SANGUDO FLOOD STUDY

FINAL REPORT

Prepared for:

Alberta

28 March 2022

NHC Ref. No. 1006073

nhc
northwest hydraulic consultants
water resource specialists

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Alberta Environment and Parks
Edmonton, Alberta

Prepared by:

Northwest Hydraulic Consultants Ltd.
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EXECUTIVE SUMMARY

Alberta Environment and Parks retained Northwest Hydraulic Consultants Ltd. in June 2021 to complete a flood study for the Pembina River at Sangudo. The 6.4 km long study reach includes the Hamlet of Sangudo and Lac Ste. Anne County. This study was facilitated under the Flood Hazard Identification Program (FHIP) with the intent to enhance public safety and reduce future flood damages within the Province of Alberta.

The Sangudo Flood Study is comprised of five major project components (***Survey and Base Data Collection, Open Water Hydrology Assessment, Open Water Hydraulic Modelling, Open Water Flood Inundation Mapping, and Design Flood Hazard Mapping***). This report summarizes the work of all five components. Together, these components include survey procedure and methodology, documentation on the collected survey and base data, flood history documentation, open water flood frequency flow estimations, construction and calibration of the hydraulic model, a sensitivity analysis, computation of flood frequency water levels, the associated inundation mapping, computation of design flood profiles and the floodway criteria and hazard mapping.

The majority of the survey program was completed in July 2021, with some follow-up work completed in September 2021. The objective of the survey program was to survey channel cross sections and hydraulic structures along the study reach to support the development of a one-dimensional (1D) hydraulic model. The DTM, aerial imagery, and other base mapping features were also collected to support the model development and flood mapping.

Open water flood frequency estimation was conducted at a single location (Pembina River at Sangudo). Flood frequencies have been estimated for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year events.

The open water hydraulic modelling component included the development of a calibrated hydraulic model, a model sensitivity analysis, and computation of flood levels. The hydraulic model (the model) was calibrated by adjusting channel roughness so that the computed flood levels matched well with the 1986 flood levels. Computed water levels were also consistent with other highwater mark surveys (1972, 1980, and 1989). Water surface profiles were calculated for 13 flood scenarios representing the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year open water flood events.

The computed flood levels were then used to determine the extent of inundation for each of the respective flood scenarios and are presented as a set of flood inundation maps for each scenario (the flood inundation map library). This library is intended primarily for stakeholders to use in emergency response planning and preparation.

The open water floodway criteria map and design flood hazard map are key deliverables for this project component and are provided as appendices to this report. Open water flood hazard identification involves defining the open water flood hazard area, which is comprised of floodway and flood fringe zones. Areas of deeper or faster moving water outside of the floodway (within the flood fringe) are identified as high hazard flood fringe areas. The design flood hazard map depicts the floodway and flood fringes based on the information resulting from the floodway criteria map. The methods summarized in this report follow the provincial Flood Hazard Identification Program guidelines, incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta.

CREDITS AND ACKNOWLEDGEMENTS

Northwest Hydraulic Consultants Ltd. would like to express appreciation to Alberta Environment and Parks for initiating this project, making extensive background information available, and providing the project team with valuable technical input throughout the project. Mr. Kurt Morrison, M.Eng., P.Eng., CFM managed and directed the Sangudo Flood Study on behalf of Alberta Environment and Parks.

The following NHC personnel were part of the study team and participated in the different components of the study:

- Robyn Andrishak (Project Manager and Technical Lead) provided the overall direction of the hazard study as the project manager. He was the technical lead at the survey and base data collection component of the project. He assisted in the model building. He was also responsible for reviewing inundation mapping, floodway criteria and flood hazard mapping.
- Gary Van der Vinne (Technical Reviewer) provided technical review and worked as the technical lead for open water hydrology assessment. He is the senior reviewer for this report.
- Makamum Mahmood (Senior Project Engineer) worked in open water hydrology assessment, hydraulic model development, flood history documentation, flood inundation mapping, and floodway criteria determination. He is the primary author of this report.
- Sarah North (GIS Specialist) reviewed the mapping products and GIS deliverables.
- Jerry Yan (GIS Analyst) created the mapping products and developed the associated digital asset deliverables (including the inundation map libraries).
- Luke Kostyk and Ken Roy (Survey Technologists) – responsible for field survey data collection.

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1 INTRODUCTION

1.1 Study Background

The Sangudo Flood Study was initiated by Alberta Environment and Parks (AEP) to identify and assess flood hazards along the Pembina River through Lac Ste. Anne County, including the Hamlet of Sangudo. A flood hazard mapping study was previously completed for the Sangudo area by Alberta Environment (AENV) in 1996; however, the present study covers an expanded study reach and represents an update to the prior work.

Results from this study are designed to inform local land use planning decisions, flood mitigation projects, and emergency response planning. This study is being undertaken as part of the Flood Hazard Identification Program (FHIP) with the intent of enhancing public safety and reducing future flood damages within the Province of Alberta.

This flood hazard study is comprised of the five major study components listed below.

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

1.2 Study Objectives

This report summarizes the work of all five components. The primary tasks, services, and deliverables associated with this report are:

- River cross section surveys
- Hydraulic structure data collection
- Survey and digital terrain model (DTM) data integration
- Documentation of flood history
- Open water hydrology assessment to provide flood frequency estimates
- Development of a calibrated, one-dimensional (1D) open water hydraulic model
- Simulation of open water floods of selected return periods, and creation of water surface profiles throughout the study reach

- Sensitivity analysis on selected modelling parameters
- Production of flood inundation maps
- Determination of floodway criteria and creation of design flood water surface profiles throughout the study reach
- Production of floodway criteria maps and flood hazard maps

1.3 Study Area and Reach

The flood hazard study area is located approximately 100 km northwest of Edmonton, AB. **Figure 1** shows the extent of the flood hazard study area. The flood hazard study reach extends along approximate 6.4 km of the Pembina River. Municipalities along this study reach include the Hamlet of Sangudo and the County of Lac Ste. Anne.

The Pembina River originates from the eastern slopes of the Rocky Mountains and is a tributary of the Athabasca River in central Alberta. It generally flows northeast through Lac Ste. Anne County and then north to the Athabasca River.

An overview of the contributing basin of the study reach is shown in **Figure 2**. Pembina River flows at Sangudo are not gauged by Water Survey of Canada (WSC). As shown in **Figure 2,** the study reach is located between WSC Station 07BB002 (Pembina River near Entwistle, drainage area 4,400 km^2) and 07BC002 (Pembina River at Jarvie, drainage area 13,100 km^2). The drainage area of the river at Sangudo is approximately 6,640 km^2 (AENV, 1996). The 4,400 km^2 Pembina River basin upstream of WSC Station 07BB002 lies mostly in the Foothills Natural Region. The downstream portion of the study basin is mostly in the Boreal Forest Natural Region and is used primarily for agriculture. However, overall, the Foothills portion is still dominant in the study reach.

2 SURVEY AND BASE DATA COLLECTION

2.1 Procedures and Methodology

The majority of the survey program was completed in July 2021, with some follow-up work completed in September 2021. The objective of the survey program was to survey channel cross sections and hydraulic structures along the study reach to support the development of a one-dimensional (1D) hydraulic model.

Ground positioning was established using Real-Time Kinematic (RTK) Global Navigation Satellite Systems (GNSS) and Trimble R8 and R10 GNSS receivers. The GNSS receivers were mounted on a survey rod to record ground elevations directly. The channel banks and a portion of the overbank floodplains were surveyed to ensure sufficient overlap with the supplied digital terrain model (DTM).

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

2.1.1 Coordinate System and Datum

Horizontal positions were referenced to the local three-degree Transverse Mercator (3TM) projection of the Canadian Spatial Reference System (CSRS) North American Datum of 1983 (NAD83), which has a central meridian of 114°W. Orthometric heights are based on the Canadian Geodetic Vertical Datum of 1928 (CGVD28) and the HTv2.0 geoid model.

2.1.2 Control Network

A control network was established from local Alberta Survey Control Monuments (ASCMs), Alberta Environment (AENV) benchmark, Alberta Transportation (AT) benchmark, and GNSS surveying to provide a spatial reference for the survey program. Two ASCMs, one AENV and one AT benchmark were used in the network along with one project control point established by NHC for the survey program. **Table 1** lists the control points in the network.

Three control point coordinates were determined by running the GNSS receivers simultaneously in static mode for more than four hours and post-processing baselines between control points using Trimble Business Center software. These control points are listed in **Table 2**. A network adjustment was made with the three control points. The final results involved constraining the survey to the NHC project control based on the CRS-PPP results. This point had a 7 hour occupation time and provided the best accuracy. The horizontal and vertical errors in the other two control points after post-processing and adjustment to the reference CSRS-PPP values are summarized in **Table 2**. The largest horizontal error was 0.0009 m and the largest vertical error was 0.0038 m.

Table 1 Control point summary

Point Name	Type	Easting (m)	Northing (m)	Elevation (m)
ASCM 441972	ASCM	-63653.863	5973469.293	696.007
ASCM 430959	ASCM	-63690.872	5972700.771	702.352
90-E-45	AENV	-60465.973	5971245.480	683.888
AT 53114-146	AT	-59106.655	5973938.425	669.407
NHC 1	Project Control Point	-59044.166	5973710.175	672.597

Table 2 Control network errors

Point Name	Easting (m)	Northing (m)	Elevation (m)
ASCM 441972	0.0006	0.0009	0.0038
90-E-45	0.0006	0.0009	0.0036
NHC 1 (constrained to)	N/A	N/A	N/A

A comparison between the surveyed coordinates (after post-processing and adjustment) and published ASCM coordinates, AENV and AT elevation is provided in **Table 3**. The mean of the elevation residuals in **Table 3** is -0.03 m, which indicates good vertical agreement between the control network and local benchmarks. Among the four comparison points the AT benchmark shows the highest vertical error. It should be noted that the AT benchmark was located on the side of a bridge making it difficult to shoot accurately with the GPS.

Table 3 Comparison between surveyed coordinates and published Alberta Survey Control Monument coordinates

Point Name	Residuals (Surveyed Minus Published)		
	Easting (m)	Northing (m)	Elevation (m)
ASCM 441972	-0.068	-0.552	-0.033
ASCM 430959	0.106	-0.522	0.022
AT 53114-146	N/A	N/A	-0.073
90-E-45	N/A	N/A	-0.047

2.2 Cross Sections

Cross section locations were selected to ensure adequate representation of the channel geometry in the hydraulic model with consideration given to the location of cross sections from the most recent floodplain study (AENV, 1996). During the survey, each cross section was assigned a number in an effort to organize the cross sections sequentially. However, cross section lines and associated survey points shown in **Figure 3** are labelled according to their river stationing.

A summary of the cross sections surveyed in the Pembina River is provided in **Table 4**. A total of 43 cross sections were surveyed. Survey point data has been assembled and provided as part of the digital file submission.

Table 4 Cross section survey summary

Reach	Reach Length (km)	Number of Cross Sections	Average Spacing (m)	Minimum Spacing (m)	Maximum Spacing (m)
Pembina River	7.3	43	170	20	438

The properties of cross sections surveyed on the Pembina River are summarized in **Table 5** below. Thalweg elevation was taken as the minimum surveyed elevation at each cross section. The top of the bank (TOB) channel width was determined based on the survey data, an inspection of the LiDAR-derived DTM data, aerial imagery and cross section profiles.

Table 5 Cross section properties

River Station (m)	Date Surveyed	Thalweg Elevation (m)	Channel Width (TOB) (m)	River Station (m)	Date Surveyed	Thalweg Elevation (m)	Channel Width (TOB) (m)
7,305	08 Sept 2021	658.20	89.2	3,583	28 July 2021	656.90	85.2
7,285	28 July 2021	658.12	88.1	3,367	28 July 2021	657.32	92.4
7,019	28 July 2021	657.93	84.1	3,205	28 July 2021	656.48	103.8
6,757	28 July 2021	658.03	84.3	3,185	28 July 2021	656.50	109.3
6,490	28 July 2021	657.73	78.3	3,080	28 July 2021	655.95	118.7
6,217	28 July 2021	657.69	89.6	2,912	28 July 2021	656.38	87.0
5,920	28 July 2021	657.45	81.0	2,746	28 July 2021	656.68	86.0
5,728	28 July 2021	657.62	77.6	2,557	28 July 2021	655.91	70.9
5,414	28 July 2021	657.44	94.5	2,442	27 & 28 July 2021	655.76	75.3
5,186	28 July 2021	657.21	95.2	2,428	27 & 28 July 2021	655.84	79.1
4,994	28 July 2021	657.32	68.9	2,361	27 & 28 July 2021	655.06	77.9
4,850	28 July 2021	656.87	89.6	2,343	27 & 28 July 2021	655.27	79.8
4,679	28 July 2021	656.91	111.3	2,199	29 July 2021	656.49	86.6

Table 5 Cross section properties (continued)

River Station (m)	Date Surveyed	Thalweg Elevation (m)	Channel Width (TOB) (m)	River Station (m)	Date Surveyed	Thalweg Elevation (m)	Channel Width (TOB) (m)
4,532	28 July 2021	657.02	117.2	2,082	29 July 2021	656.21	93.0
4,434	08 Sept 2021	655.91	117.8	1,851	29 July 2021	656.29	81.3
4,420	08 Sept 2021	656.19	118.2	1,605	29 July 2021	655.96	75.4
4,386	28 July 2021	656.75	116.5	1,400	29 July 2021	656.23	89.1
4,299	28 July 2021	656.87	103.6	1,113	29 July 2021	655.97	122.7
4,182	28 July 2021	657.06	86.5	812	29 July 2021	655.85	85.4
4,046	28 July 2021	656.92	81.6	374	29 July 2021	655.72	97.7
3,908	28 July 2021	657.26	87.9	0	29 July 2021	655.84	84.4
3,777	28 July 2021	656.87	78.3				

2.3 Hydraulic Structures

Table 6 summarizes the hydraulic structures in the study reach. Three bridges and one abandon pier were identified and surveyed within the study area. Hydraulic structure locations are shown in **Figure 3.**

Survey data for these structures has been assembled and provided as part of the digital study file; bridge and culvert details are provided in **Appendix A**.

Data collected at each bridge includes:

- Span length and deck width
- High chord (top of curb or solid guardrail) elevations (upstream and downstream)
- Low chord elevations (upstream and downstream)
- Number, location and width of piers
- Type and shape of piers
- Photographs of the bridge

Table 6 Hydraulic structure summary

Reach	Description	River Station (m)	Structure Type
Pembina River	Canadian National (CN) Railway Bridge	4,427	Bridge
	CN Railway Bridge Pier (Abandoned)	3,195	Pier Only
	HWY 43 - Eastbound Bridge	2,435	Bridge
	HWY 43 - Westbound Bridge	2,352	Bridge

2.4 Flood Control Structures

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

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

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

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

2.5 Other Survey Data

2.5.1 Discharge Measurements

A discharge measurement was conducted at the Pembina River downstream of the Highway 43 WBL bridge during the survey to support calibration of the hydraulic model. The measurement was taken on 30 July 2021, using a boat mounted Sontek M9 RiverSurveyor Acoustic Doppler Current Profiler (ADCP), which can measure water depths ranging from 0.06 m to 40 m and provide an accuracy of ±0.25% in velocity measurement. The discharge measurements followed the standard procedures of the Water Survey of Canada (WSC). A discharge of 7.2 m^3/s was measured.

2.5.2 Site Photographs

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

2.6 Other Features

2.6.1 Water Survey of Canada Benchmarks

There is no WSC gauging station within the study reach of the Pembina River at Sangudo. Thus no WSC benchmark is available to compare to the control network.

2.6.2 Aerial Imagery

Aerial imagery was acquired for AEP by OGL Engineering Ltd. On 10 Sep 2021. Fully-processed, orthophoto mosaics were provided to NHC by AEP on 22 July 2021.

2.6.3 Design Drawings

NHC requested design drawings for bridges through Alberta Environment and Parks, Alberta Transportation and CN Railway. Information was obtained for the following structures:

- Highway 43 Eastbound Bridge (BF73919)
- Highway 43 Westbound Bridge (BF78131)
- CN Railway Bridge

2.6.4 Base Mapping Features

In addition to the data sets listed above, additional base mapping data were obtained to support modelling and mapping for the study, including road network, hydrography, administrative boundaries, topographic maps, AltaLIS LiDAR15 DEM and Alberta Township System (ATS) grids within the study area.

3 FLOOD HYDROLOGY

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

3.1 Flooding History

3.1.1 General Information

A description of local flood history has been prepared to provide context for the hydraulic model creation and calibration. This flood history documentation summarizes information related to both open water and ice jam related flooding that has been documented and observed.

3.1.2 Open Water Floods

Open water flooding in the Pembina River could be divided into two distinct categories which can best be classified as upper basin floods and total basin floods.

In upper basin floods, flood waters are mostly generated upstream of Entwistle (WSC Station 07BB002 as shown in **Figure 2**) from the mountains and foothills. In upper basin floods, flood waters between Entwistle (WSC Station 07BB002) and Jarvie (WSC Station 07BC002), which is mostly dominated by the Boreal Natural Region, are not as large as the upper basin contribution. During such events, the flood peaks at Jarvie (WSC Station 07BC002) are usually lower than the flood peaks at Entwistle (WSC Station 07BB002) due to the attenuation or reduction in the peak caused by flood waters inundating the Pembina River floodplain.

In the total basin floods, flood waters are more uniformly generated throughout the basin. In these type of floods, the flood peaks at Jarvie (WSC Station 07BC002) are substantially higher than the flood peaks at Entwistle (WSC Station 07BB002).

The major floods in the Pembina River at Sangudo were found to be due to upper basin floods.

Historic and Observed Open Water Floods

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

It appears that information on historic floods prior to 1914 is not available. Between 1923-1954 (the period when systematic flow measurements were halted), it is believed that two major floods occurred (one in 1944 and the second one in 1954) based on miscellaneous high water level records at the Manola Railway bridge, located about 80 km downstream of the study site. The 1944 flood peak at

Manola was estimated to be at least 1,130 m^3/s (AENV, 1996). No recorded peak flood discharge was available or could reasonably be estimated to the study site for those two events.

Recent and Recorded Open Water Floods

As mentioned earlier, Pembina River flows at Sangudo are not gauged. However, flood characteristics in the study reach can be described from WSC streamflow gauging stations existing at several locations both upstream and downstream of Sangudo. The closest gauging station to the study site is located upstream of the Pembina River near Entwistle (WSC Station 07BB002). Systematic flow measurements on the Pembina River near Entwistle began (WSC Station 07BB002) in 1914 and were discontinued in March, 1923; and then restarted again in November, 1954.

Floods in the Pembina River more commonly occur in June-July due to summer rainfall events but have been observed as late as September and as early as April. The 1986 flood is the largest recorded flood in the Pembina River basin, followed by floods in 1980 and 1972. All these three floods occurred in June-July and are believed to be due to summer rainstorms. These three floods could be classified as upper basin floods.

The 1986 flood is the largest recorded flood in the Pembina River basin. The flood peak instantaneous discharge estimation is available for this event at Pembina River near Entwistle (WSC Station 07BB002). WSC estimated the 1986 flood peak to be 1,250 m^3/s, with daily flows of 1,100 and 1,180 m^3/s on July 19 and July 20. The WSC estimation was based on a high-water mark and was considered low when compared with the peak measurements by Alberta Environment (AENV) for the Pembina River at Belvedere Bridge (AENV, 1991), as marked in **Figure 2**. The 1986 flood peak instantaneous discharge measured for the Pembina River at Belvedere Bridge temporary gauge station was 1,450 m^3/s. AENV (1991) believed that the 200 m^3/s difference between Entwistle and Belvedere suggested by WSC could not be reasonably made up from local contributing areas when the peak-reducing influences of channel routing are added. The Hydrology Branch of AENV estimated the 1986 peak for Pembina River at Entwistle as 1,400 m^3/s based on the SSAR routing model and an iterative process to match the measured peak discharge of 1,450 m^3/s at Belvedere (AENV, 1991). The 1986 flood peak of 1,400 m^3/s at Pembina River near Entwistle is adopted for the current study.

3.1.3 Ice Jam Floods

No well-documented information on ice jam flooding is available for the Pembina River. However, a local resident who lives upstream of the study area mentioned flooding on their property on 22 April 2020. According to the resident, ice blockage at the CN Rail Bridge caused the flooding.

3.2 Flood Frequency Analysis

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

3.2.1 Flood Frequency Flow Estimates

Flood frequency estimates from the 2- to 1000-year floods were provided for the Pembina River at Sangudo. The adopted flood frequency estimates are presented in **Table 7** along with its 95% confidence limits**.**

Table 7 Adopted Flood frequency estimates for Pembina River at Sangudo

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge (m^3/s)	
		Value	95% Confidence Limit
1000	0.1	2,720	1,980 - 4,110
750	0.13	2,520	1,840 - 3,760
500	0.2	2,250	1,670 - 3,310
350	0.29	2,040	1,530 - 2,950
200	0.5	1,730	1,320 - 2,440
100	1	1,400	1,090 - 1,910
75	1.3	1,270	1,000 - 1,720
50	2	1,110	887 - 1,480
35	2.9	981	792 - 1,280
20	5	796	655 - 1,010
10	10	597	504 - 732
5	20	426	367 - 505
2	50	231	202 - 265

3.2.2 Comparison with Previous Study

A flood frequency analysis for the Pembina River at Sangudo is available from the AENV (1991) study. The adopted flow synthesis approach in the present study is similar to but more appropriate (due to use of more representative WSC gauges, details provided in Appendix C) than the SSARR modelling approach undertaken by AENV (1991) for the previous study.

The flood frequency estimates for the Pembina River at Sangudo are presented in **Table 8,** and compared with the previous study. The current flood frequency estimates are comparable with previous flood frequency estimates (AENV, 1991); but, on average, are 5% higher.

Table 8 Flood frequency estimates for Pembina River at Sangudo and compared with previous study

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge (m3/s) for Pembina River at Sangudo	AENV (1991)
1000	0.1	2,720	
750	0.13	2,520	
500	0.2	2,250	
350	0.29	2,040	
200	0.5	1,730	
100	1	1,400	1,270
75	1.3	1,270	
50	2	1,110	1,040
35	2.9	981	
20	5	796	762
10	10	597	580
5	20	426	416
2	50	231	221

4 HYDRAULIC MODELLING

4.1 Available Data

The data available to develop and calibrate the hydraulic model are described below. Additional information such as past studies, historical flood photographs, and existing hydraulic models also informed model development.

4.1.1 Digital Terrain Model

A digital terrain model (DTM) based on airborne LiDAR data was supplied by AEP for this study. The DTM was based on data collected by Airborne Imaging in 2020.

4.1.2 Existing Hydraulic Models

A previous hydraulic model was developed as part of the 1996 Pembina River at Sangudo Flood Risk Mapping Study. This study modelled a portion of the Pembina River within the current study area. Various model parameters reported in the 1996 Study were compared against current values.

4.1.3 Highwater Marks

Highwater mark observations provide documentation of the peak water levels that occurred at a given location for a particular flood of interest. These data are used for hydraulic model calibration and validation by comparing simulated water levels to the observed highwater mark elevations along the study reach. For this study, open water highwater marks were found in records from AENV and in the previous flood study (AENV, 1996). Highwater marks were available on the Pembina River at Sangudo during floods in 1972, 1974, 1980, 1986, and 1989. Among theses measurements, the 1974 measured highwater mark is not consistent with other highwater marks and has been flagged as "Probably not a H.W.L." in the highwater marks report and has not been used in this study. For the flood events of 1972, 1980, 1986 and 1989 the highwater marks were recorded near the Highway 43 bridge and also near the CN Rail bridge for the 1980 and 1986 events. For these four events the corresponding peak discharge rates were not measured, but were estimated from the open water hydrology assessment conducted as part of this study.

Table 9 provides a summary of the open water highwater mark data available for each flood event, which could be used for hydraulic model calibration.

Table 9 Summary of open water highwater marks

Location Name	River Station	Event Date	Peak Discharge	Highwater Mark Elevation	Source
	(m)		(m^3/s)	(m)	
D/S of Highway 43 bridge EBL	2,428	28 June 1972	785	661.556	AENV 1972 HWM Report
D/S of Highway 43 bridge EBL	2,428	08 June 1980	852	661.998	AENV 1996 Flood Study
D/S of CN Railway Bridge	4,420	08 June 1980	852	662.876	AENV 1980 HWM Report
D/S of Highway 43 bridge EBL	2,428	20 July 1986	1,666	664.520	AENV 1986 HWM Report
At CN Railway Bridge	4,427	20 July 1986	1,666	665.432	AENV 1986 HWM Report
D/S of Highway 43 bridge EBL	2,428	06 Aug 1989	513	660.700	AENV 1989 HWM Report

4.1.4 Low Flow Water Elevation Measurements

Flow measurements downstream of the Highway 43 WBL bridge and corresponding water elevations were surveyed during the July 2021 survey. Water level profiles have been also recorded along the study reach on three separate occasions in 27 July 1993, 14 September 1994, and 14 June 1994 (AENV, 1996). Flow rates on these dates were measured from the Highway 43 EBL bridge. These three events and 2021 water elevation measurements should not be considered as highwater events as the associated discharges with these three events are lower than the 2-year flood. However, these events can be used in the low flow calibration.

4.1.5 Gauge Data and Rating Curves

There is no WSC gauging station within the study reach of the Pembina River at Sangudo.

4.1.6 Flood Photography

Flood photographs are available for the 1980 and 1986 floods. The flood photographs are obtained from AENV highwater mark reports and are compiled in **Figure 4**.

4.2 River and Valley Features

4.2.1 General Description

The Pembina River originates from the eastern slopes of the Rocky Mountains and is a tributary of the Athabasca River in central Alberta. It generally flows northeast through Lac Ste. Anne County and then north to the Athabasca River. The study reach is located near the upper end of the transitional zone, between the sediment extraction zone in the headwaters and the deposition zone downstream (AENV, 1996).

4.2.2 Channel Characteristics

The Pembina River at Sangudo flows through predominantly flat, undulating, and mainly cultivated terrain. The river valley is stream-cut to approximately 10 m below the surrounding terrain and averages about 1500 m in width (AENV, 1996). The Pembina River channel follows an irregular meander pattern with the occurrence of occasional islands, mid-channel bars, and point bars. The reach-average channel bed slope is about 0.00036 m/m. Based on 2-year flow conditions, the average top width through the Pembina River study reach is about 83 m and the mean depth is about 3.3 m.

4.2.3 Floodplain Characteristics

The floodplain of the Pembina River is generally covered mostly in cultivated lands along with some medium to dense natural vegetation.

4.2.4 Anthropogenic Features

The hamlet of Sangudo is located within the study area. The study area also contains four hydraulic structures (including one with an abandoned pier) which have been documented along the study reach. Details on these hydraulic structures are provided in **Appendix A**. Various campgrounds, and some residential developments are also situated along the study reach.

4.3 Model Construction

4.3.1 Methodology

The U.S. Army Corps of Engineers *Hydrologic Engineering Center-River Analysis System* (HEC-RAS) computer program (Version 6.1, 2021) was used to calculate the flood levels along the study reaches. The basic inputs required by HEC-RAS are a series of cross sections with specified distances between sections, roughness coefficients for the channel and overbank areas at each cross section, inflow discharge at the upstream boundary of each reach, and a prescribed water level or normal depth condition at the downstream boundary.

HEC-RAS can perform one-dimensional (1D), two-dimensional (2D), or combined 1D and 2D hydraulic calculations for a network of channels and hydraulic structures. For this study and as per the Term of Reference (TOR), a 1D model was constructed to calculate water surface profiles for steady state gradually varied flow. The computational procedure for steady flow calculations is based on the solution of the 1D energy equation. Energy losses between river sections are calculated as friction losses (Manning's equation) and expansion/contraction losses. The momentum equation is used by the model where rapidly varied flow conditions arise, such as hydraulics through bridges, and evaluating water surface profiles at stream junctions. The analytical approach employed by HEC-RAS has the following assumptions and potential limitations:

- Flow is gradually varied so that the boundary friction losses between cross sections can be estimated by Manning's equation using section-average parameters.
- Changes in the channel and floodplain geometry resulting from erosion or mobile bed processes that might arise during a flood cannot be directly accounted for or modelled.
- The water level is constant across each cross section, with at least three separate conveyance components representing the main channel and each of the left and right overbank.
- Flow is one-dimensional, therefore only velocity components in the principal direction of flow are accounted for in the equations and calculations.

The following sections outline the model construction and parameter selection process for this study.

4.3.2 Geometric Database

The geometric database provides all of the components of the HEC-RAS model geometry, including cross sections, internal hydraulic structures, and boundary conditions. Each component is described below. Additional information and data are provided as part of the electronic deliverables of the study.

Cross Section Data

The geometric layout of the model and cross section data were developed as follows:

- Channel centerline alignment was drawn based on survey, topographic, and aerial imagery data. A single continuous centreline was created to represent the Pembina River study reach at Sangudo.
- Overbank flow path lines were drawn along the left and right floodplains so as to represent the average distance between successive cross sections in left and right overbank flow zones. Main channel distances are derived from the channel centerline alignments described above.
- Cross section alignments were digitized at each surveyed cross section. For the main channel, a straight line best-fitting the cross section survey points was drawn. The cross sections were then extended into the left and right overbank areas to cover the estimated 1000-year flood limits.
- Cross section elevation values from the survey point data were projected onto the cross section lines. The remainder of the cross section elevation data was sampled from the DTM provided by AEP, with a minimize area change filter applied, if required, to bring the number of cross section points below the 500 point per cross section limit of HEC-RAS.
- The locations of the left and right bank stations were determined by inspection of survey point codes generated in the field, aerial imagery, and simulated values for the 2- and 5-year flood levels.

Surveyed cross section details are tabulated in **Table 5**.

Bridges and Culverts

The modelled reach includes four bridge crossings. **Section 2.3** provides a summary of bridges included in the analysis. Key hydraulic structure design information incorporated into the model can be found in **Table 10** below. Any culverts in the study area that service local drainage only or were not relevant to the hydraulic model computations were not modelled.

Each bridge structure's alignment and location was established in ArcGIS. Bridge cross sections include approach roadways and abutments in the left and right overbanks, bridge piers, and bridge deck high and low chord profiles. Approach roadway profiles are based on extracted DTM elevation data supplemented with data from bridge drawings. Abutment geometry, piers, and high and low chords were determined from design drawings (if available) and/or survey data. Model bridge geometry was checked against design drawings, available AT bridge file records, and other information as available.

Table 10 Description of bridges included in the hydraulic model

Description	River Station (m)	Design Drawing /Info	Span (m)	Width (m)	No. of Piers	Pier Width (m)	Deck/Pier Skew (°)	Minimum Elevation (m)	
								High Chord	Low Chord
CN Railway Bridge	4,427	Yes	292.4	5.2	4	2.6-3.4	10°/10°	678.41	676.40
CN Railway Bridge Pier (Abandoned)	3,195	No	N/A	N/A	1	2.5	N/A	N/A	N/A
HWY 43 - Eastbound Bridge	2,435	Yes	137.2	10.5	5	0.9 – 1.5	N/A	668.64	667.24
HWY 43 - Westbound Bridge	2,352	Yes	133.6	13.6	2	0.9-1.5	N/A	669.47	666.61

For low flow conditions, the model was configured to use the highest energy solution of the energy, momentum, or Yarnell methods. The energy method was specified for conditions where a bridge is overtopped but this method was not invoked in the study.

Boundary Conditions

A normal depth boundary condition with a slope of 0.00036 m/m was used at the downstream boundary of the Pembina River. This slope was estimated from the energy grade line for the study reach and is consistent with the bed slope.

An inflow discharge was assigned at the upstream boundary of the Pembina River modelled reach.

4.3.3 Model Calibration

Methodology

Model calibration involved the selection of modelling parameters to simulate observed water levels along the study reach for both high and low flow conditions. Calibration parameters included:

- Manning's roughness coefficients for the channel and floodplain;
- Ineffective flow areas at each model cross section;
- Expansion and contraction loss coefficients; and
- Discharge coefficients for flow overtopping roadway crossings and embankments.

Of the above, the primary calibration parameter is typically Manning's roughness for the river channel, which is selected by comparing the simulated water surface profile elevations to observed water levels and highwater marks. The challenges or limitations that are typical to the calibration process include:

- The availability and accuracy of the highwater mark elevations.
- Proper identification of highwater mark locations.
- Uncertainties in estimates of the flood peak discharges.
- Insufficient channel geometry data.

The type of land cover was used to help characterize roughness in the overbank floodplain areas. Orthophotography indicates that the overbank floodplain area mainly consists of agricultural crops or pastureland along with light to medium dense grasses, light brush, and trees. The overbank areas also consist of some recreational facilities (including parks, campgrounds, etc.) located just downstream of the CN Railway bridge. No noticeable urban development was observed within the 1000-year floodplain. It is believed that a constant and single value of roughness coefficient based on values provided in reference literature (Chow, 1959) could describe the land cover type for this 6.4 km long study reach.

Note that the Highway 43 WBL bridge was not yet constructed during any of the highwater mark events and three low flow events which were captured as part of the previous flood study (AENV, 1996). So, for the calibration the Highway 43 WBL bridge was not included in the HEC-RAS the model.

Low Flow Calibration

Discharges of the Pembina River are measured at Sangudo only under exceptional circumstances, such as part of this study during the cross section survey and also as part of the previous flood study (AENV, 1996). In total, there are four events for which discharge measurements are available and summarized in **Table 11**. It should be noted that all of these four events have a flow rate lower than the 2-year flood.

For each of these events the water level profile has been recorded at the same time the discharge at Sangudo was measured.

As the water level measurements are available throughout the reach, a known water surface elevation was assigned for each low flow event based on the most downstream measured water level. This helped to calibrate Manning's roughness by removing any effect of the downstream boundary condition.

The channel Manning's roughness coefficient was calibrated separately for each low flow event to obtain the best fit between the observed and simulated water levels. A single channel roughness coefficient is used for the calibration as there was no compelling evidence to suggest that there should be any notable variation in roughness along the less than 10 km long study reach. **Table 11** summarizes the required Manning's roughness for each event which produces the lowest mean error between observed and simulated water levels. The calibrated Manning's roughness from the previous study (AENV, 1996) was also provided in the table for comparison. The previous study had calibrated the reach upstream and downstream of the CN Rail bridge separately and thus had two calibrated roughness coefficients. However, it is recommended that a single roughness coefficient be adopted for the study reach as there was no evidence of a change in flow regime within this Pembina River study reach. The calibrated Manning's roughness from the current study is comparable with the calibrated low flow Manning's roughness from the previous study (AENV, 1996).

Table 11 Calibrated channel Manning's roughness for low flow calibration

Event Date	Discharge (m^3/s)	Calibrated Channel Manning's roughness	Calibrated Channel Manning's roughness (AENV, 1996)
27 July 1993	63.7	0.034	0.030 (d/s CN Rail bridge)
14 September 1993	15.2	0.056	0.031 (u/s CN Rail bridge), 0.049 (d/s CN Rail bridge)
17 June 1994	119	0.035	0.026 (u/s CN Rail bridge), 0.033 (d/s CN Rail bridge)
30 July 2021	7.2	0.058	N/A

Among these four events, the event of 17 June 1994 is the largest one. However, the measured discharge from this event is only half that of the estimated 2-year flood. For this event, a Manning's roughness coefficient of 0.035 produced the least mean error between the simulated and observed water levels. The second-largest event in this low flow calibration is the event of 27 July 1993, for which a calibrated Manning's roughness of 0.034 was obtained. This suggests that for a flood frequency of less than 2-year, a channel roughness of around 0.034/0.035 would be reasonable.

The table also shows significantly higher calibrated Manning's roughness for 14 September 1993 and 30 July 2021 events. The discharge measured in these two events is less than one-tenth of a 2-year flood and should not be considered in the flood model calibration.

High Flow Calibration

The July 1986 flood is the largest recorded flood in the Pembina River since the continual systematic collection of gauge data. Highwater marks have been available for this event at the CN Rail bridge and downstream of the Highway 43 EBL bridge. Aerial flood photographs have also been available for this event. The 1986 flood event is believed to have a return period of close to a 200-year flood. The other flood events considered in the high flow calibration are 1972, 1980, and 1989. Note that no discharge measurements were available at Sangudo for any of these flood events. The estimated flows for these events were obtained from the open water hydrology assessment (provided in **Appendix C**).

The channel Manning's roughness coefficient was calibrated separately for each high flow event to obtain the best fit between the observed and simulated water levels. **Table 12** summarizes the calibrated Manning's roughness for each event which produces the lowest mean error between observed and simulated water levels. The calibrated Manning's roughness from previous study (AENV, 1996) was also provided in the table for comparison. Note, that the estimated flood discharges from previous study is different than the estimated flood discharges as part of current study.

Table 12 Calibrated channel Manning's roughness for high flow calibration

Event Date	Estimated Peak Discharge (m^3/s)	Calibrated Channel Manning's roughness	Estimated Discharge (m^3/s) (AENV, 1996)	Calibrated Channel Manning's roughness (AENV, 1996)
28 June 1972	785	0.024	700	0.028
08 June 1980	852	0.026	810	0.026
20 July 1986	1,666	0.029	1,540	0.028
06 Aug 1989	513	0.026	398	0.031

The flood events for high flow calibration range between about the 5-year to 200-year flood discharges. The calibrated Manning's roughness for these flood events range between 0.024 to 0.029 for the current study and 0.026 to 0.031 for the previous study. The calibrated channel Manning's roughness between current and previous study could be considered comparable, considering change in estimated discharges and possible change of channel geometry. There is no defined pattern observed between the discharge and calibrated Manning's roughness. Thus the variation in Manning's roughness on different flood events is believed to be associated with uncertainties on flow estimation, channel geometry, and accuracy of the highwater mark elevations.

Calibration Results

The high flow calibration for the Pembina River at Sangudo was carried out using highwater marks collected from the 1972, 1980, 1986, and 1989 flood events. Emphasis was placed on calibrating computed water levels to the observed highwater marks for the 1986 flood event as it is the largest recorded flood in the Pembina River. The calibrated channel Manning's roughness for this event is 0.029.

This Manning's roughness was used to simulate water surface profiles for other flood events. **Figure 5** shows the comparison between simulated water surface profiles and observed highwater marks for each event, offering a good visual fit. A tabular statistical summary of the high flow calibration is provided in **Table 13**. The difference between observed highwater marks and simulated water levels was less than 0.01 m downstream of Highway 43 bridge EBL and -0.10 m d/s of the CN Rail bridge for the 1986 flood event, which was selected as the primary calibration event. In addition, a good correlation was observed when conducting a visual comparison of 1986 simulated flood extents with flood imagery (as shown in **Figure 6**). Note that, the 1986 simulated flood extent is from the current model and 2021 DEM. The slight difference in simulated and observed 1986 flood from visual comparison could be due to number of reasons, including change in floodplain land use, construction of anthropogenic features like roadway in the floodplain, and change in grade in last 35 years, and the timing of flood photograph.

Figure 7 includes the comparison between simulated water surface profiles and surveyed water levels for the low flow events. A tabular statistical summary of the low flow calibration is also included in **Table 13.** Less emphasis was given to these low flow calibrations, as the discharge measured on these events are significantly lower than the 2-year flood.

Table 13 Calibration results for Pembina River at Sangudo

Event Type	Event Date	Estimated Peak Discharge (m^3/s)	Observed Minus Simulated Water Level (m)		
			Min	Max	Average
High Flow Events	28 June 1972	785	-0.51	-0.51	-0.51
	08 June 1980	852	-0.42	-0.31	-0.37
	20 July 1986	1,666	0.00	-0.10	-0.05
	06 Aug 1989	513	-0.26	-0.26	-0.26
Low Flow Events	27 July 1993	63.7	0.01	0.17	0.06
	14 September 1993	15.2	0.03	0.41	0.20
	17 June 1994	119	0.11	0.21	0.13
	30 July 2021	7.2	0.00	0.57	0.19

As the model is calibrated for the 1986 flood event, the simulated and observed water levels are in good agreement for this event. For other flood events (1972, 1980, and 1989), the simulated water levels are higher than observed. It indicates that a lower Manning's roughness coefficient might be more appropriate for these other lower frequency flood events. However, it contradicts with the 1986 calibration as Manning's roughness coefficient tends to decrease with increasing discharges. So choosing a lower Manning's roughness coefficient for lower flood events seems unreasonable, especially when there is uncertainty in the calibration process associated with the flow estimation, channel geometry, and accuracy of the highwater mark elevations. It is believed calibrating the model with the largest recorded flood is more appropriate. This calibrated model based on the 1986 flood event also produces conservative higher flood levels for other lower frequency highwater events. The 1986 flood event is selected as the primary calibration event as it is the largest flood event on record and has a magnitude above the estimated 100-year frequency flood which was selected as the design flood for the flood hazard maps.

For low flow events, the simulated water levels are generally lower than the observed water levels, which is well within the expected range when the model is calibrated for large flood events.

The calibrated model was also checked by generating rating curves from the simulated water levels at two locations: one at downstream of the Highway 43 EBL bridge and the second one is at downstream of the CN Rail bridge. Several highwater marks and observed water level data are available at these two location. **Figure 8** and **Figure 9** show the simulated rating curve at downstream of the Highway 43 EBL bridge and downstream of the CN Rail bridge respectively. These simulated rating curves from the calibrated model were compared with the available highwater mark measurements for the flood events and observed water levels for low flow events. Overall, good agreement was attained between the simulated rating curve and observed highwater marks and low flow water levels. This comparison verifies the adopted Manning's roughness over a range of discharges and provides confidence in the ability of the model to simulate water levels over a range of flows along the reach of the Pembina River located within the study area.

4.3.4 Model Parameters and Options

The following sections describe the key model parameters and options adopted in the HEC-RAS model. These include Manning's roughness coefficients for the channel and overbank areas, contraction and expansion loss coefficients, and ineffective areas.

Channel and Overbank Roughness Values

Manning's roughness is used to account for an array of energy losses that may vary with respect to discharge. A minimum of three (one channel and two overbank) roughness values were used within each cross section. Roughness values were assumed to be constant with discharge.

For channel, a Manning's roughness value of 0.029 was adopted for the whole study reach. This adopted channel roughness is similar as the Manning's roughness adopted in the previous flood study for downstream of CN Rail bridge reach (AENV, 1996).

For overbank floodplain areas and islands, a Manning's roughness of 0.05 was adopted based on the land use type ad recommended values in the literature (Chow, 1959).

Expansion and Contraction Coefficient

To account for the effect of flow contraction or expansion on the energy balance between successive cross sections, HEC-RAS multiplies the absolute difference in velocity head by a coefficient. The coefficients range from 0.1 for gradual transitions to 0.8 for abrupt transitions (Brunner, 2016).

The default values of 0.1 for expansion losses and 0.3 for contraction losses were used throughout the model, except for cross sections adjacent to bridge or culvert crossings where the values were increased to 0.3 and 0.5 to account for abrupt changes in flow area.

Weir Coefficient

For this study, even the 1000-year flood does not overtop any of the bridge decks. Therefore, flow overtopping road, rail, or similar embankments crossing the flow path was not simulated so the broad crested weir coefficient had no effect on the study results.

Blocked Obstructions

Blocked obstructions in the floodplain, such as buildings, walls, storage tanks, or elevated foundations were not specified in the HEC-RAS model. Obstructions associated with bridge piers and structural members were modelled using the standard bridge editor specifications in HEC-RAS.

Ineffective Flow Areas

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

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

General Criteria Used to Define Ineffective Areas

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

- Non-permanent ineffective flow areas were used to “fill” local depressions on the floodplain that are obstructed by higher ground upstream or downstream. These areas were assumed to become engaged in the active flow area (or effective) once the water level exceeded the elevation of the adjacent ground.
- Permanent ineffective flow areas were used to permanently “fill” relic channels, tributary channels or excavated holes that would otherwise have incorrectly added flow area to the cross section.

- Permanent ineffective flow areas were defined where flow patterns were likely to be influenced by nearby bridge abutments and roadway embankments crossing the floodplain. These types of obstructions tend to direct flows towards the bridge opening. Several site-specific factors were taken into account when configuring ineffective flow areas at bridges in the study area, including: distance from the cross section to the bridge, terrain features, and bridge geometry.

Non-permanent conditions often produce the undesirable result of water level profiles of high magnitudes dipping below water level profiles computed for lower flood magnitudes, so the selection of a non-permanent condition was avoided wherever possible.

4.3.5 Flood Frequency Profiles

The calibrated hydraulic model was used to generate flood frequency profiles for the thirteen open water floods of varying magnitude ranging from 2-year to 1000-year return periods. The computed flood frequency water levels at each surveyed cross section on the Pembina River are provided in **Table 14**. These results are plotted graphically in **Figure 10**.

Table 14 Computed flood frequency water levels

River Station (m)	Flood Return Period												
	2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
	Water Surface Elevation (m)												
7,305	661.26	662.46	663.29	664.11	664.76	665.15	665.51	665.83	666.58	667.17	667.56	667.98	668.28
7,285	661.25	662.45	663.28	664.10	664.75	665.14	665.50	665.81	666.55	667.16	667.56	667.98	668.28
7,019	661.18	662.37	663.20	664.02	664.67	665.06	665.42	665.73	666.49	667.11	667.49	667.93	668.23
6,757	661.11	662.30	663.13	663.95	664.60	664.98	665.34	665.65	666.42	667.06	667.45	667.90	668.20
6,490	661.02	662.21	663.04	663.86	664.51	664.89	665.25	665.58	666.38	667.04	667.43	667.89	668.20
6,217	660.95	662.15	662.99	663.81	664.46	664.85	665.22	665.57	666.38	667.04	667.45	667.89	668.20
5,920	660.84	662.03	662.86	663.67	664.34	664.76	665.16	665.52	666.36	667.03	667.43	667.88	668.19
5,728	660.77	661.96	662.79	663.61	664.28	664.72	665.13	665.49	666.34	667.01	667.41	667.87	668.18
5,414	660.69	661.90	662.74	663.56	664.23	664.67	665.07	665.44	666.28	666.95	667.35	667.81	668.12
5,186	660.64	661.85	662.69	663.51	664.19	664.63	665.02	665.38	666.22	666.89	667.27	667.73	668.05
4,994	660.54	661.72	662.53	663.34	663.99	664.40	664.75	665.09	665.90	666.56	666.96	667.44	667.78
4,850	660.49	661.69	662.51	663.32	663.98	664.41	664.78	665.14	665.96	666.63	667.03	667.50	667.83
4,679	660.45	661.65	662.48	663.30	663.98	664.42	664.79	665.15	665.98	666.64	667.05	667.51	667.84
4,532	660.41	661.62	662.45	663.28	663.95	664.38	664.75	665.10	665.91	666.56	666.96	667.40	667.71
4,434	660.37	661.58	662.42	663.25	663.93	664.36	664.72	665.07	665.88	666.52	666.92	667.36	667.67
4,420	660.35	661.52	662.35	663.17	663.85	664.27	664.64	664.98	665.79	666.43	666.82	667.26	667.57
4,386	660.34	661.51	662.34	663.17	663.84	664.27	664.63	664.98	665.79	666.44	666.84	667.30	667.61
4,299	660.32	661.49	662.31	663.13	663.81	664.23	664.58	664.93	665.75	666.40	666.80	667.25	667.57
4,182	660.26	661.43	662.25	663.06	663.72	664.14	664.49	664.83	665.64	666.30	666.70	667.15	667.47
4,046	660.22	661.38	662.20	663.01	663.68	664.10	664.44	664.77	665.51	666.17	666.56	667.01	667.32
3,908	660.18	661.35	662.17	662.98	663.65	664.06	664.40	664.73	665.48	666.10	666.48	666.91	667.23
3,777	660.12	661.28	662.09	662.90	663.56	663.97	664.29	664.62	665.35	665.99	666.38	666.83	667.15
3,583	660.05	661.23	662.04	662.85	663.51	663.92	664.24	664.58	665.34	666.00	666.39	666.84	667.16
3,367	659.96	661.15	661.98	662.79	663.46	663.87	664.19	664.52	665.28	665.93	666.32	666.75	667.06

Table 14 Computed flood frequency water levels (continued)

River Station (m)	2-year	5-year	10-year	20-year	35-year	50-year	75-year	100-year	200-year	350-year	500-year	750-year	1000-year
	Flood Return Period												
	Water Surface Elevation (m)												
3,205	659.94	661.14	661.97	662.78	663.45	663.87	664.19	664.52	665.27	665.92	666.32	666.75	667.06
3,185	659.83	661.02	661.83	662.65	663.31	663.72	664.18	664.51	665.26	665.91	666.29	666.72	667.02
3,080	659.78	660.95	661.76	662.58	663.25	663.66	664.13	664.46	665.24	665.90	666.30	666.73	667.04
2,912	659.73	660.89	661.70	662.50	663.15	663.56	664.03	664.36	665.14	665.81	666.21	666.65	666.96
2,746	659.68	660.84	661.64	662.44	663.09	663.50	663.97	664.30	665.10	665.78	666.18	666.62	666.93
2,557	659.57	660.71	661.49	662.28	662.92	663.35	663.82	664.19	665.02	665.71	666.11	666.56	666.87
2,442	659.55	660.70	661.48	662.27	662.92	663.33	663.78	664.11	664.84	665.43	665.78	666.14	666.40
2,428	659.53	660.67	661.46	662.25	662.89	663.30	663.75	664.08	664.80	665.40	665.74	666.10	666.35
2,361	659.52	660.66	661.44	662.23	662.87	663.28	663.73	664.06	664.78	665.38	665.72	666.08	666.33
2,343	659.39	660.53	661.31	662.10	662.74	663.15	663.61	663.93	664.66	665.26	665.60	665.96	666.21
2,199	659.35	660.49	661.28	662.06	662.71	663.13	663.60	663.93	664.68	665.30	665.66	666.04	666.29
2,082	659.33	660.46	661.25	662.04	662.68	663.10	663.56	663.89	664.61	665.21	665.54	665.89	666.15
1,851	659.23	660.36	661.13	661.92	662.56	662.97	663.42	663.73	664.42	664.99	665.31	665.64	665.88
1,605	659.13	660.25	661.01	661.78	662.41	662.81	663.26	663.58	664.26	664.84	665.16	665.48	665.70
1,400	659.06	660.18	660.94	661.72	662.35	662.76	663.21	663.52	664.19	664.75	665.06	665.36	665.58
1,113	658.95	660.08	660.85	661.64	662.29	662.70	663.16	663.47	664.16	664.73	665.04	665.35	665.56
812	658.84	659.96	660.73	661.50	662.14	662.55	663.00	663.31	664.00	664.57	664.89	665.20	665.43
374	658.70	659.84	660.61	661.39	662.03	662.44	662.90	663.21	663.89	664.47	664.79	665.09	665.31
0	658.56	659.69	660.46	661.23	661.86	662.27	662.72	663.03	663.72	664.30	664.63	664.96	665.19

4.3.6 Model Sensitivity

The sensitivity of the open water hydraulic model to adjustments in boundary conditions, Manning's roughness values, and weir coefficient for roadway overtopping were evaluated. These parameters could affect the computed water surface profiles, and by direct result, the predicted flood depths and inundation limits. The sensitivity analysis provides an indication of the plausible range of error in the model results and identifies the relative importance of each parameter to the overall error. When selecting the range of plausible parameters to test during the model sensitivity analyses, consideration was given to the variability of the factors with season and discharge. The 100-year flood was used as the baseline for the sensitivity analysis.

A summary of the sensitivity analysis results is tabulated in **Table 15** and described below.

Table 15 Summary of sensitivity analysis results

River	Model Parameter	Difference in Water Level from Baseline Profile (m)			
		Lower Limit		Upper Limit	
		Maximum	Average	Maximum	Average
Pembina River	Flood Frequency Estimates	-0.91	-0.80	1.25	1.15
	Downstream Normal Depth Slope	0.35	0.16	-0.28	-0.12
	Main Channel Roughness	-0.79	-0.67	0.62	0.54
	Overbank Roughness	-0.07	-0.05	0.05	0.03

Boundary Conditions

The lower and upper limits of the 95% confidence interval for the 100-year instantaneous peak discharge upstream boundary condition (as shown in **Table 7**) were examined in the sensitivity analysis. **Table 15** provides a summary of the deviation from the 100-year flood levels for the lower 95% limit and the upper 95% limit discharge. Water surface elevations are presented in **Appendix D** (**Table D-1**), and profiles are illustrated in **Figure 11**.

The adopted downstream boundary condition in the model was a normal depth, which was given by specifying an estimate of the energy grade line slope equal to 0.00036 m/m at the most downstream cross section. A plausible range of uncertainty in estimating the energy grade line slope is considered to be approximately ±20%, which resulted in a low value of 0.00029 m/m and a high value of 0.00043 m/m.

The water surface elevation profiles (baseline, low downstream normal depth slope, and high downstream normal depth slope) for Pembina River at Sangudo are presented in **Appendix D (Table D-2)** and illustrated in **Figure 12. Table 15** provides a summary of the deviation from the 100-year flood levels for the lower and upper case of the normal depth slope. The maximum deviation from the baseline of 0.35 m for the lower case and 0.28 m for the higher case occurs at the downstream boundary. The deviation from the baseline profile falls to 0.07 m at the upstream boundary of the study reach for the lower case and 0.05 m for the higher case.

Manning's Roughness

The sensitivity of the model to Manning's roughness was evaluated, with channel roughness examined independently of overbank roughness. The sensitivity of a lower and higher Manning's roughness was examined for all the modelled reaches. The results of the sensitivity analysis are discussed below.

The adopted channel roughness on the Pembina River at Sangudo study reach was 0.029. A plausible range of channel roughness for the modelled length of the Pembina River study reach is considered to be approximately ±20%, which resulted in a low channel roughness value of 0.023 and a high channel roughness value of 0.035.

Table 15 provides a summary of the deviation from the 100-year flood levels for low and high channel roughness for the Pembina River study reach. Water surface elevations are presented in **Table D-3** in **Appendix D,** and profiles are illustrated in **Figure 13.** The Pembina River reach at Sangudo is sensitive to changes in channel roughness values with average deviations from the baseline 100-year profile reaching 0.67 m and maximum deviations reaching 0.79 m. The profile in **Figure 13** indicates that the deviations from the baseline profile are uniform throughout the study reach.

The sensitivity of computed 100-year flood levels to overbank roughness variations was evaluated by selecting low and high roughness coefficients for the Pembina River at Sangudo study reach. Plausible values were generally within 20% of the overbank roughness value adopted for the baseline model (0.05), considering seasonal variations in vegetation growth and density. For simplicity, a low overbank roughness value of 0.04 and a high overbank roughness value of 0.06 were adopted for the sensitivity runs.

Table 15 presents a summary of the results of the 100-year computed flood level sensitivity analysis for varying overbank roughness values. Water surface elevations for each case are presented in **Table D-4** in **Appendix D,** and profiles are plotted in **Figure 14**. On average, flood levels were 0.03 to 0.07 m below baseline values for low overbank roughness. For high overbank roughness, computed flood levels were on average between 0.01 and 0.05 m above baseline values. Thus it can be said that the flood levels are not significantly sensitive to the variation in overbank roughness.

Roadway Weir Coefficient

No modelled bridges at the Pembina River study reach were overtopped for the selected 100-year baseline flood. Thus no sensitivity analysis for roadway weir coefficient was conducted.

5 FLOOD INUNDATION MAPS

Flood inundation mapping provides a visual display of areas that could be underwater in one or more flood scenarios. For this study, one flood inundation map series was created for each flood frequency return period; there are 13 map series, with return periods ranging from 2 to 1000-years. **Appendix E** contains the flood inundation maps. The following sections describe the flood inundation map production process.

5.1 Methodology

The flood inundation maps were created in five steps:

1. A water surface elevation (WSE) triangular irregular network (TIN) is created, representing a contiguous flood level profile along the modelled river reach.
2. A WSE grid with the same grid geometry as the underlying DTM is generated. Elevation values are assigned to each grid cell, based on the corresponding WSE TIN value.
3. A depth grid, having the same grid geometry as the WSE grid, is generated by subtracting DTM elevation values from the corresponding WSE grid value.
4. Inundation polygons are generated from the positive depths. Negative depths indicating dry cells are assigned a *NoData* value. Inundation polygons are further processed by smoothing and removing "isolated" wetted areas not directly inundated and "holes" (very small dry areas).
5. WSE and depth grids are clipped to the inundation extent using the inundation polygons.

The WSE TINs, WSE and depth grids, and the inundation polygons were created using standard ArcGIS tool sets and are stored in a conventional Esri file format.

5.2 Water Surface Elevation TIN Modifications

Necessary modifications were made to the water surface elevation TIN for areas that need manual edits (for example overbank flooding area or backwater area) so that inundation polygons could be re-generated from the data using the procedure described in **Section 5.1** above.

Areas showing extensive overbank/backwater flooding directly 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.

There is no flood control structure within the study reach and thus no water surface elevation TIN modifications were required for the potential flood control structure failure.

5.3 Flood Inundation Areas

The impacts of flooding on developed areas and infrastructure are evident in the flood inundation mapping libraries (**Appendix E**). **Table 16** lists notable flood impacted areas and provides an overview of flood magnitude ranges for residential, commercial, industrial, and other notable facilities. The Table lists areas from upstream to downstream, with left (right) floodplain areas on the left (right) side of the Table. The middle of the table shows the cross section numbers nearest to each flooded area to assist in cross-referencing with the inundation mapping libraries. The grey shaded boxes provide a graphical display of the approximate range of flood frequency magnitudes impacting each area. For all flood inundation areas please refer to **Appendix E**.

Impacts to bridges are illustrated in the computed flood level frequency profiles where low chord and high chord elevations are indicated on the profile plots **(Figure 10**). Up to and including the calculated 1,000-year flood level, no flood exceeds the high chord and low chord elevation of any bridge. The abandoned CN Rail bridge pier would be completely submerged during a 500-year flood and above.

Table 16 Overview of the range of flood magnitudes for areas impacted by flooding

Impacted Areas along Left Floodplain													River Station Reference	Impacted Areas along Right Floodplain												
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		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
Farmhouse southeast of CN Rail Bridge													RS 5186 and RS 4994													
					50	75	100	200	350	500	750	1K														
Properties adjacent and north of CN Rail Bridge													RS 4386													
								200	350	500	750	1K														
													RS 4299	Race Track												
																			50	75	100	200	350	500	750	1K
House (green roof) adjacent and east of Range Road 71													RS 4299													
								200	350	500	750	1K														
Properties adjacent and east of Range Road 71													RS 4182													
								200	350	500	750	1K														
House adjacent and south of Township Road 570													RS 4182													
										500	750	1K														
													RS 4182	Sangudo Riverside Campground												
																	20	35	50	75	100	200	350	500	750	1K
House northeast of Township Road 570 and Range Road 71 intersection													RS 4046													
									350	500	750	1K														
													RS 3908	Spirit of Sangudo Community Park												
																				75	100	200	350	500	750	1K
													RS 3777	Deep Creek Camping and Events												
																				75	100	200	350	500	750	1K
													RS 3583	Arena												
																										1K

Note: shaded areas indicate the flood frequencies impacting the respective area.

6 FLOODWAY DETERMINATION

Flood hazard identification involves the delineation of floodway and flood fringe zones for a specified design flood. A description of key terms from the FHIP Guidelines (AENV, 2011), incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta, is provided in **Sections 6.1 and 6.2** below.

6.1 Design Flood Selection

The design flood for open water flood hazard identification in Alberta is typically associated with a peak instantaneous discharge that has a one percent chance of being equaled or exceeded in any given year. This is a flood with a statistical 100-year return period, also commonly referred to as the "one in one hundred year flood".

The 100-year open water flood was selected as the design flood for the Pembina River at Sangudo. The discharge values used for the design flood correspond to the 100-year return period discharge of 1,400 m^3/s, listed in **Table 7.**

6.2 Floodway and Flood Fringe Terminology

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.

Flood Hazard Area

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

Floodway

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

Flood Fringe

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

Design Flood Levels

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

6.3 Flood Hazard Identification

6.3.1 Design Flood Profile

The design flood profile levels were those calculated for the 100-year open water flood condition. The resulting design flood level values are listed in **Table 17** and depicted visually in **Figure 15.**

Table 17 Computed design flood levels

River Station (m)	Design Flood Level (m)	River Station (m)	Design Flood Level (m)	River Station (m)	Design Flood Level (m)
7,305	665.83	4,420	664.98	2,442	664.11
7,285	665.81	4,386	664.98	2,428	664.08
7,019	665.73	4,299	664.93	2,361	664.06
6,757	665.65	4,182	664.83	2,343	663.93
6,490	665.58	4,046	664.77	2,199	663.93
6,217	665.57	3,908	664.73	2,082	663.89
5,920	665.52	3,777	664.62	1,851	663.73
5,728	665.49	3,583	664.58	1,605	663.58
5,414	665.44	3,367	664.52	1,400	663.52
5,186	665.38	3,205	664.52	1,113	663.47
4,994	665.09	3,185	664.51	812	663.31
4,850	665.14	3,080	664.46	374	663.21
4,679	665.15	2,912	664.36	0	663.03
4,532	665.10	2,746	664.30		
4,434	665.07	2,557	664.19		

6.3.2 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.
- In no case should the floodway boundary extend into the main river channel area.
- For reaches of supercritical flow, the floodway boundary should correspond to the edge of inundation or the main channel, whichever is larger.

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 floodway stations and determination criteria for each cross section are tabulated in **Table 18.** The limits of the floodway (also denoted as the floodway boundary) intersect cross sections at the floodway limit stations. In some instances the floodway limits are coincident with the inundation limits. This condition typically occurs when a floodway station (defined by the usual criteria) is very close to the extent of inundation and there is no practical width of flood fringe – along steep valley walls or high slopes, for example.

The floodway limit lines extending between cross sections were delineated based on the adjacent determination criteria and drawn such that the resulting lines followed a *hydraulically-smooth* path. For previously mapped reaches, an existing floodway from the 1996 flood study was adopted and adjusted according to the aforementioned exceptions. For newly mapped reaches, the floodway mostly followed along the 1 m depth contour. When the width of the flood fringe was impractically small, the floodway was drawn coincident with the water's edge.

Table 18 Selected floodway station and determination criteria

River Station (m)	Left		Right	
	Floodway Station (m)	Floodway Determination Criteria	Floodway Station (m)	Floodway Determination Criteria
7,305	365.80	1 m Depth	498.76	1 m Depth
7,285	378.95	1 m Depth	505.39	1 m Depth
7,019	580.12	Previous Floodway	698.00	Previous Floodway
6,757	764.59	Previous Floodway	885.71	Previous Floodway
6,490	959.81	Previous Floodway	1114.68	Previous Floodway
6,217	1114.12	Previous Floodway	1330.57	Inundation Extent[1]
5,920	1157.02	Previous Floodway	1440.22	Previous Floodway
5,728	1169.92	Previous Floodway	1420.60	Main Channel[2]
5,414	1144.81	Previous Floodway	1251.67	Previous Floodway
5,186	978.95	Previous Floodway	1091.70	Inundation Extent[1]
4,994	725.70	Main Channel[2]	822.24	Previous Floodway
4,850	581.60	Main Channel[2]	685.07	Previous Floodway
4,679	401.50	Main Channel[2]	540.70	Previous Floodway
4,532	405.90	Main Channel[2]	563.10	Previous Floodway
4,434	421.58	Previous Floodway	591.26	Previous Floodway
4,420	410.81	Previous Floodway	588.77	Previous Floodway
4,386	402.10	Main Channel[2]	575.67	Previous Floodway
4,299	541.03	Main Channel[2]	718.20	Previous Floodway
4,182	331.13	Inundation Extent[1]	468.28	Previous Floodway
4,046	105.58	Previous Floodway	219.07	Previous Floodway
3,908	30.94	Previous Floodway	138.11	Previous Floodway
3,777	74.10	Inundation Extent[1]	182.47	Previous Floodway
3,583	91.60	Previous Floodway	202.02	Previous Floodway
3,367	95.81	Inundation Extent[1]	220.40	Previous Floodway
3,205	282.09	Previous Floodway	414.78	Previous Floodway
3,185	290.44	Previous Floodway	419.82	Inundation Extent[1]
3,080	354.07	Previous Floodway	502.84	Previous Floodway
2,912	381.93	Previous Floodway	518.70	Main Channel[2]
2,746	423.72	Previous Floodway	521.60	Main Channel[2]
2,557	318.98	Previous Floodway	416.66	Previous Floodway
2,442	291.44	Inundation Extent[1]	391.85	Previous Floodway
2,428	291.47	Inundation Extent[1]	392.03	Inundation Extent[1]
2,361	359.44	Inundation Extent[1]	457.11	Previous Floodway

Table 18 Selected floodway station and determination criteria (continued)

River Station (m)	Left		Right	
	Floodway Station (m)	Floodway Determination Criteria	Floodway Station (m)	Floodway Determination Criteria
2,343	359.69	Inundation Extent[1]	461.94	Inundation Extent[1]
2,199	250.28	Previous Floodway	374.88	Previous Floodway
2,082	169.65	Previous Floodway	302.99	Previous Floodway
1,851	137.74	Inundation Extent	250.44	Previous Floodway
1,605	93.83	Previous Floodway	196.96	Inundation Extent[1]
1,400	35.17	Inundation Extent[3]	142.67	Inundation Extent[3]
1,113	47.16	Inundation Extent[3]	181.60	1 m Depth
812	134.48	1 m Depth	251.79	Inundation Extent[3]
374	218.15	1 m Depth	366.56	Inundation Extent[3]
0	541.46	1 m Depth	659.61	1 m Depth

Notes:

1. The previous floodway is outside the inundation extent.
2. Floodway limit positioned at main channel, as previous floodway limit is inside main channel.
3. No viable flood fringe.

6.3.3 Floodway Criteria Maps

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

Polygons representing areas of depth 1 m or greater and 1 m depth contour lines were derived from the flood depth grid. The depth contours were then filtered and smoothed using the same parameters and procedures as those applied to determine the inundation extents (also described in **Section 5**).

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

The floodway criteria maps provide visual documentation of the results of the floodway determination and depict the limits of the floodway and flood fringes for the design flood. The floodway criteria maps are provided in **Appendix F**. The information documented on the maps include:

The open water floodway criteria maps are provided in **Appendix F**. The information documented on the maps include:

- inundation extents of the 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 location and extent of all cross sections used in the HEC-RAS model; and
- the previous-mapped floodway boundary (where it exists).

6.3.4 Flood Hazard Maps

The flood hazard maps depict the resulting floodway and flood fringe zones for the design flood. The limits of the floodway were delineated by the floodway boundary depicted in the floodway criteria map. Areas of high ground or areas of depth less than 1 m inside the floodway boundaries were included as part of the floodway and the resulting floodway represents a single contiguous polygon.

The extent of the design flood depicted in the floodway criteria map delineates the limits of the flood fringe extending beyond the floodway. Unlike the areas of high ground found within the floodway, high ground or "dry areas" within the flood fringe are not symbolized as being inundated. High hazard flood fringe areas are differentiated with a dotted symbology.

The resulting governing flood hazard maps are provided as **Appendix G**.

Areas in the Floodway

Notable overbank areas in the floodway include:

- A small portion of the Race Track

Areas in the High Hazard Flood Fringe

The high hazard flood fringe includes all inundated areas outside the floodway but within the deeper or faster moving water. Notable inundated areas within the high hazard flood fringe include:

- Race Track

- A small portion of the Spirit of Sangudo Community Park

Areas in the Flood Fringe

The flood fringe includes all inundated areas outside the limits of the floodway and high hazard flood fringe. Notable inundated areas within the flood fringe include:

- Farmhouse southeast of CN Rail Bridge
- Sangudo Riverside Campground
- Spirit of Sangudo Community Park
- Deep Creek Camping and Events

7 POTENTIAL CLIMATE CHANGE IMPACTS

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

7.1 Comparative Scenarios

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

7.2 Results

The results of the analysis for the open water design flood hazard are provided in **Table 19**. **Figure 16** plots a comparison between the computed 100-year flood level profile and profiles computed with discharges that are 10, and 20 percent greater than the 100-year flood discharge.

Table 19 Average increases in water level associated with more severe open water design flood scenarios

Stream	Average Increase in Design Flood Level (m)	
	100-Year Plus 10%	100-Year Plus 20%
Pembina River at Sangudo	0.33	0.67

7.3 Supplementary Information

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

8 CONCLUSIONS

The Sangudo Flood Study was done according to FHIP Guidelines, incorporating technical changes implemented in 2021 regarding how floodways are mapped in Alberta. The objectives of this study were to assess river flood-related hazards along a 6.4 km long reach of the Pembina River within Lac Ste. Anne County, including the hamlet of Sangudo.

The Sangudo Flood Study was divided into five major project components: Survey and Base Data Collection, Open Water Hydrology Assessment, Open Water Hydraulic Modelling, Open Water Flood Inundation Mapping, and Design Flood Hazard Mapping. This report summarizes the work of all five components.

The collection of survey and base data primarily supports the hydraulic modelling and flood mapping. Cross sections were surveyed along the study reach. In total, 43 cross sections were surveyed to complement the LiDAR-derived DTM. In addition, geometric details were collected for three bridges and one abandoned pier.

The primary purpose of the open water hydrology assessment is to develop flood frequency estimates for Pembina River at Sangudo, in support of the hydraulic modelling and flood mapping tasks. The Pembina River at Sangudo is not gauged, and the flood frequency analysis was conducted based on synthetic flow data. The current flood frequency estimates are comparable with previous flood frequency estimates (AENV, 1991).

A numerical model was developed using the HEC-RAS computer program distributed and maintained by the U.S. Army Corps of Engineers Hydraulic Engineering Center. River bathymetry and digital terrain data from the Survey and Base Data Collection component as well as flood frequency estimates from the Open Water Hydrology Assessment component were used to develop, calibrate, and apply the open water hydraulic model. The model was mainly calibrated to the July 1986 (peak discharge 1,666 m^3/s) flood event. Water levels computed by the calibrated model were also compared with the highwater marks from June 1972, June 1980, and August, 1989 flood events and provide reasonable comparison. The calibrated model was used to calculate water surface profiles for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750-, and 1000-year flood frequency return period discharges. All three bridges along the study reach are above the computed 1000-year flood level.

Flood inundation maps were created for all the 13 flood frequency magnitudes and organized together into a single flood inundation map library. Riverside campgrounds, parks, and recreational facilities located on the right bank of the river are started getting inundated by direct inundation at the 20-year flood level. A farmhouse southeast of CN Rail Bridge would be affected by 50-year and larger floods. Few other properties and houses located on the left bank of the river and just downstream of the CN Rail Bridge started getting affected in 200-year and larger floods.

The floodway criteria maps document the open water flood hazard identification criteria and resulting floodway boundaries. These maps depict the rationale supporting the design flood hazard mapping showing the extent of the flood hazard areas (floodway, flood fringe, and high hazard flood fringe). A small portion of the Race Track is the only notable overbank area within the floodway.

9 REFERENCES

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Figures

Alberta

nhc
northwest hydraulic consultants

FLOOD FREQUENCY ESTIMATION LOCATION
STUDY AREA
STUDY REACH

SCALE - 1:15,000
0 75 150 300 450 600 750 M
N

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

Engineer	GIS	Reviewer
MMM	REH	PGV

Job: 1006073 Date: 07-FEB-2022

SANGUDO FLOOD STUDY

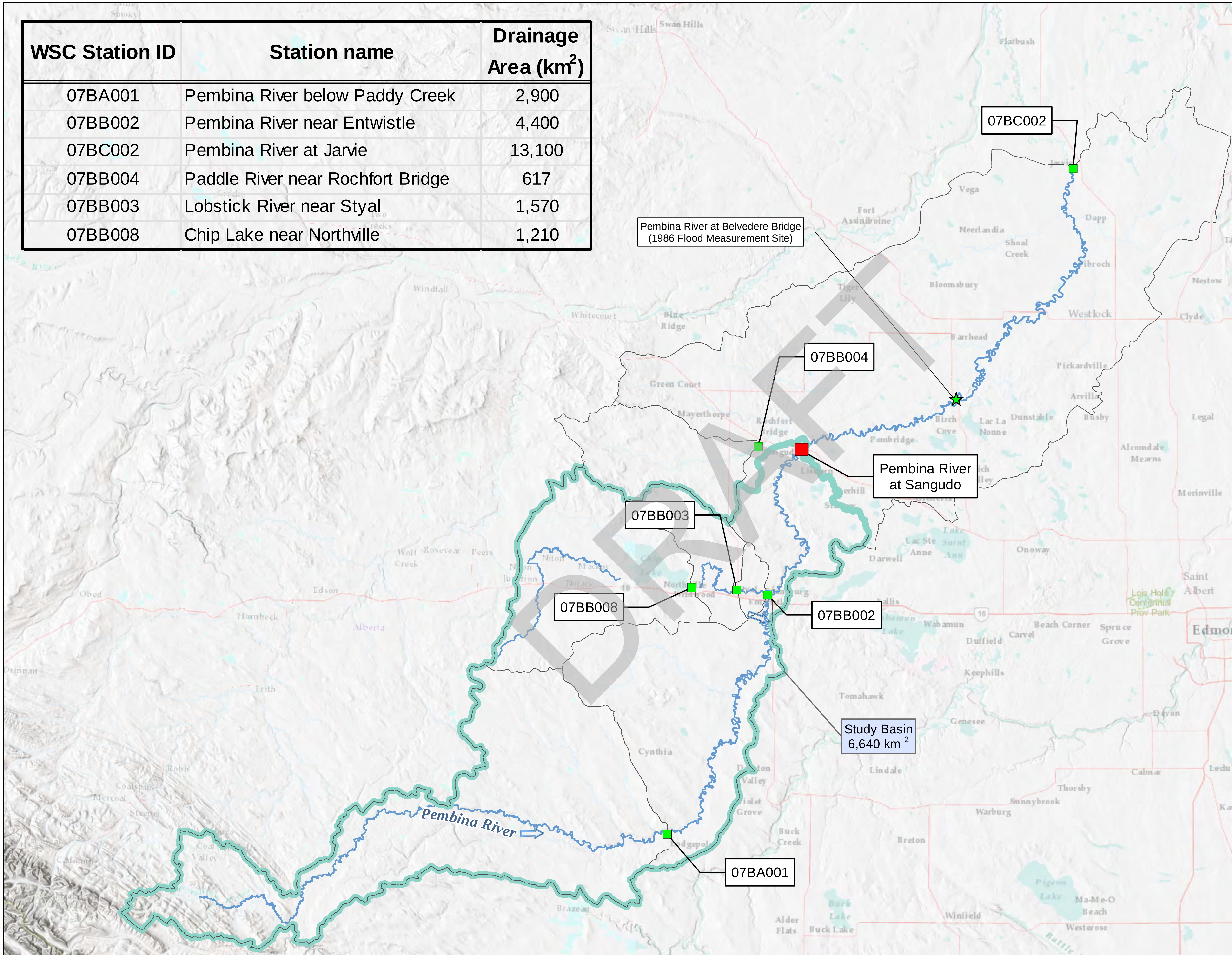
FLOOD HAZARD STUDY AREA

FIGURE 1

REH, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig1_Study_Reach.mxd

Classification: Public

WSC Station ID	Station name	Drainage Area (km^2)
07BA001	Pembina River below Paddy Creek	2,900
07BB002	Pembina River near Entwistle	4,400
07BC002	Pembina River at Jarvie	13,100
07BB004	Paddle River near Rochfort Bridge	617
07BB003	Lobstick River near Styal	1,570
07BB008	Chip Lake near Northville	1,210



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WSC GAUGE STATION (SELECTED)
FLOOD STUDY AREA
SUBBASIN

SCALE - 1:800,000
0 5 10 20 30 KM
N

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

Engineer MMM	GIS REH	Reviewer PGV

Job: 1006073 Date: 08-FEB-2022

SANGUDO FLOOD STUDY

BASIN OVERVIEW

FIGURE 2

REH, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig2_Basin_Overview.mxd

Classification: Public

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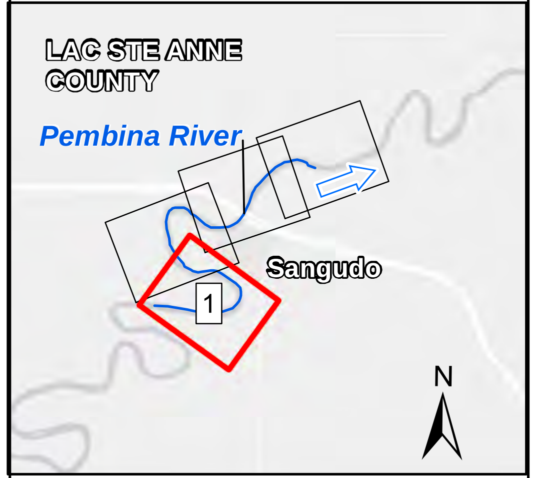


- FLOW DIRECTION
- CONTROL POINT
- SURVEY POINT
- STUDY REACH
- MODEL CROSS SECTION
- BRIDGE
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT

SCALE - 1:5,000
0 100 200 M

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

Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 09-MAR-2022	

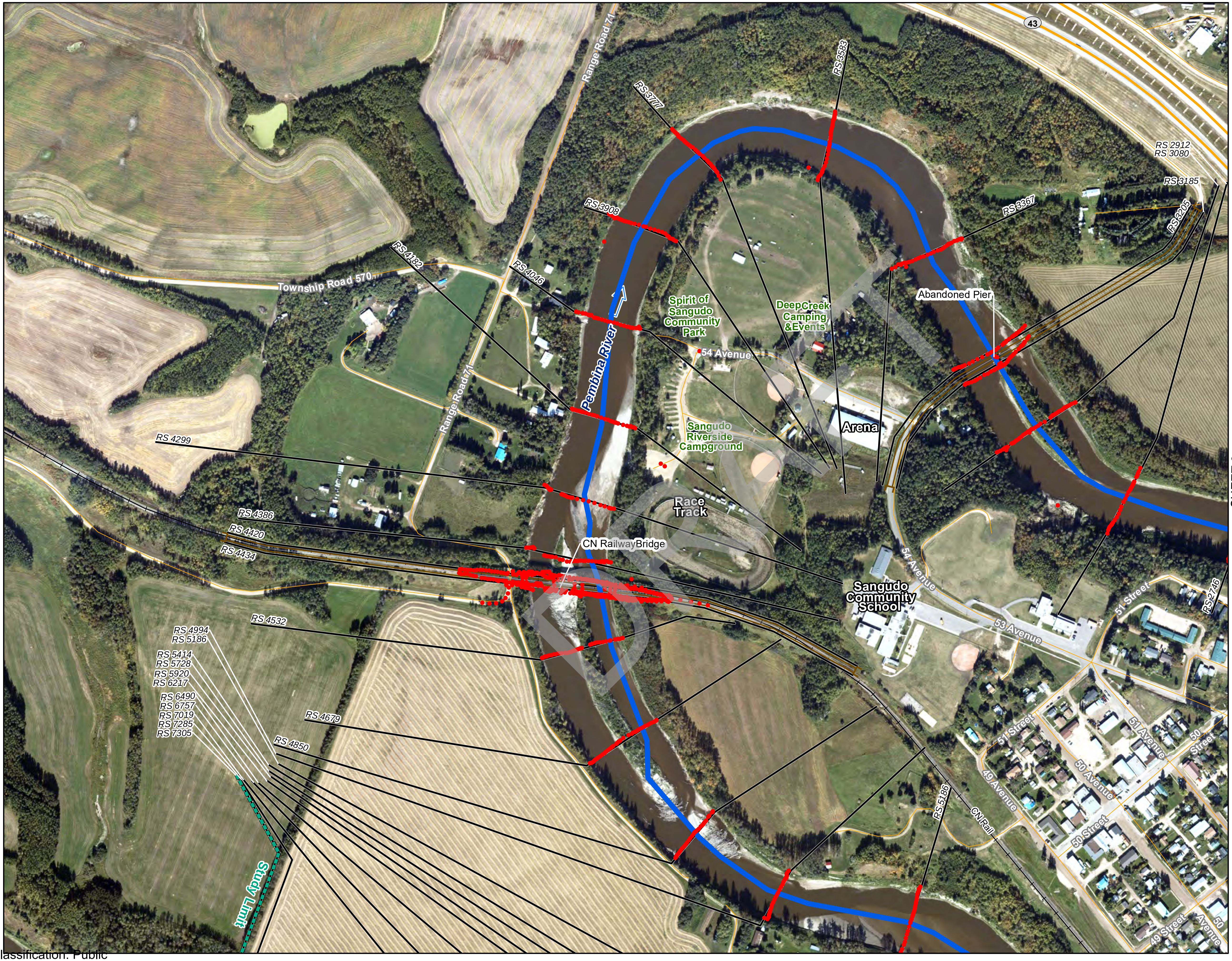
SANGUDO FLOOD STUDY

SURVEY OVERVIEW

SHEET 1 OF 4	FIGURE 3

Classification: Public

J:\Y\P_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig3_SurveyandStructure.mxd



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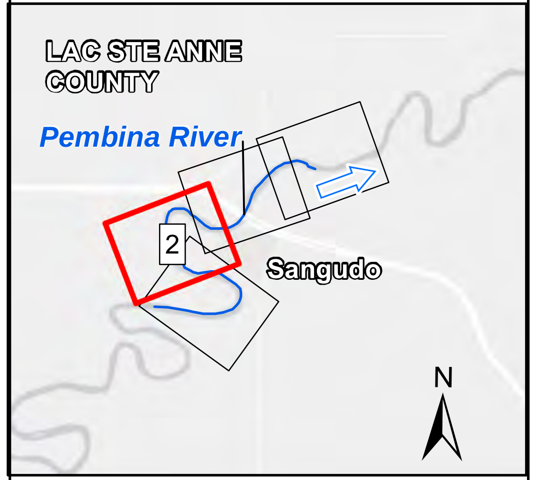


- FLOW DIRECTION
- CONTROL POINT
- SURVEY POINT
- STUDY REACH
- MODEL CROSS SECTION
- BRIDGE
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT

SCALE - 1:5,000

0 100 200 M N

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

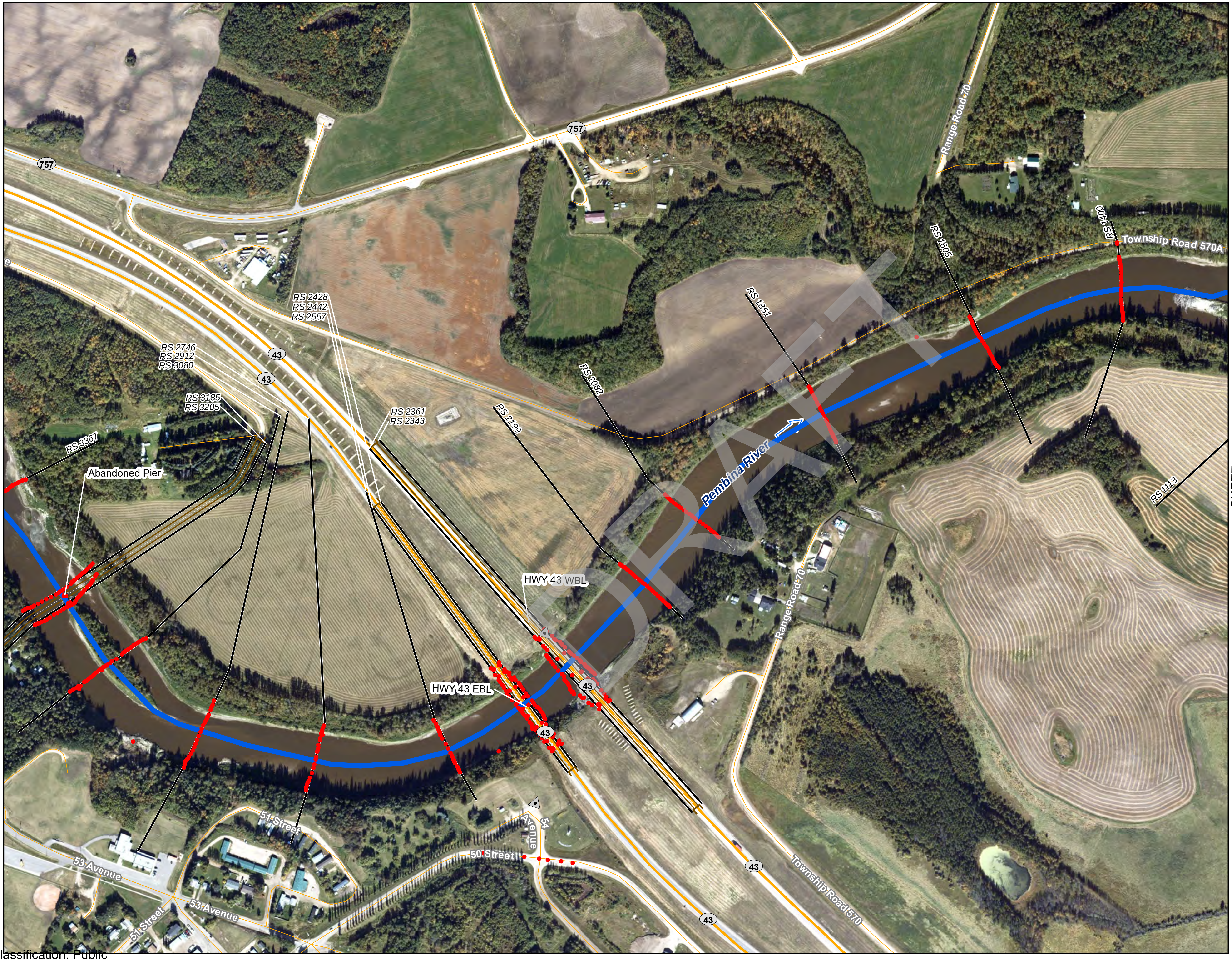
Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 09-MAR-2022	

SANGUDO FLOOD STUDY

SURVEY OVERVIEW

SHEET 2 OF 4 | FIGURE 3

JY P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig3_SurveyandStructure.mxd



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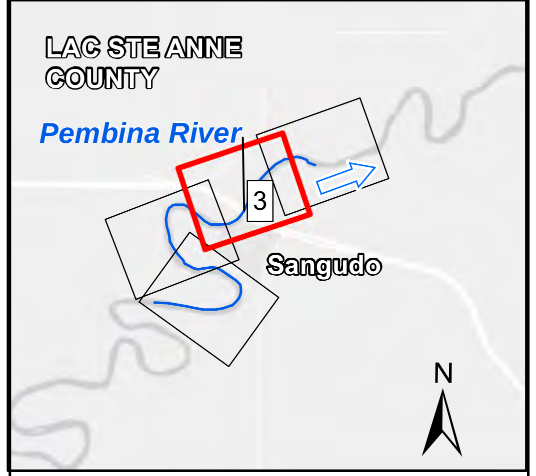


- FLOW DIRECTION
- CONTROL POINT
- SURVEY POINT
- STUDY REACH
- MODEL CROSS SECTION
- BRIDGE
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT

SCALE - 1:5,000
0 100 200 M
N

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

Engineer MMM	GIS JY	Reviewer PGV

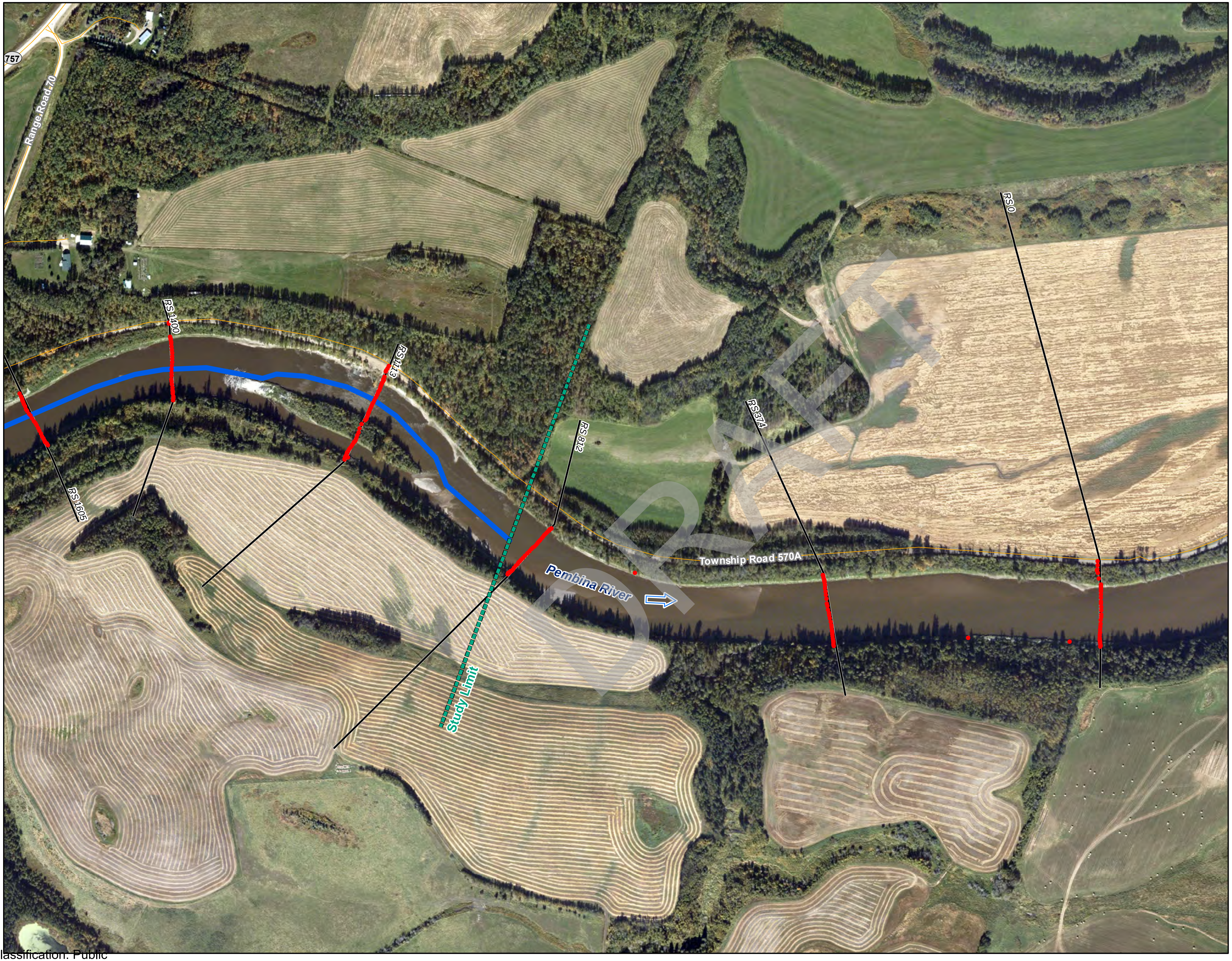
Job: 1006073	Date: 09-MAR-2022

SANGUDO FLOOD STUDY

SURVEY OVERVIEW

SHEET 3 OF 4 | FIGURE 3

JY P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig3_SurveyandStructure.mxd



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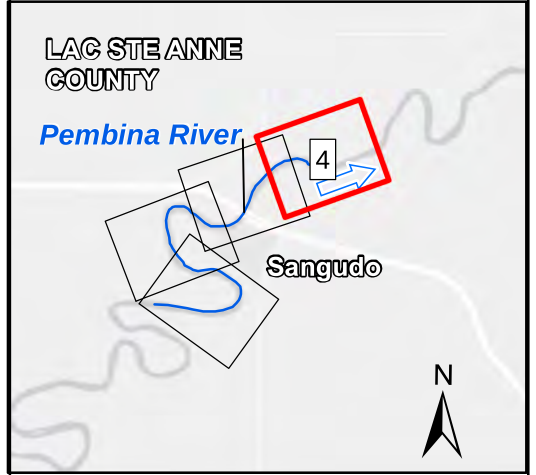

FLOW DIRECTION
CONTROL POINT
SURVEY POINT
STUDY REACH
MODEL CROSS SECTION
BRIDGE
LOCAL ROAD
PROVINCIAL HIGHWAY
RAILWAY
STUDY LIMIT

SCALE - 1:5,000
0 100 200 M
N

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

Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 09-MAR-2022	

SANGUDO FLOOD STUDY

SURVEY OVERVIEW

SHEET 4 OF 4 | FIGURE 3

Classification: Public

JY, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig3_SurveyandStructure.mxd

1980 Flood at CN Railway Bridge near Sangudo

1980 Flood at Highway 43 Bridge near Sangudo

1986 Flood Aerial View at Sangudo

1986 Flood Aerial View at Highway 43 Bridge near Sangudo

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Notes: 1. Flood photographs are obtained from Alberta Environment Highwater Mark Report.

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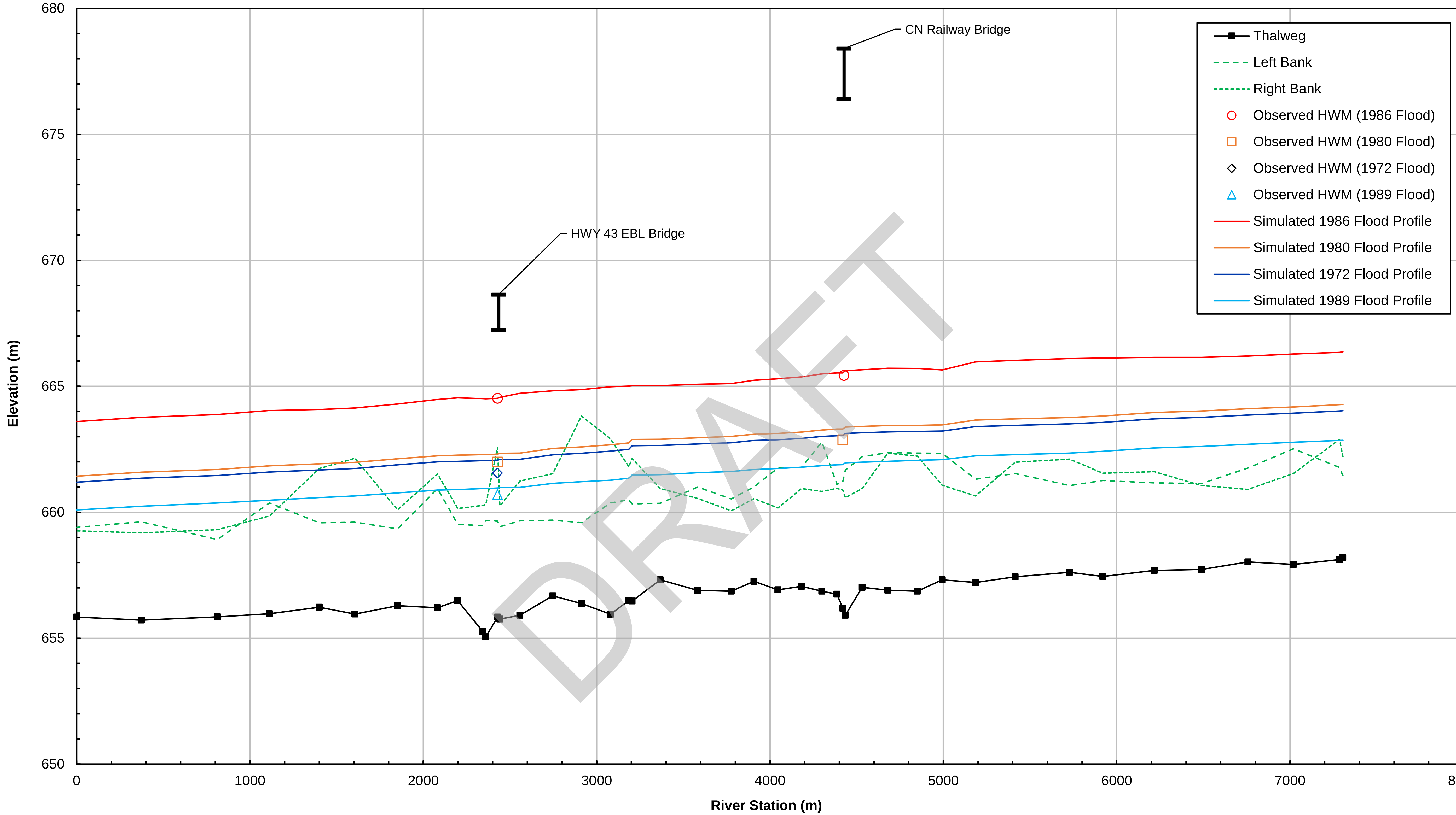
Date: 04-JAN-2022

SANGUDO FLOOD STUDY

OPEN WATER FLOOD PHOTOGRAPHS FOR PEMBINA RIVER AT SANGUDO

FIGURE 4



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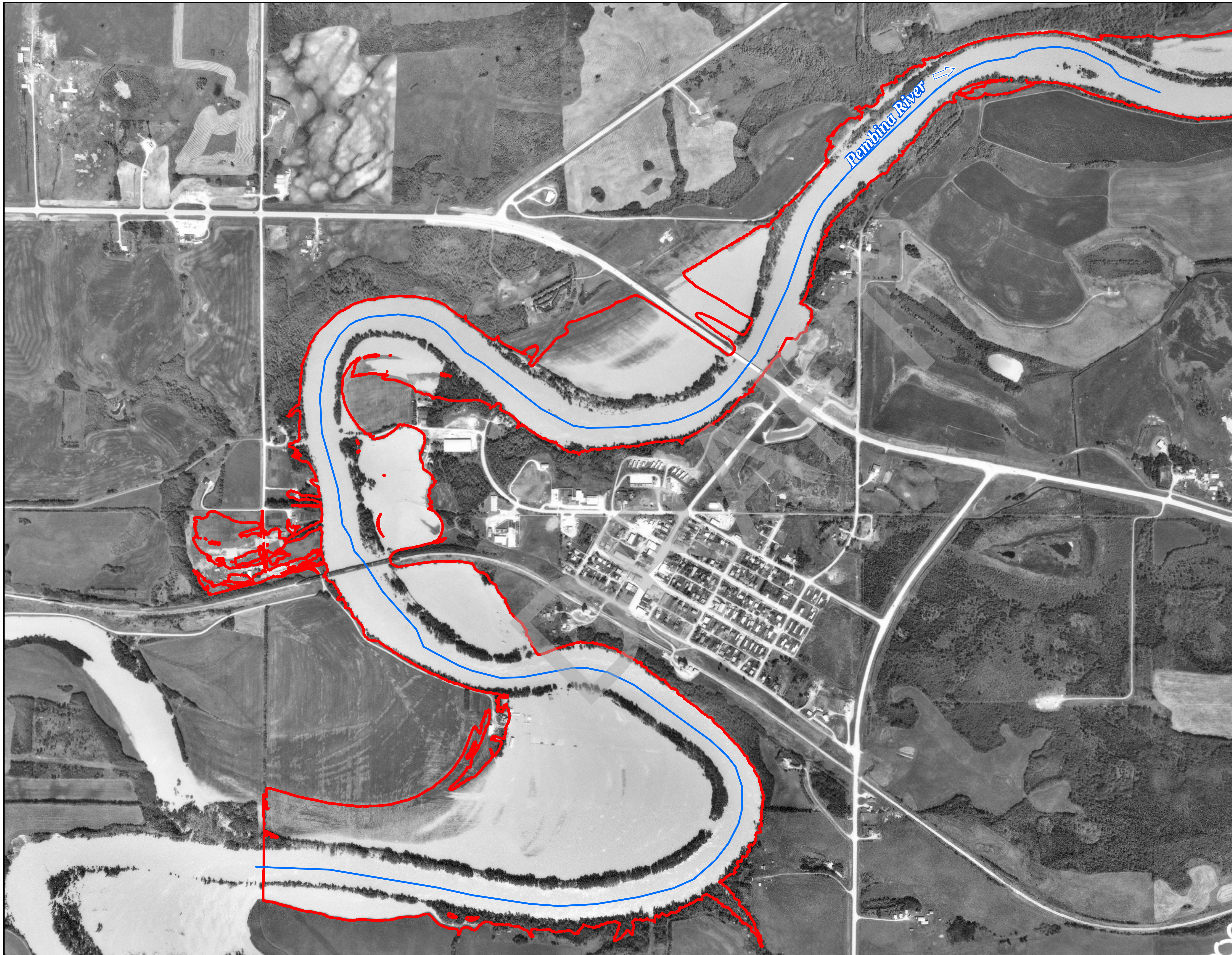
SANGUDO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – HIGH FLOW EVENTS

Job: 01006073 *Date: 04-JAN-2022*

FIGURE 5



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STUDY REACH

SIMULATED FLOOD EXTENT (1986 FLOOD)

Notes:
1. The 1986 simulated flood extent (shown as red line) is from the current model and DEM (2021).
2. The background imagery is from 1986 air photo which collected at the day of 1986 peak flood.

SCALE - 1:10,000
0 50 100 200 300 400 500 M
N

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

Engineer	GIS	Reviewer
MMM	REH	PGV

Job: 1006073	*Date: 07-FEB-2022*

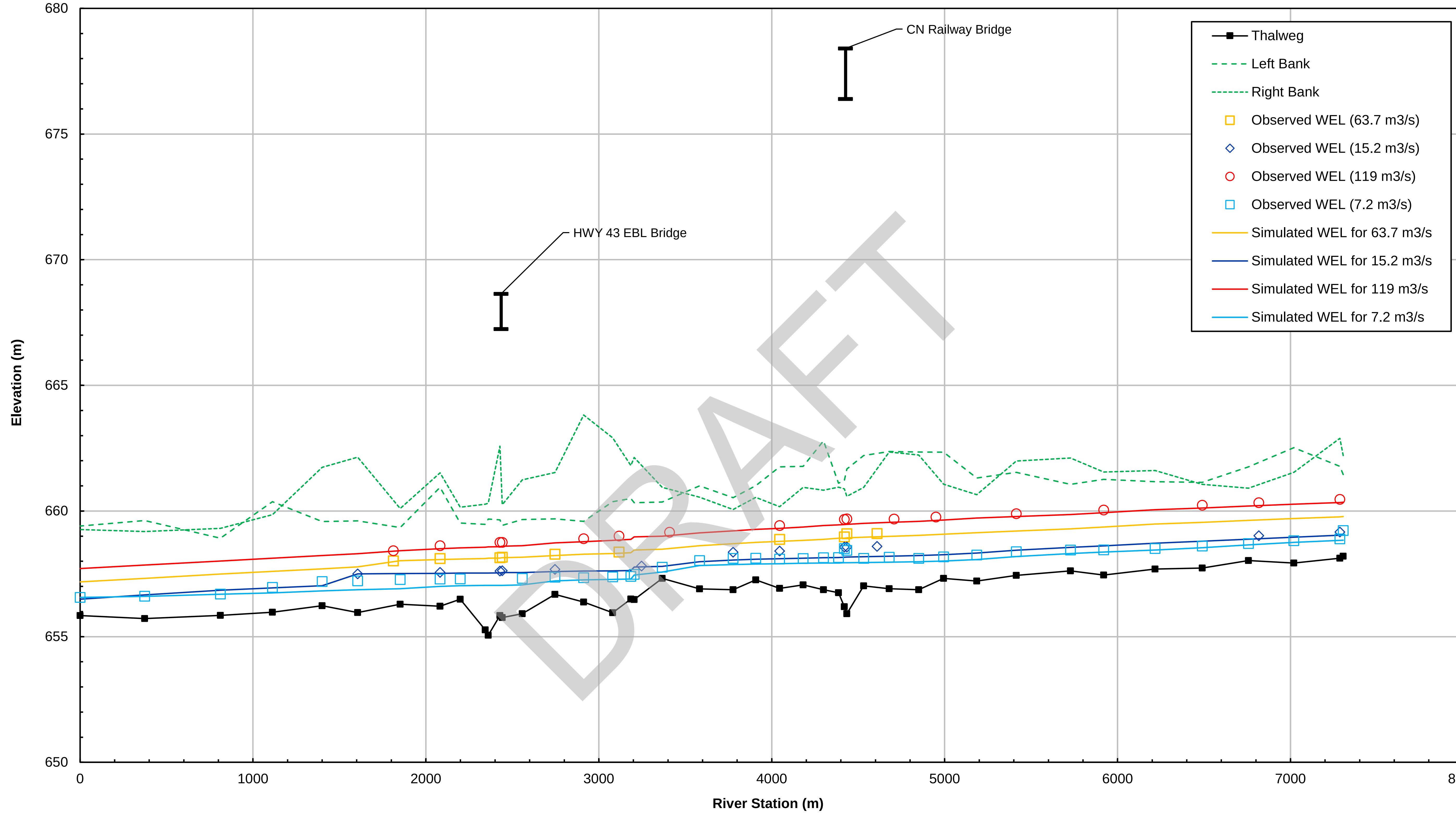
SANGUDO FLOOD STUDY

COMPARISON OF SIMULATED FLOOD EXTENT (1986) VS 1986 FLOOD AIR PHOTO

FIGURE 6

REH, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_SFS_Main_Report_Fig6_1986_Flood_Comparision.mxd

Classification: Public



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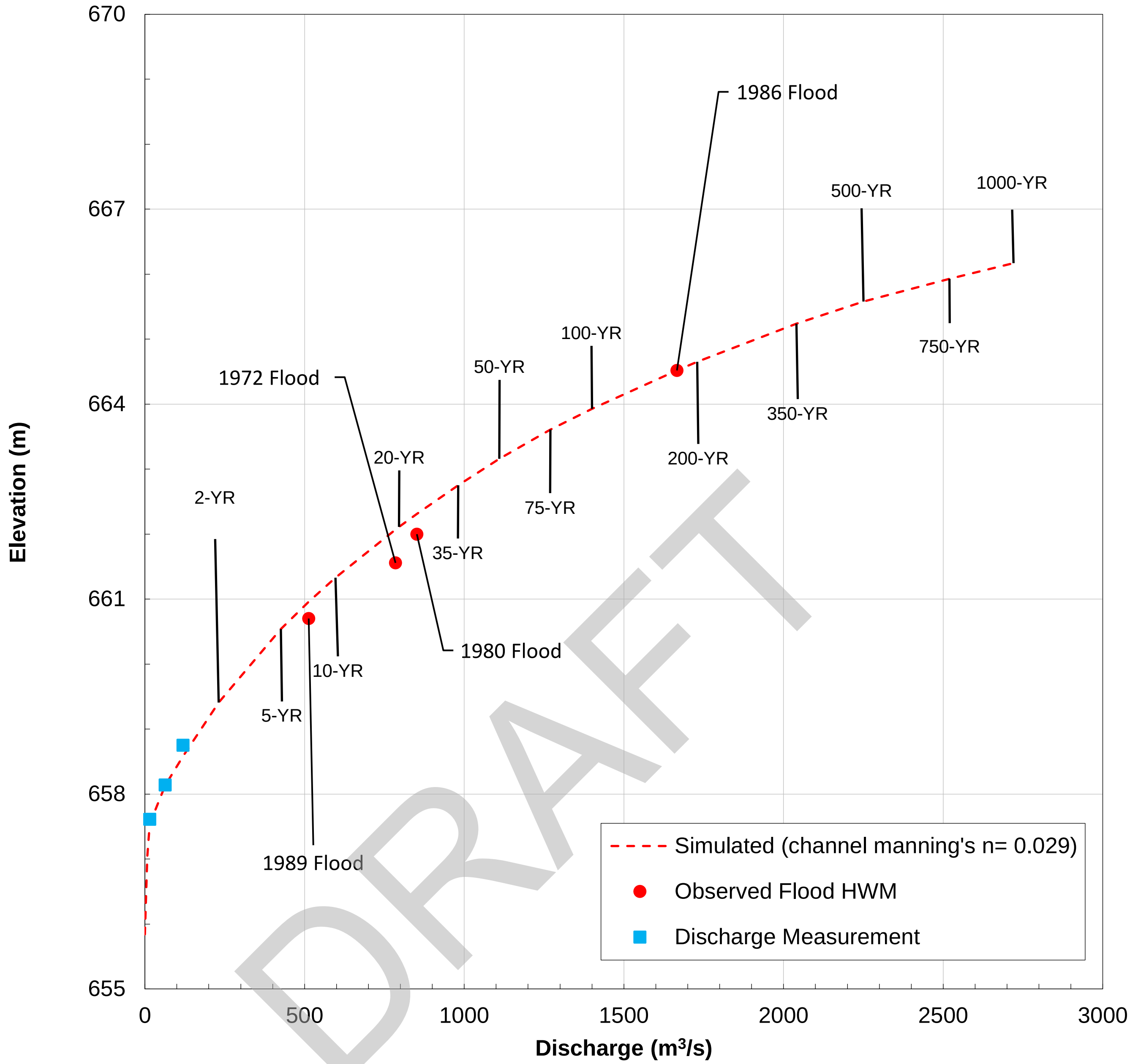
SANGUDO FLOOD STUDY

COMPUTED VS OBSERVED WATER LEVELS – LOW FLOW EVENTS

FIGURE 7



Notes: 1. The location of the 13 frequency floods are marked on the above curve.

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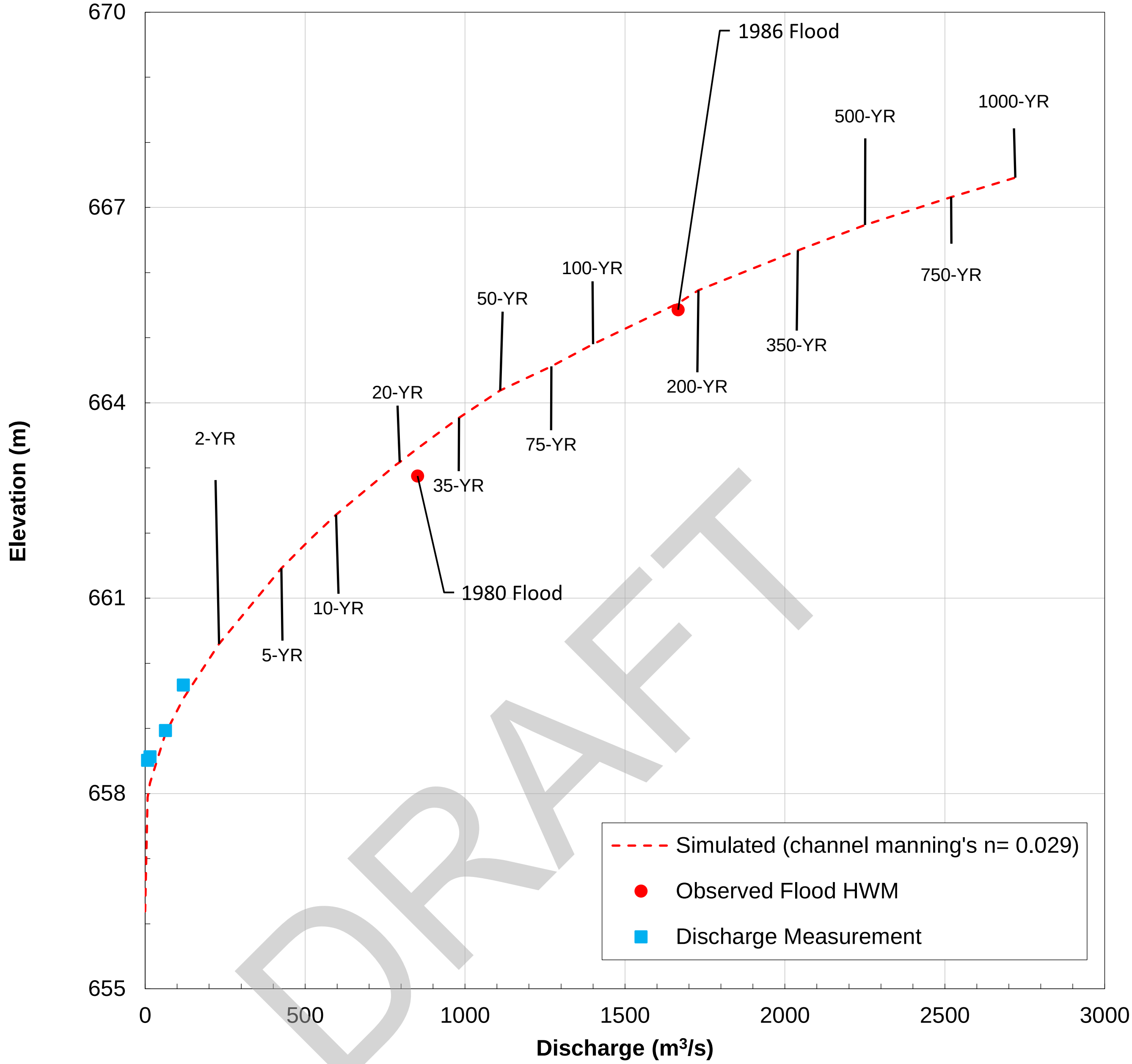
COMPARISON BETWEEN SIMULATED RATING CURVE AND OBSERVED WATER LEVELS (D/S OF HWY 43 EBL BRIDGE)

FIGURE 8



Notes: 1. The location of the 13 frequency floods are marked on the above curve.

SANGUDO FLOOD STUDY

COMPARISON BETWEEN SIMULATED RATING CURVE AND OBSERVED WATER LEVELS (D/S OF CN RAIL BRIDGE)

FIGURE 9

Job: 1006073 *Date: 29-NOV-2021*

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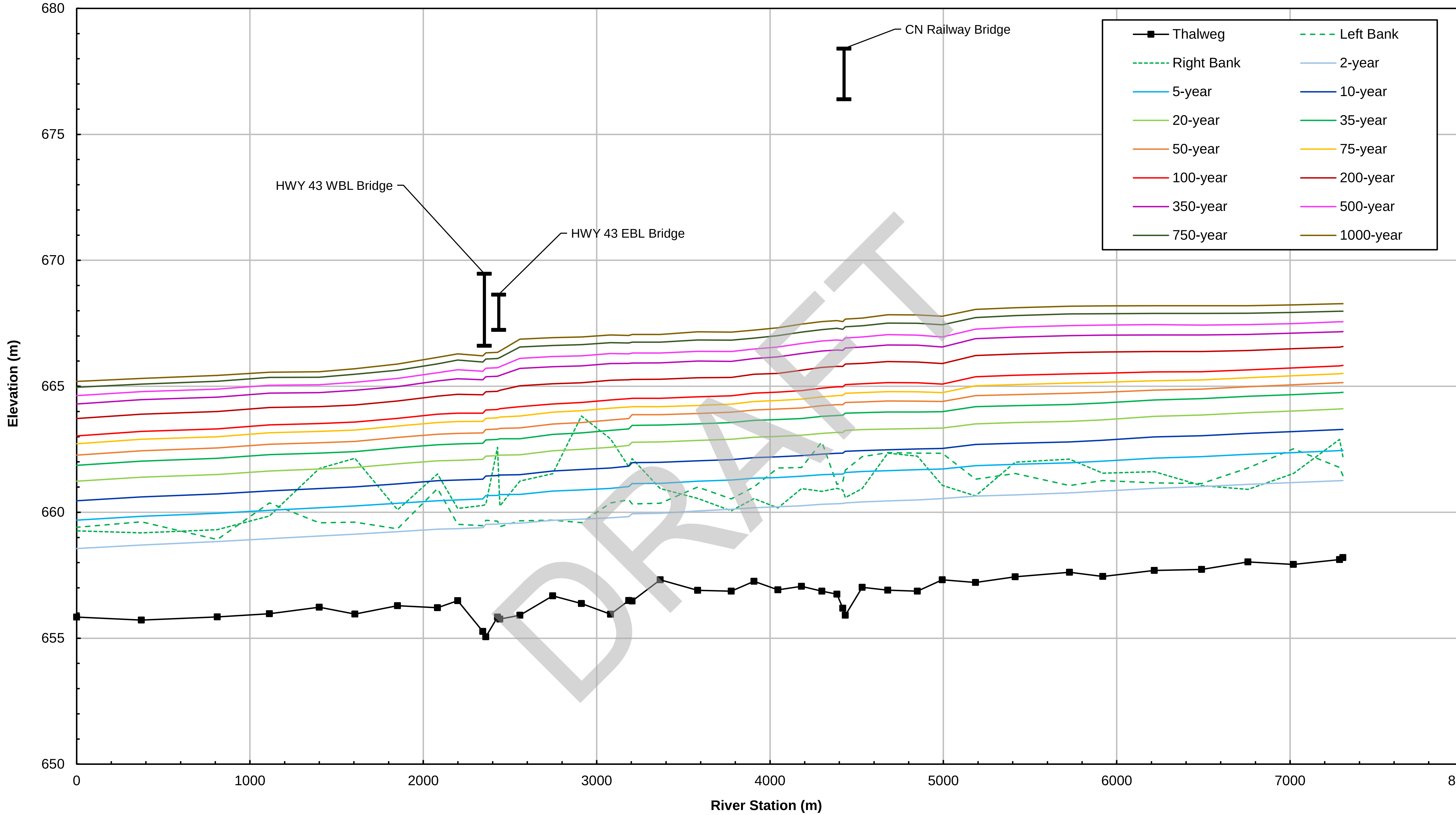

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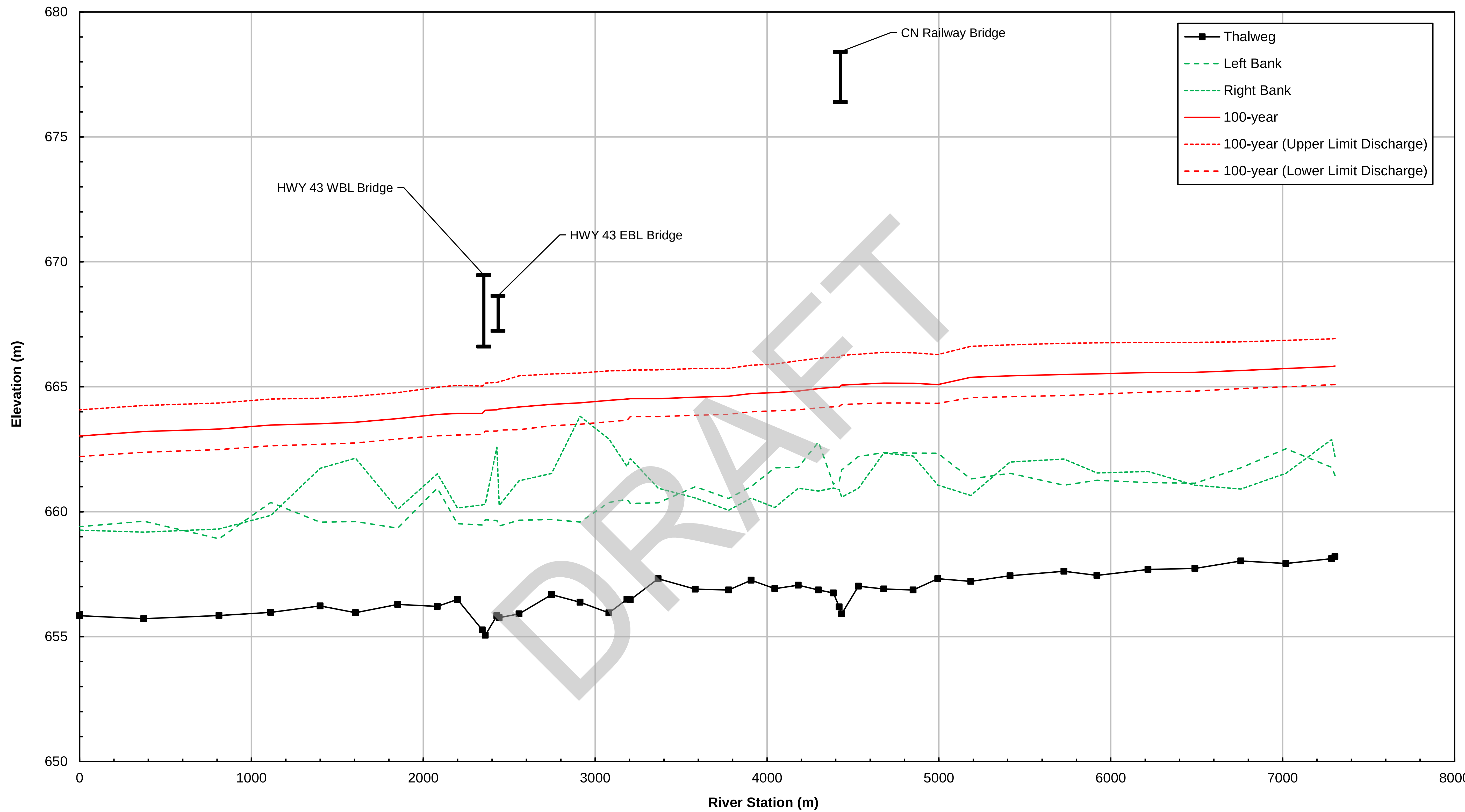
SANGUDO FLOOD STUDY

COMPUTED FLOOD FREQUENCY PROFILES

Job: 01006073 | *Date: 04-JAN-2022*

FIGURE 10



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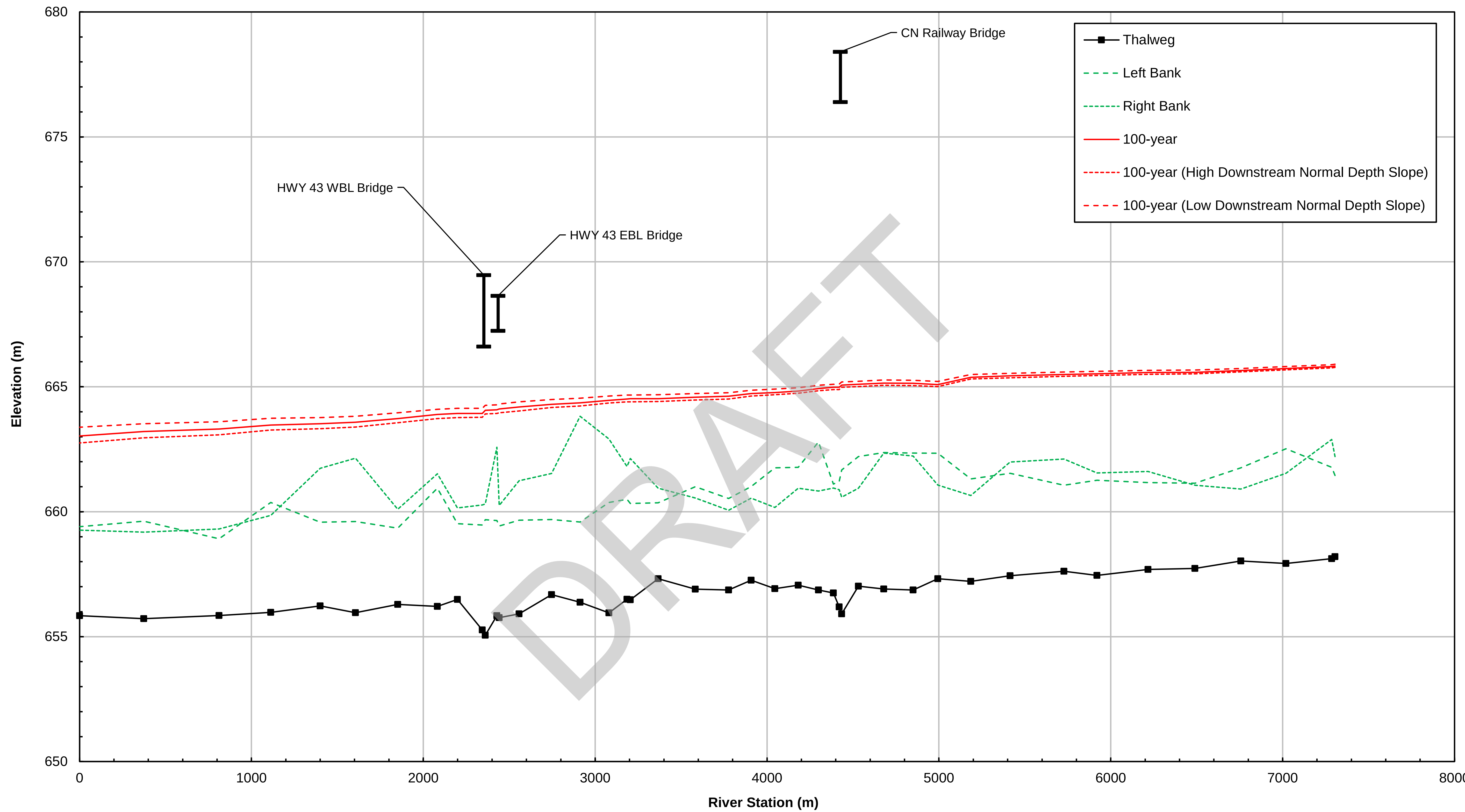
Date: 04-JAN-2022

SANGUDO FLOOD STUDY

SENSITIVITY OF COMPUTED 100-YEAR WATER SURFACE PROFILE TO CHANGES IN THE FLOOD FREQUENCY ESTIMATES

FIGURE 11



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SANGUDO FLOOD STUDY

SENSITIVITY OF COMPUTED 100-YEAR WATER SURFACE PROFILE TO CHANGES IN THE DOWNSTREAM BOUNDARY CONDITION

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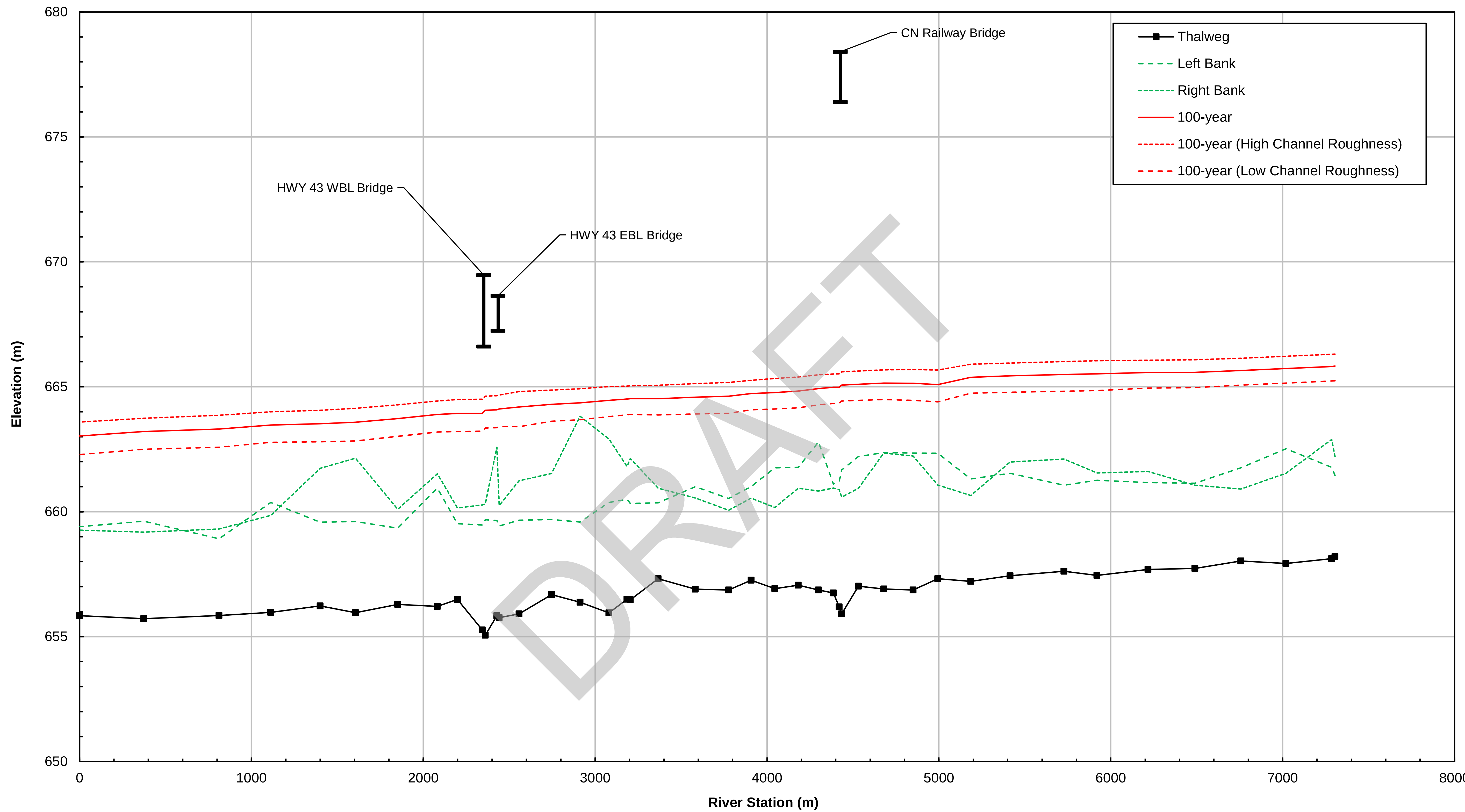
FIGURE 12



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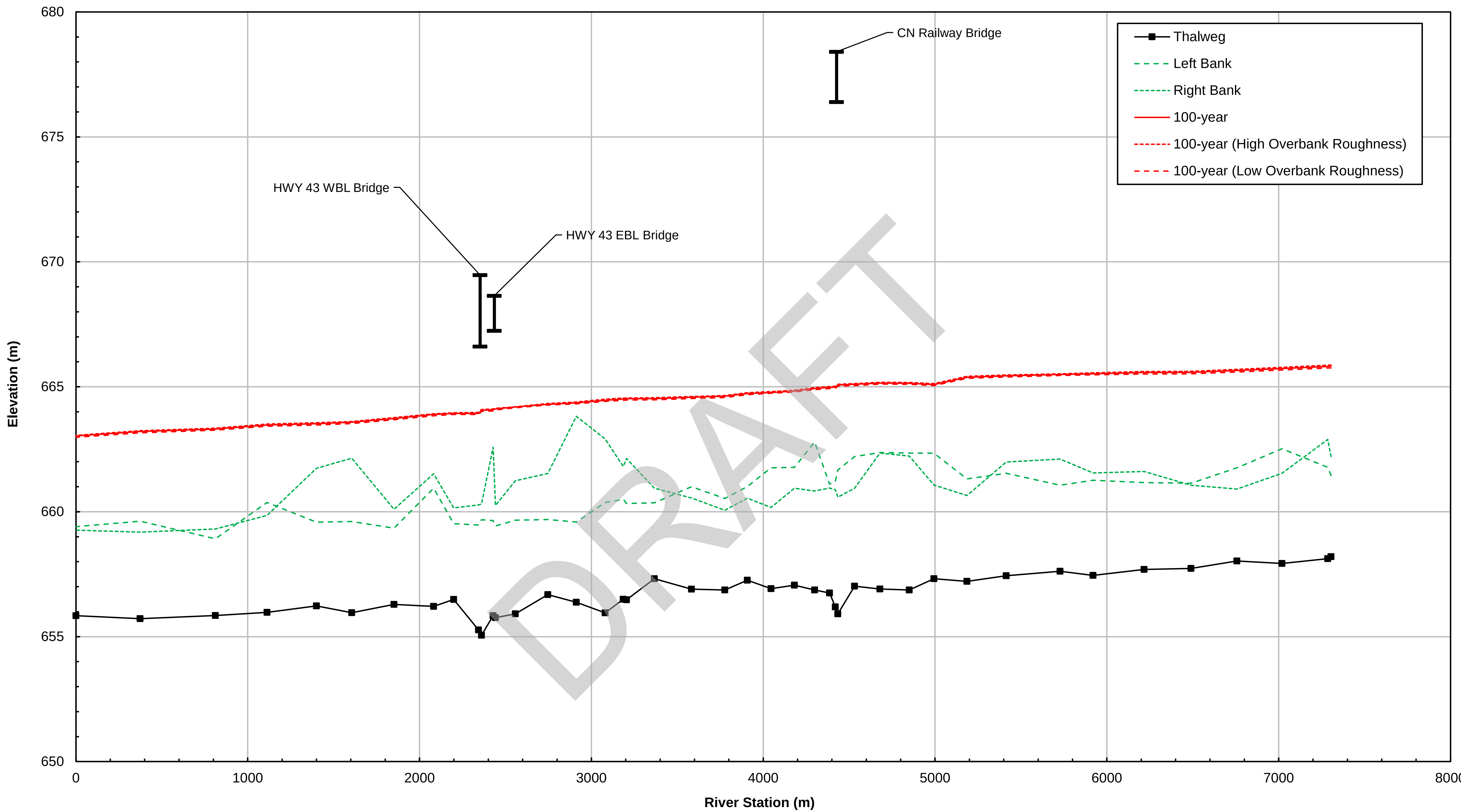
Date: 04-JAN-2022

SANGUDO FLOOD STUDY

SENSITIVITY OF COMPUTED 100-YEAR WATER SURFACE PROFILE TO CHANGES IN THE MAIN CHANNEL ROUGHNESS

FIGURE 13



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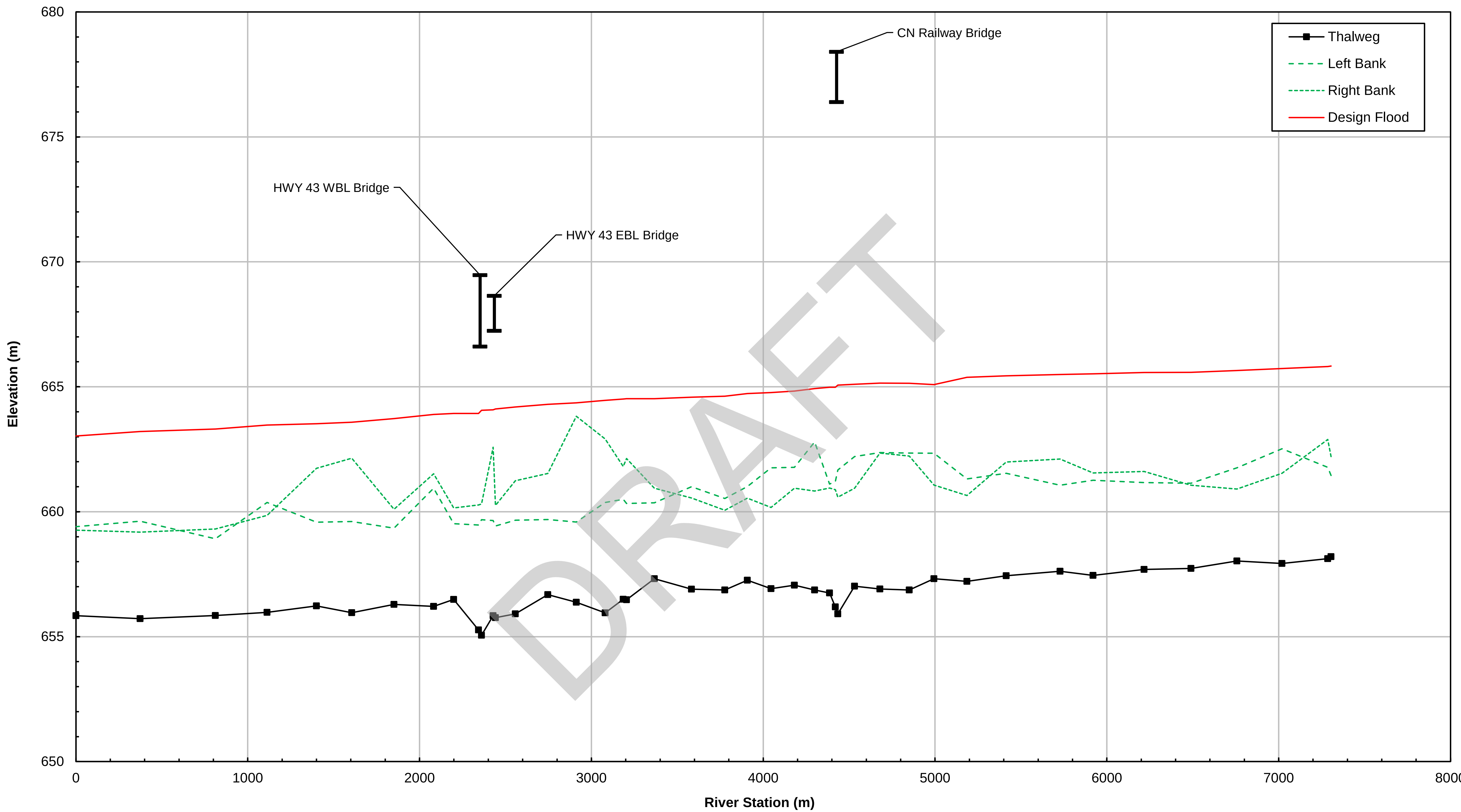
Job: 01006073

Date: 04-JAN-2022

SANGUDO FLOOD STUDY

SENSITIVITY OF COMPUTED 100-YEAR WATER SURFACE PROFILE TO CHANGES IN THE OVERBANK ROUGHNESS

FIGURE 14



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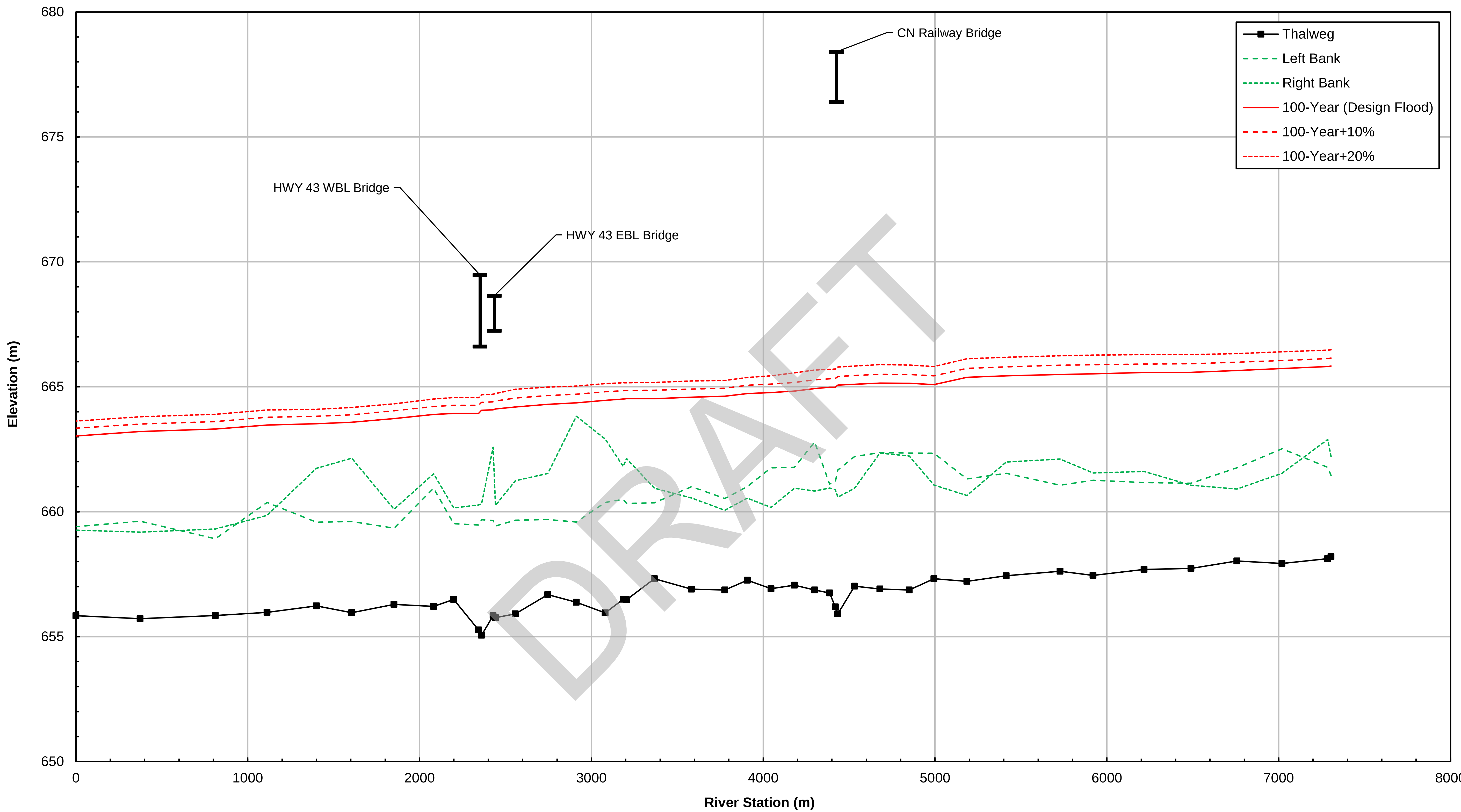
Job: 01006073

Date: 04-JAN-2022

SANGUDO FLOOD STUDY

COMPUTED DESIGN FLOOD PROFILE

FIGURE 15



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SANGUDO FLOOD STUDY

POTENTIAL CLIMATE CHANGE IMPACTS ON DESIGN FLOOD PROFILE

Job: 01006073 *Date: 04-JAN-2022*

FIGURE 16

Appendix A
Hydraulic Structure Details

Bridge Description

Name:	CN Railway Bridge	**Bridge File No.:**	N/A
River:	Pembina River	**River Station (m):**	4,427

Geometry

Span (m):	292.4	**Minimum High Chord (m):**	678.41
Width (m):	5.2	**Minimum Low Chord (m):**	676.40
Pier Type:	Concrete	**No. of Piers:**	4
Pier Shape:	Elongated with Semi Circular Ends	**Pier Width (m):**	Variable (2.6 m – 3.4 m)

Photo(s)

Looking towards the CN Railway Bridge from the right bank

Looking at the upstream side of the bridge from the left bank

Bridge Description

Name:	CN Railway Bridge Pier (Abandoned)	**Bridge File No.:**	BF71082
River:	Pembina River	**River Station (m):**	3,195

Geometry

Span (m):	N/A	**Minimum High Chord (m):**	N/A
Width (m):	N/A	**Minimum Low Chord (m):**	N/A
Pier Type:	Concrete	**No. of Piers:**	1
Pier Shape:	Triangular Nose (90° angle)	**Pier Width (m):**	2.5

Photo(s)

Looking at the pier from the right bank of the river

Looking at the upstream wedge of the pier

Bridge Description

Name:	HWY 43 EBL	**Bridge File No.:**	BF73919
River:	Pembina River	**River Station (m):**	2,435

Geometry

Span (m):	137.2	**Minimum High Chord (m):**	668.64
Width (m):	10.5	**Minimum Low Chord (m):**	667.24
Pier Type:	Concrete	**No. of Piers:**	5
Pier Shape:	Elongated with Semi Circular Ends	**Pier Width (m):**	Variable (0.9 m – 1.5 m)

Photo(s)

Looking towards the bridge (upstream side) from the right bank

Looking across the river from the right bank at the downstream side of the bridge

Bridge Description

Name:	HWY 43 WBL	**Bridge File No.:**	BF78131
River:	Pembina River	**River Station (m):**	2,352

Geometry

Span (m):	133.6	**Minimum High Chord (m):**	669.47
Width (m):	13.6	**Minimum Low Chord (m):**	666.61
Pier Type:	Concrete	**No. of Piers:**	2
Pier Shape:	Elongated with Semi Circular Ends	**Pier Width (m):**	Variable (0.9 m – 1.5 m)

Photo(s)

Looking towards the bridge (upstream side) from the right bank

Looking across the river from the right bank at the downstream side of the bridge

Appendix B
Reach-Representative Photographs

Pembina River

Pembina River (downstream view) from upstream of the study limit west of River Station 7,305 m.

Pembina River (downstream view) near CN Rail Bridge near River Station 4,386 m.

Pembina River (downstream view) near the intersection of Township Road 570 and Range Road 71 near River Station 4,046 m.

Pembina River (downstream view) from Deep Creek Campground and Events near River Station 3,583 m.

Pembina River (downstream view) looking at the CN Rail abandoned pier near River Station 3,367 m.

Pembina River (downstream view) near 54 Avenue near River Station 3,080 m.

Pembina River (downstream view) near 50A Street looking at Highway 43 EBL bridge near River Station 2,746 m.

Pembina River (downstream view) looking from Highway 43 WBL bridge near River Station 2,361 m.

Pembina River (downstream view) east of Range Road 70 near River Station 1,113 m.

Pembina River (downstream view) at the downstream study limit near River Station 812 m.

Pembina River (downstream view) east of the study limit near River Station 374 m.

Appendix C
Open Water Hydrology Assessment Memorandum

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NHC Ref. No. 1006073

MEMORANDUM

Prepared by:	Md Makamum Mahmood	Date:	01 November 2021
Reviewed by:	Gary Van Der Vinne	Client File:	22RSD861
Distribution:	Kurt Morrison (AEP)		

RE: **Sangudo Flood Study**
Open Water Hydrology Assessment

1 INTRODUCTION

In June 2021, Alberta Environment and Parks (AEP) retained Northwest Hydraulic Consultants Ltd. (NHC) to complete a flood study for the Pembina River through a portion of Lac Ste. Anne County, including the Hamlet of Sangudo. The scope of work for this study includes the following major components:

- Survey and Base Data Collection
- Open Water Hydrology Assessment
- Open Water Hydraulic Modelling
- Open Water Flood Inundation Mapping
- Design Flood Hazard Mapping
- Reporting and Documentation

This memorandum presents details of the ***open water hydrology assessment***, for which the primary objective is to develop flood frequency estimates for the Pembina River at the Hamlet of Sangudo, in support of the hydraulic modelling and flood mapping tasks of the Sangudo Flood Study.

2 STUDY AREA

As shown in **Figure 1**, the flood hazard study reach extends along approximately 6.4 km of the Pembina River from the eastern boundary of SE-35-56-7-W5M to the western boundary of NE-6-57-6-W5M. The study reach passes through the Hamlet of Sangudo and the County of Lac Ste. Anne. The study area is located about 100 km northwest of Edmonton.

The Terms of Reference (TOR) for the study does not specify the sites where flood frequency estimates are required. Tributary inflows within the relatively short reach are limited to local overland runoff that would be negligible compared to the incoming Pembina River flows. As such, the open water hydrology assessment of this study provides flood frequency estimates at a single site, namely the Pembina River at Sangudo, located at the Hwy 43 Bridge.

3 HYDROLOGIC CHARACTERISTICS

3.1 Basin Settings

The Pembina River originates from the eastern slopes of the Rocky Mountains and is a tributary of the Athabasca River in central Alberta. It generally flows northeast through Lac Ste. Anne County and then north to the Athabasca River.

As shown in **Figure 2**, Pembina River at Sangudo is located between WSC Station 07BB002 (Pembina River near Entwistle, drainage area 4,400 km^2) and 07BC002 (Pembina River at Jarvie, drainage area 13,100 km^2). The drainage area of the river at Sangudo is approximately 6,640 km^2 (AENV, 1996). The 4,400 km^2 basin area upstream of WSC Station 07BB002 lies mostly in the Foothills Natural Region; however, the downstream portion, about 23% of the basin area, is in the Boreal Forest Natural Region. From WSC Station 07BB002 to Pembina River at Sangudo, the river drainage area increases by 2,240 km^2. About 40% of this incremental area is in the Foothills Natural Region; as such, the Foothills Natural Region is still the dominant region in the basin upstream of Sangudo. From the Pembina River at Sangudo to the farther downstream station 07BC002, the incremental area is entirely in the Boreal Forest Natural Region. Boreal Forest Natural Region is used primarily for agriculture, and the runoff potential is generally lower than for the Foothills Natural Region.

3.2 Flood Characteristics

Pembina River flows at Sangudo are not gauged. However, flood characteristics at Pembina River can be described from WSC streamflow gauging stations existing at several locations both upstream and downstream of Sangudo. The closest gauge station to the study site is located upstream at Pembina River near Entwistle (WSC Station 07BB002).

Annual peak flows on the Pembina River more commonly occur in June-July due to summer rainfall events but have been observed as late as September and as early as April. The three major floods recorded at the Pembina River near Entwistle gauge occurred in 1986, 1980, and 1972. All three floods occurred in June-July and are believed to be due to summer rainstorms.

The 1986 flood is the largest recorded flood in the Pembina River basin. The flood peak instantaneous discharge estimation is available for this event at Pembina River near Entwistle (WSC Station 07BB002). WSC estimated the 1986 flood peak to be 1,250 m^3/s, with daily flows of 1,100 and 1,180 m^3/s on July 19 and July 20. The WSC estimation was based on the high-water mark and was considered low when compared with the peak measurements by Alberta Environment (AENV) for the Pembina River at Belvedere Bridge (AENV, 1991). The 1986 flood peak instantaneous discharge measured for the Pembina River at Belvedere Bridge temporary gauge station was 1,450 m^3/s. AENV (1991) believed that the 200 m^3/s difference between Entwistle and Belvedere suggested by WSC could not be reasonably made up from local contributing areas when the peak-reducing influences of channel routing are added. The Hydrology Branch of AENV estimated the 1986 peak for Pembina River at Entwistle as 1,400 m^3/s based on the Streamflow Synthesis and Reservoir Regulation (SSARR) routing model and an iterative process to match the measured peak discharge of 1,450 m^3/s at Belvedere (AENV, 1991). The 1986 flood peak of 1,400 m^3/s at Pembina River near Entwistle is adopted for the current study.

3.3 Historic Flood Events

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

Systematic continuous flow measurements on the Pembina River began in 1955. The WSC also reported some flow measurements from 1914-1922 at Pembina River near Entwistle (WSC Station 07BB002). It appears that information on historic floods prior to 1914 is not available. Between 1923-1954, it is believed that two major floods occurred (one in 1944 and the second one in 1954) based on miscellaneous high water level records at the Manola Railway Bridge. No recorded peak instantaneous discharge was available close to the study site for those two events. Therefore, historical floods were not considered in this study.

4 DATA COLLECTION

The flood frequency for the Sangudo study site cannot be determined from a single-station analysis. WSC operates streamflow gauging stations upstream (WSC Station 07BB002) and downstream (WSC Station 07BC002) of Sangudo that provide long-term records for the Pembina River but variation of the drainage areas among gauged WSC stations (07BB002 and 07BC002) and the flood study site is significant. Note that tributary areas contributing to the Pembina River between WSC Station 07BB002 and the Sangudo study site consist of the Lobstick River sub-basin (approximately 1,650 km^2) and an ungauged local sub-basin of about 590 km^2 (**Figure 2**).

As described in the next section, NHC has developed the flood frequency estimates for Pembina River at Sangudo based on regional gauge data, including a regional analysis and a flow synthesis approach.

NHC has gathered all hydrometric data, including daily and instantaneous annual peak discharges, water levels, and rating curves (as required) from the WSC and AEP for all relevant streamflow gauging stations summarized in **Figure 2** and **Table 1.** Preliminary data from the recent years (2020) and some missing years were also obtained from WSC. Data from WSC stations listed below were used by NHC as required to assess the regional characteristics of large historical floods, perform regional analyses, perform flow synthesis, or fill data gaps.

Table 1: Salient WSC streamflow gauging stations

WSC Station ID	Station Name	Drainage Area (km^2)	Period of Record
Key Stations for Regional Analysis			
07BA001	Pembina River below Paddy Creek	2,900	1956-1993, 2013-2019, 2020[1]
07BB002	Pembina River near Entwistle	4,400	1914-1922, 1955-2015, 2018-2019, 2020[1]
07BC002	Pembina River at Jarvie	13,100	1957-2020
07AG007	McLeod River near Rosevear	7,140	1984-2014, 2015[1], 2018-2019, 2020[1]
07AG004	McLeod River near Whitecourt	9,110	1968-2016, 2018-2019, 2020[1]
07AG001	McLeod River near Wolf Creek	6,310	1914-1931, 1957-1984
Supplemental Stations for Flow Synthesis			
07BB004	Paddle River near Rochfort Bridge	617	1963-2014, 2015-2017[1], 2018-2019, 2020[1]
07BB008	Chip Lake near Northville	1,210	1972-2009
07BB003	Lobstick River near Styal	1,570	1954-1986

Notes:

1. The preliminary data were obtained from WSC.

5 FLOOD FREQUENCY ANALYSIS

The objective of this task is to provide instantaneous peak discharge estimates for the 2-, 5-, 10-, 20-, 35-, 50-, 75-, 100-, 200-, 350-, 500-, 750- and 1000-year open water floods, for Pembina River at Sangudo. As mentioned above, gauge data do not exist for the Pembina River at Sangudo. Two approaches were considered in this study to develop flood frequency estimates: a regional analysis and a flow synthesis approach. In the end, one set of flood frequency estimates that best suited the study site was recommended and adopted for this study.

5.1 General Frequency Analysis Approach and Tools

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

The USGS "Guidelines for Determining Flood Frequency" Bulletin 17C (USGS, 2018) was also reviewed and considered for the study. The USGS Guidelines provide a framework primarily intended to standardize the methods to account for historic flood information, zero flows or low outliers, and high outliers, and methods to estimate population parameters. They use the LP3 as the base method for flood frequencies with the parameters being estimated from the Expected Moments Algorithm (EMA).

The goodness of fit of each of the distributions, as applied to a flood series, was compared through the Kolmogorov–Smirnov test (K-S test). The K-S test can be used to compare a sample with a reference probability distribution. It quantifies a distance between the empirical probability of the sample and the cumulative distribution function of the reference distribution. The maximum distance (referenced to as D-statistic value, D_n) can be used to describe the goodness of fit, where a smaller D_n value would indicate a better fit between the empirical distribution and the theoretical one.

The goodness of fit was also evaluated with a least square method (Kite, 1977) is based on the sum of squared errors (*SSE*) calculated by:

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

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

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

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

In this study, the applied frequency distributions were ranked first by D_n and *SSE* values separately, and the sums of the rankings were then compared to derive the final combined ranking. Note, however, that using these statistical methods tends not to provide a foolproof assessment of the goodness of fit along the tails of the distributions, which are especially important in defining the return periods of the severe floods. Therefore, the selection of the best representative distribution is based as much on judgement, visual assessment, and Bayesian concepts as it is on the statistical ranking result.

5.2 Regional Frequency Analysis

The regional frequency analysis included the candidate WSC stations summarized in **Table 1**. Their locations are shown in **Figure 2**. These gauge stations were selected considering various factors, including their proximity to the flood study site, basin size, length and period of record, basin land cover and topography, and climate condition.

Several combinations of different WSC stations were used for the regional analysis. The first attempt of regional analysis was made considering three stations on the Pembina River (WSC Station 07BA001, 07BB002, and 07BC002). The Pembina River basin upstream of Sangudo lies mostly in the Foothills Natural Region and includes the Pembina River basin below Paddy Creek (WSC Station 07BA001) and near Entwistle (WSC Station 07BB002); which both represent similar physiographic settings to the study basin area. However, from the study site to the farther downstream at Jarvie (WSC Station 07BC002), the drainage area of the Pembina River is more than doubled, with the incremental area entirely in the Boreal Forest Natural Region, which has a lower runoff potential. A comparison of Pembina River peak instantaneous flow data near Entwistle (WSC Station 07BB002, drainage area 4,400 km^2) and at Jarvie (WSC Station 07BC002, drainage area 13,100 km^2) between 1959 and 2019 was shown in **Figure 3**. The comparison suggests that the flood peak discharge at the downstream station (at Jarvie) tends to be smaller than at the upstream station when upstream flood peak discharges are higher than the 5-year flood as estimated by AENV (1991). Also, as described in the previous hydrology assessment (AENV, 1991), major floods at Sangudo are most likely to occur due to upper basin floods (where the flood waters are generated from mountain and foothills), while the major floods at Jarvie (WSC Station 07BC002) occurred due to total basin floods (where the flood waters are more uniformly generated throughout the basin). Therefore, it can be concluded that the hydrological characteristics of the Pembina River at Jarvie (WSC Station 07BC002) are significantly different from the study basin.

If a regional analysis was conducted with the Pembina River stations only, it would significantly underestimate flood frequency discharges for Pembina River at Sangudo, as the drainage basin upstream of the Pembina River gauge at Jarvie (WSC Station 07BC002) is double the size of the Pembina River at Sangudo but has lower peak instantaneous discharges for large floods compared with other two gauged basins at the Pembina River (as shown in **Figure 4**). These lower discharges are likely a result of attenuation of the flood waves due to floodplain storage downstream of Sangudo.

Another attempt of regional analysis includes two regional stations on the McLeod River (WSC Station 07AG007 near Rosevear and WSC Station 07AG004 near Whitecourt) and two Pembina River gauges (WSC Station 07BA001 below Paddy Creek and 07BB002 near Entwistle). As shown in **Figure 2**, the McLeod River basin lies parallel to the Pembina River basin and also drains to the Athabasca River. Its physiographic setting is similar to the Pembina River basin upstream of the study site. The drainage areas of the McLeod River stations are somewhat larger than that of the Pembina River at Sangudo; however, the unit flood discharges from the McLeod River stations are higher than the unit flood discharges from the Pembina River near Entwistle. This is contrary to the expectation that the additional drainage area for the Pembina River between Sangudo and Entwistle would have lower unit flood discharges because the area consists about 60% in the Boreal Forest Natural Region, which has a lower runoff potential. The additional drainage area between Sangudo and Entwistle also has a relatively large lake, Chip Lake (drainage area 1,210 km^2), which accounts for more than 50% of the total tributary area between Entwistle and Sangudo, which would tend to reduce flood peaks from the area. The use of

McLeod River stations in the regional analysis tends to increase the estimated flood frequency discharges at Sangudo, which is unreasonable.

This regional analysis is also limited by different data record lengths at different stations. For example, the Pembina River below Paddy Creek (WSC Station 07BA001) has 43 years of data, while the Pembina River near Entwistle (WSC Station 07BB002) has 73 years of data. The different years of record plot the same flood event (for example, 1986 flood event) in different return periods for these two stations resulting in a relatively steep flood frequency curve for WSC Station 07BA001, compared with the flood frequency curve for WSC Station 07BB002 (**Figure 5**).

Based on the above discussion, regional flood frequency analysis is not recommended for establishing flood frequency estimates for the Pembina River at Sangudo for use in the current study. As such, the detailed regional analysis assessment and results are not provided in this report.

5.3 Frequency Analysis based on Synthetic Flow Data

This analysis is to provide flood frequency estimates for Pembina River at Sangudo using a process-based flow synthesis approach to estimate discharges at Sangudo based on available streamflow data.

Data Series Preparation

As shown in **Figure 2**, tributary areas contributing to the Pembina River between WSC Station 07BB002 near Entwistle and the Sangudo study site consist of the Lobstick River basin at the mouth (approximately 1,650 km^2) and an ungauged local sub-basin of about 590 km^2.

The Lobstick River carries outflows from Chip Lake (drainage area 1,210 km^2) and runoff from a 440 km^2 local area. It drains to the Pembina River immediately downstream of WSC Station 07BB002 (drainage area 4,400 km^2). The Lobstick River flows were gauged at WSC Station 07BB003 (drainage area 1,570 km^2) from 1954 to 1986. These daily flows were prorated by drainage area ratio to obtain the daily flows from the Lobstick River basin at the mouth (approximately 1,650 km^2). Prorating by drainage area ratio is sufficient as the change in drainage area between the Lobstick River station and Lobstick River at the mouth is only about 5%.

Contributions from local tributary inflows in the 590 km^2 local sub-basin downstream of the Lobstick River were estimated by prorating Paddle River naturalized daily flows for WSC Station 07BB004 (1963-2017) by the drainage area ratio. As shown in **Figure 2,** the Paddle River basin above WSC Station 07BB004 (617 km^2) lies immediately north of the Lobstick River basin and near Sangudo. Its physiographic and hydro-climatic conditions are highly similar to the sub-basins contributing to the Pembina River between Entwistle and Sangudo (including the Lobstick River sub-basin). Therefore, it is a good analog for these sub-basins contributing to the Pembina River. As shown in **Table 1**, WSC Station 07BB004 on the Paddle River provides flow data from 1963 to the present, including both pre- and post-regulation periods. The river has been regulated since 1985 due to the Paddle River Dam. The proposed analysis requires natural/naturalized flows. NHC completed the Paddle River Dam flood handling options study for AEP at 2019 (NHC, 2019) and had developed naturalized Paddle River flows at WSC Station 07BB004 for the post-regulation period (1986-2017), which was used directly in the above-described analysis.

The sums of the estimated local tributary inflows, gauged Lobstick River flows, and Pembina River flows near Entwistle (WSC Station 07BB002) were taken as the estimates for flows at Sangudo for the 1963-1986 period. Note that the flow synthesis approach ignored the travel time in the Pembina River from Entwistle (WSC Station 07BB002) to Sangudo. The river channel length between these two sites is 75 km, so the flood flow travel times would be shorter than 12 hours. Therefore, the assumption that the flows occurred on the same day is reasonable for the daily flow analysis.

The proposed flow synthesis is limited to the 1963-1986 period due to Lobstick River and Paddle River flow data availability. Note that though the flow synthesis approach is only limited to 24 years (1963-1986), it covers the six largest flood events in the Pembina River during the whole systematic record period. These estimated daily discharges for Sangudo were compared with the gauged daily discharges for Pembina River near Entwistle (WSC Station 07BB002) from 1963 to 1986. The best-fit relationship, along with upper and lower bands, is shown in **Figure 6**. This relationship was used to transfer the daily discharge record from Entwistle (WSC Station 07BB002) to the Sangudo site for the periods 1914-1922, 1955-1962, 1987-2015, and 2018-2020, increasing the synthetic record length to 73 years.

A combined peak annual daily discharge series for Pembina River at Sangudo was established for the period of 1914-1922, 1955-2015, and 2018-2020 based on the developed daily data series as described above. The instantaneous peaks at Sangudo were then calculated based on the correlation between the instantaneous peak and daily discharges at Pembina River near Entwistle (WSC Station 07BB002) for years when both were measured. This relationship is shown in **Figure 7**. For the 1986 flood event, the annual instantaneous peak discharge for Pembina River at Sangudo was estimated from the adopted annual instantaneous peak discharge of 1,400 m^3/s for Pembina River near Entwistle by Alberta Environment. AENV (1991) believed that the estimated 1986 flood peak near Entwistle by WSC is low and a flood peak of 1,400 m^3/s is more reasonable and is used in the current study, assuming the ratio establish in **Figure 6** is applicable to instantaneous peak flows as well as daily flows. **Table 2** shows the annual peak flow series for the Pembina River at Sangudo.

Table 2: Annual peak instantaneous and daily discharges for Pembina River at Sangudo

Year	Annual Peak Instantaneous Discharge (m^3/s)	Date	Annual Peak Daily Discharge (m^3/s)	Date
1914	**641.4**		583.1	09-Jun
1915	**600.8**		546.2	29-Jun
1916	**269.7**		245.1	04-Jul
1917	**496.1**		451.0	19-May
1918	**96.3**		87.6	17-Apr
1919	**184.6**		167.8	06-Sep
1920	**407.1**		370.1	09-May
1921	**144.0**		130.9	17-May
1922	**102.0**		92.7	18-May
1955	**188.5**		171.4	24-May
1956	**97.5**		88.7	23-Apr
1957	**150.5**		136.9	02-May
1958	**264.4**		240.4	30-Jun
1959	**161.0**		146.4	19-Jun

Year	Annual Peak Instantaneous Discharge (m^3/s)	Date	Annual Peak Daily Discharge (m^3/s)	Date
1960	**199.0**		180.9	03-Jul
1961	**128.9**		117.2	31-Jul
1962	**106.8**		97.1	08-Aug
1963	**225.5**		*205.0*	*01-May*
1964	**280.7**		*255.2*	*08-May*
1965	**759.6**		*690.6*	*29-Jun*
1966	**291.6**		*265.1*	*07-Aug*
1967	**102.4**		*93.1*	*12-Jun*
1968	**72.7**		*66.1*	*08-Aug*
1969	**504.4**		*458.6*	*08-Aug*
1970	**195.6**		*177.8*	*18-Jun*
1971	**616.9**		*560.8*	*12-Jul*
1972	**785.2**		*713.8*	*27-Jun*
1973	**129.2**		*117.4*	*08-May*
1974	**316.5**		*287.8*	*19-Apr*
1975	**186.3**		*169.4*	*30-Jun*
1976	**69.5**		*63.2*	*15-Apr*
1977	**426.3**		*387.5*	*30-May*
1978	**238.9**		*217.2*	*12-Jul*
1979	**129.3**		*117.6*	*24-Apr*
1980	**852.4**		*775.0*	*06-Jun*
1981	**143.4**		*130.4*	*02-Jun*
1982	**674.5**		*613.2*	*06-Jul*
1983	**192.8**		*175.3*	*09-Jul*
1984	**98.6**		*89.6*	*19-May*
1985	**187.1**		*170.1*	*15-Sep*
1986	**1666.0**		*1286.6*	*20-Jul*
1987	**179.3**		163.0	04-Aug
1988	**128.4**		116.7	08-Jul
1989	**513.1**		466.5	05-Aug
1990	**553.7**		503.4	08-Jul
1991	**276.2**		251.1	08-Jul
1992	**55.4**		50.3	01-Jun
1993	**101.7**		92.5	26-Jun
1994	**192.4**		174.9	06-Jul
1995	**441.1**		401.0	10-Aug
1996	**263.1**		239.2	02-Jun
1997	**339.0**		308.2	24-Jun
1998	**227.8**		207.1	06-Jul
1999	**327.3**		297.5	09-Jul
2000	**231.7**		210.6	12-Jul
2001	**535.4**		486.7	30-Jul

Year	Annual Peak Instantaneous Discharge (m^3/s)	Date	Annual Peak Daily Discharge (m^3/s)	Date
2002	**136.1**		123.8	18-May
2003	**140.1**		127.3	27-Apr
2004	**226.5**		205.9	12-Jul
2005	**253.9**		230.9	20-Jun
2006	**94.6**		86.0	18-Jun
2007	**335.1**		304.6	06-May
2008	**221.2**		201.1	09-Jun
2009	**205.5**		186.8	11-Jul
2010	**187.2**		170.2	12-Jun
2011	**674.1**		612.9	19-Jun
2012	**253.9**		230.9	11-Jun
2013	**259.2**		235.6	27-May
2014	**184.6**		167.8	25-Apr
2015	**69.1**		62.8	02-May
2018	**308.9**		280.8	06-Jul
2019	**501.3**		455.8	23-Jun
2020	**628.3**		571.2	03-Jul

Notes:

1. No peak instantaneous and peak daily discharge was available for 1923-1954. WSC Station 07BB002 was discontinued from March, 1923-October, 1954.
2. No peak instantaneous and daily discharge was available from WSC Station 07BB002 for the period 2016-2017.
3. For 1963-1986 the annual peak daily discharge (values shown in italic) at Sangudo site were obtained from the flow synthesis approach.
4. For the period of 1914-1922, 1955-1962, 1987-2015 and 2018-2020 the annual peak daily discharges are obtained from Pembina River near Entwistle (WSC Station 07BB002) peak daily flow data based on the relationship $Q_{sangudo}=1.19Q_{Entwistle}$ established in **Figure 6**.
5. The bolded peak instantaneous values are based on the relationship $Q_i=1.10Q_d$ established in **Figure 7** from Pembina River near Entwistle (WSC 07BB002) data.
6. The peak instantaneous discharge at Sangudo for 1986 event (bolded and underlined) was obtained from adopted peak instantaneous discharge of 1,400 m^3/s at Entwistle (AENV, 1991) and based on the relationship $Q_{sangudo}=1.19Q_{Entwistle}$ established in **Figure 6.**

Single Station Frequency Analysis

A single-station frequency analysis was performed on the Pembina River peak instantaneous discharges at Sangudo, as shown in **Table 2**. **Table 3** provides a summary of the statistical parameters for the Pembina River flow series at Sangudo.

Table 3: Summary of statistical parameters of estimated annual peak instantaneous discharge series for Pembina River at Sangudo

Parameter	Estimated Annual Instantaneous Peak Flow Series (1914-1922, 1955-2015, 2018-2020)
Years of record	73
Mean (m^3/s)	307.3
Median (m^3/s)	226.5
Standard deviation (m^3/s)	253.6
Coefficient of variation	0.83
Skew coefficient (minimum, maximum, actual)	1.65, 2.01, 2.59

Each of the frequency distributions in the adopted suite was fitted to the instantaneous peak discharges shown in **Table 2**. The goodness of fit analyses described earlier were undertaken for each distribution, and the results are shown in **Table 4**.

Table 4: Goodness-of-fit comparison for probability distributions for Pembina River at Sangudo

Distribution	D_n	Normalized *SSE* (Q_{pm} = 307.3 m^3/s)	Rank by D_n	Rank by *SSE*	Combined Ranking
Normal(N)	0.186	0.428	9	9	9
Log-normal(LN)	0.071	0.180	1	4	2
Three parameter log-normal (LN3)	0.075	0.177	3	3	4
Pearson III (P3)	0.092	0.190	6	5	5
Log-Pearson III (LP3)	0.071	0.150	1	1	1
Gumbel (G)	0.140	0.272	7	8	7
Generalized extreme value (GEV)	0.089	0.191	5	6	5
Weibull (W)	0.143	0.228	8	7	7
Bulletin 17C	0.075	0.156	3	2	2

The LP3 frequency curve produced the smallest D_n and SSE value and it is ranked the best in the combined ranking, followed by the LN and Bulletin 17C distributions. These three distributions are compared in **Figure 8**. The other evaluated distributions are shown graphically in **Appendix A**.

From a visual inspection of **Figure 8**, it is clear that all three (LP3, LN and Bulletin 17C) curves are identical in the lower part. The curves diverge when the return period exceeds about 20 years. The LP3 curve appears to best fit the largest flood event (1986).

Based on the above comparisons, it is recommended that the LP3 distribution be used herein to describe the estimated flood peaks for Pembina River at Sangudo. The adopted LP3 curve is shown in **Figure 9** along with its 95% confidence limits.

Flood Frequency Estimates based on Flow Synthesis Approach

The flood frequency estimates for the Pembina River at Sangudo from the flow synthesis approach are presented in **Table 5**.

The flood frequency estimates are consistent with the closest WSC gauge station, Pembina River near Entwistle (WSC Station 07BB002). The estimated flood peaks for the Pembina River at Sangudo from the flow synthesis approach are on average 18% higher than the flood frequency estimates for the Pembina River near Entwistle (WSC 07BB002), though the drainage area increase between these two locations is about 50%. This seems reasonable considering additional boreal region with lower runoff potential between Entwistle and Sangudo and the storage effects of Chip Lake. Thus, it is expected that the flood peaks at Sangudo would only be slightly higher than the flood peaks at Entwistle (WSC Station 07BB002) and would not be as high if they are estimated from the drainage area ratio (1.5). The flow synthesis approach also produced unit flood instantaneous discharges at Sangudo that are lower than at Entwistle, as expected based on the above discussion.

Table 5: Flood frequency estimates for Pembina River at Sangudo from flow synthesis approach and compared with flood frequency estimates for Pembina River at Entwistle

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge for Pembina River at Sangudo		Peak Instantaneous Discharge for Pembina River near Entwistle (WSC Station 07BB002)	
		Value (m^3/s)	Unit Value ($m^3/s/km^2$)	Value (m^3/s)	Unit Value ($m^3/s/km^2$)
1000	0.1	2,720	0.41	2,350	0.53
750	0.13	2,520	0.38	2,170	0.49
500	0.2	2,250	0.34	1,940	0.44
350	0.29	2,040	0.31	1,750	0.40
200	0.5	1,730	0.26	1,480	0.34
100	1	1,400	0.21	1,190	0.27
75	1.3	1,270	0.19	1,080	0.25
50	2	1,110	0.17	940	0.21
35	2.9	981	0.15	828	0.19
20	5	796	0.12	669	0.15
10	10	597	0.09	499	0.11
5	20	426	0.06	354	0.08
2	50	231	0.03	191	0.04

Notes:

1. The peak instantaneous discharge for Pembina River near Entwistle is provided in the table for comparison.

6 ADOPTED FLOOD FREQUENCY ESTIMATES

The flood frequency estimates from the flow synthesis approach are adopted for the current flood study. The adopted flood frequency estimates are presented in **Table 6** and shown in **Figure 9** along with their 95% confidence limits**.**

Table 6: Adopted flood frequency estimates for Pembina River at Sangudo

Return Period (Years)	Annual Probability of Exceedance (%)	Peak Instantaneous Discharge (m^3/s)		AENV (1991)
		Value	95% Confidence Limit	
1000	0.1	2,720	1,980 - 4,110	
750	0.13	2,520	1,840 - 3,760	
500	0.2	2,250	1,670 - 3,310	
350	0.29	2,040	1,530 - 2,950	
200	0.5	1,730	1,320 - 2,440	
100	1	1,400	1,090 - 1,910	1,270
75	1.3	1,270	1,000 - 1,720	
50	2	1,110	887 - 1,480	1,040
35	2.9	981	792 - 1,280	
20	5	796	655 - 1,010	762
10	10	597	504 - 732	580
5	20	426	367 - 505	416
2	50	231	202 - 265	221

The flood frequency estimates were also compared with previous flood frequency estimates from the AENV (1991) study in **Table 6.** The adopted flood frequency estimates from the flow synthesis are slightly higher than those from the previous study. The applied flow synthesis approach in the present study is similar to but more appropriate than the SSARR modelling approach undertaken in the previous study. The AENV (1991) study used a SSARR model to route gauged Pembina River flows from Entwistle (WSC Station 07BB002) to Dapp (near WSC Station 07BC002), with ungauged tributary inflows being estimated from regional gauge data. However, they estimated the ungauged tributary inflows between Entwistle and Sangudo from gauge data for Wabash Creek located farther downstream (in the Boreal Region) and Coyote Creek which has a very small drainage area of 49 km^2 in the Boreal Region, located east of Sangudo. These reference basins are not as representative as the Paddle River near Rochefort Bridge (WSC Station 07BB004) for the estimation of ungauged tributary flows between Entwistle and Sangudo. Moreover, AENV (1991) did not consider the potential effects of the relatively large lake, Chip Lake on the estimation of ungauged tributary inflows.

7 CLIMATE CHANGE COMMENTARY

Current global climate models indicate that temperature will increase due to projected increases in CO_2 concentrations in the atmosphere. Increased temperatures in the winter months will likely result in smaller snowpacks, earlier snowmelt runoff, higher winter flows as more winter precipitation falls as rain instead of snow, lower spring flows due to reduced snow storage, and decreases in summer rainfall for some areas.

Eum et al. (2017) assessed the effects of climate change on the Athabasca River basin (including Pembina River basin) by using a process-based and distributed hydrologic model (Variable Infiltration Capacity – VIC model) forced by selected down-scaled General Circulation Model (GCM) scenarios from the latest Coupled Model Intercomparison Project (CMIP5) to a higher resolution (10 km) over Canada. The assessment includes two future periods (2050s and 2080s) and considering two emission scenarios: medium emission scenario (RCP4.5) and higher emission scenario (RCP8.5). Some of the key findings of the study are noted as follows:

- Temperature in the Athabasca River basin is likely to follow a warming trend in the future. Temperature increases over the Athabasca River basin could range from 2.7°C (RCP4.5) to 3.3°C (RCP8.5) for a projection period centered on 2050s and 3.2°C (RCP4.5) to 5.6°C (RCP8.5) for a projection period centered on 2080s. The increase in precipitation could range from approximately 6.8% (RCP4.5) to 9.7% (RCP8.5) for a projection period centered on 2050s and approximately 12.5% (RCP4.5) to 14.4% (RCP8.5) for a projection period centered on 2080s.

- The study specifically assessed the change in streamflows for the reference period (1990) and relative changes for the two future 30-year periods (2050s and 2080s) for the Pembina River basin. The projected changes in mean annual flow could be up to 10.9% for a projection period centered on 2050s and up to 23.8% for 2080s. The spring flow is projected to increase in both projection periods and both emission scenarios and could increase as much as 29.3% by 2080s. The summer flow is also projected to increase for the Pembina River basin in all scenarios except the projection scenario centered on 2050s for the lower emission scenario (RCP4.5). The flood peak is projected to increase in all scenarios and could range from approximately 7% (RCP4.5) to 14.4% (RCP8.5) for a projection period centered on 2050s and approximately 28.9% (RCP8.5) to 36.1% (RCP4.5) for a projection period centered on 2080s. The minimum flow is likely to follow a decreasing trend for the Pembina River in the future.

- An overall earlier shift in runoff timing is also projected. The projected flood peak can shift from 12 to 21 days, while the spring freshet initiation can shift from 2 to 5 days.

Shrestha et al. (2017) has also assessed the effects of climate change on freshwater resources of the Athabasca River basin (including the Pembina River basin) by using a Soil and Water Assessment Tool (SWAT) and future climate data generated by the Canadian Center for Climate Modelling and Analysis Regional Climate Model (CanRCM4) with a spatial resolution of about 25 km for same two emission scenarios (RCP 4.5 and 8.5) used by Eum et al. (2016). The study projected the climate of the Athabasca River basin to be wetter by 21–34% and warmer by 2–5.4 °C on an annual time scale.

Rood et al. (2015) performed trend analyses on historic river discharges along the Athabasca River and its major tributaries to detect significant annual or seasonal flow patterns. The trend analysis was conducted for the flow records from 1913 to 2011. The analysis revealed that the mountain and foothills reach showed slight increases in winter discharges versus larger declines in summer discharges and consequently declining annual flows. However, the further downstream reach of the Athabasca River at Athabasca (where the runoff is mostly contributed from the boreal region) displayed no overall trend in monthly or annual flows, but there was correspondence with the Pacific Decadal Oscillation that contributed to a temporary flow decline from 1970 to 2000.

Siemens (2019) has applied the Snowmelt Runoff Model (SRM) to project climate change impact on future runoff in the Upper Athabasca River. The projection period for this study was centered on 2070-2080 and different climate change scenarios have been simulated. The results demonstrated a consistent pattern of change in runoff across all scenarios, with significant increases in spring runoff, minor increases over the winter months, and decreased runoff in summer.

Poitras et al. (2011) investigated projected changes in average and extreme streamflows of ten major river basins across western Canada. The streamflows were derived from climate simulations performed with the fourth generation of the Canadian Regional Climate Model (CRCM) forced with the A2 emission scenario (an upper-mid range emission scenario representing a very heterogeneous world where economic development is regionally oriented and economic growth and technological change are relatively slow). The comparisons were made between the 1961 – 1990 period and 2041 – 2070 period. Mean annual flows were projected to increase in all basins, with an 11% increase in the Athabasca River basin. In future climate, snowmelt events in the Athabasca basin were predicted to occur earlier as well.

In summary, most of the scientific literature indicates increased temperature and precipitation in the Pembina/Athabasca River basin. Climate change has the potential to affect the timing and volume of flows in the Pembina River. In general, an increase in streamflows in spring and winter is expected, and most studies suggested a decrease in summer flows. An earlier shift of spring freshet timing is expected because of warmer air temperature. Overall, there is insufficient information to identify all the linkages between precipitation and runoff to make any forecasts about how climate change might affect flood peaks. Given the small change in median flows predicted and considering all the uncertainties associated with the climate change modelling, the most judicious approach would be to assume no changes to flood peaks for the study area over the next number of decades. This is consistent with the conclusions of the Intergovernmental Panel on Climate Change – that at present, there is low confidence in global climate model predictions of changes in flood magnitudes due to limited evidence (Jiménez et al., 2014). In general, increased precipitation may lead to higher flood peaks due to increased precipitation intensity, but this will be mitigated by reduced snowpack and drier antecedent moisture conditions due to higher temperatures. Loss of tree cover and soil changes associated with the beetle infestation, wildfires, and changing land use could also contribute to higher runoff volumes and peaks – possibly even having a greater impact than the changing climate.

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9 CLOSURE

This document has been prepared by Northwest Hydraulic Consultants Ltd. (NHC) in accordance with generally accepted engineering practices, for the benefit of Alberta Environment and Parks for specific application to the Sangudo Flood Study in Alberta. The information and data contained herein represent the best professional judgment of NHC, based on the knowledge and information available to NHC at the time of preparation.

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Sincerely,

Northwest Hydraulic Consultants Ltd.

Prepared by:	**Reviewed by:**
Md Makamum Mahmood, MEng, PEng Project Engineer	Gary Van Der Vinne, MSc, PEng Principal

Figures

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FLOOD FREQUENCY ESTIMATION LOCATION
STUDY AREA
STUDY REACH

SCALE - 1:15,000
0 75 150 300 450 600 750 M
N

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

Engineer	GIS	Reviewer
MMM	REH	PGV

Job: 1006073 | Date: 20-AUG-2021

SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

FLOOD HAZARD STUDY AREA AND FLOOD FREQUENCY ESTIMATE LOCATION

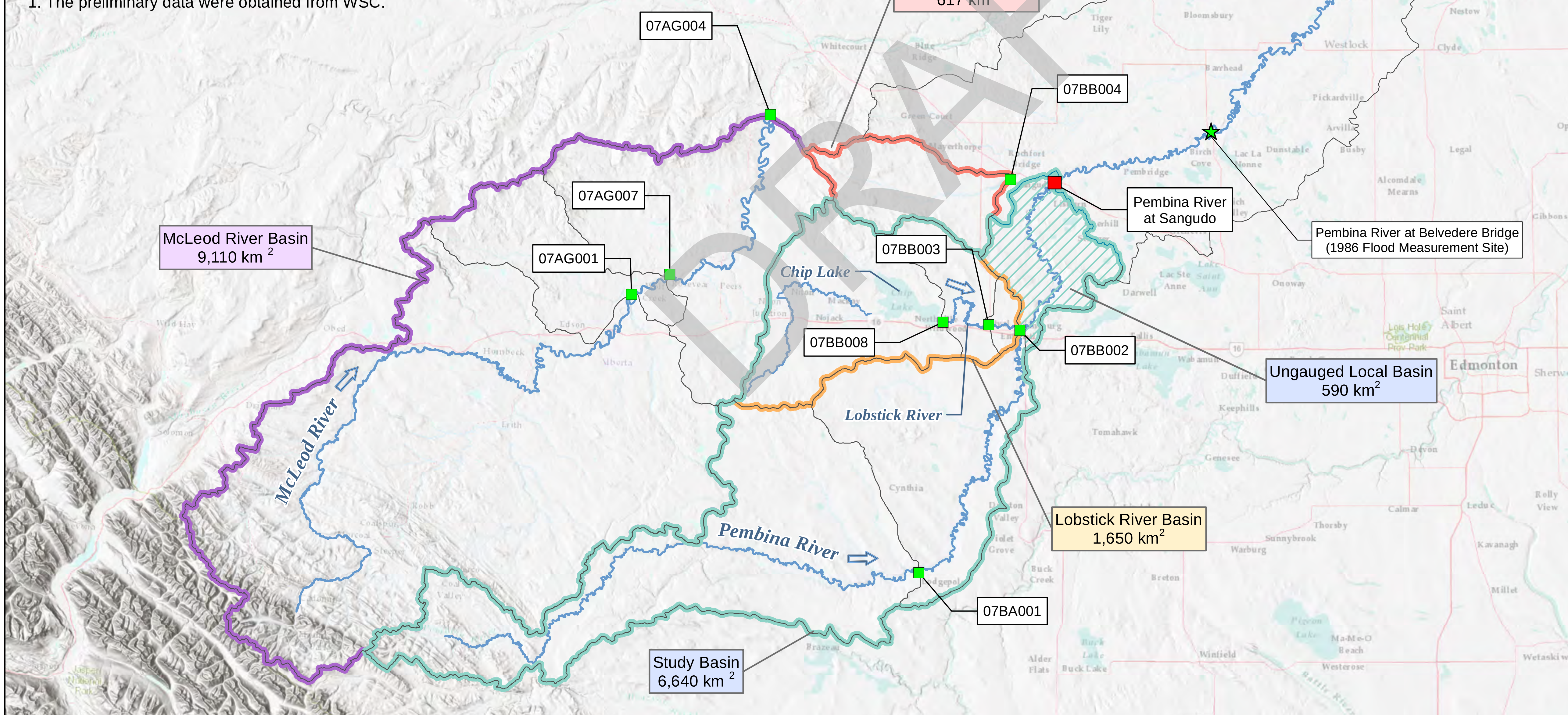
FIGURE 1

REH, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_OWH_Fig1_Reach.mxd

Classification: Public

WSC Station ID	Station name	Drainage Area (km^2)	Period of Record
Key Stations for Regional Analysis			
07BA001	Pembina River below Paddy Creek	2,900	1956-1993, 2013-2019, 2020[1]
07BB002	Pembina River near Entwistle	4,400	1914-1922, 1955-2015, 2018-2019, 2020[1]
07BC002	Pembina River at Jarvie	13,100	1957-2020
07AG007	McLeod River near Rosevear	7,140	1984-2014, 2015[1], 2018-2019, 2020[1]
07AG004	McLeod River near Whitecourt	9,110	1968-2016, 2018-2019, 2020[1]
07AG001	McLeod River near Wolf Creek	6,310	1914-1931, 1957-1984
Supplemental Stations for Flow Synthesis			
07BB004	Paddle River near Rochfort Bridge	617	1963-2014, 2015-2017[1], 2018-2019, 2020[1]
07BB008	Chip Lake near Northville	1,210	1972-2009
07BB003	Lobstick River near Styal	1,570	1954-1986

1. The preliminary data were obtained from WSC.



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WSC GAUGE STATION (SELECTED)

FLOOD STUDY AREA

SUBBASIN

SCALE - 1:1,000,000
0 5 10 20 30 40 KM

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

Engineer	GIS	Reviewer
MMM	REH	PGV

Job: 1006073 Date: 24-AUG-2021

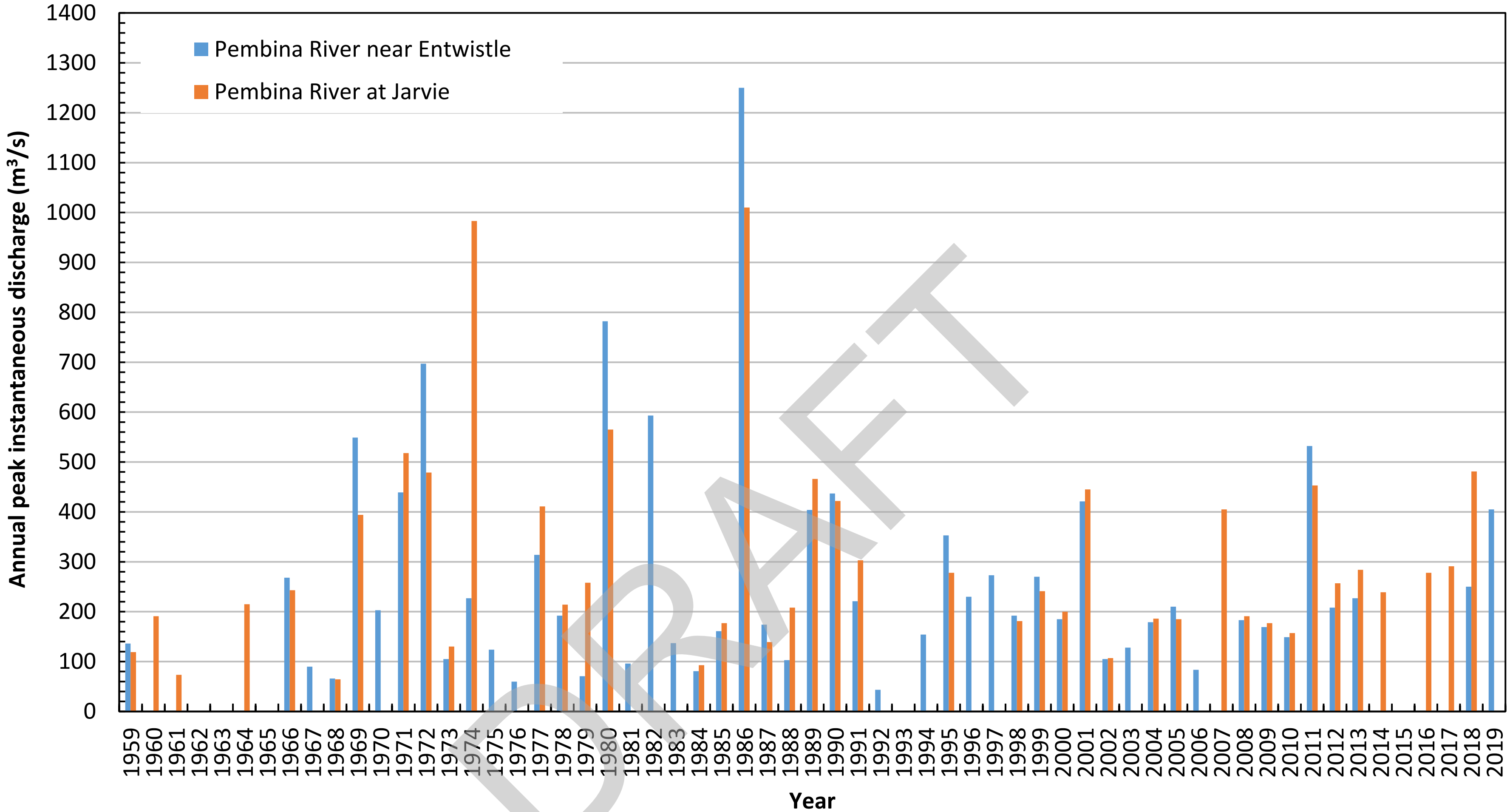
SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

BASIN OVERVIEW

FIGURE 2

REH, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_OWH_Fig2_Basin.mxd

Classification: Public



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SCALE – AS SHOWN

Coordinate System:
Units: As Shown

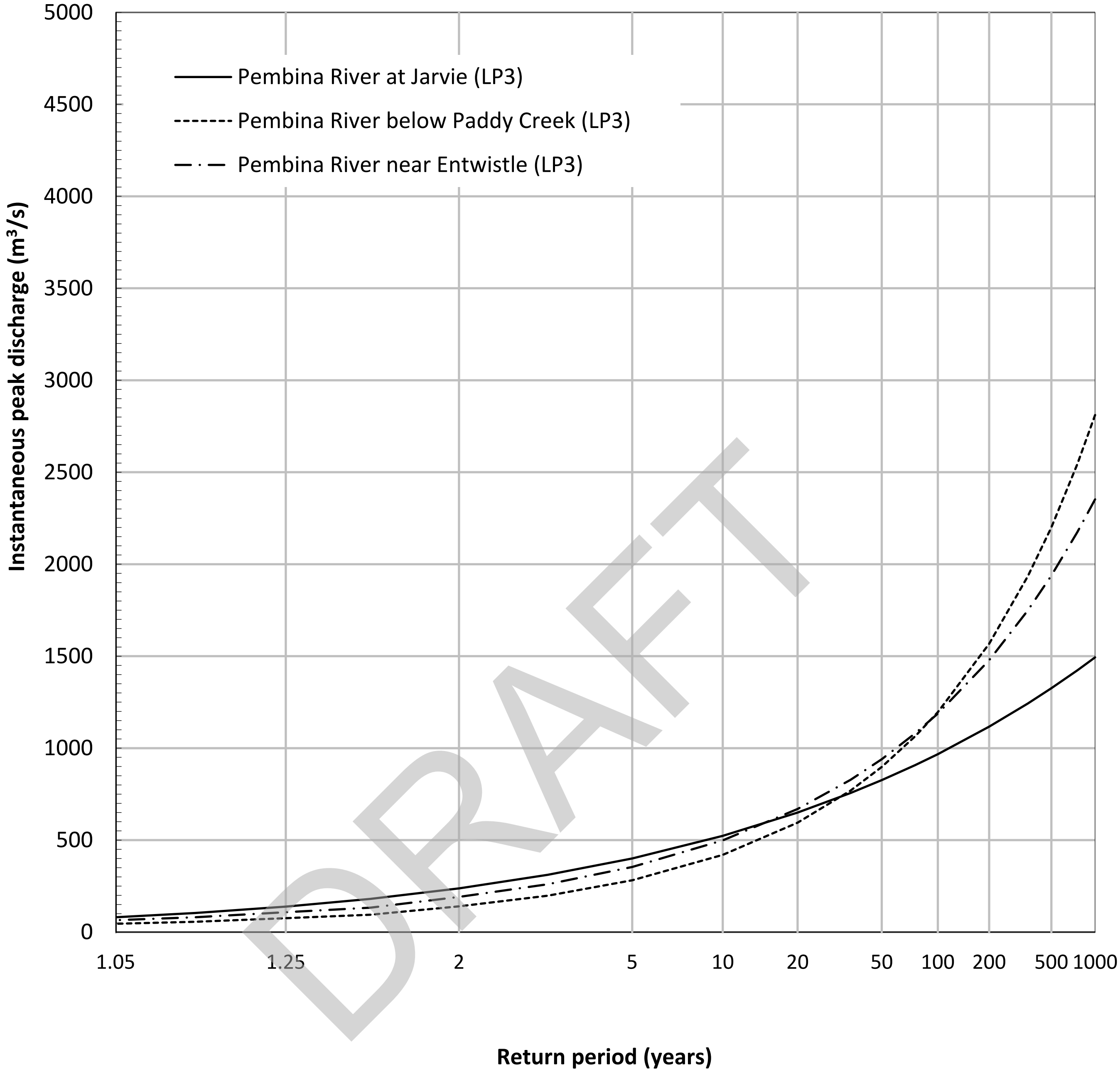
Job: 1006073

Date: 16-AUG-2021

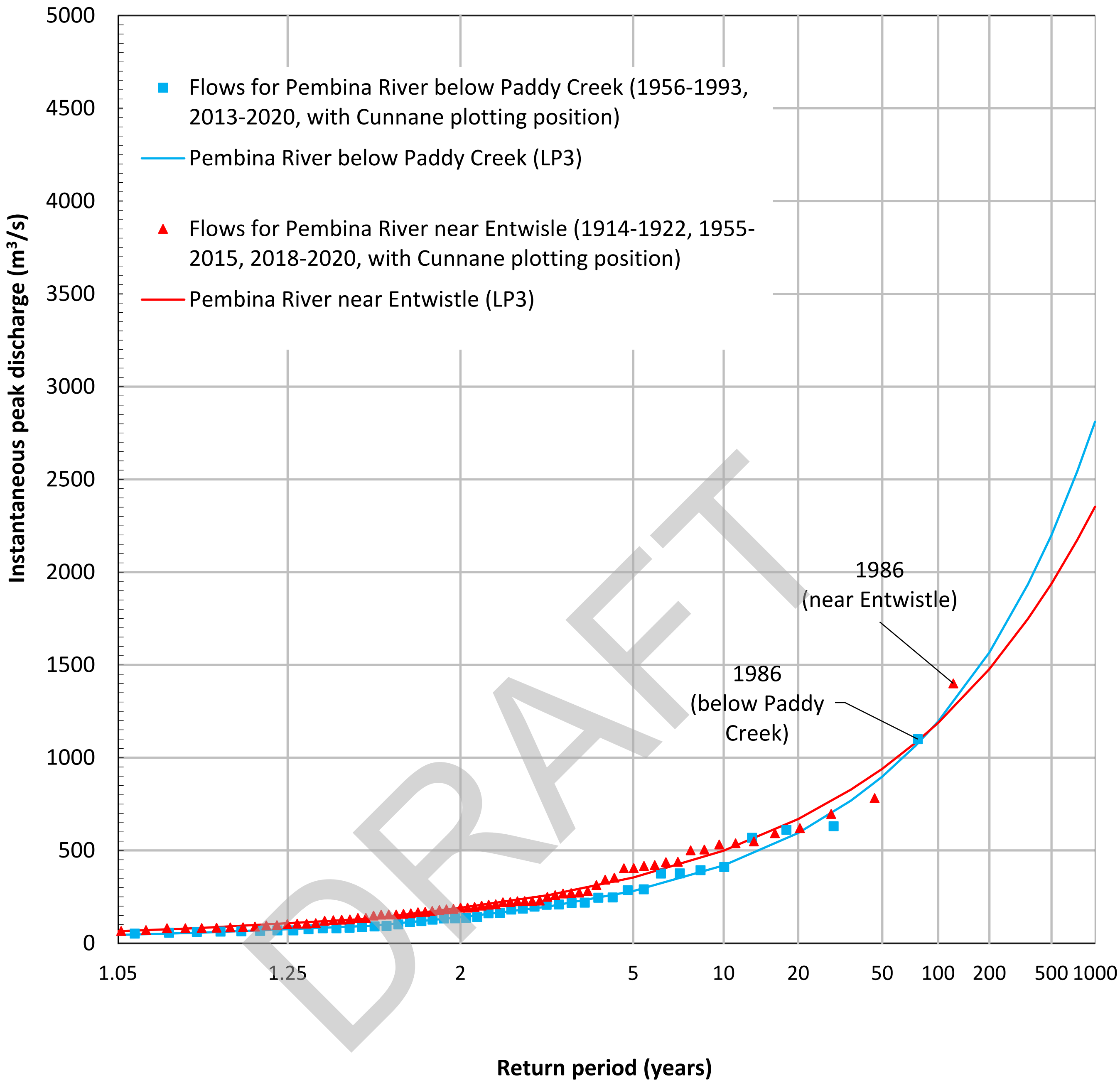
SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

COMPARISON OF PEAK INSTANTANEOUS DISCHARGES AT PEMBINA RIVER GAUGES

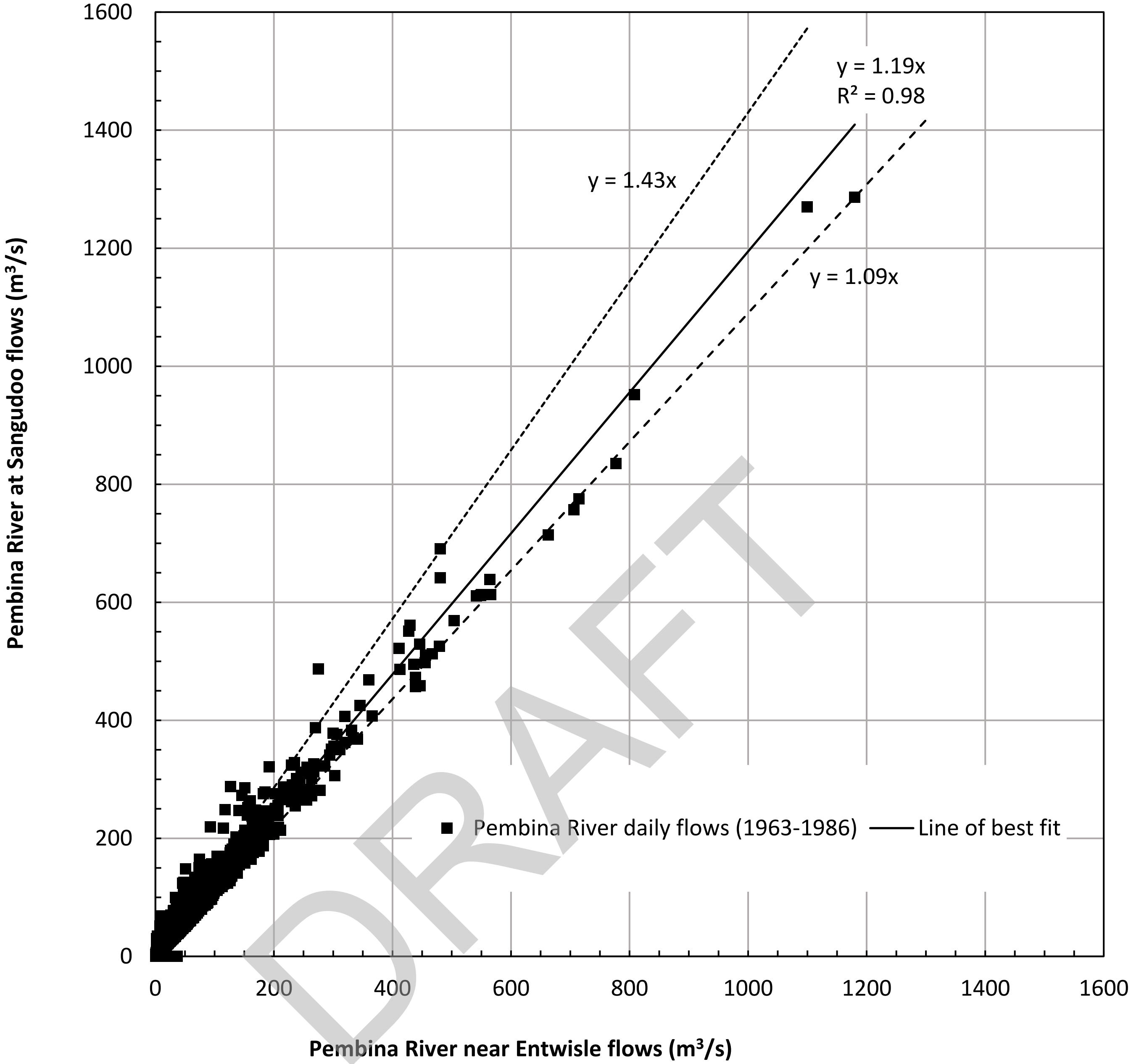
FIGURE 3



Alberta	SCALE – AS SHOWN	*SANGUDO FLOOD STUDY* *OPEN WATER HYDROLOGY ASSESSMENT* *COMPARISON OF PEMBINA RIVER FLOOD FREQUENCY CURVES*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	
	Job: 1006073 | *Date: 20-SEP-2021*	*FIGURE 4*



Alberta	SCALE – AS SHOWN	*SANGUDO FLOOD STUDY* *OPEN WATER HYDROLOGY ASSESSMENT* *COMPARISON OF PLOTTED PEMBINA RIVER FLOWS BELOW PADDY CREEK AND NEAR ENTWISTLE*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	
	Job: 1006073 | *Date: 20-SEP-2021*	*FIGURE 5*



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SCALE – AS SHOWN

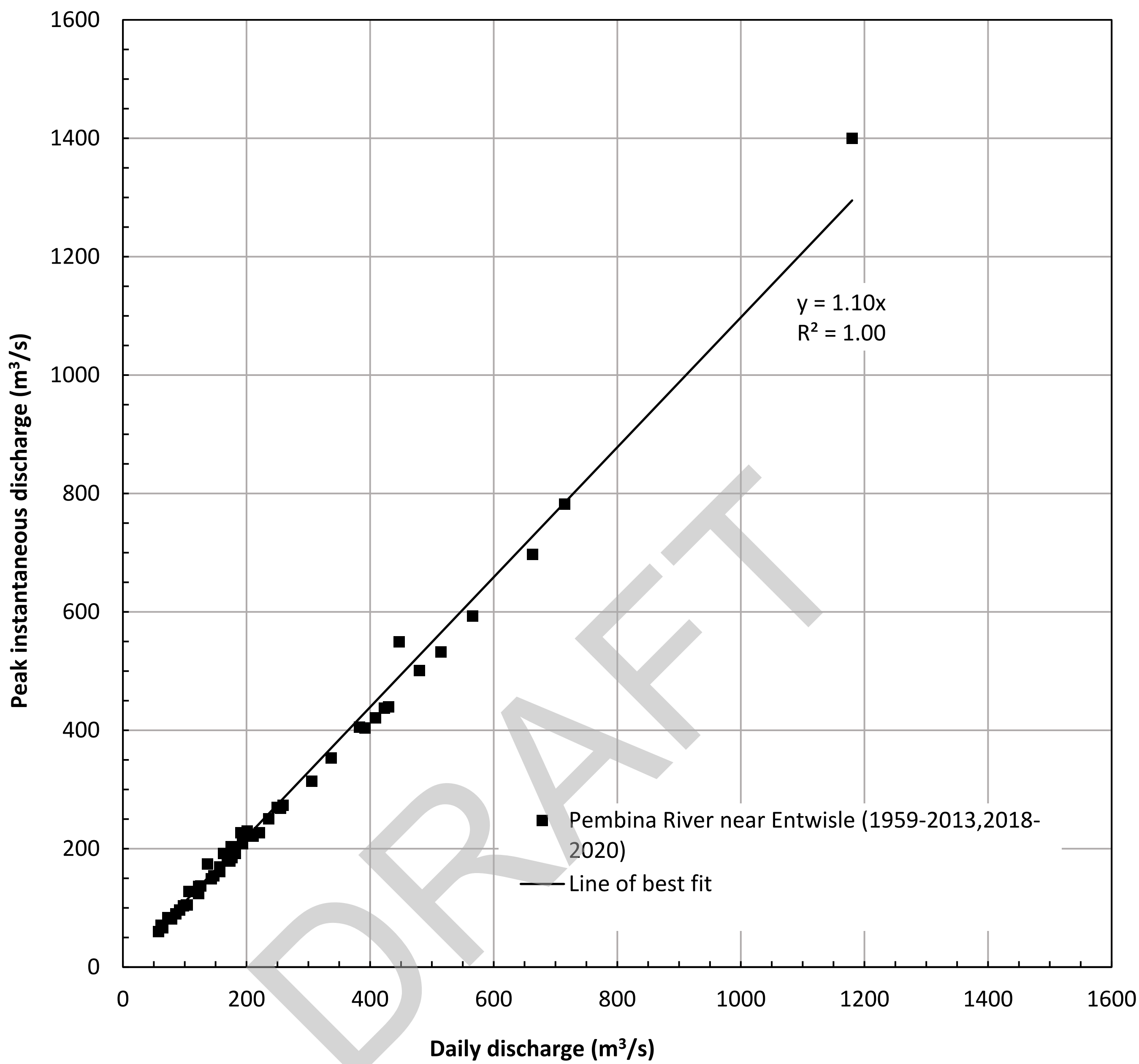
Coordinate System:
Units: As Shown

SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

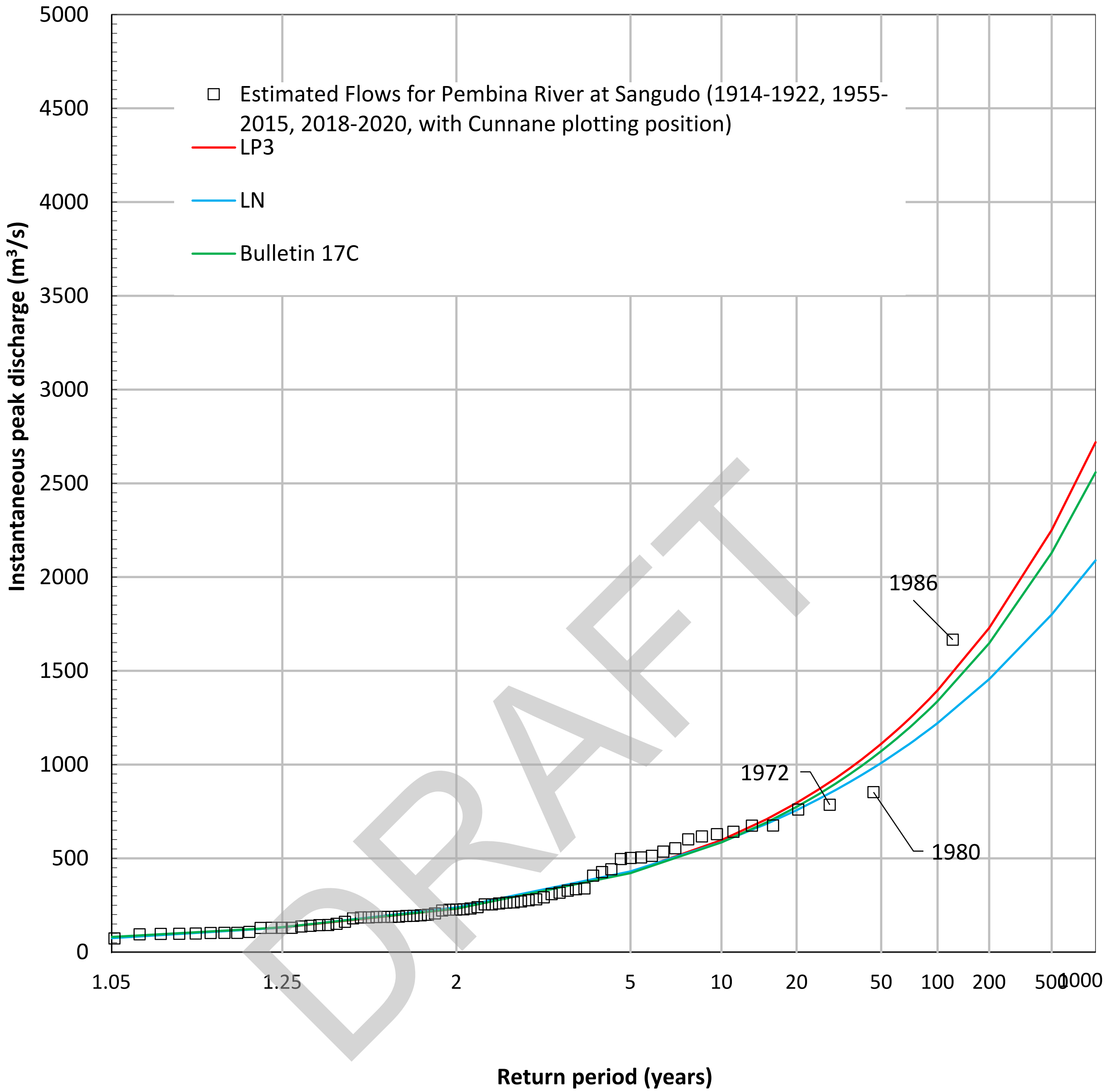
RELATIONSHIP BETWEEN DAILY DISCHARGES FOR PEMBINA RIVER AT ENTWISTLE AND SANGUDO

Job: 1006073 | *Date: 20-SEP-2021*

FIGURE 6



Alberta	SCALE – AS SHOWN	*SANGUDO FLOOD STUDY* *OPEN WATER HYDROLOGY ASSESSMENT* *COMPARISON OF MAXIMUM INSTANTANEOUS TO DAILY DISCHARGE FOR PEMBINA RIVER NEAR ENTWISTLE*
nhc northwest hydraulic consultants	Coordinate System: Units: As Shown	
	Job: 1006073 | *Date: 20-SEP-2021*	*FIGURE 7*



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

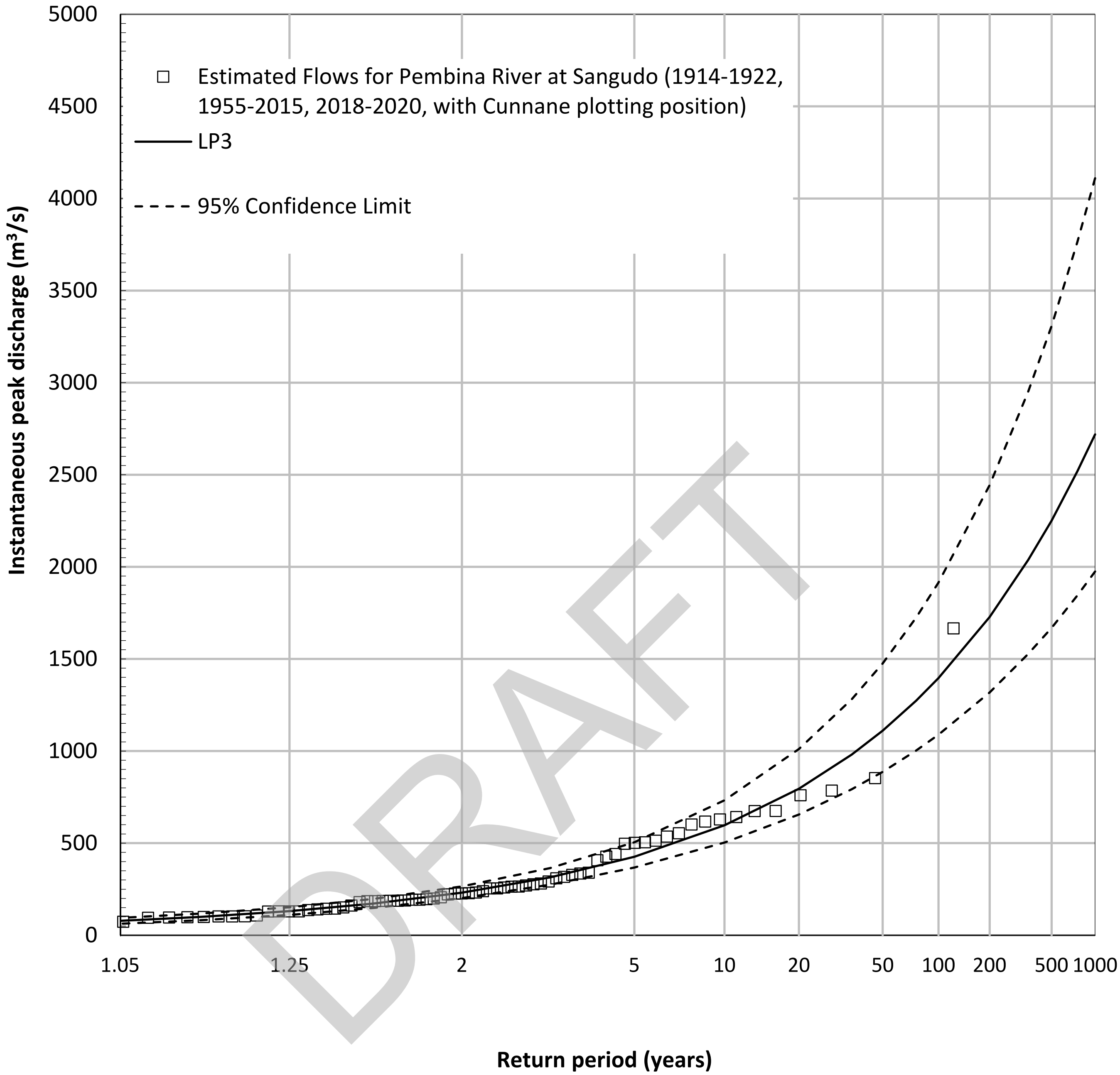
Job: 1006073

Date: 20-SEP-2021

SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

COMPARISON OF FLOOD FREQUENCY CURVES FOR PEMBINA RIVER AT SANGUDO

FIGURE 8



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1006073 | Date: 20-SEP-2021

SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

LOG PEARSON TYPE III FLOOD FREQUENCY CURVES FOR PEMBINA RIVER AT SANGUDO

FIGURE 9

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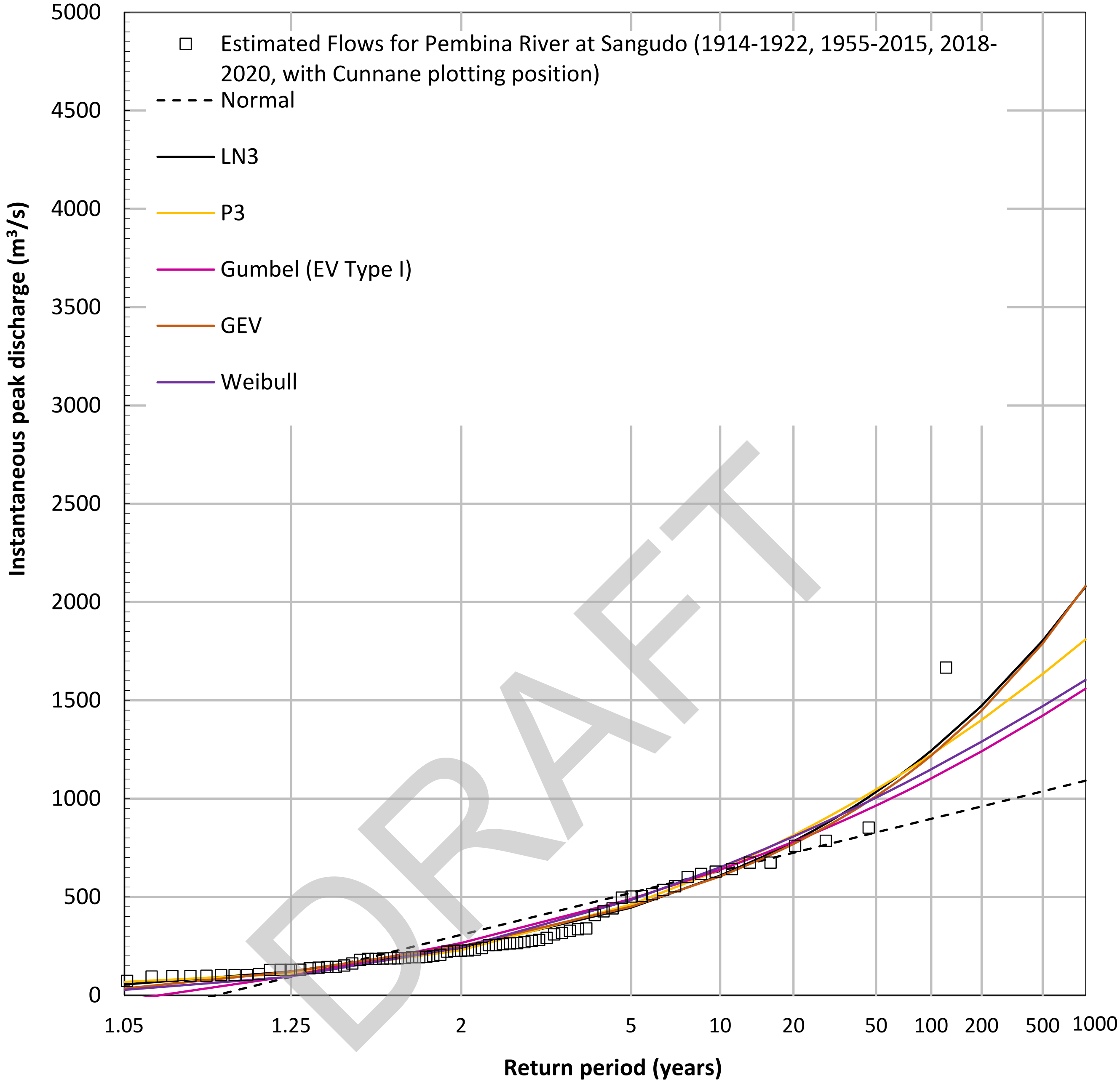
Appendix A

Additional Evaluated Frequency Distributions

For Pembina River at Sangudo



SCALE – AS SHOWN

Coordinate System:
Units: As Shown

Job: 1006073

Date: 20-AUG-2021

SANGUDO FLOOD STUDY
OPEN WATER HYDROLOGY ASSESSMENT

SINGLE STATION FLOOD FREQUENCY ANALYSIS FOR PEMBINA RIVER AT SANGUDO

FIGURE A- 1

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Appendix D
Sensitivity Analysis Results

Table D-1 Sensitivity analysis results for flood frequency estimates

River Station (m)	100-Year Flood Levels (m) for Varying Flood Frequency Estimates		
	Lower 95% Limit of Flood Frequency Estimates	Adopted Flood Frequency Estimates	Upper 95% Limit of Flood Frequency Estimates
7,305	665.09	665.83	666.93
7,285	665.08	665.81	666.92
7,019	665.00	665.73	666.86
6,757	664.93	665.65	666.80
6,490	664.83	665.58	666.78
6,217	664.79	665.57	666.78
5,920	664.70	665.52	666.76
5,728	664.65	665.49	666.74
5,414	664.60	665.44	666.68
5,186	664.56	665.38	666.62
4,994	664.34	665.09	666.29
4,850	664.35	665.14	666.36
4,679	664.35	665.15	666.38
4,532	664.32	665.10	666.30
4,434	664.29	665.07	666.26
4,420	664.21	664.98	666.18
4,386	664.20	664.98	666.18
4,299	664.16	664.93	666.14
4,182	664.08	664.83	666.04
4,046	664.04	664.77	665.91
3,908	664.00	664.73	665.86
3,777	663.90	664.62	665.74
3,583	663.86	664.58	665.73
3,367	663.81	664.52	665.68
3,205	663.81	664.52	665.67
3,185	663.66	664.51	665.65
3,080	663.60	664.46	665.64
2,912	663.50	664.36	665.55
2,746	663.44	664.30	665.51
2,557	663.28	664.19	665.44
2,442	663.27	664.11	665.20
2,428	663.23	664.08	665.17
2,361	663.22	664.06	665.15
2,343	663.09	663.93	665.03
2,199	663.07	663.93	665.06
2,082	663.04	663.89	664.98
1,851	662.91	663.73	664.77
1,605	662.75	663.58	664.62
1,400	662.70	663.52	664.54
1,113	662.64	663.47	664.51
812	662.49	663.31	664.35
374	662.38	663.21	664.25
0	662.21	663.03	664.08
Average Difference	-0.80	0.00	1.15
Maximum Difference	-0.91	0.00	1.25

Table D-2 Sensitivity analysis results for downstream boundary conditions

River Station (m)	100-Year Flood Levels (m) for Varying Downstream Boundary Condition		
	Low Normal Depth Slope (S = 0.00029 m/m)	Adopted Normal Depth Slope (S= 0.00036 m/m)	High Normal Depth Slope (S = 0.00043 m/m)
7,305	665.90	665.83	665.78
7,285	665.88	665.81	665.76
7,019	665.81	665.73	665.68
6,757	665.73	665.65	665.60
6,490	665.67	665.58	665.52
6,217	665.66	665.57	665.50
5,920	665.62	665.52	665.45
5,728	665.59	665.49	665.42
5,414	665.54	665.44	665.36
5,186	665.49	665.38	665.31
4,994	665.21	665.09	665.01
4,850	665.26	665.14	665.05
4,679	665.27	665.15	665.06
4,532	665.22	665.10	665.01
4,434	665.19	665.07	664.98
4,420	665.11	664.98	664.89
4,386	665.10	664.98	664.89
4,299	665.06	664.93	664.84
4,182	664.96	664.83	664.74
4,046	664.91	664.77	664.68
3,908	664.86	664.73	664.63
3,777	664.76	664.62	664.51
3,583	664.73	664.58	664.47
3,367	664.68	664.52	664.41
3,205	664.67	664.52	664.40
3,185	664.67	664.51	664.40
3,080	664.63	664.46	664.35
2,912	664.54	664.36	664.23
2,746	664.49	664.30	664.17
2,557	664.40	664.19	664.03
2,442	664.31	664.11	663.96
2,428	664.28	664.08	663.93
2,361	664.26	664.06	663.91
2,343	664.14	663.93	663.78
2,199	664.14	663.93	663.77
2,082	664.10	663.89	663.73
1,851	663.96	663.73	663.56
1,605	663.82	663.58	663.39
1,400	663.77	663.52	663.32
1,113	663.74	663.47	663.27
812	663.60	663.31	663.08
374	663.52	663.21	662.96
0	663.38	663.03	662.75
Average Difference	0.16	0.00	-0.12
Maximum Difference	0.35	0.00	-0.28

Table D-3 Sensitivity analysis results for channel roughness

River Station (m)	100-Year Flood Levels (m) for Varying Channel Roughness		
	Low Channel Roughness (-20%)	Adopted Roughness	High Channel Roughness (+20%)
7,305	665.24	665.83	666.31
7,285	665.23	665.81	666.30
7,019	665.15	665.73	666.22
6,757	665.07	665.65	666.14
6,490	664.97	665.58	666.08
6,217	664.95	665.57	666.06
5,920	664.85	665.52	666.04
5,728	664.82	665.49	666.01
5,414	664.78	665.44	665.95
5,186	664.74	665.38	665.90
4,994	664.40	665.09	665.67
4,850	664.46	665.14	665.69
4,679	664.49	665.15	665.68
4,532	664.46	665.10	665.63
4,434	664.43	665.07	665.60
4,420	664.34	664.98	665.52
4,386	664.33	664.98	665.51
4,299	664.28	664.93	665.48
4,182	664.16	664.83	665.39
4,046	664.11	664.77	665.33
3,908	664.08	664.73	665.26
3,777	663.94	664.62	665.17
3,583	663.91	664.58	665.13
3,367	663.87	664.52	665.06
3,205	663.89	664.52	665.04
3,185	663.88	664.51	665.03
3,080	663.81	664.46	665.01
2,912	663.68	664.36	664.92
2,746	663.62	664.30	664.87
2,557	663.40	664.19	664.81
2,442	663.41	664.11	664.67
2,428	663.36	664.08	664.64
2,361	663.35	664.06	664.62
2,343	663.22	663.93	664.50
2,199	663.21	663.93	664.49
2,082	663.19	663.89	664.43
1,851	663.02	663.73	664.28
1,605	662.83	663.58	664.14
1,400	662.80	663.52	664.06
1,113	662.78	663.47	664.00
812	662.58	663.31	663.86
374	662.50	663.21	663.74
0	662.29	663.03	663.59
Average Difference	-0.67	0.00	0.54
Maximum Difference	-0.79	0.00	0.62

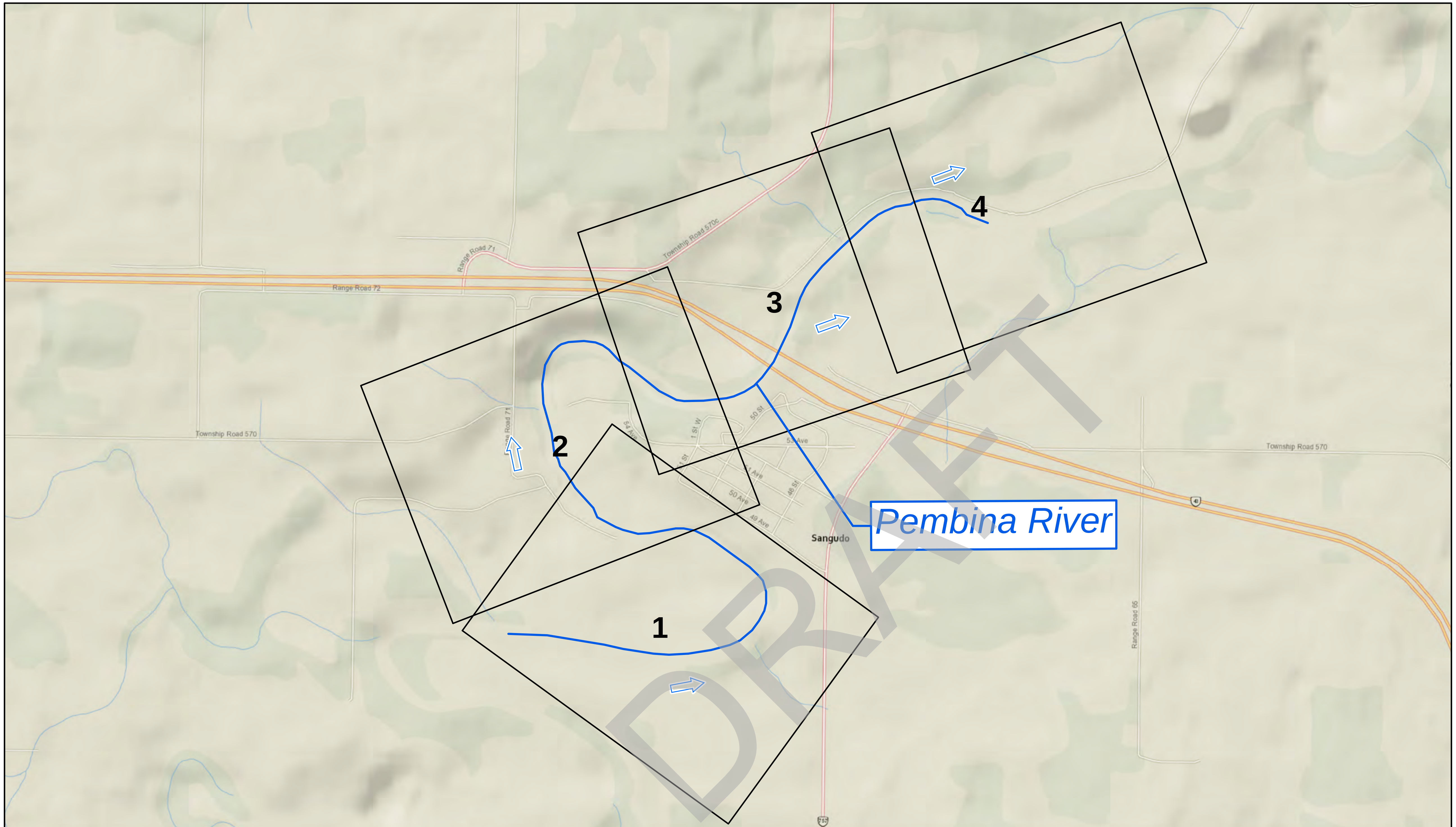
Table D-4 Sensitivity analysis results for overbank roughness

River Station (m)	100-Year Flood Levels (m) for Varying Overbank Roughness		
	Low Overbank Roughness (-20%)	Adopted/Calibrated Roughness	High Overbank Roughness (+20%)
7,305	665.76	665.83	665.87
7,285	665.75	665.81	665.86
7,019	665.67	665.73	665.78
6,757	665.59	665.65	665.70
6,490	665.52	665.58	665.62
6,217	665.51	665.57	665.61
5,920	665.48	665.52	665.56
5,728	665.45	665.49	665.52
5,414	665.39	665.44	665.47
5,186	665.34	665.38	665.42
4,994	665.05	665.09	665.13
4,850	665.10	665.14	665.17
4,679	665.11	665.15	665.18
4,532	665.05	665.10	665.13
4,434	665.02	665.07	665.10
4,420	664.94	664.98	665.02
4,386	664.93	664.98	665.01
4,299	664.89	664.93	664.97
4,182	664.79	664.83	664.86
4,046	664.74	664.77	664.81
3,908	664.68	664.73	664.76
3,777	664.57	664.62	664.65
3,583	664.53	664.58	664.61
3,367	664.48	664.52	664.56
3,205	664.47	664.52	664.55
3,185	664.46	664.51	664.55
3,080	664.42	664.46	664.50
2,912	664.32	664.36	664.39
2,746	664.27	664.30	664.33
2,557	664.16	664.19	664.20
2,442	664.07	664.11	664.14
2,428	664.03	664.08	664.11
2,361	664.02	664.06	664.09
2,343	663.89	663.93	663.97
2,199	663.89	663.93	663.96
2,082	663.84	663.89	663.92
1,851	663.68	663.73	663.77
1,605	663.53	663.58	663.61
1,400	663.47	663.52	663.56
1,113	663.42	663.47	663.51
812	663.26	663.31	663.34
374	663.16	663.21	663.24
0	662.98	663.03	663.06
Average Difference	-0.05	0.00	0.03
Maximum Difference	-0.07	0.00	0.05

Appendix E
Open Water Flood Inundation Map Library

(provided under separate cover)

Appendix F
Floodway Criteria Map



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FLOW DIRECTION
STUDY REACH
MAP SHEET

SCALE - 1:22,000
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M
N

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

Engineer	GIS	Reviewer
MMM	JY	PGV

Job: 1006073 | Date: 04-MAR-2022

SANGUDO FLOOD STUDY

FLOODWAY CRITERIA MAP

INDEX MAP

Notes to Users:

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

Definitions:

Flood Hazard Map - A flood hazard map is a specific type of flood map that identifies the area flooded for the 1:100 design flood, and divides that flood hazard area into floodway and flood fringe zones. Flood hazard maps can also show additional flood hazard information, including the incremental areas at risk for more severe floods like the 1:200 and 1:500 floods. Flood hazard maps are typically used for long-term flood hazard area management and land-use planning.
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Floodway - When a floodway is first defined on a flood hazard map, it typically represents the area of highest flood hazard where flows are deepest, fastest, and most destructive during the 1:100 design flood. When a flood hazard map is updated, the floodway will not get larger in most circumstances to maintain long-term regulatory certainty, even if the flood hazard area gets larger or design flood levels get higher.
Flood Fringe - The flood fringe is the area outside of the floodway that is flooded or could be flooded during the 1:100 design flood. The flood fringe typically represents areas with

Definitions (continued):

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

Data Sources and References:

1. Orthophoto imagery acquired by OGL Engineering for Alberta Environment and Parks: *OGL Engineering (2020). Sangudo aerial imagery acquisition memorandum, project number 2020-501, submitted to Alberta Environment and Parks, 5 pp.*
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3. Additional base mapping from Esri.

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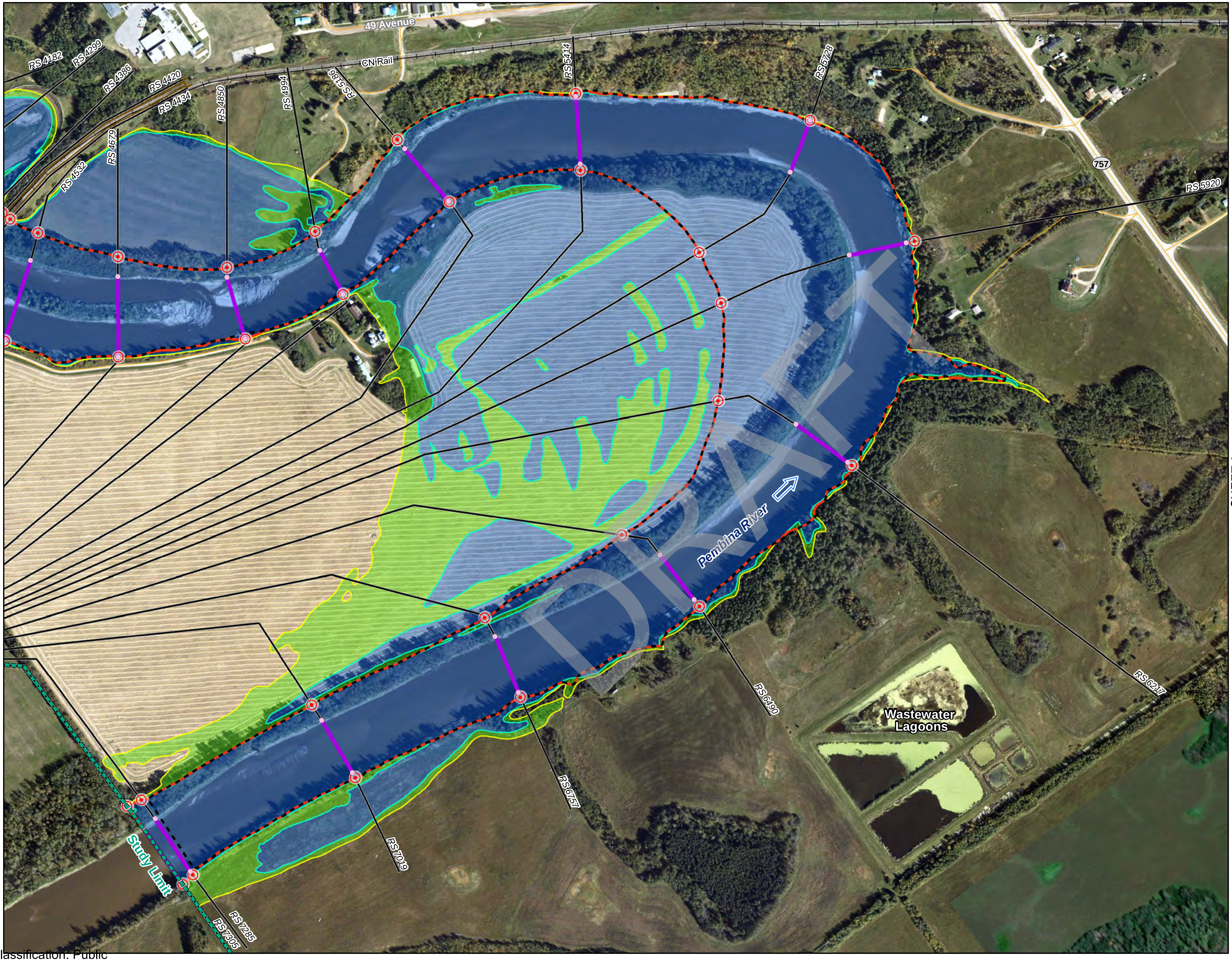

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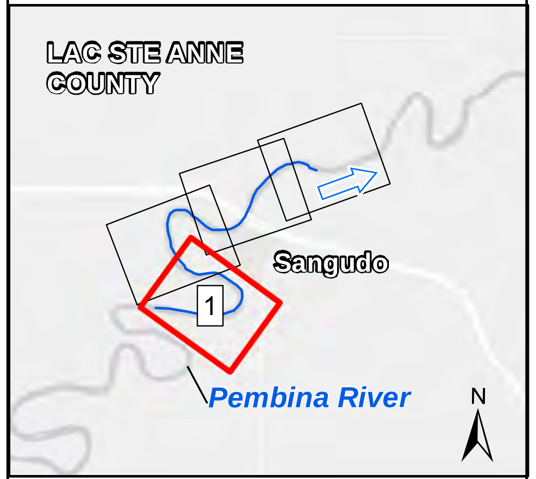


- FLOW DIRECTION
- BRIDGE
- MODEL CROSS SECTION
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT
- BANK STATION
- PROPOSED FLOODWAY LIMIT
- VELOCITY ≥ 1 m/s
- PROPOSED FLOODWAY BOUNDARY
- 1m DEPTH CONTOUR
- DEPTH ≥ 1 m
- 100-YEAR OPEN WATER DESIGN FLOOD EXTENT
- PREVIOUS FLOODWAY

SCALE - 1:5,000
0 100 200 M
N

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

Engineer MMM	GIS JY	Reviewer PGV

Job: 1006073	*Date: 15-MAR-2022*

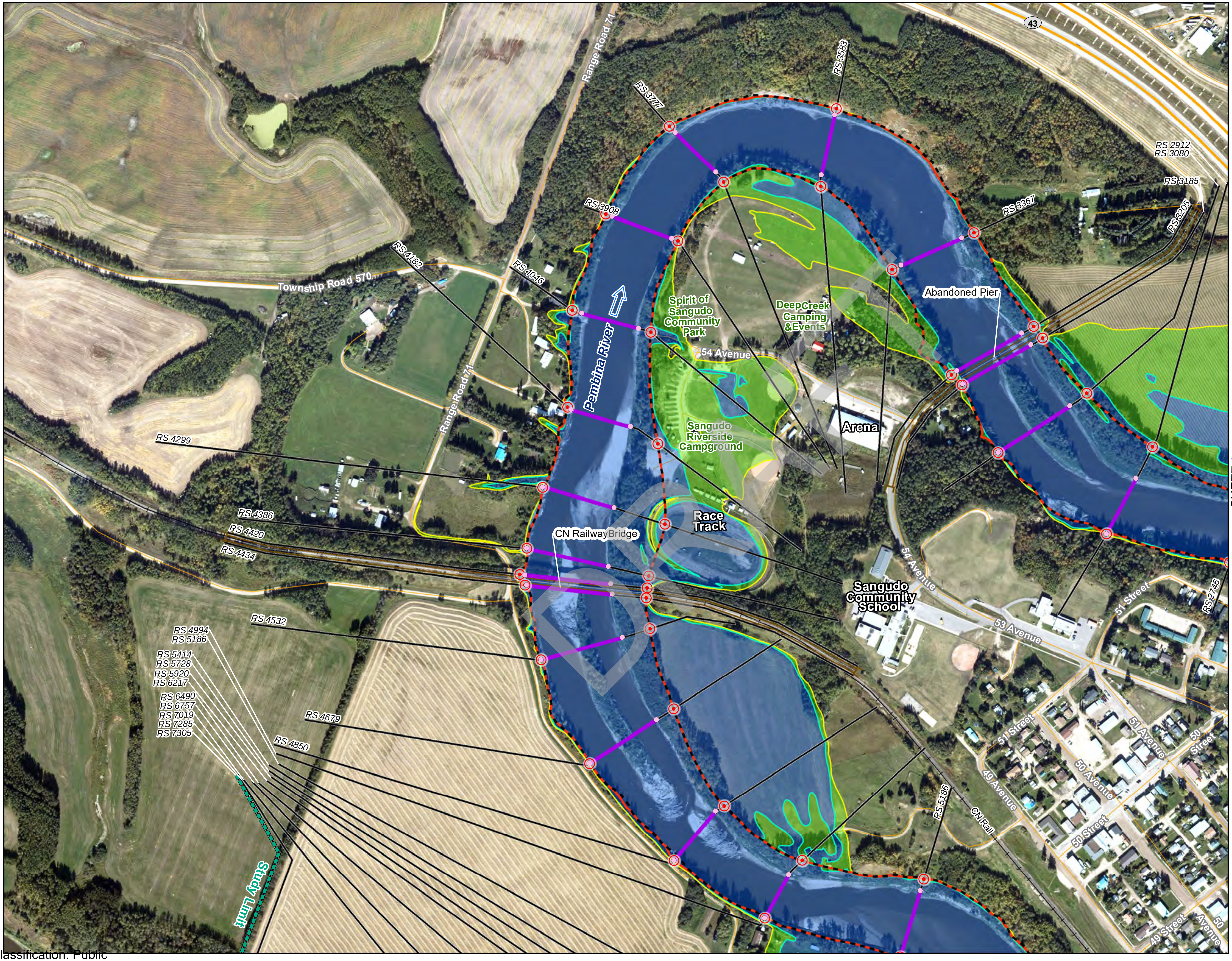
SANGUDO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 1 OF 4

JY, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_JY_Map_FloodwayCriteria.mxd

Classification: Public



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LAC STE ANNE COUNTY

2

Sangudo

Pembina River

N

FLOW DIRECTION

BRIDGE

MODEL CROSS SECTION

LOCAL ROAD

PROVINCIAL HIGHWAY

RAILWAY

STUDY LIMIT

BANK STATION

PROPOSED FLOODWAY LIMIT

VELOCITY ≥ 1 m/s

PROPOSED FLOODWAY BOUNDARY

1m DEPTH CONTOUR

DEPTH ≥ 1 m

100-YEAR OPEN WATER DESIGN FLOOD EXTENT

PREVIOUS FLOODWAY

SCALE - 1:5,000

0 100 200 M

N

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

Engineer MMM	GIS JY	Reviewer PGV

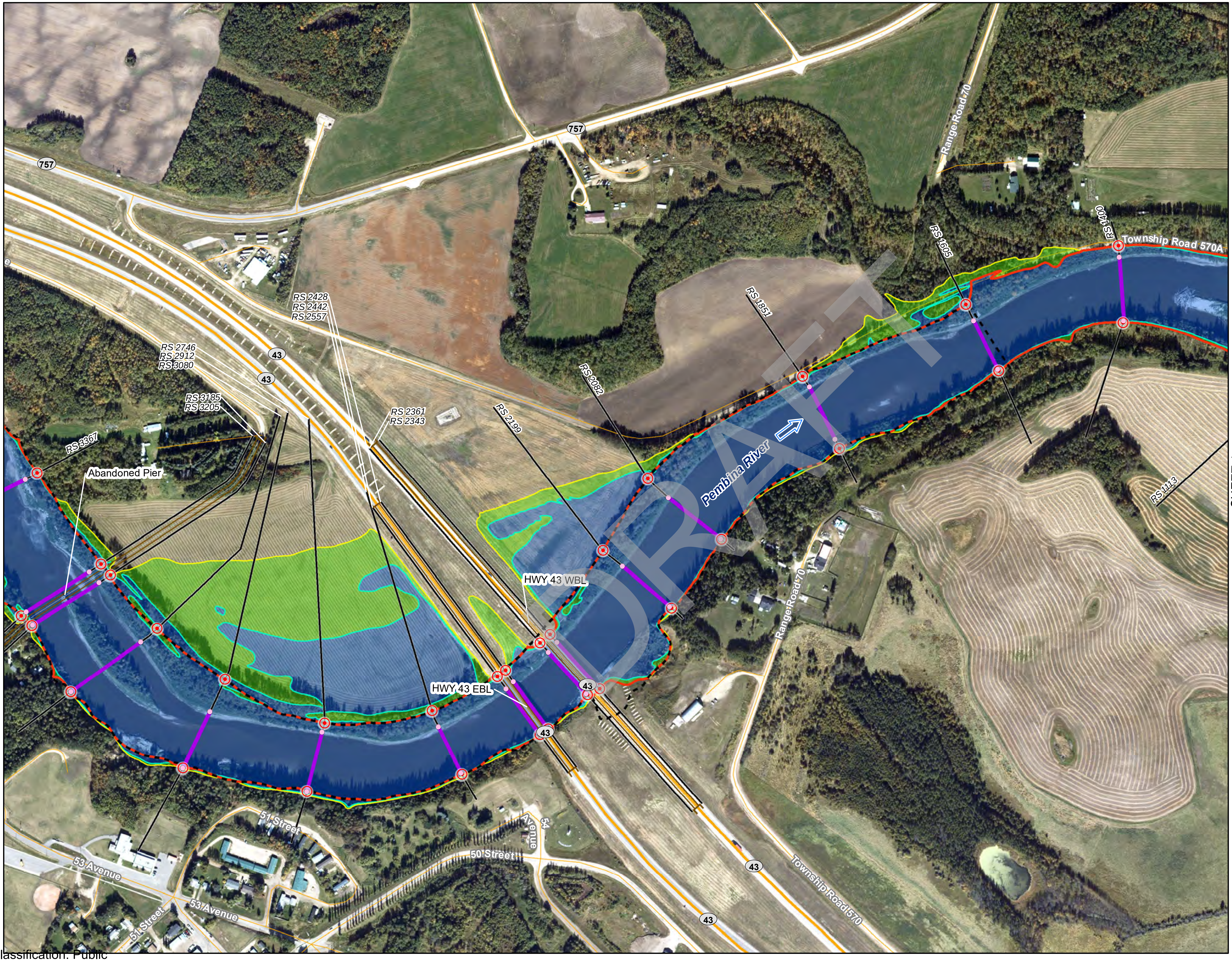
Job: 1006073 Date: 15-MAR-2022

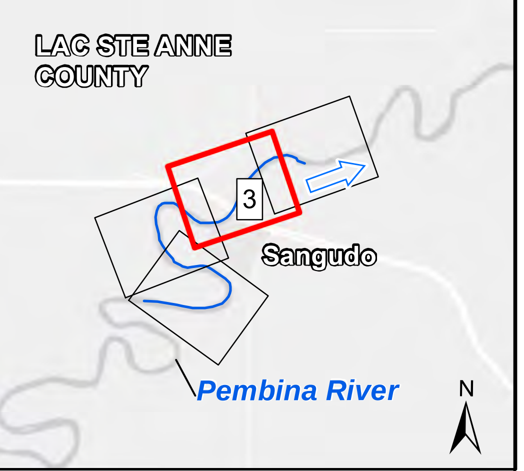
SANGUDO FLOOD STUDY

FLOODWAY CRITERIA MAP

SHEET 2 OF 4

JY, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_JY_Map_FloodwayCriteria.mxd





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- FLOW DIRECTION
- BRIDGE
- MODEL CROSS SECTION
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT
- BANK STATION
- PROPOSED FLOODWAY LIMIT
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- PROPOSED FLOODWAY BOUNDARY
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- PREVIOUS FLOODWAY

SCALE - 1:5,000
0 100 200 M
N

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

Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 15-MAR-2022	

SANGUDO FLOOD STUDY

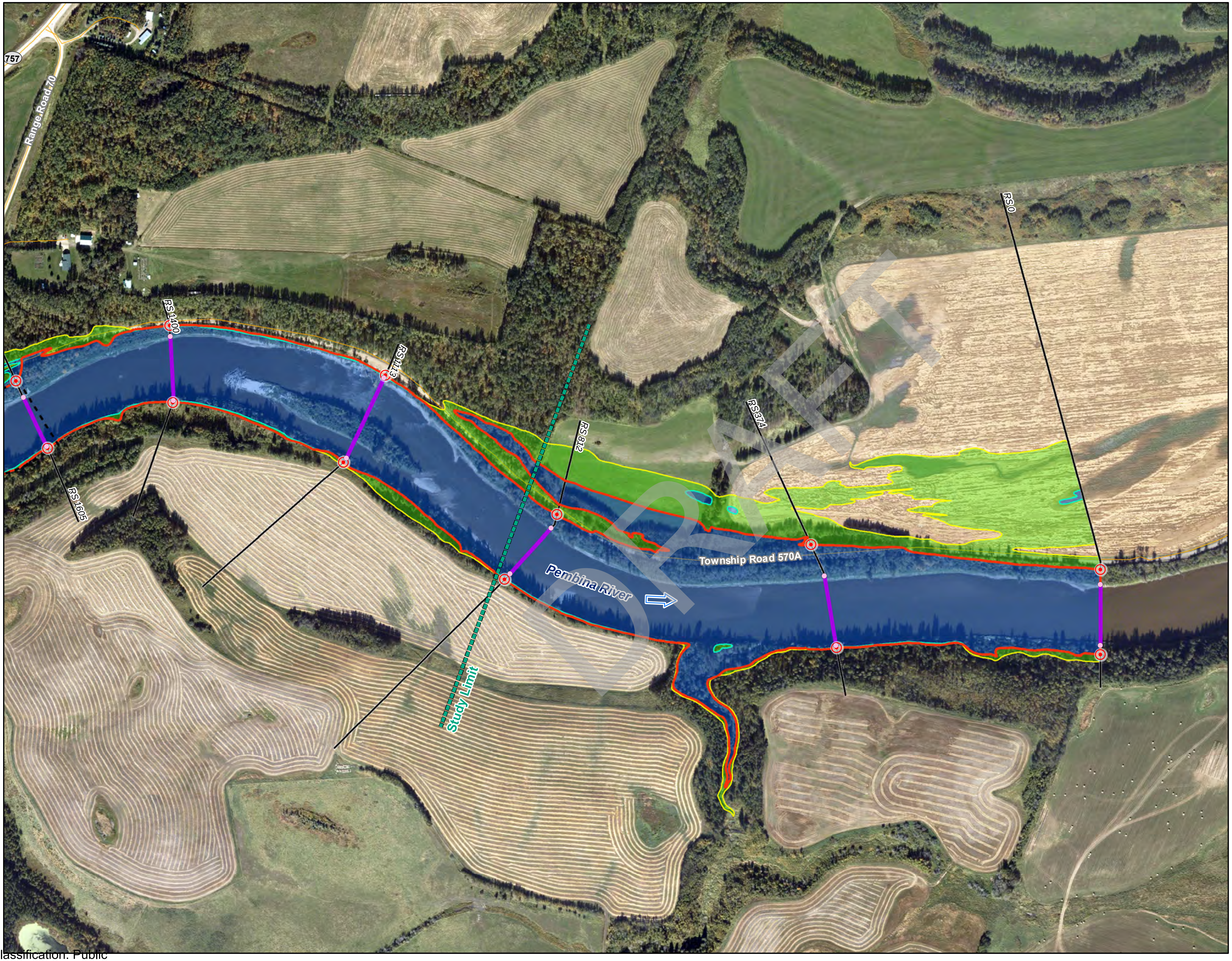
FLOODWAY CRITERIA MAP

SHEET 3 OF 4

Classification: Public

JY_P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_JY_Map_FloodwayCriteria.mxd

SHEET 3 ↑



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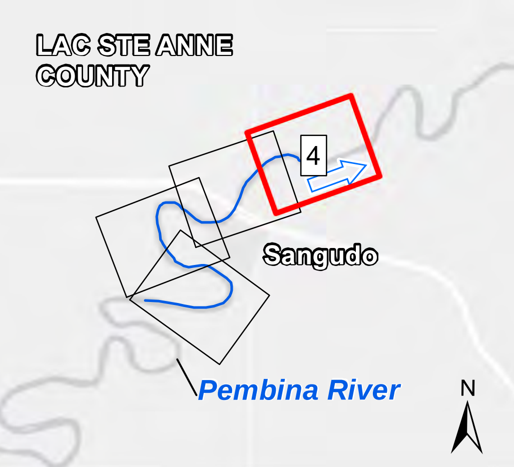

- FLOW DIRECTION
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- MODEL CROSS SECTION
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0 100 200 M
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Coordinate System: NAD 1983 CSRS 3TM 114; Vertical Datum: CGVD28 HTv2.0; Units: Metres

Engineer MMM	GIS JY	Reviewer PGV

Job: 1006073 | Date: 15-MAR-2022

SANGUDO FLOOD STUDY

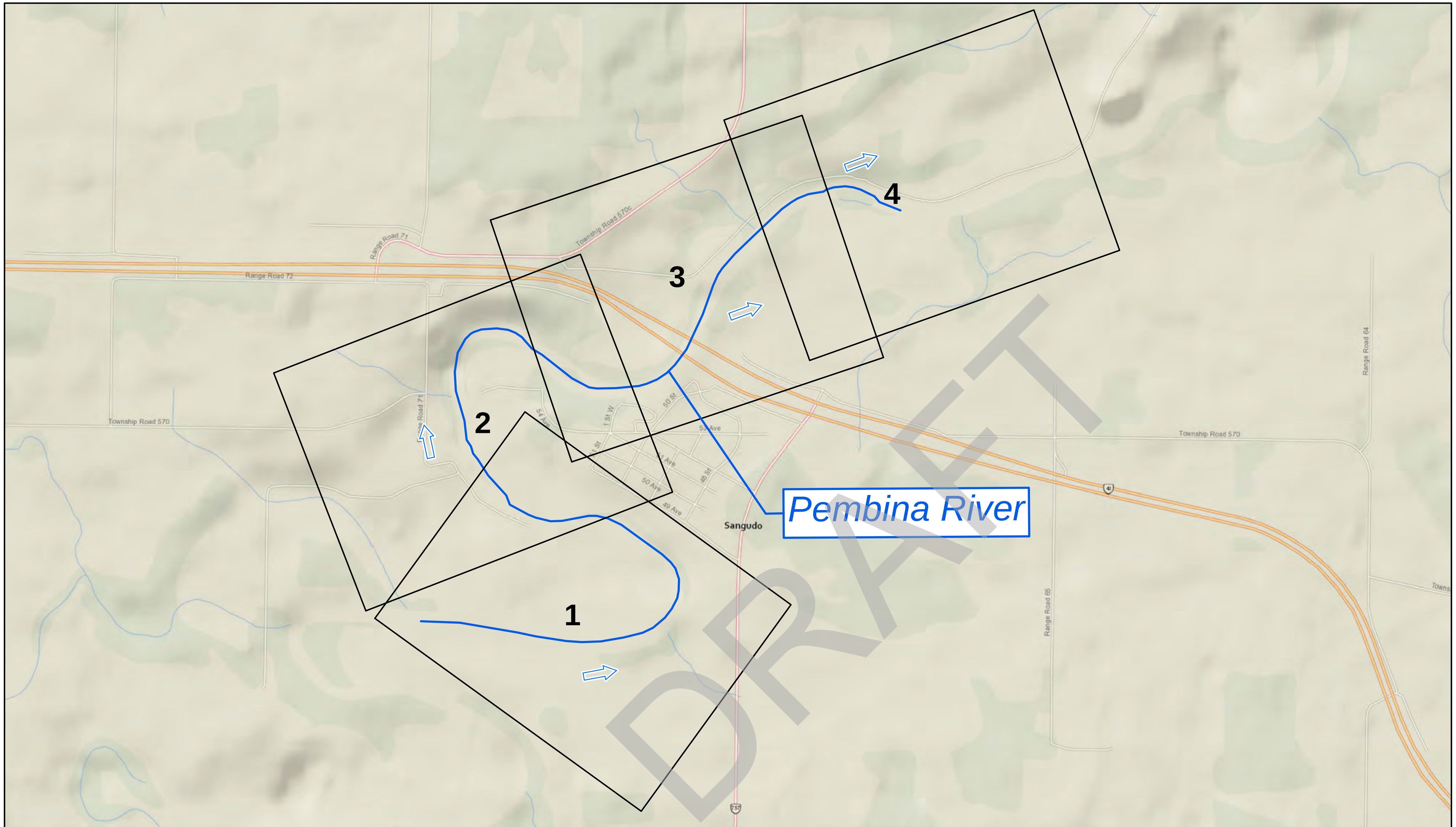
FLOODWAY CRITERIA MAP

SHEET 4 OF 4

JY_P:\Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_JY_Map_FloodwayCriteria.mxd

Appendix G

Flood Hazard Map



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FLOW DIRECTION
STUDY REACH
MAP SHEET

SCALE - 1:22,000
0 300 600 M
N

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

Engineer	GIS	Reviewer
MMM	JY	PGV

Job: 1006073 | Date: 04-MAR-2022

SANGUDO FLOOD STUDY

DESIGN FLOOD HAZARD MAP

INDEX MAP

Notes to Users:

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

Definitions:

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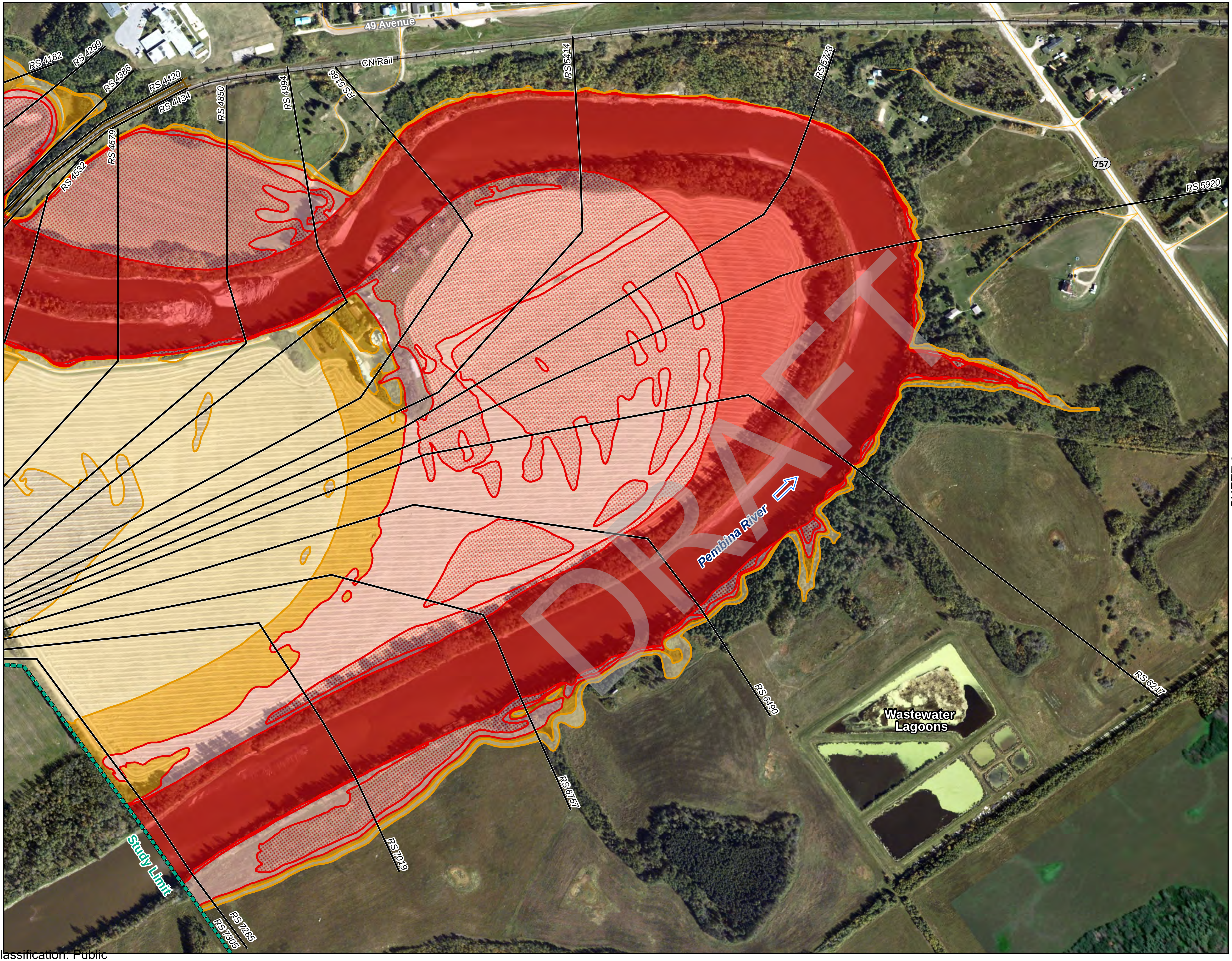
Classification: Public



SHEET 2 →

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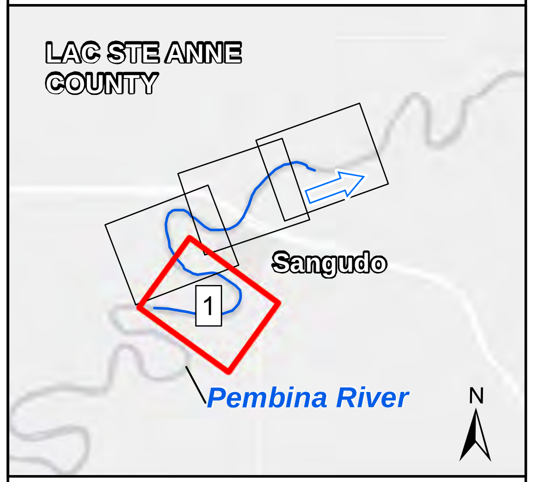


- FLOW DIRECTION
- BRIDGE
- MODEL CROSS SECTION
- LOCAL ROAD
- PROVINCIAL HIGHWAY
- RAILWAY
- STUDY LIMIT
- GOVERNING FLOODWAY
- HIGH HAZARD FLOOD FRINGE
- FLOOD FRINGE
- 200-YEAR FLOOD EXTENT
- 500-YEAR FLOOD EXTENT

SCALE - 1:5,000

0 100 200 M

N

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

Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 09-MAR-2022	

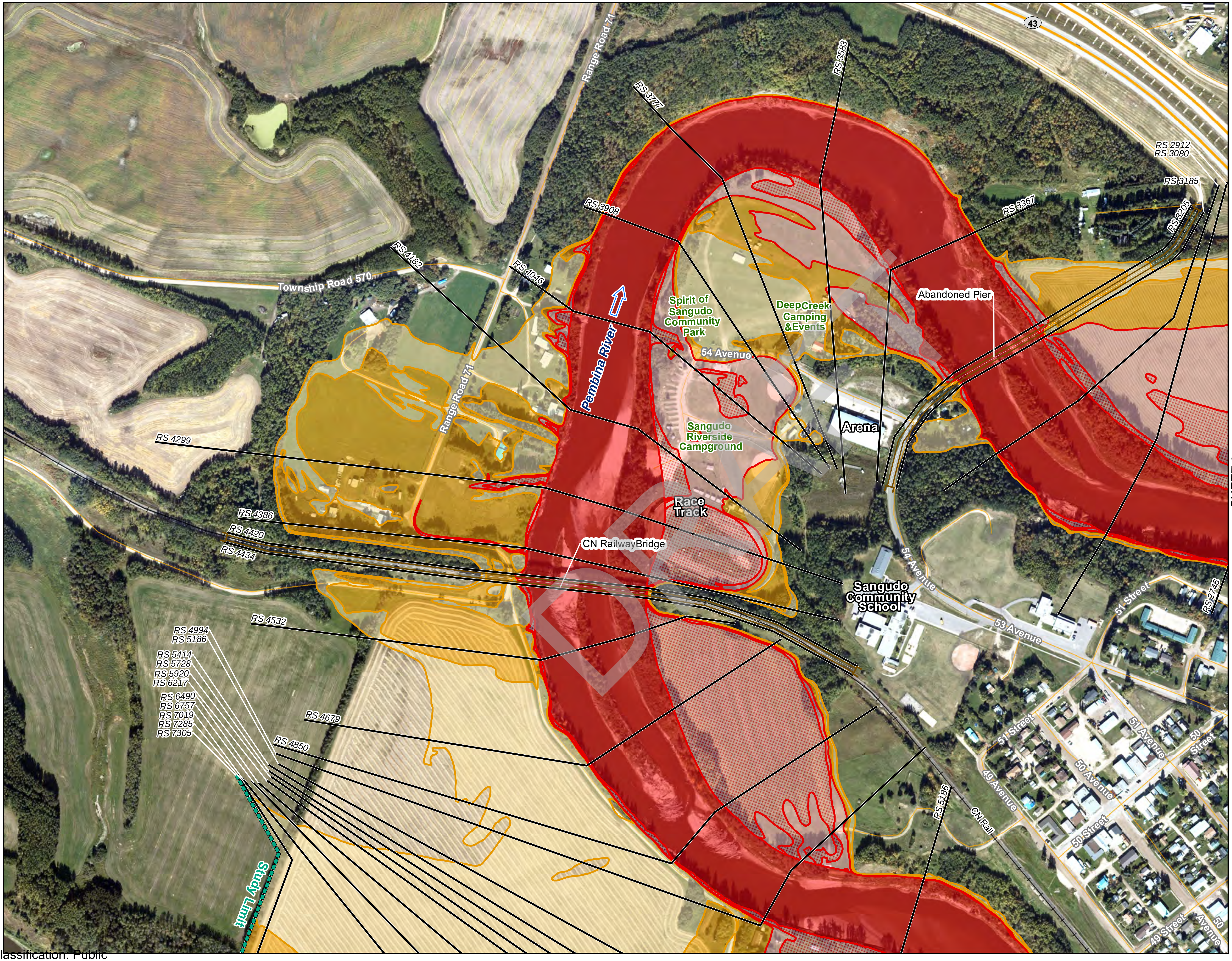
SANGUDO FLOOD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 1 OF 4

Classification: Public

JY, P:_Projects (Active)\1006073 Sangudo Flood Study\90 GIS\1006073_JY_Map_FloodHazard.mxd



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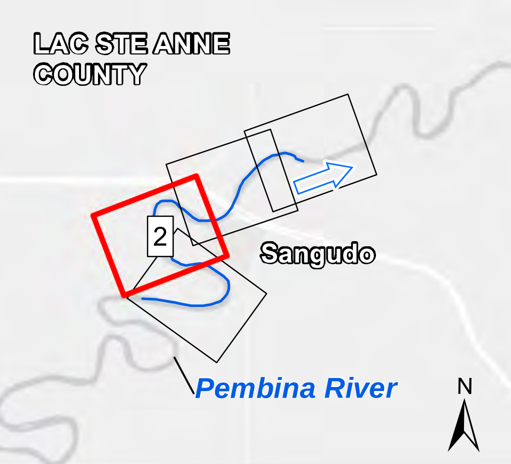

FLOW DIRECTION
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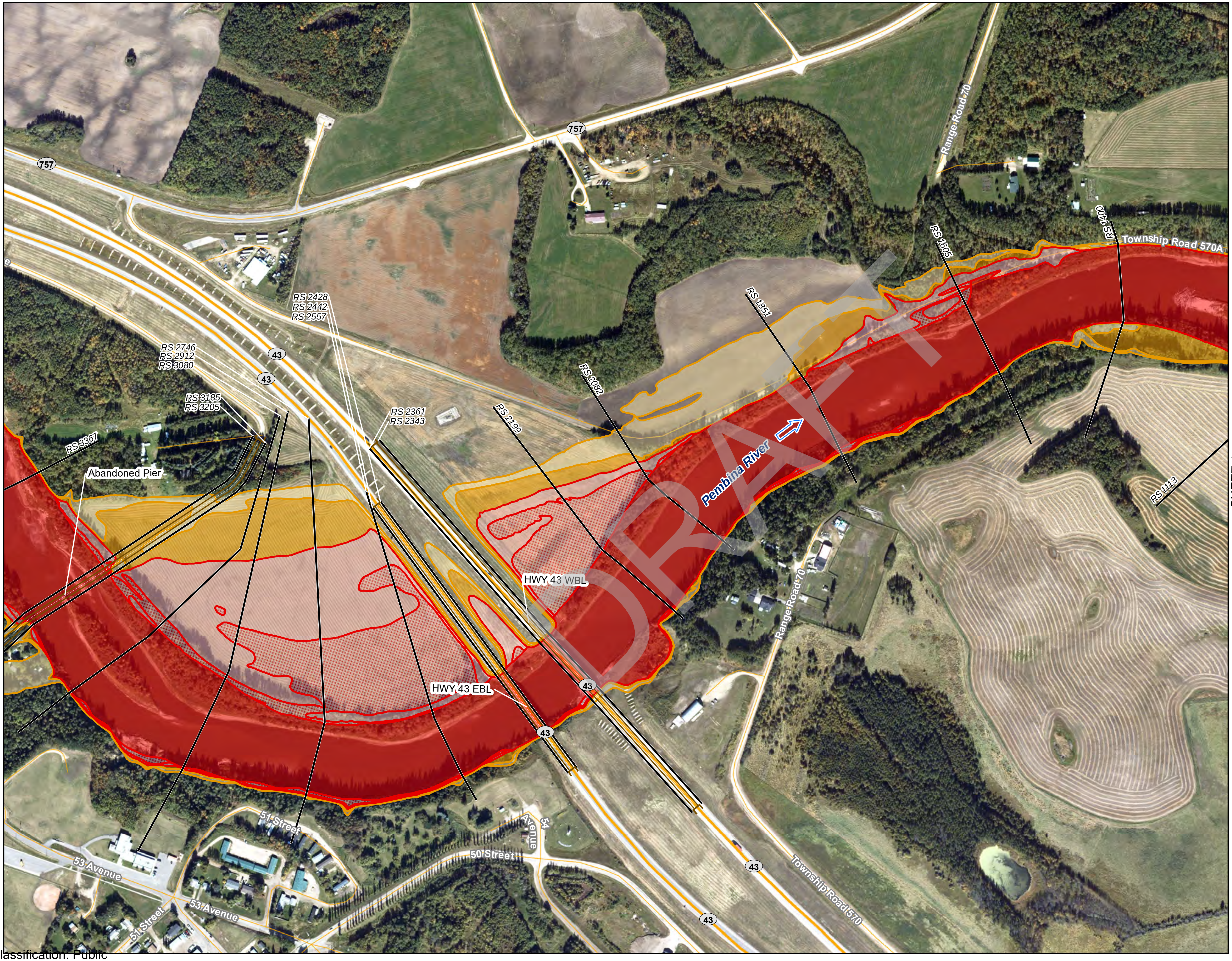
Engineer MMM	GIS JY	Reviewer PGV

Job: 1006073 | Date: 09-MAR-2022

SANGUDO FLOOD STUDY

DESIGN FLOOD HAZARD MAP

SHEET 2 OF 4



LAC STE ANNE COUNTY

3

Sangudo

Pembina River

N

FLOW DIRECTION
BRIDGE
MODEL CROSS SECTION
LOCAL ROAD
PROVINCIAL HIGHWAY
RAILWAY
STUDY LIMIT
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FLOOD FRINGE
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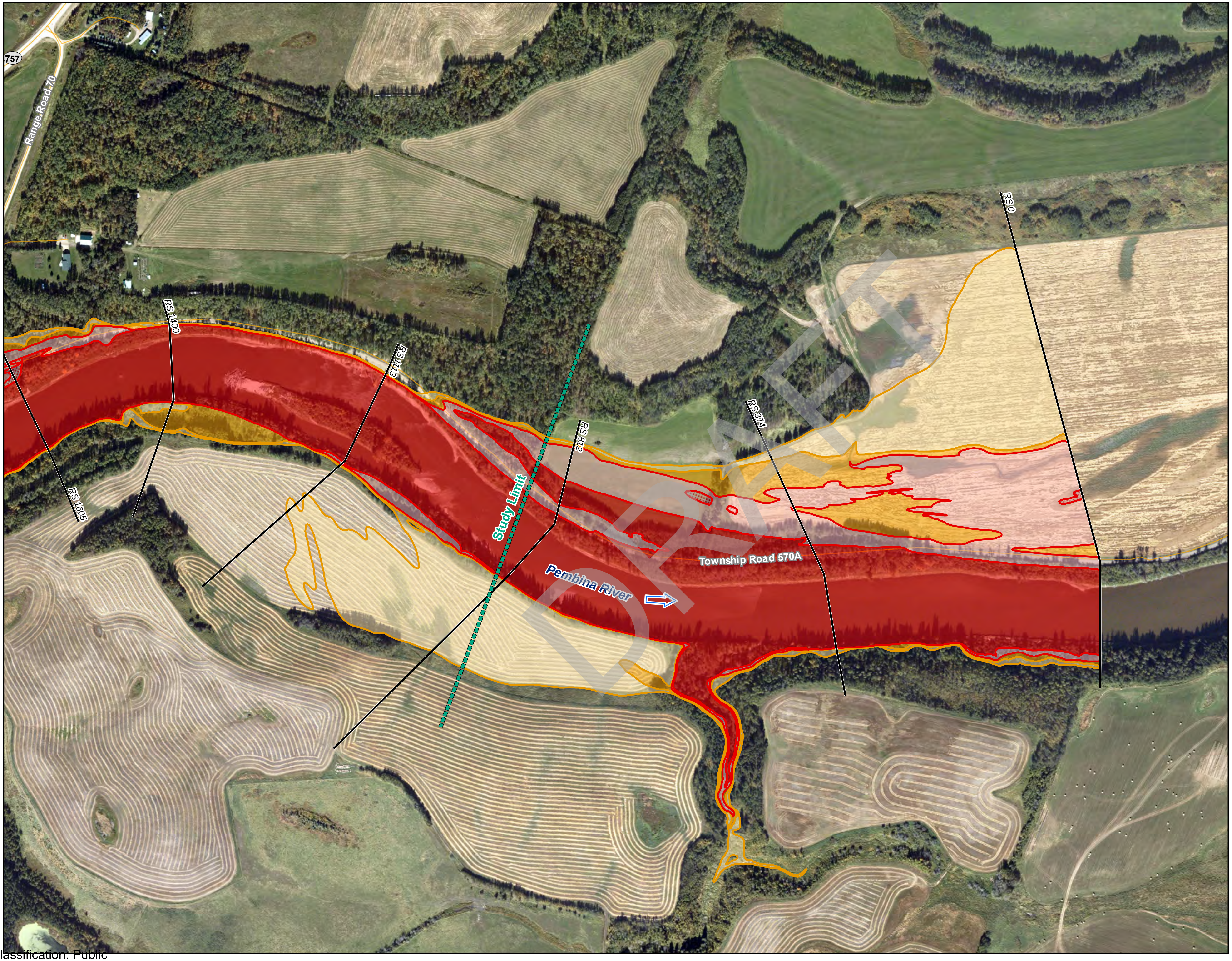
Engineer MMM	GIS JY	Reviewer PGV

Job: 1006073	Date: 09-MAR-2022

SANGUDO FLOOD STUDY
DESIGN FLOOD HAZARD MAP



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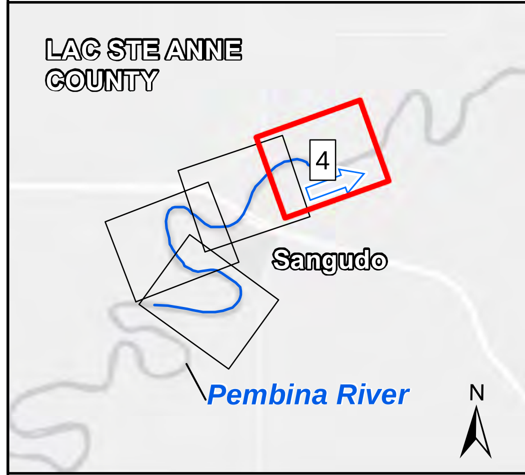

FLOW DIRECTION
BRIDGE
MODEL CROSS SECTION
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PROVINCIAL HIGHWAY
RAILWAY
STUDY LIMIT
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Engineer MMM	GIS JY	Reviewer PGV
Job: 1006073	Date: 09-MAR-2022	

SANGUDO FLOOD STUDY
DESIGN FLOOD HAZARD MAP

SHEET 4 OF 4

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