REPORT

Rochester Flood Study

Study Summary Report

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Executive Summary

Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in September 2020 to conduct the Rochester Flood Study (the study). The purpose of the study is to assess and identify river and flood hazards along the Tawatinaw River through the Hamlet of Rochester 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, Athabasca County, and the general public.

This report documents the methodology and results for all components of the study which are listed below:

- Survey and base data collection;
- Open water hydrology assessment;
- Open water hydraulic modelling;
- Open water flood inundation mapping; and
- Design flood hazard mapping.

The total length of the Rochester River study reach is approximately 7.5 km. An additional reach of approximately 1 km was included in the HEC-RAS hydraulic model beyond the downstream end of the required study reach to enable specification of reliable downstream boundary conditions and to account for backwater effect from the downstream boundary.

An open water survey was conducted in October 2020 to collect most of the bathymetric data and hydraulic structures data for model setup. Two surveys were performed in December 2020 and January 2022 under ice covered condition to collect the data from the remaining cross sections which were not accessible under open water condition.

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

An attempt was made to calibrate the HEC-RAS model based on the measured low flow water levels and discharge collected during the October 2020 survey. The calibration was not successful because of the dominant effects of the beaver dams, not channel roughness, on the low flow water levels. Attempts were made without any success to collect HWM and anecdotal flood information from the local community, Alberta Transportation and Athabasca County.

Therefore, the channel Manning's *n* value was estimated without any calibration. The selected channel Manning's *n* value is 0.050, which is deemed to account for the potential beaver dam effects on the flood levels. This value is within the typical range of roughness values for similar streams (Chow 1959).

The Manning's *n* values for the floodplain areas were estimated and selected based on the land use types.

The HEC-RAS model was used to simulate 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 for the 100-year flood event. The results of the sensitivity analysis show that variation of the channel and floodplain roughness values has small influence on the simulated water levels along the Tawatinaw River study reach. The changes of the energy slope at the downstream boundary could influence the upstream simulated flood levels up to a distance of 4.0 km immediately upstream of the downstream boundary.

Flood inundation and hazard maps were prepared for the study reach of 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 inundations areas were mapped where there is a direct connection between the main river 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 are no flood control structures in the study area, no areas of potential flooding behind such structures were identified.

Based on the simulation results, portion of a farmhouse property within the study reach would be affected starting at the 5-year flood. No commercial areas in Hamlet of Rochester would be inundated.

The floodway was defined based on the 1 m depth and 1 m/s velocity criteria and the previous floodway. The results of the design flood hazard mapping are the delineation of floodway and flood fringe zones including high hazard flood fringe areas. Based on the flood hazard maps, no residences or key structures are situated within the floodway and high hazard flood fringe zones along the Tawatinaw River study reach. A portion of a farmhouse and Township Road 623.8 are within the flood fringe zone.

Acknowledgements

Golder Associates Ltd. (Golder) acknowledges the contributions of the following staff of Alberta Environment and Parks (AEP):

- Mr. Muhammad Durrani, AEP's project manager for the study, coordinated the participation from AEP, and provided technical advice and review of this report.
- Mr. Abdullah Mamun, AEP's project manager for the study, coordinated the participation from AEP for this study during hydraulic model setup and flood inundation mapping.
- Mr. Peter Onyshko, AEP's technical advisor for the study, provided technical review and guidance.

The contributions of the following staff from Golder are acknowledged:

- Mr. Jie Chen, Golder's supporting project manager, was responsible for regular communications with AEP and overseeing the HEC-RAS modelling, flood inundation mapping, flood hazard mapping as well as preparation of this report.
- Dr. Hua Zhang, Golder's project manager, was responsible for overseeing HEC-RAS modelling, flood hazard mapping as well as preparation of this report.
- Dr. Dejiang Long, senior advisor and reviewer for this study, was responsible for providing senior inputs and review, quality control and assurance for the study, and reviewing this report.
- Dr. Wolf Ploeger, project director and senior reviewer for this study, was responsible for providing senior inputs and review, quality control and assurance for the study, and reviewing this report.
- Dr. Getu Biftu, senior hydrologist, was responsible for open water hydrology assessment.
- Mr. Amir Gharavi, a hydrodynamic modelling support, was involved in performing field surveys and conducting HEC-RAS modelling.
- Ms. Nancy Guo, a hydrodynamic modelling support, was involved in preparing floodway criteria maps and flood hazard maps to this report.
- Mr. Sean Kurash, a GIS specialist, was responsible for preparing the flood inundation maps and flood hazard maps to this report.
- Mr. Carmen Orosz, field survey lead for this study, was responsible for field survey and hydraulic structure data collection.

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1.0 INTRODUCTION

1.1 Study Background

Golder Associates Ltd. (Golder) was commissioned by Alberta Environment and Parks (AEP) in September 2020 to undertake the Rochester Flood Study (the study). The primary purpose of the study is to assess and identify river and flood hazards in the vicinity of the Hamlet of Rochester (Rochester) along a 7.5 km long reach of the Tawatinaw River.

The study was 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, Athabasca County, and the general public.

The previous provincial flood study for Rochester was completed by UMA Engineering Ltd (UMA) in 1997 (UMA, 1997). This study will replace the previous study, and expand the modelling and flood mapping coverage for open water flood scenarios.

This 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. Open Water Flood Inundation Mapping
5. 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 five study components. The primary tasks, services, and deliverables associated with this study are listed below:

- river cross-section surveys
- hydraulic structure data collection
- flood history documentation
- open water flood hydrology assessment
- HEC-RAS hydraulic model creation for open water modelling
- floods simulations, water surface profile creation, and sensitivity analysis
- open water flood inundation maps production
- floodway criteria and flood hazard mapping

1.3 Study Area

An overview of the study area is provided in Figure 1-1. The study area includes the 7.5 km long Tawatinaw River reach, extending from the north edge of NE-12-62-24-4, through Rochester, to the south edge of NW-30-62-23-W4M. An additional 1 km length of the study reach was included in HEC-RAS hydraulic model beyond the downstream end of the required study reach to appropriately account for the downstream backwater effect on the water levels in the study area. The downstream boundary of the hydraulic model terminates on the Tawatinaw River at a distance of approximate 4 km downstream of Rochester.

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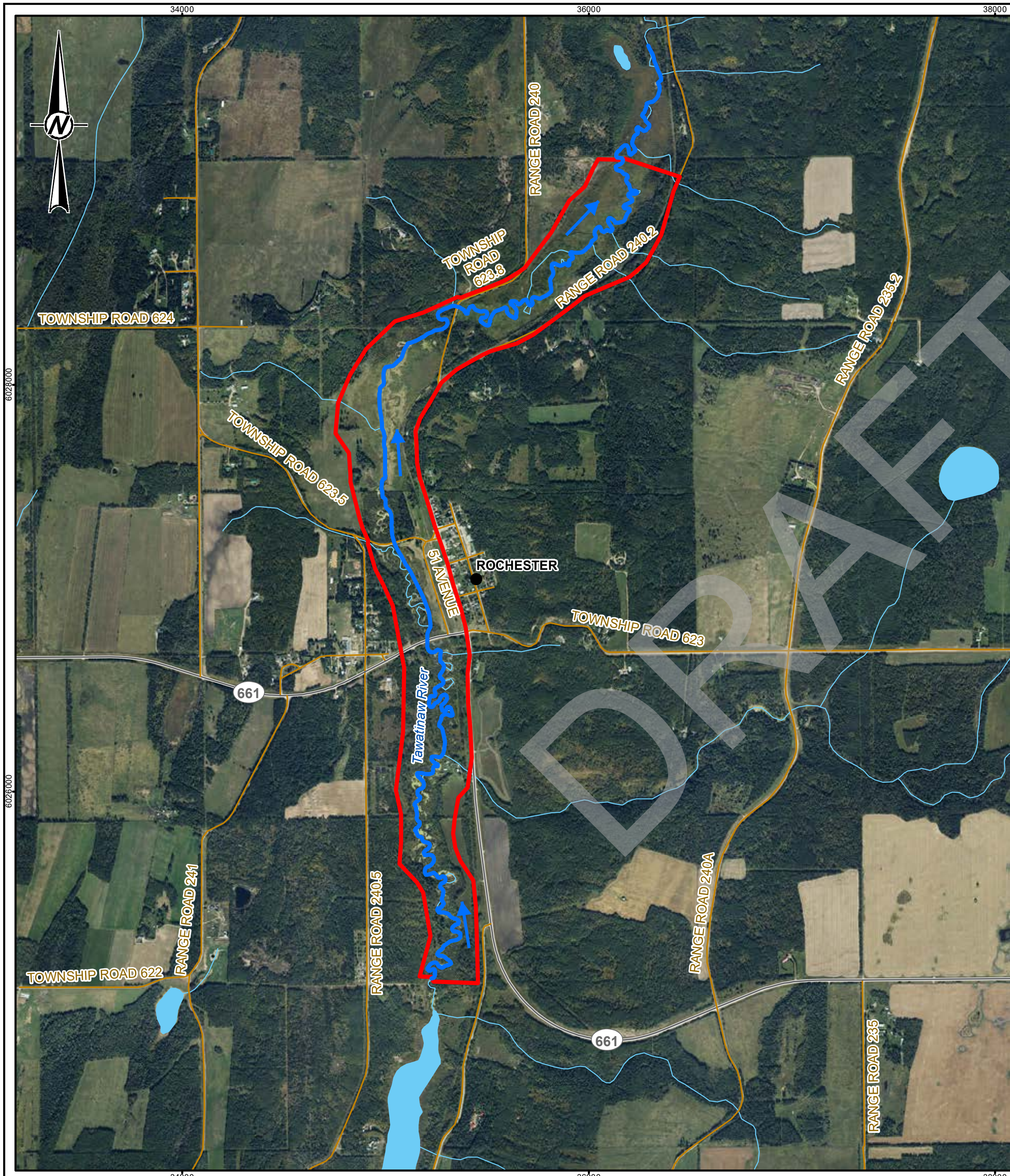

LEGEND

- POPULATED PLACE
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- SURVEY REACH
- FLOOD HAZARD STUDY AREA

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1:30,000 KILOMETRES

REFERENCE(S)

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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

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ALBERTA ENVIRONMENT AND PARKS

PROJECT
ROCHESTER FLOOD STUDY

TITLE
LOCATION MAP OF THE STUDY AREA

CONSULTANT

GOLDER MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	NB
REVIEWED	JC
APPROVED	HZ

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2.0 SURVEY AND BASE DATA COLLECTION

2.1 General

Golder conducted surveys of the Tawatinaw River study reach during three separate periods. The first survey was conducted during open water condition, and the second and third surveys were conducted during ice-cover conditions. The first survey was conducted from October 10 to 15, 2020. The survey data was collected at 33 cross sections and four hydraulic structures. The survey of the other planned cross sections could not be completed during the open water season because of unsafe conditions for site access. The second survey was conducted at 19 cross sections on December 10, 2020. The third survey was attempted at the remaining 3 cross sections on January 21, 2022, but the unsafe site and unfavourable weather conditions did not permit completion of the planned survey.

The survey scope included the following:

- survey of channel cross sections and hydraulic structures
- survey of flood control structures
- measurement of discharge and water surface profile

In addition, one Alberta Survey Control Monument (ASCM) was surveyed upon the request of AEP in support of Light Detection and Ranging (LiDAR) remote sensing data collection (by others), for confirming 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 representatives from AEP and Golder on October 9, 2020. The field visits involved the following:

- Reviewed and confirmed the preliminary survey plan.
- Confirmed the locations and numbers of channel cross sections and hydraulic structures to be surveyed.
- Identified potential flood control structures.
- Familiarized with the study area.

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 bed levels in areas where hydraulic conditions allowed the surveyors to wade the channel and walk on the banks. The RTK system was also used to survey the following:
  - Control points and benchmarks within the study area.
  - Bridge and culvert structures.

- Acoustic Doppler Velocimeter (ADV): A SonTek FlowTracker2® ADV in combination with a top-set wading rod was used to conduct discharge measurements on the Tawatinaw River.

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

All surveyed points were acquired by wading the channel and 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 in Figure 2-1. It includes a complete list of survey codes for the RTK and total station.

The data collected using typical ground-based and acoustic-based technologies were referenced to the ASCM benchmark (i.e., ASCM 263152) situated within the study area. The ASCM 263152 was also used for calibration of the collected survey data.

A local benchmark was established in the Rochester Park at the beginning of the survey. The survey crew checked the data accuracy at the local benchmark at the start and end of each survey day.

All survey data was collected in the local 3-Degree Transverse Mercator (3TM) 114° W coordinate system and referenced to NAD83 (CSRS) horizontal and the CGVD28 vertical datums. The RTK 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 reach. Considerations of changes in the channel width, cross section area, channel slope, 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 flow, as anticipated under high flow conditions. A shapefile showing the alignment of each cross section was provided to the survey crew and uploaded to the data collectors to provide guidance where to acquire data.

Each survey point collected with the RTK utilized a schematic of survey point codes and corresponding locations as shown in Figure 2-1. It also includes a complete list of survey codes for the RTK.

The quality and accuracy of all survey data were checked by using a Trimble data extraction and processing tool. All survey data was imported into ArcGIS to allow for validation and further processing. Data with horizontal or vertical accuracies of greater than ±0.05 m was rejected. Daily quality and accuracy checks were conducted in the office. In cases where multiple points with low accuracy were detected at a cross section, the survey crew repeated that survey the next day.

Survey Codes for RTK GPS River Surveys (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

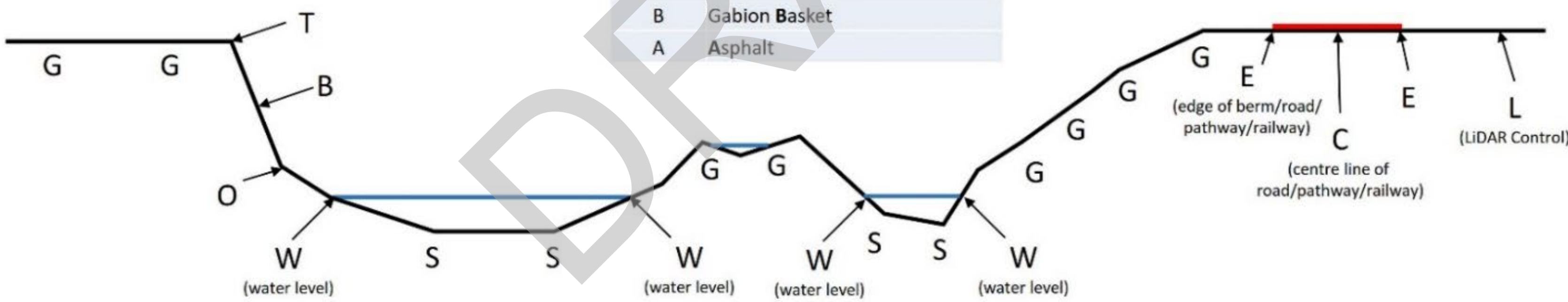


Figure 2-1: Schematic of Survey Point Locations and Code Descriptions

The Tawatinaw River were surveyed by a combination of wading and using a boat.

The main objective of the cross-section survey was to enable accurate definition of the main channel geometry. Limited overbank floodplain areas were also surveyed to overlap with the LiDAR survey 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 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 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.

Reach-representative photographs were taken at key locations within the study area during the site reconnaissance and field survey. 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 water levels along the study reach were measured during the field program to support low-flow hydraulic model calibration.

One discharge measurement on the Tawatinaw River was completed. There appeared to be no noticeable changes to the channel flow during the survey. The flow measurement was performed by wading the channel with a handheld Acoustic Doppler Velocimeter (*SonTek FlowTracker2® ADV*) and top-set wading rod in accordance with standard WSC protocols, including the following:

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

The measurement procedure involved the following:

- 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 within the study area were surveyed. Applicable structures include road bridges, pedestrian bridges, and roadway culverts.

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); and
- Top of road surface 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); and
- Top of roadway profile.

The hydraulic structures were surveyed using RTK rover unit 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 the ASCM benchmark 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 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 Tawatinaw River was approximately 9 km. An overview of the surveyed cross section locations is provided in Figures A-1 to A-3 of APPENDIX A. A total of 52 cross sections were surveyed with an average cross section spacing of 150 m.

The profiles of the surveyed main channel thalweg and measured water levels along the Tawatinaw River for the open water condition, were presented in Figure 2-2.

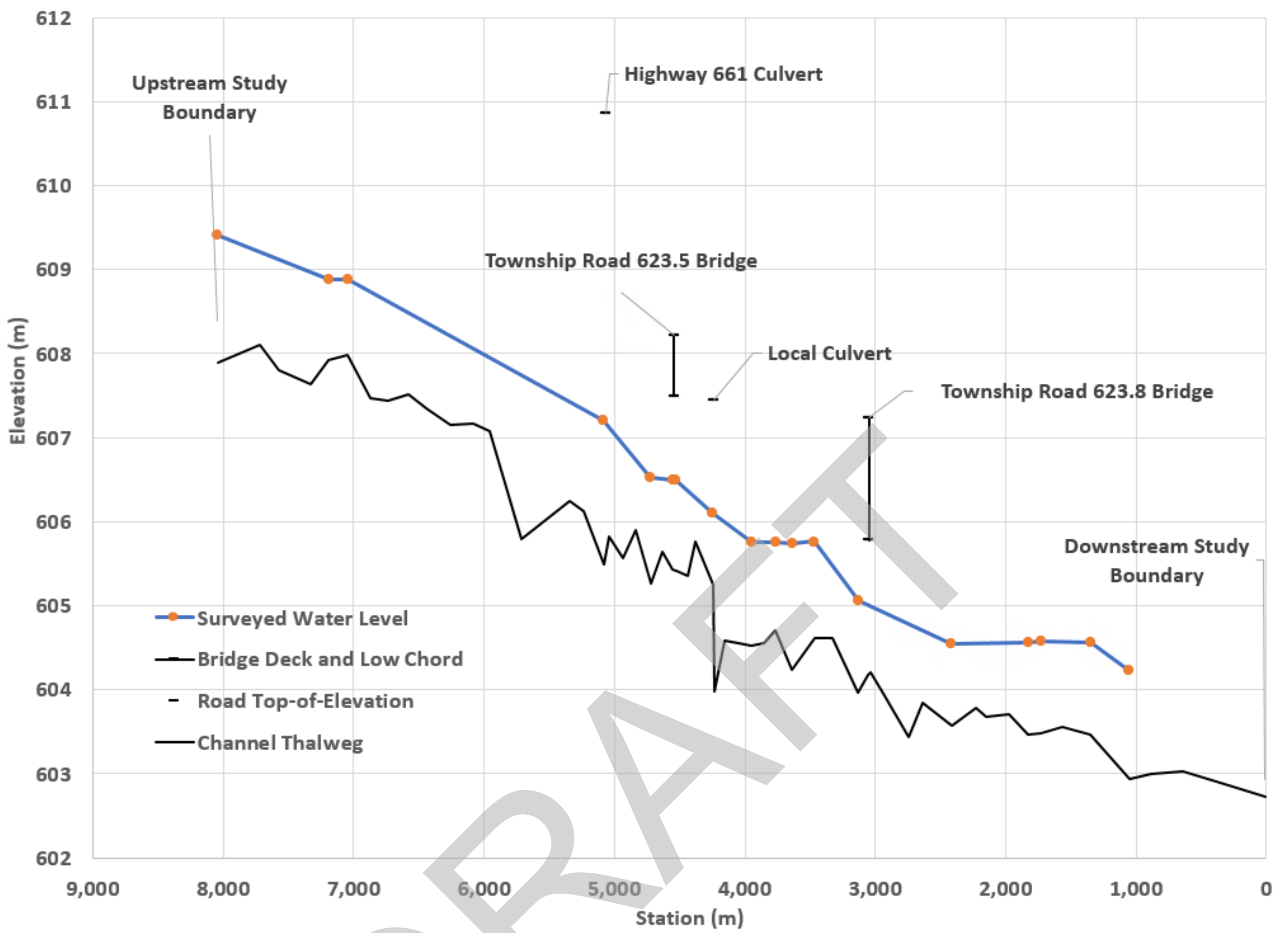


Figure 2-2 Surveyed Channel Thalweg and Surface Water Profile along the Tawatinaw River

2.5 Discharge and Water Level Measurements

One discharge measurement was made on October 12, 2020. Water levels were recorded during the cross-section surveys. Table 2-1 provides a summary of the discharge and water level measurement data.

Table 2-1: Discharge and Water Level Measurements

Waterbody	Date	Discharge Measurement Location	Water Level Measurement Locations		Measured Discharge (m^3/s)
			From Cross Section	To Cross Section	
Tawatinaw River	October 12, 2022	XS25	T1	T55	0.099

2.6 Hydraulic Structures

There are four hydraulic structures (i.e., two bridges and two culverts) in the study area. A summary of these hydraulic structures is provided in Table 2-2.

Table 2-2: Hydraulic Structures within the Study Area

Waterbody	Structure ID	Structure Name / Location	Cross Section ID	Type	No. of Spans	Corresponding Figure Number in Appendix B
Tawatinaw River	HS-01	Highway 661 Culvert	5065	Traffic	None	B-1
	HS-02	Township Road 623.5 Bridge	4542	Traffic	1	B-2
	HS-03	Local Culvert	4238	Traffic	None	B-3
	HS-04	Township Road 623.8 Bridge	3049	Traffic	1	B-4

Bridge and culvert locations are shown in Figures A-1 to A-3 of APPENDIX A. Figures The site photographs, survey data point locations superimposed onto (aerial) orthoimagery, and salient information for each hydraulic structure, are shown in Figures B-1 to B-4 of APPENDIX B.

2.7 Flood Control Structures

There was no flood control structure identified during the site visit. AEP received confirmation from the local authorities on March 1, 2021 regarding the absence of any flood-related structures within the study area.

2.8 Additional Base Data

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

- LiDAR topographic data collected by OLG Engineering (OLG) in September 2020 and provided by AEP.
- Recent orthorectified aerial imagery which was acquired by OLG in September 2020 and provided by AEP.

3.0 OPEN WATER HYDROLOGY ASSESSMENT

3.1 Overview

Documentation of a detailed open water hydrology assessment for the Tawatinaw Rive and its tributary Stony Creek, within the Hamlet of Rochester, is provided in APPENDIX C. The sections below provide a summary of that assessment.

3.2 Flooding History

3.2.1 General Information

The Tawatinaw River originates south of Rochester and flows in the northerly direction through the hamlet to the Athabasca River. It has a drainage area of approximately 445 km^2 at the Hamlet of Rochester. 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. Stony Creek is a tributary of the Tawatinaw River with its confluence at the Hamlet of Tawatinaw.

The majority of the Hamlet of Rochester is located on the east banks of the Tawatinaw River within a low terraced area. A map showing the 7.5 km reach of the Tawatinaw River through the Athabasca County, including the Hamlet of Rochester is shown in Figure 3-1

3.2.2 Open Water Flood History

The Tawatinaw River reach near the Hamlet of Rochester is not gauged, and no historic flood flow data is available. Only anecdotal information relating to approximately three significant floods between 1920 and 1950 was recalled by some local residents. They were able to recall flooding extending to the old CNR tracks and having crossed the river using boats (AEP 1997).

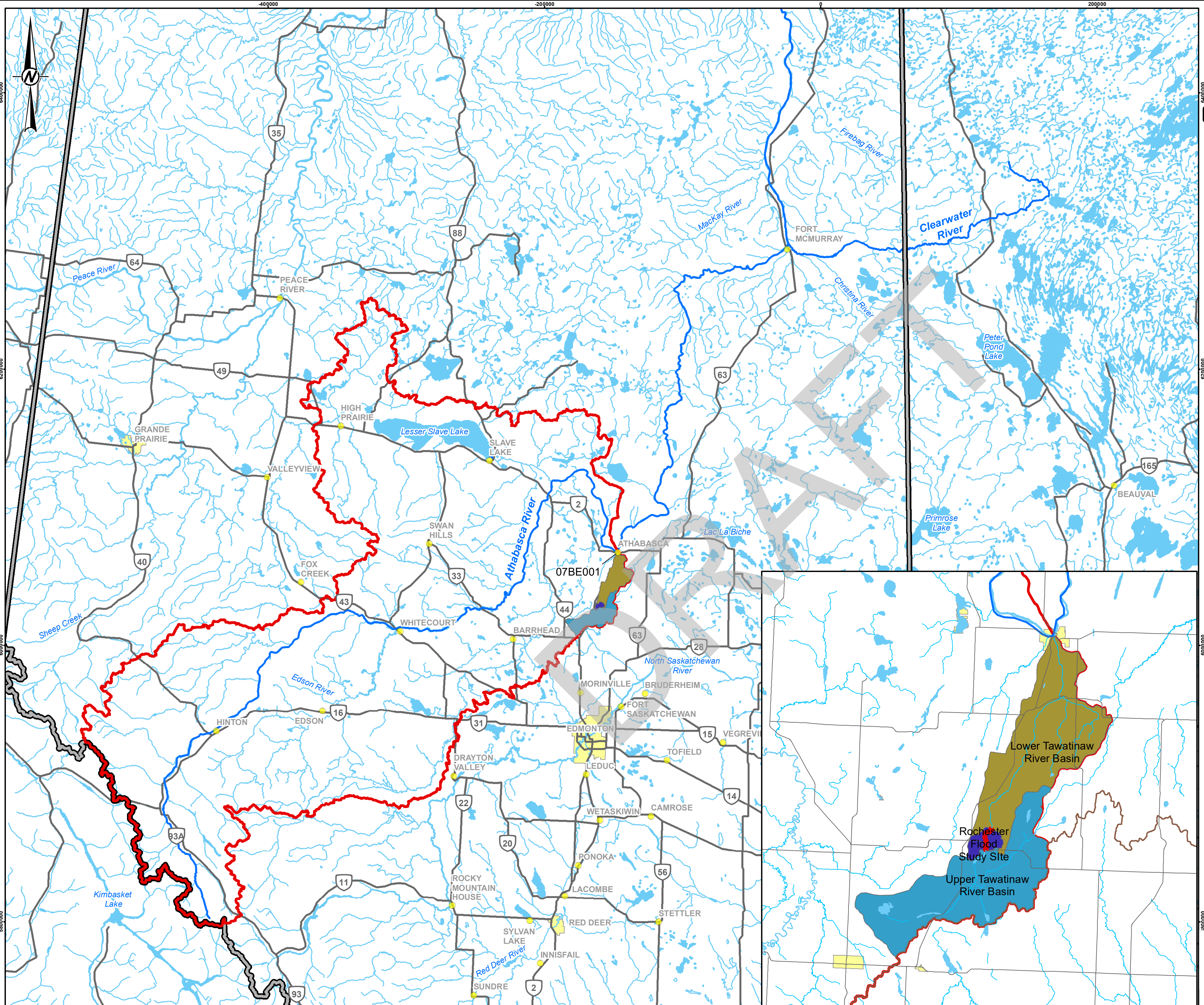
Based on the review of the regional hydrologic data, flooding in the Tawatinaw River basin could be caused by snowmelt, rainfall and snowmelt, or rainfall alone. However, the majority of the recorded annual instantaneous peak flows used for the regional analysis occurred during summer months, indicating that these floods were associated with rainfall events.

The gauging station on Stony Creek is located approximately 9 km south of the Hamlet of Rochester. The highest annual maximum instantaneous discharge in the creek was measured in 1997 and 2020. The flood frequency for the 1997 event was estimated to be in the order of 20-year return period and the 2020 event is close to 35-year return period.

Since the drainage area of Stony Creek represents about 40% of the drainage basin of the Tawatinaw River at the Hamlet of Rochester, it is reasonable to conclude that similar magnitudes of flood events also occurred in Rochester in 1997 and 2020. The 1997 flood for Stony Creek was recoded on April 21, indicating a snowmelt event or an event with combination of snowmelt and rainfall runoff. The 2020 flood for Stony Creek was recorded on June 8, indicating a rainfall event. Theses events were also recorded at the other six gauging sites considered in the regional analysis.

3.3 Open Water Flood Frequency Analysis

The flood frequency estimates for Tawatinaw River at Hamlet of Rochester were derived using the flood frequency estimates for Stony Creek near Tawatinaw and the area power factors. The area powers were developed based on the regional relationships between drainage areas and flood peak discharges using available regional flow records for various return periods from 2 to 1,000 years.



LEGEND

- MAJOR RIVER
- WATERCOURSE
- WATERBODY
- PRIMARY HIGHWAY
- POPULATED PLACE
- PROVINCIAL BOUNDARY
- Rochester_Flood_Study_Reach
- HUC8_athabasca_basin
- PRIMARY HIGHWAY

0 100 200

1:2,800,000 KILOMETRES

NOTE(S)

SUB-BASINS WERE OBTAINED FROM HUC8 WATERSHEDS (GOVERNMENT OF ALBERTA 2015) WHERE POSSIBLE. OUTSIDE OF ALBERTA, SUB-BASINS WERE OBTAINED FROM AAFC WATERSHED DATA.

REFERENCE(S)

HYDROMETRIC STATIONS OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC). BASIN AND SUB-BASIN DATA OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC) AND THE GOVERNMENT OF ALBERTA 2015.
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PROJECT
ROCHESTER FLOOD STUDY

TITLE
TAWATINAW RIVER WATERSHED AT ROCHESTER

CONSULTANT
GOLDER MEMBER OF WSP

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

Table 3-1: Recommended Flood Frequency Estimates for Tawatinaw River at Hamlet of Rochester

Return Period (years)	Annual Probability of Exceedance (%)	Value (m^3/s)	Lower 95% Limit (m^3/s)	Upper 95% limit (m^3/s)
2	50	6.9	4.8	10.1
5	20	14.7	10.7	20.0
10	10	20.9	15.2	27.1
20	5.0	27.4	19.3	34.4
35	2.9	32.9	22.2	40.5
50	2.0	36.5	23.9	44.8
75	1.3	40.7	25.8	50.2
100	1.0	43.8	26.9	54.5
200	0.50	51.4	29.3	66.5
350	0.29	57.8	30.8	77.8
500	0.20	62.0	31.7	85.7
750	0.13	66.9	33.0	96.4
1,000	0.10	70.4	33.6	103.6

3.3.1 Comparison to Previous Studies

A comparison of the flood frequency estimates obtained in this study for the Tawatinaw River with the studies previously completed by Alberta Environment and Parks (AEP 1994) as well as IBI and Golder (2014), is provided in Table 3-2.

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

Return Period (years)	Flood Peak Discharge of Tawatinaw River at the Hamlet of Rochester (m^3/s)		
	AEP (1994)	IBI and Golder (2014)	This Study
2	4.99	4.0	6.9
5	12.7	10.0	14.7
10	19.5	15.0	20.9
20	27.2	20.0	27.4
25	-	22.0	-
50	39.1	27.0	36.5
100	49.5	36.0	43.8
500	-	47.0	62.0

Notes:
1. The AEP (1994) study involved use of the recorded data up to 1989 for the regional stations.
2. The IBI and Golder (2014) study involved use of the recorded data from 1913 to 2011 for the regional stations.

The flood frequency estimates were based on the recorded data up to 1989 in the AEP study and up to 2011 in the IBI and Golder study (2014). The current study is based on the published flow data up to 2019, and the provisional flow data for 2020 for the regional gauging stations. In addition, this study includes the analyses to update the relationships between annual maximum daily and annual maximum instantaneous discharges for the regional stations.

The comparison of the studies shows that 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 estimated watershed area of the Tawatinaw River at the Hamlet of Rochester in the IBI and Golder (2014) study and this study.

4.0 OPEN WATER HYDRAULIC MODELLING

4.1 Overview

The following sections describe the methodology and results of the open water hydraulic modelling component. The scope of this component includes summary of available data and stream/valley features in the study area, hydraulic model setup, hydraulic model calibration and validation, selection of Manning's *n*, 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, and governing design flood hazard mapping 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 September 2020 by OGL Engineering.

4.2.2 Existing Model

There is one hydraulic model previously developed for the study area in 1997 as listed in Table 4-1. The study reach of the Tawatinaw River in 1997 HEC-RAS model is approximate 6.2 km, which is shorter but within the current study reach of 7.5 km. The downstream boundary locations for both studies are close to each other.

Table 4-1: Existing Hydraulic Model

No.	Study Description	Program Used for Model Development	Date	Author or Source
1	Tawatinaw River at Rochester Flood Risk Mapping Study	HEC-RAS	1997	UMA Engineering Ltd.

4.2.3 Highwater Marks

There is no highwater mark (HWM) data available along the study reach of the Tawatinaw River.

Golder and AEP attempted to identify any HWM from the residences during the site inspection and surveys without any success. Golder contacted the Athabasca County and Albert Transportation who confirmed no HWM data. Golder investigated the Hydrotechnical Information System (HIS) (a tool developed by Alberta Transportation to store historic and physical parameter data including flood information at bridge locations) to try to locate any flood information at the bridge crossings, and confirmed that no HWM data is in this system.

4.2.4 Gauge Data and Rating Curve

There is no Water Survey of Canada (WSC) hydrometric gauging station located on the Tawatinaw River within the study area.

4.2.5 Flood Photography

There is no aerial flood photography available for the study area.

4.3 River and Valley Features

4.3.1 Channel Characteristics

The Tawatinaw River study reach is approximately 7.5 km long. It extends from the upstream study boundary, approximate 200 m downstream of Tawatinaw Lake, through the Hamlet of Rochester (Rochester), to a location approximately 2.5 km downstream of Rochester. The Tawatinaw River flows in a well-defined, single channel. The reach between Highway 661 Culvert Crossing and Township Road 623.5 Bridge Crossing appears to be realigned with less sinuosity. Other reaches have relatively higher sinuosity.

The Tawatinaw River has a narrow, well-defined channel with cattail growing on both banks. The study reach has a typical channel bottom width of 5 m, bankfull width of 10 m, and bankfull depth of 1.1 m. It has an average channel bed slope of 0.07% and an average sinuosity of 1.5. The channel bed and bank materials consist of mainly sand, silt and clay. The river banks are well vegetated. Beaver dams and debris were observed in the Tawatinaw River during the site inspection and surveys.

4.3.2 Floodplain Characteristics

The Tawatinaw River study reach meanders in relative wide and flat floodplains with mountains on both sides. The floodplain width is typically 160 m (excluding the channel width) with a range of 60 to 310 m. The vegetation cover on the floodplains within the study area consists mainly of dense cattails and scattered willows.

4.3.3 Anthropogenic Features

The Hamlet of Rochester is located approximately 100 km north of Edmonton, 3 km east of Highway 2. It is situated in Athabasca County and has a population of 80, according to the 2016 Census of Population conducted by Statistics Canada. The river floodplain land use areas are mainly farmland.

4.3.4 Bridges and Culverts

The man-made structures along the study reach of the Tawatinaw River which are relevant for hydraulic modeling are listed in Table 4-2 and APPENDIX B. There are two (2) bridge crossings and two (2) culvert crossings within the study reach.

Table 4-2: Bridge and Culvert Crossings within the Study Area

No.	Name	Description	Type
1	Highway 661 Culvert	Highway 661 Crossing upstream of Hamlet of Rochester (see Figure B-1 in APPENDIX B)	6.2 span x 4.0 m rise
2	Township Road 623.5 Bridge	Township Road 623.5 at Rochester (see Figure B-2 in APPENDIX B)	1-Span
3	Local Culvert	Local culvert at Rochester (see Figure B-3 in APPENDIX B)	1.6 m diameter
4	Township Road 623.8 Bridge	Township Road 623.8 downstream of Rochester (see Figure B-4 in APPENDIX B)	1-Span

4.3.5 Weirs and Dams

There are no weirs or dams along the study reach of the Tawatinaw River.

4.3.6 Flood Control Structure

There is no flood control structure (e.g., berm or dike) along the study reach of the Tawatinaw River.

4.4 Model Construction

4.4.1 Methodology

The HEC-RAS program (Version 5.0.7, March 2019) was used to develop the one-dimensional (1D) hydraulic model for the study area. The model was not migrated to HEC-RAS version 6.1 as the modelling results and flood inundation mapping were completed before HEC-RAS Version 6.1 was released in September 2021.

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). 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. The program can be used for evaluation of floodway encroachments in floodplain management and flood hazard studies. 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 reach.

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%.
- 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 inundation extent of the 1,000-year flood event was estimated using supplemental 2D HEC-RAS modelling which was set up based on the LiDAR DEM without inclusion of the channel bathymetry, to provide conservative water level estimates 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 1 km on the Tawatinaw River) downstream of the study reach was included in the 1D model.

4.4.3.2 Reach and Branch

In this study, only one study reach along the Tawatinaw River is represented in the model. There are no separate branches or major tributary represented in the model.

4.4.3.3 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 boundary of the Tawatinaw River
- Normal flow condition (with an estimated energy slope of 0.025%) at the downstream model boundary of the Tawatinaw River.

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 was obtained from the following sources:

- River survey data collected for this study (see Section 2.0).
- 2020 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 supplemental 2D HEC-RAS modelling;
- simulated flood extents for the 1,000-year flood from supplemental 2D HEC-RAS modelling;
- topographic contours;
- estimated flow directions; and
- key structures.

There are three cross sections (i.e., XS2, XS9 and XS16) without surveyed bathymetric data. For these three cross sections, the bathymetries were defined by interpolation based on the surveyed bathymetric data of the adjacent upstream and downstream cross sections.

It is preferable that cross sections are aligned along the water surface isolines simulated using 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 channel, flow paths, bank lines, bank stations, cross section, river stations, and Manning's roughness *n*. There are total of 55 cross sections in the entire study reach represented in the model.

4.4.4.2 Roughness Coefficients

The left and right bank stations defining the main channel were determined using HEC-GeoRAS based on the 2020 LiDAR data, 2020 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
- Land use information from Government of Alberta.

Eight roughness classes were used for the model setup. The initial Manning's *n* values assigned to the classes are listed in Table 4-3. These initial values were selected based on such information as channel bed materials, and vegetation types (Chow 1959; USACE 2016b). These roughness values were modified at some locations when calibrating the Manning's *n* values. The roughness values were specified in the cross sections using HEC-GeoRAS. The distribution of the roughness classes is shown in Figure 4-1.

Table 4-3: Roughness Classes and Initial Manning's *n* Values

Number	Description	Initial Manning's *n* Value
1	River Channel	0.040
2	Urban Mixture (Residential)	0.080
3	Urban Mixture (Industrial)	0.060
4	Street	0.030
5	Grassland and Open Space	0.070
6	Pond	0.030
7	Dense Bush/Trees	0.150
8	Dense Cattails	0.100

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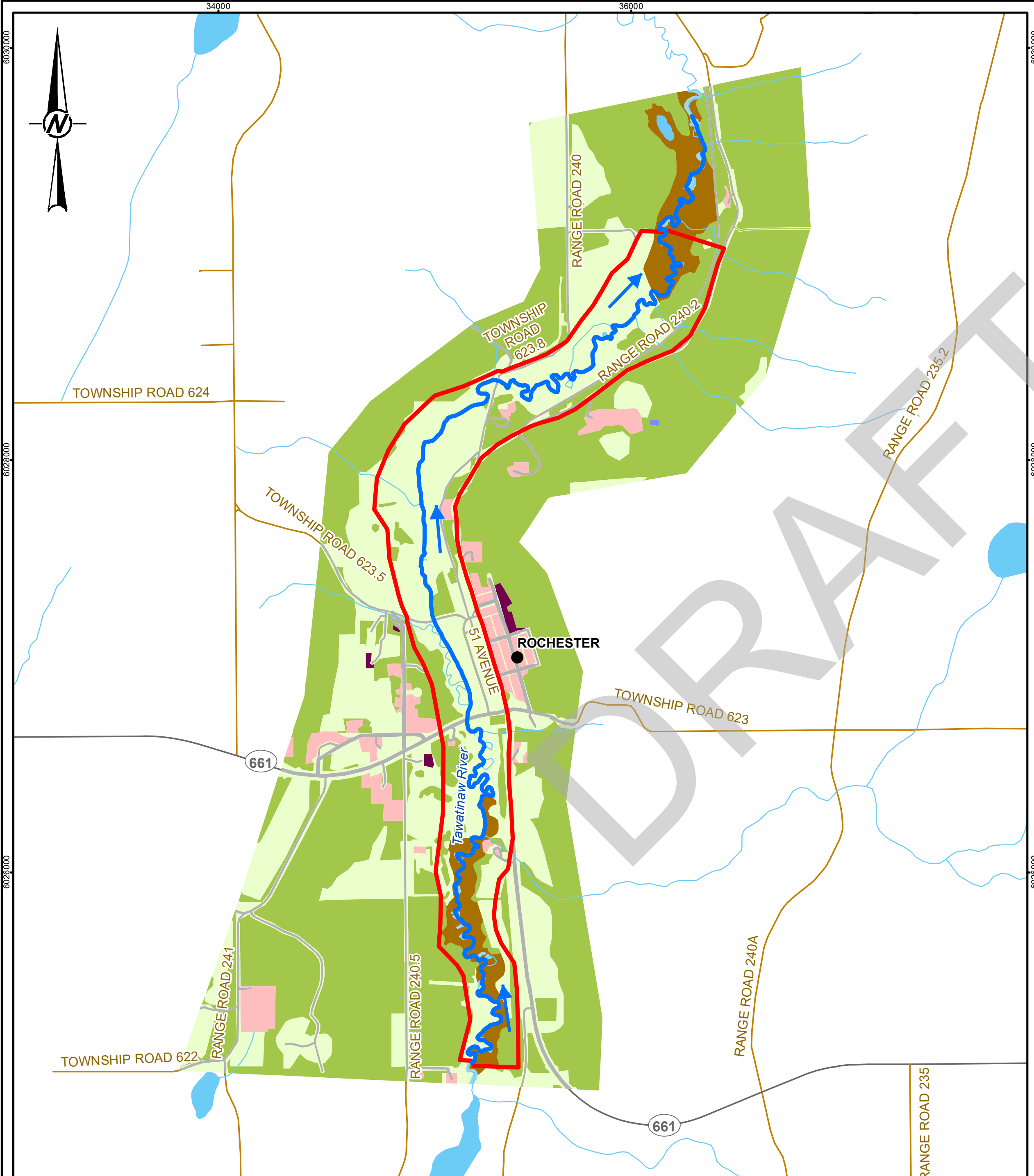

LEGEND

- POPULATED PLACE
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- SURVEY REACH
- FLOOD HAZARD STUDY AREA
- DENSE BUSH/TREES
- DENSE CATTAILS
- GRASSLAND AND OPEN SPACE
- POND
- RIVER-MAIN CHANNEL
- STREET
- URBAN MIXTURE (INDUSTRIAL)
- URBAN MIXTURE (RESIDENTIAL)

0 1 2

1:30,000 KILOMETRES

REFERENCE(S)

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ALBERTA ENVIRONMENT AND PARKS

PROJECT
ROCHESTER FLOOD STUDY

TITLE
DISTRIBUTION OF ROUGHNESS CLASSES

CONSULTANT

GOLDER
MEMBER OF WSP

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DESIGNED	AG
PREPARED	NB
REVIEWED	JC
APPROVED	HZ

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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 river and bridge surveys conducted in October 2020 (Section 2.0). Two (2) existing bridges (Section 4.3.4) were represented in the HEC-RAS model. 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 opening analysis was implemented. This allows the HEC-RAS model to calculate distribution of flows that are conveyed through the bridge openings and flows that bypass the bridges in the floodplains.

At the two bridge locations, ineffective areas upstream and downstream of the bridges were carefully defined. This included definition of permanent and non-permanent ineffective areas where appropriate.

The initial values of the contraction and expansion coefficients at both bridges were selected to be 0.3 and 0.5, respectively. These are typical values listed in the HEC-RAS User Manual.

4.4.4.3.2 Culvert

There are two culverts in the study area. The culverts were represented in the HEC-RAS model based on the survey data. 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 defined using the survey data. The ineffective areas upstream and downstream of the culvert were carefully defined in consideration of the features of the culvert and road embankment.

The multiple opening analysis approach was implemented at the culvert location to properly model overland flows that could bypass the 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 0.5.

4.4.4.3.3 Weirs and Dams

There are no weirs or dams in the study area that are represented in the HEC-RAS model.

4.4.4.4 Flood Control Structure

There is no flood control structure in the study area that is represented in the HEC-RAS model.

4.4.5 Model Calibration

The Manning's *n* and contraction/expansion coefficients are the primary model calibration parameters. Both low flow and high flow calibration should be performed to determine appropriate Manning's *n* values across a wide range of flows, if there is available data to support the model calibration. However, the hydraulic model was not calibrated in this study due to the following:

- Beaver dams are prevalent along the study reach. Many beaver dams were observed during the surveys. The beaver dam height ranged from 0.5 m to 1.5 m. The water levels were heavily influenced by the beaver dams for low flow conditions, especially for the extremely low flow rate of 0.1 m^3/s measured during October 10 and 15, 2020. This results in large differences between the simulated and surveyed water levels as shown in Figure 4-2 as beaver dams were not represented in the HEC-RAS model, in which the best estimate of Manning's *n* value of 0.040 was used. Surveyed water levels could not be match even when assigning unrealistically high Manning's *n*. Therefore, the surveyed water levels and measured low flow data were not used for the model calibration.
- There is no HWM and other flood information available within the study reach for calibrating the model for high flow conditions.
- There is no WSC gauging station situated within the study reach, so there is no stage-flow rating curve available for calibrating the model.

Therefore, Manning's *n* values for the main river channel and floodplain areas were estimated based on professional judgement and experience in this study, without any model calibration.

Accounting for the Beaver Dam Effects

Beaver dams have noticeable effects on water levels for low flow conditions along the study reach. Dimensions and configurations of the beaver dams were not surveyed as it was not safe to survey the beaver dams. To evaluate the beaver dam effects on the simulated water levels, the approximate heights and locations of the beaver dams were estimated based on the surveyed water levels (Figure 4-3) and represented in the HEC-RAS model.

The channel Manning's *n* value was selected to be 0.040, in consideration of the river bed/bank material types, vegetation cover on the banks, site information observed during the site inspection and surveys, and Golder's experience with similar modelling studies.

As shown in Figure 4-3, the simulated water levels are compared to the surveyed water levels associated with the low flow of 0.1 m^3/s (i.e., the flow conditions for the period October 11 to 15, 2020). The average water level difference between the simulated and surveyed water levels is -0.03 m, and the range of differences between -0.47 m and 0.12 m. This comparison illustrates that the HEC-RAS model with beaver dams represented in the model can be used to generate simulated water levels that match the surveyed water levels reasonably well.

In this study, the beaver dam effects were accounted for by adjusting (i.e., increasing) the estimated channel Manning's *n* value, because there was not beaver dam survey data to enable direct accounting of the beaver dam effects, and in recognition of the dynamic nature of beaver activities and changing beaver dam configurations over time. The method of above-mentioned adjustment is described in the following section.

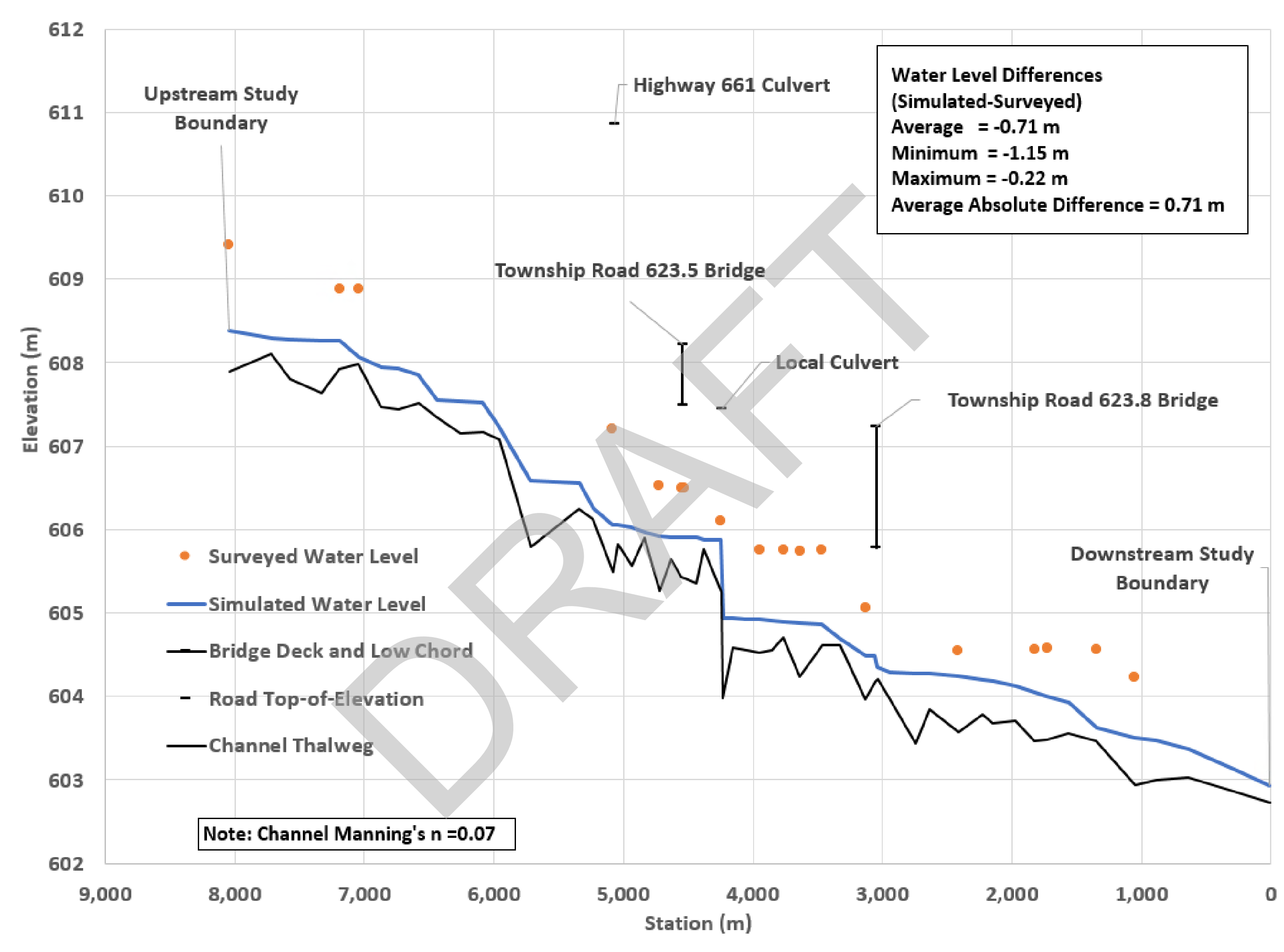

Figure 4-2: Comparison of Simulated Water Surface Profile to Surveyed Water Levels for the Surveyed Low Flow Condition

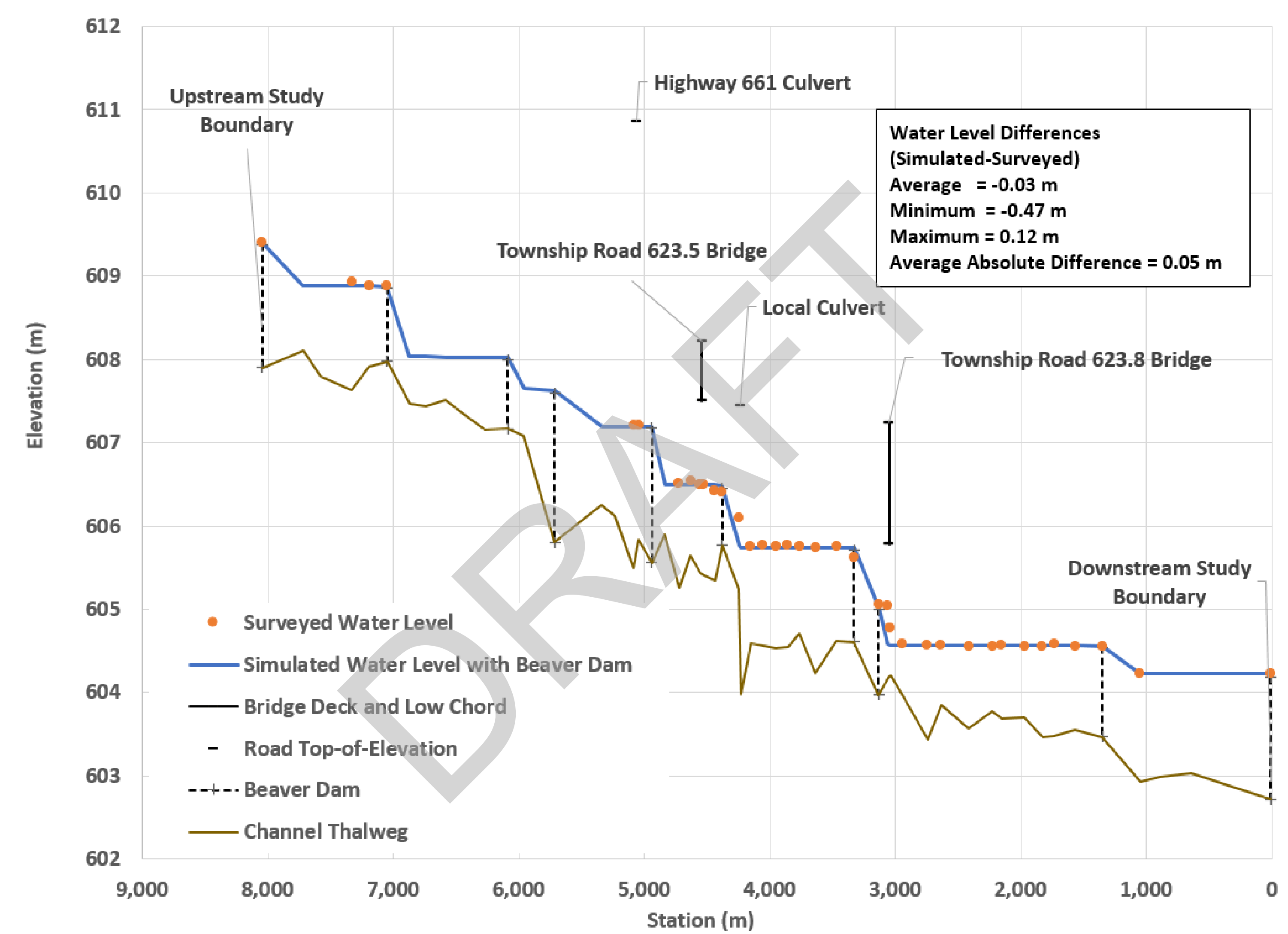

Figure 4-3: Comparison of the Simulated Water Surface Profile to the Surveyed Water Levels - Beaver Dams Assumed and Q=0.1 m³/s

4.4.6 Channel Roughness and Downstream Boundary

4.4.6.1 Selection of Channel Manning's n Value

The best estimate of the Manning's *n* value is 0.040 for the main channel based on the field observations of the channel conditions, published literature values for comparable channel conditions, and Golder's past modelling experience with similar channels. To account for the beaver dam effects, the best estimate of the Manning's *n* value of 0.040 was adjusted upward using the following procedure:

1) Simulated the 100-year flood levels along the Tawatinaw River reach using the assumed geometries of the beaver dams (see Figure 4-3) and the Manning's *n* value of 0.040.

2) Simulated the 100-year flood levels along the Tawatinaw River reach without beaver dams included in the model, and gradually increased the Manning's *n* value from 0.040.

3) Compare the simulated 100-year flood levels with and without the assumed beaver dams included, until the two sets of simulated flood levels matched closely as shown in Figure 4-4. This final step resulted in an increased Manning's *n* value of 0.050 for the main channel roughness.

Therefore, the Manning's *n* value of 0.050 was finally selected for generating the simulated water surface profiles for the 13 flood events. This approach resulted in slightly conservative estimates of the flood levels, in consideration of the following:

- The beaver dams are not permanent structures and some or all of them may be partially or completely washed away during flood events, particularly extreme floods.
- The beaver dams would have less effects on the water levels of large floods, because any potential blockage of channel flow conveyance by the beaver dams would be relatively small.

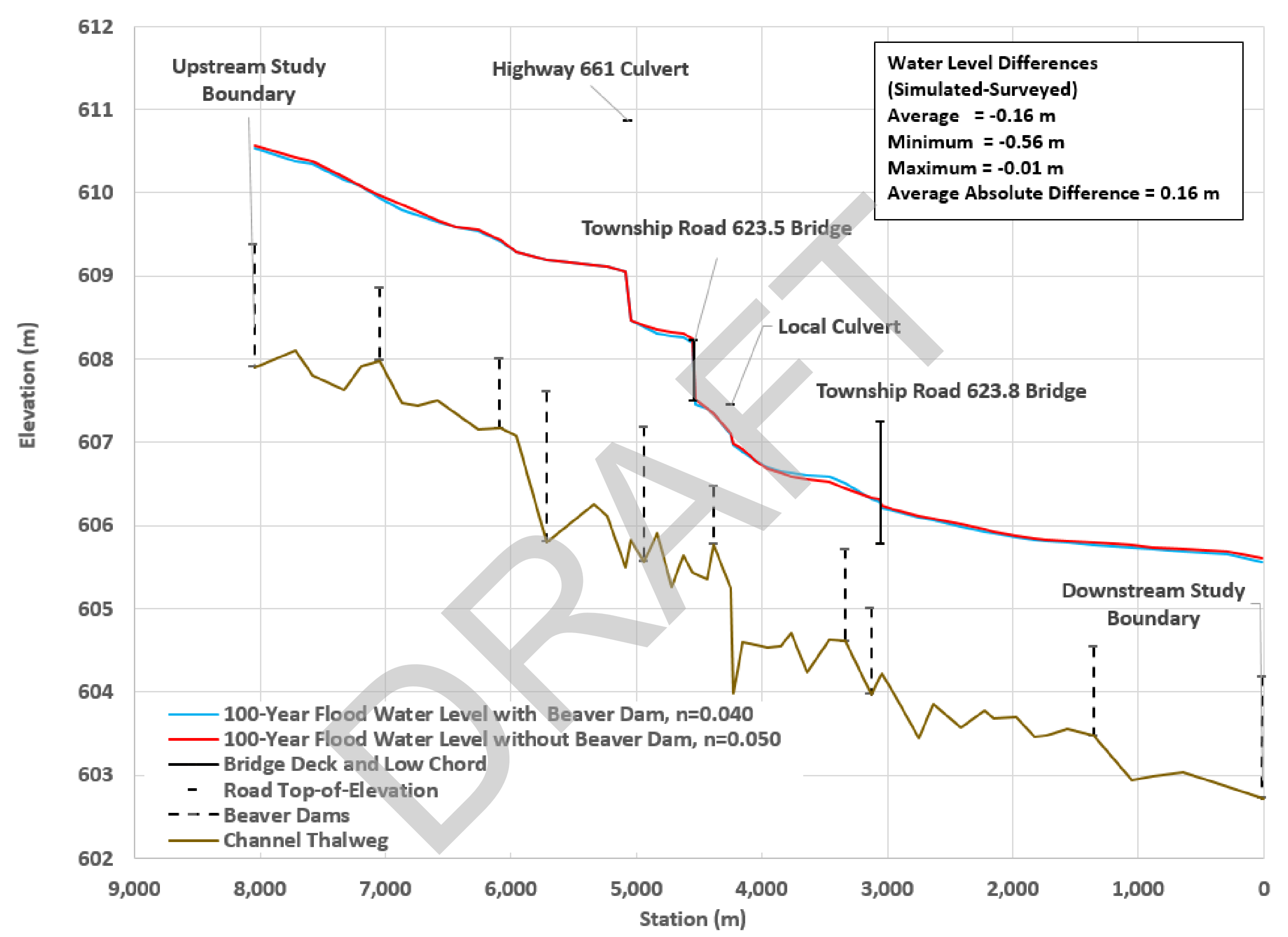

Figure 4-4: Comparison of the Simulated Water Surface Profiles with and without the Assumed Beaver Dams for the 100-Year Flood Event (Q= 43.8 m³/s)

4.4.6.2 Energy Slope at Downstream Boundary

An initial energy slope of 0.05% for the downstream boundary was estimated based on the average channel bottom slope along the 1.5 km reach immediately upstream of the downstream boundary. This energy slope was assumed for the model setup with the assumed beaver dams included. For the model setup without the assumed beaver dams included, the energy slope at the downstream boundary was adjusted as follows:

1) The water level at the downstream boundary was simulated using the model with the assumed beaver dams included and an estimated energy slope of 0.05%.

2) The water level at the downstream boundary was simulated using the model without the assumed beaver dams included, by adjusting the energy slope at the downstream boundary until the simulated water level was similar to that obtained from the above step (Figure 4-4). The final adjusted energy slope is 0.025%, which was used for the downstream boundary for the model setup without the assumed beaver dams included.

Therefore, the selected energy slope of 0.025% for the downstream boundary was used in this study for simulation of the 13 flood events.

4.4.7 Comparison with Previous Study

The simulated 100-year flood water levels using the current HEC-RAS model were compared to those of the 1997 Flood Study, as shown in Figure 4-5. This comparison shows that the simulated 100-year flood water levels using the current model are higher than those of the previous model. The differences are attributed to the following:

- The selected channel Manning's *n* value of 0.050 for the current model is higher than that in the 1997 Flood Study (*n* = 0.045).
- The cross-section data in the current model has higher resolution than that used in the 1997 Flood Study, especially for the floodplain areas. In the current study, the cross-section data for the floodplain areas was extracted from the LiDAR data with a resolution of 0.5 m, while the floodplain cross-section data in the 1997 Flood Study was based on the survey with large spacing.
- The 100-year flood peak discharges used in the two studies are different. The 100-year flood peak discharge was estimated at 43.8 m³/s in this study, which is lower than 49.5 m³/s estimated in the 1997 Flood Study.

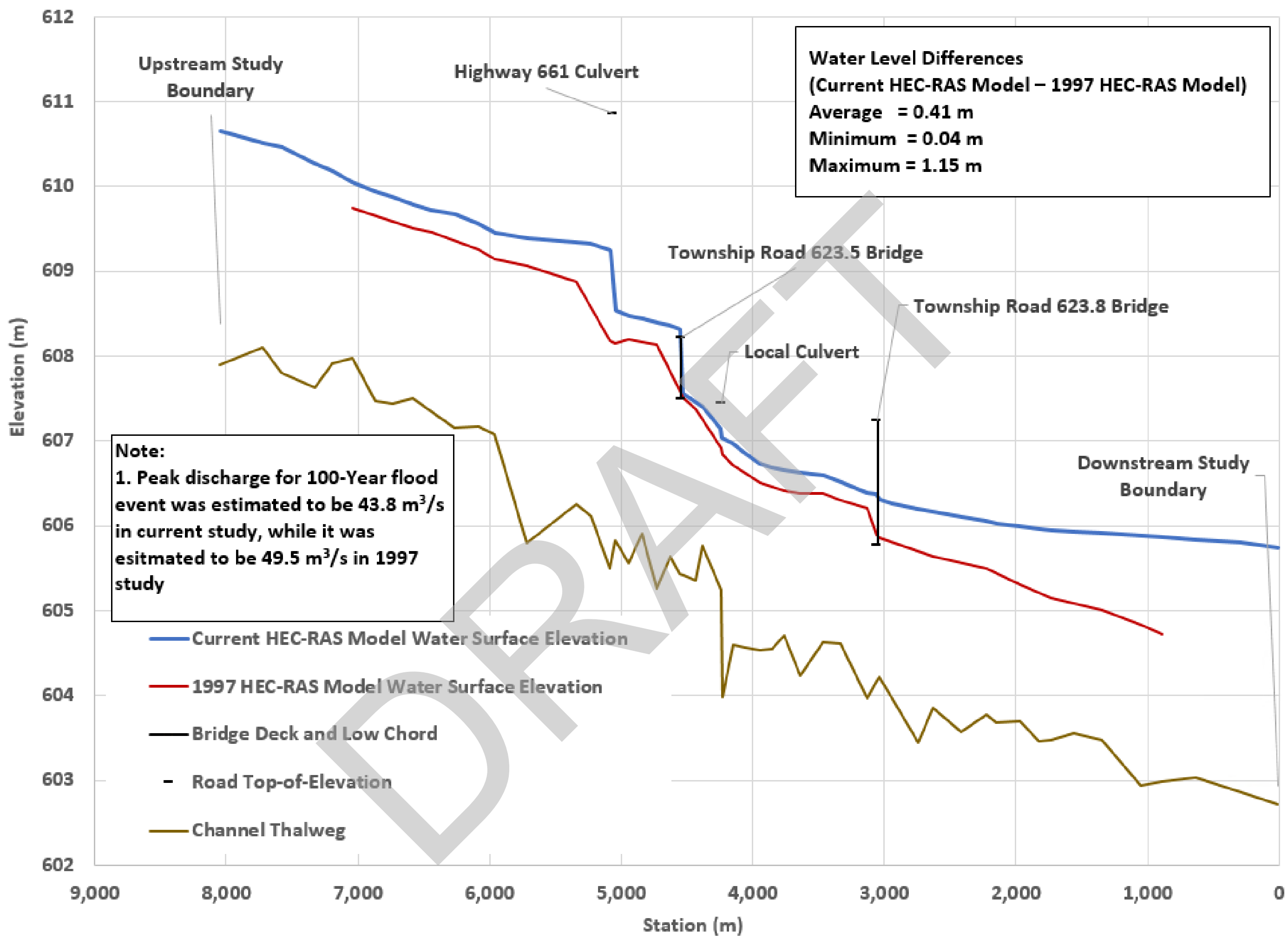

Figure 4-5: Comparison of the Simulated 100-year Flood Water Surface Profiles between the Two Models

4.4.8 Model Parameters and Options

4.4.8.1 Manning's Roughness Values

Channel Roughness

A constant Manning's *n* value of 0.050 was selected for the Tawatinaw River main channel (see Section 4.4.6.1). This value falls within a typical range for similar stream channels (Chow 1959).

Overbank Roughness

The selected overbank Manning's *n* values for the various land use types in the floodplain areas, are presented in Table 4-4.

Table 4-4: Selected Manning's *n* Values for Various Land Use Types

Land Use	Selected Manning's *n* Value
Urban Mixture (Residential)	0.080
Urban Mixture (Industrial)	0.060
Street	0.030
Grassland and Open Space	0.070
Pond	0.030
Dense Bush/Trees	0.150
Dense Cattails	0.100

4.4.8.2 Expansion and Contraction Coefficients

Typical coefficients of contraction and expansion are 0.1 and 0.3, respectively. These typical values were used for most cross sections, except for the bridges and culverts where the contraction and expansion coefficients of 0.3 and 0.5 were used, respectively.

4.4.8.3 Obstructions and Ineffective Flow Areas

The ineffective flow areas were identified and defined so that one geometry file could be used to simulate the various flood events with return periods of 2 to 1,000 years. The ineffective flow areas were defined in considerations of local topography, structure configurations, 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 are 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 certain elevation.
- Bridge decks and embankments: permanent 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.

4.4.9 Open Water Flood Frequency Profiles

4.4.9.1 Production Model

The HEC-RAS production model was based on the selected Manning's *n* values. The flood peak discharges used in the HEC-RAS production model were based on the hydrology assessment presented in Section 3.0. Surface water profiles were simulated using the production model for the 2-, 5-, 10-, 20-, 35, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events.

4.4.9.2 Flow Change Location

There is no major tributary within the study reach and the study reach is relatively short. Therefore, there is no flow change location along the study reach.

4.4.9.3 Flood Peak Discharges

The flood peak discharges listed in Table 4-5 were assigned at the model upstream boundary in the production model.

Table 4-5: Summary of the Flood Peak Discharges Used in the Production Model

Location	HEC-RAS Station	Discharges of Various Return Periods (m^3/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
Tawatinaw Upstream Boundary	8,045	6.9	14.7	20.9	27.4	32.9	36.5	40.7	43.8	51.4	57.8	62	66.9	70.4

4.4.9.4 Model Boundary Conditions

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

- The flood peak discharges specified for the upstream boundary as listed in Table 4-5.
- Normal flow condition with an energy slope of 0.025% specified for the downstream boundary.

4.4.9.5 Open Water Flood Frequency Profiles

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

4.4.10 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 100-year flood peak discharge was used for the model sensitivity analysis. The sensitivity analysis results were used to quantify the level of uncertainty associated with the simulated flood levels along the study reach of the Tawatinaw River.

The analysis of model sensitivity to Manning's *n* involves the following two sets of Manning's *n* values for the river channels and floodplains and one set of downstream boundary condition:

- 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 for the downstream boundary.

The differences between the simulated water levels for the 100-year flood along the study reach of the Tawatinaw River, are graphically presented in Figures E-1 to E-3 in APPENDIX E. The results of the sensitivity analysis indicate the following:

- The uncertainty in the simulated flood levels, on average, is within a range of ±0.03 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 ±0.03 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 influence the simulated flood levels along approximately 4.0 km study reach immediately upstream of the downstream boundary.

5.0 FLOOD INUNDATION MAPS

5.1 Scope

The scope of the open water flood inundation mapping includes the following tasks:

- open water flood inundation map production;
- flood water surface TIN development; and
- flood depth grid creation.

5.2 Methodology

The flood inundation maps were prepared based on the following information:

- The simulated 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.
- The locations and extents of individual cross sections.
- Topography from the 2020 LiDAR survey.
- Aerial imagery of the study area obtained in September 2020.

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

1) Assigned water levels at each section for all flood events to the cross-section polyline features as attributes. The result is one polyline feature that includes the simulated water levels for all flood events.
2) Created a continuous water level surface using a Triangulated Irregular Network (TIN) between cross sections.
3) Converted the TIN into a water level raster with the same resolution and cell alignment as the DTM raster.
4) Subtracted the DTM from the water level raster.
5) Assigned “NoData” to dry cells (with water depths smaller than 0.01 m).
6) Manually removed areas that are not directly connected to the main river channels. Areas where there is no direct overland connection but a hydraulic connection through culverts or other features, may be included in the inundation extent.
7) Polygons with an area smaller than 25 m^2 were deleted and holes smaller than 25 m^2 were filled.
8) The outline of the polygons was smoothed using the PEAK (Polynomial Approximation with Exponential Kernel) algorithm with a threshold of 15 m.

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

- <u>Scenario 1 (S1) – Single Overtopping Point:</u> 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.
- <u>Scenario 2 (S2) – Multiple Overtopping Points:</u> 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.

5.3 Water Surface Elevation TIN Modifications

5.3.1 Open Water Inundation Mapping

One set of open water flood inundation maps was prepared for each of the 13 flood events. The study area is covered by a total of three sheets in tabloid format (11 x 17 in). The mapping scale are 1:5,000. 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 2020 aerial imagery and other base data (roads and railways) provided by AEP. The resulting 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 a separate document (i.e., APPENDIX F: Open Water Flood Inundation Map Library).

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.

5.3.2 Manual Edits

Areas not properly delineated automatically using ArcGIS were delineated manually using break lines to properly map some complex areas.

The following manual adjustments were made in inundated areas adjacent to the Tawatinaw River:

- Scenario 1 with inundated areas mapped as backwater was done on all flood events in the treed area just East of the Tawatinaw River and North of Township Road 623.5.
- Backwater inundation for the area between Tawatinaw River and 51st Avenue, South of Township Road 623.5 was done for the 2-year and 5-year floods.
- Backwater inundation was done for six other locations for the 2-year flood event.

5.4 Areas Affected by Floods

5.4.1 Residential and Commercial Areas Affected by Floods

The residential and commercial areas affected by direct inundation are described below. Detailed inundation maps are provided in APPENDIX F.

- Portion of a farmhouse on the right floodplain near cross section 12 and upstream of Highway 661 culvert crossing, would be affected by floods with the return periods of 5 years and higher.
- No commercial areas would be inundated along the study reach.
- Water from the 100-year flood or greater would overtop the 51 Avenue on the right floodplain between Highway 661 and Township Road 623.5, and inundate portions of the land east of 51 Avenue.
- Water from the 10-year flood or greater would overtop portion of Township Road 623.8 on the right floodplain.

5.4.2 Flooding of Bridges and Culverts

A bridge is considered affected by flood when the flood water reaches its low chord. A culvert is considered affected by flood when the flood water reaches the road surface. Two (2) bridges would be affected during flood events with return periods of 5 to 10 years or higher.

The simulated water levels at the bridges and culverts along the Tawatinaw River for the various flood events, as well as the flow velocities and clearances during the 100-year flood event, are summarized in Table 5-1.

Table 5-1: Flooding at the Bridges and Culverts along the Tawatinaw River Study Reach

Bridge /Culvert Station (m)	Name	Minimum Deck/Road Surface Elevation (m)	Minimum Low Chord/ Culvert Top Elevation (m)	Simulated Water Levels 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			
5065	Highway 661 Culvert	610.9	609.6	607.4	607.8	608.1	608.4	608.6	608.8	608.9	609.1	609.3	609.6	609.7	609.9	610.1	2.74	1.81	> 1,000 years
4542	Township Road 623.5 Bridge	608.2	607.5	607.0	607.4	607.6	607.9	608.0	608.1	608.2	608.3	608.3	608.4	608.4	608.5	608.5	2.19	-0.75	10 years
4239	Local Culvert	607.4	607.2	606.8	606.9	607.0	607.0	607.0	607.1	607.1	607.1	607.2	607.2	607.3	607.3	607.3	1.24	0.33	> 1,000 years
3049	Township Road 623.8 Bridge	607.2	605.8	605.7	605.9	606.0	606.1	606.2	606.2	606.3	606.3	606.4	606.5	606.5	606.6	606.6	0.79	-0.53	5 years

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.

5.5 Flood Depth Grids

5.5.1 GIS Data Specifications

The following GIS data were provided to AEP for each of the 13 open water 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 114). 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.

5.5.2 General Comments

The flood water level data, provided as TINs (ArcGIS terrain files) 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 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).

6.0 DESIGN FLOOD HAZARD MAPPING

6.1 Flood Hazard Mapping Approach

AEP is implementing a new approach to flood hazard mapping, which is different from the approach used for previous flood studies and no longer includes an encroachment analysis. The major technical changes are described in detail in Section 6.1 in the Terms of Reference (TOR) (AEP 2020) and outlined below.

- Encroachment analysis will no longer be used to define floodway limits or determine 1:100 design flood levels. The 0.3 m water level rise criterion is no longer used to define the floodway limit.
- Existing floodways from previous flood studies will not typically get larger when flood hazard maps are updated. For areas with previously-defined floodways, the initial new floodway location will typically correspond to the existing floodway. The floodway can only get larger or smaller if it is deemed necessary with new modelling results based on consultation with local authorities.
- Areas with deeper and faster moving water outside the floodway will be identified within the flood fringe. A new high hazard flood fringe zone will highlight parts of the flood fringe with deeper or faster moving water than the rest of the flood fringe. The new high hazard flood fringe zone will be defined where the water is 1 m deep or greater, the local velocities are 1 m/s or faster in the flood fringe zone.
- The protection provided by dedicated flood berms will be reflected in new flood hazard maps. 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.
- Flood hazard maps will show areas at risk of more severe flooding than just the 1:100 design flood. Areas of incremental flood risk outside of the 1:100 flood hazard area will be highlighted, including the 1:200 and 1:500 floods.

6.2 Design Flood

The 100-year open water flood was selected as the design flood in accordance with the Flood Hazard Identification Program (FHIP) Guidelines (AEP 2011).

6.3 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: floodway and flood fringe. Flood hazard maps can also show additional flood hazard information, including areas of high hazard within the flood fringe and incremental areas at risk for more severe floods such as the 200-year and 500-year floods. Flood hazard mapping is typically used for long-term flood hazard area management and land-use planning.

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. The floodway typically includes the main channel of a stream and a portion of the adjacent overbank area. Previously mapped floodways do not typically become larger when a flood hazard map is updated, even if the flood hazard area gets larger or design flood levels get higher. New development is discouraged in the floodway and may not be permitted in some communities.

- **Flood Fringe**: The flood fringe is the portion of the flood hazard area outside of the floodway. The flood fringe typically represents areas with shallower (less than 1 m deep), slower (less than 1 m/s velocity), and less destructive flooding during the 100-year design flood. However, areas with deep or fast moving water may also be identified as high hazard flood fringe within the flood fringe. Areas at risk behind flood berms may also be mapped as protected flood fringe areas. New development in the flood fringe may be permitted in some communities.

6.4 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 stations at individual cross sections along the study reaches are listed in Table 6-1.

Table 6-1: Floodway Limits and Design Flood Water Levels

River	Cross Section	River Station	Floodway Station Limits		Floodway Determination Criteria		100-Year Open Water Design Flood Level (m)
			Left Station (m)	Right Station (m)	Left	Right	
Tawatinaw River	1	8045	57.4	75.8	Main Channel	Main Channel	610.6
Tawatinaw River	2	7720	58.5	93.1	Inundation Limit (2)	1 m Depth	610.4
Tawatinaw River	3	7577	76.8	158.6	1 m Depth	Mixed (3)	610.4
Tawatinaw River	4	7329	64.4	92.9	1 m Depth	1 m Depth	610.2
Tawatinaw River	5	7192	67.7	163.6	Previous Floodway	Previous Floodway	610.1
Tawatinaw River	6	7046	72.0	153.5	Previous Floodway	Previous Floodway	610.0
Tawatinaw River	7	6871	73.6	198.9	Previous Floodway	Previous Floodway	609.9
Tawatinaw River	8	6739	74.6	159.8	Previous Floodway	Previous Floodway	609.8
Tawatinaw River	9	6581	69.1	139.1	Mixed (3)	Previous Floodway	609.7
Tawatinaw River	10	6442	84.8	194.8	Previous Floodway	Previous Floodway	609.6
Tawatinaw River	11	6264	96.9	229.3	Previous Floodway	Previous Floodway	609.6
Tawatinaw River	12	6086	221.0	264.9	Inundation Limit (2)	Previous Floodway	609.4
Tawatinaw River	13	5961	148.6	220.2	Previous Floodway	1 m Depth	609.3
Tawatinaw River	14	5714	145.1	251.6	Previous Floodway	Previous Floodway	609.2
Tawatinaw River	15	5341	181.7	277.6	Previous Floodway	Previous Floodway	609.1
Tawatinaw River	16	5232	136.4	250.9	Previous Floodway	Previous Floodway	609.1
Tawatinaw River	17	5084	238.7	292.7	Main Channel	Previous Floodway	609.1
Tawatinaw River	18	5045	257.9	290.9	Inundation Limit (1)	Previous Floodway	608.5
Tawatinaw River	19	4940	101.8	191.8	Previous Floodway	Previous Floodway	608.4
Tawatinaw River	20	4837	75.0	163.1	Previous Floodway	Inundation Limit (1)	608.4
Tawatinaw River	21	4725	104.5	202.2	Previous Floodway	Previous Floodway	608.3
Tawatinaw River	22	4629	39.9	199.3	Previous Floodway	Previous Floodway	608.3
Tawatinaw River	23	4554	34.6	163.0	Previous Floodway	Previous Floodway	608.3
Tawatinaw River	24	4528	39.1	135.7	Previous Floodway	Inundation Limit (1)	607.5
Tawatinaw River	25	4438	49.3	161.6	Previous Floodway	Previous Floodway	607.4
Tawatinaw River	26	4380	53.8	181.0	Previous Floodway	Previous Floodway	607.4
Tawatinaw River	27	4245	142.9	224.2	Inundation Limit (1)	Previous Floodway	607.1
Tawatinaw River	28	4230	146.0	227.3	Main Channel	Previous Floodway	607.0
Tawatinaw River	29	4153	137.4	233.3	Previous Floodway	Previous Floodway	606.9
Tawatinaw River	30	4054	103.4	215.5	Previous Floodway	Previous Floodway	606.8
Tawatinaw River	31	3950	138.8	273.0	Previous Floodway	Previous Floodway	606.7

Table 6-1: Floodway Limits and Design Flood Water Levels

River	Cross Section	River Station	Floodway Station Limits		Floodway Determination Criteria		100-Year Open Water Design Flood Level (m)
			Left Station (m)	Right Station (m)	Left	Right	
Tawatinaw River	32	3855	145.5	290.0	Previous Floodway	Previous Floodway	606.6
Tawatinaw River	33	3763	137.5	283.5	Previous Floodway	Previous Floodway	606.6
Tawatinaw River	34	3637	108.7	287.6	Previous Floodway	Previous Floodway	606.6
Tawatinaw River	35	3466	97.8	256.5	Previous Floodway	Previous Floodway	606.5
Tawatinaw River	36	3332	60.0	146.9	Previous Floodway	Previous Floodway	606.5
Tawatinaw River	37	3132	19.9	116.1	Inundation Limit (2)	Previous Floodway	606.3
Tawatinaw River	38	3060	34.7	135.4	Inundation Limit (2)	Previous Floodway	606.3
Tawatinaw River	39	3039	35.0	130.8	Previous Floodway	Previous Floodway	606.2
Tawatinaw River	40	2945	46.5	194.0	Previous Floodway	Previous Floodway	606.2
Tawatinaw River	41	2746	74.1	229.6	Previous Floodway	Previous Floodway	606.1
Tawatinaw River	42	2633	73.1	216.4	Previous Floodway	Previous Floodway	606.1
Tawatinaw River	43	2415	76.2	209.6	Previous Floodway	Previous Floodway	606.0
Tawatinaw River	44	2228	77.5	203.7	Previous Floodway	Previous Floodway	606.0
Tawatinaw River	45	2150	67.2	186.5	Previous Floodway	Previous Floodway	605.9
Tawatinaw River	46	1972	71.4	182.4	Previous Floodway	Previous Floodway	605.9
Tawatinaw River	47	1825	75.8	193.4	Previous Floodway	Previous Floodway	605.9
Tawatinaw River	48	1731	99.0	216.1	Previous Floodway	Previous Floodway	605.8
Tawatinaw River	49	1565	143.5	272.7	Previous Floodway	Previous Floodway	605.8
Tawatinaw River	50	1351	150.4	267.0	Previous Floodway	Previous Floodway	605.8
Tawatinaw River	51	1050	106.7	266.1	Previous Floodway	Previous Floodway	605.8
Tawatinaw River	52	887	71.9	218.0	Mixed (3)	Previous Floodway	605.7

Notes:
1) Cross sections where the previous floodway is outside in the inundation limit.
2) No viable flood fringe.
3) To balance other criteria, or to maintain smooth floodway

6.5 Design Flood Profile

The 100-year flood water levels simulated in flood frequency analysis (Section 4.4.9.5) were selected as the final design flood levels and presented in Table 6-1.

6.6 Floodway Criteria Maps

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

- inundation extents of the 100-year open water design flood;
- areas meeting or exceeding the 1 m depth floodway criterion for the design flood;
- portions of each cross section where the computed velocity is 1 m/s or faster;
- proposed floodway boundary, as well as the associated floodway stations corresponding to the floodway determination criteria;
- isolated areas of non-flooded, high ground (i.e., “dry areas”) within the design flood extent;
- locations of the main channel top of bank at each cross section;
- location and extent of all cross sections used in the HEC-RAS model with appropriate labels;
- previous-mapped floodway boundary from 1997 study (where it exists);
- background aerial imagery collected in 2020; and
- roads, bridges, culverts and flood control structures as applicable.;

The open water design flood water surface elevations and flow velocities were generated using the HEC-RAS production model.

Flood Depth Determination

A flood depth grid was generated by subtracting the water level surface TIN (see Section 5.2.) for the design flood event from the digital terrain model. This flood depth grid was used to identify areas meeting or exceeding the 1m depth criterion and to generate 1 m depth contour lines.

Flow Velocity Computations

Flow velocities are only available at the cross-section locations in HEC-RAS as a 1D computational modelling approach was used for this Rochester Flood Study. The area with flow velocities of 1 m/s or greater between cross sections are based on the spatial output provided by HEC-RAS, which attempts to create a continuous flow velocity raster taking into consideration the cross-section lines and the main channel center line.

The floodway boundary was delineated such that a hydraulically smooth floodway boundary between cross sections was produced. The floodway criteria maps were produced using the same template as the inundation maps. The maps are provided in APPENDIX G.

6.6.1 Flood Hazard Maps

The flood hazard maps display the areas in the floodway and flood fringe. The floodway was determined as part of the floodway criteria mapping. Flood hazard maps can also show additional flood hazard information, including areas of high hazard within the flood fringe and incremental areas at risk for more severe floods, like the 200-year and 500-year floods. Flood hazard mapping is typically used for long-term flood hazard area management and land-use planning. All areas within the floodway boundary are shown as part of the floodway, even if the water levels of the design flood would not indicate a location as inundated (i.e., “islands” of dry ground within the floodway shown on the floodway criteria maps are not present on the flood hazard maps).

The flood hazard maps were produced using the same template as the inundation maps. The maps are provided in APPENDIX G.

Areas in the Floodway

No residence or key structure is situated in the floodways along the Tawatinaw River study reach.

Areas in the High Hazard Flood Fringe

There is no residence or key structure located within high hazard flood fringe.

Areas in the Flood Fringe

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

- Portion of a farmhouse property on the right floodplain near cross section 12 and upstream of Highway 661 culvert crossing
- Portion of Township Road 623.8 on the right floodplain.

6.7 Design Flood Grids

6.7.1 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.

6.7.2 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.

6.7.3 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 raster 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)].

6.8 Quantitative Climate Change Assessment

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 changes on flood levels. The effect of the 100-year flood conditions more severe than the baseline was assessed under the following two flow scenarios:

1) 100-year open water discharge +10%.

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 reach for the two additional flow scenarios. 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.

The average increases in the open water flood levels are 0.07 m for a 10% increase in flow, and 0.15 m for a 20% increase in flow.

The above analyses are not based on a regional climate change impact assessment but on a simplified assumption that climate changes would 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 climate-affected flood profiles along the Tawatinaw River study reach, are presented in Figure H-1 in APPENDIX H. The simulated climate-affected open water flood water levels at individual cross sections are compared to the baseline 100-year open water discharge in Table H-1 in APPENDIX H.

7.0 CONCLUSIONS

7.1 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:

- *Cross Section Surveys* – Cross section survey data collected in October and December 2020 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 October 2020 meet the study requirements and include the necessary details for the hydraulic modelling.
- *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.

7.2 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 Tawatinaw River at Hamlet of Rochester. These estimates provide the updated flood hydrology information as inputs to the other components of the study (e.g., hydraulic modelling). Estimates of flood peak discharges were obtained for various return periods ranging from 2 to 1,000 years, including the 95% upper and lower confidence intervals.
- The flood frequency estimates were based on the recorded data up to 1989 in the AEP study and up to 2011 in the IBI and Golder study (2014). The current study is based on the published flow data up to 2019, and the provisional flow data for 2020 for the regional gauging stations. In addition, this study includes the analyses to update the regional relationships between annual maximum daily and annual maximum instantaneous discharges.

7.3 Open Water Hydraulic Modelling

7.3.1 Selection of Manning's *n* Values

An attempt was made to calibrate the HEC-RAS model based on the measured low flow water levels and discharge collected during the October 2020 survey. The calibration was not successful because of the dominant effects of the beaver dams, not channel roughness, on the low flow water levels. Attempts were made without any success to collect HWM and anecdotal flood information from the local community, Alberta Transportation and Athabasca County.

Therefore, the channel Manning's *n* value was estimated without any calibration. The selected channel Manning's *n* value is 0.050, which accounts for the potential beaver dam effects on the flood levels. This value is within the typical range of roughness values for similar streams (Chow 1959).

The Manning's *n* values for the floodplain areas were estimated and selected based on the land use types.

7.3.2 Model Sensitivity

The model sensitivity analysis was conducted for the 100-year flood event to evaluate the effects of changing model roughness values and downstream boundary conditions on the simulated water levels. The results of the sensitivity analysis indicate the following:

- The uncertainty in the simulated flood levels, on average, is within a range of ±0.03 m along the Tawatinaw River study reach.
- The ±20% changes of the energy slope at the downstream boundary influence the simulated flood levels along approximately 4 km reach immediately upstream from the downstream boundary.

7.3.3 Flood Profiles

The HEC-RAS model is 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.

7.4 Flood Inundation Mapping

The 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:

- Portion of a farmhouse property on the right floodplain near cross section 12 and upstream of Highway 661 culvert crossing, would be affected by the 5-year flood or greater.
- No commercial areas would be inundated along the study reach.
- Water from the 100-year flood or greater would overtop the 51 Avenue on the right floodplain between Highway 661 and Township Road 623.5, and inundate portions of the land east of 51 Avenue.
- Water from the 10-year flood or greater would overtop portion of Township Road 623.8 on the right floodplain.

7.5 Design Flood Hazard Mapping

The 100-year open water flood is selected as the design flood on the Tawatinaw River in accordance with the Flood Hazard Identification Program (FHIP) Guidelines (AEP 2011).

The flood hazard maps display the areas in the floodway and flood fringe. The floodway was determined as part of the floodway criteria mapping. The flood fringe includes all other directly-inundated areas beyond the floodway limits. Flood hazard maps can also show additional flood hazard information, including areas of high hazard within the flood fringe and incremental areas at risk for more severe floods, like the 200-year and 500-year floods.

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.

Areas in the Floodway

No residence or key structure is situated on the floodways along the Tawatinaw River study reach.

Areas in the High Hazard Flood Fringe

There is no residence or key structure within high hazard flood fringe zones.

Areas in the Flood Fringe

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

- Portion of a farmhouse property on the right floodplain near cross section 12 and upstream of Highway 661 culvert crossing.
- Portion of Township Road 623.8 on the right floodplain.

7.6 Quantitative Climate Change Assessments

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 the 100-year flood peak 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 illustrate that the average increases in the open water flood levels are 0.07 m for a 10% increase in the 100-year flood peak discharge, and 0.15 m for a 20% increase in the discharge.

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

Signature Page

This report was prepared and reviewed by the undersigned.

Golder Associates Ltd.

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Reviewed by:

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PERMIT TO PRACTICE
GOLDER ASSOCIATES LTD

RM SIGNATURE: ______________________

RM APEGA ID #: ______________________

DATE: ______________________

PERMIT NUMBER: P005122
The Association of Professional Engineers and Geoscientists of Alberta (APEGA)

JC/AG/ZH/DL

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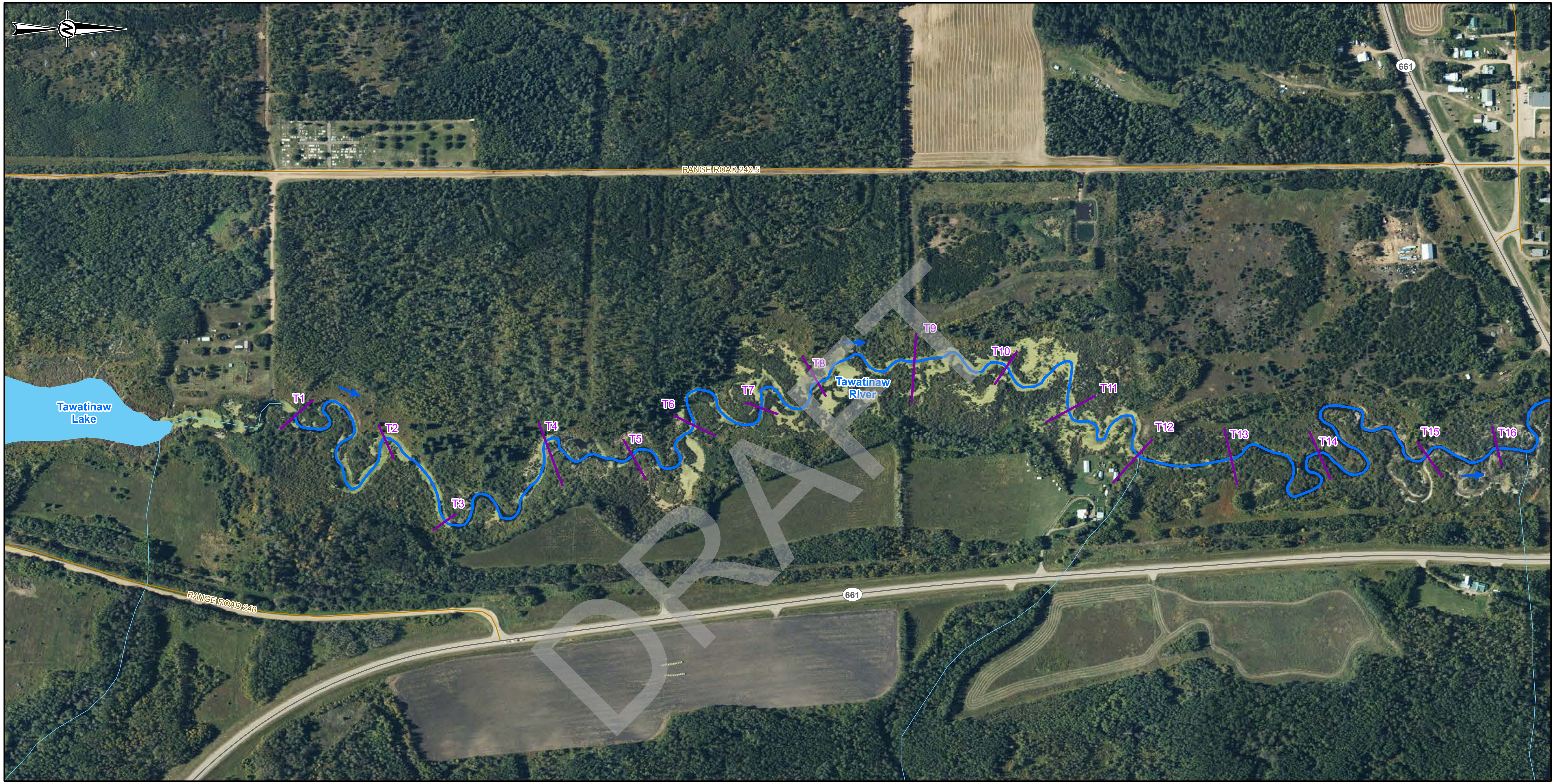
APPENDIX A

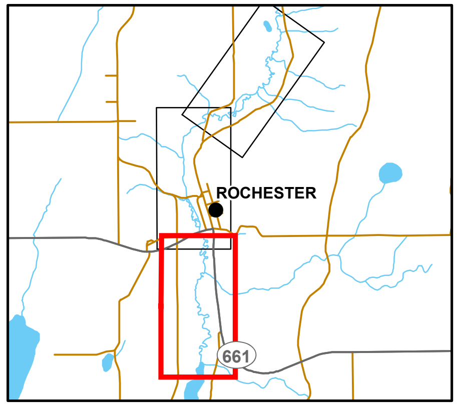
Locations of Cross Sections and Hydraulic Structures



LEGEND

- FLOW DIRECTION
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- WATERCOURSE
- WATERBODY
- SURVEY REACH
- SURVEYED CROSS SECTION

KEY MAP



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CLIENT

ALBERTA ENVIRONMENT AND PARKS

Alberta

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GOLDER MEMBER OF WSP

YYYY-MM-DD	2022-03-21
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PREPARED	SK
REVIEWED	JC
APPROVED	HZ

REFERENCE(S)

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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

PROJECT

ROCHESTER FLOOD STUDY

TITLE

LOCATIONS OF CROSS SECTIONS AND HYDRAULIC STRUCTURES

PROJECT NO.	CONTROL	REV.	FIGURE
20368084	1000	0	**A-1**

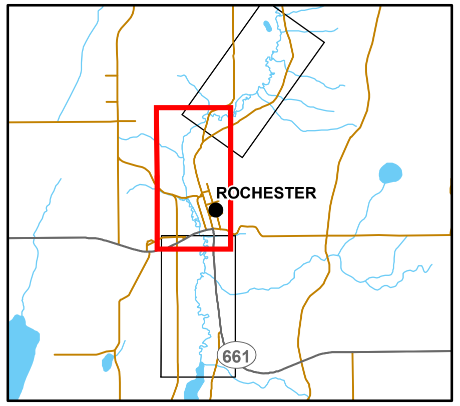
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25mm

LEGEND

- FLOW DIRECTION
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- WATERCOURSE
- WATERBODY
- BRIDGE
- CULVERT
- SURVEY REACH
- SURVEYED CROSS SECTION

KEY MAP



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ALBERTA ENVIRONMENT AND PARKS

CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

PROJECT
ROCHESTER FLOOD STUDY

TITLE
LOCATIONS OF CROSS SECTIONS AND HYDRAULIC STRUCTURES

PROJECT NO.	CONTROL	REV.	FIGURE
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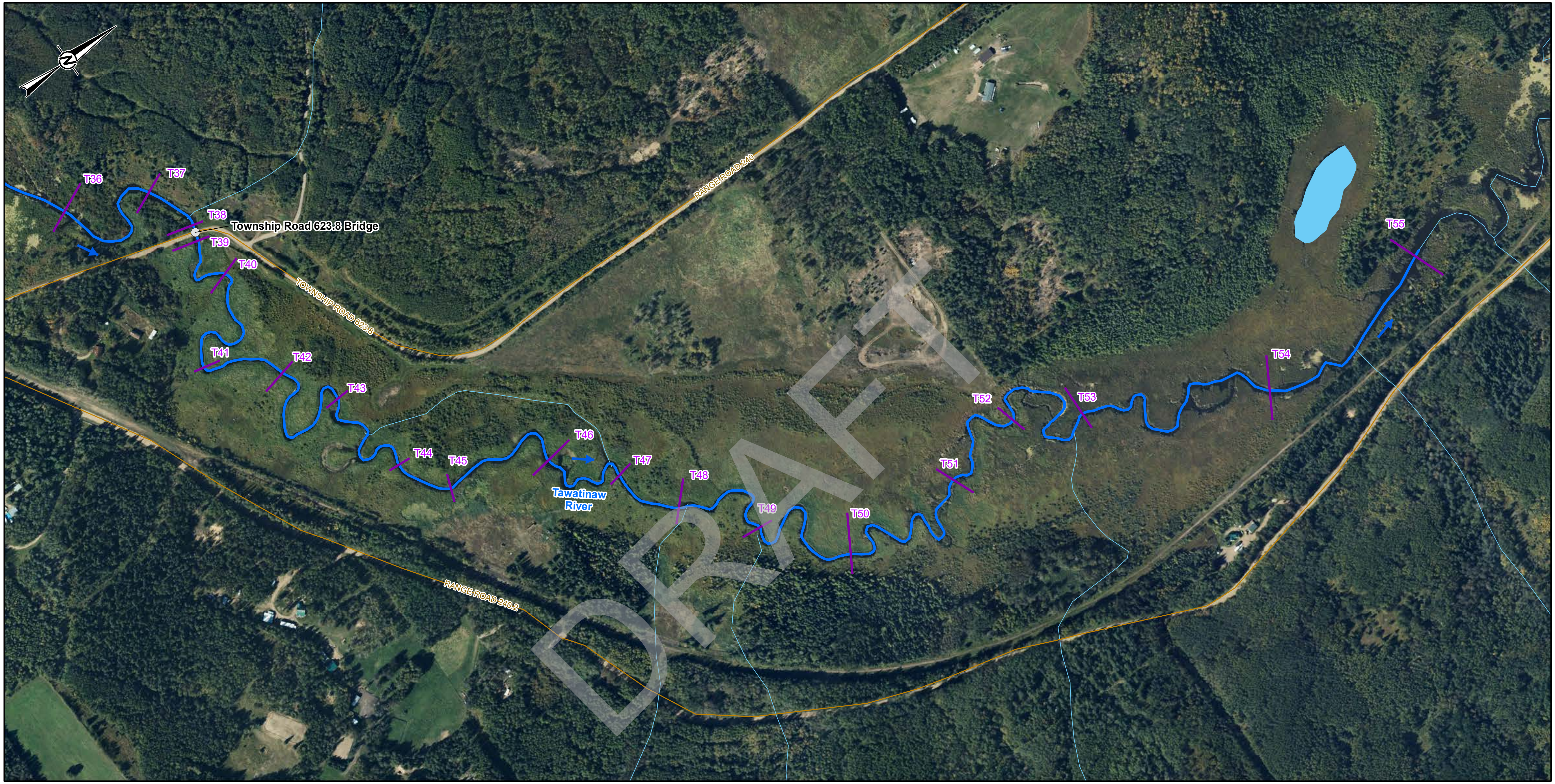
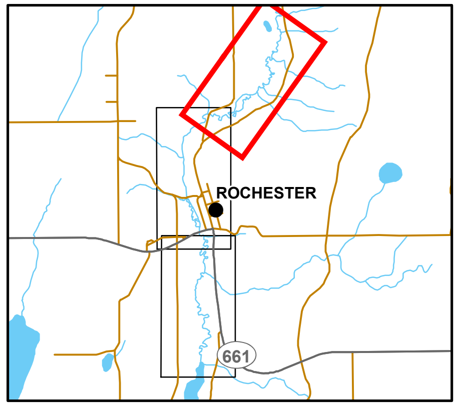


LEGEND

- FLOW DIRECTION
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- WATERCOURSE
- WATERBODY
- BRIDGE
- SURVEY REACH
- SURVEYED CROSS SECTION

KEY MAP



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CONSULTANT
GOLDER MEMBER OF WSP

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DESIGNED	AG
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REVIEWED	JC
APPROVED	HZ

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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

PROJECT
ROCHESTER FLOOD STUDY

TITLE
LOCATIONS OF CROSS SECTIONS AND HYDRAULIC STRUCTURES

PROJECT NO.	CONTROL	REV.	FIGURE
20368084	1000	0	A-3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

APPENDIX B

Hydraulic Structure Datasheets

0 50 100

1:2,000 METRES

TITLE

HYDRAULIC STRUCTURE DATASHEET - HIGHWAY 661 CULVERT

LOCATION	TAWATINAW RIVER
DESCRIPTION	HIGHWAY 661 CULVERT
TOTAL LENGTH OF CULVERT (m)	17.80
RISE OF CULVERT (m)	4.01
SPAN OF CULVERT (m)	6.27
DIAMETER OF CULVERT (m)	-
CULVERT TYPE	CORRUGATED METAL
CULVERT INVERT ELEVATION - SOUTH END (m)	605.60
CULVERT ENVERT ELEVATION - NORTH END (m)	605.57

LEGEND

- STRUCTURE SURVEY POINT
- FLOW DIRECTION
- ROADS

PHOTO 1 LEFT BANK, LOOKING DOWNSTREAM

PHOTO 2 RIGHT BANK DOWNSTREAM, LOOKING AT CULVERT

NOTE(S)

ALL DETAILS OF STRUCTURE SURVEY WILL BE USED FOR HYDRAULIC MODELLING.

REFERENCE(S)

STRUCTURE SURVEY AND STRUCTURE PHOTOS BY GOLDER ASSOCIATES LTD. OCTOBER 2020.
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CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	AL
REVIEWED	JC
APPROVED	HZ

PROJECT NO.	CONTROL	REV.	FIGURE
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0 50 100

1:2,000 METRES

TITLE

HYDRAULIC STRUCTURE DATASHEET - TOWNSHIP ROAD 623.5 BRIDGE

LOCATION	TAWATINAW RIVER
DESCRIPTION	TOWNSHIP ROAD 623.5 BRIDGE
TOTAL LENGTH OF SPAN (m)	6.00
DECK WIDTH OF BRIDGE (m)	8.10
AVERAGE TOP OF CURB OR SOLID GUARD RAIL ELEVATION (m)	608.22
AVERAGE LOW CHORD ELEVATION (m)	607.52
BRIDGE OBSTRUCTION HEIGHT (m)	0.70
NUMBER OF PIERS	0

PIER	CENTRE STATION (m)	WIDTH (m)	TYPE	SHAPE
1	-	-	-	-
2	-	-	-	-

LEGEND

- STRUCTURE SURVEY POINT
- FLOW DIRECTION
- ROADS

PHOTO 1 LEFT BANK DOWNSTREAM, LOOKING AT BRIDGE

PHOTO 2 RIGHT BANK UPSTREAM, LOOKING AT BRIDGE

NOTE(S)
ALL DETAILS OF STRUCTURE SURVEY WILL BE USED FOR HYDRAULIC MODELLING.

REFERENCE(S)
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PROJECT
ROCHESTER FLOOD STUDY

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	AL
REVIEWED	JC
APPROVED	HZ

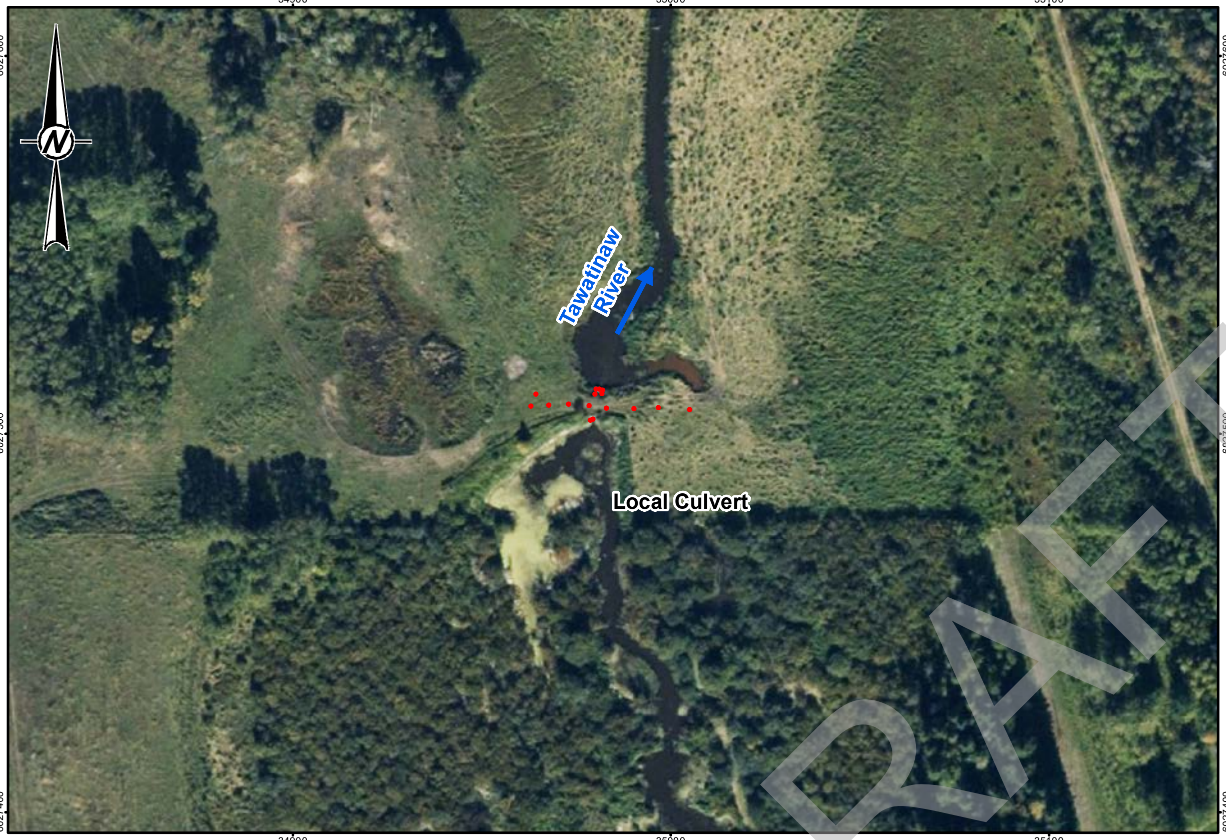
PROJECT NO.	CONTROL	REV.	FIGURE
20368084	1000	0	B-2

DRAFT

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET HAS BEEN MODIFIED FROM: ANSI A

25mm

Classification: Public



TITLE
HYDRAULIC STRUCTURE DATASHEET - LOCAL CULVERT

LOCATION	TAWATINAW RIVER
DESCRIPTION	LOCAL CULVERT
TOTAL LENGTH OF CULVERT (m)	8.15
RISE OF CULVERT (m)	-
SPAN OF CULVERT (m)	-
DIAMETER OF CULVERT (m)	1.60
CULVERT TYPE	CORRUGATED METAL
CULVERT INVERT ELEVATION - SOUTH END (m)	605.63
CULVERT ENVERT ELEVATION - NORTH END (m)	605.01

LEGEND

- STRUCTURE SURVEY POINT
- FLOW DIRECTION

PHOTO 1 LEFT BANK, LOOKING AT CULVERT

PHOTO 2 UPSTREAM, LOOKING AT CULVERT

NOTE(S)
ALL DETAILS OF STRUCTURE SURVEY WILL BE USED FOR HYDRAULIC MODELLING.

REFERENCE(S)
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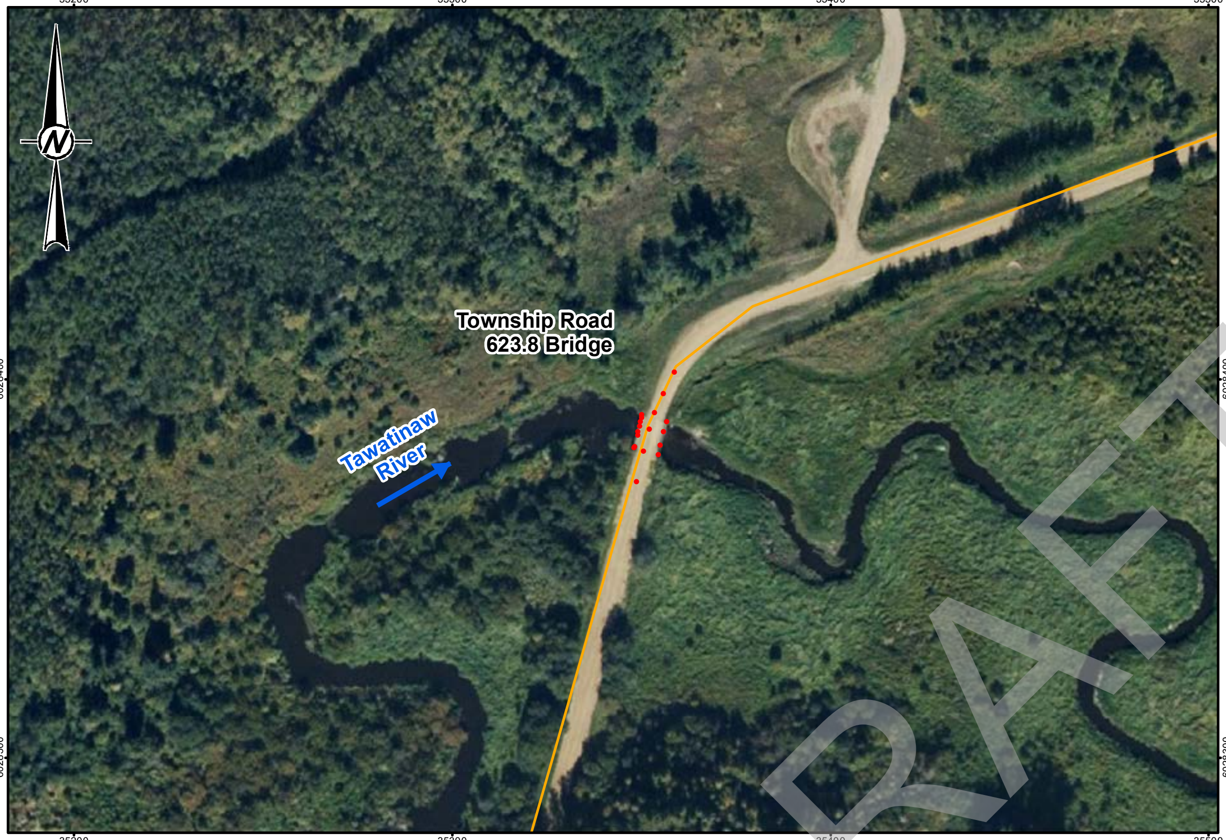

0 50 100

1:2,000 METRES

TITLE

HYDRAULIC STRUCTURE DATASHEET - TOWNSHIP ROAD 623.8 BRIDGE

LOCATION	TAWATINAW RIVER
DESCRIPTION	TOWNSHIP ROAD 623.8 BRIDGE
TOTAL LENGTH OF SPAN (m)	7.90
DECK WIDTH OF BRIDGE (m)	6.70
AVERAGE TOP OF CURB OR SOLID GUARD RAIL ELEVATION (m)	607.24
AVERAGE LOW CHORD ELEVATION (m)	605.78
BRIDGE OBSTRUCTION HEIGHT (m)	1.46
NUMBER OF PIERS	0

PIER	CENTRE STATION (m)	WIDTH (m)	TYPE	SHAPE
1	-	-	-	-
2	-	-	-	-

LEGEND

- STRUCTURE SURVEY POINT
- FLOW DIRECTION
- ROADS

PHOTO 1 LEFT BANK, LOOKING AT BRIDGE

PHOTO 2 LEFT BANK UPSTREAM, LOOKING AT BRIDGE

NOTE(S)
ALL DETAILS OF STRUCTURE SURVEY WILL BE USED FOR HYDRAULIC MODELLING.

REFERENCE(S)
STRUCTURE SURVEY AND STRUCTURE PHOTOS BY GOLDER ASSOCIATES LTD. OCTOBER 2020.
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CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ROCHESTER FLOOD STUDY

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	AL
REVIEWED	JC
APPROVED	HZ

PROJECT NO.	CONTROL	REV.	FIGURE
20368084	1000	0	**B-4**

APPENDIX C

Technical Memorandum on Open Water Hydrology Assessment

TECHNICAL MEMORANDUM

DATE March 1, 2021 **Project No.** 20368084-02

TO Abdullah Mamun
Alberta Environment and Parks

CC Jie Chen, Hua Zhang and Dejiang Long, Golder

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

OPEN WATER HYDROLOGY ASSESSMENT – ROCHESTER FLOOD STUDY

1.0 INTRODUCTION

1.1 Study Area and Scope

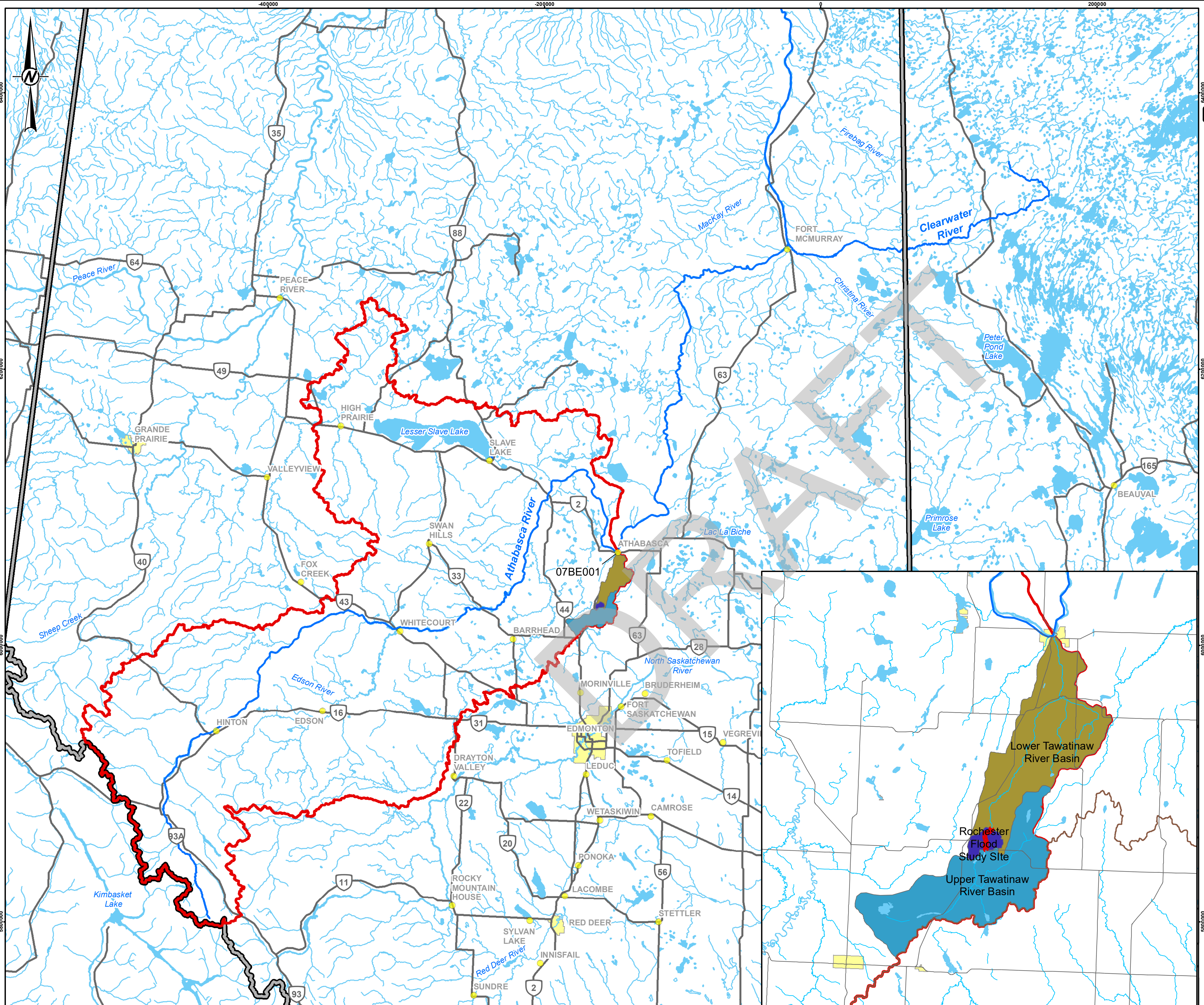
Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in October 2020 to conduct the Rochester Flood Study. The purpose of this study is to assess and identify river and flood hazards along an approximately 7.5 km reach of the Tawatinaw River through the Athabasca County, including the Hamlet of Rochester (Figure 1).

This 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, 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 peak flow information available for the 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 2020.
- **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 in this study.



LEGEND

- MAJOR RIVER
- WATERCOURSE
- WATERBODY
- PRIMARY HIGHWAY
- POPULATED PLACE
- PROVINCIAL BOUNDARY
- Rochester_Flood_Study_Reach
- HUC8_athabasca_basin
- PRIMARY HIGHWAY

0 100 200

1:2,800,000 KILOMETRES

NOTE(S)

SUB-BASINS WERE OBTAINED FROM HUC8 WATERSHEDS (GOVERNMENT OF ALBERTA 2015) WHERE POSSIBLE. OUTSIDE OF ALBERTA, SUB-BASINS WERE OBTAINED FROM AAFC WATERSHED DATA.

REFERENCE(S)

HYDROMETRIC STATIONS OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC).
BASIN AND SUB-BASIN DATA OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC) AND THE GOVERNMENT OF ALBERTA 2015.
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DATUM: NAD 83 CSRS PROJECTION: 3TM 111

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
ROCHESTER FLOOD STUDY

TITLE
TAWATINAW RIVER WATERSHED AT ROCHESTER

CONSULTANT
GOLDER

YYYY-MM-DD	2021-03-01
DESIGNED	BM
PREPARED	BM
REVIEWED	GB
APPROVED	GB

PROJECT NO.	REV.	FIGURE
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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 reach of the 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 of the annual peak flows in 2020 for the gauged regional watersheds, which were provided by Environment and Climate Change Canada, Water Survey of Canada (WSC). 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 2020 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 Tawatinaw River originates south of Rochester and flows in the northerly direction through the hamlet to the Athabasca River. It has a drainage area of approximately 445 km^2 at the Hamlet of Rochester. 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.

The majority of the Hamlet of Rochester is located on the east banks of the Tawatinaw River within a low terraced area. The Tawatinaw River reach near the Hamlet of Rochester is not gauged, and no historic flood flow data are available. Only anecdotal information relating to approximately three significant floods between 1920 and 1950 was recalled by some local residents. The local residents were able to recall flooding extending to the old CNR tracks and having crossed the river using boats (AEP 1997).

From the review of the regional hydrologic data summarized in Appendix A, it appears that the flooding in the Tawatinaw River basin could be caused by either snowmelt, rainfall and snowmelt, or rainfall alone. However, the majority of the recorded annual instantaneous peak flows used for the regional analysis occurred during summer months, indicating that they were associated with rainfall induced flood events.

Stony Creek is a tributary of the Tawatinaw River with its confluence at the Hamlet of Tawatinaw. The gauging station on the creek is located approximately 9 km south of the Hamlet of Rochester. The highest annual maximum instantaneous discharge in Stony Creek was measured in 1997 and 2020. The flood frequency for the 1997 event was estimated to be in the order of 20-year return period and the 2020 event is close to 35-year return period. Since the drainage area of Stony Creek represents about 40% of the drainage basin of the Tawatinaw River at the Hamlet of Rochester, it is reasonable to conclude that similar magnitudes of flood events also occurred in Rochester in 1997 and 2020. The 1997 flood for Stony Creek was recoded on April 21, indicating a snowmelt event or a combination of snowmelt and rainfall event. The 2020 flood for Stony Creek was recorded on June 8, indicating a rainfall event. Theses events were also recorded at the other six sites considered in the regional analysis.

2.0 AVAILABLE FLOW DATA

2.1 Recorded Data

The flood frequency estimates for the Tawatinaw River were derived based on the results of a regional analysis of flood peak discharges.

A summary of the basic hydrologic information used to derive the flood frequency estimates for the study area is provided in Table 1. The data details are provided in Appendix A. The regional hydrometric stations were selected based on their proximity (i.e., relatively close to the study area), size (i.e., reasonable range of gross and effective drainage areas that can be used to establish the regional relationships), and physiographic characteristics (e.g., similar drainage characteristics).

Table 1: Summary of Gauged Stations Considered in the Study

WSC Station Number	WSC Station Name	Latitude	Longitude	Approximate Distance from the Study Area (km)	Gross Drainage Area (km^2)	Effective Drainage Area (km^2)	Period of Record	Length of Record (years)
05EC002	Waskatenau Creek near Waskatenau	54° 07' 23"	112° 46' 58"	50	313	207	1966 – 2020	55
05EC004	Namepi Creek near the Mouth	54° 01' 47"	112° 50' 44"	55	720	586	1975 – 1995	21
05EC005	Redwater River near the Mouth	53° 53' 48"	112° 59' 46"	60	1,596	1,170	1978 – 2020	40
07BE003	Porter Creek above Baptiste Lake	54° 43' 31"	113° 35' 10"	40	57	57	1980 – 2020	38
07BE004	Stoney Creek Near Tawatinaw	54° 17' 35"	113° 27' 49"	9.0	128	113	1980 – 2020	36
07CA003	Flat Creek near Boyle	54° 35' 15"	112° 54' 24"	45	184	97	1919 – 2020	51
07CA005	Pine Creek near Grassland	54° 49' 13"	112° 46' 39"	68	1,456	995	1966 – 2020	54
07CA008	Babette Creek near Colinton	54° 39' 09"	113° 04' 46"	40	219	68	1978 – 2019	42

Note: For comparison, the study reach of the Tawatinaw River is located at latitude of approximately 54° 22' 30" and longitude of approximately 113° 27' 40", and has a gross drainage area of approximately 445 km^2 and effective drainage area of approximately 403 km^2.

2.2 Historic Data

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

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 over the last two decades. Some of these studies included assessments of open water hydrology. These studies include the following:

- Flood Frequency Analysis – Rochester Floodplain Study, Alberta Environment Protection (1994)
- Tawatinaw River at Rochester Flood Risk Mapping Study, Alberta Environment Protection (1997)

- 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)
- Regional Hydro-Climatic and Sediment Modeling (Droppo et al. 2018)
- Athabasca River Flood Hazard Study – (Golder 2020)

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 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 locations 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 peak 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 the available regional flow records and for the return periods ranging from 2 to 1,000 years. The relationships were then used to derive the flood frequency estimates for the Tawatinaw River in the study area.

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

- The drainage areas at the WSC stations were compiled. The gross drainage areas at the ungauged location of Tawatinaw River was estimated in a GIS analysis.
- The flood frequency estimates for the WSC stations (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 for the Tawatinaw River for the various return periods and the 95% confidence intervals.

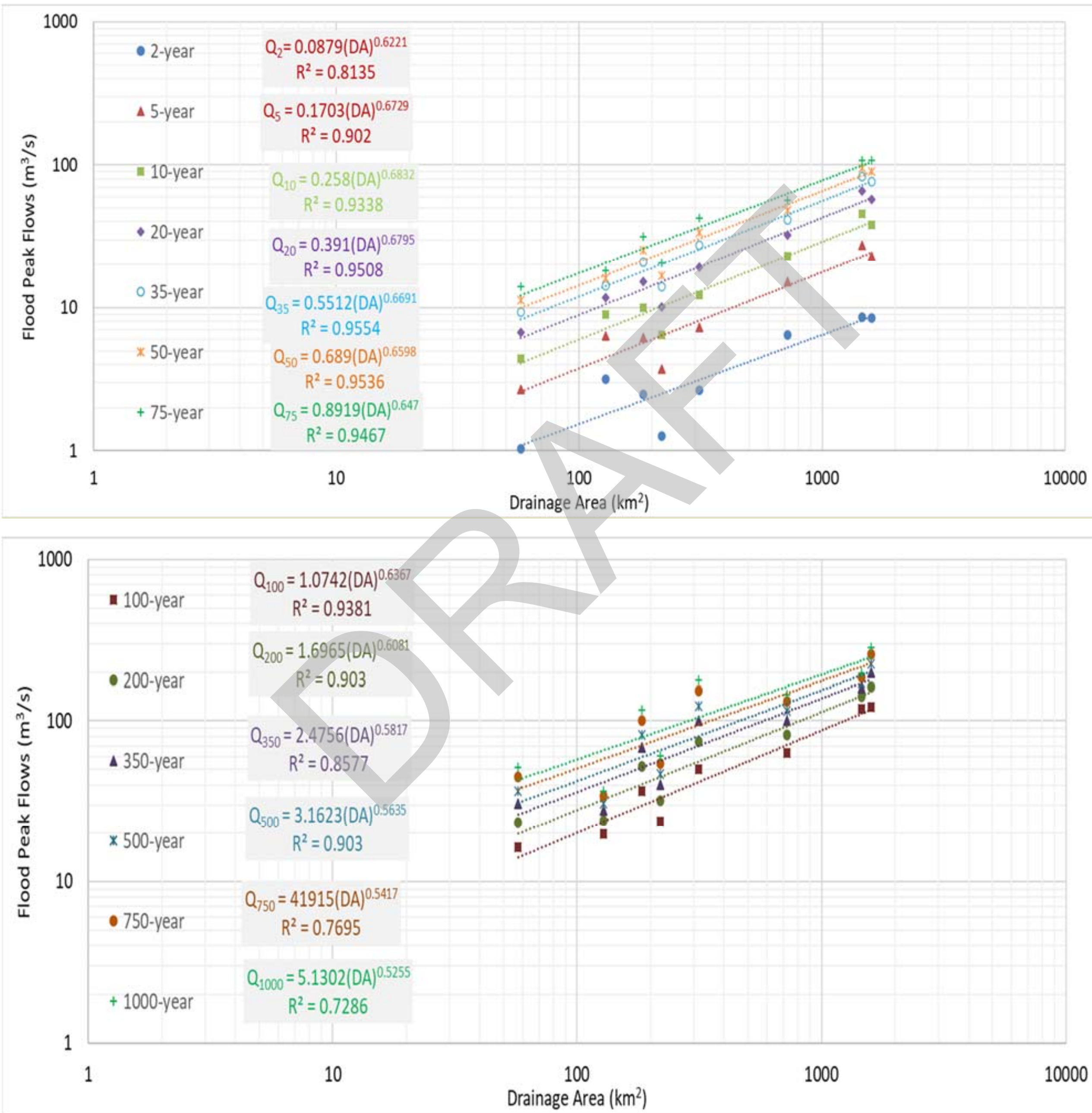

Figure 2: Empirical Relationships between Flood Peak Flows and Drainage Areas for the Regional Stations

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)
- Weibull distribution (Moment).

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

4.1.2 Results

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 discharge 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 (05EC002), Redwater River near the mouth (05EC005), and Porter Creek above Baptiste Lake (07BE003), displays dependence at both the 5% and 1% level of significance. This does not appear to be due to any large-scale climate variation (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, these stations were included from use in developing the regional relationships.
- The annual maximum instantaneous discharge series for Waskatenau Creek near Waskatenau (05EC002), displays non-randomness at the 5% and 1% level of significance. Obtaining data that is perfectly random 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 Waskatenau Creek near Waskatenau was included in developing the regional relationships.

4.2 Flood Frequency Estimates

Flood frequency analyses of the annual maximum instantaneous discharge series (that includes the preliminary estimates of the 2020 flood flows) for the regional stations, were conducted to estimate the flood peak 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). 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.

The flood frequency estimates for Tawatinaw River at Hamlet of Rochester are derived using the flood frequency estimates for Stony Creek near Tawatinaw and the area power factor developed based on the regional relationship. The flood discharge estimates and the associated upper and lower 95% confidence intervals for Tawatinaw River at Hamlet of Rochester, are summarized in Table 2.

Table 2: Recommended Flood Frequency Estimates for Tawatinaw River at Hamlet of Rochester

Return Periods (years)	Annual Probability of Exceedance (%)	Value (m^3/s)	Lower 95% Limit (m^3/s)	Upper 95% limit (m^3/s)
2	50	6.9	4.8	10.1
5	20	14.7	10.7	20.0
10	10	20.9	15.2	27.1
20	5.0	27.4	19.3	34.4
35	2.9	32.9	22.2	40.5
50	2.0	36.5	23.9	44.8
75	1.3	40.7	25.8	50.2
100	1.0	43.8	26.9	54.5
200	0.50	51.4	29.3	66.5
350	0.29	57.8	30.8	77.8
500	0.20	62.0	31.7	85.7
750	0.13	66.9	33.0	96.4
1,000	0.10	70.4	33.6	103.6

4.3 Comparison to Previous Studies

A comparison of the flood frequency estimates obtained in this study for the Tawatinaw River with the studies previously completed by Alberta Environment and Parks (AEP 1994) as well as IBI and Golder (2014), is provided in Table 3.

The flood frequency estimates were based on the recorded data up to 1989 in the AEP study and up to 2011 in the IBI and Golder study (2014). The current study is based on the published flow data up to 2019, and the provisional flow data for 2020 for the regional gauging stations. In addition, this study includes the analyses to update the relationships between annual maximum daily and annual maximum instantaneous discharges for regional stations.

The comparison in Table 3 shows that 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 estimated watershed area of the Tawatinaw River at the Hamlet of Rochester in the IBI and Golder (2014) study and this study.

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

Return Period (years)	Flood Peak Discharge of Tawatinaw River at the Hamlet of Rochester (m^3/s)		
	AEP (1994)	IBI and Golder (2014)	This Study
2	4.99	4.0	6.9
5	12.7	10.0	14.7
10	19.5	15.0	20.9
20	27.2	20.0	27.4
25	-	22.0	-
50	39.1	27.0	36.5
100	49.5	36.0	43.8
500	-	47.0	62.0

Notes:
1. The AEP (1994) study involved use of the recorded data up to 1989 for the regional stations.
2. The IBI and Golder (2014) study involved use of the recorded data from 1913 to 2011 for the regional stations.

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 that includes Tawatinaw River 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 Tawatinaw River are expected to be small for the median climate change projections.

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 Tawatinaw River at Hamlet of Rochester. These estimates provide the updated flood hydrology information as inputs to the other components of the study (e.g., hydraulic modelling). A summary of 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, is provided in Table 2.
- This study includes preliminary estimates of the annual maximum instantaneous discharges in 2020. 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 regional flood frequency analyses is less than 55 years. Therefore, there are large uncertainties (i.e., the confidence intervals are very large) with flood frequency estimates for return periods greater than 50 years.

7.0 CLOSURE

This memorandum was prepared and reviewed by the undersigned.

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GOLDER ASSOCIATES LTD

RM SIGNATURE: ____________

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DATE: March 1, 2021

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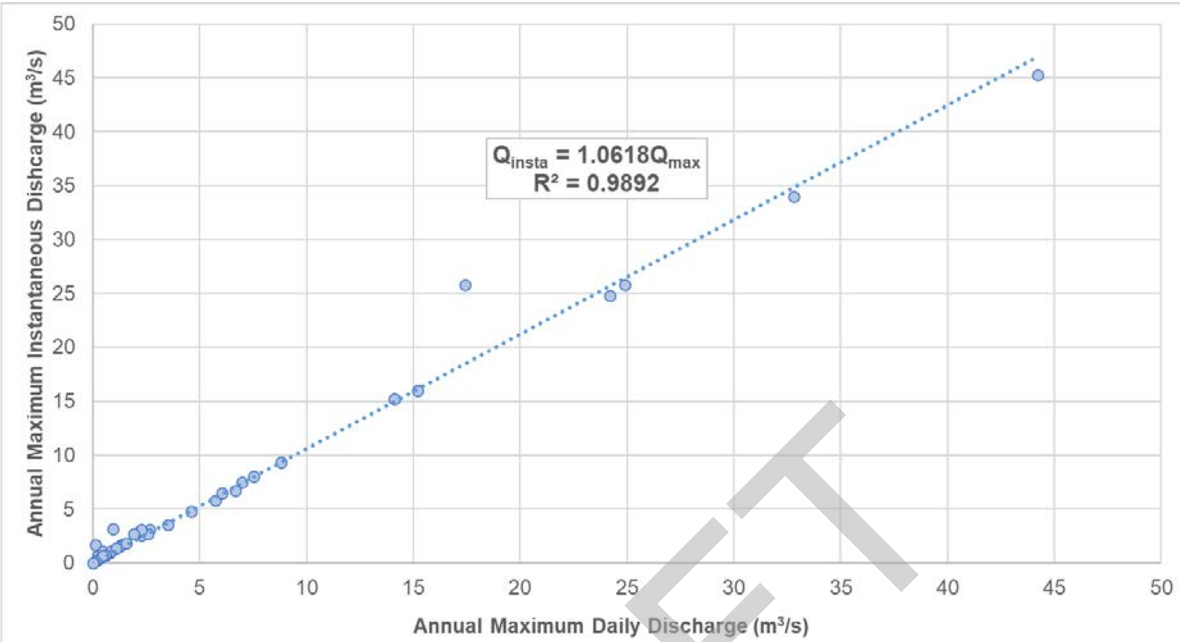
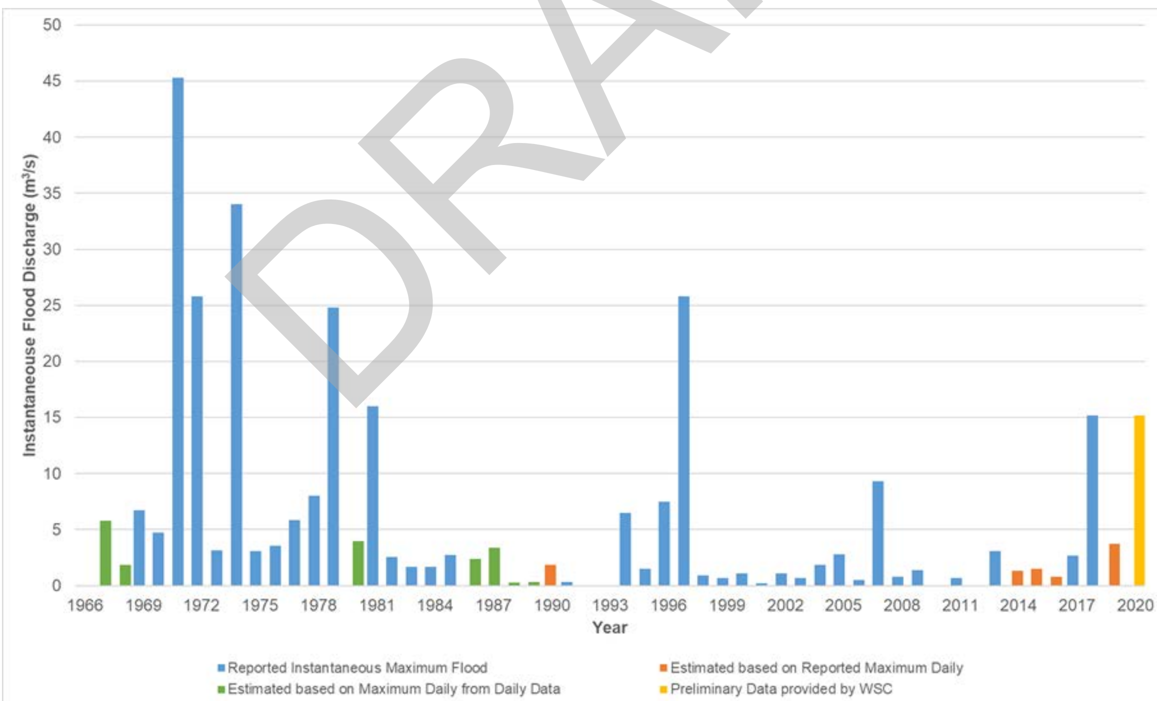
APPENDIX A

Graphical and Tabulated Summaries of Flood Flow Series at Gauged Stations

Figure A-1: WSC Station No. 05EC002, Waskatenau Creek near Waskatenau

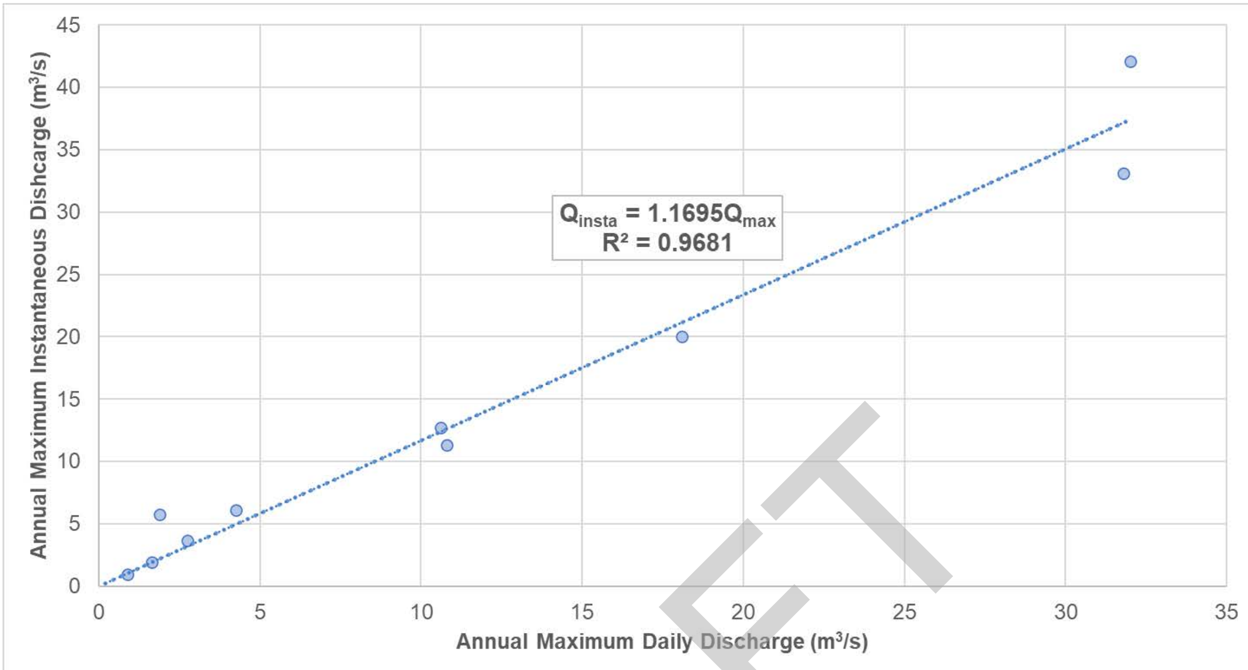


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Waskatenau Creek near Waskatenau (WSC Station No. 05EC002)

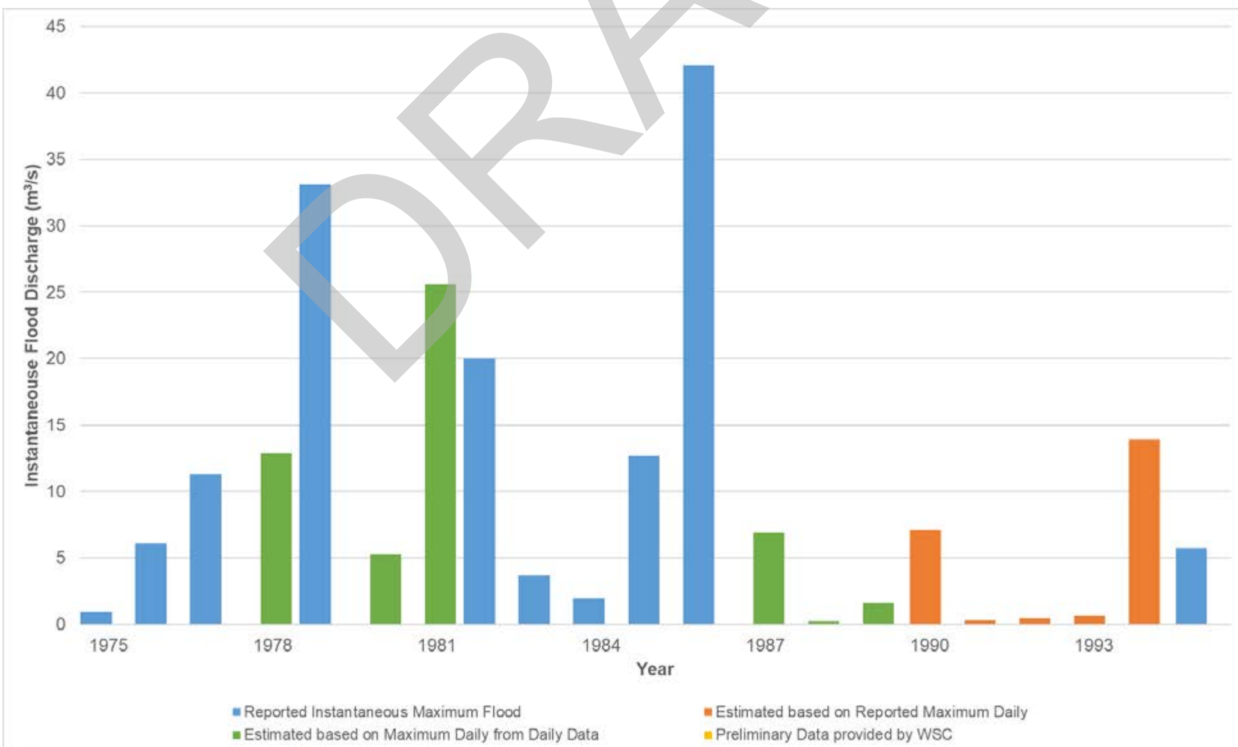


Maximum Instantaneous Flood Flow Series at Waskatenau Creek near Waskatenau (WSC Station No. 05EC002)

Figure A-2: WSC Station No. 05EC004, Namepi Creek near the Mouth

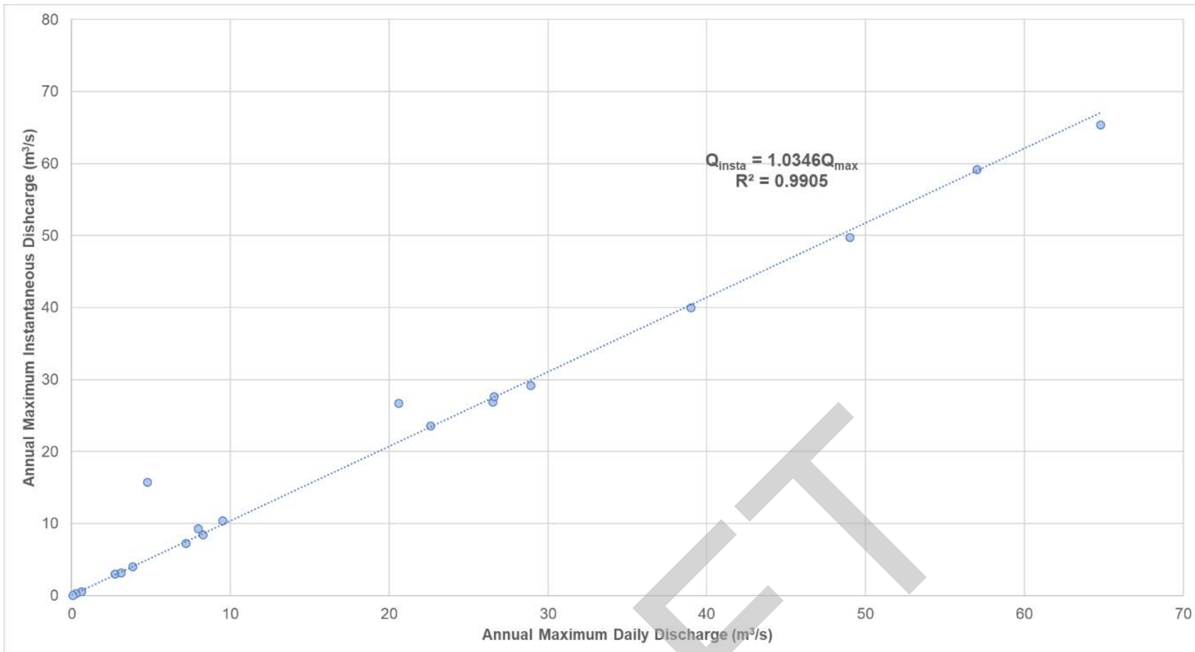


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Namepi Creek near the Mouth (WSC Station No. 05EC004)

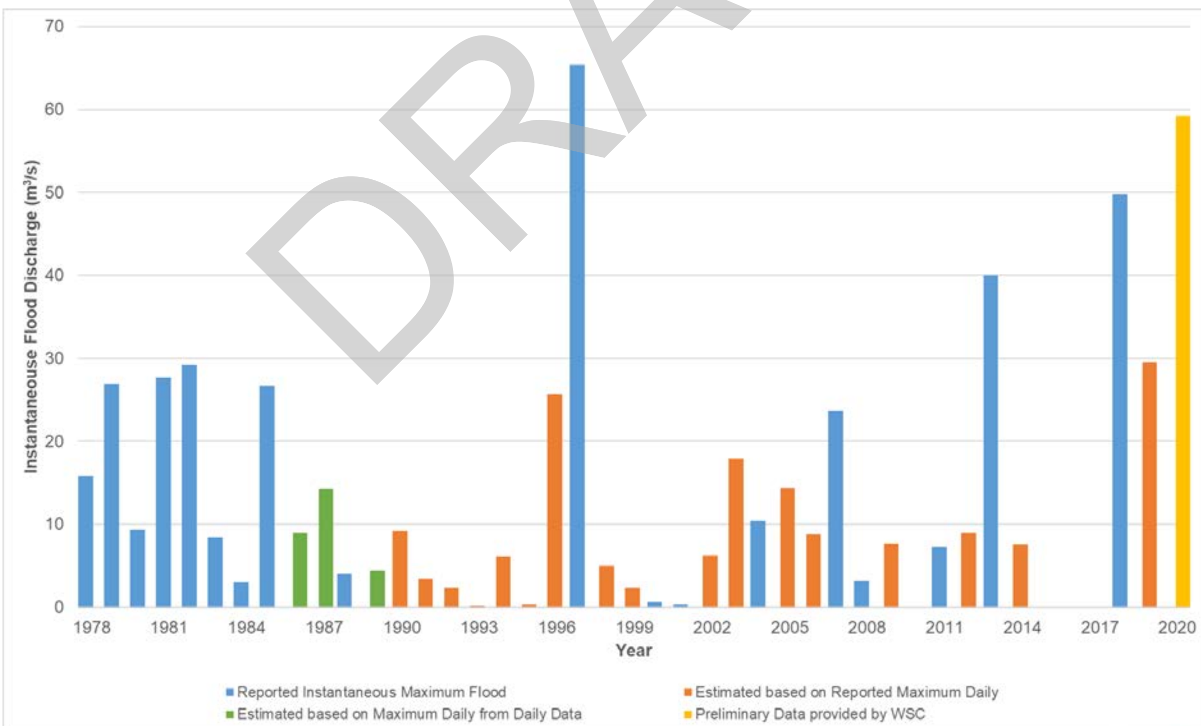


Maximum Instantaneous Flood Flow Series at Namepi Creek near the Mouth (WSC Station No. 05EC004)

Figure A-3: WSC Station No. 05EC005, Redwater River near the Mouth

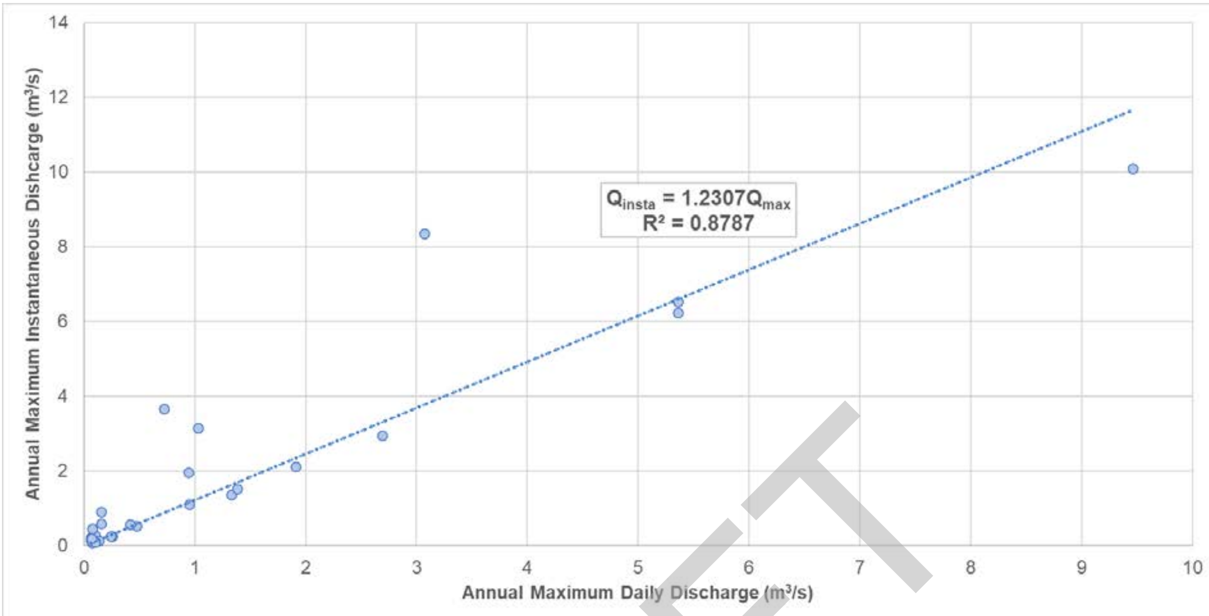


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Redwater River near the Mouth (WSC Station No. 05EC005)

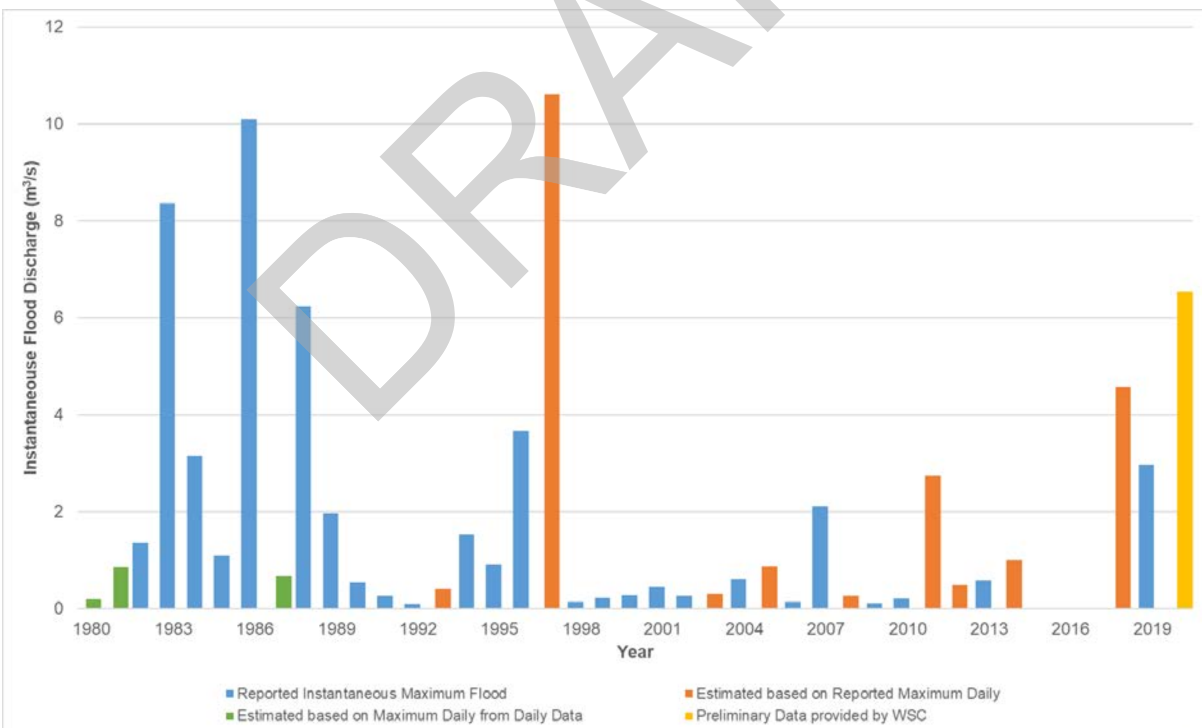


Maximum Instantaneous Flood Flow Series at Redwater River near the Mouth (WSC Station No. 05EC005)

Figure A-4: WSC Station No. 07BE003, Porter Creek above Baptiste Lake

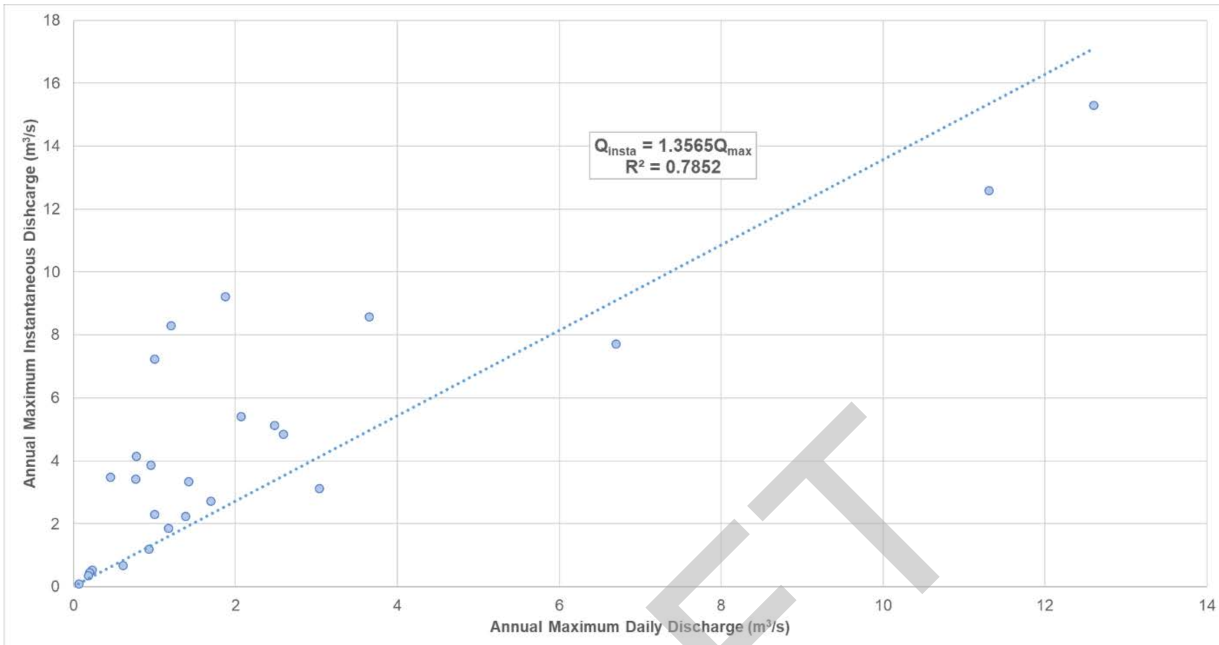


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Porter Creek above Baptiste Lake (WSC Station No. 07BE003)

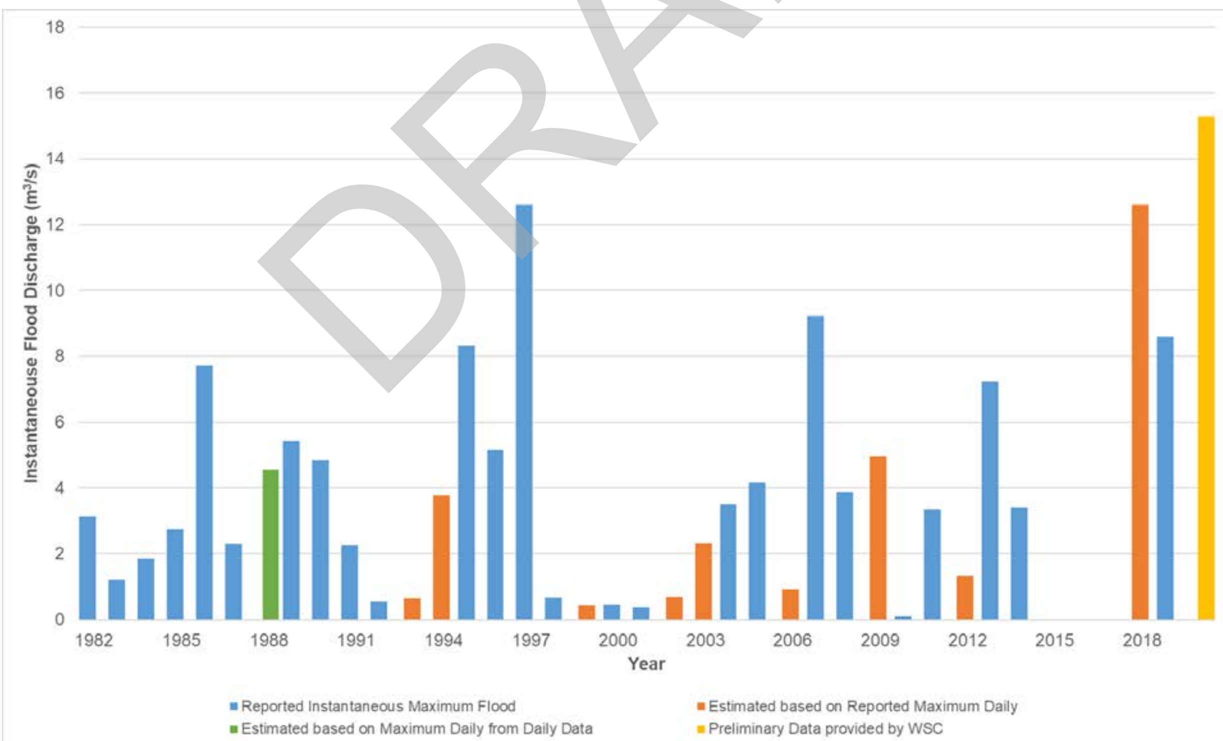


Maximum Instantaneous Flood Flow Series at Clearwater River at Porter Creek above Baptiste Lake (WSC Station No. 07BE003)

Figure A-5: WSC Station No. 07BE004, Stony Creek Near Tawatinaw

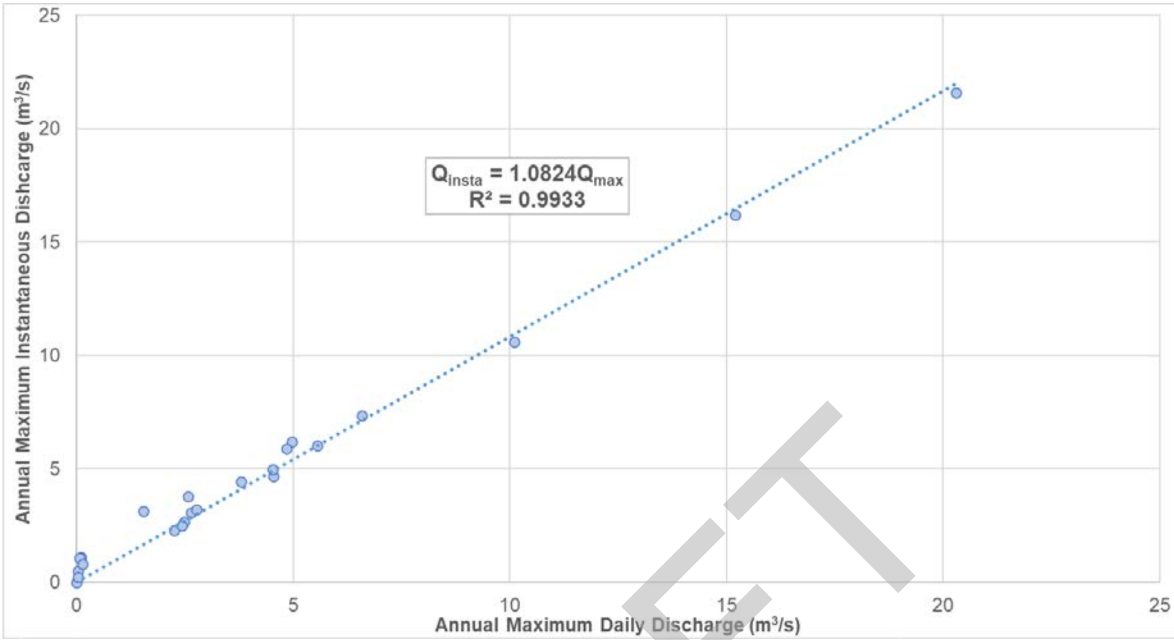


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Stony Creek near Tawatinaw (WSC Station No. 07BE004)

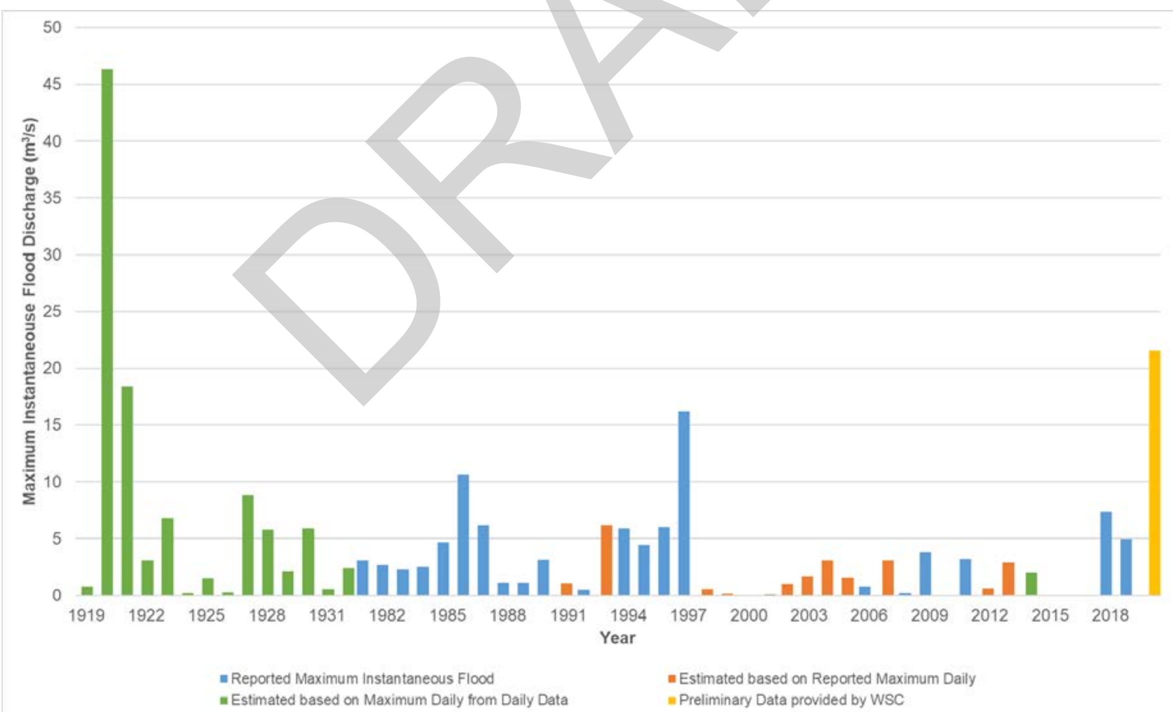


Maximum Instantaneous Flood Flow Series at Clearwater River at Stony Creek near Tawatinaw (WSC Station No. 07BE004)

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

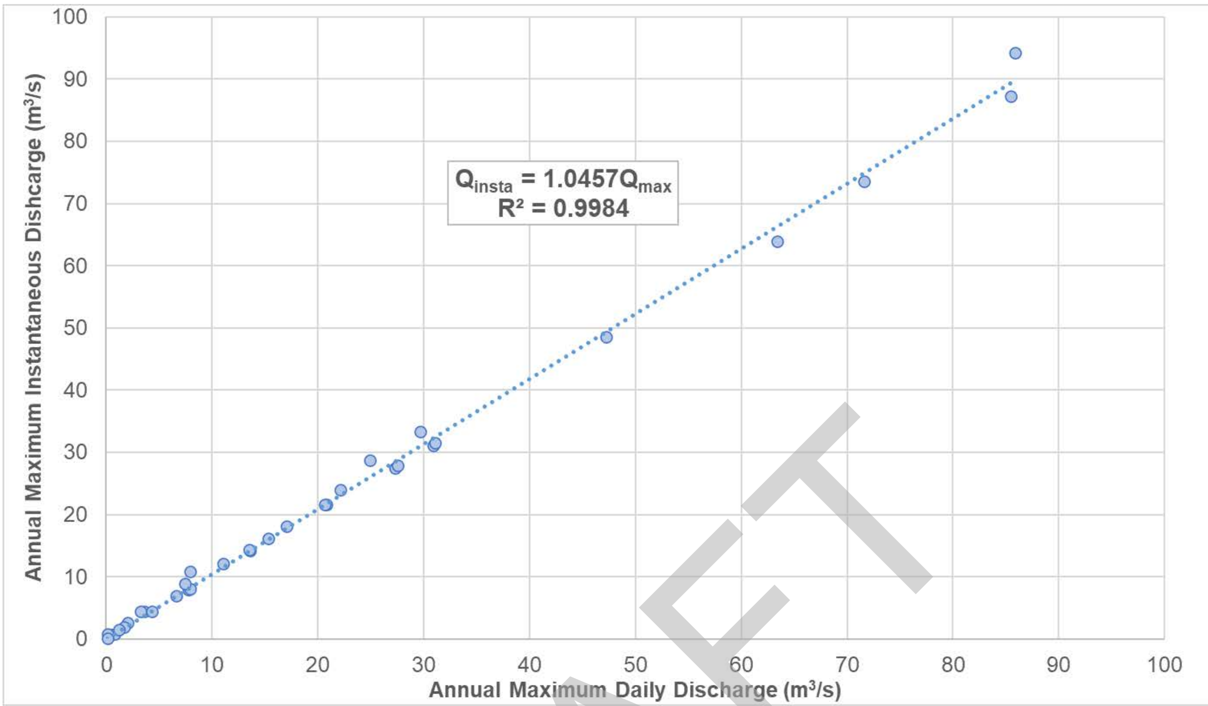


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Flat Creek near Boyle (WSC Station No. 07CA003)

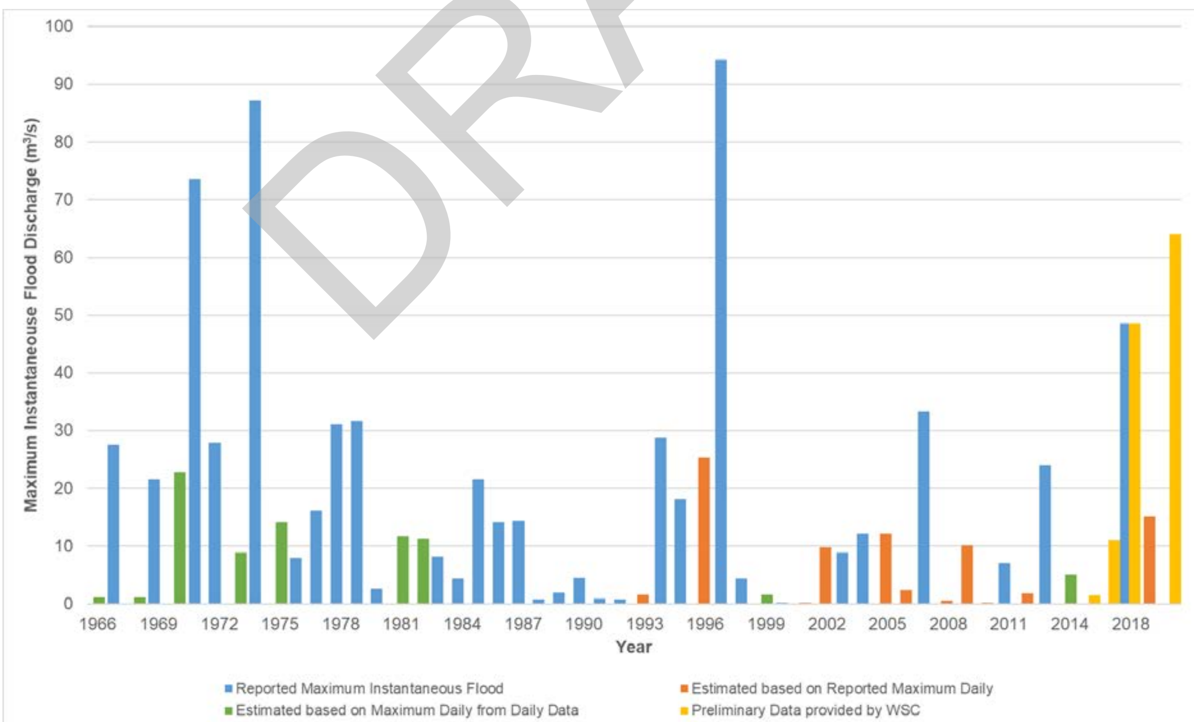


Maximum Instantaneous Flood Flow Series at Flat Creek near Boyle (WSC Station No. 07CA003)

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

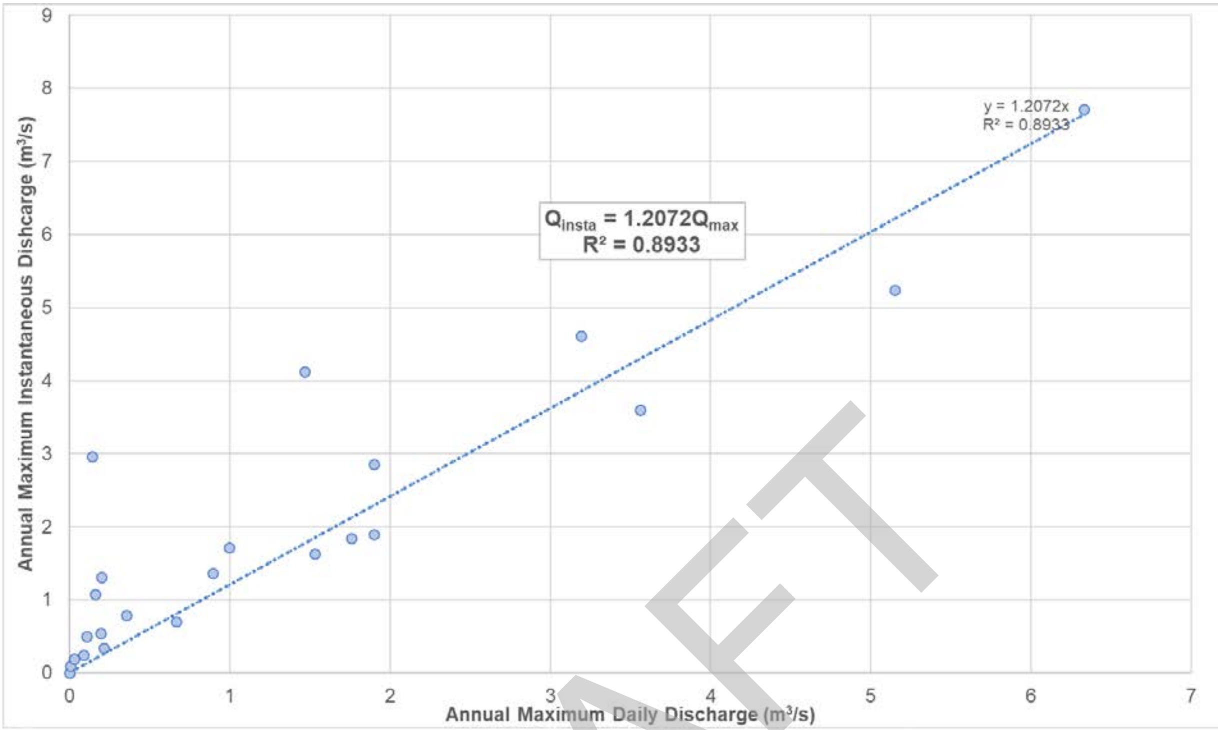


Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges at Pine Creek near Grassland (WSC Station No. 07CA005)

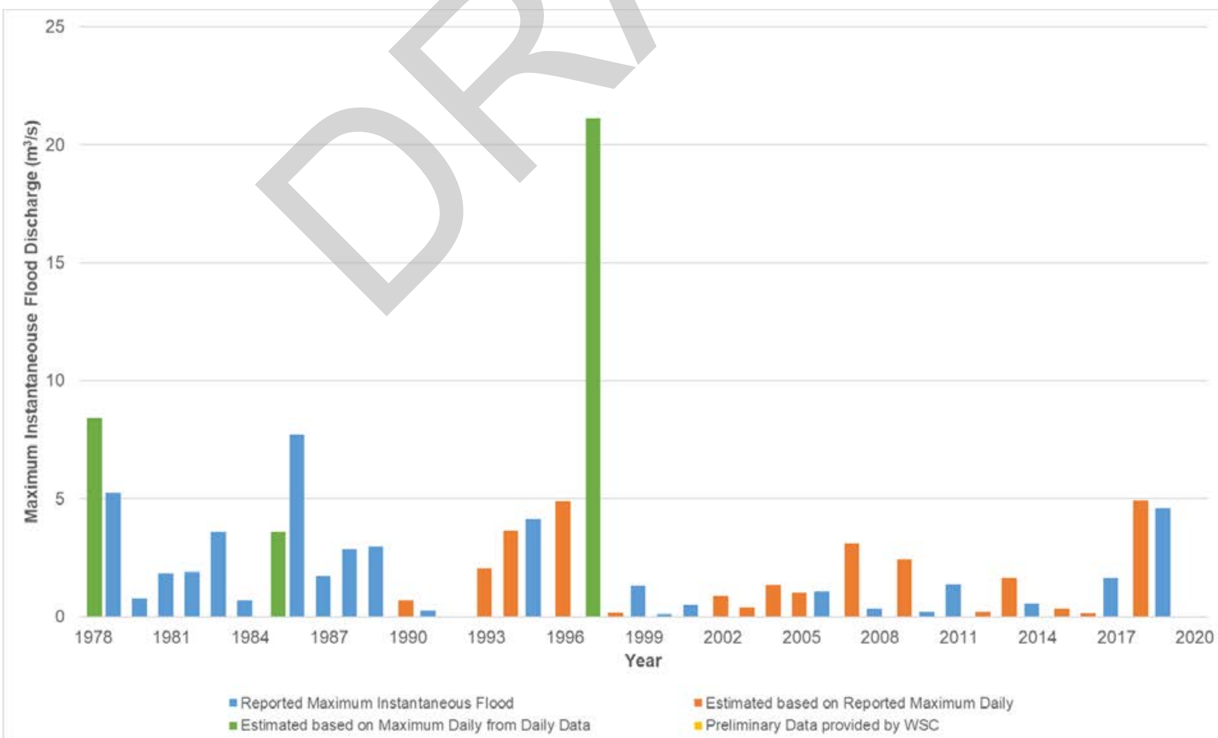


Maximum Instantaneous Flood Flow Series at Pine Creek near Grassland (WSC Station No. 07CA005)

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)

Year	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07BE004, Stony Creek near Tawatinaw	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1919	-	-	-	-	-	0.80	-	-
1920	-	-	-	-	-	46.33	-	-
1921	-	-	-	-	-	18.40	-	-
1922	-	-	-	-	-	3.10	-	-
1923	-	-	-	-	-	6.81	-	-
1924	-	-	-	-	-	0.21	-	-
1925	-	-	-	-	-	1.52	-	-
1926	-	-	-	-	-	0.28	-	-
1927	-	-	-	-	-	8.83	-	-
1928	-	-	-	-	-	5.76	-	-
1929	-	-	-	-	-	2.18	-	-
1930	-	-	-	-	-	5.92	-	-
1931	-	-	-	-	-	0.55	-	-
1932	-	-	-	-	-	-	-	-
1933	-	-	-	-	-	-	-	-
1934	-	-	-	-	-	-	-	-
1935	-	-	-	-	-	-	-	-
1936	-	-	-	-	-	-	-	-
1937	-	-	-	-	-	-	-	-
1938	-	-	-	-	-	-	-	-
1939	-	-	-	-	-	-	-	-
1940	-	-	-	-	-	-	-	-
1941	-	-	-	-	-	-	-	-
1942	-	-	-	-	-	-	-	-
1943	-	-	-	-	-	-	-	-
1944	-	-	-	-	-	-	-	-
1945	-	-	-	-	-	-	-	-
1946	-	-	-	-	-	-	-	-
1947	-	-	-	-	-	-	-	-
1948	-	-	-	-	-	-	-	-
1949	-	-	-	-	-	-	-	-
1950	-	-	-	-	-	-	-	-

Year	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07BE004, Stony Creek near Tawatinaw	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1951	-	-	-	-	-	-	-	-
1952	-	-	-	-	-	-	-	-
1953	-	-	-	-	-	-	-	-
1954	-	-	-	-	-	-	-	-
1955	-	-	-	-	-	-	-	-
1956	-	-	-	-	-	-	-	-
1957	-	-	-	-	-	-	-	-
1958	-	-	-	-	-	-	-	-
1959	-	-	-	-	-	-	-	-
1960	-	-	-	-	-	-	-	-
1961	-	-	-	-	-	-	-	-
1962	-	-	-	-	-	-	-	-
1963	-	-	-	-	-	-	-	-
1964	-	-	-	-	-	-	-	-
1965	-	-	-	-	-	-	-	-
1966	0.01	-	-	-	-	-	1.16	-
1967	5.78	-	-	-	-	-	27.50	-
1968	1.87	-	-	-	-	-	1.10	-
1969	6.74	-	-	-	-	-	21.60	-
1970	4.76	-	-	-	-	-	22.69	-
1971	45.30	-	-	-	-	-	73.60	-
1972	25.80	-	-	-	-	-	27.90	-
1973	3.14	-	-	-	-	-	8.89	-
1974	34.00	-	-	-	-	-	87.20	-
1975	3.06	0.94	-	-	-	-	14.12	-
1976	3.54	6.12	-	-	-	-	7.96	-
1977	5.83	11.30	-	-	-	-	16.20	-
1978	8.01	12.86	15.80	-	-	-	31.10	8.41
1979	24.80	33.10	26.90	-	-	-	31.60	5.24
1980	3.95	5.25	9.31	0.19	-	2.36	2.62	0.79
1981	16.00	25.61	27.70	0.86	-	3.09	11.71	1.84
1982	2.58	20.00	29.20	1.36	3.13	2.68	11.19	1.90

Year	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07BE004, Stony Creek near Tawatinaw	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
1983	1.69	3.69	8.46	8.36	1.21	2.29	8.13	3.60
1984	1.70	1.91	3.01	3.15	1.86	2.51	4.42	0.70
1985	2.74	12.70	26.70	1.10	2.73	4.66	21.60	3.59
1986	2.41	42.10	8.94	10.10	7.73	10.60	14.20	7.71
1987	3.41	6.92	14.28	0.68	2.30	6.20	14.40	1.72
1988	0.27	0.22	4.00	6.23	4.56	1.11	0.73	2.86
1989	0.35	1.64	4.37	1.96	5.42	1.08	2.00	2.96
1990	1.87	7.13	9.19	0.54	4.86	3.15	4.50	0.70
1991	0.34	0.30	3.46	0.26	2.25	1.04	0.86	0.24
1992	0.00	0.44	2.33	0.09	0.53	0.50	0.78	0.00
1993	0.02	0.65	0.11	0.40	0.66	6.21	1.54	2.05
1994	6.48	13.92	6.12	1.53	3.78	5.88	28.70	3.67
1995	1.51	5.74	0.32	0.91	8.31	4.44	18.20	4.12
1996	7.51	-	25.66	3.66	5.14	6.01	25.31	4.89
1997	25.80	-	65.40	10.62	12.60	16.20	94.20	21.13
1998	0.94	-	5.02	0.13	0.68	0.54	4.43	0.15
1999	0.70	-	2.32	0.22	0.43	0.14	1.54	1.31
2000	1.07	-	0.59	0.28	0.45	0.00	0.20	0.10
2001	0.20	-	0.27	0.45	0.37	0.07	0.14	0.50
2002	1.11	-	6.21	0.27	0.68	0.98	9.81	0.87
2003	0.70	-	17.90	0.31	2.31	1.70	8.87	0.41
2004	1.86	-	10.40	0.60	3.49	3.11	12.20	1.34
2005	2.82	-	14.38	0.86	4.15	1.57	12.13	1.03
2006	0.51	-	8.79	0.14	0.92	0.79	2.45	1.08
2007	9.32	-	23.60	2.11	9.22	3.07	33.30	3.10
2008	0.80	-	3.19	0.26	3.87	0.22	0.55	0.34
2009	1.38	-	7.64	0.10	4.95	3.80	10.13	2.41
2010	0.00	-	0.06	0.20	0.09	0.06	0.24	0.19
2011	0.68	-	7.25	2.74	3.35	3.22	7.00	1.36
2012	0.00	-	8.91	0.49	1.33	0.64	1.85	0.20
2013	3.11	-	40.00	0.57	7.25	2.89	24.00	1.63
2014	1.31	-	7.49	0.99	3.42	2.02	4.97	0.54

Year	07EC002, Waskatenau Creek near Waskatenau	07EC004, Namepi Creek near the Mouth	07EC005, Redwater River near the Mouth	07BE003, Porter Creek above Baptiste Lake	07BE004, Stony Creek near Tawatinaw	07CA003, Flat Creek near Boyle	07CA005, Pine Creek near Grassland	07CA008, Babette Creek near Colinton
2015	1.53	-	-	-	-	-	-	0.34
2016	0.78	-	-	-	-	-	1.54	0.14
2017	2.69	-	-	-	-	-	10.94	1.63
2018	15.20	-	49.80	4.61	12.62	7.35	48.54	4.93
2019	3.73	-	29.59	2.96	8.59	4.96	15.06	4.61
2020	15.20	-	59.20	6.53	15.30	21.60	64.00	-
Maximum	45.30	42.10	65.40	10.62	15.30	46.33	94.20	21.13
Mean	5.76	10.12	14.85	2.02	4.18	4.69	16.88	2.53
Minimum	0.00	0.22	0.06	0.09	0.09	0.00	0.14	0.00
Standard Deviation	9.21	11.49	16.10	2.81	3.82	7.47	21.20	3.57

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	07BE004	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	Stony Creek near Tawatinaw	Flat Creek near Boyle	Pine Creek near Grassland	Babette Creek near Colinton
Anderson-Darling statistic, $A^2 = -N - S$									
3 Parameter Log-normal		0.472	0.602	0.385	0.163	0.484	0.404	0.848	0.268
Extreme Value		0.763	0.386	0.492	1.166	0.397	0.328	0.713	0.356
Log-Pearson III		5.357	0.243	1.403	0.253	0.249	3.533	0.374	0.803
Weibull		4.271	0.538	0.849	2.568	0.431	1.165	1.328	3.343
Serial correlation coefficient test for independence									
S_1		0.4172	0.2677	0.3901	0.3888	0.3739	0.2026	0.2198	0.3051
t		3.3108	1.1786	2.5769	2.4967	2.3162	1.4333	1.6089	2.0004
t(α=0.05)		1.6747	1.7341	1.6871	1.6896	1.6924	1.6772	1.6753	1.6849
t(α=0.01)		2.4002	2.5524	2.4314	2.4377	2.4448	2.4066	2.4017	2.4258
Spearman rank order correlation coefficient test for no-trend									
r_s		0.3190	0.3065	-0.0486	0.0831	-0.2201	0.1247	0.1537	0.3036
t		2.4507	1.4035	-0.2999	0.5000	-1.3155	0.8798	1.1213	2.0154
t(α=0.05)		2.0057	2.0930	-2.0244	2.0281	-2.0322	2.0096	2.0066	2.0211
t(α=0.01)		2.6718	2.8609	-2.7116	2.7195	-2.7284	2.6800	2.6737	2.7045
Mann-Whitney split sample test for homogeneity									
Size of earlier sample		28	11	20	20	20	26	27	21
z		-1.5827	-1.4788	-0.1353	-0.9940	-1.3053	-0.6972	-0.6142	-2.4275
z(a=0.05)		-1.6449		-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	-2.3263
Test of general randomness (Runs for above or below the median)									
Median		2.4	6.1	8.9	0.8	3.4	2.7	10.5	1.5
N1(for Q>=Median)		28	11	20	19	18	26	27	21
N2(for Q<Median)		27	10	20	19	18	25	27	21
Run_ab		18	11	15	14	12	26	22	15
z		2.8567	0.2137	1.9222	1.9735	2.3674	0.1387	1.6486	2.1871
z(a=0.05)		1.9600		1.9600			1.9600	1.9600	1.9600
z(a=0.01)		2.5758		2.5758			2.5758	2.5758	2.5758

Notes:

1. *Selected distribution based on best statistical fit* *0.472*
2. *Criteria for the respective statistical tests were not met* *2.5758*

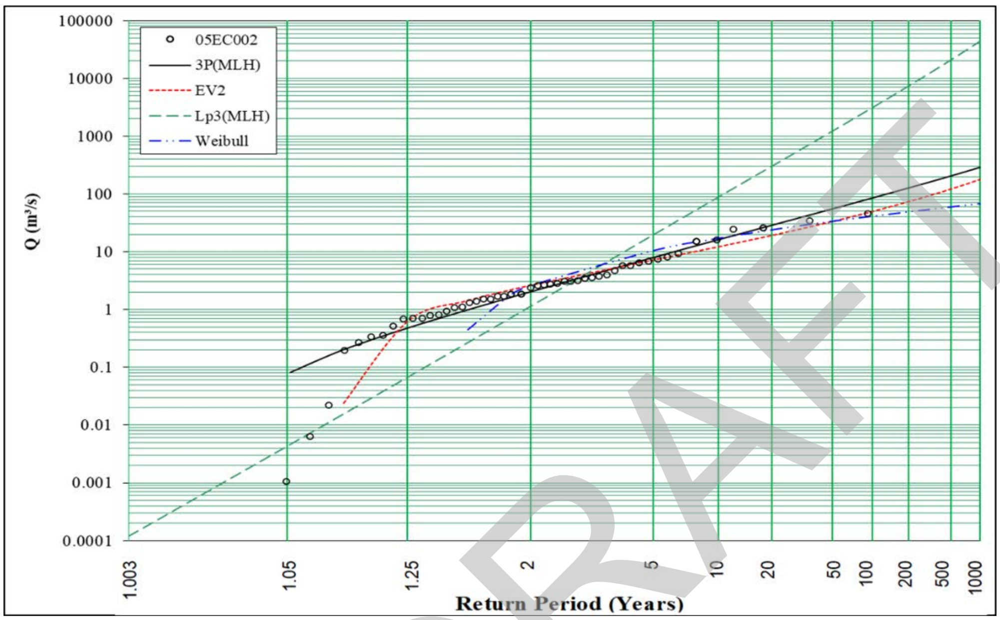
APPENDIX B

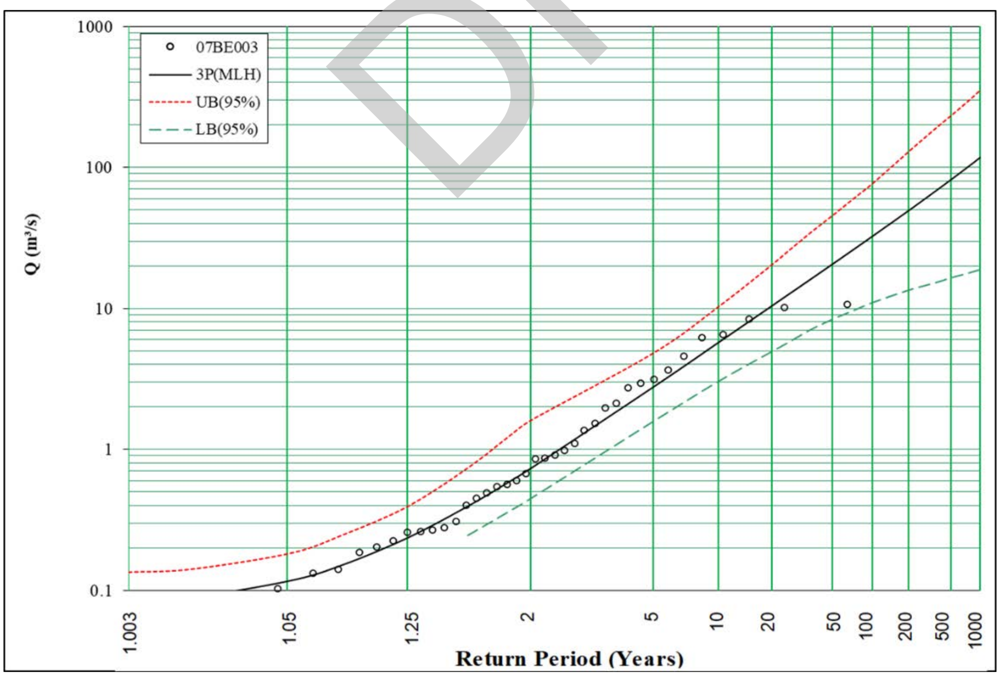
Frequency Analyses - Graphs and Tables

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.

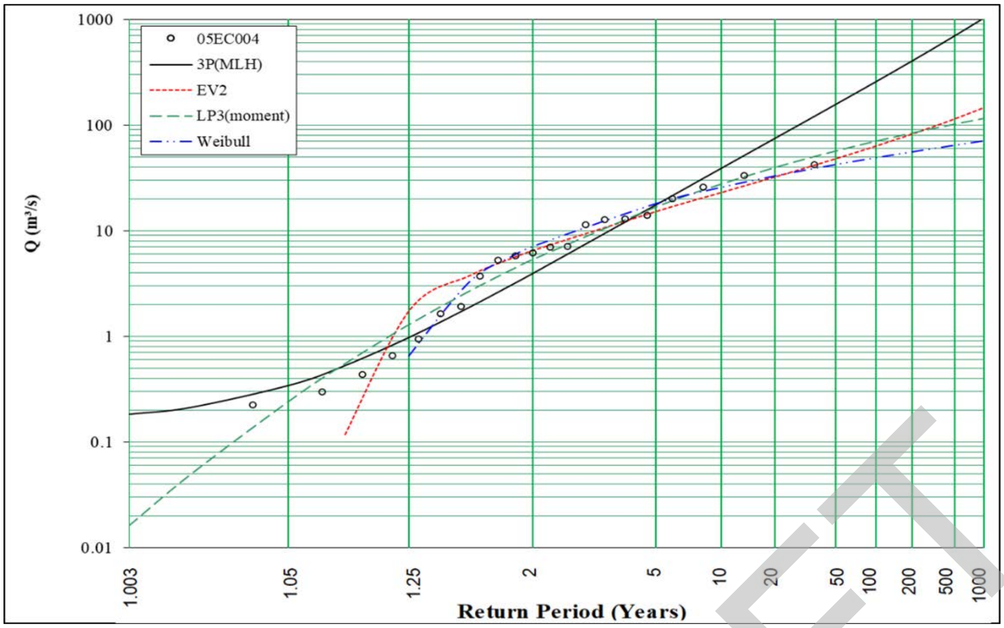
Figure B-1: WSC Station No. 05EC002, Waskatenau Creek near Waskatenau

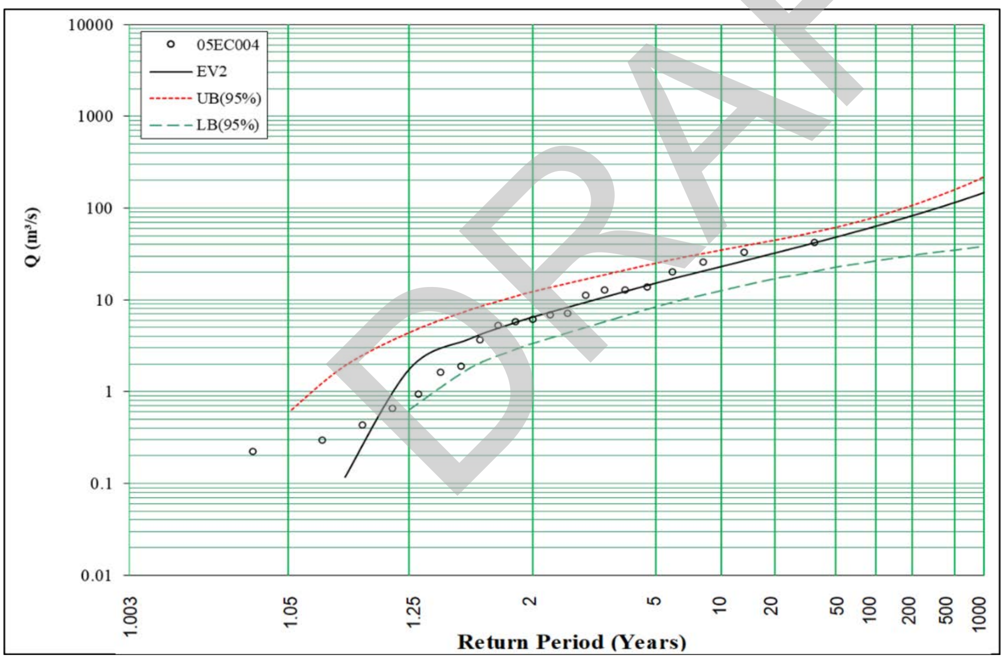




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	2.0	2.6	1.1	2.6
5	7.9	7.3	20	10.5
10	16.0	12.3	87	17.1
20	28.6	19.4	302	24.0
35	43.2	27.3	727	29.8
50	55.0	33.7	1223	33.7
75	71.2	42.6	2136	38.1
100	84.8	50.2	3117	41.2
200	126	74.1	7354	49.0
350	170	101	14053	55.5
500	204	123	20856	59.6
750	249	153	32185	64.4
1000	286	179	43400	67.8

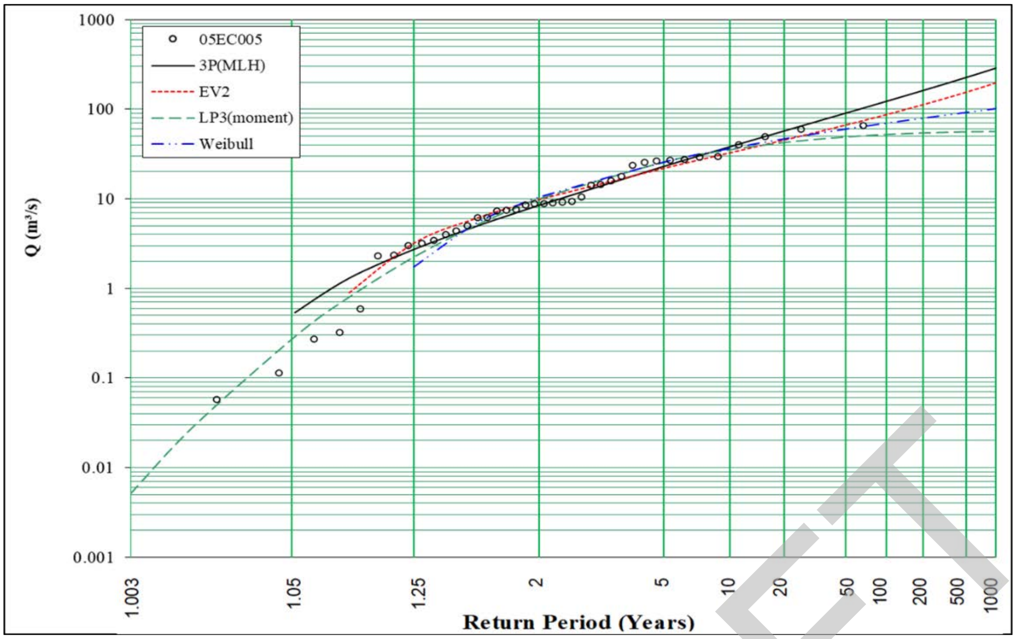
Figure B-2: WSC Station No. 05EC004, Namepi Creek near the Mouth

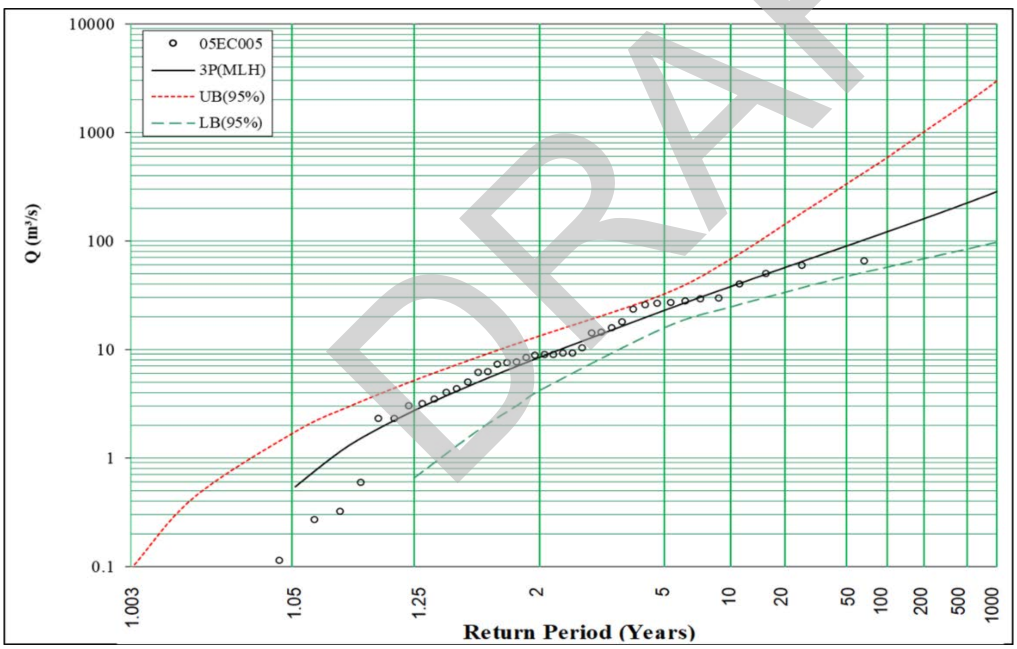




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	3.9	6.5	5.4	7.1
5	17.5	15.2	16.8	18.0
10	38.7	22.9	27.6	25.6
20	74.8	32.3	39.7	32.9
35	119.2	41.4	50.1	38.6
50	156.9	48.1	57.0	42.1
75	210.7	56.6	64.9	46.1
100	257.2	63.4	70.6	48.9
200	404	82	84	55.6
350	569	101	95	60.9
500	700	115	102	64.2
750	879	132	110	67.9
1000	1028	146	116	70.6

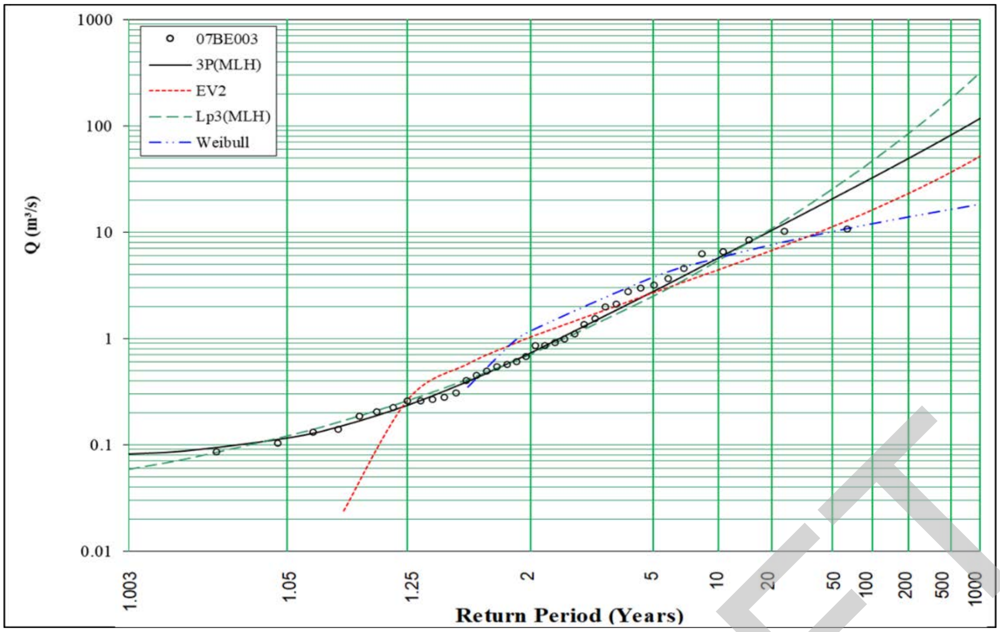
Figure B-3: WSC Station No. 05EC005, Redwater River near the Mouth

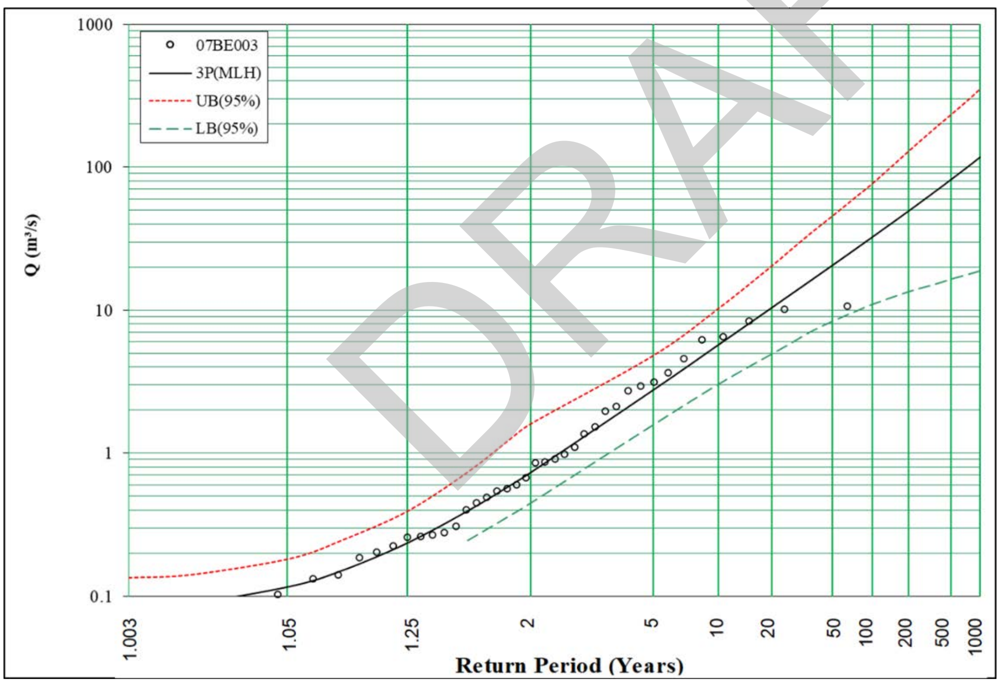




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	8.5	9.9	10.0	10.4
5	23.0	22.0	25.8	25.6
10	38.0	32.7	35.4	36.4
20	57.2	45.6	42.7	46.8
35	76.3	57.9	47.1	55.0
50	90.3	67.0	49.3	60.1
75	108	78.5	51.3	65.9
100	122	87.5	52.5	70.0
200	161	113	54.6	79.7
350	199	138	55.7	87.4
500	226	156	56.2	92.3
750	259	179	56.5	97.8
1000	285	197	56.7	102

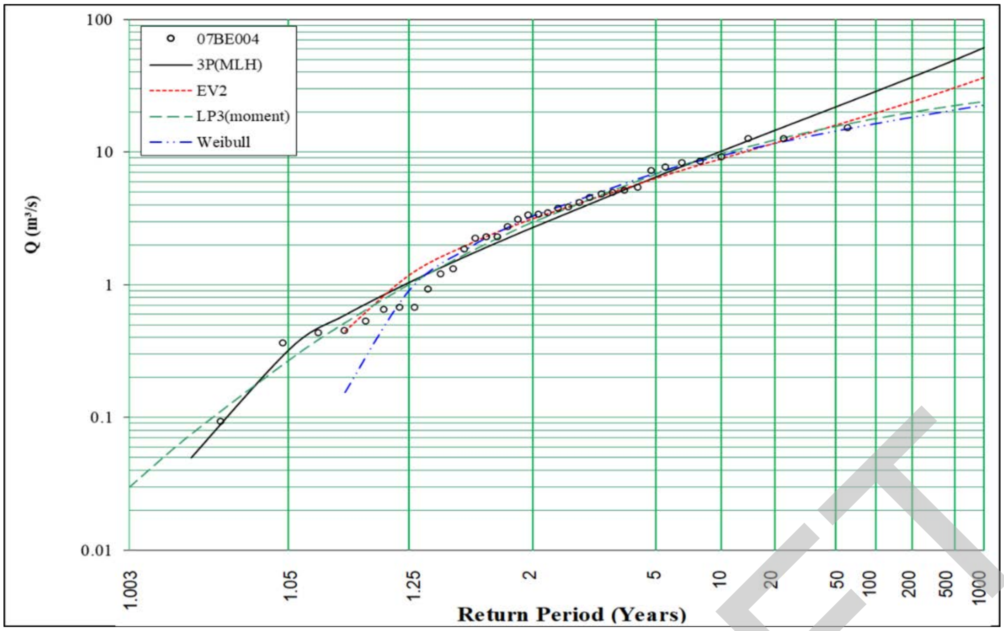
Figure B-4: WSC Station No. 07BE003, Porter Creek above Baptiste Lake

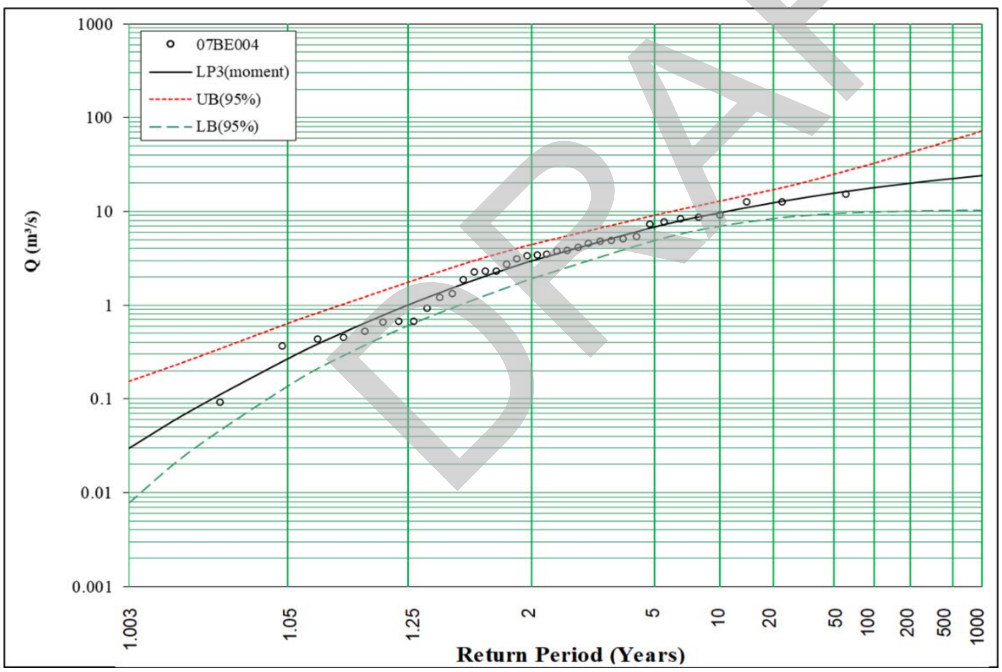




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	0.7	1.0	0.7	1.2
5	2.8	2.7	2.5	3.8
10	5.7	4.4	5.4	5.7
20	10.4	6.8	10.9	7.6
35	16.0	9.3	18.5	9.1
50	20.6	11.3	25.7	10.1
75	27.0	14.0	36.9	11.2
100	32.5	16.3	47.4	12.0
200	49.3	23.2	85.6	13.9
350	67.5	30.7	136	15.4
500	81.8	36.7	183	16.4
750	101	44.8	253	17.4
1000	117	51.7	319	18.2

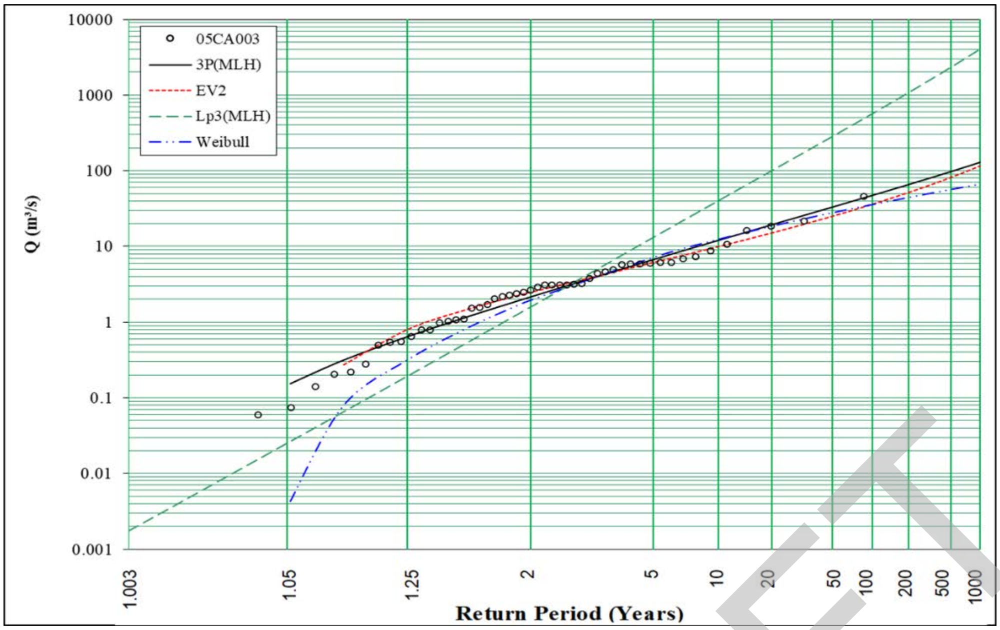
Figure B-5: WSC Station No. 07BE004, Stony Creek near Tawatinaw

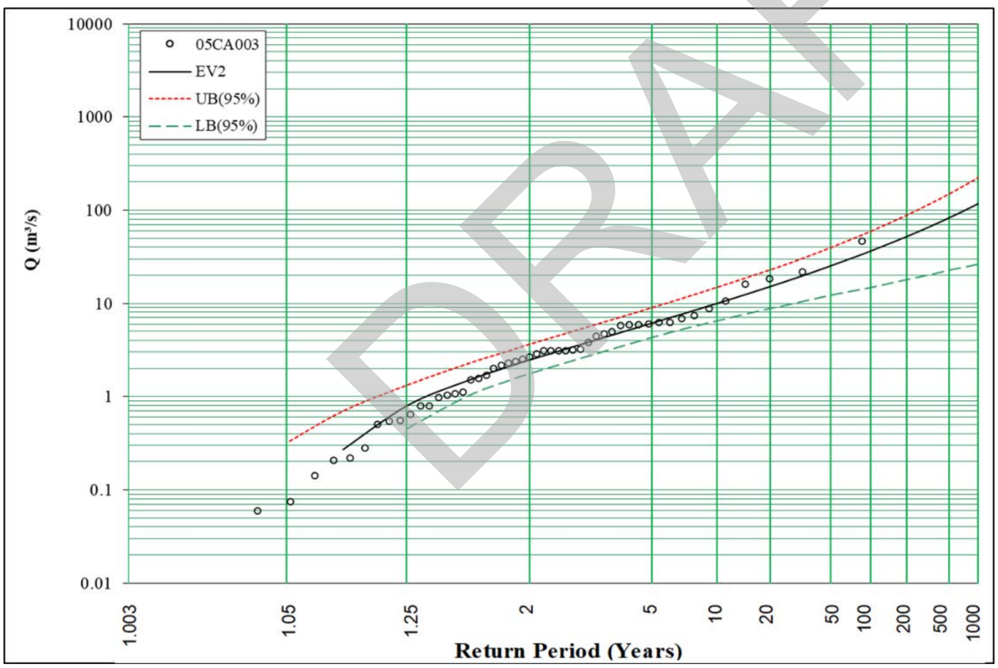




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	2.7	3.2	3.0	3.3
5	6.5	6.4	6.8	7.0
10	10.2	8.9	9.7	9.4
20	14.7	11.7	12.4	11.6
35	19.0	14.3	14.5	13.4
50	22.0	16.0	15.7	14.4
75	25.9	18.2	17.1	15.6
100	28.9	19.8	18.0	16.4
200	37.0	24.1	20.1	18.3
350	44.5	28.0	21.6	19.8
500	49.9	30.7	22.5	20.8
750	56.5	34.0	23.5	21.8
1000	61.5	36.6	24.2	22.6

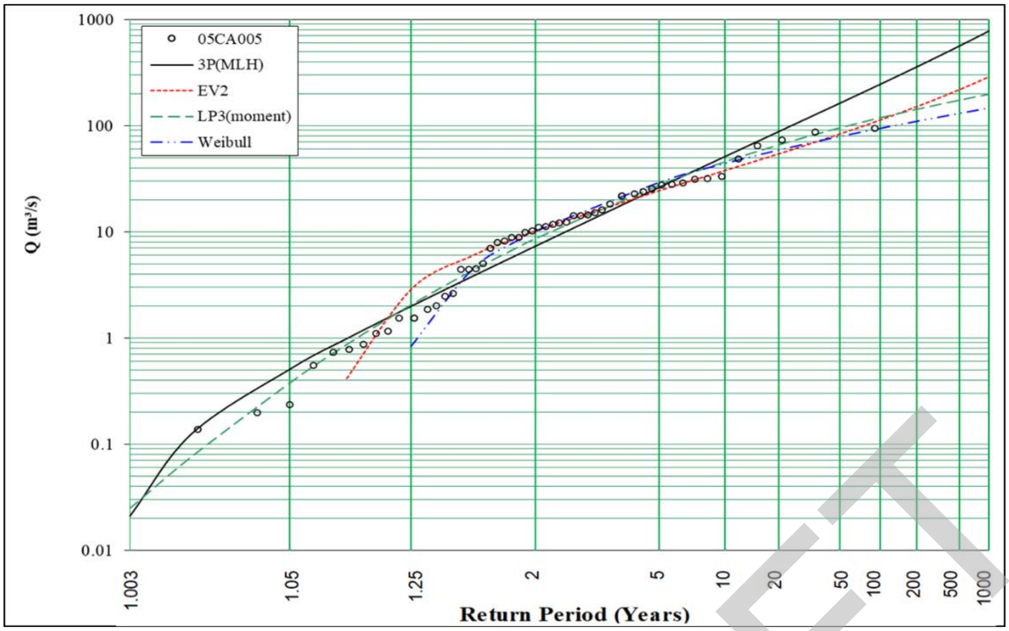
Figure B-6: WSC Station No. 07CA003, Flat Creek near Boyle

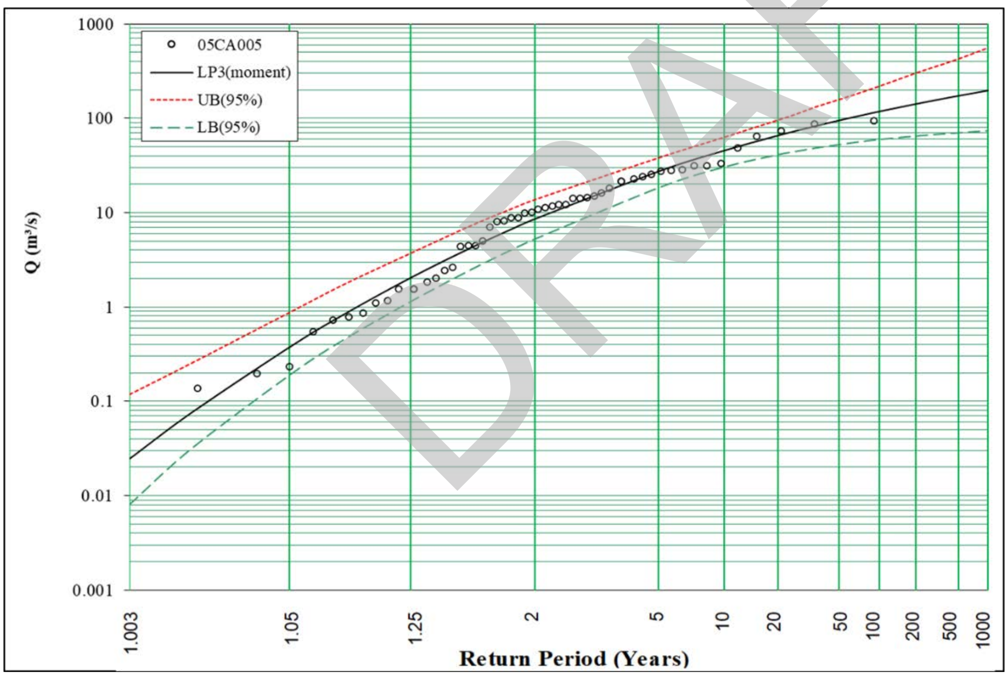




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	2.2	2.5	1.6	2.0
5	6.7	6.2	13.2	7.2
10	12.0	10.0	40.1	12.4
20	19.4	15.2	101	18.6
35	27.2	20.8	194	24.2
50	33.2	25.3	286	28.0
75	41.1	31.3	433	32.5
100	47.5	36.4	573	35.9
200	65.9	52.0	1085	44.5
350	84.4	69.0	1756	51.8
500	98.0	82.5	2354	56.7
750	116	101	3250	62.5
1000	129	116	4059	66.7

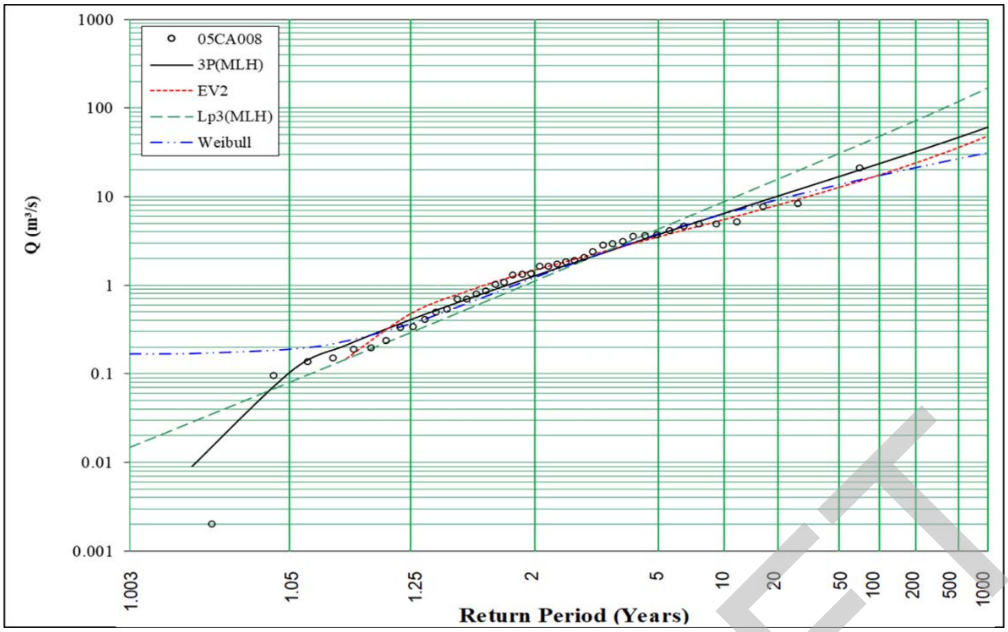
Figure B-7: WSC Station No. 07CA005, Pine Creek near Grassland

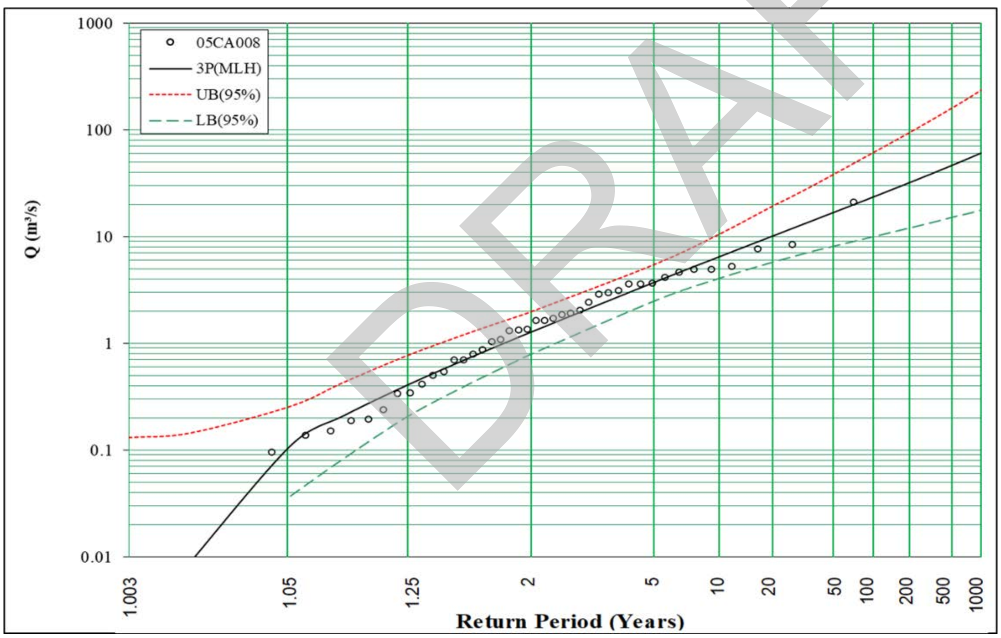




Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	7.3	10.2	8.6	10.1
5	26.2	24.5	27.3	29.2
10	50.9	37.8	45.3	44.1
20	88.0	54.5	65.6	59.2
35	130	71.3	83.2	71.5
50	163	84.0	94.8	79.5
75	208	100	108	88.5
100	246	114	118	95.0
200	357	152	142	111
350	474	191	161	123
500	563	220	173	131
750	681	259	186	141
1000	775	290	196	147

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





Return Period	3P(MLH)	EV2	LP3 (moment)	Weibull
2	1.3	1.5	1.1	1.2
5	3.7	3.5	4.3	3.8
10	6.5	5.5	8.8	6.4
20	10.1	8.1	15.8	9.3
35	14.0	10.7	24.0	11.9
50	16.8	12.7	30.7	13.7
75	20.6	15.4	40.1	15.8
100	23.6	17.6	48.0	17.3
200	32.1	24.0	72.3	21.2
350	40.5	30.7	98.4	24.6
500	46.7	35.9	119	26.8
750	54.5	42.7	146	29.4
1000	60.7	48.3	169	31.3

APPENDIX D

Open Water Flood Profiles

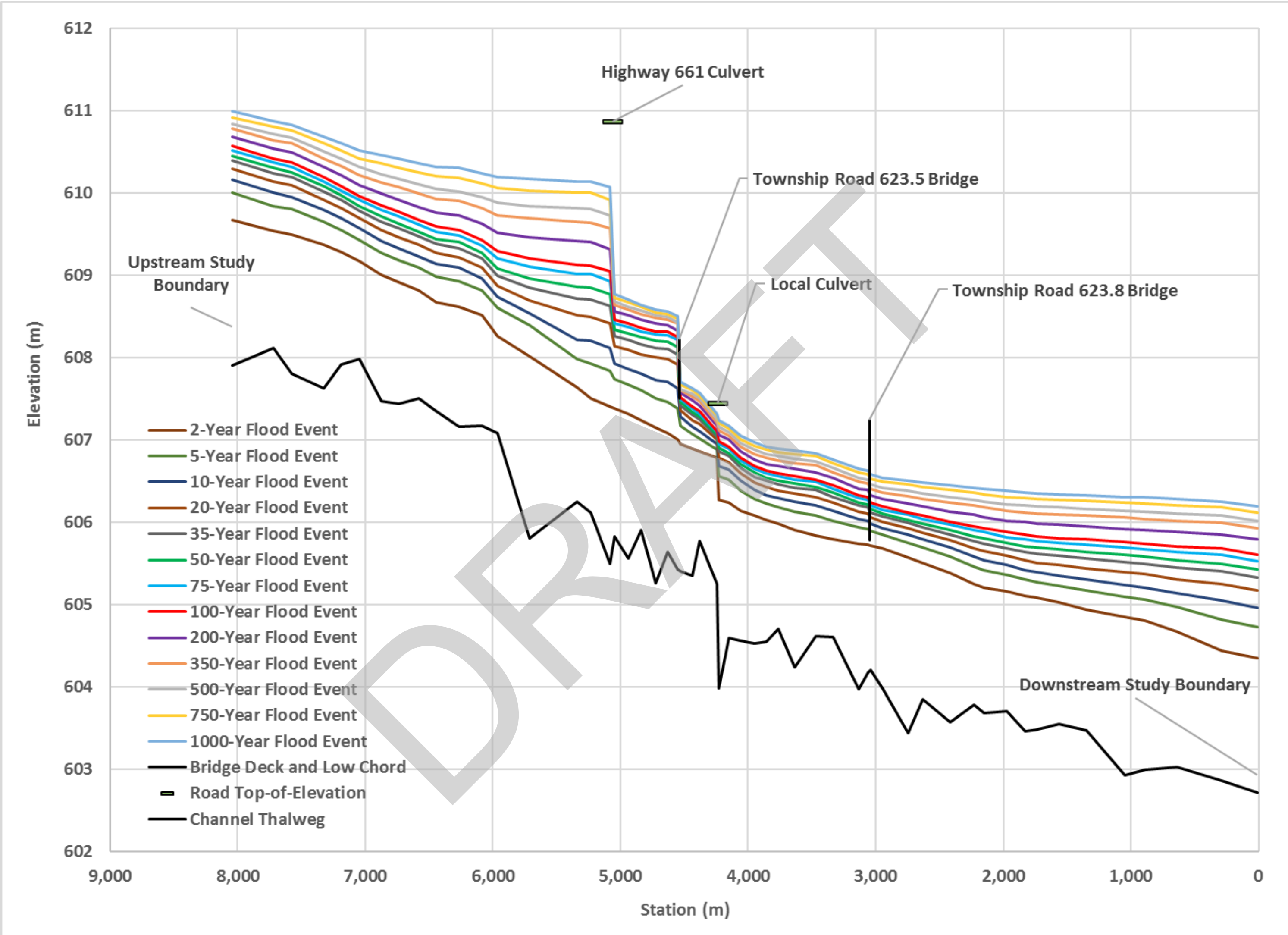


Figure D-1: Simulated Water Surface Profiles along the Tawatinaw River Study reach

Table D-1: Tawatinaw River Flood Profiles

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)												
				2-Year Flood Event	5-Year Flood Event	10-Year Flood Event	20-Year Flood Event	35-Year Flood Event	50-Year Flood Event	75-Year Flood Event	100-Year Flood Event	200-Year Flood Event	350-Year Flood Event	500-Year Flood Event	750-Year Flood Event	1000-Year Flood Event
Tawatinaw River	1	8045	607.9	609.7	610.0	610.2	610.3	610.4	610.5	610.5	610.6	610.7	610.8	610.8	610.9	611.0
Tawatinaw River	2	7720	608.1	609.5	609.8	610.0	610.1	610.2	610.3	610.4	610.4	610.5	610.6	610.7	610.8	610.9
Tawatinaw River	3	7577	607.8	609.5	609.8	610.0	610.1	610.2	610.3	610.3	610.4	610.5	610.6	610.7	610.8	610.8
Tawatinaw River	4	7329	607.6	609.4	609.7	609.8	609.9	610.0	610.1	610.1	610.2	610.3	610.4	610.5	610.6	610.7
Tawatinaw River	5	7192	607.9	609.3	609.6	609.7	609.8	609.9	610.0	610.0	610.1	610.2	610.3	610.4	610.5	610.6
Tawatinaw River	6	7046	608.0	609.2	609.4	609.6	609.7	609.8	609.8	609.9	610.0	610.1	610.2	610.3	610.4	610.5
Tawatinaw River	7	6871	607.5	609.0	609.3	609.4	609.6	609.7	609.7	609.8	609.9	610.0	610.1	610.2	610.4	610.5
Tawatinaw River	8	6739	607.4	608.9	609.2	609.3	609.5	609.6	609.6	609.7	609.8	609.9	610.1	610.2	610.3	610.4
Tawatinaw River	9	6581	607.5	608.8	609.1	609.2	609.4	609.5	609.5	609.6	609.7	609.8	610.0	610.1	610.3	610.4
Tawatinaw River	10	6442	607.4	608.7	609.0	609.1	609.3	609.4	609.4	609.5	609.6	609.8	609.9	610.1	610.2	610.3
Tawatinaw River	11	6264	607.2	608.6	608.9	609.1	609.2	609.3	609.4	609.5	609.6	609.7	609.9	610.0	610.2	610.3
Tawatinaw River	12	6086	607.2	608.5	608.8	609.0	609.1	609.2	609.3	609.4	609.4	609.6	609.8	610.0	610.1	610.2
Tawatinaw River	13	5961	607.1	608.3	608.6	608.7	608.9	609.0	609.1	609.2	609.3	609.5	609.7	609.9	610.1	610.2
Tawatinaw River	14	5714	605.8	608.0	608.4	608.5	608.7	608.9	609.0	609.1	609.2	609.5	609.7	609.8	610.0	610.2
Tawatinaw River	15	5341	606.3	607.6	608.0	608.2	608.5	608.7	608.9	609.0	609.1	609.4	609.7	609.8	610.0	610.1
Tawatinaw River	16	5232	606.1	607.5	607.9	608.2	608.5	608.7	608.9	609.0	609.1	609.4	609.6	609.8	610.0	610.1
Tawatinaw River	17	5084	605.5	607.4	607.8	608.1	608.4	608.6	608.8	608.9	609.1	609.3	609.6	609.7	609.9	610.1
Tawatinaw River	18	5045	605.8	607.4	607.7	607.9	608.1	608.3	608.3	608.4	608.5	608.6	608.6	608.7	608.7	608.8
Tawatinaw River	19	4940	605.6	607.3	607.7	607.9	608.1	608.2	608.3	608.4	608.4	608.5	608.6	608.6	608.7	608.7
Tawatinaw River	20	4837	605.9	607.2	607.6	607.8	608.0	608.2	608.3	608.3	608.4	608.5	608.5	608.6	608.6	608.6
Tawatinaw River	21	4725	605.3	607.2	607.5	607.7	608.0	608.1	608.2	608.3	608.3	608.4	608.5	608.5	608.6	608.6
Tawatinaw River	22	4629	605.6	607.1	607.5	607.7	608.0	608.1	608.2	608.3	608.3	608.4	608.5	608.5	608.5	608.6
Tawatinaw River	23	4554	605.4	607.0	607.4	607.6	607.9	608.0	608.1	608.2	608.3	608.3	608.4	608.4	608.5	608.5
Tawatinaw River	24	4528	605.4	607.0	607.2	607.3	607.4	607.4	607.5	607.5	607.5	607.6	607.6	607.6	607.7	607.7
Tawatinaw River	25	4438	605.4	606.9	607.1	607.2	607.2	607.3	607.3	607.4	607.4	607.5	607.5	607.6	607.6	607.6
Tawatinaw River	26	4380	605.8	606.9	607.0	607.1	607.2	607.3	607.3	607.3	607.4	607.4	607.5	607.5	607.5	607.6
Tawatinaw River	27	4245	605.3	606.8	606.9	607.0	607.0	607.0	607.1	607.1	607.1	607.2	607.2	607.3	607.3	607.3
Tawatinaw River	28	4230	604.0	606.3	606.6	606.7	606.8	606.9	606.9	607.0	607.0	607.1	607.1	607.2	607.2	607.2
Tawatinaw River	29	4153	604.6	606.2	606.5	606.6	606.7	606.8	606.8	606.9	606.9	607.0	607.1	607.1	607.1	607.2
Tawatinaw River	30	4054	604.6	606.1	606.4	606.5	606.6	606.7	606.7	606.8	606.8	606.9	606.9	607.0	607.0	607.1
Tawatinaw River	31	3950	604.5	606.1	606.3	606.4	606.5	606.6	606.6	606.7	606.7	606.8	606.8	606.9	606.9	607.0
Tawatinaw River	32	3855	604.6	606.0	606.2	606.3	606.4	606.5	606.5	606.6	606.6	606.7	606.8	606.8	606.9	606.9

Table D-1: Tawatinaw River Flood Profiles

River	Cross Section	River Station	Channel Thalweg (m)	Simulated Water Level (m)												
				2-Year Flood Event	5-Year Flood Event	10-Year Flood Event	20-Year Flood Event	35-Year Flood Event	50-Year Flood Event	75-Year Flood Event	100-Year Flood Event	200-Year Flood Event	350-Year Flood Event	500-Year Flood Event	750-Year Flood Event	1000-Year Flood Event
Tawatinaw River	33	3763	604.7	606.0	606.2	606.3	606.4	606.5	606.5	606.6	606.6	606.7	606.8	606.8	606.9	606.9
Tawatinaw River	34	3637	604.2	605.9	606.1	606.3	606.4	606.4	606.5	606.5	606.6	606.7	606.7	606.8	606.8	606.9
Tawatinaw River	35	3466	604.6	605.8	606.1	606.2	606.3	606.4	606.4	606.5	606.5	606.6	606.7	606.7	606.8	606.8
Tawatinaw River	36	3332	604.6	605.8	606.0	606.1	606.2	606.3	606.4	606.4	606.5	606.5	606.6	606.7	606.7	606.8
Tawatinaw River	37	3132	604.0	605.7	605.9	606.0	606.1	606.2	606.2	606.3	606.3	606.4	606.5	606.5	606.6	606.7
Tawatinaw River	38	3060	604.2	605.7	605.9	606.0	606.1	606.2	606.2	606.3	606.3	606.4	606.5	606.5	606.6	606.6
Tawatinaw River	39	3039	604.2	605.7	605.9	606.0	606.1	606.1	606.2	606.2	606.2	606.3	606.4	606.5	606.5	606.6
Tawatinaw River	40	2945	604.0	605.7	605.9	605.9	606.0	606.1	606.1	606.2	606.2	606.3	606.4	606.4	606.5	606.5
Tawatinaw River	41	2746	603.4	605.6	605.8	605.9	605.9	606.0	606.0	606.1	606.1	606.2	606.3	606.4	606.5	606.5
Tawatinaw River	42	2633	603.9	605.5	605.7	605.8	605.9	605.9	606.0	606.0	606.1	606.2	606.3	606.4	606.4	606.5
Tawatinaw River	43	2415	603.6	605.4	605.6	605.7	605.8	605.9	605.9	606.0	606.0	606.1	606.2	606.3	606.4	606.5
Tawatinaw River	44	2228	603.8	605.3	605.5	605.6	605.7	605.8	605.8	605.9	606.0	606.1	606.2	606.3	606.4	606.4
Tawatinaw River	45	2150	603.7	605.2	605.4	605.5	605.7	605.7	605.8	605.9	605.9	606.1	606.2	606.3	606.3	606.4
Tawatinaw River	46	1972	603.7	605.2	605.4	605.5	605.6	605.7	605.8	605.8	605.9	606.0	606.1	606.2	606.3	606.4
Tawatinaw River	47	1825	603.5	605.1	605.3	605.4	605.5	605.6	605.7	605.8	605.9	606.0	606.1	606.2	606.3	606.4
Tawatinaw River	48	1731	603.5	605.1	605.3	605.4	605.5	605.6	605.7	605.8	605.8	606.0	606.1	606.2	606.3	606.4
Tawatinaw River	49	1565	603.6	605.0	605.2	605.4	605.5	605.6	605.7	605.8	605.8	606.0	606.1	606.2	606.3	606.3
Tawatinaw River	50	1351	603.5	604.9	605.2	605.3	605.4	605.6	605.6	605.7	605.8	606.0	606.1	606.2	606.3	606.3
Tawatinaw River	51	1050	602.9	604.9	605.1	605.2	605.4	605.5	605.6	605.7	605.8	605.9	606.1	606.1	606.2	606.3
Tawatinaw River	52	887	603.0	604.8	605.1	605.2	605.4	605.5	605.6	605.7	605.7	605.9	606.0	606.1	606.2	606.3
Tawatinaw River	53	639	603.0	604.7	605.0	605.1	605.3	605.5	605.5	605.6	605.7	605.9	606.0	606.1	606.2	606.3
Tawatinaw River	54	290	602.9	604.4	604.8	605.1	605.3	605.4	605.5	605.6	605.7	605.9	606.0	606.1	606.2	606.3
Tawatinaw River	55	9	602.7	604.4	604.7	605.0	605.2	605.3	605.4	605.5	605.6	605.8	605.9	606.0	606.1	606.2

APPENDIX E

Open Water Sensitivity Analysis

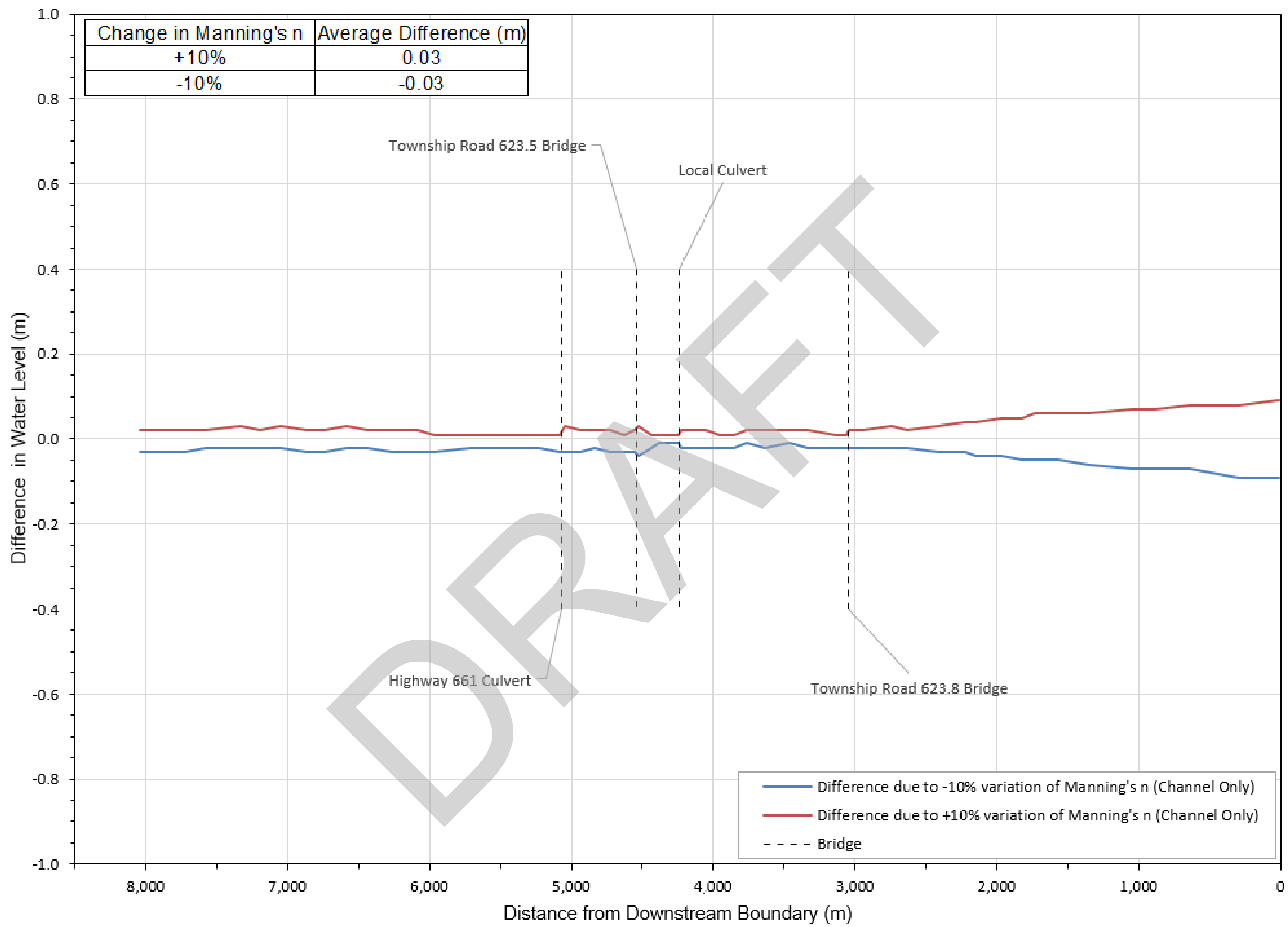


Figure E-1: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Channel Manning's *n* Only)

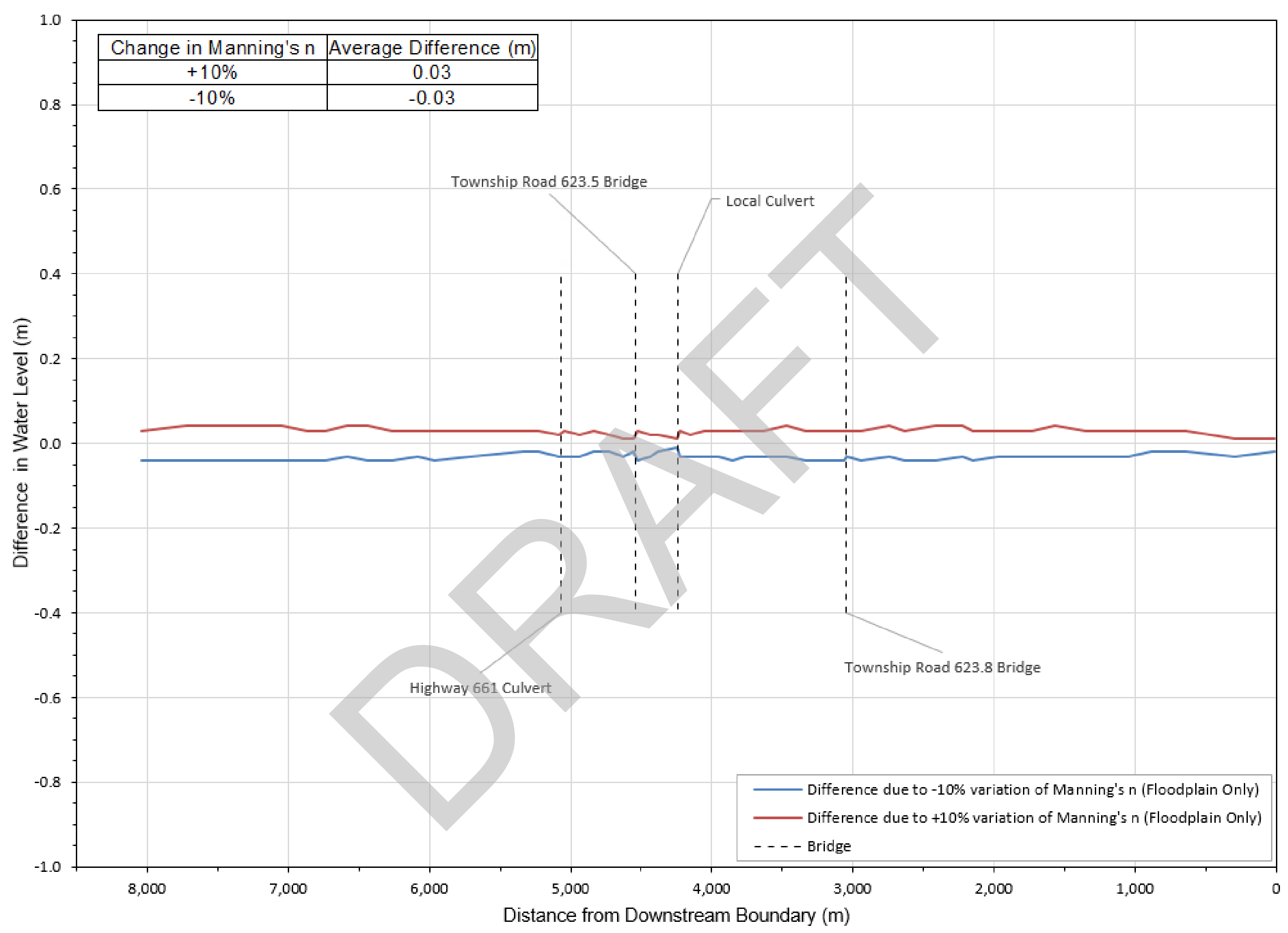


Figure E-2: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Floodplain Manning's *n* Only)

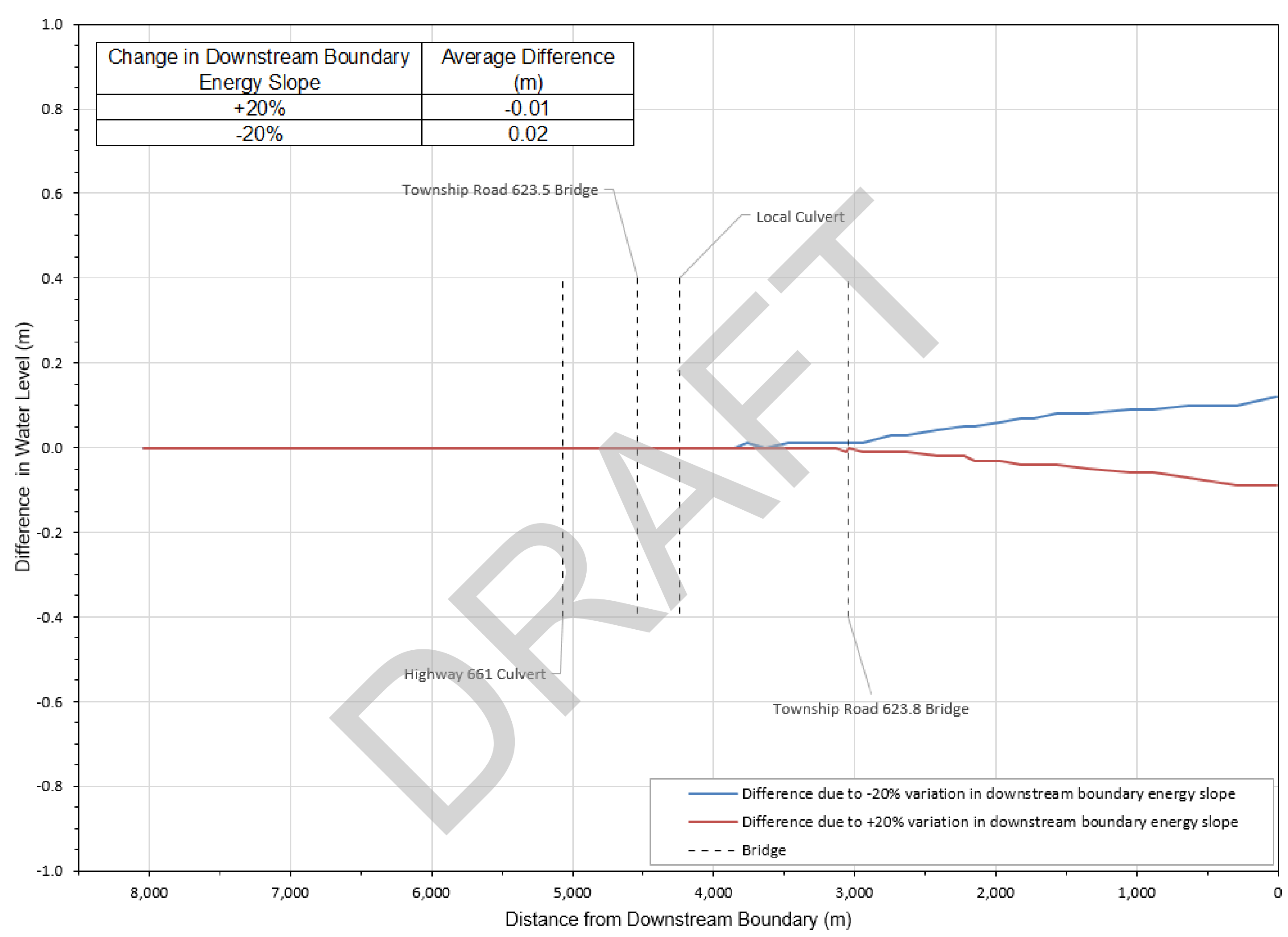


Figure E-3: Sensitivity of Simulated Water Level along the Tawatinaw River Study Reach for the 100-Year Flood Event (Downstream Boundary)

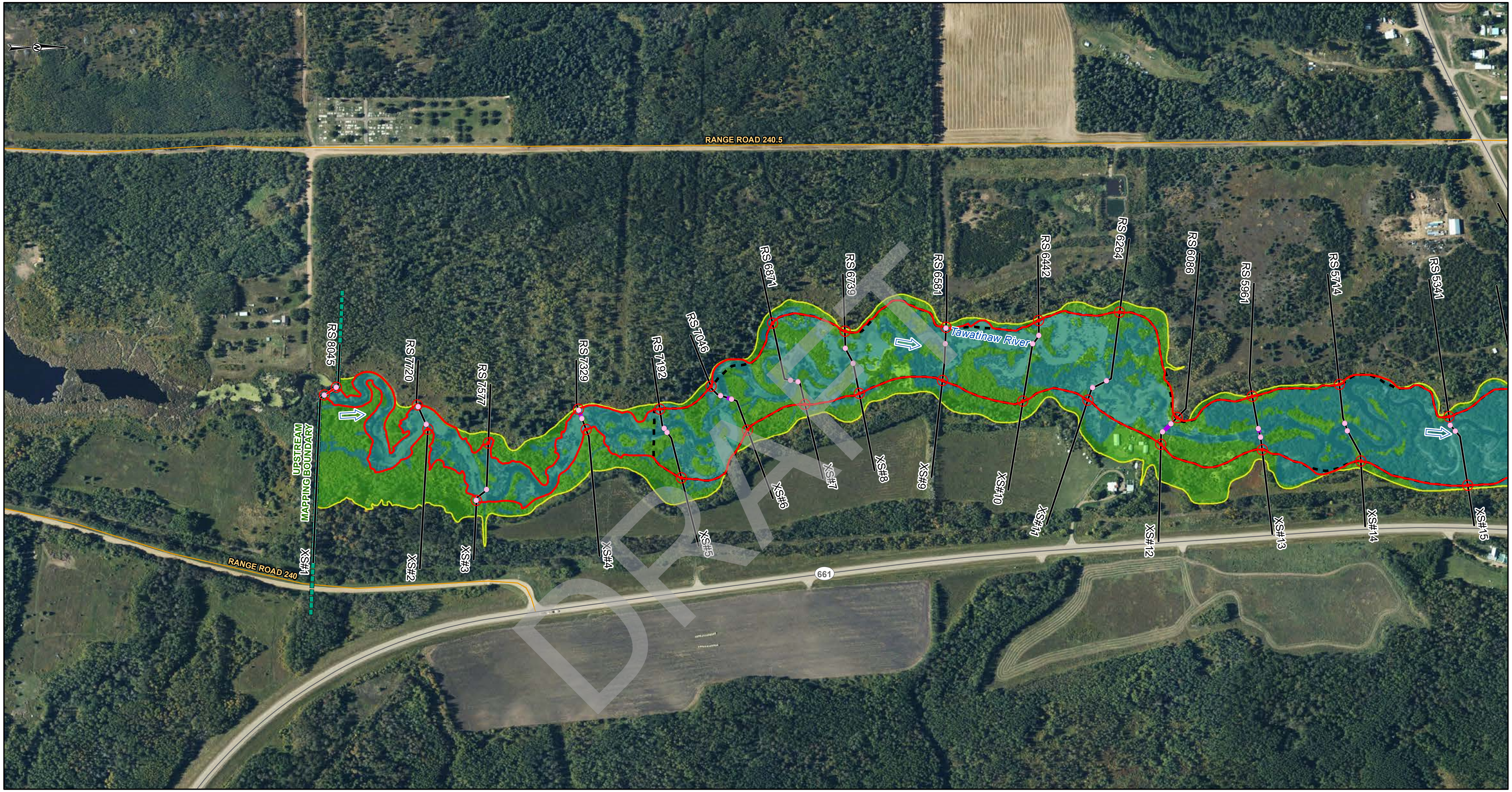
APPENDIX F

Open Water Inundation Maps

(PROVIDED SEPARATELY IN THE MAP LIBRARY)

APPENDIX G

Floodway Criteria Maps and Flood Hazard Maps



LEGEND

	CROSS SECTION
XS#52	CROSS SECTION NUMBER
RS 887	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

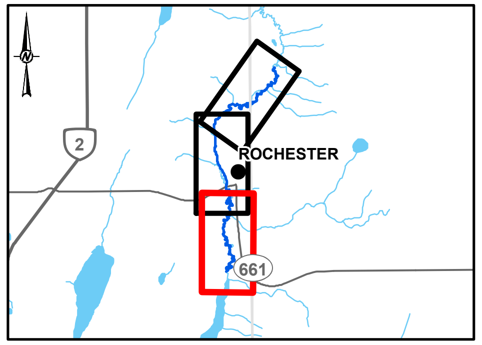
HYDRAULIC STRUCTURES

- BRIDGE
- CULVERT

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DISCHARGE

TAWATINAW RIVER = 43.8 M^3/S



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REFERENCE(S)

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CLIENT
ALBERTA ENVIRONMENT AND PARKS

Alberta

PROJECT
ROCHESTER FLOOD STUDY

CONSULTANT

GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	SP
REVIEWED	JC
APPROVED	HZ

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
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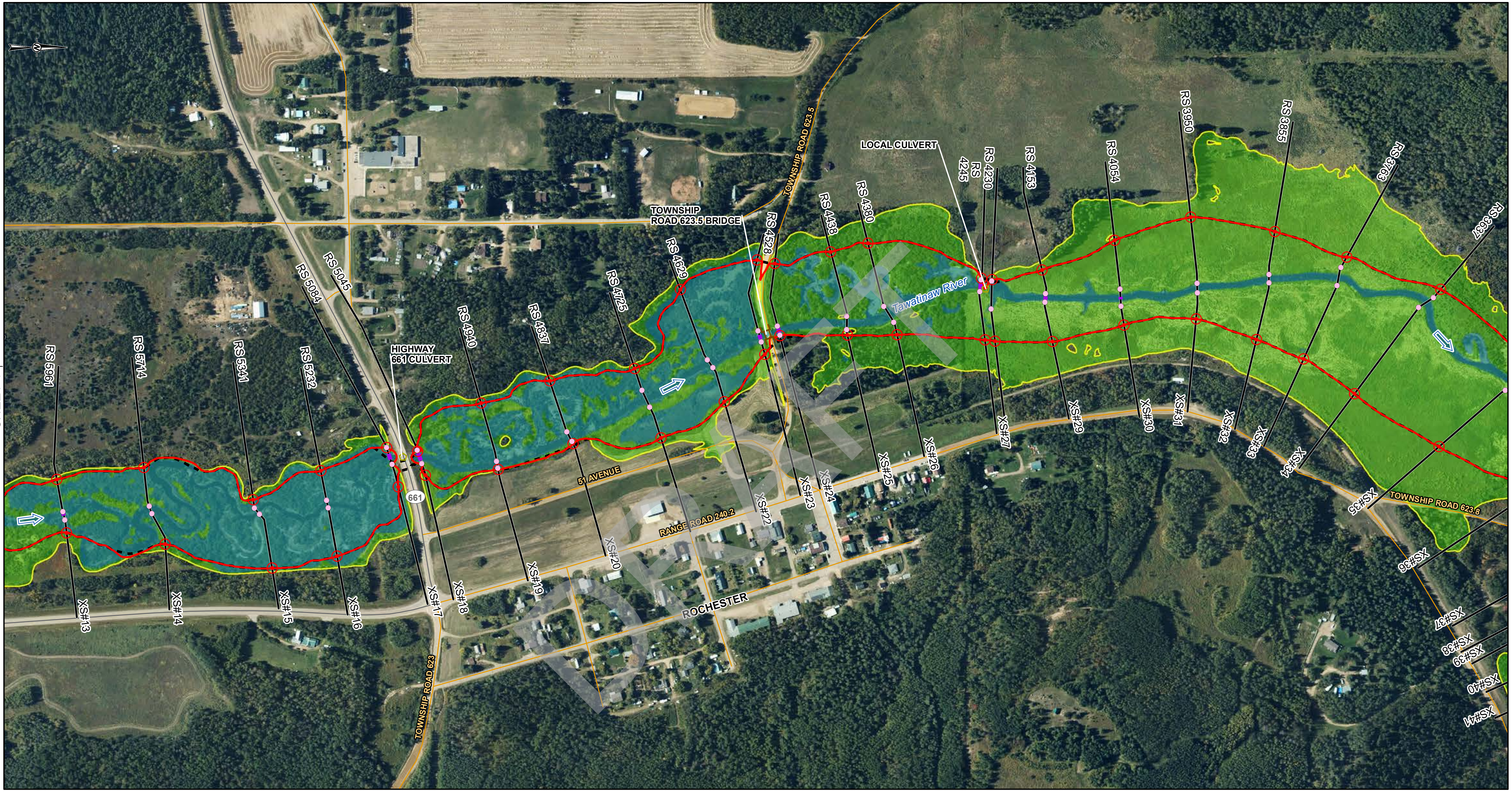


LEGEND

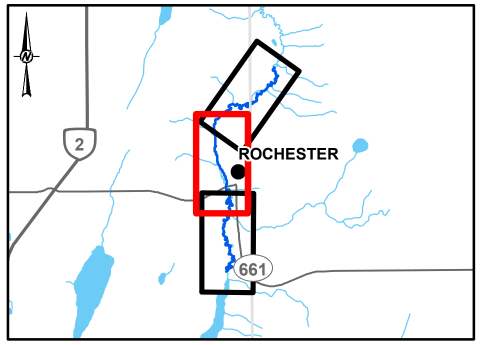
- CROSS SECTION
- XS#52 CROSS SECTION NUMBER
- RS 887 RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

HYDRAULIC STRUCTURES

- BRIDGE
- CULVERT

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DISCHARGE
TAWATINAW RIVER = 43.8 M³/S



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CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	SP
REVIEWED	JC
APPROVED	HZ

PROJECT
ROCHESTER FLOOD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
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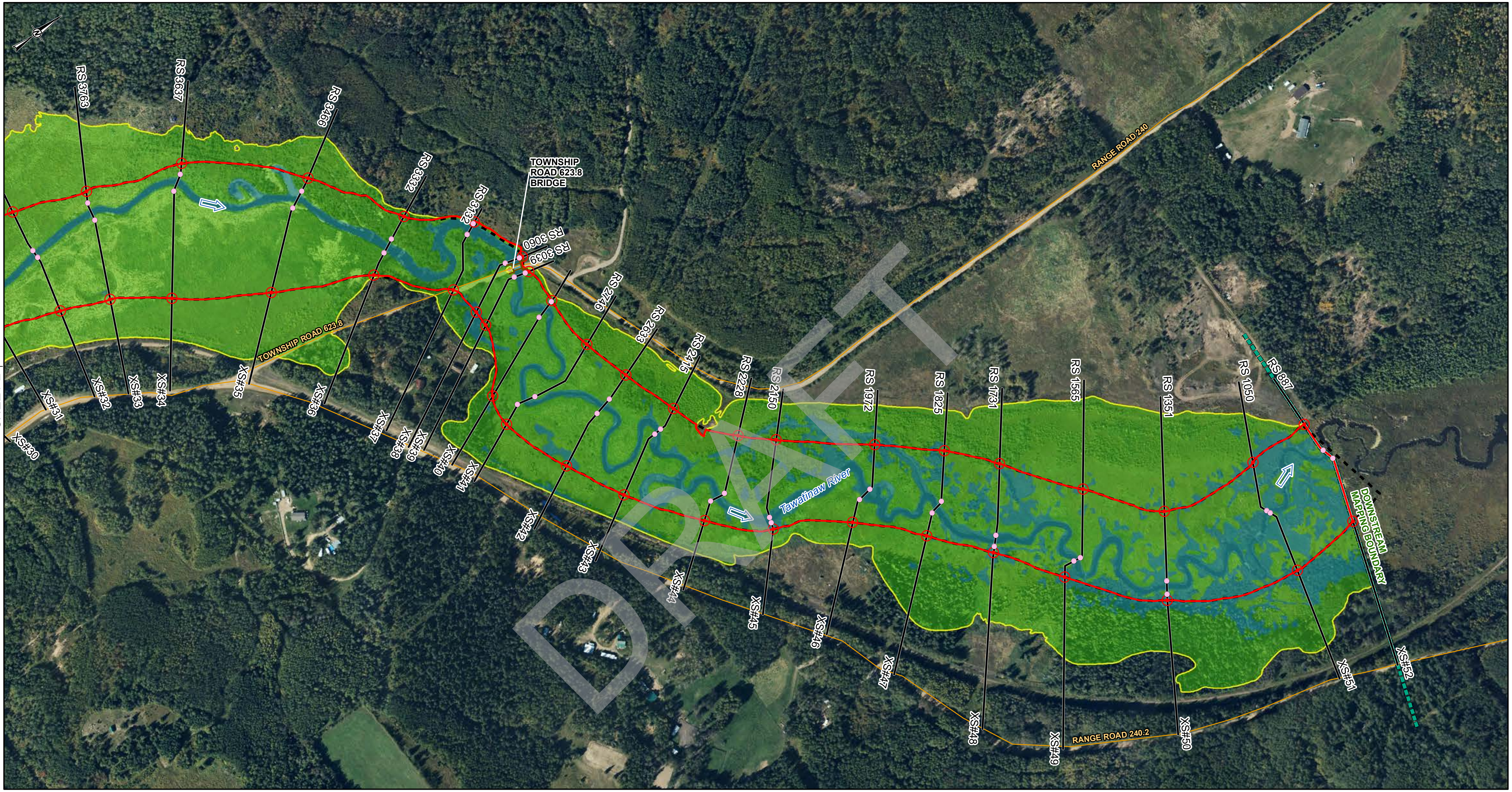


LEGEND

	CROSS SECTION
XS#52	CROSS SECTION NUMBER
RS 887	RIVER STATION (M)
	MAPPING BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY

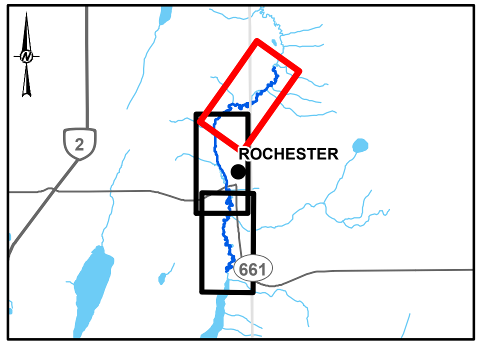
HYDRAULIC STRUCTURES

- BRIDGE
- CULVERT

- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DISCHARGE

TAWATINAW RIVER = 43.8 M^3/S



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GOLDER MEMBER OF WSP

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DESIGNED	AG
PREPARED	SP
REVIEWED	JC
APPROVED	HZ

PROJECT
ROCHESTER FLOOD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
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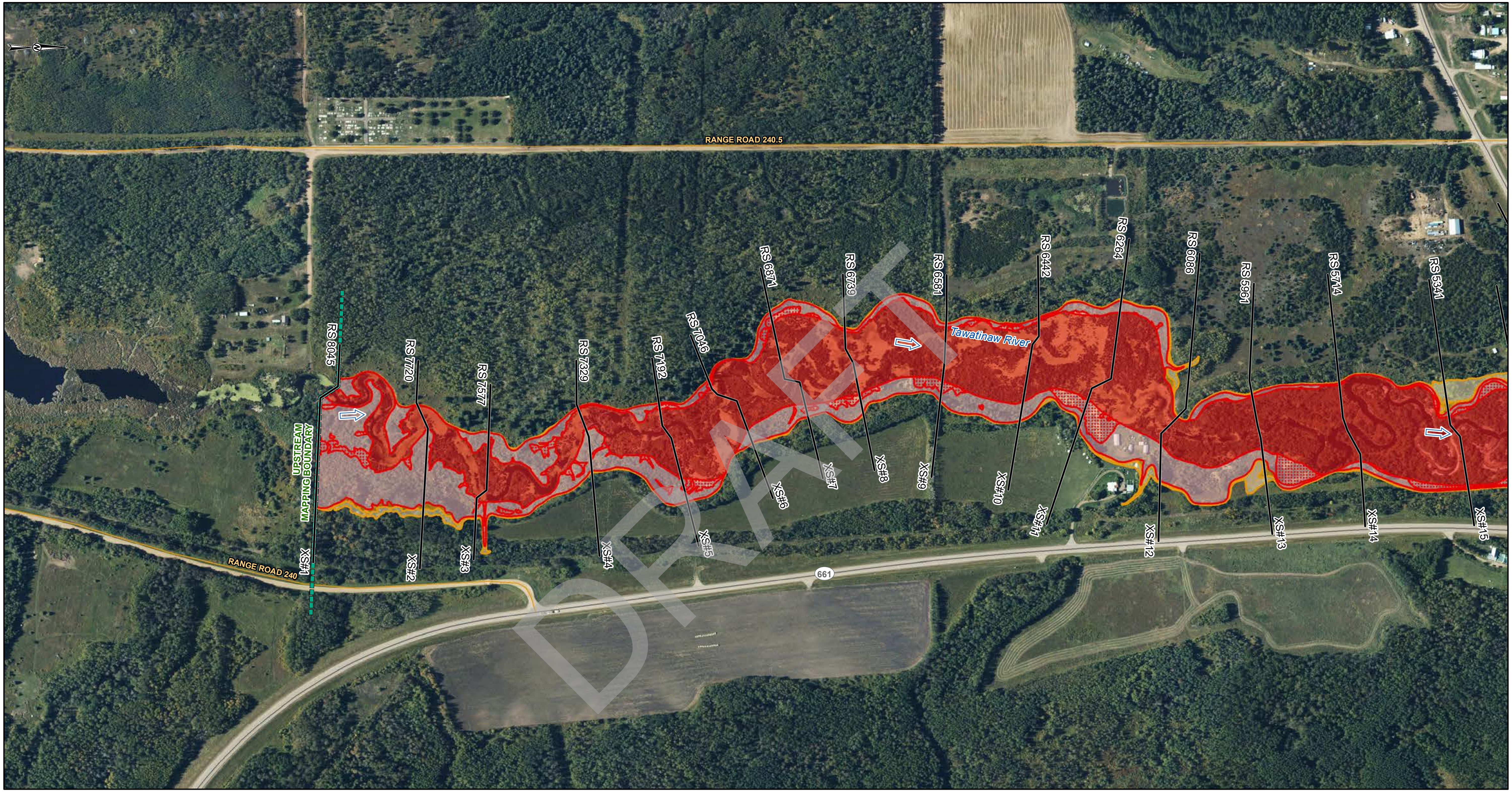
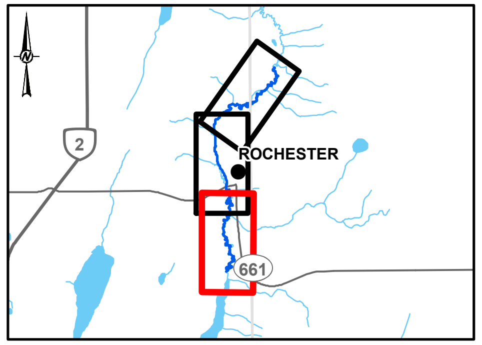


LEGEND

CROSS SECTION	**HYDRAULIC STRUCTURES**	FLOODWAY	
XS#52 CROSS SECTION NUMBER	BRIDGE	HIGH HAZARD FLOOD FRINGE	
RS 887 RIVER STATION (M)	CULVERT	FLOOD FRINGE	
MAPPING BOUNDARY		200-YEAR FLOOD EXTENT	
FLOW DIRECTION		500-YEAR FLOOD EXTENT	
LOCAL ROAD			
PRIMARY HIGHWAY		DISCHARGE	
SECONDARY HIGHWAY		TAWATINAW RIVER = 43.8 M³/S	



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1:5,000 METRES

REFERENCE(S)
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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

Alberta

CONSULTANT
GOLDER MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	SK
REVIEWED	JC
APPROVED	HZ

PROJECT
ROCHESTER FLOOD STUDY

TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
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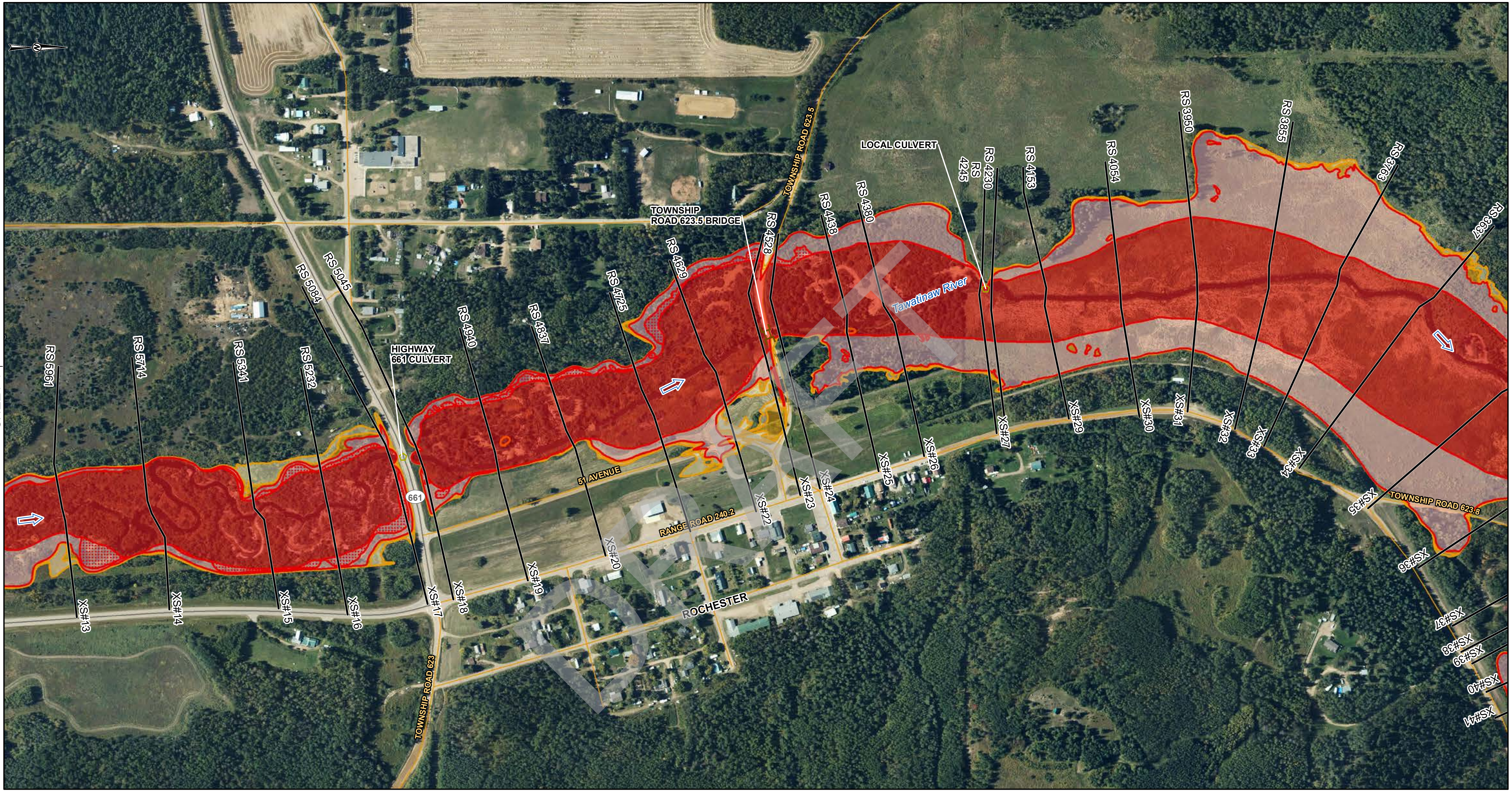
Classification: Public



LEGEND

- CROSS SECTION
- XS#52 CROSS SECTION NUMBER
- RS 887 RIVER STATION (M)
- MAPPING BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY

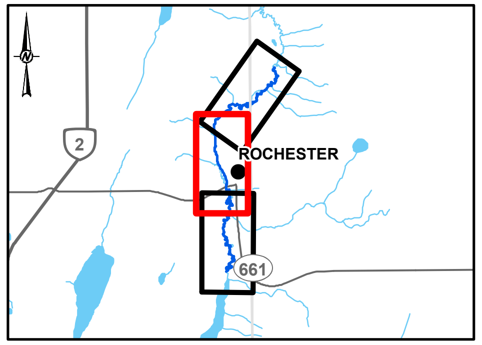
HYDRAULIC STRUCTURES

- BRIDGE
- CULVERT

- FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

DISCHARGE

TAWATINAW RIVER = 43.8 M^3/S



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1:5,000 METRES

REFERENCE(S)

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CLIENT

ALBERTA ENVIRONMENT AND PARKS

Alberta

CONSULTANT

GOLDER

MEMBER OF WSP

YYYY-MM-DD	2022-03-21
DESIGNED	AG
PREPARED	SK
REVIEWED	JC
APPROVED	HZ

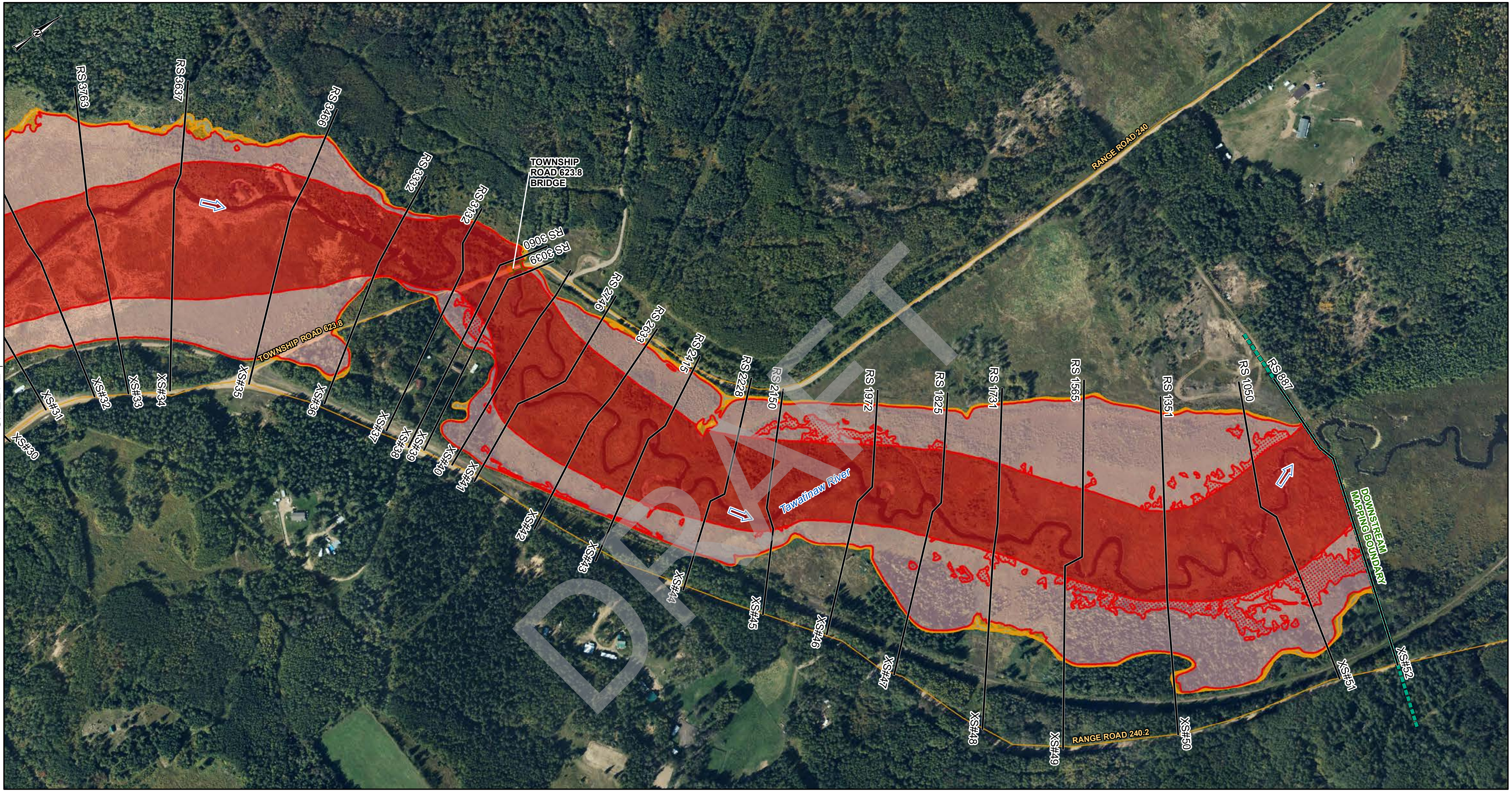
PROJECT

ROCHESTER FLOOD STUDY

TITLE

FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
20368084	5000	0	2 of 3



LEGEND

CROSS SECTION	
XS#52	CROSS SECTION NUMBER
RS 887	RIVER STATION (M)
MAPPING BOUNDARY	
FLOW DIRECTION	
LOCAL ROAD	
PRIMARY HIGHWAY	
SECONDARY HIGHWAY	

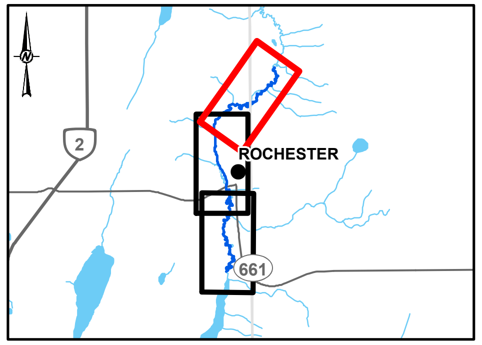
HYDRAULIC STRUCTURES

- BRIDGE
- CULVERT

- FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

DISCHARGE

TAWATINAW RIVER = 43.8 M^3/S



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TITLE

FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
20368084	5000	0	3 of 3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25mm

APPENDIX H

Climate Change Flood Profiles

Table H-1: Water Levels 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	1	8045	610.57	610.63	610.70	0.06	0.13
Tawatinaw River	2	7720	610.42	610.49	610.56	0.07	0.14
Tawatinaw River	3	7577	610.37	610.44	610.51	0.07	0.14
Tawatinaw River	4	7329	610.19	610.26	610.34	0.07	0.15
Tawatinaw River	5	7192	610.08	610.16	610.23	0.08	0.15
Tawatinaw River	6	7046	609.96	610.04	610.12	0.08	0.16
Tawatinaw River	7	6871	609.85	609.93	610.02	0.08	0.17
Tawatinaw River	8	6739	609.77	609.85	609.95	0.08	0.18
Tawatinaw River	9	6581	609.67	609.76	609.86	0.09	0.19
Tawatinaw River	10	6442	609.59	609.69	609.79	0.10	0.20
Tawatinaw River	11	6264	609.55	609.65	609.76	0.10	0.21
Tawatinaw River	12	6086	609.43	609.54	609.66	0.11	0.23
Tawatinaw River	13	5961	609.29	609.41	609.56	0.12	0.27
Tawatinaw River	14	5714	609.20	609.35	609.50	0.15	0.30
Tawatinaw River	15	5341	609.13	609.29	609.45	0.16	0.32
Tawatinaw River	16	5232	609.12	609.28	609.44	0.16	0.32
Tawatinaw River	17	5084	609.05	609.20	609.37	0.15	0.32
Tawatinaw River	18	5045	608.46	608.52	608.58	0.06	0.12
Tawatinaw River	19	4940	608.41	608.47	608.52	0.06	0.11
Tawatinaw River	20	4837	608.36	608.42	608.47	0.06	0.11
Tawatinaw River	21	4725	608.32	608.37	608.42	0.05	0.10
Tawatinaw River	22	4629	608.31	608.36	608.41	0.05	0.10

Table H-1: Water Levels 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	23	4554	608.25	608.30	608.35	0.05	0.10
Tawatinaw River	24	4528	607.52	607.55	607.57	0.03	0.05
Tawatinaw River	25	4438	607.41	607.45	607.49	0.04	0.08
Tawatinaw River	26	4380	607.35	607.39	607.42	0.04	0.07
Tawatinaw River	27	4245	607.11	607.14	607.18	0.03	0.07
Tawatinaw River	28	4230	606.98	607.03	607.07	0.05	0.09
Tawatinaw River	29	4153	606.92	606.96	607.01	0.04	0.09
Tawatinaw River	30	4054	606.78	606.83	606.87	0.05	0.09
Tawatinaw River	31	3950	606.68	606.73	606.78	0.05	0.10
Tawatinaw River	32	3855	606.63	606.68	606.72	0.05	0.09
Tawatinaw River	33	3763	606.59	606.64	606.69	0.05	0.10
Tawatinaw River	34	3637	606.56	606.61	606.66	0.05	0.10
Tawatinaw River	35	3466	606.52	606.57	606.63	0.05	0.11
Tawatinaw River	36	3332	606.45	606.50	606.55	0.05	0.10
Tawatinaw River	37	3132	606.33	606.38	606.43	0.05	0.10
Tawatinaw River	38	3060	606.31	606.35	606.40	0.04	0.09
Tawatinaw River	39	3039	606.24	606.29	606.34	0.05	0.10
Tawatinaw River	40	2945	606.19	606.24	606.30	0.05	0.11
Tawatinaw River	41	2746	606.12	606.18	606.24	0.06	0.12
Tawatinaw River	42	2633	606.08	606.14	606.21	0.06	0.13
Tawatinaw River	43	2415	606.01	606.08	606.15	0.07	0.14
Tawatinaw River	44	2228	605.95	606.03	606.11	0.08	0.16

Table H-1: Water Levels 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	45	2150	605.93	606.01	606.09	0.08	0.16
Tawatinaw River	46	1972	605.88	605.96	606.05	0.08	0.17
Tawatinaw River	47	1825	605.85	605.94	606.02	0.09	0.17
Tawatinaw River	48	1731	605.83	605.92	606.01	0.09	0.18
Tawatinaw River	49	1565	605.81	605.90	605.99	0.09	0.18
Tawatinaw River	50	1351	605.79	605.88	605.97	0.09	0.18
Tawatinaw River	51	1050	605.76	605.86	605.95	0.10	0.19
Tawatinaw River	52	887	605.74	605.84	605.93	0.10	0.19
Tawatinaw River	53	639	605.71	605.81	605.91	0.10	0.20
Tawatinaw River	54	290	605.68	605.78	605.88	0.10	0.20
Tawatinaw River	55	9	605.61	605.71	605.81	0.10	0.20

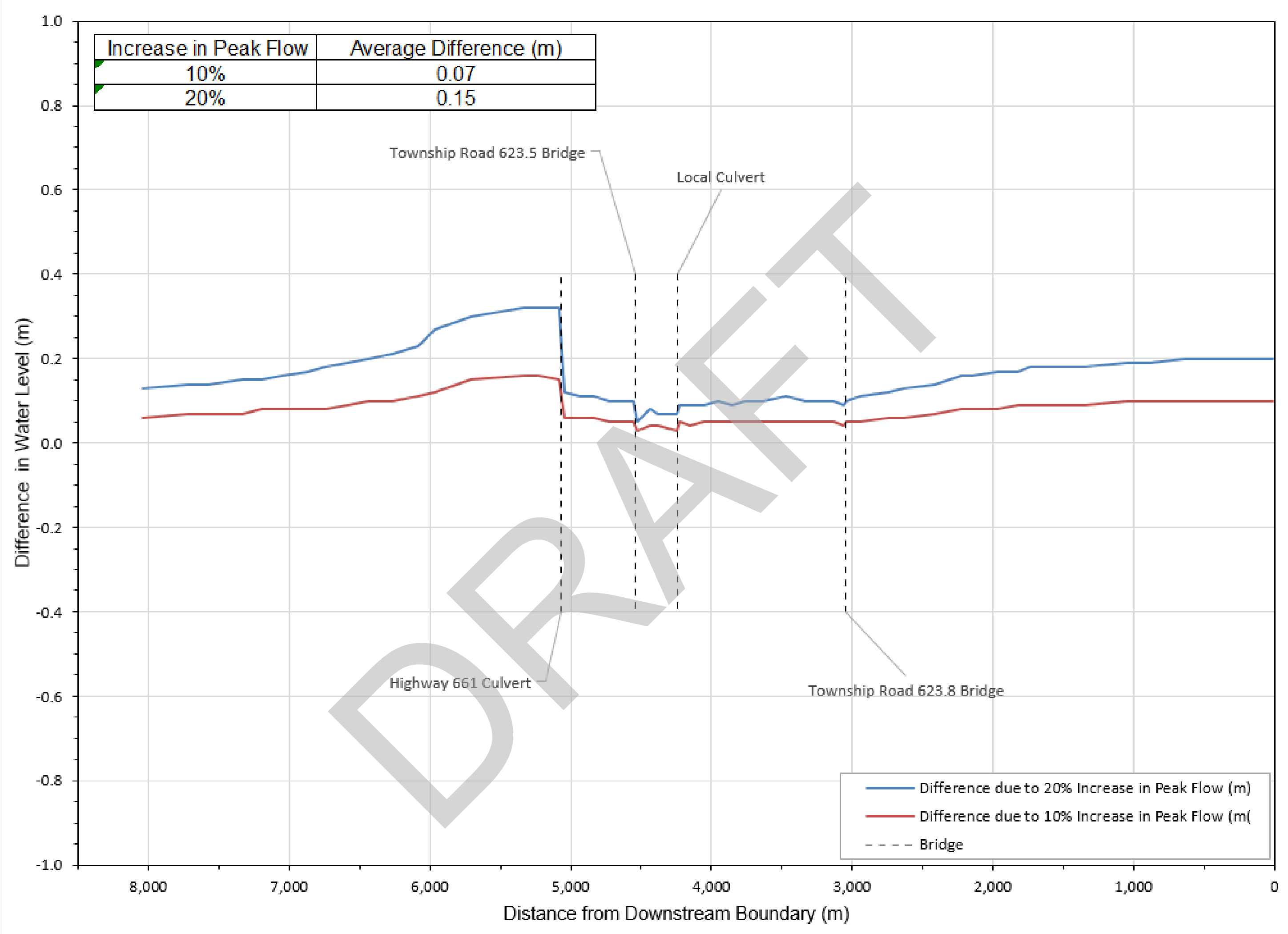

Figure H-1: Water Level Difference along the Tawatinaw River due to Climate Change

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