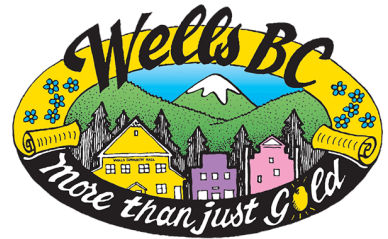




District of Wells Regional Flood Hazard Assessment

DRAFT

Prepared by BGC Engineering Inc. for:



August 17, 2026

Project 2546006

Document 2546006-001-RP-RevA



August 17, 2026

Project 2546006
Document 2546006-001-RP-RevA

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Attention: Randy Brown, Senior Project Manager

Regional Flood Hazard Assessment - DRAFT

Please find the above-referenced DRAFT report attached for your review and comment. We appreciate the opportunity to collaborate with you on this challenging and interesting project. Should you have any questions, please do not hesitate to contact the undersigned.

Yours sincerely,

BGC Engineering Inc.
per:

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

EXECUTIVE SUMMARY

BGC Engineering Inc. (BGC) completed a Regional Flood Hazard Assessment (RFHA) for the District of Wells (DoW) to support land-use planning, emergency management, and future flood mitigation initiatives. The assessment included a review of historical studies, flood records, aerial imagery, lidar data, watershed characteristics, and geomorphic conditions, along with hydrologic and hydraulic modelling to evaluate flood hazards under current and future climate conditions. The study considered flooding associated with rainfall and snowmelt runoff within the Williams Creek, Lowhee Creek, Willow River, and Jack of Clubs Creek watersheds.

A key finding of this assessment is that present-day flood hazards are not controlled solely by flood magnitude. Historical placer mining has significantly altered channel morphology and sediment transport processes throughout the watershed. More than a century after mining activities ceased, ongoing channel evolution and sediment deposition continue to influence flood hazards in several areas of the DoW.

Barkerville Town

Barkerville Town is located within the Williams Creek valley bottom downstream of Stouts Gulch and Conklin Gulch. Historical records indicate that flooding was common during the placer mining era, when large volumes of mining-derived sediment caused channel aggradation and frequent overbank flooding. Flood mitigation works, including the historic flood bulkhead and valley infilling, were subsequently constructed and continue to influence flood pathways today.

Geomorphic evidence indicates that the sediment wave associated with historical mining has largely migrated downstream and that channel narrowing and incision have substantially reduced the historical aggradation hazard within Barkerville. However, the community remains susceptible to flooding because of its location within an active floodplain.

Flood hazards are associated with Williams Creek overtopping its banks, runoff from the western hillslope exceeding the capacity of existing drainage infrastructure, and overflow from tributaries such as Stouts Gulch and Black Jack Gulch. Tributary overflows represent a significant flood hazard within Barkerville because they can bypass the Williams Creek corridor and divert floodwaters directly toward developed areas of the townsite.

The principal flood hazards within Barkerville Town are:

- Damage to buildings and heritage infrastructure located within identified flood-prone areas (particularly low-lying portions of the historic townsite)
- Localized flooding of transportation infrastructure, including sections of Highway 26 and bridge crossings.

North Barkerville

Flood hazards downstream of Bowron Lake Road differ substantially from those affecting Barkerville Town. As Williams Creek exits confinement and transitions onto a broad alluvial fan, channel gradients decrease and sediment deposition becomes the dominant geomorphic process. Historical aerial photographs, field observations, channel morphology, and sediment

transport analyses indicate that this reach remains actively aggrading and continues to accumulate sediment derived from historical placer mining activities.

The primary flood hazards within North Barkerville are:

- Flooding of critical infrastructure, including the Barkerville wastewater lagoons and associated access road
- Partial flooding of the Lowhee Campground
- Flooding of transportation infrastructure, including portions of Highway 26, Bowron Lake Road, One Mile Road, and the airstrip access road
- Ongoing channel aggradation and sediment accumulation that may alter flood behaviour and reduce channel conveyance over time.

The assessment identified sediment accumulation as a significant long-term hazard in this reach. Continued aggradation adjacent to the wastewater lagoon dikes and effluent outfall infrastructure could reduce available freeboard and increase future flood vulnerability if not monitored and managed.

Wells

Within Wells, the greatest flood hazards are associated with the Lowhee Creek alluvial fan. Historical placer mining substantially increased sediment supply within the Lowhee Creek watershed and altered natural channel processes. Sediment stored behind historical dams and debris jams continues to erode and move downstream, contributing to channel instability, fan growth, and ongoing sediment accumulation. These processes are expected to continue for decades.

The District has recently implemented significant flood mitigation measures, including the Phase 1 and Phase 2a Lowhee Creek dikes, with the Phase 2b extension currently advancing through permitting. Hydraulic modelling indicates that these works substantially reduce flood hazards to developed portions of South Wells under the design scenarios considered.

Flooding associated with the Willow River is comparatively limited. Hydraulic modelling indicates that key community infrastructure and most developed areas are located above predicted flood levels. Flooding is primarily restricted to undeveloped floodplain areas, trails, and pedestrian crossings adjacent to the river.

Transportation and Access Disruptions

Flood-related transportation disruptions represent an additional hazard throughout the study area. Localized flooding of Highway 26, Bowron Lake Road, One Mile Road, and other access routes may restrict movement during extreme flood events and could affect emergency response, evacuation, and access to critical infrastructure.

Overall, the assessment indicates that flood hazards within the DoW are controlled by both hydrologic and geomorphic processes. While inundation during extreme runoff events remains an important hazard, the long-term influence of historical placer mining, ongoing sediment deposition, and channel evolution are also key drivers of flood hazard in several areas.

TABLE OF REVISIONS

Date	Revision	Remarks
August 17, 2026	A	Draft report issued for review

CREDITS AND ACKNOWLEDGEMENTS

BGC would like to acknowledge Brett Eaton, Ph.D., P.Geo., Laura Szczyrba, Ph.D., GIT, and Maddux Swayze for their contributions to this report.

BGC would also like to express its appreciation for the support and input provided by external reviewers/contributors including Ed Coleman, Mayor of the District of Wells, and Randy Brown, Senior Project Manager, and Bob Radloff, P.Eng., of Radloff.

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GLOSSARY

Term	Definition
CDEM	Canadian Digital Elevation Model
CMIP6	Coupled Model Intercomparison Project Phase 6
CN	Curve number
CRD	Cariboo Regional District
DEM	Digital elevation model
DoW	District of Wells
DRR-CA	Disaster Risk Reduction-Climate Adaptation
ECCC	Environment and Climate Change Canada
HEC-HMS	Hydrologic Engineering Center's Hydrologic Modelling System
I_a	Initial abstraction (mm)
MAP	Mean annual precipitation
MAT	Mean annual temperature
PAS	Precipitation as snow
PCIC	Pacific Climate Impacts Consortium
RCP	Representative Concentration Pathway
RFHA	Regional Flood Hazard Assessment
S	Maximum soil moisture retention (mm)
SSP	Shared Socioeconomic Pathways
T_{lag}	Lag time
UBCM	Union of BC Municipalities
USACE	US Army Corp of Engineer

TABLE OF CONTENTS

EXECUTIVE SUMMARY	I
TABLE OF REVISIONS	III
CREDITS AND ACKNOWLEDGEMENTS	III
LIMITATIONS	IV
GLOSSARY	V
LIST OF TABLES	VIII
LIST OF FIGURES	IX
LIST OF DRAWINGS	XII
LIST OF APPENDICES	XII
1.0 INTRODUCTION	1
1.1 Background	1
1.2 Level of Detail	1
1.3 Scope of Work	3
2.0 MINING HISTORY	6
2.1 Placer Mining	6
2.1.1 Mining Activity in Lowhee Creek	9
2.1.2 Williams Creek	12
2.2 Underground Mines	14
2.3 Cariboo Gold Project	15
3.0 STUDY AREA CHARACTERIZATION	21
3.1 Watershed Description	21
3.2 Glacial History	22
3.3 Geology	23
3.4 Climate	25
3.5 Hydrologic Regime	27
3.6 Data Sources	30
4.0 EXISTING CONDITIONS	32
4.1 Barkerville Town	32
4.1.1 Overview of Assets	32
4.1.2 Historical Flood Events and Flood Mitigation	32
4.1.3 Hydraulic Structures	38
4.2 North Barkerville	43
4.2.1 Overview of Assets	43
4.2.2 Historical Flood Events and Flood Mitigation	46
4.2.3 Hydraulic Structures	48
4.3 Jack of Clubs Creek	50
4.4 Wells	52
4.4.1 Overview of Assets	52
4.4.2 Hydraulic Structures	54

4.4.3	Historical Flood Events and Flood Mitigation	58
5.0	GEOMORPHIC ASSESSMENT	61
5.1	Lowhee Creek Channel Changes.....	61
5.2	Lowhee Creek Geomorphic Assessment.....	68
5.2.1	May 2024 State of Emergency.....	68
5.2.2	Watershed Assessment	69
5.2.3	Sediment Management Plan.....	73
5.3	Williams Creek Channel Changes	77
5.4	Williams Creek Geomorphic Assessment	78
6.0	HYDROTECHNICAL METHODS AND INPUTS	83
6.1	Hydrologic Analysis	83
6.1.1	Previous Work.....	83
6.1.2	Watershed Characteristics	83
6.1.3	Rainfall Inputs	87
6.1.4	Climate Change	87
6.2	Hydraulic Modelling	88
6.2.1	Scenarios	88
6.2.2	Modelling Software	89
6.2.3	Previous Work.....	89
6.2.4	Model Terrain	89
6.2.5	Model Domain and Boundary Conditions	90
6.2.6	Manning's n Roughness	91
6.2.7	Computational Mesh	92
6.2.8	Hydraulic Structures.....	94
6.2.9	Initial Conditions.....	95
6.2.10	Simulation Settings	95
6.3	Flood Hazard Mapping.....	95
7.0	RESULTS.....	96
7.1	Peak Discharge Estimates.....	96
7.2	Hydraulic Modelling	97
7.2.1	Barkerville Town	97
7.2.2	North Barkerville	98
7.2.3	Wells	99
7.3	Flood Hazard.....	102
7.3.1	Barkerville Town	102
7.3.2	North Barkerville	103
7.3.3	Wells	104
7.3.4	Transportation and Access Disruptions	107
8.0	SENSITIVITY ANALYSIS	109
9.0	LIMITATIONS, UNCERTAINTIES AND RECOMMENDATIONS	114
9.1	Source of Uncertainty	114
9.2	Limits of Applicability.....	115

9.3 Recommendations for Hazard Management	116
10.0 CLOSURE	117
REFERENCES	118

LIST OF TABLES

Table 1-1	Hazard assessment levels of detail.	3
Table 2-1	Placer gold production (ounces) within the District of Wells between 1874 and 1945 (after Holland, 1950).	8
Table 3-1	Historical and projected change in climate variables for 2050 and 2080 (assuming SSP2-4.5) compared to the historical period 1961 to 1990 (Wang et al., 2016).	25
Table 3-2	Aerial imagery used for the District of Wells regional flood hazard assessment.	31
Table 4-1	Wells wastewater lagoon characteristics (from Radloff, September 22, 2022).	53
Table 5-1	Depth of placer outwash encountered in SoilTech boreholes.	63
Table 5-2	Estimated deposition volumes on Lowhee Creek fan for various climate-adjusted flood quantiles (after BGC, March 28, 2024).	73
Table 5-3	Sediment management measures related to the upper sub-reach of the Lowhee Creek alluvial fan.	74
Table 5-4	Williams Creek average channel width over time.	77
Table 5-5	Wolman sampling results along Williams Creek.	78
Table 6-1	Sub-watersheds characteristics.	86
Table 6-2	24-hour rainfall depths for the Willow River watershed for various AEPs.	87
Table 6-3	Increase in the rainfall depth and rain-on-snow depth from climate change to 2100 (SSP2-4.5 scenario).	88
Table 6-4	Hydraulic modelling scenarios.	89
Table 6-5	Selected Manning's roughness value for the different land cover type within the model domain.	91
Table 6-6	Terrain representation of bridges across the modelling domain.	94
Table 7-1	Peak discharges at key locations within the model.	96
Table 7-2	Summary of flood hazard.	106
Table 8-1	Change in flood extent relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope.	109
Table 8-2	Summary of the sensitivity analysis results for the water surface elevation (WSE) changes along Williams Creek and Willow River.	112

Table 8-3	Summary of the sensitivity analysis results for the water surface elevation (WSE) changes along Williams Creek and Willow River.	112
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LIST OF FIGURES

Figure 1-1	Federal flood mapping framework (NRCan, 2017).	5
Figure 2-1	Topographic map showing the distribution of major placer deposits.	6
Figure 2-2	Archival photographs of placer operations on Lowhee Creek..	10
Figure 2-3	1952 airphoto showing constructed dams within the Lowhee Creek lower watershed. Photo source: NAPL, September 1, 1952.	11
Figure 2-4	1867 photograph of Richfield and Williams Creek. Source: BC Archives: Item A-04046 – Richfield, Williams Creek, Cariboo.	12
Figure 2-5	1868 photograph looking downstream to Williams Creek and the Town of Barkerville..	13
Figure 2-6	1897 photograph of Barkerville and Williams Creek. Conklin Gulch is visible in the background. Source: BC Archives, Item I-55171 – Barkerville.	13
Figure 2-7	Regional setting for the Cariboo Gold Project.....	17
Figure 2-8	Current and proposed infrastructure for the Cariboo Gold Project.	18
Figure 2-9	Proposed surface infrastructure for the Mine Site Complex, Cariboo Gold Project (from BGM, October 2019).	19
Figure 3-1	Generalized bedrock geology map of the project area.	24
Figure 3-2	Geologic time periods.	24
Figure 3-3	Historical (1961 to 1990) data (Wang et al., 2016): (A) MAT, (B) MAP, and (C) PAS across the Willow River watershed near Wells, BC.	26
Figure 3-4	Occurrence date of annual instantaneous peak flows at WSC station <i>Little Swift River at the Mouth</i> (08KE024).	27
Figure 3-5	Distribution of daily flows over the period 1971-2026 from <i>Little Swift River at the Mouth</i> (08KE024).	28
Figure 3-6	Average daily flow at Willow River hydrometric station (H16) between 2016 and 2018 (after Golder, April 12, 2022).	29
Figure 3-7	Average daily flow for Willow River at Wells between 2006 and 2010.	30
Figure 4-1	Main Street of Barkerville during a flood on June 11, 1911..	33
Figure 4-2	Williams Creek flood breaking through the bulkhead.....	34
Figure 4-3	View looking north at the south end of the Main Street/service road loop.	35
Figure 4-4	View looking north of flood along Main Street in Barkerville.	36
Figure 4-5	Upstream view of Williams Creek from the footbridge.	37
Figure 4-6	Downstream view of Williams Creek at confluence with Conklin Gulch.	38

Figure 4-7	October 2001 photographs of A) barrier and grade in Richfield Road and B) lock blocks located downslope of the change in road grade.	39
Figure 4-8	Richfield Road realignment northeast of Stouts Gulch crossing.	40
Figure 4-9	Downstream view of Stouts Gulch overflow culvert crossing of Richfield Road. BGC photograph of June 23, 2025.	41
Figure 4-10	McElhanney design drawing for pedestrian bridge replacement.	42
Figure 4-11	Upstream view of Prosperpine Bridge crossing of Williams Creek at entrance to historic Barkerville Town. BGC photograph of June 24, 2025. ...	42
Figure 4-12	BGC June 23, 2025 drone photograph of Barkerville wastewater lagoons. ...	45
Figure 4-13	Sketch of the Lagoon dike and Airstrip dike (from Galbraith, 2001).	47
Figure 4-14	Downstream view of Williams Creek culvert crossing of Bowron Lake Road. BGC photograph of June 24, 2025.	49
Figure 4-15	Upstream view of Williams Creek at One-Mile Road bridge crossing. BGC photograph of June 23, 2025.	50
Figure 4-16	Google Earth image (September 3, 2024) of Groundhog Lake.	51
Figure 4-17	Two men and two women standing on the dam at Groundhog Lake.	52
Figure 4-18	From the Jack of Clubs bridge looking downstream (northeast) towards the aerial sewer crossing. BGC photograph of May 28, 2024.	54
Figure 4-19	From the left (west) bank of Willow River looking upstream (south) towards the piers of the Highway 26 bridge.	55
Figure 4-20	Riprap placement on the left (west) bank of Willow River immediately downstream of the Highway 26 bridge.	55
Figure 4-21	Looking along Lowhee Creek upstream (east) towards the BGM bridge.	56
Figure 4-22	Upstream view of pedestrian bridge crossing of Williams Creek.	57
Figure 4-23	Upstream view of pedestrian bridge crossing of Willow River.	57
Figure 4-24	Overview of the Lowhee Creek fan. Roads and building footprints retrieved from OpenStreetMap plugin in QGIS.	59
Figure 4-25	Downstream view of Lowhee Creek from the BGM bridge.	60
Figure 4-26	Modelled Lowhee Creek flow velocities (blue to red) during the design flood event.	60
Figure 5-1	1923 map of Lowhee Creek and Jack of Clubs Lake area.	61
Figure 5-2	SoilTech borehole locations on the alluvial fan of Lowhee Creek.	62
Figure 5-3	1940 photograph looking toward South Wells and Lowhee Creek alluvial fan.	64
Figure 5-4	Geologic cross-sections through the Lowhee Creek fan.	65
Figure 5-5	Archival photographs of the Clarke sawmill and the original outlet of Jack of Clubs Lake.	66
Figure 5-6	Excerpt of 1886 map of Wells area.	67

Figure 5-7	Willow River channel profile downstream of Jack of Clubs Lake using 2021 lidar data.	68
Figure 5-8	Looking downstream at the Lower Watershed reach with exposed terrace walls. Photo source: BGC, August 7, 2024.....	70
Figure 5-9	Looking upstream at a remnant dam and existing woody debris jam. Photo source: BGC, August 7, 2024.....	71
Figure 5-10	General arrangement of the Lowhee Creek Phase 2c sediment basin (BGC, September 19, 2025).	75
Figure 5-11	Williams Creek channel profile downstream of Stouts Gulch using 2021 lidar data.	81
Figure 5-12	Williams Creek bed texture and transport capacity downstream of Bowron Lake Road.	82
Figure 6-1	Delineation of watersheds and sub-watersheds (dashed lines) that outlet to Willow River.	84
Figure 6-2	Digital elevation model and model extent for the study area.	90
Figure 6-3	Manning's roughness layer defined for the model domain.	92
Figure 6-4	Example of mesh developed for the HEC-RAS model in the vicinity of Barkerville. ESRI satellite image of August 2024.....	93
Figure 7-1	Hydrographs for the climate change adjusted 500-year rain-on-snow scenario across the study area.	97
Figure 7-2	Modelled maximum (for all scenarios) water surface elevation at Barkerville Prosperpine Bridge (cross-section A-A').	98
Figure 7-3	Modelled maximum (for all scenarios) water surface elevation at Barkerville Lagoons (cross-section B-B'). View is looking downstream.....	99
Figure 7-4	Modelled maximum (for all scenarios) water surface elevation at Willow River at Highway 26 (cross-section C-C'). View is looking downstream.....	100
Figure 7-5	Lowhee Creek flow direction at its confluence with the Willow River.....	101
Figure 7-6	Modelled maximum (for all scenarios) water surface elevation at Wells Lift Station (cross-section D-D'). View is looking downstream.....	102
Figure 7-7	Maximum flow depth of the 500-year rain-on-snow with climate adjustment along Highway 26 centreline.	108
Figure 8-1	Change in WSE along Williams Creek centreline.	110
Figure 8-2	Change in velocity along Williams Creek centreline.	110
Figure 8-3	Change in WSE along Willow River centreline.	111
Figure 8-4	Change in velocity along Willow River centreline.	111
Figure 8-5	Model downstream boundary for evaluation of possible flow path to Slough Creek from Jack of Club Lake.	113

LIST OF DRAWINGS

- Drawing 01 Regional setting
- Drawing 02 Area of interest for regional flood hazard assessment
- Drawing 03 Existing site conditions in valley bottom
- Drawing 04 2021 hillshade DEM of Barkerville
- Drawing 05 2021 hillshade DEM of north Barkerville
- Drawing 06 2021 hillshade DEM of Wells
- Drawing 07 1952 georeferenced air photograph of Wells area
- Drawing 08 1952 georeferenced air photograph of Barkerville area
- Drawing 09 Historical channel changes of Williams Creek at Barkerville
- Drawing 10 Historical channel changes of Williams Creek at north Barkerville
- Drawing 11 Flood hazard extent maps

LIST OF APPENDICES

- Appendix A Inflow Hydrographs

1.0 INTRODUCTION

1.1 Background

The District of Wells (DoW) is located in central British Columbia within the Cariboo Regional District (CRD). Situated along BC Highway 26, the DoW covers an area of approximately 168 km² and has a population of just over 200 people (Drawing 01). The region's economy thrives on tourism, forestry, and mining, which attract an additional 200 to 3,500 people depending on the season. Small businesses play a crucial role in supporting these industries. Major changes are also coming to the community, as Osisko Development Corporation (Osisko) has been granted an operating permit by the provincial government for the underground Cariboo Gold Mine and construction has commenced.

The DoW is renowned for its rich history, vibrant art scene, and adventurous spirit, drawing many visitors each year. Barkerville Historic Town & Park, a world-class tourist attraction, borders the DoW. Moreover, it serves as the gateway to Bowron Lake Provincial Park and Cariboo Mountains Provincial Park, offering essential services to the surrounding communities. The DoW lies within the Traditional Territory of the Lhtako Dene Nation.

In March 2024, BGC Engineering Inc. (BGC) supported the DoW, with the Lhtako First Nation as a regional partner, in the development of a Union of BC Municipalities (UBCM) Disaster Risk Reduction-Climate Adaptation (DRR-CA) application for Category 1, 2 and 3 funding. This funding provides support for communities and other applicants to reduce risks associated with natural hazards and climate-related risks through the development of foundational knowledge of natural hazards and associated risks under a changing climate, as well as the development of effective strategies to prepare for, mitigate and adapt to the identified risks.

The DoW and Lhtako First Nation subsequently received funding for a Category 1 study, which is the development of a Regional Flood Hazard Assessment (RFHA) for four key watersheds within the DoW: Jack of Clubs Creek, Lowhee Creek, Williams Creek, and the Willow River (Drawing 02). This report provides the results of that RFHA.

R. Radloff & Associates Inc. (Radloff) are administering the RFHA work on behalf of the DoW. As such, this work is being conducted under the Terms and Conditions laid out in the Professional Services Agreement between Radloff and BGC, dated January 19, 2024.

1.2 Level of Detail

The primary objective of this study is to characterize clear-water flood hazards ("hazards") in the DoW. The goal is to support decisions that prevent or reduce injury or loss of life, environmental damage, and economic loss due to clear-water floods. The assessment includes floods resulting from elevated discharge on creeks and rivers due to rainfall and snowmelt runoff.

The deliverables of this study include "base level" flood hazard maps for Williams Creek and portions of Willow River and "detailed" flood hazard maps for Lowhee Creek and Willow River (within the Wells townsite). While flood mapping studies are an important tool for developing

safe and resilient communities, detailed studies are expensive and time consuming and therefore undertaken only when there are recognized hazards.

Recognizing the cost of detailed flood mapping, organizations responsible for flood management in the USA have begun to consider less costly flood mapping at a screening level. The US Federal Emergency Management Agency (FEMA) refers to this level of assessment as “Base Level Engineering” (BLE) (FEMA, 2021) and it is here referred to as “base level” hazard mapping.

While not as accurate as detailed flood studies, “base level” flood hazard maps can be completed at far lower cost per area assessed (factor of 10 lower). A key aspect of “base level” flood hazard maps is that the topographic data used for hydraulic modelling are based on available digital elevation models that generally do not account for the full river bathymetry². As such, it is possible to complete mapping over much larger areas to support decision making. Where required, “base level” flood hazard maps can also be applied to serve as a basis for more detailed mapping in the future, given it is more efficient to refine the models than prepare detailed flood maps from scratch.

The original Category 1 grant application envisioned “base level” flood hazard maps for all four watercourses. However, the DoW has also received Category 3 funding for the construction of flood mitigation infrastructure along Lowhee Creek and Willow River. As part of that work, “detailed” flood hazard assessments have been generated for both watercourses, the results of which are reported here-in.

Table 1-1 clarifies these levels of detail in terms of their applicability to decision making. Each increased level of detail is a refinement of previous work. Effort (cost) increases exponentially with the level of detail required, and this phased approach will help the DoW make progress on flood management across multiple funding cycles, focusing effort on the highest priority areas.

² In cases, where lidar data are available, a significant component of the river bathymetry can be captured if the data were acquired during a period of low flow.

Table 1-1 Hazard assessment levels of detail.

Points of Comparison	Hazard Identification Maps	Flood Hazard Assessment and Maps	
		Base Level	Detailed
Applicability for decision making	Suitable for prioritization and definition of the outer boundary of hazard areas subject to subdivision regulation in Official Community Plans (OCPs)	Suitable for application in planning and policy, and emergency management; limited application for land development regulation, and mitigation planning.	Suitable for parcel scale risk management, including risk assessment and bylaw enforcement, hazard monitoring, and detailed emergency response and mitigation planning
Level of detail	Hazard boundary (hazard extent and attributes, but not mapped flow characteristics)	Hazard characteristics (flow velocity or depth) displayed within the hazard boundary	Hazard characteristics displayed within the hazard boundary
Relative level of effort for a given study area ⁽¹⁾	\$	\$\$	\$\$\$\$\$
Examples and application to this scope of work.	Floodplain identification map	Base level flood mapping; <u>provided in this study.</u>	Detailed flood mapping; <u>provided in this study</u> ⁽²⁾ .
Inputs	Desktop analyses	Desktop analyses, limited fieldwork	Desktop analyses, field surveys of bathymetry of hydraulic structures, and field surveys of geomorphic factors.
Hazard return periods considered	Single (to compare sites)	One or more return periods	Multiple return periods and scenarios
Qualitative/Quantitative	Relative, qualitative	Quantitative	Quantitative
Map Deliverables	Hazard boundaries	Hazard maps	Hazard maps
Applicable Guidelines	NRCan (2018)	NRCan (2018); FEMA (2021)	EGBC (2017, 2018)

Notes:

1. Relative price of mapping effort.
2. For selected areas only.

1.3 Scope of Work

BGC's scope of work is outlined in the proposed work plan (BGC, January 23, 2025). Radloff is administering the RFHA work on behalf of the DoW. As such, this work is being conducted under the Terms and Conditions laid out in the Professional Services Agreement between Radloff and BGC, dated January 19, 2024.

The scope of work included:

Background Review and Analysis

- Conduct a comprehensive review and synthesis of past reports and studies. Compile and review existing spatial datasets, including lidar and aerial imagery. Assess regional characteristics such as geology, vegetation, hydrology, and climate to establish a foundational understanding of the area.
- Identify key regional legacy, current and proposed human activities that have or may affect the hydrology in the study area, such as the proposed Cariboo Gold Project, past placer mining, water quality, and sediment contamination.
- Kick off meeting with the DoW to discuss the project goals, timeline and engagement.

Regional Assessment

- Develop an overview of historical flood-related hazards and existing flood mitigation within the project area to identify historical flood trends and key events, and to consider the effectiveness of current mitigation
- Undertake a flood frequency and climate change assessment to understand potential future impacts on flood patterns and frequencies.

Technical Study of Riverine Flooding

- Plan and execute an initial site visit to gather relevant onsite observations and collect data to be used in the flood hazard assessments
- Undertake hydrological assessments of the key watersheds under current and climate change conditions to determine design flows
- Conduct base level floodplain mapping using available lidar data
 - Develop hydraulic models (i.e., 2D HEC-RAS) of selected reaches in each of the four key watersheds
 - Calibrate, validate and perform sensitivity testing for each model.
- Produce flood hazard maps to visualize potential inundation areas.

Flood Hazard Threat Assessment

- Identify and catalog valued assets, such as buildings and critical infrastructure, within potentially flood exposed areas
- Assess hazard threat for each design flood.

The flood hazard maps support the DoW with flood hazard information that can inform land use planning, policy development, and emergency management planning.

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

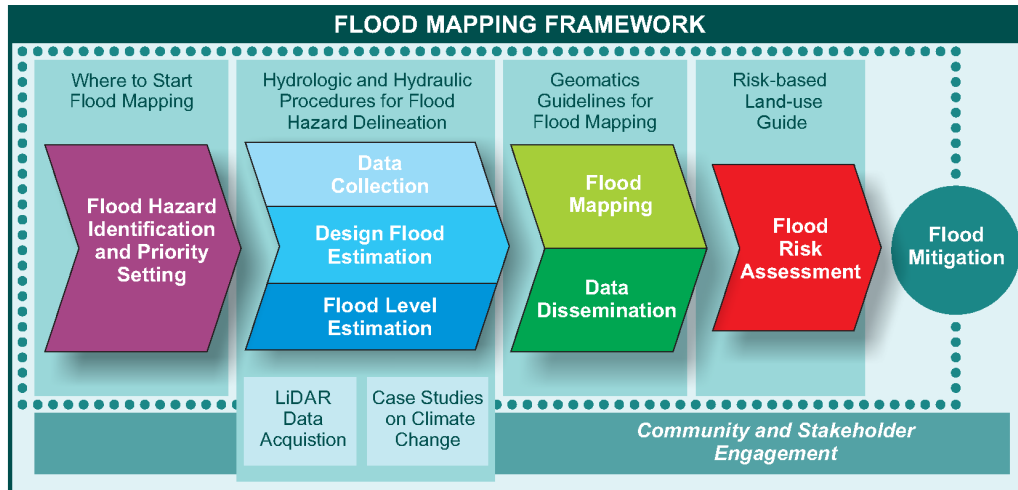


Figure 1-1 Federal flood mapping framework (NRCan, 2017).

The study scope examines flooding resulting from rainfall and snowmelt runoff. Other types of flood-generating hazards may exist in the region (such as dam breach, dike breach, landslide dam outbreak floods, sewer-related flooding, ice jam flooding or debris flows) but were not within the scope of the project herein. Potential secondary effects of high river levels, such as rising groundwater tables, were also not within the study scope.

2.0 MINING HISTORY

2.1 Placer Mining

The DoW is the epicentre of the Cariboo Gold Rush, which took place between 1861 and 1867. John Rose and Ranald McDonald were the first prospectors to discover substantial placer deposits within the Cariboo Goldfields (Brown & Ash, 2009). Late in the fall of 1860, they traversed up Keithley Creek and across the Snowshoe Plateau to Cunningham and Antler Creek, where they found some rich placer deposits at shallow depths (Figure 2-1). Immediately after their discovery a heavy snowfall forced them to retreat to Keithley Creek for supplies. In early 1861 they set off again by snowshoe and were followed by a number of other prospectors who suspected their success. As a result, a number of claims were staked and many settled in to mine Antler Creek for a period of several years. Other prospectors pushed beyond Antler in the spring of 1861 to the northwest. Discoveries were made that year on all the important creeks and many lesser ones (Figure 2-1). The largest discoveries of free gold were made at Williams, Lightning and Lowhee Creeks. Williams Creek and its tributaries were the richest and became the centre of mining operations for the district.

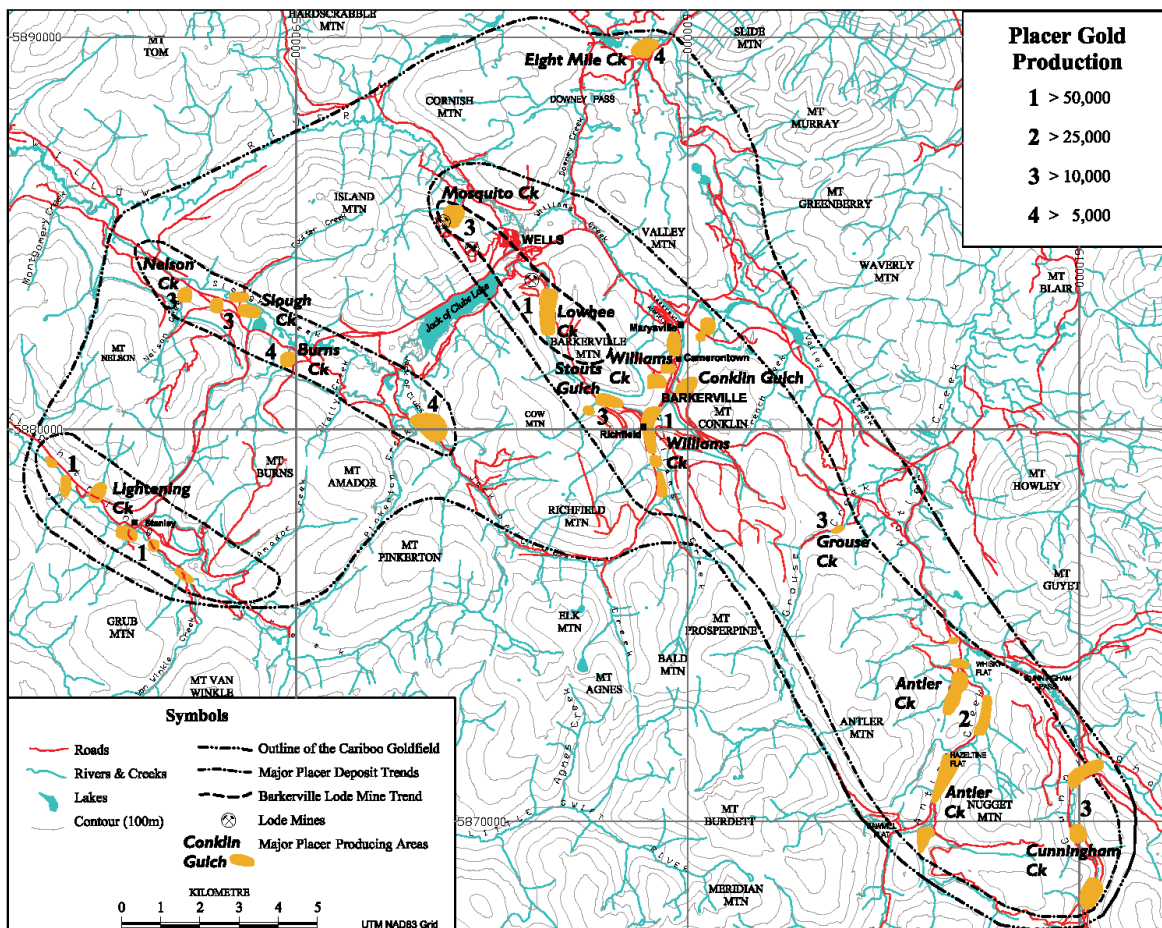


Figure 2-1 Topographic map showing the distribution of major placer deposits (from Brown & Ash, 2009).

Initial transport to the Cariboo Mining District was by a combination of foot, canoe or dog sled and the route taken was to Keithley Creek and over the Snowshoe Plateau. By 1860 gold returns from the area convinced Governor Douglas of the colonial administration to request help from the British government to construct a 650 km long wagon road (the Cariboo Road) through the Fraser Canyon to the Interior. A contingent of Royal Engineers was brought from Britain to survey the route from Yale to the administrative centre of the Cariboo. The work was begun by the army engineers in 1862, who completed the 2 most difficult stretches – 10 km from Yale to Boston Bar and 15 km from Cook's Ferry along the Thompson River. Much of the road through these sections had to be blasted through bedrock. The rest of the construction was let out to private contractors, and the road was opened in 1865. The current Highway 97 alignment largely follows the Cariboo Road route.

The initial discoveries were shallow, but mining at depths up to 20 m was soon accomplished as prospectors followed leads into deep gravel by excavating drifts³ into the surficial deposits. Water was an ongoing impediment to the drift mining, with insufficient flows to sluice in summer and too much flowing through the deep gravels, even in winter. Pumps were driven by water wheels until late in the 19th century, when steam engines and eventually diesels became available.

Significant placer mining took place on about a hundred stream beds in the district, fifteen of which produced in excess of 5,000 ounces of gold (Brown & Ash, 2009). Production before 1874, which include the most productive years of 1861 to 1867, was not accurately recorded. Placer gold production between 1874 and 1945, which was well recorded by banks and government, is tabulated by Holland (1950). Table 2-1 summarises placer gold production from creeks within the DoW boundaries for the period 1874-1945. Those records indicate that 201,000 ounces of placer gold were mined, which would be worth around \$800 million CAD at current prices (\$2,800 USD per ounce).

³ A drift mine is an underground mine that accesses mineral deposits by driving a horizontal or near-horizontal tunnel (a "drift") directly into the side of a hill or mountain where the deposit is exposed.

Table 2-1 Placer gold production (ounces) within the District of Wells between 1874 and 1945 (after Holland, 1950).

Year	Conklin Gulch	Emory Gulch	Jack of Clubs Creek	Lowhee Creek	McArthur Gulch	Mosquito Creek/Red Gulch	Stouts Gulch	Walker Gulch	Williams Creek
1874-1875	2,410			5,577			225		10,369
1876-1880	2,790		882	2,181		1,158	1,595	59	22,555
1881-1885	962		5,918	3,946		2,117	1,381		13,940
1886-1890	1,180			1,530		2,638			7,201
1891-1895				1,315		3,319	312		6,616
1896-1900				192		478			1,824
1901-1905				1,986		322	450		938
1906-1910				2,485		4,241	8,899		21,701
1911-1915				21,292		1,783	2,344		176
1916-1920				5,754				59	
1921-1925			104	4,603				174	
1926-1930				5,134			615		16
1931-1935				3,543		407			136
1936-1940		20		9,033	7	1,505		105	58
1941-1945		52	12	5,471	12	327	101	6	
Total	7,342	72	6,916	74,022	19	18,295	15,922	403	85,330

2.1.1 Mining Activity in Lowhee Creek

The first recorded mining activity on Lowhee Creek occurred in 1861, when Richard Willoughby, recovered approximately 3,000 ounces of gold from the fan of Lowhee Creek near the mouth ([Lowhee Creek – BC Gold Adventures](#)). Subsequent mining activities proceeded upstream into thick (tens of metres) sequences of gravels located along the entire length of the stream. Mining was first done at shallow depth in the channel bed near the mouth. As depth to the buried channel increased, upstream mining by drifting took place. Drift mining was eventually replaced by more efficient hydraulic mining. Hydraulic mining involved constructing a penstock (a wood and earthen reservoir of creek water) and installing a system of sectional steel pipes. This system created enough water pressure to blast away the hillside where the gold was entrained in the gravels. Miners used a large, counterweighted nozzle called a monitor to direct the piped flows. The water and gravel slurry was then directed into wooden troughs called sluice boxes. In the bottom of the box a series of riffles agitate the slurry of water and gravel, encouraging small particles of gold to fall out of solution (because they are heavier than sands and gravels). In this way gold can be captured while the waste material spills out of the end of the box. Placer mining eventually took place along most of the length of Lowhee Creek.

Archival photos of the placer operations from Lowhee Creek show the extraordinarily large volumes of sediment mined (Figure 2-2) as do historical air photos. Historical air photos also show constructed dams within the lower 500 m of the stream (Figure 2-3). The purpose of these dams is unknown, though based on anecdotal evidence from community members, the dams were likely constructed to retain sediment from the upstream hydraulic mining operations. Remnants of these dams are visible in modern imagery and on the ground.

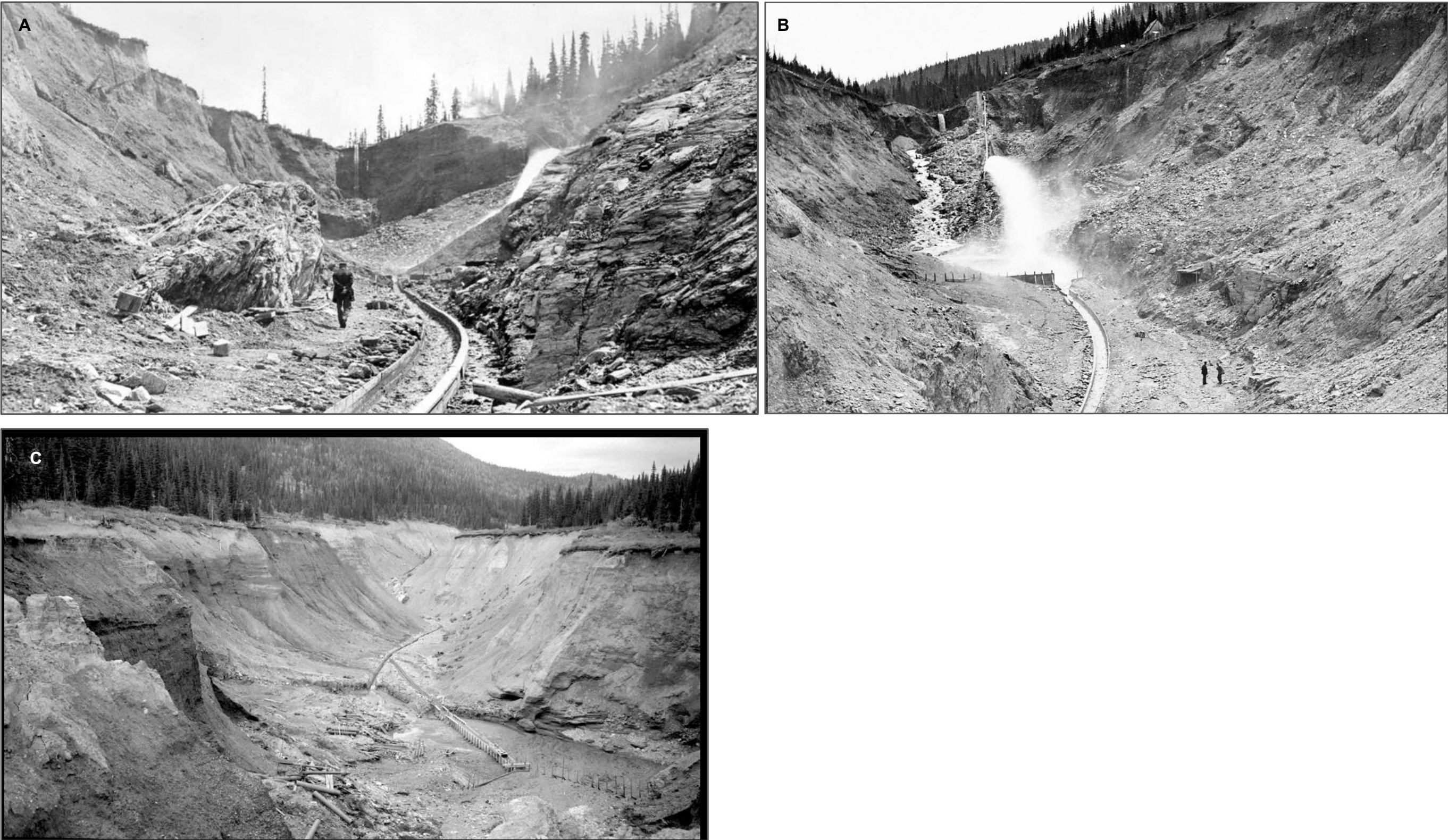


Figure 2-2 Archival photographs of placer operations on Lowhee Creek. Image A: 1916 (BC Archives, Item H-02768 – Lowhee Creek). Image B: 1912 (BC Archives, Item A-03835 – Lowhee Creek). Image C: 1946 (BC Archives, Item I-33439 – Old Lowhee Pit; Stout Creek).

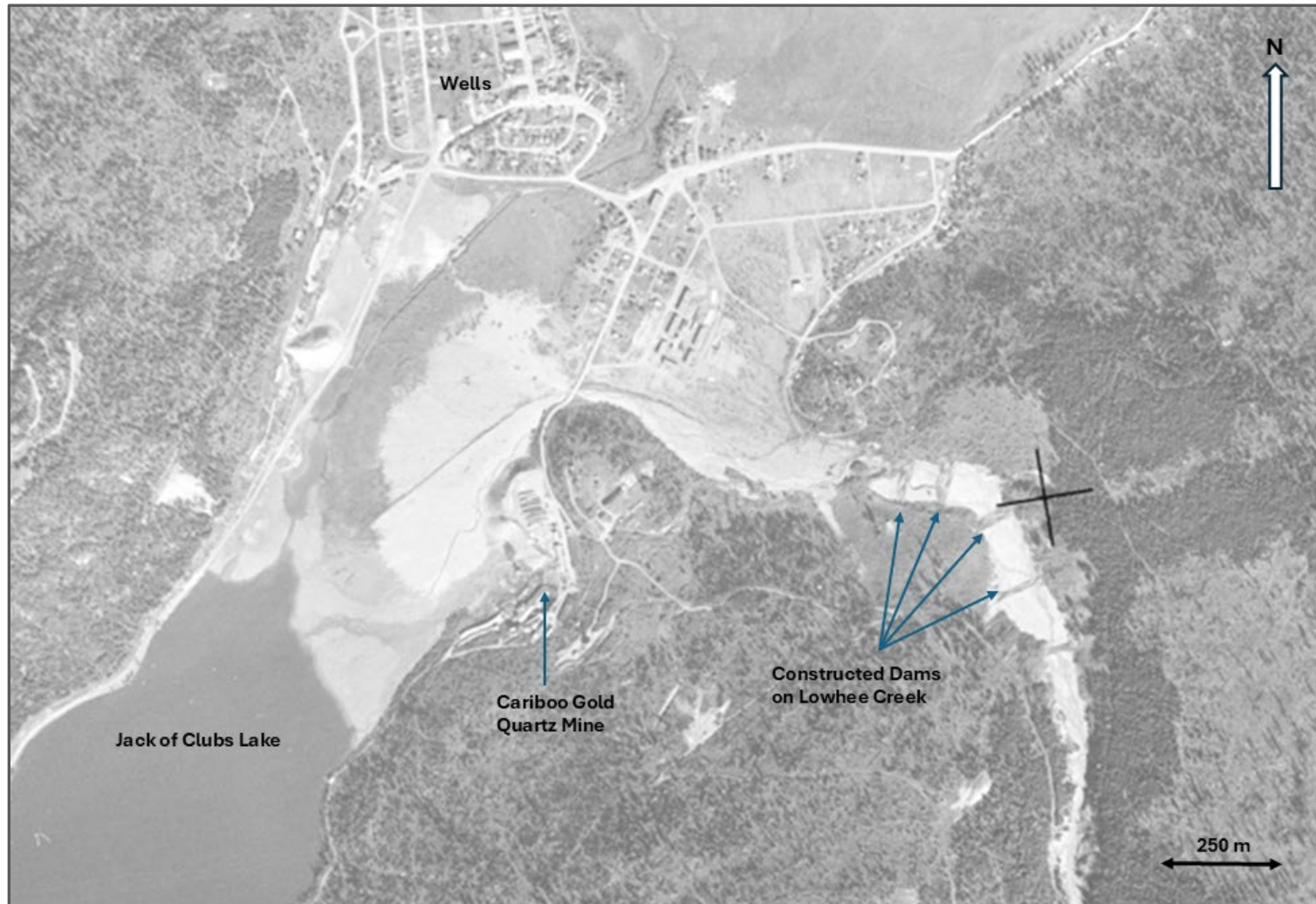


Figure 2-3 1952 airphoto showing constructed dams within the Lowhee Creek lower watershed. Photo source: NAPL⁴, September 1, 1952.

⁴ Roll Number A13524, Photo 084.

2.1.2 Williams Creek

Williams Creek was originally visited in 1861 by a party of miners headed by William Dietz who named the creek after their leader. They set to work mining the shallow ground in the vicinity of the confluence of Walker Gulch with Williams Creek. The town of Richfield quickly formed at that location, with Williams Creek becoming the epicentre of the Cariboo gold rush. Richfield had several saloons, a jail, a courthouse, two banks, a Roman Catholic church, a hotel, a post office and several stores (Figure 2-4). However, the gold around Richfield proved to be shallow and was mined out quickly, causing people to migrate further downstream as more prospectors arrived.



Figure 2-4 1867 photograph of Richfield and Williams Creek. Source: BC Archives: Item A-04046 – Richfield, Williams Creek, Cariboo.

The migrant prospectors included William Barker and John Cameron who worked the creek bed below Richfield. Working on separate claims, both prospectors found significant gold concentrations. Soon hundreds of men left Richfield and the towns of Barkerville (Figure 2-5, Figure 2-6) and Camerontown would spring up next to the discoveries. A fourth town, Marysville, formed where Williams Creek discharges from confinement into a swampy meadow. Marysville was mainly a community of residences for miners and businessmen.

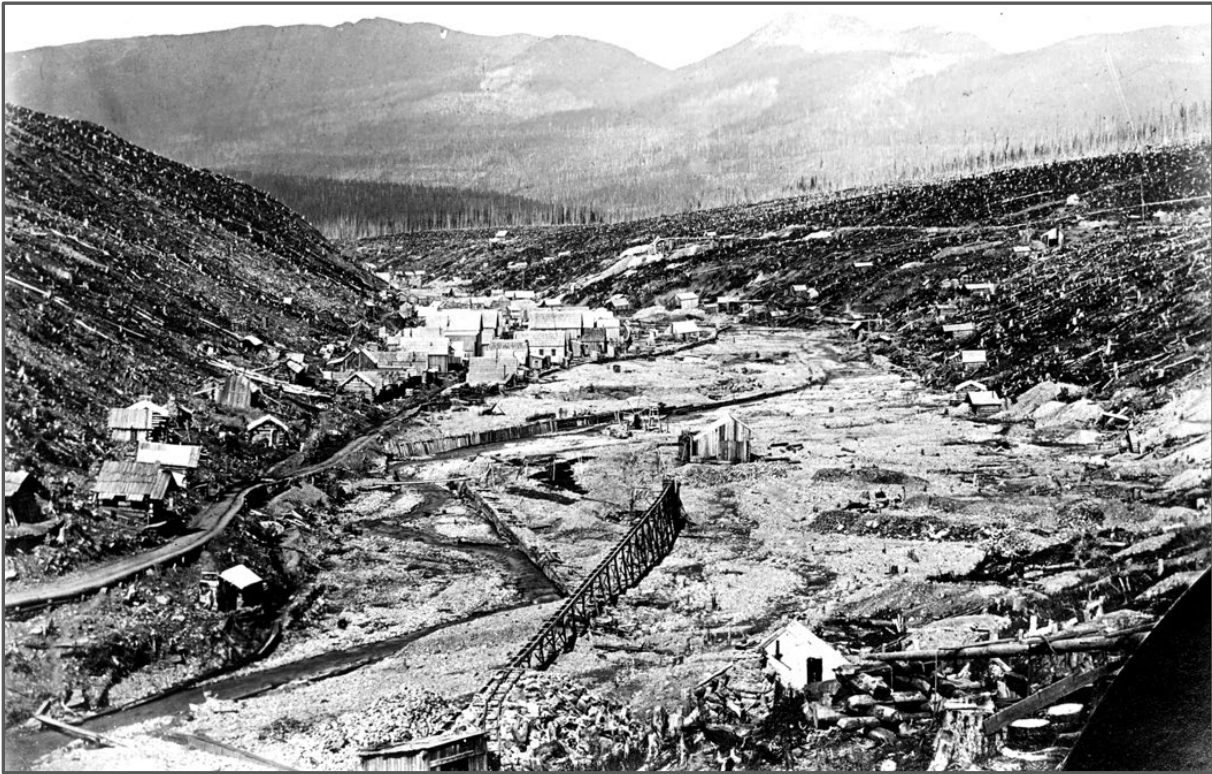


Figure 2-5 1868 photograph looking downstream to Williams Creek and the Town of Barkerville. Source: BC Archives, Item A-03748 – View on Williams Creek looking towards Barkerville.

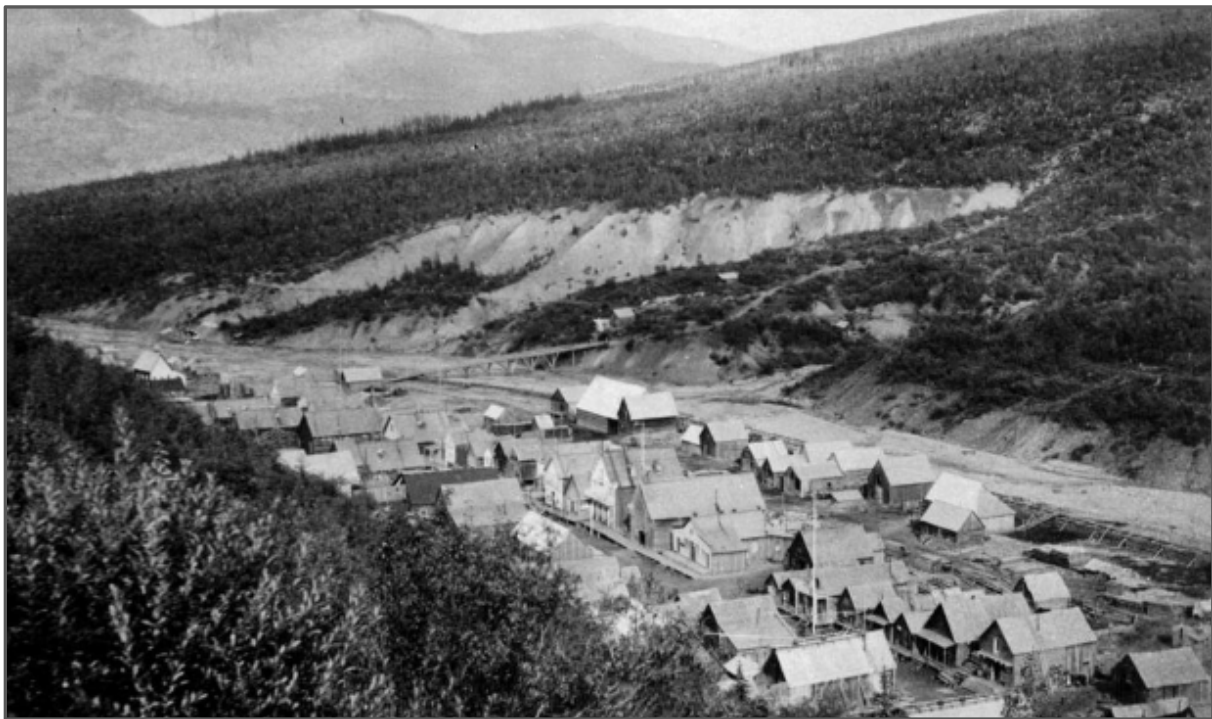


Figure 2-6 1897 photograph of Barkerville and Williams Creek. Conklin Gulch is visible in the background. Source: BC Archives, Item I-55171 – Barkerville.

By 1864, there was virtually one long street of buildings extending along the west bank of Williams Creek, from Richfield at the south end to Marysville at the north. One writer noted that the path alongside the creek was “a difficult one, over endless sluices, flumes, and ditches; across icy planks and logs.” (Sale, January 11, 2025). Because of its central location, Barkerville was the community which became the largest and lasted the longest. At its peak in the 1860s, it is estimated that the local population was on the order of 10,000 people. At least half of these inhabitants were miners working gold claims in the area while the rest were merchants and business people (Sale, January 11, 2025).

The earliest mining in the Barkerville area was at shallow depth. However, the miners soon found that the most gold was found 10 to 30 m or more below the surface. Thereafter, vertical shafts, shored by log cribbing, were sunk into the valley bottom. When bedrock was reached, drifts went out in several directions. The drifts were usually large enough so that a man could stand up to fill buckets with gravel. Water seeping into the shafts and drifts was an ongoing problem. Therefore, water wheels were often used to pump the water from the underground workings and lift up the mined gravels. Water would be fed into cup-like shelves at the top of the wheel, using water transported by a flume from a water source. The wheel would then turn a rocker arm that would pump the water from the bottom of the mine shaft and lift buckets of gravel to the surface. Eventually, hydraulic mining became the prominent mining method in the Barkerville area.

A large fire in September 1886 destroyed almost all the buildings in Barkerville. While the town was rebuilt quickly, it never regained its former size or prominence. Similar to Lowhee Creek, Williams Creek and its tributaries (Stouts Gulch, Emory Gulch, Walker Gulch, and Conklin Gulch) were ultimately hydraulically mined, drastically widening these watercourses.

2.2 Underground Mines

The search for bedrock sources followed the discovery of the placer gold. In the late 1920s, lodes were developed underground on either side of Jack of Clubs Lake which later proved economically feasible. Two mines came on stream, the Cariboo Gold Quartz Mine on Cow Mountain southeast of the lake in 1933 and Island Mountain Mine⁵ north of the lake in 1934. The former mine was founded by Fred Wells, a mining engineer and hard rock miner who purchased a claim on Lowhee Creek in 1927 and began tunnelling into Cow Mountain. At its peak in the 1940s, Wells had a population of 4,500.

The Cariboo Gold Quartz Mine purchased the Island Mountain Mine in 1954. The former mine operated until 1959, while the latter operated until 1967 before closing due to unfavourable economics. In 1980, higher gold prices allowed the Mosquito Creek Gold Mine to open further northwest on Island Mountain. This mine operated intermittently until 1987.

While the Cariboo Gold Quartz Mine and Island Mountain Mine do not connect below Jack of Clubs Lake, all the mines are on the same northwesterly trend. Between 1933 and 1987 the

⁵ Also known as Aurum Mine.

three mines produced 1.23 million ounces of gold and 101,439 ounces of silver (Brown & Ash, 2009), which would be worth more than \$5 billion CAD at current prices (\$2,800 USD per ounce). During the operation of the Cariboo Gold Quartz Mine, approximately 2.65 million t of flotation mill tailings were deposited into the northeastern end of Jack of Clubs Lake near its outlet into Willow River, filling approximately 30 ha of the original lake area (SNC Lavalin, 2011).

More recently in 2011, Barkerville Gold Mines Ltd. (BGM) received a *Mines Act* permit to develop an open pit gold mine at Bonanza Ledge, enabling a four-year mining operation. The Bonanza Ledge Mine is located at the divide between Lowhee Creek and Stouts Gulch, 3.5 km south-southeast of the community of Wells and 2 km southeast of the main workings of the past-producing Cariboo Gold Quartz Mine. Production at the Bonanza Ledge open pit mine started in March 2014, with ore processed at the company's Quesnel River (QR) Mill. The QR Mill is approximately 60 km southeast of Quesnel and 110 km from Wells (by road).

The Bonanza Ledge Mine saw limited production as an open pit mine between March 2014 and June 2015, before being placed into care and maintenance. In December 2016, BGM applied for a permit to mine the remaining resource by underground methods. BGM restarted the mine in 2017 as an underground operation, with an estimated life span of 3.5 years. However, the mine was again placed on care and maintenance in December 2018. Mining resumed in mid-2019 before being placed on care and maintenance in June 2022 (Osisko Development Corp. [Osisko], December 30, 2022).

Much of the recent activity at the Bonanza Ledge Mine has been to allow for the continuity of mining while permitting was sought for the larger Cariboo Gold Project.

2.3 Cariboo Gold Project

The Cariboo Gold Project, owned by Osisko⁶, is an advanced stage feasibility level gold project that would exploit gold resources around the historic underground mines that surround the community of Wells. The Project consists of three main zones (Cow, Shaft, and Valley) with two smaller satellite zones (Lowhee and Mosquito). The rate of exploitation of each deposit will change over time. The life-of-mine (LOM) plan has a 12-year mine life. Ore production will begin at 1,500 tpd for 2.75 years (Phase 1) and will ramp up to 4,900 tpd for 8.5 years (Phase 2). The overall mine plan comprises 16.7 million tonnes (Mt) of ore that will be processed with an average grade of 3.8 grams per tonne (g/t) gold. The mine will produce 7.1 Mt of waste from the development over the LOM.

The Project includes the following key components (Figure 2-7, Figure 2-8):

- Underground extraction infrastructure including two access portals (Valley and Cow Mountain), conveyor and crushing facility
- Access roads
- A Mine Site Complex including (Figure 2-9):

⁶ Osisko's predecessor of the Project was BGM. In December 2020, Osisko was officially created, and ownership of BGM was transferred from Osisko Gold Royalties to Osisko. BGM is a 100% owned subsidiary of Osisko.

- A services building containing a surface concentrator, paste backfill plant, and other related infrastructure
 - Electrical substation
 - Camp
 - Water Treatment Plant (WTP)
 - Sediment Pond for containment of contact water.
- A waste rock storage facility (WRSF) located at the Bonanza Ledge Site and associated water management structures (including a sediment pond), access roads, and ancillary infrastructure
- A new 69 kV Transmission Line, including access roads and a substation
- Upgrades to the existing QR Mill, water supply and management structures and facilities, access roads, ore storage, expansion of the existing WTP, and construction of a tailings dewatering plant and a filtered stack tailings storage facility (FSTSF).

Ore produced by the mine will undergo crushing, ore sorting, milling, flotation, and dewatering before being trucked as a concentrate along Highway 26 and the 500 Nyland Lake Forest Service Road to the QR Mill for the final stage of processing. A new highway bypass would be built west of Wells from Highway 26 to the Mine Site (across Willow River) to enable traffic to exit the highway before the community.

There will be two portals accessing underground ramps. The existing Cow Portal⁷, on the north side of Lowhee Creek, will allow access to the Lowhee, Shaft, and Mosquito zones in the earlier stages of the Project. The Valley portal will be built during the expansion to develop the Main ramp connecting the previous zones to the new Cow and Valley Zones. The Valley Portal will be used as the main services access.

The Project is designed in phases: in Phase 1, a 1,500 tpd crushing and ore sorting plant will be built at the Bonanza Ledge Site; and in Phase 2, a pre-concentrator designed to have a capacity of 4,900 tpd will be built at the Mine Site Complex. The QR Mill will ramp down from 859 tpd in Phase 1 to 644 tpd in Phase 2. The pre-concentration steps in Phase 2 are designed to produce less concentrate at higher grades. Phase 2 will include the construction of an underground crushing system, to be located below the services building. Ore will be brought to the crusher by underground trucks from all mining zones.

Ore will be brought to the surface using a vertical conveyor to be pre-concentrated by sorting and flotation. The material rejected by the sorter will be transferred back underground and either used as backfill material or hauled to the Bonanza Ledge WRSF, which will have capacity to store 8.5 Mt (4.25 million m³) of waste material from the Project.

⁷ Note that the EA application of Osisko proposed construction of two portals: the Valley and Island Mountain portals. The latter portal is located directly west of Wells on the north side of the Willow River (Figure 2-8). However, the recent feasibility study has replaced the Island Mountain Portal with the Cow Portal. The Cow Portal was completed by Osisko in December 2021 to access and develop a bulk sample at the Cow Mountain portion of the mineral resource.

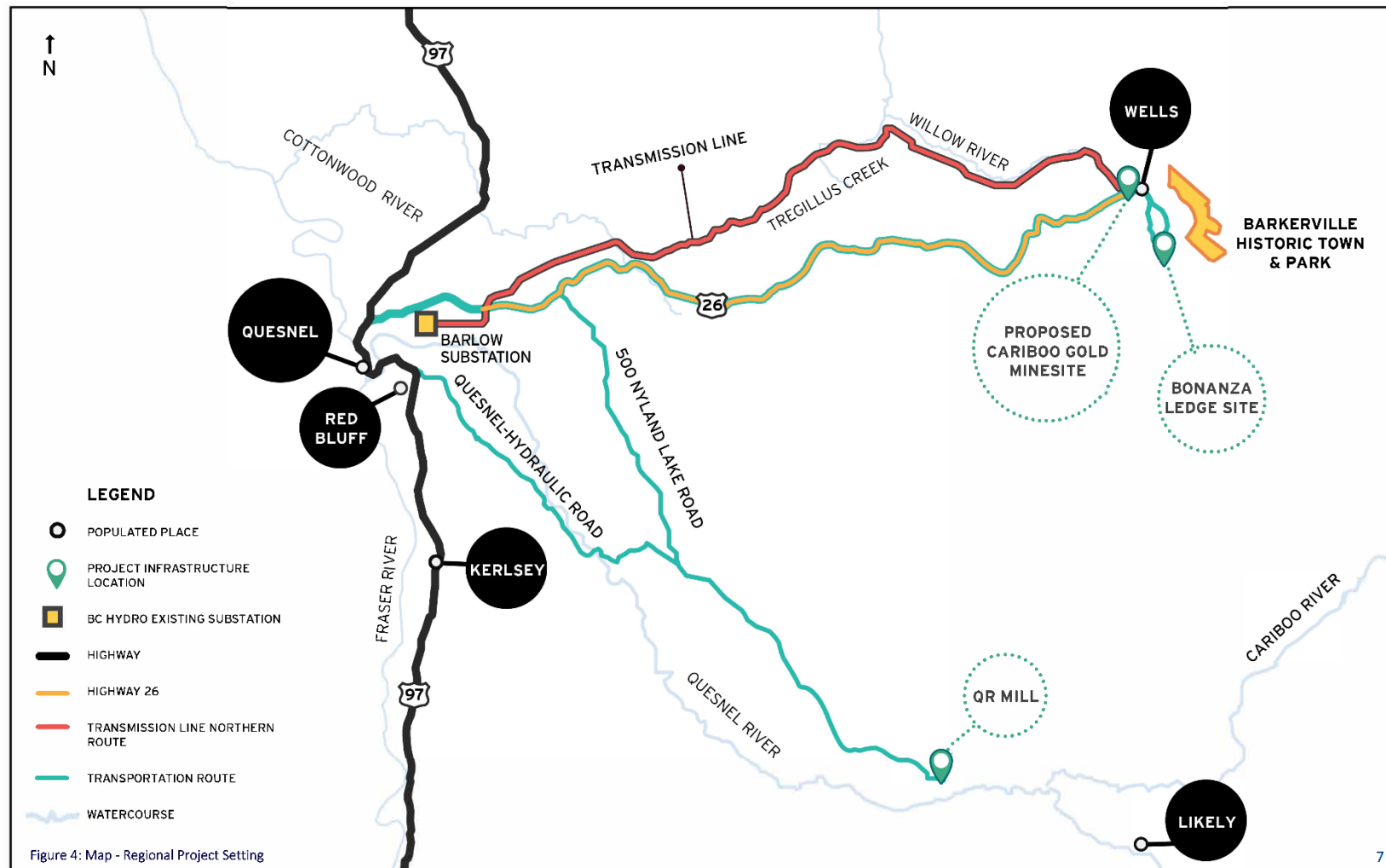


Figure 2-7 Regional setting for the Cariboo Gold Project (BC Environmental Assessment Office, September 2023).

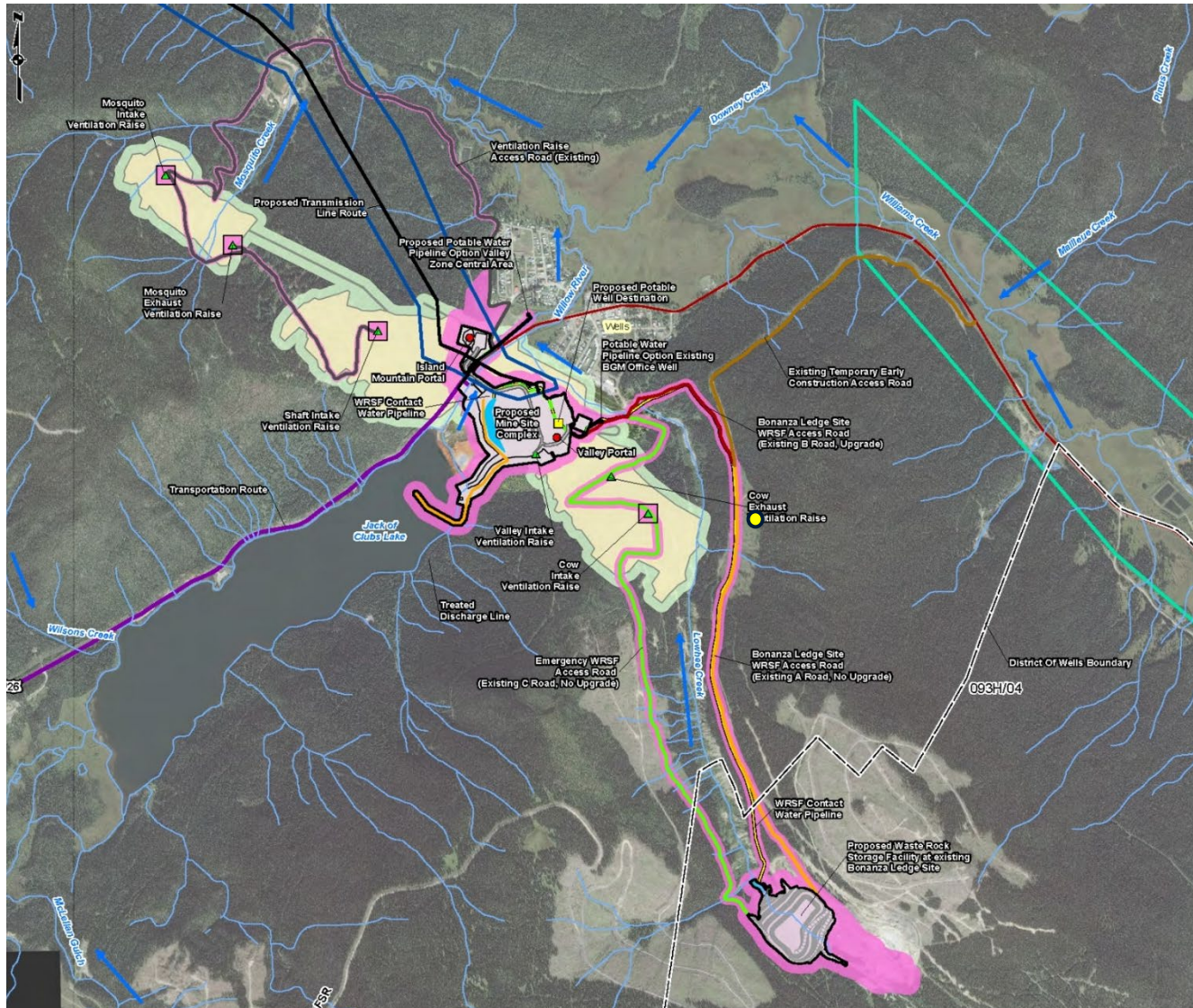


Figure 2-8 Current and proposed infrastructure for the Cariboo Gold Project (Osisko, December 30, 2022). The yellow dot is the approximate location of the Cow Portal, which is intended to replace the Island Mountain Portal shown on this figure.

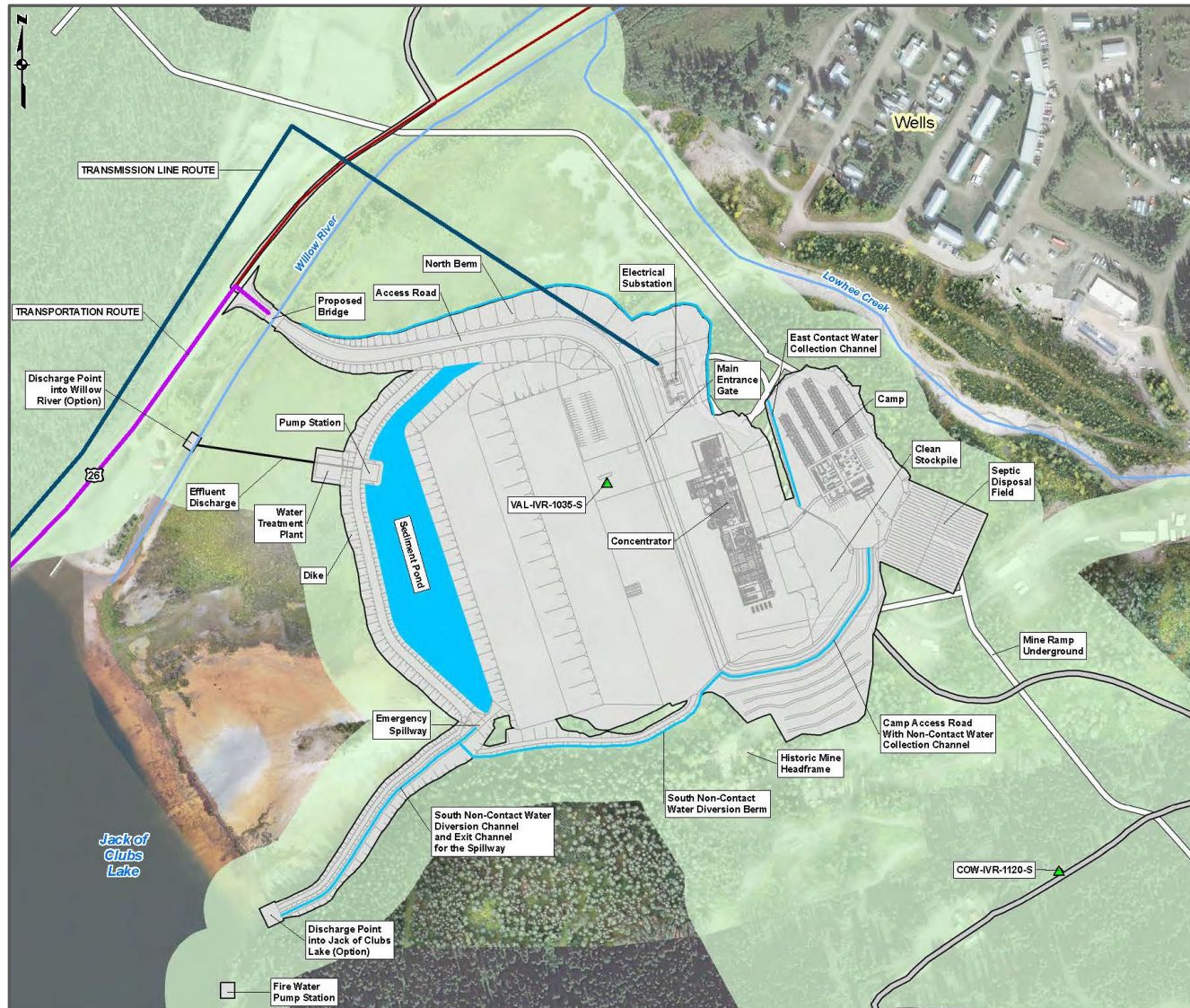


Figure 2-9 Proposed surface infrastructure for the Mine Site Complex, Cariboo Gold Project (from BGM, October 2019).

A combination of cemented rock fill (CRF), uncemented rock fill (URF), and tailings paste backfill will be used for stope backfilling. The use of CRF and URF as backfill will help to reduce the amount of waste rock and ore rejects to be stored at the WRSF. A dedicated tailings storage facility (TSF) is not required at the Mine Site Complex. The sorting of the mined ore prior to milling and flotation, as well as the use of flotation tailings for paste backfill, will eliminate the requirement for a TSF at or near the Mine Site Complex.

The water management facilities required at the Mine Site include the following:

- Two sediment ponds, one located at the Mine Site Complex and the other at the Bonanza Ledge Site, which will serve as central collection ponds for contact water at each site
- Contact and non-contact water conveyance infrastructure, including diversion berms, collection channels, pumps, and pipelines to separately manage these flows
- A mine dewatering system for the historic underground mining areas
- A WTP at the Mine Site Complex that includes a conventional high-density sludge (HDS) lime neutralization, ferric coprecipitation, organo-sulphide precipitation, nitrifying moving bed bioreactor (MBBR), and denitrifying fluidized bed reactor (FBR) system.

The Project received an Environmental Assessment (EA) Certificate on October 10, 2023, in accordance with the BC Environmental Assessment Act (2018). Receipt of the EA Certificate concluded the EA process for the Project, which was launched in October 2019. Osisko received the BC *Mines Act* permits on November 20, 2024, which grant the Company the ability to proceed with the construction and operation of the project. *Environmental Management Act* permits were received in December 2024.

Initial construction of the Mine commenced in late 2025.

3.0 STUDY AREA CHARACTERIZATION

The following sections provide a characterization of the study area including a description of the watershed, glacial history, geology, climate, and hydrologic regime.

3.1 Watershed Description

While the DoW covers an area of 168 km², the area of interest for this RFHA is the 108 km² watershed area of Willow River as delineated on Drawing 02 and the valley bottom areas prone to overland flooding (Drawing 03A and B). Major tributaries of Willow River include Jack of Clubs Creek and Williams Creek. Elevations in the study area range from approximately 1890 m to 1195 m.

Jack of Clubs Creek occupies the west half of the study area and drains from south to north, with its headwaters at an elevation of 1890 m near Groundhog Lake (Drawing 02). Jack of Clubs Creek has a watershed area of 34 km² where it discharges into Jack of Clubs Lake. The lake is approximately 2.5 km long (east-west) and 0.5 km wide (north-south). Inflows to the lake consist of flows from Jack of Clubs Creek at the west end as well as runoff from the adjacent hillslopes. During large flows, a portion of Jack of Clubs Creek may discharge into Slough Creek, which flows in a northwest direction (i.e., away from the Jack of Clubs Lake) (Drawing 02). The drainage area at the lake outlet is approximately 44.5 km².

The channel between the outlet of Jack of Clubs Lake and the confluence with Williams Creek is referred to as both Jack of Clubs Creek and Willow River in various reports and maps. For the purposes of this report, this reach is referred to as Willow River. The Willow River continues downstream of the confluence with Williams Creek. The initial reach of Willow River divides the Town of Wells into north and south sections, with Highway 26 crossing the river about 1 km downstream from the Jack of Clubs Lake outlet. Lowhee Creek discharges into the Willow River, on its right bank, about 250 m upstream of the highway crossing. Lowhee Creek drains from south to north and has an approximate watershed area of 5.8 km². As noted in Section 2.1.1, Lowhee Creek was subject to intensive placer mining in the late 1800s and early 1900s.

About 750 m downstream of the Highway 26 crossing, Williams Creek merges with Willow River. Williams Creek covers most of the eastern half of the study area and, like Jack of Clubs Creek, it generally flows from south to north (Drawing 02). Its headwaters are also at an approximate elevation of 1890 m. From its headwaters, Williams Creek flows for about 7 km before reaching the historic town of Barkerville. Here, at the confluence with Stouts Gulch (watershed area = 4.1 km²), the creek has a watershed area of 19.8 km². A couple of hundred meters further downstream, Conklin Gulch (area = 4.7 km²) discharges into Williams Creek on its right bank.

Downstream of Barkerville, Williams Creek turns to the northwest and occupies an increasingly wide valley bottom that is up to 800 m wide at the confluence with Willow River. Here, Williams Creek has an approximate drainage area of 54 km². Significant downstream tributaries of Williams Creek include Downey Creek (area = 9.4 km²) and Valley Creek (area = 6.2 km²). Valley Creek enters Williams Creek on its right bank upstream of the Barkerville sewage

lagoons, while Downey Creek discharges into Williams Creek about 1 km northeast of the confluence with Willow River.

3.2 Glacial History

British Columbia was almost completely glaciated during the Late Wisconsinan⁸ glacial stage, beginning about 29,000 years before present (BP) (Fulton et al., 2004). At the climax of this glaciation⁹, almost all of the province was covered by the Cordilleran Ice Sheet. This ice sheet was a coalescent mass of glaciers with complex flow directions. The Cariboo Mountains were an important source area for the ice sheet. Glacial deposits suggest a northeastward flow of ice across central British Columbia from the Coast Mountains toward the Rocky Mountains (Tipper, 1971). The Wells/Barkerville area was covered by ice flowing west from the Cariboo Mountains and deflected to the northwest near Cottonwood by ice flowing from the Coast Mountains.

An earlier iteration of the Cordilleran Ice Sheet also occurred in the early Wisconsinan, although the exact date of this penultimate glaciation is unknown (Ryder et al., 1991). Once the glaciers of the penultimate glaciation had receded, the region remained ice-free from about 59,000 to 29,000 years ago (Eyles & Kocsis, 1988; Clague et al., 1990). During this time, valleys were incised by rivers to levels similar to present (Clague, 1991) and the climate was colder than present-day conditions (Clague et al., 1990).

During the latest glaciation (the Fraser Glaciation), thick layers of glacial, glaciofluvial and glaciolacustrine sediments were deposited over the landscape. River valleys in the general area are filled with many tens of metres of glacial sediments. At the confluence of Mosquito Creek and the Willow River, the sediment thickness is reportedly greater than 30 m (Eyles & Kocsis, 1988). Deglaciation began about 14,500 years BP and the retreat was rapid with the ice cover mostly melted by 11,500 years BP. Downwasting of the glaciers resulted in gradual lowering of the ice surface which led to systematic exposure of higher areas, disrupting glacier flow patterns and starving thick ice masses that remained in low-lying areas (Fulton et al., 2004).

Early placer miners along Lowhee Creek, Stouts Gulch, and Williams Creek initially found gold in shallow surface gravels in the valley bottom above an uneconomic hard layer the miners referred to as blue clay. Miners initially mistook the blue clay for bedrock, when in fact it was a basal till¹⁰ from the Fraser Glaciation (Eyles & Kocsis, 1988). The early miners eventually broke through the basal till and found significant gold in underlying fluvial gravels. These underlying gravels were deposited during the interglacial period between the penultimate and Fraser glaciations.

⁸ The Wisconsinan glacial stage began about 100,000 to 75,000 years ago and ended about 11,000 years BP.

⁹ In British Columbia this glaciation is referred to as the Fraser Glaciation.

¹⁰ A basal till is glacial sediment that was carried at the very bottom (base) of a glacier and deposited directly beneath the ice. Basal tills are typically very dense given the overlying weight of the thick ice.

3.3 Geology

The DoW lies within the Interior Plateau physiographic region (Holland, 1976). The Interior Plateau is one of the major physiographic divisions of British Columbia, with a length of 900 km and a maximum width of 380 km. It is encircled by mountains, being flanked on the west by the Coast and Cascade Mountains, on the north by the Skeena and Omineca Mountains, and on the east and southwest by the Rocky Mountains and Columbia Mountains. The Interior Plateau is divided into seven subdivisions, with the Quesnel Highlands subdivision intersecting the project area. The Quesnel Highlands are situated on the eastern boundary of the Interior Plateau and lie to the west of the Cariboo Mountains. The highlands were once plateaus of moderate relief and have since been dissected, leaving upland regions that rise from approximately 1600 m in the west to 2100 m in the east (Holland, 1976).

The Cariboo Mountains are part of the Omineca Belt, which comprises a collage of allochthonous terranes¹¹ accreted to the ancient margin of the North American craton¹² (Struik, 1988). Within the project area, there are three major geologic units represented: the Cariboo, Slide Mountain, and Barkerville terranes (Bichler & Bobrowsky, 2001; Figure 3-1):

- Rocks of the Cariboo Terrane are characterized by clastic sedimentary rocks of an ancient passive continental margin typified by siltstone, sandstone, chert and shale.
- Slide Mountain rocks are characterized by oceanic marginal basin volcanic and sedimentary rocks.
- The Barkerville Terrane is an assemblage of marine clastic (sandstones, siltstones and shales) and volcanic rocks that have been intensely metamorphosed¹³. In the project area, this terrane is generally considered to be the most metamorphosed of the three terranes represented.

The main source of placer gold in the Cariboo District is the auriferous metasedimentary rocks of the Barkerville Terrane. Most of the gold was released from host rocks during a lengthy period of Tertiary weathering and denudation (Clague, 1991). This gold was concentrated in Miocene and Pliocene colluvial and alluvial sediments by mechanical processes and chemical precipitation (see Figure 3-2 for the age of these geologic time periods and epochs). During the Quaternary, streams and glaciers removed most Tertiary auriferous sediments, but much of the gold was redeposited in younger sediments (Clague, 1991). These younger sediments include the gold-rich gravels encountered by early placer miners beneath the basal till of the Fraser Glaciation.

¹¹ An allochthonous terrane is a crust fragment formed on a tectonic plate and accreted to crust lying on another plate.

¹² A craton is the ancient, stable interior portion of a continent, composed primarily of crystalline basement rocks that may be covered by younger sedimentary layers

¹³ Metamorphic rocks are rocks that have been transformed from pre-existing rocks under high heat, pressure, or chemically-active fluids without melting.

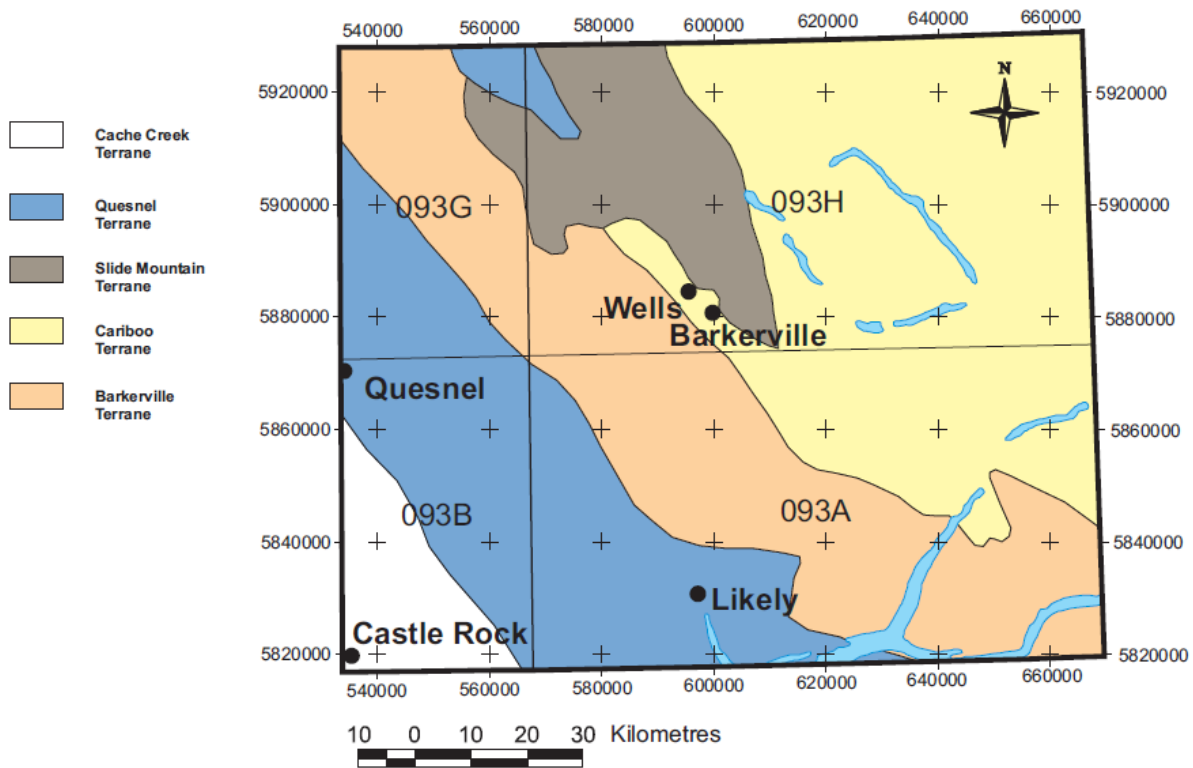


Figure 3-1 Generalized bedrock geology map of the project area (modified from Bichler & Bobrowsky, 2001).

Era	Period	Epoch	Age
Cenozoic	Quaternary	Holocene	0.01 Ma
		Pleistocene	1.8 Ma
	Tertiary	Pliocene	5 Ma
		Miocene	24 Ma
		Oligocene	34 Ma
		Eocene	55 Ma
		Paleocene	65 Ma
		Mesozoic	Cretaceous
Early	144 Ma		
Jurassic	Late		159 Ma
	Middle		180 Ma
	Early	206 Ma	

Ma: Million years before present

Figure 3-2 Geologic time periods.

3.4 Climate

Averaged over the study area watershed (an area of 108 km²), the historical mean annual temperature (MAT) is approximately 1.3°C. The historical mean annual precipitation (MAP) is 1074 mm, of which approximately 402 mm (37%) is snowfall (precipitation as snow [PAS]) (Table 3-1). The spatial distribution of historical mean annual temperature, precipitation, and snowfall across the Willow River watershed is depicted in Figure 3-3 based on climate data from Wang et al. (2016).

Climate projections over the watershed show an increase in mean temperature, a net increase in the mean precipitation, and a net decrease in the mean snowfall by 2050 with trends continuing to 2080 (Table 3-1). Winter precipitation is projected to increase while the summer precipitation is projected to decrease. The increase in winter precipitation is anticipated to fall increasingly as rain as opposed to snow (Table 3-1; Wang et al., 2016).

Table 3-1 Historical and projected change in climate variables for 2050 and 2080 (assuming SSP2-4.5) compared to the historical period 1961 to 1990 (Wang et al., 2016).

Climate Variable	Historical	2050 ^{1,2}	2080 ^{1,2}
	Ensemble Mean	Ensemble Mean	Ensemble Mean
Annual			
Temperature (°C)	1.3	4.0	4.8
Precipitation ³	1074	1196	1227
Snowfall ⁴	402	307	271
Seasonal			
Winter Precipitation	684	780	808
Summer Precipitation	390	412	418

Notes:

SSP = Shared Socioeconomic Pathways

1. The ensemble projections are averages across 13 CMIP6 models (ACCESS-ESM1-5, CNRM-ESM2-1, EC-Earth3, GFDL-ESM4, GISS-E2-1-G, MIROC6, MPI-ESM1-2-HR, and MRI-ESM2-0) that were chosen to represent all major clusters of similar Atmosphere-Ocean General Circulation Models.
2. The 2050 projection covers the 2041 to 2070 time period while 2080 projection covers the 2071 to 2100 time period.
3. Precipitation includes both rain and snow.

Precipitation occurs throughout the year within the study area, with rainfall concentrated between May and October. Mean monthly precipitation during this period ranges from approximately 67 mm to 98 mm. Historical records from the Environment and Climate Change Canada (ECCC) Barkerville climate station indicate that extreme precipitation events occur in the region, with the wettest month on record receiving 411 mm of precipitation in August 1948. Frequency analysis of the Barkerville station indicates that a 100-year, 24-hour rainfall event produces approximately 66 mm of rainfall. These climatic conditions contribute to both rainfall-driven and rain-on-snow flood hazards within the study area. A more comprehensive evaluation of climate conditions in the Wells area is provided by Golder (April 26, 2022).

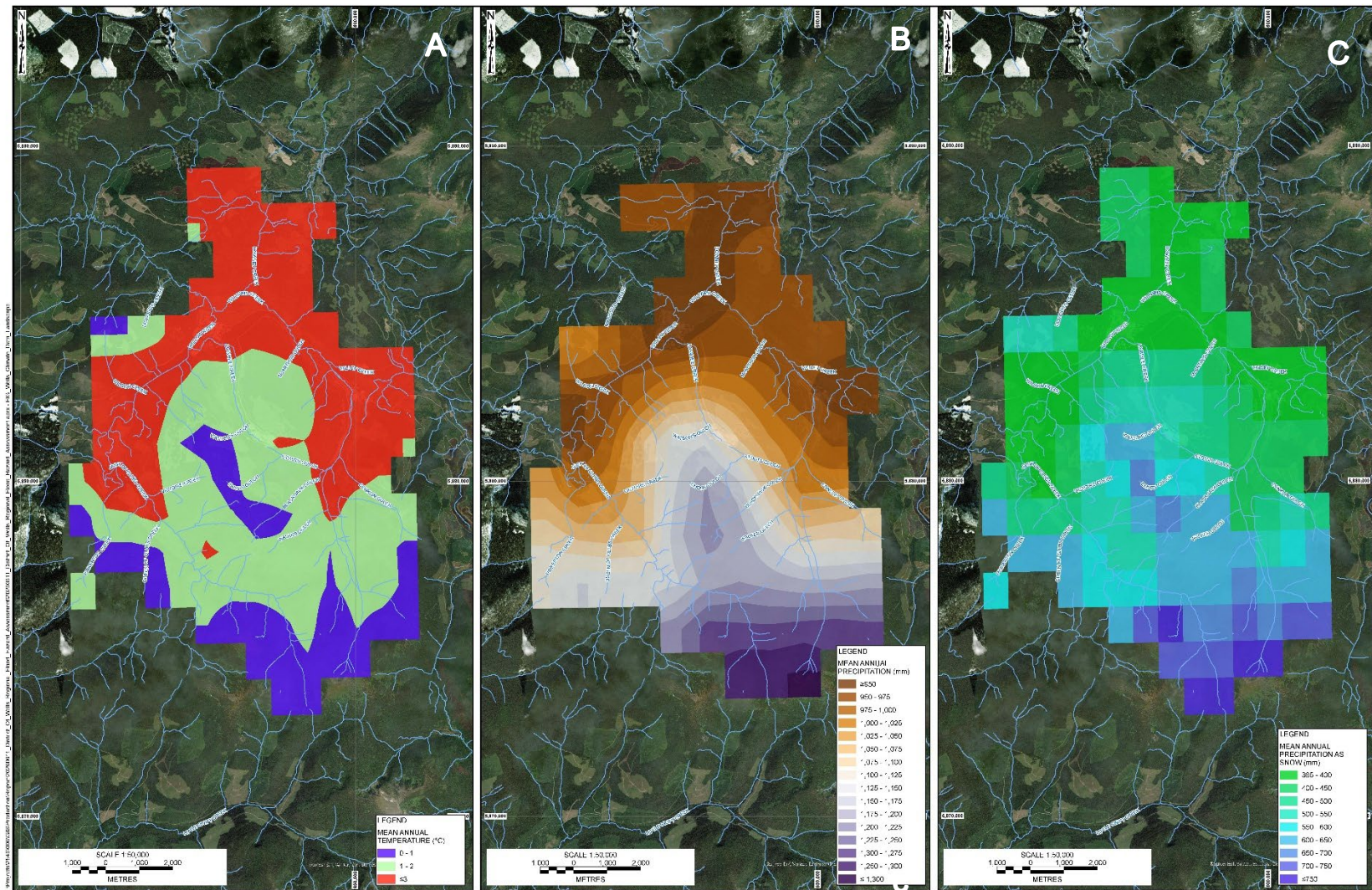


Figure 3-3 Historical (1961 to 1990) data (Wang et al., 2016): (A) MAT, (B) MAP, and (C) PAS across the Willow River watershed near Wells, BC.

3.5 Hydrologic Regime

There are no active hydrometric stations operated by the Water Survey of Canada (WSC) within the study watershed. The WSC did operate a station on Willow River below the confluence with Lowhee Creek between 1938-1941 and 1973-1975 (*Willow River at Wells*, 08KD002).

Hydrographs from this station indicate that the hydrological regime of the Willow River watershed is nival with the annual maximum flood typically occurring from snowmelt in the spring with higher peaks associated with rain-on-snow events.

The closest WSC station with long-term streamflow data is *Little Swift River at the Mouth* (08KE024). This station started operating in September 1971 and has been operated on a continuous basis until the present. The station is located approximately 23 km southwest of Barkerville (Williams Creek and Little Swift River share a watershed divide) and has a drainage area of 129 km². Almost all of the annual instantaneous peak flows at this station have occurred between May 1 and June 15, with only a few peaks occurring in the second half of July (Figure 3-4). The distribution of daily flows over the period 1971-2026 for station 08KE024 is provided in Figure 3-5.

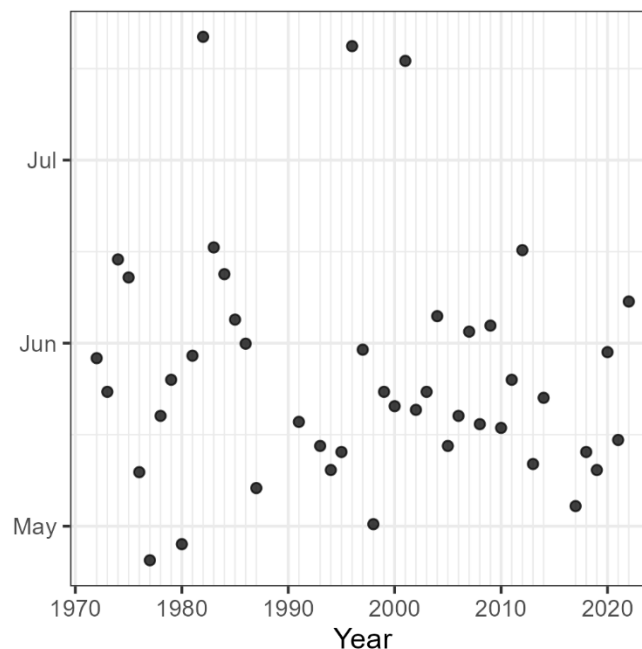


Figure 3-4 Occurrence date of annual instantaneous peak flows at WSC station *Little Swift River at the Mouth* (08KE024).

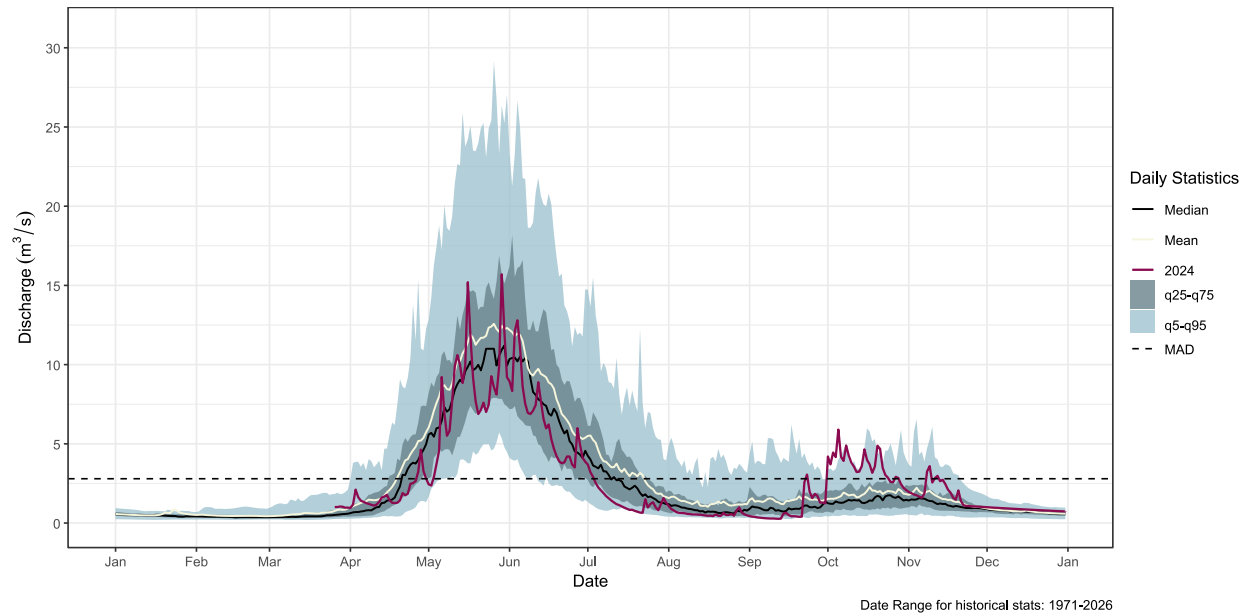


Figure 3-5 Distribution of daily flows over the period 1971-2026 from *Little Swift River at the Mouth (08KE024)*. The red line shows daily flows from year 2024, which represents a typical year. MAD = mean annual discharge.

Additional streamflow data have been gathered as part of the Cariboo Gold Project, as described below. Further details are provided in Golder (April 12, 2022).

Golder (2016-2018)

Golder (April 12, 2022) installed four hydrometric stations within the Willow River watershed:

- H12 – Black Jack Gulch upstream of Barkerville
- H21 – Jack of Clubs Creek upstream of Jack of Clubs Lake
- H25 – Mosquito Creek close to the confluence with the Willow River
- H16 – Willow River downstream of the confluence with Mosquito Creek.

These stations provide streamflow data for the open water period between July 2016 and mid-July 2018. An example of streamflow data gathered at the Willow River station (H16) is provided in Figure 3-6.

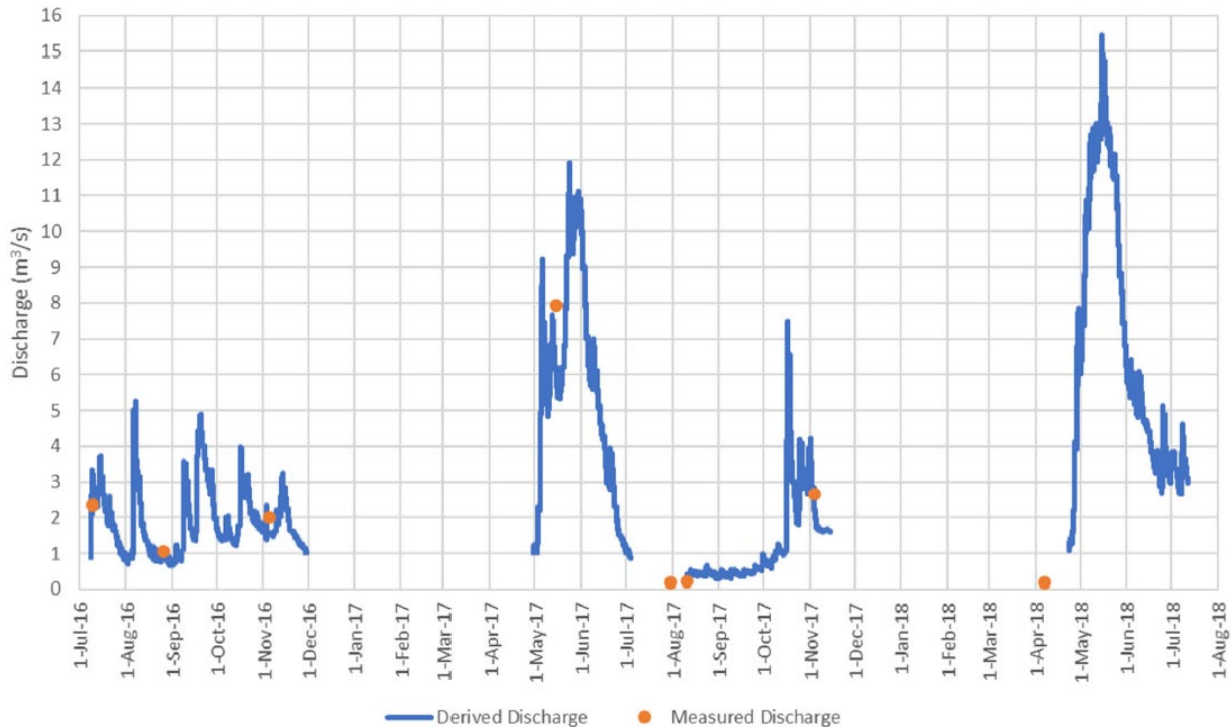


Figure 3-6 Average daily flow at Willow River hydrometric station (H16) between 2016 and 2018 (after Golder, April 12, 2022).

Osisko (2017-2018)

Osisko also operated seven hydrometric stations in 2017 and 2018 in support of the Cariboo Gold Project:

- Lowhee Creek at the BGM Bridge (LC-1.5)
- Lowhee Creek in the upper watershed (LC-3.9)
- Stouts Gulch approximately 1.35 km upstream from the confluence with Williams Creek (SG-1.35)
- Stouts Gulch approximately 250 m upstream of the confluence with Williams Creek (SG-0.25)
- Williams Creek downstream of Barkerville (WC-8.0)
- Willow River immediately upstream of the Highway 26 bridge (WR-151.5)
- Emory Gulch approximately 250 m upstream of the confluence with Stouts Gulch (EG-0.25).

However, level logger data were not provided to Golder (April 12, 2022) for LC-3.9, EG-0.25, and SG-0.25.

Knight Piésold (2006-2010)

As part of an earlier iteration of the Cariboo Gold Project, hydrometric data are also available at the following four stations (Knight Piésold, June 9, 2010):

- Willow River at Wells (downstream of Lowhee Creek) – May 2006 to April 2010

- Williams Creek at Barkerville – October 2009 to May 2010
- Lowhee Creek at BGM Bridge – May 2006 to April 2010
- Stouts Gulch – October 2006 to May 2010.

However, continuous streamflow data were only generated for the Willow River station (Figure 3-7), as insufficient discharge measurements were taken at the other stations to generate valid rating curves.

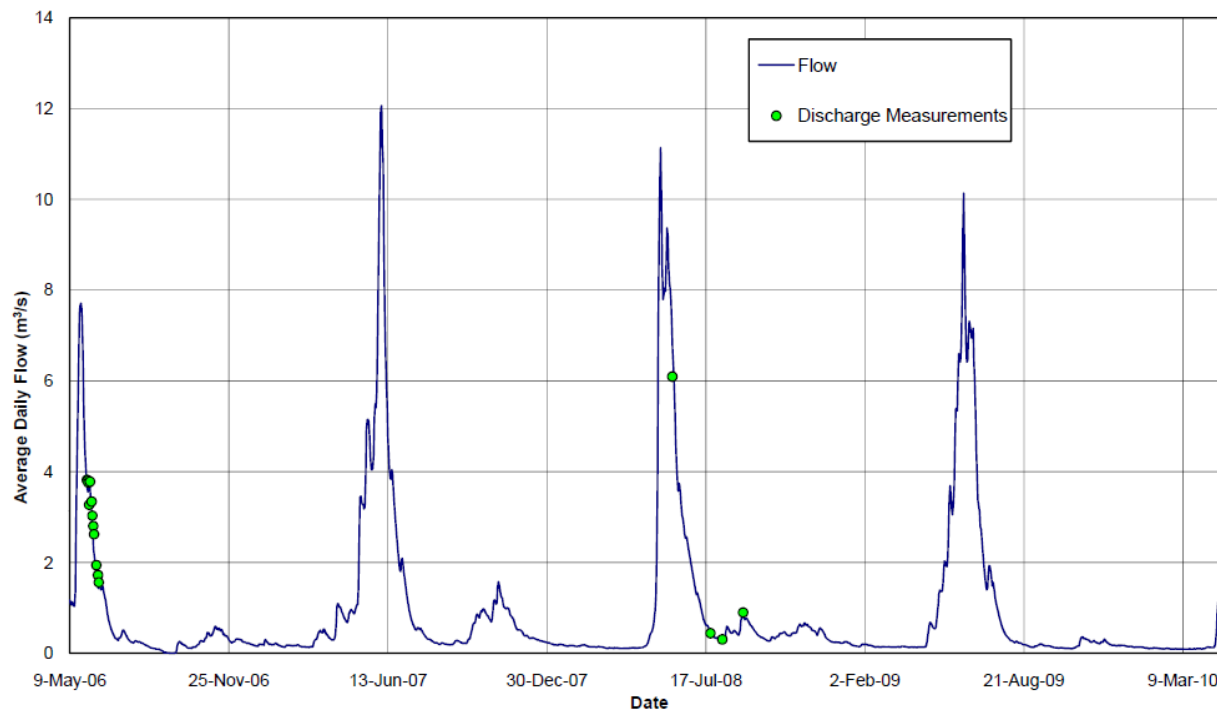


Figure 3-7 Average daily flow for Willow River at Wells between 2006 and 2010 (after Knight Piésold, June 9, 2010).

3.6 Data Sources

Sources of data used by BGC for the RFHA include past reporting, historic maps, field observations, lidar data, and historical aerial imagery.

Field Work

Fieldwork was conducted from June 22-24, 2025 by Isabelle Cheff, P.Eng., and Hamish Weatherly, P.Geo., of BGC. Mr. Ed Coleman, the Mayor of Wells, provided a site tour to BGC on June 23 to highlight areas of potential concern and facilitated a meeting with Carrie Chard and Mandy Kilby of the Barkerville Heritage Trust. The work included taking photographs and gathering sediment samples for the grain size analysis of in-channel materials. A drone survey was also conducted to capture aerial photographs of the watercourses, locations of interest, and surrounding areas.

Lidar Data Collection

Lidar data for the entire study area were graciously provided by the Quesnel Division of West Fraser Timber Co. The lidar data were acquired on various dates between June 30 and September 3, 2021 by Airborne Imaging at a density of 8 pts/m². The data were gathered using the NAD83 CSRS horizontal datum and CGVD2013 vertical datum.

Aerial Imagery

BGC obtained the aerial imagery listed in Table 3-2 for the RFHA. Of the air photographs listed in Table 3-2, BGC georeferenced the 1952, 1974, and 1991 imagery.

Table 3-2 Aerial imagery used for the District of Wells regional flood hazard assessment.

Date	Flight Line	Frame	Scale	Source
9/1/1952	A13524	83, 84	1:60,000	NAPL ¹
8/5/1967	BC7016	91, 92, 101, 102	1:15,840	GeoBC
8/4/1974	BC7599	156, 158, 159, 208, 209	1:16,000	GeoBC
9/26/1991	BCC91095	68-70, 87-90	1:15,000	GeoBC
8/26/2006	BCC06116	94, 95, 100, 101	1:20,000	GeoBC
2024	-	-	-	ESRI

Note:

1. National Air Photo Library.

4.0 EXISTING CONDITIONS

4.1 Barkerville Town

4.1.1 Overview of Assets

Barkerville is a national historic site and is the largest heritage site in BC. It is the preserved historic community that was initially established during the goldrush of the late 1800s and consists of the townsite, an abandoned airstrip, wastewater lagoon, and three campgrounds. Since 2005, Barkerville Town, the wastewater lagoon, and three campgrounds north of the townsite have been administered by the Barkerville Heritage Trust, a non-profit agency.

Barkerville Town stretches along the valley bottom of Williams Creek for about 750 m. Here the valley bottom varies in width from approximately 120 m to 140 m. The town is potentially subject to flood hazards from Williams Creek, Stout Gulch, Conklin Gulch and the hillslope on the west side. Williams Creek flows along the east side of Barkerville where it is confined by the valley wall on its right side (looking downstream). There are two bridge crossings over Williams Creek within the Barkerville Town, one being a pedestrian bridge and the other a vehicle bridge providing access to Prosperpine Trail and an emergency evacuation route. Williams Creek has a drainage area of 20.9 km² at the pedestrian bridge within the townsite (upstream of the Conklin Gulch confluence)

The primary assets in the townsite are over 100 buildings which include original historic wood structure buildings, refurbished buildings, and reproductions of historic structures (Drawing 04). A demonstration Cornish water wheel, constructed in 1985 (Galbraith, 2001), is located on the left (west) bank of Williams Creek at the south end of town. The water wheel intake includes a pond that is fed by a flexible, screened pipe intake that is lowered into the creek as needed. The intake pond is linear in shape and is divided by a causeway and culvert to permit equipment access to the creek bank. The pond feeds a PVC pipe into a sluice that powers the water wheel located approximately 190 m north of the pond. Overflow pipes at the north end of the pond control water levels when the pond is not operating.

The water source for Barkerville is a spring in Conklin Gulch. The water intake feeds to the pump house and treatment building through a 100 mm diameter pipe that crosses Williams Creek (McElhanney, July 2011).

4.1.2 Historical Flood Events and Flood Mitigation

A history of flooding in Barkerville is provided by Quackenbush (1997). Excerpts from that report are summarized below.

The first illustrated records of Williams Creek, the Royal Engineers 1863 map of the town, indicates the creek flowed in a meander near to where the Theatre Royal was constructed. This is the only reference to the creek's location as being other than the east side of the valley during historic times at the townsite. Flooding was frequent in the early years of the townsite as the buildings were constructed on the flat valley bottom (Figure 2-5 and Figure 2-6) and aggradation

of the creek bed was likely extreme due to the upstream disturbance caused by placer mining. Indeed, flood events in Barkerville prior to the 1930s arrived with such frequency that, except for the most extreme, they appear not to have been reported on (Galbraith, 2001) (Figure 4-1).



Figure 4-1 Main Street of Barkerville during a flood on June 11, 1911. Barkerville photograph collection, P2767.

The most destructive flooding appears to have occurred on April 5, 1879 (Quackenbush, 1997). Records indicate that after the flood buildings had to be raised as much as 5 m (17 feet) because of the placer tailings that had washed into town.

The primary method used to prevent flooding of the townsite was to build along the west side of the valley and to force the creek to the east side using a flood bulkhead¹⁴. The earliest photographs indicate that the bulkhead originally consisted of logs placed one above the other, horizontal to the ground, with logs placed intermittently at right angles along the wall to provide buttress support (Figure 4-2). Construction techniques evolved over time. A 1934 photograph shows horizontal boards, rather than logs, attached to widely spaced, upright supports.

¹⁴ A flood bulkhead is a watertight barrier used to prevent water from entering a property.



Figure 4-2 Williams Creek flood breaking through the bulkhead. Barkerville photograph collection, P4333. Unknown date.

Wooden bulkheads were also used to protect in-stream workings, such as mine shafts and water wheels. Quackenbush (1997) describes that the operators of the Ballarat Pit (located further downstream toward the Bowron Lake Road crossing) unearthed an entire water wheel, standing upright, that had been buried by flood debris. Log walls surrounded the wheel for its full height of about 6 m.

According to Galbraith (2001) the bulkhead was refurbished in 1897, 1915, 1929, 1934, 1965, and 1972. The 1965 work consisted of raising the ground between the townsite buildings and the left (west) bank of Williams Creek with bulk fill (burying the wooden bulkhead), providing for a backroad on the east side of town (Quackenbush, 1997). KWL (April 20, 2020) describes the Bulkhead Dike as an 820 m long structure constructed on the left (west) bank of Williams Creek with a height of 0.9 m to 1.2 m and crest width of 1.2 m to 2.4 m. Review of the lidar data indicates a slightly different reality. There are sections where an approximate 1 m high berm is evident. More commonly though, there is higher ground on the left bank of Williams Creek that gradually falls away as one moves to the west side of the valley floor (Drawing 04). This higher ground represents the bulk fill that was reportedly placed in 1965. At the south (upstream) end of the townsite, there is an approximate 2 m difference in elevation between the creek bed and the top of the left bank. This elevation difference increases to about 4 m at the north (downstream) end of town. For much of the townsite, the channel bed elevation of Williams Creek is currently similar to elevations on the far west side of the valley floor. Therefore, any overbank flows from Williams Creek would tend to avulse to the west side of the valley.

The only reported instance of Barkerville flooding between the 1930s and present is May 15, 1993, when the Stouts Gulch bridge became blocked by debris during a period of rapid snowmelt (Quackenbush, 1997). The creek flows then avulsed northeast toward the townsite (Figure 4-3), resulting in considerable damage to community infrastructure (Figure 4-4). Significant work was completed to repair the damages and restore the townsite. Overflows also reportedly occurred on Williams Creek adjacent to the water wheel.



Figure 4-3 View looking north at the south end of the Main Street/service road loop. The blue arrow shows the flow direction that allowed Stouts Gulch overflows to reach the townsite in 1993. Photograph of August 27, 2001 (Galbraith, 2001). McElhanney (2011) recommended that the road be raised in this area for flood control. However, this work was never completed.



**Figure 4-4 View looking north of flood along Main Street in Barkerville on May 15, 1993.
Photograph of Pat Taylor provided to BGC by the Barkerville Heritage Trust.**

Riprap has been placed on the left bank of Williams Creek in a few select locations within the townsite, but its placement is irregular. During BGC's 2025 site visit, heavy vegetation along the left bank made it difficult to ascertain where riprap was and was not present (Figure 4-5). Photographs from Galbraith (2001), when the left bank was much less vegetated than it is today, do illustrate the selective nature of riprap placement.



Figure 4-5 Upstream view of Williams Creek from the footbridge. BGC photograph of June 22, 2025.

In 2001, Galbraith describes how the left bank near the water wheel had only 1.5 m of clearance to the creek bed relative to upstream and downstream sections. The bank height was raised to 2 m in this area and riprap was placed from approximately 50 m upstream of the water wheel to 10 m below the footbridge (Galbraith, 2002). Bunches of slide alder were also planted and a row of boulders removed from the creek.

A decade later McElhanney (2011) noted ongoing erosion of the left bank of Williams Creek adjacent to the water wheel intake pond and recommended that 90 m of the bank be protected with riprap. Riprap has since been placed in this reach, but the year of construction is unknown.

A 2010 ice jam flood resulted in the erosion of a portion of the Williams Creek left bank at Conklin Gulch, exposing the supply line pipe. The creek bank was restored and armoured as part of emergency works in 2010 (McElhanney, July 2011) (Figure 4-6).



Figure 4-6 Downstream view of Williams Creek at confluence with Conklin Gulch. Note the riprap on the left bank. BGC photograph of June 22, 2025.

4.1.3 Hydraulic Structures

Culverts, bridges and ditches within the townsite are summarized below.

Stouts Gulch

Stouts Gulch is located at the south end of Barkerville and is crossed by Richfield Road, which is the terminus of the original Cariboo Road (Drawing 04). This creek has a drainage area of approximately 4.1 km² at the road crossing. The crossing originally consisted of a bridge that became plugged with debris on May 1, 1993, during a period of rapid snowmelt. The bridge was replaced with an 1800 mm (72") diameter culvert.

Galbraith (2001) noted that flood risk to the townsite persisted as the culvert remained susceptible to blockage. Galbraith recommended that Richfield Road be realigned to provide a dip in the road with lock blocks used to divert any potential overflows from Stouts Gulch back into Williams Creek, away from the townsite. Photographs and sketches of these constructed works are provided in Figure 4-7 and Figure 4-8.

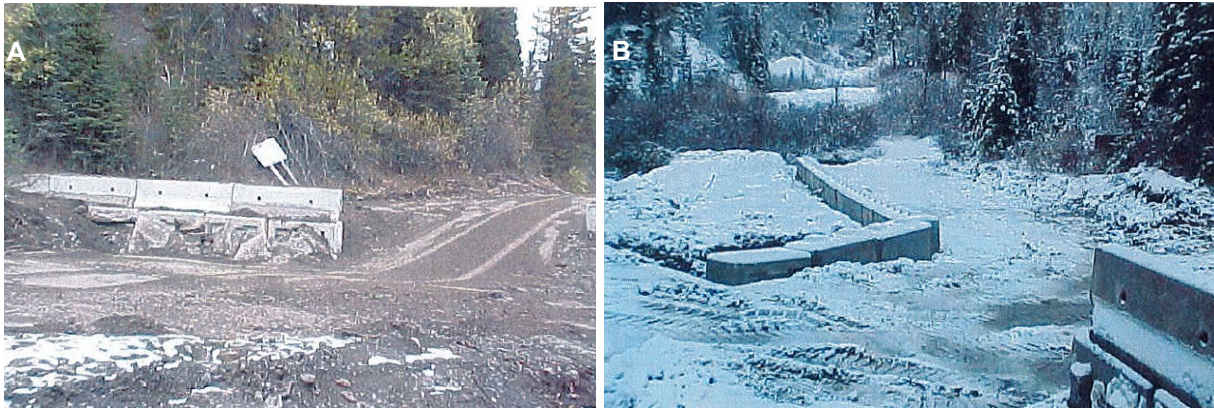


Figure 4-7 October 2001 photographs of A) barrier and grade in Richfield Road and B) lock blocks located downslope of the change in road grade (from Galbraith, 2001).

A decade later McElhanney (July 2011) noted that approximately 50 m north of Stouts Gulch was a manmade relief channel that was constructed to intercept overflow from Stouts Gulch and direct it to Williams Creek. McElhanney concluded that the effectiveness of the relief channel was limited as it was blocked by the embankment of Richfield Road. As a result, potential overflows from Stouts Gulch would spill into the relief channel but would then be blocked by the road embankment and be more likely to flow toward the townsite rather than Williams Creek.

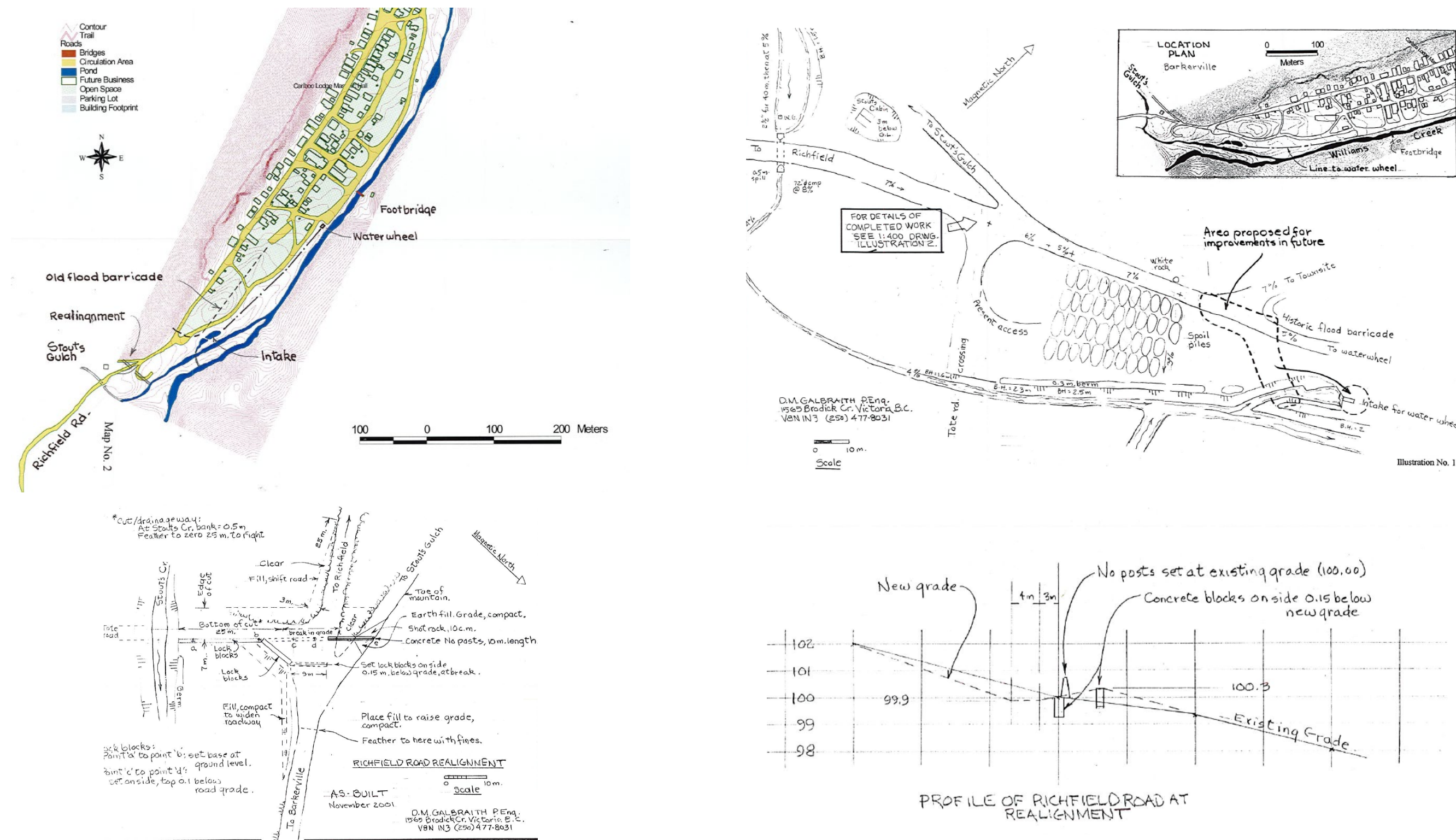


Figure 4-8 Richfield Road realignment northeast of Stouts Gulch crossing (after Galbraith, 2001).

The observations of McElhanney (July 2011) suggest that the works constructed in 2001 had been modified between 2001 and 2011 or were obscured by vegetation. Regardless, an 1800 mm diameter culvert was placed in the relief channel around 2020 (C. Chard, pers. comm.) (Figure 4-9).

In data files provided by the Barkerville Heritage Trust, there is a KWL (April 27, 2020) sketch that shows a typical section for a 1.5 m high armoured berm to be placed on the left bank of Stouts Gulch. Based on BGC's site visit and review of lidar data, this berm was not constructed.

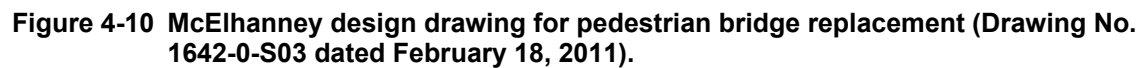


Figure 4-9 Downstream view of Stouts Gulch overflow culvert crossing of Richfield Road. BGC photograph of June 23, 2025.

Williams Creek

There are two bridge crossings of Williams Creek within the historic town of Barkerville. The upstream crossing is a pedestrian footbridge while the downstream bridge provides vehicle access to the east side of Williams Creek for logging and park maintenance activities.

The footbridge is located east of the Theatre Royal and is used to access a historic hydraulic mining site. An earlier footbridge at this location was destroyed during an ice jam flood in 2010 (McElhanney, July 2011). A replacement bridge was designed by McElhanney in 2011, and the crossing was subsequently re-instated (Figure 4-10). The McElhanney design drawing indicates that the bridge abutments are protected by Class 50 riprap, which has a median diameter (i.e., D_{50}) of approximately 350 mm.



42

West Side Drainage Ditch

The townsite also contends with nuisance seepage and runoff from the hillslope to the west. Some ditching is in place behind the buildings at the base of the hill to intercept a portion of this runoff. Where buildings abut against the slope, the ditch has been elevated above the toe of the hillslope. The natural tendency is for any water not captured by the ditch to flow towards the main street, as it is the lowest area in the valley. Several swales are therefore present to divert runoff away from individual buildings. The west side ditch is not continuous at the south end of town but is more continuous at the north end. The ditch continues along the west and north perimeter of the main parking lot and connects to ditching along Highway 26 (McElhanney, July 2011).

4.2 North Barkerville

4.2.1 Overview of Assets

North of the Barkerville townsite Williams Creek flows north parallel to Highway 26 and then under Bowron Lake Road. The creek then turns to the northwest, and parallels the Forest Rose campground, the airstrip and the wastewater lagoons for Barkerville (Drawing 03).

The old Ballarat Mine is the first asset located downstream of Barkerville and represents the downstream extent of mining on Williams Creek. Mining has occurred in four stages in this area (Quackenbush, 1997). The original work on the Ballarat property was principally drift mining, followed by the largest open pit mine on Williams Creek, the Cariboo Goldfields Company, between 1895 and 1915. A dredge was used on the claim during the late 1950s and in the final stage, areas of the floodplain were excavated to bedrock in the 1990s. This most recent stage of mining remains visible as a flooded open pit (mined out placer pit), 60 m wide by 200 m long. The open pit is located immediately south of Bowron Lake Road and to the west of Williams Creek (Drawing 05). To the east of the flooded open pit and Williams Creek is Ballarat Camp. Osisko has purchased this property, and it is being used as a construction camp and laydown area for the Cariboo Gold Project

Downstream of Bowron Lake Road, an 800 m long airstrip is located on the left (southwest) floodplain of Williams Creek. The airstrip is not in active use but is used sporadically by helicopters on an emergency basis. Campgrounds are located on either side of the valley bottom in this reach. The Forest Rose campground, with 56 campsites, is located on the right (northeast) floodplain of Williams Creek. The Lowhee campground, with 80 campsites, is located on the opposite side of the valley on the southwest side of Highway 26. In this reach, Williams Creek is 55 to 60 m wide and has a braided channel morphology.

The wastewater lagoons are located at the downstream end of the airstrip on the left floodplain of Williams Creek. The original sewage treatment facility was constructed in 1982 and included two treatment cells (~100 m long by 40 m wide, Cells #1 and #2) and a storage cell (~100 m long by 60m wide, Cell #3) (Figure 4-12). A fourth cell, on the south side of the facility, was under construction during BGC's site visit in June 2025. The storage cells are connected by cross flow pipes with the outfall for the system located at the northwest corner of the facility.

Treated effluent from the lagoon is only discharged during the spring in accordance with a Provincial permit. The lagoon system is elevated above natural ground through the construction of earthen berms, while the ponds have a synthetic liner to contain the sewage effluent. The lagoon is accessible from Highway 26 via an access road. A 200 mm sanitary sewer main that discharges into Cells #1 and 2 lies under this access road. A 300 mm diameter outfall pipe extends about 80 m downstream along the drainage ditch that has formed between the Lagoon dike and berm of Cell #3 (Figure 4-12).

Anecdotal reports indicate that the fill for construction of the wastewater lagoon berms was taken from a borrow pit between the airstrip and the lagoon. McElhanney (July 2011) indicates that it is possible that the borrow pit was isolated from Williams Creek by the dike protecting the airstrip and lagoon. The borrow pit then subsequently filled with water, either by groundwater seepage or through creek water flowing through a breach in the dike, developing into a pond with wetland areas around its perimeter (Figure 4-12).



Figure 4-12 BGC June 23, 2025 drone photograph of Barkerville wastewater lagoons.

4.2.2 Historical Flood Events and Flood Mitigation

The wastewater lagoons are protected by a 250 m long earthen dike that separates the lagoons from Williams Creek. Original design drawings for the lagoons indicate that the earthen dike was to be protected with riprap (Stanley, February 1981). However, a drone photograph of the Lagoon dike in 2001 shows no riprap protection (Galbraith, 2001).

Relative to the current bed elevation of Williams Creek, the Lagoon dike is approximately 1.5 to 2 m high. However, the land between the Lagoon dike and the outside berm of Cells #2 and #3 is about 0.5 m lower than Williams Creek, forming a ditch that flows from southeast to northwest (Figure 4-12). Galbraith (2001) notes that four cobble groynes were constructed at the upstream end of the Lagoon dike in 1999 (Figure 4-13). The Lagoon dike was extended downstream by 60 m in 2001 using borrow from the land side of the existing dike (Galbraith, 2001). Willow, alders and immature conifers were also transplanted to the Williams Creek side of the dike.

There is no known occurrence of the lagoon berms being overtopped since their construction in 1982.

An earthen dike also separates the downstream half of the airstrip and Williams Creek (Drawing 05). The Airstrip dike is approximately 500 m long and varies in height from 1.5 to 2.0 m. The dike does not extend to the upstream half of the airstrip as the height difference between the airstrip and Williams Creek in this reach is on the order of 4 to 5 m. This height difference progressively decreases downstream. At the downstream end of the dike, the airstrip and Williams Creek are at about the same elevation (Drawing 05). Four groynes, constructed of highway dividers and cobbles, were constructed in 1999 to protect the first 200 m of the dike (Figure 4-13). These groynes have since eroded away.

Figure 4-13 indicates that the Lagoon dike and Airstrip dike used to be connected; although Galbraith noted in 2001 that the downstream end had been scoured half through. There is now a 175 m gap between the Lagoon dike and Airstrip dike. Both dikes are Provincially registered structures (GPS131).

McElhanney (July 2011) describes a breach at the downstream end of the Airstrip dike that created a direct hydraulic connection between the pond southwest of the dike and Williams Creek. In earlier years, this connection periodically caused pond water levels to overtop a short, approximately 5 m section of the lagoon service road, with flows conveyed along the ditch between the lagoon and its protective dike. Although this occasional overtopping was an annoyance, it did not pose a direct risk to the lagoon berms. In the spring of 2010, however, the majority of flow in Williams Creek naturally avulsed into the pond, resulting in overtopping along approximately 100 m of the service road. Emergency works were initiated in response.

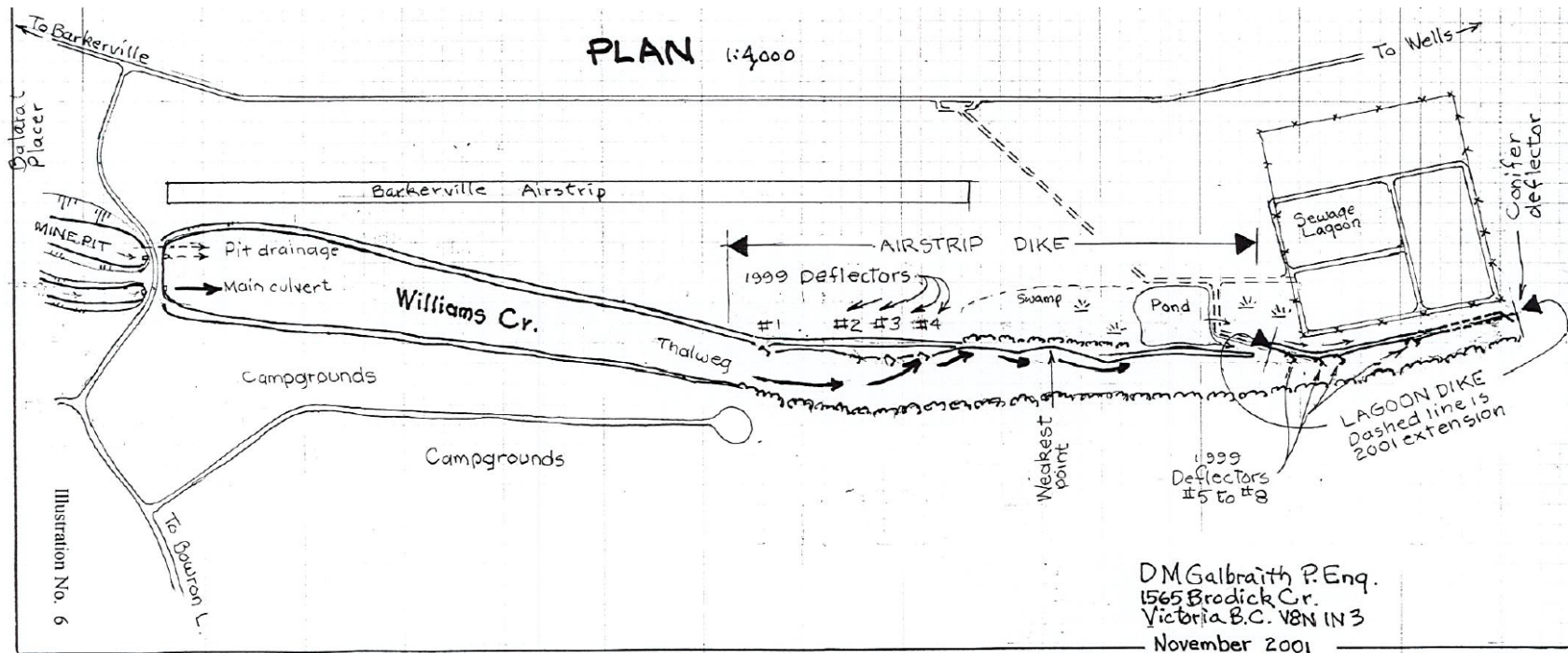


Figure 4-13 Sketch of the Lagoon dike and Airstrip dike (from Galbraith, 2001).

The pond immediately southeast of the wastewater lagoons is one of a limited number of wetland habitats along Williams Creek. Preservation of the pond was therefore identified as a high priority by Fisheries and Oceans Canada (DFO) to consider in the 2010 emergency works. Rather than repairing the dike segment between the Lagoon Dike and the Airstrip Dike, a new dike alignment was constructed around the western perimeter of the pond. The new alignment included raising approximately 220 m of the access road and service road such that it functioned as a 1 m-high dike. An additional 1 m-high service road, also acting as a dike, was constructed to the southeast to tie into the Airstrip Dike. These emergency works completed in 2010 resulted in a new, continuous 510 m-long dike around the pond between the airstrip and the lagoon, bypassing portions of the Airstrip Dike that were experiencing ongoing erosion.

A third 675 m long dike is located in this reach on the right bank of Williams Creek (Drawing 05). The dike commences immediately downstream of Bowron Lake Road and extends to the downstream end of the Forest Rose Campground. While the lidar coverage is patchy on this heavily vegetated bank, the Campground dike appears to range in height from 1 to 2 m, although some sections appear to have eroded away. Galbraith (2001) describes this dike as “half scoured away”. The construction year of this dike is not known.

4.2.3 Hydraulic Structures

Major hydraulic structures in this area include the intersection of Williams Creek with Bowron Lake Road and One-Mile Road.

At Bowron Lake Road, Williams Creek flows through a 3 m diameter corrugated steel pipe (Figure 4-14). In 1991, Bowron Lake Road crossed Williams Creek approximately 85 m to the southeast. The current road alignment, which now includes an S-bend, was presumably adjusted to accommodate excavation of the Ballarat open pit in the 1990s. Maintenance and operation of Bowron Lake Road and the 3 m diameter culvert are the responsibility of the BC Ministry of Transportation and Transit (MoTT).

The Ballarat open pit is now flooded, although water levels in the pit are typically much lower than in Williams Creek. When the 2021 lidar was acquired, water levels in the pit were approximately 4.5 m lower than in Williams Creek. At higher pond levels, there is a 600 mm diameter culvert at the north end of the pit that allows the pond to drain under Bowron Lake Road and into Williams Creek.



Figure 4-14 Downstream view of Williams Creek culvert crossing of Bowron Lake Road. BGC photograph of June 24, 2025.

Approximately 3.5 km down valley of Bowron Lake Road, Williams Creek crosses One-Mile Road. This road crossing is used by outdoor recreation vehicles such as trucks, quads, dirt bikes, etc. to access the back-country road network to the north. The road was initially built by placing logs in the swampy floodplain and then topping with gravel. During high flow periods, the road is known to overtop as it crosses the 200 m wide floodplain. This flooding has washed the road out in the past; however, the washed-out section was rebuilt in 2019 with less erodible material (KWL, April 20, 2020). The rebuilt road includes two 3-foot (0.9 m) diameter culverts within the floodplain.

The One-Mile Road crossing of Williams Creek consists of a 12 m wide bridge (Figure 4-15), located 40 m from the junction with Highway 26. During BGC's site visit, a small beaver dam was noted at the left abutment. Beaver dams are a common occurrence on Williams Creek below the wastewater lagoons.



Figure 4-15 Upstream view of Williams Creek at One-Mile Road bridge crossing. BGC photograph of June 23, 2025.

4.3 Jack of Clubs Creek

Jack of Clubs Creek has a watershed area of approximately 34 km² where it discharges into Jack of Clubs Lake. While some placer mining historically occurred along lower reaches of Jack of Clubs Creek (Table 2-1), there is currently no development in the watershed with the exception of some logging. Infrastructure in the watershed is limited to a few logging roads and Groundhog Lake Dam, located in the headwaters of the watershed at an approximate elevation of 1750 m off the northwest flank of Mt. Agnes (Figure 4-16).



Figure 4-16 Google Earth image (September 3, 2024) of Groundhog Lake.

According to the Provincial dam database, the Groundhog Lake Dam (File #D820000-0) is an earthfill structure commissioned in 1897 with a dam height of 10 m and a crest length of 100 m. The impounded lake has a surface area of 4.6 ha, and a spillway channel is located on the right (east) abutment. The Provincial database indicates that the dam, which is owned by the BC Ministry of Environment and Parks, has a Low consequence of failure classification rating and a Very Small failure probability rating. The resulting risk level rating is 5 or effectual, which is the lowest risk rating possible.

While the dam is now an earthfill structure, historical photographs indicate that it was originally constructed as a log-crib structure (Figure 4-17). BGC was unable to determine why the dam was constructed but presumably it was used to support placer mining in lower reaches of Jack of Clubs Creek. The lake is now a Provincial recreation site.



Figure 4-17 Two men and two women standing on the dam at Groundhog Lake. Barkerville photograph collection, P1055. Unknown date.

Jack of Clubs Creek discharges into Jack of Clubs Lake. Before entering Wells, Highway 26 generally follows the Jack of Clubs Lake northern shoreline and then crosses the lake outlet via the MoTT-owned Willow River bridge. The bridge is located approximately 950 m downstream from Jack of Clubs Lake.

4.4 Wells

4.4.1 Overview of Assets

The town of Wells, with an approximate population of 220 people¹⁵, is located downstream of Jack of Clubs Lake on either side of Willow River. The original town area is located on the west side of the river, perched on a hillside that lies about 10 m above the valley bottom. Over time, the town expanded to the east onto the alluvial fan of Lowhee Creek (Drawing 06). This area is known locally as South Wells.

Potable water for Wells is sourced from a 50 m deep water supply well located in South Wells on Ski Hill Road. Adjacent to the water supply well is the town's water treatment plant, which uses iron manganese filters to treat the groundwater. The town's water tower is located at an approximate elevation of 1254 m, which is about 50 m above the main town area.

¹⁵ 2021 census.

Sewage from Wells is treated by aeration at a single cell wastewater lagoon located approximately 750 m northwest of the town center. Aeration is provided from blowers, located in the adjacent blower builder, and distributed by submersed air supply lines (Radloff, September 22, 2022). Raw wastewater enters the southeast corner of the lagoon via a 200 mm diameter gravity sewer pipe. Once treated by the lagoon, the effluent exits the lagoon via a v-notch weir at the northwest corner of the lagoon. The treated effluent is then conveyed by the lagoon outfall piping (150 m of 200 mm diameter asbestos concrete piping) to the Willow River to the north. Radloff (September 29, 2025) has generated detailed engineering drawings to replace the existing outfall pipe with a 200 mm diameter HDPE pipe. The proposed updates include a new flow meter. Characteristics of the wastewater lagoon are summarized in Table 4-1. Radloff (September 22, 2022) estimates that the lagoon was originally designed to service 500 people.

Table 4-1 Wells wastewater lagoon characteristics (from Radloff, September 22, 2022).

Variable	Value
Average top width	45 m
Average length	86 m
Average depth	3.85 m
Operating volume	5,000 m ³
Available treatment volume	4,070 m ³ (average sludge depth of 0.8 m on the lagoon bottom)
Lagoon liner	2" bentonite clay liner
Permitted discharge	273 m ³ /day
Permitted water quality	45 mg/L of biochemical oxygen demand (BOD) and 60 mg/L of total suspended solids (TSS)

The town's sewage collection system drains to a lift station (Lift Station #1) at the east end of Dawson Street (~1200 m El.), where it is pumped toward the wastewater lagoon. Sewage from South Wells flows through an aerial sewer line located 20 m downstream of the Highway 26 bridge crossing of Willow River (Figure 4-18). The sewer line then runs along the west side of Willow River toward Lift Station #1. Radloff is currently designing a replacement for the aerial sewer line that would be buried beneath the river channel upstream of the bridge and feed a second lift station (Lift Station #2), to be located about 80 m west of the river (Drawing 06).



Figure 4-18 From the Jack of Clubs bridge looking downstream (northeast) towards the aerial sewer crossing. BGC photograph of May 28, 2024.

Additional critical infrastructure in the main town area includes the school, firehall, community hall, and emergency center. A water supply well, water treatment plant, fuel depot, ambulance bay, and RCMP station are located in South Wells (Drawing 06).

4.4.2 Hydraulic Structures

Bridges in the immediate vicinity of Wells include: the Highway 26 crossing of Willow River, the BGM bridge at the fan apex of Lowhee Creek, and a pedestrian bridge located 500 m downstream of the Highway 26 bridge. The pedestrian bridge connects the main townsite with South Wells and provides residents access to a network of trails constructed on the swampy floodplain of the Willow River and Williams Creek.

The Highway 26 bridge was constructed in 1994 and crosses Willow River at an oblique angle. The total span length is approximately 24 m with concrete footings on either abutment and five cylindrical steel piers mid span (Figure 4-19). High flows in May 2024 caused concerns of bank erosion impacting the bridge and downstream aerial sewer line. As a result, 250 kg¹⁶ riprap was placed on either bank of the creek for approximately 25 m downstream and 20 m (left bank) to 35 m (right bank) upstream of the bridge in early July 2024 (Figure 4-20). This work was completed on an emergency basis by MoTT and the DoW using a Radloff design (September 25, 2025).

¹⁶ The Radloff (June 2025) record drawings show both 100 kg and 250 kg riprap. It is not clear what size riprap was installed.



Figure 4-19 From the left (west) bank of Willow River looking upstream (south) towards the piers of the Highway 26 bridge. BGC photograph of May 28, 2024.



Figure 4-20 Riprap placement on the left (west) bank of Willow River immediately downstream of the Highway 26 bridge. Photo source: Radloff (unknown date).

The BGM bridge is located at the fan apex of Lowhee Creek at the end of Ski Hill Road and provides access to the abandoned workings of the Cariboo Gold Quartz Mine. This bridge is approximately 15 m long and is constructed with steel girders on a wooden deck (Figure 4-21).



Figure 4-21 Looking along Lowhee Creek upstream (east) towards the BGM bridge. Photo source: Radloff (unknown date).

There are also two pedestrian footbridges to the north of Wells. The upstream aluminum footbridge is on the Creekside trail system and crosses Williams Creek about 500 m northeast of town (Figure 4-22). The downstream footbridge is on Sugar Loaf trail, which starts at the north end of the townsite and crosses the Willow River about 400 m north. This bridge is 12 m long and constructed with 0.95 m thick steel girders that support a wooden deck (Figure 4-23). The bridge accesses trails on the north side of the river as well as a frisbee golf course.



Figure 4-22 Upstream view of pedestrian bridge crossing of Williams Creek. BGC photograph of June 24, 2025.



Figure 4-23 Upstream view of pedestrian bridge crossing of Willow River. BGC photograph of June 24, 2025.

4.4.3 Historical Flood Events and Flood Mitigation

The Wells townsite is potentially impacted by flooding from Willow River and Lowhee Creek, although there are no recorded instances of flooding impacting any infrastructure.

The headwaters of Lowhee Creek are located south of Wells and drain an approximate area of 5.8 km². Since deglaciation of the area, an alluvial fan complex has developed where the creek flows onto the valley bottom adjacent to Willow River. The creek deposits sediment on the alluvial fan due to a loss of confinement and reduced channel gradients.

In most watersheds with alluvial fans in British Columbia, sediment supply was greatest immediately following deglaciation about 12,000 years ago as the receding glaciers retreated, exposing unvegetated sediments in their wake. As the watersheds stabilized and vegetated, sediment transport rates decreased gradually. Lowhee Creek is unique in that human disturbances have greatly increased sediment supply in the last century. Due to historical upstream placer mining, sediment input to the fan have increased substantially in the past 150 years (Brown & Ash, 2009). This has led to rapid sedimentation and channel change on the fan, which has increased the likelihood for Lowhee Creek to overtop its banks and impact buildings within South Wells.

To mitigate flood hazard in residential areas adjacent to Lowhee Creek, the DoW constructed the 280 m long Phase 1 dike in 2023/2024 (Figure 4-24, Figure 4-25). While there was an existing dike along the right bank of Lowhee Creek¹⁷, that dike was in very poor shape and required significant upgrades. The remaining flood hazard is to be mitigated by way of the Phase 2 dike, which is split into two phases: Phase 2a and Phase 2b. Phase 2a extends the Phase 1 dike approximately 75 m upstream towards the BGM bridge at the fan apex, and Phase 2b extends the Phase 1 dike approximately 250 m downstream. The Phase 2a dike was constructed in early 2025 with BGC providing hydrotechnical and geomorphic support to Radloff for both the Phase 2a and 2b dikes (BGC, September 13, 2024; October 11, 2024a,b; November 8, 2024; November 26, 2024).

Issued for construction (IFC) drawings and a detailed design brief for the Phase 2a dike were prepared by Radloff (January 2025; February 25, 2025). The Phase 2b dike remains in permitting, although IFC drawings and a design brief have been prepared (Radloff, May 14, 2025; September 4, 2025).

The Phase 2a dike is intended to provide erosion protection for the bank upstream of the Phase 1 dike but not the BGM bridge itself. It is not clear who has ownership/jurisdiction over the BGM bridge. However, BGC's (October 11, 2024b) evaluation of hydraulic conditions indicates that the bridge abutments require additional erosion protection. The bridge would also benefit from a wider span, as demonstrated by the current hydraulic constriction at the bridge opening (Figure 4-26).

¹⁷ The Lowhee Creek dike is a Provincially registered structure (GPS309).



- Design flow is the 500-year instantaneous peak flow adjusted for climate change = 45 m³/s
- Minimum 1.0 m freeboard to account for uncertainty in the peak flood levels and potential future aggradation that is expected to occur on the Lowhee Creek fan
- Scour depth = 1.5 m
- Class 100 kg riprap ($D_{50} = 415$ mm) with a nominal thickness of 1.0 m overlying geotextile
- A 4 to 6 m wide access road along the dike crest for ongoing maintenance and monitoring.

59



Figure 4-25 Downstream view of Lowhee Creek from the BGM bridge. The Phase 1 and 2a dike is visible on the right bank. BGC photograph of June 22, 2025.

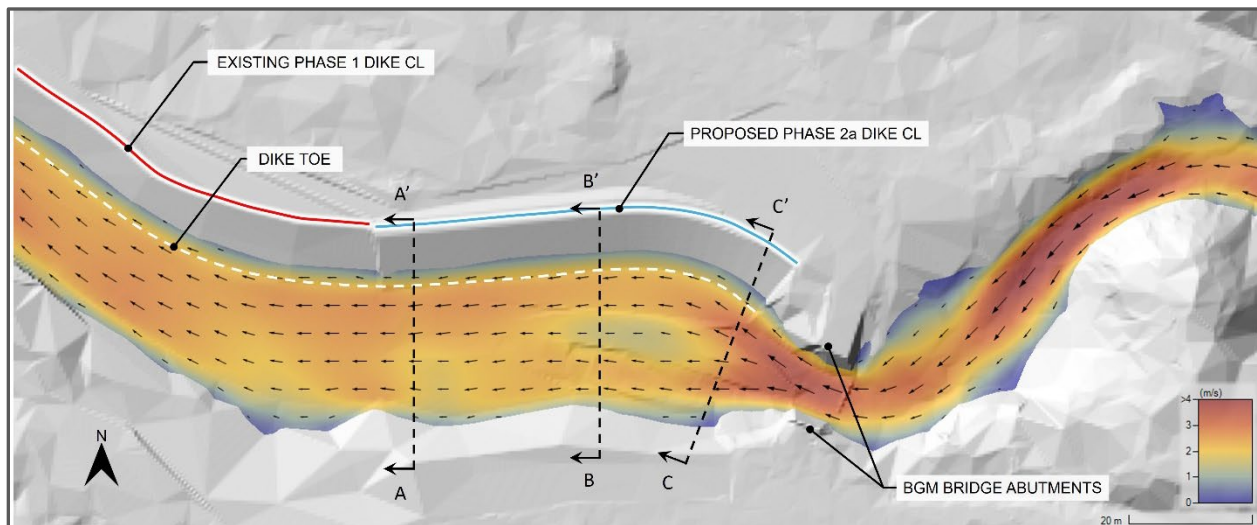


Figure 4-26 Modelled Lowhee Creek flow velocities (blue to red) during the design flood event in the vicinity of the BGM bridge overlaid on the DEM (hillshade) (from BGC, October 11, 2024a). Note the flow constriction created by the bridge.

5.0 GEOMORPHIC ASSESSMENT

The historic hydraulic placer mining has had a significant impact on the geomorphology of Lowhee Creek and Williams Creek. Drawings 07 and 08 are georectified 1952 air photographs of the area, which show the impacts of the hydraulic mining. While placer mining no longer occurs in the study area, the historical impacts continue to influence flood hazards as discussed below.

5.1 Lowhee Creek Channel Changes

Drawing 07 shows that hydraulic placer mining took place along most of the length of Lowhee Creek. Inspection of available lidar data suggests that the valley sediments were hydraulically mined to depths of up to 30 m with the sluiced sediment being transported downstream and depositing on the alluvial fan of Lowhee Creek.

The volume of sediment transported onto the alluvial fan is not known but the volume must have been very significant. A 1923 map of the area shows multiple channels on the alluvial fan, which is indicative of very high sediment transport rates (Figure 5-1). This map also shows Jack of Clubs Lake extending much further north than it currently does.

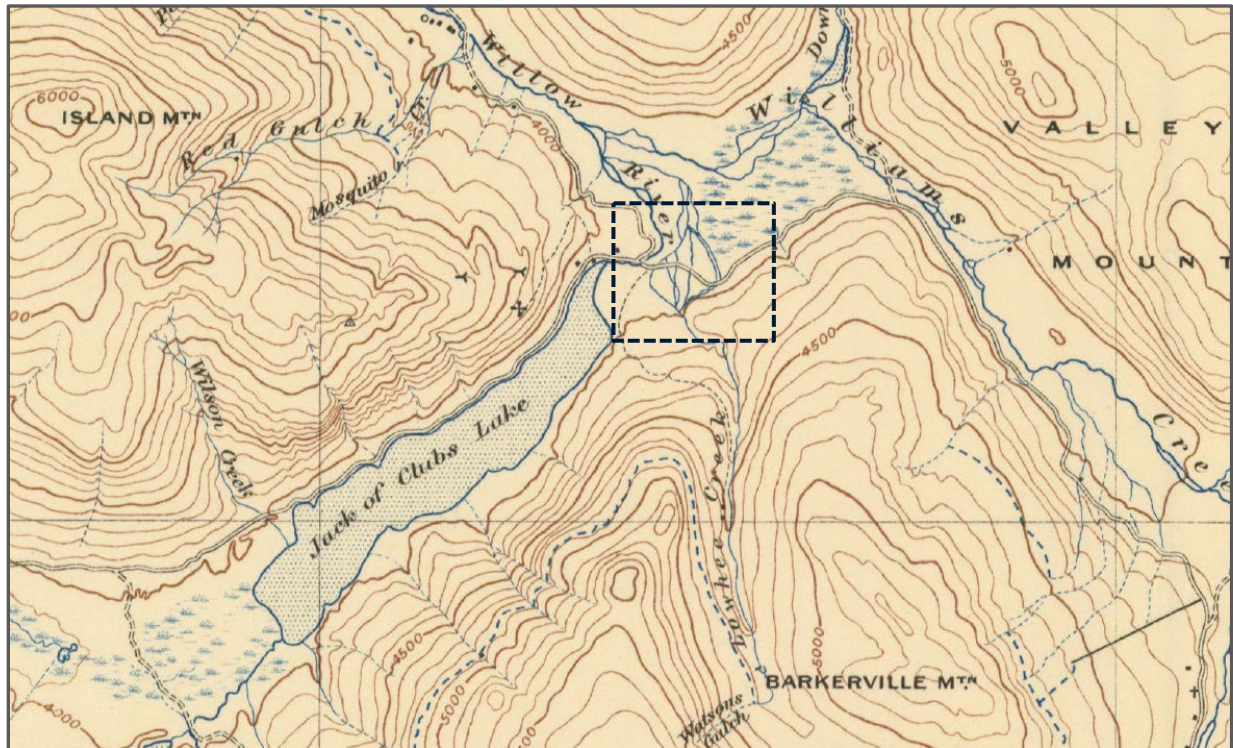


Figure 5-1 1923 map of Lowhee Creek and Jack of Clubs Lake area. Note the braided channel pattern of Lowhee Creek (black dashed rectangle) on its alluvial fan (Canada Department of Mines, 1923).

Recent geotechnical drilling by SoilTech (February 3, 2025) provides some insight into the depth of placer outwash on the alluvial fan. As part of dike construction along the right bank of Lowhee Creek, SoilTech drilled twelve (12) boreholes on the alluvial fan of Lowhee Creek

(Figure 5-2). Each of those boreholes encountered hydraulically mined sediment (i.e., placer outwash), which was described predominantly as sand and gravel with some silt and the coarse particles being highly fractured. Most of the boreholes did not encounter the full depth of the placer outwash, including BH24-07 which was still drilling into those sediments when terminated at a depth of 22 m (Table 5-1).

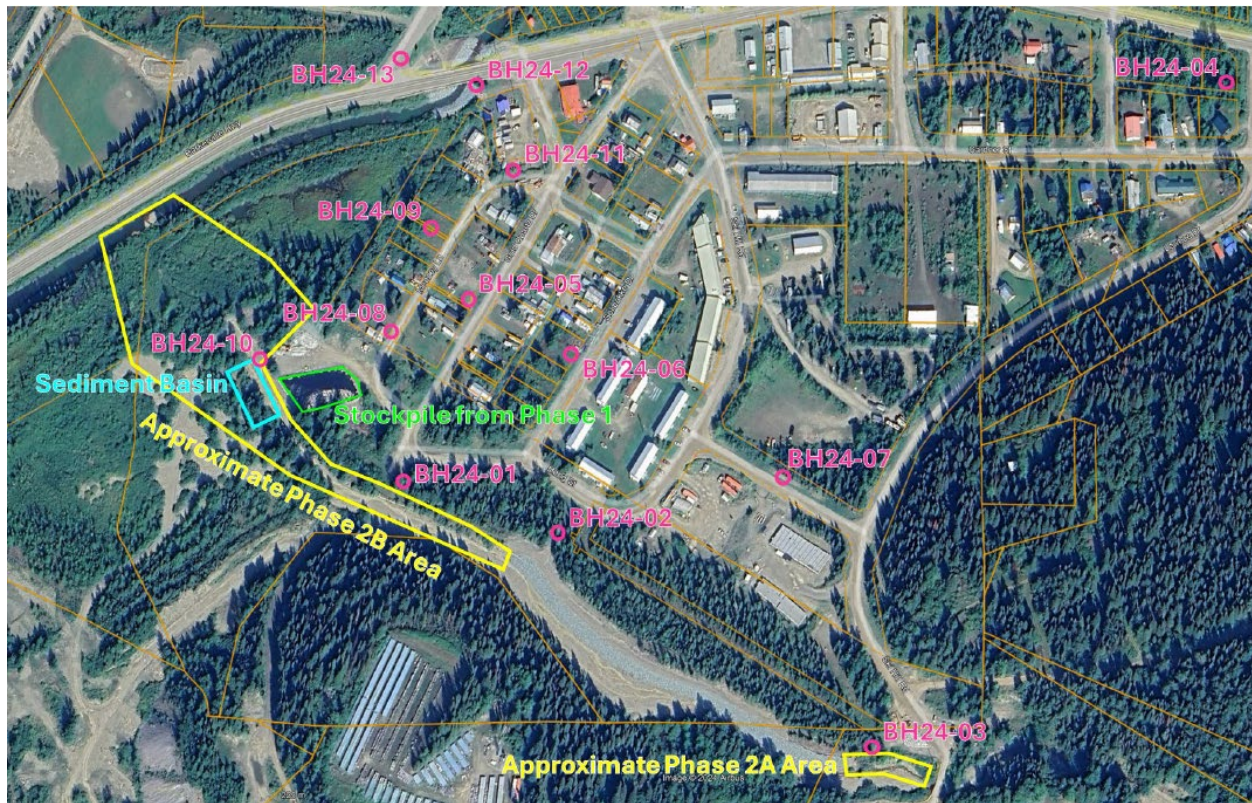


Figure 5-2 SoilTech borehole locations on the alluvial fan of Lowhee Creek (SoilTech, February 3, 2025).

Table 5-1 Depth of placer outwash encountered in SoilTech boreholes.

Borehole	Depth of Placer Outwash	Borehole	Depth of Placer Outwash ¹
BH24-01	> 11 m	BH24-08	> 7 m
BH24-02	> 11 m	BH24-09	> 7 m
BH24-03	> 7 m	BH24-10	> 7 m
BH24-04	5 m	BH24-11	> 5 m
BH24-05	> 8 m	BH24-12	2 m
BH24-06	> 11 m	BH24-13	2 m
BH24-07	> 22 m		

Note:

1. Greater than symbol (>) indicates the borehole was stopped at the tabulated depth.

A 1940 photograph of the Lowhee Creek alluvial fan shows the extent of the placer outwash deposits (Figure 5-3).

Sediment was also introduced onto the Lowhee Creek fan by the underground mining operations. During the operation of the Cariboo Gold Quartz Mine, approximately 2.65 million t of flotation mill tailings were deposited into the northeastern end of Jack of Clubs Lake near its outlet into Willow River. SNC Lavalin (2011) has reported that these tailings filled approximately 30 ha of the original lake area (SNC Lavalin, 2011). Three cross-sections of the fan showing the approximate surficial and bedrock geology of the Lowhee Creek fan are provided in Figure 5-4. These sections show the approximate distribution of the placer outwash and flotation mill tailings.

While most of the hydraulically mined sediment appears to have deposited to the east of the current Lowhee Creek channel, some of this sediment appears to have partially infilled the northeastern end of Jack of Clubs Lake also. The earliest air photographs of the area date to 1952, which post-dates the deposition of flotation tailings and the majority of the placer outwash (Figure 2-3). However, the approximate pre-development downstream extent of Jack of Clubs Lake can be inferred from several sources.

Before the town of Wells was established in the early 1930s, there were a small number of people living in the area. After the Cariboo Gold Rush had waned, there were still some miners and people living in Barkerville and, on the future site of Wells. There was a tiny camp that housed a few miners, a stopping house (or roadhouse), and a sawmill operation owned by Clarke. Early 1900 photographs of that camp and the original outlet of Jack of Clubs Lake are shown in Figure 5-5. Those photographs show the outlet of Jack of Clubs Lake at approximately the intersection of Polley Street and Highway 26.



Figure 5-3 1940 photograph looking toward South Wells and Lowhee Creek alluvial fan. Source: City of Vancouver Archives Items: CVA-289-005.602 – South Wells, BC.

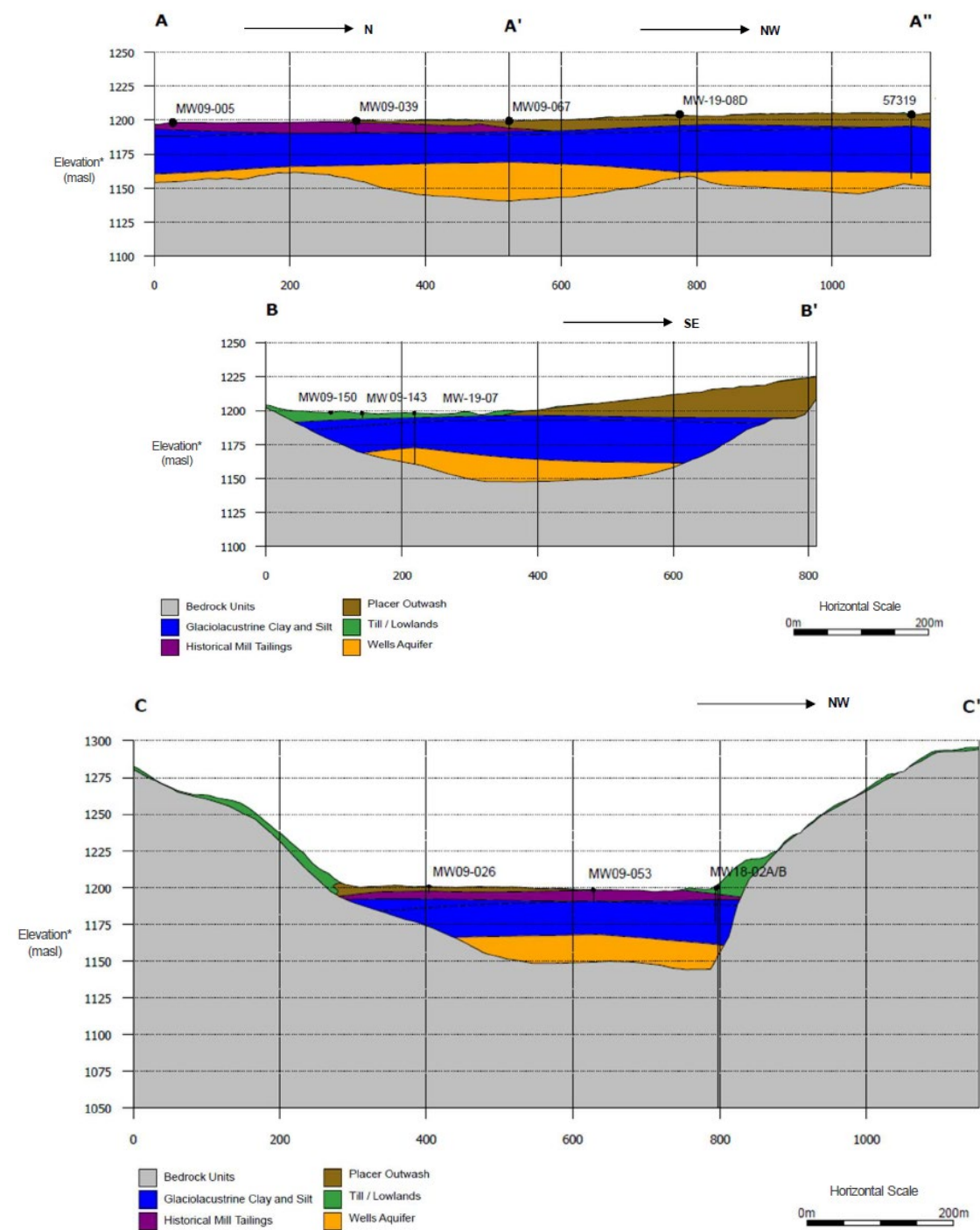


Figure 5-4 Geologic cross-sections through the Lowhee Creek fan (from Golder, June 24, 2021).

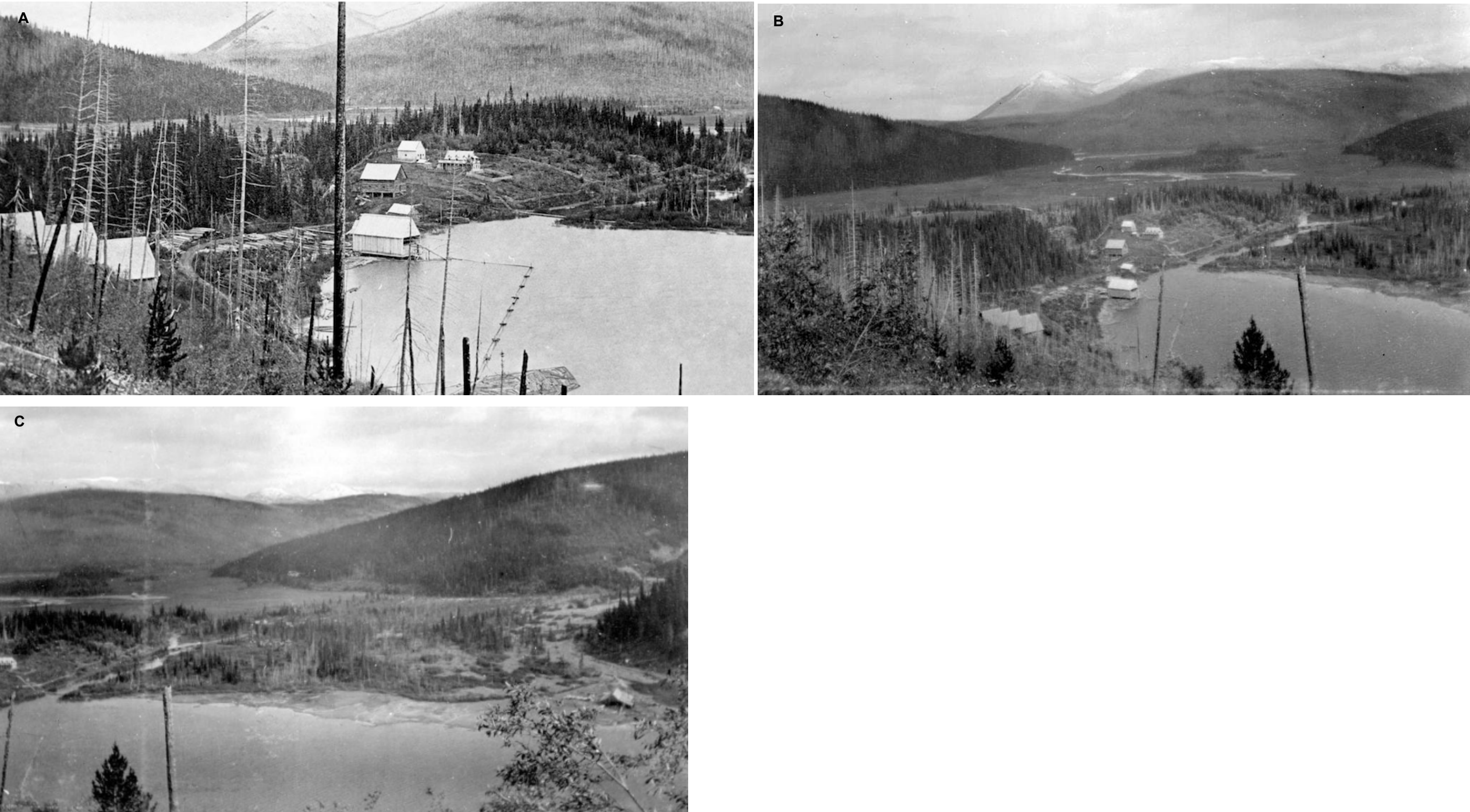


Figure 5-5 Archival photographs of the Clarke sawmill and the original outlet of Jack of Clubs Lake. Image A: 1901 (P-1326 Barkerville photograph collection. Acc. #1961.126.17.01). Image B: 1902 (BC Archives, Item I-56134). Image C: 1902 photograph looking east toward the alluvial fan of Lowhee Creek (BC Archives, I-56159).

An 1886 map also shows the outlet of Jack of Clubs Lake in the approximate location as seen in the photographs above (Figure 5-6), as does the 1923 map shown in Figure 5-1.

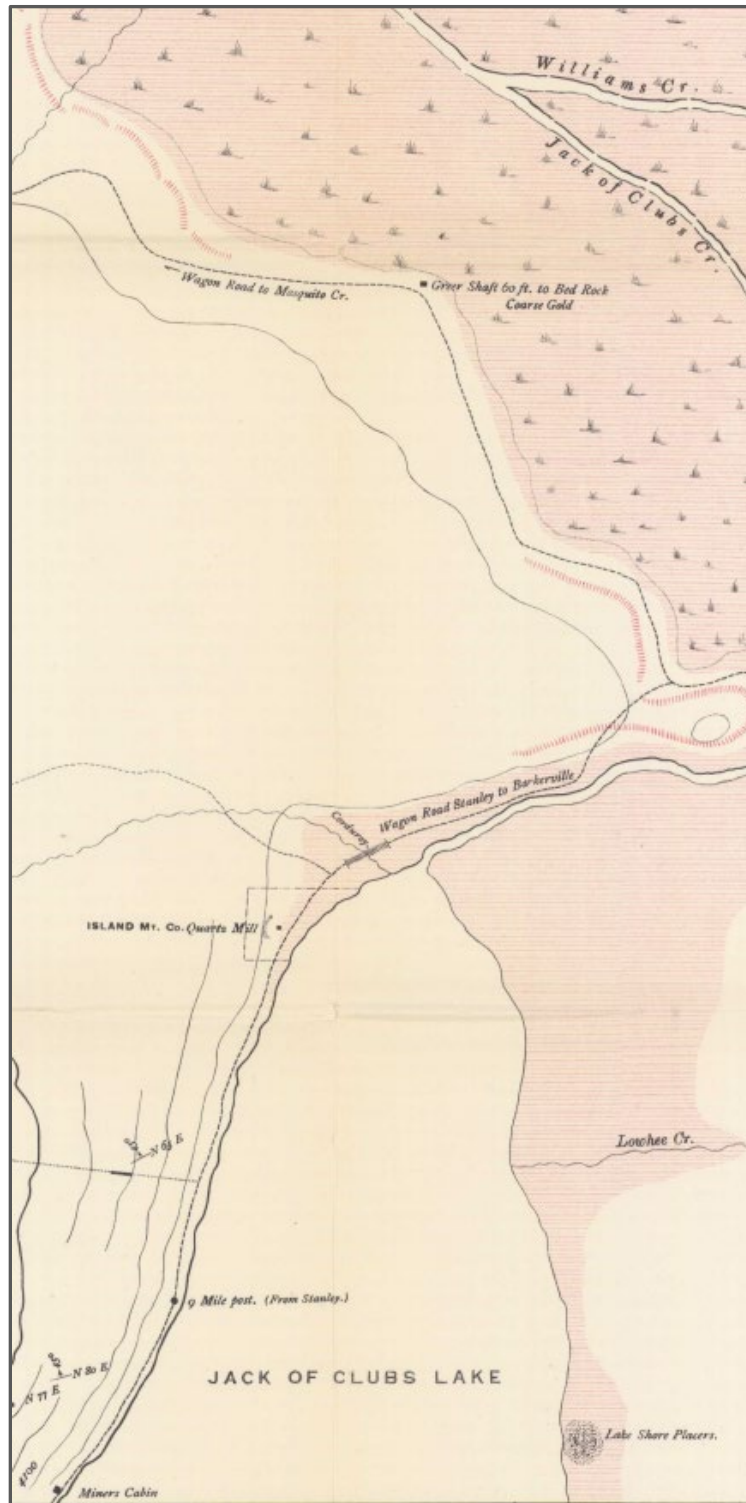


Figure 5-6 Excerpt of 1886 map of Wells area (Geological and Natural History Survey of Canada, 1886).

Finally, the Willow River has an unnaturally straight planform from the current outlet of Jack of Clubs Lake to just downstream of the Highway 26 crossing, a distance of about 1 km. This straight section of channel was constructed sometime after 1952 to allow the conveyance of lake outflows through the flotation tailings and distal hydraulically mined sediment deposits (Andrews, 1989). Drawing 07 shows how the outlet channel of Jack of Clubs Lake has changed between 1952 and 2024. The 2024 banklines are very similar to those mapped from the 1974 and 1991 air photographs.

The infilling of the lake outlet with sediments is also apparent when reviewing the channel profile downstream of Jack of Clubs Lake. The lidar profile in Figure 5-7 shows a relatively flat water surface between the lake outlet and the Highway 26 bridge (El. ~1196.4 m), before dropping abruptly about 1 m over a short distance. This change in gradient is also apparent in Figure 4-19.

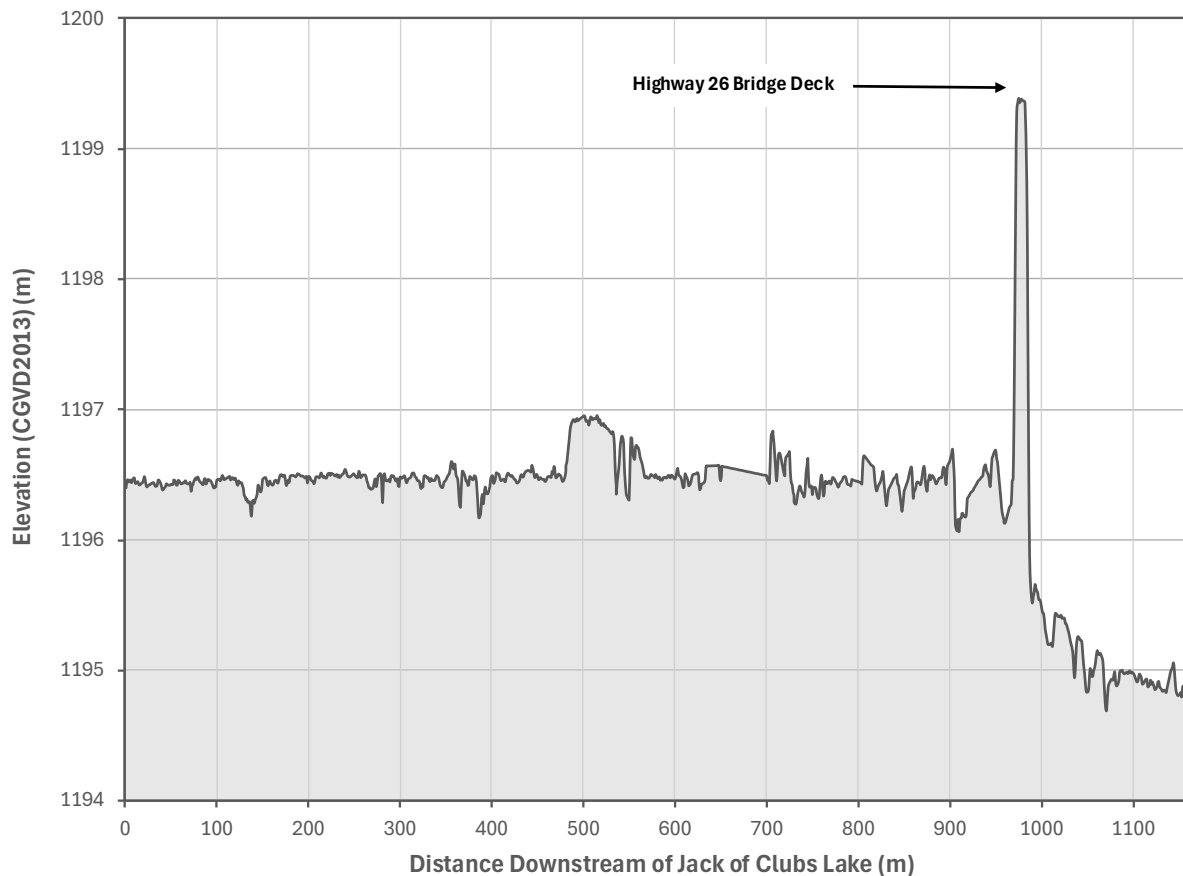


Figure 5-7 Willow River channel profile downstream of Jack of Clubs Lake using 2021 lidar data.

5.2 Lowhee Creek Geomorphic Assessment

5.2.1 May 2024 State of Emergency

Through the first part of the 21st century, Lowhee Creek generally flowed in a west to northwest direction toward Willow River and coarse sediment was broadly deposited over an area of the

fan approximately 150 m long and 75 m wide. Flows were reportedly shallowly dispersed over this area and there were no reports of gravel-sized sediment being deposited into the Willow River. During the May 2024 freshet Lowhee Creek changed course and now drains in a northwest direction and in lower reaches flows have been confined to a narrower channel. This shift has resulted in the formation of a small gravel bar in Willow River at the confluence, which has concentrated stream flow along the left bank and locally reduced the hydraulic conveyance of Willow River.

The DoW declared a state of emergency in May 2024 to address sediment deposition in Willow River from Lowhee Creek. BGC was retained by Radloff to provide hydrotechnical engineering support during the emergency response. BGC completed emergency engineering design of a temporary sediment basin to be constructed on the Lowhee Creek fan to capture sediment before it enters the Willow River (BGC, June 17, 2024). That emergency sediment basin was never constructed, although a more detailed design has since been completed (see Section 5.2.3).

The May 2024 state of emergency was the impetus for the riprap bank protection around the Highway 26 bridge crossing. It also led to a geomorphic assessment of the Lowhee Creek watershed to better understand sediment sources and expected future aggradation rates on the fan. That assessment is documented in BGC, November 8, 2024, and a summary is provided in the section below.

5.2.2 Watershed Assessment

BGC's 2024 geomorphic assessment analysis divided the watershed in three reaches: the upper watershed, the lower watershed, and the alluvial fan.

Upper Watershed

Within the Upper Watershed, the valley bottom and channel margins are densely vegetated. Some sediment reaches Lowhee Creek from landslides originating off the steep valley walls. However, the material reaching the stream channel does not appear to be readily transported downstream due to the dense riparian vegetation that traps and stores most of the sediment and woody debris. The typical gradient in this upper reach is 3%-5%, while the creek is approximately 2 m wide.

Lower Watershed

Within the Lower Watershed, the creek has incised into the sediments deposited behind the series of dams constructed to manage sediment from the historic mining activities in the watershed (Figure 2-3). Here the channel is confined by 5 to 10 m high, loose, eroding terrace walls, and the channel gradient ranges from 5 to 8%. The non-engineered dams have slowly deteriorated over the years, and the subsequent erosion of the trapped sediments is suspected to be the current primary source of sediment delivered to the fan (Figure 5-8 and Figure 5-9).

BGC also observed several old and unstable woody debris jams in the reach that appear to be holding back several thousand cubic meters of sediment. BGC suspects that some of these jams are remnants of the constructed dams previously noted.

The dams are holding back appreciable sediment and BGC believes that they will continue to contribute sediment at rates much higher than the typical background rates as Lowhee Creek erodes laterally. Sediment recruited from the Lower Watershed, as the dams fail, are expected to continue to impact the Lowhee Creek fan for decades.



Figure 5-8 Looking downstream at the Lower Watershed reach with exposed terrace walls. Photo source: BGC, August 7, 2024.



Figure 5-9 Looking upstream at a remnant dam and existing woody debris jam. Photo source: BGC, August 7, 2024.

Alluvial Fan

On the alluvial fan downstream of the BGM bridge, Lowhee Creek is approximately 20 m wide with a relatively featureless channel bed. The channel gradient is 5 to 7% for the upper 250 m of the alluvial fan, and 3 to 4% for the downstream 500 m of the fan, to the confluence with the Willow River. In this reach, there is abundant loose sediment (ranging from sand to gravel) on the channel bed that could be mobilized and transported downstream at high flow.

The rate of growth of the Lowhee fan was almost certainly increased several-fold due to the placer mining activities in the Lowhee Creek watershed. This increase in the sediment supply rate also likely increased the steepness of the fan (Whipple et al., 1998). If in future the supply of sediment to the fan is reduced (i.e., the placer deposits in the Lower Watershed become stabilized), the fan will likely respond by incising at the fan apex and extending the toe of the fan further to the north (Clarke et al., 2008; Vincent et al., 2022). As a result, sediment is likely to continue to reach the Willow River, where it will likely deposit.

The deposition of gravel into the Willow River is an issue as the creek has a very shallow gradient (0.05%) between the lake outlet and the Highway 26 bridge. This low gradient is consistent with this reach formerly being part of the lake and having infilled with placer and mining tailings, necessitating the excavation of a new channel through these sediments (Drawing 07). As a result, Willow River does not have the capacity to transport the volume and caliber of sediment delivered from Lowhee Creek during future flood events. Many alluvial fans

in British Columbia have reached a steady state configuration in which sediment moved by the fan channel is transported downstream by the watercourse flowing along the toe of the fan; as a result, such fans do not grow in size over time. Lowhee Creek is unusual in that the Willow River cannot transport the sediment volumes moved by Lowhee Creek, and the fan continues to expand over time indicating that it has not yet reached equilibrium.

Aggradation can be expected to occur on the fan along its entire length if the sediment supply from upstream remains at its current (high) level. If the upstream sediment supply is reduced, then erosion can be expected on the upper half of the fan, and deposition will continue on the lower half of the fan. In both cases, effective sediment management will be required to mitigate flood hazards in South Wells.

Sediment Transport Rates

Based on observations from a site visit and available lidar data, BGC (November 8, 2024) has estimated that roughly 186,000 m³ of sediment initially stored behind the dams was eroded and delivered to the Lowhee Creek fan between 1960 and 2019. This erosion volume represents an average deposit depth of approximately 1.7 m within the sediment deposition accommodation area (fan area).

Future releases of sediment from the Lower Watershed are likely to occur at rates that decline exponentially with time (Lisle & Church, 2002). The annual sediment volume is expected to decline from about 2500 m³/year at present to about 1100 m³/year in 100 years. BGC has estimated that it will take about 120 years for half of the sediment remaining behind the dams (i.e., 170,000 m³) to be eroded and delivered to the Lowhee Creek fan, which represents an average deposit depth of 1.6 m. Much of this accumulation is likely to be concentrated at the toe of the fan (where the transport capacity is lowest), with relatively little sediment expected to accumulate near the fan apex (where the transport capacity is highest due to steeper channel gradients and greater confinement).

Most gravel bed streams are limited by the supply of sediment to them, and transport sediment at a fraction of their capacity. BGC's estimates of the transport capacity of Lowhee Creek on the fan for a 200-year flood climate-adjusted peak flow of 35 m³/s varies from about 2,000 m³ at the toe of the fan to 5,000 m³ at the fan apex (BGC, March 28, 2024). The estimated transport capacity for Lowhee Creek is higher than the modelled sediment delivery rate, although both are reasonable estimates given the available data.

Estimated deposition volumes for various return period floods are summarized in Table 5-2. The amount of sediment delivered to the fan will vary with the magnitude of the peak flows in any given year, so these estimates reflect how the expected long-term average sediment delivery rate will change.

Table 5-2 Estimated deposition volumes on Lowhee Creek fan for various climate-adjusted flood quantiles (after BGC, March 28, 2024).

Climate-Adjusted Return Period (Years)	Potential Deposition Volume (m³)
2	170
5	660
10	1,000
25	1,400
50	1,800
100	2,200
200	2,900

5.2.3 Sediment Management Plan

As part of the permitting for the Phase 2a and 2b dikes, BGC (November 26, 2024; May 15, 2025a) developed a sediment management plan (SMP) for Lowhee Creek. The SMP is summarized in Table 5-3 and describes measures to:

- Retain and excavate sediment reaching the lower Lowhee Creek fan to reduce flood hazard to South Wells adjacent to Lowhee Creek and to limit sediment deposition in the Willow River
- Monitor sediment transport and deposition patterns to improve understanding of sediment dynamics and inform future management strategies.

Table 5-3 Sediment management measures related to the upper sub-reach of the Lowhee Creek alluvial fan.

Sediment Management Measure	Description	Priority	Timeline
Sediment Basin	A sediment basin is proposed to retain sediment delivered to the lower fan. Excavation of the sediment basin will be required.	High	1 year
Lidar survey ¹	Repeat lidar surveys should be completed to assess if the volume of sediment deposited in the Alluvial Fan is following the expected rate of deposition estimated by BGC (November 8, 2024).	High	Every 5 years starting in 2025 Post Freshet
Topographic surveys ¹	Yearly topographic surveys (i.e., survey of channel cross-sections spaced every 50 m and channel thalweg) should be completed to compare year-over-year channel bed elevations.	High	Yearly Post Freshet
Dredging	Dredging upstream of the sediment basin may be required in the channel bed, particularly if the sediment supply to the fan remains high. Frequency of dredging will be informed by the measured sediment deposition rate measured using lidar and repeat surveys (See Section 3.2).	Moderate	2-10 years

Note:

1. Data collection should be completed by a qualified survey company. Data analysis and interpretation should be completed by a qualified professional.

Sediment Basin

The sediment basin (referred to as Phase 2c of the Lowhee Creek dike work) is intended to retain sediment delivered to the toe of the fan before it enters the Willow River, supporting flood hazard mitigation along Lowhee Creek as well limiting the potential flood hazard in the Willow River. It is designed adjacent to the downstream end of the proposed Phase 2b dike (Figure 5-10). IFC drawings and a detailed design for the sediment basin have been prepared by BGC (May 15, 2025b; September 19, 2025).



Figure 5-10 General arrangement of the Lowhee Creek Phase 2c sediment basin (BGC, September 19, 2025).

The Lowhee Creek sediment basin features an upstream reach 60 m in length with a 10% slope, narrowing in width from 12 m at the upstream end to 5 m at the bottom of the basin. The upstream 10% slope will be lined with Class 500 kg riprap and will consist of a series of steps with vertical height difference of 0.3 m between steps and pool length of 1.7 m. A V-notch channel will be incorporated at the centre of the steps to convey low flows. The steps and V-notch channel are design features to allow for fish passage.

The right bank will be set back at a 2H:1V slope and armoured with Class 500 kg riprap where adjacent to the upstream 10% slope. In areas where the basin is flat, the right bank slope will be armoured with Class 100 kg riprap to match Phase 2b dike armouring. An access road lined with Class 100 kg will be constructed above the right bank of the sediment basin to facilitate excavation of sediment. The access road will provide sufficient width for an excavator to access the basin for excavation. The left bank of the sediment basin will be set back at a 3H:1V slope and will be unarmoured, as there is no critical infrastructure requiring protection beyond the bank. The outlet of the basin consists of a 0.5 m tall weir that flows into a 5 m wide low-flow outlet channel.

The basin is designed to retain up to 3,000 m³ of sediment but the total sediment deposition during any given year remains uncertain and will vary depending on sediment supply and flood magnitude. The sediment basin should be monitored yearly post freshet to determine the need for excavation (sediment removal). BGC has recommended that the DoW be prepared to excavate the sediment basin annually for the first 3 years of operation.

A temporary storage location for excavated sediment has been identified immediately north of the Phase 2b dike, adjacent to the access road (Radloff, July 16, 2025; January 21, 2026). There is approximately 7,900 m³ of available storage volume at this location, which is expected to be sufficient for the first 3 to 5 years of sediment basin operation, depending on flood magnitude and sediment deposition patterns. The DoW is coordinating with Osisko regarding long-term management and off-site disposal of sediment captured by the basin. The sediment is not suitable for general fill as environmental testing shows arsenic and iron levels that exceed certain BC Contaminant Sites Regulation (CSR) thresholds (Radloff, July 16, 2025). All other metals are below regulatory standards, and the sediments are non potentially acid generating (non-PAG).

Monitoring

Sediment transport and deposition patterns should be monitored to observe the pattern of sediment accumulation on the fan using regular repeat lidar surveys every 5 years and annual cross-section surveys spaced at 50 m intervals. If conditions change and there is excessive sedimentation along the Lowhee Creek dike system (Phase 1 or 2 dikes) that is threatening the minimum freeboard of the dike, additional dredging may be required in the channel. A minimum freeboard of 1.0 m has been specified for the dikes. The provincial standard for dike freeboard is 0.6 m. If repeat surveys detect a rise in mean bed elevation relative to the initial survey of more than 0.4 m (i.e., a reduction of freeboard to approximately 0.6 m), BGC has recommended dredging to maintain the approximate specified freeboard of 1.0 m. BGC also recommended updating the hydraulic model (using repeat lidar and cross-section survey data) every 5 years or following a significant flood event to update the estimated freeboard.

Lower Watershed Stabilization

While the SMP does not discuss stabilization of the Lower Watershed, BGC (November 8, 2024) does comment on the potential benefits of constructing instream works to stabilize the channel within the area of stored sediment. Such works would prevent lateral migration and/or vertical incision of Lowhee Creek within the area of stored sediment, reducing the sediment supply to the Lowhee Creek fan to background levels typical for undisturbed watersheds. While additional assessment and design is required, successful stabilization of the Lower Watershed would reduce the rate of sediment accumulation on the fan and the frequency and volume of any future sediment removals on the fan that may be required to maintain suitable freeboard for the Phase 1 and Phase 2 dikes.

5.3 Williams Creek Channel Changes

Similar to Lowhee Creek, placer mining has resulted in significant disturbance to Williams Creek from upstream of Stouts Gulch to a few hundred meters downstream of the Barkerville wastewater lagoons (Drawing 08). That disturbance has resulted in significant volumes of sediment being introduced into Williams Creek, which have slowly been transported downstream over the last 150 years.

The 1952 air photograph of the area shows:

- Significant disturbance by hydraulic placer mining along the full length of Stouts Gulch.
- Upstream of Stouts Gulch the right (east) bank of Williams Creek shows prior disturbance but the beginnings of revegetation.
- Within the Barkerville townsite, Williams Creek varies in width from 15 m to 60 m, with an average width of 35 m.
- Between Barkerville and Bowron Lake Road, Williams Creek widens to an average width of approximately 90 m.
- In the vicinity of the junction of Highway 26 with Bowron Lake Road, Williams Creek discharges to a much broader valley floor that is up to 600 m wide. Here, Williams Creek has formed an alluvial fan that radiates out to the far side of the valley. As a result, elevations in this area are the lowest on the north side of the valley where Valley Creek flows from east to west. The 1952 air photograph shows sediment being deposited onto the alluvial fan in two distinct splays: one to the northeast and one to the northwest.
 - Historical maps by Bowman (1895) and Johnson and Uglow (1925) suggest that the northwest flow path was the pre-disturbed alignment of Williams Creek.
- Valley Creek, located on the north side of the broad valley, is observed to be a meandering, 5 to 7 m wide creek. Valley Creek is now mostly a large wetland area.

Georeferenced aerial imagery from 1974, 1991 and 2024 show that Williams Creek has become more stable over time with a progressively narrower channel developing between Stouts Gulch and the Bowron Lake Road crossing, a distance of approximately 2.5 km (Table 5-4) (Drawing 09).

Table 5-4 Williams Creek average channel width over time.

Reach	Average Channel Width (m)			
	1952	1974	1991	2024
Stouts Gulch to Barkerville	35	15	14	8
Barkerville to Bowron Lake Road	90	63	51	16

Downstream of Bowron Lake Road, Williams Creek widens to between 50 m and 60 m for a distance of about 700 m. This section of the creek has a braided morphology and shows little change in width between 1974 and 2024 (Drawing 10). Further downstream, the creek maintains a braided morphology but narrows to between 25 m and 35 m (2024 banklines) for a further 700 m. This lower 700 m reach is up to two times wider on the 1974 and 1991 air photos (Drawing 10). The position of the 1974 banklines suggest that Williams Creek was constricted

and pushed to the northeast to accommodate construction of the Barkerville sewage lagoons in the early 1980s. Beyond this disturbed reach, Williams Creek adopts a single-thread, moderately sinuous channel planform with a bankfull width of approximately 10 m to 12 m.

5.4 Williams Creek Geomorphic Assessment

The aerial photograph and lidar evidence presented in Section 5.3 documents progressive channel narrowing and incision upstream of Bowron Lake Road over the past seven decades, reflecting the passage of mining-derived sediment through the Barkerville reach. Flooding and aggradation are no longer active concerns in the townsite. Downstream of Bowron Lake Road the situation is fundamentally different.

As Williams Creek exits the culvert beneath Bowron Lake Road it transitions onto a broad, low-gradient alluvial fan where sediment transport capacity drops sharply. To characterize the texture of bed material along this reach, BGC conducted Wolman pebble counts at four locations between approximately 700 m upstream of Bowron Lake Road and the One-Mile Road crossing (Table 5-5). The results document a pronounced downstream fining of bed material, with D_{50} declining from 41 mm at Bowron Lake Road to 20 mm just 700 m downstream. This pattern of abrupt downstream fining is characteristic of a reach transitioning from sediment transport to sediment deposition. Further, it is consistent with the braided channel morphology and low channel gradient visible in the channel profile (Figure 5-11).

Table 5-5 Wolman sampling results along Williams Creek.

Percentile	Diameter (mm)			
	Williams 1	Williams 2	Williams 3	Williams 4
D16	24	22	11	< 8
D50	47	41	20	10
D84	92	91	34	19
D90	112	106	40	24

Note: The samples were taken at four locations:

1. Williams 1 – 700 m upstream of Bowron Lake Road.
2. Williams 2 – at Bowron Lake Road.
3. Williams 3 – 700 m downstream of Bowron Lake Road.
4. Williams 4 – at 1-Mile Road crossing.

BGC estimated bedload transport capacity at nine cross-sections between Bowron Lake Road and the Barkerville wastewater lagoons using lidar-derived channel geometry, the lidar-derived longitudinal profile for local slope estimates, and D_{50} values interpolated from the Wolman data using an exponential downstream fining function fitted to the three downstream sample sites (Figure 5-12). At each cross-section, local water depths were computed directly from the lidar-surveyed bed elevations for a series of incrementally increasing water surface elevations. Cross-sectional area, wetted width, and wetted perimeter were integrated across the wet portion of the section at each increment, and reach-average hydraulic radius, velocity, and shear stress were used to estimate bedload transport using the Eaton and Church (2010) transport function.

Transport rates were then integrated over a flow duration curve¹⁸ derived from the *Little Swift River at the Mouth* record (08KE024) (see Section 3.5), scaled to Williams Creek by drainage area ratio.

The results indicate that transport capacity is approximately constant where Williams Creek is confined by dikes on the left and right banks, but then declines systematically in the downstream direction between the end of the confined section and the wastewater lagoon, implying that sediment supplied from upstream cannot be passed through this reach under typical flow conditions and that deposition is the expected outcome (Figure 5-12). The rate of aggradation implied by the loss of transport capacity between successive cross-sections ranges from approximately 0.5 m per decade in the reach immediately upstream of the lagoon to approximately 2.2 m per decade adjacent to them¹⁹. These estimates are likely upper bounds, as transport equations often overestimate capacity. Comparison with the 2021 lidar DEM provides an independent check: the Williams Creek channel bed alongside the Lagoon dike appears to be 1.0 m higher than the interpreted channel bed prior to construction of the wastewater lagoons in the early 1980s, suggesting that meaningful aggradation has occurred since the lagoon dike was constructed. Taken together, an aggradation rate of approximately 0.5 m per decade adjacent to the lagoon dike is considered the best central estimate.

Aggradation in this area has implications for the annual release of treated effluent from the lagoon system. The treated effluent is discharged in the spring through a 300 mm diameter outfall pipe that extends about 80 m downstream along the drainage ditch that has formed between the Lagoon dike and berm of Cell #3 (Figure 4-12). Aggradation at the outfall pipe outlet would compromise the ability of the system to discharge the treated effluent and accurately record the effluent volume, which is measured with a flow meter.

The lagoon dike currently protects the drainage ditch and outfall pipe from aggradation, as it stands approximately 1.5 to 2 m above the Williams Creek channel bed. If aggradation proceeds at the rates estimated above, the effective freeboard of the lagoon dike could be substantially reduced within one to two decades under existing conditions, independent of any large flood event. Also, the dike has no riprap protection and could be significantly eroded during a single flood event or series of events. Ongoing bed surveys and repeat lidar are therefore warranted to track aggradation rates and inform the timing of any required

¹⁸ Flows below approximately 2 m³/s were excluded from the calculations. BGC assumes that these flows contribute negligible bedload transport. The threshold is equivalent to the smallest daily peak flow recorded on Little Swift River.

¹⁹ The modelled aggradation rates of 0.5 to 2.2 m per decade are similar to the rates that likely prevailed during and immediately following the period of active hydraulic placer mining. Historical records document that the April 1879 flood deposited sufficient placer tailings in the Barkerville townsite that buildings had to be raised by as much as 5 m, and flooding was so frequent in the decades following the gold rush that minor events went largely unreported (Quackenbush, 1997). The progressive widening of Williams Creek to an average of 90 m between Barkerville and Bowron Lake Road by 1952 — compared to just 16 m today — reflects the enormous volumes of sediment that moved through this reach during and after the mining period. BGC assumes that the actual current rates downstream of Bowron Lake Road are lower than the model estimates (i.e., closer to 0.5 m per decade) and likely represent the tail end of a long-term sediment wave that has been attenuating since mining ceased. Nevertheless, even at these attenuated rates, the limited freeboard of the Lagoon dike makes the infrastructure vulnerable on a management-relevant timescale.

intervention. Potential interventions include erosion protection along the lagoon dike and raising of the lagoon berms.

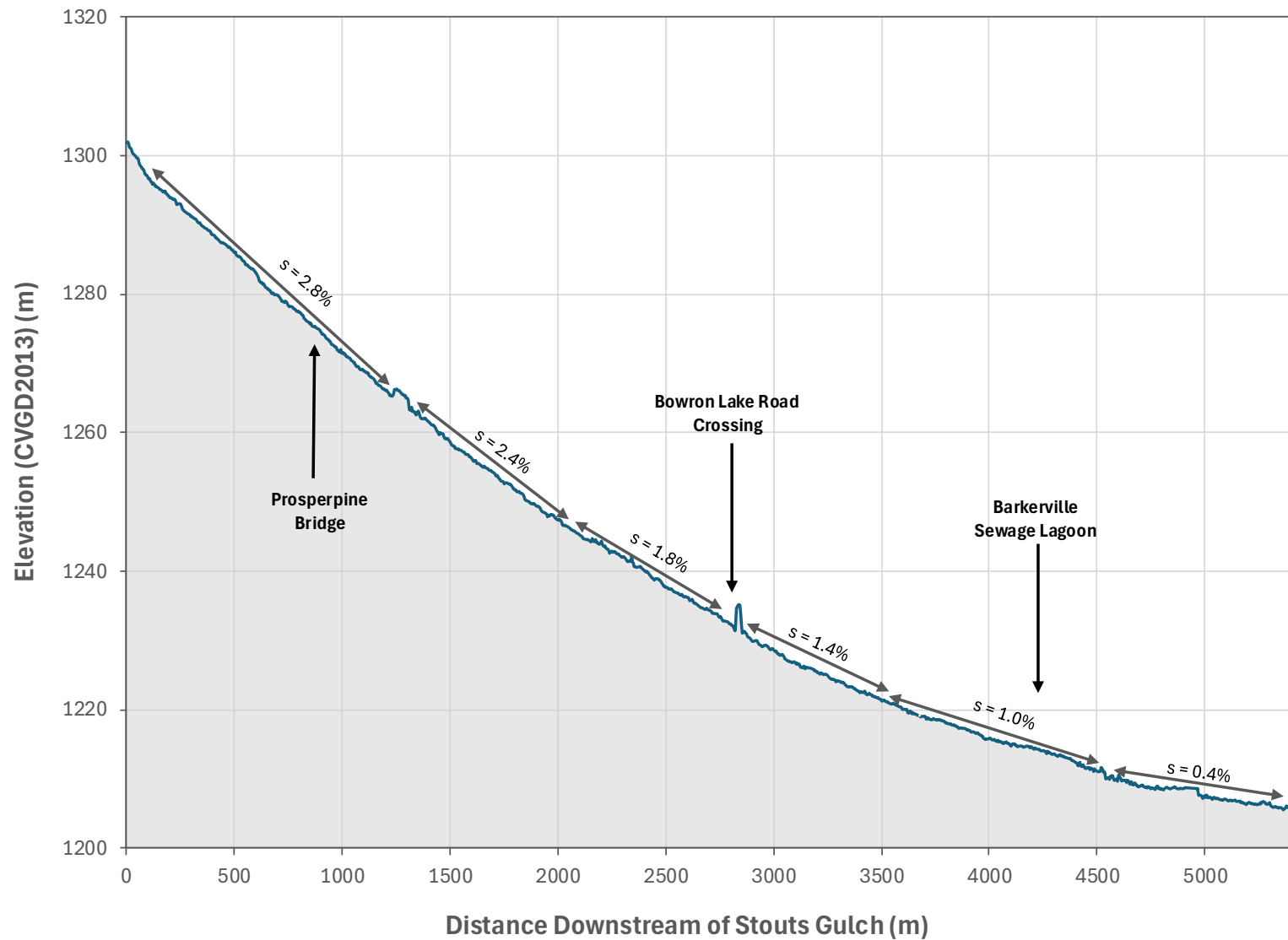


Figure 5-11 Williams Creek channel profile downstream of Stouts Gulch using 2021 lidar data.

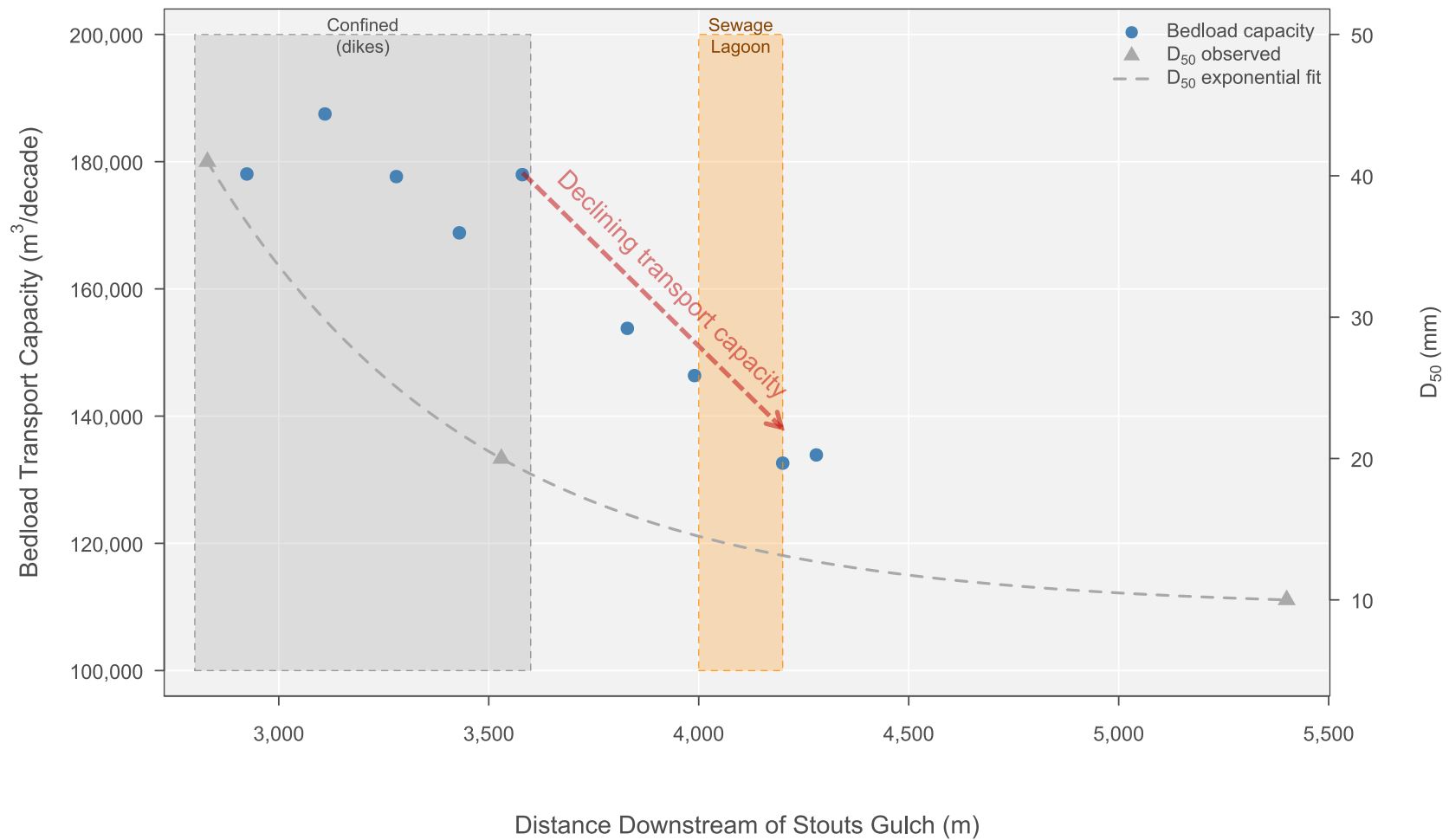


Figure 5-12 Williams Creek bed texture and transport capacity downstream of Bowron Lake Road. The section of stream confined by dikes on both sides of the creek is indicated in grey. The location of the Barkerville wastewater lagoons is indicated in orange.

6.0 HYDROTECHNICAL METHODS AND INPUTS

This section summarizes the hydrotechnical assessment methodology applied to the RFHA study area and key inputs.

6.1 Hydrologic Analysis

6.1.1 Previous Work

Peak discharge estimates for Willow River adjacent to Wells were previously developed by BGC to provide engineering support for the replacement of the sanitary sewer aerial crossing located downstream of the Highway 26 bridge (BGC, February 22, 2024). That work included the development of a rainfall runoff (hydrologic) model based on regional and local precipitation data and watershed characteristics to estimate peak flows in Willow River at the sanitary sewer aerial crossing for different return periods. The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's Hydrologic Modelling System (HEC-HMS) version 4.11 (USACE, 2022) software was used to create the model. The Willow River at sanitary sewer aerial crossing encompassed the Lowhee Creek and Jack of Clubs Creek watersheds.

6.1.2 Watershed Characteristics

The hydrologic framework that was previously developed and accepted for the Jack of Clubs and Lowhee Creek watersheds was applied consistently to the remaining watershed areas within the RFHA study area using watershed-specific inputs. HEC-HMS version 4.12 (USACE, 2024) was used for this modelling effort.

The Willow River watershed was subdivided into tributary-based sub-watersheds, including Jack of Clubs Creek, Downey Creek, Lowhee Creek, and Williams Creek (Figure 6-1). The Jack of Clubs and Williams Creek watersheds were further discretized into smaller sub-watersheds for the hydrologic analysis. These subdivisions were defined to provide appropriate flow inputs to the hydraulic model and to capture additional tributaries and spatial variability upstream of key confluences (e.g., Stouts Gulch, Conklin Gulch, and Valley Creek).

Figure 6-1 presents the sub-watersheds included in the hydrologic analysis. The delineation and characteristics of the Jack of Clubs and Lowhee Creek watersheds are consistent with those adopted in BGC's previous analysis (BGC, February 22, 2024).

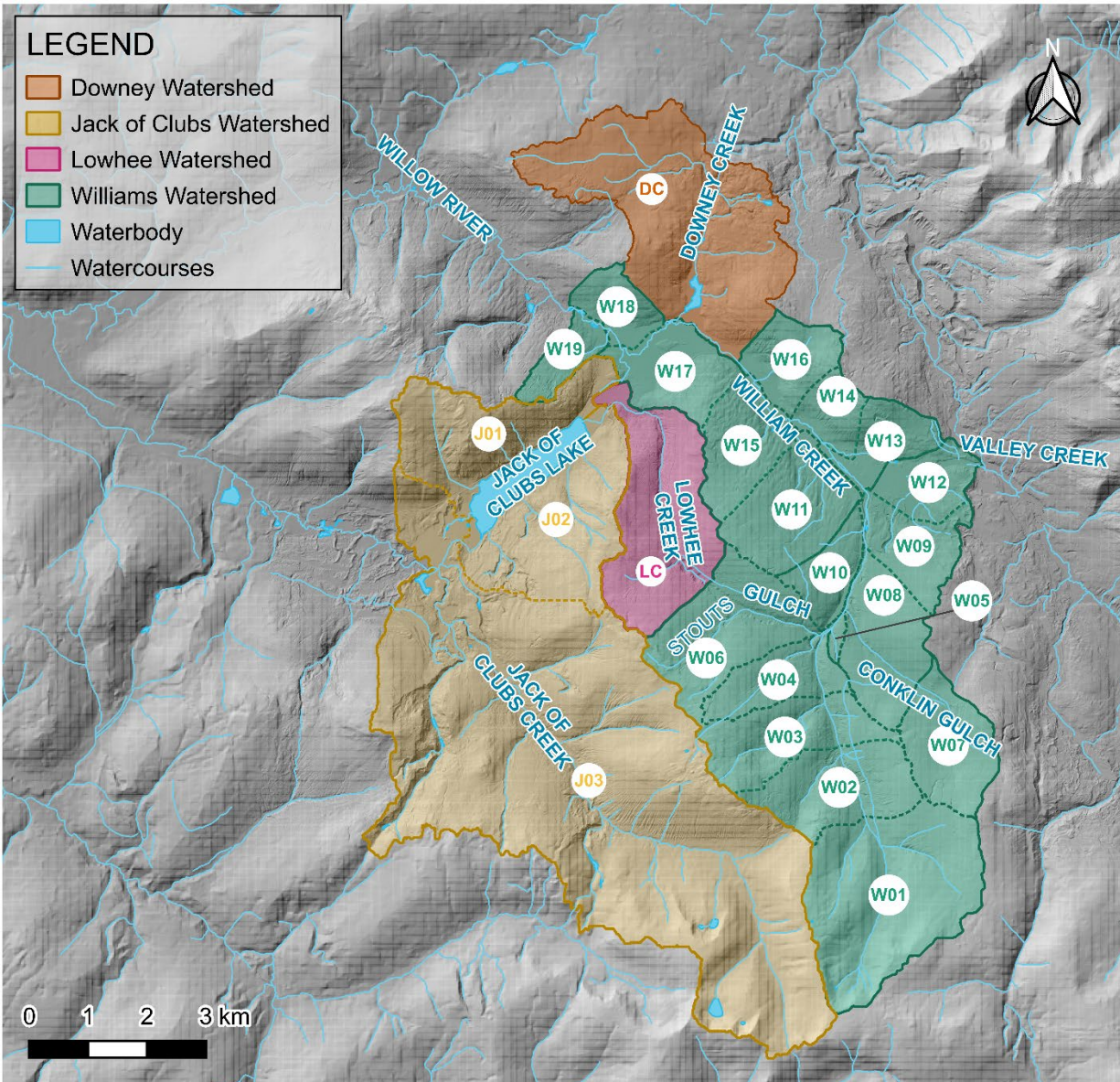


Figure 6-1 Delineation of watersheds and sub-watersheds (dashed lines) that outlet to Willow River. The background hillshade is from the Canadian Digital Elevation Model (NRCan, 2017).

For each sub-watershed the following parameters were estimated (values provided in Table 6-1):

- The sub-watershed area (A, in km²), estimated based on terrain models available from the 2021 lidar²⁰ and the Canadian Digital Elevation Model (CDEM) (NRCan, 2017). Note, the lidar covered about 75% of the watershed area.
- A runoff curve number (CN, dimensionless) value, which is an empirically derived relationship between soil type, land use (e.g., mature forest, agricultural fields, etc.),

²⁰ The 2021 lidar covers most, about 75%, but not all of the study area.

and antecedent moisture conditions (AMC). The CN is used to determine the runoff potential of a hydrological unit as a response to a given storm event (USDA, 1986). The CN value for natural ground (which represents the majority of the total watershed area) was selected based on the soil type and land cover from the Global CN gridded dataset (GCN250) (Jaafar et al., 2019). Average antecedent moisture conditions (AMCII) were used in the analysis.

- Percent imperviousness that represents the portion of the watersheds where all rainfall can be assumed to become runoff (i.e., water surfaces).
- The potential maximum soil moisture retention (S , in mm), which is a function of the CN (USDA, 1986).
- The initial abstraction (I_a , in mm) which represents the depth of precipitation that is infiltrated or intercepted (depression storage, vegetation interception) before runoff initiates. I_a is a function of S and was assumed to be 20% of S (USDA, 1986).
- The lag time (T_{lag} , in minutes), which represents the length of time between the centroid of the excess rainfall during the simulated storm event and the peak of the hydrograph at the outlet. T_{lag} is a measure of the hydrologic response of the watershed. T_{lag} depends on watershed characteristics such as longest stream length and average gradient, that were obtained using terrain models available from the CDEM. T_{lag} for the various sub-watersheds was estimated using the SCS T_{lag} equation (USDA, 1986).

Table 6-1 Sub-watersheds characteristics.

ID	Area (km ²)	Average Slope	Length (km)	CN (AMCII)	Imperviousness	Lag Time (min)
DC	10.40	20%	6.7	73	1%	54
W01	7.46	29%	3.5	72	0%	49
W02	3.23	28%	2.3	73	0%	24
W03	3.01	29%	2.6	72	0%	32
W04	1.98	26%	2.7	74	0%	31
W05	0.18	25%	1.0	76	0%	16
W06	4.12	25%	3.8	73	0%	51
W07	2.79	19%	2.6	72	0%	38
W08	3.15	20%	2.4	73	0%	36
W09	2.04	21%	2.7	73	1%	39
W10	1.39	23%	2.6	75	0%	18
W11	3.28	20%	3.2	72	0%	41
W12	1.64	11%	1.8	74	16%	313
W13	1.58	20%	1.7	76	2%	36
W14	1.17	28%	1.0	73	0%	17
W15	2.42	24%	3.0	72	0%	41
W16	1.82	25%	2.2	74	0%	40
W17	2.44	15%	3.3	74	0%	111
W18	1.29	24%	1.2	73	0%	46
W19	1.05	20%	2.1	72	1%	18
J01	4.5	32	2.3	73	13%	21
J02	6.2	27	2.6	73	13%	25
J03	34.1	30	13.0	74	0	83
LC	5.8	29	4.7	74	0%	38

6.1.3 Rainfall Inputs

BGC used precipitation data included in the Cariboo Gold Project Mine Site Climate Existing Conditions Report (Golder, April 26, 2022). Golder conducted an analysis of extreme daily rainfall, using records from the ECCC Barkerville climate station.

The Barkerville climate station, located 5.5 km southeast of Wells, provides rainfall data from 1888 to 2015. The recorded daily rainfall depths were transformed into 24-hour rainfall depths using a multiplier of 1.13 (Golder, April 26, 2022).

Golder used the rain-on-snow total depths provided by the ECCC western Canada mountain basin degree-day model for the Barkerville climate station. BGC confirmed these rain-on snow total depth by applying the Dunne and Leopold (1978) equation to the daily rainfall (BGC, February 22, 2024).

Table 6-2 summarizes the 24-hour rainfall and rain-on-snow depths for select return periods. Rain-on-snow depths are included as larger 24-hour storms typically occur between May and September, which overlaps with the period of seasonal snowpack in the Project area. No storm area reductions were applied to the rainfall depths.

Table 6-2 24-hour rainfall depths for the Willow River watershed for various AEPs.

AEP (%)	Return Period (Years)	24-hr Rainfall Depth (mm)	1-day Rain-on-Snow Depth (mm)
2.0	50	60	74
1.0	100	66	81
0.5	200	74	90
0.2	500	84	100

The SCS Storm Type 2 distribution was applied to all rainfall and the rain-on-snow depths. The SCS Type 2 storm is often applied to central British Columbia (Millar, 2017) and is consistent with the Intensity-Duration-Frequency rainfall data from the Prince George Airport climate station, located approximately 115 km northwest of Wells (BGC, February 22, 2024).

6.1.4 Climate Change

The climate change assessment was completed using the same input data and approach as in BGC (February 22, 2024). Climate change adjustment factors were derived using the Western University (UWO) IDF_CC Tool (Version 7.0; Simonovic et al., 2015), based on an ensemble of Coupled Model Intercomparison Project Phase 6 (CMIP6) global climate models with Pacific Climate Impacts Consortium (PCIC) bias correction. The Shared Socioeconomic Pathways (SSPs) SSP2-4.5 (roughly equivalent to the Representative Concentration Pathway (RCP) 4.5 emission scenario), which represents a medium (or “middle of the road”) pathway of future greenhouse gas emissions was selected. The SSP2-4.5 scenario yields approximately 2.7°C of

warming by 2100 (IPCC, 2021). This is consistent with the range of projected warming under current global policy trajectories 2.8°C to 2.9°C (UNEP, 2025; IEA, 2025).

Climate change impacts on rainfall were estimated using the IDF_CC Tool and the project rainfall increase (in mm) was added to the historical rain-on-snow daily precipitation depths (Table 6-3). Under the SSP2-4.5 scenario, the 24-hour 50-year and 100-year rainfall event was projected to increase by 6 and 8 mm, respectively. The IDF_CC tool produces results only up to the 100-year return period. To obtain an increase in daily rainfall depth to the 200-year and 500-year, BGC assumed that the climate-adjusted rainfall depths increase by 17%, consistent with the 100-year return period.

Table 6-3 Increase in the rainfall depth and rain-on-snow depth from climate change to 2100 (SSP2-4.5 scenario).

AEP (%)	Return Period (Years)	Increase in Daily Rainfall (mm)	Climate-Adjusted 24-hr Rainfall Depth (mm)	Climate Adjusted 24-hr Rain-on-Snow Depth (mm)
2.0	50	6	66	81
1.0	100	8	74	89
0.5	200	16 ¹	90	105
0.2	500	17 ¹	101	117

Note:

1. The IDF-CC tool from UWO produces results only up to the 100-year return period. To obtain an increase in daily rainfall depth, BGC assumed that the climate-adjusted rainfall depths for the 200- and 500-year return periods increases by 17%, consistent with the 100-year return period.

6.2 Hydraulic Modelling

6.2.1 Scenarios

Three scenarios were selected for flood hazard mapping (Table 6-4). The scenarios consist of rain-on-snow conditions. Climate change adjustments, based on the SSP2-4.5 scenario, were applied to the 200-year (0.5% AEP) rainfall event and the 500-year (0.2% AEP) rain-on-snow event to account for impacts from climate change. It is noted that the climate adjusted 200-year, 24-hour rainfall-only event has the same depth as the 200-year rain on snow event (no climate adjustment). As both events would be modelled using the same SCS Type II temporal distribution, they would produce identical precipitation inputs and are therefore considered equivalent scenarios. So, the flooding generated by scenario 1 in Table 6-4 would also be representative of the flooding generated by a climate-adjusted 200-year, 24-hour rainfall-only event.

Table 6-4 Hydraulic modelling scenarios.

Scenario	AEP (%)	Return Period (Years)	1-Day Rain-on-Snow Depth (mm)
1	0.5	200	90
2	0.5 + climate adjustment ¹	200 + climate adjustment ¹	105
3	0.2 + climate adjustment ¹	500 + climate adjustment ¹	117

Note:

1. Climate adjustments consist of the SSP2-4.5 climate change scenario.

6.2.2 Modelling Software

BGC developed a 2D (depth-averaged) hydraulic model of the area using HEC-RAS (Hydrologic Engineering Center – River Analysis System) version 6.6, a publicly-available software created and distributed by the USACE.

6.2.3 Previous Work

A two-dimensional (2D) hydraulic model of Lowhee Creek and Willow River was previously developed to support the design of the Phase 2 Dike on Lowhee Creek (BGC, October 11, 2024a). The model was specifically developed to inform the ultimate crest elevation, erosion protection measures, and scour depth estimates using simulated water surface elevations, flow depths, velocities, and shear-stresses during the design flood. The model domain covered lower reaches of Lowhee Creek (approximately 850 m length from upstream from the BGM Bridge to the confluence with the Willow River) and a section of the Willow River (approximately 950 m between the outlet of Jack of Clubs Lake to downstream of the Highway 26 bridge).

The modelling scenario consisted of the design event for the Lowhee Creek Dike: the 500-year rain-on-snow, climate adjusted peak flow (BGC, February 22, 2024; September 13, 2024). The inflow hydrograph for Lowhee Creek was increased by 5% to account for potential addition of sediment loading (BGC, September 13, 2024).

The model was created using a 0.5 m resolution digital Elevation Model (DEM). Because the model results were used to inform the dike design, the DEM consisted of the 2021 terrain lidar, topographic and bathymetric surveys conducted by Radloff between May 2023 and June 2024, and the proposed Phase 2 dike design.

6.2.4 Model Terrain

For the RFHA, a digital elevation model (DEM) with a resolution of 3 m was generated using the 2021 lidar (Figure 6-2). The lidar data consist of a bare earth surface (e.g., buildings and trees removed) and includes the water surface. As a result, channel bed details are not fully resolved in some areas. The following modifications were applied to the DEM:

- Disconnected channels along the floodplain adjacent to Williams Creek between Bowron Lake Road and the confluence of the Willow River were connected using representative channel geometry (i.e., average width, side slopes, and longitudinal slope).
- The elevation of the surface area of Jack of Club lake in the DEM was adjusted to be 0.05 m below the invert elevation of 1196.25 m (Golder, April 12, 2022) to allow water to freely discharge out of its outlet to Willow River.
- Bridges were removed to allow continuous flow within the channels (see Section 6.2.8).

To incorporate the proposed Phase 2 dike design along Lowhee Creek, the terrain developed for the Lowhee Creek hydraulic model (BGC, October 11, 2024a) was used, including the merged bathymetric survey data. As such, Lowhee Creek and the Willow River between the outlet of Jack of Clubs Lake and the Highway 26 bridge crossing are the only reaches where the terrain reflects the channel bed instead of the water surface.

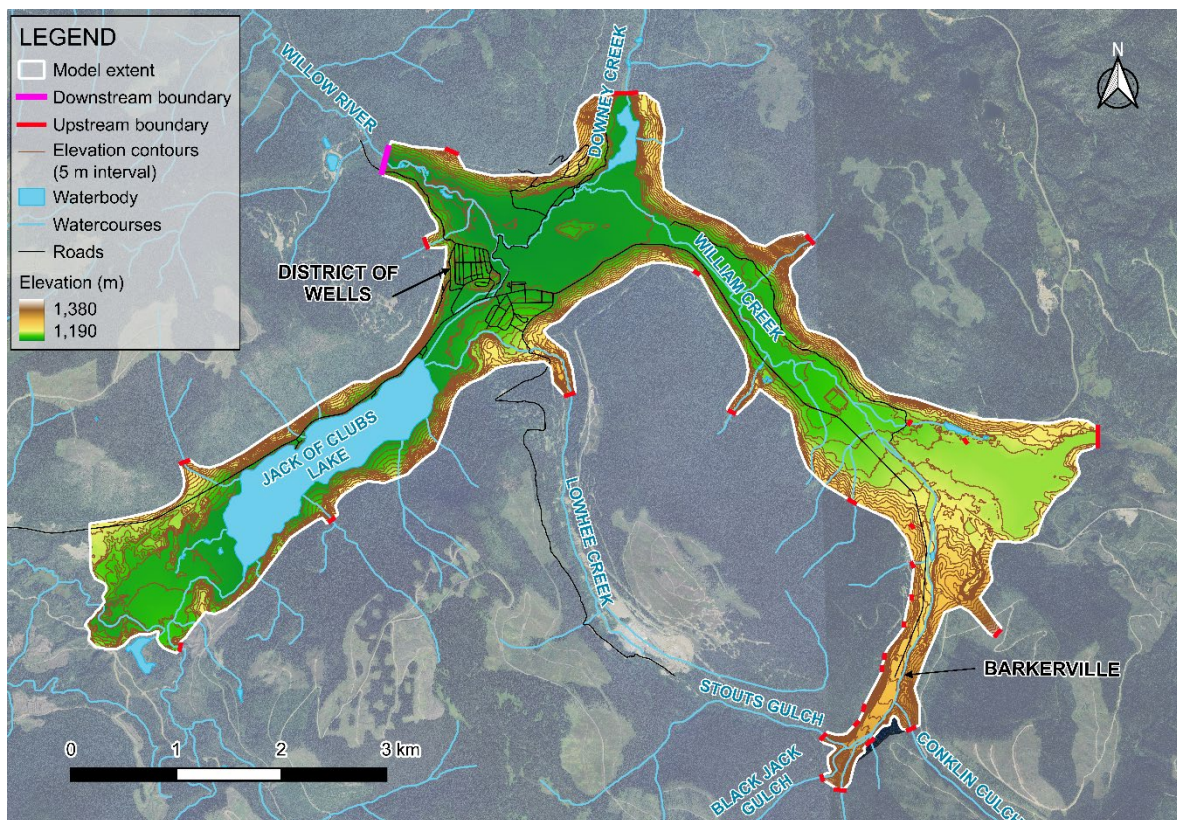


Figure 6-2 Digital elevation model and model extent for the study area. Imagery source: Google Satellite (September 2024).

6.2.5 Model Domain and Boundary Conditions

The model domain covers an approximately a 13 km² area (Figure 6-2) that includes:

- A 9 km section of Williams Creek from upstream of Barkerville (confluence of Stouts Gulch and Black Jack Gulch) to the confluence with Willow River near Wells
- Lowhee Creek from 600 m upstream of the BGM bridge to its confluence with Willow River

- Jack of Clubs Lake and the Willow River from the Jack of Clubs Lake outlet to 800 m downstream of the Wells sewage lagoons.

The downstream boundary condition (Figure 6-2) for the Willow River was set as the normal depth using an estimated friction slope of 1% (0.001 m/m), as measured from the 2021 lidar. Upstream (inflow) boundary conditions were applied as hydrographs at the downstream end of the sub-watershed (Figure 6-1). Where catchments fell outside the model domain, hydrographs were summed to define boundary conditions, including Williams Creek 1–3 and Conklin Gulch 1 and 2.

The Williams Creek 8 watershed represents the slope west of Barkerville and is not associated with a defined channel. To accurately represent runoff delivery to Barkerville, the flow from Williams Creek 8 was distributed across eight inputs within the catchment, allowing runoff to enter the model at multiple locations rather than a single point.

6.2.6 Manning's n Roughness

Hydraulic resistance to flow within the channel and floodplain is simulated in HEC-RAS using Manning's n roughness coefficients. Based on Chow (1959), Jarrett (1984), and USGS (1987), BGC selected Manning's n values between 0.03 and 0.11 to represent the different land cover type within the model domain (Table 6-5 and Figure 6-3).

Table 6-5 Selected Manning's roughness value for the different land cover type within the model domain.

Land Cover	Manning's n Value
Water surface (lakes and channels)	0.03
Infrastructure and pavement	0.018
Vegetated floodplain	0.06
Gravel floodplain	0.045
Woods	0.11

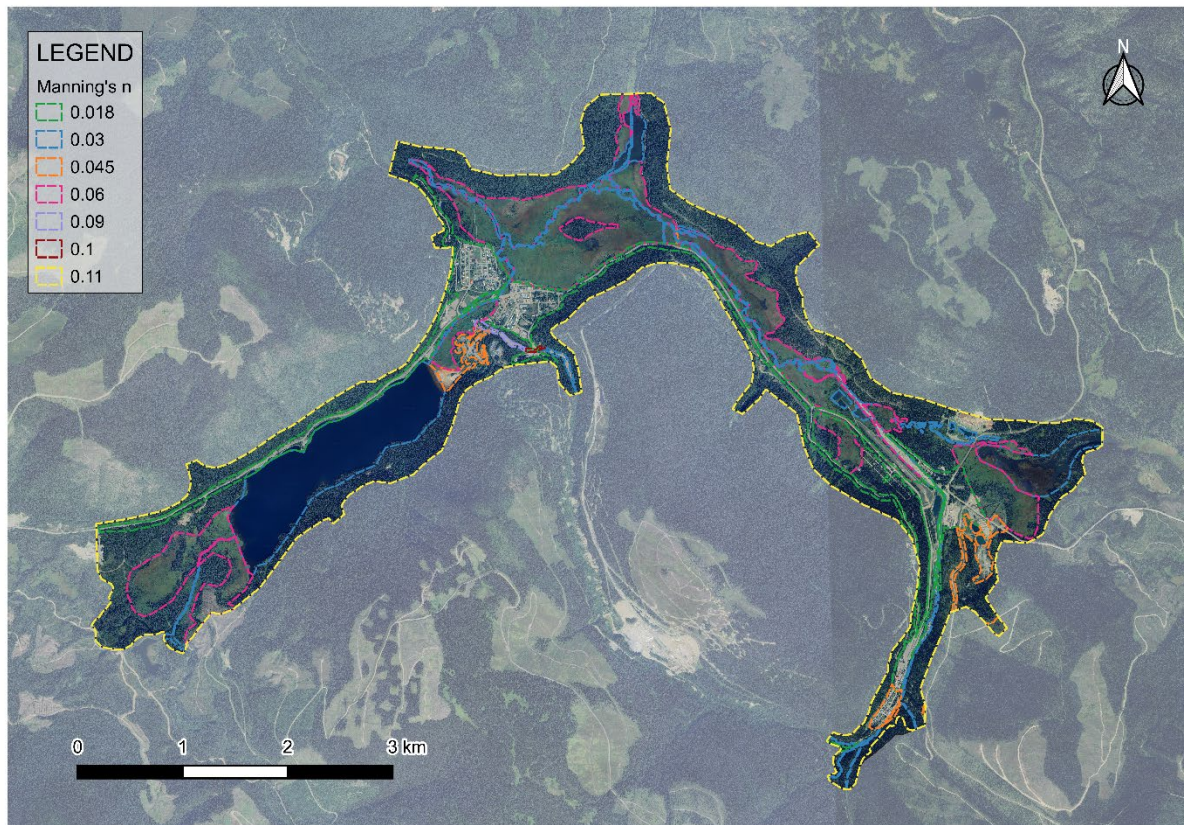


Figure 6-3 Manning's roughness layer defined for the model domain. Imagery source: Google Satellite (September 2024).

6.2.7 Computational Mesh

HEC-RAS uses a numerical mesh (or grid) to discretize the model domain into computational cells. The mesh resolution is selected to balance the desired accuracy of hydraulic results, resolution of the underlying terrain data, and computational efficiency.

Within HEC-RAS, a 2D mesh was generated based on:

- Model domain.
- Refinement areas to define sub-domains where the mesh properties (e.g., mesh resolution) are adjusted.
- Breaklines to align the mesh with terrain features which influence the flow such as channels, dikes, ditches, terraces, and embankments. HEC-RAS provides options to adjust the mesh resolution along breaklines.

From these inputs, HEC-RAS generates the mesh consisting of interconnected grid cells with computational points at the cell centroids and along the faces of the cells (i.e., along the cell sides).

To balance accuracy and model runtime, and considering the 4 m resolution of the DEM, a base mesh resolution of 10 m was applied, with a finer 5 m resolution used in areas of interest

(Barkerville, the Town of Wells, and the Barkerville Lagoons). Breaklines were iteratively placed along channel centrelines, dikes, and roads to develop a curvilinear mesh aligned with flow paths (Figure 6-4). Where required, a finer resolution (2.5 m) was applied along breaklines to allow key features to be resolved by more than one cell.

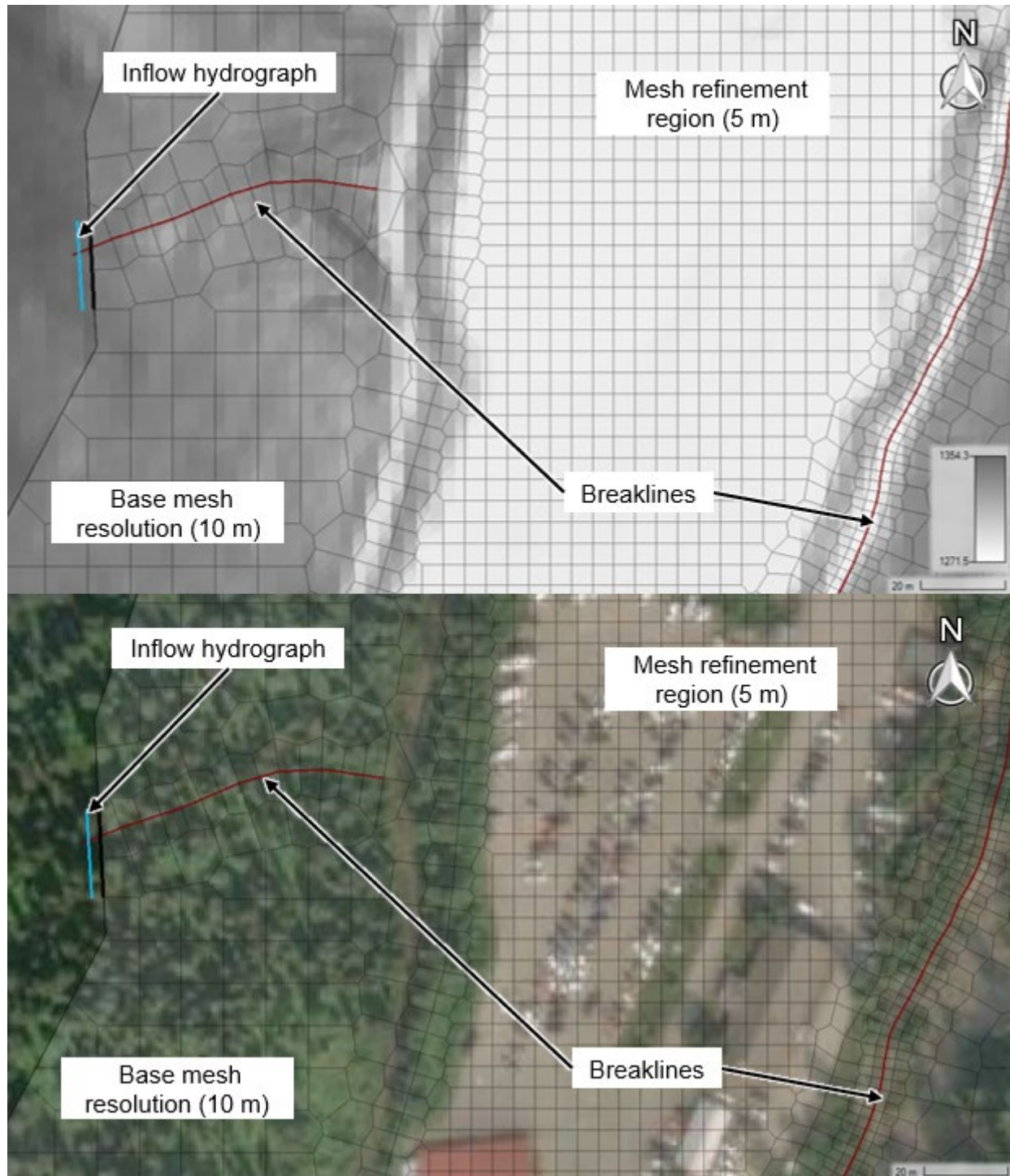


Figure 6-4 Example of mesh developed for the HEC-RAS model in the vicinity of Barkerville. ESRI satellite image of August 2024.

6.2.8 Hydraulic Structures

Bridges within the modelling domain were not explicitly represented. Instead, selected structures were removed from the terrain to allow flow to pass through the channel unobstructed. Bridge removal was implemented by interpolating average channel conditions upstream and downstream of each crossing to produce a continuous channel profile. For base level flood mapping, BGC assumed that bridge soffits are above peak flood levels and do not restrict flow, and that pier effects have a negligible influence on modelled inundation extents. The treatment of bridges within the model domain is summarised in Table 6-6.

Table 6-6 Terrain representation of bridges across the modelling domain.

Bridge	Terrain Representation	Justification
Pedestrian footbridge (Theatre Royal) across Williams Creek (Barkerville)	Retained	The bridge is not resolved in the DEM and does not introduce a constriction to channel flow.
Prosperpine Bridge across Williams Creek (Barkerville)	Removed	The bridge is resolved in the DEM and introduces a constriction to channel flow.
One-Mile Road bridge across Williams Creek	Retained	The bridge is resolved in the DEM and introduces a constriction to channel flow. The bridge soffit is close to typical water surface elevations, and the opening is often partially restricted by beaver dam activity, further limiting conveyance ¹ . Because its conveyance is often limited and because it is not in the main flood flow path, the bridge was retained in the terrain.
Highway 26 across Willow River (Wells)	Removed	The bridge is resolved in the DEM and introduces a constriction to channel flow.
BGM bridge across Lowhee Creek	Removed	The bridge is resolved in the DEM and introduces a constriction to channel flow.
Footbridge along Creekside trail across Willow River (north of Wells)	Retained	The bridge is not resolved in the DEM and does not introduce a constriction to channel flow.
Footbridge along Sugar Loaf trail across Willow River (north of Wells)	Retained	The bridge is not resolved in the DEM and does not introduce a constriction to channel flow.

Note:

1. Beaver dam activity was observed on the left abutment of the bridge during BGC's June 2025 visit.

The aerial sanitary sewer line crossing located approximately 20 m from the Willow River Bridge was not included in the model as it was not resolved in the DEM and did not introduce a constriction in the Willow River.

Culverts within the modelling domain were explicitly represented in the HEC-RAS model using available geometric and invert data. This enables direct simulation of flow conveyance at these

crossings, providing a more realistic representation of hydraulic conditions. Culverts included within the model domain are:

- Culvert along Stouts Gulch across Richfield Road (1 x 1800 mm diameter)
- Culvert along Williams Creek across Bowron Lake Road (1 x 3000 mm diameter)
- Culvert from Ballarat Pit to Williams Creek across Bowron Lake Road (1 x 600 mm diameter).

6.2.9 Initial Conditions

The base level floodplain mapping is conducted using a bare earth DEM and does not include bathymetric survey data (with the exception of Lowhee Creek and Willow River between Jack of Club Lake and Highway 26 crossing). As a result, the water surface captured in the DEM is assumed to represent typical flow conditions within the channels prior to the onset of flooding.

Jack of Clubs Lake was initialized with a starting water level of 1197.6 m, which is 1.35 m above the outlet elevation. At this level, the lake discharges approximately 4.5 m³/s to the Willow River. This initial condition reflects the expectation that Jack of Clubs Lake would already be discharging during freshet conditions prior to the onset of flood flows. It also reduces the potential for backwater effects from Lowhee Creek (Section 7.2.3) and therefore represents a more conservative scenario for evaluating flood hazards in Wells.

Initial flow conditions were applied to establish a small background flow in the stream network to avoid dry-channel conditions. The initial conditions consisted of flows set to 2% of the 200-year rain-on-snow peak flow. Only channels representing natural streams were initialized.

6.2.10 Simulation Settings

The HEC-RAS two-dimensional (2D) model was run using shallow water equations with a fixed time step of 0.5 seconds to maintain a Courant number of less than 1. The shallow water equations generally provide an accurate representation of flow dynamics, especially in areas with sharp constrictions, expansions, or changes in flow direction (e.g., meander bends, bridges, etc.).

The model was run to simulate a 24-hour period. The model was run without a turbulence model. The water surface tolerance was set to 0.003 m, and the maximum number of iterations was set to 20.

6.3 Flood Hazard Mapping

Flood hazard mapping was developed using the results from the 2D hydraulic modelling, including maximum flood extent and maximum flow depth. Modelled inundation extents were overlaid onto relevant spatial datasets (e.g., key infrastructure, building footprints) to identify assets located within the predicted floodplain. This approach allows for the spatial assessment of potential flood exposure and inundation depth at structures and infrastructure under the modelled scenarios.

7.0 RESULTS

7.1 Peak Discharge Estimates

The peak flows and catchment's hydrographs used as inflow boundary conditions are presented in Appendix A for each modelled scenario. Peak discharges for the three modelled scenarios at key locations (Drawing 11) within the model domain are summarised in Table 7-1.

Table 7-1 Peak discharges at key locations within the model.

Cross-Section	Location	Peak Flow (m ³ /s)		
		200-Year Rain-on-Snow	200-Year Rain-on-Snow + Climate Adjustment	500-Year Rain-on-Snow + Climate Adjustment
A-A'	Williams Creek at Prosperpine Bridge (Barkerville)	95	133	168
B-B'	Williams Creek at Barkerville Lagoons	91	134	167
C-C'	Willow River at Highway 26	20	26	31
E-E'	Willow River at Model Boundary	74	99	120

Note:

1. Cross-section D-D' used only for water surface elevation.

Peak precipitation occurs approximately midway through the 24-hour event. At Barkerville, peak flooding is closely aligned with this precipitation pattern, with peak discharges occurring at approximately hour 12 (Figure 7-1). Further downstream, peak discharges at the model boundary are also attenuated due to storage effects within the Williams Creek floodplain between the Barkerville lagoons and its confluence with Willow River.

In Wells, downstream of Lowhee Creek, the flood hydrograph exhibits two peaks. The first, occurring at approximately hour 13, is attributed to more rapid runoff contributions from Lowhee Creek. This is followed by a broader, higher secondary peak (approximately between 5% and 20% higher than first peak), associated with attenuated outflows from Jack of Clubs Lake.

Predicted peak flows in the Willow River at Highway 26 are substantially greater than historical observations, ranging from 1.7 to 2.6 times greater above the maximum recorded flow of 12 m³/s at the Willow River at Wells station between 2006 and 2010 (Knight Piésold, June 9, 2010).

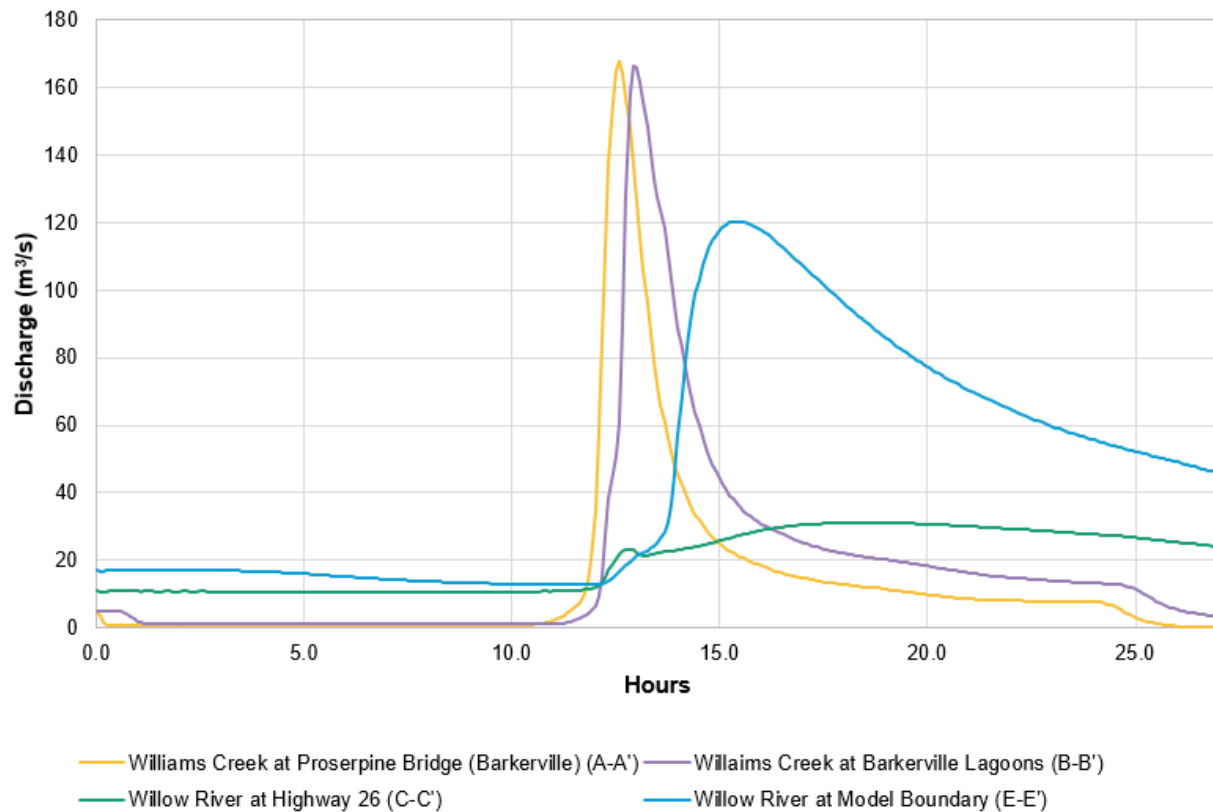


Figure 7-1 Hydrographs for the climate change adjusted 500-year rain-on-snow scenario across the study area.

7.2 Hydraulic Modelling

The maximum flow depth and flood extent for the 500-year rain-on-snow with climate adjustment scenario and flood extent for the 200-year rain-on-snow with and without climate adjustment scenarios are presented in Drawing 11. Maximum flow depths in the next sections are for the 500-year rain-on-snow with climate adjustment scenario.

7.2.1 Barkerville Town

Barkerville Town, including the parking areas, floods under all modelled scenarios, with an average flood depth between 0.1 m to 0.2 m for the modelled scenarios. The areas with the greatest flood depths are the low-lying areas at the northwest corner of the parking lot and near St. Saviour's Church and the Methodist Church with a maximum flood depth of 1.1 m (Drawing 11G and 11H). The water flooding Barkerville Town originates from the runoff from the west slope, flow from Stout Gulch and Black Jack Gulch overtopping Richfield Road, and Williams Creek overtopping its banks south of the town. Some flooding is predicted on Main Street (Highway 26), with maximum flooding depths of 0.2 to 0.3 m in some sections. Highway flooding originates from the overwhelmed roadside ditches conveying water from the west hill.

The water surface elevation across Barkerville in line with the Prosperpine Bridge (cross-section A-A') is presented in Figure 7-2. The bridge deck elevation of the Prosperpine Bridge is not presented as its elevation is not well resolved in the lidar topography.

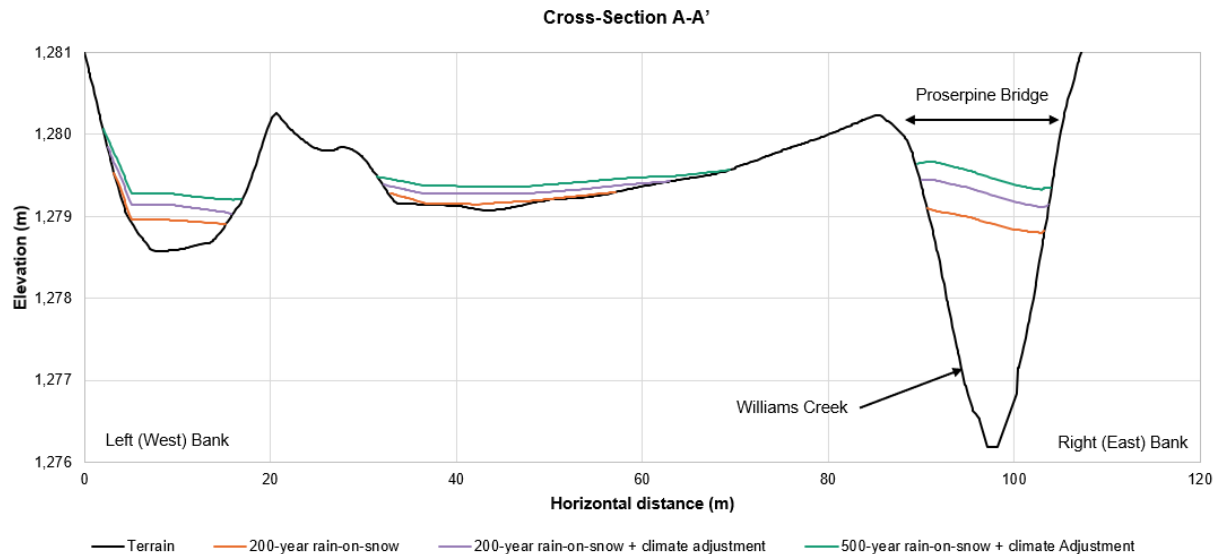


Figure 7-2 Modelled maximum (for all scenarios) water surface elevation at Barkerville Prosperpine Bridge (cross-section A-A'). View is looking downstream.

7.2.2 North Barkerville

North of Barkerville, the Bowron Lake Road is predicted to be overtopped at the crossing location with Williams Creek and the crossing with Valley Creek (Drawing 11D, 11E, 11F). The flooding is mostly contained within the floodplain, with the exception:

- The Barkerville sewage lagoon berms are overtopped and flooded.
- The northwest end of the Lowhee Campground is flooded by 0.16 m on average, with sections having a maximum depth of 0.78 m.
- Some limited sections of Highway 26 are flooded to a maximum of 0.4 m. Highway flooding originates from the overwhelmed roadside ditches conveying water from the slopes on the west side of the valley. However, these results are conservative in that there are numerous culverts along Highway 26, none of which are included in the hydraulic model.

The water surface elevation across the Barkerville lagoons and Highway 26 (cross-section B-B') is shown in Figure 7-3. Water surface elevations are higher on the right bank (looking downstream), as Williams Creek is elevated relative to the adjacent floodplain on the left bank. The sewage lagoons are therefore partially protected by the lagoon dike. Flooding of the lagoons is predicted to occur where flows overtop the lagoon dike along the southeast and eastern perimeter and from flow coming through the floodplain southeast of the lagoons. Highway 26 floods in this section from flow accumulating at the intersection of Highway 26 and Reduction Road (but again none of the culverts along Highway 26 are included in the model).

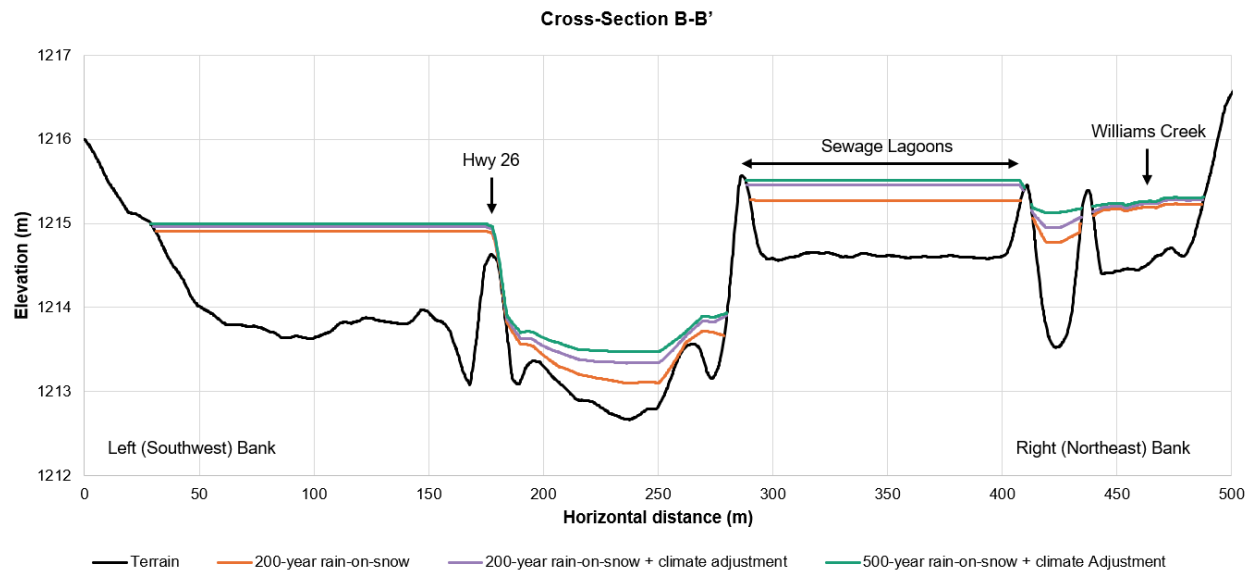


Figure 7-3 Modelled maximum (for all scenarios) water surface elevation at Barkerville Lagoons (cross-section B-B'). View is looking downstream.

7.2.3 Wells

Flooding in Wells is maintained within the Willow River floodplain and is not predicted to flood the town (Drawing 11B and 11C). At the Highway 26 crossing with the Willow River, the maximum flow surface is 1197.4 m, approximately 2.0 m below the bridge deck and highway surface (Figure 7-4). However, the bridge piers and soffit are not modelled directly, and these may result in a different water surface elevation.

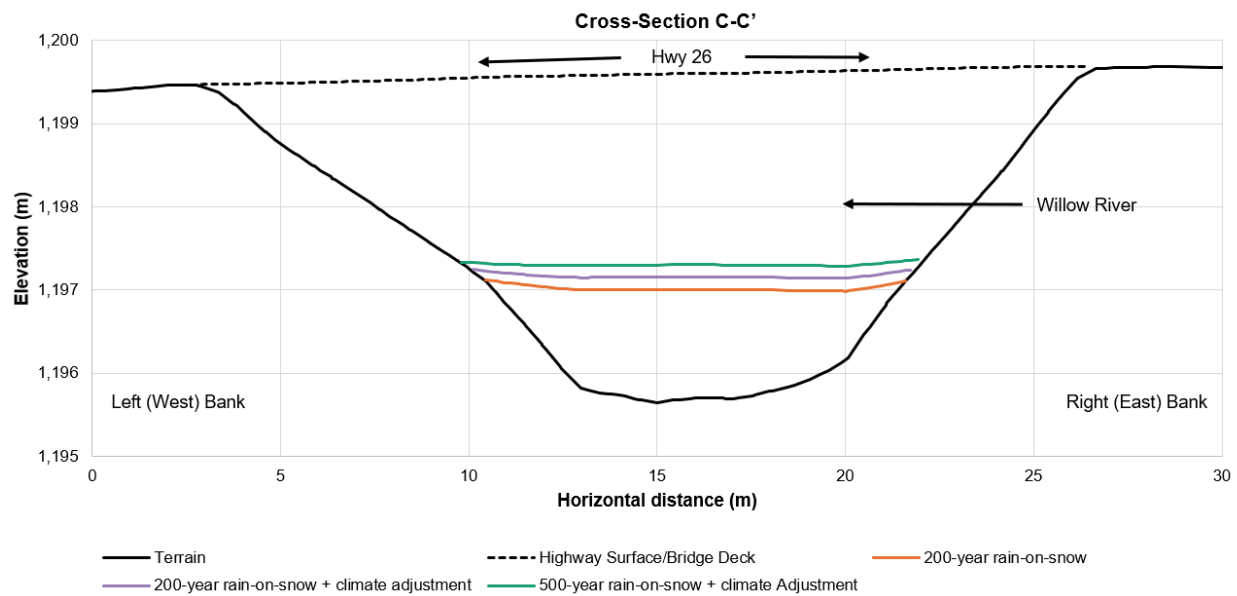


Figure 7-4 Modelled maximum (for all scenarios) water surface elevation at Willow River at Highway 26 (cross-section C-C'). View is looking downstream.

Lowhee Creek generally flows northeast at its confluence with the Willow River, consistent with the downstream drainage direction of the Willow River. The model indicates that during peak flow events, water levels in Lowhee Creek and its alluvial fan can temporarily exceed those in the Willow River, resulting in a backwater condition that diverts a portion of the flow southwest towards Jack of Clubs Lake (Figure 7-5). This backwater effects temporally diverts approximately 50% of the flow from Lowhee Creek to Jack of Clubs Lake. This flow bifurcation provides attenuation of peak flows in the Willow River near Wells.

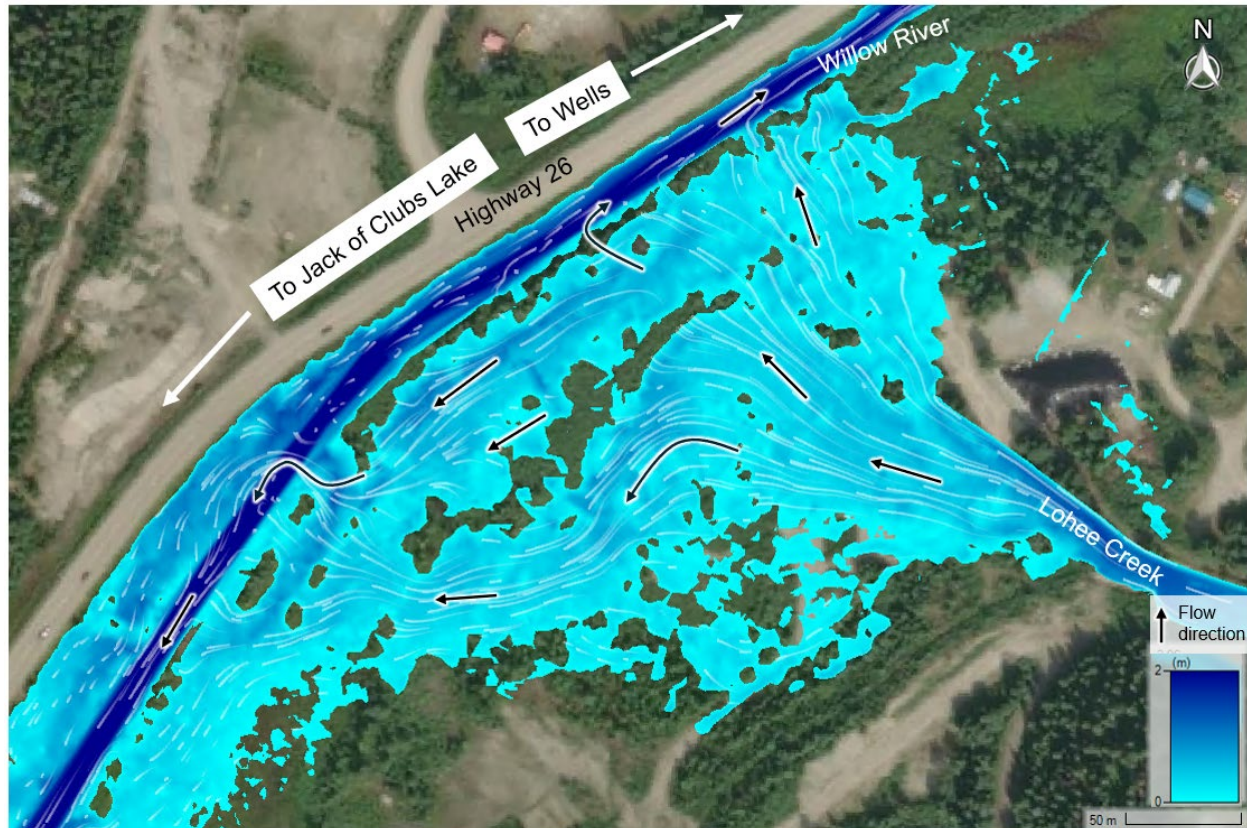


Figure 7-5 Lowhee Creek flow direction at its confluence with the Willow River (maximum flow depth from the 500-year rain-on-snow with climate adjustment scenario).

Figure 7-6 shows the water surface elevation at the north end of Wells. The lift station (#1) is 2.5 m above the maximum surface elevation, while the edge of Blair Avenue is approximately 1.5 m above. The flood is not contained within the banks of the Willow River and flows within its broad floodplain.

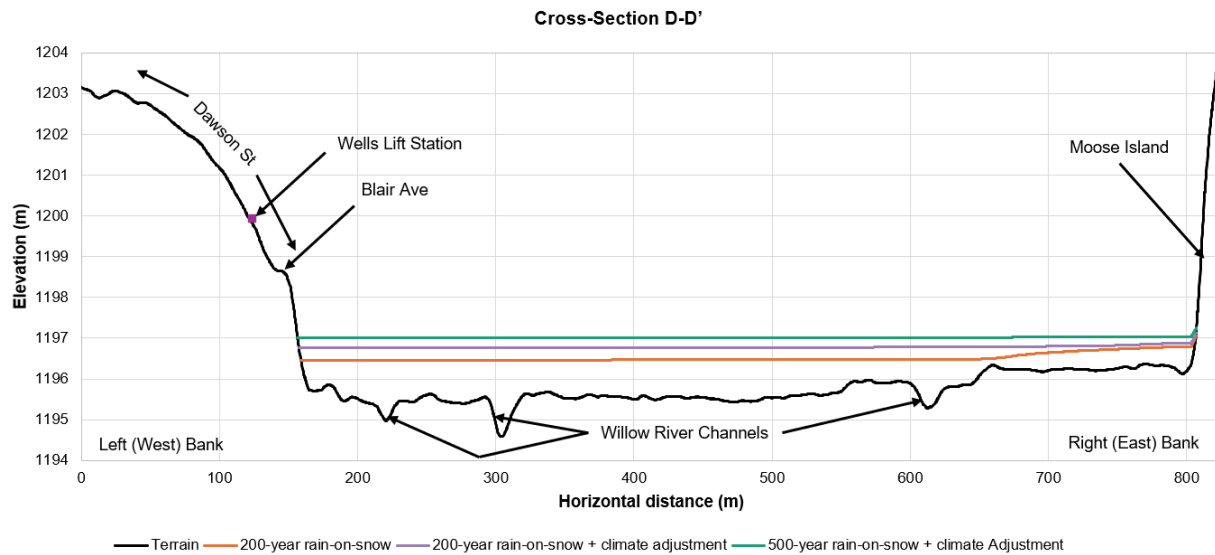


Figure 7-6 Modelled maximum (for all scenarios) water surface elevation at Wells lift station (cross-section D-D'). View is looking downstream.

7.3 Flood Hazard

The RFHA assessment identified several areas within the DoW where flooding presents an ongoing hazard to community infrastructure and land use. Flood hazards are controlled not only by flood magnitude, but also by local geomorphic conditions, historical channel modifications, sediment transport processes, and the presence or absence of flood mitigation infrastructure. Consequently, the areas of greater hazard generally correspond to locations where floodplain development coincides with active sedimentation, unstable channels, or historical flood pathways. The hydraulic modelling results provide an indication of potential inundation extents during extreme flood events; however, the assessment of flood hazard also considers historical flooding, existing conditions, and observed geomorphic processes throughout the watershed.

The flood hazard and affected infrastructure to Barkerville Town, North Barkerville and Wells are listed in the sections below and summarized in Table 7-2. Highway 26 access disruptions to the DoW are also discussed below.

7.3.1 Barkerville Town

The historic townsite of Barkerville occupies the valley floor of Williams Creek downstream of the confluences with Stouts Gulch and Conklin Gulch. Historical records indicate that flooding was common during the early period of placer mining, when large volumes of mining-derived sediment caused rapid aggradation within Williams Creek and frequent overbank flooding. Construction of the flood bulkhead and subsequent filling along the west side of the valley provided protection from these historical hazards and continues to influence flood pathways today.

Geomorphic evidence suggests that the sediment wave associated with placer mining has largely passed through the Barkerville reach and that channel narrowing and incision have

reduced the historical aggradation hazard. However, the townsite remains located within an active floodplain and is susceptible to flooding during extreme runoff events. Flood hazards are associated with:

- Williams Creek overtopping its banks
- Runoff from the western hillslope exceeding the capacity of the existing drainage infrastructure
- Overflow from tributaries such as Stouts Gulch and Black Jack Gulch overwhelming culvert crossings and overtopping Richfield Road.

Flood-related road access disruptions represent an additional hazard within Barkerville. The base level flood modelling indicates:

- Highway 26 north of the parking area may experience flooding depths of up to 0.3 m. Two sections are predicted to be inundated across the full roadway width, including a section approximately 100 m long and a second section less than 10 m long (Figure 7-7). Highway 26 access disruptions from the flood hazard are further discussed in Section 7.3.4.
- Prosperpine Bridge, a potential evacuation route, may be affected by flooding. However, bridge deck and soffit elevations have not been surveyed, and additional assessment is required to determine the potential for overtopping. Williams Creek is not predicted to overtop its banks at the bridge location.
- The Theatre Royal footbridge may be vulnerable to overtopping during extreme flood events. Although bridge elevations are unknown, overtopping of the Williams Creek banks immediately upstream and downstream of the crossing indicates that floodwaters may reach the structure.

7.3.2 North Barkerville

Flood hazards downstream of Bowron Lake Road differ substantially from those within the Barkerville townsite. As Williams Creek exits confinement and transitions onto a broad alluvial fan, channel gradients decrease and sediment deposition becomes the dominant geomorphic process. Field observations, historical aerial imagery, pebble count data, and sediment transport analyses indicate that the reach remains actively aggrading and continues to accumulate sediment derived from historical placer mining. As a result, flood hazards in this reach are influenced not only by flood inundation, but also by ongoing channel evolution and sediment accumulation, which may reduce channel conveyance and the effective freeboard of existing flood protection works over time.

The combination of active sediment deposition, a braided channel morphology, and a broad floodplain creates flood hazards for infrastructure within North Barkerville. Base level flood modelling indicates:

- Most of the Barkerville airstrip is not predicted to flood. However, access to the airstrip may be compromised, as portions of the access road from Highway 26 are predicted to be inundated.

- The Barkerville wastewater lagoons are predicted to flood, including overtopping of sections of the lagoon berms. Access to the lagoons may also be limited due to flooding of its access road.
- The Lowhee Campground is predicted to experience localized flooding, primarily within its northwestern portion. Average flood depths are approximately 0.2 m, with localized maximum depths of up to 0.8 m.

Flood-related transportation disruptions represent an additional hazard within North Barkerville. Base level flood modelling indicates:

- Bowron Lake Road is predicted to be overtopped at both the Williams Creek and Valley Creek crossings. The Williams Creek culvert is predicted to be overwhelmed, resulting in roadway flooding with depths of up to 0.7 m.
- Limited sections of Highway 26 are predicted to experience flooding, with maximum depths of up to 0.4 m. Two sections are predicted to be inundated across the full roadway width, including an approximately 100 m reach adjacent to McArthur Gulch and a 70 m reach adjacent to the wastewater lagoons (Figure 7-7). Highway 26 access disruptions from the flood hazard are further discussed in Section 7.3.4.
- Matthew Valley Road near the Ballarat Camp is predicted to have some sections flooded, with flood depths of up to 0.3 m potentially preventing access.
- One Mile Road is predicted to be overtopped north of the bridge crossing, with flood depths of up to 1.2 m potentially preventing access.

As mentioned above, flood hazards in North Barkerville are strongly influenced by ongoing sediment deposition and channel aggradation. Continued monitoring of channel conditions and flood protection infrastructure is therefore important for long-term hazard management.

7.3.3 Wells

Flood hazards within Wells are strongly influenced by the geomorphic evolution of the Lowhee Creek alluvial fan and the legacy effects of historic placer mining. The Lowhee Creek watershed has experienced extensive hydraulic mining and downstream sediment delivery, resulting in long-term fan growth, channel instability, and ongoing sediment deposition. Historical and geomorphic evidence indicates that the present-day flood hazard in South Wells is fundamentally linked to sediment management rather than flood discharge alone.

The highest hazard area within Wells is associated with the Lowhee Creek fan and adjacent portions of South Wells. The District has recently constructed the Phase 1 and Phase 2a dikes, with further works planned as part of the Phase 2b extension. These works substantially reduce the immediate flood hazard to developed areas; however, long-term flood risk remains dependent on effective management of sediment supplied from the upstream watershed. Geomorphic assessment indicates that sediment stored behind historical mining dams continues to erode and deliver material to the fan and is expected to do so for decades. Without ongoing sediment management, channel aggradation would reduce available freeboard along the dike system and increase future flood hazard.

In contrast, the Willow River corridor through Wells exhibits a relatively low hazard to flooding. The floodwater within the Willow River remains largely contained within its channel and floodplain, and the hydraulic modelling indicates that the principal community infrastructure and developed portions of the town are above predicted flood levels. Flooding is primarily limited to undeveloped floodplain areas, trails, and pedestrian crossings adjacent to the river. The pedestrian bridges servicing the Creekside trail system and Sugar Loaf trail are within the flooded floodplain and are predicted to be flooded and compromised.

Table 7-2 provides a summary of flood hazards within the DoW and Barkerville.

Table 7-2 Summary of flood hazard.

Area	Infrastructure	Summary of Hazard	Reference
Barkerville Town	Historic Townsite	Average flood depths of 0.1 to 0.2 m across portions of the townsite. Maximum flood depths (1.1 m) in the parking area, St. Saviour's Church and Methodist Church.	Drawing 11H Drawing 11G
	Operations Building	Limited flooding in the vicinity of the Operations Building (maximum depth ~0.03 m).	Drawing 11H Drawing 11G
	Highway 26	Sections of roadway inundation of 0.2–0.4 m. Flooding may compromise access.	Drawing 11H Drawing 11G
	Prosperpine Bridge	Potential overtopping of bridge deck.	Drawing 11H Drawing 11G
	Footbridge	May be overtopped based on predicted overbank flooding upstream and downstream.	Drawing 11H Drawing 11G
	Richfield Road Crossing	Stouts Gulch culvert overwhelmed, resulting in roadway overtopping.	Drawing 11H
North Barkerville	Barkerville Airport (airstrip)	Most of the airstrip remains unflooded; approximately 100 m at the northwest end may experience flood depths up to 0.4 m. The airport access road may experience flooding up to 0.2 m deep.	Drawing 11F Drawing 11E
	Highway 26	Localized flooding up to 0.3 m deep. Approximately 100 m near McArthur Gulch and 70 m near the sewage lagoons are inundated across the full width of the roadway. Flooding may compromise access.	Drawing 11F Drawing 11E
	Barkerville Wastewater Lagoons	Sewage lagoons are predicted to be flooded during the assessed design events. Access to the lagoons may also be limited due to flooding of its access road.	Drawing 11F Drawing 11E
	Lowhee Campground	Northwest portion is affected by flooding, with average depths of approximately 0.2 m and localized maximum depths up to 0.7 m.	Drawing 11F Drawing 11E
	Bowron Lake Road	Roadway overtopping is predicted at the Williams Creek and Valley Creek crossings, possible access compromised.	Drawing 11F
	Matthew Valley Road	Limited section west of Ballarat Camp predicted to be flooded with maximum depths up to 0.3 m.	
	One Mile Road	Predicted to be flooded north of bridge with its access compromised.	Drawing 11D
Wells	Pedestrian footbridge (Creekside trail and Sugar Loaf trail)	Trails and footbridges are predicted to be flooded and access compromised.	Drawing 11B

7.3.4 Transportation and Access Disruptions

Transportation disruptions represent an important flood hazard. Under the most severe scenario assessed (500-year rain-on-snow event with climate adjustment), Highway 26 remains passable through most of the study area; however, several localized sections are predicted to experience inundation (Figure 7-7). Four sections of Highway 26 are conservatively predicted to be flooded across the full roadway width, including:

- An approximately 40 m long section between One Mile Road and Reduction Road
- An approximately 60 m long section between Reduction Road and the airstrip access road
- An approximately 8 m long section south of Bowron Lake Road
- An approximately 90 m long section south of Bowron Lake Road.

While these flooded sections are relatively localized, they may impede vehicle travel and emergency response during extreme flood events (keeping in mind that these results are conservative in that no Highway 26 culverts were modelled). Of particular concern is access to the Barkerville airstrip, which, although not operated as an active airport, may provide an important emergency access location during a disaster or evacuation event. Flood-related disruptions along Highway 26 may also affect evacuation and emergency access between Barkerville and Wells.

Additional transportation hazards are associated with local roads that provide access to Highway 26. Base level flood mapping indicates:

- Bowron Lake Road is susceptible to flooding at both the Williams Creek and Valley Creek crossings, potentially restricting access to Highway 26 from properties and infrastructure north of Highway 26 and Forest Road Campground.
- One Mile Road is predicted to be overtopped north of the bridge crossing, with flood depths of up to 1.2 m potentially preventing access.
- Flooding on Matthew Valley Road may potentially restricting access to Highway 26 from properties and infrastructure west of Highway 26.

Overall, the primary transportation hazard within the DoW is the potential isolation of areas located north and west of Highway 26 through a combination of localized flooding on Highway 26 and flooding of key road crossings. These access constraints may affect emergency response, evacuation routes, and access to critical infrastructure during extreme runoff events.

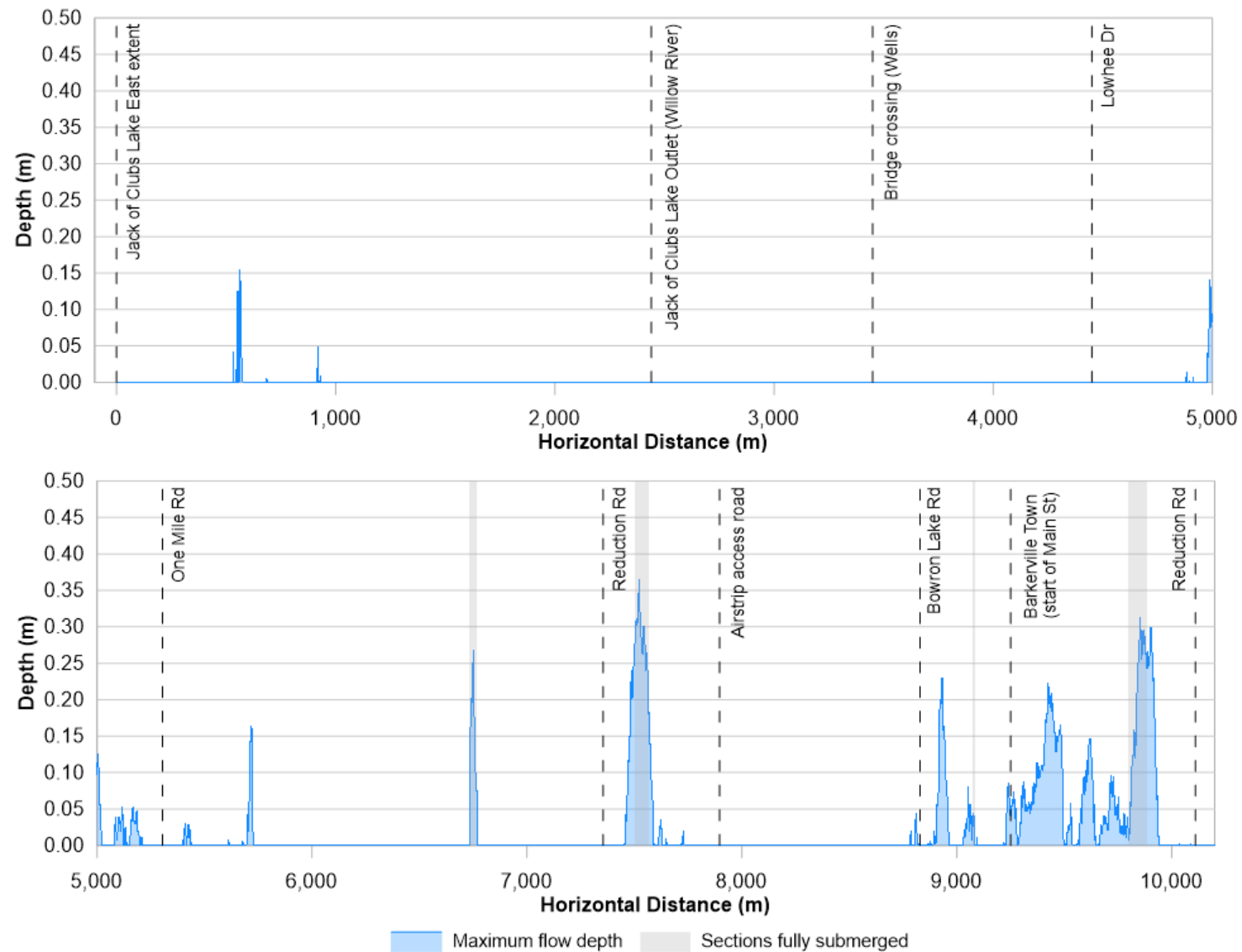


Figure 7-7 Maximum flow depth of the 500-year rain-on-snow with climate adjustment along Highway 26 centreline.

8.0 SENSITIVITY ANALYSIS

Historical imagery or documented water surface elevations during flood events are not available for calibration or verification of the model results. Therefore, BGC performed a sensitivity of water surface elevations to variations on the following parameters to assess the limitations of the uncalibrated model:

- A 20% decrease and a 20% increase in roughness (Manning's n value)
- 50% reduction in the friction slope at the downstream boundary condition.

All sensitivity scenarios were performed for the 500-year rain-on-snow with climate change adjustment scenario.

The sensitivity analysis indicated that flood extent is insensitive to variations in surface roughness and the downstream boundary condition friction slope, with changes resulting in less than a 2% difference in flood extent (Table 8-1).

Table 8-1 Change in flood extent relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope. A positive value indicates that the flood extent resulting from the sensitivity run is higher than that for the base case.

Scenario	Flood Extent (km ²)	Difference (%)
Base case	577	N/A
20% decrease in roughness ¹	569	-1.4%
20% increase in roughness ¹	585	1.4%
50% downstream friction slope ²	578	0.1%

Notes:

1. Adjustments to the model roughness was done by adjusting the Manning's n value.
2. The downstream boundary condition friction slope was reduced from 1% (0.001 m/m) to 0.5% (0.0005 m/m).

Figure 8-1 to Figure 8-4 presents the changes in flow hydraulics along the centreline of Williams Creek and Willow River and are further summarized in Table 8-2 and Table 8-3.

The sensitivity analysis results indicated that the flow hydraulics along the centreline of Williams Creek and Willow River were relatively insensitive to changes in the downstream boundary condition. Water surface elevation increases (WSE) between 0.1 m and 0.3 m were predicted within 0.5 km upstream of the model boundary, downstream of the Town of Wells. Further upstream, an increase of approximately 0.1 m was predicted in the Willow River up to the Highway 26 crossing. Changes in flow velocity were negligible throughout the model domain. Outside of the localized WSE increases, changes in modelled hydraulics were negligible.

Uncertainty in the roughness coefficients had a minor impact on the flow hydraulics:

- Both a 20% reduction and increase in the roughness results in a 4% change or less in the average flow depth in the model domain, and a 13% difference or less in the mean velocity
- The change in roughness had similar impact in Williams Creek and the Willow River:

- Change in roughness resulted in an average change of 4% in the WSE; 0.07 m in Williams Creek and 0.05 m in the Willow River (Table 8-2)
- Change in roughness resulted in an average change of 11% in velocity; 0.26 m/s in Williams Creek and 0.13 m/s in the Willow River (Table 8-3)

The largest differences in WSE and velocities are observed in Williams Creek upstream of the Barkerville wastewater lagoons where the creek is channelized and has a higher slope.

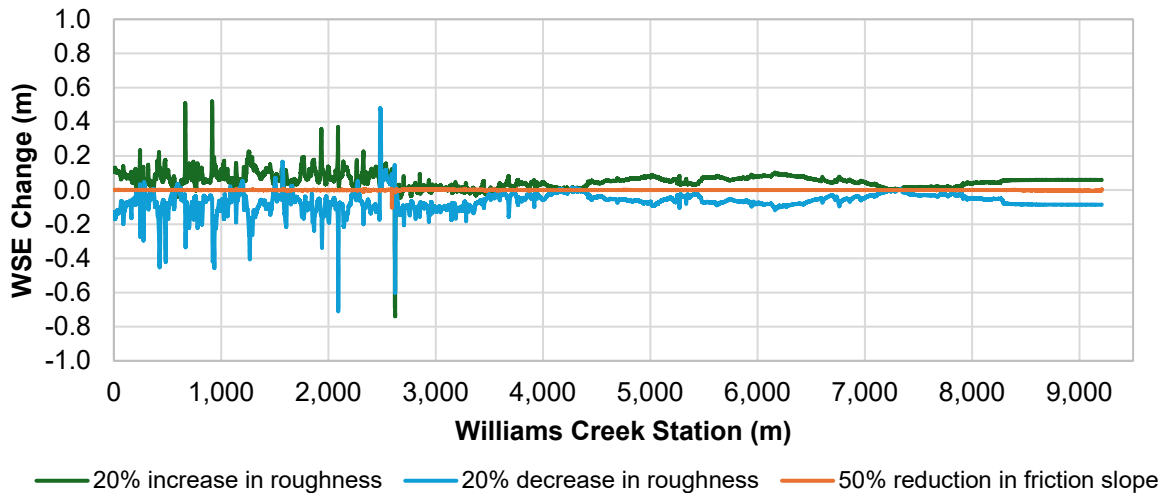


Figure 8-1 Change in WSE along Williams Creek centreline relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope. A positive value indicates that the WSE resulting from the sensitivity run is higher than that for the base case.

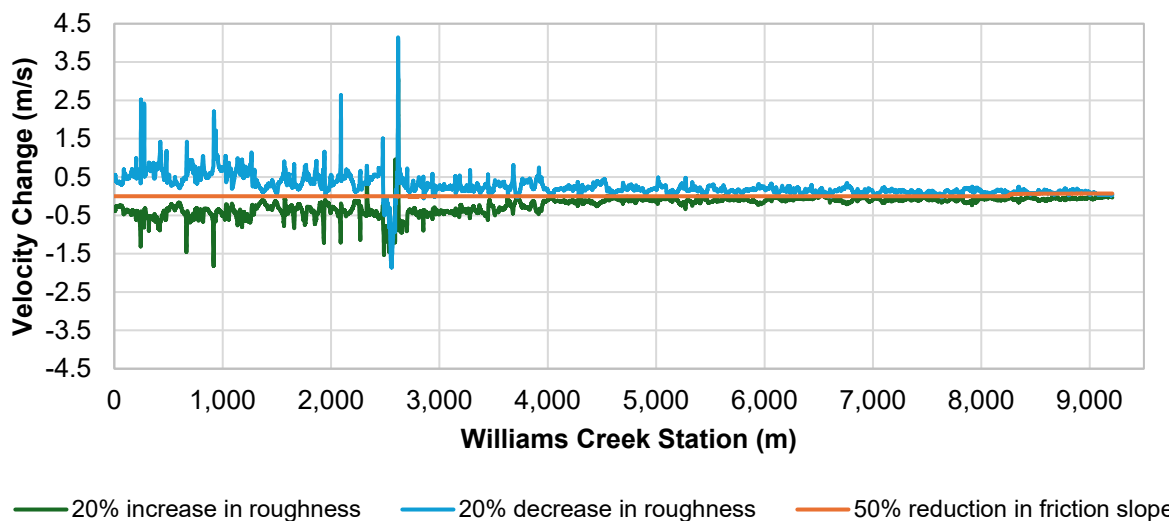


Figure 8-2 Change in velocity along Williams Creek centreline relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope. A positive value indicates that the WSE resulting from the sensitivity run is higher than that for the base case.

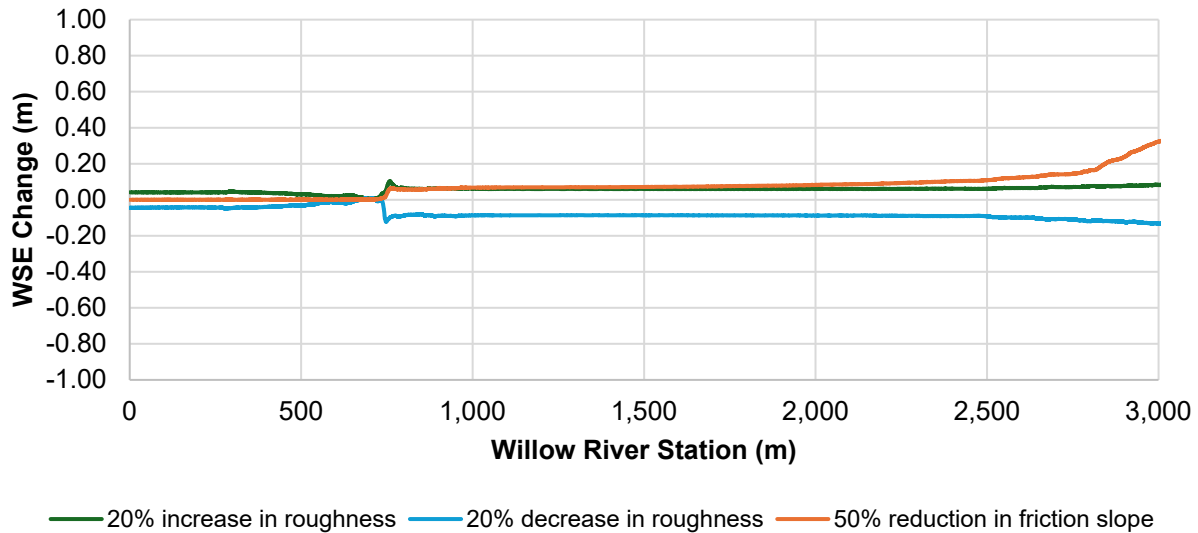


Figure 8-3 Change in WSE along Willow River centreline (starting at the outlet of Jack of Clubs Lake) relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope. A positive value indicates that the WSE resulting from the sensitivity run is higher than that for the base case.

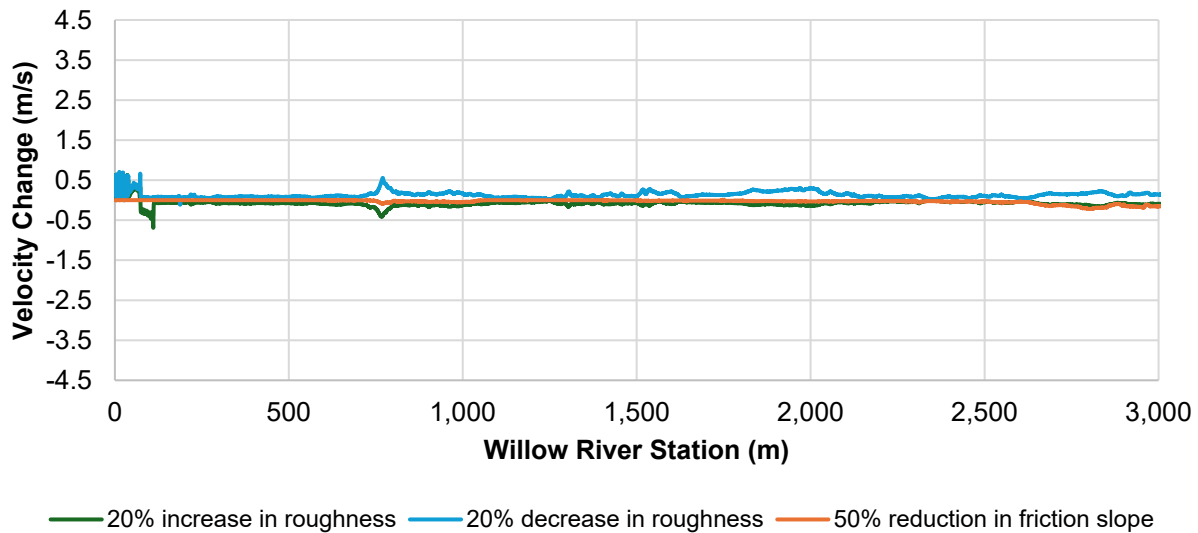


Figure 8-4 Change in velocity along Willow River centreline (starting at the outlet of Jack of Clubs Lake) relative to the base case scenario caused by varying Manning's n roughness and the downstream friction slope. A positive value indicates that the WSE resulting from the sensitivity run is higher than that for the base case.

Table 8-2 Summary of the sensitivity analysis results for the water surface elevation (WSE) changes along Williams Creek and Willow River.

Sensitivity Applied	Williams Creek		Willow River	
	Average	Std. Dev.	Average	Std. Dev.
20% decrease in roughness ¹	-0.08	0.07	-0.06	0.03
20% increase in roughness ¹	0.06	0.05	0.05	0.01
50% downstream friction slope ²	0.03	0.00	0.05	0.06

Notes:

1. Adjustments to the model roughness was done by adjusting the Manning's n value.
2. The downstream boundary condition friction slope was reduced from 1% (0.001 m/m) to 0.5% (0.0005 m/m).

Table 8-3 Summary of the sensitivity analysis results for the water surface elevation (WSE) changes along Williams Creek and Willow River.

Sensitivity Applied	Williams Creek		Willow River	
	Average	Std. Dev.	Average	Std. Dev.
20% decrease in roughness ¹	0.28	0.26	0.15	0.10
20% increase in roughness ¹	-0.24	0.23	-0.11	0.06
50% downstream friction slope ²	-0.01	0.00	0.00	0.00

Notes:

1. Adjustments to the model roughness was done by adjusting the Manning's n value.
2. The downstream boundary condition friction slope was reduced from 1% (0.001 m/m) to 0.5% (0.0005 m/m).

During large flows, a portion of Jack of Clubs Creek may discharge into Slough Creek, which flows in a northwest direction (i.e., away from Jack of Clubs Lake) (Drawing 02, Figure 8-5). An additional sensitivity analysis was conducted to evaluate this possibility. In the base-case model, a hard boundary was placed at this location, representing a conservative assumption for flood impacts to Wells by directing all flow towards the Willow River. To assess the influence of a potential secondary outlet, a downstream boundary condition was added at the Slough Creek outlet and modelled using a normal-depth boundary condition with an estimated friction slope of 2%.

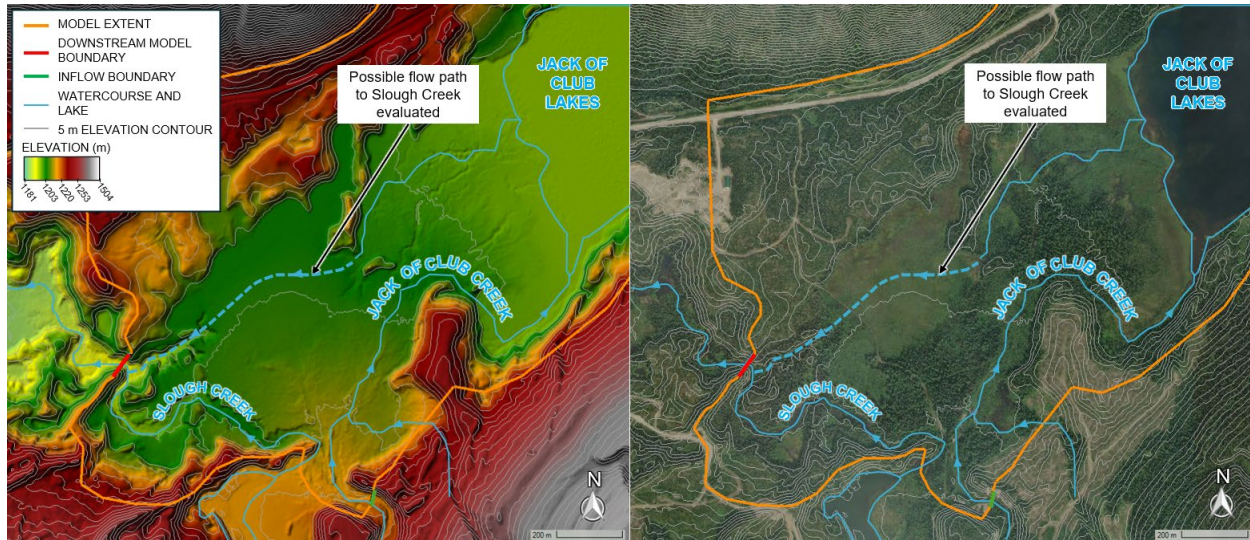


Figure 8-5 Model downstream boundary for evaluation of possible flow path to Slough Creek from Jack of Clubs Lake.

Allowing flow to discharge to Slough Creek had minimal impact on downstream inundation results in Wells. While a portion of the flow from Jack of Clubs Creek is diverted through the Slough Creek channel and exits the model domain, most flow remains within the system. This reduced the discharge at the Highway 26 bridge crossing in Wells (cross-section C–C') by 13%, resulting in a WSE decrease of 0.08 m for the 500-year rain-on-snow with climate change scenario.

9.0 LIMITATIONS, UNCERTAINTIES AND RECOMMENDATIONS

This report provides base level flood hazard maps for Williams Creek and portions of Willow River and detailed flood hazard maps for Lowhee Creek and Willow River adjacent to Wells, BC. The deliverables of this study include flood maps to support decisions that prevent or reduce injury or loss of life, environmental damage, and economic loss due to clear-water floods. The assessment includes floods resulting from elevated discharge on creeks and rivers due to rainfall and snowmelt runoff. These hazard maps can provide the basis for future work such as quantitative risk assessment and risk control (mitigation), in context of the following limitations.

9.1 Source of Uncertainty

The flood hazard mapping presented herein is subject to the following limitations and sources of uncertainty:

- **Topographic data resolution and accuracy:**
 - The hydraulic model is based primarily on 4 m resolution lidar data. While suitable for regional-scale analysis, this resolution may not capture small-scale terrain features, channel irregularities, or localized controls on flow (e.g., minor embankments, small structures), which could influence flood extents and depths. In particular, flood depths and extents along Highway 26 are conservative, as the terrain and computational mesh resolution does not include culverts along Highway 26 and likely does not fully represent roadside drainage ditches that could convey shallow overland flow.
 - The lidar-derived terrain represents the water surface at the time of acquisition. This was assumed to be equivalent to average flow conditions. Flood extents and water surface may vary based on changes in this initial flow condition. As a result, channel depths, conveyance capacity, and flow velocities may be inaccurately represented, which could affect the predicted water surface elevations, flood depths, and flood extents.
- **Hydrologic inputs and boundary conditions:**
 - Peak flows and hydrographs are derived from hydrologic modelling and available regional data and are subject to uncertainty in precipitation inputs, watershed parameters, and climate change adjustments. These uncertainties propagate into the hydraulic model results. The peak flows and hydrographs presented in this report are considered to be conservative estimates.
- **Climate change adjustments:**
 - Climate change impacts were incorporated using generalized adjustment factors derived from regional projections. These adjustments are inherently uncertain and may not fully capture future changes in storm intensity, snowmelt dynamics, or rain-on-snow processes.
- **Hydraulic structure representation:**
 - Bridges were not explicitly modelled and were either removed from the terrain or assumed not to influence flow where not resolved in the DEM. This simplification

neglects potential backwater effects, debris accumulation, or blockage during flood events, which may result in underestimation of flood levels locally.

- **Model roughness inputs:**

- Manning's n roughness values are theoretical and inherently uncertain. The roughness values affect the model results (water surface elevation, flood extent), and therefore this uncertainty is transferred to the model results. However, the sensitivity analysis on these parameters shows that varying the roughness by 20% results in less than 2% variation in the flood extent and approximately 3% in the mean flow depth in the model domain.

- **Sediment transport and channel evolution:**

- The hydraulic modelling does not explicitly account for sediment transport, channel aggradation/degradation, or debris loading, despite evidence of ongoing sediment dynamics in the study area. Changes in channel geometry over time may alter flood hazards.

- **Model calibration and validation:**

- Historical flood records, water surface elevations, or high-flow observations were not available for model calibration or validation. As a result, model outputs have not been verified against observed conditions, introducing uncertainty in predicted flood extents and depths.

9.2 Limits of Applicability

The flood hazard assessment consists of base level mapping for Williams Creek and Willow River (downstream of Highway 26 crossing), which relies on available digital elevation data and does not incorporate bathymetric surveys. Base level mapping provides a screening-level representation of flood hazards and is less accurate than detailed flood studies. Lowhee Creek and Willow River between the outlet of Jack of Clubs Lake and Highway 26 was performed using detailed level mapping.

As with all hazard assessments, the flood hazard maps prepared represents a snapshot in time. Substantial change to the conditions assumed in this flood hazard assessment can alter the flood inundation extent and associated hazards. As such, future changes, including the following may warrant re-assessment and/or updating of flood levels:

- Watershed disturbances (e.g., land use change, forestry, insect infestations, and wildfires)
- Future development in the floodplain
- Substantial flood events (i.e., a greater than 50-year flood)
- Sediment accumulation, aggradation, channel degradation, and other geomorphic changes that alter channel geometry, floodplain elevations, or flow conveyance capacity
- Bridge or culvert re-design
- Dike construction and/or construction of other flood hazard mitigation measures.

In addition, the assumption made on the impacts of climate change should be revised as scientific understanding evolves.

9.3 Recommendations for Hazard Management

Buildings and infrastructure exist within the extent of flood hazard scenarios provided in this report. Recommendations for the DoW to consider are:

- Maintain culvert conveyance capacity, particularly upstream of Barkerville, by implementing routine inspection and debris removal programs, particularly at crossings susceptible to blockage during high-flow events
- Evaluate flood mitigation options for Barkerville, including raising existing bulkheads/berms along Williams Creek and constructing a diversion structure downstream of the Stouts Gulch and Black Jack Gulch confluences to reduce potential flood impacts within the historical townsite
- Survey bridge deck and soffit elevations for key transportation and evacuation routes (e.g., Proserpine Bridge) to assess potential overtopping risks and confirm route accessibility during flood events
- Survey culverts and roadside drainage features along Highway 26 and incorporate these data into an updated hydraulic model to better assess roadway flooding hazards.
- Assess the vulnerability of critical transportation routes, including Highway 26, Bowron Lake Road, and evacuation routes, and develop contingency plans for flood-related access disruptions
- Monitor channel aggradation and sediment accumulation within Lowhee Creek and downstream of Bowron Lake Road on Williams Creek through periodic channel surveys and aerial imagery reviews to identify changes in channel capacity, flood conveyance, and the performance of existing flood protection infrastructure.

10.0 CLOSURE

We trust the above satisfies your requirements. Hamish Weatherly is the responsible geoscientist for geomorphic aspects of this report, while Isabelle Cheff is the responsible engineer for hydrotechnical aspects. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC Engineering Inc.
per:

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Principal Hydrologist

Isabelle Cheff, M.A.Sc., P.Eng.
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Reviewed by:

Paolo Chiamarello, P.Eng.
Senior Hydrotechnical Engineer

[Select a location for the Permit to Practice/Certificate of Authorization no. – see note below]

PC/th

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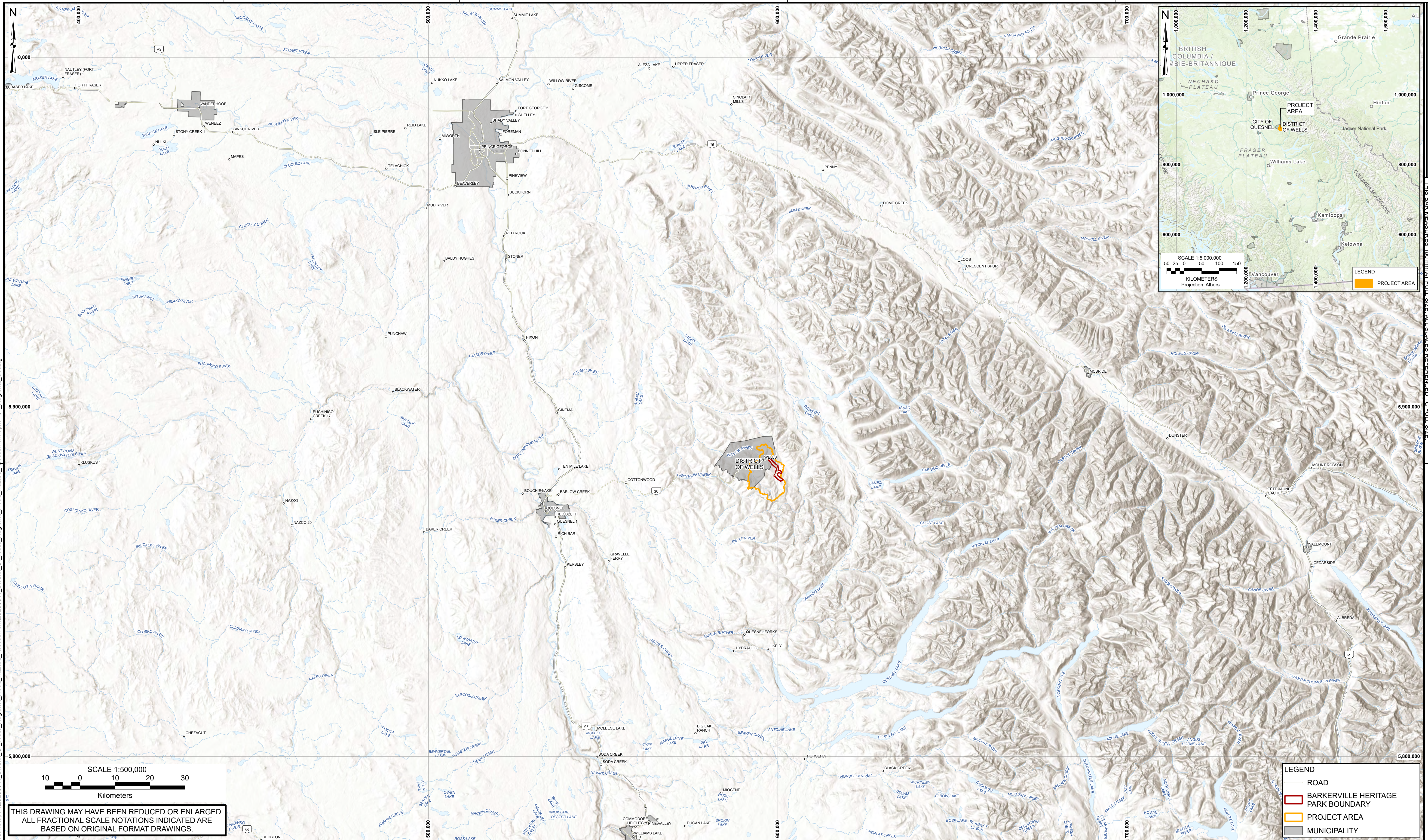
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DRAWINGS



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- NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
 2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT", AND DATED JUNE 2026.
 3. WORLD HILLSHADE BASEMAP PROVIDED BY ESRI.
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CLIENT:



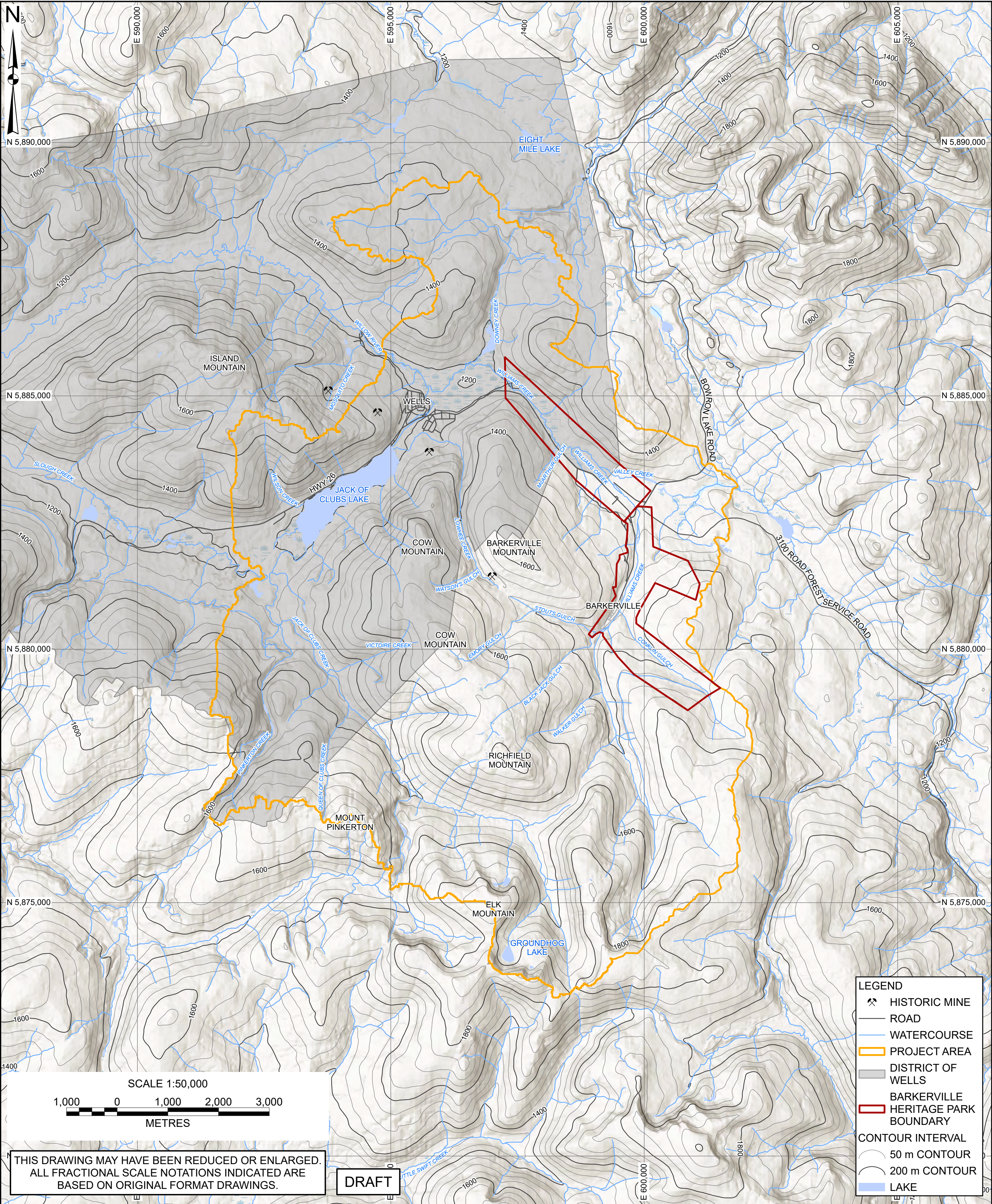
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TITLE: **REGIONAL SETTING**

PROJECT No.: **2546006**

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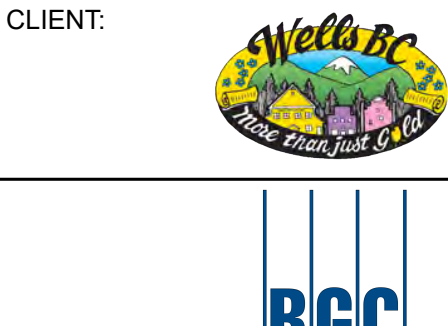
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PROJECT No.: 2546006	DWG No: 02

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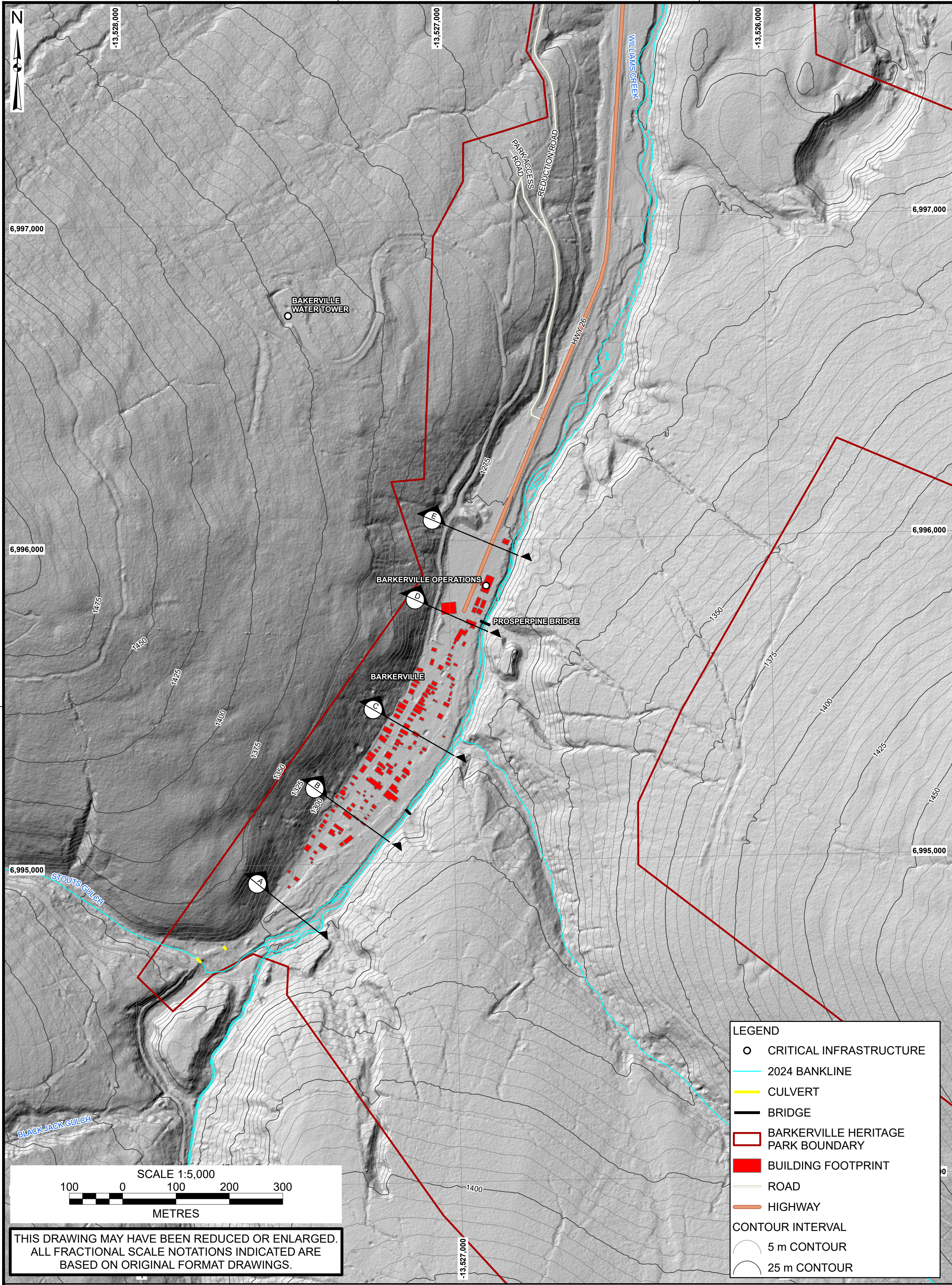
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TITLE: **EXISTING SITE CONDITIONS IN
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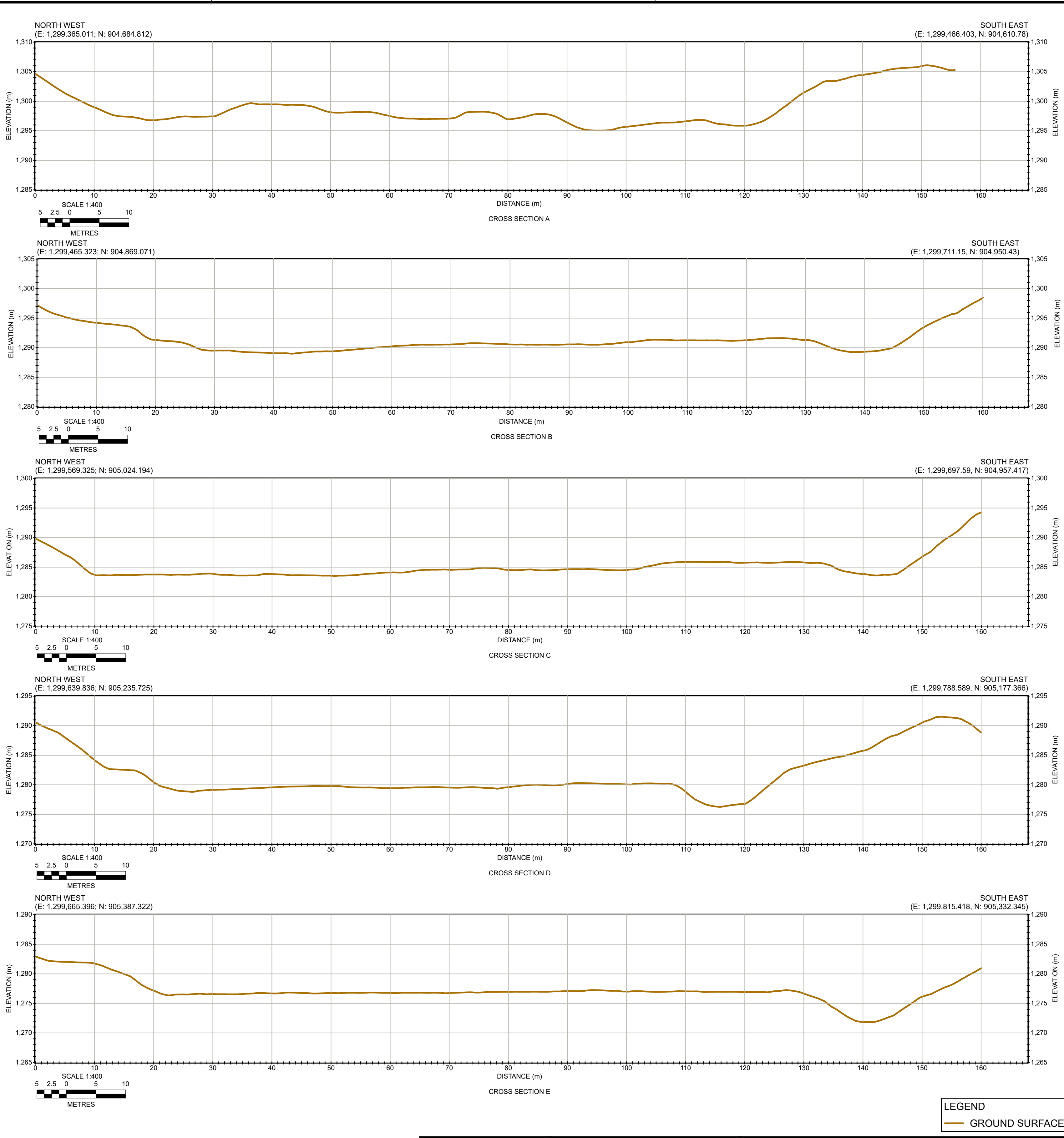
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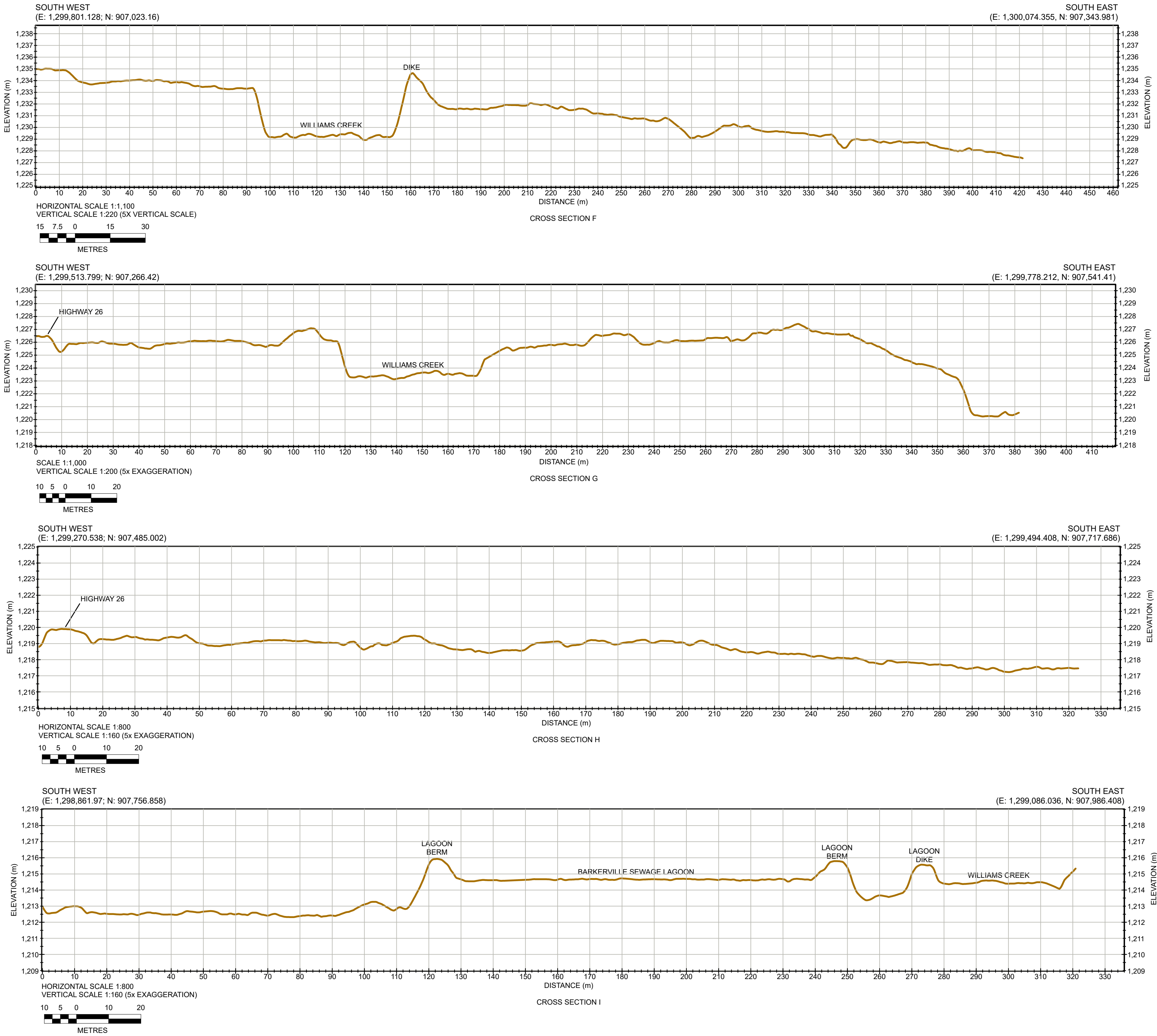
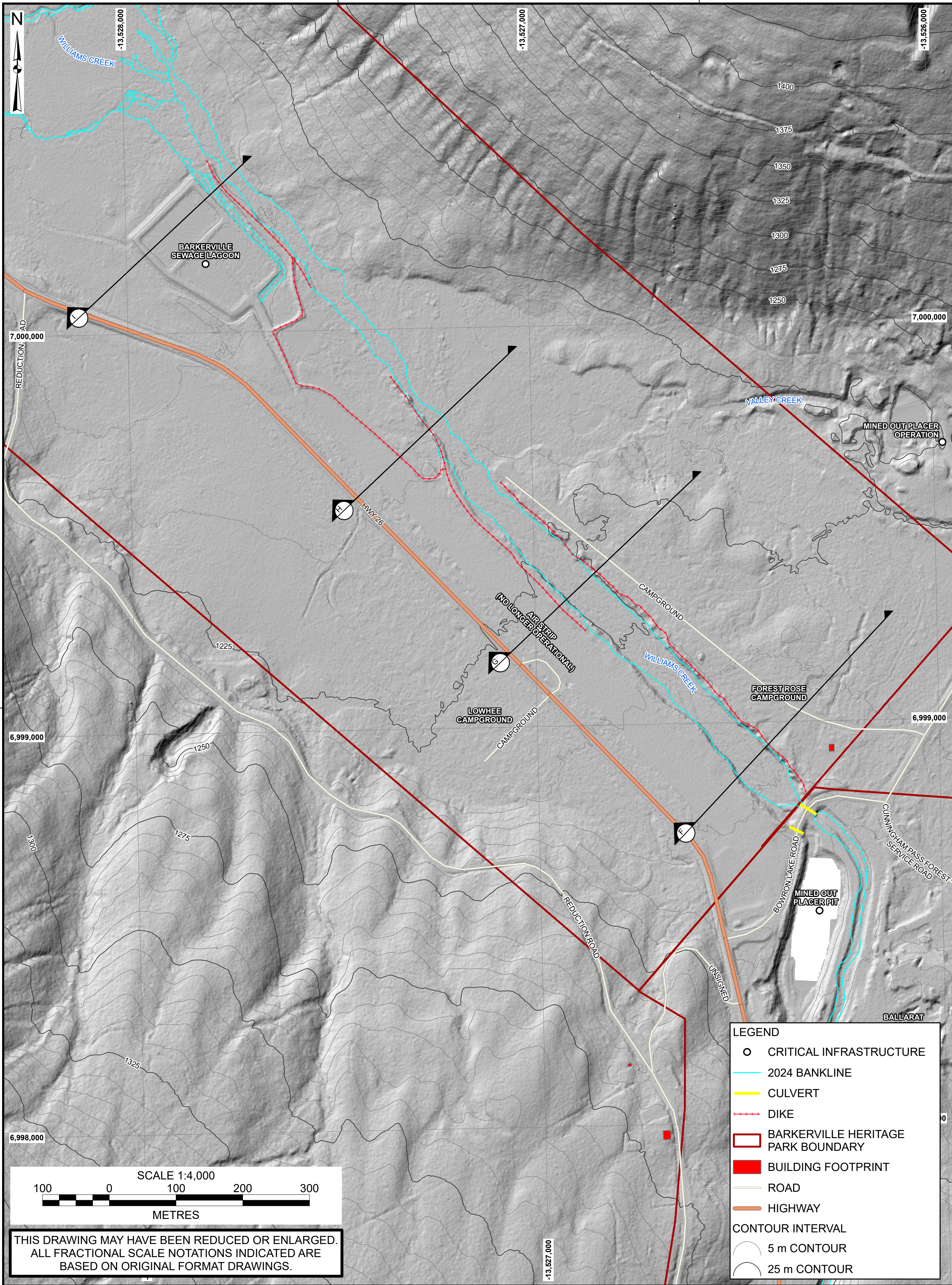
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SCALE:	AS SHOWN	CLIENT:		PROJECT:	DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT
DATE:	JUN 2026			TITLE:	2021 HILLSHADE DEM OF BARKERVILLE
DRAWN:	IL			PROJECT No.:	2546006
REVIEW:	PC			DWG No.:	04
APPROVED:	HW				

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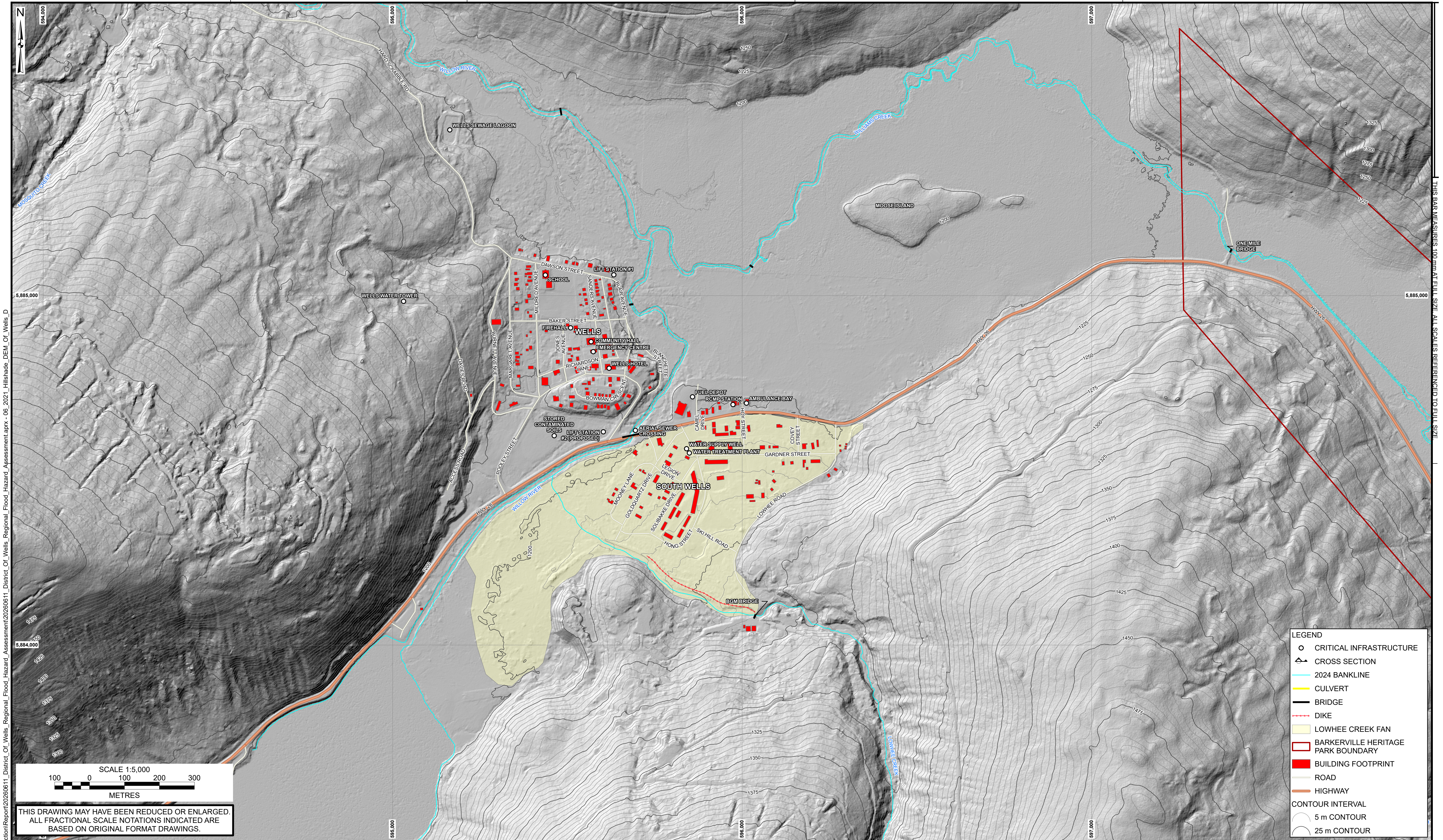


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 3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY AIRBORNE IMAGING, DATED JUNE 2021. CONTOUR INTERVAL IS 5 m.
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SCALE:	AS SHOWN	CLIENT:		PROJECT:		DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
DATE:	JUN 2026			TITLE:		2021 HILLSHADE DEM OF NORTH BARKERVILLE (D SIZE)	
DRAWN:	IL		PROJECT No.:		DWG No.:		
REVIEW:	PC		2546006		05		
APPROVED:	HW						

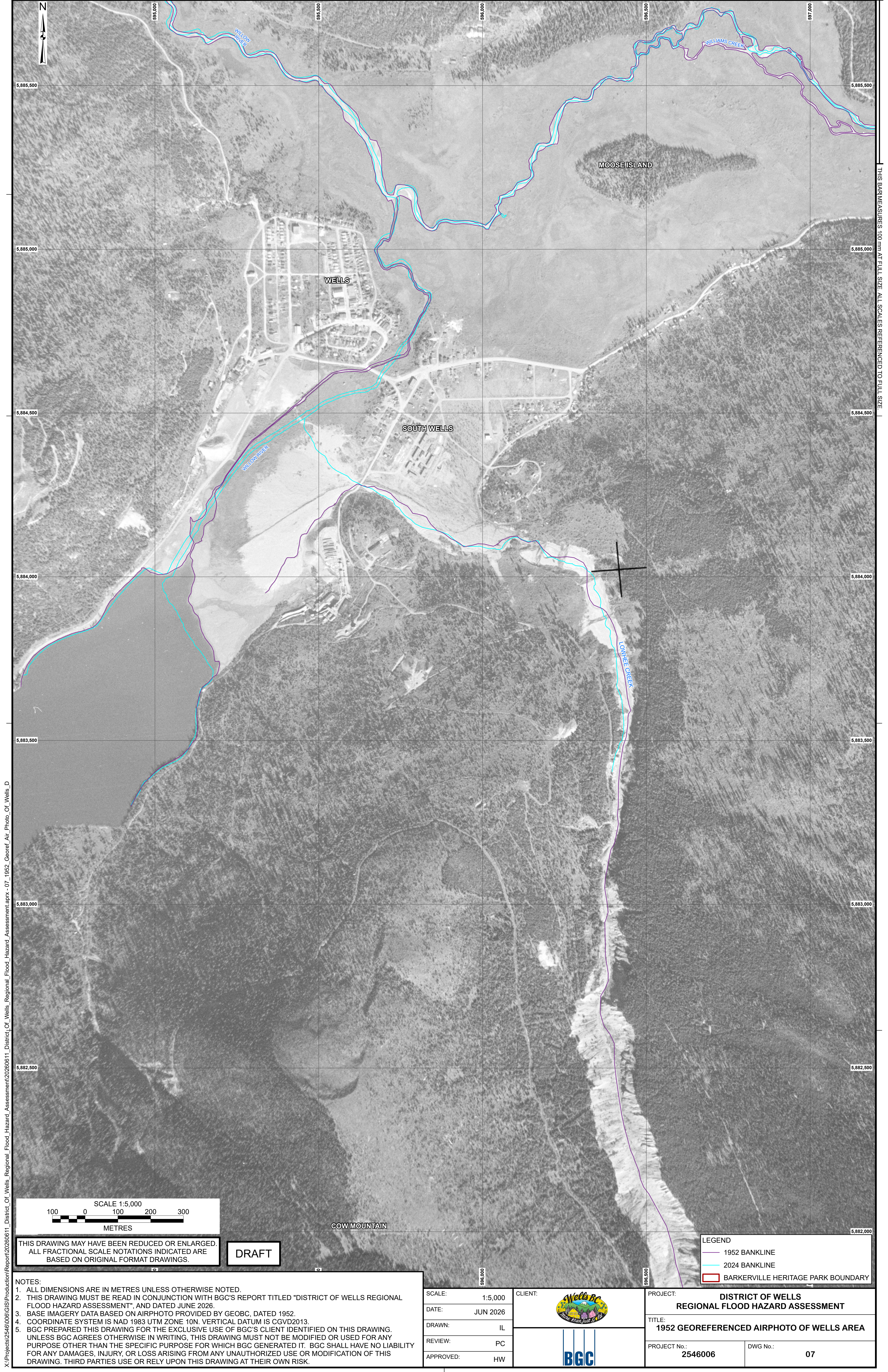


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2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT", AND DATED JUNE 2026.
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SCALE:	1:5,000	CLIENT:	
DATE:	JUN 2026		
DRAWN:	IL		
REVIEW:	PC		
APPROVED:	HW	PROJECT:	DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT
		TITLE:	2021 HILLSHADE DEM OF WELLS
		PROJECT No.:	2546006
		DWG No.:	06



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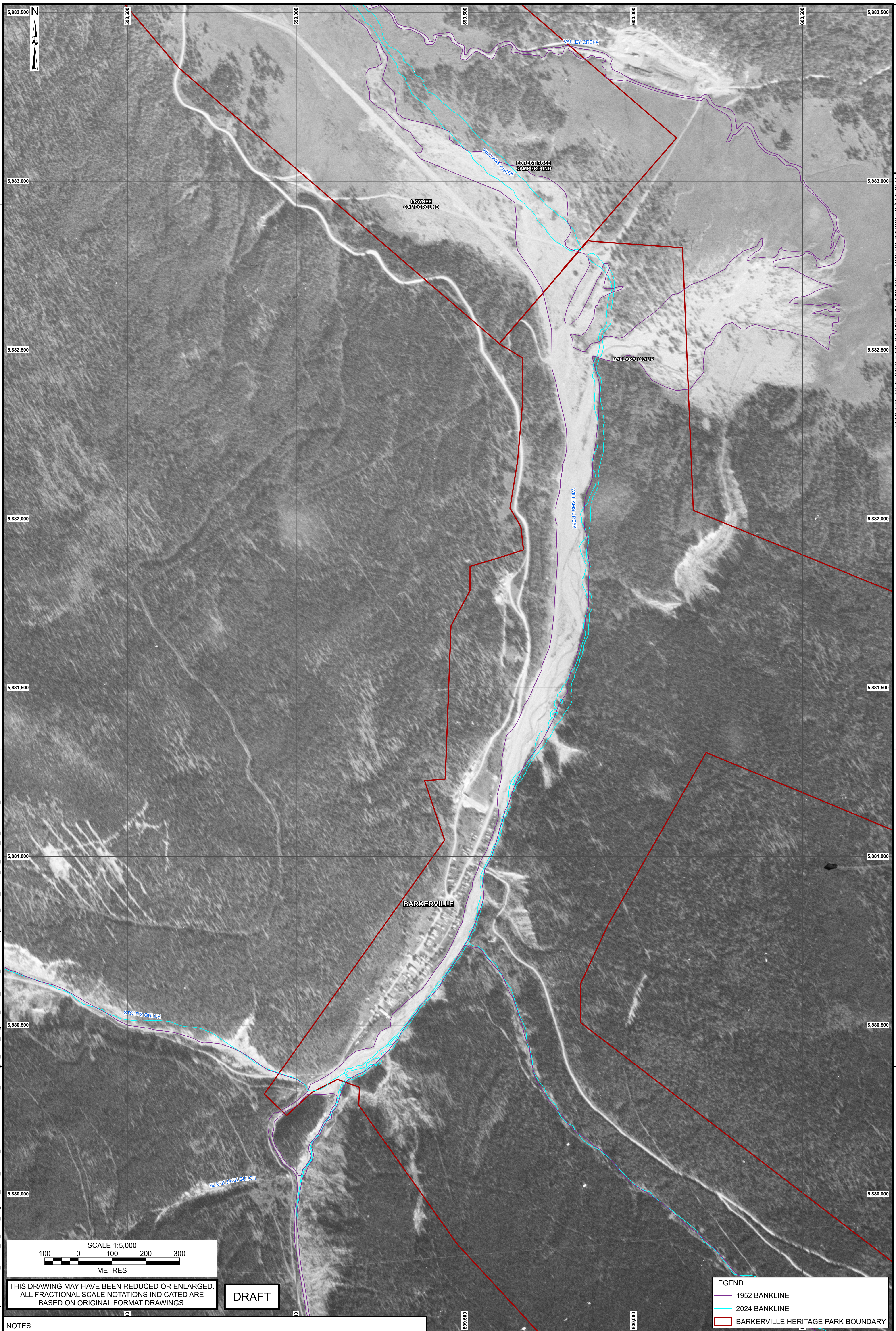
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 3. BASE IMAGERY DATA BASED ON AIRPHOTO PROVIDED BY GEOBC, DATED 1952.
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DATE:	JUN 2026
DRAWN:	IL
REVIEW:	PC
APPROVED:	HW

CLIENT:

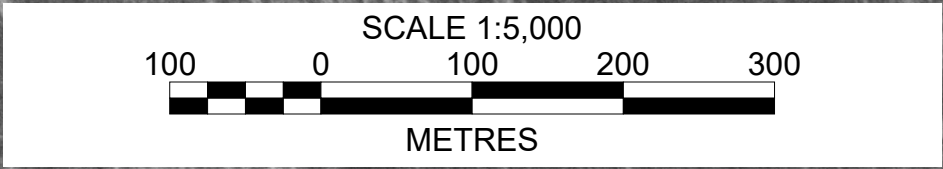


PROJECT: DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
TITLE: 1952 GEOREFERENCED AIRPHOTO OF WELLS AREA	
PROJECT No.: 2546006	DWG No.: 07



THIS BAR MEASURES 100 mm AT FULL SIZE ALL SCALES REFERENCED TO FULL SIZE

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DATE:	JUN 2026
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REVIEW:	PC
APPROVED:	HW

CLIENT:



PROJECT:

DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT

TITLE:

1952 GEOREFERENCED AIRPHOTO
OF BARKERVILLE AREA

PROJECT No.: 25

DWG No.:

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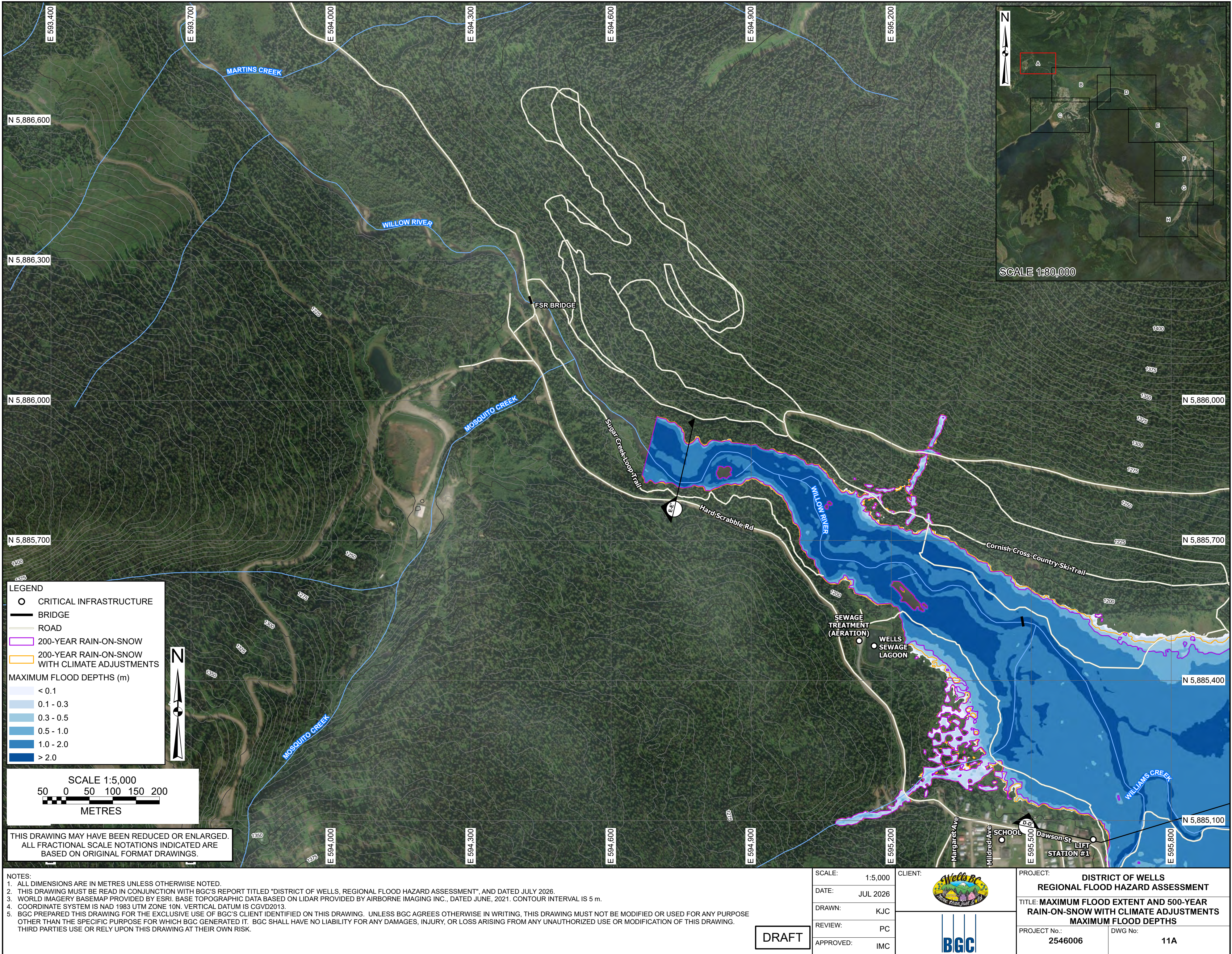
LEGEND

— 1952 BANKLINE

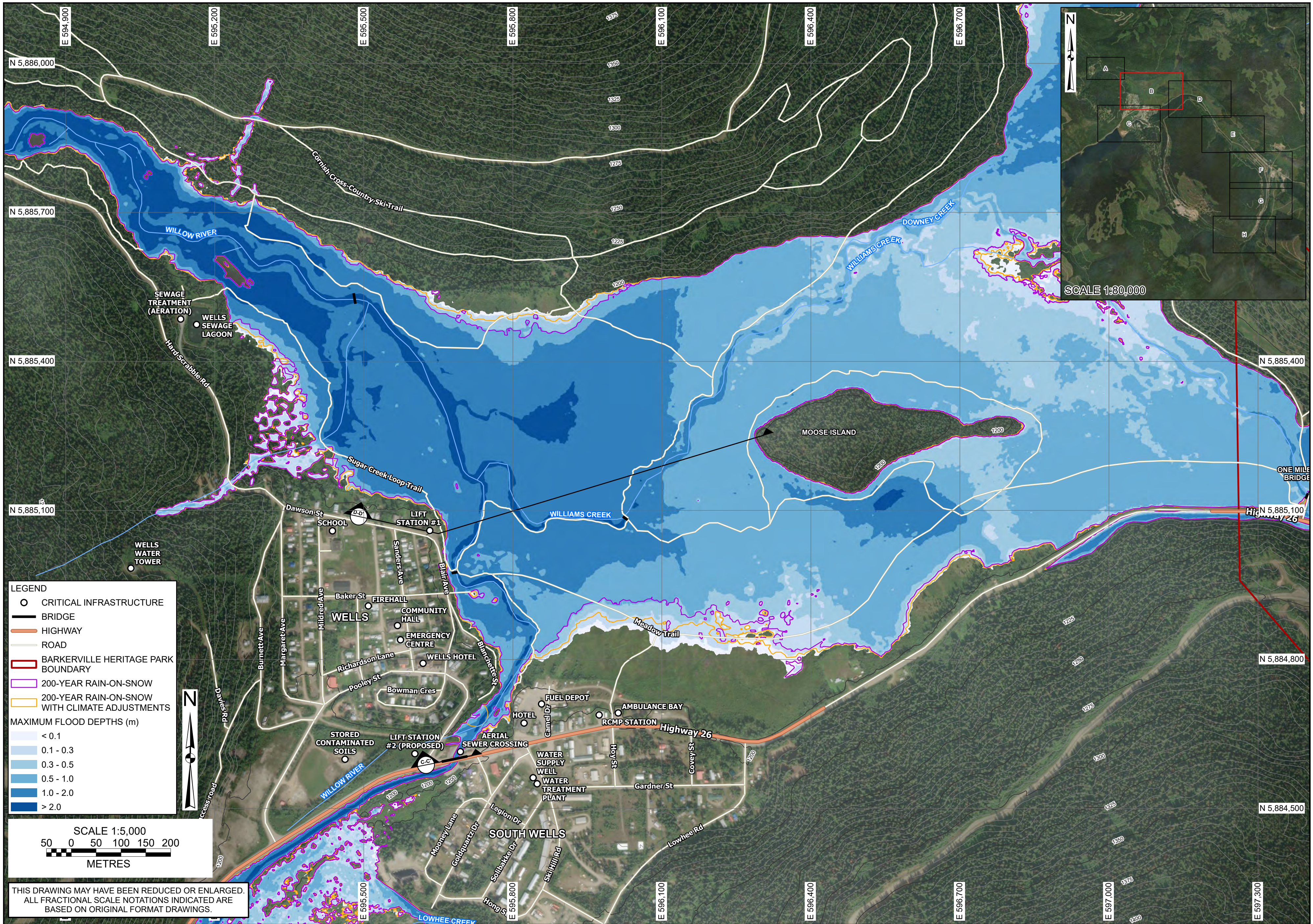
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 BARKERVILLE HERITAGE PARK BOUNDARY

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DRAWN:	KJC
REVIEW:	PC
APPROVED:	IMC

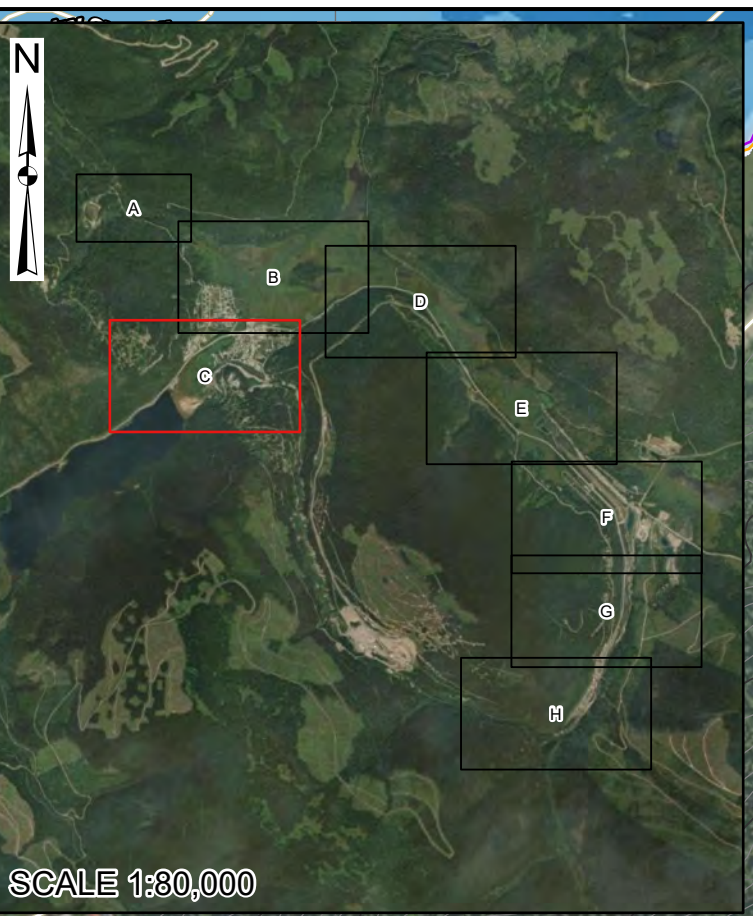
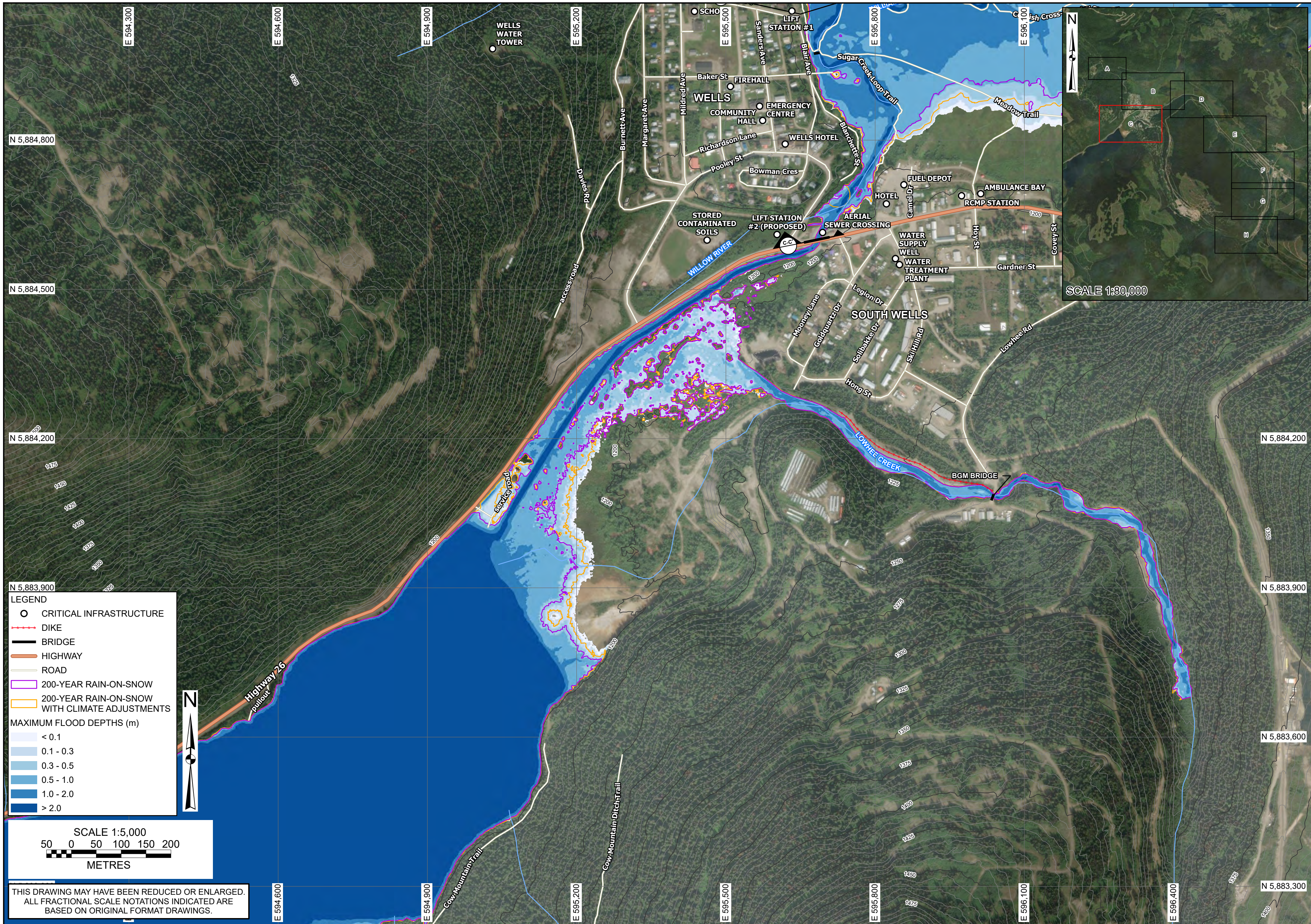
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BGC

PROJECT: DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
TITLE: MAXIMUM FLOOD EXTENT AND 500-YEAR RAIN-ON-SNOW WITH CLIMATE ADJUSTMENTS MAXIMUM FLOOD DEPTHS	
PROJECT No.: 2546006	DWG No.: 11B

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LEGEND

- CRITICAL INFRASTRUCTURE
- DIKE
- BRIDGE
- HIGHWAY
- ROAD
- 200-YEAR RAIN-ON-SNOW
- 200-YEAR RAIN-ON-SNOW WITH CLIMATE ADJUSTMENTS

MAXIMUM FLOOD DEPTHS (m)

- < 0.1
- 0.1 - 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- 1.0 - 2.0
- > 2.0

SCALE 1:5,000

50 0 50 100 150 200

METRES

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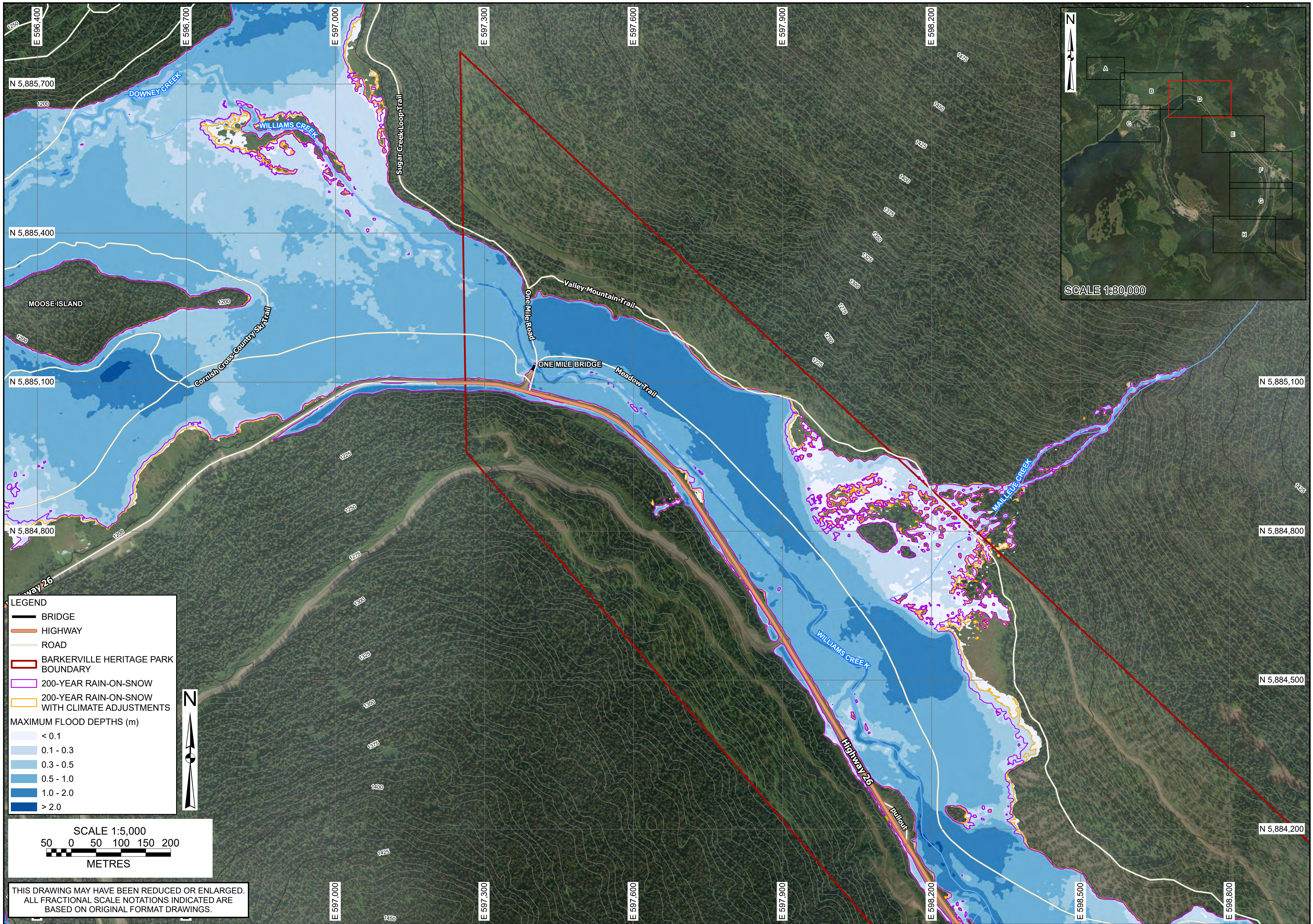
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CLIENT:




PROJECT: DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
TITLE: MAXIMUM FLOOD EXTENT AND 500-YEAR RAIN-ON-SNOW WITH CLIMATE ADJUSTMENTS MAXIMUM FLOOD DEPTHS	
PROJECT No.: 2546006	DWG No: 11C

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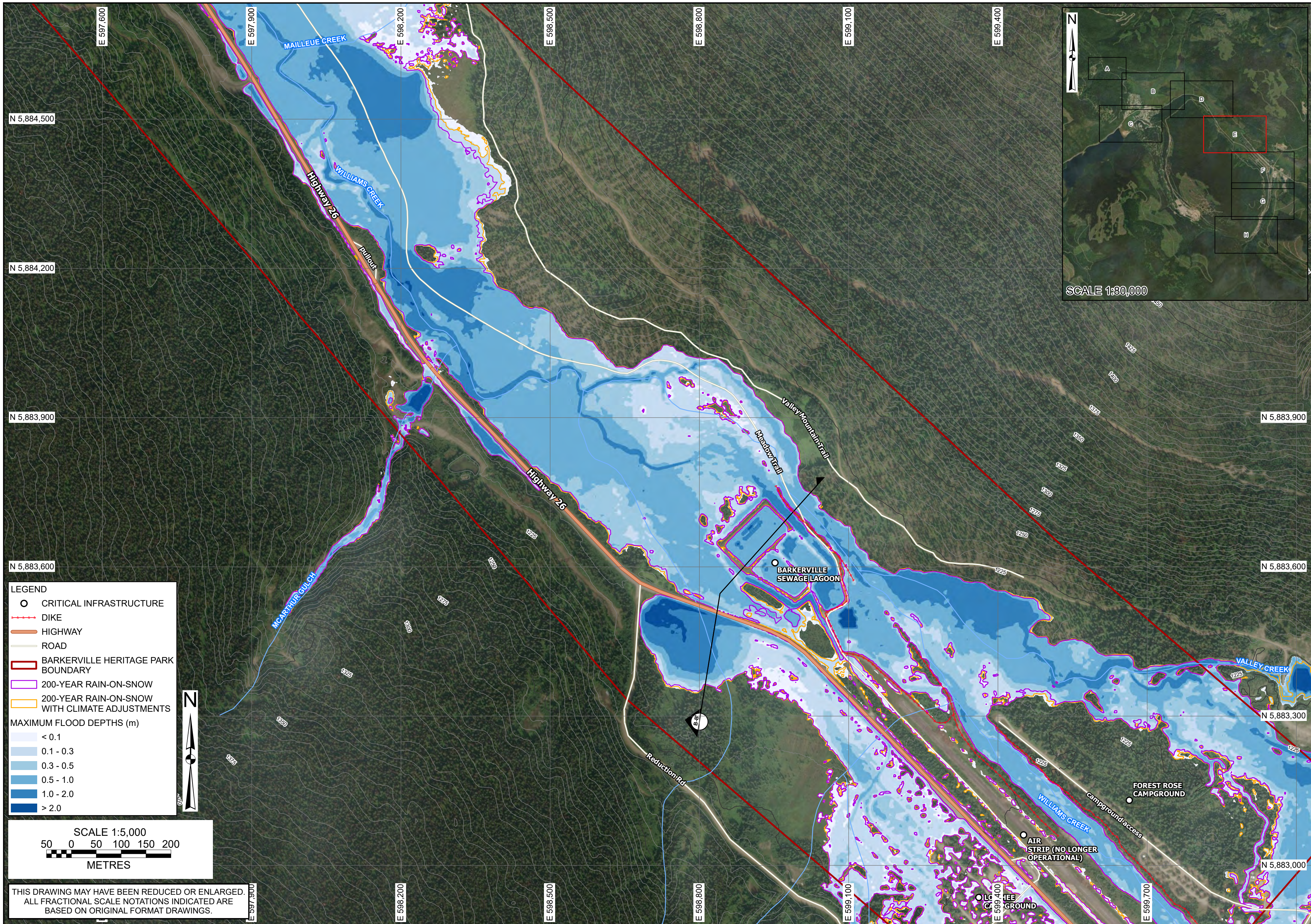
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CLIENT:



PROJECT: DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
TITLE: MAXIMUM FLOOD EXTENT AND 500-YEAR RAIN-ON-SNOW WITH CLIMATE ADJUSTMENTS MAXIMUM FLOOD DEPTHS	
PROJECT No.: 2546006	DWG No: 11D

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REVIEW:	PC
APPROVED:	IMC

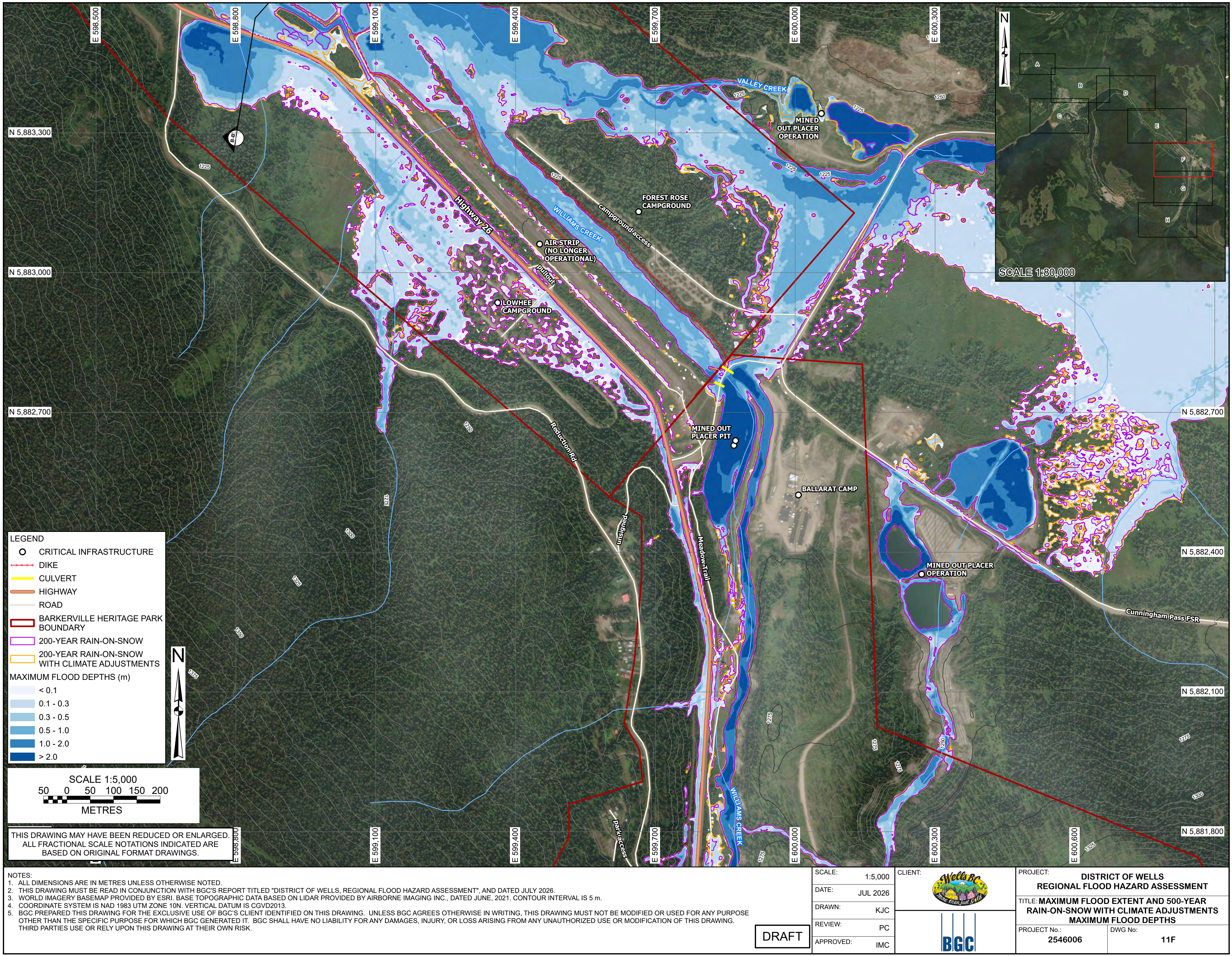
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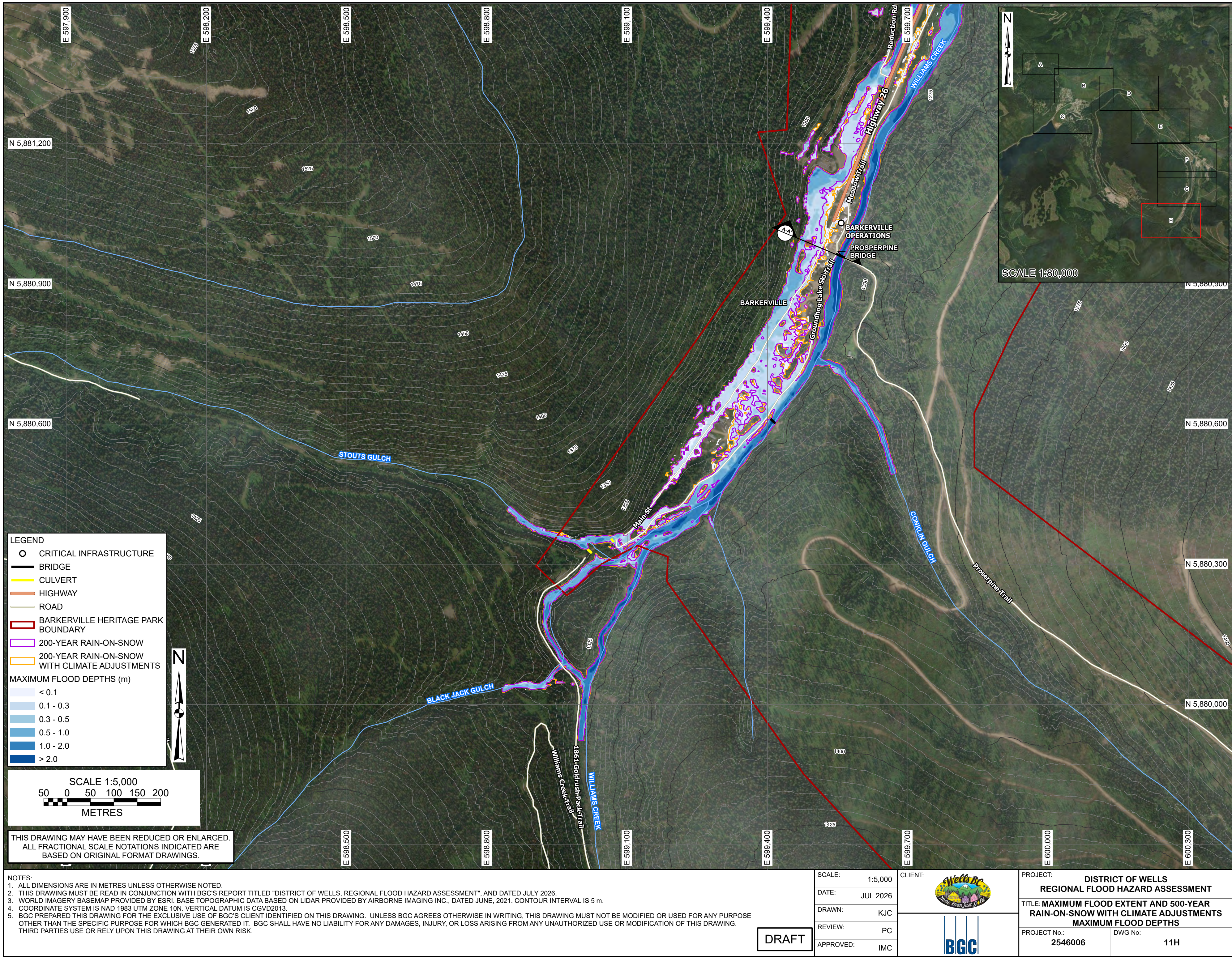
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PROJECT: DISTRICT OF WELLS REGIONAL FLOOD HAZARD ASSESSMENT	
TITLE: MAXIMUM FLOOD EXTENT AND 500-YEAR RAIN-ON-SNOW WITH CLIMATE ADJUSTMENTS MAXIMUM FLOOD DEPTHS	
PROJECT No.: 2546006	DWG No: 11E

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APPENDIX A

INFLOW HYDROGRAPHS



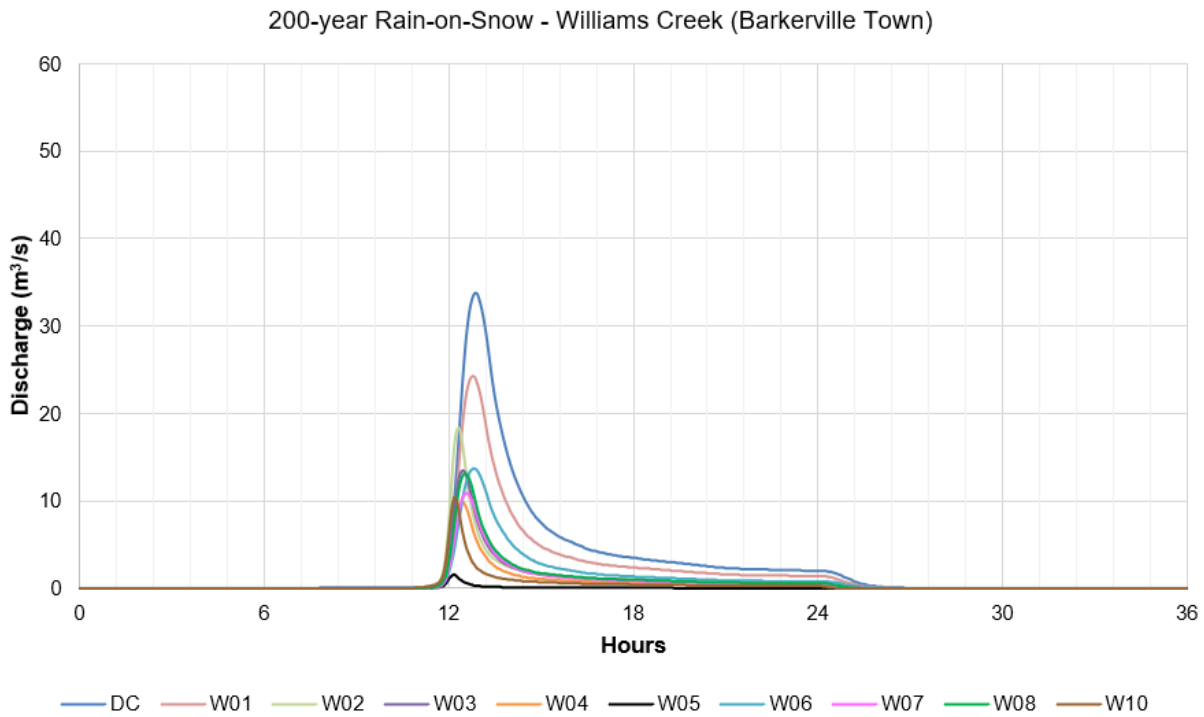


Figure A-1 Inflow hydrographs for the 200-year rain-on-snow scenario for individual watersheds along Williams Creek within Barkerville Town.

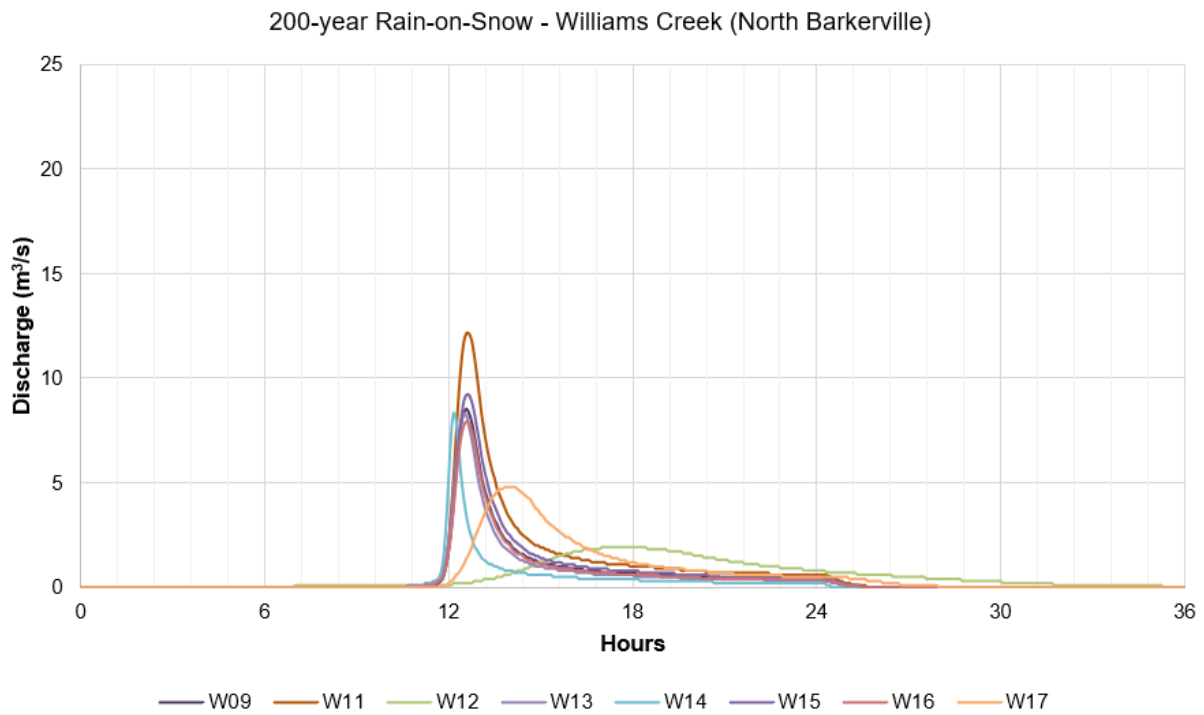


Figure A-2 Inflow hydrographs for the 200-year rain-on-snow scenario for individual watersheds along Williams Creek North Barkerville.

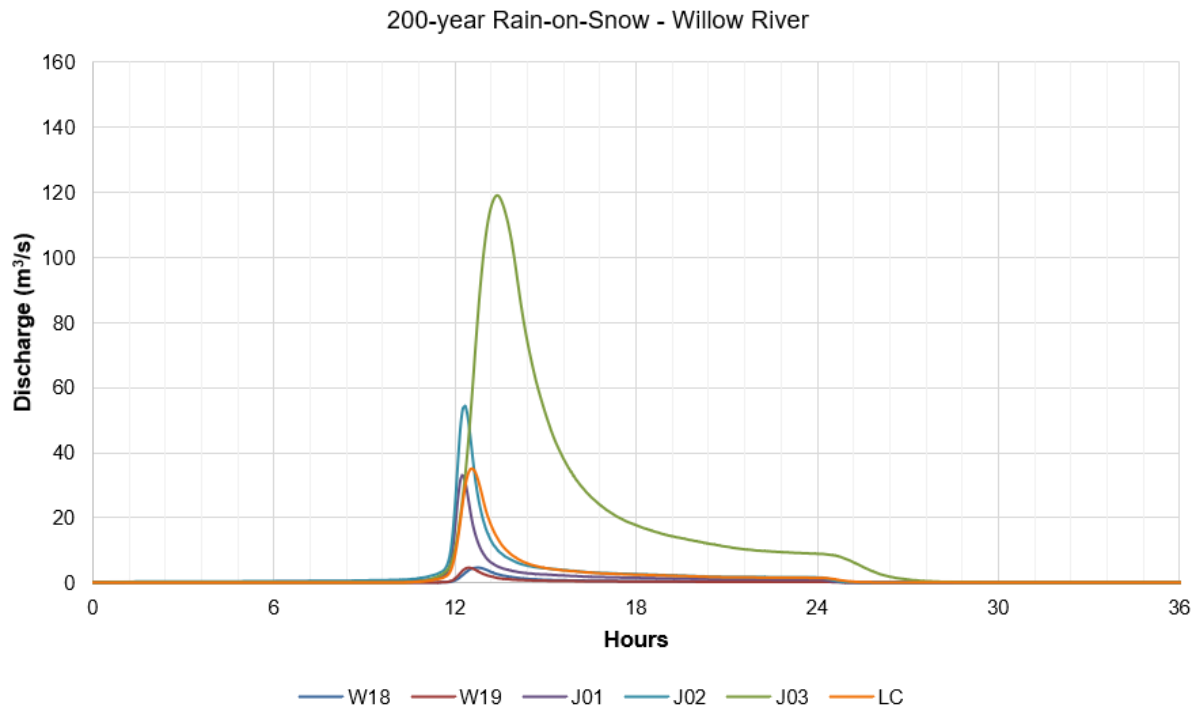


Figure A-3 Inflow hydrographs for the 200-year rain-on-snow scenario for individual watersheds along Willow River.

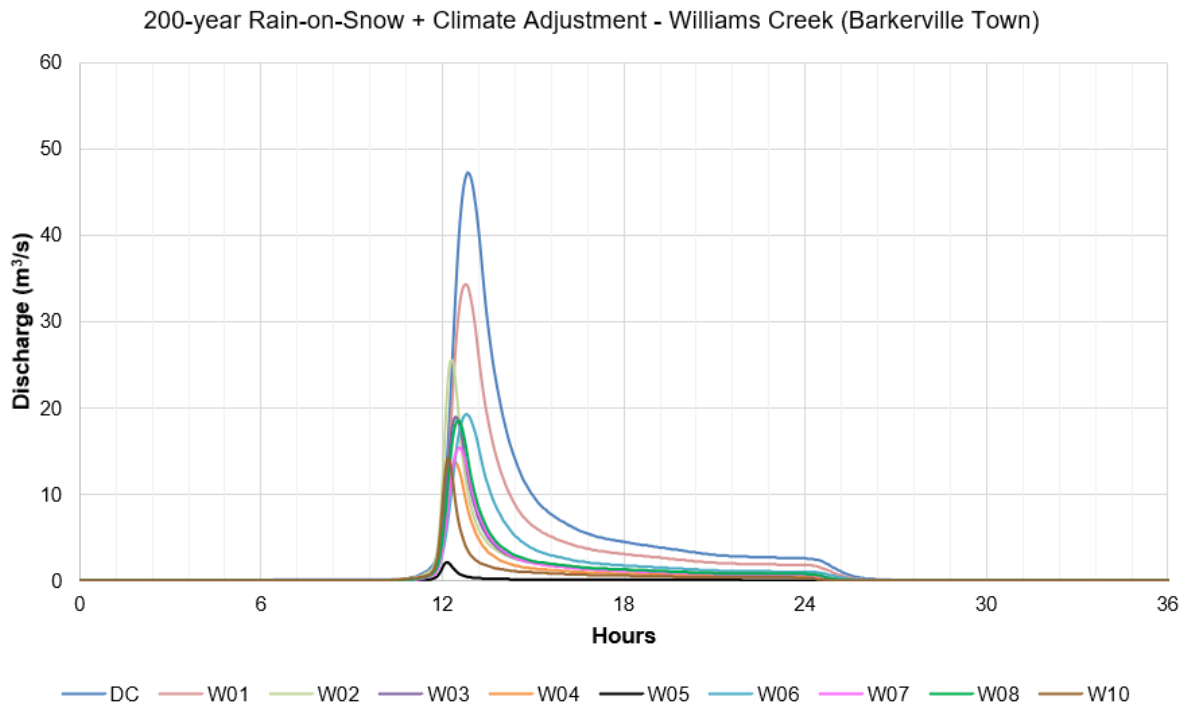


Figure A-4 Inflow hydrographs for the 200-year rain-on-snow with climate adjustment scenario for individual watersheds along Williams Creek within Barkerville Town.

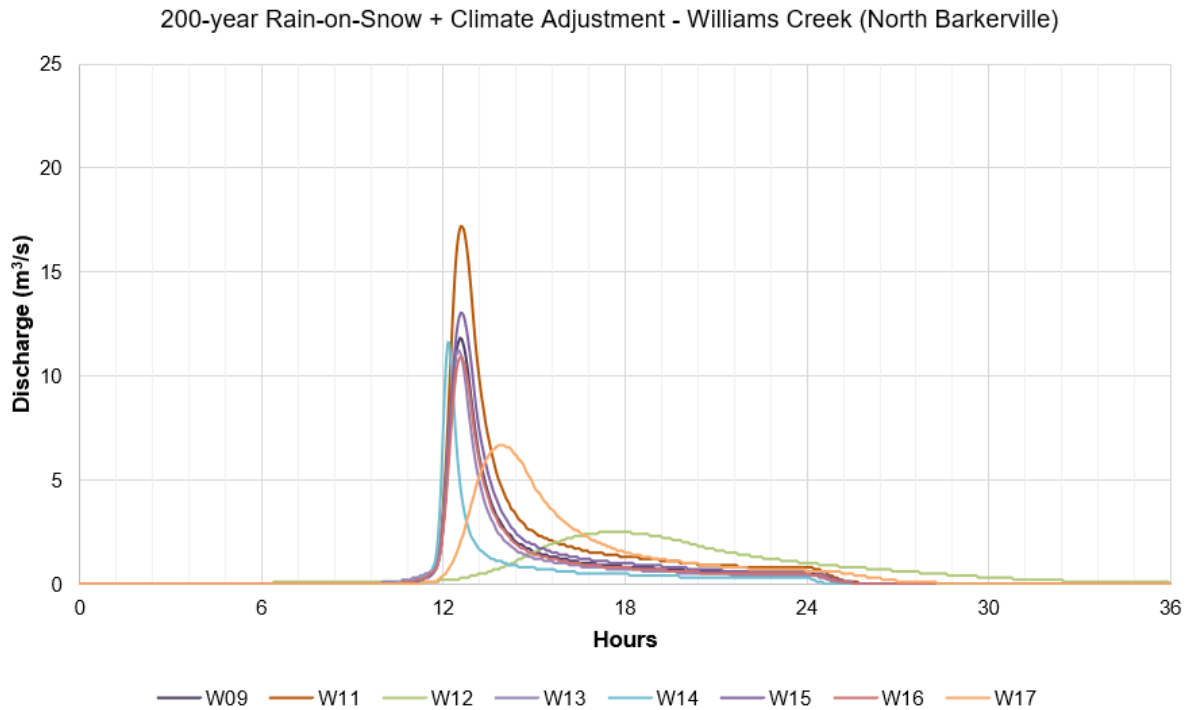


Figure A-5 Inflow hydrographs for the 200-year rain-on-snow with climate adjustment scenario for individual watersheds along Williams Creek North Barkerville.

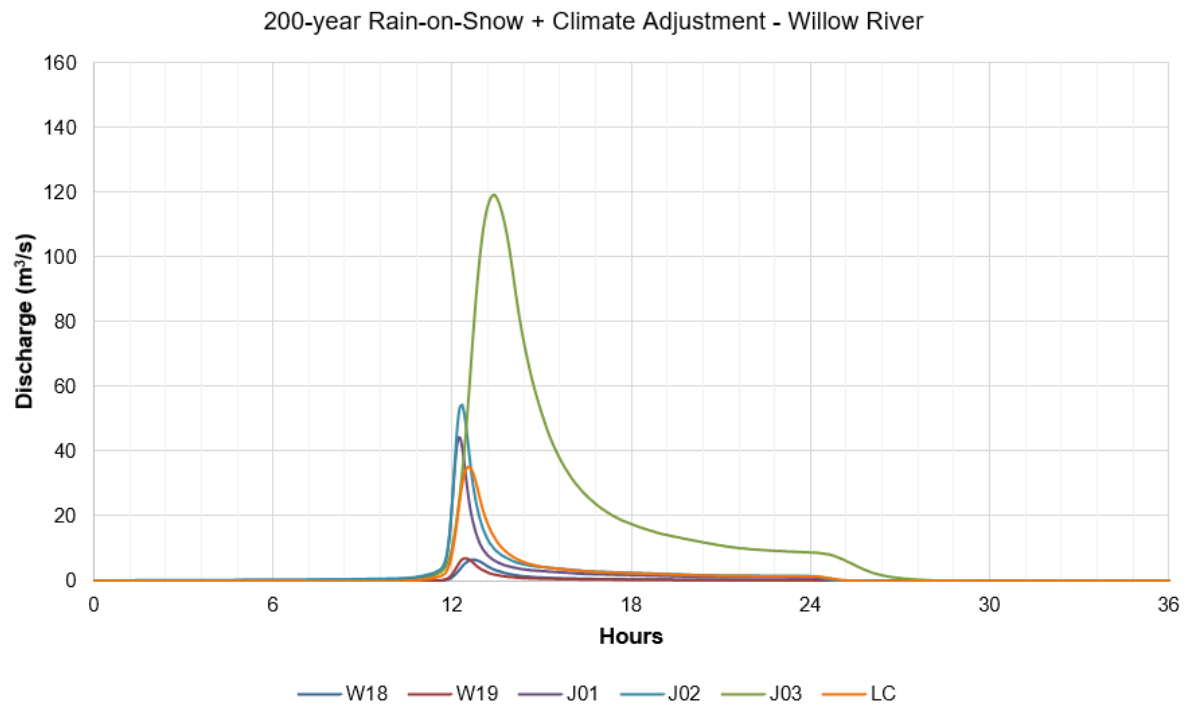


Figure A-6 Inflow hydrographs for the 200-year rain-on-snow with climate adjustment scenario for individual watersheds along Willow River.

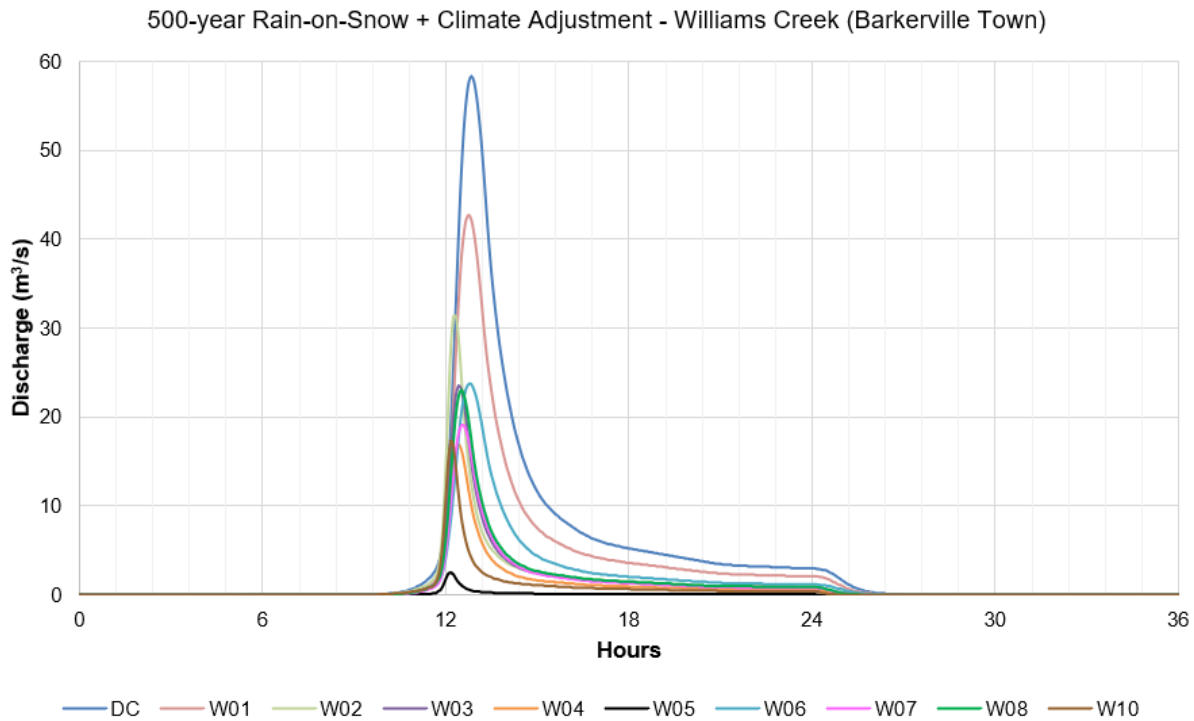


Figure A-7 Inflow hydrographs for the 500-year rain-on-snow with climate adjustment scenario for individual watersheds along Williams Creek within Barkerville Town.

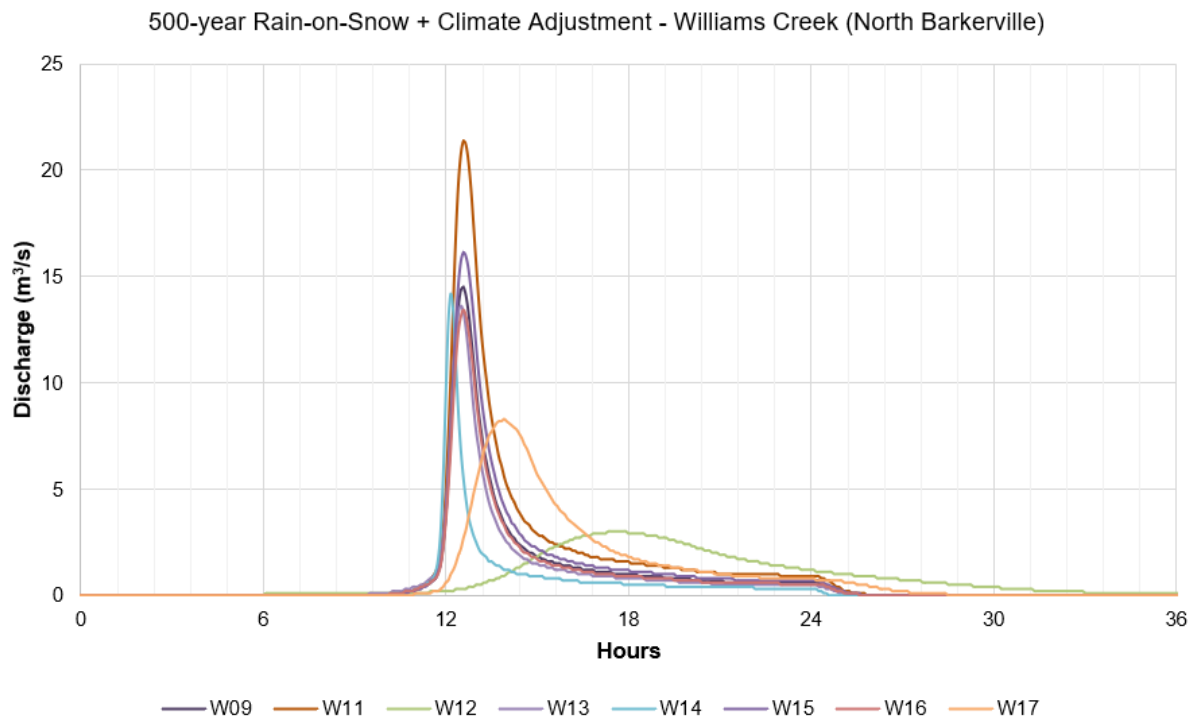


Figure A-8 Inflow hydrographs for the 500-year rain-on-snow with climate adjustment scenario for individual watersheds along Williams Creek North Barkerville.

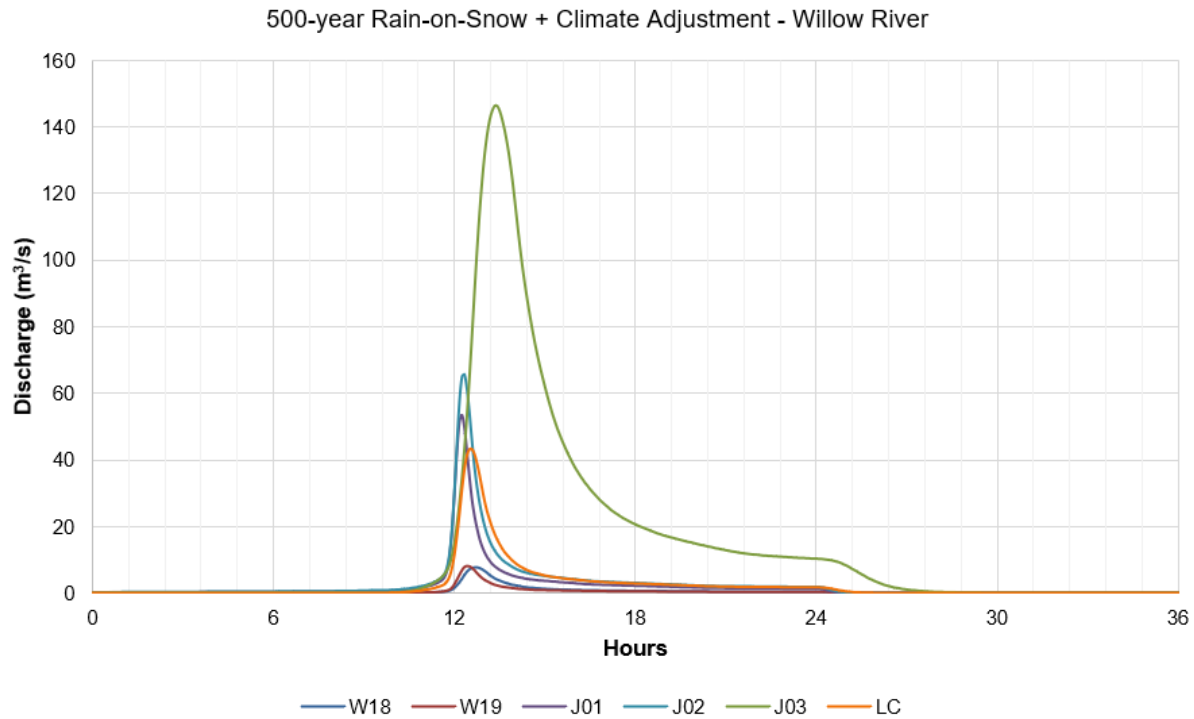


Figure A-9 Inflow hydrographs for the 500-year rain-on-snow with climate adjustment scenario for individual watersheds along Willow River.