REPORT

Cardston Flood Hazard Study

Submitted to:

Alberta Environment and Parks
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Submitted by:

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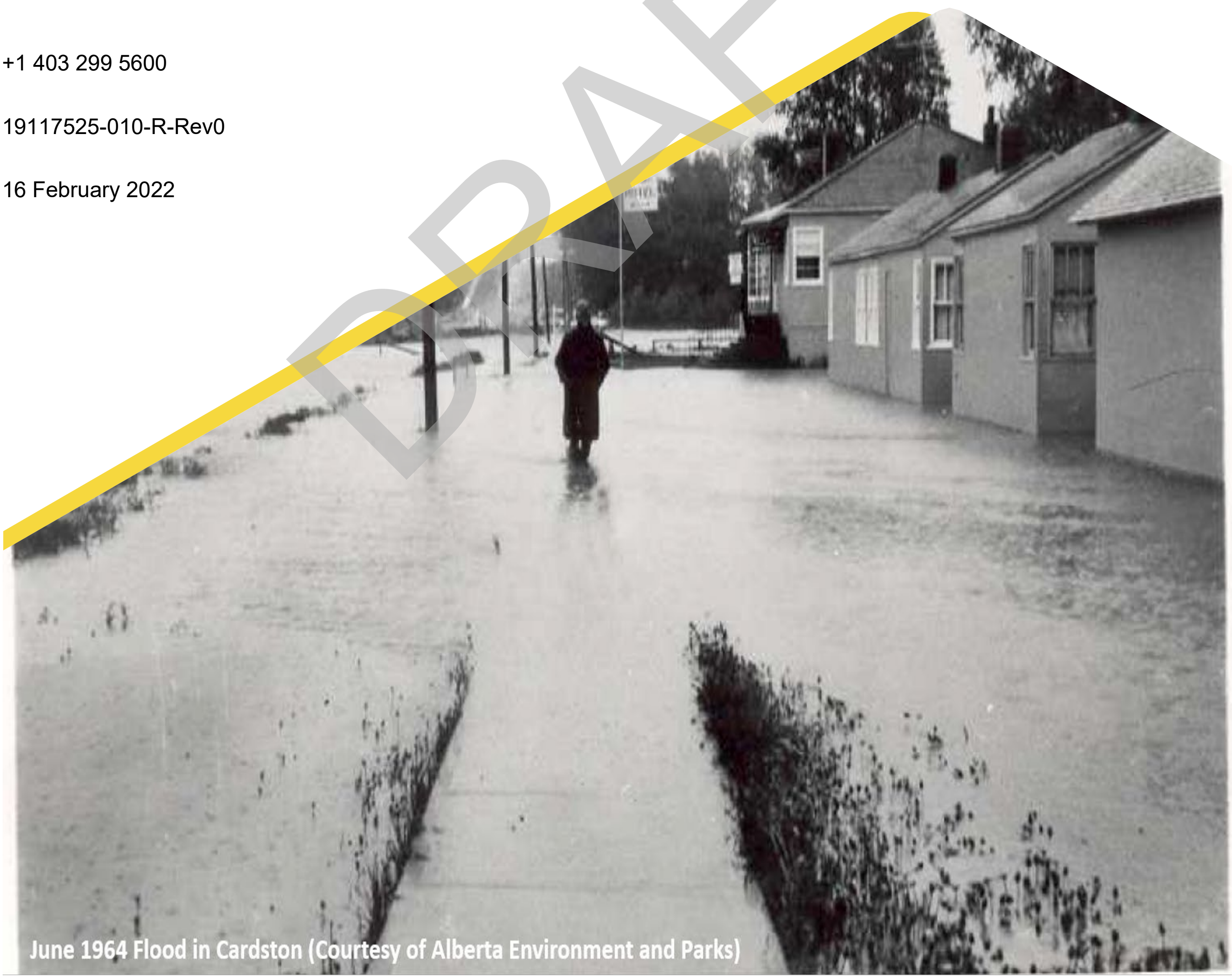
June 1964 Flood in Cardston (Courtesy of Alberta Environment and Parks)

Classification: Public

Executive Summary

Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in March 2019 to conduct the Cardston Flood Hazard Study (the study). The purpose of the study is to assess and identify river and flood hazards along Lee Creek through the Town of Cardston and adjacent areas (see Figure 1). The study is part of the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Cardston, Cardston County, and the general public. The project includes working with Kainai Nation.

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

- field survey
- hydrology assessment
- flood history documentation
- HEC-RAS model creation, calibration, and validation
- open water flood frequency modelling and profile creation
- model sensitivity analysis
- flood inundation mapping
- flood hazard mapping

The total length of Lee Creek study reach is approximately 14 km. A small reach of the St. Mary River of approximately 1 km was included in the HEC-RAS hydraulic model at the downstream end of Lee Creek to enable specification of reliable downstream boundary conditions and to account for backwater effect from the St. Mary River into Lee Creek.

The survey was completed in the spring of 2019. The hydraulic features in this study are summarized in Table i.

Table i: Summary of Survey Features

Feature	Lee Creek	St. Mary River	Total
Cross Sections	106	6	**112**
Bridges	6	1	**7**
Flood Control Structure	1	None	**1**

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

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

- the low flow conditions (i.e., water levels and discharges) measured during the 2019 spring
- the high flow conditions (i.e., high water marks collected by AEP) associated with the 1975, 1991, 1995, 2010 and 2014 flood events on Lee Creek
- the flow-stage rating curve for the Water Survey of Canada (WSC) gauging station - Lee Creek at Cardston (05AE002)

The calibrated Lee Creek channel Manning's *n* values are in the range of 0.025 to 0.040 for flood flow conditions. The calibrated model was used to simulate the water surface profiles for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events in the study area.

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

Flood inundation and hazard maps were prepared for the study reach of Lee Creek using ArcGIS. The simulated flood water levels at the cross sections were used to create a continuous water surface. The edge of inundation was delineated by subtracting the LiDAR DTM from the water surface. The following types of flood inundation were mapped:

- Direct inundation areas where there is a direct connection between the main river channels and inundated areas on the floodplains. This includes areas where inundation is caused by single or multiple topographic or structural overtopping points or backwater flooding.
- Areas of potential flooding behind dedicated flood control structures.

Based on the simulation results, various residential and commercial areas in Cardston would be flooded starting at the 35-year flood on Lee Creek. The full set of open water flood inundation maps is provided in Appendix F.

The floodway was defined based on a mix of the 1 m depth, 1 m/s velocity criteria and the previous floodway. The results of the design flood hazard mapping are the delineation of the floodway and flood fringe zones including high hazard flood fringe areas. Based on the flood hazard maps, the Lion Park is within the high hazard flood fringe zone. Cardston County Emergency Services, Westwind School, Cardston Recreation Centre (swimming pool), Southwest Concrete Products and Co-op Gas Station are within the flood fringe zone. The full sets of floodway criteria and flood hazard maps are provided in Appendix H and Appendix I, respectively.

Acknowledgements

The Cardston Flood Hazard Study was managed by Dr. Wolf Ploeger and Mr. Hossein Kheirkhah Gildeh. Overall direction and senior review were provided by Dr. Dejiang Long. The hydraulic modelling was performed by Dr. Parnian Hosseini with support from Mr. Hossein Kheirkhah Gildeh.

The authors express their special thanks to Mr. Muhammad Durrani, Project Manager for Alberta Environment and Parks (AEP), who provided overall study management, background data, and technical guidance.

The authors also thank Mr. Brandon Jensen from the Town of Cardston for providing additional background information.

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 Study Objectives

Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in March 2019 to conduct the Cardston Flood Hazard Study (the study). The purpose of the study is to assess and identify river and flood hazards along Lee Creek through the Town of Cardston and adjacent areas, including Kainai First Nation (Figure 1). The study is part of the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Cardston, Cardston County, and the general public. The project includes working with Kainai Nation.

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- flood hazard mapping

The total length of Lee Creek study reach is approximately 14 km. A small reach of St. Mary River was included in the model to enable definition of reasonable downstream boundary condition and to account for backwater effect from the St. Mary River into Lee Creek. The total length of the St. Mary River study reach is approximately 1 km. The survey was completed in Spring 2019. Appendix B describes the only flood control structure along Lee Creek.

1.2 Study Reaches

The study area includes a Lee Creek reach of about 14 km, and a St. Mary River reach of about 1 km (see Figure 1). The study reaches are summarized in Table 1.

Table 1: River Reaches in the Study Area

River	Reach Description	Length
Lee Creek	Downstream of Highway 501 to the confluence of Lee Creek with the St. Mary River	14 km
St. Mary River	Approximately 500 m upstream of its confluence with Lee Creek to a location approximately 500 m downstream of the confluence.	1 km

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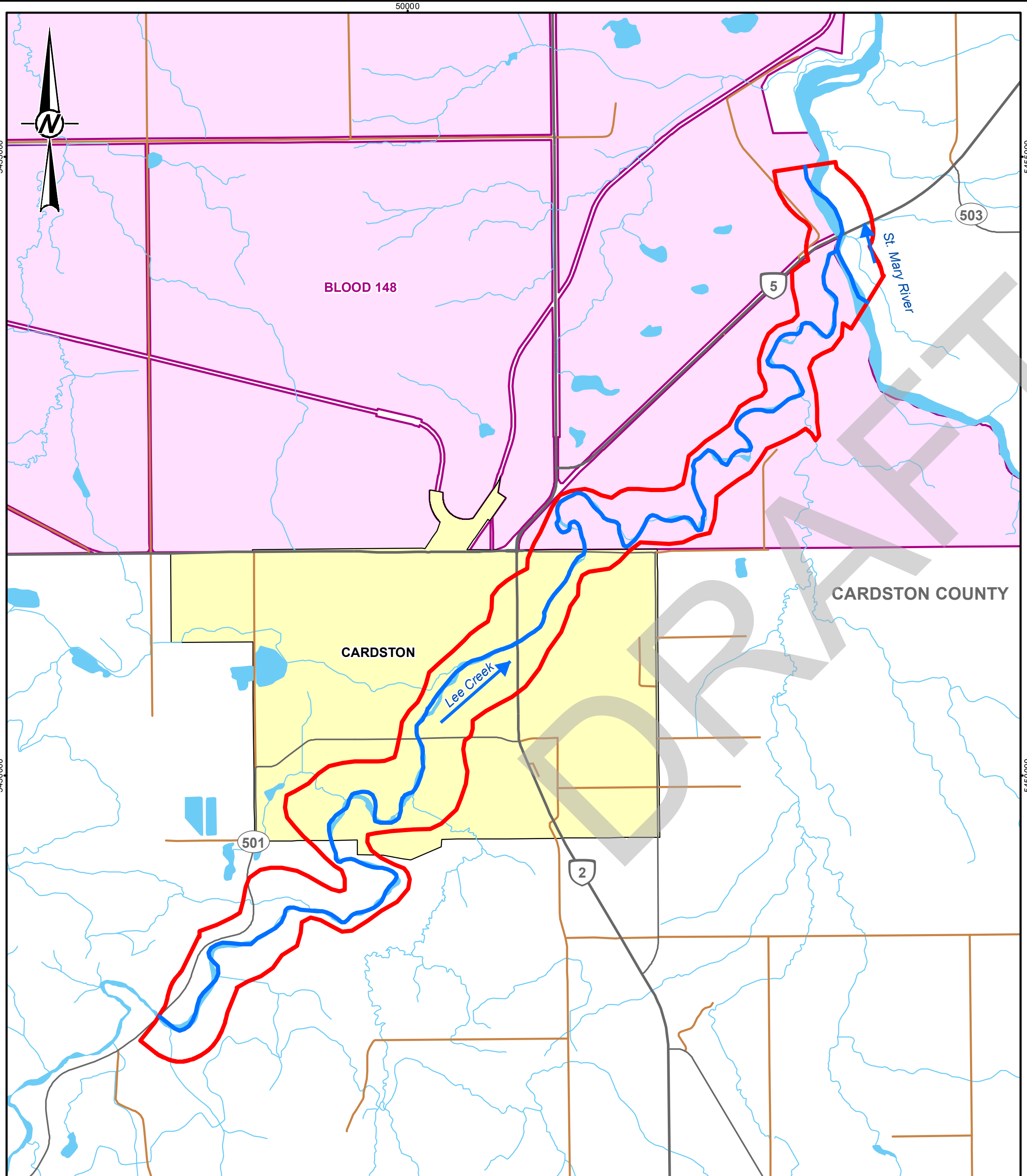

LEGEND

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

0 1 2

1:50,000 KILOMETRES

REFERENCE(S)

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

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
LOCATION MAP OF THE STUDY AREA

CONSULTANT

GOLDER

YYYY-MM-DD	2020-03-27
DESIGNED	PH
PREPARED	BP
REVIEWED	HKG
APPROVED	WP

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Classification: Public

1.3 Work Scope

The scope of the study includes the following:

- Documentation of Flooding History
- Summary of Available Data
- Documentation of River and Valley Features
- Model Setup
- Model Calibration
- Generation of Open-Water Flood Frequency Profiles
- Model Sensitivity Analysis
- Open Water Flood Inundation Mapping
- Open Water Flood Hazard Mapping

2.0 FLOODING HISTORY

2.1 General Information

Lee Creek originates in Montana, U.S.A. It flows in a north-easterly direction, passes through the Town of Cardston and then joins the St. Mary River which also originates in Montana, U.S.A. The Lee Creek drainage basin is mostly located in south-western Alberta with a small part of the upper basin in Montana. The drainage area of Lee Creek upstream of Cardston comprises mountainous terrain in the west, heavily forested foothills, parklands, and cultivated prairie. Lee Creek has a basin area of approximately 316 km^2 at the gauging station in Cardston (i.e., Lee Creek at Cardston, WSC Station 05AE002).

Lee Creek channel has a bed slope of approximately 0.004 m/m along the study reach. The St. Mary River has a bed slope of approximately 0.003 m/m at and near the Lee Creek confluence.

The recorded flow data available from the WSC website for Lee Creek is for one location (i.e., Lee Creek at Cardston) within the study area. The preliminary annual maximum instantaneous discharge data for this location was obtained from WSC for 2016, 2017 and 2018.

2.2 Historic Floods

The available records indicate that major flood events occurred on Lee Creek in 1948, 1951, 1953, 1964, 1975, 1981, 1995, 2002, 2005 and 2010. These floods were typically associated with high rainfall or rain-on snow events in June, except for the flood event on May 22, 1981. The 1964 flood had a return period of approximately 100 years, and this flood caused considerable damage in the community.

Recent flooding occurred in 2014. During that event a local state of emergency was declared and the low-lying areas along Lee Creek were under evacuation warning.

Historic flood flows for Lee Creek and the St. Mary River upstream of the study area are listed in Table 2.

Table 2: Historic Recorded Flood Flows

Year	Lee Creek (m^3/s)	St. Mary River Upstream of the Study Area (m^3/s)	Source
2014	117	241	WSC & Golder (2020)
2010	245	160	WSC & Golder (2020)
1995	303	430	WSC & Golder (2020)
1991	59	168	WSC & Golder (2020)
1975	226	660	WSC & Golder (2020)
1964	323	595	WSC & Stanley (1992)
1953	170	328	WSC & Stanley (1992)
1951	221	261	WSC & Stanley (1992)

Note: WSC = Water Survey of Canada; Golder = Golder Associates Ltd.; Stanley = Stanley Associates Engineering Ltd.

2.3 Recent Floods

The most recent flood in Cardston occurred in June 2014 when the Town declared a state of local emergency. The flood was caused by snowmelt and heavy rain. Residents were warned to prepare for possible evacuations, farmers were advised to move their livestock, and people were informed to stay away from riverbanks. Based on witnesses in the area, the rise of water occurred in a short period of time. However, no major damage was recorded during the 2014 flood.

2.4 Ice Jam Floods

Based on a review of the available documents, it is apparent that ice jams are not a significant source of flooding within the study area.

3.0 AVAILABLE DOCUMENTS AND DATA

3.1 Hydrology Summary

The flood frequency estimates for Lee Creek and the St. Mary River are documented in Appendix A. The flood flow frequency estimates at key locations in the study area are summarized in Table 3.

Table 3: Estimated Flood Peak Discharges and their 95% Confidence Interval

Return Period (years)	Lee Creek at Cardston (WSC Station No. 05AE002)			St. Mary River at Highway 501 (WSC Station No. 05AE043)			St. Mary River above the Lee Creek Confluence[1]		
	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)
2	23	28	18	108	120	98	141	156	128
5	62	80	47	176	201	154	230	263	201
10	105	145	75	240	288	194	314	377	253
20	163	241	111	321	411	231	419	537	301
35	223	352	148	402	548	260	525	715	340
50	269	440	174	463	656	280	605	856	366
75	328	559	206	543	814	303	709	1,060	396
100	375	658	232	608	941	321	793	1,230	420
200	510	967	303	794	1350	362	1,040	1,760	473
350	642	1,290	370	984	1790	396	1,280	2,340	516
500	739	1,540	418	1,127	2,160	418	1,470	2,820	546
750	862	1,870	476	1,314	2,660	443	1,710	3,470	578
1,000	959	2,130	522	1,465	3,090	462	1,910	4,030	603

Note: 1.Prorated from St. Mary River at Highway 501 based on the ratio of effective drainage areas.

3.2 DTM Data

The detailed Digital Terrain Model (DTM) for the study area was provided by AEP. It was developed from a 2019 LiDAR survey and is available as gridded raster with 0.5 m resolution, ESRI Terrain and triangulated irregular network (TIN). The DTM was delivered in the local study coordinate system and datum (3TM 114°, NAD83 CSRS).

3.3 Survey Data

The survey of the stream cross sections, hydraulic structures, and flood control structure within the study area was conducted between May 14 and June 13, 2019. Water levels measured at individual cross sections and discharge measured at a suitable location were used to support the hydraulic model calibration.

3.4 Procedures and Methodology

3.4.1 Topographic, Bathymetric and Structure Surveys

The following survey equipment were used to collect the topographic, bathymetric, and structure data for this study:

a) **Real-time Kinematic (RTK) GPS** – Trimble R8® and R10® RTK units were used to survey ground features and river bed levels in the areas where hydraulic conditions allowed the surveyors to wade the channel. The RTK units were also used to survey the control points and benchmarks found within the study area.

b) **Acoustic Doppler Profiler (ADP)** – A SonTek RiverSurveyor M9® was used in combination with a boat-mounted RTK unit to survey the St. Mary River.

c) **Remotely Operated Vehicle (ROV) with SonarMite Echo Sounder** – A Seafloor SonarMite echo sounder was used on a Seafloor Hydrone remotely operated vehicle in combination with an RTK unit to survey the river bed in the areas where water was too deep or too fast flowing to wade.

All survey data collected in this study was referenced to the Alberta Survey Control Network using Alberta Survey Control Markers (ASCMs). An RTK base station was set up over temporary benchmarks at various locations and calibrated to an ASCM that was close to the study reach or a Golder-established temporary benchmark that had been tied to an ASCM.

The survey data was acquired by RTK rover units with pre-loaded geoid files. The RTK data output for this study provides an orthometric elevation with correct northing and easting coordinates. All survey data was collected in the 3TM coordinate system with the Meridian at 114° W and referenced to NAD83 (CSRS) horizontal and CGVD28 vertical datum. Ellipsoidal heights are transformed to CGVD28 orthometric heights using the HTv2.0 geoid model. Survey data collected on the St. Mary River using the ADP/RTK combination was collected in UTM coordinates and projected into the 3TM 114° coordinate system.

Each survey point collected using the RTK utilized a schematic of survey point codes and corresponding locations as shown in Figure 2, which also includes a complete list of survey codes for the RTK.

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

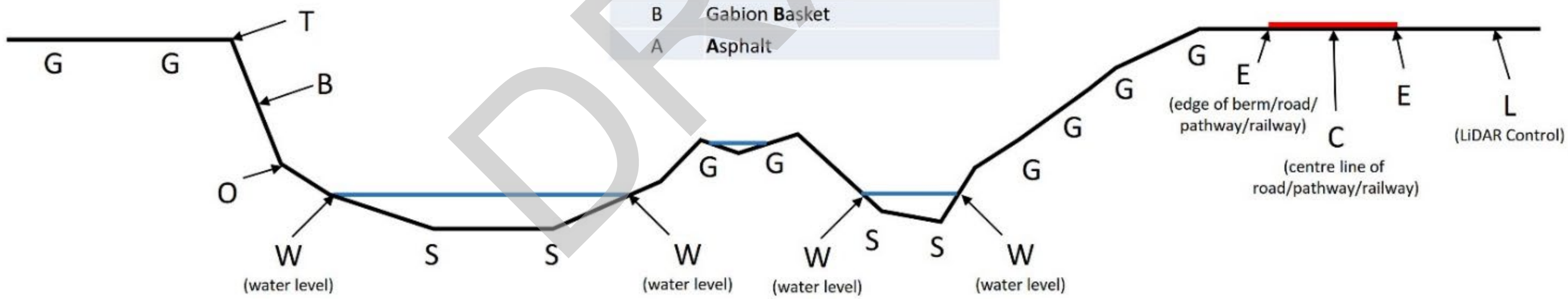
Survey Codes for RTK GPS River Surveys (No Structures)

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

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

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

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



Version 0, August 25, 2017 Golder Associates Ltd. Wolf Ploeger

Figure 2: Schematic of Survey Point Locations and Code Descriptions

Channel Cross Section Surveys

The field data was collected by surveying channel cross sections approximately perpendicular to the direction of the flow. The study reach within the St. Mary River was surveyed by boat. Most of the cross sections on Lee Creek were surveyed by wading. For some cross sections on Lee Creek where the water was too deep to wade, a remotely operated vehicle (ROV) was used.

The following procedures were applied when carrying out a bathymetric survey by wading:

- Set up the RTK-GPS base station since Can-net coverage was not available for the study area or did not provide sufficient accuracy.
- RTK rover units were used to collect cross-sectional information from a location approximately 2 to 5 m beyond the top of bank on one side of the channel to a location approximately 2 to 5 m beyond the top of bank on the other side. A minimum of 20 points were established across the channel and care was taken to reference points where the transverse bed slope changed significantly.
- Special attention was paid to surveying topographic slope breaks along the banks.
- All surveyed data points were attributed with field codes that described substrate and vegetation types.
- The water surface elevation was surveyed where the water had contact with the banks.
- The following procedure was applied when the ROV was used for deeper areas of Lee Creek:
- Set up the RTK-GPS base station since Can-net coverage was not available for the study area or did not provide sufficient accuracy.
- Mount the Sonarmite onto the frame on the Hydrone ROV.
- Place the RTK-GPS unit on top of the Sonarmite mount and measure the offset to the water surface.
- Connect both Sonarmite and RTK-GPS units to a data collector with Bluetooth transmission capability and use a field laptop or Trimble data collector for data collection.
- For each day when the ROV was used, a calibration was performed to correct the water depth measurements. This was conducted by placing the ROV over a relatively flat river bed, measuring the water depth and surveying the same point with the RTK unit. The elevation correction was then applied in the office.
- The boat survey method for the St. Mary River involved the following:
- The ADP was mounted onto a frame, which was fastened to the side of the river boat. Once the ADP was securely mounted on the boat, it was deployed in the water and the distance from the middle sensor to the water surface was measured using a standard tape measure.
- The RTK unit was attached to the top of the ADP mount at a measured offset from the water surface. This offset was measured and recorded on a daily basis.
- The ADP and RTK units were connected to a laptop data acquisition system that provided data storage and a real-time display of the position and data being collected. The RiverSurveyor software installed on the computer used on the boat was checked to make sure that both units were communicating properly, and data was being stored.

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

Hydraulic Structures

Hydraulic structures within the study area that could affect channel conveyance and water levels include two highway bridges, two road bridges and two pedestrian bridges on Lee Creek, and one highway bridge over the St. Mary River. The features of the bridges that were surveyed include the following:

- length of span (corner points, abutment-to-abutment)
- width of bridge (corner points, outside-to-outside)
- top of curb or solid guard rail elevations
- low chord elevations
- number and width of piers
- location of piers and the distance of each pier relative to the abutment
- type of piers (e.g., concrete, pile bent)
- shape of pier (e.g., round nose, wedge-shaped, circular)
- top of roadway (or path) profile

The hydraulic structures were surveyed using RTK-GPS and measuring tape. Geo-located photos of each structure were taken during the survey.

Flood Control Structures

There is one dedicated flood control structure located along Lee Creek, which was surveyed using an RTK-GPS to verify as-built elevations and to characterize its typical cross-sectional geometry. Survey data was collected along the crest of the flood control structure. Appendix B summarizes the characteristics of the flood control structure on Lee Creek.

3.4.2 Flow and Water Level Measurements

Water levels along the study reaches were measured to support the low-flow hydraulic model calibration.

One discharge along Lee Creek was measured to provide a check on the provisional data obtained from the online database of Water Survey of Canada (WSC).

Flow measurement was performed by wading the channel with a handheld Acoustic Doppler Velocimeter (*SonTek FlowTracker2® ADV*) and top-set wading rod in accordance with standard WSC protocols. This includes: (i) selecting a suitable measurement location; (ii) choosing an even number of transects with equal left bank to right transects and right bank to left transects; and (iii) ensuring that the data set of each transect is within a maximum standard deviation of five percent. The measurement procedure involved the following:

- Survey points were selected to result in a minimum of 20 panels (flow segments across the stream thus requiring a minimum of 21 velocity measurement points).
- Velocity readings were taken at 0.6 of the total depth at measurement locations since flow depth was less than 1.0 m in all cases.
- Survey points were selected such that no panel discharge exceeded 10 percent of the total discharge, six panels were within the 5 to 10 percent range and the remaining 17 panels were all less than five percent.
- The measured discharge at Lee Creek is listed in Table 4.

Table 4: Comparison between the Discharge Measurement and WSC Data

Stream	Date	Location	Discharge (m^3/s)		Difference between Golder and WSC		WSC Gauging Station
			Measured by Golder	WSC Gauge(1)	(m^3/s)	(%)	
Lee Creek	June 13, 2019	XS 63	3.22	3.05	0.22	7	05AE002 Lee Creek at Cardston

Note: 1) WSC discharge data was preliminary and subject to changes.

3.5 Cross Sections

The total length of the Lee Creek study reach is approximately 14 km. The length of the St. Mary River study reach is approximately 1 km. A summary of the surveyed channel cross sections is provided in Table 5.

Table 5: Surveyed Channel Cross Sections within the Study Area

Waterbody	Reach Description	Cross Section ID	Total Number of Cross Sections	Average Cross Section Spacing (m)	Year of Survey
Lee Creek	Downstream of Highway 501 to the confluence of Lee Creek with the St. Mary River	XS1 to XS106	106	134	2019
St. Mary River	Approximately 500 m upstream of its confluence with Lee Creek to a location approximately 500 m downstream of the confluence.	XS107 to XS112	6	135	2019

3.5.1 Thalweg and Cross Section Comparison

The surveyed thalweg and three cross sections along Lee Creek were compared between the current study and the 1992 study to characterize the changes in the channel bed elevations and overall channel/floodplain flow conveyance. As shown in Figure 3 to Figure 6, upstream of the Main Street Bridge the thalweg elevations were generally higher in the 1992 model (Stanley 1992) than those of the 2019 model by about 0.2 m on average. The thalweg elevations downstream of Main Street Bridge were generally lower in the 1992 model than the 2019 model by about 0.2 m on average.

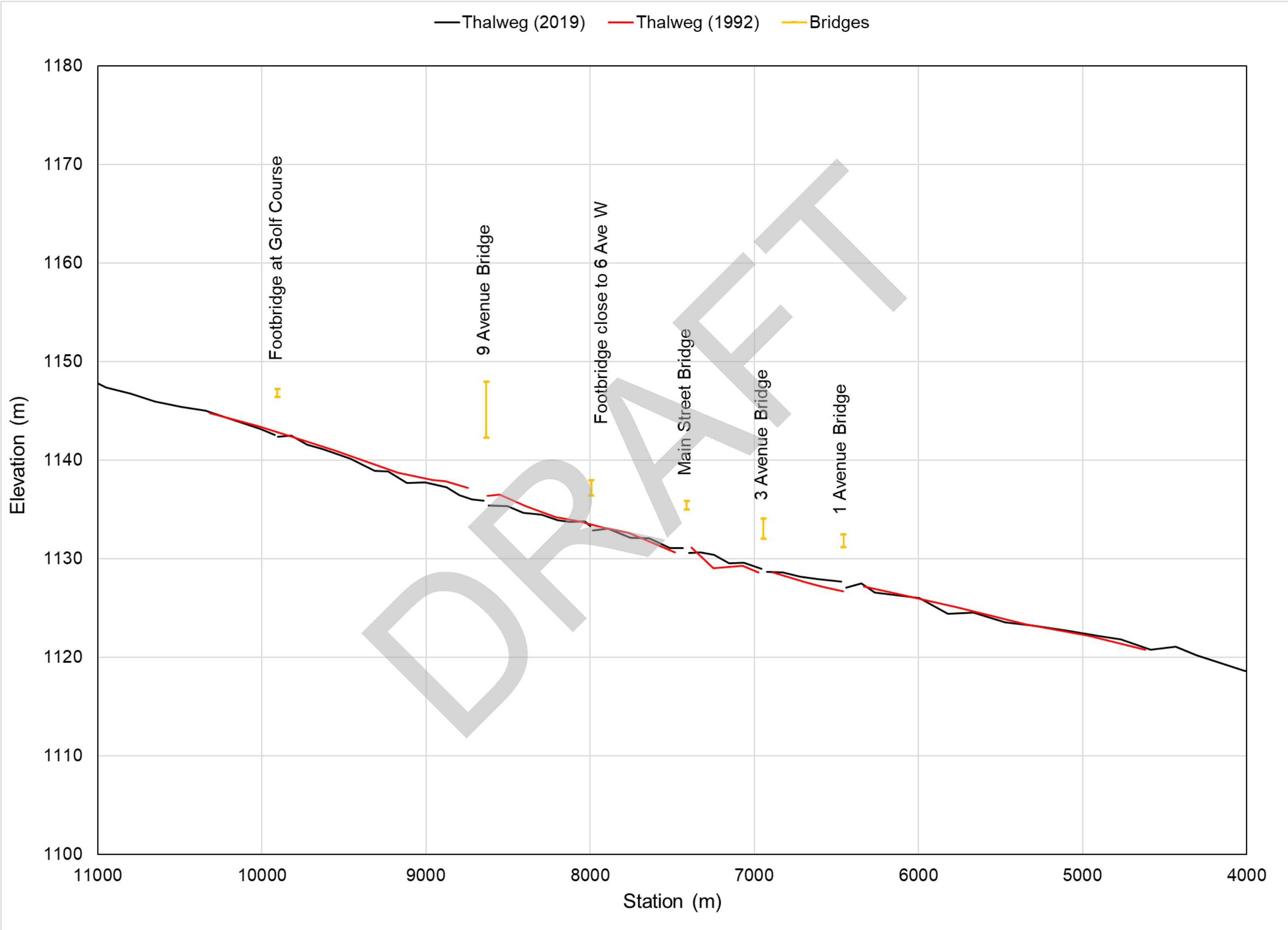


Figure 3: Comparison of the Channel Thalweg Elevations in the 1992 and 2019 Models

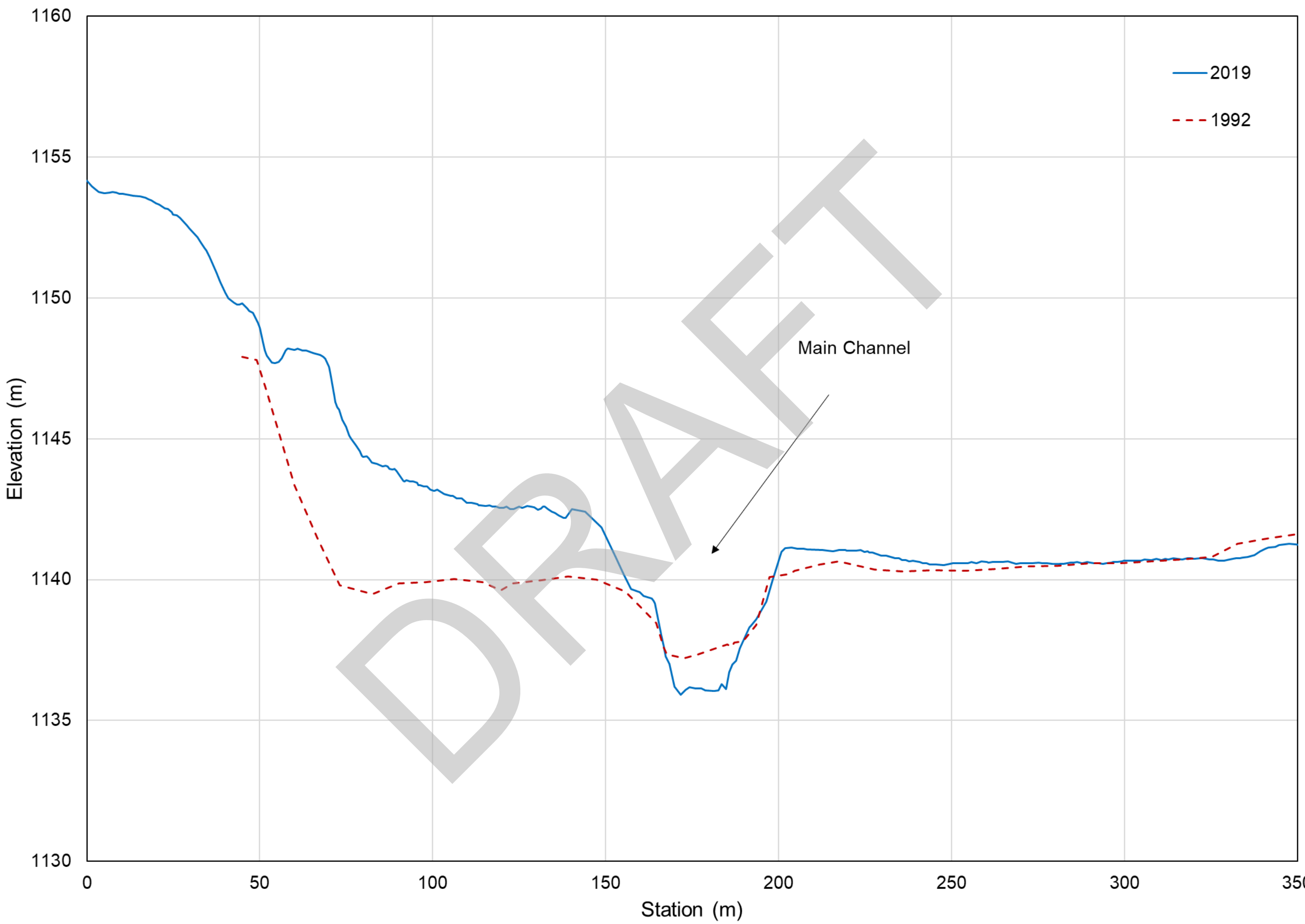


Figure 4: Comparison of Cross Sections at River Station 8,649 m (1.2 km Upstream of Main Street Bridge) between the 1992 and 2019 Studies

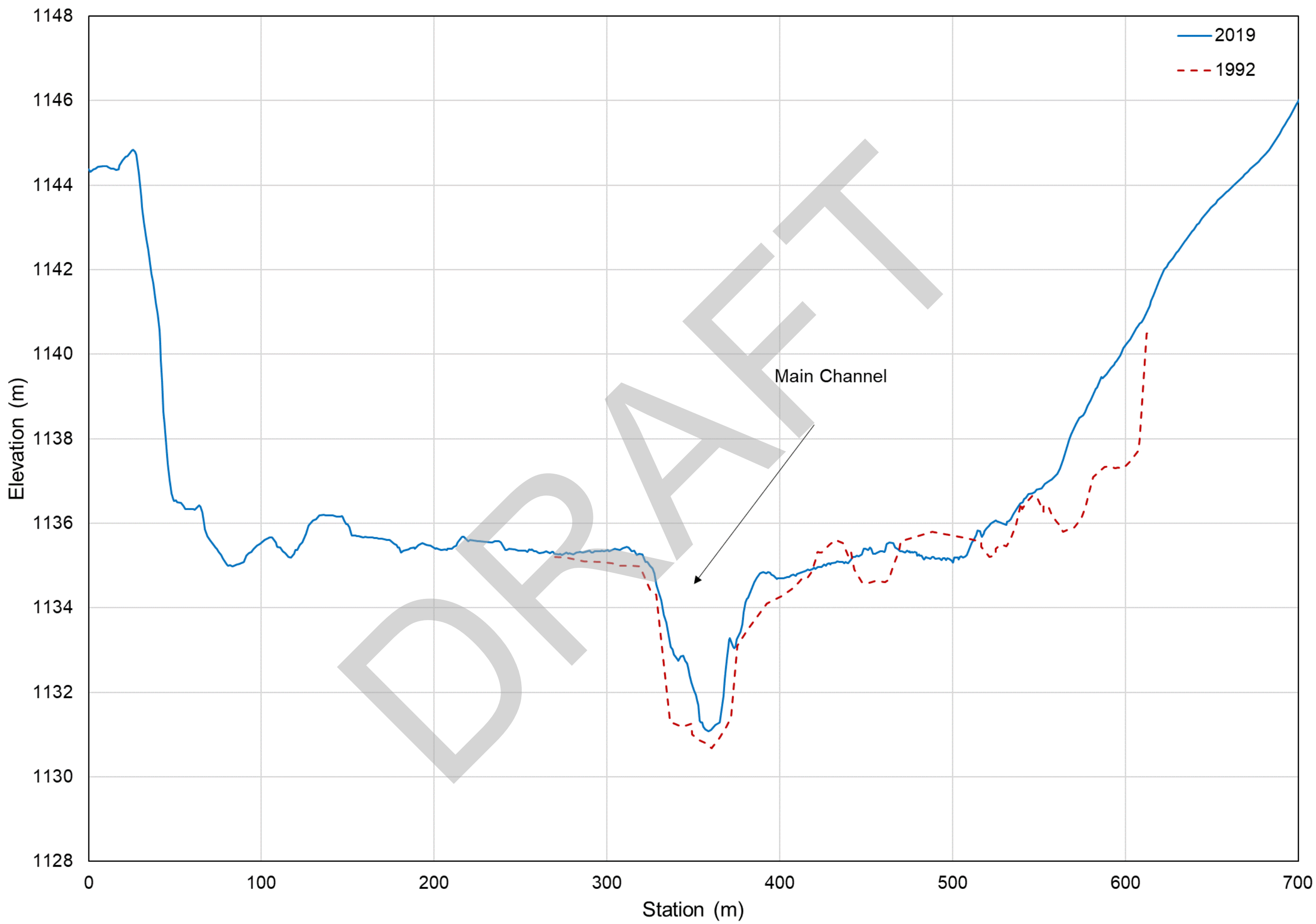


Figure 5: Comparison of Cross Sections at River Station 7,434 m (22 m Upstream of Main Street Bridge) between the 1992 and 2019 Studies

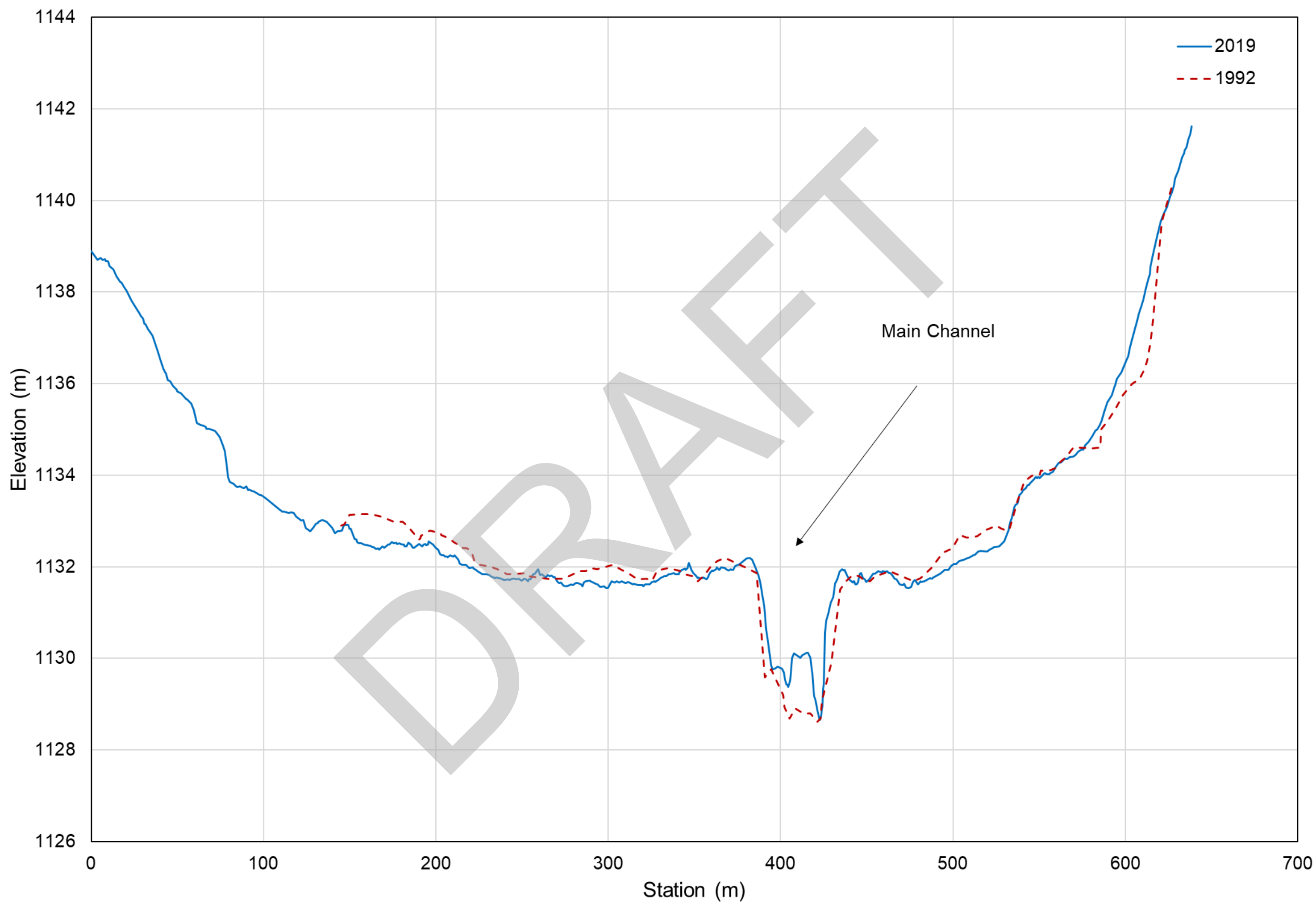


Figure 6: Comparison of Cross Sections at River Station 6925 m (0.5 km Downstream of Main Street Bridge) between the 1992 and 2019 Studies

3.6 Existing Models

The existing hydraulic model for the study area is listed in Table 6.

Table 6: Existing Hydraulic Model for the Study Area

No.	Report	Program Used	Date	Author
1	Cardston Hydraulic Study	HEC-2	1992	Alberta Environment and Parks

3.7 Highwater marks

The available high-water mark reports and data for open water flooding are listed in Table 7.

Table 7: Available High-Water Mark Reports and Data

No.	Report	Flood Event	Author
1	Highwater Mark Report for Lee Creek and St. Mary River	1975	Alberta Environment and Parks
2	Highwater Mark Report for Lee Creek	1991	Alberta Environment and Parks
3	Highwater mark Report for Lee Creek	1995	Alberta Environment and Parks
4	Highwater mark Report for Lee Creek	2010	Alberta Environment and Parks
5	Highwater mark Report for Lee Creek	2014	Alberta Environment and Parks

3.8 Gauging Station Data and Rating Curves

The active Water Survey of Canada (WSC) gauging station located within the study area is Lee Creek at Cardston (Station No. 05AE002).

3.9 Flood Photography

AEP commissioned post-flood (June 21, 2014) aerial photography along Lee Creek as part of the June 18, 2014 flood event documentation efforts. Although the near-peak aerial flood photography for Lee Creek was not captured until almost three days after the flood peak, it still provided insightful information about the event. Site flood photographs were taken as part of the AEP highwater mark surveys. Table 8 lists the available flood photography of open water flooding in the study area.

Table 8: Available Flood Photography of Open Water Flooding in the Study Area

No.	Description	Flood Event	Source
1	Photographs of Lee Creek taken during and after the 1964 flood	1964	Alberta Environment and Parks
2	Photographs of Lee Creek taken during and after the 1975 flood	1975	Alberta Environment and Parks
3	Photographs of Lee Creek taken during and after the 1991 flood	1991	Alberta Environment and Parks
4	Photographs of Lee Creek taken during and after the 1995 flood	1995	Alberta Environment and Parks
5	Photographs of Lee Creek taken during and after the 2010 flood	2010	Alberta Environment and Parks
6	Photographs of Lee Creek taken during and after the 2014 flood	2014	Alberta Environment and Parks
7	Aerial imagery captured approximately three days after the June 18, 2014 flood	2014	Alberta Environment and Parks

3.10 Aerial Imagery

AEP provided the recent aerial imagery (obtained in July 2019) for the Cardston Flood Hazard Study that was used for preparing the flood inundation as well as flood criteria and hazard maps.

4.0 RIVER AND VALLEY FEATURES

4.1 General Description

Terrain along the Lee Creek valley is composed of plains, including cultivated land and pasture (Stanley, 1992). Lee Creek, an alluvial stream, is composed of erodible soils which was deposited by the creek itself. The sediment load in the creek has the same material as the channel bed and banks.

Lee Creek has a small catchment which produces flash floods in high-intensity rainstorm events. This type of flashy runoff has a high potential for sediment removal and transportation. This may explain the changes in channel bed elevation and overall channel/floodplain flow conveyance between the 1992 and 2019 studies as compared in Section 3.5.1.

4.2 Channel and Floodplain Characteristics

Channel Characteristics

The Lee Creek channel along the study reach is situated in a valley, which is mostly confined by high terraces, while there is enough floodplain for the channel to alternate the pattern (Stanley, 1992). The Lee Creek main channel in the study area includes straight and meandered reaches. The reach of the channel that passes through the Town of Cardston is relatively straight, while the channel reaches upstream and downstream of Cardston are meandering.

Some in-channel bars and side bars are observed along the study reach that are sometimes vegetated. Based on the site reconnaissance and comparison of the channel cross sections between 1992 and 2019 surveys, the stream bed has been mobile, with some deposition and erosion occurring in various channel reaches in the study area during higher flows. This suggests that Lee Creek as an alluvial stream is affected differently during low and high flow events, including its bed forms, meanders, and other geometric parameters.

Channel bed materials vary along the study reach. In the Town area it is mostly a silty-sand bed with some boulders. More boulders are present in some areas where the channel is wider. In some areas, channel banks are unstable, and there is localized erosion. There is high vegetation along both banks of Lee Creek.

Some improvements were performed on Lee Creek in Cardston in 1983, including deepening and widening of the creek channel, construction of embankments, and a sheet pile dike (Stanley, 1992), which is the only dedicated flood control structure in the study area.

Floodplain Characteristics

Most vegetation on the Lee Creek floodplains consists of pasture. There are brushes, some trees and shrubs along the Lee Creek channel banks and partly on the floodplains. There are farmlands on the floodplains of Lee Creek, and a golf course (i.e., Lee Creek Valley Golf Course) on the left floodplain of Lee Creek upstream of 9 Avenue West.

4.3 Bridges, Culverts and Weirs

There are six bridges along Lee Creek study reach and one bridge along the St. Mary River study reach (see Table 9).

Table 9: List of Bridges within the Study Area

No.	Water Body	Location	Name / Identifier	River Station (m)	Type
1	Lee Creek	Footbridge at Golf Course	Footbridge Close to Lee Creek Valley Golf Course	9,905	3-Span
2		9 Avenue	9 Avenue Bridge (ID1669[1])	8,632	3-Span
3		Footbridge 6 Avenue W	Footbridge Close to 6 Avenue W	7,992	2-Span
4		Main Street	Main Street Bridge (ID 1303)	7,412	5-Span
5		3 Avenue	3 Avenue Bridge (ID 7652)	6,945	2-Span
6		1 Avenue	1 Avenue E Bridge (ID 78730)	6,455	2-Span
7	St. Mary River	Highway 5	Highway 5 Bridge (ID 315)	335	3-Span

Note: 1) Alberta Transportation Identification Code.

4.4 Flood Control Structures

There is one flood control structure within the study area. It is located on the left bank of Lee Creek downstream of the Main Street Bridge in the Town of Cardston (Table 10). The structure is approximately 95 m long and consists of a flood wall with steel sheet piles. The survey points were collected along the crest of the wall with a spacing of approximately 10 m. A summary of the flood control structure information is provided in Appendix B.

Table 10: Flood Control Structure within the Study Area

Stream	Name	Length (m)	Side of River [1]	Type
Lee Creek	Downstream of the Highway 2 Bridge	95	Left	Flood wall with steel sheet piles

Note: 1) Left or right refer to directions as seen by an observer looking downstream.

5.0 MODEL CONSTRUCTION

5.1 HEC-RAS Program

5.1.1 Description

The HEC-RAS program (Version 5.0.7) was used as the software platform for developing the one-dimensional hydraulic models in the study area. The HEC-RAS program was developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACE). The River Analysis System (RAS) software has a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphics and reporting facilities. HEC-RAS is a commonly used program in North America and around the world (USACE 2016).

The HEC-RAS program was designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. HEC-RAS is capable of simulating steady and unsteady flow conditions. The program can be used to calculate water surface profiles for gradually varied flow. The program is capable of calculating the water surface profiles associated with subcritical, supercritical and mixed flow regimes. In this study, the program is used in steady-state mode.

- The basic computational procedure for steady-state simulation is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning’s equation) and contraction/expansion. The momentum equation is utilized in situation where the water surface profile is rapidly varied. The program can be used to simulate the effects of various obstructions such as bridges, culverts, weirs, spillways and other structures in the floodplain.

The main assumptions in one-dimensional modelling are listed below:

- The variation of the river channel and floodplain geometries is represented by a series of cross sections.
- The water level is constant at each cross section.
- The flow is perpendicular to the cross section alignment.
- The HEC-GeoRAS module (Version 10.6) is used to prepare cross section data based on the LiDAR DEM and creek survey data. HEC-GeoRAS is an ArcGIS extension tool specifically designed to create a HEC-RAS import file from geospatial data.

5.1.2 General Model Setup

Reaches

All reaches in the study area are included in one integrated model setup. The model consists of three reaches as listed in Table 11 and shown in Figure 7.

Table 11: Reaches in the Hydraulic Model

River	Reach	Length (km)
Lee Creek	Lee Creek	14.0
St. Mary River	Upper St. Mary	0.5
St. Mary River	Lower St. Mary	0.5

Cross Sections

The cross-sectional alignments and extents were selected following the general approach listed below:

- The cross sections should be approximately perpendicular to the flow direction both in the main channel and the floodplains. This resulted in some cross sections being bended using multiple vertices.
- The cross sections must not cross each other.
- The cross sections should have sufficient lengths on the floodplains to extend beyond the limits of all simulated floods.

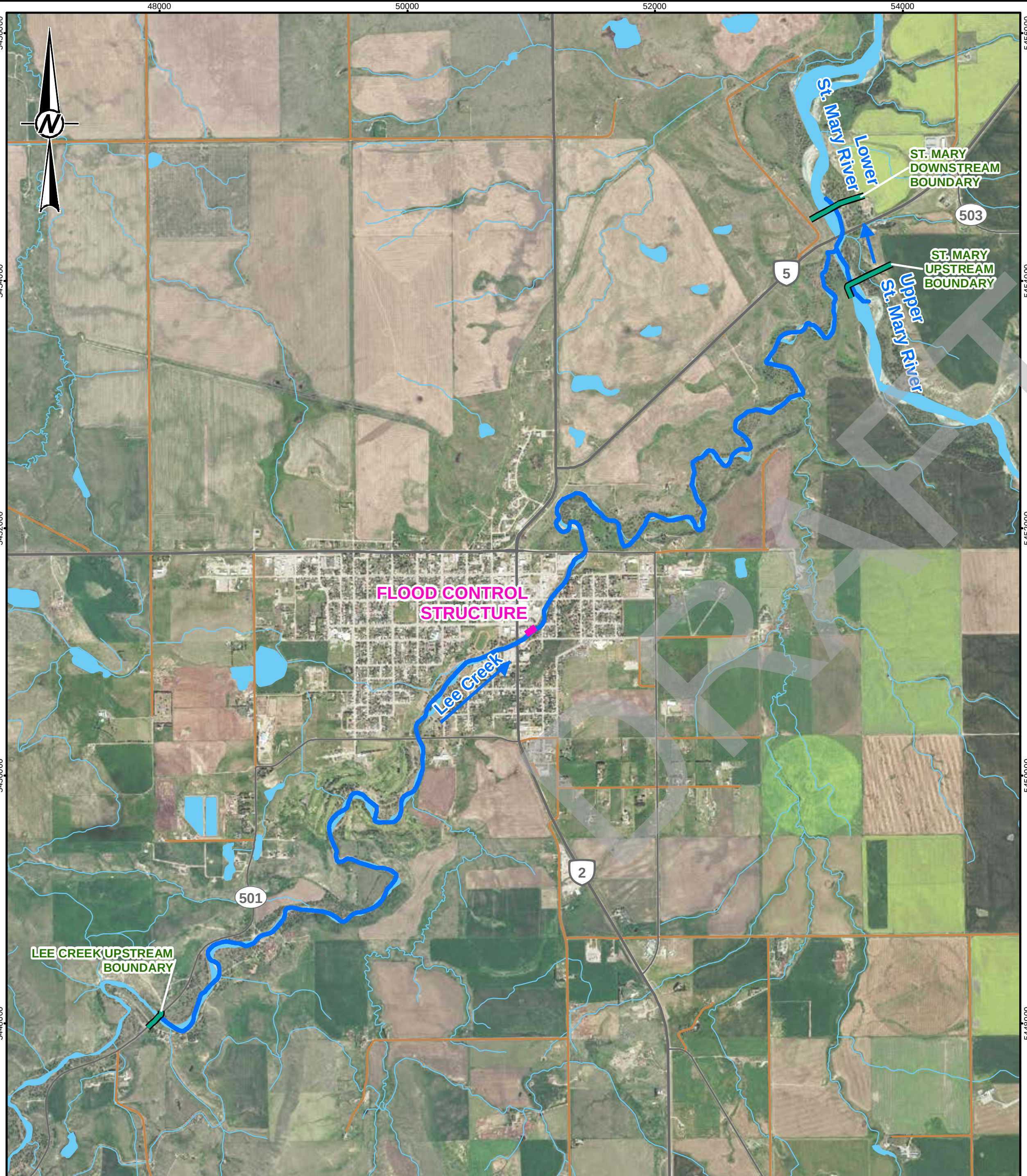
A conceptual two-dimensional hydraulic modelling was performed for the entire study area to help understand possible flood flow paths on the floodplains for the 2-, 100- and 1000-year flood events.

Boundary Conditions

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

- flow at the upstream end of Lee Creek study reach
- flow at the upstream end of the St. Mary River study reach
- slope at the downstream end of the St. Mary River study reach

A schematic of the model setup is shown in Figure 7.



LEGEND

- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOOD CONTROL STRUCTURE
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- STUDY REACH

0 1 2

1:50,000 KILOMETRES

NOTE(S)

FLOOD CONTROL STRUCTURE SURVEY PERFORMED BY GOLDER ASSOCIATES LTD. IN JUNE 2019.

REFERENCE(S)

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2018 SPOT IMAGERY PROVIDED BY AEP.
DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
RIVER REACHES AND BOUNDARY CONDITIONS

CONSULTANT

GOLDER

YYYY-MM-DD	2020-03-24
DESIGNED	PH
PREPARED	SK
REVIEWED	HKG
APPROVED	WP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	3000	0	7

5.2 Geometric Data Base

5.2.1 Cross Section Data

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

- survey data collected in 2019
- 2019 LiDAR data provided by AEP

The alignments of the cross sections on the floodplains were informed by the two-dimensional model, in combination with an examination of the topography and professional judgement. HEC-GeoRAS was used to define the main channels, overbank flow paths, bank stations, and cross section river stations.

Table 12 and Table 13 provide summaries of the stream reaches and the number of cross sections in each reach.

Table 12: Number of Cross Sections in Model Reaches

Stream Name in HEC-RAS	Reach Name In HEC-RAS	Description of Reach	From Station (m)	To Station (m)	Length (km)	Number of Cross Sections
Lee Creek	Lee Creek	Lee Creek	13,992	69	14.0	106
St. Mary River	Upper St. Mary	St. Mary River upstream of confluence of Lee Creek	771	629	0.5	2
St. Mary River	Lower St. Mary	St. Mary River Downstream of confluence of Lee Creek	455	93	0.5	4
					TOTAL	**112**

Table 13: Summary of Study Reaches

Study Reach	Reach Length (km)	Number of Cross Sections	Average Cross Section Spacing (m)
Lee Creek	14.0	106	134
St. Mary River	1.0	6	135

5.2.2 Roughness Distribution

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

- bank lines established from the LiDAR data, aerial imagery and surveyed data to identify the main channels
- available land use information from provincial data set
- information collected during the site reconnaissance in May 2019

These data sources were used to define seven roughness classes. The initial roughness values assigned to the classes are provided in Table 14. These initial values were selected based on literature and professional judgement. The roughness values were applied to the cross sections using HEC-GeoRAS and modified at some locations during the model calibration process (see Section 5.6.3). The roughness distribution is shown in Figure 8.

Table 14: Roughness Classes and Initial Manning's *n* Values

Number	Description	Initial Manning's *n*
1	Rivers – Main Channel	0.040
2	Urban Mixture	0.080
3	Golf Course	0.050
4	Grassland/Farmland	0.060
5	Ponds	0.045
6	Road Surface	0.020
7	Trees/Bushes	0.100

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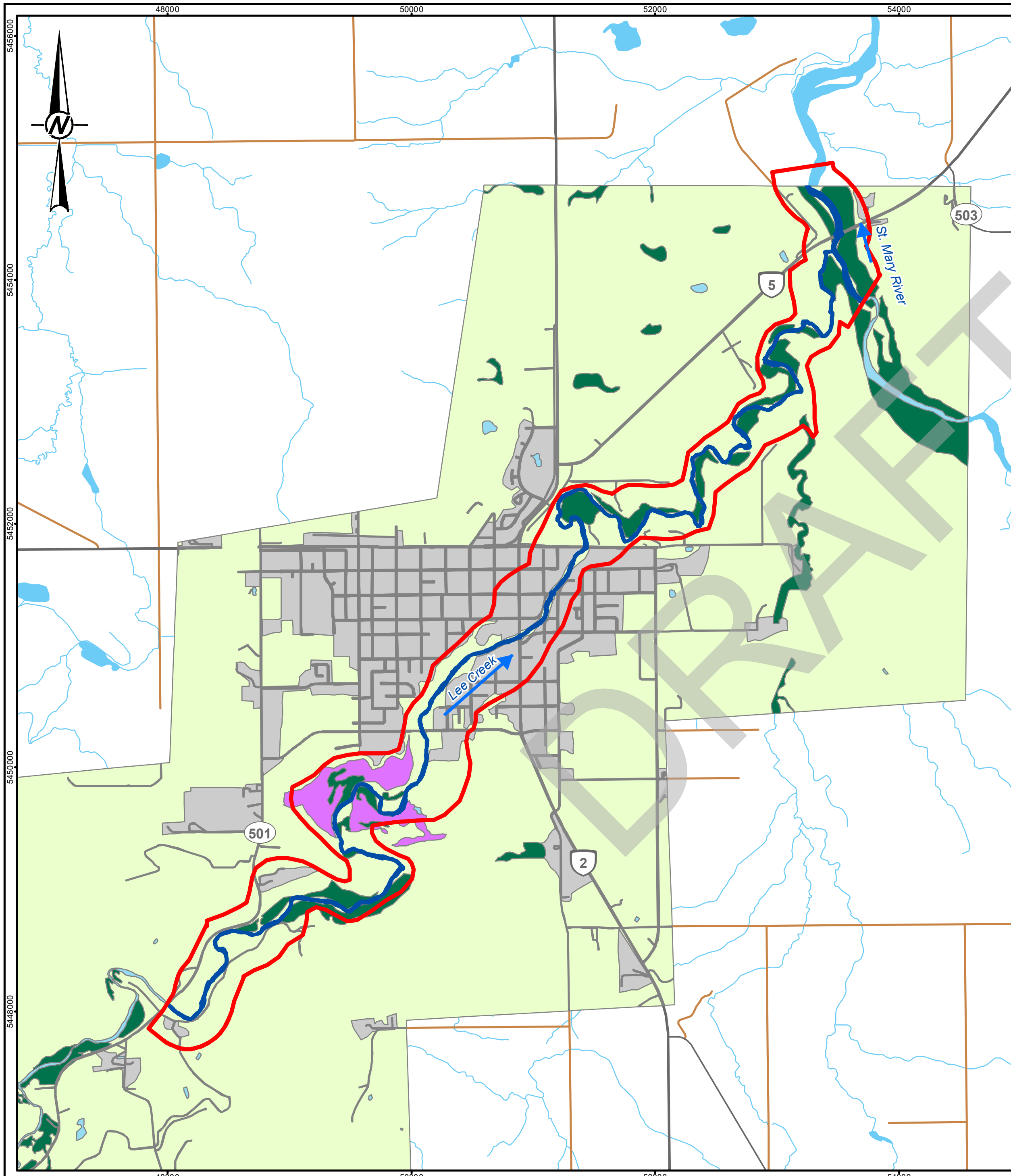

LEGEND

- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- FLOW DIRECTION
- WATERCOURSE
- WATERBODY
- FLOOD HAZARD STUDY AREA

LAND USE CLASSES

- GOLF COURSE
- GRASSLAND/FARMLAND
- PONDS
- RIVERS-MAIN CHANNEL
- ROAD SURFACE
- TREES/BUSHES
- URBAN MIXTURE

0 1 2

1:50,000 KILOMETRES

REFERENCE(S)

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DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
ROUGHNESS CLASS DISTRIBUTION IN THE MODEL DOMAIN

CONSULTANT

GOLDER

YYYY-MM-DD	2020-03-27
DESIGNED	PH
PREPARED	SK
REVIEWED	HKG
APPROVED	WP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	3000	0	8

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25mm

Classification: Public

5.2.3 Bridges

Bridges

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

- river and bridge surveys completed in 2019
- as-built drawings provided by Alberta Transportation (AT)

All existing bridges are represented in the HEC-RAS model. They include those which may not affect water levels during floods (e.g., clear span bridges with sufficient freeboards). Losses through bridges are calculated in the model using the energy equation (i.e., standard step method).

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

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

There are variations of bridge types, abutments, approaches and embankments within the study area. For each bridge, ineffective areas upstream and downstream of the bridges were carefully selected on a case-by-case basis, including the selection of permanent and non-permanent ineffective areas where appropriate.

All bridges within the study area are approximately perpendicular to the main channel flow direction, so it was not necessary to include any skew in the model.

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

The total number of bridges included in the model is summarized in Table 15.

Table 15: Bridges Included in the Hydraulic Model

Water Body	Total Number of Bridges
Lee Creek	6
St. Mary River	1

5.2.4 Flood Control Structure

The flood control structure considered in this study was based on the 2019 field data collected by Golder. The flood control structure is represented in the HEC-RAS model using a combination of the two methods listed below:

- levees
- ineffective flow areas

5.3 Model Calibration and Validation

5.3.1 Methodology

The Manning's roughness *n* values and the bridge contraction/expansion coefficients are the two primary model parameters used in calibrating the HEC-RAS model. Additionally, ineffective flow areas are used to deactivate disconnected low-lying areas on the floodplains. Selection of initial Manning's *n* values included consideration of stream bed and bank materials, vegetation cover, site information collected during the field inspection, and Golder's experience with previous hydraulic modelling studies.

The Manning's *n* value is a composite empirical parameter which may decrease with increased water depth. Model calibration was conducted based on the pertinent discharge and water level information of the low flow and high flow conditions to determine appropriate roughness values across a wide range of flows, as described below:

1) Low Flow Calibration: The surveyed water levels and measured flow during the stream survey were used for the low flow calibration.

2) High Flow Calibration: Available highwater marks on Lee Creek for the 1975, 1991, 1995, 2010 and 2014 floods and peak flow estimates for these flood events, were used for the high flow calibration.

The model calibration process involved multiple iterations to adjust the model parameter values, conduct simulations, and compare the simulated water levels with the highwater marks (for high flow calibration). The objective of the model calibration was to achieve good agreement between the simulated water levels and the measured highwater marks.

The results of the model calibration are described in the following sections.

5.3.2 Low Flow Calibration

The water level measurements on Lee Creek were conducted on May 15 to 17 and June 10 to 13, 2019. One flow measurement was conducted on Lee Creek on June 13, 2019. The measured flow and the average daily flow over the survey periods from WSC station 05AE002, were used in the model calibration for the periods June 10 to 13, 2019 and May 15 to 17, 2019, respectively (see Table 16).

Table 16: Measured Discharges and WSC Flow Data

Waterbody	Date	WSC Gauging Station	Discharge (m^3/s)		Difference		Discharge Used in Low Flow Calibration
			WSC Gauge	Survey Measurement	(m^3/s)	(%)	(m^3/s)
Lee Creek	May 15 to 17, 2019	05AE002 Lee Creek at Cardston	5.22	-	-	-	5.22
Lee Creek	June 10 to 13, 2019		3.27	3.22	0.05	2	3.22

The Lee Creek channel roughness values were calibrated based on the flow data listed in Table 16 and the water level data collected during the periods May 15 to 17 and June 10 to 13, 2019.

Figure 9 shows a comparison between the simulated water surface profiles and measured water levels for the low flow conditions. The average difference between the simulated and measured water levels is 0.00 m, with individual differences ranging from -0.40 m to +0.29 m (see Figure 10).

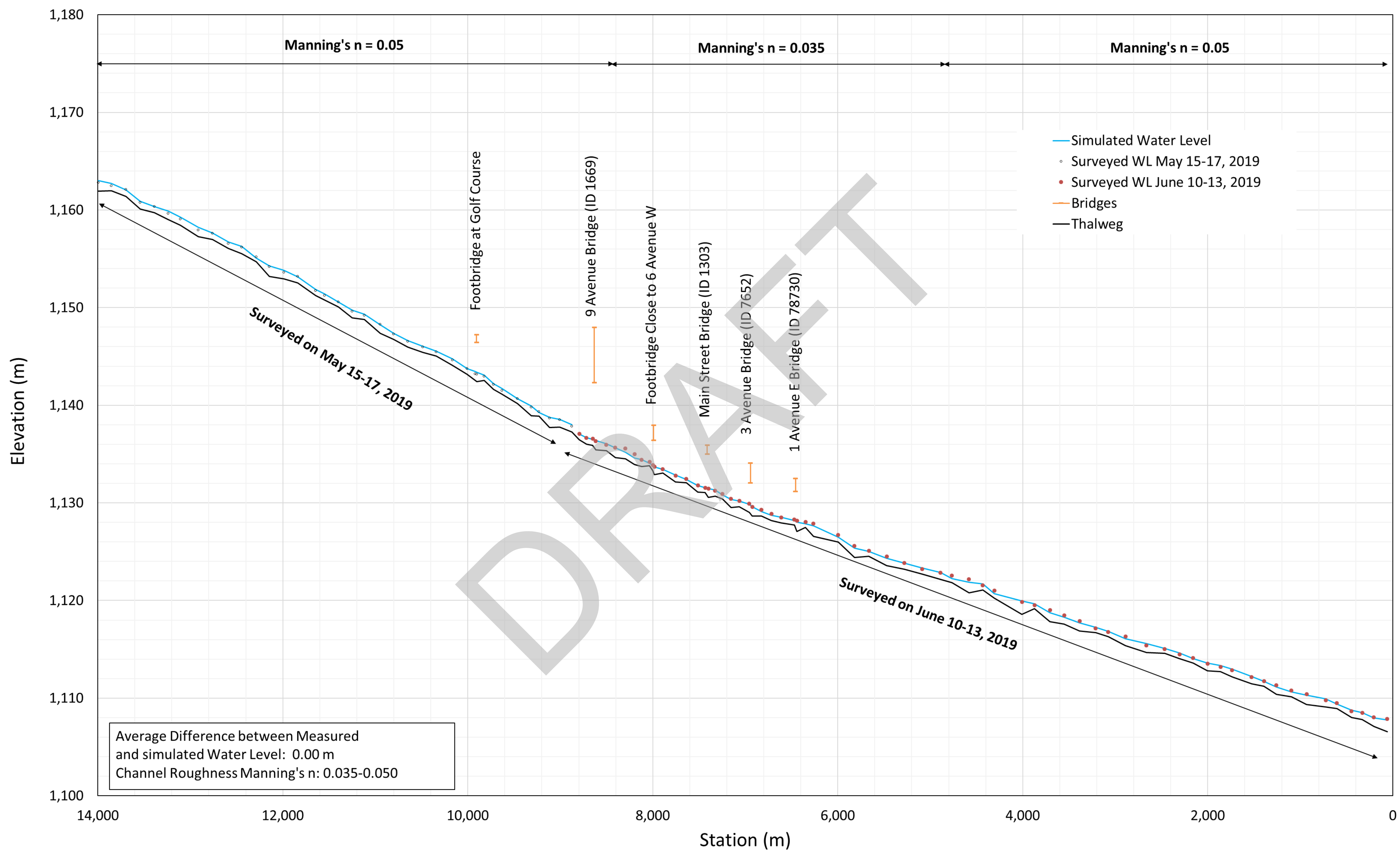


Figure 9: Comparison of Simulated Water Surface Profiles with Surveyed Water Levels for the Low Flow Conditions

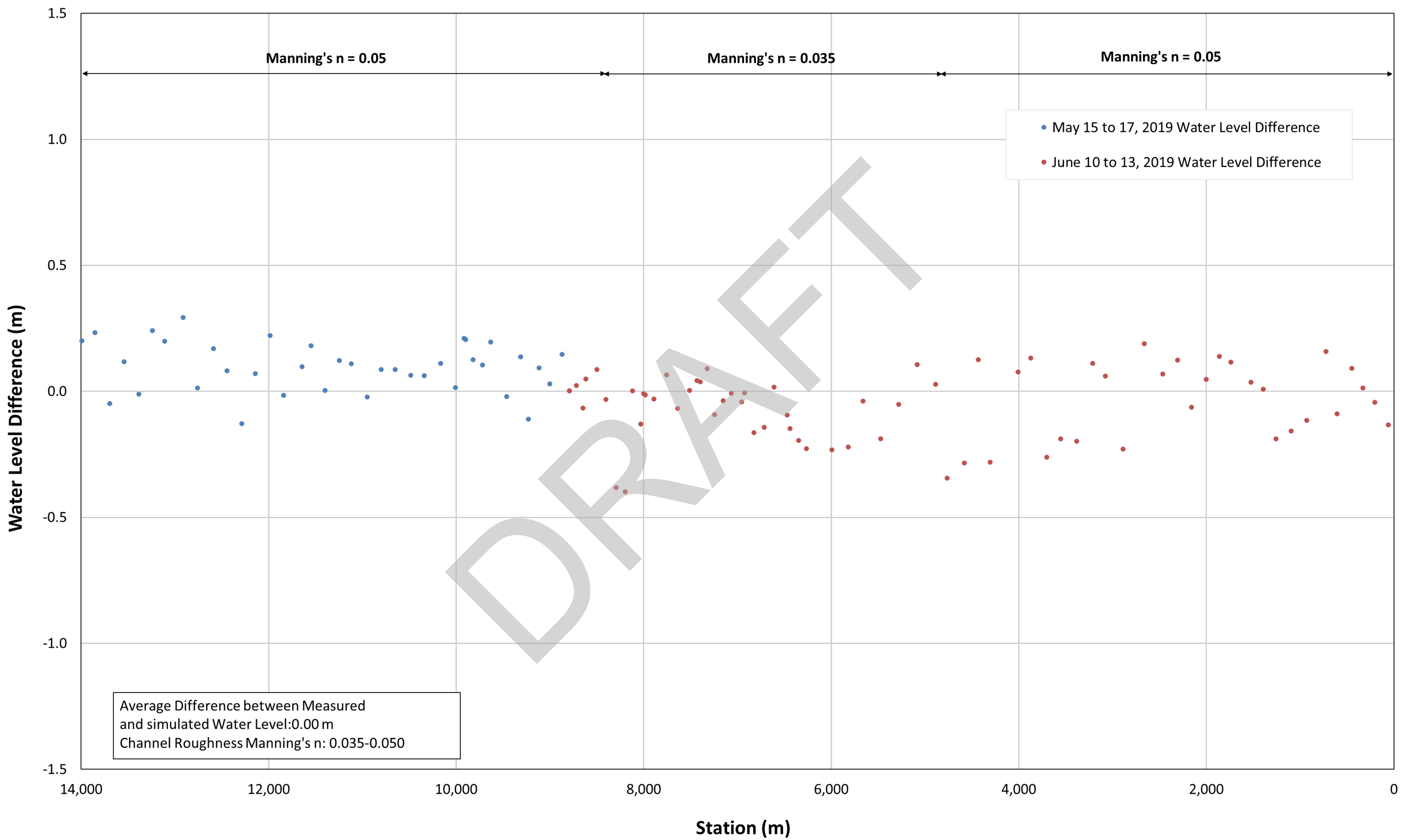


Figure 10: Difference of Simulated and Surveyed Water Levels for Low Flow Conditions

The calibrated channel Manning's *n* values for the low flow conditions range from 0.035 to 0.050. The surveyed water levels were likely impacted by small disturbances like gravel bars, low flow channel braiding and meandering, debris, and small-scale pool-riffle sequences, which may not be present during higher flow events.

5.3.3 High Flow Calibration

There are five sets of open water flood highwater marks available within Lee Creek study reach: 1975, 1991, 1995, 2010 and 2014 (AEP 1975, AEP 1991, AEP 1995, AEP 2010 and AEP 2014). The HEC-RAS model for Lee Creek was calibrated based on these five sets of highwater marks. The model calibration was achieved by adjusting the main channel Manning's *n* values so that the simulated water levels were in good agreement with the highwater marks. Floodplain roughness values were found to have a negligible impact on the model calibration because a relatively large portion of the total flow was conveyed within Lee Creek main channel during these flood events.

The model calibration is based on the peak flow estimates for the 1975, 1991, 1995, 2010 and 2014 floods listed in Table 17.

Table 17: Peak Flow Estimates for the 1975, 1991, 1995, 2010 and 2014 Floods on Lee Creek

Flood Event	Lee Creek (m^3/s)	St. Mary River (m^3/s)
2014 Flood Peak Flows	117	241
2010 Flood Peak Flows	245	160
1995 Flood Peak Flows	303	430
1991 Flood Peak Flows	59	168
1975 Flood Peak Flows	226	660

2014 Flood

Figure 11 shows a comparison between the simulated water surface profile and reported highwater marks for the 2014 flood event along Lee Creek. The average difference between the simulated and measured water levels is 0.03 m, with individual differences ranging from -0.61 m to +0.74 m (see Figure 12). Table C-1 in Appendix C lists the differences between the simulated and reported water levels for the 2014 flood event on Lee Creek.

2010 Flood

Figure 13 shows a comparison between the simulated water surface profile and reported highwater marks for the 2010 flood event along Lee Creek. The average difference between the simulated and measured water levels is 1.03 m, with individual differences ranging from +0.14 m to +1.86 m (see Figure 14). Table C-2 in Appendix C lists the differences between the simulated and reported water levels for the 2010 flood event on Lee Creek.

1995 Flood

Figure 15 shows a comparison between the simulated water surface profile and reported highwater marks for the 1995 flood event along Lee Creek. The average difference between the simulated and measured water levels is 0.37 m, with individual differences ranging from -0.37 m to +1.22 m (see Figure 16). Table C-3 in Appendix C lists the differences between the simulated and reported water levels for the 1995 flood event on Lee Creek.

1991 Flood

Figure 17 shows a comparison between the simulated water surface profile and reported highwater marks for the 1991 flood event along Lee Creek. The average difference between the simulated and measured water levels is 0.31 m, with individual differences ranging from -0.40 m to +0.82 m (see Figure 18). Table C-4 in Appendix C lists the differences between the simulated and reported water levels for the 1991 flood event on Lee Creek.

1975 Flood

Figure 19 shows a comparison between the simulated water surface profile and reported highwater marks for the 1975 flood event along Lee Creek. The average difference between the simulated and measured water levels is 0.51 m, with individual differences ranging from +0.43 m to +0.58 m (see Figure 20). Table C-5 in Appendix C lists the differences between the simulated and reported water levels for the 1975 flood event on Lee Creek.

High Flow Calibration Results

The calibrated main channel Manning's *n* values of 0.025 and 0.04 are considered reasonable. The high flow calibration results show that the simulated water levels are generally higher than the measured highwater marks, even when a relatively low Manning's *n* value of 0.025 was used for the main Lee Creek channel through the Town of Cardston.

The current model was compared to the previous model (Stanley 1992). Comparing the Lee Creek thalweg elevations and cross sections used in these two models (see Section 3.5.1) revealed that the channel bed elevations and overall channel/floodplain flow conveyance along Lee Creek have had noticeable changes since 1992. For the 1991 flood, the current model produced lower water levels upstream of the footbridge (located about 500 m upstream of the Main Street Bridge) and higher water levels downstream of this location, than the 1992 model.

As illustrated in Figure 3, the 2019 thalweg elevations were lower than the 1992 thalweg elevations upstream of the footbridge (located about 500 m upstream of the Main Street Bridge), and higher downstream of it. These differences in the channel thawleg elevations are believed to have caused the discrepancies between the two models and to explain why the current model can not produce good calibration results for the 1991 event.

The Manning's *n* values in the 1992 model vary from 0.027 to 0.062 along the model reach. The current model includes a low Manning's *n* value of 0.025 within the town area where the channel plan form is relatively straight, and a high Manning's *n* value of 0.040 for the upstream and downstream reaches where the channel is meandering.

The current model based on the updated cross-sectional data set was run for the 1991 flood event using the same flood peak discharge and Manning's n values used in the 1992 model (Stanley 1992). The simulated 1991 flood levels using the current model were compared with those using the 1992 model with the dated cross-sectional data set. The comparison shows that the current model generated lower water levels than the 1992 model by about 0.4 m upstream of footbridge (located about 500 m upstream of the Main Street Bridge), and higher water levels by about 1.0 m downstream of it.

The results of the comparison support the findings that the changes in the channel geometry (i.e., changes in channel thalweg and cross section shape/dimension) between the 1992 and current models have caused changes to the flood flow conveyance characteristics and the resulting flood levels for the same flood discharge. Therefore, the simulation results for the historical floods using the current model need to be interpreted in proper consideration of these changes for final selection of the calibrated Manning's n values, in addition to comparison to literature values for comparable streams.

The high flow calibration findings are summarized below:

i) <u>2014 flood:</u> The differences between the simulated water levels and highwater marks for the 2014 flood event do not follow the same pattern as the other flood events. The average difference of 0.03 m between the simulated water levels and highwater marks for the 2014 flood event is the smallest among all the flood events. This is because the 2014 channel bed condition was similar to the survey condition in 2019 (used in the current model).

ii) 2010 flood: The average difference between the simulated water levels and highwater marks for the 2010 flood event are the largest among all the flood events. The estimated flood peak discharge of 245 m^3/s for Lee Creek seems high when compared to that of the St. Mary River (i.e., 160 m^3/s). An overestimation of the flood peak discharge on Lee Creek could have caused the current model to produce higher water levels than the highwater marks.

iii) 1995 flood: The flood peak discharge of this event was the highest among all the flood events. The average difference between the simulated water levels and highwater marks is 0.34 m. As discussed above, the current model produced higher water levels than the highwater marks, because of the temporal changes in Lee Creek channel bed elevations overall channel/floodplain flow conveyance. Therefore, the calibration results for this flood event with higher simulated flood levels are considered reasonable.

iv) 1991 and 1975 floods: The average differences between the simulated water levels and highwater marks for these two flood events are similar to that of 1995. The calibration results for these two flood events are considered reasonable, similar to the 1995 flood.

Conclusion

The calibrated Manning's *n* values for Lee Creek main channel are 0.025 within the Town and 0.040 upstream and downstream of the Town. These values are within the typical range of roughness values for gravel and sand bed streams during high flow conditions (Chow 1959) and within the range of roughness values used in the previous model (Stanley 1992).

5.3.4 Gauge Data and Rating Curves

There is one WSC gauge within the study area. The available data at the WSC Station 05AE002 (Lee Creek at Cardston) was used to support the model calibration and to quantify the variability of the main channel roughness over a range of flows (Figure 21) The calibrated Manning's *n* value decreases from 0.06 for very low flows to 0.025 for high flows.

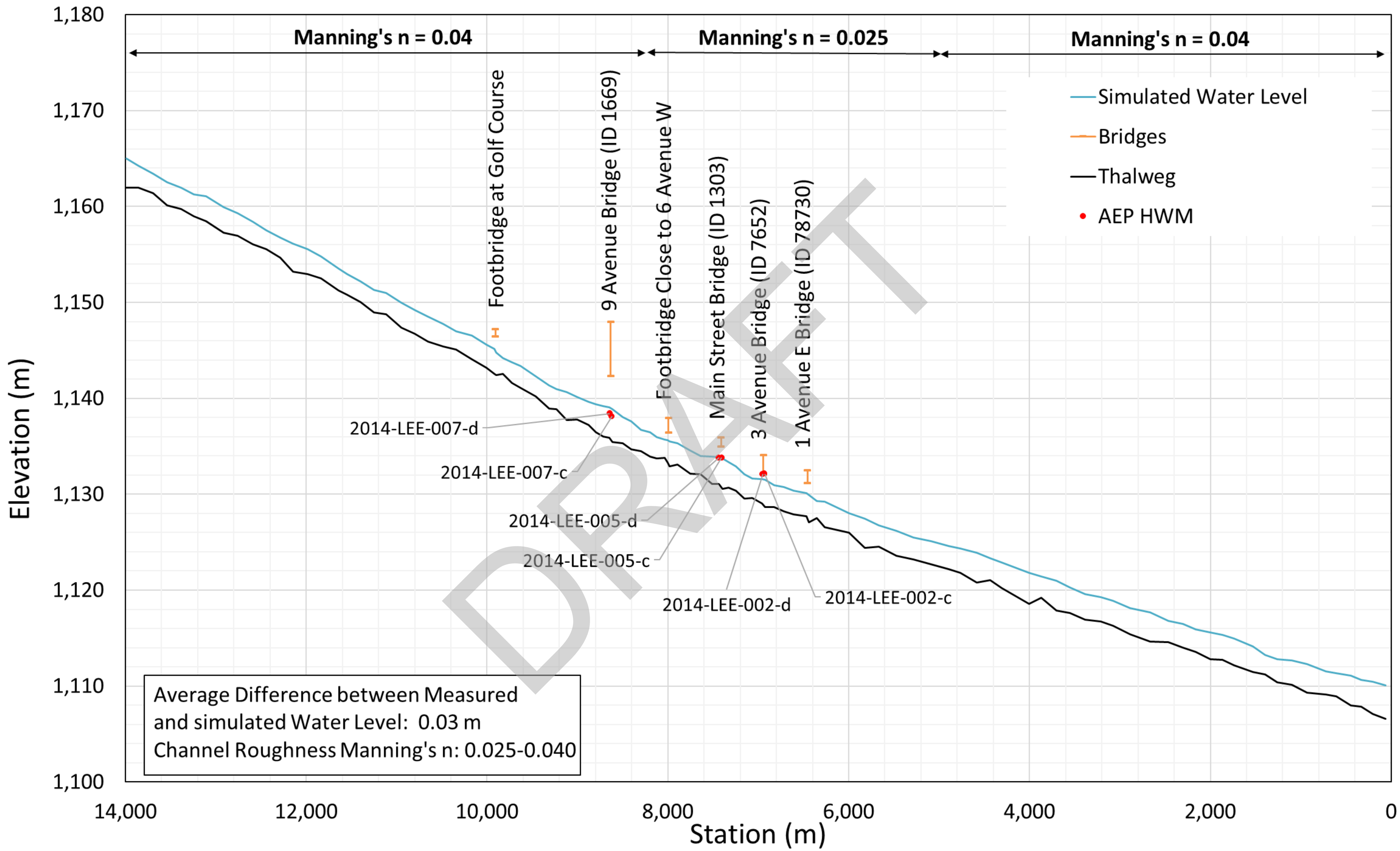


Figure 11: Comparison of Simulated Lee Creek Water Surface Profile and Reported Highwater Marks for the 2014 High Flow Event

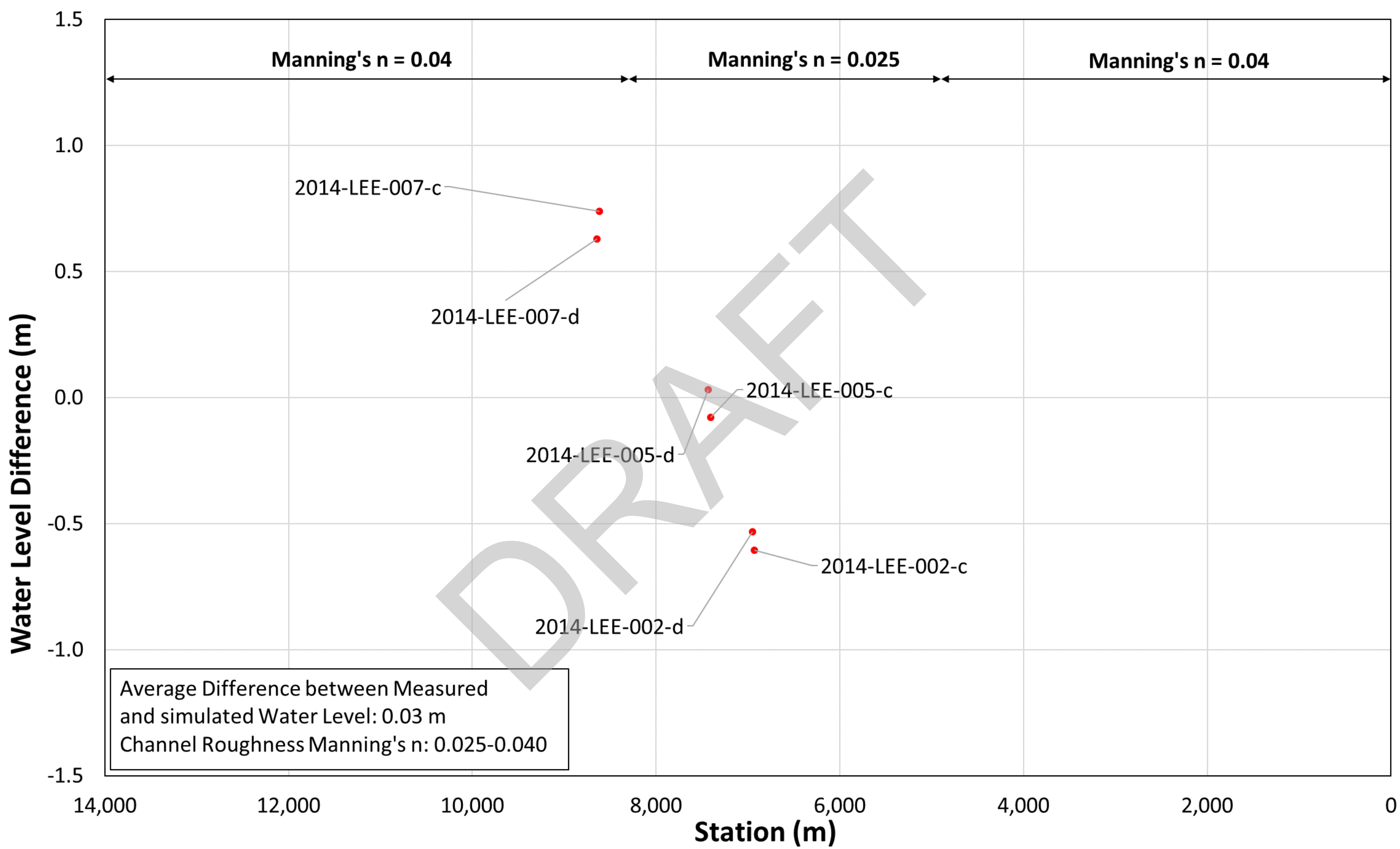


Figure 12: Difference of Simulated Lee Creek Water Levels and Reported Highwater Marks for the 2014 High Flow Event

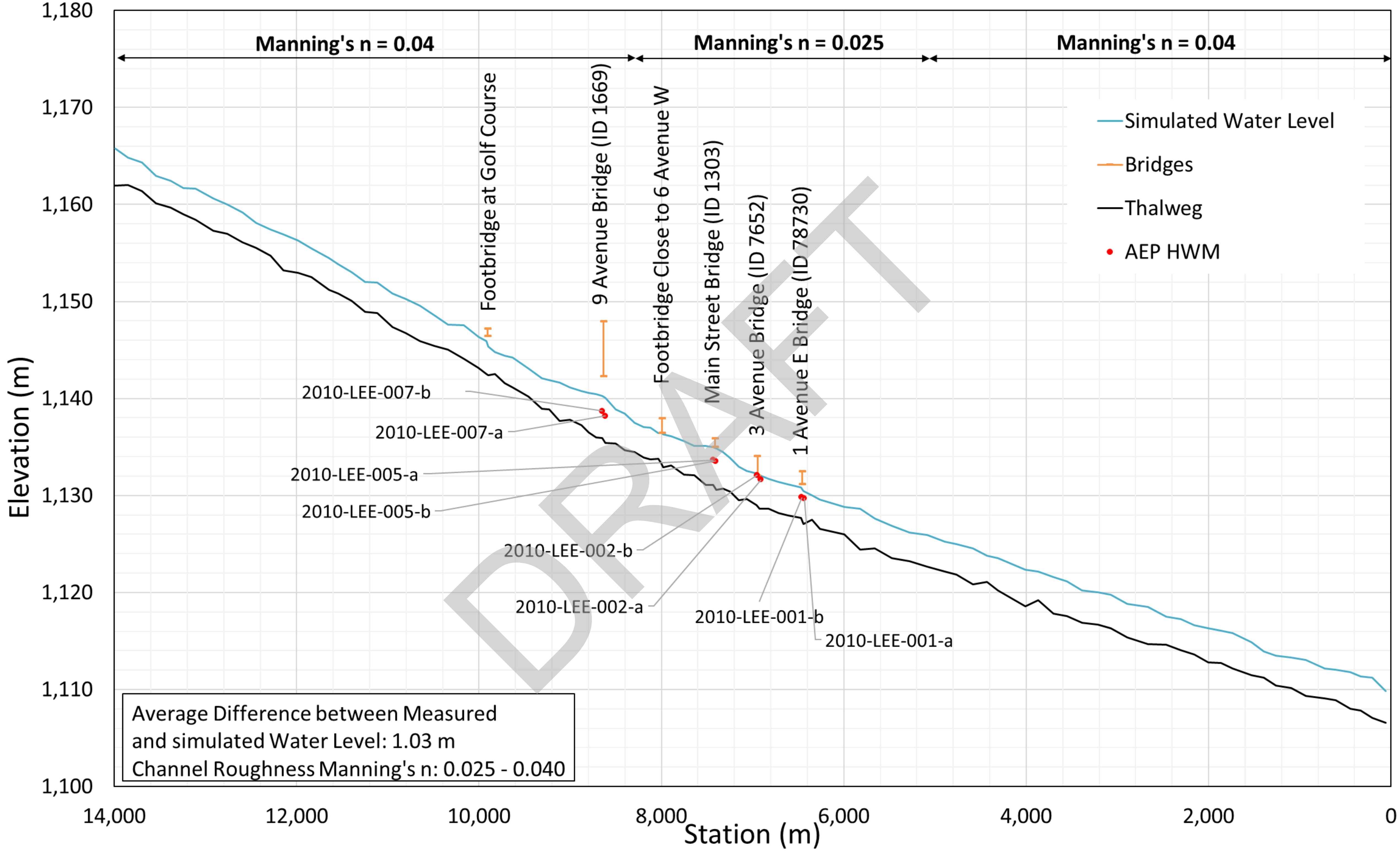


Figure 13: Comparison of Simulated Lee Creek Water Surface Profile and Reported Highwater Marks for the 2010 High Flow Event

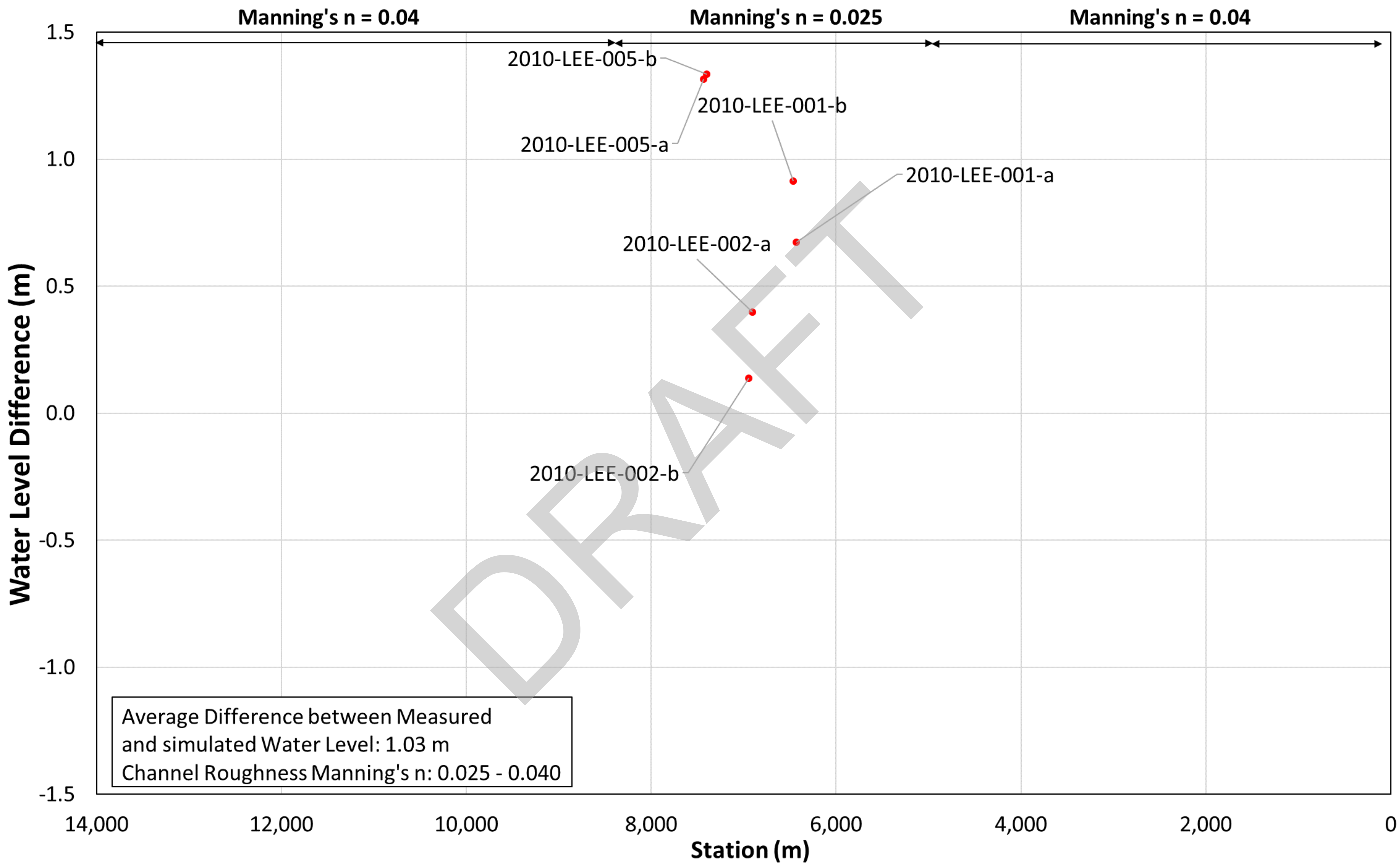


Figure 14: Difference of Simulated Lee Creek Water Levels and Reported Highwater Marks for the 2010 High Flow Event

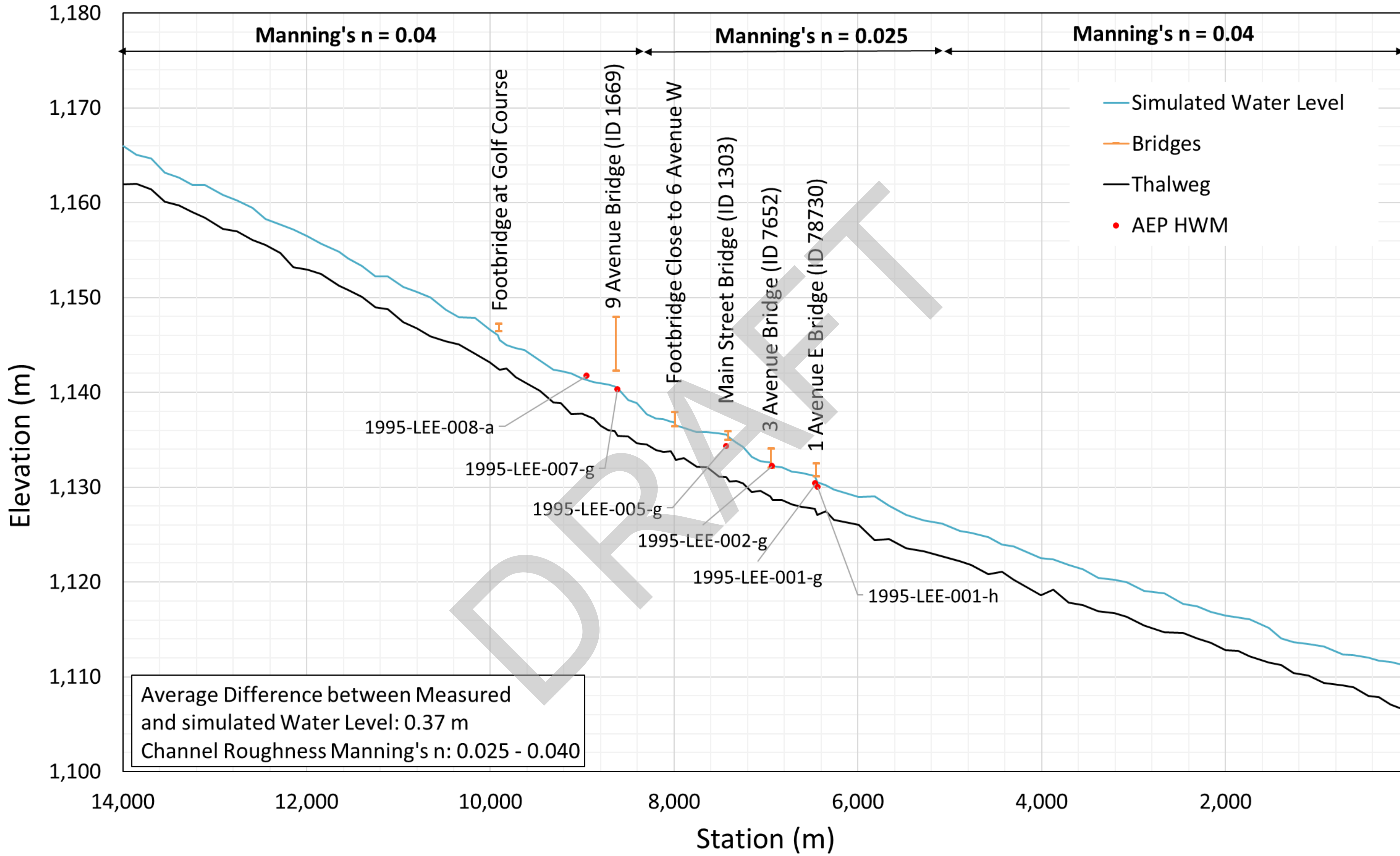


Figure 15: Comparison of Simulated Lee Creek Water Surface Profile and Reported Highwater Marks for the 1995 High Flow Event

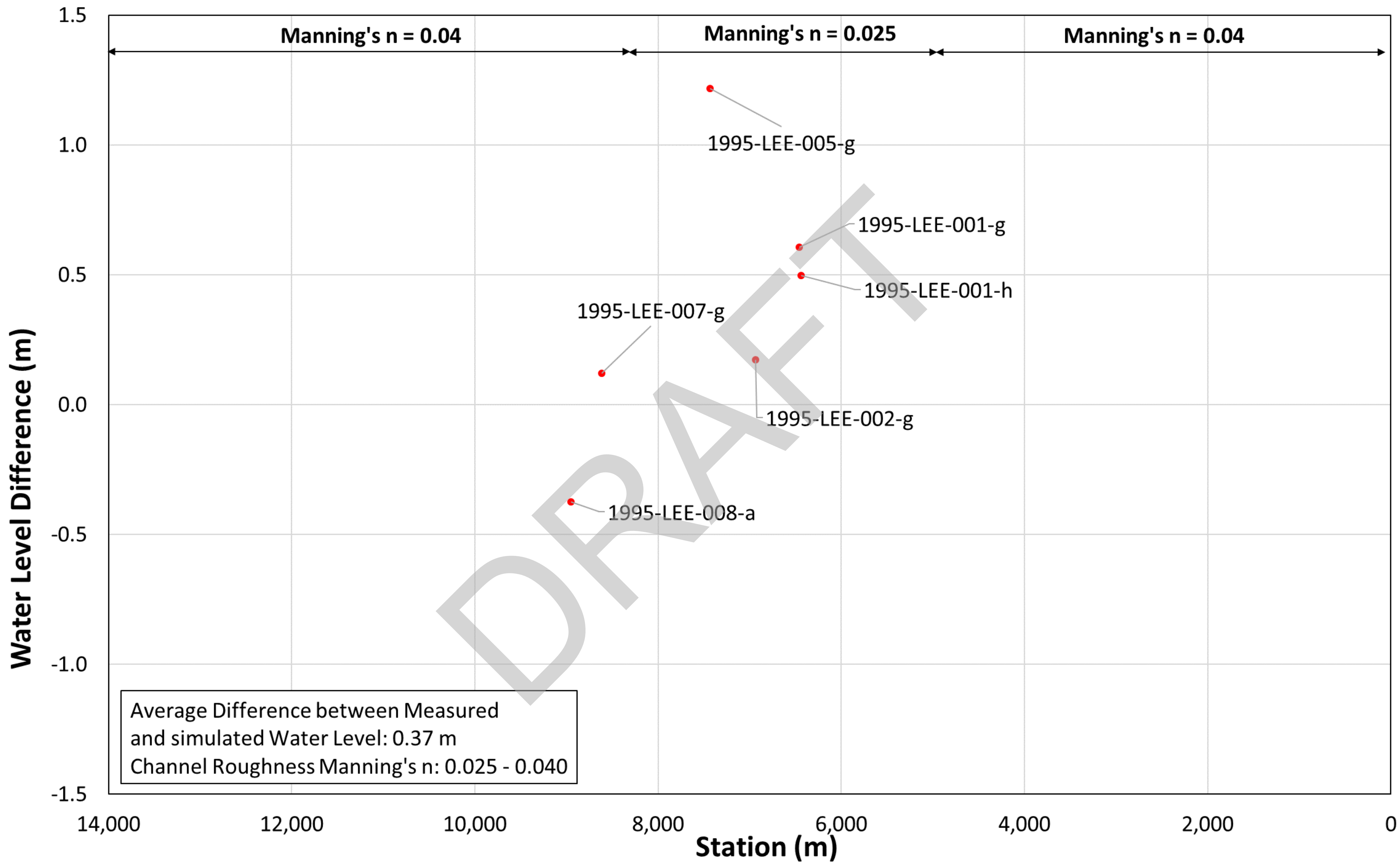


Figure 16: Difference of Simulated Lee Creek Water Levels and Reported Highwater Marks for the 1995 High Flow Event

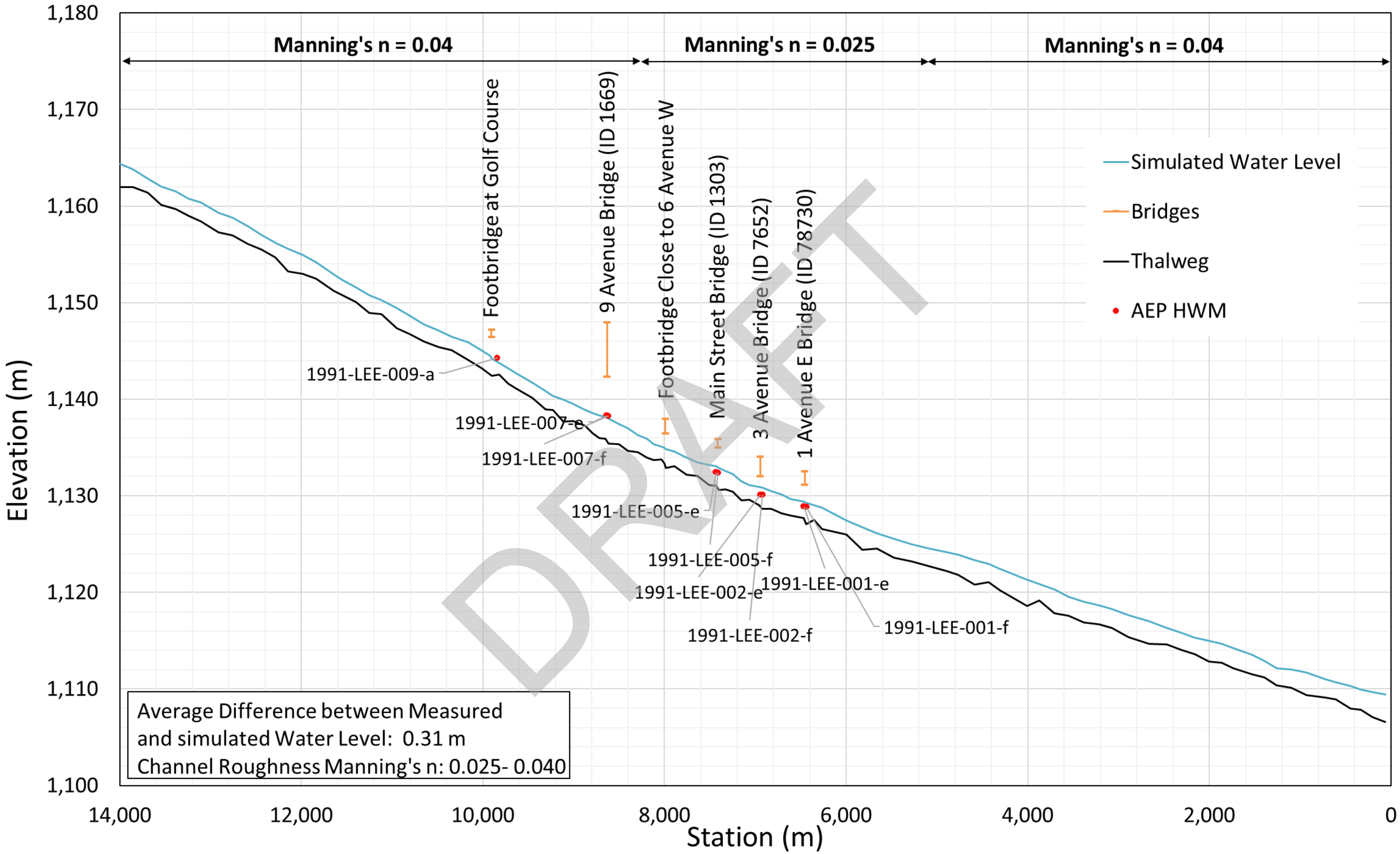


Figure 17: Comparison of Simulated Lee Creek Water Surface Profile with Reported Highwater marks for the 1991 High Flow Event

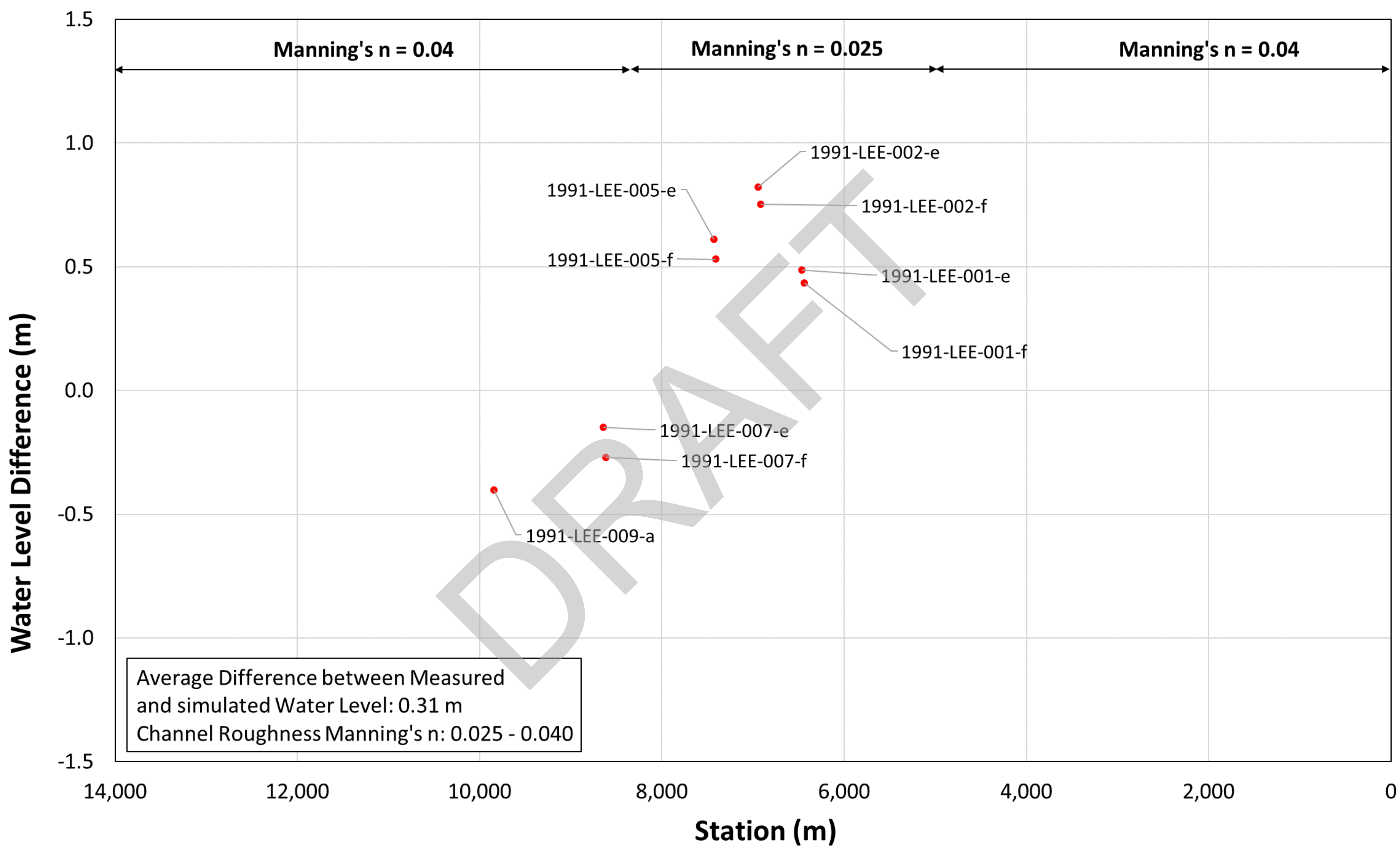


Figure 18: Difference of Simulated Lee Creek Water Levels and Reported Highwater Marks for the 1991 High Flow Event

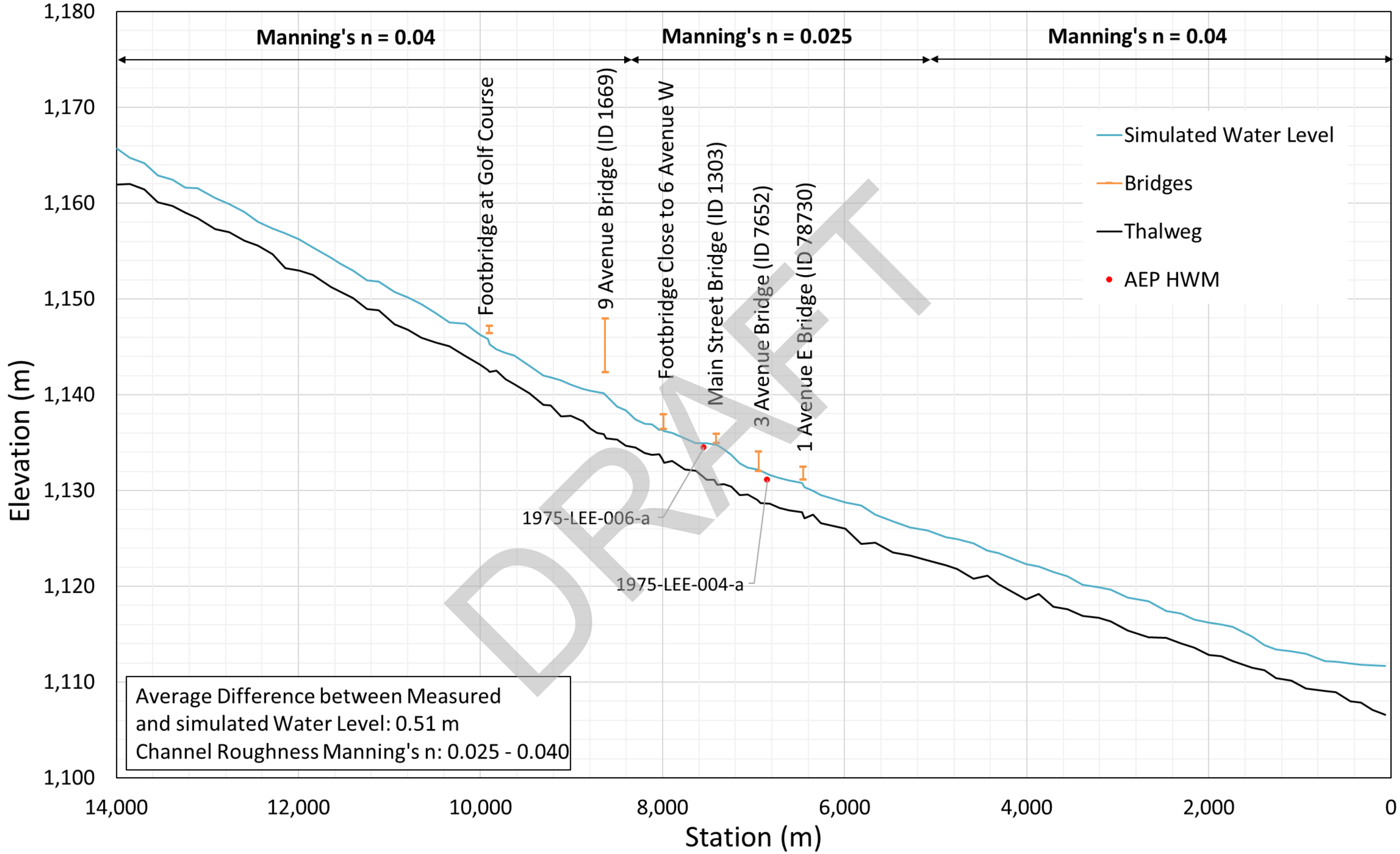


Figure 19: Comparison of Simulated Lee Creek Water Surface Profile with Reported Highwater Marks for the 1975 High Flow Event

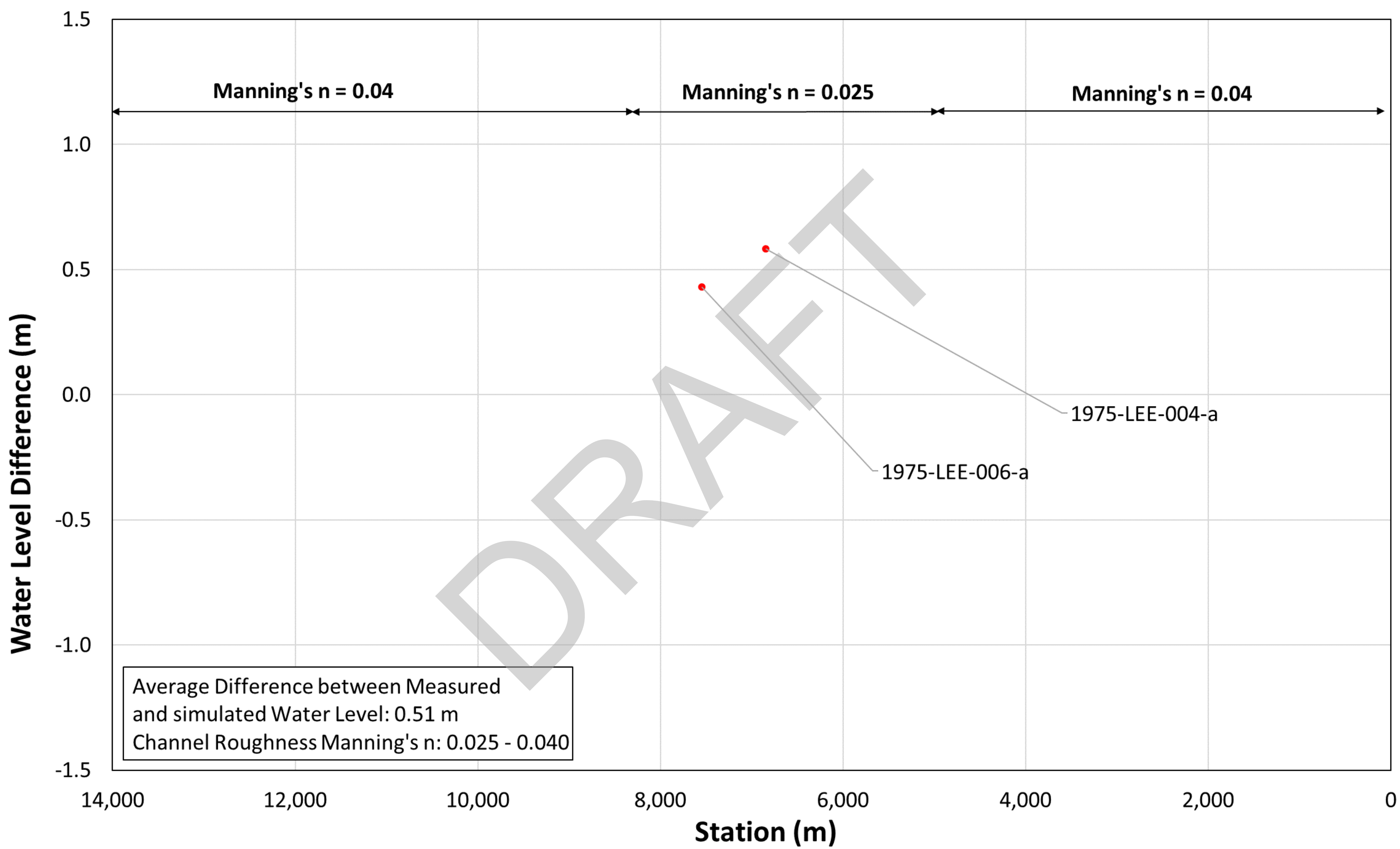


Figure 20: Difference of Simulated Lee Creek Water Levels and Reported Highwater Marks for the 1975 High Flow Event

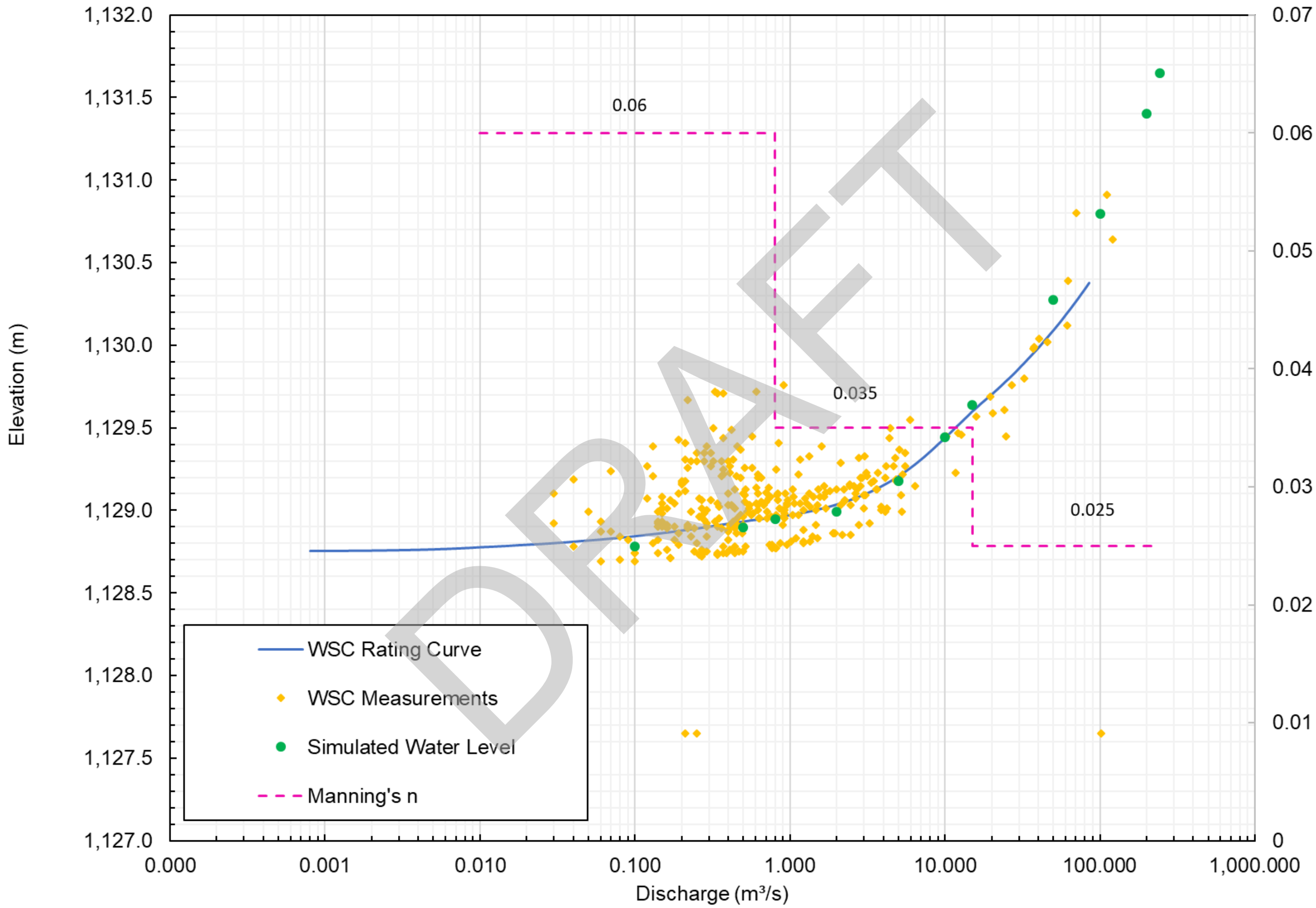


Figure 21: Calibration Results based on the Lee Creek at Cardston (05AE002) Rating Curve

5.3.5 Summary of Calibration Results

The main purpose of this study is for identification of creek flood hazards. Therefore, the focus of model calibration was to determine the appropriate Manning's *n* values for high flow conditions.

Highwater mark measurements were available for the 2014, 2010, 1995, 1991 and 1975 open water flood events on Lee Creek. The differences between the simulated water levels and measured highwater marks for these flood events are summarized in Table 18.

Table 18: Lee Creek High Flow Calibration Results

Parameter	Water Level Difference (m)				
	2014 Flood Event	2010 Flood Event	1995 Flood Event	1991 Flood Event	1975 Flood Event
Mean difference between the simulated water levels and highwater marks reported by AEP	0.03	1.03	0.37	0.31	0.51
Mean absolute difference between the simulated water levels and highwater marks reported by AEP	0.44	1.03	0.50	0.50	0.51

5.4 Model Parameters

5.4.1 Manning Roughness

5.4.1.1 Channel Roughness

The calibrated creek channel Manning's *n* values are summarized in Table 19.

Table 19: Calibrated Channel Roughness Values for High Flow Conditions

Stream	Calibrated Manning's *n* Value
Lee Creek	0.025 – 0.040

5.4.1.2 Overbank Roughness

The estimated overbank roughness values are provided in Table 20.

Table 20: Estimated Overbank Roughness Values

Number	Description	Estimated Manning's *n* Value
1	Urban Mixture	0.080
2	Golf Course	0.050
3	Grassland/Farmland	0.060
4	Ponds	0.045
5	Road Surface	0.020
6	Trees/Bushes	0.100

5.4.2 Expansion and Contraction Coefficients

The calibrated contraction and expansion coefficients for all bridges except Main Street Bridge are 0.3 and 0.5, respectively. The calibrated contraction and expansion coefficients for Main Street Bridge are 0.5 and 1.0, respectively, due to relatively small bridge openings.

The calibrated contraction and expansion coefficients for all other cross sections are 0.1 and 0.3.

5.4.3 Minor Losses

Minor losses can be added to cross sections to account for hydraulic head losses due to features in the stream that are not covered by the geometric data of the cross sections or structures such as sharp bends, local roughness elements or structures that are not defined in the geometry data.

Within the study area, minor losses were added along Lee Creek channel to account for the effects of geometry features. These loss coefficient values were estimated based on hydraulic calculations and professional judgement (Table 21).

Table 21: Minor Loss Coefficients at Select Cross Sections along Lee Creek

Station	Minor Loss Coefficient
10948	0.3
7512	0.1
7326	0.5
7247	0.5
6610	0.5

5.4.4 Obstructions and Ineffective Flow Areas

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

- Topographical low areas in which standing water may occur: Permanent ineffective flow areas were specified to block off low-lying areas that do not effectively convey flow.
- Bridge decks and embankments: Permanent ineffective flow areas were specified to block off flow through bridge embankments.

Small residential buildings and houses are not specified as building blockage, because their effects on the hydraulic conditions in the overbank areas are represented by the composite or apparent Manning's value for residential areas.

5.5 Open Water Flood Frequency Profiles

5.5.1 Hydrology Summary

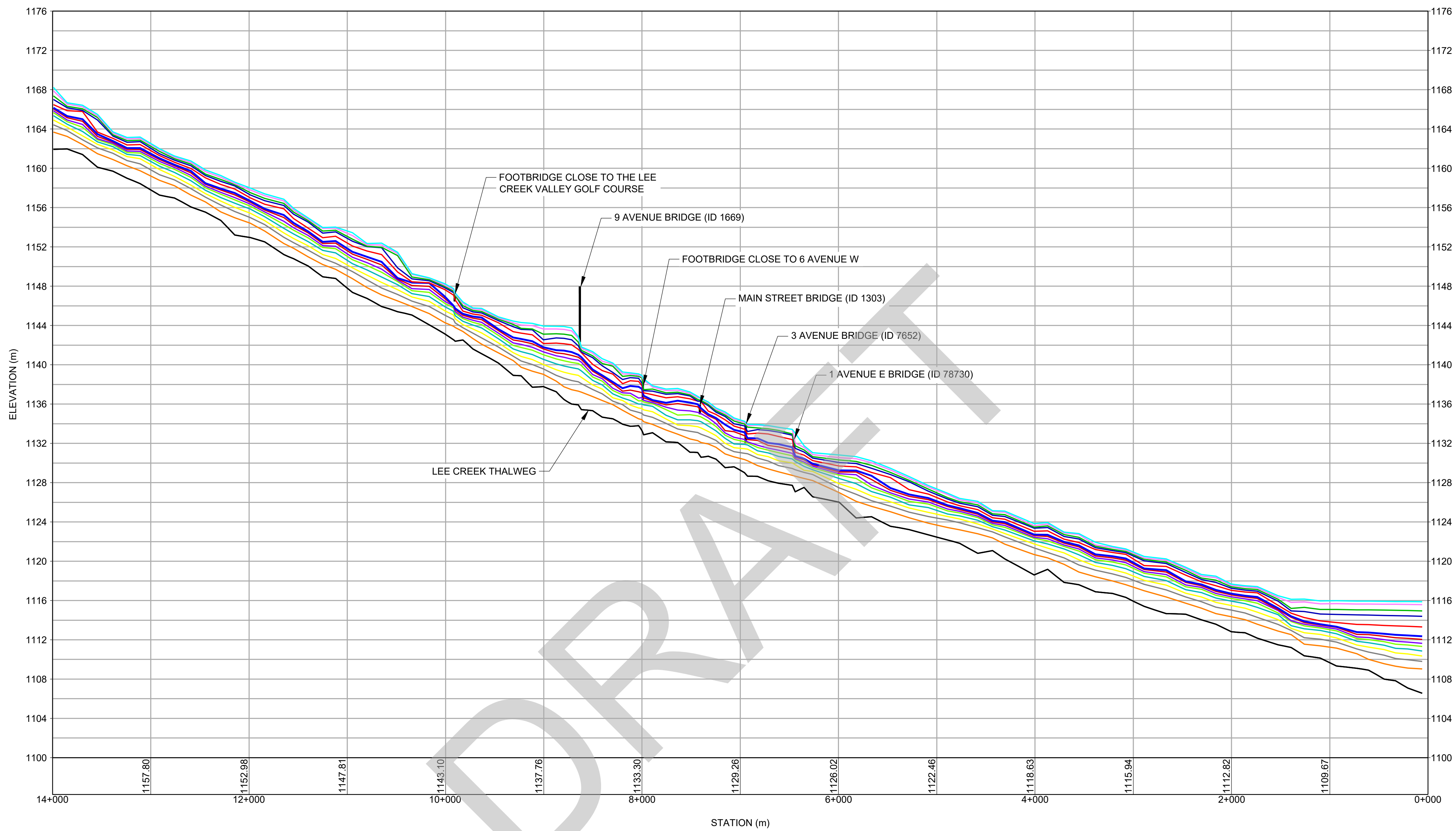
Surface water profiles were simulated for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year floods using the calibrated HEC-RAS model. The estimated peak discharges for these flood events were determined in the open water hydrology assessment (Appendix A). The flood peak discharges for the study reaches are summarized in Table 22.

5.5.2 Lee Creek

The simulated open water flood profiles of the various return periods for Lee Creek are shown in Figure 22. The open water flood water levels for individual cross sections are listed in Table D-1 in Appendix D.

Table 22: Summary of Flood Flow Frequency Estimates

Location	WSC Station ID / Node ID	Effective Drainage Area (km^2)	Flood Peak Discharges of Various Return Periods (m^3/s)												
			2-Year	5-Year	10-Year	20-Year	35-Year	50-Year	75-Year	100-Year	200-Year	350-Year	500-Year	750-Year	1,000-Year
Lee Creek at Cardston	WSC 05AE002	316	23	62	105	163	223	269	328	375	510	642	739	862	959
St. Mary River Upstream of Lee Creek Confluence	-	1,630	141	230	314	419	525	605	709	793	1,040	1,280	1,470	1,710	1,910



LEGEND

- FLOOD PROFILE - 2 YEAR
- FLOOD PROFILE - 5 YEAR
- FLOOD PROFILE - 10 YEAR
- FLOOD PROFILE - 20 YEAR
- FLOOD PROFILE - 35 YEAR
- FLOOD PROFILE - 50 YEAR
- FLOOD PROFILE - 75 YEAR
- FLOOD PROFILE - 100 YEAR
- FLOOD PROFILE - 200 YEAR
- FLOOD PROFILE - 350 YEAR
- FLOOD PROFILE - 500 YEAR
- FLOOD PROFILE - 750 YEAR
- FLOOD PROFILE - 1000 YEAR

REFERENCE

FLOOD PROFILES FROM HEC-RAS MODELLING, RIVER THALWEG FROM SURVEY DATA COLLECTED BY GOLDER FROM MAY 14-17 AND JUNE 11-13, 2019 AS PART OF THE CARDSTON FLOOD HAZARD STUDY.

0 1,000 2,000

HORIZONTAL SCALE 1:50,000 METRES
VERTICAL EXAGGERATION = 100X

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
OPEN WATER FLOOD PROFILES - LEE CREEK

CONSULTANT

GOLDER

YYYY-MM-DD	2020-01-13
PREPARED	JDS
DESIGN	PH
REVIEW	HKG
APPROVED	WP

PROJECT No.	CONTROL	Rev.	FIGURE
19117525	3000	0	22

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

5.6 Model Sensitivity

5.6.1 Purpose

Sensitivity analyses were conducted to evaluate the effects of changing model parameters on the simulated 100-year flood water levels. The model parameters included in the sensitivity analyses are the downstream boundary condition and Manning's *n* values for channels and floodplains. The results of the sensitivity analyses are used to quantify the level of uncertainty associated with the simulated 100-year flood levels.

5.6.2 Boundary Conditions

The following two model downstream boundary scenarios were evaluated:

a) Excluding the St. Mary River in the model and assuming a normal depth boundary condition at the downstream end of Lee Creek; and

b) Including the St. Mary River in the model to provide water level boundary condition at the confluence of Lee Creek and St. Mary River.

Figure 23 shows the comparison of the simulated water levels for 100-year flood event in Lee Creek between these two scenarios. As shown in Figure 23, scenario b (including the St. Mary River) resulted in higher simulated water levels in Lee Creek upstream of the confluence. The average water level difference (i.e., simulated water level including the St. Mary River minus simulated water level excluding the St. Mary River) in Lee Creek upstream of the confluence is +0.03 m, with a maximum difference of 1.42 m at the Lee Creek confluence.

As shown in Figure 23, adjusting downstream boundary would affect a distance of up to approximately one kilometer upstream of the confluence. Therefore, St. Mary River was included in the model to account for the backwater effect from the river on Lee Creek, and to provide the downstream boundary condition for simulating the Lee Creek water levels using the HEC-RAS model.

The sensitivity analyses were performed to assess the effects of varying the assumed downstream boundary condition on the upstream water levels for the following cases:

a) change of the flow in the St. Mary River by ±20% from the base value

b) change of the downstream energy slope in the St. Mary River by ±20% from the base value

By changing the flow in the St. Mary River by ±20% from the base value, the water level at the Lee Creek confluence increased by +0.40 m and reduced by -0.39 m, respectively. The effect on the Lee Creek water level was up to approximately one kilometer upstream of the Lee Creek confluence, as shown in Figure E1.

By changing the downstream energy slope in St. Mary River by ±20% from the base value, the water level at the Lee Creek confluence reduced by -0.02 m and increased by +0.03 m, respectively. The effect on the Lee Creek water level was up to approximately one kilometer upstream of the Lee Creek confluence, as shown in Figure E2.

The detailed results of the sensitivity analysis of the downstream boundary condition are presented in Figure E-1 and Figure E-2 in Appendix E.

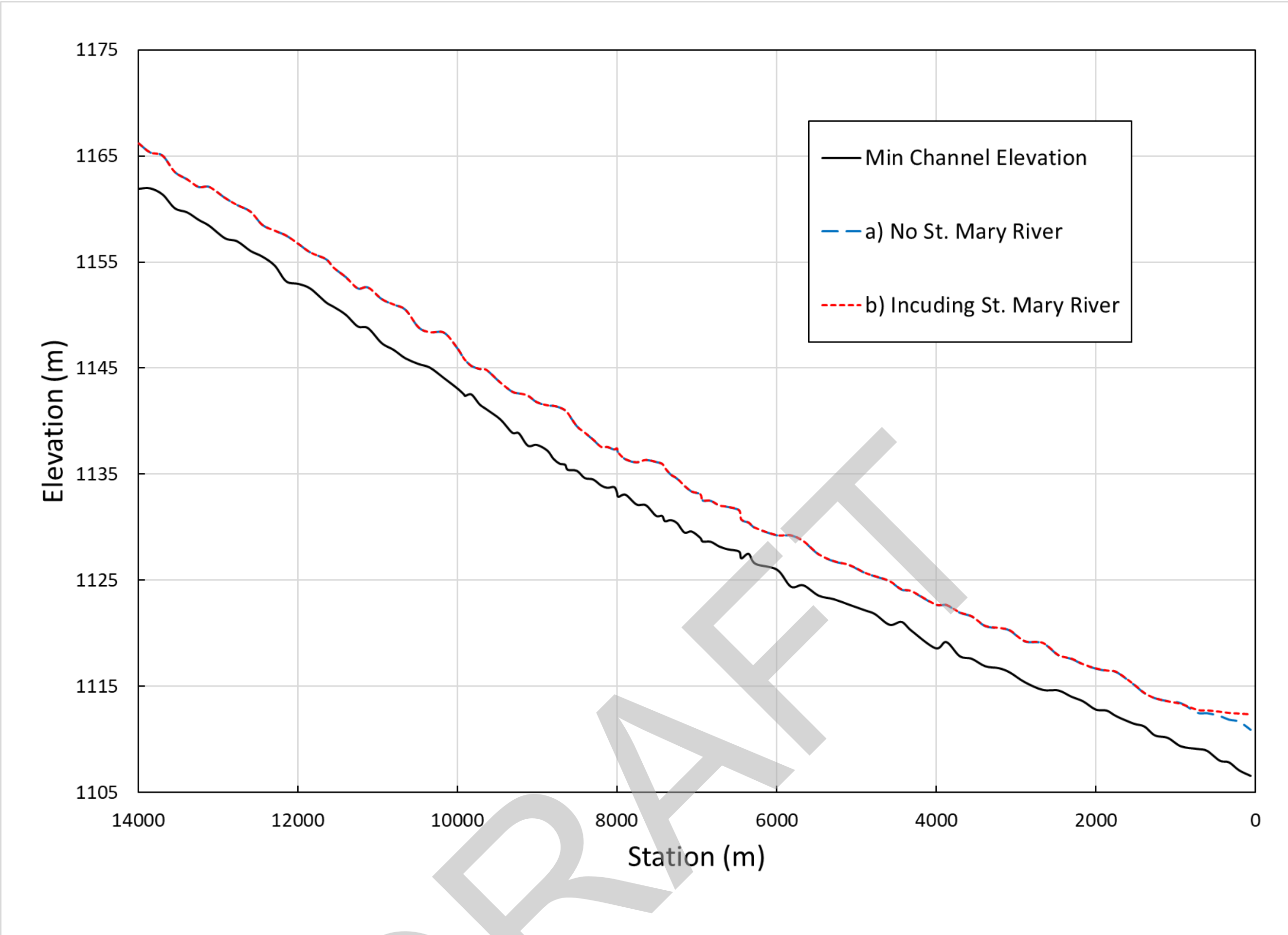


Figure 23: Effects of the Model Downstream Boundary Condition Types

5.6.3 Manning Roughness

Channel Roughness

The main channel Manning's *n* values were increased and decreased by 10% for the sensitivity analysis. The results of the sensitivity analysis of the channel Manning's *n* values are presented in Figure E-3 in Appendix E. The average water level differences to the base case are +0.06 m and -0.05 m in Lee Creek for main channel roughness increase and decrease, respectively. The maximum and minimum water level differences are +0.38 m and -0.28 m, respectively.

Floodplain Roughness

The floodplain Manning's *n* values were increased and decreased by 10% for the sensitivity analysis. The results of the sensitivity analysis of the floodplain Manning's *n* values are presented in Figure E-4 in Appendix E. The average water level differences to the base case are +0.02 m and -0.03 m in Lee Creek for floodplain roughness increase and decrease, respectively. The maximum and minimum water level differences are +0.35 m and -0.46 m, respectively.

Channel and Floodplain Roughness

All Manning's *n* values (both main channel and floodplain) were increased and decreased by 10% for the sensitivity analysis. The results of the sensitivity analysis of the combined channel and overbank Manning's *n* values are presented in Figure E-5 in Appendix E. The average water level differences to the base case are +0.08 m and -0.08 m in Lee Creek for channel and floodplain roughness increase and decrease, respectively. The maximum and minimum water level differences are +0.32 m and -0.28 m, respectively.

5.6.4 Summary

The sensitivity analysis results are summarized in Table 23.

Table 23: Summary of Sensitivity Analysis Results

Water Body	Parameter	Water Level Difference (Sensitivity Case – Base Case) (m)									
		Downstream Boundary (Flow in St. Mary River)		Downstream Boundary (St. Mary River Energy Slope)		Channel Manning's *n*		Floodplain Manning's *n*		Channel and Floodplain Manning's *n*	
		+20%	-20%	+20%	-20%	+10%	-10%	+10%	-10%	+10%	-10%
Lee Creek	Maximum	0.40	0.05	0.00	0.03	0.38	0.38	0.35	0.13	0.32	0.00
	Minimum	0.00	-0.39	-0.02	-0.01	-0.25	-0.28	-0.46	-0.18	-0.17	-0.28
	Average	0.02	-0.01	0.00	0.00	0.06	-0.05	0.02	-0.03	0.08	-0.08

6.0 FLOOD INUNDATION MAPS

6.1 Methodology

6.1.1 Map Preparation

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

- the simulated water levels at individual cross sections for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1,000-year flood events
- the locations and extents of individual cross sections
- the LiDAR DTM
- the information about dedicated flood control structures

The inundation maps show the areas along the study reach of Lee Creek. No mapping was prepared for the St. Mary River.

The purpose of the flood inundation maps is to show both direct flood inundation areas and areas at risk of flooding due to potential flood control structure failure.

The full set of open water flood inundation maps is provided in a separate document (i.e., Appendix F: Open Water Flood Inundation Map Library).

6.1.2 Direct Flood Inundation Areas

Direct flood inundation areas are identified either as being part of the actively-flowing creek channel or flooded overbank areas directly connected to the actively-flowing creek channel. The following general procedure was used in ArcGIS to develop the inundation extent for the 13 open water flood events:

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

Areas showing extensive overbank flooding connected to the channel at one distinct location (overtopping point) were adjusted such that the water surface elevation across that area was set equal to the water surface elevation at the overtopping point. This generally reduced the size of the inundated area extending upstream of an overtopping point and increased the size of the inundated area extending downstream of the overtopping point. These adjustments may result in a new overtopping point forming downstream. In these cases, the water surface elevations in the overbank area were re-adjusted such that they were interpolated linearly between the upstream overtopping point and the ground elevation at the new downstream overtopping point.

In addition to the general procedure described above, the following adjustments were made:

- Backwater inundation for relatively large tributaries was included and delineated based on the simulated water levels at the main channel at the confluences of those tributaries. This applies to Unknown Creek which is a tributary to Lee Creek.
- Areas expected to be inundated during a flood event but not delineated automatically using ArcGIS were delineated manually using break lines to properly map such complex areas. This was applied to the area downstream of 1 Avenue East.

6.1.3 Potential Flood Control Structure Failure Inundation Areas

Areas at risk of inundation due to potential flood control structure failure were mapped based on main channel water levels. Isolated areas behind flood control structures are only mapped as flood control structure failure if the flood water level in the main river channel is higher than the natural ground or the toe of the control structure, as shown in Figure 24.

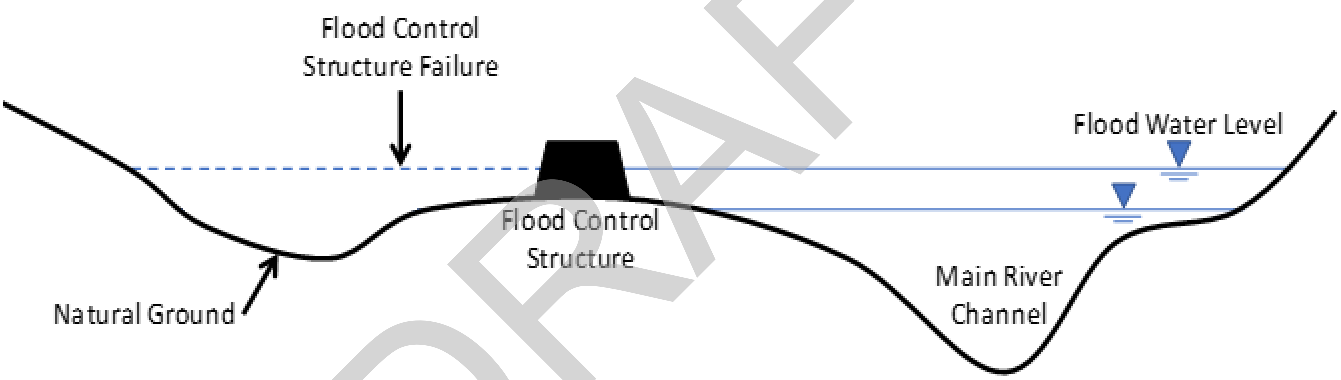
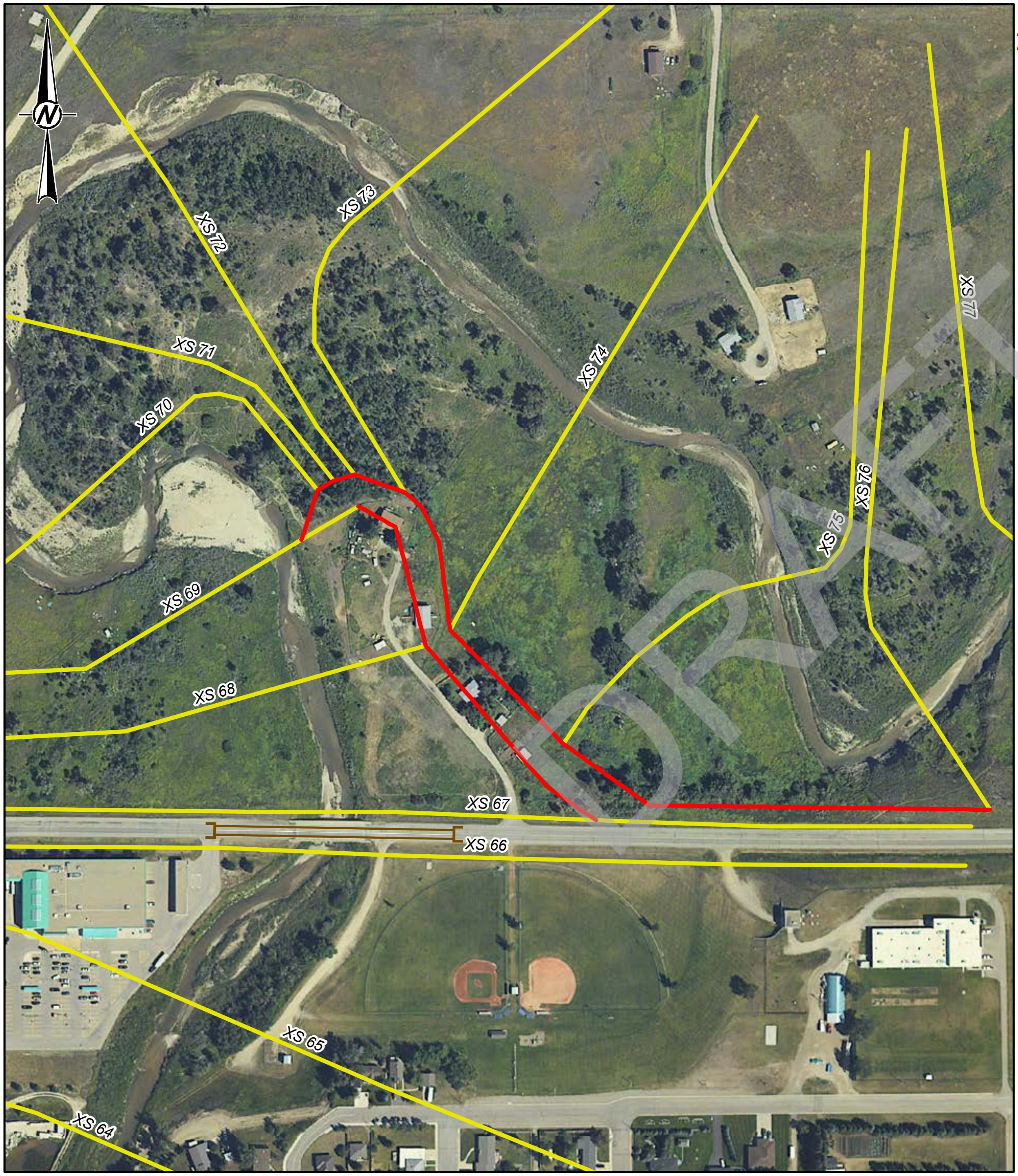


Figure 24: Illustration of Flood Control Structure Failure Inundation and Isolated Area Inundation

6.1.4 Mapping of Special Areas

A large and sharp meander immediately downstream of 1 Avenue East bridge results in dense cross sections inside the bend. To properly map water levels downstream of 1 Avenue East bridge, two breaklines were introduced at the mapping stage, as shown in Figure 25. These breaklines help to prevent unrealistic water surface interpolation between the cross section immediately downstream of the bridge and the cross sections that are approximately one kilometer downstream of the bridge (e.g., XS 74, XS 75 and XS 76). The area with the local road and buildings on the right floodplain immediately downstream of 1 Avenue East bridge is roughly level with the upstream top of right bank for three cross sections downstream of 1 Avenue East bridge, and thus the water surface elevations in this area are governed by those cross sections.



LEGEND

BRIDGE

WATER SURFACE 3D BREAKLINE

WATER SURFACE ISOLINE

0 100 200

1:4,000 METERS

NOTE(S)

WATER SURFACE BREAKLINES INSERTED TO GUIDE WATER SLOPE BETWEEN UPSTREAM AND DOWNSTREAM RIGHT BANKS OF LEE CREEK NORTH OF HIGHWAY 501. THE AREA WITH LOCAL ROAD AND BUILDINGS IS ROUGHLY LEVEL WITH THE UPSTREAM TOP OF BANK SO IT LARGELY HAS THE WATER SURFACE ELEVATIONS GOVERNED BY THE UPSTREAM CROSS SECTIONS.

REFERENCE(S)

IMAGERY CAPTURED JULY 2019 BY OGL ENGINEERING FOR THE GOVERNMENT OF ALBERTA.
DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT

ALBERTA ENVIRONMENT AND PARKS

PROJECT

CARDSTON FLOOD HAZARD STUDY

TITLE

WATER SURFACE MODIFICATIONS NORTH OF HIGHWAY 501

CONSULTANT

GOLDER

YYYY-MM-DD	2020-03-27
DESIGNED	PH
PREPARED	NB
REVIEWED	HKG
APPROVED	WP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	25

6.2 Flood Impacts

6.2.1 Direct Flood Inundation Areas

The residential and commercial areas affected by direct inundation are described below. Detailed inundation maps are provided in a separate inundation map library document.

- The residential and commercial areas on the west side of Lee Creek around 3 Avenue West would be inundated starting at the 35-year flood.
- The Lions Park would be inundated starting at the 35-year flood.
- Larger areas of the Town would be inundated starting at the 100-year flood.

6.2.2 Potential Flood Control Structure Failure

Failure of Lee Creek Dike could result in flooding of commercial and residential areas on the west side of Lee Creek upstream of 3 Avenue West. The dike would be overtopped at the 35-year flood event.

7.0 DESIGN FLOOD HAZARD DETERMINATION AND MAP PRODUCTION

7.1 Design Flood Details

The 100-year flood was selected as the open water design flood in accordance with the Flood Hazard Identification Program (FHIP) Guidelines (AEP, 2011). Flood Hazard Maps were prepared for the study reach of Lee Creek. No mapping was prepared for the St. Mary River.

7.2 Floodway and Flood Fringe Terminology

The flood hazard area is the area of land that will be flooded during the design flood event. The flood hazard area is typically divided into two zones: floodway and flood fringe. Flood hazard maps can also show additional flood hazard information, including areas of high hazard within the flood fringe and incremental areas at risk for more severe floods such as the 200-year and 500-year floods. Flood hazard mapping is typically used for long-term flood hazard area management and land-use planning. The floodway and flood fringe zones are defined as follows:

- <u>Floodway:</u> When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard where flows are deepest, fastest, and most destructive during the 100-year design flood. The floodway generally includes areas where the water is 1 m deep or greater and the local velocities are 1 m/s or faster. The floodway typically includes the main channel of a stream and a portion of the adjacent overbank area. Previously mapped floodways do not typically become larger when a flood hazard map is updated, even if the flood hazard area gets larger or design flood levels get higher. New development is discouraged in the floodway and may not be permitted in some communities
- <u>Flood Fringe:</u> The flood fringe is the portion of the flood hazard area outside of the floodway. The flood fringe typically represents areas with shallower (less than 1 m deep), slower (less than 1 m/s velocity), and less destructive flooding during the 100-year design flood. However, areas with deep or fast moving water may also be identified as high hazard flood fringe within the flood fringe. Areas at risk behind flood berms may also be mapped as protected flood fringe areas. New development in the flood fringe may be permitted in some communities.

7.3 Floodway Determination Criteria

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

- Areas in which the depth of water exceeds 1 m or the flow velocities are greater than 1 m/s shall be part of the floodway.
- Exceptions may be made for small backwater areas, ineffective flow areas, and to support creation of a hydraulically smooth floodway.
- For reaches of supercritical flow, the floodway boundary should correspond to the edge of inundation or the main channel, whichever is larger.

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

Areas of deeper or faster moving water outside of the floodway are identified as high hazard flood fringe. These high hazard flood fringe zones are identified in all areas, whether they are newly-mapped or have an existing floodway.

- The depth and velocity criteria used to define high hazard flood fringe zones will be aligned with the 1 m depth and 1 m/s velocity floodway determination criteria for newly-mapped areas.
- All areas protected by dedicated flood berms that are not overtopped during the design flood are excluded from the floodway. Areas behind flood berms will still be mapped as flooded if they are overtopped, but areas at risk of flooding behind dedicated flood berms that are not overtopped will be mapped as a protected flood fringe zone.

The floodway determination criteria for the left and right floodway limits at each cross section are provided together with the design flood levels in Table G-1 in Appendix G. The governing criteria for Lee Creek was generally based on the previous floodway (approximately 400 m upstream of the footbridge at the Lee Creek Valley Golf Course to approximately 1,500 m downstream of 1 Avenue E Bridge (ID 78730)). Where previous floodway information is not available, the 1 m depth, 1 m/s velocity or main channel criterion was used.

7.4 Floodway Criteria Maps

Floodway criteria maps are a tool for determining floodway and flood fringe extents for the design flood, including boundaries of high hazard flood fringe and protected flood fringe areas. The Open Water Floodway Criteria Maps provided in the Maps and Drawings section of this report show:

- inundation extents of the 100-year open water design flood
- areas where the depth of water is 1 m or greater and the corresponding 1 m depth contour
- the portions of each cross section where the computed velocity is 1 m/s or faster
- the proposed floodway boundary, as well as the associated floodway stations corresponding to the floodway determination criteria
- isolated areas of non-flooded, high ground (i.e., “dry areas”) within the design flood extent

- the locations of the main channel top of bank at each cross section
- the location and extent of all cross sections used in the HEC-RAS model
- the previous-mapped floodway boundary (where it exists)
- background aerial imagery collected in 2019
- roads, bridges, culverts and flood control structures as applicable

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

Flood Depth Determination

A flood depth grid was generated by subtracting the water level surface TIN (see Section 6.1.2.) for the design flood event from the digital terrain model. This flood depth grid was used to identify areas meeting or exceeding the 1m depth criterion and to generate 1 m depth contour lines.

Flow Velocity Computations

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

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

7.5 Flood Hazard Mapping

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

Based on the flood hazard maps, the Lion Park is within the high hazard flood fringe zone. Cardston County Emergency Services, Westwind School, Cardston Recreation Centre (swimming pool), Southwest Concrete Products and Co-op Gas Station are within the flood fringe zone.

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

7.6 Quantitative Climate Change Assessment

A simplified climate change assessment was completed to quantify the effects on the 100-year flood water levels for both 10% and 20% peak discharge increases for Lee Creek. Table 24 summarizes the average water level increases in Lee Creek under the assumed climate change conditions.

It is acknowledged that this simplified analysis is not based on a regional climate change impacts assessment but are based on a basic assumption that climate change will result in increased flood peak flows. The presented values can be viewed as a general range of potential climate change "freeboard" values that could be considered in addition to the computed design flood water levels.

Table 24: Effect of Increased Flows on the Water Levels in Lee Creek

Parameter	10% Increase in 100-year Flood Peak Discharge	20% Increase in 100-year Flood Peak Discharge
Average difference in water levels (m)	0.2	0.3

8.0 CONCLUSIONS

8.1 Model Calibration

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

The channel Manning's *n* roughness coefficient is the main model parameter used in calibrating the HEC-RAS model. The calibrated channel Manning's *n* values are in the range of 0.025 to 0.040 along the Lee Creek study reach. These Manning's *n* values are within the typical ranges of roughness values for similar water courses (Chow 1959).

8.2 Model Sensitivity

Model sensitivity was evaluated using the 100-year flood simulation results. The results of the sensitivity analysis show that variation of the main channel roughness values has a higher influence on the simulated water levels than variation of the floodplain roughness values along Lee Creek. A variation of the main channel and floodplain Manning's *n* values by ±10% resulted in changes of the simulated water levels within 0.08 m along Lee Creek. A variation of the flow in the St. Mary River by ±20% resulted in changes of the simulated water levels within 0.4 m at the Lee Creek confluence.

8.3 Flood Profiles

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

8.4 Flood Inundation Mapping

Flood inundation maps were prepared for the study reach of Lee Creek using ArcGIS. The simulated flood water levels at the cross sections were used to create a continuous water surface. The edge of inundation was delineated by subtracting the LiDAR DTM from the water surface.

Based on the simulation results, the main residential and/or commercial development areas that would be flooded within the study area have been identified as follows:

- The residential and commercial areas on the west side of Lee Creek around 3 Avenue West would be inundated starting at the 35-year flood.
- The Lions Park would be inundated starting at the 35-year flood.
- Larger areas of the Town would be inundated starting at the 100-year flood.

8.5 Flood Hazard Determination and Mapping

This project was undertaken per the FHIP guidelines incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta, and project Terms of Reference. The results of the design flood hazard mapping are the delineation of the floodway and flood fringe zones and determination of the design flood water levels.

Based on the flood hazard maps, the Lion Park is within the high hazard flood fringe zone. Cardston County Emergency Services, Westwind School, Cardston Recreation Centre (swimming pool), Southwest Concrete Products and Co-op Gas Station are within the flood fringe zone.

Signature Page

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WP/HKG/PHI

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APPENDIX A

Open Water Hydrology Assessment

GOLDER

TECHNICAL MEMORANDUM

DATE March 11, 2020 **Project No.** 19117525-008-TM-Rev0

TO Mr. Muhammad Durrani, M. Eng., P. Eng.
Alberta Environment and Parks

CC Wolf Ploeger and Dejiang Long

FROM Getu Biftu; Mesgana Gizaw **EMAIL** gbiftu@golder.com

OPEN WATER HYDROLOGY ASSESSMENT

1.0 INTRODUCTION

1.1 Study Area and Scope

Alberta Environment and Parks (AEP) commissioned Golder Associates Ltd. (Golder) in March 2019 to conduct the Cardston Flood Hazard Study. The purpose of the study is to assess and identify river and flood hazards along Lee Creek through the Town of Cardston and adjacent areas (see Figure 1). The study is part of the provincial Flood Hazard Identification Program (FHIP), the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Cardston, Cardston County, and the general public. The project includes working with Kainai Nation.

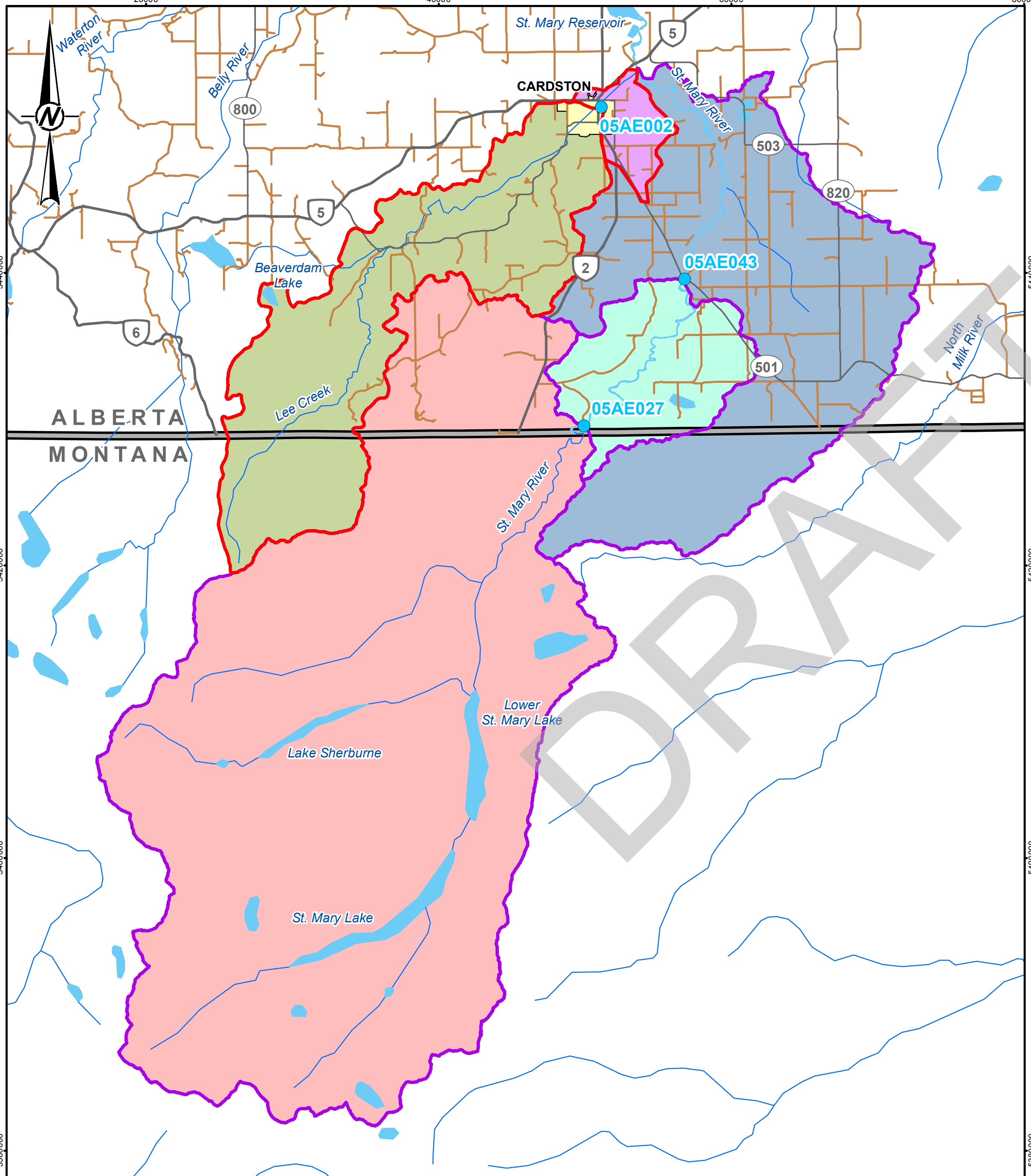
The study comprises multiple components and deliverables. This memorandum documents the methodology and results of the open water hydrology assessment that will support the hydraulic modelling and open water flood mapping. The individual tasks associated with this hydrology assessment component include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

- Compile available peak flow information for gauged locations and prepare flood flow data series;
- Conduct frequency analyses to estimate flood flows for return periods ranging from 2 to 1,000 years using the recorded and derived flood peak data for the available period of record up to 2018; and
- Provide comments and insight into how climate change processes may affect the flood peak discharges and flood frequency estimates.

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

1.2 Study Objectives and Results

The primary study objective is to identify and assess flood hazards along approximately 12 km of Lee Creek through Cardston, and adjacent areas of Cardston County and Kainai Nation. The objective of the open water hydrology assessment is to generate flood peak discharge estimates of various return period s for Lee Creek. To support hydraulic modelling, corresponding estimates are also generated for the St. Mary River above the Lee Creek confluence. The results of the frequency analysis include 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year flood peak flow estimates.



LEGEND

- HYDROMETRIC GAUGING STATION
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- LOCAL ROAD
- WATERCOURSE
- WATERBODY
- PROVINCIAL/STATE BOUNDARY
- POPULATED PLACE
- LEE CREEK BASIN AT WSC 05AE002
- LEE CREEK BASIN AT ST. MARY RIVER CONFLUENCE
- ST. MARY RIVER BASIN AT WSC 05AE027
- ST. MARY RIVER BASIN AT WSC 05AE043
- ST. MARY RIVER BASIN UPSTREAM OF LEE CREEK CONFLUENCE

DRAFT

0 15 30

1:425,000 KILOMETRES

REFERENCE(S)

HYDROMETRIC STATIONS OBTAINED FROM AGRICULTURE AND AGRI-FOOD CANADA (AAFC). POPULATED PLACES OBTAINED FROM ALTALIS, © GOVERNMENT OF ALBERTA 2017. ALL RIGHTS RESERVED.
ROADS AND HYDROGRAPHY OBTAINED FROM GEOGRATIS, © DEPARTMENT OF NATURAL RESOURCES CANADA. ALL RIGHTS RESERVED.
DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
STUDY WATERSHEDS

CONSULTANT

GOLDER

YYYY-MM-DD	2019-09-10
DESIGNED	MG
PREPARED	PT
REVIEWED	
APPROVED	

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This study includes the use of preliminary estimates of annual peak flows in 2016, 2017 and 2018 for Lee Creek at Cardston, provided by Water Survey of Canada (WSC). Including these provisional data increases the sample sizes for the flood frequency analysis and the reliability of the resulting flood frequency estimates.

However, it is important to note that provisional and preliminary data are subject to change when reviewed and revised by the WSC. Therefore, the flood frequency statistics presented in this memorandum should be used with caution and reviewed when the finalized flows are available.

1.3 Watershed Setting and Flood History

Lee Creek originates in Montana, U.S.A, and flows in a north-easterly direction and passes through Cardston and then joins the St. Mary River in Kainai Nation. The Lee Creek drainage basin is mostly located in south-western Alberta with a small part of the upper basin in Montana. The drainage areas of Lee Creek and St. Mary River upstream of Cardston are comprised of mountainous terrain in the west, heavily forested foothills and parklands and cultivated prairie. Lee Creek has a reported drainage area of 312 km^2 at the WSC gauging station at Cardston (WSC Station 05AE002).

The largest flood peak discharge recorded on Lee Creek is 323 m^3/s which occurred on June 8, 1964. The second largest recorded flood occurred on June 20, 1975, with a peak instantaneous discharge of 226 m^3/s. The recorded peak instantaneous discharge for the flood of June 21, 1991 (59.2 m^3/s) approximates the bankfull capacity of Lee Creek (Stanley 1992).

Available records indicate that major flood events occurred on Lee Creek in 1948, 1951, 1953, 1964, 1975, 1981, 1995, 2002, 2005 and 2010. These floods were typically associated with high rainfall or rain-on snow events in June, except for the flood event on May 22, 1981.

2.0 AVAILABLE FLOW DATA

2.1 Recorded Data

Recorded flow data is publicly-available for Lee Creek at Cardston between 1909 and 2015, and preliminary annual maximum instantaneous discharge data were obtained from WSC for 2016, 2017 and 2018.

Recorded flow data for the St. Mary River is also available at two locations upstream of the study area. Recorded flow data for St. Mary River are regulated since 1905 with water being diverted from St. Mary River by the St. Mary Diversion Dam just downstream from the outlet of Lower St. Mary Lake.

Table 1 provides a summary of the basic hydrologic information used to derive the flood frequency estimates for Lee Creek and St. Mary River at location within the study area. The data details are provided in Appendix A.

Table 1: Summary of the Gauged Station

WSC Station Number	WSC Station Name	Latitude	Longitude	Gross Drainage Area (km^2)	Effective Drainage Area (km^2)	Period of Record	Length of Record (year)
05AE002	Lee Creek at Cardston	49° 11' 59"	113° 17' 48"	312	312	1909-2018	105
05AE027	St. Mary River at International Boundary	49° 00' 43"	113° 17' 58"	1210	1157	1903-2018	116
05AE043	St. Mary River at Highway No. 501	49° 05' 30"	113° 13' 15"	1320	1250	1998-2017	20

2.2 Historic Data

No historic flow information is available before systematic gauging and monitoring by the WSC.

2.3 Previous Studies

This study included a review of a number of background documents, including previous hydrology and flood studies. Several hydrology studies were completed for Lee Creek over the last two decades, some of which included assessments of open water hydrology. The previous studies include:

- Flood Frequency Analysis of Lee Creek at Cardston by Alberta Environment (AENV 1991);
- Cardston Hydraulic Study by Stanley Associates Engineering Ltd. (Stanley 1992); and
- Hydro-Climate Modelling of Alberta South Saskatchewan Regional Planning Area (Golder 2010).

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

3.0 PREPARATION OF FLOOD FLOW DATA SERIES

3.1 Introduction

Preparation of the flood flow series for both Lee Creek and St. Mary River involved consideration of a large number of factors, including incomplete flow record. The methods used to compile the flood flow series and to address the data gaps are described in the following sections.

3.2 Flood Flow Series for Gauged Locations

The flood frequency estimates for the gauged locations was derived based on the recorded annual maximum instantaneous discharge series, and where there is missing data, the annual maximum daily discharges were used to estimate the instantaneous flood flows.

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

- Annual maximum daily discharge series were developed using the recorded daily flow series.
- A relationship was established between event-based annual maximum daily and annual maximum instantaneous discharges in the record (see Figure 2 and Figure 3). If the reported annual maximum daily and annual maximum instantaneous discharges for the same year are not coincident (i.e., from the same flood event), the former values are replaced by the daily flow values for the events corresponding to the annual maximum instantaneous discharges. The relationship was used to estimate the annual maximum instantaneous discharges based on the recorded annual maximum daily discharges.

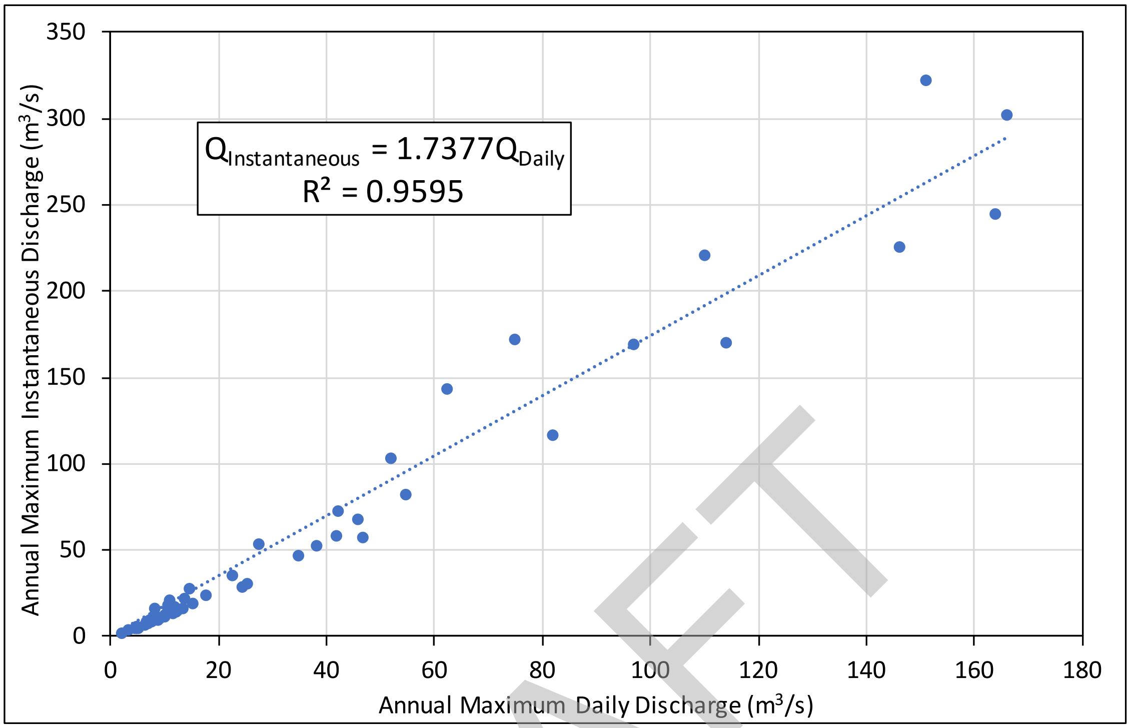

Figure 2: Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges for Lee Creek at Cardston (WSC Station No. 05AE002)

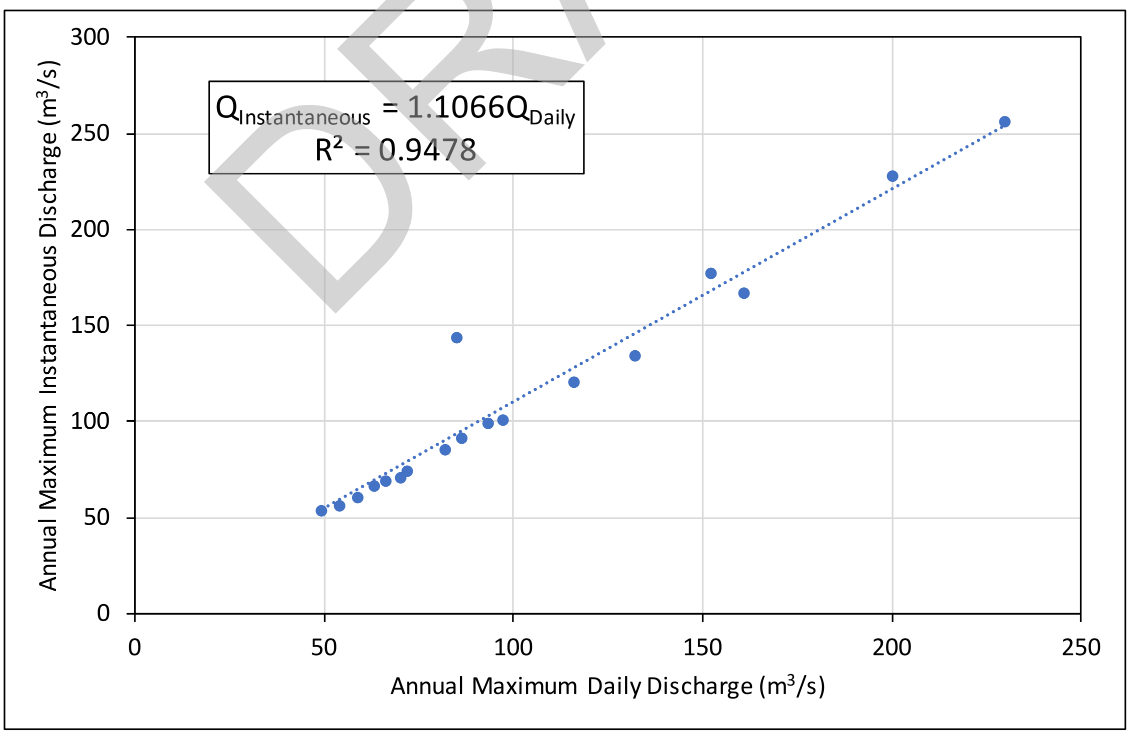

Figure 3: Relationship between Annual Maximum Daily and Annual Maximum Instantaneous Discharges for St. Mary River at Highway No. 501 (WSC Station No. 05AE043)

3.3 Flood Flow Series for the Ungauged Location

The flood frequency estimates for the ungauged location (i.e., St. Mary River above the Lee Creek confluence, with gross and effective drainage areas of 1745 km^2 and 1630 km^2, respectively) were estimated based on the recorded flood data at the two upstream gauged locations [St. Mary River at International Boundary (WSC Station No. 05AE027) and St. Mary River at Highway No.501 (WSC Station No. 05AE043)] as follows:

- A relationship was established between event-based annual maximum instantaneous discharge records at WSC Station Nos. 05AE027 and 05AE043 (see Figure 4) based on the recorded data from 1998 to 2017. If the reported annual maximum instantaneous discharges of the two stations were not coincident in any given year, the record for that year was not included in the data used to establish the relationship.
- The relationship established in Figure 4 was used to extend the annual maximum instantaneous discharge at WSC Station No. 05AE043 for the period from 1903 to1997.
- Flood frequency estimates for St. Mary River at Highway No. 501 (WSC Station No. 05AE043) for a range of return periods were derived using the extended series of maximum instantaneous discharges.
- Flood frequency estimates for St. Mary River above the Lee Creek confluence were then derived based on the estimates for WSC Station No. 05AE043 and proportion of the two drainage areas.

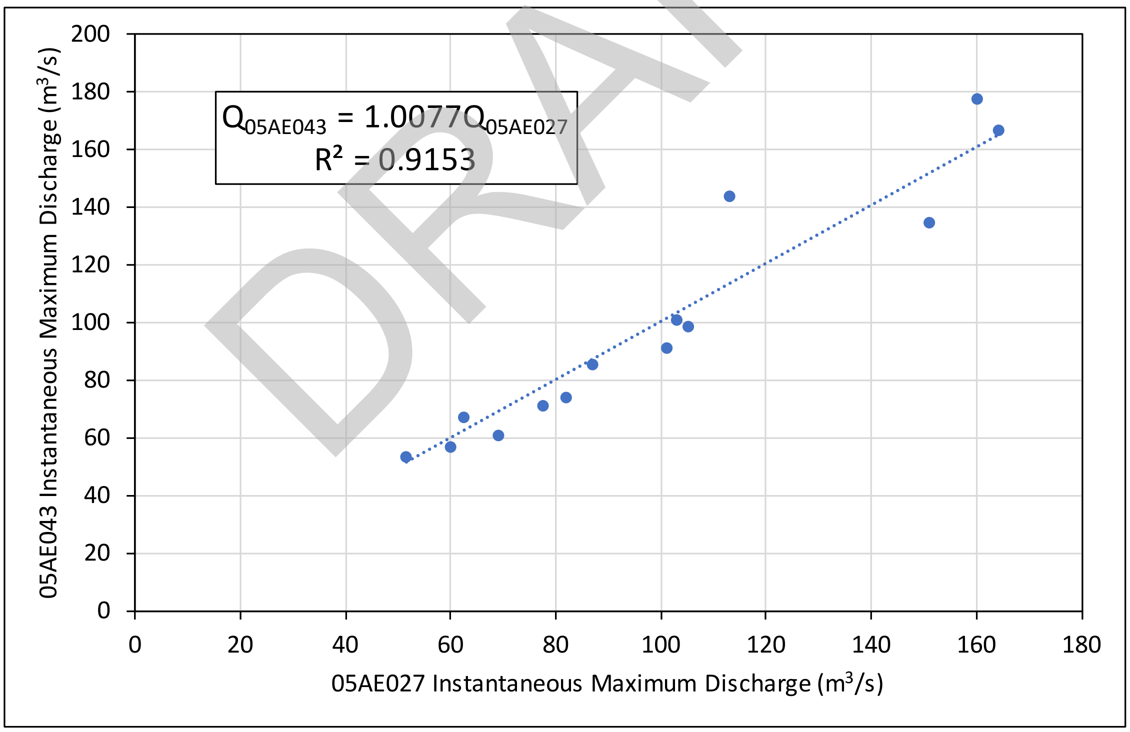

Figure 4: Relationship between Recorded Annual Maximum Instantaneous Discharges at Two WSC Stations

As indicated in Section 2.1, recorded flow data for St. Mary River are regulated. This study did not include naturalization of recorded flow data. Using recorded regulated flow data series is acceptable for current study, as the flows are simply being used to assess a downstream model boundary condition. It is expected that the annual maxima flood series for naturalized flow conditions would likely be higher and could result in flood frequency estimates higher than those estimated using regulated flow series for all return periods.

4.0 FLOOD FREQUENCY ANALYSIS

4.1 Statistical Tests

4.1.1 Methodology

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

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

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

Numerical goodness-of-fit was assessed using the non-parametric Anderson-Darling test (Stephens, 1974). In addition, engineering judgements will be used to make sure that the distribution fit selected based on numerical goodness-of-fit reflects the probability plots of recorded data for all return periods. The final frequency estimates were selected considering both the results of statistical tests and the best overall distribution fits for all return periods.

4.1.2 Results

Table 2 provides the results of statistical tests for the recorded flood flow series. The results show that the annual maximum instantaneous flood flow series are independent, random, homogeneous, and do not display any significant trends.

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

WSC Station Number	05AE002	05AE043
Station Name or Location of Interest	Lee Creek at Cardston	St. Mary River at Highway No. 501[2]
Anderson-Darling statistic, A^2 = - N -S		
3 Parameter Log-normal	0.558	0.901
Extreme Value	1.530	0.805[1]
Log-Pearson III	0.315[1]	1.583
Weibull	7.330	NA[3]
Serial correlation coefficient test for independence		
S_1	0.0411	0.0857
t	0.4092	0.9142
t(α=0.05)	1.6604	1.6585
t(α=0.01)	2.3646	2.3598
Spearman rank order correlation coefficient test for no-trend		
$_s$	0.0459	0.1363
t	0.4590	1.4695
t(α=0.05)	1.9840	1.9810
t(α=0.01)	2.6259	2.6196
Manna-Whitney split sample test for homogeneity		
Size of earlier sample	50	50
z	-0.5958	-0.4321
z(a=0.05)	-1.6449	-1.6449
z(a=0.01)	-2.3263	-2.3263
Test of general randomness (Runs for above or below the median)		
Median	19.6	117.4
N1(for Q>=Median)	51	58
N2(for Q<Median)	51	58
Run_ab	45	55
z	1.3931	0.7460
z(a=0.05)	1.9600	1.9600
z(a=0.01)	2.5758	2.5758

Notes:
1. Best distribution fit based on statistical test only.
2. Based on the recorded data for the period 1998-2017 and derived data for the period 1903-1997.

NA = not available for the distribution.

4.2 Flood Frequency Estimates

4.2.1 Analyses

Flood frequency analysis of the annual maximum instantaneous discharge series was conducted to estimate peak flow estimates for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1,000-year floods.

4.2.2 Results

Table 3 summarizes the flood peak discharge estimates and the associated upper and lower 95% confidence intervals based on the overall best fit distributions selected for Lee Creek and St. Mary River.

The Anderson-Darling test best-fit statistics for 3P and LP3 are not dramatically different (0.558 vs 0.315). Based on the frequency plots, the difference between the plots for 3P and LP3 for low flow events (i.e., less than 20-year return period) are similar. However, the LP3 distribution significantly deviates from the data for high flow events (i.e., higher than the 50-year flow, including the highest three data points). Hence, 3P is a better overall distribution fit and recommended for current study. Moreover, the flood frequency estimates using 3P distribution fit are consistent with the estimates obtained in the 1991 study.

For the St. Mary River, the selection of EV2 over 3P or LP3 is based on Anderson-Darling test best-fit statistics of 0.805 vs 0.901 and 1.583. In this case, 3P and LP3 appears to significant under-represent data above the 20-year event, including the four highest recorded flows. Hence, EV2 is a better overall distribution fit and recommended for current study.

The annual maximum instantaneous discharge series used in the flood frequency analysis, the various frequency distributions, and the best-fit distributions along with their 95% confidence intervals, are provided in Appendix B.

Table 3: Computed Flood Peak Discharge and their 95% Confidence Interval

Return Period	Lee Creek at Cardston (WSC Station No. 05AE002)			St. Mary River at Highway No. 501 (WSC Station No. 05AE043)			St. Mary River above the Lee Creek Confluence[1]		
	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)	Peak Discharge (m^3/s)	95% Upper Bound (m^3/s)	95% Lower Bound (m^3/s)
2	23	28	18	108	120	98	141	156	128
5	62	80	47	176	201	154	230	263	201
10	105	145	75	240	288	194	314	377	253
20	163	241	111	321	411	231	419	537	301
35	223	352	148	402	548	260	525	715	340
50	269	440	174	463	656	280	605	856	366
75	328	559	206	543	814	303	709	1060	396
100	375	658	232	608	941	321	793	1230	420
200	510	967	303	794	1350	362	1040	1760	473
350	642	1290	370	984	1790	396	1280	2340	516
500	739	1540	418	1127	2160	418	1470	2820	546
750	862	1870	476	1314	2660	443	170	3470	578
1000	959	2130	522	1465	3090	462	1910	4030	603

1. Prorated from St. Mary River at Highway No. 501 based on ratio of effective drainage areas.

4.3 Comparison to Previous Studies

Table 4 compares the flood frequency estimates from this study with a previous study (AENV, 1991).

The flood frequency estimates in the previous study were based on the recorded data up to 1989. The current study is based on published flow data from 1909 to 2015, and provisional flow data from 2016 to 2018. This study also includes an analysis to update the relationship between annual maximum daily and annual maximum instantaneous discharges to allow the peak flow series to be filled when only daily data are available.

Table 4: Comparison of the Flood Frequency Estimates with the Previous Study

Return Period (years)	Peak Flood Frequency Flows Estimates for Lee Creek at Cardston (m^3/s)	
	1991 Study (AENV, 1991)	Current Study
	3P-Log-Normal	3P-Log-Normal (MLH)
2	19	23
5	53	62
10	90	105
20	141	163
50	234	269
100	328	375
200	448	510

The resulting flood frequency estimates for Lee Creek at Cardston are higher than those in the previous study. The main differences in the flood frequency estimates are due to the different lengths of the recorded data used in the flood frequency analyses.

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

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

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

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

Golder (2010) completed an assessment of the effect of climate change using five selected representative GCMs and scenarios outputs from Alberta Climate Model for the Oldman River basin. The five selected scenarios represent climate conditions that were cooler and drier (CGCM2-B23), cooler and wetter (NCARPCM-A1B), warmer and wetter (HADCM3-A2A), and warmer and drier (CCSRNIES-A1F1) than median conditions (HADCM3-A2A).

The forecasted climate change is between the modelled baseline period (1961 to 1990) as represented by its 30-year average and the modelled future period (i.e., the period of 2040 to 2069 called the 2050s) as represented by its 30-year average. The results indicate that the changes in flood peaks for the Belly River watershed that is located close to Lee Creek will vary from no change for the 2-year flood to a slight decrease (i.e., less than 5%) for the 100-year flood for the median climate change conditions. Therefore, the changes in the flood peak discharges for Lee Creek are expected to be small for the median climate change projections.

The 1964 flood on Lee Creek has been the largest flood since 1910, as illustrated in Figure 5. Based on the recorded flow data for the past 109 years (i.e., 1910 to 2018), the annual peak flows on Lee Creek do not appear to be trending upward. Any upward trend shown in Figure 5 is not statistically significant.

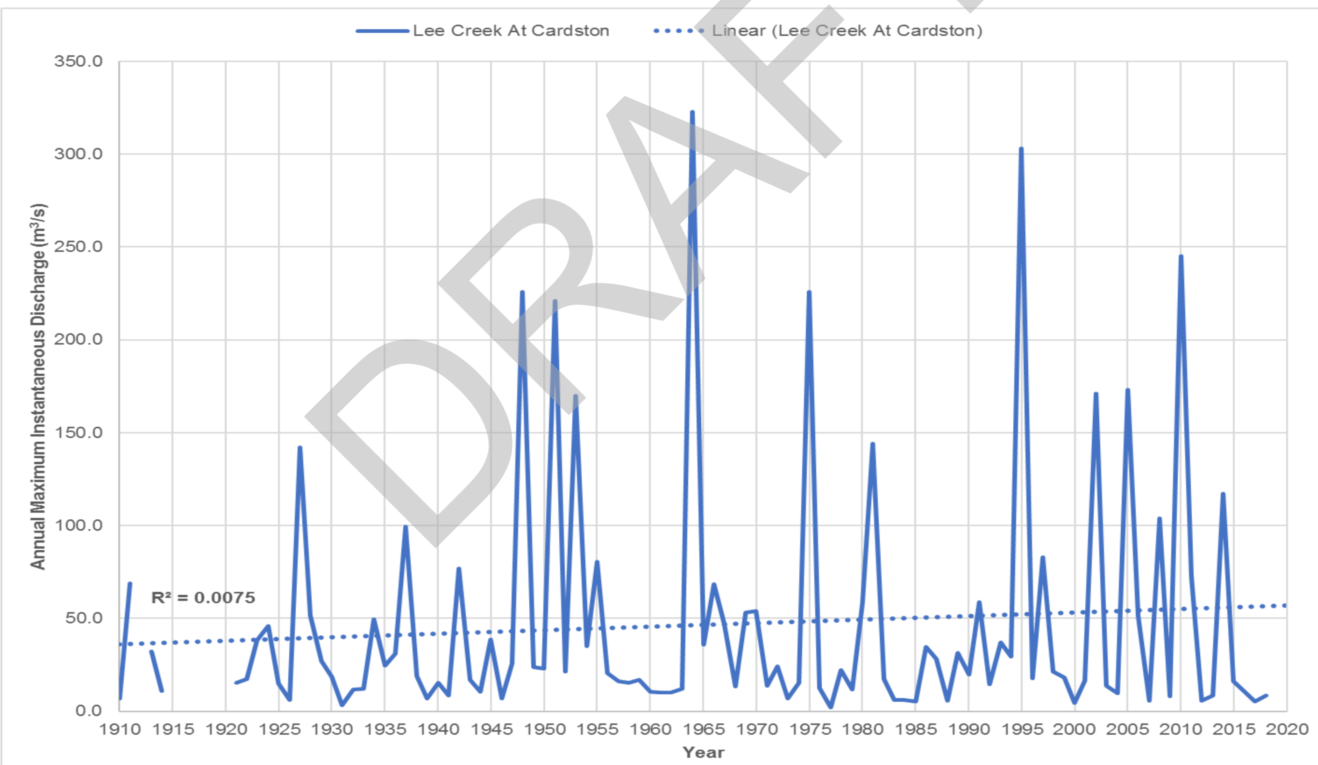

Figure 5: Annual Flood Peak Discharges on the Lee Creek at Cardston

Approximately 75% of the recorded annual maximum discharges on Lee Creek occurred between the beginning of May and end of June (see Table 5 and Figure 6). There is no clear evidence that the patterns in magnitude or timing of annual maximum discharges have changed significantly over the past 100 years. However, the frequency of annual maximum discharges occurring earlier than May has decreased since the 1990s from that for the period 1940 to 1990.

Table 5: Timing of Annual Maximum Instantaneous Flows for Lee Creek at Cardston (1910-2018)

Month	Number	Percent of Total
February	1	1
March	8	8
April	7	7
May	35	36
June	37	39
July	6	6
August	0	0
September	1	1

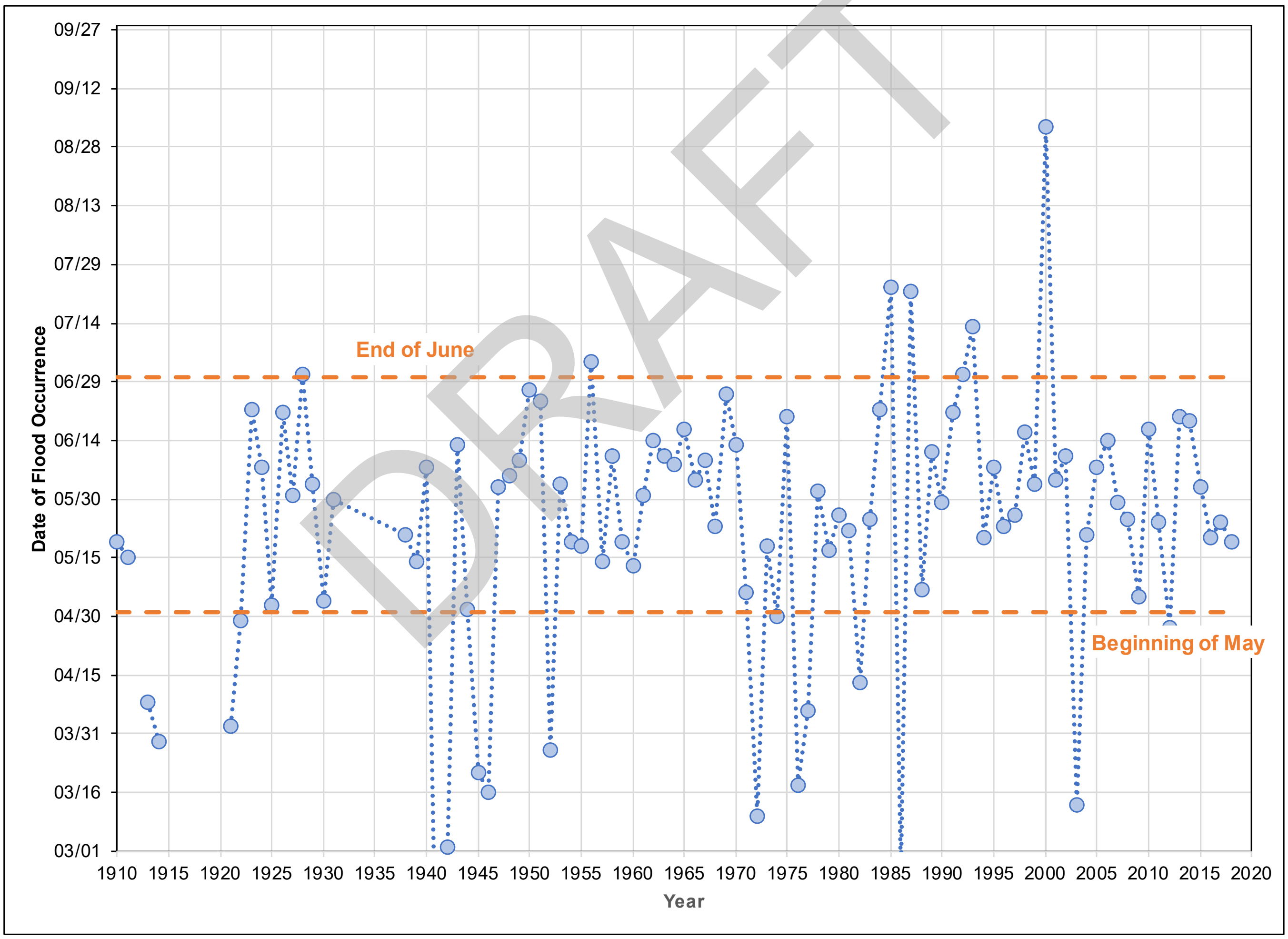

Figure 6: Timing of Past Annual Maximum Flows for Lee Creek at Cardston

6.0 CONCLUSIONS

Table 3 provides a summary of the recommended estimates of flood peak discharges for the various return periods ranging from 2 to 1,000 years, and the 95% upper and lower confidence intervals.

The results of this hydrology assessment support the following conclusions:

- The flood frequency estimates obtained in this study are the most up-to-date for Lee Creek at Cardston. These estimates provide the updated flood hydrology information as inputs for the hydraulic modelling and flood mapping components of the Cardston Flood Hazard Study.
- The period of the record for the flood flow data used in the flood frequency analyses for Lee Creek and the St. Mary River is just over 100 years. Therefore, there are large uncertainties (i.e., the confidence intervals are very large) with flood frequency estimates for return periods greater than 100 years.

7.0 CLOSURE

This memorandum is prepared and reviewed by the undersigned. If you have any questions or require additional details, please contact the undersigned.

Golder Associates Ltd.

APEGA PERMIT TO PRACTICE #05122

Prepared by: Reviewed by:

Original signed/stamped by: ***Original signed/stamped by:***

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GB/DL/al/pls

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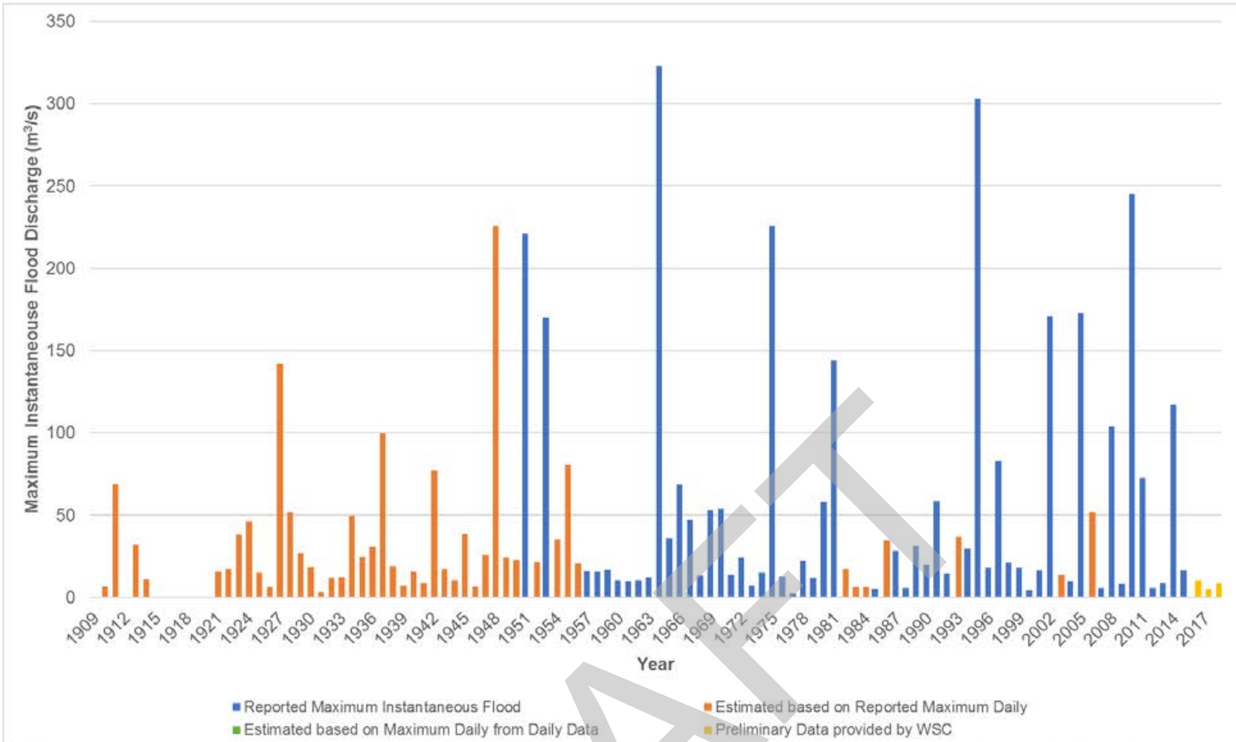
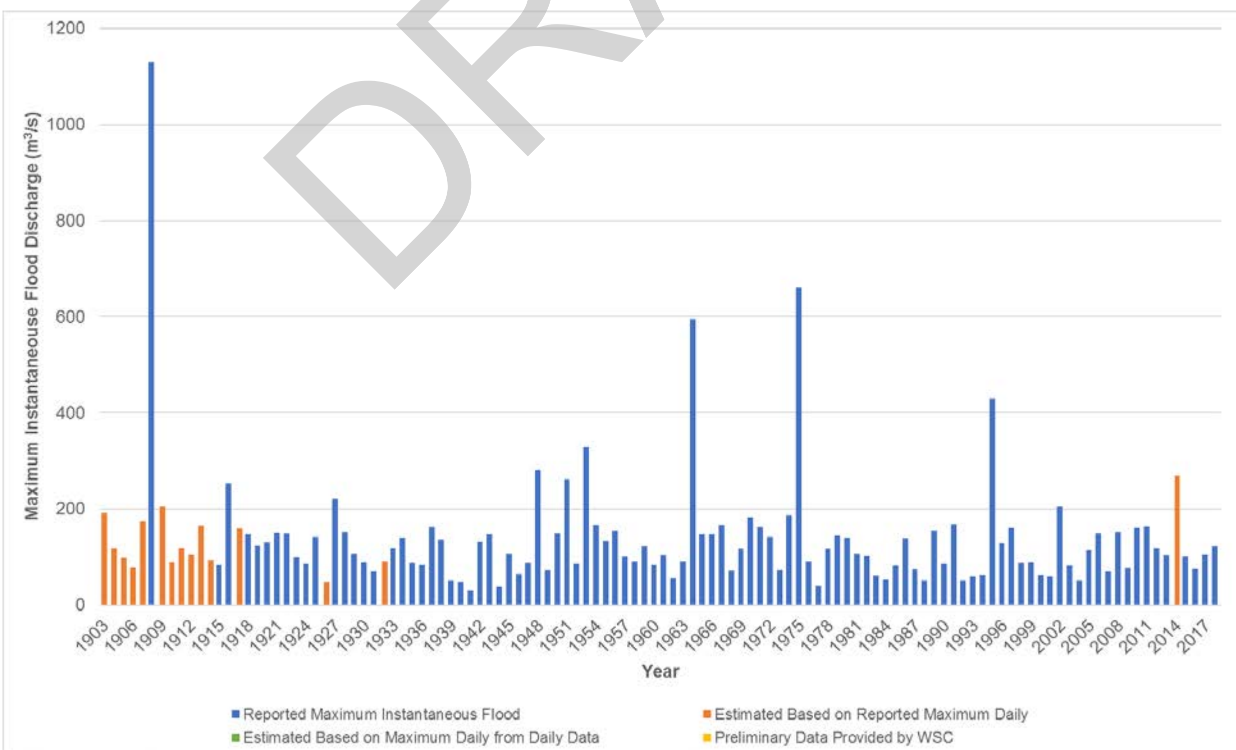
APPENDIX A

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

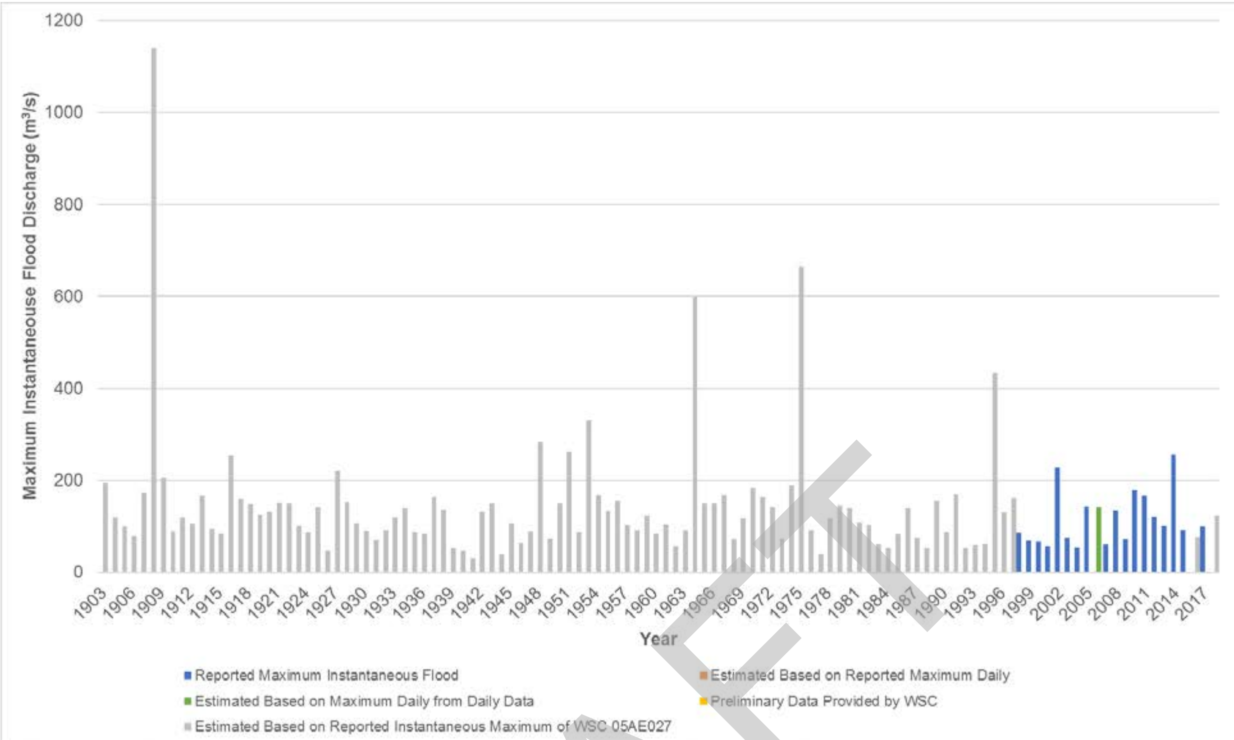
Figure A-1: WSC Station No. 05AE002 (the Lee Creek at Cardston)



Maximum Instantaneous Flood Flow Series at the Lee Creek at Cardston (WSC Station No. 05AE002)



Maximum Instantaneous Flood Flow Series at the St. Mary River at International Boundary (WSC Station No. 05AE027)



Maximum Instantaneous Flood Flow Series at the St. Mary River at Highway No. 501 (WSC Station No. 05AE043)

Table A-1: Data Used for the Flood Frequency Analysis

Year	WSC Station #05AE002 (the Lee Creek at Cardston)	WSC Station # 05AE027 (the St. Mary River at International Boundary)	WSC Station #05AE043 (the St. Mary River at Highway No. 501)
1903		191.6	193.0
1904		117.6	118.5
1905		98.1	98.9
1906		78.9	79.6
1907		173.2	174.5
1908		1130.0	1138.7
1909		204.2	205.8
1910	6.8	88.3	89.0
1911	68.8	117.6	118.5
1912	-	104.7	105.5
1913	32.1	165.2	166.5
1914	10.9	93.3	94.1
1915	-	83.0	83.6
1916	-	253.0	254.9
1917	-	159.5	160.7
1918	-	147.0	148.1
1919	-	124.0	125.0
1920	-	130.0	131.0
1921	15.6	150.0	151.2
1922	17.4	149.0	150.1
1923	38.2	99.4	100.2
1924	46.0	85.8	86.5
1925	14.9	141.0	142.1
1926	6.2	47.2	47.6
1927	142.1	220.0	221.7

Table A-1: Data Used for the Flood Frequency Analysis

Year	WSC Station #05AE002 (the Lee Creek at Cardston)	WSC Station # 05AE027 (the St. Mary River at International Boundary)	WSC Station #05AE043 (the St. Mary River at Highway No. 501)
1928	51.6	151.0	152.2
1929	26.9	106.0	106.8
1930	18.6	89.5	90.2
1931	3.1	69.1	69.6
1932	11.9	90.9	91.6
1933	12.3	118.0	118.9
1934	49.7	140.0	141.1
1935	24.7	86.7	87.4
1936	30.9	83.5	84.1
1937	99.4	162.0	163.2
1938	19.1	135.0	136.0
1939	6.9	51.0	51.4
1940	15.4	47.6	48.0
1941	8.6	30.6	30.8
1942	76.8	131.0	132.0
1943	17.2	148.0	149.1
1944	10.6	37.4	37.7
1945	38.6	106.0	106.8
1946	6.7	64.3	64.8
1947	25.7	88.1	88.8
1948	225.9	281.0	283.2
1949	24.0	72.8	73.4
1950	22.9	149.0	150.1
1951	221.0	261.0	263.0
1952	21.5	86.1	86.8
1953	170.0	328.0	330.5
1954	35.3	166.0	167.3
1955	80.6	132.0	133.0
1956	20.5	154.0	155.2
1957	16.1	101.0	101.8
1958	15.5	90.6	91.3
1959	17.0	123.0	123.9
1960	10.6	83.5	84.1
1961	10.1	103.0	103.8
1962	10.3	55.5	55.9
1963	12.3	90.6	91.3
1964	323.0	595.0	599.6
1965	36.0	148.0	149.1
1966	68.5	148.0	149.1
1967	47.3	167.0	168.3
1968	13.3	71.4	71.9
1969	53.2	117.0	117.9
1970	54.1	182.0	183.4
1971	13.8	162.0	163.2
1972	24.1	141.0	142.1
1973	7.1	72.5	73.1
1974	15.3	187.0	188.4
1975	226.0	660.0	665.1
1976	12.7	90.9	91.6
1977	2.2	39.1	39.4
1978	22.3	116.0	116.9
1979	11.6	144.0	145.1
1980	58.3	140.0	141.1
1981	144.0	107.0	107.8
1982	0.0	102.0	102.8

Table A-1: Data Used for the Flood Frequency Analysis

Year	WSC Station #05AE002 (the Lee Creek at Cardston)	WSC Station # 05AE027 (the St. Mary River at International Boundary)	WSC Station #05AE043 (the St. Mary River at Highway No. 501)
1983	0.0	60.9	61.4
1984	0.0	52.1	52.5
1985	5.4	81.3	81.9
1986	0.0	139.0	140.1
1987	28.2	73.6	74.2
1988	5.8	51.0	51.4
1989	31.5	155.0	156.2
1990	20.0	86.4	87.1
1991	58.7	168.0	169.3
1992	14.5	51.3	51.7
1993	0.0	58.9	59.4
1994	29.5	62.3	62.8
1995	303.0	430.0	433.3
1996	17.9	129.0	130.0
1997	82.8	160.0	161.2
1998	21.3	86.8	85.8
1999	18.2	89.5	69.2
2000	4.5	62.3	67.3
2001	16.5	60.0	56.9
2002	171.0	205.0	228.0
2003	0.0	81.8	74.4
2004	9.9	51.5	53.7
2005	173.0	113.0	144.0
2006	0.0	149.0	141.6
2007	5.7	69.1	61.1
2008	104.0	151.0	135.0
2009	8.3	77.6	71.6
2010	245.0	160.0	178.0
2011	72.8	164.0	167.0
2012	5.7	118.0	121.0
2013	8.7	103.0	101.0
2014	117.0	268.2	257.0
2015	16.3	101.0	91.4
2016	10.4	76.2	76.8
2017	5.3	105.0	99.1
2018	8.7	123.0	123.9
Maximum	323.0	1130.0	1138.7
Mean	46.9	139.0	139.9
Minimum	2.2	30.6	30.8
Standard Deviation	65.3	128.6	129.8

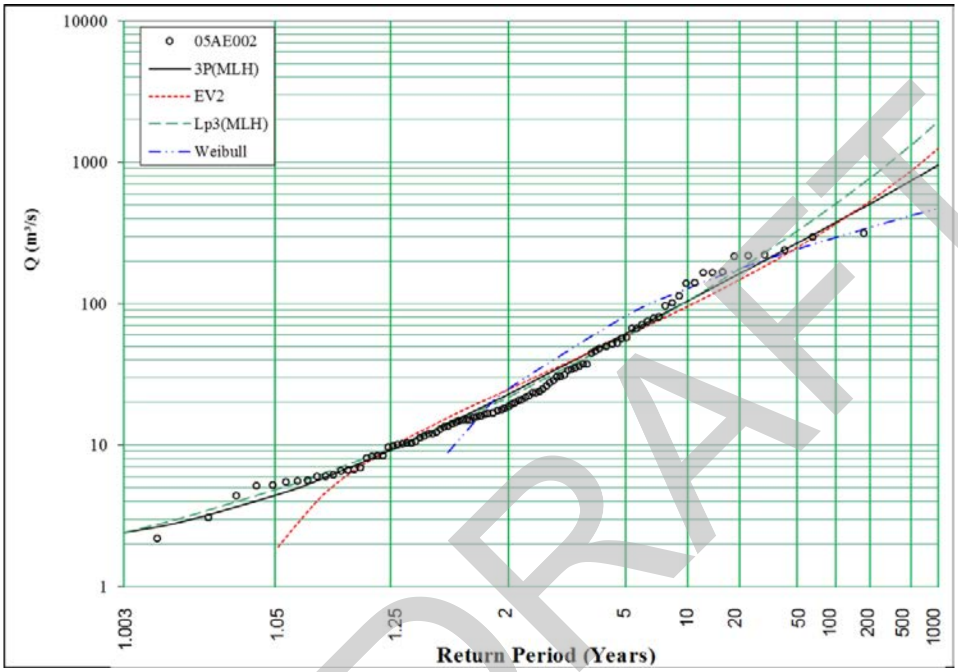
APPENDIX B

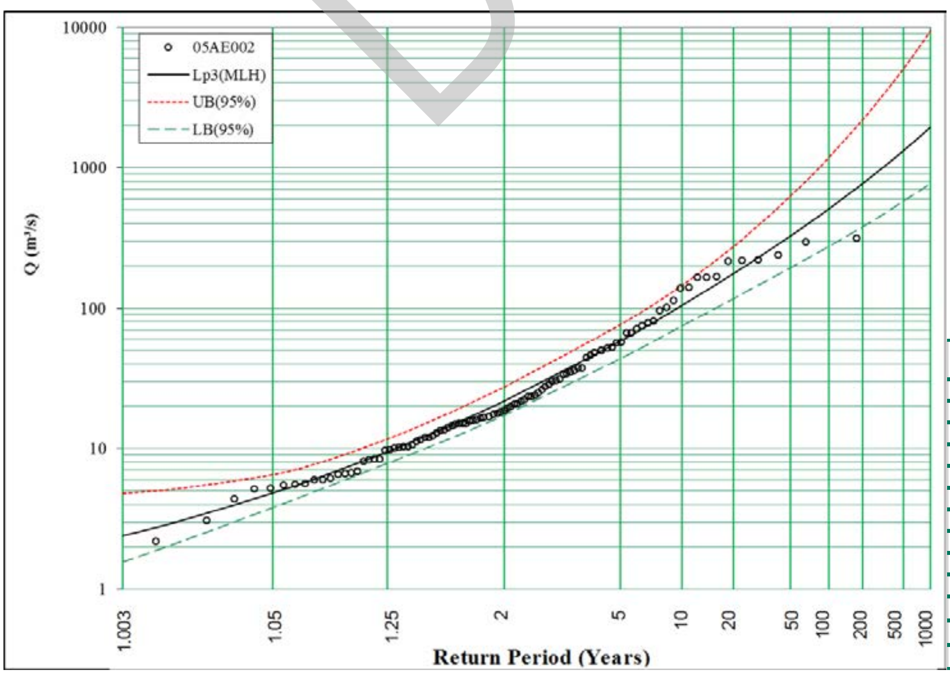
Frequency Analyses - Graphs and Tables

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

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

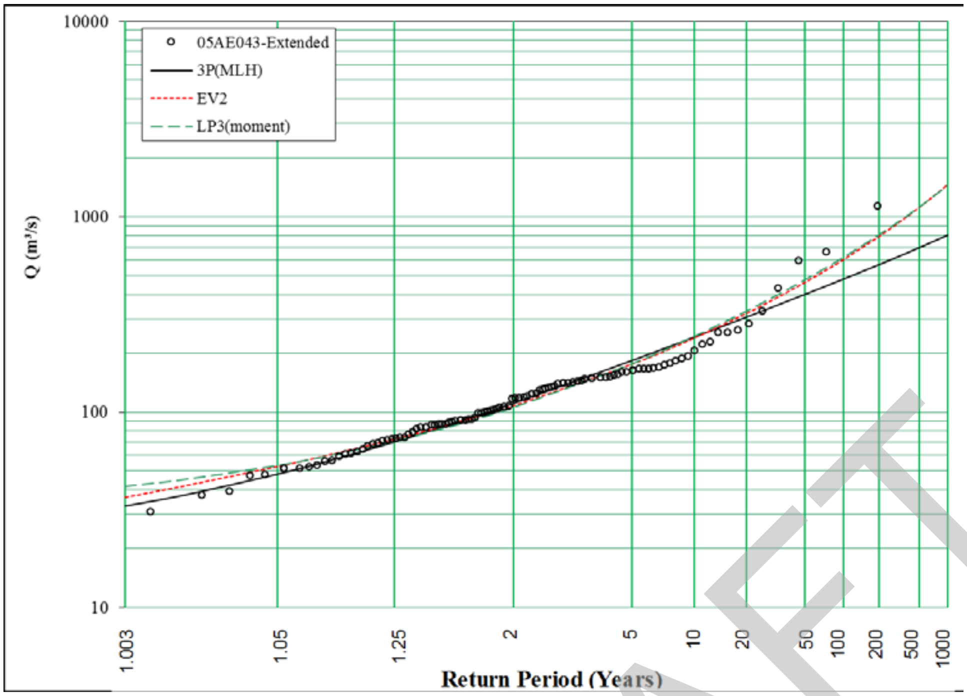
Figure B-1: WSC Station No.05AE002 (the Lee Creek at Cardston)

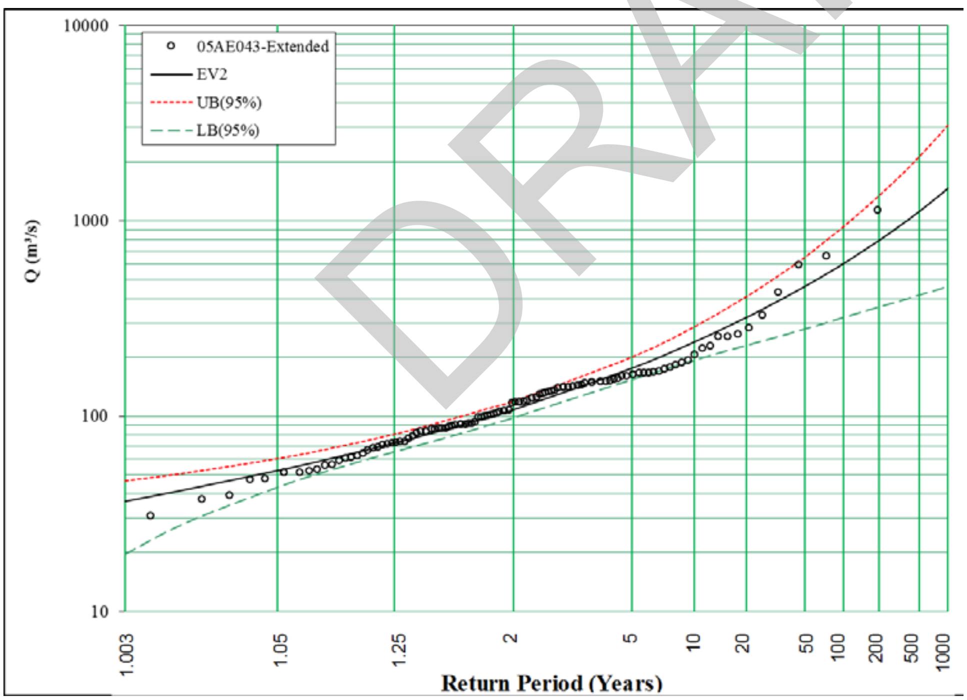




Return Period	3P(MLH)	EV2	LP3 (MLH)	Weibull
2	23	25	22	25
5	62	59	59	82
10	105	96	105	128
20	163	147	176	177
35	223	203	259	217
50	269	249	328	243
75	328	312	425	273
100	375	365	509	295
200	510	531	775	348
350	642	715	1076	391
500	739	864	1320	419
750	862	1070	1660	451
1000	959	1244	1948	474

Figure B-2: WSC Station No.05AE043 (the St. Mary River at Highway No. 501)





Return Period	3P(MLH)	EV2	LP3 (MLH)
2	111	108	105
5	184	176	177
10	243	240	245
20	308	321	330
35	364	402	415
50	403	463	478
75	449	543	559
100	483	608	624
200	571	794	809
350	648	984	994
500	700	1127	1132
750	762	1314	1310
1000	808	1465	1452

APPENDIX B

Flood Control Structures

TECHNICAL MEMORANDUM

DATE September 6, 2019 **Project No.** 19117525-005-TM-Rev0

TO Muhammad Durrani
Alberta Environment and Parks

CC Wolf Ploeger (Golder Associates Ltd.)

FROM Hossein Kheirkhah (Golder Associates Ltd.) **EMAIL** Gildeh@Golder.com

CARDSTON FLOOD HAZARD STUDY- FLOOD CONTROL STRUCTURES

1.0 INTRODUCTION

1.1 Study Background

Alberta Environment and Parks commissioned Golder Associates Ltd. in February 2019 to conduct the Cardston Flood Hazard Study.

The study is conducted under the provincial Flood Hazard Identification Program, the goals of which include enhancement of public safety and reduction of future flood damages through the identification of river and flood hazards. Project stakeholders include the Government of Alberta, the Town of Cardston, the Cardston County, Kainai Nation, and the public.

The Cardston Flood Hazard Study includes multiple components and deliverables. This memorandum documents existing flood control structures in the study area (Figure 1).

2.0 SURVEY PROGRAM

2.1 General

The survey of the stream cross sections, hydraulic structures, and flood control structures within the study area (Figure 1) was conducted between May 14, and June 13, 2019. In addition, water levels, discharges, and ASCM benchmarks were surveyed as part of this study. Surveyed cross sections, water levels and discharges will be used in the hydraulic model creation and calibration. The following section documents the existing flood control structures within the study area. The details of the stream survey are described in the hydraulic model creation and calibration report.

2.2 Flood Control Structure

There is one flood control structure within the study area. It is located on the left bank of Lee Creek downstream of the Highway 2 Bridge in the Town of Cardston. The structure is approximately 95 m long and consist of a steel sheet piles flood wall. The location of the structure within the study area is shown in Figure 2. Survey points were collected along the crest of the wall with a spacing of approximately 10 m.

Golder Associates Ltd.
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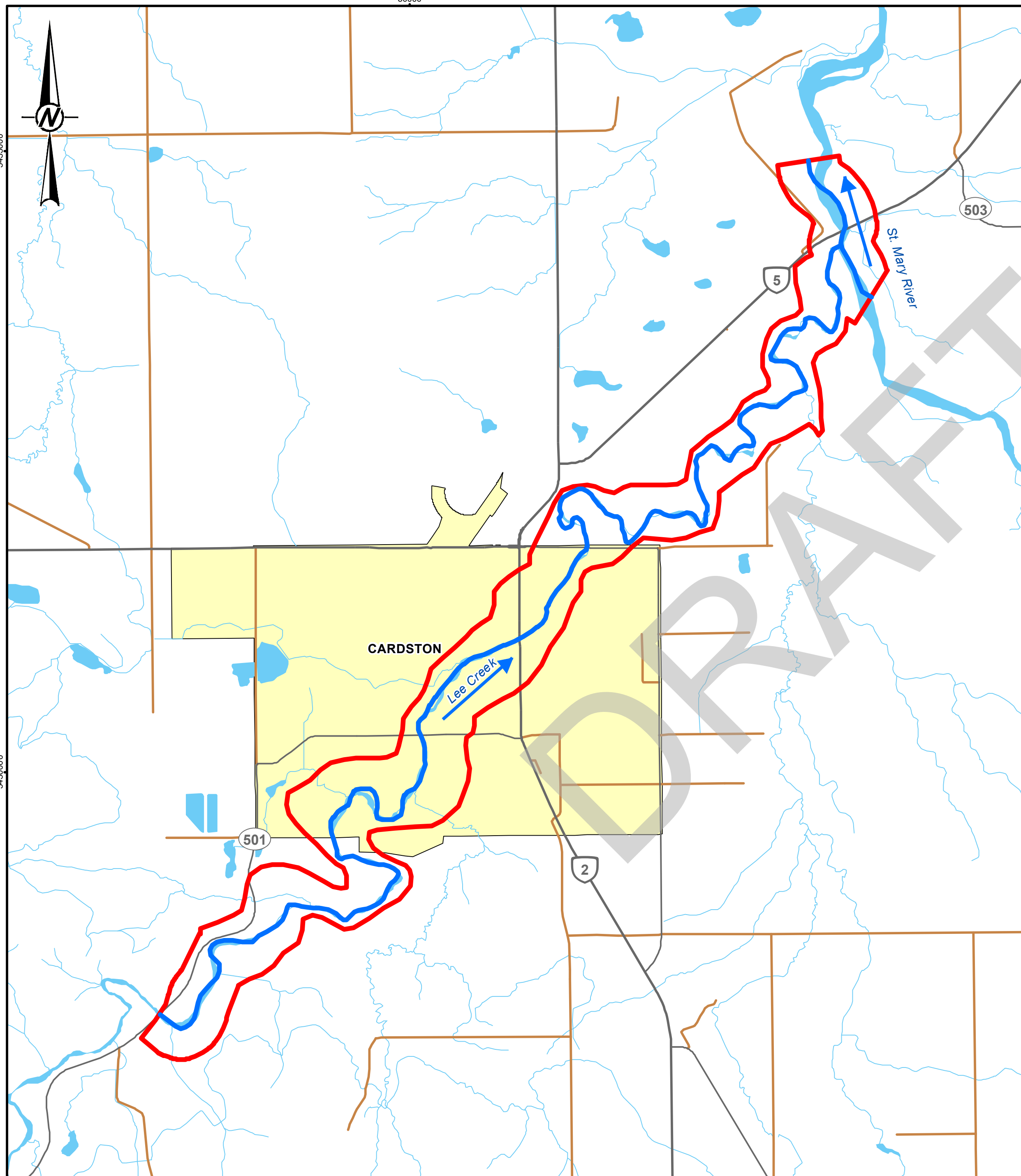

LEGEND

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

0 1 2

1:50,000 KILOMETRES

REFERENCE(S)

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

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
LOCATION MAP OF THE STUDY AREA

CONSULTANT

GOLDER

YYYY-MM-DD	2019-09-06
DESIGNED	PH
PREPARED	BP
REVIEWED	HKG
APPROVED	WP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	1000	0	1

LEGEND

- ROAD
- FLOW DIRECTION
- FLOOD CONTROL STRUCTURE

LOCATION	LEE CREEK
APPROX. LENGTH OF STRUCTURE (m)	95
TYPE OF STRUCTURE	STEEL SHEET PILES FLOOD WALL

NOTE(S)

FLOOD CONTROL STRUCTURE SURVEY COMPLETED TO SUPPORT HYDRAULIC MODELLING AND FLOOD MAPPING.

FOR MORE DETAILS SEE SECTION 2.6.

REFERENCE(S)

FLOOD CONTROL STRUCTURE SURVEY BY GOLDER ASSOCIATES LTD. JUNE 2019. ROADS OBTAINED FROM ALTALIS, © GOVERNMENT OF ALBERTA 2017. ALL RIGHTS RESERVED. IMAGERY CAPTURED 2018 FOR THE GOVERNMENT OF ALBERTA.
DATUM: NAD 83 CSRS PROJECTION: 3TM 114

0 75 150
1:2,500 METRES

TITLE
LEE CREEK DIKE

DESCRIPTION
EAST OF MAIN STREET, NORTH SIDE OF CREEK

CLIENT
ALBERTA ENVIRONMENT AND PARKS

PROJECT
CARDSTON FLOOD HAZARD STUDY

CONSULTANT

GOLDER

YYYY-MM-DD	2019-09-06
DESIGNED	PH
PREPARED	BP
REVIEWED	HGK
APPROVED	WP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	1000	0	2

3.0 SIGNATURE PAGE

APEGA PERMIT TO PRACTICE #05122

Prepared by:

ORIGINAL SIGNED

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Associate, Senior River Engineer

PH/HKG/WP/jlb

https://golderassociates.sharepoint.com/sites/106743/project files/5 technical work/01_survey & base data collection/04_reporting/rev 0/19117525-005-tm-rev0 flood control structure 20190906.docx

Third Party Disclaimer

APPENDIX C

Model Calibration Results

Table C.1: Comparison of Simulated Water Levels and Surveyed Highwater Marks along Lee Creek for 2014 High Flow Calibration

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Discharge	Date
1	8642	1138.4	1139.0	0.63	117	6/21/2014
2	8618	1138.1	1138.9	0.74	117	6/21/2014
3	7435	1133.8	1133.8	0.03	117	6/21/2014
4	7404	1133.8	1133.7	-0.08	117	6/21/2014
5	6949	1132.1	1131.6	-0.53	117	6/21/2014
6	6930	1132.1	1131.5	-0.61	117	6/21/2014

Table C.2: Comparison of Simulated Water Levels and Surveyed Highwater Marks along Lee Creek for 2010 High Flow Calibration

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Discharge	Date
1	8646	1138.7	1140.3	1.58	245	6/22/2010
2	8616	1138.2	1140.1	1.86	245	6/22/2010
3	7435	1133.7	1135.0	1.32	245	6/22/2010
4	7404	1133.5	1134.9	1.33	245	6/22/2010
5	6949	1132.1	1132.2	0.14	245	6/22/2010
6	6909	1131.6	1132.0	0.40	245	6/22/2010
7	6462	1129.8	1130.7	0.91	245	6/22/2010
8	6434	1129.7	1130.4	0.67	245	6/22/2010

Table C.3: Comparison of Simulated Water Levels and Surveyed Highwater Marks along Lee Creek for 1995 High Flow Calibration

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Discharge	Date
1	8954	1141.7	1141.3	-0.37	303	6/11/1995
2	8618	1140.3	1140.4	0.12	303	6/11/1995
3	7435	1134.3	1135.5	1.22	303	6/11/1995
4	6935	1132.2	1132.4	0.17	303	6/11/1995
5	6462	1130.4	1131.0	0.61	303	6/11/1995
6	6437	1130.0	1130.5	0.50	303	6/11/1995

Table C.4: Comparison of Simulated Water Levels and Surveyed Highwater Marks along Lee Creek for 1991 High Flow Calibration

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Discharge	Date
1	9841	1144.3	1143.9	-0.40	59	6/23/1991
2	8641	1138.3	1138.2	-0.15	59	6/23/1991
3	8615	1138.3	1138.0	-0.27	59	6/23/1991
4	7432	1132.4	1133.0	0.61	59	6/23/1991
5	7412	1132.4	1132.9	0.53	59	6/23/1991
6	6945	1130.1	1130.9	0.82	59	6/23/1991
7	6919	1130.1	1130.9	0.75	59	6/23/1991
8	6465	1128.9	1129.4	0.49	59	6/23/1991
9	6436	1128.8	1129.3	0.43	59	6/23/1991

Table C.5: Comparison of Simulated Water Levels and Surveyed Highwater Marks along Lee Creek for 1975 High Flow Calibration

No	River Station (m)	Surveyed Water Level (m)	Simulated Water Level (Interpolated from Cross Sections) (m)	Difference (Simulated – Surveyed) (m)	Discharge	Date
1	7544	1134.5	1134.9	0.43	226	6/20/1975
2	6849	1131.1	1131.7	0.58	226	6/20/1975

APPENDIX D

Flood Profiles

Table D.1: Lee Creek Flood Profiles

River	Reach	Station	Thalweg	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
LeeCreek	LeeCreek	13992	1,161.94	1163.69	1164.42	1164.92	1165.37	1165.68	1165.86	1166.05	1166.16	1166.48	1167.02	1167.37	1167.88	1168.24
LeeCreek	LeeCreek	13852	1,161.98	1163.24	1163.83	1164.19	1164.49	1164.74	1164.92	1165.15	1165.32	1165.86	1166.17	1166.32	1166.55	1166.68
LeeCreek	LeeCreek	13694	1,161.40	1162.42	1162.9	1163.31	1163.76	1164.17	1164.47	1164.79	1165.01	1165.78	1165.88	1166.05	1166.27	1166.41
LeeCreek	LeeCreek	13542	1,160.11	1161.48	1162.09	1162.43	1162.7	1162.91	1163.01	1163.25	1163.48	1163.69	1164.92	1165.17	1165.32	1165.49
LeeCreek	LeeCreek	13386	1,159.71	1160.9	1161.53	1161.86	1162.2	1162.42	1162.54	1162.68	1162.78	1163.05	1163.28	1163.42	1163.59	1163.71
LeeCreek	LeeCreek	13241	1,159.00	1160.26	1160.78	1161.19	1161.43	1161.62	1161.76	1161.93	1162.05	1162.37	1162.66	1162.82	1163	1163.13
LeeCreek	LeeCreek	13108	1,158.44	1159.76	1160.45	1160.98	1161.29	1161.55	1161.73	1161.92	1162.06	1162.41	1162.71	1162.87	1163.05	1163.19
LeeCreek	LeeCreek	12912	1,157.27	1158.77	1159.36	1159.87	1160.21	1160.5	1160.7	1160.9	1161.03	1161.29	1161.49	1161.73	1161.91	1162
LeeCreek	LeeCreek	12759	1,156.98	1158.23	1158.82	1159.18	1159.58	1159.89	1160.09	1160.27	1160.36	1160.63	1160.83	1160.97	1161.13	1161.25
LeeCreek	LeeCreek	12591	1,156.07	1157.28	1157.94	1158.3	1158.73	1159.07	1159.29	1159.54	1159.73	1159.99	1160.26	1160.43	1160.6	1160.75
LeeCreek	LeeCreek	12444	1,155.54	1156.58	1157.06	1157.43	1157.77	1158.02	1158.19	1158.37	1158.49	1159.03	1159.27	1159.42	1159.66	1159.84
LeeCreek	LeeCreek	12288	1,154.69	1155.56	1156.27	1156.67	1157.03	1157.35	1157.55	1157.78	1157.94	1158.35	1158.67	1158.86	1159.09	1159.27
LeeCreek	LeeCreek	12145	1,153.21	1154.97	1155.61	1156.02	1156.44	1156.79	1157.03	1157.3	1157.48	1157.89	1158.21	1158.34	1158.45	1158.58
LeeCreek	LeeCreek	11988	1,152.96	1154.41	1155	1155.44	1155.85	1156.16	1156.37	1156.56	1156.68	1157	1157.29	1157.52	1157.8	1158
LeeCreek	LeeCreek	11840	1,152.51	1153.61	1154.26	1154.68	1155.07	1155.38	1155.56	1155.75	1155.88	1156.34	1156.7	1156.93	1157.21	1157.4
LeeCreek	LeeCreek	11645	1,151.22	1152.35	1152.98	1153.43	1153.89	1154.3	1154.6	1154.98	1155.25	1155.87	1156.22	1156.43	1156.68	1156.85
LeeCreek	LeeCreek	11547	1,150.79	1151.85	1152.42	1152.86	1153.31	1153.7	1153.95	1154.23	1154.43	1154.88	1155.32	1155.53	1155.76	1155.92
LeeCreek	LeeCreek	11399	1,150.06	1150.98	1151.67	1152.11	1152.53	1152.92	1153.16	1153.43	1153.58	1153.9	1154.53	1154.68	1154.83	1154.94
LeeCreek	LeeCreek	11249	1,148.96	1150.21	1150.84	1151.23	1151.65	1151.94	1152.13	1152.34	1152.51	1152.94	1153.41	1153.62	1153.89	1153.97
LeeCreek	LeeCreek	11119	1,148.79	1149.72	1150.33	1150.83	1151.39	1151.81	1152.07	1152.38	1152.59	1153.08	1153.53	1153.69	1153.91	1154.02
LeeCreek	LeeCreek	10948	1,147.38	1148.76	1149.49	1149.86	1150.3	1150.69	1150.95	1151.27	1151.5	1152.09	1152.57	1152.78	1153.15	1153.44
LeeCreek	LeeCreek	10800	1,146.75	1147.85	1148.64	1149.12	1149.65	1150.11	1150.4	1150.74	1150.99	1151.59	1152.02	1152.07	1152.22	1152.34
LeeCreek	LeeCreek	10650	1,145.94	1147.11	1147.82	1148.38	1148.94	1149.42	1149.75	1150.17	1150.49	1151.23	1151.93	1151.97	1152.2	1152.38
LeeCreek	LeeCreek	10487	1,145.41	1146.51	1147.16	1147.65	1148.15	1148.46	1148.61	1148.72	1148.84	1149.44	1149.87	1151.12	1151.32	1151.46
LeeCreek	LeeCreek	10343	1,145.06	1145.92	1146.49	1146.91	1147.25	1147.53	1147.76	1148.06	1148.37	1148.44	1148.74	1148.94	1149.21	1149.28
LeeCreek	LeeCreek	10166	1,144.07	1145.22	1145.94	1146.45	1146.95	1147.38	1147.67	1147.98	1148.33	1148.35	1148.55	1148.67	1148.8	1148.86
LeeCreek	LeeCreek	10009	1,143.16	1144.31	1145.06	1145.51	1145.93	1146.26	1146.48	1146.76	1146.96	1147.66	1147.86	1147.98	1148.12	1148.26
LeeCreek	LeeCreek	9919	1,142.55	1143.91	1144.59	1145.07	1145.45	1145.8	1145.97	1146.01	1146.01	1147.09	1147.35	1147.49	1147.62	1147.75
LeeCreek	LeeCreek	9902	1,142.40	1143.79	1144.26	1144.68	1144.99	1145.25	1145.42	1145.62	1145.75	1146.6	1146.88	1147.01	1147.18	1147.31
LeeCreek	LeeCreek	9823	1,142.53	1143.4	1143.78	1144.09	1144.43	1144.7	1144.86	1145.04	1145.19	1145.47	1145.81	1146.01	1146.27	1146.39
LeeCreek	LeeCreek	9722	1,141.61	1142.67	1143.22	1143.64	1144.04	1144.33	1144.53	1144.75	1144.92	1145.17	1145.38	1145.52	1145.67	1145.79
LeeCreek	LeeCreek	9632	1,141.12	1142.09	1142.74	1143.25	1143.74	1144.09	1144.32	1144.6	1144.8	1145.05	1145.28	1145.43	1145.61	1145.75
LeeCreek	LeeCreek	9461	1,140.18	1141.21	1141.8	1142.22	1142.64	1142.98	1143.22	1143.47	1143.63	1144.27	1144.5	1144.63	1144.77	1144.87
LeeCreek	LeeCreek	9312	1,138.93	1140.42	1140.87	1141.24	1141.62	1141.97	1142.22	1142.53	1142.77	1143.36	1143.87	1144.1	1144.26	1144.44
LeeCreek	LeeCreek	9231	1,138.88	1139.75	1140.38	1140.84	1141.34	1141.77	1142.07	1142.39	1142.62	1143.22	1143.61	1143.71	1144.05	1144.31
LeeCreek	LeeCreek	9117	1,137.71	1139.35	1140.03	1140.54	1141.04	1141.48	1141.78	1142.14	1142.4	1143.05	1143.55	1143.64	1143.98	1144.22
LeeCreek	LeeCreek	9005	1,137.78	1139.02	1139.57	1140.03	1140.55	1141.02	1141.32	1141.62	1141.79	1142.16	1142.54	1143.12	1143.65	1143.95
LeeCreek	LeeCreek	8874	1,137.25	1138.33	1138.92	1139.49	1140.08	1140.59	1140.92	1141.21	1141.49	1142.2	1142.73	1143.15	1143.64	1143.92
LeeCreek	LeeCreek	8795	1,136.46	1137.77	1138.62	1139.25	1139.87	1140.4	1140.77	1141.13	1141.44	1142.15	1142.69	1143.12	1143.6	1143.89
LeeCreek	LeeCreek	8717	1,136.00	1137.47	1138.39	1139.05	1139.7	1140.24	1140.58	1140.96	1141.29	1142.03	1142.55	1142.98	1143.46	1143.75
LeeCreek	LeeCreek	8649	1,135.91	1137.32	1138.27	1138.93	1139.57	1140.12	1140.46	1140.77	1141.02	1141.58	1141.94	1142.32	1142.71	1142.87
LeeCreek	LeeCreek	8617	1,135.43	1137.2	1138.08	1138.72	1139.36	1139.91	1140.26	1140.54	1140.77	1141.24	1141.29	1141.47	1141.67	1141.8
LeeCreek	LeeCreek	8502	1,135.33	1136.77	1137.48	1137.95	1138.41	1138.76	1139	1139.34	1139.52	1140.05	1140.73	1140.92	1141.18	1141.34

Table D.1: Lee Creek Flood Profiles

River	Reach	Station	Thalweg	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
LeeCreek	LeeCreek	8405	1,134.66	1136.36	1137.05	1137.52	1137.96	1138.33	1138.59	1138.77	1138.92	1139.42	1139.89	1140.14	1140.36	1140.62
LeeCreek	LeeCreek	8297	1,134.51	1135.93	1136.3	1136.64	1137.03	1137.38	1137.62	1138.07	1138.25	1139.05	1139.22	1139.86	1140.07	1140.17
LeeCreek	LeeCreek	8197	1,133.94	1135.37	1135.95	1136.36	1136.61	1136.94	1137.15	1137.32	1137.62	1138.08	1138.5	1138.82	1139.07	1139.26
LeeCreek	LeeCreek	8120	1,133.73	1134.94	1135.39	1135.9	1136.37	1136.87	1137.1	1137.42	1137.84	1138.37	1138.7	1138.9	1139.05	1139.18
LeeCreek	LeeCreek	8034	1,133.80	1134.5	1135.15	1135.61	1136.01	1136.31	1136.62	1137.26	1137.75	1138.3	1138.62	1138.82	1138.95	1139.07
LeeCreek	LeeCreek	8000	1,133.30	1134.41	1135.04	1135.56	1136.01	1136.35	1136.65	1137.17	1137.54	1137.87	1138.13	1138.68	1138.78	1138.91
LeeCreek	LeeCreek	7983	1,132.88	1134.21	1134.89	1135.4	1135.91	1136.21	1136.38	1136.63	1136.84	1137.14	1137.35	1137.47	1138.44	1138.57
LeeCreek	LeeCreek	7893	1,133.08	1133.92	1134.65	1135.21	1135.77	1136.04	1136.16	1136.3	1136.41	1136.96	1137.32	1137.52	1137.73	1137.87
LeeCreek	LeeCreek	7755	1,132.16	1133.35	1134.03	1134.47	1134.87	1135.44	1135.69	1135.92	1136.12	1136.63	1136.99	1137.14	1137.37	1137.53
LeeCreek	LeeCreek	7637	1,132.08	1132.94	1133.47	1133.87	1134.39	1134.89	1135.38	1136.05	1136.34	1136.74	1137.06	1137.21	1137.42	1137.58
LeeCreek	LeeCreek	7512	1,131.10	1132.44	1133.22	1133.8	1134.4	1134.92	1135.31	1135.85	1136.16	1136.54	1136.83	1136.93	1137.11	1137.23
LeeCreek	LeeCreek	7434	1,131.08	1132.27	1133.1	1133.7	1134.3	1134.8	1135.16	1135.73	1135.98	1136.3	1136.3	1136.46	1136.65	1136.78
LeeCreek	LeeCreek	7399	1,130.58	1132.08	1132.94	1133.57	1134.2	1134.7	1135.03	1135.35	1135.58	1136.27	1136.28	1136.37	1136.57	1136.57
LeeCreek	LeeCreek	7326	1,130.68	1131.93	1132.65	1133.2	1133.76	1134.27	1134.6	1134.82	1134.97	1135.29	1135.83	1135.97	1136.1	1136.21
LeeCreek	LeeCreek	7247	1,130.40	1131.59	1132.3	1132.79	1133.27	1133.71	1134.02	1134.46	1134.59	1134.95	1135.18	1135.33	1135.48	1135.6
LeeCreek	LeeCreek	7155	1,129.52	1130.93	1131.55	1132	1132.51	1132.83	1133.06	1133.29	1133.91	1134.4	1134.74	1134.89	1135.05	1135.19
LeeCreek	LeeCreek	7064	1,129.61	1130.63	1131.16	1131.55	1131.94	1132.4	1132.62	1133.19	1133.37	1133.76	1134	1134.27	1134.44	1134.59
LeeCreek	LeeCreek	6957	1,129.02	1130.35	1130.99	1131.46	1131.9	1132.18	1132.4	1132.78	1133.12	1133.44	1133.77	1133.97	1134.13	1134.25
LeeCreek	LeeCreek	6925	1,128.66	1130.22	1130.94	1131.42	1131.83	1132.05	1132.15	1132.38	1132.53	1132.95	1133.2	1133.66	1133.82	1133.94
LeeCreek	LeeCreek	6825	1,128.64	1129.74	1130.51	1130.86	1131.23	1131.57	1131.82	1132.29	1132.5	1133.04	1133.39	1133.57	1133.85	1133.91
LeeCreek	LeeCreek	6717	1,128.19	1129.39	1130.2	1130.64	1131.02	1131.31	1131.52	1131.8	1132.05	1132.95	1133.31	1133.48	1133.78	1133.82
LeeCreek	LeeCreek	6610	1,127.93	1129.1	1129.73	1130.23	1130.67	1131.05	1131.3	1131.65	1131.92	1132.71	1133.14	1133.31	1133.64	1133.64
LeeCreek	LeeCreek	6470	1,127.72	1128.74	1129.45	1130.01	1130.41	1130.74	1130.98	1131.33	1131.6	1132.38	1132.82	1132.95	1133.43	1133.43
LeeCreek	LeeCreek	6441	1,127.08	1128.66	1129.32	1129.83	1130.16	1130.36	1130.45	1130.6	1130.72	1131.1	1131.49	1131.75	1132.06	1133.03
LeeCreek	LeeCreek	6350	1,127.51	1128.43	1129.03	1129.26	1129.65	1129.94	1130.1	1130.3	1130.44	1130.79	1131.11	1131.32	1131.57	1131.75
LeeCreek	LeeCreek	6264	1,126.56	1128.22	1128.82	1129.14	1129.32	1129.51	1129.67	1129.83	1129.93	1130.25	1130.49	1130.65	1130.87	1131.05
LeeCreek	LeeCreek	5995	1,126.01	1126.99	1127.47	1127.9	1128.43	1128.77	1128.91	1129.08	1129.25	1129.7	1130.05	1130.3	1130.55	1130.82
LeeCreek	LeeCreek	5820	1,124.41	1126.1	1126.81	1127.33	1127.89	1128.43	1128.8	1129.09	1129.24	1129.62	1129.94	1130.17	1130.44	1130.65
LeeCreek	LeeCreek	5664	1,124.53	1125.59	1126.17	1126.64	1127.1	1127.48	1127.75	1128.32	1128.69	1129.04	1129.47	1129.75	1130.03	1130.22
LeeCreek	LeeCreek	5472	1,123.56	1125.03	1125.62	1126.06	1126.53	1126.79	1126.97	1127.21	1127.45	1128.5	1128.82	1129.03	1129.3	1129.46
LeeCreek	LeeCreek	5281	1,123.22	1124.42	1124.99	1125.41	1125.74	1126.09	1126.32	1126.6	1126.8	1127.28	1128.08	1128.23	1128.45	1128.62
LeeCreek	LeeCreek	5086	1,122.69	1123.87	1124.55	1124.98	1125.47	1125.82	1126.04	1126.27	1126.43	1126.83	1127.14	1127.34	1127.56	1127.71
LeeCreek	LeeCreek	4890	1,122.17	1123.44	1124.19	1124.55	1124.8	1125.13	1125.32	1125.56	1125.7	1126.04	1126.33	1126.48	1126.79	1126.92
LeeCreek	LeeCreek	4767	1,121.82	1123.19	1123.92	1124.29	1124.62	1124.92	1125.08	1125.25	1125.39	1125.66	1125.9	1126.06	1126.25	1126.4
LeeCreek	LeeCreek	4583	1,120.81	1122.79	1123.4	1123.79	1124.17	1124.45	1124.6	1124.79	1124.92	1125.26	1125.54	1125.73	1125.95	1126.1
LeeCreek	LeeCreek	4433	1,121.08	1122.38	1122.99	1123.28	1123.51	1123.7	1123.85	1124.01	1124.13	1124.42	1124.67	1124.83	1125.05	1125.15
LeeCreek	LeeCreek	4308	1,120.24	1121.73	1122.48	1122.82	1123.17	1123.45	1123.65	1123.84	1123.97	1124.28	1124.55	1124.74	1124.99	1125.1
LeeCreek	LeeCreek	4008	1,118.60	1120.68	1121.35	1121.76	1122.08	1122.32	1122.44	1122.6	1122.72	1123.04	1123.36	1123.48	1123.63	1123.83
LeeCreek	LeeCreek	3872	1,119.18	1120.33	1120.89	1121.33	1121.75	1122.08	1122.28	1122.52	1122.68	1123.09	1123.44	1123.6	1123.8	1123.94
LeeCreek	LeeCreek	3705	1,117.85	1119.69	1120.34	1120.85	1121.25	1121.51	1121.66	1121.85	1121.97	1122.25	1122.51	1122.72	1122.87	1122.97
LeeCreek	LeeCreek	3553	1,117.60	1118.89	1119.62	1120.14	1120.65	1121.03	1121.24	1121.46	1121.6	1121.91	1122.26	1122.44	1122.73	1122.81
LeeCreek	LeeCreek	3385	1,116.90	1118.42	1119.1	1119.51	1119.87	1120.14	1120.33	1120.53	1120.7	1121.18	1121.38	1121.52	1121.65	1121.92
LeeCreek	LeeCreek	3212	1,116.71	1118	1118.71	1119.15	1119.58	1119.91	1120.12	1120.35	1120.51	1120.89	1121.08	1121.21	1121.39	1121.5

Table D.1: Lee Creek Flood Profiles

River	Reach	Station	Thalweg	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
LeeCreek	**LeeCreek**	**3076**	1,116.31	1117.61	1118.34	1118.79	1119.24	1119.61	1119.85	1120.1	1120.28	1120.69	1120.86	1120.99	1121.15	1121.25
LeeCreek	**LeeCreek**	**2890**	1,115.40	1116.99	1117.66	1118.06	1118.46	1118.77	1118.95	1119.15	1119.26	1119.56	1120.04	1120.19	1120.32	1120.48
LeeCreek	**LeeCreek**	**2664**	1,114.67	1116.35	1117.08	1117.57	1118.04	1118.41	1118.65	1118.91	1119.1	1119.48	1119.76	1119.92	1120.1	1120.23
LeeCreek	**LeeCreek**	**2466**	1,114.61	1115.72	1116.36	1116.75	1117.1	1117.4	1117.59	1117.84	1117.94	1118.58	1118.85	1119.06	1119.25	1119.39
LeeCreek	**LeeCreek**	**2306**	1,114.03	1115.21	1115.88	1116.37	1116.81	1117.14	1117.33	1117.5	1117.62	1117.9	1118.13	1118.29	1118.49	1118.64
LeeCreek	**LeeCreek**	**2159**	1,113.59	1114.66	1115.33	1115.82	1116.25	1116.54	1116.72	1116.94	1117.09	1117.51	1117.84	1118.07	1118.32	1118.48
LeeCreek	**LeeCreek**	**2002**	1,112.82	1114.35	1115.02	1115.5	1115.93	1116.22	1116.39	1116.57	1116.7	1117.03	1117.24	1117.39	1117.55	1117.67
LeeCreek	**LeeCreek**	**1863**	1,112.72	1114.06	1114.73	1115.22	1115.68	1115.98	1116.16	1116.36	1116.49	1116.86	1117.06	1117.21	1117.39	1117.5
LeeCreek	**LeeCreek**	**1738**	1,112.17	1113.58	1114.29	1114.82	1115.31	1115.73	1115.97	1116.21	1116.35	1116.78	1116.96	1117.13	1117.3	1117.42
LeeCreek	**LeeCreek**	**1526**	1,111.49	1112.89	1113.56	1114.05	1114.56	1114.74	1114.98	1115.07	1115.21	1115.52	1115.94	1116.12	1116.31	1116.49
LeeCreek	**LeeCreek**	**1392**	1,111.22	1112.52	1112.95	1113.17	1113.43	1113.86	1113.95	1114.28	1114.39	1114.7	1114.95	1115.2	1115.8	1116.12
LeeCreek	**LeeCreek**	**1259**	1,110.38	1111.55	1112.22	1112.71	1113.13	1113.41	1113.58	1113.76	1113.89	1114.27	1114.87	1115.31	1115.85	1116.14
LeeCreek	**LeeCreek**	**1097**	1,110.14	1111.39	1112.07	1112.56	1112.96	1113.21	1113.36	1113.51	1113.59	1113.93	1114.63	1115.1	1115.67	1115.95
LeeCreek	**LeeCreek**	**931**	1,109.34	1111.15	1111.76	1112.2	1112.62	1112.92	1113.11	1113.27	1113.34	1113.76	1114.59	1115.1	1115.68	1115.98
LeeCreek	**LeeCreek**	**724**	1,109.10	1110.58	1111.06	1111.48	1111.86	1112.12	1112.29	1112.54	1112.78	1113.57	1114.53	1115.05	1115.65	1115.94
LeeCreek	**LeeCreek**	**605**	1,108.92	1110.02	1110.76	1111.26	1111.7	1112.02	1112.24	1112.52	1112.74	1113.54	1114.51	1115.04	1115.64	1115.93
LeeCreek	**LeeCreek**	**447**	1,108.00	1109.57	1110.44	1111	1111.46	1111.8	1112.03	1112.35	1112.61	1113.46	1114.48	1115.02	1115.62	1115.92
LeeCreek	**LeeCreek**	**330**	1,107.83	1109.32	1110.1	1110.66	1111.14	1111.56	1111.85	1112.22	1112.5	1113.41	1114.45	1115	1115.61	1115.9
LeeCreek	**LeeCreek**	**205**	1,107.09	1109.13	1109.97	1110.56	1111.06	1111.47	1111.77	1112.15	1112.44	1113.38	1114.44	1114.98	1115.59	1115.89
LeeCreek	**LeeCreek**	**60**	1,106.56	1109.03	1109.8	1110.35	1110.86	1111.32	1111.65	1112.05	1112.36	1113.32	1114.4	1114.95	1115.57	1115.87

APPENDIX E

Model Sensitivity Analysis

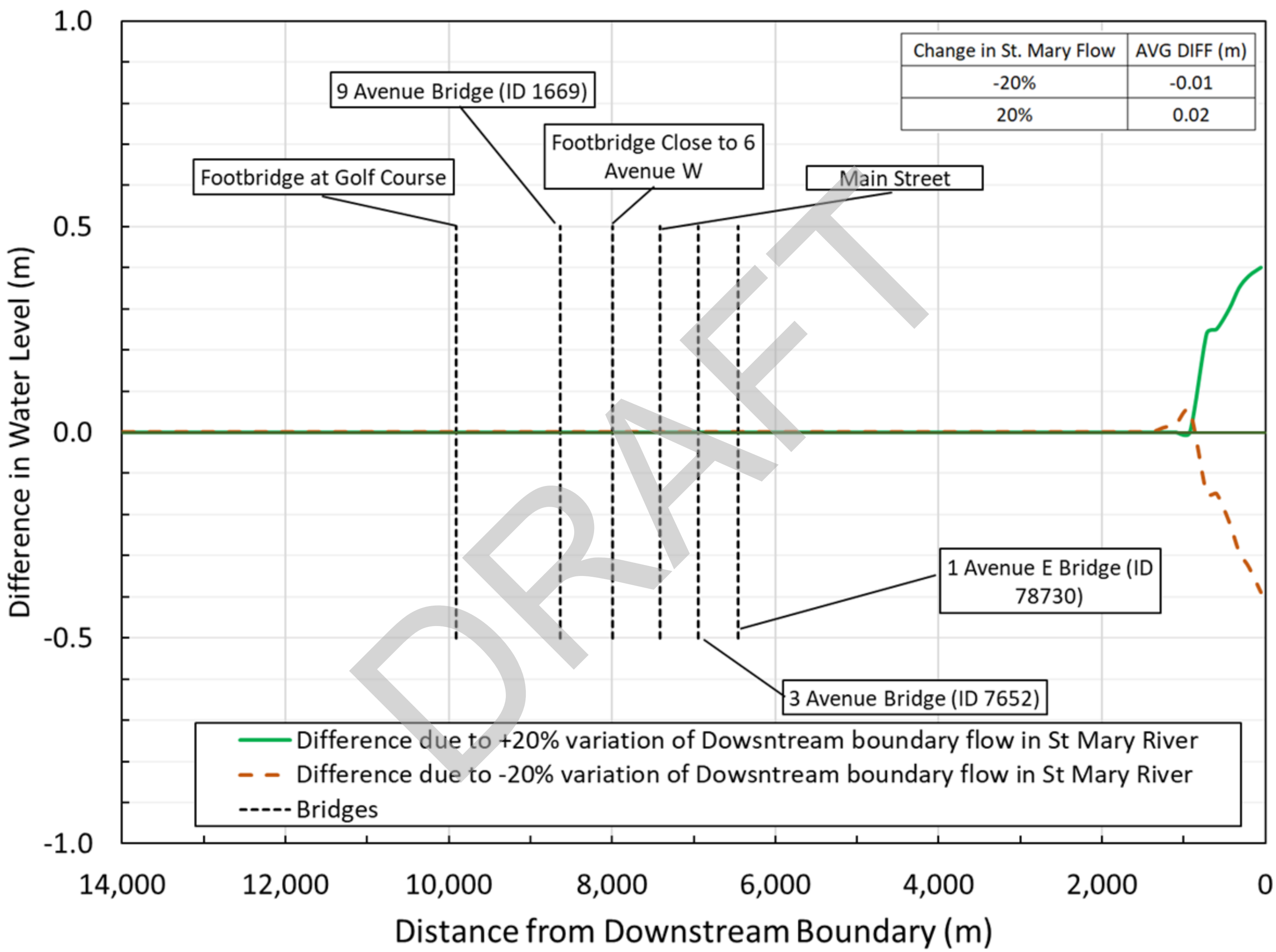


Figure E-1: Downstream Boundary (St. Mary Flow) Sensitivity Analysis for Lee Creek

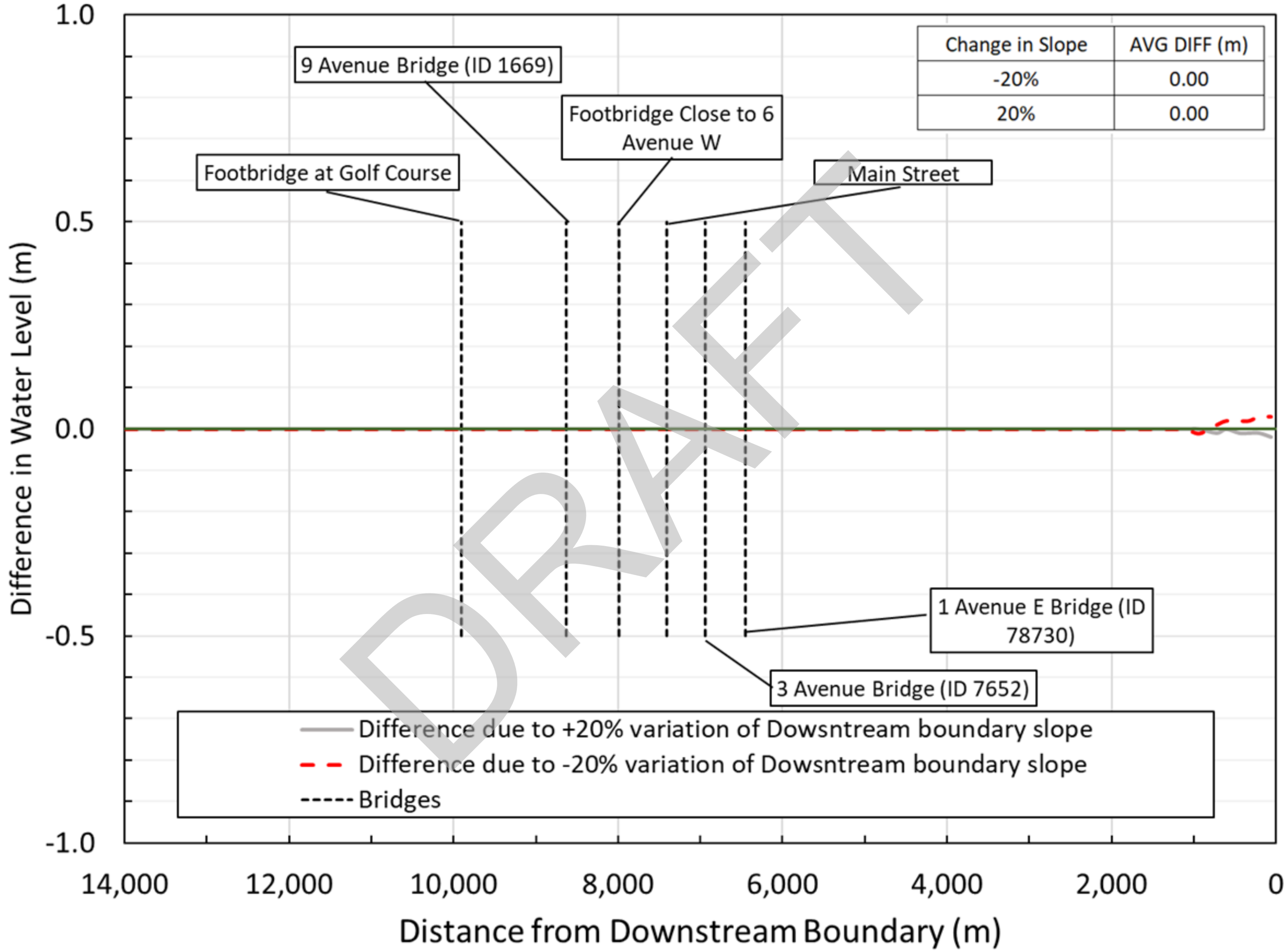


Figure E-2: Downstream Boundary (Energy Slope at St. Mary River) Sensitivity Analysis for Lee Creek

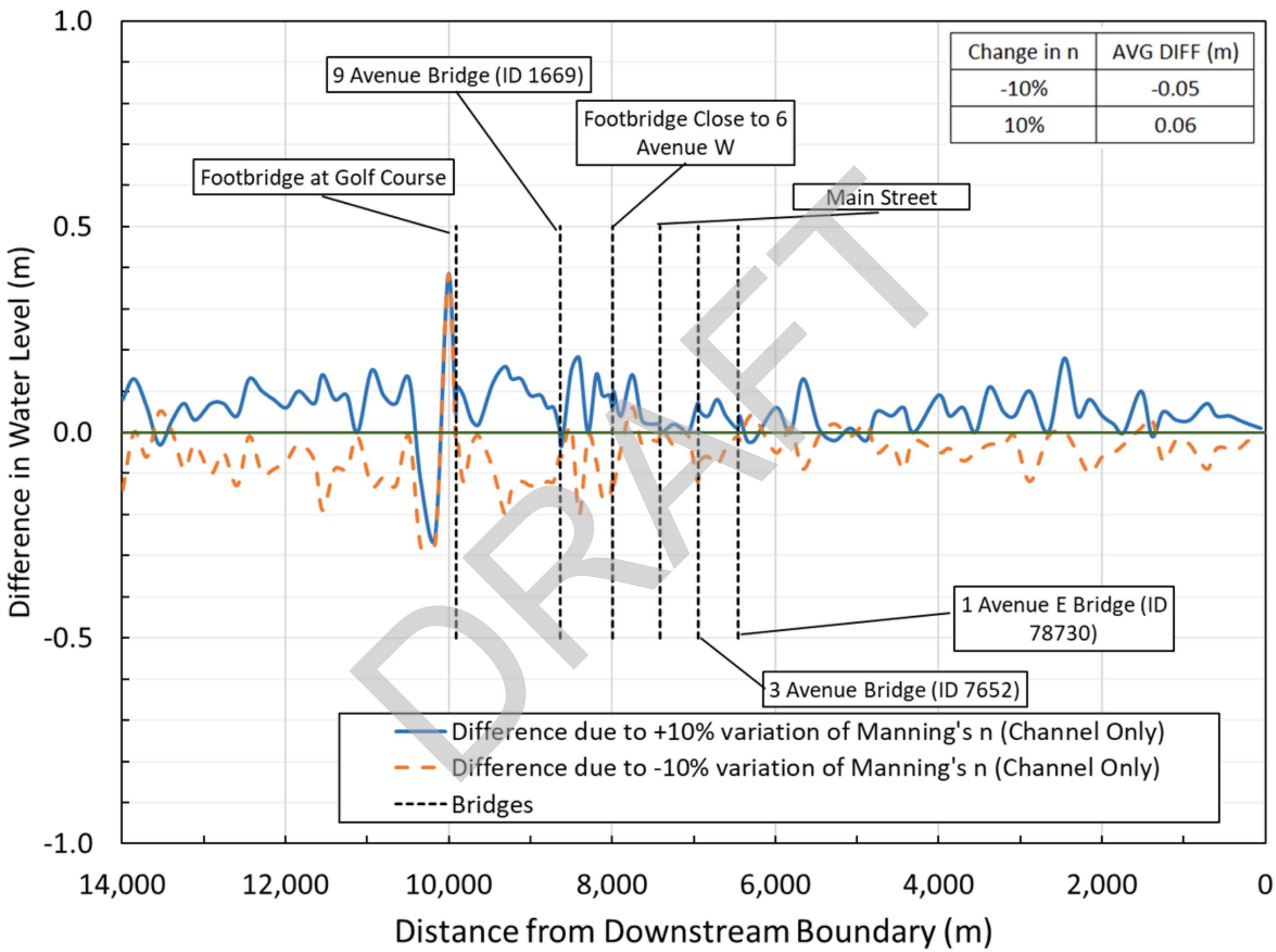


Figure E-3: Channel Roughness Sensitivity Analysis for Lee Creek

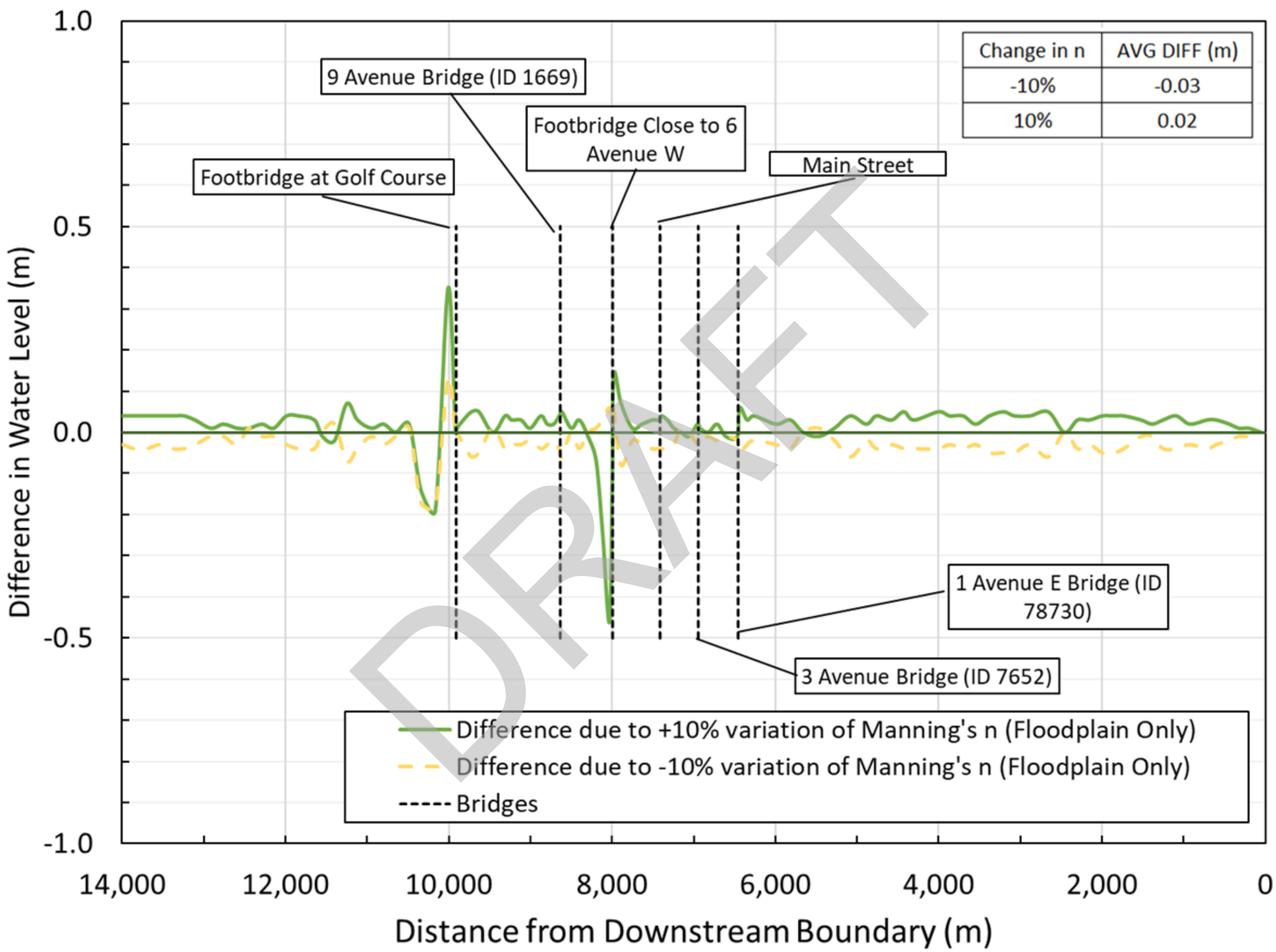


Figure E-4: Floodplain Roughness Sensitivity Analysis for Lee Creek

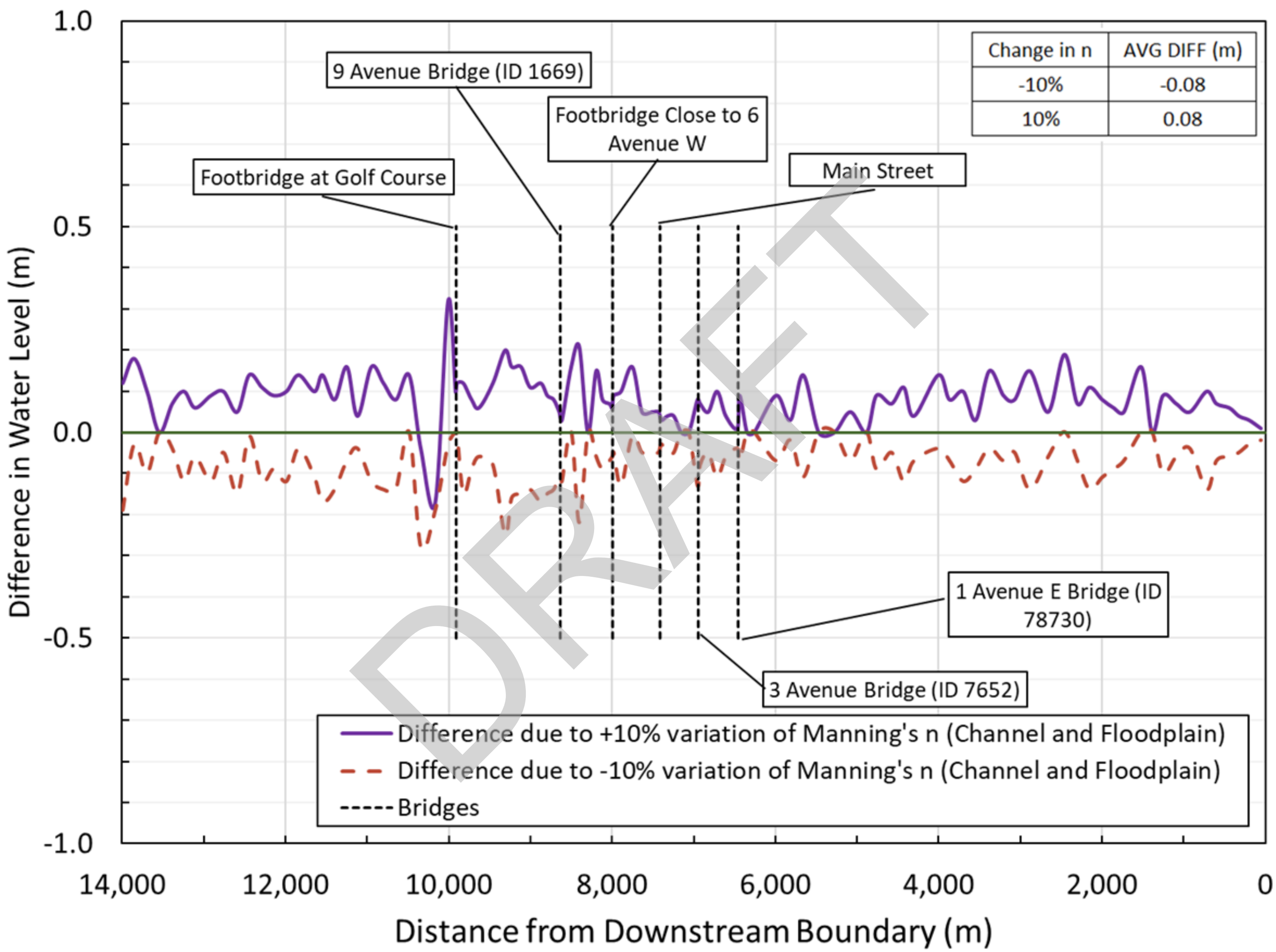


Figure E-5: Combined Channel and Floodplain Roughness Sensitivity Analysis for Lee Creek

APPENDIX F

Open Water Flood Inundation Map Library

(Provided under Separate Cover)

APPENDIX G

Floodway Criteria and Design Flood Water Levels

Table G-1: Floodway Criteria and Design Flood Water Levels

River	Reach	River Station	Left		Right		100-Year Design Flood Level (m)
			Floodway Limit (m)	Governing Criteria	Floodway Limit (m)	Governing Criteria	
LeeCreek	LeeCreek	13992	54.3	Inundation limit (2)	83.2	Inundation limit (2)	1166.16
LeeCreek	LeeCreek	13852	167.0	1 m/s velocity	235.6	1 m/s velocity	1165.32
LeeCreek	LeeCreek	13694	179.6	1 m depth	255.2	Inundation limit (2)	1165.01
LeeCreek	LeeCreek	13542	154.9	Inundation limit (2)	200.9	Inundation limit (2)	1163.48
LeeCreek	LeeCreek	13386	133.2	Mixed	213.9	Inundation limit (2)	1162.78
LeeCreek	LeeCreek	13241	58.4	Inundation limit (2)	219.7	Inundation limit (2)	1162.05
LeeCreek	LeeCreek	13108	39.6	1 m depth	318.7	1 m depth	1162.06
LeeCreek	LeeCreek	12912	25.5	1 m depth	179.9	Mixed	1161.03
LeeCreek	LeeCreek	12759	22.3	1 m depth	232.4	1 m depth	1160.36
LeeCreek	LeeCreek	12591	90.2	1 m depth	160.7	1 m/s velocity	1159.73
LeeCreek	LeeCreek	12444	140.4	1 m/s velocity	194.2	Inundation limit (2)	1158.49
LeeCreek	LeeCreek	12288	259.1	Mixed	325.6	1 m/s velocity	1157.94
LeeCreek	LeeCreek	12145	353.9	Main Channel	438.4	1 m depth	1157.48
LeeCreek	LeeCreek	11988	373.7	1 m/s velocity	425.2	Mixed	1156.68
LeeCreek	LeeCreek	11840	212.3	Mixed	303.4	1 m/s velocity	1155.88
LeeCreek	LeeCreek	11645	203.6	1 m depth	333.3	Inundation limit (2)	1155.25
LeeCreek	LeeCreek	11547	270.0	1 m/s velocity	315.2	1 m depth	1154.43
LeeCreek	LeeCreek	11399	330.1	1 m/s velocity	375.2	Inundation limit (2)	1153.58
LeeCreek	LeeCreek	11249	358.1	1 m/s velocity	411.6	1 m depth	1152.51
LeeCreek	LeeCreek	11119	273.2	1 m depth	456.1	Inundation limit (2)	1152.59
LeeCreek	LeeCreek	10948	298.9	1 m depth	372.9	Inundation limit (2)	1151.5
LeeCreek	LeeCreek	10800	220.7	1 m/s velocity	270.6	1 m/s velocity	1150.99
LeeCreek	LeeCreek	10650	117.4	1 m depth	183.3	1 m depth	1150.49
LeeCreek	LeeCreek	10487	63.8	Inundation limit (2)	97.9	1 m/s velocity	1148.84
LeeCreek	LeeCreek	10343	107.7	Inundation limit (2)	176.3	Mixed	1148.37
LeeCreek	LeeCreek	10166	263.3	Main Channel	358.5	Previous floodway	1148.33
LeeCreek	LeeCreek	10009	383.0	Previous floodway	462.9	Main Channel	1146.96
LeeCreek	LeeCreek	9919	272.6	Inundation limit (1)	334.7	Previous floodway	1146.01
LeeCreek	LeeCreek	9902	267.1	Inundation limit (1)	318.8	Inundation limit (1)	1145.75
LeeCreek	LeeCreek	9823	205.4	Main Channel	281.6	Previous floodway	1145.19
LeeCreek	LeeCreek	9722	260.7	Main Channel	375.0	Inundation limit (1)	1144.92
LeeCreek	LeeCreek	9632	394.1	Previous floodway	466.0	Previous floodway	1144.8
LeeCreek	LeeCreek	9461	452.7	Inundation limit (1)	511.2	Main Channel	1143.63
LeeCreek	LeeCreek	9312	402.2	Mixed	461.4	Main Channel	1142.77
LeeCreek	LeeCreek	9231	356.9	Previous floodway	413.0	Previous floodway	1142.62
LeeCreek	LeeCreek	9117	309.9	Main Channel	408.4	Previous floodway	1142.4

Table G-1: Floodway Criteria and Design Flood Water Levels

River	Reach	River Station	Left		Right		100-Year Design Flood Level (m)
			Floodway Limit (m)	Governing Criteria	Floodway Limit (m)	Governing Criteria	
LeeCreek	LeeCreek	9005	261.0	Previous floodway	322.2	Inundation limit (1)	1141.79
LeeCreek	LeeCreek	8874	184.4	Previous floodway	252.8	Inundation limit (1)	1141.49
LeeCreek	LeeCreek	8795	170.6	Previous floodway	231.5	Previous floodway	1141.44
LeeCreek	LeeCreek	8717	168.8	Previous floodway	218.2	Previous floodway	1141.29
LeeCreek	LeeCreek	8649	155.1	Previous floodway	201.9	Inundation limit (1)	1141.02
LeeCreek	LeeCreek	8617	173.1	Previous floodway	213.0	Inundation limit (1)	1140.77
LeeCreek	LeeCreek	8502	75.6	Previous floodway	121.5	Previous floodway	1139.52
LeeCreek	LeeCreek	8405	102.1	Inundation limit (1)	139.6	Previous floodway	1138.92
LeeCreek	LeeCreek	8297	104.5	Previous floodway	149.2	Previous floodway	1138.25
LeeCreek	LeeCreek	8197	110.2	Inundation limit (1)	180.8	Main Channel	1137.62
LeeCreek	LeeCreek	8120	82.8	Previous floodway	198.8	Previous floodway	1137.84
LeeCreek	LeeCreek	8034	69.6	Previous floodway	188.9	Previous floodway	1137.75
LeeCreek	LeeCreek	8000	83.3	Previous floodway	189.3	Previous floodway	1137.54
LeeCreek	LeeCreek	7983	84.6	Previous floodway	181.0	Previous floodway	1136.84
LeeCreek	LeeCreek	7893	155.3	Inundation limit (1)	205.8	Inundation limit (1)	1136.41
LeeCreek	LeeCreek	7755	199.0	Previous floodway	254.1	Previous floodway	1136.12
LeeCreek	LeeCreek	7637	264.4	Previous floodway	309.4	Previous floodway	1136.34
LeeCreek	LeeCreek	7512	291.3	Previous floodway	337.8	Main Channel	1136.16
LeeCreek	LeeCreek	7434	323.8	Previous floodway	386.3	Previous floodway	1135.98
LeeCreek	LeeCreek	7399	330.9	Previous floodway	394.8	Previous floodway	1135.58
LeeCreek	LeeCreek	7326	318.5	Previous floodway	379.7	Previous floodway	1134.97
LeeCreek	LeeCreek	7247	348.1	Previous floodway	398.3	Previous floodway	1134.59
LeeCreek	LeeCreek	7155	350.2	Previous floodway	398.9	Main Channel	1133.91
LeeCreek	LeeCreek	7064	350.6	Previous floodway	397.2	Previous floodway	1133.37
LeeCreek	LeeCreek	6957	391.8	Previous floodway	440.4	Previous floodway	1133.12
LeeCreek	LeeCreek	6925	385.6	Previous floodway	433.6	Previous floodway	1132.53
LeeCreek	LeeCreek	6825	303.4	Previous floodway	348.4	Previous floodway	1132.5
LeeCreek	LeeCreek	6717	312.2	Previous floodway	354.9	Previous floodway	1132.05
LeeCreek	LeeCreek	6610	372.7	Previous floodway	454.8	Previous floodway	1131.92
LeeCreek	LeeCreek	6470	452.0	Main Channel	503.7	Inundation limit (1)	1131.6
LeeCreek	LeeCreek	6441	445.2	Main Channel	491.0	Inundation limit (1)	1130.72
LeeCreek	LeeCreek	6350	443.3	Main Channel	N/A (3)	Previous floodway	1130.44
LeeCreek	LeeCreek	6264	331.1	Previous floodway	N/A (3)	Previous floodway	1129.93
LeeCreek	LeeCreek	5995	221.8	Previous floodway	N/A (3)	Previous floodway	1129.25
LeeCreek	LeeCreek	5820	151.8	Main Channel	306.4	Previous floodway	1129.24
LeeCreek	LeeCreek	5664	103.5	Main Channel	164.6	Previous floodway	1128.69
LeeCreek	LeeCreek	5472	172.2	Main Channel	226.1	Previous floodway	1127.45

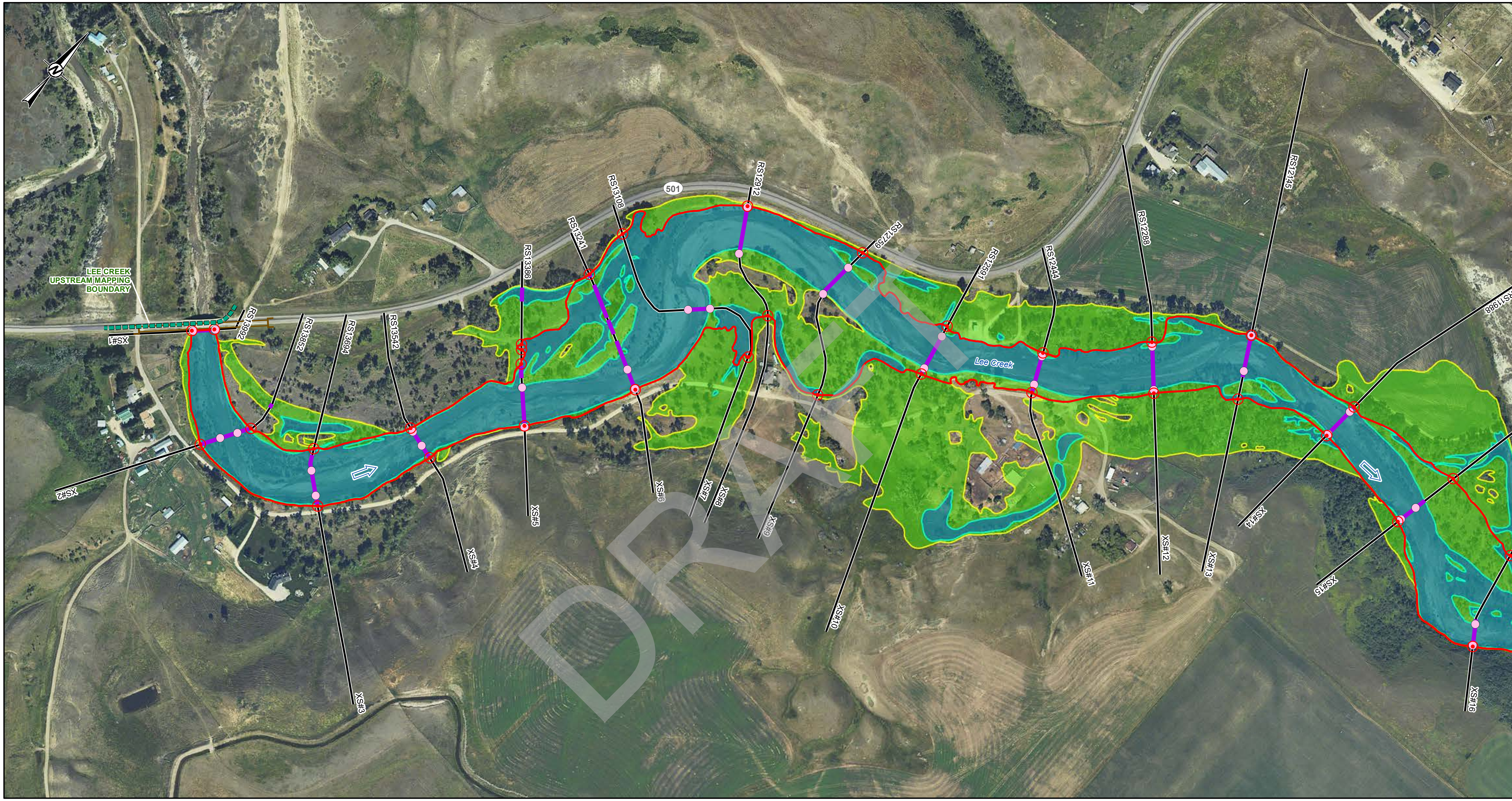
Table G-1: Floodway Criteria and Design Flood Water Levels

River	Reach	River Station	Left		Right		100-Year Design Flood Level (m)
			Floodway Limit (m)	Governing Criteria	Floodway Limit (m)	Governing Criteria	
LeeCreek	LeeCreek	5281	152.6	Previous floodway	231.6	Previous floodway	1126.8
LeeCreek	LeeCreek	5086	216.1	Previous floodway	375.9	Previous floodway	1126.43
LeeCreek	LeeCreek	4890	239.2	Previous floodway	399.6	Main Channel	1125.7
LeeCreek	LeeCreek	4767	288.9	Previous floodway	387.4	Previous floodway	1125.39
LeeCreek	LeeCreek	4583	249.9	Main Channel	366.4	Previous floodway	1124.92
LeeCreek	LeeCreek	4433	256.3	Inundation limit (1)	348.7	Previous floodway	1124.13
LeeCreek	LeeCreek	4308	300.3	1 m/s velocity	395.6	1 m depth	1123.97
LeeCreek	LeeCreek	4008	312.6	1 m depth	405.5	Inundation limit (2)	1122.72
LeeCreek	LeeCreek	3872	124.4	1 m depth	318.9	1 m depth	1122.68
LeeCreek	LeeCreek	3705	114.0	Inundation limit (2)	192.4	1 m depth	1121.97
LeeCreek	LeeCreek	3553	64.9	Inundation limit (2)	189.6	1 m depth	1121.6
LeeCreek	LeeCreek	3385	43.8	Inundation limit (2)	173.2	Mixed	1120.7
LeeCreek	LeeCreek	3212	181.4	1 m depth	288.0	1 m/s velocity	1120.51
LeeCreek	LeeCreek	3076	214.2	1 m/s velocity	373.2	Inundation limit (2)	1120.28
LeeCreek	LeeCreek	2890	254.7	1 m depth	323.4	Inundation limit (2)	1119.26
LeeCreek	LeeCreek	2664	59.4	1 m depth	216.3	1 m depth	1119.1
LeeCreek	LeeCreek	2466	52.6	1 m/s velocity	112.3	1 m depth	1117.94
LeeCreek	LeeCreek	2306	74.2	1 m depth	170.7	Mixed	1117.62
LeeCreek	LeeCreek	2159	167.6	1 m depth	233.1	1 m depth	1117.09
LeeCreek	LeeCreek	2002	184.0	Mixed	328.8	Inundation limit (2)	1116.7
LeeCreek	LeeCreek	1863	147.6	1 m depth	329.3	Main Channel	1116.49
LeeCreek	LeeCreek	1738	143.8	1 m depth	319.1	Main Channel	1116.35
LeeCreek	LeeCreek	1526	78.3	Main Channel	145.7	1 m depth	1115.21
LeeCreek	LeeCreek	1392	72.4	1 m depth	181.8	1 m depth	1114.39
LeeCreek	LeeCreek	1259	95.2	1 m/s velocity	208.3	1 m/s velocity	1113.89
LeeCreek	LeeCreek	1097	90.8	Inundation limit (2)	273.1	Inundation limit (2)	1113.59
LeeCreek	LeeCreek	931	177.5	Mixed	394.2	Inundation limit (2)	1113.34
LeeCreek	LeeCreek	724	210.5	Mixed	380.8	Inundation limit (2)	1112.78
LeeCreek	LeeCreek	605	170.8	1 m depth	369.3	Inundation limit (2)	1112.74
LeeCreek	LeeCreek	447	208.7	1 m depth	379.9	1 m depth	1112.61
LeeCreek	LeeCreek	330	266.7	1 m depth	491.8	1 m depth	1112.5
LeeCreek	LeeCreek	205	151.3	Inundation limit (2)	520.2	1 m depth	1112.44
LeeCreek	LeeCreek	60	426.2	Inundation limit (2)	712.6	1 m depth	1112.36

Notes:
1) cross sections where the previous floodway is outside the inundation limit
2) No Viable flood fringe
3) Previous floodway was used and the cross-sections didn't cross the right bank

APPENDIX H

Floodway Criteria Maps



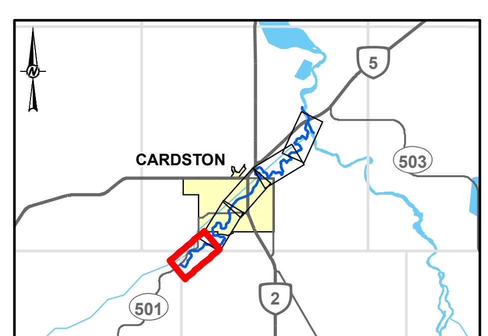
LEGEND

	CROSS SECTION
XS#100	CROSS SECTION NUMBER
RS 304	RIVER STATION (M)
	STUDY BOUNDARY
	FLOW DIRECTION
	LOCAL ROAD
	PRIMARY HIGHWAY
	SECONDARY HIGHWAY
	FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



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Alberta

CONSULTANT
GOLDER
MEMBER OF WSP

YYYY-MM-DD	2022-02-15
DESIGNED	PH
PREPARED	SK/AL
REVIEWED	WP
APPROVED	WP

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 1 OF 5

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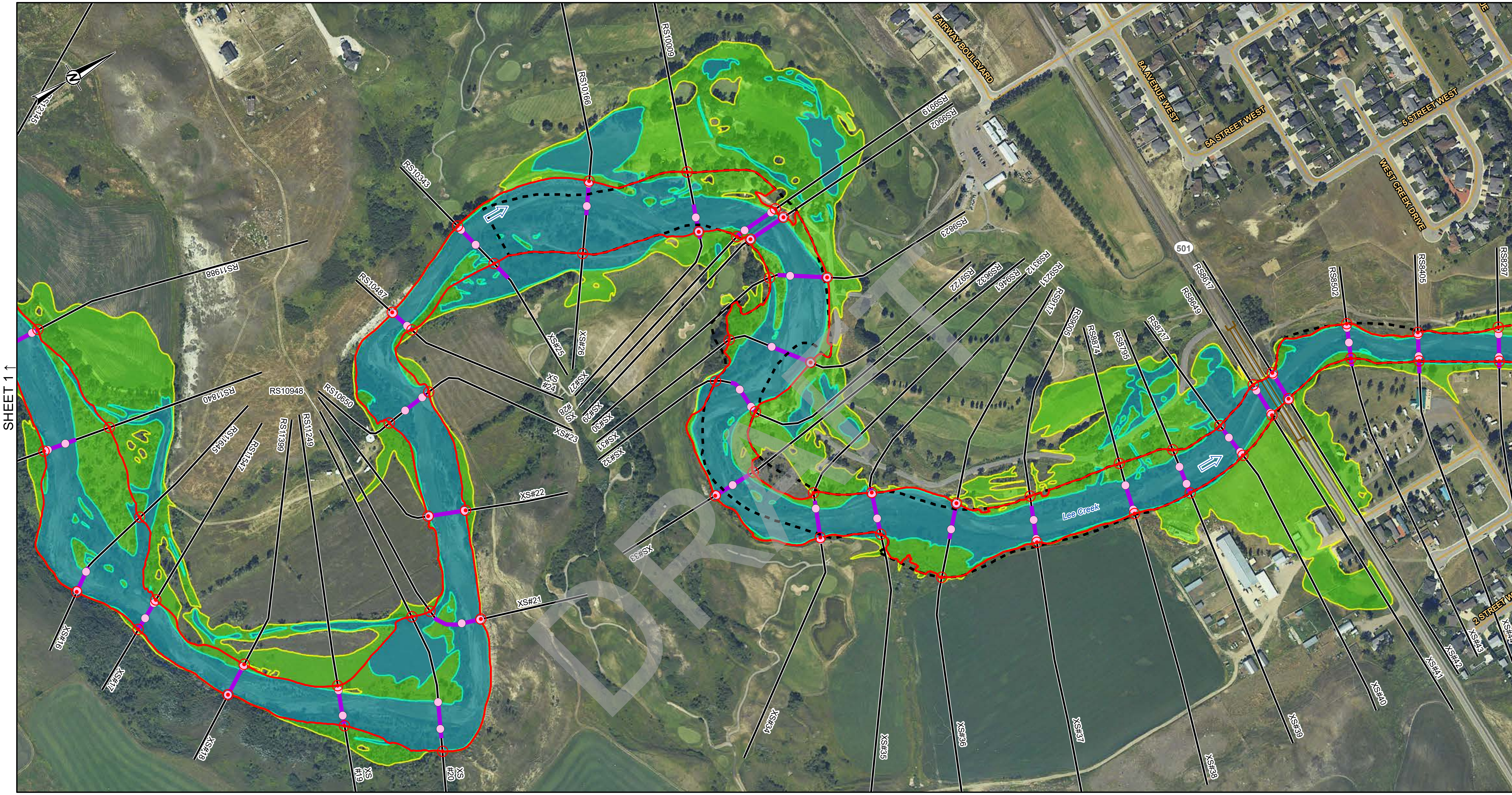
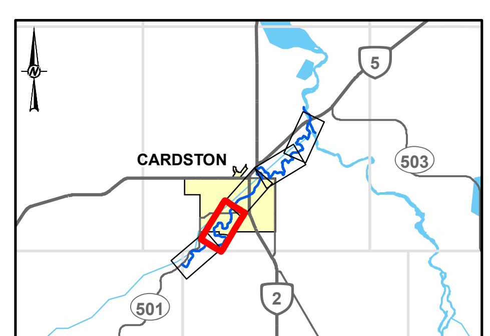


LEGEND

CROSS SECTION	FLOOD CONTROL STRUCTURE
XS#100 CROSS SECTION NUMBER	**HYDRAULIC STRUCTURES**
RS 304 RIVER STATION (M)	BRIDGE
STUDY BOUNDARY	
FLOW DIRECTION	
LOCAL ROAD	
PRIMARY HIGHWAY	
SECONDARY HIGHWAY	

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M³/S



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PREPARED	SK/AL
REVIEWED	WP
APPROVED	WP

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 2 OF 5

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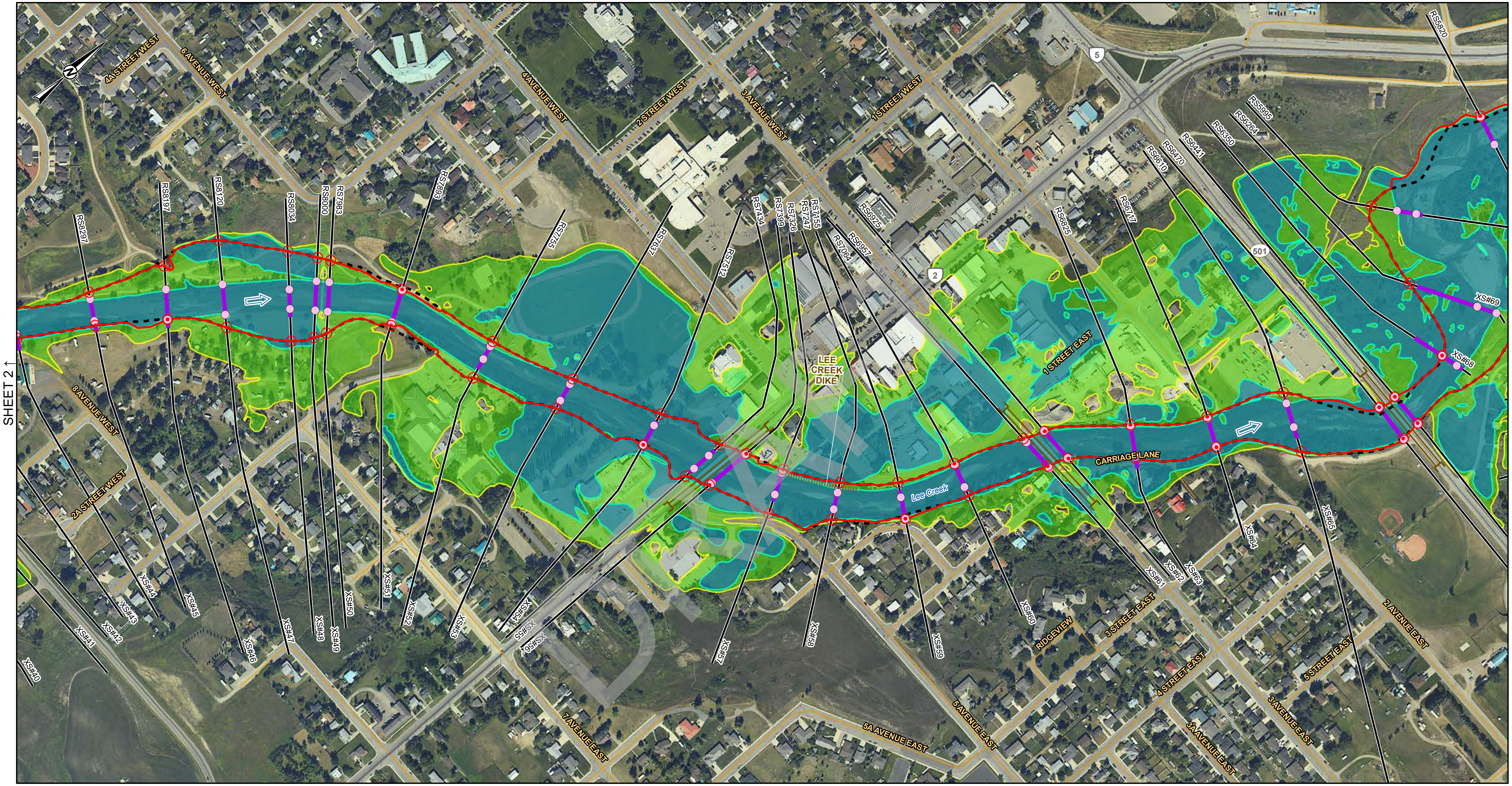
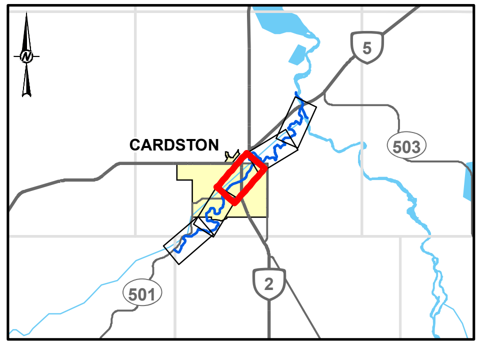


SHEET 4 →

LEGEND

- CROSS SECTION
- XS#100 CROSS SECTION NUMBER
- RS 304 RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE
- **HYDRAULIC STRUCTURES**
- BRIDGE
- PROPOSED FLOODWAY BOUNDARY
- BANK STATION
- PROPOSED FLOODWAY STATION
- PREVIOUS FLOODWAY
- DEPTH ≥ 1 M
- 100-YEAR DESIGN FLOOD EXTENT
- VELOCITY ≥ 1 M/S

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M³/S



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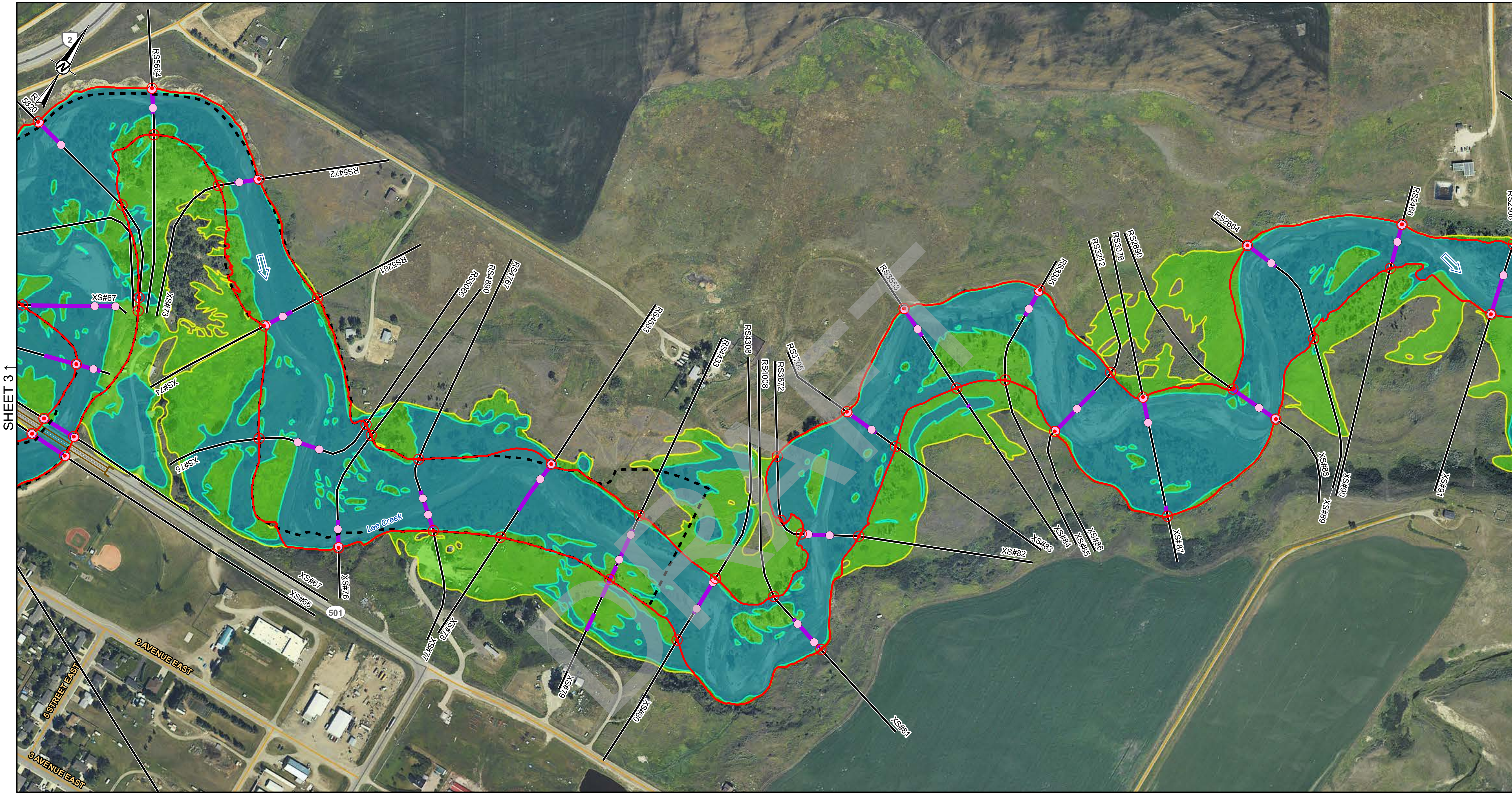
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 3 OF 5



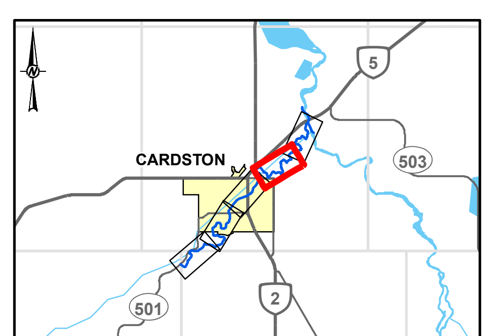
LEGEND

- CROSS SECTION
- XS#100 CROSS SECTION NUMBER
- RS 304 RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



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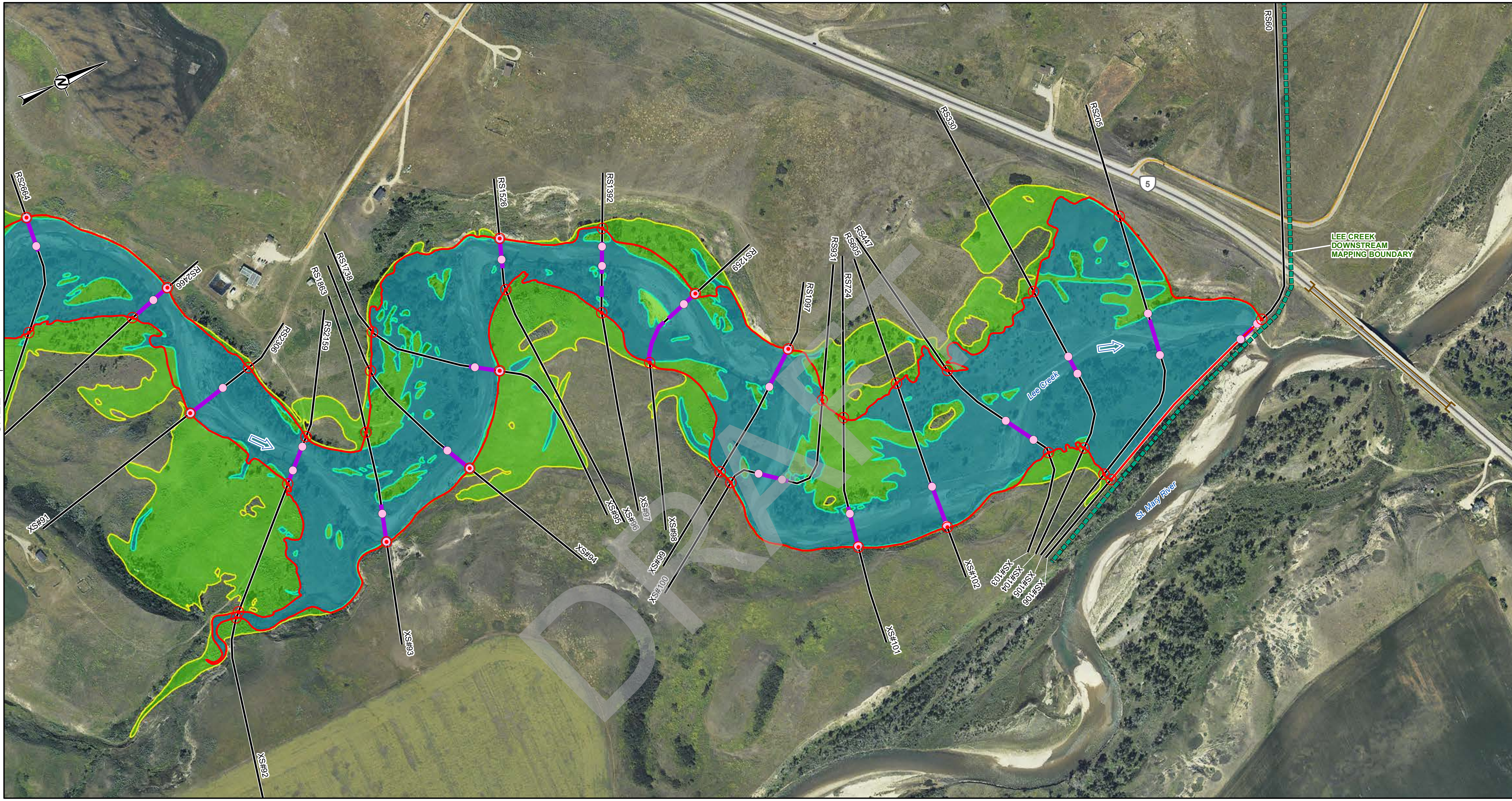
APPROVED	WP

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 4 OF 5

Classification: Public



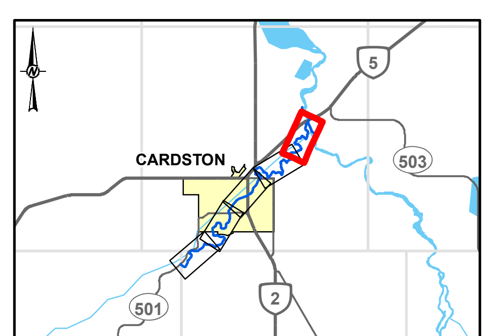
LEGEND

- CROSS SECTION
- *XS#100* CROSS SECTION NUMBER
- *RS 304* RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



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PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOODWAY CRITERIA MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 5 OF 5

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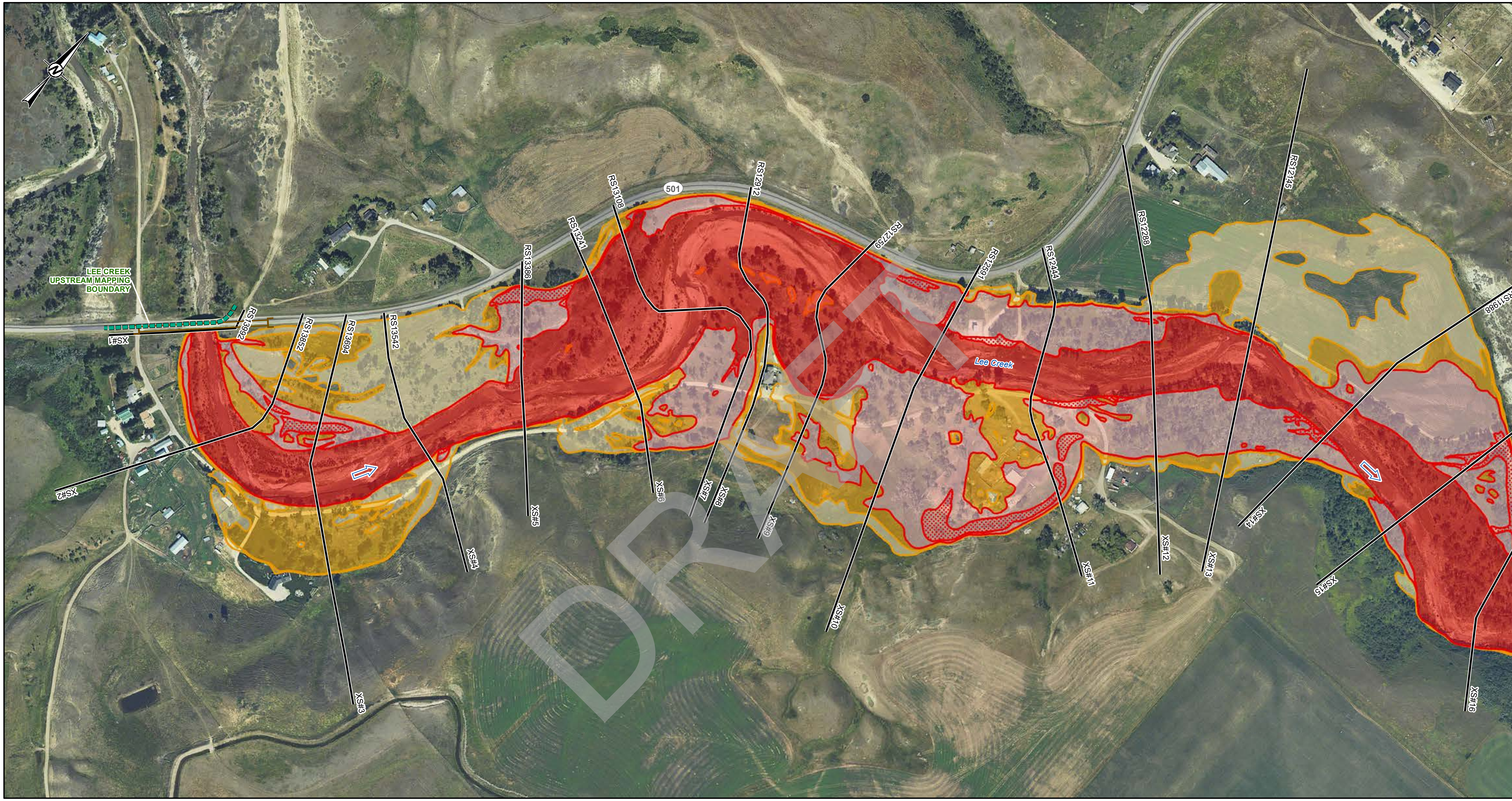
APPENDIX I

Flood Hazard Maps



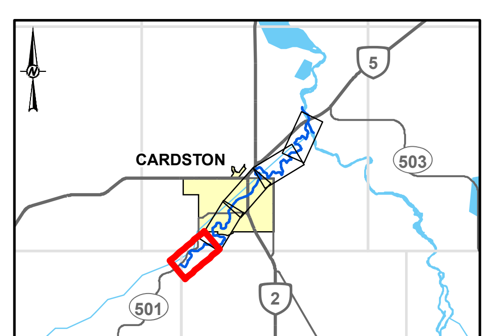
LEGEND

- CROSS SECTION
- XS#100 CROSS SECTION NUMBER
- RS 304 RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M³/S



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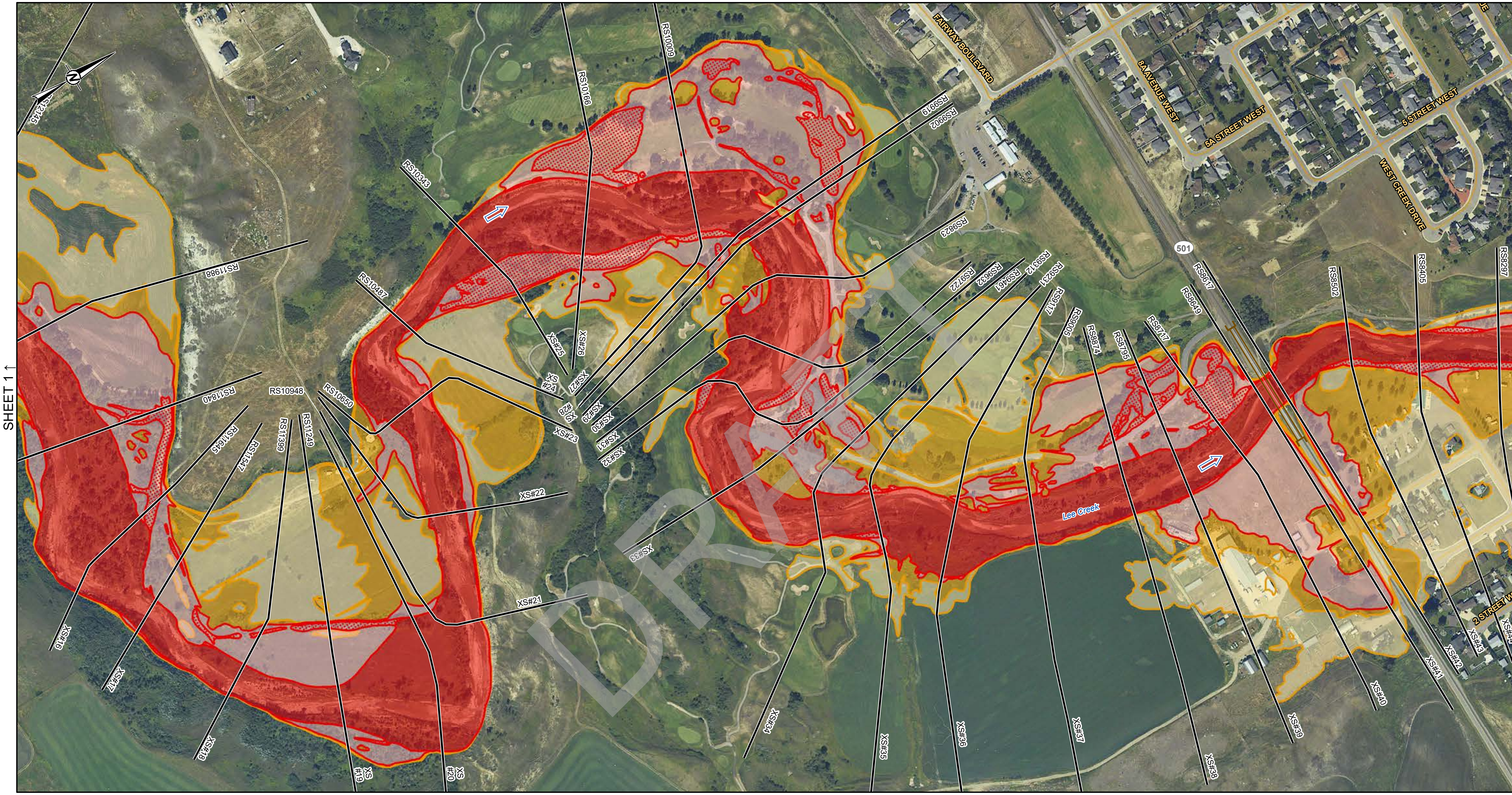
APPROVED	WP

PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 1 OF 5



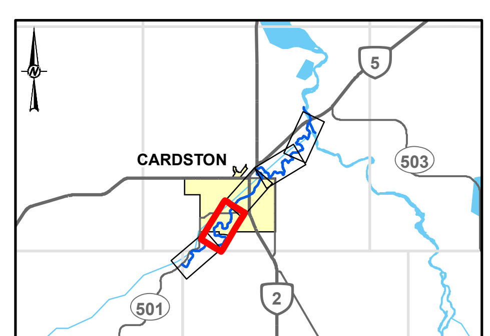
LEGEND

- CROSS SECTION
- *XS#100* CROSS SECTION NUMBER
- *RS 304* RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



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1:5,000 METRES

REFERENCE(S)

 IMAGERY CAPTURED JULY 2019 BY OGL ENGINEERING FOR THE GOVERNMENT OF ALBERTA.
DATUM: NAD 83 CSRS PROJECTION: 3TM 114

CLIENT
ALBERTA ENVIRONMENT AND PARKS

CONSULTANT
GOLDER
MEMBER OF WSP

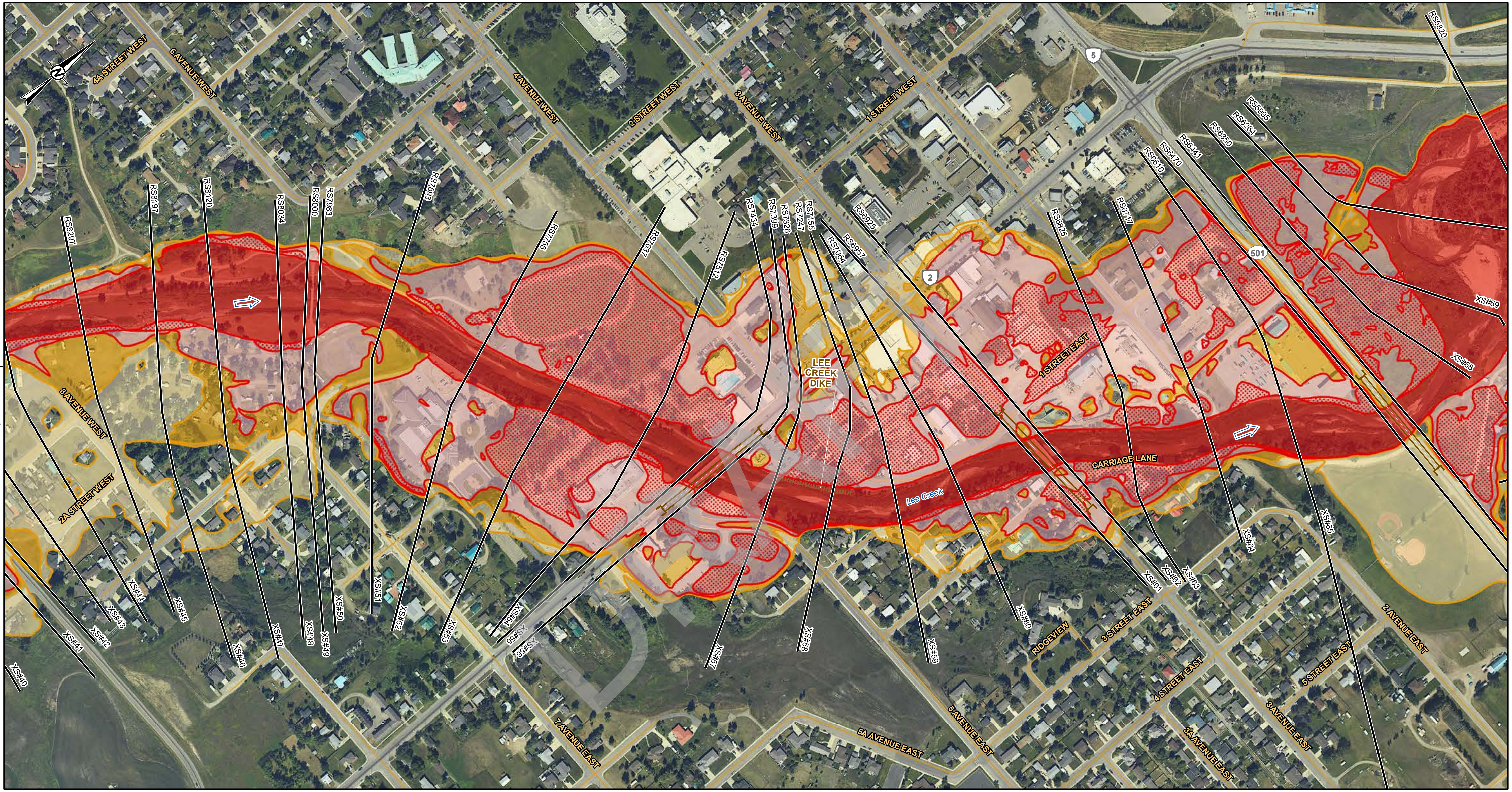
YYYY-MM-DD	2022-02-15
DESIGNED	PH
PREPARED	SK/AL
REVIEWED	WP
APPROVED	WP

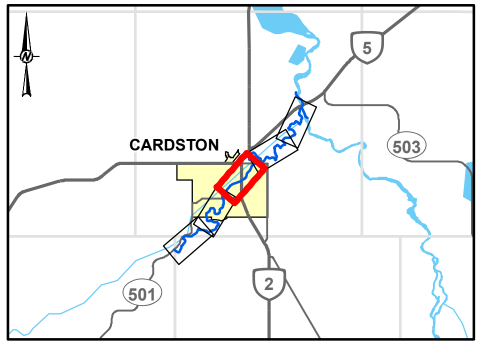
PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 2 OF 5

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





LEGEND

CROSS SECTION	FLOOD CONTROL STRUCTURE
XS#100 CROSS SECTION NUMBER	**HYDRAULIC STRUCTURES**
RS 304 RIVER STATION (M)	BRIDGE
STUDY BOUNDARY	
FLOW DIRECTION	
LOCAL ROAD	
PRIMARY HIGHWAY	
SECONDARY HIGHWAY	

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M³/S

0 100 200
1:5,000 METRES

REFERENCE(S)
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TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 3 OF 5

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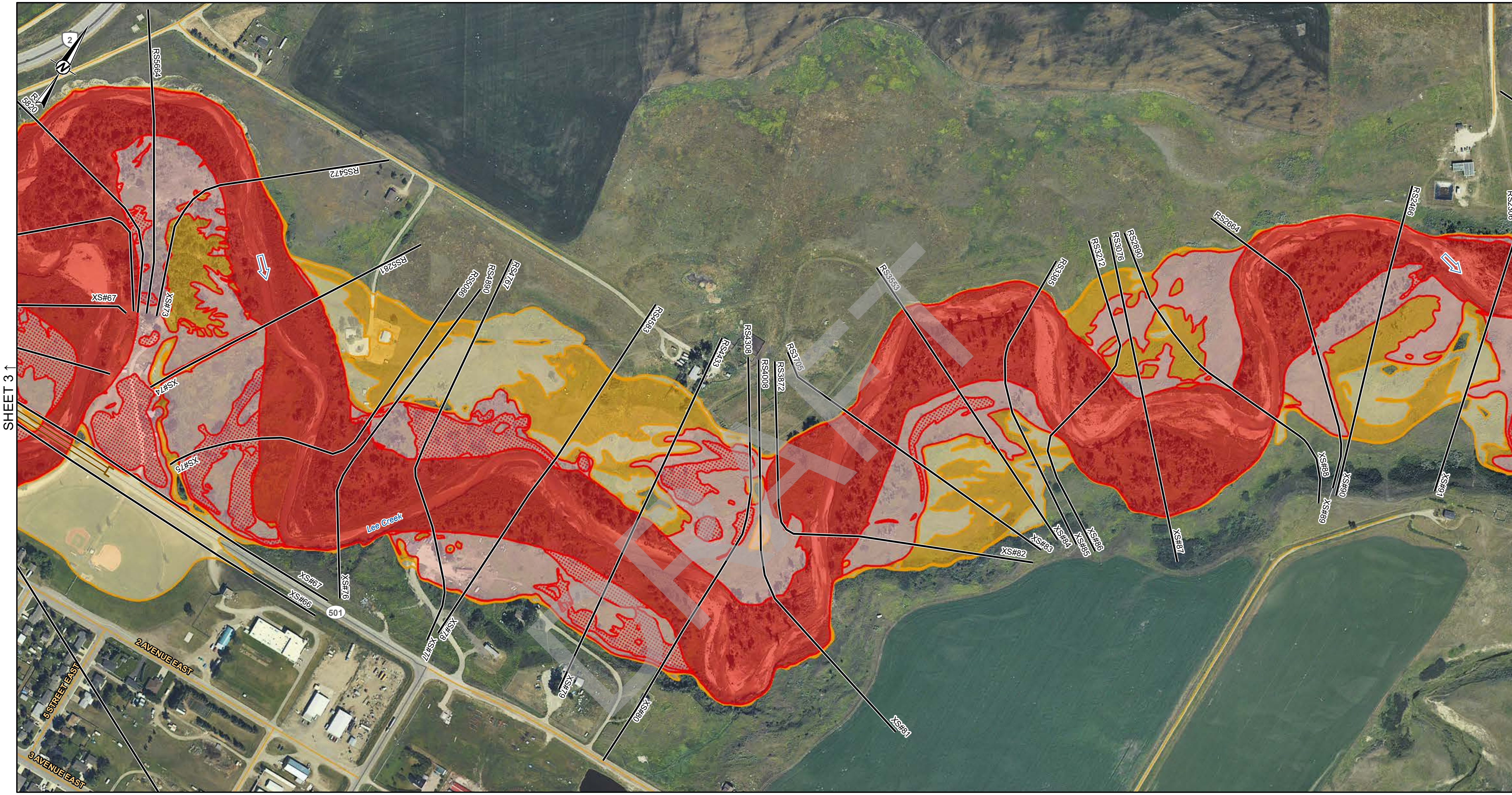
25mm

Classification: Public



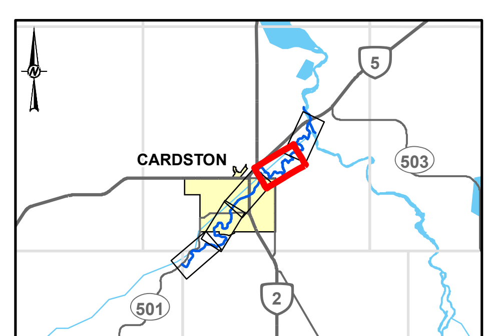
LEGEND

- CROSS SECTION
- XS#100 CROSS SECTION NUMBER
- RS 304 RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



0 100 200
1:5,000 METRES

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PROJECT
CARDSTON FLOOD HAZARD STUDY

TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 4 OF 5

LEGEND

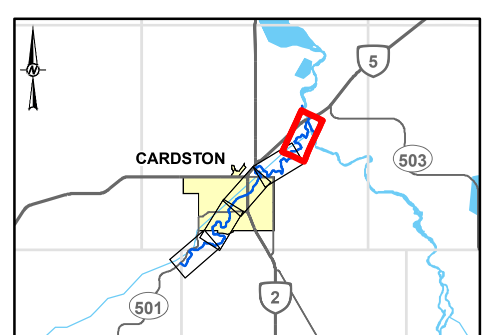
- CROSS SECTION
- XS#100 CROSS SECTION NUMBER
- RS 304 RIVER STATION (M)
- STUDY BOUNDARY
- FLOW DIRECTION
- LOCAL ROAD
- PRIMARY HIGHWAY
- SECONDARY HIGHWAY
- FLOOD CONTROL STRUCTURE

HYDRAULIC STRUCTURES

- BRIDGE

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

DESIGN DISCHARGE
LEE CREEK FLOW = 375 M^3/S



0 100 200
1:5,000 METRES

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CARDSTON FLOOD HAZARD STUDY

TITLE
FLOOD HAZARD MAP

PROJECT NO.	CONTROL	REV.	FIGURE
19117525	4000	0	SHEET 5 OF 5

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