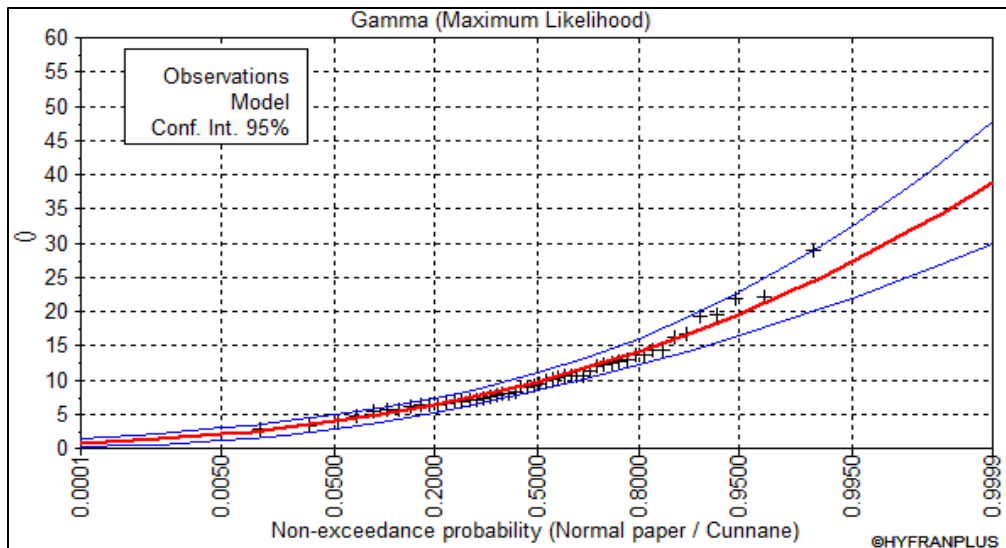


Figure 3: Graph Showing Gamma Distribution

The results of the frequency analysis for the 200-year return period for the remaining eight stations within the pooling group are presented throughout the tables (**Table 2 – Table 9**) below.

Table 2: Results of Frequency Analysis on Dataset for Station 08LC040

Distribution	Q200 for Station 08LC040	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	B C: Bayesian Information Criterion	A C: Akaike Information Criterion
Inverse Gamma (Maximum Likelihood)	22.908	7.69	36.92	232.374	228.51
Lognormal (Maximum Likelihood)	16.614	7.69	32.62	232.621	228.758
GEV (Maximum Likelihood)	20.031	7.69	7.1	235.67	229.875
Generalized Gamma (Maximum Likelihood)	18.956	7.69	6.89	235.731	229.936
Log-Pearson type 3 (Méthode SAM)	19.767	7.69	6.52	235.842	230.047
3-parameter lognormal (Maximum Likelihood)	17.991	7.69	6.16	235.956	230.161
Gumbel (Maximum Likelihood)	13.465	7.69	1.92	238.287	234.424
Gamma (Maximum Likelihood)	14.068	7.69	1.18	239.255	235.392
Exponential (Maximum Likelihood)	20.162	7.69	0.69	240.34	236.477
Weibull (Maximum Likelihood)	14.659	7.69	0.01	248.138	244.274
Normal (Maximum Likelihood)	13.13	7.69	0	272.336	268.472
Pearson type 3 (Conditional maximum likelihood)	17.146	7.69	N/D	N/D	N/D

Table 3: Results of Frequency Analysis on Dataset for Station 08NE077

Distribution	Q200 for Station 08NE077	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Gumbel (Maximum Likelihood)	80.83	7.69	28.39	518.977	514.509
Inverse Gamma (Maximum Likelihood)	79.651	7.69	26.65	519.103	514.635
Lognormal (Maximum Likelihood)	75.442	7.69	20.03	519.674	515.206
Gamma (Maximum Likelihood)	71.8	7.69	8.2	521.461	516.993
GEV (Maximum Likelihood)	78.28	7.69	3.55	523.133	516.431
3-parameter lognormal (Maximum Likelihood)	78.686	7.69	3.28	523.294	516.591
Pearson type 3 (Maximum Likelihood)	76.421	7.69	3.23	523.321	516.619
Log-Pearson type 3 (Méthode SAM)	78.704	7.69	3.22	523.328	516.626
Generalized Gamma (Maximum Likelihood)	79.403	7.69	3.21	523.336	516.633
Normal (Maximum Likelihood)	68.15	7.69	0.21	528.794	524.325
Weibull (Maximum Likelihood)	68.216	7.69	0.03	532.787	528.318
Exponential (Maximum Likelihood)	121.144	7.69	0	550.376	545.908

Table 4: Results of Frequency Analysis on Dataset for Station 08NE087

Distribution	Q200 for Station 08NE087	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Lognormal (Maximum Likelihood)	21.92	7.69	26.09	338.943	334.595
Gumbel (Maximum Likelihood)	22.373	7.69	19.43	339.533	335.184
Gamma (Maximum Likelihood)	19.942	7.69	19.31	339.545	335.197

Inverse Gamma (Maximum Likelihood)	24.711	7.69	14.67	340.095	335.746
Pearson type 3 (Maximum Likelihood)	22.508	7.69	6.71	341.661	335.137
Generalized Gamma (Maximum Likelihood)	21.388	7.69	3.29	343.083	336.56
Log-Pearson type 3 (Méthode SAM)	21.499	7.69	3.29	343.085	336.562
3-parameter lognormal (Maximum Likelihood)	22.017	7.69	3.27	343.099	336.576
GEV (Maximum Likelihood)	21.363	7.69	2.54	343.602	337.079
Weibull (Maximum Likelihood)	18.604	7.69	0.53	346.748	342.399
Exponential (Maximum Likelihood)	31.104	7.69	0.47	346.98	342.631
Normal (Maximum Likelihood)	18.224	7.69	0.42	347.203	342.854

Table 5: Results of Frequency Analysis on Dataset for Station 08NE114

Distribution	Q200 for Station 08NE114	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Inverse Gamma (Maximum Likelihood)	40.977	7.69	33.35	319.978	316.115
GEV (Maximum Likelihood)	50.986	7.69	12.26	321.98	316.184
Generalized Gamma (Maximum Likelihood)	55.541	7.69	11.84	322.05	316.255
Log-Pearson type 3 (Méthode SAM)	46.204	7.69	10.63	322.265	316.469
3-parameter lognormal (Maximum Likelihood)	44.434	7.69	10.01	322.386	316.591
Gumbel (Maximum Likelihood)	36.676	7.69	9.86	322.415	318.551
Lognormal (Maximum Likelihood)	37.882	7.69	7.04	323.09	319.227
Pearson type 3 (Maximum Likelihood)	39.286	7.69	4.31	324.069	318.274
Gamma (Maximum Likelihood)	35.348	7.69	0.64	327.872	324.008
Exponential (Maximum Likelihood)	55.008	7.69	0.04	333.248	329.384

Weibull (Maximum Likelihood)	35.053	7.69	0	339.498	335.634
Normal (Maximum Likelihood)	33.371	7.69	0	342.675	338.811

Table 6: Results of Frequency Analysis on Dataset for Station 08NH016

Distribution	Q200 for Station 08NH016	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Lognormal (Maximum Likelihood)	16.343	8.33	26.07	259.835	255.785
Gamma (Maximum Likelihood)	13.954	8.33	22.74	260.109	256.058
Gumbel (Maximum Likelihood)	15.008	8.33	22.64	260.118	256.067
Inverse Gamma (Maximum Likelihood)	20.757	8.33	6.63	262.574	258.523
Pearson type 3 (Maximum Likelihood)	14.962	8.33	4.93	263.166	257.09
Log-Pearson type 3 (Méthode SAM)	15.2	8.33	3.97	263.6	257.524
Generalized Gamma (Maximum Likelihood)	15.089	8.33	3.96	263.606	257.53
3-parameter lognormal (Maximum Likelihood)	15.534	8.33	3.71	263.734	257.658
GEV (Maximum Likelihood)	15.497	8.33	3.16	264.053	257.977
Weibull (Maximum Likelihood)	12.93	8.33	2.03	264.942	260.891
Normal (Maximum Likelihood)	12.267	8.33	0.14	270.355	266.305
Exponential (Maximum Likelihood)	22.679	8.33	0.02	273.752	269.701

Table 7: Results of Frequency Analysis on Dataset for Station 08NM142

Distribution	Q200 for Station 08NM142	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Exponential (Maximum Likelihood)	9.563	8.33	30.56	157.607	153.824
Lognormal (Maximum Likelihood)	8.751	8.33	25.9	157.938	154.154
Gamma (Maximum Likelihood)	6.83	8.33	13.7	159.212	155.428
Gumbel (Maximum Likelihood)	6.788	8.33	9.64	159.915	156.132
Generalized Gamma (Maximum Likelihood)	7.914	8.33	4.14	161.607	155.931
Log-Pearson type 3 (Méthode SAM)	8.023	8.33	4.11	161.618	155.943
3-parameter lognormal (Maximum Likelihood)	8.147	8.33	3.93	161.71	156.034
Inverse Gamma (Maximum Likelihood)	14.107	8.33	3.77	161.791	158.008
GEV (Maximum Likelihood)	8.338	8.33	3.14	162.156	156.481
Weibull (Maximum Likelihood)	6.652	8.33	1.12	164.224	160.44
Normal (Maximum Likelihood)	5.934	8.33	0	180.384	176.6

Table 8: Results of Frequency Analysis on Dataset for Station 08NM171

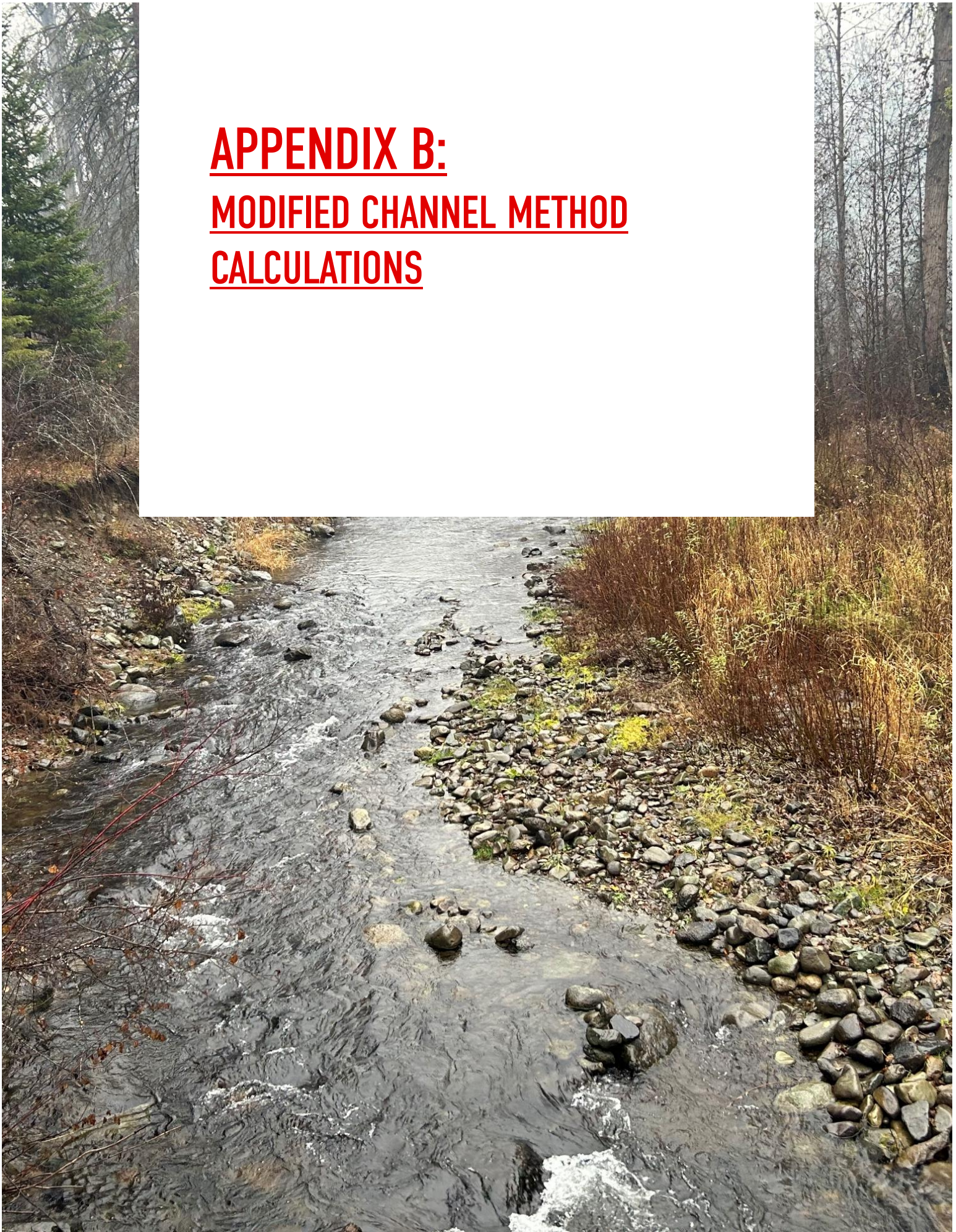
Distribution	Q200 for Station 08NM171	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Weibull (Maximum Likelihood)	33.165	8.33	26.46	341.475	337.611
Exponential (Maximum Likelihood)	56.092	8.33	19.8	342.056	338.192
Gamma (Maximum Likelihood)	38.494	8.33	18.5	342.191	338.327
Pearson type 3 (Maximum Likelihood)	49.003	8.33	11.31	343.175	337.38
Lognormal (Maximum Likelihood)	48.745	8.33	6.73	344.213	340.349

Gumbel (Maximum Likelihood)	41.237	8.33	5.36	344.67	340.807
Generalized Gamma (Maximum Likelihood)	32.392	8.33	3.74	345.388	339.593
Normal (Maximum Likelihood)	31.573	8.33	2.55	346.154	342.29
Log-Pearson type 3 (Méthode SAM)	40.554	8.33	2.2	346.452	340.656
3-parameter lognormal (Maximum Likelihood)	40.77	8.33	1.15	347.745	341.95
GEV (Maximum Likelihood)	32.598	8.33	1.13	347.777	341.981
Inverse Gamma (Maximum Likelihood)	70.515	8.33	1.07	347.889	344.026

Table 9: Results of Frequency Analysis on Dataset for Station 08NM173

Distribution	Q200 for Station 08NM173	P(Mi): A Priori Probability	P(Mi x): A Posteriori Probability	BIC: Bayesian Information Criterion	AIC: Akaike Information Criterion
Exponential (Maximum Likelihood)	3.467	11.11	75	67.013	63.072
Lognormal (Maximum Likelihood)	4.965	11.11	8.78	71.303	67.363
Gamma (Maximum Likelihood)	2.953	11.11	8.66	71.33	67.389
Weibull (Maximum Likelihood)	2.83	11.11	5.2	72.351	68.411
Generalized Gamma (Maximum Likelihood)	3.603	11.11	1.87	74.394	68.483
GEV (Maximum Likelihood)	6.056	11.11	0.24	78.464	72.553
Inverse Gamma (Maximum Likelihood)	23.652	11.11	0.2	78.876	74.935
Gumbel (Maximum Likelihood)	2.547	11.11	0.05	81.629	77.689
Normal (Maximum Likelihood)	2.218	11.11	0	100.833	96.892

APPENDIX B: MODIFIED CHANNEL METHOD CALCULATIONS



APPENDIX B: Modified Channel Calculations

METHODOLOGY

The modified channel method was used to determine the appropriate Manning's 'n' values for the study area. The method looks at different components that contribute to the overall roughness of specific land use types. The value for each component is determined through referencing standardized tables developed for the modified channel method, these include *Base n Value*, *Degree of Irregularity*, *Variation in Channel Cross Section*, *Effect of Observation*, and *Amount of Vegetation*. Each component is then summed, which is multiplied to the value for the *Degree of Meandering* to determine the Manning's 'n' value.

The tables used for reference in the analysis are provided.

Table 1. Base values of Manning's *n*
[Modified from Aldridge and Garrett, 1973, table 1; —, no data]

Bed material	Median size of bed material (in millimeters)	Base <i>n</i> value	
		Straight uniform channel ¹	Smooth channel ²
Sand channels			
Sand ³	0.2	0.012	—
	.3	.017	—
	.4	.020	—
	.5	.022	—
	.6	.023	—
	.8	.025	—
	1.0	.026	—
Stable channels and flood plains			
Concrete	—	0.012–0.018	0.011
Rock cut	—	—	.025
Firm soil	—	0.025–0.032	.020
Coarse sand	1–2	0.026–0.035	—
Fine gravel	—	—	.024
Gravel	2–64	0.028–0.035	—
Coarse gravel	—	—	.026
Cobble	64–256	0.030–0.050	—
Boulder	>256	0.040–0.070	—

¹ Benson and Dalrymple (1967).

² For indicated material; Chow (1959).

³ Only for upper regime flow where grain roughness is predominant.

Table 1, referenced during modified channel method to determine base values of Manning's 'n'

Table 2. Adjustment values for factors that affect the roughness of a channel
[Modified from Aldridge and Garrett, 1973, table 2]

Channel conditions		n value adjustment ¹	Example
Degree of irregularity (n_1)	Smooth	0.000	Compares to the smoothest channel attainable in a given bed material.
	Minor	0.001–0.005	Compares to carefully dredged channels in good condition but having slightly eroded or scoured side slopes.
	Moderate	0.006–0.010	Compares to dredged channels having moderate to considerable bed roughness and moderately sloughed or eroded side slopes.
	Severe	0.011–0.020	Badly sloughed or scalloped banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of channels in rock.
Variation in channel cross section (n_2)	Gradual	0.000	Size and shape of channel cross sections change gradually.
	Alternating occasionally	0.001–0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-sectional shape.
	Alternating frequently	0.010–0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-sectional shape.
Effect of obstruction (n_3)	Negligible	0.000–0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
	Minor	0.005–0.015	Obstructions occupy less than 15 percent of the cross-sectional area, and the spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp-edged angular objects.
	Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause the effects of several obstructions to be additive, thereby blocking an equivalent part of a cross section.
	Severe	0.040–0.050	Obstructions occupy more than 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause turbulence across most of the cross section.
Amount of vegetation (n_4)	Small	0.002–0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.010–0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season, growing along the banks, and no significant vegetation is evident along the channel bottoms where the hydraulic radius exceeds 2 ft.
	Large	0.025–0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft; bushy willows about 1 year old intergrown with some weeds along side slopes (all vegetation in full foliage), and no significant vegetation exists along channel bottoms where the hydraulic radius is greater than 2 ft.
	Very large	0.050–0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; bushy willow trees about 1 year old intergrown with weeds along side slopes (all vegetation in full foliage), or dense cattails growing along channel bottom; trees intergrown with weeds and brush (all vegetation in full foliage).
Degree of meandering ² (m)	Minor	1.00	Ratio of the channel length to valley length is 1.0 to 1.2.
	Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
	Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

¹ Adjustments for degree of irregularity, variations in cross section, effect of obstructions, and vegetation are added to the base n value (table 1) before multiplying by the adjustment for meander.

² Adjustment values apply to flow confined in the channel and do not apply where downvalley flow crosses meanders.

Table 2, referenced during modified channel method to determine different components of Mannings's 'n' for the main channel.

Table 3. Adjustment values for factors that affect roughness of flood plains

[Modified from Aldridge and Garrett, 1973, table 2]

Flood-plain conditions		<i>n</i> value adjustment	Example
Degree of irregularity (n_1)	Smooth	0.000	Compares to the smoothest, flattest flood plain attainable in a given bed material.
	Minor	0.001–0.005	Is a flood plain slightly irregular in shape. A few rises and dips or sloughs may be visible on the flood plain.
	Moderate	0.006–0.010	Has more rises and dips. Sloughs and hummocks may occur.
	Severe	0.011–0.020	Flood plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pastureland and furrows perpendicular to the flow are also included.
Variation of flood-plain cross section (n_2)		0.0	Not applicable.
Effect of obstructions (n_3)	Negligible	0.000–0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, or isolated boulders, occupy less than 5 percent of the cross-sectional area.
	Minor	0.005–0.019	Obstructions occupy less than 15 percent of the cross-sectional area.
	Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-sectional area.
Amount of vegetation (n_4)	Small	0.001–0.010	Dense growth of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation, or supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.011–0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation, or moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season.
	Large	0.025–0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation, or 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft, or mature row crops such as small vegetables, or mature field crops where depth of flow is at least twice the height of the vegetation.
	Very large	0.050–0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation, or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is below branches, or mature field crops where depth of flow is less than the height of the vegetation.
	Extreme	0.100–0.200	Dense bushy willow, mesquite, and saltcedar (all vegetation in full foliage), or heavy stand of timber, few down trees, depth of flow reaching branches.
Degree of meander (m)		1.0	Not applicable.

Table 3, referenced during modified channel method to determine different components of Manning's 'n' for floodplains.

The values in the tables have been determined through extensive field studies and validation methods. Using visual reference information and past knowledge of the Beaver Creek study area, the values for each of the different components were determined for the land use areas. **Tables 1-3** provide the values that were selected for each of the areas.

The roughness value for grassy area floodplains is 0.036, a standard value, as are the roughness values for gravel and paved surfaces.

Table 1: Modified Channel Method for Beaver Creek

Component- in St. Mary River Main Channel	Selected 'n'	Comment
Base n Value (nb) See Table 1	0.045	Bed material is all fairly uniform with large cobbles and a few boulders throughout. This was consistent for ~ 90% of the observed reach.
Degree of Irregularity (n1)	0.002	Minor: Very little erosion was observed along the banks.
Variation in Channel X-Sect (n2)	-	Not Applicable
Effect of Obstruction (n3)	-	Negligible: No islands or big sand bars were observed within the channel. No large vegetation (trees) within the channel.
Amount of Vegetation (n4)	0.002	Small: There was no vegetation within the main channel, no large bushes or plants, and small weeds were barely noticeable.
Degree of Meandering (m)	1.00	Minor: Ratio of the channel length to the valley length for the study area section is 1.0 to 1.2.
Channel Manning's 'n'	0.049	

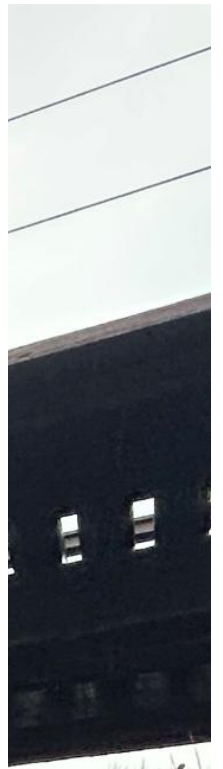
Table 2: Modified Channel Method for Forested Area Floodplain

Component- in Mark Creek Main Channel	Selected 'n'	Comment
Base n Value (nb)	0.025	Rock: The base of the forested area is rocky and hard with firm topsoil.
Degree of Irregularity (n1)	0.008	Moderate: There is some variation within the forest in terms of terrain slopes, vegetation sizes, and layout.
Variation in Flood Plain X-Sect (n2)	-	Not Applicable
Effect of Obstruction (n3)	0.025	Appreciable: There are lots of larger trees mixed within the dense vegetation and younger trees throughout the floodplain.
Amount of Vegetation (n4)	0.07	Very Large: The forest is densely vegetated with a mix of various trees and shrubs throughout the entire area. Very little empty space throughout.
Sinuosity of floodplain (m)	-	Not Applicable
Channel Manning's 'n'	0.128	

Table 3: Modified Channel Method for Willowy Area Floodplain

Component- in Forested Floodplain	Selected 'n'	Comment
Base n Value (nb)	0.025	Rock: Rocky base with firm topsoil consistent throughout this area.
Degree of Irregularity (n1)	0.006	Moderate: There are some rises and falls throughout the topography of the willowy (marsh) area.
Variation in Flood Plain X-Sect (n2)	-	Not Applicable
Effect of Obstruction (n3)	0.025	Appreciable: There are many willows and bushes close together throughout the entire area.
Amount of Vegetation (n4)	0.035	Large – The vegetation within this area is mature willows (8+ years) with tall grass and brushes intergrown throughout.
Sinuosity of floodplain (m)	-	Not Applicable
Channel Manning's 'n'	0.091	

APPENDIX C: CROSSING INFRASTRUCTURE INFORMATION



APPENDIX C: Crossing Infrastructure Information

There are 8 infrastructure crossings within the Beaver Creek study area; the images within this appendix were captured during the site visit and assisted in the creation of the crossings within the hydraulic model.



Photo C1: Railway Crossing Bridge



Photo C2: Highway 3B Outdated Bridge



Photo C3: Current Highway 3B Bridge



Photo C4: Columbia Gardens Arch Culvert



Photo C5: Kootenay Avenue Pedestrian Bridge



Photo C6: Beaver Street Pedestrian Bridge



Photo C7: Camp Tweedsmuir Pedestrian Bridge (Upstream)



Photo C8: Camp Tweedsmuir Walking Bridge



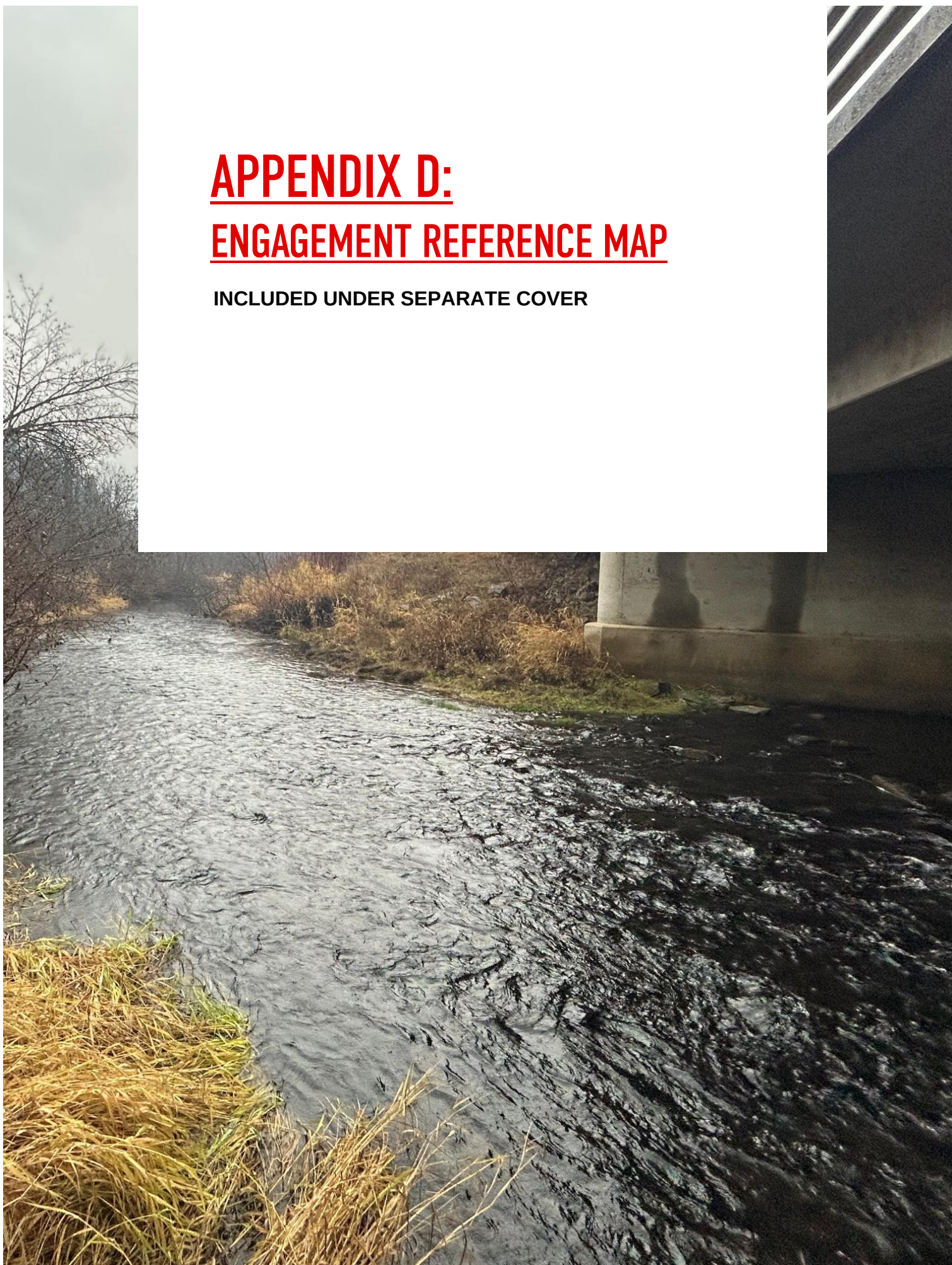
Photo C9: Bluebird Road Bridge



Photo C10: Jefferson Road Bridge

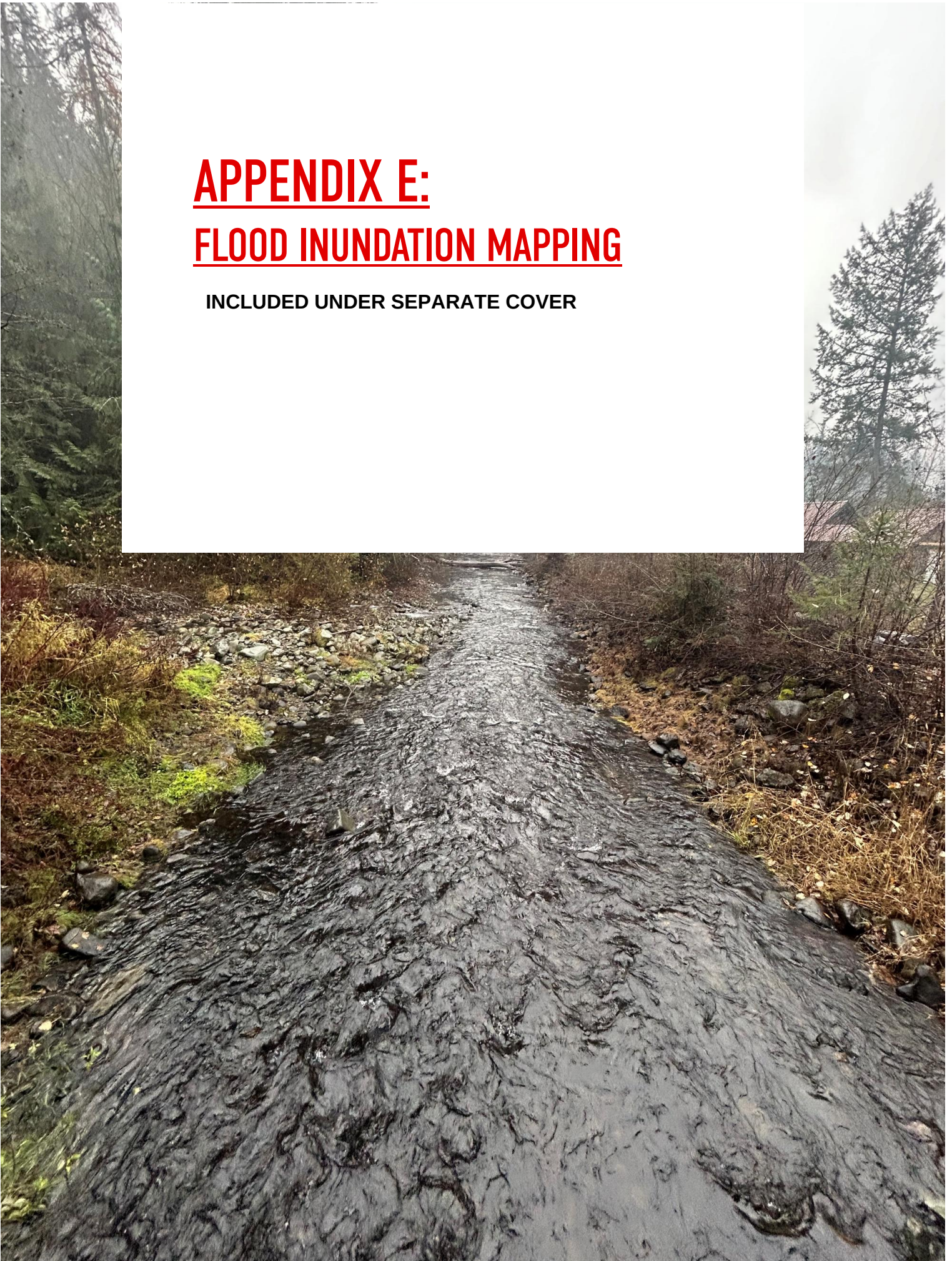
APPENDIX D: **ENGAGEMENT REFERENCE MAP**

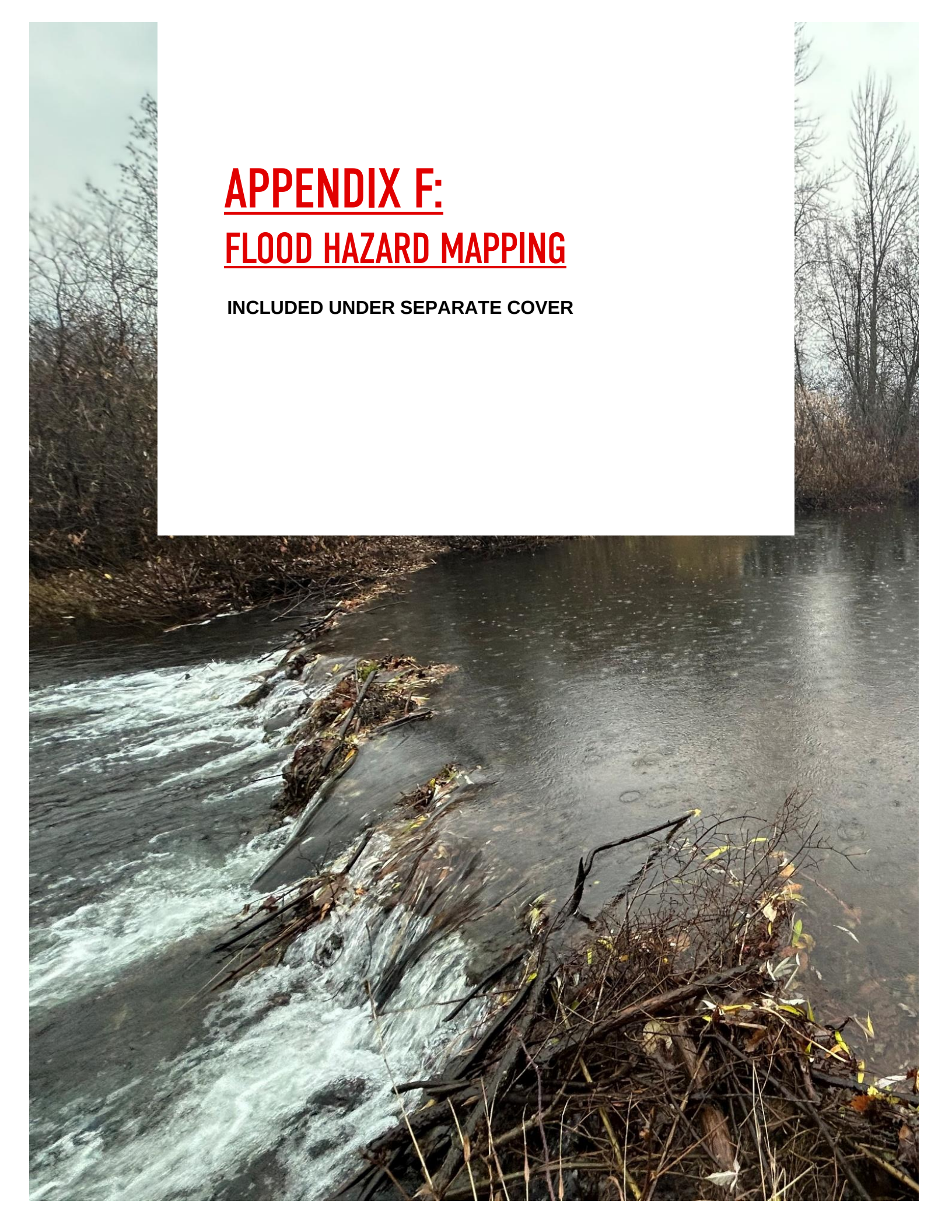
INCLUDED UNDER SEPARATE COVER



APPENDIX E: FLOOD INUNDATION MAPPING

INCLUDED UNDER SEPARATE COVER





APPENDIX F: **FLOOD HAZARD MAPPING**

INCLUDED UNDER SEPARATE COVER