# RDCK FLOODPLAIN AND STEEP CREEK STUDY

# Crawford Creek

**FINAL**
**March 31, 2020**

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# TABLE OF REVISIONS

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# EXECUTIVE SUMMARY

This report provides a detailed flood hazard assessment of Crawford Creek at the community of Crawford Bay, British Columbia. This creek was chosen as a high priority clear-water hazard amongst hundreds in the Regional District of Central Kootenay (RDCK) from a risk perspective because of its comparatively high hazards and consequences from flooding. This report describes hydrological conditions and details the methods applied to create scenario and hazard maps for Crawford Creek. This work is the foundation for possible future quantitative risk assessments or conceptualization of mitigation measures such as potential upgrades to existing dikes.

Flood mapping is used for estimating the extent and depth of different magnitude floods for application in community planning, policy development, and emergency response planning in areas subject to flood hazards. Results from a two-dimensional (2D) hydraulic model developed for about a 3 km length of Crawford Creek provides potential flood inundation extents and establishes flood construction levels (FCLs) based on the 200-year return period event or annual exceedance probability (AEP) of 0.5% and includes a freeboard allowance of 0.6 m for planning purposes.

The following types of maps were produced for Crawford Creek:

- Flood depth, velocity and intensity maps for the 20, 50, 200 and 500-year return period events
- Designated floodplain maps depicting the 200-year flood levels including a freeboard allowance of 0.6 m
- Aerial photograph interpretation and channel change mapping.

Flood mapping developed by BGC provides an update to historical floodplain mapping previously conducted for Crawford Creek. Flood extents are smaller than the 1987 designated floodplain and alluvial fan map created by the British Columbia Ministry of Environment and Parks (BC MEP). This difference is attributable to the application of a 2D instead of one-dimensional (1D) hydraulic model and access to lidar for a more detailed terrain model. Implementation of Crawford Creek’s FCLs and community planning for development outside of high hazard areas will lead to greater flood resiliency within the community of Crawford Bay. Flood mapping results are also provided digitally through a BGC web application called Cambio™.

Channel change mapping conducted by BGC indicates that the Crawford Creek channel is highly dynamic, indicating that the flood hazard assessment and modelling should be updated over time. Furthermore, the assumptions made on changes in runoff due to climate change will likely need to be updated periodically as scientific understanding evolves.

Table E-1 provides key observations derived from the hazard assessment.

**Table E-1. Summary of key hazard assessment results.**

| Process | Key Observations |
|---|---|
| Bank erosion and channel changes | • Aerial photo interpretation and channel change mapping results indicate that the identified channel reaches have experienced bank erosion resulting in lateral migration and deposition over the reviewed period (1958-2019). The dikes installed on the west bank (within Reach-7) has controlled the magnitude of these changes and promoted the stabilization of a section of the fan. However, there is an indication of ongoing erosion in several sections within the study area. High flows have the potential to exacerbate these existing processes. The sites where bank erosion has the potential to continue are: 1) where shear stresses concentrate (e.g., outside of meander bends); and, 2) where the banks are unprotected and erodible.<br>• The dikes are located outside of a meander bend and therefore exposed to high shear stresses that may compromise the integrity of these structures. As the stability of the dikes was not included in this assessment, further studies should focus on a detailed investigation of the stability of the dikes, including a bank erosion hazard assessment. |
| Clear-water inundation | • In general, modelling results and predicated floodplain extents are smaller than the results presented in historical floodplain mapping (BC MEP, 1987). These differences are attributable to a more detailed terrain model achieved from access to lidar data, the use of a 2D rather than 1D model and a lower design flows for the present work.<br>• Overbank flow is observed upstream of the Highway 3A Bridge pooling behind the highway embankment and overtopping it for even the 20-year return period flood.<br>• Flooding may occur outside the defined floodplain boundary. |
| Hydraulic Structures (Bridges) | • The Highway 3A bridge and the pedestrian footbridge over Crawford Creek are at no risk of inundation during the 200-year return period flood.<br>• There is approximately 0.6 m of clearance between the lower chord of the pedestrian bridge and the 200-year return period flood.<br>• There is greater than 3 m of clearance between the lower chord of the Highway 3A bridge and the 200-year return period flood. |
| Flood Protection Structures (Dikes) | • The dikes along Crawford Creek's west bank are in poor repair and as such a section was removed from the terrain in the hydraulic model. Details are provided in Appendix E.<br>• Outside of the removed section, the dike crest is typically around 0.4 m above the 200-year flood elevation.<br>• The section of dike located approximately 350 m upstream (north) of the Highway 21 bridge overtops in the 500-year return period event indicating that potential overtopping of the dikes occurs under a 500-year flood on Crawford River with accounting for climate change.<br>• Some floodplain inundation occurs due to flow between existing gaps in the dikes. |

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# 1. INTRODUCTION

The Regional District of Central Kootenay (RDCK, the District) is located in a mountainous region in southeastern British Columbia (BC) that is subject to damaging floods that have resulted in impacts to communities and infrastructure. In 2018, RDCK retained BGC Engineering Inc. (BGC) to carry out a regional geohazard risk prioritization study for the District (BGC, March 31, 2019). Supported by National Disaster Mitigation Program (NDMP) funding, the objective of the study was to characterize and prioritize flood and steep creek (debris-flood and debris-flow) geohazards. Through the regional study, BGC identified and prioritized 427 flood and steep creek hazard areas within the RDCK, of which, six floodplains and ten alluvial fans in the District were selected for further detailed assessment (Table 1-1, Figure 1-1).

**Table 1-1. List of study areas.**

| Site Classification | Geohazard Process | Hazard Code | Jurisdiction | Name |
|---|---|---|---|---|
| Floodplain | Clear-water Flood | 340 | Village of Salmo | Salmo River |
| | | 372 | Village of Slocan | Slocan River |
| | | 393 | Town of Creston | Goat River |
| | | 408 | RDCK Electoral Area A | Crawford Creek |
| | | 375 | Electoral Area K | Burton Creek |
| | | 423 | Village of Kaslo | Kaslo River |
| Steep Creek | Debris Flood | 212 | RDCK Electoral Area F | Duhamel Creek |
| | | 252 | RDCK Electoral Area F | Kokanee Creek |
| | | 248 | RDCK Electoral Area D | Cooper Creek |
| | | 137 | RDCK Electoral Area H | Wilson Creek |
| | | 242 | RDCK Electoral Area E | Harrop Creek |
| | | 95 | RDCK Electoral Area K | Eagle Creek |
| | | 238 | RDCK Electoral Area F | Sitkum Creek |
| | Hybrid Debris Flood/Debris Flow | 116 | RDCK Electoral Area E | Procter Creek |
| | | 251 | RDCK Electoral Area E | Redfish Creek |
| | Debris Flow | 36 | RDCK Electoral Area A | Kuskonook Creek |

The six clear-water hazard areas were prioritized either for development of new flood maps or modernization of existing historical flood maps. Flood maps provide information on the hazards associated with defined flood events, such as water depth, flow velocity, and the probability of occurrence. These maps are critical decision-making tools for local and regional governments to inform flood mitigation, land use planning, emergency management, and public awareness. Generally, the historical flood maps in the District are 20 years out-of-date and lack consideration of additional hydrological data, changes in land use such as urban development or the impacts of climate change. In response, updated floodplain mapping was conducted by BGC for each of the six prioritized clear-water hazard areas and provided under separate cover along with digital deliverables through a BGC web application called *Cambio Communities*™[1].

This report details the approach used by BGC to conduct detailed floodplain mapping for Crawford Creek located at the community of Crawford Bay, BC (Drawing 01). Crawford Creek is a tributary to Crawford Bay/Kootenay Lake and has an approximate watershed area of 182 km$^2$. Crawford Creek poses a flood hazard to properties and infrastructure constructed on the floodplain of the creek within the community of Crawford Bay. The floodplain is defined as the land adjacent to the river that extends from the banks of the channel to the base of the valley walls.

Flood mapping developed by BGC provides an update to historical floodplain mapping conducted previously for Crawford Creek in 1987 by the British Columbia Ministry of Environment and Parks (BC MEP). The BGC update is based on a two-dimensional (2D) hydraulic model, which was developed for about a 3 km length of the creek using methods described in Section 4. Modelling results described in Section 5 provide estimated flood inundation extents and establishes flood construction levels (FCLs) based on the 200-year return period event or annual exceedance probability (AEP) of 0.5% and includes a freeboard allowance of 0.6 m for planning purposes.

An outcome of the study is an improved basis for community planning, bylaw development, and emergency response planning in developed areas subject to flood hazards, with consideration of climate change. Recommendations are provided in Section 6 and include considerations for next steps from the study such as possible future quantitative risk assessments (QRAs) or conceptualization of mitigation measures such as potential upgrades to existing dikes.

BGC is providing a summary report for the entire assessment, *RDCK Floodplain and Steep Creek Study Summary Report* (referred to herein as the “Summary Report”). Readers are encouraged to read the Summary Report to obtain context about the objectives, scope of work, deliverables, and recommendations of the larger study.

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[1] *www.cambiocommunities.ca.*

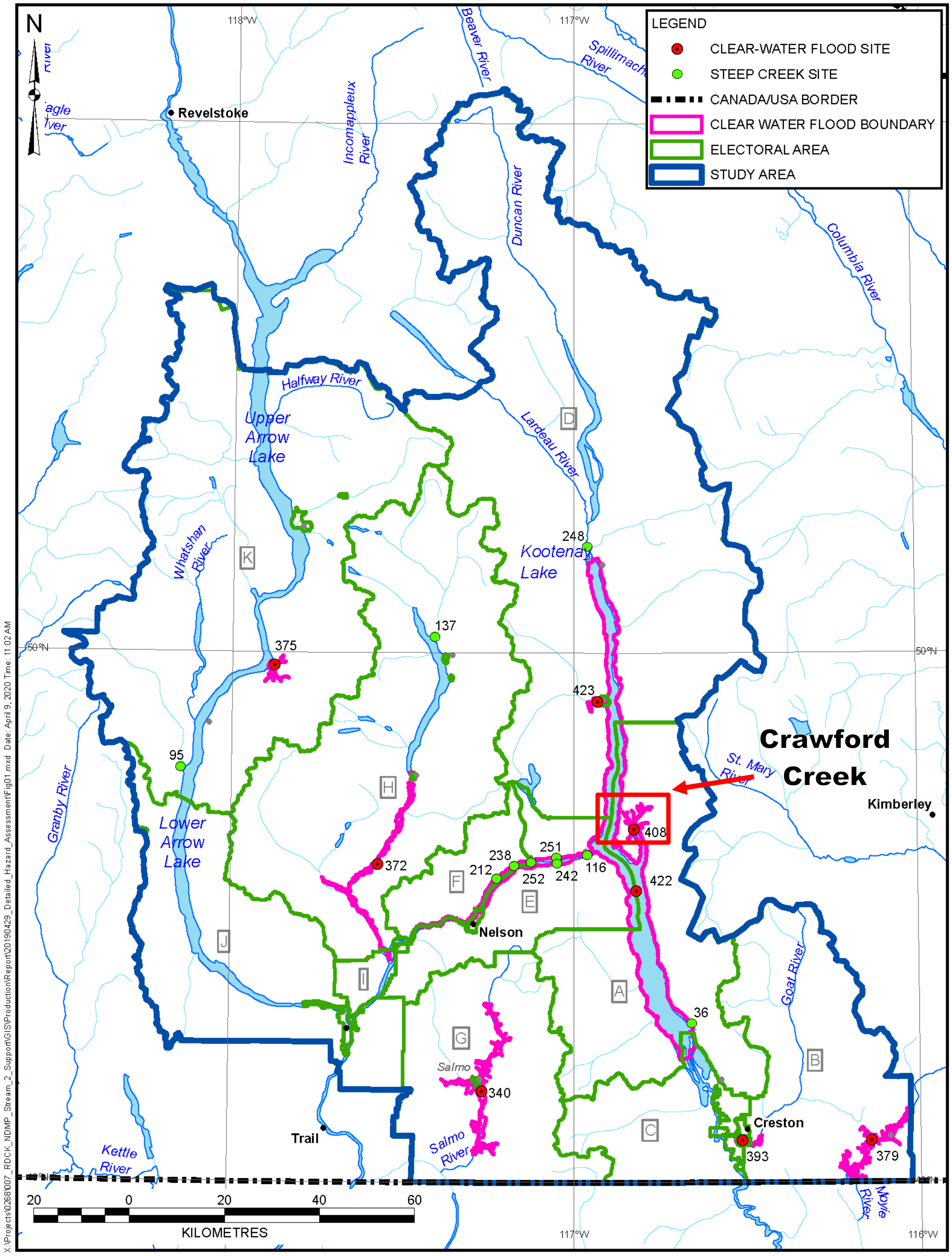

**Figure 1-1.** **Hazard areas prioritized for detailed flood and steep creek mapping. Site labels correspond to hazard identification numbers in Cambio Communities. Crawford Creek (No. 408) is labelled on the figure.**

### 1.1. Scope of Work

BGC's scope of work is outlined in the proposed work plan (BGC, May 24, 2019), which was refined to best meet RDCK's needs as the project developed (BGC, November 15, 2019). It was carried out under the terms of contract between RDCK and BGC dated June 20, 2019. The work scope was funded by Emergency Management BC (EMBC) and Public Safety Canada under Stream 2 of the NDMP.

For Crawford Creek, the scope of work included:

- Characterization of the study area including regional physiography and hydroclimate, and local watershed characteristics, geology and site characteristics.
- Development of a comprehensive site history of floods and mitigation activity.
- Compilation of data and baseline analyses required as inputs for flood geohazards assessment. This included topographic and river bathymetry data collection, terrain, hydrologic, hydraulic, and fluvial geomorphologic analyses, and consideration of climate change impacts.
- Complete hazard mapping and assessment according to provincial and national standards including mapping of inundation areas, flow velocity, and flow depth for a spectrum of return periods. Dike breach scenarios were not included.
- Integrate flood mapping results with the regional study and disseminate flood hazard mapping and data in web-accessible formats amenable to incorporation into policy and risk-informed decision making.

The study scope was informed by Engineers and Geoscientists of British Columbia (EGBC, 2018) professional practice guidelines, *Legislated Flood Assessments in a Changing Climate in BC,* and EGBC (2017) guidelines for flood map preparation. The assessment is consistent with the *Federal Floodplain Mapping Framework* (Natural Resources Canada [NRCan], 2017). Within the NRCan framework, this study provides the foundation to risk assessment and mitigation (Figure 1-2).

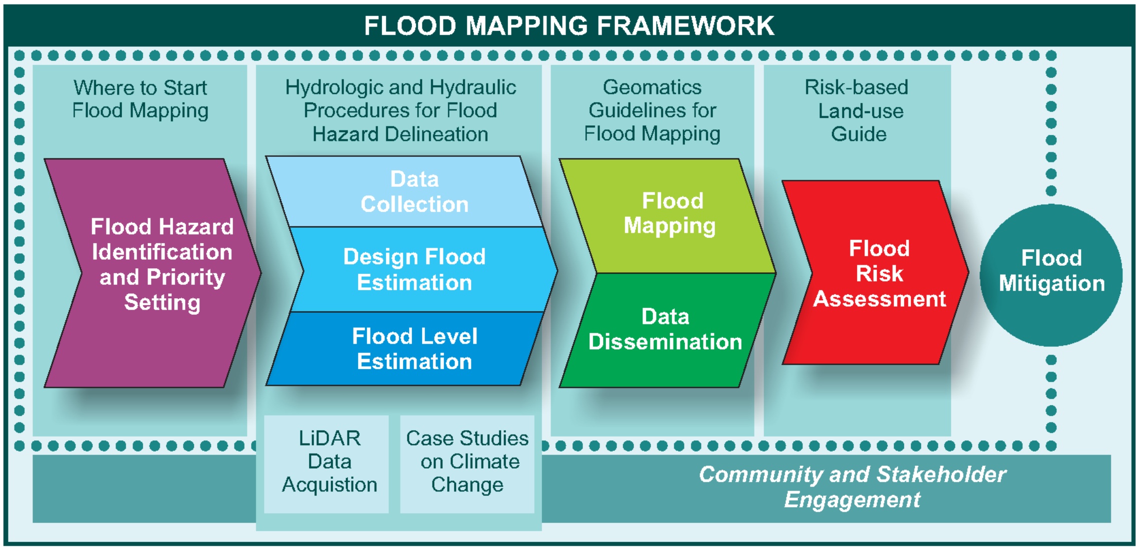

Figure 1-2. Federal Flood Mapping Framework (NRCan, 2017).

## 1.2. Terminology

This assessment uses specific hazard terminology provided in Appendix A.

## 1.3. Deliverables

The deliverables of this study include this assessment report and digital deliverables (hazard maps) provided via the Cambio web application and as geospatial data provided to RDCK.

This report is best read with access to a web application. Cambio displays the results of both the NDMP Stream 1 and Stream 2 studies. The application can be accessed at www.cambiocommunities.ca, using either Chrome or Firefox web browsers. Section 5 of the Methodology Report provides a Cambio user guide.

## 1.4. Study Team

This study was multidisciplinary. Contributors are listed below and primary authors and reviewers are listed in Table 1-2.

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# 2. STUDY AREA CHARACTERIZATION

The following section provides a characterization of the study area including physiography, hydroclimatic conditions and projected impacts of climate change, glacial history and surficial geology, and a description of Crawford Creek and its floodplain.

## 2.1. Physiography

Crawford Creek has a watershed area of 182 km$^2$ and discharges directly into Crawford Bay/ Kootenay Lake approximately 2 km after flowing through the community of Crawford Bay (Drawing 01). The community of Crawford Bay is located within the floodplain of the lower reaches of Crawford Creek. There is a wetland complex formed around Beaver Creek, which along with Beaver Pond mitigates flooding when Crawford creek overtops its banks and acts as the western boundary of Crawford Creek's flood plain. The headwaters are located in the Purcell Mountains, on the southern slope of Mount Loki and Mount Baldr at an elevation of approximately 2,500 m. Crawford Creek flows in a southward direction before flowing into Crawford Bay / Kootenay Lake at an approximate elevation of 531 m.

The Crawford Creek watershed lies in the Northern Columbia Mountains physiographic region. More specifically, the watershed spreads across a portion of the Central Columbia Mountains ecosection[2], which is drained by the north-flowing Columbia River. The ecosection is characterized by high ridges and mountains composed of a variety rock types including sedimentary, metamorphic, gneiss and granitic. Precipitation is high in this region with moist Pacific air moving over these mountains from west to east and up from the Columbia basin. Intermittently during the winter, cold arctic air reaches this area, however, it does not typically remain for long. Vegetation in the lower slopes and valleys is mostly moist Interior Cedar – Hemlock forests. Vegetation in the upper slopes is typically dominated by moist Engelmann Spruce – Subalpine Fir forests (Demarchi, 2011). The Crawford Creek watershed boundary is presented in Drawing 02 and watershed characteristics are listed in Table 2-1.

**Table 2-1. Watershed characteristics of Crawford Creek.**

| Characteristic | Value |
|---|---|
| Watershed area (km$^2$) | 182 |
| Maximum watershed elevation (m) | 2,627 |
| Minimum watershed elevation (m) | 530 |
| Watershed relief (m) | 2,092 |
| Watershed centroid elevation (m) | 1,181 |
| Average channel gradient through the modelled area (%) | 1.5 |

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[2] Ecosections are areas with minor physiographic and macroclimatic or oceanographic variations. There are 139 ecosections in British Columbia varying from pure marine units to pure terrestrial units.

## 2.2. Alluvial Fan Morphology

Below Wagar Road, Crawford Creek is confined within a narrow canyon for approximately 1 km before reaching the apex of an alluvial fan (Drawing 02)[3]. From the fan apex (which is about 40 m wide), the Crawford Creek main channel widens and flows south until the river the creek enters Crawford Bay, which is part of Kootenay Lake. At the distal section of the fan, the active area is about 220 m wide. Historically, the main channel has migrated and avulsed into new locations within this section (Figure 2-1). The active channel is actively aggrading and depositing large gravel bars, consistent with the fan being a depositional landform. Sediment sizes within the active channel range from 22 mm to 180 mm in diameter, with interstitial sand (Table 2-2). A wetland area is occupying the left side of the fan (Figure 2-1).

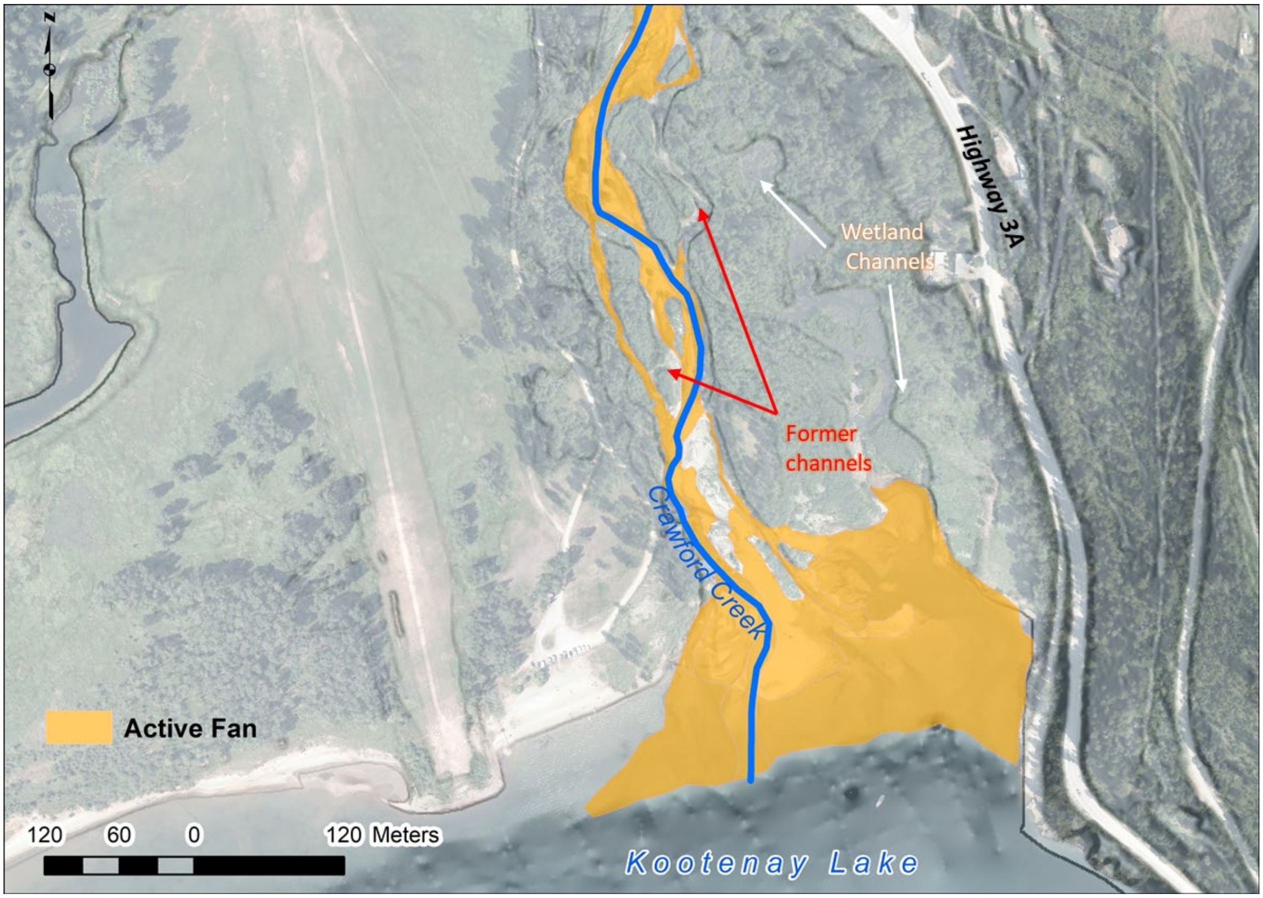

**Figure 2-1. Crawford Creek alluvial fan at its distal section. Red arrows show the former location of the Crawford Creek channel.**

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[3] A low-gradient cone-shaped depositional feature formed where the river becomes unconfined within a wide valley.

**Table 2-2. Crawford Creek sediment distribution from Wolman count data.**

| Grain Size | Crawford Creek |
|---|---|
| Location | Immediately north of Highway 3A bridge |
| Number of stones measured | 103 |
| $D_{95}$ (mm) | 128 |
| $D_{90}$ (mm) | 86 |
| $D_{84}$ (mm) | 66 |
| $D_{50}$ (mm) | 36 |
| $D_{16}$ (mm) | 19 |
| $D_{5}$ (mm) | 6 |

## 2.3. Hydroclimatic Conditions

Large-scale airflows moving in from the Pacific bring moist, marine air to the BC Interior. The Columbia Mountains, lying perpendicular to the prevailing winds, influence the distribution of precipitation and temperatures within the Columbia River watershed. Air masses rising over the Columbia Mountains produce an area of increased precipitation. The Crawford Creek watershed lies in this area. Precipitation takes the form of rain in the summer and deep snow in the winter. Cold air from the arctic infrequently enters this area because it is protected by mountain ranges from all sides.

The upper watershed (El. 2,627 m) of Crawford Creek receives a mean annual precipitation (MAP) of approximately 1,500 mm (based on 1961-1990 climate normal), whereas the community of Crawford Bay (El. 550 m) receives a MAP of approximately 750 mm due to the location of the community within the rain shadow of the Columbia Mountains. Averaged across the watershed the average MAP precipitation is 1,116 mm (Table 2-3). Average annual precipitation as snow (PAS) is 590 mm (snow water equivalent, SWE). The historical mean annual temperature (MAT) in the watershed is approximately 3ºC. The spatial distribution of historical average precipitation, temperature, and snowfall is depicted in Figure 2-2 based on climate data from Wang et al. (2016).

**Table 2-3. Historical (1961 to 1990) annual climate statistics for the Crawford Creek watershed.**

| Variable | Mean Annual Total (mm) | Percent of total annual precipitation (%) |
|---|---|---|
| Temperature | 3.0 ºC | - |
| Rainfall | 526 mm | 47 |
| Snowfall | 590 mm | 53 |
| Precipitation | 1,116 mm | 100 |

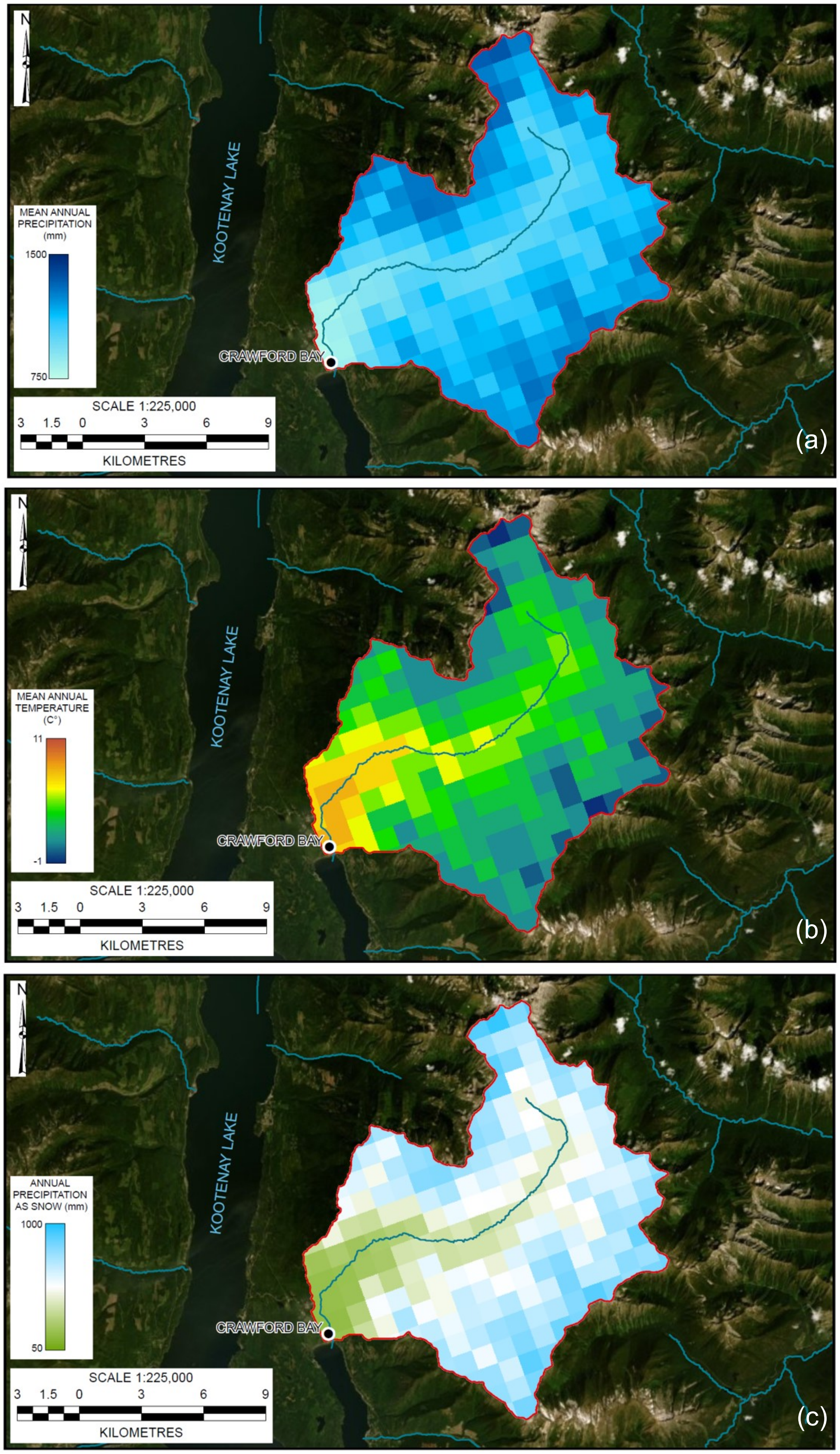


**Figure 2-2. Historical (1961 to 1990) mean annual precipitation (MAP) (a), mean annual temperature (MAT) (b), and precipitation as snow (PAS) (c) averaged over the Crawford Creek watershed (Wang et al., 2016).**

## 2.4. Climate Change Impacts

The MAT in the Crawford Creek watershed is projected to increase from 3.0ºC (based on the historical period 1961 to 1990) to 6.4ºC by 2050 (based on period 2041 to 2070) assuming representative carbon pathway 8.5 (RCP 8.5). MAP is projected to increase from 1,116 mm to 1,175 mm, while PAS is projected to decrease from 590 mm to 384 mm by 2050. Projected changes in climate variables from historical (1961 to 1990) to 2050 (2041 to 2070) conditions in the Crawford Creek watershed are presented in Table 2-4.

Extreme flood events in the Montane Cordillera are often associated with rain-on-snow events in the spring (Harder et al., 2015). Although the effects of climate change on precipitation are not clear, projected increases in temperature are expected to have the largest impact on annual minimum temperatures occurring in the winter months (Harder et al., 2015). The effects of temperature change differ throughout the region. High elevation regions throughout parts of the Montane Cordillera (e.g., Upper Columbia watershed) are projected to experience increases in snowpack, limiting the response in high elevation watersheds while lower elevations are projected to experience a decrease in snow water equivalent (Loukas & Quick, 1999; Schnorbus et al., 2014). Changes in streamflow vary spatially and seasonally based on snow and precipitation changes and topography-based temperature gradients. Researchers anticipate that streamflow will increase in the winter and spring in this region due to earlier snowmelt and more frequent rain-on-snow events, while earlier peak flow timing is expected in many rivers (Schnorbus et al., 2014; Farjad et al., 2016). Given the lack of literature and available data, there can be unforeseen consequences on the hydrology of clearwater creek watersheds.

**Table 2-4. Projected change (RCP 8.5, 2050) from historical (1961 to 1990) conditions for the Crawford Creek watershed (Wang et al, 2016).**

| Climate Variable | Projected Change |
|---|---|
| MAT | +3.4 ºC |
| MAP | +59 mm |
| PAS | -206 mm |

## 2.5. Glacial History and Surficial Geology

Between 2 million and 10,000 years ago ice sheets advanced and retreated into the Kootenay region (Turner et al., 2009). The final glaciation which ended approximately 10,000 years ago is responsible for many of the surficial materials in the area. South-flowing glaciers carved deep troughs which now hold Kootenay, Arrow and Slocan Lakes. Ice dammed the outlet to Kootenay Lake during deglaciation, which resulted in lake levels approximately 150 m higher than present, and the deposition of silts and clays in isolated terraces near the lake shore. The processes of erosion and deposition have continued since deglaciation creating the younger deposits, such as the fluvial materials found along the streams. Slopes around Crawford Creek are bedrock with a thin discontinuous cover of till and colluvium. Thicker fluvial sediments are deposited along the river valley and the fan at the head of Crawford Bay (Fulton et al., 1984).

# 3. SITE HISTORY

## 3.1. Area Development

Prior to European arrival, Crawford Bay was called k'upawi¢knuk and was the traditional territory of the Kutenai, a hunting and fishing people who formed the Ktunaxa First Nation. Settled by Europeans in the late 1800s, the community of Crawford Bay has historically had agricultural and residential development centered around the Crawford Creek fan near Crawford Bay/Kootenay Lake. Most of the farms in Crawford Bay became part of Kokanee Springs Golf Course in the 1960s. Logging in the watershed started in the late 1960's and continued through until the early 1980's when operations began to scale down. In 2018 Crawford Bay Regional Park was created by RDCK with 70 hectares of land purchased from Kokanee Springs Resort Ltd. This park includes a community beach formed by the alluvial fan of Crawford Creek. At present, the community of 304 people (Statistics Canada, 2016) has a local economy focused on tourism, and arts and crafts. Crawford Creek poses a flood hazard to this settlement as buildings in the community of Crawford Bay have been constructed with the floodplain of Crawford Creek. The recent floods of 2012 forced evacuations in the community.

## 3.2. Historical Flood Events

Crawford Creek has overtopped its banks on numerous occasions since the start of records. The first major flood recorded occurred in 1955, prompting the construction of gravel dikes along the banks of Crawford Creek between 1956-1957. These dikes are currently orphaned with no local authority responsible for maintaining them. Since the major flood event of 1955, several notable events have occurred including 1956, 1961, 1968, 1974, 1976, and the recent events of 2012 and 2013 (see Figure 3-1). The 2013 event was estimated to be approximately 75-year flood as recorded by the hydrometric station (Saint-Mary River below Morris Creek, 08NG077). Repair and reconstruction of the dikes occurred after several of these floods including 1961, 1974, and 2012. Gravel removals also occurred historically in an effort to maintain the hydraulic capacity of the channel. The provincial floodplain mapping program began in BC in 1974 aimed at identifying flood risk areas. This was in part due to the large Fraser River flood of 1972, which resulted in damage in the BC Interior. From 1975 to 2003, the province managed development in designated floodplain areas under the Floodplain Development Control Program. In 2003, the Program ended resulting in a significant change in how MFLNRO participated in land use regulation in flood-prone areas. The responsibility for developing and applying floodplain mapping tools was transferred to local governments, with the requirement that provincial guidelines be taken into consideration (EGBC, 2017).

Figure 3-2 provides a timeline summary of floods and mitigation history for Crawford Creek. The historical event inventory is based upon a variety of sources including newspaper articles, government records and consulting reports. Some sources may not be completely accurate or only provide partial records of flood events but are provided to present an overview of historic events.

**Figure 3-1.** 2013 Flooding of Crawford Creek 500 m north of Highway 3a. Photo: BC Ministry of Forest, Lands and Natural Resource Operations (2013).

## Flood History

*Crawford Creek stabilization, 2015 (Apex)*

*Flood protection berm, 2015 (Apex)*

*Dike breach along Crawford Creek, November 1961 (BC Ministry of Environment)*

*Canyon on Crawford Creek, 1897 (BC Archives)*

**May 12, 2014** - Snow avalanche caused washout in Crawford Creek

**June 20, 2013** - Heavy rain led to high water levels and washouts in creeks surrounding Crawford, including Crawford Creek; partial breach of an orphaned dike; berm at Crawford Creek affected

**June 6, 2012** - Flooding due to heavy precipitation during the normal June freshet; partial failure of a dike on Crawford Creek forced evacuations

**1981** - Flood event and potential damage to fish habitat

**March 31, 1976** - Dike breached due to seepage and erosion during flood event
**1974** - Highway 3 at Crawford Bay was cut by flooding of Crawford Creek; Kootenay Lake reaches highest level of 534.7 m
**October 25, 1972** - Flood threat to highway's dikes

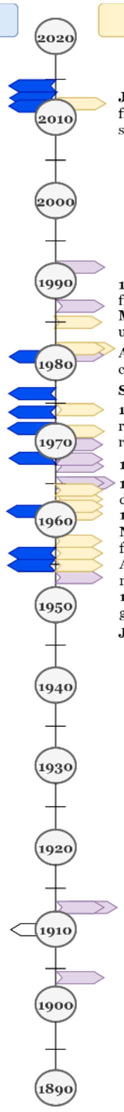
**June 1968** - Flood levels at Highway 3A reached approximately 542.9 m

**June 2, 1961** - Fan dike breached during exceptionally high freshet flow; banks overflowed and highway inundated

**January 24, 1956** - Flash flooding resulted in Crawford Creek overflowing along the west bank
**June 1955** - River overtopped bank at 90° bend; flooded land to southwest and overtopped highway

**1910** - Avulsion temporarily moved creek channel to west side of fan area

🟦 Flood (major flood)
🟫 Landslide
🟧 Debris flow or debris flood
▭ Channel location
🟨 Mitigation
🟪 Development event



## Mitigation

**June 2012** - Heavy equipment moved rock, clay and earth from neighboring property and stockpiles to fill the most serious breaches of dike

**1980s** - Department of Highways excavated material from bridge area
**May 1982** - Removal of 3 gravel bars from creek channel upstream of Highway 3A bridge

**April 13, 1982** - Bank protection and gravel dredging completed

**September 1974** - Dike repair and instream works
**1971** - Department of Highways cleared channel, will require clearing again before next freshet or might flood road and No. 6 hole of golf course

**1964** - Blockage in Crawford Creek
**1963** - Channel work at Crawford Creek to lessen flood damage
**1962** - Channel clearing undertaken
**November 7, 1961** - Repair work to dikes after spring flood
**April 3, 1958** - Flood mitigation completed near the mouth of Crawford Creek
**1956-1957** - Present channel cleared and deepened; gravel dikes built

**June 1955** - Department of Highways cleared channel

## Development

**1992** - Construction of highways bridge over Crawford Creek in order to eliminate sharp curve

**1987** - Mapping of Crawford Creek Alluvial Fan is completed to estimate 1:200 year flood

**1981** - Major bridgework involved replacing 3.6 m wide Crawford Creek bridge with a 4.3 m Bailey bridge

**1970** - A 30 m glulam and trestle structure on concrete abutments was completed over Crawford Creek
**1968** - Construction of grass airstrip; golf course opened
**1967** - Construction of the tent and trailer park and marina/ponds built
**1965** - Crawford Creek Bridge No. 427 constructed
**1965** - RDCK incorporated as a local government

**1954** - Major reconstruction completed on Woolgar Bridge No. 1 over Crawford Creek, including new log stringers and deck

**December 1913** - Crawford Creek School ready for occupancy
**October 1913** - New bridge completed over Crawford Creek

**1904** - Community of Crawford Bay was made official when the Post Office opened

Figure 3-2. Summary of recorded flood history, mitigation, and development history at the Crawford Creek.

## 3.3. Existing Flood Protection Structures

### 3.3.1. Bridges

Two bridges cross Crawford Creek within the study area: the Highway 3A bridge is located approximately 800 m upstream (north), from the creek mouth and pedestrian foot bridge is 1 km upstream from the creek mouth (Drawing 02). The dimensions of the pedestrian bridge and the Highway 3A bridge (Figure 3-3 and Figure 3-4), as given by the Ministry of Transportation and Infrastructure (MOTI) are presented in Table 3-1. As shown in Figure 3-4 there is rip-rap protecting the left bank of Crawford Creek immediately upstream of the Highway 3A bridge.

**Figure 3-3. Pedestrian Bridge crossing Crawford Creek, 200 m north of Highway 3A looking upstream. Photo: BGC, July 4, 2019.**

**Figure 3-4. Drone photo of Highway 3A bridge. Photo: Explore Survey Inc, September 25, 2019.**

**Table 3-1. Bridge dimensions as-built from MOTI.**

| Bridge | Top deck elevation (m) | Bottom deck elevation (m) | Pier Thickness (m) | Number of Piers | Shape of Piers | Deck Span (m) | Deck Width (m) |
|---|---|---|---|---|---|---|---|
| Highway 3A Bridge | 545.3 | 544.4 | 1.3 | 1 | Rectangular Concrete (with 5 pipe piles) | 58 | 13.5 |
| Pedestrian Bridge | 545.7 | 545.2 | n/a | n/a | n/a | 18[1] | 5[1] |

Note:

1. Value measured from Google Earth by BGC, February 12, 2020.

### 3.3.2. Dikes

Crawford Creek has a series of dikes along its right (west) bank spanning an approximate length of 700 m (Figure 3-5). These dikes are made primarily with gravel and cobbles taken from the river, are approximately 1.5m high and are intended to protect the community of Crawford Bay and Highway 3A from overbank flooding. The dikes were constructed in 1956 and have been repaired at least three times since their construction in 1961, 1974 and 2012. At present, the Crawford Creek dikes are deteriorating and overgrown with vegetation as shown in Figure 3-6. Given their age, it is doubtful that the dikes were ever engineered to a standard specification. Several gaps within the existing dike structure were also noted.

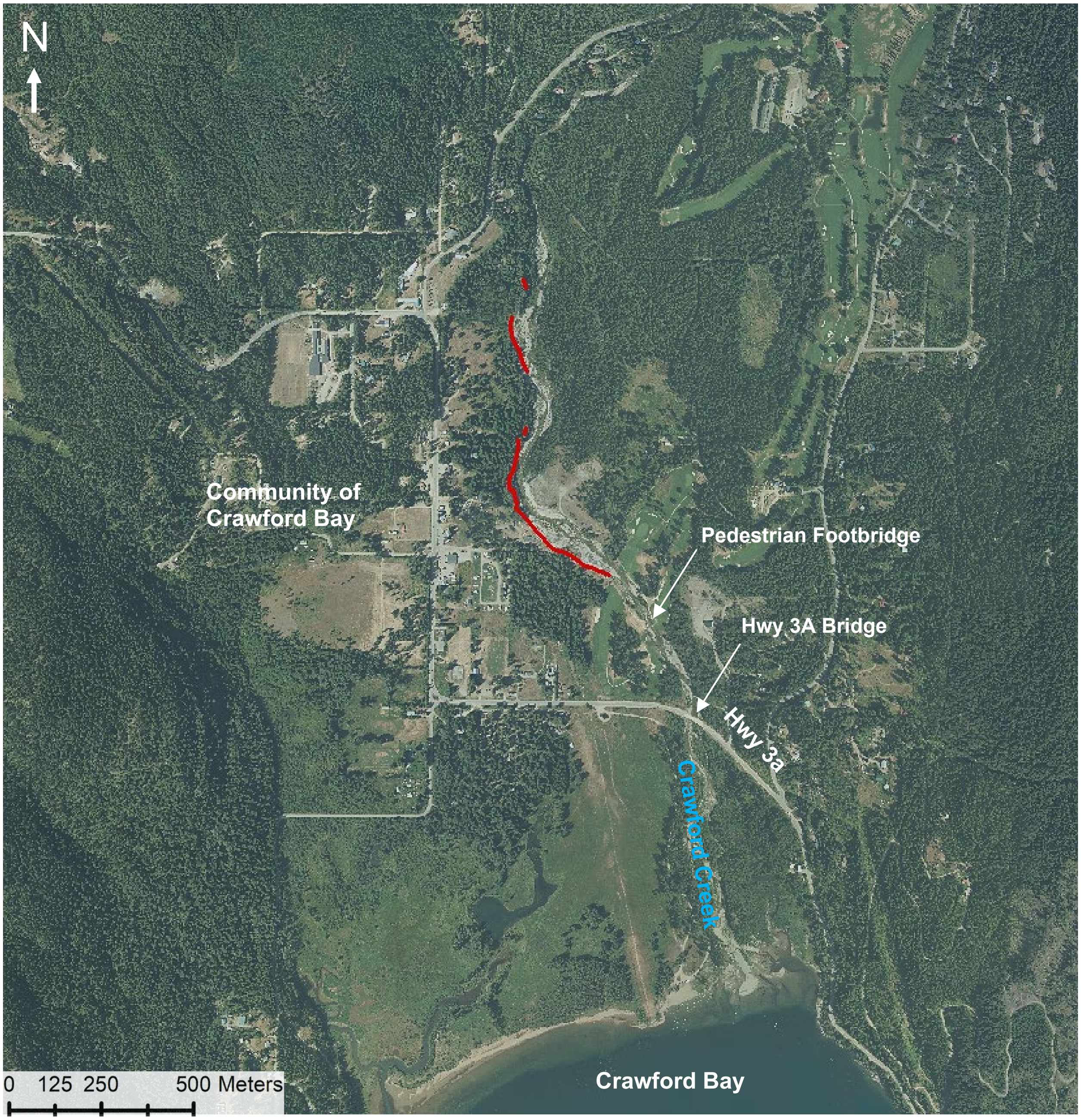


**Figure 3-5. Location of dikes (in red) on Crawford Creek. Line sections are in the provincial flood protection structure database (BC MFLNRO, 2017).**

**Figure 3-6. Overgrown section of dike along Crawford Creek's west bank, 600 m north of the Highway 3A. Photo: Explore, September 25, 2019.**

## 3.4. Previous Mitigations

Historically, gravel was excavated from Crawford Creek by the BC Department of Highways. The aim of gravel removal was to deepen the channel, reduce flood hazards, and increase the clearance of the Highway 3A Bridge crossing Crawford Creek. Dredging was continued until the mid-1980's. The dikes lining the west bank of Crawford Creek have also been periodically repaired following flooding, as noted in Section 3.3.2. Most recently, following flooding in 2012, heavy machinery was used to fill several breaches in the dike with a mix of rock, and clay.

## 3.5. Bank Erosion and Avulsion History

Lateral channel migration resulting from bank erosion and sediment deposition is a natural process in alluvial rivers. Channel migration may occur as gradual erosion at the outside of river bends, or as sudden widening of the river during floods. Gradual channel migration generally results from sediments being eroded along the outer bank of a meander bend and deposited as a point bar along the inside of the meander bend (Charlton, 2007). There are no historical studies addressing avulsion[4], bank erosion, and resulting channel changes within the entire study area. In 2015, APEX Geoscience Consultants Ltd. (APEX) conducted a hydrological and geomorphic

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[4] Lateral displacement of a stream from its main channel into a new course across its fan or floodplain (Oxford University Press (2008).

investigation within the reach located from above the golf course to Crawford Bay. The most important findings of this study, along with other relevant information obtained from BGC's review of the historical documentation, is provided in Table 3-2.

**Table 3-2. Relevant historical bank erosion and avulsion information.**

| Year of occurrence | Reported process | Reference |
|---|---|---|
| 1910 | Avulsion in 1910 resulted in the creek moving to the west side of the fan. The author reports that damage related to bank erosion, flooding and aggradation are annual problems. | Crawford Creek Alluvial Fan (Report 789), (BC MEP, 1987). |
| Late 1950's and 1960's | Channel widening and bank erosion following large flood in 1956 (Figure 3-2) | Crawford Creek Hydrogeomorphic Investigation and Identification of Habitat Restoration Opportunities (APEX Geoscience Consultants Ltd., 2015) |
| 2004 | Bank erosion common where mature vegetation is cleared. Flooding bank erosion and avulsion occur annually on the fan. | Crawford Creek Watershed assessment (Summit, 2004). |
| 2013 | Erosion of dike during high flows June 25, 2013 (Figure 3-7). | Crawford Creek dike erosion (Report 1849), (BC MFLNRO, 2013). |

Figure 3-7. June 25, 2013 photo of erosion of a historical berm, 1 km north of Highway 3A (BC MFLNRO, 2013).

# 4. METHODS

This section summarizes the assessment methodology applied to Crawford Creek. Additional details on the methodology applied are summarized in Appendices C, D, and E.

## 4.1. Field Data and River Bathymetric Surveys

### 4.1.1. Fieldwork and Site Investigations

Fieldwork on Crawford Creek was conducted on July 4, 2019 by BGC personnel (Elisa Scordo, P.Geo. and Marc Oliver Trottier, P.Eng.). Field work included Wolman sampling to characterize the grain size distribution of in-channel materials, and observations at bridge and flood protection structures (e.g., dikes, riprap armouring). The field work was also conducted to coordinate the survey extent and data collection with the survey crews.

### 4.1.2. Topographic Mapping

Detailed topographic data of the floodplain were available from a classified high-resolution lidar dataset obtained from RDCK and flown in October 2017. BGC was provided with tiles containing the classified point cloud and 1 m bare-earth Digital Elevation Model (DEM). Lidar coverage provided by RDCK for the entire study area is shown in Figure 4-1.

The lidar data were provided with the following coordinate system:

- Horizontal Datum: NAD83 CSRS
- Projection: UTM Zone 11 North
- Vertical Datum: CGVD 2013
- Geoid Model: CGG2013.

As part of the lidar acquisition, orthophotos were not collected. As a result, the classification of the raw lidar point cloud contained inaccuracies particularly around gravel bars and the location of the river shoreline. Lidar acquisition was also limited to above the waterline and channel changes occurred after the lidar was flown. In order to account for this, BGC collected additional ground and bathymetric survey data to capture in-channel features that were not classified in the lidar survey.

### 4.1.3. Ground and Bathymetric Surveying

BGC contracted Explore Surveys Inc. (Explore) to conduct a detailed survey of Crawford Creek and its entrance into Crawford Bay/Kootenay Lake (Drawing 03). The scope of work included surveying of the channel bed, bridges, and dikes. A combination of Static GNSS techniques, RTK, and RTN techniques were used to establish a precise, reliable Survey Control Network for the length of the project. The Survey Control Network was integrated with existing BC Survey Control and/or the Canadian Base Network. The survey data were provided in the 3TM NAD 83 (CSRS) UTM 11 North coordinate system with elevation in the CGVD2013 Vertical Datum.

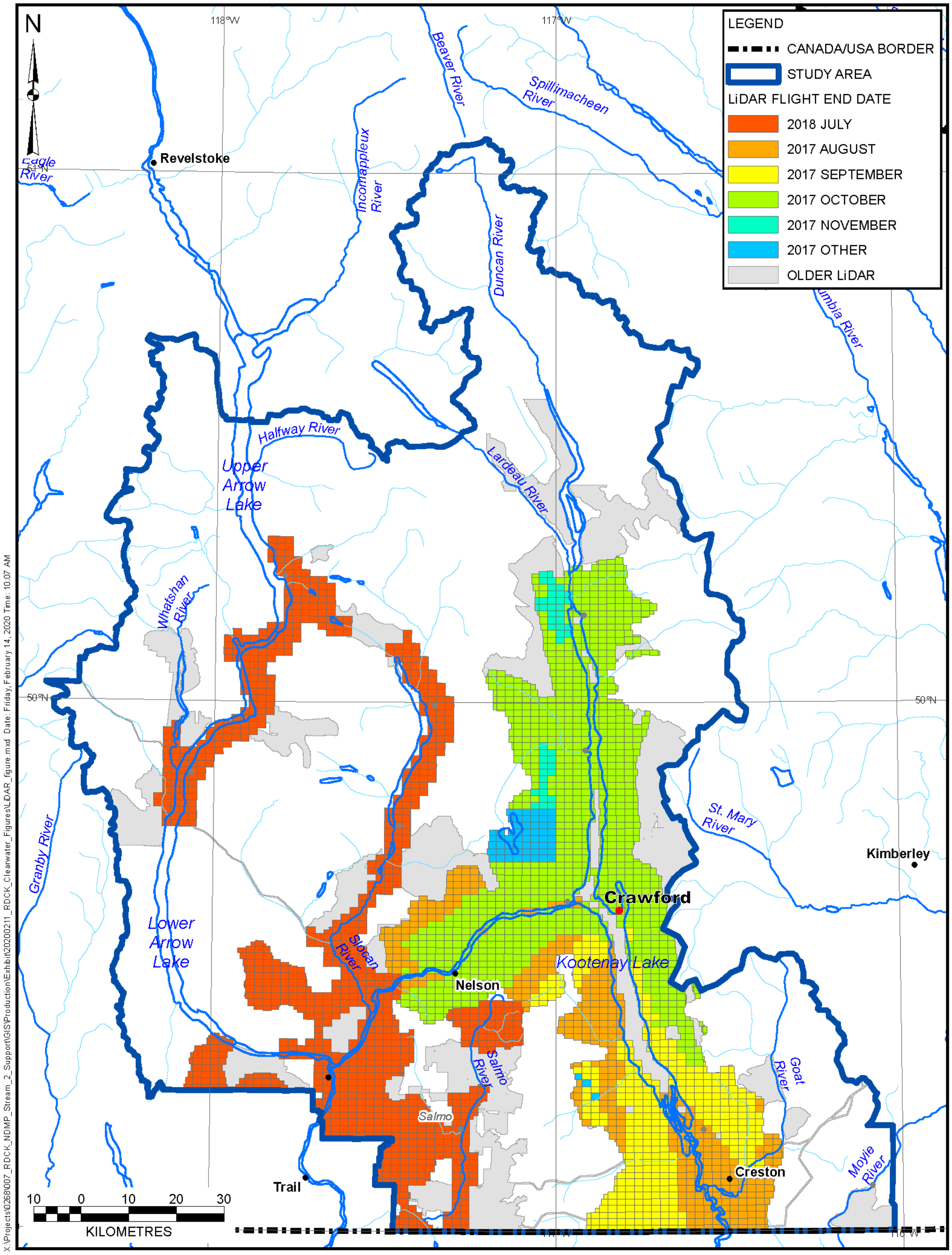

**Figure 4-1. Lidar coverage for clearwater study sites.**

The survey was conducted between August 17 and September 25, 2019. The survey covered approximately 3.5 km of Crawford Creek. The scope was expanded to include collection of the areas where the channel geometry had changed since the lidar data were flown in 2017. Surveying of the channels was completed using Static GNSS survey and in locations where the water depth was too deep to be waded safely, hydrographic surveying (sonar) from a boat was used. Drawing 03 provides a summary of locations collected using survey and sonar techniques. Bathymetric data were collected at an average density of 1 point per metre along cross-sections extending from bank to bank spaced every 5 to 10 m and perpendicular to the shoreline. Sonar data were collected along continuous transects collected at a density of 5 to 10 m.

The Highway 3A bridge was surveyed to collect details such as the length of the span, width of the bridge and a total of 700 m of dikes along the west bank were also surveyed including the crest elevation and length, type and general condition of the dike.

An offset was noted in the channel geometry between the collection of the lidar (2017) and the collection of the bathymetric survey (2019). This offset is attributed to sediment erosion and deposition and minor bank erosion that occurred during that 2-year period.

### 4.1.4. Survey Equipment, Accuracy and Processing Software

Table 4-1 provides a list of survey equipment and the reported accuracy. Hypack 2018 Hydrographic Software was used to correlate global position system (GPS) and hydrographic data together.

**Table 4-1. Summary of survey equipment.**

| Equipment Type | Reported Accuracy |
|---|---|
| | GPS |
| Trimble R10 GNSS<br>Trimble R8 GNSS<br>Trimble R6 GNSS | • Single Baseline: <30 km<br>• Horizontal: 8 mm + 1 ppm RMS<br>• Vertical: 15 mm + 1 ppm RMS<br>• Horizontal: 3 mm + 0.1 ppm RMS<br>• Vertical: 3.5 mm + 0.4 ppm RMS |
| Total Station | |
| Trimble SX10 Robotic Scanning Total Station | • Angular Accuracy: +/- 1" (0.3 mgon)<br>• EDM Range: 1 m – 5,500 m to single prism<br>• Scanning EDM Range: 1 m – 800 m<br>• Distance Accuracy: 1 mm + 1.5 ppm<br>• Distance Accuracy Scanning: 2 mm + 1.5 ppm |
| Hydrographic Equipment | |
| CEESCOPE Hydrographic System | • Depth Range: 0.20 m to 200 m<br>• Accuracy (Corrected for Sound Velocity): 0.01 m +/-0.1 % depth |

### 4.1.5. Terrain Creation

Following completion of the survey, BGC integrated the bathymetry data with the lidar bare-earth DEM to generate a continuous terrain model for use in hydraulic modelling (HEC-RAS 2D). The process to generate the terrain model from the topographic modelling and the bathymetric survey was as follows:

1. Elevation contours were generated along the banks of the surveyed channels from the ground classified lidar point cloud. The contours were clipped to the banks of the channels.
2. The survey data points and the clipped elevation contours along the channels were interpolated into a DEM representing the channel bathymetry where the surveying was performed. The interpolation was performed using the Topo to Raster tool within ArcGIS. This DEM was masked so that it only contained elevations within the channels and a portion of the banks.
3. The masked channel bathymetry DEM was merged back into the bare-earth DEM to form the complete terrain model.

The results of this process were reviewed, and adjustments made to remove artifacts from the process. Many of the artifacts encountered were due to changes in the channel alignment between the period of the lidar collection and the survey.

Hydraulic structures were not included in the terrain. Bridge decks for the Highway 3A and pedestrian bridge were removed from the DEM as they were sufficiently high above the 200-year flood water elevation and do not have an impact on flow.

## 4.2. Channel Change and Bank Erosion Analysis

Floods induce high shear stresses on channel banks, which can promote bank erosion. Non-cohesive materials such as sands and gravels are more susceptible to this process than cohesive banks. Standard hydraulic models to simulate floods do not consider bank erosion. BGC conducted a separate analysis to assess changes in the floodplain and channel and their potential influence on flooding.

Channel change mapping and bank erosion approaches using remote sensing have been widely used to detect variations in the position of channel geomorphic features (e.g., channels, banks, and bars) (Trimble & Cooke, 1991; Marcus, 2012). These methods have been reviewed and considered useful to quantify the rate of change over a study period (Lawler, 2006).

This section briefly describes the data and methods used to document planform channel changes within the study area and analyze the bank erosion processes observed between 1958 and 2019. It also outlines the limitations and uncertainties of the methodology. The investigated area comprises a 6.7 km long reach that extends from approximately 4 km above the fan apex to the toe of the alluvial fan at the shoreline of Crawford Bay (Drawing 04).

### 4.2.1. Data Sources

Aerial photographs and satellite imagery supported with lidar were used to assess historical changes in channel planform geomorphology and bank erosion within the Crawford Creek floodplain (Table 4-2). The channel mapping was also informed by the river bathymetric survey described in Section 4.1. The aerial photographs for 1958 and 2017 are included in Drawing 4.

**Table 4-2. Aerial photographs and satellite imagery used in the analysis.**

| Imagery | Year | Roll/Frame | Photo Number | Nominal Scale | Source |
|---|---|---|---|---|---|
| Aerial photographs | 1958 | BC2485 | 103-105 | Less than 1: 15,000 | BC Government |
| Aerial photographs | 1958 | BC2486 | 74-76 | Less than 1: 15,000 | BC Government |
| Orthophoto | 2017 | BC17507 | N/A | 1:6,250 | BC Government |

### 4.2.2. Methods

In this analysis, the following tasks were completed:

Data preparation:

This task involved the acquisition of historic aerial photographs and imagery for georeferencing and mosaics creation. All the imagery and photographs were georeferenced to the same coordinate systems (NAD 1983 CSRS UTM, Zone 11N).

Geomorphic analysis:

The geomorphic analysis involved three steps. First, distinct channel reaches were delineated (i.e., length of the channel with similar physical characteristics). These reaches were then used to quantify the average bank retreat in metres recorded in the analyzed period.

Second, the channel thalweg and planform were delineated. The channel planform refers to the form of a river as viewed from above (Charlton, 2007). The 2019 thalweg was generated from the river bathymetric survey data. The historical channel thalwegs were interpreted from the photographs and manually digitized on-screen. Third, geomorphic features were mapped using defined geomorphic criteria developed by BGC based on Howes and Kenk (1997) and Church (2006) (Table 4-3 and Table 4-4).

**Table 4-3. Geomorphic features used for geomorphic floodplain and channel mapping.**

| Feature | Type | Map Symbol | Description |
|---|---|---|---|
| Channel | Main-channel | Fmc | Flowing channel with distinct banks that carries most of the river discharge. This feature is always active. |
| | Side-channel | Fsc | Flowing channel with distinct banks that carries a portion of the river discharge less than the main-channel. This feature is active. |
| | Back-channel | Fbc | Abandoned-channel with distinct banks whose downstream end is connected to the river but whose upstream end is plugged. This feature is always active. |
| | Flood-channel | Ffc | Channel with distinct banks connected to a main- or side-channel only in overbank flood conditions. |
| | Abandoned-channel | Fac | Inactive channel remnant(s). No longer directly connected to active flow (e.g., oxbow lake). |
| Bars | Lateral and point-bars | Flb | Deposition and accumulation of sediments against the bank (lateral or side bars) and on the inside of a meander bend (point-bars). |
| | Mid-channel bar | Fmb | Feature characterized by the accumulation of sediments within the main channel. When the position of the bar become stable and vegetated during decades, they are commonly called islands. |
| Plain | Floodplain | Fp | Includes the level-ground area susceptible to overbank flow or flooding during high-flow events. |
| Fan | Alluvial fan/delta | Ff | A fan is a relatively smooth sector of a cone with a slope gradient from apex to toe up to and including 15°, and a longitudinal profile that is either straight, or slightly concave or convex (Howes and Kenk, 1997). |
| Terrace | Terrace | Ft, FGt<br>LGt | Flat or gently sloping areas bounded by an adjacent scarp. Fluvial terrace (Ft) deposits consist of channel deposits that may include some overbank materials. |

**Table 4-4. Levels of activity assigned to the geomorphic features.**

| Activity Class | Map Symbol | Description |
|---|---|---|
| Active | A | This indicates that the fluvial processes were active on the identified geomorphic feature at the time when the remote sensing data were collected. The floodplain and lateral, point or mid-channel bars are considered active until vegetation cover is established. Less than 75% of vegetation coverage or isolated patches of vegetation were classified as active. |
| Dormant/ Inactive | D | This indicates that there is no observable evidence of fluvial processes being active on the identified geomorphic feature at the time when the remote sensing data were collected. The floodplain and lateral, point or mid-channel bars are considered dormant when at least 75% of the mapped feature is covered by vegetation. |

### Channel Change and Bank Erosion Analysis

The channel banks and geomorphic features delineated in the previous stage were used to quantify net bank erosion between the observation periods. A spatial analysis using ArcGIS software by ESRI (Version 10.6.1) was applied to estimate the net change in riverbank positions (bank retreat) between each set of imagery. The following steps were completed:

- A numerical value of 1 (active) or 2 (dormant/inactive) was assigned to each mapped feature in the map attribute table. The values were determined based on the activity criteria described in Table 4-4. The general assumption was that unvegetated bars are active and would be submerged during bankfull conditions and, therefore, part of the active channel. A raster layer consisting of 1 and 2 values was created for each year of analysis.
- Then, the map algebra tool was used to subtract any two raster layers and estimate net change within the period. Negative values indicate bank erosion. Zero values indicate no change within the period and positive values indicate either bar stabilization, lateral accretion or deposition (Table 4-5).

**Table 4-5. Channel change classes.**

| Map Algebra Results | Class | Definition |
|---|---|---|
| -1 | Bank Erosion, Channel Migration | Lateral migration of the channel due to the removal of bank material has occurred at raster cell. |
| 0 | No Change | The channel features remained the same at the raster cell between the reviewed periods. |
| 1 | Stabilization, Bank Accretion | Two conditions are possible for this result. First, pre-existing channel bars have remained stable during the period, allowing for vegetation to grow (stabilization). Second, the fluvial processes acting during the reviewed timeframe have promoted the sideway deposition along channel meanders (lateral accretion). |

### 4.2.3. Limitations and Uncertainties

Some limitations of the interpretation of remote sensing data to the quantification of channel change include:

- The scale and resolution of available aerial photographs, which affects the level of detail that can be identified for a given year.
- The geometric distortion that results from terrain and imagery acquisition method (e.g. camera tilt in aerial photographs). These factors may result in a displacement of the geomorphic features from its true position.
- The degree to which the historical photographs represent relevant channel changes within the investigated timeframe to within tolerable levels of accuracy.

- Challenges related to the quantification of the error during the process. Possible sources of error in this analysis include scanning, georeferencing error, and on-screen digitizing errors.
- The discharge at the time of image capture. At higher discharges, most gravel bars would be inundated.

These errors were reduced in this study by applying common procedures including:

- Focusing on the central part of each aerial photograph
- Scanning the paper photographs at a high resolution
- Conducting geometric corrections on ArcGIS 10.6.1 software using the spline transformation tool, which is commonly used when local accuracy is wanted.

## 4.3. Hydrological Analysis

### 4.3.1. Flood Frequency Analysis

There are no hydrometric stations on Crawford Creek and therefore flood quantiles were estimated using regional flood frequency analysis (Regional FFA). The Regional FFA procedure is based on the index-flood method. For this project, the mean annual flood was selected as the index-flood, and dimensionless regional growth curves were developed from hydrometric station data to scale the mean annual flood to other return periods. The index-flood for each creek is determined from watershed characteristics using a regional and provincial ensemble of multiple regression models. Based on watershed characteristics the Crawford Creek watershed was assigned to the 4 East hydrologic region for watersheds less than 500 km$^2$. The flood quantile estimates were compared with historical estimates published by previous studies (BC MEP, 1987; Summit, 2004; and Obedkoff 2002). Details of the Regional FFA are presented in Appendix C.

### 4.3.2. Climate Change Considerations

Engineers and Geoscientists British Columbia (EGBC) offer guidelines that include procedures to account for climate change when flood magnitudes for protective works or mitigation procedures are required (EGBC, 2018). The impacts of climate change on peak discharge estimates in Crawford Creek were assessed using statistical and processed-based methods (Appendix D). The statistical methods included a trend assessment on historical flood events using the Mann-Kendall test as well as the application of climate-adjusted variables (mean annual precipitation, mean annual temperature, and precipitation as snow) to the Regional FFA model. The process-based methods included the trend analysis for climate-adjusted flood data offered by the Pacific Climate Impacts Consortium (PCIC).

## 4.4. Hydraulic Modelling

### 4.4.1. General Approach

The preparation of flood hazard maps requires the development of a hydraulic model. The two-dimensional (2D) hydraulic model HEC-RAS 2D (Version 5.0.7) was used to simulate the flood scenarios summarized in Table 4-6. HEC-RAS is a public-domain hydraulic modelling program developed and supported by the United States Army Corps of Engineers (Brunner & CEIWR-HEC, 2016). Each scenario was modelled with climate-change adjusted flows to represent projected future conditions in 2050 based on the average for the period 2041-2070 as described below.

**Table 4-6. Return period classes.**

| Return Period (years) | Annnual Exceedance Probability |
|---|---|
| 20 | 0.05 |
| 50 | 0.02 |
| 200 | 0.005 |
| 500 | 0.002 |

Within the Crawford Creek modelling extents there are two bridges. As model results indicated that the bridges provide sufficient of clearance for a 200-year flow event and would not affect flow, the bridge decks were removed from the HEC-RAS 2D model. Further details on modelling methods are presented in Appendix E and summarized in the sections below.

### 4.4.2. Model Inputs

Key model inputs include: (1) the topographic model to represent the floodplain and in-channel bathymetry; (2) the flood hydrology to represent peak flows for various return period; and (3) the boundary conditions at the upstream and downstream end of the study extent. Table 4-7 summarizes the key numerical modelling inputs selected for the HEC-RAS 2D model. Additional description of the topographic, flood hydrology and boundary conditions are provided in the sections below.

**Table 4-7. Summary of numerical modelling inputs.**

| Variable | HEC-RAS |
|---|---|
| Topographic Input | Lidar (2017); Bathymetry (2019) |
| Grid cells | Variable (2-30 m) |
| Manning's n | Main channel: 0.06, calculated using Jarrett's steep-creek equation (Jarrett 1984) ,Floodplain and overbank: varied based on landcover data (NALCMS, 2010) using Manning's n values from Chow (1959). |
| Upstream boundary condition | Steady Flow ($Q_{20}$, $Q_{50}$, $Q_{200}$ and $Q_{500}$) |
| Downstream boundary condition | Maximum water surface elevation (WSE) of Crawford Bay / Kootenay Lake (535 m) |

#### 4.4.2.1. Topographic Model

Following completion of the survey, BGC integrated the bathymetry data and surveyed cross sections with the lidar to generate a DEM for use in hydraulic modelling using the process described in Section 4.1.5.

#### 4.4.2.2. Boundary Conditions

The model domain covers a 3 km section of Crawford Creek starting 300 m below the bridge crossing for Wagar Road in a steep gorge above the fan apex. The downstream end of the model domain extends approximately 750 m out into Crawford Bay so that the lake level boundary condition does not affect the discharge through the Highway 3A bridge. The edges of the domain were set sufficiently far from the estimated maximum water level so as to not influence the results. The modelling extents, and the location of the upstream and downstream boundary conditions, are shown in Figure 4-2.

The upstream boundary for Crawford Creek was set as constant, steady inflow hydrograph using the climate-change adjusted peak flows summarized in Table 5-3. The downstream lake level boundary was at an elevation of 535 m, which is an intermediate scenario between BC Hydro's minimum and maximum flood scenarios, and 0.5 m above the peak recorded reservoir level (on July 4, 2012) since commissioning of the Libby Dam (BGC, January 15, 2020).

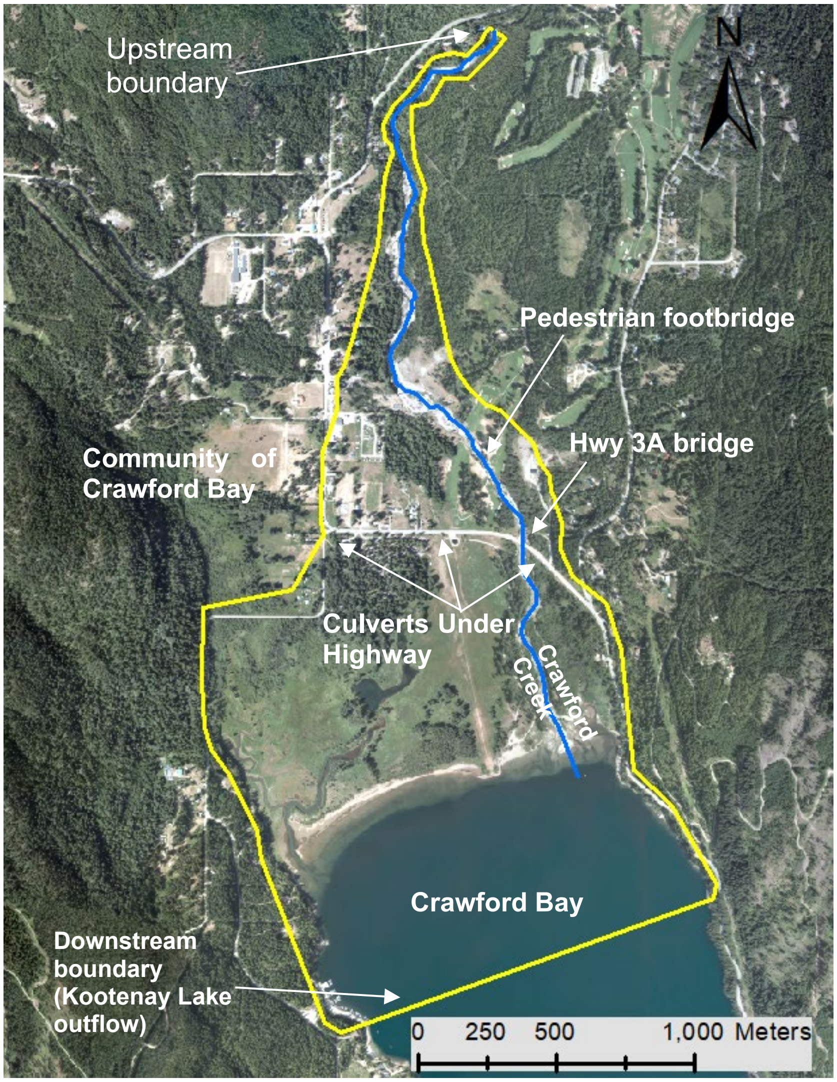

Figure 4-2. Crawford Creek study area modelling domain.

## 4.5. Flood Hazard Mapping

BGC prepared hazard maps based on the results from the numerical flood modelling. Specifically, BGC prepared two types of maps for Crawford Creek: hazard scenario maps and an FCL map. The scenario maps support emergency planning and risk analyses, while the FCL map supports communication and policy implementation, as described further below.

### 4.5.1. Hazard Scenario Maps

Hazard scenario maps display the hazard intensity (destructive potential) and extent of inundated areas for each scenario assessed. Two versions of the hazard scenario maps for each return period are provided: i) maps showing flood depth, and ii) maps showing flow impact force ($IF$) defined as the combination of fluid bulk density ($\rho$), area of impact ($A$) and velocity ($v$) shown in Equation 4-1:

$$IF \propto \rho A v^2 \qquad \text{[Eq. 4-1]}$$

For clearwater flooding, 1000 kg/m$^3$ was assumed for $\rho$ as shown Equation 4-1. The area of impact represents the area of the object that is impacted or the portion thereof. For this level of study, depth of flow from modelling results is used as a proxy for the height of the area and the impact force is then represented as an impact force per unit width, in this case 1 m.

Maps displaying flow depth support assessments where inundation is the primary mechanism of damage. Flow impact force maps highlight locations where a combination of higher flow velocity and depth may warrant additional assessment (i.e., analyses of bank stability, erosion, or life safety). Table 4-9 provides a description of the flow impact force ranges and their impacts on life safety and impacts on the built environment. A flow depth map for the 200-year peak discharge is provided in this report in Drawing 06. Flow depth and flow impact force maps for all return periods are displayed on Cambio.

**Table 4-8. Flow impact force values shown on the flood hazard scenario maps (Cambio)**

| Impact Force (kN/m) | Description |
|---|---|
| ≤ 1 | Slow flowing shallow and deep water with little or no debris. High likelihood of water damage. Potentially dangerous to people in buildings, in areas with higher water depths. |
| 1 to 10 | Mostly slow but potentially fast flowing shallow or deep flow with some debris. High likelihood of sedimentation and water damage. Potentially dangerous to people in the basement or first floor of buildings without elevated concrete foundations. |
| 10-100 | Fast flowing water and debris. High likelihood of structural building damage and severe sediment and water damage. Dangerous to people on the first floor or in the basement of buildings. Replacement of unreinforced buildings likely required. |
| >100[1] | Fast flowing debris. High likelihood of building destruction. Very dangerous to people in buildings irrespective of floor. |

Note:

1. Flow intensities greater than 100 kN/m in clear water creeks are generally confined to the main channel..

### 4.5.2. Flood Construction Level Mapping

FCLs are required for areas adjacent to river floodplains for consideration during planning. An FCL can be incorporated into regulation by authorities to provide guidance for new constructions on the extent and elevation of possible flooding in the area. In BC, FCLs have historically been calculated as the higher of the followings:

- Water surface profile for the design peak instantaneous flow plus 0.3 m of freeboard
- Water surface profile for the design daily flow plus 0.6 m of freeboard.

The freeboard is applied to the estimated water surface profile to account for uncertainties in the calculation of the water surface. As noted in EGBC (2017; 2018), for many BC rivers freeboard has been set higher than these minimum values to account for sediment deposition, debris jams, and other factors. Recently, several studies have recommended using 0.6 m of freeboard above the design peak instantaneous flow (KWL, 2014; 2017; NHC, 2008b; 2014; 2016; 2018). As such, we have selected to use this approach as well for the Crawford Creek study area.

The presence of dikes needs to be considered when defining the FCLs. Depending on the situation, the presence of a dike may lead to a local rise in the flood levels as the dike constrains the flow within the channel. Should a dike fail through overtopping or geotechnical failure, the resulting flooding depth and extent of flooding may be greater than if the dike was not present due to the elevated flood level (e.g., Figure 4-3). The numerical modelling and mapping conducted for Crawford Creek assumed a breach in the dike given their state of poor repair and history of being breached four times in the past 60 years. Several existing gaps in the dyke structure were also noted. As such a section of the dike was removed from the model geometry to simulate a pre-existing breach in the dike prior to onset of flooding, as discussed in greater detail in Appendix E. Dike breach scenarios beyond simply removing a section of the dike from the terrain were not included as part of the flood hazard assessment.

For Crawford Creek, the FCLs were generated by creating isolines from the predicted 200-year water surface plus a 0.6 m freeboard and extending the isolines across the limits of the floodplain generally perpendicular to the flow direction. The FCL maps are presented in Drawing 07.

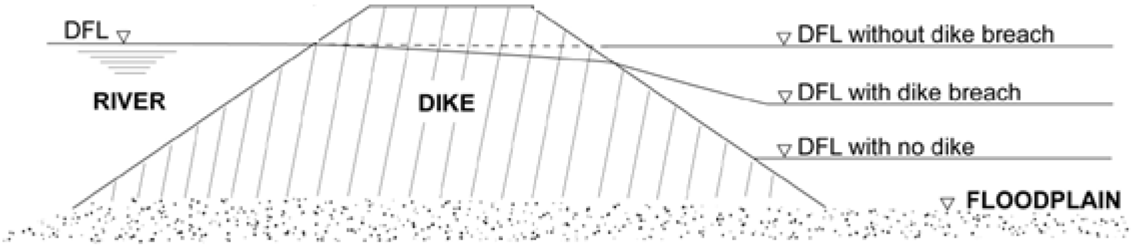

**Figure 4-3. Definition of design flood levels (DFL) in the presence of a dike. DFL refers to the estimated water levels from a design flood event such as the 200-year return period flood (Modified from Water Management Consultants, March 19, 2004).**

# 5. RESULTS

## 5.1. Channel Change Mapping and Bank Erosion

The objective of the geomorphic and bank erosion analysis was to document historical changes in channel width and related geomorphic processes using 1958 air photos and a 2017 orthophoto. The mapped geomorphic units in each of these years are illustrated in Drawing 04. The investigated area was divided into eight reaches to facilitate the analysis of the observed changes (Figure 5-1). The relevant features of these reaches, including average bank retreat based on the planimetric review, are provided in Table 5-1. A summary of the key changes observed within the 6.7 km long reach is included in Table 5-2, and a description of the relevant channel changes within the reaches on the active alluvial fan follows.

Two channel reaches were identified within the alluvial fan (Reach-7 and Reach-8) (Figure 5-1). The average bankfull width of the main-channel within these reaches is approximately 10 m (fan apex) to 40 m (distal section) wide. The average channel gradient decreases from around 2% (~1.2°) at the fan apex to 1.5% (~1°) near the channel outlet at Crawford Bay/Kootenay Lake (Figure 5-1).

Reach-7 is 1.9 km long and exhibits a straight-sinuous channel pattern (Figure 5-1). Past flood events have caused changes in this reach that indicate both lateral migration and aggradation processes. Three sections are of interest within Reach-7.

The first section of interest in Reach-7 is located at the upper part of the reach (approximately 900 m downstream of the Wagar Road). At this location, the channel thalweg shifted from right to left bank and is eroding along the toe of a terrace. If this erosion continues, there is potential to trigger slope failures leading to the incorporation of large volumes of sediments to the creek and limiting its capacity to carry flows.

The second section of interest in Reach-7 is located in the middle part of this reach, starting approximately 1 km upstream of the Highway 3A bridge. The series of channel dikes are installed in this section. Historical documentation indicates that reconstruction of the dikes has occurred after several flood events. When the 2017 and 1958 images are compared, it appears that the dikes promoted the stabilization of a large area of the fan that was impacted during the 1950s flood events. However, there are also sites along this section that have experienced bank retreatment (2017 imagery shows erosion acitivity). The current configuration of the channel suggests that this retreatment will continue as the dikes are present at a section of the channel where high shear stresses are expected to occur. The 20-year event inundation extent reaches the dike. This condition poses a threat to the integrity and stability of the dikes.

The third section of interest in Reach-7 comprises a portion of the right bank, approximately 150 m long that displays evidence of bank retreatment (Drawing 05). At this location, the channel is moving towards the right bank. The channel change analysis indicates that the bank has retread as much as 15 m within the analyzed period. The Highway 3A bridge is at this location.

Reach-8 is 0.8 km long and has a meandering pattern. It is located at the lower section of the fan and deposition processes dominate over erosion. Historically, this reach has been laterally unstable. The lidar data indicate that avulsion has occurred multiple times within this reach.

**Table 5-1. Channel reaches characterization and average bank retreat.**

| Reach | Length[1] (m) | Channel Width Variation[2] (m) | Average Bank Retreat 1958-2017 (m) | Channel Pattern |
|---|---|---|---|---|
| Reach-1 | 320 | 21-41 | 8 | Wandering[3] |
| Reach-2 | 1270 | 15-41 | 7 | Wandering[3] |
| Reach-3 | 170 | 7-9 | 3 | Single, straight (Canyon reach) [2] |
| Reach-4 | 1150 | 10-28 | 11 | Wandering[3] |
| Reach-5 | 400 | 8-18 | 6 | Single, straight (Canyon reach) |
| Reach-6 | 650 | 12-18 | 4 | Straight, sinuous |
| Reach-7 | 1890 | 20-40 | 5 | Straigh, sinuous (Fan reach) |
| Reach-8 | 810 | 12-35 | 22 | Meandering (Fan reach) |

Note:

1. Based on 2019 lidar and bathymetry data.
2. Accuracy is +/- 5 m
3. Outside area modelling domain.

**Table 5-2. Summary of key changes observed within the analyzed periods.**

| Period | Maximum Bank Retreat (m) | Highlighted observations |
|---|---|---|
| 1958 - 2019 | 38 +/- 5 m<br>(Reach-4) | Stabilization and revegetation of the channel, including the reaches within the alluvial fan.<br>Channel shifting at Reach-4, characterized by channel widening and erosion along the meander bends.<br>Channel shifting towards the right bank at Reach-7 promoting bank erosion. |

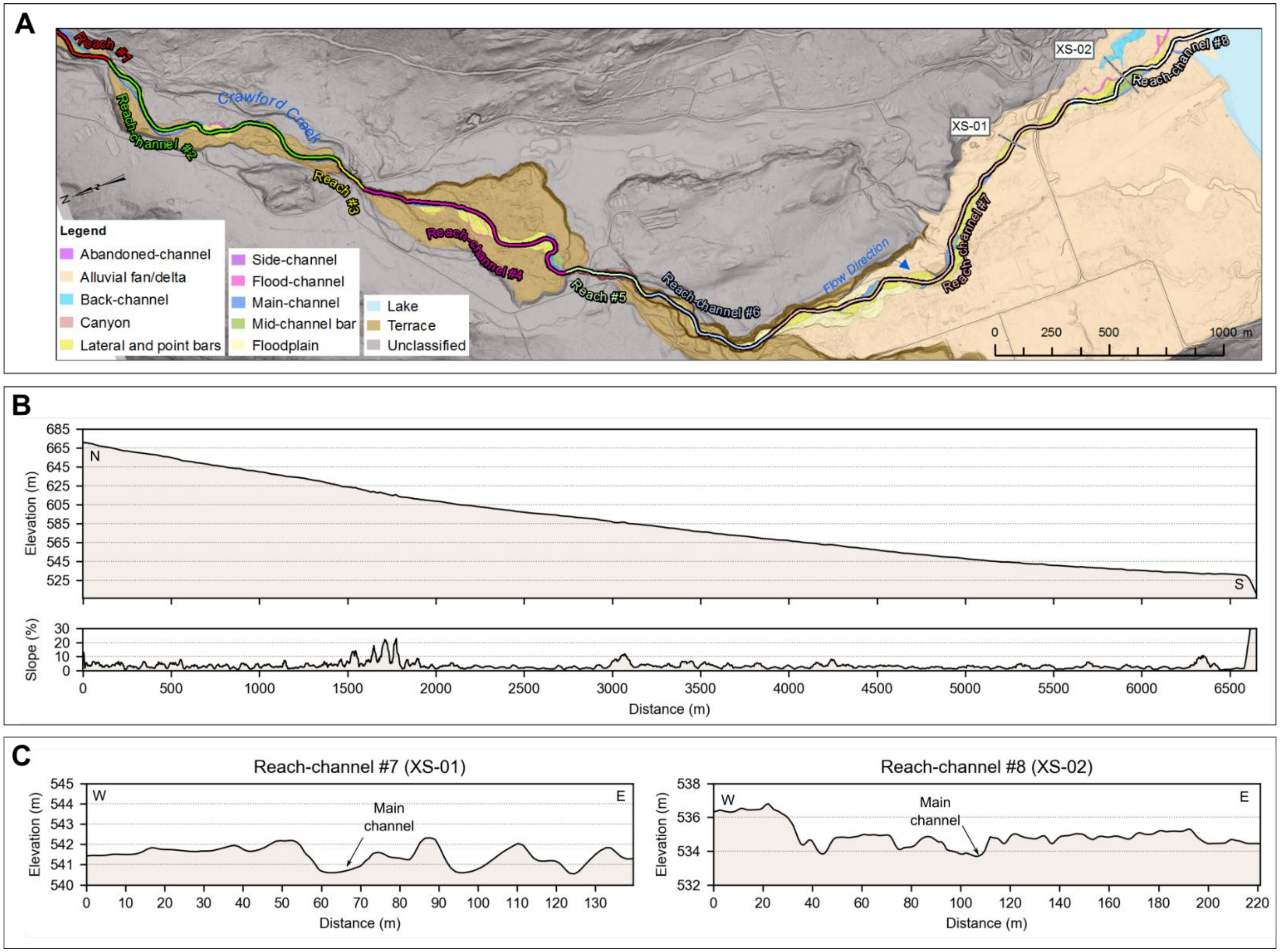

**Figure 5-1.** **Crawford Creek defined channel reaches and floodplain. (A) 2017 plan view of the creek and floodplain. (B) Channel long-profile and slope gradient.**

## 5.2. Hydrological Modelling

### 5.2.1. Historical Peak Discharge Estimates

Peak discharges for Crawford Creek were estimated based on the Global Model of the Regional FFA as discussed in Section 4.3.1. The peak discharges based on the historical data are presented in Table 5-3. A comparison of the estimates to results from previous reports (BC MEP, 1987; Summit, 2004; and Obedkoff, 2002) is also presented in Figure 5-2. The magnitude of peak discharges estimated for Crawford Creek is consistent with other estimates in the RDCK study for clearwater and steep creek watersheds. The 2-year flood is contained within the channel while the 20-year flood results in overbank flooding reaching the Highway 3A. Therefore, the peak flow estimates from the previous studies are considered to be overestimated, as the 2-year or greater floods for these studies all result in overbank flow and inundation of Highway 3A.

### 5.2.2. Accounting for Climate Change

The climate change impact assessment results based on the methods presented in Section 4.3.2 were difficult to synthesise in order to select climate-adjusted peak discharges on a site-specific basis. The assessment of the trends in the discharge records was inconclusive. The results of the statistical flood frequency modelling generally show a small decrease in the flood magnitude, while the results of the process-based discharge modelling generally show an increase with a wide range in magnitude. As a result, peak discharge estimates were adjusted upwards by 20% to account for the uncertainty in the impacts of climate change in the RDCK as per Appendix D. The climate-adjusted peak discharge estimates for various return periods are also listed in Table 5-3.

**Table 5-3. Historical and climate-adjusted peak flow estimates for Crawford Creek.**

| Return Period (years) | Annual Exceedance Probability (AEP) | Historical Peak Discharge ($m^3/s$) | Climate-adjusted Peak Discharge ($m^3/s$) |
|---|---|---|---|
| 2 | 0.5 | 25 | 30 |
| 5 | 0.2 | 35 | 42 |
| 10 | 0.1 | 43 | 51 |
| 20 | 0.05 | 50 | 60 |
| 25 | 0.04 | 53 | 63 |
| 50 | 0.02 | 61 | 73 |
| 100 | 0.001 | 69 | 83 |
| 200 | 0.005 | 78 | 94 |
| 500 | 0.002 | 91 | 109 |

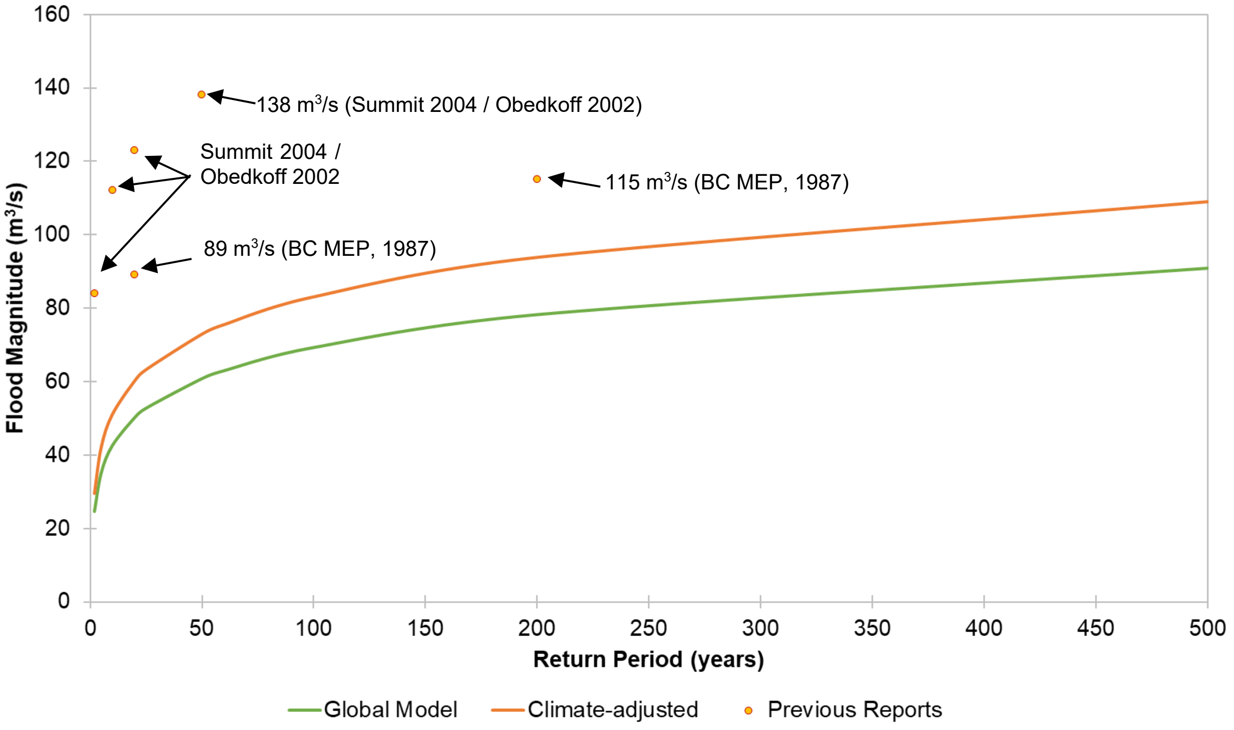

**Figure 5-2. Historical peak discharge estimates based on Regional FFA and previous reports. Climate-adjusted flows are based on the Regional FFA plus 20%.**

## 5.3. Hydraulic Modelling

A summary of the key observations from the hydraulic modelling is included in Table 5-4. A channel profile showing the 200-year water surface and dike elevations are shown on Figure 5-3.

**Table 5-4. Summary of modelling results.**

| Process | Key Observations |
|---|---|
| Clearwater inundation | • In general, modelling results and predicated floodplain extents are significantly smaller than the results presented in historical floodplain mapping (BC MEP, 1987). These differences are attributable to a more detailed terrain model achieved from access to lidar data, the use of a 2D rather than 1D model and a lower design flows for the present work.<br>• Overbank flow is observed upstream of the Highway 3A Bridge pooling behind the highway embankment and overtopping for 20-year return period or larger climate-change adjusted floods. Culverts under the highway are assumed to be blocked by debris during flood events.<br>• Flooding may occur outside the defined floodplain boundary. |
| Hydraulic Structures (Bridges) | • The Highway 3A bridge and the pedestrian footbridge over Crawford Creek are not inundated during the 200-year return period flood.<br>• There is approximately 0.6 m of clearance between the lower chord of the pedestrian bridge and the 200-year return period flood.<br>• There is greater than 3 m of clearance between the lower chord of the Highway 3A bridge and the 200-year return period flood. |

| Process | Key Observations |
|---|---|
| Flood Protection Structures (Dikes) | • The dikes along Crawford Creek's west bank are generally in poor repair and as such a section was removed from the terrain in the hydraulic model. Details are provided in Appendix E.<br>• Outside of the removed section, the dike crest is typically around 0.4 m above the 200-year flood elevation.<br>• The section of dike located approximately 350 km upstream (north) of the Highway 21 bridge overtops in the 500-year return period event indicating that potential overtopping of the dikes occurs under a 500-year flood on Crawford River with accounting for climate change.<br>• Some floodplain innundation occurs due to flow between existing gaps in the dikes. |

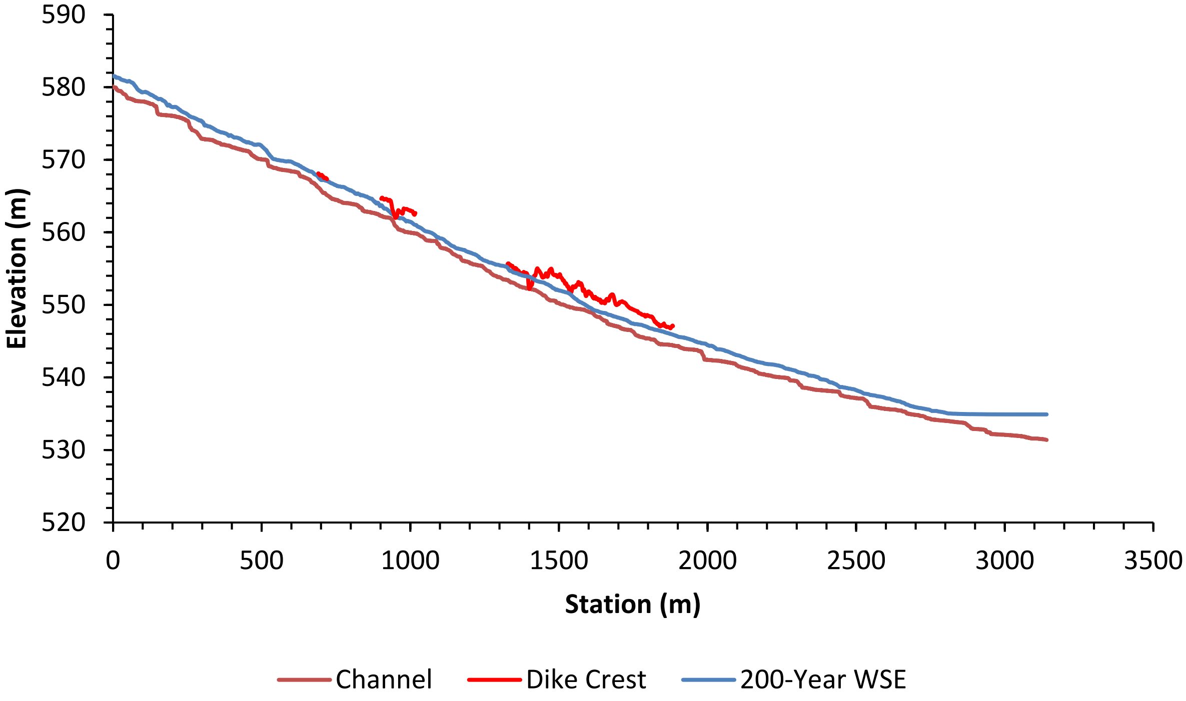

**Figure 5-3. Model profile showing channel bed, 200-year water surface elevation (WSE), and dike crest elevation.**

## 5.4. Flood Hazard Mapping

Hazard scenario results from the range of return periods modelled are presented in Cambio. Drawing 06 provides modelled water depths for the 200-year return period event.

## 5.5. Flood Construction Level Mapping

FCL results for the 200-year water surface elevation plus 0.6 m freeboard are presented on Drawing 07. Note that elevations from the FCLs have not been surveyed in the field and should not be relied upon for accuracy of ground levels at the building lot scale

# 6. SUMMARY AND RECOMMENDATIONS

This report provides a detailed flood hazard assessment of the Crawford Creek floodplain. This creek was chosen as a high priority site amongst hundreds of potential sites in the RDCK due to its comparatively high risk. This report has resulted in digital hazard maps that provide a basis for quantitative risk assessment, if required. It also provides the basis to inform the conceptualization and potential design and construction of mitigation measures should those be found to be required for Crawford Creek. A variety of analytical desktop and field-based tools and techniques were combined to understand Crawford Creek's geomorphological and hazard history, its hydrology and hydraulics.

## 6.1. Flood Hazard Assessment

### 6.1.1. Channel Change Mapping and Bank Erosion

These analyses were completed to gain an understanding of historical geomorphic changes on the fan and how these changes relate to channel migration and flooding. Channel change and bank erosion results illustrate:

- The observable geomorphic features identified from the 1958 aerial photographs and 2017 imagery (Table 5-1). These maps are useful to understand the geomorphic evolution of the channel, and how fluvial processes may influence flooding (Drawing 04). Delineating the interpreted channel thalweg from each set of photos was critical to find the major changes with the reviewed periods.
- The average bank retreat rates between observations (Table 5-2 and Drawing 05). The quantitative results indicate the rate of bank retreatment and the direction, but they do not explain the processes controlling the change. There are some limitations related to these analyses when based on desktop mapping (See Section 4.2.3). Additional steps to understand bank erosion hazard at the study area should include: 1) a characterization of bank susceptibility (erodibility); and, 2) the critical shear stresses required to erode the banks at critical erosion areas and for the different discharge events.
- Despite the acknowledged limitations, the analysis identified a few key areas for future bank erosion hazard assessment. Both bank failures and fluvial erosion processes need to be considered:
  - Reach 7- (left bank) to determine the potential of bank erosion triggering slope failures caused by undercutting of the terrace.
  - Reach 7- (right bank) to determine the integrity and stability of the dikes and their potential susceptibility to erosion under different shear stresses scenarios.

Both maps guide the analysis of channel dynamics within the analyzed area and their possible influence on flood hazards. Further efforts to assess bank erosion should be intended to estimate the erosion hazard for the different return periods.

### 6.1.2. Peak Discharge Estimates

Peak discharges for selected return periods (2- to 500-year return period) were estimated using a Regional FFA. In recognition of the projected impacts of climate change, these estimates were adjusted. Key findings include:

- The climate change impact assessment results were difficult to synthesize to select climate-adjusted peak discharges on a site-specific basis. Consequently, a 20% increase in peak discharge was adopted (Appendix D).
- The climate-change adjusted peak discharges for Crawford Creek range from 30 $m^3/s$ (2-year flood) to 109 $m^3/s$ (500-year flood) and are lower than previous flows used by BC MEP (1987) (115 $m^3/s$ for a 200 year return period flood).

### 6.1.3. Hydraulic Modelling

A 2D numerical model was employed to simulate the chosen hazard scenarios on Crawford Creek. Table 5-4 provides key observations derived from the numerical modelling. The numerical modelling demonstrates that the key hazards and associated risks at Crawford Creek stem from overtopping of Highway 3A, flow between existing gaps in the dike structures, and potential breaches of existing dike sections that are in a state of disrepair.

### 6.1.4. Flood Hazard Mapping

Model results are cartographically expressed in two ways:

1. The individual hazard scenarios are captured through hazard maps that display estimated flow velocity, flow depth and flood intensity. These maps are useful for assessments of development proposals and emergency planning.
2. An FCL map that combines the estimated water surface elevation for the 200-year return period event plus a 0.6 m freeboard. This map is useful to assist development proposals.

Both the individual scenario hazard and FCL maps serve as decision-making tools to guide subdivision and other development permit approvals.

## 6.2. Limitations and Uncertainties

While systematic scientific methods were applied in this study, some uncertainties prevail. As with all hazard assessment and concordant maps, the hazard maps prepared for Crawford Creek represent a snapshot in time. Future changes to the Crawford Creek watershed or fan including the following may warrant re-assessment and/or re-modelling:

- Future land use (urbanization) or landcover (deforestation, forest fire) changes in the floodplain or fan
- Substantial flood events
- Major changes in the channel planform or aggradation
- Bridge re-design
- Alteration to the existing dikes or the construction of new flood control structures
- Substantial changes to Kootenay Lake flood levels.

The assumptions made on changes in runoff due to climate change, while not unreasonable, are not infallible and will likely need to be updated occasionally as scientific understanding of such processes evolves. Despite these limitations and uncertainties, BGC believes that a credible hazard assessment has been achieved on which land use decisions can be made.

## 6.3. Considerations for Hazard Management

Recommendations are provided in the Summary Report as they pertain to all studied RDCK creeks. This section notes Crawford Creek-specific issues that could be considered in the short term given the findings of this report. They are purposely not named “recommendations” as those would come out of a more in-depth discussion with the District.

Key considerations are:

- Predicted floodplain extents are significantly smaller than those previously predicted for Crawford Creek by a BC MEP (1987) report. These differences are primarily attributed to the peak discharges estimated for Crawford Creek from the regional FFA study (Appendix C) which were lower than the previous studies. As discussed in Section 5.2.1, the peak discharges from the previous studies were considered to be overestimated, as the 2-year peak discharge from the previous studies results in overbank flow and inundation of Highway 3a and not felt to be realistic. Additionally, the availability of more accurate elevation from lidar and the use of a 2D rather than 1D hydraulic model result in more accurate determination of the flooding extents.
- Flooding may occur outside the defined floodplain boundary and floodplain limits were not established on the ground by legal survey.
- Numerical modelling indicates that the surveyed dike crest elevation is typically greater than 0.4 m above the 200-year return period flood depth; however, due to the poor repair of the dikes it is likely that sections of the dike will fail. There are also significant existing gaps between dikes which water will flow through during floods.
- This assessment does not include an assessment of the geotechnical stability of the dikes. Furthermore, the existing dikes were assumed to be ineffective due to their poor state of repair. Dike stability assessments could be completed in future as part of a separate scope of work. The site-specific risk geohazard management framework discussed in Section 4.2 of the Summary Report (BGC, 2020) can be used to evaluate the current flood hazard risk to the community and determine whether improving the existing dikes and/or expanding the dikes would result in a cost-effective risk reduction.
- Hydraulic modelling (2D) results indicate that the pedestrian bridge and the Highway 3A bridge are not predicted to overtop during a 200-year return period flood. This assessment does not consider the potential for the bridge to be blocked with large woody debris or the impact of accumulation of gravel.
- In 20-year or greater flood events, water is expected to pool behind and flow over Highway 3A approximately 250 m west of the bridge. Culverts running under the highway were assumed to be blocked by debris during flooding.

- The hazard mapping conducted for a range of return periods provides an improved hazard basis to apply for funding for additional risk assessment, emergency response planning and mitigation projects. Results of the hazard mapping are provided on Drawing 06 for the 200-year return period water depth and in Cambio for the range of scenarios modelled (e.g., 20-year, 50-year, 500-year).
- The FCLs presented in Drawing 07 for the 200-year return period flood event plus 0.6 m freeboard provides an improved basis for community planning, bylaw development, and emergency response planning in areas subject to flood hazards, with consideration of climate change. The application of the FCL map requires discussions and regulatory decisions for both existing and proposed development. Building and floodproofing elevations should be established from legal survey and benchmarks. Setback distances from the natural boundaries of watercourses are not shown on maps. FCLs provide a standards-based approach which are simple to apply and interpret. In some cases, the FCL may be impossible or impractical to implement for several reasons. Allowances should be permitted for stakeholders to apply for a site-specific reduction in the FCLs contingent on a report by a suitably qualified Professional Engineer, preferably using a risk-based approach.

## 6.4. Recommendations

Recommendations are provided in the Summary Report (BGC, 2020) as they pertain to all studied RDCK creeks.

# 7. CLOSURE

We trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

**BGC ENGINEERING INC.**
**per:**

| | |
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| Kenneth Lockwood, PhD., EIT<br>Hydrotechnical Engineer | Patrick Grover, M.A.Sc., P.Eng<br>Senior Hydrotechnical Engineer |
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| Rob Millar, PhD., P.Eng., P.Geo<br>Principal Hydrotechnical Engineer | Hamish Weatherly, M.Sc., P.Geo<br>Principal Hydrologist |

KL/RM/HW/mp/mm

**Final stamp and signature version to follow once COVID-19 restrictions are lifted**

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# APPENDIX A
# TERMINOLOGY

Table A-1 provides defines terms that are commonly used in geohazard assessments. BGC notes that the definitions provided are commonly used, but international consensus on geohazard terminology does not fully exist. **Bolded terms** within a definition are defined in other rows of Table A-1.

**Table A-1. Geohazard terminology.**

| Term | Definition | Source |
|---|---|---|
| Active Alluvial Fan | The portion of the fan surface which may be exposed to contemporary hydrogeomorphic or avulsion hazards. | BGC |
| Aggradation | Deposition of sediment by a (river or stream). | BGC |
| Alluvial fan | A low, outspread, relatively flat to gently sloping mass of loose rock material, shaped like an open fan or a segment of a cone, deposited by a stream at the place where it issues from a narrow mountain valley upon a plain or broad valley, or where a tributary stream is near or at its junction with the main stream, or wherever a constriction in a valley abruptly ceases or the gradient of stream suddenly decreases | Bates and Jackson (1995) |
| Annual Exceedance Probability ($P_H$) (AEP) | The Annual Exceedance Probability (AEP) is the estimated **probability** that an event will occur exceeding a specified magnitude in any year. For example, a flood with a 0.5% AEP has a one in two hundred chance of being reached or exceeded in any year. AEP is increasingly replacing the use of the term **'return period'** to describe flood recurrence intervals. | Fell et al. (2005) |
| Avulsion | Lateral displacement of a stream from its main channel into a new course across its fan or floodplain. An "avulsion channel" is a channel that is being activated during channel avulsions. An avulsion channel is not the same as a paleochannel. | Oxford University Press (2008) |
| Bank Erosion | Erosion and removal of material along the banks of a river resulting in either a shift in the river position, or an increase in the river width. | BGC |
| Clear–water flood | Riverine and lake flooding resulting from inundation due to an excess of clear-water discharge in a watercourse or body of water such that land outside the natural or artificial banks which is not normally under water is submerged. | BGC |
| Climate normal | Long term (typically 30 years) averages used to summarize average climate conditions at a particular location. | BGC |
| Consequence (C) | In relation to risk analysis, the outcome or result of a **geohazard** being realised. Consequence is a product of **vulnerability** (V) and a measure of the **elements at risk** (E) | Fell et al. (2005);<br>Fell et al. (2007),<br>BGC |

| Term | Definition | Source |
|---|---|---|
| Consultation Zone | The Consultation Zone (CZ) includes all proposed and existing development in a geographic zone defined by the approving authority that contains the largest credible area affected by specified **geohazards**, and where damage or loss arising from one or more simultaneously occurring specific **geohazards** would be viewed as a single catastrophic loss. | Adapted from Porter et al. (2009) |
| Debris Flow | Very rapid to extremely rapid surging flow of saturated, non-plastic debris in a steep channel (Hungr, Leroueil & Picarelli, 2014). Debris generally consists of a mixture of poorly sorted sediments, organic material and water (see Appendix B of this report for detailed definition). | BGC |
| Debris Flood | A very rapid flow of water with a sediment concentration of 3-10% in a steep channel. It can be pictured as a flood that also transports a large volume of sediment that rapidly fills in the channel during an event (see Appendix B of this report for detailed definition). | BGC |
| Design Peak Daily Flow | The design flow (e.g. 200-year flood) based on the analysis of annual maximum daily average discharge records. | BGC |
| Design Peak Instantaneous Flow | The design flow (e.g. 200-year flood) based on the analysis of annual maximum instantaneous discharge records. | BGC |
| Elements at Risk (E) | This term is used in two ways:<br>a) To describe things of value (e.g., people, infrastructure, environment) that could potentially suffer damage or loss due to a **geohazard**.<br>b) For risk analysis, as a measure of the value of the elements that could potentially suffer damage or loss (e.g., number of persons, value of infrastructure, value of loss of function, or level of environmental loss). | BGC |

| Term | Definition | Source |
|---|---|---|
| Encounter Probability | This term is used in two ways:<br>a) **Probability** that an event will occur and impact an element at risk when the element at risk is present in the **geohazard** zone. It is sometimes termed **"partial risk"**<br>b) For quantitative analyses, the **probability** of facilities or vehicles being hit at least once when exposed for a finite time period L, with events having a **return period** T at a location. In this usage, it is assumed that the events are rare, independent, and discrete, with arrival according to a statistical distribution (e.g., binomial or Bernoulli distribution or a Poisson process). | BGC |
| Erosion | The part of the overall process of denudation that includes the physical breaking down, chemical solution and transportation of material. | Oxford University Press (2008) |
| Flood | A rising body of water that overtops its confines and covers land not normally under water. | American Geosciences Institute (2011) |
| Flood Construction Level (FCL) | A designated flood level plus freeboard, or where a designated flood level cannot be determined, a specified height above a natural boundary, natural ground elevation, or any obstruction that could cause flooding. | BGC |
| Flood mapping | Delineation of flood lines and elevations on a base map, typically taking the form of flood lines on a map that show the area that will be covered by water, or the elevation that water would reach during a flood event. The data shown on the maps, for more complex scenarios, may also include flow velocities, depth, or other hazard parameters. | BGC |
| Floodplain | The part of the river valley that is made of unconsolidated river-borne sediment, and periodically flooded. | Oxford University Press (2008) |
| Flood setback | The required minimum distance from the natural boundary of a watercourse or waterbody to maintain a floodway and allow for potential bank erosion. | BGC |

| Term | Definition | Source |
|---|---|---|
| Freeboard | Freeboard is a depth allowance that is commonly applied on top of modelled flood depths. There is no consistent definition, either within Canada or around the world, for freeboard. Overall, freeboard is used to account for uncertainties in the calculation of a base flood elevation, and to compensate for quantifiable physical effects (e.g., local wave conditions or dike settlement). Freeboard in BC is commonly applied as defined in the BC Dike Design and Construction manual (BC Ministry of Water, Land and Air Protection [BC MWLAP], 2004): a fixed amount of 0.6 m (2 feet) where mean daily flow records are used to develop the design discharge or 0.3 m (1 foot) for instantaneous flow records. | BC Ministry of Water, Land and Air Protection [BC MWLAP] (2004) |
| Frequency (f) | Estimate of the number of events per time interval (e.g., a year) or in a given number of trials. Inverse of the **recurrence interval** (**return period**) of the **geohazard** per unit time. Recurring **geohazards** typically follow a **frequency**-magnitude (F-M) relationship, which describes a spectrum of possible **geohazard magnitudes** where larger (more severe) events are less likely. For example, annual **frequency** is an estimate of the number of events per year, for a given **geohazard event magnitude**.<br>In contrast, annual **probability** of exceedance is an estimate of the **likelihood** of one or more events in a specified time interval (e.g., a year). When the expected **frequency** of an event is much lower than the interval used to measure **probability** (e.g., **frequency** much less than annual), **frequency** and **probability** take on similar numerical values and can be used interchangeably. When **frequency** approaches or exceeds 1, defining a relationship between **probability** and **frequency** is needed to convert between the two. The main document provides a longer discussion on **frequency** versus **probability**. | Adapted from Fell et al. (2005) |
| Hazard | Process with the potential to result in some type of undesirable outcome. Hazards are described in terms of scenarios, which are specific events of a particular frequency and magnitude. | BGC |
| Hazardous flood | A flood that is a source of potential harm. | BGC |

| Term | Definition | Source |
|---|---|---|
| Geohazard | Geophysical process that is the source of potential harm, or that represents a situation with a potential for causing harm.<br>Note that this definition is equivalent to Fell et al. (2005)'s definition of Danger (threat), defined as an existing or potential natural phenomenon that could lead to damage, described in terms of its geometry, mechanical and other characteristics. Fell et al. (2005)'s definition of danger or threat does not include forecasting, and they differentiate Danger from Hazard. The latter is defined as the **probability** that a particular danger (threat) occurs within a given period of time. | Adapted from CSA (1997), Fell et al. (2005). |
| Geohazard Assessment | Combination of **geohazard analysis** and evaluation of results against a **hazard tolerance standard** (if existing). Geohazard assessment includes the following steps:<br>a. **Geohazard analysis**: identify the **geohazard** process, characterize the geohazard in terms of factors such as mechanism, causal factors, and trigger factors; estimate **frequency** and magnitude; develop **geohazard scenarios**; and estimate extent and intensity of **geohazard scenarios**.<br>b. Comparison of estimated hazards with a hazard tolerance standard (if existing) | Adapted from Fell et al. (2007) |
| Geohazard Event | Occurrence of a **geohazard**. May also be defined in reverse as a non- occurrence of a **geohazard** (when something doesn't happen that could have happened). | Adapted from ISO (2018) |
| Geohazard Intensity | A set of parameters related to the destructive power of a **geohazard** (e.g. depth, velocity, discharge, impact pressure, etc.) | BGC |
| Geohazard Inventory | Recognition of existing **geohazards.** These may be identified in geospatial (GIS) format, in a list or table of attributes, and/or listed in a **risk register**. | Adapted from CSA (1997) |
| Geohazard Magnitude | Size-related characteristics of a **geohazard**. May be described quantitatively or qualitatively. Parameters may include volume, discharge, distance (e.g., displacement, encroachment, scour depth), or acceleration. In general, it is recommended to use specific terms describing various size-related characteristics rather than the general term magnitude. Snow avalanche magnitude is defined differently, in classes that define destructive potential. | Adapted from CAA (2016) |

| Term | Definition | Source |
|---|---|---|
| Geohazard Risk | Measure of the **probability** and severity of an adverse effect to health, property the environment, or other things of value, resulting from a geophysical process. Estimated by the product of **geohazard probability** and **consequence**. | Adapted from CSA (1997) |
| Geohazard Scenario | Defined sequences of events describing a **geohazard** occurrence. Geohazard scenarios characterize parameters required to estimate risk such **geohazard extent** or **runout exceedance probability,** and **intensity.** Geohazard scenarios (as opposed to **geohazard risk scenarios**) typically consider the chain of events up to the point of impact with an element at risk, but do not include the chain of events following impact (the **consequences**). | Adapted from Fell et al. (2005) |
| Hazard | Process with the potential to result in some type of undesirable outcome. Hazards are described in terms of scenarios, which are specific events of a particular frequency and magnitude. | BGC |
| Inactive Alluvial Fan | Portions of the fan that are removed from active hydrogeomorphic or avulsion processes by severe fan erosion, also termed fan entrenchment. | BGC |
| LiDAR | Stands for Light Detection and Ranging, is a remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the Earth. These light pulses - combined with other data recorded by the airborne system - generate precise, three-dimensional information about the shape of the Earth and its surface characteristics. | National Oceanic and Atmospheric Administration, (n.d.). |
| Likelihood | Conditional **probability** of an outcome given a set of data, assumptions and information. Also used as a qualitative description of **probability** and **frequency**. | Fell et al. (2005) |
| Melton Ratio | Watershed relief divided by square root of watershed area. A parameter to assist in the determination of whether a creek is susceptible to flood, debris flood, or debris flow processes. | BGC |
| Nival | Hydrologic regime driven by melting snow. | Whitfield, Cannon and Reynolds (2002) |
| Orphaned | Without a party that is legally responsible for the maintenance and integrity of the structure. | BGC |
| Paleofan | Portion of a fan that developed during a different climate, base level or sediment transport regime and which will not be affected by contemporary geomorphic processes (debris flows, debris floods, floods) affecting the active fan surface | BGC |

| Term | Definition | Source |
|---|---|---|
| Paleochannel | An inactive channel that has partially been infilled with sediment. It was presumably formed at a time with different climate, base level or sediment transport regime. | BGC |
| Pluvial – hybrid | Hydrologic regime driven by rain in combination with something else. | BGC |
| Probability | A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty) and must refer to a set like occurrence of an event in a certain period of time, or the outcome of a specific event. It is an estimate of the **likelihood** of the magnitude of the uncertain quantity, or the **likelihood** of the occurrence of the uncertain future event.<br>There are two main interpretations:<br>i) Statistical – **frequency** or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an “objective” or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.<br>ii) Subjective (or Bayesian) probability (degree of belief) – Quantified measure of belief, judgement, or confidence in the **likelihood** of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgement regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes. | Fell et al. (2005) |
| Return Period (Recurrence Interval) | Estimated time interval between events of a similar size or **intensity**. Return period and **recurrence interval** are equivalent terms. Inverse of **frequency**. | BGC |
| Risk | Likelihood of a geohazard scenario occurring and resulting in a particular severity of consequence. In this report, risk is defined in terms of safety or damage level. | BGC |
| Rock (and debris) Slides | Sliding of a mass of rock (and debris). | BGC |
| Rock Fall | Detachment, fall, rolling, and bouncing of rock fragments. | BGC |

| Term | Definition | Source |
|---|---|---|
| Scour | The powerful and concentrated clearing and digging action of flowing air or water, especially the downward erosion by stream water in sweeping away mud and silt on the outside curve of a bend, or during a time of flood. | American Geological Institute (1972) |
| Steep-creek flood | Rapid flow of water and debris in a steep channel, often associated with avulsions and bank erosion and referred to as debris floods and debris flows. | BGC |
| Steep Creek Hazard | Earth-surface process involving water and varying concentrations of sediment or large woody debris. (see Appendix B of this report for detailed definition). | BGC |
| Uncertainty | Indeterminacy of possible outcomes. Two types of uncertainty are commonly defined:<br>a) Aleatory uncertainty includes natural variability and is the result of the variability observed in known populations. It can be measured by statistical methods, and reflects uncertainties in the data resulting from factors such as random nature in space and time, small sample size, inconsistency, low representativeness (in samples), or poor data management.<br>b) Epistemic uncertainty is model or parameter uncertainty reflecting a lack of knowledge or a subjective or internal uncertainty. It includes uncertainty regarding the veracity of a used scientific theory, or a belief about the occurrence of an event. It is subjective and may vary from one person to another. | BGC |
| Waterbody | Ponds, lakes and reservoirs | BGC |
| Watercourse | Creeks, streams and rivers | BGC |

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# APPENDIX B
# SITE PHOTOGRAPHS

**Photo 1.**

**Standing on the right bank looking upstream (north) at large woody debris in the main channel, 50m north of Crawford Bay. Photo: BGC, July 4, 2019.**

**Photo 2.**

**Standing on the right bank looking across to the left Bank (east) at large woody debris in the main channel, 50m north of Crawford Bay. Photo: BGC, July 4, 2019.**

**Photo 3.**

**Standing 50m south of Highway 3A looking south at the field below the highway. Photo: BGC, July 4, 2019.**

**Photo 4.**

**From the right Bank looking upstream 100m north of Crawford Bay. Photo: BGC, July 4, 2019.**

**Photo 5.**
**From the right Bank looking across the channel to woody debris on the left bank 100m north of Crawford Bay. Photo: BGC, July 4, 2019.**

**Photo 6.**
**Signs of bank erosion along Crawford Creek, approximately 100 m north of Crawford Bay. Photo: BGC, July 4, 2019.**

**Photo 7.**

**Measuring the size of bank protection material 75m north east of the pedestrian footbridge crossing Crawford Creek. Photo: BGC, July 4, 2019.**

**Photo 8.**

**Bank protection along Crawford Creek's west bank, 75m north east of the pedestrian footbridge crossing. Crawford Creek. Photo: BGC, July 4, 2019.**

**Photo 9.**

**Looking from the right bank across to Kootenay Springs Golf Course on the left bank of Crawford creek 75m north east of the pedestrian footbridge. Photo: BGC, July 4, 2019.**

**Photo 10.**

**Pedestrian Bridge crossing Crawford Creek, 200 m north of Highway 3A. Photo: BGC, July 4, 2019.**

**Photo 11.**

**Pedestrian Bridge crossing Crawford Creek, 200 m north of Highway 3A looking to left bank. Photo: Explore, August 28, 2019.**

**Photo 12.**

**Gravel bar in the center of Crawford Creek, 360m north of the Highway 3A. Photo: BGC, July 4, 2019.**

**Photo 13.**

**Measuring bank material along Crawford Creek's right bank, 80m north of the Highway 3A. Photo: BGC, July 4, 2019.**

**Photo 14.**

**Woolman sample of Bed Material immediately north of the Highway 3A bridge. Photo: BGC, July 4, 2019.**

**Photo 15.**

**Looking upstream of Pedestrian Bridge crossing Crawford Creek, 200 m north of Highway 3A. Photo: Explore, August 18, 2019.**

**Photo 16.**

**Bank protection along Crawford Creek's west bank,360m north of the Highway 3A. Photo: BGC, July 4, 2019.**

**Photo 17.**

**Looking upstream under Highway 3A Bridge, Photo: Explore, August 18, 2019.**

**Photo 18.**

**Drone Photo of Highway 3A. Photo: BGC, September 25, 2019.**

**Photo 19.**

**Drone Photo of Dyke along Crawford Creek's west bank, 450m north of Highway 3A. Photo: Explore, September 25, 2019.**

**Photo 20.**

**Dyke along Crawford Creek's west bank, 600m north of Highway 3A. Photo: Explore, September 25, 2019.**

**Photo 21.**

**Crawford Creek entering Kootenay Lake. Photo: Explore, August 16, 2019.**

**Photo 22.**

**Small channel 700m west of Crawford Creek. Photo: Explore, August 16, 2019.**

# APPENDIX C
# REGIONAL FLOOD FREQUENCY ANALYSIS

## C.1. INTRODUCTION

Estimating flood magnitude is of fundamental importance to reliable floodplain mapping. As most watercourses are not gauged, flood magnitude is commonly estimated for an ungauged catchment using a Regional Flood Frequency Analysis (Regional FFA). There are several methods to complete a Regional FFA. This appendix documents the methodology followed by BGC Engineering Inc. (BGC) for the regionalization of floods in British Columbia using the index-flood method (Dalrymple 1960).

This appendix begins with a description of Regional FFA and the index-flood method (Section C1.0). The study area over which the index-flood is developed is discussed in Section C2.0. The data acquisition and compilation to support the analysis is described in Section C3.0. A description of the methods and assumptions for the regionalization of floods is included in Section C4.0. Results for the different hydrologic regions that cover the Regional District of Central Kootenay (RDCK) are presented in Section C5.0, while the application of the index-flood method to ungauged catchments in the RDCK is presented in Section C6.0. Finally, the limitations of the study are discussed in Section C7.0.

### C.1.1. Regional FFA

Extreme events are rare by definition and record lengths at hydrometric stations are often short. Regional FFA accounts for short record lengths by trading space for time where flood events at several hydrometric stations are pooled to estimate flood magnitude in a homogeneous region. Homogeneous regions can be defined as geographically contiguous regions, geographically non-contiguous regions, or as hydrological neighbourhoods. Grouping catchment areas of similar catchment characteristics into homogeneous regions is a critical part of Regional FFA because hydrologic information can be transferred accurately only within a region that is homogeneous. The more homogeneous a region is, the more reliable the flood quantile estimates. Some heterogeneity may be deemed acceptable in some cases. Studies show that even moderately heterogeneous regions can yield more accurate flood quantile estimates than a single-station FFA (Hosking & Wallis, 1997).

### C.1.2. Index-flood Method

Several methods have been developed to conduct a Regional FFA in homogeneous regions. Among the quantile estimation methods, the index-flood is considered superior to other models (Ouarda et al., 2008). The index-flood is a method of regionalization with a long history in FFA (Dalrymple, 1960). The index-flood method involves the development of a dimensionless regional growth curve assumed to be constant within a homogenous region. The index-flood method also requires the selection of an index-flood which can be the mean annual flood, the median annual flood, or another quantile of choice calculated at each hydrometric station in the region.

The probability distribution of flood events at hydrometric stations in a homogeneous region are identical apart from a site-specific scaling factor, the index-flood. The parameters of the probability distribution are estimated at each hydrometric station. These at-site estimates are combined using a weighted average to generate a regional estimate. The regional growth curve is thus a dimensionless quantile function common to every hydrometric station in the region and takes on the following form (Eq. C-1):

$$X_T = Q_T \,/\, Q_m \qquad \text{[Eq. C-1]}$$

where $X_T$ is the growth factor for return period $_T$, $Q_T$ is the flood magnitude at return period $_T$, and $Q_m$ is the index-flood magnitude. The flood magnitude at any return period is calculated using this relationship given the index-flood estimate.

### C.1.3. Application to Ungauged Catchments

The index-flood method can be applied to an ungauged catchment by developing a regional relationship between the index-flood and catchment characteristics at hydrometric stations in the region. The relationship can be expressed in many forms including a multivariate linear regression. Flood events can be assumed to depend on the characteristics of individual catchments such as area, elevation, percent lake, forest coverage, mean annual precipitation, mean annual temperature, etc. Once the catchment characteristics are extracted at the ungauged site, the index-flood can be estimated. The flood magnitude of any annual exceedance probability (AEP) can be estimated for an ungauged catchment using the index-flood estimate and the regional growth curve by re-organizing equation Eq. C1-1.

## C.2. STUDY AREA

A Regional FFA for British Columbia represents a considerable challenge given its regional variations in precipitation caused by sharp changes in topography as well as diverse geology. The proportion of annual precipitation that falls as snow as opposed to rain increases with latitude, elevation, and distance from the Pacific Ocean. Significant regional variations in precipitation are observed in British Columbia, influenced by the various mountain ranges. Storms approaching the West Coast are lifted rapidly along the windward mountain slopes, resulting in widespread precipitation. A rain shadow is created on the lee side of the mountains. For example, Tofino receives an average of 3,160 mm of annual precipitation while Nanaimo, on the east coast of Vancouver Island, receives 1,060 mm.

This climate pattern is repeated several times from east to west. As the weather systems approach the Coast Mountains, orographic effects result in twice as much precipitation in North Vancouver compared to Vancouver proper. Moving to the east, the Okanagan Valley is located on the lee side of the Coast Mountains resulting in an arid to semi-arid climate with annual precipitation on the order of 350 mm. The cycle is repeated over the Monashees, the Columbia Trench, and the Rocky Mountains. These orographic effects impact flood events and complicate regionalization efforts due to significant areal variations in precipitation, even for small catchments. These significant variations in precipitation suggest that a multivariate approach to regionalization is practical for British Columbia.

Similar to precipitation, surficial geology in the province demonstrates significant spatial variability. This variability is important in that while two catchments may be located in a similar precipitation zone, the hydrologic response can be significantly different. Catchments dominated by colluvial veneers and bedrock will tend to have larger unit peak flows, than those mantled by coarse morainal sediment, with the latter tending to attenuate peak flows through available soil moisture storage. To avoid introducing boundary effects at the border with the Unites States and Alberta, the study area was extended to include the northern portion of Washington, Idaho, and Montana as well as the eastern Slopes of the Rocky Mountains. A map of the study area is presented in Figure C-1.

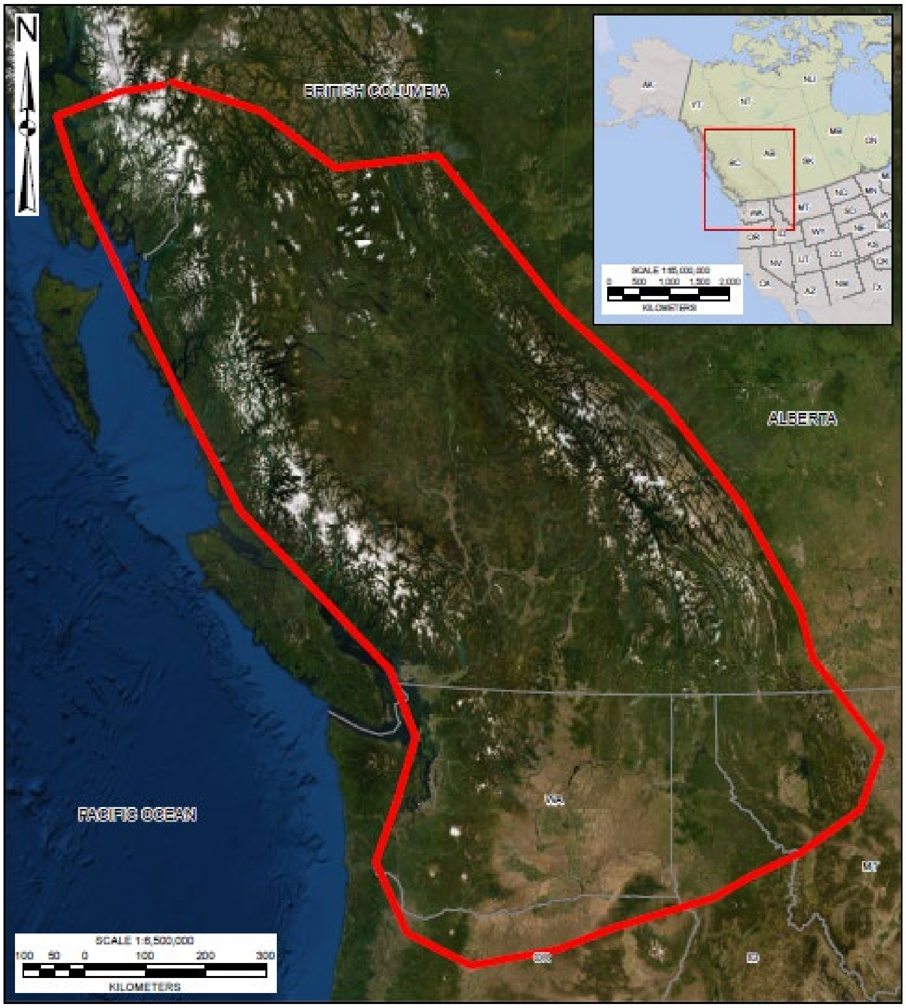

Figure C-1. Study area where the red outline defines the boundary.

## C.3. DATA ACQUISITION AND COMPILATION

A large component of this study consisted of acquiring the data and compiling it in a format that was usable for analysis. Suitable hydrometric stations in the study area were identified and the flood records were acquired from the appropriate monitoring agency. The catchment polygons upstream from the hydrometric stations were then delineated and the area calculated using methods specific to the scale of the catchment. Lastly, a suite of catchment characteristics was selected based on potential to influence flood events. These catchment characteristics were extracted for each polygon. The acquisition and the compilation of this rich dataset was the most time-consuming portion of the procedure. The following sections include a detailed description of how the data were acquired and how the dataset was compiled for analysis.

### C.3.1. Hydrometric Stations

A total of 3,309 hydrometric stations are located within the study area. Of these, 2115 are managed by the Water Survey of Canada (WSC) and the remaining 1194 are managed by the United States Geological Survey (USGS).

### C.3.2. Flood Records

As an initial step, all flood events recorded at the hydrometric stations were extracted. This extraction was challenging as records are stored differently by the WSC and USGS. In Canada, flood events are stored in the HYDAT database, which includes the annual maximum peak instantaneous streamflow, the maximum average daily streamflow, as well as the date and time of each event. The catchment area and the number of years on record are also available in the HYDAT database. The flood records were acquired directly from the HYDAT database for hydrometric stations in Canada. In the US, flood events are stored online on websites specific to each hydrometric station. The annual maximum peak instantaneous streamflow, the catchment area, and the number of years on record are also stored in this way. This information was extracted from the online storage space using a programming script for each USGS hydrometric station.

### C.3.3. Maximum Peak Instantaneous Streamflow

The preferred metric for analysis is the annual maximum peak instantaneous streamflow. However, it is not uncommon for flood records to have more annual maximum average daily streamflow records than peak instantaneous values, which are greater in magnitude. The ratio (I/D) between maximum peak instantaneous and maximum average daily streamflow is typically greater for small catchments than for very large catchments. Therefore, where only a maximum daily streamflow is reported for some years, maximum peak instantaneous streamflow values can be estimated from available maximum average daily streamflow records using regression analysis.

The reliability of the regression analysis was judged based on the coefficient of determination ($R^2$) in combination with the Cook distance (D). The $R^2$ is the proportion of the variance in the peak instantaneous streamflow that is predictable from the average daily streamflow. The D value is used to assess the influence of outliers present in the dataset. The regression analysis was deemed acceptable by BGC if the $R^2$ is above 0.95 and the D value is above 25. In this case, the maximum peak instantaneous streamflow record was extended using the regression analysis for a longer record length. Alternatively, maximum peak instantaneous streamflow record remained as-is where the regression analysis was deemed unacceptable.

### C.3.4. Catchment Polygons

The catchment polygons at hydrometric stations within the study area were estimated using two different approaches.

1. River Networks Tools™[1] (RNT)
2. Using an Environmental Systems Research Institute (ESRI) process (i.e., GIS-based).

The RNT-based approach is dependent on the delineation of a stream network, while the ESRI-based process is dependent on topographic data. Catchment polygons were defined for all hydrometric stations located within the study area. Catchment delineation based on a stream network was observed to be more reliable for small catchments, especially where topographic relief is low. The catchment polygons defined by the ESRI process were selected for larger catchments (>1,000 km$^2$), while the RNT-based approaches were selected for smaller catchment areas (<1,000 km$^2$). The selection of the best catchment polygon for analysis could not be checked directly as the monitoring agencies (WSC and USGS) do not publish polygon shape information.

### C.3.5. Catchment Areas

The catchment area was estimated for each catchment polygon (RNT, modification based on RNT, and ESRI) at each hydrometric station. The catchment area for each polygon was then compared with the value published by the respective monitoring agency. The catchment area published by monitoring agencies is generally considered most reliable (although recognizing many of the catchment areas for the WSC stations were calculated with 1:50,000 scale mapping and may not reflect more recent topographic mapping) and was used to quality check the calculated areas.

The estimated value of the catchment area was deemed acceptable if it was within ±15% of the published value. If more than 1 catchment area estimate (of the 3) was within ±15% of the published value, the catchment area with the smallest difference relative to the published value was selected as the best estimate for analysis. Approximately 90% of catchment polygons were within ±15% of the published value.

Published values are not available for all hydrometric stations. In those cases, the catchment area was deemed acceptable if the 3 estimates were within ±15% of each other. Catchment areas that did not meet the ± 15% criteria were not included in the analysis. A total of 2269 hydrometric stations were removed from the analysis because either the catchment area was deemed unreliable or water level data only was recorded at the station. Manual quality checks were not completed for these catchments due to the time-consuming nature of this effort. The number of hydrometric stations lost that could have been considered useful is considered negligible. The

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[1] The RNT is a proprietary software developed by BGC. RNT is based on publicly available 1:24,000-scale or better topographic and hydrographic datasets throughout North America that BGC has compiled and systematically developed to support a wide range of hydrotechnical calculations (e.g., catchment area) and site-specific precipitation and flood monitoring.

number of hydrometric stations in the study area is summarized in Table C-1. The ESRI catchment polygons were used for the hydrometric stations at the border between Canada and the United States because the polygons based on the two RNT approaches are observed to be poorly delineated due to differences in data resolution available between both countries.

**Table C-1. Number of hydrometric stations in the study area.**

| Criteria | Number |
|---|---|
| Hydrometric Stations in Study Area | 3284 |
| Station with Unacceptable Catchment Area Estimates | 2269 |
| Stations with Acceptable Catchment Area Estimates | 1015 |

### C.3.6. Catchment Characteristics

Catchment characteristics were selected based on potential to influence flood events. A suite of 18 catchment characteristics was ultimately selected and estimated for each hydrometric station, as summarized in Table C-2. Several data sources were used to compile the catchment characteristics which are described in the following sections.

#### C.3.6.1. Catchment Statistics

The Shuttle Radar Topography Mission (STRM) dataset (Farr et al. 2007) was used to extract the catchment elevation statistics. The catchment elevation statistics were averaged over the catchment area. This dataset was used to calculate the catchment area (just for catchments over 1000 km$^2$), relief, length, and slope. The centroid statistics were also extracted from this dataset.

#### C.3.6.2. Climate Variables

The Climate North America (ClimateNA) dataset was used to estimate the climate variables for each catchment polygon (Wang et al., 2016). The climate variables were averaged over the catchment area and were based on the average for the period 1961 to 1990.

**Table C-2. List of selected catchment characteristics.**

| Type | No. | Acronym | Characteristic | Units | Dataset |
|---|---|---|---|---|---|
| Catchment | 1 | Centroid_Lat | Latitude at the centroid location in the catchment polygon | degrees | STRM |
| | 2 | Centroid_Long | Longitude at the centroid location in the catchment polygon | degrees | |
| | 3 | Centroid_Elev | Elevation at the centroid location in the catchment polygon | m | |
| | 4 | Area | Area of the catchment polygon | km$^2$ | |
| | 5 | Relief | Maximum minus minimum catchment elevation | m | |
| | 6 | Length | Area divided by perimeter | km | |
| | 7 | Slope | Catchment length divided by relief times 100 | % | |
| Climate | 8 | MAP | Mean annual precipitation | mm | Climate NA |
| | 9 | MAT | Mean annual temperature | °C | |
| | 10 | PAS | Precipitation as snow | mm | |
| | 11 | PPT_wt | Winter precipitation (Dec, Jan, Feb) | mm | |
| | 12 | PPT_sp | Spring precipitation (Mar, Apr, May) | mm | |
| | 13 | PPT_sm | Summer precipitation (Jun, Jul, Aug) | mm | |
| | 14 | PPT_fl | Fall precipitation (Sep, Oct, Nov) | mm | |
| Physiographic | 15 | Forest | Forest cover in the catchment | % | NALCMS |
| | 16 | Water_Wetland | Wetland and open water cover in the catchment | % | |
| | 17 | Urban | Urban cover in the catchment | % | |
| | 18 | CN | Inferred based on integrating land cover and soils cover | unitless | NALCMS and HYSOGs250m |

### C.3.6.3. Land cover

The North American Land Change Monitoring System (NALCMS) land cover products include the 2005 land cover map of North America. This dataset includes 19 land cover classes derived from 250 m Moderate Resolution Spectroradiometer (MODIS) image composites (Latifovic et al. 2012).

This dataset was used to calculate the percent forest, percent wetland and lake, and the urban portion of the catchment.

### C.3.6.4. Curve Number

The curve number (CN) is an empirical parameter used for predicting runoff from rainfall. BGC integrated the land cover (NALCMS) and the hydrologic soils group (HYSOGs250m) datasets to infer the average CN over each catchment. The NALCMS dataset is described in Section C.3.6.3. The HYSOGs250m dataset represents typical soil runoff potential at a 250 m spatial resolution (Ross et al., 2018). Hydrologic soils groups are defined based on soil texture, depth to bedrock or depth to groundwater. There are four basic groups: A, B, C, D. Four additional groups are included where the depth to bedrock is considered to be less than 60 cm: AD, BD, CD, and DD. The area covered by each hydrologic soils group is summed for a total area over the catchment for each hydrologic soils group.

The CN was assigned following guidance from the USGS (1986). The CN values for soils where the depth to bedrock or depth to groundwater.is expected to be less than 0.6 m from the surface (i.e., D soils) were assumed to be the same as the case where it is not expected to be close to the ground surface. The CN value assignment for the combinations of land cover and hydrologic soils groups identified in the catchments is presented in Table C-3. The CN values were averaged over the catchment area using a weighted mean. The weight reflects the percentage of the area covered by a given CN value.

**Table C-3. CN values based on the integration between the land cover and soils datasets.**

| Land Cover (NALCMS 2005) | Cover Type (USGS 1986) | Soils | | | |
|---|---|---|---|---|---|
| | | HSG-A | HSG-B | HSG-C | HSG-D |
| Temperate or sub-polar needleleaf forest | Woods - Good | 30 | 55 | 70 | 77 |
| Temperate or sub-polar broadleaf deciduous forest | Woods - Good | 30 | 55 | 70 | 77 |
| Mixed forest | Woods - Good | 30 | 55 | 70 | 77 |
| Temperate or sub-polar shrubland | Brush - brush-weed-grass mixture with brush the major element - Fair | 35 | 56 | 70 | 77 |
| Temperate or sub-polar grassland | Pasture, grassland, or range—continuous for grazing - Good | 39 | 61 | 74 | 80 |
| Sub-polar or polar grassland-lichen-moss | Pasture, grassland, or range—continuous for grazing - Good | 39 | 61 | 74 | 80 |
| Sub-polar or polar barren-lichen-moss | Desert shrub - major plants include saltbrush. Greasewood, creosotebush, blackbrish, bursage, palo verde, mesquite, and cactus - good | 49 | 68 | 79 | 84 |
| Sub-polar taiga needleleaf forest | Woods - Good | 30 | 55 | 70 | 77 |
| Cropland | Row crops - straight row (SR) | 63 | 74 | 81 | 85 |
| Barren land | Desert shrub - major plants include saltbrush. Greasewood, creosotebush, blackbrish, bursage, palo verde, mesquite, and cactus - good | 49 | 68 | 79 | 84 |
| Urban and built-up | Urban districts - commercial and business | 89 | 92 | 94 | 95 |
| Snow and ice | NA | 0 | 0 | 0 | 0 |
| Wetland | NA | 0 | 0 | 0 | 0 |
| Water | NA | 0 | 0 | 0 | 0 |

## C.4. METHODS AND ASSUMPTIONS

Once the dataset is compiled for analysis, the regionalization of floods procedure can begin. A description of the methods and assumptions for the index-flood method is included in this section.

### C.4.1. Flood Statistics Calculations

Flood statistics were calculated using the flood record at each of the selected hydrometric stations (2101) in the study area. Flood statistics include L-moments and flood quantile estimates.

#### C.4.1.1. L-moments

The L-moment approach in the index-flood procedure was used by BGC for the regionalization of floods in British Columbia. The shape of a probability distribution has traditionally been described by the moments of the distribution including the mean, standard deviation, skewness, and kurtosis. However, moment estimators have some undesirable properties where the skewness and kurtosis can be severely biased. Both have algebraic bounds that depend on the sample size (Hosking & Wallis 1997).

L-moments are an alternative system for describing the shape of probability distributions. Studies have shown that L-moments are unbiased, less sensitive to outliers, and are better estimators of distribution parameters especially for short to moderate record length (Hosking, 1990). Furthermore, L-moments allow for the efficient computation of parameter estimates and flood quantile estimates.

L-moments evolved as modifications to the probability weighted moments (Greenwood et al., 1979). In terms of probability weighted moments, L-moments are defined as $\lambda_1$, $\lambda_2$, $\lambda_3$, and $\lambda_4$ with their mathematical expressions published for a range of probability distributions in Hosking and Wallis (1997, Appendix).

Dimensionless versions of L-moments are defined as L-moment ratios by dividing the higher order L-moments by $\lambda_2$. L-moment ratios are defined by Eq. C-2:

$$\tau_r = \lambda_r / \lambda_2 \qquad \text{[Eq. C-2]}$$

L-moment ratios depict the shape of a distribution independently of its scale measurement. Refer to Table C-4 for L-moment terminology.

**Table C-4. L-moment terminology.**

| Symbol (population) | Symbol (sample) | Definition |
|---|---|---|
| $\lambda_1$ | $\iota_1$ | L-location or the mean of the distribution |
| $\lambda_2$ | $\iota_2$ | L-scale |
| $\tau$ | $t$ | L-CV |
| $\tau_3$ | $t_3$ | L-skewness |
| $\tau_4$ | $t_4$ | L-kurtosis |

### C.4.1.2. At-site Flood Quantile Estimates

The flood quantile estimates at hydrometric stations are referred to as ‘at-site’ estimates and are used to compare with the modeled quantile estimates to assess the validity of the model. Flood quantile estimates were calculated using the flood data by means of a single-station FFA. A popular approach in FFA is the Annual Maximum Series (AMS) where the maximum peak instantaneous streamflow for each year on record is used for analysis. The basic assumption is that the flood events are independent and identically distributed from a single population of flood events.

A probability distribution is selected to describe the flood events in the record. The true form of the underlying probability distribution is not known and there is no standard distribution appropriate in all cases. The goal is to select a probability distribution that fits the observed data well but also generates robust quantile estimates that are not sensitive to physical deviations of the true probability distribution (Hosking & Wallis, 1997). In extreme value statistics, data follow one of three extremal types of distributions: Gumbel, Fréchet, or Weibull (Coles, 2001). These three distributions can be expressed as a single formula and are considered a family of distributions known as the Generalized Extreme Value (GEV) distribution. The GEV distribution is shown to arise as an asymptotic model for maximum values in a sample and hence can be viewed as a natural model for observed flood events. For these reasons, the GEV distribution was used to describe the recorded flood events. No statistical tests were used to assess this choice because the GEV distribution is considered flexible to account for the variability captured at a single hydrometric station.

The parameters of the GEV distribution were estimated using the L-moments. The flood quantiles were calculated for a range of return periods (Table C-5). The reliability of the quantile estimates depends on a range of factors including the record length and the range of flood event magnitudes captured in the record. The longer the record length, the more reliable the quantile estimates.

**Table C-5. Return period and associated AEP.**

| Return Period (Years) | AEP |
|---|---|
| 2 | 0.5 |
| 5 | 0.2 |
| 10 | 0.1 |
| 20 | 0.05 |
| 50 | 0.02 |
| 100 | 0.001 |
| 200 | 0.005 |
| 500 | 0.002 |

## C.4.2. Formation of Hydrological Regions

The catchment characteristics extracted over the catchment polygons were used to group the hydrometric stations into hydrological regions using a cluster analysis. Cluster analysis is an objective method for creating regions (Tasker, 1982) which historically were based subjectively using geographical, political, administrative or physiographic boundaries. The essence of cluster analysis is to identify clusters (groups) of hydrometric stations such that the stations within a cluster are similar while there is dissimilarity between the clusters. Hosking and Wallis (1997) suggest that cluster analysis is the most practical method of forming regions for large datasets and provides several opportunities for subjective adjustments to the regions. The algorithm used by BGC to group hydrometric stations is Agglomerative Hierarchal Clustering.

### C.4.2.1. Data Preparation

The catchment characteristics at each hydrometric station were normalized so that the average is zero and the standard deviation is approximately 1. The distance metric used is the Euclidian distance between the catchment characteristics. The suite of catchment characteristics at all hydrometric stations were compared to one another and organised using Ward's Distance measure (d) (Ward, 1963).

### C.4.2.2. Number of Hydrological Regions

Several statistical measures were used to guide the number of clusters to partition the hydrometric stations. The statistical measures include the Elbow Method, the Silhouette Score, and review of the dendrogram. The selection of the number of clusters was also subjectively assessed by reviewing the physical basis of the cluster distribution (e.g., is there a physical meaning behind the number and distribution of the clusters?).

The Elbow Method accounts for the percentage of variance explained as a function of the number of clusters. The percentage of the variance explained decreases with increasing number of clusters. The minimum number of clusters that provides the most gain in the variance explained was selected for analysis.

The Silhouette Score is a measure of how similar the catchment of a hydrometric station is to its own cluster compared to other clusters. The Silhouette Score was calculated for each hydrometric gauge station and averaged over each cluster. The Silhouette Score ranges from -1 to +1 where a high value indicates that the hydrometric stations are well matched to their own clusters and poorly matched to neighboring clusters.

The dendrogram represents how the clustering algorithm (i.e., agglomerative hierarchal clustering) groups the catchments and depicts a road map of the merging procedure showing which catchments were merged and when in order of increasing cluster distance.

The spatial distribution of the clusters was then reviewed to verify that they are physically plausible. This review was done by superimposing the clusters on a map of British Columbia to see whether there is a physical meaning supporting the cluster distributions.

### C.4.2.3. Manual Adjustments of Hydrologic Regions

The clusters identified using the clustering algorithm were adjusted manually to increase homogeneity. The manual adjustments were completed by considering the topography, spatial patterns in hydrological processes, and ecozones in Canada. The clusters were further separated based on the scale of catchment area to respect the statistical requirement for constancy in the coefficient of variation (CV) for homogeneous regions.

### C.4.2.4. Refinement of the Hydrometric Station Selection

The hydrometric station selection was refined to increase the homogeneity of the clusters by reducing the variability introduced by many hydrometric stations. The refinement process was guided by the following 5 criteria.

1. Catchments upstream of hydrometric stations with a regulation level greater than 25% were not included for analysis. The level of regulation is inferred by proportion of the catchment area upstream of the dams to the total catchment area upstream of the hydrometric station.
2. The catchment area range considered in the regionalization extends up to 5,000 km$^2$. Catchments with a greater catchment area size are most likely well gauged and studied that a regionalization of flood is not required.
3. Nested hydrometric stations along the same watercourse were also removed from the region to reduce cross-correlation.
4. A minimum of 6 years of maximum peak instantaneous streamflow data was set as a minimum for analysis. While this threshold is low, it is considered adequate since the influence of each hydrometric stations on the model reflects the record length.
5. Hydrometric stations recording water level only were excluded from the analysis at the onset. Hydrometric stations recording water level and streamflow measurements but located within or immediately at the outlet of lakes were also removed from the analysis.

The flow regime at these locations is considered heavily regulated precluding the use of frequency analysis to estimate peak flows.

In addition to these criteria, discordancy (*Di*) was considered to refine the selection. The discordancy is measured in term of the L-moments of the data at the hydrometric stations within a cluster. The formal definition for *Di* is found in Hosking and Wallis (1997, equation 3.3, page 46). A hydrometric station is considered discordant if *Di* is “large”. The definition of “large” depends on the number of hydrometric stations in the cluster. If the cluster includes more than 15 hydrometric stations, the critical value for the discordancy statistic is 3. Discordancy was calculated for each hydrometric station within each hydrologic region. Hydrometric stations with *Di* values greater than 3 were removed from the cluster. This process was re-iterated until no more hydrometric stations showed *Di* values greater than 3.

#### C.4.2.5. Testing for Homogeneity

The hypothesis for homogeneity is that the probability distribution of the flood events at the hydrometric stations within a cluster is the same except for a site-specific scale factor. The goal is to have clusters that are sufficiently homogenous that the regionalization of floods is advantageous to a single station FFA. Testing for homogeneity is done using the H-Test. The H-Test result helps assess whether the hydrometric stations in a cluster may reasonably be considered homogeneous. The formal definition for the H-Test is found in Hosking and Wallis (1997, equation 4.5, page 63). Of note, some level of heterogeneity is expected in these clusters due to the natural variability of hydrological processes that control flood events. The H-Test is not intended to be used as a significance test but rather as a guideline to inform whether the re-definition of a region could lead to a meaningful increase in the accuracy of the flood quantile estimates (Hosking and Wallis, 1993).

### C.4.3. Regionalization

Once the clusters were considered sufficiently homogeneous, they were considered “hydrologic regions”. The regionalization of floods was then completed for each region. The L-moment approach in the index-flood procedure was used by BGC for the regionalization exercise. The procedure for each hydrologic region included: averaging the L-moments, selecting a distribution, estimating the parameters, developing the growth curve, and estimating the index-flood. The mean annual flood (MAF) was selected as the index-flood for this study. The following sections describe the methods and assumptions for the regionalization of floods for a given hydrologic region.

#### C.4.3.1. Regional L-moments

The L-moment ratios were averaged over each hydrologic region. A weighted average was used where the weight reflected the number of observations at each hydrometric station. The weighted average was used to put more weight on hydrometric stations with a longer record length. The weighted average helps take advantage of all available data as it is often limited in many areas of the province. The regional average L-moment ratios are defined in Table C-6. The L-moment ratios are used rather than the L-moments because they yield slightly more accurate quantile estimates.

**Table C-6. Definition for regional average L-moment ratios.**

| Symbol (sample) | Definition |
|---|---|
| $\iota_1^R$ | L-location or the mean of the distribution |
| $\iota_2^R$ | L-scale |
| $t^R$ | L-CV |
| $t_3^R$ | L-skewness |
| $t_4^R$ | L-kurtosis |

### C.4.3.2. Distribution Selection for Growth Curves

The selection of an appropriate probability distribution for the growth curves was done using a goodness-of-fit test and review of L-moment ratio diagrams. These tests were completed to assess the variability imposed compiling the results of many hydrometric stations into a single growth curve. The goodness-of-fit test was based on 1,000 simulations and looked at a suite of candidate distributions. The candidate probability distributions included Generalised Logistic (GLO), Generalised Extreme Value (GEV), Generalised Pareto (GPA), Generalised Normal (GNO), and Pearson Type III (PE3). Probability distributions with Z statistics $\leq$1.64 were deemed acceptable (Hosking & Wallis, 1997). The regional L-moments were also plotted with the L-skewness and L-kurtosis relationships for two (Exponential (E), Gumbel (G), Logistic (L), Normal (N), and Uniform (U)) and three-parameter (GLO, GEV, GPA, GNP, PE3) candidate distributions in L-moment ratio diagrams. The plotting position of the regional L-moments was reviewed for the distribution selection that provided an acceptably close visual fit.

### C.4.3.3. Parameter Estimation

The regional L-moments were used to estimate the parameters of the selected probability distribution. The equations used to estimate the parameters for the GEV distribution are found in Hosking and Wallis (1997, A.52, A.55, and A.56, page 196) in addition to other select probability distributions.

### C.4.3.4. Growth Curves and Error Bounds

The index-flood was selected to be the MAF. As a result, the regional mean was set to 1 ($\iota_1^R = 1$). The probability distribution was fit by equating the L-moment ratios of the population ($\lambda_1$, $\tau$, $\tau_3$, $\tau_4$) to the regional average L-moment ratios ($\iota_1^R$, $t^R$, $t_3^R$, $t_4^R$).

One of the strengths of the Regional FFA completed using the regional L-moments is that the procedure is useful even when the assumptions are not all satisfied (e.g., possibility of heterogeneity, misspecification of the probability distribution, and statistical dependence between observations at different sites). An approach to estimate the accuracy of the estimated flood quantiles is by Monte Carlo simulation. A Monte Carlo simulation was therefore run to estimate the variability in the quantile estimates from the regional GEV distribution. This variability was used to set the error bounds on the regional growth curve.

### C.4.3.5. Index-flood Estimation

The index-flood was estimated using a multiple linear regression. Regression is a classic statistical method to describe the relationship between a dependent variable (index-flood) and independent variables (catchment characteristics). The multiple linear regression model is expressed as follows:

$$Q_T = aA^b B^c \dots N^n \qquad \text{[Eq. C-3]}$$

where $Q_T$ is the flood magnitude at return period $_T$, A, B, …, N are the catchment characteristics, a is the regression constant, and b, c, …, n are the regression coefficients. Base 10 logarithms are used to convert this equation to a linear form by transforming the variables to the following:

$$\log Q_T \ = \log a + b(\log A) + c\,(\log B) + \cdots + n\,(\log N) \qquad \text{[Eq. C-4]}$$

These coefficients were estimated using the Weighted Least Squares method introduced by Tasker (1980), which accounts for the sampling error introduced by unequal record lengths. Unequal record lengths mean that the sampling errors of the observations (flood quantiles) are not equal (heteroscedastic) and the assumption of constant variance in Ordinary Least Squares method is not valid.

The top 5 models were selected using consideration for the adjusted $R^2$ and the Bayesian information criterion (BIC). The 5 models with the lowest BIC were selected and the index-flood estimate was averaged. Select diagnostic plots were reviewed to control the quality of the regressions. The diagnostic plots are listed in Table C-7. The index-flood model was developed over two scales: regional and provincial. These two scales were compared to assess the influence of the distribution of hydrometric stations on the reliability of the MAF estimate.

**Table C-7. Diagnostic plots.**

| Plot | Diagnostic |
|---|---|
| At-site vs. Modeled | Inspect for a one to one relationship as close to as possible |
| At-site Quantile vs. Modeled Quantile | Inspect whether the distribution of the fitted values match the distribution of the observed values |
| At-site Quantiles vs. Modeled Residuals | Inspect for constancy in residuals. Residuals are the differences between the at-site and the modeled estimates |

#### C.4.3.6. Regional Model

The first scale considered is the regional scale where the MAF was modeled over an area consistent with the hydrologic regions defined across the province. This scale is consistent with the scale used to do develop the regional growth curves.

#### C.4.3.7. Provincial Model

The second scale considered is the provincial scale where all hydrometric stations across the province, that meet the selection criteria, were used to model the MAF. The provincial model was developed to capture the range of hydrological processes that control flood events in British Columbia.

#### C.4.3.8. Flood Quantile Estimates

Flood quantile were than estimated using the regional growth curve and index-flood estimates (both scales) for all hydrometric stations in a given region. Quantile plots were generated to compare the at-site and modeled results over the range of AEPs.

#### C.4.3.9. Catchment Characteristic Transformations

The relationship between flood events and catchment characteristics need not be linear. Experience and judgement were used to guide the selection of independent variables and inform the relationship between flood events and catchment characteristics. An exhaustive comparison of correlations between flood magnitude and catchment characteristics showed that catchment area and catchment length are proportional to flood magnitude. For this analysis, the remaining catchment characteristics needed to be log transformed.

### C.4.4. Error Statistics

The quality of the flood quantile estimates was assessed using select error statistics including the Root Mean Square Error (SRMSE), the Percent Error (SPE), and the Bias (SBIAS) for the following AEPs: 0.5, 0.1, 0.02, 0.005. The standardized version of the error statistics is used to account for the different scales (Table C-8).

**Table C-8. Error statistics, definitions, and diagnostic.**

| Error Statistic (acronym) | Definition | Diagnostic |
|---|---|---|
| SRMSE | Standard deviation of the residuals. | Inspect how concentrated the modeled estimates are around the line of best fit. |
| SPE | The difference between the modeled and at-site estimate, divided by the at-site estimate, multiplied by 100%. | Inspect how close the modeled estimate is to the at-site estimate/ |
| SBIAS | The tendency to overestimate or underestimate the modeled variable. | Inspect for a consistent over or underestimate of the modeled variable |

The mathematical expressions for the SRMSE, SPE, and SBIAS are included below in Eq. C-5, Eq. C-6, and Eq. C-7.

$$SRMSE = \sqrt{\frac{\sum_{i=1}^{Np}\left(\frac{Qm_{mod}^{i}-Qm_{at-site}^{i}}{Qm_{at-site}^{i}}\right)}{Np}} \quad \text{[Eq. C-5]}$$

$$SPE = \frac{\sum_{i=1}^{Np} abs\left(\frac{Qm_{mod}^{i}-Qm_{at-site}^{i}}{Qm_{at-site}^{i}}\right)}{Np} * 100 \quad \text{[Eq. C-6]}$$

$$SBIAS = \frac{\sum_{i=1}^{Np} \left(\frac{Qm_{mod}^{i}-Qm_{at-site}^{i}}{Qm_{at-site}^{i}}\right)}{Np} \quad \text{[Eq. C-7]}$$

### C.4.5. Decision Tree

A decision tree model was used to assign hydrologic regions to ungauged catchments. A decision tree was built using the Random Forest classification algorithm. The decision tree model was based on the catchment characteristics at the hydrometric stations in the study area. A total of 500 random samples were pulled from the dataset (with replacement). From each random sample, a decision tree was generated by using 3 variables at each decision point. The hydrologic region assignment was based on majority votes. The out-of-bag (OBB) error rate was 7.2%. The OBB is a method of measuring the prediction error specific to random forest algorithms.

### C.4.6. Statistical Software

The statistical software used by BGC for the analysis was R (R Core Team, 2019). R is a free software environment for statistical computing. The analysis is completed with support from several packages. These packages are listed in Table C-9 for reference.

**Table C-9. Analysis and associated R package.**

| Analysis | R Packages | Authors |
|---|---|---|
| Flood Statistics | Lmom | J. R. M. Hosking |
| Clustering | stats | R Core Team |
| Discordancy, H-Test, Distribution Selection, Parameter Estimation, and Growth Curve Development | lmomRFA | J. R. M. Hosking |
| Index-flood Estimation | stats and leaps | R Core Team and Alan Miller |
| Random Forest decision tree | Rpart, randomForest | Andy Liaw and Matthew Wiener |

## C.5. RESULTS

### C.5.1. Hydrometric Station Selection

A total of 1015 hydrometric stations were included in the analysis. The hydrometric stations were distributed across the study area with a greater concentration in the south compared to the north, largely reflecting population density. There is also a greater concentration of hydrometric stations in the United States than Canada (Figure C-2).

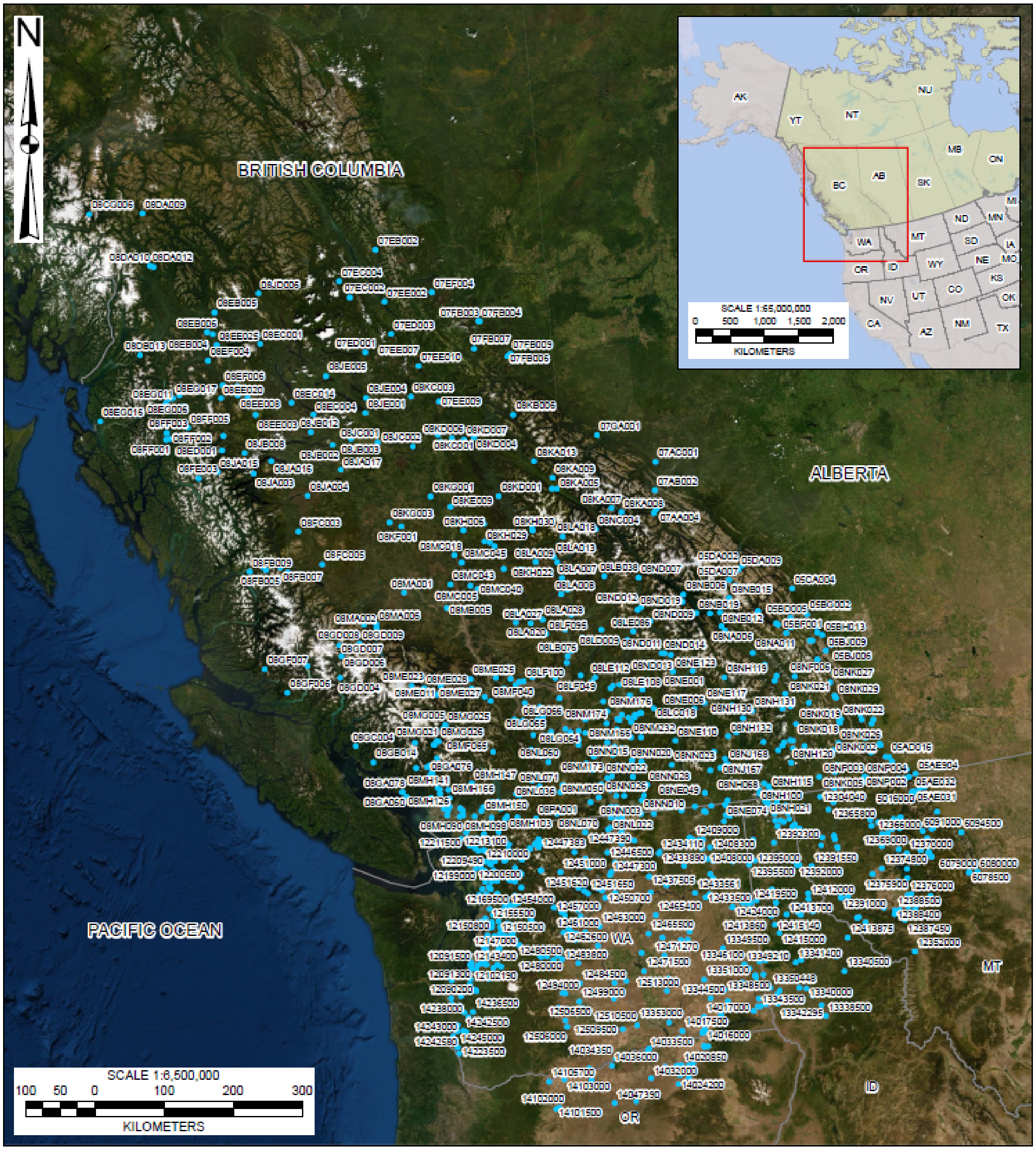

Figure C-2. Distribution of hydrometric stations within the study area.

The 18 catchment characteristics and their range in magnitude are summarized over the 1015 hydrometric stations in Table C-10. The climate catchment characteristics show a wide range in magnitude which is not surprising considering the sharp regional contrast imposed by the topography. The urban catchments are concentrated in coastal Washington.

**Table C-10. Summary of catchment characteristics, including the mean, maximum, and minimum values over all hydrometric stations considered for analysis (1,015).**

| Type | No. | Acronym | Mean | Min | Max | Standard Deviation |
|---|---|---|---|---|---|---|
| Catchment | 1 | Centroid_Lat | 49.3092758 | 43.75066 | 57.094597 | 2.3 |
| | 2 | Centroid_Long | -119.5562752 | -130.965466 | -112.917172 | 3.5 |
| | 3 | Centroid_Elev | 1,133 | 18 | 3,046 | 534 |
| | 4 | Area | 7,572 | 1.3 | 601,746 | 38,417 |
| | 5 | Relief | 1,639 | 19 | 4,355 | 791 |
| | 6 | Length | 5 | 0.2 | 71 | 7 |
| | 7 | Slope | 62 | 4 | 350 | 49 |
| Climate | 8 | MAP | 1,299 | 218 | 4,173 | 787 |
| | 9 | MAT | 4.1 | -3.0 | 10.9 | 3.0 |
| | 10 | PAS | 499 | 25 | 2191 | 323 |
| | 11 | PPT_wt | 476 | 71 | 1,683 | 328 |
| | 12 | PPT_sp | 283 | 56 | 955 | 173 |
| | 13 | PPT_sm | 185 | 31 | 522 | 77 |
| | 14 | PPT_fl | 355 | 58 | 1,329 | 249 |
| Physiographic | 15 | Forest | 61 | 0 | 100 | 25 |
| | 16 | Water_Wetland | 1 | 0 | 18 | 2 |
| | 17 | Urban | 2 | 0 | 100 | 12 |
| | 18 | CN | 68 | 55 | 94 | 6 |

### C.5.2. Formation of Hydrological Regions

Based on an interative selection process, the 1,015 hydrometric stations were ultimately organized into 10 clusters. The results of the Elbow Method showed that a selection of approximately 10 hydrological regions explained the most variance in the catchment characteristics (Figure C-3).

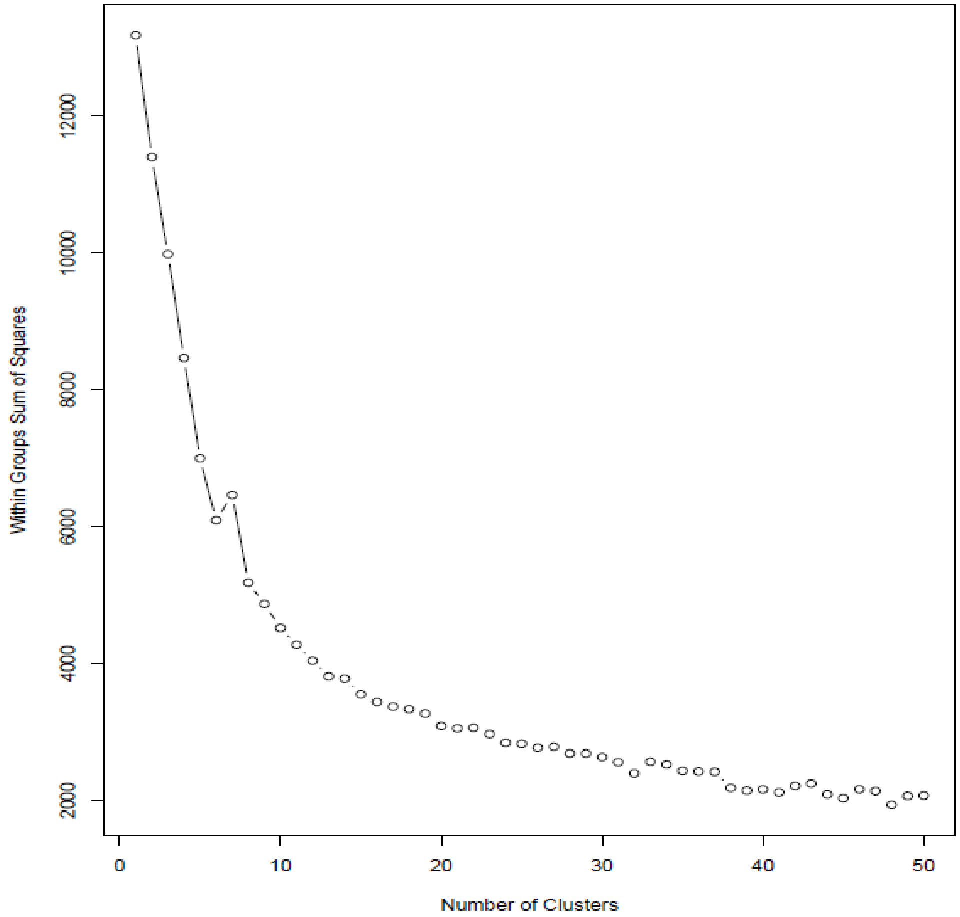

**Figure C-3. The Elbow Plot.**

The Silhouette Scores for the 10 clusters suggested some difficulty in organising the hydrometric stations based on catchment characteristics (Figure C-4). The average Silhouette Score is 0.2, suggesting that the hydrometric stations are poorly assigned to their hydrological regions. A low Silhouette Score is expected however, as it reflects the physical variability across the study area.

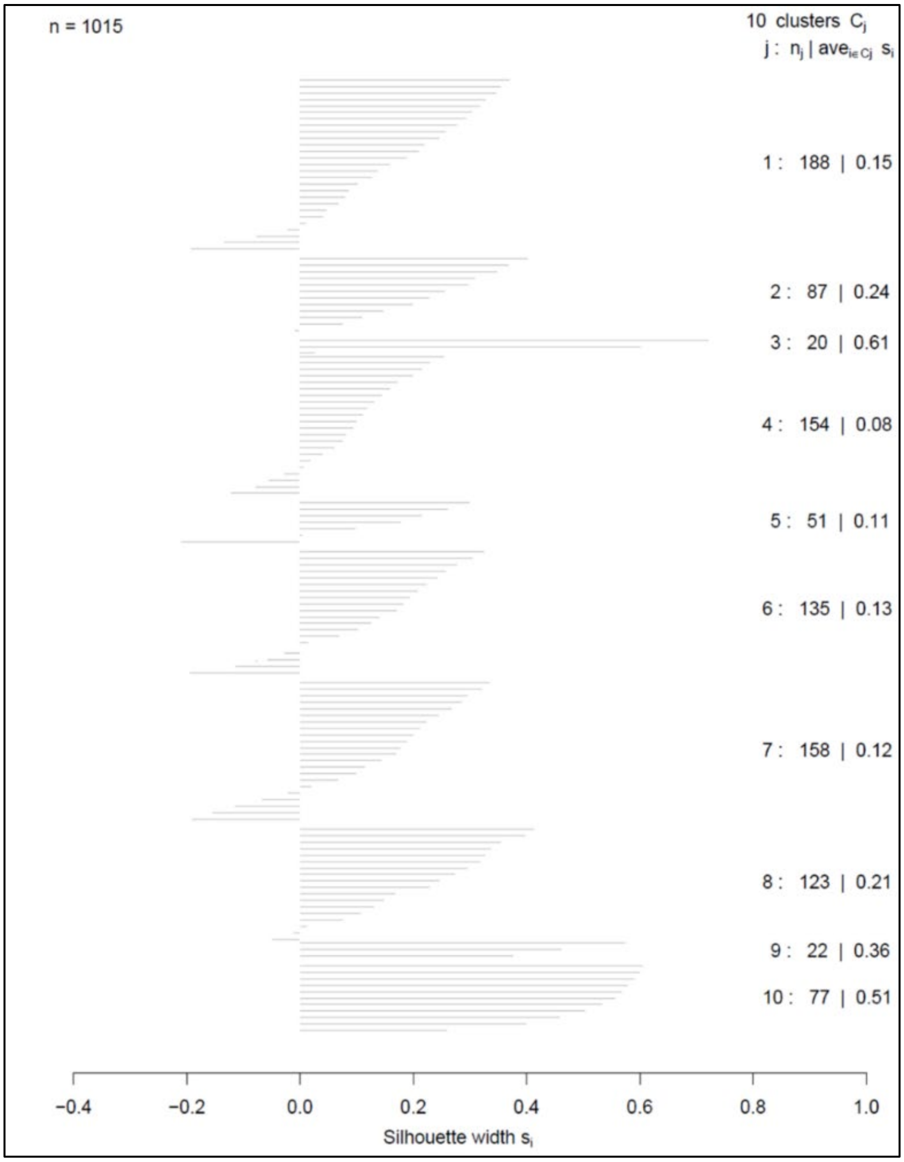

**Figure C-4. Silhouette score.**

The organization of the hydrometric stations into clusters is compiled in a dendrogram (Figure C-5). The y-axis is the dissimilarity index based on the distance metric. The horizontal axis represents the Ward's Distance (d). The green boxes separate the clusters. The 10 clusters are shown along the bottom of the dendrogram. Because we do not know how many clusters there should be in the landscape, the merging process was stopped once the clusters were more dissimilar than a threshold of approximately 90. The threshold was selected to generate a number of clusters consistent with the Elbow Plot.

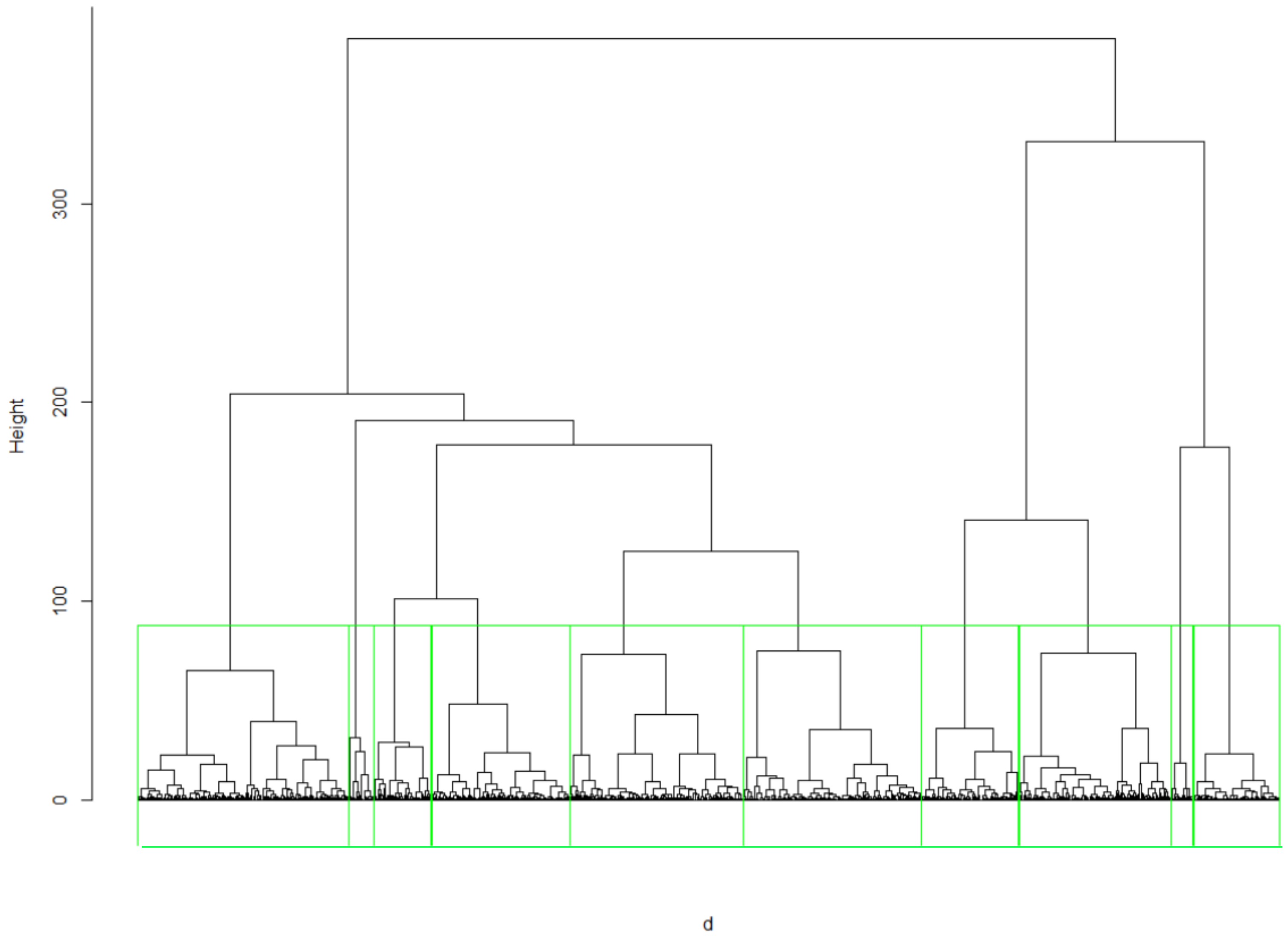

**Figure C-5. Dendrogram.**

### C.5.2.1. Physical Basis of Regions and Flood Characteristics

The spatial distribution of the clusters is considered physically plausible, considering the range in the climate catchment characteristics. Significant regional variations are expected due to the influence of the mountain ranges across the study area (e.g., Coast Mountains, Monashees, the Columbia Trench, and the Rocky Mountains). These orographic effects are expected to control, at least in part, the distribution clusters (Figure C-6).

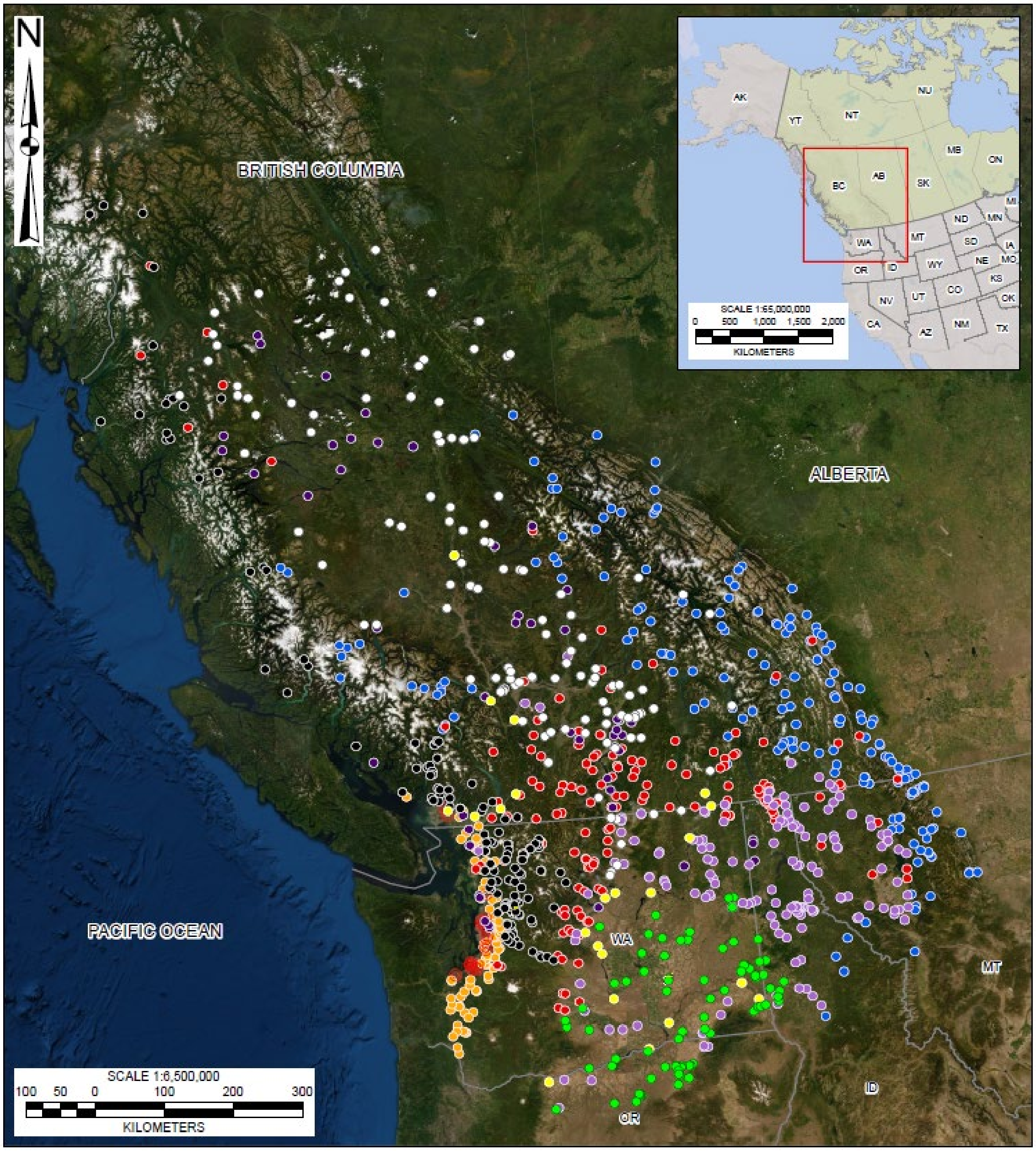

Figure C-6. Spatial distribution of 10 clusters.

The clusters that cover the RDCK region include 1 (blue), 4 (red), and 7 (lilac) with 188, 154, and 158 hydrometric stations, respectively. Cluster 1 is defined by the influence of the Rocky Mountains to the east forming the physiographic boundary with Alberta. Most flood events in this cluster are caused by snowmelt or rain-on-snow events in the spring. The eastern range of the Coastal mountains to the west also includes a small group of hydrometric station assigned to Cluster 1. Cluster 4 is defined generally by a climate characteristic of the semi-arid plateau between major mountain ranges. Most flood events are snowmelt dominated in the spring. In this drier climate, evaporation from water surfaces and from the land as well as transpiration from vegetation make up a large component of the regional water balance. Additional hydrometric stations assigned to Cluster 4 are in the montane cordillera to the east where flood events are often associated with rain-on-snow events during the spring freshet. Cluster 7 is defined by the southern edge of the Rocky Mountains in northwestern Montana. Significant floods in this region are caused by runoff from rain associated with moist air masses from the Gulf of Mexico, although most annual peak streamflow events are from snowmelt or rain-on-snow events in the spring.

### C.5.2.2. Manual Adjustments

The clusters were further separated manually due to the large number of hydrometric stations in each cluster. Cluster 1 was separated into the eastern and western ranges of the Rocky Mountains. The small group of hydrometric stations located along the eastern range of the Coastal Mountains were also separated from Cluster 1. Cluster 4 was separated into the eastern portion in the montane cordillera and the western portion in the semi-arid plateau. Cluster 7 was not separated due to the limited geographic spread of the hydrometric stations. Based on these manual adjustments, Cluster 1 West, 4 East, and 7 cover the RDCK region (Figure C-7).

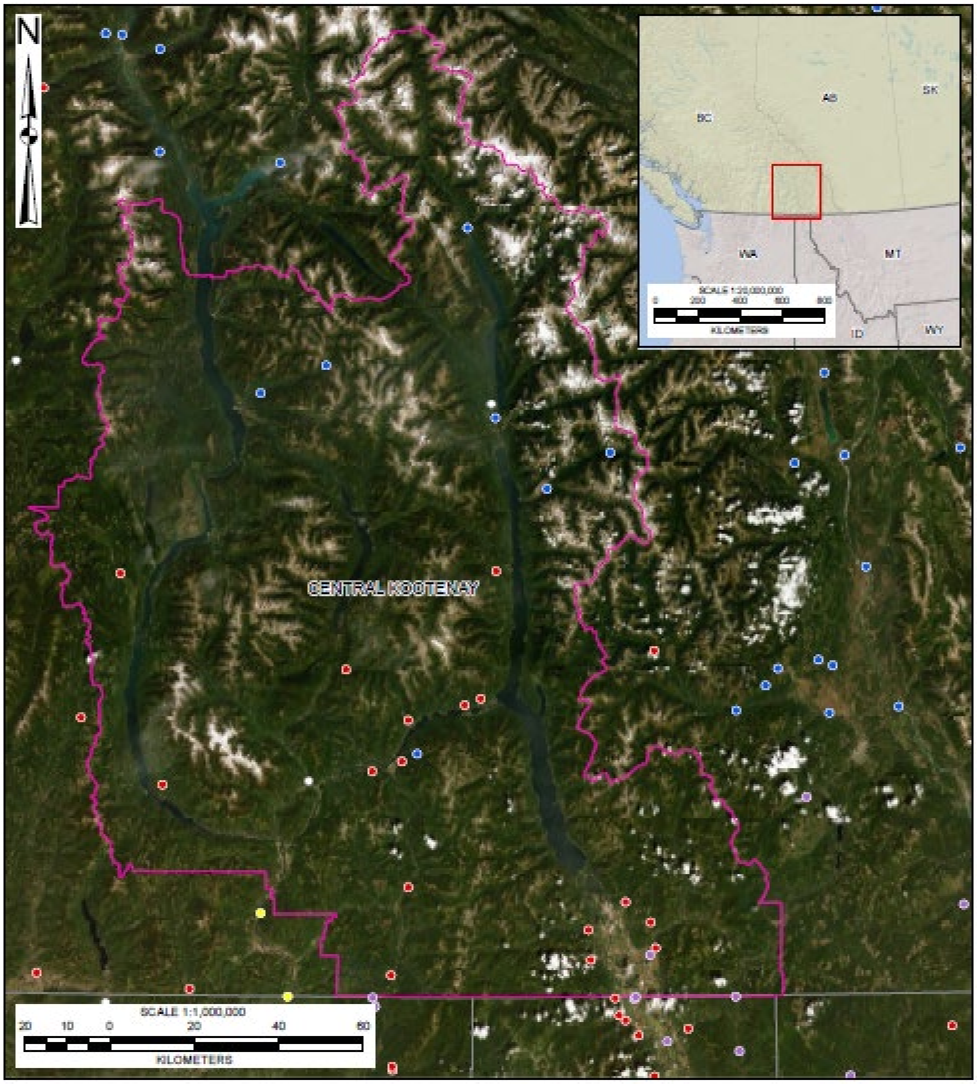

**Figure C-7. Clusters that cover the RDCK region.**

The clusters were further separated based on the scale of catchment area. The coefficient of variation (CV) is required to be constant for a given homogeneous region. A relationship between the catchment area and L-CV is observed in the clusters that cover the RDCK. However, the strength of the relationship varies considerably (Table C-11). In a flood regionalization study in British Columbia, Wang (2000) observed that in L-moment space, the L-CV varied with catchment area for the defined clusters making them heterogeneous. Wang (2000) demonstrated that the small catchments show an increase and the large catchments show a decrease in the L-CV.

**Table C-11. $R^2$ for regression between catchment area and L-CV**

| Cluster | Number of Hydrometric Stations | R2 for regression between catchment area and L-CV |
|---|---|---|
| 1 West | 88 | 0.01 |
| 4 East | 45 | 0.12 |
| 7 | 158 | 0.15 |

To account for the lack of constancy in the L-CV reported by Wang (2000) and observed in the clusters, the range in the catchment area considered in the study was modified to include two groups: 1) less than 500 $km^2$ and 2) more than 500 $km^2$ up to 5,000 $km^2$. The clusters that cover the RDCK region thus include the following which will be the focus of the results herein.

- Cluster 1 West < 500 $km^2$
- Cluster 1 West > 500 $km^2$
- Cluster 4 West < 500 $km^2$
- Cluster 4 West > 500 $km^2$
- Cluster 7 < 500 $km^2$
- Cluster 7 > 500 $km^2$.

### C.5.2.3. Refinement of the Hydrometric Station Selection

The final number of hydrometric stations, including the range of discordancy (*Di*) values, for each hydrologic region is presented in Table C-12. The number of hydrometric stations removed is based on the criteria presented in Section C.4.2.4.

**Table C-12. Final number of hydrometric stations and range in discordancy measure for each hydrologic region.**

| Cluster | Catchment Area Range | Initial Number of Hydrometric Stations | Number of Hydrometric Stations Removed | Final Number of Hydrometric Stations | Di (Min) | Di (Max) | Di (Mean) |
|---|---|---|---|---|---|---|---|
| **1 West** | < 500 $km^2$ | 36 | 10 | 26 | 0.13 | 3.0 | 1 |
| | > 500 $km^2$ | 52 | 28 | 24 | 0.09 | 3.0 | 1 |
| **4 East** | < 500 $km^2$ | 43 | 9 | 34 | 0.04 | 2.8 | 1 |
| | > 500 $km^2$ | 2 | Not enough data for regionalisation | | | | |
| **7** | < 500 $km^2$ | 75 | 35 | 40 | 0.09 | 2.6 | 1 |
| | > 500 $km^2$ | 83 | 65 | 18 | 0.11 | 2.9 | 1 |

### C.5.2.4. Homogeneity

The H-Test results are summarized in Table C-13. A cluster is declared heterogeneous if H is sufficiently "large". Hosking and Wallis (1997) recommend a cluster be considered "definitely heterogeneous" if H $\geq$ 2. Increasing the threshold implies that more heterogeneous regions are included in the analysis. Guse, Thieken, Castellarin, & Merz (2010) assessed the effect of the H-Test threshold on the performance of probabilistic regional envelope curves in Germany. Increasing the H-Test threshold from 2 to 4 resulted in a larger number of regions considered for analysis. This increase is important as it can include hydrometric stations that would have been excluded otherwise.

The reality is that while removing hydrometric stations may improve the homogeneity of a region, there may be some important reasons why the H-Test score is high. For example, the site may include a hydrometric station where a very large flood occurred. A representative heterogeneous region is better than a region that has been forced to be homogeneous (Robson and Reed 1999).

The physical variability of British Columbia was recognized by Wang (2000) where the average value for the H-Test was 6.85 based on 19 clusters. The physiographic regions in BC may be less distinct than other regions. As a result, the threshold for the H-Test was relaxed to what is practical for British Columbia.

**Table C-13. Number of hydrometric stations, Discordancy values, and H-Test results.**

| Hydrologic Region | Catchment Area Range | Number of Hydrometric Stations | H-Test |
|---|---|---|---|
| **1 West** | < 500 km$^2$ | 26 | 6.8 |
| | > 500 km$^2$ | 24 | 9.0 |
| **4 East** | < 500 km$^2$ | 34 | 13.1 |
| | > 500 km$^2$ | 2 | Not enough data |
| **7** | < 500 km$^2$ | 40 | 4.5 |
| | > 500 km$^2$ | 18 | 7.7 |

## C.5.3. Regionalization

### C.5.3.1. Regional Probability Distributions

The regionally averaged L-moments are presented in Table C-14 for hydrologic region 1 West, 4 East, and 7. For the index-flood procedure, $\iota_1$ is set to 1.

**Table C-14. Regionally averaged L-moments.**

| Hydrologic Region | Catchment Area Range | Number of Hydrometric Stations | $\iota_1$ | $\iota_2$ | $t_3$ | $t_4$ |
|---|---|---|---|---|---|---|
| 1 West | < 500 km$^2$ | 26 | 1 | 0.1796 | 0.2519 | 0.1879 |
| | > 500 km$^2$ | 24 | 1 | 0.1756 | 0.2411 | 0.2012 |
| 4 East | < 500 km$^2$ | 34 | 1 | 0.2364 | 0.2245 | 0.1624 |
| 7 | < 500 km$^2$ | 40 | 1 | 0.3014 | 0.2539 | 0.1904 |
| | > 500 km$^2$ | 18 | 1 | 0.2601 | 0.2138 | 0.1924 |

The Z-statistics for a range of candidate probability distributions is presented in Table C-15. The candidate probability distributions include GLO, GEV, GPA, GNO, and PE3. Probability distributions with Z statistics $\leq$1.64 are deemed acceptable (Hosking & Wallis 1997). All candidate distributions are deemed acceptable for the hydrologic regions that cover the RDCK based on the Z-statistic.

**Table C-15. Goodness of fit Z statistic for probability distribution selection.**

| Hydrological Region | Catchment Area Range | GLO | GEV | GNO | PE3 | GPA |
|---|---|---|---|---|---|---|
| 1 West | < 500 km$^2$ | 1.30 | -0.34 | -1.14 | -2.57 | -4.47 |
| | > 500 km$^2$ | 0.53 | -1.59 | -2.50 | -4.16 | -6.85 |
| 4 East | < 500 km$^2$ | 3.30 | 0.69 | -0.21 | -1.92 | -5.60 |
| 7 | < 500 km$^2$ | 1.41 | -0.59 | -1.59 | -3.38 | -5.66 |
| | > 500 km$^2$ | 0.62 | -1.79 | -2.55 | -4.01 | -7.54 |

To help make the decision on the most representative probability distribution, L-moment diagrams were plotted for each hydrologic region. The $t_3$ and $t_4$ position of the regional average relative to the relationships for five three-parameter (GLO, GEV, GPA, GNP, PE3) and five two-parameter (E, G, L, N, and U) candidate probability distributions are depicted in Figure C-8. The three-parameter probability distributions are depicted by the coloured lines while the two-parameter distributions are depicted by the black squares. The L-skewness and L-kurtosis ratio for each hydrologic region is depicted by the cross symbol on Figure C-8. The GEV probability distribution gives an acceptably close fit to the regional L-moments for the different hydrologic regions. As a result, the GEV probability distribution was deemed representative for all hydrologic regions.

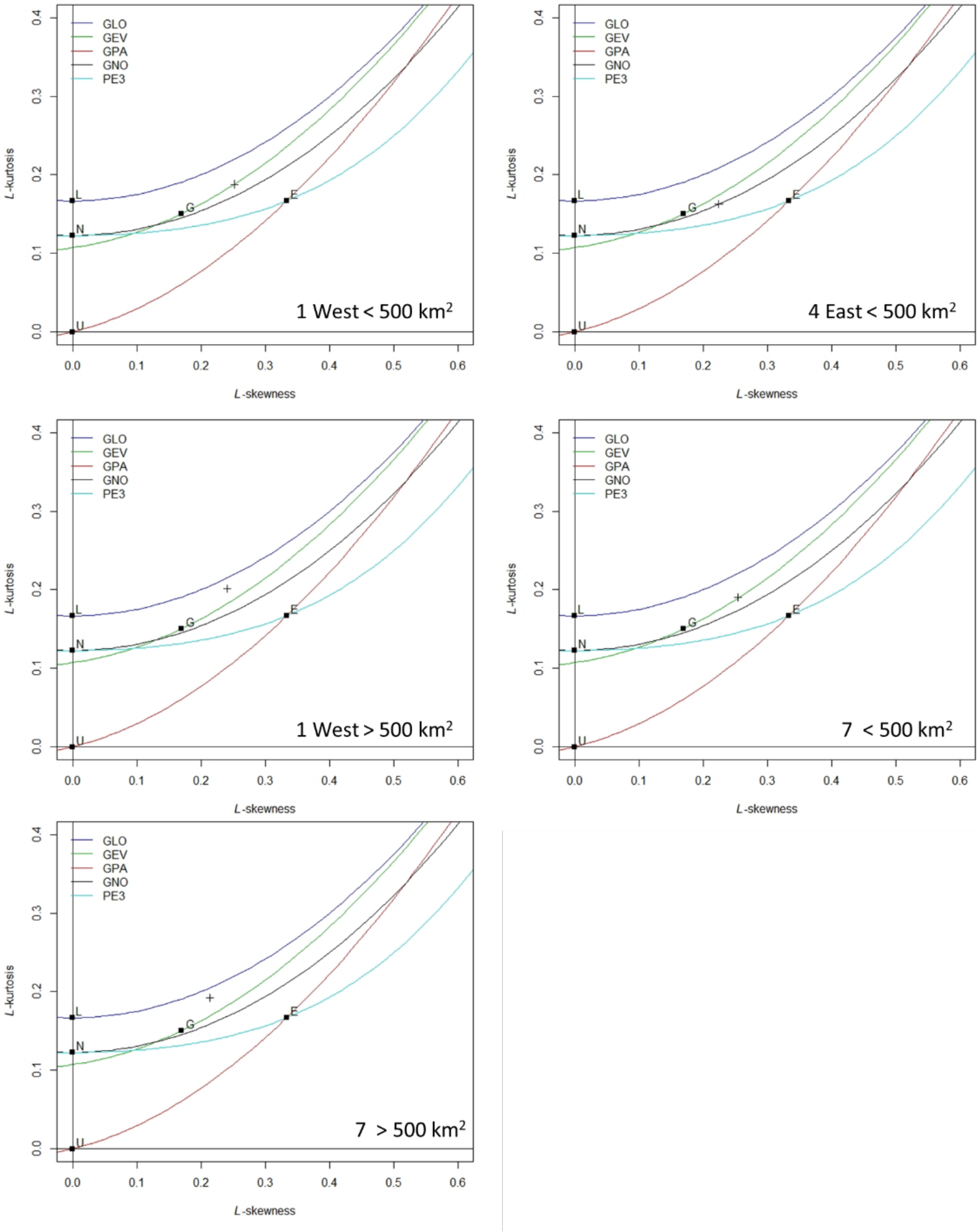

Figure C-8. L-moment ratio diagram for each hydrologic region.

### C.5.3.2. Parameter Estimation

The regionally weighted L-moments are used to estimate the parameters of the GEV probability distribution. The parameters for each hydrologic region are presented in Table C-16.

**Table C-16. Parameter estimates for the GEV distribution.**

| Hydrological Region | Catchment Area limit | $\xi$ | $\alpha$ | $\kappa$ |
|---|---|---|---|---|
| **1 West** | < 500 km$^2$ | 0.8369 | 0.2280 | -0.1236 |
| | > 500 km$^2$ | 0.8421 | 0.2269 | -0.1078 |
| **4 East** | < 500 km$^2$ | 0.7908 | 0.3139 | -0.0832 |
| **7** | < 500 km$^2$ | 0.7257 | 0.3814 | -0.1266 |
| | > 500 km$^2$ | 0.7724 | 0.3513 | -0.0671 |

### C.5.3.3. Growth Curves and Error Bounds

The regional growth curves and error bounds are presented for each region in Figure C-9.

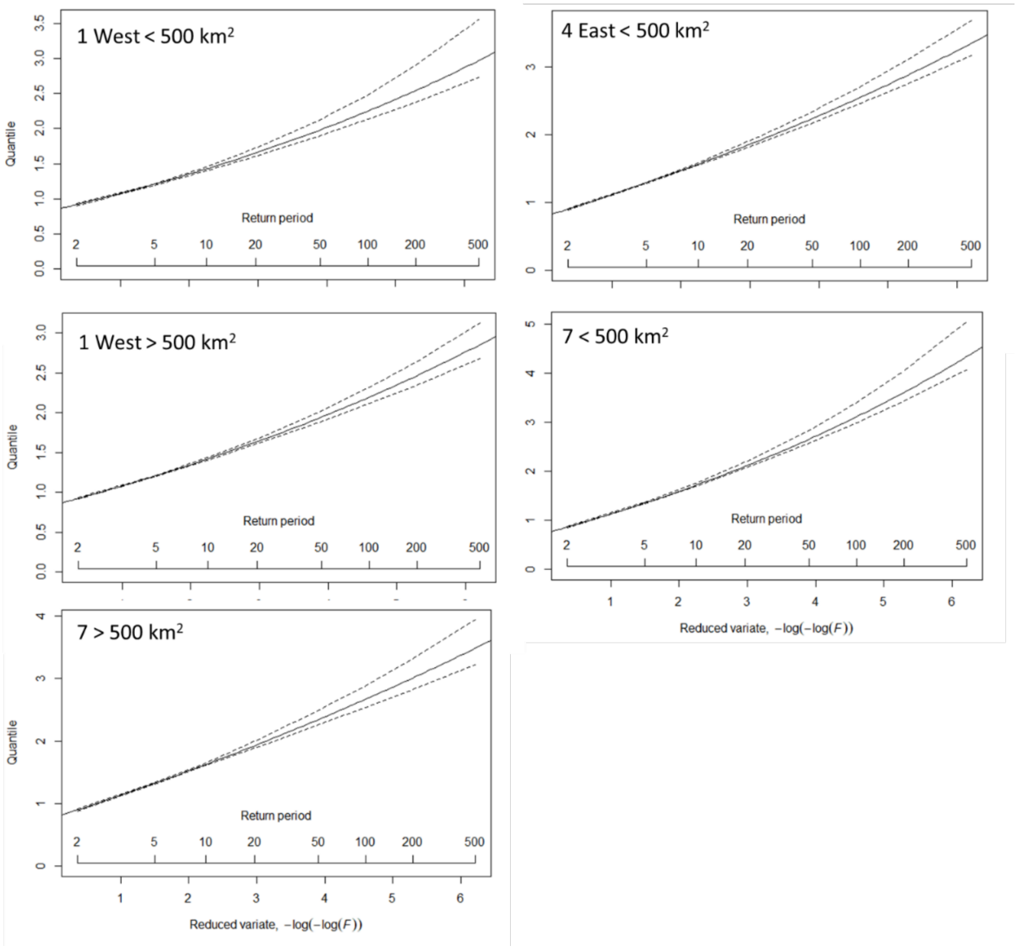

**Figure C-9. Growth curves for each hydrologic region.**

### C.5.3.4. Index Flood

The regional equations for the index-flood for each hydrologic region are presented in Table C-17. The provincial equations are also included at the end of Table C-17. The results are reported to 5 significant figures. However, a total of 5 equations are developed for each hydrologic region and across the province with the intention to average the index-flood estimates. Consequently, the results should be rounded to the nearest unit for flood magnitudes greater than 10 $m^3/s$. The adjusted $R^2$ is included for comparison of the models. Models with more catchment characteristics tend to have a lower adjusted $R^2$ as these models are penalized for increased number of independent variables.

**Table C-17. Regional and provincial equations for the index-flood including the adjusted R$^2$.**

| Hydrologic Region | Catchment Area Range | Index-flood Equations | | Adj. R$^2$ |
|---|---|---|---|---|
| **1 West < 500 km$^2$** | 42 to 454 km$^2$ | 1 | $\log Q_m = 10.169 + 1.8553(\log Area) - 0.012434(Slope) + 0.098984\,(Cen\_Long) + 0.0055555(PPT_{fl}) + 0.34911(Water\_Wetland)$ | 0.91 |
| | | 2 | $\log Q_m = 12.127 + 1.9358(\log Area) - 0.013271(Slope) + 0.11264\,(Cen\_Long) - 0.00022260(Cen\_Elev) + 0.0053230(PPT_{fl}) + 0.40695(Water\_Wetland)$ | 0.92 |
| | | 3 | $\log Q_m = 6.951 + 1.8564(\log Area) - 0.011048(Slope) + 0.071361\,(Cen\_Long) + 0.0053236(PPT_{fl})$ | 0.90 |
| | | 4 | $\log Q_m = -0.96349 + 1.7509(\log Area) - 0.0095976(Slope) + 0.0043293(PPT_{fl})$ | 0.89 |
| | | 5 | $\log Q_m = -3.2303 + 2.1932(\log Area) + 0.0015075(MAP)$ | 0.88 |
| **1 West > 500 km$^2$** | 586 to 4312 km$^2$ | 1 | $\log Q_m = -2.5781 + 2.0480(\log Area) + 0.0012740\,(MAP)$ | 0.83 |
| | | 2 | $\log Q_m = -2.3716 + 1.8939(\log Area) + 0.41806(\log Catch\_Length) + 0.0012775(MAP)$ | 0.82 |
| | | 3 | $\log Q_m = 1.3411 + 1.9306(\log Area) + 0.18827(\log Catch\_Length) + 0.0011046\,(MAP) - 0.04866(CN)$ | 0.82 |
| | | 4 | $\log Q_m = -0.70946 + 1.6015(\log Area) - 0.0081664(Slope) + 0.0013574\,(MAP) + 0.057906\,(MAT) - 0.0036032(Forest)$ | 0.83 |
| | | 5 | $\log Q_m = 0.40059 + 1.6514(\log Area) - 0.0082135(Slope) + 0.0010135\,(MAP) + 0.15045\,(MAT) - 0.016425(Forest) - 0.19361(Water\_Wetland)$ | 0.88 |

| Hydrologic Region | Catchment Area Range | Index-flood Equations | | Adj. $R^2$ |
|---|---|---|---|---|
| **4 East < 500 km$^2$** | 6 to 441 km$^2$ | 1 | $\log Q_m = -3.5763 + 2.7620(\log Area) - 0.15167(MAT) + 0.0035040(PPT_{wt}) - 0.26513(Water\_Wetland)$ | 0.96 |
| | | 2 | $\log Q_m = -4.1636 + 2.7871(\log Area) + 0.0037150(PPT_{wt}) - 0.30562(Water\_Wetland)$ | 0.96 |
| | | 3 | $\log Q_m = -1.8437 + 2.6974(\log Area) + 0.0038(PPT_{wt}) - 0.18063(MAT) + 0.0030438(PPT_{wt}) - 0.28288(Water_{Wetland}) - 0.020392(CN)$ | 0.96 |
| | | 4 | $\log Q_m = -4.0189 + 2.7063(\log Area) + 0.0047397\left(PPT_{fl}\right) - 0.3056(Water\_Wetland)$ | 0.95 |
| | | 5 | $\log Q_m = -1.3176 + 2.6880(\log Area) - 0.00069570(MAP) - 0.19022(MAT) + 0.0044279(PPT_{wt})$ | 0.96 |
| **7 < 500 km$^2$** | 8 to 471 km$^2$ | 1 | $\log Q_m = -3.8856 + 1.8844(\log Area) + 0.010435\left(PPT_{fl}\right)$ | 0.74 |
| | | 2 | $\log Q_m = -3.9002 + 1.9484(\log Area) + 0.10058\left(PPT_{fl}\right) - 0.17007(Water\_Wetland)$ | 0.74 |
| | | 3 | $\log Q_m = -4.4499 + 2.0486(\log Area) + 0.0051660(PPT_{wt}) + 0.0062765(PPT_{sm}) - 0.21014(Water\_Wetland)$ | 0.74 |
| | | 4 | $\log Q_m = -20.730 + 1.7210(\log Area) + 0.36720(Cen\_Lat) - 0.00093400(Cen_{Elev}) + 0.13920\left(PPT_{sp}\right) - 0.30900(Water\_Wetland)$ | 0.75 |
| | | 5 | $\log Q_m = -1.9967 + 2.9199(\log Area) - 0.44581(\log Catch\ Length) + 0.22219(Cen\_Lat) + 0.11838(Cen\_Long) + 0.007305(PPT_{wt}) - 0.32687(Water\_Wetland)$ | 0.75 |

| Hydrologic Region | Catchment Area Range | Index-flood Equations | | Adj. $R^2$ |
|---|---|---|---|---|
| **7 >500 $km^2$** | 529 to 4138 $km^2$ | 1 | $\log Q_m = -2.8251 + 2.0765(\log Area) - 0.65058(MAT) - 0.01087(PAS) + 0.15245(PPT_{wt}) + 0.014215(PPT_{sm}) + 0.14232(Forest)$ | 0.93 |
| | | 2 | $\log Q_m = 0.51542 + 1.4852(\log Area) - 0.024121(Slope) - 0.0078710(MAP) - 0.69867(MAT) - 0.010055(PAS)$ | 0.93 |
| | | 3 | $\log Q_m = -0.28887 + 2.1311(\log Area) - 0.00048080(Cen_{Elev}) - 0.59076(MAT) - 0.10256(PAS) + 0.14034(PPT_{wt}) + 0.14291(PPT_{sm}) + 0.018084(Forest)$ | 0.94 |
| | | 4 | $\log Q_m = -12.290 + 4.2860(\log Area) - 4.4640(\log Catch\_Length) + 0.54240(Cen\_Lat) + 0.19690(Cen\_Long) - 0.0066490(PAS) + 0.013790(PPT_{wt}) + 0.38640(Forest)$ | 0.94 |
| | | 5 | $\log Q_m = -6.0632 + 2.1265(\log Area) + 0.0053923(PPT_{wt}) + 0.030556(Forest)$ | 0.90 |
| **Provincial Model** | 1 to 4,888 $km^2$ | 1 | $\log Q_m = -10.280 + 2.0840(\log Area) - 0.052950(Cen\_Long) + 0.00078170(PAS) + 0.0045490(PPT_{sp}) - 0.077680(Water\_Wetland) + 0.015770(CN)$ | 0.88 |
| | | 2 | $\log Q_m = -10.990 + 2.0900(\log Area) - 0.054870(Cen\_Long) + 0.00079820(PAS) + 0.0045680(PPT_{sp}) + 0.0022550(Forest) - 0.079050(Water\_Wetland) + 0.020340(CN)$ | 0.88 |
| | | 3 | $\log Q_m = -9.7160 + 2.0890(\log Area) - 0.044870(Cen_{Long}) - 0.00015400(Cen\_Elev) + 0.00095000(PAS) + 0.0043910(PPT_{sp}) + 0.0027010(Forest) - 0.081050(Water\_Wetland) + 0.021030(CN)$ | 0.89 |
| | | 4 | $\log Q_m = -8.3390 + 2.0610(\log Area) - 0.047040(Cen_{Long}) + 0.00070070(PAS) + 0.0043090(PPT_{sp}) + 0.0027010(Forest)$ | 0.88 |
| | | 5 | $\log Q_m = -2.7860 + 2.0520(\log Area) - 0.0023640(PPT_{wt}) + 0.0028430(PPT_{sm}) - 0.063700(Water\_Wetland)$ | 0.88 |

### C.5.4. Error Statistics

The weighted standardized error statistics for the regional and provincial model over a range of flood quantiles for the different hydrologic regions are presented in Table C-18. The error statistics are not consistent across all hydrologic regions. The regional model may be selected for the 4 East < 500 km$^2$ hydrologic region. In the case of the 1 West region, either the regional or provincial model would be considered adequate. Lastly, the regional model is probably the model of choice for the 7 hydrologic region. As expected, the error statistics for the lower flood quantiles are lower than those for higher flood quantiles reflecting the increased uncertainty in higher quantile estimates.

**Table C-18.** Weighted standardized error statistics for the regional and provincial models over a range of flood quantiles. Green highlighted cells depict a positive bias while the red highlighted cells depict a negative bias.

| Error Stats | AEP | 1 West < 500 km$^2$ | | 1 West > 500 km$^2$ | | 4 East < 500 km$^2$ | | 7 < 500 km$^2$ | | 7 > 500 km$^2$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Regional Qm | Provincial Qm | Regional Qm | Provincial Qm | Regional Qm | Provincial Qm | Regional Qm | Provincial Qm | Regional Qm | Provincial Qm |
| SRMSE | 0.5 | 0.24 | 0.31 | 0.27 | 0.26 | 0.39 | 0.92 | 2.71 | 3.80 | 0.19 | 0.99 |
| | 0.1 | 0.28 | 0.31 | 0.26 | 0.28 | 0.33 | 0.69 | 3.08 | 4.10 | 0.21 | 0.96 |
| | 0.02 | 0.40 | 0.41 | 0.31 | 0.33 | 0.38 | 0.64 | 3.70 | 4.80 | 0.27 | 1.01 |
| | 0.005 | 0.54 | 0.53 | 0.38 | 0.39 | 0.45 | 0.66 | 4.37 | 5.59 | 0.36 | 1.09 |
| SPercent Error | 0.5 | 18 | 21 | 20 | 21 | 27 | 59 | 70 | 122 | 15 | 65 |
| | 0.1 | 22 | 24 | 20 | 24 | 22 | 45 | 74 | 128 | 14 | 65 |
| | 0.02 | 31 | 32 | 25 | 29 | 27 | 39 | 84 | 144 | 20 | 68 |
| | 0.005 | 42 | 40 | 30 | 33 | 34 | 38 | 97 | 165 | 29 | 74 |
| SBIAS | 0.5 | 0.03 | -0.08 | 0.04 | -0.09 | 0.07 | 0.30 | 0.39 | 1.03 | 0.03 | 0.39 |
| | 0.1 | 0.06 | -0.06 | 0.04 | -0.07 | 0.07 | 0.23 | 0.44 | 1.08 | 0.03 | 0.39 |
| | 0.02 | 0.09 | -0.03 | 0.06 | -0.06 | 0.08 | 0.20 | 0.52 | 1.21 | 0.04 | 0.42 |
| | 0.005 | 0.13 | 0.02 | 0.08 | -0.03 | 0.10 | 0.20 | 0.62 | 1.37 | 0.06 | 0.45 |

## C.6. APPLICATION TO UNGAUGED CATCHMENTS

The goal of the regionalization of floods is to estimate quantiles for ungauged catchments in the RDCK. A total of 12 catchments are modeled for clearwater floods. To begin, a catchment polygon was defined for each ungauged catchment, as shown in Figure C-10. The suite of 18 catchment characteristics were then extracted and averaged over the area for each ungauged catchment. The resulting catchment characteristics are presented in Table C-19.

The ungauged catchments were subsequently assigned to one of the hydrologic regions identified across the study area. The hydrologic region assignment was completed using the Random Forest classification algorithm. Once a hydrologic region was assigned to the ungauged catchment; the index-flood was estimated based on the appropriate model (regional and / or provincial). The flood quantiles were then estimated for a range of AEPs using the index-flood estimate and the appropriate regional growth curve. The hydrologic region assignment, index-flood estimate, and flood quantiles for each ungauged catchment are presented in Table C-20.

The magnitude of the flood quantiles is influenced by the catchment characteristics. This is because the index-flood is calculated using a multiple linear regression that depends on the catchment characteristics that define the best 5 models for a given region. Two catchments of similar area may have significantly different flood quantile estimates because of major differences in catchment characteristics. For example, Lost Creek and Porcupine Creek share comparable catchment areas of 62 km$^2$ and 68 km$^2$, respectively. However, flood quantiles for Porcupine Creek are 35% greater than Lost Creek, with the difference in magnitude attributed to difference in climate characteristics.

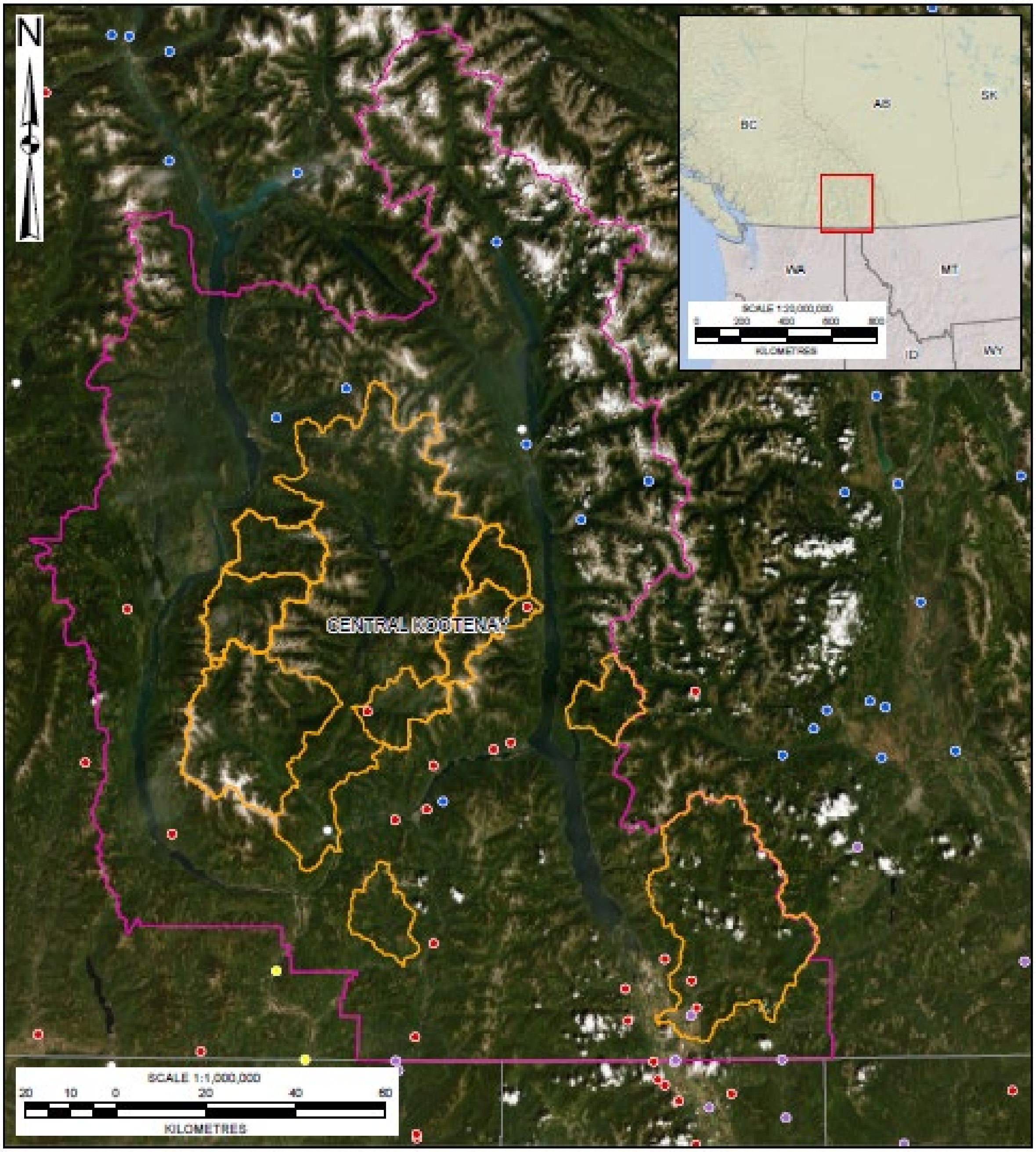

Figure C-10. Catchment polygons for the ungauged catchments.

**Table C-19. Catchment characteristics for the clearwater sites located in the RDCK region.**

| Catchment Name | Area (km$^2$) | Relief (m) | Catchment Length (km) | Slope (%) | Centroid Latitude (degrees) | Centroid Longitude (degrees) | Centroid Elevation (m) | MAP (mm) | MAT (ºC) | PAS (mm) | PPT_wt (mm) | PPT_sp (mm) | PPT_sm (mm) | PPT_fl (mm) | Forest (%) | Water and Wetland (%) | Urban (%) | CN |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Crawford Creek | 186 | 2092 | 2.53 | 83 | 49.693818 | -116.700089 | 1181 | 1116 | 3.0 | 590 | 383 | 233 | 198 | 302 | 88 | 0.0 | 0.2 | 70 |
| Keen Creek | 202 | 2066 | 2.37 | 87 | 49.861962 | -117.119617 | 1584 | 1390 | 1.3 | 857 | 460 | 307 | 240 | 384 | 66 | 0.2 | 7.7 | 67 |
| Upper Kaslo Creek | 150 | 1927 | 2.35 | 82 | 49.990505 | -117.046683 | 1182 | 1244 | 2.7 | 668 | 416 | 265 | 223 | 340 | 90 | 0.0 | 0.8 | 70 |
| Kalso Creek at Kootenay Lake | 386 | 2228 | 3.09 | 72 | 49.914818 | -117.077853 | 1280 | 1312 | 2.1 | 756 | 438 | 284 | 230 | 360 | 78 | 0.2 | 4.3 | 68 |
| Lemon Creek | 206 | 2046 | 2.58 | 79 | 49.717145 | -117.338618 | 1956 | 1322 | 2.7 | 754 | 461 | 284 | 206 | 370 | 90 | 0.1 | 0.7 | 65 |
| Burton at Arrow Lake | 530 | 2323 | 4.13 | 56 | 49.952644 | -117.773748 | 1300 | 1242 | 2.4 | 704 | 4280 | 258 | 220 | 336 | 85 | 0.3 | 1.2 | 64 |
| Caribou Creek | 238 | 2235 | 2.97 | 75 | 50.019565 | -117.726695 | 1213 | 1260 | 2.4 | 709 | 432 | 261 | 226 | 341 | 92 | 0.1 | 0.3 | 67 |
| Snow Creek | 291 | 2314 | 3.05 | 76 | 49.897831 | -117.811685 | 1742 | 1227 | 2.3 | 700 | 425 | 255 | 216 | 331 | 80 | 0.3 | 1.8 | 63 |
| Little Slocan River | 818 | 2281 | 5.40 | 42 | 49.664986 | -117.79715 | 1612 | 1161 | 2.8 | 643 | 416 | 245 | 188 | 313 | 82 | 0.5 | 1.7 | 63 |
| Slocan River | 3475 | 2544 | 8.13 | 31 | 49.85497 | -117.525816 | 1196 | 1224 | 3.0 | 666 | 431 | 256 | 206 | 332 | 81 | 2.9 | 2.1 | 66 |
| Goat River | 1259 | 2111 | 6.01 | 35 | 49.28428 | -116.347233 | 1050 | 857 | 3.2 | 433 | 284 | 194 | 163 | 217 | 88 | 0.1 | 0.2 | 69 |
| Erie Creek Upstream End | 201 | 1575 | 2.71 | 58 | 49.288665 | -117.392234 | 1010 | 1265 | 3.8 | 617 | 435 | 286 | 210 | 333 | 95 | 0.0 | 0.0 | 62 |

**Table C-20. Hydrologic region assignment for the ungauged catchments.**

| Catchment Name | Hydrometric Station | Catchment Area (km$^2$) | Hydrologic Region[1] | Qm (m$^3$/s) | Flood Quantiles | | |
|---|---|---|---|---|---|---|---|
| | | | | | 0.05 AEP (m$^3$/s) | 0.02 AEP (m$^3$/s) | 0.005 AEP (m$^3$/s) |
| Crawford Creek | - | 186 | 7 | 27 | 50 | 61 | 80 |
| Keen Creek | 08NH132 | 202 | pro-rated | - | 78 | 94 | 125 |
| Upper Kaslo Creek | 08NH005 | 150 | pro-rated | - | 99 | 120 | 160 |
| Kaslo Creek at Kootenay Lake | 08NH005 | 386 | pro-rated | - | 160 | 200 | 260 |
| Lemon Creek | 08NJ160 | 206 | pro-rated | - | 72 | 84 | 105 |
| Burton at Arrow Lake | - | 530 | 4 | 80 | 150 | 180 | 230 |
| Caribou Creek | - | 238 | 4 | 42 | 78 | 94 | 120 |
| Snow Creek | - | 291 | 4 | 45 | 83 | 100 | 130 |
| Little Slocan River | | 818 | 4 | 103 | 190 | 230 | 290 |
| Slocan River | 08NJ013 | 3475 | pro-rated | - | 685 | 770 | 880 |
| Goat River | 8NH004 | 1259 | 7 | - | 387 | 430 | 500 |
| Erie Upstream End | - | 201 | 4 | 35 | 65 | 79 | 102 |

Note:

1. A pro-rated calculation is completed when a representative hydrometric station is located upstream or downstream from the ungauged site and has a record length considered long enough for reliable frequency analysis. Flood quantile estimates calculated at the hydrometric station are transferred to the ungauged site by relating the annual maximum peak instantaneous streamflow at the hydrometric station to the ungauged site using catchment area size.

## C.7. UNCERTAINTY

The process of flood regionalization is inherently uncertain because of the several limitations. The probability distribution of flood events is unknown. While there are statistical tools to help reach a ‘best estimate’, it is not possible to know what the probability distribution is in practice. As a result, the flood quantile estimates are supported by a mathematical model that is considered reliable based on the available flood data.

The regionalisation of floods tends to underestimate peak flows for small catchments and overestimate peak flows for larger catchments. This is in part due to differences in hydrological processes that control peak flows. For example, maximum annual peak instantaneous flows in small catchments within the study area are more likely controlled by rainfall compared to larger catchment that tend to be more snowmelt-dominated in the spring. The rainfall control in small catchments reflects the greater likelihood that a rainfall event, like a convective storm, covers the entire catchment area. In the case for larger catchments, it is more likely for snowmelt to occur across the entire area in the spring.

While hydrometric stations with catchment areas starting from approximately 6 km$^2$ up to 5,000 km$^2$ are included in the analysis, it is not likely that the equations apply to catchments if they are either too small or too large. The regional models are only reliable if applied within the range of catchment areas used to build the models in the first place. Extrapolation beyond the limit of the model may yield poor or unreliable results.

The regional models are as reliable as the data that is used to support them. There is inherent measurement error in flood events, especially for larger flood events. Furthermore, the data record may simply be incorrect due to a transcription error. In addition, the measuring device may have been moved to a new location or trends over time may come about from changes in the monitoring device. It is not possible to inspect every record at every hydrometric station to control for these sources of error because so much data are pooled across such a large area.

The same applies to the catchment polygon delineation. Much of the catchment delineation was automated using tools that were developed to speed up this process (RNT and ESRI tools). Manual spot checks were completed in conjunction with quality control of the area by means of comparison with published values. Nevertheless, it was not possible to inspect every catchment polygon to control for delineation errors due to the high number of polygons that were generated for this study. It is expected that these sources of error are negligible next to the quantity of data that is processed across the study area.

Trends in the flood record imposed by climate change, land use change, wildfires, insect infestations, or urban development generally precludes the use of frequency analysis. Trend analyses were completed on the flood record to account for some level of trend. However, the flood record often captureCreate s a small window of the flood history at a given location. The limited record makes it difficult to identify a real trend from an artifact of the data record. Therefore, no hydrometric stations were discarded from the analysis due to the presence of a trend in the flood record.

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# APPENDIX D
# CLIMATE CHANGE IMPACTS ON HYDROLOGY

## D.1. INTRODUCTION

The hydroclimate of British Columbia (BC) is complex because of proximity to the Pacific Ocean, mountainous terrain, and extent in latitude. The hydrologic regime is either freshet-dominated (nival regime) or snow-influenced (hybrid nival-pluvial or nival-glacial regimes) throughout most of BC (Eaton & Moore, 2010). Hydrologic trends over recent decades generally include a warming and decreasing snowpack (Kang, Shi, Gao, & Déry, 2014) and earlier onset of spring melt (Déry et al., 2009). The hydrologic response to climate change in BC is expected to be influenced by the regional variability in projected temperature and precipitation changes and by regional variations in physical geography. For example, snow dynamics are strongly influenced by elevation-based temperature gradients resulting in large spatial variations in regions of diverse topography (Schnorbus, Werner, & Bennett, 2014). Also, warmer hybrid nival-pluvial regimes may be more sensitive to changes in regional temperature, precipitation, and rainfall trends (Whitfield, Cannon, & Reynolds, 2002).

Climate change impacts were assessed by BGC for the clearwater watersheds using statistically- and process-based methods. This appendix presents a description of these methodologies and their results. This appendix begins with a description of the anticipated climate change impacts on the hydroclimate within the RDCK (Section D.2). The climate change sensitivity of clearwater watersheds within the region is examined in Section D.3. Finally, an evaluation of the climate change impacts using statistically- and process-based methods for the clearwater watersheds is presented in Section D.4. This appendix ends with a summary of the method that was used to account for the climate change impacts on the hydrology of clearwater watersheds in the RDCK region.

## D.2. CLIMATE CHANGE IMPACTS

### D.2.1. Hydroclimate

Historical changes to climate have been documented in BC (Barnett et al., 2008). While there is a natural variability component to the changes in climate, such as El Niño Southern Oscillation (ENSO) and the Pacific Decadal Oscillation (PDO), historical trends in western North America have been attributed to climate change in the form of increased regional warming (Barnett et al., 2008).

Climate change is projected to impact the overall mean as well as the extremes for a range of climate variables including temperature, precipitation, snow, and rainfall intensities. Projected change in mean annual precipitation (MAP), temperature (MAT), and precipitation as snow (PAS) from historical conditions (1961 to 1990) for clearwater watersheds across the RDCK region for 2050 (average of years 2041 to 2070) are presented in Table D-1.

The climate-adjusted variables are calculated using projections based on the Representative Carbon Pathway (RCP) 8.5 which are averaged across 15 fifth phase Coupled Model Intercomparison project (CMIP5) models (CanESM2, ACCESS1.0, IPSL-CM5A-MR,MIROC5, MPI-ESM-LR, CCSM4, HadGEM2-ES, CNRM-CM5, CSIRO Mk 3.6, GFDL-CM3, INM-CM4, MRI-CGCM3, MIROC-ESM, CESM1-CAM5, GISS-E2R) that were chosen to represent all major clusters of similar atmosphere-ocean general circulation models (AOGCMs) (Knutti, Massin, & Gettleman, 2013), and that had high validation statistics in their CMIP3 equivalents.

**Table D-1. Projected change (RCP 8.5, 2050) from 1961 to 1990 historical conditions (Wang et al., 2016).**

| Watershed | Change in MAP (mm) | Change in MAT (ºC) | Change in PAS (Snow Water Equivalent, mm) |
|---|---|---|---|
| Crawford Creek | 59 | 3.5 | -206 |
| Keen Creek | 82 | 3.6 | -239 |
| Upper Kaslo Creek | 72 | 3.6 | -231 |
| Kalso Creek at Kootenay Lake | 76 | 3.6 | -233 |
| Lemon Creek | 82 | 3.5 | -252 |
| Burton at Arrow Lake | 73 | 3.5 | -221 |
| Caribou Creek | 75 | 3.5 | -225 |
| Snow Creek | 72 | 3.6 | -217 |
| Little Slocan River | 69 | 3.5 | -215 |
| Slocan River | 74 | 3.5 | -220 |
| Goat River | 40 | 3.5 | -151 |
| Erie Creek Upstream End | 69 | 3.6 | -247 |

Projected changes in average climate variables across the RDCK by 2050 show that there is likely to be:

- A net increase in MAP ranging from 40 mm to 82 mm
- A net increase in MAT ranging from 3.5 ºC to 3.6 ºC
- A net decrease in PAS ranging from 151 mm to 252 mm.

In addition, short-term precipitation extremes (sub-daily) are expected to increase in most of North America with a warming atmosphere. The frequency of extremes increases 5-fold in large parts of Canada in December, January, and February (Figure D-1a). The frequency of extremes decreases to approximately a 2-fold increase in southeast BC in June, July, and August (Figure D-1b). This shift in frequency covers the period January 2001 to September 2013. The increase is due to a shift towards moister and warmer climatic conditions (Prein et al., 2017). Extremes in short-term precipitation contributes to the frequency and magnitude of flood events, especially for small watersheds where soil storage is either low or full (i.e., < 250 km$^2$).

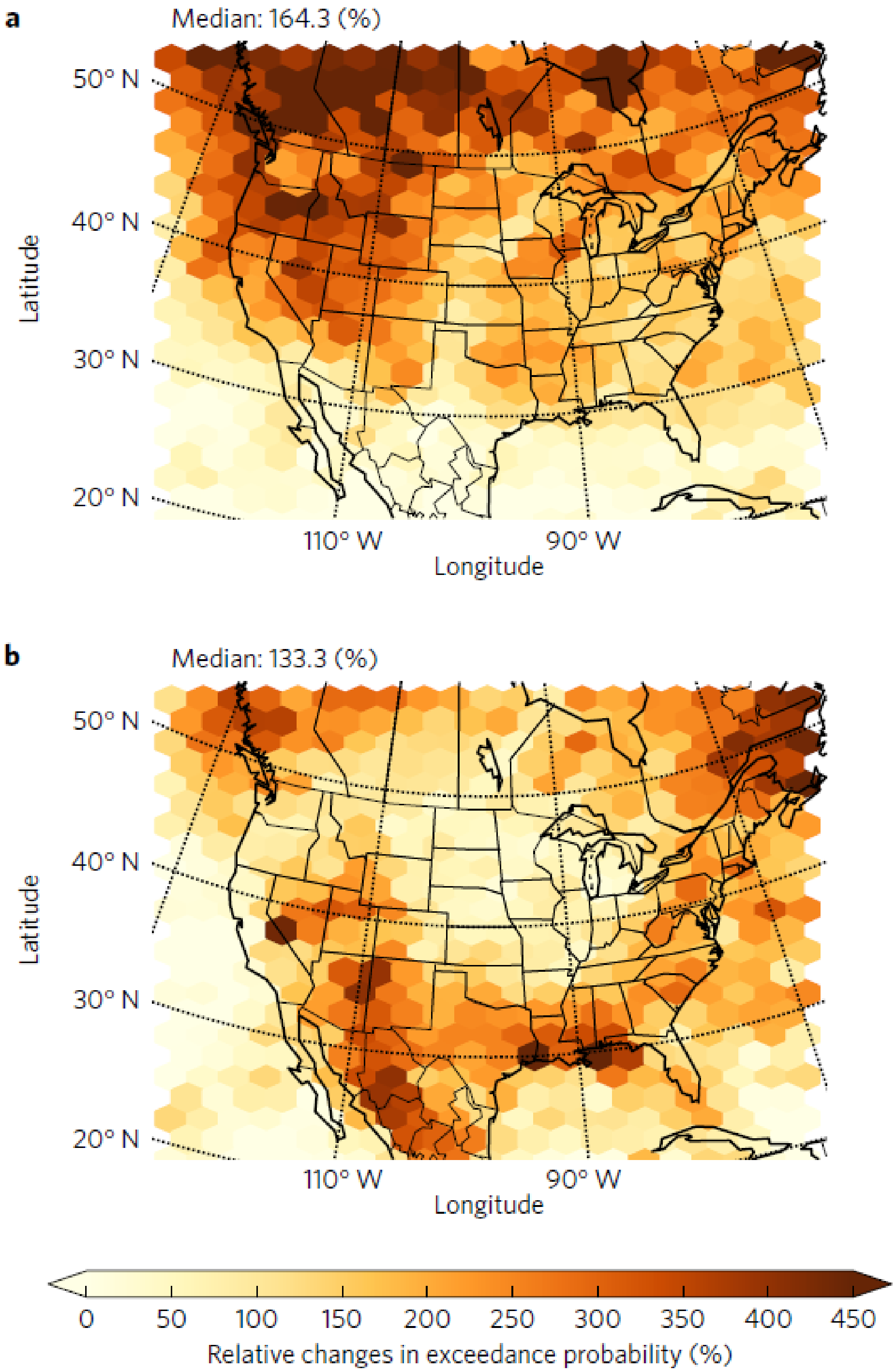

**Figure D-1.** **Change in the exceedance probability of hourly precipitation intensities for (a) December, January, and February, and (b) June, July, and August (Prein et al., 2017).**

### D.2.2. Peak Discharges

The RDCK is situated within the Montane Cordillera ecozone which covers most of southern BC. Extreme flood events in this area are often associated with rain-on-snow events in the spring (Harder et al., 2015). A hydrograph example where the regime is freshet-dominated is shown in Figure D-2. Although the effects of climate change on precipitation are not clear, projected increases in temperature are expected to have the largest impact on annual minimum temperatures occurring in the winter months (Harder et al., 2015).

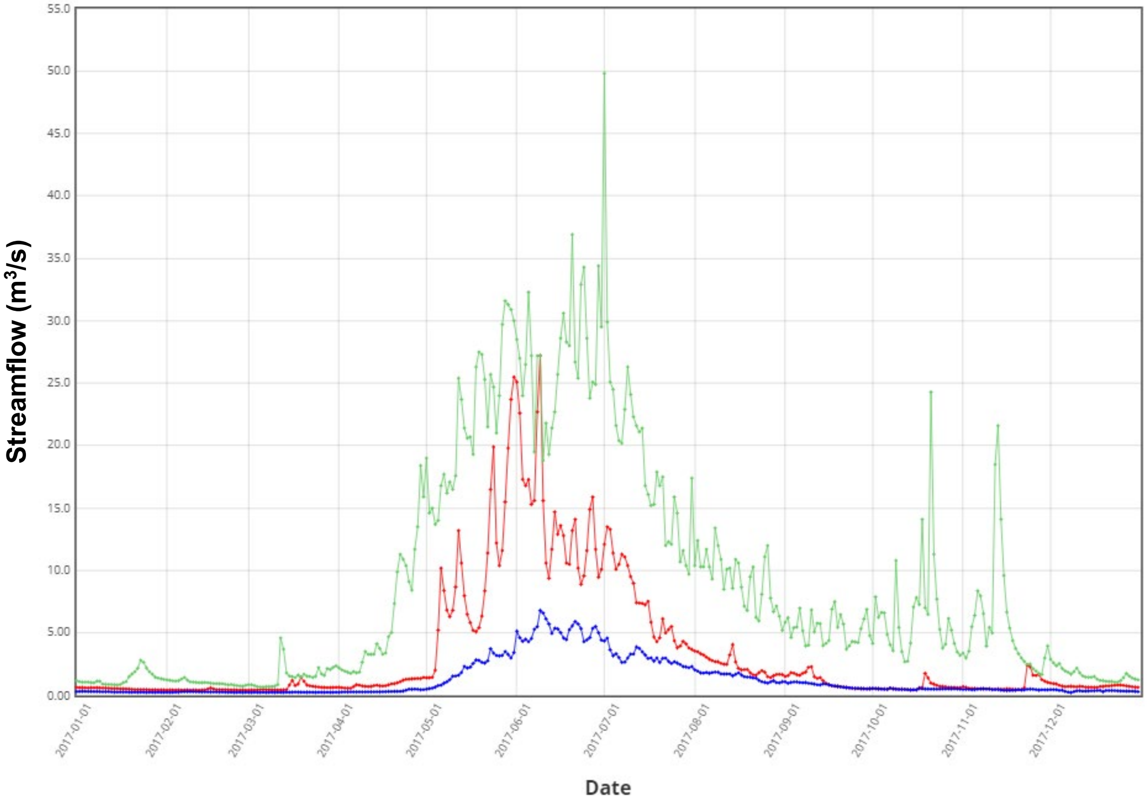

Figure D-2. Example freshet-driven hydrologic regime for Keen Creek below Kyawats Creek (08NH132). Green line is the maximum streamflow, the blue line is the minimum streamflow, and the red line is the 2017 streamflow.

The effects of temperature change differ throughout the region. High elevation regions throughout parts of the Montane Cordillera (e.g., Upper Columbia watershed) are projected to experience increases in snowpack, limiting the response in high elevation watersheds while lower elevations are projected to experience a decrease in snow water equivalent (Loukas & Quick, 1999; Schnorbus et al., 2014).

Projected changes in streamflow vary spatially and seasonally based on snow and precipitation changes and topography-based temperature gradients. Researchers anticipate that streamflow will increase in the winter and spring in the RDCK due to earlier snowmelt and more frequent rain-on-snow events, while earlier peak flow timing is expected in many rivers (Schnorbus et al., 2014; Farjad, Gupta, & Marceau, 2016).

## D.3. WATERSHED SENSITIVITY

The RDCK includes 6 detailed clearwater study areas (Crawford Creek, Kaslo Creek, Slocan River, Burton Creek, Goat River, and Salmo River). Each study area includes one or more clearwater watersheds that were assessed to inform the floodplain delineation. All clearwater watersheds in the RDCK are characterized by a freshet-dominated regime. Freshet-dominant regimes are characterized by a maximum annual streamflow in the spring

In a warmer climate, hydrologic regime shifts are likely to intensify although regional responses are expected due to each watershed's unique characteristics like elevation range and proximity to the 0$^{o}$C air temperature threshold during the cold season. The largest changes in the timing of peak floods would be expected for those areas with a hydrologic regime that shifts from a freshet-dominated to rainfall dominated regime. Therefore, those watersheds with the thinnest snowpacks would be the most sensitive.

The RDCK can be sub-divided into five regions, each with a relatively different, typical snowpack depth (Figure D-3). Two of those five regions cover the clearwater watersheds. The typical snow depths for the clearwater watersheds ranges from moderate snowpack at high elevations for Goat River and Crawford Creek to moderate to deep snowpack for the remaining sites (Table D-2). The elevation range for each clearwater watershed is included in Table D-2 for reference. The clearwater watershed with largest projected change in precipitation as snow by 2050 is Lemon Creek (decrease of 252 mm) followed by Erie Creek Upstream End (decrease of 247 mm) and Keen Creek (decrease of 239 mm) as listed in Table D-1. Hydrographs based on representative hydrometric stations for each study area are presented at the end of the appendix for reference (Figure D-8 to Figure D-11).

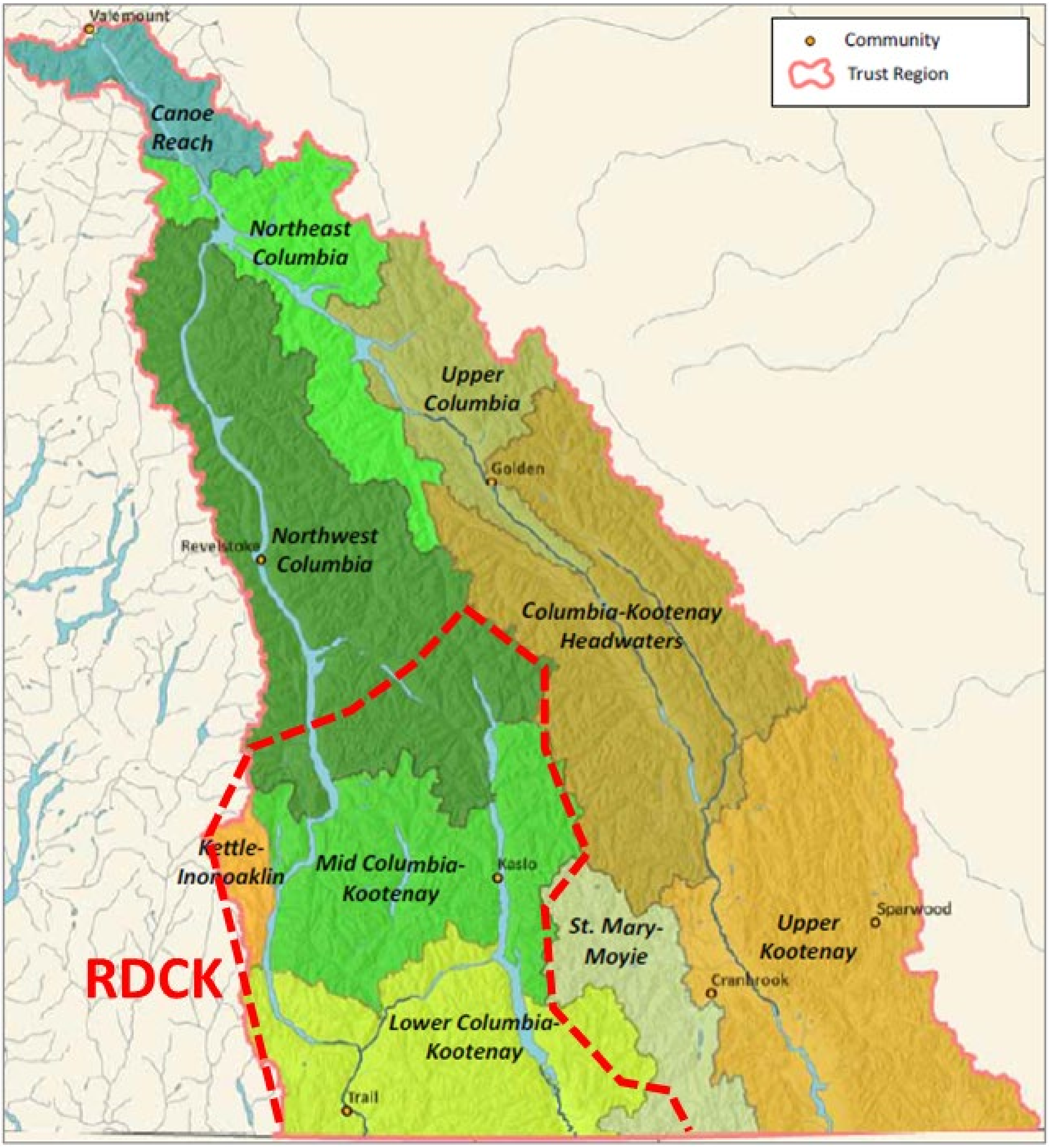

Figure D-3. Regions of the Columbia Basin as defined by patterns of climate and surface runoff. The RDCK contains 5 of these regions, 2 of which cover the clearwater watersheds (CBT, 2017)

Table D-2. Regions of the Columbia Basin covering the RDCK and their current relative snowpack depth (CBT, 2017).

| Region | Existing Relative Snowpack Depth | Study Area | Representative Hydrometric Station | Clearwater Watersheds | Elevation Range (m) |
|---|---|---|---|---|---|
| Lower Columbia-Kootenay | Moderate snowpack at higher elevations | Goat River | 08NH004 | Goat River | 532 to 2622 |
| | | Salmo River | 08NE074 | Erie Creek Upstream End | 712 to 2287 |
| Mid Columbia-Kootenay | Moderate to deep snowpack | Crawford Creek | ungauged watershed | Crawford Creek | 530 to 2627 |
| | | Kaslo Creek | 08NH005 | Keen Creek | 704 to 2797 |
| | | | | Upper Kaslo Creek | 699 to 2670 |
| | | | | Kalso Creek at Kootenay Lake | 549 to 2785 |
| | | Burton Creek | Ungauged watershed | Snow Creek | 465 to 2731 |
| | | | | Burton at Arrow Lake | 439 to 2785 |
| | | | | Caribou Creek | 1117 to 2630 |
| | | Slocan River | 08NJ013 | Lemon Creek | 538 to 2604 |
| | | | | Little Slocan River | 498 to 2803 |
| | | | | Slocan River | 450 to 2973 |

## D.4. CLIMATE CHANGE IMPACT ASSESSMENT

Assessments of climate change impacts for all clearwater watersheds were performed to quantify the anticipated changes in the annual maximum streamflow by 2050 under the RCP 8.5 emission scenario. Four different approaches were used which can be classified into statistically-based and process-based assessments.

### D.4.1. Statistically-based Assessment

Two statistically-based methods were developed to assess the effect of climate change on flood quantiles. The first method was based on an examination of the historical annual maximum flood series data to identify statistically significant trends (positive or negative). The second method was based on the index-flood model developed as part of the Regional Flood Frequency Analysis (Regional FFA) (see Appendix C) to estimate the climate-adjusted index flood using climate-adjusted variables derived from downscaled global circulation model (GCM) predictions (Wang et al., 2016). The two methods are described in more detail and results are presented in the following sections.

### D.4.1.1. Streamflow Trend Analysis

Statistical streamflow trend analysis on the annual maximum series (AMS)[1] was performed on suitable gauges (e.g., sufficient period of record, not regulated) located within the watersheds of clearwater study areas and within the hydrological regions formed as part of the Regional FFA.

The presence of a trend (positive or negative) in the AMS was inferred to be caused, at least in part, by climate change. The Mann-Kendall (M-K) statistical test was used to conduct the trend analyses. The M-K test was preferred over alternative statistical tests because it is non-parametric, and therefore does not assume a functional relationship between time and streamflow magnitude. The M-K test detects consistently increasing or decreasing trends in time series. The M-K test examines for an absence of trend in the time series (the null hypothesis) and returns the probability that the null hypothesis (that there is no monotonic trend in the series) is true. Failing the null hypothesis would in turn suggest that there is a statistically significant temporal trend in the time series. The M-K test was applied only to hydrometric stations with periods of records which spanned the year 2000 to ensure the time series included the most current climate.

Although it was assumed that statistically significant trends were at least in part caused by climate change, changes to the watershed's land cover (e.g., wildfire, insect infestations, changes in land use) were considered as possible causes to trends in peak discharges. Furthermore, the peak flow records often capture a small window of the flood history at a given location. The limited record lengths make it difficult to differentiate between a long-term trend cause by climate change and the intrinsic climate variability captured in the time series. Consequently, the presence of a statistically significant trend in the peak flow time series could not be solely attributed to climate change.

#### D.4.1.1.1 Assessment of Streamflow Gauges within Study Areas

One or more suitable streamflow gauges were identified on the Slocan, Kaslo and Salmo Rivers for trend analysis. A streamflow gauge with historical streamflow data is available on the Goat River (*Goat River Near Erickson* (08NH004)); however, this gauge cannot be used for assessment of trends as the Goat River is regulated. Of the six streamflow gauges assessed for the three rivers, none were found to show strong or even weak evidence of a trend in the AMS.

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[1] The Annual Maximum Series (AMS) is a time series of the largest peak discharge for each year.

**Table D-3. Trend results for streamflow gauges within the clearwater study areas (where suitable hydrometric station exist).**

| Hydrometric Station | Name | Start Year | End Year | p-value | Trend Direction | Sen's Slope[1] |
|---|---|---|---|---|---|---|
| **Slocan River** | | | | | | |
| 08NJ013 | Slocan River Near Crescent Valley | 1914 | 2018 | 0.18 | - | 0.48 |
| 08NJ160 | Lemon Creek Above South Lemon Creek | 1973 | 2017 | 0.23 | - | 0.17 |
| **Kaslo River** | | | | | | |
| 08NH005 | Kaslo River Below Kemp Creek | 1972 | 2017 | 0.32 | - | -0.21 |
| 08NH132 | Keen Creek Below Kyawats Creek | 1974 | 2016 | 0.79 | - | 0.04 |
| **Salmo River** | | | | | | |
| 08NE074 | Salmo River Near Salmo | 1949 | 2018 | 0.47 | - | -0.29 |
| 08NE114 | Hidden Creek Near the Mouth | 1973 | 2016 | 0.73 | - | 0.02 |

### D.4.1.1.2 Assessment of Streamflow Trends within Homogenous Regions

Each clearwater watershed was assigned to a homogeneous region as part of the Regional FFA formed using cluster analysis. (see Section 4.5 in Appendix C). A trend analysis was performed on the annual peak streamflow time series recorded at the hydrometric stations located within the homogeneous region assigned to the clearwater watersheds

#### *D.4.1.1.2.1 1 West – for Watersheds < 500 km$^2$*

Within the "1 West – for watersheds less than 500 km$^2$" hydrological region, one hydrometric station out of 15 reported a statistically significant trend ($p < 0.05$ - less than a 5% chance of rejecting the null hypothesis) in the flood series: *Kuskanax near Nakusp* (08NE006). The trend in the magnitude of the flood series for that station was in the decreasing direction (Table D-4).

**Table D-4. Trend results for the hydrometric stations in the 1 West – for watersheds < 500 km$^2$ hydrologic region.**

| Hydrometric Station Code | Start Year | End Year | p-value | Trend Direction | Sen's Slope[1] |
|---|---|---|---|---|---|
| 08LB038 | 1985 | 2016 | 0.246 | - | 0.33 |
| 08NP004 | 1995 | 2017 | 0.239 | - | 0.13 |
| 08NH131 | 1973 | 2004 | 0.444 | - | 0.19 |
| 08KA001 | 1969 | 2013 | 0.738 | - | 0.06 |
| 08NJ168 | 1983 | 2014 | 0.475 | - | 0.04 |
| 08NB014 | 1973 | 2017 | 0.431 | - | -0.25 |
| 08NH132 | 1974 | 2016 | 0.795 | - | 0.04 |
| 08ND019 | 1973 | 2005 | 0.650 | - | 0.13 |
| 08NE006 | 1968 | 2011 | 0.006 | Decreasing* | -1.33 |
| 08NK022 | 1977 | 2015 | 0.143 | - | -0.19 |
| 08NG076 | 1973 | 2017 | 0.314 | - | 0.07 |
| 08KA009 | 1967 | 2018 | 0.881 | - | -0.04 |
| 08KB006 | 1978 | 2015 | 0.386 | - | 0.20 |
| 08LE086 | 1997 | 2016 | 1.000 | - | 0.00 |
| 08KA010 | 1908 | 2015 | 0.118 | - | -0.25 |

Notes:

1. The Sen's slope is a robust estimate of the magnitude of a trend and commonly used to identify the slope of a trend line in hydrological time series (Yue et al. 2002). It is considered robust because it is sensitive to outliers.

* Strong evidence of trend ($p < 5\%$) – less than 5% chance that the null hypothesis – that there is no trend – is true.

** Weak evidence of trend ($p < 10\%$)– less than 10% chance that the null hypothesis – that there is no trend – is true.

### *D.4.1.1.2.2 1 West – for Watersheds > 500 km$^2$*

Within the "1 West – for watersheds greater than 500 km$^2$" hydrological region, one out of 15 hydrometric stations reporting a statistically significant trend in the flood series (*Fraser River at Red Pass*, 08KA007) with a trend in the decreasing direction (Table D-5).

**Table D-5. Trend results for the hydrometric stations in the 1 West – for watersheds > 500 km$^2$ hydrologic region.**

| Hydrometric Station Code | Start Year | End Year | p-value | Trend Direction | Sen's Slope[1] |
|---|---|---|---|---|---|
| 08NB019 | 1985 | 2018 | 0.836 | - | 0.20 |
| 08NB012 | 1970 | 2017 | 0.818 | - | 0.11 |
| 08LE024 | 1973 | 2017 | 0.143 | - | -1.07 |
| 08NP001 | 1929 | 2017 | 0.845 | - | -0.06 |
| 08NK018 | 1973 | 2015 | 0.530 | - | -0.23 |
| 08KA007 | 1955 | 2016 | 0.016 | Decreasing* | -0.81 |
| 08NH130 | 1973 | 2012 | 0.990 | - | 0.00 |
| 08ND012 | 1964 | 2018 | 0.670 | - | -0.11 |
| 08ND013 | 1964 | 2017 | 0.228 | - | 0.72 |
| 08NA006 | 1912 | 2017 | 0.317 | - | -0.61 |
| 12358500 | 1940 | 2017 | 0.623 | - | -0.45 |
| 08KA013 | 1998 | 2017 | 0.576 | - | 3.25 |
| 12355500 | 1911 | 2017 | 0.857 | - | -0.11 |
| 08LE027 | 1915 | 2017 | 0.598 | - | 0.15 |
| 08NA011 | 1949 | 2018 | 0.319 | - | -0.36 |

Notes:

1. The Sen’s slope is a robust estimate of the magnitude of a trend and commonly used to identify the slope of a trend line in hydrological time series (Yue et al. 2002). It is considered robust because it is sensitive to outliers.

* Strong evidence of trend ($p < 5\%$) – less than 5% chance that the null hypothesis – that there is no trend – is true.

** Weak evidence of trend ($p < 10\%$)– less than 10% chance that the null hypothesis – that there is no trend – is true.

### *D.4.1.1.2.3 4 East – for Watersheds < 500 km$^2$*

Within the “4 East – for watersheds less than 500 km$^2$” hydrological region, 19 hydrometric stations were analysed for presence of a trend (Table D-6). The M-K test identified two stations as having statistically significant trends in their time series with the first showing an increasing trend (*Boundary Creek near Porthill Idaho,* 12321500) and the second showing a decreasing trend (*Arrow Creek near Erickson*, 08NH084). Two other stations, *Redfish Creek near Harrop* (08NJ061) and *Outlet Creek near Metaline Falls* (12397100), were found to have marginally statistically significant decreasing trends ($p < 0.1$ - less than a 10% chance of rejecting the null hypothesis), while *St-Mary River below Morris Creek* (08NG077) was found to have a marginally statistically significant increasing trend ($p < 0.1$).

**Table D-6. Trend results for the hydrometric stations in the 4 East – for Watersheds > 500 km² hydrologic region.**

| Hydrometric Station Code | Start Year | End Year | p-value | Trend Direction | Sen's Slope[1] |
|---|---|---|---|---|---|
| 08NK026 | 1986 | 2018 | 0.332 | - | -0.01 |
| 08NJ130 | 1945 | 2017 | 0.177 | - | 0.01 |
| 12321500 | 1929 | 2017 | 0.002 | Increasing** | 0.23 |
| 08NH084 | 1980 | 2015 | 0.009 | Decreasing** | -0.30 |
| 08NH005 | 1972 | 2017 | 0.322 | - | -0.21 |
| 08NE110 | 1971 | 2015 | 0.567 | - | 0.14 |
| 08NJ061 | 1968 | 2017 | 0.052 | Decreasing** | -0.06 |
| 08NG077 | 1973 | 2017 | 0.083 | Increasing* | 0.50 |
| 08NN023 | 1974 | 2015 | 0.555 | - | -0.12 |
| 08NE087 | 2001 | 2017 | 0.964 | - | -0.01 |
| 08NH016 | 1947 | 2017 | 0.504 | - | -0.02 |
| 08NJ160 | 1973 | 2017 | 0.229 | - | 0.17 |
| 12313000 | 1928 | 2002 | 0.386 | - | 1.58 |
| 08NJ026 | 1995 | 2017 | 0.239 | - | 0.13 |
| 12397100 | 1959 | 2015 | 0.065 | Decreasing* | -0.07 |
| 08NE114 | 1973 | 2016 | 0.727 | - | 0.02 |
| 08NE039 | 1930 | 2017 | 0.507 | - | -0.06 |
| 12304040 | 1990 | 2000 | 0.533 | - | 0.43 |
| 08NH115 | 1964 | 2017 | 0.303 | - | 0.00 |

Notes:

1 The Sen’s slope is a robust estimate of the magnitude of a trend and commonly used to identify the slope of a trend line in hydrological time series (Yue et al. 2002). It is considered robust because it is sensitive to outliers.

* Strong evidence of trend (p < 5%) – less than 5% chance that the null hypothesis – that there is no trend – is true.

** Weak evidence of trend (p < 10%)– less than 10% chance that the null hypothesis – that there is no trend – is true.

### *D.4.1.1.2.4 7 – for Watersheds > 500 km²*

Within the “7 – for watersheds greater than 500 km²” hydrological region, 17 hydrometric stations were analysed for presence of a trend (Table D-7). The M-K test identified three USGS stations as having statistically significant decreasing trends in their time series: *Thompson River near Thompson Falls MT* (12389500), *Yaak River near Troy MT* (12304500), and *Yakima River at Umtanum, WA* (12484500). One other station, *Colville River at Kettle Falls, WA* (12409000), was found to have a marginally statistically significant increasing trend ($p < 0.1$).

**Table D-7. Trend results for the hydrometric stations in the 7 – for Watersheds > 500 km$^2$ hydrologic region.**

| Hydrometric Station Code | Start Year | End Year | p-value | Trend Direction | Sen's Slope[1] |
|---|---|---|---|---|---|
| 13339500 | 1980 | 2017 | 0.237 | - | 0.61 |
| 12414900 | 1966 | 2017 | 0.185 | - | 0.67 |
| 12433890 | 1972 | 2012 | 0.553 | - | 0.43 |
| 12354000 | 1911 | 2017 | 0.129 | - | -0.98 |
| 12388200 | 1990 | 2010 | 0.124 | - | 0.77 |
| 12301300 | 1948 | 2016 | 0.189 | - | -0.15 |
| 12365000 | 1931 | 2006 | 0.528 | - | -0.08 |
| 12306500 | 1930 | 2017 | 0.983 | - | 0.00 |
| 12389500 | 1948 | 2017 | 0.044 | Decreasing* | -0.55 |
| 12370000 | 1922 | 2017 | 0.290 | - | -0.15 |
| 12304500 | 1948 | 2017 | 0.006 | Decreasing* | -1.37 |
| 12302055 | 1948 | 2017 | 0.408 | - | -0.35 |
| 12413000 | 1912 | 2017 | 0.542 | - | 0.75 |
| 12409000 | 1923 | 2017 | 0.076 | Increasing** | 0.13 |
| 12414500 | 1911 | 2017 | 0.935 | - | 0.00 |
| 12413500 | 1911 | 2017 | 0.125 | - | 1.67 |
| 12484500 | 1906 | 2017 | 0.021 | Decreasing* | -0.70 |

Notes:

1 The Sen's slope is a robust estimate of the magnitude of a trend and commonly used to identify the slope of a trend line in hydrological time series (Yue et al. 2002). It is considered robust because it is sensitive to outliers.

* Strong evidence of trend ($p < 5\%$) – less than 5% chance that the null hypothesis – that there is no trend – is true.

** Weak evidence of trend ($p < 10\%$)– less than 10% chance that the null hypothesis – that there is no trend – is true.

### D.4.1.2. Statistical Flood Frequency Modelling

A statistical approach to estimating flood quantiles for the clearwater watersheds was performed using the Regional FFA model. The multivariate regression model to estimate the index-flood (mean annual peak flow) included three climatic variables as predictors: MAP, MAT, and PAS. This regression model was calibrated using historical values of climatic variables, thus representing current conditions.

To estimate the climate-adjusted index flood for 2050, projected values of the climatic variables were input to the regression model. These projected values were estimated from model ensemble results for the RCP 8.5 emissions scenario using the ClimateNA v5.10 software package, available at http://tinyurl.com/ClimateNA, and based on the methodology described by Wang et al. (2016). The historical and climate-adjusted MAP, MAT, and PAS for the clearwater watersheds in the RDCK region are presented in Table D-8.

**Table D-8. Climate variables used in the index flood quantile regression model with historical and climate-adjusted values for the clearwater watersheds in the RDCK.**

| Study Area | Watershed | MAP | | MAT | | PAS | |
|---|---|---|---|---|---|---|---|
| | | Historical Value | Climate-adjusted | Historical Value | Climate-adjusted | Historical Value | Climate-adjusted |
| Crawford Creek | Crawford Creek | 1116 | 1175 | 3.0 | 6.4 | 590 | 384 |
| Kaslo Creek | Keen Creek | 1390 | 1472 | 1.3 | 4.9 | 857 | 618 |
| | Upper Kaslo Creek | 1244 | 1316 | 2.7 | 6.3 | 668 | 437 |
| | Kalso Creek at Kootenay Lake | 1312 | 1389 | 2.1 | 5.7 | 756 | 523 |
| Burton Creek | Burton at Arrow Lake | 1242 | 1315 | 2.4 | 5.9 | 704 | 483 |
| | Caribou Creek | 1259 | 1334 | 2.4 | 6.0 | 709 | 484 |
| | Snow Creek | 1227 | 1299 | 2.3 | 5.8 | 700 | 483 |
| Slocan River | Little Slocan River | 1161 | 1230 | 2.8 | 6.3 | 643 | 428 |
| | Lemon Creek | 1322 | 1404 | 2.7 | 6.3 | 754 | 503 |
| | Slocan River | 1224 | 1297 | 3.0 | 6.6 | 666 | 446 |
| Goat River | Goat River | 857 | 897 | 3.2 | 6.7 | 433 | 282 |
| Salmo River | Erie Creek Upstream End | 1265 | 1334 | 3.8 | 7.4 | 617 | 371 |

Note:

1. The ensemble model projections are averages across 15 CMIP5 models (CanESM2, ACCESS1.0, IPSL-CM5A-MR, MIROC5, MPI-ESM-LR, CCSM4, HadGEM2-ES, CNRM-CM5, CSIRO Mk 3.6, GFDL-CM3, INM-CM4, MRI-CGCM3, MIROC-ESM, CESM1-CAM5, GISS-E2R).

Climate-adjusted flood quantiles were calculated using the climate-adjusted index flood and the regional growth curves. The regional growth curves are assumed to be stationary. The ratio between the magnitude of the index-flood and the other flood quantiles was assumed to be the same in a climate-adjusted context. The regional growth curves are presented in the Regional FFA (Appendix C). Historical and climate-adjusted flood quantiles are summarized in Table D-9. Results show a small decrease in magnitude between the historical and climate-adjusted flood quantiles. Examination of the regression model for the index flood revealed that both the MAP and PAS were dominant predictors. The increase in the MAP was found to offset the decrease in the PAS resulting in little change in the estimate of the climate-adjusted index flood.

**Table D-9. Historical and climate-adjusted flood quantiles for clearwater watersheds in the RDCK.**

| Study Area | Clearwater Watershed | Index-flood | | 2-year return period (0.5 AEP) | | 20-year return period (0.05 AEP) | | 200-year return period (0.005 AEP) | |
|---|---|---|---|---|---|---|---|---|---|
| | | Historical ($m^3/s$) | Climate-adjusted ($m^3/s$) | Historical ($m^3/s$) | Climate-adjusted ($m^3/s$) | Historical ($m^3/s$) | Climate-adjusted ($m^3/s$) | Historical ($m^3/s$) | Climate-adjusted ($m^3/s$) |
| Crawford Creek | Crawford Creek | 27 | 27 | 25 | 24 | 50 | 49 | 78 | 76 |
| Kaslo Creek | Keen Creek | 45 | 45 | 42 | 41 | 75 | 74 | 115 | 114 |
| | Upper Kaslo Creek | 38 | 37 | 34 | 34 | 70 | 68 | 109 | 106 |
| | Kalso Creek at Kootenay Lake | 81 | 80 | 74 | 73 | 150 | 148 | 234 | 230 |
| Burton Creek | Burton at Arrow Lake | 81 | 79 | 73 | 71 | 149 | 145 | 232 | 227 |
| | Caribou Creek | 42 | 41 | 38 | 37 | 78 | 76 | 121 | 119 |
| | Snow Creek | 45 | 44 | 41 | 40 | 83 | 81 | 129 | 126 |
| Slocan River | Little Slocan River | 103 | 100 | 94 | 91 | 191 | 186 | 297 | 289 |
| | Lemon Creek | 39 | 38 | 35 | 34 | 72 | 69 | 111 | 108 |
| | Slocan River | 347 | 339 | 315 | 308 | 642 | 627 | 1000 | 977 |
| Goat River | Goat River | 110 | 109 | 100 | 98 | 172 | 170 | 317 | 312 |
| Salmo River | Erie Creek Upstream End | 35 | 34 | 32 | 31 | 65 | 63 | 102 | 97 |

Note:

1. Final flood quantiles for Upper Kaslo Creek, Kaslo Creek at Kootenay Lake, Lemon Creek, Little Slocan River, Slocan River, and Goat River were estimated using a pro-rated calculation because they are gauged by a hydrometric station. The flood quantiles reported in Table G-7 were not used for subsequent analysis.

## D.4.2. Process-based Assessment

To complement the statistical assessment, results from process-based modelling were examined. Process-based models involve the direct application of the downscaled GCM model forecasts into hydrological models. Process-based assessments are better suited for situations where a threshold change in process is likely e.g., a transition from nival (snowmelt dominated) runoff regime to pluvial-hybrid (snow influenced) runoff regime streamflow.

### D.4.2.1. Climate-adjusted Streamflow

PCIC provides simulated daily streamflow time series for over 120 sites located in the Peace, upper Columbia, Fraser, and Campbell River watersheds. The time series are simulated at Water Survey of Canada (WSC) hydrometric stations and BC Hydro project sites. The simulated time series represent naturalized flow conditions (i.e., with effects of upstream regulation removed) for those sites affected by storage regulation. The hydrologic projections were forced with GCM data downscaled to a 1/16-degree resolution using Bias-Correction Spatial Disaggregation (BCSD) (Wood et al., 2004) following Werner (2011). Application of the Variable Infiltration Capacity (VIC) model and the generation of hydrologic projections for the Peace, Fraser, upper Columbia, and Campbell River watersheds are described in Shrestha et al. (2012) and Schnorbus et al. (2011, 2014).

An ensemble of 8 models forecasting daily streamflow time series for locations near the study area was accessed from PCIC's website. This included forecasted time series on the Slocan and Salmo Rivers, specifically:

- Slocan River Near Crescent Valley (08NJ013)
- Salmo River Near Salmo (08NE074).

The RCP 8.5 emissions scenario was not available for this dataset so the IPCC A2 Emission Scenario (business as usual) was selected as the most similar. The 200-year flood quantile was assessed for three periods between 2009-2038, 2039-2068 and 2069-2098 and compared to the 200-year flood quantile based on the historical modelling (1955-2009). Maps showing the trend in the 200-year flood for the PCIC assessed sites and the location of the clearwater watersheds in the study for the three periods are shown in Figures D-4 to D-6 for the three periods assessed.

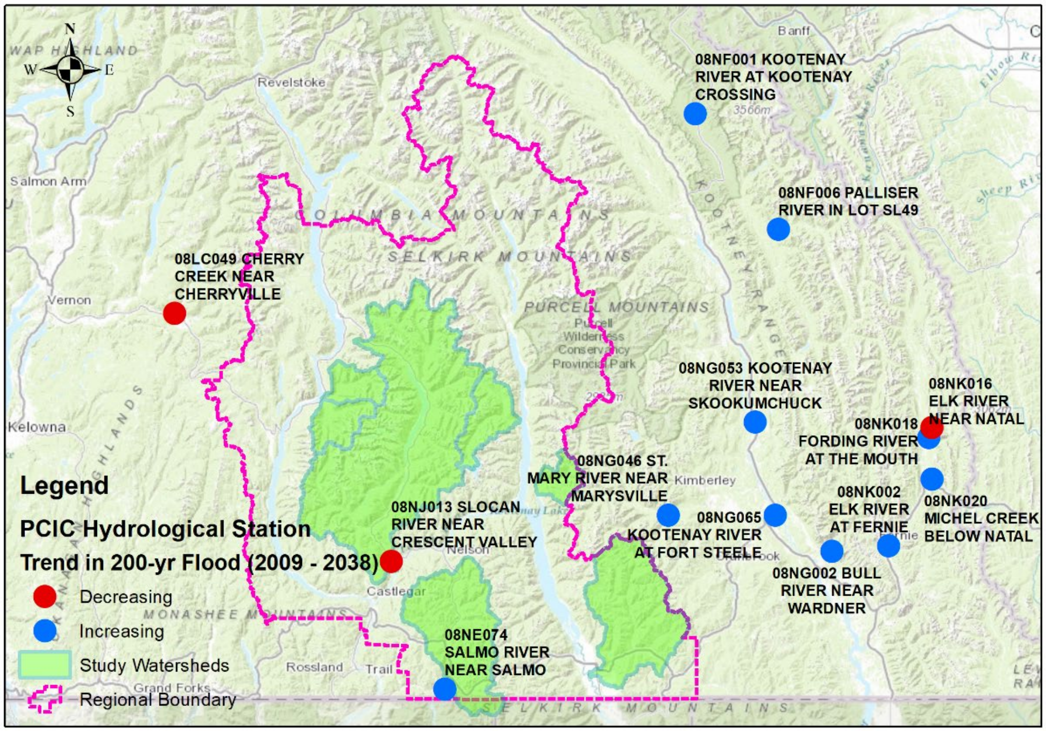

Figure D-4. Map showing nearby the PCIC hydrometric stations examined and their trend in the 200-year flood (period between 2009-2038).

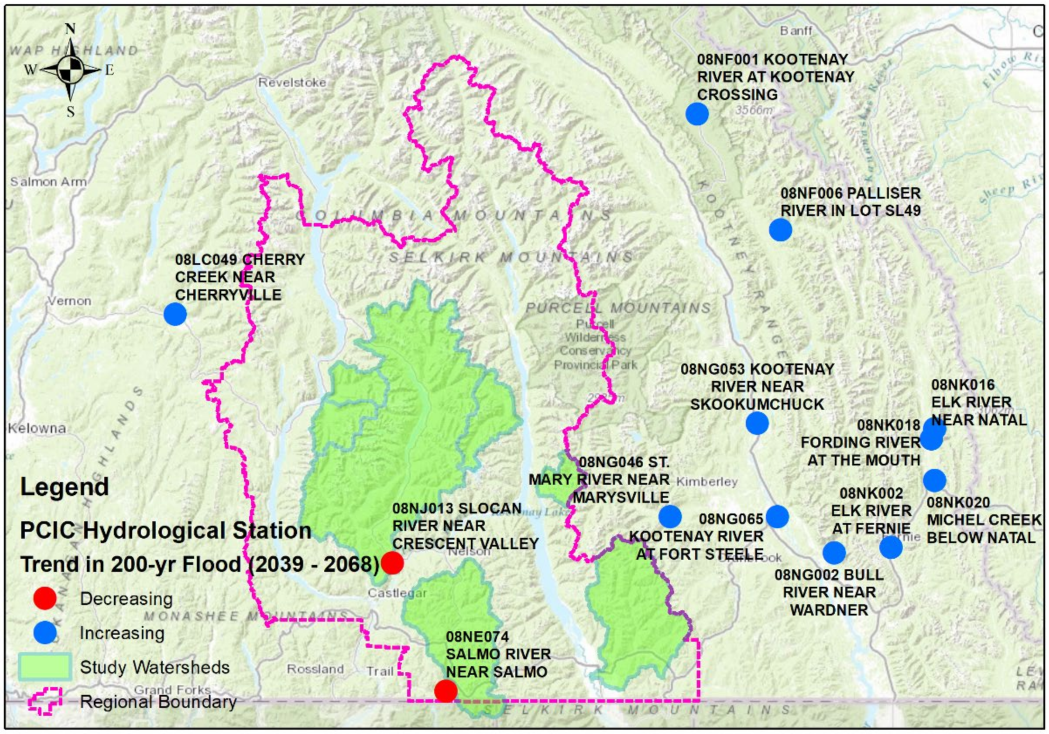

Figure D-5. Map showing nearby the PCIC hydrometric stations examined and their trend in the 200-year flood (period between 2039-2068).

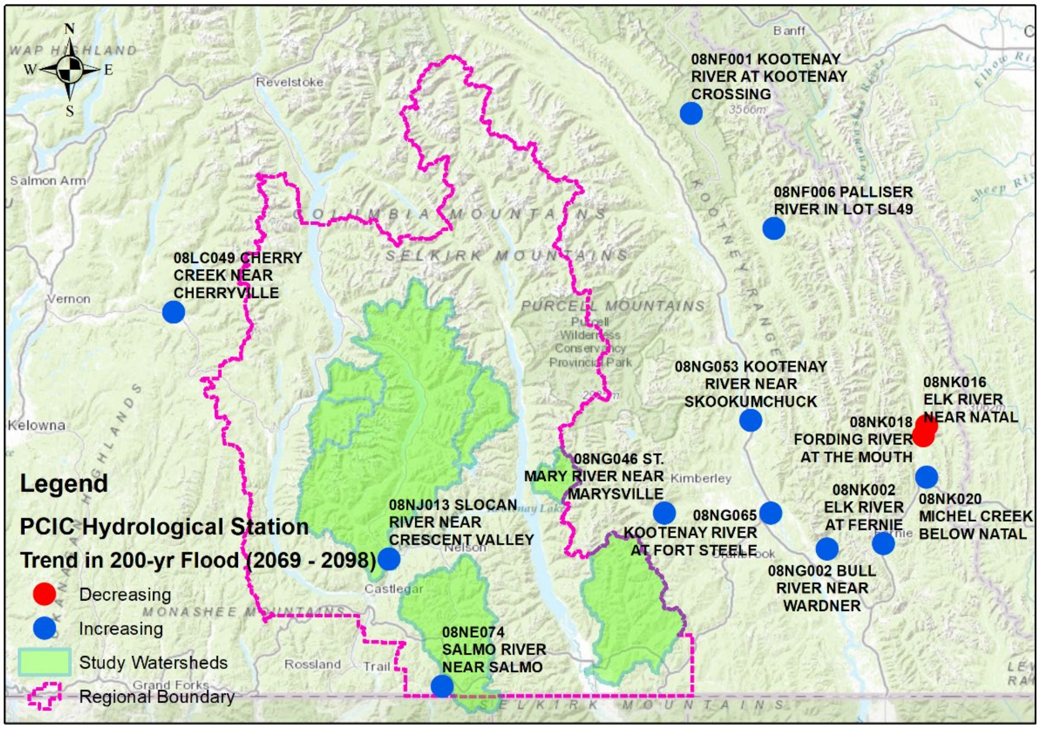

**Figure D-6.** **Map showing nearby the PCIC hydrometric stations examined and their trend in the 200-year flood (period between 2069-2098).**

The maps show that, in general, most of the thirteen stations examined show an increase in the magnitude of the 200-year flood over time with some exceptions based on an assessment of the mean of the eight models. A bar chart of the results for the individual hydrometric stations is shown in Figure D-7. The expected change in 200-year flood for the 2039-2068 period varies between –9% and +28% from the 1955-2009 period. For the 2069-2098 period, the range in the change of the 200-year flood magnitude increases from -7% and +60% from the 1955-2009 period. The mean of the predicted changes in the 200-year flood for Slocan River Near Crescent Valley (08NJ013) show virtually no change for the 2009-2038 period (-0.1%) followed by a small decrease and small increase for the 2039-2068 (-5%) and 2069-2098 (+16%) periods respectively. The mean of the predicted changes in the 200-year flood for Salmo River Near Salmo (08NE074) show a small increase for the 2009-2038 period (+8%) followed by small decrease for the 2039-2068 period (-97%) followed by a large increase for the 2069-2098 period (+60%).

Boxplots of the results for the three periods for the eight model runs are provided in Figure D-12a and Figure D-12b. The boxplots provide a sense of the uncertainty in the analysis by the considerable range in the estimated 200-year flood quantile. Of note, the PCIC hydrologic model output was found by BGC to poorly predict historical flood quantiles.

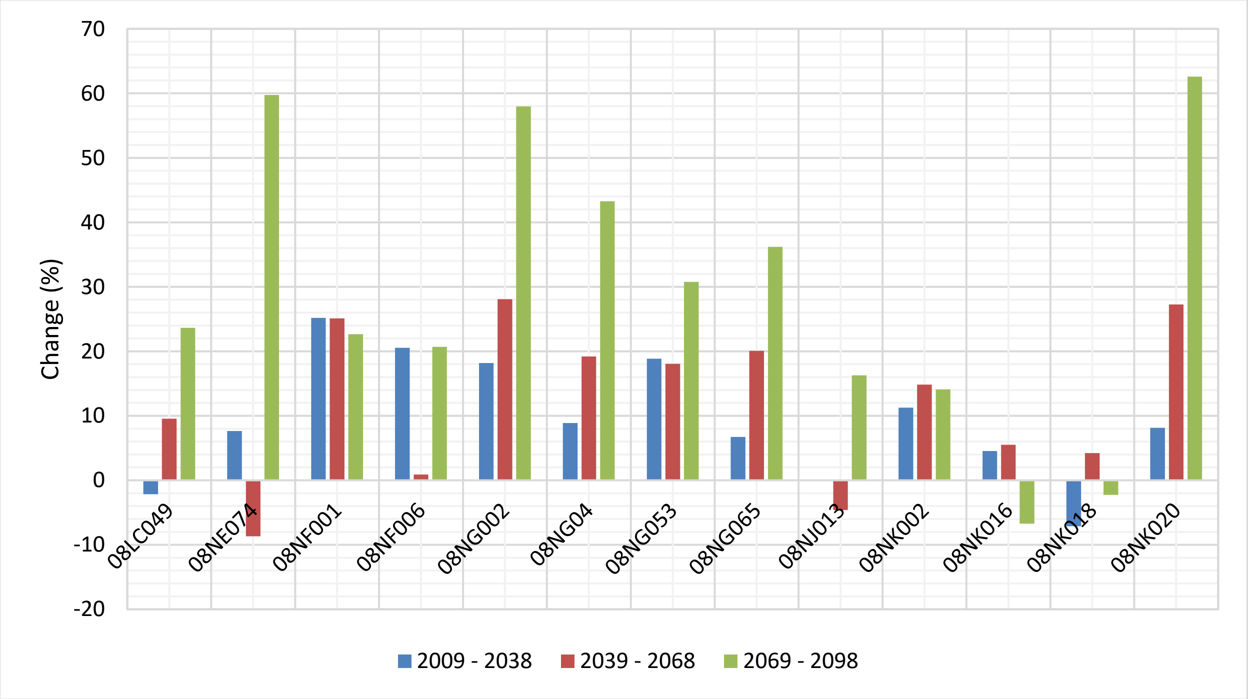

**Figure D-7.** **Bar-graph of the PCIC hydrometric stations and their change in the magnitude of the 200-year flood for the three periods examined compared to the 1955-2009 historical period.** ***Note that Station 08NJ013 and 08NE074 are stations located on the Slocan and Salmo Rivers respectively.***

### D.4.3. Legislated Guidelines

The Engineers and Geoscientists British Columbia (EGBC, 2018) guidelines state that when a historical trend is not detectable, a 10% adjustment can be applied to the design flood to account for likely future change in water input from precipitation. In cases where the information of future local conditions is inadequate to make an informed decision on the impacts of climate on hydrology, the EGBC guidelines suggest "adjusting expected flood magnitude and frequency according to the projected change in runoff during the life of the project, or by 20% in small watersheds (<50 km$^2$) for which information of future local conditions is inadequate to provide reliable guidance." These guidelines also include consideration of potential effects of land use change.

## D.5. SUMMARY

The impacts of climate change on flood quantile estimates were assessed using statistical and processed-based methods. The statistical methods included a trend assessment on historical flood events using the Mann-Kendall test as well as the application of climate-adjusted variables (mean annual precipitation, mean annual temperature, and precipitation as snow) to the Regional FFA model. The process-based methods included a trend analysis for climate-adjusted flood and precipitation data offered by PCIC.

The results of the statistical and process-based methods were found to be inconsistent across the RDCK. The results of the statistical flood frequency modelling generally show a small decrease in the flood magnitude, while the results of the process-based modelling generally show an increase with a wide range in magnitude. Although general trends for the region are predicted by GCMs and downscaled models, there is a wide range of predictions and estimation of future local conditions. The wide range in magnitude can be a function of many variables including watershed characteristics (e.g., proportion of watershed elevation above a given threshold) which were not explicitly addressed in this assessment.

## D.6. CONCLUSION

The guidance offered by the climate changes impact assessment results is considered unreliable for estimating climate-adjusted flood quantiles on a site-specific basis. As a result, flood quantile estimates were adjusted by 20% for all catchments to account for the uncertainty in the impacts of climate change as per the EGBC (2018) guidelines.

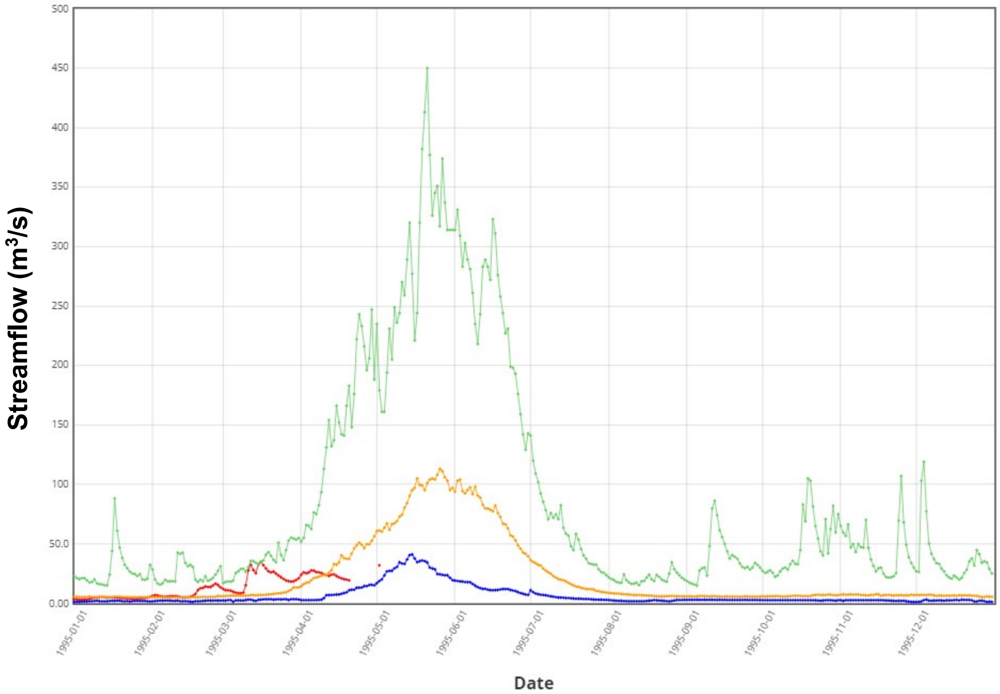

**Figure D-8.** Example freshet-driven hydrologic regime for Goat River near Erickson (08NH004). Green line is the maximum streamflow, the blue line is the minimum streamflow, the orange line is the median, and the red line is the 1995 streamflow.

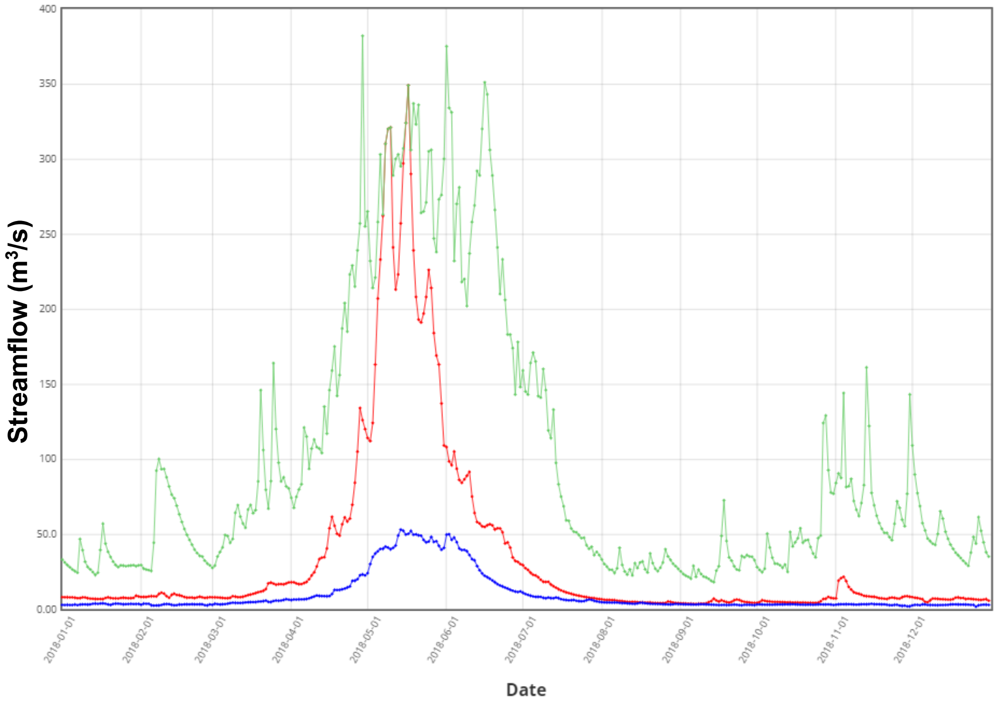

**Figure D-9.** Example freshet-driven hydrologic regime for Salmo River near Salmo (08NE074). Green line is the maximum streamflow, the blue line is the minimum streamflow, and the red line is the 2018 streamflow.

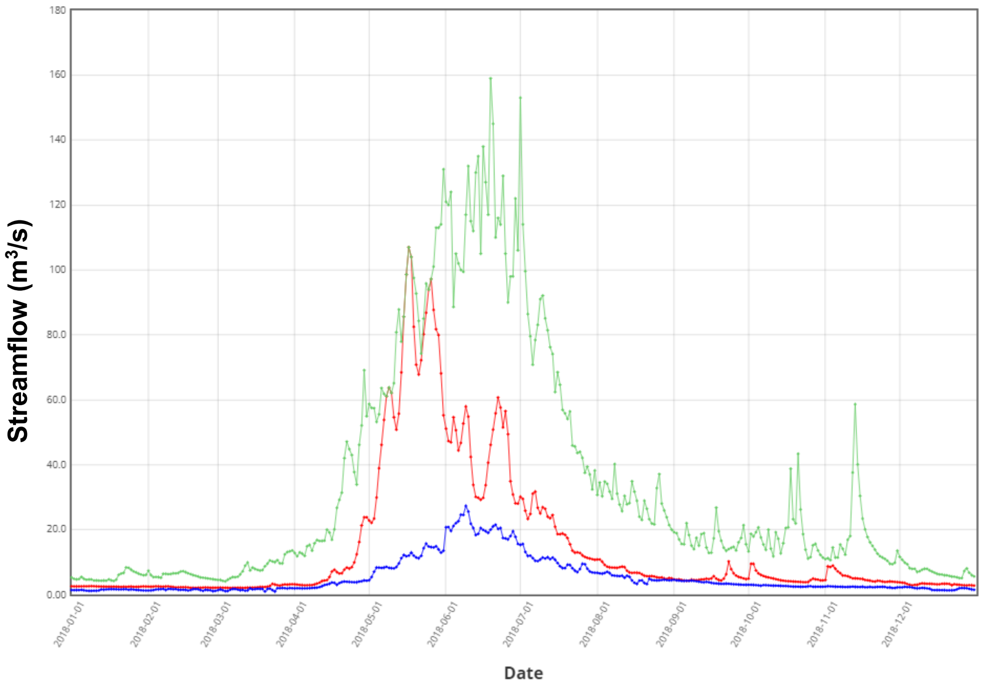

**Figure D-10.** **Example freshet-driven hydrologic regime for Kaslo below Kemp Creek (08NH005). Green line is the maximum streamflow, the blue line is the minimum streamflow, and the red line is the 2018 streamflow.**

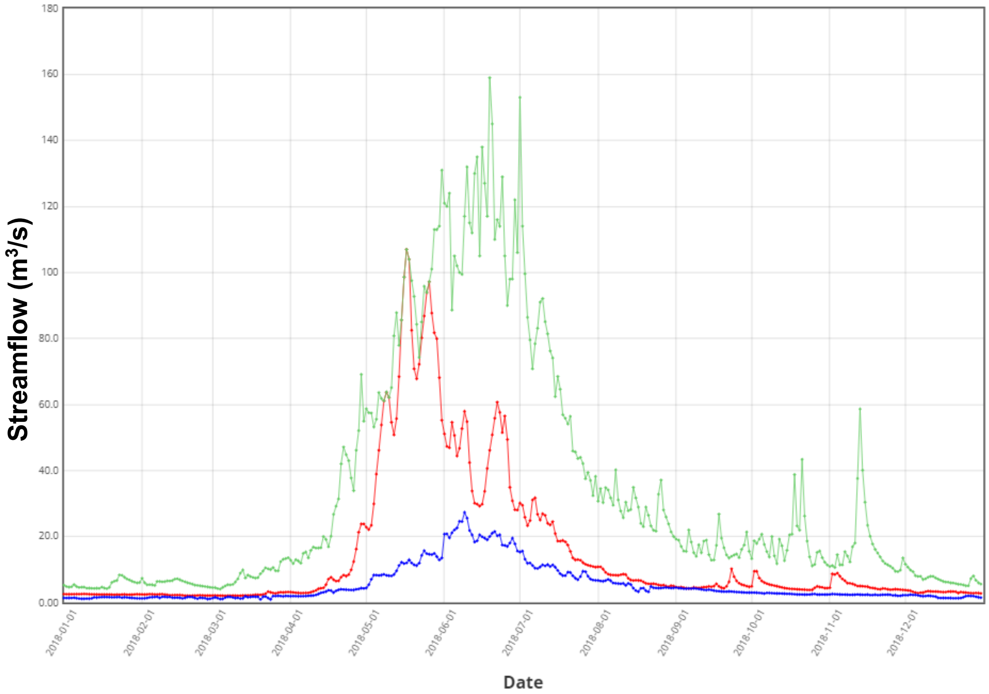

**Figure D-11.** **Example freshet-driven hydrologic regime for Slocan River near Crescent Valley (08NJ013). Green line is the maximum streamflow, the blue line is the minimum streamflow, and the red line is the 2018 streamflow.**

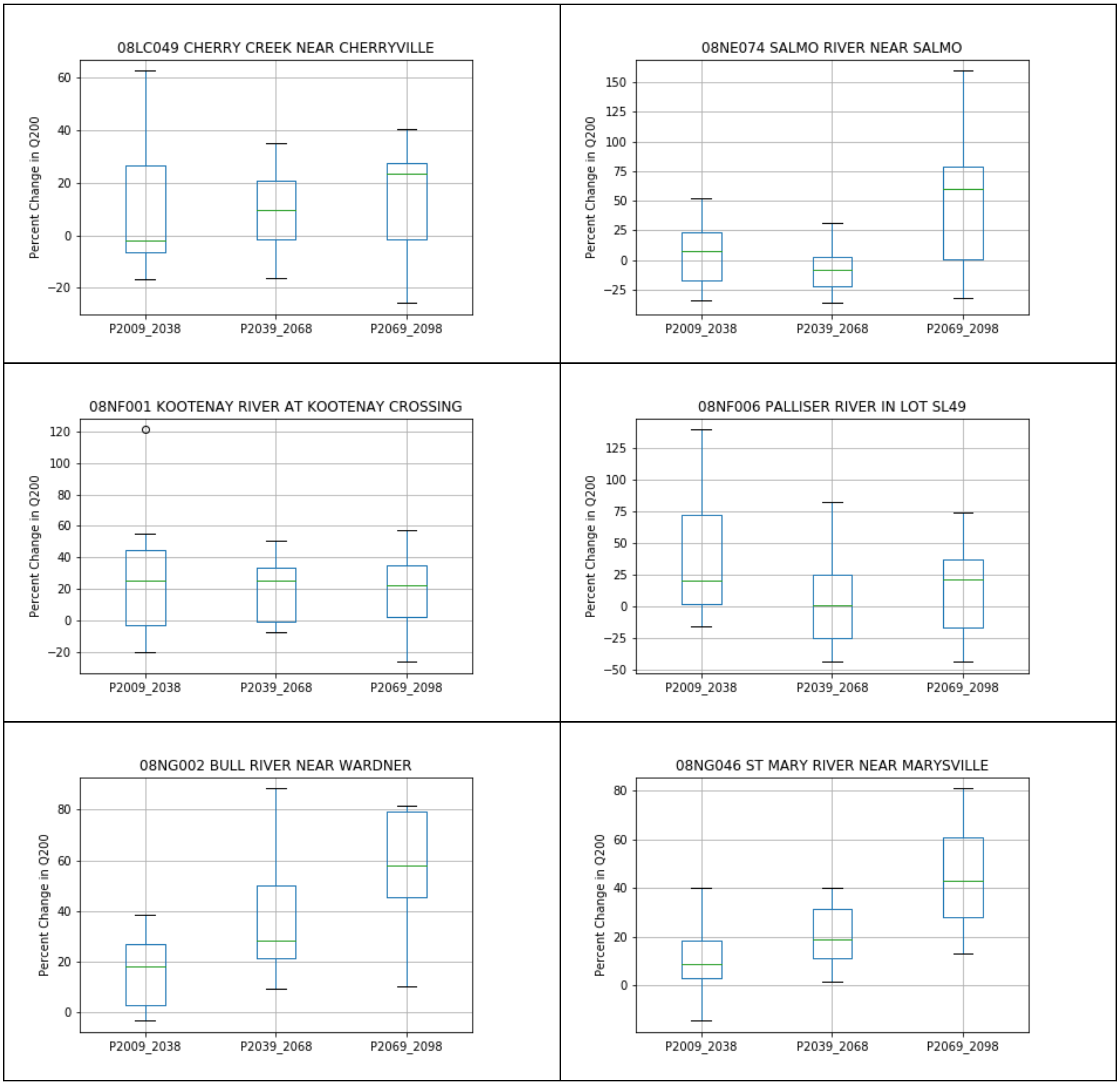

**Figure D-12a. Boxplots of the PCIC Hydrological Stations and their change in the magnitude of the 200-year flood for the three periods examined compared to the 1955-2009 historical period. Boxplots represent the interquartile range from the ensemble of 8 GCM models.**

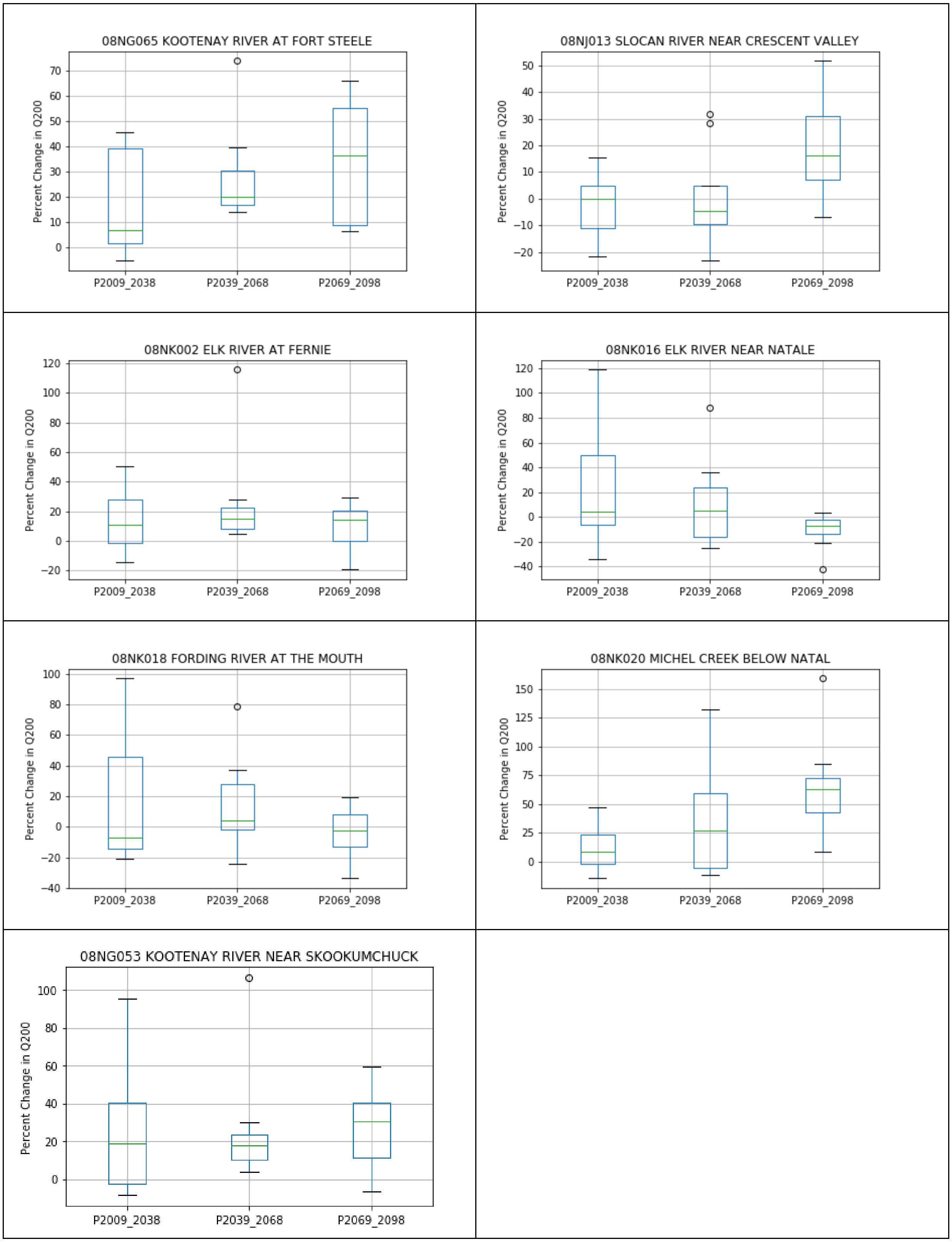

Figure D-12b. Boxplots of the PCIC Hydrological Stations and their change in the magnitude of the 200-year flood (continued).

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# APPENDIX E
# HYDRAULIC ASSESSMENT METHODS

## E.1. INTRODUCTION

This appendix describes the approach used to develop a hydraulic model to estimate flood inundation extents for 20, 50, 200 and 500-year return period floods in the Crawford Creek study area. The following sections describe the methods used to develop the hydraulic model including model selection, model domain, scenarios and sensitivity analyses.

## E.2. MODELLING SOFTWARE

Modelling results, including water surfaces profiles, water depths and flow velocities, were estimated using HEC-RAS version 5.0.7 hydraulic model. HEC-RAS is a public domain hydraulic modelling program developed and supported by the United States Army Corps of Engineers (Brunner & CEIWR-HEC, 2016). This version of HEC-RAS supports both one-dimensional (1-D) and two-dimensional (2-D) hydraulic modelling.

For this study, a 2-D hydraulic model was selected. The 2-D model provides more detailed information on the flow depths and velocities than a 1-D model. A 2-D model also removes some of the subjective modelling techniques which are involved in the development of 1-D models such as defining ineffective flow areas, levee markers and cross-section orientation.

## E.3. MODEL DOMAIN AND BOUNDARY CONDITIONS

The model domain covers a 3 km section of Crawford Creek (Figure E-1). The upstream model boundary is located in a steep gorge approximately 300m below the bridge crossing for Wagar Road. The downstream boundary of the model domain extends approximately 750 m out into Crawford Bay so that the lake level boundary condition does not affect the discharge through the Highway 3A bridge. The edges of the modelled domain were set sufficiently far from the regions of flow so as to not influence the results.

The upstream boundary condition for Crawford Creek was defined as a steady inflow hydrograph for the peak discharges of interest. The Crawford Creek watershed is ungauged, and therefore regional flood frequency analysis was used to estimate the peak discharges for the return period floods (Appendix A). The downstream lake level boundary was at an elevation of 535 m, which is intermediate scenario between BC Hydro’s minimum and maximum flood scenarios; and 0.5 m above the peak recorded reservoir level (July 4, 2012) since commissioning of the Libby Dam (BGC, January 15, 2020).

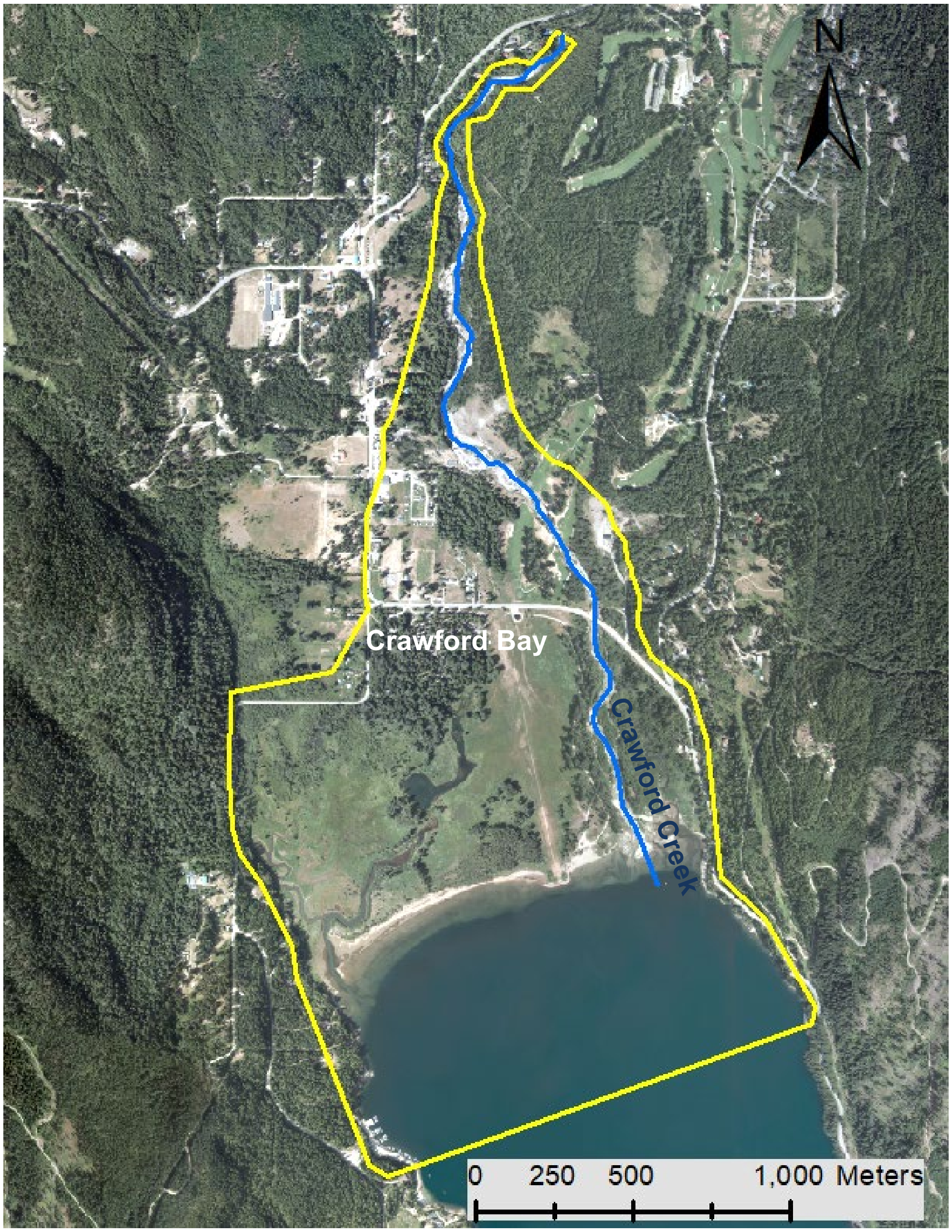


**Figure E-1. Crawford Creek Study Area Modelling Domain.**

## E.4. CHANNEL ROUGHNESS

In common with many hydraulic models, HEC-RAS 2D uses the Manning's roughness coefficient (Manning's n) to represent the hydraulic flow roughness. Measured flow and water level data for high-flow events were not available for Crawford Creek, and therefore the model is uncalibrated with the Manning's n values being selected with guidance from the literature and using empirical equations.

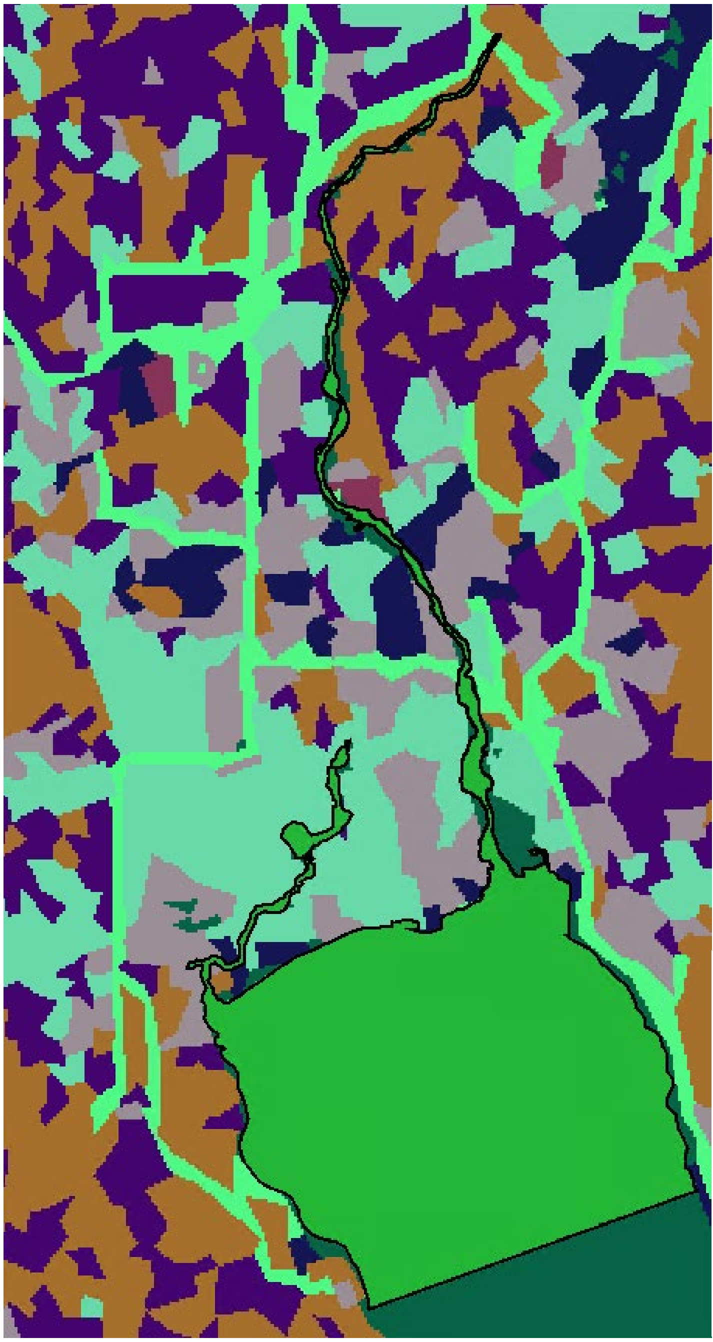
Manning's n values for floodplain areas are based on land cover types (Figure E-2) with Manning's n values for each land cover type from Chow (1959). The spatial land cover distributions were imported from digital land cover maps from the North American Land Change Monitoring System (NRCan, 2019).

The gradient of Crawford Creek main channel in the study area generally ranges between 1% to 2%. Manning's n value for the main channel was estimated using Jarrett's steep-creek equation (Jarrett 1984):

$$n = 0.39S^{0.38}R^{-0.16}$$

where S is the energy slope (assumed equal to the channel slope) and R is the hydraulic radius of the stream (in units of feet). Jarrett's equation is based on 75 observations of streams in Colorado. His streams were composed of bed material ranging from cobbles to small boulders. The range of energy slopes were 0.2 % to 9% and range of hydraulic radii were 0.15 to 2.1 m.

The main channel Manning's n value varies along the length of the channel, and for modelling an average value of 0.06 was selected for the main section of Crawford Creek. A sensitivity on the model results to the Manning's n is provided later in Section E.7 of this appendix.



| Land Class | Manning's n | Color |
|---|---|---|
| 1. Temperate or sub-polar needleleaf forest | 0.1 | 🟫 |
| 2. Temperate or sub-polar broadleaf deciduous forest | 0.1 | 🟩 |
| 3. Mixed Forest | 0.1 | 🟪 |
| 4. Temperate or sub-polar shrubland | 0.07 | ⬜ |
| 5. Temperate or sub-polar grassland | 0.035 | 🟦 |
| 6. Wetland | 0.044 | 🟫 |
| 7. Barren Lands | 0.044 | 🟥 |
| 8. Urban and Built-up | 0.025 | 🟩 |
| 9. Water | 0.044 | 🟩 |
| 10. Main Channel | 0.06 | 🟩 |

Figure E-2. Manning's n Roughness Layer defined for the model.

## E.5. MODEL MESHING

With 2D models the main challenge is to define a model with sufficient accuracy while using a mesh resolution that limits model runtime. The runtime is predominantly determined by the number of cells within the model mesh. The HEC-RAS software for 2D modelling uses an irregular mesh to simulate the flow of water over the terrain. Irregular meshes are useful for development of numerically efficient 2D models as they allow refinement of the model in locations where the flow is changing rapidly and/or where additional resolution is desired and courser in locations where a finer mesh is not necessary.

The default cell geometries created by HEC-RAS are rectangular, but other geometries can be selected to suit the problem under consideration. Within HEC-RAS, a 2D mesh is generated based on the following inputs:

- The model perimeter (the model domain or extent of the model)
- Refinement areas to define sub-domains where the mesh properties (e.g., mesh resolution) is adjusted.
- Breaklines to align the mesh with terrain features which influence the flow such as dykes, ditches, terraces and embankments. HEC-RAS provides options to adjust the mesh resolution along breaklines if the modeler chooses.

From these inputs, HEC-RAS generates the mesh consisting of computational points at the cell centroid and the faces of the cells. The mesh was cleaned and checked for errors such a cell having more than 8 faces and gaps in the mesh.

### E.5.1. Initial Mesh Development

For the Crawford Creek Study area, a base model resolution of 10 m was selected. A breakline was placed along the centerline of the main channel of the creek with a resolution of 2 m and 4 additional cells with the same resolution (repeats) on each side. Terrain features such as ridges, drainage ditches and dikes were captured using breaklines to which the mesh was aligned with a resolution of 2 m and 1 repeat on either side. An exception was the breakline for Highway 3A, which had 2 repeats on either side to fully capture the width of the highway.

### E.5.2. Mesh Refinement

The mesh was refined in two areas on the western side of Crawford Creek where flow was found to exit the main channel (Figure E-3). The upstream refinement region corresponds to an area where flow exits the main channel through existing gaps in dikes and extends south to where the water pools against the base of Highway 3A. This refinement area used a mesh resolution of 1 m. The second refinement area corresponds to an area immediately north of the pedestrian bridge where the flow overtops the creeks banks and flows overland. This refinement area used a mesh resolution of 2 m. A third region of lower refinement in Crawford Bay (30 m resolution) was implemented as the results were less sensitive to mesh size in this region. Additional breaklines were also added in several areas where the additional resolution was needed to eliminate leakage between cells.

The final mesh consisted of 130,000 computational cells with an average cell face length of 5 m and average cell area of 23 m$^2$. A summary of the mesh characteristics is given in Table E-2 and an example of the mesh developed is given in Figure 3.

**Table E-1. Mesh Characteristics.**

| | Resolution (m) | Repeats |
|---|---|---|
| Perimeter | 10 | - |
| Refinement Area (upstream) | 1 | - |
| Refinement Area (downstream) | 2 | - |
| Channel Breaklines | 2 | 4 |
| Highway Breaklines | 2 | 2 |
| Terrain Breaklines | 2 | 1 |

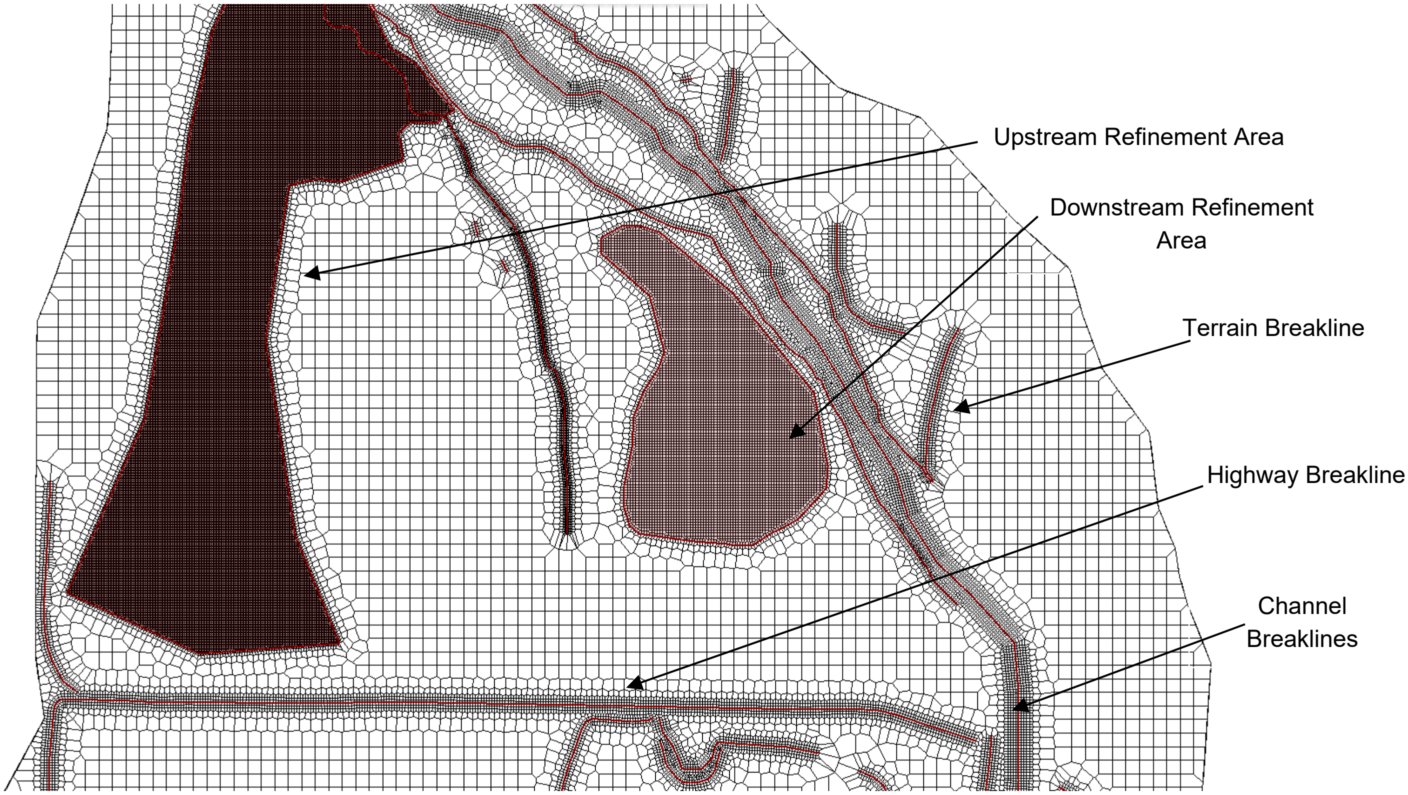


**Figure E-3. Example of the mesh developed for the model showing the breaklines and refinement areas.**

### E.5.3. Hydraulic Structures

The Crawford Creek model domain includes two bridges. As noted in Section C.2.5. the bridge decks were removed from the model as initial screening indicated that the bridge decks provide in excess of one metre of freeboard for a 500-year flow event, and therefore do not impact the results. Likewise, several small culverts are located along the Highway 3A embankment to convey local runoff. The culverts were not included in the model and were assumed to likely to become blocked by debris during flooding.

A group of orphaned dikes were observed along the west bank of Crawford Creek (Drawing 4). During field inspection by BGC, the orphaned dikes were observed to be, in general, poorly maintained (see Figure E-4), and the dikes have been breached four times in the past 60 years, including during high water events in 2012/2013. Therefore, BGC considers that the orphaned dikes could not be relied upon during the design flood events. To simulate dike breach, a section of orphaned dike was removed from the DEM (Figure E-5). The location of the removed was selected following screening to identify a critical dike breach location that would result in the greatest impact.

**Figure E-4. Photo of dike on Crawford Creek's west bank. Photo: Explore, September 25, 2019.**

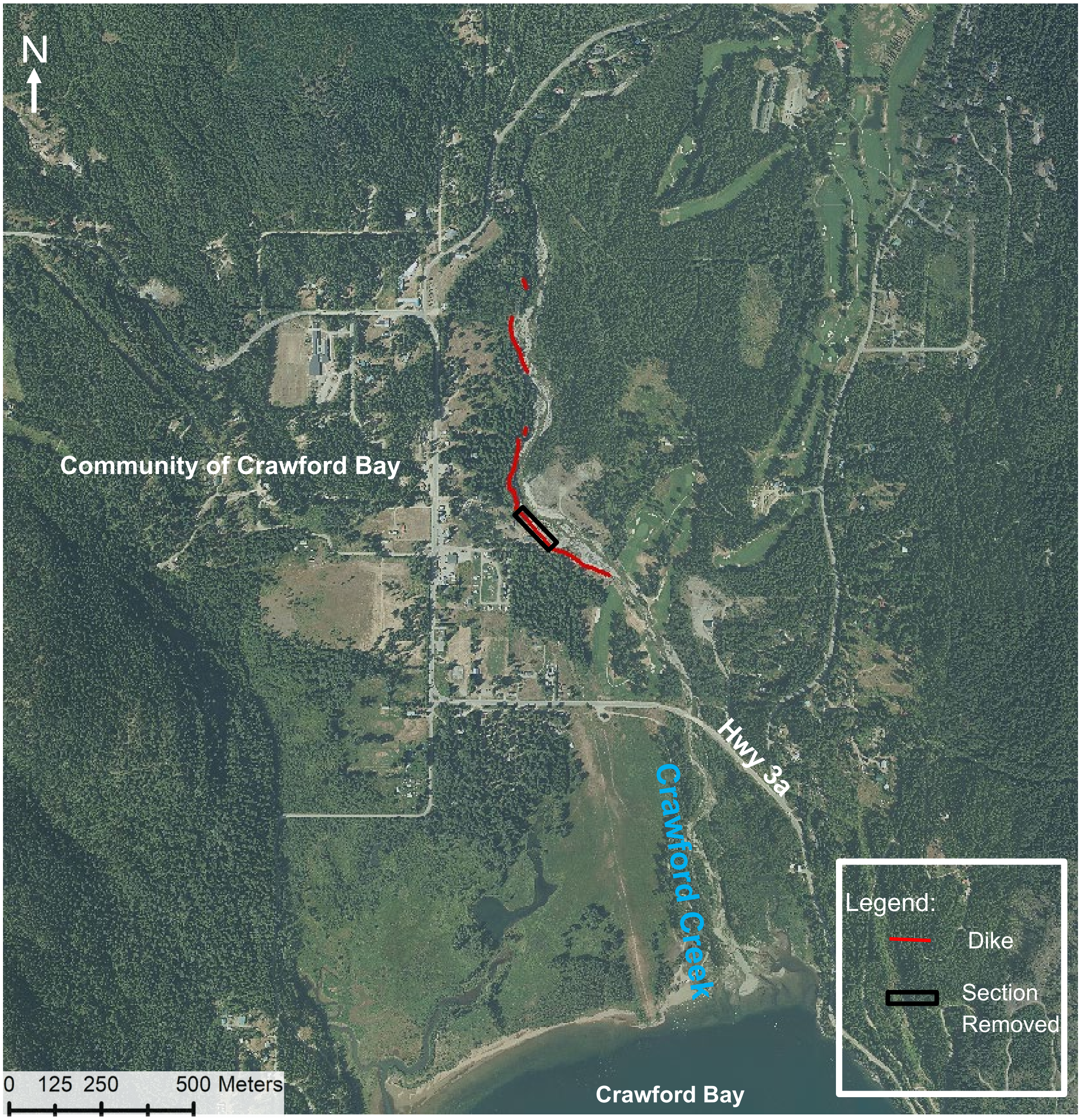


**Figure E-5. Section of dike of removed.**

### E.5.4. Simulation Settings

The HEC-RAS 2D model was run using the full momentum equation with a Courant-controlled time step. The full momentum equations provide accurate representation of flow dynamics especially where sharp construction/expansions/changes in direction are observed. The initial time step was 10 seconds, and the maximum Courant number was 2. The model was run to simulate a 24-hour period and a constant discharge was reached at the downstream boundary after 13 hours.

## E.6. MODELLING SCENARIOS

Scenarios were run for 20, 50, 200 and 500-year flood events. A summary of the modeled events is given in Table E-3. As noted above (Section C.2.7), a breach of the orphaned dikes was included in all scenarios. Sensitivity analysis was performed on the results of the 200-year flow for both Manning's n and the downstream lake level.

**Table E-2. Modelled Scenarios.**

| Return Period | Upstream Boundary Condition (m$^3$/s) | Downstream Boundary Condition (m) | Manning's n | Dike Breach (Y/N) |
|---|---|---|---|---|
| 20 year | 60 | 534.9 | 0.06 | Y |
| 50 year | 73 | 534.9 | 0.06 | Y |
| 200 year | 94 | 534.9 | 0.06 | Y |
| 200 year | 94 | 534.9 | 0.072 | Y |
| 200 year | 94 | 534.9 | 0.048 | Y |
| 200 year | 94 | 531 | 0.06 | Y |
| 500 year | 109 | 534.9 | 0.06 | Y |

## E.7. SENSITIVITY ANALYSIS

As there are no data available for model calibration, a sensitivity analysis for Manning's n was performed. For the 200-year flood event two additional scenarios were run with Manning's n increased by 20% (n=0.072) and decreased by 20% was (n=0.048). When the value of Manning's n was increased by 20% the water surface elevations were found to increase by 1-20 cm in the both the main channel and the flood plain. Similarly, a decrease in the value of Manning's n by 20% resulted in a decrease in water surface elevations of 1-20 cm. The change in the water surface elevation along the channel thalweg of Crawford Creek is shown in Figure E-6.

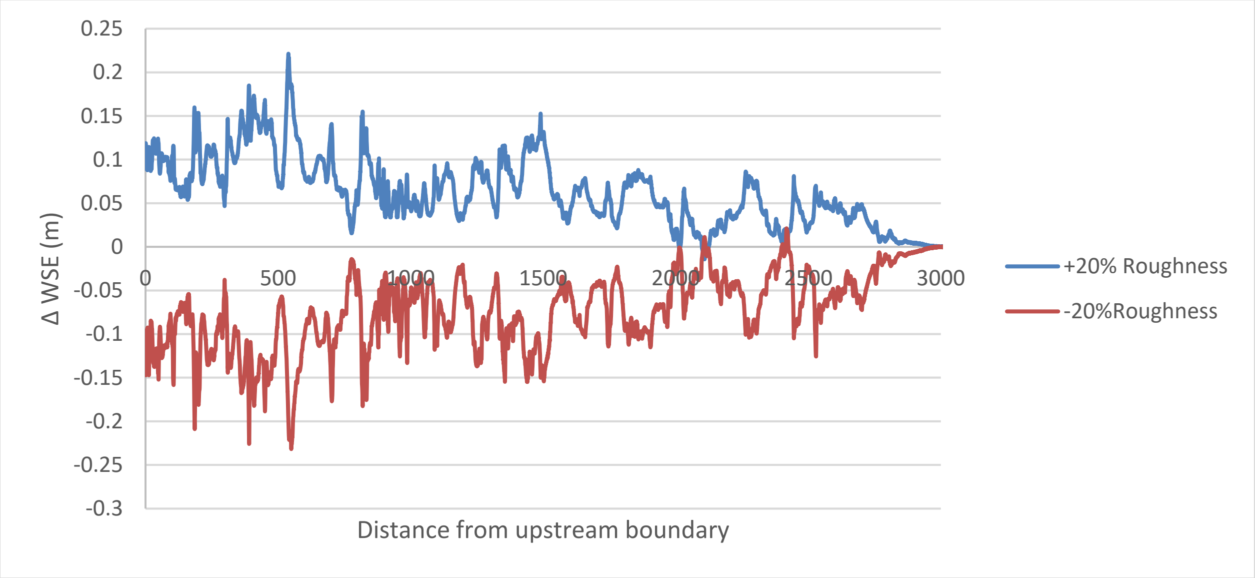


**Figure E-6. Change in WSE along the channel thalweg**

The effect of Manning's n on the WSE is generally cumulative progressing upstream, with the downstream values in all scenarios being fixed by the 535m lake elevation. The Manning's n sensitivity on the inundation areas for the 200-year flood event are shown in Figure E-7 and Figure E-8. The higher Manning's n scenario results in an increase in the predicted flood extent on the western side of the creek. The lower Manning's n scenario indicates has a limited effect on the extent of the inundation area above Highway 3A. A summary of changes to WSE at key locations is provided in Table E-4.

**Table E-3. Summary of changes to WSE at key locations.**

| Location Description | Easting NAD 83 Zone 11 | Northing NAD 83 Zone 11 | Average WSE n=0.048 | Average WSE n=0.06 | Average WSE n=0.072 |
|---|---|---|---|---|---|
| Beginning of modeled area | 513216 | 5503942 | 580.37 | 580.48 | 580.57 |
| Water flowing over bank of channel | 512855 | 5502872 | 553.86 | 553.95 | 554.02 |
| At dike breach | 512960 | 5502628 | 548.93 | 548.94 | 548.96 |
| Flowing over Highway 3A | 513091 | 5502201 | 541.92 | 541.96 | 542.00 |
| Flow under Highway 3A bridge | 513351 | 5502157 | 540.66 | 540.76 | 540.83 |
| Flood Plain Below Highway 3A | 513087 | 5501988 | 539.00 | 539.06 | 539.10 |
| Entering Crawford Bay | 513487 | 5501639 | 534.9 | 534.9 | 534.9 |

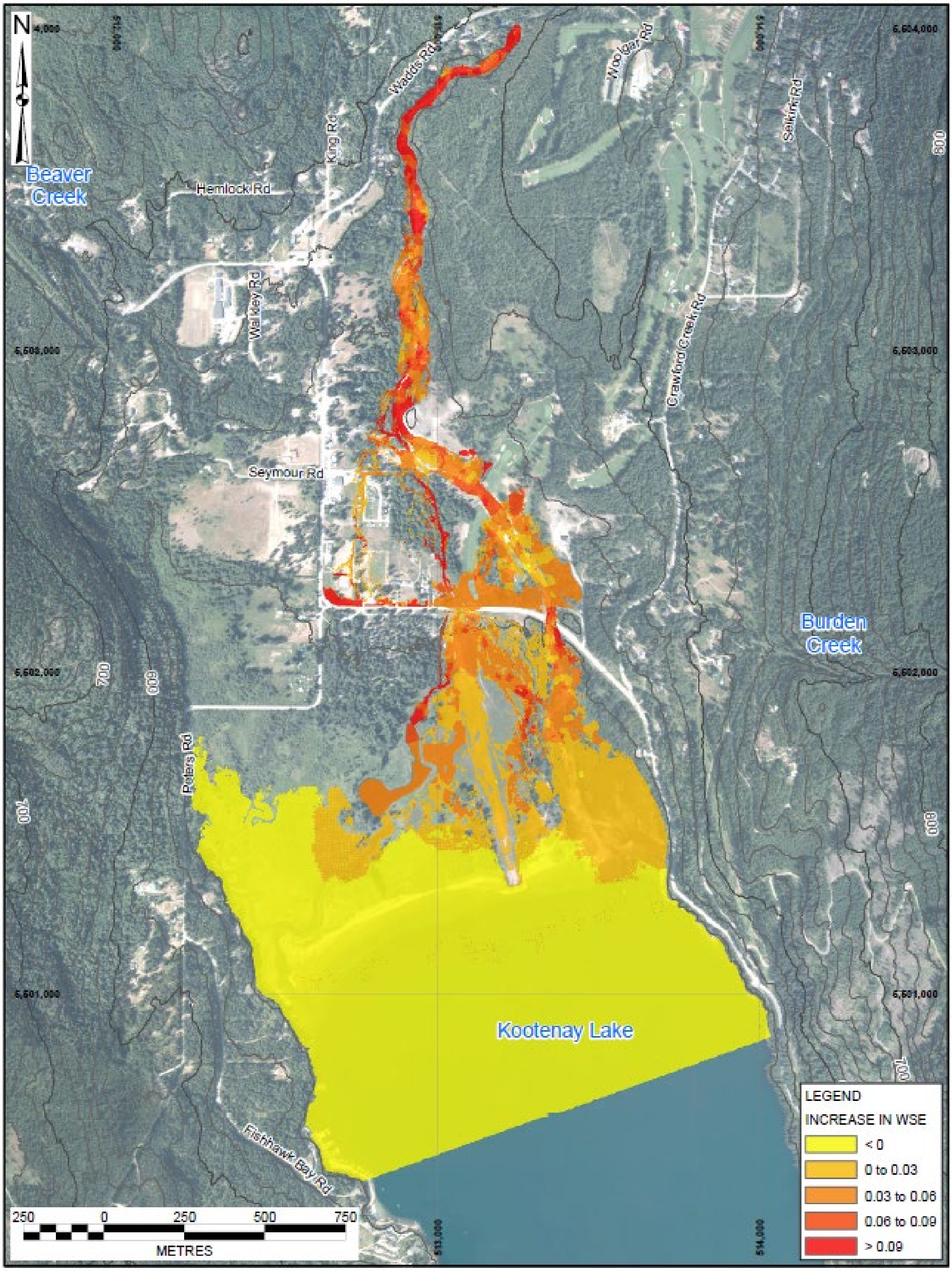


**Figure E-7.** Change in WSE for 20% increase in Manning's n (200-yr flood event).

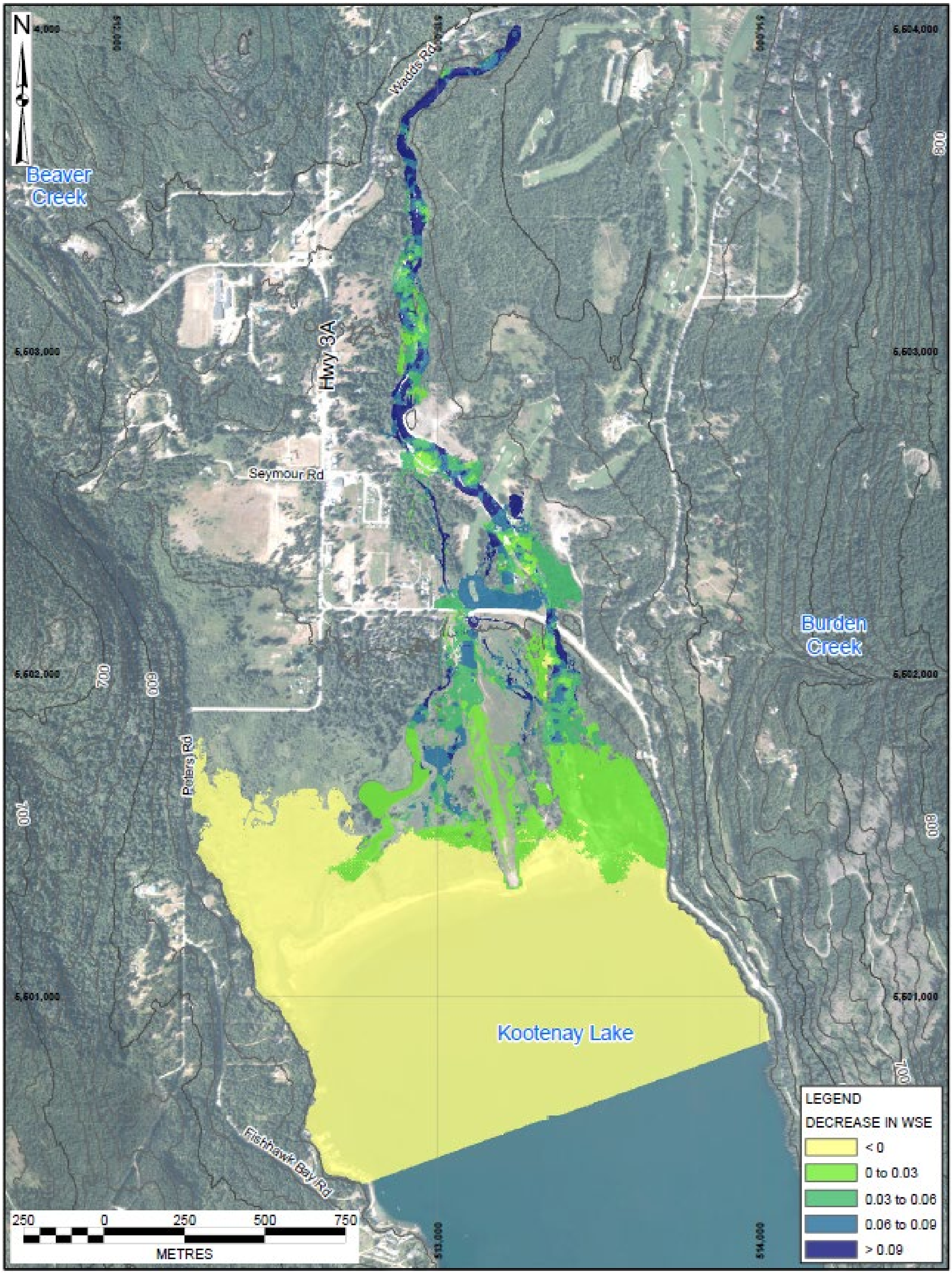


**Figure E-8. Change in WSE for 20% Decrease in Manning's n (200-yr flood event).**

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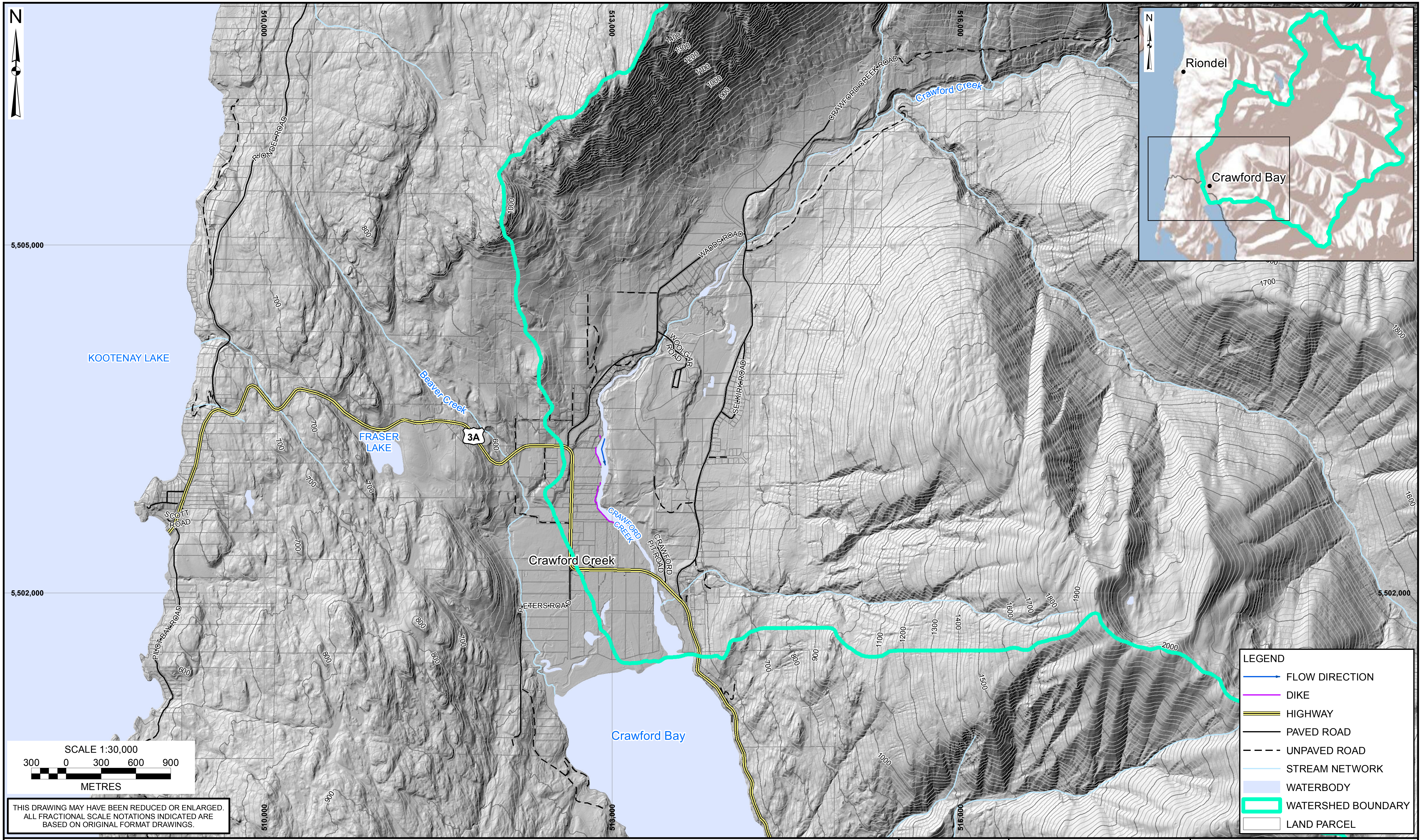
# DRAWINGS



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PROJECT: RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE: SITE LOCATION MAP

PROJECT No.: 0268-007
DWG No: 01

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CRAWFORD CREEK

TITLE: WATERSHED OVERVIEW MAP

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DWG No: 02

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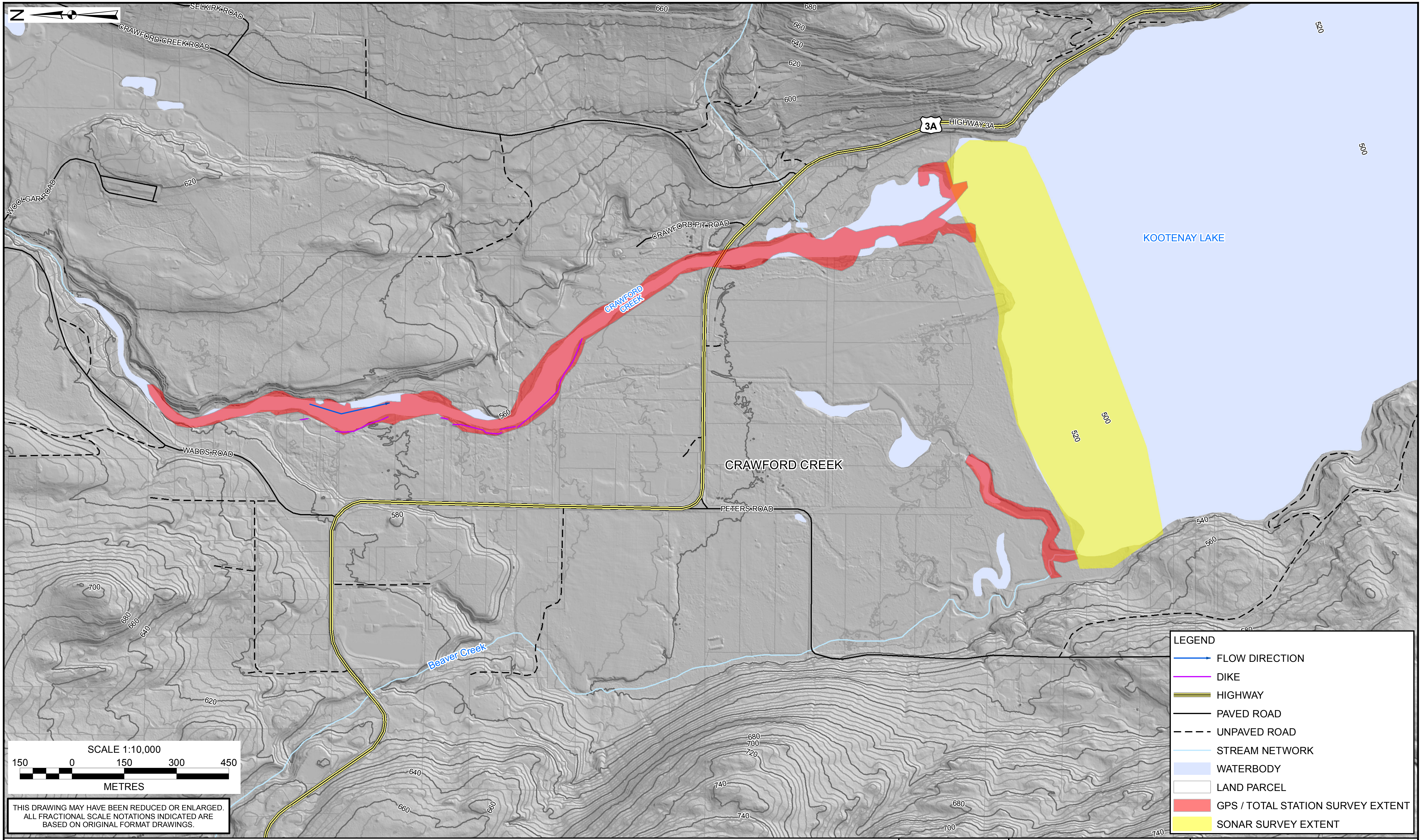

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CRAWFORD CREEK

TITLE:
SITE LOCATION MAP

PROJECT No.: 0268-007
DWG No: 03

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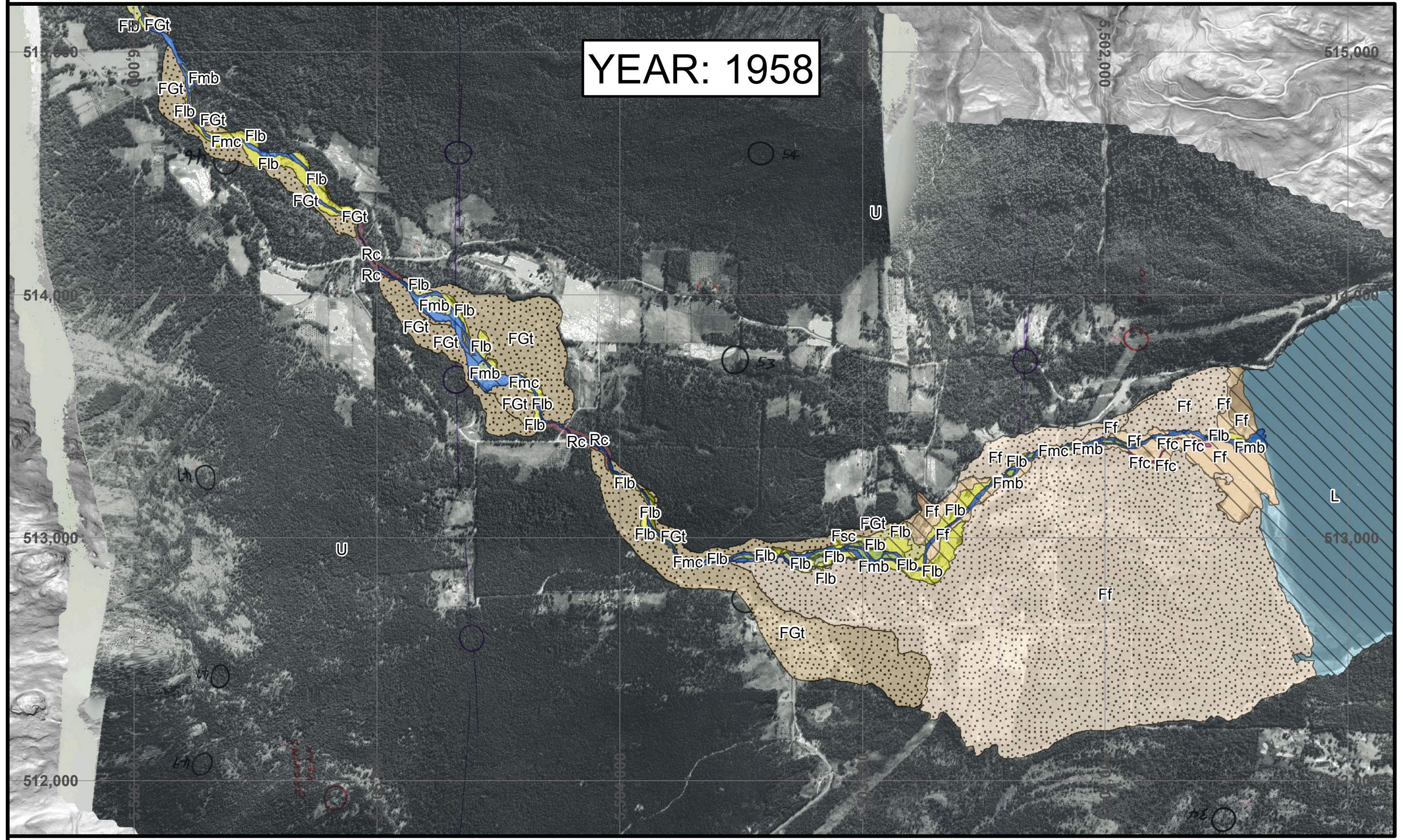

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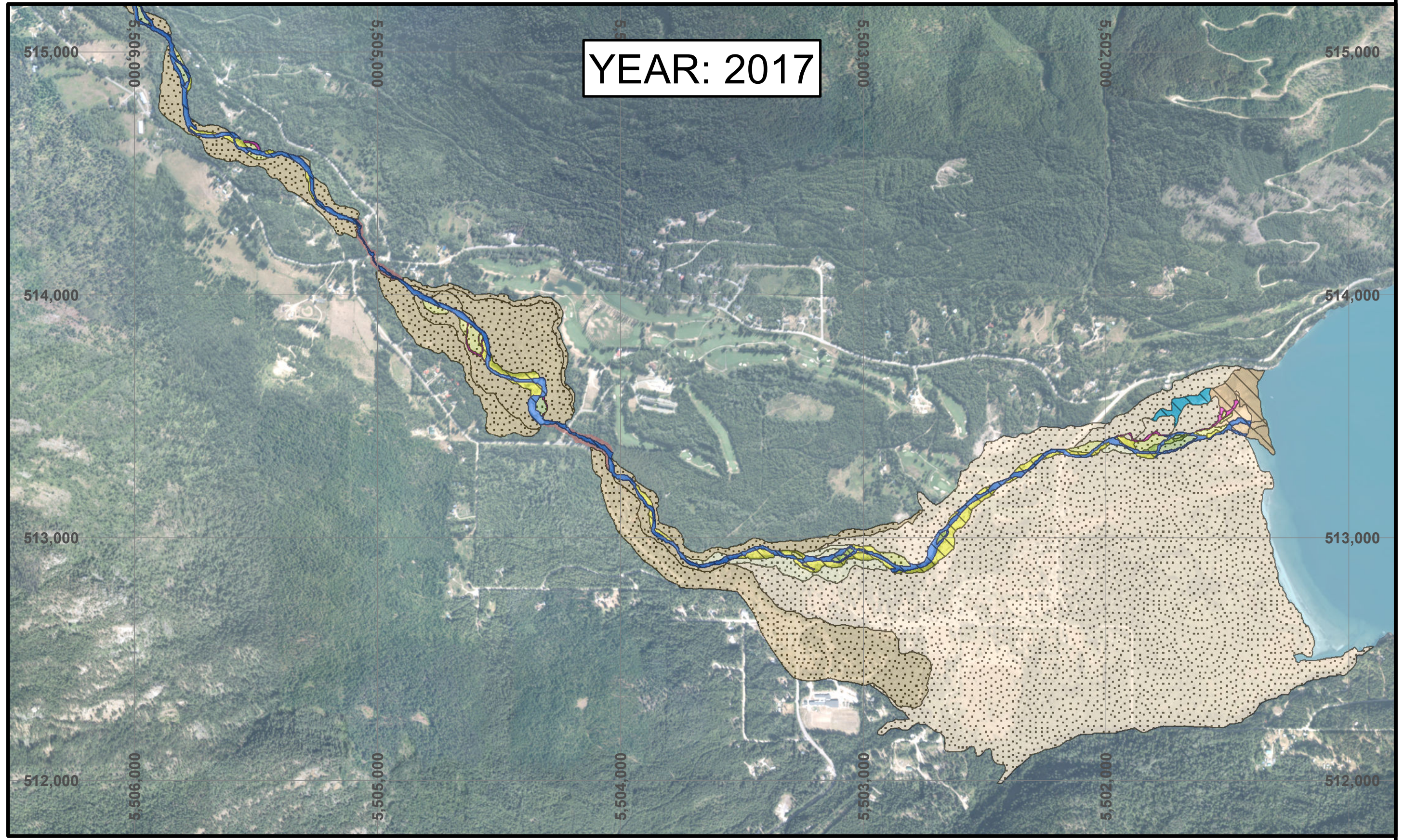

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| 🟧 | ALLUVIAL FAN/DELTA (Ff) | 🟦 | LAKE (L) | 🟫 | TERRACE (FGt) |
| 🟦 | BACK-CHANNEL (Fbc) | 🟨 | LATERAL AND POINT BARS (Flb) | | ACTIVITY LEVEL |
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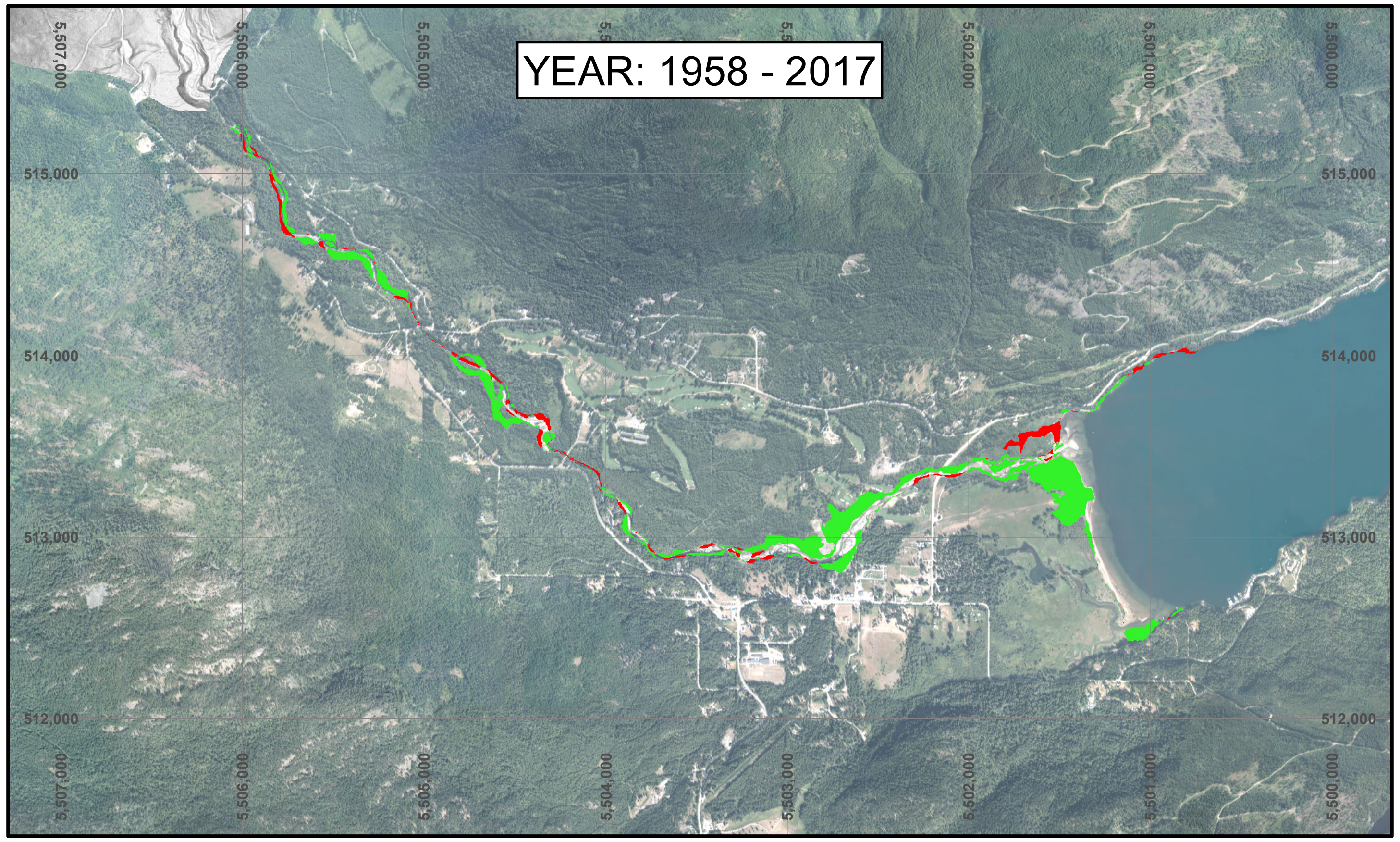

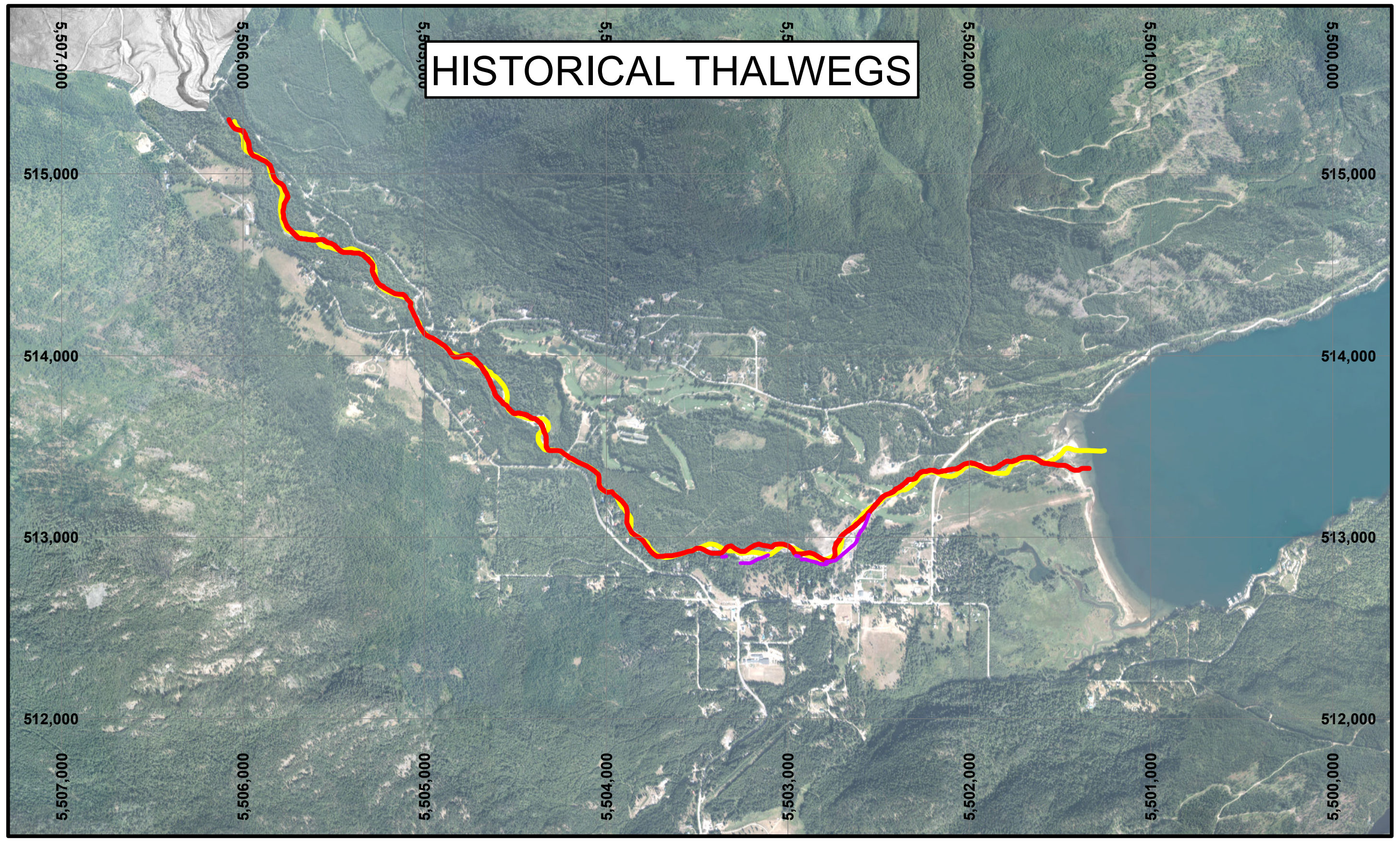

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3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY REGIONAL DISTRICT OF CENTRAL KOOTENAY DATED 2018. BASE IMAGERY REFERENCES IN TABLE 4-2 OF REPORT.
4. COORDINATE SYSTEM IS NAD 1983 UTM ZONE 11N. VERTICAL DATUM IS UNKNOWN.
5. DIKE LOCATIONS FROM FLOOD PROTECTION WORKS: STRUCTURAL WORKS DATASET PROVIDED BY BC MFLNRO (2017). CHANGE DETECTION CRITERIA DESCRIBED IN TABLE 4-5 OF REPORT. HISTORICAL THALWEGS INTERPRETED FROM PHOTOGRAPHS AND MANUALLY DIGITIZED.
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LEGEND

| | | | |
|---|---|---|---|
| ━ | DIKE | 🟥 | BANK EROSION, CHANNEL MIGRATION |
| ━ | CHANNEL THALWEG (2019) | 🟩 | DEPOSITION, STABILIZATION |
| ━ | CHANNEL THALWEG (1958) | | |

| | | | |
|---|---|---|---|
| SCALE: | 1:40,000 | BGC ENGINEERING INC.<br>AN APPLIED EARTH SCIENCES COMPANY | PROJECT: RDCK FLOODPLAIN AND STEEP CREEK STUDY CRAWFORD CREEK |
| DATE: | MAR 2020 | | TITLE: HISTORICAL CHANNEL CHANGE |
| DRAWN: | MW | CLIENT: REGIONAL DISTRICT CENTRAL KOOTENAY | PROJECT No.: 0268-007 |
| CHECKED: | ES/VC | | DWG No: 05 |
| APPROVED: | RM | | |

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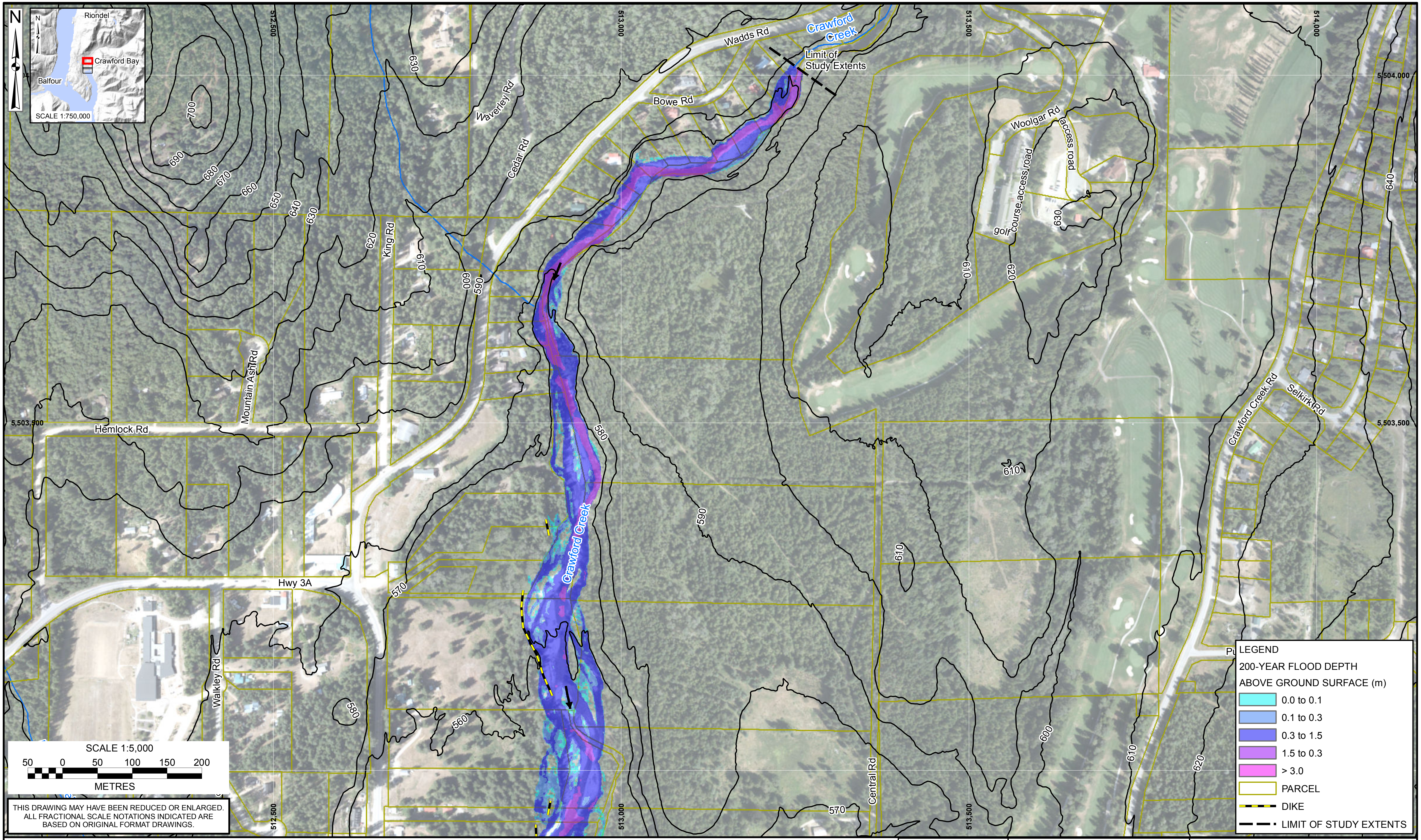

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NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "RDCK FLOODPLAIN AND STEEP CREEK STUDY CRAWFORD CREEK", AND DATED MARCH 2020.
3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY RDCK DATED 2017 AND 2018. CONTOUR INTERVAL IS 10 m. ORTHOPHOTO PROVIDED BY RDCK AND DATED 2017 AND 2018.PARCEL DATA FROM PARCELMAP BC. DIKE DATA FROM DATA BC. FLOOD DEPTH BASED ON THE 200-YEAR FLOOD USING THE INSTANTANEOUS PEAK DISCHARGE ADJUSTED FOR CLIMATE CHANGE AND A KOOTENAY LAKE ELEVATION OF 535 m.
4. PROJECTION IS NAD 1983 UTM ZONE 11N. VERTICAL DATUM IS CGVD2013.
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DATE: MAR 2020
DRAWN: LL
CHECKED: PG, KL
APPROVED: RM

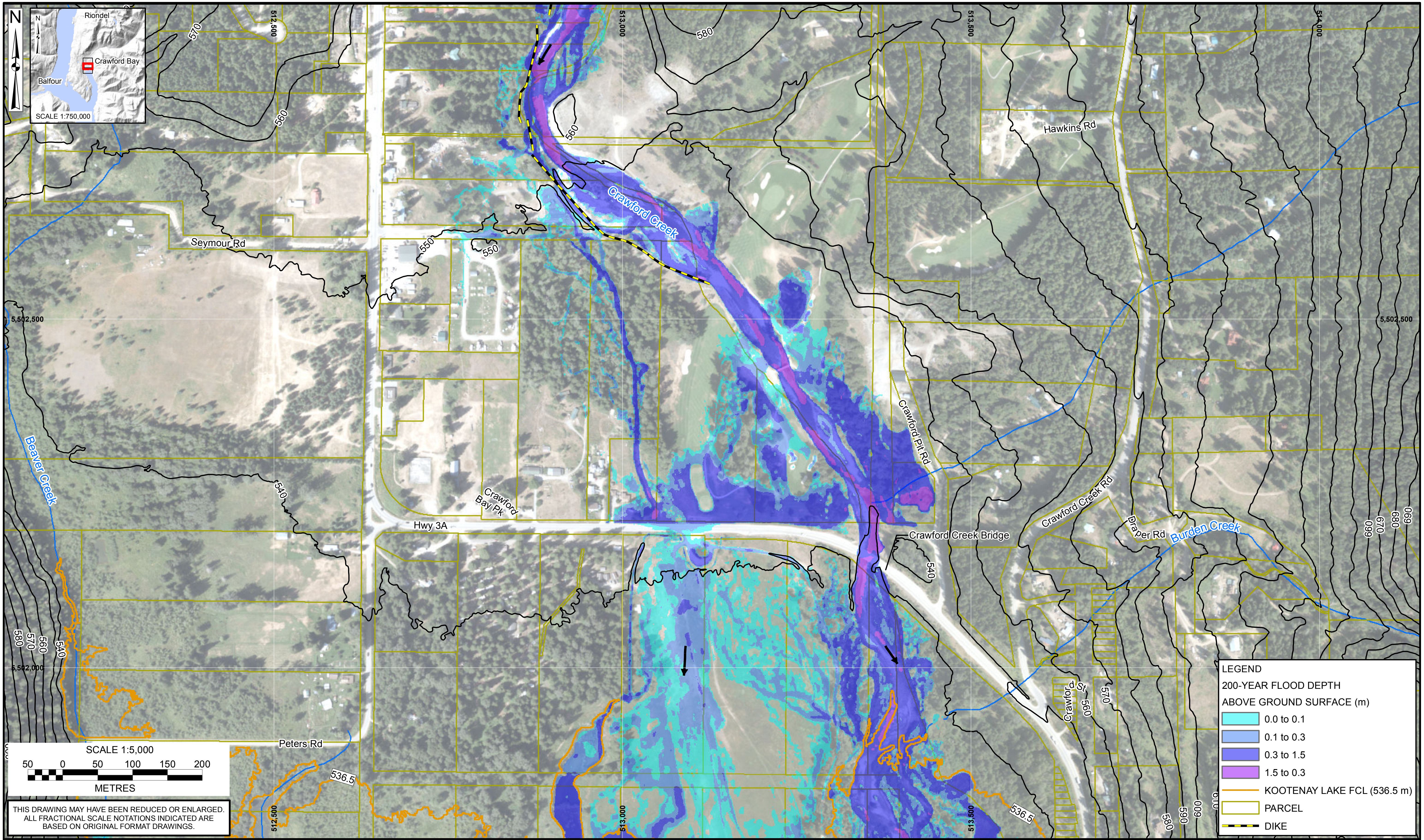
BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD HAZARD
(SHEET 1 OF 3)

PROJECT No.: 0268 007
DWG No: 06



LEGEND

200-YEAR FLOOD DEPTH
ABOVE GROUND SURFACE (m)

| | |
|---|---|
| 🟦 | 0.0 to 0.1 |
| 🟦 | 0.1 to 0.3 |
| 🟦 | 0.3 to 1.5 |
| 🟪 | 1.5 to 0.3 |
| — | KOOTENAY LAKE FCL (536.5 m) |
| ☐ | PARCEL |
| ▬ | DIKE |

SCALE 1:5,000

50 0 50 100 150 200

METRES

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SCALE: 1:5,000
DATE: MAR 2020
DRAWN: LL
CHECKED: PG, KL
APPROVED: RM

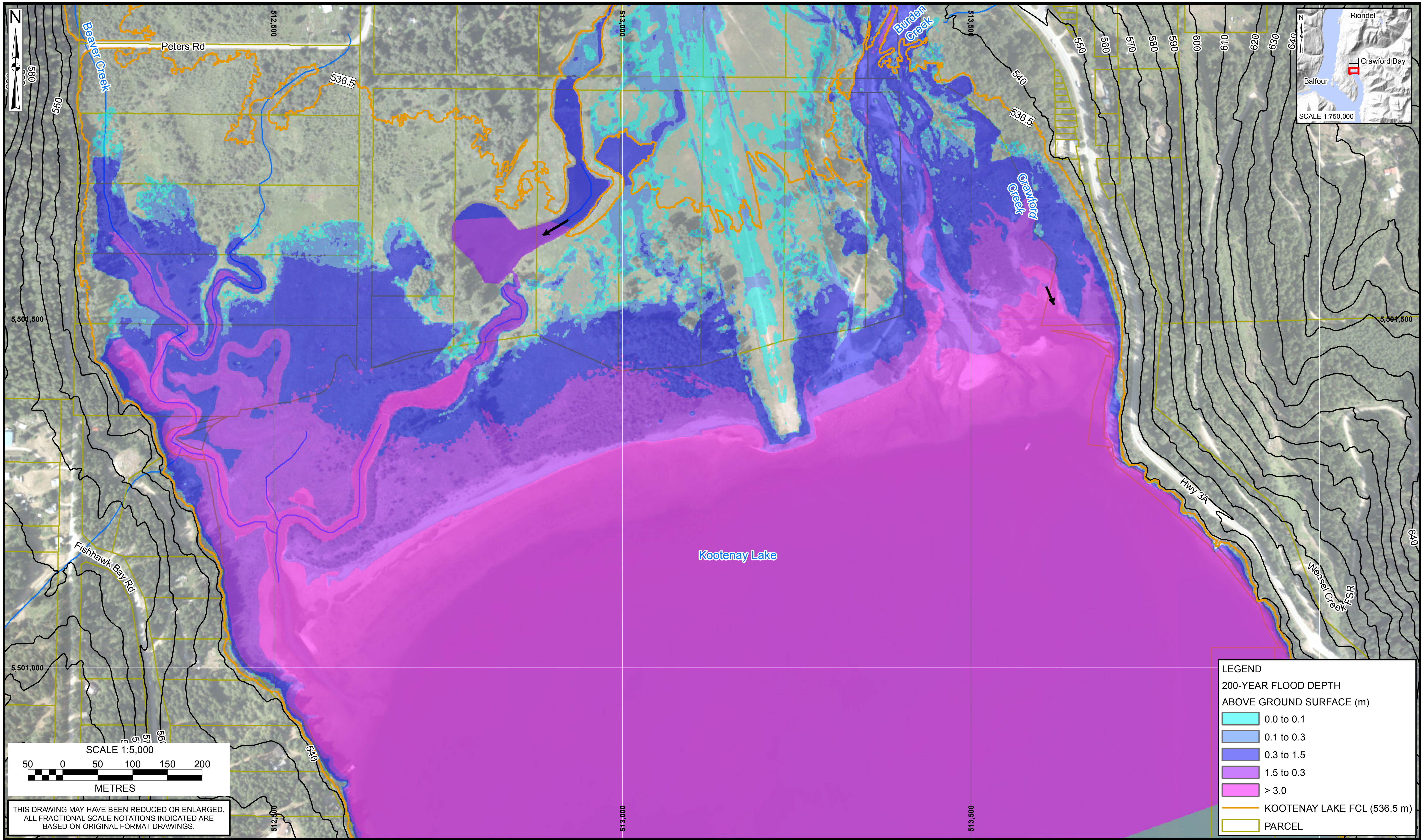
BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT: REGIONAL DISTRICT CENTRAL KOOTENAY

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD HAZARD
(SHEET 2 OF 3)

PROJECT No.: 0268 007
DWG No: 06



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LEGEND

200-YEAR FLOOD DEPTH ABOVE GROUND SURFACE (m)

| | |
|---|---|
| ▨ | 0.0 to 0.1 |
| ▨ | 0.1 to 0.3 |
| ▨ | 0.3 to 1.5 |
| ▨ | 1.5 to 0.3 |
| ▨ | > 3.0 |
| — | KOOTENAY LAKE FCL (536.5 m) |
| ☐ | PARCEL |

NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
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SCALE: 1:5,000
DATE: MAR 2020
DRAWN: LL
CHECKED: PG, KL
APPROVED: RM

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD HAZARD
(SHEET 3 OF 3)

PROJECT No.: 0268 007
DWG No: 06

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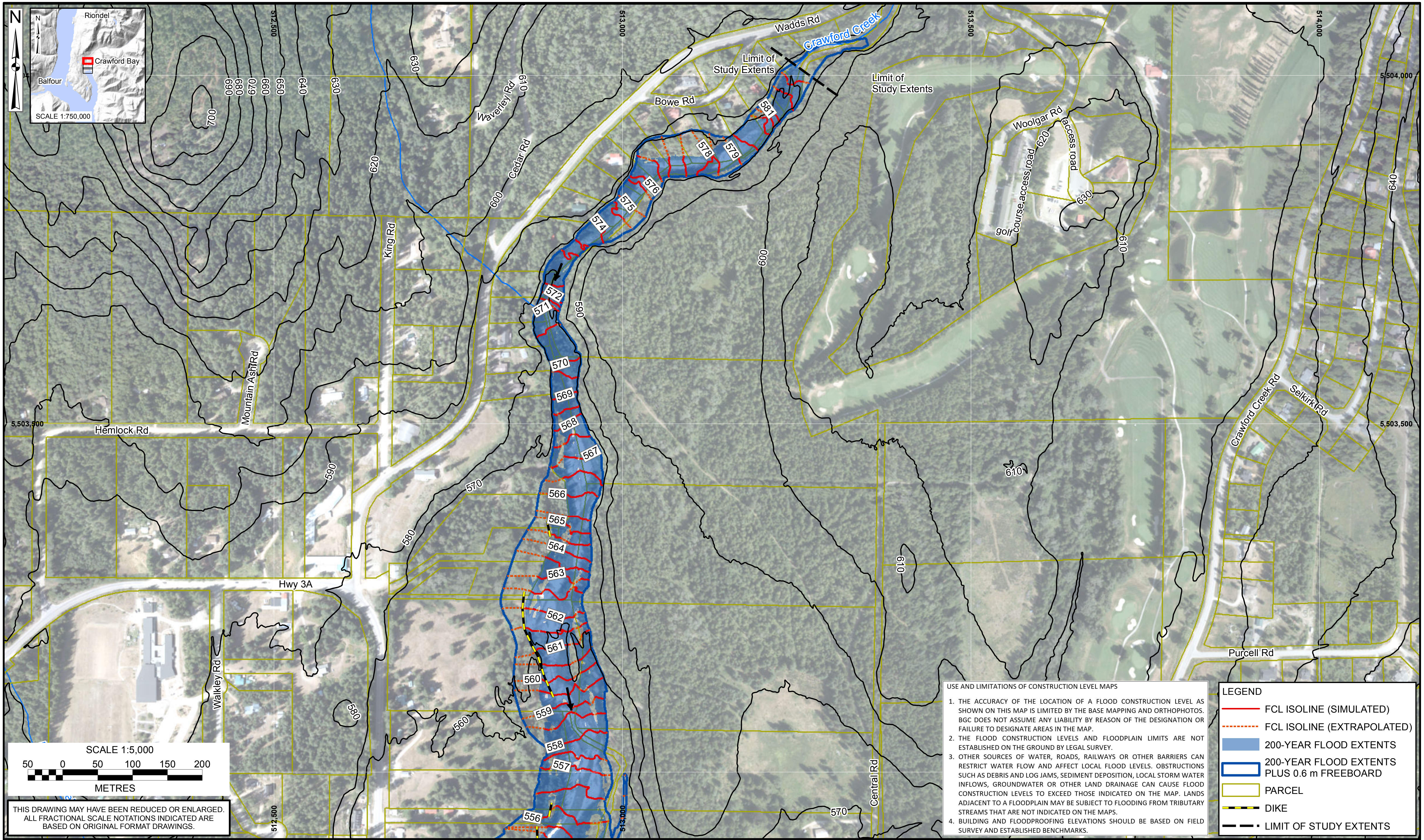

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USE AND LIMITATIONS OF CONSTRUCTION LEVEL MAPS

1. THE ACCURACY OF THE LOCATION OF A FLOOD CONSTRUCTION LEVEL AS SHOWN ON THIS MAP IS LIMITED BY THE BASE MAPPING AND ORTHOPHOTOS. BGC DOES NOT ASSUME ANY LIABILITY BY REASON OF THE DESIGNATION OR FAILURE TO DESIGNATE AREAS IN THE MAP.
2. THE FLOOD CONSTRUCTION LEVELS AND FLOODPLAIN LIMITS ARE NOT ESTABLISHED ON THE GROUND BY LEGAL SURVEY.
3. OTHER SOURCES OF WATER, ROADS, RAILWAYS OR OTHER BARRIERS CAN RESTRICT WATER FLOW AND AFFECT LOCAL FLOOD LEVELS. OBSTRUCTIONS SUCH AS DEBRIS AND LOG JAMS, SEDIMENT DEPOSITION, LOCAL STORM WATER INFLOWS, GROUNDWATER OR OTHER LAND DRAINAGE CAN CAUSE FLOOD CONSTRUCTION LEVELS TO EXCEED THOSE INDICATED ON THE MAP. LANDS ADJACENT TO A FLOODPLAIN MAY BE SUBJECT TO FLOODING FROM TRIBUTARY STREAMS THAT ARE NOT INDICATED ON THE MAPS.
4. BUILDING AND FLOODPROOFING ELEVATIONS SHOULD BE BASED ON FIELD SURVEY AND ESTABLISHED BENCHMARKS.

LEGEND

| | |
|---|---|
| 🟥 | FCL ISOLINE (SIMULATED) |
| ┅ | FCL ISOLINE (EXTRAPOLATED) |
| 🟦 | 200-YEAR FLOOD EXTENTS |
| ▭ | 200-YEAR FLOOD EXTENTS PLUS 0.6 m FREEBOARD |
| ▭ | PARCEL |
| ▬ | DIKE |
| — — · | LIMIT OF STUDY EXTENTS |

NOTES:
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| CHECKED: | PG, KL |
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AN APPLIED EARTH SCIENCES COMPANY

CLIENT:
REGIONAL DISTRICT CENTRAL KOOTENAY

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD CONSTRUCTION LEVEL
(SHEET 1 OF 3)

| PROJECT No.: | DWG No: |
|---|---|
| 0268 007 | 07 |

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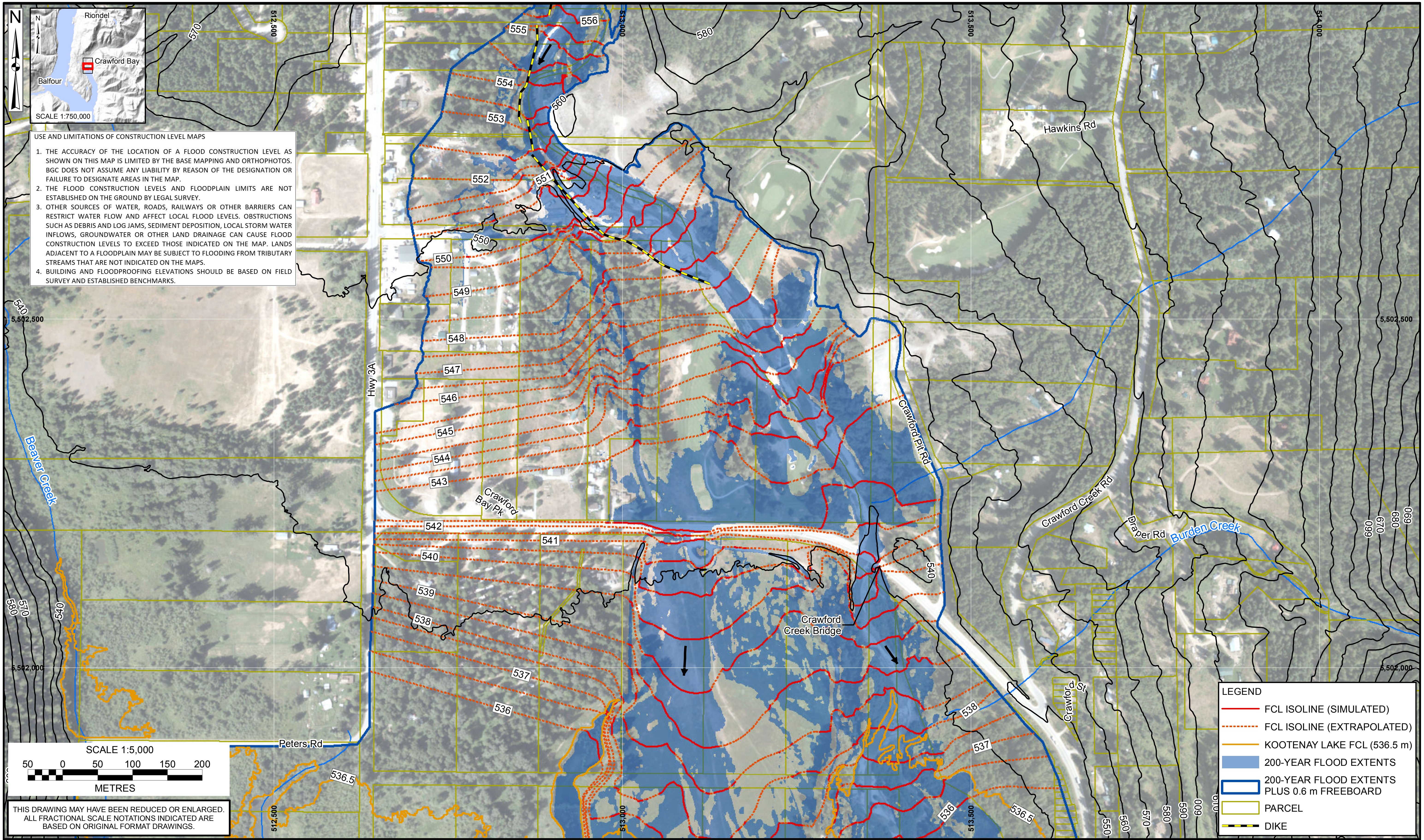

USE AND LIMITATIONS OF CONSTRUCTION LEVEL MAPS

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4. BUILDING AND FLOODPROOFING ELEVATIONS SHOULD BE BASED ON FIELD SURVEY AND ESTABLISHED BENCHMARKS.

LEGEND

- FCL ISOLINE (SIMULATED)
- FCL ISOLINE (EXTRAPOLATED)
- KOOTENAY LAKE FCL (536.5 m)
- 🟦 200-YEAR FLOOD EXTENTS
- ▭ 200-YEAR FLOOD EXTENTS PLUS 0.6 m FREEBOARD
- ▭ PARCEL
- DIKE

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| DRAWN: | LL |
| CHECKED: | PG, KL |
| APPROVED: | RM |

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AN APPLIED EARTH SCIENCES COMPANY

CLIENT:

REGIONAL DISTRICT OF CENTRAL KOOTENAY

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD CONSTRUCTION LEVEL
(SHEET 2 OF 3)

| PROJECT No.: | DWG No: |
|---|---|
| 0268 007 | 07 |

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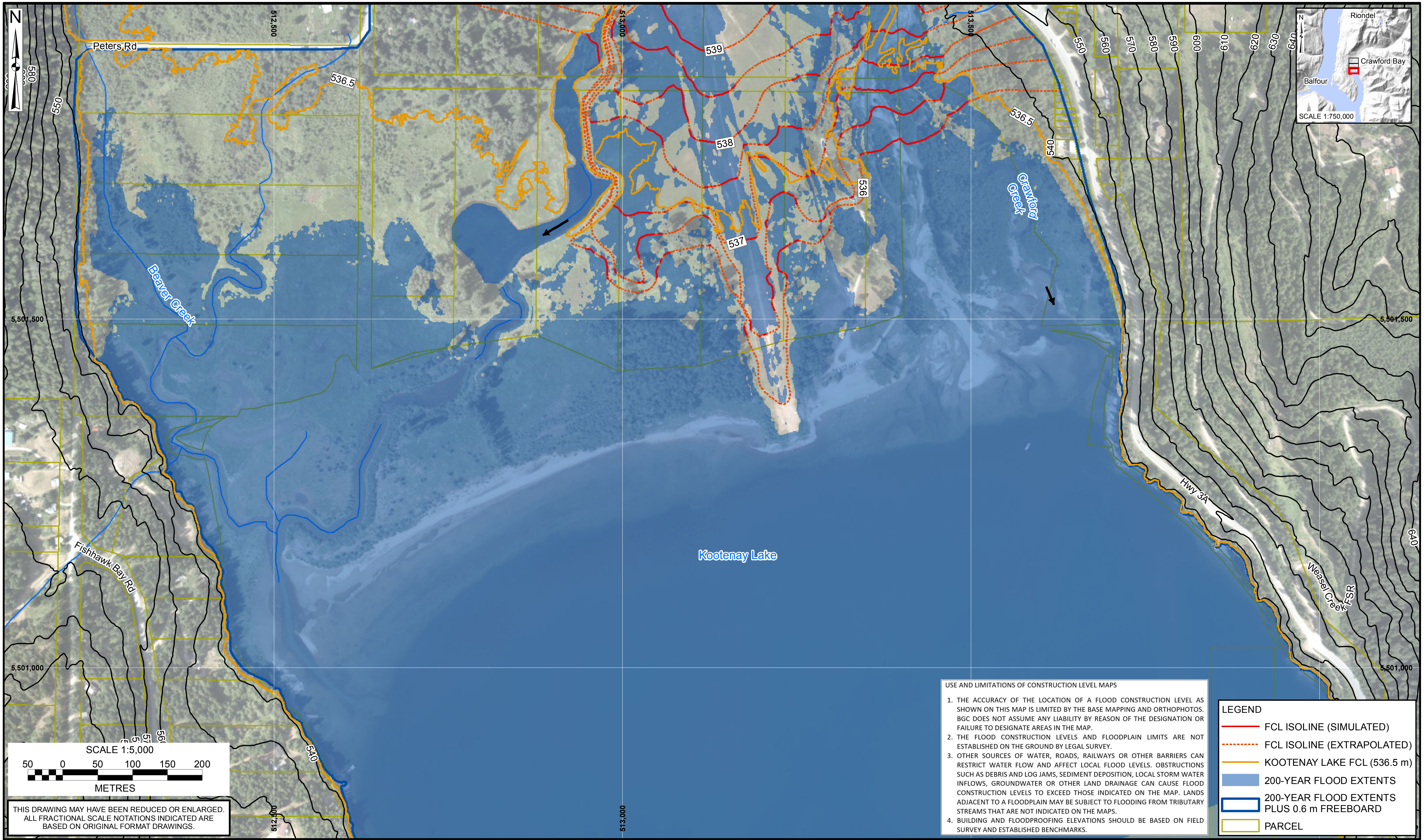

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4. BUILDING AND FLOODPROOFING ELEVATIONS SHOULD BE BASED ON FIELD SURVEY AND ESTABLISHED BENCHMARKS.

LEGEND

| | |
|---|---|
| — | FCL ISOLINE (SIMULATED) |
| - - - | FCL ISOLINE (EXTRAPOLATED) |
| — | KOOTENAY LAKE FCL (536.5 m) |
| ■ | 200-YEAR FLOOD EXTENTS |
| ☐ | 200-YEAR FLOOD EXTENTS PLUS 0.6 m FREEBOARD |
| ☐ | PARCEL |

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| APPROVED: | RM |

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:

PROJECT:
RDCK FLOODPLAIN AND STEEP CREEK STUDY
CRAWFORD CREEK

TITLE:
200-YEAR FLOOD CONSTRUCTION LEVEL
(SHEET 3 OF 3)

| PROJECT No.: | DWG No: |
|---|---|
| 0268 007 | 07 |

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