FORT VERMILION FLOOD HAZARD STUDY

FINAL REPORT

Prepared for:

Alberta

17 November 2022

NHC Ref. No. 1004659

nhc
northwest hydraulic consultants
water resource specialists

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Alberta Environment and Parks
Edmonton, Alberta

Prepared by:

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DISCLAIMER

EXECUTIVE SUMMARY

Alberta Environment and Parks retained Northwest Hydraulic Consultants Ltd. in April 2019 to complete a flood hazard study for the Peace River at Fort Vermilion, Alberta. The study area consists of a 28 km long reach of the Peace River through Mackenzie County, including Fort Vermilion and North Vermilion. This study was facilitated under the Flood Hazard Identification Program (FHIP) with the intent to enhance public safety and reduce future flood damages within the Province of Alberta. Results from this study are intended to inform local land use planning decisions, flood mitigation projects, and emergency response planning.

The Fort Vermilion Flood Hazard Study consists of seven major components, including:

- Survey and Base Data Collection
- Open Water Hydrology Assessment
- Open Water Hydraulic Modelling
- Open Water Flood Inundation Mapping
- Ice Jam Modelling
- Ice Jam Flood Inundation Mapping
- Design Flood Hazard Identification and Mapping

The collection of survey and base data is foundational to the overall study and is required to develop hydraulic models for the study reach. It includes survey of river channel cross sections, and collection of digital terrain model (DTM) data, drawings of structures interacting with river flood flows, and other base data that are essential for hydraulic model development and inundation mapping. The river survey was completed in June 2019. A total of 37 cross sections were surveyed at an average spacing of 910 m along the 28 km Peace River reach. The bridge and culvert for the Highway 88 crossing were also surveyed. Associated bridge files for these two structures were obtained from Alberta Transportation. DTM data for the study area were provided by AEP. The data were derived from a LiDAR survey conducted in October 2018. Additional base data assembled included historic flood information, base mapping data and hydrometric gauge information.

The open water hydrology assessment was required to provide estimates of flood frequencies at Fort Vermilion under an unregulated open water condition. Peace River flows have been regulated since 1968 by the W. A. C. Bennett and Peace Canyon dams. In this open water hydrology assessment, flow naturalization was performed to remove flow regulation effects on flood peak discharges at Fort Vermilion. Flood frequency analysis was then performed on a series of natural and naturalized instantaneous peak discharges that spans from 1915 to 1931 and from 1958 to 2018. Flood peak discharges for Peace River at Fort Vermilion were estimated for various return periods from 2 to 1000 years. The estimates were also applied to Peace River below Boyer River within the study reach.

The open water hydraulic modelling began with development of a one-dimensional hydraulic model for the Peace River study reach based on the 37 surveyed cross sections and DTM. The model was calibrated for low and high flow conditions using available data including: water level and discharge measurements during the river survey (for low flow calibration), surveyed highwater mark elevations for the June 1990 flood, and hydrometric gauge data for Peace River at Fort Vermilion from the Water Survey of Canada. The calibrated hydraulic model was used to calculate water surface profiles along the study reach for 13 design open water flood peak discharges corresponding to the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year return periods. The computed flood levels were then used to determine the extent of inundation for each return period. The results of this inundation analysis are presented as the open water flood inundation map library.

The Peace River at Fort Vermilion has a history of ice jam flooding. Some of the highest flood levels on record were attributed to ice jam events during breakup (e.g. 1934, 1963, 2018, and 2020). The 50-, 100- and 200-year breakup ice jam flood levels were estimated for the Peace River WSC gauge station at Fort Vermilion via frequency analysis on historical peak breakup levels. An ice jam hydraulic model was created by enhancing the open water hydraulic model and calibrated to the surveyed highwater mark elevations for the 2018 ice jam flood event. The calibrated model was then validated with highwater mark elevations for the 2020 ice jam flood event. The model was used to compute an ice jam rating curve at the gauge and ice jam profiles along the study reach corresponding to the 50-, 100- and 200-year return periods. Inundation analysis was performed based on these flood level profiles, and ice jam flood frequency maps were created for the 50-, 100- and 200-year return periods.

The flood levels for the ice jam design flood were notably higher than for the open water design flood throughout the study reach. Therefore, the 100-year ice jam design flood was adopted as the governing design flood for the Peace River at Fort Vermilion, and the floodway criteria and flood hazard identification maps were created based on this design flood event. The calculated ice jam design flood levels were used to delineate the floodway and flood fringe boundaries through the study area. Areas of deeper or faster moving water outside of the floodway (within the flood fringe) were identified as high hazard flood fringe areas.

CREDITS AND ACKNOWLEDGEMENTS

Northwest Hydraulic Consultants Ltd. would like to express appreciation to Alberta Environment and Parks for initiating this study, making extensive background information available, and providing the project team with valuable technical input throughout the project. James Choles, P.Eng., CFM managed and directed the study on behalf of Alberta Environment and Parks. Thanks are also expressed to Dr. Jennifer Nafziger and other members of Alberta Environment and Parks who provided relevant information and valuable comments throughout the course of the work.

The following NHC personnel participated in the study:

- Robyn Andrishak – Project Manager responsible for the overall direction of the project
- Michael Brayall – Hydraulic Modelling Lead responsible for hydraulic modelling
- Ken Zhao – Hydrology Specialist responsible for open water hydrology and ice jam frequency analysis
- Dan Healy – Ice Jam Modelling Specialist responsible the ice jam modelling component and support on ice jam frequency analysis
- Rebecca Himsl – GIS Specialist responsible for creation of base maps and map production
- Sarah North – GIS Specialist providing advices and senior review for inundation mapping
- Gary Van Der Vinne – Senior Reviewer providing technical advices and senior review.

This report was prepared by Mr. Michael Brayall, M.Sc., P.Eng. and Dr. C. H. (Ken) Zhao, P.Eng. with inputs of Dr. Dan Healy, P.Eng. in the chapter of Ice Jam Modelling.

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LIST OF ABBREVIATIONS AND ACRONYMS

ADCP	Acoustic Doppler Current Profiler
ADV	Acoustic Doppler velocimeter
AEP	Alberta Environment and Parks
AENV	Alberta Environment
AHS	Alberta Health Services
ASCM	Alberta Survey Control Monuments
AT	Alberta Transportation
BF	Bridge File
CGVD28	Canadian Geodetic Vertical Datum of 1928
CSRS	Canadian Spatial Refence System
CSRS-PPP	Canadian Spatial Reference System Precise Point Positioning
CP	Control point
DTM	Digital terrain model
EDBC	Engineers and Geoscientists of British Columbia
FHIP	Flood Hazard Identification Program
GNSS	Global Navigation Satellite Systems
GPS	Global Positioning System
HWM	Highwater Mark
LiDAR	Light Detection and Ranging
NAD83	North American Datum of 1983
NHC	Northwest Hydraulic Consultants Ltd.
1D	One-dimensional
PXS	Planned cross section
RS	River Station
RTK	Real Time Kinematic
SPBM	Semi-permanent benchmarks
3TM	Three-degree Transverse Mercator
TPR	Town of Peace River
TOB	Top of bank
WSC	Water Survey of Canada

1 INTRODUCTION

1.1 Study Background

The Fort Vermilion Flood Hazard Study was initiated by Alberta Environment and Parks (AEP) to identify and assess flood hazards along a 28 km long reach of the Peace River within Mackenzie County, including Fort Vermilion and North Vermilion (**Figure 1**). This study was facilitated under the Flood Hazard Identification Program (FHIP) with the intent to enhance public safety and reduce future flood damages within the Province of Alberta. Results from this study are intended to inform local land use planning decisions, flood mitigation projects, and emergency response planning.

A flood mapping study for Fort Vermilion was completed in 2000 by AEP (Alberta Environment, 2000), formerly known as Alberta Environment (AENV). The present study provides an update of this work to account for additional flow data, current survey data, and contemporary methods of data collection and analysis. Further, the current study incorporates a larger study area and includes both open water and ice jam flood scenarios. The current study is comprised of the following major study components:

1) Survey and Base Data Collection
2) Open Water Hydrology Assessment
3) Open Water Hydraulic Modelling
4) Open Water Flood Inundation Mapping
5) Ice Jam Modelling
6) Ice Jam Flood Inundation Mapping
7) Design Flood Hazard Identification and Mapping

1.2 Study Objectives

The primary tasks, services, and deliverables associated with this report are:

- river cross section surveys;
- hydraulic structure data collection;
- survey and digital terrain model (DTM) data integration;
- documentation of open water and ice jam flood history;
- open water hydrology assessment to provide flood frequency estimates;
- development of a calibrated, one-dimensional (1D) open water hydraulic model, and enhancement of the model to simulate ice jams;

- simulation of open water and ice jam floods of selected return periods, and creation of water surface profiles throughout the study reach;
- sensitivity analysis on selected modelling parameters;
- production of flood inundation maps for selected return periods;
- determination of floodway criteria and creation of design flood water surface profiles throughout the study reach; and
- production of floodway criteria maps and design flood hazard maps.

1.3 Study Area and Reach

The Fort Vermilion flood hazard study area is located approximately 77 km southeast of the town of High Level and 250 km north of the town of Peace River (TPR), in northern Alberta (**Figure 2**). It lies along approximately 28 km of the lower Peace River in Mackenzie County, including Fort Vermilion and North Vermilion situated on the south and north sides of the river, respectively (**Figure 1**).

The Peace River originates in the Rocky Mountains in northern British Columbia (BC) and flows to the northeast through northern Alberta. The headwaters of the Peace River consist of glacial fed mountain rivers and creeks that feed into Williston Lake, a large reservoir created by the W. A. C. Bennett Dam. From the W. A. C. Bennett Dam, the Peace River flows into Dinosaur Lake, the headpond of the Peace Canyon Dam located near Hudson's Hope, BC. After crossing the Alberta-BC border, the Peace River generally flows in an eastern direction toward TPR, which is located about 395 km downstream of the W. A. C. Bennett Dam. Beyond TPR, the river flows north and then northeast for about 435 km to Fort Vermilion. It flows through the study area from the southwest to the northeast. The Peace River ultimately enters the Slave River after passing the Peace-Athabasca Delta, which is located approximately 400 km downstream of the study area.

Peace River flows became regulated in 1968 when construction of the W. A. C. Bennett Dam concluded. It took four years from 1968 through 1971 for Williston Lake to be filled to its normal operating elevation range. Although this large reservoir is located about 830 km upstream of Fort Vermilion, its outflows account for approximately 60% of the runoff volume in the middle and lower reach of the Peace River (NHC, 2016). Operations at the W. A. C. Bennett Dam generally tend to increase Peace River flows through winter and decrease in late spring and summer.

2 SURVEY AND BASE DATA COLLECTION

2.1 Procedure and Methodology

The survey program was completed between 5 and 12 June 2019. The objective of the survey program was to survey channel cross sections along the study reach to support development of a 1D hydraulic model. Before commencement of the work, a survey plan was submitted to and approved by AEP. A site visit was conducted on 5 June to inspect the study reach and assess the overall condition of the river channel and floodplain.

Ground positioning for the survey was measured using Real Time Kinetic (RTK) Global Navigation Satellite Systems (GNSS) and Trimble R10 GNSS receivers. Peace River bathymetric surveys were conducted in areas generally deeper than 0.3 m using a CEESCOPE™ dual frequency digital echo sounder mounted on a boat to measure the water depth under the transducer. The position and elevation of the transducer were recorded with the GNSS receiver, which was mounted directly above the transducer. River bed elevations were derived by subtracting sounding depths from transducer elevations. Elevations of shallower areas in the river channel and the ground along the river banks and in floodplains were directly measured with the GNSS receiver attached to a survey rod. The surveys of the river banks and floodplains were conducted to ensure sufficient overlaps with the digital terrain model (DTM) provided by AEP.

The Trimble RTK GNSS receivers used for the survey can provide an accuracy of ±0.02 m under optimal operating conditions when the GNSS receiver is mounted to a tripod with a clear view of the sky and sufficient satellites to accurately establish the receiver position. Additional error may be introduced when the receiver is off-level, obstructed by nearby trees or vegetation, or the instrument height is incorrectly recorded. The expected accuracy of ground-based survey points is ±0.05 m, except in rare cases where points are surveyed in tree cover or near large vertical banks resulting in less than ideal satellite coverage. The digital echo sounder used for the boat-based surveys has an accuracy of ±0.01 m under optimal operating conditions. Due to the pitch and roll of the boat when in motion, the expected accuracy of the boat-based survey is ±0.07 m.

2.1.1 Coordinate System and Datum

Horizontal positions were referenced to the three-degree Transverse Mercator (3TM) projection with a central meridian of 117°W. The 3TM projection is part of the Canadian Spatial Reference System (CSRS) North American Datum of 1983 (NAD83), which is a three-dimensional grid on which the position of an object or feature can be precisely pinpointed. Orthometric heights are based on the Canadian Geodetic Vertical Datum of 1928 (CGVD28) and HTv2.0 hybrid geoid model.

2.1.2 Control Network

A control point (CP) network was established based on long-term GNSS observations and the CSRS Precise Point Positioning (CSRS-PPP) service provided by Natural Resources Canada (2017). Three Alberta

Survey Control Monuments (ASCM) were used in the CP network along with three semi-permanent benchmarks (SPBM) established by NHC for the survey program. The SPBM consisted of 0.9 m long rebar with an aluminum cap. A list of the CP coordinates is provided in **Table 1**.

The CP coordinates were determined by simultaneously logging static GNSS positions for at least one hour at two to four CPs. Static baselines were post-processed and control network adjustments were performed using Trimble Business Center software. The two CPs having the smallest reported CSRS-PPP coordinate error estimates were used to constrain and minimize the errors in the network adjustment.

Table 1 Control point summary

Point Name	Type	Easting (m)	Northing (m)	Elevation (m)
478024	ASCM	57807.748	6478607.833	267.487
571695	ASCM	58010.325	6474953.540	256.343
700088	ASCM	57730.038	6473124.713	279.219
NHC1	SPBM	64350.168	6478740.692	267.272
NHC2	SPBM	58777.035	6475529.360	255.392
NHC3	SPBM	50967.323	6476377.728	269.667

The horizontal and vertical errors in the control network after post-processing and adjustment to the reference ASCMs are summarized in **Table 2**. The largest horizontal error was 0.079 m and, the largest vertical error was 0.021 m.

Table 2 Control network errors

Point Name	Easting Error (m)	Northing Error (m)	Horizontal Error Magnitude (m)	Elevation Error (m)
571695	-0.020	0.005	0.021	0.021
NHC1	-0.060	0.052	0.079	0.000
NHC2	-0.057	-0.001	0.057	0.000
NHC3	-0.023	-0.048	0.053	0.004

2.2 Cross Sections

River cross section locations were selected to ensure accurate representation of the channel geometry in the hydraulic model with consideration of repeating cross section locations from the last flood risk mapping study (AENV, 2000). During the planning process for the survey, each cross section was assigned a planned cross section (PXS) identifier to organize the cross sections sequentially and to associate survey point data with a PXS. The PXS were established across the full channel width and passed through islands at several locations in the study reach. A L (left) or R (right)[1] identifier was attached to survey data measured in the secondary channel around the islands. The PXS survey point locations are shown in **Figure 3** where a river station (RS) number has been assigned to each PXS location. The RS represents distance measured in meters from the most downstream PXS (the downstream boundary of the study reach) along an established centerline for the Peace River (described in **Section 4.3.1**).

A total of 37 cross sections were surveyed. The average cross section spacing was 910 m. The minimum spacing was 42 m between the cross sections on either side of the Highway 88 bridge. The maximum spacing was 1,605 m between RS 26,255 m and RS 24,968 m. The survey point data were assembled and provided in the digital file submission. Details of the cross sections are provided in **Appendix A**.

2.3 Hydraulic Structures

The hydraulic structures measured as part of the survey program include one bridge and one culvert as listed in **Table 3**. The table also includes the corresponding Bridge File (BF) numbers of Alberta Transportation (AT). The locations of these two structures are shown in **Figure 3**. Survey data collected for the bridge included: span length; deck width; top of curb or solid guardrail elevation; low chord elevation; number, width, type, shape, and location of piers; top of deck elevation; and photographs of the bridge. Information collected for the culvert included: culvert type; shape; dimensions and length; entrance condition; upstream and downstream invert elevation; crest elevation; and photographs of the culvert. Survey data for these structures were assembled and provided in the digital file submission with details being provided in **Appendix B**.

Table 3 Hydraulic structure summary

River Station (m)	Bridge File Number	Description	Structure Type
23,609	74227	Highway 88 Bridge	Bridge
23,609	77452	Highway 88 Culvert	Culvert

2.4 Flood Control Structures

The provincial FHIP Guidelines describe flood control structures as “walls constructed to prevent water from rivers or lakes from flooding surrounding lands. Often flood control structures are earthen berms but can also be constructed of concrete and other materials.”

[1] Left and right banks or overbanks refer to the perspective of an observer looking downstream.

Dedicated flood control structures such as dikes typically require regulatory approval prior to construction, receive routine inspection and maintenance, and are officially recognized by AEP and local authorities as flood management infrastructure.

Some road and railway embankments or berms may perform as flood barriers and affect the river hydraulics but may not be classified as dedicated flood control structures. These types of infrastructure are classified as non-dedicated flood control structures. Railroad embankments are typically assumed to be permeable and are not considered natural ground features or dedicated flood control structures.

Based on the guidelines and the information available from AEP and local authorities, NHC has confirmed that there are no dedicated flood control structures within the study reach (NHC 2019a).

2.5 Other Survey Data

2.5.1 Discharge Measurements

Discharge measurements were conducted at three selected locations (**Figure 3**) during the survey to support calibration of the hydraulic model. The measurements were taken on 11 June 2019 between 10:00 and 15:00, using a boat-mounted Sontek M9 RiverSurveyor Acoustic Doppler Current Profiler (ADCP), which can measure water depths ranging from 0.06 m to 40 m and provide an accuracy of ±0.25% in velocity measurement. The discharge measurements followed the standard procedures of the Water Survey of Canada (WSC). The measured discharges are summarized in **Table 4**. Note that the measured discharges for PXS14 and PXS22 have been broken down by segments as they are divided by islands into main and side channels. Details of the discharge measurements are provided in **Appendix C**.

Table 4 Discharge measurement summary.

Planned Cross Section	River Station [1] (m)	Channel Segment	Discharge (m^3/s)	Water Elevation (m)
PXS3	2,234	Single Channel	1,824	246.875
PXS14	11,482	Main Channel	1,360	247.326
		Left Channel	271	247.354
		Right Channel	206	247.347
PXS22	18,416	Main Channel	1,633	247.628
		Left Channel	200	NA[2]

Notes:

1. Distance measured along the Peace River centreline from the downstream boundary of the study reach.
2. Surveyed water elevation was erroneous and discarded.

The measured total discharges at the three locations are consistent, with variation of 0.4% or less from the average of 1,830 m^3/s. Preliminary real-time flow data for Peace River at Fort Vermilion (WSC Station 07HF001) are published by AEP. According to the AEP data, the Peace River discharge on the 11 June

2019 between 10:00 and 15:00 varied between 1,828 m^3/s and 1,816 m^3/s. These data are consistent with the measured total discharges.

2.5.2 Site Photographs

Appendix D provides annotated reach representative photographs obtained during the site inspection and survey program. The time and other metadata information are imbedded in the electronic images.

2.6 Additional Base Data

2.6.1 Digital Terrain Model

Bare-earth and full-feature digital terrain model (DTM) data sets were provided by AEP in July 2019. The data were derived from a LiDAR survey conducted in October 2018. The bare-earth DTM was compared to ground survey points collected by NHC, and were found to be in good agreement.

2.6.2 Aerial Imagery

Aerial imagery was acquired for AEP by OGL Engineering Ltd. on 16 June 2019. AEP provided fully-processed orthophoto mosaics to NHC on 18 February 2020.

2.6.3 Structure Drawings

NHC obtained structural drawings for the Highway 88 bridge from AT. No structural drawings were available for the Highway 88 culvert, but AT provided inspection reports for the culvert that contain information required for modelling.

2.6.4 Hydrometric Gauging Station Information

Water level (stage) records, flow records, rating curves, and the station description for WSC Station 07HF001 – Peace River at Fort Vermilion were obtained to support hydraulic model calibration, open water flood hydrology assessment and ice jam analysis. Information and data for other WSC hydrometric stations were also obtained for the open water hydrology assessment (**Appendix E**) and ice jam analysis (**Section 5**).

2.6.5 Base Mapping Features

In addition to the datasets listed above, other base mapping data were obtained to support modelling and mapping for the study, including road network, hydrography, administrative boundaries, topographic maps and Alberta Township System (ATS) grids within the study area.

3 FLOOD HYDROLOGY

3.1 Flood History

3.1.1 General Information

Fort Vermilion has a history of flooding due to both ice jams and high Peace River flows under the open water conditions. A description of local flood history has been prepared to provide context for the hydraulic model creation and calibration. This flood history documentation summarizes information related to both open water and ice jam related flooding that has been documented and observed.

3.1.2 Open Water Floods

Open water floods in the study area can arise as a result of high flows originating from the W. A. C. Bennett Dam, extreme runoff from the Smoky River basin, or a combination of both. Annual peak discharges for Peace River at Fort Vermilion are most likely to occur in late May or June due to snowmelt with or without rain, but they can also occur in July and August. Floodwater at Fort Vermilion comes mostly from upstream of TPR. As described in **Appendix E**, local tributary inflows between TPR and Fort Vermilion appear to have negligible contribution to Peace River open water flood peaks.

The WSC gauge station on the Peace River at Fort Vermilion (WSC Station 07HF001) was initially established in August 1915. It provides discharge records for the periods from 1915 to 1922, 1961 to 1978 and 2006 to 2018. Only water levels (gauge heights) are available from this station for the 1979 – 1993 period. During these gauged periods, the 16 June 1990 flood was the largest open water event for Peace River at Fort Vermilion. The WSC gauge recorded a peak water level at El. 253.68 m. The peak instantaneous discharge was accordingly estimated as 13,800 m^3/s by NHC during this study, as discussed in **Section 4.1.5**. Highwater mark (HWM) elevations were surveyed and summarized in AENV (2000). Flooding at Fort Vermilion was minimal during this event. According to the AT Bridge File 74227, the highwater level at the Highway 88 bridge was about 11 m below the deck. This flood was the result of concurrent highwater events in the upper Peace and Smoky river basins. The Peace River flow peaked at TPR three days earlier on 13 June as a result of this same event, and the peak discharge was also the highest of the flow record for the TPR gauge station (WSC Station 07HA001).

The 13 July 2011 event was another high flow event on the Peace River at Fort Vermilion. The maximum water level at WSC Station 07HF001 was about El. 252.90 m according to the WSC gauge record, and the peak instantaneous discharge was 12,900 m^3/s. According to the information from the AT Bridge File 74227, the highwater level at the Highway 88 bridge was lower than the 1990 flood level. The same event resulted in a high peak discharge at TPR two days earlier.

The 16 June 1964, 13 July 1965 and 16 June 1972 floods were three other large events at Fort Vermilion with peak discharges slightly smaller than the 2011 flood peak: 12,600 m^3/s (estimated from the maximum daily discharge as described in **Appendix E**), 12,100 m^3/s and 11,200 m^3/s, respectively.

A few of other open water flood events on the Peace River at TPR were documented but not reported at Fort Vermilion, including the 1914, July 1935, and May 1948 events (NHC, 2017). The 1914 flood created problems for TPR (known as the Village of Peace River Crossing at that time). The July 1935 event was a severe flood with an estimated peak discharge just 8% smaller than the June 1990 flood peak at TPR. The May 1948 flood impacted some families in West Peace River and TPR. These floods might have resulted in noticeable high flows at Fort Vermilion, although there were no observations on record. Note that all these events occurred in the pre-regulation period.

3.1.3 Ice Jam Floods

Ice jam flooding on the Peace River can occur during freeze-up or breakup. At Fort Vermilion, freeze-up can occur any time between the middle of November and late December (Andres, 1996). Within this mild-sloped reach under the current regulated condition, river freeze-up has been characterized as an orderly progression of a stable ice cover starting from a single point located downstream of the study reach near Vermilion Chutes. Stable ice accumulations are formed by the juxtaposition of ice floes, and the rate of progression is driven by the velocity and concentration of surface ice floes. A juxtaposed ice cover is typically thin (between 0.5 and 1.0 m in thickness), and hence the freeze-up stage increase is typically low for this freeze-up condition. As a result, freeze-up ice jam flooding near Fort Vermilion is less common (or less severe) than in upper reaches (e.g. at TPR). Severe ice jams with the potential to cause flooding at Fort Vermilion are most likely to occur during breakup, which usually occurs in late April to mid-May (several weeks later than breakup at TPR). This also appears to be the case for the unregulated condition, because no information could be found on pre-regulation freeze-up but a number of breakup floods in the pre-regulation period were documented.

The documented breakup ice jam floods at Fort Vermilion include both historic and recorded events, as summarized in **Table 5**. Historic floods herein refer to major floods that occurred prior to the period of systematic hydrometric data collection.

Although WSC records at Fort Vermilion date back to 1915, there are significant gaps in the gauge data. Notwithstanding these gaps, archival sources such as journals and newspapers and information from local observers provide key details about the severity of historic ice jam floods that occurred in 1876, 1888, 1894, 1934, 1950 and 1963. Information for these historic events were obtained from AT's Bridge Files, AENV (1968), Thomson (1993) and Gerard and Karpuk (1979). During the gauged periods, the 1965, 1997, 2018, and 2020 events were recognized as major flood events.

Table 5 Historic and recorded ice jam floods at Fort Vermilion

Date	Details
May 1876	"The free traders living on the island in front of the fort were forced to evacuate and move their possessions to the roof of their house." (Thomson, 1993) The flood level at the Hudson's Bay Company (HBC) trading post (**Figure 4**) was estimated as El. 254.8 m based on Gerard and Karpuk (1979)[1].
7-9 May 1888	According to the 1888 flood documentation in AT Bridge File 74227, the flood was believed to be caused by ice jam 40 km (25 miles) downstream at Big Island, and it reached approximately the same, or higher, level as the 1934 flood. The flood level at the HBC post was estimated as El. 258.7 m based on Gerard and Karpuk (1979) [1].
1894	"An ice jam occurred near Fort Vermilion, but it does not appear any flooding resulted. Ice was forced up over the bank and water came within three feet of the top, but the ice began to move again before it could spill over." (Thomson, 1993) The flood level at the HBC post was estimated as El. 255.7 m based on Gerard and Karpuk (1979) [1].
22 April 1934	According to AENV (1968), it is the worst flood reported by long term residents who lived in Fort Vermilion since 1917. The flood was caused by an ice jam located 3.2 km below the town. It was estimated by the residents that there was 0.6 m and 1.8 m of water at the HBC post and near the present airport (Wop May Memorial Aerodrome), respectively. Previous reports have established the flood level of 256.95 m near the airport. The flood level at the HBC post was estimated as El. 258.4 m based on Gerard and Karpuk (1979) [1].
7 May 1950	The 1950 flood documentation in AT Bridge File 74227 contains photos only taken at Experimental Farm (**Figure 4**), showing a relatively high ice level and ice left on river banks. It is not expected that this flood had significant impacts. The flood level at the HBC post was estimated as El. 252.3 m based on Gerard and Karpuk (1979) [1].
23 May 1963	A resident remembered that the airport area road had two feet of water over it in 1963 or 1964 (AENV, 1968). AT Bridge File 74227 includes an estimated flood level of El. 254.20 m on the approach to south side of the ferry crossing. The flood level at the HBC post was estimated as El. 255.40 m based on Gerard and Karpuk (1979) [1].
8 May 1964	A resident remembered that the river reached bankfull in 1964 (AENV, 1968). AENV (1968) includes an estimated a flood level of El. 255.7 m at the airport area. This estimate is significantly higher than the estimate based on WSC gauge data. It is possible that the estimate of AENV (1968) was based on the information related to the 1963 flood instead (the resident interview stated that the flood was in 1963 or 1964). The maximum breakup level for 1964 was estimated as El. 248.70 m based on Gerard and Karpuk (1979) [1].

Table 5 Historic and recorded ice jam floods at Fort Vermilion (continued)

Date	Details
25 April 1997	AENV (2000) reported that the highest flood level near the HBC post was El. 254.03 m. No other information related to this event was found.
29 April 2018	The flood caused emergency evacuations, temporary closure of Highway 88 near the Peace River bridge. Nine homes in North Vermilion were flooded, and low lying areas near the airport were inundated.Flood depths on the order of 1.5 m were recorded at North Vermilion and flood levels came within 0.15 m of overtopping River Road in Fort Vermilion. The peak water level recorded at the WSC gauge was El. 255.79 m.
27 April 2020	An ice jam formed just downstream of Beaver Ranch, 44 km downstream of Fort Vermilion, and caused extensive flooding in North Vermilion, Fort Vermilion east of 50th street, the Fort Vermilion airport, and the south approach of the Hwy 88 bridge, resulting in its closure. The ice jam released in the afternoon of 28 April and water levels had fallen more than 2.5 m in Fort Vermilion by the following morning of 29 April. The peak water level recorded at the WSC gauge was El. 257.76 m.

Notes:

1. The ice jam flood level estimate has been converted into geodetic elevation from "stage above zero flow elevation" presented by Gerard and Karpuk (1979). See Table 6.

Gerard and Karpuk (1979) provides estimates of the historic ice jam flood levels based on archival sources such as journals and newspapers and descriptions and photographs provided by residents. To develop systematic estimation of the flood levels, they assumed a "perception stage" or a reference stage for each source of information, which represents the observer's probable elevation relative to a common datum. For example, the perception stage for flood levels from HBC (Hudson's Bay Company) Archives was chosen to be the level at which the island opposite Fort Vermilion begins to flood, because the camp of free traders was located on this island. The datum that Gerard and Karpuk (1979) referred all perception stages and flood levels to was a geodetic elevation 243.8 m, which was identified as the zero gauge reading elevation ("zero flow elevation") for the WSC Fort Vermilion station between 1964 and 1967. Gerard and Karpuk (1979) specified the location of their estimation at the HBC trading post, which is located near 45 Street by the Peace River, approximately 1.77 km downstream of the WSC gauge station. The estimated flood levels and source of information are summarized in **Table 6**. These estimates were used for the ice jam frequency analysis as described in **Section 5.2**. The uncertainty in these estimates is subject to determination of the perception stage and interpretation of archival and residents' information; and hence is expected to be much higher than the uncertainty in the flood levels determined from gauge records and highwater marks for recent events. Note that lower estimates for the 1934 and 1963 ice jam flood levels were provided by AENV (1968) at different locations as described in **Table 5**.

Table 6 Historic breakup ice jam flood levels at HBC post[1] estimated by Gerard and Karpuk (1979)

Year	Stage above Zero Flow Elevation[1] (m)	Geodetic Elevation (m)	Source of Information
1876	11.0	254.8	HBC Archives
1888	14.9	258.7	Ancestral communication
1894	11.9	255.7	HBC Archives
1934	14.6	258.4	Resident interview
1950	8.5	252.3	Photographs by an employee of the Experimental Farm
1963[3]	11.6	255.4	Resident interview

Notes:

1. The HBC post is located near 45 Street by the Peace River, approximately 1.77 km downstream of WSC Station 07HF001.
2. The zero flow elevation was identified by Gerard and Karpuk (1979) as El. 243.8 m.
3. The WSC operated the gauge 07HF001 in 1963 but the record was incomplete; so the 1963 event has been deemed as a historic event.

Based on the information summarized in **Table 5** and **Table 6**, the 1888 and 1934 events are the two largest ice jam flood events at Fort Vermilion. The recent 2020 event followed by the 1894 historic event are the next two largest ice jam floods at Fort Vermilion.

3.2 Open Water Flood Frequency Analysis

3.2.1 Flood Frequency Flow Estimates

An open water hydrology assessment for Peace River at Fort Vermilion (NHC, 2020) is presented in **Appendix E**. As required by the terms of reference for this study, the assessment provides open water flood frequency estimates at Fort Vermilion under an unregulated condition, for various return periods from 2 to 1000 years.

Peace River flows have been regulated since 1968 by the W. A. C. Bennett Dam. Located some 20 km downstream is a second smaller power generating facility – Peace Canyon Dam on the Peace River, which operates as a run-of-river facility and has little additional effect Peace River flows. In the open water hydrology assessment, flow naturalization was performed to remove flow regulation effects on Peace River annual peak discharges at Fort Vermilion from 1968 to 2018. Through a flow routing analysis from TPR to Fort Vermilion, natural annual peak discharges at Fort Vermilion for the pre-regulation period were also estimated for years when gauge data were not available at Fort Vermilion. These analysis resulted in a series of natural and naturalized peak instantaneous discharges for Peace River at Fort Vermilion from 1915 to 1931 and from 1958 to 2018. Frequency analysis was then performed on the instantaneous peak discharge series to provide flood frequency estimates for various return periods from 2 to 1000 years. The recommended flood frequency estimates for Peace River at Fort Vermilion (WSC Station 07HF001) are summarized in in **Table 7**. The estimates are based on a log-Pearson type III frequency curve. They are applicable through the entire study reach with no change in discharge at the confluence with the Boyer River.

Table 7 Naturalized flood frequency estimates for Peace River at Fort Vermilion

Return Period	Annual Probability of Exceedance (%)	Naturalized Peak Instantaneous Discharge (m^3/s)	
		Value	95% Confidence Limit
1,000	0.10	24,700	21,800 - 28,900
750	0.13	23,800	21,100 - 27,700
500	0.20	22,500	20,100 - 26,100
350	0.29	21,500	19,300 - 24,700
200	0.50	19,900	18,000 - 22,700
100	1.0	18,100	16,500 - 20,300
75	1.3	17,300	15,900 - 19,400
50	2.0	16,300	15,100 - 18,100
35	2.9	15,500	14,300 - 17,000
20	5.0	14,100	13,200 - 15,400
10	10	12,600	11,900 - 13,500
5	20	11,000	10,500 - 11,600
2	50	8,830	8,440 - 9,240

3.2.2 Comparison to Previous Studies

The current flood frequency estimates for Peace River at Fort Vermilion are compared to values from previous studies by AENV (2000 and 1968) in **Table 8**. The current estimates are significantly higher than those from AENV (2000). However, AENV (2000) did not consider effects of flow regulation on the Peace River, and the flood frequency estimates were stated to be preliminary. The objective of the flood frequency analysis by AENV (1968) was to provide flood peak estimates for a regulated flow condition based on limited pre-regulation (natural) flow records. The analysis simply assumed a 50% reduction on natural flood peaks to develop regulated flood frequency estimates. While the AENV (1968) study had a different objective from the current study (which is to develop naturalized flood frequency estimates), it provided a natural flood frequency curve for Peace River at Fort Vermilion based on the pre-regulation flow data for 1917-1922 and 1961-1967. The values from this curve are shown in **Table 8**. These estimates except the 2-year value are smaller than the current estimates but greater than the AENV (2000) values. Additional discussions related to this comparison are provided in **Appendix E**. The current estimates were based on more comprehensive analysis and longer gauge records. They are believed to be more reasonable.

Table 8 Comparison with previous flood frequency estimates for Peace River at Fort Vermilion

Return Period (Years)	Peak Instantaneous Discharge (m^3/s)		
	Present Study for Naturalized Flows	AENV (2000)	AENV (1968)
100	18,100	12,640	14,160
50	16,300	9,990	13,590
10	12,600	8,215	11,890
2	8,830	6,090	9,630

4 OPEN WATER HYDRAULIC MODELLING

4.1 Available Data

4.1.1 Digital Terrain Model

Bare-earth and full-feature digital terrain model (DTM) data sets were provided by AEP in July 2019. The data were derived from a LiDAR survey conducted in October 2018. The bare-earth DTM was used to extend cross sections for the hydraulic model to cover overbank flow areas.

4.1.2 Existing Models

A hydraulic model was developed as part of the previous flood risk mapping study completed for Fort Vermilion (AENV, 2000). The modelled Peace River reach extended from near the downstream boundary of the current study to an upstream location near RS 27,860 m (**Figure 3**). Model parameters from the 2000 study were referenced during the development of the hydraulic model for this study.

4.1.3 Highwater Marks

Highwater mark observations provide documentation of peak water levels at specific locations for historical floods. Highwater mark elevations observed at Fort Vermilion for the June 1990 open water flood were obtained from AEP. It is the only significant open water flood event with highwater mark observations at Fort Vermilion. **Figure 4** shows the locations of these observations. **Table 9** provides a summary of the surveyed elevations of these highwater marks and their locations with respect to river stationing established for the hydraulic model.

Table 9 Summary of highwater marks for 16 June 1990 flood

Location Name	Highwater Mark ID	River Station [1] (m)	Highwater Mark Elevation (m)
SE 24-108-14-W5 U/S FT. VERMILION RON LIZOTTE HOUSE	90-PeaVerm-1	27,826	254.492
SE 29-108-13-W5 FT. VERMILION BRIDGE CROSSING, HWY 67	90-PeaVerm-2	23,580	253.946
NE 20-108-13-W5 FT. VERMILION BRIDGE – BYPASS CHANNEL	90-PeaVerm-2A	23,647	254.236
NE 21-108-13-W5 U/S FT. VERMILION	90-PeaVerm-3	21,435	253.793
SE 23-108-13-W5 FT. VERMILION UPPER END	90-PeaVerm-4	17,946	253.720

Table 9 Summary of highwater marks for 16 June 1990 flood (continued)

Location Name	Highwater Mark ID	River Station [1] (m)	Highwater Mark Elevation (m)
SW 24-108-13-W5 FT. VERMILION TOWNSITE	90-PeaVerm-5	16,820	253.510
NE 24-108-13-W5 FT. VERMILION TOWNSITE	90-PeaVerm-6	16,349	253.523
SE 29-108-12-W5 D/S FT. VERMILION PAST LAGOON – VERMILION HOUSE	90-PeaVerm-8	14,120	253.230
NW 29-108-12-W5 D/S FT. VERMILION – FIRE BASE SIGN POST	90-PeaVerm-9	13,200	253.217
SW 28-108-12-W5 D/S FT. VERMILION – FIREFIGHTER CAMP	90-PeaVerm-10	11,916	253.194
NW 34-108-12-W5 D/S FT. VERMILION – OLD HOUSE SITE	90-PeaVerm-11	10,525	253.042
NW 32-108-12-W5 D/S FT. VERMILION – 28TH BASELINE	90-PeaVerm-12	-2,350	251.929

Notes:

1. Distance measured along the Peace River centreline from the downstream boundary of the study reach. Positive and negative values are referred to for locations upstream and downstream of the boundary, respectively.

4.1.4 Gauge Data and Rating Curves

The water level (stage) and discharge records and rating curves from the WSC hydrometric gauge station located on the Peace River at Fort Vermilion were obtained and used to support calibration of the hydraulic model. **Table 10** lists the periods in which Peace River discharges and water levels were measured at the WSC Fort Vermilion station.

Table 10 List of hydrometric gauging stations supporting model creation and calibration

Station Name (ID)	Period of Record
Peace River at Fort Vermilion (07HF001)	Discharge: 1915-1922, 1961-1978, 2006-2018
	Water level: 1979-1993, 2012-2018

4.1.5 Peak Discharge Estimate for June 1990 Flood

The 16 June 1990 flood was used as the high-flow calibration event as it is the largest open water flood event on record and is the only significant event at Fort Vermilion with highwater mark observations over a significant portion of the study reach.

The WSC only reported water levels for Peace River at Fort Vermilion from 1979 to 1993. Discharge measurements were not available for the June 1990 flood. AENV (2000) estimated the instantaneous peak discharge for this event as 12,640 m^3/s based on a water level of 253.403 m, which was claimed as the maximum instantaneous water level. The peak water level and discharge values reported by AENV (2000) appear to be incorrect according to the following observations and analysis:

- The 13 July 2011 event was another high flow event on the Peace River at Fort Vermilion. The maximum water level at WSC Station 07HF001 is about El. 252.90 m for this event according to the WSC 15-minute gauge record, and the published peak instantaneous discharge is 12,900 m^3/s. Comparison with the June 1990 flood level and discharge reported by AENV (2000) shows that the reported 1990 peak flood level is about 0.5 m higher but the estimated peak discharge is lower, which suggests that the discharge value from AENV (2000) is unreasonable.
- The peak flood level of June 1990 published by the WSC is at El. 253.68 m, which is 0.28 m higher than the water level reported by AENV (2000). The WSC flood level is consistent with the surveyed highwater mark elevations (**Table 9**) as the gauge station is located between 90-PeaVerm-4 (El. 253.72 m) and 90-PeaVerm-5 (El. 253.51 m). As described in **Appendix E** (Open Water Hydrology Assessment Report), NHC estimated the June 1990 flood peak instantaneous discharges as 13,800 m^3/s based on the WSC published water level data and rating curve (WSC Rating Table No. 9 dated 11 February 1975). This estimate is similar to the value noted on Figure 1A of AENV (2000): 13,450 m^3/s from the rating curve based on the WSC measured flood level.
- The June 1990 flood peak discharge at Fort Vermilion was also estimated by routing the Peace River flows from TPR to Fort Vermilion using the model developed for the open water hydrology assessment (**Appendix E**). As described in **Appendix E**:
    - the daily flow hydrograph recorded at TPR was routed,
    - a multiplier of 1.04 was applied to the routed peak daily discharge to account for tributary inflows, and
    - a multiplier of 1.02 was used to convert the daily value into the instantaneous value.

  This resulted in a peak instantaneous discharge estimate of 13,720 m^3/s for Fort Vermilion, which is close to the above mentioned estimate based on the WSC water level data and rating curve.

Therefore, the current study has adopted the estimate of 13,800 m^3/s based on the WSC water level data and rating curve as the peak instantaneous peak discharge for the 16 June 1990 flood at Fort Vermilion.

4.1.6 Flood Photography

Flood photography was obtained from several sources to assist with development of the hydraulic model. Selected photographs for both open water and ice jam floods are provided in **Appendix F.** The photographs were obtained from flood documentation reports from the AT bridge file for the Highway 88 bridge and from Mackenzie County from the ice jam flood in April 2018. The open water events documented by the photographs occurred in 1990 and 2011.

4.2 River and Valley Features

4.2.1 General Description

The Peace River generally flows through the Fort Vermilion study area from the southwest to the northeast. The river is partly entrenched and confined within a valley of about 30 m deep. The river valley width varies from about 2.0 km at the top to 1.6 km at the bottom. The terrain surrounding the valley is a mixture of agricultural development and sparsely forested lowlands. The valley bottom consists of two fragmented terraces with the lowest terrace being the floodplain of the channel (Kellerhals et al, 1972). Further details regarding the river channel and valley features are provided below.

4.2.2 Channel Characteristics

Kellerhals et al. (1972) describe the Peace River near Fort Vermilion as having an irregular and split channel pattern with developed point bars and islands. The sinuosity is 1.3. The river bed is comprised of predominantly sand (D_{50} = 0.31 mm) with local deposits of gravel. The banks are composed of gravels overlain by silt. The valley walls are composed of easily erodible rock and are generally bare or covered in shrub vegetation. Downstream from Fort Vermilion, the longitudinal slope of the river bed is about 0.00008 (0.08 m/km).

Several large, vegetated islands are located within the study reach, resulting in the river channel being split into two or three subchannels with comparable conveyance capacities. Otherwise, the channel is about 650 m wide on average.

4.2.3 Floodplain Characteristics

The floodplain of the Peace River at Fort Vermilion break into discreet portions as the channel moves between either toe of the valley slope (Kellerhals et al, 1972). The landcover of the floodplain is a mixture of boreal forest and agricultural development. Fort Vermilion is located on the southern floodplain of the river, while North Vermilion is located on the northern floodplain.

4.2.4 Anthropogenic Features

Descriptions of the anthropogenic features and the river stationing located within the study area are summarized in **Table 11**. Fort Vermilion is located in the study area along with the North Vermilion and the Fort Vermilion Indian Reserve #173b. The Wop May Memorial Aerodrome is located on the south side of the river downstream of the Fort Vermilion (**Figure 1**). The study area also contains one bridge and one culvert. The descriptions and locations of the hydraulic structures relative to the river stationing in the hydraulic model are provided in. Detailed information for the bridge and culvert is discussed in **Section 2.3**.

Table 11 Anthropogenic features within the study area

Description	Upstream River Station (m)	Downstream River Station (m)
Highway 88 Bridge (BF74227)	23,609	23,609
Highway 88 Culvert (BF 77452)	23,609	23,609
North Vermilion	19,950	15,450
Fort Vermilion	18,100	13,910
Fort Vermilion Indian Reserve #173b	16,730	15,770
Wop May Memorial Aerodrome	13,170	11,000

Notes:

1. River Station is the distance measured along the Peace River centreline from the downstream boundary of the study reach.

4.3 Model Construction

4.3.1 Methodology

The U.S. Army Corps of Engineer's *Hydrologic Engineering Center River Analysis System* (HEC-RAS) computer program (Version 5.0.6, November 2018) was used to perform hydraulic modelling for this study. The basic inputs required by *HEC-RAS* are cross sections spaced throughout the study reach that represent the geometry of river channel and floodplain, roughness coefficients for the channel and overbank areas at each cross section, a specified water level or slope at the downstream boundary, and an inflow discharge at the upstream boundary.

HEC-RAS can perform one-dimensional (1D), two-dimensional (2D), or combined 1D and 2D hydraulic calculations for a network of channels with or without hydraulic structures. For this study, a 1D model was constructed to compute water surface profiles for steady-state flows. The computational procedure for steady flow calculations is based on the solution of the 1D energy equation. Energy losses due to friction and expansion/contraction between cross sections are calculated. The analytical approach employed by HEC-RAS has the following assumptions and potential limitations:

- Flow is gradually varied and boundary friction losses between cross sections are estimated by Manning's equation using section-average parameters.

- The geometry is assumed to be fixed; therefore, changes in the channel and floodplain geometry (e.g. erosion or scour) that may occur during a flood are not accounted for.
- Each model cross section is apportioned into three separate conveyance components representing the main channel, left overbank, and right overbank; and the water level is assumed to be constant across all three conveyance components.
- The flow is one-dimensional; therefore only the velocity component in the principal direction of flow is accounted for in the model.

Geometric Layout

The following describes the approach for developing the key components comprising the geometric layout of the model.

- A single continuous channel centreline was created along the middle of the main channel of the Peace River to represent the study reach. The main channel was defined around islands as the channel with the largest cross sectional area. The centreline was digitized using GIS tools and visual interpretation of the DTM and aerial imagery.
- Model cross section lines were drawn at each surveyed river cross section. Initially, the cross sections were aligned perpendicular to the principle flow direction at channel survey locations. The elevation of each survey point at a cross section was projected onto the cross section line. The cross sections were then extended into the left and right overbanks and beyond the expected 1000-year flood inundation extents. Elevation data in the overbank areas was extracted from the supplied DTM along the cross section alignment.
- The locations of the left and right banks, or bank stations, were determined from the cross section survey data and examination of the DTM. The bank stations demarcate the extent of the left overbank, main channel, and right overbank portions of each model cross section.
- Overbank and main channel flow path lines were digitized to represent the lengths of the flow path in the main channel and left and right overbanks. These lengths were used to compute energy losses between cross sections through each of the three conveyance components.

Manning's Roughness Coefficient

HEC-RAS calculates energy loss due to friction on the basis of Manning's equation, which requires a roughness coefficient to be assigned to each of the three conveyance components (the main channel and left and right overbanks) for every cross section. The Manning's roughness coefficient depends primarily on bed material type and size, and land cover (e.g. vegetation and development) for overbanks. Selection of the coefficient value for a 1D river model also requires consideration of the river morphology. In addition, the Manning's roughness coefficient can vary with discharge as contributions of the abovementioned factors may decrease as flow increases. This trend is often more prominent for sand-bed rivers due to changes in bed forms (dunes and ripples) and size of deposited sediment particles. For the Peace River at Fort Vermilion, Manning's roughness coefficient varies from 0.030 to 0.017 for open water discharges between 1,000 and 10,000 m^3/s according to Trillium et al (1996).

Expansion and Contraction Coefficients

River channels generally have varying cross sections, which results in changes in velocity head (kinetic energy) from section to section, accompanied by an energy loss. This type of energy loss is accounted for in HEC-RAS using expansion and contraction coefficients for each cross section multiplied by the difference in velocity head between that cross section and the next one downstream. Expansion and contraction coefficients for model cross sections are usually kept at the default values of 0.1 and 0.3, respectively, which are typical values for gradual transitions in subcritical flow. Higher values are required for abrupt transitions. Expansion and contraction coefficients for supercritical flow are typically lower than for subcritical flow USACE (2016).

Boundary Conditions

Boundary conditions are required at the inflow (upstream) and outflow (downstream) boundaries of the model. The inflow boundary condition is the discharge measured at WSC gauge 07HF001 (Peace River at Fort Vermilion). The outflow boundary condition is a water level or a friction slope with which the water level will be calculated by HEC-RAS.

Ineffective Flow Areas

Ineffective flow areas can be specified within portions of cross sections where water will pond but the water velocity in the downstream direction will be close or equal to zero (i.e. water will not be actively conveyed). One common example of using ineffective flow area is in cross sections upstream and downstream of a bridge or culvert where flow is obstructed by elevated road embankments. In HEC-RAS, ineffective flow areas can be defined as either a permanent or non-permanent type. Permanent ineffective flow areas stay ineffective regardless of the water surface elevation, whereas temporary ineffective flow areas become effective when water surface elevation exceeds a defined elevation. The configuration of ineffective flow areas depends on site-specific circumstances and engineering judgement.

4.3.2 Geometric Database

The geometric database consists of a geodatabase and ArcMap project file that contains the components of the HEC-RAS model geometry. The information includes points, polylines, and polygons that represent model cross sections, reach lengths, channel and overbank centerlines, bank stations and banklines, structures such as bridges and culverts, etc.. The geometric database for this study is provided as part of the electronic deliverables. The following sections describe its components and associated methods of development.

Cross Section Data

A total of 37 cross sections were created and used to construct the Peace River model for this study. The steps taken to generate the cross section data were as follows:

1) Cross section alignments within the channel were established generally following the alignments of the cross section survey (**Section 2.2**). The overbank portions were aligned perpendicular to the anticipated flow direction. The cross section alignments were extended beyond the anticipated 1000-year flood inundation extents.
2) Two separate station-elevation data sets were created for each cross section.
   a. The first data set was created by projecting surveyed data points perpendicularly onto the channel portion of the cross section line.
   b. The second data set was created by extracting elevation values from the DTM along the cross section lines excluding the channel portion covered by the survey data.
3) The two station-elevation data sets were combined. For each cross section, the number of elevation points for the overbanks were reduced using the minimize-area-change point filter option in HEC-RAS, so that the total number of the points is within the HEC-RAS limit of 500 points.
4) Distances between consecutive cross sections were established within the HEC-RAS model following the established channel centerline and central flow paths for the left and right overbank areas.

Cross section details based on NHC's surveys are provided in **Table 12**.

Table 12 Model cross section details

Cross Section	River Station (m)	Thalweg Elevation (m)	Channel Width (m)	Notes
PXS37	32,785	242.15	1231	Upstream boundary
PXS36	31,762	238.79	455	
PXS35	30,910	238.85	510	
PXS34	29,866	240.85	531	
PXS33	29,050	242.52	805	
PXS32	27,860	240.35	1615	
PXS31	26,255	240.02	1440	
PXS30	24,968	240.18	974	
PXS29	24,113	238.71	588	
PXS28	23,634	240.11	481	Upstream of Hwy 88 bridge

Table 12 Model cross section details (continued)

Cross Section	River Station (m)	Thalweg Elevation (m)	Channel Width (m)	Notes
PXS27	23,591	240.06	505	Downstream of Hwy 88 bridge
PXS26	22,698	239.63	530	
PXS25	21,545	240.24	606	
PXS24	20,409	239.73	1051	
PXS23	19,433	238.71	1484	
PXS22	18,416	237.46	1343	
PXS21	17,365	238.25	1339	WSC Station 07HF001
PXS20	16,428	238.27	914	
PXS19	15,592	238.43	453	
PXS18	14,680	240.27	470	
PXS17	13,690	241.58	740	
PXS16	12,889	242.75	1177	
PXS15	11,953	241.20	2160	
PXS14	11,482	241.19	2986	
PXS13	11,025	240.41	3678	
PXS12	10,282	240.34	2972	
PXS11	9,512	237.94	2541	
PXS10	8,634	240.01	2291	
PXS9	7,958	241.18	2100	
PXS8	7,256	238.99	1663	
PXS7	6,402	239.90	1388	
PXS6	5,260	237.42	947	
PXS5	4,192	237.84	908	
PXS4	3,352	236.63	578	
PXS3	2,234	238.96	448	
PXS2	1,117	239.89	646	
PXS1	0	240.63	701	Downstream boundary

Hydraulic Structures

The modelled reach contains one bridge and one culvert as listed in **Table 3**. Both structures are part of the Highway 88 crossing. They were simulated as a single hydraulic structure with a bridge and a culvert opening. Their alignments and locations were established based on the survey data and available design drawings. Key design information of these two structures that was incorporated into the model is tabulated in **Appendix B**. The key components that define the bridge in the model are the abutments, high and low chords defining the bridge deck and the superstructure, and the arrangement, shape, and dimensions of the piers. The key components that define the culvert in the model are the length of the barrel, the shape including span and rise, and the upstream and downstream invert elevations. The cross section line that defined the centreline of this highway crossing was extended to include the approach roadway on both banks using DTM data along the road centreline.

Other Features

The water level along the Boyer River near the confluence with the Peace River was determine by assuming a horizontal condition from the Peace River. Inundation extents were mapped using the DTM upstream to the study reach boundary.

4.3.3 Model Calibration

Model calibration involves the selection of modelling parameters to simulate observed water levels along the study reach for both high and low flow conditions. The modelling parameters that affect model calibration include:

- Manning's roughness coefficient of the channel, islands, and floodplain,
- friction slope used to determine water level at the downstream boundary,
- ineffective flow areas at each cross section, and
- expansion and contraction coefficients between cross sections.

Of the above, the primary modelling calibration parameter for this study was the Manning's roughness coefficient for the river channel. The value for each cross section was adjusted until the simulated water surface profile elevations agreed with observed water levels and highwater marks.

The calibration of a hydraulic model is typically subject to:

- the accuracy of the water level and highwater mark measurements,
- uncertainties in discharge measurements or estimates associated with the water levels or highwater mark elevations that the model is calibrated against, and
- limited channel geometry data.

For this study, the hydraulic model was calibrated for low and high flow conditions using available data including: surveyed water levels and discharge, highwater marks from the June 1990 open water flood, and rating curves for Peace River at Fort Vermilion (WSC Station 07HF001) from WSC.

Low Flow Calibration

Water levels were measured during the bathymetric survey in June 2019 when the Peace River discharge was relatively low. The river discharge along the study reach was measured as about 1,830 m^3/s (**Section 2.5.1**). These data were used to facilitate the low flow calibration. Manning's roughness coefficient for the channel was adjusted until the simulated water surface profile fitted the surveyed water levels as shown in **Figure 5**. The simulated water surface elevations at the survey locations are also compared with the survey data in **Table 13**. The differences, presented as simulated water level minus surveyed level, range from -0.47 m to 0.16 m, with an average of about -0.10 m. The most significant difference (-0.47 m) occurred at the cross section immediately upstream of the Highway 88 bridge (River Station 23,634 m). The survey data show a water level drop of 0.23 m across the bridge, which is relatively large for the relatively small discharge. In comparison with the high flow calibration results discussed in the next section, the differences between the simulated and surveyed water levels for this low flow calibration are more significant. Note that the discharge for this calibration is much lower than the 2-year flood peak. The current model is a single-channel model. Using a network model (composed of separate side and main channels) and additional cross sections throughout the study reach would be required to achieve better calibration results for such a low flow; however, it would unlikely improve the channel roughness estimation, and is unnecessary for the current study, of which the modelling objective is to simulate flows at much larger magnitudes.

The low flow calibration (with a discharge of 1,830 m^3/s) resulted in a river channel Manning's roughness coefficient of 0.029, which is within the range (from 0.030 to 0.017 for discharges between 1,000 and 10,000 m^3/s) determined by Trillium et al (1996) for the Peace River. The flow was contained within the channel throughout the entire study reach. So, the roughness values for overbank areas had no effects on this calibration.

Table 13 Channel Manning's roughness coefficient calibration results - low flow conditions

River Station (m)	Measured Elevation (m)	Simulated Elevation (m)	Simulated minus Observed (m)
23,634	248.65	248.18	-0.47
23,591	248.42	248.17	-0.25
14,680	247.52	247.68	0.16
11,482	247.33	247.48	0.15
7,958	247.15	247.23	0.08
2,234	246.88	246.74	-0.13
0	246.79	246.58	-0.21

High Flow Calibration

The 16 June 1990 flood on the Peace River at Fort Vermilion was the largest open water flood event on record. Survey data for highwater marks are available through the majority of the study reach and are summarized in **Section 4.1.3**. The flood peak discharge was estimated as 13,800 m^3/s (see **Section 4.1.5**). This flood was used as the high-flow calibration event. In this calibration, Manning's roughness values for overbank areas were estimated as 0.08 and 0.06 for heavy and light vegetation conditions respectively, as described in **Section 4.3.4**. The roughness value for the river channel was adjusted until the simulated water surface profile elevations agreed with the surveyed highwater mark elevations. The simulated water surface profile is compared with the surveyed highwater mark elevations in **Figure 5** and **Table 14**. The differences between the simulated water surface profile and highwater mark elevations (presented as simulated level minus highwater mark elevation) range from -0.12 m to 0.03 m with an average of -0.02 m.

Table 14 Channel Manning's roughness coefficient calibration results - high flow conditions

Highwater Mark ID	River Station (m)	Observed Elevation (m)	Simulated Elevation (m)	Simulated minus Observed (m)
90-PeaVerm-1	27,826	254.49	254.51	0.01
90-PeaVerm-2	23,580	253.95	254.26	0.02
90-PeaVerm-2A	23,647[1]	254.24	253.94	-0.01
90-PeaVerm-3	21,435	253.79	253.80	0.01
90-PeaVerm-4	17,946	253.72	253.60	-0.12
90-PeaVerm-5	16,820	253.51	253.52	0.01
90-PeaVerm-6	16,349	253.52	253.46	-0.07
90-PeaVerm-8	14,120	253.23	253.20	-0.03
90-PeaVerm-9	13,200	253.22	253.21	-0.01
90-PeaVerm-10	11,916	253.19	253.18	-0.02
90-PeaVerm-11	10,525	253.04	253.07	0.03

Notes:

1. This highwater mark is representative of the river flood level at River Station 23,674 m, instead of the level for the nearby model cross section at River Station 23,634 m.

Note that, in this calibration, the highwater mark elevation of 90-PeaVerm-2A was compared with the computed flood level for the upstream cross section PXS30 (RS 24,968 m – see **Figure 4**) instead of the nearby PXS28 (RS 23,634 m). This highwater mark was located in the overflow channel upstream of the culvert under the south approach to the Highway 88 bridge. During the June 1990 flood, the overflow channel was connected with the river only through its inlet located near RS 23,634 m and outlet located downstream of the bridge. The flow in the overflow channel was controlled by the culvert, which is an elliptical pipe with a 1.90 m rise and 1.72 m span, with the upstream invert at El. 249.18 m. The culvert was deeply submerged during the flood. Under this condition, the water level at the culvert inlet would be close to the water level at the channel entrance. Therefore, the highwater mark should reflect the river flood level at RS 24,968 m.

The calibrated Manning's roughness coefficient for the channel is 0.0172. This value is consistent with the range (from 0.030 to 0.017 for discharges between 1,000 and 10,000 m^3/s) of Manning's roughness coefficients for the Peace River identified by Trillium (1996).

The current Manning's roughness value is slightly smaller than the values adopted in the previous study by AENV (2000), which varied from 0.018 to 0.019. The previous study also used the highwater mark elevations for the 1990 flood event to calibrate the Manning's roughness coefficient; however, the calibration was based on a smaller peak discharge estimate of values from 12,640 m^3/s, which appears to be incorrect as discussed in **Section 4.1.5**.

Calibration against WSC Gauge Rating Curve

WSC Station 07HF001 – Peace River at Fort Vermilion is located on the right bank of the river in the study reach, near the surveyed cross section PXS21 (RS 17,365 m). An open water stage-discharge rating curve for the gauge has been developed and periodically updated by the WSC based on field measurements. This information was used to determine the variation of Manning's roughness coefficient with discharge.

The WSC current rating curve (WSC Rating Table No. 14, updated on 23 May 2018) and field measurements for open water flows are shown in **Figure 6**. The field measurements were provided by the WSC as two separate data sets with one taken between 2006 and 2019 and the other between 1915 and 1990. **Figure 6** includes both data sets excluding the old measurements taken between 1915 and 1920 because they are inconsistent with the plotted data. The highest discharge among the recent 2006-2019 measurements was 9,250 m^3/s on 15 July 2011, followed by 6,420 m^3/s measured on 30 June 2011. Although the WSC indicated in their field activity report that these two measurements should not be used for curve building due to poor data quality, the current rating curve appears to fit the 15 July 2011 measurement (with the measured discharge of 9,250 m^3/s). The measurements prior to 1990 appear to be more scattered, but they are reasonably represented by the rating curve. **Figure 6** also includes a data point representing the June 1990 flood peak. As described in **Section 4.1.5**, the peak discharge for this flood was estimated from the WSC's 1975 rating curve and validated by a routing analysis. The current rating curve would predict the peak discharge of this event to be about 10% higher than the adopted estimate (13,800 m^3/s). Overall, the current rating curve represents all data points in **Figure 6** reasonably well; but it does not appear to be the best-fit. Therefore, the goal of the following model calibration was to produce a similar rating curve that can provide a good fit for all of the measured and estimated discharges, rather than to replicate the current WSC rating curve.

As discussed in the previous sections, Manning's roughness coefficient for the Peace River at Fort Vermilion increases with decreasing discharge. HEC-RAS allows the user to input "flow roughness factors" to model this variation. In this study, it was assumed that Manning's roughness coefficient changes from 0.017 at the discharge 13,800 m^3/s (the high flow calibration result) to 0.029 at the discharge 1,830 m^3/s (the low flow calibration result) following an exponential function, and it remains constant for discharges outside of this range. The resulting flow roughness factors input to the model are summarized in **Table 16** in **Section 4.3.4**. The model was then used to compute a rating curve at the WSC station (i.e. RS 17,365 m) for discharges ranging from 665 m^3/s to 13,800 m^3/s. The simulated rating curve is shown in **Figure 6.** It is similar to the current WSC rating curve for discharges up to 6,000 m^3/s. Beyond that, the simulated rating curve tends to predict higher water levels than the WSC rating curve. Overall, the simulated rating curve appears to fit the plotted data points better than the WSC rating curve. As such, the flow roughness factors based on the assumed exponential function were adopted.

4.3.4 Model Parameters and Options

The following sections describe the key model parameters and options adopted in the calibrated open water HEC-RAS model. These include Manning's roughness coefficients for the channel and overbank areas, contraction and expansion loss coefficients, boundary conditions, ineffective flow areas, and geometric configuration around flow splits and islands.

Manning's Roughness Coefficient

The type of land cover was used to help characterize roughness in the floodplain areas and along islands. Manning's roughness coefficients in the overbank and on islands in the main channel were defined based on landcover composition, professional judgement, and guidance from literature (e.g. Chow, 1959). **Table 15** presents the adopted roughness coefficients for the land cover types identified within the study area. The majority of floodplain throughout the study area was defined as densely vegetated.

Table 15 Adopted Manning roughness coefficients for floodplain areas

Land Cover Type	Manning's Roughness Coefficient
Lightly vegetated area	0.060
Densely vegetated area	0.080
Developed area	0.060

The Manning's roughness coefficient in the model was varied horizontally in accordance with spatial variation of land cover across each cross section in the overbanks and on islands within the main channel. The spatial variation between cross sections was also considered when determining the horizontal location of roughness coefficient change in the model.

The Manning's roughness coefficient in the channel or the wetted portion of each cross section was determined through the model calibration process described in **Section 4.3.3**. The calibrated coefficient varies from 0.029 at the low-flow discharge 1,830 m^3/s to 0.017 at the high-flow discharge 13,800 m^3/s. The model calibration also indicates that the variation of the Manning's roughness coefficient between the two discharges could be reasonably represented with an exponential relationship, and the resulting channel roughness values are listed in **Table 16**. Flow roughness factors in HEC-RAS were used to model this variation. As shown in **Table 16**, the roughness coefficient from the high-flow calibration was chosen as the base value with a flow roughness factor of 1.00, flow roughness factors for other discharges were then determined by prorating the Manning roughness coefficient values. It was assumed that the roughness coefficient remains constant for discharges beyond the range of the calibration discharges, i.e. 0.029 for all discharges smaller than 1,830 m^3/s and 0.017 for all discharges greater than 13,800 m^3/s. Note that, with no further reduction in the Manning's roughness coefficient for higher discharges, the model tends to predict conservatively high flood levels.

Table 16 Adopted flow roughness factors and channel Manning's roughness coefficient from model calibration

Discharge (m^3/s)	Flow Roughness Factor	Channel Manning's Roughness Coefficient
1,000	1.70	0.029
1,830[1]	1.70	0.029
2,600	1.65	0.028
3,400	1.59	0.027
4,150	1.54	0.026
5,150	1.47	0.025
6,050	1.41	0.024
7,000	1.35	0.023
8,000	1.30	0.022
9,050	1.24	0.021
10,100	1.18	0.020
11,300	1.12	0.019
12,500	1.06	0.018
13,800[2]	1.00	0.017
24,700	1.00	0.017

Notes:

1. Discharge for the low-flow calibration
2. Discharge for the high-flow calibration

Expansion and Contraction Coefficients

The default values of 0.1 and 0.3 for expansion and contraction, respectively, were applied at each cross section throughout the hydraulic model except immediately adjacent to the Highway 88 bridge. The expansion and contraction coefficients were increased to 0.3 and 0.5, respectively, for the cross sections at RS 23,634 m and RS 23,591 m to account for the mild obstruction of flow area due to the Highway 88 bridge.

Boundary Conditions

A normal depth approximation was assigned to the Peace River model downstream boundary. With this type of boundary condition, HEC-RAS will calculate a water level at the boundary for each input discharge, based on the Manning's equation and a user-entered slope. For the current study, the entered slope for the downstream boundary was selected to be 0.000084 m/m, which is the average slope between Fort Vermilion and Peace Point (excluding the Vermilion Chutes) identified by Kellerhals et al. (1972).

Ineffective Flow Areas

The study area was reviewed to identify obstructions to flow not captured by the model cross sections and no significant obstructions were found that would warrant the use of ineffective flows areas in the model. In particular, at the Highway 88 bridge, the bridges approaches are captured in the bounding cross sections; so use of ineffective flow area was not required.

Flow Splits and Islands

The study reach could be adequately represented without flow splits around islands. As such, the model was configured as a single-channel model. When a cross section intersects an island, the model assumes equal water level on both sides of the island based on the composite channel conveyance properties and computed energy losses. As flood magnitude increases or where the effective flow path distance between cross sections on either side of an island are similar, the approximation is generally accurate. As discussed in the model calibration section, with the single-channel configuration, the model may underperform in simulation of low flows; but it is adequate for the higher flows that the current study focuses on. This configuration also has the advantage that the same model geometry can be used for ice jam modelling (**Section 5**).

4.4 Open Water Flood Frequency Profiles

The calibrated hydraulic model was used to generate flood frequency profiles for the thirteen open water floods corresponding to various return periods listed in **Table 7**. The computed flood frequency water levels at each cross section are provided in **Appendix G**. The flood frequency profiles are plotted in **Figure 7**.

4.5 Model Sensitivity

Uncertainties in boundary conditions and the Manning's roughness coefficient in the hydraulic model could affect computed water levels, and consequently flood depths and inundation extents. The sensitivity of computed water levels to these parameters were evaluated to gain an indication of the plausible range of modelling errors and to identify the relative sensitivity to each parameter. The 100-year flood was used as the baseline for this sensitivity analysis. A summary of the sensitivity analysis results is provided in the following sections.

4.5.1 Boundary Conditions

The boundary conditions for the HEC-RAS model include a discharge specified at the upstream boundary and a slope at the downstream boundary used by the model to compute the normal depth corresponding to the discharge. Sensitivity analysis was conducted on both boundary conditions.

Upstream Boundary – Discharge

The discharges selected for the sensitivity analysis were corresponding to the lower and upper 95% confidence limits of the 100-year flood peak, which are 16,500 m^3/s and 20,300 m^3/s respectively (**Table 7**). The computed water levels for these discharges are compared with the baseline model results in **Figure 8** and **Table 17**. The comparison indicates that a reduction of 9% in the discharge resulted in the computed 100-year flood levels being 0.54 m lower on average, and an increase of 12% in the discharge resulted in the flood levels being 0.70 m higher on average. The changes in computed water levels are relatively uniform throughout the study reach.

Table 17 Results of sensitivity analysis on inflow discharge

Cross Section	River Station (m)	Flood Levels (m) for 100-year Discharge		
		Lower Limit 16,500 m^3/s	Baseline 18,100 m^3/s	Upper Limit 20,300 m^3/s
PXS37	32,785	255.79	256.36	257.09
PXS36	31,762	255.65	256.20	256.91
PXS35	30,910	255.56	256.09	256.79
PXS34	29,866	255.46	256.00	256.69
PXS33	29,050	255.51	256.05	256.75
PXS32	27,860	255.50	256.05	256.77
PXS31	26,255	255.31	255.87	256.58
PXS30	24,968	255.24	255.79	256.51
PXS29	24,113	255.03	255.57	256.26
PXS28	23,634	254.87	255.39	256.07
PXS27	23,591	254.83	255.35	256.01
PXS26	22,698	254.77	255.29	255.95
PXS25	21,545	254.74	255.25	255.92
PXS24	20,409	254.79	255.31	255.99
PXS23	19,433	254.75	255.28	255.97
PXS22	18,416	254.59	255.12	255.82
PXS21	17,365	254.54	255.07	255.76
PXS20	16,428	254.45	254.98	255.67
PXS19	15,592	254.16	254.67	255.34
PXS18	14,680	254.12	254.63	255.30
PXS17	13,690	254.14	254.65	255.33
PXS16	12,889	254.15	254.67	255.36
PXS15	11,953	254.14	254.68	255.37
PXS14	11,482	254.13	254.67	255.36

Table 17 Results of sensitivity analysis on inflow discharge (continued)

Cross Section	River Station (m)	Flood Levels (m) for 100-year Discharge		
		Lower Limit 16,500 m^3/s	Cross Section	River Station (m)
PXS13	11,025	254.10	254.64	255.34
PXS12	10,282	254.03	254.57	255.28
PXS11	9,512	253.89	254.45	255.16
PXS10	8,634	253.82	254.38	255.10
PXS9	7,958	253.79	254.35	255.06
PXS8	7,256	253.73	254.28	254.99
PXS7	6,402	253.69	254.24	254.94
PXS6	5,260	253.55	254.10	254.80
PXS5	4,192	253.44	253.99	254.69
PXS4	3,352	253.22	253.76	254.47
PXS3	2,234	253.02	253.55	254.25
PXS2	1,117	252.98	253.51	254.20
PXS1	0	252.91	253.45	254.14
Minimum Deviation		-0.51	-	0.66
Average Deviation		-0.54	-	0.70
Maximum Deviation		-0.57	-	0.73

Downstream Boundary – Water Level

The calibrated hydraulic model has adopted a slope of 0.000084 m/m to compute the normal depth at the downstream boundary. With this slope, the computed 100-year flood level at the downstream boundary was El. 253.38 m. The sensitivity analysis was performed for this parameter by adopting ±0.5 m as a plausible range of uncertainty, which corresponds to an energy grade slope of 0.000100 m/m (downstream water level at El. 252.88 m) and 0.000071 m/m (downstream water level at El. 253.88 m). The results are shown in **Figure 9** and **Table 18**.

Lowering the downstream water level by 0.5 m in the model resulted in lower computed 100-year water levels throughout the entire study reach, with a drop of 0.5 m at the downstream end and 0.17 m at the upstream end. The average change over the entire reach was 0.29 m downward. When the downstream boundary water level was raised by 0.5 m, the computed 100-year water levels rose throughout the entire reach with a 0.5 m increase at the downstream end and 0.20 m at the upstream end. The average change over the entire study reach was 0.32 m upward.

Table 18 Results of sensitivity analysis on downstream boundary condition

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Downstream Boundary Condition		
		0.5 m Below Baseline (Slope 0.000100 m/m)	Baseline (Slope 0.000084 m/m)	0.5 m Above Baseline (Slope 0.000071 m/m)
PXS37	32,785	256.19	256.36	256.56
PXS36	31,762	256.02	256.20	256.40
PXS35	30,910	255.92	256.09	256.30
PXS34	29,866	255.81	256.00	256.21
PXS33	29,050	255.87	256.05	256.26
PXS32	27,860	255.87	256.05	256.27
PXS31	26,255	255.67	255.87	256.10
PXS30	24,968	255.59	255.79	256.03
PXS29	24,113	255.35	255.57	255.81
PXS28	23,634	255.17	255.39	255.65
PXS27	23,591	255.12	255.35	255.60
PXS26	22,698	255.06	255.29	255.55
PXS25	21,545	255.02	255.25	255.52
PXS24	20,409	255.08	255.31	255.58
PXS23	19,433	255.04	255.28	255.55
PXS22	18,416	254.87	255.12	255.41
PXS21	17,365	254.81	255.07	255.36
PXS20	16,428	254.71	254.98	255.28
PXS19	15,592	254.38	254.67	255.00
PXS18	14,680	254.34	254.63	254.96
PXS17	13,690	254.36	254.65	254.98
PXS16	12,889	254.37	254.67	255.01
PXS15	11,953	254.37	254.68	255.01
PXS14	11,482	254.36	254.67	255.00
PXS13	11,025	254.33	254.64	254.99
PXS12	10,282	254.25	254.57	254.93
PXS11	9,512	254.10	254.45	254.82
PXS10	8,634	254.02	254.38	254.76
PXS9	7,958	253.99	254.35	254.73
PXS8	7,256	253.91	254.28	254.67
PXS7	6,402	253.86	254.24	254.63
PXS6	5,260	253.71	254.10	254.51
PXS5	4,192	253.58	253.99	254.41

Table 18 Results of sensitivity analysis on downstream boundary condition (continued)

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Downstream Boundary Condition		
		0.5 m Below Baseline (Slope 0.000100 m/m)	Baseline (Slope 0.000084 m/m)	0.5 m Above Baseline (Slope 0.000071 m/m)
PXS4	3,352	253.32	253.76	254.22
PXS3	2,234	253.08	253.55	254.04
PXS2	1,117	253.03	253.51	254.00
PXS1	0	252.95	253.45	253.95
Minimum Deviation		-0.17	-	0.20
Average Deviation		-0.29	-	0.32
Maximum Deviation		-0.50	-	0.50

4.5.2 Manning's Roughness Coefficient

Sensitivity analysis was performed on Manning's roughness coefficients for the channel and for the overbanks separately.

Channel Roughness Coefficient

The adopted channel roughness coefficient value for the 100-year flood was 0.017. This coefficient was varied by ±15% in the sensitivity analysis (i.e. the low and high channel roughness coefficients tested were 0.014 and 0.020 respectively). The results are shown in **Figure 10** and **Table 19**. The changes in the computed 100-year water levels due to the variation in the roughness coefficient are relatively uniform through the study reach. Increasing the roughness coefficient resulted in water level increases ranging from 0.73 to 0.88 m, with an average of 0.78 m. Decreasing the coefficient lowered the computed water levels by 0.78 to 0.96 m, with a 0.85 m average drop over the entire study reach.

Table 19 Sensitivity Analysis of Water Level to the Channel Manning's Roughness Coefficient

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Channel Roughness Coefficient		
		Low Roughness n = 0.014	Baseline Roughness n = 0.017	High Roughness n = 0.020
PXS37	32,785	255.52	256.36	257.12
PXS36	31,762	255.37	256.20	256.96
PXS35	30,910	255.28	256.09	256.85
PXS34	29,866	255.19	256.00	256.75
PXS33	29,050	255.26	256.05	256.78
PXS32	27,860	255.27	256.05	256.78
PXS31	26,255	255.05	255.87	256.61
PXS30	24,968	255.00	255.79	256.53

Table 19 Sensitivity Analysis of Water Level to the Channel Manning's Roughness Coefficient (continued)

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Channel Roughness Coefficient		
		Low Roughness n = 0.014	Baseline Roughness n = 0.017	High Roughness n = 0.020
PXS29	24,113	254.74	255.57	256.32
PXS28	23,634	254.55	255.39	256.17
PXS27	23,591	254.50	255.35	256.12
PXS26	22,698	254.45	255.29	256.06
PXS25	21,545	254.44	255.25	256.01
PXS24	20,409	254.53	255.31	256.04
PXS23	19,433	254.49	255.28	256.01
PXS22	18,416	254.29	255.12	255.88
PXS21	17,365	254.25	255.07	255.82
PXS20	16,428	254.16	254.98	255.74
PXS19	15,592	253.81	254.67	255.47
PXS18	14,680	253.78	254.63	255.42
PXS17	13,690	253.83	254.65	255.43
PXS16	12,889	253.85	254.67	255.44
PXS15	11,953	253.86	254.68	255.43
PXS14	11,482	253.84	254.67	255.42
PXS13	11,025	253.81	254.64	255.41
PXS12	10,282	253.74	254.57	255.35
PXS11	9,512	253.54	254.45	255.25
PXS10	8,634	253.49	254.38	255.18
PXS9	7,958	253.47	254.35	255.14
PXS8	7,256	253.40	254.28	255.08
PXS7	6,402	253.36	254.24	255.03
PXS6	5,260	253.21	254.10	254.90
PXS5	4,192	253.09	253.99	254.80
PXS4	3,352	252.81	253.76	254.61
PXS3	2,234	252.59	253.55	254.43
PXS2	1,117	252.57	253.51	254.36
PXS1	0	252.51	253.45	254.29
Minimum Deviation		-0.78	-	0.73
Average Deviation		-0.85	-	0.78
Maximum Deviation		-0.96	-	0.88

Overbank Roughness Coefficient

The adopted roughness coefficient for the overbanks and islands was 0.060 for light vegetated or developed areas, and 0.080 for densely vegetated areas. These values were varied by ±20% in the sensitivity analysis. The computed water levels for the 100-year flood discharge are compared with the baseline results in **Figure 11** and **Table 20.** The changes in the computed water levels resulting from this variation are negligible – about one to two centimeters only.

Table 20 Sensitivity Analysis of Water Level to the Floodplain Manning's Roughness Coefficient

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Floodplain Roughness Coefficient		
		Low Roughness n – 20%	Baseline Roughness n = 0.060/0.080	High Roughness n + 20%
PXS37	32,785	256.34	256.36	256.37
PXS36	31,762	256.18	256.20	256.21
PXS35	30,910	256.08	256.09	256.10
PXS34	29,866	255.98	256.00	256.01
PXS33	29,050	256.03	256.05	256.06
PXS32	27,860	256.03	256.05	256.07
PXS31	26,255	255.85	255.87	255.88
PXS30	24,968	255.78	255.79	255.80
PXS29	24,113	255.56	255.57	255.58
PXS28	23,634	255.38	255.39	255.40
PXS27	23,591	255.33	255.35	255.35
PXS26	22,698	255.27	255.29	255.30
PXS25	21,545	255.24	255.25	255.26
PXS24	20,409	255.29	255.31	255.32
PXS23	19,433	255.27	255.28	255.29
PXS22	18,416	255.11	255.12	255.13
PXS21	17,365	255.05	255.07	255.08
PXS20	16,428	254.97	254.98	254.99
PXS19	15,592	254.66	254.67	254.68
PXS18	14,680	254.62	254.63	254.64
PXS17	13,690	254.64	254.65	254.66
PXS16	12,889	254.66	254.67	254.68
PXS15	11,953	254.66	254.68	254.69
PXS14	11,482	254.65	254.67	254.67
PXS13	11,025	254.63	254.64	254.65
PXS12	10,282	254.56	254.57	254.58

Table 20 Sensitivity Analysis of Water Level to the Floodplain Manning's Roughness Coefficient (continued)

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Floodplain Roughness Coefficient		
		Low Roughness n – 20%	Baseline Roughness n = 0.060/0.080	High Roughness n + 20%
PXS11	9,512	254.43	254.45	254.46
PXS10	8,634	254.37	254.38	254.39
PXS9	7,958	254.33	254.35	254.36
PXS8	7,256	254.27	254.28	254.29
PXS7	6,402	254.22	254.24	254.24
PXS6	5,260	254.09	254.10	254.10
PXS5	4,192	253.98	253.99	253.99
PXS4	3,352	253.75	253.76	253.77
PXS3	2,234	253.55	253.55	253.56
PXS2	1,117	253.50	253.51	253.52
PXS1	0	253.43	253.45	253.45
Minimum Deviation		0.00	-	0.00
Average Deviation		-0.01	-	0.01
Maximum Deviation		-0.02	-	0.02

4.5.3 Sensitivity Analysis Summary

Among the tested parameters, the computed 100-year open water flood levels were most sensitive to the channel roughness coefficient. For a 15% increase and decrease in the channel roughness coefficient, the average changes in the computed water levels were 0.78 m and -0.85 m, respectively. The deviation was generally uniform throughout the study reach. The computed water levels were also sensitive to the upstream boundary discharge. Varying the 100-year flood peak discharge to its upper and lower 95% confidence limits resulted in average water level changes of 0.70 m and -0.54 m, respectively. Effects from varying the downstream boundary condition diminished from downstream to upstream. With a water level change of ±0.5 m at the downstream boundary, the computed water levels changed by about ±0.34 m near Fort Vermilion (RS 11,482 m) and up to ±0.20 m at the upstream end of the study reach. Finally, it was found that the modelling results were not sensitive to the overbank Manning's roughness coefficient. The computed water levels changed less than 0.02 m throughout the study reach when the roughness coefficient was varied by 20%.

5 ICE JAM MODELLING

5.1 Available Data

5.1.1 Ice Jam Highwater Marks

Highwater mark observations provide documentation of peak water levels at specific locations for selected floods. Highwater mark data were obtained for the April 2018 ice jam flood from a study conducted by NHC (2019b) for Mackenzie County. Highwater mark data were also obtained for the 2020 ice jam flood from both AEP and Mackenzie County. **Figure 4** shows the locations of these observations. **Table 21** lists the highwater mark elevations surveyed by NHC for the 2018 ice jam event. **Table 22** and **Table 23** list the highwater marks for the 2020 event surveyed by AEP and Mackenzie County, respectively. The highwater mark locations are indicated by their respective river stationing established for the ice jam model. These 2018 and 2020 highwater mark elevations were used for ice jam model calibration and validation, respectively.

Table 21 2018 ice jam flood highwater marks surveyed by NHC

River Station (m)	Event Date	Highwater Mark Elevation (m)
23,634	29 April 2018	256.172
19,183	29 April 2018	256.063
18,001	29 April 2018	255.867
17,310	29 April 2018	255.659
14,135	29 April 2018	255.298
13,325	29 April 2018	255.155

Table 22 2020 ice jam flood highwater marks surveyed by AEP

River Station (m)	Event Date	Highwater Mark Elevation (m)
23,602	27 April 2020	258.865
16,490	27 April 2020	257.770
15,698	27 April 2020	257.718
15,085	27 April 2020	257.704
14,790	27 April 2020	257.651
11,944	27 April 2020	257.474

Table 23 2020 ice jam flood highwater marks surveyed by Mackenzie County

River Station (m)	Event Date	Highwater Mark Elevation (m)
16,669	27 April 2020	257.718
16,527	27 April 2020	257.669
16,485	27 April 2020	257.634
16,480	27 April 2020	257.799
16,428	27 April 2020	257.738
16,378	27 April 2020	257.757
16,373	27 April 2020	257.676
16,313	27 April 2020	257.700
16,308	27 April 2020	257.749
16,295	27 April 2020	257.690
16,281	27 April 2020	257.737
16,218	27 April 2020	257.727
16,199	27 April 2020	257.704
15,804	27 April 2020	257.852
15,699	27 April 2020	257.699
15,665	27 April 2020	257.706
15,627	27 April 2020	257.688
15,614	27 April 2020	257.766
15,606	27 April 2020	257.755
15,587	27 April 2020	257.626
15,560	27 April 2020	257.687
15,545	27 April 2020	257.708
15,522	27 April 2020	257.597
15,493	27 April 2020	257.591
15,489	27 April 2020	257.676
15,449	27 April 2020	257.697
15,431	27 April 2020	257.573
15,418	27 April 2020	257.658
15,385	27 April 2020	257.626
15,378	27 April 2020	257.650
15,352	27 April 2020	257.633
15,327	27 April 2020	257.462
15,289	27 April 2020	257.676
15,277	27 April 2020	257.639

Table 23 2020 ice jam flood highwater marks surveyed by Mackenzie County (continued)

River Station (m)	Event Date	Highwater Mark Elevation (m)
15,245	27 April 2020	257.616
15,185	27 April 2020	257.635
15,022	27 April 2020	257.545
14,813	27 April 2020	257.731
14,172	27 April 2020	257.613
14,037	27 April 2020	257.501
13,941	27 April 2020	257.561
12,076	27 April 2020	257.476
12,013	27 April 2020	257.521
11,952	27 April 2020	257.236

Available highwater mark observations for other historical ice jam events are summarized in **Table 5** and **Table 6**. They reflect separate observations or estimates at one or two locations for each flood event and are subject to different levels of uncertainty.

5.1.2 Ice Observation Reports and Observations

Ice observation reports are produced by AEP (previously AENV) each year documenting ice conditions along the Peace River from the W. A. C. Bennett Dam to the Peace Athabasca Delta. The reports generally focus on the town of Peace River as it is the largest community along the observation reach and has a history of flooding due to ice jams. Documentation of ice conditions in Fort Vermilion are generally minimal except during years where breakup of the ice results in high water levels.

Ice observation reports were available for breakup for following years: 1975, 1982-1985, and 2004-2019. Observations relative to Fort Vermilion are summarized in **Table 24**. It was noted whether breakup was dominated by thermal or dynamic processes. Thermal breakups tend to produce only minor water level increases at breakup, while dynamic breakups can produce major water level increases. If breakup was considered dynamic, an effort was made to determine whether an ice jam or an ice run occurred at Fort Vermilion. When the breakup process was dominated by dynamic events but ice conditions were not referenced in the ice observations reports, it was assumed that an ice run occurred at Fort Vermilion because an ice jam has a much higher likelihood of being documented in the observation report.

Table 24 Summary of ice observation reports

Breakup Year	Comments from report	Dominant Breakup Process	Dynamic Event
1975	The ice cover was still intact on 22 April 1975 when monitoring was stopped	Thermal	N/A
1982	No direct reference to ice conditions at Fort Vermilion	Thermal	N/A
1983	No direct reference to ice conditions at Fort Vermilion	Thermal	N/A
1984	River ice breakup was dominated by thermal processes but an ice jam formed downstream of Fort Vermilion near the Caribou River confluence	Thermal	N/A
1985	Breakup of the ice at Fort Vermilion was reported by the RCMP on 2 May 1985	Thermal	N/A
2005	The ice front had receded almost to Fort Vermilion on 18 April 2005	Thermal	N/A
2006	No direct reference to ice conditions at Fort Vermilion	Thermal	N/A
2007	River ice breakup along the Peace River was dominated by dynamic processes but no direct reference to ice conditions at Fort Vermilion	Dynamic	Ice Run
2008	No direct reference to ice conditions at Fort Vermilion	Thermal	N/A
2009	Water levels were about 0.3 m above freeze-up levels on 21 April 2009	Thermal	N/A
2010	The ice clear from Fort Vermilion between 20 and 21 April, 2010. The orifice line of the WSC gauge was ripped out on 21 April 2010.	Thermal	N/A
2011	Ice movement was reported at Fort Vermilion between 27 and 28 April, 2011.	Thermal	N/A
2012	The ice front passed Fort Vermilion on 26 April 2012	Thermal	N/A
2013	Observations and analysis of water level gauge data indicated that an ice jam formed in the evening of 4 May 2013 and increased water level by about 0.6 m. The ice jam released the morning of 5 May 2013.	Dynamic	Ice Jam
2014	Breakup of the ice occurred between 26 and 27 April 2014 and water levels increased by about 3 m. An ice jam formed about 15 km upstream of the Vermilion Chutes resulting in a peak water level of 253.486 m at the Fort Vermilion WSC gauge. A second ice jam formed at the upstream end of the island that can be seen from Fort Vermilion and released without incident on 30 April 2014.	Dynamic	Ice Jam
2015	Ice front passed Fort Vermilion on 19 April 2015.	Thermal	N/A
2016	Ice front passed Fort Vermilion on 19 April 2016.	Thermal	N/A

Table 24 Summary of ice observation reports (continued)

Breakup Year	Comments from report	Dominant Breakup Process	Dynamic Event
2017	Ice breakup occurred the night of 29 April 2017.	Thermal	N/A
2018	An ice jam formed at Fort Vermilion on 28 April 2018. The ice jam released on 29 April 2018 and by 30 April 2018 the water level had fallen 3 m from the peak water level.	Dynamic	Ice Jam
2019	Ice runs occurred at Fort Vermilion on 19 April 2019 and 20 April 2019 but no significant rise in water level occurred.	Dynamic	Ice Run
2020	An ice run jammed just downstream of Beaver Ranch (44 km downstream of Fort Vermilion) in the morning of 27 April and caused extensive flooding in North Vermilion, Fort Vermilion east of 50th street, at the Fort Vermilion airport, and the south approach of the Hwy 88 bridge. The ice jam released in the afternoon of 28 April and water levels dropped by about 2.5 m in Fort Vermilion by the morning of 29 April.	Dynamic	Ice Jam

5.1.3 Breakup Levels at WSC Gauge

As described in **Section 3.1.3**, severe ice jams on the Peace River with the potential to cause flooding at Fort Vermilion are most likely to occur during breakup, while freeze-up ice jam flooding within the study reach is less common or less severe. As such, the current study focuses on breakup ice jam flooding. To support this, a series of historical breakup levels for Peace River at Fort Vermilion were assembled as described in this section.

Historic ice jam flood levels (prior to 1964) were estimated at the HBC trading post by Gerard and Karpuk (1979) as shown in **Table 6**. The HBC trading post is located near River Station 15,592 m, approximately 1.77 km downstream of the WSC gauge station (River Station 17,365 m). These flood levels were adjusted to provide breakup level estimates at the WSC gauge station based on a longitudinal slope of 0.0001 estimated from the surveyed highwater marks from the 2018 ice jam flood (**Table 21**). The estimated peak breakup levels for the six historic events are listed in **Table 25**.

Available strip charts that recorded gauge heights (water levels) of Peace River at Fort Vermilion (WSC Station 07HF001) from 1963 to 1993 were obtained from the WSC. No gauge data were available from 1994 to 2005; however, the gauge station was re-established in 2006 and equipped with a digital data logger. Digital data logger files that provide gauge height time series from 2006 to 2020 were also obtained from the WSC. The strip charts and data logger files were reviewed and used to determine the breakup date and peak water level of each year. These breakup levels were then compared with available daily water levels published by the WSC. For years where the maximum daily water levels during breakup were greater, the values from the published daily data records were taken as the peak breakup levels, presuming that the WSC used additional information to determine the water levels. The peak breakup levels from 1964 to 2020 (with gaps) resulting from this assessment are summarized in **Table 25**.

The strip charts for 1983 and 1984 indicate that the gauge did not function properly during breakup; but the notes on the strip charges include highwater mark levels surveyed by the WSC. For these two years, the WSC highwater mark levels were taken as the peak breakup levels. The peak breakup level for the 2020 event was provided by AEP to NHC – peak water level data was missing from the published record at the time of this report. The chart and digital logger data were also missing or unreliable due to gauge malfunction in some other years where no highwater mark information was available. For those years (1967, 1978, 1993, 2006, 2008 and 2015), breakup levels were estimated based on the incomplete records, and the estimates are expected to have higher uncertainties than the other years. These procedures are also summarized in **Table 25.**

While gauge data or systematic observations were unavailable in the period from 1994 to 2005, the April 1997 ice jam flood event at Fort Vermilion was monitored, according to AENV (2000). A peak water level at El. 254.031 m was recorded on 25 April near the HBC trading post. As shown in **Table 25**, this flood level was adjusted to provide an estimate at the WSC gauge station based on the 0.0001 longitudinal slope estimated for the 2018 ice jam flood profile.

Table 25 Peak break up levels for Peace River at Fort Vermilion (WSC Station 07HF001)

Year	Breakup Date [1]	Peak Breakup Level (m)	Breakup Level Estimation Notes
1876	May [2]	254.98	Transferred[7] from estimate by Gerard and Karpuk (1979)
1888	7-9 May [3]	258.88	Transferred[7] from estimate by Gerard and Karpuk (1979)
1894		255.88	Transferred[7] from estimate by Gerard and Karpuk (1979)
1934	22 April [4]	258.58	Transferred[7] from estimate by Gerard and Karpuk (1979)
1950	7 May [3]	252.48	Transferred[7] from estimate by Gerard and Karpuk (1979)
1963	23 May [3]	255.58	Transferred[7] from estimate by Gerard and Karpuk (1979)
1964	12 May	248.842	From WSC strip chart
1965	29 April	253.551	From WSC strip chart
1966	4 May	250.046	From WSC strip chart
1967	3 May	248.979	From WSC strip chart; the pressure line cut by ice and may not represent peak level
1974	1 May	251.098	From WSC strip chart
1975	7 May	248.034	From WSC strip chart
1976	9 May	249.065	From WSC strip chart
1977	16 May	250.366	From WSC strip chart
1978	17 May	250.061	From WSC strip chart; the pressure line cut by ice and may not represent peak level

Table 25 Peak break up levels for Peace River at Fort Vermilion (WSC Station 07HF001) (continued)

Year	Breakup Date [1]	Peak Breakup Level (m)	Breakup Level Estimation Notes
1979	11 May [5]	251.958	Based on WSC published daily level (higher than the estimate from strip chart)
1980	7 May	246.874	From WSC strip chart
1981	2 May [5]	249.508	Based on WSC published daily level (higher than the estimate from strip chart)
1982	1 May [5]	250.709	Based on WSC published daily level (higher than the estimate from strip chart)
1983	27 April	250.370	From WSC highwater mark surveyed on May 5
1984	17 April	250.421	From WSC highwater mark surveyed
1985	3 May	251.280	From WSC strip chart; pressure line cut by ice May 3 to 8
1986	28 April [5]	250.773	Based on WSC published daily level (higher than the estimate from strip chart)
1987	22 April	252.520	From WSC strip chart
1988	7 April	249.417	From WSC strip chart
1989	2 May	250.022	From WSC strip chart
1990	23 April	250.882	From WSC strip chart; ice run then jam building from downstream
1991	28 April	250.688	From WSC strip chart; likely ice run
1992	9 April	251.188	Based on WSC published daily level (higher than the estimate from strip chart)
1993	21 April	250.232	From WSC strip chart; pressure line cut by ice on April 21 so breakup not recorded
1997	25 April [6]	254.208	Transferred[7] from the peak flood level of 254.031 m recorded near the HBC post as reported by AENV (2000)
2006	18 April	249.575	From WSC data logger file; data missing from April 18 to May 2, no ice effects recorded
2007	27 April	253.208	From WSC data logger file
2008	20 April	250.288	From WSC data logger file; data missing from April 30 to May 5, no ice effects recorded
2009	28 April	250.578	From WSC data logger file; thermal breakup
2010	20 April	249.528	From WSC data logger file; thermal breakup
2011	26 April	249.903	From WSC data logger file; ice run after initial thermal breakup
2012	26 April	249.548	From WSC data logger file; thermal breakup
2013	5 May	252.218	From WSC data logger file; ice run
2014	27 April	253.478	From WSC data logger file; ice jam
2015	15 April	250.512	From WSC data logger file (records missing to 22 April except 14-15 April); ice front passed Fort Vermilion on 19 April according to ice observation report
2016	13 April	249.688	From WSC data logger file; thermal breakup

Table 25 Peak break up levels for Peace River at Fort Vermilion (WSC Station 07HF001) (continued)

Year	Breakup Date [1]	Peak Breakup Level (m)	Breakup Level Estimation Notes
2017	27 April	250.785	From WSC data logger file; thermal breakup
2018	29 April	255.787	From WSC data logger file; ice jam
2019	19 April	249.560	Provided by AEP Feb 2020; ice run
2020	27 April	257.762	Provided by AEP Feb 2020; ice jam

Notes:

1. Breakup dates, unless otherwise noted, were determined from available WSC strip charts and data logger files for Station 07HF001.
2. Thomson (1993).
3. Alberta Transportation Bridge File 74227.
4. AENV (1968).
5. WSC daily water level records.
6. AENV (2000).
7. Water level transfer from the Hudson's Bay Company trading post (RS 15,592 m) to WSC Station 07HF001 (RS 17,365 m) based on a longitudinal slope of 0.0001 that was determined from the 2018 ice jam flood profile.

The data listed in **Table 25** show that all historical breakups occurred in April or May. The table provides 46 years of peak breakup levels for WSC Station 07HF001, over a period of 145 years from 1876 to 2020. This data series covers both pre- and post-regulation periods (Peace River flows became regulated by the W. A. C. Bennett Dam in 1968). All breakup levels listed in **Table 25** are plotted in **Figure 12**. The data that were used for the ice jam frequency analysis are presented in **Section 5.2**.

5.1.4 Ice Jam Discharge

Peace River discharges are required as model inputs to simulate ice jams. The WSC gauge on the Peace River at Fort Vermilion (07HF001) provides daily discharge data for the periods of 1915 to 1922, 1961 to 1978 and 2006 to 2020. However, the gauge records miss data for winter and spring months in most years prior to 1978. Discharge measurements were not available from 1979 to 2005. In addition, discharges reported during breakup are often subject to higher uncertainty due to the effect of ice on hydraulics.

To fill the gaps in discharge data during breakup, available daily Peace River discharges at TPR (WSC Station 07HA001) were routed downstream to Fort Vermilion for the periods of 1916 to 1931 and 1958 to 2020. The routing analysis was performed at a daily time step using the routing model developed for the open water hydrology assessment (**Appendix E**), with tributary inflows being ignored. The routed Peace River daily discharges for April and May are compared with available gauge data at Fort Vermilion in **Figure 13**, which suggests that the routed daily discharges multiplied by 1.06 are representative of Peace River flows at Fort Vermilion. As such, the routed Peace River discharges factored by 1.06 were used to fill the gaps in the Fort Vermilion daily discharge records, and the results were then used to estimate breakup discharges where required for ice jam modelling described later in this chapter.

5.1.5 Flood Photography

Flood photography was obtained from several sources to assist with development of the hydraulic model. Selected photographs for both open water and ice jam floods are provided in **Appendix F.** The photographs were obtained from flood documentation reports from the AT bridge file for the Highway 88 bridge and from Mackenzie County from the ice jam flood in April 2018. The ice jam flood events documented by the photographs occurred in 1934, 1950, 1963, 2018, and 2020.

5.1.6 Historical Aerial Imagery

Historical aerial imagery was collected shortly after the peak of the 2020 ice jam flood. The imagery extends approximately from RS 26,000 to RS 9,000 and depicts flooded areas in and around Fort Vermilion and North Vermilion. The imagery was made available to NHC from AEP through public GENESIS services. Copies of the source data and information is available on request from AEP.Data@gov.ab.ca.

5.2 Ice Jam Frequency Analysis

The terms of reference for this study require determination of the 50, 100 and 200-year ice jam floods. This section provides ice jam flood level estimates for these three return periods at WSC Station 07HF001 that were developed from a frequency analysis of the breakup levels shown in **Table 25** and **Figure 12**.

5.2.1 Comparison of Pre and Post-regulation Data

Peace River flows became regulated in 1968, and normal operation of the W. A. C. Bennett Dam began after Williston Lake was filled to its normal water level in 1972. Since then, there has been ongoing debate on the extent to which flow regulation affects breakup ice jam processes on the lower Peace River. Recent discussions can be found in Wolf et al. (2005, 2006, 2012), Timoney (2009), Beltaos (2014, 2018), Hall et al. (2018), Timoney et al. (2019) and Beltaos and Peters (2020). In general, it is difficult to assess effects of the flow regulation on ice jam flooding in the lower Peace River due to many factors, including data scarcity and accuracy, complexity of ice jam mechanism, Peace River flow variability, climate variability, etc.

The focus of the current study is to determine the 50, 100 and 200-year ice jam flood levels at Fort Vermilion with the purpose to inform local land use planning decisions, flood mitigation projects, and emergency response planning. As such, instead of addressing flow regulation effects, the discussion and analysis in this section are intended to provide recommendations on selection of the ice jam flood level frequency estimates based on a reasonable and practical approach, to serve the study objective.

The breakup level dataset shown in **Table 25** and **Figure 12** spans 145 years from 1876 to 2020. However, it consists of a total of only 46 data points, with 10 of them spanning over the 92-year pre-regulation period and the other 36 in the post-regulation period. Among the pre-regulation data subgroup, only four events were identified from systematic hydrometric measurements, while the other six were documented major flood events. Given the scarcity and scatteredness of the pre-regulation data subgroup, it is difficult to evaluate the homogeneity of the dataset presented in **Figure 12** through standard statistical tests.

Gerard and Karpuk (1979) performed a frequency analysis of the 10-year pre-regulation breakup flood levels. Their estimated pre-regulation frequency distribution is illustrated in **Figure 14** (based on the 10 breakup levels from 1876 to 1967 listed in **Table 25**). To mitigate the data scarcity issue in this period, Gerard and Karpuk (1979) employed a threshold-exceedance concept for determining ice jam flood frequency. They divided the 10 data points into subgroups with a governed length of record being estimated for each to account for data gaps, and the exceedance probability for each data point was then estimated using an empirical positioning formula that is similar to the Cunnane formula. The governed length of record for a subgroup of data was defined as the number of years over which the flood levels for the events of this subgroup were known not to be exceeded. For example, Gerard and Karpuk (1979) believed that the 1888 and 1934 events were the two largest ice jam flood events over 121 years in the pre-regulation period based on available archival information and resident interviews; so the governed length of record for these two events was determined to be 121 years. Ranks of 1 and 2 over 121 years were then assigned to the 1888 and 1934 flood levels respectively, and their exceedance probabilities were determined accordingly.

The post-regulation breakup level data listed in **Table 25** were from systematic measurements (except 1997). This subgroup of data contains much less gaps (about 23%), and highly likely there were no severe floods in those gap years. As such, it is reasonable to use the data to represent the frequency distribution of the post-regulation breakup levels. The 36-year data are plotted in **Figure 14** based on the standard Cunnane formula (which is nearly the same as the positioning formula adopted by Gerard and Karpuk, 1979) to provide a comparison with the distribution of the pre-regulation events from Gerard and Karpuk (1979). As shown, the distributions of the pre- and post-regulation break levels appear to be consistent. A frequency curve that fits the data points for the pre-regulation events would also be representative for the large post-regulation events including the 2020, 2018 and 1997 ice jams.

The post-regulation subgroup of breakup levels covers a 47-year length of record. The 2020 ice jam flood is the largest event in this period. The post-regulation plot in **Figure 14** suggests that the return period of this event would be less than 100 years. However, this estimate is based on the limited length of record. The post-regulation subgroup does not contain events as large as in the 1934 and 1888. The flood levels of these two largest events on record as plotted on the pre-regulation frequency distribution do not appear to be unreasonably high in comparison with the trend of the post-regulation data points. As illustrated in **Figure 14**, the post-regulation breakup level data appear to follow the same frequency distribution as determined by Gerard and Karpuk (1979) for the pre-regulation data.

Based on the observations discussed above, the current study recommends that a frequency analysis be performed on the combined pre- and post-regulation data series listed in **Table 25**. With all available data being included, this approach is expected reduce uncertainty and provide reasonably conservative estimates for the 100 and 200-year ice jam flood levels.

It is worth noting that the data points corresponding to the four smallest pre-regulation events (1950, 1964, 1966 and 1967) appear to be more scattered in **Figure 14**. Their plotting positions are based on the governed lengths of record determined by Gerard and Karpuk (1979) as only 4 or 5 years, which are much shorter than those for the other pre-regulation events ranging from 58 to 121 years. It is easier to determine the governed length for large events than for smaller events because smaller events are generally less perceivable. The increased variability for the four smallest pre-regulation events may be related to higher uncertainties in the estimation of their governed lengths of record.

5.2.2 Frequency Analysis of Combined Data

The threshold-exceedance analysis approach undertaken by Gerard and Karpuk (1979) for the pre-regulation breakup levels requires determination of the governed length of period for each flood event. The approach is reasonable and necessary given the scarcity and scatteredness of the available pre-regulation data. However, the determination of the governed length of record is subjective to a greater extent and could have significant uncertainties for flood events with a medium to small magnitude. The combined breakup level dataset shown in **Table 25** and **Figure 12** represents a much larger sample, with the majority of the data from systematic measurements; so more reliable, standard statistical methods can be used. Nevertheless, the combined dataset still contains relatively significant data gaps that have to be addressed.

To account for data gaps encountered in flood frequency analysis, particularly gaps due to combining historic events with systematic records, the United States Geological Survey (USGS) has adopted the Hirsch-Stedinger formula (Hirsch and Stedinger, 1987) in their flood frequency guidelines, Bulletin 17C (USGS, 2018). The formula represents a threshold-exceedance-based positioning method. Based on the same concept, Guo (1990) proposed a very similar positioning formula, referred to as the modified exceedance Cunnane formula. This more recent formula was used in the current study to compute plotting positions (i.e. exceedance probabilities) of the breakup water level data listed in **Table 25** (the combined dataset). The data are then plotted in **Figure 15** based on the computed plotting positions (presented as return periods). This plot suggests that the flood level of the largest event (1888) would have a return period just under 250 years. The frequency estimates for the pre-regulation ice jam flood levels by Gerard and Karpuk (1979) are also shown in the figure for comparison.

Usually a theoretical frequency curve is used to fit a flood data sample; but it requires parameters to be estimated from the sample. In the combined breakup level dataset for this study, the large historic flood events account for a significant portion of the sample, with significant gaps between the data points. These historic flood levels tend to skew the estimation of statistical parameters (e.g. the population mean, standard deviation, skew coefficient, etc.) that are required to develop a theoretical frequency curve. On the other hand, the data gaps have been accounted for in the plotting positions based on the modified exceedance Cunnane formula as shown in **Figure 15.** As such, a theoretical curve based on the conventional approach (in which parameters would be estimated from the combined breakup level dataset) cannot fit the data points as plotted in **Figure 15**, although the computed plotting positions reasonably represent the frequency distribution of breakup levels at Fort Vermilion. In other words, the plot of the combined breakup level data shown in the figure is more reliable to represent the ice jam flood frequency distribution than a theoretical frequency curve with its parameters being estimated from the data.

In the lower tail of the plotted combined breakup level data in **Figure 15**, the five smallest events (with water level below El. 249.10 m) follow a distinctly different trend than the other data points. Excluding these five points, the remaining data points appear to follow an exponential distribution. As such, the two-parameter exponential distribution was used to fit the data points, with parameters being estimated through curve fitting. The cumulative distribution function for the two-parameter exponential distribution can be expressed as:

$$T = e^{-\frac{z-\alpha}{\lambda}} \qquad [1]$$

where:

T = return period

z = the random variable, which is breakup level herein

λ = scale factor, which equals the standard deviation of the population

α = location parameter, which equals the population mean minus λ

In this study, the scale factor and location parameter were determined through curve fitting to provide the best fit for the combined breakup data as plotted in **Figure 15**. The curve fitting process excluded the five smallest events (which could be deemed as low outliers). The resulting exponential frequency curve, as shown in **Figure 15,** is based on the adopted parameter values listed in **Table 26.** For comparison, the table also includes the parameter values estimated from the combined dataset (i.e. the conventional approach). These estimates cannot produce a frequency curve that fits the combined data plotted in **Figure 15.**

Table 26 Parameters for exponential frequency distribution

Parameter	Adopted Value[1]	Estimation from Sample	
		Based on the entire dataset[2]	Based on systematic measurements[3]
scale factor, λ	1.88	2.57	1.98
mean	251.90	251.90	251.31
location parameter, α	249.09	249.33	249.34

Notes:

1. Adopted parameter values were determined through curve fitting.
2. The combined data from 1876 to 2020, excluding the five smallest events (low outliers).
3. The combined data from systematic measurements from 1963 to 2020, excluding the five smallest events (low outliers).

The exponential frequency curve developed through curve fitting is intended to provide a smooth, continuous representation of the plotted breakup level data points (especially for higher return periods), from which breakup ice jam flood levels could be estimated by interpolation. Because the curve was developed through curve fitting, it should not be extrapolated beyond the extent of the plotted breakup data points, of which the upper limit is the return period of approximately 250 years. The curve was used herein to provide the 50, 100 and 200-year design flood levels of breakup ice jam for Peace River at Fort Vermilion. The estimated values are listed in **Table 27**. The approximate 95% confidence limits for these estimates are also included.

Table 27 Design flood levels of breakup ice jam for Peace River at Fort Vermilion (WSC Station 07HF001)

Return Period (years)	Design Flood Level (m)	
	Value	Approx. 95% Confidence Limit
200	259.07	258.72 - 259.42
100	257.76	257.47 - 258.06
50	256.46	256.22 - 256.70

5.3 Model Construction

The U.S. Army Corps of Engineer's *Hydrologic Engineering Center River Analysis System* (HEC-RAS) computer program (Version 5.0.6, November 2018) was used to generate an ice enhanced model to calculate the ice jam thickness and water surface profiles along the study reach. The basic inputs required by the ice enhanced model include those required by the HEC-RAS open water model (i.e. river cross sections along known lengths of channel, roughness coefficients for the channel and overbank areas at each cross section, a specified or computed water level at the downstream model boundary, and a discharge at all upstream model boundaries). In addition to these basic inputs, the ice enhanced model requires at each model cross section: a prescribed ice cover condition; under ice roughness; and a set of ice jam parameters characterizing the properties of the ice jam. These ice enhanced model inputs are used to solve for the under-ice hydraulics and ice jam stability relationship.

The HEC-RAS model allows the user to specify the ice cover condition as an option within the HEC-RAS cross section data editor. If no information is provided for the ice cover, then an open water condition is presumed. If the user assigns a value to the ice cover thickness, then the model assumes an ice cover condition. When an ice cover condition is defined, the user must provide additional ice-specific model parameters.

Additional detail on these parameters and how the model was enhanced for ice covered conditions is provided in the following sections.

5.3.1 Enhancement Methodology

The ice enhanced model started with the same geometry inputs used for the open water model and was then enhanced for ice jam simulation by adding a prescribed ice cover condition. The following steps were undertaken to develop the ice enhanced model.

1) Adjust and refine the open water geometry for improved performance of the ice jam thickness profile computations.
2) Define ice-specific model parameters.
3) Calibrate the model to observed recorded high water and ice levels by adjusting under ice roughness.

The following ice-specific model parameters were defined.

- Ice cover thickness in left overbank, main channel, and right overbank.
- Ice cover roughness values for the left overbank, main channel, and right overbank.
- Ice cover specific gravity.
- Ice cover condition (*known* ice thickness values or *wide-river* ice jam).
- Ice jam strength parameters (internal friction, ice jam porosity, stress ratio constants, maximum under ice velocity) for the wide-river ice jam option.
- Option to use a fixed or variable ice cover roughness

These ice enhanced model parameters are used to solve for the under ice hydraulics and ice jam stability relationship. The following provides additional detail on these parameters and how the model was enhanced to calculate ice jam profiles.

Geometry Improvements

<u>Interpolated cross sections</u>**:** The objective of the model geometry improvement was to decrease cross section spacing for improved ice jam modelling performance, by adding interpolated cross sections. Ice jam modelling experience by the authors and other investigators (Beltaos and Tang 2013; Flato and Gerard 1986) suggests that the ice jam solution algorithm requires a maximum cross section spacing of about ¼ of the ice jam width to adequately resolve the computed ice jam profile. Further, the model performs best when cross section spacing is regular and changes are gradual.

An ice jam width of 600 m was adopted to determine the model cross section spacing in this study. This value was considered to be representative of the ice jam widths used by the ice jam stability equations described in a subsequent section (equations 2 and 3). This set the target cross section spacing for ice jam modelling to 150 m. Accordingly, the open water model geometry was modified as follows for the purpose of developing the ice enhanced model:

- The cross sections upstream and downstream of the Highway 88 bridge were spaced 43 m apart in the original open water model, which is significantly less than the target cross section spacing of 150 m. Therefore, the upstream cross section was removed from the model.
- The presence of the Highway 88 bridge introduced local computational instabilities in the ice thickness computations during testing runs of the ice jam model. Therefore, the bridge was removed from the model.
- In addition to the remaining 36 cross sections from the open water model, 230 cross sections were created with the automated HEC-RAS cross section interpolation tool based on the target spacing of 150 m for the ice jam model. This includes 30 cross sections used to extend the model reach 5 km further downstream. As such, the final geometry for the ice enhanced model has a total of 266 cross sections.

<u>Main channel widths</u>: Bank stations were adjusted along the study reach to improve model stability and to provide for a more representative ice jam width. Adjustments were made so that the modelled main channel (portion between left bank and right bank stations) was representative of an average width of the floating ice jam along the river and so that changes in the ice jam width were gradual. The main channel was constrained to a single channel alongside islands and banks stations were placed within the constrained channel. This provides a reasonable approximation of field observations on ice jam widths, which are indicated by the presence of longitudinal shear walls. Observed shear wall lines generally follow a smooth pattern with gradual transitions. As ice jams form alongside islands, it is common for the ice to accumulate and shove first down one side of the island and then down the other. Bank stations within the open water main channel were necessary to allow for the transition from single channels to island splits and to ensure gradual changes in ice jam widths for model stability.

<u>Ice Cover Condition</u>: The HEC-RAS model allows the user to specify the ice cover condition as either a *"known geometry"* or a *"wide-river"* ice jam. For a known geometry condition, the user prescribes the ice thickness at each cross section along with a corresponding underside ice roughness (denoted in the model as *Ice Cover Manning's n Values)*. If option to compute an (wide-river) ice jam profile is selected, the model requires additional parameters to characterize the strength properties of the ice jam accumulation. Details on the ice specific modelling parameters are provided in the next section.

HEC-RAS uses the following form of the ice jam stability equation (for the so-called wide jam condition) to characterize the strength of the accumulation. It is a force balance equation where the stresses acting on the jam are ultimately transmitted to the channel banks (USACE 2016).

$$\frac{d(\bar{\sigma}_x t)}{dx} + \frac{2\tau_b t}{B} = \rho' g S_w t + \tau_i \qquad [2]$$

where:

$\bar{\sigma}_x$ = the longitudinal stress (along stream direction)

t = the accumulation thickness

τ_b = the shear resistance of the banks

B = the ice jam width

ρ' = the ice density

g = the acceleration of gravity

S_w = the water surface slope

τ_i = the shear stress applied to the underside of the ice by the flowing water

The ice jam stability equation can be restated in the following form which includes the ice jam parameters required by the model. This equation includes the parameters required as input to the model.

$$\frac{dt}{dx} = \frac{1}{2k_x\gamma_e}\left[\rho' g S_w + \frac{\tau_i}{t}\right]\Big] - \frac{k_o k_1}{B} t \qquad [3]$$

where:

k_x = a coefficient describing the ratio of vertical to longitudinal stress

γ_e = the effective unit weight of the accumulation

k_o = a coefficient describing the ratio of longitudinal to transverse stress

k_1 = a coefficient describing the ratio of transverse stress to shear at the banks

HEC-RAS uses an iterative approach to compute the ice jam profile thickness and under ice hydraulics. The under ice hydraulics are solved in a manner akin to the standard step method where the solution progresses in the upstream direction, then the ice jam thickness is found by solving the jam force balance equation (progressing in a downstream direction). The process is repeated until the user specified tolerances for changes in computer water levels are achieved or the maximum number of iterations is exceeded.

5.3.2 Ice-specific Model Parameters

To evaluate the formation of a consolidated ice cover, a number of ice-specific parameters are required. The primary parameters required to solve the jam stability equation are described as follows.

Composite Roughness n_o

The composite ice roughness is the combined bed and ice roughness factor resisting flow under the ice cover. HEC-RAS first computes the composite roughness, n_o, following the familiar Sabeneev relationship as follows:

$$n_o = \left(\frac{n_b^{3/2} + n_i^{3/2}}{2}\right)^{2/3} \qquad [4]$$

where n_b and n_i are the bed and bottom of ice roughness values, respectively.

Jam Stability Parameters

The jam stability parameters required as input to the HEC-RAS model to solve Equation [3] include: the internal friction angle of the jam, ϕ; the ice jam porosity (fraction of voids between ice floes), *p*; and the coefficient of lateral to longitudinal stress in the jam, k_1. All other parameters are solved internally by the model. Ice jam strength properties cannot be measured directly in the field and consequently they are not reported for observed events. However, for an idealized ***equilibrium*** thickness condition, the suite of jam stability parameters can be lumped into a single *jam stability* parameter, commonly denoted as μ. On rare occasions values for the jam stability parameter are reported. These values are deduced by assuming an equilibrium jam condition, ice jam width, and hydraulic properties. Pariset et al. (1966) first introduced this parameter and expressed it as:

$$\mu = k_1 k_x \tan\phi \qquad [5]$$

Beltaos (1978) deduced that the equilibrium jam stability relationships presented by Uzner and Kennedy (1976) could be made equivalent to those of Pariset et al. (1966) by expressing the jam stability parameter as:

$$\mu = \tan\phi\,(1-p) \qquad [6]$$

Then, Flato and Gerard (1986), following the work of Uzner and Kennedy (1976), presented a the following definition of the jam stability parameter,

$$\mu = k_v k_{xy} \tan\phi(1-p) \qquad [7]$$

Equivalence between these relationships is found when k_x/k_v=1 and $k_v k_{xy}$=1 (Healy 1997). With these assumptions it was possible to estimate the required input parameters ϕ and k_1, given the more familiar jam stability parameters μ and *p*.

Ice Jam Porosity: Ice jam porosity represents the volume fraction of the interstitial spaces in the ice accumulation. It is assumed to be the same above and below the water surface. A value of p = 0.4 is commonly used for ice jams and was also used for this study.

Jam Stability Parameter: There were no documented values found for the current study reach. Much further upstream at the Town of Peace River (TPR) on the Peace River previous investigators estimated μ = 0.93 (Neill and Andres, 1984). The same value was used for this study.

Internal Friction and Coefficient of Lateral to Longitudinal Stress: The internal friction and stress coefficient were found by substitution of the adopted values for p = 0.4 and μ = 0.93 into equations [5] through [7] resulting in values of $\phi = 57.17^o$ and $k_1 = 0.0868$.

Ice Jam Boundary Conditions

For calibration, a fully developed ice jam profile was prescribed between the downstream boundary and upstream boundary by specifying a *wide-river* ice jam condition between these boundaries. Fixed thickness values at the boundaries are required inputs to the model. For all calibration profiles the thickness at the upstream and downstream boundaries were set to 1.0 m. An initial ice thickness is required by the model at every cross section and this thickness also prescribes the minimum allowable thickness at each section. To achieve a realistic thickness profile the user must prescribe initial values that are below the fully developed ice thickness values. A initial thickness of 1 m was chosen to ensure that initial values were not set to values larger than the fully developed ice thickness values; thus, ensuring the computed thickness profile was not artificially constrained by the initial thickness. However, in the furthest upstream limits of the study reach, the computed thickness profile are forced by the model to gradually reduce down to the prescribed initial value of 1 m at the head (upstream boundary). Therefore, for some distance downstream of the upstream boundary, the computed ice thickness profile is somewhat thinner than a fully developed profile. This results in a slight under prediction in the water levels in the most upper reach. These effects did not extend downstream into reaches where recorded ice jam level data was available and thus, the calibration results were not sensitive to the adopted upstream boundary thickness.

The choice on downstream boundary ice thickness had a negligible effect on the computed ice jam profiles near the downstream limits of the study area (downstream open water model boundary) since the ice jam profile had fully developed to a near-equilibrium condition by this point.

5.3.3 Model Calibration

The ice enhanced model was calibrated to the measured 2018 ice jam flood level profile. There is no published discharge for the 2018 ice jam event; so the discharge used for calibration was estimated by routing the published discharge data at the WSC gauge at TPR downstream to the study area, as described in **Section 5.1.4**.

As is the case for open water model calibration, with all other hydraulic parameters and boundary conditions set, roughness remains the sole calibration parameter. For the ice enhanced model calibrations, the composite ice jam roughness values are reported. The composite ice jam roughness represents the combined roughness effects due to the bed and ice as represented in equation [4]. For each ice jam profile calculation, the bed roughness was kept constant and the under ice roughness values were adjusted, according to equation [4]. A composite roughness value of n_o = 0.029 resulted in the best fit to the measured 2018 recorded ice jam flood level profile as is illustrated in the comparison between the computed and measured ice jam profiles in **Figure 16** and **Table 28**. The computed and measured 2020 ice jam profiles are described in the model verification **Section 5.3.4**.

Table 28 Composite Manning's roughness coefficient calibration results (n_0=0.029) – 2018 Ice Jam Flood Levels

River Station (m)	Observed Elevation (m)	Simulated Elevation (m)	Simulated minus Observed (m)
23,634	256.172	256.28	0.11
19,183	256.063	255.93	-0.13
18,001	255.867	255.80	-0.07
17,310	255.659	255.71	0.05
14,135	255.298	255.37	0.07
13,325	255.155	255.31	0.16

The calibrated ice enhanced model was used to calculate an ice jam rating curve at the WSC gauge site for comparison to observed breakup highwater levels reported at the gauge, as shown in **Figure 17**. **Table 29** summarizes the observed water levels and discharges plotted on the figure. The gauged discharges at Fort Vermilion are only available for 2007, 2013, and 2014. For the other years, the discharges were estimated by routing the observed discharge from the TPR gauge downstream to the Fort Vermilion gauge (as described in **Section 5.1.4**). The 50-, 100-, and 200-year ice jam flood levels, as well as historic breakup water levels (including the estimate for the 1934 event), are also provided in **Figure 17** for comparison, although discharges associated with these events cannot be estimated.

Table 29 Observed Water Level and Discharge data for Dynamic Breakup Events at Fort Vermilion

Year	Breakup Date	Daily Discharge (m^3/s)		Breakup Water Level (m)	Remarks
		Fort Vermilion Gauge	Routed from TPR Gauge		
1963	23 May		5,584	255.580	ice jam
1983	27 Apr		1,845	250.370	ice run
1984	17 Apr		1,666	250.421	ice run
1985	03 May		2,338	251.280	ice run, possible ice jam
1987	22 Apr		2,976	252.520	ice jam
1990	23 Apr		1,655	250.882	ice run, possible ice jam
1991	28 Apr		2,883	250.688	possible ice run
2007	27 Apr	4,900	3,451	253.21	ice run or ice jam
2013	05 May	4,890	3,164	252.170	ice jam
2014	27 Apr	3,480	2,615	253.478	ice jam

Table 29 Observed Water Level and Discharge data for Dynamic Breakup Events at Fort Vermilion (Continued)

Year	Breakup Date	Daily Discharge (m^3/s)		Breakup Water Level (m)	Remarks
		Fort Vermilion Gauge	Routed from TPR Gauge		
2018	29 Apr		5,397	255.787	ice jam
2020	27 April	10,900	6,939	257.762	ice jam

5.3.4 Model Validation

The calibrated ice enhanced model was validated to the 2020 ice jam event through comparison between the computed and observed ice jam profile water levels. Computed water levels were calculated using the same ice jam parameters found through model calibration and the estimated 2020 breakup discharge. There is a notable difference between the estimated 2020 breakup discharge values found by hydraulic routing (6,939 m^3/s, corresponding to the gauged peak daily discharge of 7,620 m^3/s at TPR) and that reported by WSC, at the time of this report (10,900 m^3/s). The discharge adopted for model verification (7,860 m^3/s) was estimated as the calculated ice jam rating curve value corresponding to the breakup water level of 257.762 m (refer to **Figure 17**).

For the model validation using the 2020 event, the water surface profile computation followed the same approach as was used for the model calibration where the ice-affected flood level profile was estimated for a fully developed ice jam condition. The measured high water marks for the 2020 event were the result of multiple complicated unsteady ice run and ice jam events (Emmer et al. 2021). The validation run for this event provides a useful comparison between the fully developed ice jam condition and the observed, more complex ice-affected flood conditions.

The results of the model validation are depicted in **Figure 16** which provides a visual comparison between the simulated and observed ice jam profiles. **Table 30** lists a comparison between the validation profile water levels and the observed elevation profile surveyed by AEP. The water levels surveyed by Mackenzie County agree well with those surveyed by AEP. With the exception of the value at the WSC gauge, the validation profile closely follows the general shape and slope of the observed profiles. However, it is consistently lower than the surveyed levels. On average, the simulated values were 0.23 m lower than observed elevations surveyed by AEP and 0.12 m lower than observed elevations surveyed by Mackenzie County. Given the uncertainty in discharge and the apparent vertical difference between the surveyed highwater level profile and the highwater level reported by the gauge, the model was found to provide a reasonable analogue for the 2020 ice jam flood level profile.

Preliminary flood mapping extents derived from the 2020 validation profile were mapped with the 2020 aerial flood imagery (**Section 5.1.6**) and compared visually. While the flood levels had receded, those areas there were previously inundated were detectable in the images and the approximate extent of highwater could be seen. Good agreement was found between the simulated flood extents and the flood extents depicted in the imagery. Representative sample images of the comparison are provided in **Appendix F**.

The primary function of ice enhanced model is to extend the flood level frequency values derived at the WSC gauge to an ice jam flood level profile along the full study reach. The ice-enhanced model performs very well in this regard.

Table 30 Validation model results – 2020 Ice Jam Flood Levels

River Station (m)	Observed Elevation (m)	Simulated Elevation (m)	Simulated minus Observed (m)
Comparison to AEP Surveyed HWMs			
23,602	258.865	258.377	-0.49
16,490	257.770	257.669	-0.10
15,698	257.718	257.555	-0.16
15,085	257.704	257.484	-0.22
14,790	257.651	257.452	-0.20
11,944	257.474	257.239	-0.23

5.4 Ice Jam Flood Frequency Profiles

The ice-enhanced hydraulic model was used to generate flood frequency profiles for the 50-, 100-, and 200-year return periods. The profiles pass through the 50-, 100-, and 200-year return period water levels at the WSC gauge summarized in **Table 27**. The discharge for each return period ice jam profile is provided in

Table 31. The discharges were determined from the ice jam rating curve shown in **Figure 17**. They are representative discharges dependent on the adopted, ice-specific model parameters. The computed flood frequency water levels at each cross section are provided in **Table 32**. The ice jam flood frequency profiles are plotted in **Figure 18**. The design flood profiles were initially calculated using an upstream ice thickness boundary condition, t_{head} = 1 m, which resulted in an underprediction of the fully developed ice jam thickness in the most upper portion of the study reach. To account for this effect, the thickness in the head region was set to a constant fully developed ice thickness value which was approximated as the thickness at RS 29,050 m. This is the location where the computed ice thickness profile begins to taper down towards the fixed boundary condition ice thickness. While this provided a better approximation to the fully developed thickness profile in the head region, the resulting increase in the computed design level near the head region amounted to only a few centimeters.

Table 31 Representative discharge for the return period ice jam profiles

Return Period (years)	Ice Jam Water Level (m)	Representative Discharge (m^3/s)
50	256.46	6,215
100	257.76	7,860
200	259.07	9,950

Table 32 Computed Ice Jam Flood Frequency Water Levels

Cross Section	River Station (m)	Ice Jam Flood Levels (m) for Various Return Periods		
		50-year	100-year	200-year
PXS37	32,785	257.92	259.25	260.59
PXS36	31,762	257.82	259.16	260.50
PXS35	30,910	257.77	259.10	260.43
PXS34	29,866	257.69	259.01	260.34
PXS33	29,050	257.62	258.94	260.26
PXS32	27,860	257.53	258.85	260.18
PXS31	26,255	257.33	258.66	259.98
PXS30	24,968	257.20	258.53	259.86
PXS29	24,113	257.10	258.43	259.75
PXS28[1]	23,634	257.07[1]	258.40[1]	259.72[1]
PXS27	23,591	257.05	258.38	259.69
PXS26	22,698	256.97	258.29	259.60
PXS25	21,545	256.87	258.19	259.50
PXS24	20,409	256.80	258.11	259.41
PXS23	19,433	256.73	258.04	259.33
PXS22	18,416	256.61	257.92	259.22
PXS21	17,365	256.49	257.80	259.10
PXS20	16,428	256.37	257.67	258.97
PXS19	15,592	256.25	257.54	258.83
PXS18	14,680	256.15	257.44	258.73
PXS17	13,690	256.09	257.37	258.66
PXS16	12,889	256.03	257.32	258.61
PXS15	11,953	255.95	257.25	258.54

Cross Section	River Station (m)	Ice Jam Flood Levels (m) for Various Return Periods		
		50-year	100-year	200-year
PXS14	11,482	255.90	257.21	258.51

Table 32 Computed Ice Jam Flood Frequency Water Levels (continued)

Cross Section	River Station (m)	Ice Jam Flood Levels (m) for Various Return Periods		
		50-year	100-year	200-year
PXS13	11,025	255.83	257.15	258.47
PXS12	10,282	255.72	257.06	258.40
PXS11	9,512	255.54	256.88	258.25
PXS10	8,634	255.44	256.80	258.19
PXS9	7,958	255.36	256.72	258.12
PXS8	7,256	255.26	256.61	258.03
PXS7	6,402	255.14	256.50	257.93
PXS6	5,260	255.03	256.39	257.82
PXS5	4,192	254.87	256.24	257.67
PXS4	3,352	254.71	256.08	257.51
PXS3	2,234	254.50	255.86	257.31
PXS2	1,117	254.30	255.66	257.11
PXS1	0	254.11	255.48	256.93

Notes:

1. Cross section PSX28 is removed from the ice enhanced model so the reported water level at this cross section is interpolated from the upstream and downstream cross sections.

5.5 Model Sensitivity

A sensitivity analysis was conducted to determine the effects of changing the key ice jam model input parameters on computed hydraulic properties. The sensitivity of the computed hydraulic properties to changes in model parameters was evaluated in terms of changes in computed water levels, since: the computed water level is the hydraulic property of primary interest for a flood hazard study; and, changes in water level provide a good characterization of changes in other properties including: ice jam thickness, depth, velocity, flow area, and extent of inundation.

They ice jam model input parameters that were tested include the boundary conditions, ice jam stability parameters, and composite roughness coefficient. The values were selected to capture a range of plausible values for the study reach. The 100-year ice jam flood was used as the baseline for the sensitivity analysis. The ice jam profiles were computed in the same iterative manner that was used to compute the ice jam flood frequency profiles. The sensitivity analysis results are summarized in the following sections.

5.5.1 Boundary Conditions

Boundary conditions were required as inputs for the upstream and downstream boundaries of the ice enhanced model. A downstream water level was required to initiate the hydraulic calculations which progress from downstream to upstream and an upstream ice thickness was required to initiate the ice jam thickness profile calculations (jam stability equation) which progress from upstream to downstream.

Downstream Boundary – Water Level

The water level at the downstream boundary for the 100-year ice jam flood was 254.44 m and was achieved using a normal depth energy slope approximation (S_f = 0.00007). This was the baseline value for the sensitivity analysis to the downstream water level. A plausible range of uncertainty on the downstream water level of ±0.3 m was adopted. For the 100-year ice enhanced model, a +0.3 m and -0.3 m variation in the downstream water level boundary condition was ascribed by testing energy grade slopes of S_f = 0.000059 (downstream boundary water level of 255.85 m) and S_f = 0.000084 (downstream boundary water level of 255.16 m), respectively, for the normal depth approximation. The computed 100-year ice jam water levels with the varied downstream water levels are compared with the baseline results in **Figure 19** and **Table 33**.

Varying the water level at the downstream boundary by ±0.3 m from the baseline profile resulted in an average difference at the downstream boundary of ±0.34 m. The difference in water level decreased in the upstream direction with the average difference of ±0.05 m at RS 14,680 in the middle of Fort Vermilion. The difference in water level is less than ±0.01 m upstream of RS 26,255 m.

Table 33 Sensitivity Analysis of Computed Ice Jam Water Levels to Variation in Downstream Water Levels

Cross Section	River Station (m)	100-year Ice Jam Water Levels (m) for Variation in Downstream Water Levels		
		~0.3 m below Baseline S_f = 0.000084	Baseline S_f = 0.00007	~0.3 m above Baseline S_f = 0.000059
PXS37	32,785	259.23	259.23	259.24
PXS36	31,762	259.15	259.15	259.16
PXS35	30,910	259.10	259.09	259.11
PXS34	29,866	259.01	259.01	259.02
PXS33	29,050	258.94	258.94	258.95
PXS32	27,860	258.86	258.85	258.87
PXS31	26,255	258.66	258.66	258.68
PXS30	24,968	258.53	258.53	258.56
PXS29	24,113	258.43	258.43	258.46
PXS28	23,634	258.40	258.40	258.43
PXS27	23,591	258.37	258.38	258.41
PXS26	22,698	258.28	258.29	258.32

Cross Section	River Station (m)	100-year Ice Jam Water Levels (m) for Variation in Downstream Water Levels		
		~0.3 m below Baseline S_f = 0.000084	Baseline S_f = 0.00007	~0.3 m above Baseline S_f = 0.000059
PXS25	21,545	258.18	258.19	258.23
PXS24	20,409	258.10	258.11	258.15
PXS23	19,433	258.02	258.04	258.08

Table 33 Sensitivity Analysis of Computed Ice Jam Water Levels to Variation in Downstream Water Levels (continued)

Cross Section	River Station (m)	100-year Ice Jam Water Levels (m) for Variation in Downstream Water Levels		
		~0.3 m below Baseline S_f = 0.000084	Baseline S_f = 0.00007	~0.3 m above Baseline S_f = 0.000059
PXS22	18,416	257.91	257.92	257.97
PXS21	17,365	257.78	257.80	257.85
PXS20	16,428	257.64	257.67	257.73
PXS19	15,592	257.52	257.54	257.61
PXS18	14,680	257.41	257.44	257.52
PXS17	13,690	257.34	257.38	257.46
PXS16	12,889	257.28	257.32	257.41
PXS15	11,953	257.20	257.25	257.34
PXS14	11,482	257.16	257.21	257.31
PXS13	11,025	257.10	257.15	257.26
PXS12	10,282	257.00	257.06	257.18
PXS11	9,512	256.81	256.89	257.02
PXS10	8,634	256.71	256.80	256.95
PXS9	7,958	256.62	256.72	256.88
PXS8	7,256	256.50	256.61	256.79
PXS7	6,402	256.38	256.51	256.69
PXS6	5,260	256.25	256.39	256.59
PXS5	4,192	256.08	256.24	256.47
PXS4	3,352	255.89	256.08	256.33
PXS3	2,234	255.64	255.86	256.15
PXS2	1,117	255.40	255.66	255.99
PXS1	0	255.16	255.48	255.85
Minimum Deviation		0.00	-	0.01
Average Deviation		-0.07	-	0.10
Maximum Deviation		-0.32	-	0.36

Upstream Boundary – Ice Thickness

The ice thickness at the upstream boundary was varied by ±0.5 m. The baseline ice thickness was t_{head} = 1.0 m and was varied from t_{head} = 0.5 m to t_{head} = 1.5 m. The computed 100-year ice jam water levels with the varied upstream boundary ice thickness are compared with the baseline results in **Figure** 20 and **Table 34**. The computed ice jam water levels were found not to be sensitive to the ice thickness at the upstream boundary with an average variation of less than ±0.01 m.

Table 34 Sensitivity of Computed Ice Jam Water Levels to Variation in the Upstream Ice Thickness

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Floodplain Roughness Coefficient		
		0.5 m below Baseline t_{head} = 0.5 m	Baseline t_{head} = 1.0 m	0.5 m above Baseline t_{head} = 1.5 m
PXS37	32,785	259.22	259.23	259.23
PXS36	31,762	259.14	259.15	259.15
PXS35	30,910	259.09	259.09	259.09
PXS34	29,866	259.01	259.01	259.01
PXS33	29,050	258.94	258.94	258.94
PXS32	27,860	258.85	258.85	258.85
PXS31	26,255	258.66	258.66	258.66
PXS30	24,968	258.53	258.53	258.53
PXS29	24,113	258.43	258.43	258.43
PXS28	23,634	258.40	258.40	258.40
PXS27	23,591	258.38	258.38	258.37
PXS26	22,698	258.29	258.29	258.28
PXS25	21,545	258.19	258.19	258.18
PXS24	20,409	258.11	258.11	258.10
PXS23	19,433	258.04	258.04	258.03
PXS22	18,416	257.92	257.92	257.91
PXS21	17,365	257.80	257.80	257.79
PXS20	16,428	257.67	257.67	257.66
PXS19	15,592	257.54	257.54	257.53
PXS18	14,680	257.44	257.44	257.43
PXS17	13,690	257.38	257.38	257.37
PXS16	12,889	257.32	257.32	257.31

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Floodplain Roughness Coefficient		
		0.5 m below Baseline t_{head} = 0.5 m	Baseline t_{head} = 1.0 m	0.5 m above Baseline t_{head} = 1.5 m
PXS15	11,953	257.25	257.25	257.24
PXS14	11,482	257.21	257.21	257.20
PXS13	11,025	257.15	257.15	257.14
PXS12	10,282	257.06	257.06	257.05
PXS11	9,512	256.89	256.89	256.88

Table 34 Sensitivity of Computed Ice Jam Water Levels to Variation in the Upstream Ice Thickness (continued)

Cross Section	River Station (m)	100-year Flood Levels (m) for Varying Floodplain Roughness Coefficient		
		0.5 m below Baseline t_{head} = 0.5 m	Baseline t_{head} = 1.0 m	0.5 m above Baseline t_{head} = 1.5 m
PXS10	8,634	256.80	256.80	256.80
PXS9	7,958	256.72	256.72	256.72
PXS8	7,256	256.61	256.61	256.61
PXS7	6,402	256.51	256.51	256.50
PXS6	5,260	256.39	256.39	256.39
PXS5	4,192	256.24	256.24	256.24
PXS4	3,352	256.08	256.08	256.08
PXS3	2,234	255.86	255.86	255.86
PXS2	1,117	255.66	255.66	255.66
PXS1	0	255.48	255.48	255.48
Minimum Deviation		0.00	-	-0.01
Average Deviation		0.00	-	-0.01
Maximum Deviation		-0.01	-	0.00

5.5.2 Ice Jam Stability Parameters

The jam stability parameters required as input to the HEC RAS model include: the internal friction angle of the jam, ϕ; the ice jam porosity (fraction of voids between ice floes), p; and the coefficient of lateral to longitudinal stress in the jam, k_1. The combined effect of these parameters was expressed as a single jam stability parameter, μ. The calibrated jam stability parameter, μ = 0.93, was ascribed by setting the corresponding model input parameters to values of: p = 0.4, ϕ = 57.17° and k_1 = 0.0868. The model sensitivity analysis tested the range of μ between values of 0.80 and 1.2. The values of μ for the sensitivity tests were ascribed according to the following model input parameter values: p = 0.4, ϕ = 53.13° and k_1 = 0.111 for μ = 0.80; and p = 0.4, ϕ = 63.43° and k_1 = 0.0557 for μ = 1.2. The computed 100-

year ice jam water levels with the varied ice jam stability parameters are compared with the baseline results in **Figure 21** and **Table 35.**

A decrease in the jam stability parameter resulted in a thicker ice jam profile and consequently, an overall increase in ice jam flood levels. Conversely, an increase in the ice jam stability parameter resulted in a thinner ice jam profile and caused an overall decrease in ice jam flood levels. The change in water level is generally uniform throughout the study reach except for cross sections downstream of about RS 5,000 m.

Table 35 Sensitivity of Computed Water Level to the Variation in Jam Stability Parameter

Cross Section	River Station (m)	100-year Ice Jam Levels (m) for Variation in Jam Stability Parameter, μ		
		Decrease μ = 0.8	Baseline μ = 0.93	Increase μ = 1.2
PXS37	32,785	259.53	259.23	258.80
PXS36	31,762	259.46	259.15	258.71
PXS35	30,910	259.40	259.09	258.65
PXS34	29,866	259.32	259.01	258.57
PXS33	29,050	259.25	258.94	258.50
PXS32	27,860	259.17	258.85	258.41
PXS31	26,255	258.98	258.66	258.22
PXS30	24,968	258.85	258.53	258.10
PXS29	24,113	258.75	258.43	258.00
PXS28	23,634	258.72	258.40	257.97
PXS27	23,591	258.70	258.38	257.94
PXS26	22,698	258.61	258.29	257.86
PXS25	21,545	258.51	258.19	257.76
PXS24	20,409	258.43	258.11	257.68
PXS23	19,433	258.36	258.04	257.61
PXS22	18,416	258.24	257.92	257.50
PXS21	17,365	258.12	257.80	257.38
PXS20	16,428	257.99	257.67	257.26
PXS19	15,592	257.86	257.54	257.14
PXS18	14,680	257.76	257.44	257.04
PXS17	13,690	257.69	257.38	256.98
PXS16	12,889	257.64	257.32	256.93
PXS15	11,953	257.56	257.25	256.86
PXS14	11,482	257.52	257.21	256.82
PXS13	11,025	257.47	257.15	256.77

Cross Section	River Station (m)	100-year Ice Jam Levels (m) for Variation in Jam Stability Parameter, μ		
		Decrease μ = 0.8	Baseline μ = 0.93	Increase μ = 1.2
PXS12	10,282	257.38	257.06	256.68
PXS11	9,512	257.20	256.89	256.52
PXS10	8,634	257.12	256.80	256.45
PXS9	7,958	257.03	256.72	256.37
PXS8	7,256	256.93	256.61	256.28
PXS7	6,402	256.81	256.51	256.18
PXS6	5,260	256.69	256.39	256.08
PXS5	4,192	256.53	256.24	255.95

Table 35 Sensitivity of Computed Water Level to the Variation in Jam Stability Parameter (continued)

Cross Section	River Station (m)	100-year Ice Jam Levels (m) for Variation in Jam Stability Parameter, μ		
		Decrease μ = 0.8	Baseline μ = 0.93	Increase μ = 1.2
PXS4	3,352	256.35	256.08	255.81
PXS3	2,234	256.11	255.86	255.64
PXS2	1,117	255.88	255.66	255.48
PXS1	0	255.65	255.48	255.34
Minimum Deviation		0.32	-	-0.44
Average Deviation		0.31	-	-0.38
Maximum Deviation		0.17	-	-0.14

5.5.3 Composite Roughness

The composite roughness coefficient was varied above and below the calibrated value, $n_o = 0.029$, from $n_o = 0.025$ and $n_o = 0.033$. The model automatically computes the composite roughness coefficient based on the bed and ice roughness coefficients. For the sensitivity tests, the bed roughness coefficient was $n_{bed} = 0.017$, and the ice cover roughness coefficients were adjusted to $n_{ice} = 0.03187$ and $n_{ice} = 0.04571$, to achieve composite roughness coefficients of $n_o = 0.025$ and $n_o = 0.033$, respectively. The computed 100-year ice jam water levels with the varied composite roughness coefficient are compared with the baseline results in **Figure 22** and **Table 36**. Increasing the roughness coefficient resulted in water level increases ranging from 0.69 to 0.81 m, with an average of 0.74 m. Decreasing the coefficient lowered the computed water levels by 0.84 to 1.13 m, with a 0.95 m average drop over the entire study reach.

Table 36 Sensitivity of Computed Ice Jam Water Levels to Variation in the Composite Roughness Coefficient

Cross Section	River Station (m)	100-year Ice Jam Levels (m) for Variation in Composite Roughness Coefficient, n_o		
		Low Roughness n_o = 0.025	Baseline Roughness n_o = 0.029	High Roughness n_o = 0.033
PXS37	32,785	258.37	259.23	259.96
PXS36	31,762	258.30	259.15	259.88
PXS35	30,910	258.24	259.09	259.83
PXS34	29,866	258.16	259.01	259.74
PXS33	29,050	258.10	258.94	259.67
PXS32	27,860	258.01	258.85	259.58
PXS31	26,255	257.81	258.66	259.39
PXS30	24,968	257.68	258.53	259.26
PXS29	24,113	257.58	258.43	259.16
PXS28	23,634	257.51	258.40	259.13
PXS27	23,591	257.44	258.38	259.11
PXS26	22,698	257.35	258.29	259.01
PXS25	21,545	257.27	258.19	258.91
PXS24	20,409	257.20	258.11	258.82
PXS23	19,433	257.07	258.04	258.74
PXS22	18,416	256.95	257.92	258.63
PXS21	17,365	256.82	257.80	258.51
PXS20	16,428	256.70	257.67	258.39
PXS19	15,592	256.61	257.54	258.26
PXS18	14,680	256.55	257.44	258.15
PXS17	13,690	256.50	257.38	258.07
PXS16	12,889	256.41	257.32	258.02
PXS15	11,953	256.36	257.25	257.95
PXS14	11,482	256.29	257.21	257.91
PXS13	11,025	256.18	257.15	257.87
PXS12	10,282	255.98	257.06	257.79
PXS11	9,512	255.91	256.89	257.65
PXS10	8,634	255.82	256.80	257.57
PXS9	7,958	255.71	256.72	257.50
PXS8	7,256	255.59	256.61	257.41
PXS7	6,402	255.48	256.51	257.30

Table 36 Sensitivity of Computed Ice Jam Water Levels to Variation in the Composite Roughness Coefficient (continued)

Cross Section	River Station (m)	100-year Ice Jam Levels (m) for Variation in Composite Roughness Coefficient, n_o		
		Low Roughness $n_o = 0.025$	Baseline Roughness $n_o = 0.029$	High Roughness $n_o = 0.033$
PXS6	5,260	255.32	256.39	257.19
PXS5	4,192	255.16	256.24	257.04
PXS4	3,352	254.95	256.08	256.89
PXS3	2,234	254.76	255.86	256.67
PXS2	1,117	254.59	255.66	256.46
PXS1	0	254.52	255.48	256.27
Minimum Deviation		-0.84	-	0.69
Average Deviation		-0.95	-	0.74
Maximum Deviation		-1.13	-	0.81

5.5.4 Sensitivity Analysis Summary

The results of the sensitivity analysis of each of the ice jam model parameters was compared and it was found that the results were most sensitive to the composite roughness coefficient. For about a 15% increase and decrease in the composite roughness coefficient, the water level changed by an average of 0.74 m and -0.95 m, respectively. The deviation was generally uniform throughout the study reach. The results were less sensitive to the ice jam strength parameter with an average deviation along the study reach of ±0.34 m. The results were similarly sensitive to the downstream boundary water level with an average deviation of ±0.34 m at the downstream extent of the study reach. The affects of the downstream boundary water level decreased in the upstream direction. Finally, it was found that the results were not sensitive to the ice thickness at upstream boundary.

6 OPEN WATER AND ICE JAM FLOOD INUNDATION MAPS

Flood inundation mapping shows areas of ground that could be covered by open water and ice jam floods under existing conditions. Flood inundation maps were created for each of the thirteen open water and three ice jam flood scenarios, corresponding to various return periods. The methodology used to generate the flood inundation maps are summarized below. The open water inundation maps are provided in **Appendix H** and the ice jam flood inundation maps are provided in **Appendix I**.

6.1 Methodology

The methodology used to create the flood inundation maps followed the four basic steps listed below.

1) Create a water surface elevation (WSE) triangular irregular network (TIN) representing a contiguous surface of water elevation along the modelled river reach.
2) Generate a WSE grid with the same grid geometry as the underlying DTM. Assign elevation values to each grid cell based on the corresponding value taken from the WSE TIN.
3) Generate a depth grid (with the same geometry as for the WSE grid) by subtracting elevation values from the underlying DTM from the corresponding WSE grid value. Calculated negative values were replaced with a value of *NoData* to represent dry cells.
4) Generate inundation polygons based on the depth grids by converting cells with depth value greater than 0 m into inundation polygons.

The inundation polygons were further processed by smoothing, filtering out wetted areas that were not directly inundated (isolated areas), and removing isolated dry areas smaller than 100 m^2. The inundation polygons were then used to clip the WSE grids and depth grids to the full inundation extent. The WSE TINs, WSE grids, depth grids, and inundation polygons were created using standard ArcGIS tool sets and are provided in standard Esri file format as study deliverables.

6.2 Water Surface Elevation TIN Modifications

The WSE TIN represents the contiguous surface of water elevation resulting from interpolation along the TIN between water elevations computed at the model cross sections. The WSE TIN assumes that the water elevation changes linearly between adjacent cross sections. This assumption is accurate in the channel area and in the floodplain where inundated areas are connected by more than one distinct flow path to the channel area. If an inundated area is only connected to the channel area by one distinct flow path, then the inundated area was categorized as an overtopping area. The location where the overtopping area connects to the channel area via the distinct flow path is referred to as an overtopping point.

In overtopping areas, the assumption that the water elevation changes linearly between adjacent cross sections is invalid. The water elevation was instead set to be equal to the water elevation of the WSE TIN at the overtopping point. This generally reduced the inundated area upstream of the overtopping point and increased the inundated area downstream of the overtopping point.

Modifications are usually made to the WSE TIN to account for the failure of flood control structures; but such modifications are not required as there are not flood control structures located within the study reach (NHC 2019a).

6.3 Flood Inundation Areas

Flood inundation areas are categorized as residential areas and industrial/commercial areas. These areas have the potential to be impacted by open water and ice jam floods of various magnitude.

6.3.1 Residential Areas

Fort Vermilion

- Residences near 47 Street would be inundated during the 350-year open water flood and greater.
- Residences near River Road, 52 Avenue, and 45 Street would start becoming inundated during the 750-year open water flood and greater and during the 50-year ice jam flood and greater.
- Residences along River Road near RS 14,680 m would be inundated during the 750-year open water flood and greater and during the 50-year ice jam flood and greater.
- The inundation extends south along 45 Street past 50 Avenue during the 200-year ice jam flood resulting in the inundation of residences on both the east and west side of 45 Street.

North Vermilion

- Residences between RS 19,433 m and RS 17,365 m would start becoming inundated during the 50-year open water flood and 50-year ice jam flood. The extent and depth of the inundation increases for larger open water and ice jam floods.

Rural Area

- A residence along River Road near RS 13,690 m would be inundated during the 75-year open water flood and greater and during the 50-year ice jam flood and greater.

6.3.2 Industrial and Commercial Areas

Fort Vermilion

- Industrial land near RS 14,680 m would be inundated during the 200-year open water flood and greater.

- Commercial buildings near 47 Street and a gravel pit near Range Road 124A would be inundated during the 350-year open water flood and greater and 50-year ice jam flood and greater. A portion of a golf course near 47 Street would be inundated during the 1000-year open water flood and is mostly inundated during the 200-year ice jam flood.
- Commercial buildings near 45 Street would be inundated during the 750-year open water flood and greater.
- Commercial buildings on the east side of the intersection of River Road and 50 Street become inundated during the 50-year ice jam flood.

North Vermilion

- Agriculture land between RS 19,433 m and RS 17,365 m would be inundated during the 35-year open water flood and greater.

Rural Area

- Agricultural land on the left overbank upstream of RS 29,866 m would be inundated starting during the 5-year open water flood and greater.
- A gravel pit on the right overbank between RS 26,255 m and RS 24,968 m would be inundated during the 50-year open water flood and greater.
- Agricultural land on the right overbank near RS 24,113 m and RS 23,634 m would be inundated starting during the 35-year open water flood and greater.
- Agricultural land on the right overbank near RS 22,698 m and RS 21,545 m would be inundated starting during the 5-year open water flood and greater.
- Agricultural land on the left overbank between RS 14,680 m and RS 11,482 m would be inundated during the 5-year open water flood and greater.
- Portions of the Wop May Memorial Aerodrome start becoming inundated during the 50-year open water flood. During the 75-year open water flood, the edges of the runway become inundated along with several of the buildings on the property. By the 200-year open water flood, the runway is nearly fully inundated. Most of the runway is inundated at the 50-year ice jam flood. For larger open water and ice jam return period floods, the depth of flooding increases.
- Agricultural land on the right overbank between RS 5,260 m and RS 3,352 m would be inundated during the 75-year open water flood and greater. A significant percentage of this agricultural land is inundated during the 1000-year open water and 200-year ice jam floods.

7 FLOODWAY DETERMINATION

Flood hazard identification involved the delineation of floodway and flood fringe zones for the ice jam design flood under the FHIP Guidelines (Alberta Environment, 2011), incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta.

7.1 Design Flood Selection

The design flood for flood hazard identification is typically a flood that has a one percent annual exceedance probability. This is a flood with a statistical 100-year return period, also commonly referred to as the “one in one hundred year flood”.

The 100-year ice jam flood was selected as the design flood for flood hazard identification as ice-affected water levels are higher than open water levels for the same return period. For ice jam conditions, the 100-year design flood is the water level which has a one percent chance of being equalled or exceed in a given year. It is not necessarily meant to represent a static ice jam over the full length of the study reach at a single point in time. Instead, the 100-year ice jam design flood should be interpreted such that, anywhere along the study reach, an ice jam may develop and produce the 100-year ice jam flood levels over some distance within the study reach. It is assumed that there is an equal likelihood of ice jams occurring anywhere within the study reach.

7.2 Floodway and Flood Fringe Terminology

Flood Hazard Area

The flood hazard area is the area of land that would be flooded during the design flood. It is composed of the floodway and the flood fringe zones, which are defined below.

Flood Hazard Mapping

Flood hazard mapping identifies the area flooded for the design flood and is typically divided into floodway and flood fringe zones. Flood hazard maps can also show additional flood hazard information, including areas of high hazard within the flood fringe and incremental areas at risk for more severe floods, like the 200-year and 500-year floods. Flood hazard mapping is typically used for long-term flood hazard area management and land-use planning.

Floodway

When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard where flows are deepest, fastest, and most destructive during the 100-year design flood. The floodway generally includes the main channel of a stream and a portion of the adjacent overbank area. Previously mapped floodways do not typically become larger when a flood hazard map is updated, even if the flood hazard area gets larger or design flood levels get higher.

Flood Fringe

The flood fringe is the portion of the flood hazard area outside of the floodway. The flood fringe typically represents areas with shallower, slower, and less destructive flooding during the 100-year design flood. However, areas with deep or fast moving water may also be identified as high hazard flood fringe within the flood fringe. Areas at risk behind flood berms may also be mapped as protected flood fringe areas.

Design Flood Levels

Design flood levels are the computed water levels associated with the design flood.

7.3 Flood Hazard Identification

7.3.1 Floodway Determination Criteria

In areas being mapped, the floodway typically represents the area of highest hazard where flows are deepest, fastest, and most destructive during the design flood. The following criteria, based on those described in current FHIP guidelines, are used to delineate the floodway in such cases:

- Areas in which the depth of water exceeds 1 m or the flow velocities are greater than 1 m/s shall be part of the floodway.
- Exceptions may be made for small backwater areas, ineffective flow areas, and to support creation of a hydraulically smooth floodway.
- In no case should the floodway boundary extend into the main river channel area.
- For reaches of supercritical flow, the floodway boundary should correspond to the edge of inundation or the main channel, whichever is larger.

Flood hazard areas in which the depth of water exceeds 1 m or the flow velocities greater than 1 m/s but excluded from the floodway are identified as high-hazard flood fringe.

Criteria governing floodway determination for different cross sections are listed in **Table 37**. The table also includes the locations where the floodway boundaries intersect each model cross section, presented as the left and right floodway limit stations. For all cross sections, the floodway boundaries are governed by the 1 m water depth criterion. The 100-year ice jam flood was selected as the design flood because the ice-affected flood levels exceed the 100-year open water flood levels. The previous floodway limits were determined for open water flooding (Alberta Environment, 2000). Therefore, the floodway determined from this study extends beyond the existing floodway limits.

Table 37 Design floodway limit stations and governing criteria at the model cross sections

Cross Section	River Station (m)	Left		Right	
		Floodway Limit Station (m)	Governing Criteria	Floodway Limit Station (m)	Governing Criteria
PXS37	32,785	132.47	1 m Depth	2,031.31	1 m Depth
PXS36	31,762	81.63	1 m Depth	1,932.17	1 m Depth
PXS35	30,910	80.58	1 m Depth	1,828.60	1 m Depth
PXS34	29,866	57.19	1 m Depth	1,437.95	1 m Depth
PXS33	29,050	31.89	1 m Depth	1,520.78	1 m Depth
PXS32	27,860	19.27	1 m Depth	1,844.54	1 m Depth
PXS31	26,255	44.73	1 m Depth	2,024.76	1 m Depth
PXS30	24,968	20.86	1 m Depth	1,816.66	1 m Depth
PXS29	24,113	284.18	1 m Depth	1,920.33	1 m Depth
PXS28[1]	23,634	353.77	1 m Depth	2,114.10	1 m Depth
PXS27	23,591	358.01	1 m Depth	2,157.58	1 m Depth
PXS26	22,698	158.52	1 m Depth	1780.6	1 m Depth
PXS25	21,545	184.1	1 m Depth	1749.09	1 m Depth
PXS24	20,409	102.91	1 m Depth	1,964.81	1 m Depth
PXS23	19,433	149.73	1 m Depth	2,224.62	1 m Depth
PXS22	18,416	432.04	1 m Depth	2,512.34	1 m Depth
PXS21	17,365	563.08	1 m Depth	2,601.23	1 m Depth
PXS20	16,428	163.32	1 m Depth	2,392.06	1 m Depth
PXS19	15,592	200.38	1 m Depth	2,483.21	1 m Depth
PXS18	14,680	144.16	1 m Depth	2,631.79	1 m Depth
PXS17	13,690	277.05	1 m Depth	3,088.61	1 m Depth
PXS16	12,889	24.39	1 m Depth	2,982.49	1 m Depth
PXS15	11,953	92.86	1 m Depth	3,665.03	1 m Depth
PXS14	11,482	150.75	1 m Depth	3,803.87	1 m Depth
PXS13	11,025	99.97	1 m Depth	4,234.40	1 m Depth
PXS12	10,282	82.30	1 m Depth	3,176.53	1 m Depth
PXS11	9,512	25.16	1 m Depth	2,624.95	1 m Depth
PXS10	8,634	1.89	1 m Depth	2,310.78	1 m Depth
PXS9	7,958	0	1 m Depth	2,183.04	1 m Depth

Table 37 Design floodway limit stations and governing criteria at the model cross sections (continued)

Cross Section	River Station (m)	Left		Right	
		Floodway Limit Station (m)	Governing Criteria	Floodway Limit Station (m)	Governing Criteria
PXS8	7,256	96.54	1 m Depth	1,881.55	1 m Depth
PXS7	6,402	13.59	1 m Depth	1,421.68	1 m Depth
PXS6	5,260	61.84	1 m Depth	1,607.57	1 m Depth
PXS5	4,192	55.39	1 m Depth	1,518.86	1 m Depth
PXS4	3,352	51.72	1 m Depth	1,600.89	1 m Depth
PXS3	2,234	31.22	1 m Depth	2,039.84	1 m Depth
PXS2	1,117	54.19	1 m Depth	1,553.28	1 m Depth
PXS1	0	11.52	1 m Depth	1,465.67	1 m Depth

Notes:

1. Cross section PSX28 is removed from the ice enhanced model so the reported water level at this cross section is interpolated from the upstream and downstream cross sections.

7.3.2 Design Flood Profile

The design flood profile, corresponding the 100-year ice jam design flood profile, is shown in **Figure 23** and the design flood levels at each cross section are provided in **Table 38**.

Table 38 Design flood levels

Cross Section	River Station (m)	Design Flood Level (m)
PXS37	32,785	259.25
PXS36	31,762	259.16
PXS35	30,910	259.10
PXS34	29,866	259.01
PXS33	29,050	258.94
PXS32	27,860	258.85
PXS31	26,255	258.66
PXS30	24,968	258.53
PXS29	24,113	258.43
PXS28[1]	23,634	258.40[1]
PXS27	23,591	258.38
PXS26	22,698	258.29
PXS25	21,545	258.19
PXS24	20,409	258.11
PXS23	19,433	258.04
PXS22	18,416	257.92
PXS21	17,365	257.80
PXS20	16,428	257.67
PXS19	15,592	257.54
PXS18	14,680	257.44
PXS17	13,690	257.37
PXS16	12,889	257.32
PXS15	11,953	257.25
PXS14	11,482	257.21
PXS13	11,025	257.15
PXS12	10,282	257.06
PXS11	9,512	256.88

Table 38 Design flood levels (continued)

Cross Section	River Station (m)	Design Flood Level (m)
PXS10	8,634	256.80
PXS9	7,958	256.72
PXS8	7,256	256.61
PXS7	6,402	256.50
PXS6	5,260	256.39
PXS5	4,192	256.24
PXS4	3,352	256.08
PXS3	2,234	255.86
PXS2	1,117	255.66
PXS1	0	255.48

Notes:

1. Cross section PSX28 is removed from the ice enhanced model so the reported water level at this cross section is interpolated from the upstream and downstream cross sections.

7.3.3 Floodway Criteria Maps

Floodway criteria maps are a tool for documenting the results of the floodway determination and depict the floodway and flood fringe extents for the design flood. The floodway criteria maps are provided in **Appendix J**. The floodway criteria maps include:

- the inundation extents for the design flood;
- the areas where the depth of water is 1 m or greater and the corresponding 1 m depth contour;
- the floodway limit station locations;
- floodway boundaries from the previous flood hazard study;
- stranded areas of dry ground within the flood hazard area; and
- the location and extent of the model cross sections.

The floodway criteria maps were produced using the following procedure:

- The extent of inundation for the design flood was mapped using the procedure described in **Section 6.1**. The procedure included generation of the water surface elevation (WSE) triangular irregular network (TIN), WSE grid, and flood depth grid which were provided as study deliverables.
- Inundated areas where the depth of water was equal to or exceeded 1 m and the corresponding 1 m depth contour were generated from the flood depth grid. The depth

contours were filtered and smoothed using the same parameters and procedures as those applied to the inundation extents, as described in **Section 6.1**.

The floodway boundary was mostly defined by the 1 m depth contour. When the width of the flood fringe was impractically small, the floodway line was drawn coincident with the edge of inundation. In backwater zones where smaller water bodies flow into the Peace River, the floodway limit boundary extents upstream along the water body to the study area boundary.

7.4 Design Flood Hazard Determination

7.4.1 Design Flood Hazard Maps

The design flood hazard maps illustrate the flood hazard area under the design flood hazard scenario corresponding to the design flood levels listed in **Table 38**. The design flood hazard maps were developed using the following process.

The design flood hazard maps were developed from the design flood criteria maps. The extent of the floodway was delineated from the floodway boundary on the design floodway criteria maps. Areas of high ground or areas of depth less than 1 m within the extent of the floodway boundary were incorporated into the floodway. The floodway is represented as a single contiguous polygon. The limit of the flood fringe follows the design flood extent and isolated areas were not included. Areas of high ground above the design flood water level within the high hazard flood fringe were included in the high hazard flood fringe area; while, areas of high ground above the design flood water level were excluded from the flood fringe. The WSE TINs, WSE grids, depth grids, and inundation polygons were created for the design flood hazard are provided as study deliverables.

The design flood hazard maps are provided in **Appendix K**.

Areas in the Floodway

- Agricultural land in the rural area on the left overbank between RS 32,785 m and RS 29,866 m, and on the left overbank between RS 14,680 m and RS 11,482 m.
- Agricultural land in the rural area on the right overbank between RS 24,968 m and RS 20,409 m.
- Industrial development in the rural area on the right overbank between RS 26,255 and RS 24,133 m.
- Agricultural land and several residences in North Vermilion between RS 19,433 m and RS 16,428 m.
- Residences and commercial/industrial businesses downstream of RS 16,428 m along River Road in Fort Vermilion, including the water treatment plant lagoons.
- Residential properties in Fort Vermilion near 47 Street, 45 Street, and 52 Avenue.
- Structures and the majority of the runway of the Wop May Memorial Aerodrome.

- Agricultural land in Fort Vermilion between RS 15,592 m and RS 13,690 m.

Areas in the High Hazard Flood Fringe

- Agricultural land in Fort Vermilion between RS 6,402 and RS 3,352.

Areas in the Flood Fringe

- Agricultural land in North Vermilion between RS 19,433 and 16,423.
- Agricultural land in Fort Vermilion between RS 15,592 m and RS 13,690 m.

8 POTERNTIAL CLIMATE CHANGE IMPACTS

To address the potential impacts of climate change on flood levels, more severe open water and ice jam flood scenarios were compared to the current design flood estimates in order to obtain a measure of "freeboard" that may be generally appropriate for long-term planning purposes. To obtain information appropriate for other applications, the simplified approach taken herein could be supplemented in the future by a more rigorous regional climate analysis and site-specific impact assessment.

8.1 Comparative Scenarios

Comparative scenarios were selected for both open water and ice jam conditions. For open water conditions, the baseline 100-year design flood water levels were compared to water levels computed for discharges of 19,900 m^3/s and 21,700 m^3/s, which are respectively 10 and 20 percent greater than the adopted 100-year flood discharge. This approach is consistent with guidelines prepared by Engineers and Geoscientists British Columbia (EGBC, 2018). EGBC recommends that for basins where no historical trend is detectable in local or regional streamflow magnitude frequency relations, a 10 percent upward adjustment in design discharge be applied to account for likely future changes in water input from precipitation. On the other hand, if a statistically significant trend is detected, a 20 percent adjustment may be appropriate, particularly for smaller basins. A more detailed discussion of the potential climate change impacts on open water floods can be found in **Appendix E**. The general conclusion was that the effect of climate change on discharges on the Peace River was uncertain. For ice jam conditions, the baseline ice jam design flood levels were compared to the 200-year ice jam flood levels.

8.2 Results

The computed water surface profiles of the selected comparative scenarios are compared to the open water and ice jam design flood water surface profiles in **Figure 24**. The water level increase for the comparatives scenarios is generally constant throughout the study reach. The increased open water discharge of 10 and 20 percent cause the water levels to increase relative to the open water design water levels on average by 0.57 m and 1.34 m, respectively. The 200-year ice jam flood is on average 1.08 m higher than the ice jam design flood.

8.3 Supplementary Information

Climate change has the potential to affect many factors related to flood severity and ice jam propensity. For open water floods, more frequent and greater intensity summer rain storms are commonly attributed to future climate flood risks. A comprehensive analysis would consider meteorological and hydrological factors at the basin scale to assess changes in flood peak discharges and their associated return periods. For ice jam induced flooding, the effects of climate change are even more complex as precipitation, temperature, streamflow regulation, and antecedent conditions affecting ice cover thickness and integrity all come into play.

9 CONCLUSIONS

The objectives of this study were to assess river flood-related hazards along a 28 km long reach of the Peace River within Mackenzie County, including Fort Vermilion and North Vermilion. A flood hazard mapping study was previously completed for Fort Vermilion by AEP in 2000. The present study provides an update of this work and includes both open water and ice jam flood scenarios.

The Fort Vermilion Flood Hazard Study consists of seven major project components: Survey and Base Data Collection, Open Water Hydrology Assessment, Open Water Hydraulic Modelling, Open Water Flood Inundation Mapping, Ice Jam Modelling, Ice Jam Flood Inundation Mapping, and Design Flood Hazard Identification and Mapping. This report summarizes the work of all seven components.

The collection of survey and base data primarily supports the hydraulic modelling and flood mapping. A total of 37 cross sections were surveyed along the study reach using a combination of boat-based bathymetric and ground surveys to complement the LiDAR-derived DTM of the overbank area. In addition, geometric details were collected for one bridge and one culvert in the study reach.

An open water hydrology assessment was conducted for the Peace River at Fort Vermilion to determine the naturalized discharge for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200- 350-, 500, 750-, and 1000-year year return periods. The naturalized discharges were significantly greater than the discharges determined in the previous flood hazard study carried out in 2000 for the 2-, 10-, 50-, and 100-year return periods.

A hydraulic model of the study reach was developed using the HEC-RAS computer program from the U.S. Army Corps of Engineers. River bathymetry and digital terrain data were used to develop the geometry of the hydraulic model. The channel roughness coefficient was calibrated for low flow conditions using water elevations collected during the survey and for high flow conditions using high water marks collected during the June 1990 flood. The overbank roughness coefficients were defined based on landcover composition, professional judgement, and guidance from literature. Water surface profiles were prepared for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year open water flood frequency return period discharges.

An ice jam frequency analysis was conducted on available breakup levels at the WSC gauge to determine the 50-, 100-, and 200-year ice jam flood levels at the WSC gauge. To extend the flood level estimation from the gauge to the upstream and downstream extent of the study area, an ice enhanced model was developed by modifying the calibrated open water hydraulic model. The model was enhanced by removing closely spaced cross sections at hydraulic structures, interpolating cross sections to decrease spacing, and modifying bank stations to reflect the expected active width of the ice jam. The under ice roughness coefficient was calibrated to match highwater marks from the 2018 ice jam and validated with highwater marks and aerial imagery from the 2020 ice jam. The calibrated ice enhanced model was used to simulate the 50-, 100-, and 200-year ice jam water surface profiles.

Flood inundation maps were created for the open water and the ice jam flood frequency return periods. In Fort Vermilion, the first residential property and land around the Wop May memorial Aerodrome become inundated starting at the open water 50-year return period. Additional inundation of residential and commercial/industrial properties occurs during the open water 200-year return period. In North Vermilion, agricultural land becomes inundated starting during the open water 20-year return period. Several residences become inundated during the open water 35-year return period. Within rural areas, agricultural land starts becoming inundated during the open water 2-year return period. The area of agricultural land inundated increases with increasing return period. Residential properties start becoming inundated in rural areas during the open water 200-year return period. More severe floods including the assessed ice jam events do not inundate more structures; however, they increase the depth of inundation.

Floodway criteria maps were developed for the design flood which illustrate the criteria used to define the floodway and flood fringe. The design hazard flood corresponded to the 100-year ice jam design flood. The floodway boundary was primarily governed by the 1 m depth criteria throughout the study reach. The floodway included several residences in North Vermilion and residential and commercial buildings downstream of 47 Street in Fort Vermilion, including the Wop May Memorial Aerodrome. Agricultural land was also located within the design floodway throughout the study reach.

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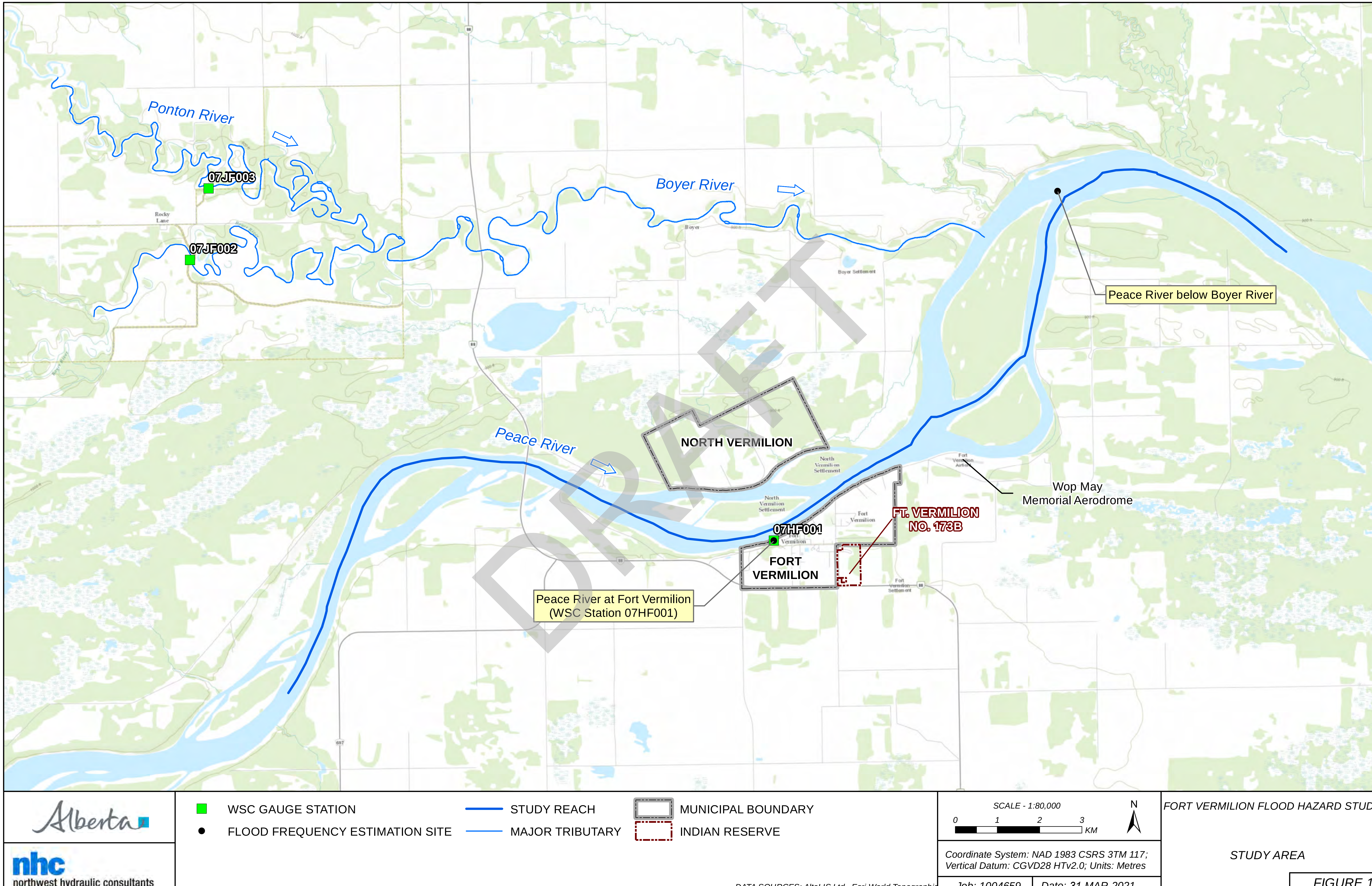
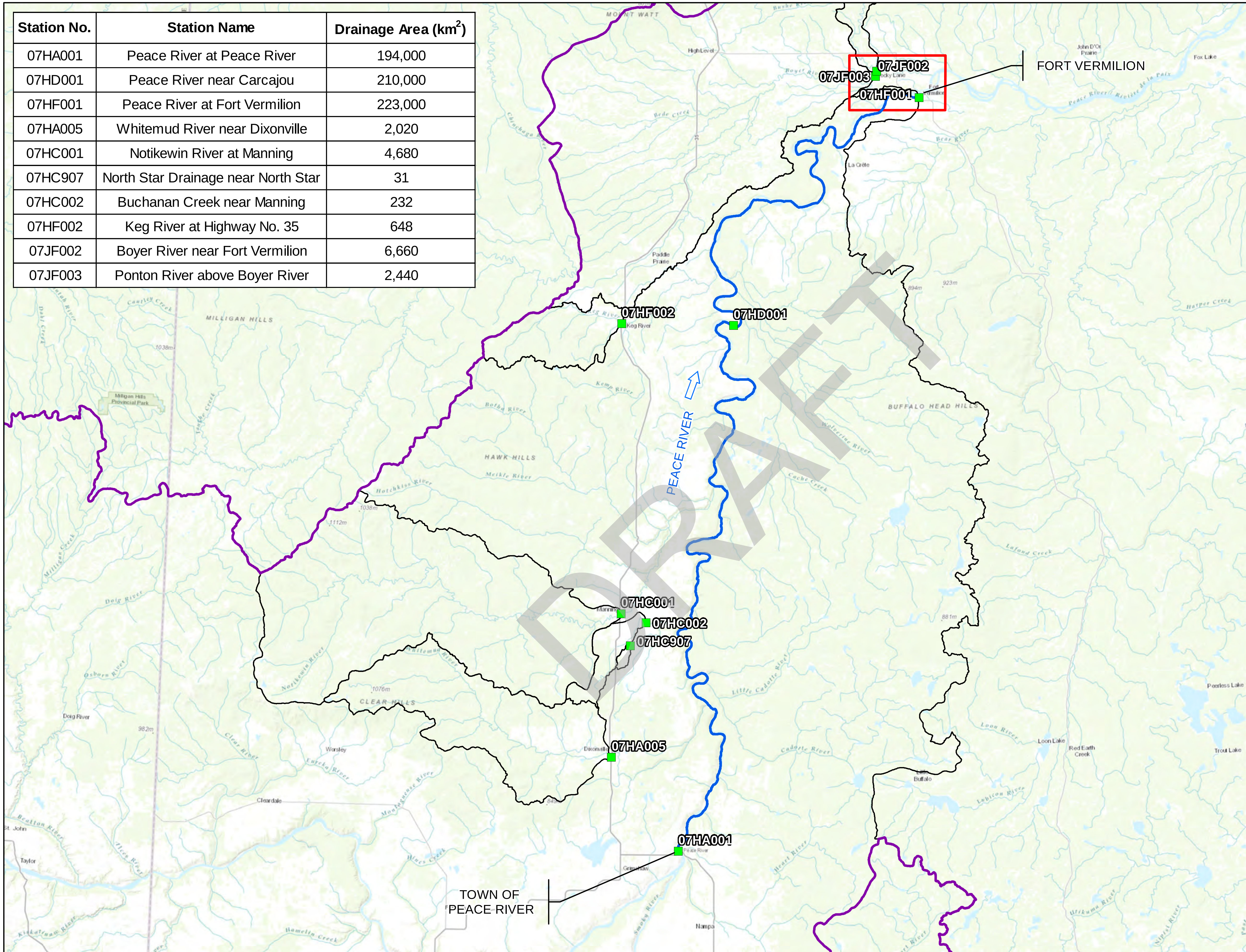
Figures



Classification: Public

Station No.	Station Name	Drainage Area (km^2)
07HA001	Peace River at Peace River	194,000
07HD001	Peace River near Carcajou	210,000
07HF001	Peace River at Fort Vermilion	223,000
07HA005	Whitemud River near Dixonville	2,020
07HC001	Notikewin River at Manning	4,680
07HC907	North Star Drainage near North Star	31
07HC002	Buchanan Creek near Manning	232
07HF002	Keg River at Highway No. 35	648
07JF002	Boyer River near Fort Vermilion	6,660
07JF003	Ponton River above Boyer River	2,440



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- WSC GAUGE STATION
- ROUTING REACH
- FLOOD HAZARD STUDY AREA
- PEACE RIVER BASIN
- GAUGED SUB-BASIN

DATA SOURCES: Esri National Geographic (primary map) and World Topographic Basemaps (inset)

SCALE - 1:1,150,000
0 10 20 40 KM
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

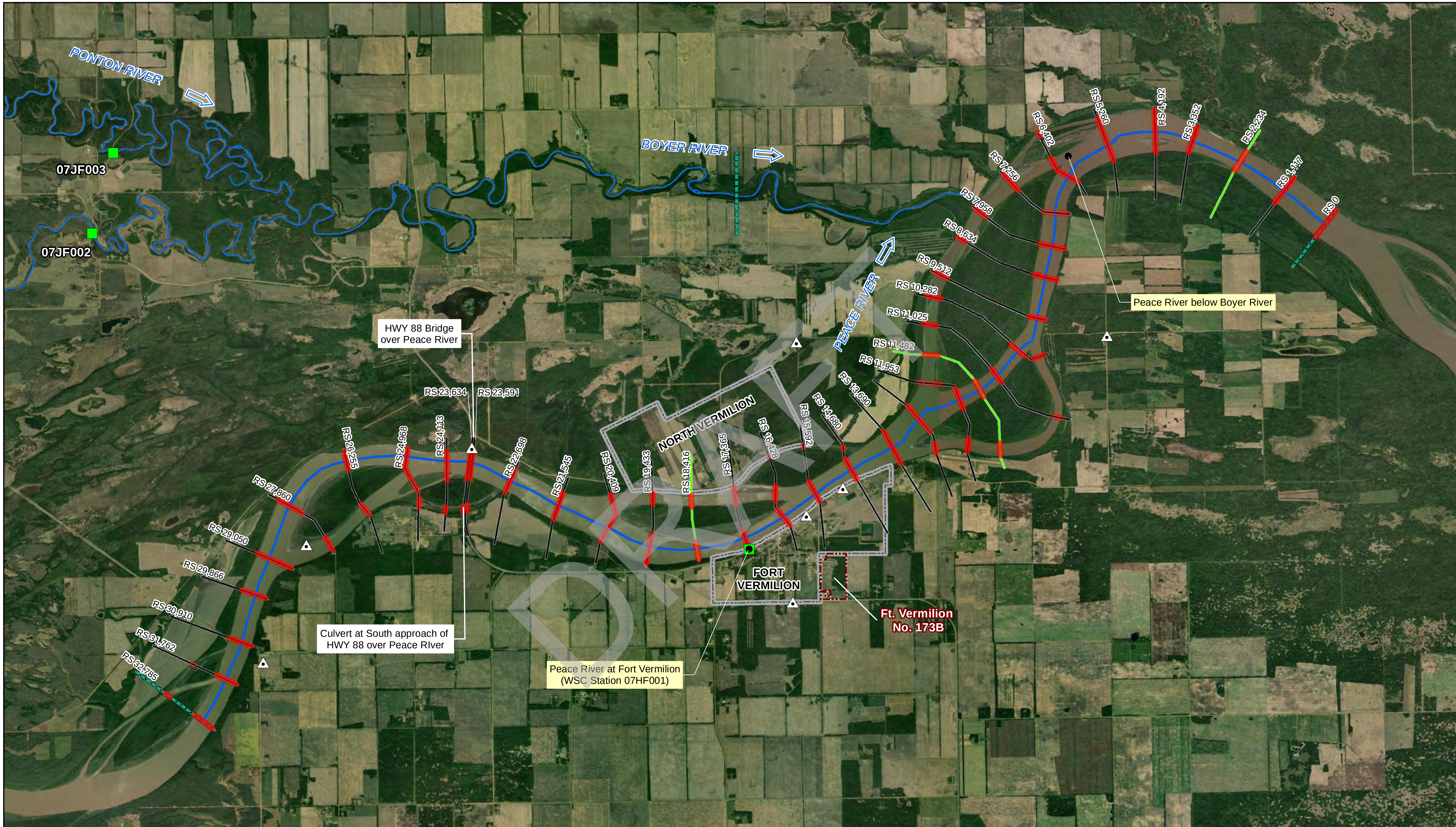
Job: 1004659 | *Date: 21-APR-2020*

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER HYDROLOGY ASSESSMENT AREA

FIGURE 2

REH, P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\90 GIS\1004659_REH_Fig_T200_SummaryReport_ProjectLoc.mxd



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CONTROL POINT	CULVERT
FLOOD FREQUENCY ESTIMATION SITE	BRIDGE
WSC GAUGE STATION	CROSS SECTION WITH DISCHARGE MEASUREMENT
SURVEY POINT	STUDY LIMIT
	CROSS SECTION

STUDY REACH
MAJOR TRIBUTARY
INDIAN RESERVE
MUNICIPAL BOUNDARY

DATA SOURCES: AltaLIS Ltd. & Esri World Imagery

SCALE - 1:75,000

0 1 2 3 KM

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Job: 1004659 Date: 31-MAR-2021

FORT VERMILION FLOOD HAZARD STUDY

SURVEY PLAN

FIGURE 3

REH, P:_Projects (Active)\1006302 Flood Hazard Study Updates\700 - Fort Vermilion\GIS\1006302_REH_Fig_SurveyPlan.mxd

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HIGHWATER MARK LOCATION
- 1990 OPEN WATER FLOOD
- 2018 ICE JAM FLOOD
- 2020 ICE JAM FLOOD

CULVERT
BRIDGE
STUDY LIMIT
MODEL CROSS SECTION

STUDY REACH
MAJOR TRIBUTARY
INDIAN RESERVE
MUNICIPAL BOUNDARY

DATA SOURCES: AltaLIS Ltd. & Esri World Imagery

SCALE - 1:75,000
0 1 2 3 KM
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Job: 1004659 Date: 20-APR-2021

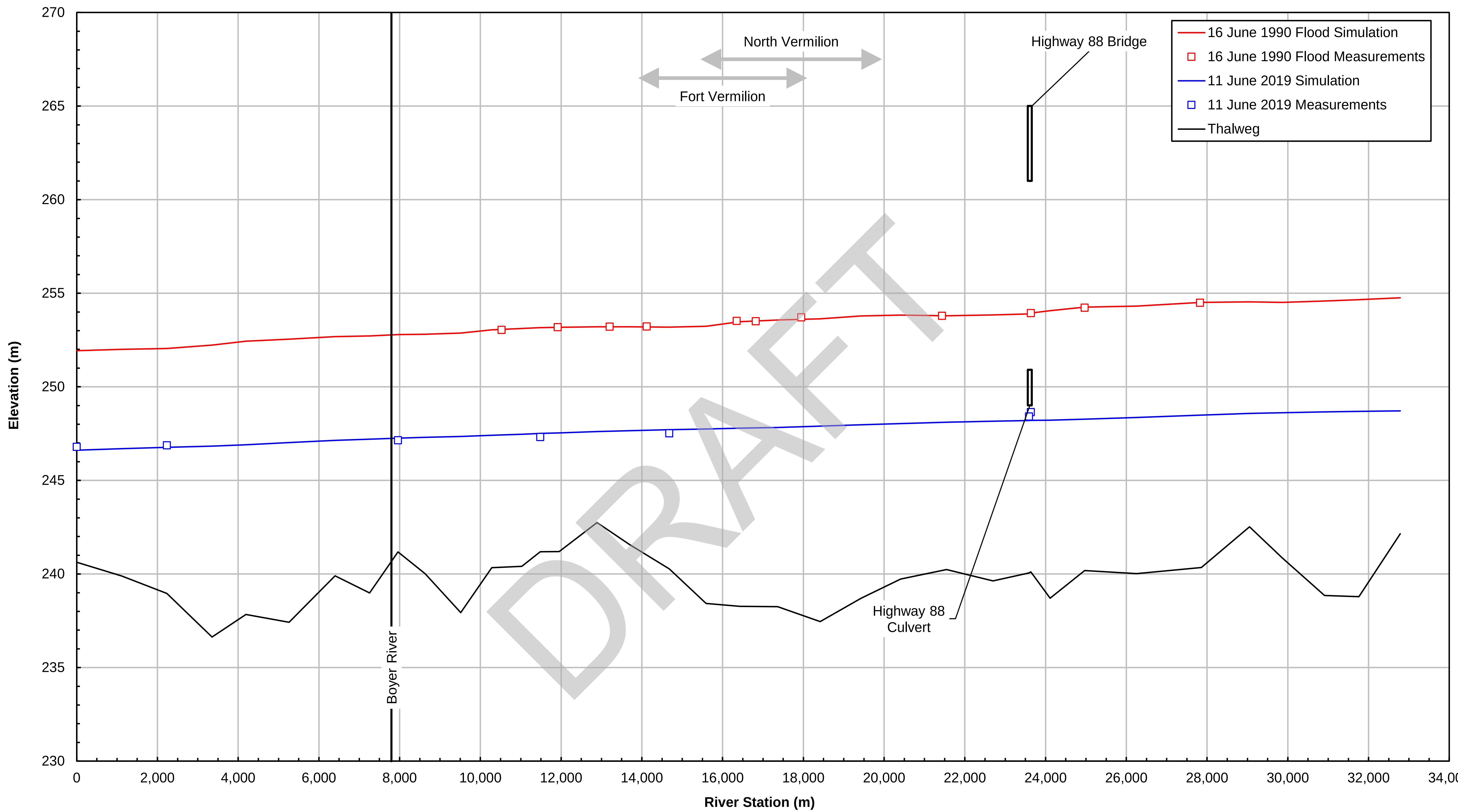
FORT VERMILION FLOOD HAZARD STUDY

HIGHWATER MARK LOCATIONS

FIGURE 4

REH, P:_Projects (Active)\1006302 - Fort Vermilion\GIS\1006302_REH_Fig_HWM.mxd

Classification: Public



SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659

Date: 31 Mar 2021

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER CALIBRATION PROFILES

FIGURE 5

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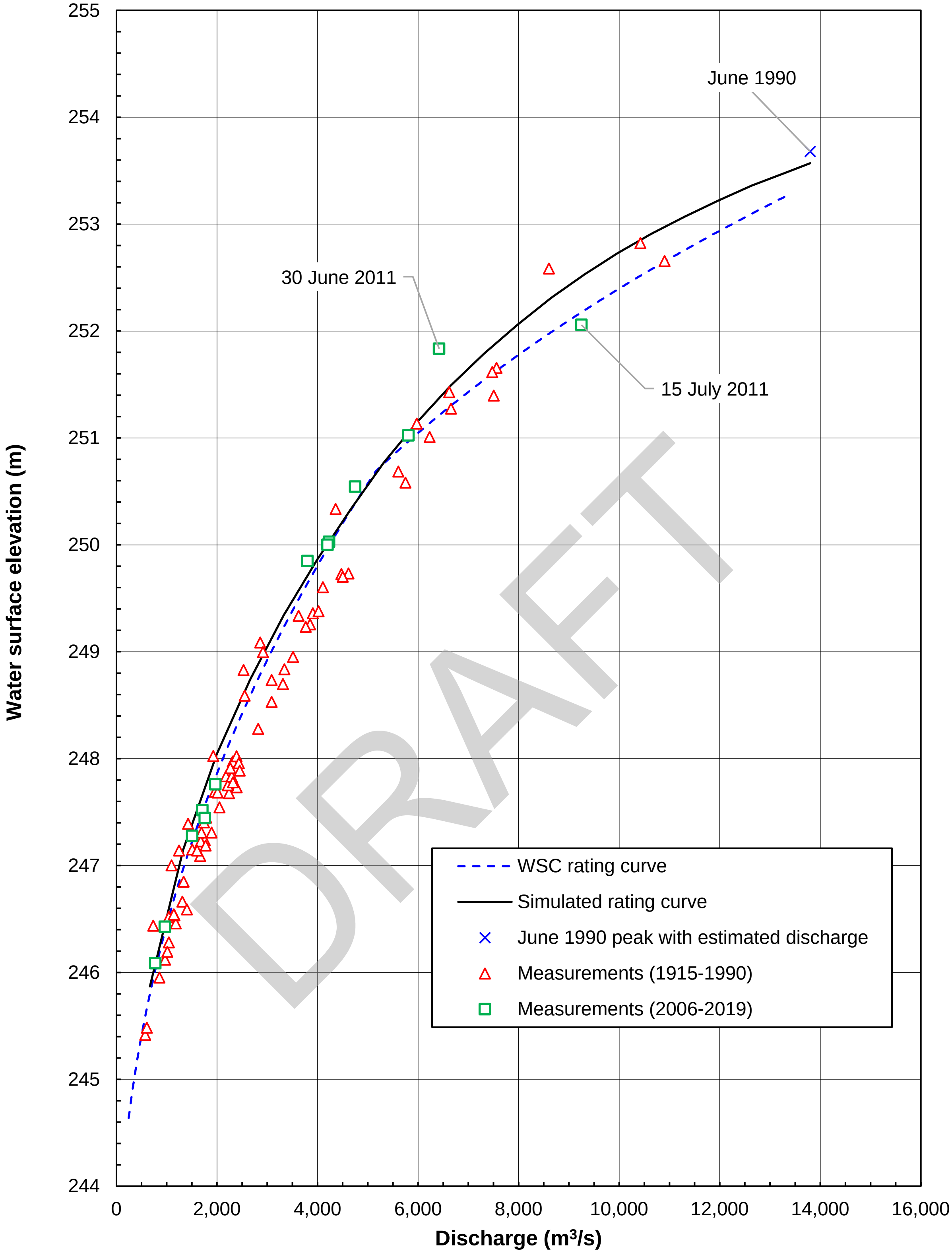
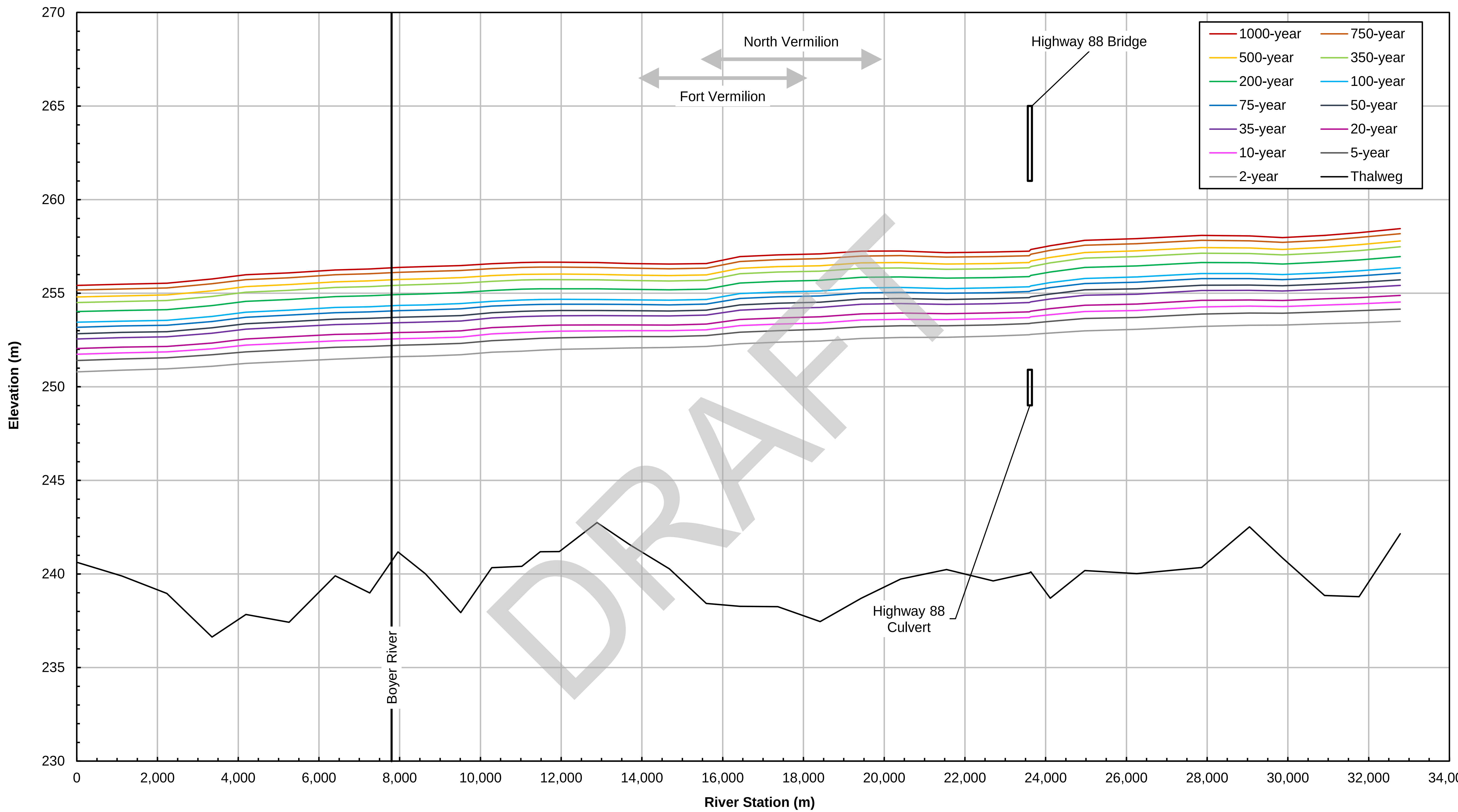


Alberta	SCALE – AS SHOWN	*FORT VERMILION FLOOD HAZARD STUDY*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	*OPEN WATER SYNTHETIC RATING CURVES ON PEACE RIVER AT FORT VERMILION (07HF001)*
	Job: 1004659 | *Date: 31-Mar-2021*	*FIGURE 6*



SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659

Date: 31 Mar 2021

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER FLOOD FREQUENCY PROFILES

FIGURE 7

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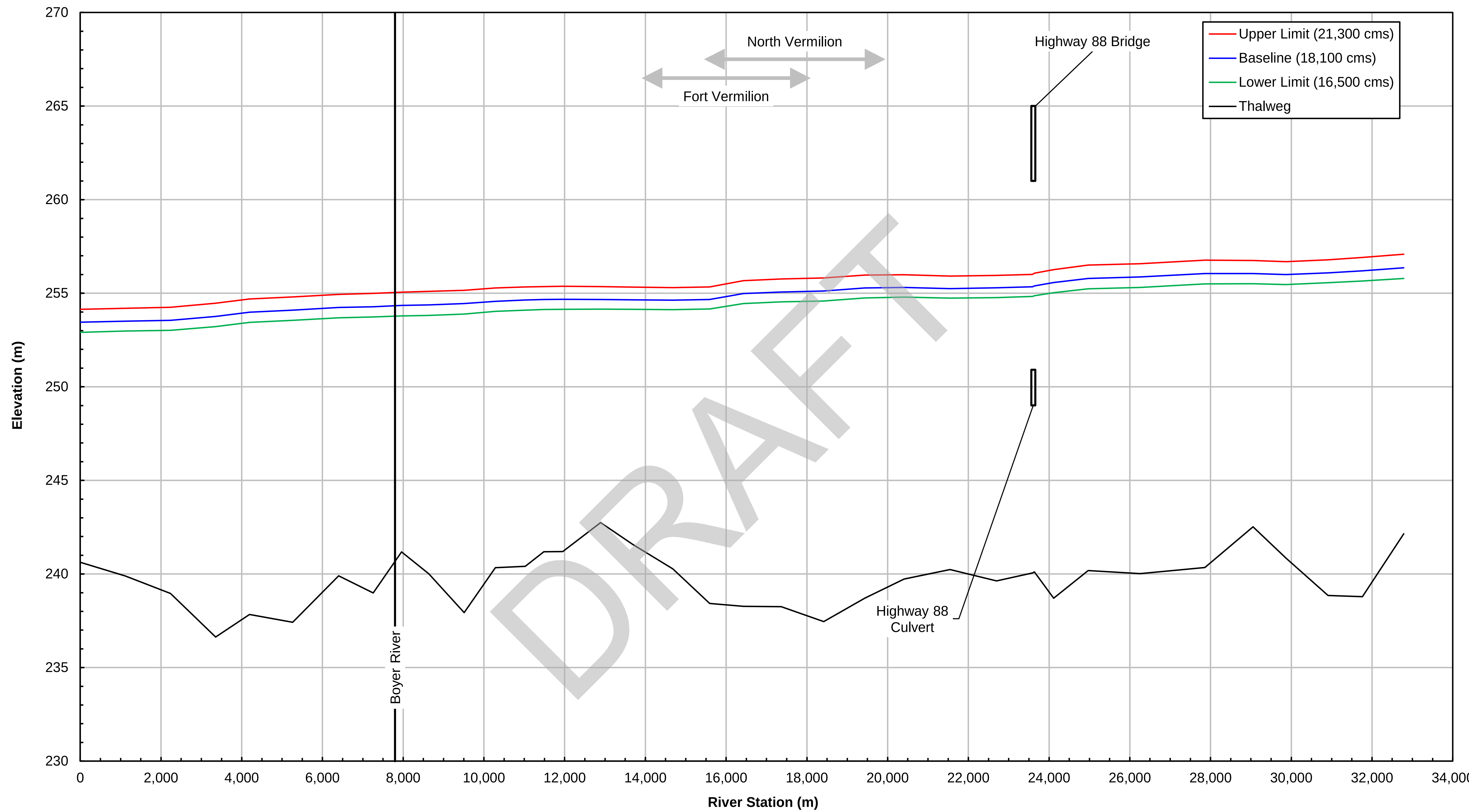
Classification: Public



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SCALE – AS SHOWN

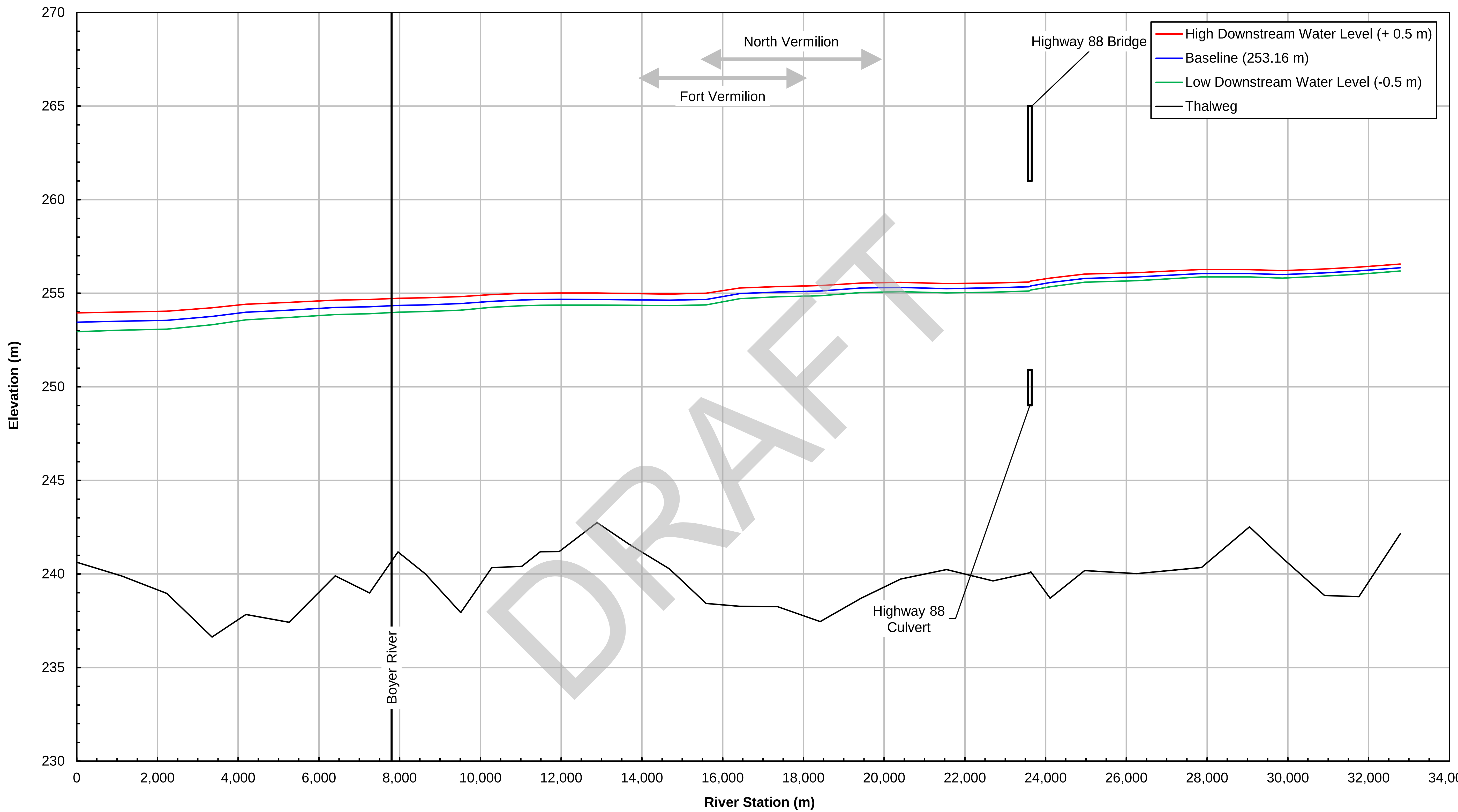
Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER SENSITIVITY ANALYSIS PROFILE - UPSTREAM BOUNDARY DISCHARGE

FIGURE 8



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SCALE – AS SHOWN

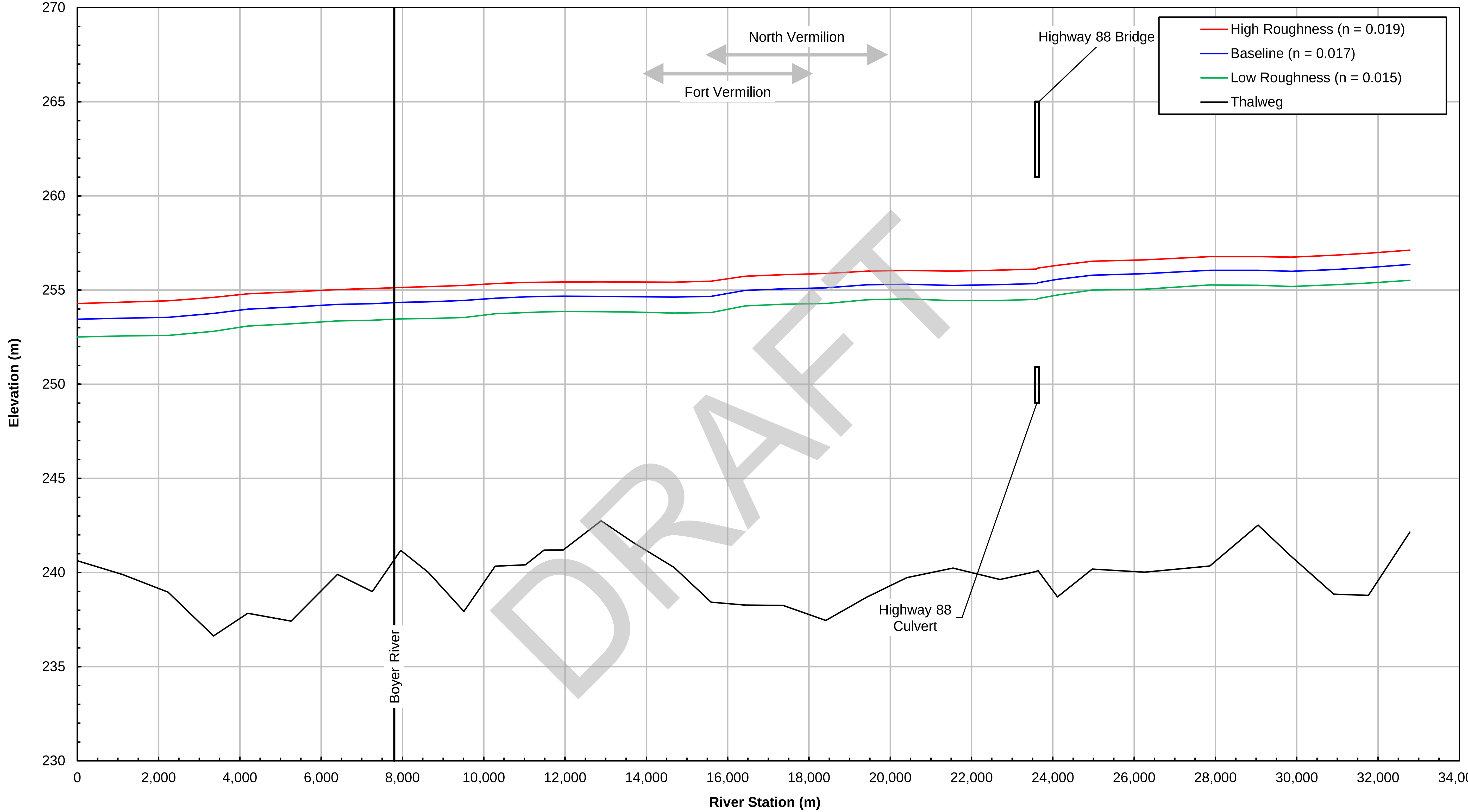
Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER SENSITIVITY ANALYSIS PROFILE – DOWNSTREAM BOUNDARY WATER LEVEL

Job: 01004659	*Date: 31 Mar 2021*

FIGURE 9



SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER SENSITIVITY ANALYSIS PROFILES – CHANNEL ROUGHNESS COEFFICIENT

FIGURE 10

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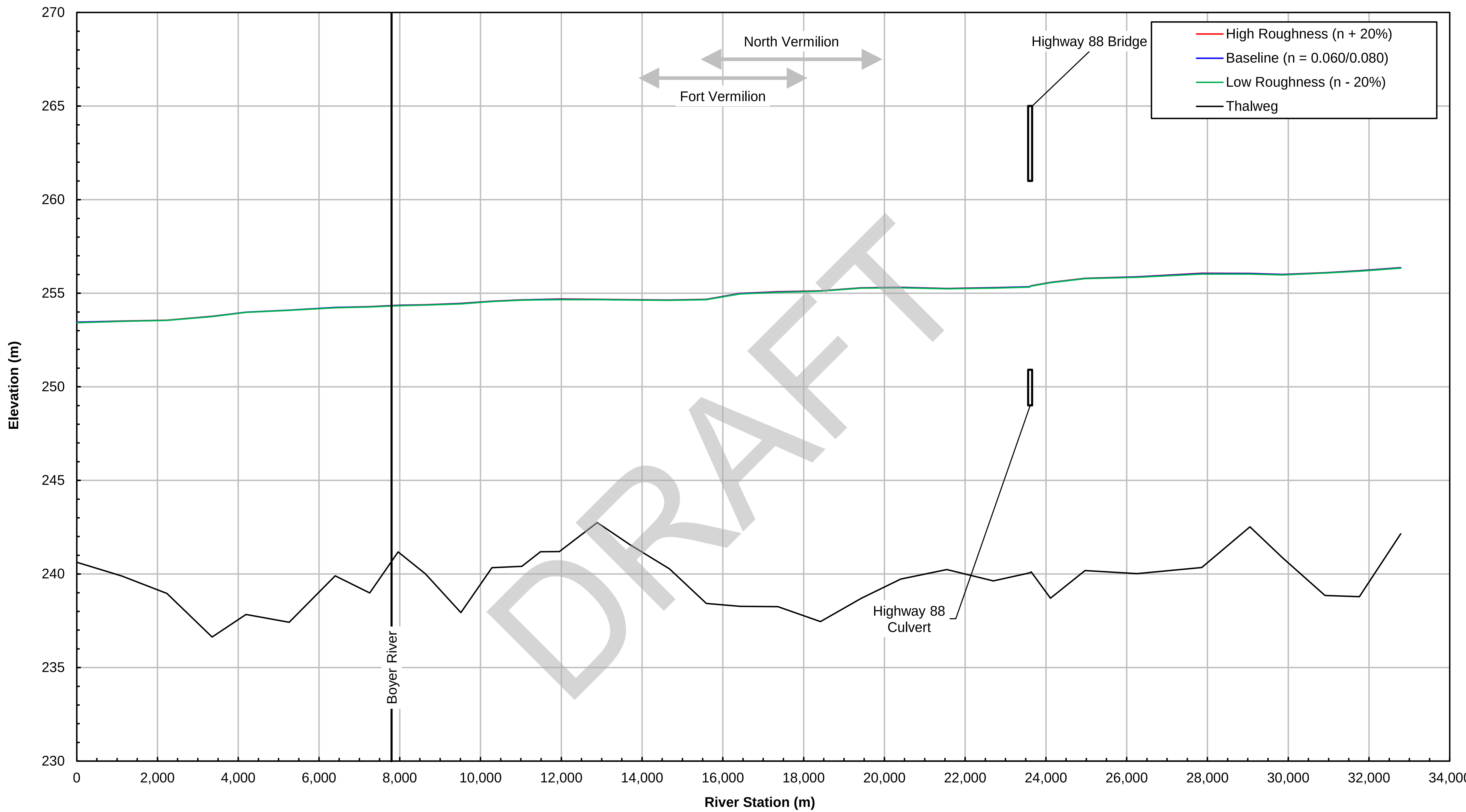

SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER SENSITIVITY ANALYSIS PROFILES – OVERBANK ROUGHNESS COEFFICIENT

FIGURE 11

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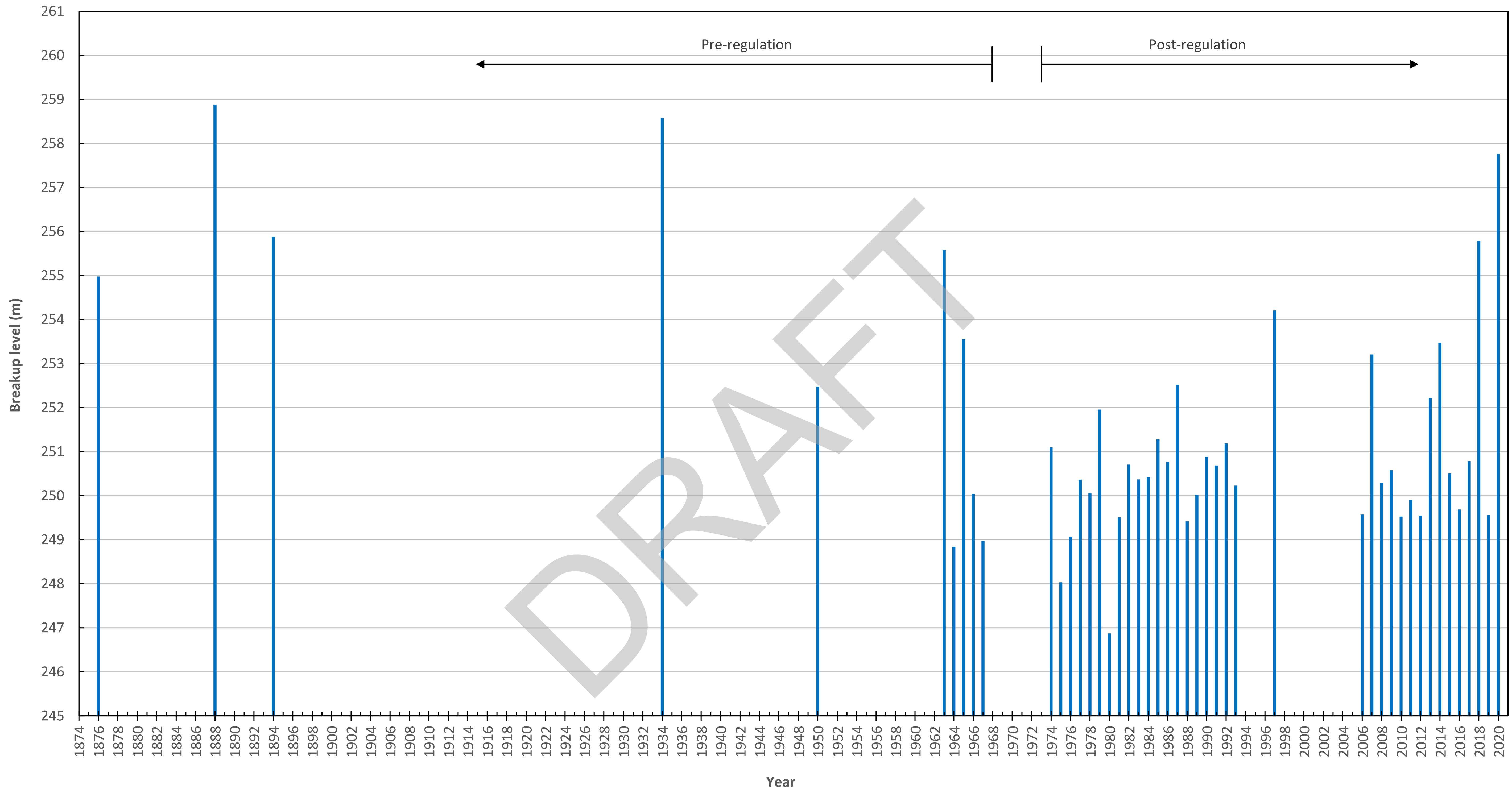
Classification: Public



Notes: 1. The pre-regulation period ended in 1967, and the post-regulation period began in 1972. The 1968-1971 period was the reservoir filling period

SCALE – AS SHOWN

Units: As Shown

Job: 01004659

Date: 31 Mar 2021

FORT VERMILION FLOOD HAZARD STUDY

PEAK BREAKUP WATER LEVELS FOR PEACE RIVER AT FORT VERMILION (WSC STATION 07HF001)

FIGURE 12

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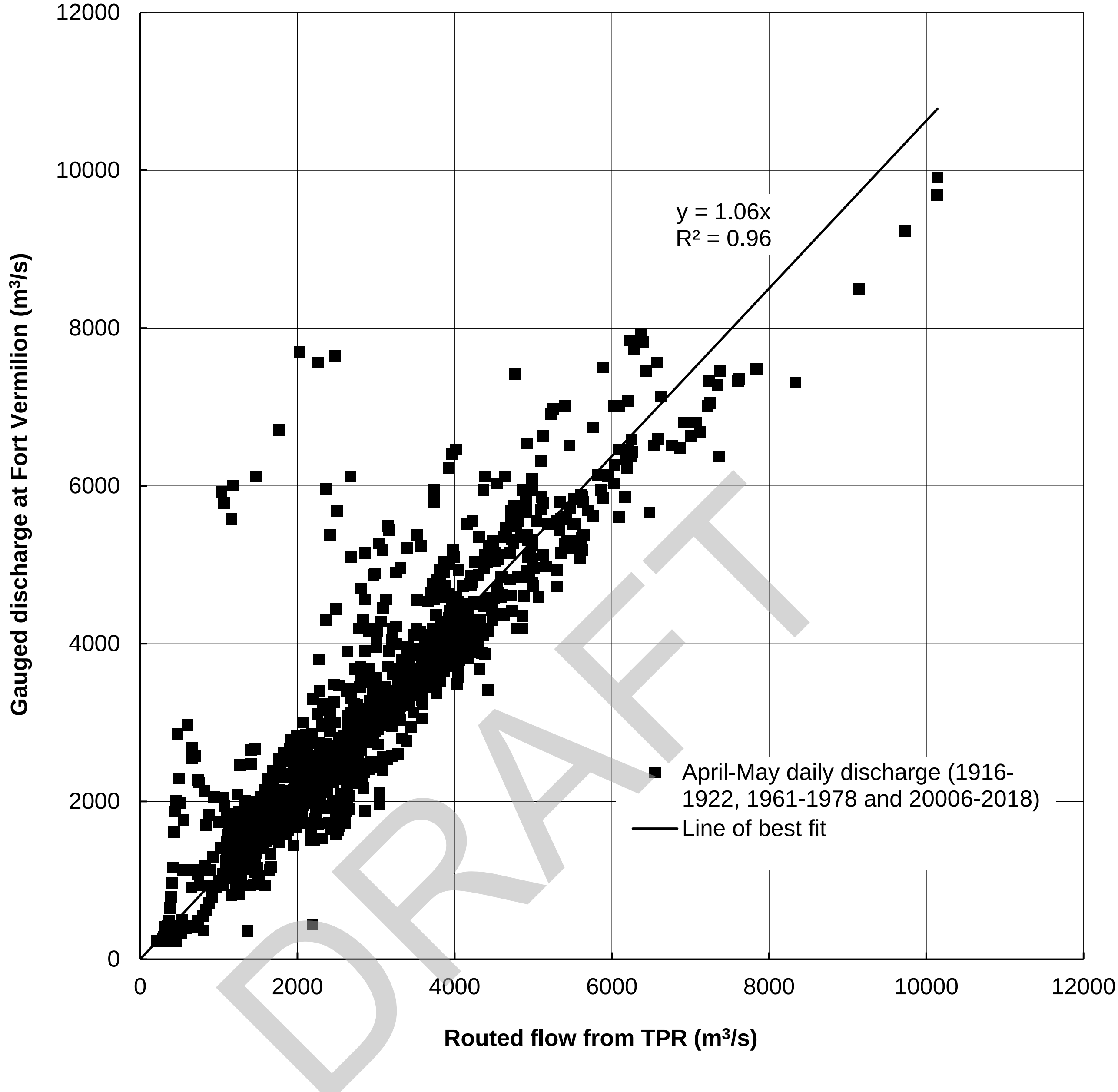

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SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1004659

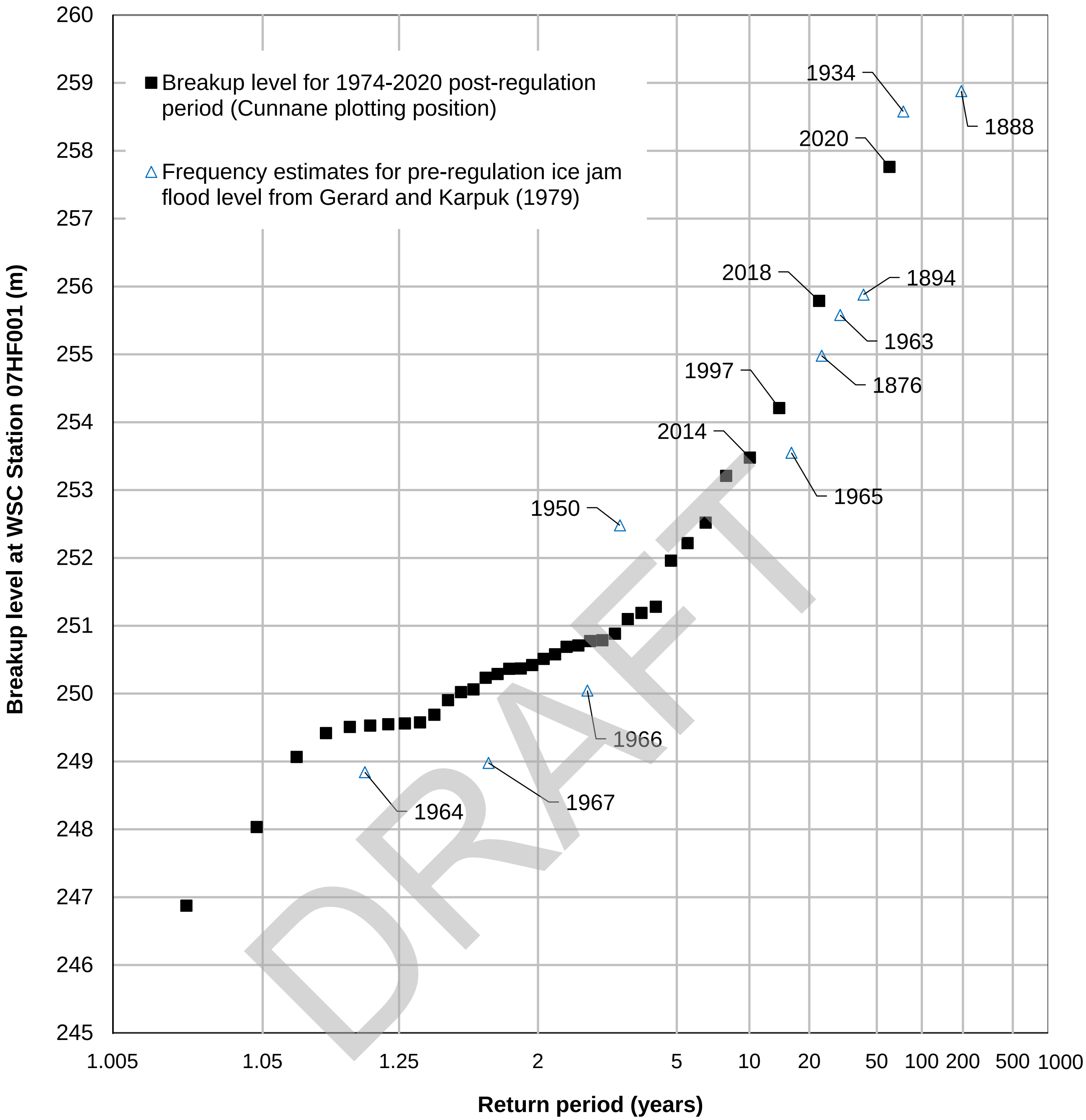
Date: 31-Mar-2021

FORT VERMILION FLOOD HAZARD STUDY

COMPARISON OF ROUTED AND GAUGED PEACE RIVER FLOWS IN APRIL AND MAY AT FORT VERMILION

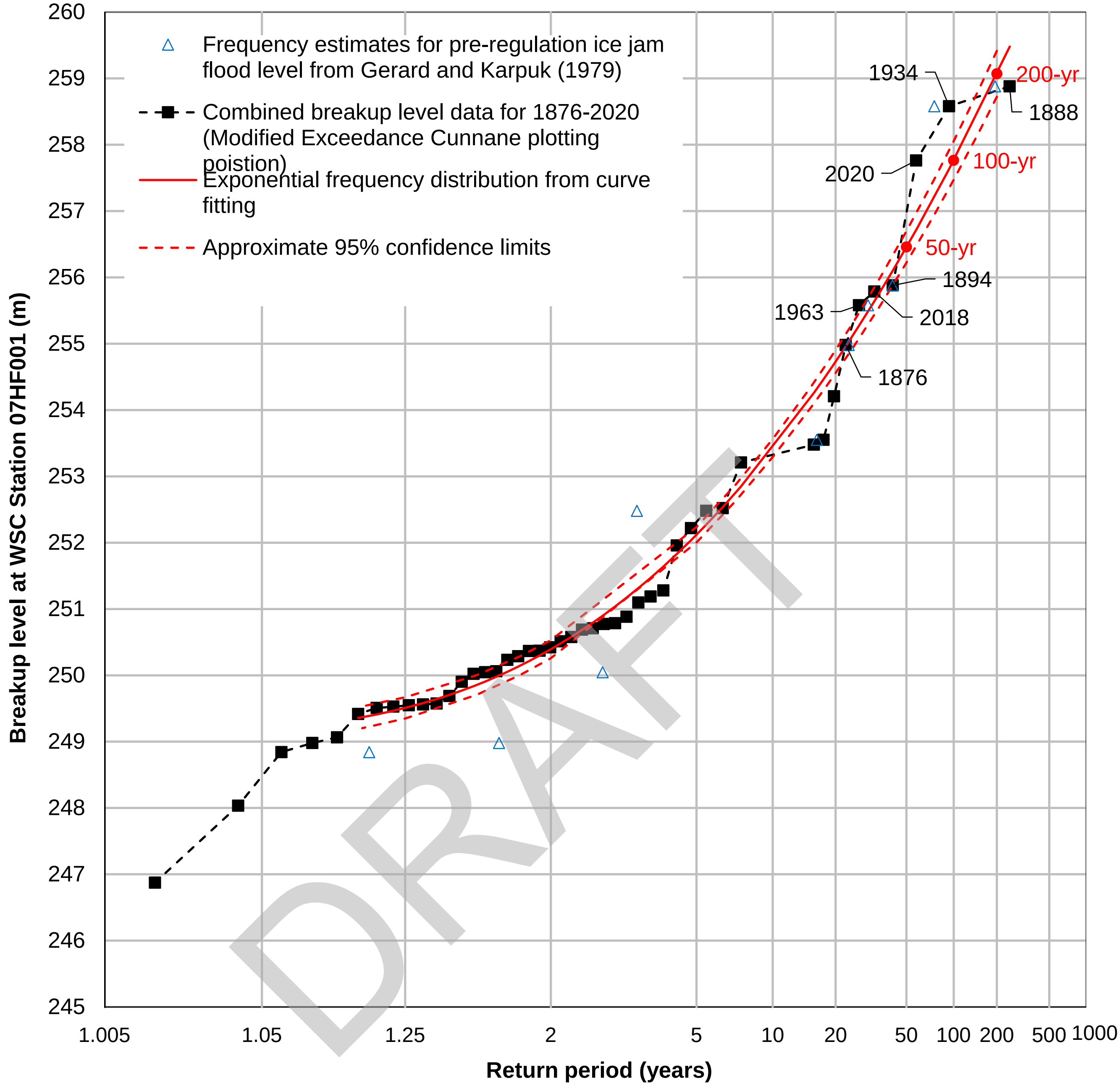
FIGURE 13

Classification: Public



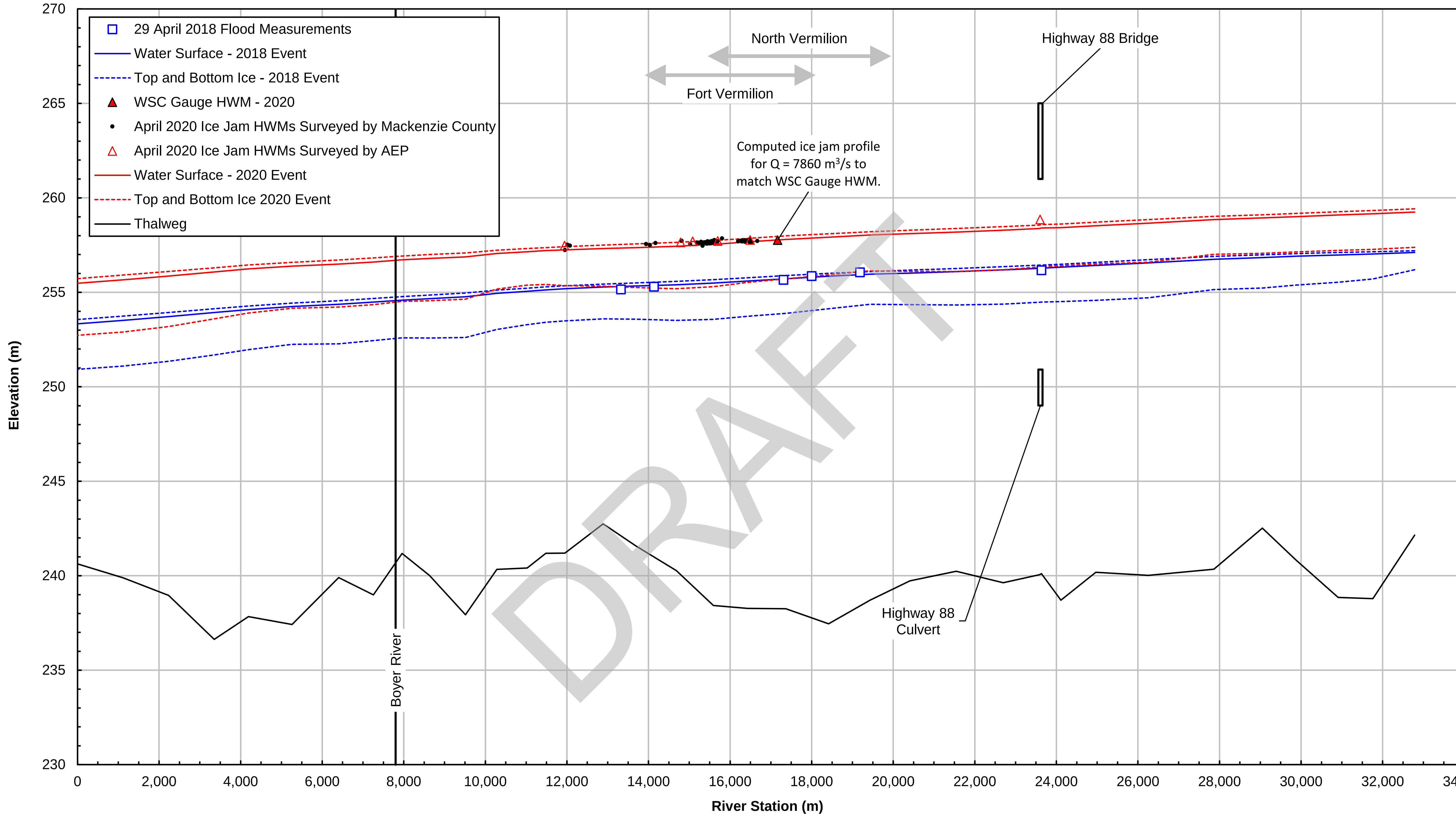
Notes: 1. Breakup water level data from Table 23
2. Plotting positions (return periods) for pre-regulation data from Gerard and Karpuk (1979)

Alberta	SCALE – AS SHOWN	FORT VERMILION FLOOD HAZARD STUDY
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	COMPARISON OF PRE- AND POST-REGULATION BREAKUP LEVEL FREQUENCY DISTRIBUTIONS (WSC STATION 07HF001)
Job: 1004659	Date: 31-Mar-2021	FIGURE 14



Notes:
1. Breakup water level data from Table 23
2. Plotting positions (return periods) for pre-regulation data from Gerard and Karpuk (1979)

Alberta	SCALE – AS SHOWN	FORT VERMILION FLOOD HAZARD STUDY
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	FREQUENCY DISTRIBUTION OF COMBINED BREAKUP LEVEL DATA (WSC STATION 07HF001)
	Job: 1004659	Date: 31-Mar-2021	FIGURE 15



Alberta

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SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

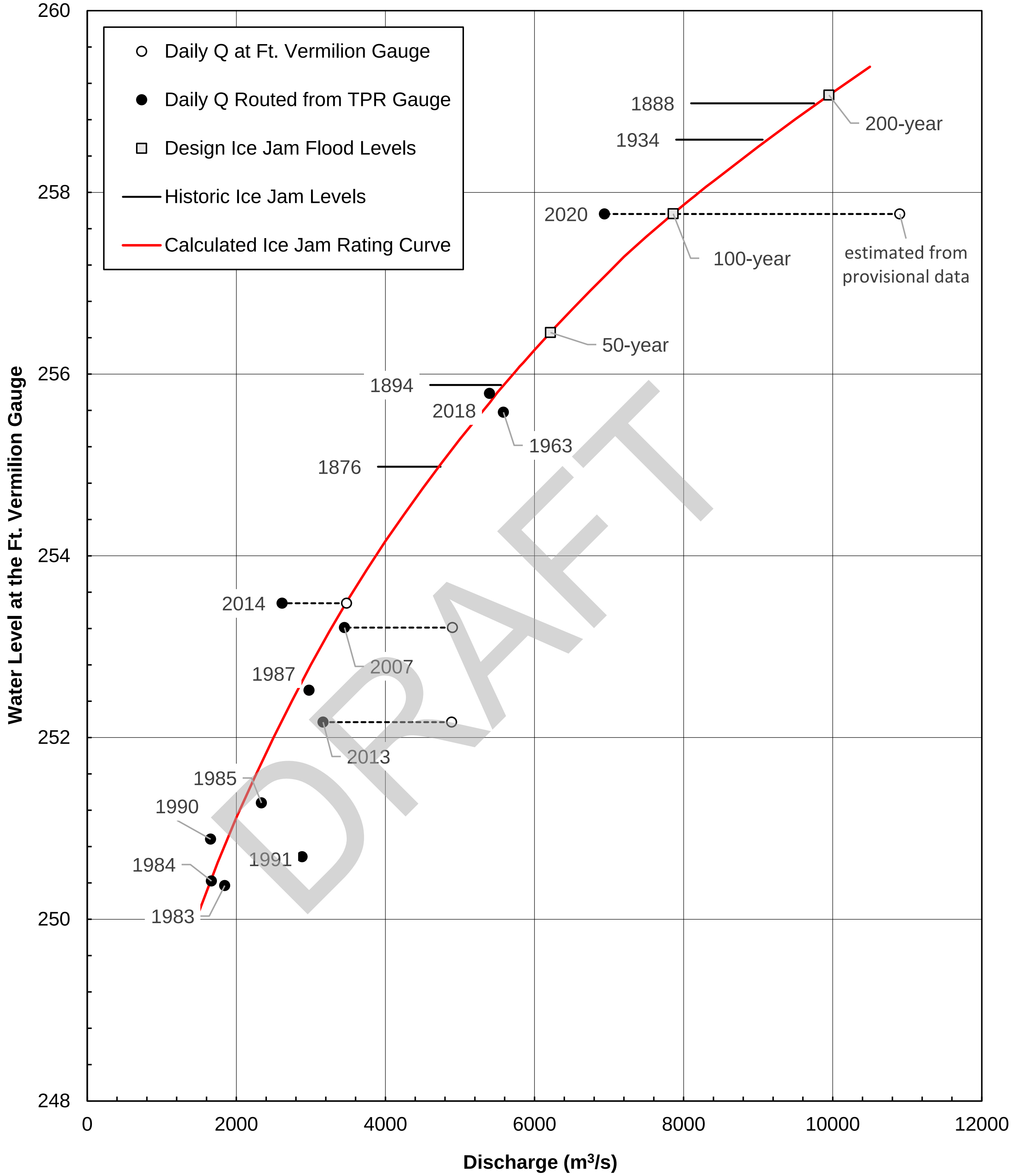
Job: 01004659

Date: 31 Mar 2021

FORT VERMILION FLOOD HAZARD STUDY

2018 ICE JAM CALIBRATION AND 2020 ICE JAM VALIDATION PROFILES

FIGURE 16



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1004659

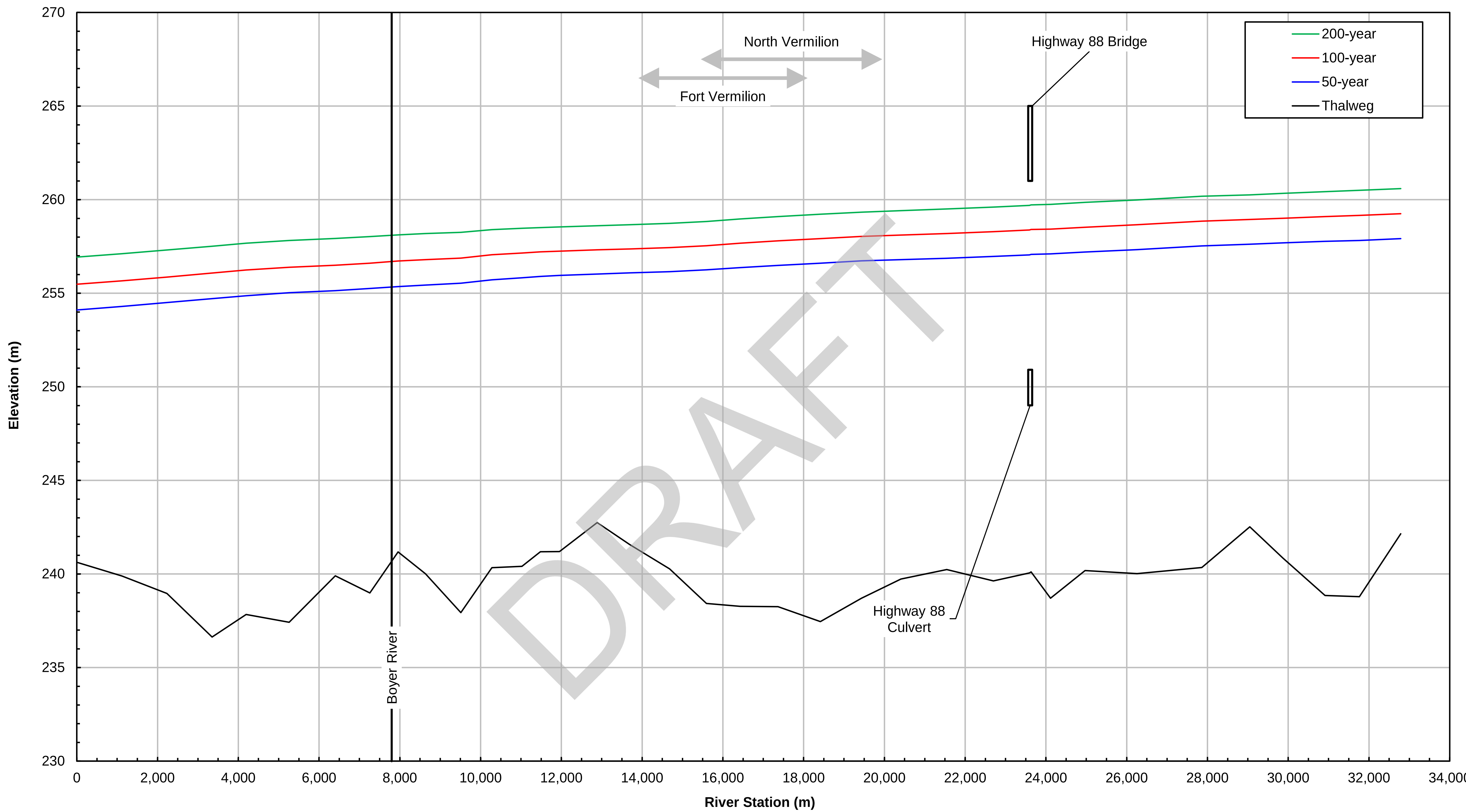
Date: 31-Mar-2021

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM RATING CURVE ON PEACE RIVER AT FORT VERMILION (07HF001)

FIGURE 17

northwest hydraulic consultants



SCALE – AS SHOWN

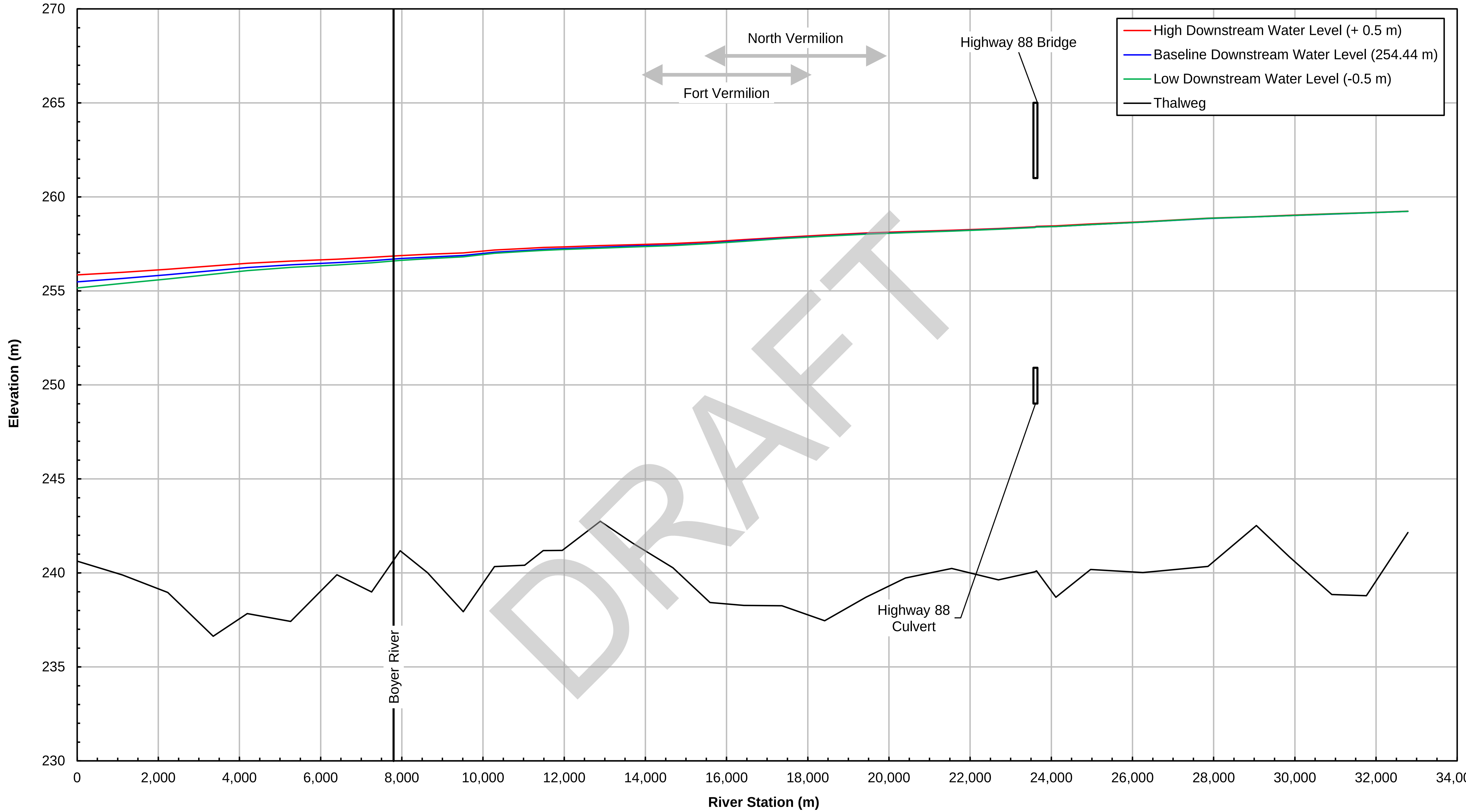
Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | Date: 31 Mar 2021

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOOD FREQUENCY PROFILES

FIGURE 18



SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

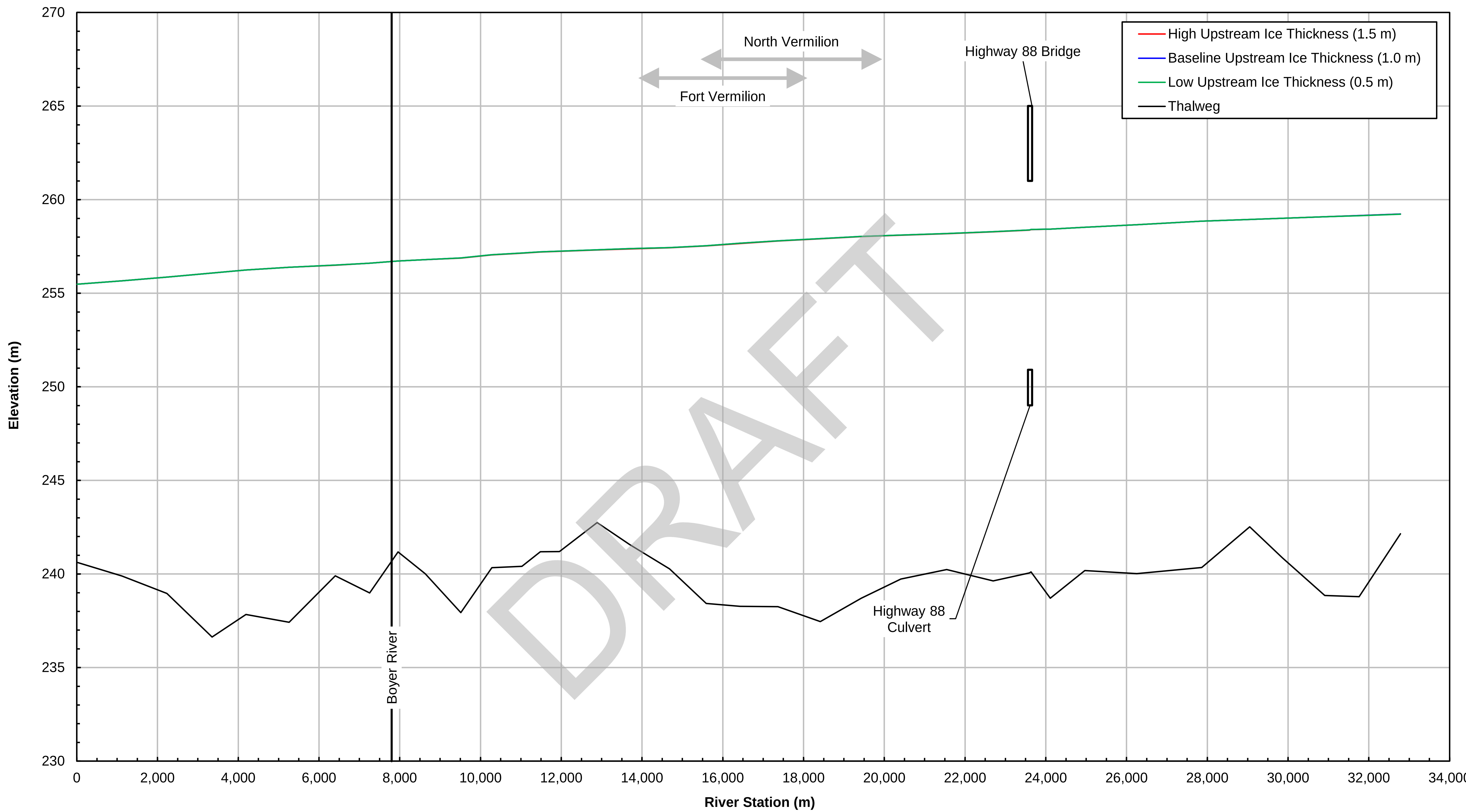
FORT VERMILION FLOOD HAZARD STUDY

SENSITIVITY OF COMPUTED 100-YEAR ICE JAM FLOOD LEVELS TO DOWNSTREAM WATER LEVEL

Job: 01004659 | *Date: 31 Mar 2021*

FIGURE 19

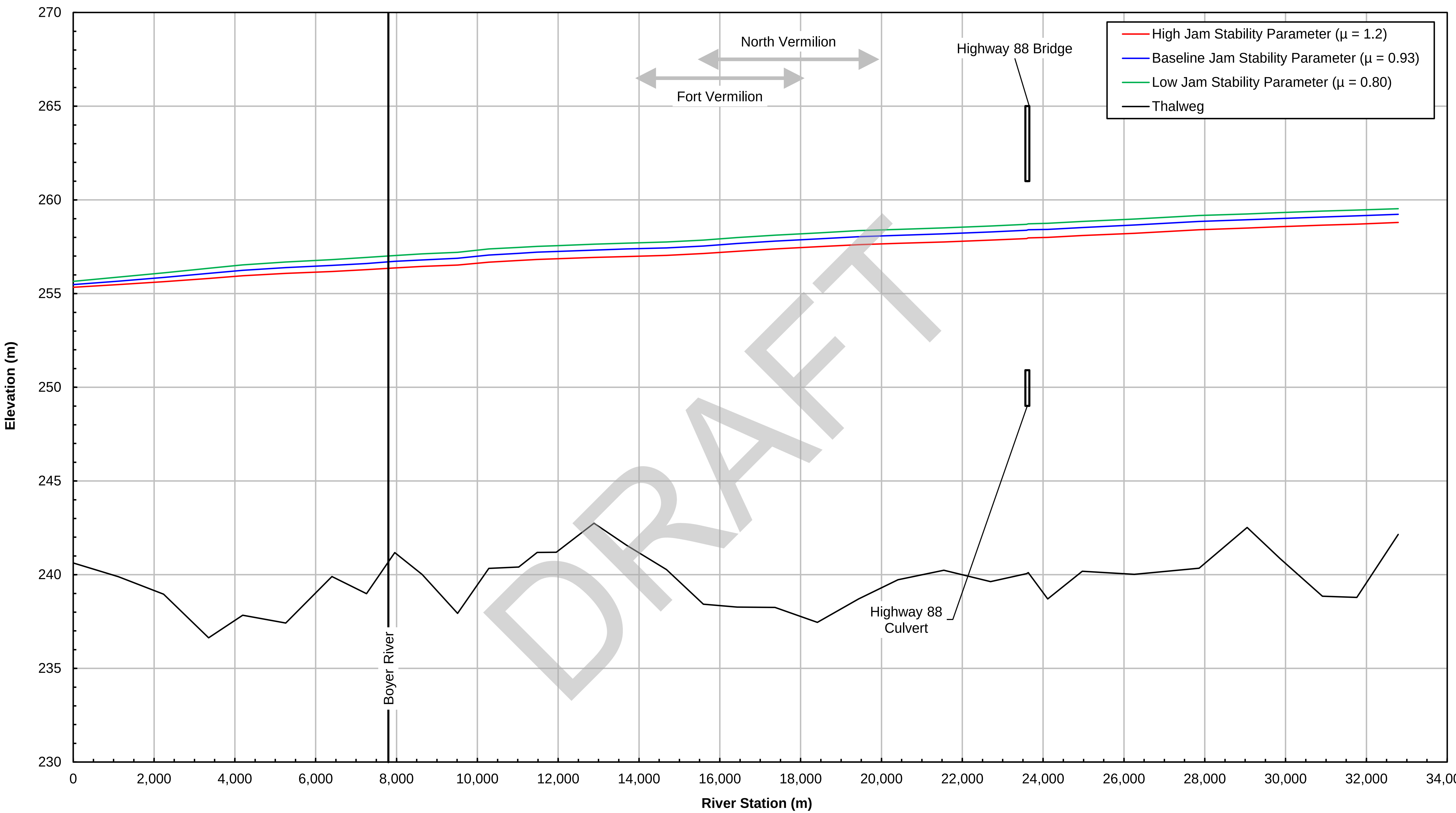
northwest hydraulic consultants



SCALE – AS SHOWN		FORT VERMILION FLOOD HAZARD STUDY
Elevation Datum: CGVD28 (HTv2.0) Units: As Shown		SENSITIVITY OF COMPUTED 100-YEAR ICE JAM FLOOD LEVELS TO UPSTREAM ICE THICKNESS
Job: 01004659	Date: 31 Mar 2021	FIGURE 20

Alberta

nhc northwest hydraulic consultants



Alberta

nhc northwest hydraulic consultants

SCALE – AS SHOWN

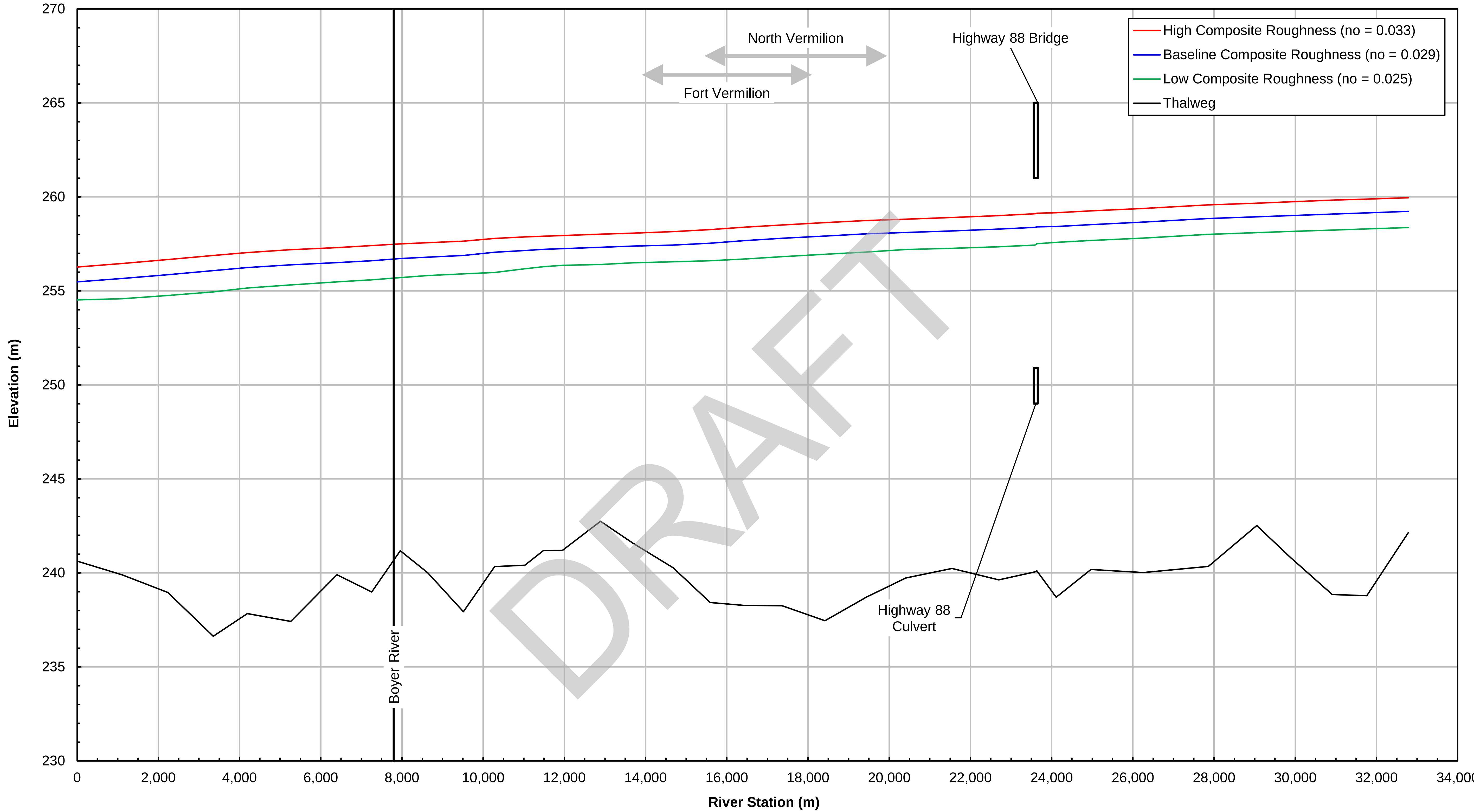
Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY

SENSITIVITY OF COMPUTED 100-YEAR ICE JAM FLOOD LEVELS TO JAM STABILITY PARAMETERS

FIGURE 21



northwest hydraulic consultants

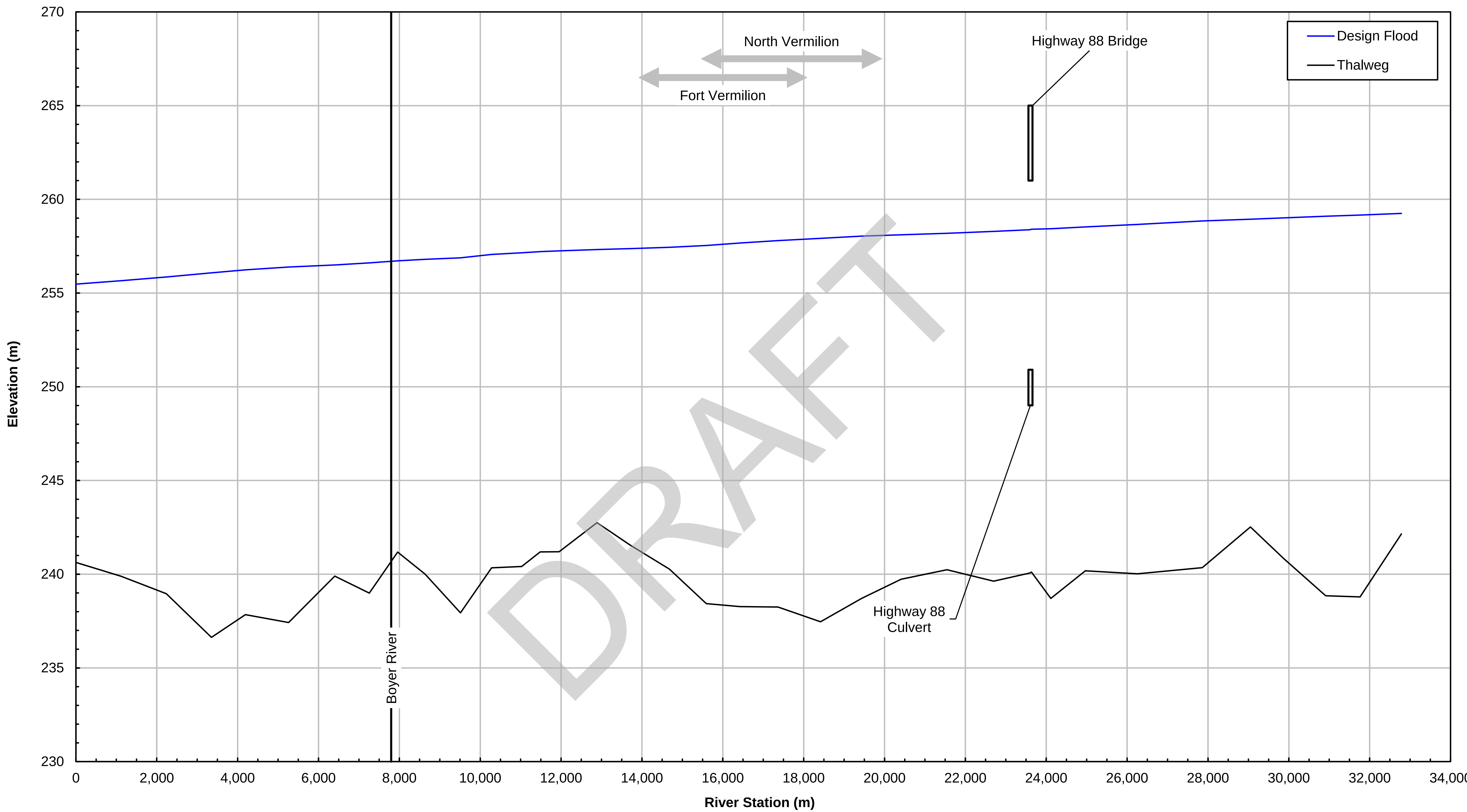
SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY SENSITIVITY OF COMPUTED 100-YEAR ICE JAM FLOOD LEVELS TO COMPOSITE ROUGHNESS COEFFICIENT

FIGURE 22



SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659

Date: 31 Mar 2021

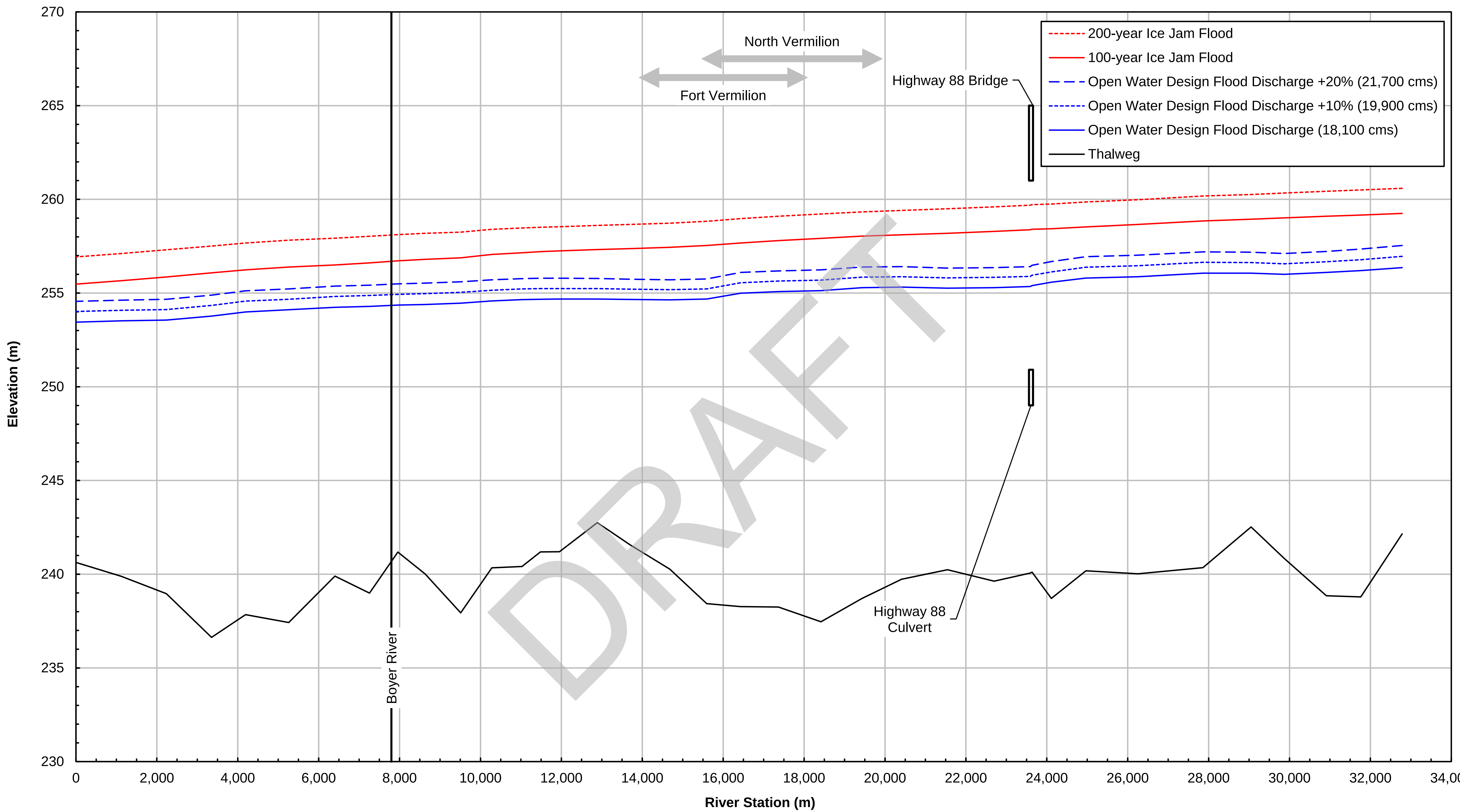
FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD PROFILE

FIGURE 23

northwest hydraulic consultants

Classification: Public



northwest hydraulic consultants

SCALE – AS SHOWN

Elevation Datum: CGVD28 (HTv2.0)
Units: As Shown

Job: 01004659 | *Date: 31 Mar 2021*

FORT VERMILION FLOOD HAZARD STUDY

COMPARISON OF DESIGN FLOODS AND POTENTIAL CLIMATE CHANGE FLOODS

FIGURE 24

Appendix A
Survey Data

Electronic File Submission

- Complete list of surveyed points (Fort Vermilion RHS Survey Data.xlsx)
- Geodatabase containing cross section, bridge, culvert, weir, and flood control structure alignments; infrastructure and site photographs (FortVermilionRHS_Survey.gdb)

Appendix B
Hydraulic Structure Details

Table B-1 Bridge details

Description	River Station (m)	Municipality	Design Drawing/Info	Span (m)	Width (m)	Number of Piers	Pier Width (m)	Deck Skew (°)	Pier Skew (°)	Minimum Elevation (m)		Low Flow Modelling Approach	High Flow Modelling Approach
										Top Chord	Low Chord		
Highway 88 Bridge	23,609	Mackenzie County	BF74227	510	10.7	4	3.25 m at elev. 244.0 m to 1.4 m at low chord	0	0	265.01	261.00	Energy	Energy

Table B-2 Culvert details

Description	River Station (m)	Municipality	Design Drawing/Info	Culvert Shape	Culvert Type	Entrance Condition	Number of Barrel	Barrel Length (m)	Diameter, Rise, or Height (m)	Span or Width (m)	Upstream Invert Elevation (m)	Downstream Invert Elevation (m)	Loss Coefficient		Manning's n	
													Entrance	Exit	Top	Bottom
Highway 88 Culvert	23,609	Mackenzie County	BF77452	Ellipse	CSP	Pipe projecting from fill	1	42	1.901	1.724	249.176	248.839	0.9	1.0	0.02	0.02

Appendix C
Discharge Measurement Summary

PXS3 – R.S. 2,234

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information

Site Name	Fort Vermilion
Station Number	1
Location	XS-3

Measurement Information

Party	JM/AV
Boat/Motor	Harbercraft
Meas. Number	1

System Information

System Type	RS-M9
Serial Number	3790
Firmware Version	4.10
Software Version	4.1

System Setup

Transducer Depth (m)	0.21
Screening Distance (m)	0.00
Salinity (ppt)	0.0
Magnetic Declination (deg)	15.4

Units

Distance	m
Velocity	m/s
Area	m2
Discharge	m3/s
Temperature	degC

Discharge Calculation Settings

Track Reference	Bottom-Track	Left Method	Sloped Bank
Depth Reference	Vertical Beam	Right Method	Sloped Bank
Coordinate System	ENU	Top Fit Type	Power Fit
		Bottom Fit Type	Power Fit
		Start Gauge Height (m)	0.00
		End Gauge Height (m)	0.00

Discharge Results

Width (m)	434.048
Area (m2)	2,537.765
Mean Speed (m/s)	0.719
Total Q (m3/s)	1,824.490
Maximum Measured Depth	7.967
Maximum Measured Speed	1.594

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge							%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal	Measured
1	L	10:02:30 AM	0:05:10	14.6	436.46	421.83	428.825	2,527.497	1.408	0.722	0.00	0.13	173.20	1,379.40	271.85	1,824.579	--	75.6
2	R	10:09:16 AM	0:05:20	14.4	441.85	428.68	435.679	2,547.759	1.381	0.712	0.02	0.15	170.68	1,366.67	276.88	1,814.408	--	75.3
3	L	10:14:55 AM	0:05:58	14.5	448.14	425.77	432.771	2,525.076	1.252	0.734	0.06	0.13	176.31	1,390.12	286.12	1,852.746	--	75.0
4	R	10:20:56 AM	0:04:52	14.3	446.02	431.92	438.915	2,550.726	1.527	0.708	0.03	0.09	170.41	1,353.76	281.95	1,806.226	--	74.9
			Mean	14.4	443.12	427.05	434.048	2,537.765	1.392	0.719	0.03	0.12	172.65	1,372.49	279.20	1,824.490	0.000	75.2
			Std Dev	0.1	4.46	3.72	3.717	11.557	0.098	0.010	0.02	0.02	2.38	13.63	5.36	17.561	0.000	0.3
			COV	0.0	0.010	0.009	0.009	0.005	0.070	0.014	0.794	0.193	0.014	0.010	0.019	0.010	0.000	0.003

Exposure Time: 0:21:20

Tr1=20190611100230r.rivr; Tr2=20190611100920r.rivr; Tr3=20190611101502r.rivr; Tr4=20190611102107r.rivr;

Comments

Tr1=20190611100230r.rivr - 1; Tr2=20190611100920r.rivr - 1; Tr3=20190611101502r.rivr - 1; Tr4=20190611102107r.rivr - 1;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

PXS14 – R.S. 11,482

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information		Measurement Information	
Site Name	Fort Vermilion	Party	JM/AV
Station Number	3	Boat/Motor	Harbercraft
Location	XS-14	Meas. Number	3

System Information		System Setup		Units	
System Type	RS-M9	Transducer Depth (m)	0.21	Distance	m
Serial Number	3790	Screening Distance (m)	0.00	Velocity	m/s
Firmware Version	4.10	Salinity (ppt)	0.0	Area	m2
Software Version	4.1	Magnetic Declination (deg)	15.4	Discharge	m3/s
				Temperature	degC

Discharge Calculation Settings				Discharge Results	
Track Reference	Bottom-Track	Left Method	Sloped Bank	Width (m)	364.700
Depth Reference	Vertical Beam	Right Method	Sloped Bank	Area (m2)	1,676.953
Coordinate System	ENU	Top Fit Type	Power Fit	Mean Speed (m/s)	0.811
		Bottom Fit Type	Power Fit	Total Q (m3/s)	1,360.357
		Start Gauge Height (m)	0.00	Maximum Measured Depth	6.229
		End Gauge Height (m)	0.00	Maximum Measured Speed	1.895

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge								%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal		Measured
1	R	11:40:06 AM	0:04:15	14.7	362.58	356.41	365.414	1,674.898	1.422	0.810	0.16	1.97	149.42	990.52	214.84	1,356.905	–		73.0
2	L	11:44:30 AM	0:04:02	14.6	367.42	355.21	364.208	1,674.553	1.518	0.817	0.00	2.43	151.04	993.11	221.15	1,367.727	–		72.6
3	R	11:48:41 AM	0:04:26	14.5	363.99	356.02	365.021	1,687.672	1.368	0.801	0.18	2.29	147.73	984.30	217.83	1,352.331	–		72.8
4	L	11:53:15 AM	0:04:06	14.4	361.87	355.16	364.159	1,670.689	1.471	0.817	0.00	2.91	150.43	991.07	220.06	1,364.466	–		72.6
			Mean	14.5	363.97	355.70	364.700	1,676.953	1.445	0.811	0.08	2.40	149.66	989.75	218.47	1,360.357	0.000		72.8
			Std Dev	0.1	2.14	0.54	0.536	6.405	0.056	0.006	0.08	0.34	1.26	3.29	2.41	6.073	0.000		0.2
			COV	0.0	0.006	0.002	0.001	0.004	0.039	0.008	1.002	0.140	0.008	0.003	0.011	0.004	0.000		0.002

Exposure Time: 0:16:49

Tr1=20190611114035r.rivr; Tr2=20190611114502r.rivr; Tr3=20190611114915r.rivr; Tr4=20190611115353r.rivr;

Comments

Tr1=20190611114035r.rivr - 3; Tr2=20190611114502r.rivr - 3; Tr3=20190611114915r.rivr - 3; Tr4=20190611115353r.rivr - 3;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

DRAFT

PXS14L – R.S. 11,482

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information	
Site Name	Fort Vermilion
Station Number	6
Location	XSL-22L

Measurement Information	
Party	JM/AV
Boat/Motor	Harbercraft
Meas. Number	6

System Information	
System Type	RS-M9
Serial Number	3790
Firmware Version	4.10
Software Version	4.1

System Setup	
Transducer Depth (m)	0.21
Screening Distance (m)	0.00
Salinity (ppt)	0.0
Magnetic Declination (deg)	15.4

Units	
Distance	m
Velocity	m/s
Area	m2
Discharge	m3/s
Temperature	degC

Discharge Calculation Settings			
Track Reference	Bottom-Track	Left Method	Sloped Bank
Depth Reference	Vertical Beam	Right Method	Sloped Bank
Coordinate System	ENU	Top Fit Type	Power Fit
		Bottom Fit Type	Power Fit
		Start Gauge Height (m)	0.00
		End Gauge Height (m)	0.00

Discharge Results	
Width (m)	170.015
Area (m2)	520.738
Mean Speed (m/s)	0.520
Total Q (m3/s)	270.612
Maximum Measured Depth	4.154
Maximum Measured Speed	1.032

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge							%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal	Measured
1	R	11:07:19 AM	0:03:21	15.3	166.99	162.39	168.387	530.932	0.831	0.511	0.00	0.00	31.95	197.91	41.55	271.411	--	72.9
2	L	11:10:45 AM	0:02:42	15.8	169.02	163.22	170.224	514.262	1.043	0.534	0.00	-0.18	33.31	204.10	37.19	274.412	--	74.3
3	R	11:13:32 AM	0:03:01	15.0	166.98	163.47	170.474	518.076	0.923	0.519	0.00	0.00	32.36	195.58	40.80	268.749	--	72.8
4	R	11:19:43 AM	0:03:18	14.8	167.74	162.98	170.977	519.682	0.847	0.515	0.00	0.00	32.42	194.53	40.93	267.876	--	72.6
			Mean	15.2	167.68	163.02	170.015	520.738	0.911	0.520	0.00	-0.05	32.51	198.03	40.12	270.612	0.000	73.1
			Std Dev	0.4	0.83	0.40	0.978	6.206	0.084	0.008	0.00	0.08	0.50	3.71	1.71	2.581	0.000	0.7
			COV	0.0	0.005	0.002	0.006	0.012	0.092	0.016	0.000	-1.732	0.015	0.019	0.043	0.009	0.000	0.009

Exposure Time: 0:12:22

Tr1=20190611110737r.rivr; Tr2=20190611111105r.rivr; Tr3=20190611111353r.rivr; Tr4=20190611112009r.rivr;

Comments

Tr1=20190611110737r.rivr - 2; Tr2=20190611111105r.rivr - 1; Tr3=20190611111353r.rivr - 1; Tr4=20190611112009r.rivr - 1;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

/

PXS14R – R.S. 11,482

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information

Site Name	Fort Vermilion
Station Number	4
Location	XSL-14

Measurement Information

Party	JM/AV
Boat/Motor	Harbercraft
Meas. Number	4

System Information

System Type	RS-M9
Serial Number	3790
Firmware Version	4.10
Software Version	4.1

System Setup

Transducer Depth (m)	0.21
Screening Distance (m)	0.00
Salinity (ppt)	0.0
Magnetic Declination (deg)	15.4

Units

Distance	m
Velocity	m/s
Area	m2
Discharge	m3/s
Temperature	degC

Discharge Calculation Settings

Track Reference	Bottom-Track	Left Method	Sloped Bank
Depth Reference	Vertical Beam	Right Method	Sloped Bank
Coordinate System	ENU	Top Fit Type	Power Fit
		Bottom Fit Type	Power Fit
		Start Gauge Height (m)	0.00
		End Gauge Height (m)	0.00

Discharge Results

Width (m)	239.065
Area (m2)	406.146
Mean Speed (m/s)	0.507
Total Q (m3/s)	206.071
Maximum Measured Depth	3.102
Maximum Measured Speed	1.508

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge							%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal	Measured
1	R	12:09:37 PM	0:03:12	15.1	238.68	231.97	237.972	404.313	1.243	0.529	0.00	0.32	42.70	140.22	30.43	213.683	--	65.6
2	L	12:12:58 PM	0:03:12	14.7	237.47	233.08	239.080	409.158	1.237	0.481	0.07	0.14	38.52	131.73	26.55	197.007	--	66.9
3	R	12:16:18 PM	0:02:53	15.0	238.20	233.53	239.534	407.113	1.377	0.523	0.00	0.18	44.04	136.64	32.06	212.927	--	64.2
4	L	12:19:20 PM	0:02:51	14.7	237.18	233.68	239.677	404.000	1.387	0.497	0.00	0.18	39.42	133.48	27.59	200.669	--	66.5
			Mean	14.9	237.89	233.07	239.065	406.146	1.311	0.507	0.02	0.21	41.17	135.52	29.16	206.071	0.000	65.8
			Std Dev	0.2	0.59	0.67	0.669	2.120	0.071	0.019	0.03	0.07	2.27	3.24	2.20	7.353	0.000	1.0
			COV	0.0	0.002	0.003	0.003	0.005	0.054	0.038	1.732	0.330	0.055	0.024	0.075	0.036	0.000	0.016

Exposure Time: 0:12:08

Tr1=20190611121018r.rivr; Tr2=20190611121340r.rivr; Tr3=20190611121703r.rivr; Tr4=20190611122006r.rivr;

Comments

Tr1=20190611121018r.rivr - 4; Tr2=20190611121340r.rivr - 4; Tr3=20190611121703r.rivr - 4; Tr4=20190611122006r.rivr - 4;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

PXS22 – R.S. 18,416

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information	
Site Name	Fort Vermilion
Station Number	5
Location	XSL-22

Measurement Information	
Party	JM/AV
Boat/Motor	Harbercraft
Meas. Number	5

System Information	
System Type	RS-M9
Serial Number	3790
Firmware Version	4.10
Software Version	4.1

System Setup	
Transducer Depth (m)	0.21
Screening Distance (m)	0.00
Salinity (ppt)	0.0
Magnetic Declination (deg)	15.4

Units	
Distance	m
Velocity	m/s
Area	m2
Discharge	m3/s
Temperature	degC

Discharge Calculation Settings			
Track Reference	Bottom-Track	Left Method	Sloped Bank
Depth Reference	Vertical Beam	Right Method	Sloped Bank
Coordinate System	ENU	Top Fit Type	Power Fit
		Bottom Fit Type	Power Fit
		Start Gauge Height (m)	0.00
		End Gauge Height (m)	0.00

Discharge Results	
Width (m)	345.183
Area (m2)	1,813.427
Mean Speed (m/s)	0.901
Total Q (m3/s)	1,633.346
Maximum Measured Depth	10.640
Maximum Measured Speed	1.862

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge							%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal	Measured
1	R	1:24:03 PM	0:04:24	17.6	351.68	344.12	351.122	1,833.465	1.332	0.905	0.00	0.07	143.47	1,264.75	250.76	1,659.046	--	76.2
2	L	1:28:34 PM	0:04:28	16.5	351.84	341.73	348.734	1,831.866	1.313	0.899	0.00	0.16	142.37	1,257.19	247.98	1,647.695	--	76.3
3	R	1:33:11 PM	0:04:18	16.2	339.06	335.92	342.922	1,803.010	1.314	0.909	0.00	0.12	141.97	1,253.62	243.44	1,639.151	--	76.5
4	L	1:37:39 PM	0:04:29	15.6	343.37	334.02	341.024	1,808.285	1.276	0.894	0.00	0.11	140.48	1,225.61	249.72	1,615.923	--	75.8
5	R	1:42:26 PM	0:04:00	15.6	340.52	336.11	342.115	1,790.512	1.419	0.896	0.00	0.27	139.31	1,228.17	237.16	1,604.915	--	76.5
			Mean	16.3	345.29	338.38	345.183	1,813.427	1.331	0.901	0.00	0.15	141.52	1,245.87	245.81	1,633.346	0.000	76.3
			Std Dev	0.7	5.46	3.86	3.992	16.743	0.048	0.006	0.00	0.07	1.46	15.93	5.00	20.060	0.000	0.2
			COV	0.0	0.016	0.011	0.012	0.009	0.036	0.006	0.000	0.467	0.010	0.013	0.020	0.012	0.000	0.003

Exposure Time: 0:21:39

Tr1=20190611132452r.rivr; Tr2=20190611132927r.rivr; Tr3=20190611133407r.rivr; Tr4=20190611133838r.rivr; Tr5=20190611134327r.rivr;

Comments

Tr1=20190611132452r.rivr - 5; Tr2=20190611132927r.rivr - 5; Tr3=20190611133407r.rivr - 5; Tr4=20190611133838r.rivr - 5; Tr5=20190611134327r.rivr - 5;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

PXS22L – R.S. 18,416

Discharge Measurement Summary

Date Measured: Tuesday, June 11, 2019

Site Information

Site Name	Fort Vermilion
Station Number	6
Location	XSL-22L

Measurement Information

Party	JM/AV
Boat/Motor	Harbercraft
Meas. Number	6

System Information

System Type	RS-M9
Serial Number	3790
Firmware Version	4.10
Software Version	4.1

System Setup

Transducer Depth (m)	0.21
Screening Distance (m)	0.00
Salinity (ppt)	0.0
Magnetic Declination (deg)	15.4

Units

Distance	m
Velocity	m/s
Area	m2
Discharge	m3/s
Temperature	degC

Discharge Calculation Settings

Track Reference	Bottom-Track	Left Method	Sloped Bank
Depth Reference	Vertical Beam	Right Method	Sloped Bank
Coordinate System	ENU	Top Fit Type	Power Fit
		Bottom Fit Type	Power Fit
		Start Gauge Height (m)	0.00
		End Gauge Height (m)	0.00

Discharge Results

Width (m)	192.520
Area (m2)	526.987
Mean Speed (m/s)	0.380
Total Q (m3/s)	200.078
Maximum Measured Depth	5.120
Maximum Measured Speed	1.229

Measurement Results

Tr		Time			Distance				Mean Vel		Discharge							%
#		Time	Duration	Temp.	Track	DMG	Width	Area	Boat	Water	Left	Right	Top	Middle	Bottom	Total	MBTotal	Measured
1	L	2:23:50 PM	0:03:12	16.6	191.91	187.32	192.320	526.234	1.000	0.384	0.00	0.02	28.27	145.08	28.96	202.322	-	71.7
2	R	2:27:10 PM	0:03:10	16.2	192.63	187.97	192.967	526.513	1.014	0.376	0.08	-0.01	27.57	141.54	28.94	198.120	-	71.4
3	L	2:30:40 PM	0:03:11	16.0	189.95	187.46	192.456	529.354	0.995	0.372	0.00	-0.03	27.78	139.72	29.68	197.151	-	70.8
4	R	2:34:00 PM	0:03:12	15.7	189.20	187.34	192.338	525.848	0.985	0.386	0.03	-0.08	27.95	144.50	30.31	202.718	-	71.2
			Mean	16.1	190.92	187.52	192.520	526.987	0.998	0.380	0.03	-0.03	27.89	142.71	29.47	200.078	0.000	71.3
			Std Dev	0.3	1.40	0.26	0.263	1.387	0.010	0.005	0.03	0.04	0.26	2.18	0.57	2.470	0.000	0.3
			COV	0.0	0.007	0.001	0.001	0.003	0.010	0.014	1.089	-1.439	0.009	0.015	0.019	0.012	0.000	0.004

Exposure Time: 0:12:45

Tr1=20190611142455r.rivr; Tr2=20190611142817r.rivr; Tr3=20190611143150r.rivr; Tr4=20190611143512r.rivr;

Comments

Tr1=20190611142455r.rivr - 6; Tr2=20190611142817r.rivr - 6; Tr3=20190611143150r.rivr - 6; Tr4=20190611143512r.rivr - 6;

Compass Calibration

Passed Calibration
Error from calibration: 0.12 deg
Mean Magnitude: 9661.38

Pitch: 0/7
Roll: -1/6

System Test

System Test: PASS

Parameters and settings marked with a * are not constant for all files.

Report generated using SonTek RiverSurveyor Live v4.1

Appendix D
Reach Representative Photographs

Looking upstream at the right bank near PXS35.

Looking upstream at the right bank downstream of PXS34.

Looking downstream between PXS31 and PXS30 at the upstream end of the small island along the right (south) side of the channel.

Looking downstream at the Highway 88 bridge.

Looking at the right (south) bank at the ferry crossing near PXS26.

Looking downstream from PXS21 at the right bank near the upstream end of Fort Vermilion.

Looking upstream from PXS19 at the downstream end of the large island.

Looking at the left (north) bank near PXS17.

Looking at the left (north) bank in the north branch of the river near PXS11.

Looking at the right (south) bank in the middle branch of the river near PXS13.

Looking at the right (south) bank near PXS8.

View of the left (north) bank near PXS4.

Appendix E
Open Water Hydrology Assessment Report

FORT VERMILION FLOOD HAZARD STUDY

OPEN WATER HYDROLOGY ASSESSMENT
FINAL REPORT

Prepared for:

Alberta

30 April 2020

NHC Ref. No. 1004659

nhc
northwest hydraulic consultants
water resource specialists

FORT VERMILION FLOOD HAZARD STUDY
19TDRSTR826

OPEN WATER HYDROLOGY ASSESSMENT
FINAL REPORT

Prepared for:

Alberta Environment and Parks
River Engineering and Technical Services
Edmonton, AB

Prepared by:

Northwest Hydraulic Consultants Ltd.
Edmonton, Alberta

30 April 2020

NHC Ref No.1004659

Report prepared by:

Can Hua (Ken) Zhao, Ph.D., P.Eng.
Principal

Sarah Grass, MASc., E.I.T
Project Engineer

Reviewed by:

Gary Van Der Vinne, M.Sc., P.Eng.
Principal

DISCLAIMER

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1 INTRODUCTION

1.1 Background

The Fort Vermilion Flood Hazard Study was initiated by Alberta Environment and Parks (AEP) to identify and assess flood hazards along a 28 km long reach of the Peace River through Mackenzie County, including the hamlet of Fort Vermilion and the settlement of North Vermilion. This study was facilitated under the Flood Hazard Identification Program (FHIP) with the intent to enhance public safety and reduce future flood damages within the Province of Alberta. Results from this study are intended to inform local land use planning decisions, flood mitigation projects, and emergency response planning.

A flood mapping study for Fort Vermilion was completed in 2000 by AEP, formerly know as Alberta Environment (AENV). The present study provides an update of this work to account for additional flow data, current survey data, and contemporary methods of data collection and analysis. Further, the current study incorporates a larger study area and includes both open water and ice jam flood scenarios. The current study is comprised of the following major study components:

1) Survey and Base Data Collection
2) Open Water Hydrology Assessment
3) Open Water Hydraulic Modelling
4) Open Water Flood Inundation Mapping
5) Ice Jam Modelling
6) Ice Jam Flood Inundation Mapping
7) Design Flood Hazard Identification and Mapping

This report summarizes the work of the second component – ***Open Water Hydrology Assessment***.

1.2 Study Objectives

The objective of this component of the overall flood hazard study is to provide open water flood frequency estimates for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year floods along all modelled streams under naturalized conditions. According to the terms of reference for the Fort Vermilion Flood Hazard Study, the flood frequency estimates are required above and below major tributaries and at all locations where flow changes are substantial in comparison to the flood flow and hence necessary to be accounted for in the hydraulic modelling. Based on this criterion, the following locations have been selected for flood frequency estimates:

- Peace River at Fort Vermilion (Station 07HF001)
- Peace River below Boyer River

These locations are shown in **Figure 1**.

1.3 Scope of Report

Flows in the Peace River are regulated by the Bennett Dam and Peace Canyon Dam located on its headwaters. As the flow regulations could have effects on flood peaks at Fort Vermilion, the flood hazard study requires flow naturalization to remove effects of the flow regulation, and subsequent flood frequency analysis under the naturalized conditions.

In presenting the development of the flood frequency estimates, this report contains the following:

- a description of the hydrologic characteristics of the study area and the prevailing flood generating mechanisms,
- routing of naturalized flows from Peace River at Peace River to Fort Vermilion and the creation of naturalized annual maximum discharge series at the abovementioned flood frequency estimate sites,
- statistical descriptions of the naturalized flood peaks, and corresponding frequency curves, at the flood frequency estimate sites, and
- a brief discussion of the effects of climate change on the flood regime.

1.4 Study Area and Reach

The Peace River originates in the Rocky Mountains in northern British Columbia (BC) and flows to the northeast through northern Alberta. The headwaters of the river consist of glacial fed mountain rivers and creeks that feed into Williston Lake, a large reservoir created by the Bennett Dam. From the Bennett Dam, the Peace River flows into Dinosaur Lake, the headpond of the Peace Canyon Dam. After crossing the Alberta-BC border, the Peace River generally flows in an eastern direction toward the town of Peace River (TPR), which is located about 395 km downstream of the Bennett Dam. Beyond TPR, the river flows north and then northeast for about 435 km to Fort Vermilion. It ultimately enters the Slave River after passing the Peace-Athabasca Delta.

Regulation of Peace River flows began in 1968 due to BC Hydro's operation at the Bennett and Peace Canyon dams. Primarily due to the significant storage capacity of Williston Lake, the operation imposes relatively significant effects on Peace River flows at downstream locations including TPR and Fort Vermilion. Flow naturalization for the Peace River was completed by NHC (2016) as part of the Peace River flood hazard study, which provides naturalized daily discharge timeseries at TPR (WSC Station 07HA001) from 1968 to 2015.

While the flood hazard study area is limited to an approximately 28 km long sub-reach of the Peace River through the hamlet of Fort Vermilion, the open water hydrology assessment covers a larger area along the river, which extends from TPR to Fort Vermilion. According to the WSC, the drainage area of the

Peace River increases from approximately 194,400 km^2 at TPR (WSC Station 07HA001) to 227,000 km^2 at Fort Vermilion (WSC Station 07HF001). A basin map is shown in **Figure 2**.

2 DATA COLLECTION

2.1 Hydrometric Data

As part of the Peace River flood hazard study, NHC (2016) produced a naturalized daily discharge timeseries for the Peace River at Peace River (WSC Station 07HA001) from 1968 to 2015. This data set was used as the starting point for the current open water hydrology assessment for Fort Vermilion. This study has also relied on published streamflow and water level data obtained from the WSC for the hydrometric stations listed in **Table 1**. Locations of these stations are shown in **Figure 2**.

Table 1: WSC streamflow stations describing Peace River flows

WSC Station No.	Station Name	Drainage Area (km^2)[1]	Period of Record
07HA001	Peace River at Peace River	194,000	1915-1931, 1958-2016 and 2017-2018[2]
07HD001	Peace River near Carcajou	210,000	1960-1967
07HF001	Peace River at Fort Vermilion	223,000	1917-1922, 1961-1978, 1979-1993[3], 2006-2017 and 2018[2]
07HA005	Whitemud River near Dixonville	2,020	1967-2018
07HC001	Notikewin River at Manning	4,680	1961-2017 and 2018[2]
07HC907	North Star Drainage near North Star	31	1991-2011 and 2012-2018[2]
07HC002	Buchanan Creek near Manning	232	1985-2016 and 2017-2018[2]
07HF002	Keg River at Highway No. 35	648	1971-2018
07JF002	Boyer River near Fort Vermilion	6,660	1962-2015
07JF003	Ponton River above Boyer River	2,440	1962-2015

Notes:

1. Drainage area based on information from WSC
2. Preliminary data from WSC
3. Only water level data are available for 1979-1993. The data were used to estimate daily discharges for this study.

2.2 Historic Flood Data

Historic floods herein refer to major floods that occurred prior to the period of systematic hydrometric data collection. If the magnitude of a historic flood can be estimated based on available information, the estimate could be used to improve the flood frequency estimates.

The WSC gauge station on the Peace River at Fort Vermilion (07HF001) was initially established in August 1915. It appears that information on historic floods prior to 1915 is not available. Therefore, historic floods were not considered in this study.

According to AENV (2000), the worst flood reported at Fort Vermilion was caused by an ice jam in 1934. The June 1990 event was a record high open-water flood event; however, the Fort Vermilion gauge station provided only water level records for the 1979-1993 period without discharge measurements. AENV (2000) estimated the maximum instantaneous discharge for this event as 12,640 m^3/s based on a water level of El. 253.403 m recorded on 16 June 1990. The water level used by AENV (2000) to estimate this flood peak discharge is 0.28 m lower than the 1990 peak water level published by the WSC. As discussed in Section 3.3.1, the peak discharge for the June 1990 flood was estimated in this study as 13,800 m^3/s based on the WSC published peak water level.

2.3 Previous Flood Frequency Estimates

Previous flood frequency estimates for the Peace River at Fort Vermilion were found in the following studies:

- The Peace River at Fort Vermilion flood risk mapping study by AENV (2000)
- Fort Vermilion flood and bank erosion study by AENV (1968)

3 FLOW NATURALIZATION

3.1 Naturalized Flows at TPR

Flow naturalization to remove the effects of BC Hydro's operation at the Bennet and Peace Canyon dams was completed as part of the Peace River flood hazard study by NHC (2016), which resulted in a naturalized daily discharge timeseries for the 1968-2015 period at TPR (WSC Station 07HA001). The analysis was extended for this study using 2016-2018 preliminary flow and water level data for the Peace River and Williston Lake obtained from the WSC. The extended naturalized TPR daily discharge timeseries (1968-2018) was used as inputs for the Fort Vermilion flow naturalization.

3.2 Routing Model Configuration

Flow naturalization for this study is to estimate naturalized flood peak discharges for Peace River at Fort Vermilion by routing the naturalized flows from TPR to Fort Vermilion, together with tributary inflows and gauge correction (as discussed in Section 3.3). The analysis generally follows the same approach undertaken by NHC (2016) for the Peace River Flood Hazard study. The routing analysis was performed at a daily time step using HEC-ResSim. **Figure 3** depicts the structure of the routing model.

HEC-ResSim currently supports eight hydrologic channel routing methods. In this study, the Streamflow Synthesis and Reservoir Regulation (SSARR) method was used. This method was developed by the U.S. Army Corps of Engineers (USACE) and has been widely used across Alberta by AEP for water supply studies, flood forecasting and other studies. It uses a Muskingum-type of channel routing method to simulate channel storage effects based on reach-specific discharge-travel time relationships that are provided as input. The relationship can be defined by a table of discharge vs. travel time, or by the following formula:

$$T_s = \frac{KTS}{Q^n} \qquad \textbf{(Equation 1)}$$

where T_s is travel time (or time of storage) in hours; Q is discharge in m^3/s; and KTS and n are coefficients that need to be input to the model. The method also requires to specify the number of routing phases (N). The routing parameters (KTS, n and N) or the discharge vs. travel time table are usually determined by calibration against observed hydrographs or from average flow velocity estimates based on channel geometry data.

A HEC-RAS open-channel hydraulic model of the Peace River was used to obtain initial estimates of the routing parameters for the sub-reaches between TPR and Fort Vermilion. This HEC-RAS model was also used for the NHC 2016 Peace River study. It represents the channel geometry of the Peace River from Peace Canyon Dam to just downstream of Fort Vermilion. The channel geometry is based on a large number of cross sections of varying quality that were surveyed along the Peace River by a number of agencies over the past 30 years. The model has been used for a number of major projects and studies completed by BC Hydro (2002, 2012) and Glacier Power Ltd. (NHC, 2006). The estimated SSARR routing parameters based on the HEC-RAS model were then further refined through calibration against WSC gauge data. **Figure 4** demonstrates the comparisons of routed and observed flow hydrographs at Fort Vermilion for the three largest calibration events. The adopted SSARR routing parameters are summarized in **Table 2**.

Table 2: SSARR routing parameters for Peace River from Peace River to Fort Vermilion

Sub-reach	KTS	n	Number of Routing Phases
Peace River (07HA001) to Carcajou (07HD001)	588	0.38	1
Carcajou (07HD001) to Fort Vermilion (07HF001)	714	0.38	1

3.3 Estimation of Naturalized and Natural Flows at Fort Vermilion

3.3.1 Flow Naturalization for Regulated Period

Estimation for 1968-1978 and 2006-2018

The flow naturalization for Fort Vermilion follows the Project Depletion approach, which has been used in various studies in Alberta, including the previous Peace River flood hazard study (NHC, 2016). In this approach, both gauged (regulated) and naturalized flows, together with gauged or estimated natural tributary inflows, are routed from an upstream gauge station to a downstream gauge station; and gauge corrections (adjustments based on differences between routed gauged flows and downstream gauge data) are then applied to the routed naturalized flows to derive natural flow estimates for the downstream station. The gauge correction is to account for additional ungauged tributary inflows and errors from the routing model.

In this study, the 1968-2018 naturalized Peace River daily flows for TPR were routed to Fort Vermilion. Available daily flow data from the Fort Vermilion gauge station (07HF001) were used to perform gauge correction. This resulted in naturalized daily flows at Fort Vermilion for the 1968-1978 and 2006-2018 periods. Naturalized annual maximum daily discharges at Fort Vermilion for these two regulated periods were extracted from the naturalized daily discharge timeseries.

The naturalized flow routing analysis was also performed for 1968-1978 and 2006-2018 without the gauge correction. The resulting annual maximum values for Fort Vermilion are compared with the naturalized maximum discharges (with gauge correction) in **Figure 5** to assess effects of the gauge correction applied in the flow naturalization process. The comparison indicates that the gauge correction represents only about 3% of the naturalized maximum daily discharge at Fort Vermilion, which is relatively small.

Estimation for 1979-1993

Flow data for 1979-1993 are not available for Peace River at Fort Vermilion (WSC Station 07HF001); however, this gauge station provides daily gauge height readings (water levels) for this period. According to the WSC (Appendix A), a data review was completed in 1985 with its focus on the high uncertainty in winter discharges. The review noted that the significant discrepancies between winter discharge measurements and the rating curve for this gauge station cast doubt on the overall data quality. The WSC also indicated that discharge measurements were not available to validate the stage-discharge rating curve for the 1979-1993 period; therefore, only gauge heights were published for this period.

In this study, the 1979-1993 daily gauge height data were used to estimate Peace River discharges at Fort Vermilion, and the results were used to perform gauge correction for flow naturalization.

Based on a review of the WSC historical rating curves for the Fort Vermilion gauge station, the rating curve No. 9 dated February 11, 1975 was used to convert the gauge heights into discharges. It is the last curve produced by the WSC before they restarted reporting Peace River discharges at Fort Vermilion in 2006. This rating curve is shown in **Figure 6** and compared with the curve dated August 18, 2015, which is currently used by the WSC. As shown in **Figure 6**, the differences between the 1975 and 2015 rating curves are relatively small. So, it is reasonable to assume that the 1975 rating curve is representative of the 1979-1993 gauging condition. Note that the 1975 rating curve is nearly identical to the rating curve used by AENV (2000).

The estimated 1979-1993 daily discharges for Fort Vermilion were compared with reported discharges for the TPR gauge station. Some high discharge estimates in March or early April were discarded because they were probably affected by ice conditions and appeared to be erroneous in comparison with TPR. Other than that, the estimates appeared to be reasonable. The 1990 maximum daily discharge (the record high event) was estimated to be 13,200 m^3/s from the adopted 1975 rating curve. Note that based on the WSC published peak instantaneous gauge height (10.169 m, which is converted into a geodetic elevation of 253.68 m), the maximum instantaneous discharge for the 1990 event would be 13,800 m^3/s from the rating curve. This discharge value is about 9% higher than the estimate from AENV (2000); however, the AENV estimate was based on a water level 0.28 m lower than the WSC published peak water level.

Note that estimation of Fort Vermilion discharges for this regulated period is intended to provide data for gauge correction in the flow naturalization process. As discussed in the previous section, the gauge correction represents only about 3% of the naturalized maximum daily discharge at Fort Vermilion. So, errors in the results related to the discharge estimation from gauge heights would be negligible. Therefore, although the estimated discharges probably do not meet the WSC's standard of data quality, they are believed to be adequate for gauge correction in the flow naturalization process of this study.

The 1979-1993 naturalized daily flows at TPR together with gauged tributary inflows were routed to Fort Vermilion to provide naturalized flow estimates, with gauge correction being applied based on the discharges estimated from the gauge heights. The naturalized annual maximum daily discharges for Fort Vermilion were then derived. The routing analysis was also performed without the gauge correction, and the two sets of resulting maximum discharges (with and without gauge correction) are plotted against each other in **Figure 5**. The plot, which illustrates effects of the gauge correction, is consistent with the linear relationship developed from the results for the other periods (1968-1978 and 2006-2018), in which the WSC discharge data area available for gauge correction. This also indicate that the results for 1979-1993 are reasonable.

Estimation for 1994-2005

Gauge correction cannot be performed in the flow naturalization process for Fort Vermilion from 1994 through 2005, because neither flow nor gauge height records are available for Peace River at Fort Vermilion (WSC Station 07HF001). For this period, the naturalized Peace River flows for TPR (WSC Station 07HA001) were routed to Fort Vermilion together with available gauged tributary inflows. The resulting annual maximum daily discharges from the routed flow series were multiplied by 1.03 to account for gauge correction based on the relationship shown in **Figure 5**. The adjusted values were taken as the final maximum discharge estimates for Fort Vermilion.

3.3.2 Natural Flow Estimation for Pre-regulation Period

For the pre-regulation period, the Fort Vermilion station (07HF001) provides natural annual maximum discharges (daily or instantaneous value, or both) for 1917-1922 and 1961-1967 missing 1963. Available natural daily flow data from the TPR gauge station (07HA001) were used for the routing analysis to estimate annual maximum discharges at Fort Vermilion for 1915, 1916, 1923-1931,1958-1960 and 1963. As gauge correction and tributary inflow data are not available for those years, the routed discharges needed to be adjusted.

Figure 7 shows a comparison of naturalized annual maximum discharges for Fort Vermilion with maximum discharges resulting from routing of TPR flows with no gauge correction and no gauged tributary inflows, for the 1971-1993 and 2006-2018 periods. Based on the relationship from this figure, the routed maximum discharges for the gap years listed above were multiplied by 1.04 to obtain the estimates for Fort Vermilion. Note that the relationship shown in **Figure 7**, in comparison with that in **Figure 5** (which illustrates effects of gauge correction only, resulting in a ratio of 1.03), indicates that gauged tributary inflows between TPR and Fort Vermilion represent only about 1% of maximum discharges at Fort Vermilion.

3.4 Summary of Natural and Naturalized Peak Discharges for Fort Vermilion

Table 3 provides a summary of the natural and naturalized annual maximum daily discharges for Peace River at Fort Vermilion, derived from the analysis described above (for the pre and post-regulation periods respectively). The table also includes the reported natural annual maximum daily and instantaneous discharges for the Fort Vermilion gauge station (07HF001).

For the pre-regulation years when the instantaneous value were missing and for the post-regulation period (1968-2018), the instantaneous values were calculated with the daily values multiplied by 1.02. This factor (1.02) is the ratio of annual maximum instantaneous (Q_i) against daily discharge (Q_d) for the natural flow condition estimated by NHC (2016) based on available pre-regulation flow data for hydrometric stations on the Peace River from Taylor to Peace Point (including the Fort Vermilion station). The relationship is illustrated in **Figure 8**, which is a reproduction of the plot from NHC (2016). The estimated instantaneous discharges are shown in **Table 3.** All of the natural and naturalized annual maximum daily and instantaneous discharges are plotted in **Figure 9**. Note that most of the annual maximum discharges occurred in late May or June.

Table 3: Annual peak discharges of natural/naturalized flows for Peace River at Fort Vermilion

Year	Maximum Daily Discharge (m^3/s)(1)	Date	Maximum Instantaneous Discharge (m^3/s)(2)	Date
1915	8,880	Jul-17	9,060	
1916	7,180	Jul-10	7,330	
1917	**8,470**	**Jun-08**	8,640	
1918	**11,100**	**Jun-22**	**11,200**	**Jun-22**
1919	**9,060**	**Jun-27**	9,240	
1920	**9,940**	**Jun-21**	10,100	
1921	**8,980**	**Jun-12**	9,160	
1922	**9,260**	**Jun-08**	**9,340**	**Jun-08**
1923	8,400	Jun-18	8,570	
1924	7,750	May-21	7,900	
1925	7,790	May-23	7,940	
1926	7,240	Jun-21	7,380	

Year	Maximum Daily Discharge (m^3/s)[(1)]	Date	Maximum Instantaneous Discharge (m^3/s)[(2)]	Date
1927	8,090	Jul-11	8,250	
1928	6,850	Jun-02	6,990	
1929	6,390	Jun-10	6,510	
1930	8,580	Jun-22	8,750	
1931	7,570	Jun-22	7,720	
1958	11,200	Jun-02	11,400	
1959	8,120	Jun-08	8,280	
1960	11,100	Jun-25	11,300	
1961	**9,910**	**May-31**	10,100	
1962	**8,890**	**Jul-01**	9,070	
1963	8,770	May-28	8,940	
1964	**12,400**	**Jun-16**	12,600	
1965	**11,500**	**Jul-13**	**12,100**	**Jul-12**
1966	**7,480**	**May-15**	**7,790**	**May-16**
1967	**10,400**	**Jun-06**	10,600	
1968	12,500	Jun-16	12,700	
1969	6,710	Jun-08	6,850	
1970	8,550	Jun-09	8,720	
1971	12,100	Jun-20	12,300	
1972	18,500	Jun-16	18,800	
1973	8,530	Jun-21	8,700	
1974	7,930	Jun-22	8,090	
1975	5,530	May-22	5,640	
1976	9,390	Jun-30	9,570	
1977	9,810	Jun-13	10,000	
1978	6,640	Jun-10	6,770	
1979	8,470	Jun-09	8,640	
1980	5,630	Jun-23	5,740	
1981	11,100	May-31	11,300	
1982	10,000	Jul-19	10,200	
1983	6,980	Jun-25	7,120	
1984	9,510	Jun-17	9,700	
1985	7,800	Jun-08	7,960	
1986	8,680	Jun-04	8,850	
1987	10,400	Aug-05	10,600	
1988	8,530	Jun-15	8,700	
1989	6,380	Jun-08	6,510	
1990	19,400	Jun-15	19,700	

Year	Maximum Daily Discharge (m^3/s)[(1)]	Date	Maximum Instantaneous Discharge (m^3/s)[(2)]	Date
1991	7,560	May-18	7,720	
1992	8,370	Jun-20	8,540	
1993	6,620	May-25	6,750	
1994	8,260	Jun-17	8,420	
1995	7,230	May-20	7,370	
1996	11,000	Jun-10	11,200	
1997	10,600	Jun-05	10,800	
1998	8,070	May-31	8,230	
1999	10,700	Jun-21	10,900	
2000	8,810	Jun-14	8,990	
2001	10,700	Jun-15	10,900	
2002	12,100	Jun-20	12,300	
2003	7,650	Jun-14	7,800	
2004	7,100	Jun-12	7,240	
2005	8,660	Jun-05	8,830	
2006	6,760	May-31	6,900	
2007	13,700	Jun-10	14,000	
2008	9,900	Jun-03	10,100	
2009	8,190	Jun-16	8,350	
2010	5,790	May-26	5,900	
2011	14,900	Jul-13	15,200	
2012	12,900	Jun-11	13,200	
2013	9,840	Jun-04	10,000	
2014	7,440	May-28	7,590	
2015	8,500	May-30	8,670	
2016	8,210	Jun-20	8,370	
2017	10,600	Jun-14	10,800	
2018	6,550	May-18	6,680	

Notes:

1. The bolded values are gauge data from the pre-regulation record for Peace River at Fort Vermilion (WSC Station 07HF001). The other daily values are based on routed natural/naturalized flows from TPR.
2. The instantaneous discharges except the bolded vales are based on $Q_i = 1.02Q_d$.

4 FLOOD FREQUENCY ANALYSIS

4.1 Analysis for Peace River at Fort Vermilion

Frequency analysis was performed for the naturalized annual maximum instantaneous discharges for Peace River at Fort Vermilion shown in **Table 3**. The analysis was conducted using the USACE HEC-SSP (version 2.1) flood frequency program and a spreadsheet model developed by NHC. In accordance with the Hydrologic and Hydraulic Guidelines for Flood Hazard Area Delineation by AENV (2008) and Guidelines on Flood Frequency Analysis by Alberta Transportation (AT, 2001), various theoretical probability distributions were tested, including the normal (N), log-normal (LN), three-parameter log-normal (LN3), Pearson type III (P3), log-Pearson type III (LP3), Gumbel (G), generalized extreme value (GEV), and Weibull (W) distributions. In accordance with AT (2001), the method of moments was used in the calculation of means, variances, and skew coefficients with theoretical limits being considered. The Cunnane positioning formula was used to plot data points for visualization purposes.

The USGS "Guidelines for Determining Flood Frequency" Bulletin 17C (USGS, 2018) was also used for the present study. The Bulletin 17C provide a framework primarily intended to standardize the methods to account for: historic flood information, zero flows or low outliers, and high outliers; and methods to estimate population parameters. It uses the LP3 as the base method for flood frequencies and recommend use of a weighted average of the station skew and a regional skew. The Bulletin 17C uses the Expected Moments Algorithm (EMA) to extends the method of moments so that it can better handle low outlier adjustments, regional skew information and historical information. The primary difficulty with the application of the Bulletin 17C is that regional skew estimates are not available in Alberta. As a result, only the station skewness was used in the present study. Note that, when the station skewness is used and no outliers are detected in the population, the resulting Bulletin 17C curve is often very close or identical to a standard LP3 curve based on the method of moments.

Table 3 includes a total of 78 naturalized annual maximum instantaneous discharges for Peace River at Fort Vermilion. This data series spans 104 years from 1915 to 2018 (missing 1932-1957). Each of the frequency distributions in the adopted suite were fitted to the data, as shown in **Figure 10**. The goodness of fit of each of the distributions, as applied to a flood series, was compared through the Kolmogorov–Smirnov test (K-S test) and a least squares method.

The K-S test can be used to compare a sample with a reference probability distribution. It quantifies a distance between the empirical probability of the sample and the cumulative distribution function of the reference distribution. The maximum distance (referenced to as D-statistic value, D_n) can be used to describe the goodness of fit: a smaller D_n value would indicate a better fit between the empirical distribution and the theoretical one.

The least squares method (Kite, 1977) is based on the sum of squared errors (*SSE*) calculated by

$$SSE = \sqrt{\frac{1}{n-m}\sum_{i=1}^{n}(x_i - y_i)^2} \quad \textbf{(Equation 2)}$$

where n is the number of recorded events, m is the number of parameters used by a frequency distribution, x_i is the i^{th} recorded peak discharge, and y_i is the discharge computed from the frequency distribution at the probability equal to the empirical probability of discharge x_i.

The *SSE* values of the tested probability distributions were then normalized by the mean peak discharge (Q_{pm}) to provide a dimensionless *SSE*. In this approach a lower dimensionless *SSE* would indicate a better fit between the empirical distribution and the theoretical one.

The applied frequency distributions were ranked first by D_n and *SSE* values separately and the sums of the rankings were then compared to derive the final combined ranking, as shown in **Table 4**. The LP3 distribution and Bulletin 17C are identical, and they produced the lowest *SSE* value and second lowest D_n value. As such they are ranked the best in the combined ranking, followed by the GEV and LN3 distributions. As shown in **Figure 10**, these four frequency curves are very similar, and they all fit the data reasonably well. It is recommended that the LP3 distribution be used herein to described the natural/naturalized flood peaks for Peace River at Fort Vermilion. This selection is consistent with the flood frequency curve for Peace River at Peace River (WSC Station 07HA001) adopted by NHC (2016). The adopted LP3 curve for Fort Vermilion (WSC Station 07HF001) with 95% confidence limits is shown in **Figure 11**.

Table 4: Goodness-of-fit comparison for probability distributions for natural/naturalized Peace River flood peaks at Fort Vermilion

Distribution	D_n	Normalized *SSE* (Q_{pm} = 9,370 m^3/s)	Rank by D_n	Rank by *SSE*	Combined Ranking
Normal (N)	0.125	0.100	8	8	8
Log-normal(LN)	0.080	0.069	7	7	7
Three parameter log-normal (LN3)	0.061	0.040	4	4	4
Pearson III (P3)	0.074	0.048	6	5	5
Log-Pearson III (LP3)	0.057	0.039	2	1	1
Gumbel (G)	0.063	0.052	5	6	5
Generalized extreme value (GEV)	0.052	0.039	1	3	3
Weibull (W)	0.134	0.110	9	9	9
Bulletin 17C	0.057	0.039	2	1	1

4.2 Estimation for Peace River Below Boyer River

The Boyer River joins the Peace River at a location about 9 km downstream of the Fort Vermilion gauge station (07HF001). It drains an area of about 9,300 km^2, which is about 4% of the Peace River drainage area at Fort Vermilion. Daily inflows to the Peace River could be estimated as the sum of the gauged discharges for WSC Stations 07JF002 (Boyer River near Fort Vermilion) and 07JF003 (Ponton River above Boyer River), which are located northwest of Fort Vermilion (**Figure 2**). Based on the data published by

WSC, the natural/naturalized annual maximum discharges for Peace River below Boyer River were estimated by adding the same-day discharges to the daily discharges for Peace River at Fort Vermilion, for the 1962-2015 period. As shown **Figure 12**, there is no significant difference between the natural/naturalized maximum daily discharges at Fort Vermilion and downstream of the Boyer River. Therefore, it is recommended that the frequency estimates for Peace River at Fort Vermilion be applied to Peace River below Boyer River.

5 SUMMARY OF FLOOD FREQUENCY ESTIMATES

5.1 Recommended Flood Frequency Estimates

The recommended flood frequency estimates for Peace River at Fort Vermilion (WSC Station 07HF001) for the naturalized flow condition are summarized in **Table 5**. The estimates are based on the adopted LP3 (log-Pearson III) frequency curve. These estimates are also applicable for the study reach below the Boyer River.

Table 5: Naturalized flood frequency estimates for Peace River at Fort Vermilion

Return Period (Years)	Annual Probability of Exceedance (%)	Naturalized Peak Instantaneous Discharge (m^3/s)	
		Value	95% Confidence Limit
1000	0.10	24,700	21,800 - 28,900
750	0.13	23,800	21,100 - 27,700
500	0.20	22,500	20,100 - 26,100
350	0.29	21,500	19,300 - 24,700
200	0.50	19,900	18,000 - 22,700
100	1.0	18,100	16,500 - 20,300
75	1.3	17,300	15,900 - 19,400
50	2.0	16,300	15,100 - 18,100
35	2.9	15,500	14,300 - 17,000
20	5.0	14,100	13,200 - 15,400
10	10	12,600	11,900 - 13,500
5	20	11,000	10,500 - 11,600
2	50	8,830	8,440 - 9,240

5.2 Comparison with Previous Studies

The flood frequency estimates for Peace River at Fort Vermilion are compared with values from the previous studies by AENV (2000 and 1968) in **Table 6**. The values calculated in this study are significantly higher than those from AENV (2000). However, AENV (2000) did not consider effects of flow regulation on the Peace River, and the flood frequency estimates were stated to be preliminary. The objective of the flood frequency analysis by AENV (1968) was to provide flood peak estimates for a regulated flow condition based on limited pre-regulation (natural) flow records. The analysis simply assumed a 50% reduction on natural flood peaks to develop regulated flood frequency estimates. While the AENV (1968) study had a different objective from the current study (which is to develop naturalized flood frequency estimates), it provided a natural flood frequency curve for Peace River at Fort Vermilion based on the pre-regulation flow data for 1917-1922 and 1961-1967. The values from this curve are shown in **Table 6.** These estimates except the 2-year value are smaller than the current estimates but greater than the AENV (2000) values. The largest flood peak discharge in the data used by AENV (1968) is 12,000 m^3/s for the 1964 event, which is exceeded 9 times in the naturalized flood peak data series used for the current study (**Table 3**). The AENV (1968) 100-year value is smaller than the naturalized flood peak discharges for 1972, 1990 and 2011. It should also be noted that the 1964 and 1965 peak discharges for Peace River at Fort Vermilion presented by AENV (1968) are smaller than the WSC published values. Therefore, the current estimates are more reasonable.

Table 6: Comparison with previous flood frequency estimates for Peace River at Fort Vermilion

Return Period (Years)	Peak Instantaneous Discharge (m^3/s)		
	Present Study for Naturalized Flows	AENV (2000)	AENV (1968)
100	18,100	12,640	14,160
50	16,300	9,990	13,590
10	12,600	8,215	11,890
2	8,830	6,090	9,630

The flood frequency estimates for Fort Vermilion were also compared with the naturalized flood peaks for Peace River at TPR (WSC Station 07HA001) from NHC (2016) in **Table 7**. While the drainage area increases by about 17% when the Peace River flows from TRP to Fort Vermilion, the estimated peak discharges for Fort Vermilion are smaller than for TPR. The differences are smaller for shorter return periods (e.g. about 2% between the 2-year values) but are significant for longer return periods. The 100 and 1000-year values for Fort Vermilion are about 14% and 21% smaller than the TRP values respectively. The decreases imply that flood waves attenuate noticeably when propagating from TPR to Fort Vermilion, while contribution of tributary inflows between two site is small.

Based on the discussion presented in Section 3.3.2, contribution of tributary inflows between TPR and Fort Vermilion during the Peace River annual peak flow event would generally account for about 4% of the peak discharge at Fort Vermilion (the combined effect of gauged tributary inflows and gauge correction used to compensate for ungauged tributary flows and routing errors in the flow naturalization process). The tendency of flood peak attenuation from TPR to Fort Vermilion appears to be supported by historical gauge data, as shown in **Figure 13**. The figure shows the annual peak instantaneous discharge ratios of Fort Vermilion versus TPR, based on available WSC data for both the pre- and post-regulation periods from 1917 to 2017 (missing instantaneous values were estimated from daily values based on historical Q_i/Q_d ratios). For most of the years, the Fort Vermilion to TPR peak discharge ratio is smaller than 1.0. The flood peak discharges at Fort Vermillion were smaller than at TPR by up to 33% (corresponding to the ratio of 0.67). The maximum TPR discharge in this plot is 15,600 m^3/s (occurred in 1972), which is equal to the 20-year naturalized flood peak (**Table 7**). The corresponding peak discharge measured at Fort Vermilion is 28% smaller. There are 9 events for which the Peace River peak discharges at Fort Vermillion were greater than at TPR (the peak discharge ratio greater than 1.0). For those events, the TPR peak discharges range from 3,710 to 9,730 m^3/s, which are noticeably smaller than the TPR 5-year naturalized flood peak (**Table 7**). Note that 5 of these events occurred in 5 consecutive years from 1917 to 1921 with the Fort Vermilion to TPR peak discharge ratio as high as 1.37; and the data were from manual gauge readings which may not be as accurate as recording (automatic) gauge data for the 1964-2017 period.

Table 7: Comparison of flood frequency estimates for Peace River at TPR and at Fort Vermilion

Return Period (Years)	Naturalized Peak Instantaneous Discharge (m^3/s)		Peak Discharge Ratio of Fort Vermilion to TPR
	At TPR (NHC 2016)	At Fort Vermilion (current study)	
1000	31,600	25,000	0.79
750	30,100	24,100	0.80
500	28,100	22,800	0.81
350	26,400	21,700	0.82
200	23,900	20,100	0.84
100	21,100	18,200	0.86
75	20,100	17,500	0.87
50	18,600	16,400	0.88
35	17,400	15,600	0.90
20	15,600	14,200	0.91
10	13,500	12,600	0.93
5	11,600	11,000	0.95
2	9,050	8,840	0.98

6 CLIMATE CHANGE COMMENTARY

NHC (2016) provides a qualitative interpretation of climate and hydrologic projections obtained from the scientific literature that is pertinent to evaluating future changes in flood hazards in the Peace River basin. Although the commentary was made for the Peace River Flood Hazard Study, the large-scale climate change assessments are also applicable to the flows downstream; because, as discussed in this report, tributary inflows between TPR and Fort Vermilion do not have significant effects on Peace River annual peak discharges at Fort Vermilion. The open-water flood risk at Fort Vermilion is governed by runoff derived from the basin area above TPR. As such, potential impacts from climate change would be similar for the two sites. Therefore, the commentary for the previous Peace River study (NHC, 2106) is applicable to the Fort Vermilion study. The key points are summarized as follows:

Current global climate models indicate that both temperature and precipitation will increase in the upper Peace River Basin due to projected increases in CO_2 concentrations in the atmosphere. Increased temperatures in the winter months will likely results in smaller snow packs and earlier snowmelt runoff. Climate models differ in their predictions of changes to median monthly runoff, with some models predicting increases in runoff (e.g. Poitras et al., 2011) and others predicting decreases. However, most models predict a shift in peak runoff from June to May.

The implications of climate change on the hydrologic characteristics of the Smoky River basin – the other main contributor to flood peaks at TPR and Fort Vermilion – is not so well defined. Like the upper Peace River, the Smoky River is part of the Mackenzie River basin, which has been studied extensively to evaluate the impact of a changing climate on Mackenzie River flows. The studies by Aziz and Burn (2005) and Yip, et al. (2012) are typical examples. However, given the small drainage area of the Smoky basin relative to that of the overall Mackenzie River basin, few details specific to the Smoky River are provided. However, Kerkhoven (2014) assessed the implications of climate change on the Wapiti River basin, which forms a large part of the Smoky basin. The overall conclusions for the Wapiti basin can be used to generalize the expected climate change outcomes for the entire Smoky basin.

Overall, the annual and season temperatures are expected to increase over the next 80 years at rates similar to what has been experienced in the last 100 years or so. The expected changes in annual precipitation are somewhat equivocal with the GCM Climate PP models suggesting that the annual precipitation could change by between a five percent decrease and 30 percent increase over the next 80 years, reflecting an increase in rainfall and a decrease in snowfall. There is no consensus on changes to runoff, with some of the projections calling for a systematic decrease in runoff and some calling for an increase, although overall the trend suggests a slight increase in annual runoff from the current average of 95 mm to something close to 100 mm – contrary to recent trends. Clearly, changes to future precipitation and runoff are not well quantified. Overall, Kerkhoven suggests that there will be no systematic changes to runoff volumes in the foreseeable future – likely because of the reduction in snowmelt runoff.

With the climate change forecasts calling for slightly more rainfall, it would be expected that runoff from the Smoky basin would increase. Since most of the summer rainfall occurs during large storms, it could be reasonably assumed that more rainfall would also mean more severe storms that could in turn produce more severe floods. However, with the higher temperatures, evapotranspiration would be greater, thereby possibly reducing runoff coefficients and limiting the expected increases to flood peaks. On the whole, there is insufficient information to be able to identify all the linkages between precipitation and runoff to make any forecasts about how climate change might affect flood peaks.

In general, the effect of climate change on Peace River inflows is uncertain. Increased precipitation may lead to higher flood peaks due to increased precipitation intensity but this will be mitigated by reduced snowpack and drier antecedent moisture conditions due to higher temperatures. Loss of tree cover and soil changes associated with beetle infestation, wildfires, and changing land use could also contribute to higher runoff volumes and peaks.

A more detailed descriptions of climate characteristics, its effect on flood hazards, and specific hydrologic projections developed by B.C.'s Pacific Climate Impacts Consortium are provided in Appendix C of the NHC 2016 report.

7 REFERENCES

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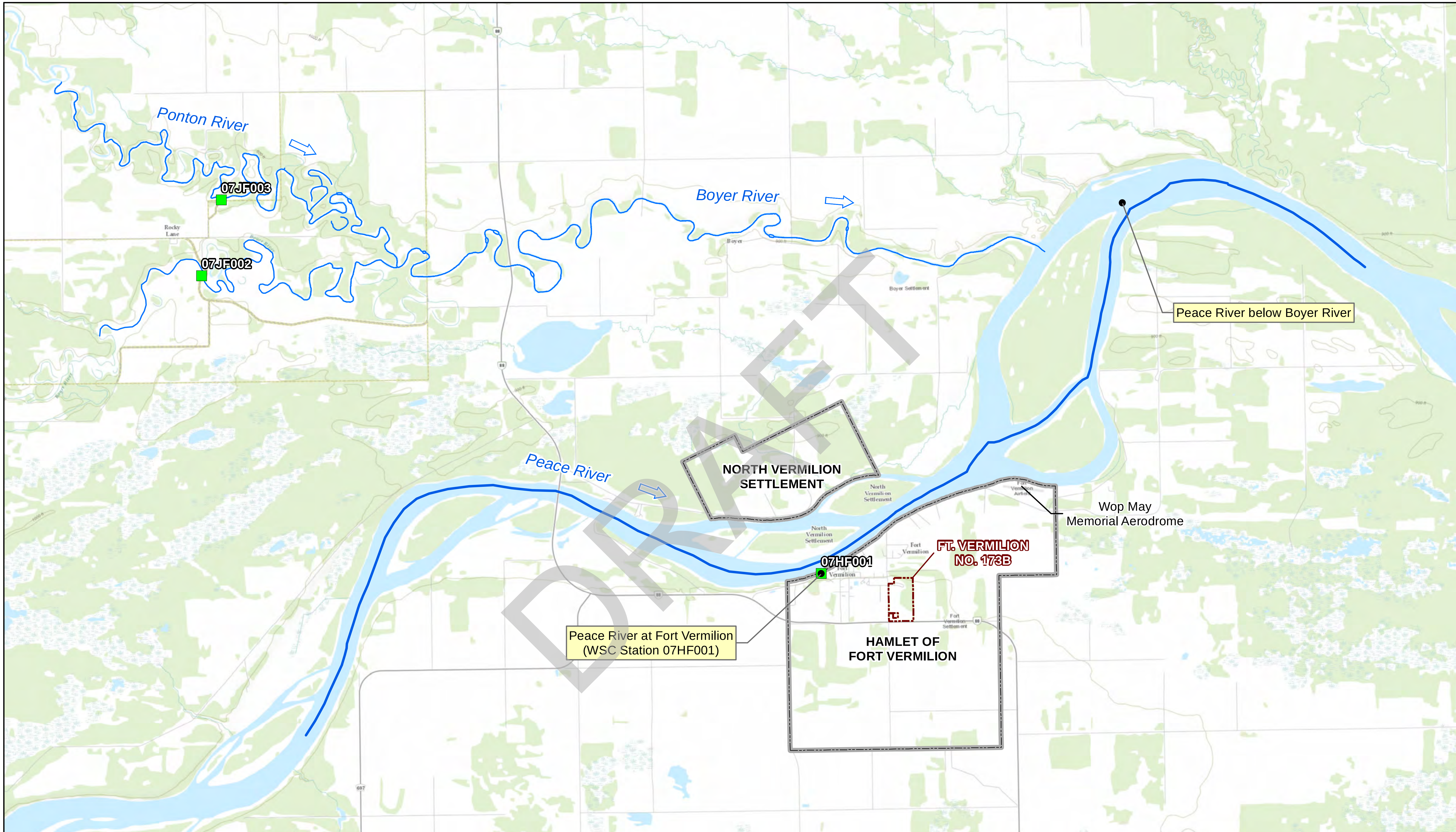
Figures



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WSC GAUGE STATION	STUDY REACH	MUNICIPAL BOUNDARY
FLOOD FREQUENCY ESTIMATION SITE	MAJOR TRIBUTARY	INDIAN RESERVE

DATA SOURCES: AltaLIS Ltd., Esri World Topographic Basemap

SCALE - 1:80,000

0 1 2 3 KM

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD29 HTv2.0; Units: Metres

Job: 1004659 | Date: 21-APR-2020

FORT VERMILION FLOOD HAZARD STUDY

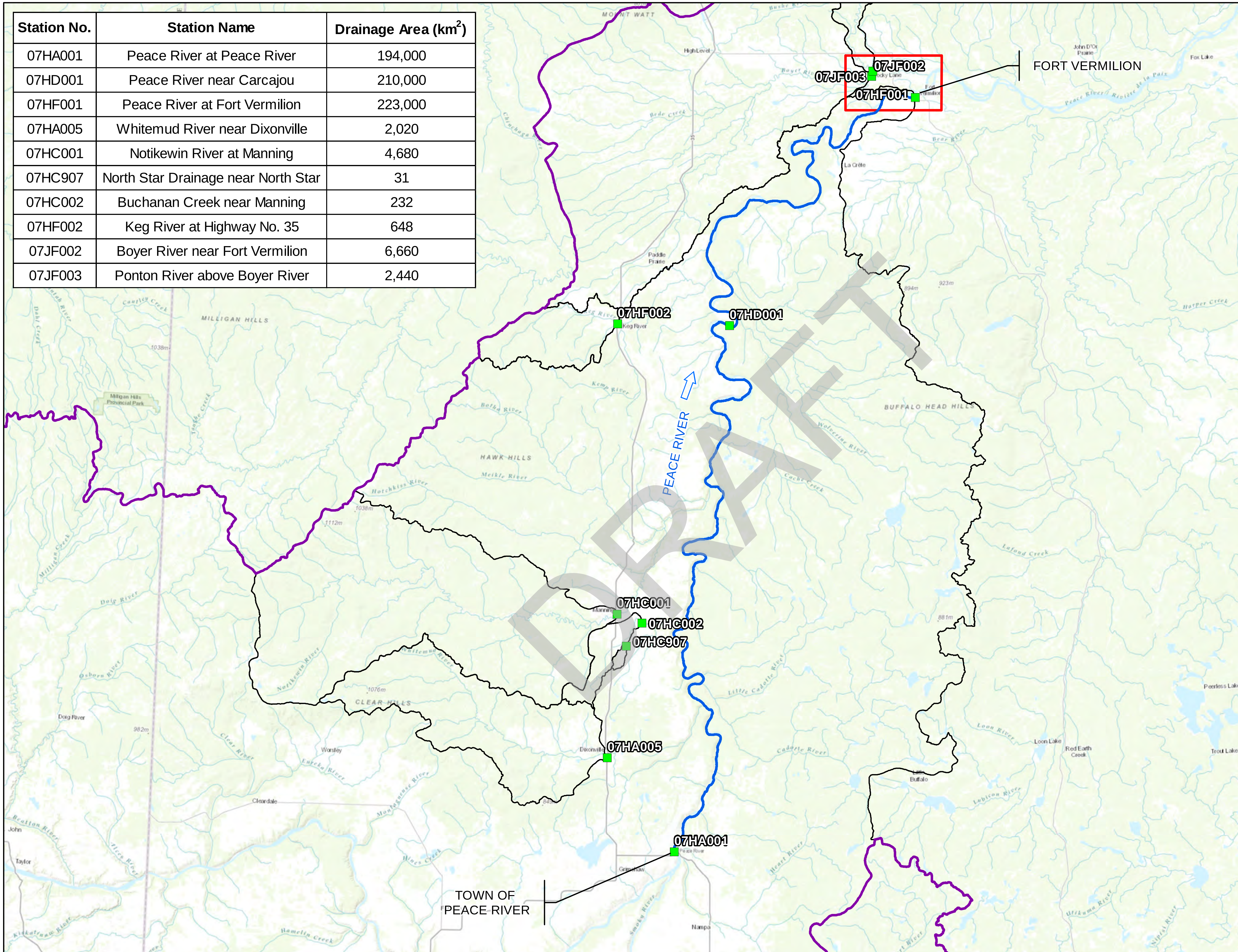
STUDY AREA

FIGURE 1

REH_P:_Projects (Active)\1004662 Camrose Flood Hazard Study\90 GIS\1004659_REH_Fig_T200_StudyArea_R1.mxd

Classification: Public

Station No.	Station Name	Drainage Area (km^2)
07HA001	Peace River at Peace River	194,000
07HD001	Peace River near Carcajou	210,000
07HF001	Peace River at Fort Vermilion	223,000
07HA005	Whitemud River near Dixonville	2,020
07HC001	Notikewin River at Manning	4,680
07HC907	North Star Drainage near North Star	31
07HC002	Buchanan Creek near Manning	232
07HF002	Keg River at Highway No. 35	648
07JF002	Boyer River near Fort Vermilion	6,660
07JF003	Ponton River above Boyer River	2,440



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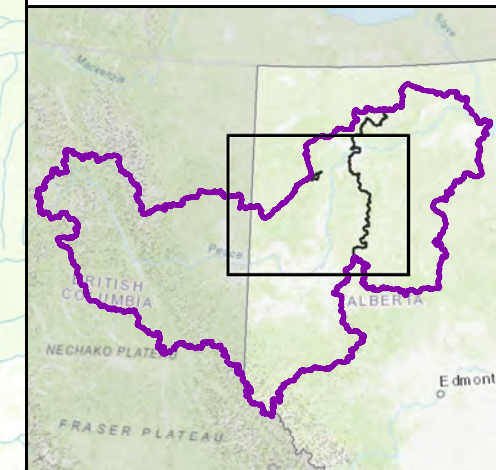


- WSC GAUGE STATION
- ROUTING REACH
- FLOOD HAZARD STUDY AREA
- PEACE RIVER BASIN
- GAUGED SUB-BASIN

DATA SOURCES: Esri National Geographic (primary map) and World Topographic Basemaps (inset)

SCALE - 1:1,150,000
0 10 20 40 KM
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

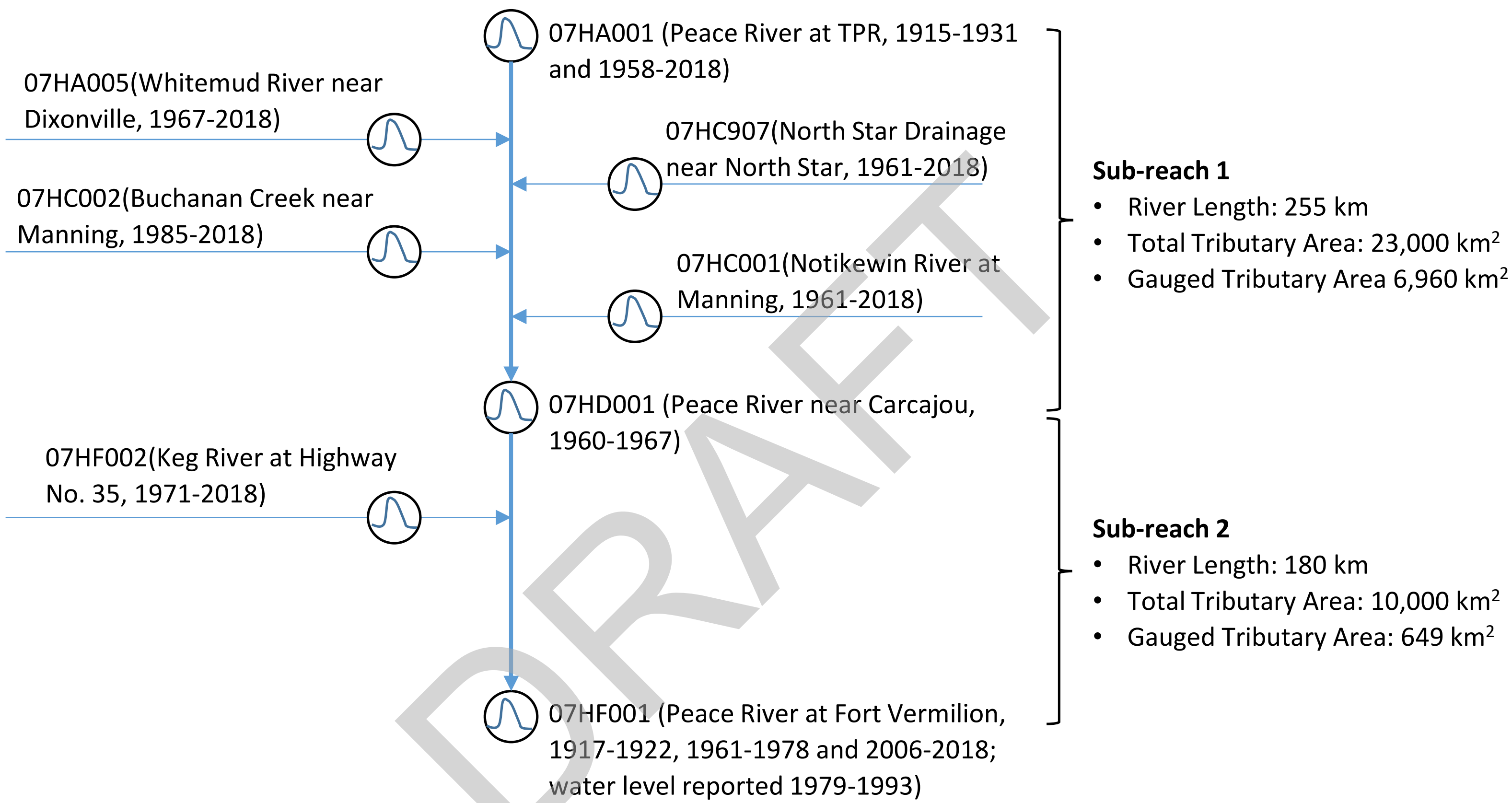
Job: 1004659 | *Date: 21-APR-2020*

FORT VERMILION
FLOOD HAZARD STUDY
OPEN WATER HYDROLOGY ASSESSMENT AREA

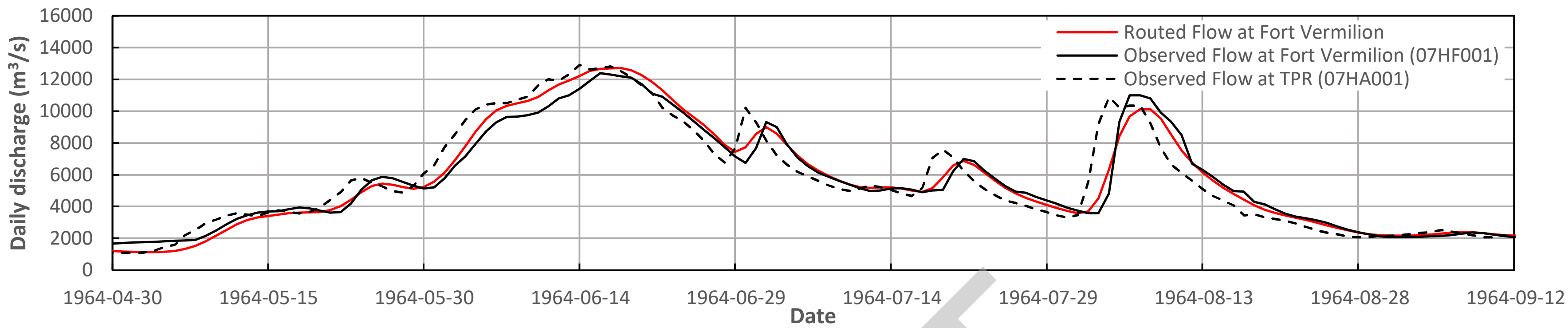
FIGURE 2

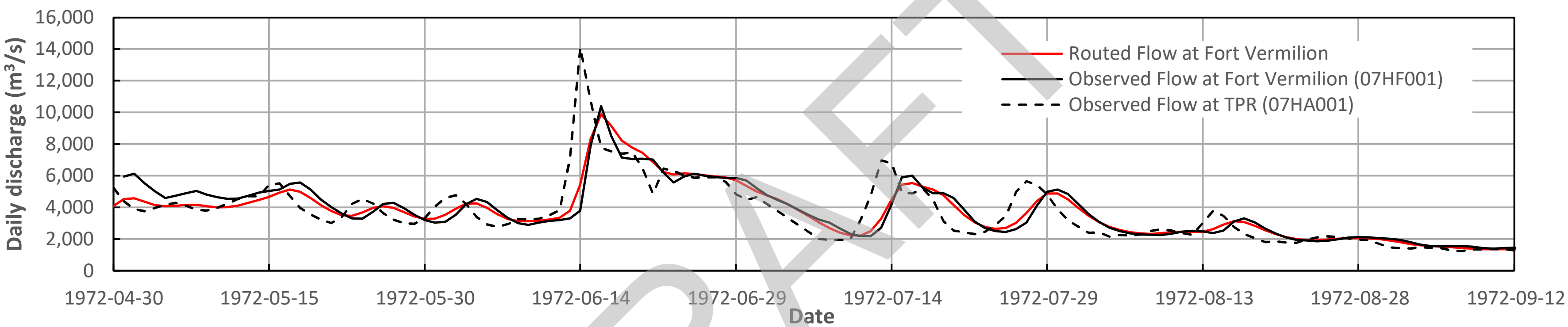
REH, P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\90 GIS\1004659_REH_Fig_T200_ProjectLoc.mxd

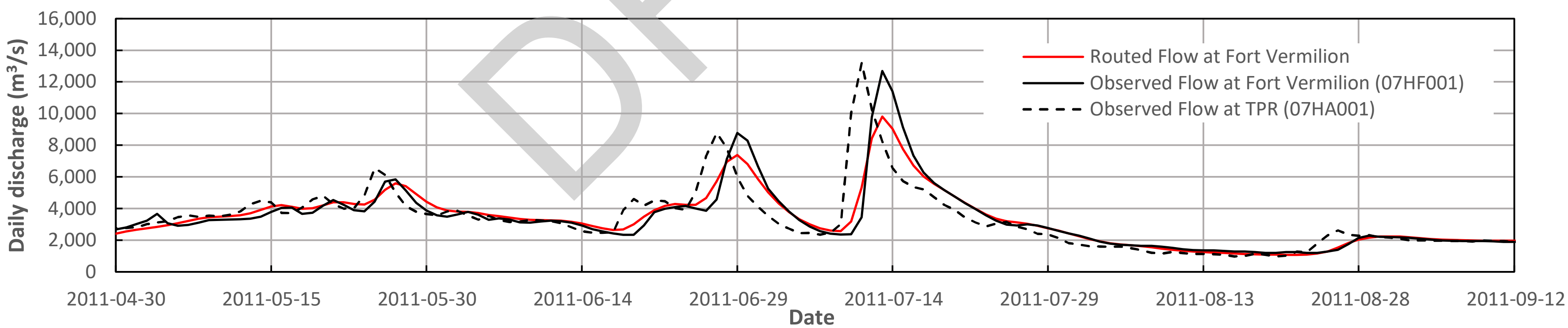
Classification: Public



Alberta	SCALE – AS SHOWN	*FORT VERMILION FLOOD HAZARD STUDY* *OPEN WATER HYDROLOGY ASSESSMENT* *FLOW ROUTING MODEL SCHEMATIC*
	Coordinate System: Units: As Shown	
nhc northwest hydraulic consultants	*Job: 1004659* | *Date: Mar-2020*	*FIGURE 3*







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SCALE – AS SHOWN

Coordinate System:
Units: As Shown

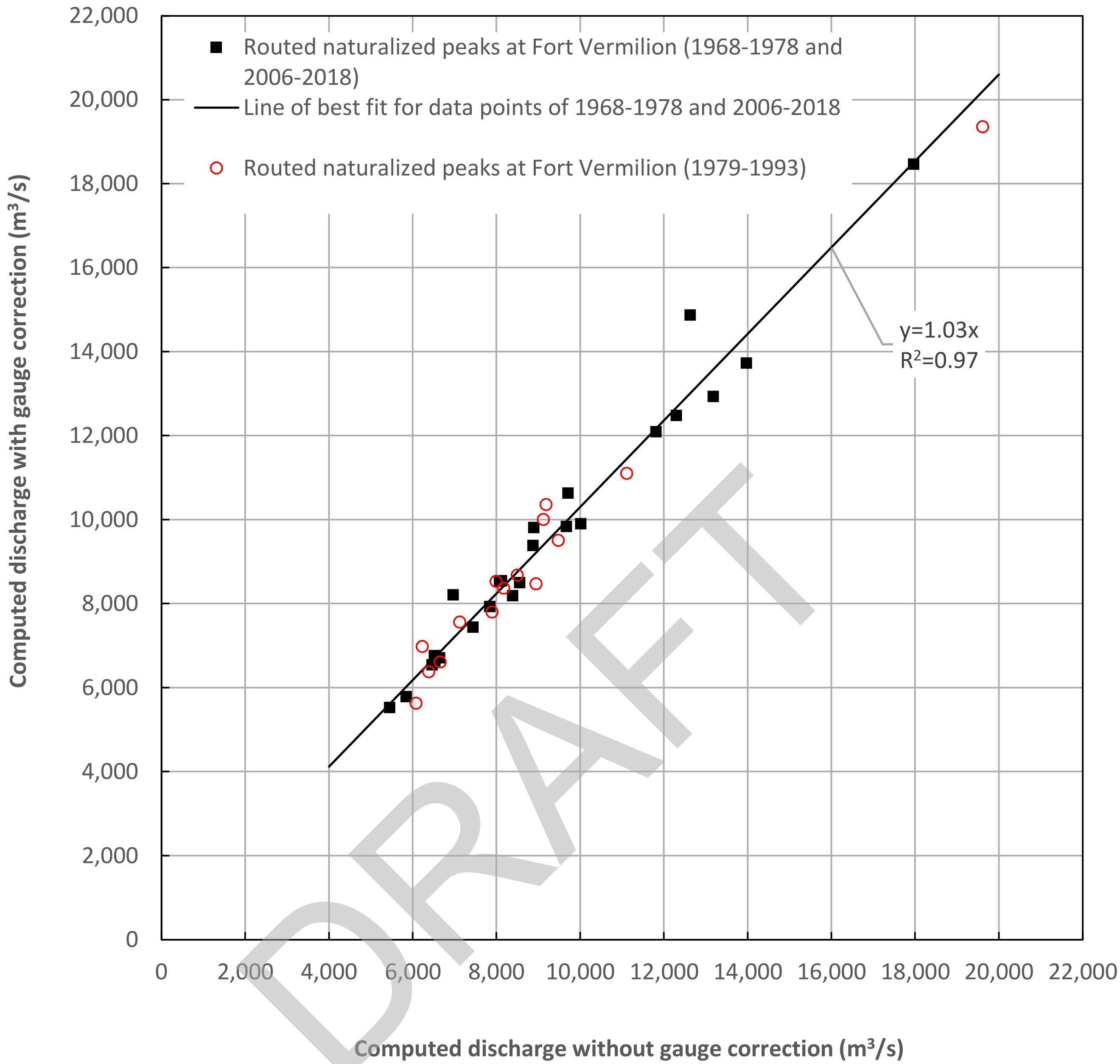
Job: 1004659

Date: Mar-2020

FORT VERMILION FLOOD HAZARD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

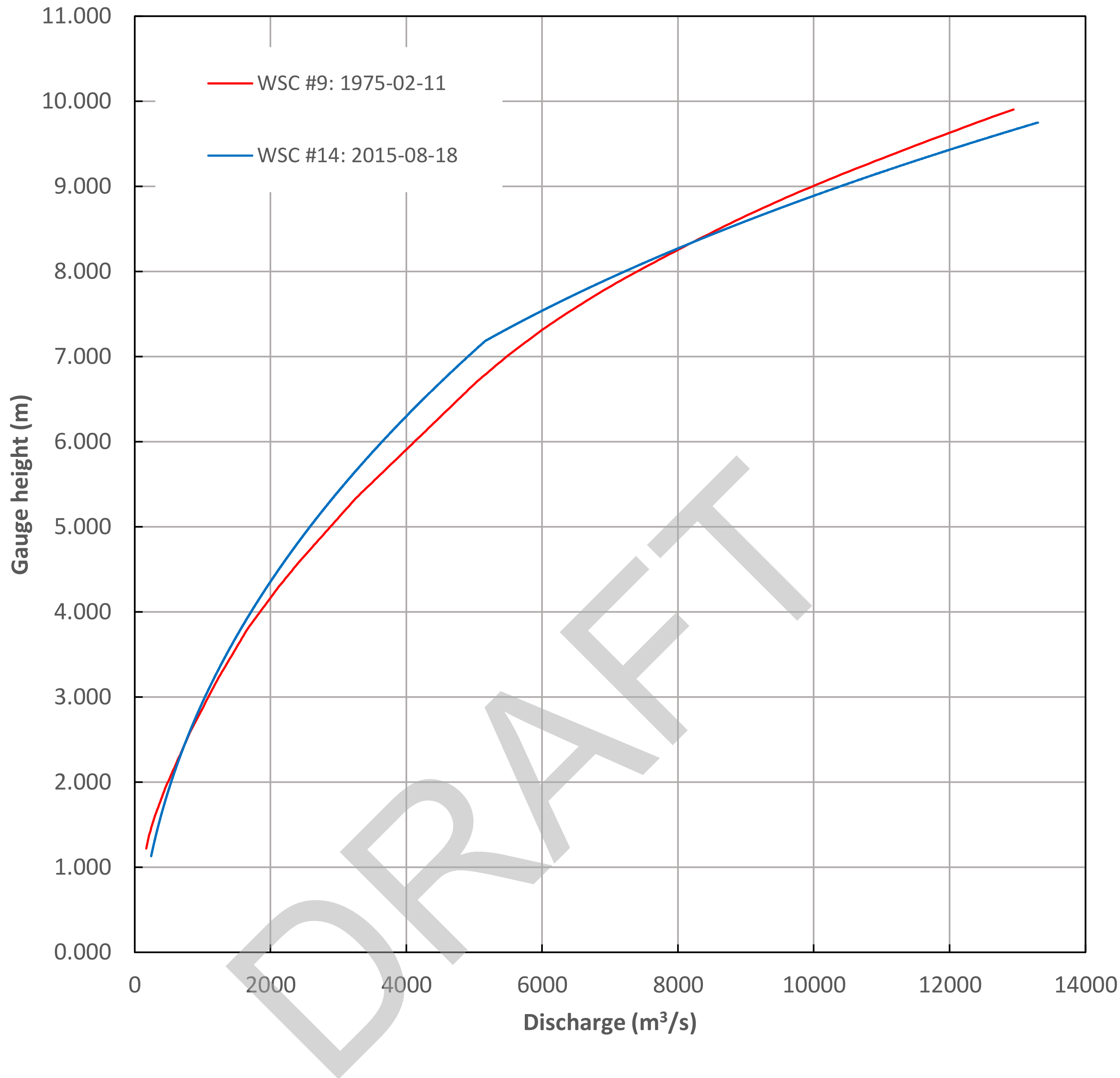
EXAMPLES OF ROUTING MODEL CALIBRATION RESULTS

FIGURE 4



Notes:
1. Discharges at Fort Vermilion were computed by routing the naturalized flows for the town of Peace River (WSC Station 07HA001) to Fort Vermilion (WSC Station 07HF001).
2. Gauge correction for 1968-1978 and 2005-2018 was based on WSC reported discharges at Fort Vermilion.
3. Gauge correction for 1979-1993 was based on discharges estimated from WSC reported gauge heights for Fort Vermilion.

Alberta	SCALE – AS SHOWN	FORT VERMILION FLOOD HAZARD STUDY OPEN WATER HYDROLOGY ASSESSMENT
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	EFFECTS OF GAUGE CORRECTION
	Job: 1004659 / Date: Mar-2019	FIGURE 5



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1004659 | *Date: Mar-2019*

FORT VERMILION FLOOD HAZARD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

COMPARISON OF SELECTED WSC RATING CURVES

FIGURE 6

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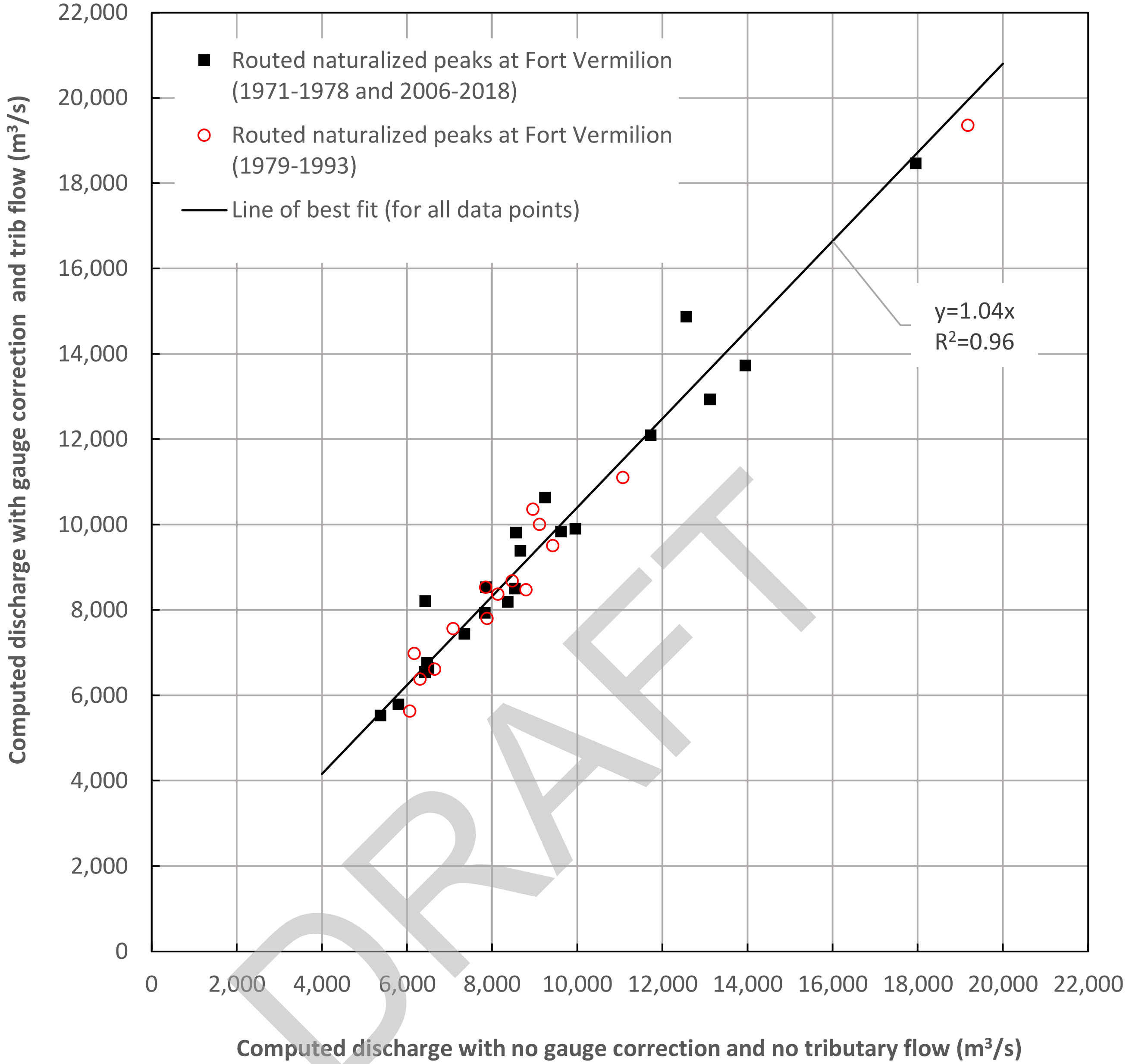
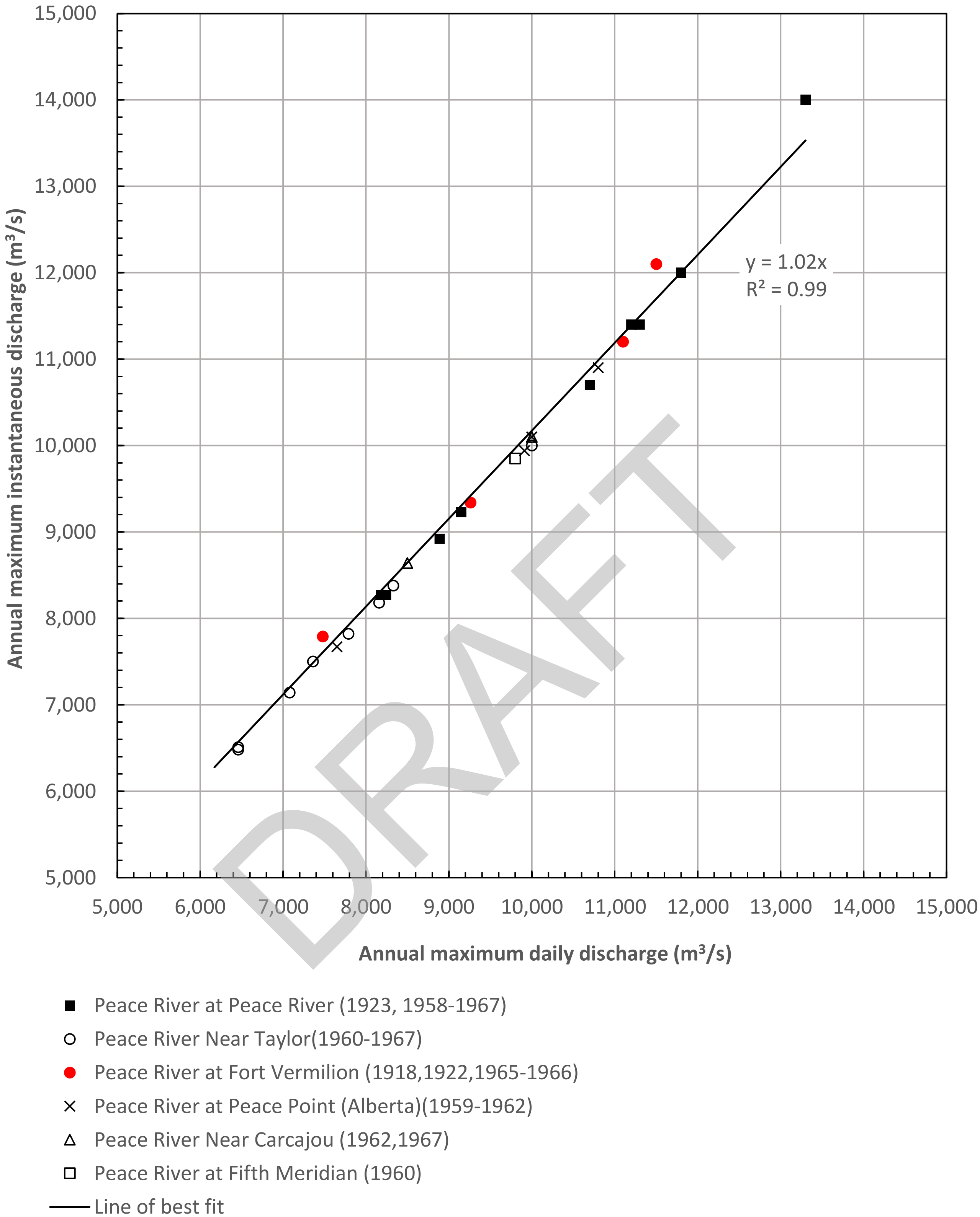
Classification: Public



Notes:
1. Discharges at Fort Vermilion were computed by routing the naturalized flows for the town of Peace River (WSC Station 07HA001) to Fort Vermilion (WSC Station 07HF001).
2. Gauge correction for 1968-1978 and 2005-2018 was based on WSC reported discharges at Fort Vermilion.
3. Gauge correction for 1979-1993 was based on discharges estimated from WSC reported gauge heights for Fort Vermilion.

Alberta	SCALE – AS SHOWN	FORT VERMILION FLOOD HAZARD STUDY OPEN WATER HYDROLOGY ASSESSMENT
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	EFFECTS OF GAUGE CORRECTION AND GAUGED TRIBUTARY INFLOWS
	Job: 1004659 / Date: Mar-2019	FIGURE 7



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SCALE – AS SHOWN

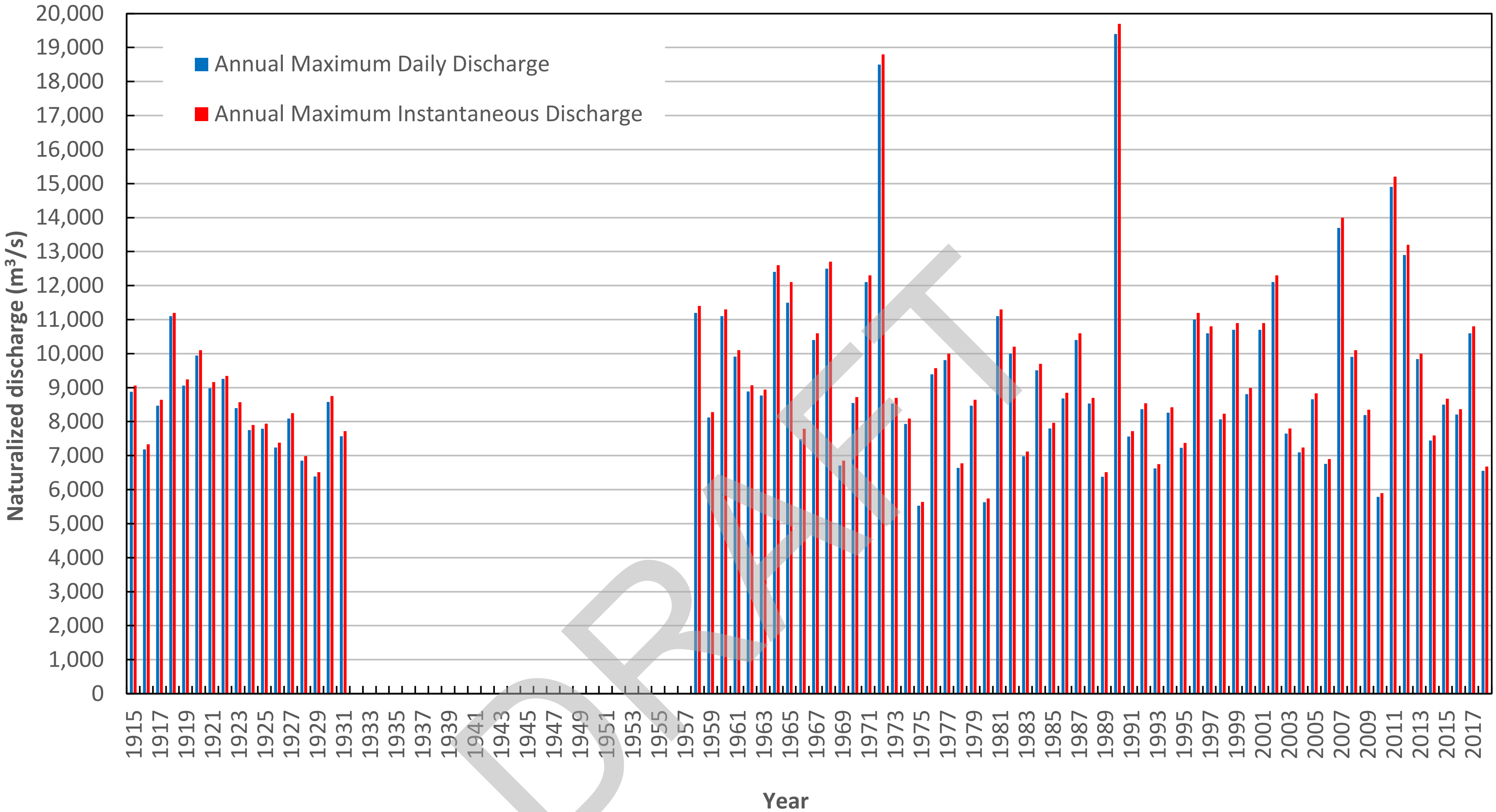
Coordinate System:
Units: As Shown

Job: 1004659 | *Date: Mar-2019*

FORT VERMILION FLOOD HAZARD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

RELATIONSHIPS BETWEEN MAXIMUM INSTANTANEOUS AND DAILY DISCHARGES FOR PEACE RIVER PRE-REGULATION FLOWS

FIGURE 8



Notes:
1. The annual maximum daily discharges are estimates based on routing of flows from the Town of Peace River, except the values for 1917-1922, 1961-1962 and 1964-1967 which are from the pre-regulation record for Peace River at Fort Vermilion (WSC Station 07HF001).
2. The instantaneous discharges were estimated with the daily values being multiplied by 1.02, except for the values of 1918, 1922, 1965 and 1966 which are available from WSC records.

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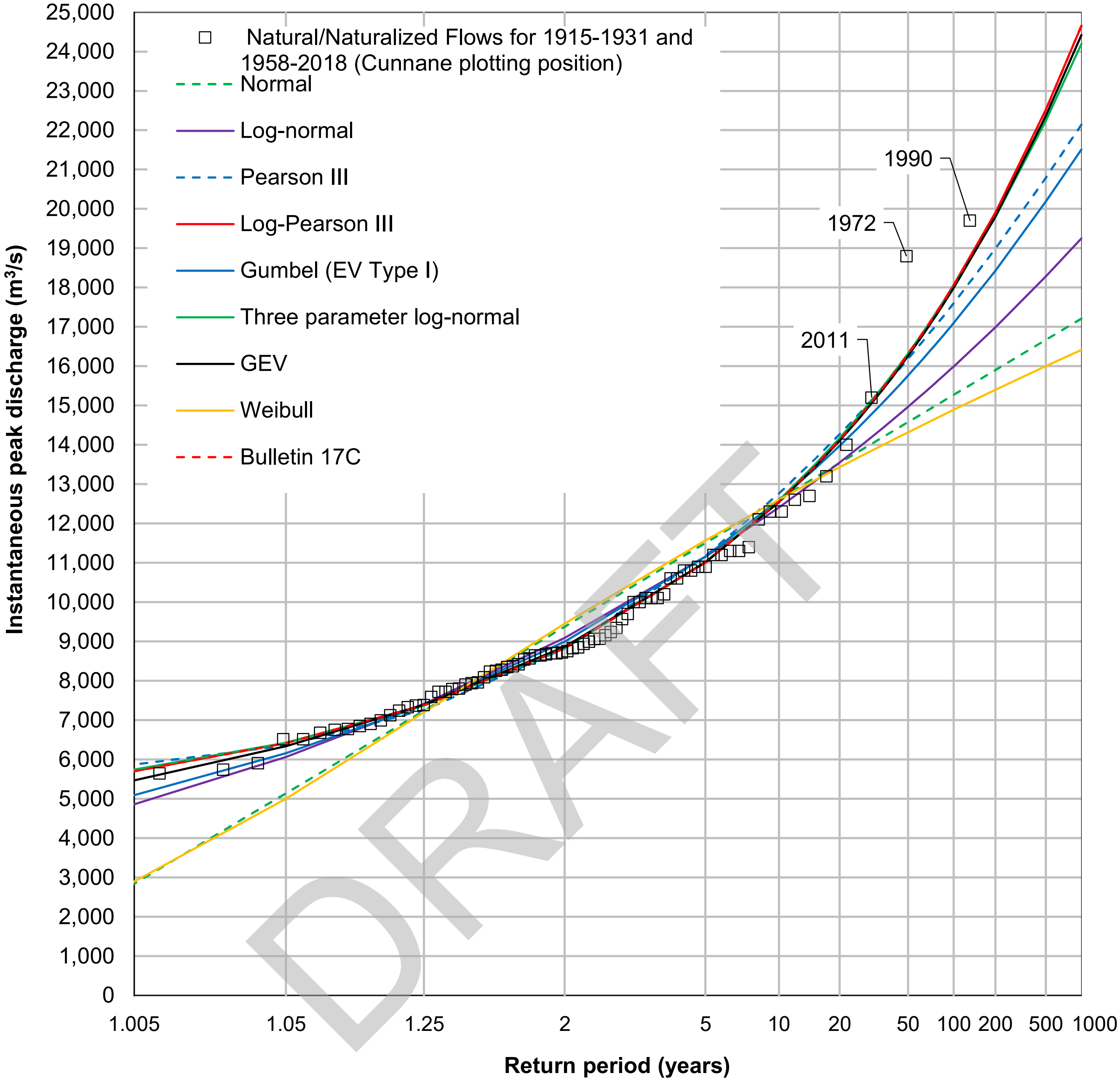
Coordinate System:
Units: As Shown

Job: 1004659

Date: Mar-2020

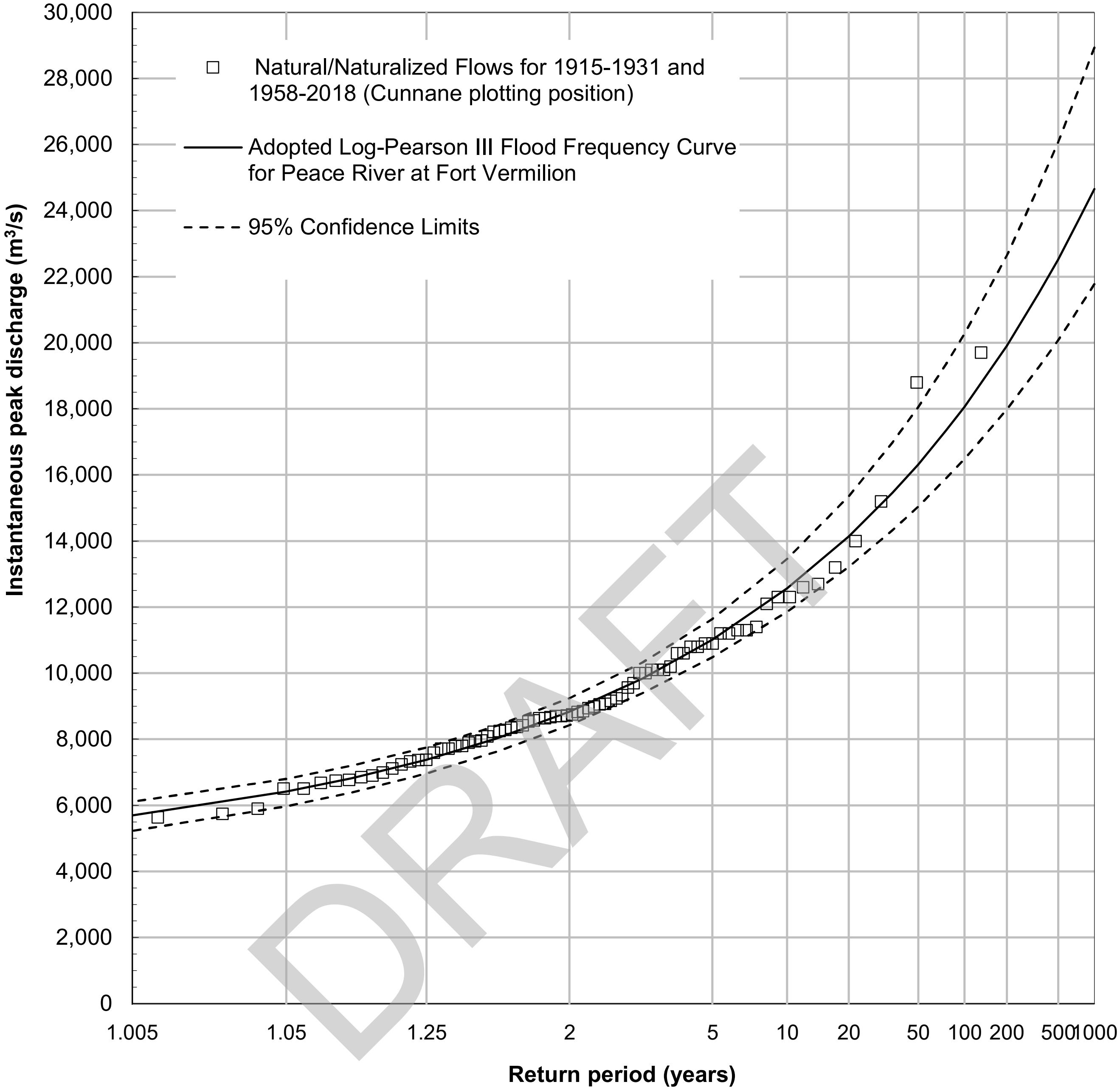
FORT VERMILION FLOOD HAZARD STUDY
OPEN WATER HYDROLOGY ASSESSMENT
ESTIMATED NATURAL AND NATURALIZED MAXIMUM INSTANTANEOUS AND DAILY DISCHARGES FOR PEACE RIVER AT FORT VERMILION

FIGURE 9

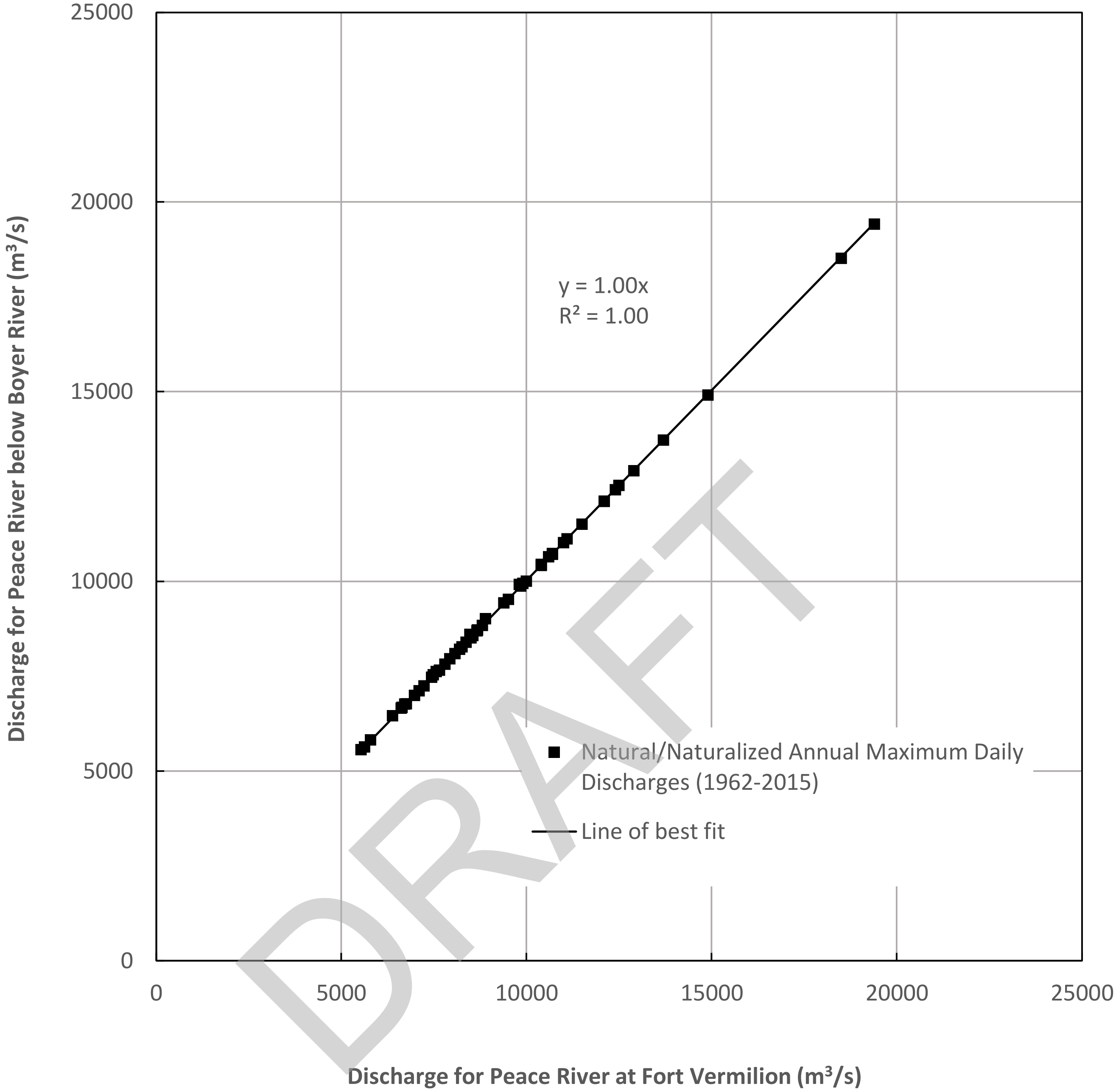


Notes: 1. The log-Pearson III and Bulletin 17C curves are identical.

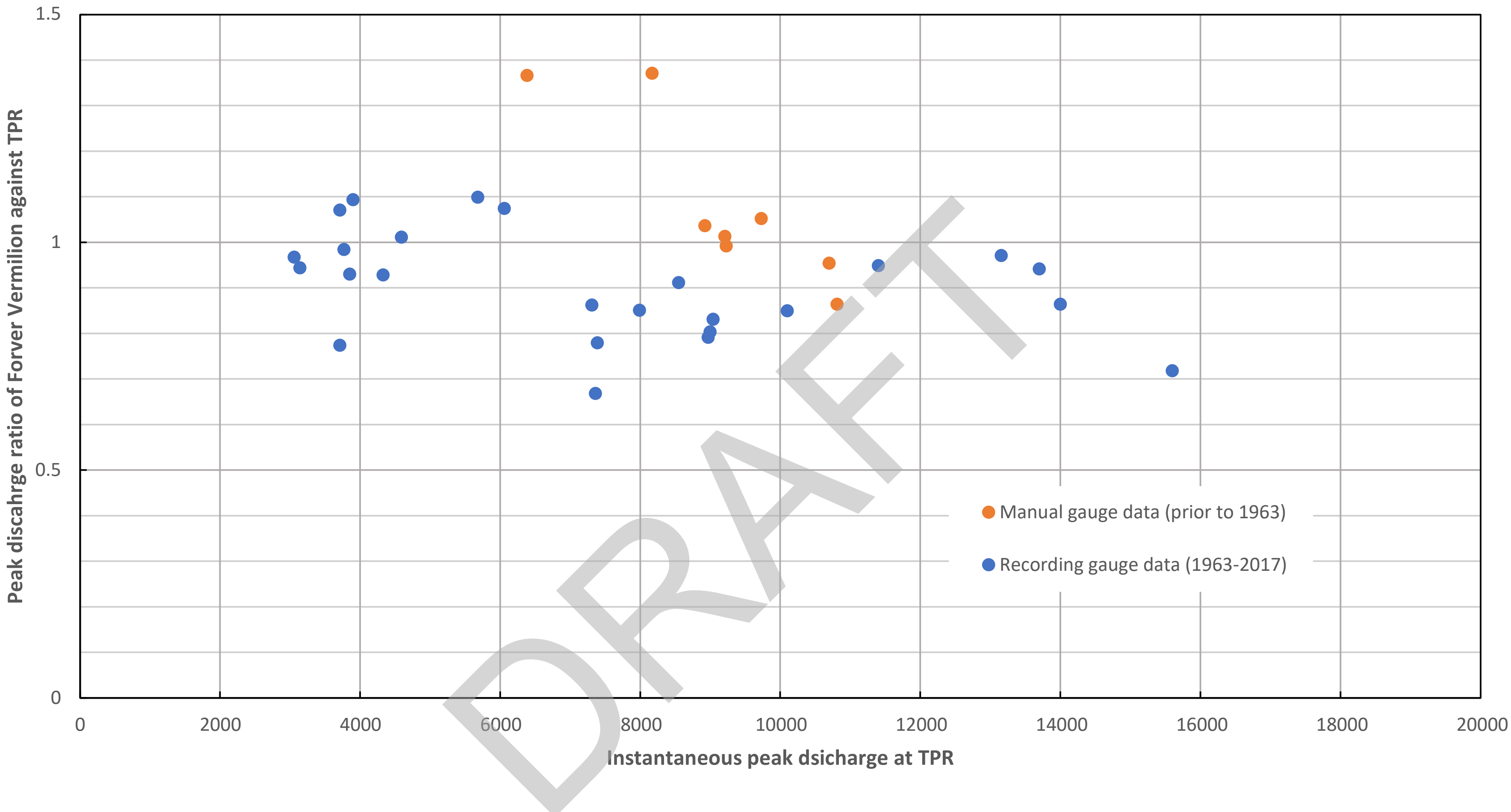
Alberta	SCALE – AS SHOWN	FORT VERMILION FLOOD HAZARD STUDY OPEN WATER HYDROLOGY ASSESSMENT COMPARISON OF FREQUENCY CURVES FOR NATURAL/NATURALIZED PEACE RIVER FLOOD PEAKS AT FORT VERMILION
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	
	Job: 1004659 | *Date: Mar-2019*	*FIGURE 10*



Alberta	SCALE – AS SHOWN	*FORT VERMILION FLOOD HAZARD STUDY OPEN WATER HYDROLOGY ASSESSMENT*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	*ADOPTED NATURAL/NATURALIZED FLOOD FREQUENCY CURVE FOR PEACE RIVER AT FORT VERMILION*
	Job: 1004659 *Date: Mar-2019*	*FIGURE 11*



Alberta / nhc northwest hydraulic consultants	SCALE – AS SHOWN	*FORT VERMILION FLOOD HAZARD STUDY OPEN WATER HYDROLOGY ASSESSMENT*
	Coordinate System: Units: As Shown	*COMPARISON OF PEACE RIVER PEAK DISCHARGES AT FORT VERMILION AND BELOW BOYER RIVER*
	Job: 1004659 *Date: Mar-2019*	*FIGURE 12*



Notes:

1. The plot is based on WSC published gauge data available for both Peace River stations at TPR and at Fort Vermilion, including the periods of 1917-1962, 1964-1978 (missing 1974) and 2006-2017 (excluding 2014 because the WSC indicates that the 2014 instantaneous peak is not valid).
2. Data prior to 1963 were from manual gauge readings and may be less accurate than the data from recording (automatic) gauges used since 1963.

Alberta	SCALE – AS SHOWN	*FORT VERMILION FLOOD HAZARD STUDY* *OPEN WATER HYDROLOGY ASSESSMENT*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	*COMPARISON OF GAUGED PEACE RIVER PEAK DISCHARGES AT TPR WITH AT FORT VERMILION*
	Job: 1004659 *Date: Mar-2020*	*FIGURE 13*

Appendix A
1979-1993 Fort Vermilion Gauge Information from Water Survey of Canada

Ken Zhao

From: Bishop, Zachary (EC) <zachary.bishop@canada.ca>
Sent: October 28, 2019 11:40 AM
To: Ken Zhao
Cc: Hussey, Samantha (EC); SHN Hydrologique AB / NHS Hydrological AB (EC)
Subject: RE: WSC Data Request for AEP Study

Hi Ken,

Unfortunately, the data review does not provide such information. The review specifically focused on the high uncertainty of ice measurements during that period, resulting in the majority of winter discharge being based on comparison stations. The poor monitoring conditions were not expanded on but mentioned as a contributing factor to the degraded data quality. I looked more into the review and it cites poor rating definition contributed it to being a stage-only station for many years. It seems the station was not measured frequently and the measured discharge was often significantly different from the rated discharge, giving low confidence in the derived discharge record. For the period when the station was published as stage-only (1979-1993) it does not appear any discharge measurement were made and therefore we have no idea of the validity of the rating curve during that period.

Although applying the rating during the stage-only period may produce reasonable values, I would still stand by that the rating curve was not validated with measurements and was determined to not meet out QAQC standards at the time of publication. I would not recommend using the 1975 rating curve to derive discharge record or peak flows, I understand a large data gap is limiting but the data does not meet our standards so I would use caution if including it in your study.

Thanks,
Zac Bishop
A/ Hydrological Services Supervisor
Water Survey of Canada, National Hydrological Services
Environment and Climate Change Canada / Government of Canada
854, 220-4th Avenue SE, Calgary, Alberta T2G 4X3
Zachary.bishop@canada.ca/ (403) 819-4003
Environment and Climate Change Canada Environnement et Changement climatique Canada

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From: Ken Zhao <KZhao@nhcweb.com>
Sent: Monday, October 28, 2019 10:34 AM
To: Bishop, Zachary (EC) <zachary.bishop@canada.ca>
Cc: Hussey, Samantha (EC) <samantha.hussey@canada.ca>
Subject: RE: WSC Data Request for AEP Study

Thanks Zac. The information is very useful.

Our goal is to estimate naturalized flood peaks at Fort Vermilion (07HF001). In this flow naturalization process, we route measured flood flows from the town of Peace River to Fort Vermilion, compare the routed flows with Fort Vermilion gauge data, and then use the differences to improve our natural flow estimates. So, we were hoping to be able to use the water level data improve our estimation for those years and our focus is on high flows. The 1990 peak was the highest discharge on the record. Based on the reported water level and historical rating table, we estimated the peak discharge as 12,940 m3/s, which is fairly close to the estimate of 12,640 by Alberta Environment (I believe it was from surveyed highwater mark).

I was wondering if the 1985 data review provided any plots of measurements vs gauge data to show the gauge issue. It would be good to know the range of discharges that the 1985 data review cover.

Again, your help is much appreciated.

Regards,
Ken

Can Hua (Ken) Zhao |Ph.D., P.Eng. |Principal
kzhao@nhcweb.com | Tel: 587-759-7511

From: Bishop, Zachary (EC) <zachary.bishop@canada.ca>
Sent: October 28, 2019 8:57 AM
To: Ken Zhao <KZhao@nhcweb.com>
Cc: Hussey, Samantha (EC) <samantha.hussey@canada.ca>
Subject: RE: WSC Data Request for AEP Study

Hi Ken,

We looked into the missing data and found a data review completed in 1985 which indicated poor gauging conditions, reduced confidence in discharge measurements and low correlation of open water data to composite curves. For those reasons the rating curve did not meet our quality control measures and was not used to convert water level into discharge for that period, and by the same logic I would not recommend using the water level or historical rating to produce discharge.

Hope that helps, let me know if you have any further questions

Zac Bishop
A/ Hydrological Services Supervisor
Water Survey of Canada, National Hydrological Services
Environment and Climate Change Canada / Government of Canada
854, 220-4th Avenue SE, Calgary, Alberta T2G 4X3
Zachary.bishop@canada.ca/ (403) 819-4003
Environment and Climate Change Canada Environnement et Changement climatique Canada

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From: Ken Zhao <KZhao@nhcweb.com>
Sent: October 24, 2019 12:30 PM
To: Hussey, Samantha (EC) <samantha.hussey@canada.ca>; Brown, Amber (EC) <amber.brown@canada.ca>
Subject: RE: WSC Data Request for AEP Study

Hi Amber and Samantha,

We are working on the Fort Vermilion flood hazard study. First of all, thanks for all supports that you have been providing to us.

I noticed that the gauge 07HF001 stopped reporting discharges between 1979 and 2005 but water levels were reported from 1979-1993. I was wondering why the water level data were not converted into discharges for that period. We would like to use the rating table #9 (dated 1975, which is very close to the current rating table) for this gauge to estimate discharges from this water level data. Could you please comment on that?

Regards,
Ken

Can Hua (Ken) Zhao |Ph.D., P.Eng. |Principal
kzhao@nhcweb.com | Tel: 587-759-7511

Appendix F
Flood Photography

a) Old Sheridan Lawrence Ranch
(Photo Date: 22 April 1934)


c) Roman Catholic Church at Fort Vermilion (Photo Date: 22 April 1934)

b) Hudson Bay Company Building (22 April 1934)

Alberta Government

nhc northwest hydraulic consultants

Notes: 1. Images from AT Bridge File 74227-1934 Flood Documentation

FORT VERMILION FLOOD HAZARD STUDY
SUMMARY REPORT

1934 ICE JAM FLOOD

Job: 01004659 | Date: Mar-2021

FIGURE F-1


a) Ice Breakup Opposite the Experimental Farm(Photo Date: 7 May 1950)


b) Ice Breakup Opposite the Experimental Farm (Photo Date: 7 May 1950)

Alberta Government

nhc northwest hydraulic consultants

Notes: 1. Images from AT Bridge File 74227-1950 Flood Documentation

Job: 01004659	Date: Mar-2021

FORT VERMILION FLOOD HAZARD STUDY SUMMARY REPORT

1950 ICE JAM FLOOD

FIGURE F-2

a) Looking Upstream from East of Town (Photo Date: 7 May 1950)

b) River Ice Breakup at Fort Vermilion (Photo Date: 7 May 1950)

c) Looking West on New Approach to South Side of Ferry Crossing – Esimate Water Elevation of 834 ft (Photo Date: 7 May 1950)

d) Looking Southeast at Flooding at Prairie Point Fields (Photo Date: 23 May 1963)

Alberta Government

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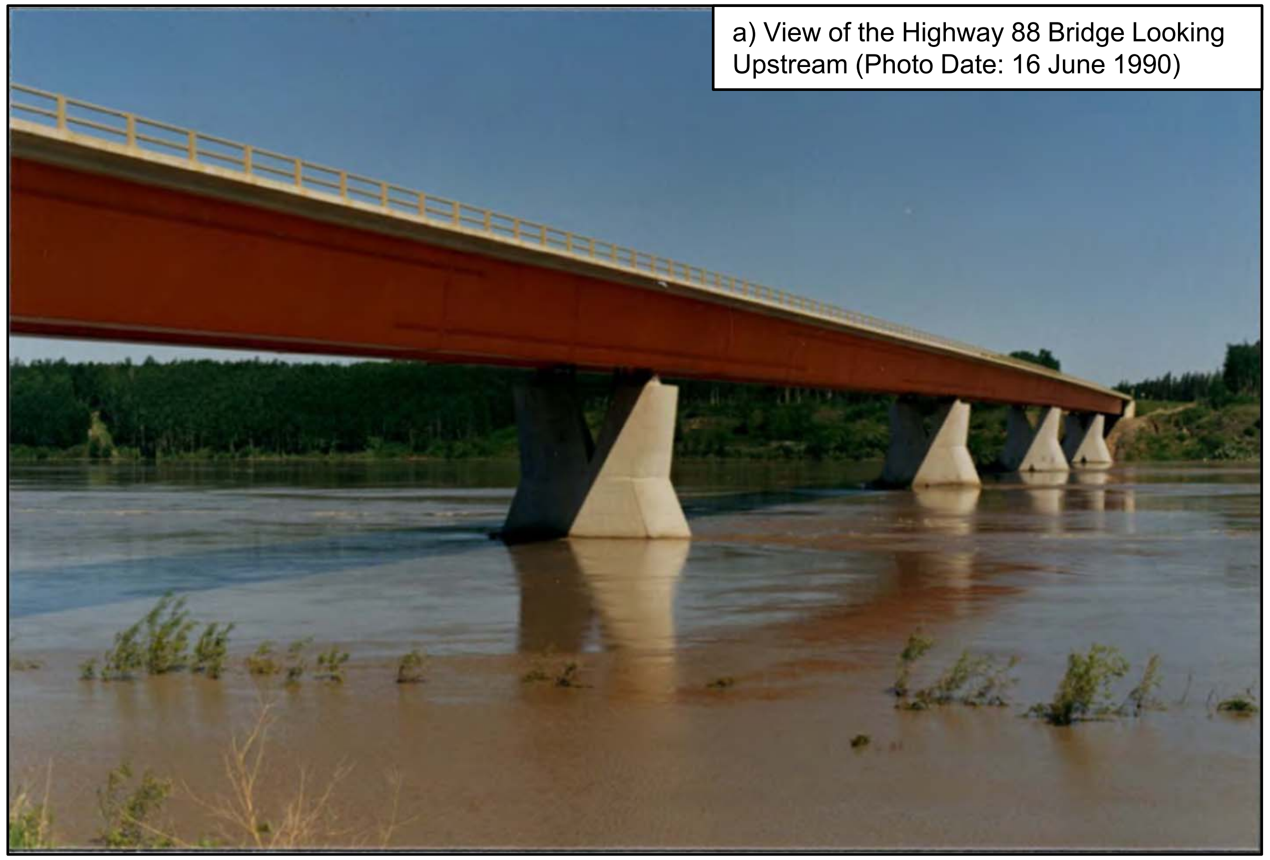
Notes: 1. Images from AT Bridge File 74227-1963 Flood Documentation

Job: 01004659 | Date: Mar-2021

FORT VERMILION FLOOD HAZARD STUDY
SUMMARY REPORT

1963 ICE JAM FLOOD

FIGURE F-3


a) View of the Highway 88 Bridge Looking Upstream (Photo Date: 16 June 1990)


b) View Looking Upstream from Highway 88 Bridge (Photo Date: 16 June 1990)


c) View Looking Downstream from Highway 88 Bridge (Photo Date: 16 June 1990)


d) Looking at the Upstream Side of Culvert through the South Approach to the Highway 88 Bridge (Photo Date: 16 June 1990)

Alberta Government

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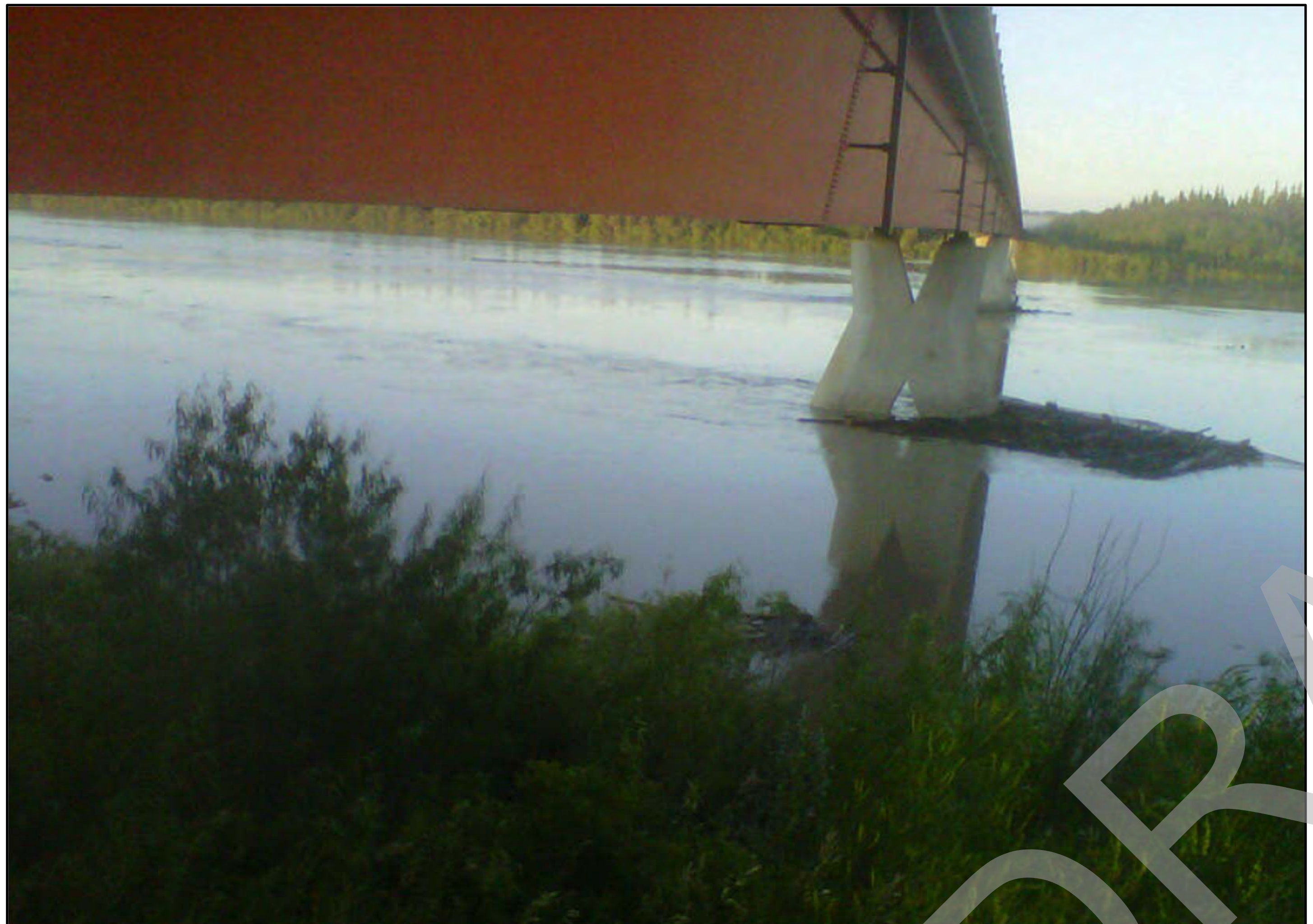
Notes: 1. Images from AT Bridge File 74227-1990 Flood Documentation

FORT VERMILION FLOOD HAZARD STUDY SUMMARY REPORT

1990 OPEN WATER FLOOD

Job: 01004659 | *Date: Mar-2021*

FIGURE F-4


a) Looking South at the Highway 88 Bridge (Photo Date: 14 July 2011)


b) Looking South at the Upstream side of the Culvert through the South Approach of the Highway 88 Bridge (Photo Date: 14 July 2011)

Alberta Government	Notes: 1. Images from AT Bridge File 74227-2011 Flood Documentation		*FORT VERMILION FLOOD HAZARD STUDY SUMMARY REPORT*
nhc northwest hydraulic consultants			*2011 OPEN WATER FLOOD*
		Job: 01004659 / *Date: Mar-2021*	*FIGURE F-5*

a) Ice Accumulation adjacent to DA Thomas Park, R.S. 14,790 m (Photo Date: April 2018)

b) Looking Downstream at Ice Accumulation near the Wop May Memorial Aerodrone (Photo Date: April 2018)

c) Looking East at Flooding on Range Road 131, R.S. 17,365 m (Photo Date: April 2018)

Alberta Government

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Notes: 1. Images from Mackenzie County.

FORT VERMILION FLOOD HAZARD STUDY
SUMMARY REPORT

2018 ICE JAM FLOOD

Job: 01004659 | Date: Mar-2021

FIGURE F-6

a) Damage to Tree along River Road, R.S. 16,820 m (Photo Date: April 2018)

b) Flooding along Township Road 1084A, R.S. 19,720 m (Photo Date: April 2018)

c) Flooded land north of Township Road 1084A, R.S. 17,830 (Photo Date: April 2018)

DRAFT

Alberta Government

nhc northwest hydraulic consultants

Notes: 1. Images from Mackenzie County.

FORT VERMILION FLOOD HAZARD STUDY
SUMMARY REPORT

2018 ICE JAM FLOOD

Job: 01004659

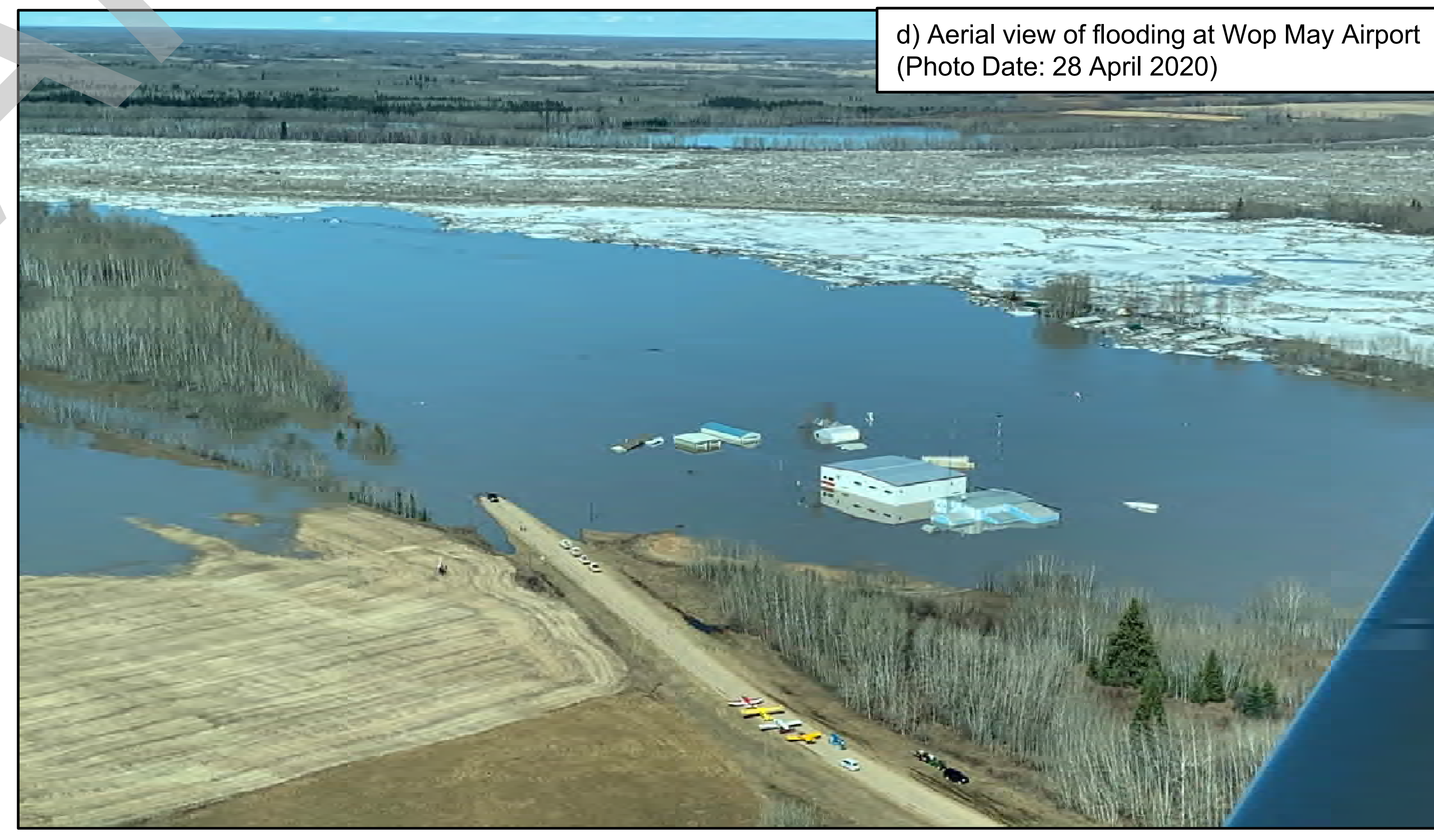
Date: Mar-2021

FIGURE F-7


a) Aerial view of flooding in Fort Vermilion (Photo Date: 28 April 2020)


b) Aerial view of flooding in Fort Vermilion (Photo Date: 28 April 2020)


c) South approach to Hwy 88 bridge flooded (Photo Date: 28 April 2020)


d) Aerial view of flooding at Wop May Airport (Photo Date: 28 April 2020)

Alberta Government

nhc northwest hydraulic consultants

Notes: 1. Images provided by AEP.

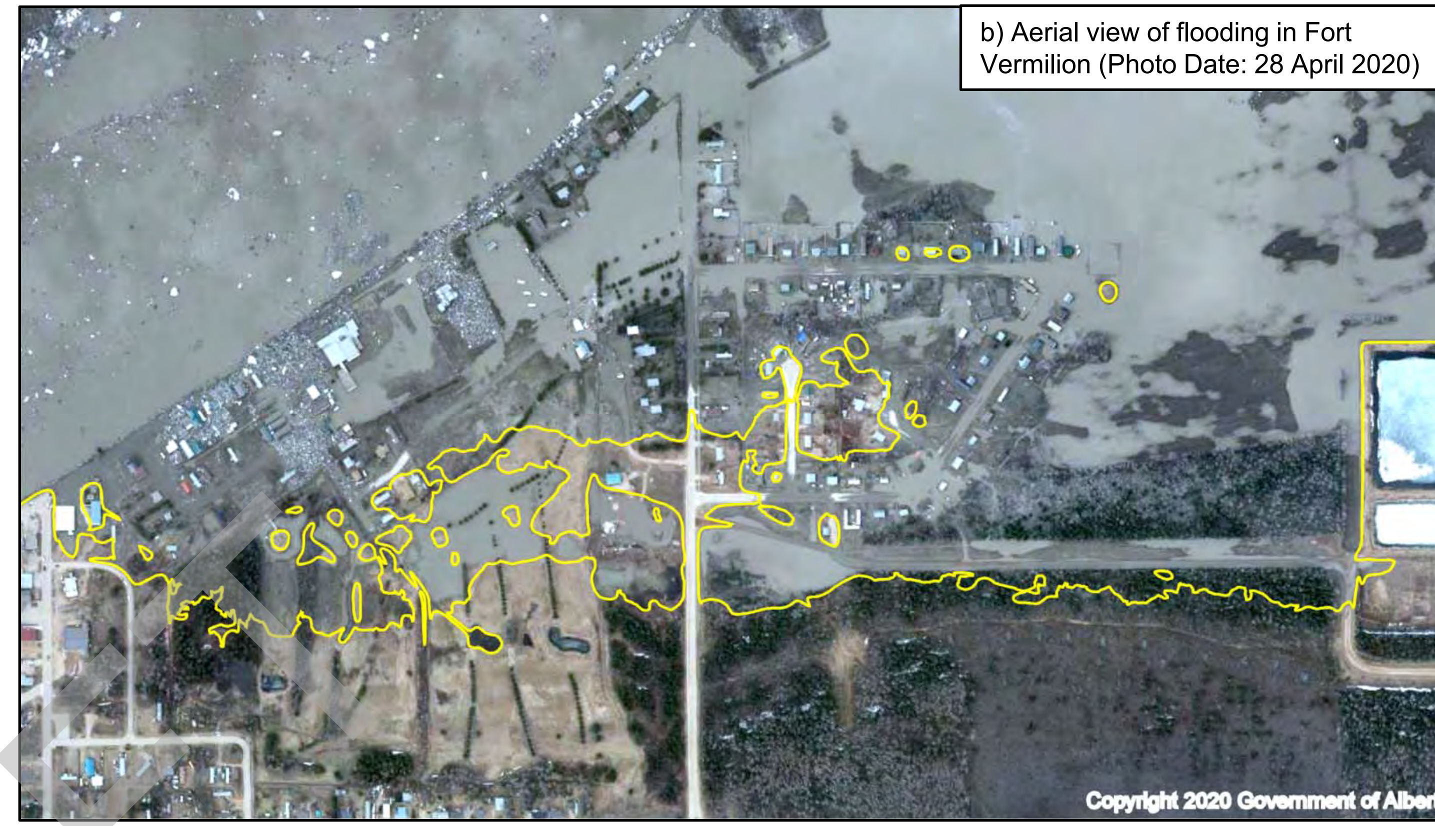
FORT VERMILION FLOOD HAZARD STUDY
SUMMARY REPORT

2020 ICE JAM FLOOD

Job: 01004659 | Date: Mar-2021

FIGURE F-8

a) Ground view of ice floes in Fort Vermilion (Photo Date: 28 April 2020)



b) Aerial view of flooding in Fort Vermilion (Photo Date: 28 April 2020)

c) Aerial view of flooding near Wop May Airport (Photo Date: 28 April 2020)

d) Aerial view of flooding in North Vermilion (Photo Date: 28 April 2020)

Alberta Government

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Notes:
1. Images provided by AEP.
2. Photos (b), (c), and (d): thin yellow line delineates the flood extents calculated during 2021 model validation (refer to report Section 5.3.4)
3. Photos (b), (c), and (d): approximate scale is 1:10,000

FORT VERMILION FLOOD HAZARD STUDY SUMMARY REPORT

2020 ICE JAM FLOOD

Job: 01004659 | *Date: Mar-2021*

FIGURE F-9

Appendix G
Open Water Flood Frequency Water Levels

Table G- 1 Computed open water flood frequency water levels – Peace River at Fort Vermilion

River Station (m)	Flood Return Period												
	2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
	Water Surface Elevation (m)												
32,785	253.50	254.15	254.53	254.88	255.42	255.72	256.08	256.36	256.96	257.48	257.79	258.18	258.45
31,762	253.42	254.07	254.43	254.77	255.29	255.58	255.93	256.20	256.78	257.28	257.59	257.98	258.24
30,910	253.37	254.01	254.37	254.70	255.21	255.49	255.83	256.09	256.67	257.16	257.46	257.83	258.09
29,866	253.30	253.93	254.29	254.61	255.12	255.40	255.73	256.00	256.56	257.05	257.34	257.72	257.97
29,050	253.29	253.94	254.31	254.64	255.16	255.44	255.78	256.05	256.63	257.12	257.42	257.80	258.06
27,860	253.23	253.89	254.27	254.62	255.15	255.43	255.78	256.05	256.64	257.14	257.44	257.83	258.09
26,255	253.07	253.71	254.08	254.42	254.95	255.24	255.59	255.87	256.46	256.96	257.27	257.65	257.92
24,968	252.99	253.65	254.02	254.36	254.89	255.18	255.52	255.79	256.38	256.88	257.18	257.57	257.83
24,113	252.87	253.50	253.85	254.18	254.69	254.96	255.30	255.57	256.13	256.62	256.91	257.29	257.54
23,634	252.80	253.40	253.74	254.04	254.54	254.81	255.13	255.39	255.95	256.42	256.72	257.09	257.34
23,591	252.78	253.38	253.70	254.01	254.50	254.77	255.09	255.35	255.89	256.36	256.64	257.00	257.25
22,698	252.71	253.31	253.64	253.95	254.44	254.71	255.03	255.29	255.84	256.31	256.59	256.96	257.20
21,545	252.65	253.26	253.59	253.91	254.40	254.67	255.00	255.25	255.81	256.28	256.56	256.93	257.17
20,409	252.64	253.26	253.61	253.94	254.45	254.72	255.05	255.31	255.87	256.35	256.64	257.01	257.26
19,433	252.58	253.21	253.57	253.90	254.41	254.69	255.02	255.28	255.85	256.33	256.62	256.99	257.25
18,416	252.45	253.07	253.41	253.74	254.24	254.52	254.86	255.12	255.69	256.18	256.47	256.85	257.10
17,365	252.38	253.00	253.35	253.68	254.19	254.47	254.81	255.07	255.64	256.13	256.42	256.80	257.05
16,428	252.30	252.92	253.27	253.60	254.10	254.38	254.72	254.98	255.55	256.04	256.33	256.70	256.96
15,592	252.16	252.74	253.05	253.35	253.83	254.10	254.42	254.67	255.22	255.69	255.98	256.34	256.59
14,680	252.10	252.68	253.00	253.30	253.79	254.05	254.38	254.63	255.18	255.65	255.94	256.31	256.56
13,690	252.08	252.68	253.00	253.31	253.80	254.07	254.40	254.65	255.21	255.68	255.97	256.34	256.59
12,889	252.04	252.66	252.99	253.31	253.81	254.08	254.41	254.67	255.24	255.72	256.01	256.38	256.64
11,953	252.00	252.62	252.97	253.30	253.80	254.08	254.41	254.68	255.24	255.73	256.03	256.40	256.66
11,482	251.96	252.59	252.94	253.27	253.78	254.06	254.40	254.67	255.24	255.73	256.02	256.40	256.66
11,025	251.90	252.54	252.90	253.23	253.75	254.03	254.38	254.64	255.22	255.71	256.01	256.38	256.64

Table G- 1 Computed open water flood frequency water levels – (continued)

River Station (m)	Flood Return Period												
	2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
	Water Surface Elevation (m)												
10,282	251.85	252.47	252.83	253.16	253.68	253.96	254.31	254.57	255.15	255.64	255.94	256.32	256.58
9,512	251.71	252.32	252.66	252.99	253.52	253.81	254.17	254.45	255.04	255.54	255.84	256.22	256.48
8,634	251.64	252.26	252.60	252.93	253.46	253.75	254.11	254.38	254.97	255.47	255.77	256.16	256.42
7,958	251.61	252.22	252.57	252.90	253.43	253.72	254.07	254.35	254.93	255.43	255.74	256.12	256.38
7,256	251.55	252.16	252.51	252.84	253.37	253.66	254.01	254.28	254.87	255.36	255.66	256.04	256.30
6,402	251.48	252.11	252.46	252.80	253.33	253.62	253.96	254.24	254.82	255.31	255.61	255.99	256.24
5,260	251.36	251.99	252.34	252.67	253.20	253.48	253.83	254.10	254.67	255.16	255.46	255.83	256.09
4,192	251.25	251.87	252.23	252.56	253.08	253.37	253.72	253.99	254.57	255.06	255.36	255.73	255.99
3,352	251.10	251.71	252.03	252.34	252.86	253.15	253.49	253.76	254.34	254.83	255.13	255.51	255.76
2,234	250.96	251.55	251.87	252.16	252.67	252.95	253.29	253.55	254.12	254.61	254.91	255.28	255.54
1,117	250.89	251.49	251.81	252.12	252.63	252.91	253.25	253.51	254.08	254.56	254.86	255.23	255.48
0	250.80	251.41	251.74	252.05	252.56	252.84	253.18	253.45	254.02	254.50	254.80	255.17	255.42

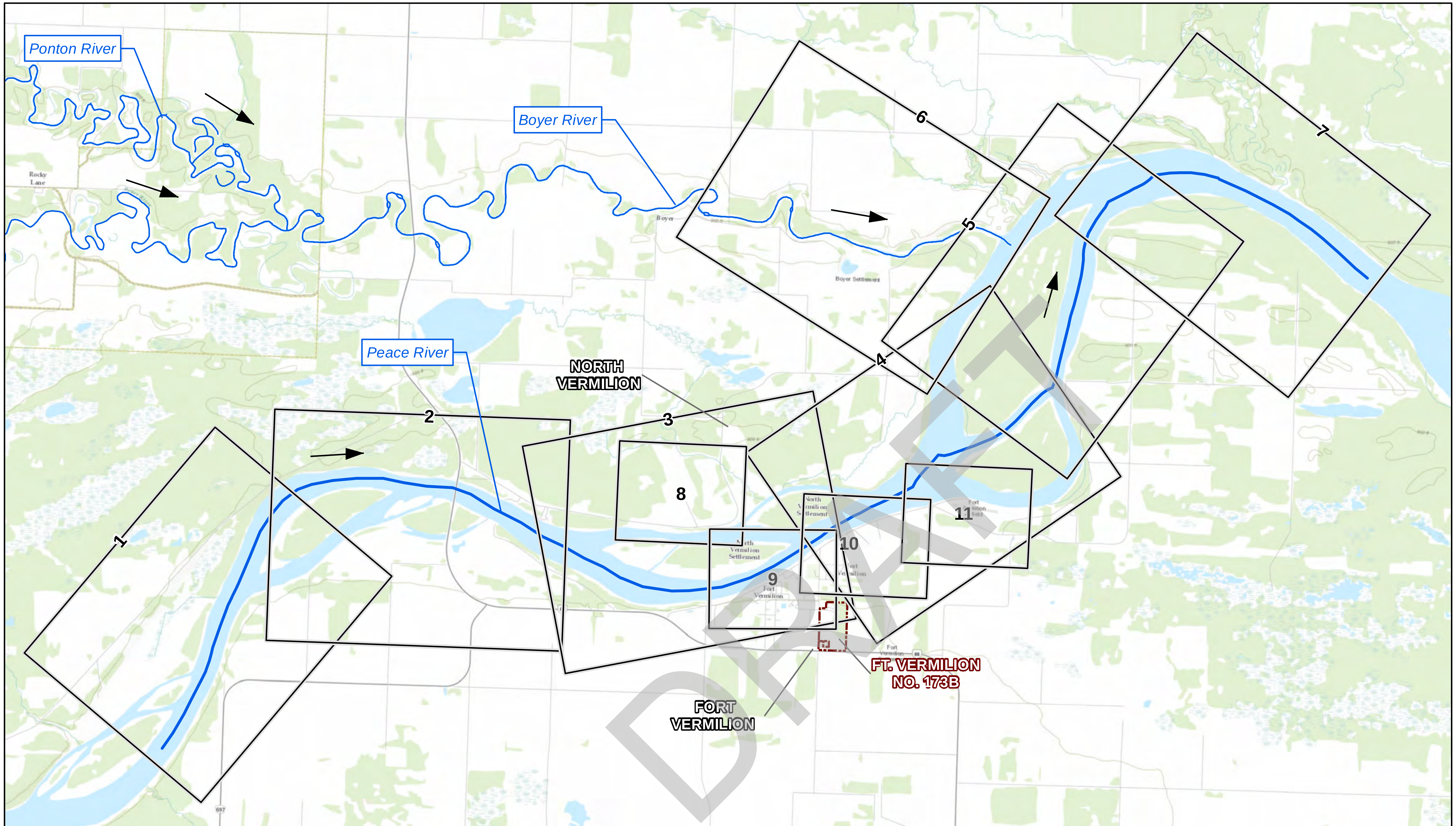
Appendix H
Open Water Flood Inundation Map Library

(submitted under separate cover)

Appendix I
Ice Jam Flood Inundation Map Library

(submitted under separate cover)

Appendix J
Ice Jam Floodway Criteria Maps



Alberta

nhc northwest hydraulic consultants

FLOW DIRECTION
MAP SHEET
INDIAN RESERVE
STUDY REACH
MAJOR TRIBUTARY

SCALE - 1:85,000
0 1 2 4 KM
N

Coordinate System: NAD 1983 CSRS 3TM 114; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	*GIS* REH/MMM	*Reviewer* DJH

Job: 1006302 | Date: 04-NOV-2022

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

INDEX MAP

Notes to Users:

1. Please refer to the accompanying **Fort Vermilion Flood Hazard Study** for important information concerning these maps.
2. Within the flood inundation areas shown on this map, there may be isolated pockets of high ground. To determine whether or not a particular site is subject to flooding, reference should be made to the computed flood levels in conjunction with site-specific surveys where detailed definition is required.
3. Non-riverine and local sources of water have not been considered, and structures such roads and railways can restrict water flow and affect local flood levels. Channel obstruction, local stormwater inflow, groundwater seepage or other land drainage can cause flood levels to exceed those indicated on the map. Lands adjacent to a flooded area may be subject to flooding from tributary streams not indicated on the maps.
4. The flood inundation area is shown above the linework for bridges and flood control structures that are below flood levels.

Definitions:

Flood Hazard Map - A flood hazard map is a specific type of flood map that identifies the area flooded for the 1:100 design flood, and divides that flood hazard area into floodway and flood fringe zones. Flood hazard maps can also show additional flood hazard information, including the incremental areas at risk for more severe floods like the 1:200 and 1:500 floods. Flood hazard maps are typically used for long-term flood hazard area management and land-use planning.

Design Flood - The design flood standard in Alberta is the 1:100 flood, which is a flood that has a 1% chance of being equaled or exceeded in any given year. The design flood is typically based on the 1:100 open water flood, but it can also reflect 1:100 ice jam flood levels or be based on a historical flood event. Different sized floods have different chances of occurring – for example, a 1:200 flood has a 0.5% chance of occurring in any given year and a 1:500 flood has a 0.2% chance of occurring in any given year – but only the 1:100 design flood is used to define the floodway and flood fringe zones on flood hazard maps.

Floodway - When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard where flows are deepest, fastest, and most destructive during the 1:100 design flood. When a flood hazard map is updated, the floodway will not get larger in most circumstances to maintain long-term regulatory certainty, even if the flood hazard area gets larger or design flood levels get higher.

Flood Fringe - The flood fringe is the area outside of the floodway that is flooded or could be flooded during the 1:100 design flood. The flood fringe typically represents areas with

Definitions (continued):

shallower, slower, and less destructive flooding, but it may also include "high hazard flood fringe" areas. Areas at risk of flooding behind flood berms may also be mapped as "protected flood fringe" areas.

High Hazard Flood Fringe - The high hazard flood fringe identifies areas within the flood fringe with deeper or faster moving water than the rest of the flood fringe. High hazard flood fringe areas are likely to be most significant for flood maps that are being updated, but they may also be included in new flood maps.

Protected Flood Fringe - The protected flood fringe identifies areas that could be flooded if dedicated flood berms fail or do not work as designed during the 1:100 design flood, even if they are not overtopped. Protected flood fringe areas are part of the flood fringe and do not differentiate between areas with deeper or faster moving water and shallower or slower moving water.

Data Sources and References:

1. Orthophoto imagery acquired by OGL Engineering for Alberta Environment and Parks: *Imagery acquired on 16 June 2019.*
2. Base data from Natural Resources Canada, Alberta Environment and Parks, and Altalis.
3. Additional base mapping from Esri.

Alberta

nhc
northwest hydraulic consultants

FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500
0 100 200 400 600 800 M

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

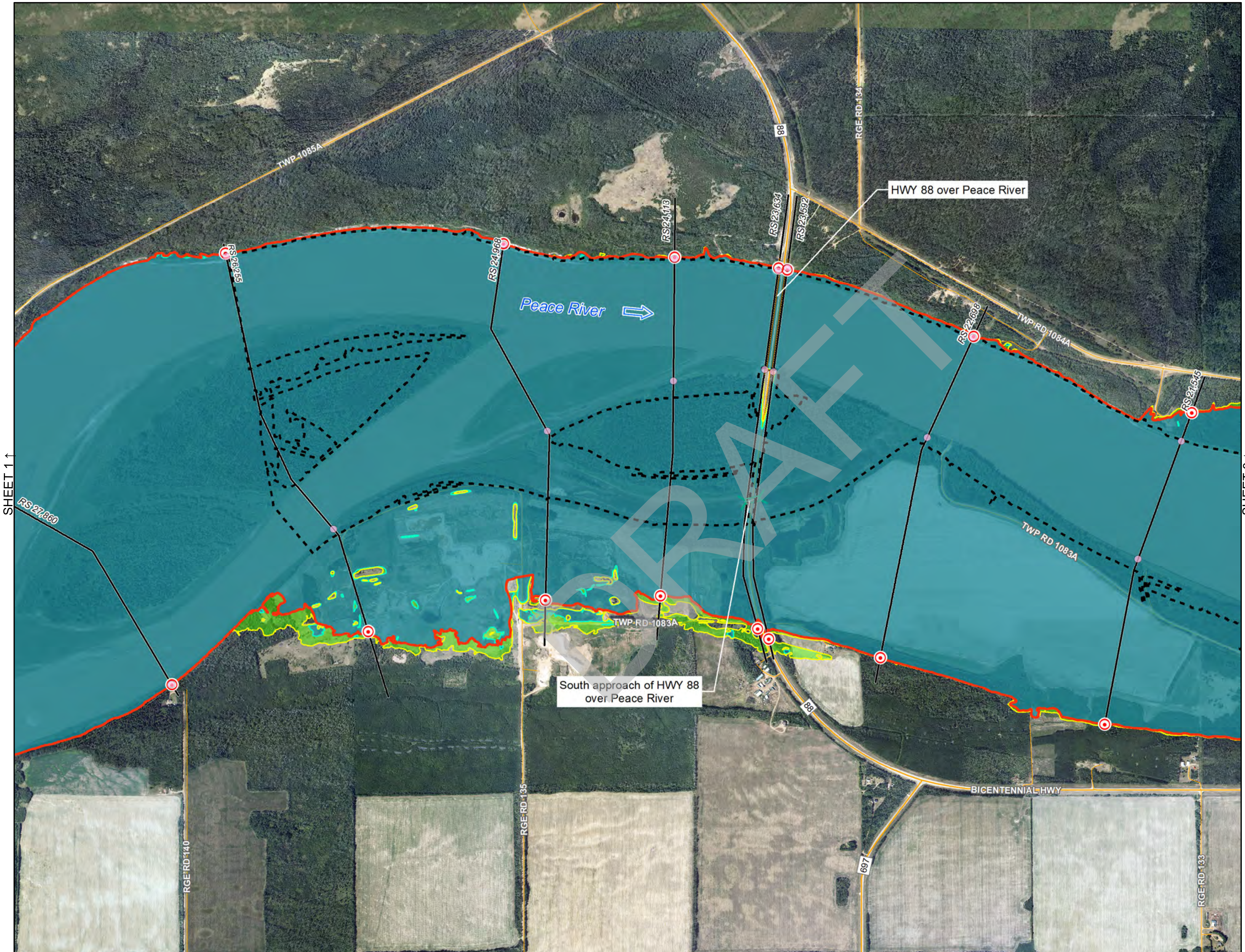
FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

SHEET 1 OF 11

M:\PJ_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_IJFloodway_2022Update.mxd

Classification: Public



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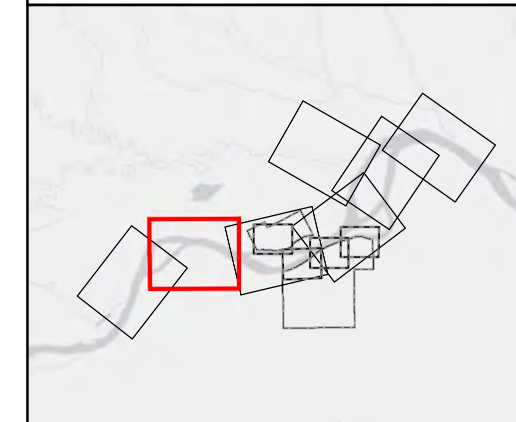

- FLOW DIRECTION
- BRIDGE
- CULVERT
- STUDY LIMIT
- CROSS SECTION
- LOCAL ROAD
- PROVINCIAL ROAD
- INDIAN RESERVE
- MUNICIPAL BOUNDARY
- ZOOMED-IN MAP SHEET
- PROPOSED FLOODWAY BOUNDARY
- PROPOSED FLOODWAY STATION
- BANK STATION
- 100-YEAR DESIGN FLOOD EXTENT
- DEPTH ≥ 1 m
- PREVIOUS FLOODWAY

SCALE - 1:17,500

0 100 200 400 600 800 M

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

SHEET 2 OF 11

M:\P_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_UFloodway_2022Update.mxd

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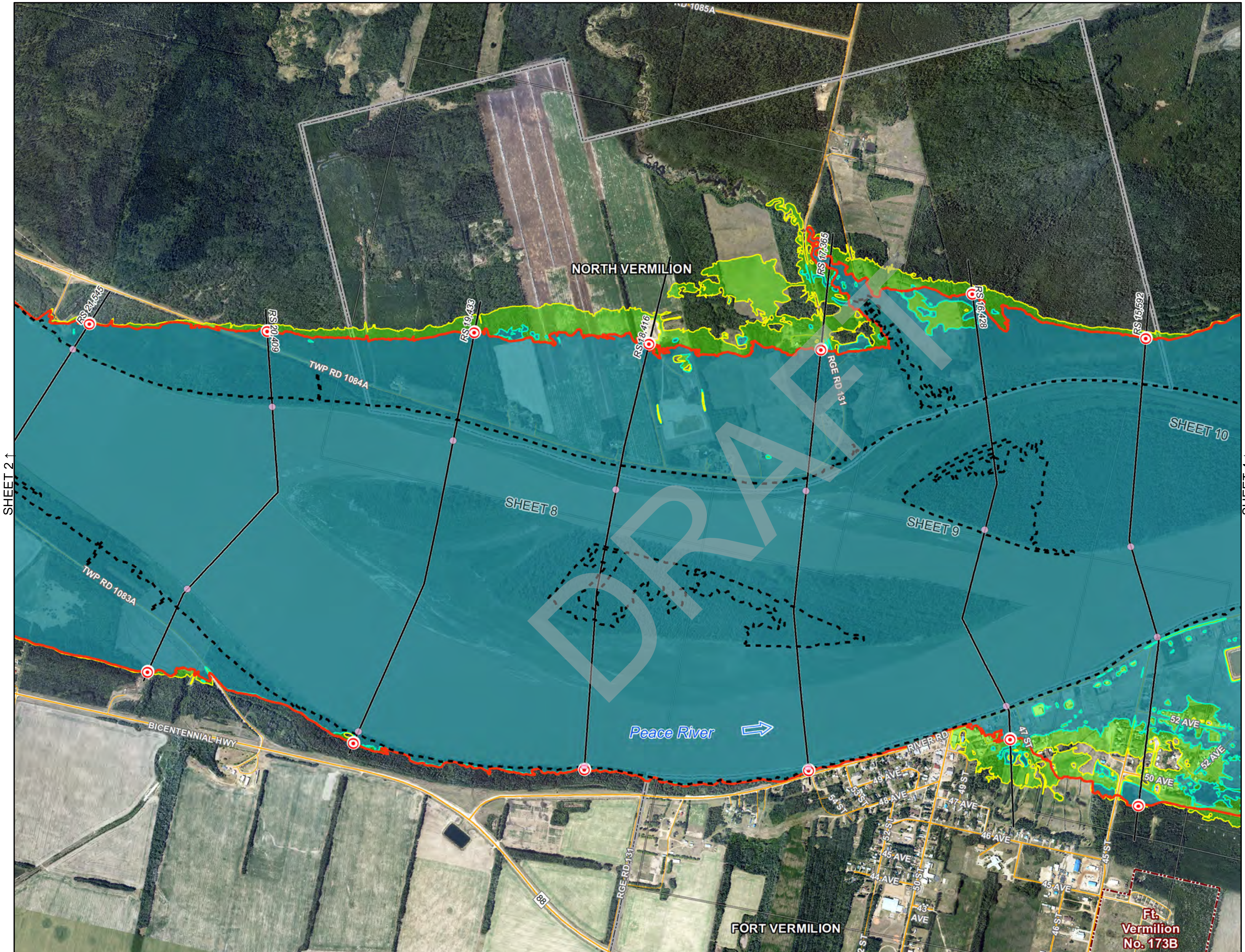

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FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500
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N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

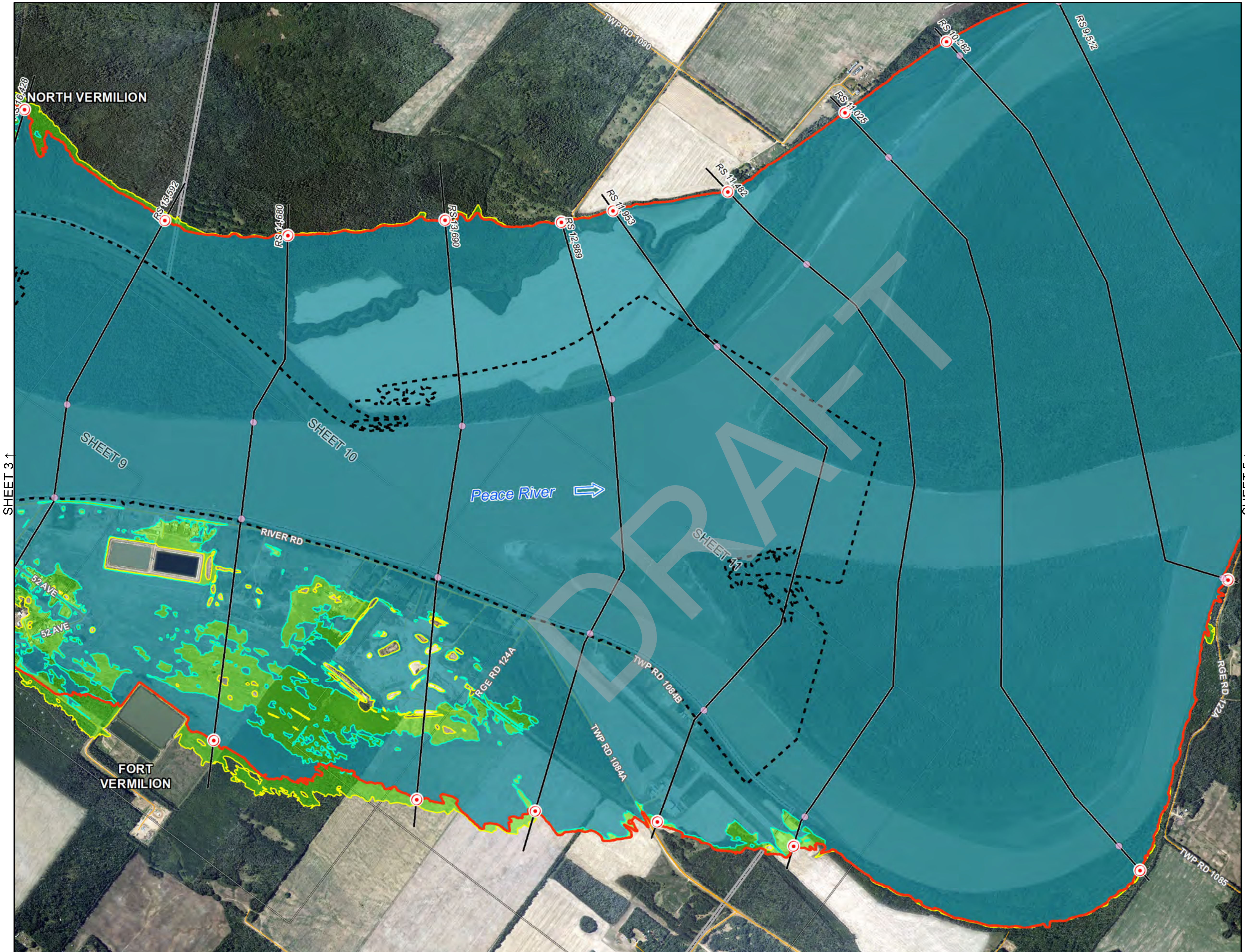
Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

SHEET 3 OF 11

Classification: Public



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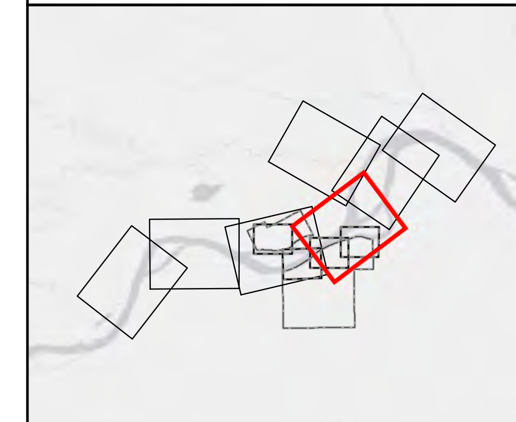

FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500
0 100 200 400 600 800 M
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

SHEET 4 OF 11

MMM P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_IJFloodway_2022Update.mxd

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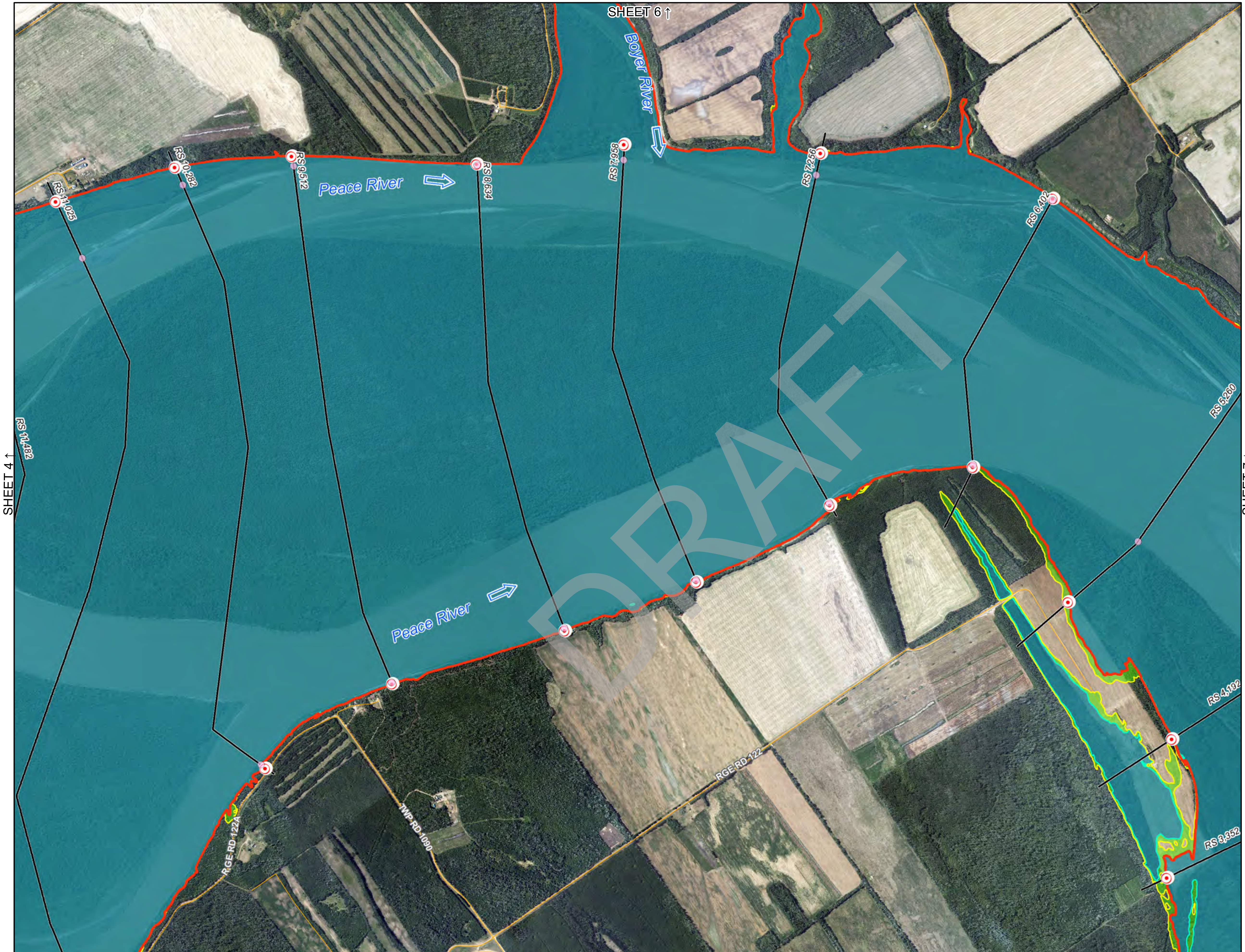

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FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500
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Coordinate System: NAD 1983 CSRS 3TM 117;
Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

SHEET 5 OF 11

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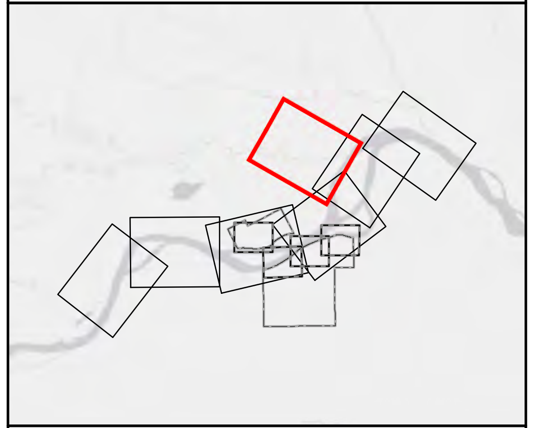


FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500

0 100 200 400 600 800 M

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	*Date: 04-NOV-2022*	

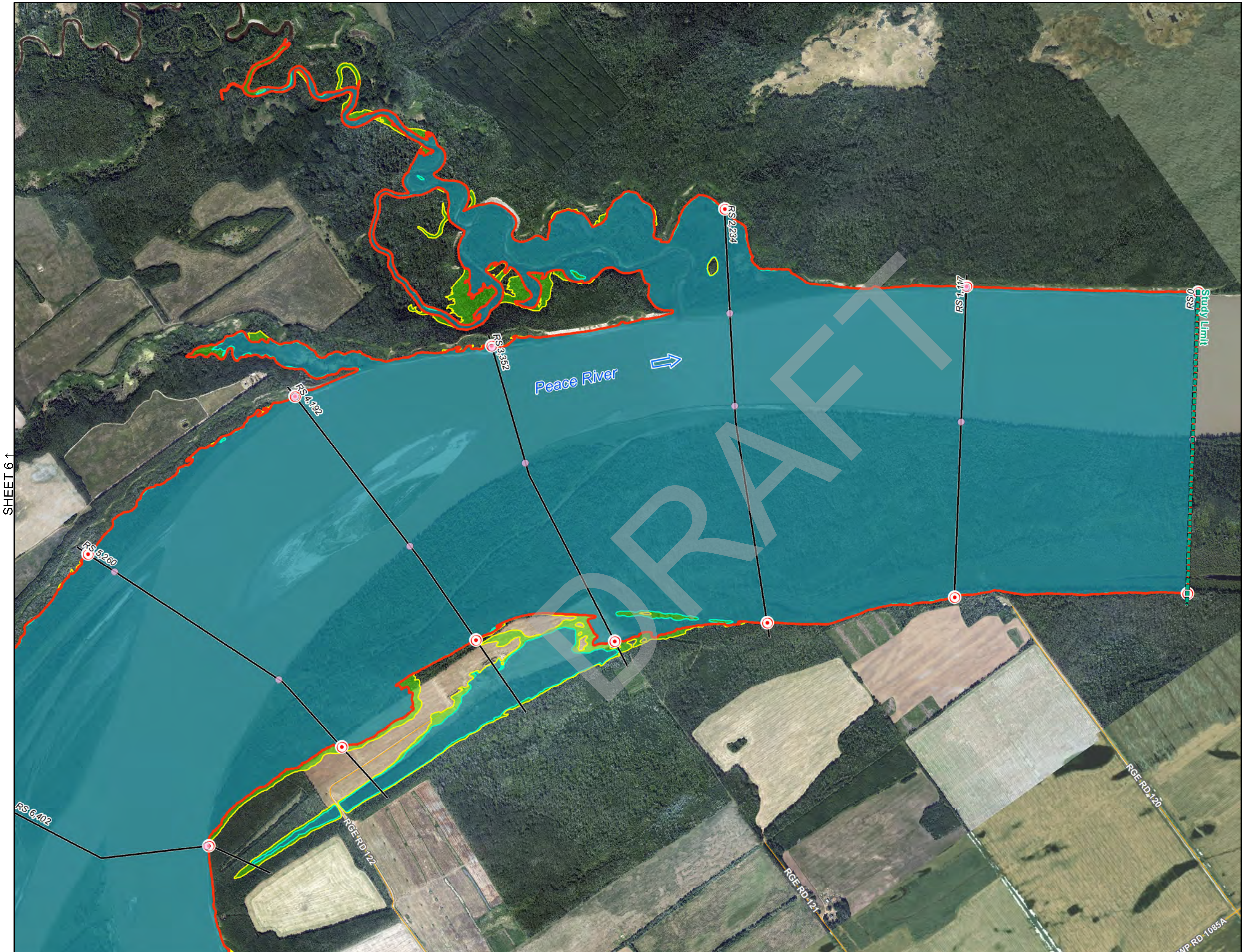
FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

SHEET 6 OF 11

M:\M:\P_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilion\FHS_Map_IJFloodway_2022Update.mxd

SHEET 6 ↑



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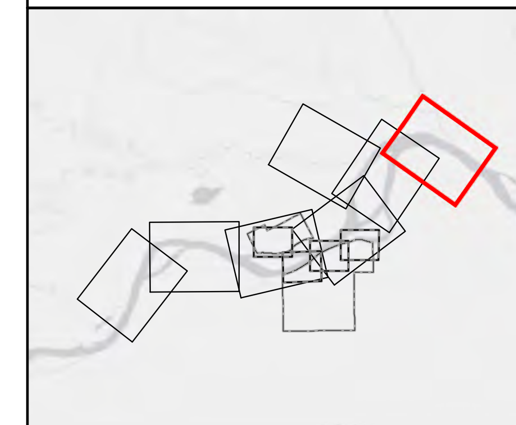

FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:17,500

0 100 200 400 600 800 M

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	*GIS* MMM/REH	*Reviewer* DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

SHEET 7 OF 11

MMM P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_IJFloodway_2022Update.mxd

Classification: Public

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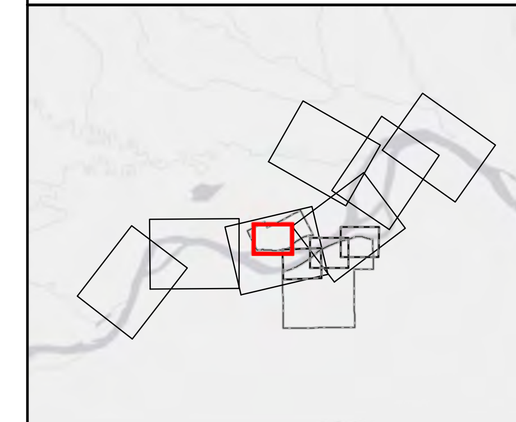


FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:7,500
0 100 200 M
N

Coordinate System: NAD 1983 CSRS 3TM 117;
Vertical Datum: CGVD28 HTv2.0; Units: Metres

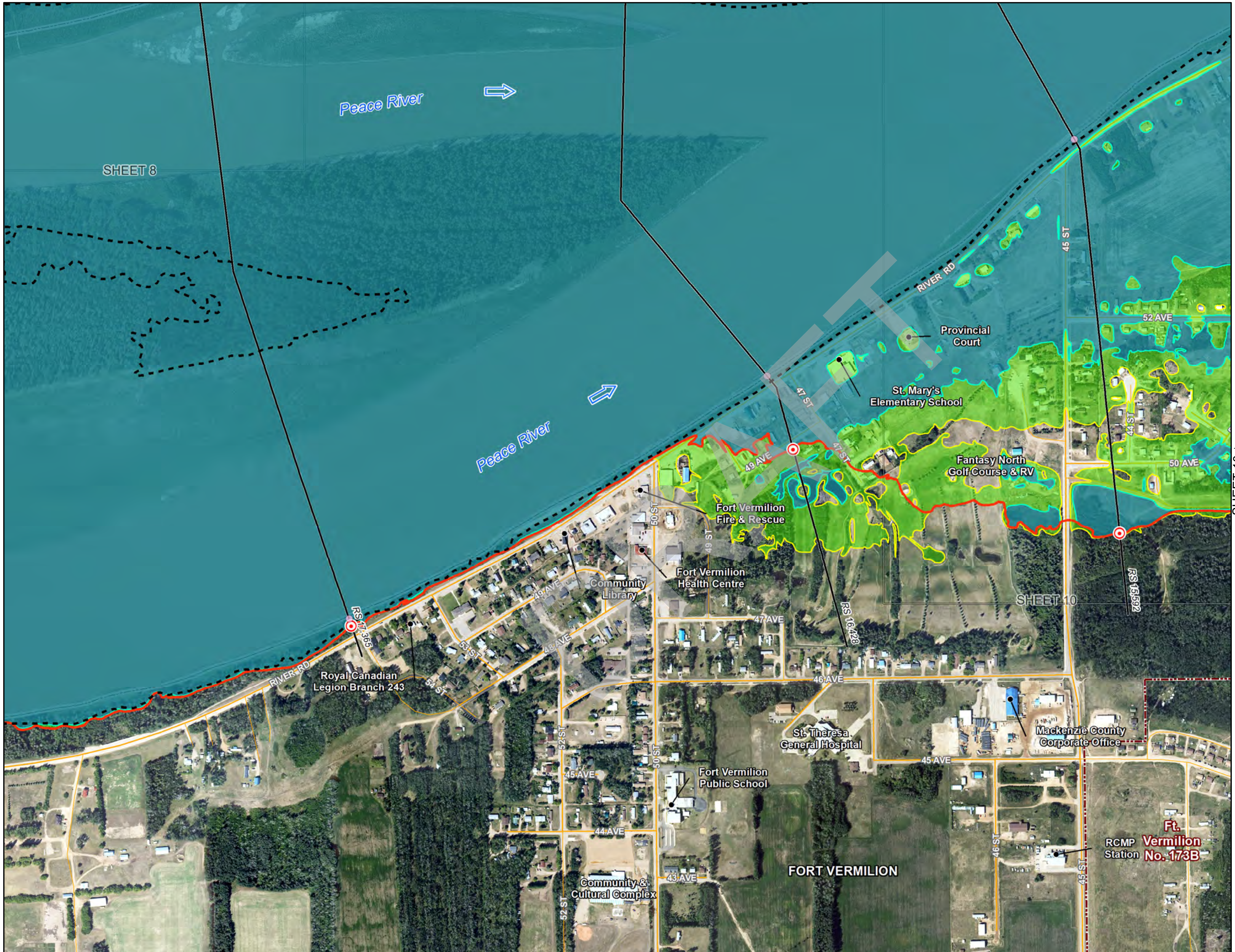
Engineer	GIS	Reviewer
MGB	MMM/REH	DJH

Job: 1006302 Date: 04-NOV-2022

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

SHEET 8 OF 11



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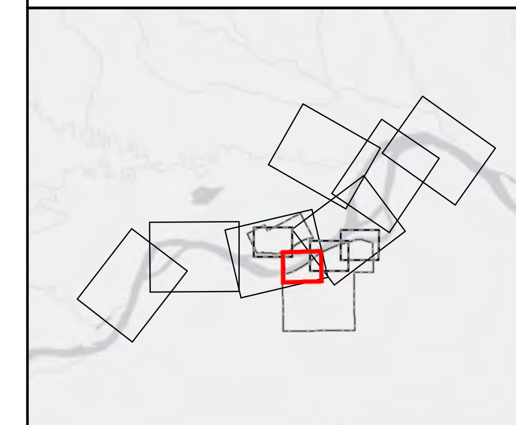

FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:7,500
0 100 200 M
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM FLOODWAY CRITERIA MAP

SHEET 9 OF 11

M:\P_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_IJFloodway_2022Update.mxd

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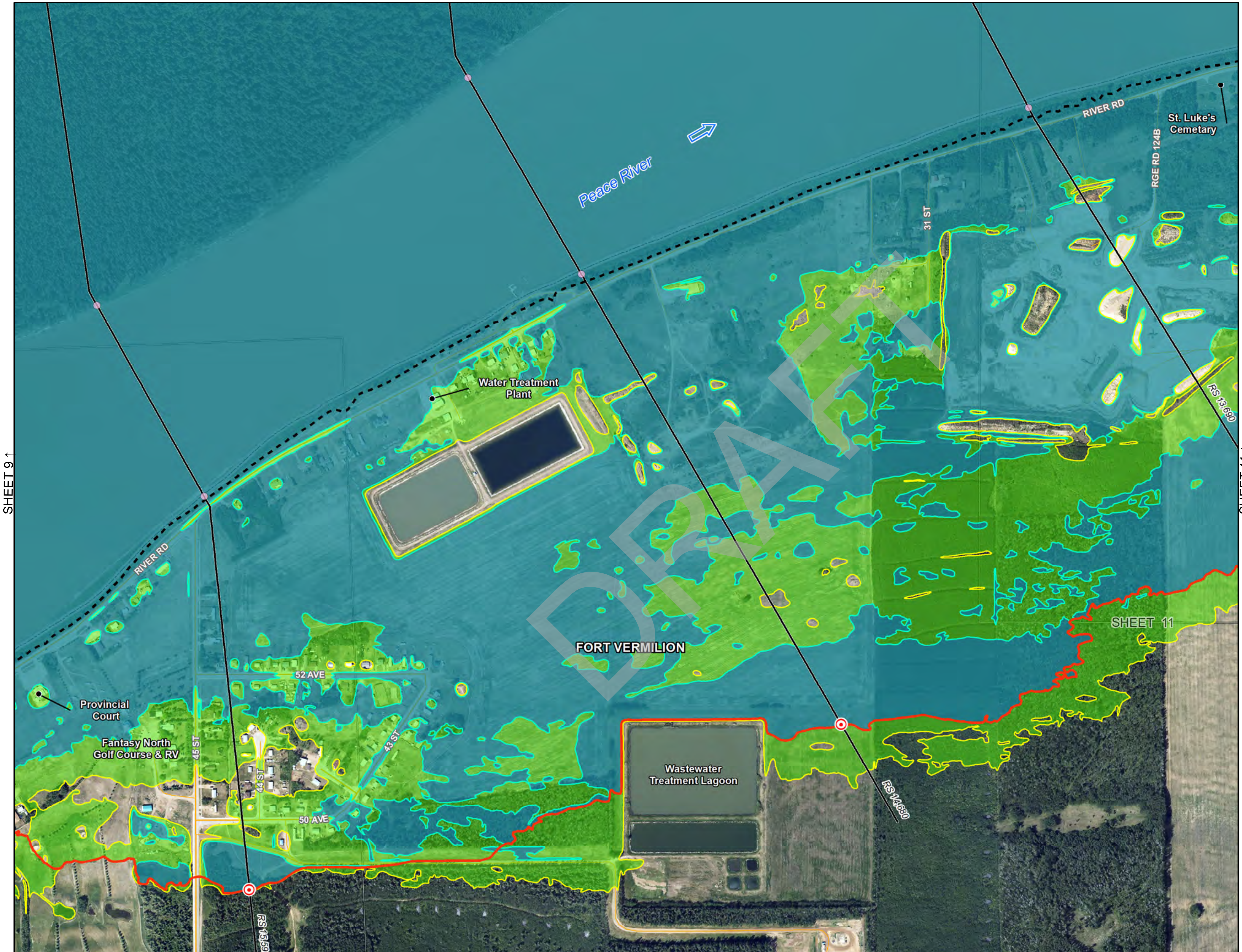

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FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
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BANK STATION
100-YEAR DESIGN FLOOD EXTENT
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PREVIOUS FLOODWAY

SCALE - 1:7,500
0 100 200 M
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH

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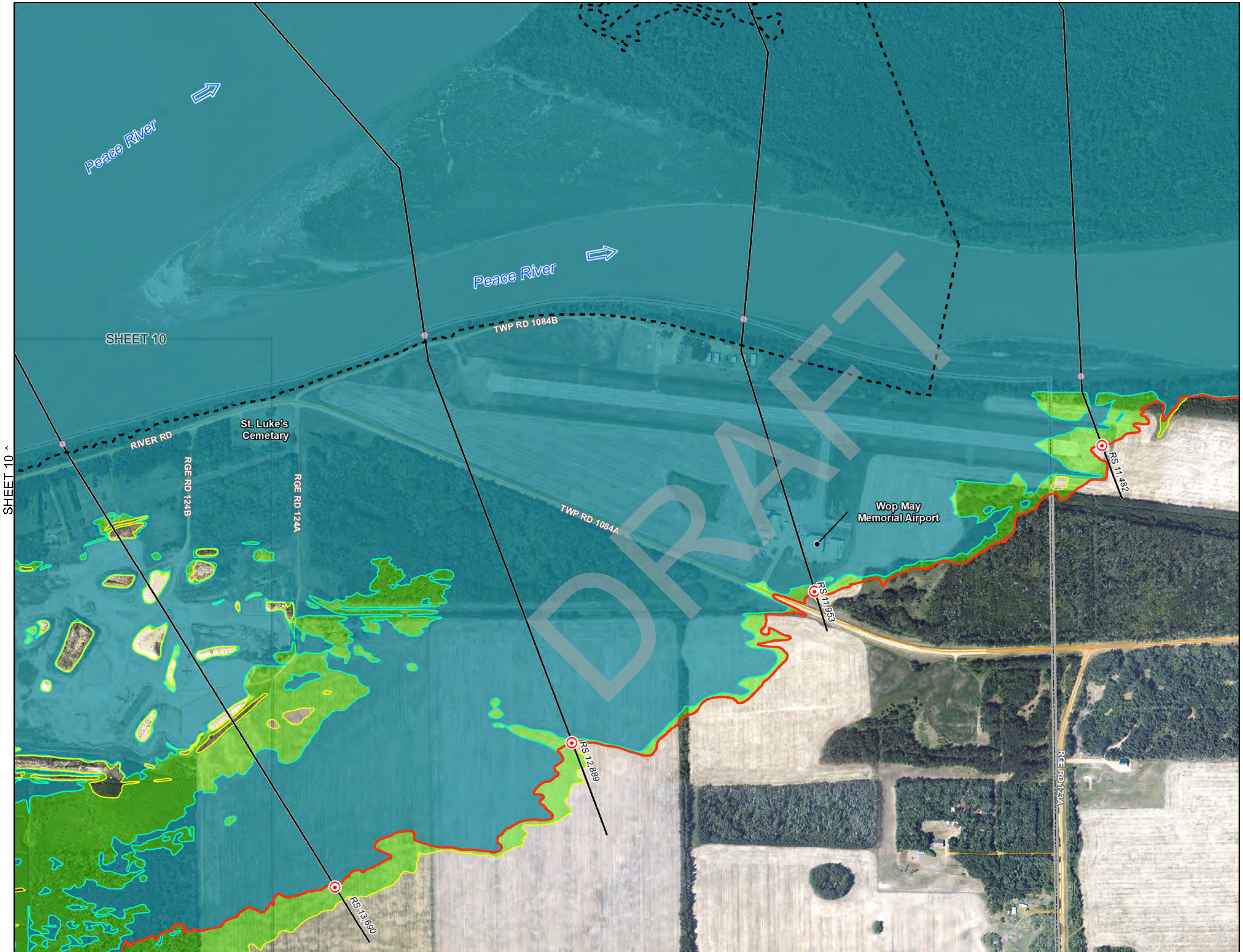
FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

SHEET 10 OF 11

Classification: Public



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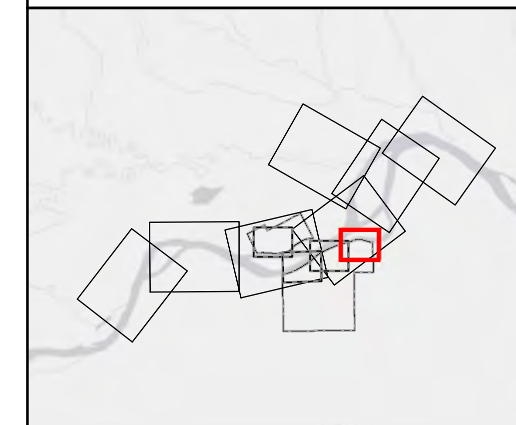

FLOW DIRECTION
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CULVERT
STUDY LIMIT
CROSS SECTION
LOCAL ROAD
PROVINCIAL ROAD
INDIAN RESERVE
MUNICIPAL BOUNDARY
ZOOMED-IN MAP SHEET
PROPOSED FLOODWAY BOUNDARY
PROPOSED FLOODWAY STATION
BANK STATION
100-YEAR DESIGN FLOOD EXTENT
DEPTH ≥ 1 m
PREVIOUS FLOODWAY

SCALE - 1:7,500
0 100 200 M
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS MMM/REH	Reviewer DJH

Job: 1006302 | Date: 04-NOV-2022

FORT VERMILION FLOOD HAZARD STUDY

ICE JAM
FLOODWAY CRITERIA MAP

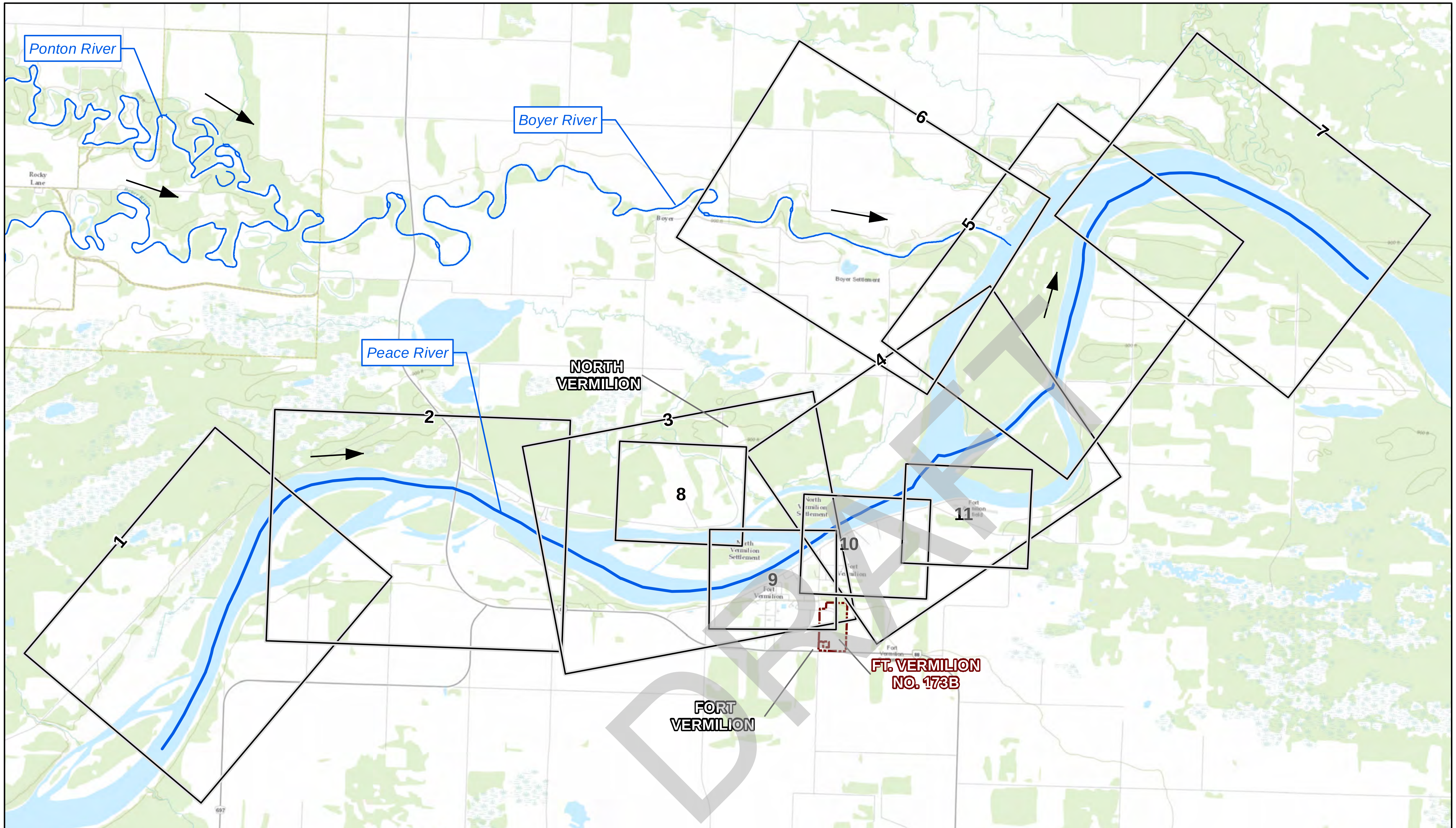
SHEET 11 OF 11

MMM:P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_IJFloodway_2022Update.mxd

Classification: Public

Appendix K

Design Flood Hazard Maps



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FLOW DIRECTION
MAP SHEET
INDIAN RESERVE
STUDY REACH
MAJOR TRIBUTARY

SCALE - 1:85,000
0 1 2 4 KM
N

Coordinate System: NAD 1983 CSRS 3TM 114;
Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer	GIS	Reviewer
MGB	REH/MMM	DJH

Job: 1006302 | Date: 04-NOV-2022

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

INDEX MAP

Notes to Users:

1. Please refer to the accompanying **Fort Vermilion Flood Hazard Study** for important information concerning these maps.
2. Within the flood inundation areas shown on this map, there may be isolated pockets of high ground. To determine whether or not a particular site is subject to flooding, reference should be made to the computed flood levels in conjunction with site-specific surveys where detailed definition is required.
3. Non-riverine and local sources of water have not been considered, and structures such roads and railways can restrict water flow and affect local flood levels. Channel obstruction, local stormwater inflow, groundwater seepage or other land drainage can cause flood levels to exceed those indicated on the map. Lands adjacent to a flooded area may be subject to flooding from tributary streams not indicated on the maps.
4. The flood inundation area is shown above the linework for bridges and flood control structures that are below flood levels.

Definitions:

Flood Hazard Map - A flood hazard map is a specific type of flood map that identifies the area flooded for the 1:100 design flood, and divides that flood hazard area into floodway and flood fringe zones. Flood hazard maps can also show additional flood hazard information, including the incremental areas at risk for more severe floods like the 1:200 and 1:500 floods. Flood hazard maps are typically used for long-term flood hazard area management and land-use planning.
Design Flood - The design flood standard in Alberta is the 1:100 flood, which is a flood that has a 1% chance of being equaled or exceeded in any given year. The design flood is typically based on the 1:100 open water flood, but it can also reflect 1:100 ice jam flood levels or be based on a historical flood event. Different sized floods have different chances of occurring – for example, a 1:200 flood has a 0.5% chance of occurring in any given year and a 1:500 flood has a 0.2% chance of occurring in any given year – but only the 1:100 design flood is used to define the floodway and flood fringe zones on flood hazard maps.
Floodway - When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard where flows are deepest, fastest, and most destructive during the 1:100 design flood. When a flood hazard map is updated, the floodway will not get larger in most circumstances to maintain long-term regulatory certainty, even if the flood hazard area gets larger or design flood levels get higher.
Flood Fringe - The flood fringe is the area outside of the floodway that is flooded or could be flooded during the 1:100 design flood. The flood fringe typically represents areas with

Definitions (continued):

shallower, slower, and less destructive flooding, but it may also include "high hazard flood fringe" areas. Areas at risk of flooding behind flood berms may also be mapped as "protected flood fringe" areas.
High Hazard Flood Fringe - The high hazard flood fringe identifies areas within the flood fringe with deeper or faster moving water than the rest of the flood fringe. High hazard flood fringe areas are likely to be most significant for flood maps that are being updated, but they may also be included in new flood maps.
Protected Flood Fringe - The protected flood fringe identifies areas that could be flooded if dedicated flood berms fail or do not work as designed during the 1:100 design flood, even if they are not overtopped. Protected flood fringe areas are part of the flood fringe and do not differentiate between areas with deeper or faster moving water and shallower or slower moving water.

Data Sources and References:

1. Orthophoto imagery acquired by OGL Engineering for Alberta Environment and Parks: *Imagery acquired on 16 June 2019.*
2. Base data from Natural Resources Canada, Alberta Environment and Parks, and Altalis.
3. Additional base mapping from Esri.

MMM_P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Index_GVDFloodHazard_2022Update.mxd

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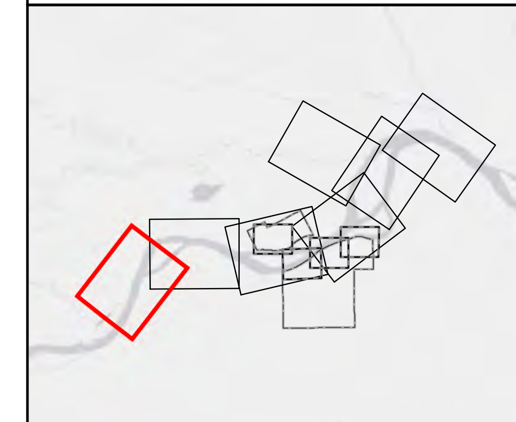


- FLOW DIRECTION
- BRIDGE
- CULVERT
- STUDY LIMIT
- CROSS SECTION
- FLOODWAY
- FLOOD FRINGE
- HIGH HAZARD FLOOD FRINGE
- 200-YEAR ICE JAM EXTENT
- LOCAL ROAD
- PROVINCIAL ROAD
- ZOOMED-IN MAP SHEET
- INDIAN RESERVE
- MUNICIPAL BOUNDARY

The design event is the 100-year ice jam flood.

SCALE - 1:17,500

0 100 200 400 600 800 M

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

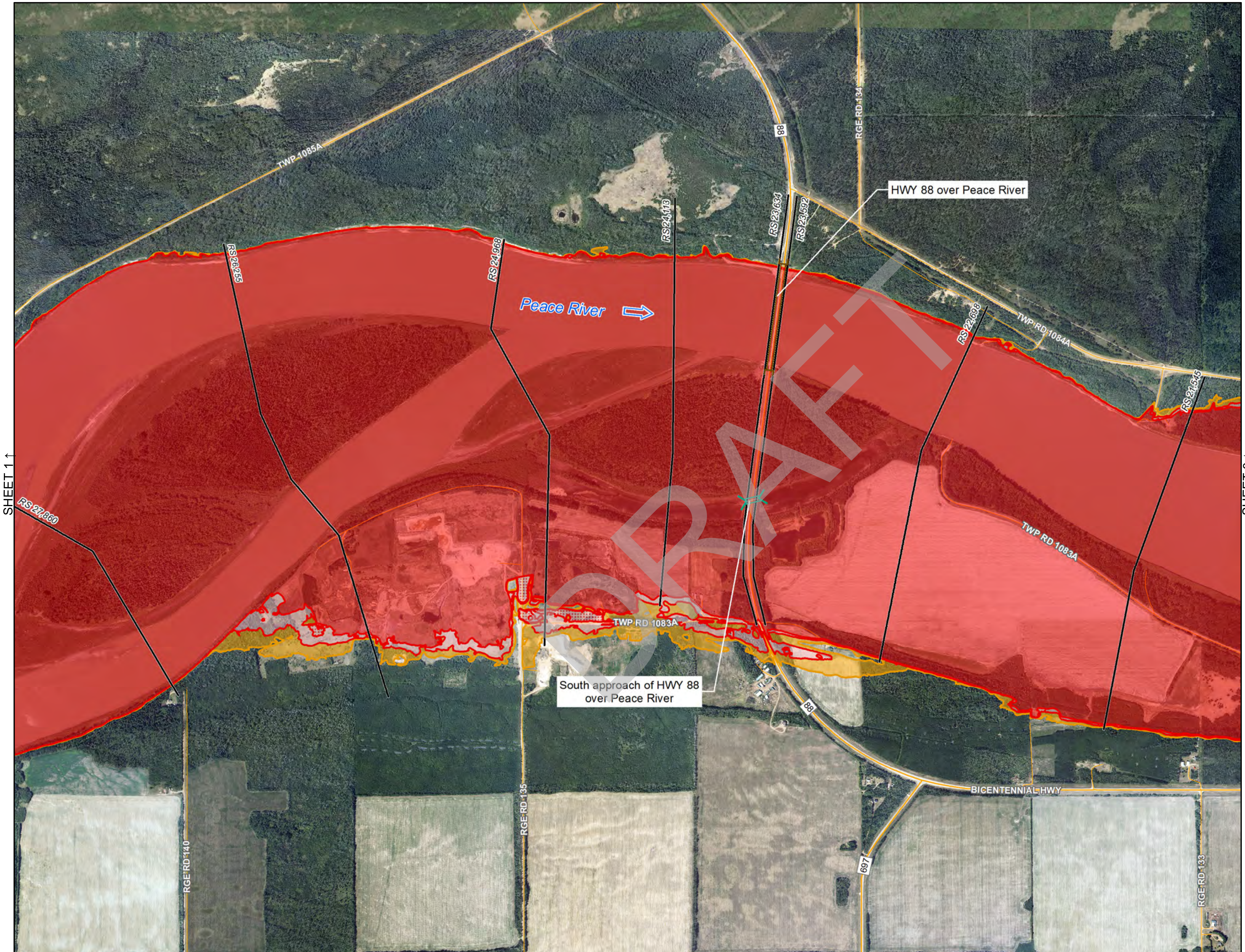
Engineer MGB	GIS REH/MMM	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 1 OF 11

MMM, P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd



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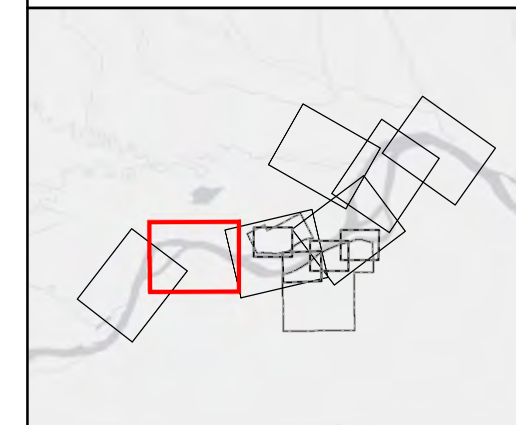

FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
FLOODWAY
FLOOD FRINGE
HIGH HAZARD FLOOD FRINGE
200-YEAR ICE JAM EXTENT
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The design event is the 100-year ice jam flood.

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Engineer MGB	*GIS* REH/MMM	*Reviewer* DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 2 OF 11

MMM_P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd

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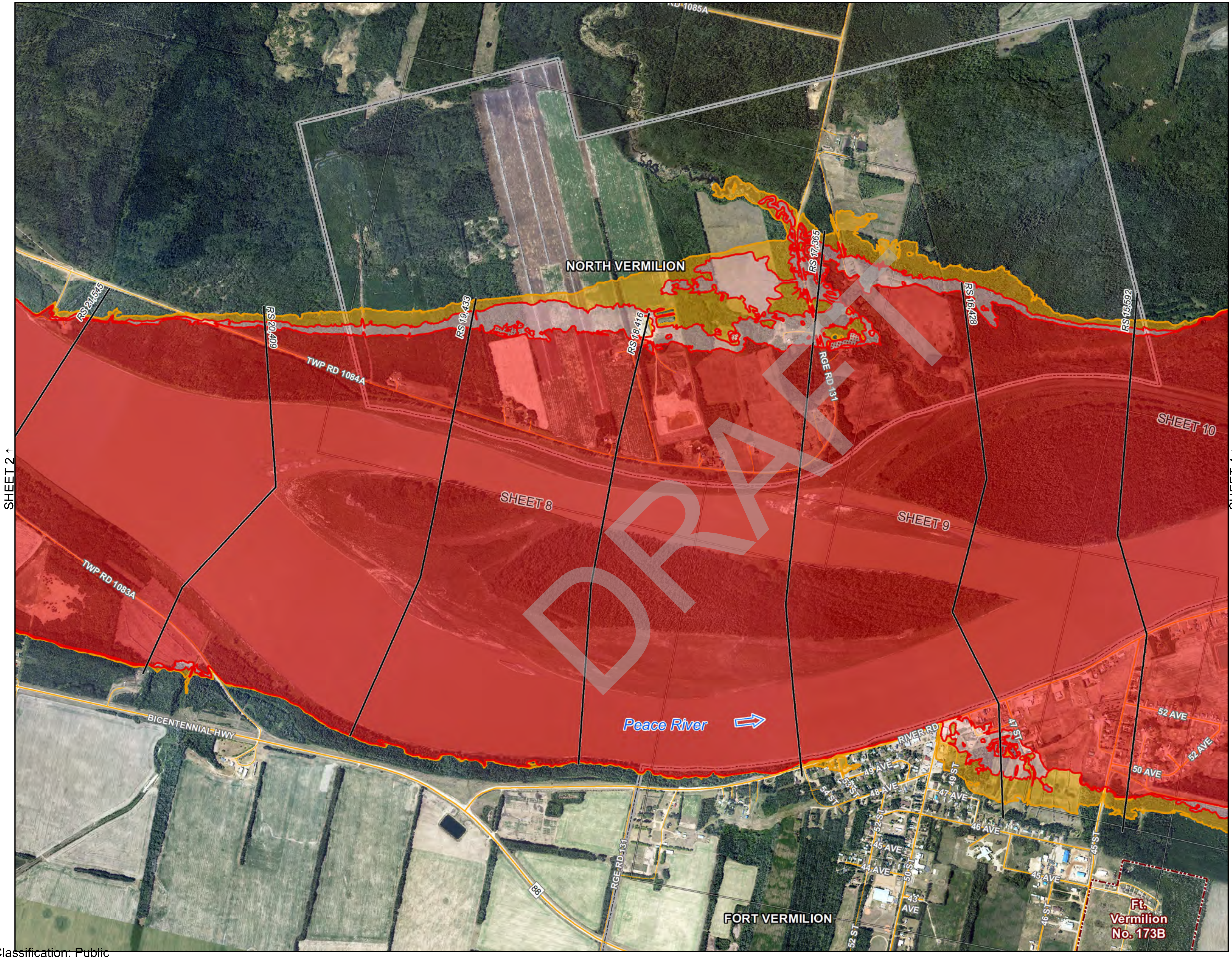

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- FLOW DIRECTION
- BRIDGE
- CULVERT
- STUDY LIMIT
- CROSS SECTION
- FLOODWAY
- FLOOD FRINGE
- HIGH HAZARD FLOOD FRINGE
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- LOCAL ROAD
- PROVINCIAL ROAD
- ZOOMED-IN MAP SHEET
- INDIAN RESERVE
- MUNICIPAL BOUNDARY

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SCALE - 1:17,500
0 100 200 400 600 800 M
N

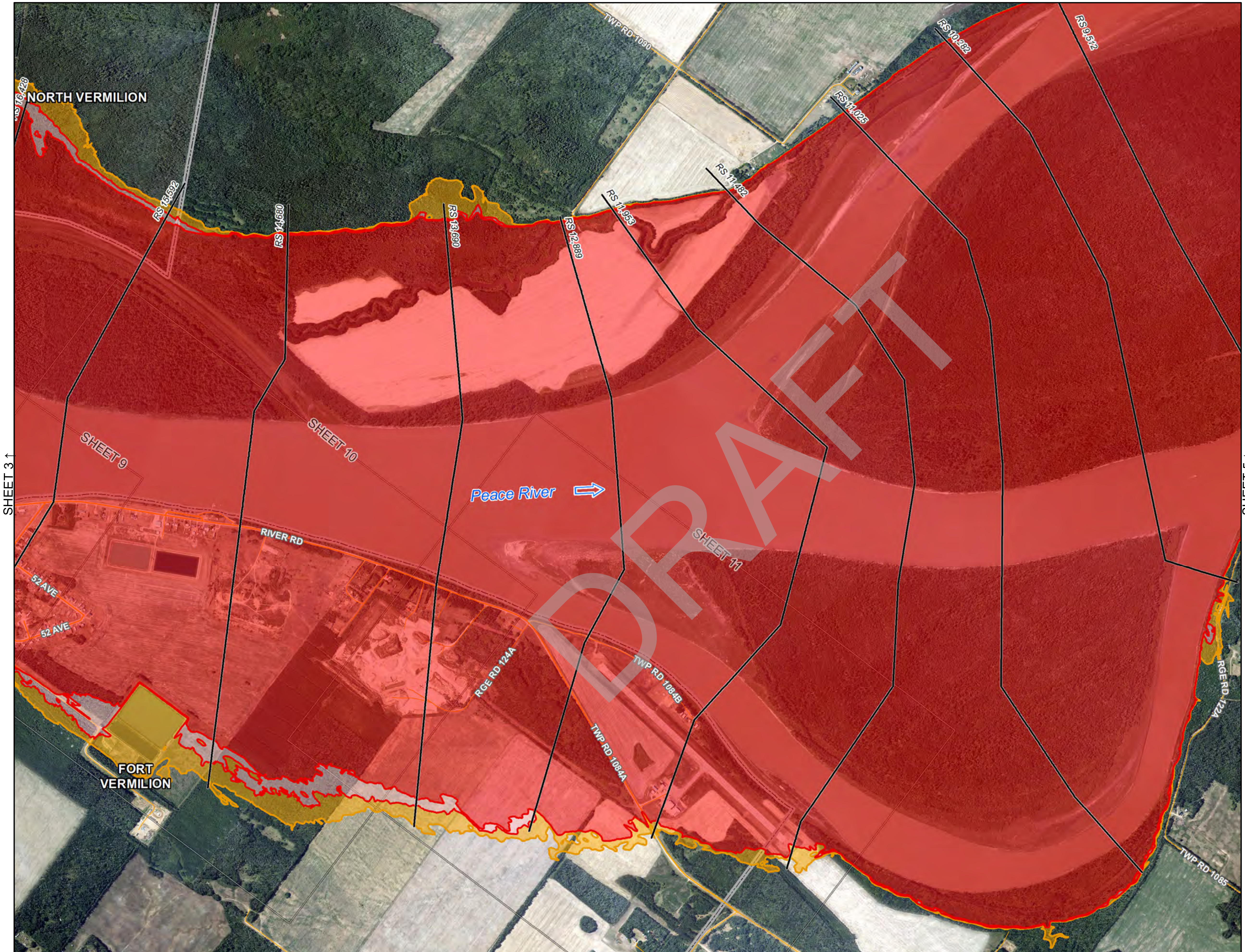
Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer	GIS	Reviewer
MGB	REH/MMM	DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 3 →



SHEET 5 →

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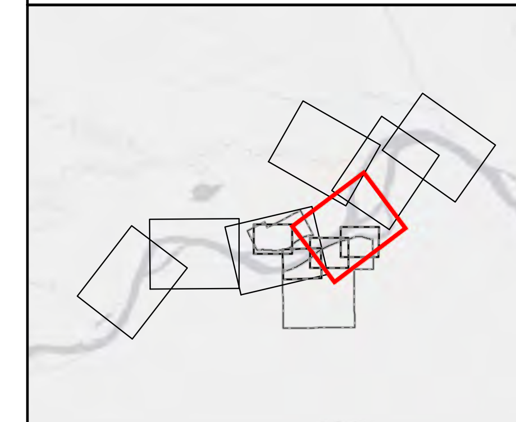

- FLOW DIRECTION
- BRIDGE
- CULVERT
- STUDY LIMIT
- CROSS SECTION
- FLOODWAY
- FLOOD FRINGE
- HIGH HAZARD FLOOD FRINGE
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- PROVINCIAL ROAD
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- INDIAN RESERVE
- MUNICIPAL BOUNDARY

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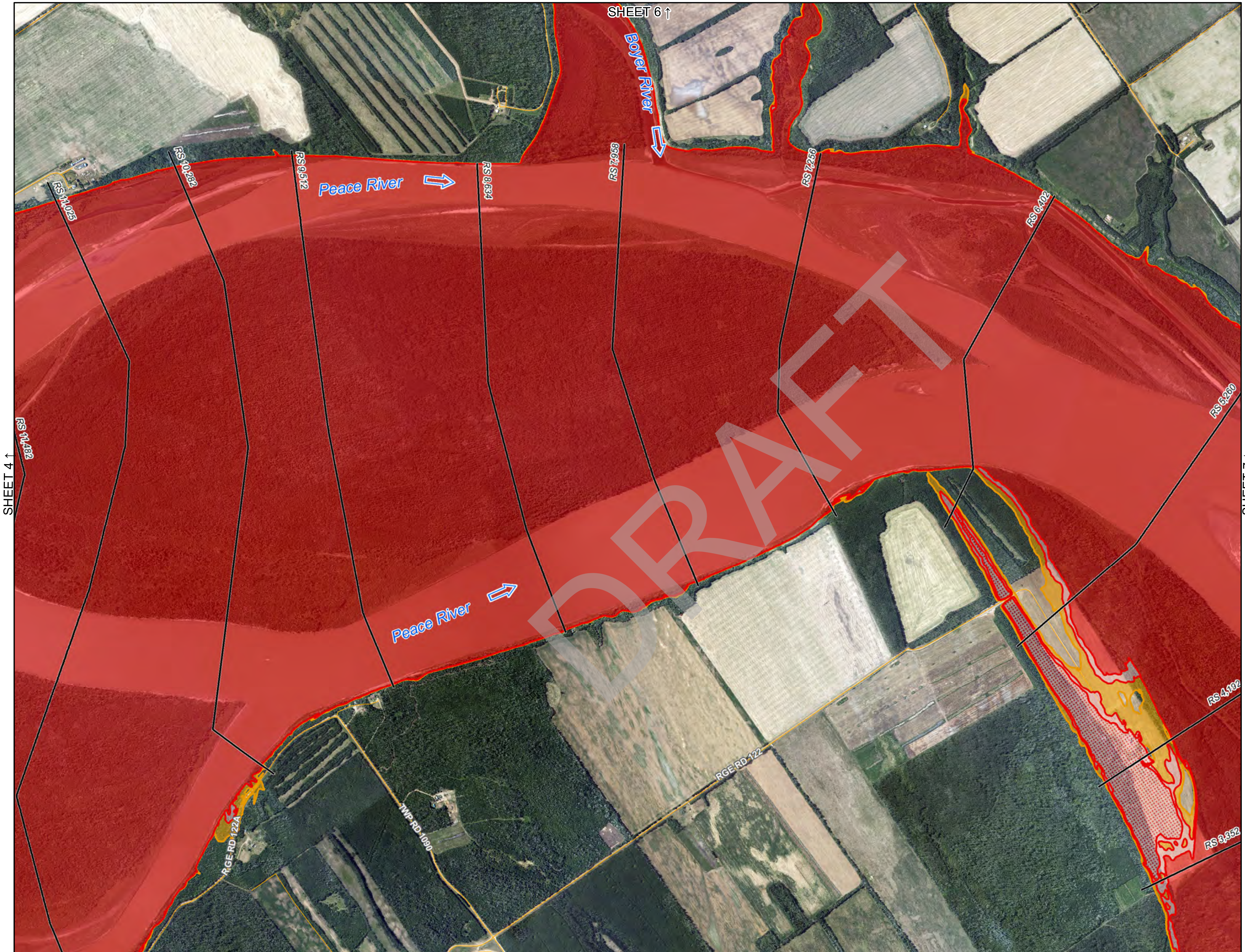
N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	GIS REH/MMM	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP



FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
FLOODWAY
FLOOD FRINGE
HIGH HAZARD FLOOD FRINGE
200-YEAR ICE JAM EXTENT
LOCAL ROAD
PROVINCIAL ROAD
ZOOMED-IN MAP SHEET
INDIAN RESERVE
MUNICIPAL BOUNDARY

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Coordinate System: NAD 1983 CSRS 3TM 117;
Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer	GIS	Reviewer
MGB	REH/MMM	DJH

Job: 1006302	*Date: 04-NOV-2022*

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

MMM_P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd

SHEET 5 →

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FLOW DIRECTION
BRIDGE
CULVERT
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CROSS SECTION
FLOODWAY
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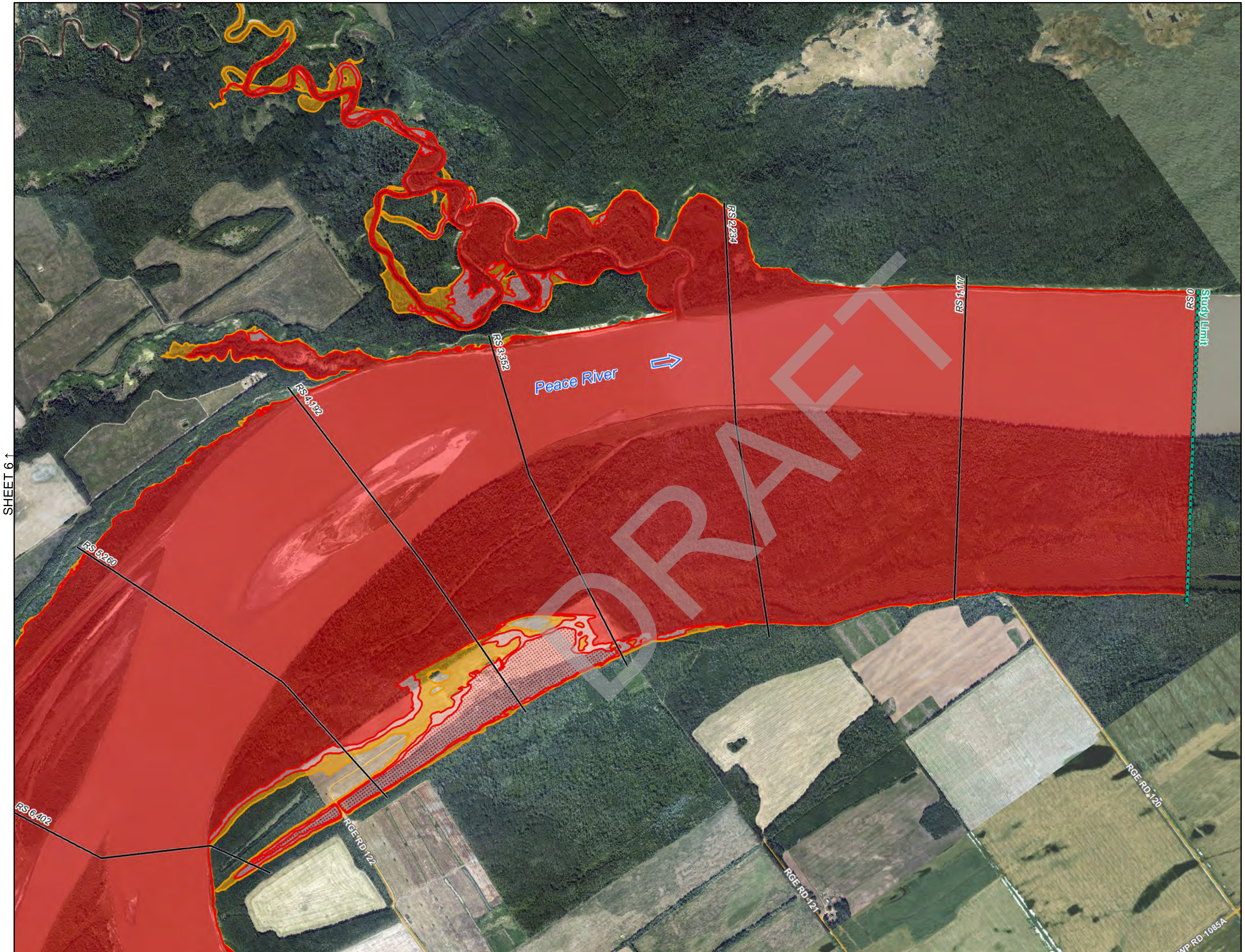
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 6 OF 11



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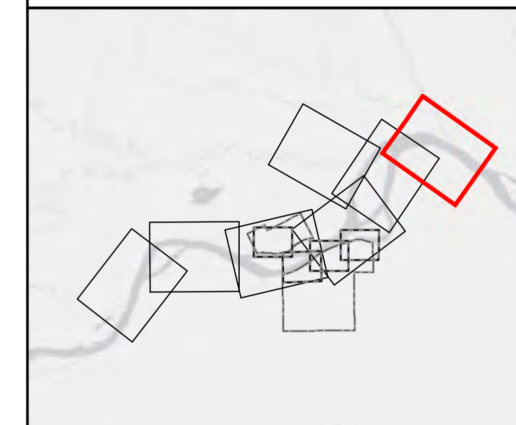


FLOW DIRECTION
BRIDGE
CULVERT
STUDY LIMIT
CROSS SECTION
FLOODWAY
FLOOD FRINGE
HIGH HAZARD FLOOD FRINGE
200-YEAR ICE JAM EXTENT
LOCAL ROAD
PROVINCIAL ROAD
ZOOMED-IN MAP SHEET
INDIAN RESERVE
MUNICIPAL BOUNDARY

The design event is the 100-year ice jam flood.

SCALE - 1:17,500

0 100 200 400 600 800 M

N

Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	*GIS* REH/MMM	*Reviewer* DJH
Job: 1006302	*Date: 04-NOV-2022*	

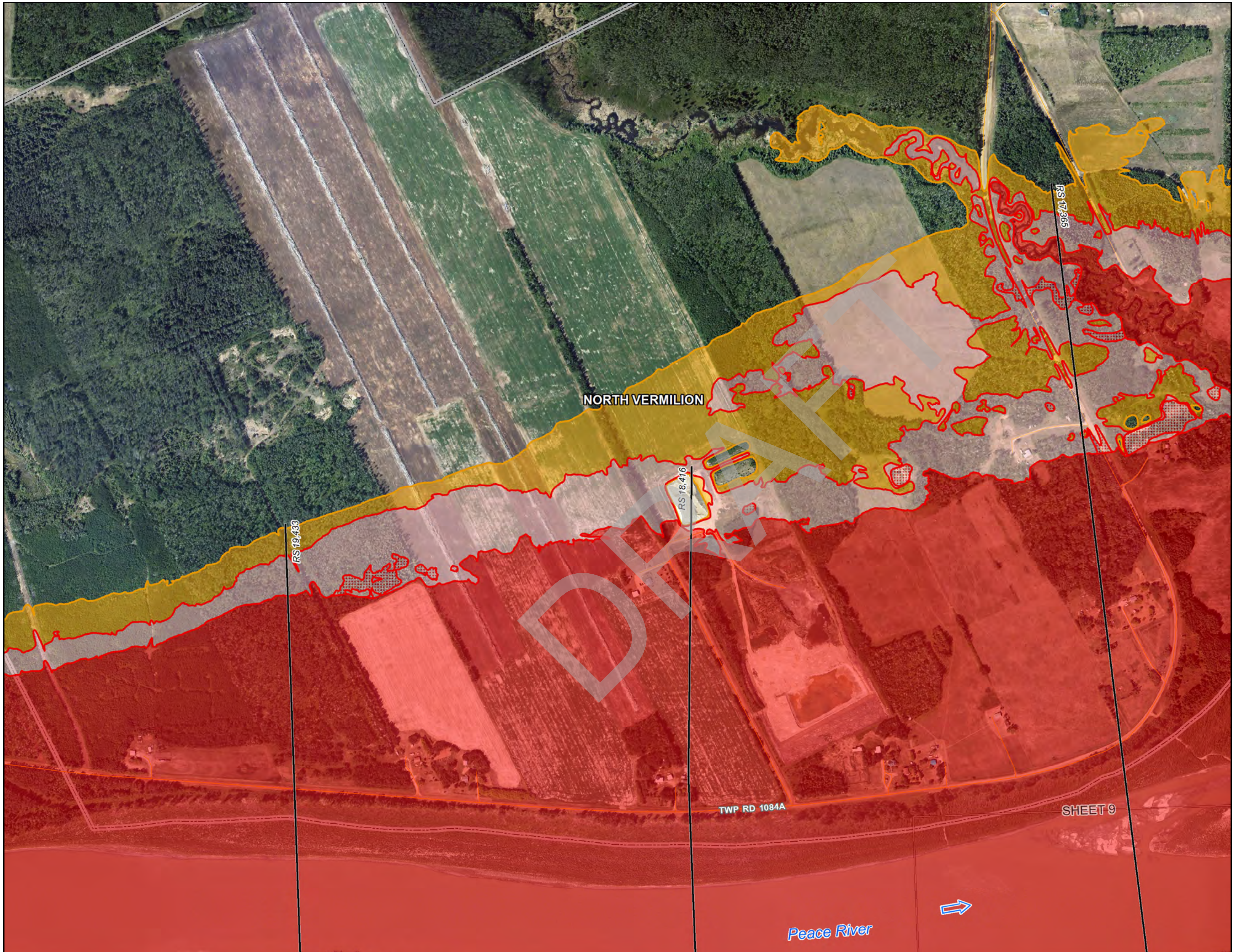
FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 7 OF 11

MMM, P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd

Classification: Public



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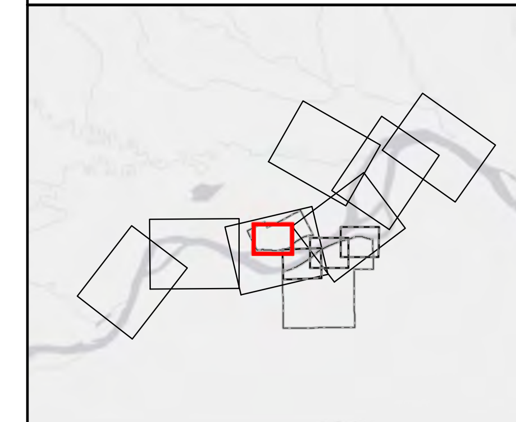

- FLOW DIRECTION
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- CULVERT
- STUDY LIMIT
- CROSS SECTION
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The design event is the 100-year ice jam flood.

SCALE - 1:7,500

0 100 200 M

N

Coordinate System: NAD 1983 CSRS 3TM 117;
Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MGB	*GIS* REH/MMM	*Reviewer* DJH

Job: 1006302	*Date: 04-NOV-2022*

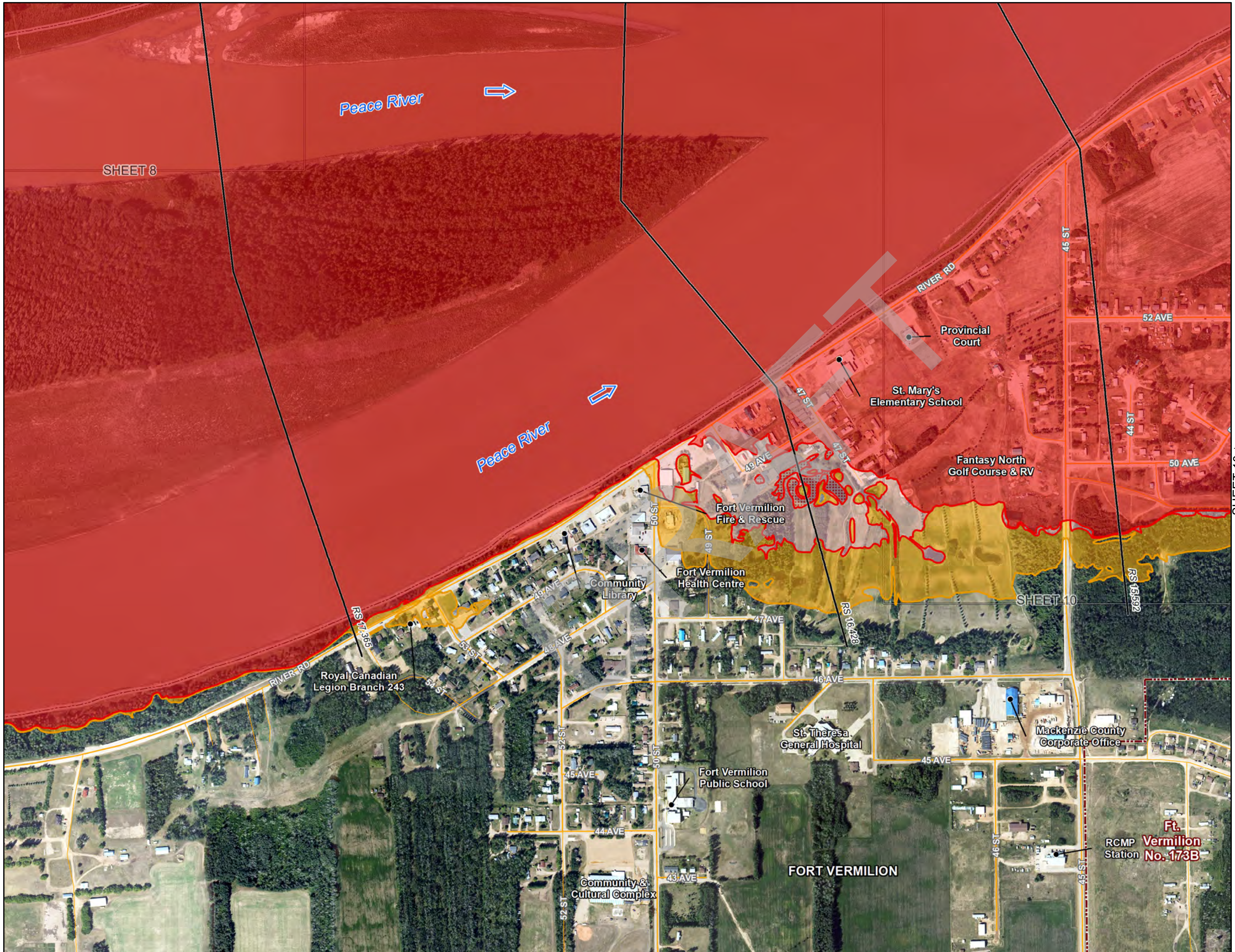
FORT VERMILION FLOOD HAZARD STUDY

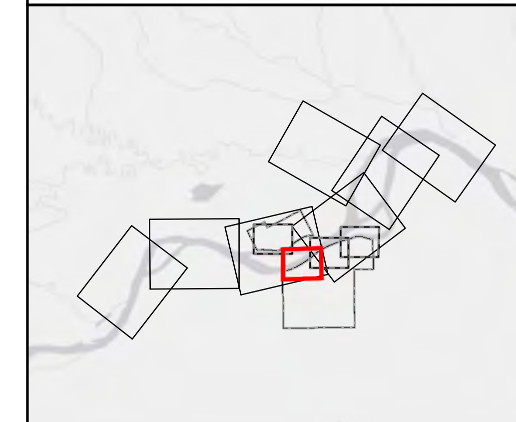
DESIGN FLOOD HAZARD MAP

SHEET 8 OF 11

MMM, P:_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd

Classification: Public





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Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

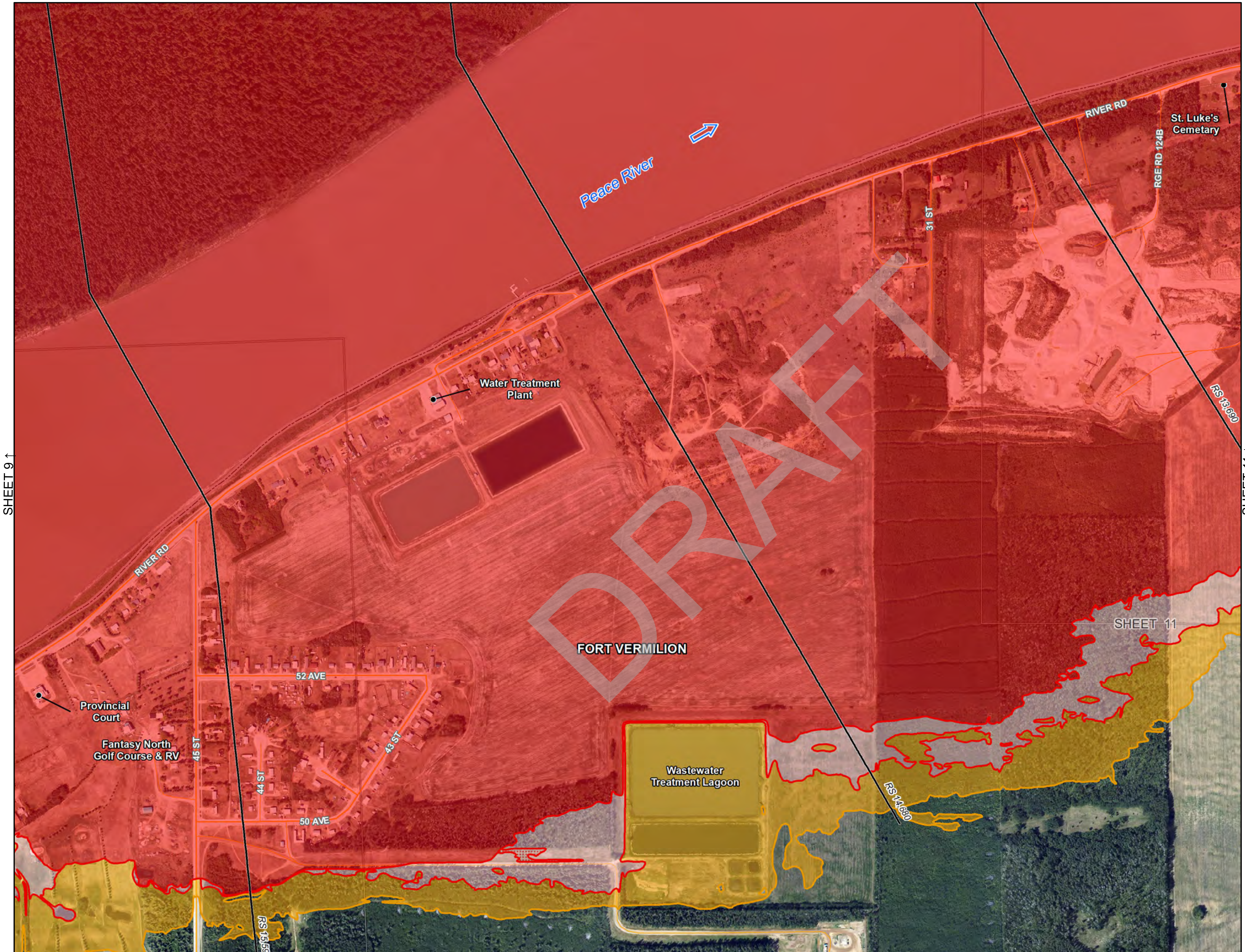
Engineer MGB	GIS REH/MMM	Reviewer DJH
Job: 1006302	Date: 04-NOV-2022	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 9 OF 11

Classification: Public



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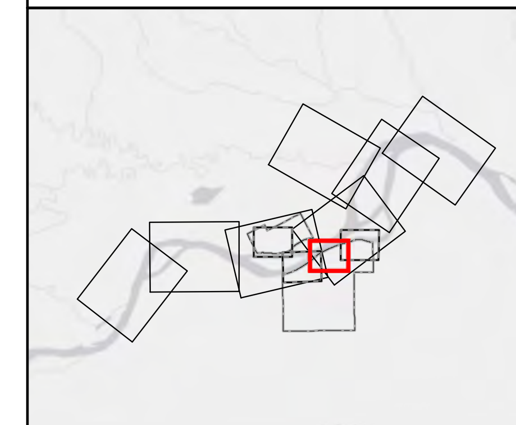

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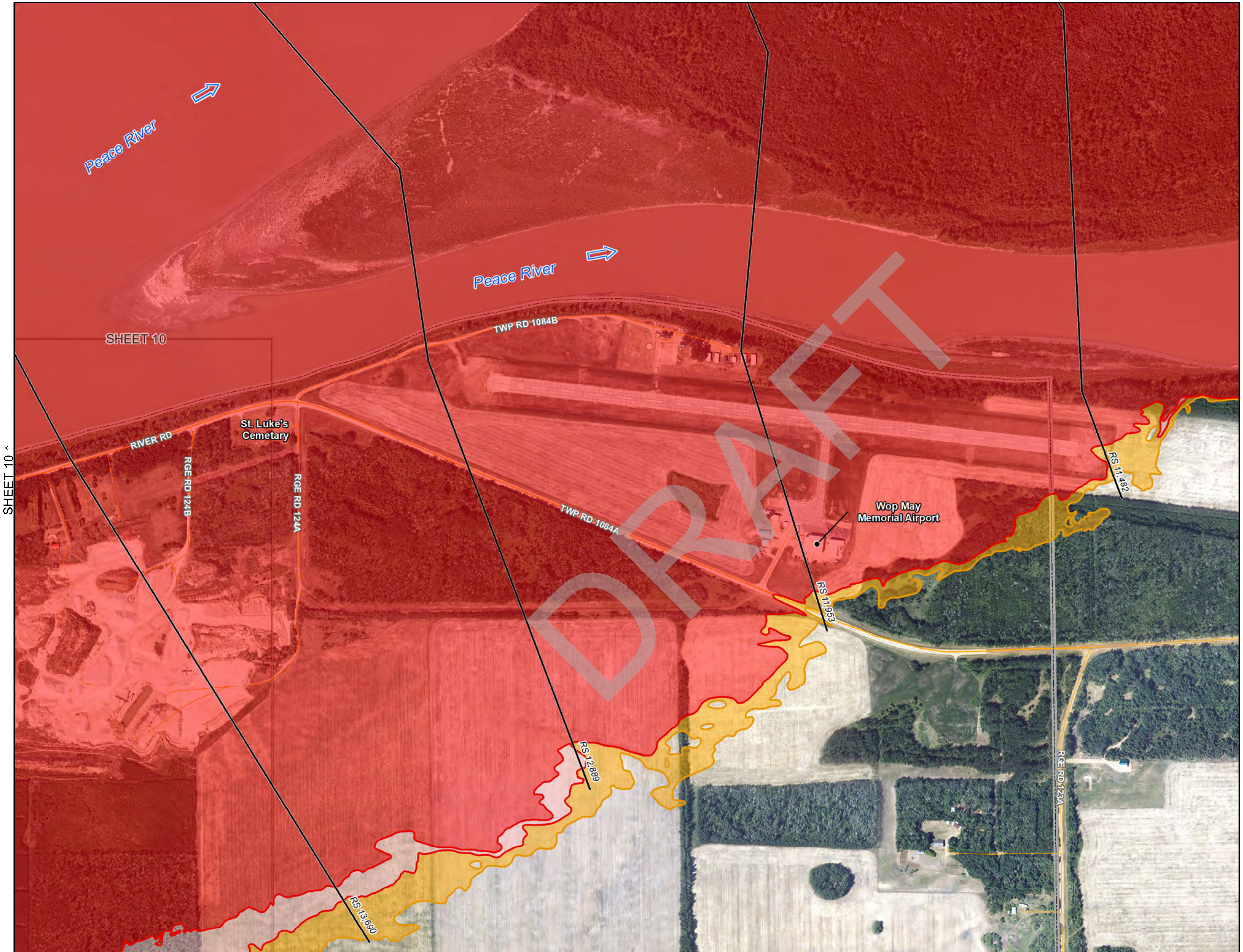
Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

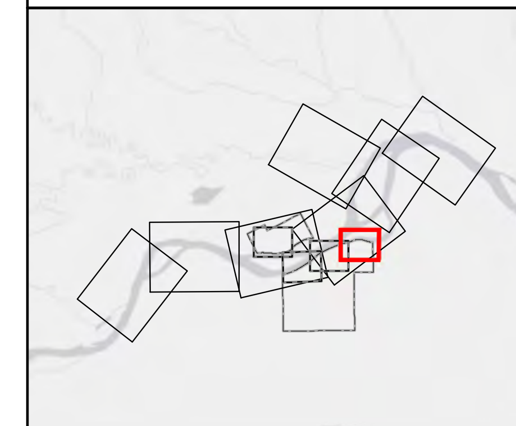
Engineer	GIS	Reviewer
MGB	REH/MMM	DJH

Job: 1006302 | *Date: 04-NOV-2022*

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP





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Engineer MGB	GIS REH/MMM	Reviewer DJH
Job: 1006302	*Date: 04-NOV-2022*	

FORT VERMILION FLOOD HAZARD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 11 OF 11

M:\M_P_Projects (Active)\1004659 Fort Vermilion Flood Hazard Study\2022 Municipal Review Updates\90 GIS\FortVermilionFHS_Map_GVDFloodHazard_2022Update.mxd

Classification: Public

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water resource specialists