Madawaska-Edmundston International Bridge
Hydrology, Hydraulics and Scour Report.

For HNTB and Maine DOT
August 14, 2020

WIN 21736.00

MaineDOT Bridge Number 2399

NBDTI Asset E320

Madawaska-Edmundston International Bridge Hydrology, Hydraulics and Scour Report.

Summary-1.0 Hydrology:

The proposed Madawaska-Edmundston International Bridge crosses the Saint John River approximately ¼ mile upstream of the confluence with the Madawaska River, which flows south from Canada. The drainage are for this site is 5,976-mi^2 (15,500-km^2).

Hydrologic analysis was based on weighting the USGS gage data on the St. John River to reflect the drainage basin at the project site. The technique for weighting the gage and utilizing regression formula data was prescribed by USGS in their Regression analysis for peak flows on Maine Rivers. Flows on the Madawaska River at time of peak flow on the Saint John River were estimated by scaling USGS gage data just below the Fish River (1014000) and the Canadian gage at Grand Falls. Federal Emergency Management (FEMA) Flood Insurance reports were not available for this river.

The Saint John River typically floods in the spring due to the combination of heavy rainfall, snowmelt and ice. Tables 1 and 2 summarize design flows at the project. Table 2. Summarizes flows within the model domain: in the Madawaska River, in the St. John River and the combined St. John and Madawaska River flows downstream of the Madawaska River junction. Note that the Madawaska River flood peak would occur prior to the St. John so flows on the Madawaska River do not represent peak flows on that river.

St. John River at Madawaska-Edmundston, US of Madawaska River	SI Units	Metric Units
Drainage Area	5976 sq mi.	15500 km^2
Design Discharge, Q50	163,120 cfs	4620 cms
Check Discharge, Q100	176,980 cfs	5020 cms
Scour Check Discharge, Q500	208,780 cfs	5920 cms
Ordinary High Water (Q1.1)	67,985 cfs	1930 cms
Flood of Record, Q250, 2008	190,320 cfs	5393 cms

Table 1. Summary of Design Hydrology, Madawaska-Edmundston International Bridge

	Flow on St. John River, cfs	**Flow on Madawaska River, cfs**	**Combined flow, cfs**
2008	190320	38000	228320
1-year	67895	13000	81000
2-year	87840	16500	104340
5-year	113330	21100	134430
10-year	129360	27000	156360
25-year	148900	28000	176900
50-year	163120	32400	195520
100-year	176980	35000	211980
500-year	208780	41500	250280

Table 2. Summary of Model Flows on St. John River upstream and downstream of confluence with Madawaska River.

Summary-2.0 Hydraulics:

Hydraulic modeling was conducted to develop project design parameters associated with existing and proposed conditions. Modeling provided a basis for water level projections, velocity distributions and directions and scour variables. An SMS –SRH2-D model for 2-Dimensional flow analysis was utilized for this project. The 2-dimensional flow model was selected to account for:

- Proposed bridge at angle to river flow – difficult to simulate with 1-dimensional model.
- River confluence downstream of project site could impact distribution of flow.
- Islands downstream of project site also impact flow distribution.

This bridge is not subject to pressure flow or overtopping. Model options for incorporation of ice cover or ice jam data are limited, but include options such as increasing the Mannings “n” value, increasing debris on piers via pier width, or addition of artificial debris in other locations. Due to model limitations, ice is included in discussion only and has not been modeled.

The model was based on a terrain model for the river, islands, bridges and overbanks. Two sources of elevation data were incorporated into the model, including project survey where available, and DEMs (Digital Elevation Models) published by the Province of New Brunswick and the State of Maine for overbank areas not included in the project survey.

Figure 1 below shows the model layout including islands downstream, Madawaska River junction, existing bridges and proposed bridge alignment (red). The model computes water surface elevation for selected storms, and velocity magnitude and direction.

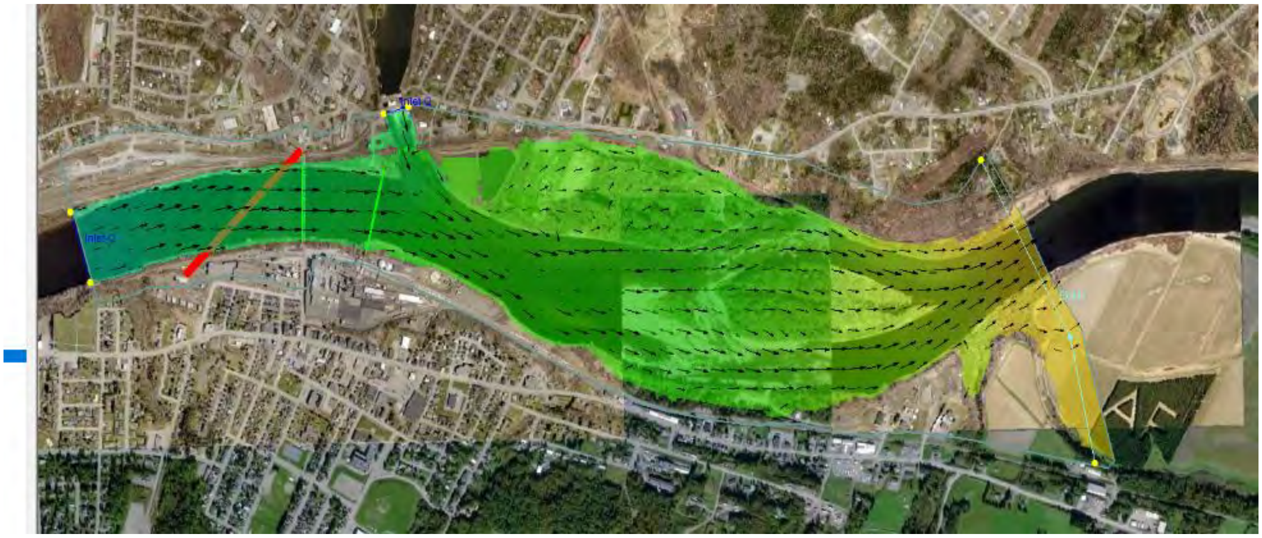

Figure 1. Model layout. Note island downstream, river junction with Madawaska River. Utility Bridge and Existing Bridge shown in green, proposed new alignment in red.

Bathymetric data was collected for approximately ½ mile upstream and downstream of the existing bridge and was used for model elevation data within that area. Downstream of the full channel bathymetric data, the model includes both surveyed cross section data and the DEMs.

The goals of the hydraulic analyses include evaluating potential existing vs proposed water levels, evaluation of potential water levels for setting freeboard, and calculation of velocities and scour variables including those needed for design of abutments and piers.

The model includes the Twin Rivers utility bridge downstream of the project site, the existing International Bridge, the proposed bridge and possible construction phase access roads. The proposed bridge includes 5 piers. Summarizing model configurations for hydraulic analysis (not full scour analysis):

- Existing conditions, with utility bridge
- Proposed 6 span bridge with utility bridge
- Proposed 6 span bridge with existing bridge, utility bridge and construction phase access roads and possible trestle construction bridge .

The Bridge Design Guide (BDG) states that major riverine bridges must provide a minimum of 4 feet of freeboard over the 50-year storm event. In both the existing and proposed conditions, the bridge provides over 30 feet of freeboard at the 50-year storm, to the low chord of the respective structures. This is due to the need to meet vertical clearance requirements over the railroad as well as the high banks of the river. Note that the low chord of the proposed structure is similar, and generally higher than the existing bridge. The existing bridge is well above flood stage even with ice cover or potential ice jamming.

Table 3 summarizes hydraulic data.

Madwaska-Edmundston International Bridge		Headwater Just Upstream of Each Bridge		Water Surface Elevation at Upstream Model Boundary	
		Existing Structure	Proposed Structure$^{@}$	Existing Structure	Proposed Structure
		928 -ft, 4 - Span, through Truss, LC 499	6- span, LC 503.5	928 -ft, 4 - Span, through Truss	6-span
Total Area of Waterway Opening	ft2	20,375	18,650		
Headwater elevation @ Q1.1 [1]	ft	455.3	455.7	456.8	456.8
Headwater elevation @ Q2 [1]	ft	458.8	459.2	460.1	460.1
Headwater elevation @ Q10	ft	464.9	465.1	466.0	466.0
Headwater elevation @ Q25	ft	466.3	466.5	467.4	467.4
Headwater elevation @ Q50	ft	467.4	467.6	468.6	468.6
Headwater elevation @ Q100	ft	468.4	468.6	469.6	469.6
Headwater elevation @ Q500	ft	470.5	470.8	471.9	471.9
Freeboard @ Q50	ft	Approx 31.9	35.9		
Freeboard @ Q100	ft	Approx 31.0	34.9		
Flood of Record, 1/25/2008	ft	469.1	469.3	470.6	470.6
Velocity @ Q1.1	fps	6.7	7.6		
Velocity @ Q2	fps	7.2	7.8		
Velocity @ Q10	fps	8.1	8.5		
Velocity @ Q25	fps	8.8	9.3		
Velocity @ Q50	fps	9.3	9.7		
Velocity @ Q100	fps	9.8	10.1		
Velocity @ Q500	fps	10.7	11.0		

[1] Note that the Edmundston gage has recorded higher water levels for 1-2 year flows, likely due to ice jamming. The gage shows up to 462.5’ for this level of flow, likely due to ice.

[2] Elevations listed are highest across the bridge section. Because the bridge is skewed to flow, these elevations generally occur towards the southern or U.S. bank of the river.

Table 3. Summary of Hydraulic Data

Summary-3.0 Ice Jams

NBDTI commissioned an ice study for the project site. According to this ice study, historically, ice jams have broken up with as little as 7000 cfs or as high as a 1-2-year flow. Higher flows are not likely to include ice jams, as the ice will already have broken up before those high flows are reached on the river.

Based on water levels recorded by the Canadian river gage in Edmundston and flows recorded by USGS at Fort Kent it is likely that potential 1-2 year flows with ice may reach elevation 462.5 ft (141m), more typical of a 25-year type of flow.

Summary-4.0 Scour Analysis

Scour was analyzed for the proposed 5-pier, 6-span bridge. Data from model SMS SRH-2D was used to generate scour variables, including water depth, water elevation, velocity, and Froude number at selected locations. The Bridge Scour option in SMS SRH-2D was used to generate composite data such as flow through a section, bottom width and section area. Geotechnical data was furnished for final design scour analysis. Contraction scour is estimated as minimal (0.7' for the 100-year storm and 1.0' for the 500-year storm). Potential pier scour is computed as 10.5 to 15.4' depending on pier number and storm frequency. No abutment scour is expected, due to location of abutments and planned embankment protection with heavy riprap. Data on long term bed changes was not available but was assumed to be approximately 2' of degradation.

Summary- 5.0 Notes on Project Design Related to Climate Change and Water Levels.

The proposed bridge is located on the Saint John River, approximately 250 miles upstream of the river's mouth in Saint John, NB and is at least 120 meters above sea level. The river is deeply incised, with tall steep banks. These factors mitigate potential impacts of sea level rise or higher flows due to more intense storms. The project is designed such that high water levels will not rise due to the project. The proposed bridge has significant freeboard above normal, high and ice affected water levels. While there is some basis for anticipating sea level rise at sites that may be impacted by ocean levels, there is little information available to predict the potential for climate change effects on a river such as the Saint John. Providing adequate freeboard and designing for no change compared to the existing bridge geometry are strategies employed at this project.

Madawaska-Edmundston International Bridge, Preliminary Design Report, Hydrology, Hydraulics and Scour.

This report details investigations and results of preliminary engineering analysis related to hydrologic, hydraulic and scour for replacement of the Madawaska-Edmundston International Bridge on the St. John River.

1.0. Introduction

It is proposed by Maine Department of Transportation (MDOT) and New Brunswick Dept. of Transportation and Infrastructure (NBDTI) to replace the existing international bridge that crosses the St. John River between Madawaska in Maine and Edmundston in New Brunswick. The bridge connects two border crossing stations administered by the respective governments of the United States and Canada. This multi-faceted project includes not only the new bridge, but also approach roadways, and a new border station on the United States side. The project team thus includes not only the two state/provincial transportation agencies, but also the U.S. General Services Administration (GSA), Public Services and Procurement Canada (PSPC), U.S. Customs and Border Protection (CBP), Canada Border Services Agency (CBSA). The project design team includes EXP for the design of the Canadian Port of Entry site redevelopment,, HNTB Corporation for bridge and approach roadway design, and MPdL Studio for design of the new LPOE (Land Port of Entry). Northstar Hydro, Inc. is subcontracted to HNTB specifically to analyze the hydrology, hydraulics and scour potential of the existing and proposed bridges. Additional project stakeholders include the Twin Rivers Paper Company, The City of Edmundston, The Town of Madawaska, CN Railroad and Maine Northern Railways, all of which are directly in the path of bridge reconstruction.

Site location and project understanding:

The project is located at the U.S. Canadian border in Madawaska, adjacent to the neighboring City of Edmundston, NB. The project is overseen by a collaboration of State, Provincial and Federal stake holders from both sides of the border. It is our understanding that approximately twelve alternative sites were considered for the final border crossing bridge, with the selected option being a bridge crossing the St. John River at an angle, upstream of the current location. It is also our understanding that the current bridge has reached the end of its useful life and will be removed once the new bridge is complete and open to traffic.

The existing bridge crosses the St. John River essentially perpendicular to flow. The bridge is supported on two abutments that are well above typical flood levels and three piers in the river channel. The bridge deck is also well above typical flood levels. A privately-owned utility bridge is located approximately 900 feet downstream from the existing international bridge and will not be removed or replaced as part of the project. Water levels in the St. John River are also impacted by several islands in the channel downstream of the Madawaska River. The project area is shown in Figure 2.

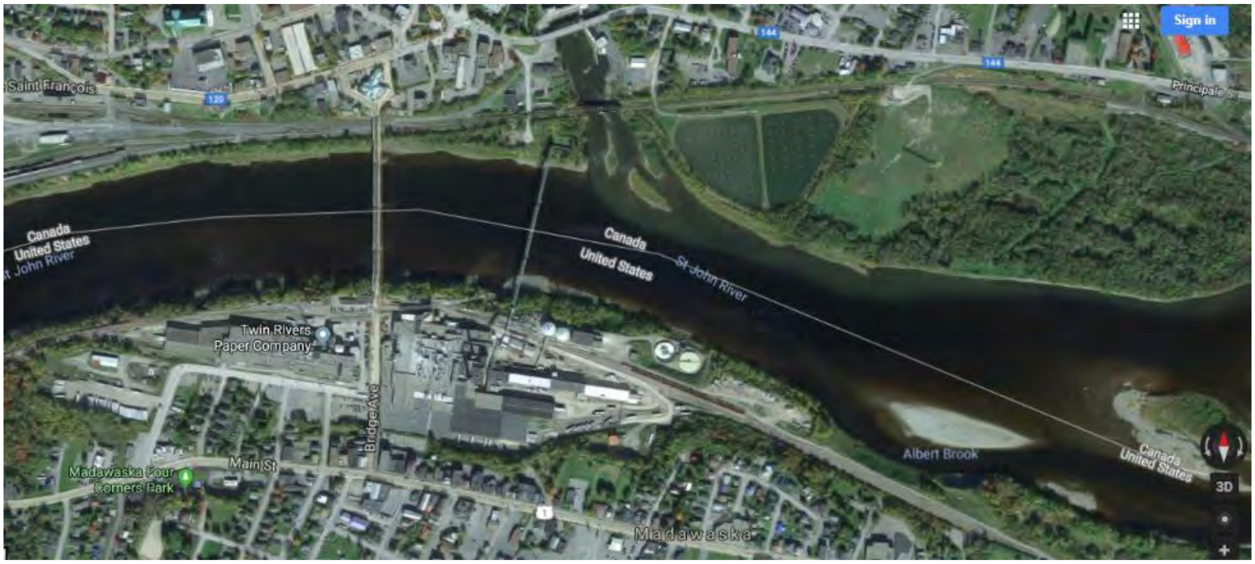

Figure 2 Existing bridge and utility bridge at project site. Note confluence of Madawaska River downstream, existing utility bridge, and several islands in downstream channel.

2.0 Existing Data Review:

HNTB furnished the following study information developed prior to this phase of the project:

- Appendix E of the Final Draft Feasibility Report, including Conceptual Plans, dated January 19, 2018
- Hydrology Report prepared by MaineDOT for Madawaska-Edmundston International Bridge, 21736, March, 2017
- Madawaska/Edmundston International Bridge and Border Crossing, Feasibility and Planning Study. Final Draft, Version 2.0. April 2018

This report is based on the following plan set:

- HNTB International Bridge Saint John River Madawaska, ME Edmundston, NB. 60% plans, 6/3/2020. Pier design sheets 63 through 67

The following existing data *were collected and reviewed*:

- Records from Hydrometric Data Site - 01AD004 Saint John River at Edmundston from Government of Canada, water level and flow records. https://wateroffice.ec.gc.ca/report/real_time_e.html?stn=01AD004
- FEMA Flood Study – This area of the St. John River is listed as Zone A by FEMA with no flood elevation determined.
- Anecdotal historical flood data from Madawaska and Edmundston were collected as the study progressed.
- USGS Gaging data for the station near Fort Kent, https://waterdata.usgs.gov/me/nwis/uv?site_no=01014000
- *E-mail furnished by HNTB from Opus Engineering of Canada offered assistance in establishing flow curves for the Madawaska River.*

- Project hydrology report prepared by Maine DOT for the St. John River. Note that this report did not cover the Madawaska River.
- A GIS database of information was compiled to assist in model building. The database includes: project survey including hydrographic survey, LiDAR-based DEM data from Canada and Maine, and aerial photography.

The project site includes complex river hydraulics, steep river banks, and urban landscapes on either side of the river. The general proposed alignment corridor is shown in Figure 3 and the final alignment is shown in figure 3a.

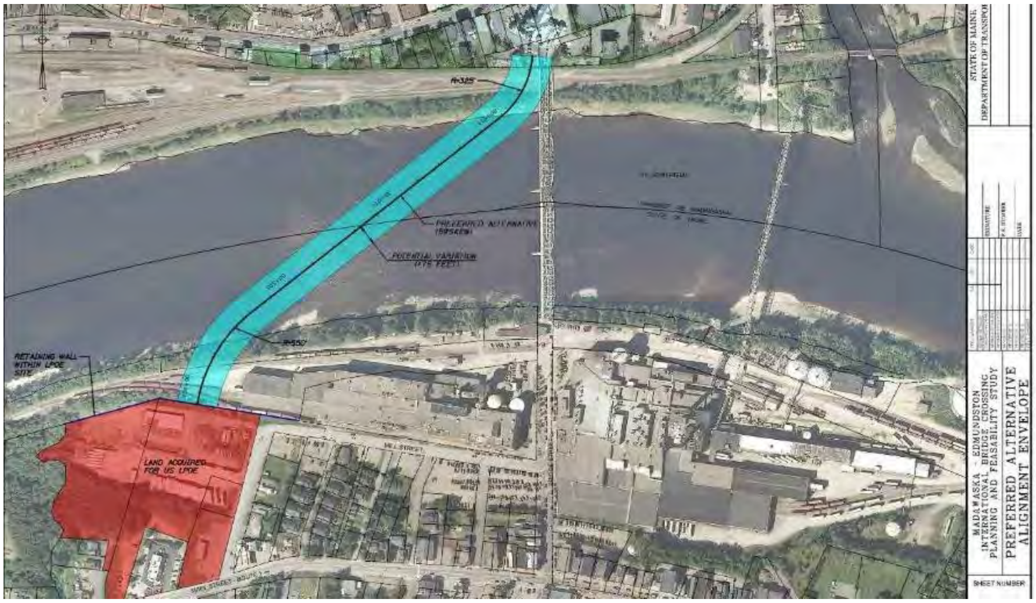

Figure 3. Preferred alternative from Planning Report.

Figure 3a. Final alignment for new bridge

3.0 Site Visit, Project Meetings and Survey

NHI attended a project kick off meeting at MaineDOT on August 28, 2018 where team members were introduced and project components discussed. NHI also has attended regular internal team meetings at HNTB to review project progress.

NHI and HNTB conducted a site visit on October 29, 2018. The following photos (figures 4 through 14) were taken during the site visit.

Figures 15 and 16 were furnished by Twin Rivers Paper via email.

Figure 4. Looking downstream at International Bridge and Twin Rivers Utility Bridge

Figure 5. Looking downstream towards islands in channel from junction with Madawaska River.

Figure 6. Twin Rivers Utility Bridge and downstream river reach

Figure 7. Looking upstream from Utility Bridge towards International Bridge.

Figure 8. Looking upstream from deck of International Bridge

Figure 9. Example of steep bank and riprap protection on U.S. side of St. John River, upstream of International Bridge

Figure 10. Steep bank below Abutment on Canadian side of International Bridge.

Figure 11. Piers of International Bridge, note triangular nose and apparent ice damage.

Figure 12. Twin Rivers Utility Bridge with rounded piers.

Figure 13. Looking upstream into Madawaska River outlet. Note Railroad bridge and dam.

Figure 14. Looking across the Madawaska River mouth towards downstream reach of St. John River.

Figure 15. Water level on May 1, 2008 at Twin Rivers, South (US) side of river downstream of utility bridge.

Figure 16. Water and ice at Twin Rivers manhole, spring, 2019

Survey Notes

Project survey related to river hydrography is shown in figure 17. Within the blue box, detailed channel and bank survey was collected. Further downstream, only river cross sections were collected as shown with the blue lines.

Figure 17. Survey related to River geometry.

Project Units and Datum:

Models and computations were run in English units with datum set at NAVD 1988. Key findings are listed in metric and English. Project datum is NAVD 1988. Correspondence with MaineDOT and NBDTI indicates that this datum is approximately equal to the datum used for the Canadian gaging station, CGVD 1928. (email Tracy McDonald, NBDTI, 12/6/18)

4.0 Hydrologic Analysis

The proposed Madwaska-Edmundston International Bridge will cross the Saint John River approximately ¼ mile upstream of the confluence with the Madawaska River, which flows south from Canada. The drainage area for the project site is 5,976 mi^2.

Maine DOT provided project hydrology for the St. John River. The New Brunswick Department of Transportation and Infrastructure (NBDTI) assisted with interpretation of St. John River gage data at Edmundston. Maine DOT also assisted with estimation of flows on the Madawaska River at times of high flows on the St. John River.

Information used to develop project hydrology, water level and model calibration data include:

- MaineDOT Hydrologic Analysis, <u>21736 Madawaska International Bridge #2399 Hydrology,</u> From Charles Hebson to Nate Howard, MaineDOT, 3/23/17

    - Analysis based on USGS Regression Formulas for Maine and Analysis of Fort Kent Gage for St. John River

- USGS Gaging Station # 0101400, Fort Kent Gage on St. John River https://waterdata.usgs.gov/usa/nwis/uv?01014000

- Canadian Gaging Station at Edmundston on St. John River (*Datum of Canadian Gage is CGVD1928)*. http://wateroffice.ec.gc.ca/report/historical_e.html?stn=01AD004

Hydrologic analysis was based on weighting the USGS gage data on the St. John River to reflect the drainage basin at the project site. The technique for weighting the gage and utilizing regression formula data was prescribed by USGS in their Regression analysis for peak flows on Maine Rivers. Flows on the Madawaska River at time of peak flow on the Saint John River were estimated by scaling USGS gage data just below the Fish River (1014000) and the Canadian gage at Grand Falls. Detailed study Federal Emergency Management (FEMA) Flood Insurance reports were not available for this river.

The Saint John River typically floods in the spring due to the combination of heavy rainfall, snowmelt and ice. April of 2019 saw a flood of this type, where water levels with ice reached 466.4' (142.2m) at the Edmunston gage, but the river flow peaked after the high was recorded at approximately a 25-year level. April/May of 2018 also saw high water levels, and high flow, but a record of flow level at the Edmundston gage was not available through the gage website.

Flow information for the Saint John River is included in Table 4 below, which details flow frequency distribution at project site and at the Fort Kent Gage, 19 miles upstream of the site on the St. John River. Figure 18 shows the two locations.

Figure 18. Fort Kent Gage and Madawaska/Edmundston Bridge Location

Location	St. John River at Madawaska-Edmundston, US of Madawaska River		USGS Gate at Fort Kent	
	SI Units	Metric Units	SI Units	Metric Units
Drainage Basin	5976 mi$^{2.}$	15,500 km^2	5676 mi^2	14,722 km^2
Frequency, years	Flow, cfs	Flow, cms	Flow, cfs	Flow, cms
1.1	67,985	1,930	57,280	1,623
2	87,840	2,489	84,590	2,393
10	129,360	3,665	124,500	3,527
25	148,940	4,219	143,300	4,059
50	163,120	4,620	156,900	4,448
100	176,980	5,020	170,200	4,822
500	208,780	5,920	200,700	5,686
Flood of Record, Q250, 2008	190,320	5,393	183,000	5,184
April, 2019			137,000	3,881
May, 2018			127,000	3,598

Table 4. Summary of St. John River Flows at Madawaska (project site) and USGS Gage at Fort Kent.

The Canadian Water Office maintains a gaging station in Edmundston just upstream of the junction of the Madawaska River. The record for this station includes measured flows for the period 1968 to 1980 and water levels for the period 1980 to present. *The datum for these measurements is Canadian Geodetic Vertical Datum which coincides with NAD83.*

The station location is shown in Figure 19.

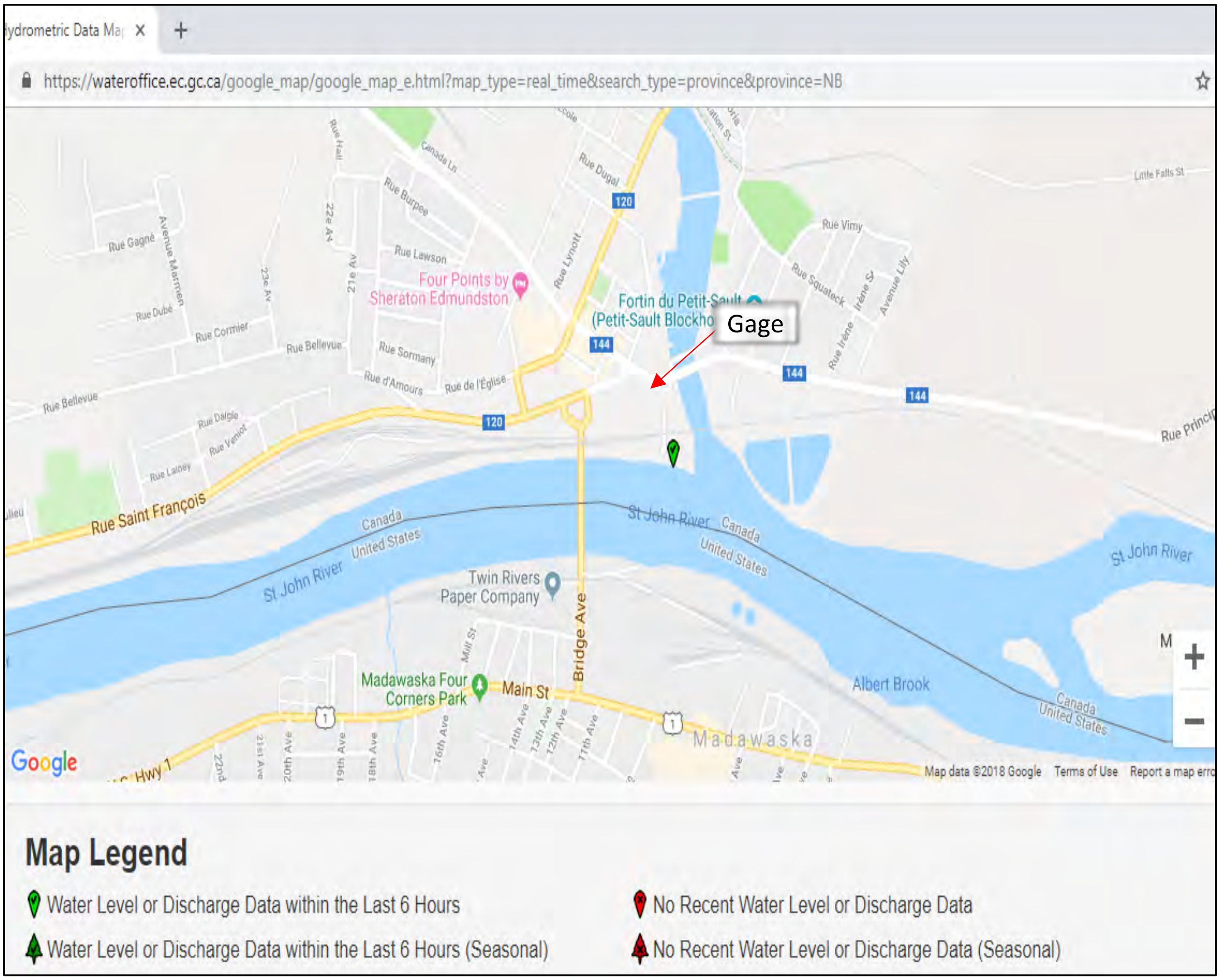

Figure 19. Location of Canadian Gage at Edmundston

Table 5 lists highest water levels for this gaging station. Note that the table is arranged from highest water level to lower water levels. The Canadian gaging station publishes water levels in meters at CGVD1928 datum which is approximately equal to NAVD88. Table 5 also includes rate of flow at the USGS gaging station at Fort Kent. Several of these flows were identified as being approximately equal to selected storm frequencies based on the gage flow-frequency curve in table 4. The data indicates likely effect of ice on water levels at this location as higher flows do not necessarily coincide with higher water levels.

Canadian Gaging Station at Edmundston Data, DS of International Bridge, on Canadian side of St. John River.			**Flow and Frequency at USGS Gage at Fort Kent**	
Date	water level, CGVD1928	Water Level, CGVD, Ft	Gaging Station Flow Record, cfs	Estimated Frequency for Fort Kent Flows, years
5/1/2008	142.888	468.67	183000	250+/-
4/19/19	142.2	466.4	45000	<1, ice
4/23/19	141.0	462.5	137000	25
4/19/1983	141.041	462.61	141000	25
4/11/1991	140.700A	461.50	87400	2
5/6/2011	140.506	460.86	114000	
5/2/1984	140.194	459.84	110000	
4/13/1993	140.047	459.35	86600	2
12/27/1990	140.011	459.24	85400	2
4/27/2005	139.953	459.05	116000	
4/8/2009	139.736	458.33	96000	
4/24/1992	139.498	457.55	102000	
4/18/1994	139.469	457.46	144000	25
4/7/2010	139.09	456.22	90700	
4/19/2002	139.046	456.07	86600	2
4/4/1986	138.886	455.55	63000	1
4/27/2007	138.881	455.53	88800	2
2/26/1981	138.838	455.39	88900	2
4/17/2006	138.61	454.64	74700	
4/28/1985	138.541	454.41	78200	
4/21/2004	138.436	454.07	80500	
1/5/2015	138.27	453.53	68100	
1/14/2016	138.196	453.28	63600	
12/13/1980	138.12	453.03	56000	1.1

Table 5. Summary of Edmundston Gage Highest Water Levels and Corresponding flow at USGS Fort Kent Gage.

USGS published a report detailing flood flows and levels for the flood of April and May, 2008 in Northern Maine. The 2008 USGS report identified a flow of 183,000 cfs at Fort Kent (approximately 250-year storm) and a flood level of 468.89’ downstream of the existing International Bridge at Right Edge of Water (REW).

Additional flood marks were noted by Twin Rivers Paper workers at a Manhole that is downstream of the junction with the Madawaska River and on the United States side of the river. Levels of 466.1 and 467 are noted. The flood events were identified as "recent" but not dated.

The Madawaska River joins the St. John River just downstream of the project location. The contributing flow from the Madawaska River during peak flooding on the St. John River was computed by examining data from gages on the St. John River and estimating the contributing flow of the Madawaska River at time of peak on the St. John. Table 6 summarizes flows within the model domain: the Madawaska River, the St. John River, and the combined St. John and Madawaska River flows downstream of the Madawaska River junction. Note that the Madawaska River flood peak would occur prior to the St. John so flows on the Madawaska River do not represent peak flows on that river.

Peak Flow Frequency on St. John River	**Flow on St. John River, cfs**	**Flow on Madawaska River, cfs**	**Combined flow, cfs**
2008, FOR	190320	38000	228320
1-year	67895	13000	81000
2-year	87840	16500	104340
5-year	113330	21100	134430
10-year	129360	27000	156360
25-year	148900	28000	176900
50-year	163120	32400	195520
100-year	176980	35000	211980
500-year	208780	41500	250280

Table 6. Summary of Model Flows on St. John River upstream and downstream of confluence with Madawaska River at time of peak flow on St. John River.

Figure 20 shows the two rivers and the project site.

Figure 20. Aerial view of existing bridges and Madawaska River junction

Table 7 summarizes design flows for the project bridge.

St. John River at Madawaska-Edmundston, US of Madawaska River	SI Units	Metric Units
Drainage Area	5976 sq mi.	15500 km^2
Design Discharge, Q50	163,120 cfs	4620 cms
Check Discharge, Q100	176,980 cfs	5020 cms
Scour Check Discharge, Q500	208,780 cfs	5920 cms
Ordinary High Water (Q1.1)	67,985 cfs	1930 cms
Flood of Record, Q250, 2008	190,320 cfs	5393 cms

Table 7. Summary of Design Hydrology, Madawaska-Edmundston International Bridge

5.0 Low Water Level Information, Average Monthly Water Levels

Average Monthly Water Levels at Piers of New Bridge.

The gage at Edmundston has recorded water levels on the St. John River from 1980 to the present. Data from 1980 to 2016 was downloaded and analyzed to estimate average monthly water levels at the gage. The SMS hydraulic model was used to estimate difference in water level from the gage to each pier for the new bridge. Estimated water levels for each month were compiled into tables showing Average, Highest and Lowest water level at each pier.

Results are included in the following pages. Table 7a is data for the gage, in feet and meters. Table 7b is elevations at each pier in feet and table 7c is the same data in meters.
In addition to the analysis NHI did with the Edmundston gage data, Vincent Balland from NBDTI provided the graphic in figure 21 below to illustrate plotted typical water levels. This information is to be used to assist in navigational opening.

Month	**Flow**		**Elevation at Edmundston Gage (near Utility Bridge)- 1980 to 2016**							
	cfs	**cms**	Highest Recorded		Lowest Recorded		Average Monthly Leve		Range	
			Meters	Feet	Meters	Feet	Meters	Feet	Meters	Feet
Jan	5220	148	139.0	456.0	134.4	440.9	135.8	445.4	4.6	15.0
Feb	3600	102	138.8	455.4	134.6	441.6	135.4	444.3	4.2	13.8
Mar	6780	192	137.8	452.0	134.6	441.3	135.4	444.0	3.3	10.7
Apr	39400	1116	141.8	465.0	134.8	442.0	136.8	448.7	7.0	23.0
May	24700	700	142.9	468.7	134.7	441.8	136.3	447.0	8.2	26.8
Jun	10100	286	137.2	449.9	134.6	441.4	135.3	443.7	2.6	8.5
Jul	7890	224	137.3	450.4	134.4	441.0	135.1	443.0	2.9	9.5
Aug	6370	180	138.5	454.2	134.3	440.5	134.9	442.6	4.2	13.8
Sep	4380	124	138.0	452.5	134.3	440.4	134.8	442.0	3.7	12.2
Oct	7070	200	137.7	451.7	134.3	440.4	135.0	442.9	3.4	11.2
Nov	10800	306	137.3	450.5	134.4	440.7	135.3	443.7	3.0	9.7
Dec	9610	272	140.0	459.2	134.6	441.5	135.6	444.9	5.4	17.7

Table 7a

Month	Estimated Monthly Average Water Levels at New Bridge Piers- Elevations in Feet, Project Datum											
	Pier 5	**0.3**	**0.3**	**Pier 4**	**0.6**	**0.4**	**Pier 3**	**0.7**	**0.6**	**Pier 2**	**0.9**	**1.7**
	Ave +.3	**High**	**Low**	**Ave +.4**	**High**	**Low**	**Ave +.6**	**High**	**Low**	**Ave +1.7**	**High**	**Low**
Jan	445.7	456.3	441.2	445.8	456.6	441.3	446.0	456.7	441.5	447.1	456.9	442.6
Feb	444.6	455.7	441.9	444.7	456.0	442.0	444.9	456.1	442.2	446.0	456.3	443.3
Mar	444.3	452.3	441.6	444.4	452.6	441.7	444.6	452.7	441.9	445.7	452.9	443.0
Apr	449.0	465.3	442.3	449.1	465.6	442.4	449.3	465.7	442.6	450.4	465.9	443.7
May	447.3	469.0	442.1	447.4	469.3	442.2	447.6	469.4	442.4	448.7	469.6	443.5
Jun	444.0	450.2	441.7	444.1	450.5	441.8	444.3	450.6	442.0	445.4	450.8	443.1
Jul	443.3	450.7	441.3	443.4	451.0	441.4	443.6	451.1	441.6	444.7	451.3	442.7
Aug	442.9	454.5	440.8	443.0	454.8	440.9	443.2	454.9	441.1	444.3	455.1	442.2
Sep	442.3	452.8	440.7	442.4	453.1	440.8	442.6	453.2	441.0	443.7	453.4	442.1
Modeled	442.4			442.5			442.6			443.3		
Oct	443.2	452.0	440.7	443.3	452.3	440.8	443.5	452.4	441.0	444.6	452.6	442.1
Nov	444.0	450.8	441.0	444.1	451.1	441.1	444.3	451.2	441.3	445.4	451.4	442.4
Dec	445.2	459.5	441.8	445.3	459.8	441.9	445.5	459.9	442.1	446.6	460.1	443.2

Table 7b. Estimated Monthly Water Levels at Piers, Proposed Madawaska-Edmundston International Bridge

Month	**Estimated Monthly Average Water Levels at New Bridge Piers- Elevations in Meters**											
	Pier 5	**0.09**	**0.09**	**Pier 4**	**0.18**	**0.12**	**Pier 3**	**0.21**	**0.18**	**Pier 2**	**0.27**	**0.52**
	Ave +.09	**High**	**Low**	**Ave +.12**	**High**	**Low**	**Ave +.18**	**High**	**Low**	**Ave +.52**	**High**	**Low**
Jan	135.9	139.1	134.5	135.9	139.2	134.5	136.0	139.2	134.6	136.3	139.3	134.9
Feb	135.5	138.9	134.7	135.6	139.0	134.8	135.6	139.1	134.8	136.0	139.1	135.1
Mar	135.5	137.9	134.6	135.5	138.0	134.7	135.5	138.0	134.7	135.9	138.1	135.1
Apr	136.9	141.9	134.9	136.9	142.0	134.9	137.0	142.0	134.9	137.3	142.1	135.3
May	136.4	143.0	134.8	136.4	143.1	134.8	136.5	143.1	134.9	136.8	143.2	135.2
Jun	135.4	137.3	134.7	135.4	137.4	134.7	135.4	137.4	134.8	135.8	137.4	135.1
Jul	135.1	137.4	134.5	135.2	137.5	134.6	135.2	137.5	134.6	135.6	137.6	135.0
Aug	135.0	138.6	134.4	135.1	138.7	134.4	135.1	138.7	134.5	135.5	138.8	134.8
Sep	134.9	138.1	134.3	134.9	138.1	134.4	135.0	138.2	134.4	135.3	138.2	134.8
Oct	135.1	137.8	134.4	135.2	137.9	134.4	135.2	137.9	134.5	135.6	138.0	134.8
Nov	135.4	137.4	134.5	135.4	137.5	134.5	135.4	137.6	134.6	135.8	137.6	134.9
Dec	135.7	140.1	134.7	135.8	140.2	134.7	135.8	140.2	134.8	136.2	140.3	135.1

Table 7c

	Flows, US, Madawaska River and DS of River- cfs	WSEL at DS boundary of model	Elev at MH for Twin Rivers	Elev at Utility BR - centerline	Elevation at Canadian Gage	Elevation at Existing Bridge		Elevations at Proposed Bridge Location				Modeled WSEL at upstream model boundary
Existing Bridge						DS	US	pier 5	pier 4	pier 3	pier 2	
September Median,	2333/420/2753	432.1	440.7	440.8	441	441	441	441.1	441.2	441.5	442.6	444.8
September Mean	4380/790/5170	434.8	441.9	442.1	442.1	442.2	442.3	442.4	442.4	442.6	443.3	444.7
Proposed Bridge												
September Median,	2333/420/2753	432.1	440.7	440.8	440.9	441	441	441.1	441.2	441.5	442.6	444.5
September Mean	4380/790/5170	434.8	441.9	442.1	442.1	442.2	442.3	442.4	442.5	442.6	443.3	444.7
Construction Phase												
September Median,	2333/420/2753	432.1	440.7	440.8	440.9	441	441.1	441.2	441.2	441.5	442.6	444.5
September Mean	4380/790/5170	434.8	441.9	442.1	442.1	442.1	442.3	442.4	442.5	442.7	443.3	444.7

Table 7d.

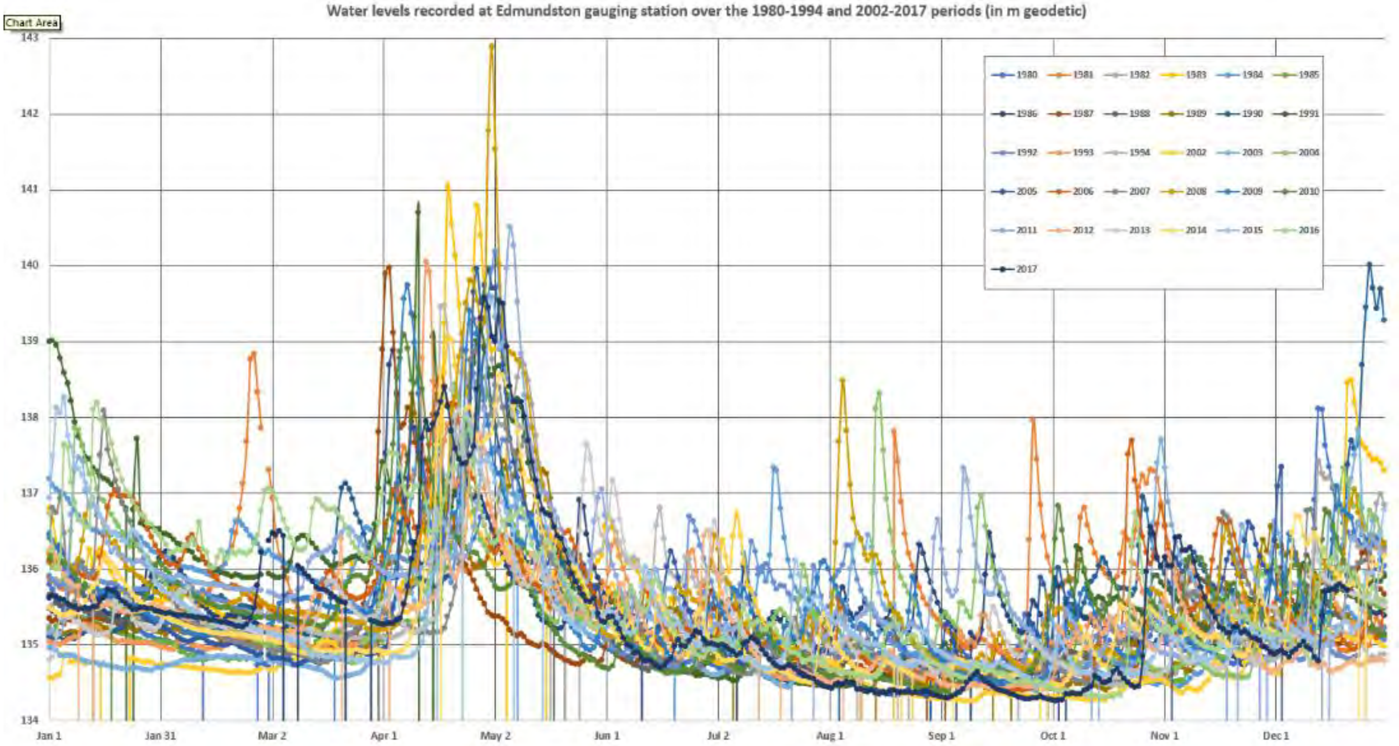

Figure 21. Plot of recorded water levels at Edmundston Gage on St. John River. Plot by Vincent Balland, NBDTI. email to HNTB, 1/6/2020. Datum not specified.

6.0 Hydraulic Analysis

Note: (OHW) was developed by others for inclusion on project documents as this is generally a field determined elevation based on bank characteristics, vegetation and high water marks. The 1-year or annual peak flow elevation was modeled based on project hydrology and hydraulic modeling as a comparison with field determined OHW.

Hydraulic modeling was conducted to develop project design parameters associated with existing and proposed conditions. Modeling provided a basis for water level projections, velocity distributions and directions and scour variables. NHI employed model SMS –SRH2-D for 2-Dimensional flow analysis. The 2-dimensional flow model was selected to account for:

- Proposed bridge at angle to river flow – difficult to simulate with 1-dimensional model.
- River confluence downstream of project site could impact distribution of flow.
- Islands downstream of project site also impact flow distribution.

This bridge is not subject to pressure flow or overtopping. Model options for incorporation of ice cover or ice jam data are limited, but include options such as increasing the “n” value, increasing debris on piers via pier width, or addition of artificial debris in other locations. Due to model limitations, ice is included in discussion only and has not been modeled.

The model was based on a terrain model for the river, islands, bridges and overbanks as discussed above under existing data and GIS model compilation. Two sources of "z" or elevation data were incorporated into the model, including project survey where available and a DEMs published by the Province of New Brunswick and State of Maine for overbank areas not included in project survey.

Bathymetric data was collected within the blue square outlined in Figure 17 above and was used for model elevation data within that area. Downstream of the square area, the model includes both survey data and the DEM.

The goals of the hydraulic analysis include evaluating potential existing vs proposed water levels, evaluation of potential water levels for setting freeboard, and calculation of velocities and scour variables including those needed for design of abutments and piers to withstand local and contraction scour.

The model includes the Twin Rivers utility bridge downstream of the project site, the existing International Bridge, the proposed bridge and possible construction phase bridges such as a temporary construction trestle. The proposed bridge includes 5 piers. Summarizing model configurations for hydraulic analysis (not full scour analysis):

1. Existing conditions with Utility Bridge
2. Proposed Bridge 6- span with utility bridge.
3. Proposed 6-span bridge, plus existing bridge, utility bridge and construction phase access roads. Note that a construction phase trestle was also modeled and is discussed in the supplemental memorandum in the appendix.

Figures 22 through 29 show extent of model and selected model results.

Figure 22. Model domain (green) shows boundaries and bridges. Proposed bridge location is shown in red, existing and utility bridges are shown in aerial photo.

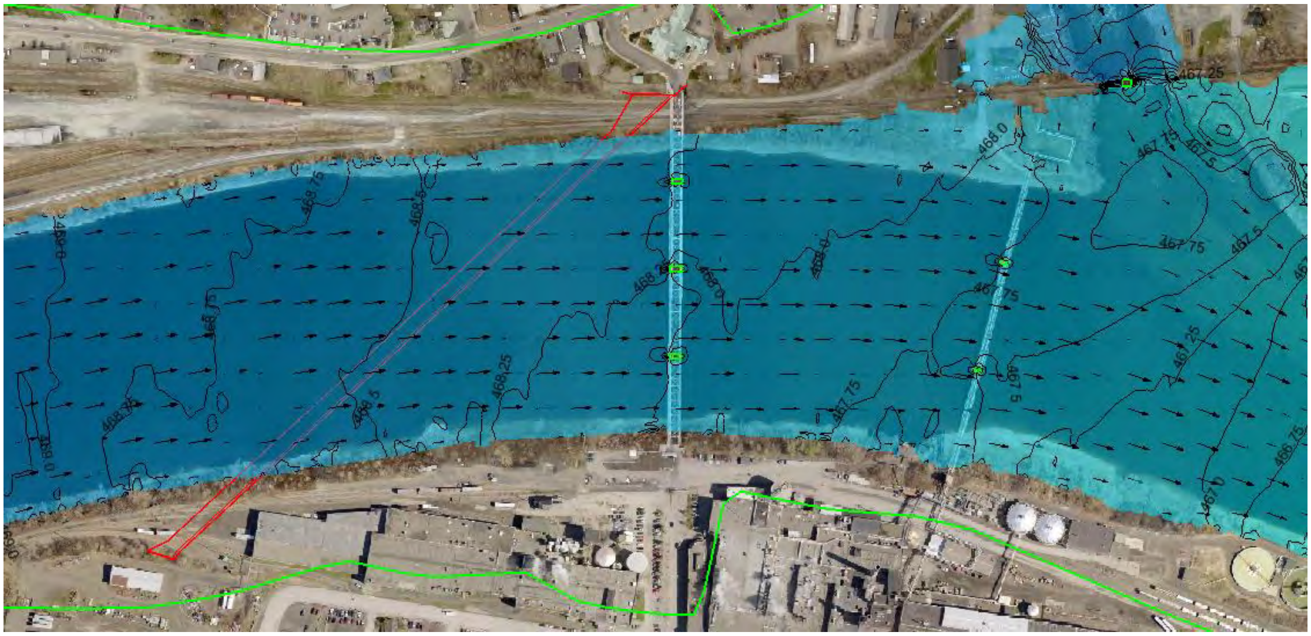

Figure 23. Close-up of project area showing 100-year, 0.25' water elevation contours for existing conditions, bridges and flow direction and magnitude for the existing bridge. Proposed bridge location is shown in red.

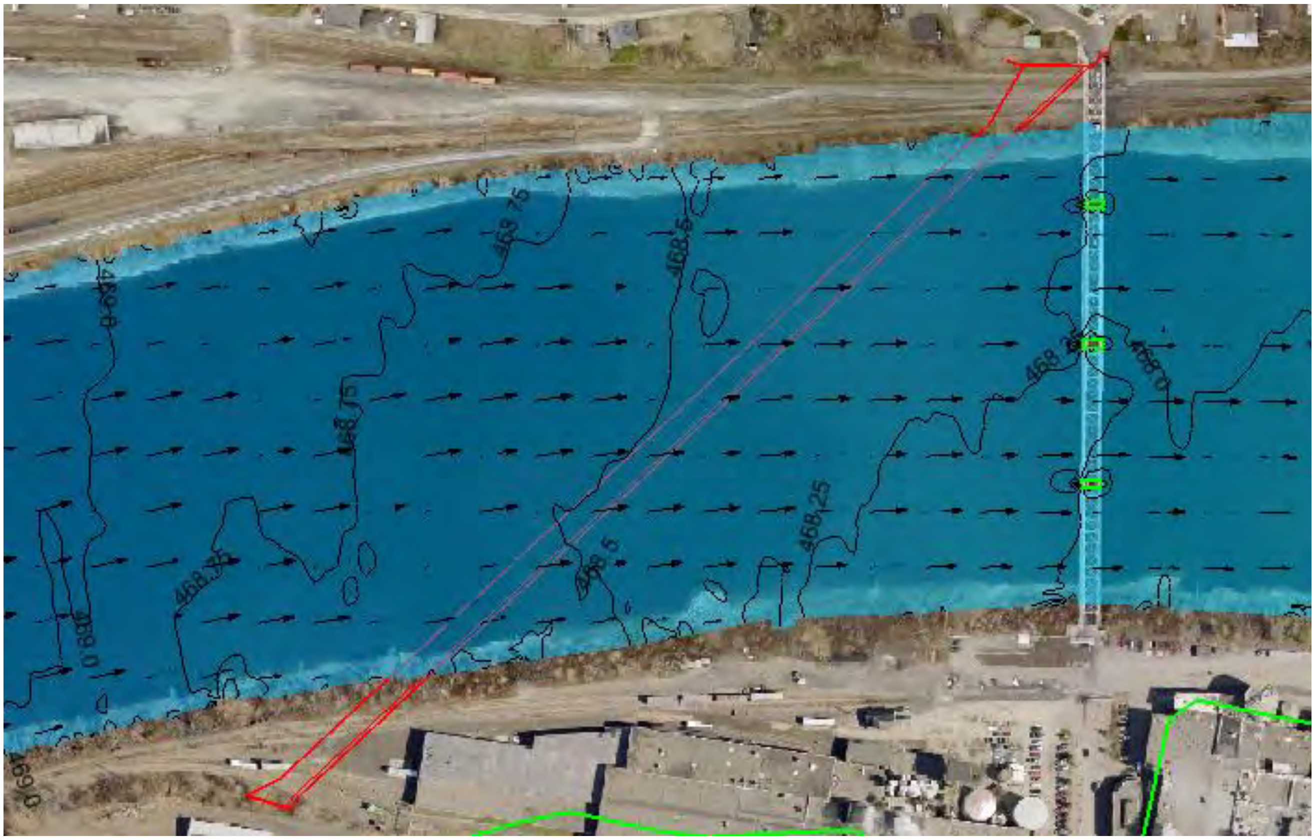

Figure 24: Water surface elevation contours (0.25') for the 100-year Existing Bridge (proposed bridge shown in Red).

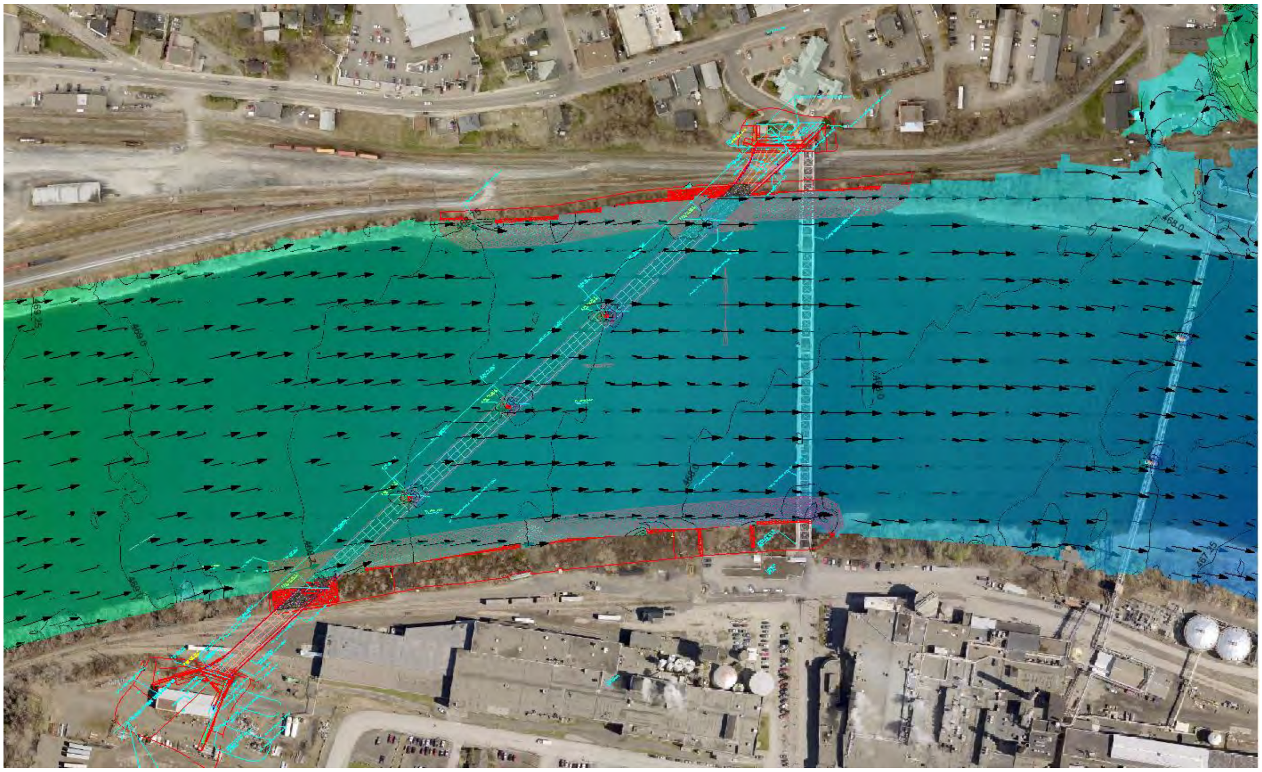

Figure 25: Water surface elevation contours (0.25') for the 100-year Proposed Bridge (existing bridge shown in aerial photo). Figure also shows proposed construction scenario with access roads and piers from both the proposed and existing bridges.

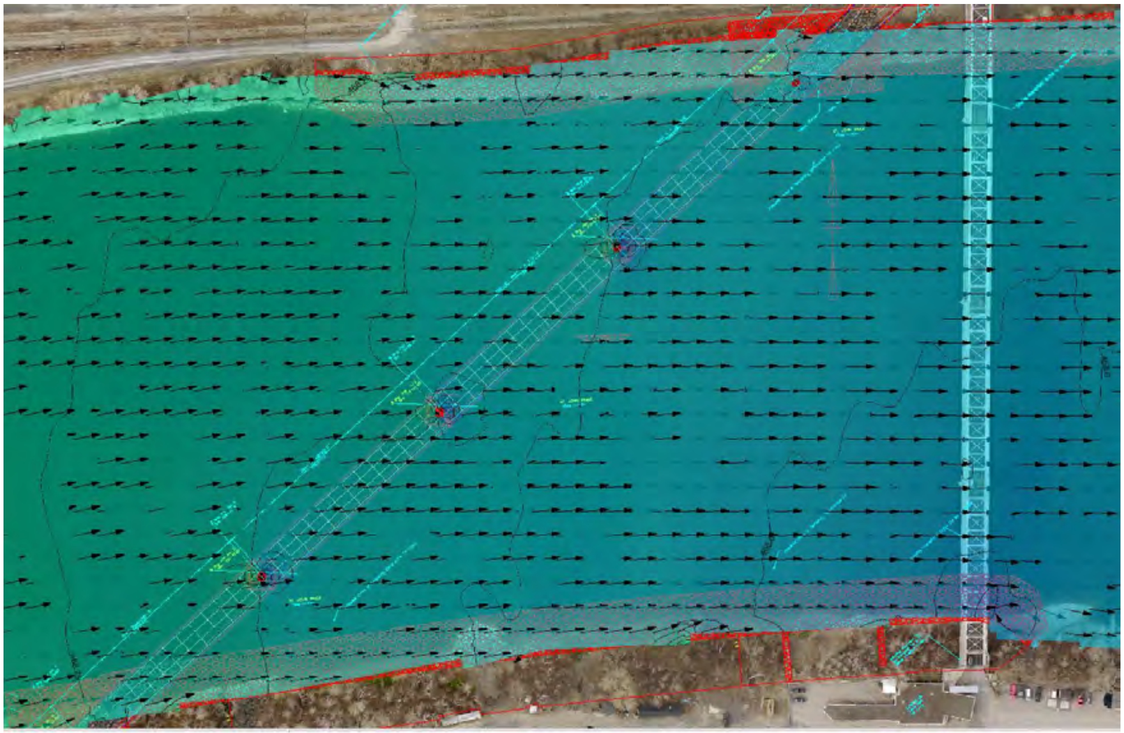

Figure 26. Proposed bridge showing piers and construction access roads with 100-year final water surface elevation contours.

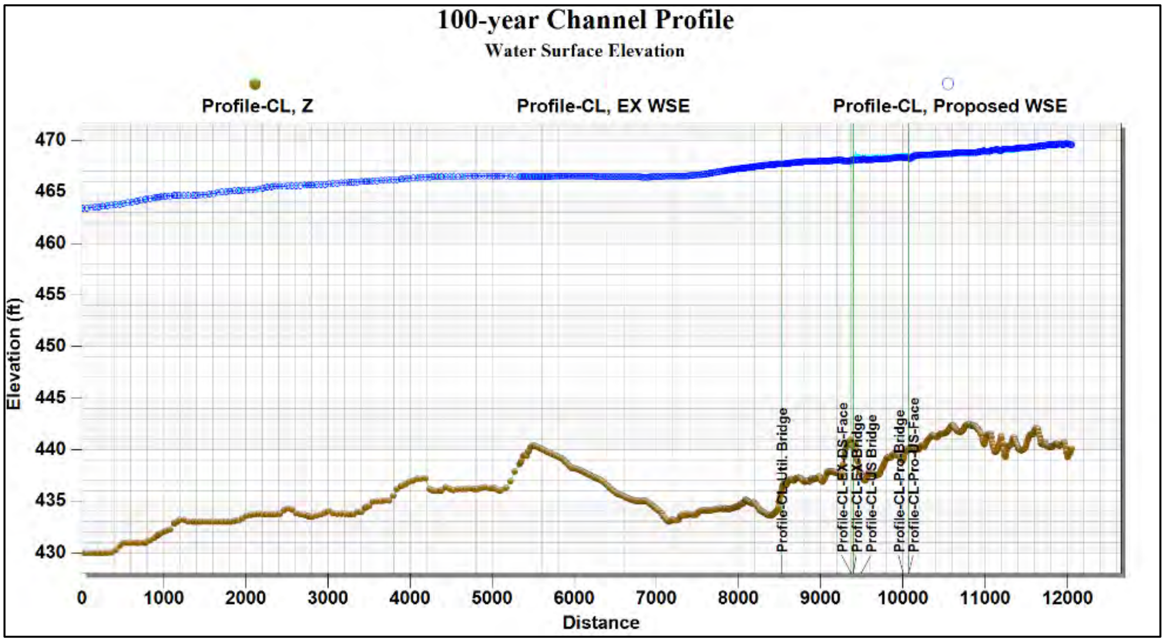

Figure 27: Comparison of Existing vs. Proposed water surface elevations along the channel centerline for the 100-year event. Note that profile is plotted from downstream (left) to upstream (right) per model constraints. Note also that water surface elevations are nearly identical. For detail on water levels, see table 8. In this plot, "z" represents channel elevation in the center of the channel. "EX WSE" is existing condition water elevation and "Proposed WSE" is for the proposed 6 span bridge, all at the channel center.

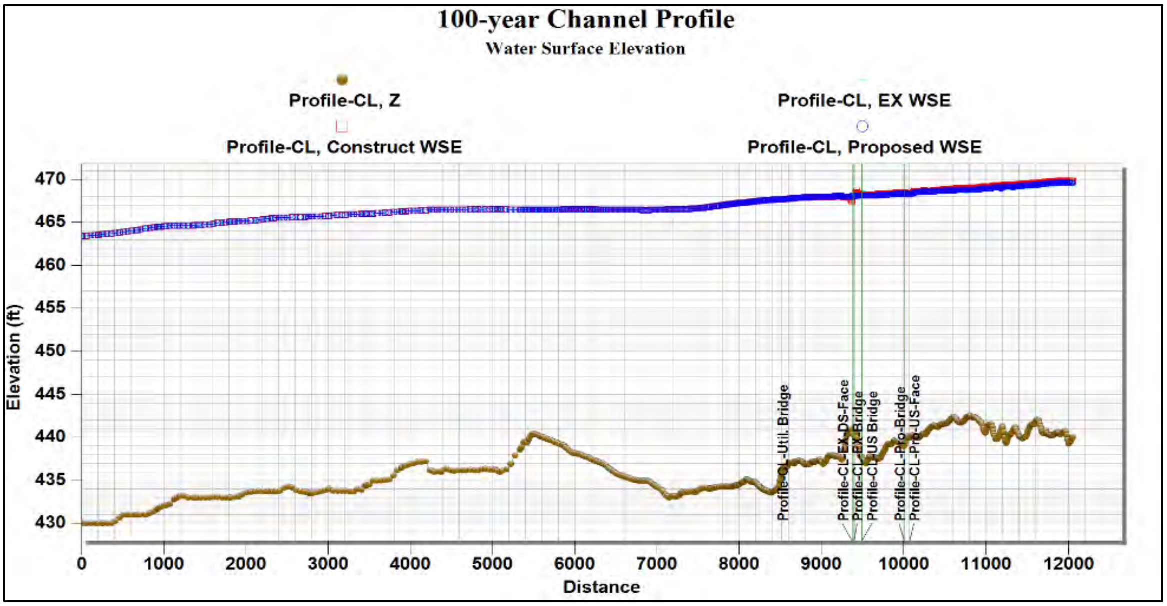

Figure 28: Comparison of Existing vs. Proposed water surface elevations along the channel centerline for the 100-year event including the construction scenario with access roads and piers on both the existing and proposed bridges. Note that profile is plotted with downstream to left, upstream to right. Water levels for existing and proposed are very similar. Detail at bridges is in table 8. "Z" is channel bottom elevation. "EX WSE" is existing conditions 100-year water level. "Proposed WSE" is modeled elevation for the proposed 6-span bridge after project is complete. "Construct WSE" is 100-year water level assuming existing bridges, proposed bridge and construction phase access roads are all in place.

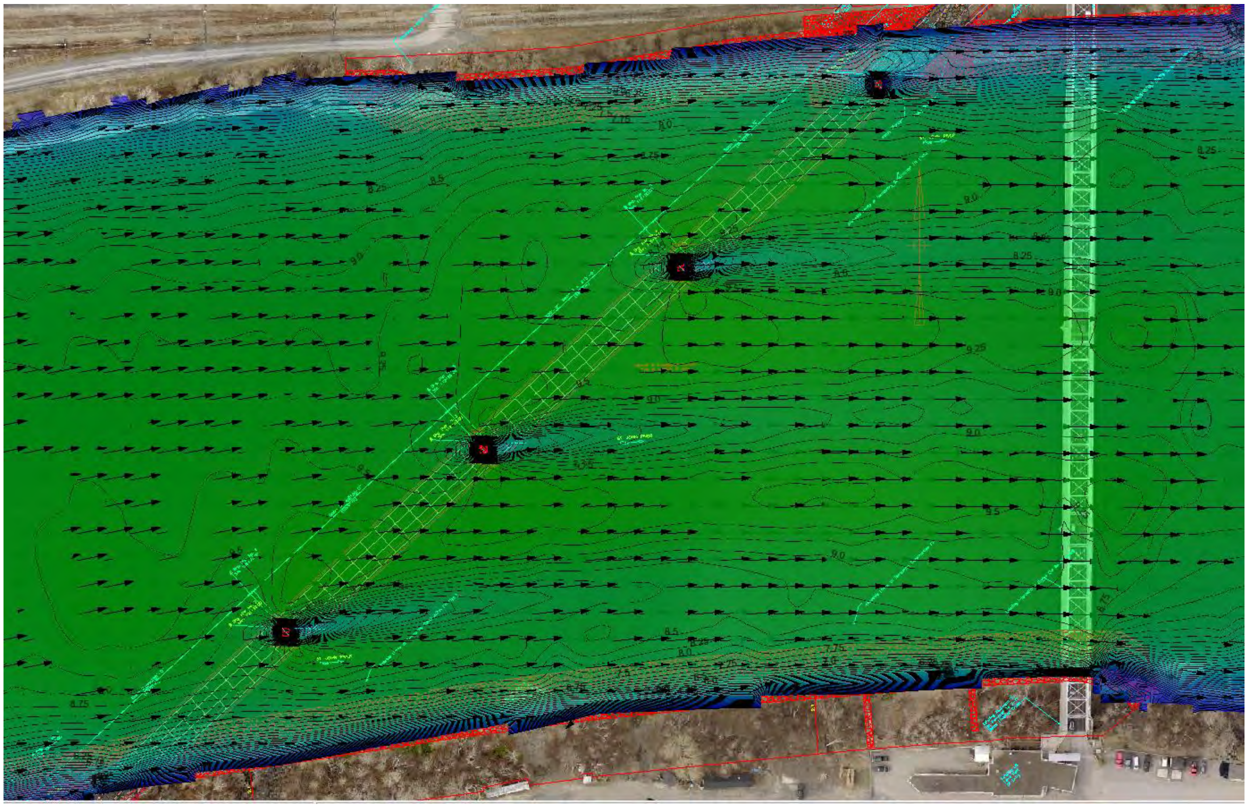

Figure 29. Velocity contours, 100-year flood at proposed 6-span bridge

The MaineDOT Bridge Design Guide (BDG) states that major riverine bridges must provide a minimum of 4 feet of freeboard over the 50-year storm event. In both the existing and proposed conditions, the bridge provides over 30 feet of freeboard at the 50-year storm, to the low chord of the respective structures. This is due to the need to meet vertical clearance requirements over the railroad, the low chord of the proposed structure is similar, and generally higher than the existing bridge.

Modeling indicates that the river’s water levels with the new bridge will not exceed existing condition water levels once the project is complete aside from a limited area in close proximity to the two bridges.

The results highlighted in the Summary of Hydraulic Data represent the maximum water surface elevations upstream of the individual bridges. The maximum water surface elevations occur just upstream of the piers. Thus the Summary Table compares water surface elevations upstream of the proposed piers to water surface elevations upstream of the existing piers. Since the proposed bridge is 170’ to 1150’ upstream of the existing bridge, it is expected that water surface elevations would be higher. Figure 30 is a cross section drawn perpendicular to flow at the southern point of the proposed bridge showing water surface elevations essentially unchanged from existing conditions.

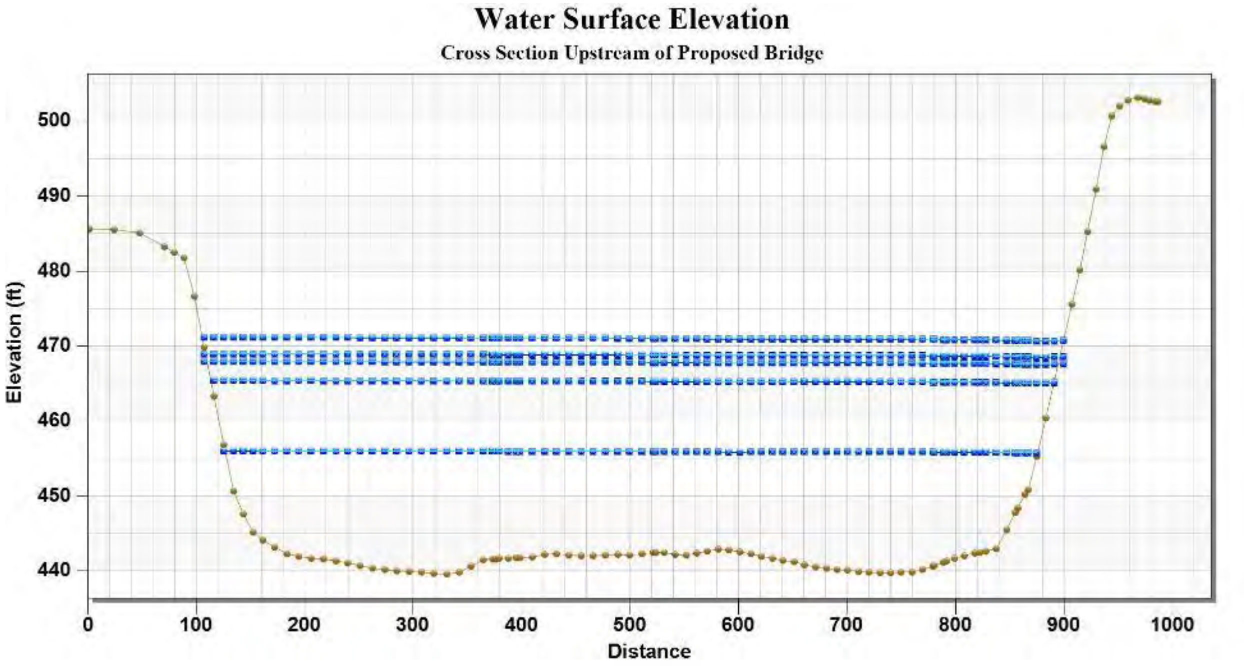

Figure 30: Cross section perpendicular to flow at the southern end of the proposed bridge. Water surface elevations for the existing and proposed condition 1-year, 10, year, 50-year, and 500-year. Existing WSE are blue boxes, proposed WSE are light blue X's. Water surface elevations are essentially the same and vary by less than 0.05 feet at this section near the proposed bridge's most upstream pier.

A plot of the difference in water surface elevations (figure 31) for the 100-year event (proposed minus existing) shows water surface elevations are very similar between the two bridge configurations. The most significant differences are as expected just upstream and downstream of the existing and proposed piers. As expected WSE increases upstream of the new piers and decreases upstream of the removed piers. WSELs between the proposed bridge and existing bridge are approximately 0.1 ft lower than existing when the existing piers are removed, but are equal upstream of the new bridge once the existing bridge has been removed.

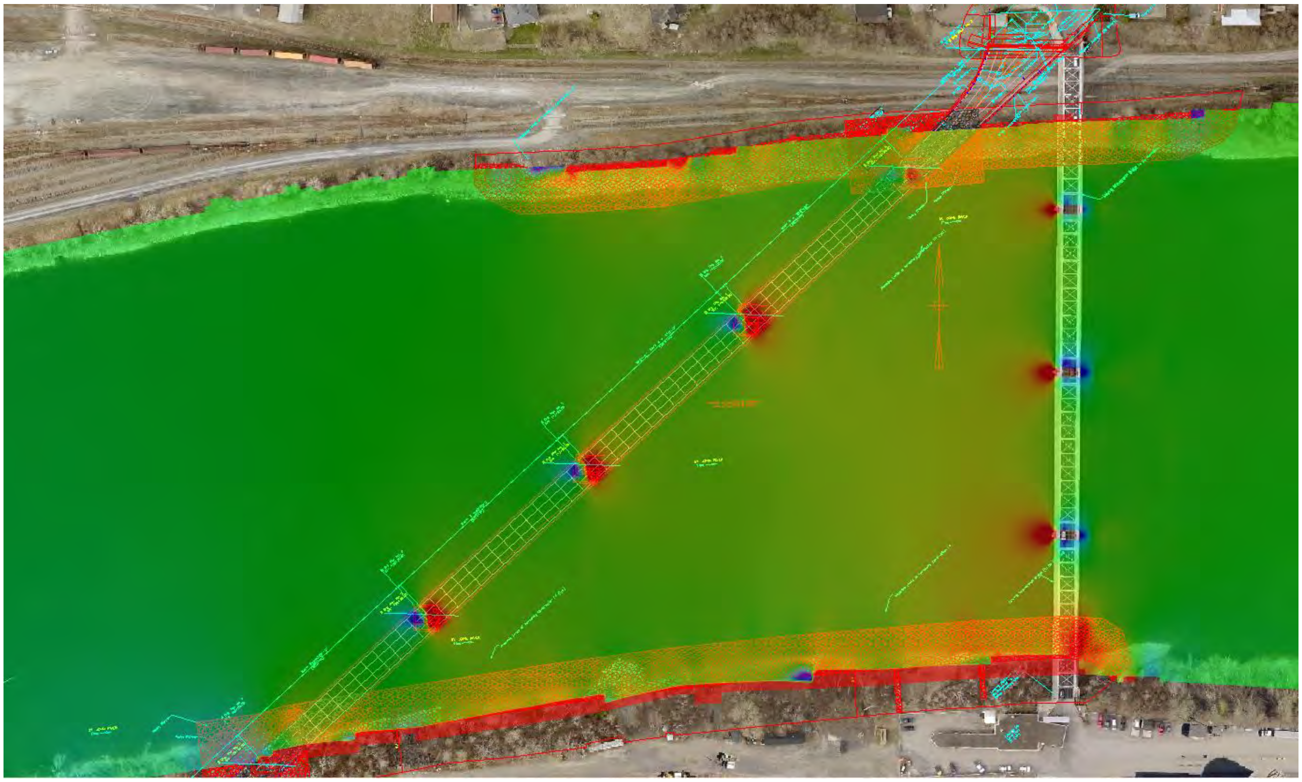

Figure 31: Comparison of the Proposed to Existing 100-year water surface elevations. Green represents essentially no change between proposed and existing. Red represents an decrease in water surface elevations of approximately 0.5 ft and blue represents decrease in WSE of approximately -0.5 ft.

Water surface elevations were examined across a cross section at the southern start of the proposed bridge and 1,350 feet upstream of the northern end of the new bridge. The table below shows those elevations for each storm event. At the upstream bridge section, water surface elevations are very slightly increased with the proposed bridge. All increases are under 0.05 feet. Further upstream there is no difference in water surface elevations. For the temporary construction scenario, the hydraulic model predicts water surface elevations to increase by less than 0.25 feet over existing conditions for all scenarios.

Hydraulic data is summarized in table 8. Note that table 8 includes water levels upstream of each bridge and at the boundary of the hydraulic model for comparison. Because the new bridge is upstream of the existing bridge, water levels are higher, but do not represent higher river levels. Low flow water levels are estimated in table 8a based on preliminary design.

Madwaska-Edmundston International Bridge Note that proposed bridge is upstream of existing bridge. Ice jams not included. See note*		Headwater Just Upstream of Each Bridge – NOTE new bridge is upstream of existing bridge, therefore water levels are higher at the new bridge.		Water Surface Elevation at Upstream Model Boundary	
		Existing Structure	Proposed Structure	Existing Structure	Proposed Structure
		928 -ft, 4 - Span, through Truss, LC 499	6- span, LC 503.5	928 -ft, 4 - Span, through Truss	6-span
Total Area of Waterway Opening	ft2	20,375	18,650		
Headwater elevation @ Q1.1*	ft	455.3	455.7	456.8	456.8
Headwater elevation @ Q2*	ft	458.8	459.2	460.1	460.1
Headwater elevation @ Q10	ft	464.9	465.1	466	466
Headwater elevation @ Q25	ft	466.3	466.5	467.4	467.4
Headwater elevation @ Q50	ft	467.4	467.6	468.6	468.6
Headwater elevation @ Q100	ft	468.4	468.6	469.6	469.6
Headwater elevation @ Q500	ft	470.5	470.8	471.9	471.9
Freeboard @ Q50	ft	approx 31.9	35.9		
Freeboard @ Q100	ft	approx 31.0	34.9		
Flood of Record, 1/25/2008	ft	469.1	469.3	470.6	470.6
Velocity @ Q1.1	fps	6.7	7.6		
Velocity @ Q2	fps	7.2	7.8		
Velocity @ Q10	fps	8.1	8.5		
Velocity @ Q25	fps	8.8	9.3		
Velocity @ Q50	fps	9.3	9.7		
Velocity @ Q100	fps	9.8	10.1		
Velocity @ Q500	fps	10.7	11.0		

*Note that data recorded by the Edmundston gage and the Fort Kent gage indicate that 1- to2-year flows may generate higher water levels during ice jams, with recorded levels as high as 462.5' for an approximate 2-year flow.

Table 8, Summary of Hydraulic Data

7.0 Construction Phase Hydraulic Modeling, Assuming no Ice and Ice

One additional hydraulic scenario was simulated with the 2_D model. This simulation included the existing bridges, the proposed 6-span bridge and the construction access roads assuming all are in place, i.e. the most conservative conditions for simulating water levels. Table 9 details modeled water levels at selected locations for existing, proposed and construction phase conditions. The lower frequency floods (1-, 2-, and 10-year floods) are often used for construction phase planning because this represents a temporary condition. Table 10 presents model results for several lower flow events, the September mean flow and September median flow. Additional lower flow data is included in the report section on hydrology based on recorded/gage water levels, rather than modeled levels.

The model indicates that a small rise in water levels may occur during construction. Once the existing bridge and construction access roads are removed, water levels will not be higher than existing.

Construction Phase Hydraulic Modeling, No Ice

To model construction impacts, access roads and trestle pier geometry from December 2019 plans were coded into the SMS SRH2D model. The piers of both the existing bridge and proposed bridge were also included in the model. The 2-yr and 10-yr events were modeled with the proposed access roads and with and without the trestle piers. It is assumed that the trestle deck would be removed during projected high flow conditions. The results of the storm events are compared to the existing condition and completed proposed condition at a section upstream of the proposed bridge located where the bridge begins on the southern bank.

Water Surface Elevations	US of Proposed Bridge		At Upstream Model Boundary	
Location	2-yr	10-yr	2-year	10-year
Existing Bridge Geometry	459.3	465.3	460.1	466.0
Completed Proposed Geometry (no existing)	459.3	465. 2	460.1	466.0
Construction Access Roads with EX and Pro bridges	459.4	465.4	460.2	466.1
Construction Access Roads and Trestle Piers with both bridges	459.8	465.7	460.4	466.4

Table 9. Construction Phase Hydraulic Model water level, assuming no ice.

Construction Phase Hydraulic Modeling, Assuming Ice Jamming

Modeling was based on information contained in NBDTI's study on Ice for the proposed the Madawaska-Edmundston International Bridge. The study was conducted by HILCON Limited and was published in December of 2018. Based on the discussion included in the Hilcon Ice Study, Northstar Hydro, Inc. assumed a river-wide ice jam during breakup conditions to be the most severe likely scenario to occur during construction. The location and spacing of the piers and narrowing of the channel width presents a location where ice would be more likely to accumulate creating a jam as described above. The Hilcon study describes the average flow at the Edmundston gage during breakup to be 25,780 cfs (730 m^3/s) which is less than the 1-year annual peak flow

The thickness of the potential jam was calculated using the Equilibrium Ice Jam Theory described in the 2002 Army Corps of Engineers Ice Engineering Manual. Using this theory, the thickness of the ice jam is a function of ice strength, density and cohesion, depth of water below ice, and channel geometry. The ice jam was modeled in SRH2D as a pressure flow boundary condition just upstream of the construction zone. An initial estimate of the depth of water below the ice was used to calculate an ice thickness at the probable jam location. Several iterations of the model were performed until the water surface elevation just upstream of the jam was just below the ice surface at the crest of the jam. As a simplification, the model assumes ice free conditions downstream of the construction zone.

The table below summarizes the results of the average winter breakup flows at a cross section of the river just upstream of the start of the bridge on the southern bank. Note that this is an estimate only, based on available information and modeling assumptions to simulate effects of ice.

Modeled Condition (25, 780 cfs)	US WSE (ft)	Ice Thickness (ft)	Ice Crest Elev. (ft)
Existing Conditions	448.7	No ice	
Construction Access Roads and Trestle Piers with both bridges, no ice	449.52	No ice	
Possible Ice Impacts on construction with Piers, Q=25,780 cfs, (< 1-year peak flow).	452.13	6.9	452.4

Table 9a. Construction Phase Approximate Hydraulic Model, water levels assuming ice. Estimated only. Ice conditions vary dramatically.

The predicted thickness of the ice and the predicted impact on water surface elevations is consistent with the Hilcon Report showing ice jam thicknesses on the St. John River ranging from 4.3 to 12.7 feet.

Note again that an example of past ice jams is April, 1991, when an approximately 2-year flow reached elevation 140.7 m (461.5’) at the gage, whereas a non-ice 2-year flow would be about 458.4’ or about 3’ lower than the iced condition.

8.0 Ice Jams

The NBDTI commissioned Hilcon Ice study for the proposed the Madawaska-Edmunston International Bridge was published in December of 2018. According to this ice study, historically, ice jams have broken up with as little as 7000 cfs or as high as a 1-2-year flow. Higher flows are not likely to include ice jams, as the ice will already have broken up before those high flows are reached on the river. However, it is estimated that the high water level in 1991 caused a water level close to the 2008 level, and flow was less than a 5-year event. Table 10 summarizes data that is included in the Ice Study. The last line of the table is given for reference and notes findings of this Hydrology/Hydraulics and Scour study.

Data Notes	Flow, cfs	Flow, cms	Elev, ft	Elev, m	Approx Freq
HECRAS approx rating curve figure 3.1 on page 15 of Ice Study	88,000	2493	462.5	141	2 year
	70,000	1983	459.2	140	1 year
	40000	1130	453.3	138.2	<1-year
Min breakup flow	6987	198	437.9	133.3	< 1-year
Max breakup flow	81161	2300	460.8	140.5	2-year

Note: Current NHI Hydraulic study, no ice	68000	1926	454.9	138.7	1-year

Table 10. Summary of data included in Ice Study and comparison to Hydraulic data from this study. Elevation at Canadian gage, downstream of existing bridge.

Key points in the HILCON STUDY include:

- Flood stage is noted to be 139.0 m. (455.9'). Note that elevations are based on "mean sea level" but the datum is not defined. (NHI estimates that it is approximatly 0.7' higher than project datum.) The study also noted these floods
  - 1991: 2200 cms, 141.0 m, (77,000 cfs, 462.5').
  - May 2008 – 143.1 m (469.4')
  - 1993: 2490 cms (87,900), 140.5 (460.8')
  - 1996: 723 cms (25,500 cfs), 138.5 m, (454.3')
- Breakup flows: 550 cms to 1440 cms (19,400 to 50,800) (less than 1-year)
- Minimum breakup flow: 198 cms (6987 cfs) < 1-year
- Maximum breakup flow: 2300 csm (81161 cfs) approx 2-year
- Est breakup stage: 133.3 to 140.5 m. (437.2 to 460.8')
- Ave breakup flow: 1000 cms (35,300 cfs)
- The study notes that the maximum scour depth at piers in general is about 2.3 times pier width even with ice (page 29). The study also notes possible increase of 25 to 35% or .3 to .5 times pier width with ice and scour may occur up to 21% more than without ice.
- Flow below ice jams is not well understood.
- Scour under ice may be similar to a submerged bridge
- The study suggest a design value for scour maybe 1.2 times calculated pier scour without ice.
- Section 5.0 calculates that the three bridges may be sufficient to trigger an ice jam, although they also note that this is not likely with only the new bridge.
- A computation process for predicting the potential for ice jamming is given in Appendix C of that report.
- In the summary, the report calculations predict that the new bridge is not likely to increase the likelihood of ice jams.

- A key finding, #11 on page 48 of the report notes: "*During the construction period, there will be ice seasons during which piers of both the proposed and existing bridges are in place. Furthermore, temporary works to facilitate construction (e.g. rock roads, trestles) may also contribute to the total resisting force and thus increase the possibility of ice jamming during the construction period*."
- The report recommends mitigation measures for ice during the construction period.

NHI's hydraulic study notes that water levels recorded by the Canadian river gage in Edmundston compared to flows recorded by USGS at Fort Kent indicates potential 1-2 year flows with ice may reach elevation 141 m (462.5') at the gage (slightly higher at the bridge), more typical of a 25-year type of flow without ice.

NHI prepared additional information related to ice by reviewing several documents for potential impacts of construction phase piles being in the river during the winter season. Documents reviewed include the U.S. Army Corps of Engineers Engineering and Design Manual on Ice Engineering., the Hilcon Ice Study commissioned by NBDTI, FHWA Scour Manual HEC-18, the Maine DOT Bridge Design Manual and information developed by NHI during preliminary design for the Madawaska-Edmundston International Bridge. The following comments summarize information that may assist in decisions related to construction phase piles remaining in the river in the winter season.

Key points from the ACOE Ice manual include:

- Breakup type of ice jams may be created with planned Ice Control Structure (ICS). While an ICS may be used to control where ice jams occur, the same idea may mean that leaving piles in the river during the winter may trigger this type of affect.
- "The factors and relationships that determine the probability of ice jams and ice jam flooding are more complex than those for open-water flooding. This means that the extensive statistical analysis methods applied to normal flooding phenomena are not readily applicable to ice-related occurrences."
- "Many structures such as dams, bridge piers, and tower foundations, although not specifically designed to control ice, do serve that purpose. In addition, piers, piles, and pile clusters … have been used to stabilize a sheet ice cover."
- "Ice jams form when the moving ice floes reach a location in the river where its ice transport capacity is exceeded...geometric constraints etc." These types of jams are known as break up ice jam.
- The manual illustrates several ice control structures. These structures are designed to "capture" moving ice floes and create a jam at a planned location, with a goal of protecting downstream areas from damage due to ice. Figures 3-35 and 3-36 below show conceptual views of structures that use piles. These structures may be very similar to the construction phase structures.

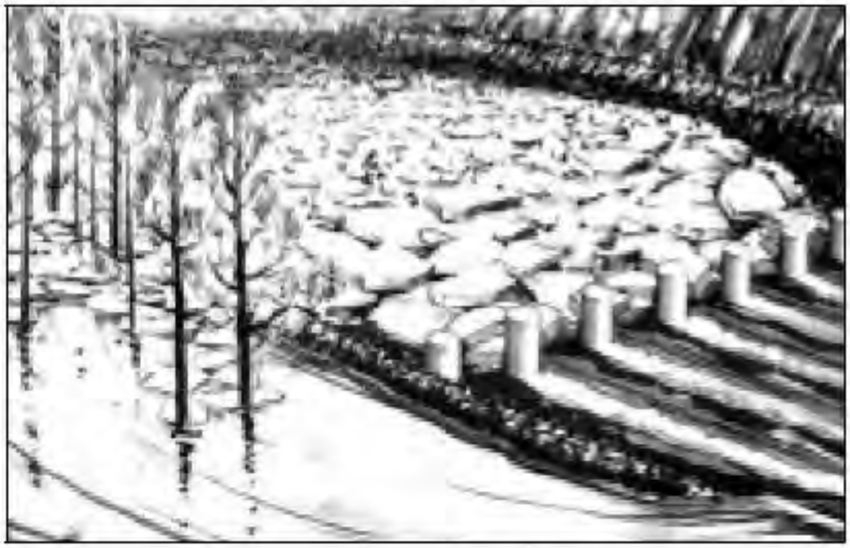

Figure 3-35. Conceptual drawing of Cazenovia Creek ICS.

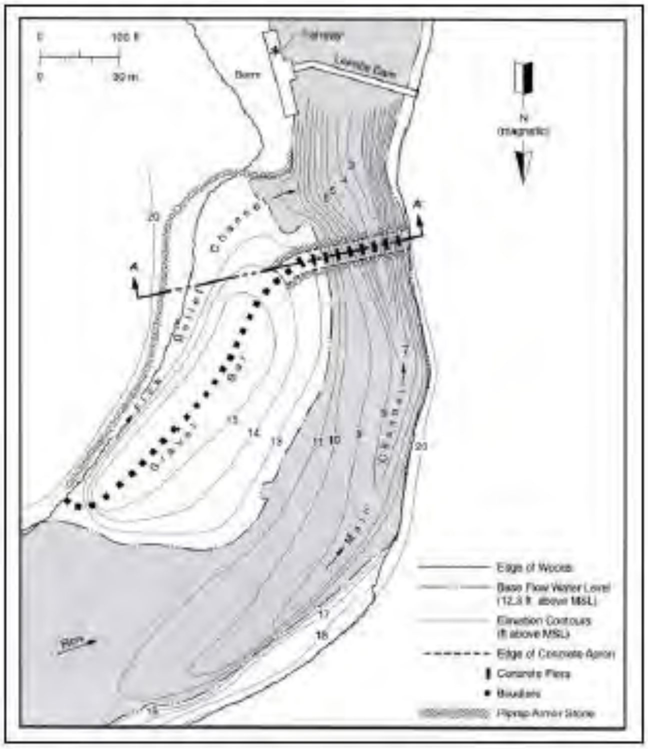

Figure 3-36. Plan view of the Salmon River ICS.

A photo of an ice control structure is shown in figure 3-37 from the manual.

Figure 3-37. Weir with piers ice control structure on the Ste. Anne River, St. Raymond, Quebec. (Photo courtesy of Marc Delagrave, Roch ltée Groupe-conseil, Sainte-Foy, Quebec.)

Another source of information related to potential impact of ice on piers may be found in FHWA's HEC-18 manual, <u>Evaluating Scour at Bridges</u>. While the text is specifically about debris on piers, ice accumulation on piers could have a similar impact.

- "Floating woody debris (drift) that lodges and accumulates at bridge piers creates additional obstruction to flow, and transforms the pier geometry into one that is effectively wider than if debris were not present."
- Equations for estimating pier scour contain a variable for pier width. Adding debris directly affects potential width of the pier and therefore scour depth.

In their Bridge Design Guide, Maine DOT recommends using 1.25 pier width to account for potential debris.

The COE Ice Manual suggests a method that my help to alleviate flow at an ice jam created by piers/piles and ice control structures (ICS), by creating relief flow around the main jam.

- A new concept termed "in channel relief flow" uses a longitudinal row of piers aligned parallel to one bank. These longitudinal piers provide an open water flow path around the jam that forms behind the piers across the main channel." (page 3-63, and figure 3-42). This concept may make sense if the piles are to be left in the river over the winter. The manual cautions on this same page "Possible outcomes range from gradual melting in place or metered release through one or two of the pier gaps (in the ICS), to a massive release between multiple piers. Great care must be taken to avoid this third type of scenario, as the ICS may pose a greater public hazard than the one it is trying to prevent. In light of this, a careful and conservative design approach is advisable".

The manual notes the following as well:

- A planned ICS that causes an ice jam will also cause an upstream rise in water level due to the ice jam. If the planned construction piles act as an ICS, water levels can be expected to rise upstream.

- Options for modeling ice effects include the simple 1-D flow model HECRAS which requires input variables that include detailed knowledge of what the ice jam would look like such as ice thickness. More robust numerical models have been developed by CRREL and others, but all require detailed knowledge of ice processes at the site. Physical modeling may provide more insight into potential ice impacts, but this is an expensive and time consuming option for what is planned as a temporary structure for construction.
- “Neither HEC-RAS nor ICETHK can estimate where an ice jam will occur”.
- The COE manual notes that optimal spacing for creating an ice jam is 12 feet or 3.7 meters.
- Detailed instructions for ICS design are included in Table 3-8 of the COE manual.
- “Ice jams initiate at locations in the river where the ice transport capacity or ice conveyance of the river is exceeded by the ice transported to that location by the river’s flow. “
- “Very few methods for predicting ice jams exist, and those that do are highly site-specific, requiring knowledge of the location of the jam initiation point.”

9.0 Scour Analysis

Scour was analyzed for the proposed 5-pier, 6-span bridge. Data from model SMS SRH-2D was used to generate scour variables, including water depth, water elevation, velocity, and Froude number at selected locations. The Bridge Scour option in SMS SRH-2D was used to generate composite data such as flow through a section, bottom width and section area.

Long Term Bed: This component of scour can be difficult to estimate. No specific data was located for the Saint John River during preliminary design. Based on field observation, it appears that smaller particles may be removed from the bed, leaving an armored layer behind. Figure 32 shows typical bed material on the surface of the river bed. Note boot for scale. This element of scour was estimated at 2’.

Bed Material: Pebble counts were used to estimate the size of the bed material as shown in Figure 27. The distribution in table 12 resulted from the count.

Percent Smaller	Size, mm	Size, Feet
D16	34	.11
D50	79	.26
D84	135	.44

Table 11. Bed material size distribution based on pebble counts during site visit.

Bed material was also sampled and analyzed in the Geotechnical Report prepared by GZA for this bridge project. Table 12a summarizes bed material data gathered from borings.

Boring no	DOT pier no	Sample depth	D10, mm	D30, mm	D50, mm	D85, mm	D90, mm
205	2	0-2'	0.002	0.029	0.463	3.970	5.800
		3-5'	0.004	0.150	1.290	19.500	28.700
206	3	0-2'		0.003	0.009	0.051	0.064
		3-5'		0.003	0.007	0.331	1.370
207	4	no samples					
208	5	0-2'		0.738	4.190	22.900	23.700
		5-7'		0.004	0.015	1.490	1.960
Ave mm				0.154	0.996	8.040	10.266
Range, mm				.003-0.74	.01-4.2	.05-22.9	.064-28.7
Median, mm				0.372	2.100	11.500	14.400
Ave ft				0.001	0.003	0.026	0.034

Table 11a. Summary of boring data, GZA Geotechnical Report.

Contraction Scour: The proposed bridge appears to cause little to no contraction in the river. Computation of contraction scour using model parameters yielded the following minimal amounts compared to the size of the river:

100-year	0.9 feet
500-year	1.0 feet

Abutment Scour: Abutments are located well above flood levels and outside of the flood zone, so no abutment scour was computed. Abutments do not cause any contraction within the river’s flood plain. Abutments will be protected against potential channel migration by heavy riprap. This section of river appears stable as far as potential channel migration, with its relatively straight alignment and deeply incised bed. NHI assumes that scour protection will designed by others in the project team. NHI will provide assistance with and hydraulic parameters for scour protection design.

Pier Scour: Piers were evaluated individually for potential scour. Scour estimates are based on details provided in 60% design plans dated 6/3/2020 and provided to NHI by HNTB,).

In accordance with HEC18, pier scour estimates for piers 2-5 were prepared based on “complex pier scour” recommended analysis. The piers include either a pier stem, footing and seal or a pier stem, footing, seal and piles, all of which have different widths and shape, requiring analysis of each component and its potential impact on scour. Pier 1 (U.S. side of river) was analyzed using the basic pier scour equations. The base of this pier is only partially submerged during 100- and 500-year floods, but may be susceptible to scour. Worst case scour for this pier was computed. Hydraulic data from model SMS SRH 2-D was used for pier scour analysis.

Tables 12 and 13 on the following pages detail estimated scour at each pier as well as other scour components. .

Figures 32, 33, and 34 show ongoing pier scour and scour protection at the existing bridge.

Figure 32. Typical distribution of bed material.

Figure 33. Note scour hole at nose of pier of existing bridge

Figure 34. Riprap placed at nose of existing pier

Figure 35. Pier shows evidence of scour at base as well as pointed nose shape with ice/debris protection

Location	Pier Number	Bottom Elev- US of pier (plans)	Top of Footer	Top of Seal	Base of Seal	Rock Elev	Pier Scour, assuming Debris @1.25Xpier stem width		Width of Scour Hole*		Contraction Scour		Aggradation/Degradation -Assumed		Total Scour		Bottom Elevation after Scour	
							100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Canadian side	5	441	441	434	429	407	12.7	12.6	20.1	19.8	0.9	1.0	2.0	2.0	15.6	15.6	425.4	425.4
	4	438	439	432	427	410	16.6	17.5	27.5	28.8	0.9	1.0	2.0	2.0	19.5	20.5	418.5	417.5
	3	441	441	434	429	410	15.9	16.8	25.4	26.7	0.9	1.0	2.0	2.0	18.8	19.8	422.2	421.2
	2	441	441	434	429	412	15.5	16.4	24.8	26.0	0.9	1.0	2.0	2.0	18.4	19.4	422.6	421.6
US side	1	474	441	458	453	413	8.6	9.4	17.2	18.8	0.9	1.0	2.0	2.0	11.5	12.4	462.5	461.6

* Assume scour occurs beside footing/seal to 1/2 of this width on each side of pier.

Table 12. Pier Scour Summary for Piers with Stem, footing, seal and piles

note: pier 1 scour per basic pier scour equation, piers 2-5 via complex pier scour
conservative value at pier 1 for seal would be 18.5 and 20.5

Location	Pier Number	Bottom Elev- US of pier (plans)	Top of Footer	Top of Seal	Base of Seal	Rock Elev	Pier Scour, assuming Debris @1.25Xpier stem width		Width of Scour Hole*		Contraction Scour		Aggradation/Degradation -Assumed		Total Scour		Bottom Elevation after Scour	
							100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Canadian side	5	441	441	434	407	407	10.0	9.9	20.1	19.8	0.9	1.0	2.0	2.0	12.9	12.9	428.1	428.1
	4	438	434	432	410	410	13.7	14.4	27.5	28.8	0.9	1.0	2.0	2.0	16.6	17.4	421.4	420.6
	3	441	429	434	410	410	12.7	13.4	25.4	26.7	0.9	1.0	2.0	2.0	15.6	16.4	425.4	424.6
	2	441	407	434	412	412	12.4	13.0	24.8	26.0	0.9	1.0	2.0	2.0	15.3	16.0	425.7	425.0
US side	1	474	32	458	413	413	8.6	9.4	17.2	18.8	0.9	1.0	2.0	2.0	11.5	12.4	462.5	461.6

* Assume scour occurs beside footing/seal to 1/2 of this width on each side of pier.

Table 13. Pier Scour Summary for Piers with Stem, footing, and seal to rock

note: pier 1 scour per basic pier scour equation, piers 2-5 via complex pier scour
conservative value at pier 1 for seal would be 18.5 and 20.5

10.0 References

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Canadian Gaging Station at Edmundston on St. John River (*Datum of Canadian Gage is CGVD1928)*. http://wateroffice.ec.gc.ca/report/historical_e.html?stn=01AD004

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NORTHSTAR
— H Y D R O —

Appendix:

Detailed Scour Computations

Location	Pier Number	Bottom Elev- US of pier (plans)	Top of Footer	Top of Seal	Base of Seal	Rock Elev	Pier Scour, assuming Debris @1.25Xpier stem width		Width of Scour Hole*		Contraction Scour		Aggradation/Degradation -Assumed		Total Scour		Bottom Elevation after Scour	
							100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Canadian side	5	441	441	434	429	407	12.7	12.6	20.1	19.8	0.9	1.0	2.0	2.0	15.6	15.6	425.4	425.4
	4	438	439	432	427	410	16.6	17.5	27.5	28.8	0.9	1.0	2.0	2.0	19.5	20.5	418.5	417.5
	3	441	441	434	429	410	15.9	16.8	25.4	26.7	0.9	1.0	2.0	2.0	18.8	19.8	422.2	421.2
	2	441	441	434	429	412	15.5	16.4	24.8	26.0	0.9	1.0	2.0	2.0	18.4	19.4	422.6	421.6
US side	1	474	441	458	453	413	8.6	9.4	17.2	18.8	0.9	1.0	2.0	2.0	11.5	12.4	462.5	461.6

* Assume scour occurs beside footing/seal to 1/2 of this width on each side of pier.

Table 12. Pier Scour Summary for Piers with Stem, footing, seal and piles

note: pier 1 scour per basic pier scour equation, piers 2-5 via complex pier scour

conservative value at pier 1 for seal would be 18.5 and 20.5

Location	Pier Number	Bottom Elev- US of pier (plans)	Top of Footer	Top of Seal	Base of Seal	Rock Elev	Pier Scour, assuming Debris @1.25Xpier stem width		Width of Scour Hole*		Contraction Scour		Aggradation/Degradation -Assumed		Total Scour		Bottom Elevation after Scour	
							100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Canadian side	5	441	441	434	407	407	10.0	9.9	20.1	19.8	0.9	1.0	2.0	2.0	12.9	12.9	428.1	428.1
	4	438	434	432	410	410	13.7	14.4	27.5	28.8	0.9	1.0	2.0	2.0	16.6	17.4	421.4	420.6
	3	441	429	434	410	410	12.7	13.4	25.4	26.7	0.9	1.0	2.0	2.0	15.6	16.4	425.4	424.6
	2	441	407	434	412	412	12.4	13.0	24.8	26.0	0.9	1.0	2.0	2.0	15.3	16.0	425.7	425.0
US side	1	474	32	458	413	413	8.6	9.4	17.2	18.8	0.9	1.0	2.0	2.0	11.5	12.4	462.5	461.6

* Assume scour occurs beside footing/seal to 1/2 of this width on each side of pier.

Table 13. Pier Scour Summary for Piers with Stem, footing, and seal to rock

note: pier 1 scour per basic pier scour equation, piers 2-5 via complex pier scour

conservative value at pier 1 for seal would be 18.5 and 20.5

Piers are numbered from L to R facing DS (Canadian side to American side)

Updated with final SMS model 6/3/2020 EO

Pier number	Bottom US	Bottom DS	WSEL UP	WSEL dn	Depth Up	Depth DS	Velocity US	Vel DS	Froude No US
100-year	out of water								
L Abut (facing DS)	499	479.5							
Proj Pier 5	444.8	445.3	468.3	468.2	23.5	22.9	5.4	4.7	0.2
Proj Pier 4	437.7	437.7	468.6	468.1	30.9	30.4	8.4	6	0.2
Proj Pier 3	440.7	440.7	468.6	468.2	27.9	27.5	8.7	6.5	0.29
Proj Pier 2	441.3	441	468.8	468.4	27.5	27.4	8.3	6	0.28
Project Pier 1	466	474.3	468.9	470	2.9	-4.3	1.5	0	0.22
R Abut	out of water								
500-year									
L Abut (facing DS)	499	479.5							
Proj Pier 5	444.8	441.8	470.5	470.3	25.7	28.5	5.15	3.7	0.21
Proj Pier 4	437.7	437.7	470.8	470.2	33.1	32.5	9.2	6.5	0.2
Proj Pier 3	440.7	440.7	470.8	470.4	30.1	29.7	9.6	7.1	0.31
Proj Pier 2	441.3	441	471.1	470.5	29.8	29.5	9.1	6.5	0.29
Project Pier 1	466	474.3	471.1	471.6	5.1	-2.7	2.1	0	0.17
R Abut	out of water								

Summary of Scour Variables from Model

Madawaska International Bridge
Contraction Scour

filled in variables
key computed variables

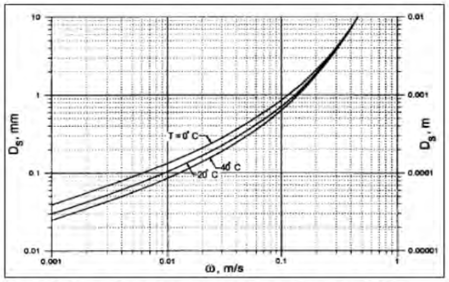
EOB update 5-28-20

Vc = Ku * y^(1/6) * D ^(1/3) Critical Velocity in fps

	english	m	
Ku	11.17	6.19	
D50 ft, mm	0.003267	0.9957	
D50 M	0.003266	0.000996	
y, ft	27.76	8.463415	ave depth in us section
Y^/167	1.742012	1.428565	
D^.333	0.148662	0.100087	
Vc = Ku * y^(1/6) * D ^(1/3)			
VC	2.892704	0.885049	live bed scour

Slope
0.000659
0.00072

		V*/T<.5	V*/T>.5<2	V*/T>2
k1		0.59	0.64	0.69
V*=(gyS1)^.5	100		0.76750352	
	500		0.8343953	
T, fig 6.8	mps		0.15	
	fps		0.492	
V*/T	100		1.5599665	
	500		1.69592541	
K1			0.64	
			0.64	



ure 6.8. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

y2	K1	Y0	Ys	HECRAS ys
final BR depth		ex depth in	scour in BR	
#DIV/0!	0.59		#DIV/0!	
#DIV/0!	0.64		#DIV/0!	
#DIV/0!	0.64		#DIV/0!	
#DIV/0!	0.64		#DIV/0!	

y2/y1 = (Q2/Q1)^(6/7) * (W1/W2)^k1

Live Bed Scour		100	500
y1,y	ave depth in US channel	27.76	30.03
y2	ave depth in contracted section after scour	28.7	31.0
y0	existing depth in contracted section	27.9	30.0
Q1	flow in US channel section	172211.0	202610.0
Q2	flow in contracted channel section	175876.0	207123.0
W1	bottom width of US main channel transporting bed material	726.5	726.5
W2	bottom width of main channel in contracted section	710.9	710.9
Ys	scour depth	0.9	1.0

D=particle size in ft/m
D50=particles size 50%smaller, ft, m

	(W1/W2)^k1	(Q2/Q1)^(6/7)
100	1.013989241	1.018214234
500	1.013989241	1.019062153

From boring logs, preliminary design

Onsite Pebble count D50 approx 79 mm, or 0.26'

Boring no	DOT pier no	Depth	D10	D30	D50	D85	D90
205	2	0-2'	0.0018	0.0285	0.463	3.97	5.8
		3-5'	0.004	0.15	1.29	19.5	28.7
203	Aubt R						
206	3	0-2'		0.0029	0.0088	0.051	0.064
		3-5'		0.0032	0.0074	0.331	1.37
207	4						
208	5	0-2'		0.738	4.19	22.9	23.7
		5-7'		0.0036	0.015	1.49	1.96
Ave mm				0.154367	0.9957	8.040333	10.26567
Range				.003-0.74	.01-4.2	.05-22.9	.064-28.7
Median				0.3715	2.1	11.5	14.4
Ave ft				0.000506	0.003267	0.026379	0.03368

Boring no	**DOT pier no**	**Sample depth**	**D10, mm**	**D30, mm**	**D50, mm**	**D85, mm**	**D90, mm**
205	2	0-2'	0.002	0.029	0.463	3.970	5.800
		3-5'	0.004	0.150	1.290	19.500	28.700
206	3	0-2'		0.003	0.009	0.051	0.064
		3-5'		0.003	0.007	0.331	1.370
207	4	no samples					
208	5	0-2'		0.738	4.190	22.900	23.700
		5-7'		0.004	0.015	1.490	1.960
Ave mm				0.154	0.996	8.040	10.266
Range, mm				.003-0.74	.01-4.2	.05-22.9	.064-28.7
Median, mm				0.372	2.100	11.500	14.400
Ave ft				0.001	0.003	0.026	0.034

Madawaska International Bridge EOB 6/4/2020

FINAL DESIGN 60% plans

HEC 18 page 7.3. 169 out of 340

This spreadsheet computes simple pier scour.

For Madawaska, 3 trials are done, one for the pier stem, one for the footings, one for the seal

Pier numbers L to R facing DS

Ys/y1=2.0 K1 K2 K3((a/y1)^.65) * FR^0.43

Compute pier scour for seal

			Proj pier 5		proj pier 4		proj pier 3		proj pier 2		proj pier 1	
			100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Ys<= 2.4 times pier width for Fr < 0.8 and <3.0 for FR > 0.8		FR<0.8	86.4	86.4								
		FR>0.8	108	3.3								
Ys = scour depth, ft												
Y1 = flow depth just US of pier, ft			23.5	25.7	30.9	33.1	27.9	30.1	27.5	29.8	2.9	5.1
K1 = correction factor for pier nose shape (fig 7.3 or table 7.1)			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
K2= correction factor for angle of attack of flow from Table 7.2 or equation 7.4			1	1	1	1	1	1	1	1	1	1
K3= Correction factor for bed condition from table 7.3			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
a = pier width, ft	for seal	check thes	36	36	36	36	36	36	36	36	36	36
L = length of pier, ft			36	36	36	36	36	36	36	36	36	36
L/a			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
FR1 = Froude Number just US of pier			0.2	0.21	0.2	0.2	0.29	0.31	0.28	0.29	0.22	0.17
V1 = mean velocity of flow just US of pier, ft			5.4	5.15	8.4	9.2	8.7	9.6	8.3	9.1	1.5	2.1
g = acceleration of gravity, ft/s2			32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2
a/y1			1.5	1.4	1.2	1.1	1.3	1.2	1.3	1.2	12.4	7.1
a/y1^.65			1.3	1.2	1.1	1.1	1.2	1.1	1.2	1.1	5.1	3.6
FR^.43			0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.5	0.5
Ys/Y1			1.6	1.5	1.3	1.3	1.7	1.6	1.7	1.6	6.5	4.0
ys			37.6	39.6	41.3	42.3	46.8	49.5	45.9	47.9	18.8	20.5

Compute pier scour for footing

Ys/y1=2.0 K1 K2 K3((a/y1)^.65) * FR^0.43

			Proj pier 5		proj pier 4		proj pier 3		proj pier 2		proj pier 1	
			100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Ys<= 2.4 times pier width for Fr < 0.8 and <3.0 for FR > 0.8		FR<0.8	76.8	76.8								
		FR>0.8	96	3.3								
Ys = scour depth, ft												
Y1 = flow depth just US of pier, ft			23.5	25.7	30.9	33.1	27.9	30.1	27.5	29.8	2.9	5.1
K1 = correction factor for pier nose shape (fig 7.3 or table 7.1)			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
K2= correction factor for angle of attack of flow from Table 7.2 or equation 7.4			1	1	1	1	1	1	1	1	1	1
K3= Correction factor for bed condition from table 7.3			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
a = pier width, ft	for footing	check these	32	32	32	32	32	32	32	32	32	32
L = length of pier, ft			32	32	32	32	32	32	32	32	32	32
L/a			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
FR1 = Froude Number just US of pier			0.2	0.21	0.2	0.2	0.29	0.31	0.28	0.29	0.22	0.17
V1 = mean velocity of flow just US of pier, ft			5.4	5.15	8.4	9.2	8.7	9.6	8.3	9.1	1.5	2.1
g = acceleration of gravity, ft/s2			32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2
a/y1			1.4	1.2	1.0	1.0	1.1	1.1	1.2	1.1	11.0	6.3
a/y1^.65			1.2	1.2	1.0	1.0	1.1	1.0	1.1	1.0	4.8	3.3
FR^.43			0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.5	0.5
Ys/Y1			1.5	1.4	1.2	1.2	1.6	1.5	1.5	1.5	6.0	3.7
ys			34.8	36.7	38.3	39.2	43.3	45.8	42.5	44.4	17.4	19.0

Compute pier scour for pier stem

Ys/y1=2.0 K1 K2 K3((a/y1)^.65) * FR^0.43

			Proj pier 5		proj pier 4		proj pier 3		proj pier 2		proj pier 1	
			100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Ys<= 2.4 times pier width for Fr < 0.8 and <3.0 for FR > 0.8		FR<0.8	30	30								
		FR>0.8	37.5	37.5								
Ys = scour depth, ft												
Y1 = flow depth just US of pier, ft			23.5	25.7	30.9	33.1	27.9	30.1	27.5	29.8	2.9	5.1
K1 = correction factor for pier nose shape (fig 7.3 or table 7.1)			1	1	1	1	1	1	1	1	1	1
K2= correction factor for angle of attack of flow from Table 7.2 or equation 7.4			1	1	1	1	1	1	1	1	1	1
K3= Correction factor for bed condition from table 7.3			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
a = pier width, ft	pier stem		12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
L = length of pier, ft			10	10	10	10	10	10	10	10	10	10
L/a			0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
FR1 = Froude Number just US of pier			0.2	0.21	0.2	0.2	0.29	0.31	0.28	0.29	0.22	0.17
V1 = mean velocity of flow just US of pier, ft			5.4	5.15	8.4	9.2	8.7	9.6	8.3	9.1	1.5	2.1
g = acceleration of gravity, ft/s2			32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2
a/y1			0.5	0.5	0.4	0.4	0.4	0.4	0.5	0.4	4.3	2.5
a/y1^.65			0.7	0.6	0.6	0.5	0.6	0.6	0.6	0.6	2.6	1.8
FR^.43			0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.5	0.5
Ys/Y1			0.7	0.7	0.6	0.6	0.8	0.8	0.8	0.7	3.0	1.8
ys			17.2	18.1	18.9	19.4	21.4	22.6	21.0	21.9	8.6	9.4

This spreadsheet is used to assist with computation of complex pier scour, HEC-18 pages 7.22-7.23 EO 6/4/2020 Check NB 6/9/2020 corrections to final spreadsheet 6/10/2020

Complex Pier Scour per HEC18

For Madawaska, this sheet computes complex pier scour with piles

SCOUR VARIABLES		Scour variables for each pier									
pier L to R		1		2		3		4		5	
Dot pier # US to Canada		5		4		3		2		1	
		100-year	500	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Ys = scour depth, ft											
Y1 = flow depth just US of pier, ft		23.5	25.7	30.9	33.1	27.9	30.1	27.5	29.8	2.9	5.1
K1 = correction factor for pier nose shape (fig 7.3 or table 7.1)		0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
K2= correction factor for angle of attack of flow from Table 7.2 or equation 7.4		1	1	1	1	1	1	1	1	1	1
K3= Correction factor for bed condition from table 7.3		1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
a = pier width, ft	pier stem per MEDOT use 1.25 X pier width for debris	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
L = length of pier, ft		12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
L/a		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
FR1 = Froude Number just US of pier		0.2	0.21	0.2	0.2	0.29	0.31	0.28	0.29	0.22	0.17
V1 = mean velocity of flow just US of pier, ft		5.4	5.15	8.4	9.2	8.7	9.6	8.3	9.1	1.5	2.1
g = acceleration of gravity, ft/s2		32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2
f = distance from front end of footing to pier stem		13	13	13	13	13	13	13	13	13	13
h0= height of base of pile cap/footer above bed at start of computation (BOF-Bed elev)		-7	-7	-6	-6	-7	-7	-7	-7	-16	-16
T= height of footer or pile cap (TOF-BOF)	USE TOF-BOS - combine seal and footing	12	12	12	12	12	12	12	12	12	12
Ground elevation. Model	bottom 1	444.8	444.8	437.7	437.7	440.7	440.7	441.3	441.3	466	466
Ground elevation plans		441	441	438	438	441	441	441	441	474	474
top of footer	TOF	441	441	439	439	441	441	441	441	465	465
top of seal/base of footer	BOF	434	434	432	432	434	434	434	434	458	458
base of seal	BOS	429	429	427	427	429	429	429	429	453	453
rock elev		407	407	410	410	410	410	412	412	413	413
width of footer	Assume flow hits square edge	32									
width of seal	Combine Footing and Seal	36									
length cap	Assume Seal width for both	32									
length seal		36									

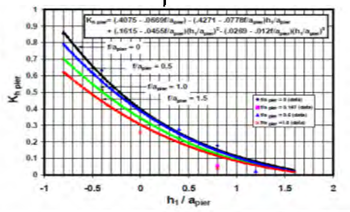


Figure 7.6. Suspended pier scour ratio (Jones and Sheppard 2000).

Pier Stem Scour

HEC 18 EQUA 7.1: Yspier/y1= Khpier(2 X K1 X k2 X K3 (apier/y1)^.65 X (v1/(gy1)^.5)0.43)

Pier Stem Scour, ys pier	y1, starting depth	h1, pier stem above bed	h1/a	f/a	Kh pier	2 X K1 X k2 X k3	(a/y1)^.65	((v1/(g*y1)^.5)^.43	ys pier, equa 7.33
		H0 + T			fig 7.6				
100									
5	23.5	0	0	1.04	0.34	1.98	0.7	0.50	5.21
4	30.9	1	0.08	1.04	0.38	1.98	0.6	0.57	7.31
3	27.9	0	0	1.04	0.34	1.98	0.6	0.59	6.55
2	27.5	0	0	1.04	0.34	1.98	0.6	0.58	6.40
500			0						
5	25.7	0	0	1.04	0.34	1.98	0.6	0.48	5.17
4	33.1	1	0.08	1.04	0.38	1.98	0.5	0.58	7.67
3	30.1	0	0	1.04	0.34	1.98	0.6	0.60	6.90
2	29.8	0	0	1.04	0.34	1.98	0.6	0.59	6.74

checked formulas w/HEC18 1/28

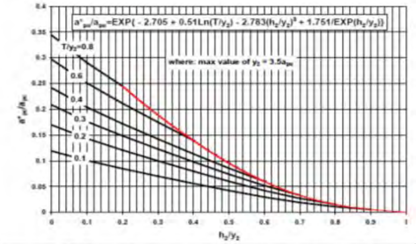


Figure 7.7. Pile cap (footing) equivalent width (Jones and Sheppard 2000).

Pile Cap or or Footing Scour Depth component

y2	y2	water surface	bottom 2	h0	h2	h2/y2	footing exposed, T1	T1/y2	V2	a*pc/apc	a*pc	apc-projected
Determine case 1 variables - bottom of pile cap or footing above the bed	y2= y1 +ys pier/2		bottom 1-y2	=bottom footing-bottom 1	ho+yspier/2	to use in fig 7.7	top of footing - bottom 2	to use in fig 7.7	V2=V1(y1/y2) - note case 1 and 2 are same	from fig 7.7	a*pc=(a*pc/apc)apc	
100				using bottom of seal		Assume 0 is less than 0						
5	26.11	468.3	442.19	-12	-9.39	-0.36	-1.19	-0.05	4.9	0.12	4.32	
4	34.55	468.6	434.05	-11	-7.35	-0.21	4.95	0.14	7.5	0.12	4.32	
3	31.17	468.6	437.43	-12	-8.73	-0.28	3.57	0.11	7.8	0.12	4.32	
2	30.70	468.8	438.10	-12	-8.80	-0.29	2.90	0.09	7.4	0.12	4.32	
500											0	
5	28.28	470.5	442.22	-12	-9.42	-0.33	-1.22	-0.04	4.7	0.12	4.32	
4	36.94	470.8	433.86	-11	-7.16	-0.19	5.14	0.14	8.2	0.12	4.32	
3	33.55	470.8	437.25	-12	-8.55	-0.25	3.75	0.11	8.6	0.12	4.32	
2	33.17	471.1	437.93	-12	-8.63	-0.26	3.07	0.09	8.2	0.12	4.32	

Determine Case 2 footing variables - bottom of footer/pile cap below bed	y2=y1+yspier/2	yf = h1 +yspier/2	Ks= D84,mm	Ks,ft	ln(10.93*yf/ks+1)	ln(10.93* y2/ks+1)	Vf= Equa 7.25 - V2 X colF/colG	y2/a footer	Kw	Kw primarily <1, therefore no wide pier correction
100-year - pier 5	26.11	2.6	8	0.026	7.0	9.3	3.7	0.73	0.81	
4	34.55	4.7	8	0.026	7.6	9.6	5.9	0.96	0.89	
3	31.17	3.3	8	0.026	7.2	9.5	5.9	0.87	1.10	
2	30.70	3.2	8	0.026	7.2	9.5	5.7	0.85	1.07	
500								0.00		
5	28.28	2.6	8	0.026	7.0	9.4	3.5	0.79	0.86	
4	36.94	4.8	8	0.026	7.6	9.6	6.5	1.03	0.91	
3	33.55	3.5	8	0.026	7.3	9.5	6.6	0.93	1.18	
2	33.17	3.4	8	0.026	7.3	9.5	6.2	0.92	1.12	

Pile Cap or footing scour, Yspc= (case 1, pile cap/footing above ground)	y2	X 2 X K1 X K2 X K3 X Kw	(a*pc/y2)^.65	(v2/(gy2)^.5)^.43	Yspc, Case 1, equation 7.24 pg 7.15
100		note: Kw not used, primarily <1			
5	26.11	1.98	0.31	0.46	7.45
4	34.55	1.98	0.26	0.53	9.33
3	31.17	1.98	0.28	0.55	9.34
2	30.70	1.98	0.28	0.54	9.14

Pile cap/footing below ground, Yspc, Case 2	yf	X2 X K1 X K2 X K3 X Kw	(a*pc/yf)^.65	(vf/(gyf)^.5)^.43	Yspc, Case 2, equation 7.26 pg 7.16
100		Kw assummed = 1			
1	2.61	1.98	1.39	0.67	4.83
2	4.65	1.98	0.95	0.73	6.43
3	3.27	1.98	1.20	0.79	6.13
4	3.20	1.98	1.21	0.78	5.99

500					
5	28.28	1.98	0.29	0.45	7.41
4	36.94	1.98	0.25	0.54	9.80
3	33.55	1.98	0.26	0.56	9.85
2	33.17	1.98	0.27	0.55	9.62

500					
1	2.58	1.98	1.40	0.66	4.73
2	4.84	1.98	0.93	0.76	6.73
3	3.45	1.98	1.16	0.82	6.45
4	3.37	1.98	1.18	0.80	6.28

Pile Group Scour Depth

Pile Group Effective width of equivalent pier= a*pg	Madawaska, 4 piles per pier, ALT 2b, 4 6' wide piers, projected is 12' due to layout			
aproj = sum of non overlapping widths of piles (fig 7.9 and 7.10)			12	
a pile			6	
aproj/a			2	
S			24	
S/a			4	
Ksp = coeff for pile spacing (fig 7.11)			0.62	0.62 calculated value for fig 7.11
Number of Pile Rows			2	
Km= coeff for number of aligned rows (fig 7.12)			1.13	1.13 calculated value for fig 7.12
Km = 1.0 for skewed of staggered pile groups				
a*pg=aproj*Ksp*Km			8.444114175	

y3 = adjusted flow depth and velocity for pile group- Equa. 7.29	Y3=Y2+Yspc/2 - recheck comps to use Case 1 or 2 above- for this project use Case 1 as Case 2 assumes no pile group scour	WSEL	bottom elev = WSEL-y3	h3 = bottom of cap-new bottom elev	H3/y3	Khpg, Fig 7.13	V3= V1(y1/y3), equa 7.30
100							
5	29.83	468.3	438.47	-9.47	-0.32	0	4.25
4	39.22	468.6	429.38	-2.38	-0.06	0	6.62
3	35.85	468.6	432.75	-3.75	-0.10	0	6.77
2	35.27	468.8	433.53	-4.53	-0.13	0	6.47
500							
5	31.99	470.5	438.51	-9.51	-0.30	0	4.14
4	41.83	470.8	428.97	-1.97	-0.05	0	7.28
3	38.48	470.8	432.32	-3.32	-0.09	0	7.51
2	37.98	471.1	433.12	-4.12	-0.11	0	7.14

NOTE: all scour elevations are above base of seal so no pile scour

Pile Group Scour, Yspg, equa 7.31	X 2 X K1 X K3	(a*pg/y3)^.65	(v3/(gy3)^.5)^.43	Yspg=y3 * Khpg *(2 X K1 X K3 Xcol C X colD)
100				
5	1.98	0.440284375	0.43	0
4	1.98	0.368545508	0.49	0
3	1.98	0.390732295	0.50	0
2	1.98	0.394848392	0.49	0
500				
5	1.98	0.420743749	0.41	0
4	1.98	0.353398986	0.50	0
3	1.98	0.373142002	0.51	0
2	1.98	0.376324525	0.51	0

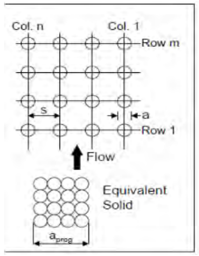


Fig 7.9

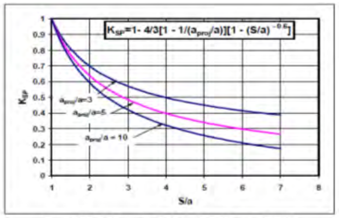
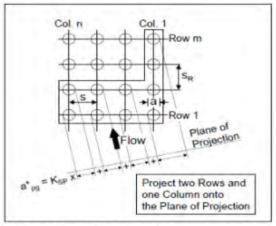


Figure 7.11. Pile spacing factor (refer to Sheppard 2001).



igure 7.10. Projected width of piles for the general case of skewed flow.

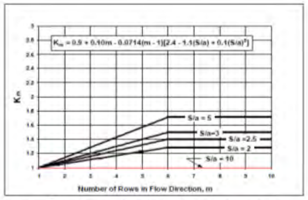


Figure 7.12 Adjustment factor for number of aligned rows of piles (refer to Sheppard 2001).

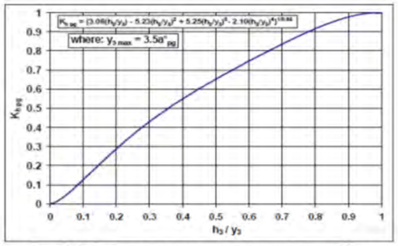


Figure 7.13. Pile group height adjustment factor (refer to Sheppard 2001)

Calculate Total Pier Scour

Total Scour , equa 7.22. ys=yspier + yspc + yspg	Yspier	Yspc, Case 1	Yspc Case 2	Yspg	Case 1 or 2 for pile cap?	Ytotal, case 1 - equa 7.22	Ytotal case 2-equa 7.27 = yspier+yspc(2)	bottom 1	Elev after scour	Rock Elev
100									sub for case 1 or 2	
5	5.21	7.45	4.83	0	1	12.66	10.04	444.8	434.76	407
4	7.31	9.33	6.43	0	1	16.64	13.74	437.7	423.96	410
3	6.55	9.34	6.13	0	1	15.89	12.68	440.7	428.02	410
2	6.40	9.14	5.99	0	1	15.54	12.40	441.3	428.90	412
500										
5	5.17	7.41	4.73	0	1	12.58	9.90	444.8	434.90	407
4	7.67	9.80	6.73	0	1	17.47	14.40	437.7	423.30	410
3	6.90	9.85	6.45	0	1	16.76	13.35	440.7	427.35	410
2	6.74	9.62	6.28	0	1	16.36	13.02	441.3	428.28	412

Summary	Bed Elev	Contractio	Pier Scour				Bottom of Seal	Rock Elev
100-year			scour with debris factor (1.25 x a	Aggredation/ Degredation - assume 2'.	Total Scour	Bed Elev after Scour		
pier 5	444.8	0.90	12.66	2.00	15.56	429.24	429	407
pier 4	437.7	0.90	16.64	2.00	19.54	418.16	427	410
pier 3	440.7	0.90	15.89	2.00	18.79	421.91	429	410
pier 2	441.3	0.90	15.54	2.00	18.45	422.85	429	412
500-year								0
pier 5	444.8	1.00	12.58	2.00	15.58	429.22	429	407
pier 4	437.7	1.00	17.47	2.00	20.47	417.23	427	410
pier 3	440.7	1.00	16.76	2.00	19.76	420.94	429	410
pier 2	441.3	1.00	16.36	2.00	19.36	421.94	429	412

Width of Scour hole -Total Sco	YS	2*ys (fig 7.18)
100		
1	10.0	20
2	13.7	27
3	12.7	25
4	12.4	25
500		
5	9.9	20
4	14.4	29
3	13.4	27
2	13.0	26

pier 5, pier stem, regular pier scour (recheck, final design for complex pier scour)

Pier 1-100	8.6	17
Pier 1-500	9.4	19

Effective Width of Pier with Debris: ICE?

Ice likely breaks up before 100 or 500 year flood, would not apply to footings- below ice

a	10	NOTE: a*d is approx
K1	0.79	same as 1.25 X a or
H	6.56	12.5
Width	25	USE 1.25 X a
y		a*d
100		
1	23.5	13.31
2	30.9	12.52
3	27.9	12.79
4	27.5	12.83
500		
1	25.7	13.02
2	33.1	12.35
3	30.1	12.58
4	29.8	12.61

$$a^*_d = \frac{K_1(HW) + (y - K_1H)a}{y}$$

where:

a^*_d	=	Effective width of pier when debris is present, ft (m)
a	=	Width of pier perpendicular to flow, ft (m)
K_1	=	0.79 for rectangular debris, 0.21 for triangular debris
H	=	Height (thickness) of the debris, ft (m)
W	=	Width of debris perpendicular to the flow direction, ft (m)
y	=	Depth of approach flow, ft (m)

This spreadsheet is used to assist with computation of complex pier scour, HEC-18 pages 7.22-7.23 EO 6/4/2020 Check NB 6/9/2020 corrections to final spreadsheet 6/10/2020

Complex Pier Scour per HEC18

For Madawaska, This sheet computes complex pier scour with full depth seal to bedrock.

SCOUR VARIABLES			Scour variables for each pier									
pier L to R			1		2		3		4		5	
Dot pier # US to Canada			5		4		3		2		1	
			100-year	500	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Ys = scour depth, ft												
Y1 = flow depth just US of pier, ft			23.5	25.7	30.9	33.1	27.9	30.1	27.5	29.8	2.9	5.1
K1 = correction factor for pier nose shape (fig 7.3 or table 7.1)			0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
K2= correction factor for angle of attack of flow from Table 7.2 or equation 7.4			1	1	1	1	1	1	1	1	1	1
K3= Correction factor for bed condition from table 7.3			1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
a = pier width, ft		pier stem per MEDOT use 1.25 X pier width for debris	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
L = length of pier, ft			12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
L/a			1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
FR1 = Froude Number just US of pier			0.2	0.21	0.2	0.2	0.29	0.31	0.28	0.29	0.22	0.17
V1 = mean velocity of flow just US of pier, ft			5.4	5.15	8.4	9.2	8.7	9.6	8.3	9.1	1.5	2.1
g = acceleration of gravity, ft/s2			32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2	32.2
f = distance from front end of footing to pier stem			13	13	13	13	13	13	13	13	13	13
h0= height of base of pile cap/footer above bed at start of computation (BOF-Bed elev)			-7	-7	-6	-6	-7	-7	-7	-7	-16	-16
T= height of footer or pile cap (TOF-BOF)		USE TOF-BOS - combine seal and footing	34	34	29	29	31	31	29	29	52	52
Ground elevation. Model	bottom 1		444.8	444.8	437.7	437.7	440.7	440.7	441.3	441.3	466	466
Ground elevation plans			441	441	438	438	441	441	441	441	474	474
top of footer	TOF		441	441	439	439	441	441	441	441	465	465
top of seal/base of footer	BOF		434	434	432	432	434	434	434	434	458	458
base of seal	BOS		407	407	410	410	410	410	412	412	413	413
rock elev			407	407	410	410	410	410	412	412	413	413
width of footer		Assume flow hits square edge	32									
width of seal		Combine Footing and Seal	36									
length cap		Assume Seal width for both	32									
length seal			36									

Pier Stem Scour

HEC 18 EQUA 7.1: Yspier/y1= Khpier(2 X K1 X k2 X K3 (apier/y1)^.65 X (v1/(gy1)^.5)0.43)

Pier Stem Scour, ys pier	y1, starting depth	h1, pier stem above bed	h1/a	f/a	Kh pier	2 X K1 X k2 X k3	(a/y1)^.65	((v1/(g*y1)^.5)^.43	ys pier, equa 7.33	
		H0 + T			fig 7.6					
100										
5	23.5	0	0	1.04	0.34	1.98	0.7	0.50	5.21	
4	30.9	1	0.08	1.04	0.38	1.98	0.6	0.57	7.31	
3	27.9	0	0	1.04	0.34	1.98	0.6	0.59	6.55	
2	27.5	0	0	1.04	0.34	1.98	0.6	0.58	6.40	
500			0							
5	25.7	0	0	1.04	0.34	1.98	0.6	0.48	5.17	
4	33.1	1	0.08	1.04	0.38	1.98	0.5	0.58	7.67	
3	30.1	0	0	1.04	0.34	1.98	0.6	0.60	6.90	
2	29.8	0	0	1.04	0.34	1.98	0.6	0.59	6.74	checked formulas w/HEC18 1/28

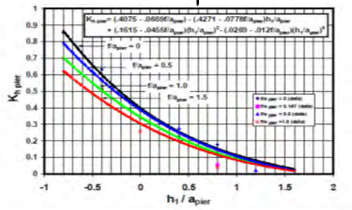


Figure 7.6. Suspended pier scour ratio (Jones and Sheppard 2000).

Pile Cap or or Footing Scour Depth component												
y2	y2	water surface	bottom 2	h0	h2	h2/y2	footing exposed, T1	T1/y2	V2	a*pc/apc	a*pc	apc-projected
Determine case 1 variables - bottom of pile cap or footing above the bed	y2= y1 +ys pier/2		bottom 1-y2 (using bottom in model)	=bottom footing-bottom 1	ho+yspier/2	to use in fig 7.7	top of footing - bottom 2	to use in fig 7.7	V2=V1(y1/y2) - note case 1 and 2 are same	from fig 7.7	a*pc=(a*pc/apc)apc	
100				using bottom of seal		Assume 0 is less than 0						
5	26.11	468.3	442.19	-34	-31.39	-1.20	-1.19	-0.05	4.9	0.12	4.32	
4	34.55	468.6	434.05	-28	-24.35	-0.70	4.95	0.14	7.5	0.12	4.32	
3	31.17	468.6	437.43	-31	-27.73	-0.89	3.57	0.11	7.8	0.12	4.32	
2	30.70	468.8	438.10	-29	-25.80	-0.84	2.90	0.09	7.4	0.12	4.32	
500											0	
5	28.28	470.5	442.22	-34	-31.42	-1.11	-1.22	-0.04	4.7	0.12	4.32	
4	36.94	470.8	433.86	-28	-24.16	-0.65	5.14	0.14	8.2	0.12	4.32	
3	33.55	470.8	437.25	-31	-27.55	-0.82	3.75	0.11	8.6	0.12	4.32	
2	33.17	471.1	437.93	-29	-25.63	-0.77	3.07	0.09	8.2	0.12	4.32	

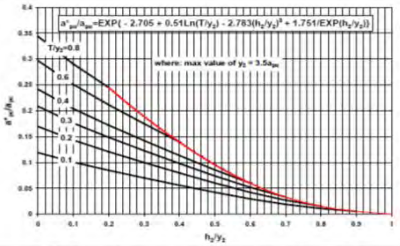


Figure 7.7. Pile cap (footing) equivalent width (Jones and Sheppard 2000).

Determine Case 2 footing variables - bottom of footer/pile cap below bed	y2=y1+yspier/2	yf = h1 +yspier/2	Ks= D84,mm	Ks,ft	ln(10.93*yf/ks+1)	ln(10.93* y2/ks+1)	Vf= Equa 7.25 - V2 X colF/colG	y2/a footer	Kw	Kw primarily <1, therefore no wide pier correction
100-year - pier 5	26.11	2.6	8	0.026	7.0	9.3	3.7	0.73	0.81	
4	34.55	4.7	8	0.026	7.6	9.6	5.9	0.96	0.89	
3	31.17	3.3	8	0.026	7.2	9.5	5.9	0.87	1.10	
2	30.70	3.2	8	0.026	7.2	9.5	5.7	0.85	1.07	
500								0.00		
5	28.28	2.6	8	0.026	7.0	9.4	3.5	0.79	0.86	
4	36.94	4.8	8	0.026	7.6	9.6	6.5	1.03	0.91	
3	33.55	3.5	8	0.026	7.3	9.5	6.6	0.93	1.18	
2	33.17	3.4	8	0.026	7.3	9.5	6.2	0.92	1.12	

Pile Cap or footing scour, Yspc= (case 1, pile cap/footing above ground)	y2	X 2 X K1 X K2 X K3 X Kw	(a*pc/y2)^.65	(v2/(gy2)^.5)^.43	Yspc, Case 1, equation 7.24 pg 7.15
100		note: Kw not used, primarily <1			
5	26.11	1.98	0.31	0.46	7.45
4	34.55	1.98	0.26	0.53	9.33
3	31.17	1.98	0.28	0.55	9.34
2	30.70	1.98	0.28	0.54	9.14

Pile cap/footing below ground, Yspc, Case 2	yf	X2 X K1 X K2 X K3 X Kw	(a*pc/yf)^.65	(vf/(gyf)^.5)^.43	Yspc, Case 2, equation 7.26 pg 7.16	USE Case 2 variables for full depth seal
100		Kw assummed = 1				
1	2.61	1.98	1.39	0.67	4.83	
2	4.65	1.98	0.95	0.73	6.43	
3	3.27	1.98	1.20	0.79	6.13	
4	3.20	1.98	1.21	0.78	5.99	

500						
5	28.28	1.98	0.29	0.45	7.41	
4	36.94	1.98	0.25	0.54	9.80	
3	33.55	1.98	0.26	0.56	9.85	
2	33.17	1.98	0.27	0.55	9.62	

500					
1	2.58	1.98	1.40	0.66	4.73
2	4.84	1.98	0.93	0.76	6.73
3	3.45	1.98	1.16	0.82	6.45
4	3.37	1.98	1.18	0.80	6.28

Pile Group Scour Depth

Pile Group Effective width of equivalent pier= a*pg — Madawaska, 4 piles per pier, ALT 2b, 4 6' wide piers, projected is 12' due to layout

aproj = sum of non overlapping widths of piles (fig 7.9 and 7.10)			12		
a pile			6		
aproj/a			2		
S			24		
S/a			4		
Ksp = coeff for pile spacing (fig 7.11)			0.62	0.62	calculated value for fig 7.11
Number of Pile Rows			2		
Km= coeff for number of aligned rows (fig 7.12)			1.13	1.13	calculated value for fig 7.12
Km = 1.0 for skewed of staggered pile groups					
a*pg=aproj*Ksp*Km			8.444114175		

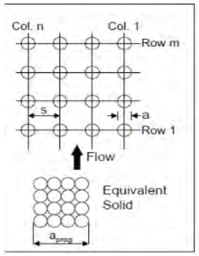


Fig 7.9

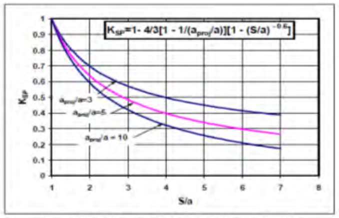
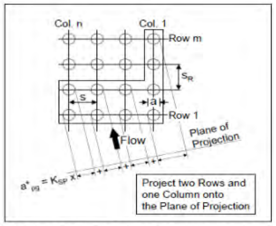


Figure 7.11. Pile spacing factor (refer to Sheppard 2001).

y3 = adjusted flow depth and velocity for pile group- Equa. 7.29	Y3=Y2+Yspc/2 - this set of comps uses case 2 for full depth seal	WSEL	bottom elev = WSEL-y3	h3 = bottom of cap-new bottom elev	H3/y3	Khpg, Fig 7.13	V3= V1(y1/y3), equa 7.30
100							
5	28.52	468.3	439.78	-32.78	-1.15	0	4.45
4	37.77	468.6	430.83	-20.83	-0.55	0	6.87
3	34.24	468.6	434.36	-24.36	-0.71	0	7.09
2	33.70	468.8	435.10	-23.10	-0.69	0	6.77
500							
5	30.65	470.5	439.85	-32.85	-1.07	0	4.32
4	40.30	470.8	430.50	-20.50	-0.51	0	7.56
3	36.78	470.8	434.02	-24.02	-0.65	0	7.86
2	36.31	471.1	434.79	-22.79	-0.63	0	7.47

NOTE: all scour elevations are above base of seal so no pile scour

Pile Group Scour, Yspg, equa 7.31	X 2 X K1 X K3	(a*pg/y3)^.65	(v3/(gy3)^.5)^.43	Yspg=y3 * Khpg *(2 X K1 X K3 Xcol C X colD)
100				
5	1.98	0.453315143	0.44	0
4	1.98	0.377661018	0.50	0
3	1.98	0.402533476	0.51	0
2	1.98	0.406738529	0.51	0
500				
5	1.98	0.432612831	0.43	0
4	1.98	0.362092918	0.51	0
3	1.98	0.384275041	0.53	0
2	1.98	0.387484548	0.52	0



igure 7.10. Projected width of piles for the general case of skewed flow.

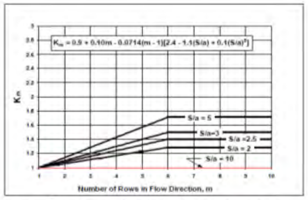


Figure 7.12. Adjustment factor for number of aligned rows of piles (refer to Sheppard 2001).

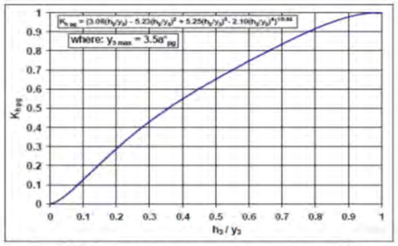


Figure 7.13. Pile group height adjustment factor (refer to Sheppard 2001).

Calculate Total Pier Scour

Total Scour , equa 7.22. ys=yspier + yspc + yspg	Yspier	Yspc, Case 1	Yspc Case 2	Yspg	Case 1 or 2 for pile cap?	Ytotal, case 1 - equa 7.22	Ytotal case 2-equa 7.27 = yspier+yspc(2)	bottom 1	Elev after scour	Rock Elev
100									sub for case 1 or 2	
5	5.21	7.45	4.83	0	2	10.04	10.04	444.8	434.76	407
4	7.31	9.33	6.43	0	2	13.74	13.74	437.7	423.96	410
3	6.55	9.34	6.13	0	2	12.68	12.68	440.7	428.02	410
2	6.40	9.14	5.99	0	2	12.40	12.40	441.3	428.90	412
500										
5	5.17	7.41	4.73	0	2	9.90	9.90	444.8	434.90	407
4	7.67	9.80	6.73	0	2	14.40	14.40	437.7	423.30	410
3	6.90	9.85	6.45	0	2	13.35	13.35	440.7	427.35	410
2	6.74	9.62	6.28	0	2	13.02	13.02	441.3	428.28	412

Summary	Bed Elev	Contractio	Pier Scour				Bottom of Seal	Rock Elev
100-year			scour with debris factor (1.25 x a	Aggredation/ Degredation - assume 2'.	Total Scour	Bed Elev after Scour		
pier 5	444.8	0.90	10.04	2.00	12.94	431.86	407	407
pier 4	437.7	0.90	13.74	2.00	16.64	421.06	410	410
pier 3	440.7	0.90	12.68	2.00	15.58	425.12	410	410
pier 2	441.3	0.90	12.40	2.00	15.30	426.00	412	412
500-year								0
pier 5	444.8	1.00	9.90	2.00	12.90	431.90	407	407
pier 4	437.7	1.00	14.40	2.00	17.40	420.30	410	410
pier 3	440.7	1.00	13.35	2.00	16.35	424.35	410	410
pier 2	441.3	1.00	13.02	2.00	16.02	425.28	412	412

Width of Scour hole -Total Sco	YS	2*ys (fig 7.18)
100		
1	10.0	20
2	13.7	27
3	12.7	25
4	12.4	25
500		
5	9.9	20
4	14.4	29
3	13.4	27
2	13.0	26

pier 5, pier stem, regular pier scour (recheck, final design for complex pier scour)

Pier 1-100	8.6	17
Pier 1-500	9.4	19

Effective Width of Pier with Debris: ICE?

Ice likely breaks up before 100 or 500 year flood, would not apply to footings- below ice

a	10	NOTE: a*d is approx
K1	0.79	same as 1.25 X a or
H	6.56	12.5
Width	25	USE 1.25 X a
y		a*d
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1	23.5	13.31
2	30.9	12.52
3	27.9	12.79
4	27.5	12.83
500		
1	25.7	13.02
2	33.1	12.35
3	30.1	12.58
4	29.8	12.61

$$a^*_d = \frac{K_1(HW) + (y - K_1H)a}{y}$$

where:

a^*_d = Effective width of pier when debris is present, ft (m)
a = Width of pier perpendicular to flow, ft (m)
K_1 = 0.79 for rectangular debris, 0.21 for triangular debris
H = Height (thickness) of the debris, ft (m)
W = Width of debris perpendicular to the flow direction, ft (m)
y = Depth of approach flow, ft (m)