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REPORT

Athabasca Flood Hazard Study

Study Summary Report – Volume 1: Technical Report

Submitted to:

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Distribution List

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Executive Summary

Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in March 2019 to conduct the Athabasca Flood Hazard Study (the study). The purpose of the study is to assess and identify river and flood hazards along the Athabasca River, Muskeg Creek and the Tawatinaw River through the Town of Athabasca and adjacent areas. The study is part of the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Athabasca, Athabasca County, and the general public.

This report documents the methodology and results for all components of the study. The study includes the following components:

- Survey and base data collection;
- Open water hydrology assessment;
- Open water hydraulic modelling;
- Ice jam modelling;
- Open water and ice jam flood inundation mapping; and
- Floodway determination and governing design flood hazard mapping.

The total length of the Athabasca River study reach is approximately 8.3 km, the total length of the Muskeg Creek study reach is approximately 7.5 km, and the total length of the Tawatinaw River study reach is approximately 6.1 km. The survey was completed in the spring of 2019. The hydraulic features in this study are summarized in Table i. There is no flood control structure identified in the study area.

Table i: Summary of Survey Features

Feature	Athabasca River	Muskeg Creek	Tawatinaw River	Total
Cross Sections	24	37	32	93
Bridges / Culverts	1	4	2	7

A hydrology assessment was completed to provide the flood peak discharge estimates at key locations in the study area as inputs to the HEC-RAS model.

The HEC-RAS model setup for the study area was informed by the supplementary two-dimensional modelling results. The HEC-RAS model includes the Athabasca River, Muskeg Creek and Tawatinaw River reaches. The model was calibrated based on the following:

- The low flow conditions (i.e., water levels and discharges) measured during the 2019 spring survey;
- The high flow conditions (i.e., high water marks collected by AEP) associated with the 1980 and 1990 flood events on the Athabasca River; and
- The flow-stage rating curve for the Water Survey of Canada (WSC) gauging station – Athabasca River at Athabasca (07BE001).

The calibrated Athabasca River channel Manning's *n* value was 0.026 for flood flow conditions. In the absence of flood data or high water marks for model calibration, a channel Manning's *n* value of 0.050 was estimated for Muskeg Creek and the Tawatinaw River for flood flow conditions. The calibrated model was used to simulate the water surface profiles for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events in the study area.

The model sensitivity was evaluated using the 100-year open water flood simulation results. The results of the sensitivity analysis show that variation of the channel roughness values has a higher influence on the simulated water levels than variation of the floodplain roughness values along the Athabasca River, Muskeg Creek and Tawatinaw River.

An assessment of historical ice jam flooding was used to provide a basis for ice jam hydraulic modeling. The assessment considered major ice jam events that occurred in 1904, 1943, 1963 and 2018. The 1904 event was determined to fall between the 500- and 1,000-year events. These historical ice jam events, along with historical records from WSC station 07BE001, was used as a basis for calibrating the HEC-RAS hydraulic model. The calibrated model was used to simulate the water surface profiles for the 50-, 100- and 200-year ice jam flood events in the study area.

Flood inundation and hazard maps were prepared for the study reaches of the Athabasca River, Muskeg Creek and the Tawatinaw River using ArcGIS. The simulated flood water levels at the cross sections were used to create a continuous water surface. The edge of inundation was delineated by subtracting the LiDAR DTM from the water surface. Direct inundation areas were mapped where there is a direct connection between the main river/creek channels and inundated areas on the floodplains. This includes areas where inundation is caused by single or multiple topographic or structural overtopping points or backwater flooding. Because there is no flood control structure in the study area, no area of potential flooding behind such structure was identified.

Based on the simulation results, various park areas in Athabasca would be flooded starting at the 10-year flood and various commercial areas would be flooded starting at the 75-year flood. The full set of open water flood inundation maps was prepared in this study.

Flood hazard maps were developed for the design flood event. These maps show the floodway and flood fringe, including high hazard flood fringe areas. Based on the flood hazard maps, no residences are situated within the floodways along the Athabasca River, Muskeg Creek and the Tawatinaw River. The areas within the floodway include a walkway on the right bank of the Athabasca River between the Muskeg Creek confluence and the Tawatinaw River confluence, and a wetland area on the right side of the Tawatinaw River upstream of the Highway 55 bridge crossing.

Developed areas in the flood fringe of the Athabasca River include portions of parking lot of the Independent Grocery Store, the Athabasca Home Hardware Building Centre, and 51st Avenue, all on the right floodplain. The Muskeg Creek flood fringe includes small portions on both sides upstream of Highway 2 Culvert, including 57 Street and Muskeg Creek trails, and small portions on the right side downstream of Highway 2 Culvert. The Tawatinaw River flood fringe includes small portions on the left bank upstream of Highway 55 Bridge. The full sets of floodway criteria maps and flood hazard maps are provided in this report.

Acknowledgements

The Athabasca Flood Hazard Study was prepared by Golder Associates Ltd. (Golder) in collaboration with SG1 Water Consulting Ltd. (SG1 Water). Overall project management was provided by Dr. Nathan Schmidt and Dr. Wolf Ploeger, with direction by Dr. Dejiang Long. Hydrological tasks were led by Dr. Getu Biftu, with support from Dr. Mesgana Gizaw. Hydraulic modeling was led by Dr. Hua Zhang with support from Mr. Jie Chen, Dr. Tebikachew Tariku and Ms. Nancy Guo. GIS tasks were led by Mr. Peter Thiede. Field studies were completed by Mr. Carmen Orosz with support from Ms. Jenna Pearse and Mr. Sean Denis.

Mr. David Andres of SG1 Water conducted the ice-related flood hazard assessment, including the ice jam flood frequency analysis, developed the HEC-RAS model, and reviewed ice-related sections of the study report.

Mr. Darren Shepherd of SG1 Water reviewed the Ice-related Flood Hazard Assessment memorandum and served as the technical lead and author of the Survey and Base Data Collection component.

The authors express their special thanks to Mr. Abdullah Mamun, project manager for Alberta Environment and Parks, who provided overall study management, background data, and technical guidance, and to Ms. Nadia Kovachis Watson and Dr. Jennifer Nafziger, who provided technical review of the ice-related flood hazard assessment.

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1.0 INTRODUCTION

1.1 Study Background

Golder Associates Ltd. (Golder), in collaboration with SG1 Water Consulting Ltd. (SG1), was commissioned by Alberta Environment and Parks (AEP) in March 2019 to undertake the Athabasca Flood Hazard Study (the study). The primary purpose of the study is to assess and identify river and flood hazards in the vicinity of the Town of Athabasca along an 8.3 km long reach of the Athabasca River and along two tributaries (i.e., a 6.1 km long reach of the Tawatinaw River and a 7.5 km long reach of Muskeg Creek). The study area for these two tributaries extends from their respective confluences with the Athabasca River to locations upstream of the municipal boundary of the Town.

The study is conducted under the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, local authorities, and the public. Key municipal stakeholders are the Town of Athabasca and the County of Athabasca.

The previous provincial flood hazard study for Athabasca for open water flood conditions was completed in 1993 (EC 1993). This study will replace the previous study, expand the modelling and flood mapping coverage, and include both open water and ice jam flood scenarios.

The study is comprised of multiple components and deliverables. This report documents the methodology and results of all major study components listed below.

1. Survey and Base Data Collection
2. Open Water Hydrology Assessment
3. Open Water Hydraulic Modelling
4. Ice Jam Modelling
5. Open Water and Ice Jam Flood Inundation Mapping
6. Floodway Determination and Governing Design Flood Hazard Mapping

1.2 Study Objectives

The overall goal of the study is to enhance public safety and support the assessment and identification of flood hazards in the study area. The study results are intended to reduce potential future flood damages and associated disaster assistance costs, to mitigate flood impacts by informing land use planning decisions, and for emergency preparation.

This report summarizes the work of all six components. 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 flood hydrology assessment and ice jam flood frequency analysis

- Creation, calibration, and validation of a HEC-RAS hydraulic model for both the open water and ice jam hydraulic modelling
- Simulation of selected return-period floods and the creation of water surface profiles throughout the study reach and sensitivity analysis of the model inputs
- Production of open water and ice jam flood inundation maps
- Determination of floodway criteria and creation of design flood water surface profiles throughout the study reach
- Production of floodway criteria maps for both open water and ice jam flood and governing design flood hazard maps

1.3 Study Area and Reaches

Figure 1-1 provides an overview of the study area, including the following:

- an 8.3 km long reach of the Athabasca River, extending from a short distance upstream of the municipal boundary at SE 30-66-22-W4M to a short distance downstream of the Athabasca Golf and Country Club at SE-34-66-22-W4M;
- a 6.1 km long reach of the Tawatinaw River, from the north limit of NE-4-66-22-W4M to its confluence with the Athabasca River; and
- a 7.5 KM long reach of Muskeg Creek, from the north limit of NW-6-66-22-W4M to its confluence with the Athabasca River.

Surveying, hydraulic modelling, and flood mapping were conducted over the study area. The downstream boundary of the hydraulic model terminates on the Athabasca River at a distance of approximately 5 km downstream of the Secondary Highway 813 bridge crossing.

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NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.
2. THE STUDY AREA IS COMPRISED OF AN 8.3 km LONG REACH OF THE ATHABASCA RIVER, A 6.1 km LONG REACH OF THE TAWATINAW RIVER, AND A 7.5 km LONG REACH OF MUSKEG CREEK WITHIN ATHABASCA COUNTY AND THE TOWN OF ATHABASCA.
3. A HYDROMETRIC STATION, OWNED AND OPERATED BY THE WATER SURVEY OF CANADA (WSC), IS LOCATED ON THE ATHABASCA RIVER WITHIN THE WATER TREATMENT PLANT BUILDING (WSC 07BE001). REFER TO FIGURE B-2 FOR DETAILS.

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1 : 35,000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Location Map of Study Area

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-101	FIGURE NO: 1.1
DATE:	29-JUL-2020	

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-101.dwg - 101, PLOT DATE: 28-Jul-2020

2.0 SURVEY AND BASE DATA COLLECTION

2.1 General

Surveying of the Athabasca River, the Tawatinaw River, and Muskeg Creek within the study area was conducted by Golder during the periods May 21 to 24 and June 3 to 4, 2019. The survey extents along each of the three study reaches are shown in Figure 1-1. The field program included the following:

- Surveying of cross sections and hydraulic structures within the study area; and
- A one-time discharge measurement on each waterbody in conjunction with a corresponding water surface profile survey along a portion of the study reach.

In addition, selected Alberta Survey Control Monuments (ASCM) were surveyed upon the request of AEP in support of Light Detection and Ranging (LiDAR) remote sensing data collection (by others) to confirm that the LiDAR-based digital terrain model (DTM) meets FHIP accuracy standards and that there is consistency between the LiDAR and ground surveys.

A site reconnaissance was conducted by key personnel from AEP, Golder, and SG1 on May 16, 2019. This field visit involved the following:

- Reviewed and confirmed the preliminary survey plan;
- Confirmed the locations and quantities of channel cross sections and hydraulic structures to be surveyed;
- Identified any flood control structures; and
- Familiarized the project team with the study reaches.

2.2 Procedures and Methodology

2.2.1 Survey Equipment and Control

The survey equipment used in collecting the topographic, bathymetric, and structure data for this study included the following:

- Real-time Kinematic (RTK) Global Positioning System (GPS): A Trimble® R8 RTK base station and Trimble® R10 RTK rover units, the latter of which were paired to Trimble® TSC3 hand-held data collectors running Trimble Access® survey software, were used to survey ground features, water levels, and river/creek bed levels in areas where hydraulic conditions allowed the surveyors to wade the channel and walk the banks. The RTK system was also used to survey the following:
  - control points and benchmarks that were found or placed within the study area; and
  - portions of the bridge structures and the lone culvert.
- Acoustic Doppler Current Profiler (ADCP): A SonTek RiverSurveyor M9® was used in combination with a boat-mounted RTK unit to survey the bed of the Athabasca River in the areas where wading was not possible. The ADCP was also used to conduct discharge measurements at a selected cross section on the Athabasca River within the study area.
- Acoustic Doppler Velocimeter (ADV): A SonTek FlowTracker2® ADV in combination with a top-set wading rod was used to conduct discharge measurements at a selected cross section on the Tawatinaw River and on Muskeg Creek within the study area.

- Total Station: A Nikon Nivo 5M® reflectorless total station was used to survey areas of the bridge structures and culvert that could not be accessed with an RTK rover unit.

The proposed locations of all cross sections were identified in a digital georeferenced vector format, which the survey crew utilized on their data collectors to guide the survey. Uploading a georeferenced survey plan into the data collector aided the surveyor in maintaining precise spacing and alignment of cross sections along each study reach.

The survey crew set up an RTK base station daily and operated in the traditional RTK mode. All surveyed points were acquired by boating the channel (in the case of the Athabasca River), wading the channel, or walking on the banks. Each survey data point collected was attributed a specific code. A schematic of survey point codes and corresponding descriptions is shown on Figure 2-1, which includes a complete list of survey codes for the RTK and total station.

Data that were collected using typical ground-based and acoustic-based technologies were referenced to one of five ASCM benchmarks situated within or close to the study area (i.e., ASCM 804708, ASCM 533448, ASCM 837047, ASCM 855155, and ASCM 663328). ASCM 804708 was used for calibration of the collected survey data. Quality assurance conducted by Golder during the field program confirmed that these ASCMs compared well to one another, which contributed to maintaining survey precision. The calibration process involved having the field crew check the survey equipment readings against one of these benchmarks.

In addition to checking against an ASCM at the start and end of each survey day, the survey crew obtained a secondary check on data accuracy by having the static (temporary) RTK base station log data continuously over the course of the day.

All survey data was collected in the 3TM 114° W coordinate system and referenced to NAD83 (CSRS) horizontal and CGVD28 vertical datums. The RTK and total station survey data outputs provided an orthometric elevation with correct northing and easting coordinates. The survey data were acquired by pre-loading geoid files into the survey equipment. Ellipsoidal heights were transformed to CGVD28 orthometric heights using the HTv2.0 geoid model.

2.2.2 River Cross Sections and Longitudinal Profiles

The locations of representative cross sections were selected to capture the variations in the physical characteristics of the channel and floodplains that could affect flood levels along the study reaches. Considerations of changes to the channel width, cross section area, channel bed and bank materials, and the presence of any confluences or islands, flood control structures, bridges, and other channel irregularities contributed to the selection of the cross section locations.

The alignment of each cross section was established so that it would be orientated perpendicular to the direction of river/creek flow, as anticipated under high-flow conditions. A shapefile showing the alignment of each cross section was provided to the survey crew at the outset of the field work and uploaded to the data collectors to provide guidance on where along the study reach to acquire data.

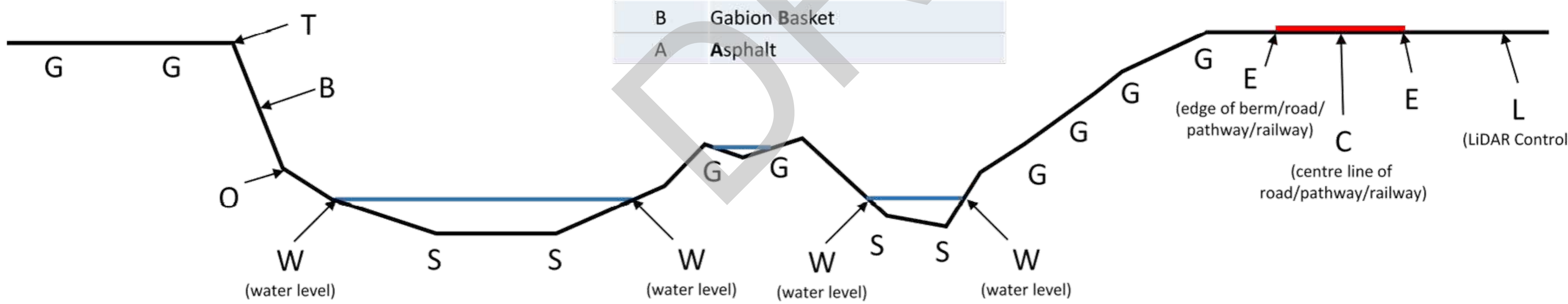
Survey Codes for River Surveys using RTK-GPS or Total Station (No Structures)

Purpose: - Create common definitions for survey points collected in the field for easier data processing in the office
- Reduce confusion or uncertainty for field staff regarding coding of points

Location Code	
G	**G**round
T	**T**op of Bank
B	**B**ank
O	T**o**e of Bank
W	**W**ater Level
S	**S**tream Bottom (under water)
E	**E**dge of Road/Berm/Pathway/Railway
C	**C**entre Line of Road/Berm/Pathway/Railway
L	**L**iDAR control point

Material Code	
1	Mud/Silt (<0.063 mm)
2	Sand (0.063 mm - 2 mm)
3	Gravel (2 mm - 6.4 cm)
4	Cobble (6.4 cm - 25 cm)
5	Boulder (> 25 cm)
6	Bedrock
C	**C**oncrete
G	**G**rass
R	**R**iprap
T	**T**rees (large, trunk > 10 cm)
W	**W**illows and Shrubs
B	Gabion **B**asket
A	**A**sphalt

Examples	
G2	Ground, Sand
G4	Ground, Cobble
W3	Water Level, Gravel
GG	Ground, Grass
GT	Ground, Trees
CA	Centre Line, Asphalt
BR	Bank, Riprap
LC	LiDAR control, Concrete



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NOTES:

1. THE SCHEMATIC AND SURVEY INFORMATION PROVIDED HEREIN WERE PROVIDED BY GOLDER ASSOCIATES LTD.
2. REFER TO SECTION 2.2 OF THE STUDY REPORT FOR MORE INFORMATION.

PREPARED FOR:
Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Schematic of Survey Point Locations and Code Descriptions

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-102	FIGURE NO: 2.1
DATE:	29-JUL-2020	

Muskeg Creek and the Tawatinaw River were surveyed by wading the channels. The banks and shallow areas along the banks of the Athabasca River were also surveyed by wading. However, it was necessary to deploy a river boat to survey most of the channel bed at each cross section.

The main objective of the cross section surveys was to capture the characteristics of the main channel. However, limited overbank floodplain areas were also surveyed to overlap with the LiDAR survey (provided by AEP) where LiDAR coverage was assured. The cross sections were extended into the overbank areas during the hydraulic model development phase using the topographic (LiDAR) data provided by AEP. A breakline survey technique was utilized to capture variances in the bank geometry (i.e., slope breaks), with enough data points collected along each cross section to properly define the channel geometry and the near-bank floodplain.

Each recorded survey data point included Northing and Easting coordinate positions, water surface, and/or ground elevation and was attributed with a survey code that denotes its location (e.g., bank, stream bottom, edge of water, water level, top of bank, etc.).

The field program specifically included a cross section survey at the single Water Survey of Canada (WSC) hydrometric station within the study area. The gauge on the Athabasca River (WSC Station 07BE001) is located on the right[1] (south) bank within the Town's Water Treatment Plant building, as shown in Figure B-2 in Appendix B.

The following procedures were adhered to when conducting a bathymetric survey by wading:

- RTK rover units were used to collect cross-sectional information from a location approximately 2 to 5 m beyond water's edge on one side of the river/creek channel, to a location approximately 2 to 5 m beyond water's edge on the other side. A minimum of 15 survey data points were established across the channel, and care was taken to reference points where the transverse bed slope changed significantly.
- Special attention was paid to surveying topographic slope breaks along the banks.
- Each of the surveyed data points was attributed with field codes that described substrate and vegetation types (see Figure 2-1).
- The water surface elevation was surveyed at all points along the cross section where the water had contact with the bank.

The adopted boat survey method used to define the bathymetry along the Athabasca River study reach involved the following:

- The ADCP was mounted onto a frame, which was fastened to the side of the river boat. The ADCP was deployed in the water once it was securely mounted on the boat, and the distance from the middle sensor to the water surface was measured using a standard tape measure.
- The RTK unit was attached to the top of the ADCP mount at a measured offset from the water surface. This offset was measured and recorded daily.
- The ADCP and RTK units were connected to a laptop-based data acquisition system that provided data storage and a real-time display of the position and data being collected. The system was checked to make sure that both units were communicating properly, and data were being stored.

[1] *Left* or *right* refer to directions as seen by an observer looking downstream.

- A short calibration profile was run at the beginning of each day to verify that both the ADCP offset and the level of the sounding head below the water surface remained consistent while the boat was in motion. Furthermore, the sounding depths were verified by direct measurements during the calibration process.
- The bathymetric data were collected by the ADCP and RTK units at a frequency of one Hertz along the prescribed cross sections (i.e., a data point was collected every second). At a nominal boat speed of 0.75 m/s, this would correspond to a measured depth at horizontal intervals of about 0.75 m. In areas where water depth was less than approximately 0.5 m, survey data points were collected by wading the channel, as described above.
- Bank topographic data were obtained using RTK rover units, as described above.
- The water surface elevation was surveyed at all points along the cross section where the water had contact with the bank.

Processing of the data collected using ADCP and RTK included the following steps:

- Data were sorted using the UTM easting values and any points with UTM coordinates of zero were removed.
- Data were sorted by altitude, which corresponds to the elevation value supplied to the ADCP from the RTK unit (instrument offsets were applied to the data during post-processing).
- Data were sorted by combined depth, and those points with a zero depth or depths well outside of the possible range were discarded.
- Data were sorted according to the difference between the vertical beam (VB) depth and the averaged bottom track (BT) depth. The BT depth was used in cases where the VB depth returned an inaccurate value (i.e., shallow areas).
- Data were sorted by mean velocity. The ADCP returns a value of zero when it cannot compute a flow velocity and vector. These values were removed, and the remaining values within the data set were retained.

Reach-representative photographs were taken at key locations within the study area during the site reconnaissance on May 16, 2019 and throughout the field program. The photographs, which include salient details and features at surveyed cross sections, are georeferenced with appropriate metadata.

2.2.3 Discharge and Water Level Measurements

Discharge and corresponding water levels along a portion of each study reach were measured once during the field program to provide additional data to support the low-flow hydraulic model calibration. The discharge data collected on the Athabasca River also provided a check on the provisional real-time hydrometric data obtained from WSC's online database.

Flow depths were sufficient on the Athabasca River to allow discharge to be measured by deploying the ADCP (SonTek RiverSurveyor M9®) from the river boat. However, the relatively shallow depth and narrow width of the Tawatinaw River and Muskeg Creek required that discharge measurements be made by wading the channel with the handheld ADV (SonTek FlowTracker2®) and top-set wading rod. Both methods provide a measurement accuracy of ±5 percent of the total discharge.

Water levels were measured along the study reach while the discharge measurement was being conducted. Prior to taking the discharge measurement, the crew member who was not directly involved in acquiring discharge data was assigned to measuring water levels along the river. The crew members coordinated their efforts so that the measured water levels and discharge correspond to one another. This approach is superior to only measuring water levels when surveying the cross sections, as doing so minimizes time discrepancies and results in a corresponding water surface profile for the measured discharge.

Discharge Measurement Using ADCP

The ADCP was mounted on the river boat and synchronized, in the same manner as previously described, with regard to the boat survey method used to survey cross sections. The survey crew ensured that an even number of transects (a minimum of four), with an equal number of left-to-right transects as right-to-left transects, were measured so that each individual measurement was within five percent of the others.

Bathymetric (flow depth) and flow velocity data collected by the ADCP on the Athabasca River were exported using the SonTek RiverSurveyorLive® software. The exported data were processed in a spreadsheet application as described in Section 2.2.2. In total, less than five percent of the collected discharge data were removed during the above-mentioned process.

Discharge Measurement Using ADV

Discharge measurements using the ADV were performed in accordance with standard WSC protocols, including the following:

- Selection of a suitable measurement location;
- Choosing an even number of transects with equal left-to-right transects and right-to-left transects; and
- Ensuring that the data set of each transect is within a maximum standard deviation of five percent.

Discharges on the Tawatinaw River and Muskeg Creek were measured using standard WSC measurement techniques as described below.

- Survey points were selected to result in a minimum of 20 panels (flow segments across the stream thus requiring a minimum of 21 velocity measurement points).
- Velocity readings were taken at 0.6 of the total depth at measurement locations, because flow depth was less than 1.0 m in all cases.
- Survey points were selected such that no panel discharge exceeded 10 percent of the total discharge (six panels were within the 5-10 percent range; the remaining 17 panels were all less than five percent).

2.2.4 Hydraulic Structures

All hydraulic structures with the ability to affect channel conveyance and water levels within the study area were surveyed as part of the field program. Applicable structures include traffic bridges, pedestrian bridges, and a roadway culvert.

The features of each bridge structure surveyed included the following:

- Length of span (corner points, abutment to abutment)
- Width of bridge (corner points, outside to outside)
- Top of curb or solid guard rail elevations
- Low chord elevations

- Number and width of piers
- Location of piers and the distance of each pier relative to the left abutment
- Type of piers (e.g., concrete, pile bent, steel column)
- Shape of pier (e.g., round nose, wedge, circular)
- Top of roadway profile

The following data were collected on the roadway culvert within the study area:

- Number of culverts
- Barrel length
- Culvert opening dimensions
- Upstream and downstream invert elevations
- Culvert type (e.g., corrugated steel pipe, concrete box, timber-framed)
- Culvert shape (e.g., circular, arch, elliptical, square, rectangular)
- Entrance condition (e.g., projecting from fill, mitered to conform to slope)
- Top of roadway profile

The hydraulic structures were surveyed using a total station, RTK rover unit, or a combination thereof. The total station, when deployed, was used in reflectorless mode to collect survey points that were difficult to access. In this mode, the user targets the point to be surveyed and a laser beam is transmitted to the object and reflected from the structure without having to use a traditional total station prism (or reflector target). The RTK rover unit was used to collect structural data in clear sky areas where it was possible to connect to the GPS satellites. Georeferenced photographs of each hydraulic structure were taken during the field program.

Two cross sections were surveyed at each bridge or culvert, each located within a short distance upstream and downstream of the bridge face or culvert opening. Ground and structure data were also collected at the inlet and outlet of the culvert to capture key elevations and dimensions.

2.3 Survey Standards and Accuracy

Quality control and quality assurance (QA/QC) of collected data were conducted in the field at the time of data collection and in the office during data processing. QA/QC of field data was conducted as described below.

- Position and elevation from the RTK rover unit were checked for accuracy each day, based on one of the five ASCM benchmarks mentioned previously. All survey data collected during the field program were tied to an ASCM benchmark. Temporary benchmarks were established by the field crew along the watercourses as required to maintain data accuracy.
- The field crew was provided with a shapefile showing cross section alignment for the purpose of guiding the survey along the selected cross sections.
- The RTK data collectors were set up to provide a warning when calculated maximum error exceeded 0.05 m for a manually recorded point. When notified, the surveyor either adjusted their location or waited for a better solution before surveying a point.

The RTK control network is considered accurate to within ±0.02 m at 95 percent confidence in both horizontal and vertical directions. A high level of accuracy was maintained throughout the field program by calibrating the spatial position and elevation of each RTK rover unit to an ASCM benchmark daily. Furthermore, the daily protocol required that the survey crew calibrate to, and then open and close on, an ASCM benchmark to maintain absolute positional accuracy. The hydraulic structures surveyed using a total station are accurate to ±0.01 m, comparable to that of the RTK system.

The collected survey data were imported into a Geographic Information System (GIS) to allow for validation and further processing. In addition to the QA/QC procedures for field data collection, the technical lead for the field program reviewed the survey data within 24 hours of it being collected to check for outliers (including erroneous or missing data) and to ensure appropriate coverage along each cross section and on the hydraulic structures.

2.4 Cross Sections and Longitudinal Profiles

The surveyed length of the Athabasca River was 8.3 km, the surveyed length of the Tawatinaw River was 6.1 km, and the surveyed length of Muskeg Creek was 7.5 km (see Figure B-1). A total of 93 cross sections were surveyed. Table 2-1 provides a summary of surveyed cross sections.

Table 2-1: Surveyed Cross Sections within Study Area

Waterbody	Reach Description	Cross Section ID	No. of Cross Sections	Average Cross Section Spacing (m)
Athabasca River	8.3 km long reach extending 3.5 km upstream and 4.8 km downstream of SH 813 bridge	A-1 to A-23 and Q1	24	400
Tawatinaw River	6.1 km long reach extending upstream from the Athabasca River confluence	T-1 to T-32	32	250
Muskeg Creek	7.5 km long reach extending upstream from the Athabasca River confluence	M-1 to M-37	37	275

Appendix A contains plots of the surveyed main channel thalweg and measured water levels along the Athabasca River (see Figure A-1), the Tawatinaw River (see Figure A-2), and Muskeg Creek (see Figure A-3) during the hydrographic survey. An overview of the surveyed cross section locations is provided in Figures B-2 to B-8 of Appendix B.

2.5 Discharge and Water Level Measurements

A one-time discharge measurement was made on the Tawatinaw River and Muskeg Creek on June 3, 2019 and on the Athabasca River on June 4, 2019 as part of the field program. Water levels were recorded in consort with each discharge measurement to obtain a corresponding water surface profile along each of the reaches. Table 2-2 provides a summary of the discharge and water level measurement data.

Table 2-2: Discharge and Water Level Measurements within Study Area

Waterbody	Date (2019)	Measurement Location	Water Level Measurements		Measured Discharge (m^3/s)
			From	To	
Athabasca River	June 4	Adjacent to WSC Station 07BE001	A9	A23	968
Tawatinaw River	June 3	15 m upstream of footbridge	T3	T15	0.22
Muskeg Creek	June 3	40 m upstream of culvert	M2	M13	0.037

2.6 Hydraulic Structures

The study area includes seven hydraulic structures (i.e., one bridge on the Athabasca River, two bridges on the Tawatinaw River, three bridges on Muskeg Creek, and one road culvert on Muskeg Creek). A summary of the general characteristics of these hydraulic structures is provided in Table 2-3.

Table 2-3: Hydraulic Structures within the Study Area

Waterbody	Structure ID	Structure Name / Location	Cross Section ID	Type	No. of Spans	Corresponding Figure Nos.
Athabasca River	HS-01	SH 813 Bridge	A13	Traffic	8	B-2, C-1
Tawatinaw River	HS-02	Highway 55 Bridge	T4	Traffic	1	B-3, C-2
	HS-03	Trans Canada Trail Footbridge	T8	Pedestrian	1	B-3, C-3
Muskeg Creek	HS-04	Highway 2 (50 Avenue) Culvert	M7	Traffic	-	B-6, C-4
	HS-05	Lower (North) Footbridge	M11	Pedestrian	1	B-6, C-5
	HS-06	Middle Footbridge	M16	Pedestrian	1	B-6, C-6
	HS-07	Upper (South) Footbridge	M26	Pedestrian	1	B-7, C-7

Bridge and culvert locations are shown in Figures B-2, B-3, B-6, and B-7 of Appendix B. Figures C-1 to C-7 of Appendix C include site photographs, survey data point locations superimposed onto (aerial) orthoimagery, and salient information regarding each hydraulic structure.

Background information and data (i.e., detailed design and/or as-built survey drawings) obtained from Alberta Transportation (AT) for several traffic bridges and the lone culvert within the study area are not included in this report. Data for the four pedestrian bridges were not available from AT or the local authorities.

2.7 Flood Control Structures

No flood control structure was identified during the site visit. AEP received confirmation from the local authorities in late July 2019 of the absence of any flood-related structure.

2.8 Additional Base Data

Additional base data collected in support of hydraulic modelling and mapping included the following:

- Infrastructure datasets consisting of construction drawings and/or as-built drawings of traffic bridges and the lone culvert, as provided by AT;
- LiDAR topographic data collected by Airborne Imaging Inc. (Airborne) on October 10, 2018 and provided by AEP;
- Recent orthorectified aerial imagery, which was acquired by Airborne on May 26, 2019 and provided by AEP; and
- Provisional streamflow data at the hydrometric station, which is operated by WSC (Station 07BE001 – Athabasca River at Athabasca).

3.0 OPEN WATER HYDROLOGY ASSESSMENT

3.1 Overview

Documentation of a detailed, comprehensive open water hydrology assessment for the Town of Athabasca, including the Athabasca River, Tawatinaw River and Muskeg Creek, is provided in Appendix D. The sections below provide a summary of that assessment. Ice jam floods on the Athabasca River are discussed in Section 5.1 and Appendix H.

3.2 Flooding History

3.2.1 General Information

The Athabasca River has its sources in the Rocky Mountains near Mount Columbia (elevation 3,747 m) and flows northeast for 1,300 km before discharging into the Peace-Athabasca Delta and Lake Athabasca (elevation 208 m) (RAMP 2016a). A map of the Athabasca River basin upstream of the Town of Athabasca is shown in Figure 3-1. The river drains an area of approximately 74,602 km^2 at the gauging station at Athabasca (i.e., Athabasca River at Athabasca, WSC Station 07BE001; period of record 1913-2018). No historical gauging data are available for the Tawatinaw River or Muskeg Creek.

As a major river system, the Athabasca River is influenced by a variety of climate, terrain and landscape characteristics of its basin (RAMP 2016b). The seasonal climate is a major factor affecting the river flow conditions. The climate is characterized by cold winters when most of the seasonal precipitation falls as snow, followed by warm summers when snow and glacial melt from the river's headwaters combine with runoff from localized snowmelt and rainfall events throughout the basin.

The Athabasca River flows through the Town of Athabasca. The downtown and residential sections of the Town are located south of the former industrial section and rail yards that were located on the south bank of the Athabasca River and have been redeveloped into the highway, commercial, industrial and park uses. The Tawatinaw River and Muskeg Creek flow north from the eastern and western boundaries of the Town into the Athabasca River.

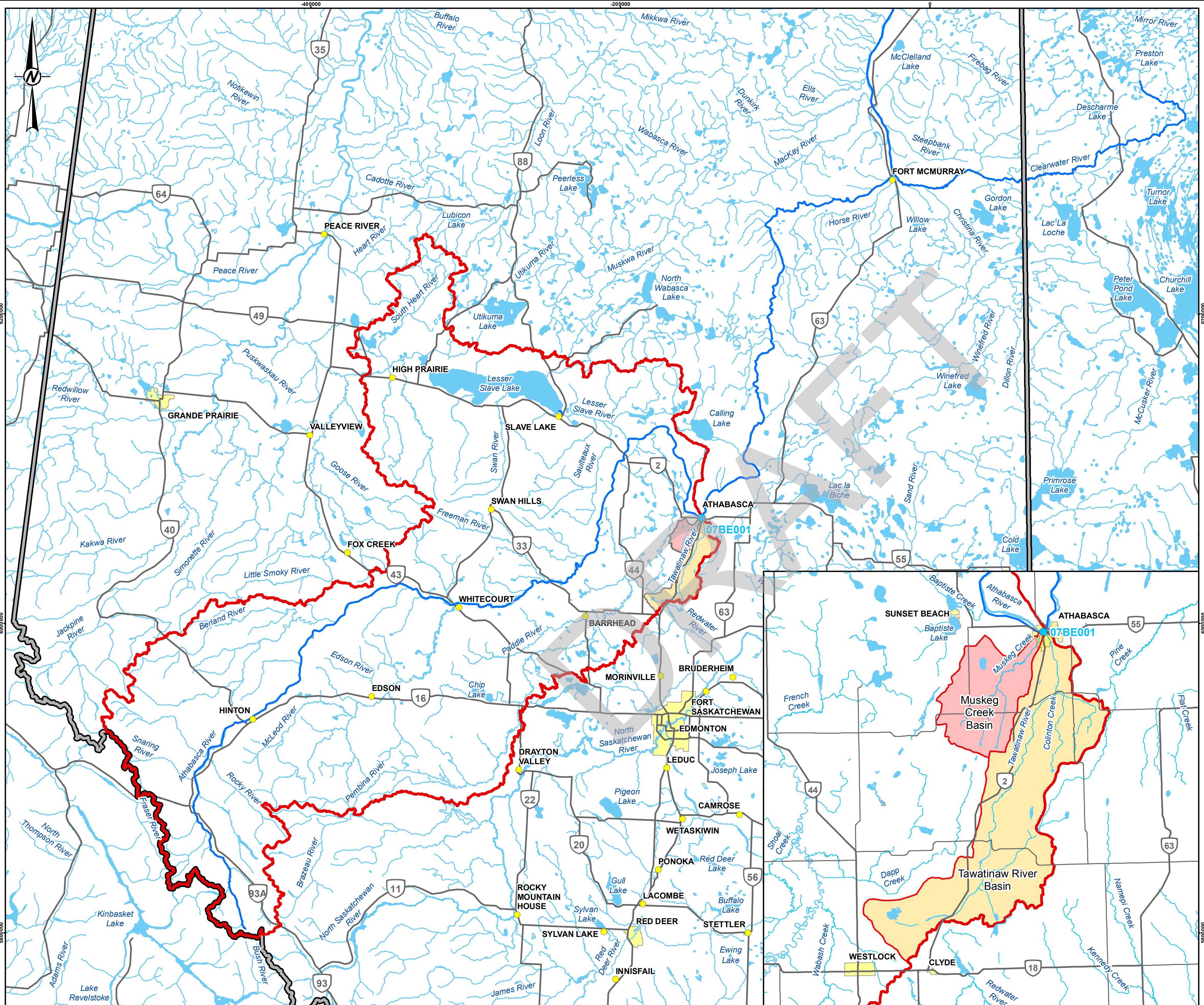
The Tawatinaw River originates south of the Hamlet of Rochester and flows in the northerly direction through the Hamlet. It has a drainage area of approximately 865 km^2 at the Town of Athabasca. The drainage basin primarily consists of pasture land, with some forested areas. The drainage basin is not well drained, and includes several sloughs and lakes.

Muskeg Creek has similar drainage characteristics as Tawatinaw River and has a drainage area of approximately 275 km^2 at the Town of Athabasca.

3.2.2 Open Water Flood History

The Town has experienced Athabasca River flooding periodically. The last large flood event occurred in 1991. Available records indicate that major open-water floods occurred in 1914, 1944, 1948, 1954, 1965, 1969, 1971, 1972, 1980, 1982, 1986, 2011, and 2012, with the following highlights:

- The 1954 flood was the highest on record, with a reported maximum mean daily discharge of 5,440 m^3/s and a maximum instantaneous discharge of 5,660 m^3/s.
- The 1944 flood was the second largest on record, with a reported maximum mean daily discharge of 4,810 m^3/s and a maximum instantaneous discharge of 5,040 m^3/s.
- The 2011 flood was the third largest on record, with a reported maximum mean daily discharge of 3,810 m^3/s and a maximum instantaneous discharge of 4,850 m^3/s.



LEGEND

- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- WATERCOURSE
- MAJOR RIVER
- WATERBODY
- POPULATED PLACE
- PROVINCIAL BOUNDARY
- HYDROMETRIC GAUGING STATION 07BE001
- ATHABASCA RIVER UPSTREAM OF WSC 07BE001
- TAWATINAW RIVER AT ATHABASCA RIVER CONFLUENCE
- MUSKEG CREEK AT ATHABASCA RIVER CONFLUENCE

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1:2,500,000 KILOMETRES

REFERENCE(S)

HYDROMETRIC STATIONS AND BASIN DATA OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC).
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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
ATHABASCA RIVER BASIN AT ATHABASCA INCLUDING TAWATINAW RIVER AND MUSKEG CREEK SUB-BASINS

CONSULTANT

GOLDER

YYYY-MM-DD	2019-07-25
DESIGNED	MG
PREPARED	PT
REVIEWED	GB
APPROVED	NS

PROJECT NO.	CONTROL	REV.	FIGURE
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These floods were typically associated with high rainfall or rain-on snow events in the Athabasca River headwaters in June and July.

As noted, no gauging data are available for the Tawatinaw River or Muskeg Creek. Alberta Transportation records for the Tawatinaw River (AT 2019) indicate the following:

- A washout of the west approach of the original Tawatinaw River bridge in 1921;
- High water due to an Athabasca River ice jam in 1956, with water levels approximately 6.2 m above the river bed; and
- April 1971 high water not exceeding bankfull elevation at the bridge.

No Alberta Transportation flood history is available for Muskeg Creek.

3.3 Open Water Flood Frequency Analysis

3.3.1 Athabasca River

Flood frequency analyses of the annual maximum instantaneous discharge series, including the preliminary estimates of the 2017 and 2018 flood flows, at one location within the study area (i.e., Athabasca River at Athabasca) were conducted to estimate the flood peak discharges of various return periods (i.e., 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year floods).

Table 3-1 summarizes the flood peak discharge estimates and the associated upper and lower 95% confidence intervals. The annual maximum instantaneous discharge series used in the flood frequency analyses, the various frequency distributions, and the best-fit distributions along with their 95% confidence intervals are provided in Appendix D.

3.3.2 Tawatinaw River and Muskeg Creek

A regional hydrological analysis was used to develop flood peak discharge estimates based on drainage areas, for the two ungauged tributaries (i.e., Muskeg Creek and the Tawatinaw River). Empirical relationships between drainage areas and flood peak discharges were established based on available regional flow records for the various return periods ranging from 2 to 1,000 years. The relationships were then used to derive the flood frequency estimates for the tributaries in the study area.

Table 3-1 summarizes the flood peak discharge estimates and the associated upper and lower 95% confidence intervals. The annual maximum instantaneous discharge series used in the flood frequency analyses, the various frequency distributions, and the best-fit distributions along with their 95% confidence intervals are provided in Appendix D.

3.3.3 Comparison to Previous Studies

Table 3-2 presents a comparison of the flood frequency estimates obtained in this study for the Athabasca River at Athabasca, Muskeg Creek, and the Tawatinaw River with the studies previously completed by Environment Canada (EC 1993) as well as IBI and Golder (2014).

Table 3-1: Flood Peak Discharge Estimates and their 95% Confidence Intervals

WSC Station ID / Node ID	WSC Station Name or Location of Interest	Gross Drainage Area (km²)	Flow Type	Distribution[1]	Recommended Instantaneous Flood Peak Discharges (m³/s)																																						
					1000-yr			750-yr			500-yr			350-yr			200-yr			100-yr			75-yr			50-yr			35-yr			20-yr			10-yr			5-yr			2-yr		
					Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower
07BE001 / Node 100	Athabasca River at Athabasca (WSC Station 07BE001)	74,602	Natural	EV2	9,340	12,500	6,080	8,820	11,600	5,850	8,120	10,500	5,540	7,540	9,520	5,280	6,700	8,260	4,890	5,760	6,880	4,420	5,400	6,390	4,230	4,920	5,750	3,960	4,530	5,220	3,720	3,940	4,460	3,350	3,280	3,670	2,880	2,670	2,960	2,410	1,880	2,050	1,740
Muskeg Creek / Node 200	Muskeg Creek at the Town of Athabasca	275	Natural	Regional	98.8	125	66.2	87.9	112	58.3	74.3	95	48.8	64	82.1	41.8	50.2	64.7	32.8	36.7	47.3	24.2	32.1	41.3	21.3	26.4	33.9	17.7	22	28.2	14.9	16.3	20.8	11.2	10.8	13.6	7.54	6.56	8.27	4.63	2.46	3.11	1.71
Tawatinaw River / Node 300	Tawatinaw River at the Town of Athabasca	865	Natural	Regional	160	180	123	146	164	112	129	144	99	114	128	89	94.3	105	74	73.1	81	58	65.2	72	52	55.1	60	45	47.1	51	39	36.0	39	30	24.5	26	21	15.1	16	13	5.5	6	5

Note:
1. Confidence intervals for the regional analysis were determined based on confidence interval of best fit regional curve.

Table 3-2: Comparison of the Flood Frequency Estimates of Various Studies

Return Period (years)	Athabasca River at Athabasca (WSC Station 07BE001)			Muskeg Creek at the Town of Athabasca			Tawatinaw River at the Town of Athabasca		
	EC (1993)	IBI and Golder (2014)	This Study	EC (1993)	IBI and Golder (2014)	This Study	EC (1993)	IBI and Golder (2014)	This Study
	Log Pearson Type III	EV2	EV2	Log Normal	Regional Index Station	Regional Analysis	Log Pearson Type III	Regional Analysis	Regional Analysis
2	-	1,873	1,880	-	2	2.46	-	8	5.5
5	-	2,729	2,670	-	4	6.56	-	21	15.1
10	3,420	3,383	3,280	10	5	10.8	35	33	24.5
20	-	4,070	3,940	-	7	16.3	-	48	36.0
25	-	4,301	-	-	8	-	-	53	-
50	5,250	5,050	4,920	23	10	26.4	55	70	55.1
100	6,200	5,846	5,760	30	12	36.7	65	98	73.1
500	-	7,961	8,120	-	19	74.3	-	144	129

Notes:
1. The EC (1993) study involved use of the recorded data up to 1989 for the Athabasca River at Athabasca.
2. The IBI and Golder (2014) study involved use of the recorded data from 1913 to 2011 for the Athabasca River at Athabasca. In the IBI and Golder (2014) study, flood estimates for Muskeg Creek were assumed to be the same as flood estimates for Babette Creek near Colinton (i.e., WSC Station No. 07CA008).

The flood frequency estimates were based on the recorded data up to 1989 in the EC (1993) study and up to 2011 in the IBI and Golder (2014) study. The current study is based on the published flow data up to 2016, the provisional flow data for 2017 to 2018 from WSC for Athabasca River at Athabasca, and the published data up to 2018 for some of the regional gauging stations. In addition, this study includes the analyses to update the relationships between annual maximum daily and annual maximum instantaneous discharges.

The results of the comparison are summarized below:

- The resulting flood frequency estimates of this study for the Athabasca River at Athabasca are lower than those in the EC (1993) and IBI and Golder (2014) studies.
- The resulting flood frequency estimates of this study for Muskeg Creek at the Town of Athabasca are higher than those in the EC (1993) and IBI and Golder (2014) studies. In the IBI and Golder (2014) study, flood flow estimates for Muskeg Creek assumed to be the same as those for Babette Creek near Colinton (i.e., WSC Station No. 07CA008 assumed as regional index station).
- The resulting flood frequency estimates of this study for the Tawatinaw River at the Town of Athabasca are comparable to the results in the EC (1993) study but lower than the IBI and Golder (2014) study.

The main differences in the flood frequency estimates are due to the different periods of record of data used in the flood frequency analyses, and the selections of different frequency curve distributions. There is also a difference in the watershed area of the Tawatinaw River at Athabasca between this study and the IBI and Golder (2014) study.

4.0 OPEN WATER HYDRAULIC MODELLING

4.1 Overview

The following sections document the methodology and results of the open water hydraulic modelling component. The scope of this component includes summary of available data, description of the flooding history and river/creek and valley features in the study area, hydraulic model setup, hydraulic model calibration and validation, sensitivity analysis, and generation of open water flood frequency profiles. The results of this component are used in the flood inundation mapping, flood hazard identification, governing design flood hazard mapping, and flood risk assessment components.

4.2 Available Data

4.2.1 Digital Terrain Model

Digital Terrain Model (DTM) data was provided by AEP for this study. The DTM was derived from survey-verified high-accuracy Light Detection and Ranging (LiDAR) remote sensing data set acquired during October 2018 by Airborne Imaging (2020).

4.2.2 Existing Model

There is one hydraulic model previously developed for the study area as listed in Table 4-1.

Table 4-1: Existing Hydraulic Model

No.	Study Description	Program Used for Model Development	Date	Author or Source
1	Athabasca Hydraulic Study	HEC-2	1993	Environment Canada

4.2.3 Highwater Marks

There are two sets of historic open water flood highwater mark (HWM) data available (1980 and 1990) along the Athabasca River study reach. There is no historic HWM data available along the study reaches of Muskeg Creek and the Tawatinaw River.

The HWM reports and data available for this study are listed in Table 4-2 and locations of HWMs are shown in Figure 4-1.

Table 4-2: Available Highwater Mark Reports and Data

No.	Report Title	Flood Year	Author or Source
1	High Water Mark Data – Athabasca River	1980	Alberta Environment
2	High Water Mark Data – Athabasca River	1990	Alberta Environment

4.2.4 Gauge Data and Rating Curves

There is one active Water Survey of Canada (WSC) hydrometric gauging station located within the study area (i.e., WSC Station 07BE001 - Athabasca River at Athabasca), and shown in Figure 3-1 and in more detail in Figure B-2 in Appendix B. This station is located on the right bank of the Athabasca River, approximately 1.1 km upstream of the Highway 813 bridge. The rating curve of river stage versus discharge for this station was used for the model calibration.

There is no hydrometric gauging station located on Muskeg Creek or the Tawatinaw River within the study area.

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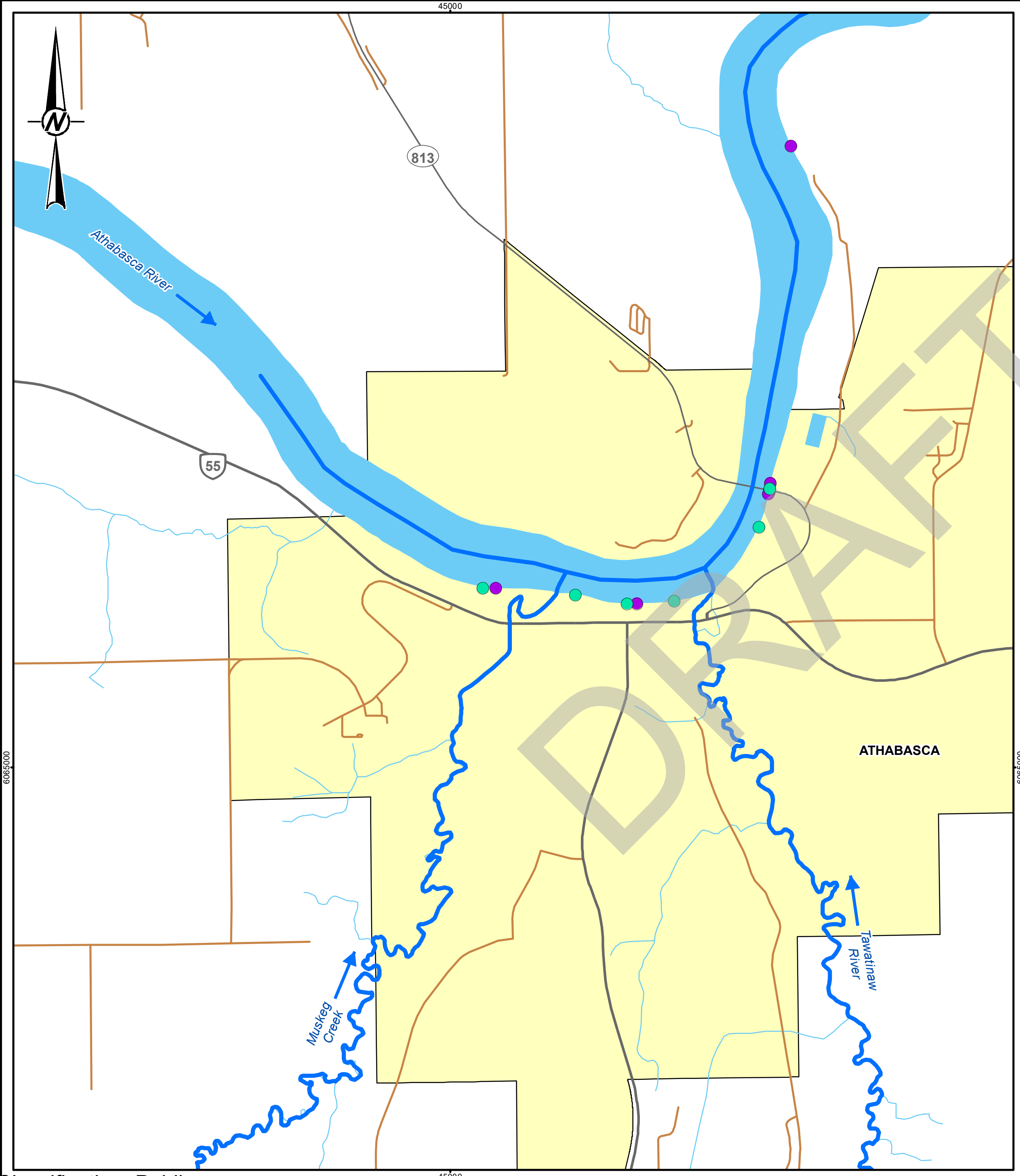

LEGEND

- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- POPULATED PLACE
- SURVEY REACH

HIGH WATER MARK

- 1980
- 1990

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1:35,000 KILOMETRES

REFERENCE(S)

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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
LOCATIONS OF SURVEYED HIGHWATER MARKS

CONSULTANT

GOLDER

YYYY-MM-DD	2020-08-10
DESIGNED	JC
PREPARED	PT
REVIEWED	GB
APPROVED	NS

PROJECT NO.	CONTROL	REV.	FIGURE
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Classification: Public

4.2.5 Flood Photography

There is no aerial flood photography available to this study.

4.3 River and Valley Features

4.3.1 General Description

The Athabasca Hydraulic Study (EC 1993) provided a general description of the Athabasca River basin. The Athabasca River originates in the glacial dome of the Columbia Icefield and winds its way for a stream distance of 1,280 km across Alberta to Lake Athabasca. It flows in a north-easterly direction, passing Jasper, Hinton and Whitecourt. Downstream of the Lesser Slave River confluence, Athabasca River loops to the southeast and flows through the Town of Athabasca. Downstream of the Town of Athabasca, it continues flowing in a predominantly northern direction, passing Fort McMurray and discharging into Lake Athabasca.

Starting as a rapid mountain stream, it gradually matures as it receives flow from tributaries, until its mouth at Lake Athabasca, where it conveys water from a drainage area of approximately 155,000 km^2. The Athabasca River near the Town of Athabasca has a stream-cut valley with occasional slumps and moderately forested valley walls. The river is entrenched and exhibits irregular meandering.

The two tributaries included in this study, Muskeg Creek and the Tawatinaw River, are of special interest because they bracket the lower Town site on the east and west, and could potentially cause flooding in some areas.

The man-made structures along the study reaches of Athabasca River and the two tributaries which are relevant for hydraulic modeling include highway/local road bridges and culverts (see Table 4-3 and Appendix C).

4.3.2 Channel Characteristics

The Athabasca River study reach is approximately 8.3 km long. It extends from the upstream study boundary, through the Town of Athabasca, to a location approximately 5 km downstream of the Highway 813 bridge crossing. The Athabasca River flows in a well-defined, single channel. The study reach has a typical channel bottom width of 240 m, bankfull width of 320 m, and bankfull depth of 6.5 m. It has an average channel bed slope of 0.025% and sinuosity of 1.03. The channel bed and bank materials consist of gravel, sand, silt and clay. The river banks are well vegetated.

The Muskeg Creek study reach is approximately 7.5 km long. It extends from the upstream study boundary to its confluence with Athabasca River in the Town of Athabasca. Muskeg Creek has an incised single channel, confined on both sides by valley walls. The study reach has a typical channel bottom width of 4.2 m, bankfull width of 10 m, and bankfull depth of 1.2 m. It has an average channel bed slope of 0.49% and sinuosity of 1.71. The channel bed and bank materials consist of gravel, sand, silt and clay. Beaver dams and debris were observed in the upper reach of Muskeg Creek during the field inspection and survey for this study. The creek banks are partially vegetated.

The Tawatinaw River study reach is approximately 6.1 km long. It extends from the upstream study boundary to its confluence with Athabasca River in the Town of Athabasca. The Tawatinaw River has an incised single channel, confined on both sides by valley walls. The study reach has typical channel bottom width of 5.6 m, bankfull width of 18 m, and bankfull depth of 2.4 m. It has an average channel bed slope of 0.19% and sinuosity of 1.58. The channel bed and bank materials consist of gravel, sand, silt and clay. Beaver dams and debris were observed in the upper reach of the Tawatinaw River during the field inspection and survey for this study. The creek banks are partially vegetated.

4.3.3 Floodplain Characteristics

The Athabasca River near the Town of Athabasca has a stream-cut valley with occasional slumps and moderately forested valley walls. The floodplains along the study reach are relatively small. The floodplain width is typically 15 m (excluding the channel width) with a range of 5 to 80 m. The vegetation cover on the floodplains of the study reach consists mainly of dense forest.

The Muskeg Creek study reach has steep and moderately forested valley walls. The floodplains along the study reach are narrow. The floodplain width is typically 40 m (excluding the channel width) with a range of 20 to 100 m. The vegetation cover on the floodplains of the study reach consists mainly of dense forest/willows and scattered grasses.

The Tawatinaw River study reach has steep and forested valley walls. The floodplains along the study reach are wide relative to those of Muskeg Creek. The floodplain width is typically 50 m (excluding the channel width) with a range of 30 to 120 m. The width of floodplains increases gradually from about 30 m at the upstream study boundary to about 120 m near the Town of Athabasca. The vegetation cover on the floodplains of the river reach consists mainly of dense forest and scattered willows/grasses.

4.3.4 Anthropogenic Features

The Town of Athabasca is located approximately 140 km north of Edmonton, on the banks of the Athabasca River. It is the centre of Athabasca County and has a population of 3,000, according to the 2016 Census of Population conducted by Statistics Canada. The area immediately adjacent to the right river bank consists of a low level flatland, followed by high level terrace. The river floodplain areas between Muskeg Creek and the Tawatinaw River are urbanized.

4.3.5 Bridges and Culverts

There are six bridge crossings and one culvert crossing in the study area (see Table 4-3) as summarized below:

- One highway bridge (Secondary Highway 813 bridge) crossing on the Athabasca River reach;
- One highway culvert (Highway 2 culvert) crossing and three footbridge crossings on the Muskeg Creek reach; and
- One highway bridge (Highway 55 bridge) and one footbridge crossing on the Tawatinaw River reach.

Table 4-3: Bridge and Culvert Crossings within the Study Area

No.	Name	Description	Type
1	Secondary Highway 813 Bridge on the Athabasca River	650 m downstream of the Tawatinaw River (see Figure C-1 in Appendix C)	8-Span
2	Highway 2 (50 Avenue) Culvert on Muskeg Creek	Between 56th Street and 57th Street in Athabasca (see Figure C-4 in Appendix C)	8.5 span x 5.2 m rise
3	Upper (South) Footbridge on Muskeg Creek	Approximately 2.3 km upstream of the Highway 2 culvert crossing (see Figure C-7 in Appendix C)	1-Span
4	Middle Footbridge on Muskeg Creek	Approximately 0.7 km upstream of the Highway 2 culvert crossing (see Figure C-6 in Appendix C)	1-Span

Table 4-3: Bridge and Culvert Crossings within the Study Area

No.	Name	Description	Type
5	Lower (North) Footbridge on Muskeg Creek	Approximately 0.2 km upstream of the Highway 2 culvert crossing (see Figure C-5 in Appendix C)	1-Span
6	Highway 55 Bridge on the Tawatinaw River	East of 48th Street in Athabasca (see Figure C-2 in Appendix C)	1-Span
7	Trans Canada Trail Footbridge on the Tawatinaw River	Approximately 0.2 km upstream of the Highway 55 bridge crossing (see Figure C-3 in Appendix C)	1-Span

4.3.5.1 Weir and Dam

There is no weir or dam along the study reaches of the Athabasca River, Muskeg Creek and the Tawatinaw River.

4.3.5.2 Flood Control Structure

There is no flood control structure (e.g., berm or dike) along the study reaches of the Athabasca River, Muskeg Creek, and the Tawatinaw River.

4.4 Model Construction

4.4.1 Methodology

The latest HEC-RAS program (Version 5.0.7, March 2019) was used to develop the one-dimensional (1D) hydraulic models for the study area.

The HEC-GeoRAS module (Version 10.1) was used to prepare cross section data based on the recent LiDAR and river survey data. HEC-GeoRAS is an ArcGIS extension tool specifically designed to create a HEC-RAS import file from geospatial data.

4.4.2 HEC-RAS Program

The HEC-RAS program was developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACE 2016a). The software has a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphics and reporting facilities. HEC-RAS is a commonly-used program in North America and around the world.

The HEC-RAS program was designed to perform one-dimensional (1D), two dimensional (2D) or combined 1D and 2D hydraulic calculations for a full network of natural and constructed channels. The program supports steady-state and unsteady-state hydraulic simulation. HEC-RAS can be used to calculate water surface profiles for gradually varied flow. In this study, the program was used for 1D steady-state simulation. However, preliminary 2D runs were completed to guide 1D model cross section alignments and spacing.

The basic computational procedure for 1D steady-state simulation is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion. The momentum equation is utilized in situation where the water surface profile is rapidly varied. The program can be used to simulate the effects of various obstructions such as bridges, culverts, weirs, levees and other structures. The program is capable of simulating the water surface profiles associated with subcritical, supercritical and mixed flow regimes. In this study, the program was run in sub-critical flow only, as the calculated Froude Number values are less than one along the study reaches.

The main assumptions of 1D steady-state modelling are listed below:

- Flow is steady;
- The variation of the river channel and floodplain geometries is represented by a series of cross sections;
- The water level is constant at each cross section;
- Flow is gradually varied except at hydraulic structures;
- The channel slope is less than 10%; and
- The flow is perpendicular to the cross section alignment.

4.4.3 General Model Setup

4.4.3.1 Model Domain

It is generally desirable to use a single geometry file to simulate floods of various return periods. Therefore, the model domain needs to be defined for covering inundation extents of the largest flood event to be simulated. The 1D model domain was defined in consideration of the simulation results of a supplemental HEC-RAS 2D model, which was set up based on the LiDAR DEM without inclusion of the channel bathymetry, to provide conservative water level estimates.

The inundation extents of the 1,000-year flood event were estimated using the supplemental HEC-RAS 2D modelling results. The 1D model domain was defined to include the flood extents from the supplemental 2D model and a buffer zone covering additional areas with elevations of 2 to 3 m higher than the flood levels from the supplemental 2D model.

To account for the downstream boundary effects, a short river reach (i.e., approximately 2 km on the Athabasca River) downstream of the study reach (as specified in the Terms of Reference) was included in the 1D model.

4.4.3.2 Reaches

The 1D model domain was divided into a number of reaches, as described in Table 4-4. The reach breaks in the Athabasca River portion of the model correspond to the confluences with the tributaries, with Muskeg Creek joining the Athabasca River between the Upper Athabasca and Middle Athabasca reaches, and the Tawatinaw River joining the Athabasca River between the Middle Athabasca and Lower Athabasca reaches.

Table 4-4: HEC-RAS Model Reach Descriptions for Athabasca River, Muskeg Creek and Tawatinaw River

Reach	Cross Sections	Upstream River Station	Downstream River Station	Reach Length (m)
Upper Athabasca	XS 1 to XS 5	8339.925	6111.871	2228.054
Middle Athabasca	XS 6 to XS 8	6111.871	5352.214	759.657
Lower Athabasca	XS 9 to XS 23	5352.214	6.481	5345.733
Lower Muskeg	XS 24 to XS 57	7593.556	141.030	7452.526
Lower Tawatinaw	XS 58 to XS 89	6229.830	152.927	6076.903

4.4.3.3 Separate Branch

The 1D HEC-RAS model is based on assumed constant water level at each cross section, including both main channel and overbank areas. This assumption also applies to any side channels included in each cross section, unless these side channels are explicitly represented with separate branches in the model.

There is no separate branch along the study reaches represented in the model.

4.4.3.4 Boundary Conditions

The HEC-RAS model requires specification of boundary conditions at all open and internal boundaries. The open boundaries of the hydraulic model are listed below:

- Discharges at the upstream model boundaries of the Athabasca River, Muskeg Creek and the Tawatinaw River; and
- Normal flow condition (with an estimated energy slope of 0.026%) at the downstream model boundary of the Athabasca River.

A schematic of the HEC-RAS model is shown in Figure 4-2, corresponding to the reach descriptions provided in Table 4-4.

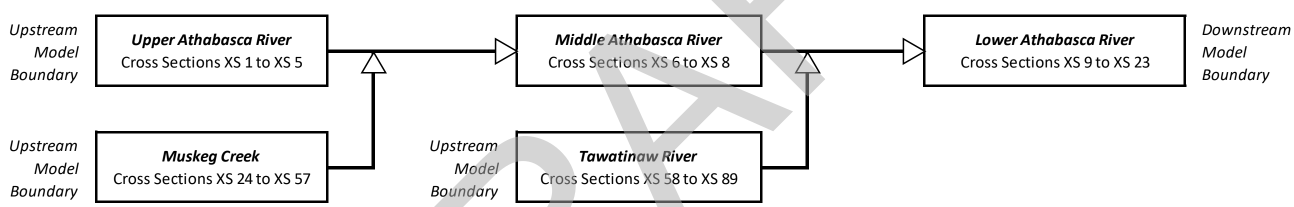

Figure 4-2: Schematic of HEC-RAS Model Setup

4.4.4 Geometric Data Base

4.4.4.1 Cross Section Data

The locations of the cross sections in the model were selected based on the locations of the surveyed cross sections and modelling requirements. The cross section data were obtained from the following sources:

- River survey data collected for this study (see Section 2.0); and
- 2018 LiDAR data provided by AEP.

The alignments of the cross sections in the floodplain areas were defined in consideration of the following:

- Simulated water surface isolines generated from the supplemental HEC-RAS 2D model;
- Topographic contours;
- Estimated flow directions; and
- Key structures.

It is preferable that cross sections are aligned along the water surface isolines simulated from the supplemental 2D model, so that the water levels along the alignments are approximately the same at individual cross sections. HEC-GeoRAS was used to define the main channels, overbank flow paths, bank stations, cross section river stations, and the connections between the main and branch channels. Table 4-5 provides an overview of the river reaches and the number of cross sections in each study reach represented in the model.

Table 4-5: Cross Sections Used in the HEC-RAS Model

River Name in HEC-RAS	Reach Name in HEC-RAS	Description of Reach	From River Station (m)	To River Station (m)	Length (m)	Number of Cross Sections
Athabasca River	Upper, Middle and Lower Athabasca River	Upstream boundary to downstream boundary	8340	6.5	8,333	24
Muskeg Creek	Lower Muskeg	Upstream boundary to its confluence with the Athabasca River	7594	141	7,453	37
Tawatinaw River	Lower Tawatinaw	Upstream boundary to its confluence with the Athabasca River	6230	153	6,077	32
					TOTAL	93

4.4.4.2 Roughness Coefficients

The left and right bank stations defining the main channel were determined using HEC-GeoRAS based on the 2018 LiDAR data, 2019 aerial imagery and survey data. Manning's *n* values were specified using the distributed roughness approach, which allows for multiple, varying roughness values within each cross section. The initial roughness distribution was specified based on the following data:

- Bank lines established from the LiDAR data, aerial imagery and surveys to identify the main channels; and
- Land use information from Government of Alberta.

Five roughness classes were used for the model setup. The initial Manning's *n* values assigned to the classes are listed in Table 4-6. These initial values were selected based on channel bed materials, vegetation types, etc. (Chow 1959; USACE 2016b). These roughness values were modified at some locations during the model calibration process. The roughness values were specified in the cross sections using HEC-GeoRAS. Figure 4-3 shows the distribution of the roughness classes.

Table 4-6: Roughness Classes and Initial Manning's n Values

Number	Description	Initial Manning's *n*
1	Rivers-Main Channel	0.030
2	Urban Mixture (Residential)	0.080
3	Urban Mixture (Commercial)	0.060
4	Streets	0.030
5	Grassland and Open Space	0.050
6	Golf Course	0.050
7	Ponds	0.030
8	Dense Vegetation	0.150

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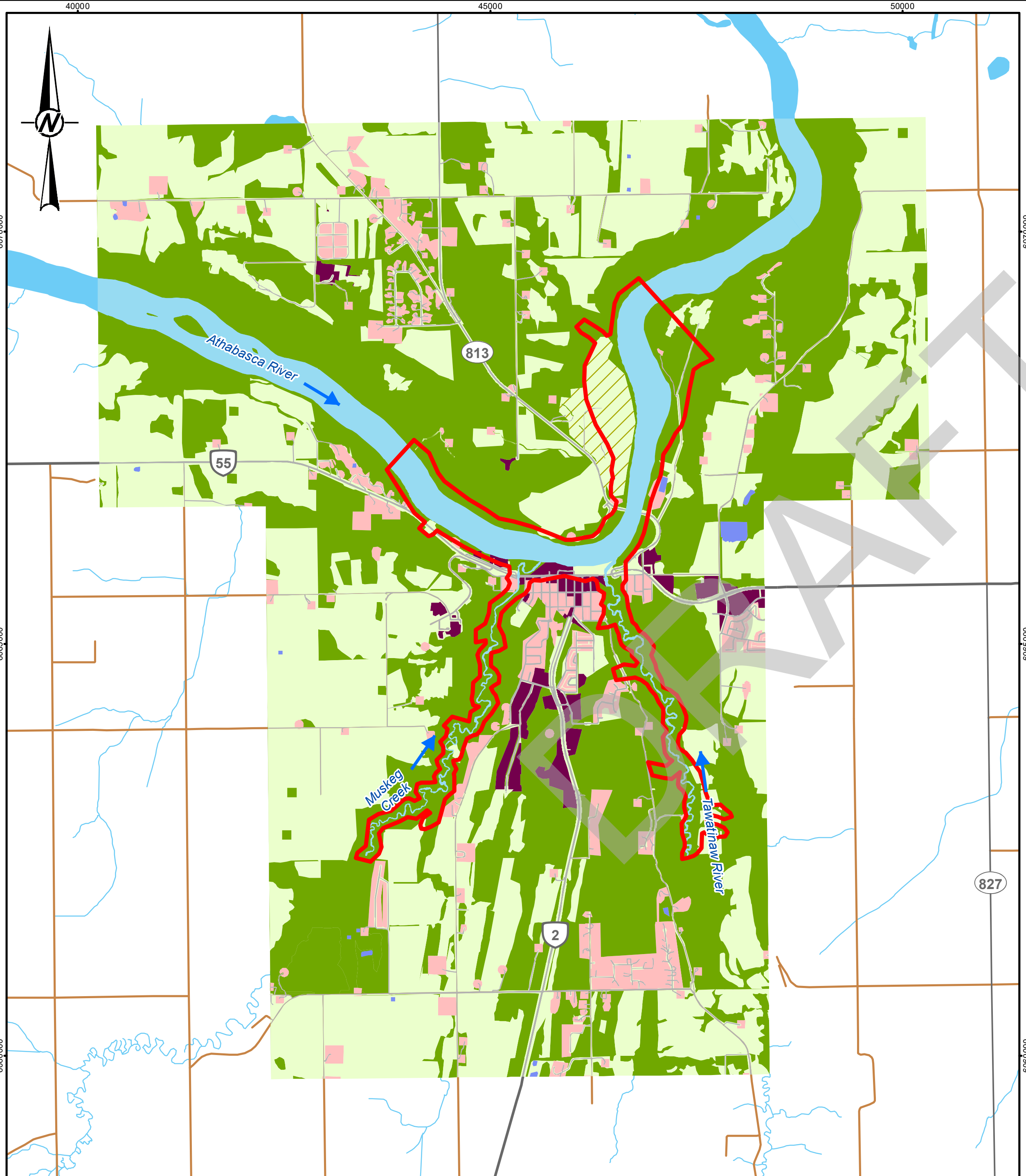

LEGEND

- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- RIVER HAZARD STUDY AREA
- DENSE VEGETATION
- GOLF COURSE
- GRASSLAND AND OPEN SPACE
- POND
- RIVERS - MAIN CHANNEL
- STREET
- URBAN MIXTURE (COMMERCIAL)
- URBAN MIXTURE (RESIDENTIAL)

0 2.5 5

1:75,000 KILOMETRES

REFERENCE(S)

POPULATED PLACES AND HYDROGRAPHY OBTAINED FROM ALTALIS, © GOVERNMENT OF ALBERTA 2019. ALL RIGHTS RESERVED.
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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
DISTRIBUTION OF ROUGHNESS CLASSES

CONSULTANT

GOLDER

YYYY-MM-DD	2020-08-10
DESIGNED	JC
PREPARED	PT
REVIEWED	GB
APPROVED	NS

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4.4.4.3 Hydraulic Structures

4.4.4.3.1 Bridges

The bridge geometries used in the HEC-RAS model were defined based on the following data sources:

- River and bridge surveys conducted in 2019 (refer to Section 2.0); and
- As-built drawings provided by AEP in 2019.

All existing bridges (see Section 4.4) were represented in the HEC-RAS model, including those which may not affect water levels during floods. The bridge deck, pier and abutment information were included in the model. Losses through bridges were calculated in the model using the energy equation (i.e., standard step method). Flows over the bridge and approach embankment were calculated using the standard weir equation.

The bridges were modelled using the cross sections upstream and downstream of the bridges. Cross sections cut along the centerlines of the bridges were not used. This is because the lengths of upstream and downstream cross sections are different in some cases, and this would result in levees and ineffective flow areas being misplaced along the bridge cross sections.

To properly model overland flows that can bypass bridges on floodplains, the multiple flow analysis was implemented. This allows the HEC-RAS model to calculate distribution of flows that are conveyed through the bridge openings and bypassed around the bridges in the floodplains. The bypassed flow in multiple flow analysis was modelled as an open channel flow. Not using the multiple flow analysis approach would result in bypassed flows being modelled as flows over a broad-crested weir.

There are large variations of bridge types, abutments, approaches and embankments within the study area. At individual bridge locations, ineffective areas upstream and downstream of the bridges were carefully selected. This included selection of permanent and non-permanent ineffective areas where appropriate.

The initial values of the contraction and expansion coefficients at bridges were selected to be 0.3 and 0.5, respectively. These are typical values listed in the HEC-RAS User Manual. They were modified at some locations during the model calibration process.

4.4.4.3.2 Culvert

There is one major culvert (i.e., Highway 2 culvert crossing on Muskeg Creek) in the study area. This culvert was represented in the HEC-RAS model based on the survey conducted in 2019. The pertinent culvert information, including size, length, upstream invert and downstream invert elevations, was specified in the model.

The culvert was modelled using the cross sections upstream and downstream of the culvert, with the top of the embankment based on the road surface elevations extracted from the LiDAR DEM. The ineffective areas upstream and downstream of the culvert were carefully selected in consideration of the features of the culvert and road embankment.

The multiple flow analysis approach was implemented at the culvert location to properly model overland flows that could bypass culverts on the floodplains. The initial value of the contraction coefficient at the culvert location was selected to be 0.3, and the initial value of the expansion coefficient to be 0.5. These were modified during the model calibration process.

4.4.4.3.3 Weir and Dam

There is no weir or dam in the study area. Therefore, there is no weir or dam represented in the HEC-RAS model.

4.4.4.4 Flood Control Structure

There is no flood control structure identified in the study area. Therefore, there is no flood control structure represented in the HEC-RAS model.

4.4.5 Model Calibration

4.4.5.1 Methodology

The Manning's *n* and contraction/expansion coefficients are the two primary model parameters with values that were adjusted, if necessary, in calibrating the HEC-RAS model. Selection of the initial Manning's *n* values included consideration of river bed/bank materials, vegetation cover, site information collected during the field inspection, and Golder's experience from previous hydraulic modelling studies.

Manning's *n* values may reduce with increased stage. Both low flow and high flow calibrations were performed to determine appropriate Manning's *n* values across a wide range of flows. The following scenarios were included in the model calibration and validation:

- Low/Normal Flow Calibration: The surveyed water levels and measured flows during the river/creek surveys were used for the low/normal flow calibration.
- High Flow Calibration: Available HWMs and peak flow estimates for the 1980 and 1990 flood events on the Athabasca River were used for the high flow calibration. These two flood events were selected because they were the largest events in recent history and were well-documented in terms of peak flow estimates and available HWMs. There is no HWM information available for calibrating the model for Muskeg Creek and Tawatinaw River.
- Validation Based on Gauging Station Data and Rating Curve: The stage-flow rating curve at one active WSC gauging station (i.e., WSC Station 07BE001) was used for validating the calibrated model for the Athabasca River.

The model calibration process involved multiple iterations to adjust the model parameter values, conduct simulations, and compare the simulated water levels to the HWMs (for the high flow calibration), or the surveyed water levels (for the low/normal flow calibration). The objective of the model calibration was to achieve good matches between the simulated water levels and the HWMs or measured water levels.

The model validation process involved simulation of the flood conditions not used in the model calibration, by maintaining the calibrated model parameter values, and comparing the simulated water levels to the surveyed or recorded water levels. The objective of the model validation was to confirm if the calibrated model can be reliably used to simulate other flood flow conditions.

4.4.5.2 Low/Normal Flow Calibration

The water level and discharge measurements on the Athabasca River were conducted on June 4, 2019, and on the two tributaries (i.e., Muskeg Creek and the Tawatinaw River) on June 3, 2019 (see Table 4-7). For simulating the water levels along the two tributaries on June 3, 2019, the Athabasca River discharge reported for the WSC Station 07BE001 for that date was used in the model for the Athabasca River reach.

Table 4-7: Measured Discharges for the HEC-RAS Model Calibration

Waterbody	Date	WSC Gauging Station	Discharge (m^3/s)	
			WSC Gauge	Measurement
Muskeg Creek	June 3, 2019	Not Applicable	Not Applicable	0.04
Tawatinaw River	June 3, 2019	Not Applicable	No Applicable	0.22
Athabasca River	June 3, 2019	WSC Station 07BE001	954	Not Applicable
Athabasca River	June 4, 2019	WSC Station 07BE001	972	968

Athabasca River

The Athabasca River channel Manning's *n* value was calibrated based on the water level data measured on June 3 and 4, 2019, the recorded discharge at the WSC Station 07BE001 on June 3, 2019, and the measured discharge on June 4, 2019. Figure 4-4 compares the simulated water surface profile to the surveyed water levels for the June 3/4 flow conditions.

The average difference between the simulated and surveyed water levels was 0.2 cm and ranged between -3 cm and +6 cm. The calibrated channel Manning's n value was 0.026 for the June 3/4 flow conditions on the Athabasca River.

Muskeg Creek

The measured water levels on Muskeg Creek were associated with the extremely low flow of 0.04 m^3/s on June 3, 2019. The water levels were strongly influenced by local low-flow features such as channel forms, debris and beaver dams. As a result, the calibrated channel Manning's *n* value for the low flow condition was 2.0. This high value reflects large apparent roughness and local effects under such low flow condition.

Figure 4-5 compares the simulated water surface profile for $n = 0.070$ (a reasonable value for low flow within a rough channel without beaver dams or debris accumulations) to surveyed water levels for the surveyed low flow condition on Muskeg Creek. The average difference between the simulated and survey water levels was 29 cm.

Tawatinaw River

Similar to Muskeg Creek, the measured water levels on the Tawatinaw River were associated with the extremely low flow of 0.22 m^3/s on June 3, 2019. The water levels were strongly influenced by local low-flow features such as channel forms, debris and beaver dams. As a result, the calibrated channel Manning's *n* value for the low flow condition was 0.5. This high value reflects large apparent roughness and local effects under such low flow condition.

Figure 4-6 compares the simulated water surface profile for $n = 0.070$ (a reasonable value for low flow within a rough channel without beaver dams or debris accumulations) to surveyed water levels for the surveyed low flow condition on the Tawatinaw River. The average difference between the simulated and survey water levels was 9 cm.

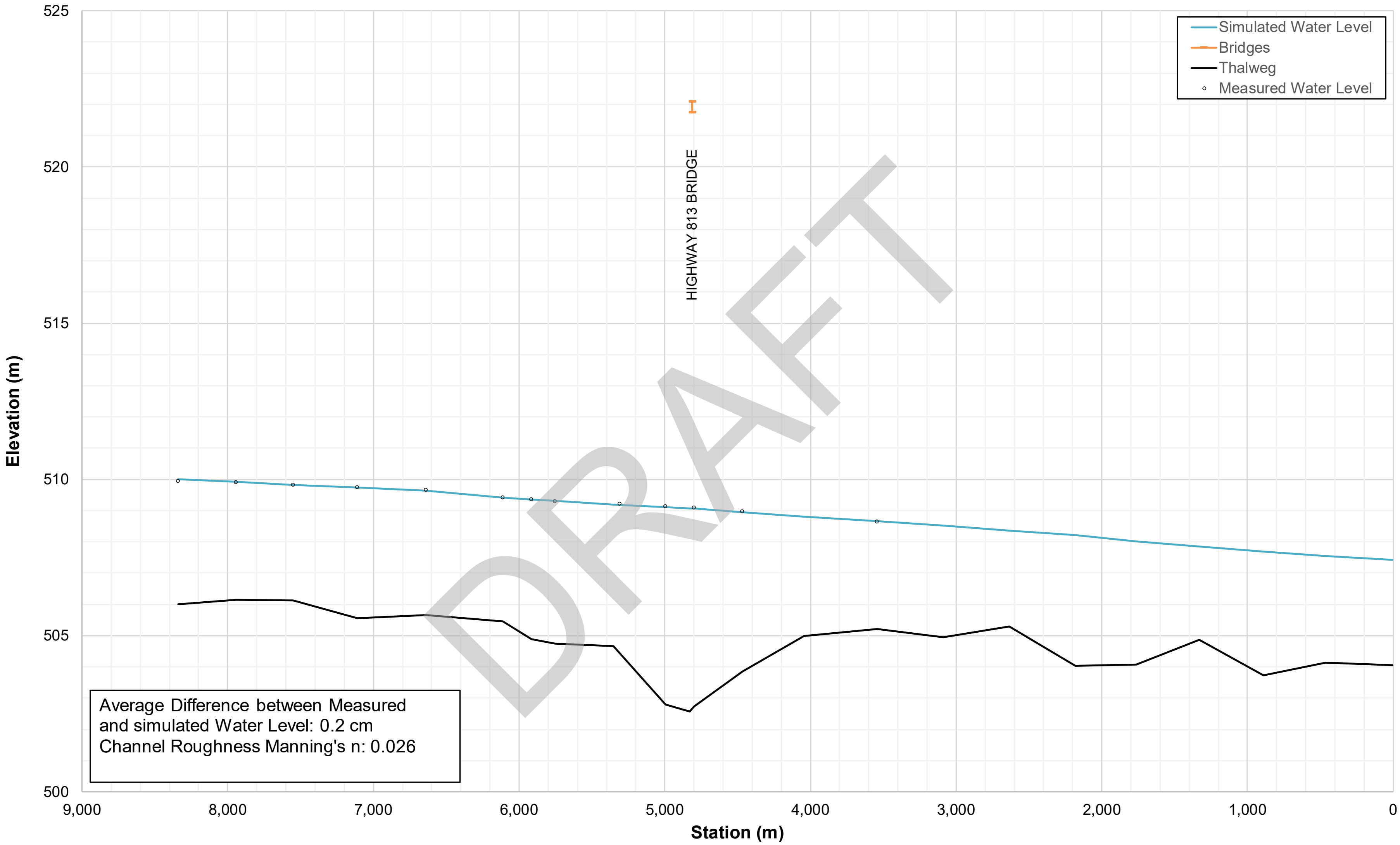

Figure 4-4: Comparison of the Simulated Water Surface Profile to the Surveyed Water Levels on the Athabasca River for the June 4 Flow Condition

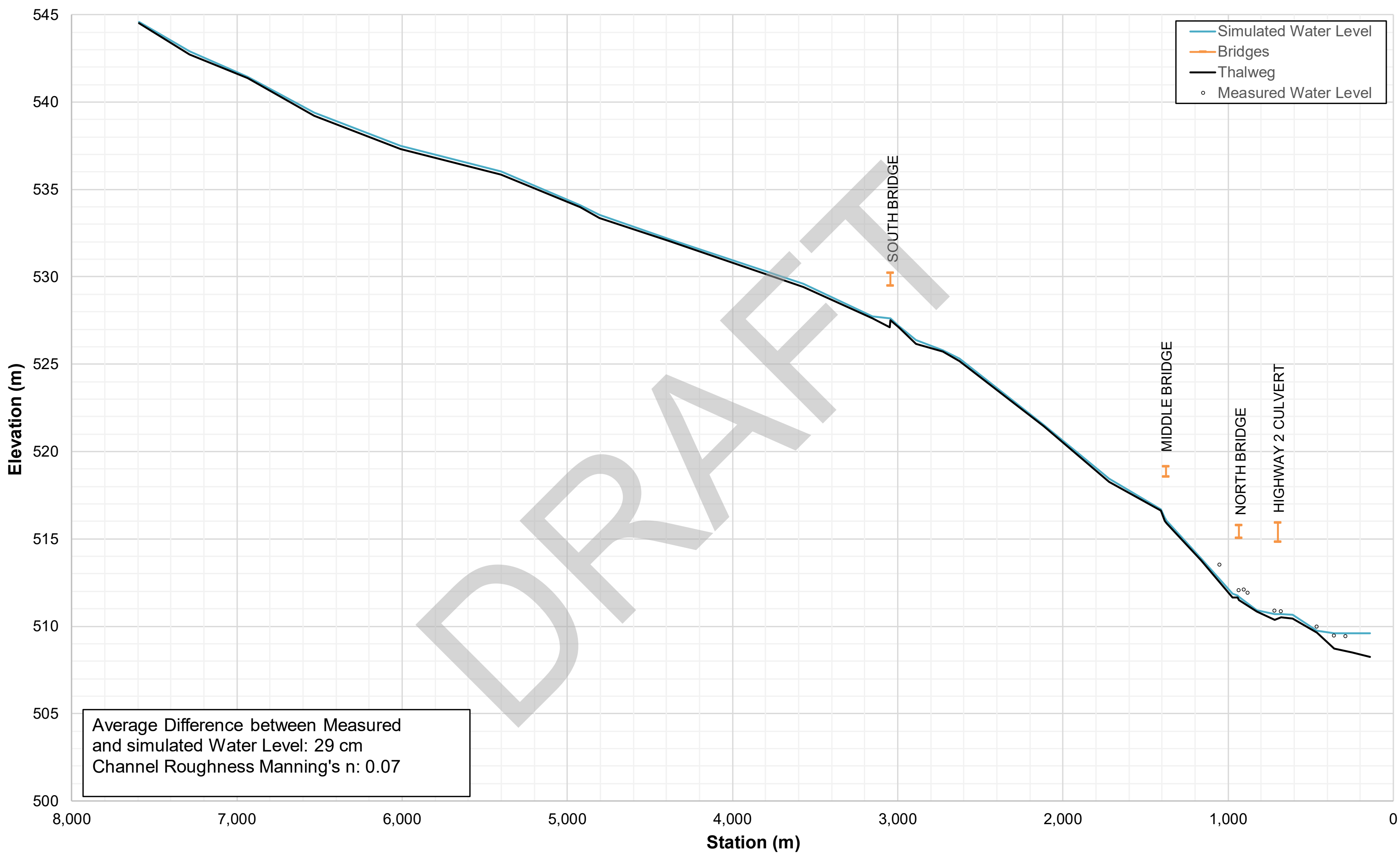

Figure 4-5: Comparison of Simulated Water Surface Profile to Surveyed Water Levels on Muskeg Creek for the Low Flow Condition

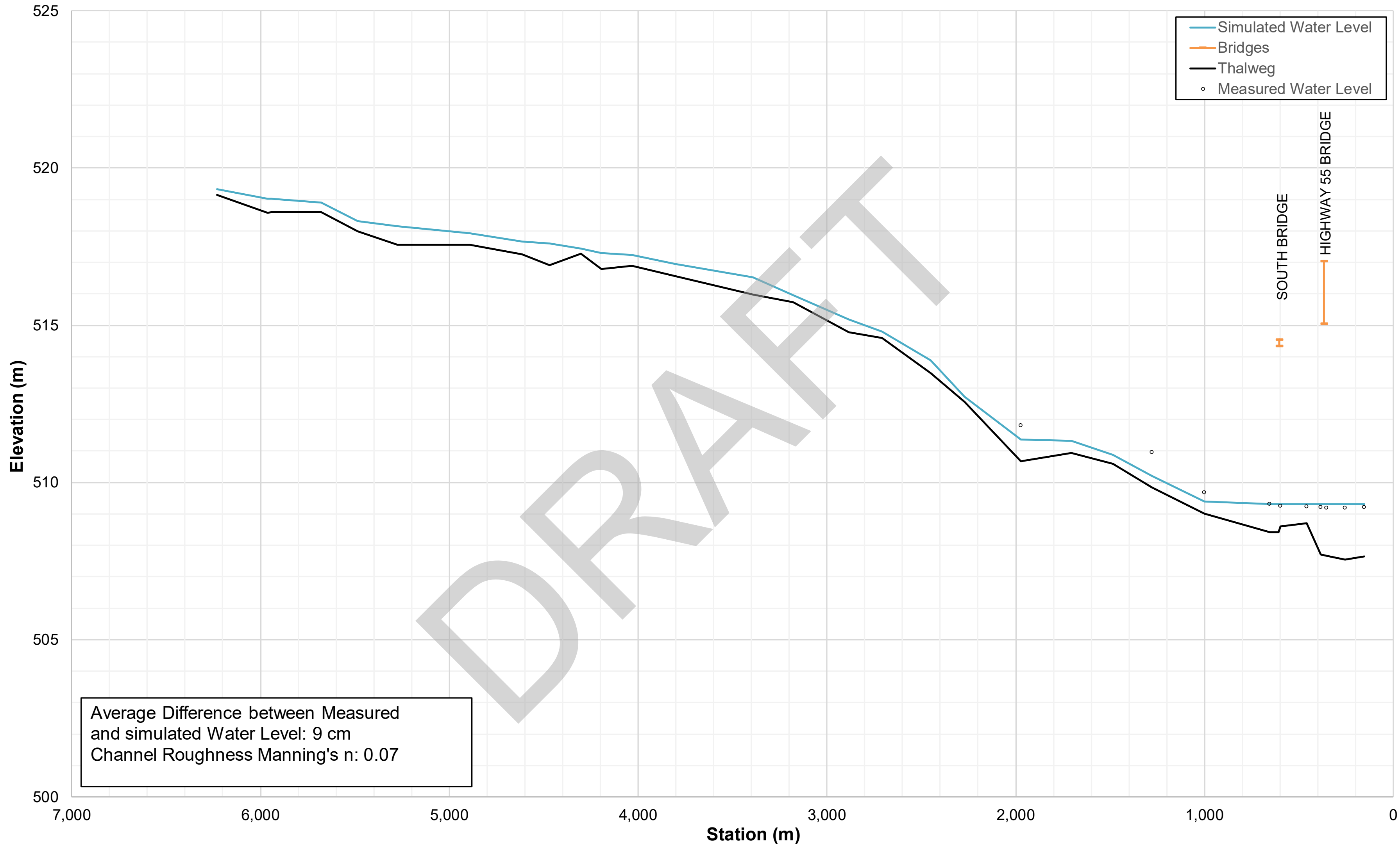

Figure 4-6: Comparison of Simulated Water Surface Profile to Surveyed Water Levels on the Tawatinaw River for the Low Flow Condition

4.4.5.3 High Flow Calibration

The HEC-RAS model for the Athabasca River study reach was calibrated based on the 1980 and 1990 HWMs. The estimated flood peak discharges on the Athabasca River were 4,190 m^3/s on June 8, 1980; 2,340 m^3/s on July 11, 1990; and 2,790 m^3/s on July 10, 1990. The 1990 and 1980 floods on the Athabasca River at the Town of Athabasca had estimated return periods of approximately 35 years and more than 100 years, respectively.

The 1980 HWMs were first used to closely match the simulated water levels by adjusting the initial values of the channel Manning's *n* and bridge contraction/expansion coefficients, where necessary. The 1990 HWMs were subsequently used to confirm the adjusted values of the channel Manning's *n* and the bridge contraction and expansion coefficients. The model calibration was achieved by adjusting the model parameter values, such that the simulated water levels were in good agreement with the 1980 and 1990 HWMs. Floodplain roughness values were found to have small effects on the model calibration.

Figure 4-7 compares the simulated water surface profile to the surveyed HWMs for the 1980 flood event along the Athabasca River. Based on the calibrated channel Manning's *n* value of 0.026, the average difference between the simulated and surveyed HWMs is -1 cm, and the differences range from -10 cm to +7 cm.

Figure 4-8 and Figure 4-9 compare the simulated water surface profile based on the calibrated channel Manning's *n* values of 0.026 to the surveyed HWM data on July 10 (Figure 4-8) and July 11 (Figure 4-9), 1990, along the Athabasca River. For July 10, 1990, the average difference between the simulated and measured water levels is -1 cm and the differences range from -15 cm to +13 cm. For July 11, 1990, the average difference between the simulated and measured water levels is -4 cm and the differences range from -7 cm to -1 cm.

Table E-1 to Table E-3 in Appendix E summarize the differences between the simulated and surveyed HWMs for the 1980 and 1990 flood events. The calibrated channel Manning's *n* value for the high flow conditions is 0.026, which is within the typical range of roughness values for similar rivers with gravel, sand, silt and clay bed materials under high flow conditions (Chow 1959).

There is no historic HWM data available along the study reaches of Muskeg Creek and the Tawatinaw River for model calibration. Therefore, Manning's *n* value of 0.050 for the Muskeg Creek and Tawatinaw River channels was selected for flood flow simulations based on published range of values for similar streams (e.g., HEC-RAS Hydraulic Reference) as well as Golder's modelling experience and professional judgement.

4.4.5.4 Validation Using WSC Gauge Data and Rating Curve

The available flow data and rating curve at WSC Station 07BE001 (Athabasca River at Athabasca) were used to validate the model calibration and to quantify the variability of the main channel roughness over a range of flows. The simulated water levels at the station for the various discharges are compared with the recorded water levels in Figure 4-10.

The comparison shows that the simulated and measured water levels at the station are comparable particularly during high flows, and the simulated water levels are slightly higher than the recorded water levels for the river flow of 2,500 m^3/s and higher. The simulated water levels were obtained based on a constant Manning's *n* value of 0.026. This comparison validates applicability of the calibrated Manning's *n* value for a full range of river flow conditions, particularly during floods.

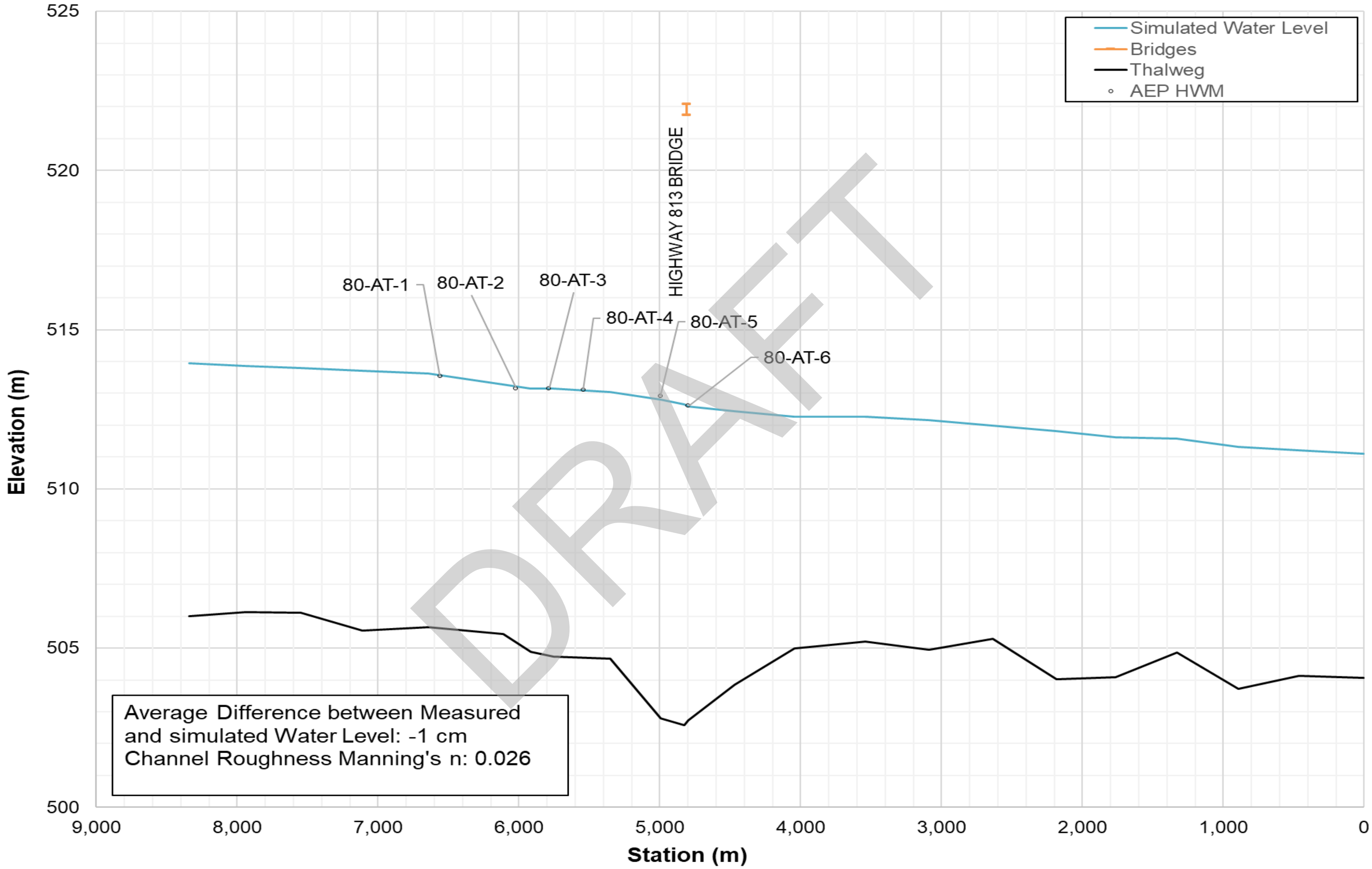

Figure 4-7: Comparison of Simulated Athabasca River Water Surface Profile and Surveyed Highwater Marks for the 1980 Flood

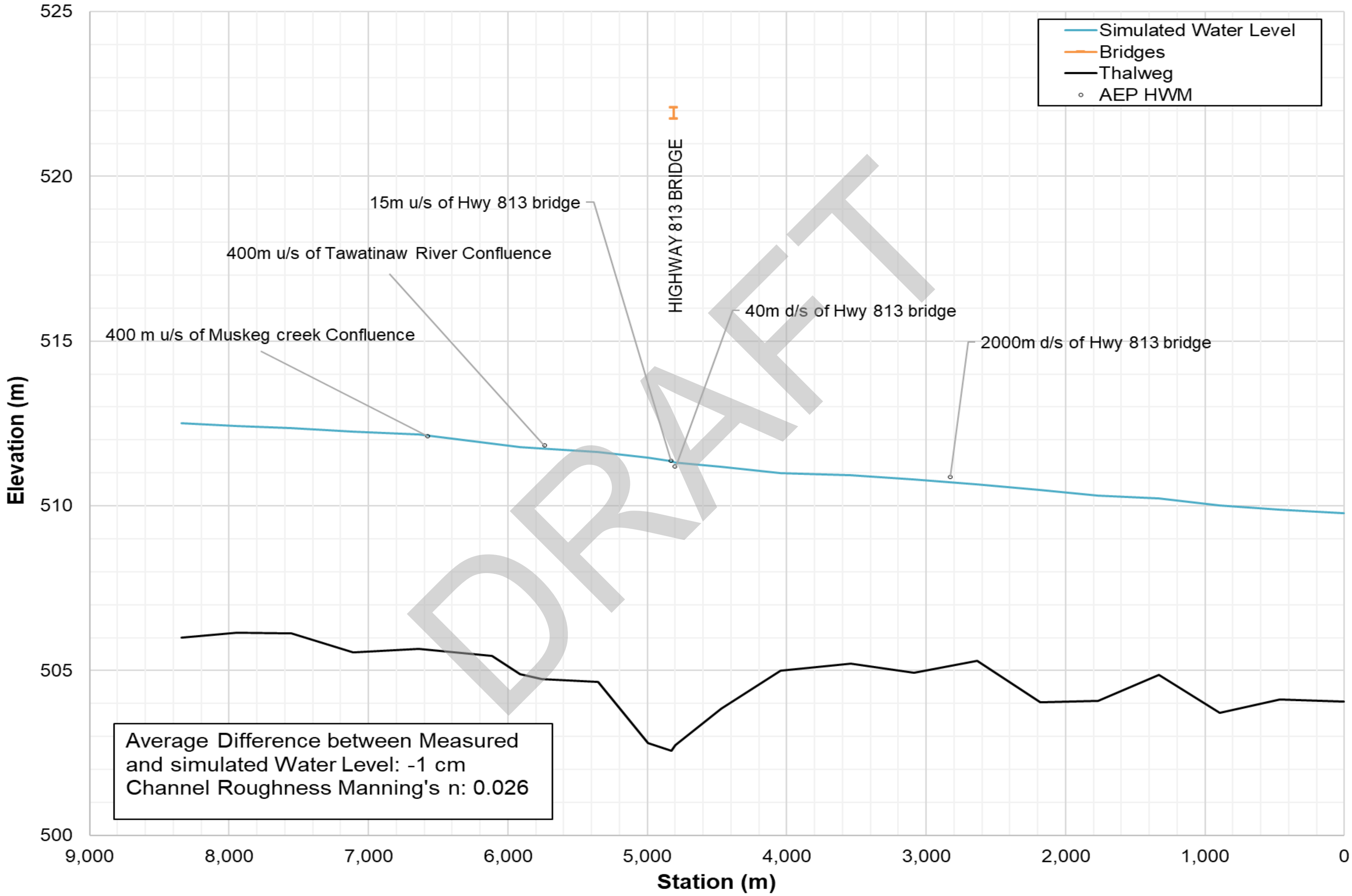

Figure 4-8: Comparison of Simulated Athabasca River Water Surface Profile and Surveyed Highwater Marks for July 10, 1990 Flood

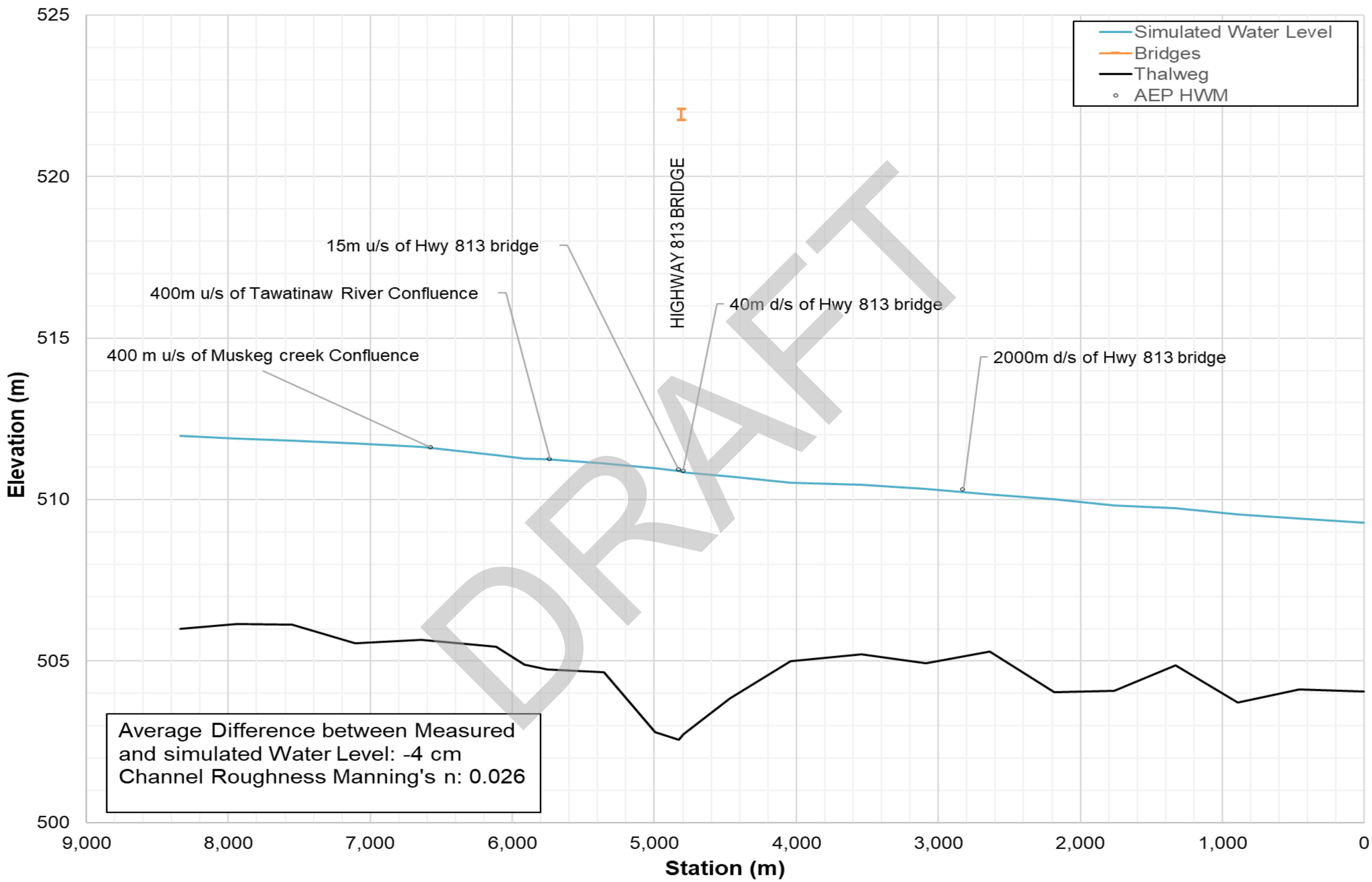

Figure 4-9: Comparison of Simulated Athabasca River Water Surface Profile and Surveyed Highwater Marks for July 11, 1990 Flood

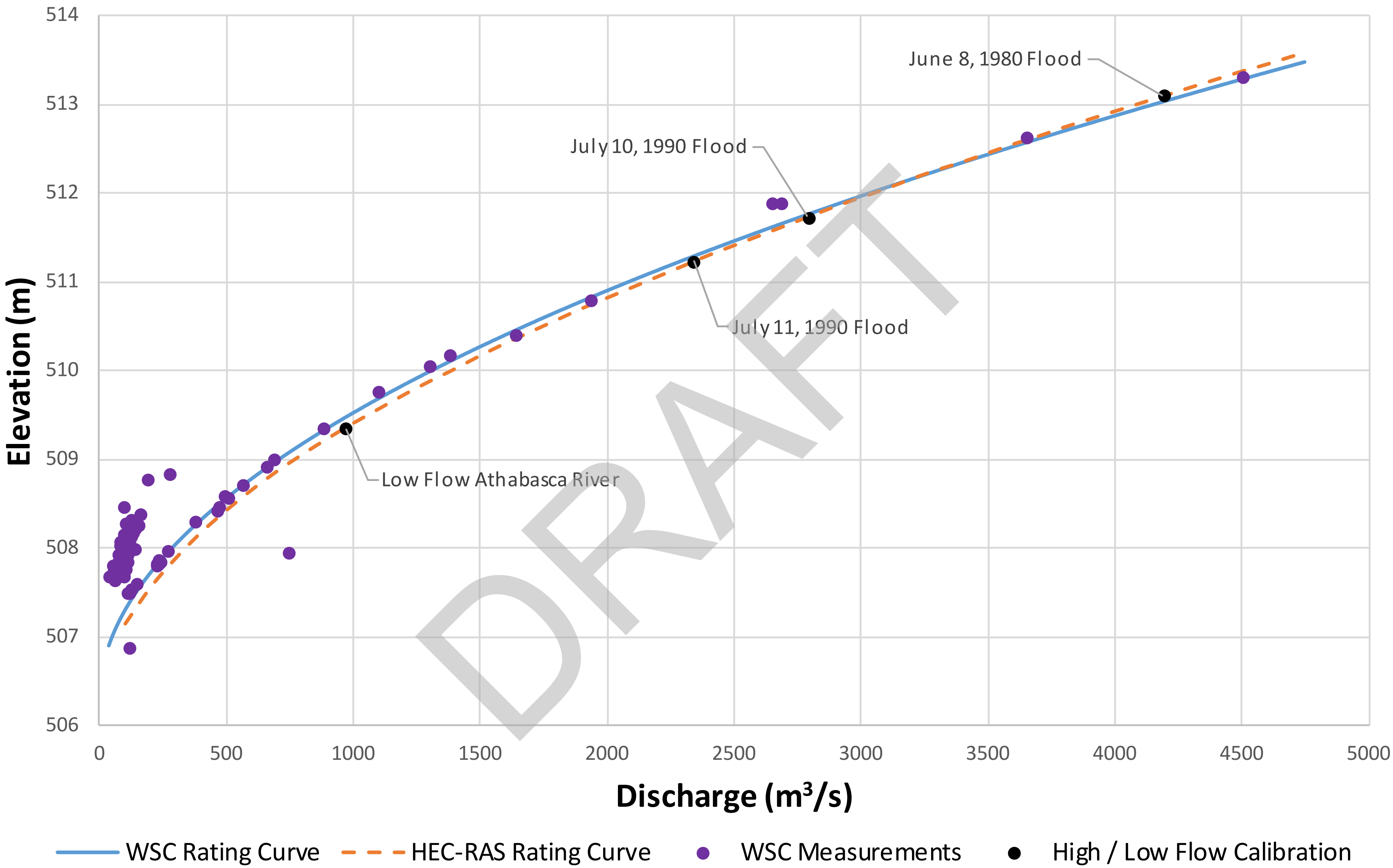

Figure 4-10: Model Validation Results Based on WSC Station 07BE001 (Athabasca River at Athabasca) Rating Curve

4.4.5.5 Summary of Calibration Results

The HEC-RAS model for the Athabasca River study reach was calibrated and validated. The results are summarized below:

- The high flow calibration results show that the simulated water levels compare well to the available HWMs on the Athabasca River. The channel Manning's *n* value, as well as contraction and expansion loss coefficients at bridges and other locations, were calibrated based on the 1980 and 1990 flood HWMs.
- The validation results based on the stage-flow rating curve for the WSC gauging station on the Athabasca River show that the simulated water levels generally compare well with the measured water levels.
- A constant Manning's *n* value of 0.026 for the Athabasca River main channel can be reliably used for simulating flood flows. The calibrated channel Manning's *n* value is within the typical range of roughness values for similar rivers (Chow 1959).

No high flow data is available for calibrating the HEC-RAS model for Muskeg Creek and the Tawatinaw River study reaches. Therefore, a Manning's n value of 0.050 for the Muskeg Creek and Tawatinaw River channels was selected as a reasonable value for flood flow simulations.

4.4.6 Model Parameters and Options

4.4.6.1 Manning's Roughness Coefficient

4.4.6.1.1 Channel Roughness

Channel Roughness

A constant Manning's *n* value of 0.026 was selected for the Athabasca River channel, and a constant Manning's *n* value of 0.050 for the Muskeg Creek and Tawatinaw River channels. The selections were based on the model calibration and validation results (see Section 4.4.5), literature values, and Golder's modelling experience and professional judgement.

The selected Manning's n values are in the reasonable range in comparison to typical values of comparable streams (Chow 1959).

Overbank Roughness

Table 4-8 presents the selected overbank Manning's *n* values based on various land uses on the floodplains.

Table 4-8: Manning's *n* Values for Various Land Uses on the Floodplains

Land Use	Initial Manning's n Value	Selected Manning's n Value
Urban Mixture (Residential)	0.080	0.080
Urban Mixture (Commercial)	0.060	0.060
Streets	0.030	0.030
Grassland and Open Space	0.050	0.050
Golf Course	0.050	0.050
Ponds	0.030	0.030
Dense Vegetation	0.150	0.150

4.4.6.2 Expansion and Contraction Coefficients

During the calibration process, some of the estimated values of the contraction/expansion coefficients were adjusted where appropriate. The selected contraction coefficient values range from 0.1 to 0.3, and the selected expansion coefficient values from 0.3 to 0.5.

4.4.6.3 Obstructions and Ineffective Flow Areas

Considerable efforts were spent to identify and define the ineffective flow areas, so that one geometry file can be used to simulate various floods with return periods of 2 to 1,000 years. The ineffective flow areas were defined in considerations of local topography, structures, and flow connection between adjacent cross sections.

The following three types of ineffective flow areas were implemented in the model setup:

- Topographical low areas such as ponds: permanent ineffective flow areas were specified to block off low-lying areas that do not effectively convey flows.
- Topographical low areas that can be activated: non-permanent ineffective flow areas are specified to block off low-lying areas that can become active after the water level is above a certain elevation.
- Bridge decks and embankments: ineffective flow areas are specified at the cross sections upstream and downstream of the bridges to block off the flow areas if the water level is lower than the top-of-embankment elevation.

4.4.7 Open Water Flood Frequency Profiles

4.4.7.1 Production Model

The HEC-RAS production model was developed based on the calibrated and estimated Manning's *n* values. The flood peak flows used in the HEC-RAS production model were estimated based on the hydrology assessment presented in Section 3. Surface water profiles were simulated for the 2-, 5-, 10-, 20-, 35 , 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events using the production model.

4.4.7.2 Flow Change Locations

Four flow change locations along the study reaches were selected and included in the production model. They include two locations on the Athabasca River, one on Muskeg Creek, and one on the Tawatinaw River. The number of flow zones proposed along the study reaches matches with the number of assigned flow nodes identified in Section 3.

4.4.7.3 Flood Peak Flows

The estimates of flood peak flows at the four flow change locations included in the HEC-RAS production model are summarized in Table 4-9.

Table 4-9: Summary of Flood Peak Flows at the Flow Change Locations Used in the HEC-RAS Production Model

Location	HEC-RAS Station	Discharges of Various Return Periods (m³/s)												
		2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1,000-year
Athabasca River Upstream Boundary	8,340	1,880	2,670	3,280	3,940	4,530	4,920	5,400	5,760	6,700	7,540	8,120	8,820	9,340
Athabasca River below Tawatinaw River Confluence	5,352	1,886	2,685	3,305	3,976	4,577	4,975	5,465	5,833	6,794	7,654	8,249	8,966	9,500
Muskeg Creek Upstream Boundary	7,594	2.46	6.56	10.8	16.3	22.0	26.4	32.1	36.7	50.2	64.0	74.3	87.9	98.8
Tawatinaw River Upstream Boundary	6,230	5.50	15.1	24.5	36.0	47.1	55.1	65.2	73.1	94.3	114	129	146	160

4.4.7.4 Model Boundary Conditions

The boundary conditions of the HEC-RAS production model are listed below:

- The discharges specified at the four locations as listed Table 4-9.
- Normal flow condition with an energy slope of 0.026% specified at the model downstream boundary on the Athabasca River.

4.4.7.5 Open Water Flood Frequency Profiles

Athabasca River

The simulated open water flood profiles along the study reach of the Athabasca River are presented in Figure F-1 in Appendix F. The simulated open water flood water levels at individual cross sections along the study reach of the Athabasca River are listed in Table F-1 in Appendix F.

Muskeg Creek

The simulated open water flood profiles along the study reach of Muskeg Creek are presented in Figure F-2 in Appendix F. The simulated open water flood water levels at individual cross sections along the study reach of Muskeg Creek are listed in Table F-2 in Appendix F.

Tawatinaw River

The simulated open water flood profiles along the study reach of the Tawatinaw River are presented in Figure F-3 in Appendix F. The simulated open water flood water levels at individual cross sections along the study reach of the Tawatinaw River are listed in Table F-3 in Appendix F.

4.4.8 Model Sensitivity

A model sensitivity analysis was conducted to evaluate the effects of changing model roughness values and downstream boundary conditions on the simulated water levels. The discharges used for the model sensitivity analysis were the 100-year flood peak flows. The results of the sensitivity analysis were used to quantify the level of uncertainty associated with the simulated flood levels along the study reaches of the Athabasca River, Muskeg Creek, and the Tawatinaw River.

The analysis of sensitivity to Manning's *n* involves the following three sets of Manning's *n* values for the river/creek channels and floodplains and two set of downstream boundary conditions:

- First set corresponding to ±10% changes of the base channel Manning's *n* values only.
- Second set corresponding to ±10% changes of the base floodplain Manning's *n* values only.
- Third set corresponding to ±20% changes of the specified energy slope at the downstream boundary.

Figures G-1 to G-3 in Appendix G graphically present the differences between the simulated water levels for the 100-year flood along the study reach of the Athabasca River. The results of the sensitivity analysis indicate the following:

- The uncertainty in the simulated flood levels, on average, is within a range of ±0.43 m (with standard deviation of 0.02 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base channel Manning's *n* values only.
- The uncertainty in the simulated flood levels, on average, is within a range of ±0.00 m (with standard deviation of 0.00 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base floodplain Manning's *n* values only.

- The ±20% changes of the energy slope at the downstream boundary influence the simulated flood levels in in the entire reach.

Figures G-4 to G-6 in Appendix G graphically present the differences between the simulated water levels for the 100-year flood along the study reach of Muskeg Creek. The results of the sensitivity analysis indicate the following:

- The uncertainty in the simulated flood levels, on average, is within a range of ±0.18 m (with standard deviation of 0.14 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base channel Manning's *n* values only.
- The uncertainty in the simulated flood levels, on average, is within a range of 0.00 m (with standard deviation of 0.01 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base floodplain Manning's *n* values only.
- The ±20% changes of the energy slope at the downstream boundary of Athabasca River influence the simulated flood levels along approximately 1.3 km reach of Muskeg Creek upstream of its confluence with the Athabasca River.

Figures G-7 to G-9 in Appendix G graphically present the differences between the simulated water levels for the 100-year flood along the study reach of the Tawatinaw River. The results of the sensitivity analysis indicate the following:

- The uncertainty in the simulated flood levels, on average, is within a range of ±0.21 m (with standard deviation of 0.13 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base channel Manning's *n* values only.
- The uncertainty in the simulated flood levels, on average, is within a range of ±0.01 m (with standard deviation of 0.01 m) along the entire study reach, based on the differences in the simulated flood levels for the ±10% changes to the base floodplain Manning's *n* values only.
- The ±20% changes of the energy slope at the downstream boundary of Athabasca River influence the simulated flood levels along approximately 2.4 km reach of the Tawatinaw River upstream of its confluence with the Athabasca River.

5.0 ICE JAM MODELLING

5.1 Overview

Ice jam modelling of the Athabasca River at Athabasca involved defining river breakup characteristics and deriving an ice jam rating curve at WSC Station 07BE001, based on year-to-year variations in discharge and ice conditions during breakup. A stage frequency analysis of both historical and systematic measurements of peak annual ice-related breakup water levels at the WSC station was undertaken to provide estimates of the flood severities at the gauge.

Appendix H (SG1 2020) of this report presents a comprehensive review of historical river ice data and observations with the objectives of:

- summarizing the measured ice-related annual peak water levels along the Athabasca River at Athabasca based on measurements at the WSC Station 07BE001 and any additional anecdotal or historical evidence;
- describing the breakup mechanisms and ice-related hydraulic characteristics of the study reach within the context of the current understanding of ice jam mechanics; and
- assessing the annual ice-related flood hazards using the USGS (1982) approach to carry out a frequency analysis of the annual peak ice-related water levels.

The following sections provide a summary of data and analysis presented in Appendix H (SG1 2020), a description of the gradually varied, ice jam related flow modelling that was undertaken to extrapolate frequency-specific water levels from the WSC gauge throughout the flood hazard domain, and flood water surface elevation estimates for ice jam floods with return periods of 50, 100 and 200 years.

While the focus of the simulations is to estimate the ice jam water levels throughout the flood hazard domain, results of sensitivity analyses are also presented to illustrate the effects that the adopted boundary conditions would have on the ice jam water level simulations within the flood hazard domain.

5.2 Available Data

5.2.1 Gauge Data and Rating Curves

A detailed review of ice-related water levels at Athabasca, based on the data reported for WSC Station 07BE001 (Athabasca River at Athabasca), is provided in Appendix H (SG1 2020). A graphical summary of the available ice-related water level data is shown in Figure 5-1. Peak water surface elevations during breakup have ranged from 506.9 m to 511.8 m (mean daily values) and from 508.0 m to 517.5 m (maximum instantaneous values).

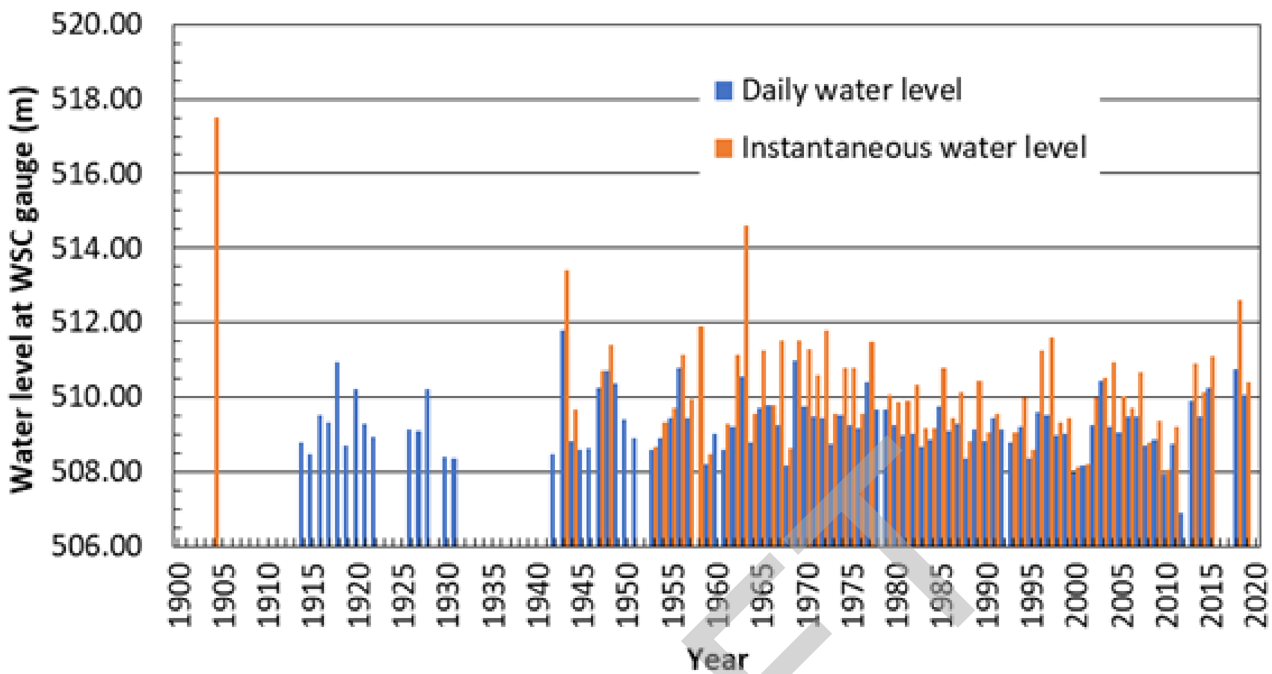

Figure 5-1: Summary of Peak Ice-Related Water Levels during Breakup Period at WSC Station 07BE001

The annual ice-related peak water levels presented in Figure 5-1 were correlated with the adopted discharge at breakup in each year to define a number of plausible ice-related rating curves, as presented in Figure 5-2. The upper-bound envelope curve is most important of those curves. This curve represents the equilibrium ice jam stage-discharge rating curve, which relates the water level within a fully develop jam to the discharge during its formation. The shape and position of the curve is determined on the basis of the local channel geometry and appropriately defined ice jam parameters. The test that determines if the curve is a valid representation of a equilibrium ice jam condition is if the theoretically defined curve reflects the shape of the upper bound of the data while using reasonable ice jam parameters.

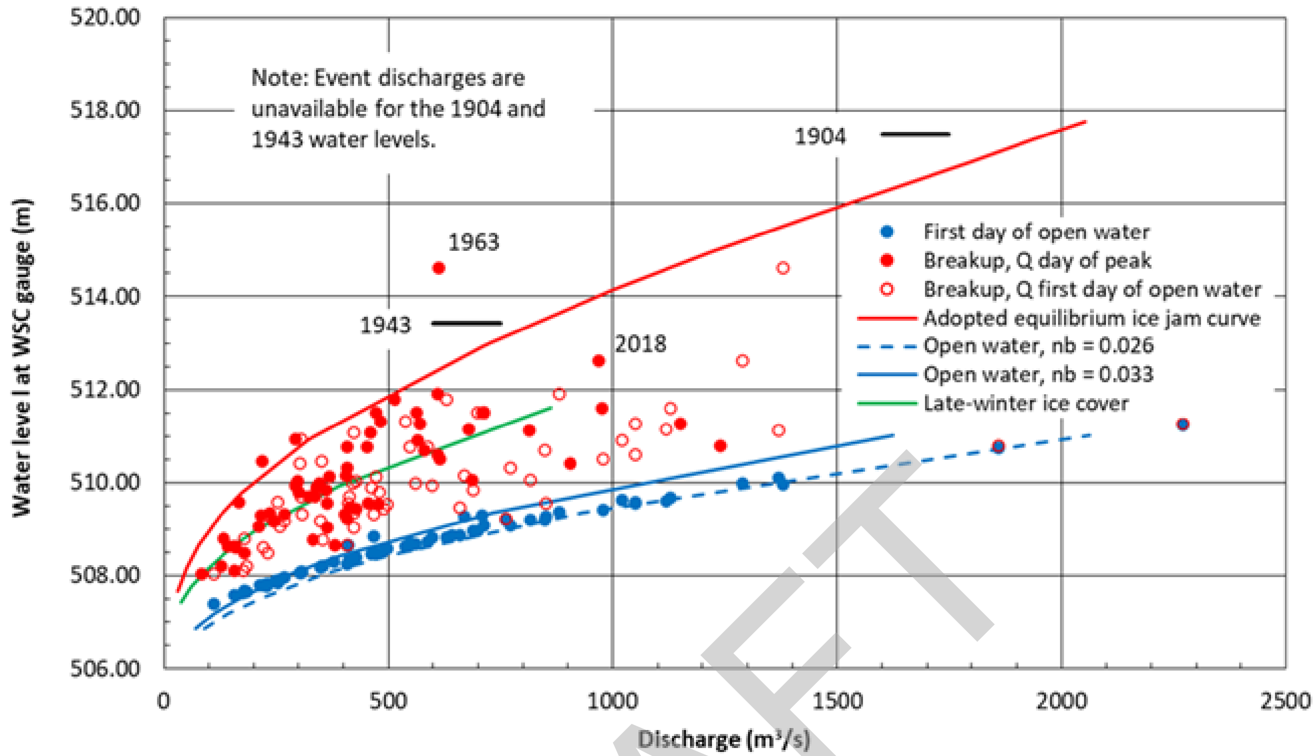

Figure 5-2: Derived Ice-Related Stage-Discharge Rating Curve, Compared to Open Water Rating Curve

5.2.2 Ice Observation Reports and Documentation

Incorporated into the water level records and assessment summarized in Section 5.2.1 are data from the ice jam flood of record in 1904, which predated the establishment of WSC Station 07BE001. The water level associated with the 1904 event was estimated by reconciling information derived from contemporaneous photographs and written accounts of the event with historical mapping and the current LiDAR-derived DEM. The detailed analysis of the 1904 event, including discussion of uncertainty, is presented in Appendix H (SG1 2020).

The derived frequency curve of annual peak water level, considering uncertainty in the 1904 event, is presented in Figure 5-2. The peak water level of the 1904 ice jam flood was estimated to be 517.5 ± 0.5 m, reflecting a return period of between 500 and 1,000 years.

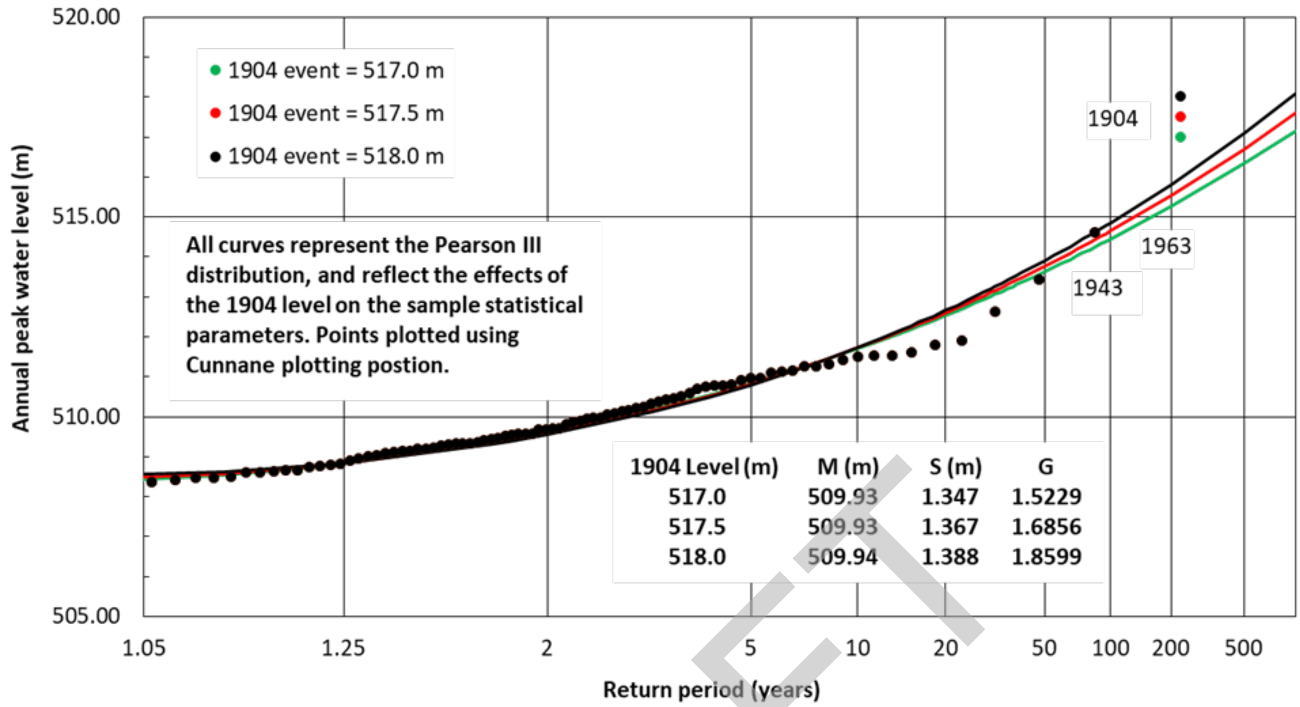

Figure 5-3: Frequency Curves for a Range of Possible 1904 Breakup Levels

In Figure 5-3, the red curve represents the adopted ice jam flood frequency curve, while the black and green curves represent those derived based on higher and lower potential 1904 peak instantaneous breakup levels. In Figure 5-3, M represents the sample average, S the sample standard deviation, and G the sample skew coefficient.

5.2.3 Ice Jam Highwater Marks

No systematic, ice-related highwater marks, or ice jam profiles have been measured on the Athabasca River at Athabasca.

5.3 Ice Jam Model Setup

5.3.1 Computer Program

The Hydrologic Engineering Center River Analysis System (HEC-RAS) computer modelling program, developed by the U.S. Army Corps of Engineers, was selected to carry out ice jam hydraulic modelling, including extrapolation of flood water levels from those derived at WSC Station 07BE001.

With respect to ice modelling, HEC-RAS allows for the hydraulic simulation of ice-covered channels with either a solid floating ice cover or a gradually varying ice accumulation [i.e. the so-called wide-river ice jam, where the ice jam thickness is determined by solving the ice jam force balance equation (USACE 2016b)]. In both cases, a range of parameters that describe the ice characteristics are required to carry out meaningful hydraulic simulations.

5.3.2 Ice Jam Frequency Analysis

The detailed ice jam frequency analysis is presented in Appendix H (SG1 2020). The resulting ice-related water level frequencies at WSC Station 07BE001, as calculated by procedures outlined in Bulletin 17B (USGS 1982), is presented in Table 5-1.

Table 5-1: Ice Jam Frequency Analysis for Athabasca River at Athabasca (WSC Station 07BE001)

Return Period (years)	Annual Probability being Equaled or Exceeded (%)	Water Level at WSC Station 07BE001 (m)
50	2	513.76
100	1	514.64
200	0.5	515.53

5.3.3 Methodology

An open water HEC-RAS model for the Athabasca River at Athabasca was developed as part of this study using survey data collected by Golder in June 2019 and LiDAR data acquired by Airborne Imaging Inc. on October 18, 2018 (on behalf of AEP). The model was calibrated for open water low flow conditions and is appropriate for use on the ice jam modelling task. The calibrated low flow Manning's roughness was 0.033 for the Athabasca River main channel bed (Appendix H, SG1 2020).

The following simplifications were made to the open water model geometry file to create an ice jam model geometry file that is appropriate for simulating ice-related water levels:

- Removal of the bridge: the Athabasca River bridge at Secondary Highway 813 has four instream piers, each with a width of 1.35 m, for a total width of 5.4 m. This compares to a channel width of 200 m at the bridge, meaning the piers occupy approximately 2.7% of the channel width. Each concrete bridge pier features a sloping, steel-jacketed wedge-shaped nose, intended to cause impinging ice floes to fail in bending (uplift by sloped nose) or crushing (concentration of force by narrow wedge). Based on this, it is expected that the bridge will have a negligible effect on the local characteristics of an ice jam.
- Removal of the Muskeg Creek and Tawatinaw River tributaries: because their inflows have minimal to no effects during the severe ice jam events that occur in the flood hazard reach.
- Because of the flows and the thickness of the ice accumulations during severe breakup events, most of the flow occurs within the main channel. Therefore, floodplain roughnesses were simplified. In addition, channel bank locations at each cross section were selected to represent the locations where the ice accumulation within the jam would most likely ground against or on the bank, and a shear line would develop. In-river flows would be affected by the roughness of the ice and the bed, and floodplain flows, if any, would be affected only by the roughness of the floodplain. Table 5-2 summarizes the bank locations for both the open water and ice jam models. Ice jam bank locations within the flood hazard model domain were, on average, extended 30 m outwards compared to the open water left bank locations and 43 m outwards compared to the open water right bank locations.
- Cross section extrapolation: Additional cross sections were added by extrapolation beyond the upstream boundary of the flood hazard domain to ensure that equilibrium ice thicknesses were being simulated throughout the domain. The geometry of the most upstream cross section (XS1 at river station 8339.925) was extrapolated upstream at intervals of 200 m from river station 8540 and to river station 12540, at a bed slope of 0.00037 m/m. The extrapolated reach including these cross sections is shown in Appendix J. As a sensitivity test, cross sections were also added at the downstream end of the survey reach using the same extrapolation technique (refer to Section 5.6.3). The objective was to determine if the surveyed cross sections extended far enough downstream to provide sufficient distance for the simulations to transition from

the solid ice at the jam toe to an equilibrium jam thickness at the downstream end of the flood hazard domain. The results indicated that the survey domain did extend far enough downstream of the flood hazard domain to ensure equilibrium conditions throughout the flood hazard domain.

Table 5-2: HEC-RAS Model Bank Locations for the Open Water and Ice Jam Models

River Station	Cross Section ID	Open Water Bank Location (m)		Ice Jam Bank Location (m)	
		Left Bank	Right Bank	Left Bank	Right Bank
8340	1	117.8	451.5	96.4	504.6
7941	2	169.9	511.2	129.9	547.7
7550	3	161.6	505.1	102.4	549.2
7109	4	133.4	484.3	103.5	510.8
6640	5	156.0	518.8	92.0	530.9
6112	6	51.8	313.5	26.7	313.5
5915	7	58.8	292.1	33.3	313.1
5756	8	49.2	298.6	35.2	452.9
5352	9	158.8	446.7	141.0	544.1
4994	10	207.4	431.2	187.9	448.2
4829	11	282.0	483.3	260.4	532.3
4796	12	262.9	457.2	239.9	506.3
4468	13	250.5	488.7	224.6	512.6
4043	14	347.2	607.0	317.4	658.3
3542	15	610.4	1013.3	576.9	1074.3
3087	16	579.8	980.5	530.7	997.0
2636	17	369.8	721.0	312.6	735.1
2182	18	86.3	399.9	75.6	442.8
1764	19	95.1	410.4	77.5	458.2
1329	20	144.5	545.9	130.5	597.6
892	21	204.9	504.9	188.9	541.4
461	22	107.3	433.7	35.5	577.2
6	23	160.2	495.6	120.3	634.6

- Cross section Interpolation: interpolated cross sections were not used, and the ice jam hydraulic model was based on the same cross sections as the open water hydraulic model. That cross section spacing was selected to accommodate the ice jam simulations, as inferred from the observations of Beltaos et al. (2011), which noted that use of an average cross section spacing of less than 1.0 times the channel width resulted in implausible ice jam thickness predictions, and reduced accuracy of water level predictions. The cross section spacing for the Athabasca River model averaged 1.2 times the channel width, with a range from 0.5 to 1.8 times the channel width[2], based on a mean cross section spacing of 380 m and a mean channel width of 306 m. This is discussed further in Section 5.6.4.

- Limiting Velocity: The default value for the limiting (maximum) velocity at the toe of the ice jam is 1.5 m/s. This value was increased to 2.0 m/s. Beltaos and Tang (2013) justified the use of virtually any limiting velocity "so long as it is understood that the HEC-RAS solution at and very near the toe may be unrealistic" and "an arbitrarily high V_{max} value is definitely advantageous… [if] it produces acceptable results over the bulk of the jam profile and does resolve the snags that arise from the default value". Because the toe of the ice jam in the Athabasca River model is located over a kilometre downstream of the flood hazard domain, the model is insensitive to use of a value greater than the default value.

5.4 Ice Jam Model Calibration

The HEC-RAS ice jam model calibration is partially described in Appendix II of Appendix H (SG1 2020). The model assumes no ice cover in the floodplain, thereby confining the ice accumulation to the main channel, as per local observations. This assumption allows for open water flow in the floodplain if water levels exceed bankfull, thereby providing a slight amount floodplain conveyance. Note that the ice model was not calibrated against any historical ice jam events, due to lack of data, but it reconciles well with the uniform flow analysis reported in Appendix H (SG1 2020).

Figure 5 from Appendix H (SG1 2020) is reproduced in Figure 5-4, and shows winter discharge and water level measurements at WSC Station 07BE001, along with the open water rating curve for the station and a rating curve generated using HEC-RAS to model flow under a solid ice cover. The ice cover rating curve considers a late winter submerged ice thickness of 0.57 m and reflects a composite Manning roughness of 0.039, based on a low flow channel bed roughness of 0.033 and a solid ice cover roughness of 0.045. The two outliers in March, which fall below the bulk of the March data, were measured at a location not typically used during the winter measurement program.

The ice jam model was set up by populating the HEC-RAS model using ice-specific channel geometry (i.e., modifying open water model channel bank stations as discussed above) and adopting ice-specific model parameters as discussed in Table 5-4.

Ice-related water level frequencies, as calculated by procedures outlined in Bulletin 17B (USGS 1982), were presented in Table 5-1. The uniform flow analysis (Appendix H, SG1 2020) was used to calculate the corresponding discharges required to achieve those levels. Discharges were adjusted to force target water levels at WSC Station 07BE001 to match those derived for various return periods. The resulting discharge was compared to that from the uniform flow analysis to validate the model. A summary of the ice-related water levels at the WSC gauge (model XS7 at river station 5914.59) for a range of return periods is provided in Table 5-3.

[2] The 0.5 channel width spacing occurred at WSC Station 07BE001, where the cross section was required for calibration and validation purposes. The next smallest spacing used in the model domain was 0.9 channel widths.

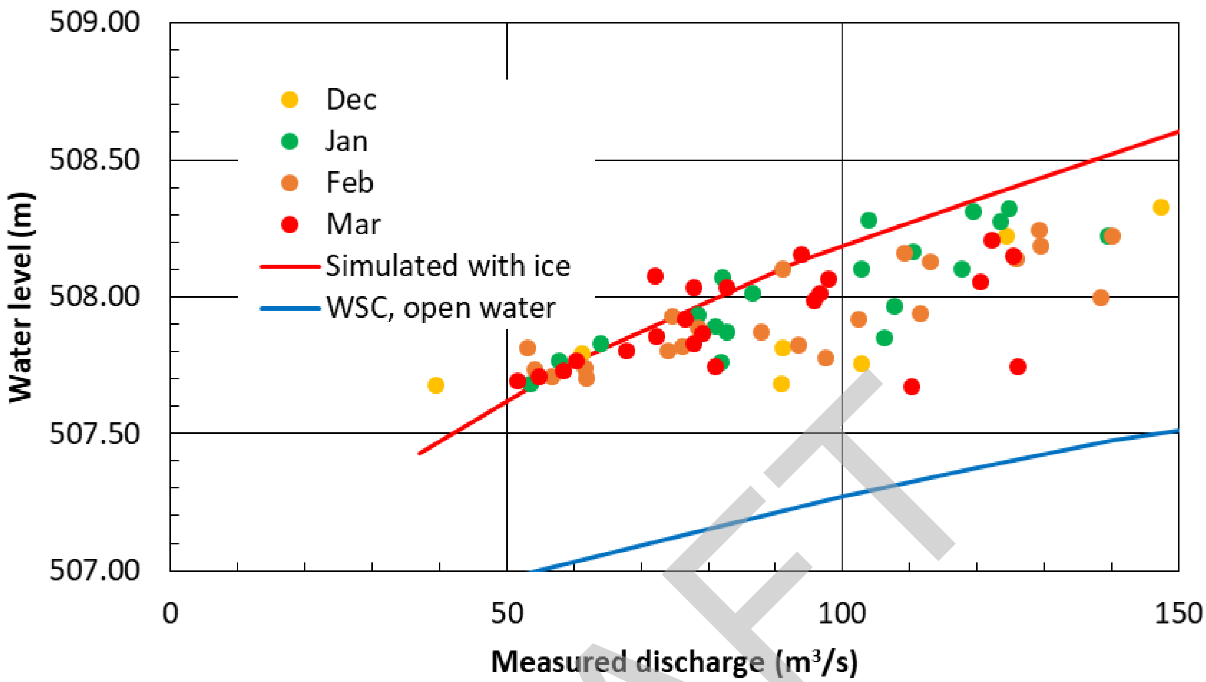

Figure 5-4: Winter Discharge Measurements at WSC Station 07BE001 with Simulated Solid Ice Rating Curve

Table 5-3: Ice-Related Water Levels at the WSC Station 07BE001 for Salient Return Periods

Return Period (years)	Water Level from Frequency Analysis (m)	Corresponding Discharge from Uniform Flow Analysis (m^3/s)	HEC-RAS Ice Jam Model	
			Adopted Discharge (m^3/s)	Simulated Water Level (m)
50	513.76	898	915	513.76
100	514.64	1,120	1,150	514.64
200	515.53	1,370	1,410	515.53

Table 5-3 summarizes the following information:

- the expected (target) water level for the various return periods, based on the results of the frequency analysis described in Appendix H (SG1 2020);
- the corresponding discharge required to produce that water level, based on the reach-average, uniform flow hydraulic analysis described in Appendix H (SG1 2020);
- the discharge in the HEC-RAS analysis required to match the target water levels at the WSC gauge; and
- the HEC-RAS simulated water level at the gauge for that discharge.

The two discharges reconcile very well, suggesting that the uniform flow analysis is credible, and the HEC-RAS simulated water levels are all within a few centimetres of the target water levels.

HEC-RAS solves for the local ice thickness and flow depth by solving the ice jam force balance equation and the energy equation. The two equations are solved when the calculated ice jam thickness converges. The maximum number of ice jam iterations is defined as the maximum number of times for successive solutions of the two equations (USACE 2016a). The default value of the maximum number of ice jam iterations is 50, which is 2.5 times the maximum number of iterations. The ice jam calculations are mostly converged within the default ice jam iterations and show no errors during model simulation. Increasing the maximum number of ice jam iterations show little to no change in the simulated ice jam water levels.

Table 5-4 summarizes the adopted HEC-RAS ice jam model parameter values.

Table 5-4: HEC-RAS Model Parameters for Ice Jam Modelling

Model Parameters	Units	Value
Channel Bed Roughness Under the Ice Cover (determined from late-winter WSC discharge measurements)	-	0.033 (Appendix H, SG1 2020)
Ice Underside Roughness (1)	-	0.060 (ice jam); 0.045 (solid ice cover) (Appendix H, SG1 2020)
Channel Ice Thickness	m	0.62 (Appendix H, SG1 2020)
Ice Cover Specific Gravity	-	0.916 (default)
Internal Friction Angle of Ice Jam	°	45 (default)
Ice Jam Porosity	Φ	0.40 (default)
Coefficient K1, Longitudinal to Lateral stress in Jam	-	0.33 (default)
Jam Strength Coefficient (defined as a function of the internal friction angle, ice jam porosity, and K1.)	μ	1.2
Limiting Velocity under Jam	m/s	2.0
Ice Jam in Main Channel	-	Yes
Ice Jam or Solid Ice on Floodplain	-	No

Note: Adopted ice roughnesses were identical throughout the flood hazard domain, based on consistency of channel geometry. Adopted Manning's roughness was n = 0.033 for the main channel and overbank areas and n = 0.060 for ice underside roughness. This resulted in a composite roughness of n = 0.047, as calculated using the Belokon-Sabaneev equation.

5.5 Ice Jam Flood Frequency Profiles

For the HEC-RAS gradually varied flow analysis, the ice jam water levels throughout the hazard domain for each return period were similarly determined, based on the adopted discharge in the HEC-RAS ice model.

Ice jam profiles for 50-year (Figure 5-5), 100-year (Figure 5-6), and 200-year (Figure 5-7) return periods were simulated using the HEC-RAS model. The profiles are shown relative to open water bank lines, to best represent the degree of overbank flooding during these flood events. A combined plot of ice jam profiles for all return periods is provided in Figure 5-8. Tabulated model outputs are provided in Appendix I.

The adopted discharges presented in Table 5-3 indicate that the adopted 50-, 100-, and 200-year ice jam flood discharges fall below the 2-year discharge for the open water event (1,880 m^3/s). Figure 5-5 through Figure 5-7 show that for all ice jam flood scenarios, the ice jam gradually thickens in the downstream direction through the flood hazard domain as follows:

- For the 50-year event, the ice jam thickness varies from 1.90 m to 3.14 m (average 2.26 m).
- For the 100-year event, the ice jam thickness varies from 2.00 m to 3.56 m (average 2.47 m).
- For the 200-year event, the ice jam thickness varies from 2.14 m to 4.07 m (average 2.75 m).

Ice jam water levels generally increase in elevation by about 1 m from the 50- to 100-year event and additional 1 m from the 100- to 200-year event. For the 100-year event, the water surface elevation is approximately 4 m above the right downstream bank elevation at WSC Station 07BE001, with the bottom of ice approximately 2 m above the right downstream bank elevation. This indicates that the River Front Park area and adjacent areas north of Highway 2 and Highway 55 would be at risk of flooding.

5.6 Ice Jam Model Sensitivity

5.6.1 Scenarios

Three model sensitivity scenarios were evaluated to:

- assess the implications of assuming open water flow in the floodplain;
- ensure that the ice jam toe location was far enough downstream of the flood hazard reach to allow equilibrium ice conditions; and
- examine the effect of including interpolated cross sections in the ice jam hydraulic model.

The results of the model sensitivity analysis are discussed in the following sections.

5.6.2 No Flow Conveyance in the Floodplain

The ice jam model was set up so that neither solid ice nor ice floes were allowed to accumulate in the floodplain. If ice jam levels exceeded bankfull, water would be allowed to flow along the floodplain as open water flow (i.e. Base Case). However, because of the short lengths and built-up characteristics of parts of the floodplain, water might pond on the floodplain. Therefore, allowing open water flow might not provide a conservative assessment of the resulting flood levels. To evaluate the effects of not allowing flows along the floodplain (i.e., Scenario 1), flow conveyance in the floodplain was restricted by making the floodplain ineffective either by adding levees or designating the floodplain as being ineffective.

Water level comparisons between the two conditions (i.e., flow and no flow within the floodplain) are summarized in Table 5-5, with a detailed figure showing water level profiles provided in Appendix J, Figure J-1. Because of the small differences in the simulated water levels between the two conditions, it can be concluded that any flow conveyance in the floodplain has minimal effects on the resulting ice jam water levels within the ice jam hazard domain.

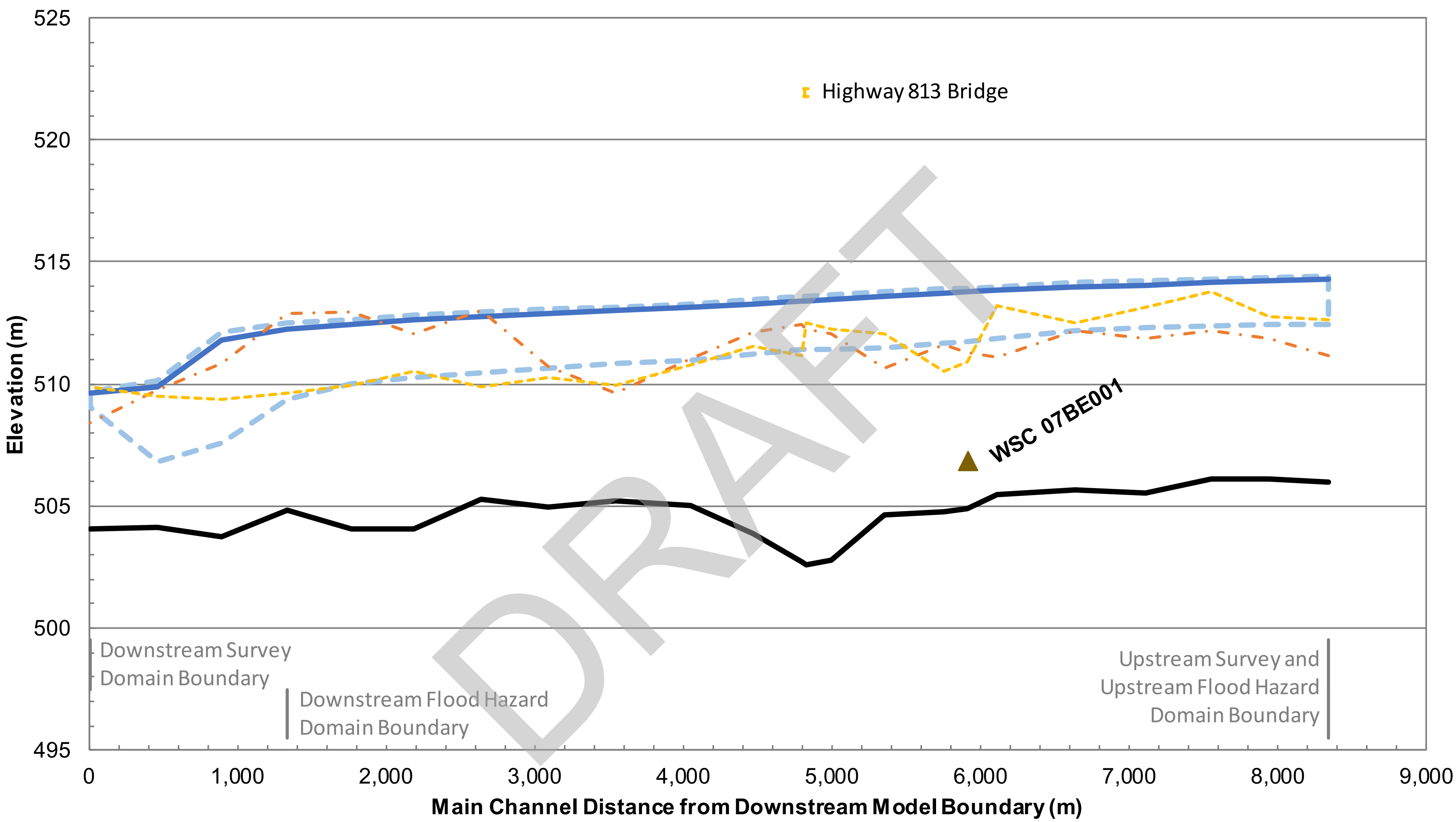

Figure 5-5: 50-Year Ice Jam Flood Profile, Q_{50} = 915 m³/s

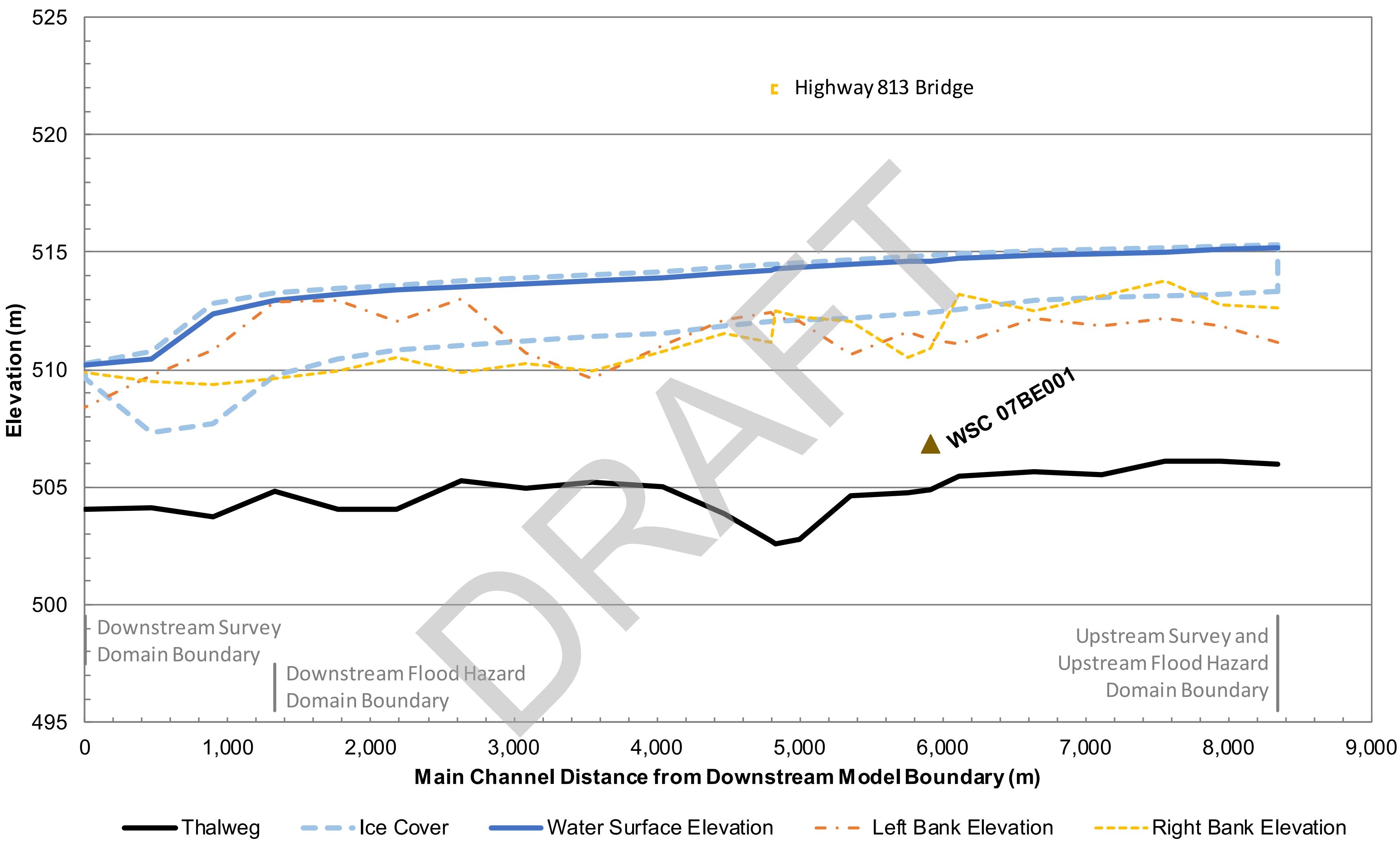

Figure 5-6: 100-Year Ice Jam Flood Profile, Q_{100} = 1,150 m³/s

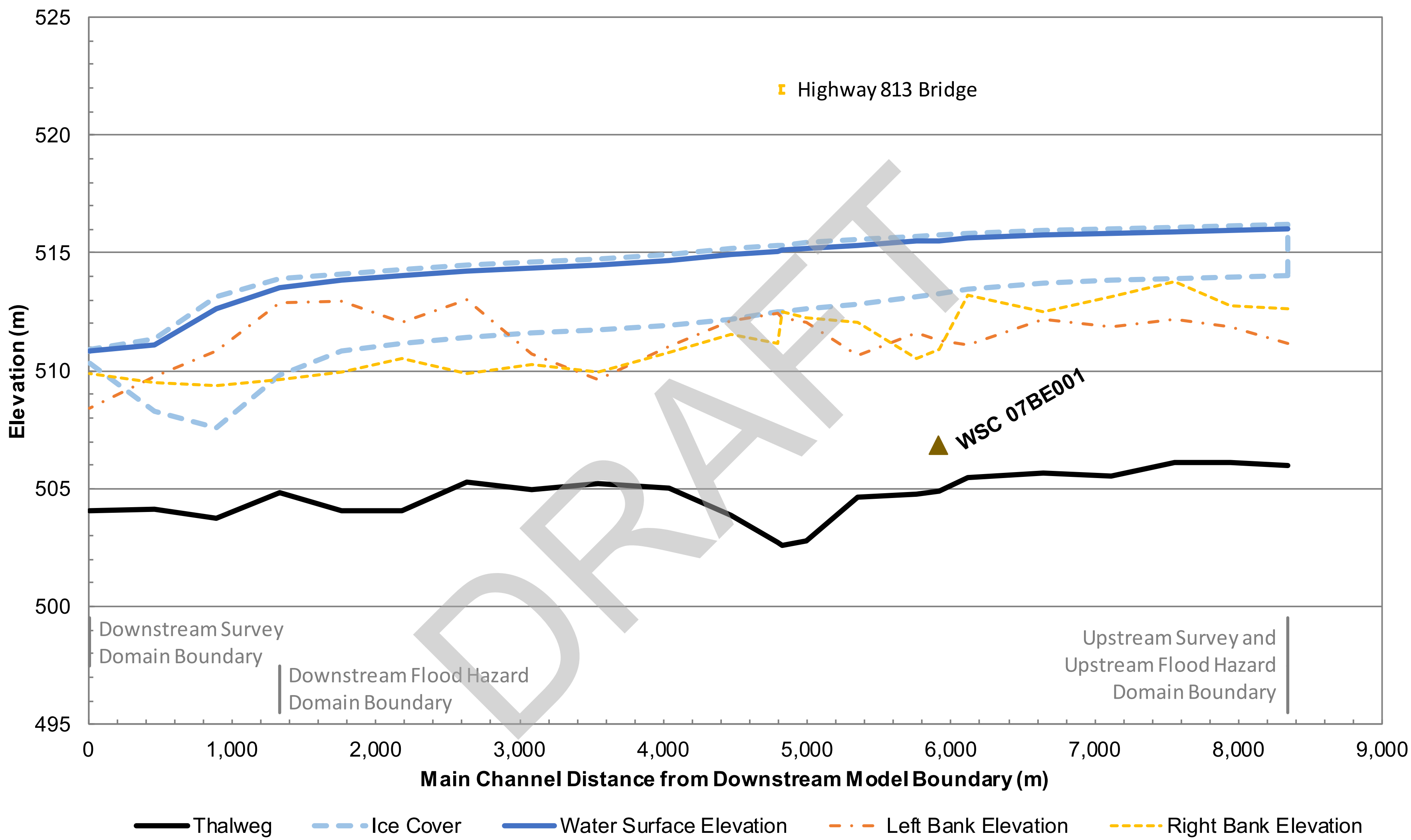

Figure 5-7: 200-Year Ice Jam Flood Profile, Q_{200} = 1,410 m^3/s

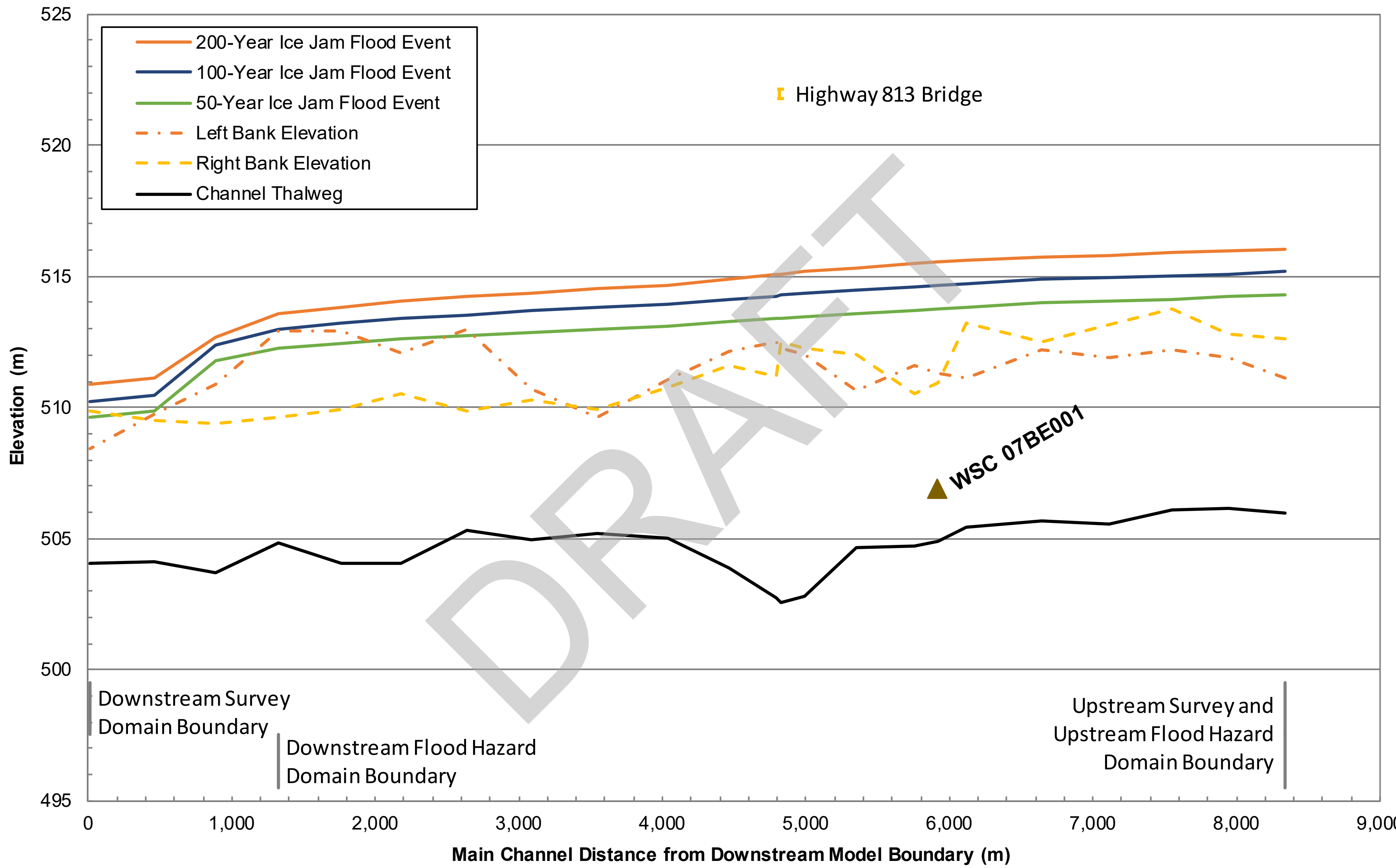

Figure 5-8: Comparison of 50-, 100-, and 200-year Ice Jam Flood Water Surface Profiles

Table 5-5: Water Level Comparison between Base Case and Scenario 1, Evaluating Floodplain Ineffective Flow Assumption

Return Period (years)	Water Level at WSC Gauge (m)		Difference in Water Levels within the Ice Jam Hazard Domain (m)[1, 2]		
	Flow within Floodplain (Base Case)	No Flow within Floodplain (Scenario 1)	Minimum	Average	Maximum
50	513.77	513.77	0[(3)]	0	0.01[(3)]
100	514.70	514.71	0[(4)]	0.01	0.02[(5)]
200	515.67	515.69	0.02[(6)]	0.03	0.06[(7)]

Notes:

[1] Flood hazard domain is the area between XS1 (river station 8339.925) and XS20 (river station 1329.168).
[2] Absolute difference in water levels calculated for cross sections within the flood hazard domain.
[3] 0.01 m difference in water levels occur at the following cross sections: XS2, XS3, XS4, XS5, XS9, XS11, XS12, XS14 through XS19, and XS21; and no difference in water levels occur at cross sections that are not aforementioned.
[4] 0.00 m difference in water levels occur at the following cross sections: XS1, XS6, XS22, and XS23.
[5] 0.02 m difference in water levels occur at the following cross sections: XS14, and XS16 through XS20.
[6] 0.02 m difference in water levels occur at the following cross sections: XS7 through XS13, XS20, and XS21.
[7] 0.06 m difference in water levels occur at the following cross sections: XS17 and XS18.

5.6.3 Extended Downstream Model Boundary

The downstream end of the model domain was established at a considerable distance (1.3 km) downstream of the downstream end of the flood hazard domain (i.e., Base Case) to ensure that the ice jam toe would be located far enough downstream of the flood hazard domain for an equilibrium ice jam thickness to develop within the flood hazard domain. To evaluate if the model domain was far enough downstream to achieve this objective, additional cross sections, as represented by the cross section at XS1 (river station 6.481), were added at 200 m intervals to extend the model boundary by 2,000 m downstream of the model domain, at an average slope of 0.00020 m/m (i.e., Scenario 2).

Water level comparisons within the flood hazard domain for the two model configurations (i.e., Base Case and Scenario 2 with the extended downstream model domain) are summarized in Table 5-6, with a detailed figure showing water level profiles provided in Appendix J, Figure J-2. Based on the comparison results, it was concluded that the ice jam toe was placed far enough downstream such that equilibrium ice thickness was reached within the ice jam hazard domain.

Table 5-6: Water Level Comparison between Base Case and Scenario 2, Evaluating Toe Location Assumption

Return Period (years)	Water Level at Downstream End of the Flood Hazard Domain (m)[1]		Difference in Water Level at Downstream End of the Flood Hazard Domain (m)[2]
	Adopted Model (Base Case)	Extended Model (Scenario 2)	
50	512.26	512.20	0.06
100	513.02	513.00	0.02
200	513.67	513.74	0.07

Notes:

[1] Downstream end of the flood hazard domain is located on XS20 (river station 1329.168).
[2] Absolute difference in water level at the selected cross section.

5.6.4 Interpolated Cross Sections

As discussed in Section 5.3.3, additional cross sections were not interpolated in the model between survey cross sections (i.e. Base Case), because of potential adverse effects on model accuracy should the ratio of cross section spacing to river width be reduced below approximately 1.0. The ice jam model had an average cross section spacing to river width ratio of 1.2.

To evaluate the model sensitivity, the ice jam model was run with interpolated cross sections where the Base Case model spacing exceeded 200 m. This resulted in 32 additional cross sections, with a total of 55 cross sections between river station 6.481 and river station 8339.925, for an average cross section spacing of 155 m and an average spacing to channel width ratio of 0.5 (i.e., Scenario 3).

Water level comparisons within the flood hazard domain for the two model configurations (i.e., Base Case and Scenario 3) are summarized in Table 5-7, with a detailed figure showing water level profiles provided in Appendix J, Figure J-3. Based on the comparison results, it was concluded that the ice jam model would not have benefited from additional interpolated cross sections, as within the ice jam hazard domain, the model was relatively insensitive to reducing the cross section spacing ratio from 1.2 to 0.5.

Table 5-7: Water Level Comparison between Base Case and Scenario 3, Evaluating Cross Section Spacing Sensitivity

Return Period (years)	Water Level at WSC Gauge (m)		Difference in Water Levels within the Ice Jam Hazard Domain (m)[1, 2]		
	Adopted Model (Base Case)	Interpolated Model (Scenario 3)	Minimum	Average	Maximum
50	513.76	513.82	0[(3)]	0.06	0.10[(4)]
100	514.64	514.71	0.01[(5)]	0.06	0.09[(6)]
200	515.53	515.58	0[(7)]	0.05	0.11[(8)]

Notes:

1 Flood hazard domain is the area between XS1 (river station 8339.925) and XS20 (river station 1329.168).
2 Absolute difference in water levels calculated for cross sections within the flood hazard domain.
3 No difference in water levels occurs at the following cross section: XS17.
4 0.10 m difference in water levels occurs at the following cross section: XS14.
5 0.01 m difference in water levels occur at the following cross sections: XS16 and XS17.
6 0.09 m difference in water levels occurs at the following cross section: XS20.
7 No difference in water levels occur at the following cross sections: XS20.
8 0.11 m difference in water levels occurs at the following cross section: XS14.

6.0 OPEN WATER AND ICE JAM FLOOD INUNDATION MAPS

The scope of the open water and ice jam flood inundation mapping component include the following tasks:

- open water flood inundation map production;
- ice jam flood inundation map production;
- flood water surface TIN development (open water and ice jam floods); and
- flood depth grid creation (open water and ice jam floods).

6.1 Methodology

The open water and ice jam flood inundation maps were prepared based on the following information:

- Simulated open water levels at individual cross sections for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year flood events;
- Simulated ice jam water levels at individual cross sections for the 50-, 100-, and 200-year flood events (Athabasca River only);
- Locations and extents of individual cross sections;
- Topography from the 2018 LiDAR survey; and
- Aerial imagery of the study area collected in May 2019.

The following procedure was used in ArcGIS to develop inundation extents for the open water design flood:

1) Assign design flood water levels at each section to the cross section polyline features as attributes. The result was one polyline feature that included the simulated water levels.
2) Create a continuous water surface elevation TIN between cross sections.
3) Manually adjust the water surface elevation TIN in special areas as described in the inundation mapping section for this study. Adjustments were made by using 3D break lines to separate manually adjusted TIN sub-elements from the automatically generated TIN, and appropriately interpolated water surface elevations between select cross sections were applied.
4) Convert the adjusted TIN into a water surface elevation raster with the same resolution and cell alignment as the DTM raster.
5) Subtract the DTM raster from the water surface elevation raster to define wet areas.
6) Assign “NoData” to dry raster cells (with water depths smaller than 0.01 m).
7) Convert the wet areas into a flood inundation polygon dataset and deleted all features not directly connected to the main river channels (Scenario 1).
8) Simplify polygons by filing holes smaller than 25 m^2.
9) Smoothed polygons outlines using the PAEK (Polynomial Approximation with Exponential Kernel) algorithm with a threshold of 15 m.

In addition to the above general procedure, backwater inundation for relatively large tributaries was included and based on the design water levels at the main channel at the confluences of those tributaries. This method was applied to Muskeg Creek and the Tawatinaw River, which are the only named tributaries that join the Athabasca River study reach.

The delineated inundation areas were then carefully reviewed and modified for the following scenarios:

- Scenario 1 (S1) – Single Overtopping Point: At locations where inundated areas are connected to the main channel at a single overtopping point (spill point), the inundation extent was re-evaluated using a constant water level which is equal to that at the spill point.
- Scenario 2 (S2) – Multiple Overtopping Points: If there are multiple overtopping points related to a single overflow area, the inundation extent was based on the hydraulic gradient in the main channel between the overtopping points. The inundation extent upstream of the most upstream overtopping point and downstream of the most downstream overtopping point were evaluated using the estimated water levels at these bounding spill points.
- Scenario 3 (S3) – Single Overtopping Point Causing Overtopping Downstream: At some locations, Scenario 1 can lead to the following situation: if the area behind the single overtopping location would be (after some time) completely inundated and pooled with a constant water level elevation similar to the water level at the spill point, this may cause a second overtopping further downstream and flow back into the main channel, because at that point the water level behind the embankment may be higher than that in the main channel. In this case, the inundation extent was re-evaluated using a linear interpolation between the water level at the upstream spill point and the ground elevation at the downstream re-entry point.
- Scenario 4 (S4) – Potential Flood Inundation due to Flood Control Structure Failure: In areas where permanent flood control structures have been identified and are not overtopped, the protected areas are shown as potentially flooded. The inundation extent is determined by assuming that the flood control structure is ineffective. No permanent flood control structures have been identified in the study area.

6.2 Water Surface Elevation TIN Modifications

6.2.1 Open Water and Ice Jam Inundation Mapping

One set of open water flood inundation maps was prepared for each of the 13 flood events and a set of ice jam flood inundation maps were prepared for each of the three ice jam flood events. The study area is covered by 4 sheets (11 inch x 17 inch). The mapping scale is 1:10,000. The maps were prepared using the local 3-Degree Transverse Mercator (3TM) zone and the Canadian Spatial Reference System North American Datum of 1983 (NAD83 CSRS) coordinate system and datum.

The maps include the 2019 aerial imagery and other base data (roads and railways) provided by AEP. The resulting open water inundation maps for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year flood events are presented in Volume 2 of this report and the resulting ice jam inundation maps for the 50-, 100-, and 200-year events are presented in Volume 3 of this report.

The flood inundation maps were prepared in a geographical information system (ESRI ArcGIS 10.7). The maps, including all layers, were provided to AEP as digital files in the ESRI ArcGIS file format and in hardcopy in the map libraries provided in Volume 2 and Volume 3 of this report.

6.2.2 Manual Edits

6.2.2.1 Locations

For the open water flood events there are no areas under the four scenarios (i.e., S1, S2, S3 and S4 described in Section 6.1) that required manual edits, as the flood flows would be contained in and directly connect to the river/creek channels along the study reach. Removal of cross sections and flood water replacement was required in inundated areas adjacent to the two major tributary confluences with the Athabasca River in the Town of Athabasca (see Section 6.2.2.2).

For the ice jam flood events, water levels were simulated for the Athabasca River only. Inundation extents along the two major tributaries were delineated based as backwater effects from the Athabasca River. The channel of Muskeg Creek forms a wide opening at the confluence with the Athabasca River, causing continuous overtopping of the bank of the Athabasca River along the opening. Backwater levels in the Muskeg Creek channel were therefore based on the continuous interpolation between the Athabasca River cross sections at this location (cross sections 5 and 6). At the confluence of the Tawatinaw River with the Athabasca River, the channel narrows at the Highway 55 Bridge, and therefore the inundation in the Tawatinaw River channel behind this narrow point was mapped based on a single split point (S1).

6.2.2.2 Removal of Cross Sections and Flood Water Level Replacements at Tributary Confluences

During open water flood events with a 20-year return period or higher at Muskeg Creek confluence, and a 10-year return period or higher at Tawatinaw River confluence, certain cross sections on both tributaries are fully located within the inundation extents of the Athabasca River, near their respective confluences. As the flood inundation water levels in these areas would be governed by the Athabasca River, the water surface elevation TINs were modified based on the Athabasca River cross sections. The cross sections of the tributaries were removed from the TINs when they were fully located within the inundation extent of the Athabasca River, depending on the flood events.

The flood water levels for the removed cross sections are reported based on the interpolated flood water levels on the Athabasca River at the intersection of the respective cross section and centreline of the tributary. Table F-2 and Table F-3 in Appendix F provide the detailed cross section water level replacements at the two tributary confluences with the Athabasca River.

6.3 Direct Flood Inundation Areas

6.3.1 Definition

The direct flood inundation areas are attributed to direct overland flow connection to the main channel flow, including Scenarios 1, 2 and 3.

6.3.2 Major Direct Inundation Areas

Athabasca River Reach

There are some low-lying areas on the floodplains where ponding water would occur, in particular at the Riverfront Park in the Town of Athabasca.

Open Water Floods: Some trails at Riverfront Park could be inundated during open water floods with return periods as low as 10 years. During the 100-year open water flood, Athabasca River flooding would be limited to areas north of 50th Avenue (Highway 2). Some flooding south of 50th Avenue would occur for open water floods exceeding the 350-year return period.

Ice Jam Floods: Some trails at Riverfront Park could be inundated during open water floods with return periods as low as 50 years. During the 100-year ice jam flood, Athabasca River flooding would be limited to areas north of 50th Avenue (Highway 2). Some flooding south of 50th Avenue would occur for the 200-year ice jam flood.

Muskeg Creek Reach

This creek reach has an incised channel, generally confined on both sides by the valley walls. It has limited floodplains. There are many small, low-lying areas on the floodplains where ponding water would occur.

Open Water Floods: The inundated areas on the floodplains would be direct flood inundation areas for open water floods with return periods of 10 years or higher. During open water floods with return periods of 100 years or higher, sections of the three local roads (56th Street, 57th Street and the adjacent 50th Avenue south of 50th Avenue) would be inundated. During open water floods with return periods of 200 years or higher, sections of Highway 2 and its culvert crossing would be inundated.

Ice Jam Floods: The inundated areas on the floodplains would be direct flood inundation areas for ice jam floods with return periods of 50 years or higher. During ice jam floods with return periods of 100 years or higher, sections of the three local roads (56th Street, 57th Street and the adjacent 50th Avenue south of 50th Avenue) would be inundated. During ice jam floods with return periods of 200 years or higher, sections of Highway 2 and its culvert crossing would be inundated.

Tawatinaw River Reach

This river reach has an incised channel, generally confined on both sides by the valley walls. It has limited floodplains. There are many small low-lying areas on the floodplains where ponding water would occur.

Open Water Floods: The inundated areas on the floodplains would be direct flood inundation areas for open water floods with return periods of 10 years or higher. During open water floods with return periods of 20 years or higher, sections of the pedestrian trail east of the river would be inundated. During open water floods with return periods of 200 years or greater, flooding could extend to buildings on the west side of the river at 50th Avenue (Highway 55). During open water floods with return periods of 350 years or greater, sections of Highway 55 and its bridge crossing would be inundated.

Ice Jam Floods: The inundated areas on the floodplains would be direct flood inundation areas for ice jam floods with return periods of 50 years or higher. During ice jam floods with return periods of 200 years or greater, flooding could extend to buildings on the west side of the river at 50th Avenue (Highway 55).

6.3.3 Areas Affected by Flooding

6.3.3.1 Residential Areas

Town of Athabasca

There is one major residential area (i.e., Town of Athabasca) situated on the Athabasca River floodplains. Flooding of residential areas would be limited to properties along Muskeg Creek, upstream of the confluence with the Athabasca River, as noted below.

Athabasca River Floodplains

There is no other major residential area situated on the floodplains along this reach except for the Town of Athabasca.

Muskeg Creek Floodplains

Several residences are located on the left downstream side of Muskeg Creek, south of 50th Avenue.

Open Water Floods: Residences on the west side of 57th Street and east side of 58th Street, between 49th Avenue and 50th Avenue, would be inundated for open water floods with return periods of 200-years or greater. Homes on the west side of 58th Street would not be inundated for the 1000-year open water flood.

Ice Jam Floods: Residences on the west side of 57th Street and east side of 58th Street, between 49th Avenue and 50th Avenue, would be inundated for the 200-year ice jam flood.

Tawatinaw River Reach

There is no major residential area situated on the floodplains along this reach. No residences would be inundated for the 1000-year open water flood.

6.3.3.2 Commercial and Industrial Areas

Athabasca River Floodplains

Open Water Floods: Portions of the Town of Athabasca Riverfront Park, situated on the right floodplain, would be inundated during the flood events with return periods of 10 years or higher. Municipal facility buildings located in Riverfront Park east of 48th Street would be inundated for open water floods with return periods of 75 years or greater.

North of 50th Avenue, the Home Hardware building would be inundated for open water floods with return periods of 75 years or greater. Buildings that would be inundated for open water floods with return periods of 200 years or greater include the Town of Athabasca Pumphouse (WSC Station), the Independent Grocers and Gregg Distributors buildings, and the Petro Canada bulk fuel facility. For open water floods with return periods exceeding 350 years, widespread inundation of buildings on the south side of 50th Avenue would be expected, including some buildings on intersecting streets up to 100 m south of 50th Avenue. The Tizzco Riverside Machine & Welding shop would be inundated for open water floods with return periods of 750 years or greater.

The Athabasca Golf Course, situated on the left floodplain downstream of Highway 813, would not be inundated during the 1000-year open water flood.

Ice Jam Floods: Municipal facility buildings located in Riverfront Park east of 48th Street would be inundated for ice jam floods; the west building would be inundated for the 100-year ice jam flood and the east building would be inundated for the 50-year ice jam flood.

North of 50th Avenue, the Home Hardware building would be inundated for ice jam floods with return periods of 100 years or greater. Buildings that would be inundated for ice jam floods with return periods of 200 years or greater include the Town of Athabasca Pumphouse (WSC Station), the Independent Grocers and Gregg Distributors buildings, and the Petro Canada bulk fuel facility.

The Athabasca Golf Course, situated on the left floodplain downstream of Highway 813, would not be inundated during the 200-year ice jam flood.

Muskeg Creek Floodplains

Open Water Floods: Two commercial/industrial buildings are located on the east side of Muskeg Creek, immediately south of 50th Avenue and west of 56th Street. The southern of these properties would be inundated for open water floods with return periods of 100 years or greater. The northern of these properties would be inundated for open water floods with return periods of 200 years or greater.

Ice Jam Floods: Two commercial/industrial buildings are located on the east side of Muskeg Creek, immediately south of 50th Avenue and west of 56th Street. Both of these properties would be inundated for the 200-year ice jam flood.

Tawatinaw River Reach

Open Water Floods: For open water floods with return periods of 200 years or greater, the SS Athabasca Car Wash building would be inundated.

Ice Jam Floods: No commercial/industrial buildings were identified as being at risk for ice jam floods up to the 200-year return period.

6.3.3.3 Bridges and Culverts

A bridge is considered affected by flood when flood waters reach its low chord. A culvert is considered affected by flood when flood waters reach the road surface.

Athabasca River

Table 6-1 summarizes the simulated water levels at Highway 813 bridge along the Athabasca River for the 2-year to 1,000-year open water floods, and Table 6-2 summarizes simulated water levels for the 50-, 100- and 200-year ice jam floods. Flow velocities and clearances during the 100-year flood event are also provided.

Open Water Floods: There is one bridge (i.e., Highway 813 bridge), which would not be affected during the 1000-year open water flood.

Ice Jam Floods: There is one bridge (i.e., Highway 813 bridge), which would not be affected during the 200-year ice jam flood.

Muskeg Creek

Table 6-1 summarizes the simulated water levels at the three footbridges and one culvert along the Muskeg Creek reach for the 2-year to 1,000-year open water floods, and Table 6-2 summarizes simulated water levels for the 50-, 100- and 200-year ice jam floods. Flow velocities and bridge clearances or flow depths above the road surface for the 100-year flood event are also provided.

Open Water Floods: None of the footbridges along the Muskeg Creek reach would be affected during open water flood events with return periods of 75 years or lower. Two pedestrian bridges would be affected during open water flood events with return periods of 100 years or higher. The Highway 2 culvert along the Muskeg Creek reach would not be affected during open water flood events with return periods of 50 years or lower. The 200-year open water flood would overtop the road surface at of Highway 2 culvert crossing along the Muskeg Creek reach.

Ice Jam Floods: Backwater conditions from ice jams on the Athabasca River may affect water levels in lower Muskeg Creek. The Highway 2 culvert along the Muskeg Creek reach would reach the culvert crown for the 100-year ice jam flood and would surcharge the culvert and overtop Highway 2 to the east of the culvert during the 200-year ice jam flood.

The lower chord of the first footbridge on Muskeg Creek would be inundated for the 200-year ice jam event. The second and third footbridges are upstream of the effects of the 200-year ice jam event.

Tawatinaw River Reach

Table 6-1 summarizes the simulated water levels at the two bridges along the Tawatinaw River reach for the 2-year to 1,000-year open water floods, and Table 6-2 summarizes simulated water levels for the 50-, 100- and 200-year ice jam floods. Flow velocities and bridge clearances or flow depths above the road surface for the 100-year flood event are also provided.

Open Water Floods: None of the bridges along the Tawatinaw River reach would be affected during open water flood events with return periods of 50 years or lower. The footbridge would be affected during open water flood events with return periods of 75 years or higher. The Highway 55 bridge would be affected during open water flood events with return periods of 200 years or higher.

Ice Jam Floods: Backwater conditions from ice jams on the Athabasca River may affect water levels in the lower Tawatinaw River. The lower chord of the Highway 55 bridge along the Tawatinaw River reach would be inundated during the 200-year ice jam flood. The lower chord of the Tawatinaw River footbridge would be inundated during the 100-year ice jam flood.

Table 6-1: Open Water Flooding at the Bridges and Culvert along the Study Reaches of Athabasca River, Muskeg Creek and Tawatinaw River

Bridge /Culvert Station (m)	Name	Minimum Deck/Road Surface Elevation (m)	Minimum Low Chord/ Culvert Top Elevation (m)	Simulated Water Level at the Bridges/Culverts for the Various Flood Events (m)													Average Flow Velocity for the 100-year Flood Event (m/s)	Clearance for 100-year Flood Event[1] (m)	Return Period of Flood Event Causing Pressure Flow or Overtopping Road Surface (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			
4810.9	Highway 813 Bridge on Athabasca River	522.7	521.8	510.3	511.2	511.8	512.4	512.9	513.2	513.6	513.9	514.6	515.1	515.5	516.0	516.3	3.49	7.89	>1000-year
699.56	Highway 2 Culvert on Muskeg Creek	515.9	514.8	511.2	512.1	512.8	513.5	514.1	514.5	514.9	515.2	516.1	516.2	516.7	517.1	517.5	0.72	-0.41	75-year
936.64	1st Footbridge on Muskeg Creek (North Bridge)	515.8	515.1	512.3	512.7	513.1	513.7	514.2	514.6	515.0	515.4	516.2	516.8	517.1	517.6	518.0	1.14	-0.31	100-year
1377.5	2nd Footbridge on Muskeg Creek (Middle Bridge)	519.1	518.6	516.7	517.0	517.3	517.6	517.8	518.0	518.1	518.2	518.5	518.8	519.2	519.6	520.0	2.02	0.34	350-year
3046.1	3rd Footbridge on Muskeg Creek (South Bridge)	530.2	529.5	528.1	528.4	528.7	528.9	529.1	529.3	529.5	529.6	530.1	530.5	530.7	530.8	530.9	1.66	-0.12	100-year
366.75	Highway 55 Bridge on Tawatinaw River	517.1	515.1	510.7	511.7	512.4	513.1	513.7	514.1	514.5	514.8	515.6	515.8	516.2	516.7	517.0	0.52	0.26	200-year
603.96	Footbridge on Tawatinaw River	514.6	514.3	510.7	511.8	512.4	513.1	513.7	514.1	514.5	514.8	515.6	516.3	516.7	517.2	517.6	0.47	-0.48	75-year

Note 1: The clearances for the 100-year flood event are the elevation differences between bridge low chord elevations or culvert road surface elevations and simulated water levels. A negative value indicates that the water depth above the low chord for a bridge or above the road surface for a culvert.

Table 6-2: Ice Jam Flooding at the Bridges and Culvert along the Study Reaches of Athabasca River, Muskeg Creek and Tawatinaw River

Bridge /Culvert Station (m)	Name	Minimum Deck/Road Surface Elevation (m)	Minimum Low Chord/ Culvert Top Elevation (m)	Simulated Water Level at the Bridges/Culverts for the Various Flood Events (m)			Average Flow Velocity for the 100-year Flood Event (m/s)	Clearance for 100-year Flood Event[1] (m)	Return Period of Flood Event Causing Pressure Flow or Overtopping Road Surface (Return Period)
				50-year	100-year	200-year			
4810.9	Highway 813 Bridge on Athabasca River	522.7	521.8	513.4	514.3	515.1	0.89	7.5	>200-year
699.56	Highway 2 Culvert on Muskeg Creek	515.9	514.8	513.9	514.8	515.6	1.06	0	100-year
936.64	1st Footbridge on Muskeg Creek (North Bridge)	515.8	515.1	513.9	514.8	515.6	1.19	0.3	200-year
366.75	Highway 55 Bridge on Tawatinaw River	517.1	515.1	513.7	514.6	515.4	0.41	0.5	200-year
603.96	Footbridge on Tawatinaw River	514.6	514.3	513.7	514.6	515.4	0.41	-0.3	100-year

Note 1: The clearances for the 100-year flood event are the elevation differences between bridge low chord elevations or culvert road surface elevations and simulated water levels. A negative value indicates that the water depth above the low chord for a bridge or above the road surface for a culvert.

6.4 Flood Depth Grids

6.4.1 GIS Data Specifications

The following GIS data were provided to AEP for each of the 13 open water flood events and each of the three ice jam flood events:

- Inundation polygons;
- Water surface elevations TINs;
- Water surface elevation rasters; and
- Flood depth rasters.

All GIS data were created in ArcGIS 10.7 compatible format in the native study coordinate system (Canadian Spatial Reference System, North American Datum of 1983 (CSRS NAD83), Epoch 2002 and 3-Degree Transverse Mercator projection with the Central Meridian of 111° (3TM 111)). All raster files have a spatial resolution of 0.5 m.

The inundation polygons and raster files were stored in ArcGIS file geodatabases, Version 10.7. The flood water level TINs were stored as ArcGIS terrain datasets in the file geodatabases, Version 10.7.

6.4.2 General Comments

The flood water level data, provided as terrains and rasters, cover all areas between cross section lines and in special inundation areas within the study area including dry areas. The flood water depth rasters only include the areas with a water depth of more than 0.01 m.

The calibrated HEC-RAS model and the LiDAR DTM provided a good basis for simulating the flood levels and preparing the flood inundation maps for the 13 open water flood events (i.e. 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year open water floods) and three ice jam flood events (50-, 100- and 200-year events). The simulation results show that some residential and commercial/industrial areas along the study reach would be affected by open water flooding, particularly within the Town of Athabasca. The ice jam simulation results show that commercial/industrial areas along the Athabasca River, between the two tributaries, would be affected by ice jam water flooding.

7.0 FLOODWAY DETERMINATION

7.1 Design Flood Selection

The 100-year flood, under open water or ice jam condition, was selected as the design flood for this study. The corresponding peak instantaneous flood frequency discharge estimates from Table 4-9 (open water) and Table 5-3 (ice jam) were used for each flow zone within the study reaches.

7.2 Floodway and Flood Fringe Terminology

The flood hazard area is the area of land that will be flooded during the design flood event. The flood hazard area is typically divided into two zones (i.e., floodway and flood fringe).

The floodway and flood fringe zones are defined as follows:

- 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 areas where the water is 1 m deep or greater and the local velocities are 1 m/s or faster. Typically, the floodway includes the river channel and adjacent overbank areas. 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. New development is discouraged in the floodway and may not be permitted in some communities.
- Flood Fringe: The flood fringe is the land along the edges of the flood hazard area that has relatively shallow water (less than 1 m deep) with lower velocities (less than 1 m/s). 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. New development in the flood fringe may be permitted in some communities.

7.3 Open Water Flood Hazard Identification

7.3.1 Open Water Floodway Determination Criteria

In areas being mapped for the first time, 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.
- For reaches of supercritical flow, the floodway boundary should correspond to the edge of inundation or the main channel, whichever is larger.

When a flood hazard map is updated, an existing floodway will not change in most circumstances. Exceptions to this would be: (1) a floodway could get larger if a main channel shifts outside of a previously-defined floodway or (2) a floodway could get smaller if an area of previously-defined floodway is no longer flooded by the design flood.

Areas of deeper or faster moving water outside of the floodway are identified as high hazard flood fringe. These high hazard flood fringe zones are identified in all areas, whether they are newly-mapped or have an existing floodway. The depth and velocity criteria used to define high hazard flood fringe zones will be aligned with the 1 m depth and 1 m/s velocity floodway determination criteria for newly-mapped areas.

All areas protected by dedicated flood berms that are not overtopped during the design flood are excluded from the floodway. Areas behind flood berms will still be mapped as flooded if they are overtopped, but areas at risk of flooding behind dedicated flood berms that are not overtopped will be mapped as a protected flood fringe zone.

The floodway determination criteria for the left and right floodway limits at individual cross sections along the study reaches are listed in Table 7-1 (Athabasca River), Table 7-2 (Muskeg Creek) and Table 7-3 (Tawatinaw River).

Table 7-1: Open Water Floodway Limits and Design Flood Water Levels along the Athabasca River

River	Cross Section	River Station	Floodway Limit		Floodway Determination Criteria	100-Year Open Water Design Flood Level (m)
			Left (m)	Right (m)		
Athabasca River	1	8340	106.92	469.44	Inundation Limit (1)	515.30
Athabasca River	2	7941	143.18	532.88	Inundation Limit (1)	515.23
Athabasca River	3	7550	135.17	504.82	Previous Floodway	515.15
Athabasca River	4	7109	121.41	496.38	Previous Floodway	515.07
Athabasca River	5	6640	144.86	519.78	Previous Floodway	514.99
Athabasca River	6	6112	40.97	318.64	Previous Floodway	514.61
Athabasca River	7	5915	47.30	302.25	Previous Floodway	514.47
Athabasca River	8	5756	41.99	324.81	Previous Floodway	514.46
Athabasca River	9	5352	151.61	501.43	Previous Floodway	514.38
Athabasca River	10	4994	201.72	436.66	Previous Floodway	514.08
Athabasca River	11	4829	269.33	489.28	Previous Floodway	513.87
Athabasca River	12	4796	258.19	469.74	Previous Floodway	513.78
Athabasca River	13	4468	245.16	492.66	Previous Floodway (Left) – Inundation Limit (2) (Right)	513.65
Athabasca River	14	4043	338.78	619.53	Previous Floodway	513.48
Athabasca River	15	3542	598.68	1023.40	Previous Floodway	513.53
Athabasca River	16	3087	574.24	986.53	Previous Floodway (Left) – Inundation Limit (2) (Right)	513.42
Athabasca River	17	2636	371.58	733.04	Previous Floodway (Left) – Inundation Limit (2) (Right)	513.24
Athabasca River	18	2182	85.36	422.39	Inundation Limit (1) (Left) – 1 m Depth (Right)	513.06
Athabasca River	19	1764	95.00	436.02	Inundation Limit (1) – 1 m Depth (Right)	512.87
Athabasca River	20	1329	144.39	571.80	Inundation Limit (1)	512.85

Notes: (1) No viable flood fringe.
(2) Previous floodway is outside the inundation limit.

Table 7-2: Open Water Floodway Limits and Design Flood Water Levels along Muskeg Creek

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Open Water Design Flood Level (m)
			Left (m)	Right (m)		
Muskeg Creek	24	7594	63.55	84.60	Main Channel (Left) – Inundation Limit (1) (Right)	546.31
Muskeg Creek	25	7286	109.10	126.68	Main Channel (Left) – Inundation Limit (1) (Right)	545.14
Muskeg Creek	26	6935	158.85	173.92	1 m/s Velocity(Left) – Inundation Limit (1)	543.65
Muskeg Creek	27	6532	61.69	75.88	Main Channel	541.65
Muskeg Creek	28	6009	207.52	223.88	Main Channel	540.00
Muskeg Creek	29	5401	128.53	147.37	Main Channel (Left) – Inundation Limit (1) (Right)	537.89
Muskeg Creek	30	4921	108.20	124.88	Main Channel	536.22
Muskeg Creek	31	4806	107.20	121.88	Main Channel	535.79
Muskeg Creek	32	4382	78.56	95.02	Inundation Limit (1) (Left) – Main Channel (Right)	534.35
Muskeg Creek	33	3571	208.20	226.75	Inundation Limit (1)	531.58
Muskeg Creek	34	3153	37.50	53.18	Main Channel (Left) – Inundation Limit (1) (Right)	530.10
Muskeg Creek	35	3049	49.12	67.16	Main Channel	529.64
Muskeg Creek	36	3043	55.76	71.28	Main Channel	529.56
Muskeg Creek	37	2993	98.81	117.58	Inundation Limit (1)	529.22
Muskeg Creek	38	2891	133.80	147.78	Inundation Limit (1) (Left) – Main Channel (Right)	528.65
Muskeg Creek	39	2728	18.74	33.39	Inundation Limit (1) (Left) – Main Channel (Right)	527.75
Muskeg Creek	40	2628	62.35	78.80	Inundation Limit (1)	527.02
Muskeg Creek	41	2116	125.24	143.72	Inundation Limit (1)	523.20
Muskeg Creek	42	1722	297.23	312.85	Inundation Limit (1)	520.65
Muskeg Creek	43	1406	122.15	138.53	Inundation Limit (1) (Left) – Main Channel (Right)	518.43
Muskeg Creek	44	1383	126.66	138.71	Main Channel (Left) – Inundation Limit (1) (Right)	518.24
Muskeg Creek	45	1374	125.23	137.15	Main Channel	518.14
Muskeg Creek	46	1162	156.03	166.79	Inundation Limit (1)	515.97
Muskeg Creek	47	973	136.17	149.30	Inundation Limit (3)	515.41
Muskeg Creek	48	943	151.77	165.99	Inundation Limit (3) (Left) – Previous Floodway (Right)	515.38
Muskeg Creek	49	934	156.15	170.60	Inundation Limit (3) (Left)– Previous Floodway (Right)	515.34
Muskeg Creek	50	825	182.83	203.06	Previous Floodway	515.28
Muskeg Creek	51	718	251.94	275.90	Inundation Limit (1)	515.24
Muskeg Creek	52	680	259.86	277.42	Previous Floodway (Left) – 1 m Depth (Right)	514.83
Muskeg Creek	53	610	259.18	-	Inundation Limit (3) (Left) – Mixed (2) (Right)	514.84
Muskeg Creek	54	463	-	-	Mixed (2)	514.80
Muskeg Creek	55	357	-	77.21	Mixed (2) (Left) – Previous Floodway (Right)	514.76
Muskeg Creek	56	249	-	63.54	Mixed (2) (Left) – Previous Floodway (Right)	514.70
Muskeg Creek	57	141	-	145.26	Mixed (2) (Left) – Previous Floodway (Right)	514.66

Notes: (1) No viable flood fringe.
(2) Floodway extents pass the cross section line.

(3) Previous floodway is outside the inundation limit.

Table 7-3: Open Water Floodway Limits and Design Flood Water Levels along the Tawatinaw River

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Open Water Design Flood Level (m)
			Left (m)	Right (m)		
Tawatinaw River	58	6230	126.59	157.73	Main Channel (Left) - Mixed (2) (Right)	523.05
Tawatinaw River	59	5962	143.73	169.31	1 m Depth	522.80
Tawatinaw River	60	5944	152.87	176.58	1 m Depth	522.79
Tawatinaw River	61	5679	174.97	198.27	1 m Depth	522.48
Tawatinaw River	62	5485	159.46	182.72	1 m Depth	522.18
Tawatinaw River	63	5274	92.07	112.99	Main Channel	521.97
Tawatinaw River	64	4893	170.53	258.80	1 m Depth (Left) – Main Channel (Right)	521.64
Tawatinaw River	65	4615	116.49	148.02	Mixed (2) (Left) – Main Channel (Right)	521.35
Tawatinaw River	66	4468	170.71	196.15	Main Channel	521.20
Tawatinaw River	67	4303	150.48	179.10	1 m Depth	520.99
Tawatinaw River	68	4196	118.19	140.33	Main Channel	520.85
Tawatinaw River	69	4033	131.74	184.91	Mixed (2) (Left) – Main Channel (Right)	520.66
Tawatinaw River	70	3805	207.95	232.63	Mixed (2) (Left) - Inundation Limit (1) (Right)	520.36
Tawatinaw River	71	3393	132.90	164.75	Previous Floodway (Left) - Inundation Limit (4) (Right)	519.37
Tawatinaw River	72	3179	259.92	293.67	Mixed (2) (Left) – 1 m Depth (5) (Right)	518.78
Tawatinaw River	73	2884	188.82	215.95	Previous Floodway	518.14
Tawatinaw River	74	2709	201.34	226.19	Inundation Limit (4) (Left) - Previous Floodway (Right)	517.73
Tawatinaw River	75	2450	210.27	223.64	Previous Floodway	516.73
Tawatinaw River	76	2270	248.64	274.99	Previous Floodway (Left) - 1 m/s Velocity (5) (Right)	515.59
Tawatinaw River	77	1973	270.06	301.77	Previous Floodway (Left) - Inundation Limit (5) (Right)	515.18
Tawatinaw River	78	1706	117.05	183.70	Previous Floodway	515.02
Tawatinaw River	79	1484	110.87	174.36	Previous Floodway	514.93
Tawatinaw River	80	1278	148.96	233.37	Previous Floodway	514.89
Tawatinaw River	81	1000	77.91	200.37	Inundation Limit (4) (Left) - Previous Floodway (Right)	514.86

Table 7-3: Open Water Floodway Limits and Design Flood Water Levels along the Tawatinaw River

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Open Water Design Flood Level (m)
			Left (m)	Right (m)		
Tawatinaw River	82	657	30.16	139.03	Inundation Limit (4) (Left) - Previous Floodway (Right)	514.83
Tawatinaw River	83	607	25.88	110.69	Previous Floodway	514.82
Tawatinaw River	84	599	26.03	119.30	Previous Floodway	514.82
Tawatinaw River	85	457	181.11	309.09	Previous Floodway (Left) - Inundation Limit (4) (Right)	514.81
Tawatinaw River	86	383	315.61	347.34	Main Channel (5) (Left) - Inundation Limit (4) (Right)	514.80
Tawatinaw River	87	355	374.01	411.71	Mixed (2) (Left) - Inundation Limit (4) (Right)	514.80
Tawatinaw River	88	257	18.40	73.37	Previous floodway (Left) - Inundation Limit (4) (Right)	514.40
Tawatinaw River	89	153	-	77.71	Mixed (3) (Left) – Previous Floodway (Right)	514.39

Notes: (1) No viable flood fringe.
(2) To balance other criteria, or to maintain smooth floodway.
(3) Floodway extents pass the cross section line.
(4) Previous floodway is outside the inundation limit.
(5) Main channel shift outside of the previous floodway.

7.3.2 Open Water Design Flood Profile

The open water design flood profile information along the study reaches of Athabasca River, Muskeg Creek and Tawatinaw River is provided in Table 7-1 (Athabasca River), Table 7-2 (Muskeg Creek), and Table 7-3 (Tawatinaw River).

7.3.3 Open Water Floodway Criteria Maps

Floodway criteria maps show the basis for determining the floodway, high hazard flood fringe zone and flood fringe zone for the design flood and documenting the results of water levels, depths and flow velocities. The open water floodway criteria maps include the following information:

- the extent of the open water design flood;
- areas meeting or exceeding the 1 m depth floodway criterion for the design flood ;
- the locations along each model cross section where flow velocities are calculated to meet the 1 m/s velocity floodway criterion;
- the locations of the main channel top of bank at each cross section;
- the floodway boundary for the open water design flood;
- the location and extent of all cross sections used in the HEC-RAS model with appropriate labels;
- background aerial imagery collected in 2019; and
- roads, bridges, culverts and flood control structures as applicable.

The study area is covered by a total of four sheets (11 inch x 17 inch) with the mapping scale of 1:10,000. The maps were prepared using the local 3-Degree Transverse Mercator (3TM) zone and the Canadian Spatial Reference System North American Datum of 1983 (NAD83 CSRS) coordinate system and datum. The maps are provided in Appendix K.

7.3.4 Open Water Flood Hazard Maps

The flood hazard maps display the areas in the floodway, high hazard flood fringe and flood fringe. The floodway was determined as part of the floodway criteria mapping. The high hazard flood fringe is part of flood fringe areas with deeper (1 m deep or greater) and faster moving water (the local velocities of 1 m/s or faster) outside the floodway limits. The flood fringe includes all other directly-inundated areas beyond the floodway limits.

The mapping scale is 1:10,000. The maps were prepared using the local 3-Degree Transverse Mercator (3TM) zone and the Canadian Spatial Reference System North American Datum of 1983 (NAD83 CSRS) coordinate system and datum. The maps are provided in Appendix L.

Areas in the Floodway

No residences are situated on the floodways along the Athabasca River, Muskeg Creek and the Tawatinaw River.

The development areas and structures within the floodway zone are listed below.

- A walkway on the right bank of the Athabasca River between the Muskeg Creek confluence and the Tawatinaw River confluence; and
- A wetland area on the right side of the Tawatinaw River upstream of the Highway 55 bridge crossing.

Areas in the High Hazard Flood Fringe

- There are not any residence and key structures within high hazard flood fringe.

Areas in the Flood Fringe

The development areas in the flood fringe within the study area are listed below.

Athabasca River

- Portions of parking lot of the Independent Grocery Store on the right floodplain;
- Athabasca Home Hardware Building Centre on the right floodplain; and
- 51st Avenue on the right floodplain.

Muskeg Creek

- Small portions on both sides of upstream of Highway 2 Culvert, including 57 Street and Muskeg Creek trails; and
- Small portions on the right side downstream of Highway 2 Culvert.

Tawatinaw River

- Small portions on the left bank upstream of Highway 55 Bridge.

7.3.5 Open Water Design Flood Grids

Water Surface Elevation Grids

The water surface elevation grid was created by converting the water surface elevation TIN into a raster file with the same resolution (0.5 m) and alignment as the DTM. The water surface elevation raster was then clipped to the directly-inundated areas and the areas of potential flood control structure failures.

Flood Depth Grids

The flood depth grid was created by subtracting the DTM from the water surface elevation grid. The flood depth grid has the same resolution (0.5 m) and alignment as the DTM. The extent of the depth grid is limited to the directly-inundated areas and the areas of potential flood control structure failures.

General Comments

Water surface elevation TINs cover all areas between cross sections and in special inundation areas within the study area, including dry areas. Corresponding rasters were clipped to the inundation extents of the design flood.

All GIS data were created in ArcGIS Version 10.7 compatible format in the native study coordinate system [Canadian Spatial Reference System, North American Datum of 1983 (CSRS NAD83), Epoch 2002 and 3-Degree Transverse Mercator projection with the Central Meridian of 114° (3TM 114)].

7.4 Ice Jam Flood Hazard Identification

7.4.1 Ice Jam Floodway Determination Criteria

The ice jam flood hazard area is the area of land that will be flooded during the passage of the 100-year ice jam flood event. This flooded area is divided into two zones (i.e., the floodway and the flood fringe). A high hazard flood fringe is part of flood fringe areas with deeper (1 m deep or greater) outside the floodway limits. For the ice jam flood, the boundary between the floodway and the flood fringe is defined using different floodway determination criteria and mapping standards than those outlined in FHIP guidelines as well as the existing floodway.

The floodway is delineated primarily as the area of highest hazard, defined where flood depths are 1 m or deeper. Given the backwater associated with a fully developed, wide channel ice jam, flow velocities are not typically relevant when defining the ice-affected floodway. The governing criterion under ice jam conditions is thus defined by flood depths of 1 m or greater.

The new floodway typically corresponds to the existing floodway, which defined in 1993 flood study (EC, 1993). The new floodway would be modified from the existing floodway if a main channel shifts outside of the existing floodway or the existing floodway is no longer flooded by the design flood.

It is important to note that not all floodway determination criteria and mapping standards are different than those outlined in FHIP guidelines. The floodway must include the main river channel area, and allowances may be made for small backwater areas, ineffective flow areas, and hydraulic smoothing when delineating the floodway boundary.

The left and right stations resulting from the floodway determination at the left and right stations at individual cross sections along the study reaches are listed in Table 7-4 (Athabasca River), Table 7-5 (Muskeg Creek) and Table 7-6 (Tawatinaw River). For Muskeg Creek and the Tawatinaw River, cross sections are only presented where ice jam backwater from the Athabasca River is present.

Table 7-4: Ice Jam Floodway Limits and Design Flood Levels along the Athabasca River

River	Cross Section	River Station	Floodway Limits[1]		Floodway Determination Criteria	100-Year Ice Jam Design Flood Level (m)
			Left (m)	Right (m)		
Athabasca River	1	8340	106.92	469.50	Inundation Limit (1)	515.17
Athabasca River	2	7941	144.08	532.66	Inundation Limit (1)	515.10
Athabasca River	3	7550	135.17	504.82	Previous Floodway	515.02
Athabasca River	4	7109	121.41	496.07	Previous Floodway	514.95
Athabasca River	5	6640	144.86	519.78	Previous Floodway	514.87
Athabasca River	6	6112	40.97	318.64	Previous Floodway	514.72
Athabasca River	7	5915	47.30	302.25	Previous Floodway	514.64
Athabasca River	8	5756	41.99	324.81	Previous Floodway	514.59
Athabasca River	9	5352	151.61	501.43	Previous Floodway	514.46
Athabasca River	10	4994	201.72	436.94	Previous Floodway	514.34
Athabasca River	11	4829	269.33	489.28	Previous Floodway	514.27
Athabasca River	12	4796	258.19	469.74	Previous Floodway	514.26
Athabasca River	13	4468	245.16	493.79	Previous Floodway (Left) - Inundation Limit (2) (Right)	514.12
Athabasca River	14	4043	338.78	619.53	Previous Floodway	513.94
Athabasca River	15	3542	598.68	1023.40	Previous Floodway	513.79
Athabasca River	16	3087	574.24	986.87	Previous Floodway (Left) - Inundation Limit (2) (Right)	513.68
Athabasca River	17	2636	371.58	733.26	Previous Floodway (Left) - Inundation Limit (2) (Right)	513.54
Athabasca River	18	2182	85.26	423.41	Inundation Limit (1) (Left) – 1 m Depth (Right)	513.38
Athabasca River	19	1764	94.08	441.88	Inundation Limit (1) (Left) – 1 m Depth (Right)	513.20
Athabasca River	20	1329	144.39	571.80	Inundation Limit (1)	512.98

Notes: (1) No viable flood fringe.
(2) Previous floodway is outside the inundation limit.

Table 7-5: Ice Jam Floodway Limits and Design Flood Levels along Muskeg Creek

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Ice Jam Design Flood Level (m)
			Left (m)	Right (m)		
Muskeg Creek	47	973	136.75	148.04	Inundation Limit (3)	514.77
Muskeg Creek	48	943	152.58	165.49	Inundation Limit (3)	514.76
Muskeg Creek	49	934	157.52	169.65	Inundation Limit (3)	514.76
Muskeg Creek	50	825	182.83	203.06	Previous Floodway	514.78
Muskeg Creek	51	718	255.29	270.00	1 m depth	514.80
Muskeg Creek	52	680	259.86	277.64	Previous Floodway (Left) – 1 m Depth (Right)	514.80
Muskeg Creek	53	610	259.21	-	Inundation Limit (3) (Left) - Mixed (2) (Right)	514.81
Muskeg Creek	54	463	-	-	Mixed (2)	514.80
Muskeg Creek	55	357	-	77.21	Mixed (2) (Left) – Previous Floodway (Right)	514.77
Muskeg Creek	56	249	-	63.54	Mixed (2) (Left) – Previous Floodway (Right)	514.74
Muskeg Creek	57	141	-	145.26	Mixed (2) (Left) – Previous Floodway (Right)	514.73

Notes: (1) No viable flood fringe.
(2) Floodway extents pass the cross section line.
(3) Previous floodway is outside the inundation limit.

Table 7-6: Ice Jam Floodway Limits and Design Flood Levels along the Tawatinaw River

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Ice Jam Design Flood Level (m)
			Left (m)	Right (m)		
Tawatinaw River	75	2450	212.13	218.52	Inundation Limit (1)	514.55
Tawatinaw River	76	2270	249.19	274.10	Inundation Limit (4)	514.55
Tawatinaw River	77	1973	270.11	300.70	Previous Floodway (Left) - Inundation Limit (4) (Right)	514.55
Tawatinaw River	78	1706	122.12	183.70	Inundation Limit (1) (Left) – Previous Floodway (Right)	514.55
Tawatinaw River	79	1484	111.84	174.36	Inundation Limit (1) (Left) – Previous Floodway (Right)	514.55
Tawatinaw River	80	1278	148.97	233.37	Previous Floodway	514.55
Tawatinaw River	81	1000	78.21	200.37	Inundation Limit (1) (Left) – Previous Floodway (Right)	514.55
Tawatinaw River	82	657	30.57	139.03	Inundation Limit (1) (Left) – Previous Floodway (Right)	514.55
Tawatinaw River	83	607	25.88	110.69	Previous Floodway	514.55
Tawatinaw River	84	599	26.03	119.30	Previous Floodway	514.55
Tawatinaw River	85	457	196.34	308.23	Inundation Limit (1)	514.55

Table 7-6: Ice Jam Floodway Limits and Design Flood Levels along the Tawatinaw River

River	Cross Section	River Station	Floodway Limits		Floodway Determination Criteria	100-Year Ice Jam Design Flood Level (m)
			Left (m)	Right (m)		
Tawatinaw River	86	383	315.61	346.61	Main Channel (4) (Left) - Inundation Limit (1) (Right)	514.55
Tawatinaw River	87	355	373.89	411.31	Mixed (2) (Left) - Inundation Limit (1) (Right)	514.55
Tawatinaw River	88	257	18.40	73.96	Previous Floodway (Left) - Inundation Limit (1) (Right)	514.54
Tawatinaw River	89	153	-	77.71	Mixed (3) (Left) – Previous Floodway (Right)	514.51

Notes: (1) Previous floodway is outside the inundation limit.
(2) To balance other criteria, or to maintain smooth floodway.
(3) Floodway extents pass the cross section line.
(4) Main channel shift outside of the previous floodway.

7.4.2 Ice Jam Design Flood Profile

The ice jam design flood water levels along the Athabasca River, Muskeg Creek and the Tawatinaw River are identical to those presented in Appendix I and in Table 7-4 (Athabasca River), Table 7-5 (Muskeg Creek) and Table 7-6 (Tawatinaw River).

7.4.3 Ice Jam Floodway Criteria Maps

Floodway criteria maps are a tool for documenting the basis for the location of the boundary between the floodway, high hazard flood fringe and flood fringe, and illustrate the following:

- the location and extent of all ice jam hydraulic model cross sections;
- the inundation extent of the design flood, showing areas of dry ground;
- areas where design flood depths are 1 m or greater; and
- the proposed floodway boundary, as well as the associated floodway stations corresponding to the alternate floodway determination criteria.

The floodway criteria maps were developed considering the criteria listed above. To develop the depth isolines, water depth rasters were first created. Following this, the 1 m depth boundaries were identified for each cross section. The floodway criteria lines were then developed. The floodway lines were digitized electronically using ArcMap tools, by inspection of the computed flood extents, and flow depths. It should be noted that delineation of the floodway boundary is not a fully automatic procedure, and in some cases, manual interpretation and judgement was needed to project these water levels onto the surface topography, and to ensure a smooth hydraulic transition between cross sections.

The final ice jam flood criteria maps are provided in Appendix K of this report. The maps illustrate the location of the floodway boundaries and limits of the flood fringe corresponding to the 100-year ice jam event.

7.4.4 Ice Jam Flood Hazard Maps

The ice jam flood hazard maps display the areas in the floodway, high hazard flood fringe and flood fringe for the design ice jam flood along the Athabasca River. The ice jam floodway was determined as part of the ice jam floodway criteria mapping.

The mapping scale is 1:10,000. The maps were prepared using the local 3-Degree Transverse Mercator (3TM) zone and the Canadian Spatial Reference System North American Datum of 1983 (NAD83 CSRS) coordinate system and datum. The maps are provided in Appendix K.

7.4.5 Ice Jam Design Flood Grids

The water surface elevation and flood depth grids for ice jam conditions were created using the same method as described in Section 7.3.5. for open water.

All GIS data were created in ArcGIS Version 10.7 compatible format in the native study coordinate system [Canadian Spatial Reference System, North American Datum of 1983 (CSRS NAD83), Epoch 2002 and 3-Degree Transverse Mercator projection with the Central Meridian of 114° (3TM 114)].

7.5 Governing Design Flood Hazard Determination

7.5.1 Governing Design Flood

Both the open water and ice jam flood events were considered in the development of the governing design flood hazard map. The development of open water flood profiles is described in detail in the Section 7.3 and development of ice jam related flood profiles in Section 7.4.

Either open water or ice jam flooding is the governing design flood hazard scenario along the Athabasca River study reach. If ice jam flooding is more severe than open water flooding in the Athabasca River, the ice jam flood levels may also govern along the lower reaches of these tributary streams because of the backwater effects.

In general, the higher design water level between open water and ice jam conditions was used as a governing design flood level. However, exceptions to the application of this general rule were made for ensuring consistency to upstream or downstream criteria where the differences of design water level between open water and ice jam conditions are small.

7.5.2 Comparison of Open Water and Ice Jam Design Flood Level

Table 7-7 and Figure 7-1 compare the Athabasca River flood levels during the open water and ice jam flood events. In addition, Figure 7-1 compares the 100-year design flood water levels to the 50-year and 200-year flood levels. As shown, open water design flood levels exceed the ice jam design water levels on the Athabasca River for the reach upstream of Muskeg Creek Confluence. The ice jam water levels for the reach downstream of Muskeg Creek confluence on the Athabasca River are higher than the open water levels.

Table 7-7: Open Water and Ice Jam Design Flood Levels along the Athabasca River

River	Cross Section	River Station (m)	Open Water Design Flood Level (m)	Ice Jam Design Flood Level (m)	Difference (Ice Jam - Open Water) (m)
Athabasca River	1	8340	515.30	515.17	-0.13
Athabasca River	2	7941	515.23	515.10	-0.13
Athabasca River	3	7550	515.15	515.02	-0.13
Athabasca River	4	7109	515.07	514.95	-0.12
Athabasca River	5	6640	514.99	514.87	-0.12
Athabasca River	6	6112	514.61	514.72	0.11
Athabasca River	7	5915	514.47	514.64	0.17
Athabasca River	8	5756	514.46	514.59	0.13
Athabasca River	9	5352	514.38	514.46	0.08
Athabasca River	10	4994	514.08	514.34	0.26

Table 7-7: Open Water and Ice Jam Design Flood Levels along the Athabasca River

River	Cross Section	River Station (m)	Open Water Design Flood Level (m)	Ice Jam Design Flood Level (m)	Difference (Ice Jam - Open Water) (m)
Athabasca River	11	4829	513.87	514.27	0.40
Athabasca River	12	4796	513.78	514.26	0.48
Athabasca River	13	4468	513.65	514.12	0.47
Athabasca River	14	4043	513.48	513.94	0.46
Athabasca River	15	3542	513.53	513.79	0.26
Athabasca River	16	3087	513.42	513.68	0.26
Athabasca River	17	2636	513.24	513.54	0.30
Athabasca River	18	2182	513.06	513.38	0.32
Athabasca River	19	1764	512.87	513.20	0.33
Athabasca River	20	1329	512.85	512.98	0.13

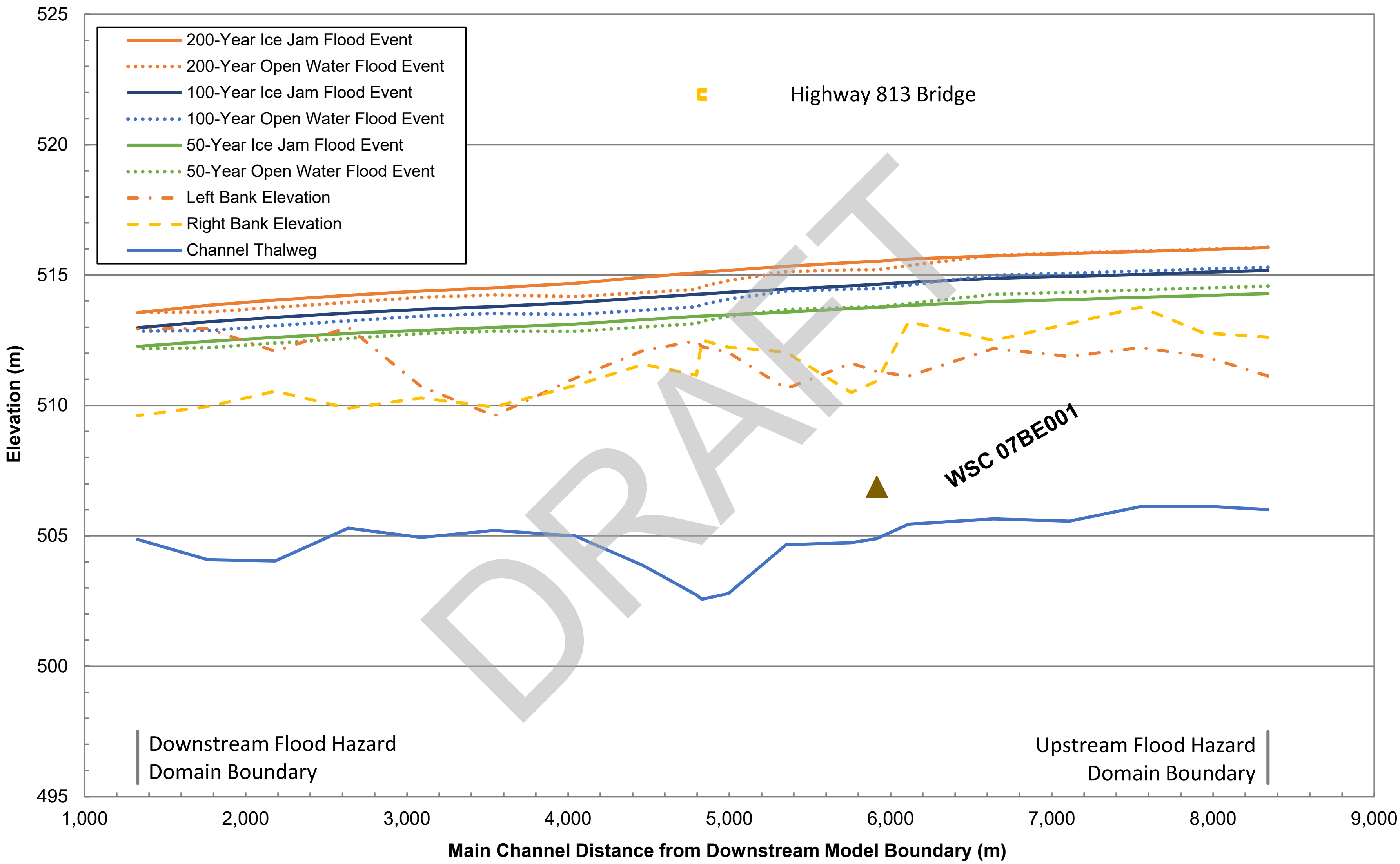

Figure 7-1: Comparison of Water Surface Profiles for the 50-, 100-, and 200-year Ice Jam and Open Water Flood Events

Table 7-8 compares Muskeg Creek flood levels during the open water and ice jam flood events. Table 7-9 compares Tawatinaw River flood levels during the open water and ice jam flood events. The design water levels on Muskeg Creek between its confluence and Cross Section 54 are governed by ice jam flooding. The remaining areas of Muskeg Creek are governed by open water flooding. The design water levels on the Tawatinaw River between its confluence and Cross Section 88 are governed by ice jam flooding. The remaining areas of the Tawatinaw River are governed by open water flooding. For Muskeg Creek and the Tawatinaw River, cross sections are presented where ice jam flooding on the Athabasca River could affect the areas adjacent to their confluences with Athabasca River.

Table 7-8: Open Water and Ice Jam Design Flood Levels along Muskeg Creek

River	Cross Section	River Station (m)	Open Water Design Flood Level (m)	Ice Jam Design Flood Level (m)	Difference (Ice Jam - Open Water) (m)
Muskeg Creek	52	680	514.83	514.80	-0.03
Muskeg Creek	53	610	514.84	514.81	-0.03
Muskeg Creek	54	463	514.80	514.80	0.00
Muskeg Creek	55	357	514.76	514.77	0.01
Muskeg Creek	56	249	514.70	514.74	0.04
Muskeg Creek	57	141	514.66	514.73	0.07

Table 7-9: Open Water and Ice Jam Design Flood Levels along the Tawatinaw River

River	Cross Section	River Station (m)	Open Water Design Flood Level (m)	Ice Jam Design Flood Level (m)	Difference (Ice Jam - Open Water) (m)
Tawatinaw River	75	2450	516.73	514.55	-2.18
Tawatinaw River	76	2270	515.59	514.55	-1.04
Tawatinaw River	77	1973	515.18	514.55	-0.63
Tawatinaw River	78	1706	515.02	514.55	-0.47
Tawatinaw River	79	1484	514.93	514.55	-0.38
Tawatinaw River	80	1278	514.89	514.55	-0.34
Tawatinaw River	81	1000	514.86	514.55	-0.31
Tawatinaw River	82	657	514.83	514.55	-0.28
Tawatinaw River	83	607	514.82	514.55	-0.27
Tawatinaw River	84	599	514.82	514.55	-0.27
Tawatinaw River	85	457	514.81	514.55	-0.26
Tawatinaw River	86	383	514.80	514.55	-0.25
Tawatinaw River	87	355	514.80	514.55	-0.25
Tawatinaw River	88	257	514.40	514.54	0.14
Tawatinaw River	89	153	514.39	514.51	0.12

7.5.3 Governing Design Flood Profile

In general, the higher design water levels between open water and ice jam conditions are used as governing design flood levels to represent the higher river hazard. In this study, the Athabasca River reach upstream of the Muskeg Creek confluence, where the ice jam design water levels were used as the governing design flood levels, although the open water design flood levels exceed the ice jam design flood levels for the following reasons:

- The difference of design water levels between open water and ice jam conditions is very small (<0.13 m) for this short reach, and the Highway 813 bridge was not simulated in the ice jam hydraulic model;
- The difference in lateral extent of flooding in this short reach between open water and ice jam design floods is small; and
- Considering that the remaining Athabasca River study reach is governed by ice jam design flood, the ice jam design flood levels were selected as the governing design flood levels for this short reach upstream of the Muskeg Creek based on our communications with AEP specialists.

For Muskeg Creek and the Tawatinaw River, the open water design flood levels were used as the governing design flood levels for most cross sections except at their confluence areas where ice jam flooding on Athabasca River would cause higher water levels.

The final governing design flood levels are presented in Table 7-10 to Table 7-12. The ice jam flood is the governing event for the entire study reach along the Athabasca River. For Muskeg Creek and the Tawatinaw River, the open water flood is the governing event except for the immediate areas adjacent to their confluences with the Athabasca River. The final floodway extent stations relative to the cross-sectional length in the model are also summarized in Table 7-10 to Table 7-12.

Table 7-10: Governing Design Flood Levels along the Athabasca River

River	Cross Section	River Station (m)	Governing Design Flood Level (m)	Governing Flood	Floodway Limit (m)		Notes
					Left	Right	
Athabasca River	1	8340	515.17	Ice Jam(1)	106.92	469.50	Inundation Limit (2)
Athabasca River	2	7941	515.10	Ice Jam(1)	144.08	532.66	Inundation Limit (2)
Athabasca River	3	7550	515.02	Ice Jam(1)	135.17	504.82	Previous Floodway
Athabasca River	4	7109	514.95	Ice Jam(1)	121.41	496.07	Previous Floodway
Athabasca River	5	6640	514.87	Ice Jam(1)	144.86	519.78	Previous Floodway
Athabasca River	6	6112	514.72	Ice Jam	40.97	318.64	Previous Floodway
Athabasca River	7	5915	514.64	Ice Jam	47.30	302.25	Previous Floodway
Athabasca River	8	5756	514.59	Ice Jam	41.99	324.81	Previous Floodway
Athabasca River	9	5352	514.46	Ice Jam	151.61	501.43	Previous Floodway
Athabasca River	10	4994	514.34	Ice Jam	201.72	436.94	Previous Floodway
Athabasca River	11	4829	514.27	Ice Jam	269.33	489.28	Previous Floodway
Athabasca River	12	4796	514.26	Ice Jam	258.19	469.74	Previous Floodway
Athabasca River	13	4468	514.12	Ice Jam	245.16	493.79	Previous Floodway (Left) - Inundation Limit (3) (Right)
Athabasca River	14	4043	513.94	Ice Jam	338.78	619.53	Previous Floodway
Athabasca River	15	3542	513.79	Ice Jam	598.68	1023.40	Previous Floodway
Athabasca River	16	3087	513.68	Ice Jam	574.24	986.87	Previous Floodway (Left) - Inundation Limit (3) (Right)
Athabasca River	17	2636	513.54	Ice Jam	371.58	733.26	Previous Floodway (Left) - Inundation Limit (3) (Right)
Athabasca River	18	2182	513.38	Ice Jam	85.26	423.41	Inundation Limit (2) (Left) – 1 m Depth (Right)
Athabasca River	19	1764	513.20	Ice Jam	94.08	441.88	Inundation Limit (2) (Left) - 1 m Depth (Right)
Athabasca River	20	1329	512.98	Ice Jam	144.39	571.80	Inundation Limit (2)

Notes: (1) The ice jam design water levels were used as the governing design flood levels although the open water design flood levels are slightly higher than the ice jam design flood levels based on communications with AEP specialists.
(2) No viable flood fringe.
(3) Previous floodway is outside the inundation limit.

Table 7-11: Governing Design Flood Levels along Muskeg Creek

River	Cross Section	River Station (m)	Governing Design Flood Level (m)	Governing Flood	Floodway Limit (m)		Notes
					Left	Right	
Muskeg Creek	24	7594	546.31	Open Water	63.55	84.60	Main Channel (Left) - Inundation Limit (1) (Right)
Muskeg Creek	25	7286	545.14	Open Water	109.10	126.68	Channel Bank (Left) - Inundation Limit (1) (Right)
Muskeg Creek	26	6935	543.65	Open Water	158.85	173.92	1 m/s Velocity (Left) - Inundation Limit (1) (Right)
Muskeg Creek	27	6532	541.65	Open Water	61.69	75.88	Main Channel
Muskeg Creek	28	6009	540.00	Open Water	207.52	223.88	Main Channel
Muskeg Creek	29	5401	537.89	Open Water	128.53	147.37	Main Channel (Left) - Inundation Limit (1) (Right)
Muskeg Creek	30	4921	536.22	Open Water	108.20	124.88	Main Channel
Muskeg Creek	31	4806	535.79	Open Water	107.20	121.88	Main Channel
Muskeg Creek	32	4382	534.35	Open Water	78.56	95.02	Inundation Limit (1) (Left) - Main Channel (Right)
Muskeg Creek	33	3571	531.58	Open Water	208.20	226.75	Inundation Limit (1)
Muskeg Creek	34	3153	530.10	Open Water	37.50	53.18	Main Channel (Left) - Inundation Limit (1) (Right)
Muskeg Creek	35	3049	529.64	Open Water	49.12	67.16	Main Channel
Muskeg Creek	36	3043	529.56	Open Water	55.76	71.28	Main Channel
Muskeg Creek	37	2993	529.22	Open Water	98.81	117.58	Inundation Limit (1)
Muskeg Creek	38	2891	528.65	Open Water	133.80	147.78	Inundation Limit (1) (Left) - Main Channel (Right)
Muskeg Creek	39	2728	527.75	Open Water	18.74	33.39	Inundation Limit (1) (Left) - Main Channel (Right)
Muskeg Creek	40	2628	527.02	Open Water	62.35	78.80	Inundation Limit (1)
Muskeg Creek	41	2116	523.20	Open Water	125.24	143.72	Inundation Limit (1)
Muskeg Creek	42	1722	520.65	Open Water	297.23	312.85	Inundation Limit (1)
Muskeg Creek	43	1406	518.43	Open Water	122.15	138.53	Inundation Limit (1) (Left) - Main Channel (Right)
Muskeg Creek	44	1383	518.24	Open Water	126.66	138.71	Main Channel (Left) - Inundation Limit (1) (Right)
Muskeg Creek	45	1374	518.14	Open Water	125.23	137.15	Main Channel
Muskeg Creek	46	1162	515.97	Open Water	156.03	166.79	Inundation Limit (1)
Muskeg Creek	47	973	515.41	Open Water	136.17	149.30	Inundation Limit (3)
Muskeg Creek	48	943	515.38	Open Water	151.77	165.99	Inundation Limit (3) (Left) – Previous Floodway (Right)
Muskeg Creek	49	934	515.34	Open Water	156.15	170.60	Inundation Limit (3) (Left) – Previous Floodway (Right)
Muskeg Creek	50	825	515.28	Open Water	182.83	203.06	Previous Floodway
Muskeg Creek	51	718	515.24	Open Water	251.94	275.90	Inundation Limit (1)
Muskeg Creek	52	680	514.83	Open Water	259.86	277.64	Previous Floodway (Left) – 1 m Depth (Right)
Muskeg Creek	53	610	514.84	Open Water	259.21	-	Inundation Limit (3) (Left) - Mixed (2) (Right)

Table 7-11: Governing Design Flood Levels along Muskeg Creek

River	Cross Section	River Station (m)	Governing Design Flood Level (m)	Governing Flood	Floodway Limit (m)		Notes
					Left	Right	
Muskeg Creek	54	463	514.80	Ice Jam	-	-	Mixed (2)
Muskeg Creek	55	357	514.77	Ice Jam	-	77.21	Mixed (2) (Left) - Previous Floodway (Right)
Muskeg Creek	56	249	514.74	Ice Jam	-	63.54	Mixed (2) (Left) - Previous Floodway (Right)
Muskeg Creek	57	141	514.73	Ice Jam	-	145.26	Mixed (2) (Left) - Previous Floodway (Right)

Notes: (1) No viable flood fringe.
(2) Floodway extents pass the cross section line.
(3) Previous floodway is outside the inundation limit.

Table 7-12: Governing Design Flood Levels along the Tawatinaw River

River	Cross Section	River Station (m)	Governing Design Flood Level (m)	Governing Flood	Floodway Limit (m)		Notes
					Left	Right	
Tawatinaw River	58	6230	523.05	Open Water	126.59	157.73	Main Channel (Left) - Mixed (2) (Right)
Tawatinaw River	59	5962	522.80	Open Water	143.73	169.31	1 m Depth
Tawatinaw River	60	5944	522.79	Open Water	152.87	176.58	1 m Depth
Tawatinaw River	61	5679	522.48	Open Water	174.97	198.27	1 m Depth
Tawatinaw River	62	5485	522.18	Open Water	159.46	182.72	1 m Depth
Tawatinaw River	63	5274	521.97	Open Water	92.07	112.99	Main Channel
Tawatinaw River	64	4893	521.64	Open Water	170.53	258.80	1 m Depth (Left) - Main Channel (Right)
Tawatinaw River	65	4615	521.35	Open Water	116.49	148.02	Mixed (2) (Left) - Main Channel (Right)
Tawatinaw River	66	4468	521.20	Open Water	170.71	196.15	Main Channel
Tawatinaw River	67	4303	520.99	Open Water	150.48	179.10	1 m Depth
Tawatinaw River	68	4196	520.85	Open Water	118.19	140.33	Main Channel
Tawatinaw River	69	4033	520.66	Open Water	131.74	184.91	Mixed (2) (Left) - Main Channel (Right)
Tawatinaw River	70	3805	520.36	Open Water	207.95	232.63	Mixed (2) (Left) - Inundation Limit (1) (Right)
Tawatinaw River	71	3393	519.37	Open Water	132.90	164.75	Previous Floodway (Left) - Inundation Limit (4) (Right)
Tawatinaw River	72	3179	518.78	Open Water	259.92	293.67	Mixed (2) (Left) - 1 m Depth (5) (Right)
Tawatinaw River	73	2884	518.14	Open Water	188.82	215.95	Previous Floodway

Table 7-12: Governing Design Flood Levels along the Tawatinaw River

River	Cross Section	River Station (m)	Governing Design Flood Level (m)	Governing Flood	Floodway Limit (m)		Notes
					Left	Right	
Tawatinaw River	74	2709	517.73	Open Water	201.34	226.19	Inundation Limit (4) (Left) – Previous Floodway (Right)
Tawatinaw River	75	2450	516.73	Open Water	210.27	223.64	Previous Floodway
Tawatinaw River	76	2270	515.59	Open Water	248.64	274.99	Previous Floodway (Left) - 1 m Velocity (5) (Right)
Tawatinaw River	77	1973	515.18	Open Water	270.06	301.77	Previous Floodway (Left) - Inundation Limit (5) (Right)
Tawatinaw River	78	1706	515.02	Open Water	117.05	183.70	Previous Floodway
Tawatinaw River	79	1484	514.93	Open Water	110.87	174.36	Previous Floodway
Tawatinaw River	80	1278	514.89	Open Water	148.96	233.37	Previous Floodway
Tawatinaw River	81	1000	514.86	Open Water	77.91	200.37	Inundation Limit (4) (Left) – Previous Floodway (Right)
Tawatinaw River	82	657	514.83	Open Water	30.16	139.03	Inundation Limit (4) (Left) – Previous Floodway (Right)
Tawatinaw River	83	607	514.82	Open Water	25.88	110.69	Previous Floodway
Tawatinaw River	84	599	514.82	Open Water	26.03	119.30	Previous Floodway
Tawatinaw River	85	457	514.81	Open Water	181.11	309.09	Previous Floodway (Left) - Inundation Limit (4) (Right)
Tawatinaw River	86	383	514.80	Open Water	315.61	347.34	Main Channel (5) (Left) - Inundation Limit (4) (Right)
Tawatinaw River	87	355	514.80	Open Water	373.89	411.31	Mixed (2) (Left) - Inundation Limit (4) (Right)
Tawatinaw River	88	257	514.54	Ice Jam	18.40	73.96	Previous Floodway (Left) - Inundation Limit (4) (Right)
Tawatinaw River	89	153	514.51	Ice Jam	-	77.71	Mixed (3) (Left) - Previous Floodway (Right)

Notes: (1) No viable flood fringe.
(2) To balance other criteria, or to maintain smooth floodway.
(3) Floodway extents pass the cross section line.
(4) Previous floodway is outside the inundation limit.
(5) Main channel shift outside of the previous floodway.

7.5.4 Governing Design Flood Hazard Maps

7.5.4.1 Flood Mapping Methodology

Governing design flood mapping was typically based on either open water or ice jam flood hazards, but a combination scenario was also required at some locations. In that case hybrid mapping was necessary to merge polygons from different floodway criteria maps with only one flood scenario governing along one sub-reach.

Governing flood hazard maps were prepared and are provided in Appendix L of this report. These maps were developed in accordance with the provincial standards, with the exception of the most upstream reach of the Athabasca River, as discussed in Section 7.5.3. The maps illustrate the locations of the floodway boundaries and limits of the flood fringe corresponding to the governing event (open water or ice jam) in each river reach. These maps include the following:

- the extent of the governing design flood;
- the floodway boundary for the governing design flood;
- the location and extent of all cross sections used in the HEC-RAS model with appropriate labels;
- background aerial imagery collected for the study in May 2019; and
- roads, bridges and flood control structures.

The governing floodway criteria maps were prepared in a scale of 1:10,000 in the study datum and coordinate system (CSRS NAD83, 114° 3TM). The maps are provided in Appendix L.

7.5.4.2 Areas in the Floodway

The following areas are in the governing floodway:

- the main channels of the Athabasca River, Muskeg Creek and the Tawatinaw River;
- a walkway on the right bank of the Athabasca River between the Muskeg Creek confluence and the Tawatinaw River confluence;
- a wetland area on right side of the Tawatinaw River upstream of the Highway 55 bridge crossing; and
- the confluence areas of Muskeg Creek and the Tawatinaw River.

7.5.4.3 Areas in the Flood Fringe

The development areas in the flood fringe within the study area are listed below.

Athabasca River

- Portions of parking lot of the Independent Grocery Store on the right floodplain;
- Athabasca Home Hardware Building Centre on the right floodplain; and
- The 51st Avenue on the right floodplain.

Muskeg Creek

- Small portions on both sides of upstream of Highway 2 Culvert, including 57 Street and Muskeg Creek trails; and
- Small portions on the right side downstream of Highway 2 Culvert.

Tawatinaw River

- Small portions on the left bank upstream of Highway 55 Bridge.

7.5.5 Governing Design Flood Grids

Water Surface Elevation Grids

The water surface elevation grid was created by converting the water surface elevation TIN into a raster file with the same resolution (0.5 m) and alignment as the DTM. The water surface elevation raster was then clipped to the directly-inundated areas and the areas of potential flood control structure failures.

Flood Depth Grids

The flood depth grid was created by subtracting the water surface elevation grid from the DTM. The flood depth grid has the same resolution (0.5 m) and alignment as the DTM. The extent of the depth grid is limited to the directly-inundated areas and the areas of potential flood control structure failures.

General Comments

Water surface elevation TINs cover all areas between cross sections and in special inundation areas within the study area, including dry areas. Corresponding rasters were clipped to the inundation extents of the design flood.

All GIS data were created in ArcGIS Version 10.7 compatible format in the native study coordinate system [Canadian Spatial Reference System, North American Datum of 1983 (CSRS NAD83), Epoch 2002 and 3-Degree Transverse Mercator projection with the Central Meridian of 114° (3TM 114)].

8.0 POTENTIAL CLIMATE CHANGE IMPACTS

8.1 Open Water Floods

A cursory examination of potential increases in 100-year design water levels associated with climate change were performed to understand the possible impacts of climate change on flood levels. The effect of the 100-year flood conditions more severe than the baseline was assessed under two additional flow scenarios:

1) 100-year open water discharge +10%; and

2) 100-year open water discharge +20%.

No hydraulic modelling parameters were varied other than discharges under the open water conditions. Water level profiles were produced along the study reaches for all two additional flow scenarios. If a lower flow profile resulted in higher water levels, or if numerical instabilities or unreasonable results occurred, model parameters were adjusted to resolve these issues. The water level differences compared to the baseline 100-year open water discharge were calculated and summarized below. These water level differences were identified as potential “freeboards” that could be applied to the design water levels to account for flow changes that could result from climate change.

- For the Athabasca River reach, the average increases in open water flood levels are 0.44 m for a 10 percent increase in flow, and 0.87 m for a 20 percent increase in flow.
- For the Muskeg Creek reach, the average increases in open water flood levels are 0.22 m for a 10 percent increase in flow, and 0.43 m for a 20 percent increase in flow.
- For the Tawatinaw River reach, the average increases in open water flood levels are 0.26 m for a 10 percent increase in flow, and 0.52 m for a 20 percent increase in flow.

The above analyses are not based on a regional climate change impact assessment but on a simplified assumption that climate change will result in increased flood peak flows. The presented values can be viewed as a general range of potential climate change “freeboard” that could be considered in addition to the computed design flood water levels.

The simulated two additional climate-affected flood profiles along the Athabasca River reach, are presented in Figure M-1 in Appendix E. The simulated two additional climate-affected open water flood water levels at individual cross sections are compared to the baseline 100-year open water discharge in Table M-1 in Appendix M.

The simulated two additional climate-affected flood profiles along the Muskeg Creek and Tawatinaw River study reaches are presented in Figure M-2 (Muskeg Creek) and Figure M-3 (Tawatinaw River) in Appendix M. The simulated two additional climate-affected open water flood water levels at individual cross sections are compared to the baseline 100-year open water discharge in Table M-2 (Muskeg Creek) and Table M-3 (Tawatinaw River) in Appendix M.

8.2 Ice Jam Floods

Ice jam floods under climate change scenarios may be influenced by multiple factors including winter temperature regime and season duration, snowpack depth, ice thickness and other factors, introducing challenges to projecting the effects of climate change on ice jam frequency and severity. Turcotte et al. (2019) conducted a comprehensive review of the state of knowledge and research approaches addressing climate change impacts on breakup ice jams in Canada.

Turcotte et al. (2019) summarized recent changes in ice jam conditions in the Athabasca River basin by recognizing:

- warmer winters and earlier spring breakup;
- stable snow water equivalent;
- reduced summer precipitation, increasing the capacity of the landscape to absorb spring snowmelt and thus reducing the potential for rapid snowmelt runoff; and
- general downward trend in maximum spring ice-induced water levels, though this was influenced by low water years from 1998 to 2012 and did not consider the extreme flooding of spring 2020.

Eum et al. (2017) projected an earlier freshet in the middle reach of the Athabasca, by up to 12 days in the 2080s, accompanied by increased spring flows, while at Fort McMurray, Das et al. (2017) projected an earlier freeze and later breakup, extending the ice cover season by up to 12 days and increasing the likelihood of a thermal breakup and reduction in severity of ice jams.

Turcotte et al. (2019) identified these differing conclusions, and examined the various factors affecting ice jam frequency and severity. Turcotte et al. (2019) concluded that more research was required to define future trends in ice jam frequency and severity.

Based on the review of applicable literature, it would be imprudent to provide predictions of potential future ice jam flood water levels under climate change scenarios. As for the case of open water floods, the variability of modeled discharges under historical climate conditions may be used as an indicator of potential "freeboards" that could be applied to the design water levels to account for flow changes that could result from climate change.

The baseline 200-year ice jam flood water levels are used as possible 100-year ice jam flood water levels along the Athabasca River reach due to climate change. The average difference between the baseline 200-year and 100-year ice jam water flood levels is 0.80 m. The simulated ice jam climate-affected flood profiles along the Athabasca River reach, are presented in Figure I-1 in Appendix I. The simulated ice jam climate-affected flood water levels at individual cross sections are compared to the baseline 100-year open water discharge in Table I-1 in Appendix I.

9.0 CONCLUSIONS

9.1 Survey and Base Data Collection

Topographic, bathymetric, and supporting base data required for this study were collected in accordance with the requirements by AEP. The following conclusions are made:

- *River Cross Section Surveys* – Cross section survey data collected for this study in May/June 2019 meet the current study requirements with regard to cross section spacing and alignment, extents of cross sections on the floodplains, labeling of survey points, and data accuracy.
- *Hydraulic and Flood Control Structure Surveys* – Hydraulic structure survey data collected in May/June 2019 meet the study requirements and include the necessary details for the hydraulic modelling. It was confirmed that there is no flood control structure in the study area and therefore no flood control structure survey was conducted.
- *Digital Terrain Model* – The differences in elevation between the selected survey points and the DTM data are considered to be within an acceptable range. Therefore, the DTM is considered suitable for overbank cross section data extraction and flood mapping.

9.2 Open Water Hydrology Assessment

The results of the open water hydrology assessment completed in this study support the following conclusions:

- The flood frequency estimates obtained in this study are the most up-to-date for the Athabasca River at Athabasca, Muskeg Creek, and the Tawatinaw River. These estimates provide the updated flood hydrology information as inputs to the other components of the study (e.g., hydraulic modelling). Table 3-1 summarizes the estimates of flood peak discharges for various return periods ranging from 2 to 1,000 years, and the 95% upper and lower confidence intervals.
- This study includes preliminary estimates of the annual maximum instantaneous discharges in 2017 and 2018. Inclusion of the additional discharge information increases the sample size for the flood frequency analyses and reliability of the resulting flood frequency estimates.
- The length of time period of the recorded flood flow data available and used in the flood frequency analyses for the Athabasca River at Athabasca, is 100 years. Therefore, there is uncertainty with flood frequency estimates for return periods greater than 100 years.

9.3 Open Water Hydraulic Modelling

9.3.1 Model Calibration

The HEC-RAS model, set up for the study reaches of the Athabasca River, Muskeg Creek and the Tawatinaw River, was calibrated based on the available low flow, high flow, and rating curve data. The calibrated HEC-RAS model can be reliably used in this study for simulating various flood events with return periods ranging from 2 to 1,000 years.

River channel Manning's n roughness coefficient is the main model parameter used in calibrating the HEC-RAS model. The calibrated river channel Manning's n values for the low flow conditions on the Athabasca River, Muskeg Creek and the Tawatinaw River are generally higher than those for the high flow conditions.

The calibrated channel Manning's *n* values for the high flow conditions is 0.026 along the Athabasca River study reach. No high flow data were available for calibrating the hydraulic model for Muskeg Creek or the Tawatinaw

River. Therefore, a representative Manning's *n* value of 0.050 was estimated for the Muskeg Creek and Tawatinaw River channels. These Manning's n values are within the typical range of roughness values for similar streams (Chow 1959).

9.3.2 Model Sensitivity

A model sensitivity was evaluated using the 100-year flood simulation results. The results of the sensitivity analysis show the following:

- Variation of the river channel roughness values has a much higher influence on the simulated flood levels than variation of the floodplain roughness values; and
- On average, the 100-year flood levels are estimated to be within a range of ±0.43 m of the simulated values along the Athabasca River, ±0.18 m along Muskeg Creek, and ±0.21 m along the Tawatinaw River.

9.3.3 Flood Profiles

The calibrated HEC-RAS model provides a reliable tool for simulating the flood profiles of the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events in the study area.

9.4 Ice Jam Modelling

9.4.1 Model Calibration

The HEC-RAS model for the study reaches of the Athabasca River was modified to represent ice jam conditions, and was calibrated based on plausible estimates of ice jam model parameters. The calibrated HEC-RAS model can be reliably used in this study for simulating various flood events with return periods ranging from 50 to 200 years.

The calibrated channel Manning's *n* value for ice jam conditions is 0.033 along the Athabasca River study reach, corresponding to the open water low flow value, with an ice underside Manning's n value of 0.060. The ice jam model used default values of ice cover specific gravity, internal friction angle, porosity, ratio of longitudinal to lateral stress and jam strength coefficient. The limiting mean flow velocity under the ice jam was set to 2.0 m/s, and it was assumed that there was no flow on the floodplain during the ice jam.

9.4.2 Model Sensitivity

Sensitivity tests were performed to evaluate the impact on simulated water levels along the Athabasca River based on variation of model setup and assumptions. The model was tested for changes in floodplain conveyance, extension of the downstream model boundary, and addition of interpolated cross sections, and it was demonstrated that model results were insensitive to these changes.

9.4.3 Flood Profiles

The calibrated HEC-RAS model provides a reliable tool for simulating the ice jam flood profiles of the 50-, 100- and 200-year flood events in the study area.

9.5 Flood Inundation Mapping

9.5.1 Open Water Flood Inundation Mapping

The calibrated HEC-RAS model and the LiDAR DTM provided a good basis for simulating the flood levels and preparing the inundation maps for the 13 open water flood events (i.e. 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year open water floods), including direct flood inundation areas and other indirect flood inundation areas.

Based on the simulation results, the main areas to be affected by open water flooding have been identified as follows:

- Along the Athabasca River, some trails at Riverfront Park could be inundated during open water floods with return periods as low as 10 years. During the 100-year open water flood, flooding would be limited to areas north of 50th Avenue (Highway 2). Some flooding south of 50th Avenue would occur for open water floods exceeding the 350-year return period. The Highway 813 bridge would remain above the 1,000-year flood water level.
- Along Muskeg Creek, during open water floods with return periods of 100 years or higher, sections of the three local roads (56th Street, 57th Street and the adjacent 50th Avenue south of 50th Avenue) would be inundated. During open water floods with return periods of 200 years or higher, sections of Highway 2 and its culvert crossing would be inundated.
- Along the Tawatinaw River, during open water floods with return periods of 20 years or higher, sections of the pedestrian trail east of the river would be inundated. During open water floods with return periods of 200 years or greater, flooding could extend to buildings on the west side of the river at 50th Avenue (Highway 55). During open water floods with return periods of 350 years or greater, sections of Highway 55 and its bridge crossing would be inundated.

9.5.2 Ice Jam Flood Inundation Mapping

The calibrated, ice-enhanced HEC-RAS model and the LiDAR DTM provided a good basis for simulating the flood levels and preparing inundation maps for the three ice-related flood events (i.e., 50-, 100-, and 200-year floods), including direct flood inundation areas and other indirect flood inundation areas. Based on the simulation results, the main areas to be affected by ice-related flooding have been identified as follows:

- Along the Athabasca River, some trails at Riverfront Park could be inundated during ice jam floods with return periods as low as 50 years. During the 100-year ice jam flood, flooding would be limited to areas north of 50th Avenue (Highway 2). Some flooding south of 50th Avenue would occur for ice jam floods exceeding the 200-year return period. The Highway 813 bridge would remain above the 200-year ice jam flood water level.
- Along Muskeg Creek, during ice jam floods with return periods of 100 years or higher, sections of the three local roads (56th Street, 57th Street and the adjacent 50th Avenue south of 50th Avenue) would be inundated. During ice jam floods with return periods of 200 years or higher, sections of Highway 2 and its culvert crossing would be inundated.
- Along the Tawatinaw River, during ice jam floods with return periods of 200 years or greater, flooding could extend to buildings on the west side of the river at 50th Avenue (Highway 55).

9.6 Governing Design Flood Event

In assessing the governing design flood event, it has been determined that the ice jam flood is the governing design flood on the Athabasca River, and at the reaches of Muskeg Creek and the Tawatinaw River immediately upstream of their confluences with the Athabasca River. For areas on Muskeg Creek and the Tawatinaw River that are further upstream of their respective confluences with the Athabasca River, 100-year open water flood levels are higher than the ice jam flood levels.

In the areas where open water floods govern (such as the areas noted above), the boundary between the floodway and flood fringe was determined. The floodway typically includes areas where water depths are greater than 1 m or where flow velocities are greater than 1 m/s. Exceptions may be made for ineffective flow areas, backwater areas or to accommodate a hydraulically smooth floodway line.

In the areas where the ice jam flood governs, the flood fringe is the area between the inundation extent and the floodway, which was identified based on the 1 m depth. Exceptions to these criteria were applied in some areas (ineffective flow areas, inundations due to backwater, etc.) to ensure that the floodway would be hydraulically smooth.

In the areas where an existing floodway from previous flood study governs, the floodway typically corresponds to the existing floodway. The new floodway would be modified from the existing floodway if a main channel shifts outside of the existing floodway or the existing floodway is no longer flooded by the design flood.

9.6.1 Areas in the Floodway

No residences are situated on the floodways along the Athabasca River, Muskeg Creek and the Tawatinaw River. The development areas and structures within the floodway zone are listed below.

- A walkway on the right bank of the Athabasca River between the Muskeg Creek confluence and the Tawatinaw River confluence; and
- A wetland area on the right side of the Tawatinaw River upstream of the Highway 55 bridge crossing.

9.6.2 Areas in the Flood Fringe

The development areas in the flood fringe within the study area are listed below.

Athabasca River

- Portions of parking lot of the Independent Grocery Store on the right floodplain;
- Athabasca Home Hardware Building Centre on the right floodplain; and
- 51st Avenue on the right floodplain.

Muskeg Creek

- Small portions on both sides of upstream of Highway 2 Culvert, including 57 Street and Muskeg Creek trails; and
- Small portions on the right side downstream of Highway 2 Culvert.

Tawatinaw River

Small portions on the left bank upstream of Highway 55 Bridge.

9.7 Climate Change Effects

9.7.1 Open Water Floods

Potential effects of climate change on open water floods were assessed through a sensitivity analysis of flood water level differences due to 10- and 20-percent increases in discharge. These water level differences were identified as potential “freeboards” that could be applied to the design water levels to account for flow changes that could result from climate change. The results of the climate change effect assessment are summarized below:

- For the Athabasca River reach, the average increases in open water flood levels are 0.44 m for a 10 percent increase in flow, and 0.87 m for a 20 percent increase in flow.
- For the Muskeg Creek reach, the average increases in open water flood levels are 0.22 m for a 10 percent increase in flow, and 0.43 m for a 20 percent increase in flow.

- For the Tawatinaw River reach, the average increases in open water flood levels are 0.26 m for a 10 percent increase in flow, and 0.52 m for a 20 percent increase in flow.

The analysis in this study was not based on a regional climate change impact assessment but on a simplified assumption that climate change will result in increased flood peak flows.

9.7.2 Ice Jam Floods

Ice jam floods under climate change scenarios may be influenced by multiple factors including winter temperature regime and season duration, snowpack depth, ice thickness and other factors, introducing challenges to projecting the effects of climate change on ice jam frequency and severity. A literature review suggests that it would be imprudent to provide predictions of potential future ice jam flood water levels under climate change scenarios.

As for the case of open water floods, the variability of modeled discharges under historical climate conditions may be used as an indicator of potential “freeboards” that could be applied to the design water levels to account for flow changes that could result from climate change.

The baseline 200-year ice jam flood water levels were used as possible 100-year ice jam flood water levels along the Athabasca River reach due to climate change. The average difference between the baseline 200-year and 100-year ice jam water flood levels is 0.80 m.

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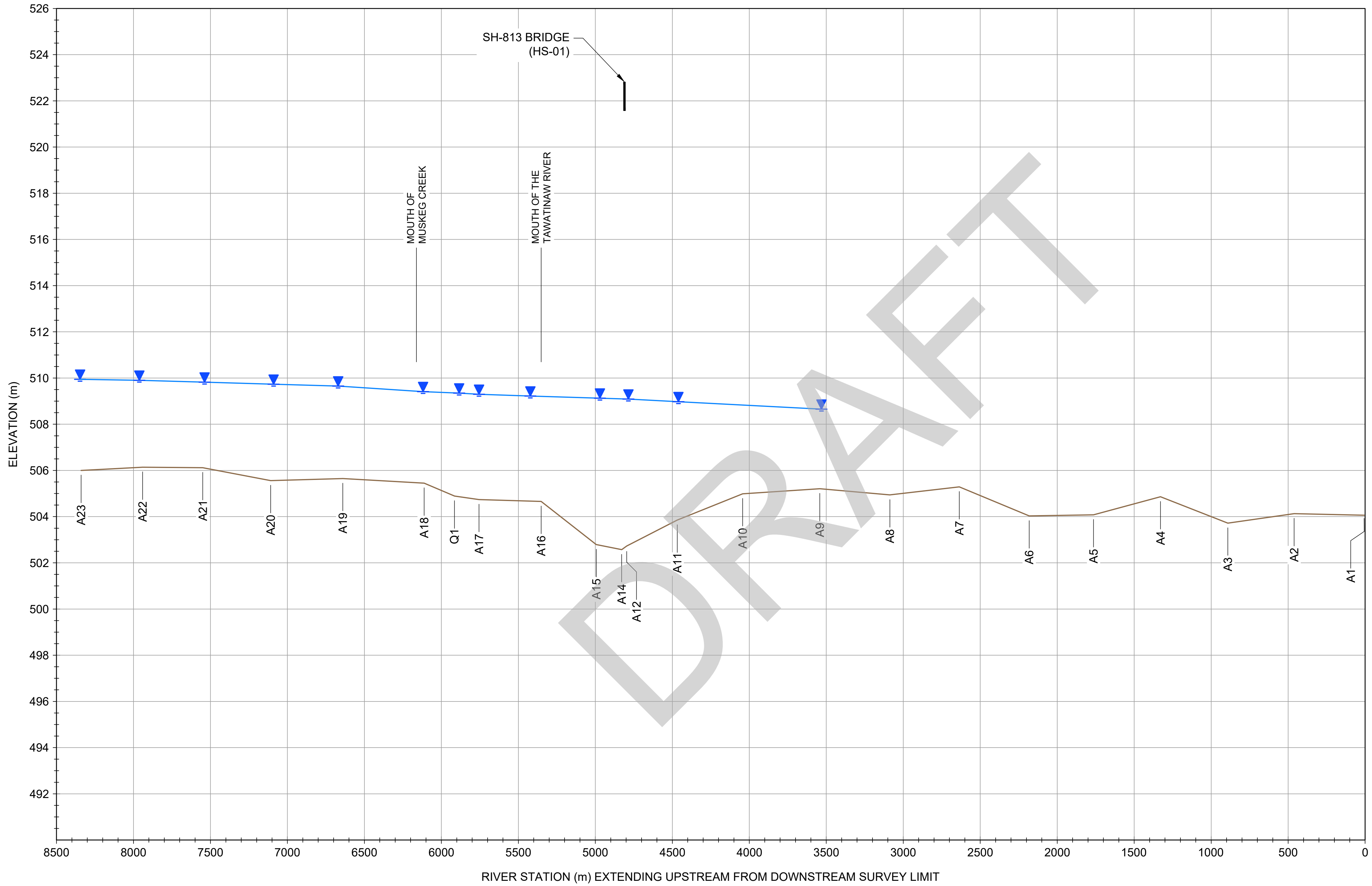
APPENDIX A

Surveyed Thalweg and Water Surface Profiles



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NOTES:

1. GROUND AND WATER LEVEL DATA ON THE ATHABASCA RIVER WERE COLLECTED BY GOLDER ASSOCIATES LTD. IN MAY AND JUNE 2019.
2. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
3. A DISCHARGE OF 968 m^3/s WAS MEASURED ON THE ATHABASCA RIVER AT THE TIME OF THE WATER SURFACE PROFILE SURVEY ON 4-JUN-2019. THE MEASUREMENT WAS MADE AT THE WSC 07BE001 HYDROMETRIC STATION LOCATION (CROSS SECTION Q1) USING AN ACOUSTIC DOPPLER CURRENT PROFILER (ADCP), DEPLOYED FROM A RIVER BOAT.

LEGEND:

SURVEYED THALWEG (MAY/JUNE 2019)
SURVEYED WATER LEVEL (4-JUN-2019)

0 m 300 600 900 1200
1 : 30,000 HORIZONTAL

0 m 2 4 6 8
1 : 200 VERTICAL

PREPARED FOR:

Alberta

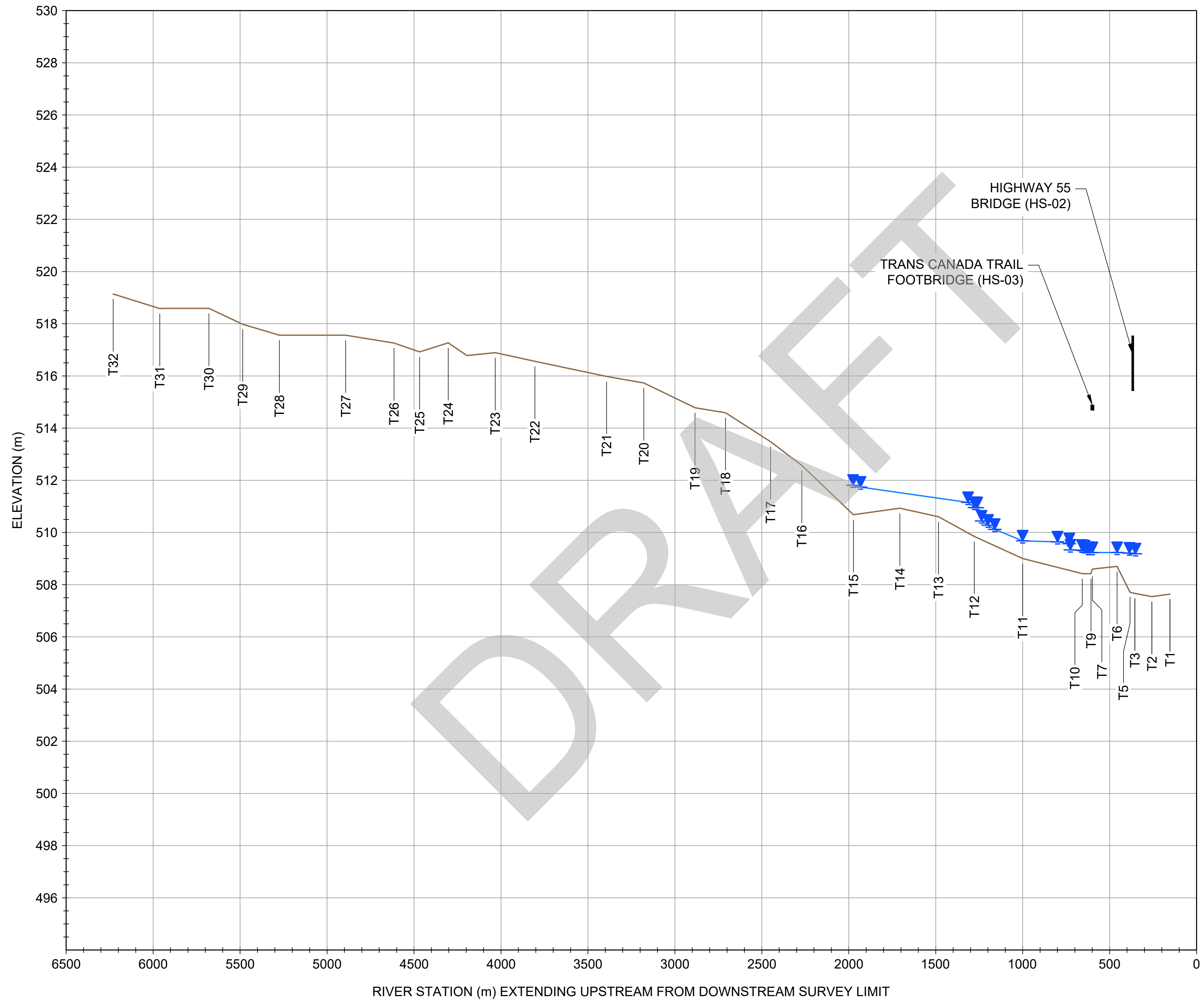
PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Surveyed Thalweg and Water Surface Profile
Athabasca River

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-103		FIGURE NO: A-1
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-103.dwg - 103, PLOT DATE: 28-Jul-2020

Classification: Public



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NOTES:

1. GROUND AND WATER LEVEL DATA ON THE TAWATINAW RIVER WERE COLLECTED BY GOLDER ASSOCIATES LTD. IN MAY AND JUNE 2019.
2. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
3. A DISCHARGE OF 0.22 m^3/s WAS MEASURED ON THE TAWATINAW RIVER AT THE TIME OF THE WATER SURFACE PROFILE SURVEY ON 3-JUN-2019. THE MEASUREMENT WAS MADE 15 m UPSTREAM OF THE FOOTBRIDGE USING AN ACOUSTIC DOPPLER VELOCIMETER (ADV).
4. SURVEYED WATER LEVELS WERE INFLUENCED BY BEAVER DAMS OBSERVED AT SEVERAL LOCATIONS ALONG THE CREEK.

LEGEND:

SURVEYED THALWEG (MAY 2019)
SURVEYED WATER LEVEL (3-JUN-2019)

0 m 300 600 900 1200
1 : 30,000 HORIZONTAL

0 m 2 4 6 8
1 : 200 VERTICAL

PREPARED FOR:

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PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Surveyed Thalweg and Water Surface Profile
Tawatinaw River

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-104	FIGURE NO: A-2
DATE:	29-JUL-2020	

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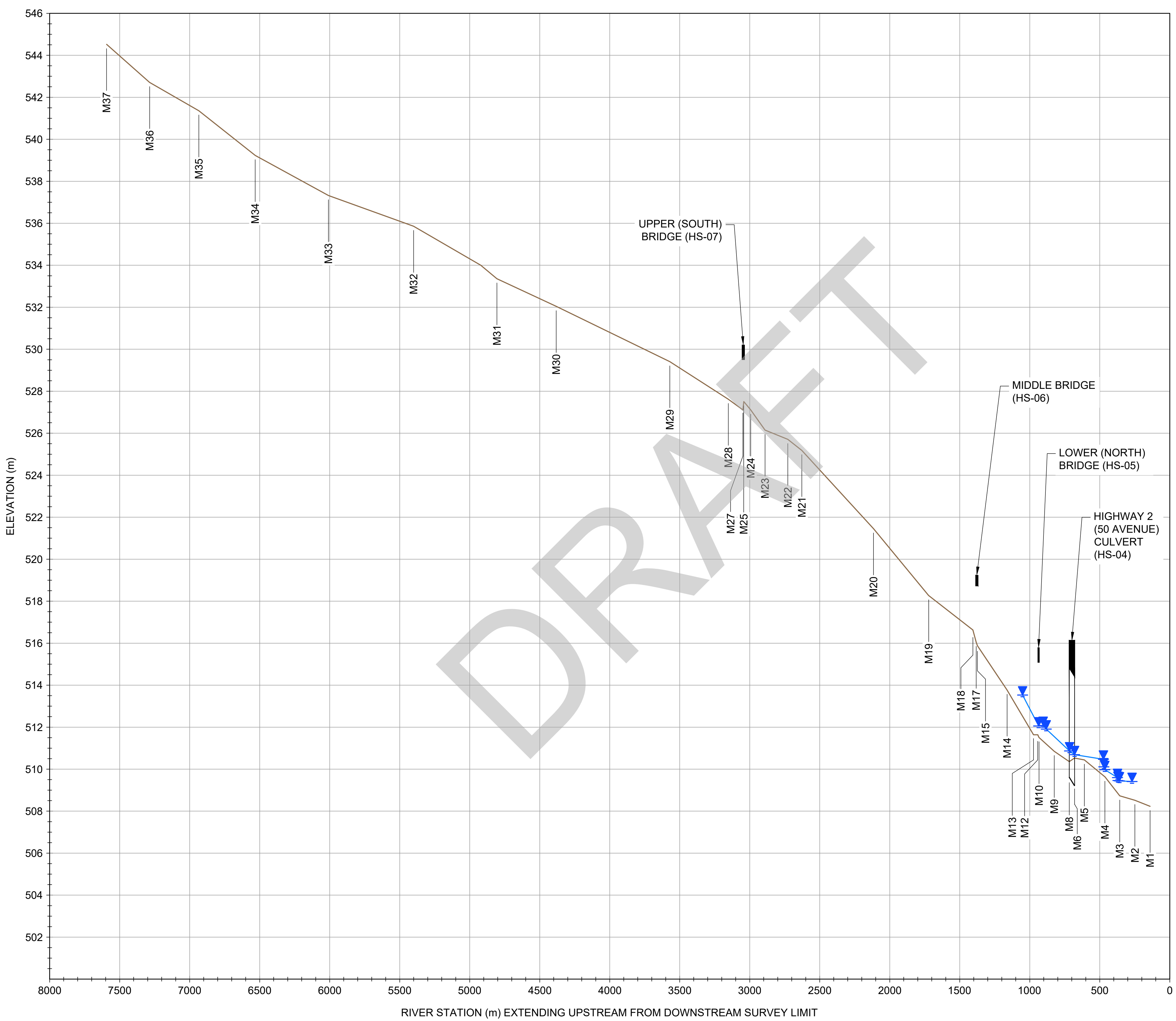

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NOTES:

1. GROUND AND WATER LEVEL DATA ON MUSKEG CREEK WERE COLLECTED BY GOLDER ASSOCIATES LTD. IN MAY AND JUNE 2019.
2. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
3. A DISCHARGE OF 0.037 m^3/s WAS MEASURED ON MUSKEG CREEK AT THE TIME OF THE WATER SURFACE PROFILE SURVEY ON 3-JUN-2019. THE MEASUREMENT WAS MADE APPROXIMATELY 40 m UPSTREAM OF THE HIGHWAY 2 CULVERT USING AN ACOUSTIC DOPPLER VELOCIMETER (ADV).
4. SURVEYED WATER LEVELS WERE INFLUENCED BY BEAVER DAMS OBSERVED AT STA 0+374 AND STA 0+473 ALONG THE CREEK.

LEGEND:

SURVEYED THALWEG (MAY 2019)

SURVEYED WATER LEVEL (3-JUN-2019)

0 m 300 600 900 1200

1 : 30,000 HORIZONTAL

0 m 2 4 6 8

1 : 200 VERTICAL

PREPARED FOR:

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PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

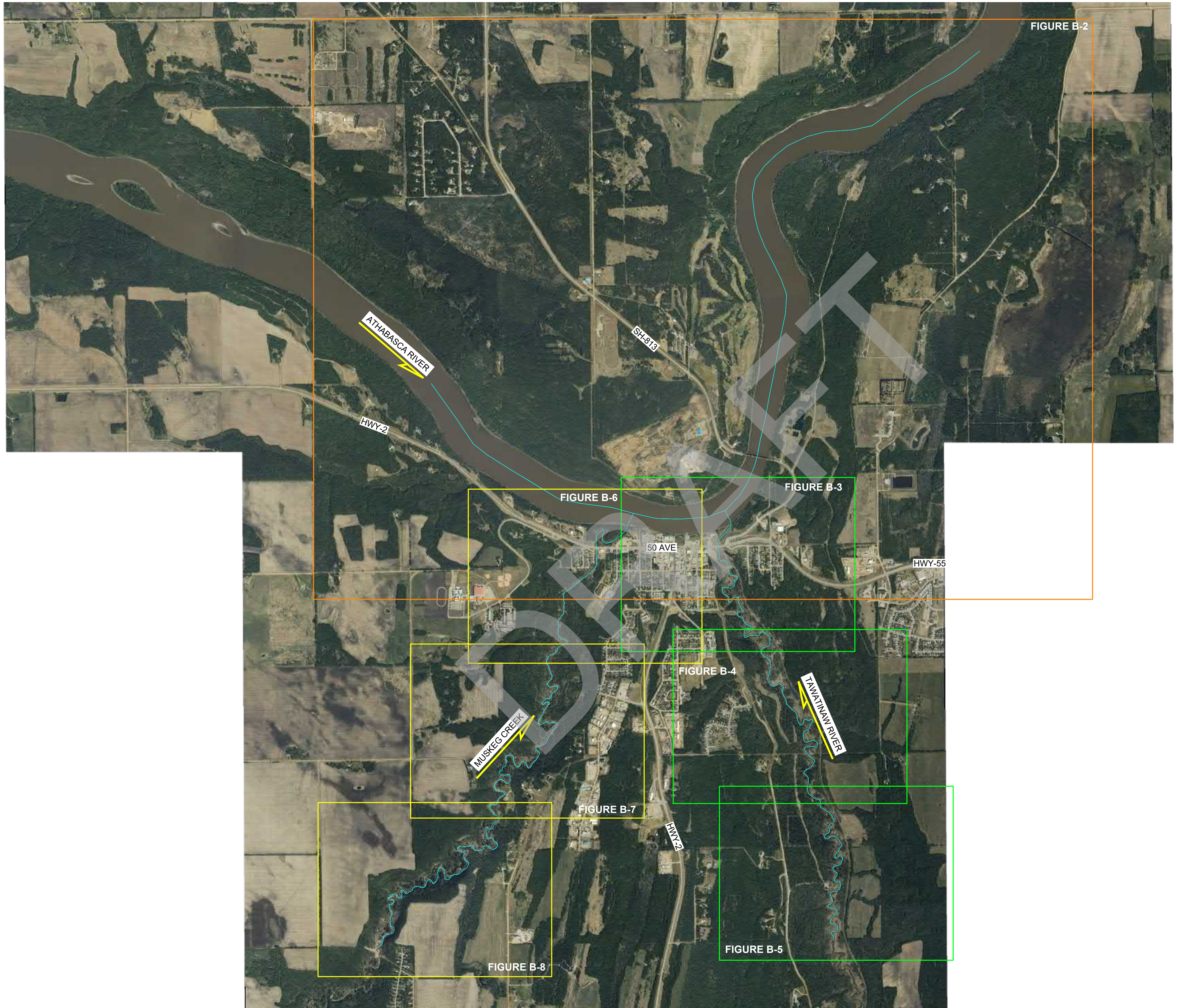
Surveyed Thalweg and Water Surface Profile
Muskeg Creek

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-105		FIGURE NO: A-3
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-105.dwg - 105, PLOT DATE: 28-Jul-2020

APPENDIX B

Cross Section and Hydraulic Structure Locations



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NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.
2. THE STUDY AREA IS COMPRISED OF AN 8.3 km LONG REACH OF THE ATHABASCA RIVER, A 6.1 km LONG REACH OF THE TAWATINAW RIVER, AND A 7.5 km LONG REACH OF MUSKEG CREEK WITHIN ATHABASCA COUNTY AND THE TOWN OF ATHABASCA.
3. A HYDROMETRIC STATION, OWNED AND OPERATED BY THE WATER SURVEY OF CANADA (WSC), IS LOCATED ON THE ATHABASCA RIVER WITHIN THE WATER TREATMENT PLANT BUILDING (WSC 07BE001). REFER TO FIGURE B-2 FOR DETAILS.

0 m 500 1000 1500

1 : 35,000

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PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Location Map

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-106	FIGURE NO: B-1
DATE:	29-JUL-2020	

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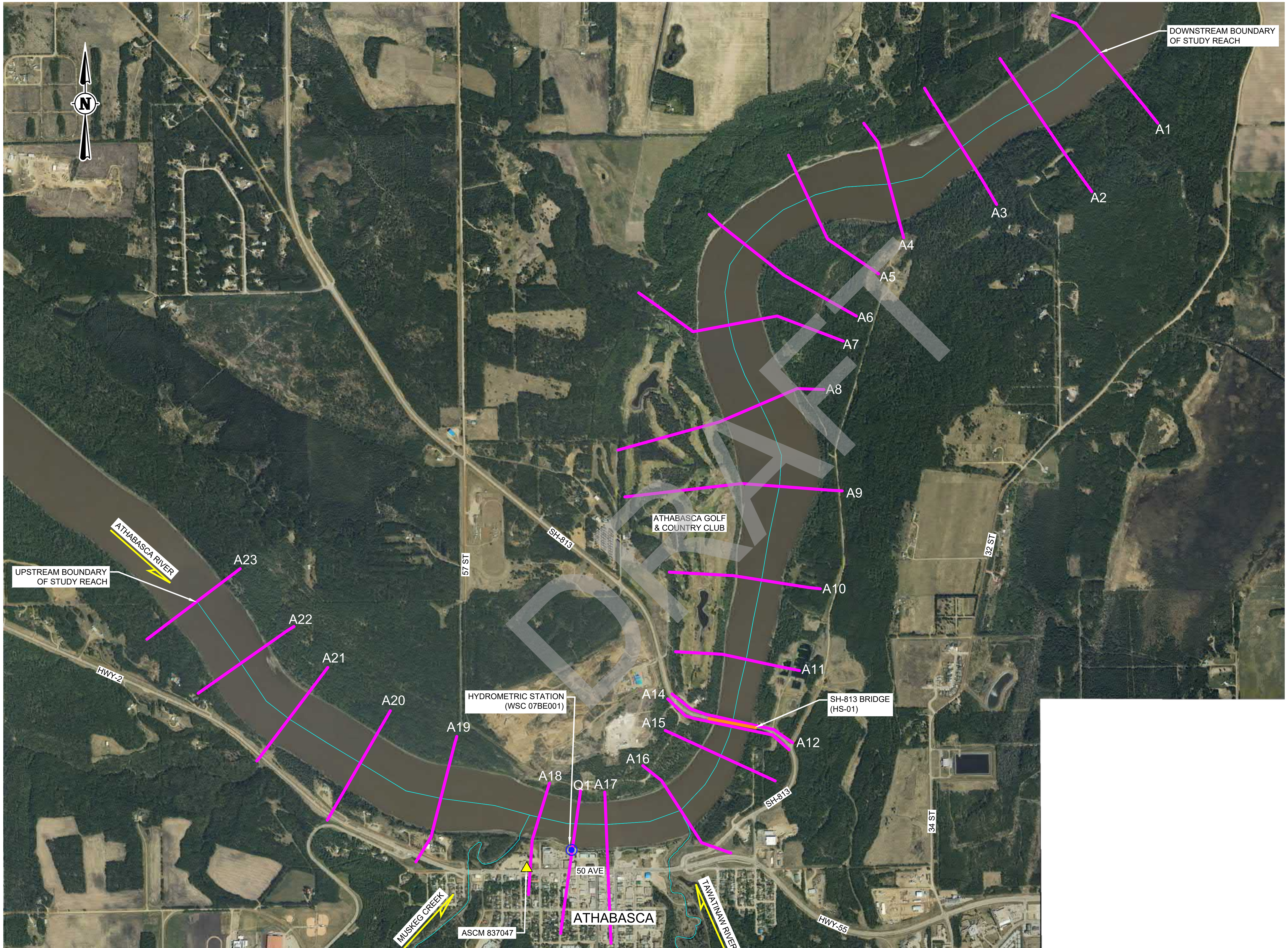

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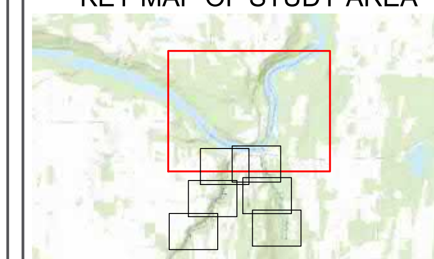
NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.

LEGEND:

- CHANNEL CROSS SECTION (AS MODELLED)
- HYDRAULIC STRUCTURE (BRIDGE/CULVERT)
- HYDROMETRIC STATION
- SURVEY CONTROL (ASCM)

KEY MAP OF STUDY AREA



0 m 200 400 600 800

1 : 20,000

PREPARED FOR:

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PROJECT:

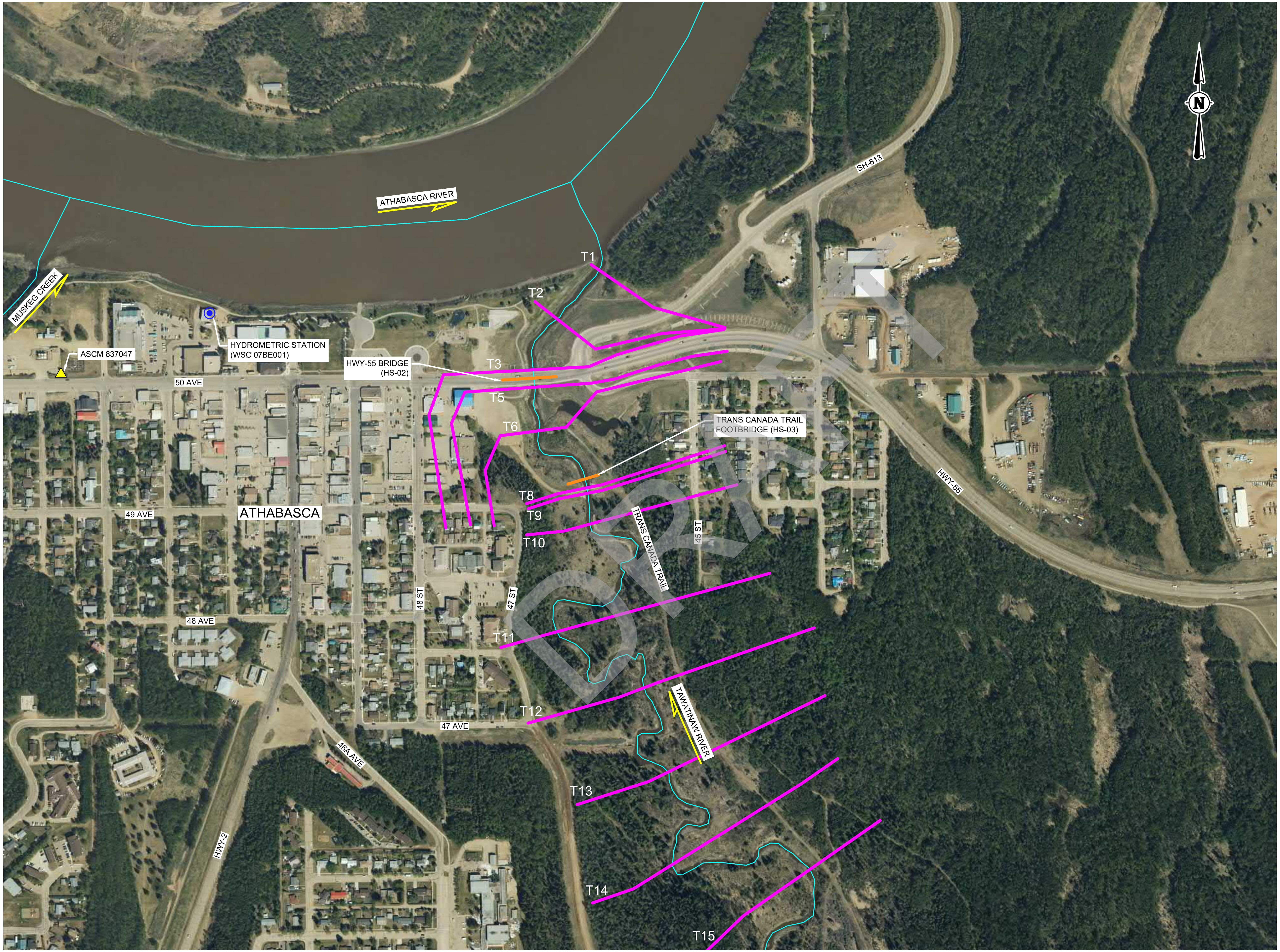
ATHABASCA FLOOD HAZARD STUDY

TITLE:

Cross Section and Hydraulic Structure Locations
Athabasca River

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-107		FIGURE NO: B-2
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-107.dwg - 107, PLOT DATE: 28-Jul-2020



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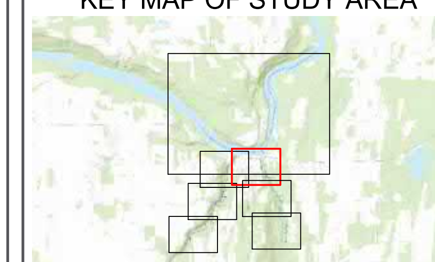
NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.

LEGEND:

- CHANNEL CROSS SECTION (AS MODELLED)
- HYDRAULIC STRUCTURE (BRIDGE/CULVERT)
- HYDROMETRIC STATION
- SURVEY CONTROL (ASCM)

KEY MAP OF STUDY AREA



0 m 50 100 150 200

1 : 6000

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Alberta

PROJECT:

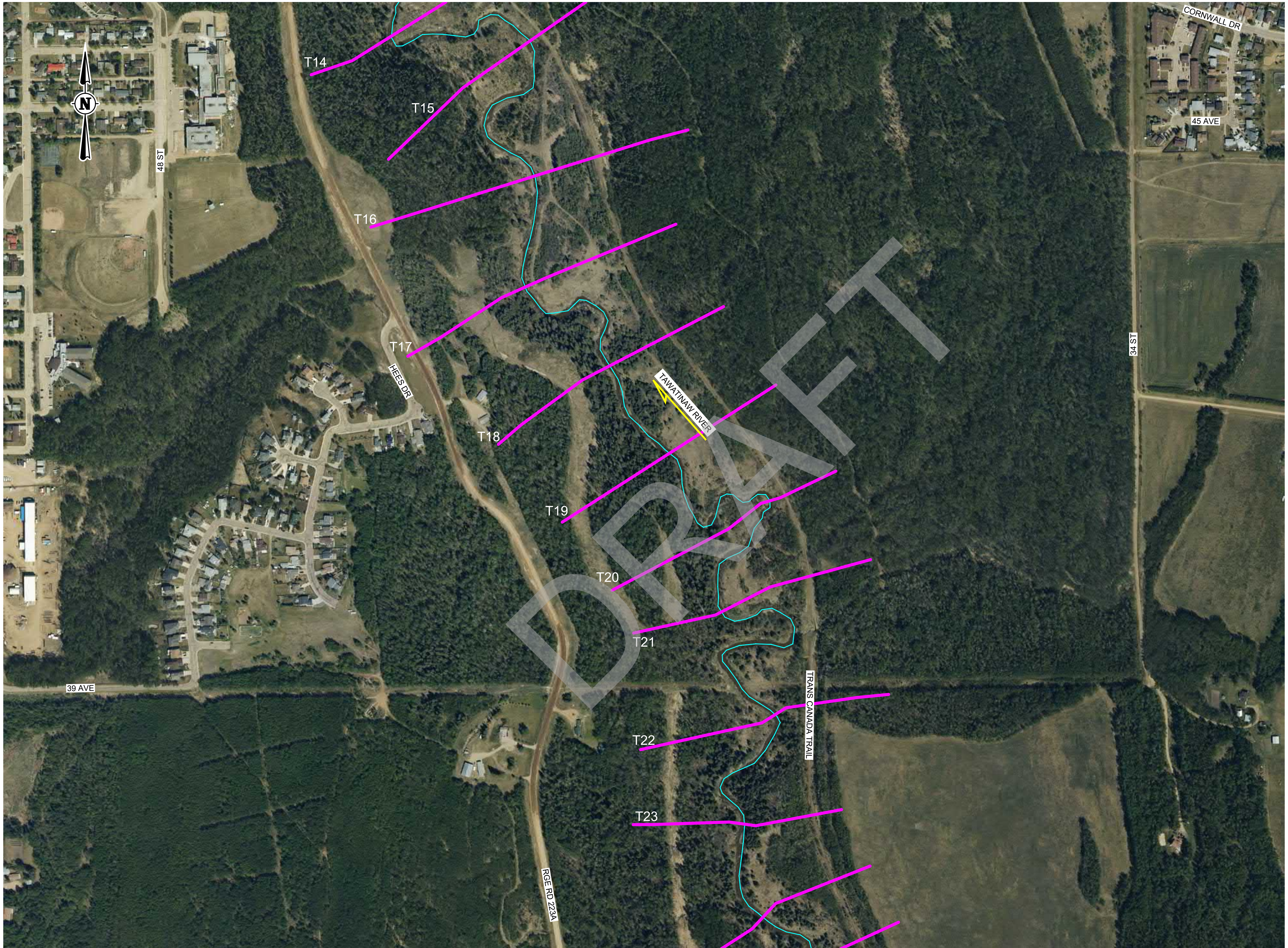
ATHABASCA FLOOD HAZARD STUDY

TITLE:

Cross Section and Hydraulic Structure Locations
Tawatinaw River
(Sheet 1 of 3)

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-108		FIGURE NO: B-3
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-108.dwg - 108, PLOT DATE: 28-Jul-2020



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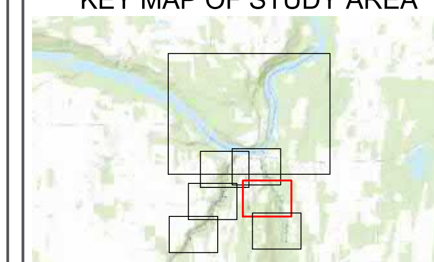
NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.

LEGEND:

CHANNEL CROSS SECTION (AS MODELLED)

KEY MAP OF STUDY AREA



0 m 50 100 150 200

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PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

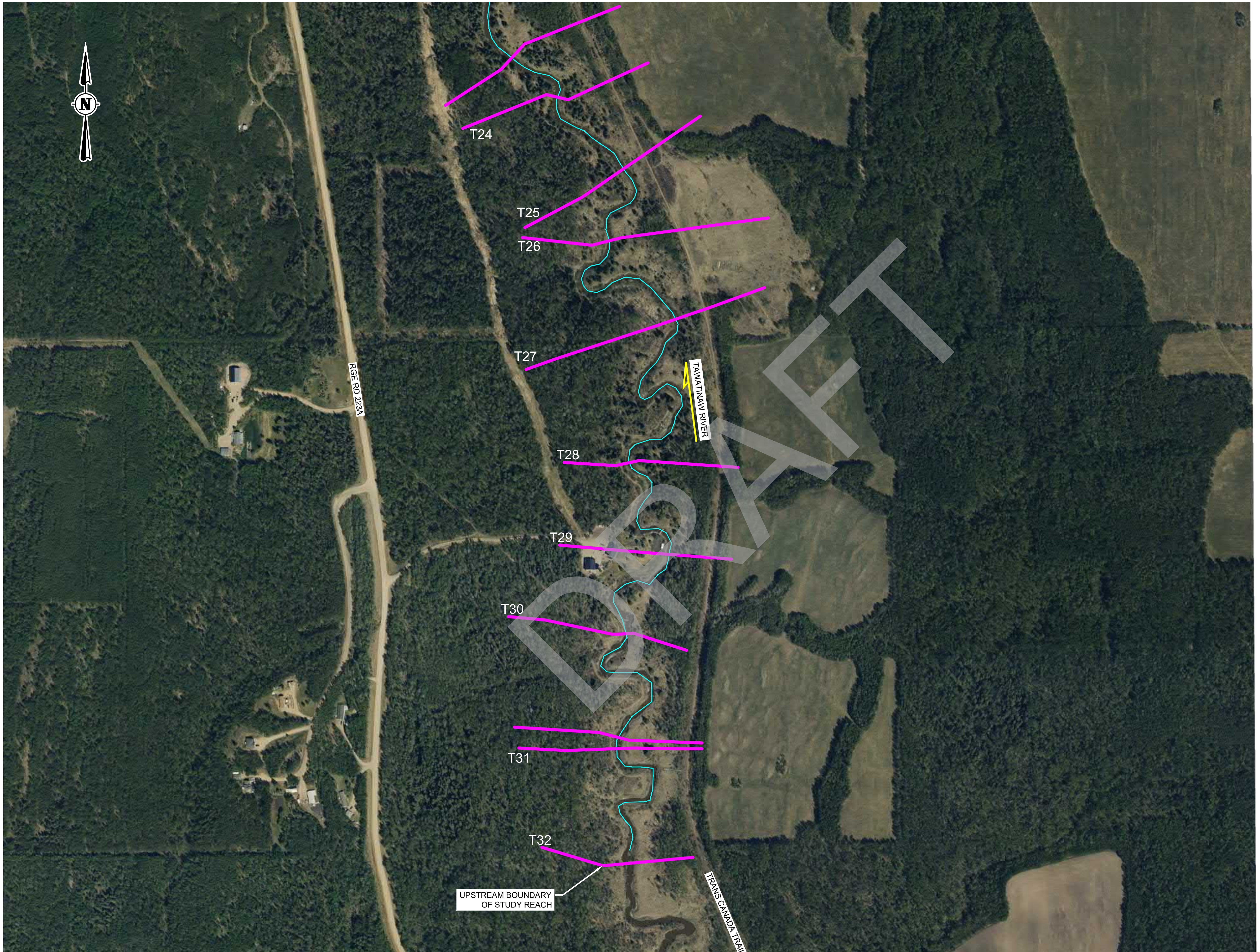
TITLE:

Cross Section and Hydraulic Structure Locations
Tawatinaw River
(Sheet 2 of 3)

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-109	FIGURE NO: B-4
DATE:	29-JUL-2020	

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-109.dwg - 109, PLOT DATE: 28-Jul-2020

Classification: Public



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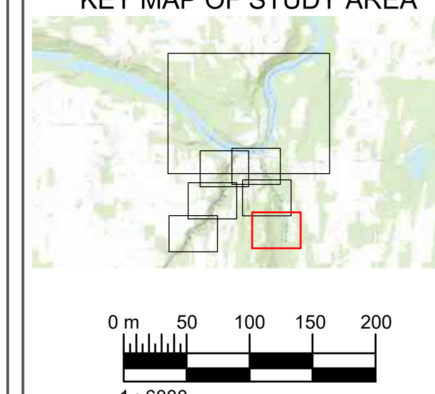
LEGEND:

CHANNEL CROSS SECTION (AS MODELLED)

KEY MAP OF STUDY AREA



PREPARED FOR:

Alberta

PROJECT:

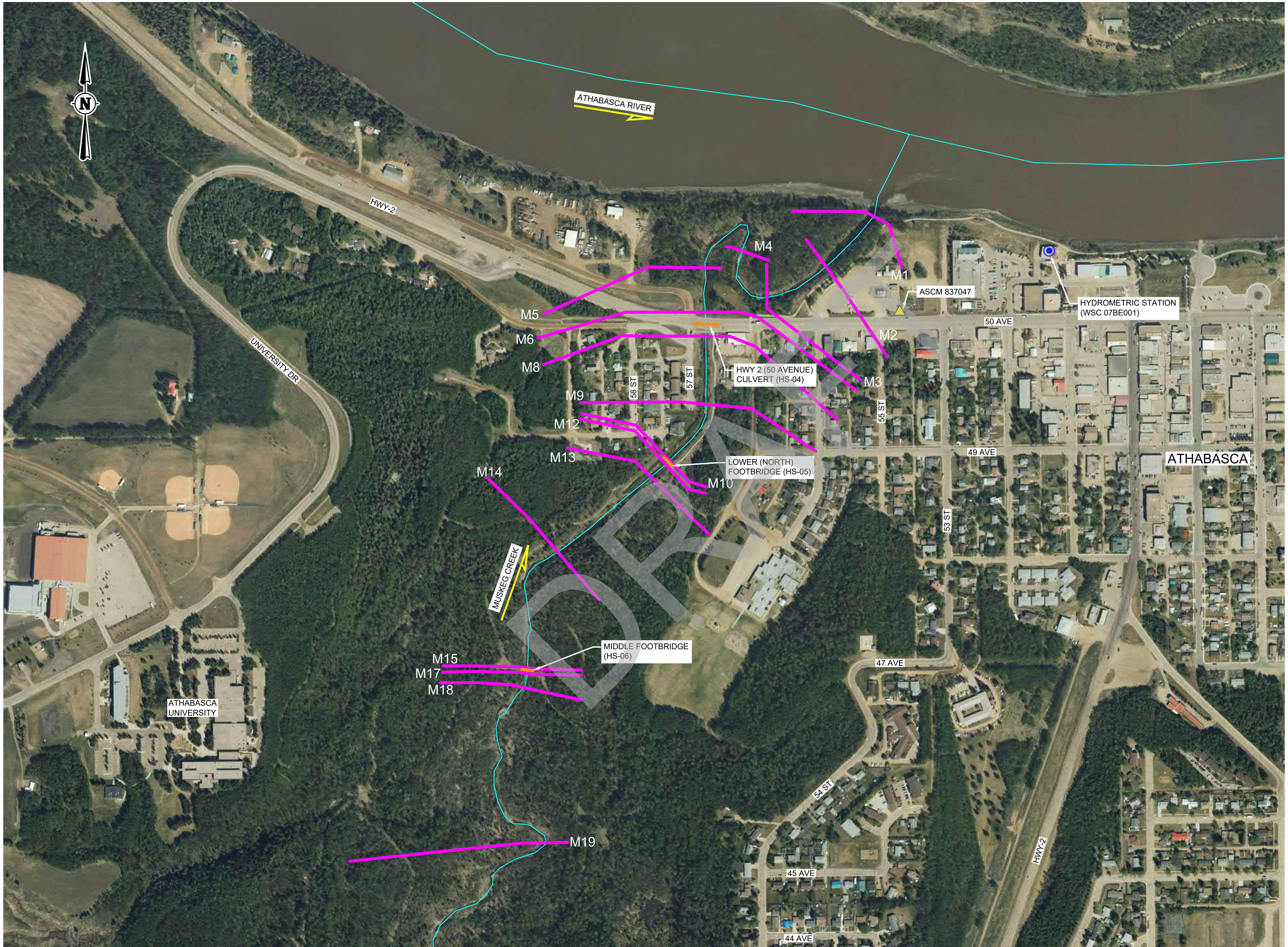
ATHABASCA FLOOD HAZARD STUDY

TITLE:

Cross Section and Hydraulic Structure Locations
Tawatinaw River
(Sheet 3 of 3)

DWN BY:	CHK'D BY:	REV NO:
RDJ	DMS	1
DWG NO: 10140-02-110		FIGURE NO: B-5
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-110.dwg - 110, PLOT DATE: 28-Jul-2020



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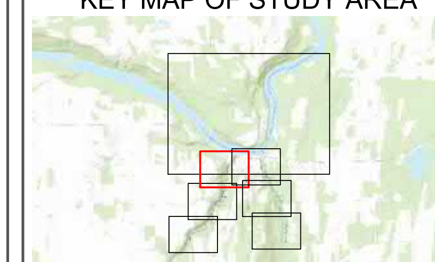
LEGEND:

- CHANNEL CROSS SECTION (AS MODELLED)
- HYDRAULIC STRUCTURE (BRIDGE/CULVERT)
- HYDROMETRIC STATION
- SURVEY CONTROL (ASCM)

KEY MAP OF STUDY AREA



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1 : 6000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Cross Section and Hydraulic Structure Locations
Muskeg Creek
(Sheet 1 of 3)

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-111	FIGURE NO: B-6
DATE:	29-JUL-2020	

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-111.dwg - 111, PLOT DATE: 28-Jul-2020

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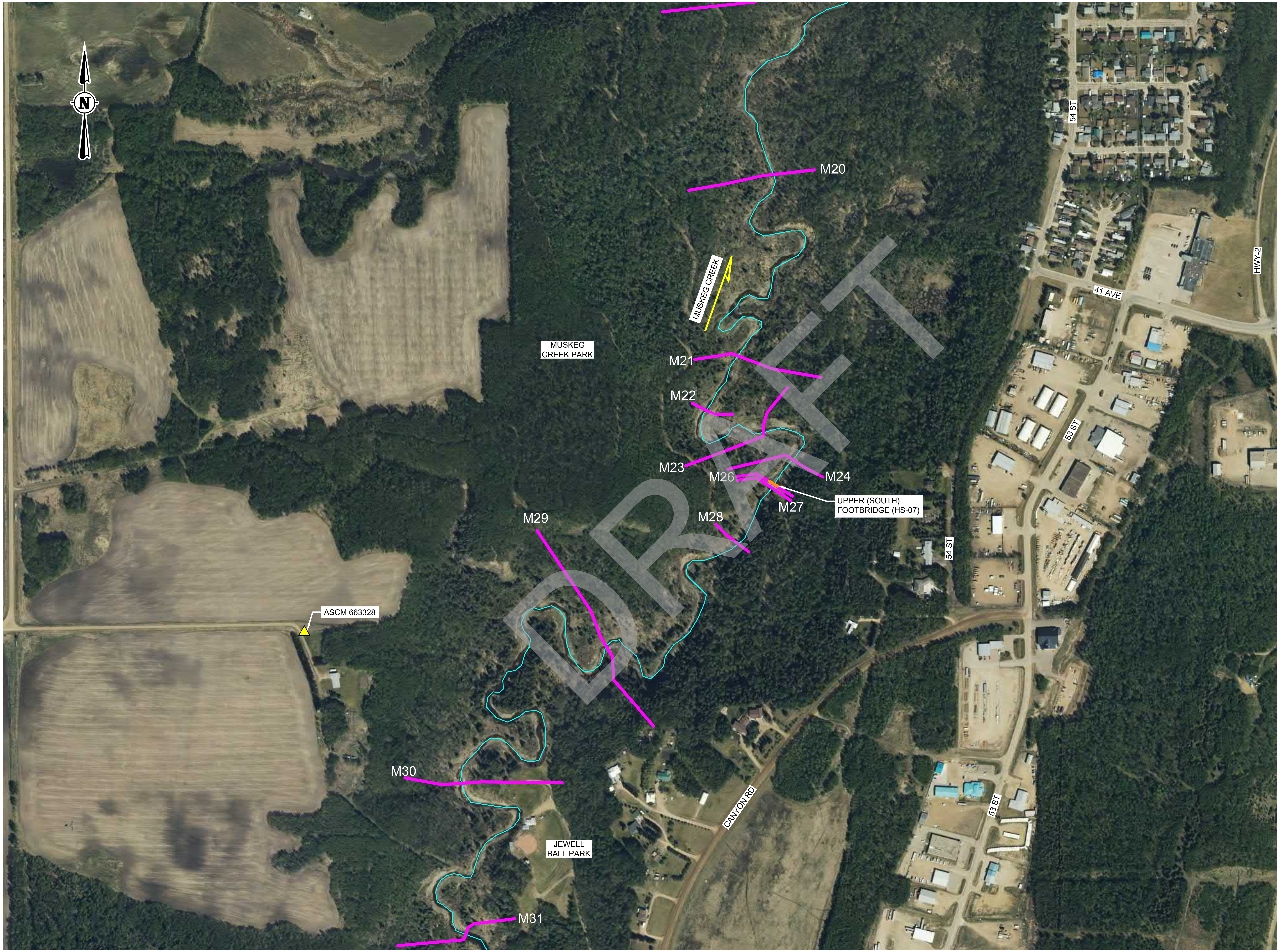

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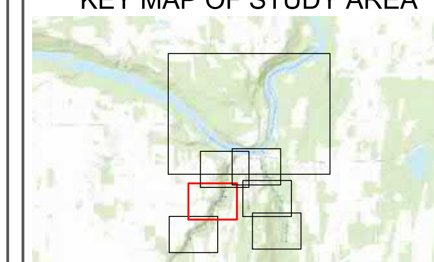
LEGEND:

- CHANNEL CROSS SECTION (AS MODELLED)
- HYDRAULIC STRUCTURE (BRIDGE/CULVERT)
- SURVEY CONTROL (ASCM)

KEY MAP OF STUDY AREA



0 m 50 100 150 200

1 : 6000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

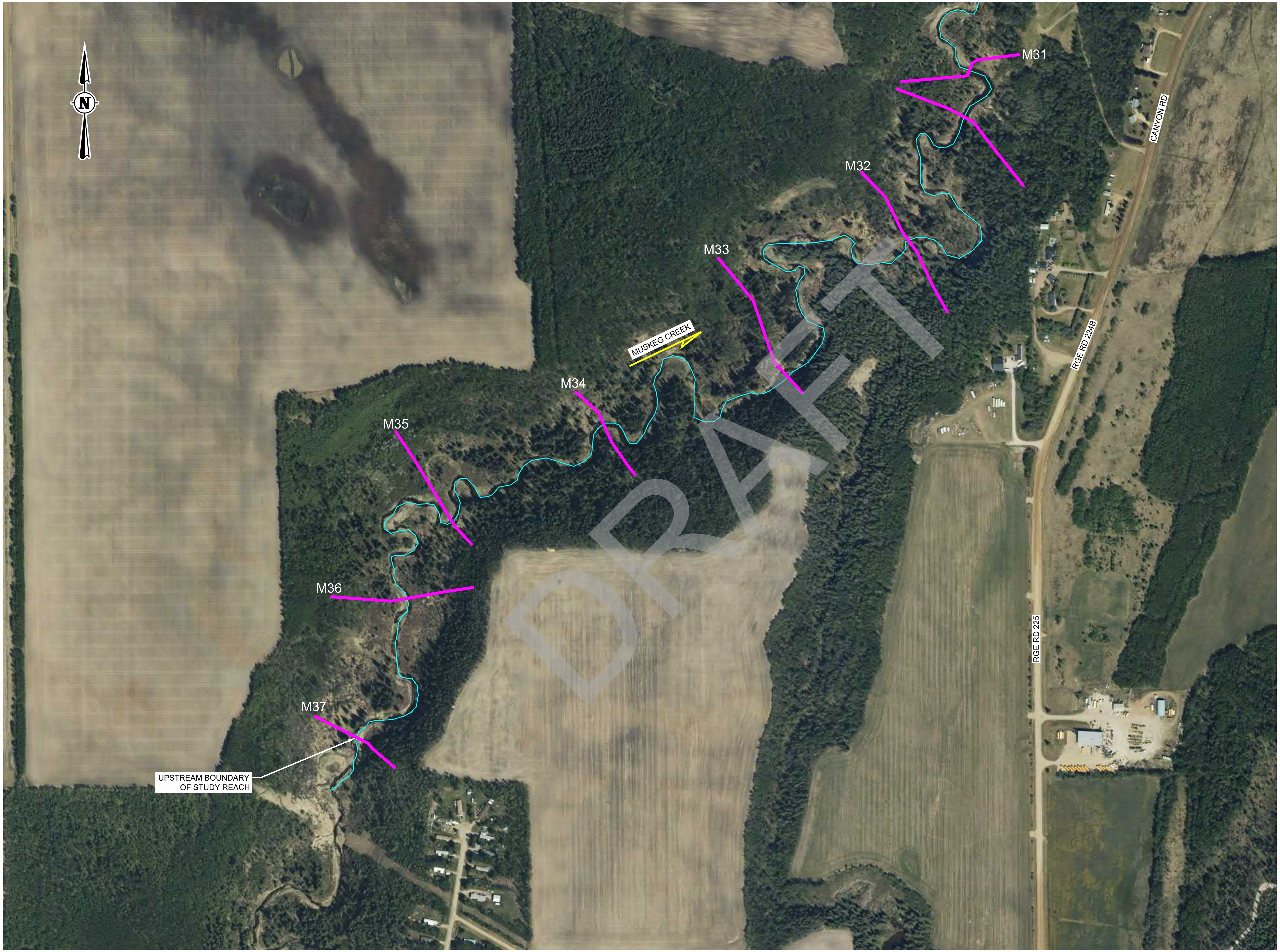
TITLE:

Cross Section and Hydraulic Structure Locations
Muskeg Creek
(Sheet 2 of 3)

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-112		FIGURE NO: B-7
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-112.dwg - 112, PLOT DATE: 28-Jul-2020

Classification: Public



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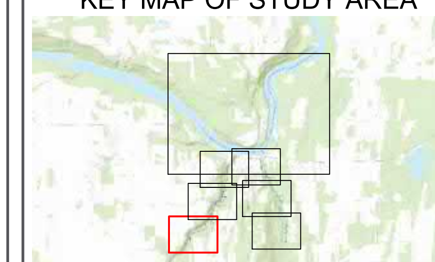
LEGEND:

CHANNEL CROSS SECTION (AS MODELLED)

KEY MAP OF STUDY AREA



0 m 50 100 150 200

1 : 6000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Cross Section and Hydraulic Structure Locations
Muskeg Creek
(Sheet 3 of 3)

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO:	10140-02-113	FIGURE NO: B-8
DATE:	29-JUL-2020	

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-113.dwg - 113, PLOT DATE: 28-Jul-2020

Classification: Public

APPENDIX C

Hydraulic Structure Datasheets

IMAGE REF: 10140-SV.039

1. View looking downstream from the right bank (near the mouth of the Tawatinaw River).

IMAGE REF: 10140-SV.055

2. View looking obliquely upstream from the top of the right bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS[1]				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
ATHABASCA RIVER	SH-813 BRIDGE	09905	ALBERTA TRANSPORTATION	TRAFFIC	1950	288.42	7.30	521.57	522.82	521.11	521.99	522.46	522.90	8	7	1	30.8	3.00	1.35	Lower portion: Solid concrete wall with triangular wedge nose and tail (sloped) set on H-pile foundation Upper portion: Two rectangular concrete columns at both ends of pier cap, extending up to bridge structure
																2	92.4	3.00	1.35	
																3	154.1	3.00	1.35	
																4	215.6	3.00	1.35	
																5	246.5	3.00	1.35	
																6	261.7	n/a	n/a	Two rectangular concrete columns at both ends of pier cap
																7	277.0	n/a	n/a	

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

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NOTES:

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2. TOPOGRAPHIC SURVEY DATA WERE COLLECTED BY TROUT HYDROGRAPHY INC. ON 25-MAY-2019.
3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
4. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

● SURVEY DATA POINT

0 m 50 100 150 200

1 : 5,000

PREPARED FOR:

Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Hydraulic Structure Datasheet (HS-01)
Traffic Bridge on Athabasca River
SH-813 Bridge

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-114		FIGURE NO: C-1
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-114.dwg - 118, PLOT DATE: 28-Jul-2020

DRAFT

IMAGE REF: 10140-SV.051

1. View looking downstream from the left bank.

IMAGE REF: 10140-SV.042

2. View looking upstream from the top of the left bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS(1)				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
TAWATINAW RIVER	HIGHWAY 55 BRIDGE	01517	ALBERTA TRANSPORTATION	TRAFFIC	2013	40.00	21.20	515.43	517.54	515.06	515.83	517.12	517.94	1	0	-	-	-		-

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

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NOTES:

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2. TOPOGRAPHIC SURVEY DATA WERE COLLECTED BY TROUT HYDROGRAPHY INC. ON 24-MAY-2019.
3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
4. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

● SURVEY DATA POINT

0 m 20 40 60 80
1 : 2000

PREPARED FOR:
Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Hydraulic Structure Datasheet (HS-02)
Traffic Bridge on Tawatinaw River
Highway 55 Bridge

DWN BY: RDJ | CHK'D BY: DMS | REV NO: 1
DWG NO: 10140-02-115 | FIGURE NO: C-2
DATE: 29-JUL-2020

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-115.dwg - 119, PLOT DATE: 28-Jul-2020

IMAGE REF: 10140-SV.072

1. View looking downstream from the left bank.

IMAGE REF: 10140-SV.068

2. View looking upstream from the left bank.

DRAFT

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS[1]				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
TAWATINAW RIVER	TRANS CANADA TRAIL FOOTBRIDGE	n/a	n/a	PEDESTRIAN	2017	36.40	3.00	514.68	514.89	514.34	514.71	514.56	514.92	1	0	-	-	-		-

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

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NOTES:

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3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
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5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

SURVEY DATA POINT

0 m 10 20 30 40

1 : 1000

PREPARED FOR:

Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Hydraulic Structure Datasheet (HS-03)
Pedestrian Bridge on Tawatinaw River
Trans Canada Trail Footbridge

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-116		FIGURE NO: C-3
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-116.dwg - 120, PLOT DATE: 28-Jul-2020

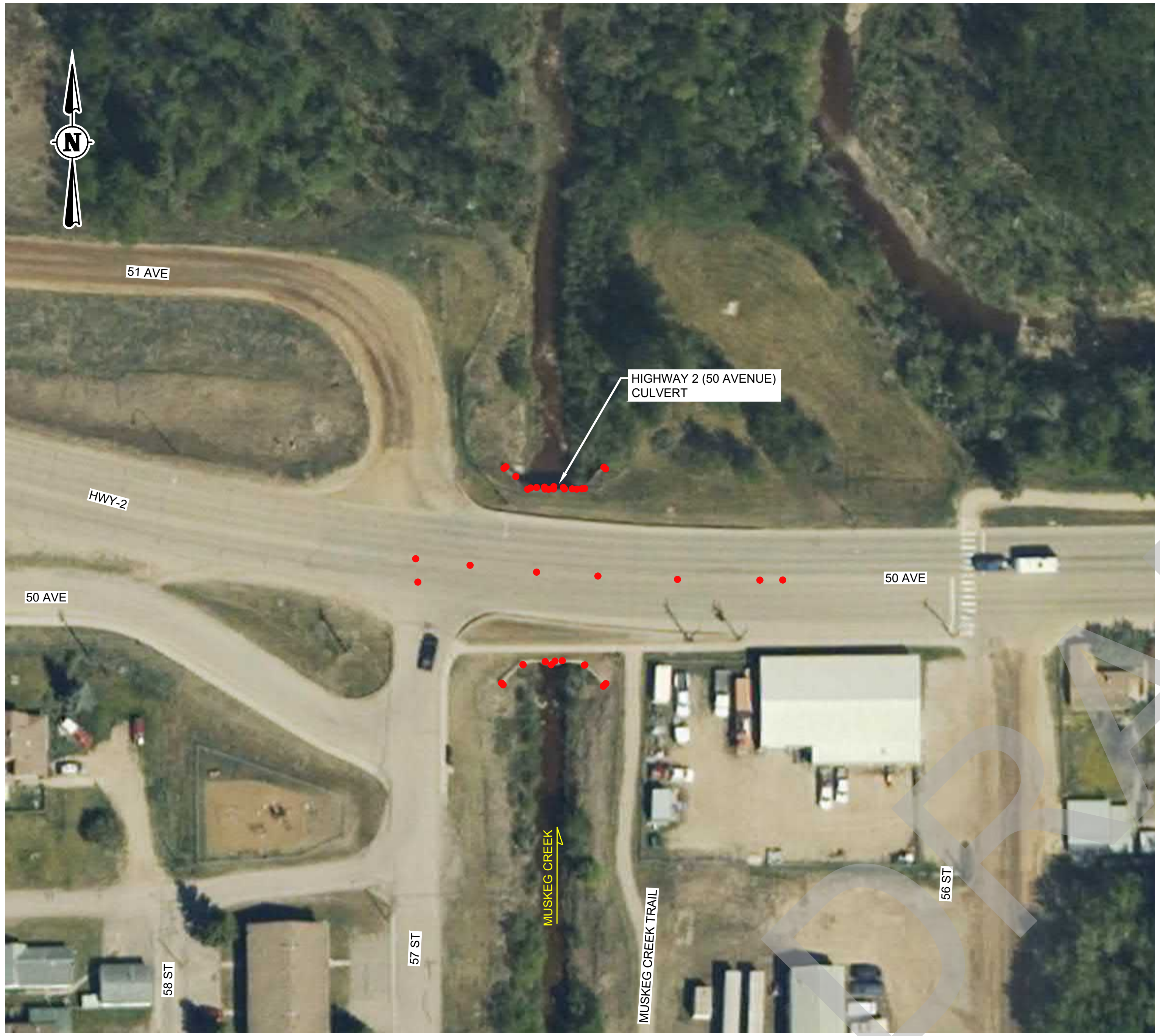

IMAGE REF: 10140-SV.087

1. View looking downstream from the right bank.

IMAGE REF: 10140-SV.017

2. View looking upstream from the right bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	NO. OF CULVERTS	BARREL LENGTH (m)	MAXIMUM WIDTH (m)	MAXIMUM HEIGHT (m)	PIPE SLOPE (m/m)	CULVERT TYPE	CULVERT SHAPE	ENTRANCE CONDITION	CULVERT INVERT ELEVATION (m)		TOP OF ROADWAY ELEVATION (m)
														INLET	OUTLET	
MUSKEG CREEK	HIGHWAY 2 (50 AVENUE) CULVERT	08394	ALBERTA TRANSPORTATION	TRAFFIC	1983	1	30.00	8.56	5.21	0.0137	CORRUGATED STEEL PIPE ENCASED IN CONCRETE BOX CHANNEL	ELLIPSE	VERTICAL CONCRETE HEADWALL WITH FLARED WINGWALLS; PROJECTING OUT FROM ROADWAY BANK SLOPE	509.62	509.21	516.13

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3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
4. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

● SURVEY DATA POINT

0 m 10 20 30 40

1 : 1000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Hydraulic Structure Datasheet (HS-04)
Culvert on Muskeg Creek
Highway 2 (50 Avenue) Culvert

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-117		FIGURE NO: C-4
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-117.dwg - 117, PLOT DATE: 28-Jul-2020

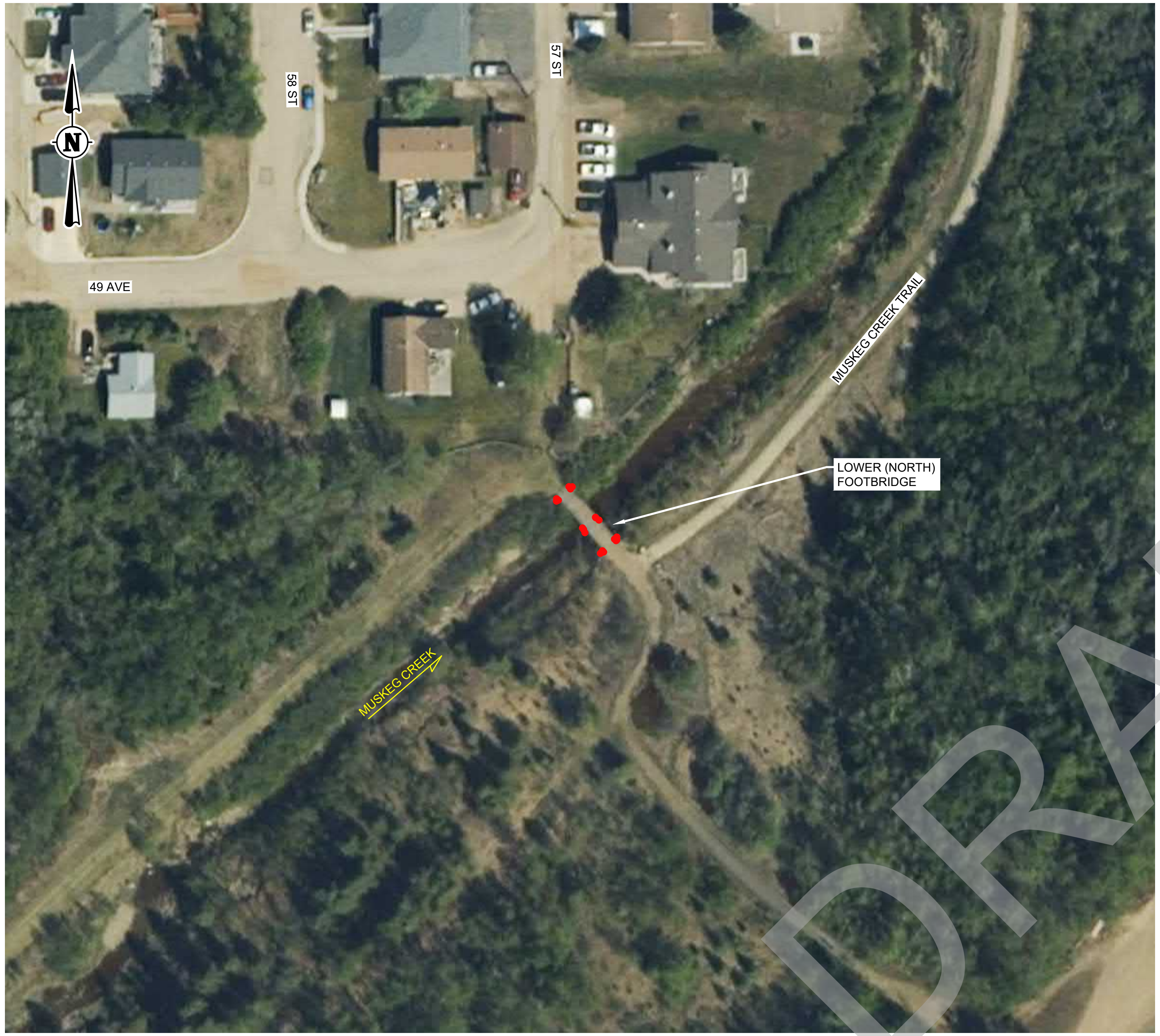

IMAGE REF: 10140-SV.095

1. View looking downstream from the top of the right bank.

IMAGE REF: 10140-SV.094

2. View looking obliquely upstream from the top of the right bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS(1)				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
MUSKEG CREEK	LOWER (NORTH) FOOTBRIDGE	n/a	n/a	PEDESTRIAN	n/a	11.75	3.00	515.08	515.79	515.08	515.07	515.79	515.78	1	0	-	-	-		-

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

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6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

SURVEY DATA POINT

0 m 10 20 30 40

1 : 1000

PREPARED FOR:

Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Hydraulic Structure Datasheet (HS-05)
Pedestrian Bridge on Muskeg Creek
Lower (North) Footbridge

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-118		FIGURE NO: C-5
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-118.dwg - 122, PLOT DATE: 28-Jul-2020

IMAGE REF: 10140-SV.109

1. View looking downstream from the left bank.

IMAGE REF: 10140-SV.110

2. View looking upstream from the left bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS[1]				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
MUSKEG CREEK	MIDDLE FOOTBRIDGE	n/a	n/a	PEDESTRIAN	n/a	13.43	3.50	518.72	519.24	518.90	518.69	519.19	519.38	1	0	-	-	-		-

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

PREPARED BY:
SG1
SG1 Water Consulting Ltd.
7303 118A St NW, Edmonton, AB, Canada T6G 1V3
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NOTES:
1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.
2. TOPOGRAPHIC SURVEY DATA WERE COLLECTED BY TROUT HYDROGRAPHY INC. ON 21-MAY-2019.
3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
4. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

● SURVEY DATA POINT

0 m 10 20 30 40

1 : 1000

PREPARED FOR:
Alberta

PROJECT:
ATHABASCA FLOOD HAZARD STUDY

TITLE:
Hydraulic Structure Datasheet (HS-06)
Pedestrian Bridge on Muskeg Creek
Middle Footbridge

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-119		FIGURE NO: C-6
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-119.dwg - 123, PLOT DATE: 28-Jul-2020

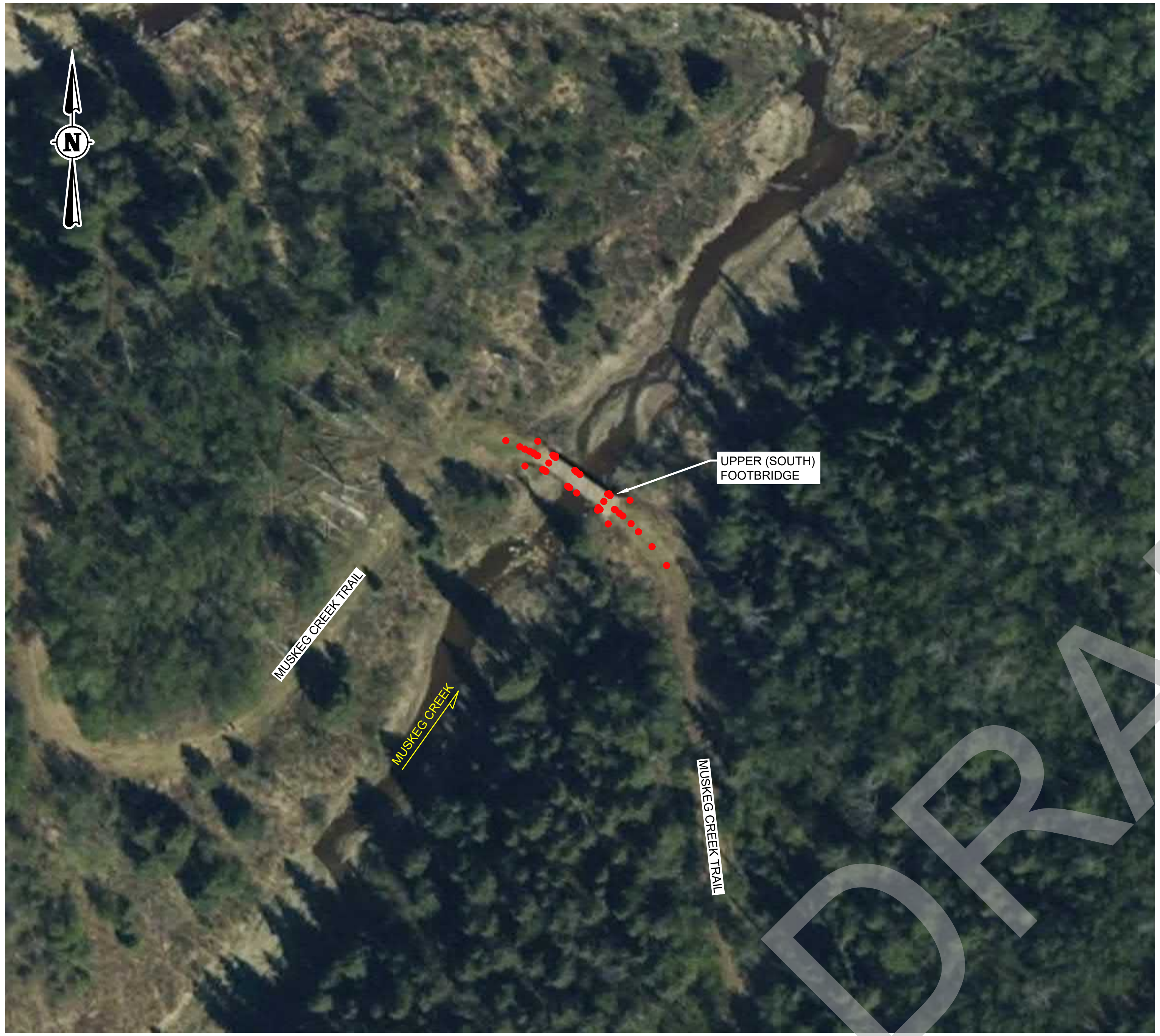

IMAGE REF: 10140-SV.121

1. View looking downstream from the right bank.

IMAGE REF: 10140-SV.118

2. View looking upstream from the right bank.

WATERBODY	STRUCTURE NAME / LOCATION	BRIDGE FILE NUMBER	RECORD HOLDER	USAGE	YEAR CONSTRUCTED	TOTAL LENGTH OF SPAN (m)	DECK WIDTH (m)	AVERAGE LOW CHORD ELEVATION (m)	AVERAGE TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)	LOW CHORD ELEVATION (m)		TOP-OF-CURB OR SOLID GUARD RAIL ELEVATION (m)		NUMBER OF SPANS	NUMBER OF PIERS	PIER DETAILS(1)				
										LEFT ABUTMENT	RIGHT ABUTMENT	LEFT ABUTMENT	RIGHT ABUTMENT			#	CENTRE STATION (m)	WIDTH (m) UPPER PORTION	WIDTH (m) LOWER PORTION	DESCRIPTION
MUSKEG CREEK	UPPER (SOUTH) FOOTBRIDGE	n/a	n/a	PEDESTRIAN	n/a	13.45	3.00	529.50	530.21	529.50	529.49	530.21	530.21	1	0	-	-	-		-

(1) Pier stationing is with respect to the centreline of the left abutment, viewed looking downstream (i.e., left abutment corresponds to station zero).

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NOTES:

1. ORTHOIMAGERY SHOWN HEREIN WAS ACQUIRED ON 25-MAY-2019. SOURCE: ALBERTA ENVIRONMENT AND PARKS.
2. TOPOGRAPHIC SURVEY DATA WERE COLLECTED BY TROUT HYDROGRAPHY INC. ON 22-MAY-2019.
3. DETAILS OF THE BRIDGE SURVEY WERE USED FOR HYDRAULIC MODELLING. PIER CENTRE STATIONS, AS SHOWN TABLE BELOW, ARE WITH RESPECT TO VALUES USED IN THE NUMERICAL MODEL.
4. THE COORDINATE SYSTEM IS 3TM MERIDIAN 114° W REFERENCED TO VERTICAL AND HORIZONTAL DATUMS OF CGVD28 AND NAD83 (CSRS), RESPECTIVELY.
5. REFER TO SECTION 2.3 OF THE STUDY REPORT AND THE HYDRAULIC (HEC-RAS) MODEL FOR MORE INFORMATION.
6. LEFT OR RIGHT REFER TO DIRECTIONS AS SEEN BY AN OBSERVER LOOKING DOWNSTREAM.

LEGEND:

SURVEY DATA POINT

0 m 10 20 30 40

1 : 1000

PREPARED FOR:

Alberta

PROJECT:

ATHABASCA FLOOD HAZARD STUDY

TITLE:

Hydraulic Structure Datasheet (HS-07)
Pedestrian Bridge on Muskeg Creek
Upper (South) Footbridge

DWN BY: RDJ	CHK'D BY: DMS	REV NO: 1
DWG NO: 10140-02-120		FIGURE NO: C-7
DATE: 29-JUL-2020		

FILE LOC: H:\SG1\OwnCloud\Drafting\10140 Athabasca FHS\Task 2\10140-02-120.dwg - 124, PLOT DATE: 28-Jul-2020

APPENDIX D

Open Water Hydrology Assessment

GOLDER

TECHNICAL MEMORANDUM

DATE 31 March 2020 **Project No.** 19117524-2000

TO Abdullah Mamun
Alberta Environment and Parks

CC Nathan Schmidt and Dejiang Long

FROM Getu Biftu **EMAIL gbiftu@golder.com**

OPEN WATER HYDROLOGY ASSESSMENT

1.0 INTRODUCTION

1.1 Study Area and Scope

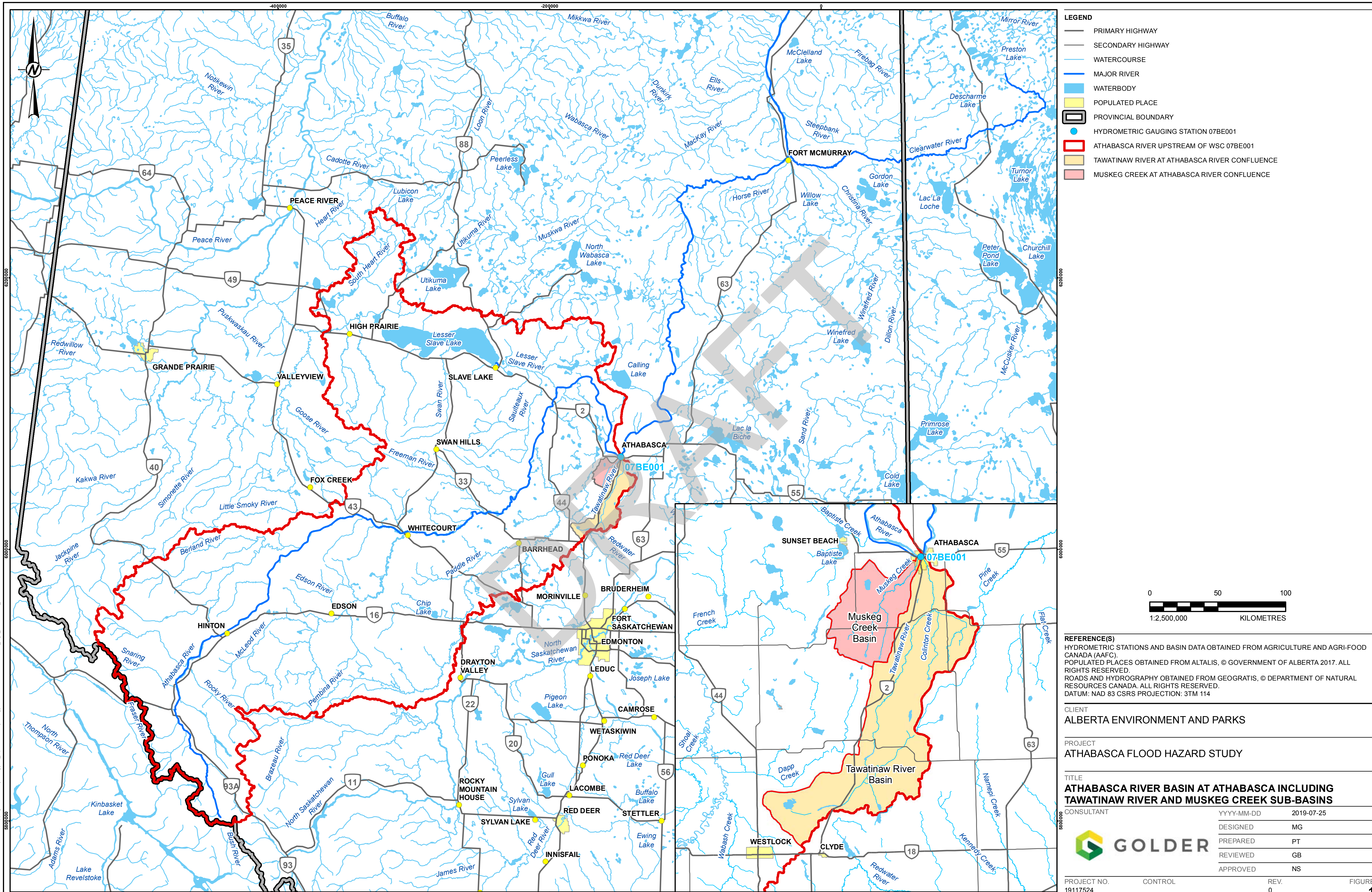
Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder), in March 2019 to conduct the Athabasca Flood Hazard Study. The purpose of the study is to assess and identify river and flood hazards along an approximately 6 km reach of the Athabasca River, an approximately 7 km reach of the Muskeg Creek, and an approximately 6 km reach of the Tawatinaw River through the Town of Athabasca and adjacent areas of Athabasca County (see Figure 1).

The study is part of the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Athabasca, Athabasca County, and the public.

The study comprises multiple components and deliverables. This memorandum documents the methodology and results of the open water hydrology assessment that will support the hydraulic modelling and open water flood mapping. The individual tasks associated with this hydrology assessment component include the following:

- **Data Series Preparation:** Compile available peak flow information for gauged locations and prepare flood flow data series.
- **Flood Frequency Analysis:** Conduct frequency analyses to estimate flood flows for return periods ranging from 2 to 1,000 years using the recorded and derived flood peak flow data for the available periods of record up to 2018.
- **Climate Change Commentary:** Provide comments and insight into how climate change processes may impact the flood peak discharges and flood frequency estimates.

The flood frequency estimates obtained in this study are the most up-to-date for the locations in the study area. These estimates provide the updated flood hydrology information as flow inputs to hydraulic modelling.



Classification: Public

1.2 Study Objectives and Results

The primary study objective is to identify and assess river-related hazards. The objective of the open water hydrology assessment is to generate flood peak discharge estimates along the study reaches of the Athabasca River, Muskeg Creek and Tawatinaw River. The results of the frequency analysis include estimates of the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year open water flood peak discharges.

This study includes the use of preliminary estimates provided by Environment and Climate Change Canada, Water Survey of Canada (WSC) of the annual peak flows in 2017 and 2018 for the Athabasca River at Athabasca. Including these provisional data increases the sample sizes for the flood frequency analyses and reliability of the resulting flood frequency estimates.

It is important to note that the 2017 and 2018 annual maximum instantaneous discharges used in this study are provisional and preliminary and may be subject to change when reviewed and corrected by the WSC. Therefore, the flood frequency statistics presented in this memorandum should be used with caution and reviewed when the finalized discharge values are available.

1.3 Watershed Setting and Historical Floods

The Athabasca River has its sources in the Rocky Mountains near Mount Columbia (elevation 3,747 m) and flows northeast for 1,300 km before discharging into the Peace-Athabasca Delta and Lake Athabasca (elevation 208 m) (RAMP 2016a). The river drains an area of approximately 74,602 km² at the gauging station at Athabasca (i.e., Athabasca River at Athabasca, WSC Station No. 07BE001).

As a major river system, the Athabasca River is influenced by a variety of climate, terrain and landscape characteristics of its basin (RAMP 2016b). The seasonal climate is a major factor affecting the river flow conditions. The climate is characterized by cold winters when most of the seasonal precipitation falls as snow, followed by warm summers when snow and glacial melt from the river's headwaters combine with runoff from localized snowmelt and rainfall events throughout the basin.

The Athabasca River flows through the Town of Athabasca. The downtown and residential sections of the Town are located south of the former industrial section and rail yards that were located on the south bank of the Athabasca River and redeveloped into the highway, commercial, industrial and park uses. The Tawatinaw River and Muskeg Creek flow north from the eastern and western boundaries of the Town into the Athabasca River.

The Tawatinaw River originates south of Rochester and flows in the northerly direction through the hamlet. It has a drainage area of about 865 km^2 at the Town of Athabasca. The drainage basin primarily consists of pasture land with some areas covered by trees. The drainage basin is not well drained as there are several sloughs and lakes. Muskeg Creek has similar drainage characteristics as Tawatinaw River and has a drainage area of about 275 km^2 at the Town of Athabasca.

The Town has experienced flooding periodically since 1914. The last large flood event occurred in 2011. Available records indicate that major open-water floods occurred in 1914, 1944, 1948, 1954, 1965, 1969, 1971, 1972, 1980, 1982, 1986, 2011 and 2012.

The 1954 flood was the highest on record to date, based on local newspapers and the available WSC records. The 1944 flood was the second largest and the 2011 flood the third largest[1] recorded for the Athabasca River at Athabasca. These floods were typically associated with high rainfall or rain-on snow events in June and July.

2.0 AVAILABLE FLOW DATA

2.1 Recorded Data

Recorded flow data from the WSC website are available at one location (i.e., the Athabasca River at Athabasca) within the study area. Preliminary annual maximum instantaneous discharge data in 2017 and 2018 were obtained from WSC. The flood frequency estimates for the Muskeg Creek and Tawatinaw River were derived based on the results of a regional analysis of flood peak discharges.

Table 1 provides a summary of the basic hydrologic information used to derive the flood frequency estimates for the study area. The data details are provided in Appendix A.

Table 1: Summary of Gauged Stations used in the Study

WSC Station Number	WSC Station Name	Latitude	Longitude	Gross Drainage Area (km^2)	Effective Drainage Area (km^2)	Period of Record	Length of Record (years)
07BE001	Athabasca River at Athabasca	54° 43' 19"	113° 17' 16"	74,602	73,332	1913 – 2018	100
Gauging Stations Included in the Regional Analysis							
05EC002	Waskatenau Creek near Waskatenau	54° 07' 23"	112° 46' 58"	313	207	1966 – 2018	53
05EC004	Namepi Creek near the Mouth	54° 01' 47"	112° 50' 44"	720	586	1975 – 1995	21
05EC005	Redwater River near the Mouth	53° 53' 48"	112° 59' 46"	1,596	1,170	1978 – 2018	38
07BE003	Porter Creek above Baptiste Lake	54° 43' 31"	113° 35' 10"	57	57	1980 – 2018	36
07CA003	Flat Creek near Boyle	54° 35' 15"	112° 54' 24"	184	97	1919 – 2018	49
07CA005	Pine Creek near Grassland	54° 49' 13"	112° 46' 39"	1,456	995	1966 – 2018	52
07CA008	Babette Creek near Colinton	54° 39' 09"	113° 04' 46"	219	68	1978 – 2018	41

2.2 Historic Data

There are no additional historic flow data available for the study area before systematic gauging and monitoring by the WSC.

[1] Note that the 2011 flood was the third largest on record, with a maximum instantaneous flow of 4,850 m^3/s on July 12. However, the corresponding maximum mean daily flow of 3,810 m^3/s on July 11 was the sixth largest on record, after the 1954, 1944, 1971, 1986 and 1971 floods. The anomalous difference between the maximum instantaneous and maximum mean daily flows may be related to the timing of the 2011 peak (01:00 on July 12), that split the flood volume over two calendar days. The other major floods peak between 08:00 to 18:09, meaning the rising and falling limbs of the hydrograph were largely contained within one calendar day.

2.3 Previous Studies

This study included a review of a number of background documents, including previous hydrology and flood studies. Several hydrology studies were completed for the Athabasca River over the last two decades. Some of these studies included assessments of open water hydrology. These studies include the following:

- Athabasca Hydraulic Study, Environment Canada (1993);
- Climate Change Assessment for Athabasca River Basin (Golder 2013);
- Hydro-Climate Model Selection and Application on the Athabasca and Beaver River Basins (Golder 2009);
- Athabasca River Basin - Feasibility Study (IBI and Golder 2014); and
- Regional Hydro-Climatic and Sediment Modeling (Droppo et al. 2018).

The review involved documentation of the assumptions, limitations, and understanding of the hydrologic techniques applied in the past studies. The results of these past studies provided a frame of reference for interpretation of the results and comparison to this study. The review helped identify data gaps and apparent discrepancies in the data that may affect their use in subsequent analyses.

3.0 PREPARATION OF FLOOD FLOW DATA SERIES

3.1 Introduction

Preparation of the flood flow series involved consideration of a large number of factors, including flow changes at the confluences of the Athabasca with the Muskeg Creek and Tawatinaw River, unequal and non-overlapping record lengths, and incomplete flow records. The methods used to compile the flood flow series and to address the data gaps are described in the following sections.

3.2 Flood Flow Series for the Gauged Location

The flood frequency estimates for the gauged location [i.e., the Athabasca River at Athabasca (WSC Station No. 07BE001 / Node 100)], were derived based on the recorded annual maximum instantaneous discharge series, and where there are missing data, the annual maximum daily discharges that were used to estimate the instantaneous flood flows.

The following method was used for estimating the annual maximum instantaneous discharges based on the annual maximum daily discharges to fill the data gaps in the record:

- Annual maximum daily discharge series were developed using the recorded daily flow series.
- A relationship was established between event-based annual maximum daily and annual maximum instantaneous discharges in the record. If the reported annual maximum daily and annual maximum instantaneous discharges for the same year were not coincident (i.e., from the same flood event), the former values were replaced by the daily flow values for the events corresponding to the annual maximum instantaneous discharges. This relationship was used to estimate the annual maximum instantaneous discharges based on the recorded annual maximum daily discharges.

3.3 Flood Flow Series for the Ungauged Locations

Empirical relationships between drainage areas and flood peak discharges were established based on available regional flow records for the return periods ranging from 2 to 1,000 years. The relationships were then used to derive the flood frequency estimates for the two ungauged tributaries in the study area (i.e., the Muskeg Creek and Tawatinaw River).

The flood frequency estimates for the Muskeg Creek and Tawatinaw River were obtained as follows:

- The drainage areas at the WSC stations were compiled. The gross drainage areas at the ungauged tributary locations were estimated in a GIS analysis.
- The flood frequency estimates for the WSC stations (see Appendix B) were obtained based on the annual maximum instantaneous flow series.
- Regional relationships between drainage area and peak discharge for a range of return periods (i.e., 2 to 1,000 years) were developed, as shown in Figure 2.
- The resulting regional relationships were then used to estimate the flood peak discharges at the two ungauged locations for the various return periods and the 95% confidence intervals.

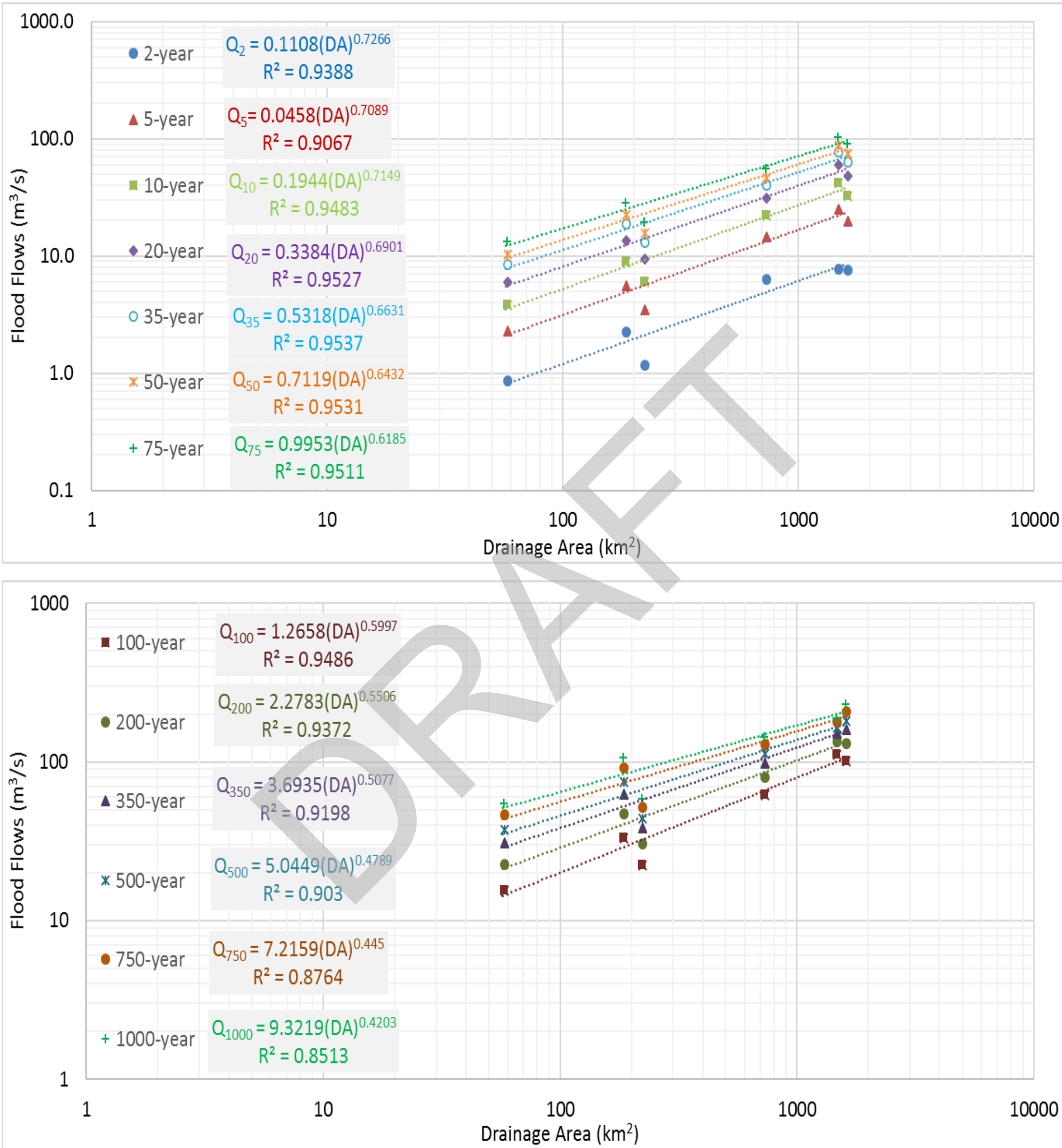


Figure 2: Empirical Relationships between Flood Flows and Drainage Areas for Regional Stations Identified in Table 1

4.0 FLOOD FREQUENCY ANALYSIS

4.1 Statistical Tests

4.1.1 Methodology

Prior to fitting the appropriate frequency distribution to the flood flow data, a number of statistical tests were performed to determine the quality of the developed annual maximum instantaneous discharge series. Software developed by Golder that is similar to Environment Canada's Consolidated Frequency Analysis (CFA), but with enhanced methodology, was used for: (i) flood frequency analyses and statistical tests for independence (not serially correlated); and (ii) trend, randomness, and homogeneity tests. Golder's software includes modern boot-strapping method and estimation of confidence intervals.

The following probability distributions were analyzed with select parameter estimation methods (i.e., method of moments [Moment], maximum likelihood estimation [MLH], and Method of L-moments [MLM]):

- Three-parameter Log Normal distribution (3P, Moment and MLH);
- Generalized Extreme Value distribution, which includes Extreme Value 1, 2, and 3 distributions (EV, MLM);
- Log-Pearson Type III distribution (LP3, Moment, and MLH); and
- Weibull distribution (Moment).

Numerical goodness-of-fit tests were performed using the non-parametric Anderson-Darling test (Stephens 1974).

4.1.2 Results

Table 2 provides the results of statistical tests for the recorded flood flow series for Athabasca River at Athabasca. The results of statistical analysis for the regional stations are provided in Table A-2, Appendix A. The results show that most of the annual maximum instantaneous flood flow series are independent, random, homogeneous, and do not display any significant trends. The results are highlighted and discussed below:

- The annual maximum instantaneous discharge series for Waskatenau Creek near Waskatenau, displays dependence, trend, non-homogeneity, and non-randomness at both the 5% and 1% level of significance and non-randomness at the 5% level of significance. This does not appear to be due to any large-scale climate change (i.e., relatively long dry and wet hydrologic cycles) and appears to be somehow affected by anthropogenic change (i.e., depression storage for agricultural development). Therefore, this station was excluded from use in developing the regional relationships.
- The annual maximum instantaneous flow series for the Redwater River near the mouth and Porter Creek above Baptiste Lake display dependence at the 5% level of significance but not at the 1% level of significance. Obtaining data that is perfectly independent is almost impossible, because such factors as data length and period of record can affect the outcome of the statistical tests. Because of this consideration, the flow series for the Redwater River and Porter Creek were included in developing the regional relationships.

Table 2: Results of Statistical Tests of Annual Maximum Instantaneous Discharges and Goodness-of-Fit of Probability Distribution Functions

WSC Station ID	**07BE001 (Node 1)**
WSC Station Name or Location of Interest	**Athabasca River at Athabasca**
Anderson-Darling statistic, $A^2 = -N - S$	
3 Parameter Log-normal	0.320
Extreme Value	0.255
Log-Pearson III	0.275
Weibull	1.478
Serial correlation coefficient test for independence	
S_1	0.016
t	0.157
t(α=0.05)	1.661
t(α=0.01)	2.366
Spearman rank order correlation coefficient test for no-trend	
r_s	0.020
t	0.201
t(α=0.05)	1.985
t(α=0.01)	2.627
Mann-Whitney split sample test for homogeneity	
Size of earlier sample	50
z	-0.420
z(a=0.05)	-1.645
z(a=0.01)	-2.326
Test of general randomness (Runs for above or below the median)	
Median	1856.7
N1(for Q>=Median)	50
N2(for Q<Median)	49
Run_ab	52
z	0.304
z(a=0.05)	1.960
z(a=0.01)	2.576

Notes: 1. *Selected distribution based on best statistical fit 0.320*

4.2 Flood Frequency Estimates

Flood frequency analyses of the annual maximum instantaneous discharge series (that includes the preliminary estimates of the 2017 and 2018 flood flows) at one location within the study area (i.e., Athabasca River at Athabasca) and for the regional stations, were conducted to estimate the discharges of various return periods of floods (i.e., 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year floods).

Table 3 summarizes the flood discharge estimates and the associated upper and lower 95% confidence intervals. The annual maximum instantaneous discharge series used in the flood frequency analyses, the various frequency distributions, and the best-fit distributions along with their 95% confidence intervals are provided in Appendix B.

4.3 Comparison to Previous Studies

Table 4 presents a comparison of the flood frequency estimates obtained in this study for the Athabasca River at Athabasca, Muskeg Creek, and Tawatinaw River with the studies previously completed by Environment Canada (EC 1993) as well as IBI and Golder (2014).

The flood frequency estimates were based on the recorded data up to 1989 in the EC study (1993) and up to 2011 in the IBI and Golder study (2014). The current study is based on the published flow data up to 2016, the provisional flow data for 2017 to 2018 from WSC for Athabasca River at Athabasca, and the published data up to 2018 for some of the regional gauging stations. In addition, this study includes the analyses to update the relationships between annual maximum daily and annual maximum instantaneous discharges.

The results of the comparison are summarized below:

- The resulting flood frequency estimates of this study for the Athabasca River at Athabasca are lower than those in the EC (1993) and IBI and Golder (2014) studies.
- The resulting flood frequency estimates of this study for the Muskeg Creek at the Town of Athabasca are higher than those in the EC (1993) and IBI and Golder (2014) studies. In the IBI and Golder (2014) study, flood estimates for Muskeg Creek assumed to be the same as flood estimates for Babette Creek near Colinton (i.e., WSC Station No. 07CA008 assumed as regional index station).
- The resulting flood frequency estimates of this study for the Tawatinaw River at the Town of Athabasca are comparable to the results in the EC (1993) study but lower than the IBI and Golder (2014) study.

The main differences in the flood frequency estimates are due to the different lengths of the recorded data used in the flood frequency analyses and the selections of different frequency curve distributions. In addition, there is a difference in the watershed area of the Tawatinaw River at Athabasca in the IBI and Golder (2014) study and this study.

Table 3: Flood Peak Discharge Estimates and their 95% Confidence Intervals

WSC Station ID / Node ID	WSC Station Name or Location of Interest	Gross Drainage Area (km^2)	Flow Type[1]	Distribution[(2)]	Recommended Instantaneous Flood Flows (m^3/s)																	
					1000-yr			750-yr			500-yr			350-yr			200-yr			100-yr		
					Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower
07BE001 / Node 100	Athbasca River at Athbasca (WSC Station No. 07BE001)	74602	N	EV2	9,340	12,500	6,080	8,820	11,600	5,850	8,120	10,500	5,540	7,540	9,520	5,280	6,700	8,260	4,890	5,760	6,880	4,420
Muskeg Creek / Node 200	Muskeg Creek at the Town of Athabasca	275	N	Regional	98.8	125	66.2	87.9	112	58.3	74.3	95	48.8	64	82.1	41.8	50.2	64.7	32.8	36.7	47.3	24.2
Tawatinaw River / Node 300	Tawatinaw River at the Town of Athabasca	865	N	Regional	160	180	123	146	164	112	129	144	99	114	128	89	94.3	105	74	73.1	81	58

WSC Station ID / Node ID	75-yr			50-yr			35-yr			20-yr			10-yr			5-yr			2-yr		
	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower	Flood Est.	Upper	Lower
07BE001 / Node 100	5,400	6,390	4,230	4,920	5,750	3,960	4,530	5,220	3,720	3,940	4,460	3,350	3,280	3,670	2,880	2,670	2,960	2,410	1,880	2,050	1,740
Muskeg Creek / Node 200	32.1	41.3	21.3	26.4	33.9	17.7	22	28.2	14.9	16.3	20.8	11.2	10.8	13.6	7.54	6.56	8.27	4.63	2.46	3.11	1.71
Tawatinaw River / Node 300	65.2	72	52	55.1	60	45	47.1	51	39	36.0	39	30	24.5	26	21	15.1	16	13	5.5	6	5

Note:
1. Flow Type: N: Natural.
2. Confidence intervals for the regional analysis were determined based on confidence interval of best fitted regional curve.

Table 4: Comparison of the Flood Frequency Estimates of Various Studies

Return Period (years)	Athabasca River at Athabasca (WSC Station 07BE001)			Muskeg Creek at the Town of Athabasca			Tawatinaw River at the Town of Athabasca		
	EC (1993)	IBI and Golder (2014)	This Study	EC (1993)	IBI and Golder (2014)	This Study	EC (1993)	IBI and Golder (2014)	This Study
	Log Pearson Type III	EV2	EV2	Log Normal	Reginal Index Station	Regional Analysis	Log Pearson Type III	Regional Analysis	Regional Analysis
2	-	1,873	1,877	-	2	2.46	-	8	5.5
5	-	2,729	2,669	-	4	6.56	-	21	15.1
10	3,420	3,383	3,280	10	5	10.8	35	33	24.5
20	-	4,070	3,942	-	7	16.3	-	48	36.0
25	-	4,301	-	-	8	-	-	53	-
50	5,250	5,050	4,924	23	10	26.4	55	70	55.1
100	6,200	5,846	5,765	30	12	36.7	65	98	73.1
500	-	7,961	8,121	-	19	74.3	-	144	129

Notes:
1. The EC (1993) study involved use of the recorded data up to 1989 for the Athabasca River at Athabasca.
2. The IBI and Golder (2014) study involved use of the recorded data from 1913 to 2011 for the Athabasca River at Athabasca. In the IBI and Golder (2014) study, flood estimates for Muskeg Creek were assumed to be the same as flood estimates for Babette Creek near Colinton (i.e., WSC Station No. 07CA008).

5.0 POTENTIAL EFFECTS OF CLIMATE CHANGE ON FLOOD PEAK DISCHARGES AND FLOOD FREQUENCY ESTIMATES

Recent studies on the effects of climate change (e.g., Martz et al. 2007; Droppo et al. 2018) indicate that climate change could result in increased air temperature, more frequent drought and water shortages, increased precipitation in some areas, and increased flooding. As a result of the expected change in both the systematic climate and its variability, many regions of Canada, including the Prairies, could experience warmer air temperatures and changes in stream flow magnitude and timing (e.g., higher winter stream flows, early spring peak streamflow, and lower summer stream flows).

The Droppo et al. (2018) review of several studies indicates with high confidence that projected increases in extreme precipitation are expected to increase the potential for future urban flooding. There is medium confidence that projected higher temperatures will result in a shift toward earlier floods associated with spring snowmelt, ice jams, and rain-on-snow events. However, it is uncertain how projected higher temperatures and reductions in snow cover will affect the frequency and magnitude of future snowmelt-related flooding.

Assessment of future climate scenarios depends on the climate model used for the prediction. Regardless, precipitation is projected to increase in Alberta, with less precipitation falling as snow and more rain-on-snow precipitation events (Valeo et al. 2007). Therefore, it is anticipated that such changes in precipitation patterns could increase the frequency and intensity of extreme events (i.e., flood, drought, hail, and windstorms). It is also predicted that the flood events for the Athabasca River watershed could occur earlier in the spring than in the past if rain-on-snow events occur more frequently and the snowpack begins to melt earlier.

Golder (2013) completed an assessment of the effect of climate change using five selected representative GCMs and scenarios outputs from the IPCC Fourth Assessment Report (AR4) (IPCC 2007) for the Athabasca River basin. The five selected scenarios represented climate conditions that were cooler and drier (BCM2.0 SR-B1), cooler and wetter (INMCM3.0 SR-A2), warmer and wetter (MIROC3.2 hires SR-A1B), and warmer and drier (CNRMCM3 SR-A2) than median conditions (CGCM3T47 SR-B1).

The forecasted total climate change is between the modelled baseline period (1961 to 1990) as represented by its 30-year average and the modelled future period (i.e., the period of 2051 to 2080 called the 2060s) as represented by its 30-year average. The results indicate that the changes in flood peaks for the Athabasca River watershed will vary from a slight decrease for the 2-year flood (i.e., less than 5%) to a slight increase (i.e., less than 10%) for the 100-year flood for the median climate change conditions. Therefore, the changes in the flood peak discharges for the Athabasca River are expected to be small for the median climate change projections.

The 1954 flood in the Athabasca River basin was the largest flood since 1913, as illustrated in Figure 3. Based on the recorded flow data for the past 104 years (i.e., 1913 to 2018), the annual peak flows for the Athabasca River do not appear to be trending upward. Any upward trend shown in Figure 3 is not statistically significant.

About 74 percent of the recorded annual peak discharges in the Athabasca River occurred between the beginning of June and the end of July (see Table 5 and Figure 4). The frequency of annual peak discharges occurring outside this period (earlier or later) does not appear to be changing with time. The recent pattern in the timing of these peak discharges is similar to what was observed at the beginning of the century. There is no clear evidence that the patterns in magnitude or timing of annual peak discharges have changed significantly over the past hundred years.

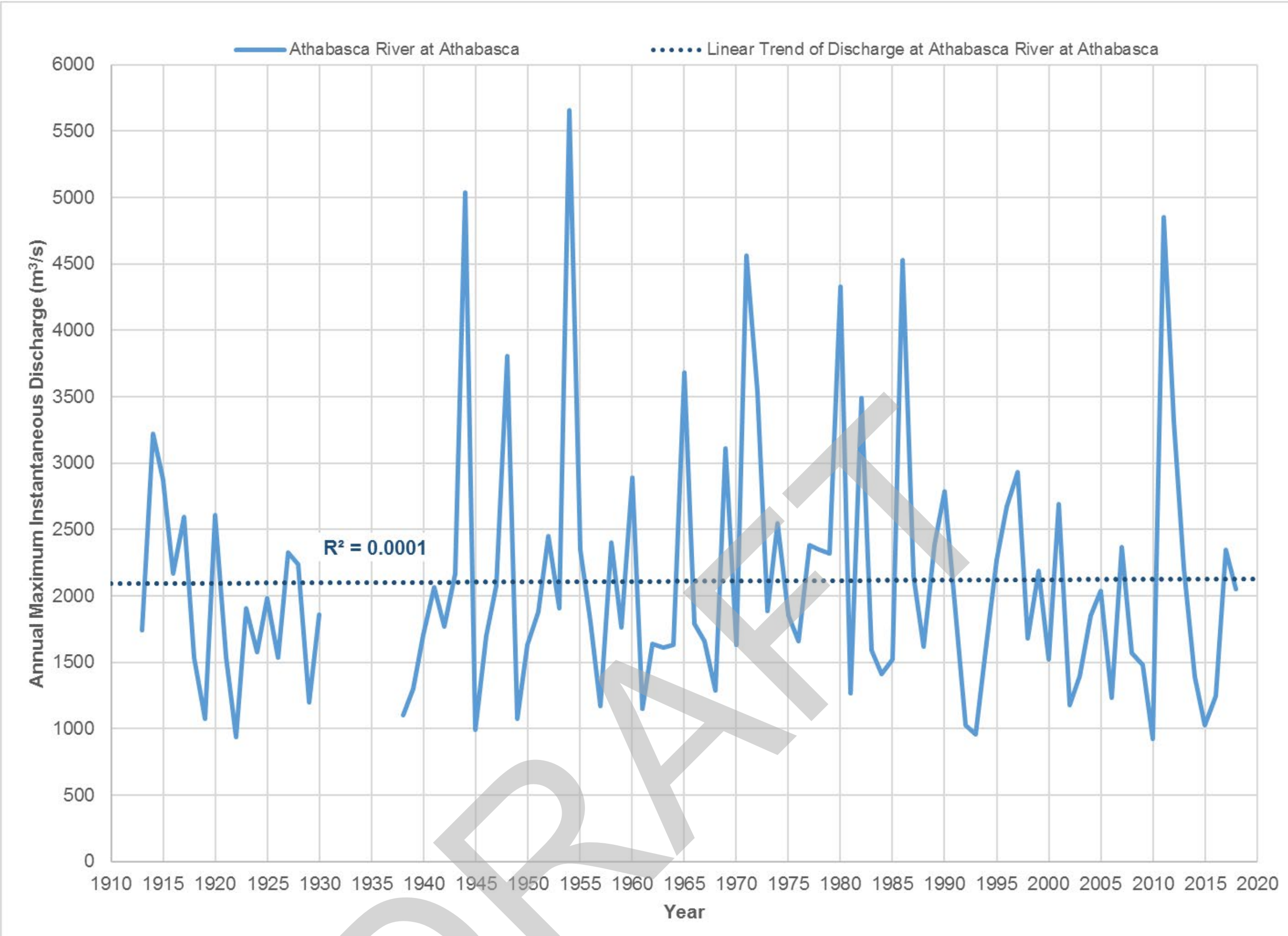


Figure 3: Annual Flood Peak Discharges on the Athabasca River at Athabasca

Table 5: Timing of Annual Maximum Instantaneous Flows in the Athabasca River at Athabasca (1913 – 2018)

Month	Number	%
April	3	3
May	10	10
June	47	47
July	26	26
August	11	11
September	2	2

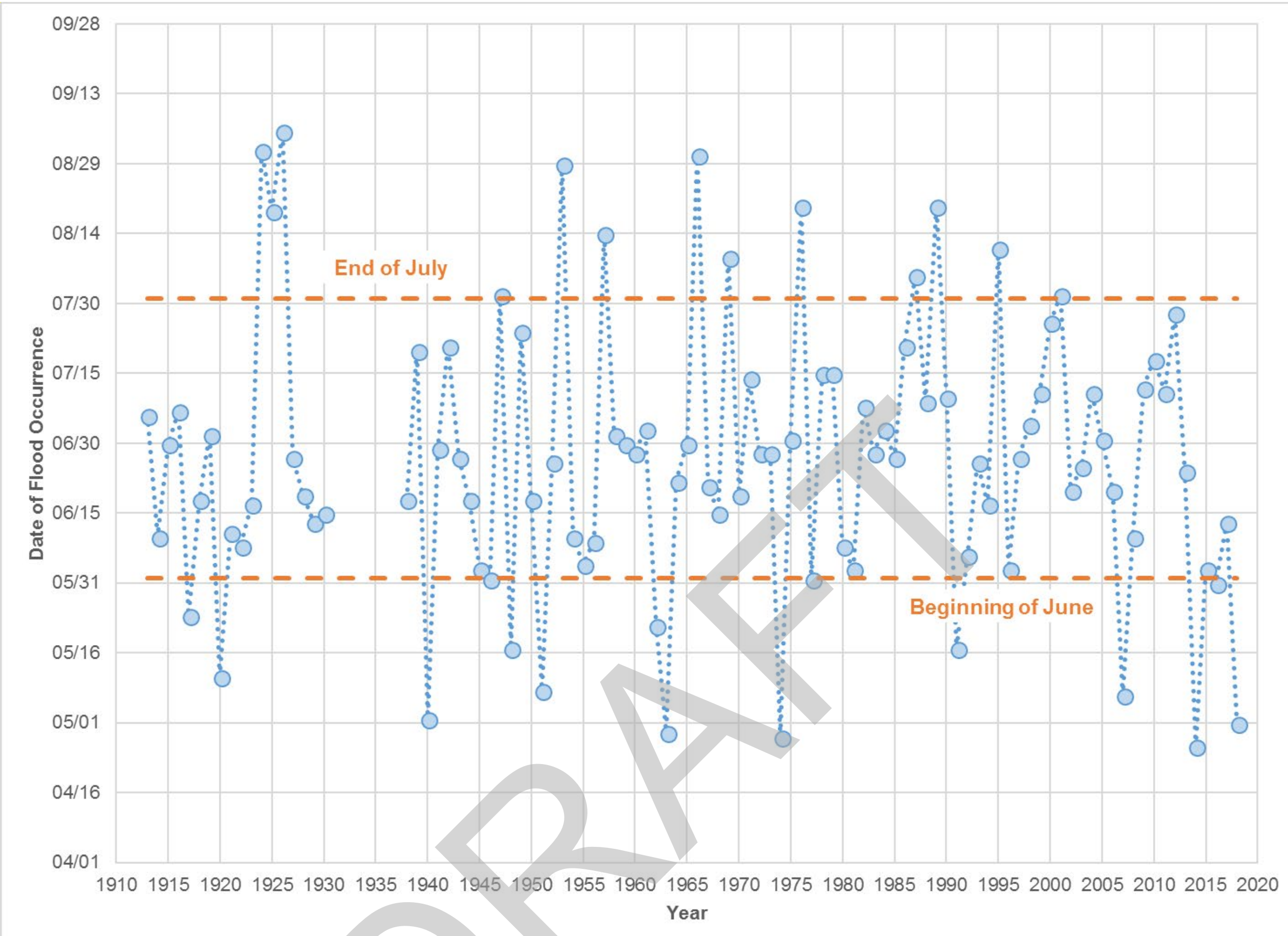


Figure 4: Timing of Past Annual Peak Flows in Athabasca River at Athabasca

6.0 CONCLUSIONS

The results of this hydrology assessment support the following conclusions:

- The flood frequency estimates obtained in this study are the most up-to-date for the Athabasca River at Athabasca, Muskeg Creek, and Tawatinaw River. These estimates provide the updated flood hydrology information as inputs to the other components of the study (e.g., hydraulic modelling). Table 3 summarizes the estimates of flood peak discharges for various return periods ranging from 2 to 1,000 years, and the 95% upper and lower confidence intervals.
- This study includes preliminary estimates of the annual maximum instantaneous discharges in 2017 and 2018. Inclusion of the additional discharge information increases the sample size for the flood frequency analyses and reliability of the resulting flood frequency estimates.
- The length of time period of the recorded flood flow data available and used in the flood frequency analyses for the Athabasca River at Athabasca, is 100 years. Therefore, there are large uncertainties (i.e., the confidence intervals are very large) with flood frequency estimates for return periods greater than 100 years.

7.0 CLOSURE

This memorandum is prepared and reviewed by the undersigned.

GOLDER ASSOCIATES LTD.

Prepared by: Reviewed by:

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https://golderassociates.sharepoint.com/sites/106774e/shared documents/phase 8000 - reporting and documentation/task 8.2 hydrology memorandum/final/athabasca river technical memorandum_rev0.docx

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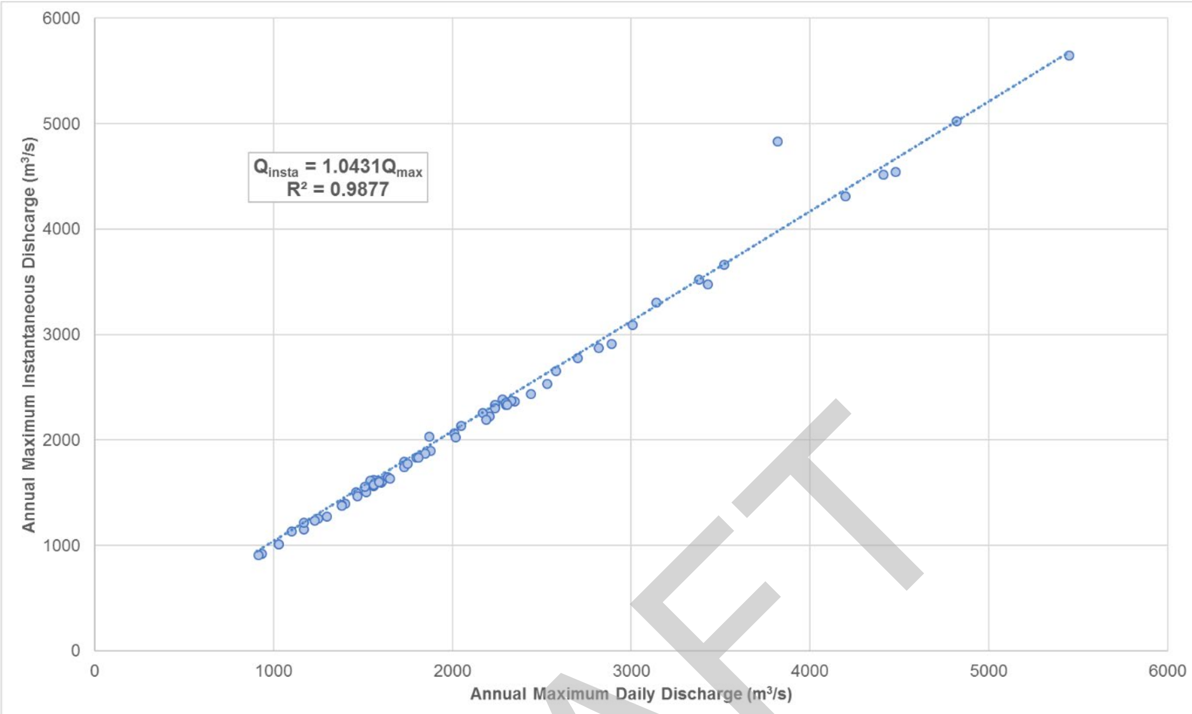
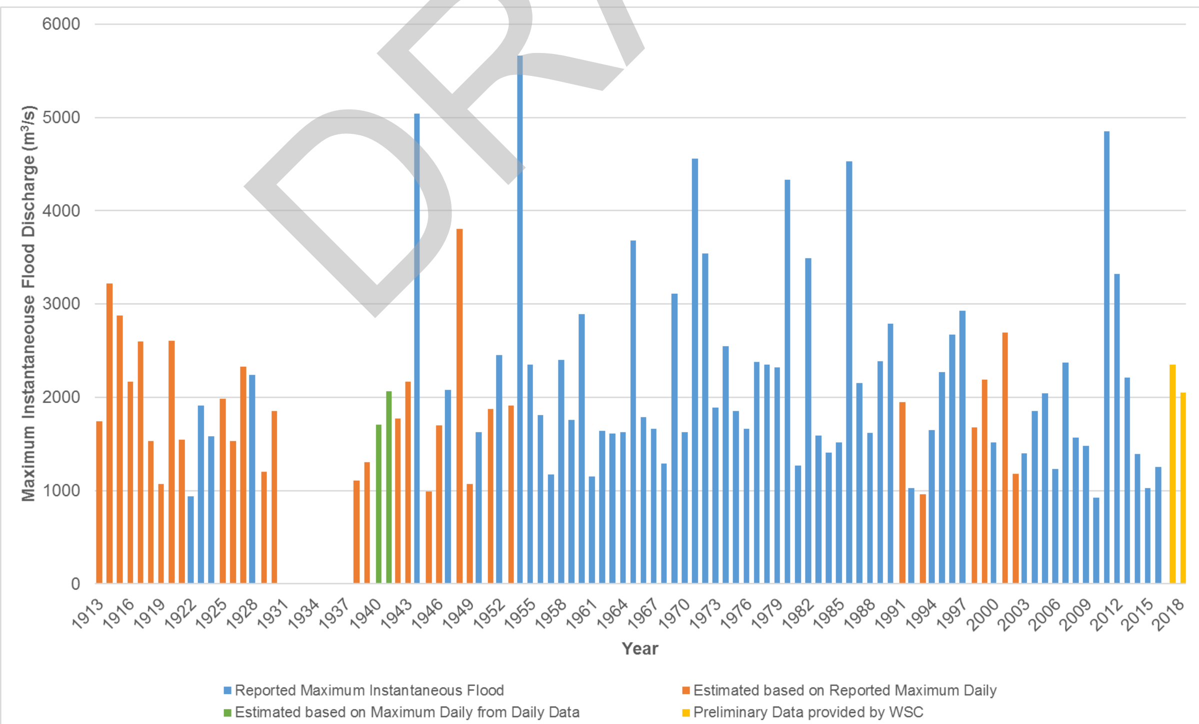
APPENDIX A

Graphical Summaries of Flood Flow Series at Gauged Stations and Locations of Interest

Figure A-1: WSC Station No. 07BE001, Athabasca River at Athabasca

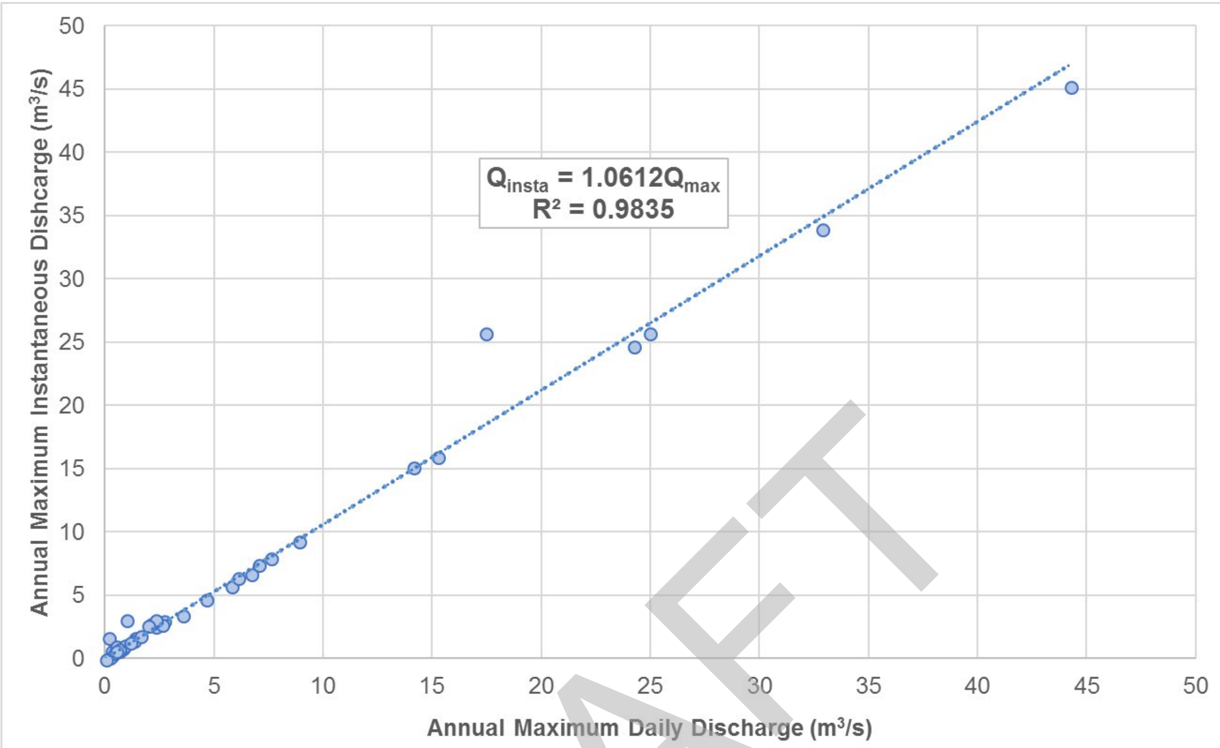


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Athabasca River at Athabasca (WSC Station No. 07BE001)

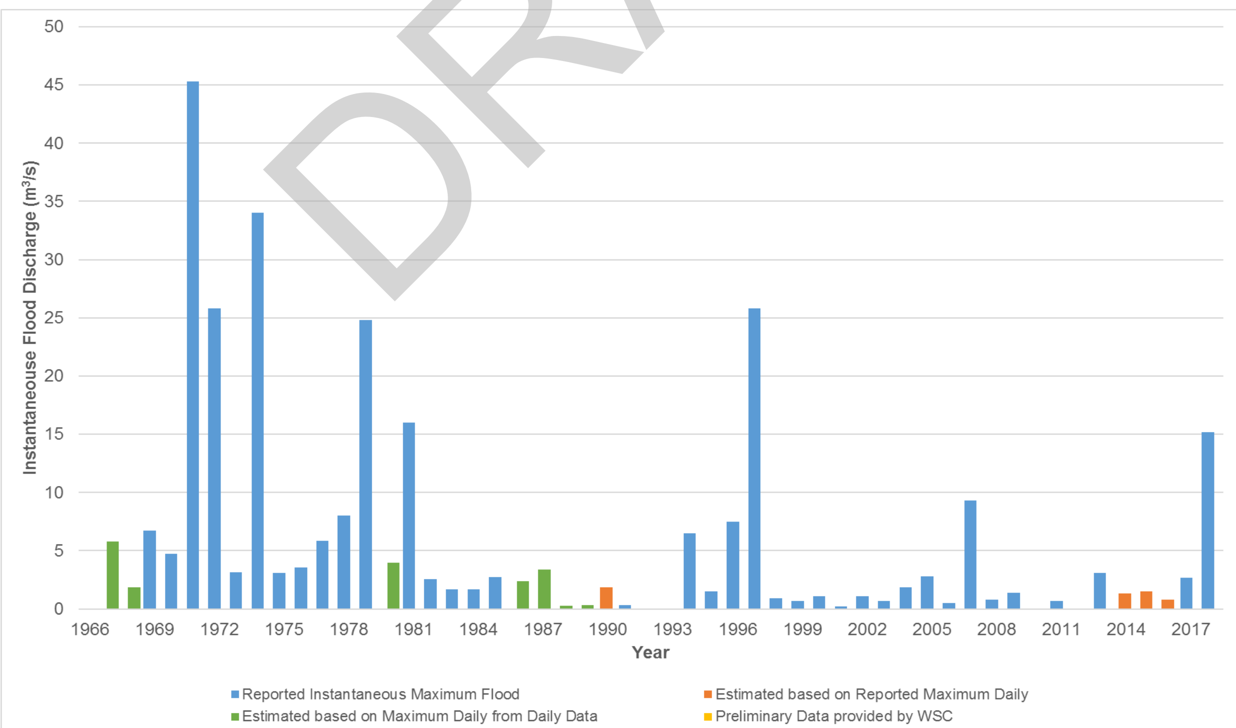


Maximum Instantaneous Flood Flow Series at Athabasca River at Athabasca (WSC Station No. 07BE001)

Figure A-2: WSC Station No. 05EC002, Waskatenau Creek near Waskatenau

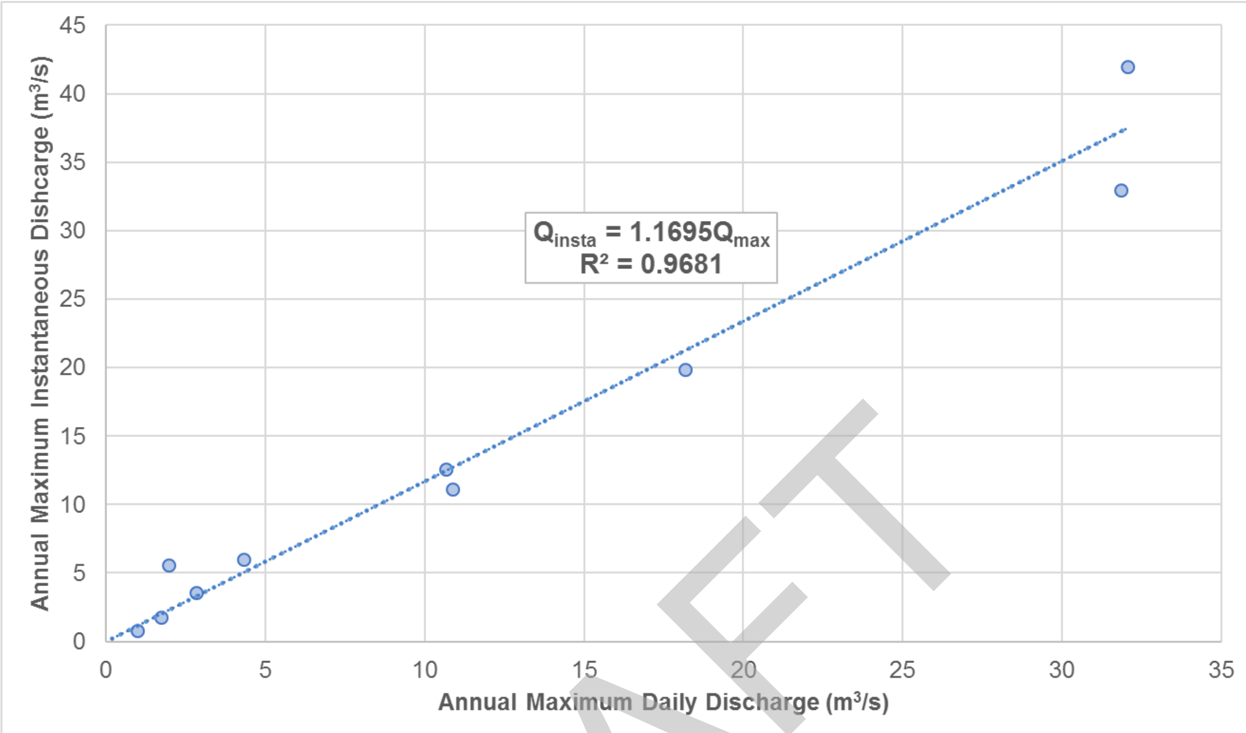


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Waskatenau Creek near Waskatenau (WSC Station No. 05EC002)

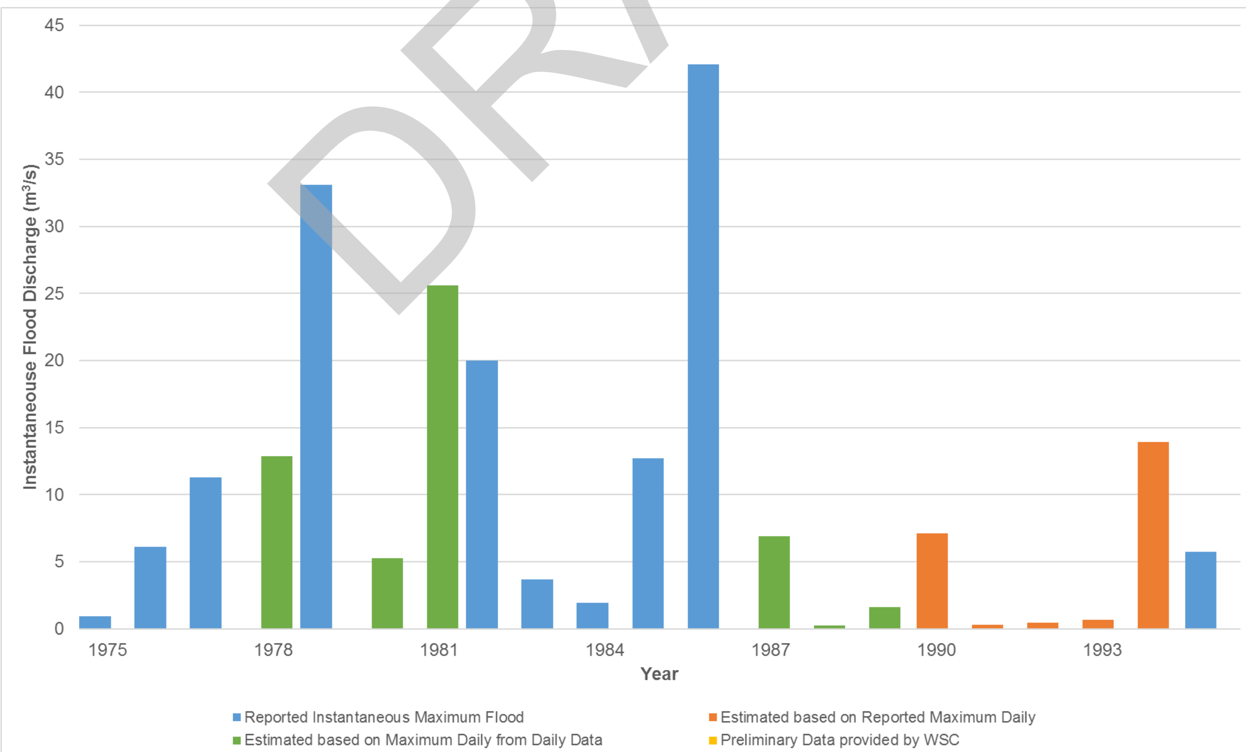


Maximum Instantaneous Flood Flow Series at Waskatenau Creek near Waskatenau (WSC Station No. 05EC002)

Figure A-3: WSC Station No. 05EC004, Namepi Creek near the Mouth

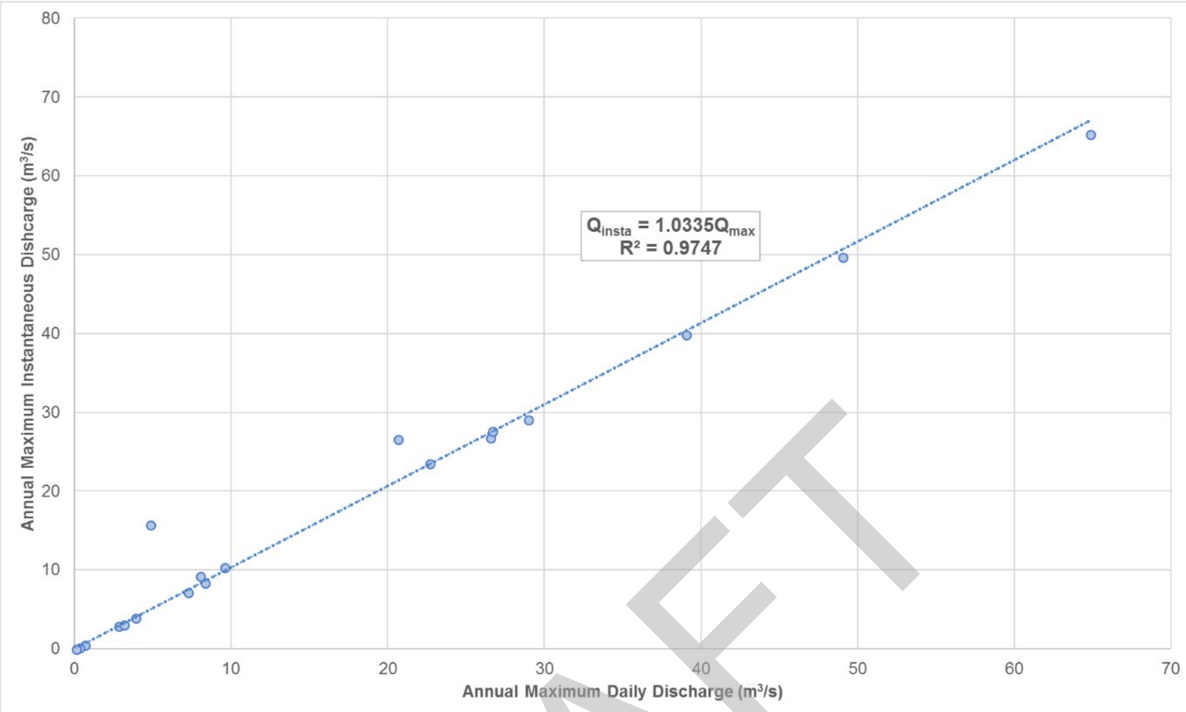


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Namepi Creek near the Mouth (WSC Station No. 05EC004)

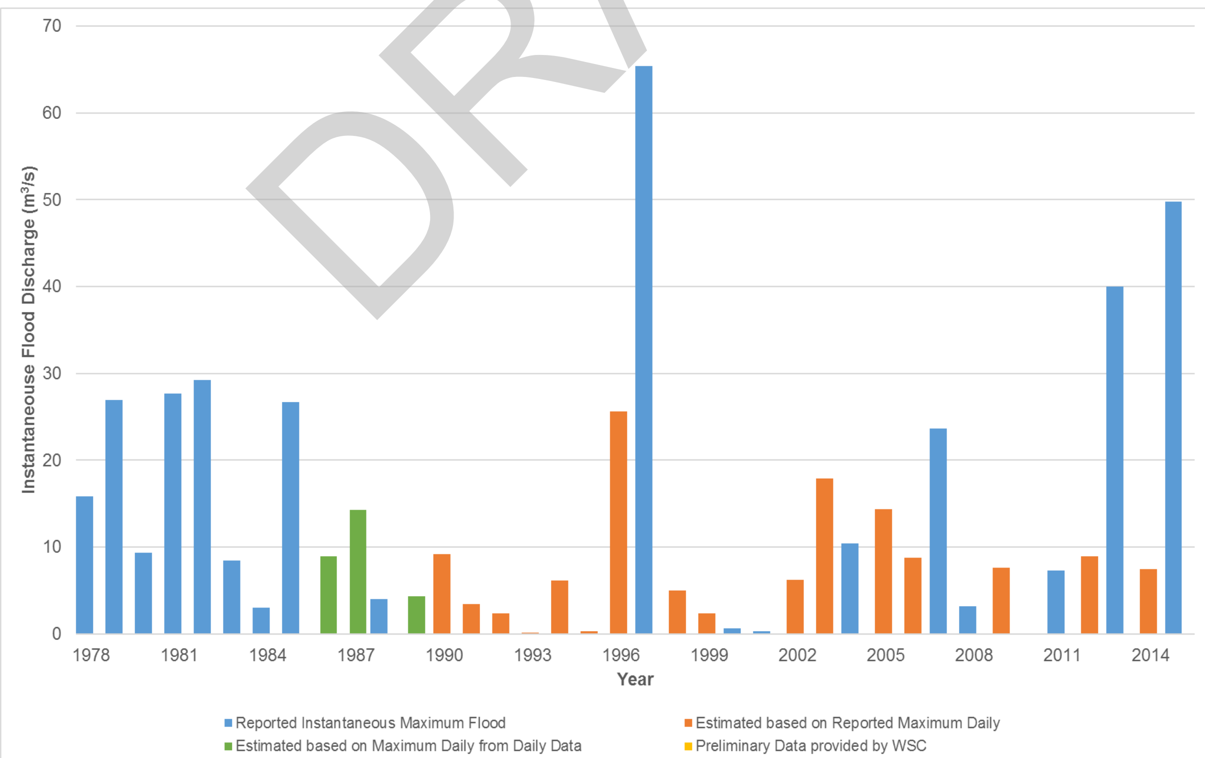


Maximum Instantaneous Flood Flow Series at Namepi Creek near the Mouth (WSC Station No. 05EC004)

Figure A-4: WSC Station No. 05EC005, Redwater River near the Mouth

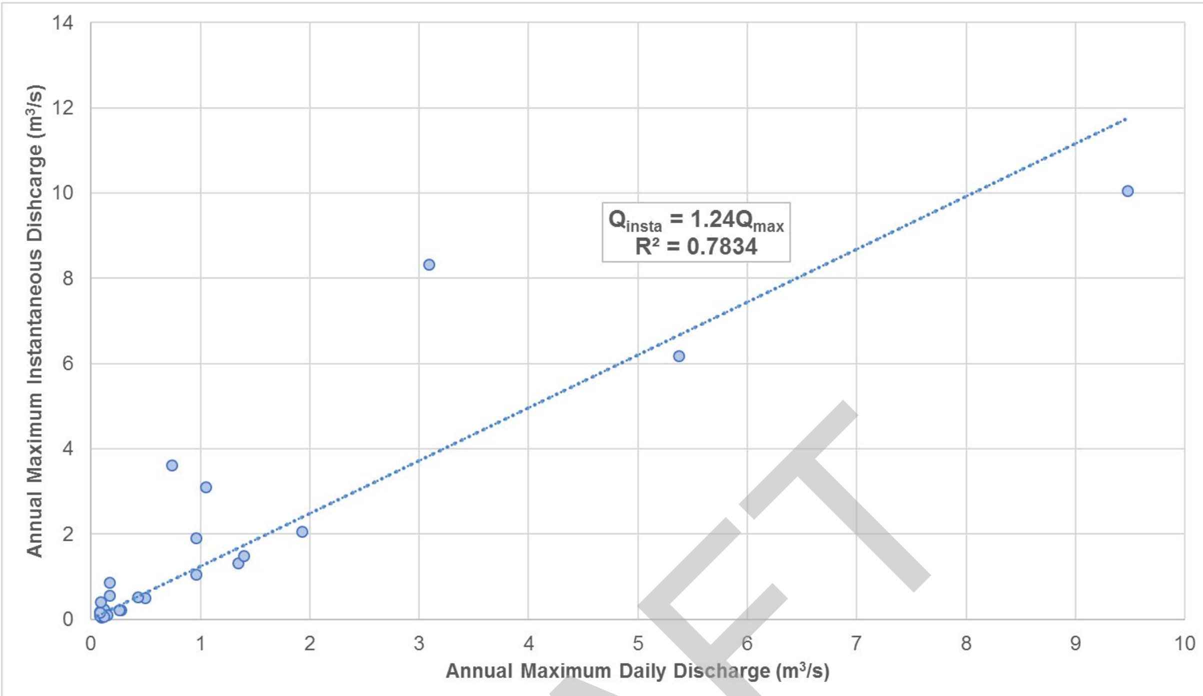


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Redwater River near the Mouth (WSC Station No. 05EC005)

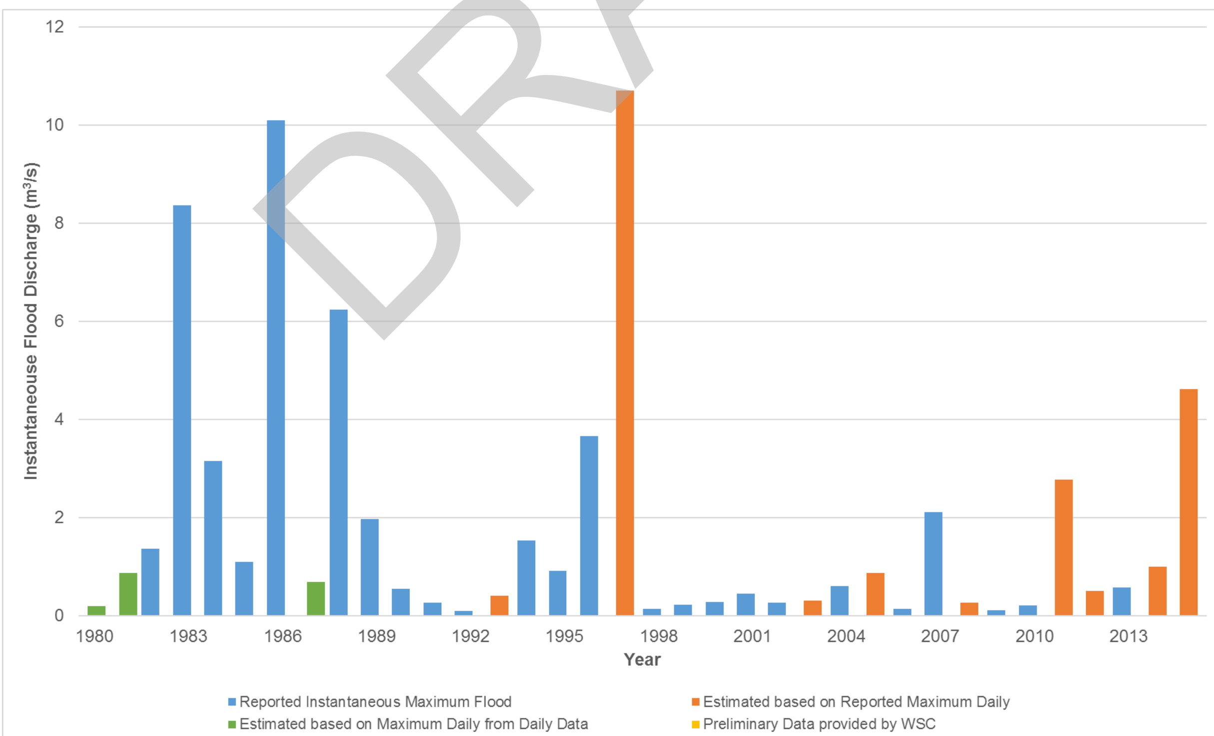


Maximum Instantaneous Flood Flow Series at Redwater River near the Mouth (WSC Station No. 05EC005)

Figure A-5: WSC Station No. 07BE003, Porter Creek above Baptiste Lake

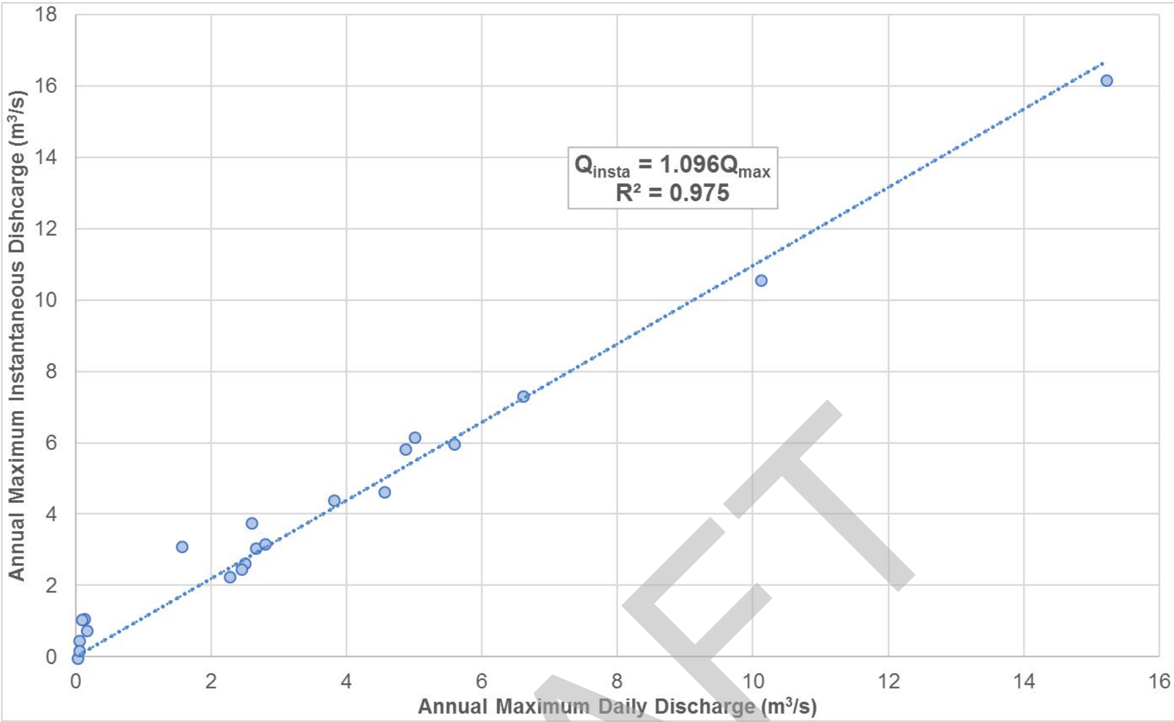


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Porter Creek above Baptiste Lake (WSC Station No. 07BE003)

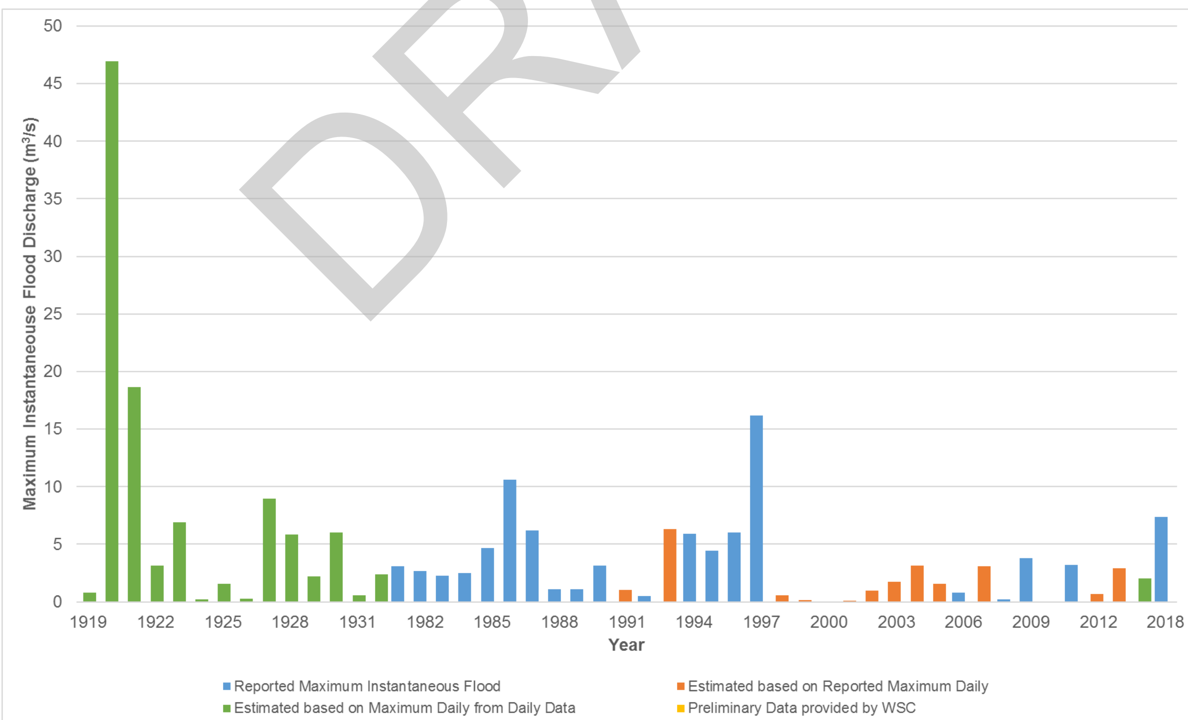


Maximum Instantaneous Flood Flow Series at Clearwater River at Porter Creek above Baptiste Lake (WSC Station No. 07BE003)

Figure A-6: WSC Station No. 07CA003, Flat Creek near Boyle

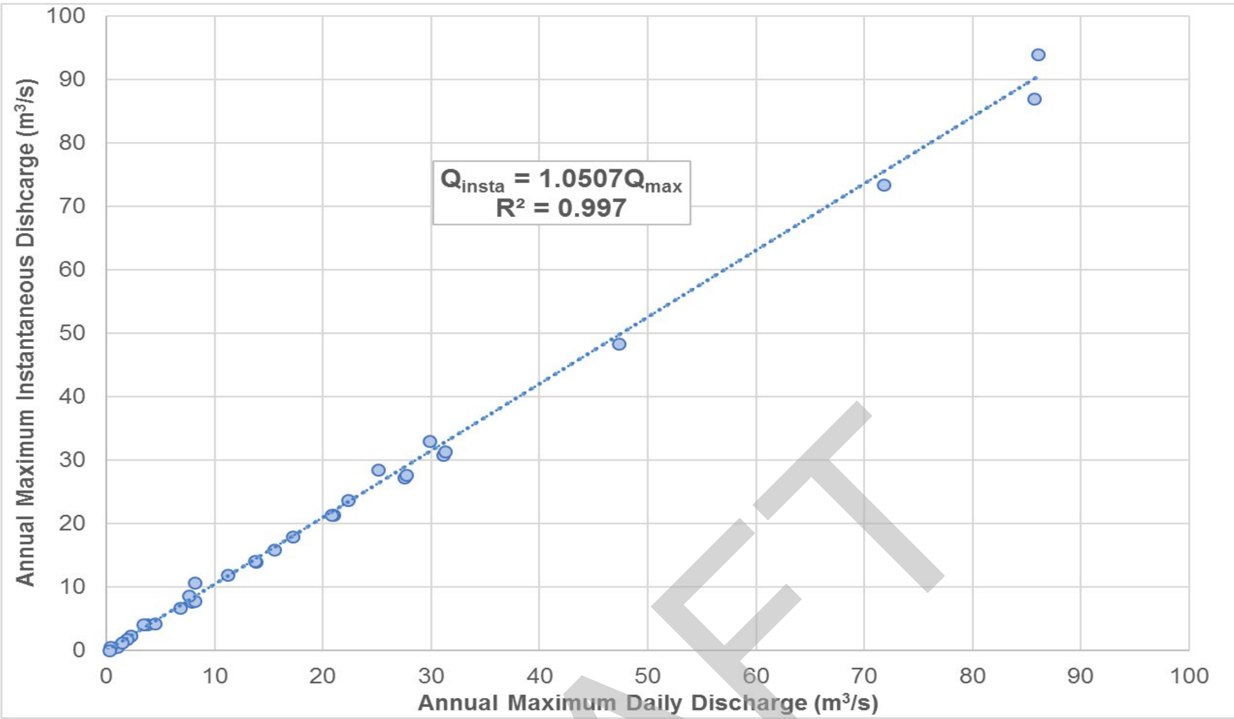


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Flat Creek near Boyle (WSC Station No. 07CA003)

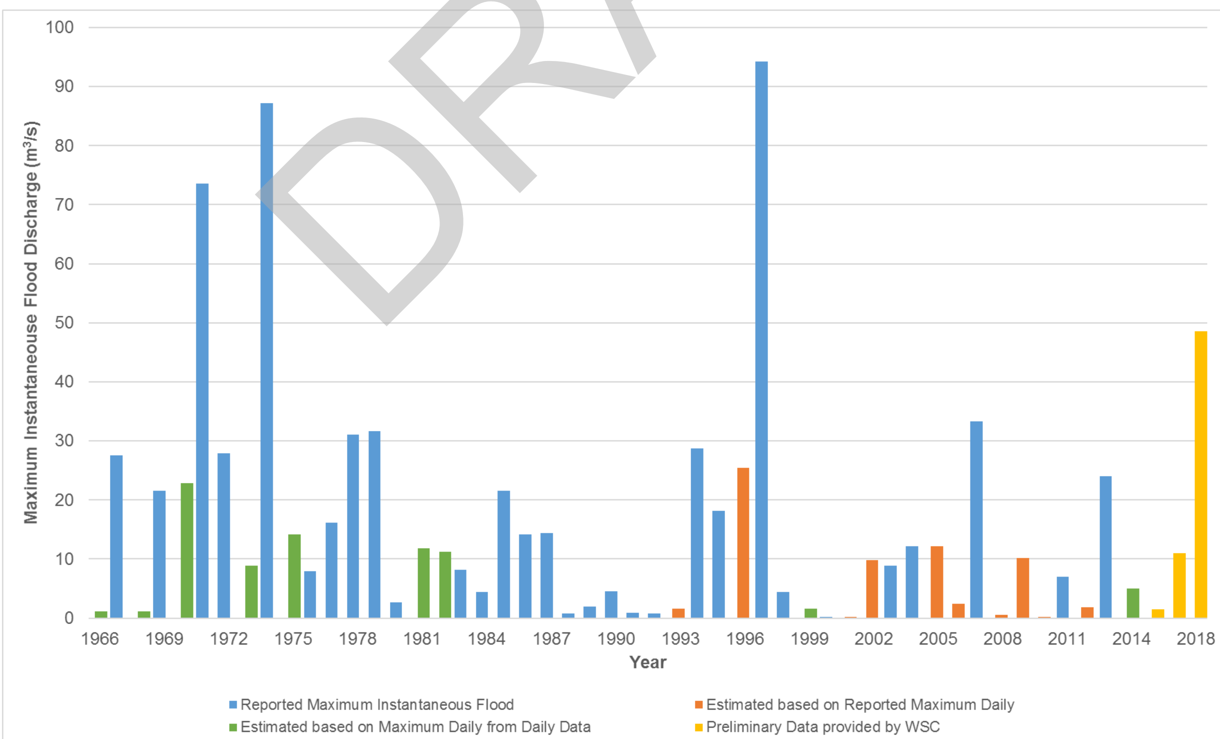


Maximum Instantaneous Flood Flow Series at Flat Creek near Boyle (WSC Station No. 07CA003)

Figure A-7: WSC Station No. 07CA005, Pine Creek near Grassland

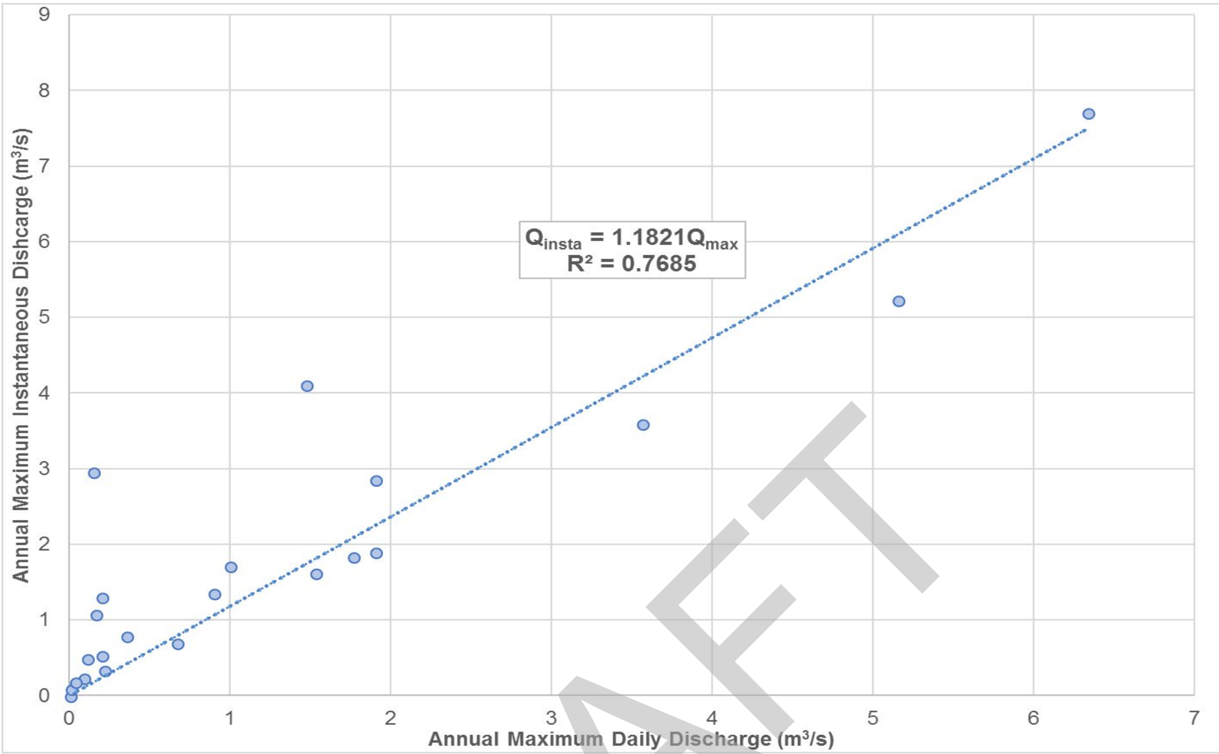


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Pine Creek near Grassland (WSC Station No. 07CA005)

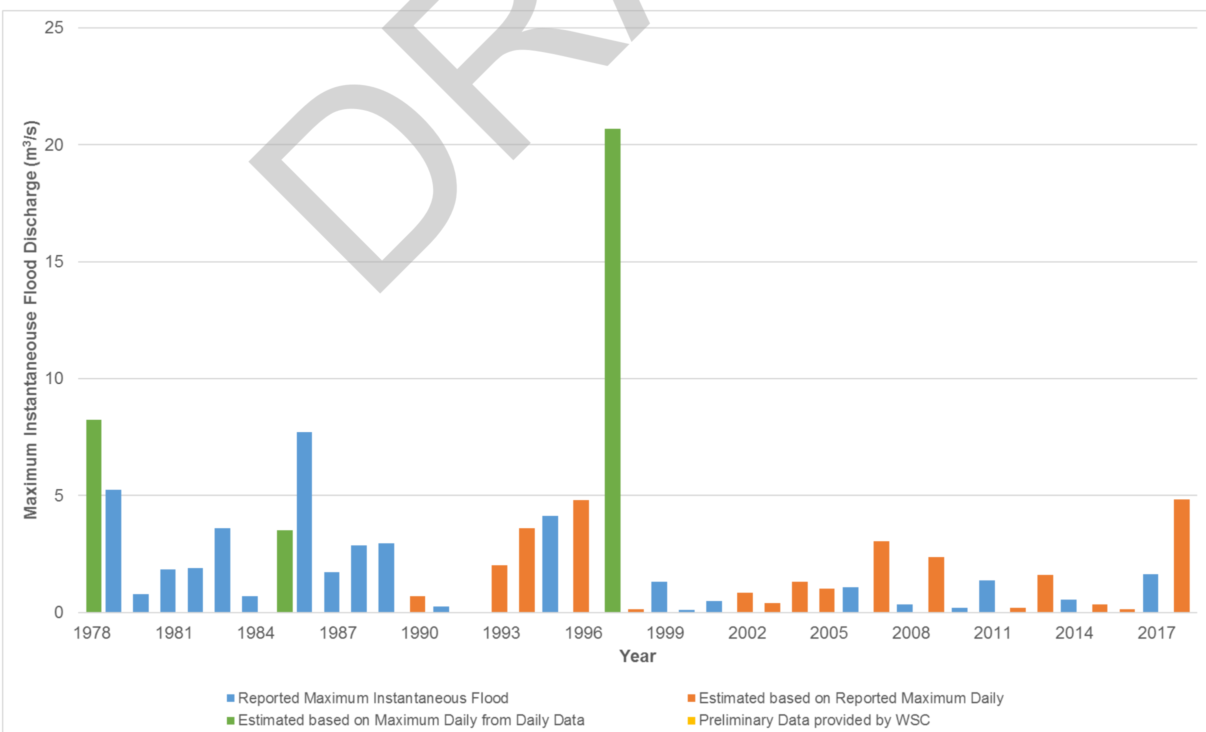


Maximum Instantaneous Flood Flow Series at Pine Creek near Grassland (WSC Station No. 07CA005)

Figure A-8: WSC Station No. 07CA008, Babette Creek near Colinton



Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Babette Creek near Colinton (WSC Station No. 07CA008)



Maximum Instantaneous Flood Flow Series at Babette Creek near Colinton (WSC Station No. 07CA008)

Table A-1: Annual Maximum Instantaneous Flood Data used for Flood Frequency Analysis

Year	07BE001, Athabasca River at Athabasca	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1913	1,741	-	-	-	-	-	-	-
1914	3,222	-	-	-	-	-	-	-
1915	2,878	-	-	-	-	-	-	-
1916	2,169	-	-	-	-	-	-	-
1917	2,597	-	-	-	-	-	-	-
1918	1,533	-	-	-	-	-	-	-
1919	1,074	-	-	-	-	0.81	-	-
1920	2,607	-	-	-	-	46.91	-	-
1921	1,543	-	-	-	-	18.63	-	-
1922	940	-	-	-	-	3.13	-	-
1923	1,910	-	-	-	-	6.89	-	-
1924	1,580	-	-	-	-	0.21	-	-
1925	1,981	-	-	-	-	1.53	-	-
1926	1,533	-	-	-	-	0.28	-	-
1927	2,325	-	-	-	-	8.94	-	-
1928	2,240	-	-	-	-	5.83	-	-
1929	1,199	-	-	-	-	2.20	-	-
1930	1,856	-	-	-	-	6.00	-	-
1931		-	-	-	-	0.56	-	-
1932		-	-	-	-	-	-	-
1933		-	-	-	-	-	-	-
1934		-	-	-	-	-	-	-
1935		-	-	-	-	-	-	-
1936		-	-	-	-	-	-	-
1937		-	-	-	-	-	-	-
1938	1,105	-	-	-	-	-	-	-
1939	1,304	-	-	-	-	-	-	-
1940	1,710	-	-	-	-	-	-	-
1941	2,065	-	-	-	-	-	-	-
1942	1,773	-	-	-	-	-	-	-
1943	2,169	-	-	-	-	-	-	-
1944	5,040	-	-	-	-	-	-	-
1945	992	-	-	-	-	-	-	-

Table A-1: Annual Maximum Instantaneous Flood Data used for Flood Frequency Analysis

Year	07BE001, Athabasca River at Athabasca	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1946	1,700	-	-	-	-	-	-	-
1947	2,080	-	-	-	-	-	-	-
1948	3,806	-	-	-	-	-	-	-
1949	1,074	-	-	-	-	-	-	-
1950	1,630	-	-	-	-	-	-	-
1951	1,877	-	-	-	-	-	-	-
1952	2,450	-	-	-	-	-	-	-
1953	1,908	-	-	-	-	-	-	-
1954	5,660	-	-	-	-	-	-	-
1955	2,350	-	-	-	-	-	-	-
1956	1,810	-	-	-	-	-	-	-
1957	1,170	-	-	-	-	-	-	-
1958	2,400	-	-	-	-	-	-	-
1959	1,760	-	-	-	-	-	-	-
1960	2,890	-	-	-	-	-	-	-
1961	1,150	-	-	-	-	-	-	-
1962	1,640	-	-	-	-	-	-	-
1963	1,610	-	-	-	-	-	-	-
1964	1,630	-	-	-	-	-	-	-
1965	3,680	-	-	-	-	-	-	-
1966	1,790	0.01	-	-	-	-	1.17	-
1967	1,660	5.77	-	-	-	-	27.50	-
1968	1,290	1.87	-	-	-	-	1.10	-
1969	3,110	6.74	-	-	-	-	21.60	-
1970	1,630	4.76	-	-	-	-	22.80	-
1971	4,560	45.30	-	-	-	-	73.60	-
1972	3,540	25.80	-	-	-	-	27.90	-
1973	1,890	3.14	-	-	-	-	8.93	-
1974	2,550	34.00	-	-	-	-	87.20	-
1975	1,850	3.06	0.94	-	-	-	14.18	-
1976	1,660	3.54	6.12	-	-	-	7.96	-
1977	2,380	5.83	11.30	-	-	-	16.20	-
1978	2,350	8.01	12.86	15.80	-	-	31.10	8.24

Table A-1: Annual Maximum Instantaneous Flood Data used for Flood Frequency Analysis

Year	07BE001, Athabasca River at Athabasca	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1979	2,320	24.80	33.10	26.90	-	-	31.60	5.24
1980	4,330	3.95	5.25	9.31	0.19	2.39	2.62	0.79
1981	1,270	16.00	25.61	27.70	0.87	3.09	11.77	1.84
1982	3,490	2.58	20.00	29.20	1.36	2.68	11.24	1.90
1983	1,590	1.69	3.69	8.46	8.36	2.29	8.13	3.60
1984	1,410	1.70	1.91	3.01	3.15	2.51	4.42	0.70
1985	1,520	2.74	12.70	26.70	1.10	4.66	21.60	3.51
1986	4,530	2.41	42.10	8.93	10.10	10.60	14.20	7.71
1987	2,150	3.41	6.92	14.26	0.68	6.20	14.40	1.72
1988	1,620	0.27	0.22	4.00	6.23	1.11	0.73	2.86
1989	2,390	0.35	1.64	4.36	1.96	1.08	2.00	2.96
1990	2,790	1.87	7.13	9.18	0.54	3.15	4.50	0.69
1991	1,950	0.34	0.30	3.45	0.26	1.05	0.86	0.24
1992	1,030	0.00	0.44	2.33	0.09	0.50	0.78	0.00
1993	958	0.02	0.65	0.11	0.41	6.29	1.54	2.01
1994	1,650	6.48	13.92	6.12	1.53	5.88	28.70	3.59
1995	2,270	1.51	5.74	0.32	0.91	4.44	18.20	4.12
1996	2,670	7.51	-	25.63	3.66	6.01	25.43	4.79
1997	2,930	25.80	-	65.40	10.70	16.20	94.20	20.69
1998	1,679	0.94	-	5.01	0.13	0.55	4.43	0.15
1999	2,190	0.70	-	2.32	0.22	0.14	1.54	1.31
2000	1,520	1.07	-	0.59	0.28	0.00	0.20	0.10
2001	2,690	0.20	-	0.27	0.45	0.08	0.14	0.50
2002	1,178	1.11	-	6.20	0.27	0.99	9.86	0.85
2003	1,400	0.70	-	17.88	0.31	1.72	8.87	0.40
2004	1,850	1.86	-	10.40	0.60	3.15	12.20	1.31
2005	2,040	2.82	-	14.37	0.87	1.59	12.19	1.00
2006	1,230	0.51	-	8.78	0.14	0.79	2.46	1.08
2007	2,370	9.32	-	23.60	2.11	3.11	33.30	3.04
2008	1,570	0.80	-	3.19	0.26	0.22	0.55	0.34
2009	1,480	1.38	-	7.63	0.10	3.80	10.18	2.36
2010	924	0.00	-	0.06	0.20	0.06	0.24	0.19
2011	4,850	0.68	-	7.25	2.77	3.22	7.00	1.36

Table A-1: Annual Maximum Instantaneous Flood Data used for Flood Frequency Analysis

Year	07BE001, Athabasca River at Athabasca	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
2012	3,320	0.00	-	8.90	0.50	0.65	1.86	0.19
2013	2,210	3.11	-	40.00	0.57	2.93	24.00	1.60
2014	1,390	1.31	-	7.48	1.00	2.05	4.99	0.54
2015	1,030	1.53	-	-	-	-	-	0.33
2016	1,250	0.78	-	-	-	-	1.54	0.13
2017	2,349	2.69	-	-	-	-	10.94	1.63
2018	2,049	15.20	-	49.80	4.61	7.35	48.54	4.82
Maximum	5,660	45.30	42.10	65.40	10.70	46.91	94.20	20.69
Mean	2,112	5.62	10.12	13.29	1.87	4.38	16.02	2.45
Minimum	924	0.00	0.22	0.06	0.09	0.00	0.14	0.00
Standard Deviation	952	9.29	11.49	14.55	2.78	7.30	20.56	3.54

Table A-2: Results of Statistical Tests of Annual Maximum Instantaneous Discharges and Goodness-of-Fit of Probability Distribution Functions

WSC Station ID		07EC002	07EC004	07EC005	07BE003	07CA003	07CA005	07CA008
WSC Station Name or Location of Interest		Waskatenau Creek near Waskatenau	Namepi Creek near the Mouth	Redwater River near the Mouth	Porter Creek above Baptiste Lake	Flat Creek near Boyle	Pine Creek near Grassland	Babette Creek near Colinton
Anderson-Darling statistic, $A^2 = - N - S$								
3 Parameter Log-normal		0.514	0.602	0.373	0.129	0.422	0.827	0.229
Extreme Value		0.727	0.386	0.380	0.974	0.330	0.734	0.315
Log-Pearson III		5.226	0.243	1.386	0.176	3.577	0.387	0.759
Weibull		4.587	0.538	0.604	2.881	2.400	1.173	0.966
Serial correlation coefficient test for independence								
S_1		0.3996	0.2677	0.3151	0.3387	0.1566	0.1939	0.2666
t		3.0829	1.1786	1.9641	2.0676	1.0757	1.3836	1.7052
t(α=0.05)		1.6759	1.7341	1.6896	1.6924	1.6787	1.6766	1.6860
t(α=0.01)		2.4033	2.5524	2.4377	2.4448	2.4102	2.4049	2.4286
Spearman rank order correlation coefficient test for no-trend								
r_s		0.4079	0.3065	0.1003	0.2100	0.2194	0.2356	0.3754
t		3.1905	1.4035	0.6051	1.2527	1.5416	1.7145	2.5296
t(α=0.05)		2.0076	2.0930	2.0281	2.0322	2.0117	2.0086	2.0227
t(α=0.01)		2.6757	2.8609	2.7195	2.7284	2.6846	2.6778	2.7079
Mann-Whitney split sample test for homogeneity								
Size of earlier sample		26	11	20	20	25	26	21
z		-2.7312	-1.4788	-0.6724	-1.4326	-1.4000	-1.2994	-2.6604
z(a=0.05)		-1.6449		-1.6449	-1.6449	-1.6449	-1.6449	-1.6449
z(a=0.01)		-2.3263		-2.3263	-2.3263	-2.3263	-2.3263	-2.3263
Test of general randomness (Runs for above or below the median)								
Median		1.9	6.1	8.6	0.6	2.5	10.0	1.4
N1(for Q>=Median)		28	11	19	18	25	26	21
N2(for Q<Median)		25	10	19	18	24	26	20
Run_ab		18	11	13	12	26	24	17
z		2.6203	0.2137	2.3024	2.3674	0.1474	0.8403	1.4205
z(a=0.05)		1.9600				1.9600	1.9600	1.9600
z(a=0.01)		2.5758				2.5758	2.5758	2.5758

Notes:

1. *Selected distribution based on best statistical fit* *0.320*
2. *Criteria for the respective statistical tests were not met* *1.9600*

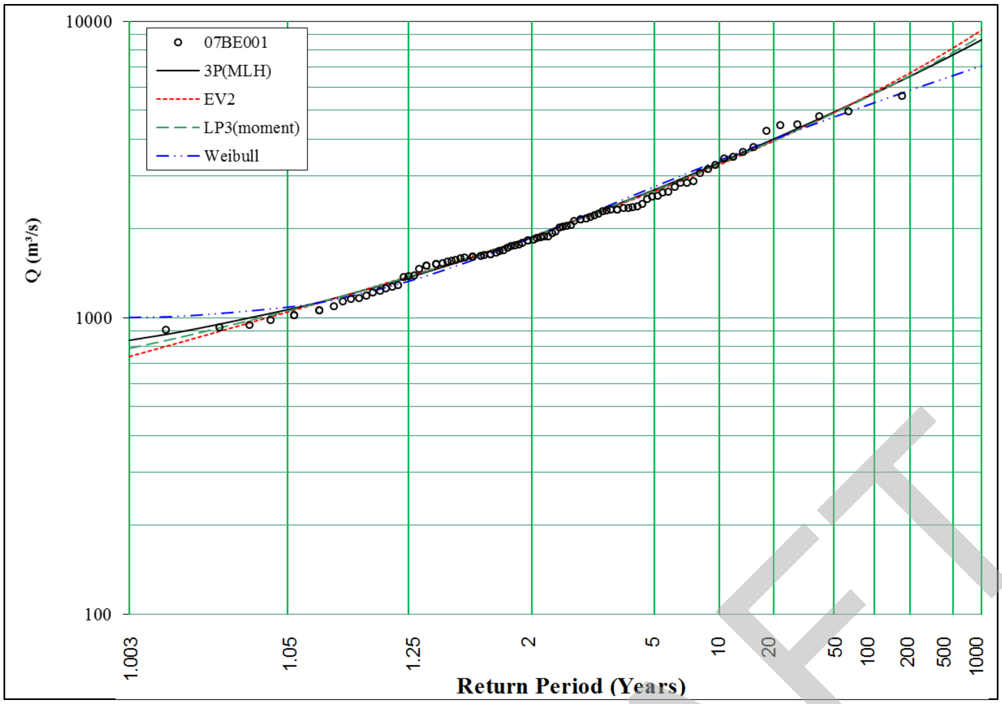
APPENDIX B

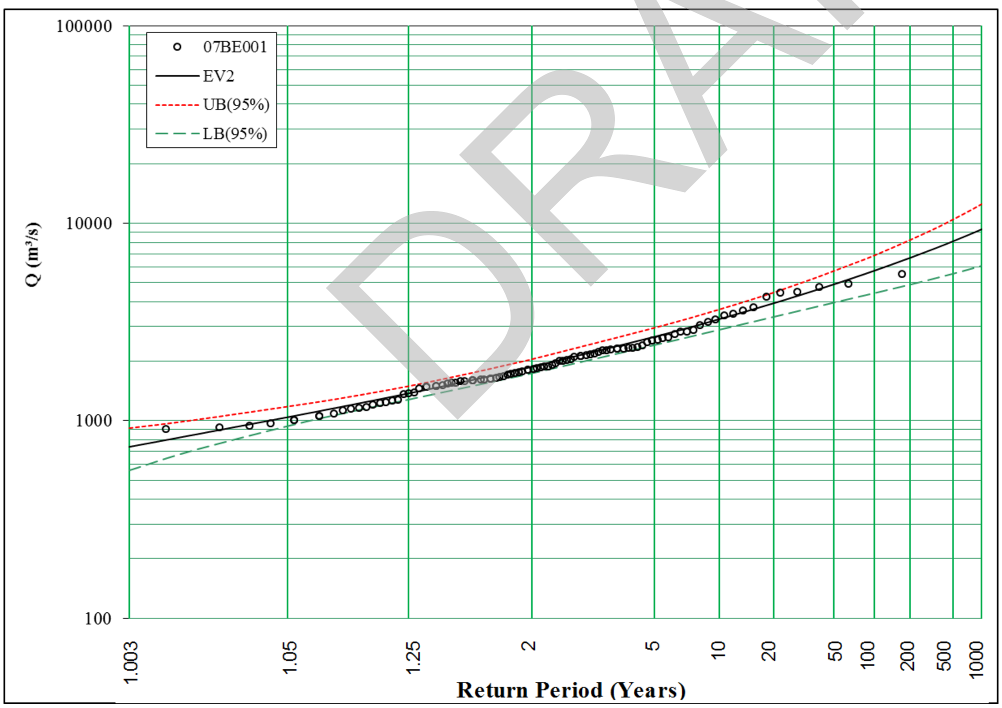
Frequency Analyses - Graphs and Tables

This appendix includes the graphs and results from the frequency analysis of the compiled/derived maximum instantaneous flood flow series at either the gauged stations or locations of interest within the study area. For each flood flow series, the following information is presented:

- Frequency distribution graph – all distributions;
- Frequency distribution graph – best fit graph with confidence interval; and
- Flood flow estimates – all distributions.

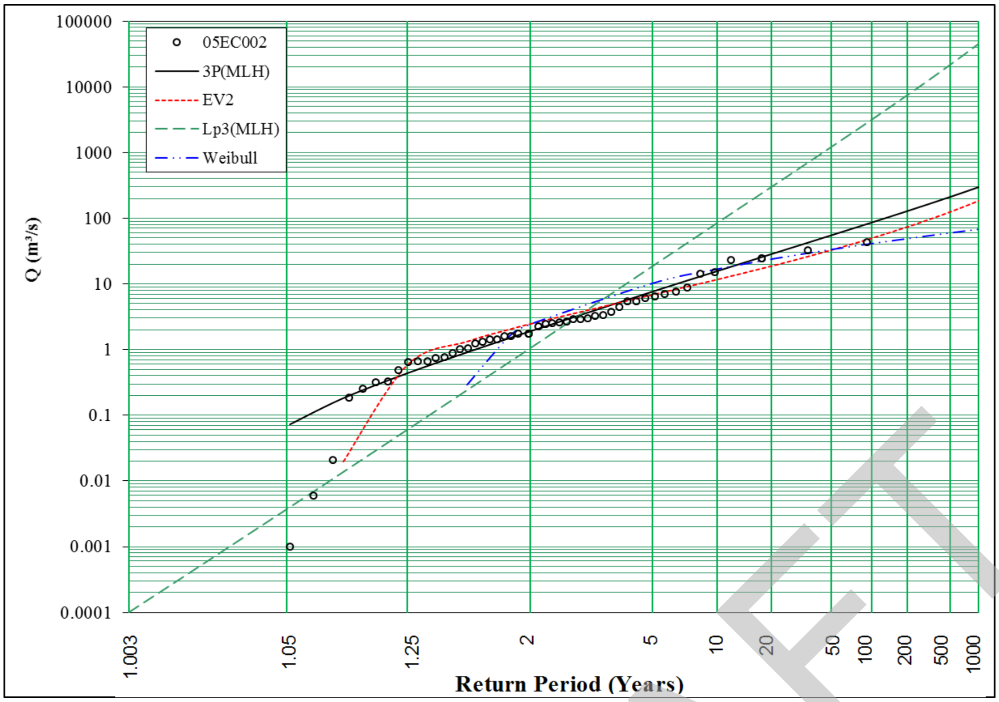
Figure B-1: WSC Station No.07BE001, Athabasca River at Athabasca

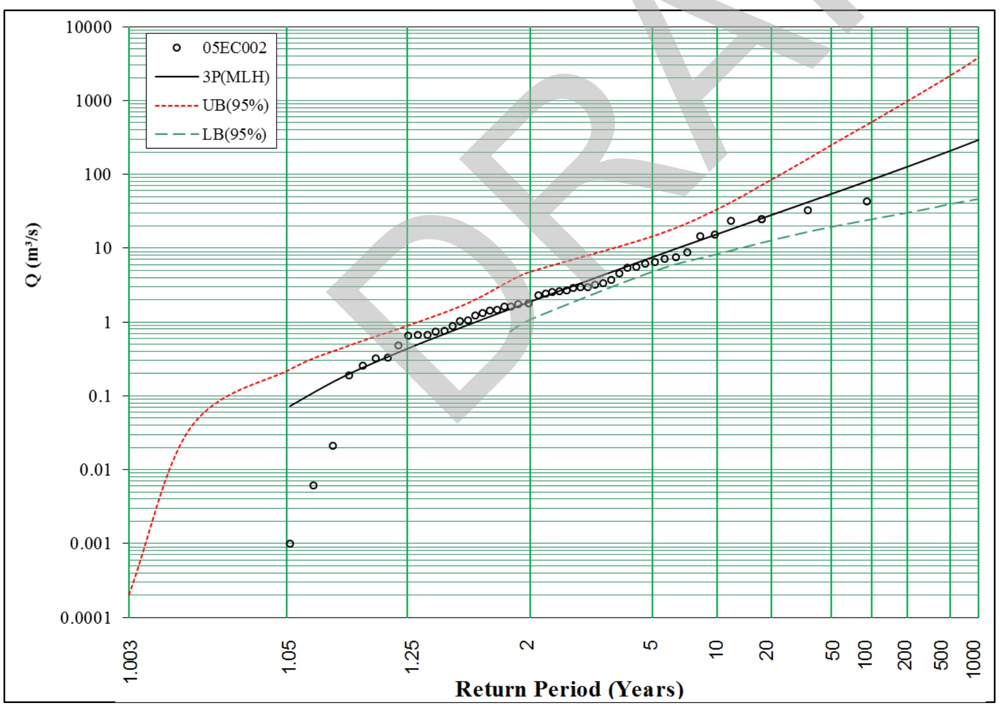




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	1870	1877	1880	1862
5	2704	2669	2687	2767
10	3338	3280	3301	3394
20	4001	3942	3952	3993
35	4567	4525	4516	4461
50	4942	4924	4896	4753
75	5385	5405	5350	5080
100	5710	5765	5687	5309
200	6530	6705	6554	5853
350	7233	7545	7316	6284
500	7702	8121	7834	6555
750	8254	8819	8454	6860
1000	8659	9344	8916	7075

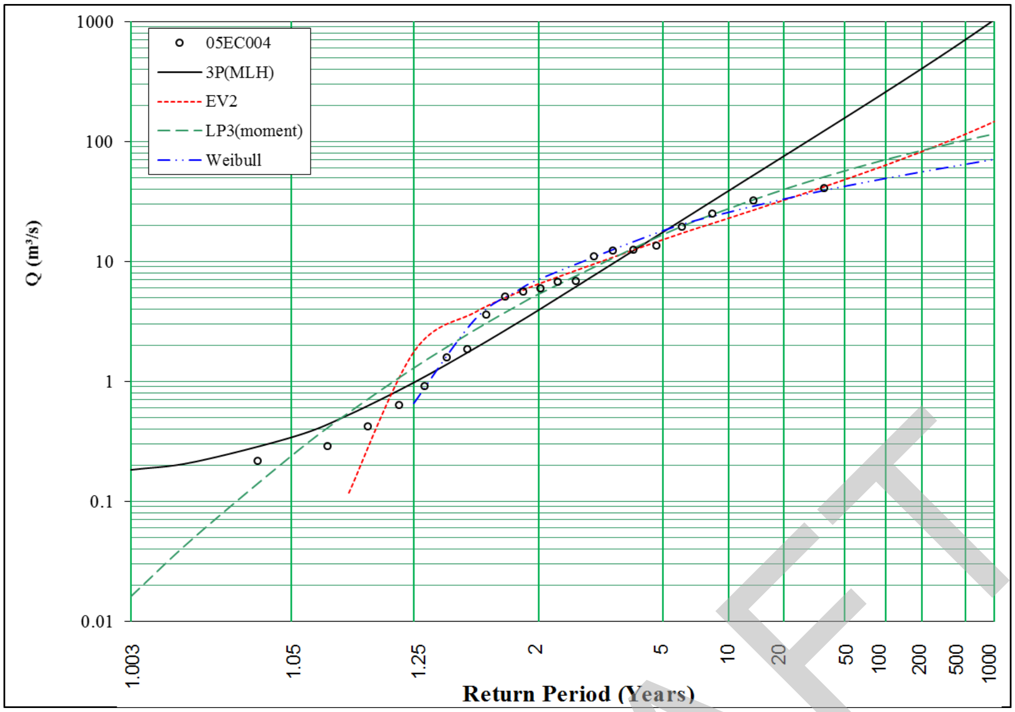
Figure B-2: WSC Station No. 05EC002, Waskatenau Creek near Waskatenau

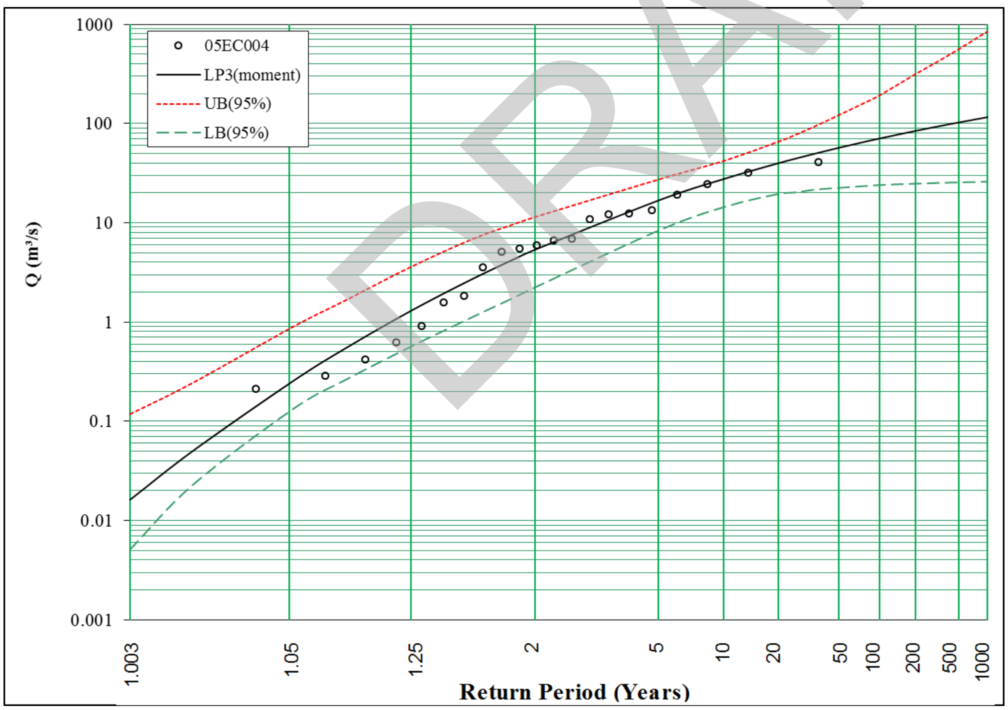




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	2	2	1	2
5	8	7	19	10
10	16	12	85	17
20	28	19	299	24
35	43	27	729	30
50	55	33	1232	34
75	71	42	2167	38
100	85	50	3175	42
200	128	75	7563	50
350	173	103	14558	56
500	209	126	21703	60
750	256	158	33656	65
1000	294	186	45537	69

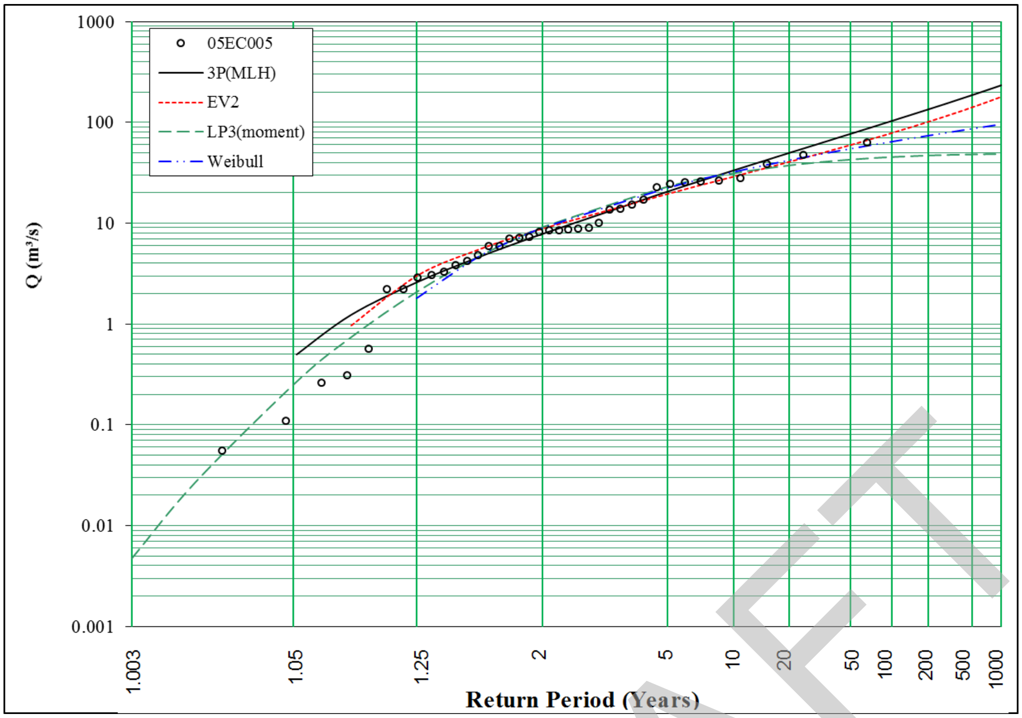
Figure B-3: WSC Station No. 05EC004, Namepi Creek near the Mouth

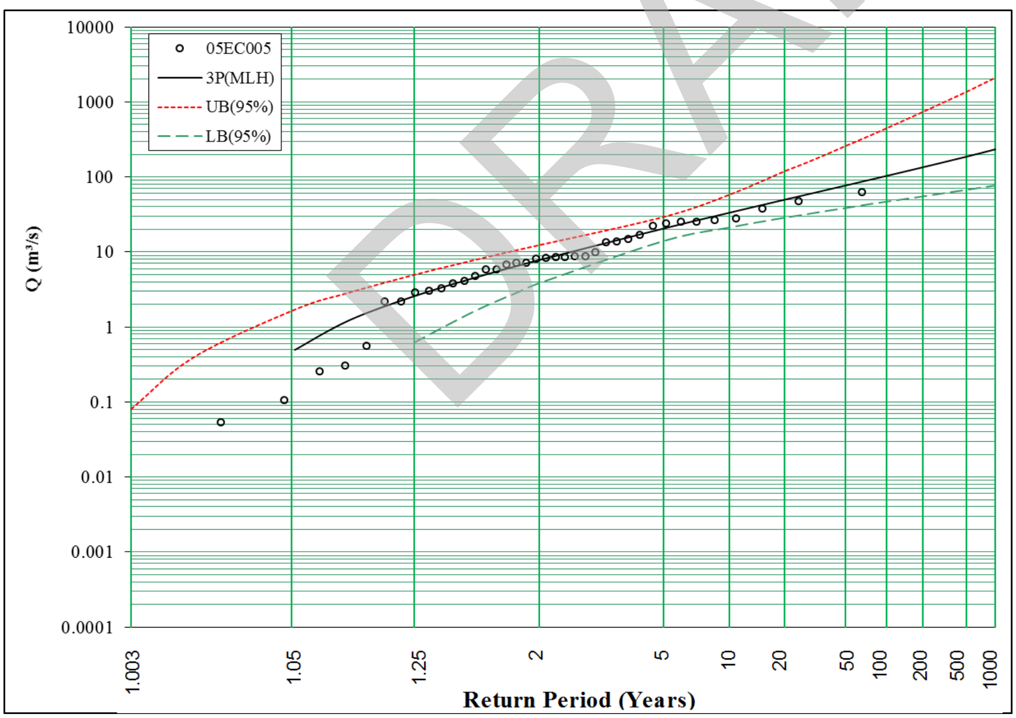




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	4	6	5	7
5	18	15	17	18
10	39	23	28	26
20	75	32	40	33
35	119	41	50	39
50	157	48	57	42
75	211	57	65	46
100	257	63	71	49
200	404	82	84	56
350	569	101	95	61
500	700	115	102	64
750	879	132	110	68
1000	1028	146	116	71

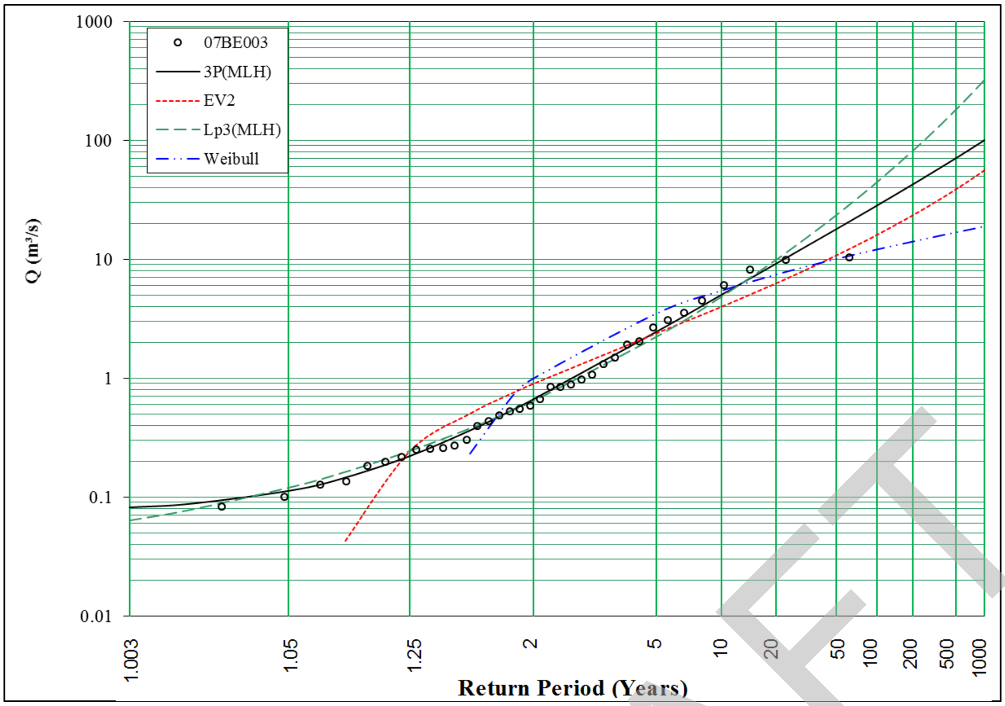
Figure B-4: WSC Station No. 05EC005, Redwater River near the Mouth

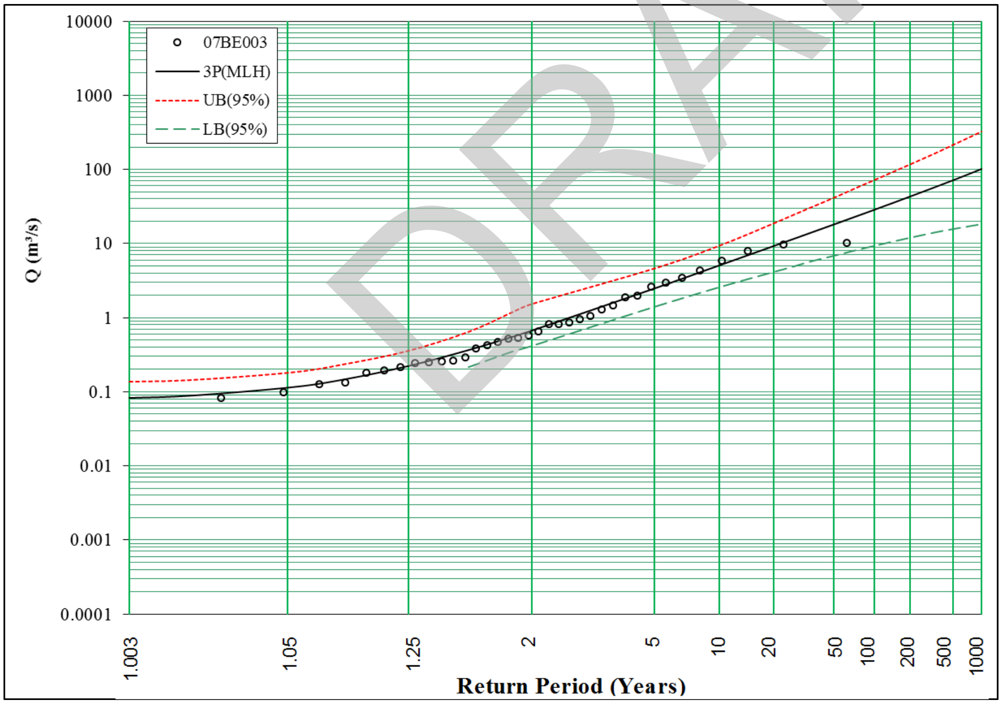




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	8	9	9	9
5	21	20	23	22
10	33	29	31	32
20	50	41	37	42
35	65	52	41	50
50	77	60	43	55
75	92	70	44	61
100	103	79	45	65
200	135	102	47	75
350	165	124	48	82
500	186	141	48	87
750	213	162	48	93
1000	233	179	49	97

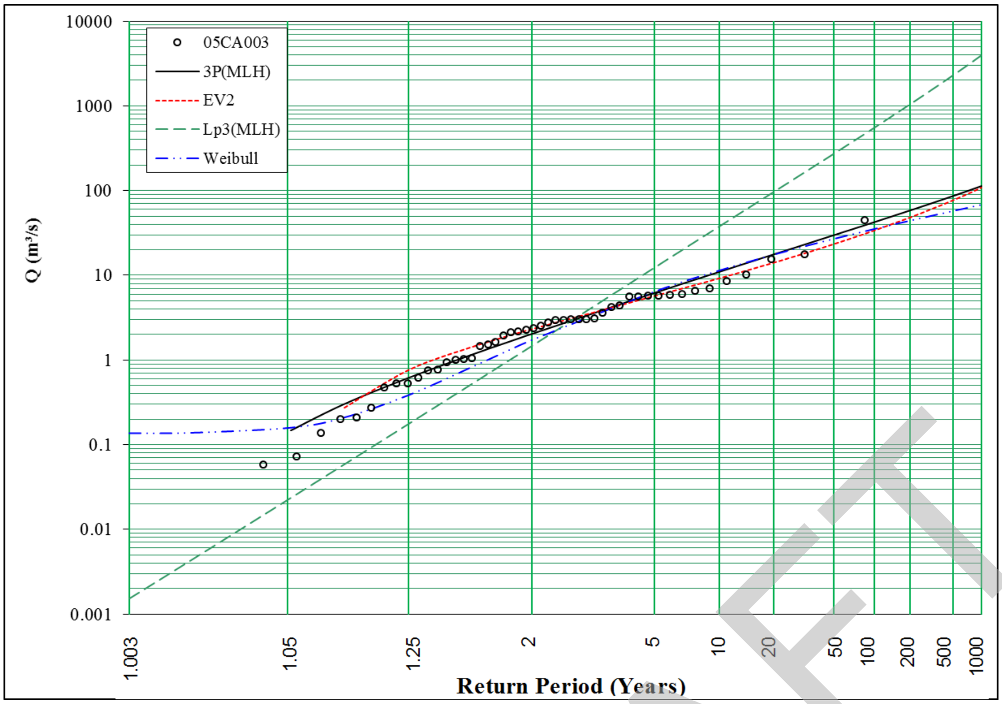
Figure B-5: WSC Station No. 07BE003, Porter Creek above Baptiste Lake

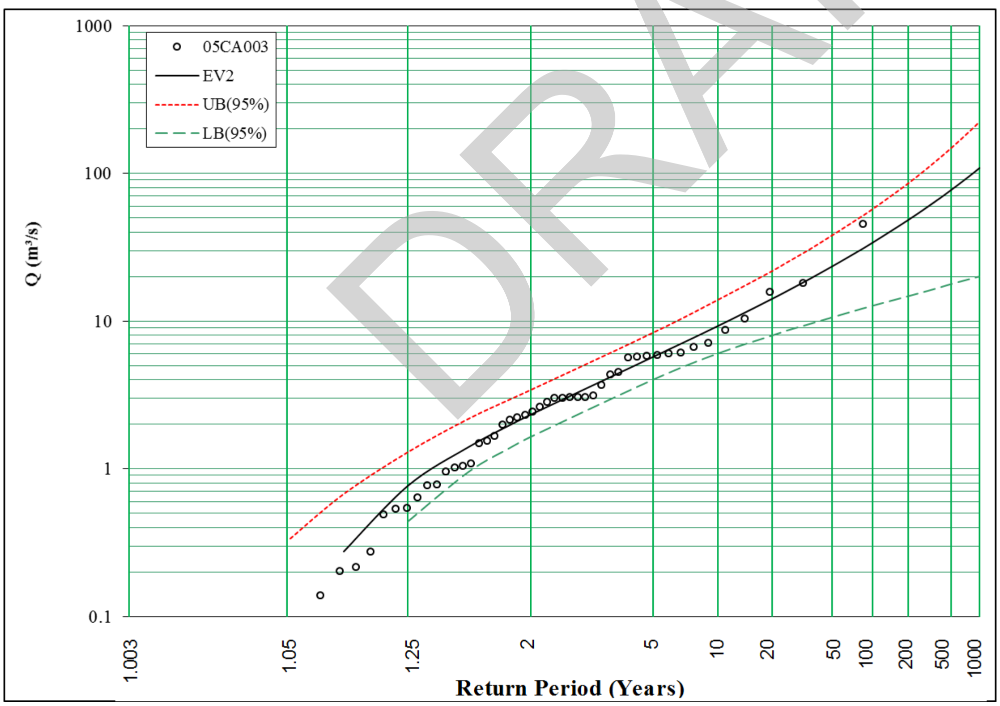




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	1	1	1	1
5	2	2	2	3
10	5	4	5	5
20	9	6	10	7
35	14	9	17	9
50	18	11	24	10
75	24	14	34	11
100	28	16	44	12
200	43	23	82	14
350	59	32	133	16
500	71	38	180	17
750	87	48	253	18
1000	101	56	322	19

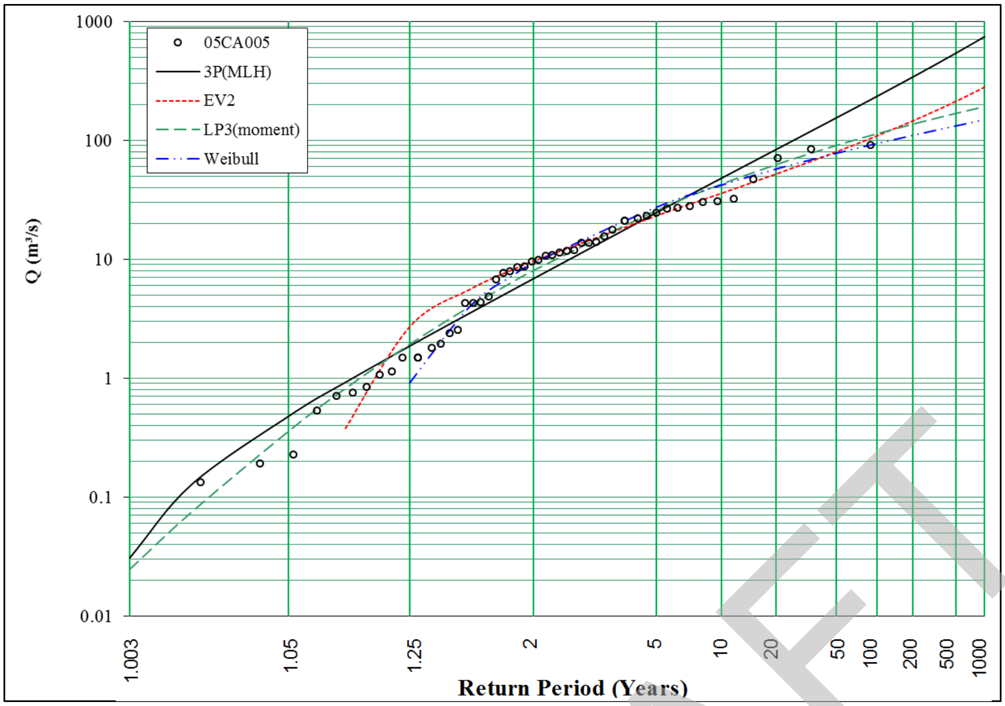
Figure B-6: WSC Station No. 07CA003, Flat Creek near Boyle

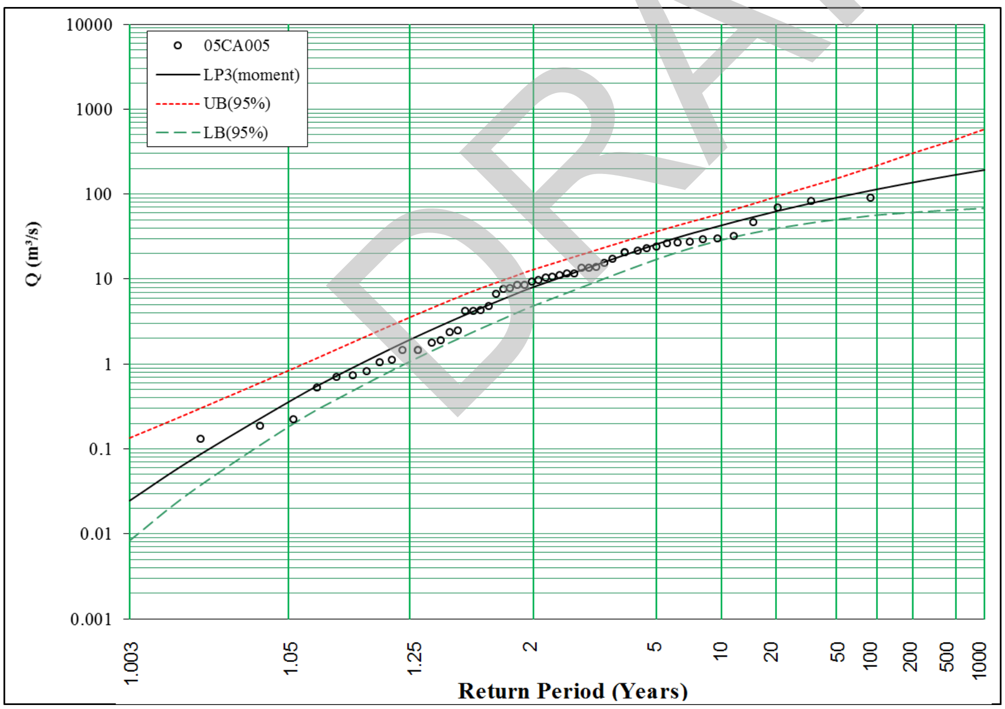




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	2	2	1	2
5	6	6	12	6
10	11	9	38	11
20	18	14	97	18
35	25	19	187	23
50	30	23	276	27
75	37	29	418	32
100	43	34	555	35
200	59	48	1055	44
350	75	64	1713	52
500	87	77	2301	58
750	102	94	3183	64
1000	114	108	3981	68

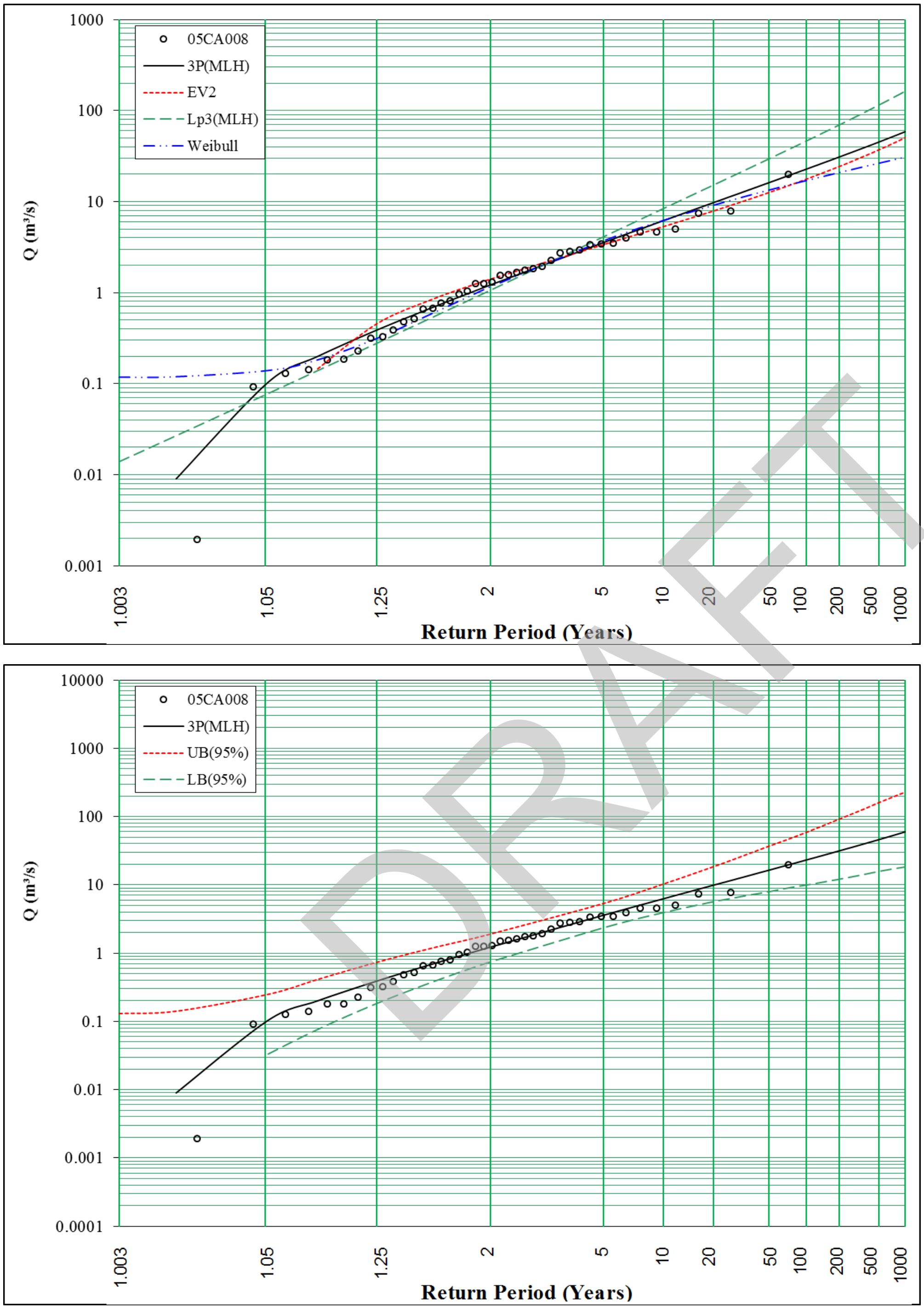
Figure B-7: WSC Station No. 07CA005, Pine Creek near Grassland





Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	7	10	8	9
5	25	23	26	27
10	48	36	43	42
20	84	52	62	57
35	124	68	80	69
50	155	80	91	78
75	199	96	104	87
100	235	109	114	93
200	343	146	137	110
350	455	183	156	123
500	541	212	168	132
750	655	249	182	141
1000	746	279	192	148

Figure B-8: WSC Station No. 07CA008, Babette Creek near Colinton



Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	1	1	1	1
5	4	3	4	4
10	6	5	8	6
20	10	8	15	9
35	13	10	23	12
50	16	13	30	13
75	20	15	39	16
100	23	18	46	17
200	31	24	69	21
350	39	31	95	24
500	45	37	114	26
750	53	44	141	29
1000	59	50	162	31

APPENDIX E

Highwater Mark Comparison

Table E-1: Comparison of Simulated and Surveyed Highwater Marks along the Athabasca River Study Reach for the June 1980 Flood Event

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Measured Discharge (m^3/s)	Survey Date	Description
1	6559	513.54	513.57	0.03	4190	1980-06-08	80-AT-1
2	6021	513.16	513.22	0.07	4190	1980-06-08	80-AT-2
3	5785	513.16	513.14	-0.02	4190	1980-06-08	80-AT-3
4	5543	513.11	513.09	-0.02	4190	1980-06-08	80-AT-4
5	4994	512.91	512.81	-0.10	4190	1980-06-08	80-AT-5
6	4796	512.61	512.58	-0.03	4190	1980-06-08	80-AT-6

Table E-2: Comparison of Simulated and Surveyed Highwater Marks along the Athabasca River Study Reach for the July 1990 Flood Event

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Measured Discharge (m^3/s)	Survey Date	Description
1	6575	512.11	512.13	0.02	2790	1990-07-10	400 m u/s of Muskeg creek Confluence
2	5736	511.82	511.74	-0.07	2790	1990-07-10	400 m u/s of Tawatinaw River Confluence
3	4829	511.36	511.36	0.00	2790	1990-07-10	15 m u/s of Hwy 813 bridge
4	4796	511.18	511.31	0.13	2790	1990-07-10	40 m d/s of Hwy 813 bridge
5	2825	510.87	510.72	-0.15	2790	1990-07-10	2000 m d/s of Hwy 813 bridge

Table E-3: Comparison of Simulated and Surveyed Highwater Marks along the Athabasca River Study Reach for the July 1990 Flood Event

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Measured Discharge (m^3/s)	Survey Date	Description
1	6575	511.62	511.61	-0.01	2340	1990-07-11	400 m u/s of Muskeg creek Confluence
2	5736	511.26	511.23	-0.03	2340	1990-07-11	400 m u/s of Tawatinaw River Confluence
3	4829	510.93	510.88	-0.05	2340	1990-07-11	15 m u/s of Hwy 813 bridge
4	4796	510.89	510.84	-0.05	2340	1990-07-11	40 m d/s of Hwy 813 bridge
5	2825	510.31	510.24	-0.07	2340	1990-07-11	2000 m d/s of Hwy 813 bridge

APPENDIX F

Open Water Flood Profiles

Figure F-1: Simulated Water Surface Profiles along the Athabasca River Study Reach

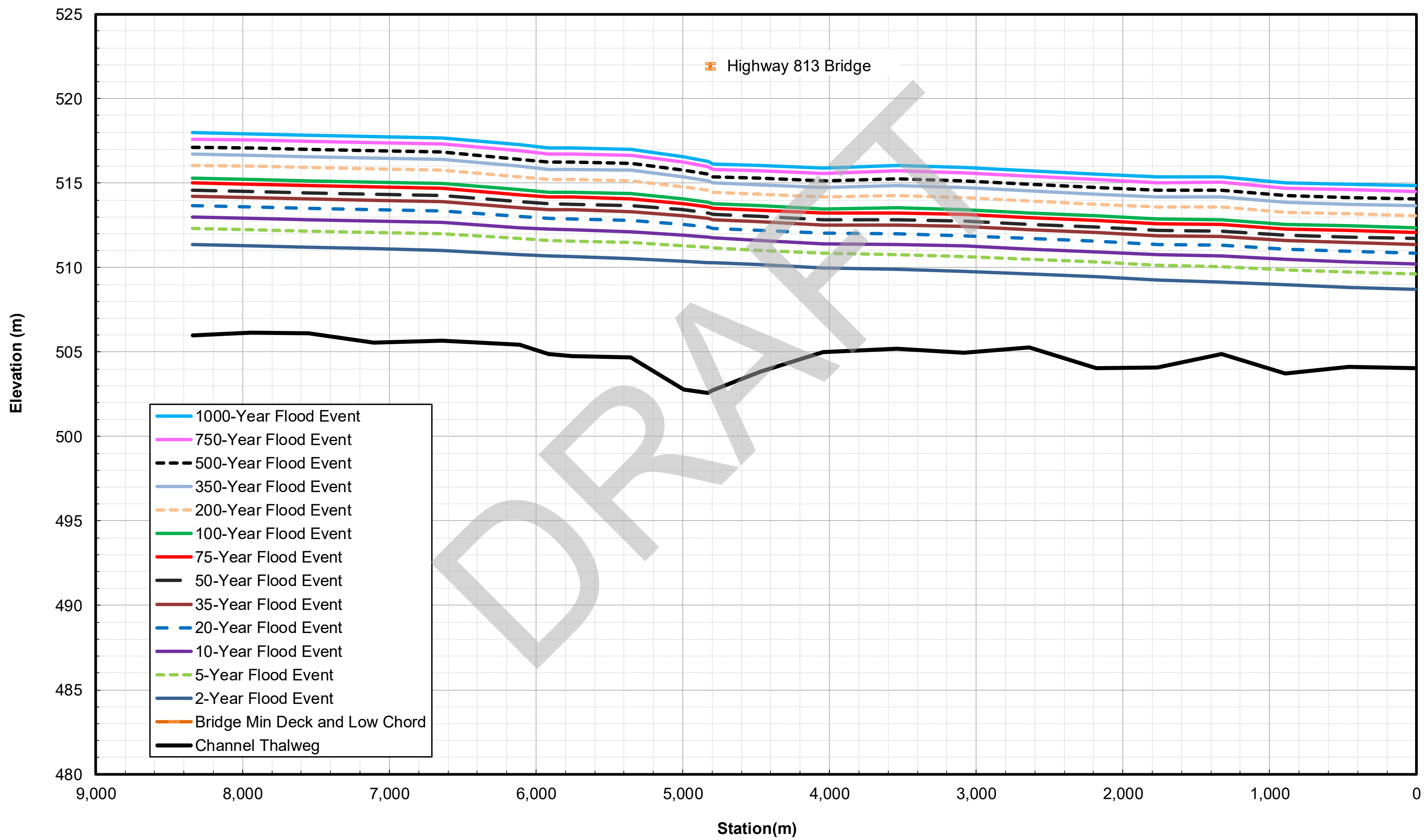


Figure F-2: Simulated Water Surface Profiles along the Muskeg Creek Study Reach

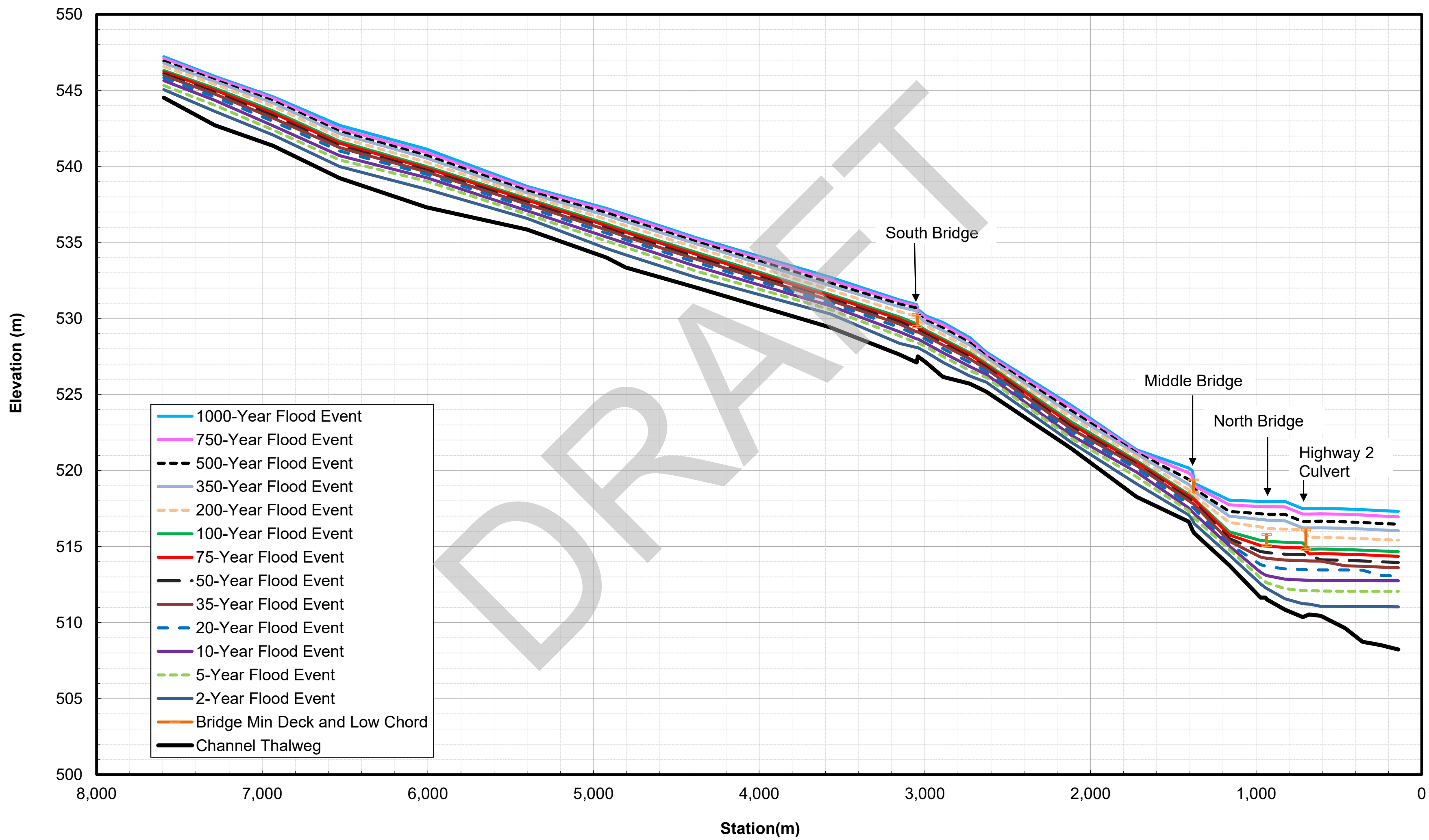


Figure F-3: Simulated Water Surface Profiles along the Tawatinaw River Study Reach

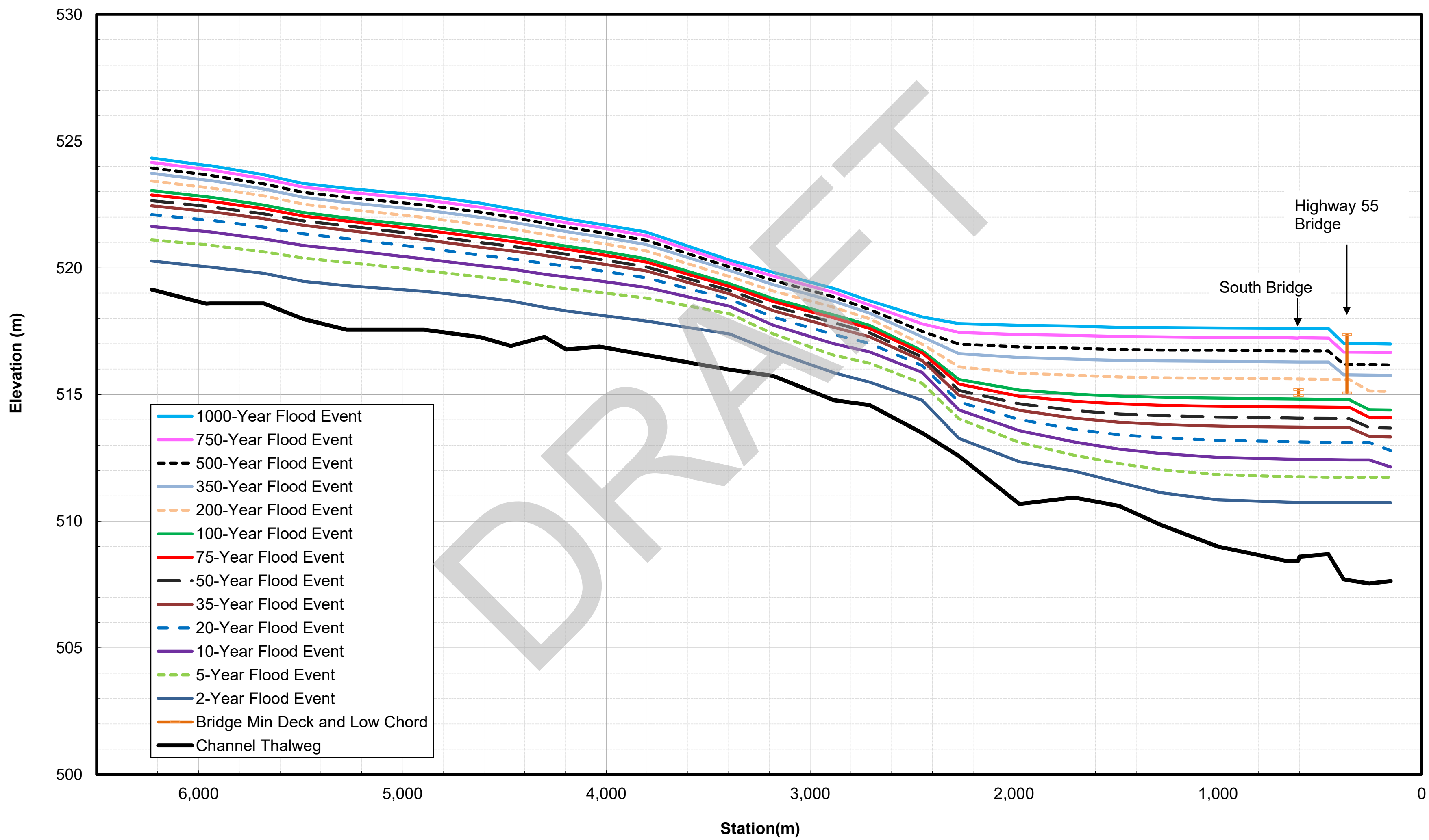


Table F-1: Athabasca River Flood Profiles

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)												
				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
Athabasca River	1	8340	506.0	511.4	512.3	513.0	513.7	514.2	514.6	515.0	515.3	516.1	516.7	517.1	517.6	518.0
Athabasca River	2	7941	506.1	511.3	512.3	512.9	513.6	514.2	514.5	514.9	515.2	516.0	516.6	517.1	517.6	517.9
Athabasca River	3	7550	506.1	511.2	512.2	512.8	513.5	514.1	514.4	514.9	515.2	515.9	516.6	517.0	517.5	517.8
Athabasca River	4	7109	505.6	511.1	512.1	512.8	513.4	514.0	514.3	514.8	515.1	515.8	516.5	516.9	517.4	517.8
Athabasca River	5	6640	505.7	511.0	512.0	512.7	513.3	513.9	514.3	514.7	515.0	515.8	516.4	516.8	517.3	517.7
Athabasca River	6	6112	505.5	510.8	511.7	512.4	513.0	513.6	513.9	514.3	514.6	515.4	516.0	516.4	516.9	517.3
Athabasca River	7	5915	504.9	510.7	511.6	512.3	512.9	513.4	513.8	514.2	514.5	515.2	515.8	516.2	516.7	517.1
Athabasca River	8	5756	504.7	510.6	511.6	512.2	512.9	513.4	513.8	514.2	514.5	515.2	515.8	516.2	516.7	517.1
Athabasca River	9	5352	504.7	510.5	511.5	512.1	512.8	513.3	513.7	514.1	514.4	515.1	515.8	516.2	516.7	517.0
Athabasca River	10	4994	502.8	510.4	511.3	511.9	512.6	513.1	513.4	513.8	514.1	514.8	515.4	515.8	516.2	516.6
Athabasca River	11	4829	502.6	510.3	511.2	511.8	512.4	512.9	513.2	513.6	513.9	514.6	515.1	515.5	516.0	516.3
Athabasca River	12	4796	502.7	510.3	511.2	511.7	512.3	512.8	513.1	513.5	513.8	514.5	515.0	515.4	515.8	516.1
Athabasca River	13	4468	503.9	510.2	511.0	511.6	512.2	512.7	513.0	513.4	513.7	514.3	514.9	515.3	515.7	516.0
Athabasca River	14	4043	505.0	510.0	510.8	511.4	512.0	512.5	512.8	513.2	513.5	514.2	514.8	515.1	515.6	515.9
Athabasca River	15	3542	505.2	509.9	510.8	511.4	512.0	512.5	512.9	513.2	513.5	514.2	514.8	515.2	515.7	516.1
Athabasca River	16	3087	504.9	509.8	510.7	511.3	511.9	512.4	512.8	513.1	513.4	514.1	514.7	515.1	515.6	515.9
Athabasca River	17	2636	505.3	509.6	510.5	511.1	511.7	512.2	512.6	513.0	513.2	514.0	514.5	514.9	515.4	515.7
Athabasca River	18	2182	504.0	509.5	510.3	510.9	511.6	512.1	512.4	512.8	513.1	513.8	514.4	514.7	515.2	515.5
Athabasca River	19	1764	504.1	509.3	510.2	510.8	511.4	511.9	512.2	512.6	512.9	513.6	514.2	514.6	515.0	515.4
Athabasca River	20	1329	504.9	509.2	510.1	510.7	511.3	511.8	512.2	512.6	512.9	513.6	514.2	514.6	515.0	515.4
Athabasca River	21*	892	503.7	509.0	509.9	510.5	511.1	511.6	511.9	512.3	512.6	513.3	513.9	514.2	514.7	515.0
Athabasca River	22*	461	504.1	508.8	509.7	510.4	511.0	511.5	511.8	512.2	512.5	513.2	513.8	514.2	514.6	514.9
Athabasca River	23*	6.48	504.1	508.7	509.6	510.2	510.9	511.4	511.7	512.1	512.4	513.1	513.7	514.1	514.5	514.8

* These cross sections are outside of the Athabasca Flood Hazard Study Boundary

Table F-2: Muskeg Creek Flood Profiles

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)												
				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
Muskeg Creek	24	7594	544.5	545.1	545.3	545.6	545.8	546.0	546.1	546.2	546.3	546.6	546.8	546.9	547.1	547.2
Muskeg Creek	25	7286	542.7	543.6	544.0	544.4	544.6	544.8	544.9	545.0	545.1	545.4	545.6	545.7	545.8	545.9
Muskeg Creek	26	6935	541.4	542.1	542.4	542.7	543.0	543.2	543.3	543.5	543.7	544.0	544.2	544.3	544.5	544.6
Muskeg Creek	27	6532	539.2	540.0	540.4	540.7	541.0	541.2	541.4	541.5	541.7	541.9	542.2	542.3	542.5	542.7
Muskeg Creek	28	6009	537.3	538.5	539.0	539.3	539.5	539.6	539.8	539.9	540.0	540.3	540.6	540.7	540.9	541.1
Muskeg Creek	29	5401	535.9	536.6	536.9	537.1	537.3	537.5	537.7	537.8	537.9	538.1	538.3	538.4	538.6	538.7
Muskeg Creek	30	4921	534.0	534.6	535.1	535.4	535.6	535.8	536.0	536.1	536.2	536.5	536.8	536.9	537.1	537.2
Muskeg Creek	31	4806	533.4	534.2	534.7	534.9	535.2	535.4	535.5	535.7	535.8	536.1	536.4	536.5	536.7	536.8
Muskeg Creek	32	4382	532.0	532.7	533.1	533.4	533.7	533.9	534.1	534.2	534.4	534.7	534.9	535.1	535.2	535.3
Muskeg Creek	33	3571	529.4	530.3	530.6	530.8	531.0	531.2	531.3	531.5	531.6	531.9	532.2	532.3	532.6	532.7
Muskeg Creek	34	3153	527.6	528.4	528.9	529.1	529.4	529.6	529.8	530.0	530.1	530.5	530.8	531.0	531.1	531.2
Muskeg Creek	35	3049	527.1	528.1	528.4	528.7	528.9	529.1	529.3	529.5	529.6	530.1	530.5	530.7	530.8	530.9
Muskeg Creek	36	3043	527.5	528.1	528.4	528.7	528.9	529.1	529.3	529.4	529.6	529.9	530.2	530.3	530.5	530.6
Muskeg Creek	37	2993	527.1	527.8	528.1	528.4	528.6	528.8	529.0	529.1	529.2	529.5	529.7	529.9	530.0	530.2
Muskeg Creek	38	2891	526.2	527.1	527.5	527.8	528.0	528.3	528.4	528.5	528.7	529.0	529.2	529.4	529.6	529.7
Muskeg Creek	39	2728	525.7	526.2	526.6	526.8	527.1	527.3	527.5	527.6	527.8	528.0	528.3	528.4	528.6	528.7
Muskeg Creek	40	2628	525.2	525.8	526.1	526.3	526.5	526.7	526.8	526.9	527.0	527.2	527.4	527.5	527.7	527.8
Muskeg Creek	41	2116	521.5	521.9	522.1	522.3	522.5	522.7	522.9	523.0	523.2	523.5	523.7	523.9	524.1	524.3
Muskeg Creek	42	1722	518.3	519.1	519.6	519.9	520.2	520.3	520.4	520.6	520.7	520.9	521.1	521.2	521.3	521.4
Muskeg Creek	43	1406	516.6	517.1	517.3	517.5	517.8	518.0	518.2	518.3	518.4	518.7	519.1	519.4	519.8	520.2
Muskeg Creek	44	1383	516.1	516.7	517.0	517.3	517.6	517.8	518.0	518.1	518.2	518.5	518.8	519.2	519.6	520.0
Muskeg Creek	45	1374	515.9	516.5	516.9	517.2	517.5	517.8	517.9	518.1	518.1	518.4	518.6	518.8	519.0	519.2
Muskeg Creek	46	1162	513.8	514.4	514.8	515.0	515.2	515.4	515.5	515.8	516.0	516.6	517.0	517.3	517.7	518.1
Muskeg Creek	47	973.3	511.6	512.6	513.0	513.3	513.8	514.3	514.7	515.1	515.4	516.3	516.8	517.2	517.6	518.0
Muskeg Creek	48	942.6	511.6	512.3	512.7	513.1	513.7	514.2	514.6	515.0	515.4	516.2	516.8	517.1	517.6	518.0
Muskeg Creek	49	934.2	511.5	512.2	512.6	513.1	513.7	514.2	514.6	515.0	515.3	516.2	516.7	517.1	517.6	518.0
Muskeg Creek	50	825.3	510.9	511.6	512.2	512.9	513.5	514.1	514.5	514.9	515.3	516.2	516.7	517.1	517.6	518.0
Muskeg Creek	51	718.1	510.4	511.2	512.1	512.8	513.5	514.1	514.5	514.9	515.2	516.1	516.2	516.6	517.1	517.5
Muskeg Creek	52	680.2	510.5	511.2	512.1	512.8	513.5	514.1	514.4	514.5	514.8	515.6	516.2	516.7	517.1	517.5
Muskeg Creek	53	609.9	510.4	511.1	512.1	512.8	513.5	514.0	514.1	514.5	514.8	515.6	516.2	516.7	517.2	517.5
Muskeg Creek	54	462.9	509.6	511.1	512.1	512.8	513.5	513.7	514.1	514.5	514.8	515.6	516.2	516.6	517.1	517.5
Muskeg Creek	55	357.2	508.7	511.1	512.1	512.8	513.5	513.7	514.1	514.5	514.8	515.5	516.2	516.6	517.1	517.4
Muskeg Creek	56	249.4	508.5	511.1	512.1	512.8	513.1	513.6	514.0	514.4	514.7	515.5	516.1	516.5	517.0	517.4
Muskeg Creek	57	141.0	508.2	511.0	512.1	512.8	513.1	513.6	513.9	514.4	514.7	515.4	516.0	516.5	517.0	517.3

Note: The highlighted values were interpolated based on the water level at the Athabasca River upstream (XS 5) and downstream (XS 6) cross sections

Table F-3: Tawatinaw River Flood Profiles

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)												
				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
Tawatinaw River	58	6230	519.1	520.3	521.1	521.6	522.1	522.5	522.7	522.9	523.1	523.4	523.7	523.9	524.2	524.3
Tawatinaw River	59	5962	518.6	520.0	520.9	521.4	521.9	522.2	522.4	522.7	522.8	523.2	523.5	523.7	523.9	524.0
Tawatinaw River	60	5944	518.6	520.0	520.9	521.4	521.9	522.2	522.4	522.6	522.8	523.2	523.5	523.7	523.9	524.0
Tawatinaw River	61	5679	518.6	519.8	520.6	521.1	521.6	521.9	522.1	522.3	522.5	522.8	523.1	523.3	523.5	523.7
Tawatinaw River	62	5485	518.0	519.5	520.4	520.9	521.3	521.7	521.9	522.0	522.2	522.5	522.8	523.0	523.2	523.3
Tawatinaw River	63	5274	517.6	519.3	520.2	520.7	521.2	521.5	521.7	521.8	522.0	522.3	522.6	522.8	523.0	523.1
Tawatinaw River	64	4893	517.6	519.1	519.9	520.4	520.8	521.1	521.3	521.5	521.6	522.0	522.3	522.5	522.7	522.9
Tawatinaw River	65	4615	517.3	518.8	519.6	520.1	520.5	520.8	521.0	521.2	521.4	521.7	522.0	522.2	522.4	522.6
Tawatinaw River	66	4468	516.9	518.7	519.5	520.0	520.4	520.7	520.9	521.1	521.2	521.5	521.8	522.0	522.2	522.3
Tawatinaw River	67	4303	517.3	518.4	519.3	519.8	520.2	520.5	520.7	520.9	521.0	521.3	521.6	521.8	521.9	522.1
Tawatinaw River	68	4196	516.8	518.3	519.2	519.6	520.1	520.4	520.5	520.7	520.9	521.2	521.4	521.6	521.8	521.9
Tawatinaw River	69	4033	516.9	518.1	519.0	519.5	519.9	520.2	520.3	520.5	520.7	521.0	521.2	521.4	521.6	521.7
Tawatinaw River	70	3805	516.6	517.9	518.8	519.2	519.6	519.9	520.0	520.2	520.4	520.7	520.9	521.1	521.3	521.4
Tawatinaw River	71	3393	516.0	517.4	518.2	518.5	518.8	519.0	519.1	519.3	519.4	519.7	519.9	520.0	520.2	520.3
Tawatinaw River	72	3179	515.7	516.7	517.4	517.7	518.1	518.3	518.5	518.7	518.8	519.1	519.3	519.5	519.7	519.8
Tawatinaw River	73	2884	514.8	515.9	516.6	517.0	517.4	517.7	517.8	518.0	518.1	518.4	518.7	518.9	519.0	519.2
Tawatinaw River	74	2709	514.6	515.5	516.2	516.7	517.0	517.3	517.4	517.6	517.7	518.0	518.2	518.4	518.5	518.7
Tawatinaw River	75	2450	513.5	514.8	515.4	515.9	516.1	516.3	516.5	516.6	516.7	517.0	517.3	517.5	517.8	518.1
Tawatinaw River	76	2270	512.6	513.3	514.0	514.4	514.7	515.0	515.2	515.4	515.6	516.1	516.6	517.0	517.5	517.8
Tawatinaw River	77	1973	510.7	512.4	513.1	513.6	514.0	514.4	514.6	514.9	515.2	515.8	516.5	516.9	517.4	517.7
Tawatinaw River	78	1706	510.9	512.0	512.6	513.1	513.6	514.1	514.4	514.7	515.0	515.8	516.4	516.8	517.3	517.7
Tawatinaw River	79	1484	510.6	511.5	512.3	512.8	513.4	513.9	514.2	514.6	514.9	515.7	516.4	516.8	517.3	517.7
Tawatinaw River	80	1278	509.9	511.1	512.0	512.7	513.3	513.8	514.2	514.6	514.9	515.7	516.3	516.8	517.3	517.6
Tawatinaw River	81	999.7	509.0	510.8	511.8	512.5	513.2	513.8	514.1	514.5	514.9	515.6	516.3	516.8	517.3	517.6
Tawatinaw River	82	656.7	508.4	510.8	511.8	512.5	513.1	513.7	514.1	514.5	514.8	515.6	516.3	516.7	517.2	517.6
Tawatinaw River	83	607.5	508.4	510.7	511.8	512.4	513.1	513.7	514.1	514.5	514.8	515.6	516.3	516.7	517.2	517.6
Tawatinaw River	84	598.8	508.6	510.7	511.8	512.4	513.1	513.7	514.1	514.5	514.8	515.6	516.3	516.7	517.2	517.6
Tawatinaw River	85	457.5	508.7	510.7	511.7	512.4	513.1	513.7	514.1	514.5	514.8	515.6	516.3	516.7	517.2	517.6
Tawatinaw River	86	383.0	507.7	510.7	511.7	512.4	513.1	513.7	514.1	514.5	514.8	515.6	515.8	516.2	516.7	517.0
Tawatinaw River	87	355.4	507.7	510.7	511.7	512.4	513.1	513.7	514.1	514.5	514.8	515.6	515.8	516.2	516.7	517.0
Tawatinaw River	88	257.3	507.5	510.7	511.7	512.4	513.1	513.3	513.7	514.1	514.4	515.1	515.8	516.2	516.7	517.0
Tawatinaw River	89	152.9	507.6	510.7	511.7	512.1	512.8	513.3	513.7	514.1	514.4	515.1	515.8	516.2	516.7	517.0

Note: The highlighted values were interpolated based on the water level at the Athabasca River upstream (XS 8) and downstream (XS 9) cross sections

APPENDIX G

Open Water Sensitivity Analysis Flood Profiles

Figure G-1: Sensitivity of Simulated Water Level along the Athabasca River Study Reach for the 100-Year Flood Event (Channel Manning's n Only)

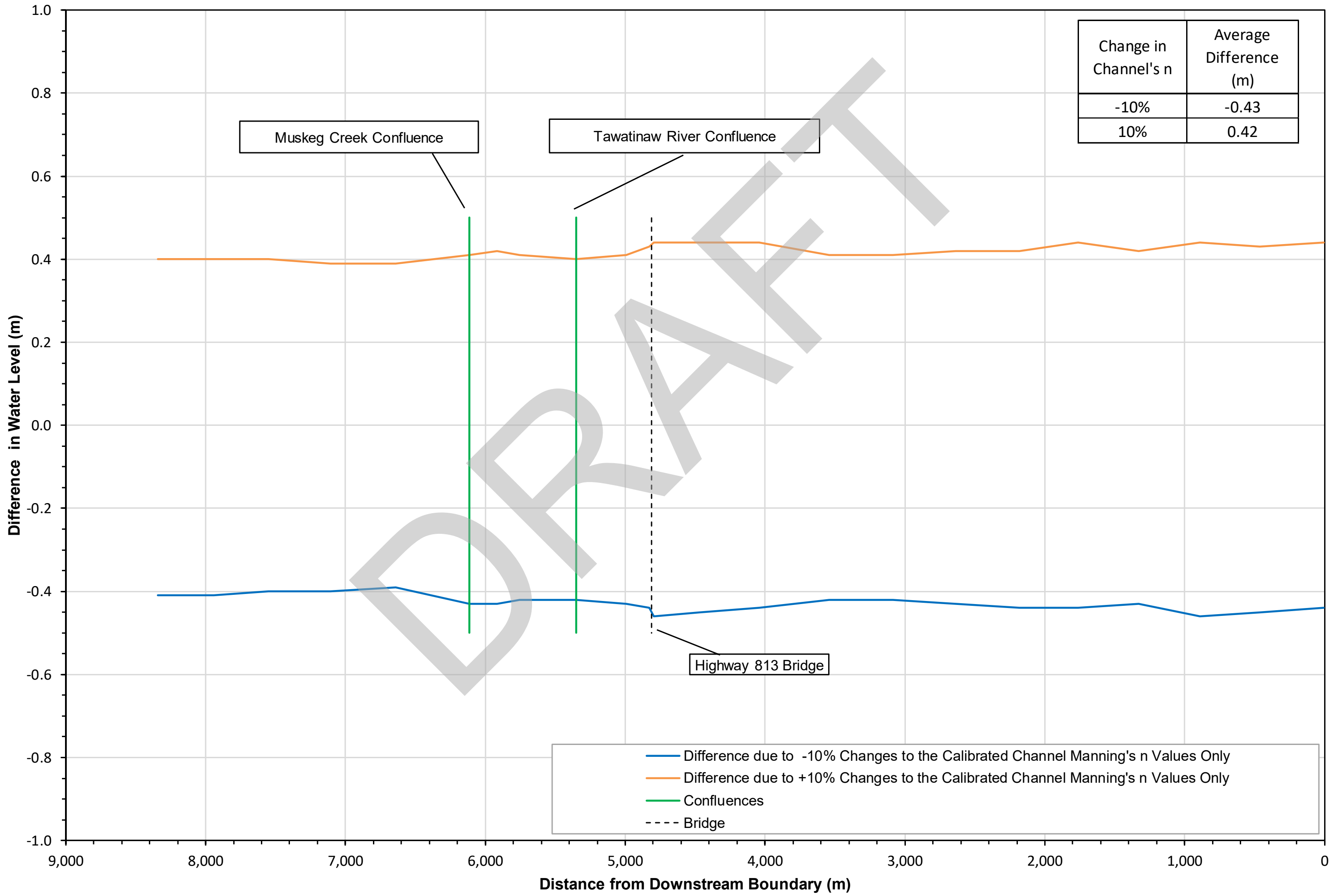


Figure G-2: Sensitivity of Simulated Water Level along the Athabasca River Study Reach for the 100-Year Flood Event (Floodplain Manning's n Only)

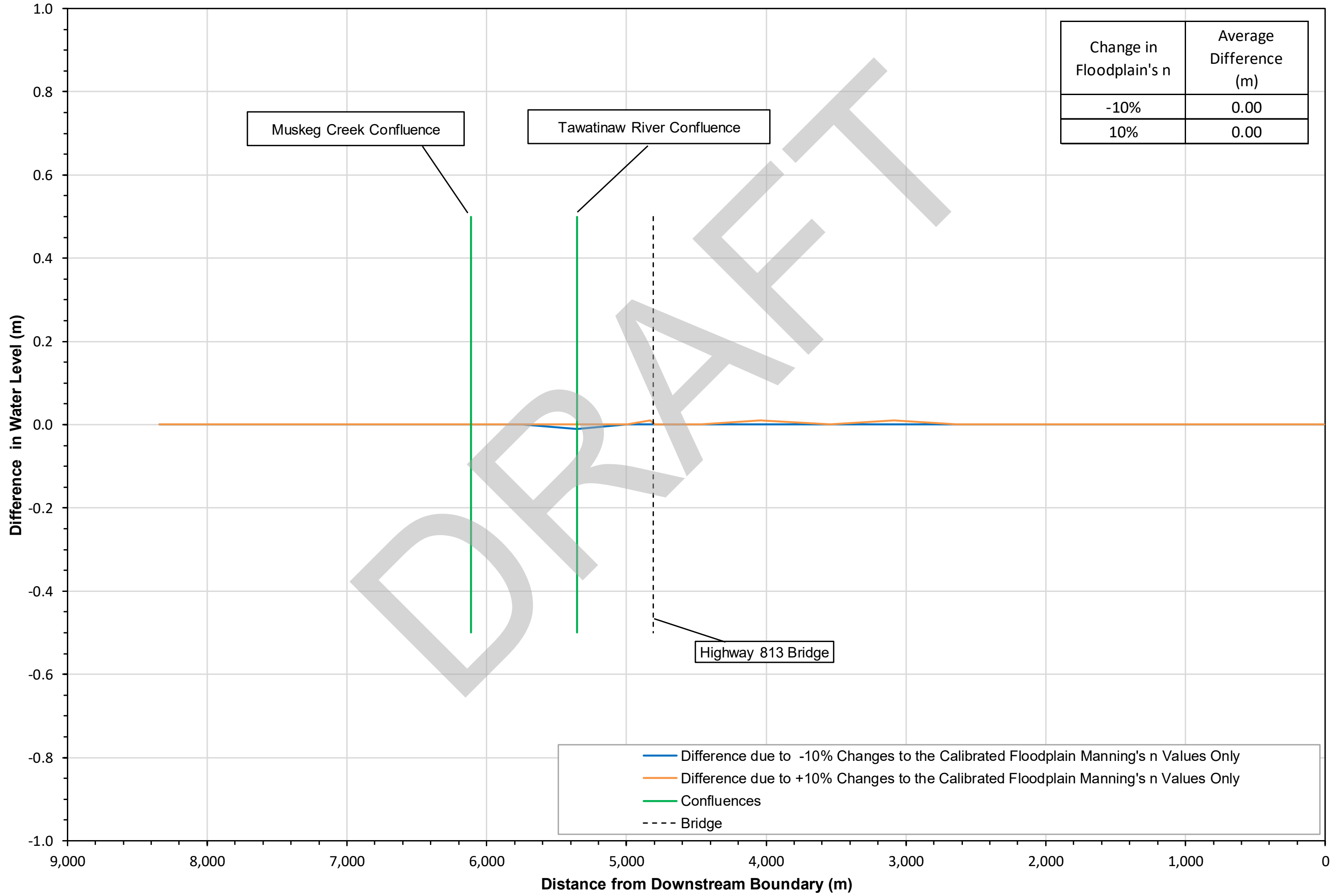


Figure G-3: Sensitivity of Simulated Water Level along the Athabasca River Study Reach for the 100-Year Flood Event (Downstream Boundary)

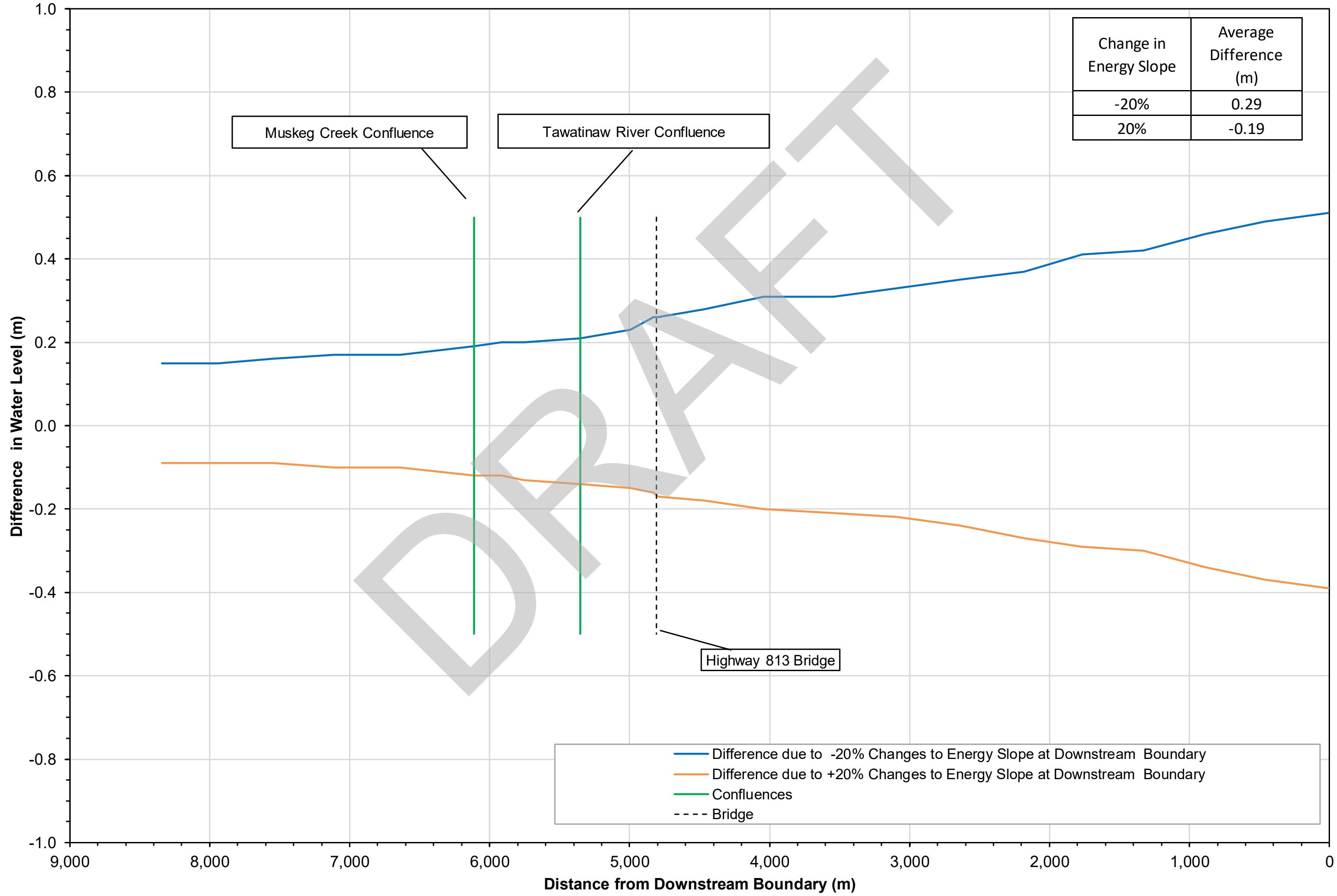


Figure G-4: Sensitivity of Simulated Water Level along the Muskeg Creek Study Reach for the 100-Year Flood Event (Channel Manning's n Only)

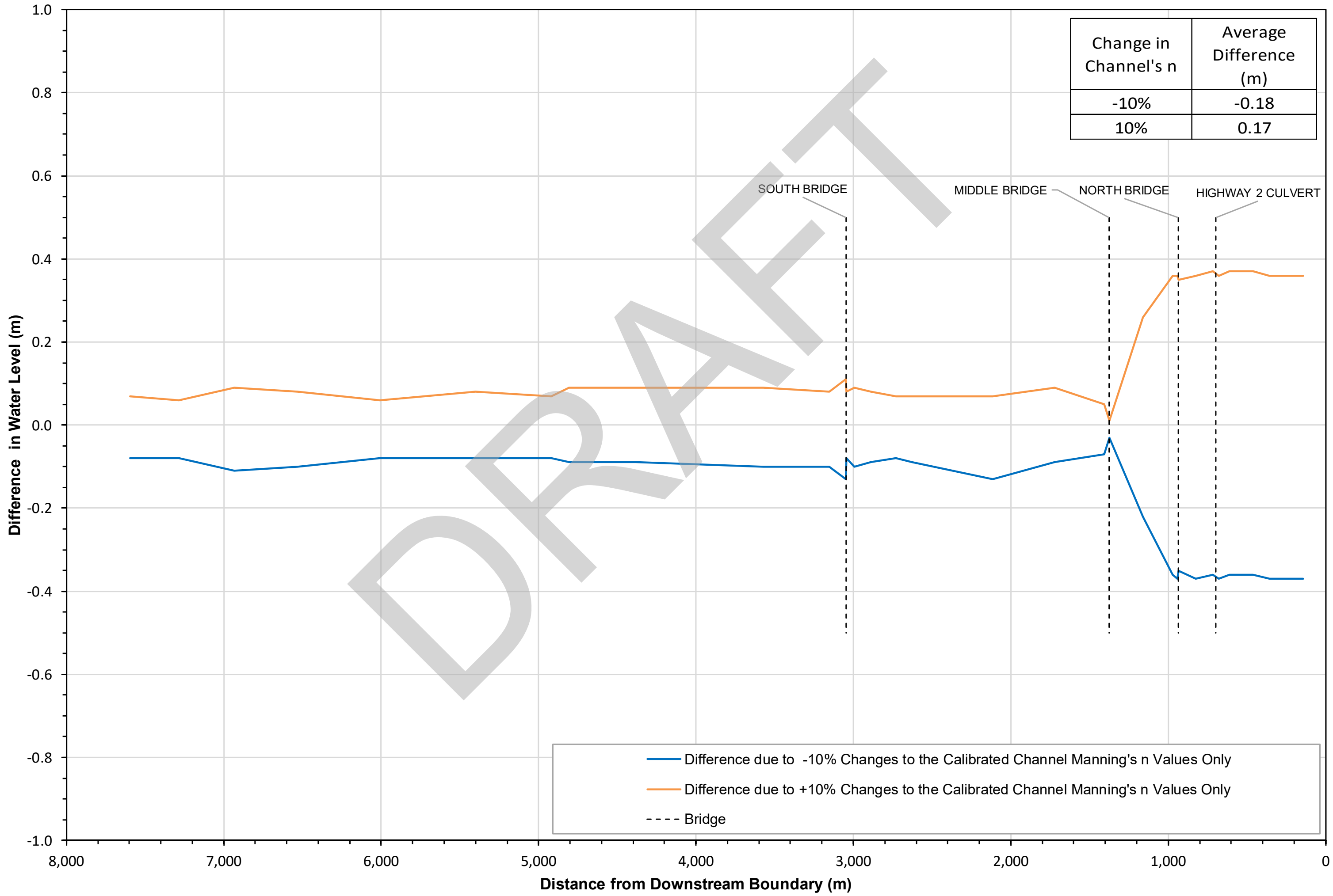


Figure G-5: Sensitivity of Simulated Water Level along the Muskeg Creek Study Reach for the 100-Year Flood Event (Floodplain Manning's n Only)

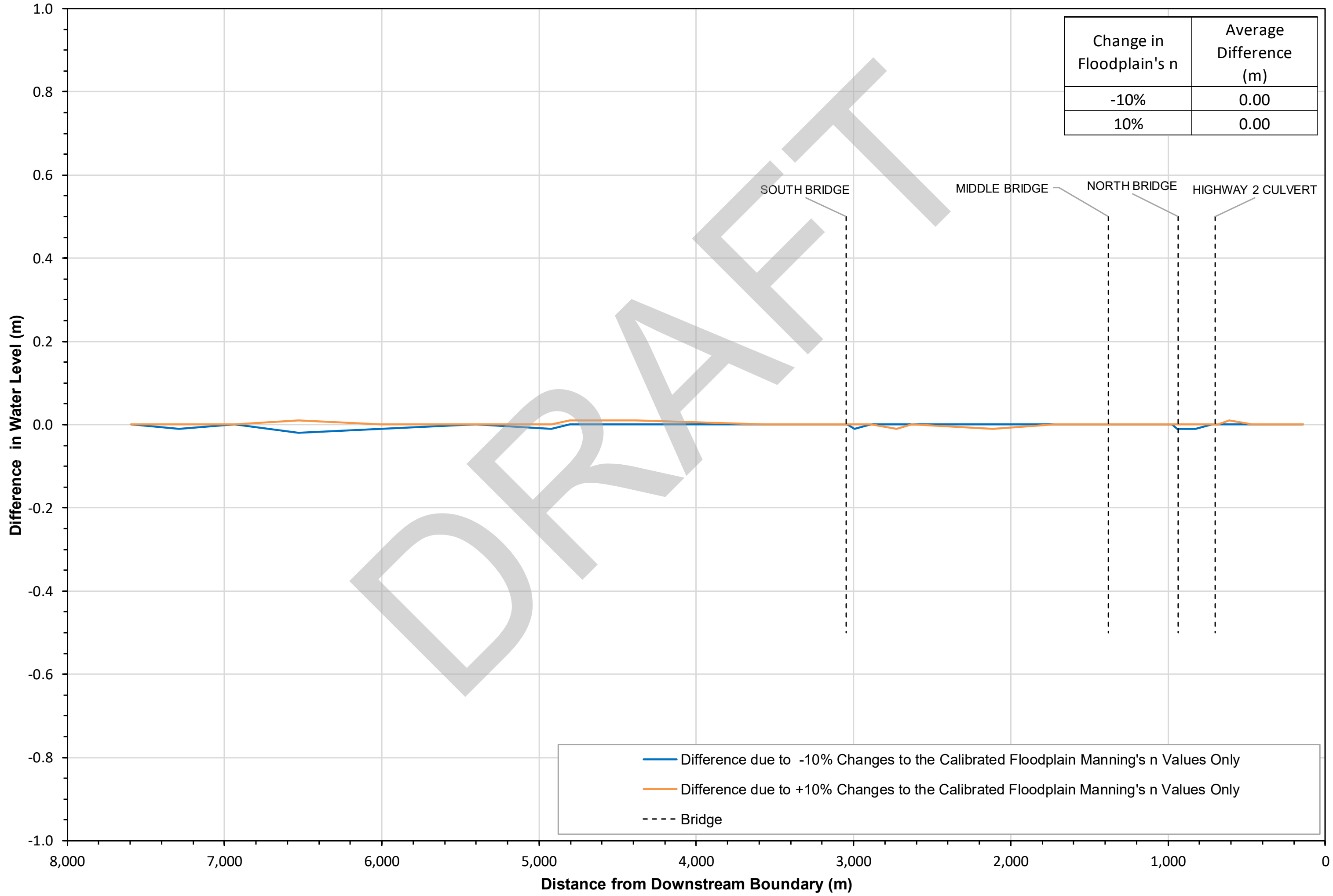
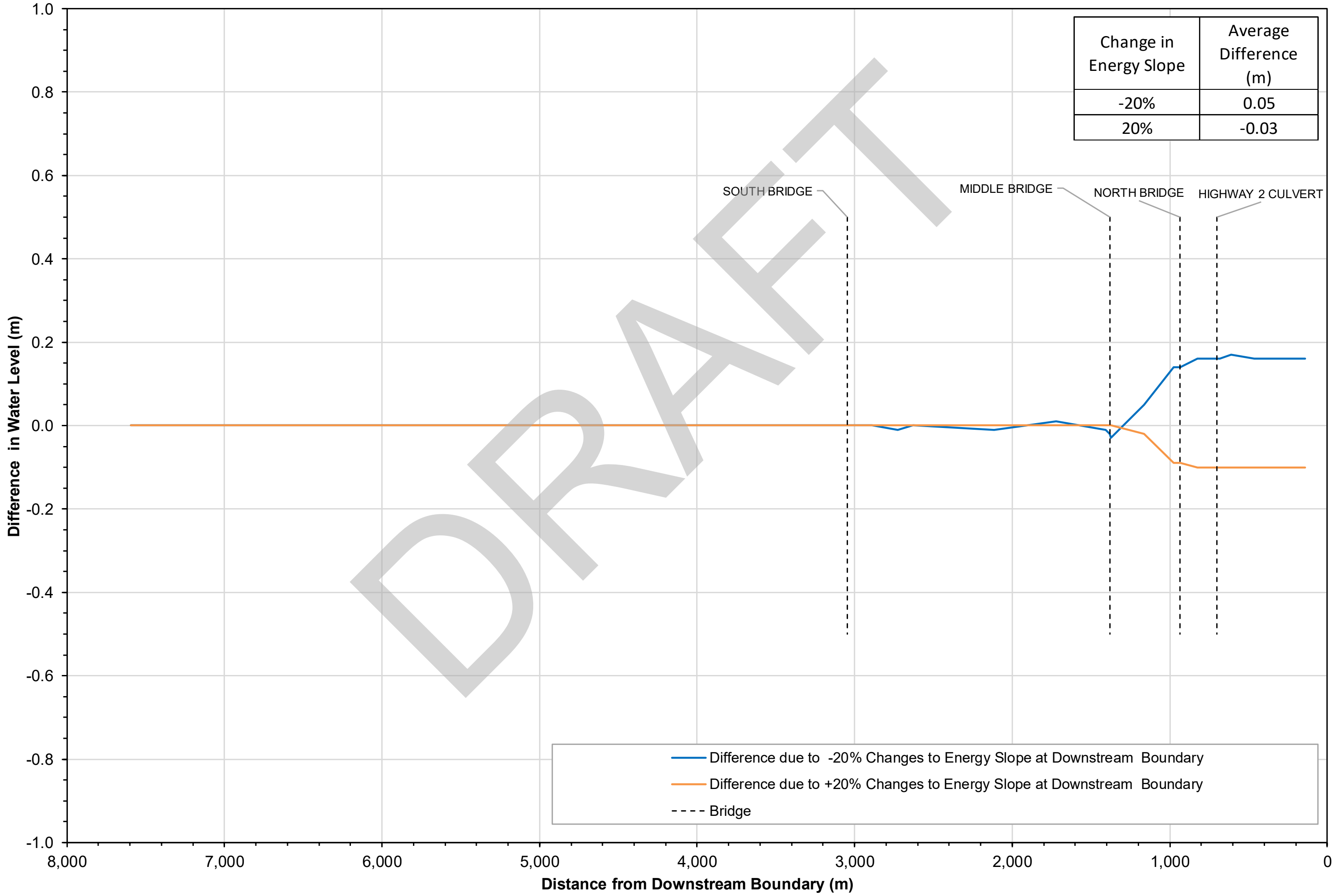
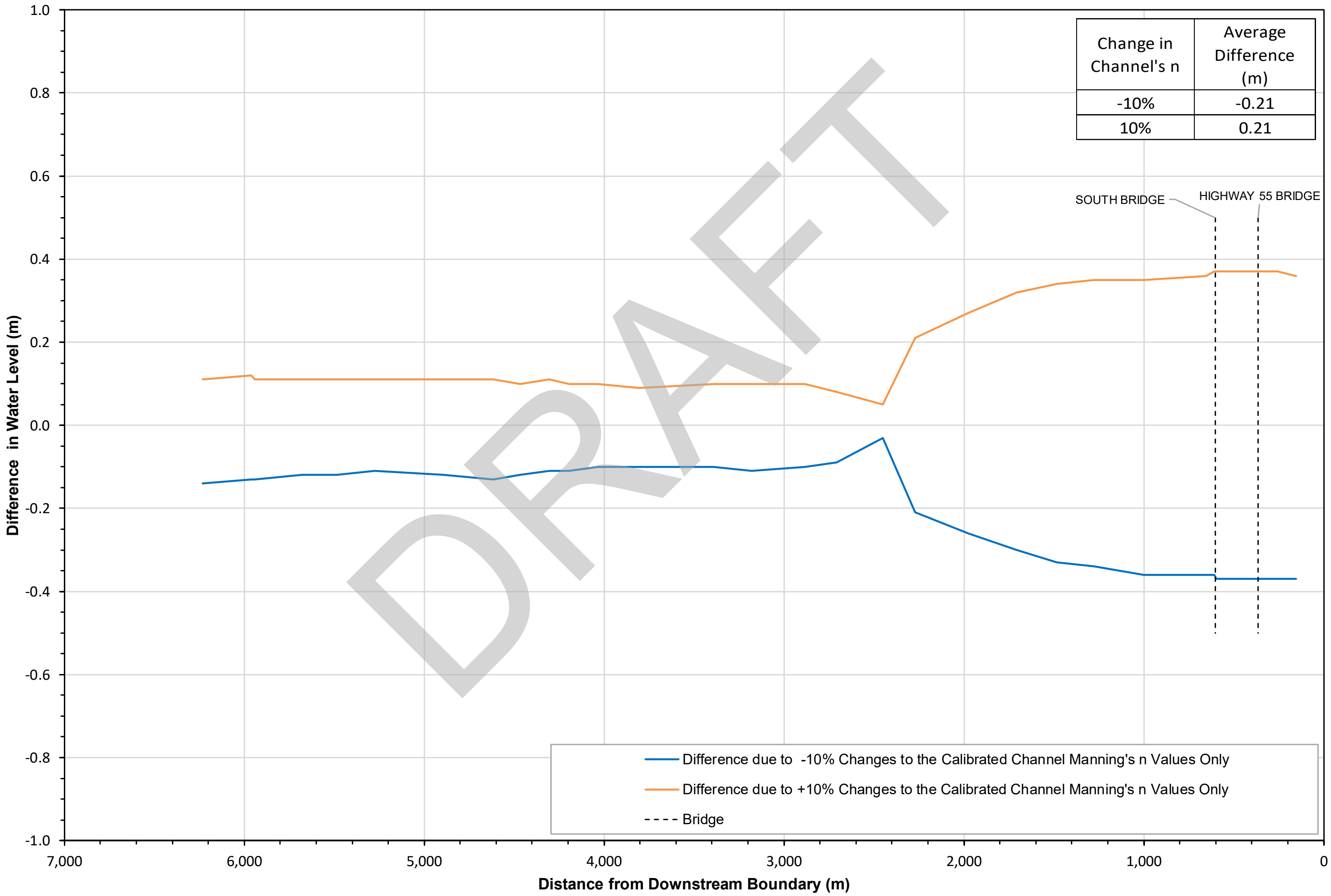


Figure G-6: Sensitivity of Simulated Water Level along the Muskeg Creek Study Reach for the 100-Year Flood Event (Downstream Boundary)



Change in Energy Slope	Average Difference (m)
-20%	0.05
20%	-0.03

Figure G-7: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Channel Manning's n Only)



Change in Channel's n	Average Difference (m)
-10%	-0.21
10%	0.21

Figure G-8: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Floodplain Manning's n Only)

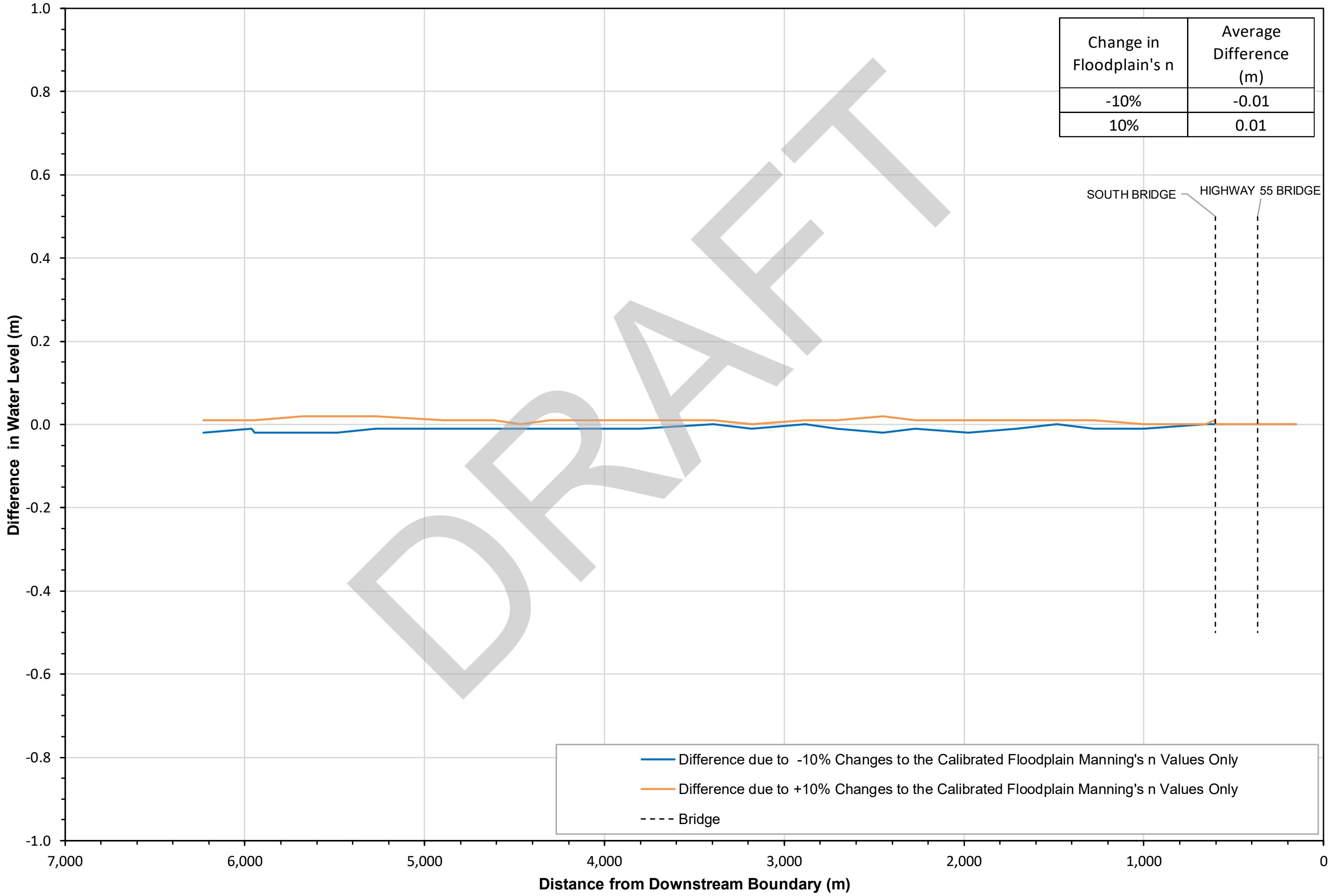
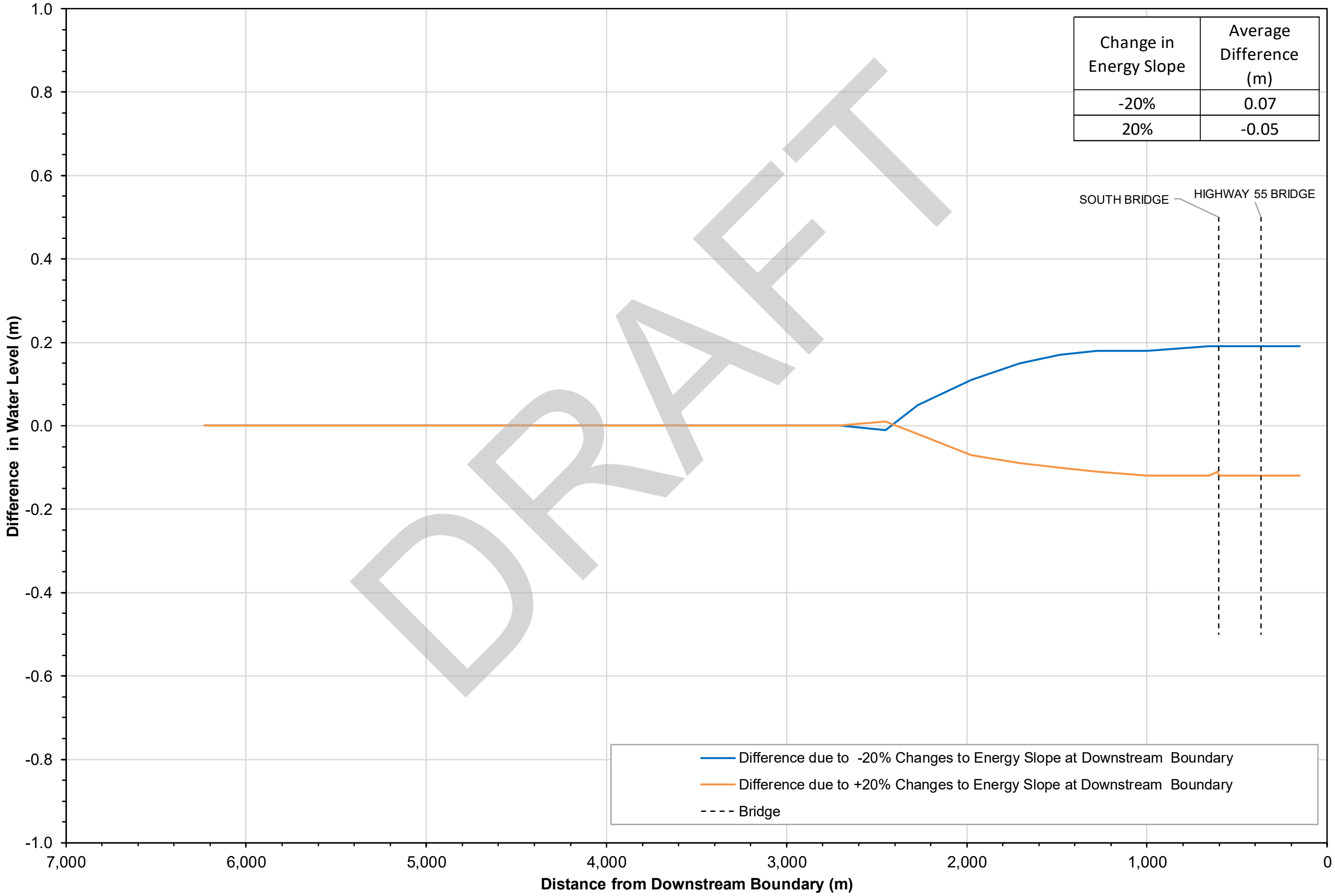


Figure G-9: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Downstream Boundary)



APPENDIX H

Ice Jam Water Level Frequency Analysis

SG1 Ref. No. 10140

7 August 2020

GOLDER ASSOCIATES LTD.
16820 107 Avenue NW
Edmonton, AB
T5P 4C3

Attention: Nathan Schmidt, Ph.D., P.Eng., CPESC
Project Manager

Via email: Nathan_Schmidt@golder.com

Re: **Athabasca Flood Hazard Study**
Ice-related Flood Hazard Assessment – Frequency Analysis of Water Levels
Technical Memorandum

Dear Dr. Schmidt:

1 INTRODUCTION

The objectives of the analysis described herein are to (i) summarize the measured ice-related annual peak water levels along the Athabasca River at the Town of Athabasca (the Town) based on measurements at the Water Survey of Canada (WSC) hydrometric station and any additional anecdotal or historical evidence, (ii) describe the breakup mechanisms and ice-related hydraulic characteristics of the study reach within the context of the current understanding of ice jam mechanics, and (iii) assess the annual ice-related flood hazards using the Bulletin 17B approach (USGS, 1982) to carry out a frequency analysis of the annual peak ice-related water levels. As part of this analysis, the scope of this report includes the following:

1) A summary of measured salient ice-related water levels and flows on the Athabasca River in the vicinity of the WSC hydrometric station.
2) A description of ice-related flood processes and breakup mechanisms that is supported by a quantitative hydraulic assessment of breakup characteristics at the Town.
3) A description of the 1904 ice jam event (thought to be the highest on record) along with its historical context, which supports the choice of the record length and the historical threshold ice jam level that is fundamental to the application of Bulletin 17B methodologies.
4) A description of the analytical technique that was adopted (i) to assign a frequency to historical water levels and (ii) to define the ice-related frequency curve at the WSC hydrometric station.

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5) A description of the simulation of steady-state, gradually varied ice jam profiles to extrapolate ice-related water levels at the WSC gauge to other locations within the flood hazard mapping domain.

2 ICE-RELATED WATER LEVELS AT THE TOWN

Ice-related water levels during breakup have been recorded at the WSC gauge (#07BE001, Athabasca River at Athabasca) since 1914. The existing gauge is located at the water intake operated by the Town and has been more or less at that same location since the station was established. Operation of the gauge, however, has changed over time in response to changes in monitoring priorities and to stay current with emerging technologies. Regardless of how the gauging methodology varied throughout the period of record, the gauge datums were maintained in a systematic fashion, with adjustments made to the gauge zero to compensate for changing datums. The current gauge datum, which was established in 1971, has "gauge zero" set at elevation 506.873 m. The following summarizes the characteristics of the gauge record in terms of the measurements that were undertaken and reported over the period of record:

1) 1913: The gauge was installed by WSC.
2) 1914-1931: Water levels were measured manually once or twice a day, and reported as daily water levels and discharges, from just prior to breakup, during the breakup period, and throughout the open water period. No peak instantaneous water levels were reported, although it is possible that in some years the maximum daily water level reported during the breakup period may represent an instantaneous peak.
3) 1932-1937: The gauge was not operating.
4) 1938-1941: The gauge was not operational during the breakup period. No ice-related water levels were reported. Water levels were recorded manually in May to October inclusive and reported as daily water levels and discharges.
5) 1942-1951: Water levels were measured manually. Only daily water levels were reported during the breakup period. Daily water levels and discharges were reported for the May to October period. Instantaneous peak water levels during the breakup period were reported in some years.
6) 1952-1958: Water levels were measured manually over the entire year. Instantaneous peak water levels during the breakup period were contained in the gauge file for some years, but the source of that data is not clear.
7) 1959-2019: Water levels were measured with an automatic recording gauge throughout the entire year. The data in this period provide the best indication of general breakup characteristics. Peak data are missing during breakup in some years when ice disrupted the gauge. Water levels were recorded on strip charts for the period between 1959 and 1999, whereas water levels were recorded digitally from 2000 onwards.

Salient ice-related information that would be useful in developing an understanding of the breakup characteristics and that could be used in the statistical analysis to quantify the ice-related hazards is summarized in Table 1. The table provides the following information for each year that ice-related observations are available:

1) Last date of stable ice cover when the late-winter ice-related rating curve is assumed to apply. This parameter is deduced from the water level trends and represents the ice conditions just before the ice cover appears to destabilize and significant shifts from the winter rating curve start to occur.
2) Water level on date of last stable ice cover.
3) Reported discharge on date of last stable ice cover, based on WSC extrapolations of the winter rating curve. This is an estimate of the likely minimum flow that occurred during the breakup period.
4) Date of peak daily water level during the breakup period.
5) Peak daily water level during the breakup period. No discharge is attached to this event due to imprecise estimation procedures.
6) Date of peak instantaneous water level during the breakup period.
7) Peak instantaneous water level, either based on gauge data or measured from highwater marks. No discharge is attached to this water level due to imprecise estimation procedures.
8) Date of first open water when ice effects have vanished and, therefore, the open water rating curve would apply.
9) Water level on first day of open water.
10) Reported discharge on first day of open water.
11) Salient notes about the quality of the data and/or the breakup characteristics. These comments also provide an indication of the reliability of the monitoring protocols, particularly during the period when the automatic recording gauge was operating.

In addition to the systematic ice-related measurements reported by WSC, anecdotal evidence on breakup severities could have been available prior to the installation of the WSC gauge and in periods when it was not operating. Appendix I provides a brief summary of the settlement history at the Town and how local presence may have affected the interpretation of what would have been a significant event worthy of documentation. Based on that information, severe ice events could have been observed since about 1884, when the area was first settled. However, there is little anecdotal information about severe breakup events except for the 1904 event when the ice-related water level reached an estimated elevation of 517.5 m.

Table 1. Summary of ice-related water levels and discharges at the WSC gauge during breakup of the Athabasca River at Athabasca

Year	Last Pre-Breakup Stable Ice Condition			Peak Daily Water Level		Peak Instantaneous Water Level			First Day of Open Water			Comments
	Date	Water Level (m)	Discharge (m^3/s)	Date	Water Level (m)	Date	Water Level (m)	Estimated Discharge (m^3/s)	Date	Water Level (m)	Discharge (m^3/s)	
1900												
1901												
1902												
1903												
1904						17-Apr	517.5					See Appendix I for description
1905												
1906												
1907												
1908												
1909												
1910												
1911												
1912												
1913												Gauge installed, no record at breakup
1914	20-Apr	508.610	95.4	23-Apr	508.763				30-Apr	508.168	348	
1915	2-Apr	508.275	442	6-Apr	508.458				5-Apr	508.397	442	
1916	21-Apr	508.397	124	25-Apr	509.516				1-May	508.772	577	Jammed, Apr 22-25
1917	17-Apr	508.367	113	28-Apr	509.327				1-May	508.498	468	
1918	13-Apr	508.437	173	17-Apr	510.957				21-Apr	508.519	477	
1919	13-Apr	508.123	164	16-Apr	508.717				21-Apr	508.107	297	
1920	5-May	509.037	414	8-May	510.226				9-May	510.811	1880	
1921	15-Apr	508.979	334	16-Apr	509.269				23-Apr	509.086	829	
1922	20-Apr	508.004	101	4-May	508.952				4-May	508.952	677	
1923	20-Apr	507.790	116						26-Apr	508.540	549	Missing data Apr 22-25
1924	26-Apr	507.629	113						4-May	509.458	1040	Missing data Apr 27 to May 3
1925	10-Apr	508.528	351						21-Apr	510.512	1880	Missing data Apr 11-20
1926	13-Apr	508.159	215	20-Apr	509.129				24-Apr	508.848	657	Missing data Apr 11-18, peak likely higher
1927	13-Apr	508.632	139	23-Apr	509.104				28-Apr	509.555	1060	Missing data Apr 14-15, Apr 26-27, peak may have been higher in late April

Table 1. Summary of ice-related water levels and discharges at the WSC gauge during breakup of the Athabasca River at Athabasca

Year	Last Pre-Breakup Stable Ice Condition			Peak Daily Water Level		Peak Instantaneous Water Level			First Day of Open Water			Comments
	Date	Water Level (m)	Discharge (m^3/s)	Date	Water Level (m)	Date	Water Level (m)	Estimated Discharge (m^3/s)	Date	Water Level (m)	Discharge (m^3/s)	
1928	25-Apr	508.449	247	29-Apr	510.195				30-Apr	509.525	1040	Missing data Apr 26
1929	23-Apr	508.519	286						2-May	508.653	583	Missing data Apr 24 to May 1
1930	7-Apr	508.278	182	15-Apr	508.394				21-Apr	508.211	385	Missing data Apr 08-14, peak likely higher
1931	15-Apr	508.248	206	16-Apr	508.345							No record after Apr 16
1932												Gauge discontinued
1933												Gauge not operating
1934												Gauge not operating
1935												Gauge not operating
1936												Gauge not operating
1937												Gauge not operating
1938												No data before May 1
1939												No data before May 1
1940												No data before May 1
1941												No data before May 1
1942	18-Apr	508.007		26-Apr	508.458							Water levels only before May 1
1943	10-Apr	508.351		16-Apr	511.792	15-Apr	513.426					Water levels only before May 1
1944	8-Apr	508.397		12-Apr	508.833	12-Apr	509.665					Water levels only before May 1
1945	22-Apr	507.922		27-Apr	508.598							Water levels only before May 1
1946	27-Apr	508.309		30-Apr	508.610							Water levels only before May 1
1947	16-Apr	508.534		20-Apr	510.247	17-Apr	510.729					Water levels only before May 1
1948	23-Apr	508.013		1-May	510.713	1-May	511.402					Water levels only before May 1
1949	14-Apr	509.107		16-Apr	510.372							Water levels only before May 1
1950	20-Apr	508.184		27-Apr	509.394							Water levels only before May 1
1951	17-Apr	508.455		19-Apr	508.882							Water levels only before May 1
1952	10-Apr	508.833	291						20-Apr	509.720	1180	Missing data Apr 13-19
1953	24-Apr	508.153	209	27-Apr	508.574	30-Apr	508.650	380	1-May	508.245	408	
1954	5-May	508.129	153	8-May	508.882	9-May	509.324	404	10-May	508.857	467	
1955	13-Apr	508.970	390	18-Apr	509.455	22-Apr	509.692	321	24-Apr	508.095	306	
1956	17-Apr	508.854	258	19-Apr	510.774	19-Apr	511.125	815	21-Apr	510.098	1370	

Table 1. Summary of ice-related water levels and discharges at the WSC gauge during breakup of the Athabasca River at Athabasca

Year	Last Pre-Breakup Stable Ice Condition			Peak Daily Water Level		Peak Instantaneous Water Level			First Day of Open Water			Comments
	Date	Water Level (m)	Discharge (m^3/s)	Date	Water Level (m)	Date	Water Level (m)	Estimated Discharge (m^3/s)	Date	Water Level (m)	Discharge (m^3/s)	
1957	23-Apr	508.757	190	24-Apr	509.439	25-Apr	509.933	292	1-May	508.818	597	
1958	8-Apr	508.562	231			15-Apr	511.902	610	20-Apr	509.372	881	Missing data, Apr 12-18; peak likely higher
1959	9-Apr	508.141	142	11-Apr	508.220	12-Apr	508.488	180	16-Apr	507.827	231	Missing data, Apr 12; first year that paper strip charts are available
1960	13-Apr	508.744	254	15-Apr	509.007				24-Apr	507.797	215	Missing data, Apr 18
1961	19-Apr	508.385	178	24-Apr	508.604	26-Apr	509.296	217	28-Apr	507.781	228	Missing data, Apr 26 and Apr 30
1962	18-Apr	509.071	385	19-Apr	509.214	20-Apr	511.140	678	23-Apr	509.610	1120	Missing data, Apr 21-22
1963	17-Apr	509.686	230	20-Apr	510.558	19-Apr	514.609	613	23-Apr	509.970	1380	Missing data, Apr 19; reported peak determined manually; highest recorded event on record
1964	17-Apr	508.434	98.0	20-Apr	508.775	21-Apr	509.570	167	26-Apr	507.858	254	
1965	23-Apr	509.555	479	26-Apr	509.695	26-Apr	511.262	1150	1-May	511.253	2270	Record is complex; secondary peak on Apr 30 appears to be flow related
1966	8-Apr	509.656	270	11-Apr	509.799	12-Apr	509.799	301	5-May	508.485	479	Clock stopped; peak is a point measurement
1967	29-Apr	509.129	597	1-May	509.226	2-May	511.506	714	2-May	509.080	714	
1968	18-Apr	508.050	139	18-Apr	508.147	19-Apr	508.641	143	22-Apr	507.583	157	Thermal breakup
1969	11-Apr	508.174	323	15-Apr	510.988	15-Apr	511.515	473	21-Apr	508.973	699	
1970	16-Apr	509.369	470	17-Apr	509.729	17-Apr	511.308	481	22-Apr	508.626	538	
1971	19-Apr	508.769	170	22-Apr	509.461	21-Apr	510.592	610	23-Apr	509.589	1050	
1972	18-Apr	509.305	340	22-Apr	509.421	21-Apr	511.780	513	23-Apr	508.830	629	Jam likely formed and failed on Apr 22
1973	15-Apr	508.757	385	15-Apr	508.757	20-Apr	509.537	478	21-Apr	508.580	496	
1974	17-Apr	509.061	612	19-Apr	509.528	19-Apr	510.799	1240	21-Apr	510.744	1860	
1975	23-Apr	508.830	306	25-Apr	509.247	26-Apr	510.774	452	28-Apr	508.693	549	
1976	10-Apr	508.790	377	15-Apr	509.159	11-Apr	509.555	456	16-Apr	509.269	852	
1977	10-Apr	508.717	275	12-Apr	510.415	12-Apr	511.500	564	13-Apr	509.308	708	
1978	12-Apr	508.958	340	14-Apr	509.662				19-Apr	508.299	379	Gauge not functioning; data recorded manually
1979	23-Apr	509.176	426	26-Apr	509.686	27-Apr	510.067	687	29-Apr	509.200	817	
1980	13-Apr	508.851	253	15-Apr	509.229	15-Apr	509.843	362	21-Apr	508.970	688	
1981	2-Apr	508.798	279	4-Apr	508.971	4-Apr	509.887	340	8-Apr	508.470	463	
1982	22-Apr	508.269	164	25-Apr	509.027	24-Apr	510.313	408	27-Apr	509.076	773	
1983	16-Apr	508.550	140	13-Apr	508.669	19-Apr	509.183	245	22-Apr	508.188	349	

Table 1. Summary of ice-related water levels and discharges at the WSC gauge during breakup of the Athabasca River at Athabasca

Year	Last Pre-Breakup Stable Ice Condition			Peak Daily Water Level		Peak Instantaneous Water Level			First Day of Open Water			Comments
	Date	Water Level (m)	Discharge (m^3/s)	Date	Water Level (m)	Date	Water Level (m)	Estimated Discharge (m^3/s)	Date	Water Level (m)	Discharge (m^3/s)	
1984	5-Apr	508.488	225	7-Apr	508.843	7-Apr	509.183	243	10-Apr	507.974	269	
1985	11-Apr	509.615	230	12-Apr	509.729	13-Apr	510.773	409	15-Apr	508.735	588	
1986	13-Apr	509.102	350	13-Apr	509.102	18-Apr	509.433	428	22-Apr	508.531	490	Missing strip chart data, Apr 15-17
1987	13-Apr	509.284	265	13-Apr	509.284	15-Apr	510.132	369	17-Apr	508.490	472	
1988	7-Apr	507.998	96.3	10-Apr	508.349	11-Apr	508.803	133	16-Apr	507.682	179	
1989	19-Apr	508.734	165	21-Apr	509.122	21-Apr	510.453	218	26-Apr	508.193	351	
1990	14-Apr	508.685	304	11-Apr	508.835	18-Apr	509.033	364	22-Apr	508.379	424	
1991	11-Apr	508.883	303	14-Apr	509.439	15-Apr	509.564	365	18-Apr	508.657	411	
1992	13-Apr	507.880	253	13-Apr	509.130				14-Apr	507.820	228	Clock stopped; missing data, Apr 14-17
1993	14-Apr	508.375	184	16-Apr	508.797	17-Apr	509.068	212	22-Apr	507.922	258	
1994	8-Apr	509.221	328	8-Apr	509.221	9-Apr	509.975	346	21-Apr	508.679	560	
1995	11-Apr	508.066	128	14-Apr	508.348	14-Apr	508.600	159	20-Apr	507.802	222	
1996	14-Apr	508.348	380	20-Apr	509.601	16-Apr	511.261	571	21-Apr	509.551	1050	Possible jam lasting for about 2 hours
1997	17-Apr	509.061	669	19-Apr	509.517	19-Apr	511.593	976	20-Apr	509.672	1130	
1998	7-Apr	508.545	164	9-Apr	508.972	12-Apr	509.323	267	14-Apr	508.075	308	
1999	15-Apr	508.553	246	17-Apr	509.015	17-Apr	509.453	412	20-Apr	508.866	660	Last year of paper strip charts
2000	17-Apr	507.859	96.3	19-Apr	507.982	19-Apr	508.113	157	24-Apr	507.640	178	First open water could be as early as Apr 22; first year of digital charts
2001	17-Apr	508.179	97.1	21-Apr	508.179	21-Apr	508.213	128	28-Apr	507.665	184	Ice went out on Apr 19; jam occurred Apr 19-22; surge on Apr 16 raised water level to El. 508.2 m
2002	29-Apr	509.117	172	1-May	509.234	1-May	509.975	296	3-May	508.369	420	
2003	19-Apr	509.342	250	22-Apr	510.436	23-Apr	510.507	615	27-Apr	509.422	980	Ice run on Apr 22; secondary peak on Apr 24; gauge not operative Apr 21-27; data appears to be collected manually
2004	8-Apr	508.463	284	14-Apr	509.187	14-Apr	510.944	293	23-Apr	508.068	306	Instantaneous peak water level is higher than the reported hourly peak level shown on Figure 4
2005	3-Apr	508.590	165	6-Apr	509.061	6-Apr	510.034	297	9-Apr	508.390	428	
2006	11-Apr	508.639	182	15-Apr	509.458	15-Apr	509.704	336	17-Apr	508.351	413	Digital record is suspect between Apr 14-15; peak may have been higher
2007	15-Apr	508.390	180	18-Apr	509.464	18-Apr	510.687	581	20-Apr	509.207	848	
2008	15-Apr	508.209	180	19-Apr	508.693	29-Apr	508.776	332	1-May	508.203	354	

Table 1. Summary of ice-related water levels and discharges at the WSC gauge during breakup of the Athabasca River at Athabasca

Year	Last Pre-Breakup Stable Ice Condition			Peak Daily Water Level		Peak Instantaneous Water Level			First Day of Open Water			Comments
	Date	Water Level (m)	Discharge (m^3/s)	Date	Water Level (m)	Date	Water Level (m)	Estimated Discharge (m^3/s)	Date	Water Level (m)	Discharge (m^3/s)	
2009	17-Apr	508.552	181	19-Apr	508.848	19-Apr	509.345	235	26-Apr	508.376	423	
2010	5-Apr	507.884	80.0	7-Apr	507.943	7-Apr	508.039	84.0	19-Apr	507.408	110	
2011	21-Apr	508.657	207	25-Apr	508.740	25-Apr	509.224	409	2-May	509.190	763	
2012	20-Apr	508.958	350		506.873				26-Apr	508.627	547	Missing record Apr 22-24, peak may have been higher
2013	25-Apr	509.112	262	27-Apr	509.915	27-Apr	510.909	565	30-Apr	509.622	1020	
2014	20-Apr	509.479	143	20-Apr	509.479	22-Apr	510.144	406	24-Apr	509.279	668	
2015	3-Apr	509.123	469	5-Apr	510.265	5-Apr	511.078	459	12-Apr	508.392	423	Missing record Apr 10-14, WSC estimate used for first day of open water
2016									25-Apr	507.907	249	Missing record Apr 5-13
2017									22-Apr	508.875	643	Missing record Apr 8-12
2018	22-Apr	508.838	249	1-May	510.736	1-May	511.093 (512.62)	970	5-May	509.976	1290	Missing record, Apr 23-26, peak may be underestimated, ice-related highwater mark downstream of gauge surveyed to El. 512.62 m
2019	5-Apr	509.838	948	7-Apr	510.039	7-Apr	510.413	905	5-May	508.059	303	Missing record Apr 8-16, peak appears to have occurred prior to Apr 8

The historical record of peak daily and the peak instantaneous ice-related breakup levels is shown on Figure 1. The 1904 event is almost 2.9 m higher than the next highest event, which occurred in 1963. In years when both a daily and instantaneous peak are reported, the instantaneous levels are higher than the mean daily levels by an average 0.82 m, with the minimum and maximum differences being 0.03 m and 4.05 m, respectively. The most severe event recorded at the gauge was in 1963 when an instantaneous peak level of 514.61 m was measured.

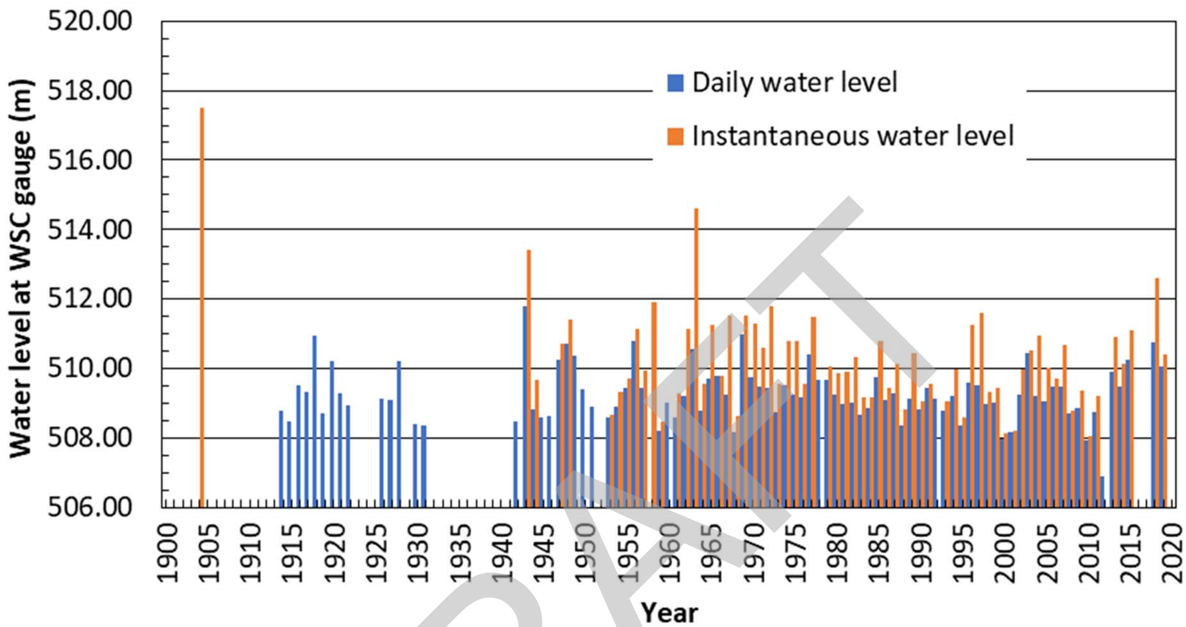

Figure 1. Peak ice-related water levels during the breakup period.

3 ICE-RELATED FLOOD MECHANISMS AND REACH-AVERAGE ICE CHARACTERISTICS

3.1 General

Breakup on the Athabasca River progresses in an upstream to downstream direction (from west to east) beginning at the mouth of the Pembina River, which provides the earliest substantial contribution of snowmelt runoff to the Athabasca River upstream of the Town, (Andres and Rickert, 1984). During the period prior to breakup, the ice cover deteriorates while flows increase. Breakup occurs once the ice cover within the Town destabilizes. The mechanisms of breakup at the Town are somewhat complex and depend on a range of hydrometeorological factors such as the amount of snow available for runoff, the meteorological characteristics of the melt, the prevailing late-winter ice thickness, and the freeze-up level.

Breakup at the Town can be characterized generally as either thermal or mechanical. Thermal breakup occurs in years when there is low snowmelt runoff, or it develops slowly, and the ice cover can deteriorate before it is dislodged by rising water levels. In this situation, the deteriorating ice cover provides little impediment to the flushing of the ice cover and, consequently, water levels at breakup do not rise significantly above the pre-breakup levels.

In years when snowmelt runoff is high, or it develops rapidly prior to a significant deterioration of the ice cover, the only way to dislodge the ice is by mechanical means, hence the term “mechanical breakup.” Mechanical breakup occurs at the Town through the action of surging ice and water, which occurs when an upstream jam fails. The ice floes within the surge are driven up against the intact ice cover, thereby creating a breaking front that moves downstream. If the intact ice cover is deteriorated, only a surge (or an ice run) will develop. However, if the intact ice cover is competent, or the surge is weak, a jam will form once the surge ceases. The height of the jam will be related to the local channel geometry and the ambient (carrier) discharge that sustains the surge. If the carrier discharge during breakup is generally low, the resulting peak water level derived either from a surge or a stable ice jam would be relatively low compared to a situation where the carrier discharge is high.

Flows are unsteady during the breakup period as the snowmelt runoff develops in response to the cumulative energy inputs to the snowpack. Nevertheless, flows generally increase monotonically (with short term variations) over time from pre-breakup through the breakup period and, finally, to the point when open water conditions develop. Typically, the carrier discharge during breakup is low relative to the spring peak, because the ice-related hydraulic processes occur early in the spring runoff period, well before the arrival of the spring runoff peak from the upper part of the Athabasca River basin. Should ice jams form under a high carrier discharge (in years with substantial snowmelt) the resulting jams will be higher and the subsequent surges will be more severe when the jams release. In years with low snowmelt runoff, the carrier discharge will be less and both the jam heights and corresponding surges will be less severe.

Defining the carrier discharge is difficult, because of the complexities introduced to its measurement by the rapidly changing ice conditions. Typically, the best estimate of the carrier discharge can only be made on the first day of open water when the open water rating curve would apply. A less accurate estimate can be made on the last day of a stable ice cover when the late-winter ice rating curve would provide an indication of the discharge. In between those dates, flow estimates are difficult, but a reasonable first order estimate would be to assume that the carrier discharge varies linearly with time between those two dates.

In summary, ice-related breakup events can be classified into three categories. The most benign is a thermal breakup, which is defined as one where there is an insufficient increase in the carrier discharge to mechanically break the ice cover before significant amounts of ice melts in situ. The resulting ice-related stage increase above pre-breakup water levels is small, with a typical water level response at the WSC gauge for this situation shown on Figure 2. The next most severe condition occurs during a mechanical breakup, in which there is a sufficient rise in the carrier discharge to mobilize the ice cover while it still has some structural integrity; a surge occurs, but an ice jam does not form. The most severe condition occurs during a mechanical breakup when the downstream passage of a surge is stopped and a stable or equilibrium jam develops on the river throughout the Town. Depending on the resistance of the ice downstream of the Town and the strength of the surge, the Town may experience only a surge or a fully developed equilibrium jam of some consequence. Typical water level responses to the passage of a surge and to the formation of a stable ice jam are shown on Figures 3 and 4, respectively.

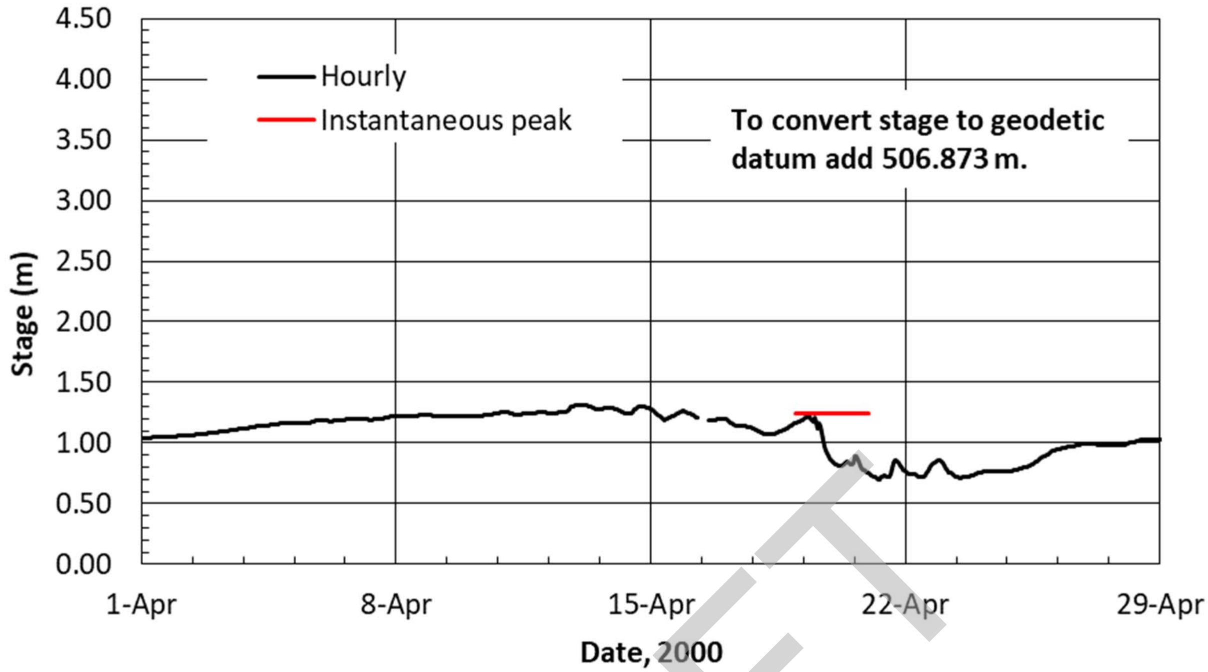

Figure 2. Typical water levels during a thermal breakup. *Flows remain relatively steady until ice melts out on Apr 17-18, and water levels drop.*

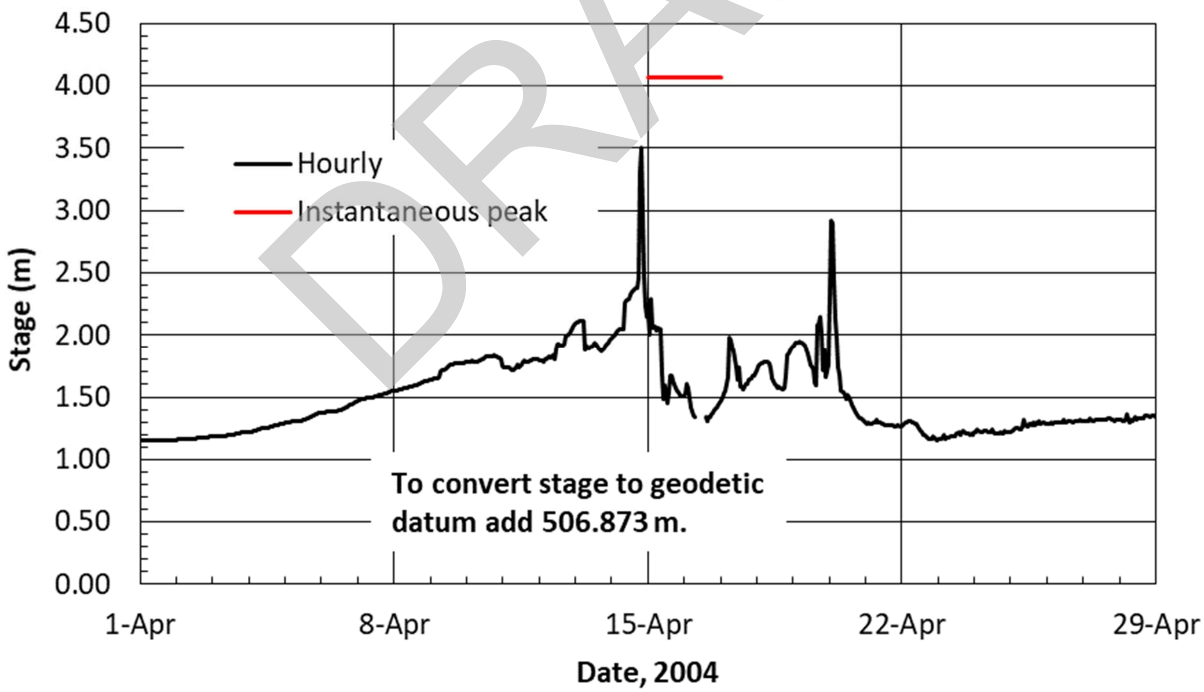

Figure 3. Typical water levels during the passage of surges *Jams form upstream and then release, followed by a water level rise as the surge passes by the gauge. In this case, a large surge occurred on April 14 due to the upstream release of a large jam, followed by smaller surges on subsequent days.*

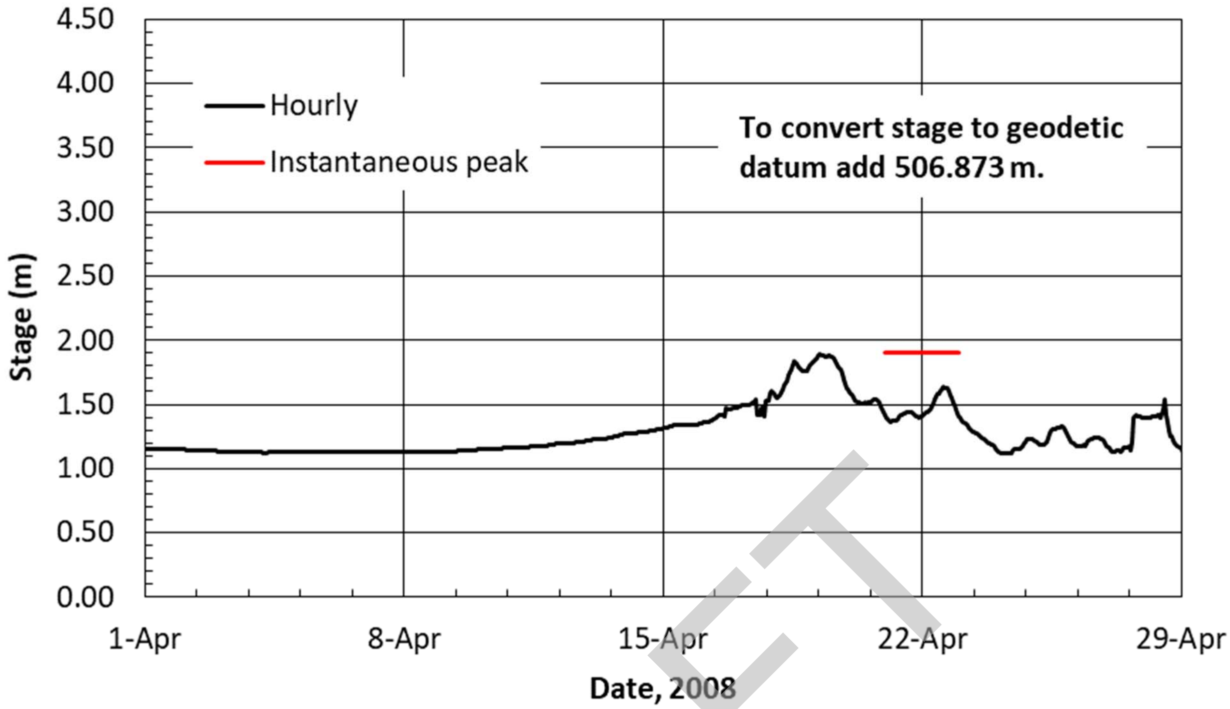

Figure 4. Typical water levels during the formation of an ice jam. *A relatively small ice jam occurred on April 17-18. Water levels increased by about 0.9 m between the pre-breakup period and the formation of the jam. The plot indicates that the discharge was very low during this event given the relatively small increase in river stage.*

3.2 Pre-breakup Ice Conditions

WSC carries out winter discharge measurements to assist in the interpretation of the relationship between ice-affected water levels and discharge, when the relationship between the flow and the water level is affected by both the thickness of the ice cover and the additional flow resistance provided by the presence of the ice – both of which change throughout the winter. Measurements of ice thickness, under-ice water depths, and flow velocities typically are made monthly from December to April at a preferred location near the gauge, which typically does not change from year to year unless extenuating circumstances develop (i.e., the existence of an open lead or difficulty in accessing the ice cover at the preferred location). The last winter measurement – typically made in late March – provides the best indication of the relationship between the measured water level and the discharge that would prevail during the pre-breakup period. An indication of the ice thickness variation throughout the winter is shown in Table 2. The data are based on winter measurements carried out between January 1998 and March 2017. On average, the reported submerged ice thickness varies from 0.36 m in December to 0.57 m in March.

Table 2. Summary of ice thickness data on the Athabasca River at Athabasca, 1998-2017

Month	Number of Measurements	Mean Submerged Ice Thickness (m)	Standard Deviation (m)
December	7	0.36	0.12
January	19	0.47	0.059
February	22	0.54	0.11
March	22	0.57	0.081

The effects of a solid ice cover on the winter rating curve are shown on Figure 5, with the water levels differentiated by month to represent increasing ice thickness between December and March. There is considerable scatter in the data due to the highly variable ice conditions. The March rating curve is of most interest. Aside from the two outliers, the data are in a relatively tight band along the upper bounds of the plot, indicating that the combination of ice thickness and its roughness found in late winter minimizes the conveyance capacity in the reach. For interest (see later discussion), the March water levels can be reasonably well simulated in the reach-averaged uniform flow hydraulic analysis using the average submerged ice thickness of 0.57 m shown in Table 2 and a composite Manning roughness of 0.039. The WSC open water low-flow rating curve is also shown on Figure 5. It suggests that a solid ice cover will increase water levels by between 0.5 and 1.0 m above open water levels. The WSC open water rating curve is based on a downward extrapolation from the lowest recently measured open water discharge of 271 m^3/s, which was made at a water level of 507.97 m on October 13, 1998.

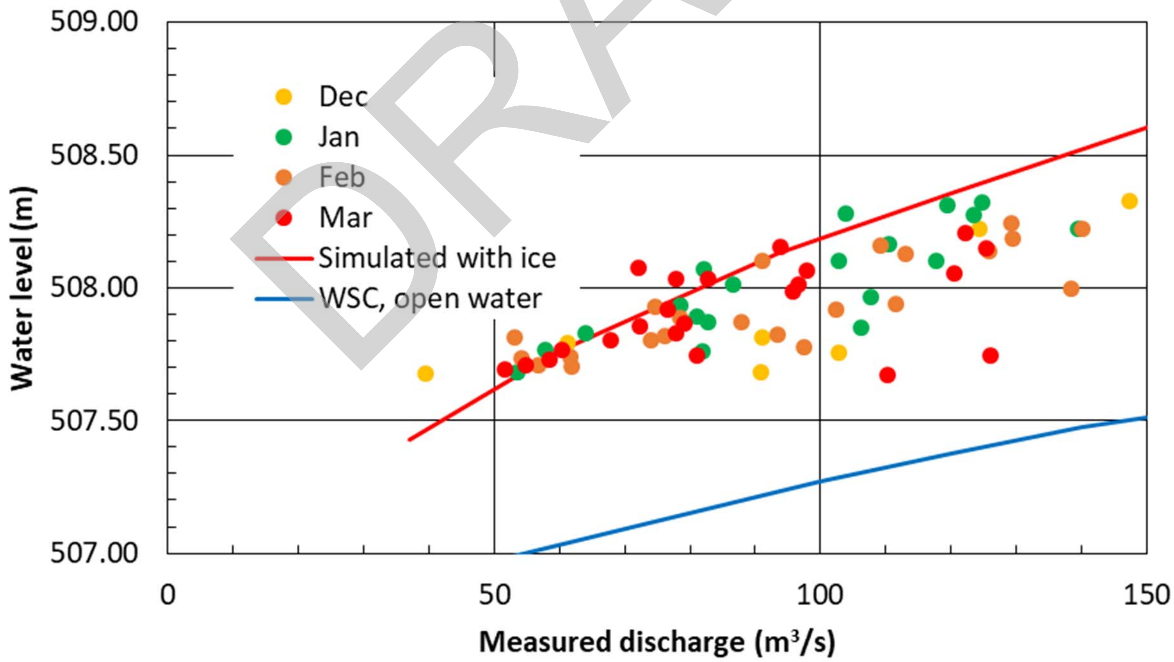

Figure 5. Winter discharge measurements stratified by month. *The simulated ice-related rating curve is based on a late-winter submerged ice thickness of 0.57 m and reflects a composite Manning roughness of 0.039. The two outliers in March, which fall below the bulk of the March data, were measured at a location not typically used during the winter measurement program.*

4 REACH-AVERAGE HYDRAULIC CHARACTERISTICS

The surveys carried out in the summer of 2019 provide a basis to examine the reach-average hydraulic characteristics that provide a framework for examining the relationships between flows and water levels for a variety of open water and ice conditions germane to the analysis of ice-related breakup levels. The thalweg, surveyed water level, and spill level profiles for the entire reach are shown on Figure 6, along with salient place names and a variety of measured open and ice-related water levels.

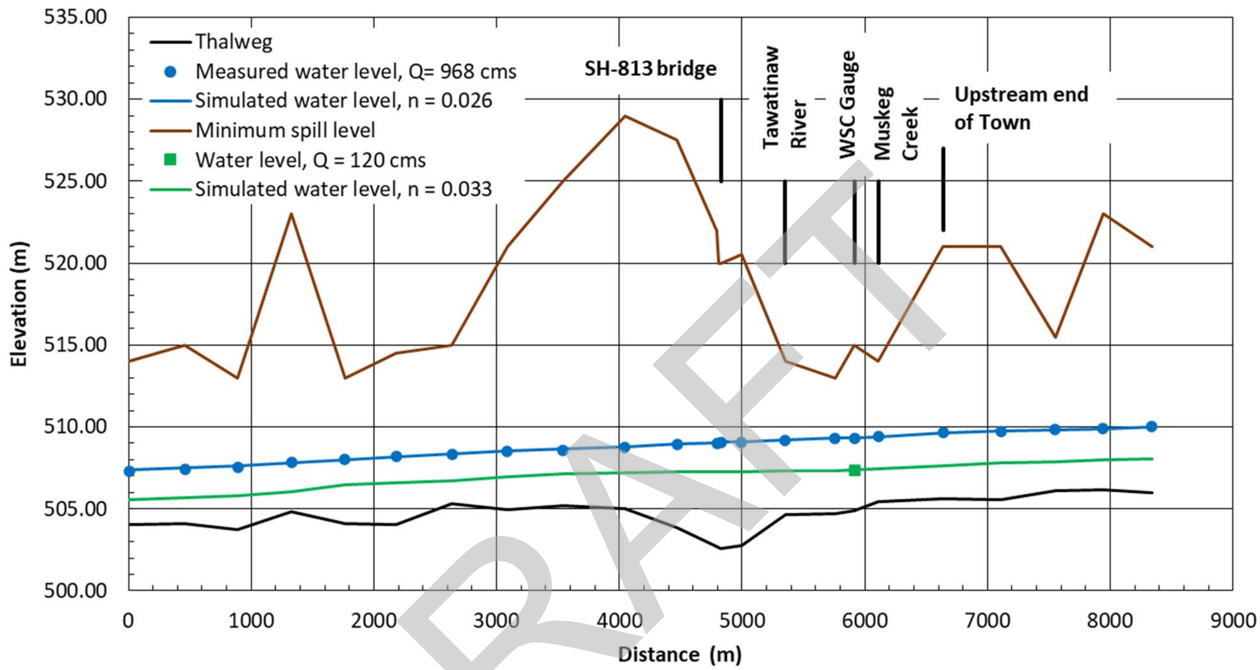

Figure 6. Open water profiles along the study reach. *The spill level defines the degree of horizontal constriction provided by the channel. High spill levels ensure that flow is not lost across the floodplain during an ice jam event, thereby preventing floodplains at low elevations from limiting ice jam levels throughout the reach.*

The open water surface profile was measured at a discharge of 968 m^3/s. The reach-average flow depth during the survey was about 2.7 m, which is substantially greater than the bed-affected flow depths that would be expected during an ice jam event. This precludes the use of that flow scenario to determine the bed roughness that would be applicable during an ice jam event. Furthermore, WSC has not made any open water flow measurements at such low water depths so it is difficult to find an analogue for determining the representative bed roughness values that would apply to low bed-affected flow depths under an ice jam.

However, the WSC open water low-flow rating curve reproduced on Figure 5 provides estimates of discharges that would occur at low water depths at the gauge. Choosing a nominal low discharge of 120 m^3/s, and a corresponding water level of 507.37 m from the open water rating curve on Figure 5, allows for an estimate of a reach-average Manning bed roughness that would apply to the bed-affected flow depth under an ice jam. The HEC-RAS simulation (Figure 6) indicates that a Manning bed roughness of 0.033 would appropriately define the resistance provided by the bed during an ice jam event.

Cross sections that are representative of the channel and overbank characteristics within the Town, in the vicinity of the WSC gauge, are shown on Figure 7. There has been little change in the shape of the cross section since 1914 in terms of bed levels and bank lines. The spill level, or top of bank, is at an elevation of about 515.0 m, and the floodplain slopes up at a mild grade of about 35H:1V. On average, the top width and the mean depth of the channel at its spill level would be about 350 m and 6.8 m, respectively.

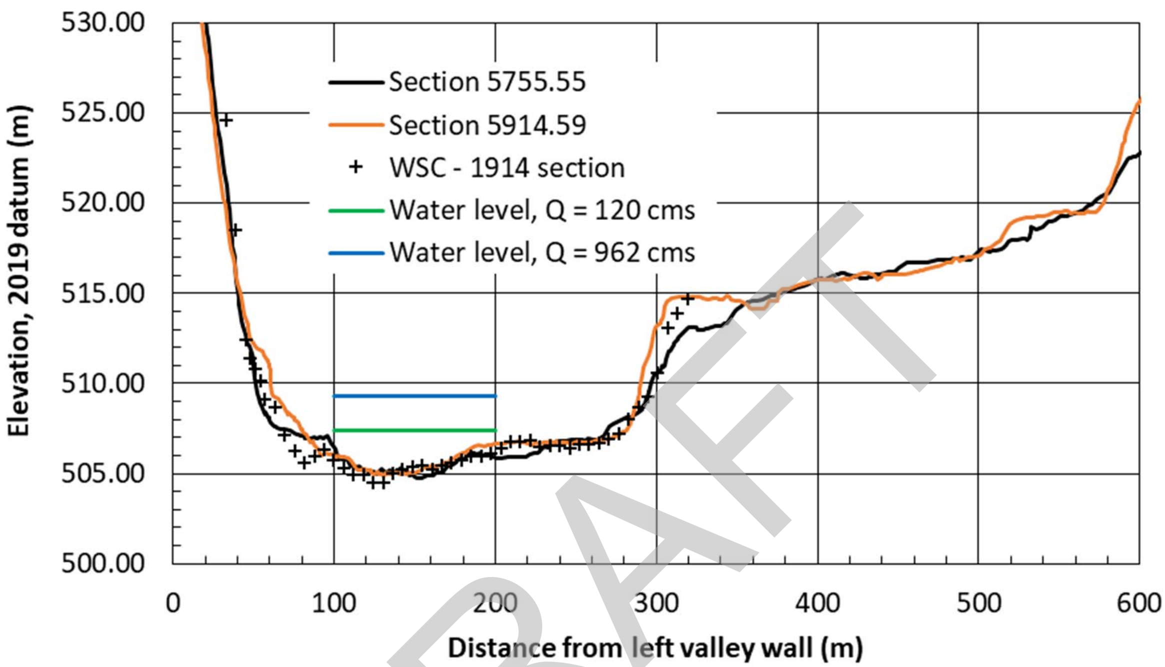

Figure 7. Typical cross sections at the WSC gauge at the Town.

A more detailed profile of the study reach within the Town is shown on Figure 8. That figure provides context to the high ice-related water levels observed in the past. The spill level, which defines the degree of horizontal constriction provide by the channel ranges in elevation from 513.0 to 515.0 m. The 2018 event referenced in Table 1 was just below the minimum spill level at elevation 512.6 m, and the best estimate of the 1904 event (see Appendix I) appears to be about 4.5 m above the minimum spill level. For interest, the 1912 WSC benchmark elevations, which provide an indication of the flood levels that would be expected and tolerated by WSC, are also shown relative to the adopted spill levels. The benchmarks are located just above the highest spill level.

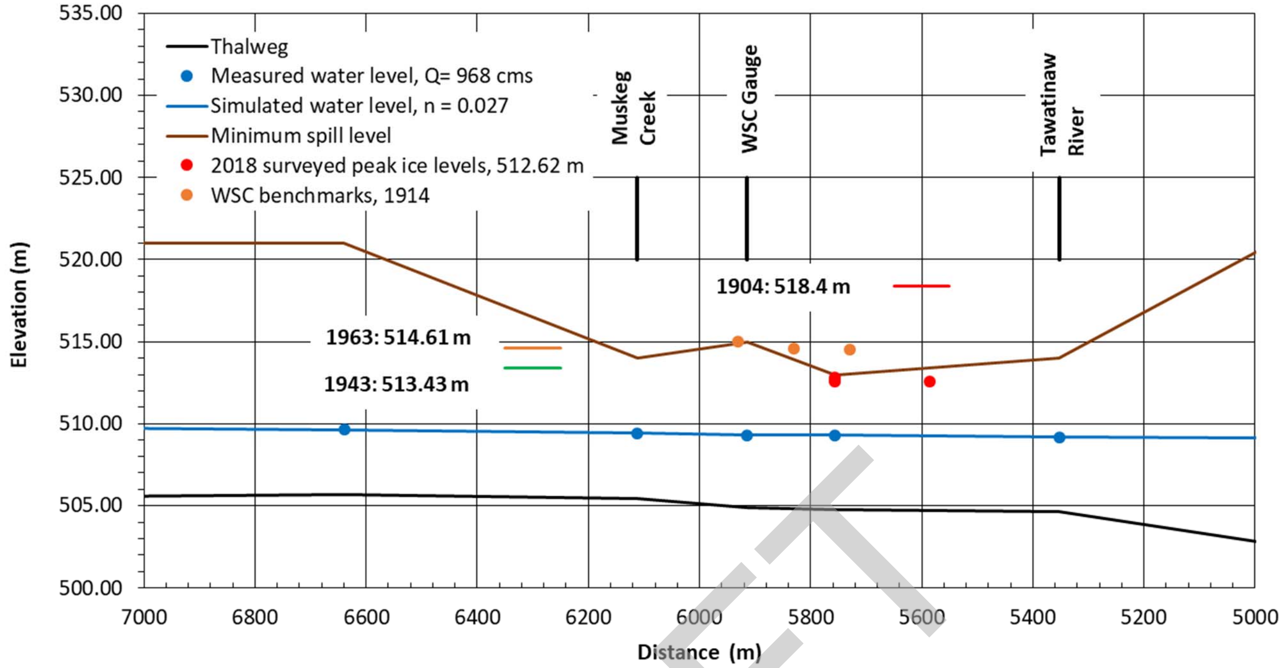

Figure 8. Detailed channel profile within the Town. *Most of the historical high ice-related events, with the exception of 1904, are approximately at the adopted spill level.*

5 ICE JAM CHARACTERISTICS

An assessment of the reach-average ice conditions, based on reach-average channel characteristics and uniform flow analysis of both open water and ice-related water levels, provides context to the measured ice-related water levels during breakup at the WSC gauge (Figure 9). This analysis helps inform the selection of reasonable ice jam parameters for the subsequent HEC-RAS modelling. The figure contains the following information:

1) Measured water levels on the first day of open water plotted against the reported discharge on that day. The consistency of the plotted data indicates no systematic shifts in the rating curve and that the reported discharges are based on a rating curve that has not changed significantly over time.
2) Simulated open water levels in HEC-RAS for two Manning bed roughness coefficients: 0.033 and 0.026. The former represents the open water levels at low flows when flow depths also would be low – something akin to the bed-affected flow depths that would be expected during an ice jam. The latter represents the Manning bed roughness determined by calibrating to the 2019 surveyed water surface profile. It appears that the calibrated value remains valid for a uniform flow analysis up to a discharge of at least 2500 m^3/s. A Manning bed roughness value of 0.033 is used herein for both uniform and gradually varied ice jam simulations.
3) A late winter rating curve. This curve represents the water level with a solid ice cover and is an extrapolation of the ice-related rating curve shown in Figure 5. The curve is based on a

late-winter ice thickness of 0.62 m and Manning bed and ice roughness values of 0.033 and 0.045, respectively, which result in a composite roughness under a solid ice cover of 0.039. This ice roughness appears to be quite high, relative to experience elsewhere, and likely reflects influences of water level controls that cannot be quantified in a uniform flow model.

4) Measured instantaneous peak ice jam levels reported in Table 1, plotted against two alternative discharges: (i) the mean daily interpolated discharge on the day that the instantaneous peak occurred; and (ii) the mean daily discharge on the day of first open water. Regardless of which discharge is adopted, the considerable scatter in the data reflects both a highly variable breakup process and errors in assigning a discharge to the breakup event. The lower bound of this data would represent a thermal breakup that may be defined by the late-winter rating curve. The upper bound is represented by the equilibrium ice jam rating curve. The data points that fall in the middle reflect surge events that fail to achieve the equilibrium ice jam level at the adopted discharge. However, the inherently poor representation of the discharge during the breakup period contributes to the scatter in the data and obscures its differentiation into the three ice states. Nevertheless, regardless of which definition is adopted, the location of the upper envelope curve would not change significantly.

5) The theoretical equilibrium ice jam rating curve that accounts for the contributions of the combined hydraulic resistance of the bed and jam underside, and the equilibrium ice jam thickness, to the overall water level. Theoretically, for any ambient discharge at breakup, the equilibrium ice jam condition should produce the highest water level. Therefore, the upper envelope of the breakup data points should be represented by the equilibrium ice jam rating curve. The shape of the curve reflects a dimensionless internal ice strength coefficient of 1.2 and Manning bed and ice underside roughness values of 0.033 and 0.060, respectively. The ice jam equations that underpin this analysis, and the methodology used to derive the ice jam rating curve, are summarized in Appendix II.

The representation of the equilibrium ice jam rating curve is the most significant feature on Figure 9. While it is possible to derive a frequency curve of annual peak breakup levels at the WSC gauge (see later discussion), extrapolation from that location requires the computation of gradually varied ice jam profiles to capture fully the influences of changes in channel geometry throughout the flood hazard domain. It is likely that the severe events reflective of long return periods (50 or 100 years) would be represented by fully developed equilibrium ice jams that would extend throughout the entire reach. These jams likely would have the same hydraulic characteristics (i.e., similar bed and under-ice Manning roughness coefficients and the same dimensionless internal friction coefficient). The difference in water levels for the range of return-period ice jam events would be due solely to differences in the carrier discharge.

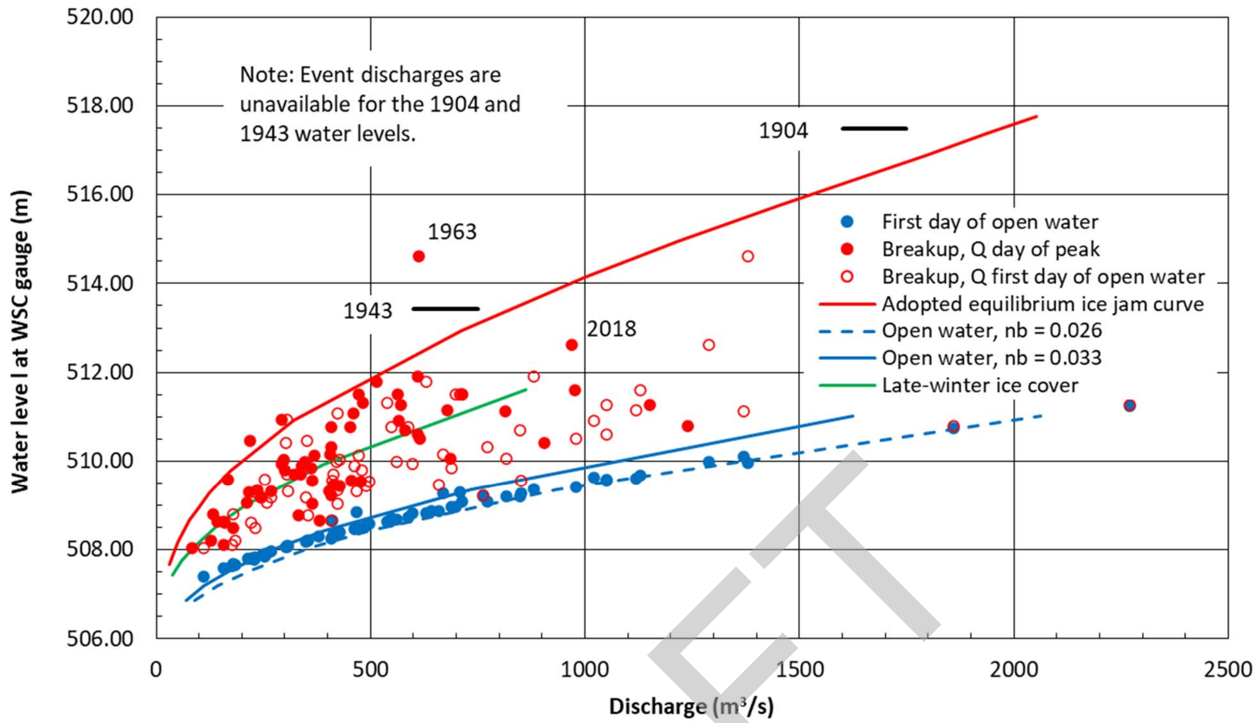

Figure 9 Comparison of open water and ice-related rating curves.

The curve representing the adopted upper bound of the data on Figure 9 corresponds to the equilibrium ice jam rating curve. The location and shape of the curve reconciles with accepted ice jam parameters in the literature – a dimensionless coefficient of internal friction of 1.2, a bed roughness of 0.033, and a bottom-of-jam roughness of 0.060. The thickness of the ice jams in their equilibrium state varies between 1.9 m at a discharge of 500 m^3/s to 2.3 m at a discharge of 1000 m^3/s. The carrier discharge associated with the 1904 event is about 2000 m^3/s. The discharge attributed to the 1963 event by WSC appears to be inaccurate (i.e., too low) given its relationship to the equilibrium ice jam rating curve.

6 FREQUENCY ANALYSIS OF PEAK ICE-RELATED WATER LEVELS

6.1 Ice-Related Water Level Record

The record of breakup water levels in Table 1 is composed of the 90 known anecdotal, daily, and instantaneous ice-related peak water levels. The record is not homogeneous, but contains one anecdotal historical instantaneous peak, 65 systematically measured instantaneous peaks at or close to the WSC gauge, and 24 systematically measured daily peaks at the WSC gauge. The instantaneous peaks are most relevant, but to include only them creates an artificially short record, which will affect the shape of the frequency curve at return periods greater than about 20 years. A choice needs to be made in the frequency analysis to balance the effects of the shorter (and higher) instantaneous record against the longer (but lower) combined record.

Given the duration of ice jam events, and the fact that manual water level measurements typically were carried out twice daily, it is unlikely that a non-reported instantaneous peak would have been significantly greater than the corresponding daily peak in those years when manual measurements were being made. Also, if a very severe event had occurred, it would have been noted by the observer – at least anecdotally. Given that there is a lack of anecdotal evidence of extremely high breakup levels during the period when manual observations were being made, it is unlikely that instantaneous peaks of any consequence would have gone unreported in the systematic record. Furthermore, these unrecorded instantaneous peaks would contribute largely to defining the lower end (short return period) of the water level spectrum. Therefore, since the statistical characteristics of the severe breakup events are of most interest, it is deemed appropriate to combine the two data sets to create one long record.

If the two records are treated as being distinct, such that there is no differentiation between the significance of any extraordinary levels in the historical period and those in the systematic record, all data points carry the same weight. Thus, there are essentially two records that could be adopted for analysis: the 66-year long instantaneous record and the 90-year long combined record. The statistical characteristics of these two records are shown on Figure 10.

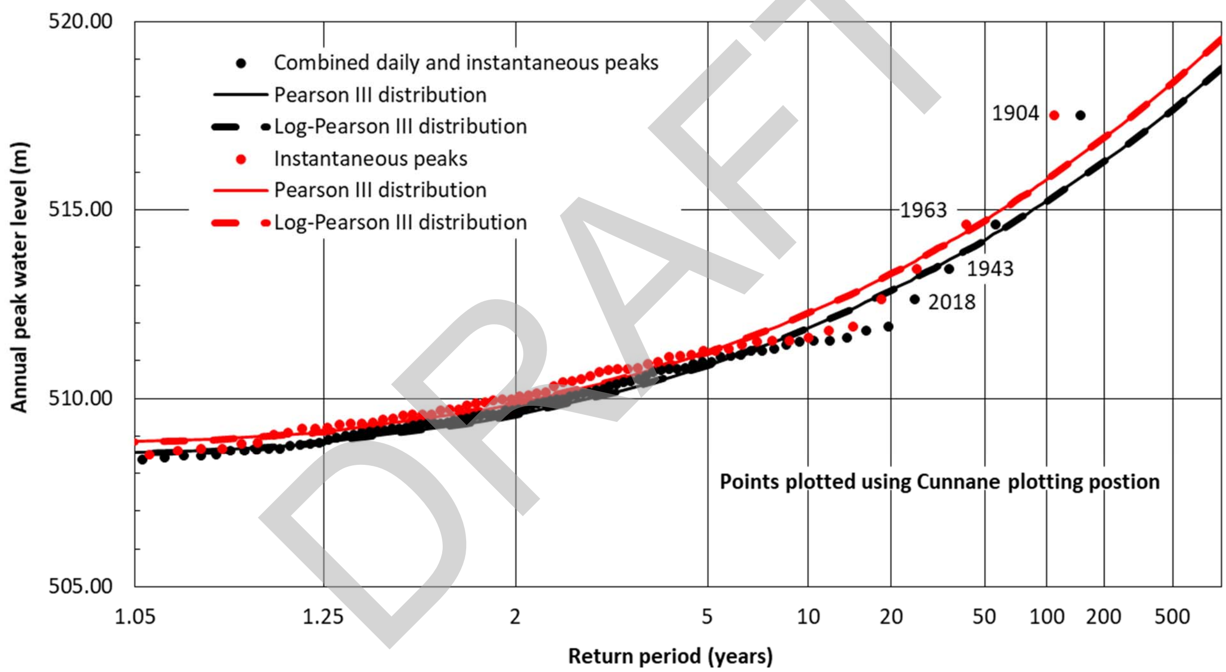

Figure 10 **Comparison of the frequency distributions of the peak instantaneous and combined peak water levels in the record.** *The mean, standard deviation, and skew coefficient for the instantaneous peaks are 510.30 m, 1.53 m, and 1.99, respectively. For the combined peaks, the mean, standard deviation, and skew coefficient are 509.98 m, 1.47 m, and 1.92, respectively. There is no difference between the Pearson III and log-Pearson III distributions for each data set.*

As expected, the curves representing the instantaneous peaks on Figure 10 plot slightly higher than those of the combined peaks, because many of the lower data points in the daily record are excluded from the record of instantaneous peaks. Also, because of the shorter record length associated with the instantaneous peaks, that record exhibits a greater skew coefficient than that of the combined record.

This results in a more conservative estimates of water levels associated with the long return periods, even though the highest four or five data points are the same in both records.

Although the short record of instantaneous peaks results in a more conservative analytical outcome (i.e., results in higher water levels) than that provided by the combined record, it is believed that the longer combined record, which contains more of the low events, is a better representation of the true record and, therefore, provides a better estimate of water levels for the longer return periods.

The results of the analysis shown on Figure 10 also provide a comparison between the Pearson III and log-Pearson III distributions. Since there essentially is no difference between the two curves, the more flexible Pearson III distribution is adopted herein.

6.2 Bulletin 17B Procedure

Because of the existence of the 1904 event, the combined record needs to be treated as a censored data set. The strategy employed in the Bulletin 17B approach is used herein to carry out the frequency analysis of censored combined record, using the Pearson III distribution to quantify the statistical characteristics of the record. The procedure is described in Appendix 6 of Bulletin 17B (USGS, 1982, 2017) and in the user's manual for the PeakFQ computer program (Flynn et al., 2006). Stedinger and Cohn (1987) and Cohn et al. (1997) also provide a succinct description of the statistical theory that underpins the application of the Bulletin 17B procedures.

For the situation herein there is an incomplete record of ice-related water levels and a mix of both systematic and historical measurement (the former representing events of all magnitude and the latter only notably high events). The total record of length, N, is composed of two periods as follows:

1) A historical record of length, N_H, in which only high events above the adopted threshold level T are measured.
2) A systematic record of length, N_S, where all events are measured. Missing years in the systematic record would be treated as belonging to the historical period.

Therefore, the entire record consists of four types of data that include:

1) $N_H^>$ documented events above the threshold level in the historical part of the record,
2) $N_H^<$ undocumented events in the historical part of the record, which are assumed not to be notable and therefore fall below the threshold level,
3) $N_S^>$ documented events above the threshold level in the systematic part of the record, and
4) $N_S^<$ documented events below the threshold level in the systematic part of the record.

The challenge is to quantify the statistical characteristics of the unmeasured events in the historical period, which are then combined with the measured data in both the historical and systematic record periods by applying a weighting factor, $W = [1+ (N_H^< / N_S^<)]$, to the measured points in the systematic record that are below the adopted threshold level. The equations for determining the weighting factor and the formulae for determining the plotting positions are summarized in Table 3. The weighted mean, standard deviation, and skew are calculated following the procedure described by Cohn et al. (1997)

using the "method of moments" approach. The weighting factor varies according to (i) the range of data in the historical and systematic measurement periods and (ii) the choice of the threshold level, since that determines the number of data points in each period.

As the threshold level goes up, $N_H^>$ and $N_H^<$ remain relatively constant, but $N_S^>$ and $N_S^<$ change as events in the systematic record drop out of the above-threshold category and into the below-threshold category. As the threshold level increases, the weighting factor becomes smaller, which ultimately affects the shape of the frequency curve.

After the statistics are defined, any selected reasonable distribution can be adopted to represent the data, although Bulletin 17B suggests either a Pearson III or a log-Pearson III distribution. As per the earlier discussion, the Pearson III distribution was adopted, along with the Cunnane plotting position.

Given the above assumptions, the two main factors that could bias the results of the threshold-based frequency analysis are the length of the historical period and the adopted threshold. A review of the history of Athabasca region (Appendix I) suggest that 1884 would have been the earliest year in which a severe ice-related flood would have been observed – suggesting a record length of 136 years (1884-2019). The length of the historical period in which no systematic record exists at the WSC gauge is 46 years. This is composed of 40 years in the periods 1884 to 1913, 1932, and 1941 (when the gauge was not operating) and 6 years between 1942 and 2019 (when the gauge malfunctioned during breakup). In that period, only one event – the 1904 event – was significant enough to have been noted. The length of the systematic record at the WSC gauge is 89 years. The number of events in that period exceeding a given threshold would depend on the choice of the threshold.

Table 3. Summary of methods to compute the weighting factor applied to the below-threshold points in the systematic record, Bulletin 17B, 1982 and 2006

Parameter	Formulation
Total record length, N	$N_H^> + N_H^< + N_S^> + N_S^<$
Number of recorded points, N_R	$N_H^> + N_S^> + N_S^<$
Number of points below threshold level in systematic record	$N_S^<$
Number of undocumented events below threshold in historical record	$N_H^<$
Number of points above a given threshold in both historical and systematic record, Z	$N_H^< + N_S^<$
Weighting factor, W	$1 + (N_H^< / N_S^<)$
Plotting position, PP, where r is the rank, m is the weighted rank, and a = 0.40 to represent the Cunnane plotting formula	If $r \leq Z$, $m = r$ If $r > Z$, $m = Z + W (r - Z - 0.50) + 0.5$ $PP = (m-a)/(N+1-2a)$

The threshold applied to any period would depend on the nature of the settlement. As described in Appendix I, the potential threshold elevation could range between 514.0 and 514.6 m – the former related to the elevations of the warehouses and storage facilities along the riverbank, and the latter related to the elevation of the railway track. However, as shown in Table 1 and on Figure 8, only four severe breakup events occurred (1904, 1943, 1963, and 2018) that could potentially fall above a reasonable threshold level. From a statistical perspective, a threshold level above elevation 515.0 m would capture only one data point, making any statistical analysis somewhat moot. Choosing a lower threshold elevation that is well below the spill level (e.g., 512.0 m) would make it more likely that some events in the historical period could have exceeded that level but would not have been noted because they were of no concern.

Weighting factors and the factors contributing to its determination are summarized in Table 4 for three potential threshold levels. The statistical parameters (mean, standard deviation, and skew coefficient) of the distributions that resulted from the application of the adopted threshold levels also are summarized in that table.

Table 4. Summary of calculated weighting factors for various threshold levels

Parameter	Parameter Value for Given Threshold Level		
	512.0 m	514.0 m	515.0 m
Length of record, N (years)	136	136	136
Number of documented events above threshold in the historical part of the record, $N_H^>$	1	1	1
Number of undocumented events below the threshold in the historical part of the record, $N_H^<$	46	46	46
Number of documented events above threshold in the systematic part of the record, $N_S^>$	3	1	0
Number of documented events below threshold in the systematic part of the record, $N_S^<$	86	88	89
Weighting factor, W	1.535	1.523	1.517
Weighted mean, M (m)	509.91	509.93	509.95
Weighted standard deviation, S (m)	1.342	1.367	1.396
Weighted skew, G	1.7355	1.6856	1.6911

The effect on the frequency curve of changing the threshold level is shown on Figure 11. There are no significant effects on either the plotting positions or the shape of the frequency curve for the adopted range of threshold levels. Therefore, a threshold level of 514.0 m, which is just at the lower end of the range of spill levels within the Town, was adopted for analytical purposes. It appears that the number of historical measurements that are not part of the long systematic record is insufficient to affect how the historical data is treated. However, adopting the Bulletin 17B approach reduces the water levels at the long return periods relative to those that would be calculated by simply adopting the shorter systematic record at the gauge.

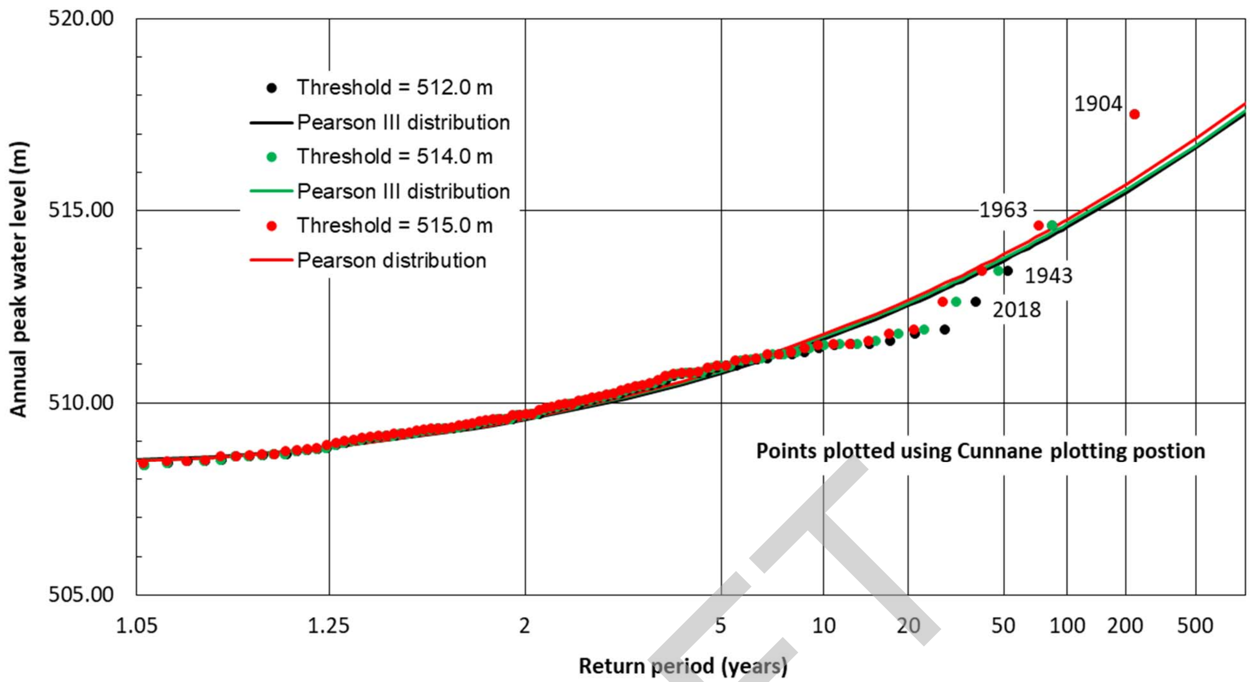

Figure 11. Comparison of frequency curves for a range of threshold elevations. *There are few effects on either the plotting positions or the shape of the frequency curve for the adopted range of threshold elevations.*

Notwithstanding the aforementioned, there could be some speculation about the elevation of the 1904 event. While the case can be made that the peak level was likely 517.5 m, its estimation – both in 1904 and today – could easily be in error by at least ±0.5 m. The effects on the frequency curve of adopting a range between 517.0 and 518.0 m for the 1904 event are shown on Figure 12. Changing the value of the 1904 event between those bounds has little effect on sample average. However, as the flood level of the 1904 event increases, both the standard deviation and the skew coefficient increase. This leads to higher values at the longer return periods. For example, at return periods greater than 100 years, the range in peak water level would be about 0.5 m. There is little difference in water levels at return periods shorter than 50 years

Given that the long return-period floods tend to decrease as more years are added to the analysis, it is likely that the significance of the 1904 historical flood will diminish over time. Furthermore, climate trends that tend to decrease the late-winter ice thickness (thereby reducing the likelihood of developing stable equilibrium ice jams) and decrease snowmelt runoff (thus reducing jam levels should a jam occur) will contribute to less severe breakups and lower ice-related water levels in the future.

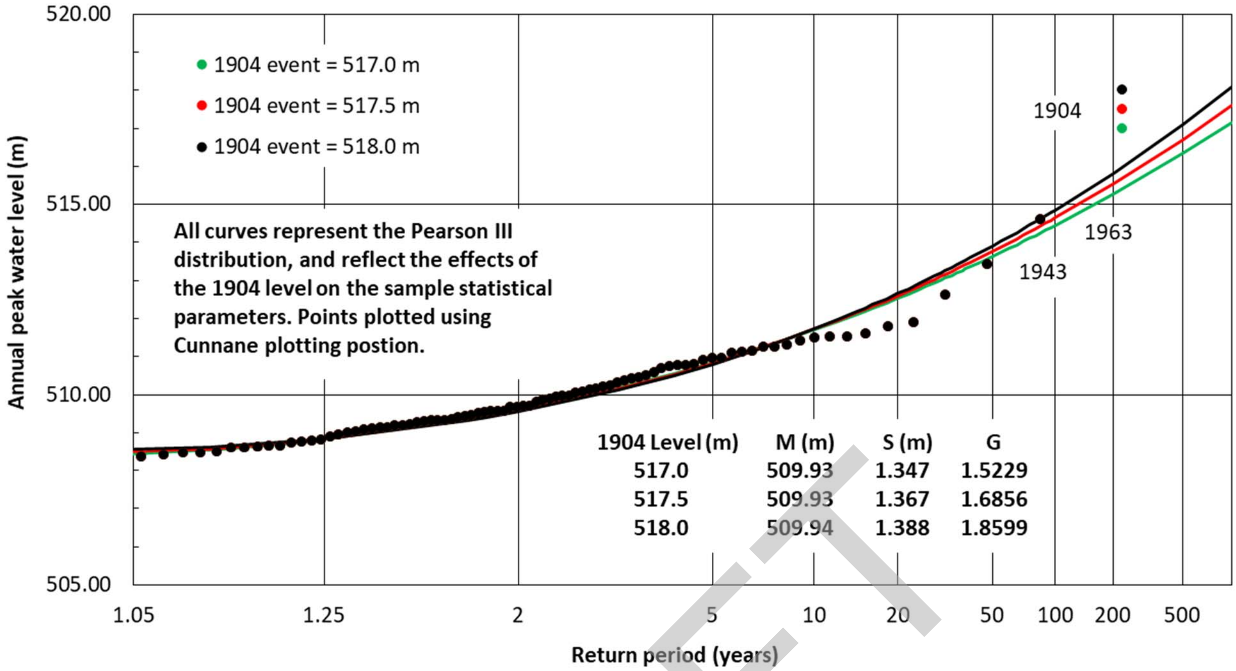

Figure 12. Comparison of frequency curves for a range of possible 1904 breakup levels. *The adopted threshold level is 514.0 m. An elevation of 517.5 m for the 1904 event represents the best estimate based on the assessed quality of the anecdotal evidence. Note that M = sample average, S = sample standard deviation, and G = sample skew coefficient. See Table 4 for other statistical parameters.*

7 CONCLUSIONS

The historically-based ice-related water level frequencies that reflect a 1904 breakup elevation of 517.5 m and represent a range of possible thresholds, as calculated by procedures outlined in Bulletin 17B, are summarized in Table 5. The analysis indicates that the 1904 flood would be between a 500- and 1000-year event. The next two largest floods (1963 and 1943) would have return periods of about 85 and 35 years, respectively. The 2018 event would have a return period of about 20 years.

Table 5. Summary of historically based ice-related water level frequencies at the WSC gauge

Return Period (years)	Annual Probability Being Equaled or Exceeded (percent)	Water Level (m) at WSC Gauge
2	50	509.58
5	20	510.83
10	10	511.72
20	5	512.60
50	2	513.76
100	1	514.64
200	0.5	515.53

8 CLOSURE

This document has been prepared by SG1 Water Consulting Ltd. (SG1) for the sole use and benefit of Golder Associates Ltd. and its client, Alberta Environment and Parks, in regard to the Athabasca Flood Hazard Study on the Athabasca River at Athabasca, Alberta. Information contained herein has been prepared in accordance with generally-accepted engineering practices and represents SG1's best judgement based on information and data provided to or obtained by SG1 during the project. This document is provided "as is" without any guarantee, representation, condition, or warranty of any kind, either express, implied, or statutory. Except as required by law, SG1 assumes no liability or responsibility for the reliance or use of this document by any parties other than those specifically named herein.

We trust that the information contained in this memorandum is sufficient for your present needs. Please contact the undersigned should you have any questions or wish to discuss.

Sincerely,

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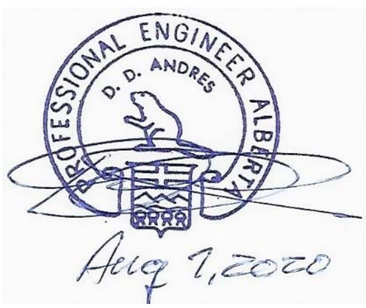
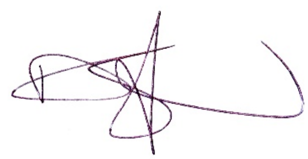


David Andres, M.Sc.CE, P.Eng.
River Ice Engineering Specialist

Reviewed by:



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ENCLOSURE

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APPENDIX I

Assessment of Ice-Related Threshold (Perception) Levels
and the 1904 Ice Jam Flood Event – Historical Context
Athabasca River at Town of Athabasca

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Assessment of Ice-Related Threshold (Perception) Levels and the 1904 Ice Jam Flood Event – Historical Context Athabasca River at Town of Athabasca

1 INTRODUCTION

The objectives of this document are (i) to determine the potential length of the historical record, (ii) to evaluate the elevation of the 1904 breakup event, and (iii) to provide a context to determine potential ice-related threshold or perception levels at the Town of Athabasca[1] (the Town) to inform the ice-related frequency analysis that is being undertaken as part of the Athabasca Flood Hazard Study. The 1904 breakup event, which produced the highest ice-related water levels on record and is the only event documented in the historical record, was described anecdotally, but the peak water levels were not recorded. Threshold levels are defined as salient elevations that, if exceeded during a flood, would cause residents to take notice and the occurrence of the event would have been recorded. These data can contribute to the development of a historical record of ice jam events to enable a frequency analysis of those levels. To provide context for the anecdotal ice-related flood evidence, a brief historical account of activities at Athabasca is provided.

Much of the information provided herein is based on:

1) "Athabasca Landing: An Illustrated History" that was published 1986 by the Athabasca Historical Society (AHS, 1986),
2) Information provided by archivists working at the Hudson's Bay Company Archives in Winnipeg,
3) Photographs and newspaper clippings gleaned from the archives of the Athabasca Historical Society located in Athabasca, and
4) Photographs held by the Provincial Archives of Alberta in Edmonton.

Acknowledgements of individual pieces of information will be made where possible, but the reader is directed to the above noted publication for primary sources for much of the background history of the Athabasca region. Margaret Anderson (Archivist at the Athabasca Archives) and Lisa Friesen (Archivist at the Hudson's Bay Company Archives, Archives of Manitoba) provided valuable assistance in scouring salient archival records, and their time and contributions are gratefully acknowledged.

[1] The Town of Athabasca originally was called Athabasca Landing. Its original name was changed to its present name in 1913.

2 HISTORY

The first Hudson's Bay Company (HBC) warehouse at Athabasca (Figure I-1) was established in 1877. Prior to that time, the area was inhabited for brief periods by nomadic First Nation peoples, including the Blackfoot, who were based in the Saskatchewan River valley, the Beaver, who occupied the Peace River and Beaver River areas, and the Chipewyan, who lived in the area northeast of Athabasca. The portion of the Athabasca River in the vicinity of Athabasca was not an active trade area during the early days of the fur trade. Most of the earlier traders (late 1700s and early 1800s) bypassed the area, preferring to use the North Saskatchewan River and the Peace River in their travels west and/or accessing the Peace River country and the Mackenzie River area via the Churchill, Clearwater, lower Athabasca, and Slave River systems. Nevertheless, there is evidence that David Thompson navigated the river in 1799 on his way to what is now Lake Athabasca after travelling from Lac la Biche overland to the North Saskatchewan River, then across to the Athabasca River via what is now the Pembina River before continuing his way northward on the Athabasca River. Also, during this time, the Northwest Company had established a route from Lac la Biche to Lesser Slave Lake via the La Biche River, the Athabasca River, and the Lesser Slave River.

During this period the plains Cree were being pushed further west by the increasing numbers of Metis hunters and the westward expansion of the Red River settlement. This westerly migration was resisted by the Beaver, the Chipewyans, and the Blackfoot; however, by the middle part of the 1800s, the area in the vicinity of Athabasca had been overtaken by the Cree, and the language changed from Athapaskan to Algonquian. To quote AHS, 1986:

> "Neglected by the HBC during the forty-year period [1820-1860] of George Simpson's administration, the central section of the Athabasca River valley remained in its natural state, uncultivated and unsettled except for the Indian encampment at Calling Lake, until the 1870s."

Evidently, because the area was not rich in furs and other older well-established routes north from Fort Edmonton had been established, there was little desire to open up new posts in the area after the amalgamation of the two fur trading companies. Furthermore, George Simpson, the HBC governor, decided to access the area northwest of Lesser Slave Lake via an overland route from Fort Edmonton to Fort Assiniboine, then downstream on the Athabasca River, upstream along the Lesser Slave River, and then along Lesser Slave Lake. This strategy delayed settlement in the vicinity of the Town.

The original "post": Hudson's Bay Company storage shed, Athabasca Landing (built 1877). R.C.M.P. Archives: Photograph Collection.

Figure I-1. Original HBC warehouse at Athabasca, precise location is unknown, after AHS, 1986.

By the early 1870s, experience had proven that the Fort Edmonton-Fort Assiniboine route became more or less impassable because of "mud and swollen river crossings." An alternative was required and in 1874, HBC commissioned a survey of the 100-mile long overland route between Fort Edmonton and "the Elbow" of the Athabasca River. The route, now known as the Athabasca Trail, was completed in 1877 and, from then on, it was the preferred route from Fort Edmonton to the north and northwest.

The terminus of the trail became known as Athabasca Landing, and despite all the activity at the site, it was mainly a staging area for the transport of goods northward. There was no continuous habitation until 1884, when navigation problems at Grand Rapids were overcome, the warehouse facilities on the river were converted to an actual trading post, and Leslie Wood took up permanent residence on behalf of HBC. Within the settlement itself, the Athabasca Trail became the main north-south thoroughfare, and additional facilities were constructed up from the river on both sides of the trail (Figure I-2). By 1888, sternwheelers and scows were being constructed at the settlement to facilitate trade and travel along the Athabasca River. The North West Mounted Police established a post in 1892 to provide law and order, just in advance of the influx of Klondikers.

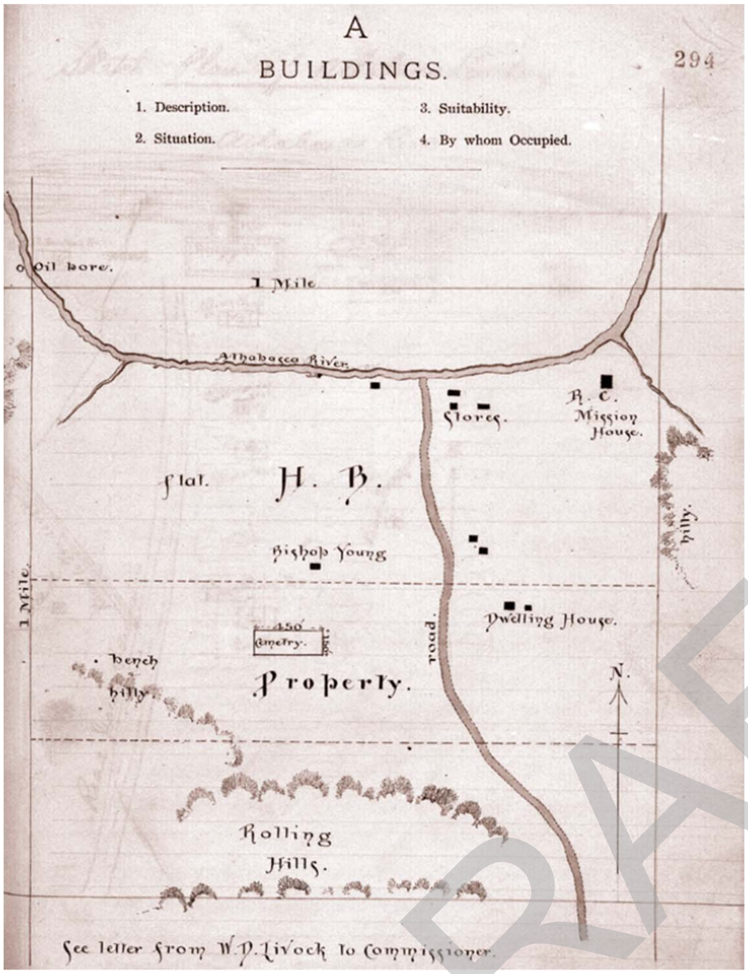

Figure I-2. HBC Buildings at Athabasca, 1894. Hudson's Bay Company sketch-map. *Archives of Manitoba Hudson's Bay Company Archives, D. 25i19 fa. 294, after AHS, 1986.*

Gold was discovered in 1896 near Dawson City, Yukon, precipitating the Klondike Gold Rush. An all-Canadian route to the Klondike was established from Edmonton that passed through the settlement, which became the gateway to the Athabasca River. In 1897, a large number of gold-seekers arrived in preparation to go north. The local population grew substantially to accommodate the thousands of gold-seekers, and this stimulated a significant amount of construction to expand local businesses. In 1898, the area was surveyed, the Athabasca Trail was renamed Strathcona Street (now 50th Street), lots were established on both sides of the street, and the location of additional streets were defined by surveys. The east-west trail along the river front became Lichfield Street (now 50th Avenue), and the intersection of the two streets more or less became the centre of development (Figure I-3). Commercial development along Strathcona Street in 1900 is shown Figure I-4. The HBC store (Lot 1W on Figure I-3) is prominent in the photo. It is flanked by the Imperial Bank of Canada building, which in turn is situated south of the two-story telegraph office.

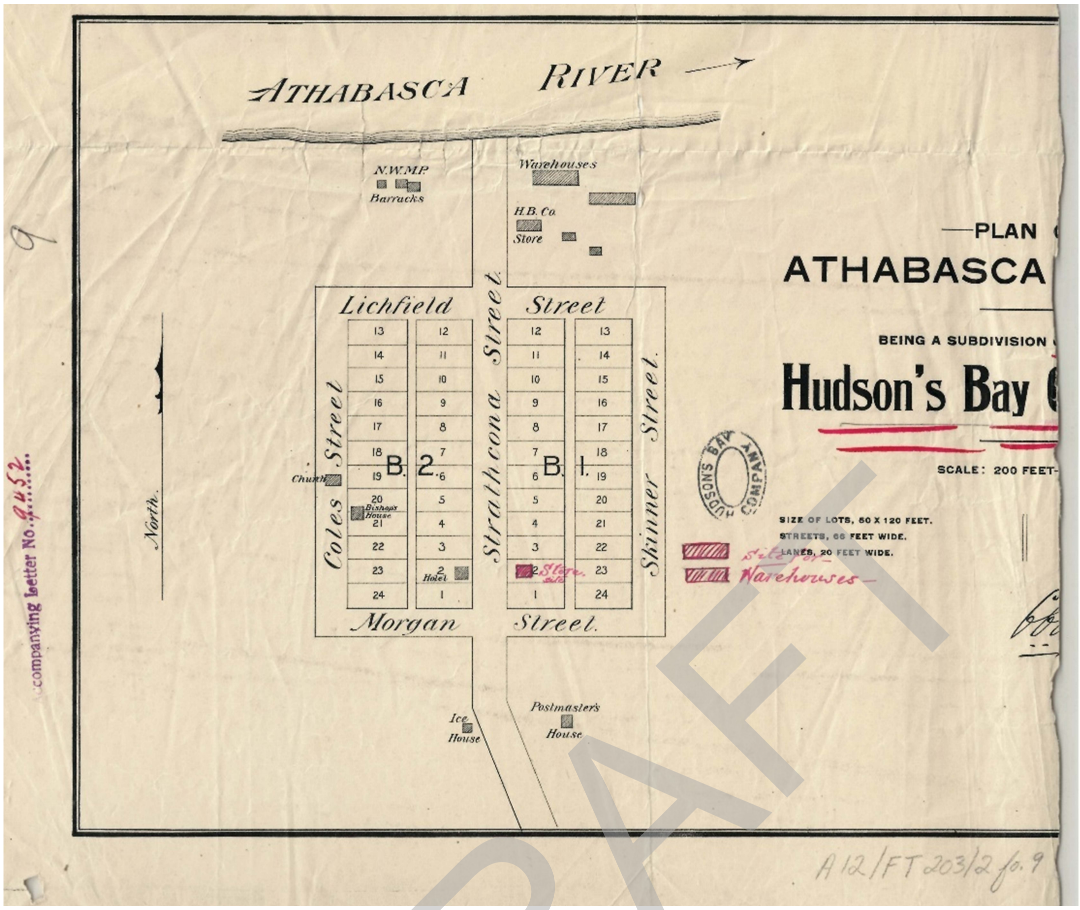

Figure I-3. Map showing street layout and HBC structures in 1898. *Figure provided courtesy of the HBC Archives. The annotations in red refer to plans to relocate buildings following the 1904 ice jam flood.*

Figure I-4. Looking north along Strathcona Street (50th Street) in 1900. *The main HBC store, which was constructed prior to 1900, sits on the corner. It burned down in the 1912 fire.*

After the end of Klondike gold fever, activities at the settlement slowed considerably, but the population continued to grow as the settlement established itself as a transportation hub. Despite the severe ice jam in 1904 that destroyed much of the commerce along the river (Figures I-5 to I-8), the population grew from 250 in 1905 to 450 in 1911, and then to 2000 by the end of 1913. The river front area, as it appeared in 1905, is shown on Figure I-9. It is not clear if the date of the photo is correct, since the area appears to be rather undeveloped – perhaps as a result of the 1904 flood. There is no clear evidence of the existence of the structure shown on Figures I-6 and I-7, and it is difficult to identify any salient structures at the upper end of Strathcona Street that might correspond to the buildings on Figure I-4.

Figure I-5. HBC warehouse structure during 1904 ice jam flood. *The location of this structure is unknown, but evidently it was dismantled after the flood. See Figure I-6 for its pre-flood condition.*

Figure I-6. HBC warehouse. *Ostensibly, this photo was taken before 1904.*

Figure I-7. Unknown structure encased in ice – ostensibly during the 1904 ice jam event. *The location of this structure is unknown.*

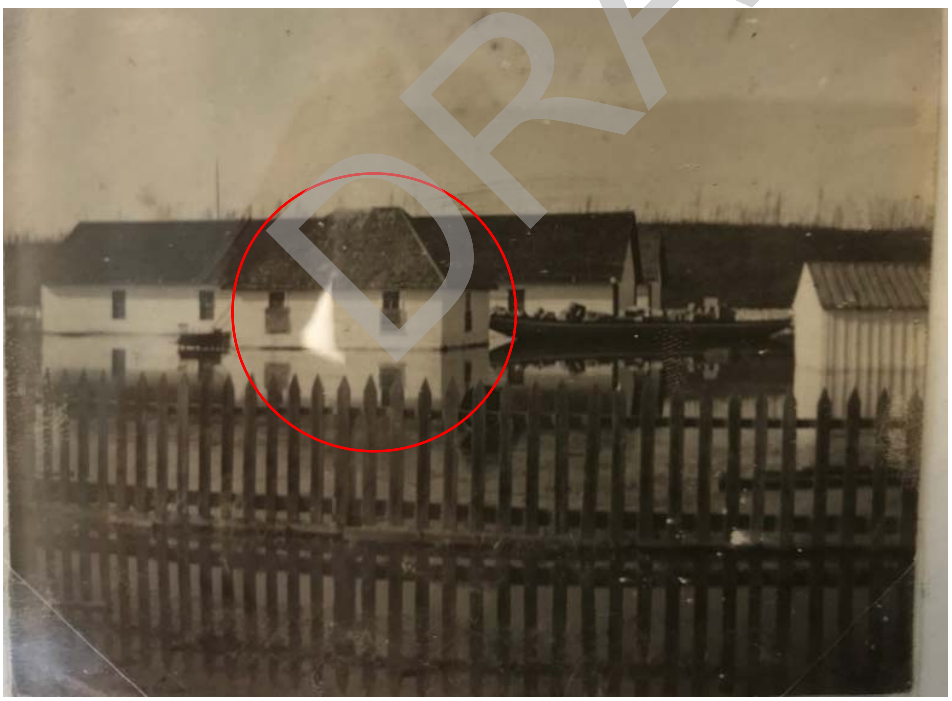

Figure I-8. Warehouses inundated by flood – ostensibly during the 1904 ice jam event. *The building with the hip roof in the centre of the photo looks to be the original HBC warehouse shown on Figure I-1.*

Figure I-9. Athabasca Landing waterfront, 1905, looking south along Strathcona Street. *It appears that the Grand Union Hotel is under construction. Note the largely undeveloped area between the river and Strathcona Street and the rather mildly sloping floodplain.*

A commercial ferry went into operation in 1906 and the Athabasca was incorporated into a town in 1911, and the name officially changed to Town of Athabasca in 1913. Rail service arrived in May 1912 (Figures I-10 and I-11), and then disaster struck – a large fire swept through the town in 1913. The fire destroyed most of the business core, which was mostly rebuilt after the fire. The Grand Union Hotel, which stood on the southeast corner of Strathcona and Lichfield Streets was one of the notable reconstructions (Figures I-12 and I-13). Also notable in 1913 was the installation of the WSC hydrometric station. Details of the station layout are shown on Figures I-14 and I-15.

Figure I-10. Lichfield Street before 1913, looking east with the NAR right-of-way on its left. *Note the elevation difference between the street and the railway right-of-way. The original Grand Union Hotel is in the background of the photo. The hip-roofed structure located left of centre is likely the original HBC warehouse.*

Figure I-11. Lichfield Street in the early 1920s, looking east with the NAR right-of-way on its left. *The railway station is on the left, with the tracks located behind the station. The new post-fire Grand Union Hotel is in the background of the photo.*

Figure I-12. The Grand Union Hotel before the fire of 1913. *The original hotel was a two-storey structure. The third storey was added sometime prior to 1913. The front of the hotel is facing Strathcona Street.*

Figure I-13. Albertans preparing to leave for World War I. *The rebuilt Grand Union Hotel is in the background. Note the bottom of the windowsills along Lichfield Street. It appears that street and sidewalk levels have not changed significantly since this photo was taken.*

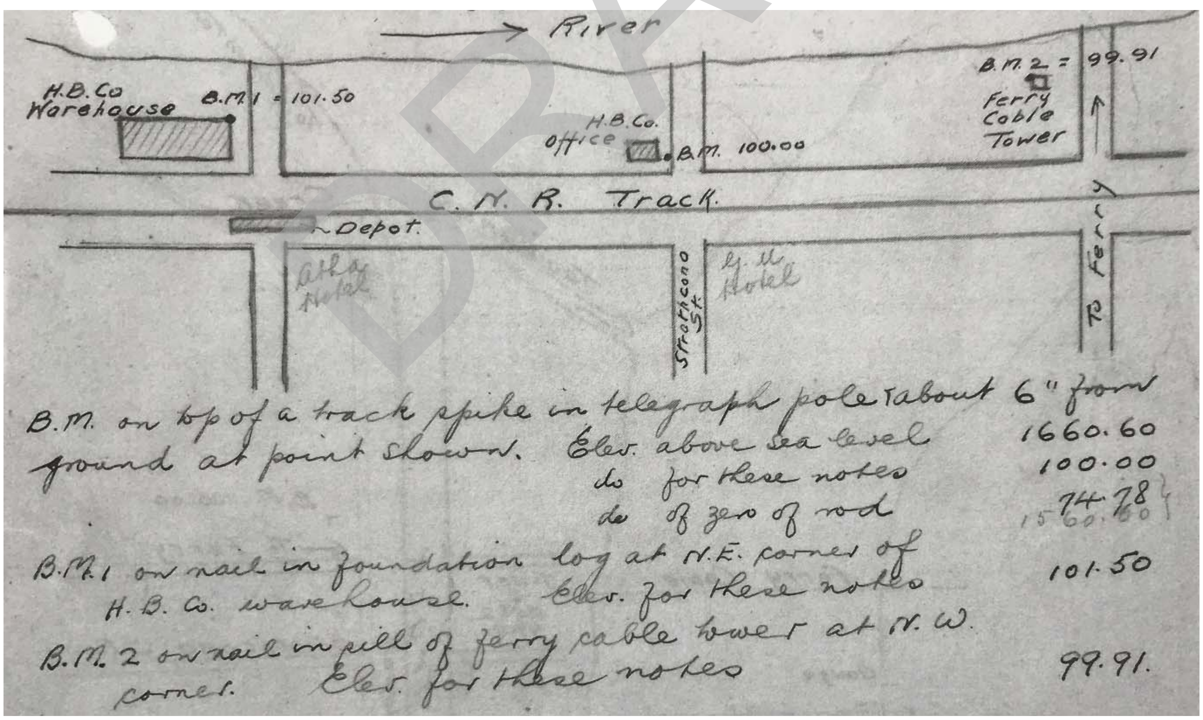

Figure I-14. Locations of Water Survey of Canada (WSC) benchmarks on the river-side of the railway right-of-way, 1914. *Location of hotels are shown, along with various HBC structures. To convert the elevation of B.M. located near the HBC office to 2019 datum, add 27.50 feet.*

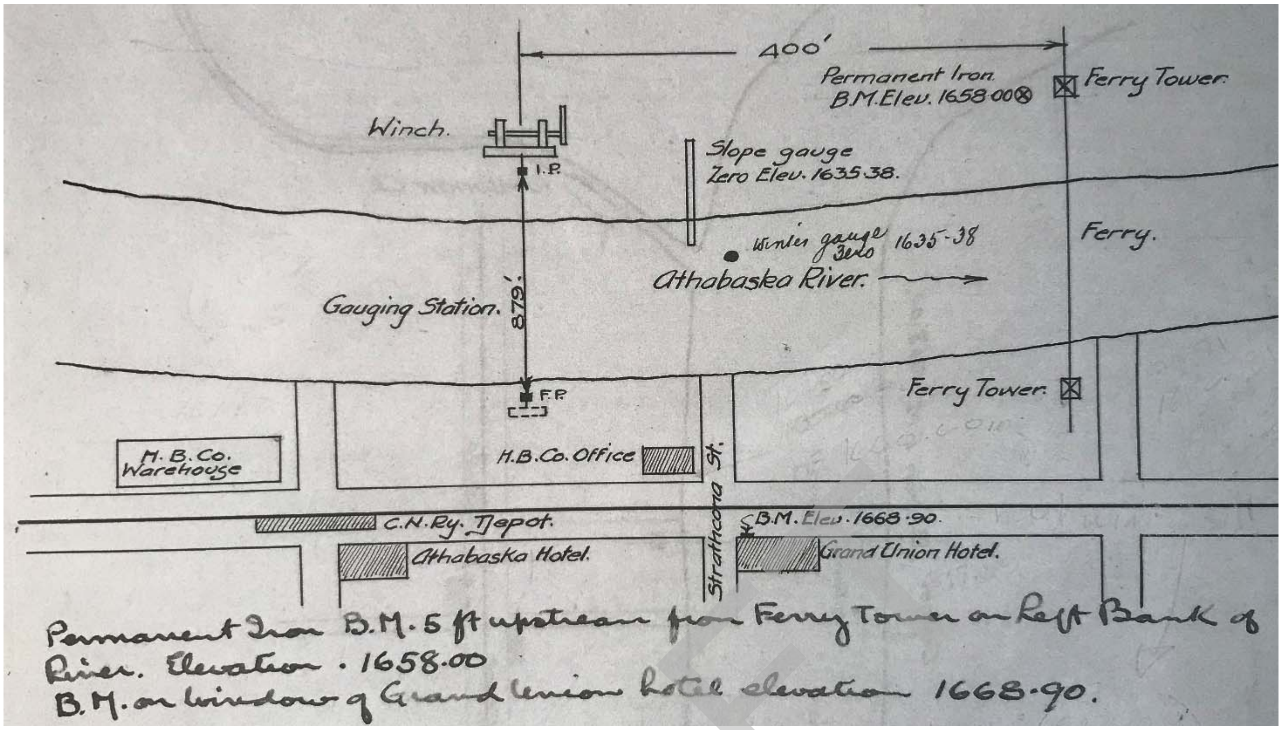

Figure I-15. Locations of WSC gauges and cross sections, 1914. *Note the elevation of the benchmark on the sill of the Grand Union Hotel. It was, and still is, at elevation 517.09 m based on the 2019 datum. The line IP-FP shows the location of a 1914 cross section surveyed by WSC.*

The 1914 waterfront is shown on Figure I-16. The main features described in the WSC records are evident in the photo, along with other notable structures. Of interest is the difference in elevation between the railway grade and the street level at the Grand Union Hotel. Clearly, the waterfront area between the river and Lichfield Street is lower than the elevation of Lichfield Street. It would not be surprising that structures on the river side of Lichfield Street would be susceptible to flooding.

Figure I-16. Athabasca waterfront in 1914 at the foot of Strathcona Street. *Note the presence of the train in front of the Grand Union Hotel, the Athabasca Hotel, and the newly constructed train station. The two HBC structures identified on Figure I-15 are also evident, along with other miscellaneous structures. Riverfront development has increased significantly since 1905 (Figure I-9).*

3 COMPARISON OF TERRAIN – EARLY 1900S AND 2019

A comparison of the general terrain in the early 1900s with the current terrain is possible using the existing LiDAR data and the benchmarks recorded by WSC. The comparisons are summarized in Table I-1. All the WSC benchmarks were located close to the ground on foundations or sill levels of structures. It is expected that the benchmarks would represent the ground levels in the early 1900s to within ±0.15 m. Overall, it appears that the area along the railway right-of-way has been raised by about a metre or so, but even with redevelopment, the elevations along the riverfront north of the railway right-of-way either have not changed significantly or the landscape has been lowered over time.

A comparison of the 1914 WSC cross section along section IP-FP (see location on Figure I-15) with a 2019 cross section is shown on Figure I-17. The WSC section falls roughly midway between Golder sections 5755.55 and 5914.59. The horizontal distances in the WSC section were adjusted so that the high left banks were superimposed. It is remarkable how little the river has changed in the last 100 or so years. The channel width, the position of the right bank, and bed levels have barely changed. Furthermore, it appears that the area in the vicinity of the top of bank has not changed significantly over time. The present floodplain slopes upward to the south at a gradient of about 35H:1V.

Table I-1. Comparison of waterfront elevations at the foot of Strathcona Street (50th Street)

Feature	1914 Elevation (m)	Approximate 2019 Ground Elevation (m)
WSC BM #1 (see Figure I-14 for location)	515.02	514.5 to 515.0
WSC BM	514.56	-
Ground near WSC BM (see Figure I-14 for location)	514.41	515.0 to 515.5
WSC BM #2 (see Figure I-14 for location)	514.53	513.5 to 514.0
Windowsill of Grand Union Hotel	517.09	517.09

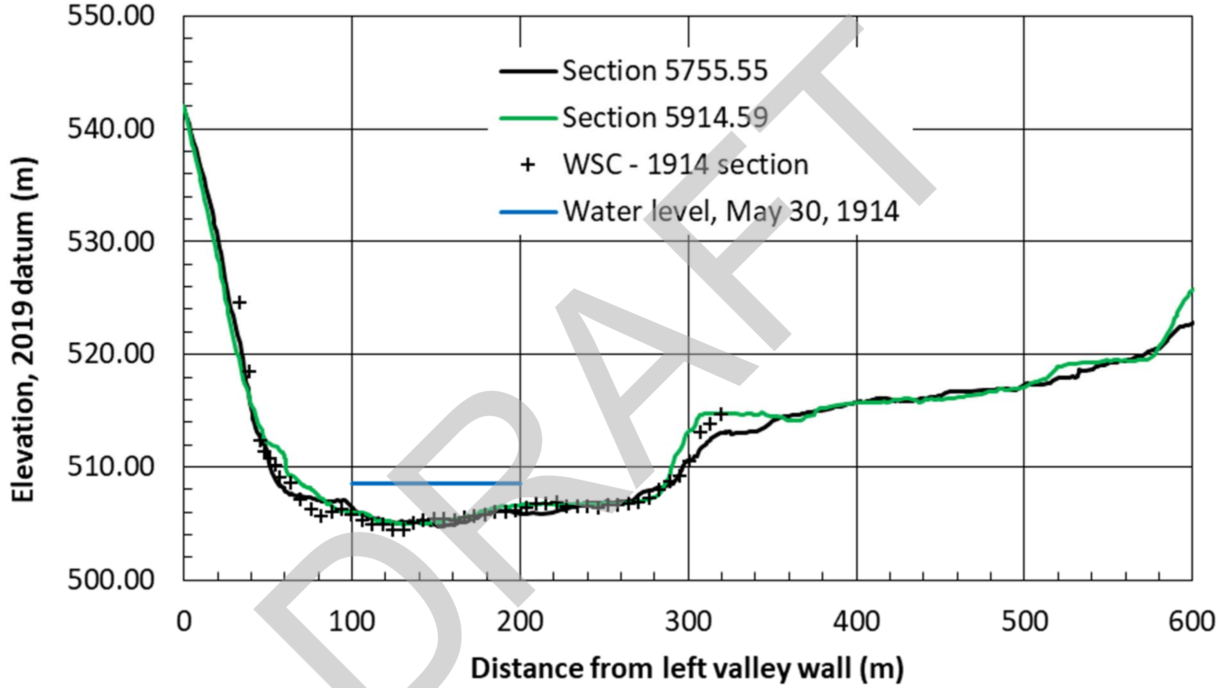

Figure I-17. Comparison of 1914 WSC cross section with 2019 surveyed cross sections.

A comparison of 2019 levels along Strathcona Street with 1914 spot elevations is shown on Figure I-18. Again, the levels along Strathcona Street between the river and todays 51st Avenue are based on LiDAR data, while the 1914 spot elevation were extracted from the WSC files. It appears that the WSC section at the channel margin reconciles well with today's elevations. Also, the elevation of the windowsill of the Grand Union Hotel reconciles with today's street level. In other words, the street level in 1914 (and likely in 1904) would be close to today's level. However, along the railway right-of-way, it appears that the landscape has been raised by about 1.5 m.

There is about a 6 m rise in elevation between today's 50th Avenue and 49th Avenue, with elevations increasing towards the south. If the overall grade of 50th Street has not changed significantly between 1914 and today, the elevation of each of the lots shown on Figure I-3 can be determined by interpolation. The adopted 1904 lot levels to the 2019 datum are summarized in Table I-2.

Table I-2. Estimated 1904 lot levels along Strathcona (50th) Street using 2019 LiDAR data

Lot Number	Description	Distance (m) South of South Edge of 50th Avenue	Street Elevation (m) at Centre of Lot
12	Grand Union Hotel – 12E	0 – 15.2	516.4
11	-	15.2 – 30.5	516.7
10	-	30.5 – 45.7	517.0
9	-	45.7 – 61.0	517.4
8	-	61.0 – 76.2	517.9
7	-	76.2 – 91.4	518.4
6	-	91.4 – 106.7	518.8
5	-	106.7 – 121.9	519.3
4	-	121.9 – 137.2	519.9
3	Telegraph Office – 3W	137.2 – 152.4	520.5
2	Imperial Bank of Canada – 2W HBC Warehouse – 2E	152.4 – 167.6	521.1
1	Main HBC Store – 1W	167.6 – 182.9	521.9

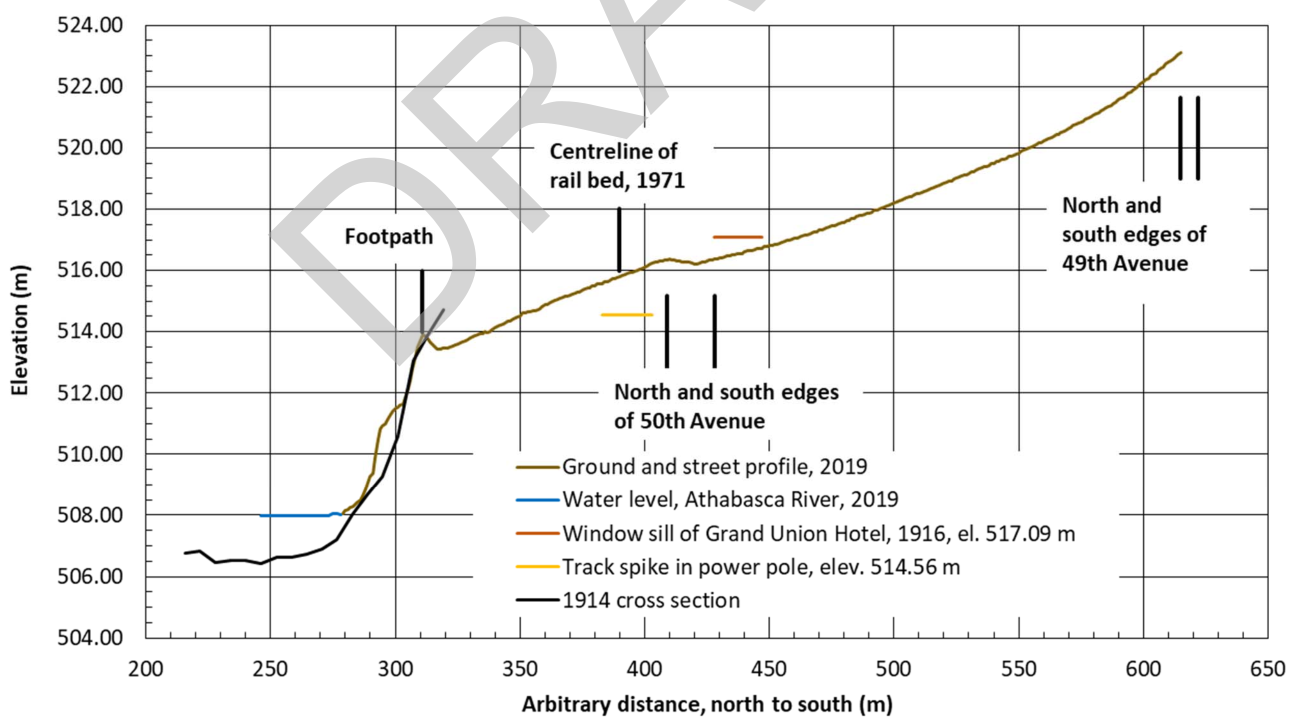

Figure I-18. Centreline profile along Strathcona (50th) Street based on 2019 LiDAR data with comparison to salient 1914 elevations.

4 DESCRIPTIONS OF THE 1904 ICE JAM EVENT

4.1 Extracts from the Hudson's Bay Company Archives (Source: Lisa Friesen)

4.1.1 Letter to Wm. Ware, Esq., dated April 22, 1904.

As of April 21, 1904:

"Serious flooding prevails Athabasca Landing. Warehouses with Northern freight in danger. Further details will be sent as soon as possible by telegraph."

"there was an ice jam which had flooded the flats, water in warehouse to beams, warehouses expecting to be swept away"

"the ice jam has broken, the damage to buildings less than feared."

4.1.2 Letter to Wm. Ware, Esq., dated April 26, 1904.

As of April 25, 1904:

"Athabasca Flood may now be considered to have subsided. Freight has been under water. A very great portion must be replaced immediately. I shall be able to meet the emergency."

"The information at hand is now to the following effect: The river ice started on the 17th inst., ran for about two hours and jammed. Thereafter the water rose suddenly and within another two hours the whole river flats became flooded, the ice being forced onto the land, and breaking down and carrying away several buildings and their contents. To what exact depth the water rose in the buildings I do not know yet. So much ice has been stranded on the flats that the warehouses had not yet been examined at time of writing but there is every reason to fear that the great bulk of the Northern freight stored at the landing has been under water."

"The damage to the Company's buildings is stated to be:

1) The Saw-Mill torn down, machinery of Mill rolled down the flat and buried under tons of ice.
2) Warehouse No. 1. The one next to the river. Moved 10 or 12 feet in towards the next warehouse, part of the east end torn out and some bacon and other goods swept down the river.
3) Workshop and Forge knocked down.
4) Steamer "Athabasca" thrown in onto bank 20 or 30 feet."

"It may be well to quote the latest communication from the Clerk in Charge at Athabasca Landing to the Officer in Charge of the District:

> "19th April. Just back from office. Managed to scramble in. Grinds the heart out of a man. Men and team have been at work since 7 o'clock cleaning out a 10 feet space to get to the shop, -- as yet they are only at the corner and will do well if get to the office door tomorrow.

Everything up at McDougall & Secord's with the exception of Gagnon's Mill, which has disappeared, is safe. One of Louttit's two scows is wrecked but may be repaired. The other is buried somewhere out in front of the office. Part of the Mission warehouse was turned right about. Wood's Steamer is away up the bank, but safe; all his workshops, shanties, etc., were swept away, -- we have all had a trying time."

Upon the foregoing data anything like an approximate estimate of the loss cannot be given."

4.1.3 Letter to Wm. Ware, Esq., dated June 4, 1904.

"Regarding the buildings, it is recommended that the warehouses be removed to a plot marked on the accompanying plan *[east of Skinner Street, opposite Lots 22E and 23E, as shown on Figures I-3 and I-19]*. The level of this plot is 8 feet higher than where the water reached at the highest point of the flood. The inner of the two main warehouses can be removed standing, but the outer one has been so wrecked that it will have to be replaced by another which must be larger as more storage is required. A new shop will also have to be built. The site suggested for it is Lot 2 *[Lot 2E as shown on Figure I-3]*."

Figure I-19. Present day lot locations of the relocated HBC warehouses, east of 49th (Skinner) Street. *The green lines represent approximate lot lines and the* ***X****s mark the intended locations of the structures. The minimum elevation of the most northerly lot was about 520.0 m.*

The 2019 LiDAR data (Figure I-18) suggest that the elevation east of Skinner Street (49th Street) is about 520.0 m. Adopting a 2.4 m (8 foot) water level difference between the area east of Skinner Street and the level of the 1904 flood, this observation would suggest that the HBC estimate of the peak water level associated with the 1904 flood would have been about 517.5 m.

4.2 Extracts from page 79 of "Athabasca Landing: An Illustrated History. Athabasca Historical Society, 1986. Revised, 2018. Athabasca, Alberta."

"We know relatively little about the pioneers of this early wave. One of them, however, has left us a memoir of his first years at the Landing. Scottie Willey, one of the first homesteaders in the South Athabasca area, came to the Landing in 1903. Here are some extracts from his reminiscences, written more than fifty years later:

> On New Year's Day in 1904 we squatted on unsurveyed land in what is known as the South Athabasca district. It was surveyed several years later. That winter we took a contract to cut and deliver thirty miles of telegraph poles for the first telegraph line to Edmonton. The spring of 1904 after an extra severe winter, making the ice very thick, the ice jammed on an island down the river, causing the water to back up and flood the town. This lasted for three days before the jam broke and the water released. The water backed up as far as the H.B. Store, now the Parker Store. There was six feet of water in the hotel on the corner. Boats were used for transportation in the street. … At that time of year the warehouses were full of goods and the river bank was lined with boats being built to take this cargo north. Many boats were smashed and goods in post and warehouses were lost or damaged. H.B.Co. brought in a crew of men from Edmonton to unpack and dry out these goods. They were most of the summer at this job. The late David Hay, of Colinton, came here on that work. … At this time a steamboat was being built across the river. The builder was Sam Emmerson, who afterwards homesteaded in West Athabasca. … This was one of the several boats plying the river at that time and later. These boats went up the river as far as Little Slave river (now Smith) and down the river to Pelican Rapids. … Athabasca Landing was the distribution point for all freight going north. By boats in summer and teams on ice in winter."
>
> Willey, "Reminiscing"., Athabasca Echo, 26 August 1959.
> Athabasca Echo, 2 September, 1959. Athabasca Echo, 8 June 1960.

If the water had "backed up" to the HBC store, then located in Lot 1W along Strathcona Street, the water level would have been at an elevation somewhere between 521 and 522 m (Table I-2). With the main floor elevation of the Grand Union Hotel being close to the current street level of 516.3 m, 1.8 m of water in the hotel would have produced a water level of 518.2 m. It appears that the two anecdotal data points do not reconcile. However, given the HBC record described above, it is likely that the lower elevation is more plausible.

The most compelling evidence of the 1904 flood level is the photograph of the HBC building shown on Figure I-5. Unfortunately, it is difficult to establish the precise location of that structure. Evidently it was removed or altered to such a degree that its location is not apparent in any photographs taken after 1904. Considering the two most plausible alternative pieces of evidence summarized above, it is expected that the 1904 ice jam flood likely produced a maximum water level of between 517.5 and 518.2 m. Given the potential changes in the elevation of the lots to which the HBC warehouses were relocated, the 2019 LiDAR data probably provides an upper limit to what the elevation would have been in 1904. Furthermore, it is likely that the ground floor elevation in the pre-1913 Grand Union Hotel was lower than its current elevation, which also would make the Grand Union estimate on the high side. From these considerations, 517.5 m would be the most plausible elevation of the 1904 peak. Even at that level, it is evident that the flood would have encroached about halfway up Strathcona Street and inundated all the lots located north of Lot 8. This would have caused considerable damages to both the infrastructure along the river and well-established businesses south of Lichfield Street.

5 HISTORICAL THRESHOLD (PERCEPTION) LEVELS

Identifying threshold levels based on what might constitute a severe and notable event during a particular time period is somewhat of a hazardous exercise that requires an understanding of the tolerance of residents to floods during a particular historical era. The following thinking may have prevailed during the historical period, based on the current relevance of the 1914 cross section and the sensitivity to flooding of the development between the river and Strathcona Street:

- Prior to 1884 there would have been little notice of flooding events since there were no significant facilities that could sustain damage of any consequence.
- In the 1890s, during which time the HBC and other companies established a number of warehouses and storage sheds immediately adjacent to the bank (Figure I-20), any floods above bankfull (about elevation 514 m) would have been noted. Given that the structures were located so close to the top of bank, it appears there was no concern that water levels would exceed 514 m. That is, between 1884 and 1904 (or thereabouts), no flood would have exceeded this elevation.
- The large flood in 1904 must have created some concerns for the residents. However, there is no evidence that it served to dampen the development along the river for an extended period of time (Figures I-21 and I-22), except for HBC moving some of its facilities to higher ground after the flood.
- With the installation of the WSC hydrometric gauge in 1913 and its first full year of operation in 1914, systematic water level monitoring was typically being undertaken, such that any large flood would have been recorded. However, by that time, the railway and train station had been constructed, all to an elevation of about 514.6 m. If any floods had exceeded that level, reports would have been made regardless of whether or not the WSC gauge was in operation.

Based on the above, elevation 514.0 m appears to be a reasonable threshold level to provide context to the historical floods to satisfy the Bulletin 17B statistical methodology.

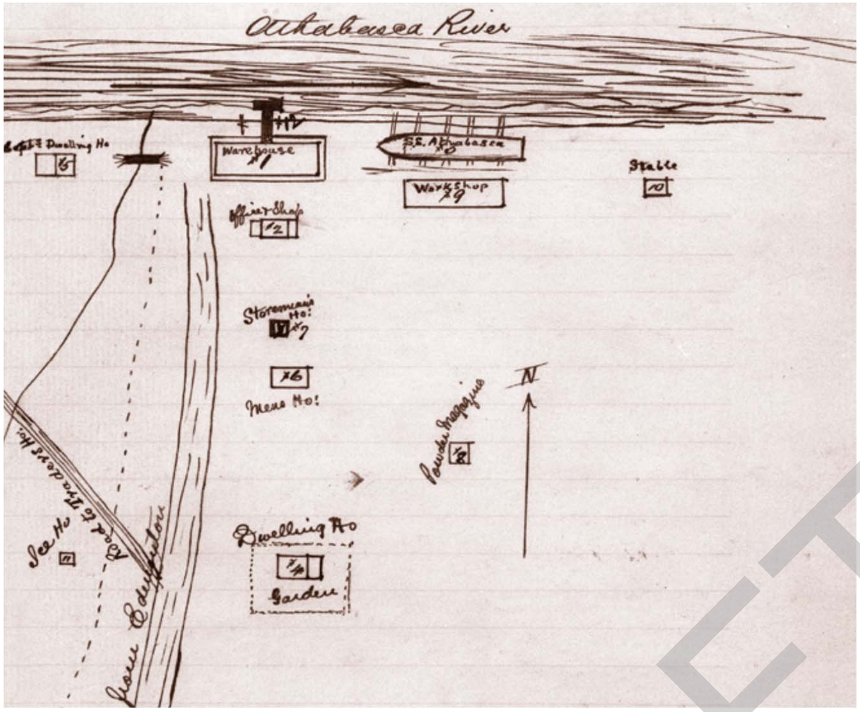

Figure I-20. Inspecting Chief Factor James McDougall. Hudson's Bay Company "Sketch Plan of Athabasca Landing." 1891. *Archives of Manitoba Hudson's Bay Company Archives, D. 24/19 fo. 295.*

Figure I-21. Athabasca Landing viewed from the river, 1899. *Glenbow Archives, NA-949-7. Compare the photo to the sketch on Figure I-20.*

Figure I-22. A general view of Athabasca Landing, 1911, looking west. *Provincial Archives of Alberta Brown Collection, B 2620. Note the bed slope between the river and the base of the bank, and the slope of the floodplain between the top of the bank and Lichfield Street. The floodplain is sparsely developed, except for the area close to Lichfield Street. Also, note the two large structures on the left of the photo. They appear to be the relocated HBC warehouses that are meant to be 2.4 m above the 1904 flood level. West of the warehouses, the white building with the dark roof and the black window in the gable is likely the HBC Store; the high building just south of it is likely the telegraph building. The lower-lying Imperial Bank of Canada building is hidden from view.*

APPENDIX II

Formulation of Equilibrium Ice Jam Equations

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Formulation of Equilibrium Ice Jam Equations

The thickness of a "wide channel" ice jam is a function of the force balance between (i) longitudinal driving forces arising from shear along the jam underside and the streamwise weight of the jam that promote thickening and (ii) friction and cohesion forces between the jam and the bank that resist thickening. The driving and resisting forces are both partly a function of the thickness. A stable jam develops when the driving and resisting forces are in equilibrium and the jam achieves its "equilibrium ice thickness."

The formulation of the ice jam stability equation used in HEC-RAS to calculate steady state, gradually varied ice jam thickness profiles in a streamwise direction *x* is:

$$\frac{d(\sigma_x\, t)}{dx} + \frac{2\,\tau_b\, t}{B} = \rho_i\, g\, S\, t + \rho_w\, g\, H_i\, S$$

where t is jam thickness, σ_x is the depth-average internal longitudinal stress arising out of the hydrostatic pressures in the ice column, τ_b is the shear resistance along the banks that confine the jam, B is the channel width, ρ_i and ρ are the density of ice and water respectively, g is the acceleration of gravity, S is the slope, and H_i is the flow depth under the jam that is associated with the jam underside.

The two terms on the left side of the equation represent the contribution that the internal strength of the jam has on its stability. The first and second terms on the right side represent the streamwise weight of the jam and the shear force on the underside of the jam, respectively (i.e., the driving forces).

The equation is solvable for the jam thickness if equations can be derived (i) for the relationship between the internal depth-averaged longitudinal stresses and depth-averaged lateral stresses, (ii) the way these internal forces are related to the thickness of the jam, and (iii) the mechanisms by which the depth-averaged lateral stresses are transferred to the channel banks. This is done using the equations listed below, which relate the various internal longitudinal, vertical, and lateral stresses resulting from the driving forces to the thickness of the jam that can just resist the driving forces. These stress-thickness relationships have their own coefficients that need to be defined either from fundamental ice parameters or by calibration.

The vertical stress, σ_z , is related to the thickness by:

$$\sigma_z = \gamma\, t$$

where

$$\gamma = 0.5\,\rho_i\, g\,(1 - \rho_i/\rho_w)\,(1 - p)$$

where p is the jam porosity.

The horizontal stress is related to the vertical stress by

$$\sigma_x = K_x \, \sigma_z$$

where

$$K_x = tan^2 \, (\pi/2 + \phi/2)$$

where ϕ is the angle of internal friction of the ice.

Furthermore, with respect to the shear stresses along the bank, the stresses orthogonal to the bank, σ_y , can be written as:

$$\sigma_y = K_1 \, \sigma_x$$

and finally

$$\tau_b = K_o \, \sigma_y$$

where the lateral thrust coefficient K_1 is given by

$$K_1 = [(1 - sin^2 \, \phi)/((1 + sin^2\phi)]$$

but is typically given a value of 0.33, and

$$K_o = \tan \phi$$

Solution of the gradually varied flow equation requires values for the angle of internal friction and the jam porosity. The default values in HEC-RAS, which have been selected based on numerous field measurements of ice jams, are porosity (0.4) and angle of internal friction (45 degrees). This results in values for K_x of 1.0, K_1 of 0.33, and K_o of 1.6.

For uniform flow analysis, where the hydraulics are based on reach-average channel characteristics (top width and average depth) that vary with the water level, the gradually varied flow equation is simplified to represent only the equilibrium portion of the ice jam where the jam thickness does not vary with distance along the channel. This simplification results in the following relatively quadratic equation to calculate the thickness:

$$\mu \, \rho_i \, (1 - (\rho_i/\rho) \, g \, t^2 = B \, (\rho_i \, g \, S \, t + \rho_w \, g \, H_i \, S)$$

The dimensionless coefficient of internal friction, μ , embodies the porosity and the angle of internal friction of the ice using:

$$\mu = K_o \, K_1 \, K_p \, (1 - p)$$

Additional equations are required to calculate the flow depth under the jam for a given discharge, Q, using the following modified Manning equation, which represents flow under an ice cover:

$$Q = (1/n_o)\,(H_o/2)^{2/3}\,A\,S^{1/2}$$

where

$$A = B\,H_o$$

$$n_o = \left(\left(n_i^{3/2} + n_b^{3/2}\right)/2\right)^{2/3}$$

$$H_i/H_o = (n_i/n_o)^{2/3}$$

and

$$H_o = H_i + H_b$$

Finally,

$$H_j = H_o + 0.92\,t$$

where n_i and n_b are the roughness of the jam underside and the bed, respectively, n_o is the composite roughness of the flow under the jam, A is the flow area under the jam, H_i is the flow depth associated with the jam, H_o is the total flow depth, and H_j is the height of the jam.

The computation to derive the ice jam rating curve, as outlined below, is quite straightforward once the relationship between elevation and the reach-average channel geometry is defined.

1) The flow depths associated with both the bed and the underside of the ice jam are calculated for each unique combination of water level, top width, and mean depth H_o that represent the stage at the underside of the ice jam.
2) Knowing the slope and the flow depth associated with the ice underside, which quantify the shear stress on the ice underside, the thickness of the jam is calculated from the stability equation for an adopted value of the dimensionless coefficient of internal friction.
3) The submerged thickness of the jam is added to the water level to calculate the jam level.
4) Adjust either the dimensionless coefficient of internal friction to values within the range of 0.9 to 1.3 – and/or adjust the jam roughness coefficient – to achieve a reasonable reconciliation between the ice jam rating curve and the measured (or expected) ice jam levels.

APPENDIX I

Ice Jam Flood Profiles

Figure I-1: Simulated Ice Jam Water Surface Profiles along the Athabasca River Study

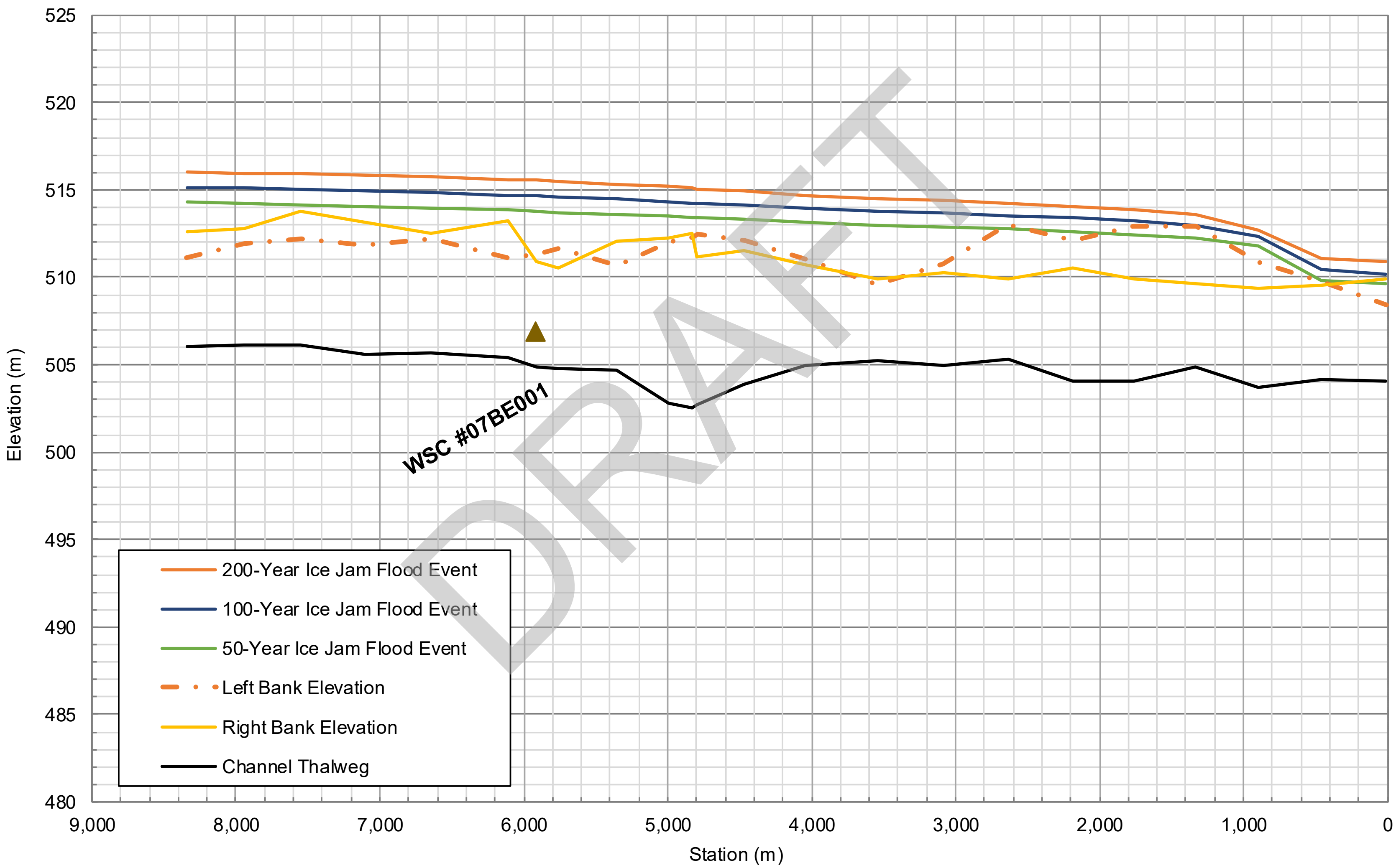


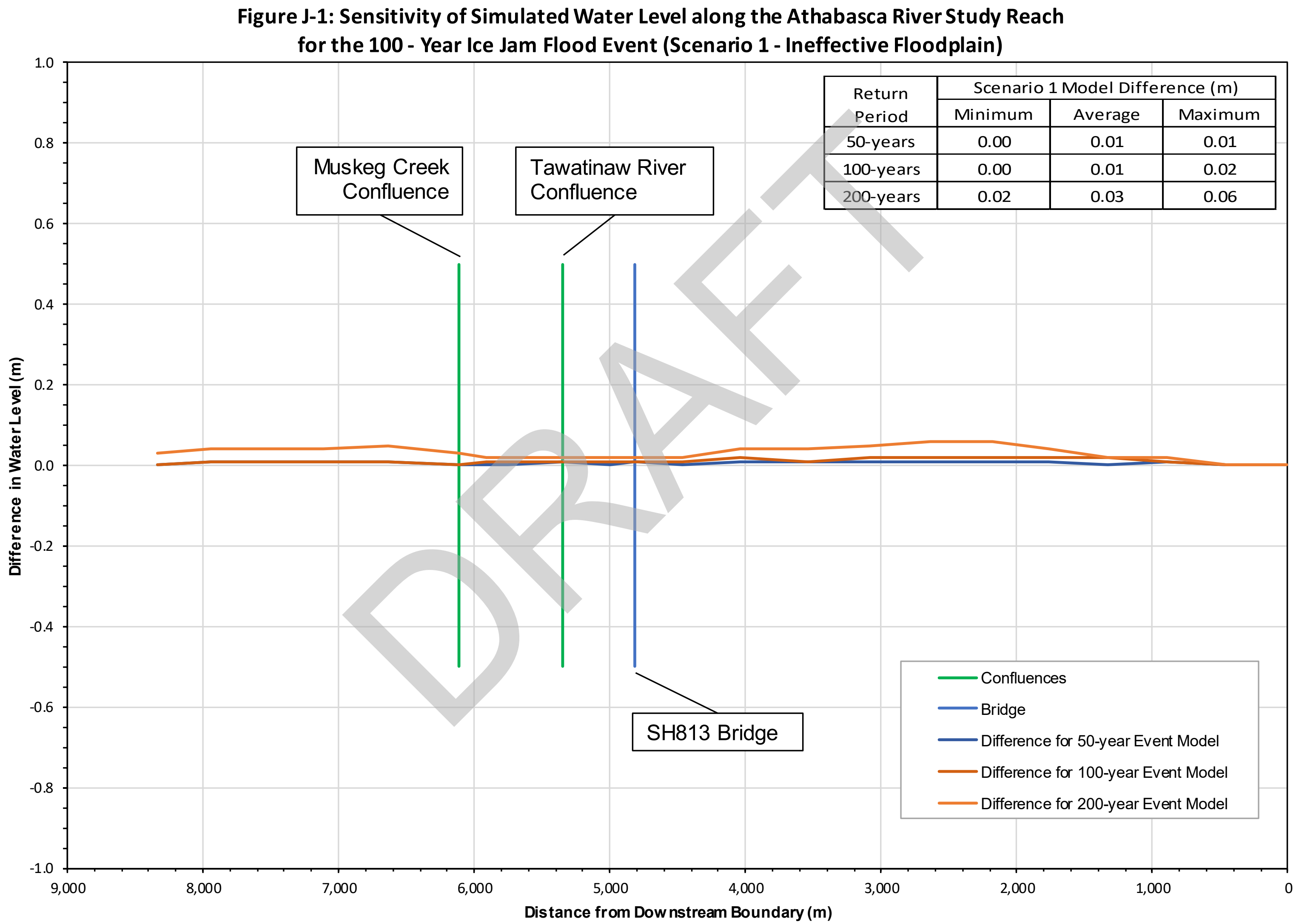
Table I-1: Simulated Ice Jam Water Levels along the Athabasca River

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)		
				50-Year	100-Year	200-Year
Athabasca River	1	8340	506.0	514.3	515.2	516.1
Athabasca River	2	7941	506.1	514.2	515.1	516.0
Athabasca River	3	7550	506.1	514.1	515.0	515.9
Athabasca River	4	7109	505.6	514.1	515.0	515.8
Athabasca River	5	6640	505.7	514.0	514.9	515.7
Athabasca River	6	6112	505.5	513.8	514.7	515.6
Athabasca River [a]	7	5915	504.9	513.8	514.6	515.5
Athabasca River	8	5756	504.7	513.7	514.6	515.5
Athabasca River	9	5352	504.7	513.6	514.5	515.3
Athabasca River	10	4994	502.8	513.5	514.3	515.2
Athabasca River	11	4829	502.6	513.4	514.3	515.1
Athabasca River	12	4796	502.7	513.4	514.3	515.1
Athabasca River	13	4468	503.9	513.3	514.1	514.9
Athabasca River	14	4043	505.0	513.1	513.9	514.7
Athabasca River	15	3542	505.2	513.0	513.8	514.5
Athabasca River	16	3087	504.9	512.9	513.7	514.4
Athabasca River	17	2636	505.3	512.8	513.5	514.2
Athabasca River	18	2182	504.0	512.6	513.4	514.0
Athabasca River	19	1764	504.1	512.5	513.2	513.8
Athabasca River	20	1329	504.9	512.3	513.0	513.6
Athabasca River	21	892	503.7	511.8	512.4	512.7
Athabasca River	22	461	504.1	509.9	510.5	511.1
Athabasca River	23	6.48	504.1	509.6	510.2	510.9

(a) XS7 is located at WSC Station 07BE001

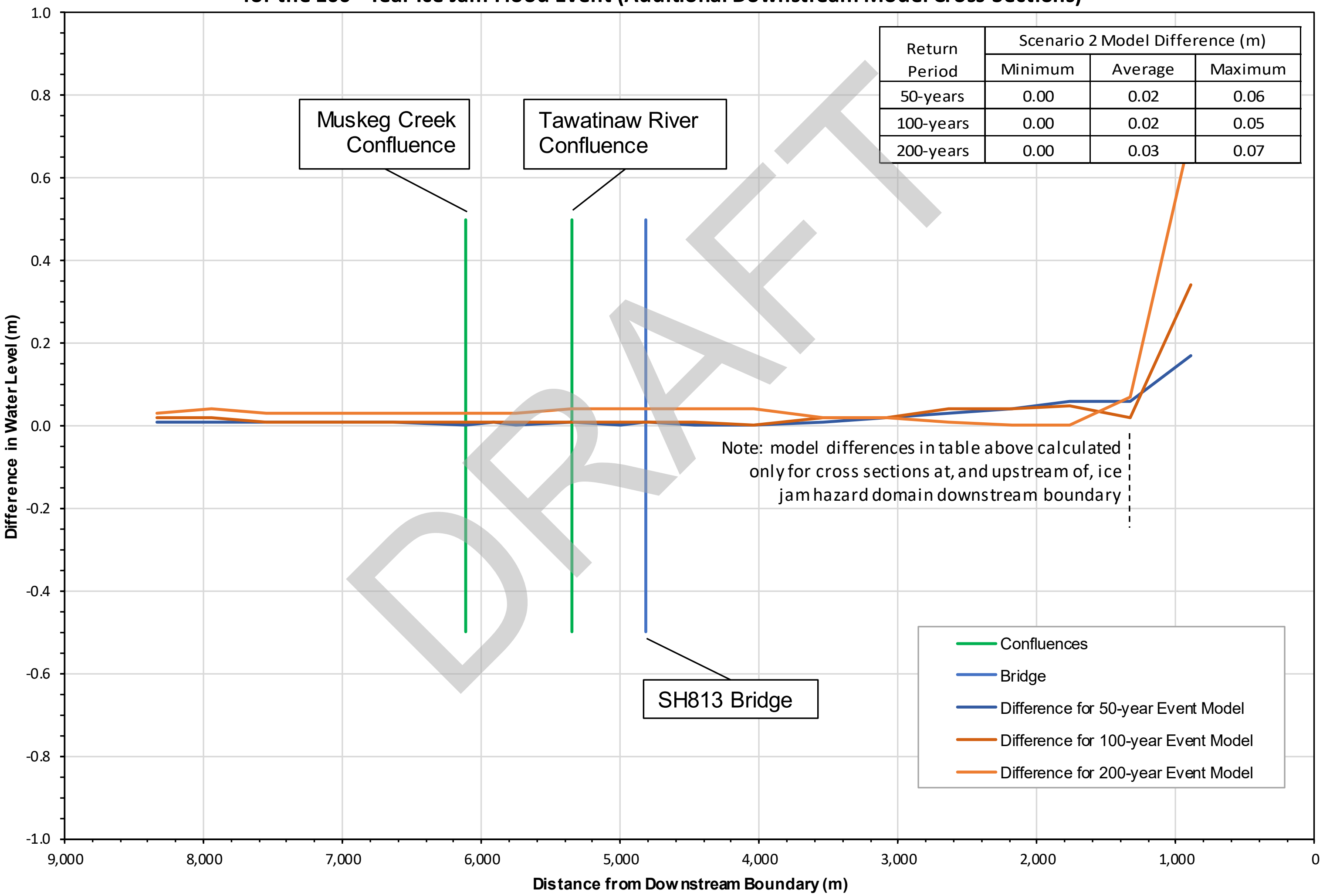
APPENDIX J

Ice Jam Sensitivity Analysis Flood Profiles



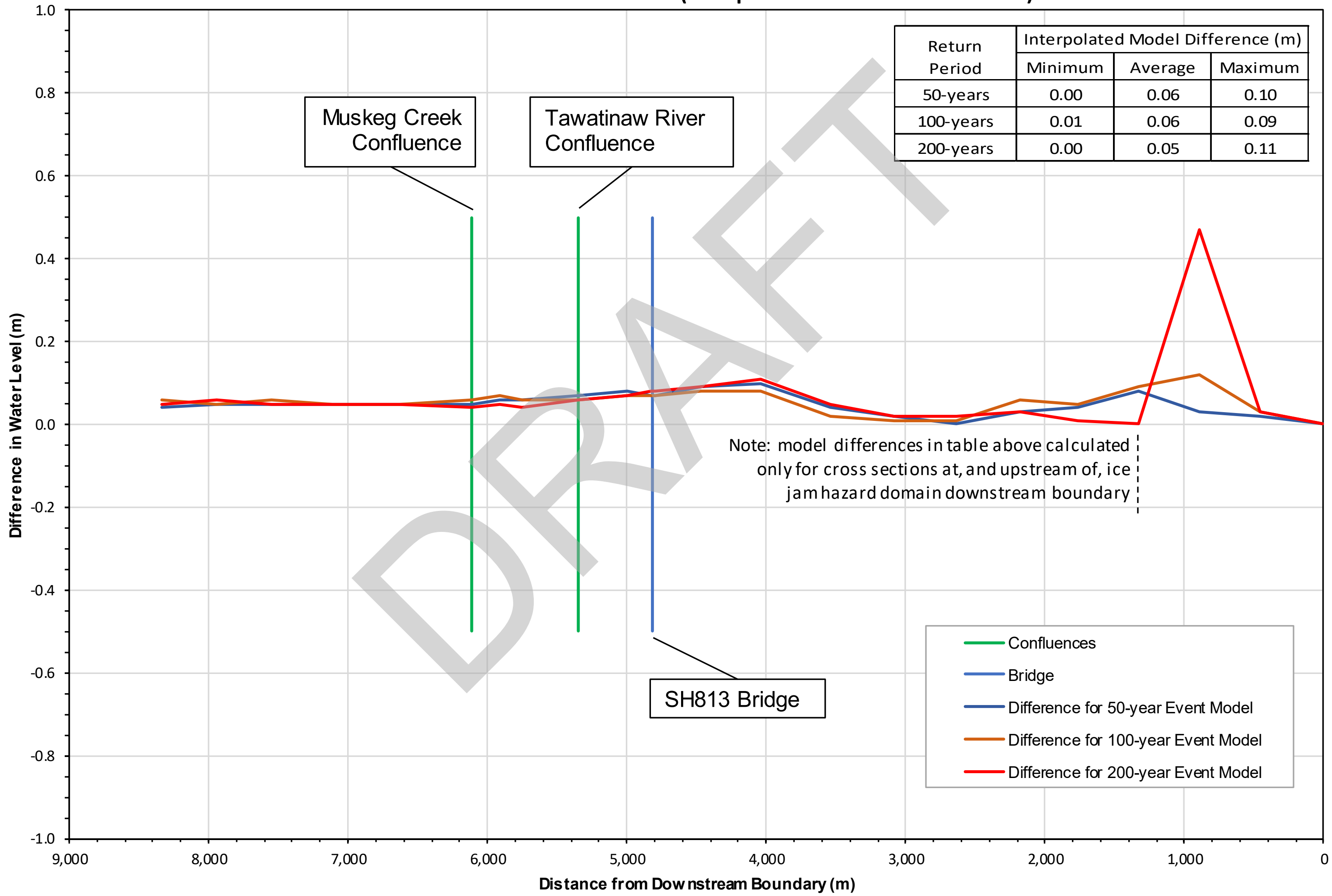
Return Period	Scenario 1 Model Difference (m)		
	Minimum	Average	Maximum
50-years	0.00	0.01	0.01
100-years	0.00	0.01	0.02
200-years	0.02	0.03	0.06

Figure J-2: Sensitivity of Simulated Water Level along the Athabasca River Study Reach for the 100 - Year Ice Jam Flood Event (Additional Downstream Model Cross-Sections)



Return Period	Scenario 2 Model Difference (m)		
	Minimum	Average	Maximum
50-years	0.00	0.02	0.06
100-years	0.00	0.02	0.05
200-years	0.00	0.03	0.07

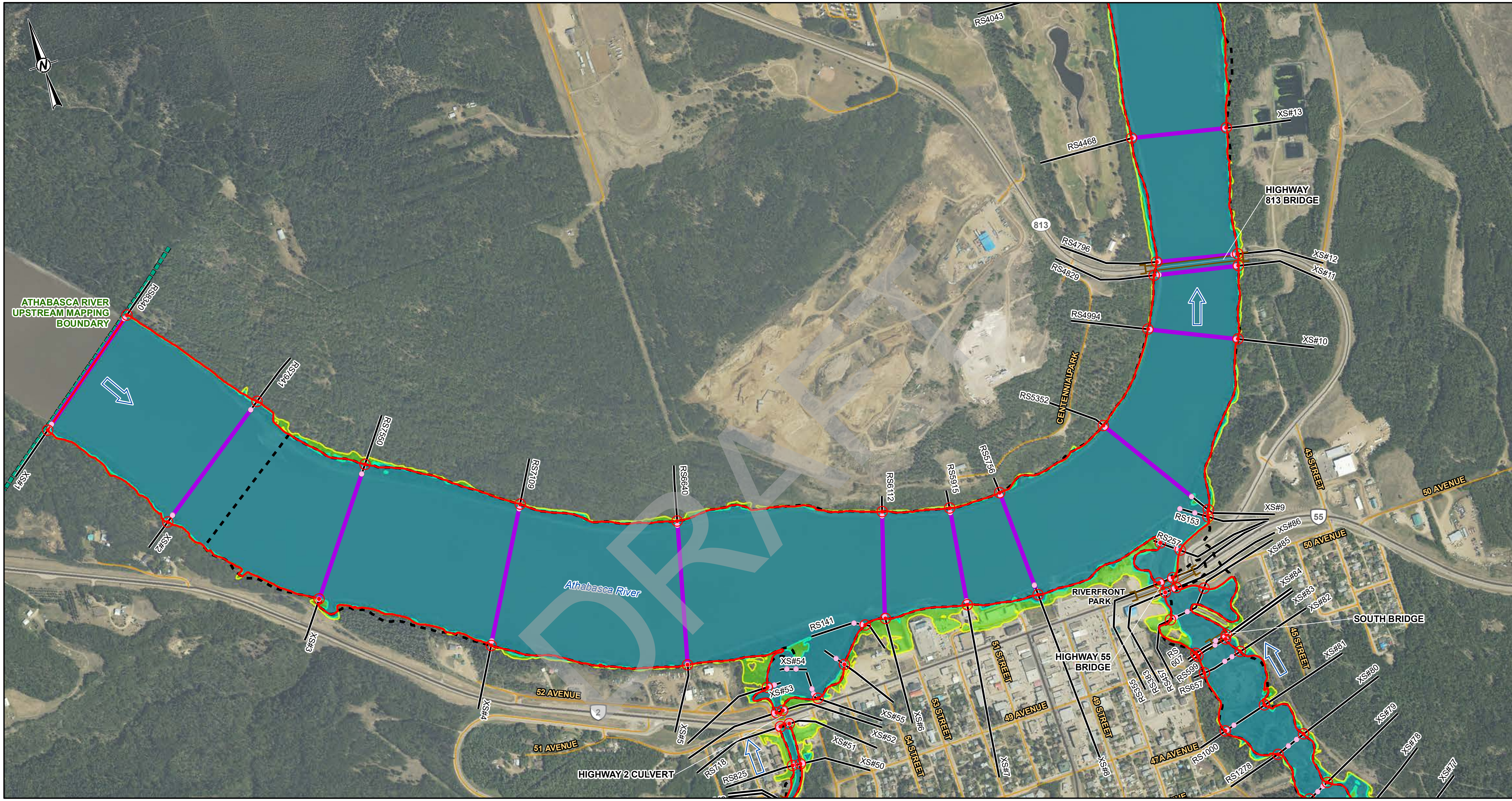
Figure J-3: Sensitivity of Simulated Water Level along the Athabasca River Study Reach for the 100 - Year Ice Jam Flood Event (Interpolated Model Cross-Sections)



APPENDIX K

Open Water and Ice Jam Flood Criteria Mapping

SHEET 2 ↑



SHEET 3 & 4 ↓

LEGEND

- CROSS SECTION
- XS#10 — CROSS SECTION NUMBER
- RS 4994 — RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

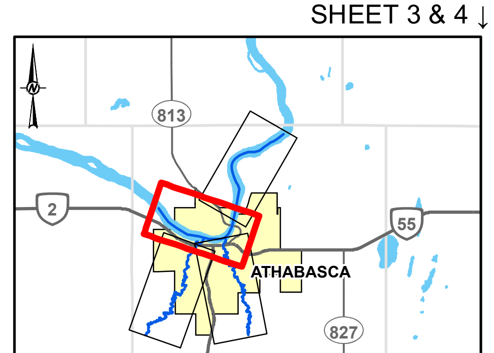
HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DESIGN DISCHARGE

ATHABASCA RIVER = 5,760 M^3/S
MUSKEG CREEK = 36.7 M^3/S
TAWATINAW RIVER = 73.1 M^3/S



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CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ATHABASCA FLOOD HAZARD STUDY

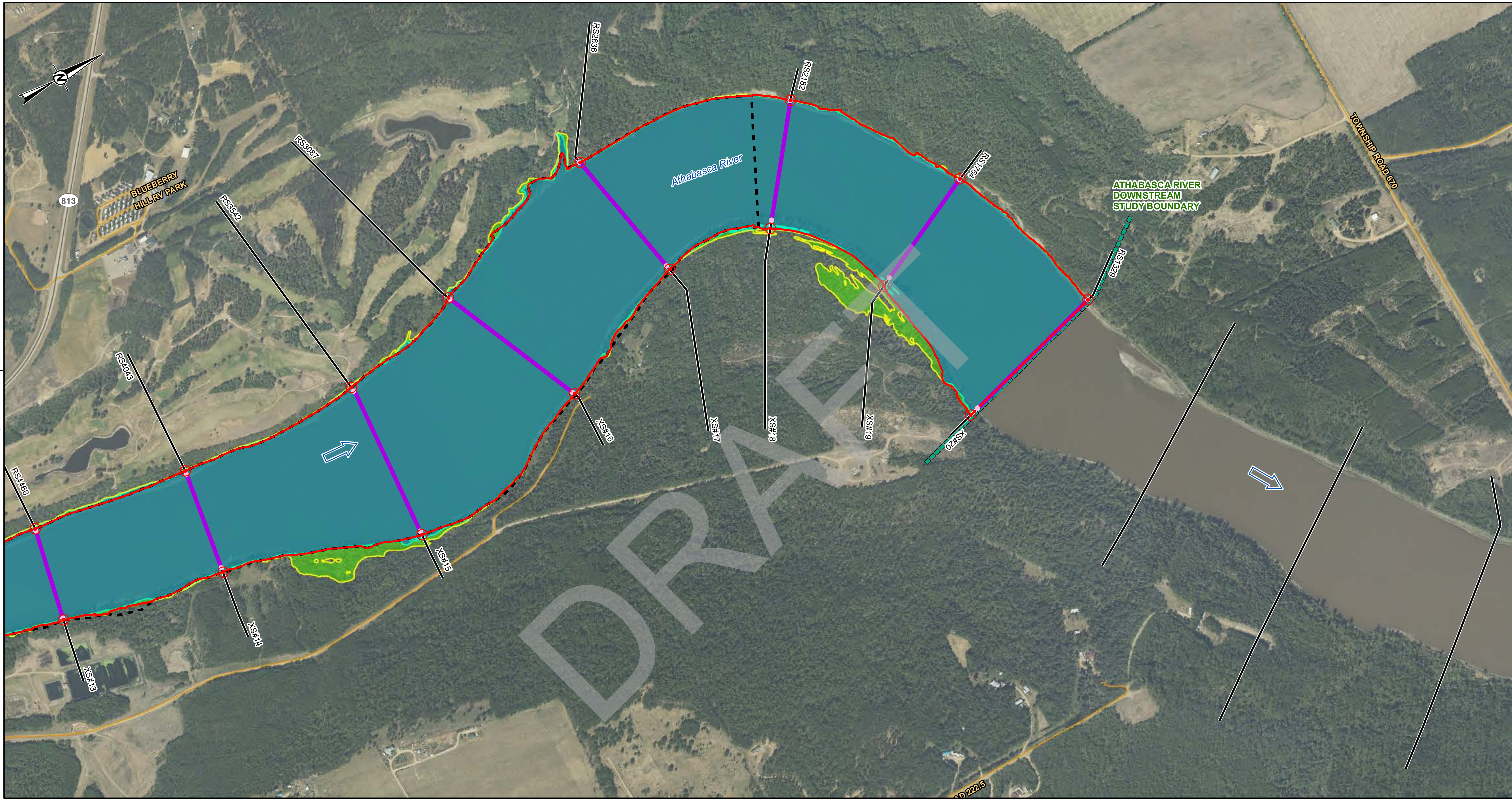
CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2020-03-31
DESIGNED	TT
PREPARED	PT/BP
REVIEWED	JC/WP
APPROVED	WP

TITLE
OPEN WATER FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 1 OF 4

Classification: Public



LEGEND

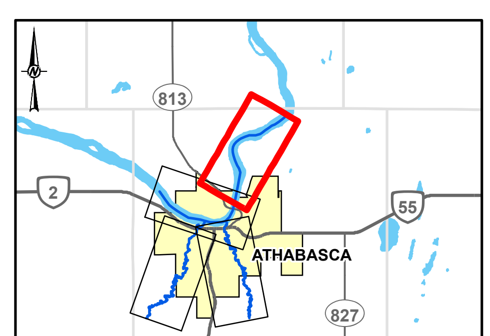
	CROSS SECTION
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DESIGN DISCHARGE
ATHABASCA RIVER = 5,760 M^3/S



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PROJECT
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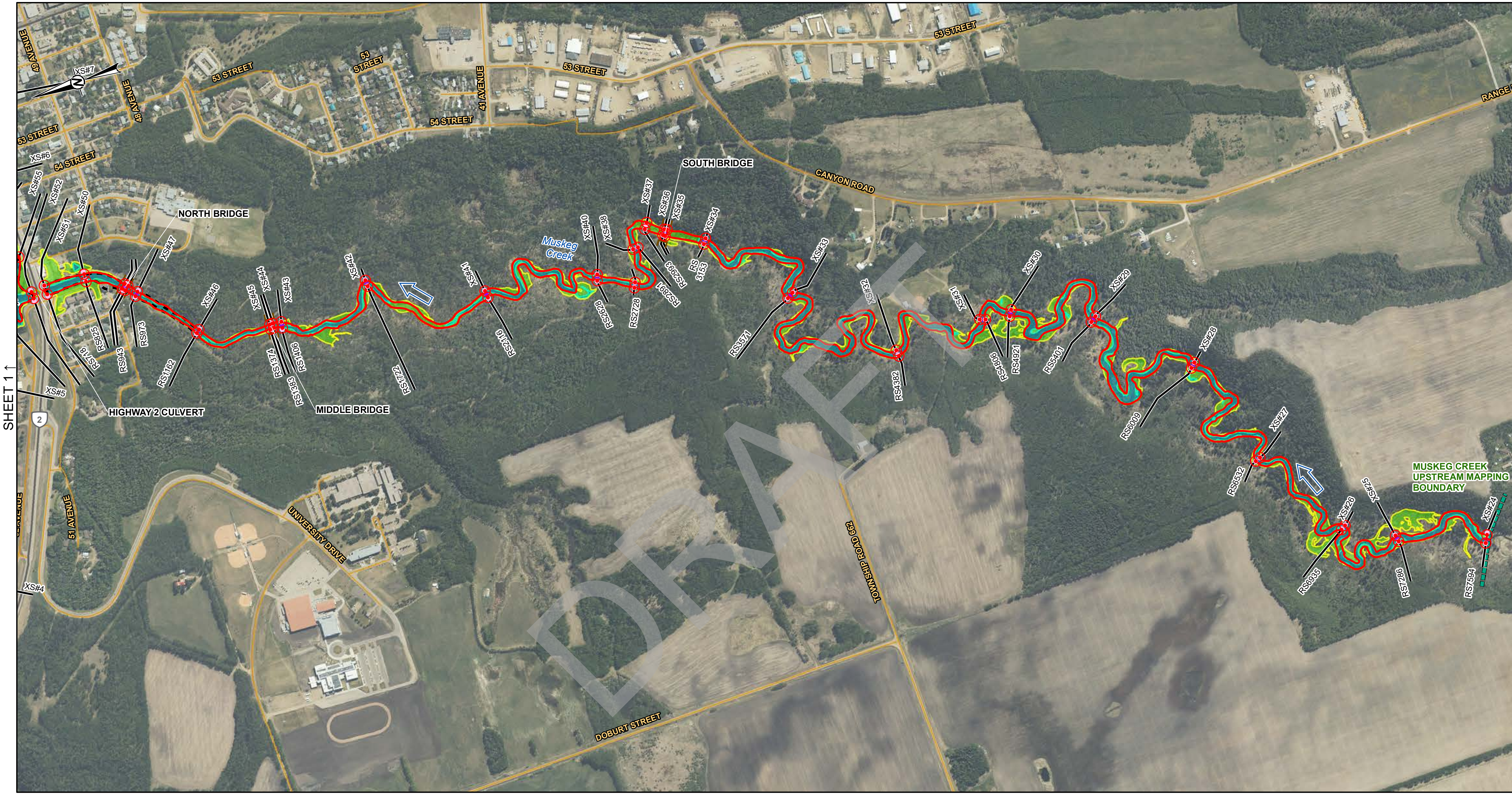
TITLE
OPEN WATER FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 2 OF 4

Classification: Public

SHEET 4 ↑



LEGEND

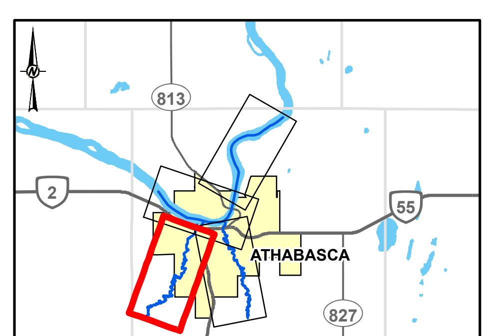
—	CROSS SECTION
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
▬▬▬	MAPPING BOUNDARY
⇨	FLOW DIRECTION
—	LOCAL ROAD
—	PRIMARY HIGHWAY
—	SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

⬠	CULVERT
⊢⊣	BRIDGE

▭	PROPOSED FLOODWAY BOUNDARY
●	BANK STATION
⊙	PROPOSED FLOODWAY STATION
▭	PREVIOUS FLOODWAY
■	DEPTH ≥ 1 M
■	100-YEAR DESIGN FLOOD EXTENT
—	VELOCITY ≥ 1 M/S

DESIGN DISCHARGE
MUSKEG CREEK = 36.7 M^3/S



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PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
OPEN WATER FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 3 OF 4

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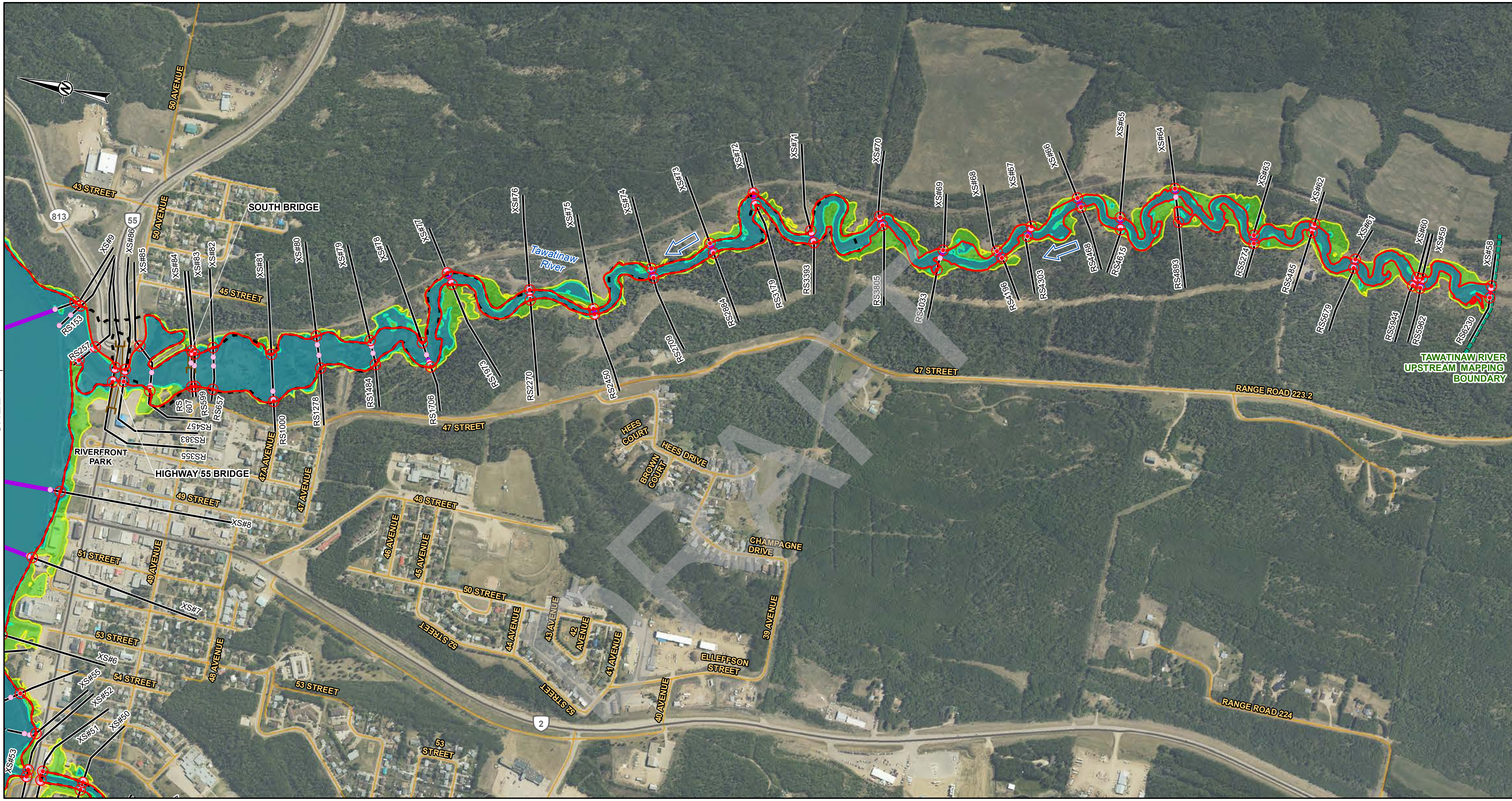
Classification: Public



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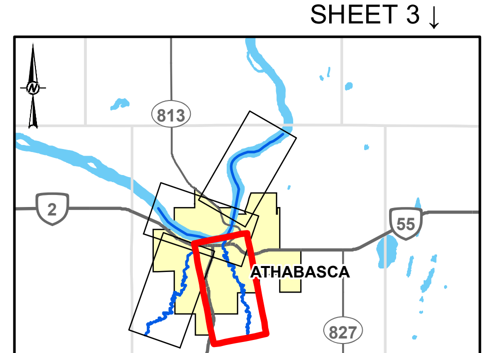

LEGEND

	CROSS SECTION
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DESIGN DISCHARGE
ATHABASCA RIVER = 5,760 M^3/S
TAWATINAW RIVER = 73.1 M^3/S

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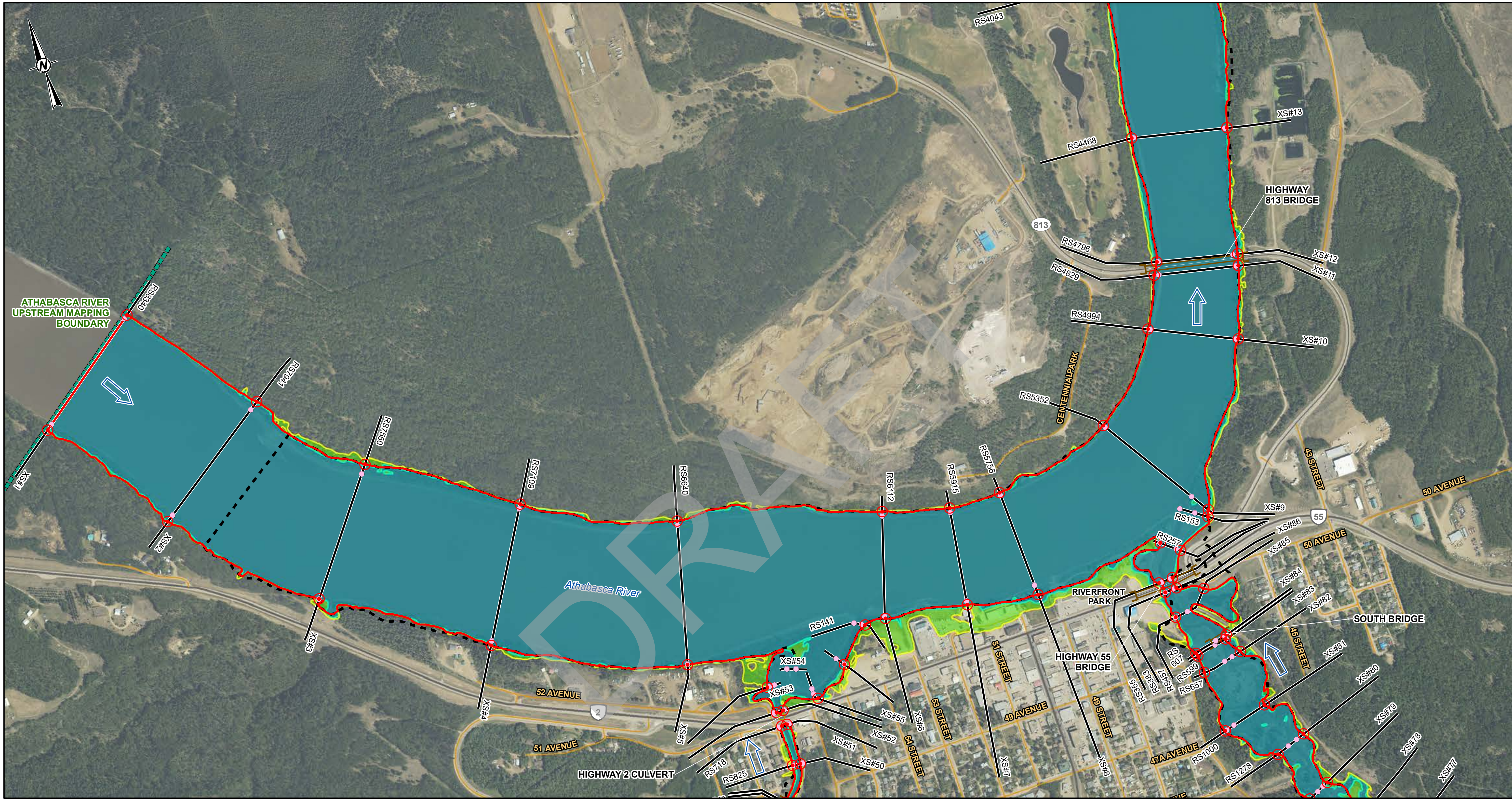
PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
OPEN WATER FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 4 OF 4

SHEET 2 ↑



SHEET 3 & 4 ↓

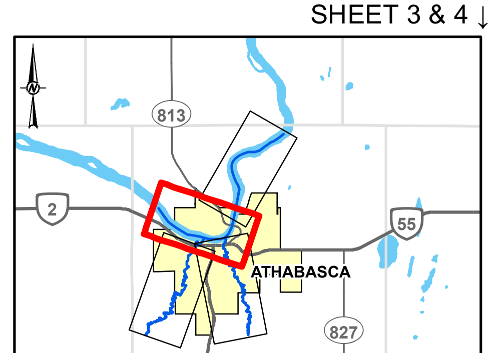
LEGEND

- CROSS SECTION
- XS#10 — CROSS SECTION NUMBER
- RS 4994 — RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED ICE JAM FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT



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CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2020-03-31
DESIGNED	TT
PREPARED	PT
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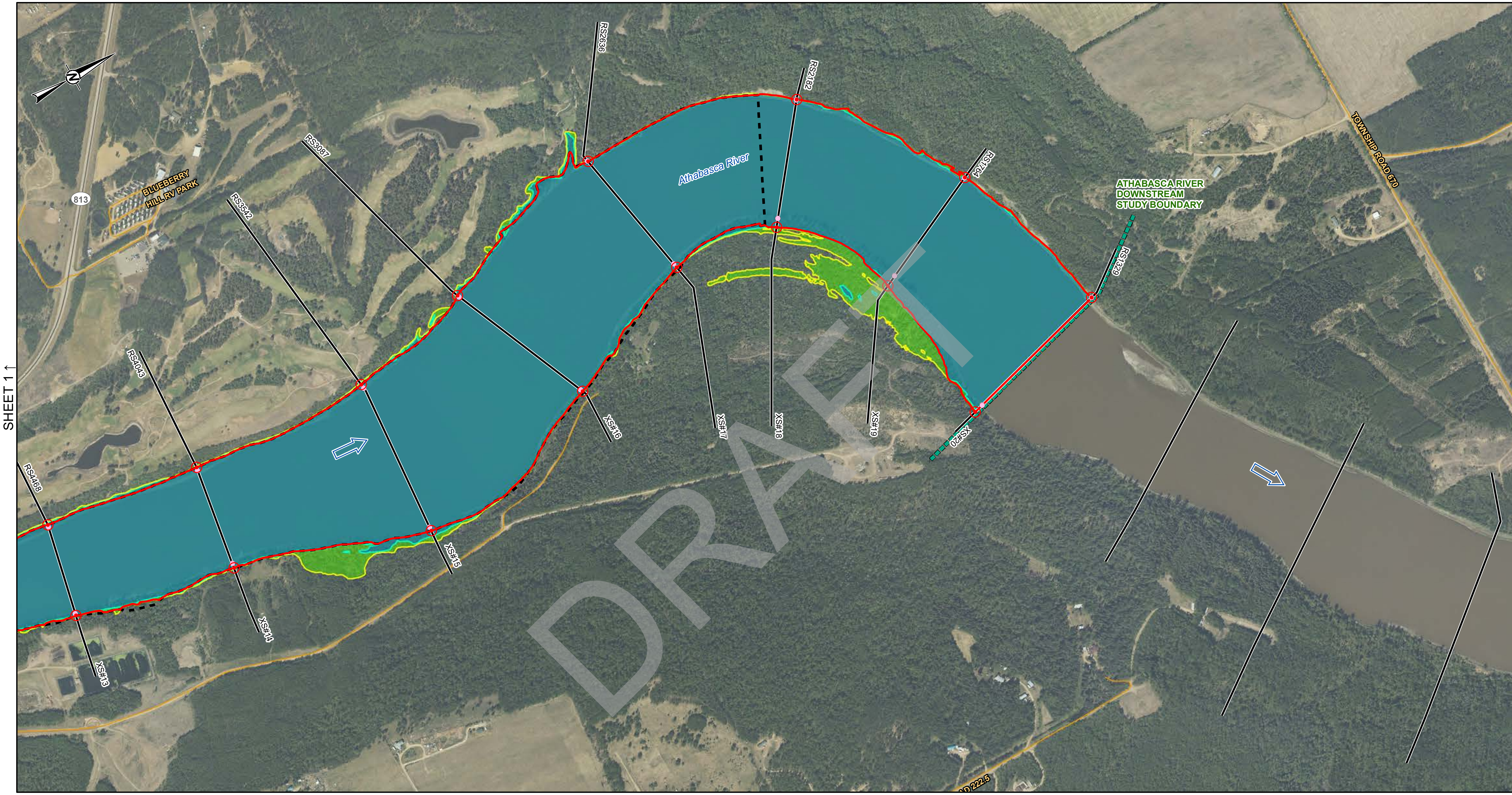
APPROVED	WP

TITLE
ICE JAM FLOODWAY CRITERIA MAP

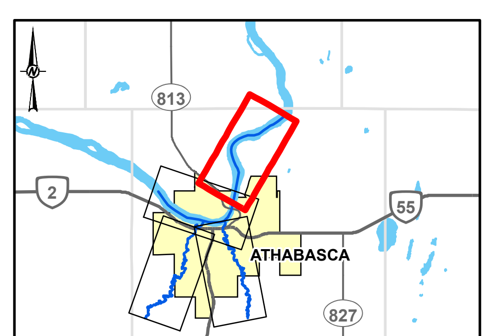
PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 1 OF 4

Classification: Public



LEGEND

CROSS SECTION	HYDRAULIC STRUCTURES	PROPOSED FLOODWAY BOUNDARY	
XS#10 CROSS SECTION NUMBER	CULVERT	BANK STATION	
RS 4994 RIVER STATION (M)	BRIDGE	PROPOSED ICE JAM FLOODWAY STATION	
MAPPING BOUNDARY		PREVIOUS FLOODWAY	
FLOW DIRECTION		DEPTH ≥ 1 M	
LOCAL ROAD		100-YEAR DESIGN FLOOD EXTENT	
PRIMARY HIGHWAY			
SECONDARY HIGHWAY			



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Alberta

PROJECT
ATHABASCA FLOOD HAZARD STUDY

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2020-03-31
DESIGNED	TT
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APPROVED	WP

TITLE
ICE JAM FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 2 OF 4

25mm
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Classification: Public

SHEET 4 ↑

SHEET 1 ←

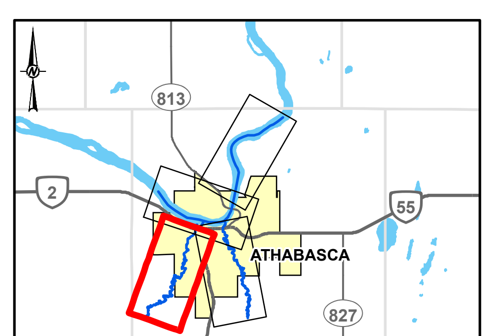
LEGEND

	CROSS SECTION
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED ICE JAM FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT



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TITLE
ICE JAM FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 3 OF 4

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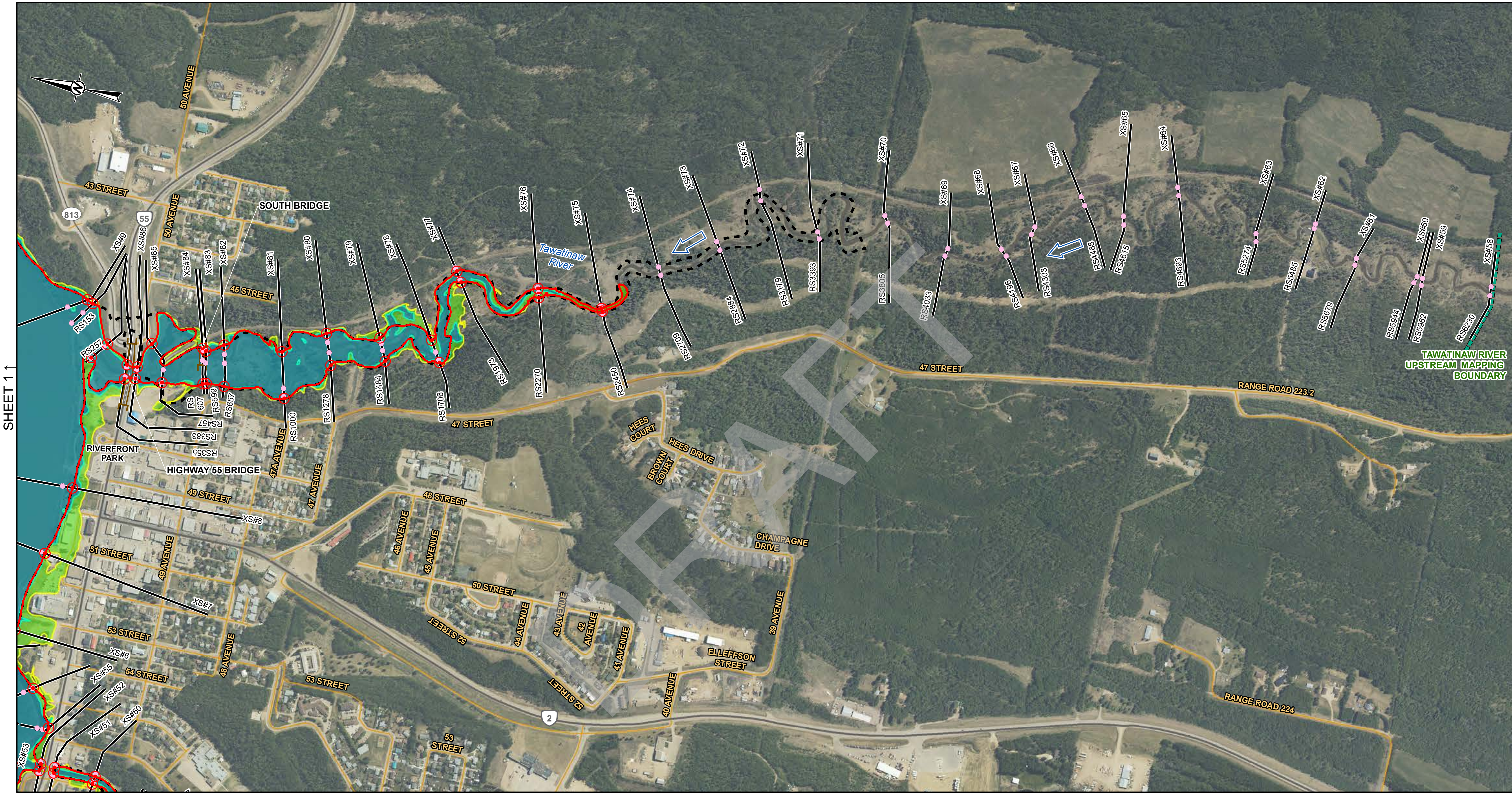


SHEET 1 ↑

SHEET 3 ↓

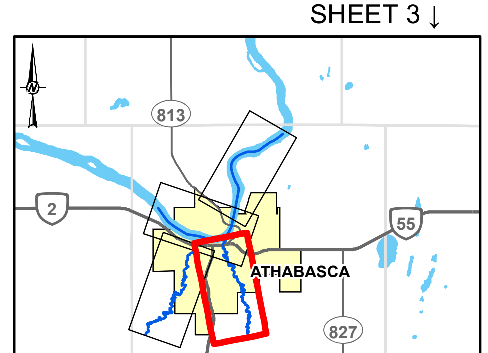
LEGEND

- CROSS SECTION
- XS#10 CROSS SECTION NUMBER
- RS 4994 RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED ICE JAM FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT



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APPROVED	WP

PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
ICE JAM FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 4 OF 4

APPENDIX L

Governing Design Flood Hazard Mapping

SHEET 2 ↑

SHEET 3 & 4 ↓

LEGEND

—	CROSS SECTION
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

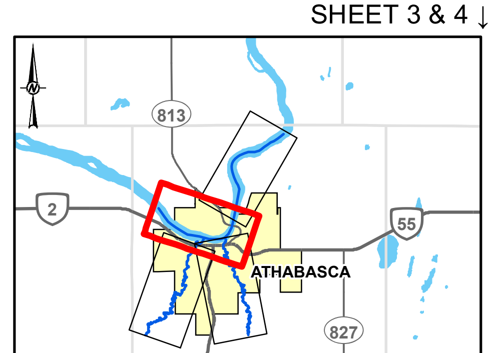
HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

DISCHARGE

ATHABASCA RIVER – ICE JAM GOVERNS
MUSKEG CREEK = 36.7 M^3/S
TAWATINAW RIVER = 73.1 M^3/S



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PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
GOVERNING DESIGN FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 1 OF 4

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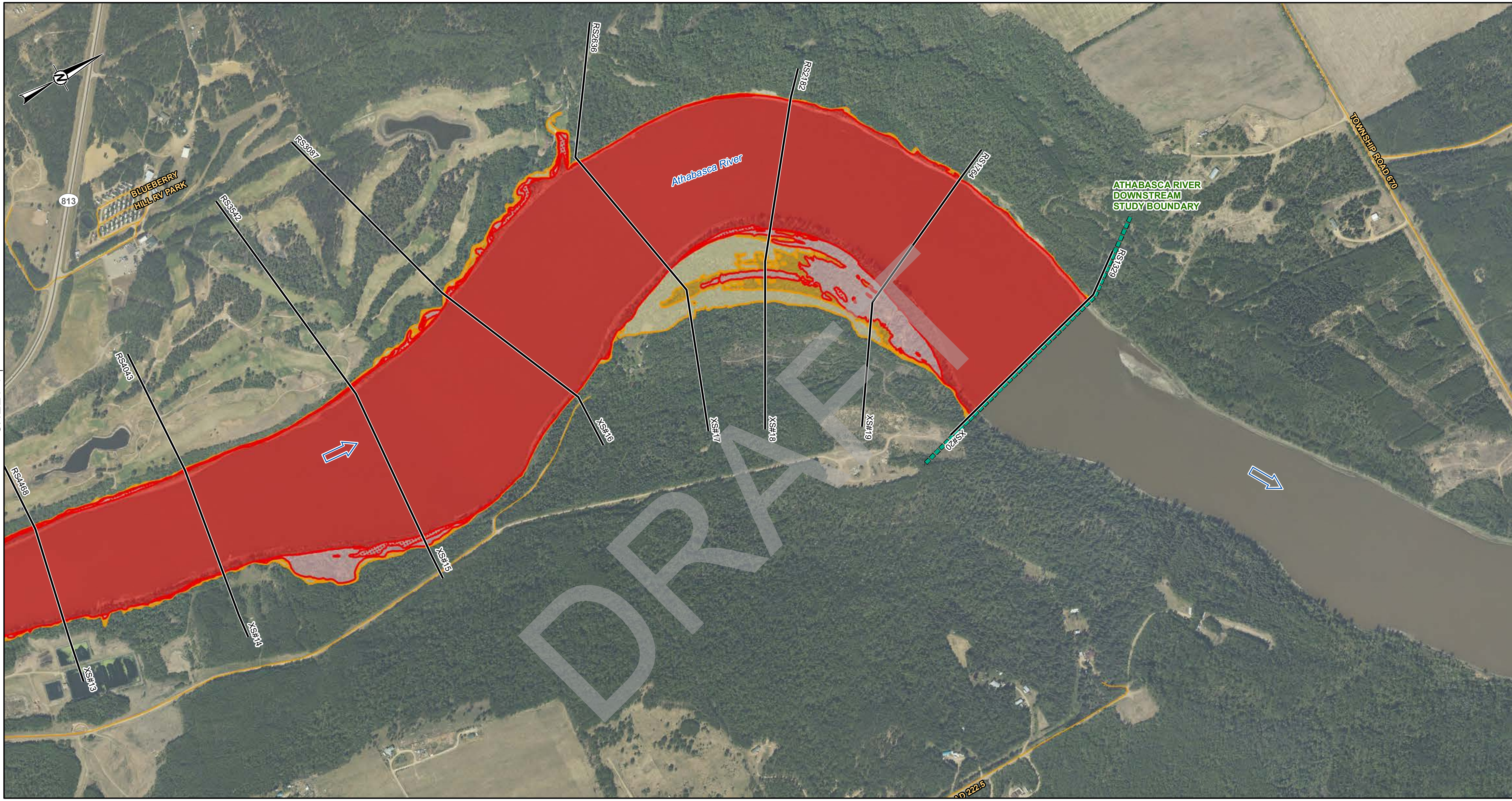
Classification: Public



SHEET 1

LEGEND

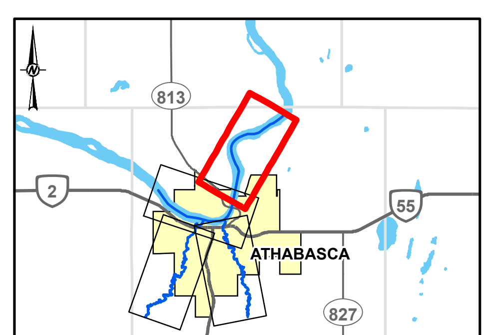
CROSS SECTION
XS#10 CROSS SECTION NUMBER
RS 4994 RIVER STATION (M)
MAPPING BOUNDARY
FLOW DIRECTION
LOCAL ROAD
PRIMARY HIGHWAY
SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

CULVERT
BRIDGE

FLOODWAY
HIGH HAZARD FLOOD FRINGE
FLOOD FRINGE
200-YEAR FLOOD EXTENT
500-YEAR FLOOD EXTENT

DISCHARGE
ATHABASCA RIVER – ICE JAM GOVERNS



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PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
GOVERNING DESIGN FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 2 OF 4

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Classification: Public

SHEET 4 ↑

SHEET 1 ↑

LEGEND

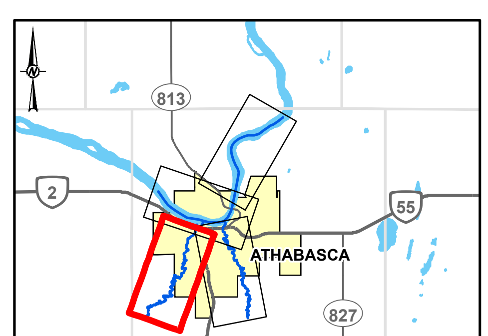
CROSS SECTION	
XS#10	CROSS SECTION NUMBER
RS 4994	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

DISCHARGE
MUSKEG CREEK = 36.7 M^3/S



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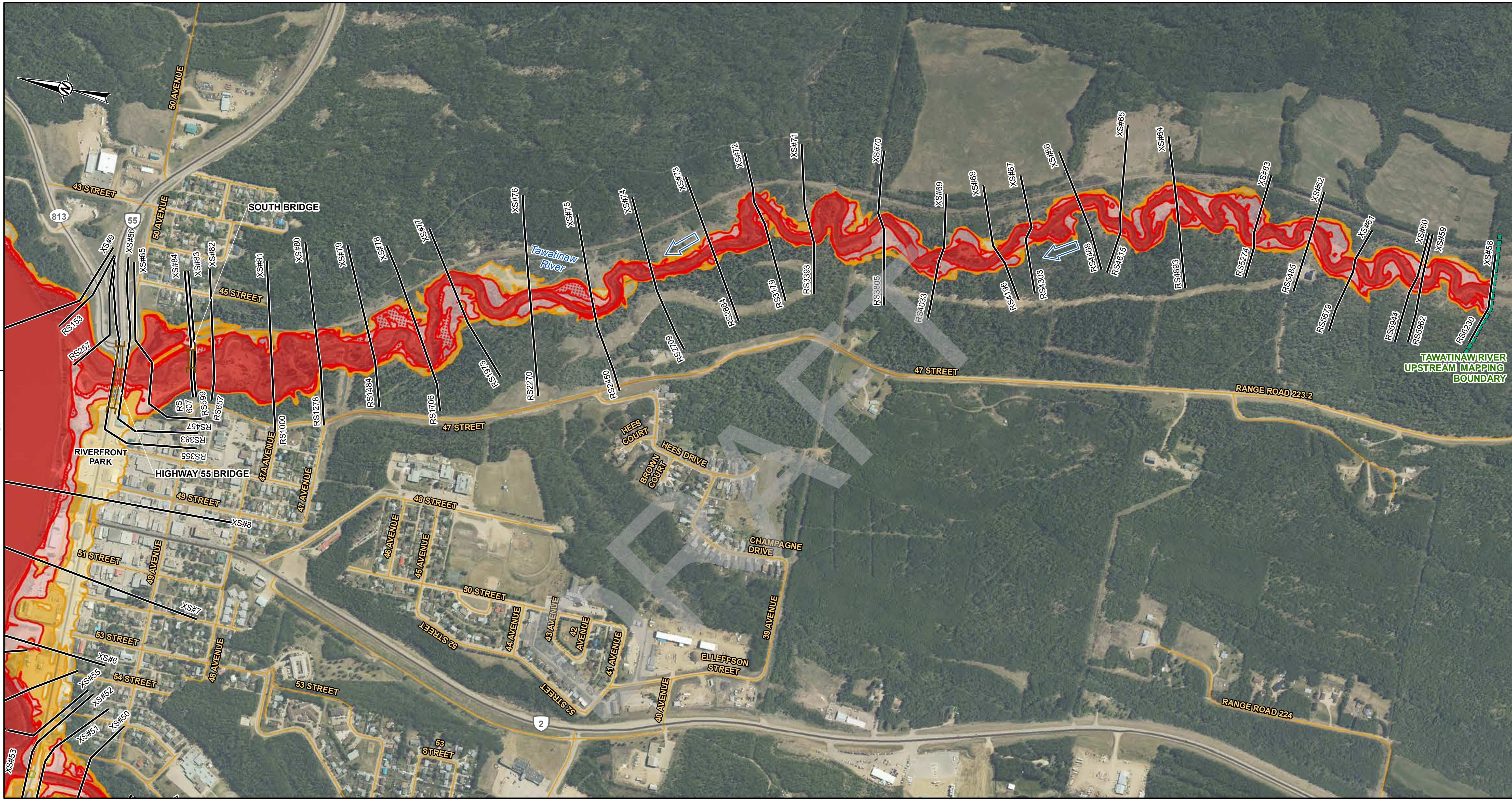
PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
GOVERNING DESIGN FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 3 OF 4



SHEET 3 ↓

LEGEND

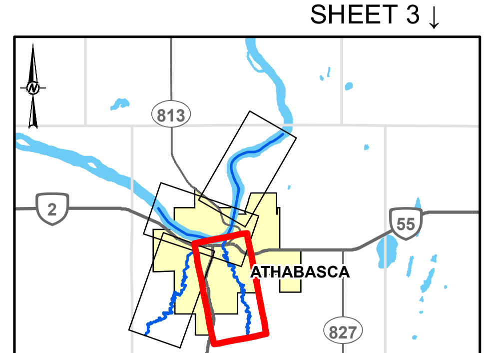
- CROSS SECTION
- XS#10 CROSS SECTION NUMBER
- RS 4994 RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- CULVERT
- BRIDGE

- FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

DISCHARGE
ATHABASCA RIVER – ICE JAM GOVERNS
TAWATINAW RIVER = 73.1 M^3/S



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PROJECT
ATHABASCA FLOOD HAZARD STUDY

TITLE
GOVERNING DESIGN FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117524	7000	0	SHEET 4 OF 4

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APPENDIX M

Open Water Climate Change Sensitivity Analysis

Figure M-1: Simulated Water Surface Profiles along the Athabasca River Study Reach due to Climate Change

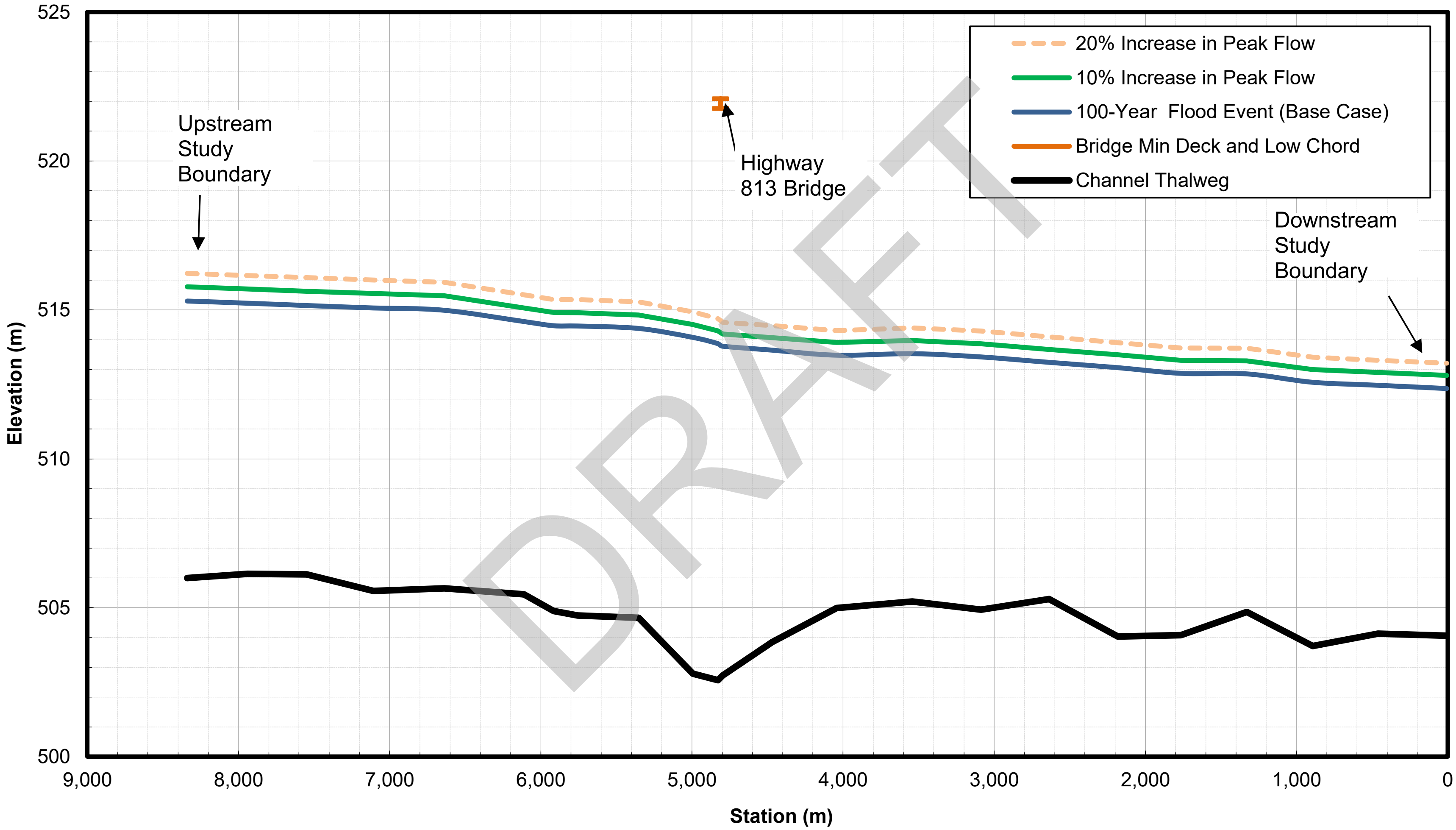

Figure M-2: Simulated Water Surface Profiles along the Muskeg Creek Study Reach due to Climate Change

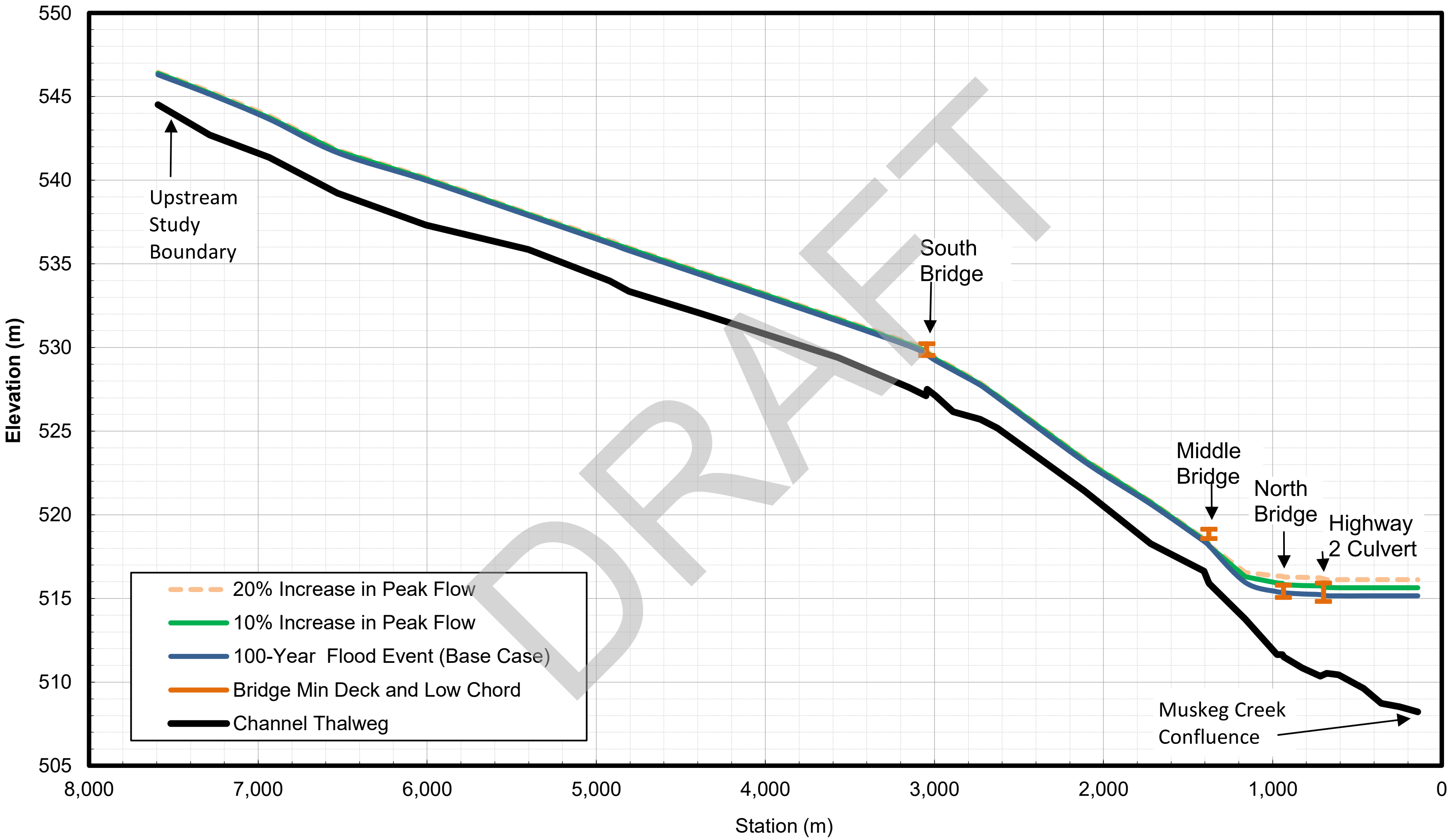

Figure M-3: Simulated Water Surface Profiles along the Tawatinaw River Study Reach due to Climate Change

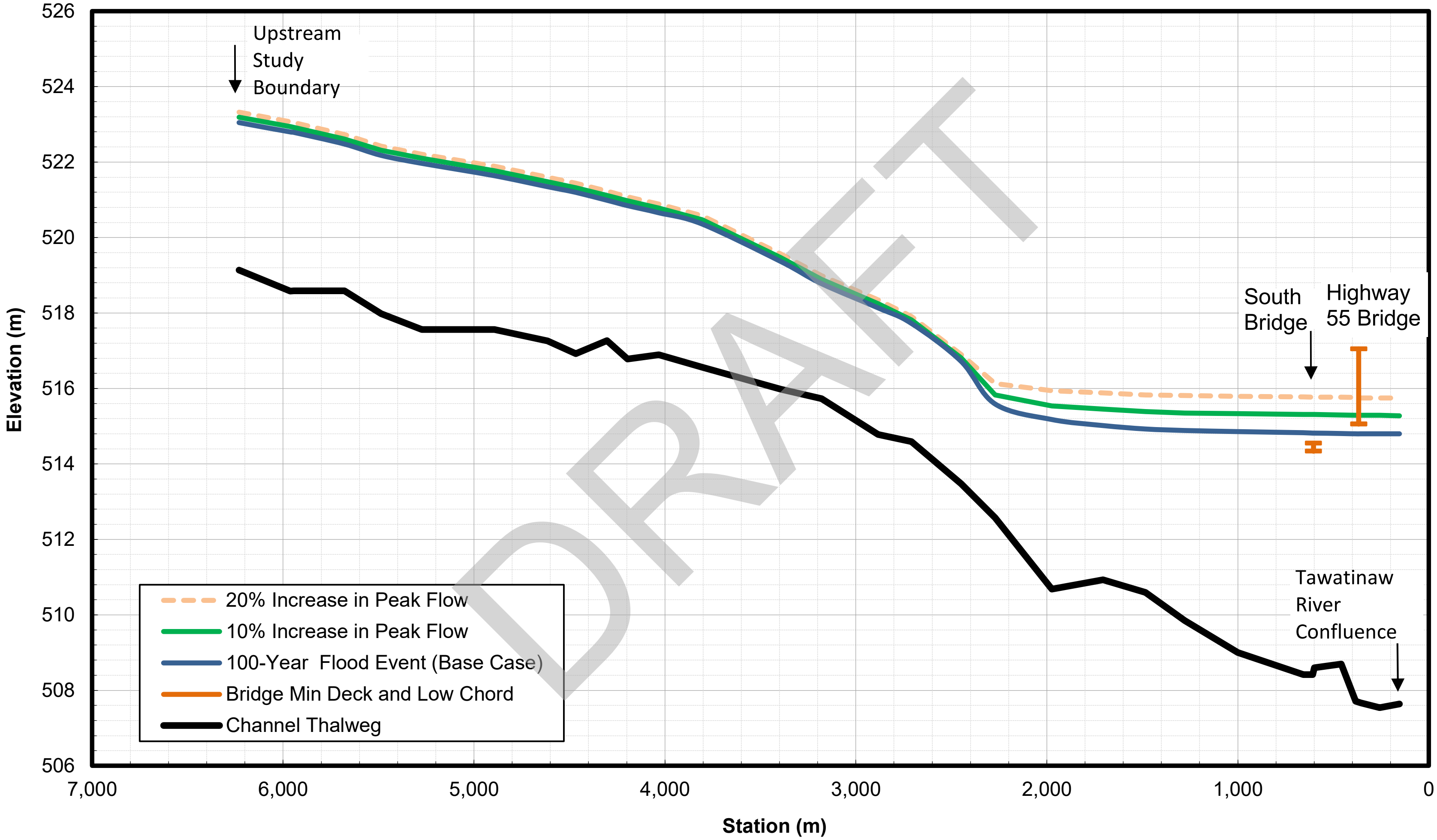

Table M-1: Water Level Difference along the Athabasca River due to Climate Change

River	Cross Section	River Station	Water Level for 100-Year (Base Case) (m)	Water Level for 10% Increase in Peak Flow (m)	Water Level for 20% Increase in Peak Flow (m)	Difference due to +10% increase in Peak Flow (m)	Difference due to +20% increase in Peak Flow (m)
Athabasca River	1	8340	515.30	515.77	516.22	0.47	0.92
Athabasca River	2	7941	515.23	515.70	516.15	0.47	0.92
Athabasca River	3	7550	515.15	515.62	516.08	0.47	0.93
Athabasca River	4	7109	515.07	515.55	516.00	0.48	0.93
Athabasca River	5	6640	514.99	515.47	515.92	0.48	0.93
Athabasca River	6	6112	514.61	515.07	515.51	0.46	0.90
Athabasca River	7	5915	514.47	514.92	515.35	0.45	0.88
Athabasca River	8	5756	514.46	514.91	515.35	0.45	0.89
Athabasca River	9	5352	514.38	514.83	515.27	0.45	0.89
Athabasca River	10	4994	514.08	514.51	514.93	0.43	0.85
Athabasca River	11	4829	513.87	514.30	514.70	0.43	0.83
Athabasca River	12	4796	513.78	514.19	514.58	0.41	0.80
Athabasca River	13	4468	513.65	514.07	514.47	0.42	0.82
Athabasca River	14	4043	513.48	513.91	514.31	0.43	0.83
Athabasca River	15	3542	513.53	513.97	514.39	0.44	0.86
Athabasca River	16	3087	513.42	513.86	514.29	0.44	0.87
Athabasca River	17	2636	513.24	513.67	514.09	0.43	0.85
Athabasca River	18	2182	513.06	513.49	513.90	0.43	0.84
Athabasca River	19	1764	512.87	513.31	513.72	0.44	0.85
Athabasca River	20	1329	512.85	513.29	513.71	0.44	0.86
Athabasca River	21	892	512.57	513.00	513.41	0.43	0.84
Athabasca River	22	461	512.47	512.90	513.31	0.43	0.84
Athabasca River	23	6	512.36	512.80	513.21	0.44	0.85

Table M-2: Water Level Difference along the Muskeg Creek due to Climate Change

River	Cross Section	River Station	Water Level for 100-Year (Base Case) (m)	Water Level for 10% Increase in Peak Flow (m)	Water Level for 20% Increase in Peak Flow (m)	Difference due to +10% increase in Peak Flow (m)	Difference due to +20% increase in Peak Flow (m)
Muskeg Creek	24	7594	546.31	546.38	546.45	0.07	0.14
Muskeg Creek	25	7286	545.14	545.21	545.31	0.07	0.17
Muskeg Creek	26	6935	543.65	543.74	543.85	0.09	0.20
Muskeg Creek	27	6532	541.65	541.74	541.81	0.09	0.16
Muskeg Creek	28	6009	540.00	540.08	540.15	0.08	0.15
Muskeg Creek	29	5401	537.89	537.96	538.02	0.07	0.13
Muskeg Creek	30	4921	536.22	536.30	536.39	0.08	0.17
Muskeg Creek	31	4806	535.79	535.88	535.97	0.09	0.18
Muskeg Creek	32	4382	534.35	534.45	534.54	0.10	0.19
Muskeg Creek	33	3571	531.58	531.66	531.75	0.08	0.17
Muskeg Creek	34	3153	530.10	530.19	530.29	0.09	0.19
Muskeg Creek	35	3049	529.64	529.78	529.93	0.14	0.29
Muskeg Creek	36	3043	529.56	529.66	529.76	0.10	0.20
Muskeg Creek	37	2993	529.22	529.30	529.38	0.08	0.16
Muskeg Creek	38	2891	528.65	528.74	528.82	0.09	0.17
Muskeg Creek	39	2728	527.75	527.82	527.90	0.07	0.15
Muskeg Creek	40	2628	527.02	527.09	527.14	0.07	0.12
Muskeg Creek	41	2116	523.20	523.27	523.35	0.07	0.15
Muskeg Creek	42	1722	520.65	520.74	520.81	0.09	0.16
Muskeg Creek	43	1406	518.43	518.49	518.57	0.06	0.14
Muskeg Creek	44	1383	518.24	518.27	518.33	0.03	0.09
Muskeg Creek	45	1374	518.14	518.13	518.17	-0.01	0.03
Muskeg Creek	46	1162	515.97	516.30	516.56	0.33	0.59
Muskeg Creek	47	973	515.41	515.93	516.35	0.52	0.94

Table M-2: Water Level Difference along the Muskeg Creek due to Climate Change

River	Cross Section	River Station	Water Level for 100-Year (Base Case) (m)	Water Level for 10% Increase in Peak Flow (m)	Water Level for 20% Increase in Peak Flow (m)	Difference due to +10% increase in Peak Flow (m)	Difference due to +20% increase in Peak Flow (m)
Muskeg Creek	48	943	515.38	515.90	516.33	0.52	0.95
Muskeg Creek	49	934	515.34	515.82	516.29	0.48	0.95
Muskeg Creek	50	825	515.28	515.78	516.26	0.50	0.98
Muskeg Creek	51	718	515.24	515.75	516.23	0.51	0.99
Muskeg Creek	52	680	515.17	515.66	516.12	0.49	0.95
Muskeg Creek	53	610	515.16	515.65	516.12	0.49	0.96
Muskeg Creek	54	463	515.16	515.65	516.12	0.49	0.96
Muskeg Creek	55	357	515.16	515.65	516.12	0.49	0.96
Muskeg Creek	56	249	515.16	515.65	516.12	0.49	0.96
Muskeg Creek	57	141	515.16	515.65	516.12	0.49	0.96

Table M-3: Water Level Difference along the Tawatinaw River due to Climate Change

River	Cross Section	River Station	Water Level for 100-Year (Base Case) (m)	Water Level for 10% Increase in Peak Flow (m)	Water Level for 20% Increase in Peak Flow (m)	Difference due to +10% increase in Peak Flow (m)	Difference due to +20% increase in Peak Flow (m)
Tawatinaw River	58	6230	523.05	523.19	523.32	0.14	0.27
Tawatinaw River	59	5962	522.80	522.94	523.07	0.14	0.27
Tawatinaw River	60	5944	522.79	522.92	523.05	0.13	0.26
Tawatinaw River	61	5679	522.48	522.61	522.73	0.13	0.25
Tawatinaw River	62	5485	522.18	522.31	522.42	0.13	0.24
Tawatinaw River	63	5274	521.97	522.10	522.21	0.13	0.24
Tawatinaw River	64	4893	521.64	521.77	521.89	0.13	0.25
Tawatinaw River	65	4615	521.35	521.48	521.60	0.13	0.25
Tawatinaw River	66	4468	521.20	521.32	521.44	0.12	0.24
Tawatinaw River	67	4303	520.99	521.11	521.23	0.12	0.24
Tawatinaw River	68	4196	520.85	520.97	521.08	0.12	0.23
Tawatinaw River	69	4033	520.66	520.78	520.89	0.12	0.23
Tawatinaw River	70	3805	520.36	520.47	520.58	0.11	0.22
Tawatinaw River	71	3393	519.37	519.47	519.57	0.10	0.20
Tawatinaw River	72	3179	518.78	518.89	518.99	0.11	0.21
Tawatinaw River	73	2884	518.14	518.25	518.35	0.11	0.21
Tawatinaw River	74	2709	517.73	517.82	517.91	0.09	0.18
Tawatinaw River	75	2450	516.73	516.81	516.90	0.08	0.17
Tawatinaw River	76	2270	515.59	515.83	516.13	0.24	0.54
Tawatinaw River	77	1973	515.18	515.54	515.94	0.36	0.76
Tawatinaw River	78	1706	515.02	515.45	515.88	0.43	0.86
Tawatinaw River	79	1484	514.93	515.39	515.83	0.46	0.90
Tawatinaw River	80	1278	514.89	515.35	515.81	0.46	0.92
Tawatinaw River	81	1000	514.86	515.33	515.79	0.47	0.93

Table M-3: Water Level Difference along the Tawatinaw River due to Climate Change

River	Cross Section	River Station	Water Level for 100-Year (Base Case) (m)	Water Level for 10% Increase in Peak Flow (m)	Water Level for 20% Increase in Peak Flow (m)	Difference due to +10% increase in Peak Flow (m)	Difference due to +20% increase in Peak Flow (m)
Tawatinaw River	82	657	514.83	515.31	515.78	0.48	0.95
Tawatinaw River	83	607	514.82	515.31	515.77	0.49	0.95
Tawatinaw River	84	599	514.82	515.31	515.77	0.49	0.95
Tawatinaw River	85	457	514.81	515.30	515.77	0.49	0.96
Tawatinaw River	86	383	514.80	515.29	515.76	0.49	0.96
Tawatinaw River	87	355	514.80	515.29	515.75	0.49	0.95
Tawatinaw River	88	257	514.80	515.29	515.75	0.49	0.95
Tawatinaw River	89	153	514.80	515.28	515.75	0.48	0.95

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