Watino Flood Study

Main Report

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EXECUTIVE SUMMARY

In March 2023, Alberta Environment and Protected Areas (EPA) enlisted the services of Northwest Hydraulic Consultants Ltd. (NHC) to complete a flood study for areas along approximately 15 km of the Smoky River through the Municipal District of Smoky River and Birch Hills County, including the Hamlet of Watino. This initiative was conducted under the Flood Hazard Identification Program (FHIP) with the goals of enhancing public safety and mitigating potential flood damages in the province.

The Watino Flood Study is comprised of five major project components: 1) survey and base data collection; 2) open water hydrology assessment; 3) open water hydraulic modelling; 4) open water flood inundation mapping; and 5) design flood hazard mapping. This report summarizes the work of all five components, and together they include the following details:

- descriptions of survey procedures and methodology
- documentation of the collected survey and base data
- summary of flood history
- open water flood frequency flow estimations
- construction of the hydraulic model
- calibration and validation of the hydraulic model
- computation of flood frequency water levels
- a model sensitivity analysis
- associated inundation mapping
- computation of design flood profiles
- floodway criteria and hazard mapping

The majority of the survey program was completed in May and June 2023, with additional follow-up work completed in August 2023. The objective of the survey program was to survey river cross sections and hydraulic structures along the Smoky River study reach to support the development of a one-dimensional (1D) hydraulic model. In addition to this, data such as Digital Terrain Model (DTM), aerial imagery, and other mapping features were gathered for model development and flood mapping efforts. The model calibration and validation data, including highwater marks (HWMs), were collected for the floods in 2020, 1990, 1982, and 1972. Furthermore, the rating curve and direct discharge measurements were obtained for the Water Survey of Canada (WSC) Station 07GJ001 (Smoky River at Watino).

Open water flood frequency estimates were completed for the Smoky River at Watino based on a single station frequency analysis. Flood magnitudes for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year events were estimated using a Bulletin 17C distribution, which best fits the data. The estimated peak discharges from this study are smaller than those reported by AEP (1994), except for the 2-year peak discharges. The AEP 1994 estimates had been derived using a flood discharge data series up to 1993, which is 30 years shorter than the period used in this study. The shorter data series from the 1994 study included the same three largest flood events as the data series used in this study (1990, 1972, and 1982 events). Consequently, these events were previously assigned a lower return period than in the current study. As a result, the flood peak estimates from the 1994 flood frequency analysis are too high and are not representative of the current data series. This study's flood frequency estimates are slightly higher than the estimates produced by NHC in 2016 for return periods up to 50 years, while the estimates are lower for return periods greater than 50 years (NHC, 2016). These differences can be largely attributed to the 1935, 1954, and 2020 flood peak data, which were included in the current study analysis but not in the 2016 study.

The open water hydraulic modelling task involved the creation, calibration, and validation of a hydraulic model, computation of flood levels, and execution of a model sensitivity analysis. A 1D hydraulic model was developed based on the survey data and DTM. The model was then calibrated using HWMs collected during the 2020 flood, the fourth-largest recorded flood on the Smoky River at Watino. NHC selected the 2020 flood as the primary calibration event due to the availability of most recent, reliable, and comprehensive HWM data. Channel roughness values were used as the calibration parameter, resulting in Manning's roughness coefficients of 0.035 upstream and 0.032 downstream of the Highway 49 bridge. Calibration achieved a difference of approximately 0.03 m between observed and simulated water levels, indicating good agreement. Validation using low-flow water level survey data and WSC station 07GJ001 rating curves also demonstrated reasonable agreement. Later, the calibrated model was used to calculate water surface profiles for 13 flood scenarios representing the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year open water flood events. Lastly, a sensitivity analysis was conducted on the simulated water surface elevation profiles for 100-year flood by varying inflow discharges, downstream boundary conditions, and channel and overbank roughness values. The sensitivity analysis on simulated water surface elevations shows the model is most sensitive to variations in inflow discharge and channel roughness values.

The computed flood levels were then used to determine the extent of inundation for each of the respective flood scenarios and are presented as a set of flood inundation maps for each scenario (the flood inundation map library). This library is intended primarily for stakeholders to use in emergency response planning and preparation. The inundation maps indicate that some buildings and residences in the Hamlet of Watino could be affected during a 200-year flood. Few additional buildings/residences may be impacted in 350-year and higher flood events. The Highway 49 bridge would be affected during a 350-year flood, with floodwaters reaching its low chord and overtopping the right bank approach road. The bridge itself would only be overtopped during 1000-year or larger flood.

The design flood hazard mapping component provided a design flood hazard map – a key deliverable for this flood study. The design flood hazard map depicts the floodway and flood fringes (including high hazard areas) for the open water design flood. The supporting rationale for the flood hazard map is depicted on the open water floodway criteria map. The methods used to develop the flood hazard map follow the provincial Flood Hazard Identification Program guidelines, incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta. The floodway criteria and flood hazard maps show no notable overbank areas within the floodway or flood fringes.

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APPENDICES

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ABBREVIATIONS

Acronym / Abbreviation	Definition
1D	One-dimensional
2D	Two-dimensional
3TM	Three-degree Transverse Mercator
AEP	Alberta Environment and Parks; formerly Alberta Environmental Protection
ASCM	Alberta Survey Control Monument
ATS	Alberta Township System
BF	Bridge File
CGVD28	Canadian Geodetic Vertical Datum 1928
CORS	Continuously Operating Reference Station
CSRS	Canadian Spatial Reference System
DTM	Digital Terrain Model
EPA	Alberta Environment and Protected Areas
FHIP	Flood Hazard Identification Program
GNSS	Global Navigation Satellite System
HEC-RAS	Hydrologic Engineering Center River Analysis System
NAD83	North American Datum 1983
NHC	Northwest Hydraulic Consultants Ltd.
PPP	Precise Point Positioning
PXS	Planned Cross Section
RS	River Station
RTK	Real-Time Kinematic
TEC	Alberta Transportation and Economic Corridors
TIN	Triangular Irregular Network
WSC	Water Survey of Canada
WSE	Water Surface Elevation

SYMBOLS AND UNITS OF MEASURE

Symbol / Unit of Measure	Definition
°	degree
±	plus or minus
%	percent
cm	centimetre
km	kilometre
km^2	square kilometre
m	metre
m^3	cubic metre
m/s	metres per second
m^3/s	cubic metres per second
m/m	metres per metre

1 INTRODUCTION

This section provides comprehensive insights into the background of the Watino Flood Study, outlines its specific objectives, and includes a detailed description of the study area and its reaches.

1.1 Study Background

The Watino Flood Study was initiated by Alberta Environment and Protected Areas (EPA) to identify and assess flood hazards for areas along approximately 15 km of the Smoky River through the Municipal District of Smoky River and Birch Hills County, including the Hamlet of Watino. This study is being undertaken as part of Alberta's Flood Hazard Identification Program (FHIP), which is intended to enhance public safety and reduce future flood damages in the province. Results from this study are also intended to inform local land use planning decisions, flood mitigation projects, and emergency response planning.

The Watino Flood Study is comprised of five major components: 1) survey and base data collection; 2) open water hydrology assessment; 3) open water hydraulic modelling; 4) open water flood inundation mapping; and 5) design flood hazard mapping.

A previous provincial flood hazard mapping study was completed for the Watino area by Northwest Hydraulic Consultants (NHC) in 1996, however, the present study covers an expanded study reach and represents an update to the prior work.

1.2 Study Objectives

The study objectives include the following primary services and deliverables:

- Conduct river cross section surveys.
- Collect data on hydraulic structures.
- Conduct a field survey and integrate data from the digital terrain model (DTM).
- Document flood history.
- Conduct open water hydrology assessment to determine flood frequency estimates.
- Develop a one-dimensional (1D) open water hydraulic model.
- Simulate open water floods for 13 return periods and create water surface profiles throughout the study reach.
- Perform sensitivity analysis on selected modelling parameters.
- Produce flood inundation maps for selected return periods.

- Determine floodway criteria and boundary line.
- Produce floodway criteria and design flood hazard maps.

1.3 Study Reach

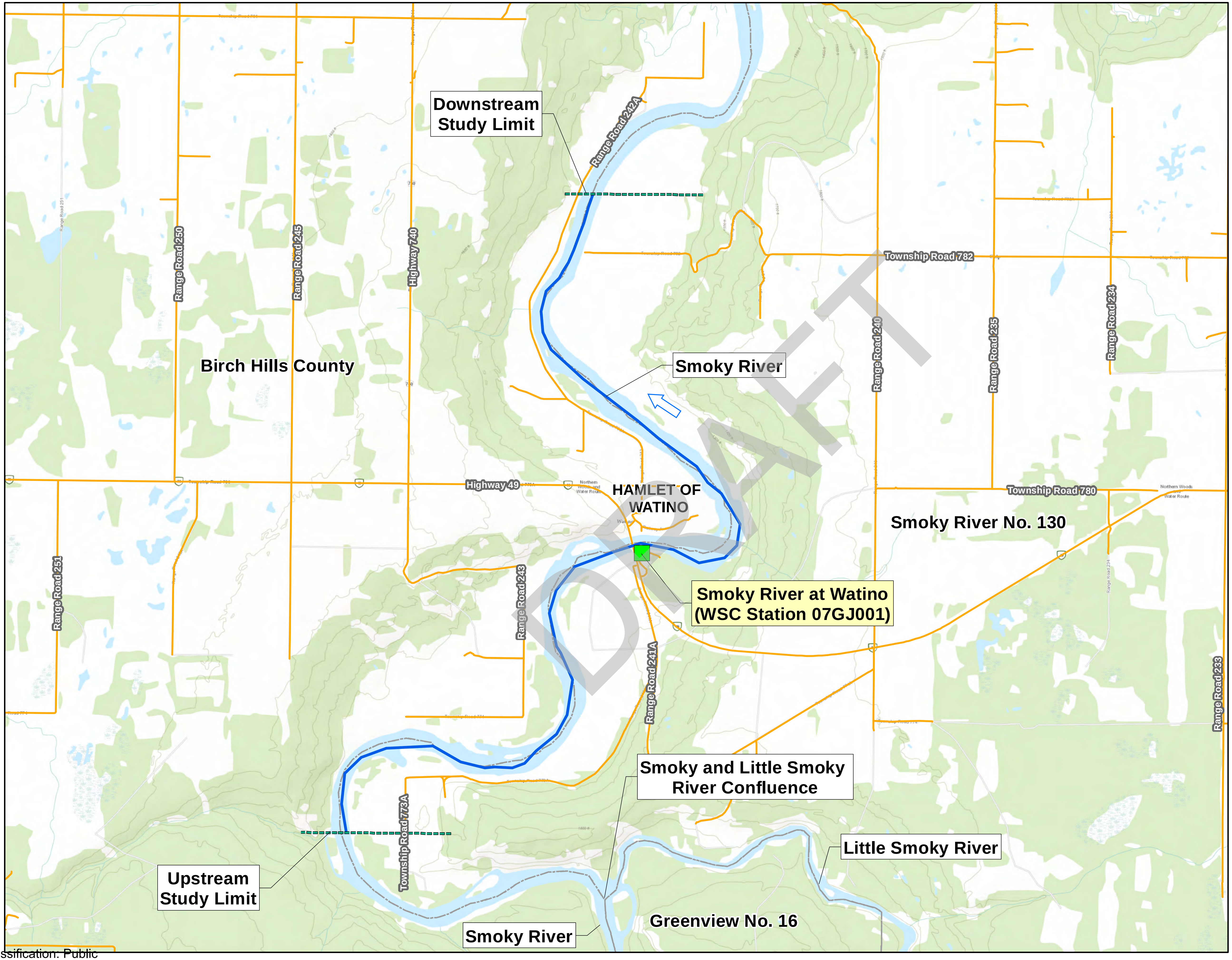
Figure 1.1 shows the location and boundaries of approximately 15 km long study reach of the Smoky River, which extends from 5 km downstream of the confluence with Little Smoky River to 1 km downstream of Township Road 782. Water Survey of Canada (WSC) gauging station 07GJ001 is located in the middle of the study reach.

River cross section surveys were extended beyond these boundaries to accommodate hydraulic modelling and inundation mapping requirements.

1.4 River and Basin Settings

The Smoky River is a major tributary of the Peace River, originating in a steep mountainous region southwest of Grand Cache, Alberta. From there, the river flows northeast through Alberta's foothills and prairie landscapes before discharging into the Peace River, approximately 5 km upstream of the Town of Peace River.

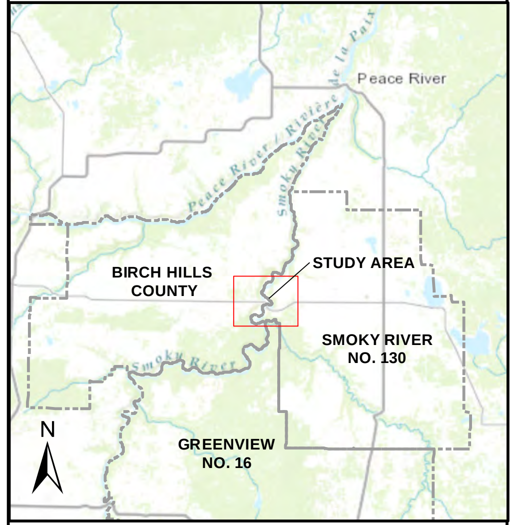
Approximately 51,850 km^2, the Smoky River basin (Figure 1.2) extends across the Rocky Mountains and Foothills, the Western Alberta Plains, Wapiti Plain Physiographic Regions of Alberta, and the Boreal Forest Natural Region (NHC, 1996). The Hamlet of Watino is located about 65 km upstream of the Smoky River confluence with the Peace River. Above this point, the basin area of the Smoky River is approximately 50,300 m^2 and includes three major tributary sub-basins: the Wapiti River sub-basin (14,830 km^2), the Simonette River sub-basin (5,390 km^2), and the Little Smoky River sub-basin (13,375 km^2).



Classification: Public

Alberta Canada

nhc



🟩 WSC GAUGE STATION
- - - STUDY LIMIT
— STUDY REACH
— ROAD
⬚ MUNICIPAL BOUNDARIES

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:50,000
0 1 2 KM
N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

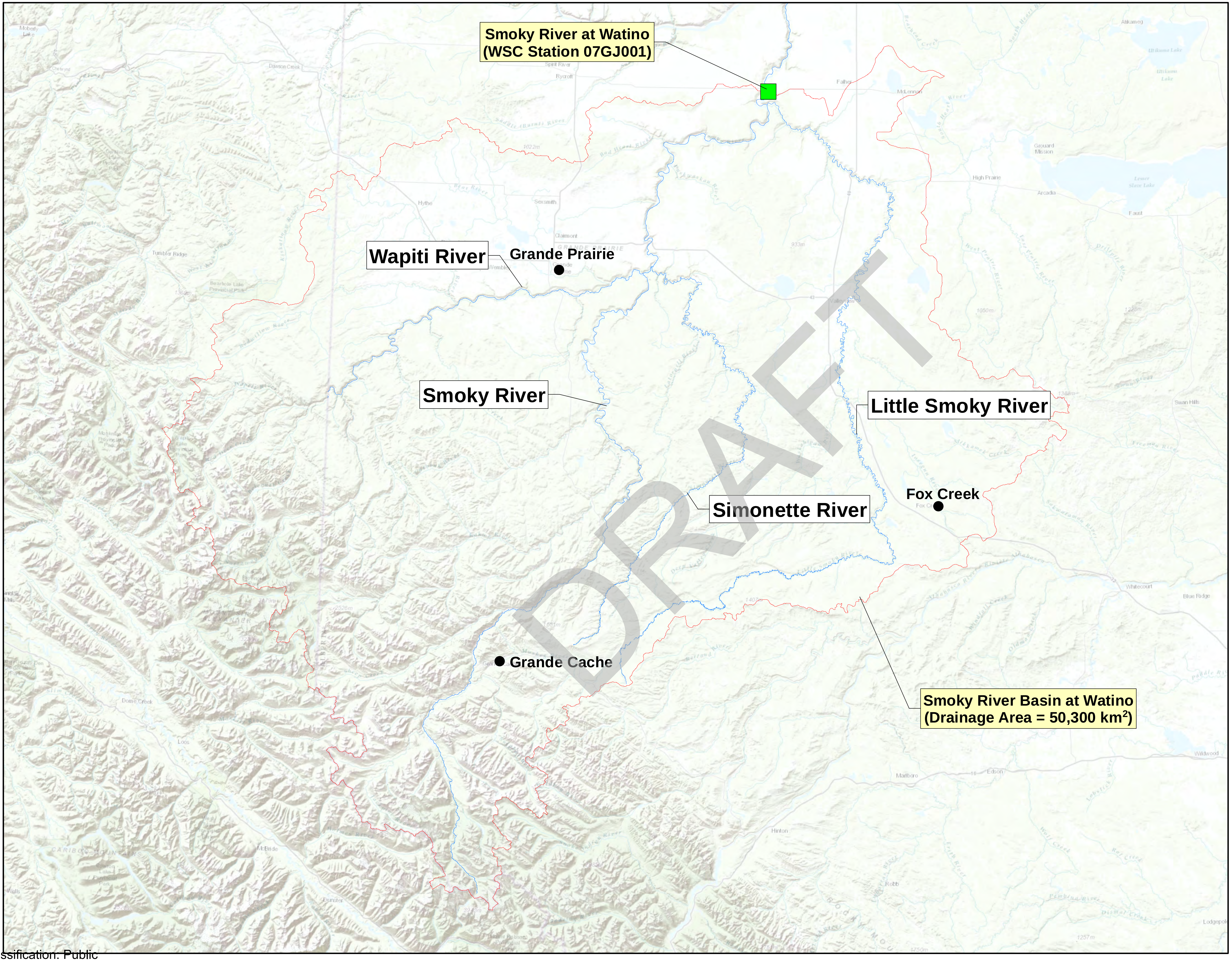
Job Number	Date
1008016	*14-JAN-2025*

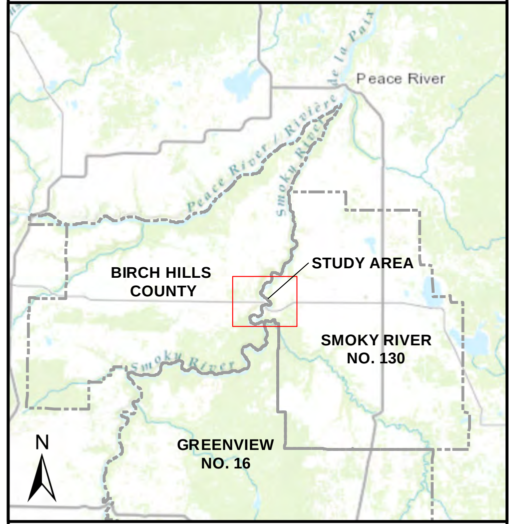
WATINO FLOOD STUDY

FLOOD STUDY REACH

FIGURE 1.1

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS FIG 1_1 FLOOD STUDY REACH.mxd





🟥 SMOKY RIVER BASIN AT WATINO
● MUNICIPAL LOCATIONS
🟩 WSC GAUGE STATION
— MAJOR RIVERS

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:1,260,000
0 20 40 KM
N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*14-JAN-2025*

WATINO FLOOD STUDY

SMOKY RIVER BASIN OVERVIEW

FIGURE 1.2

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS FIG 1_2 SMOKY RIVER BASIN OVERVIEW.mxd

2 SURVEY AND BASE DATA COLLECTION

This section presents details on the survey procedures and methodology, including the coordinate system and datum, as well as a summary of the control network points and discussion of survey accuracy and comparisons with published benchmarks. Also included in this section are descriptions of the collected baseline data, such as DTM, aerial imagery, surveyed cross sections, hydraulic structures, flood control structures, site photographs, hydrometric gauging station information, and other base mapping features.

2.1 Survey Procedures and Methodology

The survey program was conducted between 29 May and 2 June 2023, with additional data collected on 29 August 2023. The objective was to survey channel cross sections along the study reach to support the development of a 1D hydraulic model. Prior to the survey, a site visit was carried out on 1 and 2 May 2023, to inspect the study reach, assess the overall condition of the river channel and floodplain, and plan the survey program. After the site visit, the survey plan was submitted to and approved by EPA.

Ground positioning for the survey was measured using Global Navigation Satellite Systems (GNSS) and Trimble R10 and R12 Real-Time Kinematic (RTK) GNSS receivers. In addition, the bathymetric surveys were performed in areas deeper than 0.5 meters using an Odom echo sounder mounted on a jet boat to measure water depth under the transducer. The position and height of the transducer were recorded with the RTK GNSS receiver mounted directly above it and paired through Hypack (a software suite used for hydrographic surveying). Riverbed elevations were derived by subtracting sounding depths from transducer elevations. In shallower, wadable areas, elevations were directly measured with the RTK GNSS receiver attached to a survey rod. The surveyed cross sections included the riverbanks and extended into the floodplain, overlapping with the DTM provided by EPA.

The Trimble RTK GNSS receivers used in the survey can provide an accuracy of ±0.02 m under optimal conditions when mounted on a tripod with a clear view of the sky, sufficient satellites, and proper static data collection. Additional errors may occur if the receiver is off-level or; obstructed by trees or vegetation, or if the receiver height is incorrectly recorded. The expected accuracy of ground-based survey points is ±0.05 m, except in cases where points are surveyed in tree cover or near large vertical banks, resulting in poor satellite coverage. The digital echo sounder used for the boat-based surveys has an accuracy of ±0.01 m under optimal conditions. Due to the pitch and roll of the boat, the expected accuracy of the boat-based survey is ±0.07 m. The collected survey data's accuracy in both ground-based and boat-based surveys falls within the expected accuracy specified in the FHIP *Flood Study Technical Guidelines* (AEP, 2022).

2.1.1 Coordinate System and Datum

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

2.1.2 Control Network

A control point network was established from local available benchmarks and GNSS surveying to provide a spatial reference for the survey program. Four NHC project survey control points, four WSC benchmarks, and two Alberta Transportation and Economic Corridors (TEC) bridge benchmarks were tied into the survey. A list of the control point coordinates is provided in Table 2.1.[1]

Table 2.1 Control point summary

Name	Type	Easting (m)	Northing (m)	Elevation (m)	Measurement Type
NHC Control 1	Project Control Point	-39993.317	6181360.977	386.812	Static Measurement
NHC Control 2		-39116.687	6176344.789	382.121	
NHC Control 3		-41513.324	6173245.439	388.742	
NHC Control 4		-38262.999	6182983.87	374.085	RTK Measurement
WSC BM06-2	WSC Benchmark	-39154.501	6176342.626	382.670	
WSC BM12-1		-39146.155	6176351.381	382.320	
WSC A-2020-9		-39198.550	6176364.909	383.150	
WSC A-2020-8		-39185.358	6176374.823	380.300	
HWY Bridge NW Corner	AT Benchmark	-39299.902	6176647.057	391.512	
HWY Bridge SE Corner		-39212.432	6176353.046	388.396	

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[1] All coordinates presented in Table 2.1 are NHC survey coordinates.

The coordinates for three of the NHC control points were determined by running the GNSS receivers simultaneously in static mode for over two hours at each control point. This was done to obtain precise point positioning (PPP) results from the Canadian Spatial Reference System (CSRS). The data were then post-processed using Trimble Business Center software to adjust the network by establishing baselines between control points.

The adopted coordinates were constrained to NHC Control Point 2, which was central to the survey area and had the longest occupation time and the most redundant data collected. The CSRS-PPP results estimated the total standard deviation of 95%, accounting for both PPP and epoch transformation uncertainties. At NHC Control Point 2, the total horizontal uncertainty was determined to be 0.040 m (easting) and 0.029 m (northing), with a total vertical uncertainty of 0.036 m. The horizontal and vertical errors at the other two control points, after post-processing and adjustment to the reference CSRS-PPP values, are summarized in Table 2.2. The largest horizontal error was 0.0028 m, and the largest vertical error was 0.0059 m.

Table 2.2 Control network errors

Name	Easting (m)	Northing (m)	Elevation (m)
NHC Control 1	0.0026	0.0028	0.0059
NHC Control 2 (constrained to)	N/A	N/A	N/A
NHC Control 3	0.0025	0.0028	0.0056

A comparison between the surveyed elevations (after post-processing and adjustment) and published WSC and TEC benchmark elevations is provided in Table 2.3. While no published horizontal coordinates are available for these benchmarks, NHC's comparison was based only on the vertical coordinates (elevations).

Table 2.3 Comparison between surveyed coordinates and published local benchmark coordinates

Name	Residuals (Surveyed Minus Published) Elevation (m)
WSC BM06-2	-0.202
WSC BM12-1	-0.192
WSC A-2020-9	-0.169
WSC A-2020-8	-0.165
HWY Bridge NW Corner	-0.070
HWY Bridge SE Corner	-0.096

The mean of the elevation residuals in Table 2.3 is -0.149 m, which is relatively high. Due to these higher residuals between the surveyed and published elevations, NHC resurveyed the benchmarks on August 29, 2023. The average difference between NHC's original and resurveyed elevations is -0.010 meters, which is well within the survey tolerance.

Higher residuals were observed for the WSC benchmarks compared to the TEC benchmarks. WSC explained that the published elevations of their benchmarks are not intended for land use surveys or monitoring the true elevation of the benchmark relative to the earth's fluctuating surface. The difference between the surveyed and WSC published elevations could be approximately 20 cm, influenced by ground movement since 1955 and potential errors in the Geodetic Survey of Canada (GSC) benchmarks established in 1955. Therefore, the residuals between NHC's surveyed elevations and the WSC's published benchmark elevations are not unusual.

Overall, survey data obtained from NHC's field program is more accurate than the WSC published benchmarks, and it was used for model development and adjusting calibration data as required.

2.1.3 Temporary Benchmark Survey

As requested by EPA, NHC surveyed three temporary benchmarks established by EPA during their 2020 flood highwater marks (HWM) survey. EPA did not have geodetic elevations for these benchmarks and requested NHC to collect these data, which they require to finalize their 2020 HWM report. These HWMs were used for calibration of the hydraulic model for the current flood study. The coordinates of these temporary benchmarks are provided in Table 2.4 below. Note that the names of the temporary benchmarks were provided by EPA.

Table 2.4 Coordinates of surveyed temporary benchmarks

Name	Easting (m)	Northing (m)	Elevation (m)
Watino -1	-40756.340	6174899.885	384.637
Watino -3	-40370.576	6178779.731	378.692
Watino -4	-38178.120	6177076.862	382.746

2.2 Digital Terrian Model

For this study, a non-hydro-flattened DTM was provided by EPA, constructed from airborne LiDAR data gathered in the fall of 2023.

Elevations extracted from the DTM were compared with selected ground survey points collected by NHC on roads and other high ground. On average, the elevation difference between the DTM and ground survey points was approximately 0.07 m within the sampled data. However, a positive bias was noted in the DTM, meaning it generally shows higher elevations compared to ground survey points. Despite this positive bias, the overall average error falls within an acceptable range, and the DTM was deemed suitable for model development.

2.3 Aerial Imagery

Aerial imagery was acquired for EPA by OGL Engineering on 08 June 2023. Fully processed orthophoto mosaics were provided to NHC by EPA in July 2024.

2.4 Cross Sections and Bathymetry

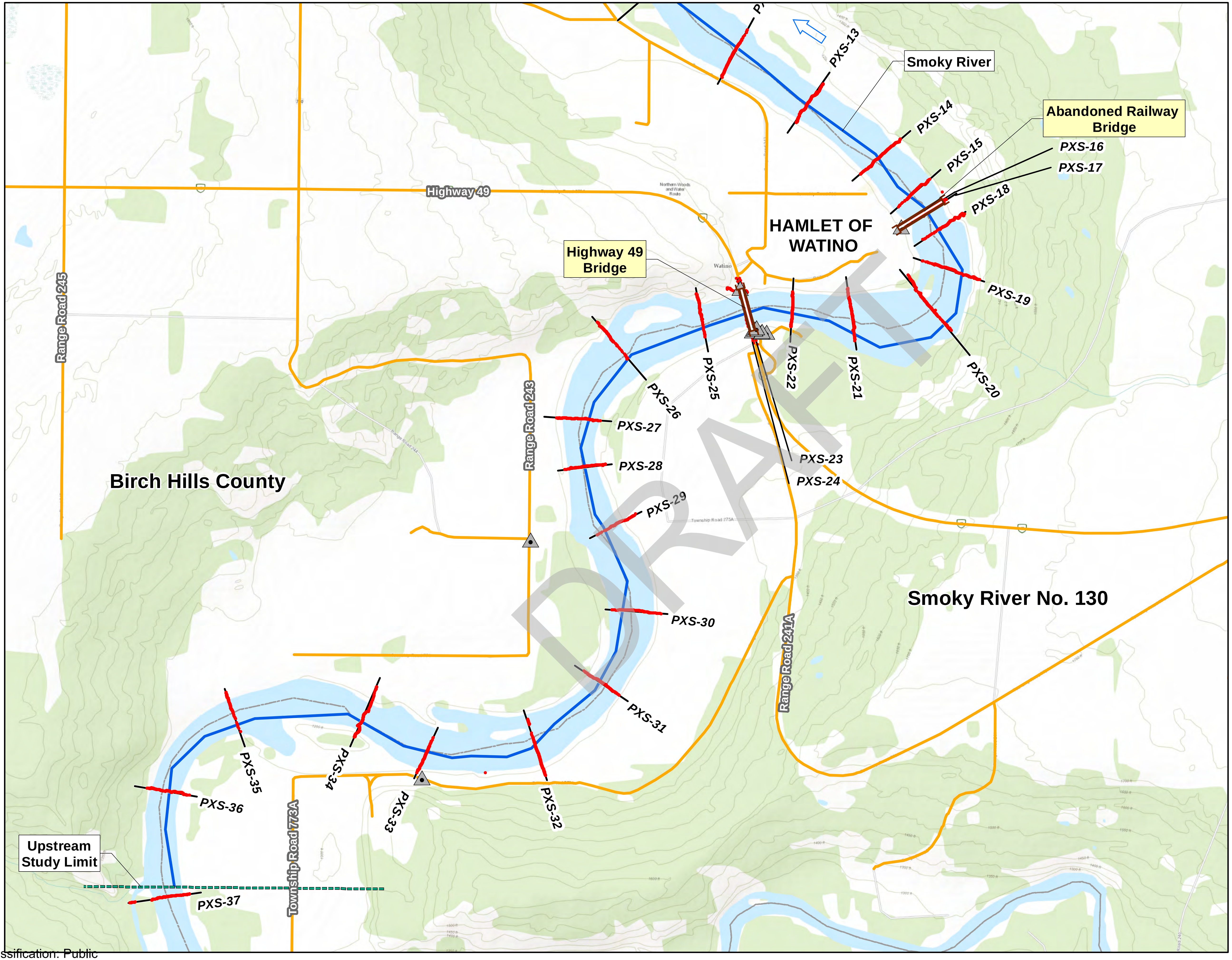
Cross section locations were selected to ensure adequate representation of the channel geometry in the hydraulic model with consideration of repeating cross section locations from the last flood risk mapping study (NHC, 1996), as appropriate. During the planning process for the survey, each cross section was assigned a planned cross section (PXS) identifier to organize the cross sections sequentially and to associate survey point data with a PXS. Most of the cross sections were surveyed between 30 May and 1 June 2023; while one cross section was re-surveyed on 29 August 2023. The PXS cross section and survey point locations are shown on Figure 2.1.

A total of 37 cross sections were surveyed along the Smoky River study reach. The survey was extended about 2 km below the downstream study limit to reduce the uncertainties in hydraulic modelling results within the study reaches caused by downstream boundary condition parameters. The average cross section spacing was approximately 490 m. The minimum spacing was 9.5 m between the cross sections (PXS-24 and PXS-23) on either side of the Highway 49 bridge. The maximum spacing was 980 m between PXS-35 and PXS-34.

The properties of cross sections surveyed are summarized in Appendix A. Thalweg elevation was taken as the minimum surveyed elevation at each cross section. The top of the bank (TOB) channel width was determined based on the survey data, an inspection of the LiDAR-derived DTM data, aerial imagery and cross section profiles. All the survey point data were assembled and provided in the digital file submission.

2.5 Hydraulic Structures

The hydraulic structures surveyed during the field program include two bridge crossings as listed in Table 2.5, along with the corresponding TEC Bridge File numbers when available. The locations of these structures are shown in Figure 2.1. Survey data collected for the bridges included: span length; deck width; top of curb or solid guardrail elevation; low chord elevation; number, width, type, and location of piers; top of deck elevation; and photographs of the bridge. In addition, bridge design drawings were collected as available to complement the survey data. Hydraulic structure details are provided in Appendix B**.**



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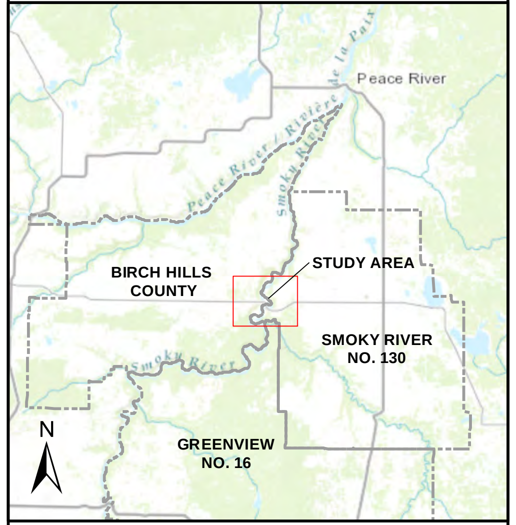


▲	CONTROL POINT
•	SURVEY POINT
⊏⊐	BRIDGE
▬ ▬	STUDY LIMIT
▬	STUDY REACH
▬	SURVEY CROSS SECTION
▬	ROAD
⬚	MUNICIPAL BOUNDARIES

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:25,000

0 1 KM

N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

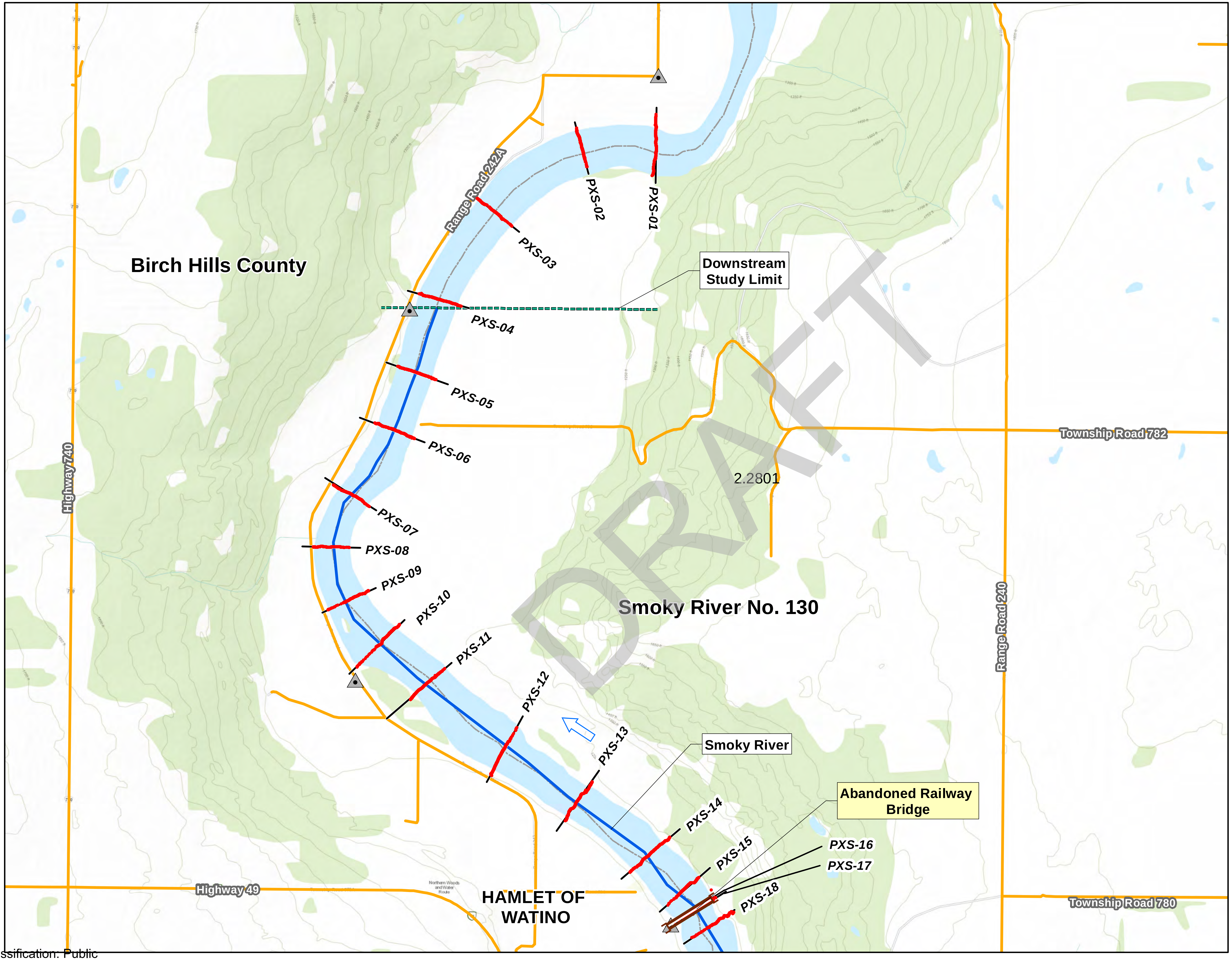
Job Number	Date
1008016	*19-MAR-2025*

WATINO FLOOD STUDY

SURVEY OVERVIEW (1/2)

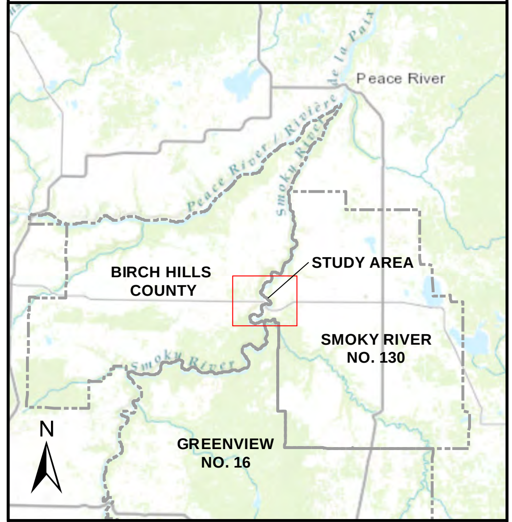
FIGURE 2.1

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS FIG 2_1 SURVEY OVERVIEW 1 of 2.mxd



Alberta Canada

nhc



- ▲ CONTROL POINT
- • SURVEY POINT
- BRIDGE
- STUDY LIMIT
- STUDY REACH
- SURVEY CROSS SECTION
- ROAD
- MUNICIPAL BOUNDARIES

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:25,000

0 1 KM

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*19-MAR-2025*

WATINO FLOOD STUDY

SURVEY OVERVIEW (2/2)

FIGURE 2.1

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS FIG 2_1 SURVEY OVERVIEW 2 of 2.mxd

Classification: Public

Table 2.5 Hydraulic structure survey summary

River	Crossing Description	Structure Type	TEC Bridge File No.
Smoky River	Highway 49 Bridge	Bridge	70241
	Abandoned Railway Bridge	Bridge (Abandoned)	N/A

2.6 Flood Control Structures

The FHIP Guidelines (AEP, 2022) describe flood control structures as a permanent barrier or engineering system that keep water from entering and flooding an area up to a design water level and can be overtopped if flood levels exceed the height of the berms at one or more locations. These flood control structures are mostly earth berms but can also be constructed of concrete and other materials.

Dedicated flood control structures, such as flood berms, typically require regulatory approval prior to construction, are owned, operated, and maintained by a local authority or other government entity, and are officially recognized by EPA and local authorities as flood management infrastructure.

Some road and railway embankments or berms may function as flood barriers and affect the river hydraulics but may not be dedicated flood control structures. Railroad embankments are typically assumed to be permeable and are not treated as natural ground features or dedicated flood control structures in flood studies.

Based on the site visit, survey, and discussion with EPA, the Municipal District of Smoky River, and Birch Hills County, NHC determined that there are no dedicated flood control structures within the Watino flood study area.

2.7 Additional Data

Additional data collected for this study include site photographs taken during the site visit and survey, encompassing both ground shots and aerial drone shots. NHC also collected hydraulic structure drawings and base mapping features as integral components of the study.

2.7.1 Hydrometric Gauging Station Information

A WSC gauging station, 07GJ001, is located within the study reach. Data collected for this station includes discharges, water levels, rating curves, station details, and benchmark elevations.

2.7.2 Water Level Measurements

As part of the survey program, NHC measured water levels throughout the Smoky River study reach. The corresponding discharge information associated with the water level survey were obtained from the WSC Station 07GJ001 located in the middle of the study reach, just downstream of the Highway 49 bridge crossing.

2.7.3 Site Photographs

Appendix C provides annotated reach representative photographs obtained during NHC's site visit and survey program. The location, time, and other metadata information are embedded in the electronic images that are included in the digital file submission.

2.7.4 Hydraulic Structure Design Drawings

NHC collected the most up to date bridge design drawings for Highway 49 bridge from TEC. NHC also obtained a sketch drawing of the abandoned railway bridge, which had been collected as part of the previous flood study (NHC, 1996).

2.7.5 Base Mapping Features

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

3 FLOOD HYDROLOGY

This section provides a summary of flood hydrology for the study area. A more detailed assessment of open water hydrology is provided in the Open Water Hydrology Assessment Memorandum in Appendix D.

3.1 Flood History

A summary of the local flood history, encompassing both open water and ice affected floods, was prepared to provide context for the development and calibration of the hydraulic model. NHC explored and reviewed the following sources to document major flood events in the Smoky River at the Watino study reach:

- Watino Flood Risk Mapping Study (NHC, 1996).
- Flood of June 1972 in the Southern Peace (Smoky River) Basin, Alberta (Warner and Thompson, 1974).

- TEC flood history documentation (1954, 1972, 1982, 1990, and 1997) and hydrotechnical summary for Highway 49 bridge crossing (Bridge File 70241).
- HWM reports for 1972, 1982, and 1990 floods.
- EPA field notes and flood photographs for July 2020 flood.
- Discussion with local people and stakeholders.

3.1.1 Open Water Floods

Annual peak discharges on the Smoky River usually occur between May and July; but they can occur as early as April and as late as August. Most of the Smoky River flows originate from the mountainous and foothills portions of the basin. Runoff from the mountainous region is generally dominated by snowmelt in the spring and early summer, and when combined with major rainfall storms in the foothills region, peak discharges on the Smoky River typically occur.

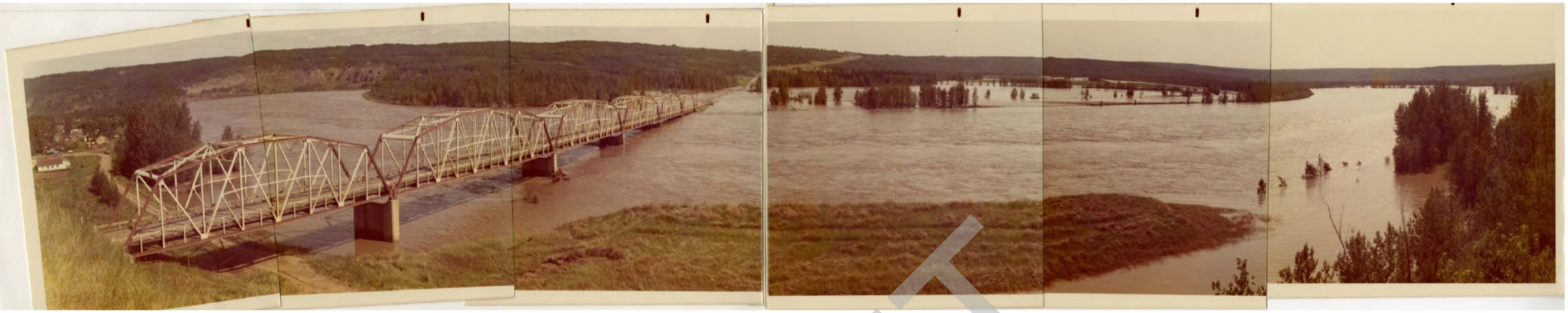
Recent and Recorded Open Water Floods

Systematic flow monitoring data for the Smoky River at Watino (WSC station 07GJ001) are available from 1915 to 1922 and from 1955 to the present. The data do not show any significant trends over time. The three largest floods on record occurred in 1990, 1972, and 1982, and all three events resulted in published peak instantaneous discharges greater than 9,000 m^3/s. The largest flood on record occurred on 6 June 1990, with a peak instantaneous discharge of 9,400 m^3/s. The most recent flood occurred on 3 July 2020, with a published peak instantaneous discharge of 7,950 m^3/s. Details of these four floods are presented below.

1972 Flood Event

In June 1972, an intense rainfall event occurred over the Smoky and Wapiti River basins. The basin's response to this intensive precipitation event commenced on Monday, 12 June, as water levels began to rise in both the Smoky and Wapiti rivers. The Smoky River at Watino started rising during the morning of 14 June 1972, peaking at a height of 384.1 m at the Watino WSC Station 07GJ001, with an estimated discharge of 9,200 m^3/s. The flood receded, and the flow almost halved on Thursday morning, 15 June 1972.

The flood significantly damaged the Northern Alberta Railway Bridge at Watino (presently abandoned), causing portions of the bridge to completely wash out, cutting the rail line at the crossing. The flood also overtopped the southeast approach of the old Highway 49 bridge. Figure 3.1 illustrates the impacts of the 1972 flood at Watino (including the overtopping of old highway bridge approach and washed out railway bridge). Based on the estimated flow in the Smoky River, the 1972 flood ranks as the second-highest event on record.



(a)

(b)

Notes:
1. (a): Panoramic view of the Old Highway 49 Bridge looking from right bank - 14 June 1972 - 5:00 PM (Photo By: A. Mah).
2. (b): Bridge southeast approach is overtopped and HWM is evident - estimated high water level: 384.11 m - 14 June 1972 (Photo By: A. Mah).

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Coordinate System:
Units: As Shown

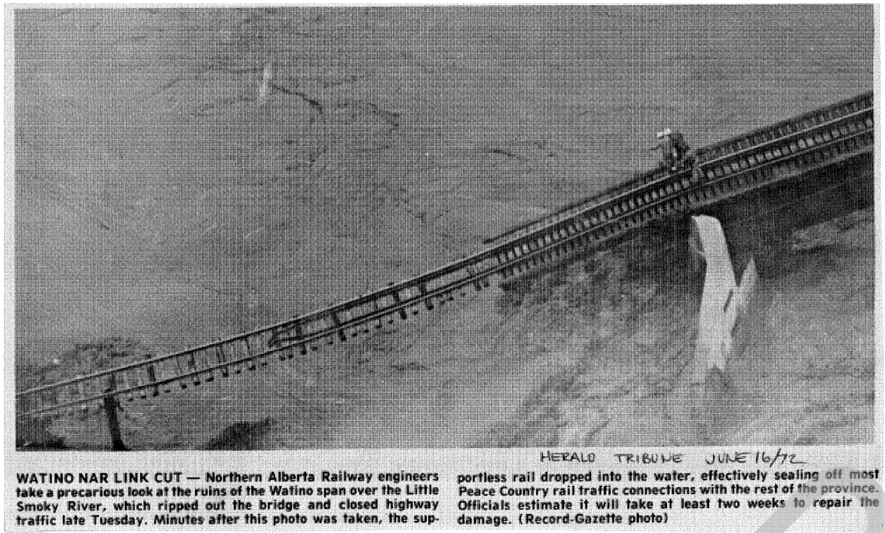
Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1972 FLOOD PHOTOGRAPHS (1/3)

FIGURE 3.1



HERALD TRIBUNE JUNE 16/72

WATINO NAR LINK CUT — Northern Alberta Railway engineers take a precarious look at the ruins of the Watino span over the Little Smoky River, which ripped out the bridge and closed highway traffic late Tuesday. Minutes after this photo was taken, the supportless rail dropped into the water, effectively sealing off most Peace Country rail traffic connections with the rest of the province. Officials estimate it will take at least two weeks to repair the damage. (Record-Gazette photo)

(a)

FORECAST: RAIN 92 PAGES EDMONTON, ALBERTA, SATURDAY, JUNE 17, 197

Track hanging after washout

Three-hundred feet of track were left hanging 40 feet in the air when the Northern Alberta Railway bridge at Watino was washed out. The flooding Smoky River washed away 300 feet of fill beneath the track, 299 miles north of Edmonton. NAR manager Ken Perry took this picture while surveying the damage to the railway's line to Dawson Creek. His repair crews hope to have the bridge in operation again by June 27. The damage to the bridge was $100,000.

(b)

Notes:
1. (a): Herald Tribune Newspaper Article - 16 June 1972 depicting the damage on the Northern Alberta Railway Bridge.
2. (b): Edmonton Post Article - 17 June 1972 showing the damage to the Northern Alberta Railway Bridge - looking east.

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Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1972 FLOOD PHOTOGRAPHS (2/3)

FIGURE 3.1

(a)

(b)

(c)

Notes:
1. (a) and (b): Washed out right bank approach of Northern Alberta Railway Bridge - 6 June 1972 (Photo by: J. Phelps).
2. (c): Panoramic view of impacted Northern Alberta Railway Bridge from right bank.

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Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1972 FLOOD PHOTOGRAPHS (3/3)

FIGURE 3.1

1982 Flood Event

The 1982 flood resulted from summer storms that occurred during mid-July of 1982. The flow reached an estimated 9,020 m^3/s on 16 July 1982, leading to a highwater elevation approximately 383.74 m at Watino (roughly 0.4 m lower than the 1972 flood).

Although no structural damage was reported at the Old Watino Highway 49 bridge, traffic on the bridge was halted for several hours due to flooding of the approach grades. The flooding also caused severe damage to the railroad bridge at Watino (now abandoned), resulting in closure of the bridge for several months. Scouring at Pier No. 1 (at right bank) caused the structure to drop by approximately 1 m and shift downstream by some 2 m. The scour hole depth at the pier measured approximately 4 m after the flood. The 1982 flood ranks as the third largest flood on record. Figure 3.2 shows the impact of 1982 flooding on the two bridges, both situated in the study reach.

1990 Flood Event

The 1990 flood event was characterized by a relatively "wet" start to the month of June, with an already high base flow in the Smoky River prior to the flood. The rise in water levels at Watino began on 10 June 1990, reaching peak levels of 384.13 m at the old Highway 49 bridge and 382.54 m at the rail bridge at 1:00 pm on 13 June 1990. The southeast approach of Highway 49 was completely flooded to an extent of approximately 600 m. The recorded flood flow reached as high as 9,400 m^3/s at the Watino WSC Station 07GJ001, making it the highest recorded event.

Following the 1990 flood event, it took approximately 10 days for the floodwaters to recede and water levels to return to pre-flood levels. The gradual decrease in water levels allowed for the HWM survey to be conducted shortly after the peak of the event. Figure 3.3 depicts the impact and documented HWMs following the 1990 event at the Hamlet of Watino.

2020 Flood Event

The most recent flood event in Watino occurred in July 2020. The Smoky River exhibited an upward trend in water levels from 2 July 2020, at the Watino Highway 49 bridge site. The water continued to rise throughout the night until reaching its maximum height of 382.65 m at the bridge on 3 July 2020, at 5:40 pm. At the peak on 3 July, the flood flow was estimated to be approximately 7,950 m^3/s.

The flood event resulted in significant bank failure along 1 km of Range Road 242 and 242A, located approximately 8 km downstream of the Watino Highway 49 bridge. As a result, the road shifted about 50 m west of its original location. Additionally, around 1.5 km of Range Road 243 along the left bank of the Smoky River and upstream of the Highway 49 bridge was also inundated.

Figure 3.4 presents images showing the extent of flooding along the Smoky River within the Watino study area, captured a few days after the flood peak.

(a)

(b)

Notes:
1. (a): View of Old Highway 49 bridge pier 2 nose - from right (south) bank - 17 July 1982
2. (b): Service station, southeast of bridge on upstream side of the road - HWM showed in blue - 17 July 1982

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WATINO FLOOD STUDY

1982 FLOOD PHOTOGRAPHS (1/3)

FIGURE 3.2

Classification: Public

(a)

(b)

Notes:
1. (a): Looking south along Highway 49, HWM shown as the drift on signposts - 17 July 1982.
2. (b): Looking south along Highway 49, HWM showed in blue - 17 July 1982.

SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1008016

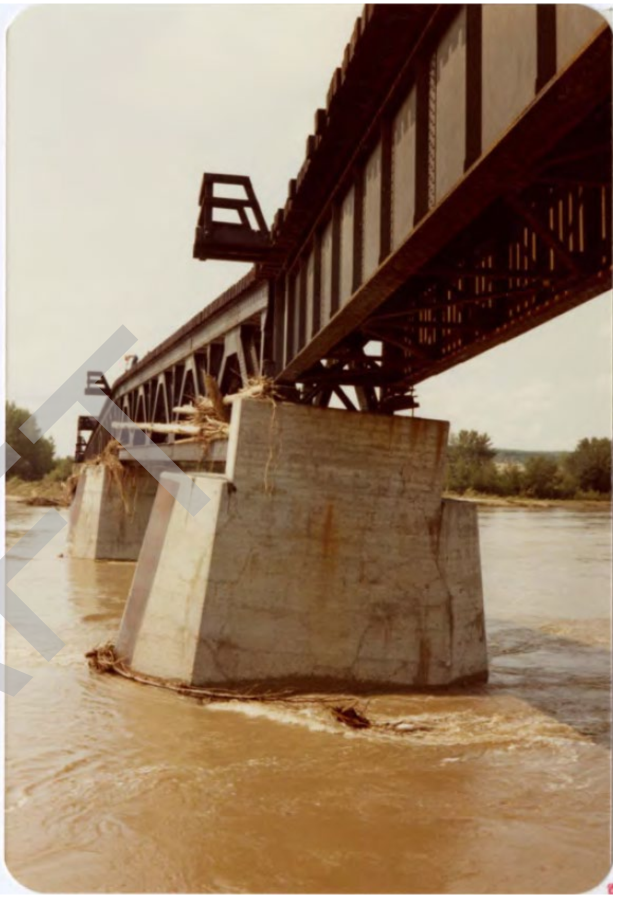
Date: 19-Mar-2025

WATINO FLOOD STUDY

1982 FLOOD PHOTOGRAPHS (2/3)

FIGURE 3.2

(a)



(b)

Notes:
1. (a): Looking west at the railway bridge - 0.9 m drop was observed at Pier 1 - 24 July 1982.
2. (b): Looking southwest along the railway bridge Pier 1, pier movement can be seen in the picture - 24 July 1982.

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Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1982 FLOOD PHOTOGRAPHS (3/3)

FIGURE 3.2

(a)

(b)

Notes: 1. (a) and (b): Old Highway 49 Bridge at Watino during the peak of 1990 flood - 13 June 1990.

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Coordinate System:
Units: As Shown

Job: 1008016

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WATINO FLOOD STUDY

1990 FLOOD PHOTOGRAPHS (1/3)

FIGURE 3.3

(a)

(b)

Notes:
1. (a): Looking southeast- upstream of Old Highway 49 bridge - debris caught on Piers - 15 June 1990.
2. (b): Looking east along the railway bridge - turbulent water patterns evident downstream of piers - 15 June 1990.

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Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1990 FLOOD PHOTOGRAPHS (2/3)

FIGURE 3.3

(a)

(b)

Notes: 1. (a) and (b): Looking southeast- Highway 49 bridge approach road - 13 June 1990 - 11:00 am.

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Coordinate System:
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Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

1990 FLOOD PHOTOGRAPHS (3/3)

FIGURE 3.3

(a)

(b)

Notes:
1. (a): Bank failure along the left bank of the Smoky River near Range Road 242 - 4 July 2020.
2. (b): Range Road 242 A failure along the left bank of the Smoky River, following the flood - 4 July 2020.

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Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

2020 FLOOD PHOTOGRAPHS (1/3)

FIGURE 3.4

(a)

(b)

Notes:
1. (a): Township Road 775 flooding extent following 2020 flood event- looking downstream - 4 July 2020.
2. (b): Flooding along Range Road 243 following the flood- looking upstream - 4 July 2020.

SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

2020 FLOOD PHOTOGRAPHS (2/3)

FIGURE 3.4

(a)

(b)

Notes:
1. (a): Bridge abutment failure along the left bank following the 2020 flood – 4 July 2020.
2. (b): HWM along the north (left) bank, downstream of the Highway 49 Bridge – 4 July 2020.

SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

2020 FLOOD PHOTOGRAPHS (3/3)

FIGURE 3.4

Historical and Observed Open Water Floods

Historic floods refer to major floods that occurred prior to the period of hydrometric data collection and systematic recording of water level and discharge. The magnitude of historic floods can be estimated based on observations and anecdotal information.

Large floods occurred at Watino in some years outside the periods of record, including 1935 and 1954 (Warner and Thompson, 1974). Limited information is available regarding these two flood events. Local inhabitants' knowledge of HWMs suggests that an estimated flow of 7,080 m^3/s (250,000 cfs) could have occurred during the July 1935 event at Watino. Estimated flow for the 1935 event, ranks it the fifth highest flood on record at Watino.

Similarly, the 1954 event at the study site during the month of May 1954 is likely associated with an estimated flow of 6,370 m^3/s. The 1954 event was the sixth largest flood in recorded flood history at Watino.

Visual evidence from available photographs indicates that water levels rose more than 2 m within a span of approximately 2 hours during the 1954 flood, resulting in the complete flooding of Pier No. 2 and the coffer dam for Pier No.1 during the construction phase of Old Highway 49 Bridge. Figure 3.5 presents the images captured during the 1954 flood at the bridge construction site, which shows the pre- and post- flood condition.

3.1.2 Ice Affected Floods

The Smoky River at Watino has also experienced river ice affected floods over the years, commonly caused by ice jams and ice runs. The ice affected floods were not as significant as the open water floods; however, notable highwater levels were observed at the WSC gauge during spring breakup (March-April) in 2018 and 2020.

Recent and Recorded Ice Jam Floods

2018 Flood Event

The 2018 breakup season commenced in late April. Throughout the last two weeks of April, water levels at Watino rose by approximately 3.5 m, reaching a level of 379.73 m at the Highway 49 bridge site. The peak flow was estimated to be approximately 2,780 m^3/s, which corresponds to a larger than 2-year but less than 5-year open water flood event.

(a)

(b)

(c)

Notes:
1. (a): Pier #2 during the construction- picture taken from Pier # 3 - 5 May 1954 (prior to the flood).
2. (b): Pier #2 during the construction- picture taken from Pier # 3 - 14 May 1954 - 11:30 AM.
3. (c): Pier #2 during the construction- picture taken from Pier # 3 - 14 May 1954 - 1:30 PM.

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Date: 19-Mar-2025

WATINO FLOOD STUDY

1954 FLOOD PHOTOGRAPHS

FIGURE 3.5

2020 Flood Event

Water levels during the spring of 2020 experienced a substantial rise, reaching approximately 383.40 m, which is slightly below the 75-year open water flood level. An analysis of the winter 2019 and 2020 data indicated higher-than-average water levels, likely attributed to thicker and rougher consolidated ice conditions. The spring ice cover in 2020 was possibly remnants of a freeze-up ice jam formation at the study site. Gauge data also indicate the likelihood of a juxtaposed and mechanically thickened ice cover formation during the freeze-up period in late October 2019. River flow doubled in a matter of several days during the freeze-up, which potentially contributed to the mechanical ice thickening during this period.

Throughout November 2019, water levels continuously rose for multiple consecutive weeks while the flow was continuously reducing during the winter. The trend was followed by a sudden plunge of the water surface by approximately 1 m in late November, which could be an indication of a possible freeze-up jam release. From December 2019 to the end of February 2020, water levels consistently rose, suggesting the reconsolidation of the ice cover at Watino. Subsequently, the water surface elevation (WSE) dropped by about 1 m in February and March, likely due to gradual ice cover smoothening and deterioration. Nevertheless, water levels remained above the typical levels for the end of winter months at the study site.

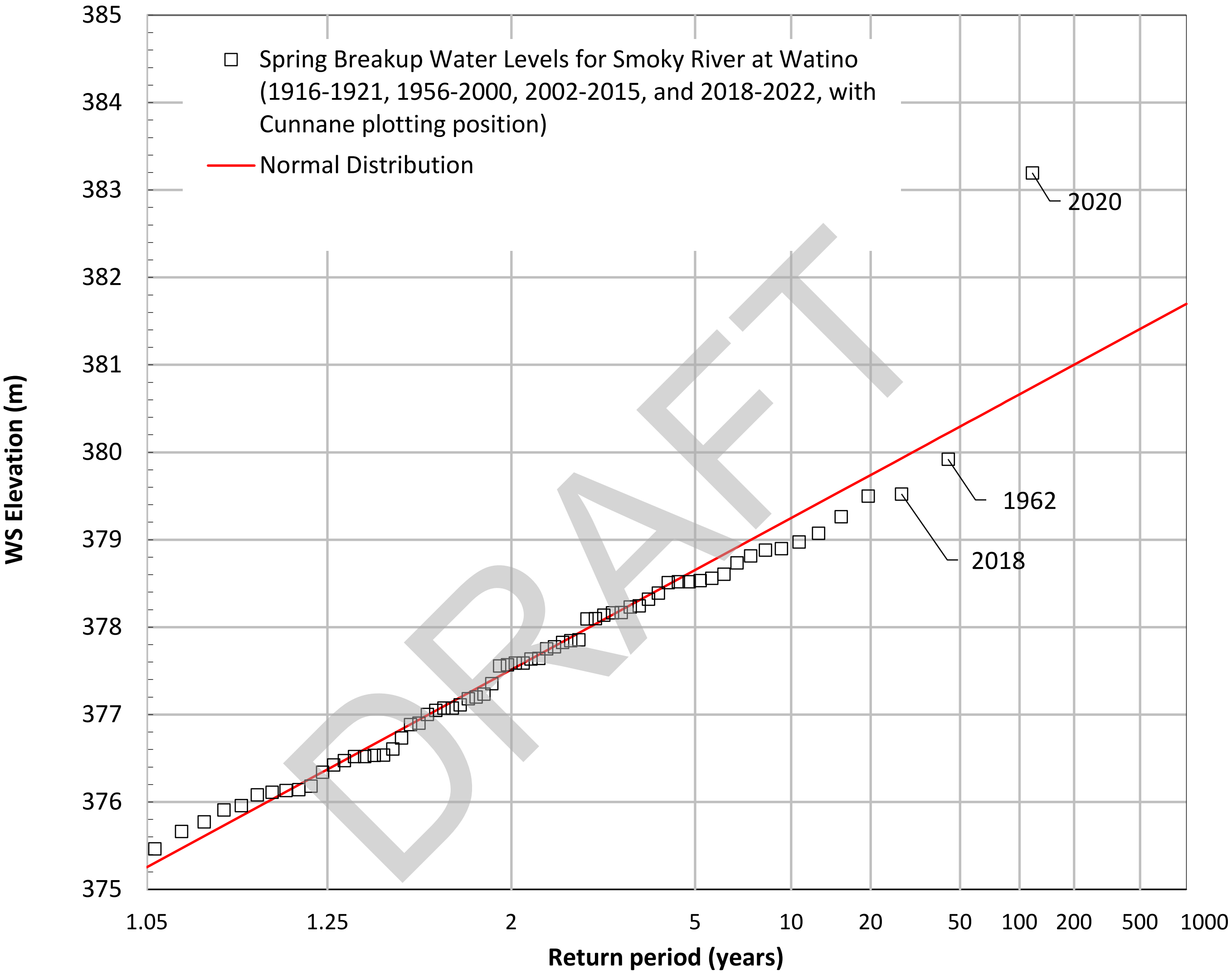
The ice cover condition and associated water levels in spring 2020 corresponded to one of the highest pre-break up water levels recorded in the past decade at Watino. Consequently, when the breakup occurred, the ice cover condition was still competent and solid, resulting in the mechanical breakup that caused the water level to rise to the highest breakup levels at the study site, recorded on 21 April 2020. Field evidence and recordings showed a considerable ice run on 19 April, several days before the peak at the site (presented in Figure 3.6). After reviewing the WSC Station 07GJ001 gauge water level data, NHC concluded that no ice jam had formed at the site during the spring of 2020. If such a phenomenon had occurred, it could have led to significantly higher WSEs, resulting in an index year when both a high ice jam and an open water flood take place.

Ice Breakup Water Level Frequency Analysis

An ice breakup water level frequency analysis was performed for the Smoky River at Watino (WSC Station 07GJ001) using gauge data. Annual maximum ice breakup levels were sourced from the Canadian River Ice Database (CRID), WSC water level data, and preliminary data from EPA. The data were collected for the periods 1916 to 1921, 1956 to 2000, 2002 to 2015, and 2018 to 2022. The analysis, based on the normal frequency distribution, is illustrated in Figure 3.7. It is important to note that the 2020 breakup water level deviates from the normal distribution and is significantly higher than other recorded breakup levels for the study reach. The resulting curve shows a 100-year breakup water level of 380.90 m, which is approximately in between a 10-year to 20-year open water flood and is significantly lower than the major recorded open water floods of 1972, 1982, 1990, and 2020.

Notes: 1. Ice run at Smoky River from north (left) bank - 19 April 2020.

Alberta Canada	SCALE – AS SHOWN	WATINO FLOOD STUDY
nhc	Coordinate System: Units: As Shown	**2020 BREAK UP EVENT PHOTOGRAPHS**
	Job: 1008016 | *Date: 19-Mar-2025*	FIGURE 3.6



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WATINO FLOOD STUDY

ICE BREAKUP WATER LEVEL FREQUENCY ANALYSIS

FIGURE 3.7

3.2 Flood Frequency Analysis

An open water flood frequency analysis was carried out to determine estimates of flood frequencies for a range of return periods up to 1000 years. Details on the flood frequency analysis are provided in the Open Water Hydrology Assessment Memorandum in Appendix D.

3.2.1 Flood Frequency Flow Estimates

Table 3.1 presents the flood frequency estimates for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1000-year open water floods for the Smoky River at Watino. The flood frequency estimates are from a single station frequency analysis based on peak instantaneous discharges at the Smoky River at Watino (WSC Station 07GJ001) gauge and the adopted Bulletin 17C distribution.

Table 3.1 Flood frequency estimates for the Smoky River at Watino

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge (m^3/s)	
		Value	95% Confidence Limit
1000	0.1	19,400	13,400 - 44,000
750	0.13	18,100	12,700 - 39,100
500	0.2	16,300	11,800 - 33,100
350	0.29	14,900	11,000 - 28,500
200	0.5	12,900	9,820 - 22,500
100	1	10,700	8,450 - 16,800
75	1.3	9,830	7,920 - 14,900
50	2	8,740	7,180 - 12,500
35	2.9	7,850	6,570 - 10,700
20	5	6,570	5,640 - 8,350
10	10	5,170	4,560 - 6,130
5	20	3,930	3,540 - 4,440
2	50	2,450	2,230 - 2,690

3.2.2 Comparison to Previous Studies

Table 3.2 presents a comparison of the flood frequency estimates for the Smoky River at Watino with the estimates from previous AEP (1994) and NHC (2016) studies. The estimated peak discharges from this study are smaller than those from AEP (1994) except for the 2-year peak discharges. The AEP 1994 estimates were derived from a Log-Pearson Type III (LP3) frequency curve constructed using a flood discharge data series up to 1993, which is 30 years shorter than that the data series used in this study. The shorter data series used in the 1994 study contained the same three largest flood events as the data series used in this study (the 1990, 1972, and 1982 events), and thus the return periods for these events were under-estimated in the previous study (AEP, 1994). As such, the flood peak estimates from the 1994 flood frequency analysis are therefore too high, which is not representative of the current data series.

The flood frequency estimates from this study are slightly higher than the estimates by NHC (2016) for return periods up to 50 years but are lower for longer return periods. The differences could be largely attributed to the 1935, 1954 and 2020 flood peaks, which are included in the analysis of this study but not in the NHC 2016 study. These three events were relatively large. The differences are probably also related to the difference between the Bulletin 17C and LP3 distribution adopted by this and the 2016 study, respectively.

Table 3.2 Comparison with previous flood frequency estimates

Return Period (Years)	Peak Instantaneous Discharge (m^3/s)		
	This Study	AEP (1994)	NHC (2016)
1000	19,400		21,300
750	18,100		19,700
500	16,300		17,500
350	14,900		15,800
200	12,900	15,400	13,400
100	10,700	12,300	10,800
75	9,830		9,860
50	8,740	9,730	8,650
35	7,850		7,690
20	6,570	7,010	6,340
10	5,170	5,350	4,900
5	3,930	3,950	3,680
2	2,450	2,390	2,270

4 HYDRAULIC MODELLING

This section provides an overview of the hydraulic modelling relevant to this study. It includes discussions on the data used in the modelling, the characteristics of the rivers and valleys, the model development and calibration process, the results obtained from the model, and the model sensitivity analysis.

4.1 Available Data

Data pertinent to the development of a hydraulic model includes basin hydrology, recent high-resolution terrain data representing the floodplain, survey data, existing hydraulic models, and calibration data. The data available for this study are summarized below.

4.1.1 Survey and Base Data

The hydraulic modelling utilized the following survey and base data:

- cross section survey data gathered by NHC
- survey data on hydraulic structures such as bridges, collected by NHC
- hydraulic structure design drawings
- water level survey data collected by NHC throughout the study reach

In addition, the non-hydro-flattened DTM received in March 2024 was used for constructing the model. For additional details regarding the acquired survey and base data, refer to **Section 2**.

4.1.2 Previous Models

In 1996, NHC completed a flood risk mapping study for the provincial government and developed a HEC-2 numerical hydraulic model for the Smoky River at Watino. The 1996 model covers a segment of the Smoky River within the present study area. During the development of the hydraulic model for this study, parameters from the NHC (1996) model were referenced and compared to the values selected for the present analysis. This comparison serves as a validation check for the parameters chosen in the current study.

4.1.3 Highwater Marks

HWM observations serve as documentation of the highest water levels reached at specific locations during a particular flood event. These observations are instrumental in calibrating and validating hydraulic models because they enable comparisons between simulated and observed water levels along a specific study reach. To gather HWM data, various sources such as previous studies, the EPA database, flood history documentation from TEC, and local historical archives were thoroughly searched. HWM data are available from TEC flood history documentation for the Smoky River study reach at Watino for 1972 and 1982, and HWM data collected by EPA/AENV are available for 1990 and 2020.

Table 4.1 lists the HWM elevation values reported for the June 1972, July 1982, June 1990, and July 2020 floods and Figure 4.1 depicts their locations along the river. The locations were initially plotted from approximate observation coordinates included in the available HWM reports. These locations were further validated based on the information available specific to each HWM observation site (e.g., site photographs, comments on their location with respect to prominent features, such as bridges or buildings). Each HWM location was then assigned a river station (RS) value representative of its location alongside the model channel centreline; that is, the streamwise distance from the downstream model boundary. At one HWM location, the reported elevation value appeared suspiciously low in relation to the neighboring HWMs. It is plausible that a reported elevation value at this location is incorrect or the streamwise location might not have been accurately deduced from the information available in the HWM report. This location is denoted as "suspect" in Table 4.1.

The available HWMs tabulated below are associated with the four largest recorded events in the Smoky River at Watino, with the 1990 event identified as the largest, followed by the 1972, 1982, and 2020 flood events. The corresponding peak discharge rates for these HWM events were obtained from the published peak instantaneous flows at the WSC Station 07GJ001 (Smoky River at Watino).

Among all the available HWMs, the 2020 HWMs are the most suitable for calibration in this study because they were collected most recently; they are available throughout the reach; they accurately reflect the current channel, floodplain, and hydraulic structures; and they were tied to the NHC survey. In contrast, the HWMs from 1972, 1982, and 1990 were collected more than 30 years ago. Since then, the Highway 2 bridge has been reconstructed, and the channel bed may have changed due to several reported scour events near the bridge. Additionally, the benchmarks and datum used in those surveys are unknown, making it difficult to precisely align them with current model results. For these reasons, NHC has determined that the older HWMs cannot be relied upon with the same level of confidence for this study.

4.1.4 2020 Post Flood Surveyed Water Levels

The HWM elevations for 2020 flood along the Smoky River at Watino study reach were collected on 8 July 2020, 5 days after the peak of the 2020 flood and after water levels had receded. During the HWM survey, EPA also surveyed the river water levels at three locations, and these surveyed water levels provided additional calibration data. The discharge reported at the gauge on the day of the EPA survey was 1,920 m^3/s, which is below the 2-year event discharge. Thus, these data could be used for low-flow validation. Table 4.2 lists the surveyed water elevation values from the survey on 8 July 2020 survey. The location of these surveyed water levels is added to the Figure 4.1.

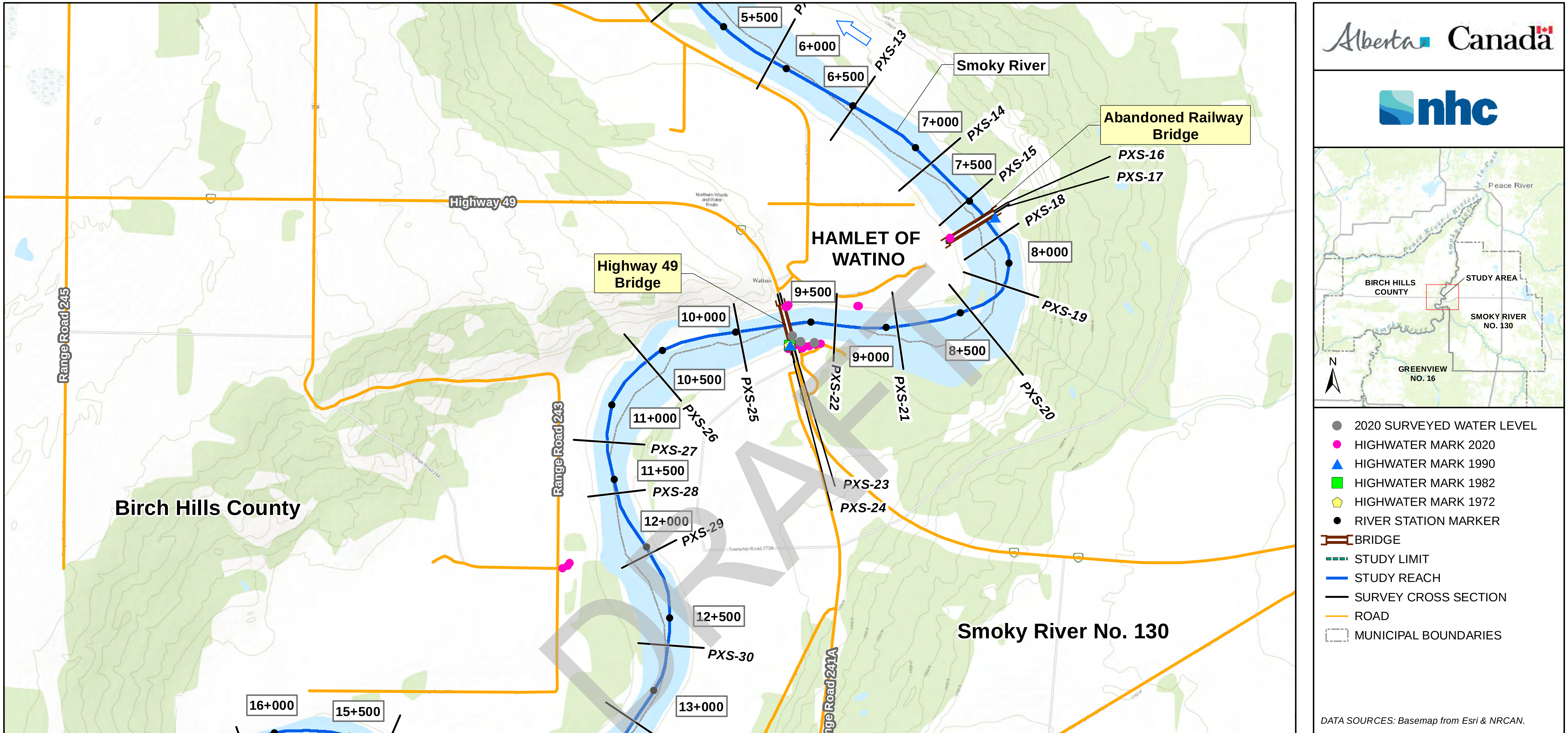
Table 4.1 Summary of available HWMs

Event	HWM ID	Location Description	River Station (m)	Discharge (m^3/s)	HWM Elevation (m)
14 June 1972	N/A	Southeast approach to Highway 49 bridge	9,664	9,200	384.110
16 July 1982	N/A	Southeast approach to Highway 49 bridge	9,659	9,020	383.740
13 June 1990	N/A	Southeast approach to Highway 49 bridge	9,654	9,400	384.110
	90-SM-15A	Highway 49 bridge crossing, downstream edge of bridge (right bank)	9,657		384.130
	90-SM-16	Upstream of Northern Alberta Railway bridge crossing (right bank)	7,722		382.544
3 July 2020	HWM3 - Watino 1	East of Range Road 243 (erosion line in sand/gravel pile)	11,810	7,950	384.783
	HWM2 - Watino 1	East of Range Road 243 (erosion line in sand/gravel pile)	11,803		384.765
	HWM1 - Watino 1	East of Range Road 243 (silt line on excavator boom)	11,801		385.065
	HWM3 - Watino 5	Upstream, river right side of Highway 49 bridge (debris in grass)	9,674		382.638
	HWM2 - Watino 5	Upstream, river right side of Highway 49 bridge (debris in grass)	9,671		382.569
	HWM1 - Watino 5	Upstream, river right side of Highway 49 bridge (debris in grass)	9,669		382.521
	HWM8 - Watino 2	Downstream of Highway 49 bridge (debris and pressed grass line)	9,648		382.517
	HWM9 - Watino 2	Downstream of Highway 49 bridge (pressed grass line)	9,634		382.627
	HWM10 - Watino 2	Downstream of Highway 49 bridge (pressed grass line)	9,628		382.704
	HWM7 - Watino 2	Downstream of Highway 49 bridge (debris line)	9,612		382.447
	HWM6 - Watino 2	Downstream of Highway 49 bridge (pressed grass line)	9,599		381.869 (Suspect)
	HWM5 - Watino 2	Downstream of Highway 49 bridge (needle bed at gauge shelter)	9,577		382.455
	HWM4 - Watino 2	Downstream of Highway 49 bridge (debris in grass)	9,560		382.449
	HWM3 - Watino 2	Downstream of Highway 49 bridge (course debris in tree)	9,503		382.278
	HWM1 - Watino 2	Downstream of Highway 49 bridge (debris in tree)	9,448		382.384
	HWM2 - Watino 2	Downstream of Highway 49 bridge (course debris in tree)	9,414		382.313
	HWM12 - Watino 2	Downstream of Highway 49 bridge (erosion line and pressed grass)	9,202		381.875
	HWM11 - Watino 2	Downstream of Highway 49 bridge (pressed grass line)	9,192		382.057
	HWM1 - Watino 4	Left bank of abandoned railway bridge (wash line)	7,630		380.043

Event	HWM ID	Location Description	River Station (m)	Discharge (m^3/s)	HWM Elevation (m)
	HWM2 - Watino 4	Left bank of abandoned railway bridge (Wash line)	7,624		380.109
	HWM3 - Watino 4	Left bank of abandoned railway (coarse debris in tree)	7,579		380.009
	HWM1 - Watino-3	East of Township Road 780A (debris line)	4,768		377.296
	HWM2 - Watino-3	East of Township Road 780A (peak water level staked by country staff)	4,760		377.361

Table 4.2 Summary of surveyed water levels on 8 July 2020

Event	ID	Location Description	River Station (m)	Discharge (m^3/s)	Water Level Elevation (m)
8 July 2020	WL1	Downstream edge of Highway 49 bridge (right bank)	9,458	1920	377.861
	WL2	Downstream edge of Highway 49 bridge (right bank)	9,583		377.888
	WL3	Highway 49 bridge (reflector off bridge pier)	9,633		377.941



DATA SOURCES: Basemap from Esri & NRCAN.

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

Engineer	GIS	Reviewer
MMM	JY	RBA

Job Number	Date
1008016	14-JAN-2025

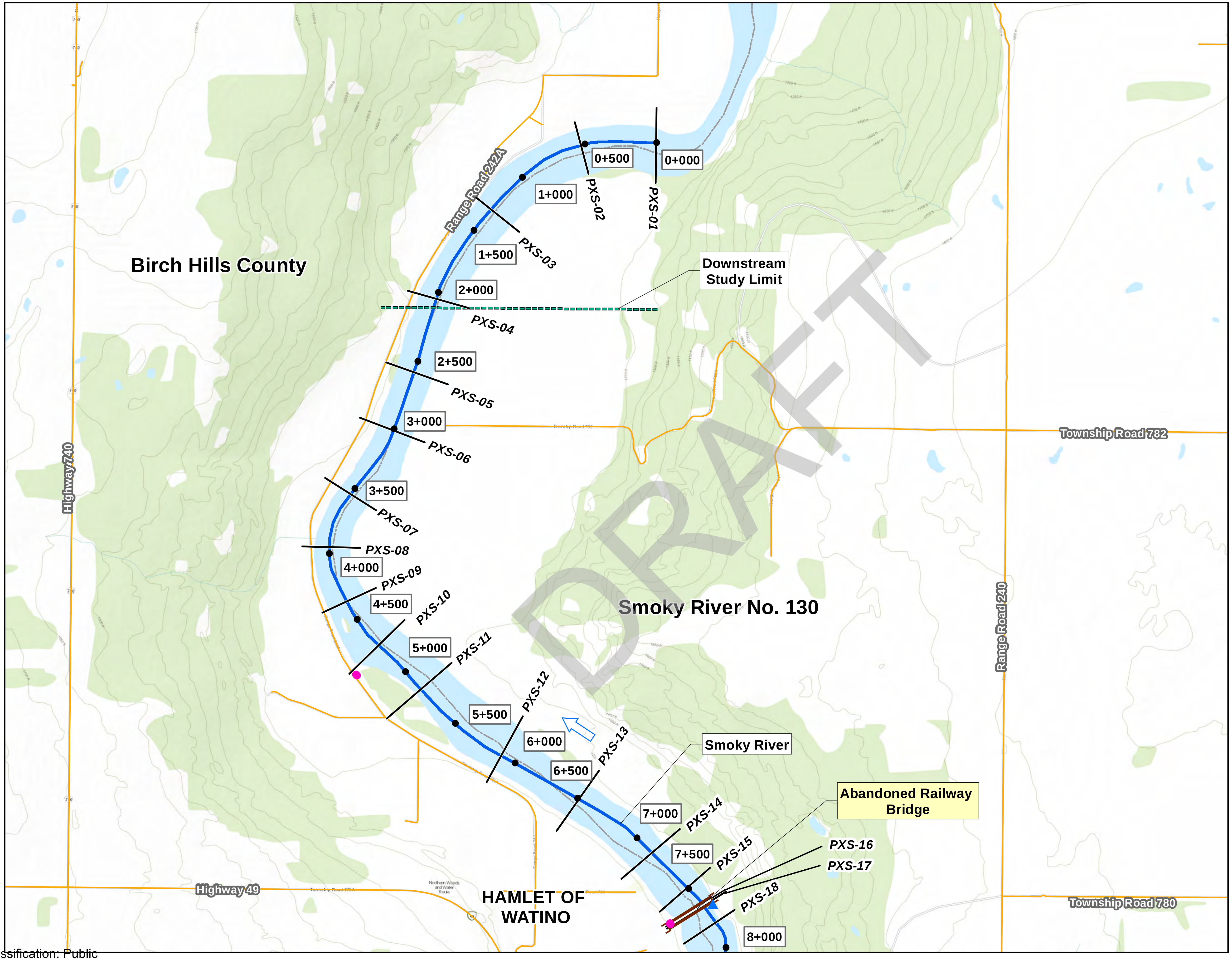
WATINO FLOOD STUDY

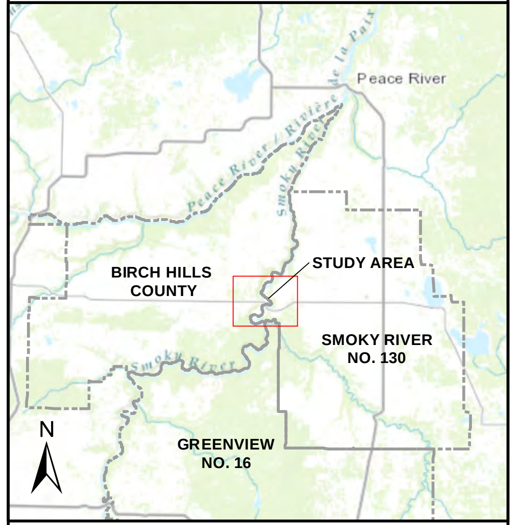
HIGHWATER MARK LOCATIONS (1/2)

FIGURE 4.1

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS FIG 4_1 HIGHWATER MARK LOCATIONS 1 of 2.mxd

Classification: Public





- ● HIGHWATER MARK 2020
- ▲ HIGHWATER MARK 1990
- 🟩 HIGHWATER MARK 1982
- ⬠ HIGHWATER MARK 1972
- ● RIVER STATION MARKER
- BRIDGE
- STUDY LIMIT
- STUDY REACH
- SURVEY CROSS SECTION
- ROAD
- MUNICIPAL BOUNDARIES

DATA SOURCES: Basemap from Esri & NRCAN.

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*14-JAN-2025*

WATINO FLOOD STUDY

HIGHWATER MARK LOCATIONS (2/2)

FIGURE 4.1

4.1.5 Gauge Data and Rating Curves

WSC Station 07GJ001 (Smoky River at Watino), is the lone streamflow gauge located within the study reach. WSC has published discharge record for this gauge from 1915 to 1922 and from 1955 to the present, and the water level record from 2012 to present.

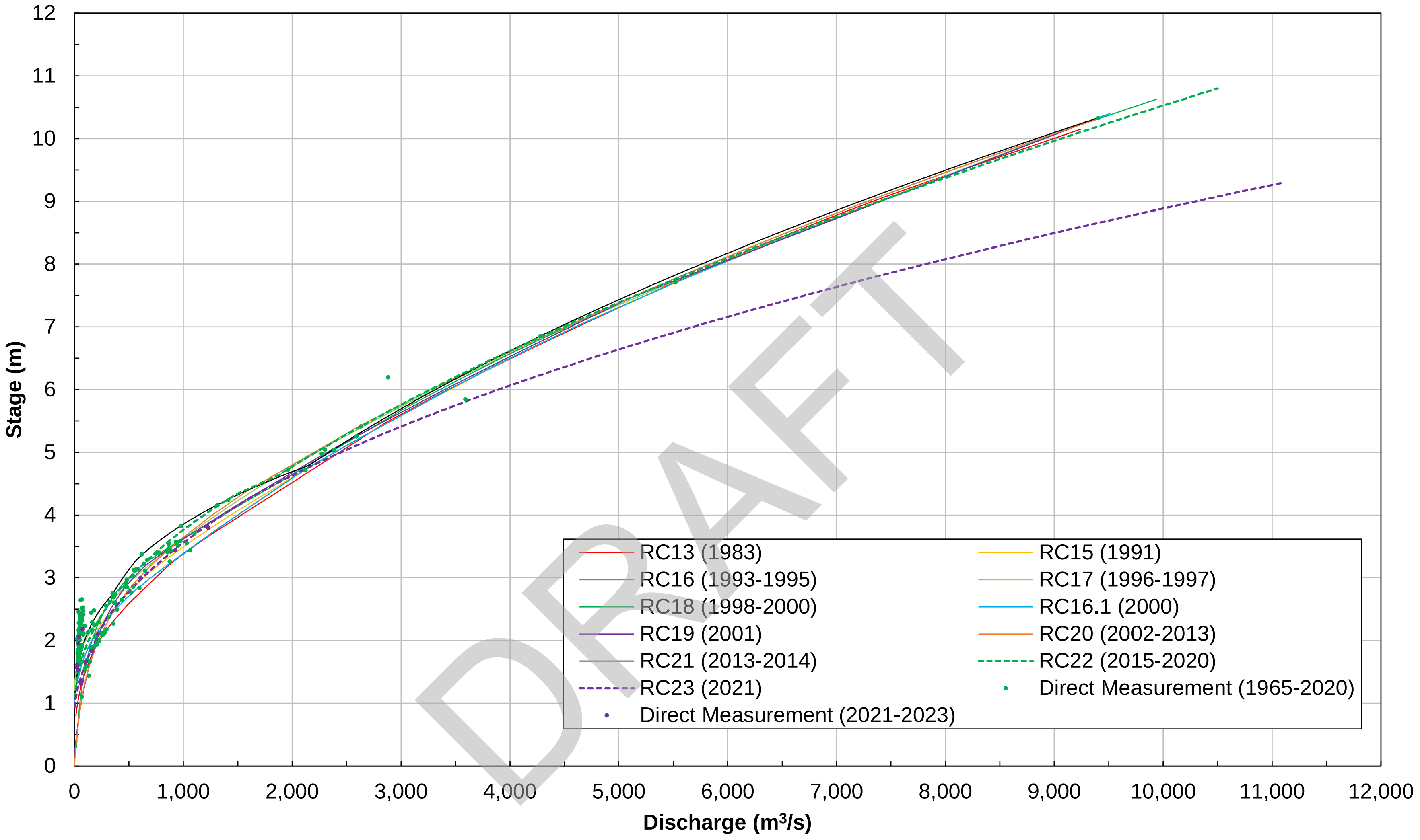
The relationship between stage (or height) and discharge at the gauging station is determined by the WSC, based on recorded stage and direct discharge measurements. This relationship is represented by a curve fit through the observed data, commonly called a rating curve. New direct discharge measurements are continually added to the dataset, and the rating curve is adjusted periodically to fit the additional data. The rating curve relationship allows for discharge (streamflow) to be estimated from the recorded gauge height.

Historical rating curves for WSC Station 07GJ001 were collected from 1983 to the most recent one in 2021. These curves, along with the corresponding discharge measurements, are plotted in Figure 4.2. The plot indicates that the rating curves remained largely consistent from 1983 to 2020. However, the 2021 rating curve (currently used by the WSC) shows significant adjustment. WSC noted that this adjustment was necessary to account for severe scouring that occurred in 2019 and 2020.

4.1.6 Flood Photography

Aerial flood photographs are available for the 1982 and 1990 floods.

In addition, ground flood photographs are available for all the major floods in the study area including 1954, 1972, 1982, 1990, and 2020 open water floods and 2020 ice breakup events. These photographs are referenced in the flood history section (Section 3.1).



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Date: 19-Mar-2025

WATINO FLOOD STUDY

HISTORICAL RATING CURVES FOR WSC STATION 07GJ001

FIGURE 4.2

4.2 River and Valley Features

The Smoky River in western Alberta is a major tributary of the Peace River. At Watino, the river flows through a deep, stream-cut valley, with continuous terraces lining the valley walls. While some areas of the valley walls are covered by forests, shrubs are generally more prevalent across the landscape.

4.2.1 Channel Characteristics

The Smoky River study reach exhibits an irregular meander pattern and features occasional islands, mid-channel bars, and point bars. The average channel slope within the study reach is approximately 0.0009 m/m and the top of bank width is roughly 300 m. The riverbed is composed of gravel and gravelly-boulder material, while the banks primarily consist of gravel overlain by silt and sand. In areas where the channel meets the valley wall, soft bedrock is exposed (NHC, 1996).

4.2.2 Floodplain Characteristics

The floodplain of the Smoky River within the study area is primarily cultivated, with dense forested areas along some portions of the channel banks.

4.2.3 Anthropogenic Features

The Hamlet of Watino is located within the study area. Two bridges cross the Smoky River: one is Highway 49 bridge, and the other is an abandoned railway bridge. Details of these are provided in Appendix B.

4.3 Model Construction

This section describes NHC's process for developing the hydraulic model and includes details on calibration, model parameter selections, model results, and model sensitivity analysis.

4.3.1 General Methodology

The U.S. Army Corps of Engineer's Hydrologic Engineering Center River Analysis System (HEC-RAS) computer program (Version 6.5, 2024) was used to perform hydraulic modelling for this study. HEC-RAS can be used to perform one-dimensional (1D), two-dimensional (2D), or combined 1D and 2D hydraulic calculations for a network of channels with or without hydraulic structures. For this study, a 1D model was constructed to compute water surface profiles for steady-state flows.

The basic inputs required by HEC-RAS are cross sections spaced throughout the study reach that represent the geometry of the river channel and floodplain, as well as roughness coefficients for the channel and overbank areas at each cross section, and the upstream and downstream boundary conditions.

The computational procedure for 1D steady flow calculations is based on the solution of the energy equation, which calculates energy losses due to friction and expansion/contraction between cross sections. The analytical approach employed in HEC-RAS is associated with the following assumptions and potential limitations:

- Flow is gradually varied and boundary friction losses between cross sections are estimated by Manning's equation using section-average parameters.
- The geometry is assumed to be fixed; therefore, changes in the channel and floodplain geometry (e.g. erosion or scour) that may occur during a flood are not accounted for.
- Each model cross section is apportioned into three separate conveyance components representing the main channel, left overbank, and right overbank; and the water level is assumed to be constant across all three conveyance components.
- Since the modelled flow is 1D, only the velocity component in the principal direction of flow is accounted for in the model.

Supplementary 2D Modelling

A supplementary 2D model was developed for the Smoky River study reach based on the DTM. The model was preliminary in nature and was used to guide the initial 1D model construction, such as defining overbank cross section alignments, overbank flow paths, and ineffective flow areas.

1D Geometric Layout

NHC employed the following approach to develop key components of the model's geometric layout:

- Defined the channel centreline along the middle of the main channel and digitized it using ArcGIS tools and by visually referencing the DTM, hillshade, and aerial imagery.
    - Created a single continuous centreline to represent the Smoky River modelled reach.
    - Extended the study reach from just above the upstream study limit to 2 km downstream of the study limit; the length of the total modelled reach is approximately 18 km.
- Digitized the model cross section transects at each surveyed cross section as follows:
    - Digitized the main channel section over the surveyed channel and bank point data.
    - Extended the main channel portion left and right across the floodplain (overbank areas).

  - Aligned the overbank portions perpendicular to the anticipated path of the floodplain flows based on preliminary 2D modelling results and projected them far enough to extend beyond the 1000-year flood inundation extents.
  - Projected cross section elevation values from the survey point data onto the cross section lines using the HEC-GeoRAS toolset through a conflation process.
  - Determined elevations in the overbank areas by extracting elevation values from the underlying DTM along the cross section polylines.
- Determined the left and right banks (referred to as bank stations) by examining the geometry of cross sections and analyzing the DTM, aerial imagery, and survey data. These bank stations were strategically positioned to delineate the boundaries of the modelled left overbank, main channel, and modelled right overbank sections of the cross sections.
- Created flow paths coincident with the river centerline and along the left and right floodplains, representing the length of the main channel, left overbank, and right overbank flow paths.
  - Measured distances between cross sections along flow path lines. The model requires these distances for calculating energy losses between cross sections within the main channel and the left and right overbank areas.

Channel and Overbank Roughness

Manning’s roughness values were used to simulate roughness in the modelled reaches. A single roughness value was assigned to the channel in each cross section, while multiple roughness values were applied to the overbank areas to account for variations in land use across the floodplain. Manning’s roughness is an empirical coefficient used to account for energy losses due to a combination of factors including surface roughness and channel sinuosity. The Manning’s roughness values adopted for the present study are discussed further in Section 4.3.4.

Expansion and Contraction Coefficients

HEC-RAS applies a coefficient to the absolute difference in velocity head to account for flow contraction or expansion losses in the energy balance between successive cross sections. This coefficient can range from 0.10 for gradual transitions to 0.80 for abrupt transitions (Brunner, 2016).

Boundary Conditions

Boundary conditions are required at the inflow (upstream) and outflow (downstream) boundaries of the model. The inflow boundary condition for this model is the discharge. The outflow boundary condition can be a water level or a friction slope with which the water level will be calculated by HEC-RAS assuming a normal depth condition. NHC used a normal depth friction slope as the downstream boundary condition for this study.

Ineffective Flow Areas

Ineffective flow areas can be specified within portions of cross sections where water will pond but there is no appreciable flow. One common example of using an ineffective flow area is in cross sections upstream and downstream of a bridge or culvert where flow is obstructed by elevated road embankments. In HEC-RAS, ineffective flow areas can be defined as either a permanent or non-permanent type. Permanent ineffective flow areas stay ineffective regardless of the WSE, whereas temporary ineffective flow areas become effective when the WSE exceeds a defined elevation. NHC selected the configuration of ineffective flow areas in the model based on site-specific circumstances and engineering judgement.

Levees

HEC-RAS enables users to define levees to confine flow within a specified portion of the cross section by specifying left or right levee stations along with their corresponding elevations at a cross section. When levees are set, water cannot flow beyond either the left or the right levee station until the water level exceeds the designated levee elevation. NHC used natural levees in the Watino model development to restrict overland flows behind high grounds.

4.3.2 Geometric Database

The geometric database of the HEC-RAS model geometry primarily includes model cross sections and hydraulic structures. Each component is described below. Additional information and data are provided as part of the electronic deliverables of the study.

Cross Section Data

NHC created and used a total of 37 cross sections to construct the model for this study, taking the following steps to generate the cross section data:

- Established cross section alignments within the channel by generally following the alignments of the cross-section survey (Section 2.4).
    - Aligned the overbank portions perpendicular to the anticipated flow direction based on preliminary 2D models.
    - Extended the cross section alignments beyond the anticipated 1,000-year flood inundation extents.
- Created two separate station elevation datasets for each cross section.
    - Created the first data set by projecting surveyed data points perpendicularly onto the channel portion of the cross-section line.
    - Created the second data set by extracting elevation values from the DTM along the cross section lines excluding the channel portion covered by the survey data.

- Combined the two station-elevation datasets. Reduced the number of elevation points for each overbank cross section using the minimize-area-change point filter option in HEC-RAS, so that the total number of the points is within the HEC-RAS limit of 500 points.
- Assigned the bank stations.
- Determined distances between consecutive cross sections within the HEC-RAS model following the channel centerline and central flow paths for the left and right overbank areas.

Hydraulic Structures

The modelled reach includes two bridge crossings (Highway 49 bridge and abandoned railway bridge). Each bridge structure's alignment and location were established in ArcGIS. Bridge cross sections included approach roadways and abutments in the left and right overbanks, bridge piers, and bridge deck high and low chord profiles. Approach roadway profiles were extracted based on DTM elevation data supplemented with data from bridge drawings. Abutment geometry, piers, and high and low chords were determined from surveyed data and drawings. Model bridge geometry was checked against design drawings, available TEC bridge file records, and other information as available.

Key hydraulic structure design information incorporated into the model can be found in Table 4.3 below. Any culverts in the study area that service local drainage only or were not relevant to the hydraulic model computations were not modelled.

Table 4.3 Description of bridges included in the hydraulic model

Name	River Station (m)	Design Drawing Info	Span (m)	Width (m)	No of Piers	Pier Width (m)	Skew (°)	Minimum Elevation (m)	
								High Chord	Low Chord
Highway 49 Bridge	9,661	Yes	307	12.0	4	1.5	None	388.20	385.40
Abandoned Railway Bridge	7,651	Yes	N/A	3.7	7	2.7-3.2	None	385.60	381.35

For both low and high flow computation in the bridges, the model was configured to use the energy method.

4.3.3 Model Calibration

This section discusses the general model calibration methodology, high flow calibration, low flow validation, and validation against the WSC rating curve.

Methodology

Calibration Parameters

Model calibration involves the selection and adjustment of model parameters such that calculated flood levels agree well with observed flood levels. Calibration parameters could include:

- Manning's roughness coefficient for the channel and floodplain
- friction slope associated with the downstream normal depth boundary condition
- ineffective flow areas
- expansion and contraction coefficients

NHC selected Manning's roughness as the primary calibration parameter for this study; however, other model parameters from the list were also adjusted within plausible limits to achieve reasonable results.

Roughness Calibration Challenges and Limitations

Roughness calibration is associated with the following challenges and limitations:

- accuracy of HWM elevations
- improper identification of HWMs
- uncertainties in estimates of instantaneous flood peak discharge

Channel Manning's Roughness

NHC's general calibration approach involved assigning Manning's roughness values to ensure computed water levels closely matched observed levels during the selected high flow calibration event. These assignments were made on a reach-averaged basis through visual comparison of computed and observed water levels. A single calibrated model was developed to represent both high and low flow conditions. The calibrated model was validated against the published rating curves for WSC Station 07GJ001.

Overbank / Floodplain Roughness

NHC characterized roughness in the floodplain areas (model overbanks) using land cover type determined by ground observations, collected aerial drone videos, and visual inspection of aerial imagery. These overbank and floodplain roughness values were not adjusted during the model calibration process to simplify the calibration, also recognizing that simulated water surface elevations are not sensitive to variations in overbank roughness values (see Section 4.3.6).

High Flow Calibration

NHC selected the July 2020 flood event as the primary calibration event because it is the most recent significant flood event, the fourth largest on record, and had HWMs collected throughout the study reach. Additionally, the model geometry used in this study reflects the channel and floodplain conditions during the 2020 flood, and the HWMs collected are tied to the NHC survey. While other flood events (June 1972, July 1982, and June 1990) were used to compare computed and observed water levels, they were not considered for selecting Manning's roughness because the HWMs from those events are affected by the replacement of the highway bridge and subsequent scouring. The benchmarks used to collect those HWMs are also unknown.

Among these HWM events, the main selected calibration event of July 2020 has a reported instantaneous peak discharge of 7,950 m^3/s, which is just above the estimated 35-year discharge. The June 1972 and July 1982 flood had a reported peak instantaneous discharge of 9,200 and 9,020 m^3/s, respectively, which exceed the 50-year flood discharge but fall short of the 75-year discharge. The June 1990 flood is the largest recorded flood within the study reach and had a peak instantaneous discharge of 9,400 m^3/s, this value is just below the estimated 75-year discharge.

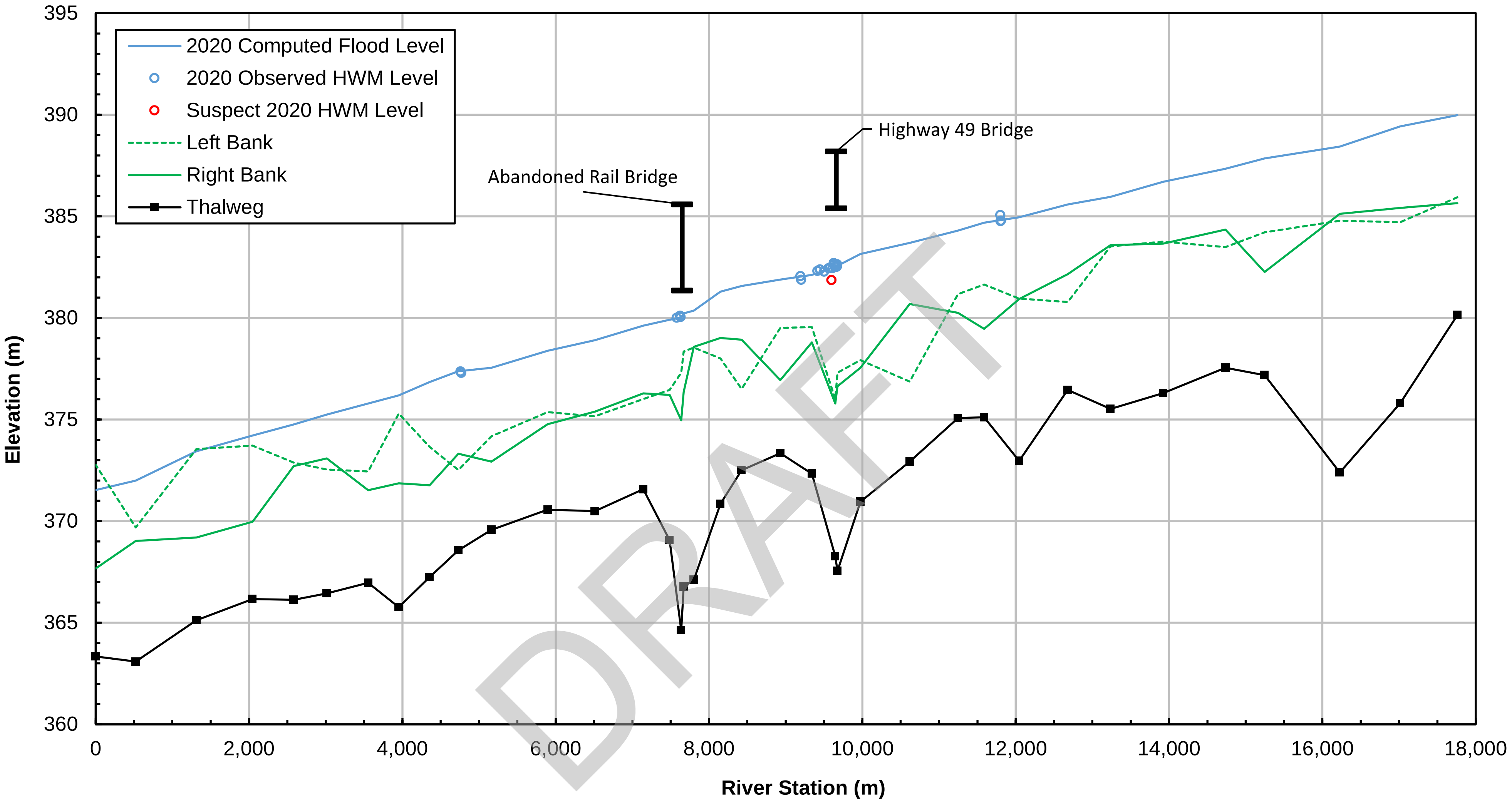
The calibration results are summarized in Table 4.4 by comparing the computed WSEs with the observed HWM elevations. Figure 4.3 shows a comparison between the computed water surface profile and the observed HWM elevations for the main calibration event (July 2020 flood), while Figure 4.4 shows comparisons for other floods (June 1972, July 1982, and June 1990). Excluding one HWM location NHC deemed to be *suspect*, computed water levels were on average 0.03 m below the observed 2020 flood HWM elevations. For the 2020 flood event, the average absolute difference between computed and observed HWM elevations was 0.07 m; the largest positive difference was 0.17 m, and largest negative difference was -0.25 m. For the other flood events (June 1972, July 1982, and June 1990) the computed water levels were significantly lower than the observed HWM elevations.

Table 4.4 High flow calibration results for Smoky River at Watino

HWM ID	River Station (m)	Observed HWM Elevation (m)	Computed WS Elevation (m)	Computed minus Observed (m)	Remarks
2020 Flood Event (Main Calibration Event)					
HWM3 - Watino 1	11,810	384.78	384.82	0.04	
HWM2 - Watino 1	11,803	384.76	384.82	0.05	
HWM1 - Watino 1	11,801	385.06	384.82	-0.25	
HWM3 - Watino 5	9,674	382.64	382.57	-0.07	
HWM2 - Watino 5	9,671	382.57	382.56	-0.01	
HWM1 - Watino 5	9,669	382.52	382.56	0.04	
HWM8 - Watino 2	9,648	382.52	382.52	0.01	
HWM9 - Watino 2	9,634	382.63	382.51	-0.12	
HWM10 - Watino 2	9,628	382.70	382.50	-0.21	
HWM7 - Watino 2	9,612	382.45	382.48	0.03	
HWM6 - Watino 2	9,599	381.87	382.46	0.59	Suspect
HWM5 - Watino 2	9,577	382.45	382.43	-0.02	
HWM4 - Watino 2	9,560	382.45	382.41	-0.04	
HWM3 - Watino 2	9,503	382.28	382.33	0.06	
HWM1 - Watino 2	9,448	382.38	382.26	-0.12	
HWM2 - Watino 2	9,414	382.31	382.22	-0.10	
HWM12 - Watino 2	9,202	381.88	382.04	0.17	
HWM11 - Watino 2	9,192	382.06	382.04	-0.02	
HWM1 - Watino 4	7,630	380.04	380.01	-0.03	
HWM2 - Watino 4	7,624	380.11	380.01	-0.10	
HWM3 - Watino 4	7,579	380.01	379.98	-0.03	
HWM1 - Watino-3	4,768	377.30	377.39	0.10	
HWM2 - Watino-3	4,760	377.36	377.39	0.03	
1972 Flood Event					
N/A	9,664	384.11	383.20	-0.91	
1982 Flood Event					
N/A	9,659	383.74	383.10	-0.64	
1990 Flood Event					
N/A	9,654	384.11	383.28	-0.83	
90-SM-15A	9,657	384.13	383.29	-0.84	
90-SM-16	7,722	382.54	381.04	-1.50	

Notes:

1. “suspect” denotes HWM observations that are likely to be in error in elevation or location.



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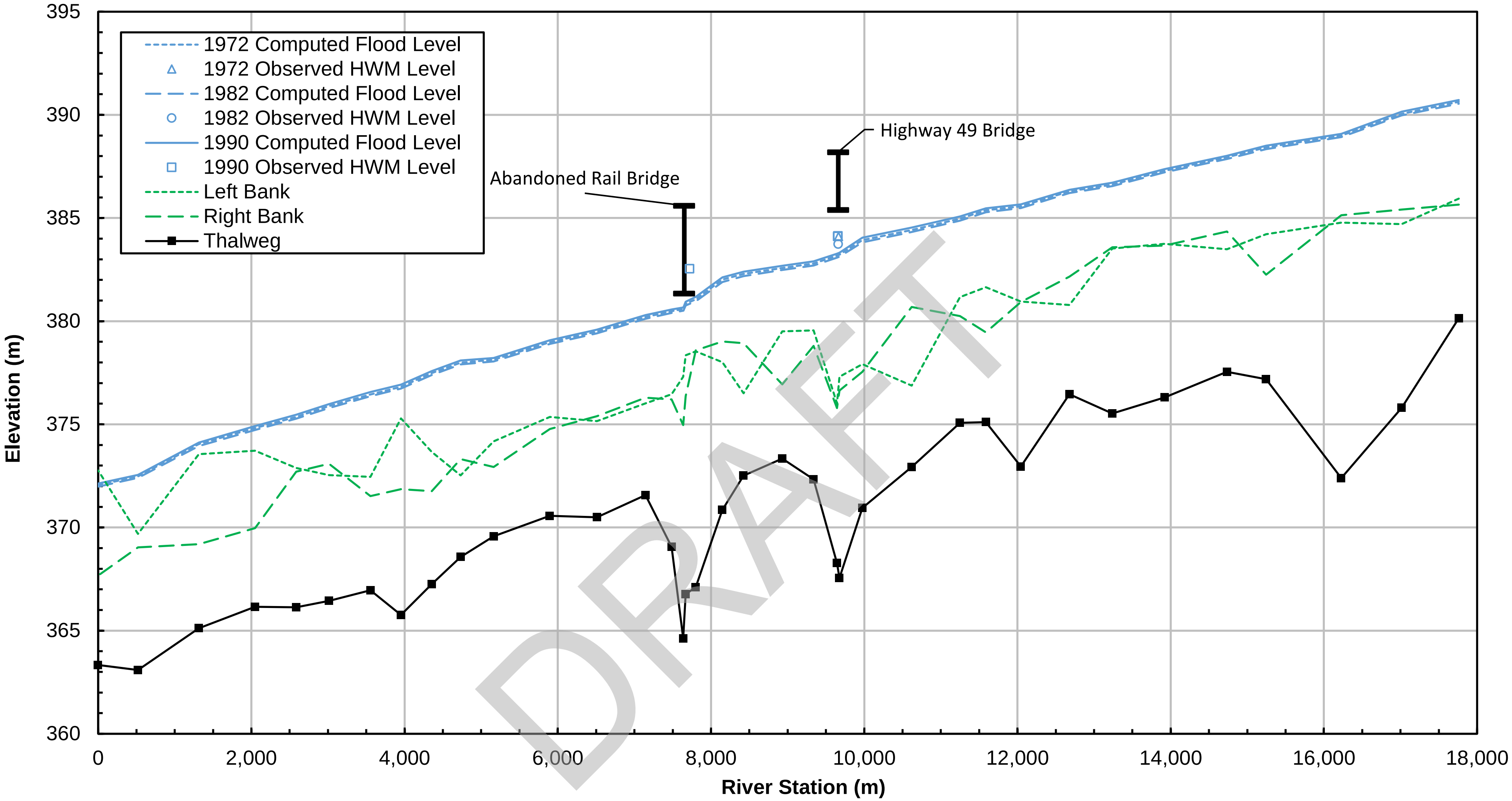
Date: 19-Mar-2025

WATINO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – JUNE 2020 FLOOD

FIGURE 4.3



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WATINO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – OTHER FLOODS (1972, 1982, 1990)

FIGURE 4.4

Low Flow Validation

Low flow validation is an important step, as it demonstrates how the calibrated model, based on a specific HWM event, performs under different scenarios, such as smaller floods. For low flow validation, NHC used surveyed water levels from their 2023 survey program (as part of this study) and additional surveyed water levels from 8 July 2020 (collected during the 2020 HWM survey by EPA).

As previously discussed, NHC used the high flow calibrated model to compute water levels for low flow conditions during the 2023 cross sections survey (30 May -1 June). This comparison assessed whether the Manning's roughness values calibrated for high flows were suitable for simulating low flows as well. The discharges corresponding to the observed water levels during the 2023 survey were obtained from the published daily discharges at Smoky River at Watino gauge (WSC Station 07GJ001), which ranged from 614 m^3/s to 775 m^3/s, significantly below the 2-year flood peak.

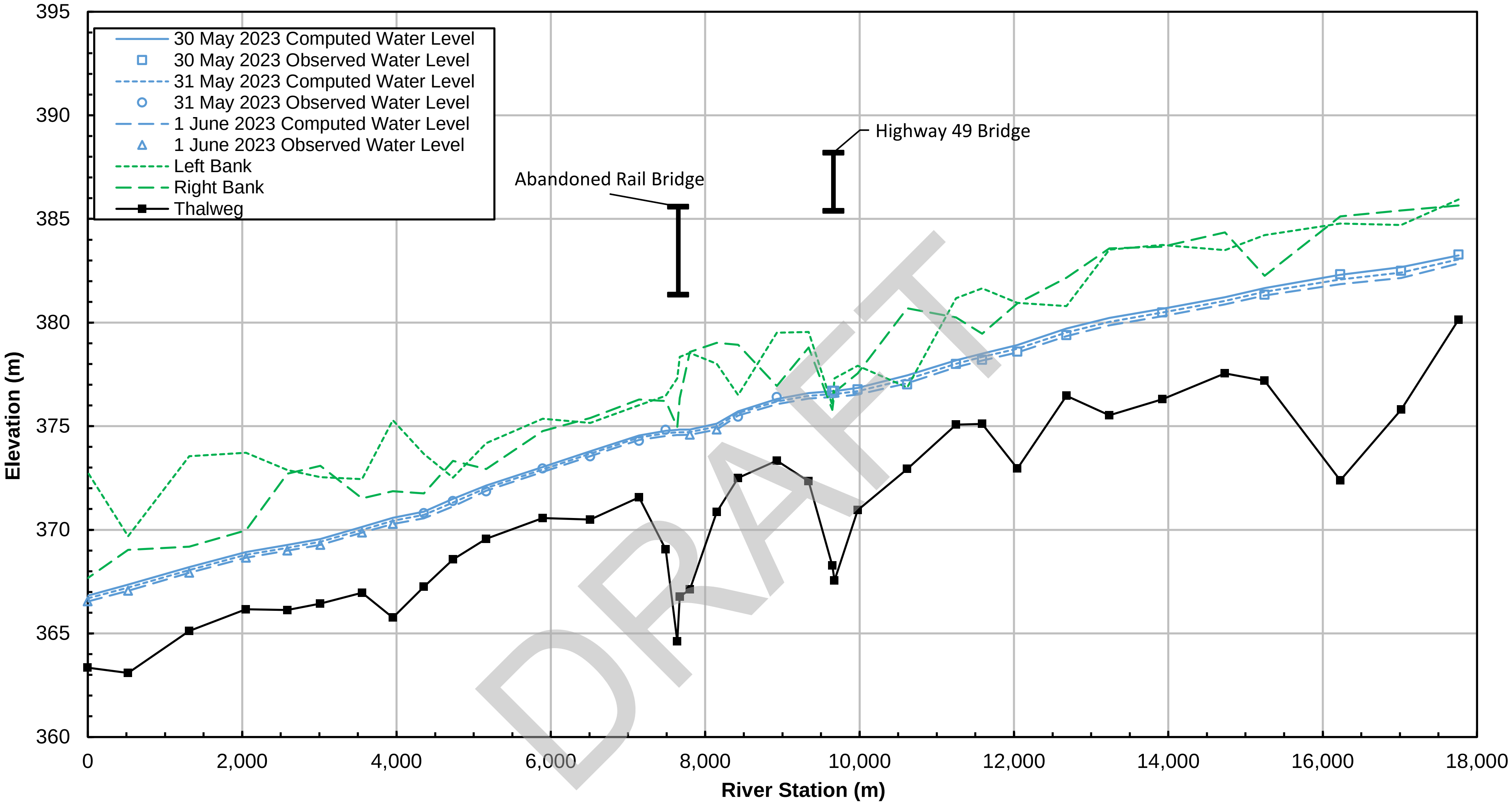
The calibrated model was also used to compute water levels along the reach for conditions observed on 8 July 2020. These water levels were surveyed by EPA when they conducted a HWM survey for the 2020 flood, which peaked on 3 July 2020. The model for this low flow validation used the WSC discharge reported for the day of the survey (1,920 m^3/s), which was on the recession limb of the 2020 flood hydrograph, just below the estimated 2-year flood peak.

The validation results are shown in Table 4.5 by comparing the computed WSEs with the observed water levels. Figure 4.5 provides a comparison of the computed water surface profile with the observed water levels for NHC's survey period (30 May – 1 June), and Figure 4.6 compares the computed and observed water levels for 8 July 2020. For the 2023 NHC survey period, computed water levels were on average 0.06 m above the observed levels; while, the average absolute difference between computed and observed water levels was 0.16 m; the largest positive difference was 0.44 m, and the largest negative difference was -0.28 m. For the 8 July 2020 survey period, computed water levels were higher than observed levels at all three survey locations, with an average difference of 0.15 m.

Overall, the low flow validation results show good agreement between computed and observed water levels. Generally, models that are calibrated to high flow conditions underpredict water levels when used to simulate low flow conditions. However, the low flow validation results indicate that the computed water levels tend to be higher than the observed low flow levels. This suggests that channel bed roughness has a minimal effect on the modelled water levels for the given flows. Instead, variations in form roughness within the study reach play a more significant role in influencing simulated water levels. A reach-specific form roughness approach appears to be effective across a wide range of flow conditions, from the 2-year to the 1,000-year flood events.

Table 4.5 Low flow validation results for Smoky River at Watino

Survey Date	Discharge (m^3/s)	River Station (m)	Observed WS Elevation (m)	Computed WS Elevation (m)	Computed minus Observed (m)
NHC Survey (30 May 2023 - 01 June 2023)					
30 May 2023	775	17,761	383.28	383.24	-0.04
		17,016	382.50	382.67	0.17
		16,228	382.33	382.31	-0.02
		15,247	381.34	381.66	0.32
		13,921	380.49	380.66	0.17
		12,679	379.39	379.71	0.32
		12,045	378.60	378.91	0.31
		11,588	378.20	378.50	0.30
		11,249	378.00	378.18	0.18
		10,618	377.01	377.45	0.44
		9,974	376.78	376.84	0.06
		9,676	376.71	376.69	-0.02
		9,646	376.71	376.69	-0.02
31 May 2023	693	8,929	376.40	376.19	-0.21
		8,425	375.45	375.63	0.18
01 June 2023	614	8,148	375.11	374.83	-0.28
		7,801	374.78	374.58	-0.20
31 May 2023	693	7,487	374.82	374.66	-0.16
		7,143	374.29	374.45	0.16
		6,510	373.54	373.67	0.14
		5,895	372.96	372.91	-0.05
		5,161	371.86	372.02	0.16
		4,732	371.38	371.32	-0.06
		4,355	370.81	370.72	-0.09
01 June 2023	614	3,951	370.47	370.28	-0.19
		3,554	370.07	369.86	-0.21
		3,013	369.18	369.27	0.09
		2,584	368.93	368.99	0.06
		2,048	368.71	368.65	-0.06
		1,314	367.73	367.92	0.19
		521	367.04	367.06	0.02
		0	366.37	366.54	0.17
08 July 2020 Survey					
08 July 2020	1,920	9,458	377.86	377.99	0.13
		9,583	377.89	378.06	0.18
		9,633	377.94	378.09	0.15



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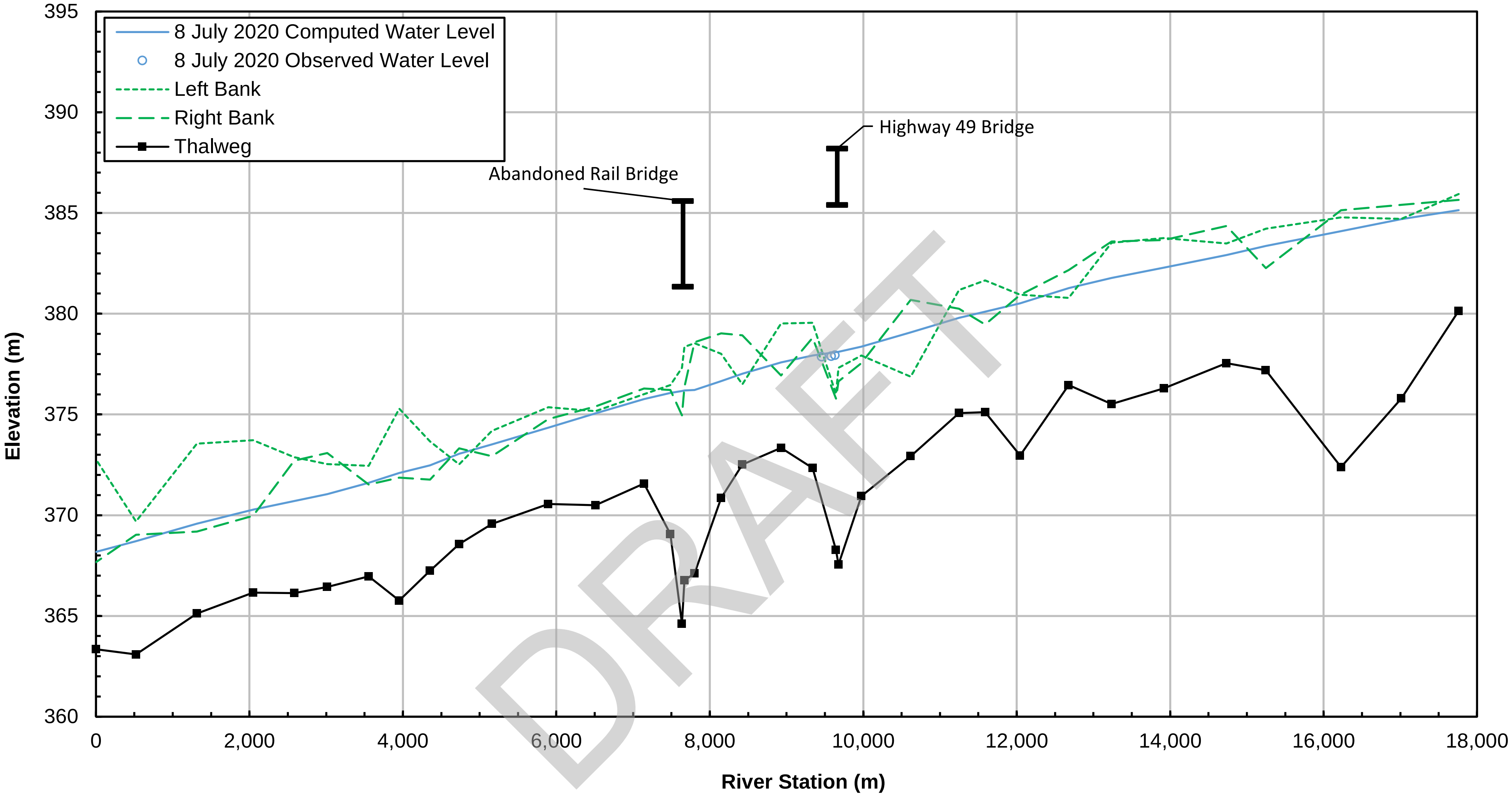
WATINO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – 2023 NHC SURVEYED WATER LEVELS

FIGURE 4.5



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WATINO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – 08 JULY 2020 EPA SURVEYED WATER LEVELS

FIGURE 4.6

WSC Rating Curve Validation

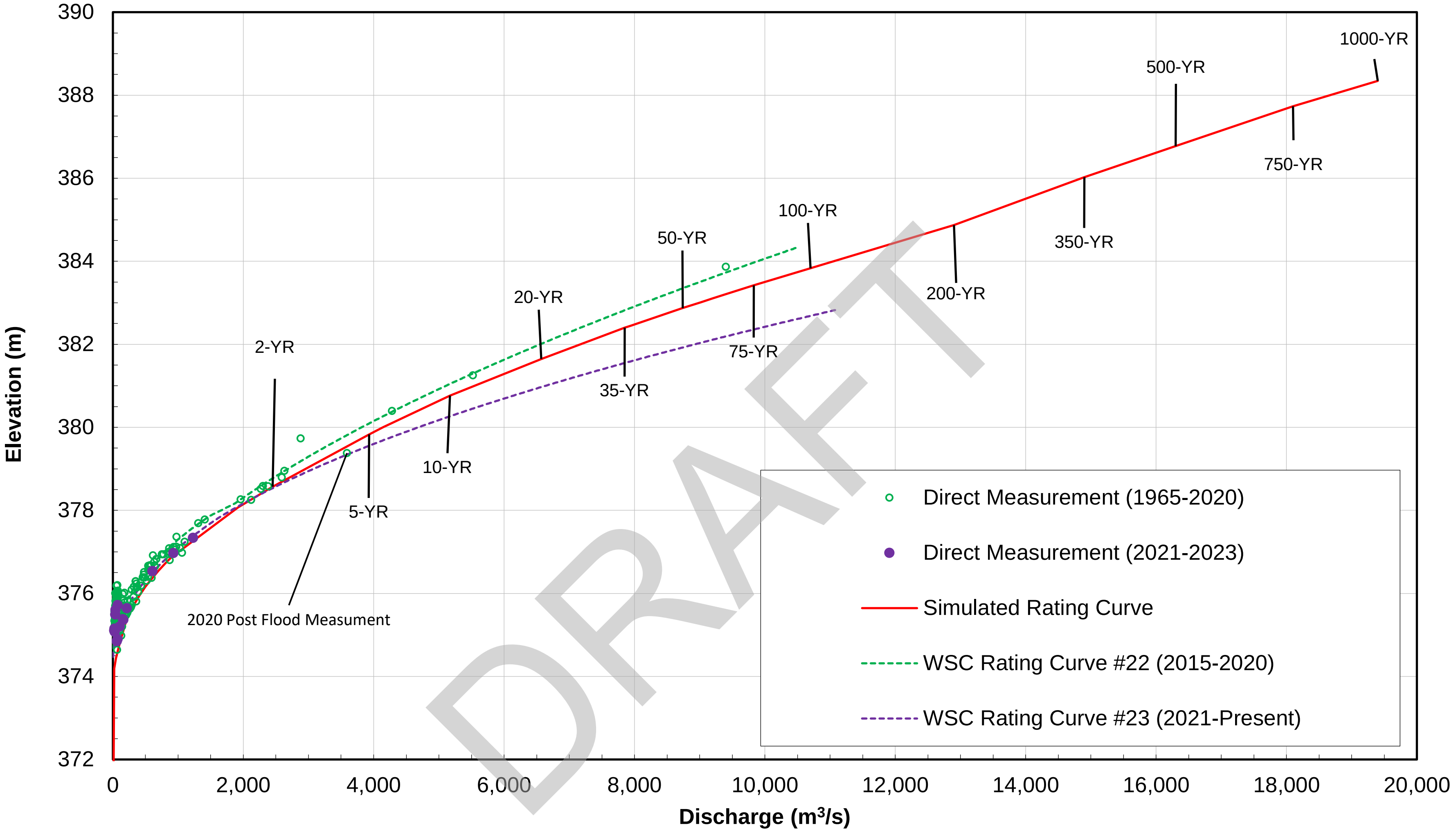
NHC computed a rating curve using the calibrated model and compared it to the published rating curves for the WSC gauge at Smoky River at Watino (WSC Station 07GJ001). As illustrated in Figure 4.7, two WSC published rating curves were used for this comparison: Rating Curve #22 (used by WSC from 2015 to 2020) and Rating Curve #23 (used by WSC from 2021-present). While these are the most recent rating curves, they differ significantly because WSC adjusted the rating curve to account for severe scouring at all gauge heights during the 2019 and 2020 flood events.

To compare the WSC rating curves with the computed rating curve, the WSC rating curve stages were converted to WSEs using a WSC reported conversion factor of 373.737 m at the gauge (WSC Station 07GJ001). Additionally, the WSEs were further adjusted based on the difference between the WSC published benchmark elevation and the NHC surveyed elevation (-0.202 m).

The computed curve aligns well with the most recent Rating Curve #23 for the 2-year flood and lower flows. However, for floods greater than the 5-year return period, the computed rating curve begins to diverge from Rating Curve#23. Note that this curve was established without any flows higher than 1,230 m^3/s, which means the curve beyond that point is extrapolated and may have greater uncertainty.

The computed rating curve falls below Rating Curve #22, with a relatively consistent difference (approximately 0.2 to 0.3 m) between 2-year to 5-year flood level, after which the curves diverge. This behavior is justifiable considering the large scour hole created during the 2019 and 2020 events, as reported by WSC on the provided Rating Curve#23 data.

The computed rating curve closely matches the direct discharge measurement taken by WSC on 5 July 2020 (after the peak of 2020 flood). The difference between the computed rating curve and measured discharge is approximately 0.17 m. On that day, a discharge measurement of 3,950 m^3/s was recorded, which corresponds to approximately a 5-year return period. This agreement adds confidence that the computed rating curve from the calibrated model is reasonable and can be used to simulate a range of discharges from the 2- to the 1,000-year return periods.



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WATINO FLOOD STUDY

COMPARISON BETWEEN COMPUTED AND REPORTED WSC RATING CURVE DATA FOR SMOKY RIVER AT WATINO (WSC STATION 07GJ001)

FIGURE 4.7

4.3.4 Model Parameters and Options

This section provides descriptions of the key parameters and options adopted in the HEC-RAS model. These include Manning's roughness values for channel and overbank areas; contraction and expansion loss coefficients; ineffective areas; and a roadway weir coefficient.

Manning's Roughness Coefficient

Manning's roughness is used to account for an array of energy losses that may vary with respect to discharge. A minimum of three (one channel and two overbank) roughness values were used within each cross section. Where appropriate, roughness was varied horizontally more than three times across the cross section to capture changes in river and floodplain characteristics. Roughness values were kept constant with discharge.

Channel Roughness

NHC calibrated channel roughness along the study reach using high flow events and later verified with low flow events and the rating curve at WSC Station 07GJ001. For the reach upstream of the Highway 49 bridge, a channel roughness value of 0.035 was used, while a roughness value of 0.032 was applied to the reach downstream of the bridge. These roughness values provided the best match between the computed water levels and the 2020 HWM data. In NHC's previous study (1996), channel Manning's roughness values ranged from 0.030 to 0.034 and varied with discharge. The calibrated roughness values in this study are comparable to those used in the previous study.

Overbank Roughness

NHC selected Manning's roughness values for the various overbank characteristics based on values from available literature (Chow 1959; Arcement and Schneider 1989). Overbank roughness values were selected based on land cover and vegetation type. To limit the number of overbank roughness values in each cross section, sometimes composite roughness values based on weighted average of different land use types were used. The selected overbank roughness values for each land cover type are summarized in Table 4.6 below.

Table 4.6 Description of land cover types and selected Manning's roughness values within study area

Land Cover Type	Description	Manning's Roughness
Trees	Dense, willows, straight tree covered areas	0.15
Cultivated Areas	Agricultural crops or pastureland within the overbank with grasses	0.04
Built Area	Development within the wetted width of the design flood with buildings, yards, and transportation corridors	0.07

Land Cover Type	Description	Manning's Roughness
Bare Ground	Cleared ground with no or short grasses	0.028

Expansion and Contraction Coefficients

To account for the effects of flow contraction and expansion losses on the energy balance between successive cross-sections, HEC-RAS uses a coefficient to multiply the absolute difference in velocity head. Initially, the expansion and contraction coefficients were adopted from the previous study and then slightly adjusted to better reflect the channel geometry as visible in the DTM and aerial imagery. Table 4.7 below summarizes the selected expansion and contraction coefficients for different sub-reaches and provides a comparison with the values used in the previous study by NHC (1996). For most of the study reach, the selected coefficients remain the same, except for sub-reaches with bridges, where the coefficients are slightly lower.

Table 4.7 Selected expansion and contraction coefficient values within study reach

River Station (RS)	Sub-reach Description	Expansion and Contraction Coefficients	
		This Study	NHC (1996)
RS 17,761 to RS 11,249	Sub-reach upstream of Highway 49 bridge	0.2 and 0.4	0.2 and 0.4
RS 10,618 to RS 9,646	Highway 49 bridge sub-reach	0.3 and 0.5	0.4 and 0.6
RS 9,340 to RS 7,669	Sub-reach between Highway 49 bridge and abandoned railway bridge	0.5 and 0.7	0.6 and 0.8
RS 7,638 to RS 7,487	Sub-reach immediately downstream of abandoned railway bridge	0.3 and 0.7	0.6 and 0.8
RS 7,143 to RS 0	Sub-reach downstream of abandoned railway bridge	0.2 and 0.4	0.2 and 0.4

Weir Coefficient

HEC-RAS uses a broad crested weir formulation to simulate flow overtopping the hydraulic structures. Typical discharge coefficients range between 1.4 and 1.8, with larger values generating less backwater. Flow overtopping a bridge deck is not ideally represented by a broad crested weir overtopping scenario, and it is generally recommended that lower values be used when an increased resistance to flow is anticipated from obstructions such as bridge railings, curbs, and debris (FHA, 2012). The bridges in the Smoky River at Watino could cause significant resistance to flow, thus a weir coefficient of 1.4 was deemed appropriate for the Highway 49 bridge and the abandoned railway bridge in this study.

Upstream Boundary Condition

A specified discharge is required at the upstream end of each modelled reach. For this study, there is a single modelled reach for the Smoky River, and the estimated flood frequencies, as shown in Table 3.1, were used as the upstream boundary condition.

Downstream Boundary Condition

At the downstream boundary, a normal depth water level approximation was assigned as the boundary condition. The slope used for calculating normal depth was set to 0.00112 m/m representing the estimated energy grade line slope near the downstream boundary. This slope was determined using the measured water surface slope derived from the DTM data.

Ineffective Flow Areas

Ineffective flow areas were specified at cross sections in the HEC-RAS model based on preliminary 2D model simulation results and a review of the local terrain and floodplain features both at and between cross sections. Ineffective flow areas can be specified within portions of cross sections where water is expected to pond, and where the velocity of that water, in the downstream direction, is expected to be close to or equal to zero (Brunner, 2016). The downstream direction is taken relative to the cross section lines defined in the model, so the orientation of cross sections was considered when specifying ineffective flow areas.

Ineffective flow areas in the model may be specified as either permanent or non-permanent. Permanent ineffective flow areas apply regardless of the WSE, whereas non-permanent ineffective flow areas become effective above a defined elevation. The configuration of permanent and non-permanent ineffective flow areas was specified depending on site-specific circumstances and based on engineering judgement.

The general principles for determining ineffective flow areas were as follows:

- Non-permanent ineffective flow areas were used to “fill” local floodplain depressions that are obstructed by higher ground upstream or downstream. These areas were assumed to become effective once the water level exceeded the elevation of the adjacent ground.
- Permanent ineffective flow areas were used to permanently “fill” relic channels, tributary channels or excavated holes that would have otherwise incorrectly added flow area to the cross section.
- Permanent ineffective flow areas were defined where flow patterns would likely be influenced by nearby bridge abutments and roadway embankments crossing the floodplain. These types of obstructions tend to direct flows towards the bridge opening. Several site-specific factors were taken into account when configuring ineffective flow areas at bridges and culverts in the study area, including distance from the cross section

to the bridge, terrain features, bridge geometry, and skew of the bridge opening relative to the river.

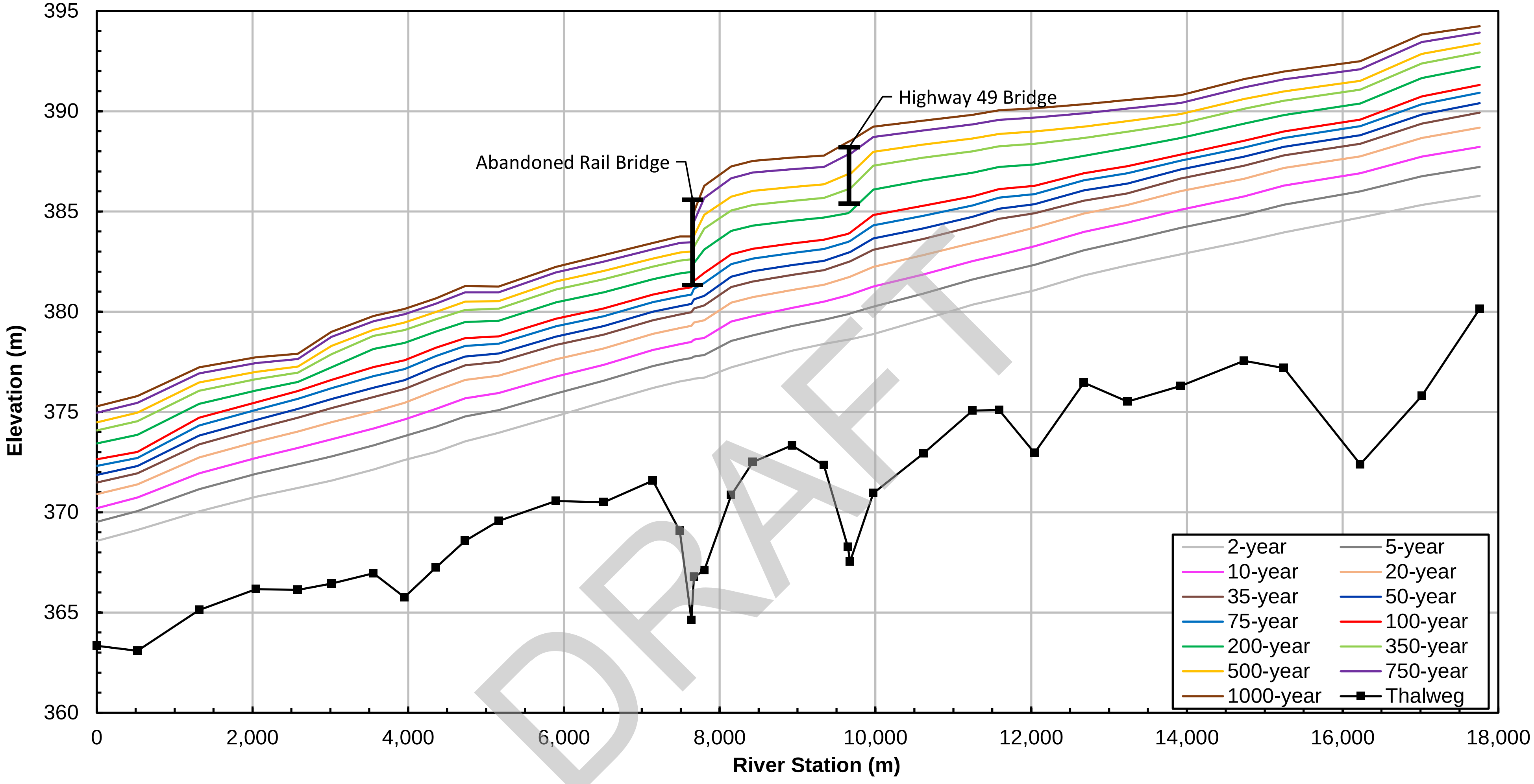
- Ineffective flow areas behind railroad and highway embankments were assessed on an individual basis. Aerial imagery, LiDAR DTM data, and historic information were used to determine if there were indications of flow behind or above embankments. Areas behind and below the height of the embankment were modelled as effective flow only if there was no downstream obstruction or if there was an indication of flow moving in the downstream direction. Otherwise, permanent ineffective flow areas were set to the top of embankment elevation, allowing areas behind embankments (assumed permeable) to be shown as wet and isolated but not conveying flow. Areas above embankments generally conveying flow once the embankment is overtopped, unless an upstream or downstream obstruction causes local ponding or dead zones with limited flow.

Levees

Although there are no berms, dikes, or other constructed levees within the study area, the levee feature was used in the model where high natural ground and roadways were assumed to limit flooding in low lying areas behind them. In HEC-RAS, levees were modelled using levee markers to control flow behavior and prevent water from appearing in adjacent low areas of the cross section unless there is a reasonable pathway for this to occur. This was done in the geometric data editor by selecting the cross section and defining left and right levee stations. It is a general practice to use levees to define natural berm/road embankments which confine water to the channel and overbank until they are overtopped.

4.3.5 Flood Frequency Profiles

NHC used the hydraulic model to generate flood frequency profiles for the 13 open-water floods of varying magnitudes, ranging from 2-year to 1000-year return periods. The computed flood frequency water levels at each cross section on the Smoky River are provided in Appendix E. The results are plotted in Figure 4.8.



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WATINO FLOOD STUDY

COMPUTED FLOOD FREQUENCY WATER LEVELS

FIGURE 4.8

4.3.6 Model Sensitivity

A sensitivity analysis was conducted to assess the impact of key parameters on the hydraulic model results. Variations in boundary conditions (flood frequency estimates and downstream boundary conditions) and Manning’s roughness impact computed water levels, which in turn affect flood depths and inundation extents. The sensitivity of computed water levels was analyzed to estimate the model's error range and determine the relative influence of each parameter. The 100-year flood was used as the baseline for this analysis. However, sensitivity to the roadway weir coefficient at bridges was not evaluated, as the 100-year flood did not overtop either of the two bridges within the study reach.

A summary of the sensitivity analysis results is provided in Table 4.8 below and discussed in the following sections. Detailed results of all the sensitivity analysis are provided in Appendix F.

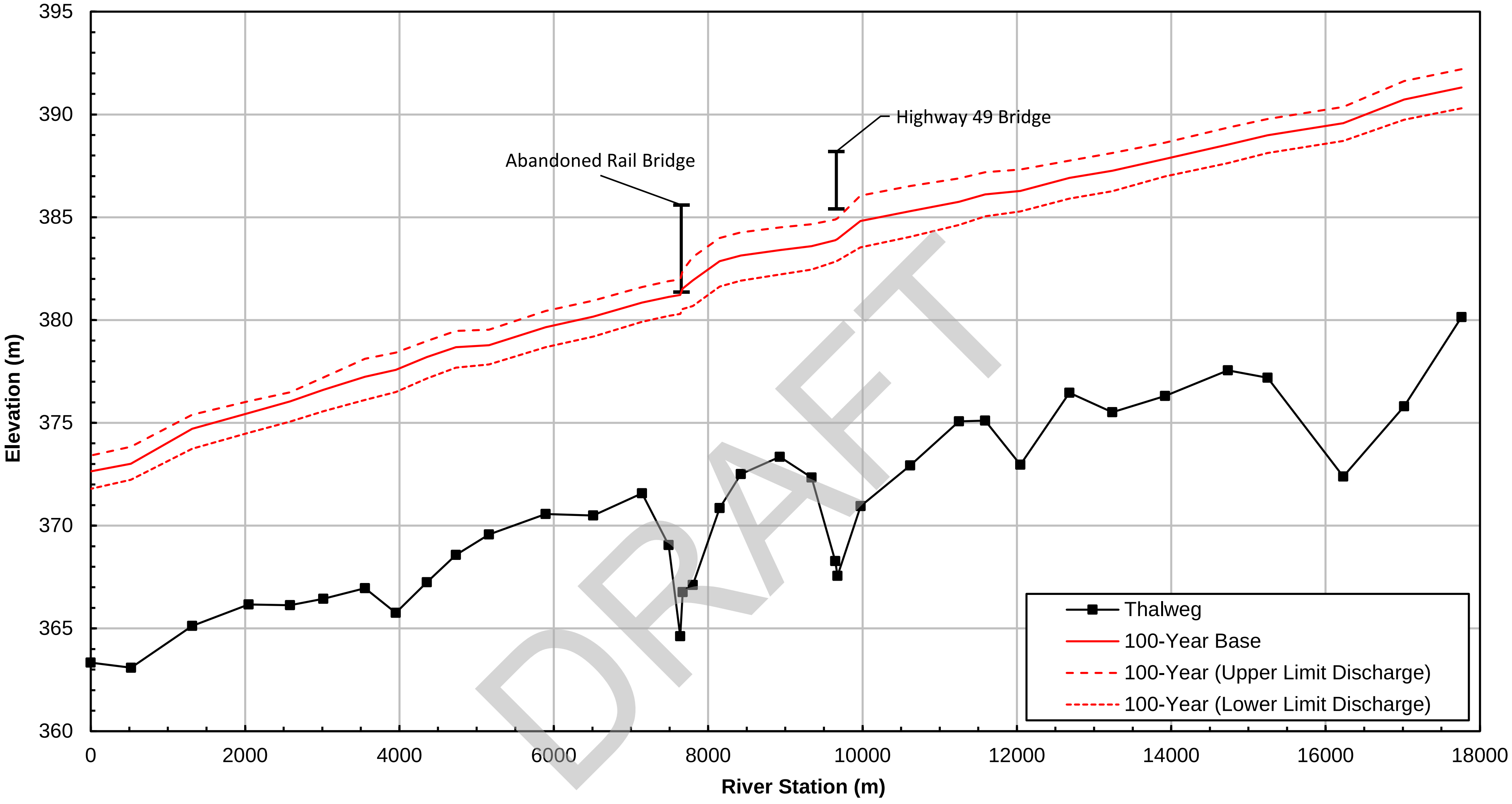
Table 4.8 Summary of sensitivity analysis results

Model Parameter	Difference from 100-year Baseline Profile (m)			
	Lower Limit		Upper Limit	
	Maximum	Average	Maximum	Average
Upstream Boundary Flood Frequency Estimates	-1.29	-1.02	1.23	0.88
Downstream Boundary Normal Depth	-0.30	-0.02	0.30	0.02
Channel Manning’s Roughness Coefficient	-0.48	-0.34	0.43	0.34
Overbank Manning’s Roughness Coefficient	-0.15	-0.06	0.12	0.04

Boundary Conditions

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

The discharges used for the sensitivity analysis were ±20% of the 100-year flood peak (10,700 m^3/s), resulting in an upper limit of 12,840 m^3/s and a lower limit of 8,560 m^3/s. This range of discharges (8,560 to 12,840 m^3/s) was chosen instead of the 95% confidence limit discharges because the upper 95% limit of the 100-year peak discharge (16,800 m^3/s) even exceeds the 500-year flood frequency estimate. This was considered unrealistic and beyond the plausible range for a meaningful sensitivity analysis of the 100-year flood. The selected lower limit for the sensitivity analysis (8,560 m^3/s) is comparable to the lower 95% confidence limit (8,450 m^3/s). NHC compared the computed water levels for these discharges with the baseline model results in Figure 4.9 (Table 4.8) and concluded that the lower limit of 100-year discharges resulted in the computed 100-year flood levels being 1.02 m lower on average, and the upper limit of 100-year discharges resulted in the flood levels being 0.88 m higher on average.



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WATINO FLOOD STUDY

SMOKY RIVER SENSITIVITY ANALYSIS PROFILE – UPSTREAM BOUNDARY DISCHARGE

FIGURE 4.9

The adopted downstream boundary condition for this study was based on a normal depth approximation, with the downstream boundary water level calculated by the Manning's equation with a specified energy slope equal to 0.00112 m/m. The 100-year flood level calculated at the downstream boundary was 372.64 m. The sensitivity to this parameter was assesses by adopting ±0.30 m as a plausible range of uncertainty; results are presented in Figure 4.10.

Departures of the computed WSEs from the baseline condition steadily decrease to below 0.1 m approximately 1,314 m upstream of the model boundary, near cross-section XS-03. Beyond this point, computed WSEs are not significantly affected by downstream boundary changes. Extending the boundary beyond the study area (XS-02 and XS-01) confirms its negligible influence within the study area (XS-03 and upstream).

Manning's Roughness

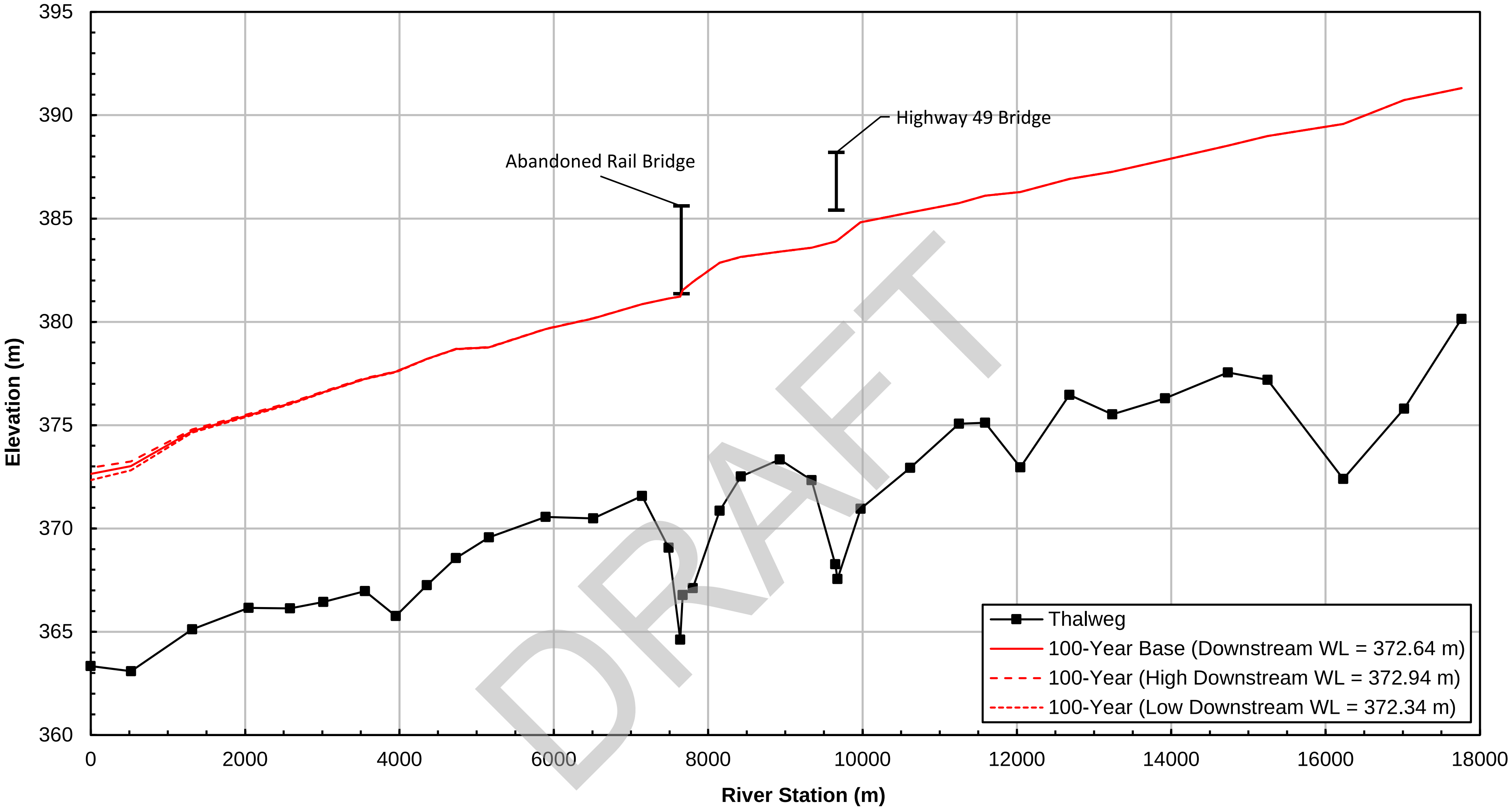
The sensitivity of the model to channel and overbank Manning's roughness was evaluated independently.

The sensitivity of 100-year flood levels to channel roughness was assessed by varying roughness coefficients ±10%, with base values of 0.035 upstream and 0.032 downstream of the Highway 49 Bridge at Smoky River.

The sensitivity results show an average deviation from the 100-year profile of 0.34 m for both low and high channel roughness values; maximum deviations reach 0.48 m and 0.43 m respectively. The sensitivity results are illustrated in Figure 4.11 and summarized in Table 4.8.

The sensitivity of computed 100-year flood levels to overbank roughness variations was evaluated by selecting low and high roughness coefficients for the modelled river reach. The range of varying overbank roughness coefficients were selected considering seasonal variations in vegetation growth and density. For the low and high roughness sensitivity runs, the overbank roughness values were adjusted by ± 20% from the base case to reflect this range.

The average changes in the simulated 100-year water levels resulting from variations in the overbank roughness values are 0.06 m (maximum 0.15 m) for the low overbank roughness values and 0.04 m (maximum 0.12 m) for the high overbank roughness values. The sensitivity results are illustrated in Figure 4.12 and summarized in Table 4.8.



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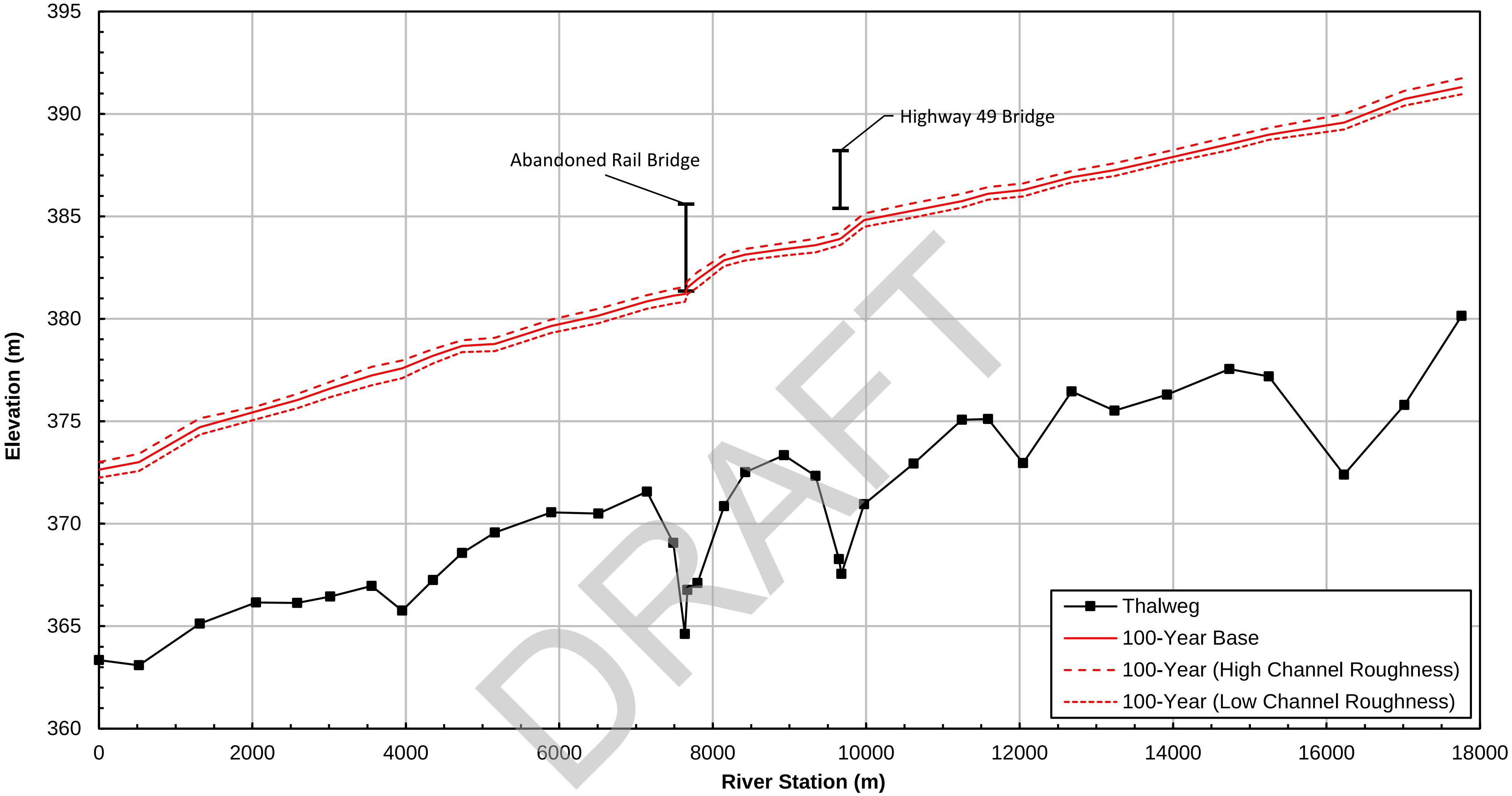
WATINO FLOOD STUDY

SMOKY RIVER SENSITIVITY ANALYSIS PROFILE – DOWNSTREAM BOUNDARY WATER LEVEL

FIGURE 4.10



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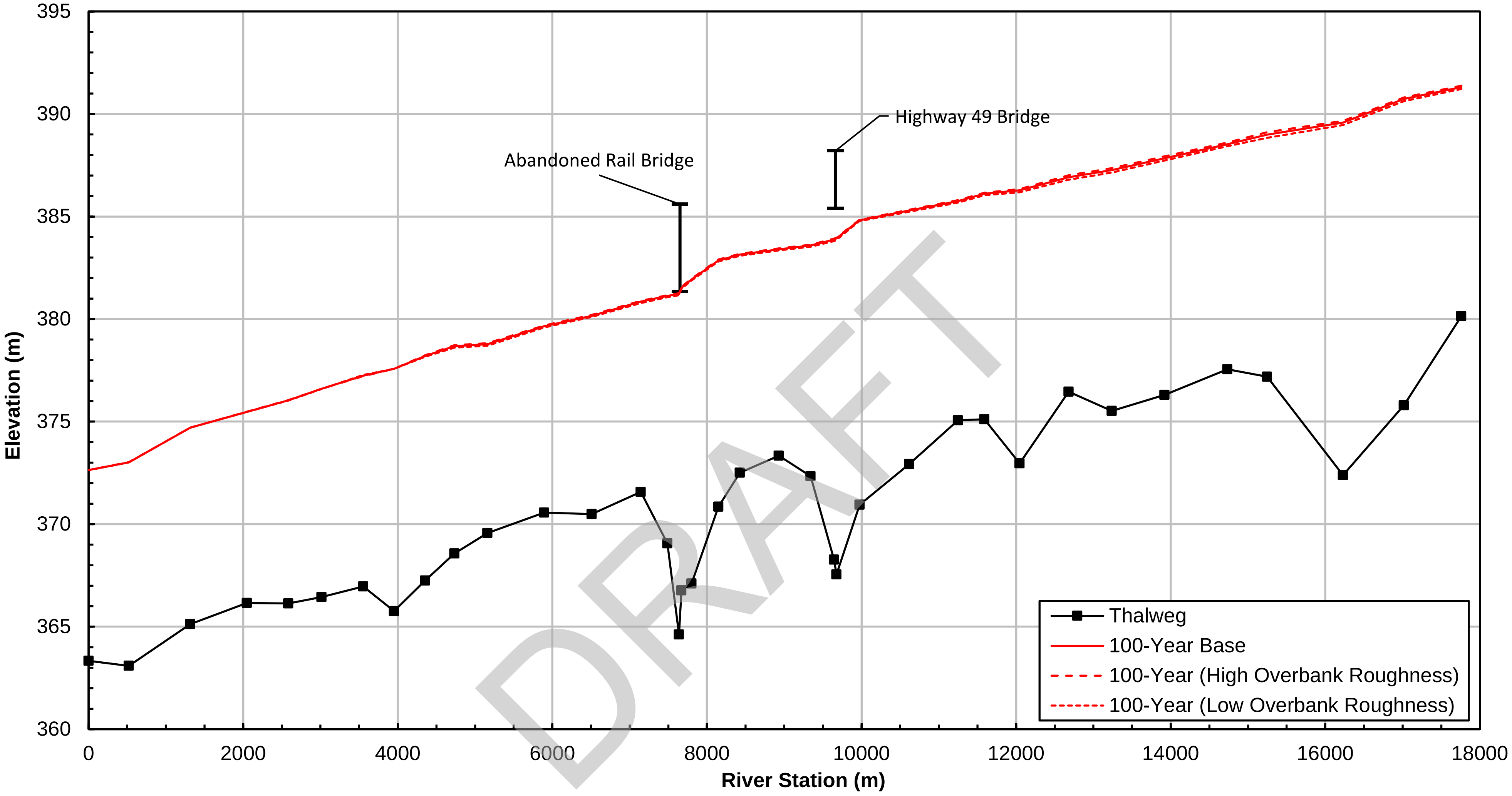
WATINO FLOOD STUDY

SMOKY RIVER SENSITIVITY ANALYSIS PROFILE – CHANNEL ROUGHNESS VALUE

FIGURE 4.11



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WATINO FLOOD STUDY

SMOKY RIVER SENSITIVITY ANALYSIS PROFILE – OVERBANK ROUGHNESS VALUE

FIGURE 4.12

5 FLOOD INUNDATION MAPS

This section presents the methods and results of the flood inundation mapping, including the individual flood inundation map series created for each of the 13 flood frequency return periods from the 2-year through 1000-year scenarios. The open water inundation map series is provided in Appendix G. Flood inundation mapping shows areas of ground that could be covered by water under one or more flood scenarios for existing conditions.

5.1 Methodology

NHC used the following procedures to create the flood inundation maps:

- Created a WSE triangular irregular network (TIN) representing a contiguous flood level profile along the modelled Smoky River reach.
- Generated a WSE grid with the same grid geometry as the underlying DTM. Assigned elevation values to each grid cell based on the corresponding value taken from the WSE TIN.
- Generated a depth grid with the same grid geometry as the WSE grid by subtracting DTM elevations from the corresponding WSE grid values. Negative depth values represent dry cells and were assigned a value of *NoData*.
- Generated inundation polygons based on the depth grids by converting depths greater than 0 m into inundation polygons.
- Further processed inundation polygons by smoothing and filtering out wetted areas that were not directly inundated (or isolated) and removed very small dry areas (or holes).
- Mapped wetted areas that were not directly connected by overland flows as directly inundated if there was evidence of subsurface hydraulic connectivity through structures such as culverts. NHC reviewed aerial imagery, DTM, hillshade, and other available information to assume hydraulic connectivity.
- Used these inundation polygons to clip the WSE grids and depth grids to the inundation extent. Produced the WSE TINs, WSE grids, depth grids, and inundation extent polygons in standard Esri file formats using ArcGIS tool sets.

5.2 Water Surface Elevation TIN Modifications

NHC modified the WSE TIN in areas needing manual edits (e.g., overbank flooding or backwater areas) to regenerate inundation polygons from the data using the procedures described above.

Areas showing extensive overbank or backwater flooding directly connected to the channel at one distinct location (the overtopping point) were adjusted such that the WSE across that area was set equal to the WSE at the overtopping point. This modification generally reduced the size of the inundated area upstream of an overtopping point and increased the size of the inundated area downstream of the overtopping point. When the size of the inundated area expanded downstream, NHC verified whether the expanded area could potentially reconnect to the channel, ensuring a realistic connection in such instances.

The adjusted TIN surfaces were then used to generate the adjusted flood depth grids and flood inundation polygons so that each product incorporates the adjustments.

5.3 Flood Inundation Area

The impacts of flooding on developed areas and infrastructure are evident in the flood inundation maps (Appendix G).

5.3.1 Direct Flood Inundation Areas

Table 5.1 provides a list of notable areas that would be impacted by floods and includes a range of flood return periods or frequencies that could impact residences, buildings, and other notable facilities. The table lists areas from upstream to downstream, with reference river stations to assist in cross-referencing with the inundation mapping libraries. The shaded boxes provide a graphical display of the approximate range of flood frequency magnitudes impacting each area. For all flood inundation areas, please refer to Appendix G.

Study results show that the Hamlet of Watino, including residences and buildings along Railway Avenue will likely be impacted as early as a 200-year flood, as well as several other buildings and residences in the study reach (see Table 5.1 and Appendix G).

5.3.2 Hydraulic and Flood Control Structures

Two crossings are situated along the Smoky River at the Watino study reach: one is active, and the other is abandoned. Study results show that the Highway 49 bridge would be affected during a 350-year flood, with floodwaters reaching the bridge’s lower chord and overtop the approach road along the right bank. The bridge itself would be overtopped during a 1000-year. At present, the abandoned railway bridge has only one remaining span standing in the river; this portion of the deck would not be overtopped, even during a 1000-year flood.

There were no known flood control structures within the study reach.

Table 5.1 List of notable areas impacted by the range of flood magnitudes

Reach	Cross-Section Reference	Impacted Areas along the Floodplain for Different Flood Frequency Events												
		2-yr	5-yr	10-yr	20-yr	35-yr	50-yr	75-yr	100-yr	200-yr	350-yr	500-yr	750-yr	1000-yr
Smoky River	XS-35 (RS 16,228)	Residence near Range Road 244 and Township Road 773A (right bank)												
											350	500	750	1000
	XS-22 (RS 9,340)	Multiple residences along Railway Avenue at Watino (left bank)												
										200	350	500	750	1000
	XS-21 (RS 8,929)	Buildings along Railway Avenue at Watino (left bank)												
										200	350	500	750	1000
	XS-16 (RS 7,638)	Building northwest of Railway Avenue (left bank)												
														1000
	XS-10 (RS 4,732)	Building west of Township Road 780A (left bank)												
													750	1000

6 FLOODWAY DETERMINATION

Flood hazard identification involves the delineation of floodway and flood fringe zones for a specified design flood. A description of key terms from the FHIP Flood Study Technical Guidelines (AEP, 2022), incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta, is provided below.

6.1 Design Flood Selection

As defined in the guidelines, the design flood standard in Alberta is the 1:100 flood, which is a flood that has a 1% chance of occurring in any given year. The design flood can be based on either open water or ice jam flooding scenarios, depending on which scenario results in more severe flooding. The 1:100 flood has a statistical 100-year return period and is also referred to as the 100-year flood.

The 100-year open water flood was selected as the design flood for the study area. The discharge value for the 100-year open water flood for Smoky River at Watino is 10,700 m^3/s.

6.2 Floodway and Flood Fringe Terminology

Flood Hazard Map

A flood hazard map is a specific type of flood map that identifies the area flooded or at risk of flooding for the 100-year design flood and divides that flood hazard area into floodway and flood fringe zones, and flood fringe sub-zones like the high hazard flood fringe and protected flood fringe. Flood hazard maps can also illustrate additional flood hazard information, including incremental areas at risk for more severe floods than the design flood, including the 200- and 500-year floods.

Floodway

When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard for the 100-year design flood, where flows are deepest, fastest, and most destructive. When a flood hazard map is updated, the floodway may no longer represent the area of highest hazard based on new information.

Flood Fringe

The flood fringe is the area outside of the floodway that is flooded or could be flooded during the 100-year design flood. The flood fringe typically represents areas with shallower, slower, and less destructive flooding, but it may also include high hazard flood fringe areas. Areas at risk of flooding behind flood berms may also be mapped as protected flood fringe areas.

High Hazard Flood Fringe

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

Protected Flood Fringe

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

6.3 Flood Hazard Identification

6.3.1 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 (AEP, 2022), were 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.
- The floodway must always include the main channel of a stream.
- For reaches of supercritical flow, the floodway boundary should typically 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 were identified as high hazard flood fringe. These high hazard flood fringe zones were identified in all areas, whether they are newly-mapped or have an existing floodway.

The final floodway limits were determined in consultation with the EPA project team. The floodway limit stations and limiting criteria for each cross section are tabulated in **Appendix H**. The limits of the floodway (also denoted as the floodway boundary) intersect cross sections at the floodway limit stations. In most instances, the floodway limits were coincident with the inundation limits. This condition typically occurs when there is no viable flood fringe, or the previous floodway was drawn outside the current inundation limit. In some instances, the floodway followed along the 1 m depth contour and 1 m/s velocity contour.

6.3.2 Design Flood Profile

The open water design flood levels presented in **Appendix H** were extracted from the HEC-RAS model for the 100-year flood. Figure 4.8 earlier in the document depict the 100-year flood profile, which is the design flood.

6.3.3 Floodway Criteria Maps

A floodway criteria map documents the technical information that helps define the floodway (AEP, 2022). The information on the maps include:

- Inundation extents for the design flood.
- Areas where the depth of water is 1 m or greater and the corresponding 1 m depth contour.
- Areas where the computed velocity is 1 m/s or faster.
- The proposed floodway limits and, where applicable, the previous floodway boundary.
- Stranded areas of dry ground within the flood hazard area.
- The location and extent of all cross sections used in the HEC-RAS model.
- The extent of main channel at each cross section.

The floodway criteria maps for this study are provided in **Appendix I**.

The mapping exercise began with the computed water surface elevations and flow velocities for the open water design flood. The extent of inundation was then mapped using the general procedure described in Section 5. This procedure included generation of the corresponding water surface elevation (WSE) triangular irregular network (TIN), WSE grid, and flood depth grid.

Polygons representing 1 m or more than 1 m deep areas, and 1 m depth contour lines were derived from the flood depth grid. The depth contours were then filtered and smoothed using the same parameters and procedures as those applied to determine the inundation extents.

Since a one-dimensional computational modelling approach was used for this study, flow velocities were only available at the cross section locations. HEC-RAS can apportion channel and overbank discharge into a maximum of 45 sub-sections at any cross section location. Discharge is apportioned based on the computed water level and a weighted flow area approach. This provides a convenient means to estimate the lateral variation in velocity across a section. For this study, the maximum number of velocity subsections were specified in the overbanks. The velocity values were assigned to the corresponding segments along each cross section. Those segments with velocities of 1 m/s or greater were emphasized on the maps to help visualize where local flow velocities were greater than or equal to 1 m/s.

6.3.4 Flood Hazard Maps

The flood hazard maps depict the floodway and flood fringe, including the high hazard flood fringe areas, for the design flood. The map also shows incremental areas at risk for more severe floods: the 200- and 500-year floods. The flood hazard maps are provided as **Appendix J**.

The limits of the floodway were delineated by the floodway boundary developed for the open water floodway criteria map. The floodway was represented as a contiguous polygon by including areas of high ground or areas of depth less than 1 m inside the floodway.

The design flood extent developed for the floodway criteria maps was adjusted to create the flood fringe. The limits of the flood fringe followed the extent of direct inundation of the design flood. Areas of high ground within the direct inundation extent (and outside of the floodway) were preserved and not indicated as flood fringe in the flood hazard map. High hazard flood fringe areas were differentiated in the flood hazard maps.

Areas in the Floodway

The floodway primarily includes the main channel and parts of the overbank areas, which are mostly forested or farmland, with no significant areas of interest.

Areas in the High Hazard Flood Fringe

There were no notable areas of interest within the overbank areas in the high hazard flood fringe.

Areas in the Flood Fringe

The flood fringe includes all inundated areas outside the limits of the floodway and high hazard flood fringe. There were no notable areas of interest within the overbank areas in the flood fringe.

7 POTENTIAL CLIMATE CHANGE IMPACTS

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

7.1 Comparative Studies

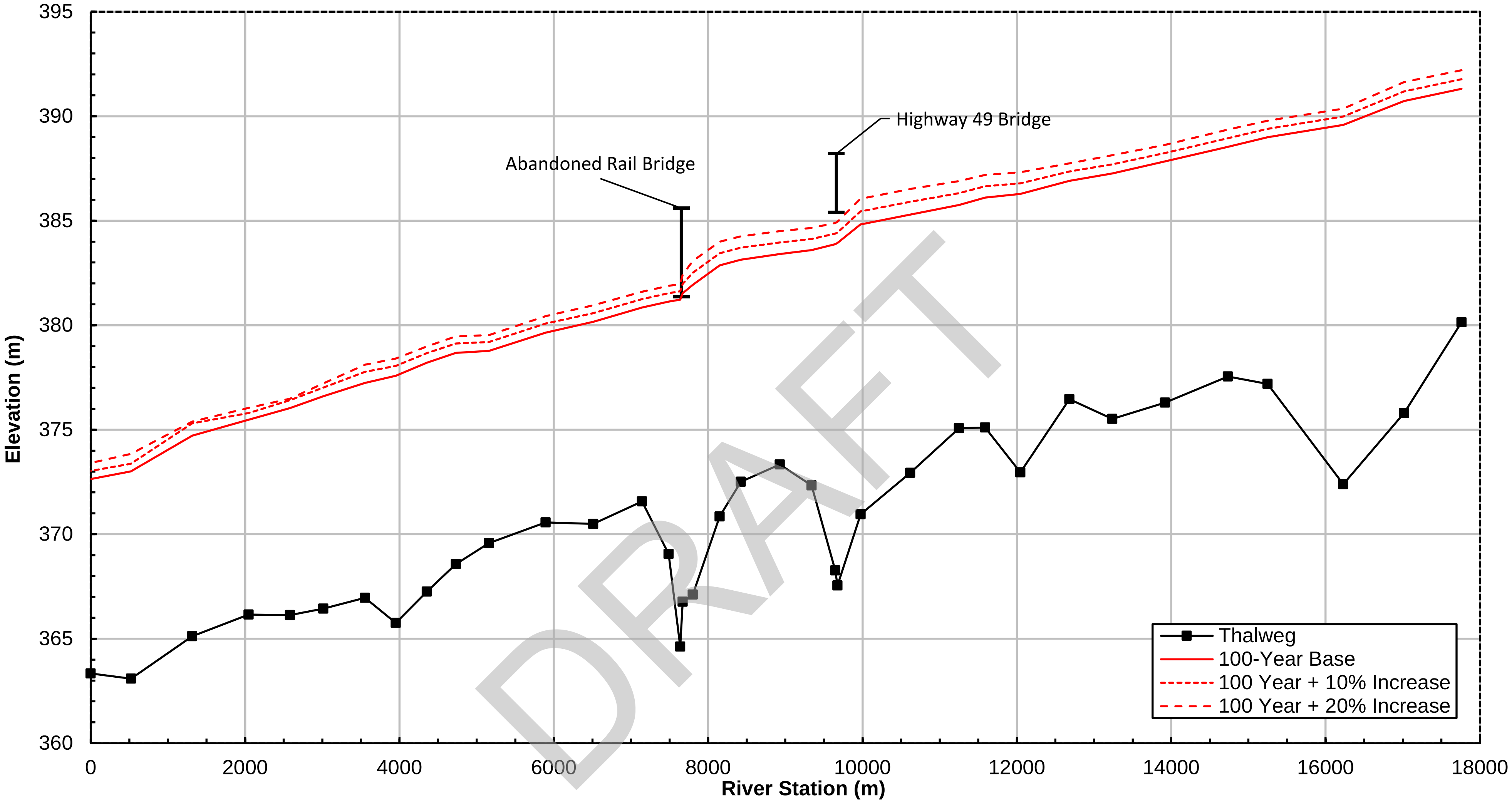
For the open water flood hazard, the current 100-year design flood water levels were compared to those associated with discharges that are 10% and 20% higher than the current 100-year flood estimates. This approach is consistent with guidelines prepared by Engineers and Geoscientists British Columbia (EGBC, 2018), which recommends that for basins where no historical trend is detectable in local or regional streamflow magnitude frequency relations, a 10% increase in design discharge be applied to account for likely future changes in water input from precipitation. On the other hand, if a statistically significant trend is detected, a 20% increase may be appropriate, particularly for smaller basins.

7.2 Results

NHC found the magnitude of the increases to be fairly uniform along the study reach. The average increase in water levels due to 10% and 20% increase in the 100-year flood discharge were 0.47 m and 0.88 m, respectively. Figure 7.1 plots a comparison between the computed 100-year flood level profile and profiles computed with discharges that are 10% and 20% greater than the 100-year flood discharge.

7.3 Supplementary Information

Climate change has the potential to affect many factors related to flood severity. For open water floods, more frequent and greater intensity summer rain storms are commonly attributed to future climate flood risks. A comprehensive analysis would consider meteorological and hydrological factors at the basin scale to assess changes in flood peak discharges and their associated return periods.



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1008016

Date: 19-Mar-2025

WATINO FLOOD STUDY

SMOKY RIVER POTENTIAL CLIMATE CHANGE IMPACTS ON DESIGN FLOOD PROFILE

FIGURE 7.1

8 CONCLUSIONS

The objectives of this study were to assess river flood-related hazards along 15 km of the Smoky River through the Municipal District of Smoky River and Birch Hills County, including the Hamlet of Watino. A provincial flood hazard mapping study was previously completed for Watino in 1996; however, the present study covers an expanded study reach and represents an update to the prior work.

The Watino Flood Study is comprised of five major project components: 1) survey and base data collection; 2) open water hydrology assessment; 3) open water hydraulic modelling; 4) open water flood inundation mapping; and 5) design flood hazard mapping. This report summarizes the work of all five components.

The collection of survey and base data primarily supports the hydraulic modelling and flood mapping. A total of 37 cross sections were surveyed along the study reaches using a combination of ground surveys and boat-based surveys to complement the LiDAR-derived DTM of the overbank area. In addition, geometric details were collected for 2 bridge crossings (one highway bridge and one abandoned bridge) in the study reach. No dedicated flood control structures were identified during the site visit, survey, and discussion with EPA and the stakeholders. In addition to the survey data, DTM, aerial imagery, hydrometric data, HWM data, WSC rating curves, and other mapping features were gathered.

The primary purpose of the open water hydrology assessment was to develop flood frequency estimates for the Smoky River study reach, in support of the hydraulic modelling and flood mapping tasks. A single station frequency analysis at WSC Station 07GJ001 (Smoky River at Watino) was conducted and a Bulletin 17C distribution was adopted to estimate the flood magnitudes for the return periods of 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year. These updated estimates are comparable with those from NHC (2016) but are notably lower than AEP 1994 estimates, which were used in the 1996 flood study. This discrepancy is particularly pronounced for the 20-year and higher return periods. The 1994 flood peak estimates, now 30-years old, are not representative of the updated data series.

A hydraulic model of the study reach was developed using the HEC-RAS computer program from the U.S. Army Corps of Engineers. Channel bathymetry and non-hydro-flattened DTM data were used to develop the geometry of the hydraulic model. Upstream and downstream boundary conditions were specified in the model. An inflow discharge was assigned at the upstream boundary and a normal depth slope of 0.00112 m/m was applied at the downstream boundary. The downstream boundary was positioned far enough outside the downstream study limit to reduce the impact of downstream boundary condition on the model results. The channel roughness values were determined through model calibration using the 2020 flood event, with values of 0.035 assigned upstream of the Highway 49 bridge and 0.032 downstream. The overbank roughness coefficients were defined based on landcover composition, professional judgement, and guidance from literature.

Water surface profiles were prepared for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year open water flood frequency return period discharges. A model sensitivity analysis was performed on several model parameters including boundary conditions (upstream and downstream) and the Manning's roughness values (channel and overbanks). It was concluded that the model is most sensitive to discharge and channel roughness values. The sensitivity of the model to other parameters analyzed was minimal.

Flood inundation maps were generated for various open water flood frequency return periods, ranging from 2- to 1000-year, based on water surface profiles derived from hydraulic modelling. No significant areas or buildings are expected to be inundated up to the 100-year flood event. However, buildings and residences in the Hamlet of Watino may be impacted during floods with return periods of 200 years or greater. The Highway 49 bridge would be affected as early as during a 350-year flood, with floodwater reaching its low chord and overtopping the right bank approach road.

Floodway criteria maps were developed for the 100-year design flood, showing the criteria used to define the floodway and flood fringe. The floodway boundary largely follows the inundation extent criteria. Flood hazard maps were later created to illustrate the floodway, flood fringe, and high hazard flood fringe areas for the design flood, along with incremental risk areas for the 200- and 500-year floods. No significant overbank areas are located within the floodway or flood fringes.

9 REFERENCES

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APPENDIX A

CROSS SECTION PROPERTIES

Table A-1 Cross section properties – Smoky River

Survey XS ID	Date Surveyed	Thalweg Elevation (m)	Top of Bank Width (m)
PXS-37	2023-05-30	380.14	260.09
PXS-36	2023-05-30	375.80	268.27
PXS-35	2023-05-30	372.39	242.07
PXS-34	2023-05-30	377.19	381.80
PXS-33	2023-05-30	377.55	270.25
PXS-32	2023-05-30	376.30	315.60
PXS-31	2023-05-30	375.52	286.85
PXS-30	2023-05-30	376.46	332.07
PXS-29	2023-05-30	372.96	283.97
PXS-28	2023-05-30	375.11	284.17
PXS-27	2023-05-30	375.07	296.34
PXS-26	2023-05-30	372.93	303.08
PXS-25	2023-05-30	370.95	291.09
PXS-24	2023-05-30	367.55	227.00
PXS-23	2023-05-30	368.27	231.70
PXS-22	2023-05-31	372.34	239.92
PXS-21	2023-05-31	373.34	351.47
PXS-20	2023-05-31	372.51	471.03
PXS-19	2023-05-31	370.85	379.02
PXS-18	2023-06-01	367.11	288.80
PXS-17	2023-05-31	366.77	212.30
PXS-16	2023-05-31	364.62	208.00
PXS-15	2023-05-31	369.06	271.54
PXS-14	2023-05-31	371.57	366.60
PXS-13	2023-05-31	370.49	324.08
PXS-12	2023-05-31	370.56	363.08
PXS-11	2023-05-31	369.57	306.42
PXS-10	2023-05-31	368.57	423.42
PXS-9	2023-05-31	367.25	312.55
PXS-8	2023-06-01	365.76	247.51
PXS-7	2023-06-01	366.96	278.81

Table A-1 Cross section properties – Smoky River (Continued)

Survey XS ID	Date Surveyed	Thalweg Elevation (m)	Top of Bank Width (m)
PXS-6	2023-08-29	366.44	294.46
PXS-5	2023-06-01	366.13	280.54
PXS-4	2023-06-01	366.16	299.08
PXS-3	2023-06-01	365.12	316.77
PXS-2	2023-06-01	363.09	278.15
PXS-1	2023-06-01	363.34	412.16

APPENDIX B

HYDRAULIC STRUCTURE DETAILS

Bridge Description

Name:	Highway 49 bridge	**Bridge File No.:**	70241
River:	Smoky River	**River Station (m):**	9,661

Geometry

Span (m):	307	**Minimum High Chord (m):**	388.197
Width (m):	12.0	**Minimum Low Chord (m):**	385.397
Pier Type:	Concrete	**No. of Piers:**	4
Pier Shape:	Triangular nose	**Pier Width (m):**	1.5

Photo(s)

Aerial image of the downstream side of the bridge

Aerial image of the upstream side of the bridge

Bridge Description

Name:	Abandoned CP Rail Bridge	**Bridge File No.:**	N/A
River:	Smoky River	**River Station (m):**	7,651

Geometry

Span (m):	N/A	**Minimum High Chord (m):**	385.600
Width (m):	3.7	**Minimum Low Chord (m):**	381.350
Pier Type:	Concrete	**No. of Piers:**	7
Pier Shape:	Elongated Semi Circular	**Pier Width (m):**	2.7-3.2

Photo(s)

Aerial image of the upstream side of the bridge

Aerial image of the downstream side of the bridge

APPENDIX C

REACH REPRESENTATIVE PHOTOS

1) Looking upstream on the Smoky River near PXS-36 (at the upstream study boundary).

2) Looking upstream on the Smoky River from PXS-33.

3) Looking upstream on the Smoky River from PXS-31.

4) Looking downstream at right bank bar on the Smoky River from PXS-26.

5) Looking downstream at Highway 49 bridge and the hamlet of Watino on the Smoky River from PXS-25

6) Looking downstream on the Smoky River from sand bar at PXS-19.

Note: Photos taken by NHC during the site visit on 1 May 2023, and the survey conducted between 30 May and 1 June 2023.

WATINO FLOOD STUDY

REACH REPRESENTATIVE PHOTOS

1008016	August, 2024	**C-1**

7) Looking upstream on the Smoky River near PXS-19.

8) Looking upstream on the Smoky River from PXS-09.

9) Looking downstream on the Smoky River from PXS-08.

10) Looking upstream on the Smoky River from PXS-08.

11) Looking upstream on the Smoky River from PXS-01 (at the downstream model boundary).

Note: Photos taken by NHC during the site visit on 1 May 2023, and the survey conducted between 30 May and 1 June 2023.

WATINO FLOOD STUDY

REACH REPRESENTATIVE PHOTOS

1008016	August, 2024	**C-2**

APPENDIX D

OPEN WATER HYDROLOGY ASSESSMENT

Classification: Public

Northwest Hydraulic Consultants Ltd.
9819 – 12 Avenue SW
Edmonton, AB T6X 0E3
Tel: 780.436.5868
www.nhcwater.com

NHC Reference No. 1008016.01

June 27, 2024

Alberta Environment and Protected Areas
11th floor, Oxbridge Place
9820-106th Street NW
Edmonton, AB T5K 2J6

Attention: Hammad Javid and Massoud Shafieifar
Via email: Hammad.Javid@gov.ab.ca, Massoud.Shafieifar@gov.ab.ca

Re: **Watino Flood Study**
Open Water Hydrology Assessment Final Report

1 INTRODUCTION

In March 2023, Alberta Environment and Protected Areas (EPA) retained Northwest Hydraulic Consultants Ltd. (NHC) to complete a flood study for areas along approximately 15 km of the Smoky River through the Municipal District of Smoky River and Birch Hills County, including the Hamlet of Watino. This study is part of Alberta's Flood Hazard Identification Program (FHIP), which is intended to enhance public safety and reduce future flood damages in the province. Results from this study are also intended to inform local land use planning decisions, flood mitigation projects, and emergency response planning.

The scope of work for this study includes the following major components:

- survey and base data collection
- open water hydrology assessment
- open water hydraulic modelling
- open water flood inundation mapping
- design flood hazard mapping.
- reporting and documentation

This letter report presents the results of the open water hydrology assessment, the primary objective of which is to develop flood frequency estimates for the Smoky River at Watino in support of the hydraulic modelling and flood mapping tasks for the Watino flood study.

2 STUDY AREA

Figure 1 shows the location and boundaries of the study reach for the Smoky River, which extends a distance of approximately 15 km from 5 km downstream of the confluence with Little Smoky River to 1 km downstream of Township Road 782. A Water Survey of Canada (WSC) gauging station 07GJ001 is located in the middle of the study reach.

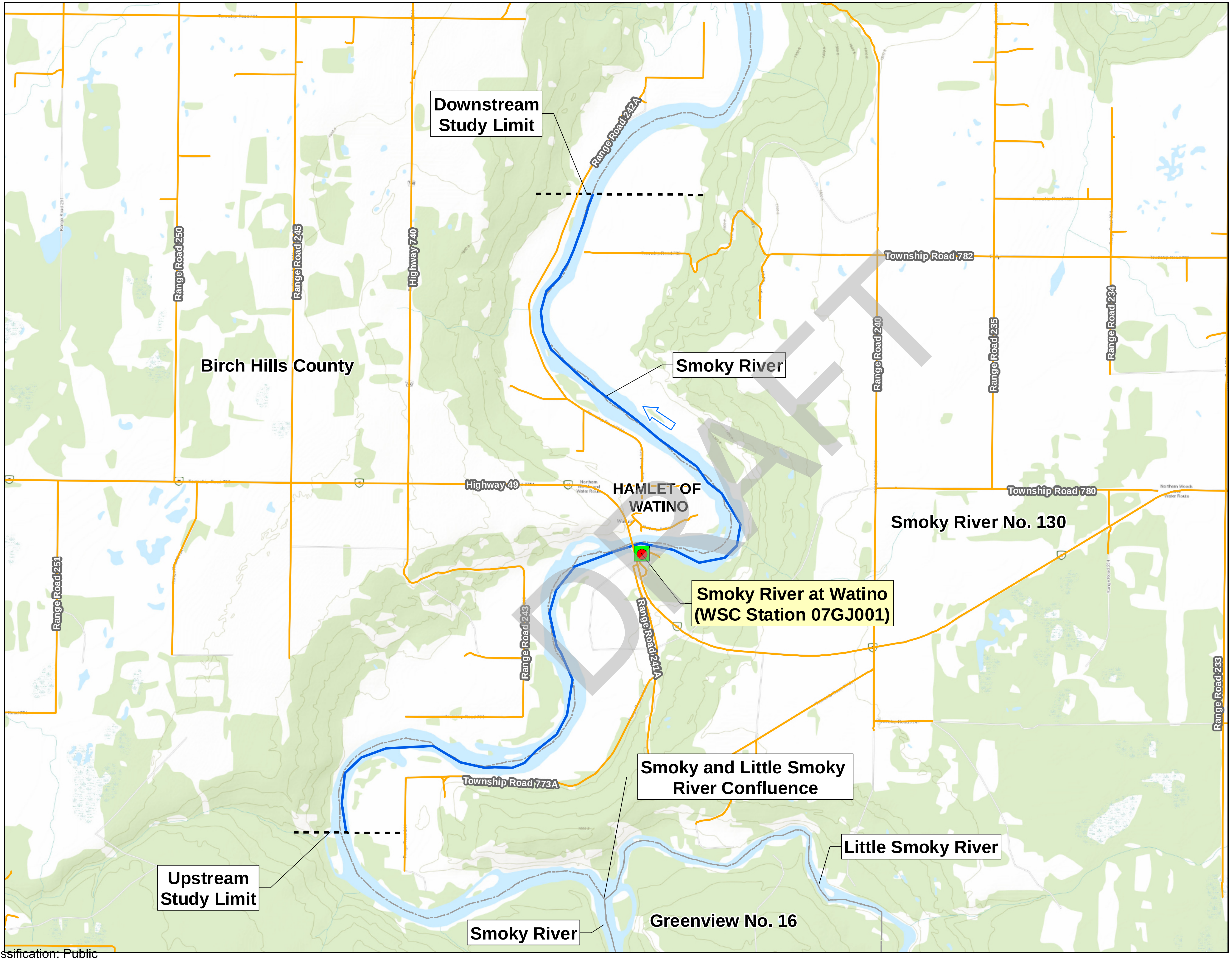
3 BASIN SETTINGS

The Smoky River basin is depicted in Figure 2. The Smoky River is a major tributary of the Peace River, originating in a steep mountainous region southwest of Grand Cache, Alberta. From there, the river flows northeast through foothills and prairie landscapes before discharging into the Peace River, approximately 5 km upstream of the Town of Peace River. The total basin area of the Smoky River is approximately 51,850 km^2. It extends across the Rocky Mountains and foothills, the western Alberta plains, Wapiti Plain physiographic regions of Alberta, and the Boreal Forest Natural Region (NHC, 1996). The Hamlet of Watino is located about 65 km upstream of the Smoky River confluence with the Peace River. Above this point, the basin area of the Smoky River is approximately 50,300 km^2. The basin has three major tributary sub-basins: the Wapiti River sub-basin (14,830 km^2), the Simonette River sub-basin (5,390 km^2), and the Little Smoky River sub-basin (13,375 km^2).

4 FLOOD CHARACTERISTICS

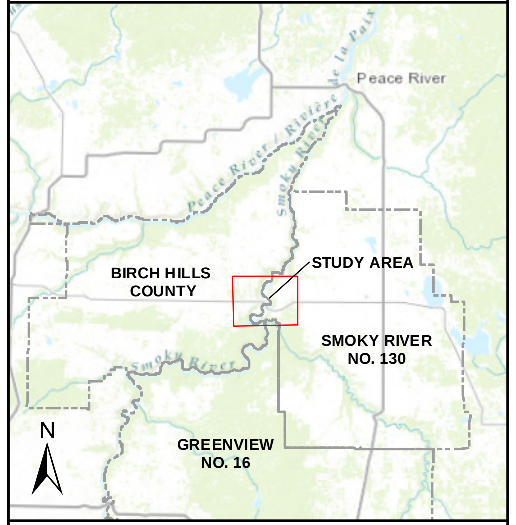
Annual peak discharges on the Smoky River usually occur between May and July; however, they can occur as early as April and as late as August. Most of the Smoky River flows are derived from the mountainous and foothill portions of the basin. Runoff from the mountainous region is generally dominated by snowmelt in the spring and early summer, and when combined with major rainfall storms in the foothill's region, peak discharges on the Smoky River typically occur.

Systematic flow monitoring data for the Smoky River at Watino (WSC station 07GJ001) are available from 1915 to 1922 and from 1955 to the present. The available daily discharge and instantaneous peak data from WSC station 07GJ001 are illustrated in Figure 3. The data do not show any significant trends over time. The three largest floods of the record occurred in 1990, 1972, and 1982, and all three events resulted in published peak instantaneous discharges greater than 9,000 m^3/s. The largest flood on record occurred on June 6, 1990, with a peak instantaneous discharge of 9,400 m^3/s. The most recent flood occurred on July 3, 2020, with a published peak instantaneous discharge of 7,950 m^3/s.



Alberta Canada

nhc



● FLOOD FREQUENCY ESTIMATE LOCATION

🟩 SMOKY RIVER GAUGE AT WATINO

STUDY REACH

ROAD

- - STUDY LIMITS

MUNICIPAL BOUNDARIES

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:50,000

0 1 2 KM

N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*26-JUN-2024*

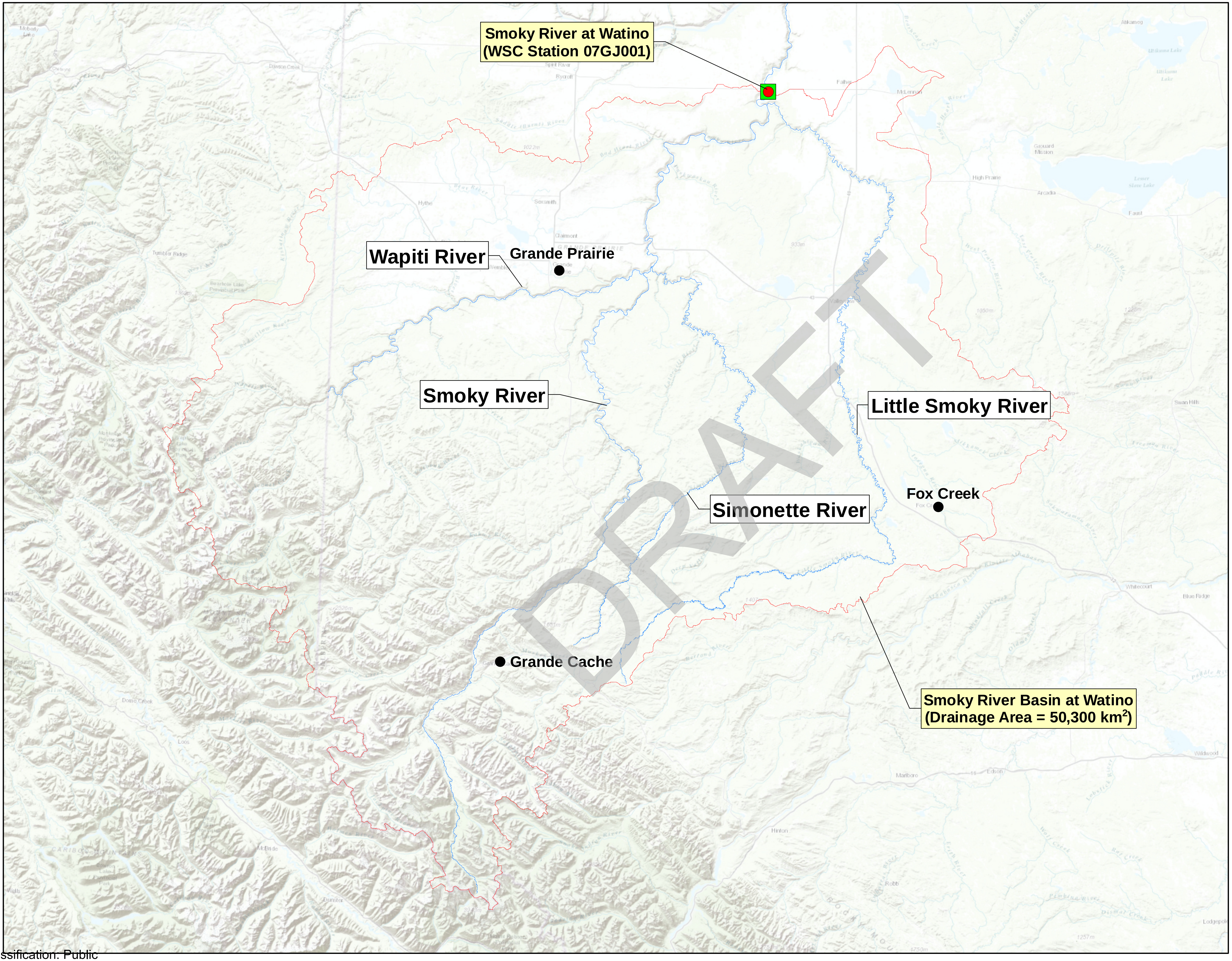
WATINO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

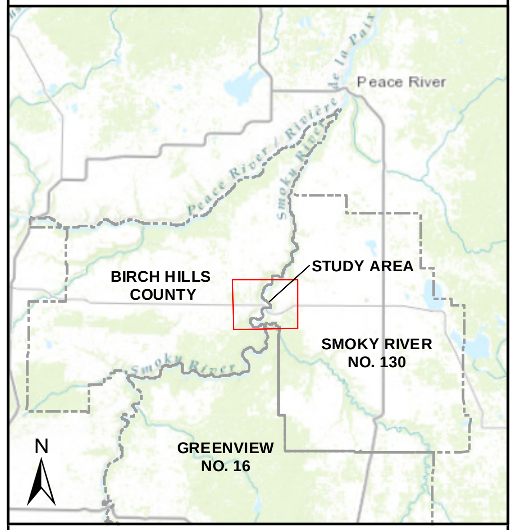
FLOOD STUDY REACH AND FLOOD FREQUENCY ESTIMATE LOCATION

FIGURE 1

Classification: Public

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS OWH FIG 01 STUDY REACH AND FF ESTIMATE LOCATION.mxd





🟥 SMOKY RIVER BASIN AT WATINO

● FLOOD FREQUENCY ESTIMATE LOCATION

● MUNICIPAL LOCATIONS

🟩 SMOKY RIVER GAUGE AT WATINO

— MAJOR RIVERS

DATA SOURCES: Basemap from Esri & NRCAN.

SCALE - 1:1,260,866

0 20 40 KM

N

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

Engineer	GIS	Reviewer
MMM	JY	RBA

Job Number	Date
1008016	26-JUN-2024

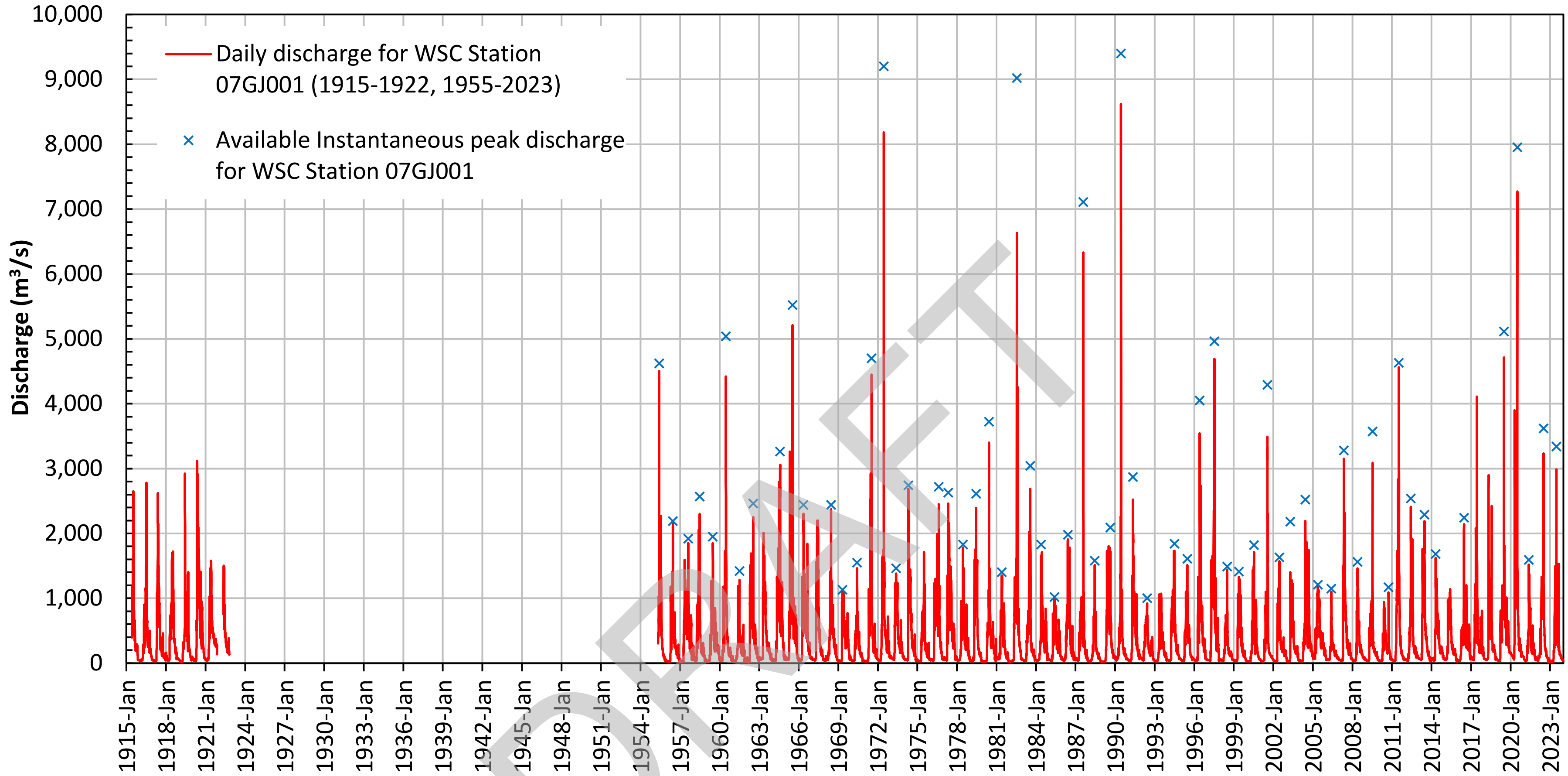
WATINO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

BASIN MAP

FIGURE 2

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016 WFS OWH FIG 02 BASIN MAP.mxd

Classification: Public



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1008016

Date: 26-JUN-2024

WATINO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

DAILY AND INSTANTANEOUS PEAK DISCHARGES FOR THE SMOKY RIVER AT WATINO

FIGURE 3

Large floods occurred at Watino in some years outside the periods of record, including 1935 and 1954 (Warner and Thompson, 1974). Limited information is available regarding these two flood events. Local inhabitants' knowledge of highwater marks suggests that the July 1975 flood peak probably reached 7,080 m^3/s, while the May 1954 flood peak probably reached 6,370 m^3/s. These events would be ranked as the fifth and sixth largest floods if included in the record for the Smoky River at Watino.

The scope of the study is limited to the open water hydrology assessment, which is the dominant flood mechanism in the study reach. However, it is important to note that the Smoky River at Watino also experienced river ice-induced floods over the years, commonly caused by ice jams and ice runs. The ice induced floods were not as significant as the open water floods. However, notable high-water levels were observed at the WSC gauge during the spring breakup (March-April) in 2018 and 2020. The water levels reached an elevation of 379.70 m and 383.40 m, respectively. The 2020 breakup level surpassed the previously estimated 100-year ice jam level (381.1 m as estimated by NHC, 1996) by more than 2 m at the gauge, although it was lower than the previously estimated 100-year open water level (NHC 1996) at the same location.

5 DATA SERIES PREPARATION

Flood frequency estimates for Smoky River at Watino could be determined based on a single station frequency analysis at WSC Station 07GJ001 (Smoky River at Watino). The station is located in the middle of the study reach (Figure 1). According to the WSC, the drainage area above this station is 50,300 km^2. The study reach is relatively short, and the local drainage area contributing to this reach is negligible in comparison with the gauged basin area. Thus, flows reported at WSC station 07GJ001 are representative of the entire Smoky River study reach, and flood frequency estimation at this station is considered adequate for this study (Figure 1).

The WSC Station 07GJ001 has 76 years of published data. All data published by WSC have gone through their standard quality assurance and quality control process, and using these published data for flood frequency analysis is a standard practice. NHC also collected unpublished preliminary data from the WSC for 2023. Based on a cursory review, these data are believed to be reasonable and can be included in the flood frequency analysis.

The annual peak discharges reported by the WSC for Station 07GJ001 and the estimated peak discharges for the two historic floods (1935 and 1954) discussed above are listed in Table 1 and illustrated in Figure 4. The data series covers 79 years from 1915 to 1922, 1935, and from 1954 to 2023. However, in many of these years, instantaneous peaks were not reported. NHC has estimated the missing instantaneous peaks for the years where daily values were available, based on the relationship between the instantaneous peak (Q_i) and daily discharges (Q_d) for years when both were reported. This relationship is illustrated in Figure 5 and represented by (**Equation 1**).

$$Q_i = 1.09Q_d \qquad \textbf{(Equation 1)}$$

When developing this relationship, the data point representing the July 1982 event (the third largest event) was deemed an outlier and ignored. A review of the WSC published daily and annual maximum daily data for the 1982 event indicates that the maximum daily discharge of this event was calculated from a partial record, which may lead to underestimation of the daily discharge. The previous hydrology study (AEP, 1994) did not address this outlier and lacks a detailed discussion or plot regarding the relationship between instantaneous and maximum daily discharge data.

Table 1 Annual peak instantaneous and peak daily discharges for the Smoky River at Watino.

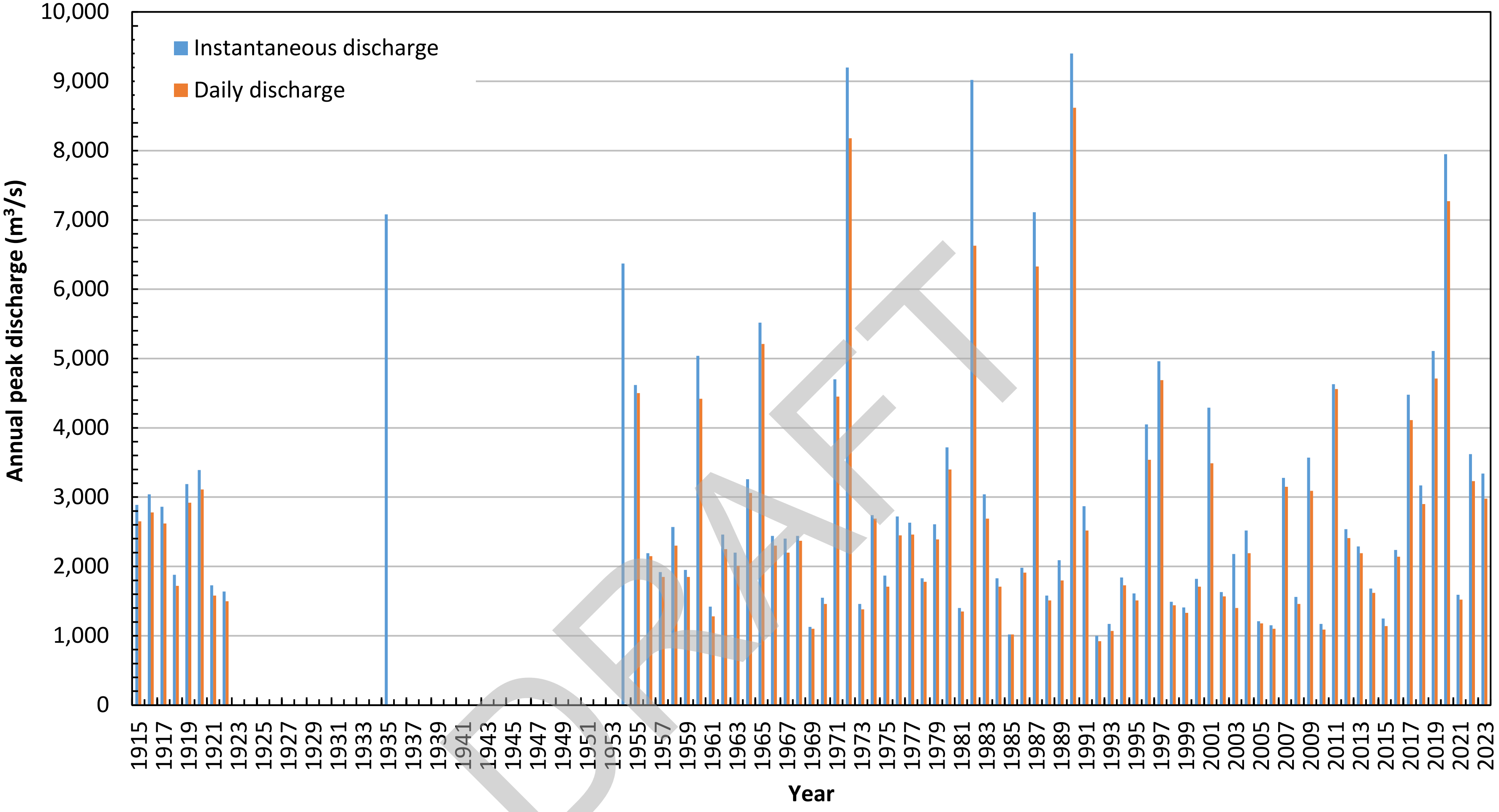
Year[1]	Peak Instantaneous Discharge (m^3/s)	Date	Peak Daily Discharge (m^3/s)	Date	Daily Discharge[2] (m^3/s)
1915	**2,890**[3]		2,650	July 14	
1916	**3,040**[3]		2,780	July 5	
1917	**2,860**[3]		2,620	May 22	
1918	**1,880**[3]		1,720	July -14	
1919	**3,190**[3]		2,920	June 12	
1920	**3,390**[3]		3,110	May 9	
1921	**1,730**[3]		1,580	June 8	
1922	**1,640**[3]		1,500	May 17	
1923 – 1934	No data				
1935	*7,080*[4]			July	
1936 – 1953	No data				
1954	*6,370*[4]			May	
1955	4,620	June 2	4500	June 2	
1956	2,190	June 13	2150	June 13	
1957	1,920	Aug. 13	1850	Aug. 13	
1958	2,570	June 30	2,300	June 30	
1959	1,950	June 28	1,850	June 28	
1960	5,040	June 23	4,420	June 23	
1961	1,420	July 2	1,280	July 3	
1962	2,460	July 20	2,250	July 20	
1963	**2,200**[3]		2,010	May 1	
1964	3,260	Aug. 3	3,060	Aug. 4	
1965	5,520	July 10	5,210	July 10	
1966	2,440	May 12	2,300	May 12	
1967	**2,400**[3]		2,200	June 3	

Year[1]	Peak Instantaneous Discharge (m^3/s)	Date	Peak Daily Discharge (m^3/s)	Date	Daily Discharge[2] (m^3/s)
1968	2,440	June 14	2,370	June 14	
1969	1,130	April 30	1,100	April 30	
1970	1,550	June 5	1,460	June 5	
1971	4,700	July 13	4,450	July 13	
1972	9,200	June 14	8,180	June 14	
1973	1,460	May 18	1,380	May 18	
1974	2,740	April 27	2,690	April 27	
1975	**1,870**[3]		1,710	June 30	
1976	2,720	Aug.18	2,450	Aug. 18	
1977	2,630	May 7	2,460	May 7	
1978	1,830	June 17	1,780	June 17	
1979	2,610	June 15	2,390	June 15	
1980	3,720	June 6	3,400	June 6	
1981	1,400	May 27	1,350	June 3	1,350
1982	9,020	July 16	6,630	July 17	
1983	3,040	July 27	2,690	July 28	
1984	1,830	May 18	1,710	June 10	1,660
1985	1,020	May 24	1,020	May 26	
1986	1,980	May 30	1,910	May 30	
1987	7,110	Aug. 3	6,330	Aug. 3	
1988	1,580	June 13	1,510	June 13	
1989	2,090	Aug. 24	1,800	July 4	1,720
1990	9,400	June 13	8,620	June 13	
1991	2,870	May 11	2,520	May 11	
1992	1,000	June 3	921	June 4	
1993	**1,170**[3]		1,070	June 26	
1994	1,840	July 6	1,730	July 6	
1995	1,610	June 22	1,510	June 22	
1996	4,050	June 1	3,540	June 1	
1997	4,960	July 13	4,690	July 13	
1998	1,490	July 3	1,440	July 3	
1999	1,410	May 26	1,330	May 26	
2000	1,820	July 11	1,710	July 11	
2001	4,290	July 19	3,490	July 20	
2002	1,630	June 19	1,570	June 19	
2003	2,180	April 18	1,400	April 18	
2004	2,520	June 7	2,190	June 8	
2005	1,210	May 18	1,180	May 18	

Year[1]	Peak Instantaneous Discharge (m^3/s)	Date	Peak Daily Discharge (m^3/s)	Date	Daily Discharge[2] (m^3/s)
2006	1,150	May 28	1,100	May 28	
2007	3,280	May 6	3,150	May 6	
2008	1,560	May 25	1,460	May 25	
2009	3,570	July 9	3,090	July 9	
2010	1,170	Sept. 30	1,090	Sept. 30	
2011	4,630	July 12	4,560	July 11	
2012	2,540	June 8	2,410	June 8	
2013	2,290	June 16	2,190	June 16	
2014	1,680	April 25	1,620	April 25	
2015	**1,250**[3]		1,140	June 1	
2016	2,240	June 18	2,140	June 18	
2017	**4,480**[3]		4,110	June 11	
2018	**3,170**[3]		2,900	April 29	
2019	5110	June 30	4,710	June 30	
2020	7,950	July 3	7,270	July 3	
2021	1,590	May 20	1,520	May 20	
2022	3,620	Jun 30	3,230	July 1	
2023[5]	3,340	Jun 20	2,980	Jun 21	

Notes:

1. No peak or daily discharges are available from WSC 07GJ001 station from 1923 to 1954.
2. Daily discharge on the same event of peak instantaneous discharge is reported in the table where the WSC reported annual peak instantaneous and daily discharges do not correspond to the same event.
3. The bolded and underlined peak instantaneous discharges were estimated from daily values based on the relationship shown in Equation 1, established in Figure 5.
4. Peak discharges for historic floods (1935 and 1954) are presented in italics and were estimated from highwater mark data according to Warner and Thompson (1974). The peak discharges have been rounded to the nearest tens.
5. The peak instantaneous and daily discharge for 2023 is obtained from WSC preliminary data and rounded to the nearest tens.



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WATINO FLOOD STUDY
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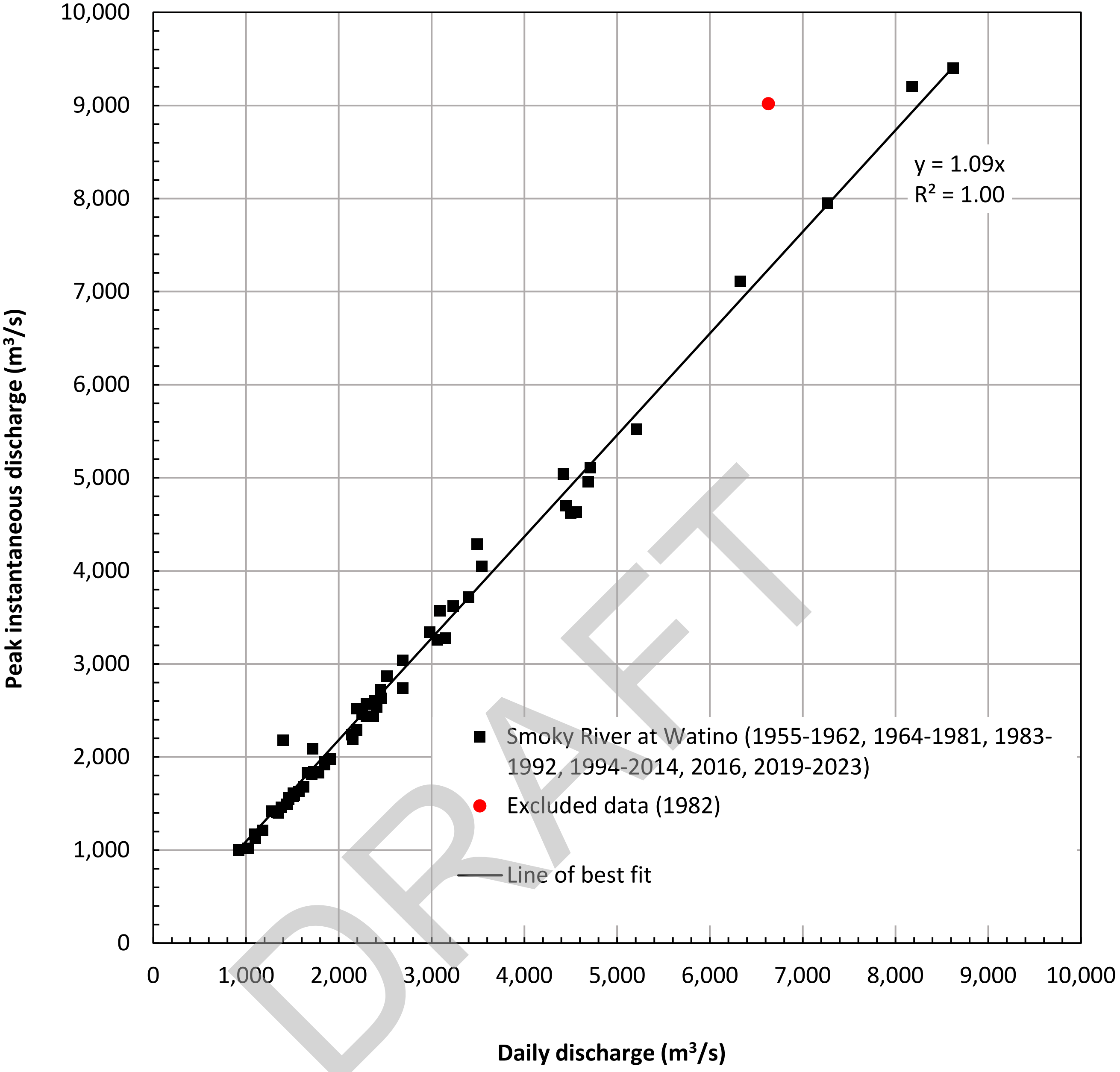
ANNUAL PEAK DISCHARGES FOR THE SMOKY RIVER AT WATINO

FIGURE 4



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WATINO FLOOD STUDY
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CORRELATION BETWEEN ANNUAL MAXIMUM DAILY AND PEAK INSTANTANEOUS DISCHARGES

FIGURE 5

6 FLOOD FREQUENCY ANALYSIS

A single-station flood frequency analysis was selected for this study as discussed in Section 5. The frequency analysis was conducted based on the Smoky River at Watino peak instantaneous discharges shown in Table 1, using the USACE HEC-SSP (version 2.2) flood frequency program and a spreadsheet model developed by NHC. In accordance with the Hydrologic and Hydraulic Guidelines for Flood Hazard Area Delineation by Alberta Environment and Parks (AEP, 2008) and Guidelines on Flood Frequency Analysis by Alberta Transportation (AT, 2001), NHC tested the following theoretical probability distributions:

- normal (N)
- log-normal (LN)
- three-parameter log-normal (LN3)
- Pearson type III (P3)
- log-Pearson type III (LP3)
- Gumbel (G)
- generalized extreme value (GEV)
- Weibull (W)

In accordance with AT, 2001, NHC used the method of moments when calculating means, variances, and skew coefficient. The theoretical limits of the skew coefficients were also calculated and considered in the analysis. In addition, to support a visual assessment of the flood frequency analysis results, NHC used the Cunnane positioning formula (Cunnane, 1978) to plot data points representing reported flows. The study team also evaluated the method presented in the United States Geological Survey Bulletin 17C (USGS, 2018). Table 2 provides a summary of the statistical parameters for the Smoky River peak flow series.

Table 2 Summary of statistical parameters of annual instantaneous peak flow series for the Smoky River at Watino.

Parameter	Annual Instantaneous Peak Flow Series (1915 – 1922, 1935, 1954 – 2023)
Total years of record	79
Mean (m^3/s)	3,010
Median (m^3/s)	2,440
Standard deviation (m^3/s)	1,940
Coefficient of variation	0.64
Skew coefficient (minimum, maximum, actual)	1.29, 1.93, 1.73

NHC compared the goodness of fit for each distribution (as applied to the peak flow series) based on the Kolmogorov–Smirnov test (K-S test). The K-S test can be used to compare a sample with a reference probability distribution by quantifying a distance between the empirical probability of the sample and the cumulative distribution function of the reference distribution. The maximum distance (referenced to as D-statistic value, D_n) can be used to describe the goodness of fit, where a smaller D_n value would indicate a better fit between the empirical distribution and the theoretical one.

NHC also evaluated the goodness of fit with a least squares method (Kite, 1977) based on the sum of squared errors (*SSE*) calculated as follows:

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

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

NHC then normalized the *SSE* values of the tested probability distributions using the mean peak discharge (Q_{pm}, the average of the annual peak discharges). In this approach a lower dimensionless *SSE* would indicate a better fit between the empirical distribution and the theoretical one.

Each of these methods has its own advantages and disadvantages. The D_n value from the K-S test is defined as the maximum discrepancy between the predicted probabilities for given flood peaks by the frequency curve and empirical probabilities from the data sample, while the SSE value represents the average deviation of predicted flood peaks from the measured or estimated discharges.

In this study, the applied frequency distributions were ranked first by D_n and *SSE* values separately; the sums of the ranks were then compared to derive the final combined ranking. However, these statistical methods can not be used as a foolproof assessment of the goodness of fit along the tails of the distributions, which are especially important when defining the return periods for severe floods. Therefore, the selection of the best representative distribution is based as much on judgement and visual assessment as it is on the statistical ranking result. Table 3 shows the ranking of the frequency distributions based on the D_n and *SSE* values. The P3 and Bulletin 17C (USGS, 1982) distribution are ranked the best in the combined ranking. The P3 distribution has the lowest *SSE* and relatively small D_n values. The Bulletin 17C distribution also has relatively small D_n and *SSE* values. The LP3 distribution has the lowest D_n value; however, it is ranked third in the combined ranking due to its higher SSE value. The LN3 distribution is also ranked third in the combined ranking.

Table 3 Goodness-of-fit comparison for probability distributions for the Smoky River at Watino.

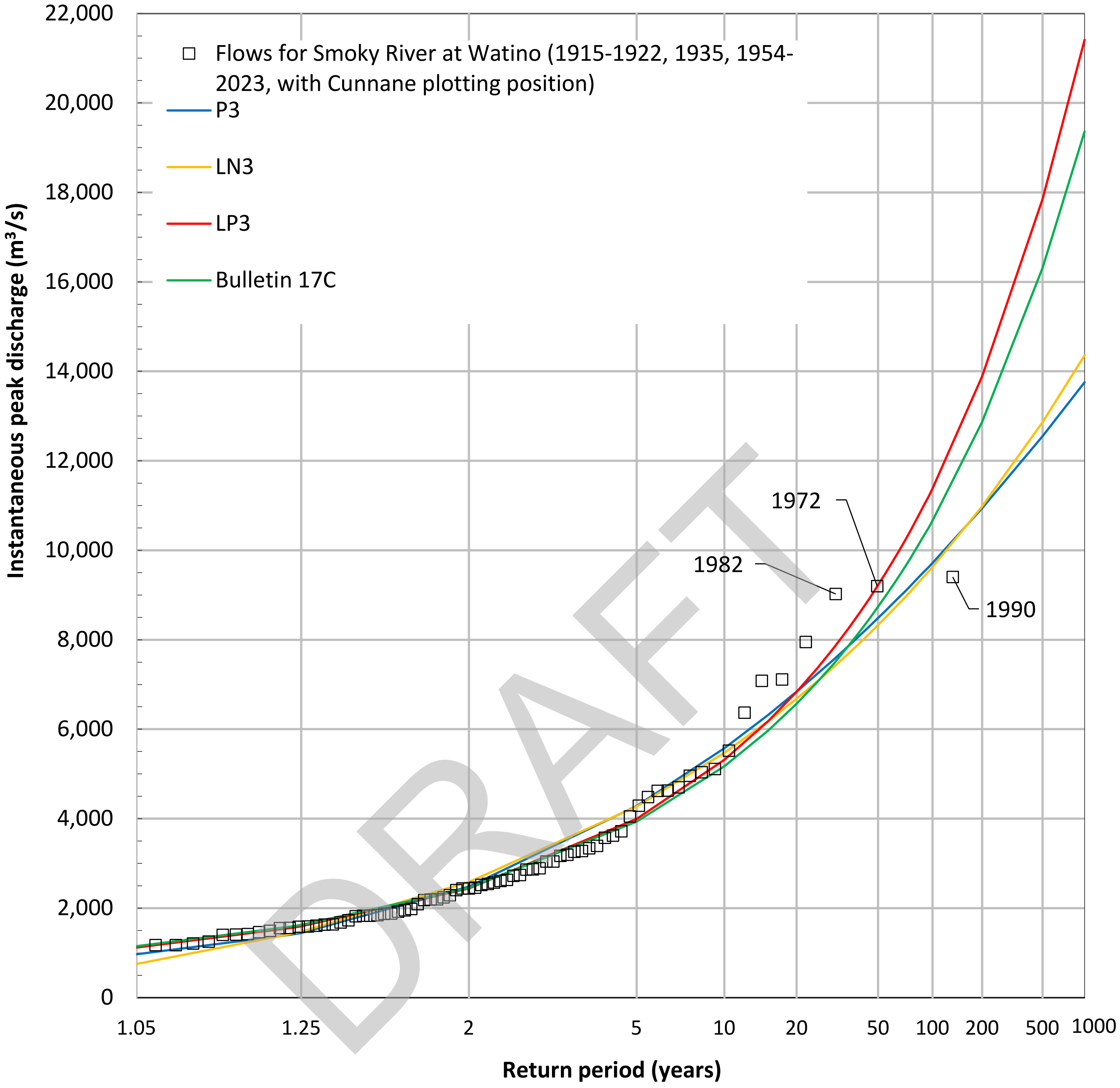
Distribution	D_n	Normalized SSE (Q_{pm} = 3,010 m^3/s)	Rank by D_n	Rank by SSE	Combined Ranking
Normal (N)	0.158	0.289	9	9	9
Log-normal (LN)	0.059	0.137	3	5	5
Three-parameter log-normal (LN3)	0.091	0.126	5	2	3
Pearson III (P3)	0.083	0.104	4	1	1
Log-Pearson III (LP3)	0.037	0.139	1	6	3
Gumbel (G)	0.120	0.161	7	7	7
Generalized extreme value (GEV)	0.094	0.135	6	4	6
Weibull (W)	0.137	0.163	8	8	8
Bulletin 17C	0.044	0.134	2	3	1

The top four distributions from the combined ranking (P3, Bulletin 17C, LP3, and LN3) are compared in Figure 6. The other distributions are shown graphically in Figure 7.

Since both the Bulletin 17C and P3 distributions are ranked the best in the combined ranking, a further comparison was required to select the best distribution. Based on a visual inspection of Figure 6, the Bulletin 17C curve generally fits the data better than the P3 curve. Their difference is relatively small for the lower part but becomes more noticeable for return periods longer than 50 years.

The previous Watino flood study (AEP, 1994) and Peace River hazard study (NHC, 2006) adopted the LP3 distribution in their flood frequency analyses for the Smoky River at Watino. The Bulletin 17C uses the LP3 distribution as the base method with the parameters being estimated from the expected moments algorithm with consideration of historic flood information.

Based on the above discussion, NHC recommends using the Bulletin 17C to estimate the flood peaks for the Smoky River at Watino. The adopted Bulletin 17C curve is shown in Figure 8 along with its 95% confidence limits.



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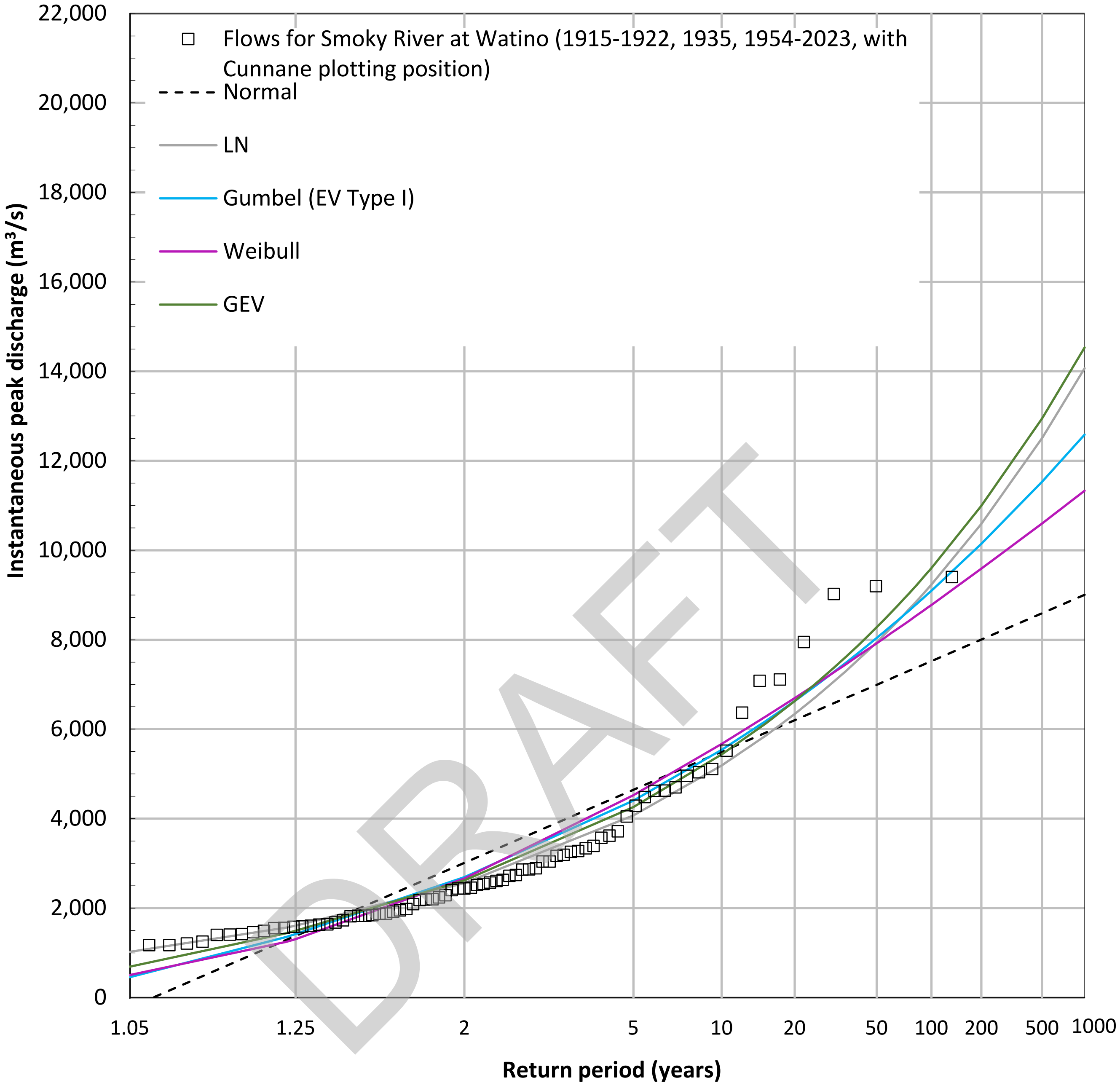
COMPARISON OF SELECTED FLOOD FREQUENCY CURVES FOR THE SMOKY RIVER AT WATINO

FIGURE 6



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WATINO FLOOD STUDY
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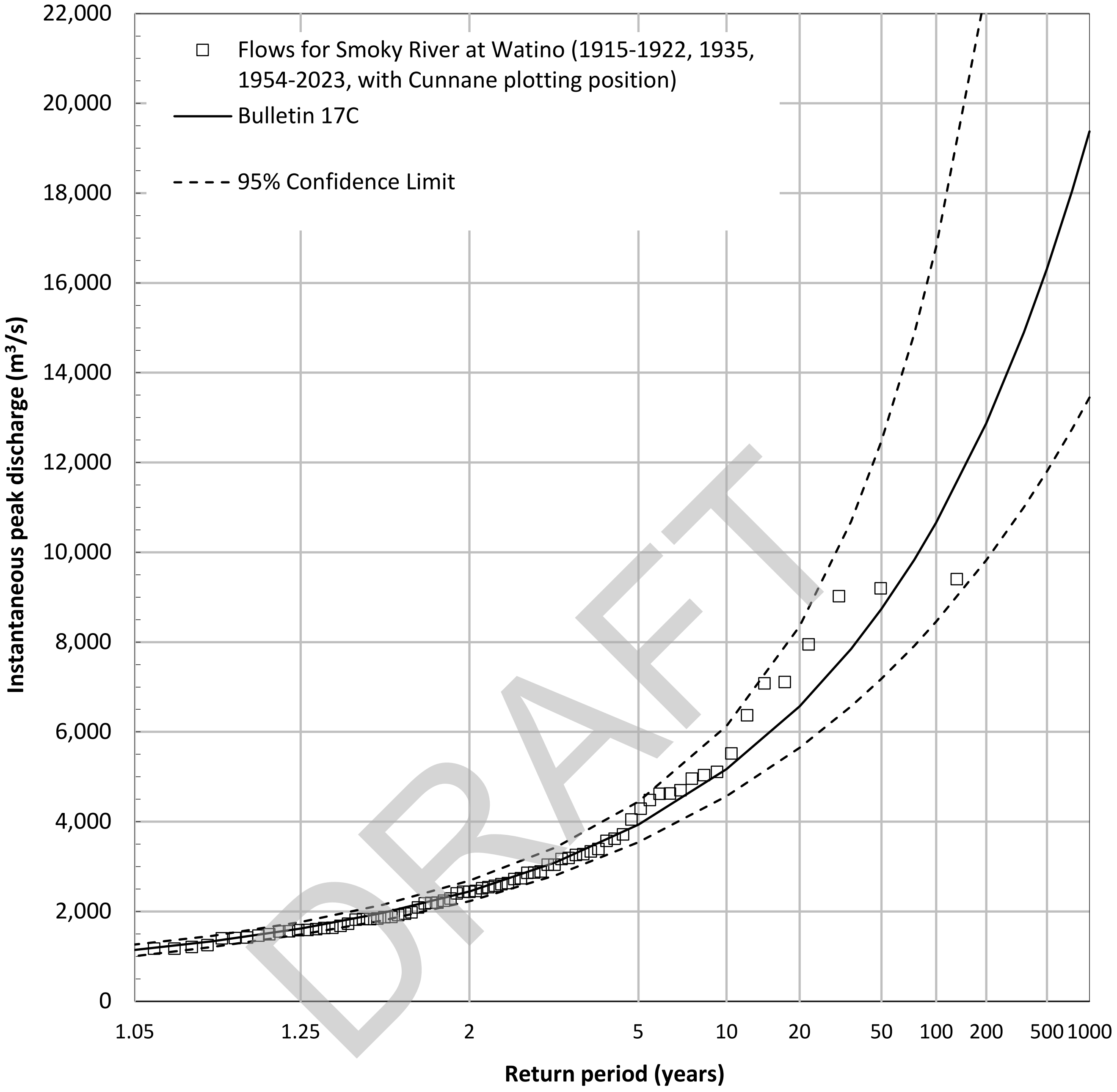
COMPARISON OF OTHER FLOOD FREQUENCY CURVES FOR THE SMOKY RIVER AT WATINO

FIGURE 7



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WATINO FLOOD STUDY
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ADOPTED BULLETIN 17C FLOOD FREQUENCY CURVE FOR THE SMOKY RIVER AT WATINO

FIGURE 8

7 FLOOD FREQUENCY ESTIMATES

Table 4 presents the flood frequency estimates for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year open water floods for the Smoky River at Watino.

Table 4 Flood frequency estimates for the Smoky River at Watino.

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge (m^3/s)	
		Value	95% Confidence Limit
1,000	0.1	19,400	13,400 - 44,000
750	0.13	18,100	12,700 - 39,100
500	0.2	16,300	11,800 - 33,100
350	0.29	14,900	11,000 - 28,500
200	0.5	12,900	9,820 - 22,500
100	1	10,700	8,450 - 16,800
75	1.3	9,830	7,920 - 14,900
50	2	8,740	7,180 - 12,500
35	2.9	7,850	6,570 - 10,700
20	5	6,570	5,640 - 8,350
10	10	5,170	4,560 - 6,130
5	20	3,930	3,540 - 4,440
2	50	2,450	2,230 - 2,690

7.1 Comparison With Previous Studies

Table 5 presents a comparison of the flood frequency estimates for the Smoky River at Watino with the estimates from previous AEP 1994 and NHC 2016 studies. The estimated peak discharges from this study are smaller than those from AEP (1994) except the 2-year peak discharges. The AEP 1994 estimates were derived from an LP3 frequency curve constructed using a flood discharge data series up to 1993, which is 30 years shorter than that used in this study. The shorter data series of the 1994 study contained the same three largest flood events as the data series used in this study (the 1990, 1972, and 1982 events), and the return periods for these events were underestimated. As such, the flood peak estimates from the 1994 flood frequency analysis are too high and not representative of the current data series.

The flood frequency estimates from this study are slightly higher than the estimates by NHC (2016) for return periods up to 50 years but are lower for longer return periods. The differences could be largely attributed to the 1935, 1954 and 2020 flood peaks, which were included in the analysis of this study but not in the NHC 2016 study. These three events were relatively large. The differences are probably also related to the difference between the Bulletin 17C and LP3 distribution adopted by this and the 2016 study, respectively.

Table 5 Comparison with previous flood frequency estimates.

Return Period (Years)	Peak Instantaneous Discharge (m^3/s)		
	This Study	AEP (1994)	NHC (2016)
1,000	19,400		21,300
750	18,100		19,700
500	16,300		17,500
350	14,900		15,800
200	12,900	15,400	13,400
100	10,700	12,300	10,800
75	9,830		9,860
50	8,740	9,730	8,650
35	7,850		7,690
20	6,570	7,010	6,340
10	5,170	5,350	4,900
5	3,930	3,950	3,680
2	2,450	2,390	2,270

8 CLIMATE CHANGE COMMENTARY

This section provides a summary of a qualitative interpretation of climate change. They are based on climate and hydrologic projections from scientific literatures. The implications of climate change on the hydrologic characteristics of the Smoky River basin have been studied recently by Schnorbus and Ben Alaya (2023). Some scientific literatures on climate and hydrologic projections are also available for the Peace River (of which the Smoky River is a tributary) and Wapiti River (a tributary to the Smoky River) basins, and they are believed to be relevant.

Schnorbus and Ben Alaya (2023) is the most recent study on the effect of climate change on flood peaks and future design flood values in the Smoky River basin, including Watino. The flood peaks and future design flood values were produced by using a tool called CanESM2-LE (the second-generation Canadian Earth System Model) for the BC Ministry of Transportation and Infrastructure. It included a baseline period (1951 to 2000) and three projected periods (2011 to 2040, 2041 to 2070, and 2071 to 2100). The study examined the flood frequency estimates for return periods between 2 and 200 years. The study projected an increase in the 2 to 200-year flood peaks for the Smoky River at Watino for the 2011 to 2041 and 2041 to 2070 periods. For the 2071 to 2100 period, the flood peaks for return periods longer than 50 years were projected to increase; but the flood peaks for shorter return periods were projected to decrease.

In the Peace River basin, current global climate models predict that both temperature and precipitation will increase due to projected increases in carbon dioxide concentrations in the atmosphere. Increased temperatures in the winter months will likely result in smaller snowpacks and earlier snowmelt runoff. Climate models differ in their predictions of changes to median monthly runoff, with some models predicting increases in runoff (e.g., Poitras et al., 2011) and others predicting decreases (e.g., Schnorbus and Ben Alaya, 2023). However, most models predict a shift in peak runoff from June to May. The projected change in average temperature and precipitation due to potential climate change for the Peace River region (including Watino) can also be readily viewed using Plan2Adapt[1]. This tool generates maps, plots, and data describing projected future climate conditions. Plan2Adapt predicts an average annual temperature increase of 3.3°C and an average precipitation increase of 9% for the Peace River region for the 2050s (2040-2069), compared to the 1961 to 1990 baseline period. This tool also shows an average annual temperature increase of 5.5°C and an average precipitation increase of 13% for the Peace River region for the 2080s (2070-2099). Precipitation as snow is expected to decrease 19 – 26 % annually (7 - 9 % in winter and 35 – 47 % in spring) with the increases in average temperatures.

Kerkhoven (2014) assessed the implications of climate change on the Wapiti River basin, which forms a large part of the Smoky River basin. The overall conclusions for the Wapiti River basin can be used to generalize the expected climate change outcomes for the Smoky River basin. Kerkhoven (2014) analyzed historical data and future projections from General Circulation Models (GCMs) for temperature, precipitation, and river flow. The study utilized Environment Canada's CanGrid dataset, downscaled meteorological data, and future climate projections from the Pacific Climate Impacts Consortium (PCIC). Historical data indicated a significant warming trend of +1.76°C per century and a non-significant increase in precipitation, driven mainly by increased rainfall rather than snowfall. Future projections suggest continued warming with potential acceleration and a range of precipitation changes. Stream flow trends showed a historical decline, inconsistent with the slight increases projected under climate change

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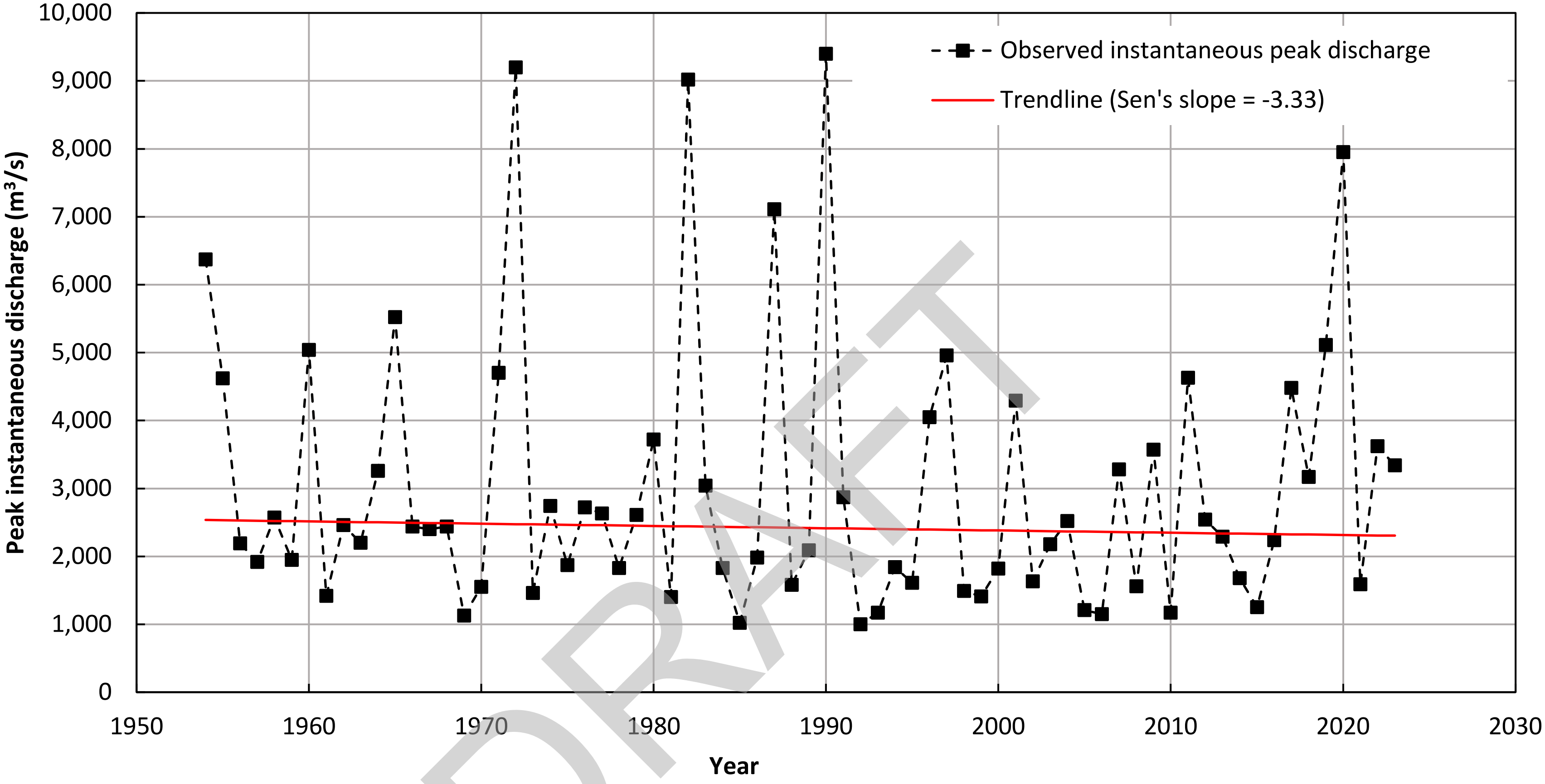
[1] Pacific Climate Impacts Consortium, University of Victoria. Plan2adapt tool. https://services.pacificclimate.org/plan2adapt/app/. Accessed on 28 November, 2022.

projections from the PCIC climate change simulation for the Upper Peace River Basin. Long-term projections indicate that temperature and precipitation will likely follow the historical trends in future, maintaining the river flow within historical variability but with significant uncertainty due to snowfall variability. Overall, Kerkhoven suggests that river flows are expected to remain more or less in the same range as they have been historically.

As part of this assessment, a trend analysis was performed on available annual peak instantaneous flow data for WSC Station 07GJ001 – Smoky River at Watino using the Mann-Kendall (MK) test and Sen's slope. The MK test is a non-parametric statistic test that is commonly used in detecting trends in hydrologic data. Sen's slope is a nonparametric estimate of the slope of a trend. Annual instantaneous peak discharges for the 1954-2023 period were used for the trend analysis. The data and trendline based on Sen's slope are shown in Figure 9. While the data series appears to have a slightly downward trend (with Sen's slope of -3.3 m3/s per year as shown in the figure), the MK test indicates that it likely has no trend (p-value = 0.7).

With the climate change forecasts calling for slightly more rainfall, it can be expected that runoff from the Smoky River basin would increase. Since most of the summer rainfall occurs during large storms, it can be reasonably assumed that more rainfall would also mean more severe storms. which could in turn produce more severe floods. However, with the higher temperatures, evapotranspiration would be greater, thereby possibly reducing runoff coefficients and limiting the expected increase in flood peaks. On the whole, there is insufficient information available to enable identification of all the linkages between precipitation and runoff to make any forecasts about how climate change might affect flood peaks. The most judicious approach would be to assume no changes will occur to flood peaks in the Smoky River over the next number of decades. This assumption is consistent with the performed trend analysis on available annual peak instantaneous flow data for WSC Station 07GJ001 – Smoky River at Watino, which shows insignificant trend in the last 70 years.

In general, increased precipitation may lead to higher flood peaks due to increased precipitation intensity, but this will be mitigated by reduced snowpack and drier antecedent moisture conditions due to higher temperatures. Loss of tree cover and soil changes associated with beetle infestation, wildfires, and changing land use could also contribute to higher runoff volumes and peaks – possibly even having a greater impact than the changing climate.



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TRENDOF FLOOD PEAKS FOR THE SMOKY RIVER AT WATINO

FIGURE 9

CLOSURE

We trust this report meets your needs. If you have any questions or requests, please feel free to contact the undersigned at (780) 436-5868.

Sincerely,

Northwest Hydraulic Consultants Ltd.

Report prepared by:	**Report reviewed by:**
Makamum Mahmood, MEng, PEng Senior Project Engineer	C.H. (Ken) Zhao, PhD, PEng Principal

APEGA Permit to Practice Number: P654

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DISCLAIMER

APPENDIX E

FLOOD FREQUENCY WATER SURFACE ELEVATIONS

Table E-1 Smoky River Flood Frequency Water Surface Elevations

Cross Section Number	River Station (m)	Flood Return Period												
		2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
		Water Surface Elevation (m)												
XS-37	17,761	385.78	387.22	388.22	389.18	389.93	390.40	390.92	391.31	392.22	392.93	393.38	393.92	394.24
XS-36	17,016	385.33	386.75	387.73	388.66	389.38	389.83	390.34	390.73	391.65	392.38	392.86	393.45	393.82
XS-35	16,228	384.69	386.00	386.90	387.75	388.38	388.80	389.25	389.58	390.38	391.07	391.52	392.09	392.49
XS-34	15,247	383.96	385.34	386.29	387.17	387.80	388.22	388.66	388.99	389.80	390.52	390.99	391.59	391.98
XS-33	14,735	383.50	384.83	385.75	386.63	387.28	387.73	388.19	388.53	389.38	390.12	390.61	391.19	391.60
XS-32	13,921	382.86	384.18	385.09	386.01	386.64	387.09	387.53	387.84	388.66	389.38	389.86	390.41	390.80
XS-31	13,237	382.31	383.56	384.44	385.33	385.90	386.39	386.90	387.26	388.16	388.98	389.51	390.13	390.56
XS-30	12,679	381.81	383.07	383.98	384.90	385.54	386.05	386.55	386.91	387.77	388.66	389.23	389.89	390.34
XS-29	12,045	381.07	382.34	383.26	384.19	384.91	385.36	385.87	386.28	387.35	388.38	388.99	389.68	390.15
XS-28	11,588	380.65	381.92	382.83	383.74	384.63	385.14	385.69	386.11	387.22	388.25	388.87	389.57	390.04
XS-27	11,249	380.35	381.61	382.52	383.43	384.25	384.73	385.30	385.75	386.93	388.00	388.64	389.34	389.82
XS-26	10,618	379.61	380.91	381.86	382.82	383.63	384.16	384.79	385.29	386.56	387.69	388.34	389.04	389.53
XS-25	9,974	378.88	380.25	381.25	382.23	383.09	383.65	384.31	384.82	386.09	387.28	387.97	388.72	389.23
XS-24	9,676	378.62	379.91	380.85	381.75	382.51	382.98	383.53	383.94	384.98	386.14	386.85	387.86	388.51
XS-23	9,646	378.61	379.88	380.82	381.71	382.47	382.94	383.48	383.88	384.91	386.10	386.86	387.84	388.46
XS-22	9,340	378.39	379.60	380.50	381.35	382.07	382.54	383.13	383.59	384.69	385.67	386.35	387.22	387.79
XS-21	8,929	378.05	379.28	380.19	381.08	381.83	382.32	382.93	383.40	384.53	385.53	386.22	387.11	387.69
XS-20	8,425	377.53	378.82	379.78	380.73	381.51	382.02	382.65	383.14	384.30	385.32	386.03	386.94	387.52
XS-19	8,148	377.23	378.55	379.51	380.45	381.23	381.74	382.37	382.86	384.03	385.05	385.74	386.65	387.24
XS-18	7,801	376.71	377.84	378.70	379.58	380.31	380.79	381.43	381.93	383.10	384.15	384.84	385.68	386.28
XS-17	7,669	376.66	377.77	378.61	379.46	380.17	380.62	381.14	381.53	382.43	383.23	383.78	384.48	384.98
XS-16	7,638	376.62	377.69	378.49	379.30	379.97	380.39	380.86	381.22	381.99	382.61	383.00	383.46	383.76
XS-15	7,487	376.52	377.59	378.38	379.19	379.86	380.28	380.76	381.13	381.91	382.55	382.95	383.43	383.75
XS-14	7,143	376.20	377.29	378.09	378.90	379.57	380.00	380.48	380.85	381.62	382.25	382.65	383.11	383.43

Table E-1 Smoky River Flood Frequency Water Surface Elevations (Continued)

Cross Section Number	River Station (m)	Flood Return Period												
		2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
		Water Surface Elevation (m)												
XS-13	6,510	375.49	376.56	377.35	378.17	378.86	379.28	379.78	380.16	380.97	381.62	382.03	382.50	382.82
XS-12	5,895	374.79	375.92	376.76	377.63	378.34	378.76	379.27	379.65	380.46	381.11	381.51	381.96	382.24
XS-11	5,161	373.96	375.09	375.94	376.81	377.50	377.92	378.40	378.77	379.55	380.15	380.53	380.97	381.26
XS-10	4,732	373.54	374.78	375.68	376.60	377.33	377.77	378.29	378.68	379.49	380.09	380.50	380.97	381.28
XS-9	4,355	373.01	374.26	375.16	376.07	376.78	377.25	377.79	378.20	379.01	379.62	379.99	380.40	380.67
XS-8	3,951	372.61	373.80	374.63	375.46	376.14	376.59	377.14	377.58	378.44	379.08	379.46	379.87	380.14
XS-7	3,554	372.13	373.33	374.17	375.02	375.74	376.21	376.79	377.24	378.14	378.79	379.10	379.52	379.79
XS-6	3,013	371.57	372.78	373.63	374.49	375.19	375.65	376.19	376.60	377.23	377.88	378.29	378.74	379.00
XS-5	2,584	371.21	372.39	373.21	374.03	374.71	375.15	375.66	376.04	376.50	376.96	377.26	377.64	377.91
XS-4	2,048	370.77	371.91	372.71	373.51	374.17	374.60	375.10	375.48	376.07	376.63	377.00	377.44	377.73
XS-3	1,314	370.05	371.15	371.94	372.73	373.39	373.82	374.32	374.71	375.41	376.06	376.47	376.93	377.23
XS-2	521	369.12	370.06	370.73	371.39	371.94	372.30	372.71	373.01	373.86	374.54	374.96	375.46	375.79
XS-1	0	368.57	369.52	370.20	370.90	371.48	371.86	372.31	372.64	373.43	374.08	374.48	374.96	375.28

APPENDIX F

SENSITIVITY ANALYSIS RESULTS

Table F-1 Sensitivity analysis results for flood frequency estimates

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Flood Frequency Estimates		
		Lower Limit of Flood Frequency Estimates (- 20%)	Adopted Flood Frequency Estimates	Upper Limit of Flood Frequency Estimates (+20%)
XS-37	17,761	390.30	391.31	392.20
XS-36	17,016	389.74	390.73	391.63
XS-35	16,228	388.71	389.58	390.36
XS-34	15,247	388.13	388.99	389.78
XS-33	14,735	387.63	388.53	389.36
XS-32	13,921	386.99	387.84	388.63
XS-31	13,237	386.27	387.26	388.13
XS-30	12,679	385.91	386.91	387.75
XS-29	12,045	385.28	386.28	387.32
XS-28	11,588	385.04	386.11	387.19
XS-27	11,249	384.63	385.75	386.89
XS-26	10,618	384.05	385.29	386.52
XS-25	9,974	383.53	384.82	386.05
XS-24	9,676	382.89	383.94	384.95
XS-23	9,646	382.84	383.88	384.89
XS-22	9,340	382.45	383.59	384.66
XS-21	8,929	382.22	383.40	384.50
XS-20	8,425	381.91	383.14	384.26
XS-19	8,148	381.63	382.86	383.99
XS-18	7,801	380.68	381.93	383.07
XS-17	7,669	380.54	381.53	382.41
XS-16	7,638	380.30	381.22	381.97
XS-15	7,487	380.20	381.13	381.89
XS-14	7,143	379.91	380.85	381.60
XS-13	6,510	379.20	380.16	380.95
XS-12	5,895	378.68	379.65	380.44
XS-11	5,161	377.84	378.77	379.53
XS-10	4,732	377.69	378.68	379.47
XS-9	4,355	377.16	378.20	378.98

Table F-1 Sensitivity analysis results for flood frequency estimates (Continued)

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Flood Frequency Estimates		
		Lower Limit of Flood Frequency Estimates (- 20%)	Adopted Flood Frequency Estimates	Upper Limit of Flood Frequency Estimates (+20%)
XS-8	3,951	376.50	377.58	378.41
XS-7	3,554	376.12	377.24	378.11
XS-6	3,013	375.56	376.60	377.21
XS-5	2,584	375.06	376.04	376.49
XS-4	2,048	374.52	375.48	376.05
XS-3	1,314	373.74	374.71	375.39
XS-2	521	372.23	373.01	373.84
XS-1	0	371.79	372.64	373.41
Average Difference		-1.02	0.00	0.88
Maximum Difference		-1.29	0.00	1.23

Table F-2 Sensitivity analysis results for downstream boundary conditions

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Downstream Conditions		
		0.3 m below Adopted W.S. (W. S. = 372.34) m)	Adopted W.S.=372.64 m	0.3 m above Adopted W.S. (W. S. = 372.94) m)
XS-37	17,761	391.31	391.31	391.31
XS-36	17,016	390.73	390.73	390.73
XS-35	16,228	389.58	389.58	389.58
XS-34	15,247	388.99	388.99	388.99
XS-33	14,735	388.53	388.53	388.53
XS-32	13,921	387.84	387.84	387.84
XS-31	13,237	387.26	387.26	387.26
XS-30	12,679	386.91	386.91	386.91
XS-29	12,045	386.28	386.28	386.28
XS-28	11,588	386.11	386.11	386.11
XS-27	11,249	385.75	385.75	385.75
XS-26	10,618	385.29	385.29	385.29
XS-25	9,974	384.81	384.82	384.82
XS-24	9,676	383.94	383.94	383.94
XS-23	9,646	383.88	383.88	383.88
XS-22	9,340	383.59	383.59	383.59
XS-21	8,929	383.40	383.40	383.40
XS-20	8,425	383.13	383.14	383.14
XS-19	8,148	382.86	382.86	382.86
XS-18	7,801	381.93	381.93	381.94
XS-17	7,669	381.53	381.53	381.53
XS-16	7,638	381.22	381.22	381.22
XS-15	7,487	381.13	381.13	381.13
XS-14	7,143	380.85	380.85	380.85
XS-13	6,510	380.16	380.16	380.17
XS-12	5,895	379.65	379.65	379.66
XS-11	5,161	378.76	378.77	378.78
XS-10	4,732	378.67	378.68	378.69
XS-9	4,355	378.19	378.20	378.21

Table F-2 Sensitivity analysis results for downstream boundary conditions (Continued)

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Downstream Condition		
		0.3 m below Adopted W.S. (W. S. = 372.34) m)	**Adopted W.S.=372.64 m**	**0.3 m above Adopted W.S. (W. S. = 372.94) m)**
XS-8	3,951	377.56	377.58	377.60
XS-7	3,554	377.23	377.24	377.27
XS-6	3,013	376.57	376.60	376.63
XS-5	2,584	376.01	376.04	376.09
XS-4	2,048	375.44	375.48	375.54
XS-3	1,314	374.64	374.71	374.79
XS-2	521	372.81	373.01	373.24
XS-1	0	372.34	372.64	372.94
Average Difference		-0.02	0.00	0.02
Maximum Difference		-0.30	0.00	0.30

Table F-3 Sensitivity analysis results for channel roughness values

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Channel Roughness Values		
		Low Channel Roughness (-10%)	Adopted Roughness	High Channel Roughness (+10%)
XS-37	17,761	390.96	391.31	391.74
XS-36	17,016	390.41	390.73	391.13
XS-35	16,228	389.24	389.58	390.00
XS-34	15,247	388.74	388.99	389.31
XS-33	14,735	388.23	388.53	388.89
XS-32	13,921	387.59	387.84	388.17
XS-31	13,237	386.97	387.26	387.60
XS-30	12,679	386.66	386.91	387.21
XS-29	12,045	385.97	386.28	386.61
XS-28	11,588	385.82	386.11	386.43
XS-27	11,249	385.43	385.75	386.10
XS-26	10,618	384.95	385.29	385.65
XS-25	9,974	384.50	384.82	385.14
XS-24	9,676	383.63	383.94	384.24
XS-23	9,646	383.58	383.88	384.18
XS-22	9,340	383.25	383.59	383.91
XS-21	8,929	383.09	383.40	383.69
XS-20	8,425	382.85	383.14	383.41
XS-19	8,148	382.57	382.86	383.14
XS-18	7,801	381.54	381.93	382.28
XS-17	7,669	381.20	381.53	381.84
XS-16	7,638	380.83	381.22	381.56
XS-15	7,487	380.75	381.13	381.46
XS-14	7,143	380.49	380.85	381.16
XS-13	6,510	379.78	380.16	380.49
XS-12	5,895	379.31	379.65	379.96
XS-11	5,161	378.43	378.77	379.08
XS-10	4,732	378.38	378.68	378.96
XS-9	4,355	377.83	378.20	378.52
XS-8	3,951	377.10	377.58	377.97

Table F-3 Sensitivity analysis results for channel roughness values (Continued)

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Channel Roughness Values		
		Low Channel Roughness (-10%)	Adopted Roughness	High Channel Roughness (+10%)
XS-7	3,554	376.76	377.24	377.66
XS-6	3,013	376.18	376.60	376.93
XS-5	2,584	375.64	376.04	376.34
XS-4	2,048	375.10	375.48	375.72
XS-3	1,314	374.35	374.71	375.14
XS-2	521	372.58	373.01	373.42
XS-1	0	372.25	372.64	373.01
Average Difference		-0.34	0.00	0.34
Maximum Difference		-0.48	0.00	0.43

Table F-4 Sensitivity analysis results for overbank roughness values

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Overbank Roughness Values		
		Low Overbank Roughness (-20%)	Adopted Roughness	High Overbank Roughness (+20%)
XS-37	17,761	391.22	391.31	391.38
XS-36	17,016	390.63	390.73	390.80
XS-35	16,228	389.46	389.58	389.67
XS-34	15,247	388.84	388.99	389.11
XS-33	14,735	388.43	388.53	388.61
XS-32	13,921	387.73	387.84	387.94
XS-31	13,237	387.14	387.26	387.36
XS-30	12,679	386.79	386.91	387.01
XS-29	12,045	386.18	386.28	386.34
XS-28	11,588	386.04	386.11	386.16
XS-27	11,249	385.68	385.75	385.80
XS-26	10,618	385.24	385.29	385.33
XS-25	9,974	384.78	384.82	384.84
XS-24	9,676	383.87	383.94	383.98
XS-23	9,646	383.81	383.88	383.93
XS-22	9,340	383.53	383.59	383.63
XS-21	8,929	383.35	383.40	383.44
XS-20	8,425	383.08	383.14	383.18
XS-19	8,148	382.81	382.86	382.90
XS-18	7,801	381.89	381.93	381.97
XS-17	7,669	381.48	381.53	381.57
XS-16	7,638	381.17	381.22	381.26
XS-15	7,487	381.07	381.13	381.17
XS-14	7,143	380.78	380.85	380.89
XS-13	6,510	380.11	380.16	380.20
XS-12	5,895	379.59	379.65	379.69
XS-11	5,161	378.70	378.77	378.82
XS-10	4,732	378.61	378.68	378.73
XS-9	4,355	378.16	378.20	378.23
XS-8	3,951	377.58	377.58	377.57

Table F-4 Sensitivity analysis results for overbank roughness values (Continued)

Cross Section Number	River Station (m)	100-Year Flood Levels (m) for Varying Overbank Roughness Values		
		Low Overbank Roughness (-20%)	Adopted Roughness	High Overbank Roughness (+20%)
XS-7	3,554	377.27	377.24	377.22
XS-6	3,013	376.59	376.60	376.60
XS-5	2,584	376.03	376.04	376.05
XS-4	2,048	375.47	375.48	375.49
XS-3	1,314	374.70	374.71	374.71
XS-2	521	373.01	373.01	373.02
XS-1	0	372.63	372.64	372.64
Average Difference		-0.06	0.00	0.04
Maximum Difference		-0.15	0.00	0.12

APPENDIX G

OPEN WATER FLOOD INUNDATION MAP LIBRARY

(provided under separate cover)

APPENDIX H

FLOODWAY DETERMINATION CRITERIA AND DESIGN FLOOD LEVELS

Table H-1 Floodway Limits and Determination Criteria

Cross Section Number	River Station (m)	Left		Right	
		Floodway Limit (m)	Determining Criteria	Floodway Limit (m)	Determining Criteria
XS-37	17761	121.3	1 m Depth	668.2	1 m Depth
XS-36	17016	142.0	1 m Depth	822.5	1 m Depth
XS-35	16228	221.3	1 m Depth	929.0	1 m Depth
XS-34	15247	233.3	1 m Depth	1116.4	1 m Depth
XS-33	14735	377.6	Inundation Extent[1]	1399.7	1 m Depth
XS-32	13921	607.1	Inundation Extent[1]	1749.4	1 m Depth
XS-31	13237	747.5	Inundation Extent[2]	2284.5	1 m Depth
XS-30	12679	928.1	Inundation Extent[2]	2429.5	Inundation Extent[2]
XS-29	12045	407.3	Inundation Extent[1]	2373.6	Inundation Extent[2]
XS-28	11588	66.6	Inundation Extent[2]	2348.2	Inundation Extent[1]
XS-27	11249	168.7	Inundation Extent[2]	2157.0	Inundation Extent[1]
XS-26	10618	93.4	Inundation Extent[2]	1585.6	Inundation Extent[2]
XS-25	9974	113.1	Inundation Extent[1]	1272.3	Inundation Extent[2]
XS-24	9676	230.9	Inundation Extent[2]	496.5	Inundation Extent[2]
XS-23	9646	230.1	Inundation Extent[2]	500.8	Inundation Extent[2]
XS-22	9340	256.7	Inundation Extent[2]	1246.2	Inundation Extent[2]
XS-21	8929	499.0	Inundation Extent[2]	1350.6	Inundation Extent[2]
XS-20	8425	604.4	Previous Floodway	1378.0	Inundation Extent[2]
XS-19	8148	621.3	Inundation Extent[1]	1326.3	Inundation Extent[1]
XS-18	7801	647.8	Inundation Extent[1]	1423.0	Inundation Extent[2]
XS-17	7669	667.2	Inundation Extent[2]	1118.6	Inundation Extent[2]
XS-16	7638	685.6	Inundation Extent[1]	1169.9	Inundation Extent[2]
XS-15	7487	596.7	Inundation Extent[2]	1326.0	Inundation Extent[1]
XS-14	7143	469.9	1 m/s Velocity	1043.9	Inundation Extent[2]
XS-13	6510	312.0	Inundation Extent[2]	891.4	Inundation Extent[2]
XS-12	5895	501.6	Inundation Extent[2]	1118.1	Inundation Extent[2]
XS-11	5161	686.6	Inundation Extent[1]	1397.3	Inundation Extent[2]
XS-10	4732	740.3	Previous Floodway	1484.1	1 m Depth
XS-09	4355	539.2	1 m Depth	1235.0	1 m/s Velocity
XS-08	3951	344.0	1 m Depth	1028.4	Inundation Extent[1]

Table H-1 Floodway Limits and Determination Criteria (Continued)

Cross Section Number	River Station (m)	Left		Right	
		Floodway Limit (m)	Determining Criteria	Floodway Limit (m)	Determining Criteria
XS-07	3554	551.7	1 m Depth	1092.8	1 m/s Velocity
XS-06	3013	508.3	1 m Depth	847.7	1 m/s Velocity
XS-05	2584	288.1	1 m Depth	640.5	1 m/s Velocity
XS-04	2048	156.5	1 m Depth	472.6	1 m/s Velocity

Notes:

1. No viable flood fringe.
2. The previous floodway is outside the inundation extent.

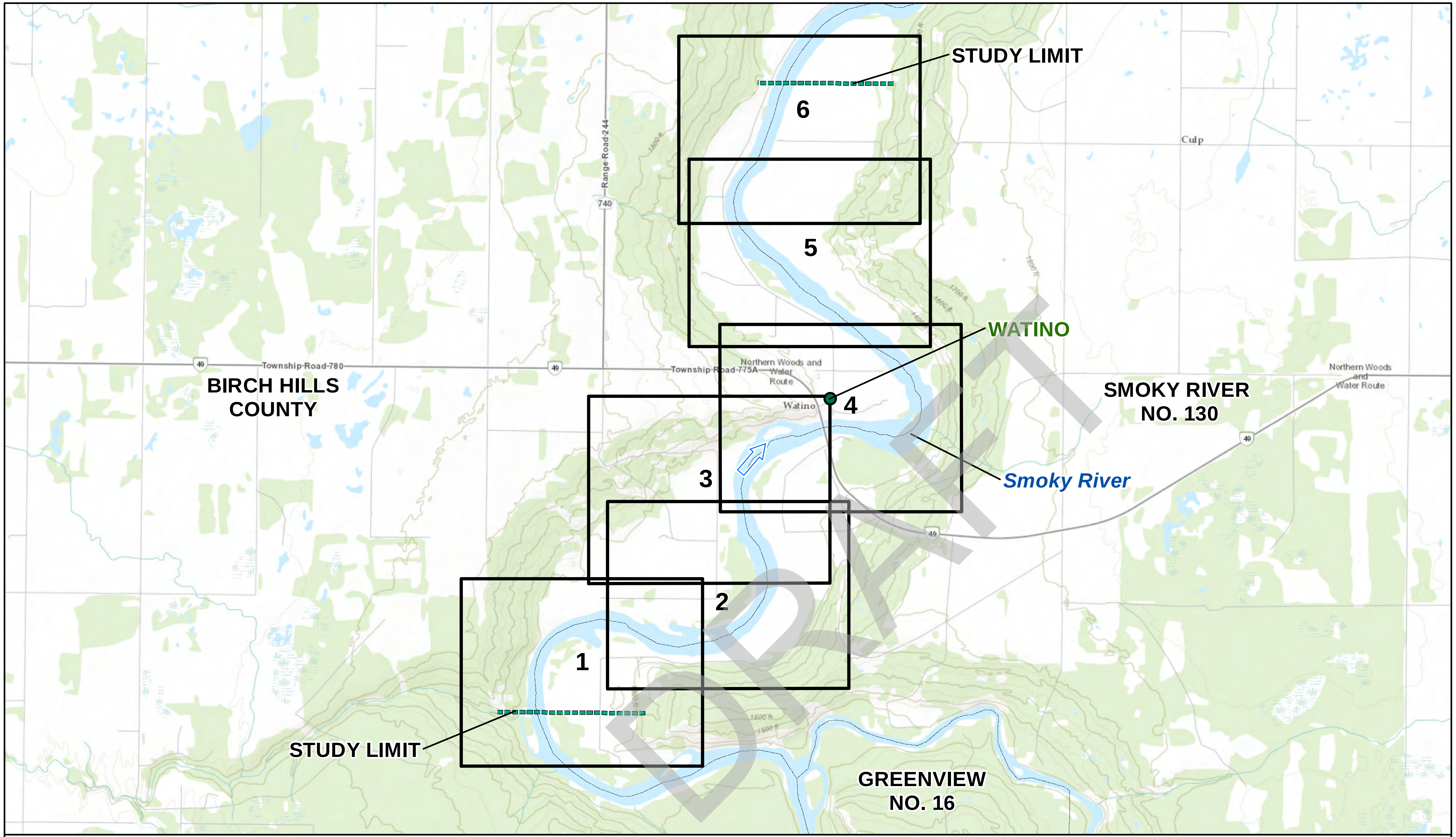
Table H-2 Computed Design Flood Levels

Cross Section Number	River Station (m)	Design Flood Level (m)	Cross Section Number	River Station (m)	Design Flood Level (m)
XS-37	17761	391.31	XS-20	8425	383.14
XS-36	17016	390.73	XS-19	8148	382.86
XS-35	16228	389.58	XS-18	7801	381.93
XS-34	15247	388.99	XS-17	7669	381.53
XS-33	14735	388.53	XS-16	7638	381.22
XS-32	13921	387.84	XS-15	7487	381.13
XS-31	13237	387.26	XS-14	7143	380.85
XS-30	12679	386.91	XS-13	6510	380.16
XS-29	12045	386.28	XS-12	5895	379.65
XS-28	11588	386.11	XS-11	5161	378.77
XS-27	11249	385.75	XS-10	4732	378.68
XS-26	10618	385.29	XS-9	4355	378.20
XS-25	9974	384.82	XS-8	3951	377.58
XS-24	9676	383.94	XS-7	3554	377.24
XS-23	9646	383.88	XS-6	3013	376.60
XS-22	9340	383.59	XS-5	2584	376.04
XS-21	8929	383.40	XS-4	2048	375.48

APPENDIX I

FLOODWAY CRITERIA MAPS

Classification: Public



⇨ FLOW DIRECTION

▪▪▪▪ STUDY LIMIT

☐ MAP SHEET

⌜⌟ COUNTY OR MUNICIPAL DISTRICT

SCALE - 1:60,000

0 1 2 Km

N

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

Engineer *MMM*	*GIS* *JY*	*Reviewer* *RBA*
Job Number *1008016*	*Date* *07-JAN-2025*	

WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

INDEX MAP

Notes to Users:

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

Definitions:

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

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

Definitions (continued):

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

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

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

Data Sources and References:

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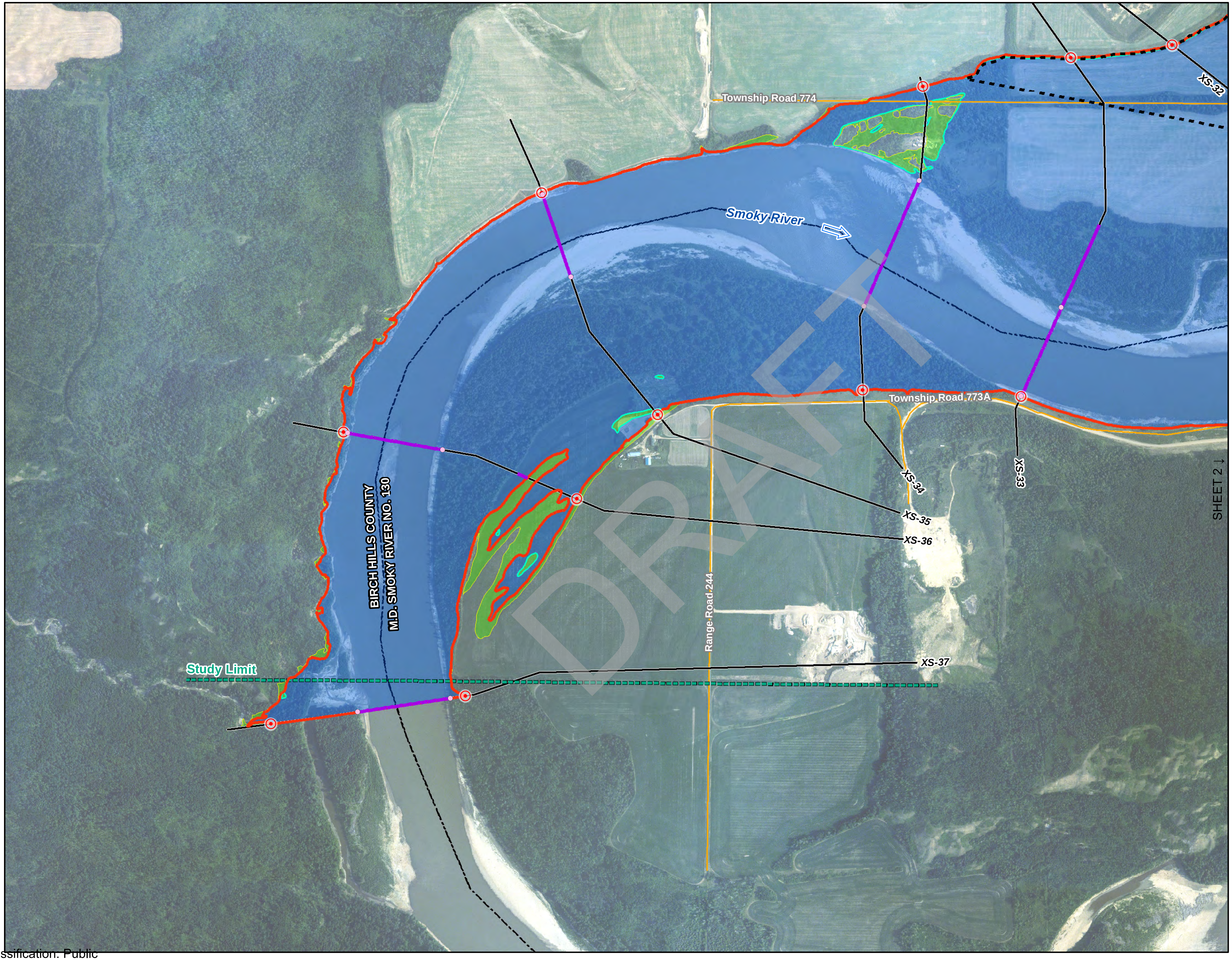
3. Base data from Hamlet of Watino, Natural Resources Canada, Alberta Environment and Protected Areas, and Altalis.

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria_IndexMap.mxd



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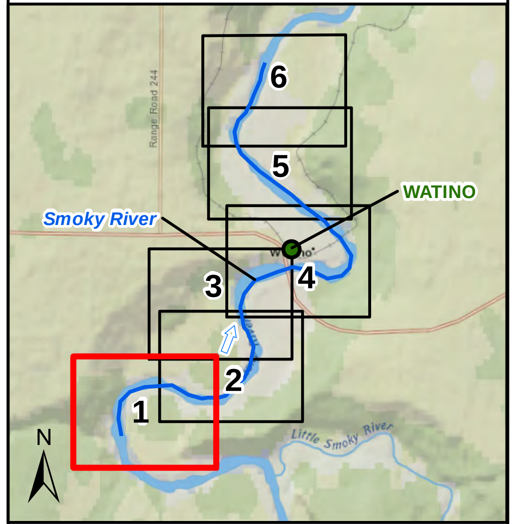


⇨	FLOW DIRECTION
▬ ▬	STUDY LIMIT
●	BANK STATION
⊙	PROPOSED FLOODWAY LIMIT
⬚	PREVIOUS FLOODWAY
▬	PROPOSED FLOODWAY BOUNDARY
⊏⊐	BRIDGE
▬	VELOCITY ≥ 1 m/s
▬	MODEL CROSS SECTION
▬	1 m DEPTH CONTOUR
🟦	DEPTH ≥ 1 m
🟩	100-YEAR OPEN WATER DESIGN FLOOD EXTENT
▬	PROVINCIAL HIGHWAY
▬	LOCAL ROAD
⬚	COUNTY OR MUNICIPAL DISTRICT

DISCHARGE
SMOKY RIVER = 10,700 m^3/s

SCALE - 1:10,000
0 200 400 M
N

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

Engineer *MMM*	*GIS* *JY*	*Reviewer* *RBA*

Job Number *1008016*	*Date* *09-MAR-2025*

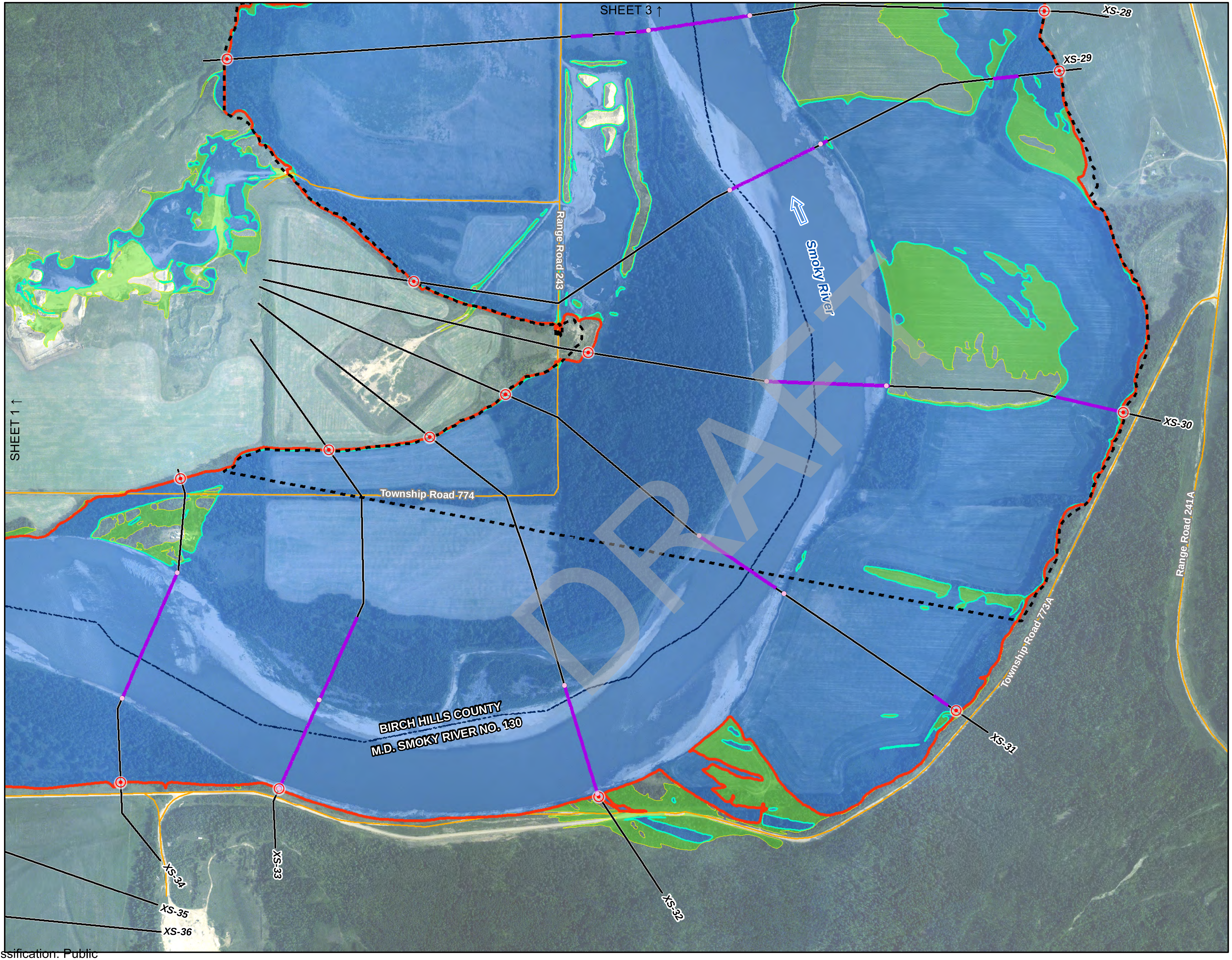
WATINO FLOOD STUDY

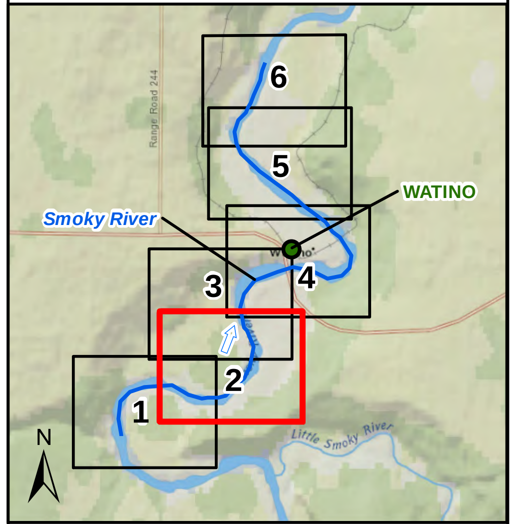
FLOODWAY CRITERIA MAP

SHEET 1 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria.mxd

Classification: Public





⇨ FLOW DIRECTION
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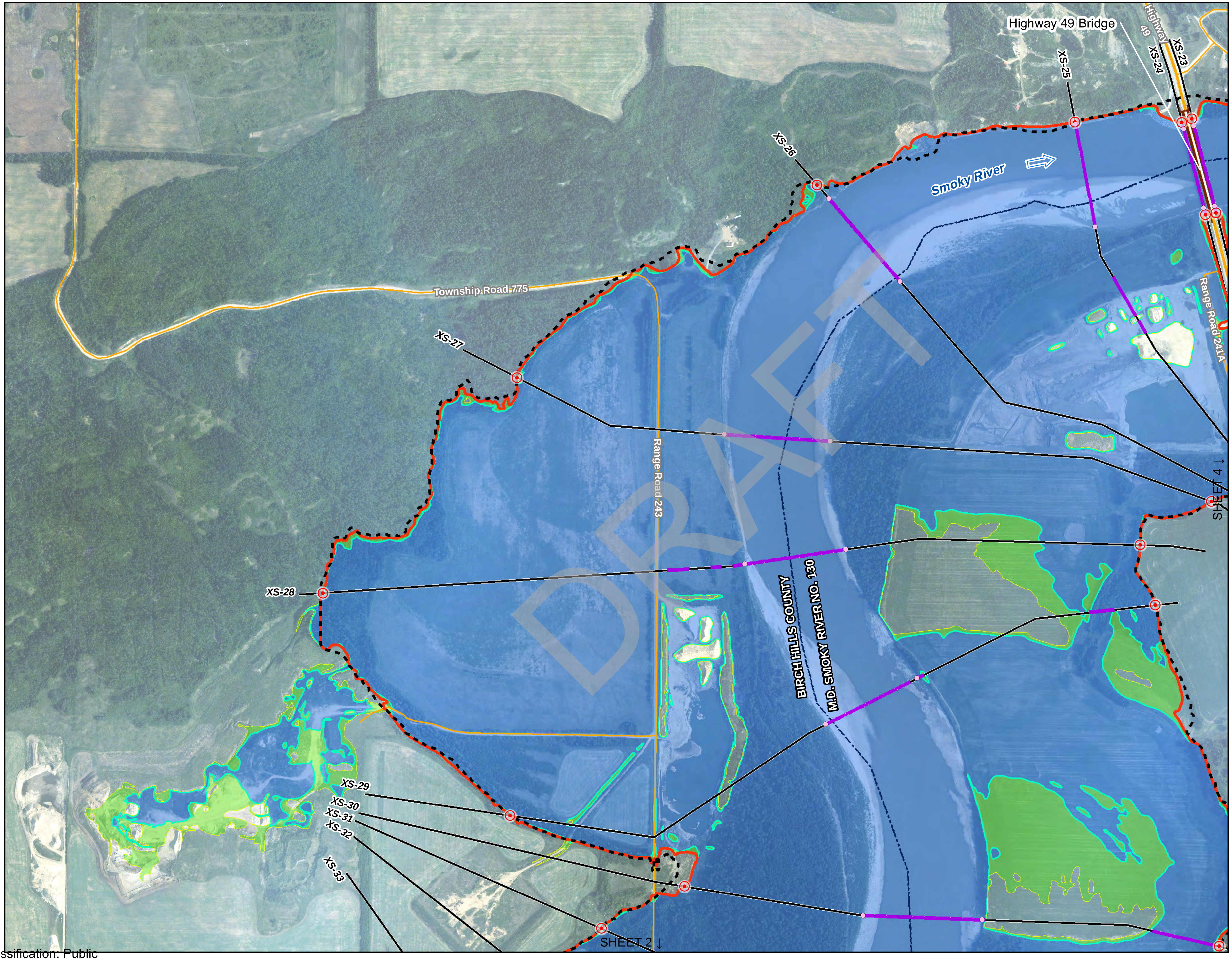
WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 2 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria.mxd

Classification: Public



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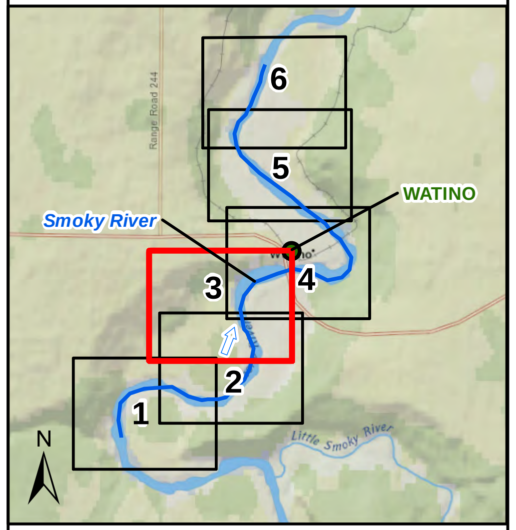

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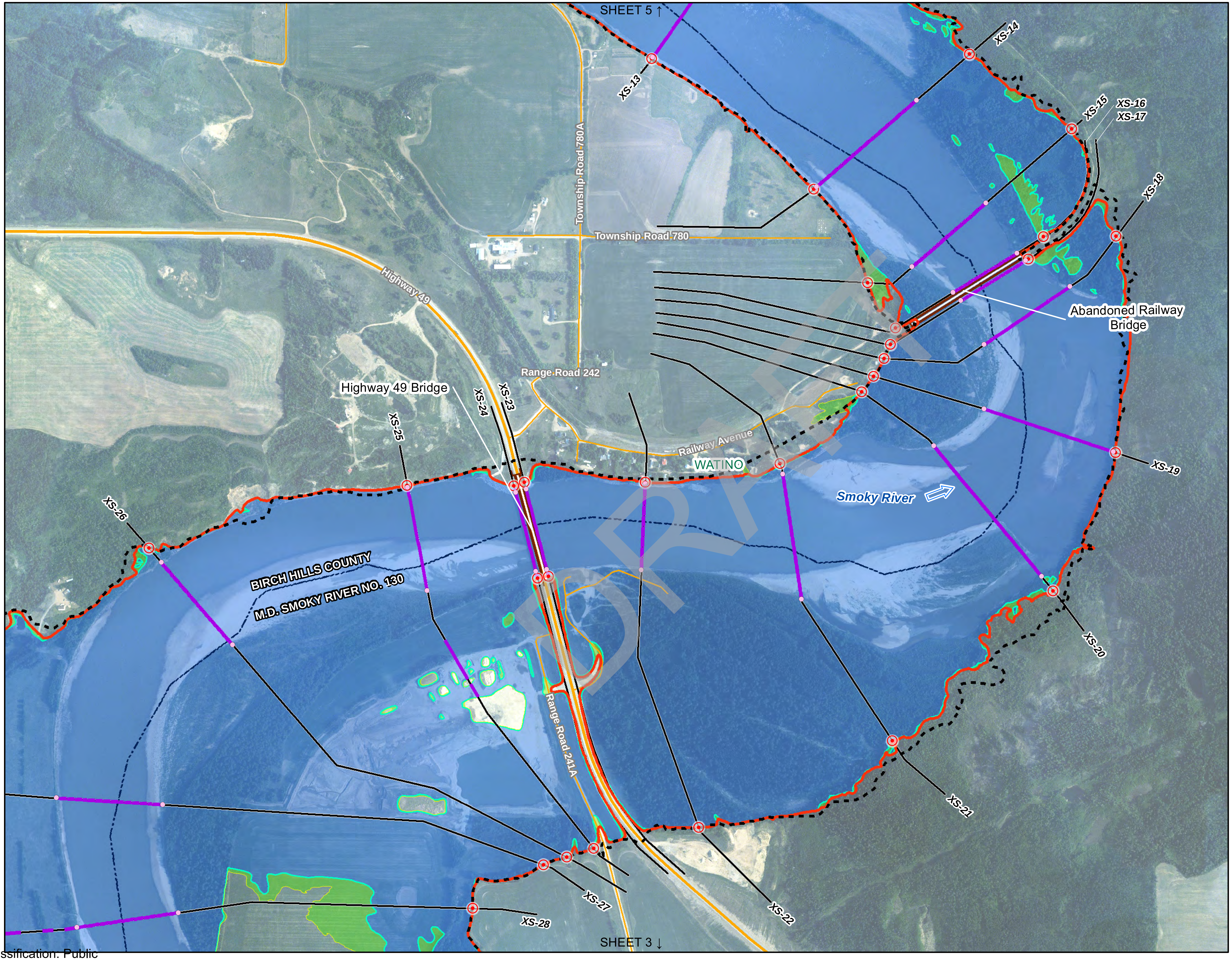
1008016	*09-MAR-2025*

WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 3 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria.mxd



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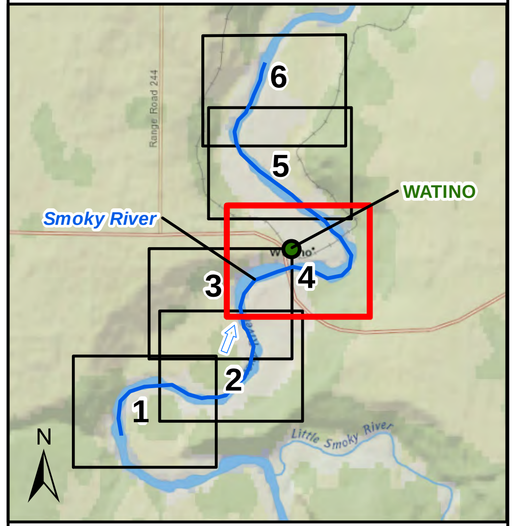

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MMM	*JY*	*RBA*

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1008016	*09-MAR-2025*

WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 4 OF 6

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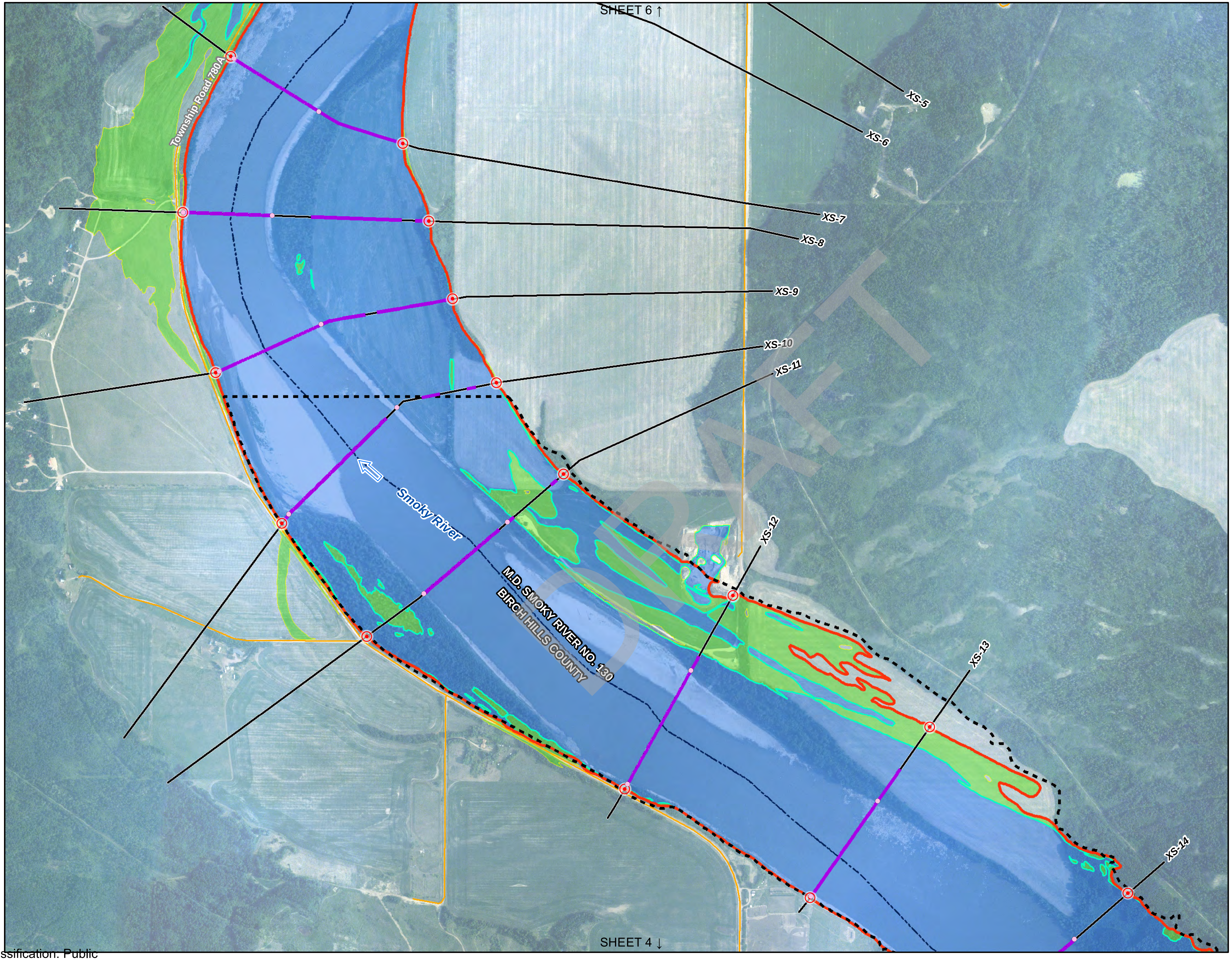

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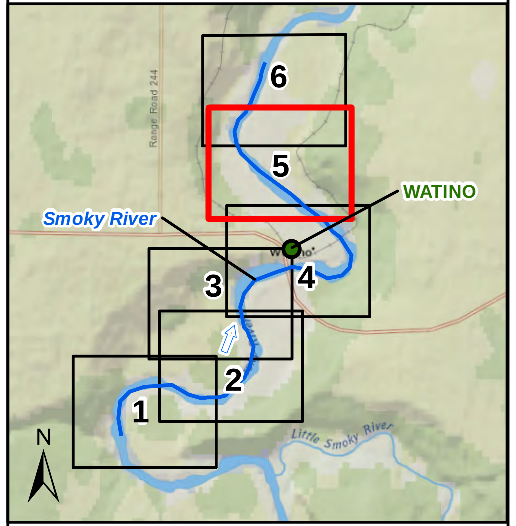

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MMM	*JY*	*RBA*

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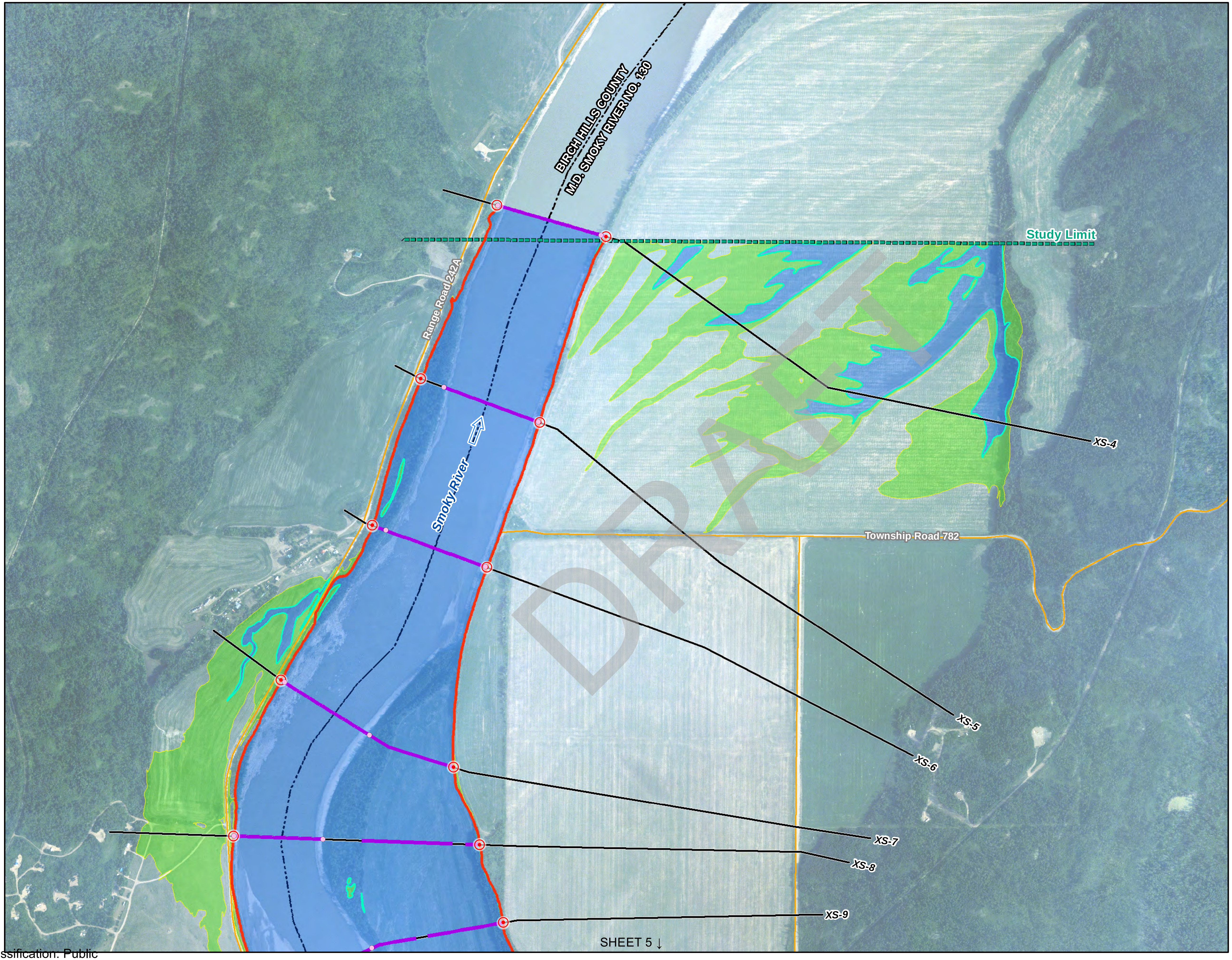
1008016	*09-MAR-2025*

WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 5 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria.mxd



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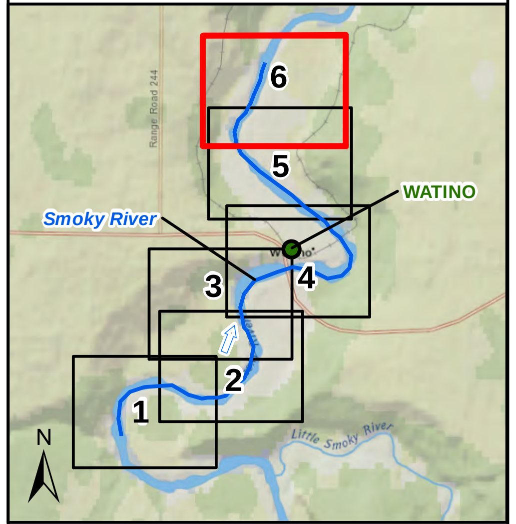

- FLOW DIRECTION
- STUDY LIMIT
- BANK STATION
- PROPOSED FLOODWAY LIMIT
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- PROPOSED FLOODWAY BOUNDARY
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Coordinate System: NAD 1983 CSRS 3TM 117; Vertical Datum: CGVD28 HTv2.0; Units: Metres

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Job Number	Date
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WATINO FLOOD STUDY

FLOODWAY CRITERIA MAP

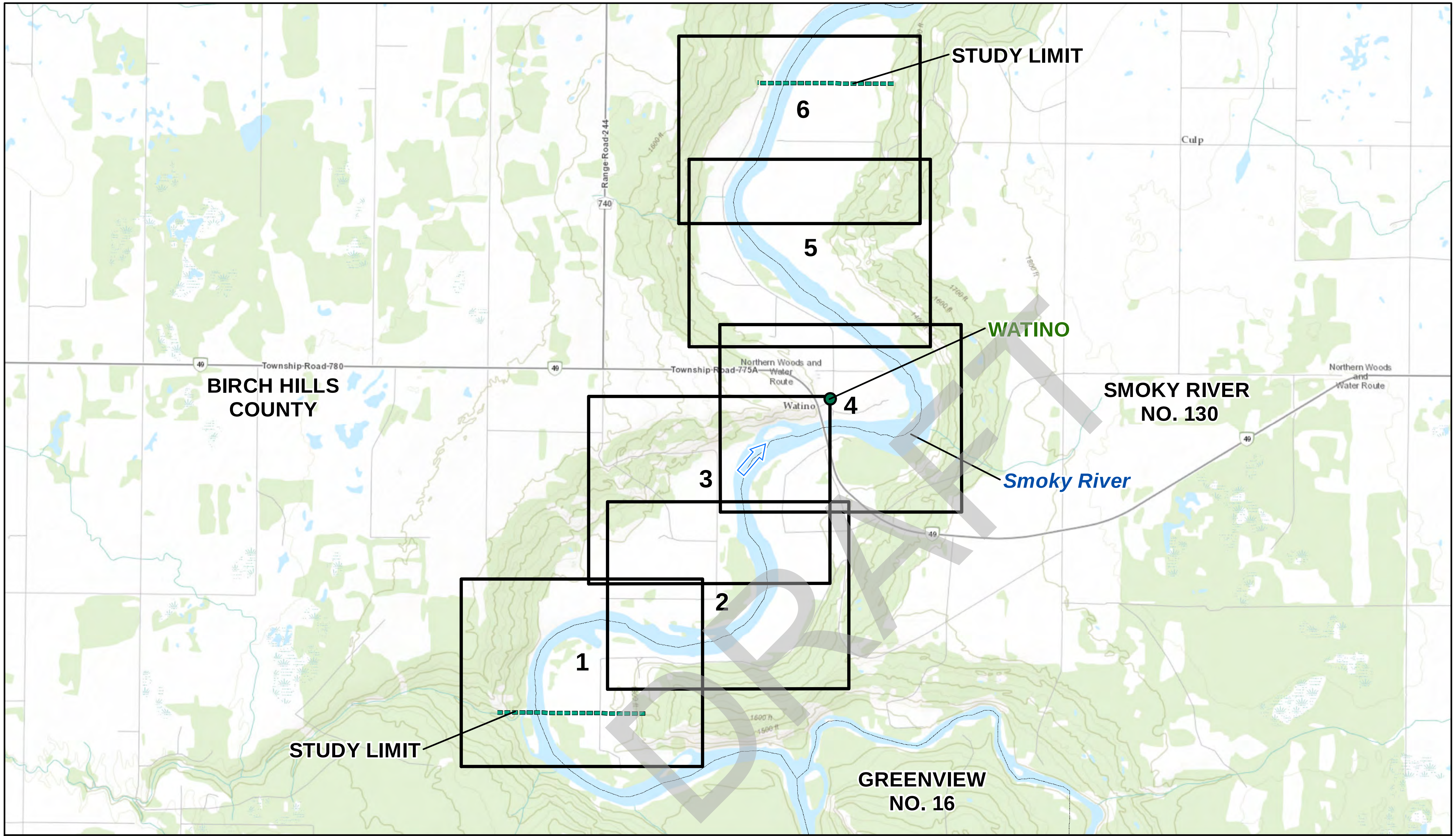
SHEET 6 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodwayCriteria.mxd

Classification: Public

APPENDIX J

FLOOD HAZARD MAPS



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FLOW DIRECTION

STUDY LIMIT

MAP SHEET

COUNTY OR MUNICIPAL DISTRICT

SCALE - 1:60,000

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N

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Definitions (continued):

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Coordinate System: NAD 1983 CSRS 3TM 117;
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Engineer	*GIS*	*Reviewer*
MMM	*JY*	*RBA*

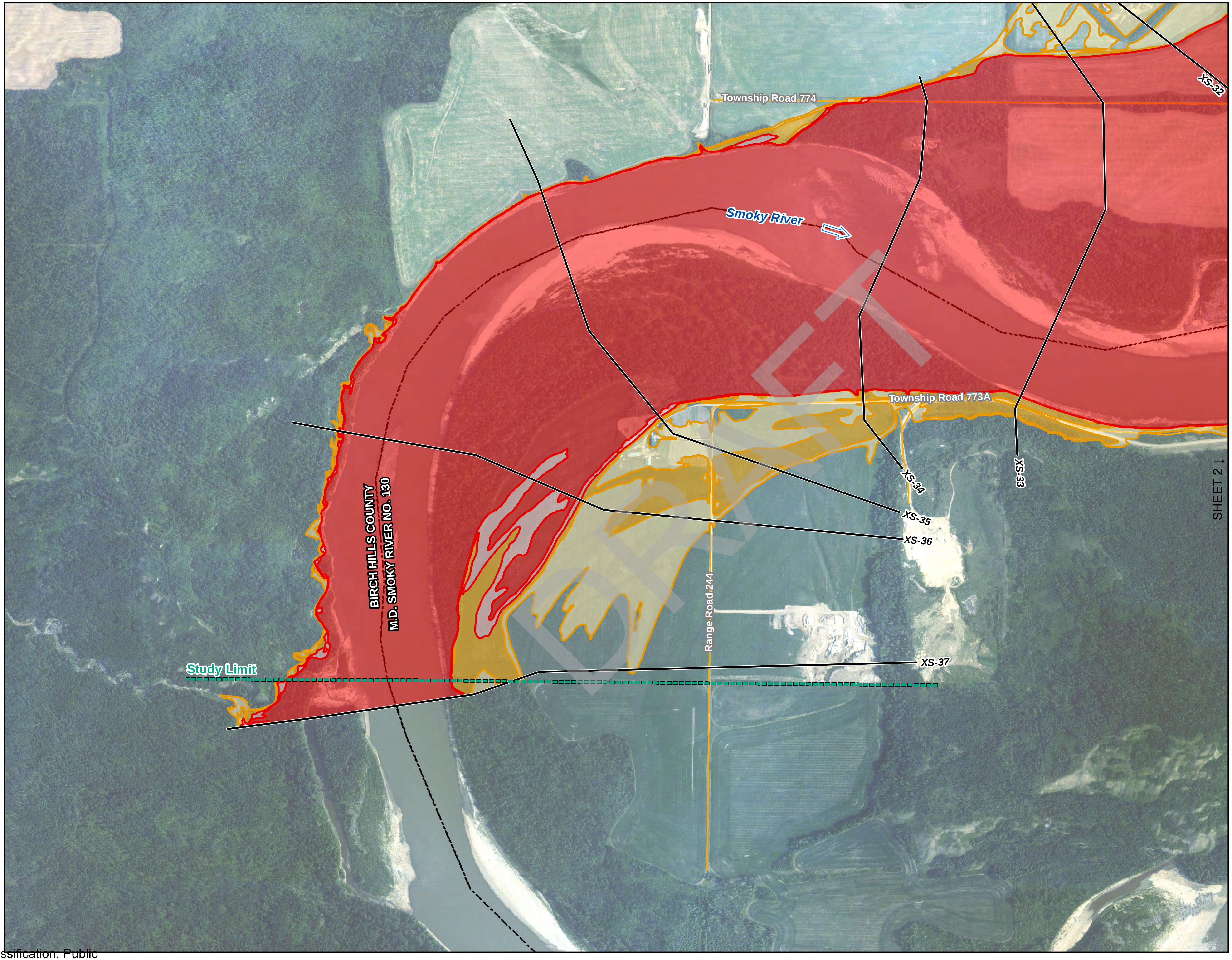
Job Number	*Date*
1008016	*07-JAN-2025*

WATINO FLOOD STUDY

FLOOD HAZARD MAP

INDEX MAP

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard_IndexMap.mxd



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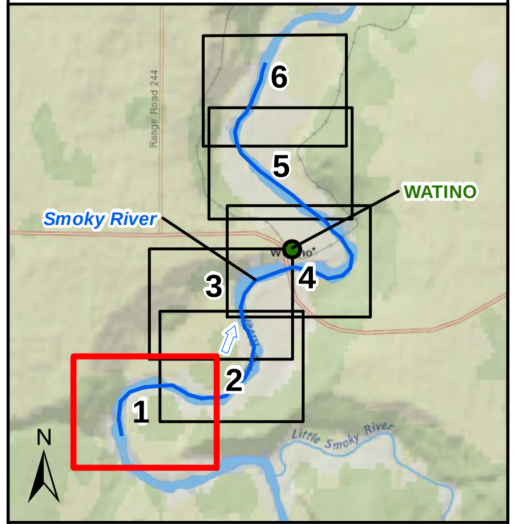

- ⇨ FLOW DIRECTION
- ▬ STUDY LIMIT
- ▬ BRIDGE
- — MODEL CROSS SECTION
- 🟥 FLOODWAY
- ☐ FLOOD FRINGE
- ▨ HIGH HAZARD FLOOD FRINGE
- 🟧 200-YEAR FLOOD EXTENT
- ☐ 500-YEAR FLOOD EXTENT
- — PROVINCIAL HIGHWAY
- — LOCAL ROAD
- ⬚ COUNTY OR MUNICIPAL DISTRICT

DISCHARGE
SMOKY RIVER = 10,700 m^3/s

SCALE - 1:10,000
0 200 400 M
N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

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1008016	*09-MAR-2025*

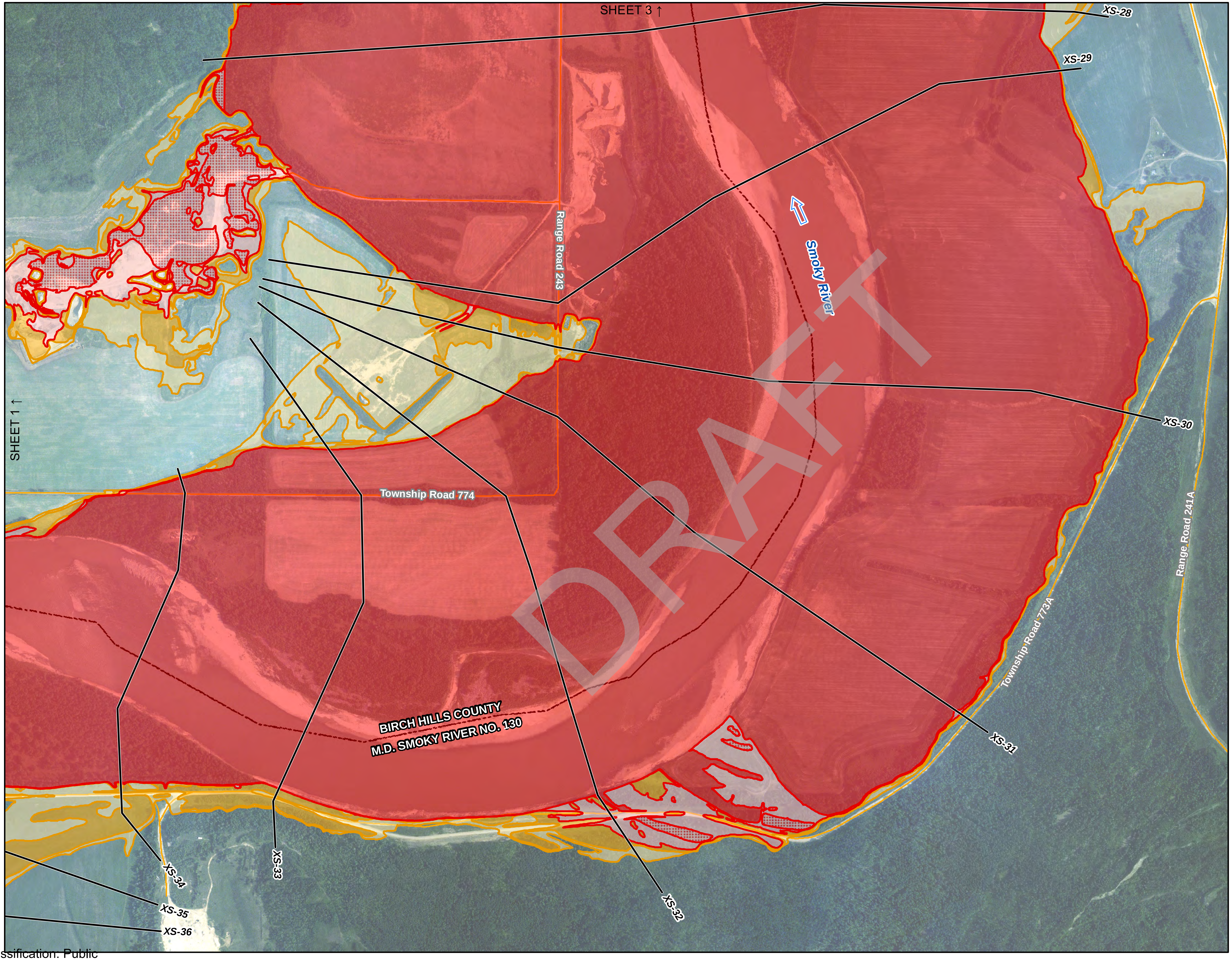
WATINO FLOOD STUDY

FLOOD HAZARD MAP

SHEET 1 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd

Classification: Public



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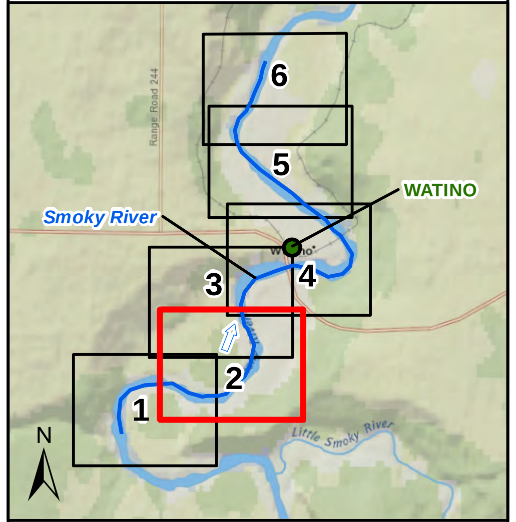

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DISCHARGE
SMOKY RIVER = 10,700 m^3/s

SCALE - 1:10,000
0 200 400
M
N

Coordinate System: NAD 1983 CSRS 3TM 117;
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Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

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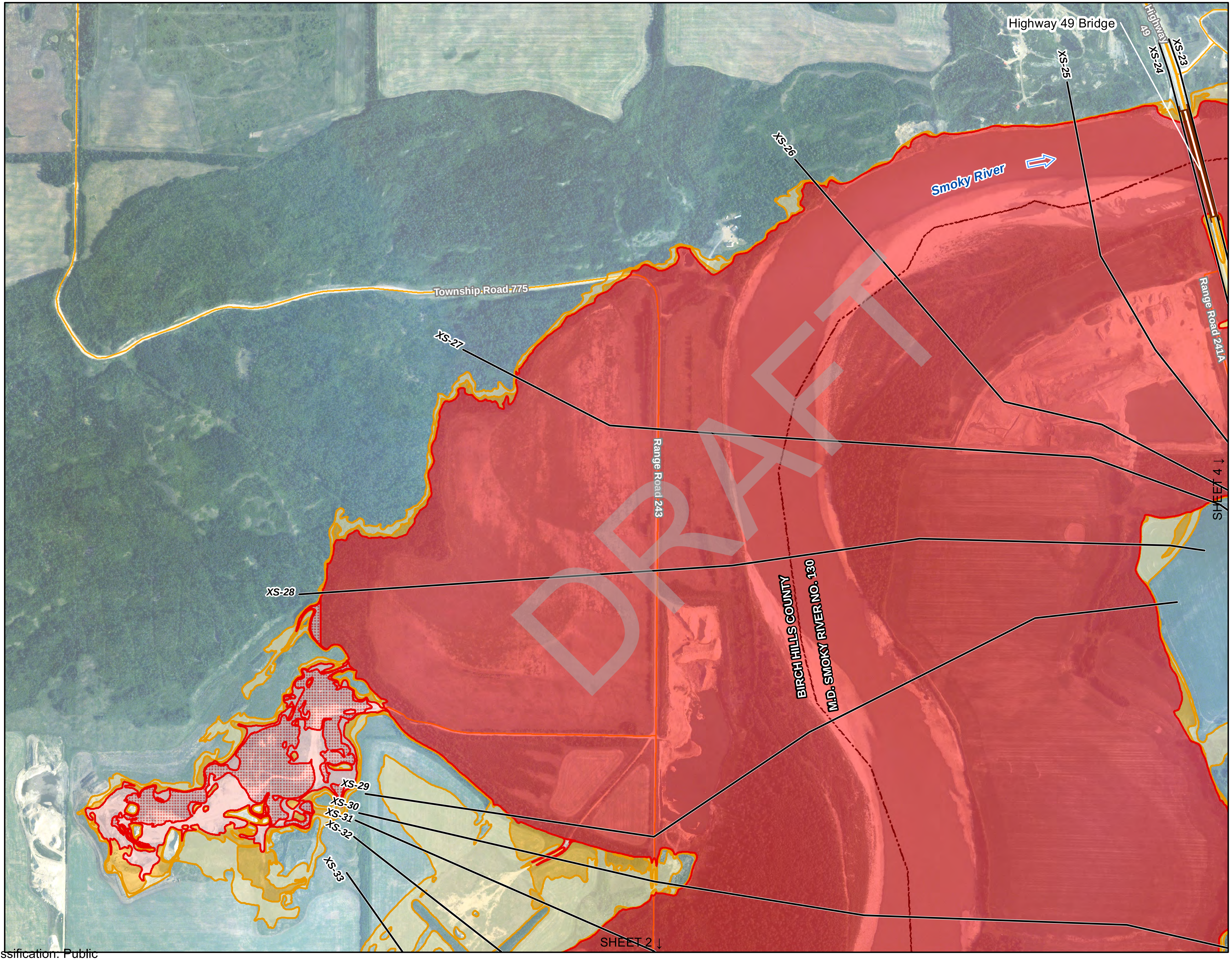
WATINO FLOOD STUDY

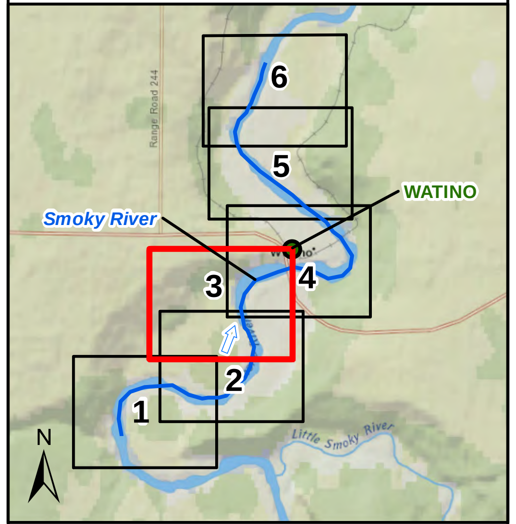
FLOOD HAZARD MAP

SHEET 2 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd

Classification: Public





- FLOW DIRECTION
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DISCHARGE
SMOKY RIVER = 10,700 m^3/s

SCALE - 1:10,000
0 200 400 M
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Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*09-MAR-2025*

WATINO FLOOD STUDY

FLOOD HAZARD MAP

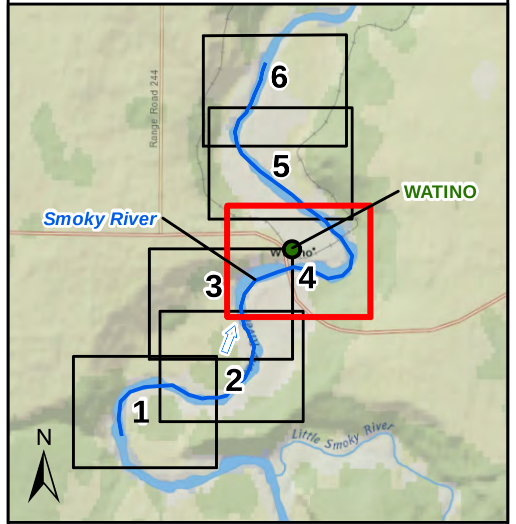
SHEET 3 OF 6

Classification: Public

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd

Alberta Canada

nhc



- ⇨ FLOW DIRECTION
- ▬ ▬ STUDY LIMIT
- ╤ BRIDGE
- — MODEL CROSS SECTION
- 🟥 FLOODWAY
- □ FLOOD FRINGE
- ▨ HIGH HAZARD FLOOD FRINGE
- 🟧 200-YEAR FLOOD EXTENT
- 🟨 500-YEAR FLOOD EXTENT
- — PROVINCIAL HIGHWAY
- — LOCAL ROAD
- ⬚ COUNTY OR MUNICIPAL DISTRICT

DISCHARGE
SMOKY RIVER = 10,700 m^3/s

SCALE - 1:10,000
0 200 400 M
N

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

Engineer	GIS	Reviewer
MMM	*JY*	*RBA*

Job Number	Date
1008016	*09-MAR-2025*

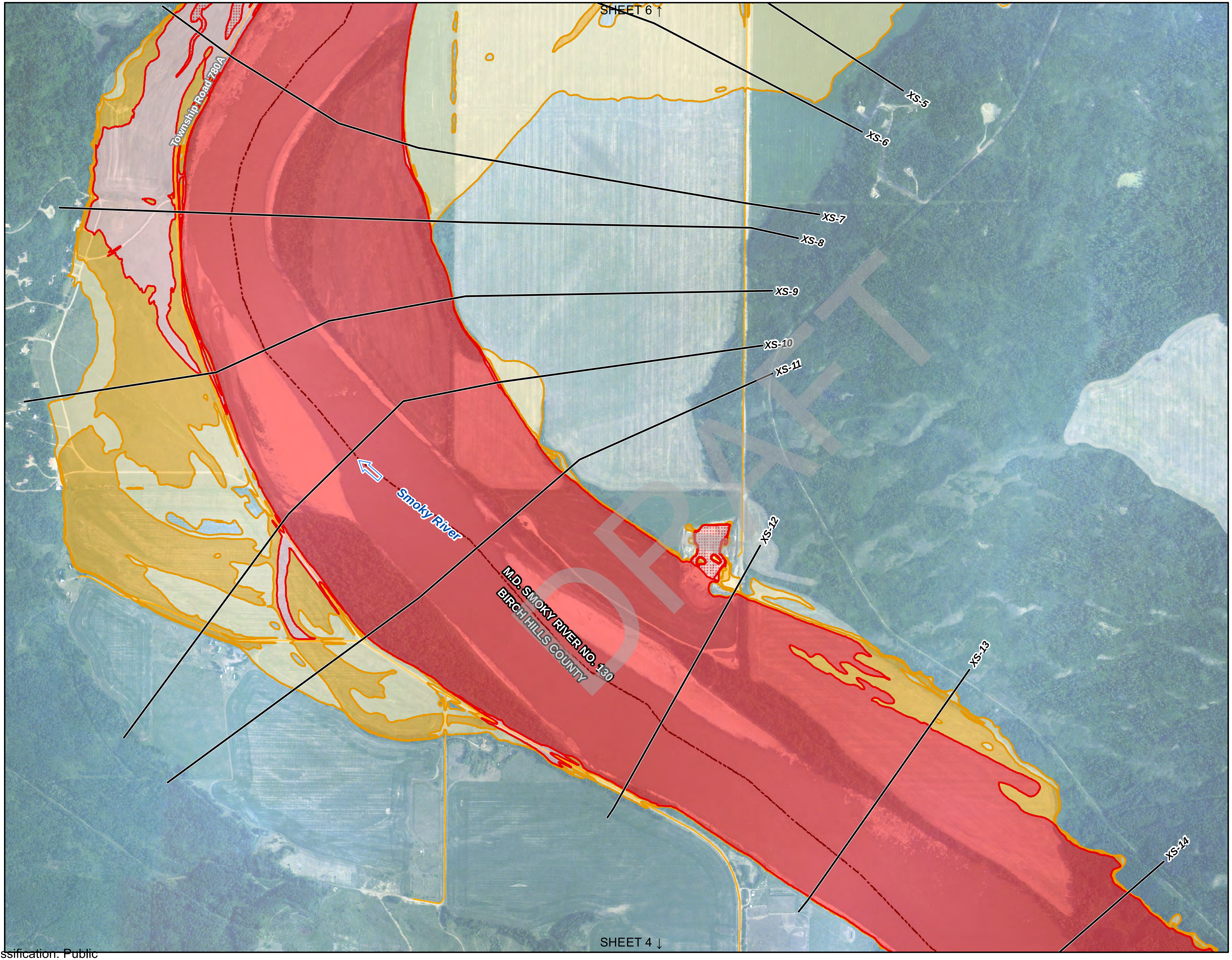
WATINO FLOOD STUDY

FLOOD HAZARD MAP

SHEET 4 OF 6

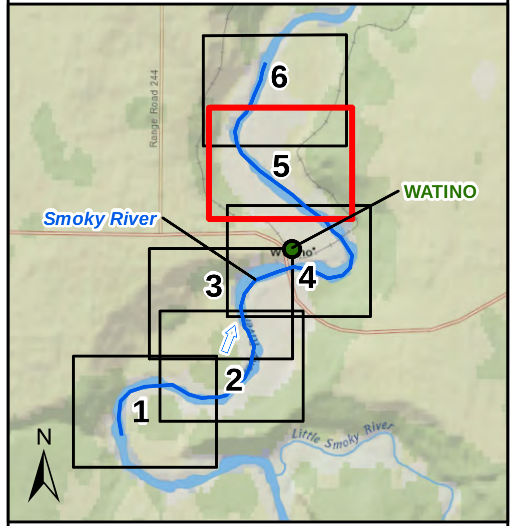
JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd

Classification: Public



Alberta Canada

nhc



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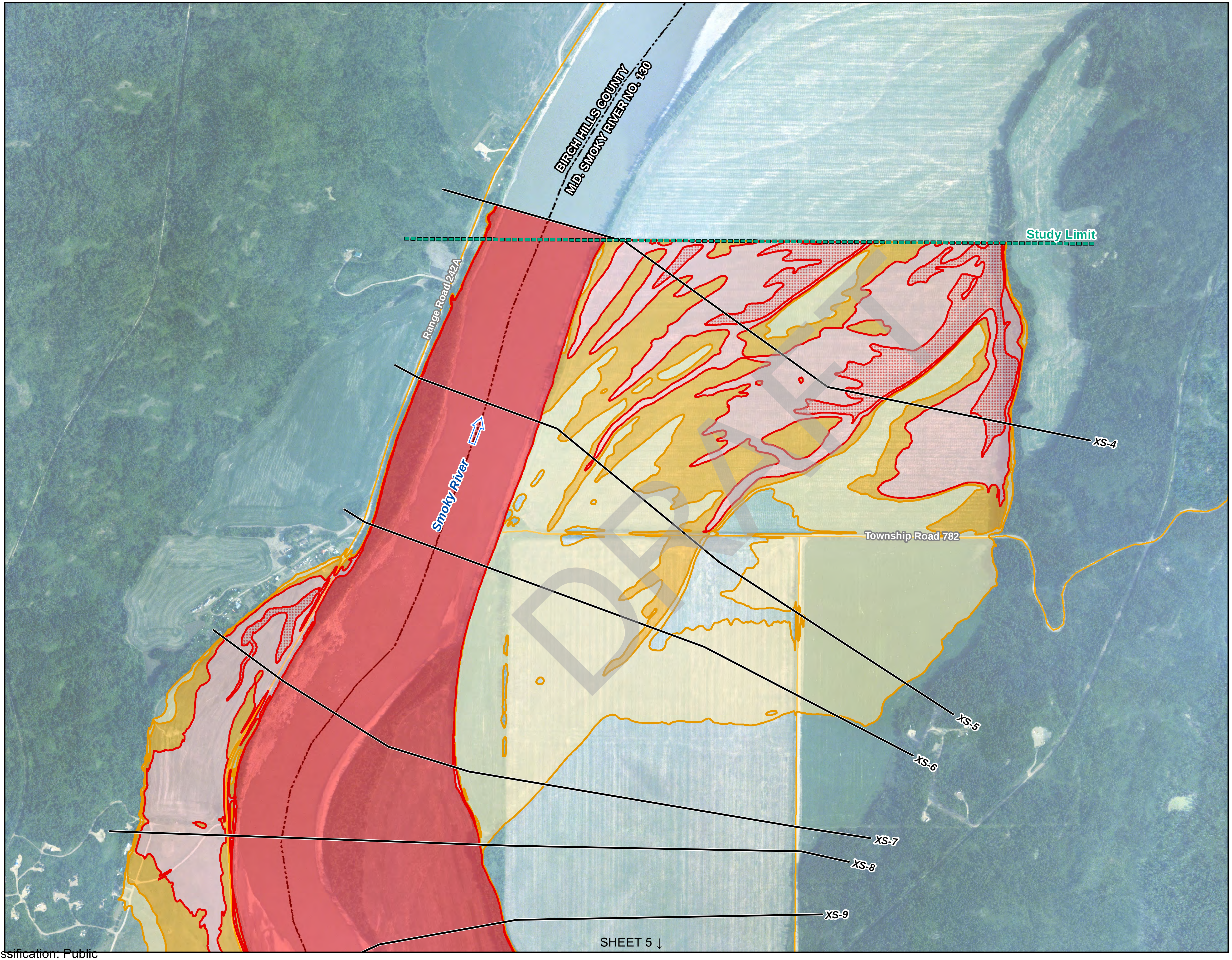
Engineer *MMM*	*GIS* *JY*	*Reviewer* *RBA*
Job Number *1008016*	*Date* *09-MAR-2025*	

WATINO FLOOD STUDY

FLOOD HAZARD MAP

SHEET 5 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd



Classification: Public

Alberta Canada

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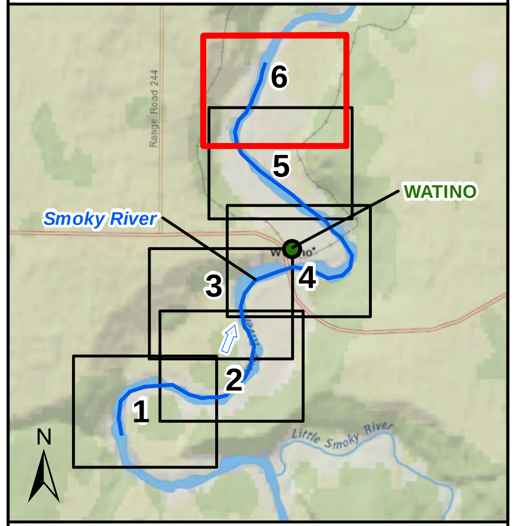

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WATINO FLOOD STUDY

FLOOD HAZARD MAP

SHEET 6 OF 6

JY, P:_Projects (Active)\1008016 Watino Flood Study\90_GIS\1008016_WFS_Map_FloodHazard.mxd