BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DUNN LAKE ROAD

HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT

FINAL

PROJECT NO.:	0272033	DATE:	May 14, 2021
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May 14, 2021
Project No.: 0272033

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Dear Mr. Sullivan,

Re: Dunn Lake Road, Hydrotechnical and Geomorphic Assessment

Please find attached our report that details Phase 1 hydrotechnical and geomorphic assessment and Phase 2 detailed design of the Dunn Lake Road crossings of Newhykulston and Skowootum Creeks, located north of Barriere, BC. This work was completed as part of planned upgrades to Dunn Lake Road.

Yours sincerely,

BGC ENGINEERING INC.
per:

Evan Shih, M.Eng., P.Eng.
Senior Hydrotechnical/Geological Engineer

EXECUTIVE SUMMARY

As part of an in-progress upgraded highway design, BC Ministry of Transportation and Infrastructure (MOTI) retained BGC Engineering Inc. (BGC) to complete hydrotechnical and geomorphological assessments, and detailed design for the replacement of the Newhykulston Creek and Skowootum Creek culvert crossings along Dunn Lake Road. The overall objective of the work was to assess and provide various culvert replacement options for both crossings and prepare detailed designs for the selected replacement options.

BGC estimated design flows for the replacement culverts using a rainfall-runoff model and completed a climate change analysis (and vulnerability risk assessment) to determine the appropriate upward adjustment of design flows based on modelled increases of annual maximum daily rainfall over the anticipated service life of the structures. For both creeks, estimates of the 100-year peak instantaneous flow (Q_{i100}) were found to exceed the 200-year maximum daily flow (Q_{200}) and, therefore, were selected as the design flows (Table E-1). Based on the results of the climate change vulnerability risk assessment it is recommended that potential climate change impacts be incorporated into the design of the Newhykulston Creek crossing, but not at the Skowootum Creek crossing due to the likelihood of the design flood event to trigger a debris flow that would be more damaging than climate change adjusted clearwater flows.

Table E-1. Design flows based on observed data and with climate change adjustment.

Parameters	Skowootum Creek	Newhykulston Creek
Q_{i100} (m^3/s)	**9.5**	12.6
Q_{i100} (m^3/s) with Climate Change Adjustment	16.5	**21.9**

Note:

1. Bolded values indicate flows used for culvert sizing

Results from the geomorphological assessment suggest that Newhykulston Creek is prone to debris floods, whereas Skowootum Creek is prone to both debris flows and debris floods. At Newhykulston Creek, the volumetric sediment concentration of debris floods are expected to be less than 5%; therefore, no adjustment was made to the 100-year return period peak instantaneous flow, particularly as climate change was accounted for in the peak flow estimates. It is important to note that irrespective of the preferred culvert replacement option, the design event could result in an upstream avulsion due to sudden channel aggradation. In this case, the avulsion channel could cause damage to Dunn Lake Road and result in flood damage to buildings on the fan.

At Skowootum Creek, given the potential size of future debris flows and the steep fan topography, debris-flow mitigation would be costly (>$1 million). Because of the low volume use of Dunn Lake Road and general lack of development on the fan, mitigation was not a recommended option from a cost-benefit perspective. Rather, the crossing was recommended to be designed for clearwater floods only while acknowledging that maintenance will be required following future debris flow events. This recommendation was based on economic consequences only and did not consider the potential for loss of life, which was outside the scope of work for this study.

Various culvert replacement options were compared based on culvert type, size, construction considerations, and fish passage considerations. MOTI ultimately selected a 2.7 m diameter round corrugated steel pipe (CSP) culvert to replace the Skowootum Creek crossing, and a 5 m (span) x 2 m (rise) low-profile open bottom arch culvert to replace the Newhykulston Creek crossing. Detailed design drawings are appended to this report.

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LIMITATIONS

BGC Engineering Inc. (BGC) prepared this document for the account of BC Ministry of Transportation and Infrastructure. The material in it reflects the judgment of BGC staff in light of the information available to BGC at the time of document preparation. Any use which a third party makes of this document or any reliance on decisions to be based on it is the responsibility of such third parties. BGC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

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1.0 INTRODUCTION

1.1. Background

The BC Ministry of Transportation and Infrastructure (MOTI) has been working with the Simpcw First Nation for many years to coordinate improvements to the Dunn Lake Road and to establish a Section 35 Land Transfer, which has established a Dunn Lake Road Right-of-Way (ROW). Any proposed improvements to the road that extend beyond the established ROW would require further negotiation with the Simpcw First Nation Band. To support the highway upgrade design, MOTI requested BGC complete hydrotechnical and geomorphological assessments, and detailed design for the Newhykulson Creek[1] and Skowootum Creek culvert crossings along Dunn Lake Road. The crossings are located approximately 10 to 15 km north of Barriere, BC.

BGC commenced Phase 1 of the project - preliminary hydrotechnical and geomorphic assessment of the two crossings in late 2018. The overall objective of Phase 1 was to assess and provide various culvert replacement options for both crossings. Phase 2 – hydrotechnical detailed design commenced in late 2019 after preferred culvert replacement options were selected by MOTI.

1.2. Scope of Work

BGC's scope of work for this project was detailed in proposals dated December 24, 2018 for Phase 1 and November 15, 2019 for Phase 2. All works and services were provided in accordance with Contract No. 831CS0990, As & When Hydrotechnical Engineering and Design Services.

[1] This creek is known as Neqwey-que-lsten Creek by the Simpcw First Nation. Some records may also refer to it as Coal Creek.

1.3. Design Considerations

Per discussions with MOTI, BGC's work considered the following design considerations:

- Dunn Lake Road is assumed to be a Low Volume Road and therefore MOTI's Low Volume Road standards will apply to this project.
- Based on MOTI standards and BC Water Sustainability Act regulations, the design flow rates will either be the 100-year peak instantaneous flow (Q_{i100}) or the 200-year maximum daily flow (Q_{200}), whichever is the higher of the two.
- This project will be funded through MOTI's Rehabilitation Funding and overall project cost is important to keep to a minimum.
- All hydrotechnical designs shall follow the following design guidelines:
  - Canadian Highway Bridge Design Code (CHBDC), CSA S6-14 and the BC MoTI Bridge Standards and Procedures Manual, Supplement to CHBDC S6-14;
  - Guide to Bridge Hydraulics, Transportation Association of Canada; and
  - BC Supplement to TAC Geometric Design Guide (MOTI, 2019).
- The Newhykulston Creek culvert needs to be designed to allow for fish passage.

1.4. Highway Infrastructure Climate Change - Resilient Design

As specified in EGBC (2020), the deliverables for every MOTI Highway Infrastructure design project should include, at a minimum:

- A Highway Infrastructure Climate Change – Resilient Design Report (Climate Change Design Report);
- A Climate Change Vulnerability Risk Assessment Assurance Statement (Assurance Statement); and,
- A MOTI Design Criteria Sheet for Climate Change Resilience (Design Criteria Sheet).

Given that the scope of work is limited to hydrotechnical assessment and design, documentation requirements for the Climate Change Design Report are incorporated into this hydrotechnical report. The Assurance Statement and Design Criteria Sheet are provided in Appendix A.

2.0 SITE CHARACTERIZATION

2.1. Site Visits

BGC mobilized to site on November 28, 2018 to observe ground conditions prior to snowfall. The site visit was conducted by Hamish Weatherly, M.Sc., P.Geo. and Rob Millar, Ph.D., P.Eng. of BGC, accompanied by Mike Sullivan, Bonnie Meints, and Shawn Clough of MOTI. The site visit included an office meeting with three members of the Simpcw First Nation.

On June 24, 2020, Evan Shih, M.Eng., P.Eng. of BGC conducted a follow up site visit accompanied by Mike Sullivan, Bonnie Meints, and Donna Olsen of MOTI. The purpose of the follow up site visit was 1.) to allow Evan, as the hydrotechnical Engineer of Record (EoR) for the culvert replacement designs, to become familiarized with the site and 2.) to determine the optimal approach, from both engineering and environmental perspectives, to transition instream works at Newhykulston Creek to the natural channel upstream.

2.2. Physiography

Newhykulston and Skowootum creeks are located in the Interior Plateau physiographic region of British Columbia. This physiographic region has two large sub-regions: the Shuswap Highlands and the Thompson Plateau. This region is characterized by little to moderate topographic relief.

At the maximum of the Fraser Glaciation, ice flowed across the Interior Plateau as a continental ice dome to the south, at times discordantly across present topographic features (Tipper, 1971; Ryder et al., 1991). Deglaciation was accomplished through retreat of valley glaciers and downwasting on the plateau (Fulton, 1986; Ryder et al., 1991). The Interior Plateau was ice free at the North Thompson Valley between 11,000 and 10,000 years before present (Fulton, 1986). During glaciation and post-glaciation, till and glaciofluvial materials were deposited in large volumes.

2.3. Climate

Newhykulston Creek and Skowootum Creek are located at the boundary between the Shuswap Highlands and the Thompson Plateau sub-regions, challenging the characterization of the prevailing climate and hydrology of the watersheds.

The Thompson Plateau has a distinct climate boundary that is characterized by a large scale easterly flowing weather pattern, which generates cool, dry winters and warm to hot, dry summers (Demarchi, 2011). As the westerly winds pass over the Coast Mountains, the air descends into the plateau and is warmed by compression, creating the driest zone in British Columbia, averaging 300 - 500 mm per year (Moore et al., 2010), although some higher elevations will see as much as 700 mm annually or more. This sub-region of the Interior Plateau lies within the rain shadow of the Coast Mountains, creating a hydrographic zone called the “Southern Interior Low Flow Zone” (Moore et al., 2010). However, along the eastern and western margins of the plateau, it may be slightly moister and cooler due to elevation and topographic controlled rainfall.

In contrast, the Shuswap Highlands does not have distinct climatic boundaries. The sub-region is characterized by a large scale easterly flowing weather pattern which generates a moderately dry and cool to cold winter and warm summer (Demarchi, 2011). Precipitation increases from west to east with distance from the Coastal Mountains and an increase in topography approaching the Columbia Mountains. Although relatively dry, this sub-region also experiences significant precipitation events during the early summer which produce localized flooding. These localized storms are created as air is forced to rise before entering the Cariboo Mountains, increasing precipitation throughout the highlands (Coulson & Obedkoff, 1998). Precipitation in this region is affected by elevation and averages between 500 and 1,000 mm per year at lower elevations and as high as 1,400 mm at higher elevations.

The closest Environment and Climate Change Canada (ECCC) meteorological station to the study watersheds is *Darfield* (#1162265). This station is located immediately adjacent to the study creeks on the west side of the North Thompson Valley at an elevation of 466 m (Drawing 01). Temperature and precipitation records at this station date back to 1956. Climate normals for the period 1981-2010 are shown in Figure 2-1. Average annual precipitation for this period is 485 mm.

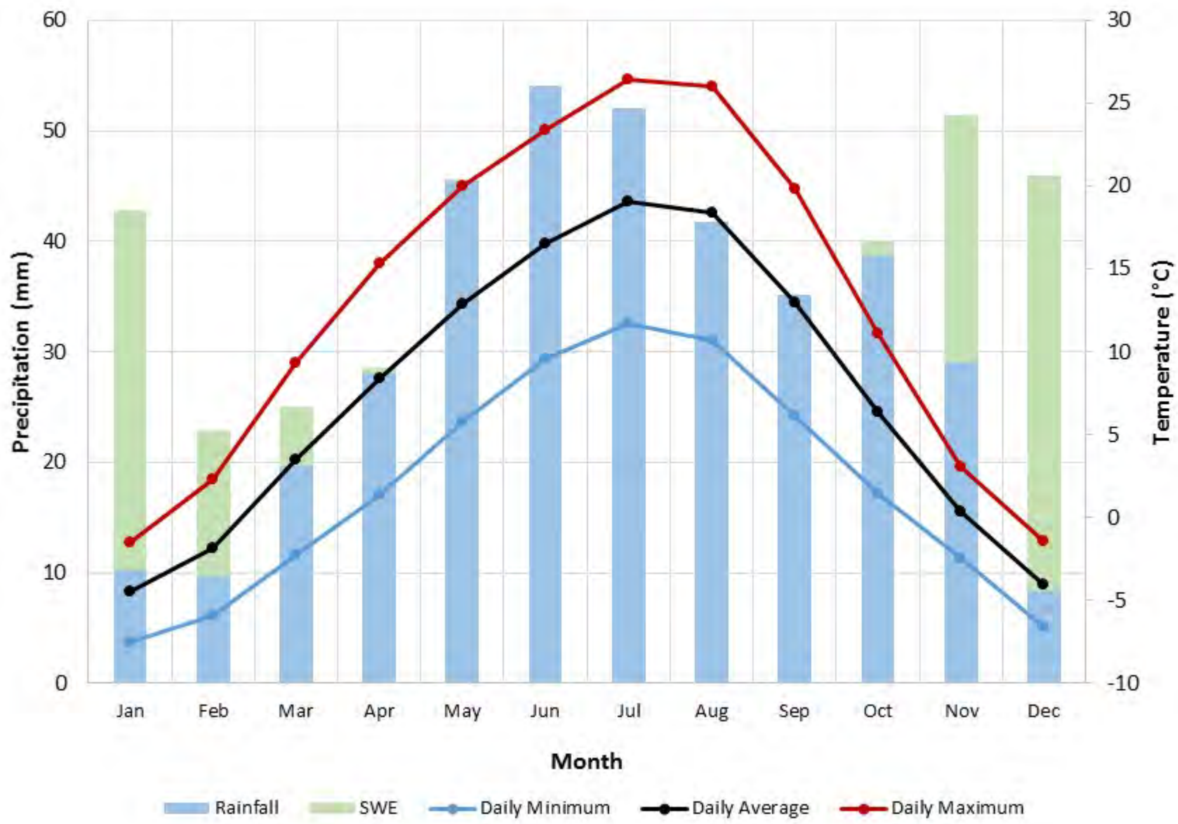

Figure 2-1. Climate normals for ECCC Darfield station (1981-2010). SWE = snow water equivalent.

Flow regimes in both sub-regions with catchments less than 1,000 km^2 are influenced by spring snowmelt (nival) and summer rainfall. Due to low to moderate regional precipitation (which increases from west to east), moderate elevation influences on precipitation and the occurrence of intense localized events during the summer, significant variations in flow regimes between individual watercourse crossings within a given year are common.

2.4. Geomorphology

2.4.1. Watersheds

Both creeks originate in the plateau region on the east side of the North Thompson River valley (Drawing 01). Newhykulston Creek drains an area of 14.3 km^2. Total relief in the watershed is 1460 m, with elevations ranging from 380 m at the confluence with the North Thompson River to 1840 m at its highest point. The terrain in the watershed above elevations of approximately 1600 m is gently-sloping plateau. Between 1600 m and 700 m elevation the creek is confined by moderately steep terrain, as it flows to the west for approximately 2 km before turning to the north for another 2 km. The average channel gradient in this 4 km reach is 20%. Below an elevation of 700 m, the creek turns to the west and becomes increasingly confined within a narrow, V-shaped valley. Sideslopes are steep in this reach and are mantled by a thick and dense till. Several slope instabilities are visible in this reach on both historic aerial photographs and recent Google Earth™ imagery. The fan apex of Newhykulston Creek is located at an approximate elevation of 460 m. A channel profile of the creek is provided in Figure 2-2.

Like Newhykulston Creek, the upper watershed of Skowootum Creek consists of gently to moderate sloping terrain (Drawing 01, Figure 2-3). Below an elevation of 1100 m, however, the channel becomes confined and the slopes are steep and rugged. Sideslopes in this steep reach are dominated by bedrock cliffs, which supply episodic rockfall and colluvium to the channel. The fan apex of Skowootum Creek is located at an approximate elevation of 500 m, where it emerges from its upstream confinement.

The respective watershed and fan morphology values, which were used in the hydrogeomorphic process identification (Section 3.3.1) and hazard assessment (Section 0), are summarized in Table 2-1. Elevation and slope values for the fan were measured from the 2014 LiDAR digital elevation model (DEM) provided by MOTI to BGC, and for the watershed from the approximately 20 m resolution Canadian Digital Elevation Dataset (CDED).

Table 2-1. Watershed morphometry.

Characteristic	Unit	Skowootum Creek	Newhykulston Creek
Watershed area	km^2	11.5	14.3
Fan area	ha	21	46
Maximum watershed elevation	m	1800	1840

Minimum watershed elevation (at fan apex)	m	500	460
Watershed relief	m	1300	1380
Melton Ratio (Watershed Relief/ (Watershed Area$^{0.5}$))	Unitless	0.38	0.36
Longest continuous stream length above fan apex	km	8.7	8.4
Average channel gradient mainstem above fan apex	%	14	16
Average channel gradient on fan	%	14	7

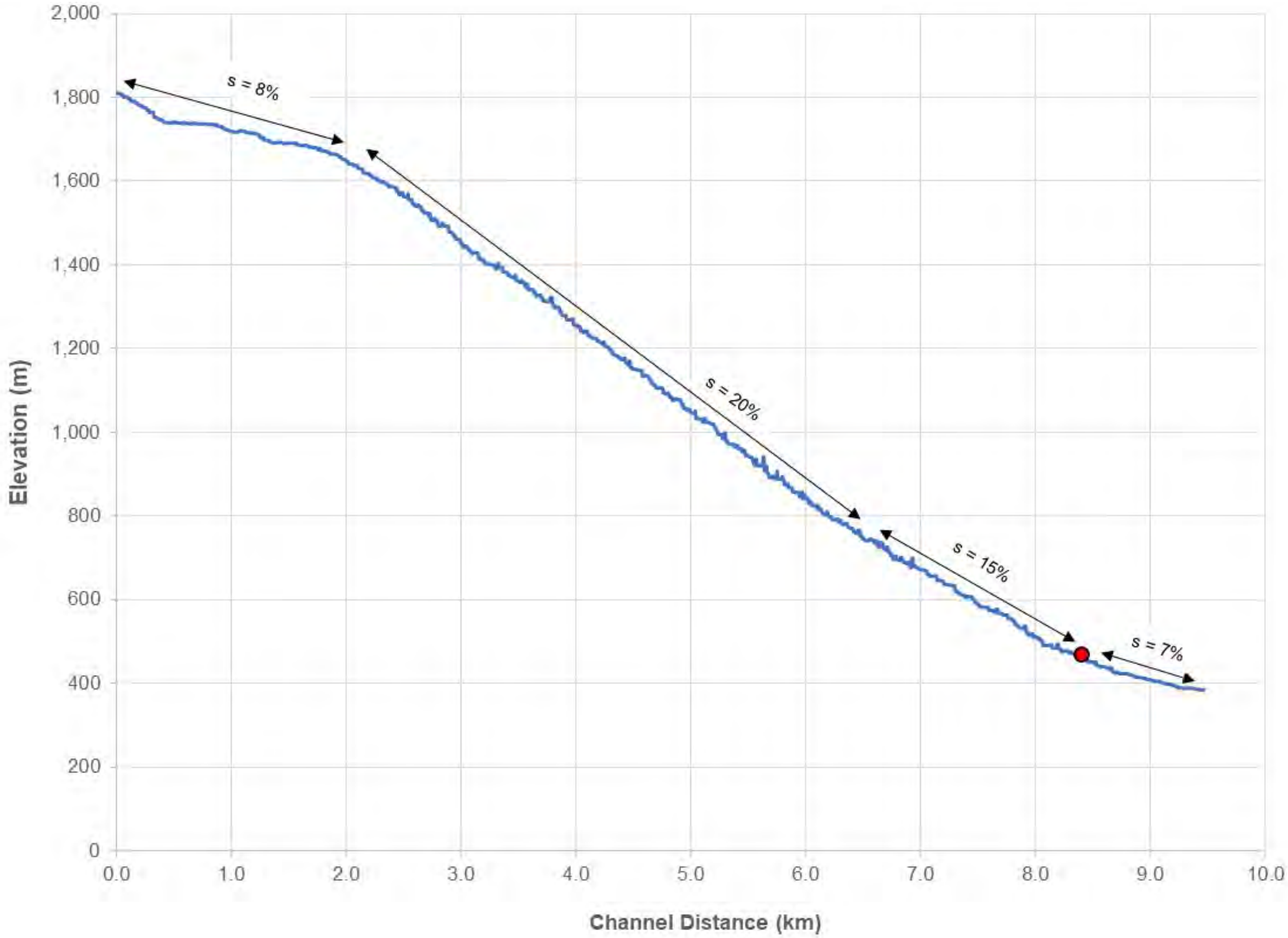

Figure 2-2. Longitudinal profile of Newhykulston Creek. The red dot delineates the fan apex. The channel profile ends at the CP Railway.

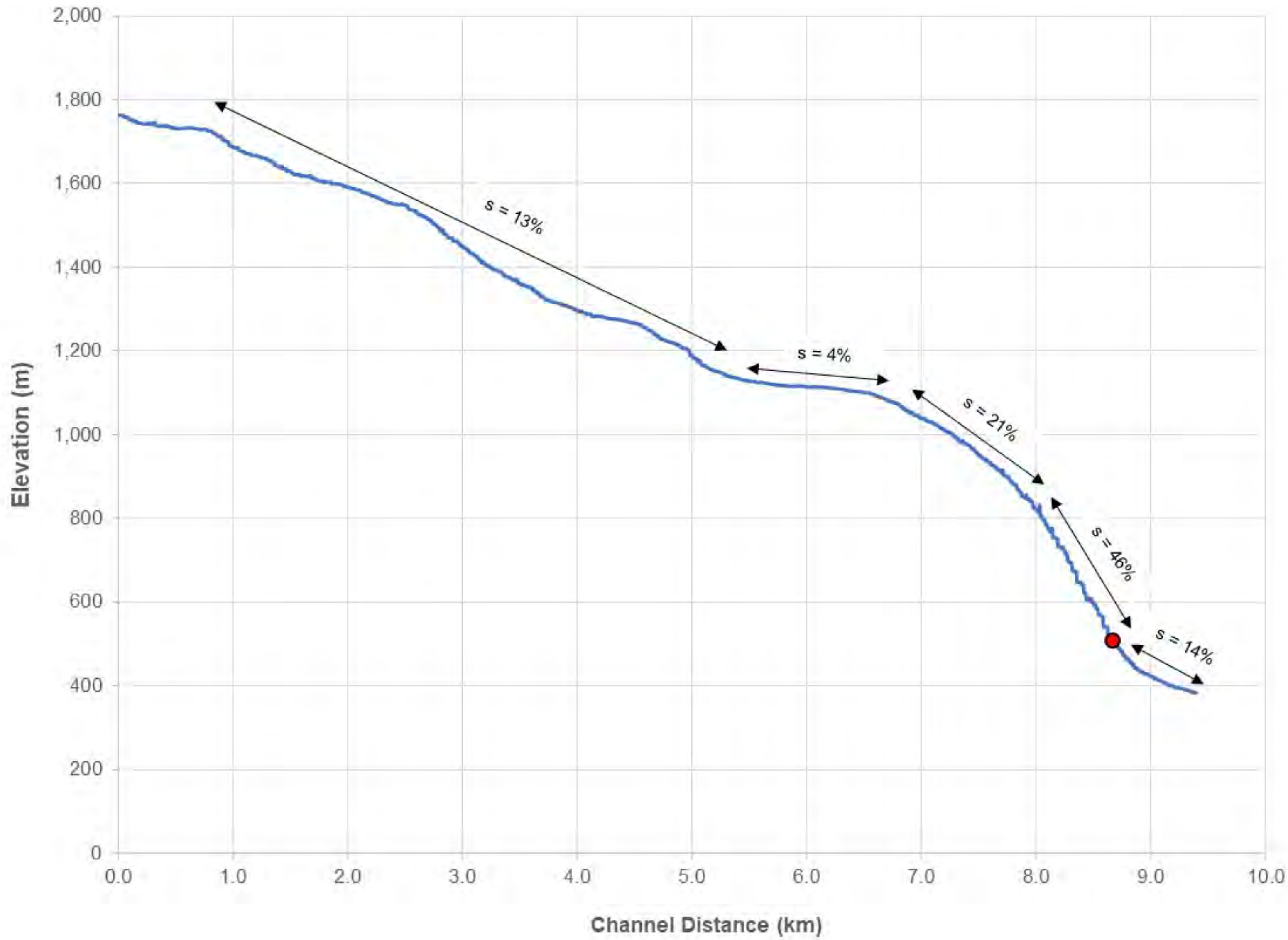

Figure 2-3. Longitudinal profile of Skowootum Creek. The red dot delineates the fan apex. The channel profile ends at the CP Railway.

Surficial material in the watersheds is dominated by till with glaciofluvial and fluvial deposits in the valley bottom flanking and underlying the North Thompson River valley. Currently the only land use in the watershed is logging. Based on a review of historical aerial photographs (Section 3.4), logging of the watersheds commenced in the 1960s. Logging in the valley bottom commenced much earlier, likely in the late 1800s to early 1900s.

M.J. Milne & Associates (Milne) conducted a watershed condition and forest development planning report for both study watersheds in 2012. For Newhykulston Creek, that report notes the following: “Runoff and peak flow levels were likely increased by past forest harvesting above the snowline (1,440 m) in Newhykulston Creek but recovery has been good in old cutblocks resulting in a current equivalent clearcut area (ECA) of 31% in this portion of the watershed. Some salvage of mountain pine beetle (MPB) affected stands has recently occurred at and below the snowline, and approximately 350 ha of MPB affected stands remain in the watershed above and below the snowline. A small increase in ECA above the snowline should be expected with the demise of pine – effects on runoff and peak flow are expected to be minor”. The Milne report also notes that forest road-related contributions to sediment loading on Newhykulston Creek are limited, although specific road deactivations are also recommended.

For Skowootum Creek, the report notes that a significant amount of logging has occurred in the upper watershed in the last 5 to 10 years for MPB salvage. Approximately 56% of the area above the snowline has been logged, resulting in an ECA of 37%. Milne (2012) notes that the MPB and associated salvage has likely increased runoff in the watershed and may be affecting peak flow levels. As a result, Milne recommended that no logging should occur in the watershed for 15 to 20 years to allow for hydrologic recovery in MPB salvage areas. However, the forest roads are noted to have been well built and maintained, resulting in low road-related sediment delivery to the creek at this time.

2.4.2. Fan Geomorphology and Development

<u>Newhykulston Creek Fan</u>

Where Newhykulston Creek debouches onto the valley bottom of the North Thompson River, an alluvial fan has developed. From its fan apex, Newhykulston Creek flows directly west for approximately 1 km before discharging onto the North Thompson River floodplain, immediately downstream of the CN Rail crossing (Drawings 02 and 03). The creek continues to flow to the west and southwest for 400 m before discharging into the North Thompson River. The average channel gradient on the fan is 7% (Figure 2-2). Fan formation is constrained by a sand and gravel paleofan to the north (Drawing 02). This paleofan likely formed at the start of deglaciation when the valley floor was either occupied by a glacial lake or a valley bottom glacier. Much of this paleofan was likely eroded immediately following retreat of the valley bottom ice/lake through incision by Newhykulston Creek and an unnamed creek to the north, as well as the North Thompson River. The approximate extents of both fans are delineated on Drawing 02.

Newhykulston Creek currently crosses Dunn Lake Road at an approximate elevation of 415 m. Here, the creek discharges through a 1.8 m diameter corrugated steel pipe (CSP) culvert (Figure 2-4). A 1 m vertical drop is located at the culvert outlet (Figure 2-5). Downstream at the railway crossing, three culverts have been installed through the rail grade (Figure 2-6). The middle box culvert is the original culvert, with newer steel culverts installed on either side likely to increase flow capacity during peak runoff events. BGC did not visit this lower crossing, so the size of the culverts or date of installation is not known.

Figure 2-4. Downstream view of 1.8 m diameter CSP culvert inlet on Newhykulston Creek at Dunn Lake Road. BGC photograph of November 28, 2018.

Figure 2-5. Upstream view of Newhykulston Creek culvert outlet at Dunn Lake Road. Note the approximate 1 m drop. BGC photograph of November 28, 2018.

Figure 2-6. Downstream view of Newhykulston Creek at the CN Rail crossing (from M.J. Milne & Associates Ltd., 2012). Channel aggradation appears to be occurring upstream of the crossing.

Downstream of Dunn Lake Road, Newhykulston Creek is confined within a 2.5 to 3 m deep channel. According to the Simpcw First Nation, this confinement was created by channel excavations conducted by CN Rail in the 1960s. Conversely, above Dunn Lake Road, the creek is generally incised less than 1 m allowing for overbank flooding and avulsion channels to form during peak flow events (Figure 2-7). This active channel zone varies in width but is typically about 20 m wide. Beyond this active zone, the fan surface is about 1 m higher in elevation. Throughout the fan, the low flow channel width is approximately 3 m.

Development on the Newhykulston Creek fan includes (Drawing 03):

- Dunn Lake Road
- Two secondary roads – Coal Creek Crescent and Neqweyqweistem Road
- CN Rail
- A water treatment plant downstream of the fan apex on the north side of the creek. According to the Simpcw First Nation, water used to be supplied to the plant from the creek directly but is now sourced from groundwater wells.
- Approximately 23 buildings including the Simpcw First Nation administrative offices and a primary school, both of which are located on the south half of the fan.

Most of the fan downslope of Dunn Lake Road is unforested – the original clearing of the fan was likely done to create land for grazing.

Figure 2-7. Downstream view of Newhykulston Creek approximately 100 m upstream of Dunn Lake Road. BGC photograph of November 28, 2018.

Skowootum Creek Fan

The Skowootum Creek fan is located 5 km to the south of Newhykulston Creek. The creek flows in a westerly direction for approximately 800 m on the south half of the fan, before discharging directly into the North Thompson River (Drawings 04 and 05). Skowootum Creek has an average gradient of 14% on the fan, but with an overall concave profile. Gradients are steepest immediately below the fan apex and decrease toward the North Thompson River (Figure 2-3). Development on the fan includes:

- Dunn Lake Road
- CN Rail
- three buildings (south of the creek)
- a sand and gravel quarry.

The quarry occupies more than half of the fan and is located to the north of the creek. At Dunn Lake Road, the creek flows through two 1.2 m diameter steel corrugated steel culverts (Figure 2-8 and Figure 2-9). During BGC's November 28, 2018 site visit, the creek was dry at the Dunn Lake Road crossing but flowing near the fan apex. Between the road and the fan apex, Skowootum Creek is typically 2 m wide.

Figure 2-8. Upstream view of Skowootum Creek at Dunn Lake Road. BGC photograph of November 28, 2018.

Figure 2-9. Downstream view of Skowootum Creek twin 1.2 m CSP culvert inlets at Dunn Lake Road. BGC photograph of November 28, 2018.

3.0 HYDROGEOMORPHIC ASSESSMENT

3.1. Introduction

Steep creeks (here-in defined as having channel gradients steeper than 5%) are typically subject to a spectrum of mass movement processes ranging from clearwater floods to debris floods to hyper-concentrated flows to debris flows, in order of increasing sediment concentration. They are referred to collectively as hydrogeomorphic[2] floods or processes because water ("hydro") and sediment ("geo") are being transported, which causes local landscape changes. Both Newhykulston Creek and Skowootum Creek are steep creeks that are subject to debris floods and debris flows, respectively. This section provides a preliminary assessment of the potential hazards associated with these processes at the two study creeks.

3.2. Definitions

A continuum prevails between hydrogeomorphic processes in space and time, with floods transitioning into debris floods upon exceedance of bed shear stress thresholds and eventually debris flows through progressive sediment entrainment in channels steeper than approximately 15° (27%). Conversely, dilution of a debris flow through partial sediment deposition on lower gradient (approximately less than <15° (27%)) channels, and tributary injection of water can lead to a transition towards hyper-concentrated flows and debris floods and eventually floods. Some steep creeks can be classified as hybrids, implying variable hydrogeomorphic processes. Creeks classified as subject to debris flows may also be subject to floods and debris floods at lower return periods, or debris flows may transition to debris floods in the lower runout zone and after the main debris surge. Those classified as subject to debris floods may be subject to clearwater floods but are only under specific circumstances subject to debris flows.

Figure 3-1 summarizes the different hydrogeomorphic processes by their appearance in plan form, velocity and sediment concentration.

[2] Hydrogeomorphology is an interdisciplinary science that focuses on the interaction and linkage of hydrologic processes with landforms or earth materials and the interaction of geomorphic processes with surface and subsurface water in temporal and spatial dimensions (Sidle & Onda, 2004).

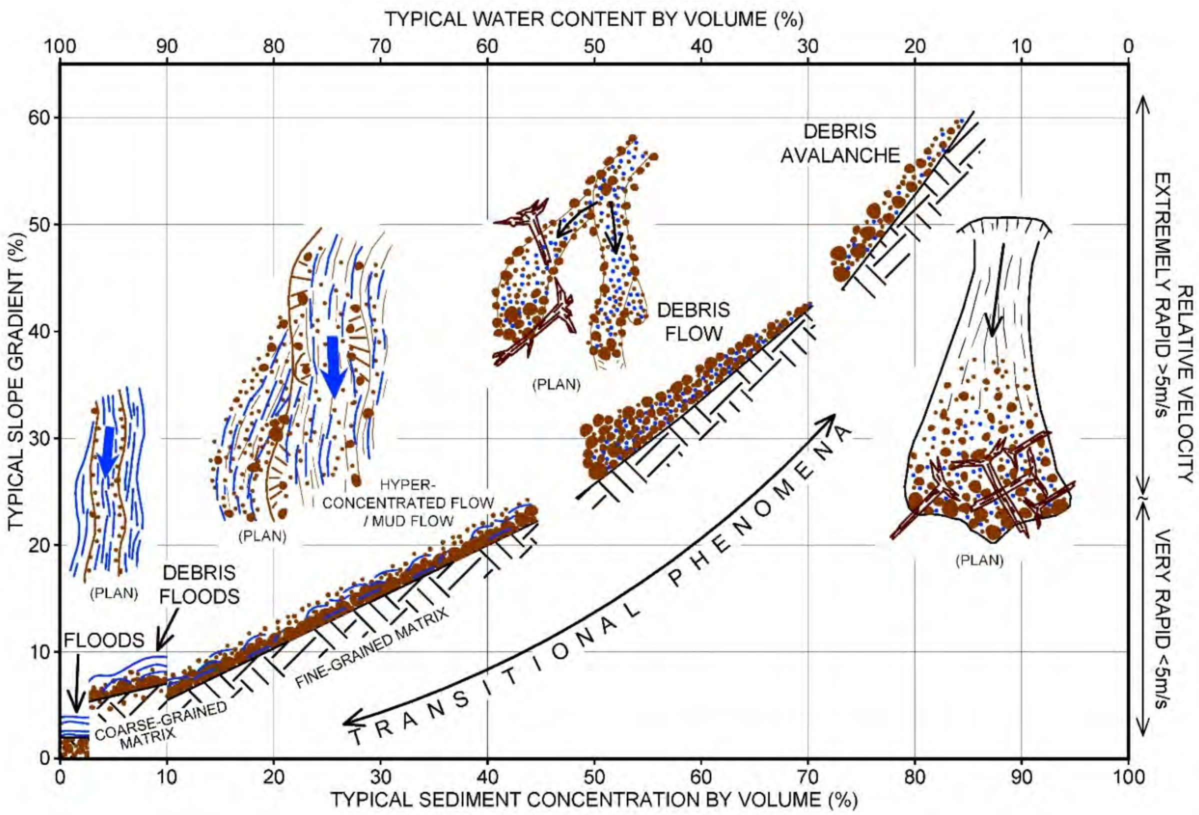

Figure 3-1. Hydrogeomorphic process classification by sediment concentration, slope velocity and planform appearance.

3.2.1. Debris Flows

'Debris flow', as defined by Hungr et al. (2014), is a very rapid, channelized flow of saturated debris containing fine grained sediment (i.e., sand and finer fractions) with a plasticity index of less than 5%. Debris flows originate from a single or distributed source area(s) from sediment mobilized by the influx of ground- or surface water. Liquefaction occurs shortly after the onset of landsliding due to turbulent mixing of water and sediment, and the slurry begins to flow downstream, 'bulking' by entraining additional water and channel debris.

Sediment bulking is the process by which rapidly flowing water entrains bed and bank materials either through erosion or preferential "plucking" until a certain sediment conveyance capacity (saturation) is reached. At this time, further sediment entrainment may still occur through bank undercutting and transitional deposition of debris, with a zero net change in sediment concentration. The volume of the flowing mass is thereby increased (bulked). Bulking may be limited to partial channel substrate mobilization of the top gravel layer, or – in the case of debris flows – may entail entrainment of the entire loose channel debris. Scour to bedrock in the transport zone is expected in the latter case.

Typical debris flows require a channel gradient of at least 27% (15º) for transport over significant distances (Takahashi, 1991) and have volumetric sediment concentrations in excess of 50%.

Between the main surges a fluid slurry with a hyperconcentration (>10%) of suspended fines occurs. Transport is possible at gradients as low as 20% (11^{o}), although some type of momentum transfer from side-slope landslides is needed to sustain flow on those slopes.

Due to their high flow velocities, peak discharges during debris flows are at least an order of magnitude larger than those of comparable return period floods and can be upwards of 50 to 100 times larger (Jakob & Jordan, 2001; Jakob, Clague, & Church, 2016). Further, the large caliber of transported sediment and wood means that debris flows are highly destructive along their channels and on fans.

3.2.2. Debris Floods

A 'debris flood' is "a very rapid surging flow of water heavily charged with debris in a steep channel" (Hungr et al., 2014). Transitions from clearwater floods to debris floods occur at minimum volumetric sediment concentrations of 3 to 10%, the exact value depending on the particle size distribution of the entrained sediment[3]. Because debris floods are characterized by heavy bedload transport, rather than by a more homogenous mixture of suspended sediments typical of hyperconcentrated flows (Pierson, 2005), the exact definition of sediment concentration depends on how sediment is transported in the water column. Debris floods typically occur on creeks with channel gradients between 5 and 30%. More formally, BGC defines debris flood onset when at least the grain size corresponding to the D_{84} (the 84th percentile of all bedload grain sizes) is mobilized. When this occurs, most of the stream bed becomes mobile, and the mobile layer is a few D_{84} grains thick (Mackenzie, Eaton & Church, 2018). Debris floods can occur due to exceptionally high discharge associated with rain or rain-on-snow events, but also due to debris flow dilution, or as a consequence of outbreaks of natural or man-made dams.

The term "debris flood" is similar to the term "hyperconcentrated flow", defined by Pierson (2005) on the basis of sediment concentration as "a type of two-phase, non-Newtonian flow of sediment and water that operates between normal streamflow (water flow) and debris flow (or mudflow)". Debris floods (as defined by Hungr et al., 2014) have lower sediment concentrations than hyperconcentrated flows (as defined by Pierson). Thus, there is a continuum of geomorphic events that progress from floods to debris floods to hyperconcentrated flows to debris flows, as volumetric sediment concentrations increase.

3.3. Process Identification

3.3.1. Statistical

Steep creek processes can be differentiated based on physical measurements of the watershed. Figure 3-2 contextualizes the span of processes based on a comprehensive analysis of the

[3] The yield strength is the internal resistance of the sediment mixture to shear stress deformation; it is the result of friction between grains and cohesion (Pierson, 2005).

stream length and Melton[4] ratio of over 700 watersheds in Alberta prone to floods, debris floods or debris flows. These terrain factors are a good screening level indicator of the propensity of a creek to dominantly produce floods, debris floods or debris flow (Holm et al., 2016).

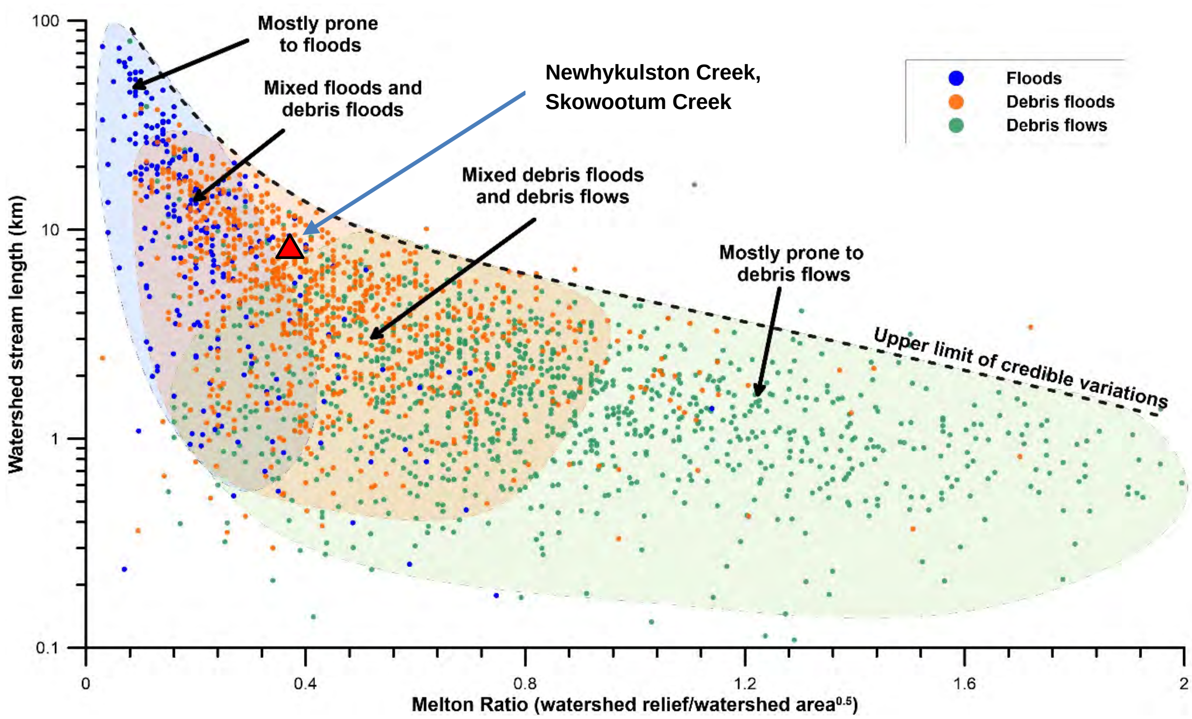

Figure 3-2. Steep creek processes as a function of Melton ratio and stream length. Data points and process boundaries are derived from 2,258 steep creeks in mountainous regions of Alberta and British Columbia (Holm et al., 2016; Lau, 2017; BGC data).

Although there is overlap, creeks with the highest Melton ratio and shortest watershed stream length are mostly prone to debris flows, and those with the lowest Melton ratio and longest watershed stream lengths are mostly prone to floods. Debris floods fall between these types.

As shown in Figure 3-2, both study creeks plot in the spectrum of debris floods. As terrain factors are only a screening level indicator of the propensity of a creek to produce a hydrogeomorphic event, both creeks were visited in the field to look for evidence of past debris flood or debris flow behaviour.

[4] Melton (1957) ratio – The ruggedness of the basin can be characterized by the dimensionless ratio, $H/\sqrt{A}$ (H is the watershed relief in km, and A is watershed area in km^2).

3.3.2. Field Evidence

Table 3-1 provides a detailed list of geomorphic characteristics that can be used to distinguish between debris flow, debris flood, and flood process types. Creeks classified as subject to debris flows may also be subject to floods and debris floods at lower return periods, or debris flows may transition to watery afterflow in the lower runout zone and after the main debris surge. Those classified as subject to debris floods may be subject to clearwater floods but will generally not be subject to debris flows.

Table 3-1. Sediment and geomorphic characteristics for different steep creek processes.

Sediment or Geomorphic Characteristic	Debris Flows	Debris Floods	Clearwater Floods
Matrix-supported deposit stratigraphy	Yes	Rarely	No
Clast-supported deposit stratigraphy	Rarely	Often	Yes
Inverse grading of deposit	Yes	No	No
Clast imbrication	No	Sometimes	Usually
Defined boulder lobes	Yes	Sometimes, but with less sharp boundaries than for debris flows	No
Boulder levées	Yes	No	No
Terraces on both sides of the channel at the same elevation (“paired terraces”)	Rarely	Often	Only if stream is incising into alluvial bed
Buried vegetation	Yes	Yes	Sometimes
Impact-scarred riparian vegetation	Yes	Often	Rarely
Creek channel scour	Mostly in transport zone	Yes	Yes
Fine-grained overbank deposits	Rarely	Sometimes	Usually
Channel gradient (watershed)	Typically >30%	Typically <30%	Typically <30%
Channel gradient (fan)	> 9% (>5^{o})	5 to 9% (3 to 5^{o})	< 5% (< 3^{o})

<u>Newhykulston Creek</u>

The following field evidence indicates that Newhykulston Creek is prone to debris floods:

- The channel gradient in the confined reach between elevation 460 m and 700 m is not steep enough at 15% to sustain debris flow transport nor are there any steep tributaries that appear prone to debris flows.
- The average fan channel gradient (7%) falls within the expected range for debris floods.
- The fan deposits are clast-supported.

- Buried tree bases were observed within the active channel zone upstream of Dunn Lake Road (Figure 3-3).

Figure 3-3. Buried conifer tree bases on the right bank of Newhykulston Creek approximately 50 m above Dunn Lake Road (site WP 1, Drawing 02). BGC photograph of November 28, 2018.

Debris floods on Newhykulston Creek most likely occur during extreme floods (>50-year return period) when a critical shear stress is reached that mobilizes the D_{84} grain sizes. When this occurs, most of the stream bed (several grains deep) becomes mobile (Mackenzie, Eaton & Church, 2018). While sediment supply to the creek is not unlimited, there are several slope instabilities between the fan apex and an elevation of 700 m that provide a continuous source of sediment to the creek. One such slope is located on the left bank of the creek below the fan apex (site WP 6, Drawing 03). Here the creek is eroding a 30 m high slope composed of dense till (silt and clay matrix with coarser clasts) (Figure 3-4).

Figure 3-4. Unstable valley slope on south side of Newhykulston Creek (site WP 6, Drawing 03). BGC photograph of November 28, 2018.

<u>Skowootum Creek</u>

While the watershed morphometrics of Skowootum Creek place it within the debris flood spectrum, its morphometry statistics are misleading. The reason is that the creek has a hanging valley morphology, where channel gradients are generally less than 15% in mid to upper reaches of the watershed (Figure 2-3). However, below elevation 1100 m, the creek steepens considerably on its descent to the valley bottom. Between elevations 820 m and 500 m, the average channel gradient is 46%, which is more than sufficiently steep for debris flow transport. Fan gradients (average of 14%) and field evidence of boulder lobes are also indicative of debris flow processes.

While Skowootum Creek is prone to debris flows, a mixed population of debris flows and debris floods (or hyperconcentrated flows) is highly likely. The reason is that the steep confined reach includes large rockfall boulders greater than 1 m in diameter (Figure 3-5, Figure 3-6). These angular boulders are likely stabilizing structures that prevent full mobilization of the bed except during extreme return period events (i.e., return periods estimated as greater than several hundred years). BGC did not observe any boulders of this size on the surface of the fan or in the exposed cuts of the quarry. This observation suggests that when large debris flow events do occur, not all sediment in the channel is mobilized.

Figure 3-5. Upstream view of Skowootum Creek at site WP 17 (Drawing 03). BGC photograph of November 28, 2018.

Figure 3-6. Upstream view of Skowootum Creek at site WP 18 (Drawing 03). BGC photograph of November 28, 2018.

3.4. Frequency-Magnitude

Frequency-magnitude (F-M) relations answer the question “how often and how big can steep creek hazard events become?”. The ultimate objective of an F-M analysis is to develop a graph that relates the return period of the hazard to its magnitude. For all watersheds, the event magnitude levels off at some point because of either sediment supply or water limitations. This means that debris flows and debris floods from a given watershed have a maximum possible sediment volume and peak discharge.

The previous sections have demonstrated that Newhykulston Creek is prone to debris floods, while debris flows can occur on Skowootum Creek. In this section, the approximate magnitudes of both processes are estimated within the design return period of 100 years. The intent is not to provide detailed F-M relationships for the two creeks, but rather to determine how potential future hydrogeomorphic events might influence or impact the recommended culvert replacement option.

3.4.1. Newhykulston Creek

3.4.1.1. Debris Flood Frequency

The frequency of past debris floods on Newhykulston Creek was investigated using two different methods: historical accounts and an analysis of historic air photographs.

<u>Historical Accounts</u>

During the site visit of November 28, 2019, BGC and MOTI met with members of the Simpcw First Nation. In their recollection (dating back to the 1960s), they could not recall any flood events on Newhykulston Creek that resulted in an avulsion on the fan or blockage of the culvert at Dunn Lake Road. MOTI personnel could also not recollect any problems at the road crossing, although systematic government records do not exist for the road.

The Simpcw First Nation did note that historically the creek used to flow on the south half of the fan, before CP Rail straightened the creek so that it flowed due west through the centre of the fan. This observation is consistent with a July 5, 1877 survey plan of the area, which shows the creek flowing to the southwest on the fan (Figure 3-7). Simpcw members also noted that CP Rail used to regularly excavate sediment out of the creek until about the mid-1960s.

A 2012 report by M.J. Milne & Associates noted that a debris flood occurred on Newhykulston Creek in the late 1990s, but no other details were provided in the report.

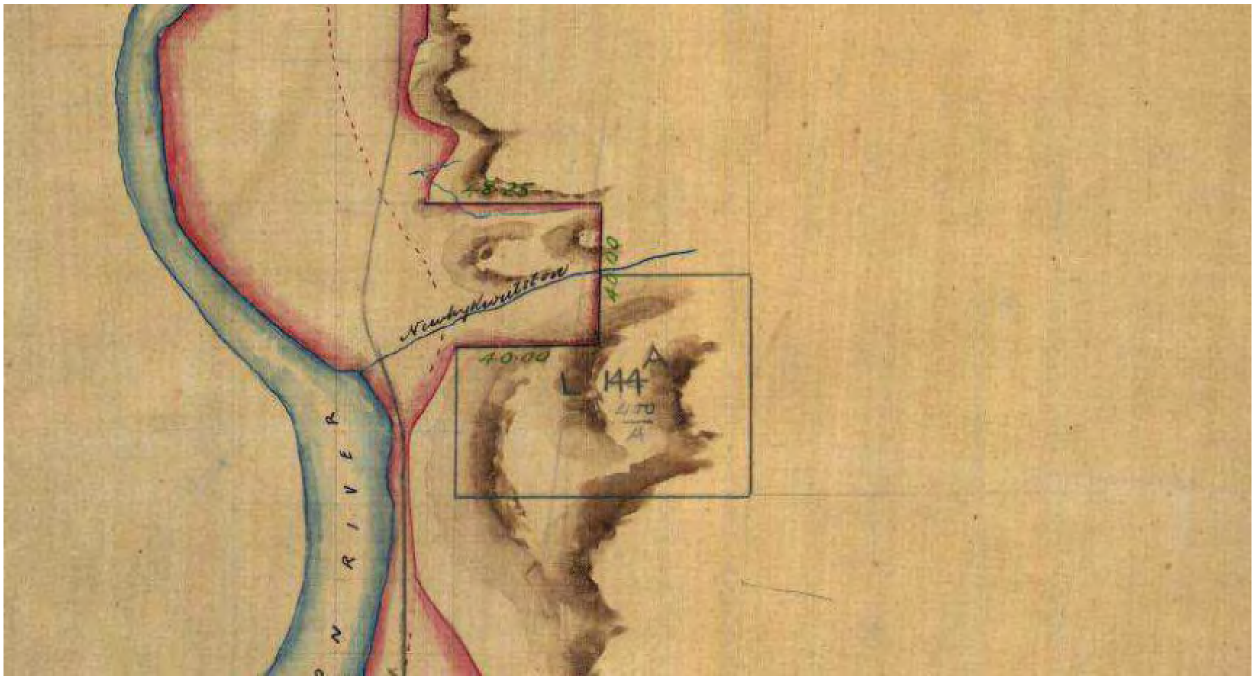

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Figure 3-7. July 5, 1877 survey plan of Newhykulston Creek (survey accessed from ParcelMap BC web site).

Air Photographs

Air photographs dating back to 1947 were also reviewed by BGC. A summary of observations from that analysis is provided in Table 3-2. The air photos show a lot of geomorphic activity early in the photographic record (late 1940's to late 1960's) followed by a half-century period of stabilization. The 1947 air photographs[6] show a major avulsion on the southwest portion of the fan. It is presumed that the extent of overbank flooding seen on these air photographs represents a major debris flood event that caused avulsion channels to develop upstream of Dunn Lake Road. As there is no obvious source for the hydrogeomorphic activity, the debris flood was likely the result of an extreme flood sufficiently large to mobilize the entire bed surface layer. An active channel and avulsion channels are also visible on the 1952, 1965, and 1967 air photographs. Since 1967, none of the air photographs show evidence of hydrogeomorphic activity.

Summary

The available evidence indicates that debris floods occur on Newhykulston Creek. However, the historical frequency of damaging events (i.e., major channel avulsions) appears to be on the order of 50 to 100 years.

[5] Lateral displacement of a creek from its main channel into a new course across its fan.
[6] Unfortunately, the scale of the 1947 air photographs is too large to show a meaningful visual representation of the magnitude of the channel avulsion.

Table 3-2. Observations from air photograph interpretation, Newhykulston Creek 1947-2011.

Series	Numbers	Year	Date	Scale	Observations
X4	27-26	1947	Jul-14		Dunn Lake Road constructed; no buildings on the fan; main channel runs due west, but large avulsion to the south; about two-thirds of the fan area cleared for grazing; these fields are covered in gravel sheets; avulsion channel starts about 100 m upstream of Dunn Lake Road; no obvious source for presumed debris flood activity.
BC1502	2-4	1952	Jun-8		Same extent of disturbance; additional avulsion to the north of the main channel; a few buildings constructed to the southwest of road crossing
BC4306	149-152	1965	Jul-3	1:15,840	Main channel looks like it has been recently excavated; avulsion channel starts about 100-150 m above Dunn Lake Road; two small avulsion channels apparent - one trending south and one trending southwest; neither avulsion appears that significant.
BC4343	111-113	1965	Aug-31	1:15,840	No change.
BC5255	8-7	1967	Jul-10	1:32,000	Minor avulsion activity to north of main channel; avulsion starts above Dunn Lake Road.
BC7757	256-257	1975	Jul-23	1:20,000	Fan has stabilized since 1967; ~ 16 buildings on fan now including the band office and new buildings above Dunn Lake Road and to the north of the main channel.
BC81013	37-35	1981	May-28	1:40,000	No obvious change.
30BCC880	96-93	1988	Aug-24	1:15,000	Continued stabilization pattern; small subdivision on gravel terrace to north of fan
30BCC95046	98-96	1995	Sep-2	1:15,000	Continued stabilization; a few more buildings.
30BCC00055	208-205	2000	Aug-17	1:15,000	No change.
15BCC04054	56-57	2004	Jul-16	1:30,000	No change.

3.4.1.2. Debris Flood Magnitude

Prediction of bedload transport can be important for hazard assessments and engineering applications although knowledge on sediment transport is still limited, particularly from a modelling perspective (see Church, 2013, for tables of data on steep creek sediment transport). Furthermore, few sediment transport studies have been completed for steep (> 5%) mountain creeks, and as noted by Hassan *et al.* (2005), sediment transport in such channels may be quite different from low-gradient channels. However, BGC has developed empirical relations between rainfall/runoff volume and sediment volume based on a dataset from Switzerland. BGC has successfully applied these relations to watersheds in both Alberta and British Columbia.

During August 21-23, 2005 severe flooding occurred in a large area of northern Switzerland with significant morphological changes in stream channels (Jäggi, 2007). This flood event was associated with more than 200 mm of rain within three days with corresponding return periods exceeding 100 years. As many mountain creek hazards in Switzerland have been mitigated by catchment basins, the sediment volumes could be determined.

A database was subsequently created with 33 debris-flows and 39 fluvial sediment transport events, details of which are reported in Rickenmann and Koschni (2010). These authors used a variety of transport movement equations to compare modeled and predicted sediment transport volumes. Rickenmann and Koschni (2010) found reasonable agreements between modelled and measured sediment volumes for channels with less than 5% gradient using the Meyer-Peter and Mueller equations. In contrast, for steeper channels, the observed sediment volumes transported by fluvial processes are over-predicted by bedload equations developed for steep channels.

Given the value of the Rickenmann and Koschni (2010) database, BGC analyzed the data further. First, BGC separated the debris-flow events from the mostly fluvial transport (i.e., debris flood) data. Watersheds with very large areas and correspondingly low gradients (< 1%) were also deleted from the dataset. These deletions provided a final dataset of 32 cases. Multivariate regression analysis was then applied to the log-transformed dataset to determine sediment volumes based on catchment area, rainfall volume, runoff volume, and channel gradient. This analysis yielded the following relation:

$$logV_S = 0.755\ logV_R - 0.406,\ R^2 = 0.81 \qquad \text{[Eq. 3-1]}$$

where V_S is the total sediment volume (m^3) displaced and V_R is the total runoff volume (m^3). The formula presented above is appropriate for debris floods with channel gradients from approximately 2 to 24%.

Using Equation 3-1 and a runoff volume of 210,000 m^3 (see Section 4.3.3), it is estimated that a 100-year flood on Newhykulston Creek could transport approximately 4,100 m^3 of sediment. In BGC's experience, debris flood hydrographs typically have volumetric sediment concentration of between 3% and 5%. As a secondary check, BGC also applied a volumetric sediment concentration of 3% to 5% to the simulated 100-year storm hydrograph (see Section 4.3.3). That application results in a sediment volume range of 3,300 m^3 to 5,400 m^3, which is consistent with the empirical relation derived from the Swiss dataset.

3.4.2. Skowootum Creek

3.4.2.1. Debris Flow Frequency

Similar to Newhykulston Creek, the frequency of past debris flows on Skowootum Creek was investigated using two different methods: historical accounts and an analysis of historic air photographs.

Historical Accounts

MOTI (Sean Clough, pers. comm) noted to BGC that a minor debris flow event occurred on Skowootum Creek in April 2012, in response to a rain-on-snow event. No official records exist of this event, but the crossing was damaged, and flooding occurred on the property to the immediate to the south of the creek below Dunn Lake Road. Mr. Clough also noted that a small debris flow event may have occurred in 2005 or 2006; this event did not damage the crossing, but the road suffered some damage due to an avulsion.

Members of the Simpcw First Nation recalled the 2012 event but were not aware of any other flood/hydrogeomorphic events on Skowootum Creek.

Air Photographs

BGC reviewed air photographs dating back to 1947. A summary of observations from that analysis is provided in Table 3-3. Dunn Lake Road is visible on the earlier air photographs (1947) but no evidence of hydrogeomorphic activity is apparent on any of the imagery.

Summary

A debris flow event of a magnitude[7] to be visible on satellite imagery or air photographs has not occurred on Skowootum Creek in more than 70 years. This observation is consistent with site observations, where the steep confined reach above the fan apex includes large rockfall boulders greater than 1 m in diameter (Section 3.3.2). These angular boulders are interpreted to be stabilizing structures that prevent full mobilization of the bed except during extreme return period events (i.e., return periods greater than several hundred years). However, smaller events capable of damaging the road and crossing do occur, likely on the order of a couple to several decades.

[7] It is estimated that a debris flow volume in excess of several thousand m^3 would likely be visible on air photographs.

Table 3-3. Observations from air photograph interpretation, Skowootum Creek 1947-2011.

Series	Numbers	Year	Date	Scale	Observations
BC355	110-111	1947	Jul-14		No development on fan or logging in watershed; Dunn Lake Road and railway visible, but no houses on fan; older avulsion channel on north part of fan(?)
BC1502	5-7	1952	Jun-8		No change. No debris flow activity.
BC4343	209-211, 217-219	1965	Aug-31	1:15,840	No change.
BC5255	5-6	1967	Jul-10	1:32,000	No change.
BC7757	252-253	1975	Jul-23	1:20,000	Road accessing upper watershed with some clear cut logging; selective logging on fan (roads visible); house below Dunn Lake Road but not above.
BC81013	234-235	1981	May-28	1:40,000	Very small quarry; recent logging in upper watershed; no debris flow activity.
30BCC875	205-208, 109-111	1988	Aug-24	1:15,000	Nascent development of quarry; no debris flow activity; logging in upper watershed.
30BCC95022	48-50	1995	Jun-30	1:15,000	Increased quarry development.
30BCC00055	106, 104-103	2000	Aug-17	1:15,000	No obvious instabilities; quarry fully developed as well as house below road; logging in upper watershed.
15BCC04030	75-76	2004	Jul-23	1:30,000	No change.

3.4.2.2. Debris Flow Magnitude

As noted in the previous section, large debris flows on Skowootum Creek are estimated to have a return period greater than several hundred years. Sediment volumes associated with such rare events can be estimated by summing yield rates along the channel. That is, how much sediment could a debris flow entrain along its channel on its descent. This variable is referred to as entrainment or yield rate and has units of m^3/m (Hungr, McDougall, and Bovis, 2005).

Based on the channel profile (Figure 2-3), a debris flow on Skowootum Creek could travel a maximum distance of about 1 km. While BGC's traverse of Skowootum Creek was limited, a yield rate of 12 m^3/m appears reasonable and is within the range of those observed elsewhere (Hungr et al., 2005). Assuming a point source (debris avalanche/debris slide/rock fall) volume of 3,000 m^3 (i.e., a landslide event that initiates a debris flow), the resulting debris flow volume range is estimated at approximately 15,000 m^3. Without much more detailed investigations (dendrochronology and test trenching with organic material dating, and a traverse of the entire lower 1 km of the channel), it is not possible to provide a more reliable estimate.

The approximate maximum debris flow discharge for a 15,000 m^3 event was then estimated based on the peak discharge (Q) to sediment volume (V) relationship for coarse-grained debris flows developed by Bovis and Jakob (1999), as shown in Equation 3-2.

$$V = 28 * Q^{1.11} \qquad \text{[Eq. 3-2]}$$

Application of this equation yields a peak discharge of approximately 480 m^3/s.

The volume and peak discharge of smaller events, such as occurred in 2012 and possibly 2005 or 2006, are difficult to estimate, as high-water marks and debris have been obscured to allow reasonable magnitude estimates. However, such events are of sufficient magnitude to damage Dunn Lake Road.

3.5. Design Implications

This chapter has assessed the hydrogeomorphic processes that occur on both study creeks and provided approximate magnitudes of such events. The design implications of these processes are discussed below.

3.5.1. Newhykulston Creek

Field observations and historical air photographs indicate that debris floods occur on Newhykulston Creek. However, the frequency of damaging events (i.e., channel avulsions) appears to be on the order of 50 to 100 years. For a 100-year flood event, it is estimated that 4,000 to 5,000 m^3 of sediment could be deposited on the fan. A debris flood event of this magnitude could result in channel aggradation above Dunn Lake Road and cause an avulsion on either bank. From a design perspective, it is important that the culvert replacement option at Dunn Lake Road not compromise the sediment transport capacity of the creek. This can be accomplished by ensuring that the culvert/bridge is sufficiently large for the design event and that the channel slope through the crossing is similar to upstream and downstream reaches (i.e., not induce a backwater effect at the crossing).

Because the volumetric sediment concentration of debris floods on Newhykulston Creek are expected to be less than 5%, no adjustment is recommended to the 100-year return period peak instantaneous flow estimate discussed in the next chapter, particularly as climate change is accounted for in the peak flow estimates.

It is important to note that irrespective of the preferred culvert replacement option, the design event could result in an upstream avulsion due to sudden channel aggradation. In this case, the avulsion channel could cause damage to Dunn Lake Road and result in flood damage to buildings on the fan. Economic consequences would be greater if the avulsion happened on the left (south) bank where there is a higher density of buildings, including the band office and local school. Based on the creek's past behaviour, an avulsion to the south is considered more likely.

3.5.2. Skowootum Creek

Unlike Newhykulston Creek, Skowootum Creek is prone to debris flows, which tend to be much more damaging than debris floods due to their higher peak discharge. Based on the historical air photograph record, a large debris flow event (> several thousand m^3) has not occurred on Skowootum Creek in more than 70 years. Such events are estimated to have a return period greater than several hundred years with an associated sediment volume of approximately 15,000 m^3 and a peak discharge of 480 m^3/s. These numbers are considered preliminary values only. Without a detailed investigation, it is not possible to provide a more reliable estimate. However, even if a third of this estimated volume was mobilized, the estimated peak flow still exceeds 100 m^3/s. Furthermore, smaller debris flow events capable of damaging the road and crossing also occur, likely on the order of a couple to several decades.

Given the potential size of future debris flows and the steep fan topography, debris-flow mitigation would be costly (>$1 million). Because of the low volume use of Dunn Lake Road and general lack of development on the fan, mitigation is not a recommended option from a cost-benefit perspective. Rather, the crossing should be designed for clearwater floods only while acknowledging that maintenance will be required following future debris flow events. This recommendation is based on economic consequences only and does not consider the potential for loss of life, which is outside the scope of work for this study.

3.5.3. Post-Fire Response

It should be noted that the estimated magnitudes of hydrogeomorphic events on the study creeks represents existing conditions. However, this area is highly susceptible to forest fires, as demonstrated by the 2003 Barriere wildfire, which was devasting to the populated valley floor around Barriere. A high intensity wildfire in either of the two study watersheds is likely to increase the frequency of damaging debris floods on Newhykulston Creek and debris flows on Skowootum Creek.

4.0 HYDROLOGICAL ANALYSIS

4.1. Introduction

As noted in Section 1.3, the design flow rates for the two crossings will either be the 100-year peak instantaneous flow (Q_{i100}) or the 200-year maximum daily flow (Q_{200}), whichever is the higher of the two. Extreme peak flows for ungauged watercourses can either be estimated by regional flood frequency analysis (RFFA) or by rainfall-runoff modelling, as described below.

4.2. Regional Flood Frequency Analysis

The current standard practice in British Columbia for RFFA's is to identify nearby hydrometric stations that have a common hydrologic regime to the ungauged watershed being studied. Individual flood frequency analyses are conducted on the gauges and a logarithmic relation developed where peak flows for a given return period are regressed against drainage area. While this methodology appears to work well for medium to large watersheds, BGC has repeatedly experienced that this approach under-estimates extreme peak flows in smaller watershed (i.e., drainage area < 100 km^2), particularly those prone to hydrogeomorphic events. Sicamous Creek is a case study that illustrates this point.

Sicamous Creek is located on the east side of Mara Lake to the immediate south of Sicamous, BC. The creek drains a watershed area of 65 km^2 and is ungauged. This watershed has been the focus of litigation the past two years due to a debris flood that occurred between June 23 and June 25, 2012, which caused major (millions of dollars) damage to existing developments on the alluvial fan where Sicamous Creek enters Mara Lake. Much of this damage was the result of bank erosion and substantial sediment transport and deposition causing channel avulsions. While the event was categorized as a debris flood, the peak flow associated with the storm was not significantly bulked by the sediment transport, which was likely no more than 2 or 3% volumetric concentration (BGC, November 30, 2017).

Immediately following the event, Golder (July 30, 2012) estimated that the Q_{i200} for Sicamous Creek was 31 m^3/s, based on regional analysis. However, this peak flow estimate was subsequently found to be inconsistent with observations of water levels at the peak of the event (BGC, November 30, 2017). Subsequent detailed hydrologic simulation using the Raven platform[8] estimated the peak flow of the 2012 event at approximately 68 m^3/s, while a longer-term simulation for the period 1997-2012 indicated that the 2012 event was: a) not unusually high and b) the Q_2 was in the range of 25 to 30 m^3/s (WaterSmith, November 30, 2017).

Of further interest is that the 2012 peak flow event was initiated by a convective cell within a larger synoptic storm system that was isolated to the Sicamous Creek watershed. No climate stations in the region recorded the magnitude of this storm event, as measured in the Sicamous Creek watershed using radar data from Silver Star Mountain.

[8] http://raven.uwaterloo.ca/

Both examples highlight the potential issues arising from the use of traditional RFFA practices in mountainous terrain:

- steep creek hydrology is poorly characterized
- observation periods typically do not extend more than two or three decades, as such hydrometric stations in smaller watersheds may not experience an extreme event in the observation period
- hydrometric station density is typically quite low throughout BC.

In light of these potential issues in smaller mountainous catchments, BGC decided to employ a rainfall-runoff model for Newhykulston Creek and Skowootum Creek.

4.3. Rainfall-Runoff Model

Rainfall-runoff analysis uses physical watershed characteristics and a design storm to calculate peak flows. For this study, the U.S. Soil Conservation Service (SCS) unit hydrograph method was implemented using the hydrologic model HEC-HMS (Version 4.2.1) developed by the U.S. Army Corps of Engineers. Required inputs to the model include:

- The design storm event hyetograph (rainfall distribution).
- Sub-basin or basin catchment areas.
- Curve numbers (CN) based on the soils, plant cover and impervious areas within the watershed, which determines the effective rainfall (runoff) of the design storm event (USDA, 1986). As with the rational method, the choice of the CN value (or runoff coefficient) has a large impact on the final discharge value especially in a small catchment area.
- Initial abstraction (I_a), which represents the depth of rainfall that is infiltrated and intercepted before runoff begins.
- Lag time, which is the time between the centroid of the rainfall amount and the centroid of the outflow hydrograph.

4.3.1. Design Hyetograph

The closest climate station with rainfall intensity-duration-frequency (IDF) data to the project site is ECCC's *Kamloops A* (#1163780), which is located approximately 70 km the south. The 100 year 24-hour rainfall depth from this station is estimated to be 49 mm, based on 38 years of historical data (1965 to 2002). The closest ECCC station with daily precipitation data is *Darfield* (#1162265) with a 63-year precipitation record (1956 to 2018). Precipitation and rainfall records are available at a daily timestep for this station, but no IDF data have been published by ECCC.

BGC estimated daily rainfall depths for various return periods at the *Darfield* station using frequency analysis and a scaling factor of 1.13 to adjust daily recorded values to 24-hour estimates (as per World Meteorological Organization, 2009). A comparison of the 24-hour rainfall depths for both stations is provided in Table 4-1, while Figure 4-1 provides a comparison of the annual maximum 24-hour rainfall.

Table 4-1. 24-hour rainfall depths

Return Period (years)	ECCC[1]	BGC[2]
	Kamloops (mm)	Darfield (mm)
2	20	25
5	28	33
10	33	39
25	40	49
50	44	56
100	49	64
200	Not Available	73

Notes:

1) Gumbel distribution used by ECCC to develop IDF curve
2) Generalized Extreme Value (GEV) distribution used for frequency analysis

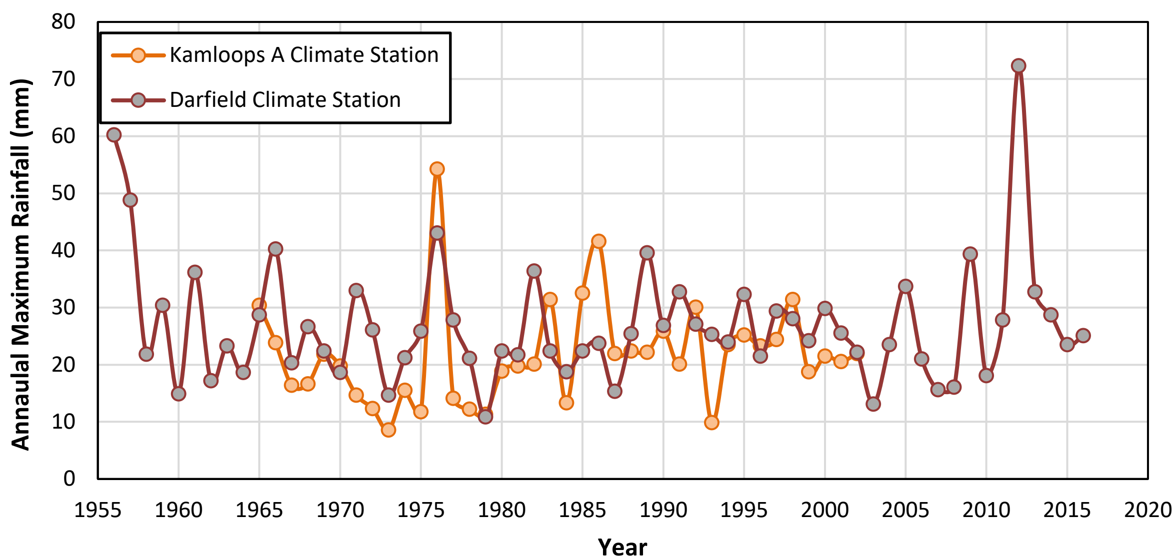

Figure 4-1. Annual maximum 24-hour rainfall at Darfield and Kamloops A climate stations.

Due to the proximity of the *Darfield* station to site and its significantly longer rainfall record, a 100-year 24-hour rainfall depth of 64 mm was chosen for modelling of existing conditions. As the rainfall data for the *Darfield* station is only available at a daily interval, the temporal distribution of the 100-year storm was adopted from *Kamloops A* climate station which has published IDF curves with a temporal breakdown to the 5-minute interval. As recommended in MOTI (2007) for areas where critical runoff values are the result of summer storms (e.g., interior areas), Antecedent Moisture Conditions (AMC) II were assumed for the 100-year return period storm.

4.3.2. Watershed Characteristics

Watershed parameters for both creeks were delineated based on Geobase CDED data at a pixel resolution of 23 m. The required parameters for hydrological analysis using the SCS method are summarized in Table 4-2.

Table 4-2. Hydrological parameters of the watersheds.

Parameters	Skowootum Creek	Newhykulston Creek
Watershed Area (km^2)	11.5	14.3
Average Gradient (%)	18	27
Longest Hydraulic Path (km)	9.4	10.1
Composite SCS Curve Number (CN II)[1]	73	73
Initial Abstraction (mm)	19	19
Lag time (min)[2]	85	74

1) AMC II
2) SCS Lag Formula

4.3.3. Model Results

HEC-HMS model results are shown in Table 4-3. The Q_{i100} peak flow estimates prevail, which is expected given the relatively small size of both watersheds and lag times that are considerably less than 24 hours. The 100-year return period storm hydrographs are also shown in Figure 4-2.

Table 4-3. HEC-HMS modelling results.

Parameters	Skowootum Creek	Newhykulston Creek
100-year, 24-hr Peak Instantaneous Flows (m^3/s)	9.5	12.6
200-year, 24-hr Maximum Daily Flows (m^3/s)	3.9	4.9

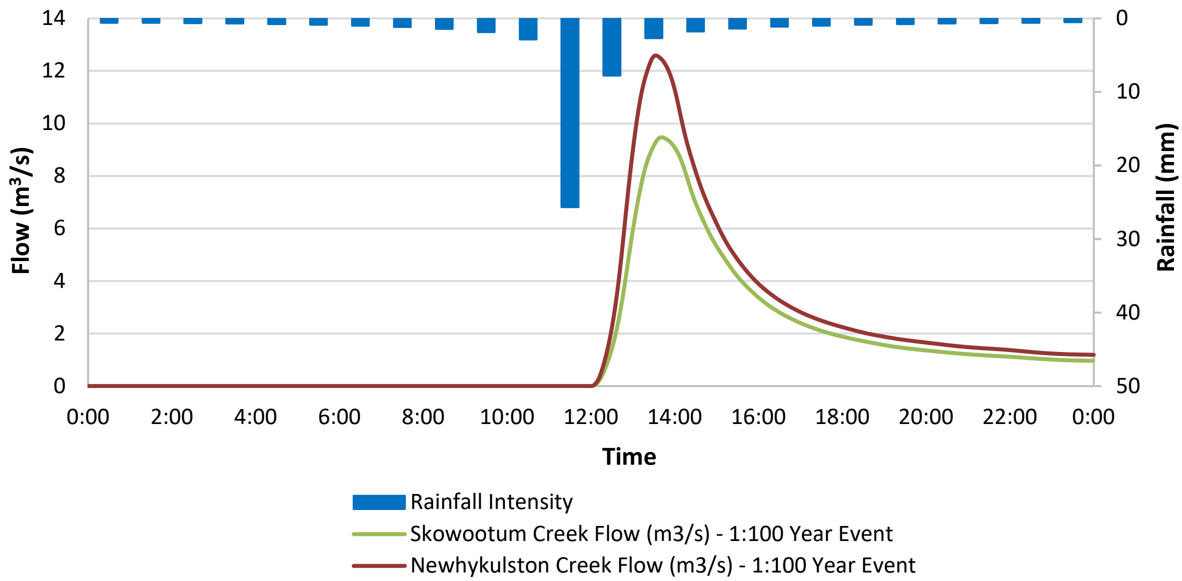

Figure 4-2. Simulated 100-year return period hydrographs for Skowootum Creek and Newhykulston Creek.

4.3.4. Peak Flow Secondary Check

As a secondary check on the peak flow estimate derived using the HEC-HMS model, the peak flow was compared to an independent estimate from the B.C. Streamflow Inventory (Coulson & Obedkoff, 1998). An online map[9] displays 100-year peak flow isolines, which are normalized for a 100 km^2 catchment. The two crossings correspond approximately to the 25 m^3/s isoline (Q_{100k} on the equation below). The discharge is then adjusted for catchment area using the following equation:

$$Q_{site}=Q_{100k}*\left(\frac{A}{100}\right)^{0.785} \quad \text{[Eq. 4-1]}$$

where:

Q_{site} is the discharge for the study sites

Q_{100k} is the isoline value corresponding to a normalized 100 km^2 catchment

A is the watershed area of the study site.

The resulting Q_{i100} peak flow estimates for Newhykulston Creek and Skowootum Creek are 5.4 m^3/s and 4.6 m^3/s, respectively. These peak flow estimates are approximately 50% lower than those generated using HEC-HMS, which is consistent with regional analysis[10] potentially under-estimating peak flows in the smaller watersheds.

[9] https://catalogue.data.gov.bc.ca/dataset/hydrology-100-year-peak-flow-isolines-historical
[10] The hydrologic assessment of Coulson and Obedkoff (1998) is essentially a regional analysis.

4.4. Climate Change Assessment

The crossings of Skowootum and Newhykulston Creeks are vulnerable to potential increases in peak flows related to climate change. As requested by MOTI, an assessment was undertaken by BGC to estimate climate-adjusted design flows. Two assessment methods were conducted, as discussed below. Both methods considered an ensemble of global climate models (GCMs) as listed in Table 4-4, but only representative concentration pathway (RCP) 8.5 was considered. EGBC (2020) notes that RCP 8.5 is generally regarded as the most relevant scenario for infrastructure design as it represents the worst-case RCP scenario and is consistent with current trends in greenhouse gas emissions. From discussions with MOTI, BGC understands that the typical expected service life of a highway culvert is 75 years. Therefore, a time horizon of 2100 was considered for the assessment.

Based on historical daily precipitation data obtained at the *Darfield* climate station, the ten largest rain events on record occurred from the months of May to October. Eight of those events occurred during the summer months (June to August), including the three largest events on record. For the climate assessment, BGC assumed that future peak flows will also be driven by intense convective summer rain storms, rather than rain-on-snow or pure snow melt events.

Table 4-4. GCM ensembles used for each climate change assessment method.

PCIC Statistically Downscaled GCM Scenarios	University of Western Ontario IDF_CC Tool
CNRM-CM5	CNRM-CM5
CanESM2	CanESM2
ACCESS1-0	CSIRO-Mk3-6-0
Inmcm4	CCSM4
CSIRO-Mk3-6-0	MIROC5
CCSM4	MPI-ESM-LR
MIROC5	MRI-CGCM3
MPI-ESM-LR	GFDL-ESM2G
HadGEM2-CC	HadGEM2-ES
MRI-CGCM3	
GFDL-ESM2G	
HadGEM2-ES	

4.4.1. PCIC Statistically Downscaled GCM Scenarios

The Pacific Climate Impacts Consortium (PCIC) provides statistically downscaled daily climate scenarios at a gridded resolution of approximately 10 km. A climate change analysis was conducted using model outputs of daily precipitation from the grid element in which the Darfield station is situated. For all GCM outputs, simulated maximum daily precipitation values were extracted for each year from 1956 to 2100. Frequency analyses were then conducted to estimate the 24-hour rainfall depths for various return periods associated with the historical timeframe (1956-2016) and three future timeframes (2011-2040, 2041-2070, and 2071-2100). The range of results from model ensemble are presented in terms of percentiles in Appendix B (Figures B1 to B5).

As shown on Figure B1 and in Table 4-5, the results indicate that 24-hour rainfall depth magnitudes estimated by BGC, based on historical data (1956-2016), align well with the 75th percentile of model simulation results over the same period. On that basis, BGC has assumed that it is reasonable for future conditions to also be estimated by the 75th percentile of model simulation results. The future timeframes over which the analysis the was conducted do not show a consistent upward trend in 24-hour rainfall depth magnitudes. Values appear to peak during the 2041-2070 period before dropping during the 2071-2100 period. The peak value was compared to the historical simulation results to estimate a scaling factor with which to adjust the design rainfall depths. For the 100-year 24-hour rainfall depth, a scaling factor of 1.2 (i.e., 20%) was used on BGC's estimate (64 mm), resulting in an upward adjustment to 77 mm. A comparison of the observed and climate change adjusted 24-hour rainfall depths is provided in Table 4-6.

Table 4-5. 75th percentile of model ensemble results for 24-hour rainfall depths (mm), RCP 8.5.

Return Period (Years)	2	5	10	25	50	100	200
BGC Historical 1956 - 2016	25	33	39	49	56	64	73
CC_Model Historical 1956 -2016	23	32	38	48	55	63	74
CC_Model 2011-2040	27	37	44	55	64	75	87
CC_Model 2041-2070	26	36	44	59	67	76	90
CC_Model 2071-2100	25	34	41	50	58	69	82
Maximum Scaling Factor	1.12	1.14	1.15	1.22	1.21	1.20	1.22

Table 4-6. Observed and climate change adjusted 24-hour rainfall depths (mm) for *Darfield* (#1162265).

Return Period (years)	Historical Observed[1]	Climate Change Projection
2	25	27
5	33	38
10	39	45
25	49	59
50	56	68
100	64	77
200	73	89

Notes:

1. Based on the period 1956-2016

4.4.2. University of Western Ontario IDF_CC Tool

A climate change analysis was also completed using the University of Western Ontario IDF_CC Tool (IDF_CC Tool, www.idf-cc-uwo.ca). The IDF_CC Tool allows for historical and climate change adjusted IDF data to be generated for ungauged sites at any location in Canada. An ungauged site was created at the approximate location of the *Darfield* climate station and IDF data were generated using the model ensemble listed in Table 4-4. As the IDF_CC Tool requires a minimum projection period of 50 years for climate change assessments, the period from 2050 to 2100 was selected. The results show an upward adjustment of 30% for the 100 year 24-hour rainfall depth compared to historical data. The IDF_CC Tool results are in general agreement with the results from BGC's analysis of PCIC data in showing an increase in precipitation over time. However, the results from BGC's analysis were used given the transparency of the process and the following capabilities, which were not available using the IDF_CC Tool:

- Ability to compare historical and future model simulations by percentile
- Ability to analyze timeframes smaller than 50 years
- Inclusion of all GCMs recommended by PCIC for western North America.

This choice is also consistent with the MOTI climate change guidelines (EGBC, 2020), which note the following: "there are a number of uncertainties associated with the method that the tool uses to produce sub-daily projections; therefore, it is recommended that this tool be used for exploratory rather than design purposes".

4.4.3. Hydrological Model Results

The rainfall-runoff model was rerun using a climate change adjusted design hyetograph and assuming existing hydrological watershed characteristics. The resulting design flows are provided in Table 4-7. The Q_{i100} peak flow estimates prevail as the design event for both Skowootum Creek and Newhykulston Creek. The climate change adjusted 100-year return period storm hydrographs are also shown in Figure 4-3.

Table 4-7. HEC-HMS modelling results for the projected climate change scenario.

Parameters	Skowootum Creek	Newhykulston Creek
1:100-year, 24-hr Peak Instantaneous Flows (m^3/s)	16.5	21.9
1:200-year, 24-hr Maximum Daily Flows (m^3/s)	6.1	7.6

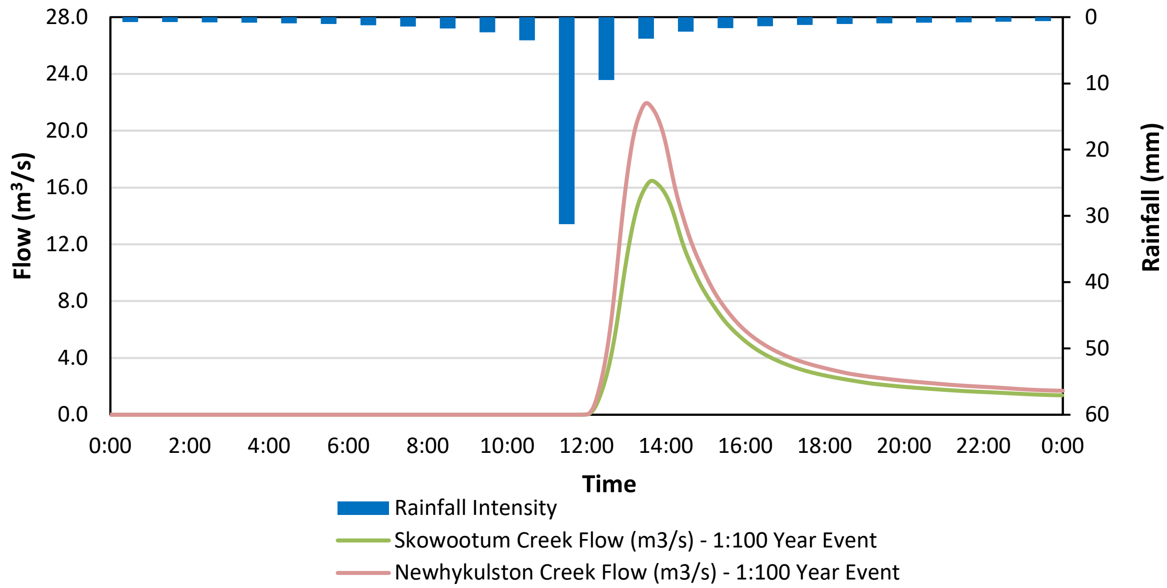

Figure 4-3. Climate change adjusted 100-year return period hydrographs for Skowootum Creek and Newhykulston Creek.

4.4.4. Risk Analysis

EGBC (2020) guidelines stipulate that a risk assessment should be conducted to determine if climate change should be included in the infrastructure design. Risk is a measure of the probability and severity of an adverse effect to health, property or the environment, and is estimated by the numerical product of hazard probability times consequences.

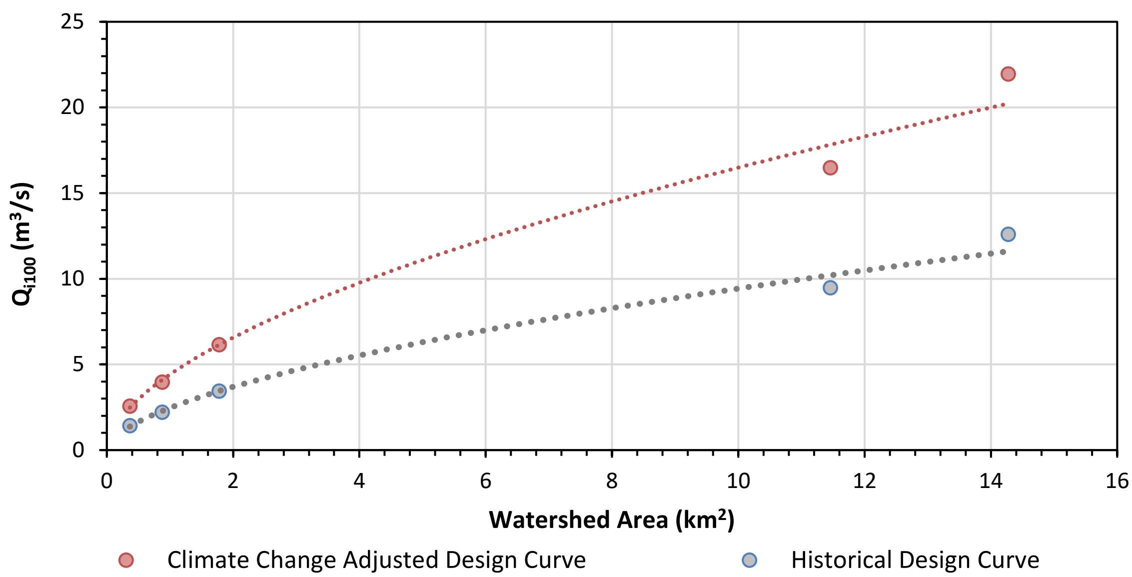

Figure 4-5 shows a risk evaluation matrix commonly used by BGC to combine the likelihood of an unwanted outcome and consequence assessment to determine a risk rating for flooding hazards. The probability of the undesirable outcome and the severity of the consequence define an intersection point in the matrix that ranks the risk scenario from “Very Low” to “Very High”. The top five rows of Figure 4-4 guide possible responses by the proponent to each risk level but depend on the proponent’s risk tolerance criteria. Based on the EGBC (2020) guidelines, infrastructure with “High” or “Very High” risk scores would be candidates for a robust design, while “Moderate” risk may be candidate for a flexible design or may be evaluated further to determine if additional assessment, such as using engineering analysis, is required to clarify risk and identify appropriate adaptation measures.

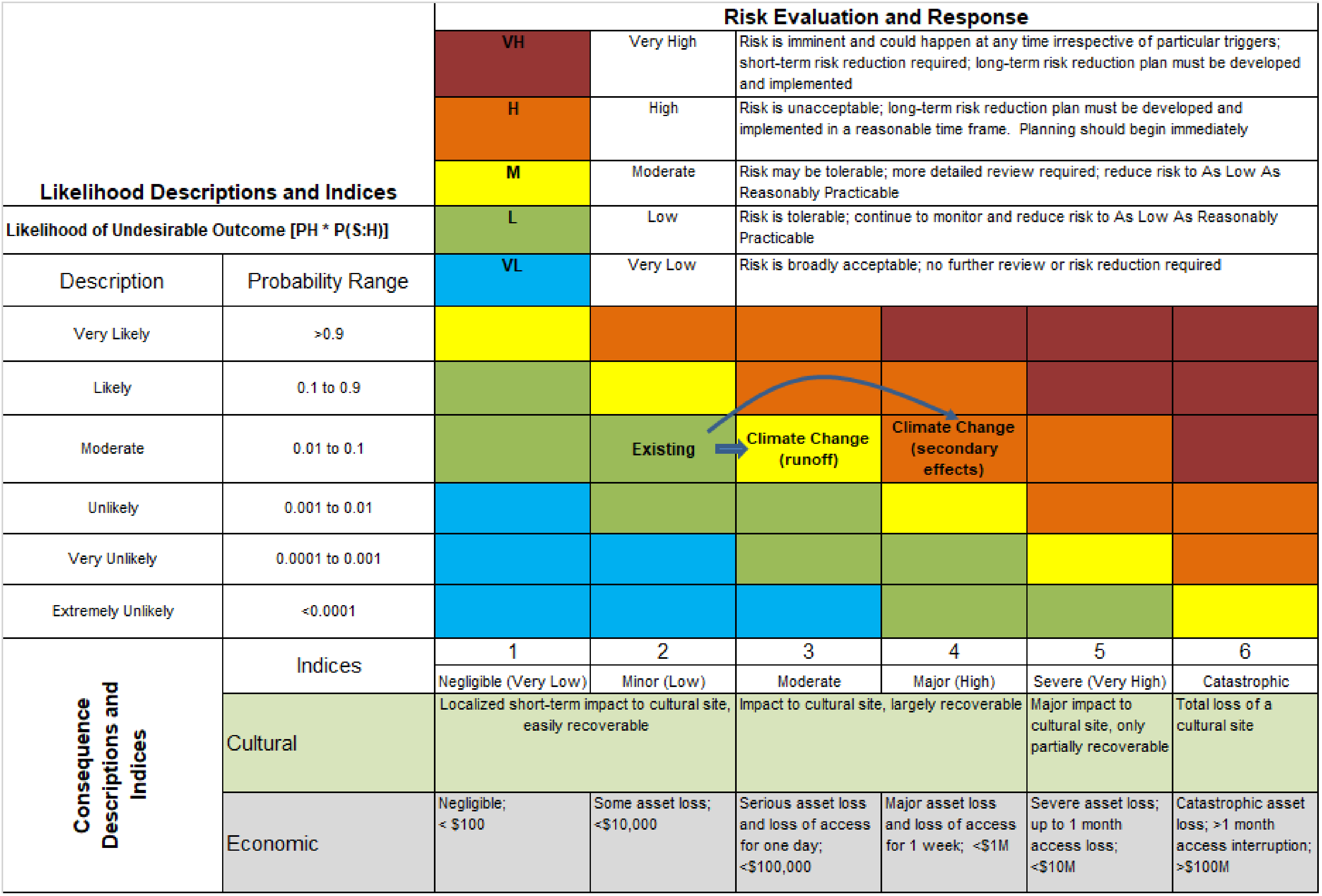

Figure 4-4. BGC's semi-quantitative risk matrix for geohazard risk assessments with application to Newhykulston Creek.

BGC first evaluated flooding hazards for existing conditions on Newhykulston Creek, that is assuming the existing culvert would be upgraded to convey a peak flow of 12.6 m^3/s. Under this condition, the likelihood is Moderate while the potential cultural and economic consequences are Low – the resulting risk is Low. If the culvert was not designed to account for climate change, then the resulting consequences and risk would be Moderate, as an avulsion could occur during the design event due to inadequate flow capacity at the crossing. Secondary effects of climate change, such as the increased frequency of a major debris flood and associated avulsion, could result in a major consequence and therefore High risk. However, the selected crossing does not influence the consequences of such climate change-induced secondary effects.

Based on this risk analysis, it seems appropriate to design the Newhykulston Creek crossing for the projected climate change peak flow of 21.9 m^3/s, as the risk would then drop back down to Low.

For Skowootum Creek, the hydrogeomorphic analysis concluded that large debris flow events have a return period greater than several hundred years. However, it was also noted smaller debris flow events capable of damaging the road and crossing also occur, likely on the order of couple to several decades. It was concluded that the crossing should be designed for clearwater floods only while acknowledging that maintenance will be required following future debris flow events. Given the potential for small debris flow events to occur with a relatively high frequency and that a 100-year peak flow is likely to trigger either a debris flow or hyperconcentrated flow, it is recommended that the crossing for Skowootum Creek not be designed for projected climate change.

4.5. Regional Design Flow Curve

Regional design flow curves (

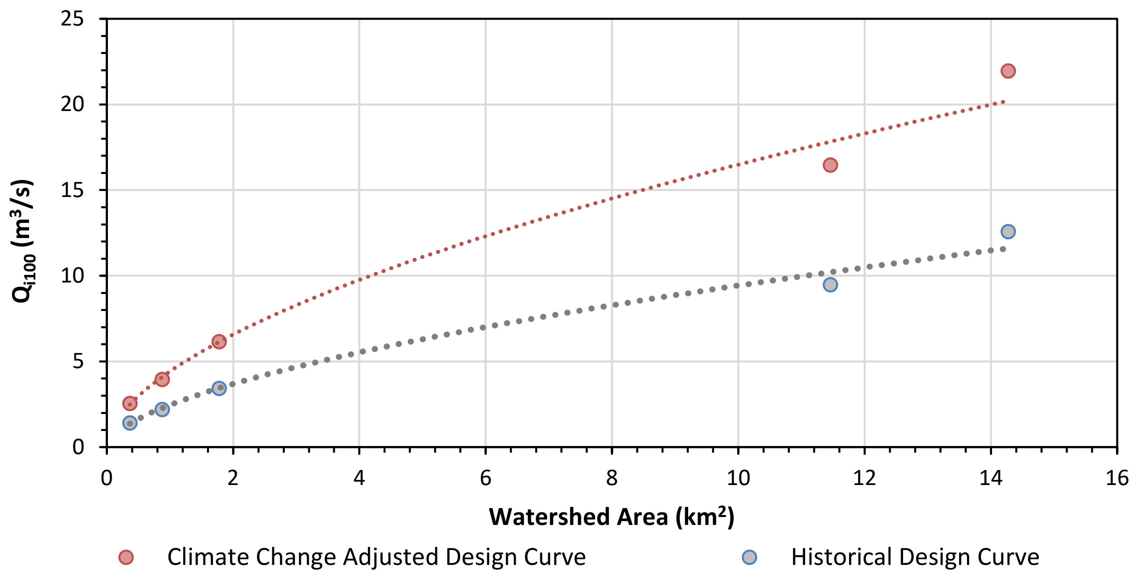

Figure 4-5) for both historical peak flow [Eq. 4-2] and climate change adjusted [Eq. 4-3] scenarios were generated to assist MOTI in estimating design flows for small drainage basins along Dunn Lake Road. The design curves are based on the 100-year peak instantaneous flows for five catchments using the methodology described in Section 4.3. In addition to the Skowootum Creek and Newhykulston Creek catchments, three additional surrounding catchments were analyzed to capture the spectrum of spatial variability in catchments. The average watershed gradient for the catchments used to develop this curve varies from 18% to 53%.

Historical design flows:

$$Q_{i100} = 2.46 \times Watershed\ Area^{0.58} \qquad \text{[Eq. 4-2]}$$

Climate change adjusted design flows:

$$Q_{i100} = 4.42 \times Watershed\ Area^{0.57} \qquad \text{[Eq. 4-3]}$$

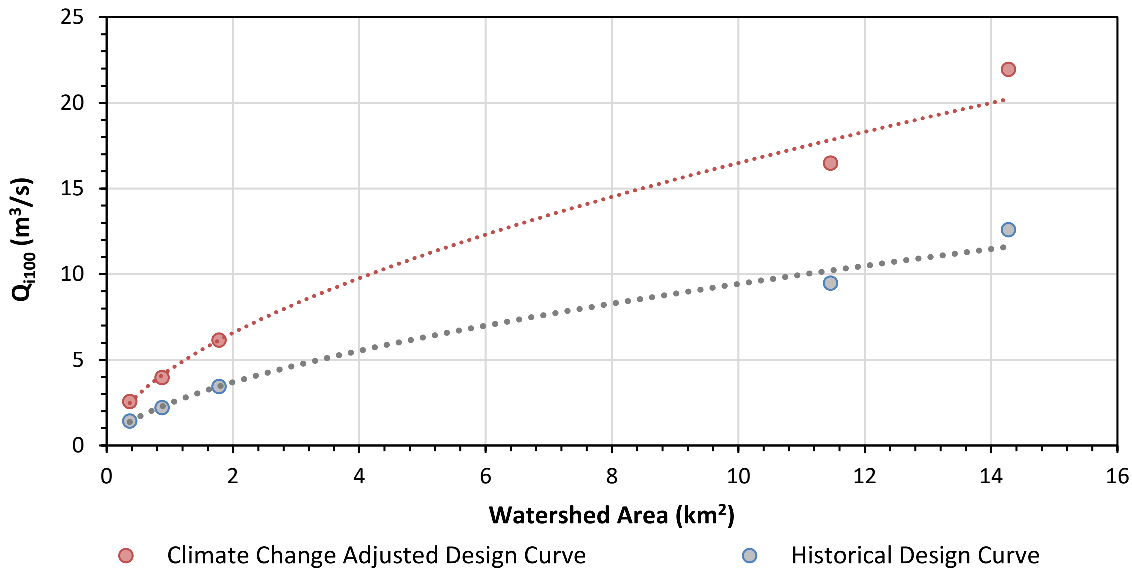

Figure 4-5. Historical and climate change adjusted regional (Q_{i100}) design curves along Dunn Lake Road for small watersheds.

5.0 CULVERT REPLACEMENT OPTIONS ASSESSMENT

Preliminary hydraulic analyses were completed to inform an assessment of potential culvert replacement options for Skowootum and Newhykulston Creeks. Culvert sizing was completed using both HY-8 and HEC-RAS software. Preliminary culvert dimensions were optimized for the options assessment based on the natural widths of the creeks, maintaining minimum depths of cover beneath the road surface, and maintaining headwater-to-diameter (HW/D) ratios less than 1.0. Comparisons between options were made primarily based on culvert type, size, construction considerations, and fish passage considerations.

The following culvert replacement options were considered at each of the crossings:

1) A) Single round CSP culvert (20% and 40% embedded)
2) A) Single concrete box culvert (20% embedded)
3) Open-bottom arch culvert (including low profile options)
4) Pipe arch CSP or structural plate CSP (SPCSP) (20% embedded)

Bridge options were not considered for these crossings due to the high relative cost and limited benefits in passing clear water flows and debris floods (i.e., at Newhykulston Creek) compared to the culvert options listed. As both creeks are subject to hydrogeomorphic processes capable of causing avulsions upstream from the crossings or significant damage to the crossings themselves, potential bridge replacement would also provide limited benefits for passing such events.

For the Skowootum Creek crossing, MOTI selected a 2.7 m diameter CSP culvert (20% embedded) to replace the existing culverts. As Skowootum Creek is prone to debris flows, the necessity for small machinery (i.e., a bobcat) to occasionally enter and clean out the culvert was a consideration of the culvert sizing. Based on discussions with MOTI, BGC understands that fish passage of the crossing is preferred but not required. Concrete collar end treatments were selected in order to mitigate the reduced ring compression that would result from step-beveling the culvert inlet and outlet. Concrete cutoff walls were also incorporated at the inlet and outlet to reduce of the potential for piping around the culvert.

For the Newhykulston Creek crossing, MOTI selected a 5 m x 2 m (Span x Rise) low profile open-bottom arch culvert to replace the existing CSP culvert and support fish passage of the crossing. Headwall (with wingwall) end treatments were selected in order to minimize the length of the culvert and associated pile foundation.

6.0 DETAILED HYDROTECHNICAL DESIGN

Detailed design of the Skowootum Creek and Newhykulston Creek culverts required a multi-disciplinary approach. The designs were developed in coordination with Parsons Corporation (Parsons, project structural engineering consultant), Thurber Engineering Ltd. (Thurber, project geotechnical engineering consultant), and MOTI's highway design group. Detailed hydrotechnical design of the culverts and instream armouring are discussed herein.

6.1. HEC-RAS Modelling

As mentioned in Section 5.0, BGC developed one-dimensional hydraulic models of both creeks using the US Army Corps of Engineers HEC-RAS (version 5.0.7) modelling software to estimate hydraulic conditions in the vicinity of the road crossings as part of the culvert replacement options assessment task. The models were subsequently refined during detailed design to optimize the culvert alignment, the local channel grade, and instream armouring.

The models were initially developed using channel cross-sections cut from a digital elevation model (DEM) created using the following survey data sources provided by MOTI:

- Topographic and bathymetric survey of Skowootum Creek extending approximately 155 m upstream and 150 m downstream from the road crossing (date unknown to BGC)
- Topographic and bathymetric survey of Newhykulston Creek extending approximately 200 m upstream and 80 m downstream from the road crossing (dated January 22, 2019)
- A LiDAR survey conducted along the full 12.5 km extent of the project in an average 500 m wide swath (dated June 2014).

The following model parameters were used:

- Manning's n values of 0.08 within the main channel and 0.1 beyond the banks of both creeks
- Model boundary conditions set as normal depth based on the surveyed upstream and downstream channel gradients
- Culvert entrance loss coefficient of 0.5 and exit loss coefficient of 1.0 for both creek crossings.

Both models were run over a range of flow conditions including the 100-year peak instantaneous design flow (including the upward adjustment for climate change for Newhykulston Creek). Water surface profiles for both crossings are provided in Appendix C.

6.2. Scour Analysis

6.2.1. Newhykulston Creek Scour

At the Newhykulston Creek crossing, mean contraction scour within the open-bottom low profile arch culvert was estimated using the U.S. Federal Highways Administration equation provided in Equation 1 (FHWA, 2012).

$$d_{s,C} = d_1 \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^k \qquad \text{Eq.1}$$

Where:

$d_{s,C}$ is the depth of contraction scour below design water surface elevation (WSE) (m)
d_1 is the average depth of the approach flow (m)
Q_2 is the design flood discharge through the bridge opening (m^3/s)
Q_1 is the design flood discharge through the approach channel (not including overbank flow) (m^3/s)
W_2 is the net (top) width at the bridge opening (m)
W_1 is the net (top)width of the approach channel (m)
k is the sediment transport exponent (0.59 for mainly bedload)

Input parameters used in the contraction scour estimate are provided in Table 6-1. Based on a channel bed elevation of 411.9 m at the culvert inlet, the predicted scour elevation is 411.3 m, or 0.6 m below the average channel bed elevation.

Table 6-1. Input parameters used for contraction scour analysis

Parameter	Value
WSE (m)	413.1
d_1 (m)	1.3
Q_2 (m^3/s)	21.9
Q_1 (m^3/s)	21.90
W_2 (m)	5
W_1 (m)	9
k	0.59
$d_{s,C}$ (m)	1.8

Natural scour was also estimated within the Newhykulston Creek crossing reach using the Blench Regime Depth method (Blench, 1969) based on Equations 2 and 3. The estimated natural scour depth beneath the channel bed was estimated based on a typical channel cross section located approximately 10 m upstream from the culvert inlet. The natural scour depth is estimated to be approximately 1.0 m below the channel thalweg during the design flood event.

$$d_f = \left(\frac{{q_f}^2}{F_{b0}}\right)^{1/3} \qquad \text{Eq.2}$$

$$d_{s,G} = Zd_f \qquad \text{Eq.3}$$

Where:

- $d_{s,N}$ is the depth of natural scour below regime water surface elevation (m)
- d_f is the mean regime depth (m)
- q_f is the design flood discharge per unit width (m^3/s/m)
- F_{b0} is the zero bed factor (m/s^2)
- Z is the channel morphology Z factor

Additional intermediate parameters are shown below:

- Q is the design flow in the main channel (m^3/s)
- W is the top width of flow within the main channel (m)
- D_{50} is the median bed material particle size (mm)

Input parameters used in the natural scour estimate are provided in Table 6-2.

Table 6-2. Input parameters used for natural scour analysis – Newhykulston Creek

Parameter	Value
Q (m^3/s)	21.9
W (m)	9
D_{50} (mm)	100
q_f (m^3/s/m)	2.4
F_{b0} (m/s^2)	1.7
Z	1.25
$d_{s,N}$ (m)	1.9

Given that the estimated natural scour within the crossing reach is greater than the estimated contraction scour within the culvert, the natural scour depth estimate has been considered for both design of the culvert foundation (by Parsons) and the instream armouring.

6.2.2. Skowootum Creek Scour

Given that the selected replacement culvert for the Skowootum Creek crossing is a 2.7 m diameter CSP culvert, contraction scour was not estimated as this scour component would be restricted by the form of the closed bottom culvert. Similar to the Newhykulston Creek crossing, natural scour was estimated for the Skowootum Creek crossing reach using Equations 2 and 3. The estimated natural scour depth beneath the channel bed was estimated based on a typical channel cross section located approximately 20 m upstream from the culvert inlet. The natural scour depth is estimated to be approximately 0.7 m below the channel thalweg during the design flood event. Input parameters used in the natural scour estimate are provided in Table 6-3.

Table 6-3. Input parameters used for natural scour analysis – Skowootum Creek

Parameter	Value
Q (m³/s)	9.5
W (m)	7.5
D_{50} (mm)	100
q_f (m³/s/m)	1.3
F_{b0} (m/s²)	1.7
Z	1.5
$d_{s,N}$ (m)	1.5

6.3. Riprap Sizing and Placement

Riprap sizing for scour and erosion protection was estimated based on Equation 4 provided in Robinson et al. (1998), which is commonly used to size riprap in steep creeks with gradients steeper than ~5%. Input parameters and resulting riprap sizes are provided in Table 6-4.

$$D_{50} = \left[\frac{qS^{0.58}}{8.07 \times 10^{-6}}\right]^{1/1.89} \quad \text{Eq.4}$$

Where:

D_{50} is the median riprap particle size (mm)
q is the design flood discharge per unit bottom width (m³/s/m)
S is the energy gradient

Table 6-4. Input parameters for riprap sizing

Parameter	Newhykulston Creek	Skowootum Creek
q (m³/s/m)	3.1	3.2
S (m/m)	0.14	0.07
D_{50} (mm)	495	403

Note:

1. Energy gradient slopes shown represent the steepest conditions on which riprap will be placed along both creeks

USACE 1994 recommendeds that rounded riprap be sized about 25 percent larger than angular riprap with a comparable increase in thickness of the riprap layer. Riprap sizes selected for detailed design of instream works at the culvert crossings are provided in Table 6-5. Geotextile filter fabric should be installed beneath all riprap bank protection to reduce the potential for migration of soil particles from the underlying insitu soils. Recommended geotextile filter fabric specifications are provided in Table 6-6.

Table 6-5. Riprap armouring considerations.

Crossing	Riprap Armouring Considerations
Skowootum Creek	Angular Riprap - 100 kg Class (D_{50} ~ 425 mm)
Newhykulston Creek	Angular Riprap - 250 kg Class (D_{50} ~ 575 mm) Rounded Riprap – 500 kg Class (D_{50} ~ 725 mm)

Table 6-6. Recommended Geotextile Filter Fabric Specifications

Parameter	ASTM Test Method	Value[1]
Grab Strength	D4632	1110 N
Sewn Seam Strength	D4632	890 N
Tear Strength	D4533	440 N
Static CBR Puncture Strength	D6241	3110 N
Permittivity	D4491	0.7 s^{-1}
Apparent Opening Size (AOS)	D4751	0.15 mm

Notes:
1. All values minimum average roll value except AOS which is a maximum average roll value.
2. Tencate Mirafi 1100N or approved equivalent.

6.3.1. Newhykulston Creek Instream Design

The instream design for the Newhykulston Creek crossing is composed of (i) riprap bank protection, (ii) a series of grade control weirs, and (iii) an energy dissipation pool at the culvert outlet as shown on the attached. Riprap bank protection at the crossing consists of 1 m thick layers of 250 kg Class angular riprap extending approximately 14 m upstream from the culvert inlet and 16 m downstream from the culvert oulet along both the left and right banks. Due to the steep average gradient of the creek (~8%), four grade control structures have been incorporated into the design to reduce the potential for scour and degradation within the culvert and downstream from the culvert. The grade control structures consist of 500 kg Class rounded riprap with a typical thickness of 1.5 m and range in length from 4 m to 6 m. An approximately 7 m long pool is incorporated into the instream design downstream from the culvert. The purpose of the pool is to dissipate discharge energy from the culvert and to provide a resting area for fish prior to navigating the culvert and steep sections of the creek upstream from the culvert.

Following discussions with MOTI's environmental group, a decision was made to tie the instream works into the natural channel bed at a location approximately 20 m upstream from the culvert inlet so as to minimize the extent of construction disturbance. This configuration results in a relatively steep (~14%) section of creek immediately upstream from the culvert. It is anticipated the inflection point created in the creek bed at the upstream end of the instream works will gradually degrade (and headcut) overtime thereby naturally reducing the slope of the creek immediately upstream of the culvert.

6.3.2. Skowootum Creek Instream Design

Based on design files provided by MOTI's highway engineering group, the upgraded road centreline is located approximately 13 m west of the existing road centreline at the Skowootum Creek crossing. The instream design for the Skowootum Creek crossing therefore includes an open channel upstream from the culvert inlet, to allow for flow conveyance through the existing road fill. As the surface elevation of the existing road is slightly lower than the design riprap elevation (i.e., design water surface elevation + 0.3 m), thin layers of fill will be required along both banks to achieve the design grade. Riprap armour at the crossing consists of 0.9 m thick (minimum thickness) layers of 100 kg Class angular riprap extending approximately 15 m upstream from the culvert inlet and 11 m downstream from the culvert outlet.

7.0 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:

Evan Shih, M.Eng., P.Eng.
Senior Hydrotechnical/Geological Engineer

Hamish Weatherly, M.Sc., P.Geo.
Principal Hydrologist

Reviewed by:

Robert Millar, Ph.D., P.Eng., P.Geo.
Principal Hydrotechnical Engineer

ES/HW/AM/RM/cr/admin

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APPENDIX A
CLIMATE CHANGE ASSURANCE STATEMENT AND DESIGN CRITERA SHEET

Design Criteria Sheet for Climate Change Resilience

Highway Infrastructure Engineering Design and Climate Change Adaptation
BC Ministry of Transportation and Infrastructure
(Separate Criteria Sheet per Discipline)
(Submit all sheets to the Chief Engineers Office at:
BCMoTI-ChiefEngineersOffice@gov.bc.ca)

Project:	*Newhykulston Creek Culvert (Structure No. 10276-21) Replacement*
Type of work:	*Culvert Replacement Project*
Location:	*Dunn Lake Road, Barriere*
Discipline:	*Hydrotechnical*

Design Component	Design Life or Return Period	Design Criteria + (Units)	Design Value Without Climate Change	Change in Design Value from Future Climate	Design Value Including Climate Change	Adaptation Cost Estimate ($)	Comments / Notes / Deviations / Variances
Replacement Bridge	100 year RP	24-hour Rainfall Depth (mm) Flow Rate (m^3/s)	64 12.6	+20% on 24-hour Rainfall Depth)	77 21.9	$35,100 (cost of raising bank riprap by 0.1 m and increasing thickness by 0.3 m)	- Design flood elevation rises by 0.1 m with climate change adjustment (accounting for increase in culvert size) - Riprap class increases from 100 kg to 250 kg

Explanatory Notes / Discussion:

The BC Ministry of Transportation and Infrastructure (MOTI) plans to replace the Newhykulston Creek culvert on Dunn Lake Road in Barriere, BC. The existing culvert is a 1.8 m diameter corrugated steel pipe culvert with an estimated capacity of approximately 5.5 m^3/s. The proposed replacement culvert is an approximately 16.5 m long, open-bottom low profile arch culvert with a span of 5 m and a rise of 2 m.

The Newhykulston Creek crossing is vulnerable to potential changes in future peak flows as a consequence of climate change. As requested by MOTI, an assessment was undertaken by BGC to estimate climate-adjusted design flows. Pacific Climate Change Consortium (PCIC) provides statistically downscaled daily climate scenarios at a gridded resolution of approximately 10 km. A climate change analysis was conducted using model outputs of daily precipitation from the grid element in which the reference climate station (Darfield Station) was situated. For all GCM outputs, simulated maximum daily precipitation values were extracted for each year from 1956 to 2100. Frequency analyses were then conducted to estimate the 24-hour rainfall depths for various return periods associated with the historical timeframe (1956-2016) and three future timeframes (2011-2040, 2041-2070, and 2071-2100).

The results indicate that 24-hour rainfall depth magnitudes estimated by BGC, based on historical data (1956-2016), aligned well with the 75th percentile of model simulation results over the same period. On that basis, BGC assumed that it is reasonable for future conditions to also be estimated by the 75th percentile of model simulation results. The future timeframes over which the analysis the was conducted do not show a consistent upward trend in 24-hour rainfall depth magnitudes. Values appear to peak during the 2041-2070 period before dropping during the 2071-

2100 period. The peak value was compared to the historical simulation results to estimate a scaling factor with which to adjust the design rainfall depths. For the 100-year 24-hour rainfall depth, a scaling factor of 1.2 (i.e., 20%) was used on BGC's estimate (64 mm), resulting in an upward adjustment to 77 mm. A rainfall-runoff model was run using the climate change adjusted 100-year 24-hour rainfall depth, resulting in a design (peak instantaneous) flows of 21.9 m^3/s.

Based on hydraulic modelling results, an equivalent open-bottom low profile arch culvert with a span of 3.9 m and rise of 1.6 m would have sufficient capacity to convey the non-climate change adjusted design flow. However, based on input provided by Atlantic Industries Ltd., BGC understands that the cost difference of increasing the culvert size is negligible.

The climate change adjusted design flows also result in an approximate 0.1 m rise in the design flood elevation at the culvert inlet (with the increase in culvert size accounted for) and an increase in riprap size along the creek banks from 100 kg to 250 kg Class (with a corresponding increase in thickness of 0.3 m). The approximate additional riprap volume to accommodate climate change adjusted design flows is estimated to be 90 m^3. Considering a \$390/m^3 unit price for 250 kg Class riprap, the total cost for adapting to climate change is estimated to be \$35,100.

For a complete discussion of the results of the hydrotechnical assessment, please refer BGC's report titled "Dunn Lake Road – Hydrotechnical and Geomorphic Assessment", dated May 14, 2021.

Recommended by: Engineer of Record: Evan Shih, P.Eng.

Date: May 14, 2021

Engineering Firm: BGC Engineering Inc.

Accepted by BCMoTI Consultant Liaison: ______________________________
(For External Design)

Deviations and Variances Approved by the Chief Engineer: ______________________________
Program Contact: Chief Engineer BCMoTI

APPENDIX B
PCIC CLIMATE CHANGE ANALYSIS RESULTS

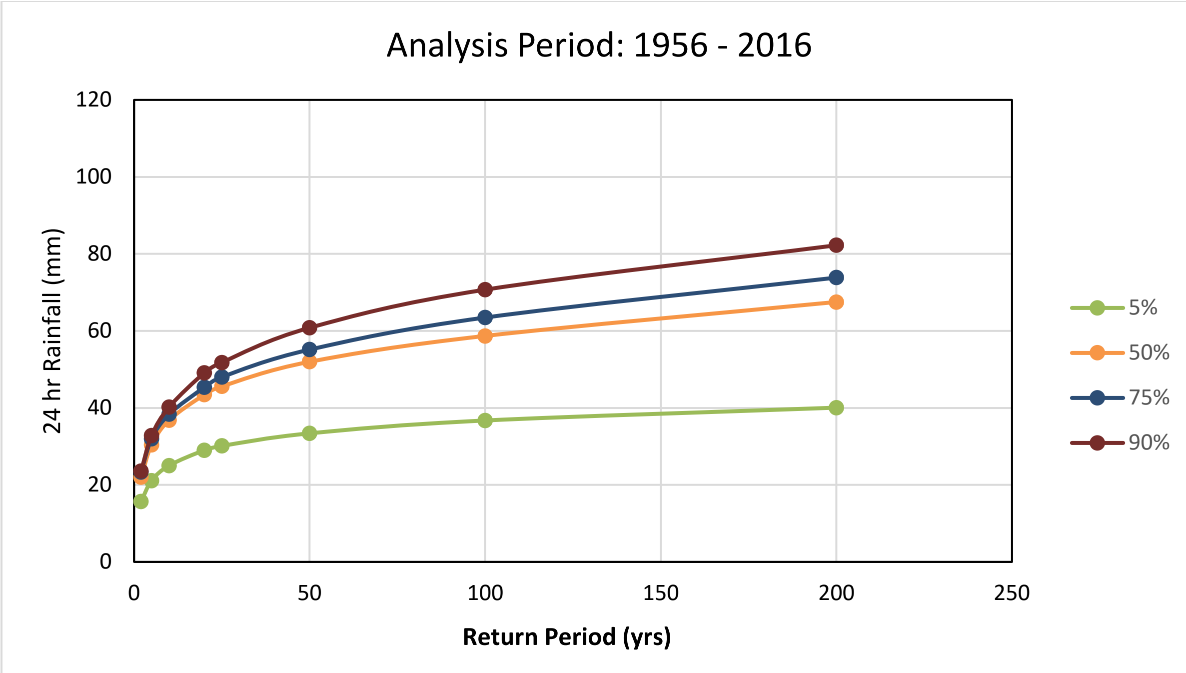


Figure B1. Climate change model ensemble results for the analysis period 1956-2016

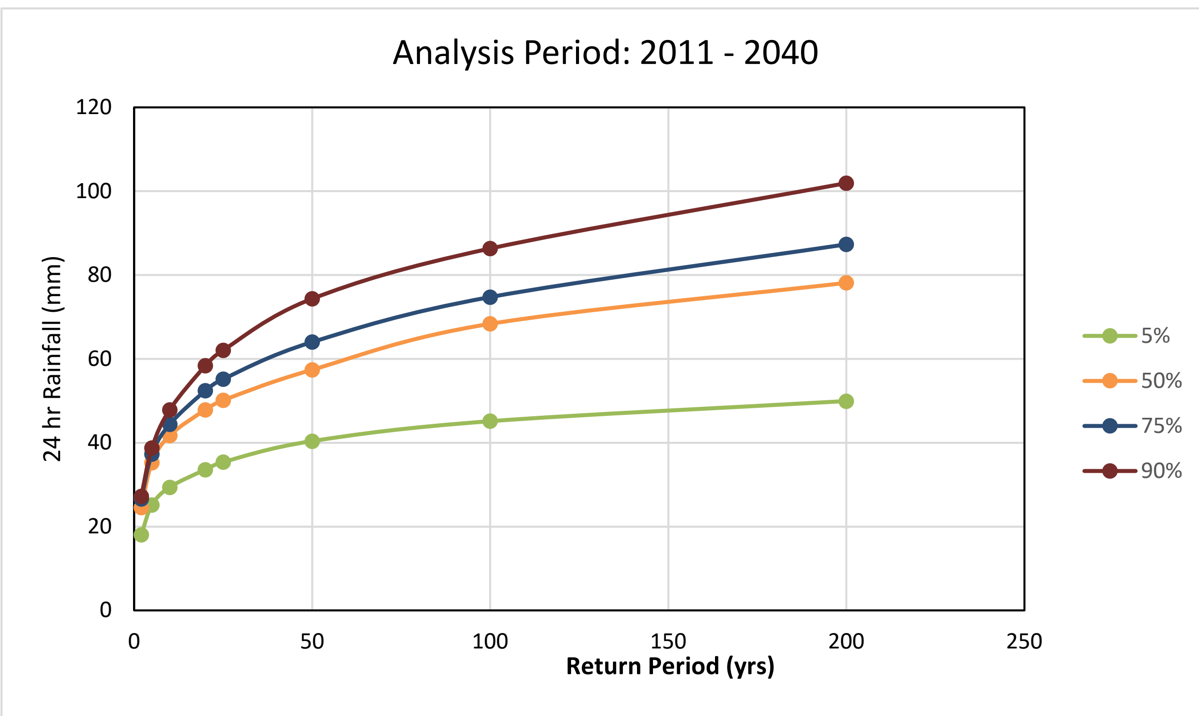


Figure B2. Climate change model ensemble results for the analysis period 2011-2040

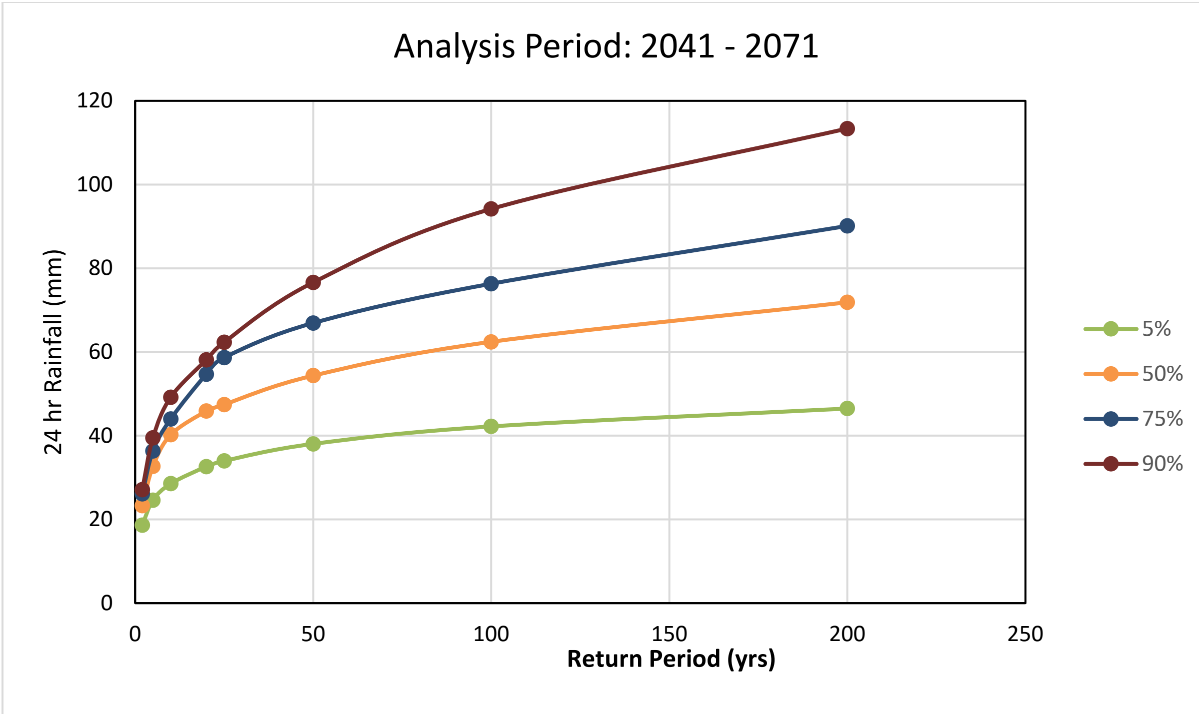
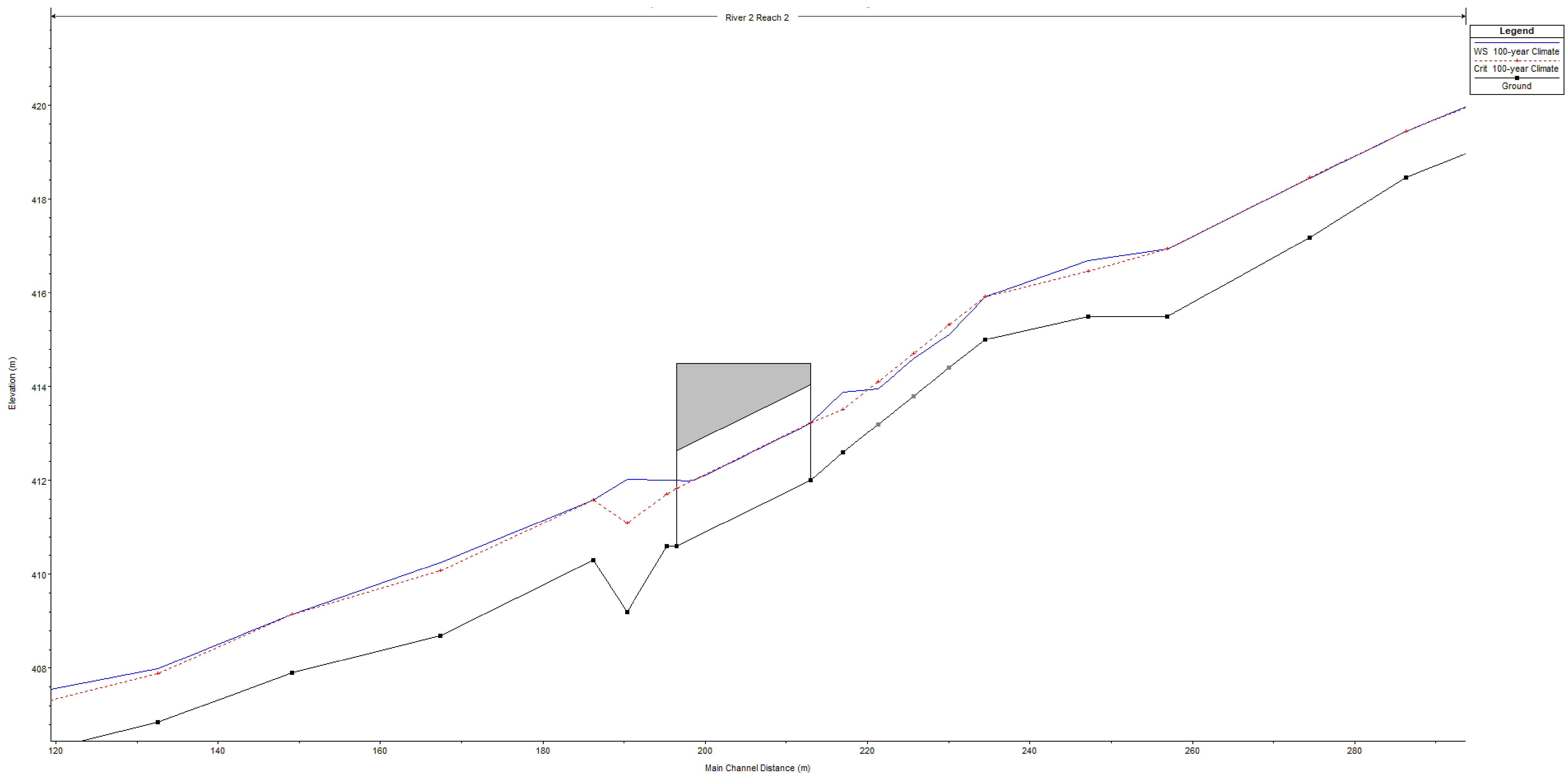


Figure B3. Climate change model ensemble results for the analysis period 2041-2070

Figure B2. Climate change model ensemble results for the analysis period 2071-2100

APPENDIX C
HEC-RAS WATER SURFACE PROFILES



NOTES
1. This Figure should be read in conjunction with BGC's report titled "Dunn Lake Road – Hydrotechnical and Geomorphic Assessment" and dated May 2021.

PREPARED BY: ES	FIGURE TITLE: HEC-RAS RESULTS PROFILE NEWHYKULSTON CREEK			
CHECKED BY: ES	CLIENT: BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE			
APPROVED BY: RM	SCALE: N/A	PROJECT NO: 0272-042	FIGURE NO: C-1	

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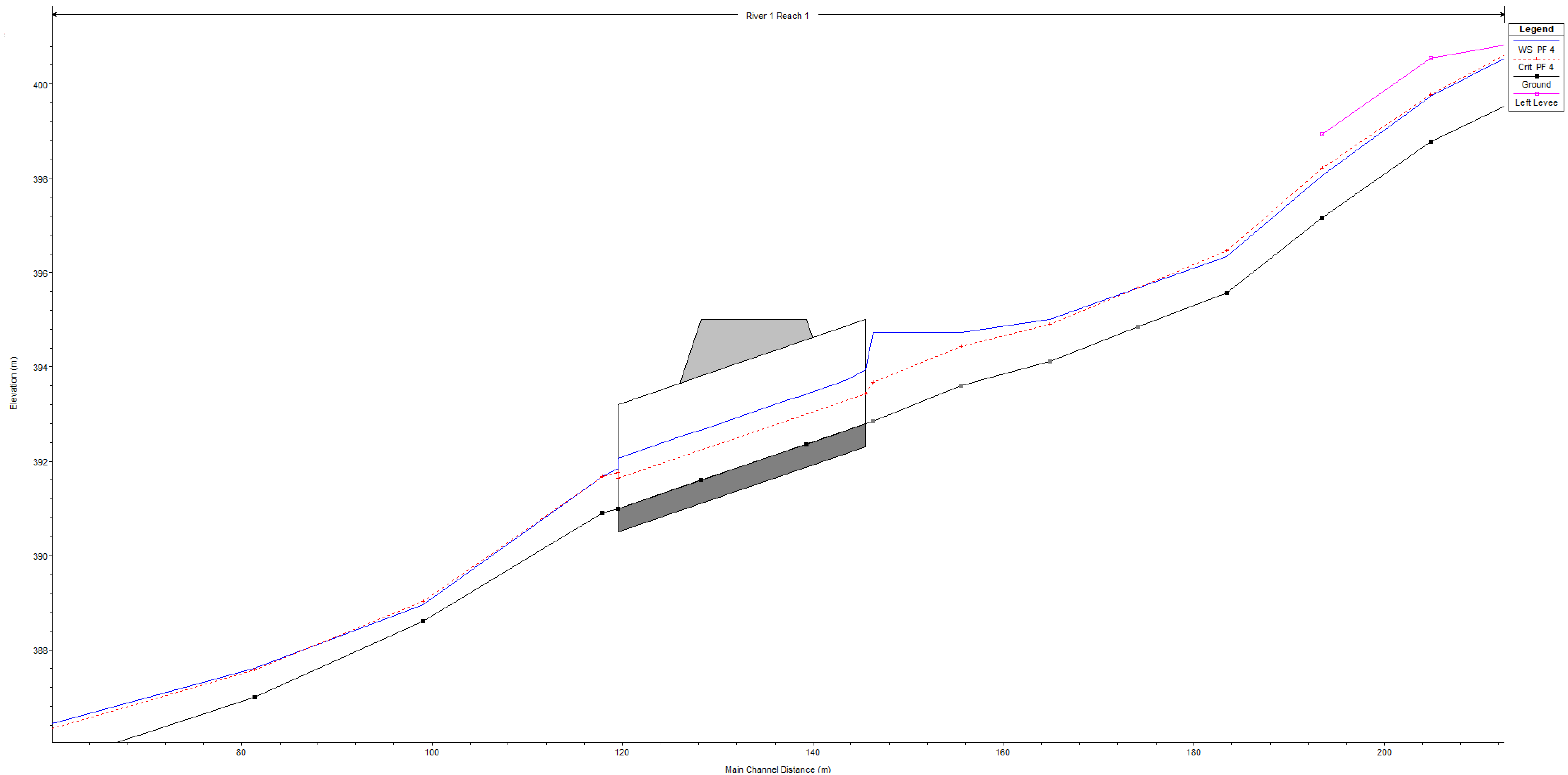

NOTES

1. This Figure should be read in conjunction with BGC's report titled "Dunn Lake Road – Hydrotechnical and Geomorphic Assessment" and dated May 2021.

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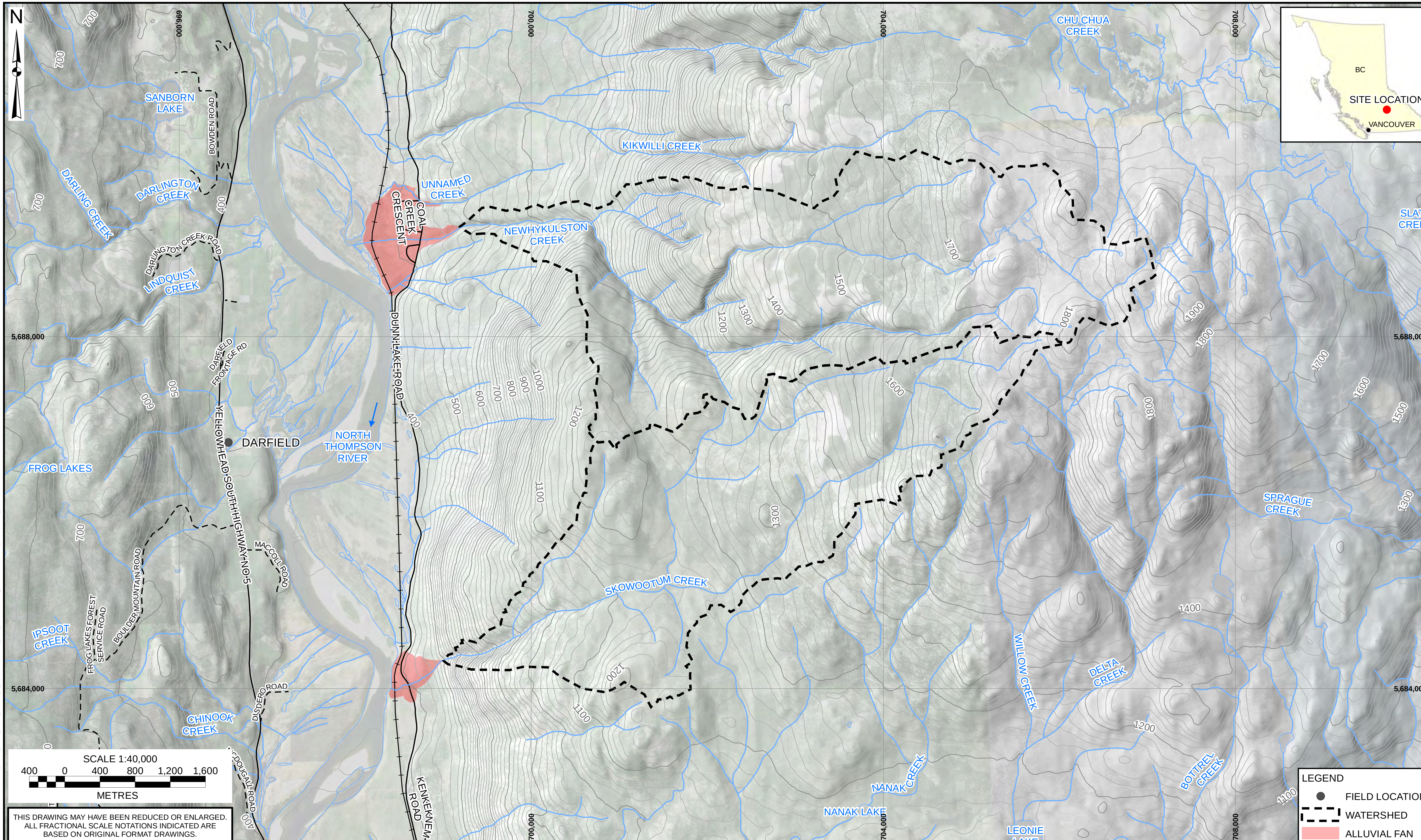
CHECKED BY: ES	CLIENT: BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE		
APPROVED BY: RM	SCALE: N/A	PROJECT NO: 0272-042	FIGURE NO: C-2

0 5 10 mm in ANSI B sized paper

DRAWINGS



NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "DUNN LAKE ROAD, PHASE 1 HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT", AND DATED MAY 2021.
3. BASE TOPOGRAPHIC DATA BASED ON CANADA DIGITAL ELEVATION DATA (CDED). CONTOUR INTERVAL IS 20 m.
4. STREAM, RAILWAY AND ROAD DATA FROM CANVEC.
5. PROJECTION IS PROJECTION IS UTM NAD 83 ZONE 10. VERTICAL DATUM IS CGVD28.
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PROJECT: DUNN LAKE ROAD, PHASE 1 HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT

TITLE: WATERSHED MAP

PROJECT No.: 0272 033

DWG No: 01

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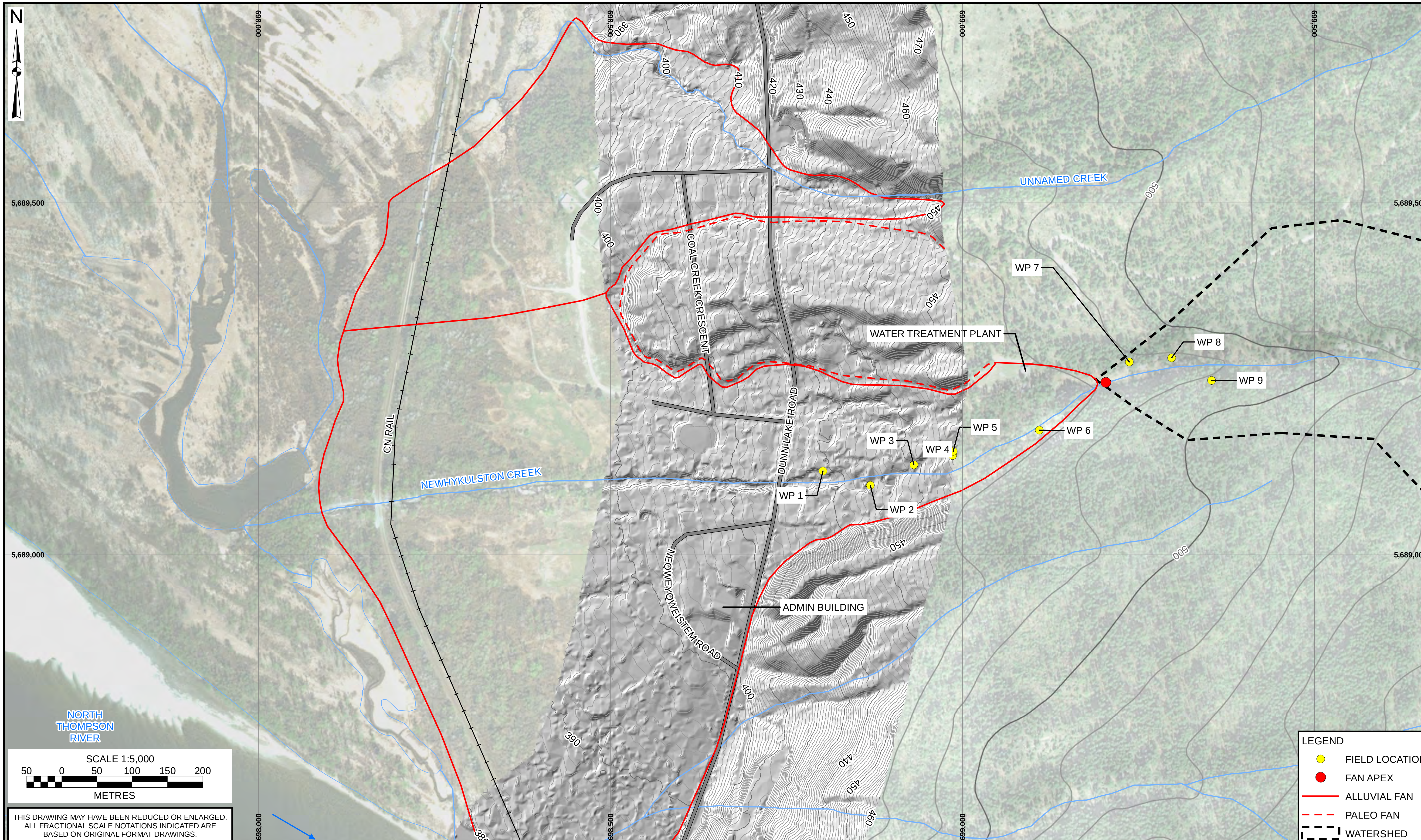

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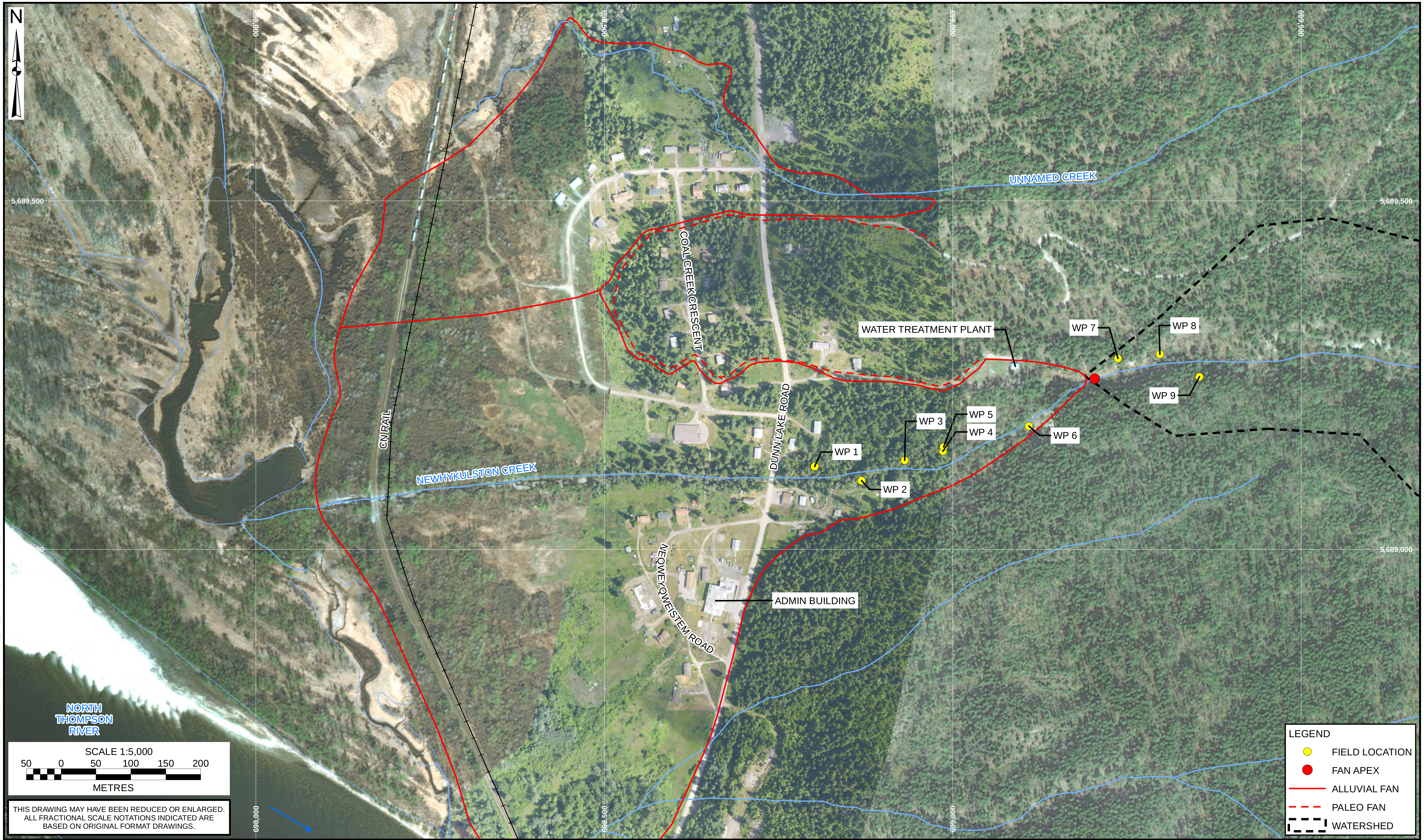
PROJECT: DUNN LAKE ROAD, PHASE 1 HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT

TITLE: NEWHYKULSTON CREEK FAN - HILLSHADE

PROJECT No.: 0272 033

DWG No: 02



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TITLE: NEWHYKULSTON CREEK FAN - ORTHOPHOTO

PROJECT No.: 0272 033

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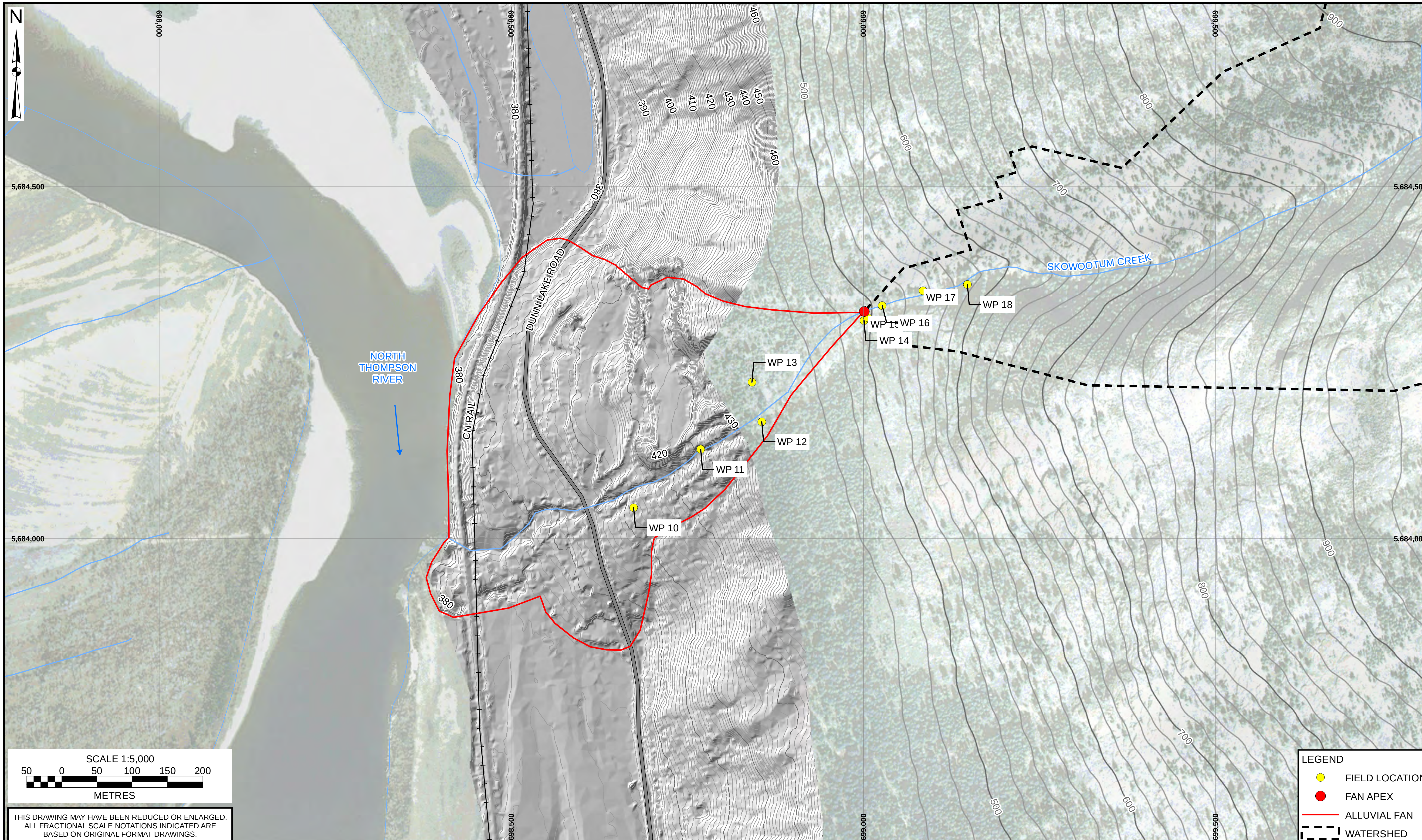

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PROJECT: DUNN LAKE ROAD, PHASE 1 HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT

TITLE: SKOWOOTUM CREEK FAN - HILLSHADE

PROJECT No.: 0272 033

DWG No: 04

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PROJECT: DUNN LAKE ROAD, PHASE 1 HYDROTECHNICAL AND GEOMORPHIC ASSESSMENT

TITLE: SKOWOOTUM CREEK FAN - ORTHOPHOTO

PROJECT No.: 0272 033

DWG No: 05

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