Preliminary Hydrologic and Hydraulic Design Report

Mill Cove New Bridge #6205 over Mill Cove, WIN 026630.06

Robbinston, Maine

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Revised August 2024
June 2024
Project No. 2400963

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1. Introduction

This report provides the methods and findings for a Level 2 hydrologic and hydraulic analysis (Basic Analysis) to support the proposed replacement of Mill Cove New Bridge (MaineDOT Bridge #6205), which carries Route 1 over Western Stream, which outlets to Mill Cove, in Robbinston, Washington County, Maine (Figure 1).

Mill Cove New Bridge is a 15-foot-diameter, 156-foot-long, aluminum alloy structural plate pipe culvert that was constructed in 1967. Appendix A includes the 1967 bridge design plans. The existing site survey indicates the top of roadway is roughly El. 37, and the culvert inverts are El. 3.5 and El. 0.5 on the upstream and downstream sides, respectively.

According to the Highway Bridge Inspection Report dated September 7, 2021, the downstream 10 feet of culvert was “hanging” or unsupported based on culvert soundings. There were also missing and rusted bolts, a 2-inch drilled hole that was seeping water and gravel, and the concrete collar around the upstream end of the culvert was severely deteriorated.

From our discussions with Thornton Tomasetti, we understand the replacement bridge will likely consist of a single span, integral abutment bridge with a span length of about 135 feet. Appendix B includes information on the preliminary bridge design provided by Thornton Tomasetti.

This hydrology and hydraulic study relied on the field observations and gran size analyses conducted as part of the preliminary geotechnical studies.

Elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

2. Existing Data Review

This study relied on several readily available data sources to establish the basis for the hydraulic study and scour analysis, including the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for Washington County (FEMA, 2017) and National Oceanic and Atmospheric Association (NOAA) tides and currents data. MaineDOT provided peak flows for Western Stream and measured downstream water surface observations. Measured water surface elevations upstream of the crossing were not available.

The tidal datums near Mill Cove New Bridge were estimated using the NOAA Online Vertical Datum Transformation (VDATUM) (NOAA, 2023). The Highest Astronomical Tide (HAT) elevation was based on the HAT elevation reported at NOAA Station 8410834, Pettegrove Point, Dochet Island, ME (NOAA, 2010). This was listed as 24.14 ft Mean Lower Low Water (MLLW), which was then converted to an elevation referenced to NAVD88 by subtracting 13.82 ft following the datum conversion provided by the NOAA VDATUM tool. The tidal datums are summarized in the table below.

Table 2-1. Tidal Datums near Mill Cove New Bridge

Tidal Datum	Elevation (NAVD88, ft)
Mean Lower Low Water (MLLW)	-10.3
Mean Low Water (MLW)	-9.8
Mean Tide Level (MTL)	0.3
Mean High Water (MHW)	9.3
Mean Higher High Water (MHHW)	9.7
Highest Astronomical Tide (HAT)	13.9

3. Hydrology

MaineDOT developed a range of flows for Western Stream that flows under Mill Cove Bridge based on USGS Regression Equations and StreamStats (MaineDOT, 2023). The range of flows for Western Stream are summarized in the table below and included in Appendix C.

Table 3-1. Hydrology

Summary	
Drainage Area	6.2 mi^2
Q1.1	160 ft^3/s
Q10	595 ft^3/s
Q25	750 ft^3/s
Q50	880 ft^3/s
Q100	1,010 ft^3/s
Q500	1,300 ft^3/s

4. Hydraulic Analysis

This study included hydraulic analysis to estimate peak water surface elevation and velocity at the Mill Cove Bridge for a range of freshwater flows and tidal conditions for the existing and replacement crossing configurations. The hydraulic model was developed using the U.S. Bureau of Reclamation (USBR) Sedimentation and River Hydraulics – Two-Dimension (SRH-2D) version 13.3.10 (SMS, 2024). The model was used to simulate 2-dimensional (2D) flows with unsteady analysis, the full momentum wave equation set, and a 2-second fixed computation interval.

4.1. Model Setup

The model simulated flow along a 0.3-mile-reach of Western Stream from a location upstream of the Mill Cove Bridge crossing and extended 360 ft downstream of the crossing into the tidal side of Mill Cove. The model was used to simulate flows through the existing hydraulic structure and a proposed single-span bridge configuration. The model element sizes ranged from approximately 2 ft near the hydraulic crossings to approximately 50 ft near the boundaries of the model.

The digital elevation model (DEM) (i.e., terrain) for the model was compiled from the following data sources: a survey performed by MaineDOT of the channel bottom and existing crossing elevations and LiDAR collected in 2017 (OCM Partners, 2024). The terrain was further refined for both existing and proposed crossing configurations for an assumed removal of the Old Route 1 Bridge located approximately 100 ft upstream of the Mill Cove Bridge crossing with a revised channel width of 33.5 ft and bank slopes of 2H:1V at the Old Route 1 Bridge location. The existing culvert dimensions were based on the 1967 as-built design plans (ME State Highway Commission, 1967). The culvert was modeled as a 15-ft-diameter, 156-ft-long pipe with an upstream invert El. 3.5 ft and a downstream invert El. 0.5 ft. The culvert was represented in the model as a 3D structure with a Manning's n-value of 0.02. Figure 4-1 displays the topographic and bathymetric surface used in the SRH-2D model for the existing conditions.

The proposed single span bridge for Mill Cove was represented in the model by modifying the terrain to represent a single span bridge with a channel bottom width of 15 ft and side slopes of 1.75H:1V. The channel bottom elevation ranged from El. 0.25 ft upstream to El. -0.25 ft downstream. Based on information from Thornton Tomasetti, the low chord of the proposed bridge would be El. 28.0 ft, which would be above the peak water surface elevation for the scenarios included in this study. Because of this, the superstructure of the proposed bridge was not included in the model. Figure 4-2 displays the topographic and bathymetric surface used in the SRH-2D model for the proposed structure.

The model mesh was created using quadrilateral and triangular elements, with primarily quadrilateral elements within the channel to align the cell faces of the mesh perpendicular to the direction of water flow and in line with linear features such as the crossing and channel banks. Triangular elements were used above the channel banks and in the downstream, tidal portion of the mesh where the flow direction was less linear. The model mesh is shown in Figure 4-3.

The 2D flow area included spatially varied Manning's n-values based on the 2006 Maine Land Cover Database (MELCD) (MEGIS, 2006). Manning's n-values were assigned to land cover groups based on Chow (1959), USGS (2015), NRCS (2010), and our engineering judgement.

Figure 4-1. SRH-2D model topography and bathymetry for existing conditions

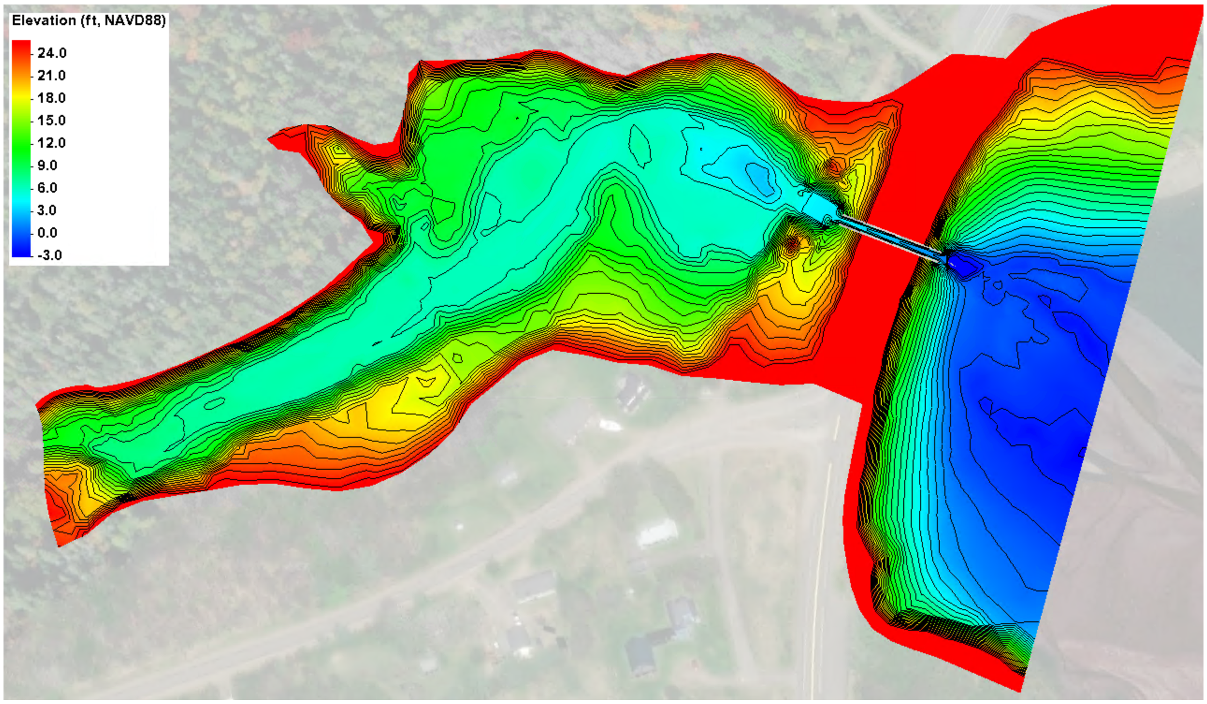


Figure 4-2. SRH-2D model topography and bathymetry for proposed structure

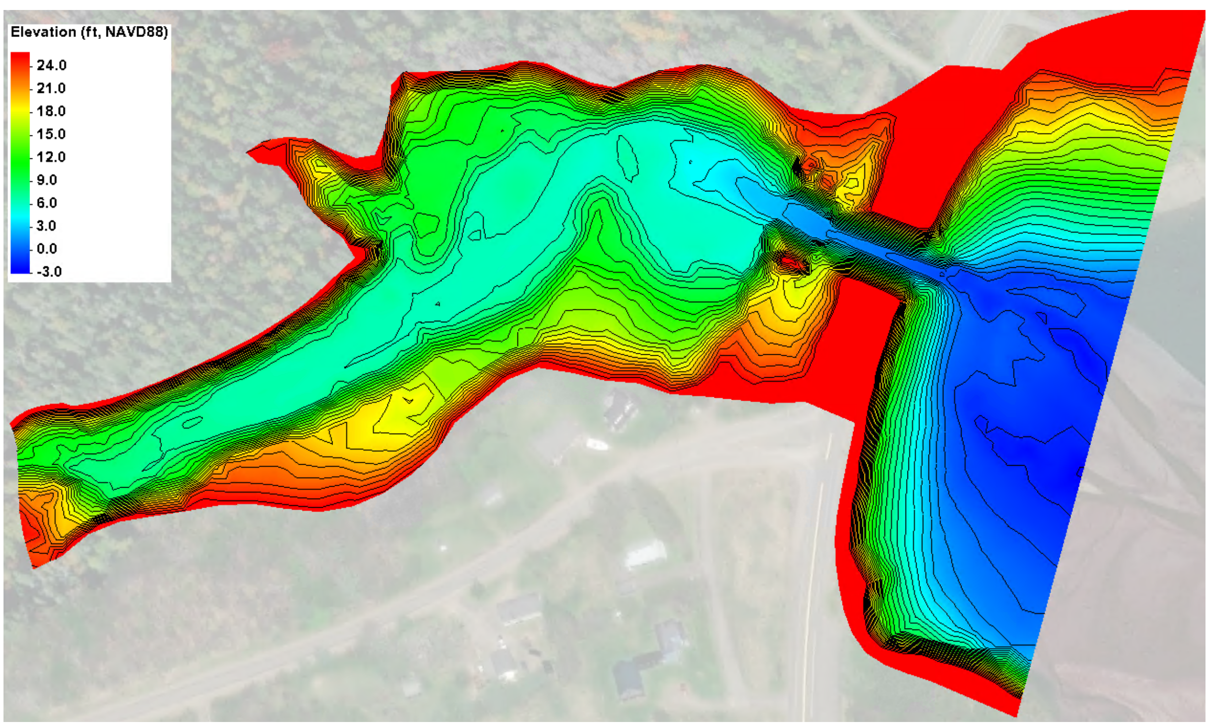
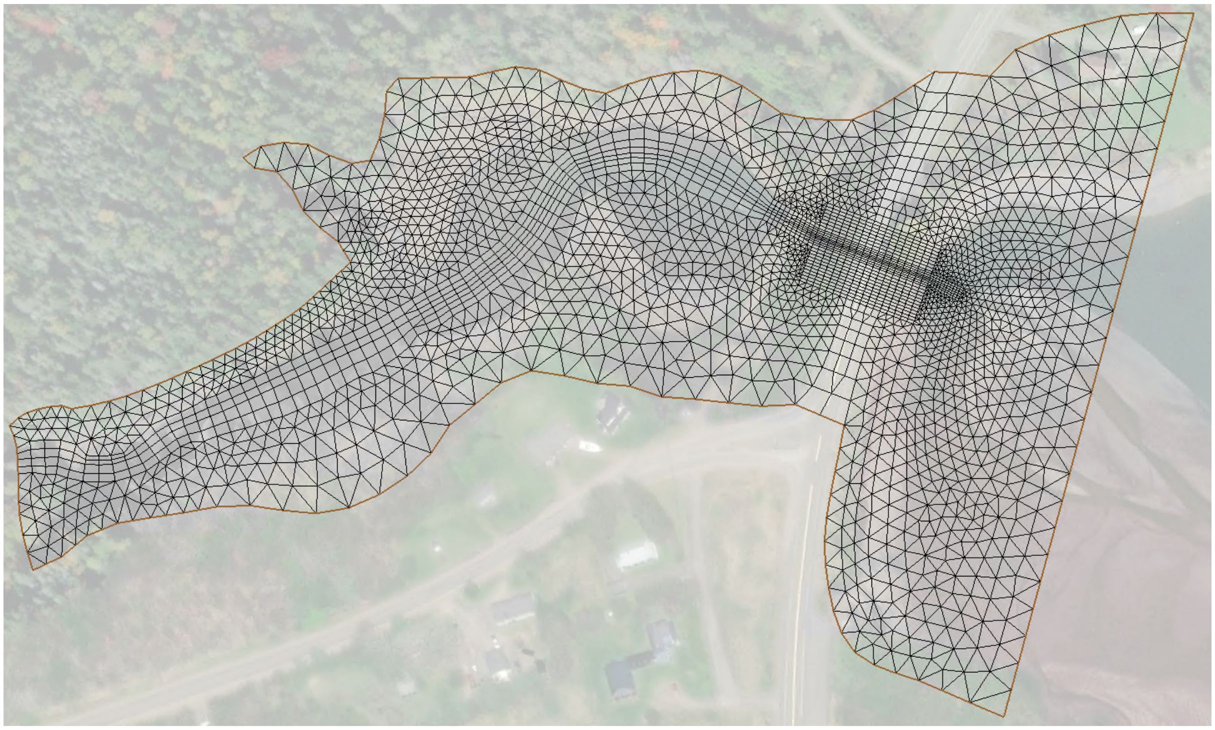


Figure 4-3. SRH-2D model mesh



4.2. Boundary Conditions

Freshwater flows were input as constant flows to the upstream boundary of the model. A stage hydrograph was used at the downstream end of the model to represent tidal scenarios. Tidal stage hydrographs were developed using the NOAA Tide Gage in Eastport, ME and water elevation measurements collected by MaineDOT in Perry, Maine as part of this study (NOAA, 2024). Four tidal stage hydrographs were developed: "average" tides, "average" tides plus 4.0 ft of sea level rise (SLR), 2% annual chance ("50-yr") storm tides, and 2% annual chance storm tides plus 4.0 ft of SLR.

The stage hydrographs representing "average" tides were developed by adjusting observed water surface elevations from the NOAA Tide Gage in Eastport, ME, between the period of January 20 and January 23, 2024, up 0.2 ft so that the high tides more closely represented the high tides measured by MaineDOT in Perry, Maine. To represent SLR, the stage hydrograph was adjusted up linearly by 4.0 ft.

The stage hydrographs representing 2% annual chance storm surge events were developed using tidal observations collected from the NOAA Tide Gage in Eastport, ME, between the period of January 12 and January 15, 2024, during a period of coastal surge. According to the NOAA Tide Gage, tidal observations during the January 2024 event peaked at 13.99 ft, which is 0.31 ft below the 2% annual chance stillwater elevation of 14.3 ft as reported by the FEMA in the Washington County Flood Insurance Study (FEMA, 2017). The tidal observations were adjusted linearly upwards by 0.31 ft so that the peak water surface elevation reached El. 14.3 ft. To represent SLR, the stage hydrograph was adjusted up linearly by 4.0 ft for a peak El. 18.3 ft.

Twelve scenarios representing a range of freshwater flows and tidal conditions were simulated in the model for the existing and proposed structure. The scenarios were selected based on a review of the MaineDOT Bridge Design Guide (2003), Hydraulic Circular No. 18 (HEC-18), and Hydraulic Circular No. 25 (HEC-25) under the U.S. Department of Transportation, Federal Highway Administration.

Model calibration was not performed as part of this study due to lack of water surface observations upstream of the crossing.

4.3. Model Results

The model results suggest that the crossing is tidally influenced during high-tides and has no tidal-influence during low tides. Results are summarized for both high tide and low tide conditions in Table 4-1 below.

The hydraulic results of these scenarios are summarized in the table below. Appendix D includes maximum water depths at high tide for the modeled scenarios. Appendix E includes maximum velocities at low tide for the modeled scenarios.

Table 4-1. Hydraulic Analysis Summary

Scenario	Value	Existing Structure 15-ft-diameter, 156-ft-long Culvert	Proposed Structure Single Span Bridge
Total Area of Waterway Opening, ft^2		176.7	1,777.1
$Q_{1.1}$ Avg. Tides	Peak Headwater: High Tide, ft	10.30	10.28
	Peak Tailwater: High Tide, ft	10.28	10.28
	PHHD: High Tide, ft	0.02	0.00
	Freeboard: High Tide, ft	26.7	17.7
	Velocity at Peak High Tide, ft/s	2.3	0.4
	Headwater: Low Tide, ft	6.86	2.96
	Tailwater: Low Tide, ft	2.41	0.67
	Peak Velocity at Low Tide, ft	15.1	7.4
$Q_{1.1}$ Avg. Tides 4 ft SLR	Peak Headwater: High Tide, ft	14.29	14.28
	Peak Tailwater: High Tide, ft	14.28	14.28
	PHHD: High Tide, ft	0.01	0.00
	Freeboard: High Tide, ft	22.7	13.7
	Velocity at Peak High Tide, ft/s	0.3	0.04
	Headwater: Low Tide, ft	6.86	2.96
	Tailwater: Low Tide, ft	2.40	0.67
	Peak Velocity at Low Tide, ft	15.2	7.4
$Q_{1.1}$ 2% annual chance coastal storm event	Peak Headwater: High Tide, ft	14.41	14.40
	Peak Tailwater: High Tide, ft	14.39	14.39
	PHHD: High Tide, ft	0.01	0.00
	Freeboard: High Tide, ft	22.6	13.6
	Velocity at Peak High Tide, ft/s	0.6	0.3

Scenario	Value	Existing Structure 15-ft-diameter, 156-ft-long Culvert	Proposed Structure Single Span Bridge
	Headwater: Low Tide, ft	6.86	2.96
	Tailwater: Low Tide, ft	2.40	0.67
	Peak Velocity at Low Tide, ft	15.2	7.4
$Q_{1.1}$ 2% annual chance coastal storm event 4 ft SLR	Peak Headwater: High Tide, ft	18.41	18.39
	Peak Tailwater: High Tide, ft	18.39	18.39
	PHHD: High Tide, ft	0.02	0.00
	Freeboard: High Tide, ft	18.6	9.6
	Velocity at Peak High Tide, ft/s	0.8	0.1
	Headwater: Low Tide, ft	6.86	2.96
	Tailwater: Low Tide, ft	2.40	2.52
	Peak Velocity at Low Tide, ft	15.1	7.4
Q_{10} Avg. Tides	Peak Headwater: High Tide, ft	10.85	10.29
	Peak Tailwater: High Tide, ft	10.27	10.27
	PHHD: High Tide, ft	0.58	0.01
	Freeboard: High Tide, ft	26.2	17.7
	Velocity at Peak High Tide, ft/s	9.9	2.6
	Headwater: Low Tide, ft	10.32	5.12
	Tailwater: Low Tide, ft	3.88	2.17
	Peak Velocity at Low Tide, ft	21.8	12.1
Q_{10} Avg. Tides 4 ft SLR	Peak Headwater: High Tide, ft	14.47	14.28
	Peak Tailwater: High Tide, ft	14.29	14.28
	PHHD: High Tide, ft	0.18	0.00
	Freeboard: High Tide, ft	22.5	13.7
	Velocity at Peak High Tide, ft/s	5.7	1.3
	Headwater: Low Tide, ft	10.33	5.12
	Tailwater: Low Tide, ft	3.88	2.17
	Peak Velocity at Low Tide, ft	21.8	12.1
Q_{50} Avg. Tides	Peak Headwater: High Tide, ft	12.20	10.30
	Peak Tailwater: High Tide, ft	10.25	10.27
	PHHD: High Tide, ft	1.94	0.03
	Freeboard: High Tide, ft	24.8	17.7
	Velocity at Peak High Tide, ft/s	11.2	3.9
	Headwater: Low Tide, ft	12.18	6.04
	Tailwater: Low Tide, ft	4.66	2.88
	Peak Velocity at Low Tide, ft	23.8	13.7
Q_{50} Avg. Tides 4 ft SLR	Peak Headwater: High Tide, ft	14.78	14.28
	Peak Tailwater: High Tide, ft	14.31	14.27
	PHHD: High Tide, ft	0.47	0.01
	Freeboard: High Tide, ft	22.2	13.7
	Velocity at Peak High Tide, ft/s	8.8	2.0
	Headwater: Low Tide, ft	12.18	6.04

Scenario	Value	Existing Structure 15-ft-diameter, 156-ft-long Culvert	Proposed Structure Single Span Bridge
	Tailwater: Low Tide, ft	4.66	2.88
	Peak Velocity at Low Tide, ft	23.8	13.7
Q_{100} Avg. Tides	Peak Headwater: High Tide, ft	12.95	10.31
	Peak Tailwater: High Tide, ft	10.18	10.26
	PHHD: High Tide, ft	2.77	0.04
	Freeboard: High Tide, ft	24.1	17.7
	Velocity at Peak High Tide, ft/s	11.7	4.5
	Headwater: Low Tide, ft	12.94	6.40
	Tailwater: Low Tide, ft	4.97	3.15
	Peak Velocity at Low Tide, ft	24.6	14.2
Q_{100} Avg. Tides 4 ft SLR	Peak Headwater: High Tide, ft	14.99	14.28
	Peak Tailwater: High Tide, ft	14.32	14.27
	PHHD: High Tide, ft	0.67	0.01
	Freeboard: High Tide, ft	22.0	13.7
	Velocity at Peak High Tide, ft/s	10.0	2.3
	Headwater: Low Tide, ft	12.94	6.40
	Tailwater: Low Tide, ft	4.97	3.15
	Peak Velocity at Low Tide, ft	24.5	14.2
Q_{500} Avg. Tides	Peak Headwater: High Tide, ft	14.61	10.34
	Peak Tailwater: High Tide, ft	9.87	10.26
	PHHD: High Tide, ft	4.74	0.08
	Freeboard: High Tide, ft	22.4	17.7
	Velocity at Peak High Tide, ft/s	12.6	5.8
	Headwater: Low Tide, ft	14.61	7.20
	Tailwater: Low Tide, ft	5.63	3.66
	Peak Velocity at Low Tide, ft	26.0	15.2
Q_{500} Avg. Tides 4 ft SLR	Peak Headwater: High Tide, ft	15.61	14.29
	Peak Tailwater: High Tide, ft	14.35	14.27
	PHHD: High Tide, ft	1.26	0.02
	Freeboard: High Tide, ft	21.4	13.7
	Velocity at Peak High Tide, ft/s	12.2	3.0
	Headwater: Low Tide, ft	14.61	7.20
	Tailwater: Low Tide, ft	5.62	3.66
	Peak Velocity at Low Tide, ft	26.0	15.2

Notes:
1. Peak Hydraulic Head Difference (PHHD) refers to the Hydraulic Head Difference between the peak headwater elevation and the peak tailwater elevation.
2. Freeboard is measured as the distance from the peak headwater elevation to the road surface elevation (approximately El. 37.0 ft) for the existing culvert crossing and from the peak headwater elevation to the low chord El. 28.0 ft for the proposed bridge structure.

5. Coastal Waves

We have included an investigation of coastal waves at this crossing location to evaluate the water surface elevation at the downstream side of the crossing in relation to the MaineDOT tidal hydraulic guidance and to estimate coastal abutment scour.

The wave height used in the freeboard evaluation was based on the significant wave height from the FEMA FIS transect data for Coastal Transect 4 (FEMA, 2017). This significant wave height was calculated using wave transformation modeling for a 1% annual chance coastal storm event with a stillwater El. 14.7 ft and wind speeds representative of a 1% annual chance coastal storm. The wave transformation modeling used to obtain this significant wave height is less conservative than assuming a significant wave height equal to the breaking wave height (which is an adequate assumption to use for shallower water depths and when more detailed modeling data is not present) but should still be considered conservative compared to wave heights that may occur during Mean-High Water (MHW) conditions for present-day sea levels (El. 9.3 ft) and for MHW with 4 ft of SLR (El. 13.3 ft), the latter of which is the suggested scenario to use when evaluating freeboard within the MaineDOT tidal hydraulic guidance.

The significant wave height from FEMA Coastal Transect 4 is 5.2 ft. Considering a 5.2 ft wave, an existing MHW El. 9.3 ft, and 4.0 ft of SLR, the peak water surface elevation, including wave heights, along the downstream side of the structure would be approximately 18.5 ft, which is 9.5 ft below the low chord elevation of the proposed bridge (El. 28.0 ft). This analysis indicates the proposed design would meet the MaineDOT tidal hydraulic guidance for freeboard of 2.0 ft above Q_{10} flow for MHW tidal conditions with sea level rise, including wave heights.

6. Scour Analysis

A scour analysis was performed for the proposed single-span bridge to evaluate contraction and abutment scour depths. Since there are no piers in the proposed design, pier scour was not performed. Additionally, abutment scour along the coastal-facing abutment due to wave action was estimated for the scenarios included in this study. Pressure scour would not be present for the flow scenarios evaluated as part of this study since the low chord of the proposed bridge would be higher than the water surface elevations. The scour analysis methodology is discussed below followed by a summary of estimated scour depth results.

6.1. Contraction and Abutment Scour

A scour analysis was performed for the proposed single-span bridge. The scour estimates were developed based on Hydraulic Guidance Circular No. 18 (HEC-18) under the U.S. Department of Transportation, Federal Highway Administration (FHWA, 2012). The scour analysis was performed using the Bridge Scour Analysis program within the FHWA Hydraulic Toolbox Program using the results of the SRH-2D hydraulic model.

The scour analysis was performed for two separate grain size distributions from boring samples collected at the site to understand the sensitivity of grain size in the analysis and to obtain a range of results for representative grain sizes present at the site. The grain size distributions used were from boring BB-RMC-103 Sample 8D, which was collected from El. 0.1 ft to El. 2.0 ft, and BB-RMC-104 Sample 5D, which was collected from El. 5.8 ft to El. 7.8 ft. The grain size analyses indicated that the samples had D_{50} values of approximately 3.49 mm and 0.28 mm, respectively. Additional information on boring locations, boring logs, and grain size is provided in GEI's preliminary geotechnical design report developed under a separate task from this hydrologic and hydraulic report.

Scour was calculated using hydraulic model results for low tide conditions when the flows would not be influenced by the tide. These conditions were found to be the governing conditions for scour depth due to the higher velocities that would likely be present within the contracted channel.

The hydraulic model results indicated that approach velocities were greater than the critical velocity for the grain size and, therefore, contraction scour depth within the Bridge Scour Analysis program was based on a live bed contraction scour analysis. Abutment scour was calculated for the National Cooperative Highway Research Program (NCHRP) Scour Condition A.

The scour analysis results for contraction and abutment scour are summarized in Table 6-1.

6.2. Coastal Abutment Scour

Coastal abutment scour was estimated for the abutments on the downstream (ocean) side by approximating scour depth due to wave height, as suggest by the Coastal Engineering Manual (CEM) (USACE, 2002).

The wave height used to estimate scour depth for most scenarios included in this study was based on the FEMA FIS significant wave height data for Coastal Transect 4, which represents a wave height for a 1% annual chance coastal storm with a stillwater El. 14.7 ft. The scenarios evaluated as part of this study had peak water surface elevations downstream of the crossing less than El. 14.7 ft, except for the $Q_{1.1}$ condition with 2% annual chance coastal storm tides plus 4.0 ft of SLR, which had a peak downstream water surface at El. 18.4 ft. Apart from this scenario, coastal abutment scour depth was approximated as 5.2 ft, which is equal to the Coastal Transect 4 significant wave height.

For the $Q_{1.1}$ condition with 2% annual chance coastal storm tides plus 4.0 ft of SLR with a downstream water surface El. 18.4 ft, coastal abutment scour depth can be conservatively approximated as the depth-limited breaking wave height. For an approximate abutment toe at El. -1.0 ft and a peak water surface at El. 18.4 ft, the depth at the structure toe would be approximately 19.4 ft. The depth-limited breaking wave height, approximately 0.8 times the breaking wave depth, was calculated to be 15.5 ft. For lack of more detailed coastal modeling, the coastal abutment scour depth for this flow scenario would be approximately 15.5 ft.

The scour analysis results for coastal abutment scour are summarized in Table 6-1.

6.3. Scour Analysis Results

The scour analysis results suggest that the Q_{500} flow conditions would likely cause the greatest amount of scour compared to the other flow scenarios included in this evaluation. Under Q_{500} conditions, contraction scour would be approximately 7.8 ft deep and abutment scour would be approximately 9.8 ft deep. Appendix F includes a scour profile at the bridge centerline for the Q_{500} conditions.

Coastal abutment scour due to wave action on the downstream face of the abutments was approximated to be 5.2 ft for the scenarios included in this study, except for when the tidal conditions represented a 2% annual chance coastal storm plus 4 ft of sea level rise when the coastal abutment scour was approximated to be 15.5 ft.

The difference in scour depths for the grain size distributions of the two sample locations evaluated in this study was minimal. The scour analysis results suggest that scour depth would be the same for many of the flow scenarios included in this study for the two separate grain size distributions evaluated, with a difference of up to 0.5 ft occurring for the Q_{50} flow scenario.

The scour results are likely conservative. The abutment scour computations do not take scour protection into account, such as riprap or other erosion protection, which would be expected to be used along the abutment. Additionally, the tidal nature of the crossing presents several areas of uncertainty in the scour evaluation. The time-nature of tides means that peak velocities occur for discrete amounts of time during the tidal cycle and may not have duration long enough for scour to reach the estimated depths presented above. The coastal abutment scour depths are also likely conservative due to lack of detailed coastal modeling available for the scenarios included in this evaluation. These values could be refined by extracting wave heights from the Maine Coastal Flood Risk Model (ME-CFRM) when they become available.

The scour analysis results for are summarized in the Table 6-1.

Table 6-1. Scour Analysis Summary

Scenario	Live Bed Contraction Scour Depth (ft)		Abutment Scour Depth (ft)		Coastal Abutment Scour Depth (ft)
	BB-RMC-103 8D	BB-RMC-104 5D	BB-RMC-103 8D	BB-RMC-104 5D	
$Q_{1.1}$ Avg. Tides	0.5	0.5	1.4	1.4	5.2
$Q_{1.1}$ Avg. Tides 4 ft SLR	0.5	0.5	1.4	1.4	5.2
$Q_{1.1}$ 2% annual chance coastal storm event	0.5	0.5	1.4	1.4	5.2
$Q_{1.1}$ 2% annual chance coastal storm event 4 ft SLR	0.5	0.5	1.4	1.4	15.5
Q_{10} Avg. Tides	4.1	4.1	5.5	5.5	5.2
Q_{10} Avg. Tides 4 ft SLR	4.1	4.1	5.5	5.5	5.2
Q_{50} Avg. Tides	6.2	6.7	8.4	8.4	5.2
Q_{50} Avg. Tides 4 ft SLR	6.2	6.7	8.4	8.4	5.2
Q_{100} Avg. Tides	6.8	7.1	9.0	9.0	5.2
Q_{100} Avg. Tides 4 ft SLR	6.8	7.0	8.9	8.9	5.2
Q_{500} Avg. Tides	7.7	7.8	9.8	9.8	5.2
Q_{500} Avg. Tides 4 ft SLR	7.6	7.6	9.6	9.6	5.2

6.4. Stone Size Recommendation

To evaluate scour protection against erosion of abutments, we performed analysis to size riprap at the abutments using guidance from the HEC-23 Volume II Design Guideline 14 (FHWA, 2009). The analysis considered protection against abutment erosion for the proposed bridge design. This analysis did not evaluate abutment overtopping protection since none of the flow scenarios would be expected to overtop the proposed bridge. The existing culvert was not evaluated. This analysis did not address potential geotechnical instability but rather evaluated riprap sizes that would minimize the potential for contraction and local scour at abutments.

The approach taken in this report was based on Equation 14.1 from the HEC-23 guidance as follows:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

where:

D_{50} = median stone diameter, ft
V = characteristic average velocity in the contracted section, ft/s
S_s = specific gravity of rock riprap, 2.65 (unitless)
g = gravitational acceleration, 32.2 ft/s^2
y = depth of flow in the contracted bridge opening, ft
K = 0.61 for spill-through abutments for Froude Numbers >0.81

We performed the scour analysis in a Microsoft Excel Spreadsheet using the HEC-23 equation and the velocities and depths from the hydraulic model for the five flow scenarios. The results from this analysis are summarized in the table below.

Table 6-2. Riprap Size Analysis Summary

Parameter	Q500	Q100	Q50	Q10	Q1.1
K	0.61	0.61	0.61	0.61	0.61
S_s	2.65	2.65	2.65	2.65	2.65
g (ft/s^2)	32.2	32.2	32.2	32.2	32.2
y (ft)	5.1	4.5	4.2	3.3	1.7
V (ft/s)	11.9	11	10.7	9.8	6.5
D_{50}	**1.8**	**1.6**	**1.5**	**1.2**	**0.6**

The analysis indicated that a median stone diameter of 1.8 ft would provide a stable stone size for riprap layers to protect the proposed abutments from scour for the Q_{500}. The other flow conditions, with lower annual recurrence interval flows, would warrant median stone diameters ranging from 0.6 ft to 1.6 ft, but it would be prudent to design for the flow condition resulting in the largest riprap size.

These results are preliminary and will be further reviewed and developed upon receiving the final bridge design. We recommend that the bridge design group review these estimates for riprap median stone diameter. Furthermore, we assume that the bridge design group will develop design information for the riprap that is consistent with, for example, the Maine Bridge Design Guide (MaineDOT, 2003), which indicate, for example, recommendations for the design of riprap thickness, toe construction, bedding material, geotextile filter material.

7. Summary of Findings and Recommendations

- The hydraulic characteristics of Western Stream that outlets to Mill Cove are dependent on the stage of the tide. During low tide, Western Stream at Mill Cove New Bridge is not influenced by the tide and the hydraulic characteristics (i.e. velocity, depth of flow) are depended on inflow conditions. As the tide rises above the elevation of the channel at the downstream end of the crossing, approximate El. -1.0 ft, Western Stream becomes tidally influenced and the stage of the water will increase with the rising tide.
- The model results suggest that the proposed bridge structure meets the MaineDOT general tidal hydraulic guidance of a maximum 0.25-inch head difference between upstream and downstream at MHHW. This condition is met for the $Q_{1.1}$ and Q_{10} flow scenarios evaluated as part of this study, including for the four tidal scenarios associated with these two inflow conditions. The peak hydraulic head difference (PHHD) for average tidal conditions and present-day sea levels for the Q_{50}, Q_{100}, and Q_{500} flow scenarios exceeds 0.25-inches (0.02 ft).
- The freeboard guidance within the MaineDOT tidal hydraulic guidance document states a minimum freeboard of 2.0 ft above Q_{10} flow for mean-high-water (MHW) tidal conditions with sea level rise, including wave heights. Considering a 5.2 ft wave, an existing MHW El. 9.3 ft, and 4.0 ft of SLR, the peak water surface elevation, including wave heights, along the downstream side of the structure would be approximately 18.5 ft, which is 9.5 ft below the proposed low chord elevation of the bridge (El. 28.0 ft). This would meet the MaineDOT tidal hydraulic guidance for freeboard.
- The MaineDOT tidal hydraulic guidance document states that the roadway finished grade shall be 4.0 ft above the Highest Astronomical Tide (HAsT). HAsT is approximately El. 13.9 ft based on a VDATUM conversion of water elevations from the Pettegrove Point NOAA Station (NOAA, 2010). The finished grade of the proposed structure would be approximately El. 36.5 which would meet this guidance.
- The scour analysis results suggest that contraction scour of up to 7.1 ft and 7.8 ft would be likely for Q_{100} conditions and Q_{500} conditions, respectively. The scour analysis results suggest that abutment scour of up to 9.0 ft and 9.8 ft would be likely for Q_{100} conditions and Q_{500} conditions, respectively.
- Abutment scour along the coastal face of the abutment due to wave action was approximated using wave heights. For most flow scenarios evaluated, coastal abutment scour was estimated to be 5.2 ft. Coastal abutment scour was estimated to be 15.5 ft for tidal conditions representing a 2% annual chance coastal storm event plus 4 ft of sea level rise. These scour estimates could be refined with more detailed coastal modeling. The estimated wave heights used in this analysis should be checked against results from the ME-CFRM when they become available.
- The riprap along the bridge abutments should be sized with a D50 of 1.8 ft.

8. Limitations

This report presents the preliminary hydrologic and hydraulic analysis for the proposed Mill Cove New Bridge replacement. The results are based on readily available online information, the proposed bridge design information provided by the design team at the time of this report, field observations and data collected during our preliminary geotechnical investigation, and survey information provided by MaineDOT. This analysis and report may require modification if there are any changes in the nature, design, and/or location of the data or proposed design. We recommend that members of the design team be engaged to review the final plans and specifications to evaluate whether changes in the project affect the validity of the methods, findings, and/or recommendations in this study.

The recommendations in this report are based in part on the data obtained from the preliminary borings. The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. Therefore, it is recommended that a geotechnical engineer be engaged to make site visits during construction to check that the subsurface conditions exposed during construction are in general conformance with the study assumptions.

The professional services for this project have been performed in accordance with generally accepted engineering practices; no warranty, express or implied, is made. Actual conditions are expected to vary from the flow scenarios presented in this report.

Reuse of this report for any purposes, in part or in whole, is as the sole risk of the user.

9. References

Chow, V.T. (1959). Open Channel Hydraulics. New York: McGraw-Hill Book Company, Inc.

Federal Emergency Management Agency (FEMA) (2017). Flood Insurance Study. Washington County, Maine. U. S. Department of Homeland Security, July.

Maine Department of Transportation (MaineDOT) (2023). "WIN 26093 Mill Cove New Bridge (#6205) Hydro Report," August.

Maine Department of Transportation (MaineDOT) (2003). Bridge Design Guide, August.

ME State Highway Commission (1967). Robbinston. Washington County. Maine Federal Highway Aid Primary Project No. F-044-2(13) and Project No. LSF-044-2(I).

MEGIS (2006). Maine Land Cover Data. MELCD). May 2006. Downloaded on April 5, 2024 from: http://megis.maine.gov/catalog/.

Natural Resources Conservation Service (NRCS), (2010). "Part 630 Hydrology National Engineering Handbook (NEH), Chapter 15, Time of Concentration," May.

NOAA (2024). Station 8410140. Eastport, ME. Water level data accessed April 2024 from https://tidesandcurrents.noaa.gov/stationhome.html?id=8410140.

NOAA (2023). VDATUM. Online Vertical Datum Transformation. Version 4.6.1, September. Accessed May 2024 from: https://vdatum.noaa.gov/vdatumweb/.

NOAA (2010). Station 8410834. Pettegrove Point, Dochet Island, ME. Water level datums accessed May 2024 from https://tidesandcurrents.noaa.gov/datums.html?id=8410834.

OCM Partners (2024). 2017 Lidar: Eastern Maine. Downloaded March 14 2024 from https://coast.noaa.gov/dataviewer/#/.

SMS (2024). Surface-Water Modeling System Version 13.3.10. March.

United States Army Corps of Engineers (USACE) (2002). Coastal Engineering Manual. Engineer Manual 1110-2-1100, USACE, Washington, D.C.

U.S. Department of Transportation. Federal Highway Administration (FHWA) (2020). "Highways in the Coastal Environment, 3rd edition. HEC-25." Publication No. FHWA-HIF-19-059, January.

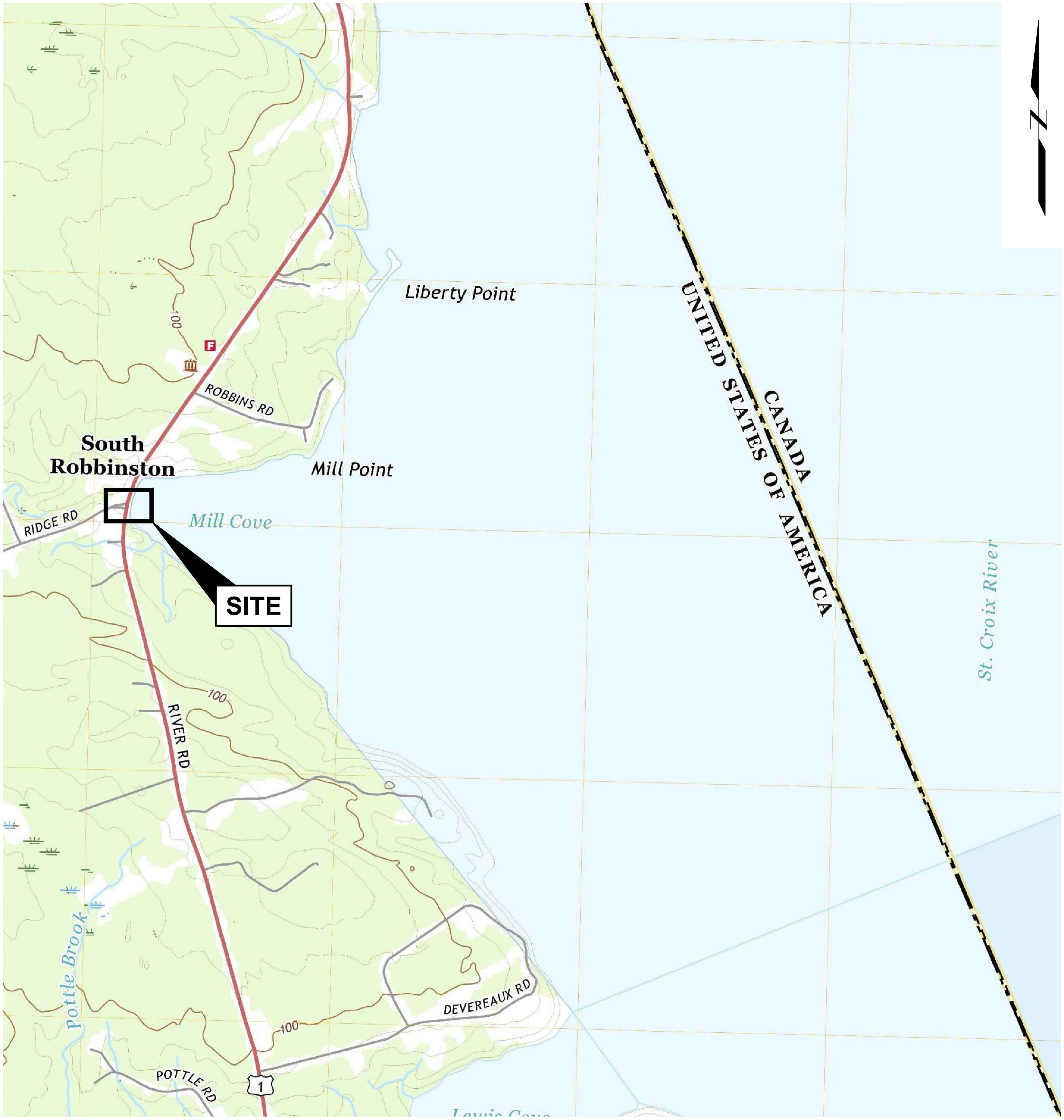
U.S. Department of Transportation. Federal Highway Administration (FHWA) (2012). "Evaluating Scour at Bridges, 5th edition. HEC-18." Publication No. FHWA-HIF-12-003, April.

U.S. Department of Transportation. Federal Highway Administration (FHWA) (2009). "Bridge Scour and Stream Instability Countermeasures: Experience Selection, and Design Guidance-Third Edition. Volume II. HEC-23" Publication No. FHWA-NHI-09-112, September.

United States Geological Survey (USGS) (2015). Surface-Water Field Techniques, Verified Roughness Characteristics of Natural Channels. Accessed variable dates from http://wwwrcamnl.wr.usgs.gov/sws/fieldmethods/Indirects/nvalues/.

Figures

Figure 1. Site Location Map



0 2000 4000

SCALE, FEET

SOURCE:

USGS TOPOGRAPHIC QUADRANGLE, 7.5 MINUTE SERIES: ROBBINSTON QUADRANGLE, MAINE-WASHINGTON COUNTY, 2021

NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88)

20-FOOT CONTOUR INTERVAL

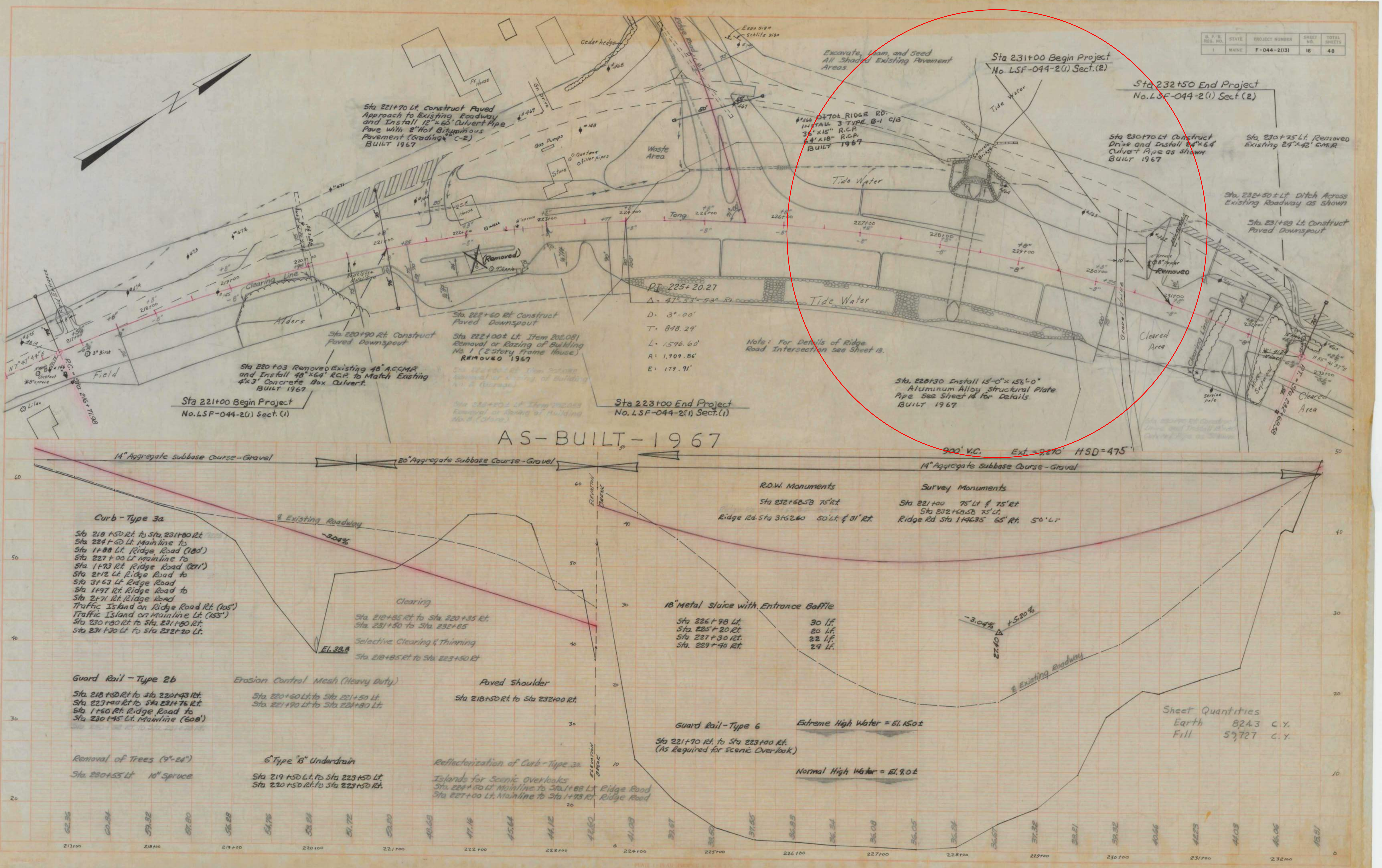
QUADRANGLE LOCATION

Mill Cove New Bridge (#6205) over Mill Cove WIN 026630.06 Robbinston, Maine	GEI Consultants	SITE LOCATION MAP
Thornton Tomasetti Portland, Maine	Project 2400963	August 2024 Fig. 1

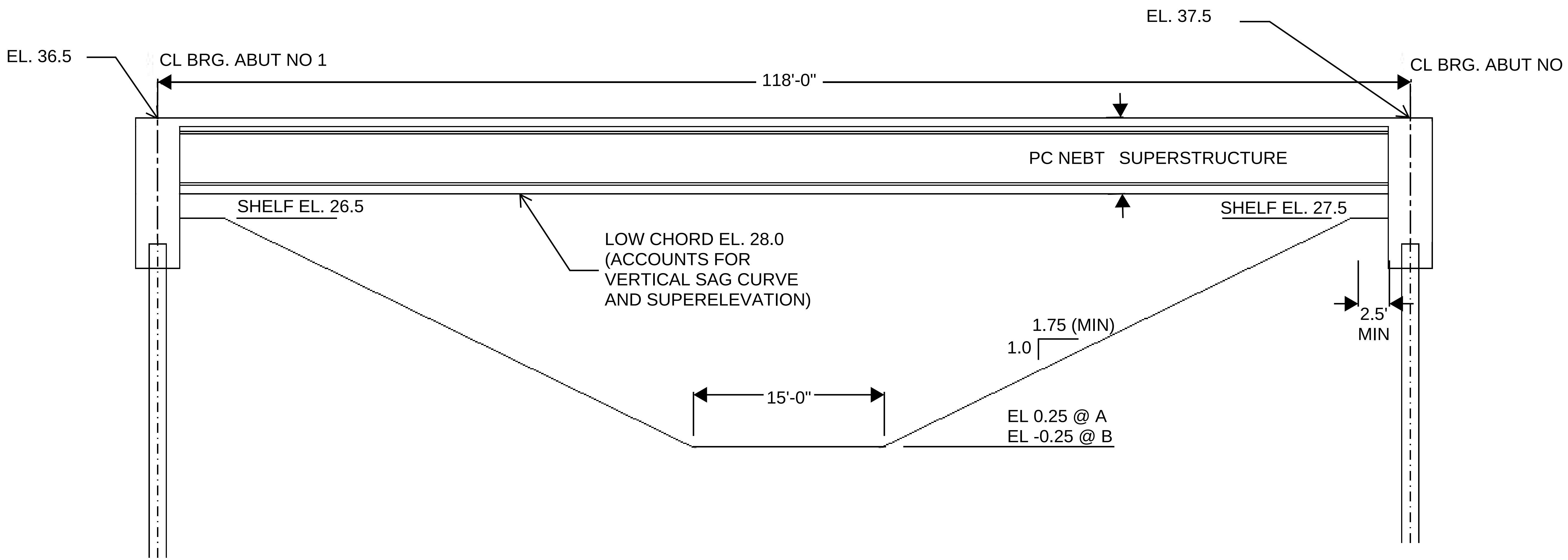
CARVAJAL, SEBASTIAN B:\Working\THORNTON TOMASETTI\2400963 MaineDOT Downeast Bridges\00_CAD\Figures\Sheet 1 Site Location\Site Location Map - Mill Cove New Bridge.dwg - 4/30/2024

Appendix A 1967 Bridge Design Plans

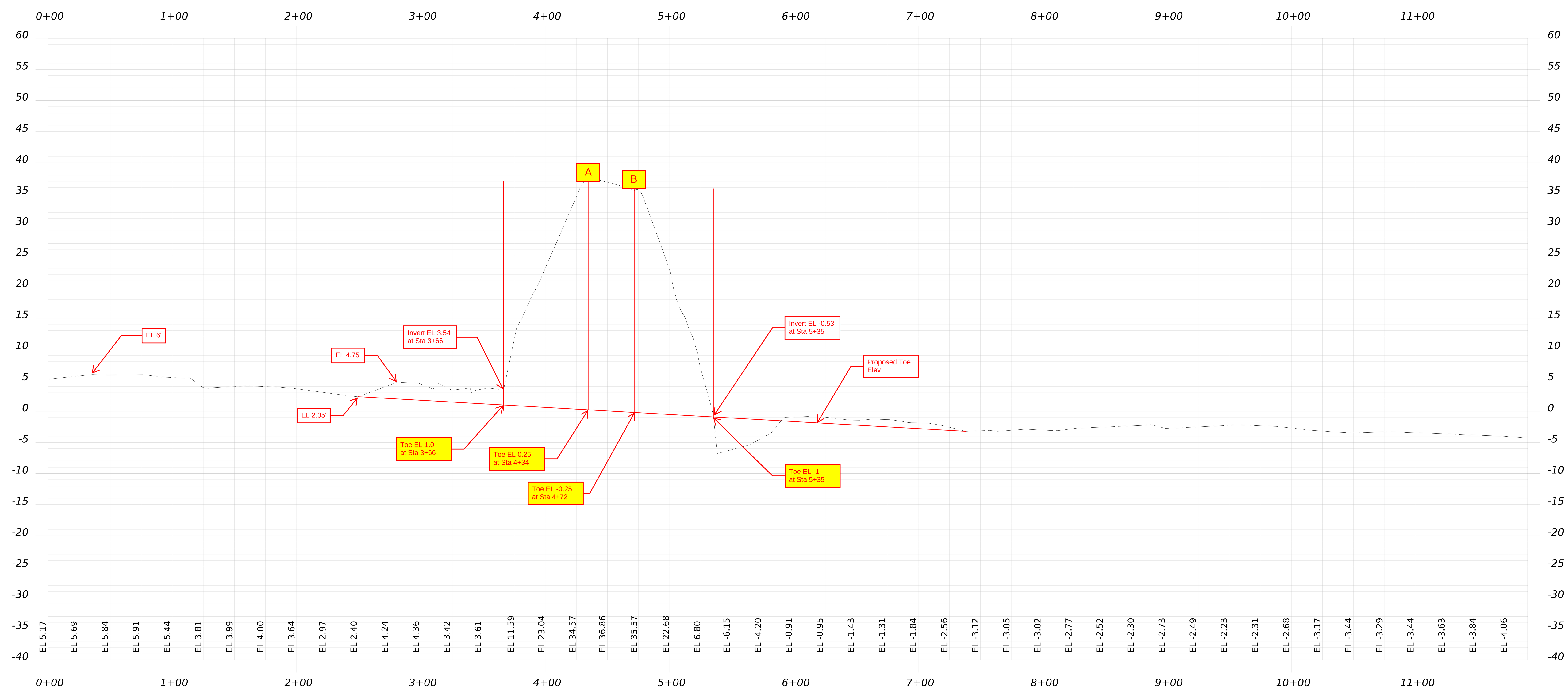
B. P. R. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	F-044-2(13)	16	48



Appendix B Preliminary Design Single Span Bridge



WIN 026630.06
ROBBINSTON - MILL COVE
BRIDGE ELEVATION
05-26-2024



EL 6'
EL 4.75'
Invert EL 3.54 at Sta 3+66
EL 2.35'
Toe EL 1.0 at Sta 3+66
Toe EL 0.25 at Sta 4+34
Toe EL -0.25 at Sta 4+72
A
B
Invert EL -0.53 at Sta 5+35
Proposed Toe Elev
Toe EL -1 at Sta 5+35
EL 5.17
EL 5.69
EL 5.84
EL 5.91
EL 5.44
EL 3.81
EL 3.99
EL 4.00
EL 3.64
EL 2.97
EL 2.40
EL 4.24
EL 4.36
EL 3.42
EL 3.61
EL 11.59
EL 23.04
EL 34.57
EL 36.86
EL 35.57
EL 22.68
EL 6.80
EL -6.15
EL -4.20
EL -0.91
EL -0.95
EL -1.43
EL -1.31
EL -1.84
EL -2.56
EL -3.12
EL -3.05
EL -3.02
EL -2.77
EL -2.52
EL -2.30
EL -2.73
EL -2.49
EL -2.23
EL -2.31
EL -2.68
EL -3.17
EL -3.44
EL -3.29
EL -3.44
EL -3.63
EL -3.84
EL -4.06
0+00
1+00
2+00
3+00
4+00
5+00
6+00
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-40

Appendix C MaineDOT Hydrology

WIN:	26093.00		**Project Name:**	
Town:	Robbinston		**Stream Name:**	Western Stream
Route No.	US1		**Bridge Name:**	Mill Cove New
Asset ID:	6205		**Analysis by:**	csh
Lat:	45.05814	**Long:** -67.11840	**Date:**	8/15/2023

Peak Flow Calculations by USGS Regression Equations (Lombard/Hodgkins, 2021; Hodgkins, 1999 & Lombard/Hodgkins, 2015)

Enter data in blue cells only!

ver. 2021 Jan 01

	km^2	mi^2	ac	*Enter data in [mi^2]*
A	16.11	6.22	3980.8	Watershed Area *DRNAREA*
W	1.07	0.4	265.5	Wetlands area (by NWI)
P_c	645504	4992436		watershed centroid (E, N; UTM 19N; meters)
County	Washington			*choose county from drop-down menu*

Worksheet prepared by:
Charles S. Hebson, PE
Environmental Office
Maine Dept. Transportation
Augusta, ME 04333-0016
207-557-1052
Charles.Hebson@maine.gov

Watershed Characteristics from StreamStats

STORAGE	6.27	
STORNWI	6.67	NWI Wetlands %
SANDGRAVF	0.00	sand & gravel aquifer as decimal fraction of watershed A
ELEV	242.7	mean basin elevation (ft)
BSLDEM10M	7.33	mean basin slope (%)
COASTDIST	42.80	distance from the coast (mi)
ELEVMAX	526.4	maximum basin elevation (ft)
LC06WATER	2.55	percent of drainage basin land cover as open water
PRECIP	43.7	mean annual precipitation
STATSGOA	6.65	mean basin percentage of hydrological soil group A

References:

Hodgkins, G.A., 1999.
Estimating the magnitude of peak flows for streams in Maine
for Selected Recurrence Intervals
WRIR 99-4008, USGS Augusta, ME

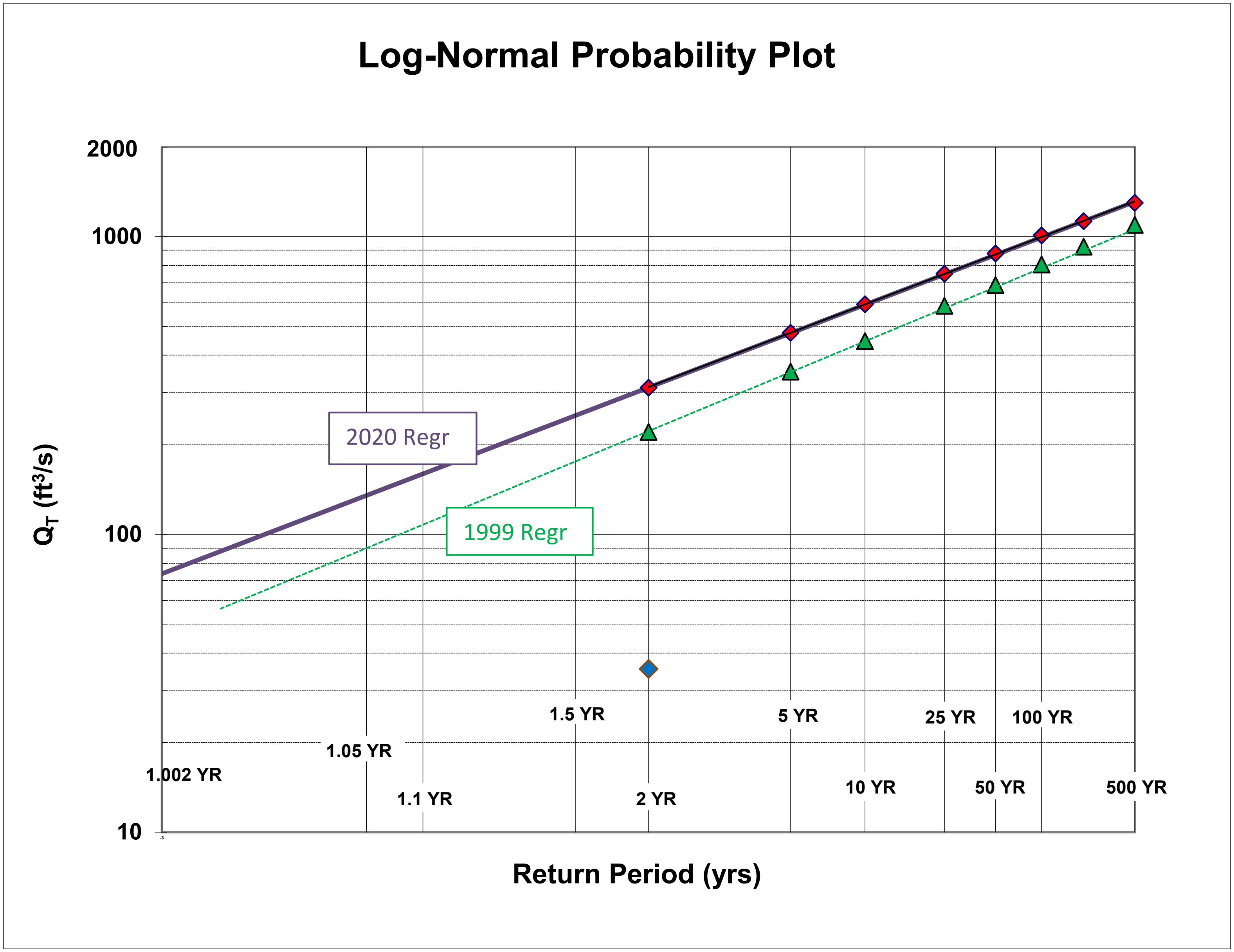
Lombard, P.J. & G.A. Hodgkins, 2015.
Peak flow regression equations for small, ungaged streams:
in Maine: Comparing Map-Based to Field-Based Variables
SIR 2015-4059, USGS, Augusta, ME

Lombard, P.J. & G.A. Hodgkins, 2020.
Estimating Flood Magnitude and Frequency on Gaged and
Ungaged Streams in Maine
SIR 2020-5092, USGS, Augusta, ME.

Ret Pd T (yr)	I24	Q_T (ft^3/s) *1999 / 2015*	Q_T (ft^3/s) *2020*	Q_T (ft^3/s) ***Design***
1.1			160	***160***
2	3.18	221	311	***310***
5	3.89	352	475	***475***
10	4.48	446	593	***595***
25	5.29	586	751	***750***
50	5.91	688	879	***880***
100	6.55	806	1009	***1010***
200	7.23	926	1128	***1130***
500	8.22	1094	1299	***1300***

Calculated Bankfull Width: 23.2 **ft**

Instructions:
Enter values in blue cells only, watershed data from StreamStats
Copy I24 values from Stream Stats
Use results under "Design"
Check against gage data and FEMA studies if available
Questions? Check with ENV / Hydrology Section



Log-Normal Probability Plot
2000
1000
100
10
Q_T (ft³/s)
2020 Regr
1999 Regr
1.002 YR
1.05 YR
1.1 YR
1.5 YR
2 YR
5 YR
10 YR
25 YR
50 YR
100 YR
500 YR
Return Period (yrs)

WIN:	26093.00	**Project Name:**	0
Town:	Robbinston	**Stream Name:**	Western Stream
Route No.	US1	**Bridge Name:**	Mill Cove New
Asset ID:	6205	**Analysis by:**	csh
Lat:	45.05814 **Long:** -67.11840	**Date:**	8/15/2023

DO NOT ENTER ANY DATA ON THIS PAGE; EVERYTHING IS CALCULATED

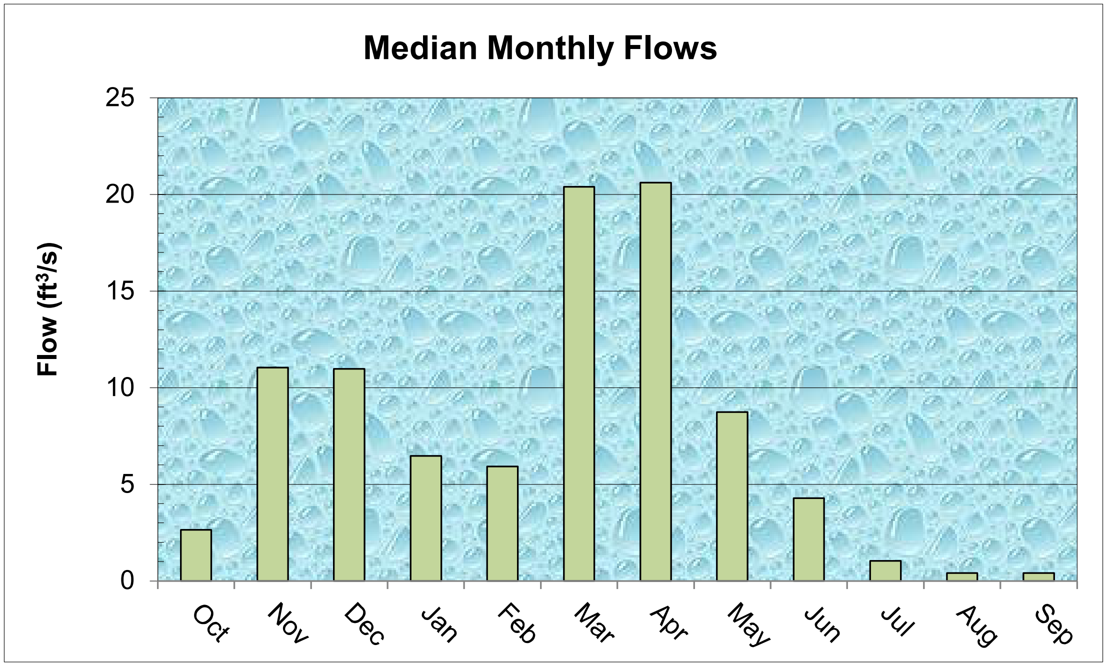
MAINE MONTHLY MEDIAN FLOWS and HYDRAULIC GEOMETRY BY USGS REGRESSION EQUATIONS (2004, 2013, 2015)

	Value	Variable	Explanation
	6.22	*A*	Area (mi^2)
645504	4992436	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	42.13	*DIST*	Distance from Coastal reference line (mi)
	43.7	*pptA*	Mean Annual Precipitation (inches)
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median} (ft3/s)	(m^3/s)
Jan	**6.47**	0.1833
Feb	**5.91**	0.1676
Mar	**20.39**	0.5779
Apr	**20.61**	0.5841
May	**8.74**	0.2478
Jun	**4.28**	0.1212
Jul	**1.04**	0.0295
Aug	**0.40**	0.0113
Sep	**0.41**	0.0116
Oct	**2.64**	0.0749
Nov	**11.04**	0.3128
Dec	**10.97**	0.3109

Q_{bf}	**35.4**	
ann avg	**13.2**	
ann med	**5.5**	
$Q_{1.002}$	**73.8**	
$Q_{1.01}$	**97.1**	
$Q_{1.05}$	**135.2**	
$Q_{1.1}$	**159.7**	
Q_{bf}	**87.6**	assume v = 4ft/s

W_{bf}	**23.2**	estimated bankfull width (ft)
d_{bf}	**1.1**	estimated bankfull depth (ft)
A_{bf}	**21.9**	estimated bankfull flow area (ft^2)

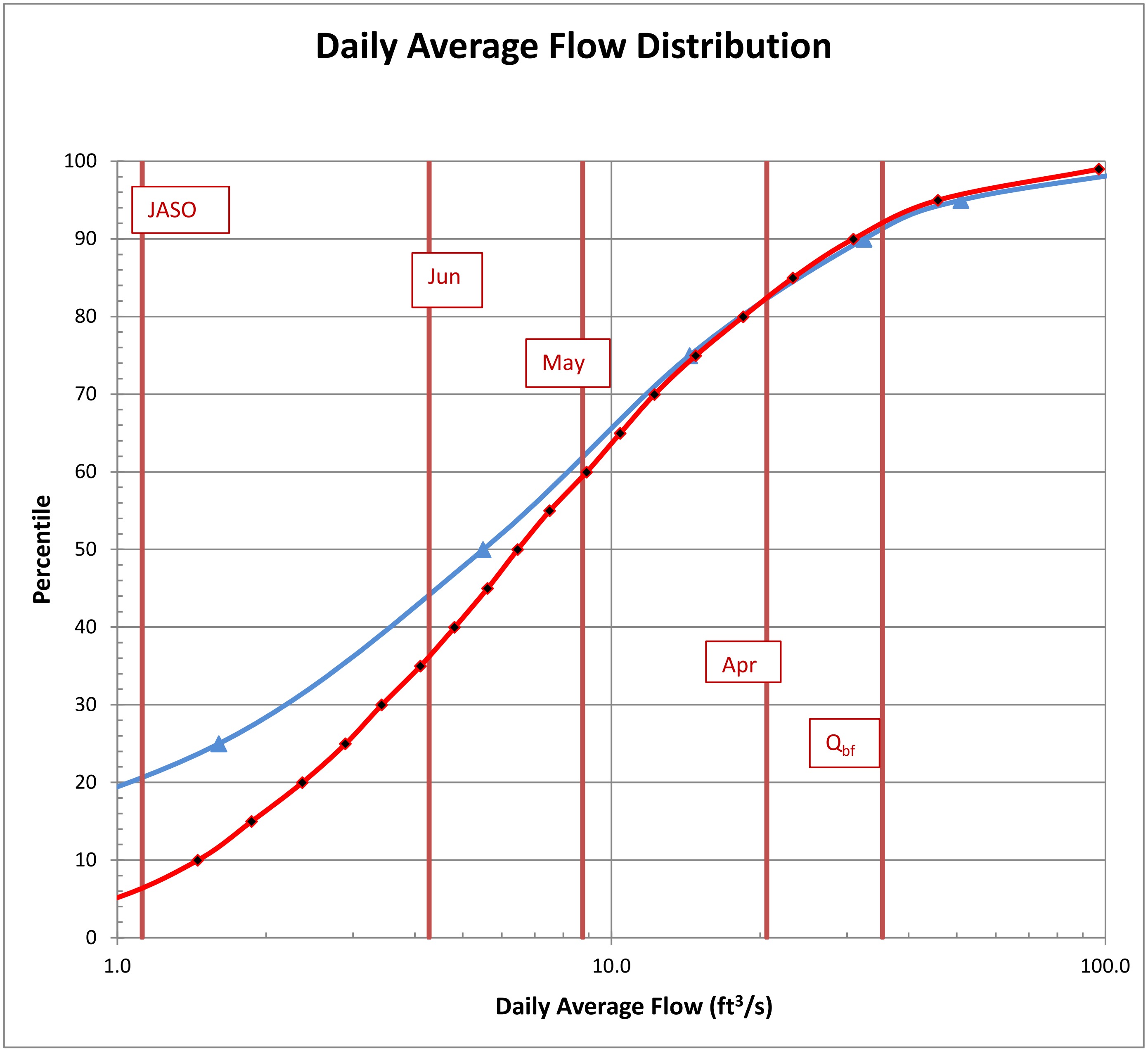


References

Dudley, 2013. FY2013 Progress Report - Phase 1 …, USFWS QRP Project

Dudley, 2004. Estimating Monthly Streamflows … , SIR 2004-5026

Dudley, 2015. Regression Equations for Monthly & Annual Mean..., USGS SIR 2015-5151



Daily Avg Flow Dist

A_{ws} = (mi^2) 6.2

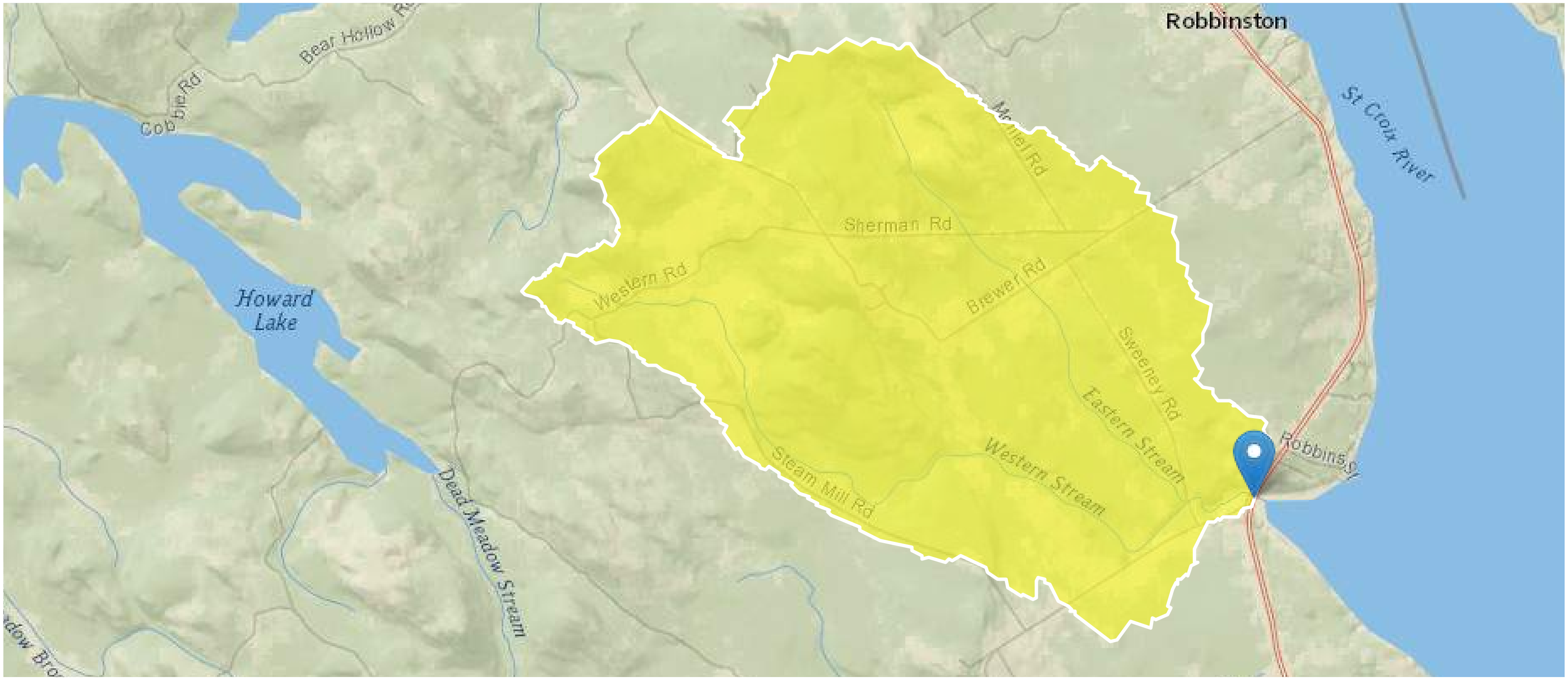
Q (ft^3/s)

Pctl	Median	84th pctl
1.00E-06	0.00	0.00
1	0.53	0.94
5	0.98	1.58
10	1.45	2.19
15	1.87	2.73
20	2.37	3.31
25	2.90	3.88
30	3.43	4.42
35	4.11	5.05
40	4.81	5.81
45	5.61	6.57
50	6.46	7.76
55	7.50	9.03
60	8.91	10.60
65	10.43	12.35
70	12.23	14.40
75	14.82	17.32
80	18.49	20.68
85	23.33	26.50
90	30.90	35.58
95	45.86	55.34
99	96.97	127.67

Q_{bf}	35.4
$Q_{1.002}$	73.8
$Q_{1.1}$	159.7
Q_2	311.3

Robbinston 26093 Mill Cove New Bridge #6205

Region ID: ME
Workspace ID: ME20230815123920413000
Clicked Point (Latitude, Longitude): 45.05814, -67.11840
Time: 2023-08-15 08:39:51 -0400



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
BSLDEM10M	Mean basin slope computed from 10 m DEM	7.33	percent
CENTROIDX	Basin centroid horizontal (x) location in state plane coordinates	645504	meters
CENTROIDY	Basin centroid vertical (y) location in state plane units	4992436.3	meters
COASTDIST	Shortest distance from the coastline to the basin centroid	42.8	miles
DRNAREA	Area that drains to a point on a stream	6.22	square miles
ELEV	Mean Basin Elevation	242.7	feet
ELEVMAX	Maximum basin elevation	526.4	feet
I24H100Y	Maximum 24-hour precipitation that occurs on average once in 100 years	6.55	inches
I24H10Y	Maximum 24-hour precipitation that occurs on average once in 10 years	4.48	inches
I24H200Y	Maximum 24-hour precipitation that occurs on average once in 200 years	7.23	inches
I24H25Y	Maximum 24-hour precipitation that occurs on average once in 25 years	5.29	inches
I24H2Y	Maximum 24-hour precipitation that occurs on average once in 2 years - Equivalent to precipitation intensity index	3.18	inches
I24H500Y	Maximum 24-hour precipitation that occurs on average once in 500 years	8.22	inches
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	5.91	inches
I24H5Y	Maximum 24-hour precipitation that occurs on average once in 5 years	3.89	inches
JULAVPRE	Mean July Precipitation	2.98	inches
LC06WATER	Percent of open water, class 11, from NLCD 2006	2.55	percent
LC11DEV	Percentage of developed (urban) land from NLCD 2011 classes 21-24		percent

Parameter Code	Parameter Description	Value	Unit
LC11IMP	Average percentage of impervious area determined from NLCD 2011 impervious dataset		percent
PCTSNDGRV	Percentage of land surface underlain by sand and gravel deposits	0	percent
PRDECFEB90	Basin average mean precipitation for December to February from PRISM 1961-1990	11.5	inches
PRECIP	Mean Annual Precipitation	43.7	inches
SANDGRAVAF	Fraction of land surface underlain by sand and gravel aquifers	0	dimensionless
SANDGRAVAP	Percentage of land surface underlain by sand and gravel aquifers	0	percent
STATSGOA	Percentage of area of Hydrologic Soil Type A from STATSGO	6.65	percent
STORAGE	Percentage of area of storage (lakes ponds reservoirs wetlands)	6.271	percent
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	6.67	percent

General Disclaimers

The delineation point is in an exclusion area. Warning! Coastal/Tidal areas are outside the hydrologic region defined by the study. Accuracy of regression equations is not defined.

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Statewide multiparameter peakflows SIR 2020 5092]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	6.22	square miles	0.26	5680
I24H2Y	24 Hour 2 Year Precipitation	3.18	inches	1.92	4.17
STORAGE	Percent Storage	6.271	percent	0	29.4
I24H5Y	24 Hour 5 Year Precipitation	3.89	inches	2.48	5.38
I24H10Y	24 Hour 10 Year Precipitation	4.48	inches	2.84	6.38
I24H25Y	24 Hour 25 Year Precipitation	5.29	inches	3.3	7.75
I24H50Y	24 Hour 50 Year Precipitation	5.91	inches	3.65	8.79
I24H100Y	24 Hour 100 Year Precipitation	6.55	inches	3.99	9.88
I24H200Y	24 Hour 200 YearPrecipitation	7.23	inches	5.26	11.1
I24H500Y	24 Hour 500 Year Precipitation	8.22	inches	5.95	13.1

Peak-Flow Statistics Flow Report [Statewide multiparameter peakflows SIR 2020 5092]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, ASEp: Average Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	ASEp
50-percent AEP flood	311	ft^3/s	167	579	39.1
20-percent AEP flood	475	ft^3/s	259	871	38.1
10-percent AEP flood	593	ft^3/s	319	1100	38.9
4-percent AEP flood	751	ft^3/s	399	1410	39.9

Statistic	Value	Unit	PII	PIu	ASEp
2-percent AEP flood	878	ft^3/s	459	1680	39.7
1-percent AEP flood	1010	ft^3/s	532	1920	40.7
0.5-percent AEP flood	1130	ft^3/s	576	2220	42.8
0.2-percent AEP flood	1300	ft^3/s	653	2590	43.8

Peak-Flow Statistics Citations

Lombard, P.J., and Hodgkins, G.A.,2020, Estimating flood magnitude and frequency on gaged and ungaged streams in Maine: U.S. Geological Survey Scientific Investigations Report 2020–5092, 56 p. (https://doi.org/10.3133/sir20205092)

Annual Flow Statistics

Annual Flow Statistics Parameters [Statewide Annual SIR 2015 5151]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	6.22	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	242.7	feet	239	2120

Annual Flow Statistics Disclaimers [Statewide Annual SIR 2015 5151]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Annual Flow Statistics Flow Report [Statewide Annual SIR 2015 5151]

Statistic	Value	Unit
Mean Annual Flow	13.2	ft^3/s

Annual Flow Statistics Citations

Dudley, R.W.,2015, Regression equations for monthly and annual mean and selected percentile streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2015–5151, 35 p. (http://dx.doi.org/10.3133/sir20155151)

Flow Percentile Statistics

Flow Percentile Statistics Parameters [Statewide Annual SIR 2015 5151]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	6.22	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	242.7	feet	239	2120

Flow Percentile Statistics Disclaimers [Statewide Annual SIR 2015 5151]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Flow Percentile Statistics Flow Report [Statewide Annual SIR 2015 5151]

Statistic	Value	Unit
1st Percentile Flow	0.0194	ft^3/s
5th Percentile Flow	0.117	ft^3/s
10th Percentile Flow	0.321	ft^3/s
25th Percentile Flow	1.6	ft^3/s
50th Percentile Flow Median	5.5	ft^3/s

Statistic	Value	Unit
75th Percentile Flow	14.4	ft^3/s
90th Percentile Flow	32.5	ft^3/s
95th Percentile Flow	51	ft^3/s
99th Percentile Flow	124	ft^3/s

Flow Percentile Statistics Citations

Dudley, R.W.,2015, Regression equations for monthly and annual mean and selected percentile streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2015–5151, 35 p. (http://dx.doi.org/10.3133/sir20155151)

Application Version: 4.16.1
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.1

Appendix D Proposed Structure Maximum Water Depths

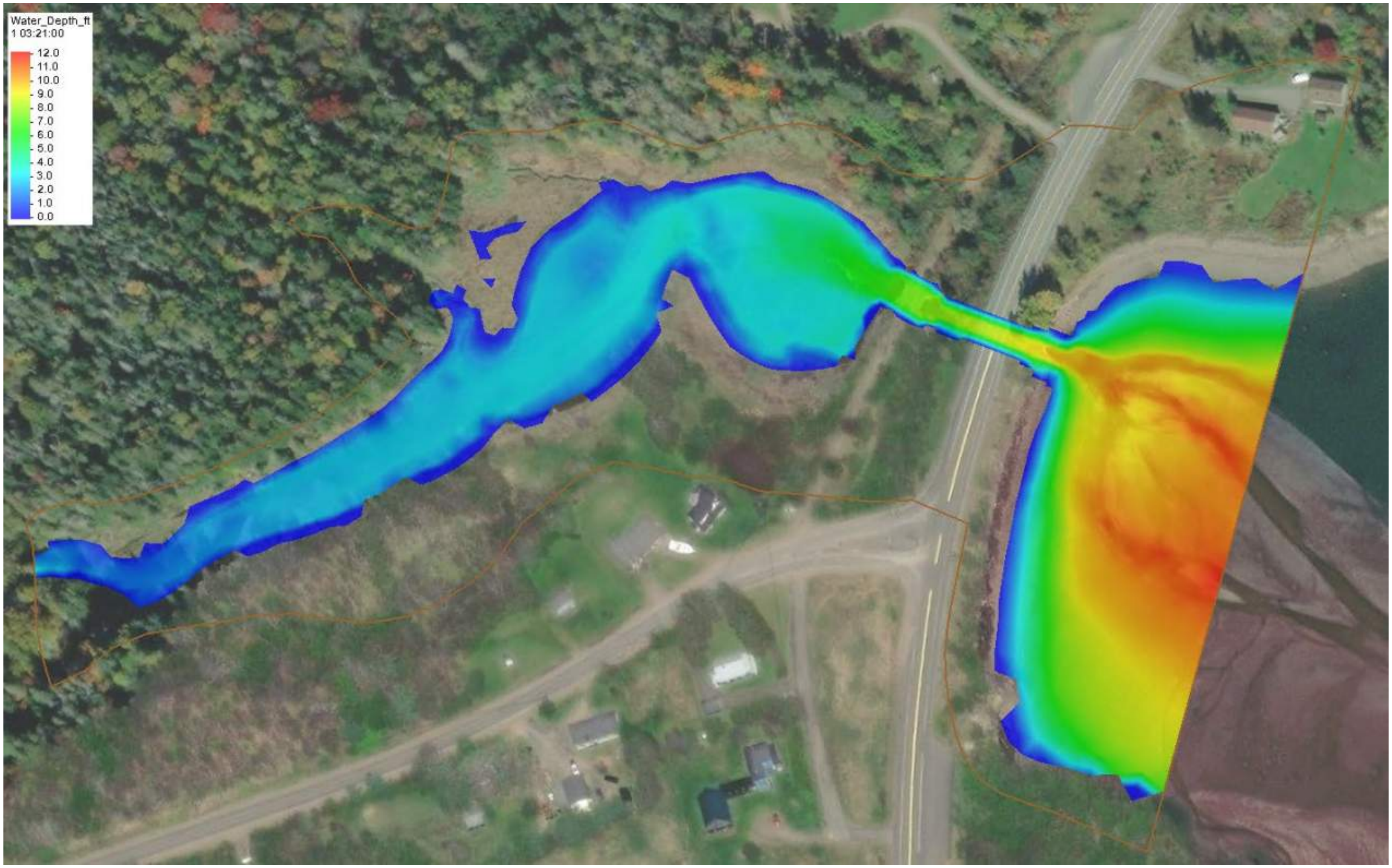


Figure D-1: $Q_{1.1}$, Average Tides – Maximum Water Depths, Proposed Single Span Bridge Structure

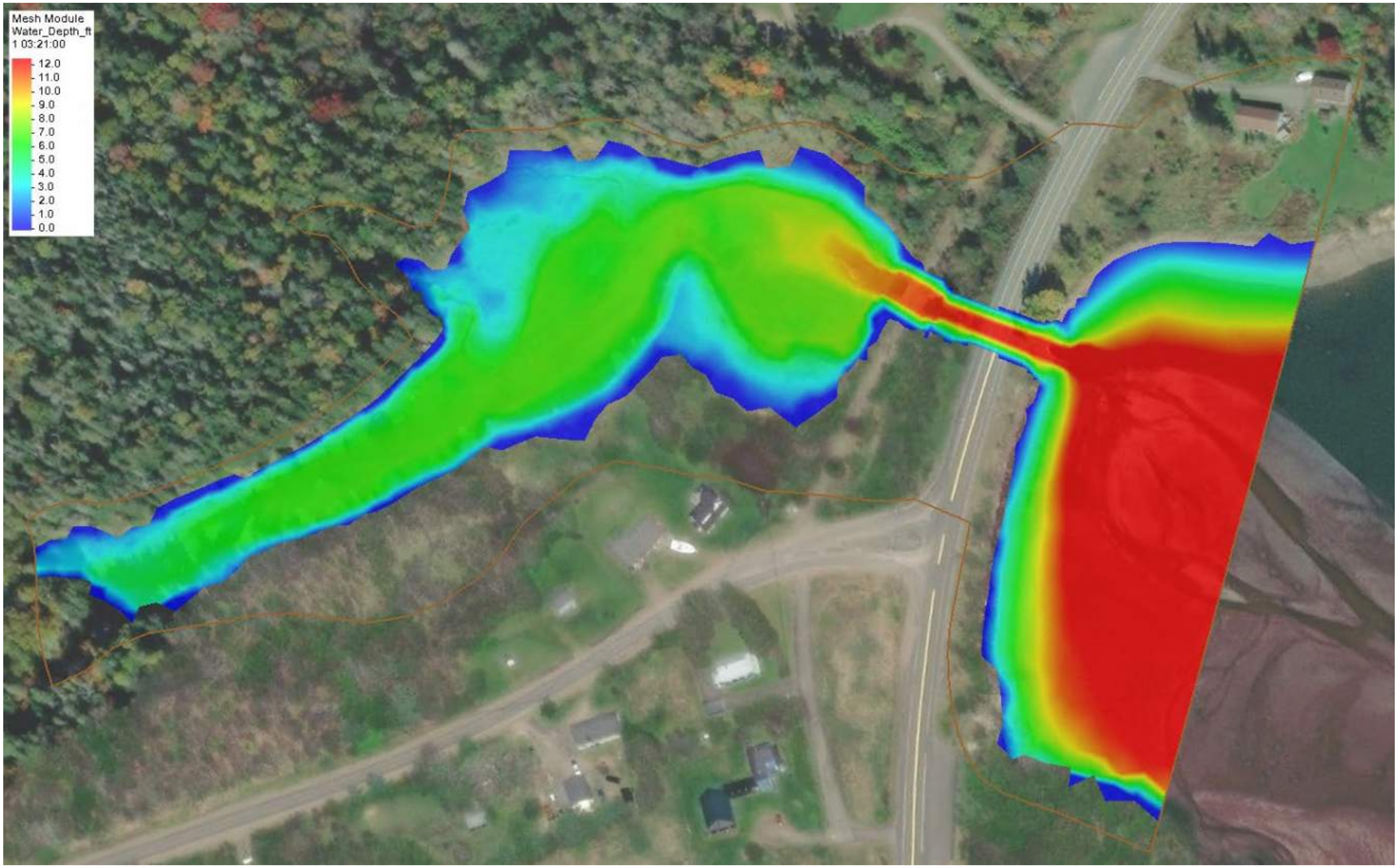


Figure D-2: $Q_{1.1}$, **Average Tides plus 4 ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure**

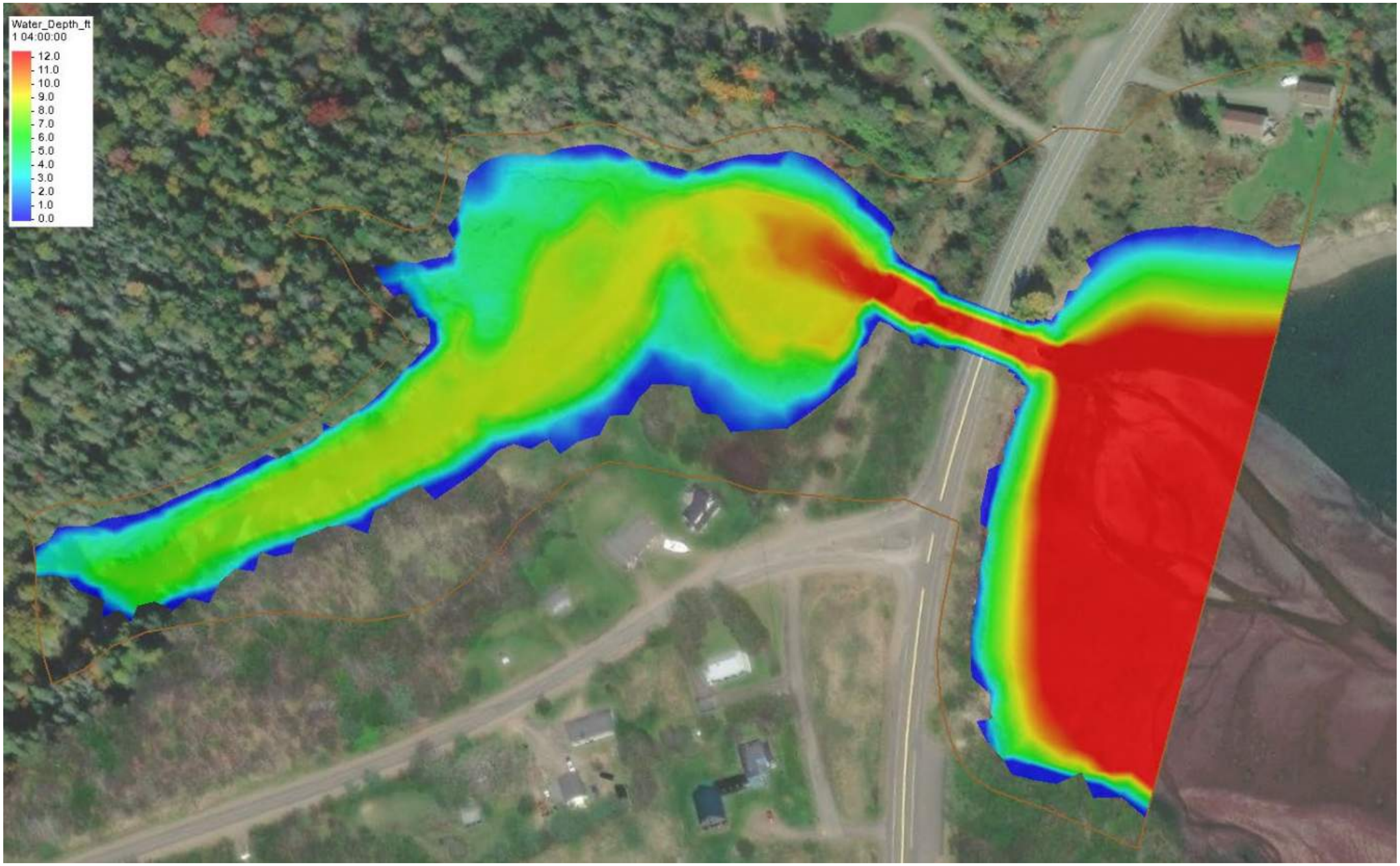


Figure D-3: $Q_{1.1}$, 2% annual chance coastal storm event – Maximum Water Depths, Proposed Single Span Bridge Structure

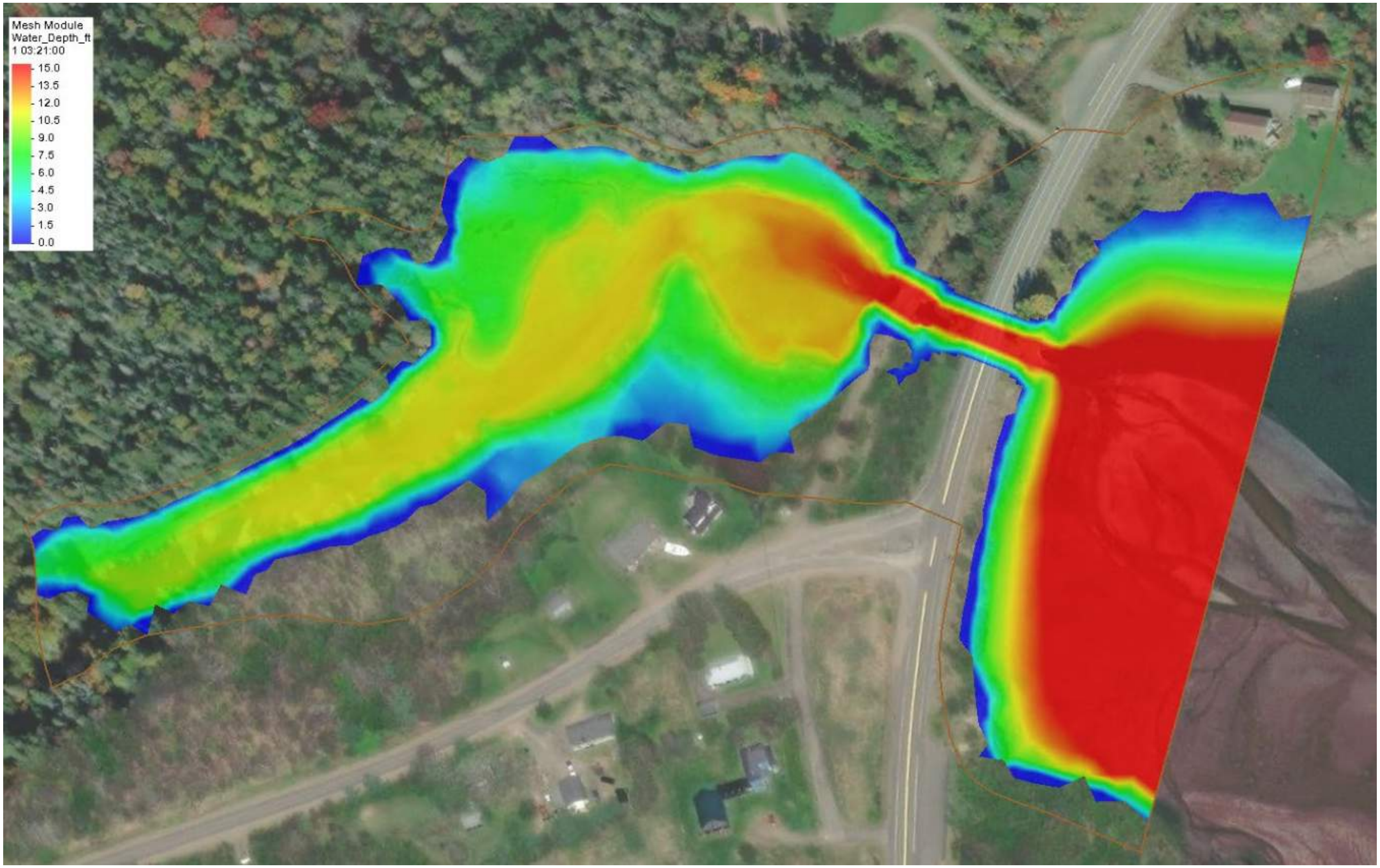


Figure D-4: **$Q_{1.1}$, 2% annual chance coastal storm event plus 4 ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure**

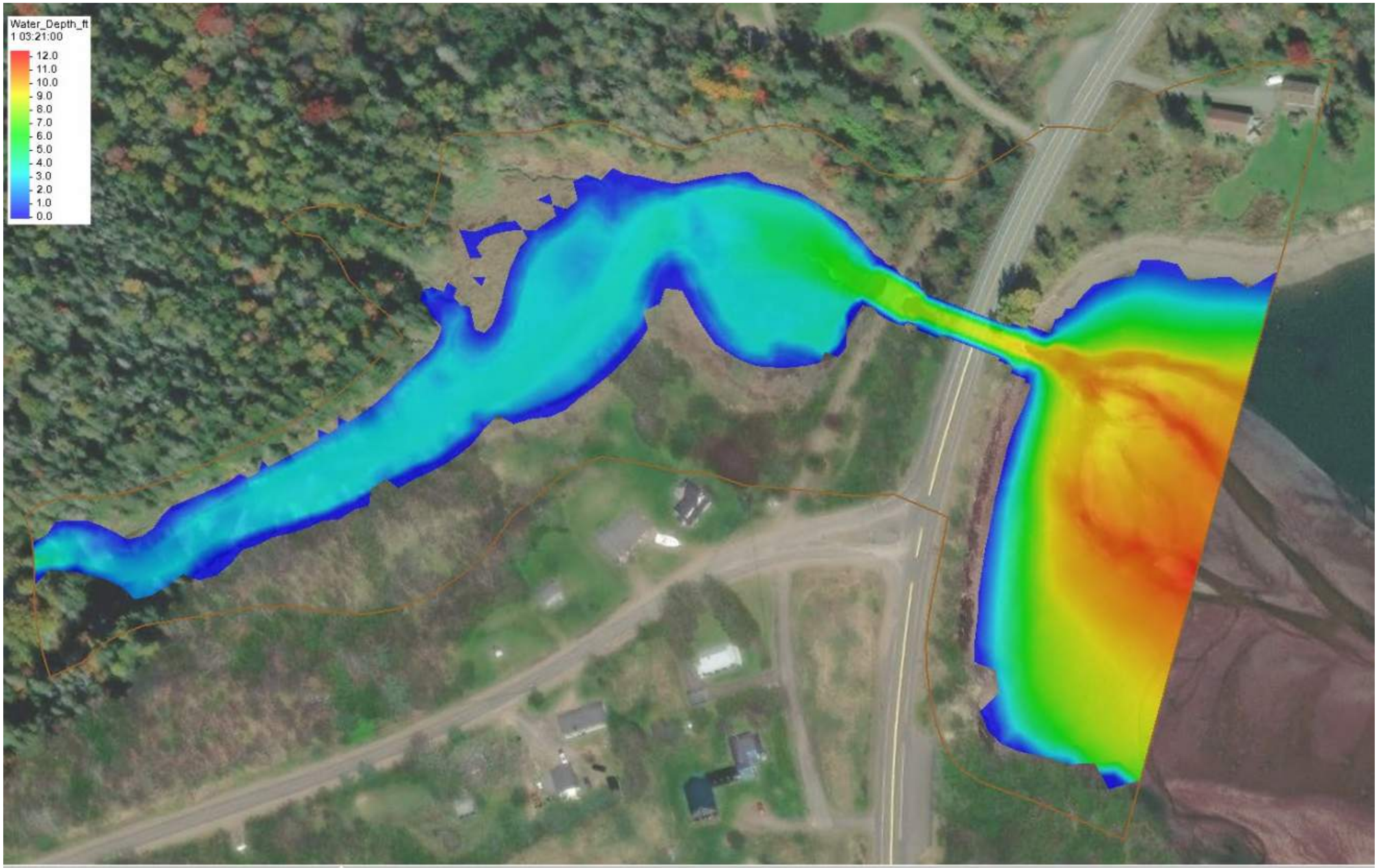


Figure D-5: Q_{10}, Average Tides – Maximum Water Depths, Proposed Single Span Bridge Structure

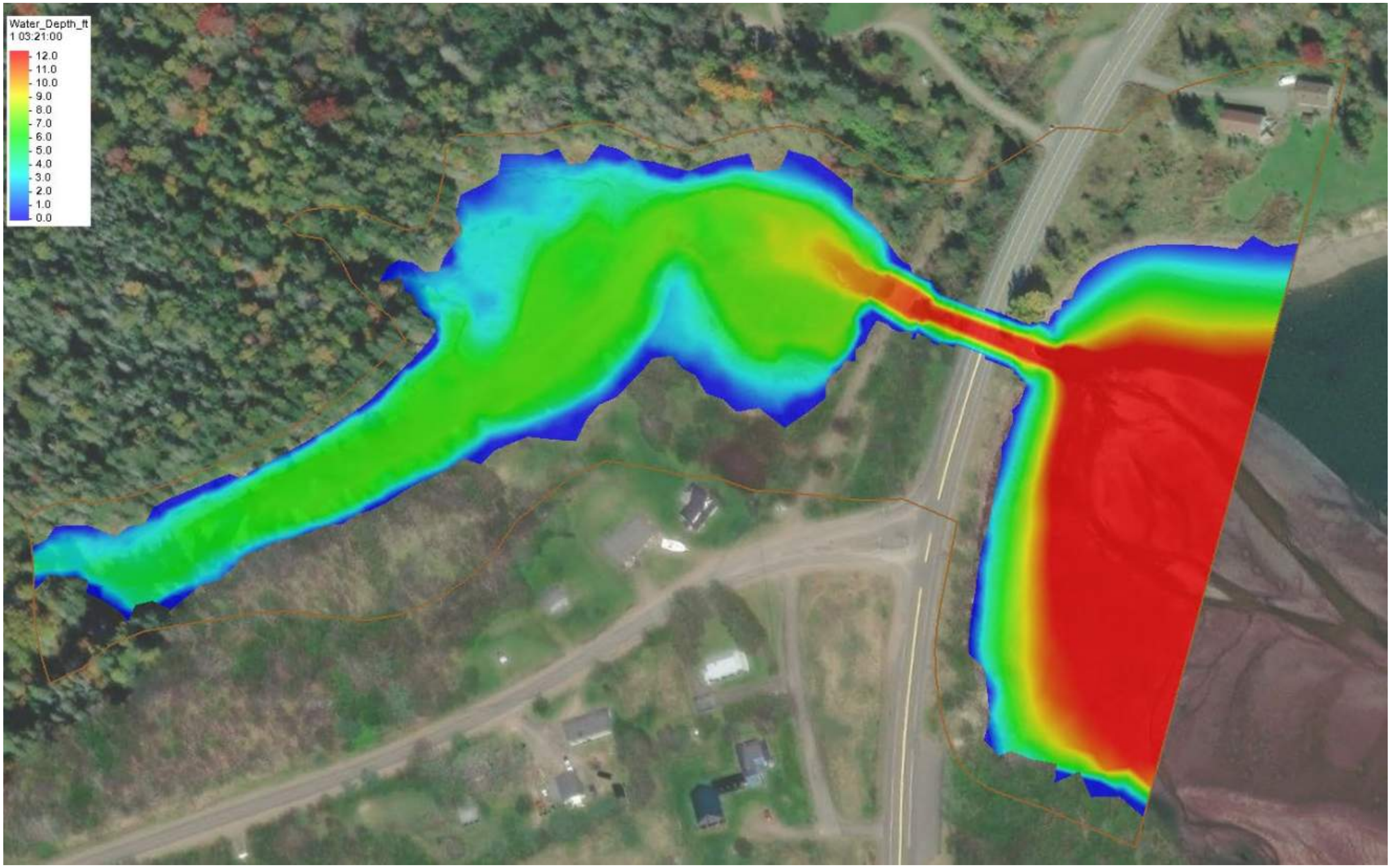


Figure D-6: **Q_{10}, Average Tides plus 4ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure**

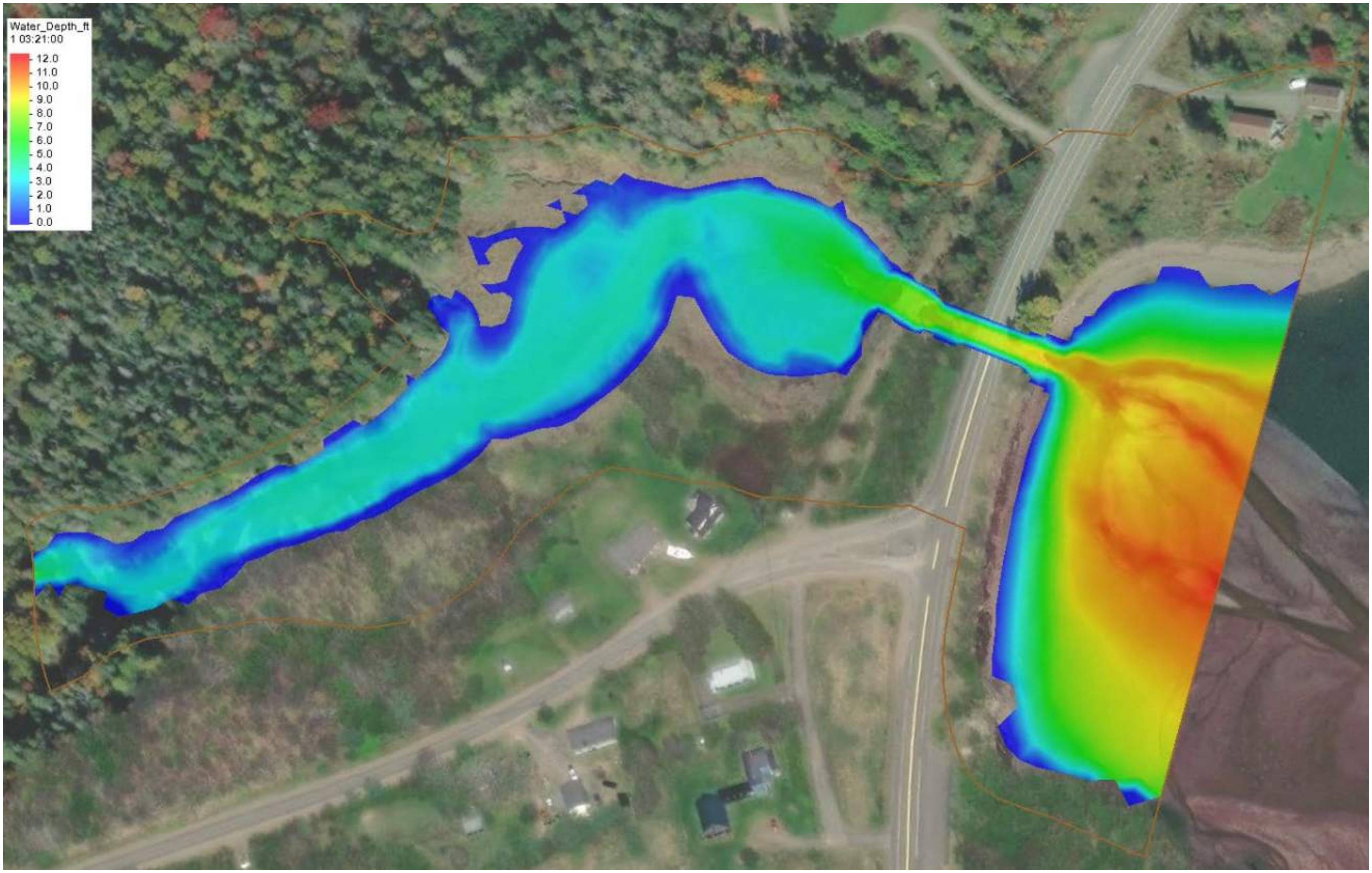


Figure D-7: **Q_{50}, Average Tides – Maximum Water Depths, Proposed Single Span Bridge Structure**

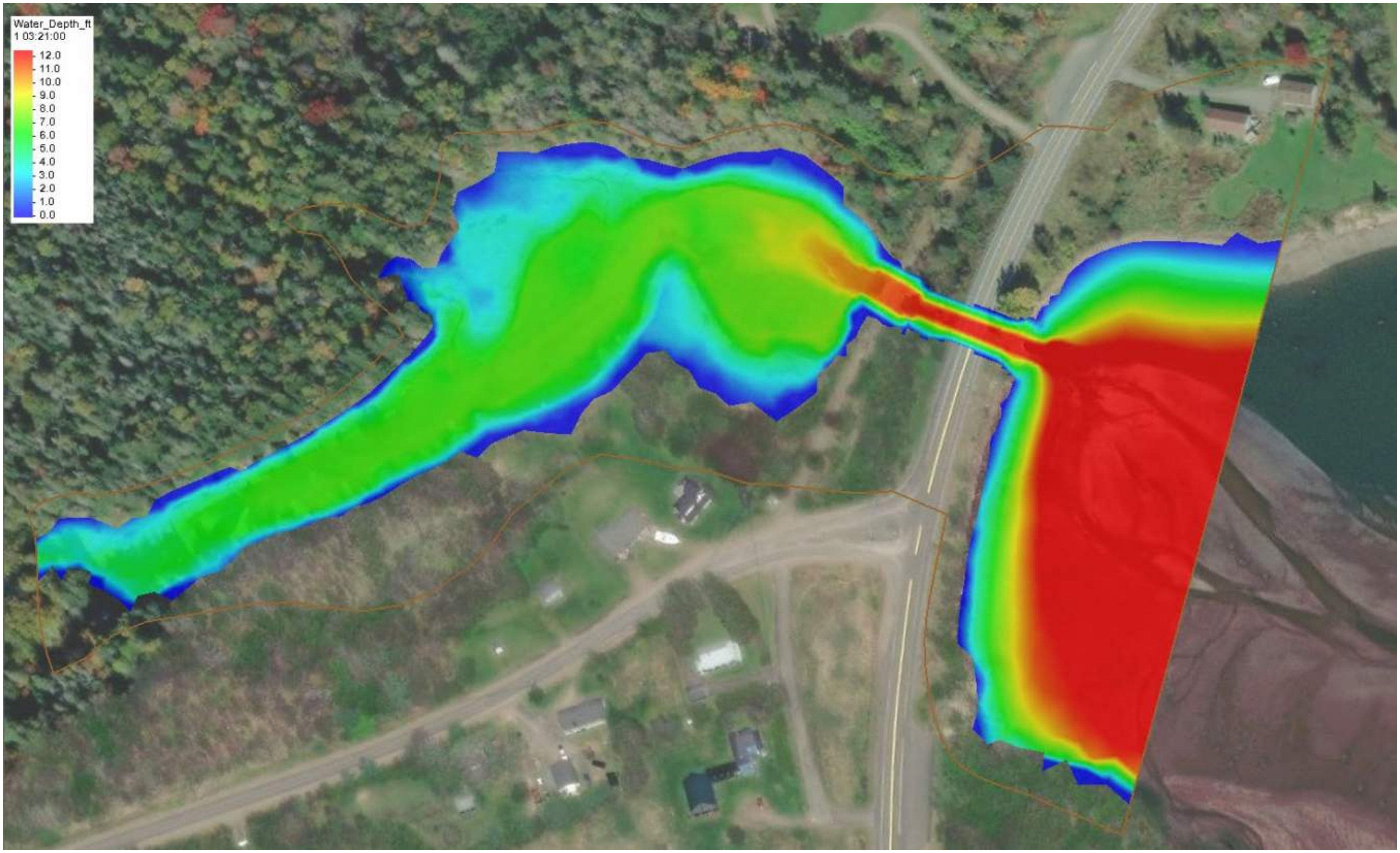


Figure D-8: **Q_{50}, Average Tides plus 4ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure**

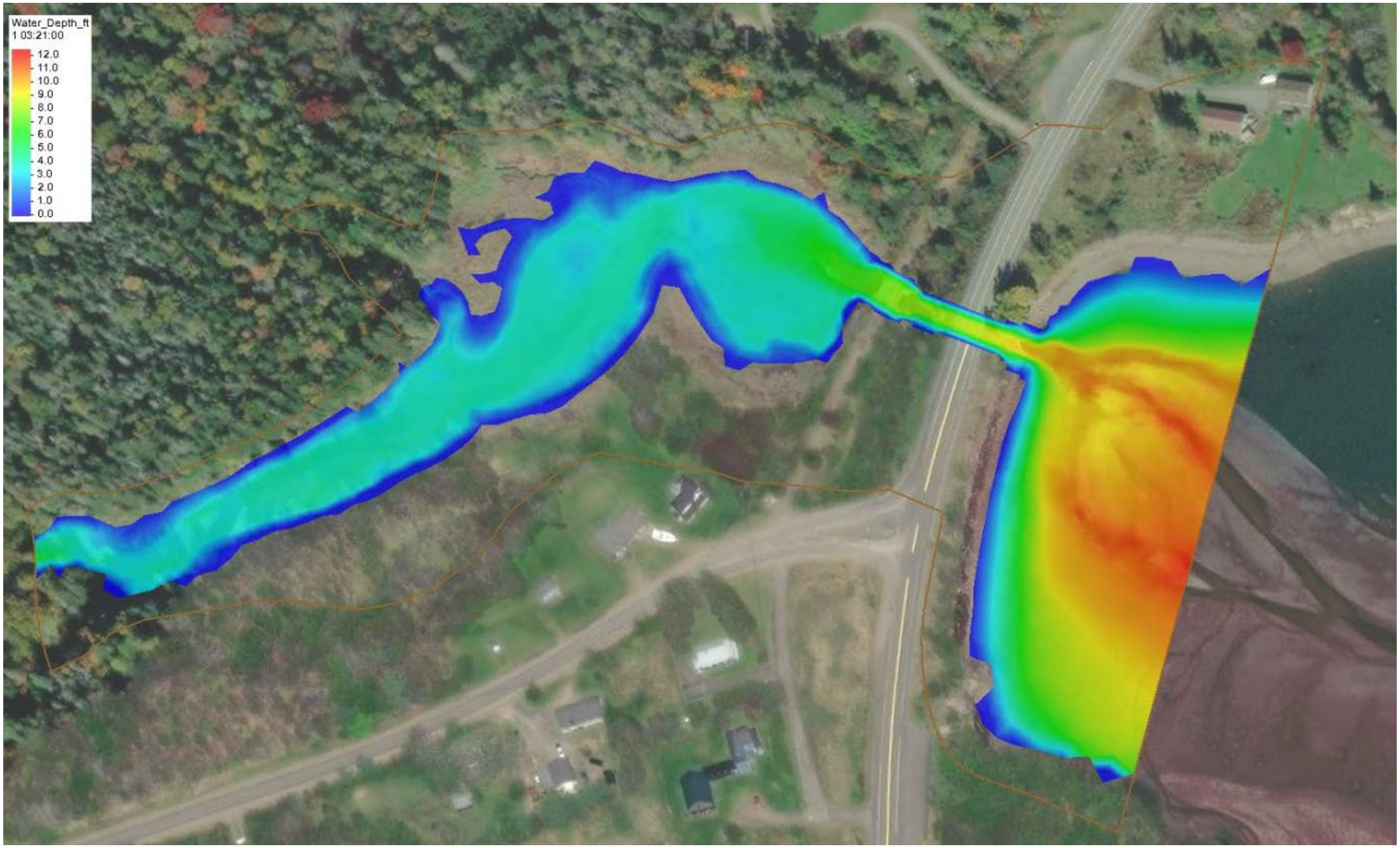


Figure D-9: **Q_{100}, Average Tides – Maximum Water Depths, Proposed Single Span Bridge Structure**

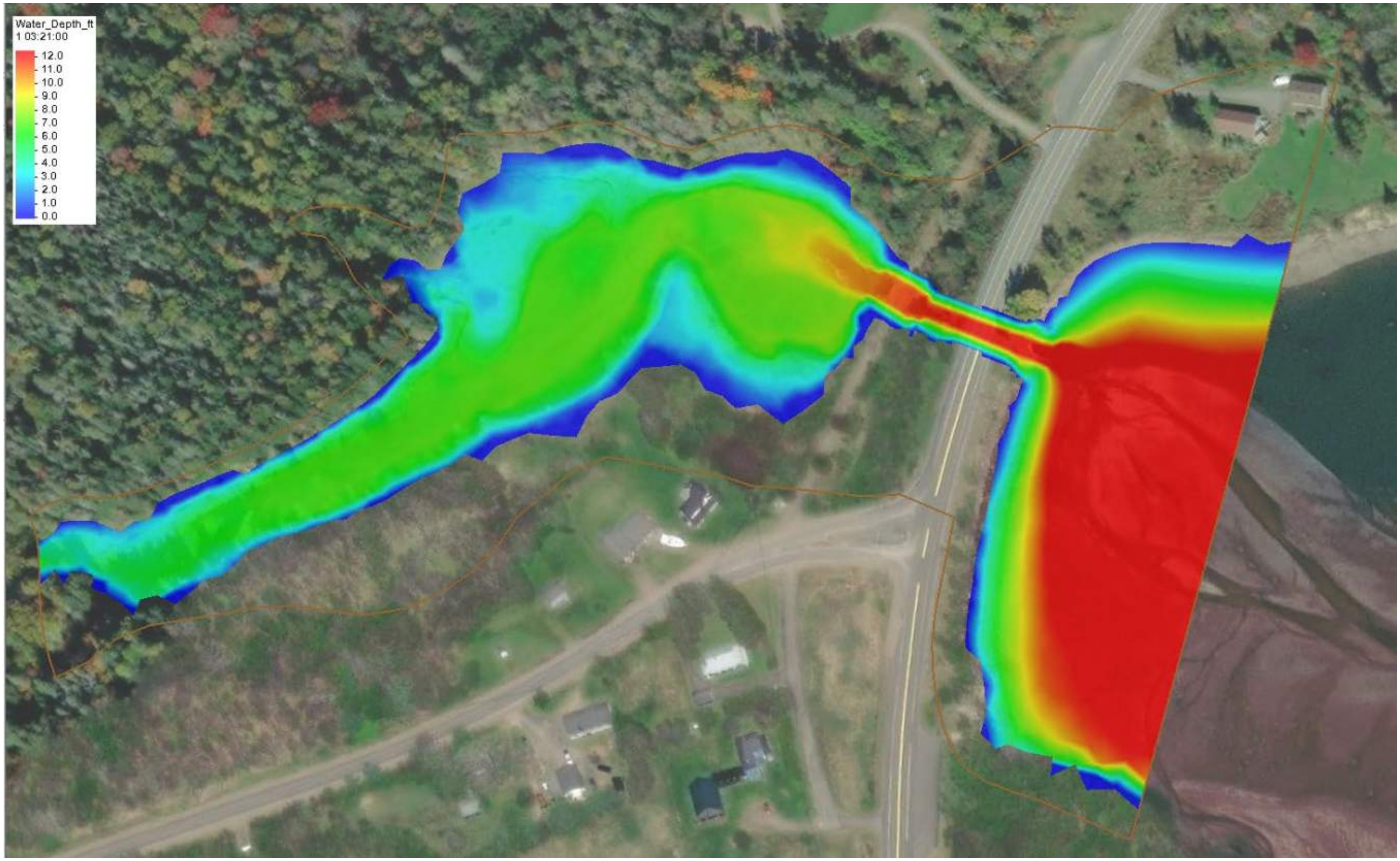


Figure D-10: Q_{100}, Average Tides plus 4ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure

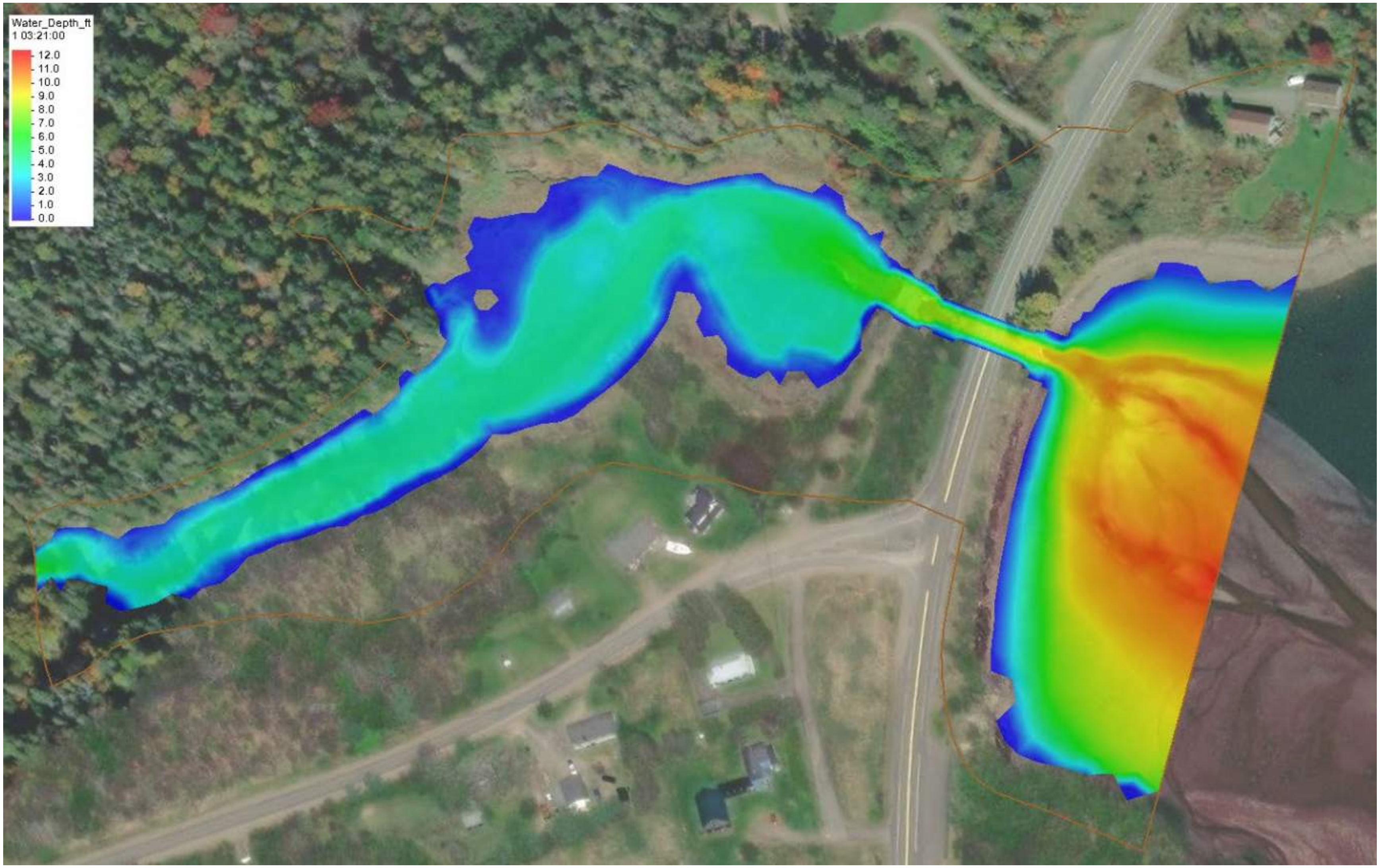


Figure D-11: Q_{500}, Average Tides – Maximum Water Depths, Proposed Single Span Bridge Structure

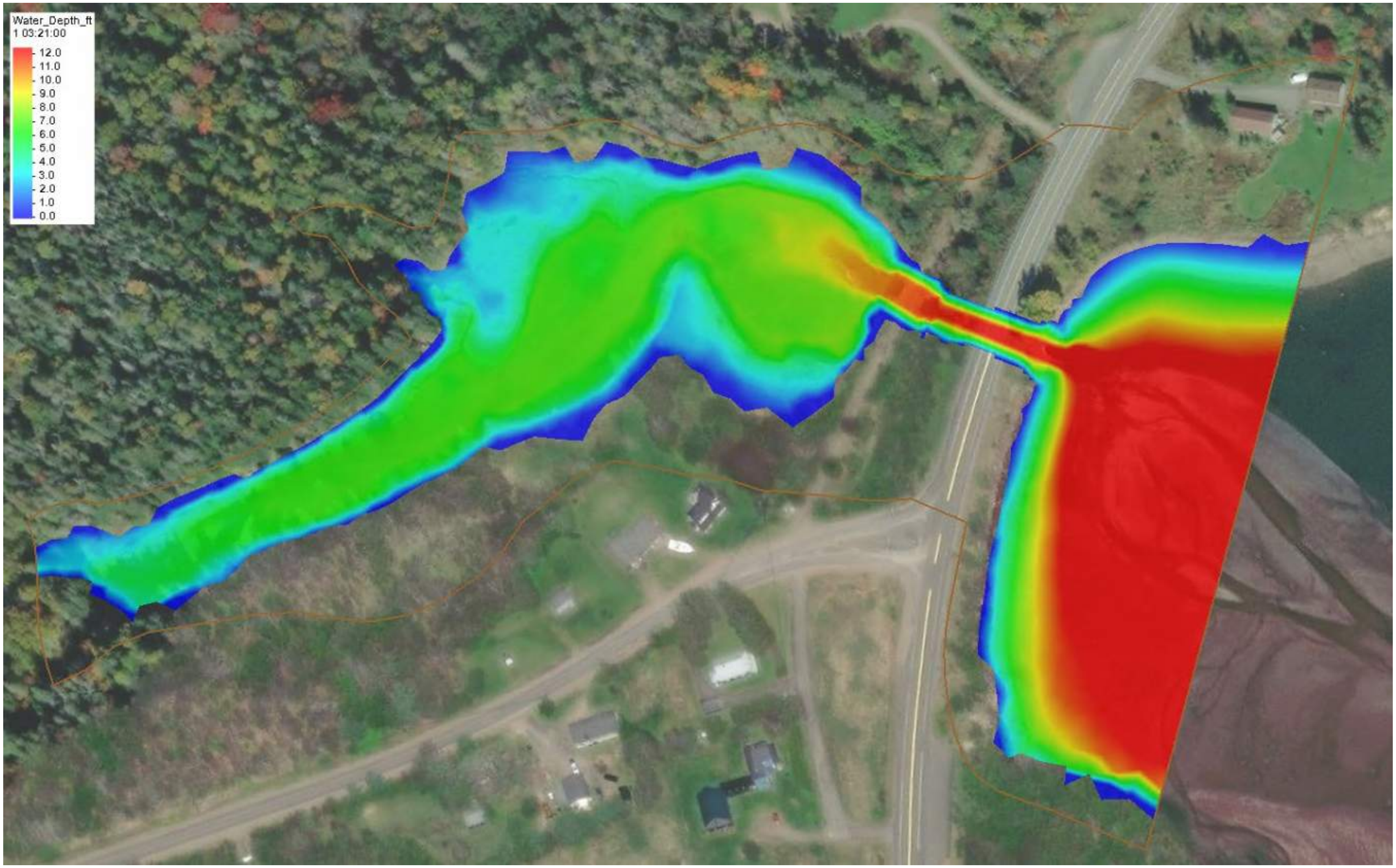


Figure D-12: Q_{500}, Average Tides plus 4ft SLR – Maximum Water Depths, Proposed Single Span Bridge Structure

Appendix E Proposed Structure Maximum Water Velocities at Low Tide

Figure E-1: **$Q_{1.1}$, Average Tides – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

Figure E-2: **$Q_{1.1}$, Average Tides plus 4 ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

Figure E-3: **$Q_{1.1}$, 2% annual chance coastal storm event – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

Figure E-4: **$Q_{1.1}$, 2% annual chance coastal storm event plus 4 ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

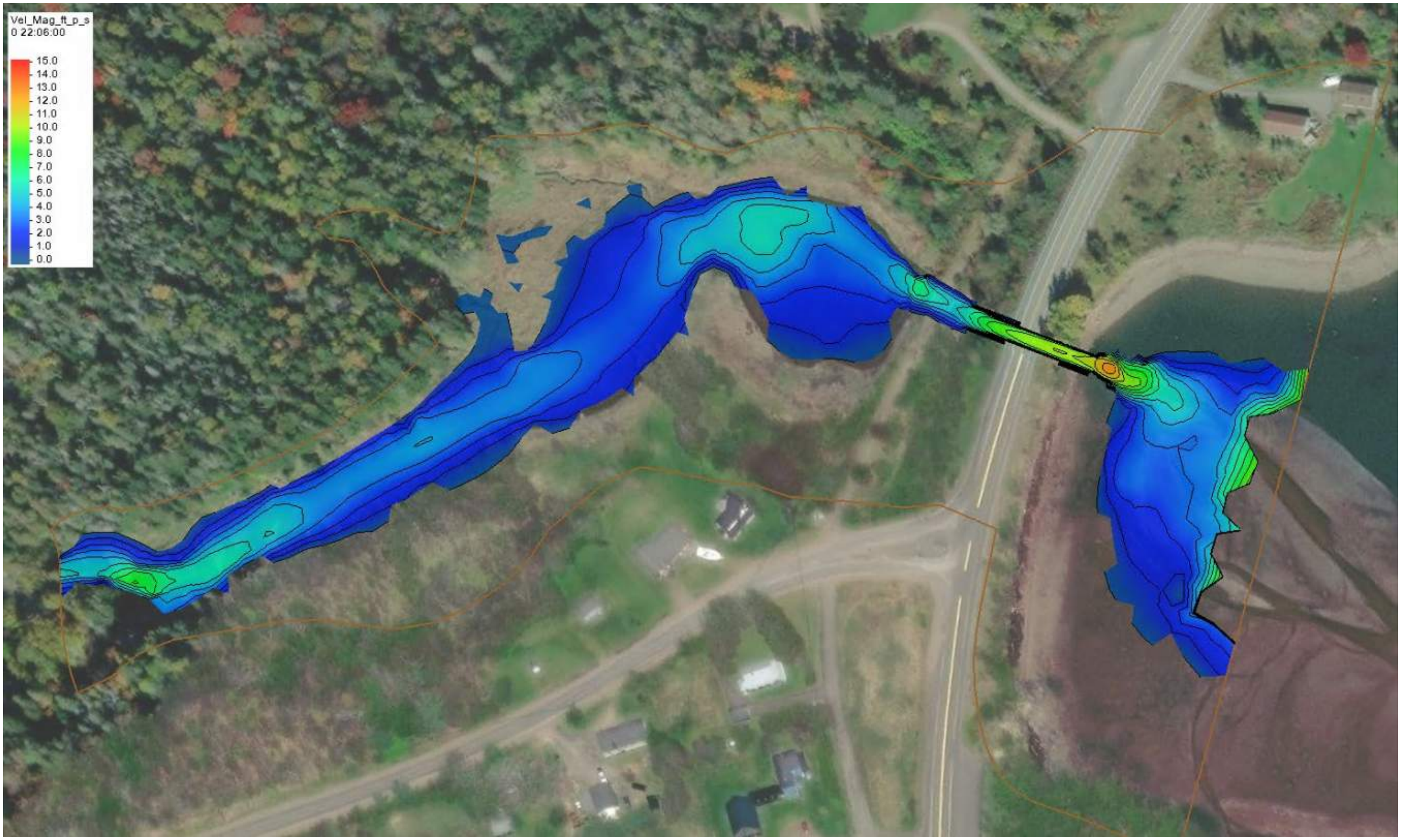


Figure E-5: Q_{10}, Average Tides – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure

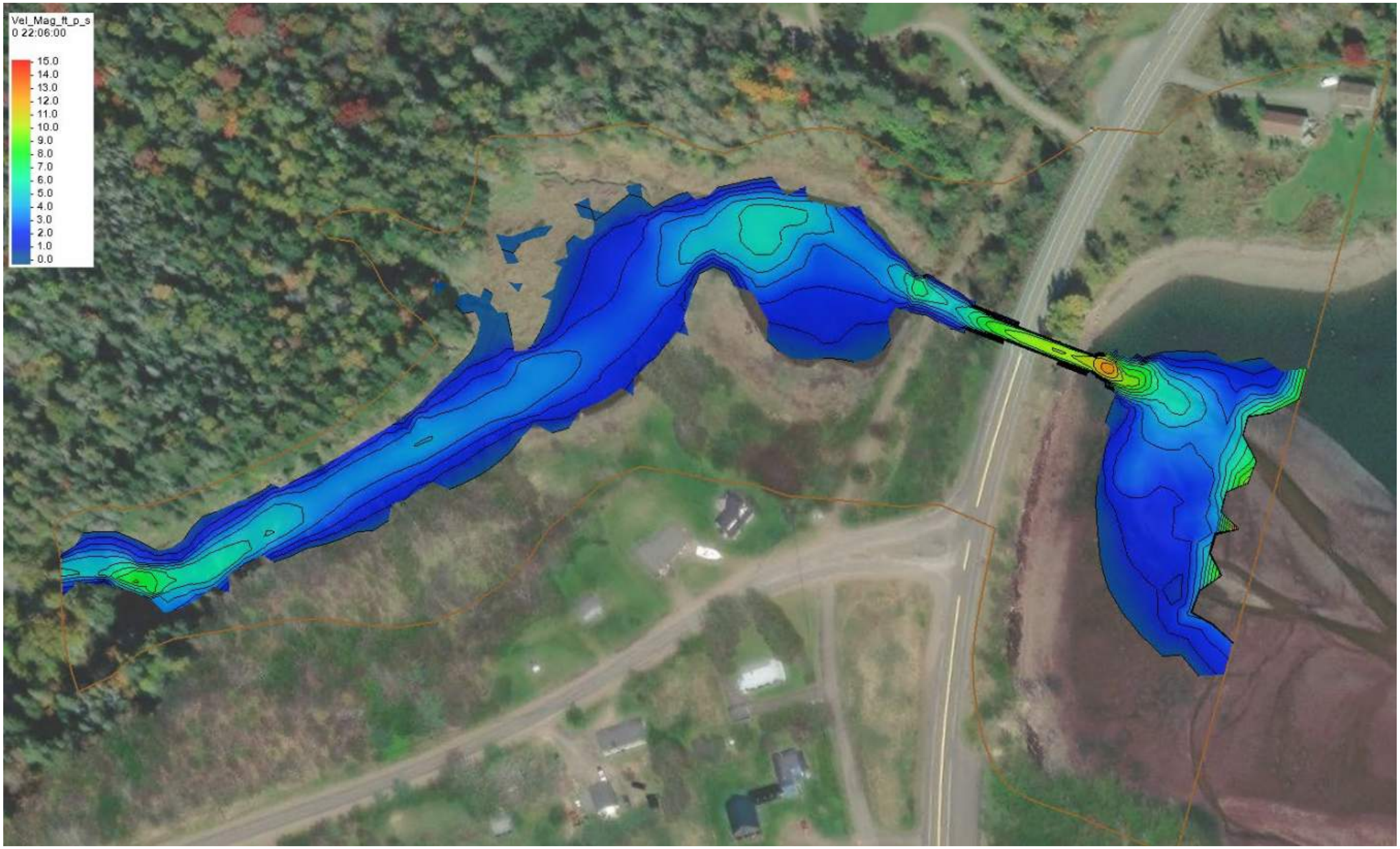


Figure E-6: **Q_{10}, Average Tides plus 4ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

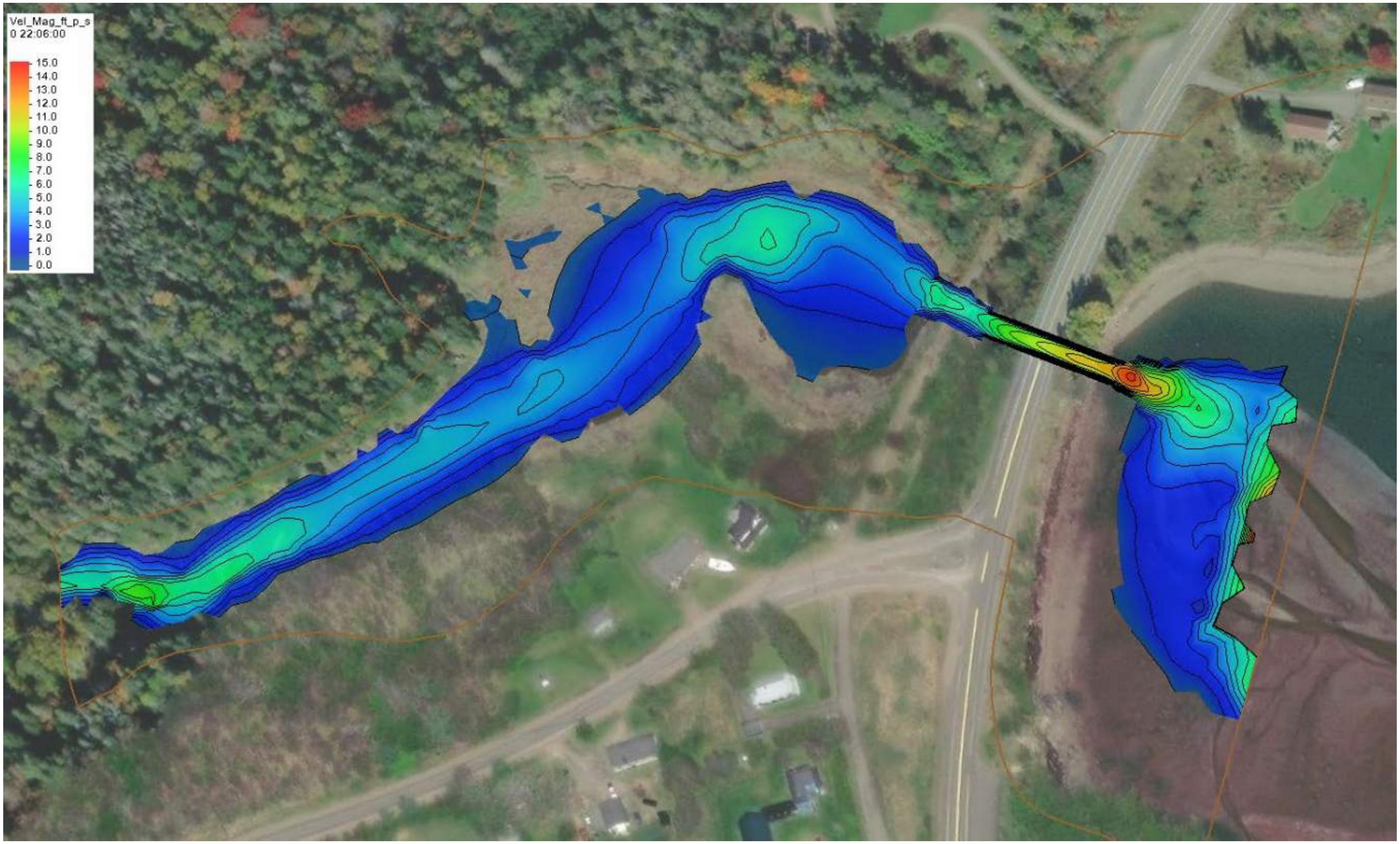


Figure E-7: Q_{50}, Average Tides – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure

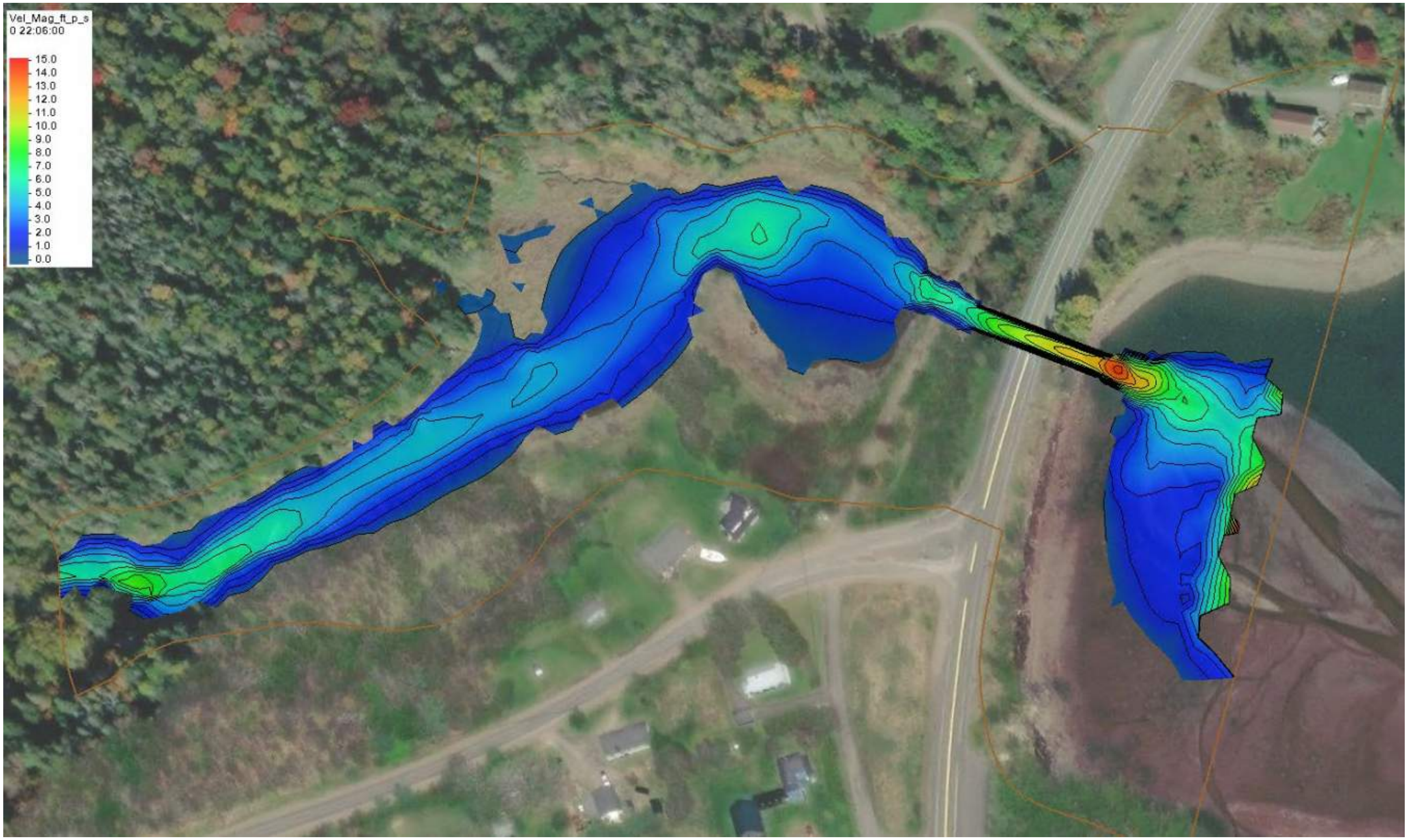


Figure E-8: Q_{50}, Average Tides plus 4ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure

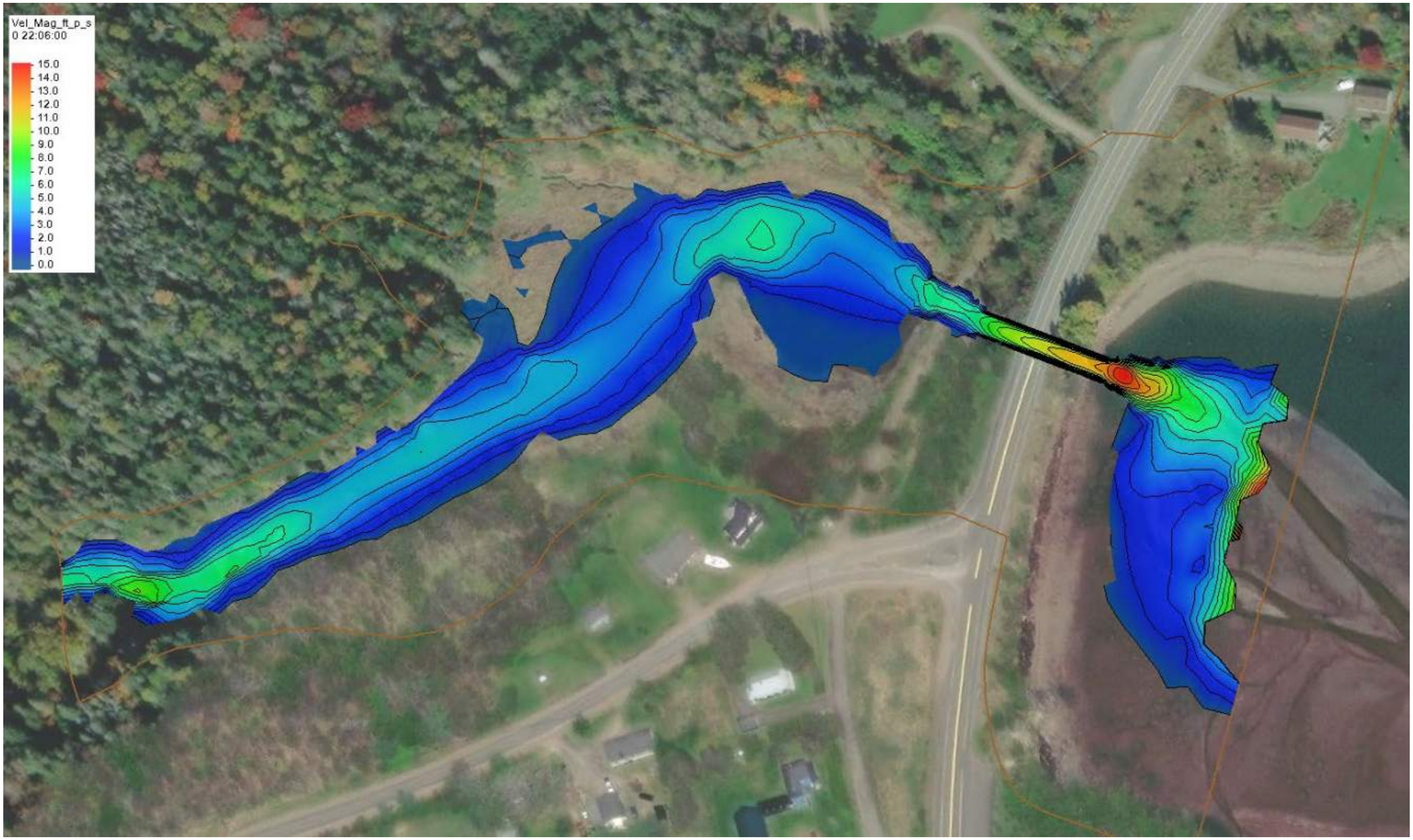


Figure E-9: **Q_{100}, Average Tides – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

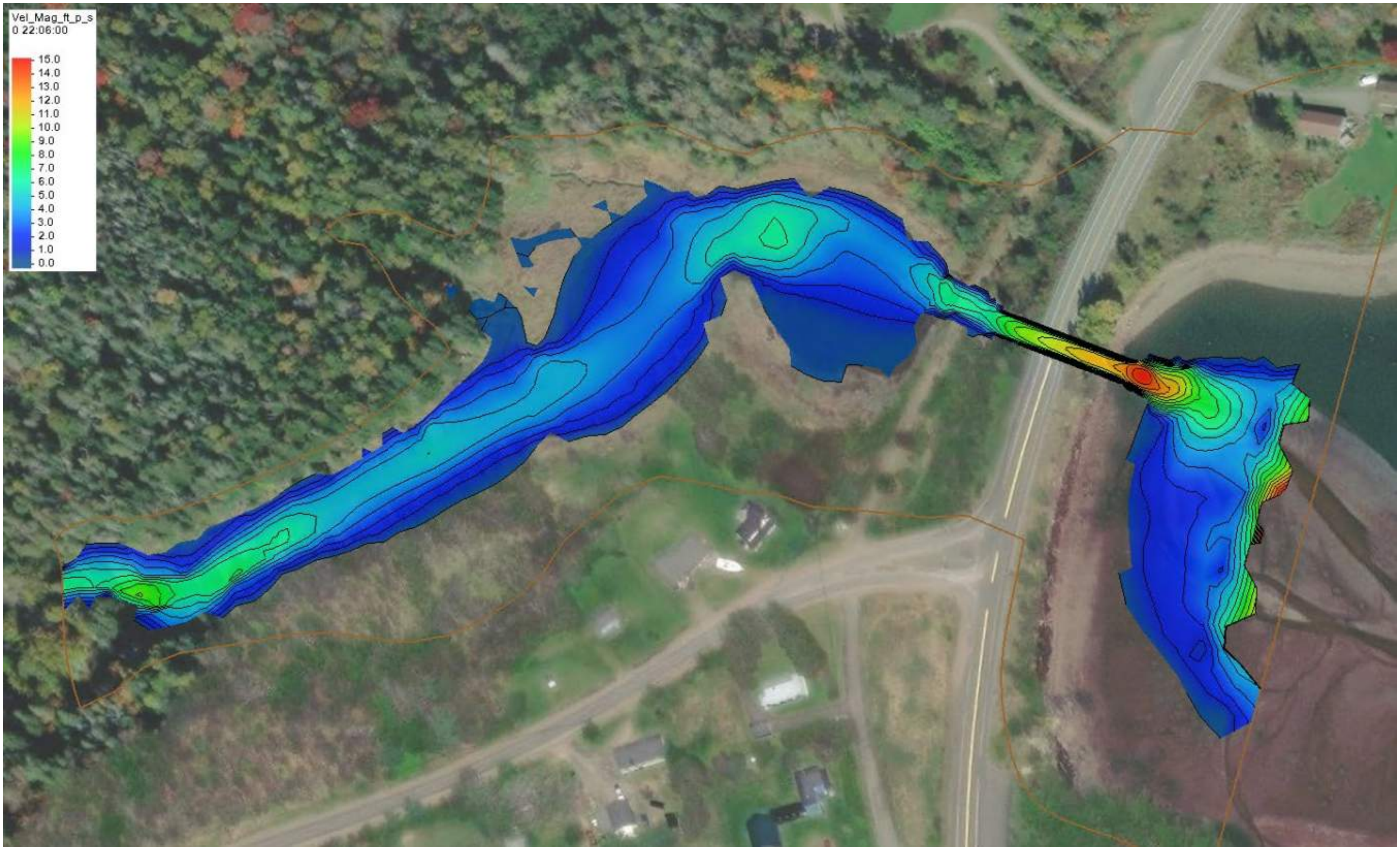


Figure E-10: **Q_{100}, Average Tides plus 4ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

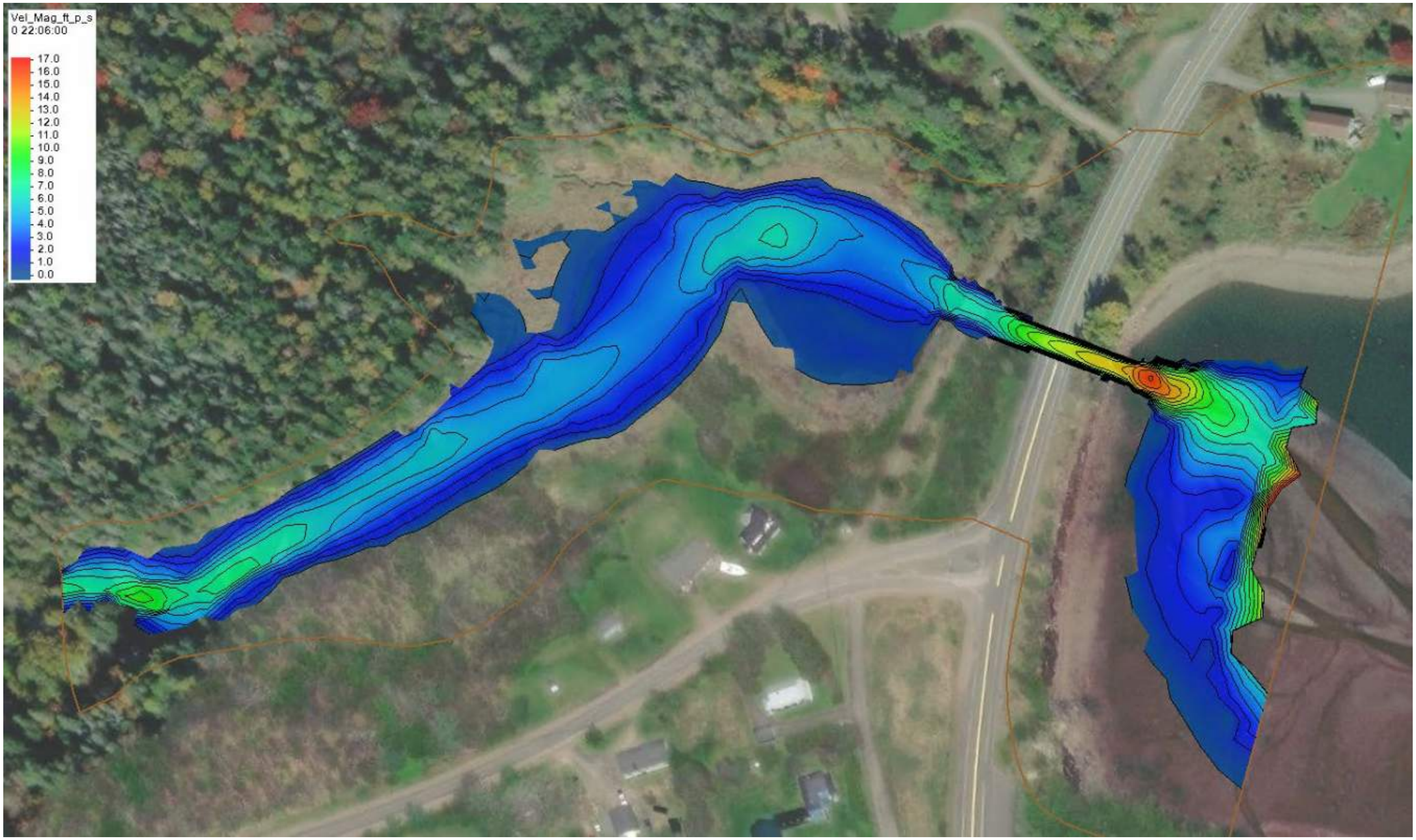


Figure E-11: Q_{500}, Average Tides – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure

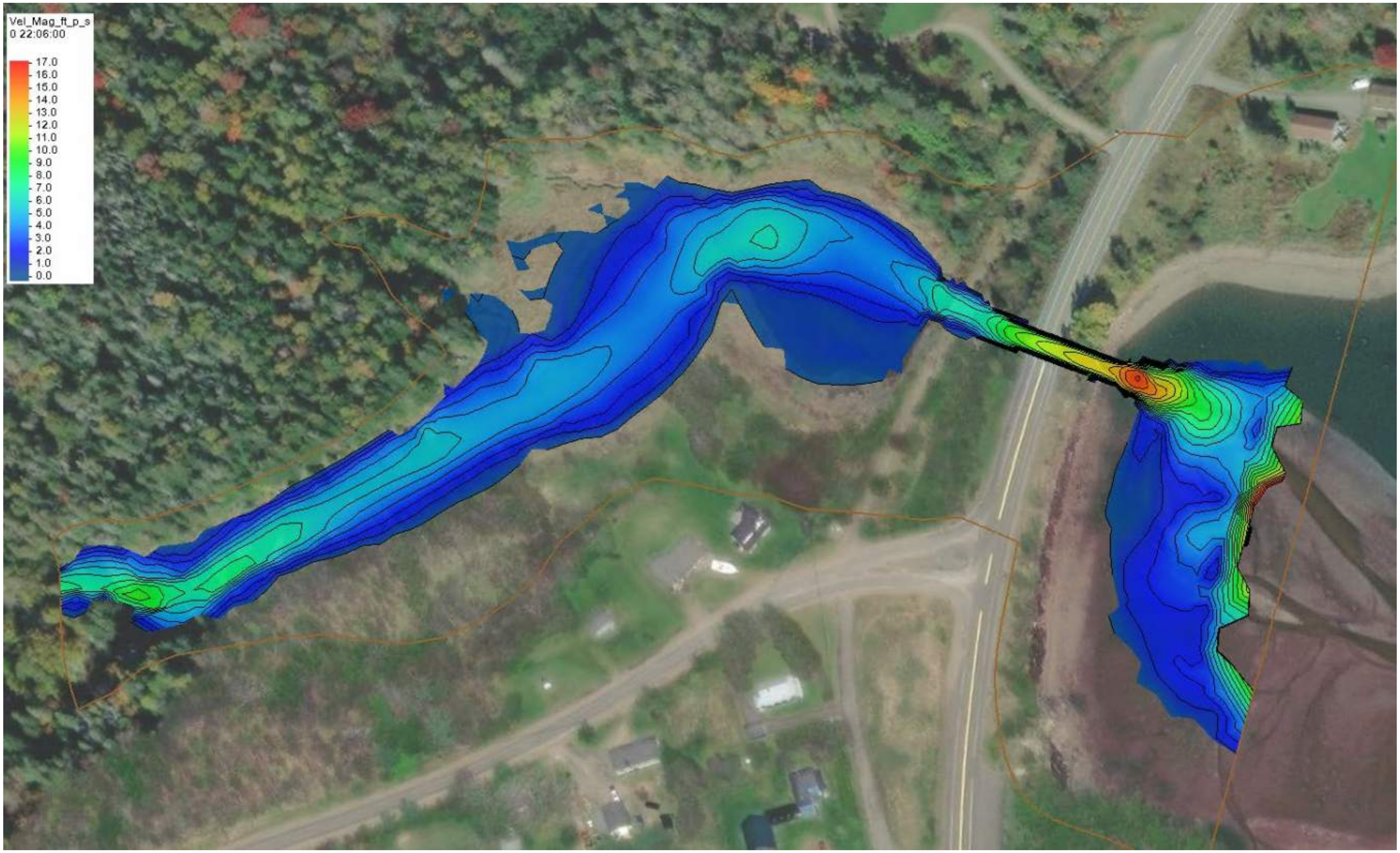


Figure E-12: Q_{500}, **Average Tides plus 4ft SLR – Maximum Velocities at Low Tide (ft/s), Proposed Single Span Bridge Structure**

Appendix F Q500 Scour Profile

Figure F-1: Q500, Average Tides – Bridge Scour Plot

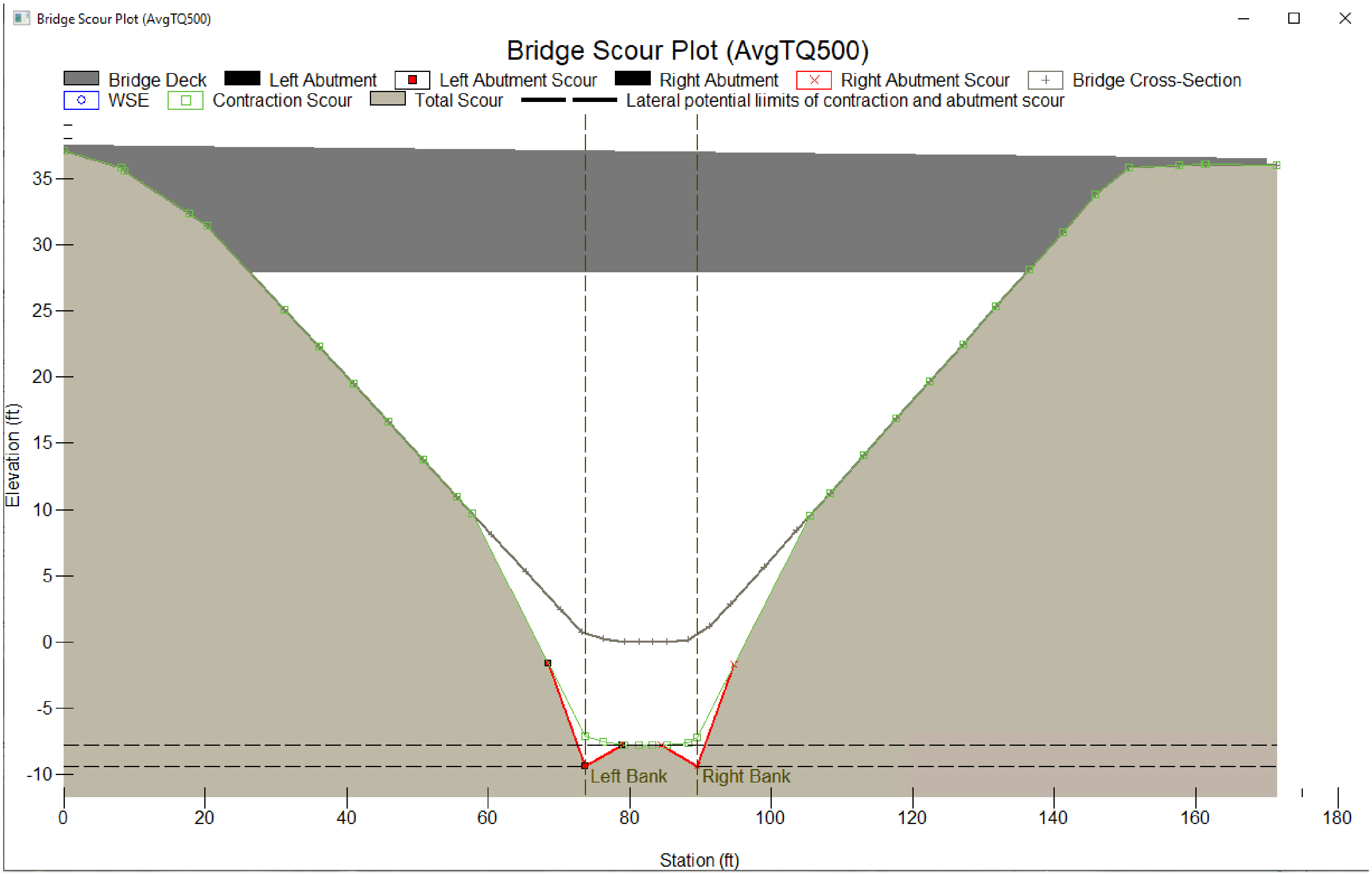


Figure F-1: **Q_{500}, Average Tides – Bridge Scour Plot**