Attachment 2:
US 74 Lumber River Crossing
Hydraulic Design Alternatives Report

North Carolina Flood Mitigation:
PROTECTing US 74 at the Lumber River

US-74 Lumber River Crossing

Alternatives Report

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1 INTRODUCTION

NCDOT is working to fulfill items required as part of the governor's Executive Order 80 (October 29, 2018) and to establish and maintain a resilient roadway network throughout North Carolina. To meet these goals, NCDOT has requested RK&K incorporate resiliency into the updated STIP cost for I-6011: upgrading US-74 to interstate standards in Columbus and Robeson Counties. The US-74 crossing of the Lumber River floodplain includes four pairs of bridges, four box culverts, and seven pipe culverts. Table 1 summarizes these existing structures; Figure 1A shows the location of each crossing by HEC-RAS SA/2D Connection name, as used in the HEC-RAS 2D model discussed in Section 3.

TABLE 1: SUMMARY OF EXISTING STRUCTURES, US-74 CROSSING OF LUMBER RIVER FLOODPLAIN

HEC-RAS SA/2D Conn. Name	NCDOT Structure #	Description	Build Date
Bridge #1	230018	3@45ft (135ft OAL), 46ft Out-Out, RC deck on steel girders, RC caps on PPC piles	1969
	230398	3@45ft (135ft OAL), 41ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	1998
Bridge #2	230004	2@67.5ft (135ft OAL), 41.08ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	2000
	230397	2@67.5ft (135ft OAL), 41ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	1998
Bridge #3	770118	3@91.67ft (275ft OAL), 41ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	2000
	770466	3@91.67ft (275ft OAL), 41ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	1998
Bridge #4	770110	3@45ft (135ft OAL), 41ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	2000
	770465	3@45ft (135ft OAL), 41.08ft Out-Out, RC deck on PPC girders, RC caps on PPC piles	1998
Culvert #1	770469	2@12ft x 6ft RCBC w/ wingwalls, 145ft OAL	1958
Culvert #2	UNK	2@6ft x 4ft RCBC w/ wingwalls, ~160ft OAL	UNK
Culvert #3	UNK	2@6ft x 4ft RCBC w/ wingwalls, ~160ft OAL	UNK
Culvert #4	770099	2@12ft x 6ft RCBC w/ wingwalls, 156.2ft OAL (Note: Inspection Report states 2@12ft x 7ft. Field measurements and centerline shots indicate 2@12ft x 6ft, consistent with 770469.)	1958
Pipe #1	N/A	1@36in RCP, OAL ~240ft	UNK
Pipe #2	N/A	2@36in RCP, OAL ~160ft	UNK
Pipe #3	N/A	1@42in RCP, OAL ~160ft	UNK
Pipe #4	N/A	1@42in RCP, OAL ~160ft	UNK
Pipe #5	N/A	1@42in RCP, OAL ~160ft	UNK
Pipe #6	N/A	1@42in RCP, OAL ~160ft	UNK
Pipe #7	N/A	2@42in RCP, OAL ~160ft	UNK

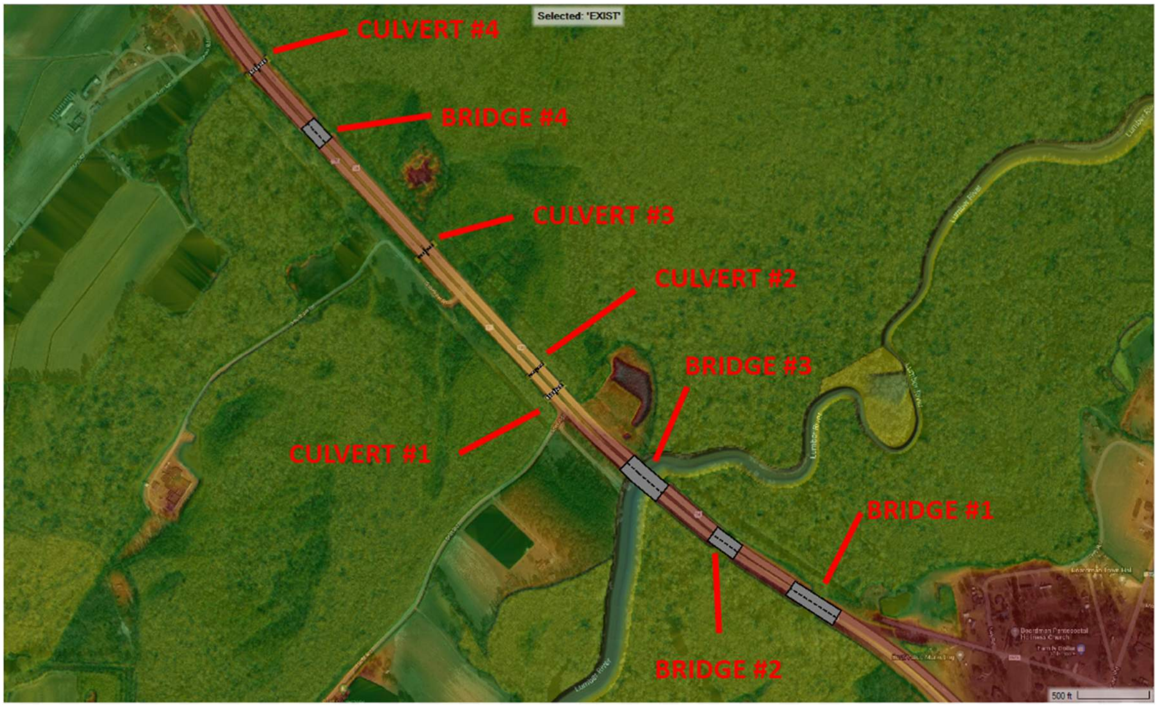


Figure 1A. Structure Map, US-74 Lumber River Crossing

The project site is located between the towns of Boardman in Columbus County and Orrum in Robeson County, North Carolina. The project location is shown in Figures 1B and 1C, the latter including an overlay of the 2D area used in the model. A structure map (Figure 1A) of all major structures along the crossing is included above.

The Lumber River, which features a large drainage area, a relatively small channel, and a wide, flat floodplain, is known for flooding local communities and overtopping crossings in strong, lower-frequency storms. The US-74 crossing between Columbus and Robeson counties is no exception, with multiple flooding events in recent years causing significant damage to hydraulic structures and the roadbed, necessitating costly repairs. As US-74 is the primary crossing of the Lumber River in the area, with few alternatives, the flooding and resulting damage can present as major obstacles to the flow of East-West traffic. This has the potential of greatly increasing travel times along the corridor, especially for motorists travelling between the Charlotte and Wilmington areas.

This Alternative Report summarizes the modeling assumptions made, the development of the existing model, and the incorporation and evaluation of proposed alternatives for damage mitigation.

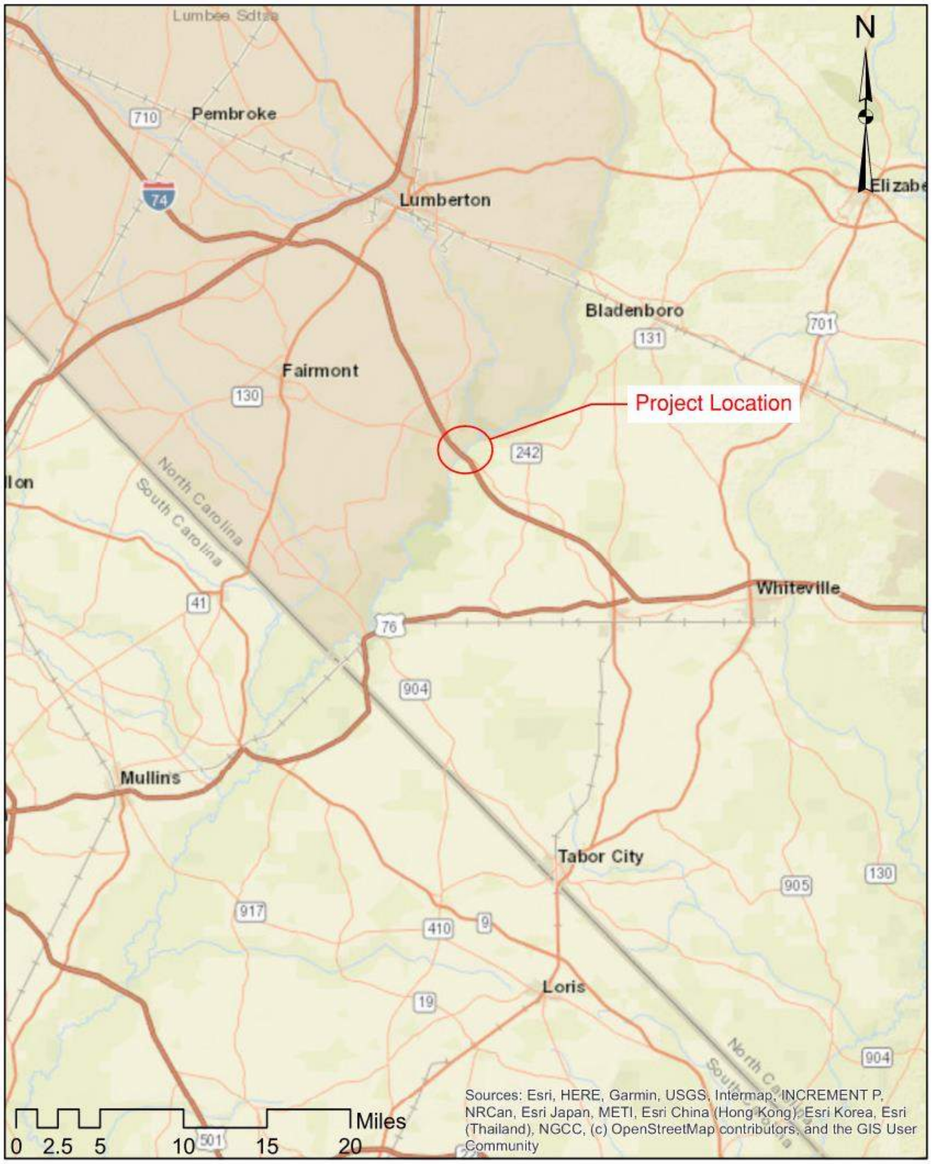


Figure 1B. Vicinity Map, US-74 Lumber River Crossing

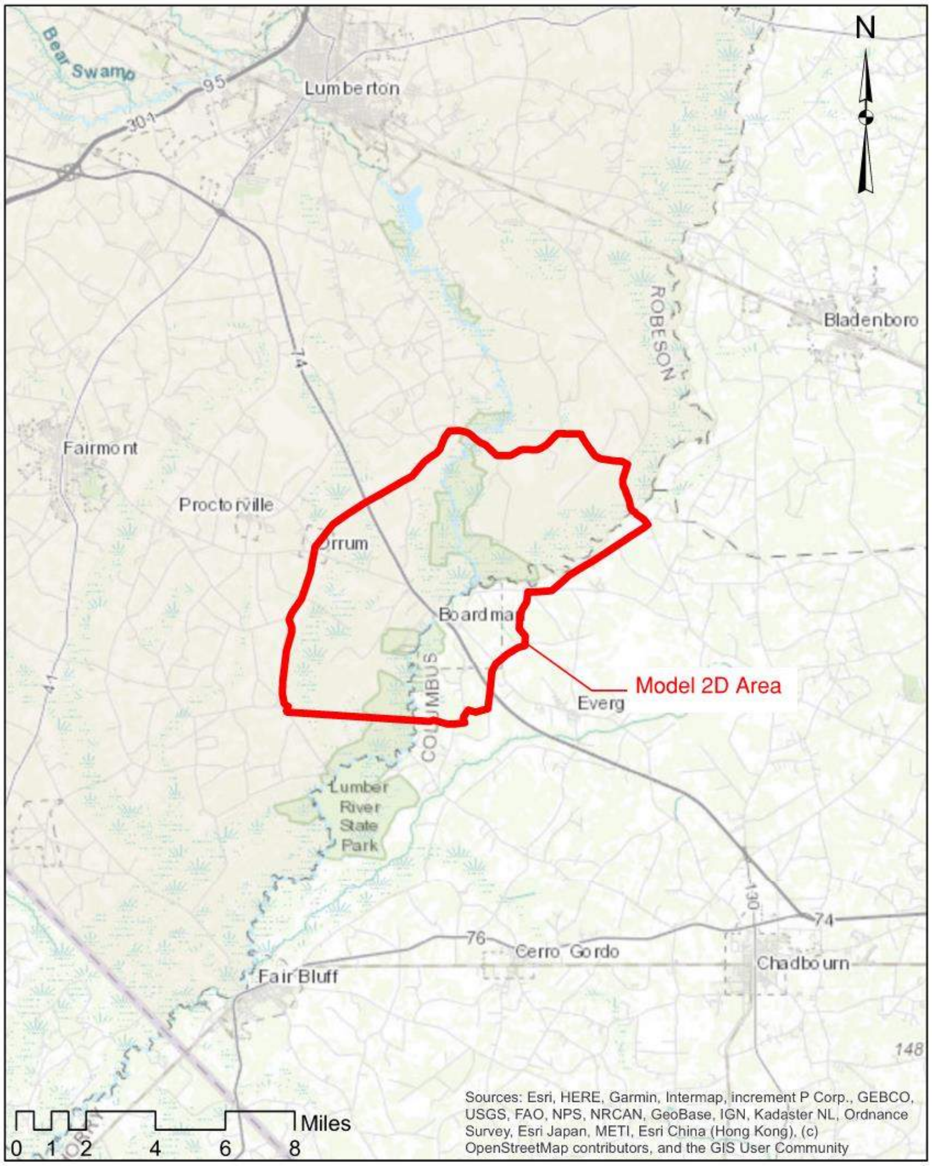


Figure 1C. Location Map, US-74 Lumber River Crossing

1.1 PROJECT OBJECTIVES

The main objectives of the project include:

1. Develop a 2D HEC-RAS model to provide an accurate representation of current flow patterns and facilitate the estimation of future flow patterns for the proposed improvement of US-74 to Interstate standards. The model is to be a Level 2 model approaching the detail of a Level 3.
2. Identify locations along the existing roadway that are most susceptible to overtopping and scouring. Determine the flows and storm frequency with which these actions occur. Document water depth in events in which overtopping occurs.
3. Develop alternatives to reduce road closures and damages during low frequency storm events.

2 PRELIMINARY ASSESSMENT

2.1 FEMA Floodplain

FEMA has performed a detailed hydrologic and hydraulic study (37155CV000C) on the Lumber River in the vicinity of the US-74 crossing. The site location is found on the Flood Insurance Rate Map (FIRM) on Panels 0205, 0206, and 0215, which collectively include Community Numbers 370659 (Town of Boardman), 370305 (Columbus County), and 370202 (Robeson County). The Lumber River floodplain is mapped as flood zone AE with a revised date of December 6, 2019. FEMA AE flood zones are areas within the floodplain where the 100-year flood boundary has been delineated and base flood elevations have been determined. Figures 2.1A through 2.1C show the FIRM at the US-74 crossing of the Lumber River and its floodplain.

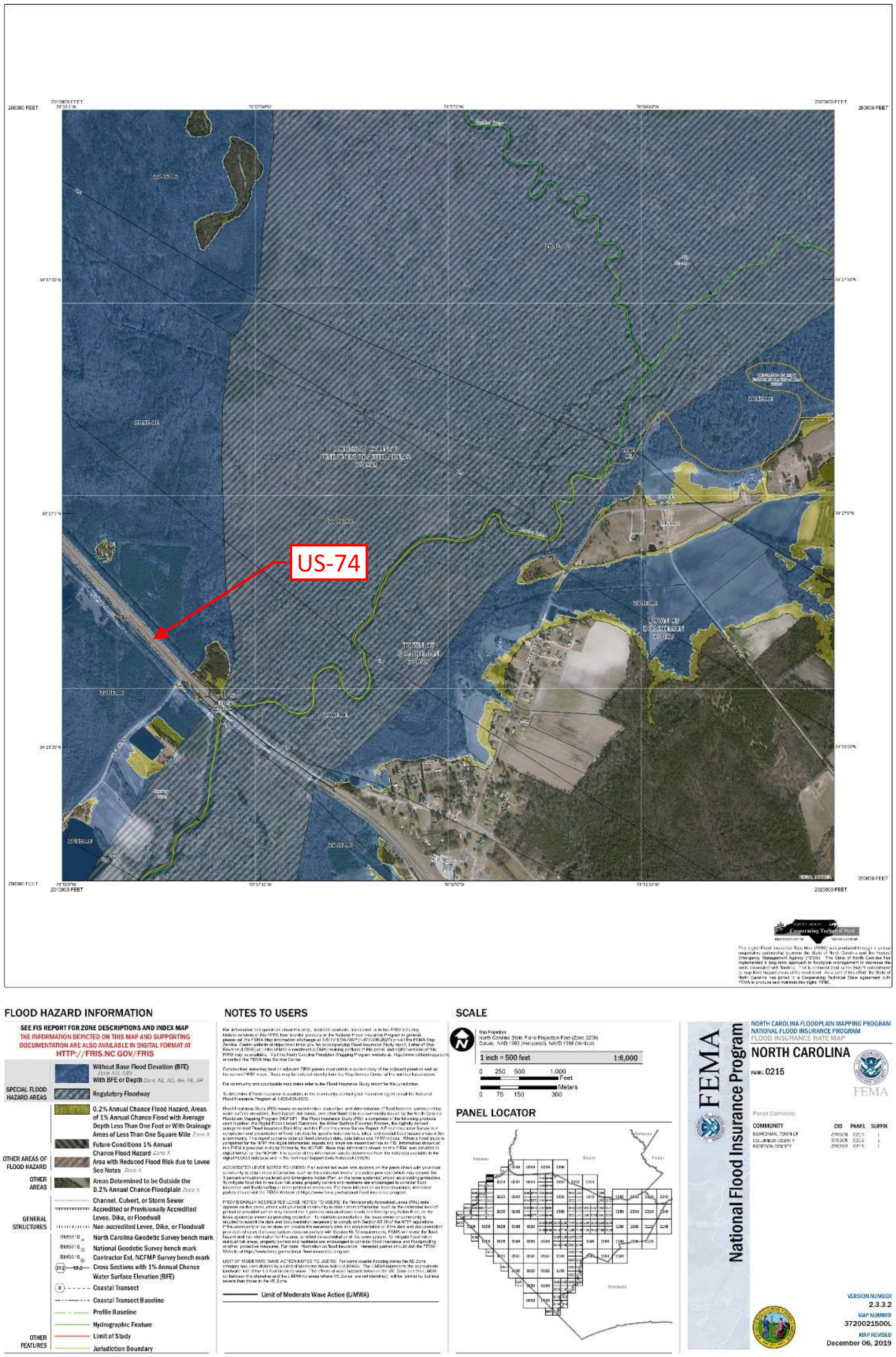


Figure 2.1A. FEMA Flood Insurance Rate Map (Panel 0215)

Figure 2.1B. FEMA Flood Insurance Rate Map (Panel 0205)

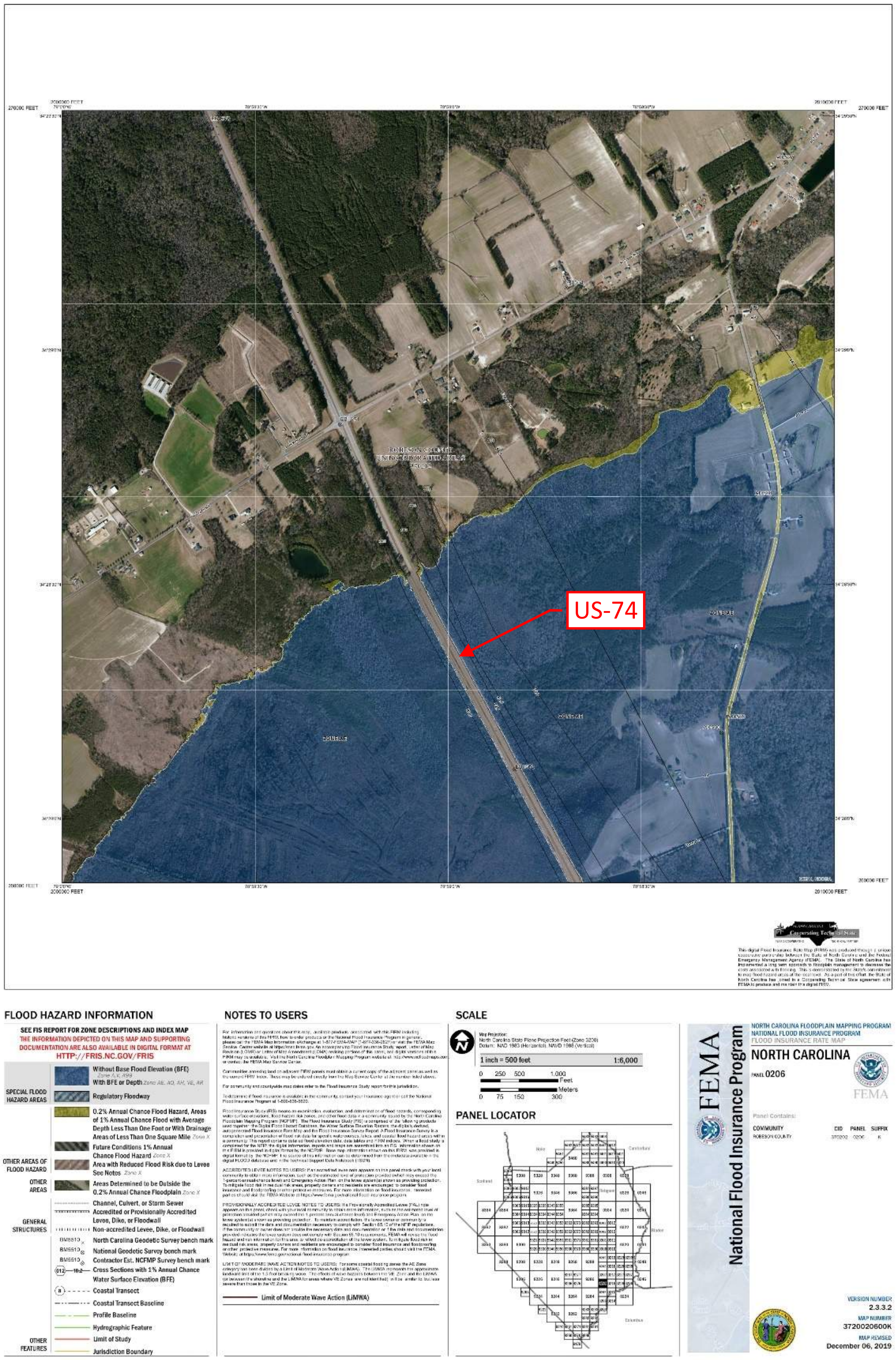


Figure 2.1C. FEMA Flood Insurance Rate Map (Panel 0206)

2.2 Historical Storm Events

The destructive impacts of strong storms and the ensuing floodwaters have been well-documented for the US-74 crossing of the Lumber River. Fast-moving water overtopped the highway for approximately one mile during Hurricane Matthew in 2016 in an incident period that lasted 10 days. This caused significant damage to the shoulder of the roadway, washing away much of the shoulder material and leaving 2,525 ft of guardrail in need of resetting or replacement. Culvert 770469 (Culvert #1) experienced severe scour that undermined the earth supporting it, causing it to shift from its original position. A smaller non-inventory culvert (Culvert #3) also experienced significant scour requiring repairs. Much of the riprap used for slope protection under bridge 770465 (Bridge #4 DS) was blown out, leaving the abutments susceptible to scour in future events. The combination of saturated soils and strong velocities resulted in the road surface lifting in two locations, producing holes in the eastbound lanes measuring approximately 450ft x 6ft. The cumulative cost of these repairs totaled $1,962,091. Figure 2.2A below shows an example of the roadway and shoulder damage incurred due to Hurricane Matthew.

Figure 2.2A. Large Hole in Eastbound Lanes Following Hurricane Matthew

Similar damage was seen over the 22-day incident period in 2018 when floodwaters from Hurricane Florence eroded 4,000ft of roadway shoulder and embankment, requiring guardrail replacement along the entire length, as well as subgrade and road surface repair of an area measuring approximately 120ft x 6ft. Repairs due to scour were necessary on several structures, including culvert 770099 (Culvert #4), bridge 770110 (Bridge #4 US), bridge 770118 (Bridge #3 US), bridge 770466 (Bridge #3 DS), culvert 770469 (Culvert #1), and bridge 770465 (Bridge #4 DS). The cumulative cost of these repairs totaled $2,696,974, of which $2,258,978 was solely for repair of the shoulder and embankment. Figure 2.2B shows an example of should damage during Hurricane Florence. See Appendix B for additional photos of damage from both events.

Figure 2.2B. Extensive Shoulder Damage following Hurricane Florence

2.3 Existing Conditions

The project reach in the vicinity of the US-74 crossing of the Lumber River largely consists of swamp land/wooded wetlands throughout the floodplain except for some agricultural and forestry lands and sparse residential. Many of the residential properties within the 100-yr floodplain are in Robeson County along NC-72, just upstream of US-74, and along Ann Rd (SR-2244) and VC Britt Rd (SR-2245) on the downstream side of US-74. A wastewater treatment plant is in the floodplain at the end of Woodrow Rd (SR-2312) on the downstream side of US-74 but sits above the 500-yr WSEL. The Lumber River channel is stable and meanders significantly along the wide, flat floodplain. The channel is well-defined, but relatively small, necessitating use of the floodplain to convey the discharge from larger storm events.

As part of developing the model of the US-74 crossing of the Lumber River, a field assessment was conducted by RK&K in May 2022. During this visit, data was collected on the existing structures along the crossing, including sizes and invert elevations. A boat was used to collect bathymetric data of the Lumber River to confirm channel geometry in the vicinity of the crossing. Elevations were taken along the high points of the roadway to confirm LiDAR data and ensure accuracy in determination of overtopping locations.

2.4 RK&K AGOL Map

To facilitate streamlined data collection and presentation, RK&K developed an ArcGIS Online (AGOL) map for the US-74 crossing of the Lumber River. This map merges GIS datasets from FEMA, USFWS, NRCS, and local municipalities. Nodes representing historical damage have been geolocated based on reports provided by NCDOT; included are images and specific details that characterize the damage and subsequent repairs. This map was used during the field assessment to collect data and imagery of each hydraulic structure along the crossing via a tablet. Not only does this eliminate the need for later processing and uploading of field data, but it also permits anyone with access to the AGOL to retrieve this data in real time, as it is collected. The map, shown in Figure 2.4, is secure, but access can be provided by RK&K on request.

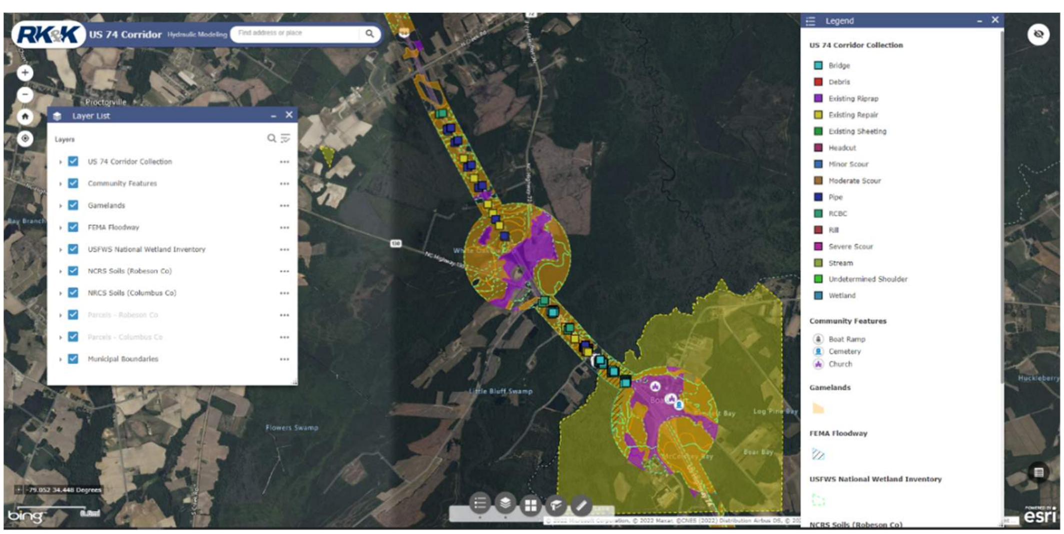


Figure 2.4. RK&K AGOL Map

2.5 Hydrology Analysis

The contributing watershed boundary was delineated using StreamStats by USGS. StreamStats delineated a total drainage area of 1230 square miles for the Lumber River at US-74. Considering the proximity of the Big Swamp confluence to the US-74 crossing and the significant width of the Lumber River floodplain, the choice was made to perform calculations well upstream of the confluence and to provide unique flows for both the Lumber River and Big Swamp. The FIS report (37155CV000C) shows that Big Swamp has been evaluated as a Limited Detailed study, and therefore only 100-yr discharges are available. Because of this, StreamStats was used to determine flows for both Big Swamp and the Lumber River, with calculations taken at the upstream structures for both reaches. See section 3.1.1.2 for additional information regarding upstream boundary locations.

The USGS SIR 2009-5158 regression method was utilized by StreamStats for flow calculations due to an impervious area of less than 10%, as determined from the NLCD 2006 impervious dataset, and a drainage area exceeding 1 square mile. A report for each reach can be found in Appendix A. Table 2A below compares the regression method (Fixed Region Equations) to flows from gage analysis and FEMA flows for the Lumber River at US-74. This table shows that the flows from the regression method are consistently largest, and, therefore, suitably conservative for this application. To further validate the use of StreamStats, the 100-yr discharges from StreamStats and FEMA were compared for both reaches (Table 2B). Again, the StreamStats flows were found to be larger and therefore conservative. Finally, the sum of the Big Swamp and Lumber River StreamStats discharges sampled at the upstream structures was found to be larger than the Lumber River StreamStats flows taken at the US-74 crossing (Table 2C), further supporting the use of unique flows and upstream boundary conditions for each reach. Flows highlighted green represent flows used in the model.

TABLE 2A: PEAK FLOWS FOR LUMBER RIVER AT US-74 CROSSING

Return Period (years)	Fixed Region 95% Prediction Intervals		Fixed Region Equations (cfs)	USGS Gage Analysis (cfs)	FEMA (cfs)
	Lower Limit (cfs)	Upper Limit (cfs)			
2	3,100	11,500	5,980	4,970	N/A
10	6,830	25,900	13,300	10,000	10,100
25	8,470	34,900	17,200	12,900	N/A
50	9,760	43,500	20,600	15,300	15,400
100	10,900	52,700	24,000	17,700	18,000
500	13,200	77,600	32,000	23,900	24,500

TABLE 2B: 100-YEAR DISCHARGE COMPARISON – STREAMSTATS VS FEMA

Reach	100-yr Discharge	
	StreamStats	FEMA
Lumber River	16900	15500
Big Swamp	13800	9423

TABLE 2C: DISCHARGE COMPARISON – US-74 LUMBER CROSSING VS SUM OF UPSTREAM FLOWS

Site	StreamStats Discharges			
	10-yr	50-yr	100-yr	500-yr
Lumber River @ Willouby Road	9410	14600	16900	22500
Big Swamp @ Old Whiteville Road	7240	11600	13800	18600
SUM (Upstream Boundaries)	**16650**	**26200**	**30700**	**41100**
Lumber River @ US-74	**13300**	**20600**	**24000**	**32000**

3 DESIGN APPROACH

3.1 Two-Dimensional Hydrodynamic Modeling

A two-dimensional (2D) hydrodynamic analysis of the US-74 Lumber River crossing was performed using HEC-RAS. A 2D model offers the following benefits for design as compared to the HEC-RAS 1D analysis:

- Analyzes shear stress, velocity, velocity vectors and flow depth along the entire stream channel and floodplain surface areas and not only at HEC-RAS cross section locations.
- Calculates varying shear stress, velocity, velocity vectors and flow depth values laterally across the stream channel and floodplain compared to one average shear stress value in each channel and overbank area as calculated by HEC-RAS 1D.
- More effectively models flow transitions, ineffective flow areas, channel and floodplain bend losses, and flow expansion and contraction using finite difference solution.

This 2D analysis for the project reach was performed using HEC-RAS 6. The results of the analysis provide a distribution of shear stress on the topographic digital terrain model, direction and magnitude of velocity, depth, and water surface elevation for multiple storm events. The results of the model have been used to inform design floodplain widths, valley slopes, and structure stability.

3.1.1 Model Setup

3.1.1.1 Downstream Boundary Condition

The downstream boundary condition was computed using the normal depth slope downstream of the project limits.

3.1.1.2 Upstream Boundary Conditions

The upstream boundary conditions consist of two input hydrographs based on discharges from StreamStats reports. The Lumber River upstream boundary condition is located along the Willoughby Road (SR-2121) crossing, approximately 5.2 miles upstream of the US-74 crossing, and 3.8 miles upstream of the confluence with Big Swamp. The Big Swamp upstream boundary condition is located along the Old Whiteville Road (SR-1002) crossing, approximately 4.2 miles upstream of the confluence with the Lumber River. The input hydrograph for each stream is a step hydrograph that includes four discharges – 10-year, 50-year, 100-year, and 500-year peak flows, with each step of the hydrograph lasting 5 days. The "stepped flow" hydrograph simulates steady flow for each of the modeled events while incorporating the effects of floodplain wetting and volume fluxes as the discharges ramp up to the next design storm. The time component was evaluated to ensure that the model is run long enough for steady state conditions to occur during each return interval. Steady state for the 100-year event is reached on Jan 16, 2023 at 00:00, while the steady state for the 500-year event is reached on Jan 21, 2023 at 00:00. Due to the variable timestep, the latest date available for results is Jan 20, 2023 just past 23:30, which is used for all 500-year results in this report. The time component was modified based on Hurricane Florence, which saw peak discharges on the 4th day of the event.

The step hydrographs for the Lumber River and Big Swamp are shown in Figures 3.1.1A and 3.1.1B, respectively. Discharges used for each modeled input are summarized in Table 3.

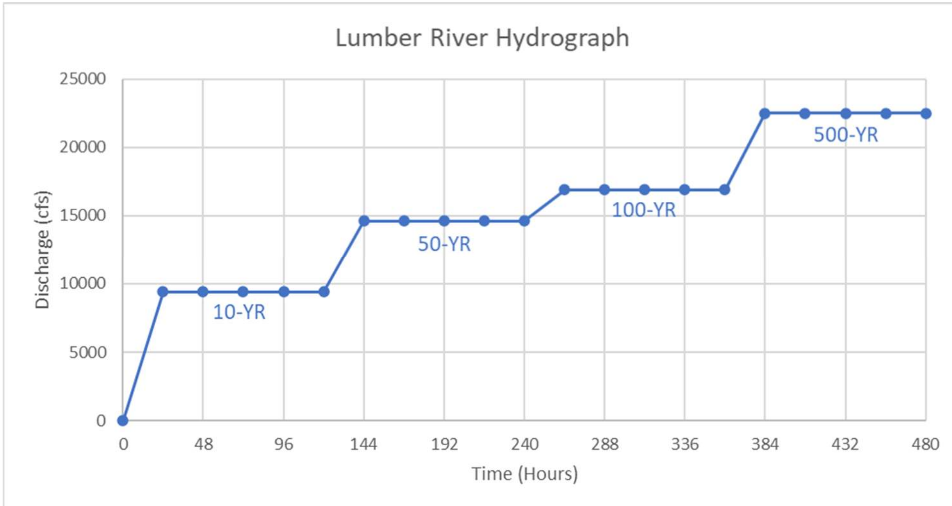


Figure 3.1.1A. Stepped Flow Hydrograph Upstream Boundary Condition on Lumber River

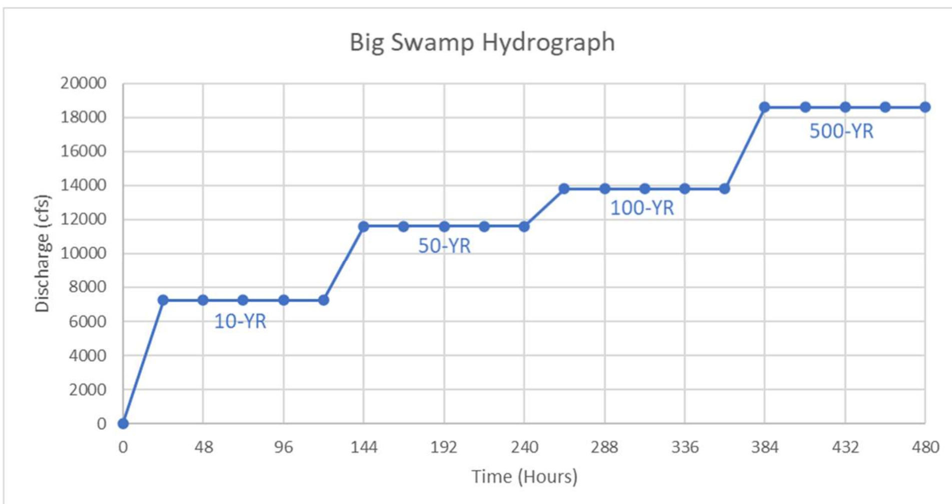


Figure 3.1.1B. Stepped Flow Hydrograph Upstream Boundary Condition on Big Swamp

TABLE 3: SUMMARY OF MODELED DISCHARGES

Design Reach	10-Yr Discharge (cfs)	50-Yr Discharge (cfs)	100-Yr Discharge (cfs)	500-Yr Discharge (cfs)
Lumber River	9410	14600	16900	22500
Big Swamp	7240	11600	13800	18600

3.1.1.3 Computational Grid and Roughness Distribution

The existing conditions computational grid was created using Bare Earth LiDAR data available on the North Carolina Flood Risk Information System and supplemented with channel data from the FEMA FRIS Study of the Lumber River. The land use was delineated based on the National Land Cover Database available through NC State and site observations. Manning's n values were assigned to the computational surface based on the delineated land use categories. The model uses a 300-foot grid across the floodplain with a minimum 10-foot grid in the vicinity of the bridges, culverts and breaklines. Breaklines were used to provide greater definition of the Lumber River channel, the toe of fill of the US-74 crossing, and distinct floodplain features in vicinity of the crossing, such as other roadways and irrigation ditches. As secondary alignments were not investigated for existing hydraulic structures as part of the US-74 field assessment, crossings along NC-72 and NC-130 in Robeson County were located through use of aerials and StreetView. To mimic the conveyance of these structures, terrain modifications were used to cut narrow trenches through the roadway at the lowest adjacent elevation. Breaklines along the alignments were split at these crossings, with additional breaklines plotted tangent to flow, reducing the cell size to better define the cuts (Figure 3.1.3A).

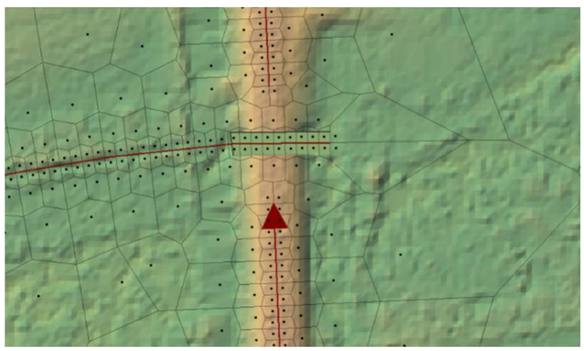


Figure 3.1.1.3A. Example Terrain Modification Across NC-72

3.1.1.4 Model Calibration and Validation

RK&K evaluated the existing model results using known high-water marks along the roadway profile provided by NCDOT for two storm events, Hurricane Matthew and Hurricane Florence. Using gage data, the maximum flow during both Hurricane Matthew (37,800 cfs) and Hurricane Florence (35,400 cfs) was determined. Based on the flows, the high-water marks were compared to the 500-year event (37,900 cfs). In addition, NCDOT provided information of the peak storm at the Lumber River Crossing for US-74 Corridor for both events which was used to encompass each event in the full range of flows. Using these storm events, RK&K was able to apply model refinements and carefully evaluate the areas of complex hydraulics.

While in development, a significant water surface drop of 1-2ft across Bridge #3 was experienced in all plans, especially in low flow. Evaluation determined that the Momentum equation was being overly conservative for this bridge. Therefore, Bridge #3 was updated to run Energy only in low flow conditions, which eliminated the drop and produced results more in-line with expectations. All other bridges are stable, and therefore retain larger of Momentum and Energy for low flow conditions.

3.1.2 HECRAS 2D Model Results

RK&K has developed the HEC-RAS 2D model to be able to identify existing sites along the crossing prone to damage in large storm events. This model provides visual and numerical outputs indicating flow patterns and velocities around abutments, through structures, and across the roadway, especially in the 100-year and 500-year events. Water surface and depth results facilitate the evaluation of upstream and downstream impacts from altering the existing grade, adding a new structure, or resizing an existing structure. The following sections detail the assessment of known issues along the crossing using this model.

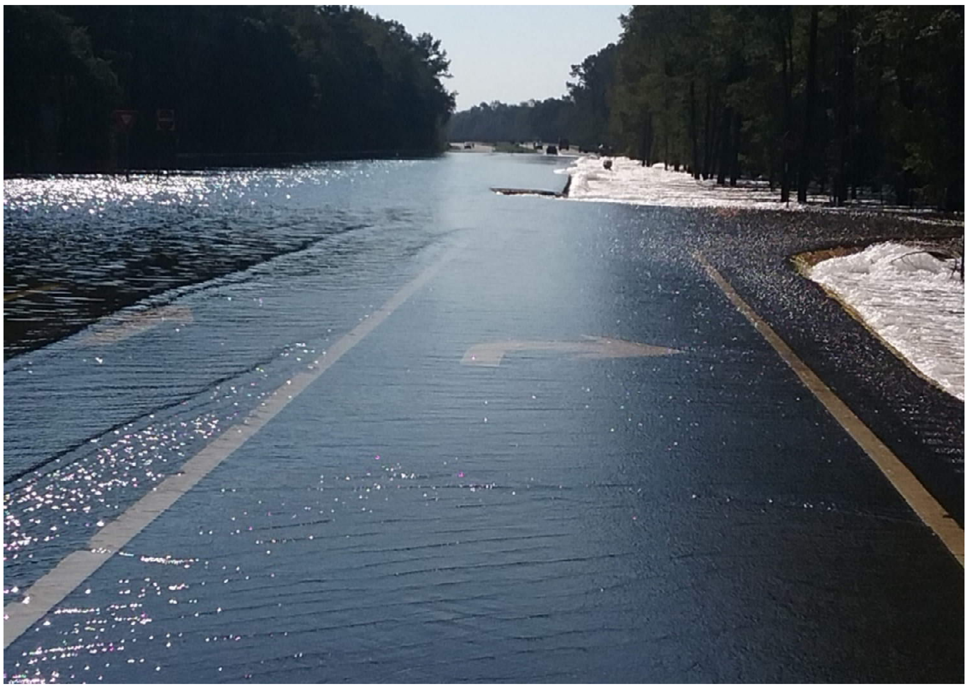


Figure 3.1.2.1A. US-74 Overtopping & Shoulder Scour at Woodrow Road – Hurricane Florence

3.1.2.1 Overtopping & Shoulder Scour

The model shows that the road begins to overtop the westbound lanes of US-74 along nearly the entire 1100ft stretch between Britt Road and Woodrow Road as headwater builds from the 50-year event (Figure 3.1.2.1B). This is soon followed by overtopping of the eastbound lanes, first spilling over immediately southwest of Woodrow Road (Figure 3.1.2.1C). This is the location pictured above in Figure 3.1.2.1A, beyond the end of the turn lane. Overtopping is largely contained between these two roads through the 100-year event, extending approximately 1500ft along the westbound lanes and 1200ft along the eastbound lanes (Figure 3.1.2.1D).

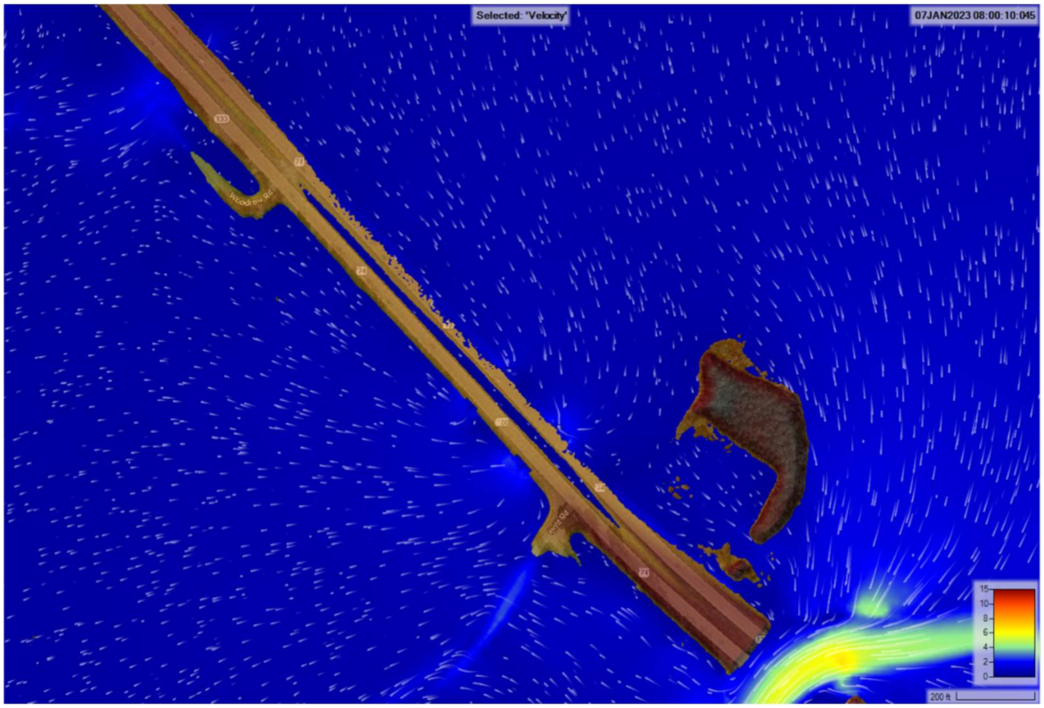


Figure 3.1.2.1B. Overtopping of Westbound Lanes During 50-Year Event

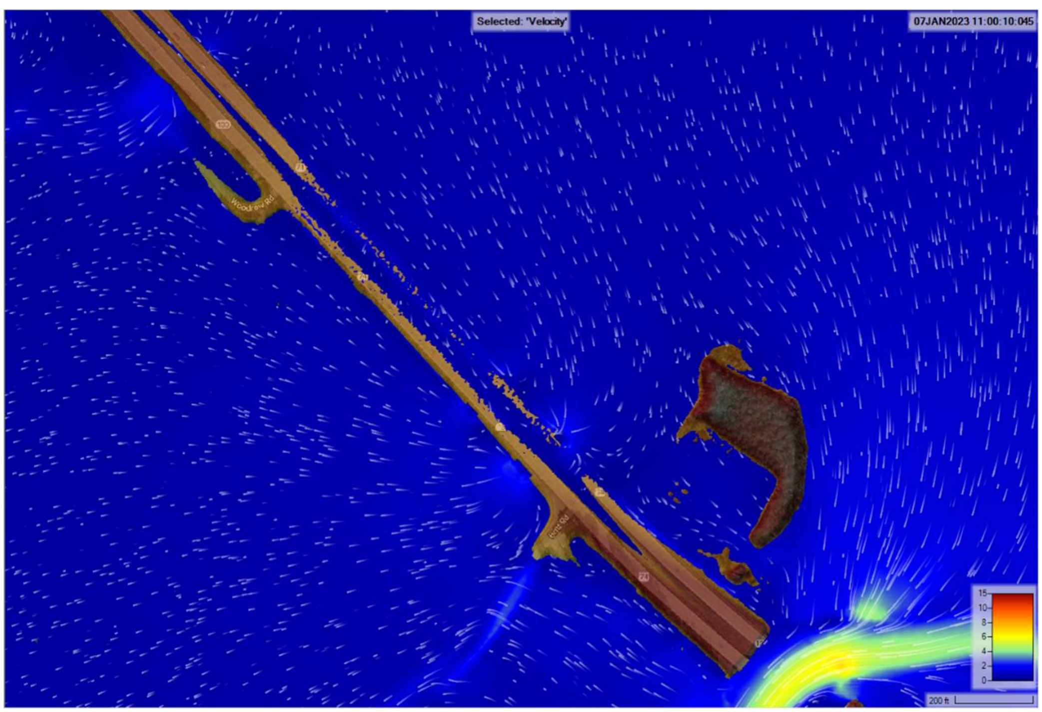


Figure 3.1.2.1C. Overtopping of Eastbound Lanes During 50-Year Event

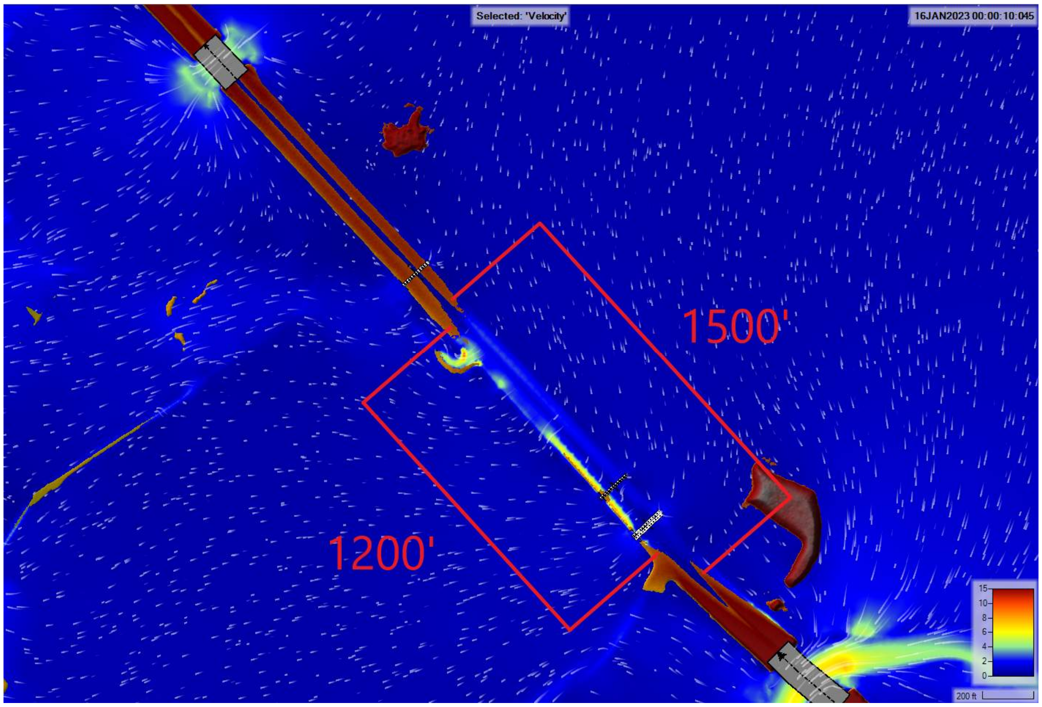


Figure 3.1.2.1D. 100-Year Overtopping Extents

Flow depth for overtopping in the 100-year event varies but reaches up to 2ft along the shoulder and downstream embankment (Figure 3.1.2.1E). Larger peaks are consistent with the locations of the box culverts within the overtopping zone and can be ignored. Figure 3.1.2.1F depicts an example cross section of the roadway within the overtopping zone, including water surface and velocity. Note that this section does not necessarily represent the actual low point. Shear stress along the downstream embankment, shown in Figure 3.1.2.1G, reaches a maximum of approximately 4.7 lb/ft^2. The largest peak can be found on the shoulder just southeast of Woodrow Road. The shear stress results map, shown in Figure 3.1.2.1H, highlights the hot spots at this location.

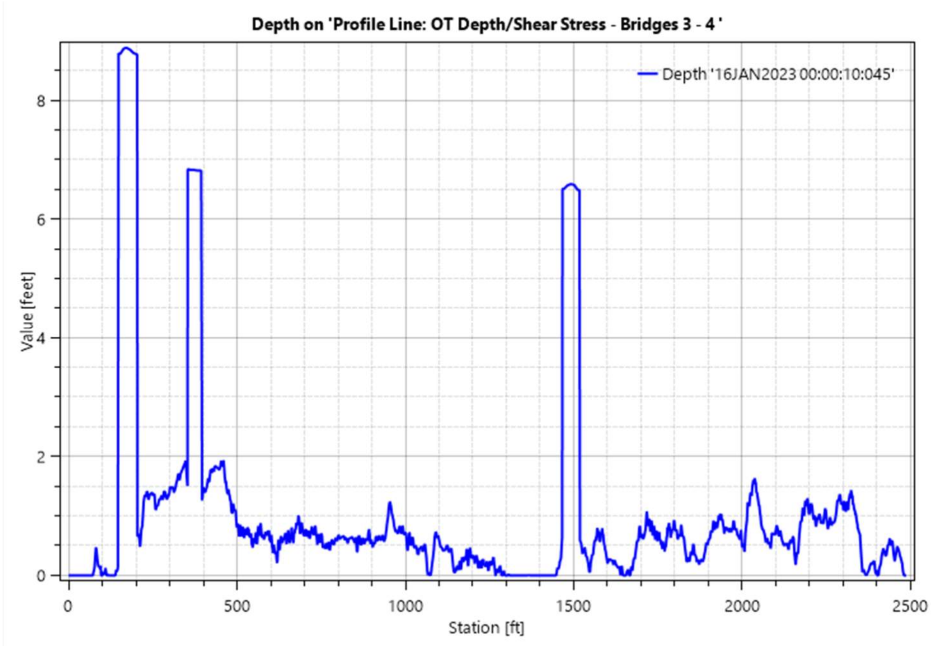


Figure 3.1.2.1E. 100-Year Overtopping Depth, Shoulder of EB Lanes

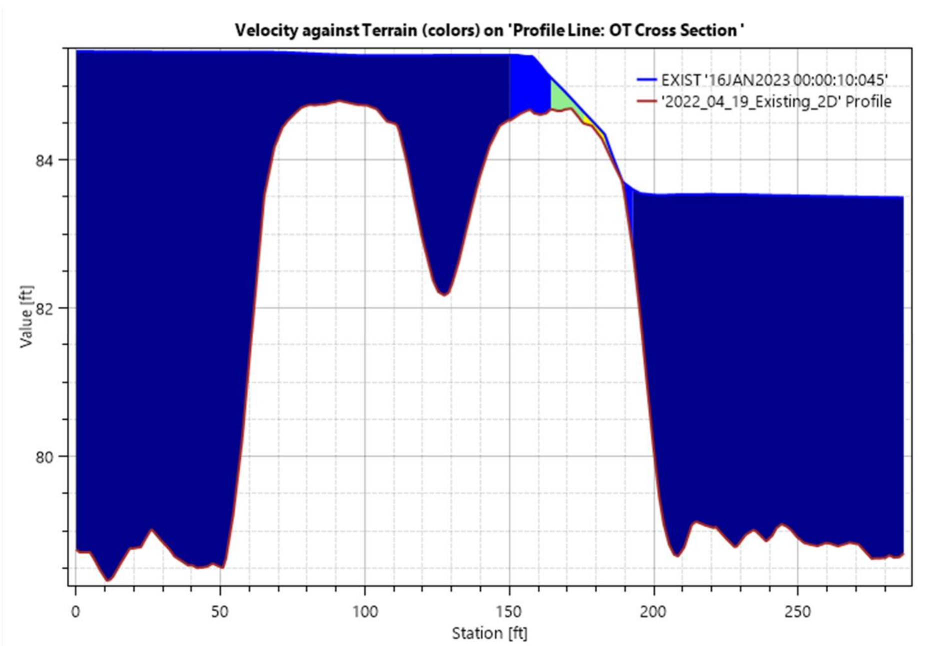


Figure 3.1.2.1F. Example Roadway Cross Section with 100-year Water Surface & Velocity

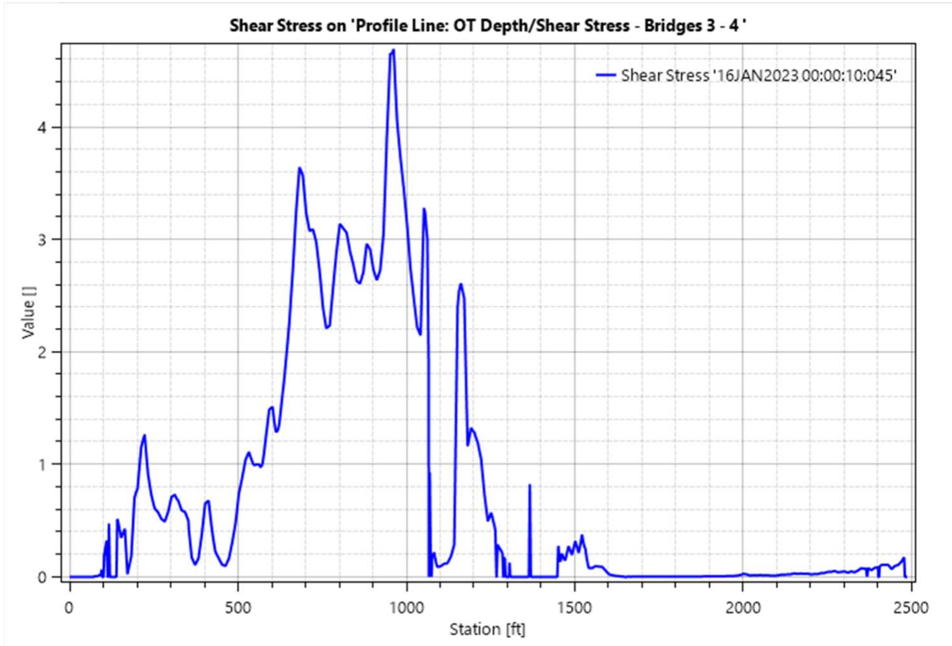


Figure 3.1.2.1G. 100-Year Overtopping Shear Stress, Shoulder of EB Lanes

Figure 3.1.2.1H. 100-Year Shear Stress, Woodrow Road

The 500-year event sees overtopping extend to an approximate length of 2500ft along the westbound lanes, covering almost the entire roadway between Bridge #3 and Bridge #4 (Figure 3.1.2.1I). Overtopping along the eastbound lanes does not reach as far towards Bridge #4, resulting in a total length of 1500ft. Figure 3.1.2.1J depicts overtopping depth along the downstream embankment, with the large peaks again correlating with the locations of box culverts between Bridge #3 and Bridge #4. Excluding these outliers, the profile line output shows that the overtopping depth reaches a maximum of roughly 3ft near Culvert #2. The roadway cross section in Figure 3.1.2.1K shows the approximate 1.5ft drop across the downstream shoulder and the increase in velocity due to weir flow across the eastbound lanes. The resulting shear stress generated along the shoulder peaks at about 6.3 lb/ft^2 (Figure 3.1.2.1L). The two largest peaks on the chart correlate with the shoulders of Woodrow Road, with the saddle between representing the actual road surface. The shear stress results map is included (Figure 3.1.2.1M) to provide a visual summary of the peaks in the vicinity of Woodrow Road.

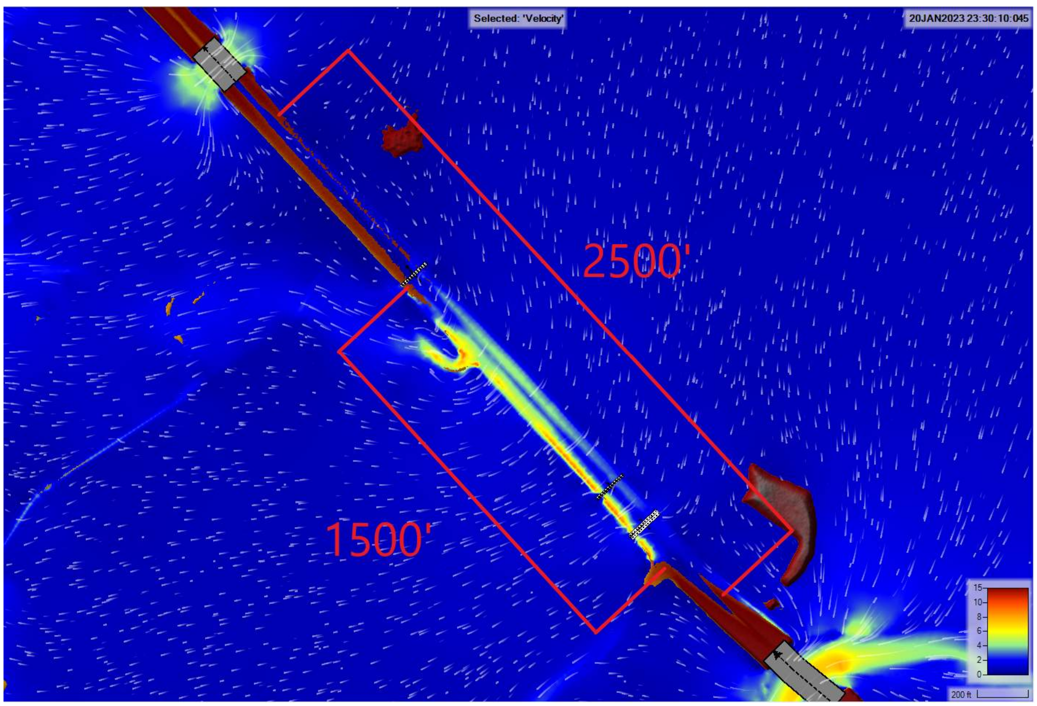


Figure 3.1.2.1I. 500-Year Overtopping & Velocities (Bridge #3 to Bridge #4)

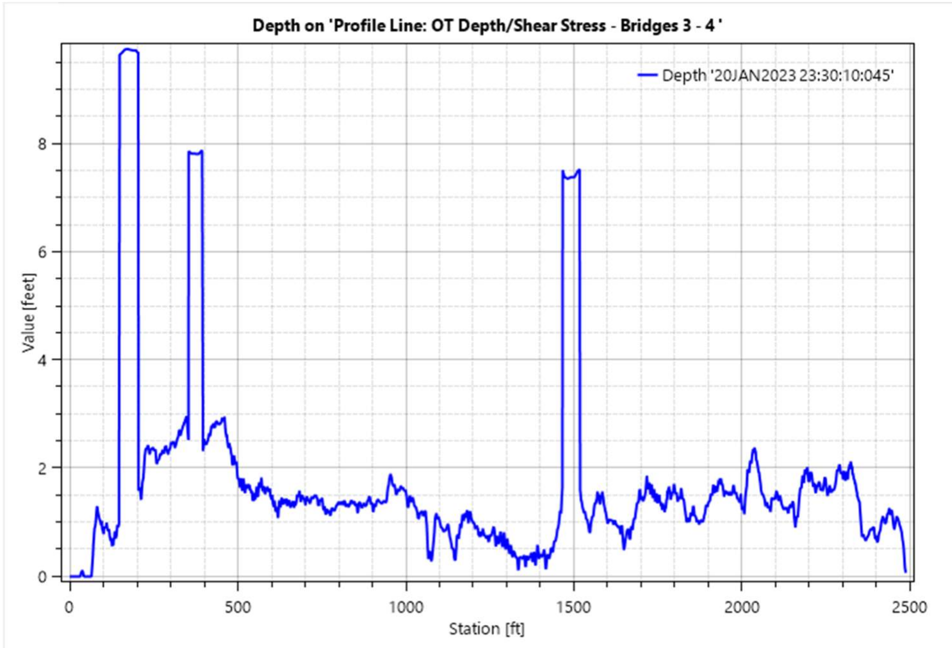


Figure 3.1.2.1J. 500-Year Overtopping Depth, Shoulder of EB Lanes (Bridge #3 to Bridge #4)

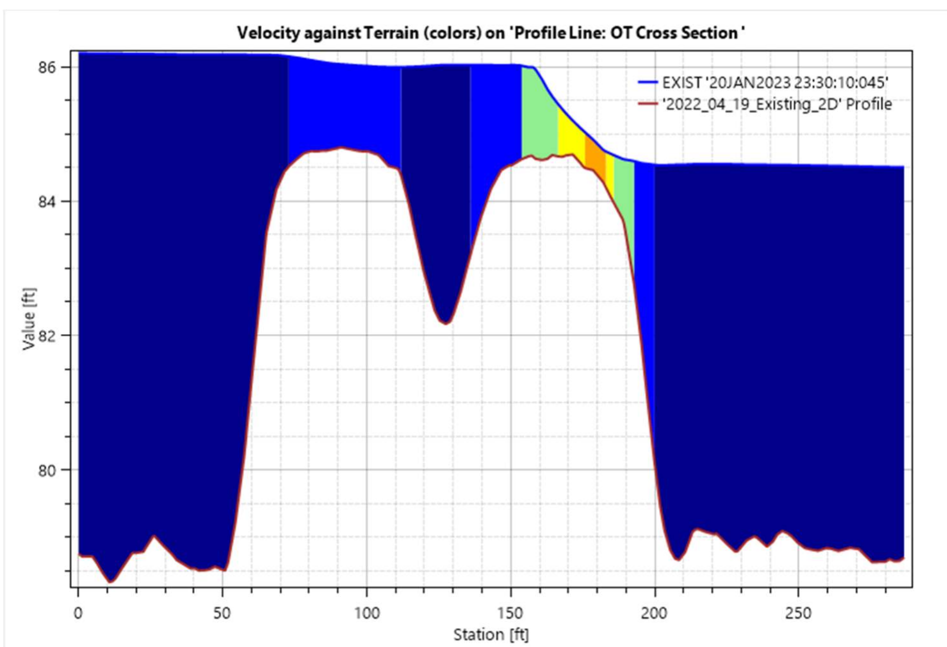


Figure 3.1.2.1K. 500-Year Water Surface & Terrain Example Across Overtopping Location

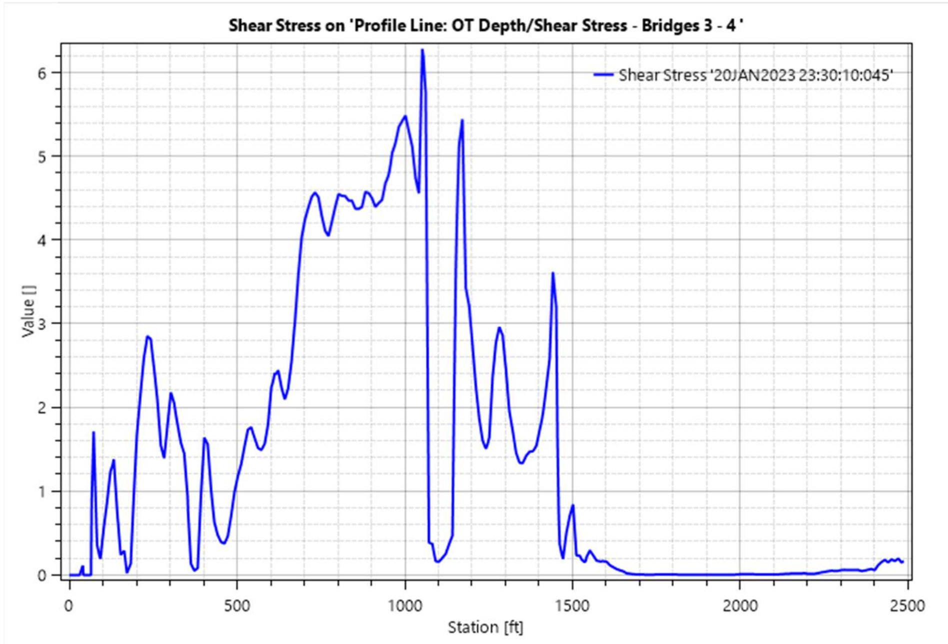


Figure 3.1.2.1L. 500-Year Overtopping Shear Stress, Shoulder of EB Lanes (Bridge #3 to Bridge #4)

Figure 3.1.2.1M. 500-Year Shear Stress, Woodrow Road

The northwest side of the floodplain between NC-130 and Creek Road also begins to overtop early in the 500-year event, roughly at the location of Pipe #6 (Figure 3.1.2.1N). Ultimately, the 500-year event produces shallow overtopping along much of the segment, covering nearly 7,000ft of the approximately 11,500ft of highway between the roads, as shown in Figure 3.1.2.1O. Most of this flow is only around 0.1-0.2ft in depth and slow moving; however, Figure 3.1.2.1P suggests that 1500ft of roadway between Pipe #5 and Pipe #7 overtops at a greater depth, up to 0.5ft, likely due to a slight local sag. While most of the crossing sees minimal shear stress, rarely exceeding 0.5 lb/ft^2, this section sees an average shear stress over 1 lb/ft^2 with peaks up to 5 lb/ft^2 (Figure 3.1.2.1Q). While many of the peaks correspond to pipe outlets, hot spots can also be seen in Figure 3.1.2.1R on the embankment adjacent to outlets.

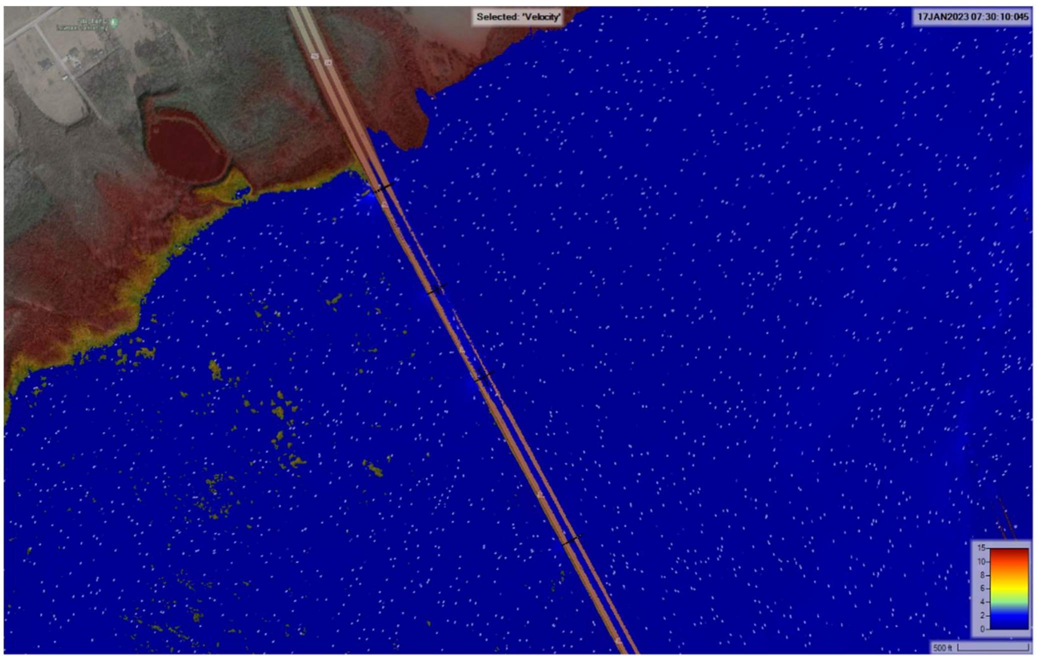


Figure 3.1.2.1N. 500-Year Initial Overtopping (NC-130 to Creek Rd.)

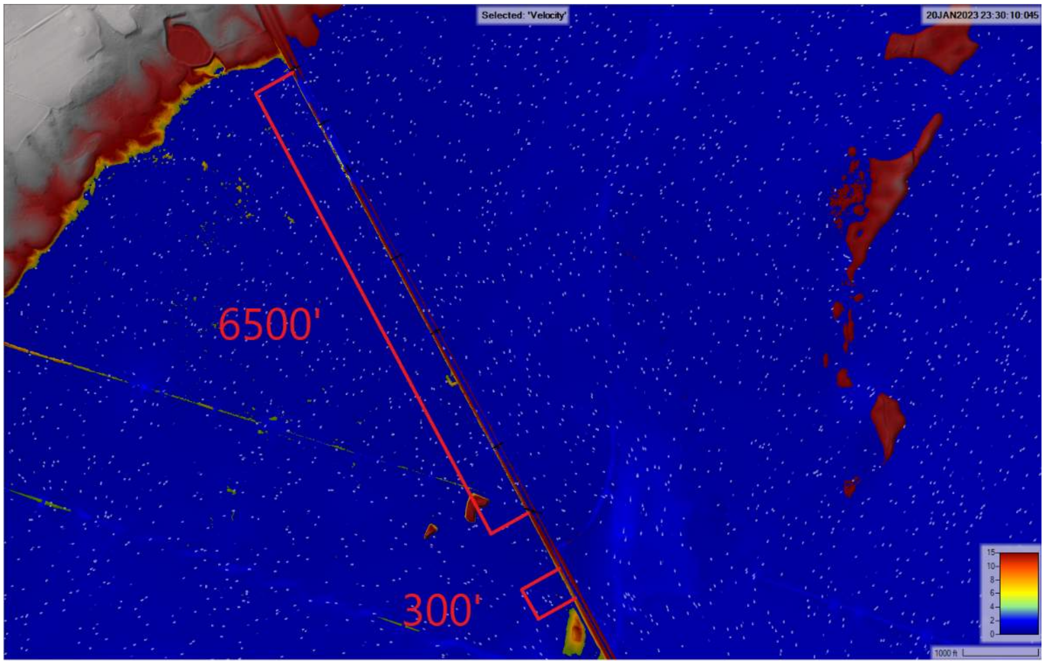


Figure 3.1.2.1O. 500-Year Overtopping & Velocities (NC-130 to Creek Rd.)

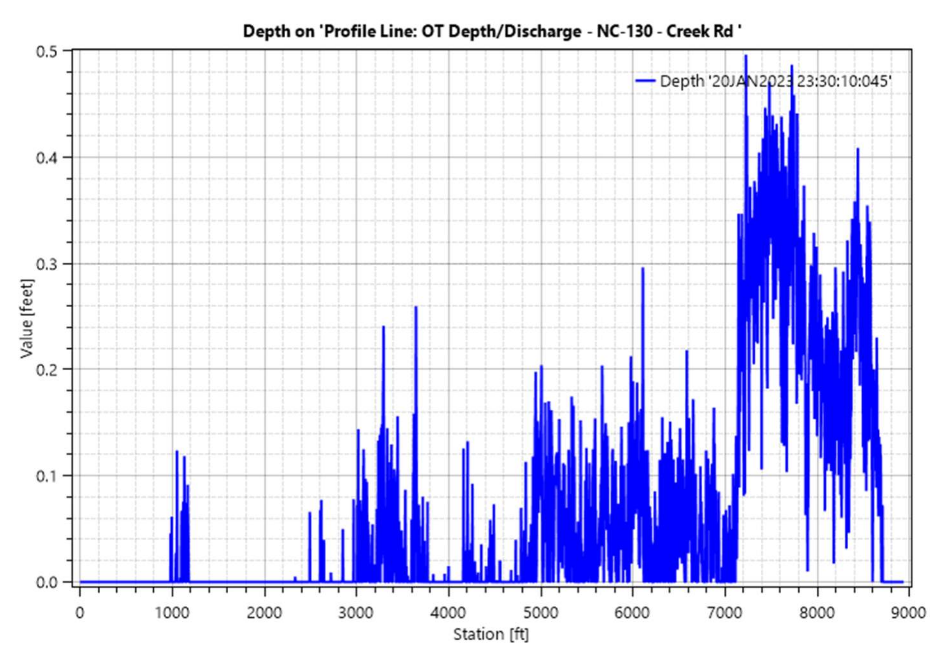


Figure 3.1.2.1P. 500-Year Overtopping Depth, EB Lanes (NC-130 to Creek Rd.)

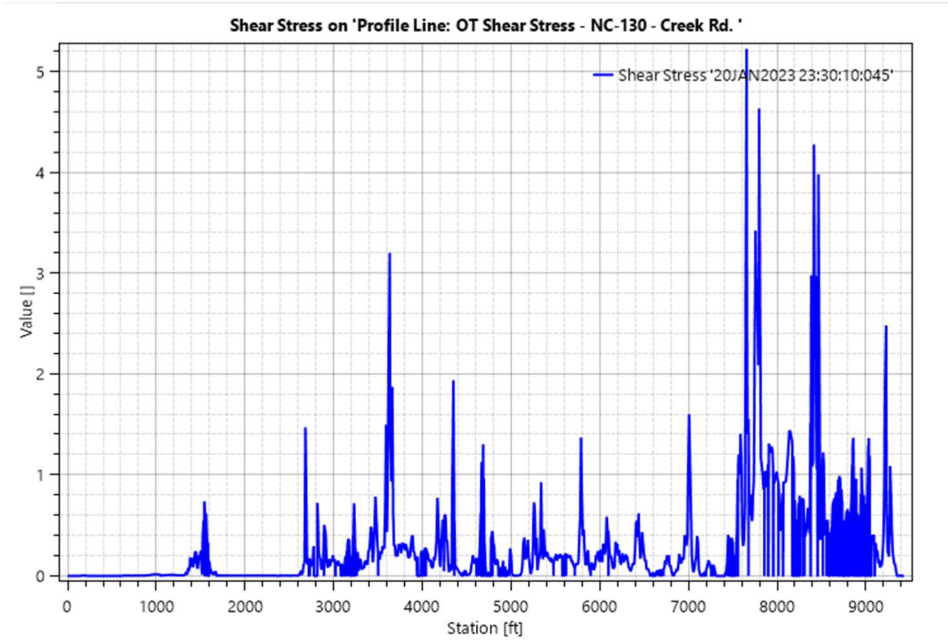


Figure 3.1.2.1Q. 500-Year Overtopping Shear Stress, Shoulder of EB Lanes (NC-130 to Creek Rd.)

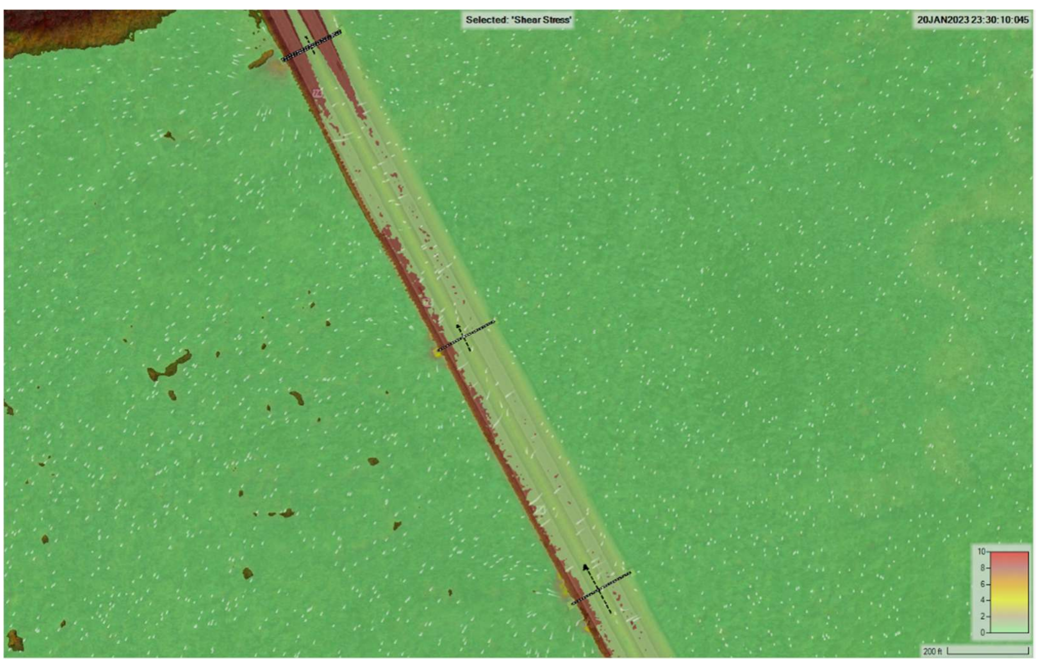


Figure 3.1.2.1R. 500-Year Shear Stress (Pipe #5 - Pipe #7)

Overtopping discharge between Bridge #3 and Bridge #4 is documented in Figure 3.1.2.1S. This area sees about 300 cfs over the road in the 50-year event, 1,400 cfs in the 100-year event, and 5,500 cfs in the 500-year event. The section on US-74 northwest of NC-130 only sees overtopping in the 500-year event, reaching a total discharge of roughly 370 cfs (Figure 3.1.2.1T). Therefore, the crossing in its entirety sees approximately 1,400 cfs of overtopping flow in the 100-year event and 5,900 cfs in the 500-year.

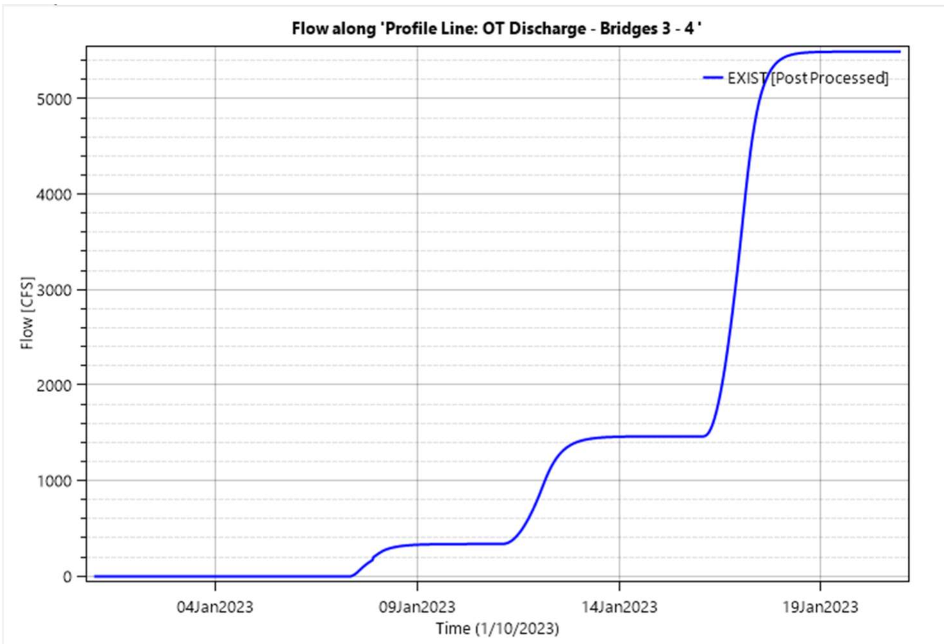


Figure 3.1.2.1S. Overtopping Flow Across EB Lanes (Bridge #3 to Bridge #4)

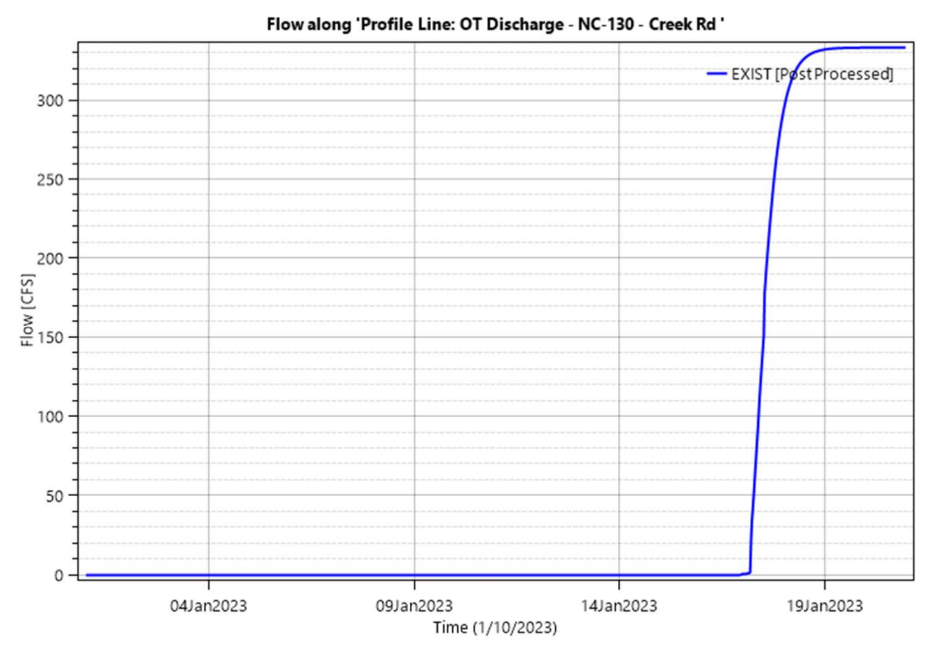


Figure 3.1.2.1T. Overtopping Flow Across Shoulder of EB Lanes (NC-130 to Creek Rd.)

3.1.2.2 Bridge Abutment Scour

The susceptibility of existing structures to scour was determined through evaluation of velocities, flow patterns, and shear stresses in the vicinity of structures for the 100-year and 500-year storms. Figures 3.1.2.2A and 3.1.2.2B, respectively, display the 100-year and 500-year velocity distribution in the vicinity of Bridges #1 through #3, while Figures 3.1.2.2E and 3.1.2.2F, respectively, display the 100-year and 500-year velocity distribution in the vicinity of Bridge #4 and Culvert #4. These velocity outputs show flow cutting past the upstream bridge corners for each bridge at around 5-6 fps, well above 2 fps, the limit for non-erosive velocity. Similar velocities can be seen at the downstream bridge corners for Bridge #4.

Figures 3.1.2.2C and 3.1.2.2D show shear stresses at Bridges #1-3 in the 100-year and 500-year events, respectively, while Figures 3.1.2.2G and 3.1.2.2H show the same for Bridge #4. These outputs suggest that shear stresses in proximity of the bridges reach a maximum of approximately 5 lb/ft^2, primarily at the transition from excavation to natural ground under the flood bridges. This wraps around the limit of excavation towards the bridge abutments.

These areas would be vulnerable to scour in large events, as supported by a record of abutment damage and blowout following hurricanes Matthew and Florence. This problem is exacerbated by the limited amount of combined flow area at this crossing in relation to the size of the floodplain, which forces water laterally along the upstream embankment towards the main channel bridges. Once this large amount of lateral flow hits a structure, it speeds up and turns hard, cutting into the bridge corners and scouring out the abutments.

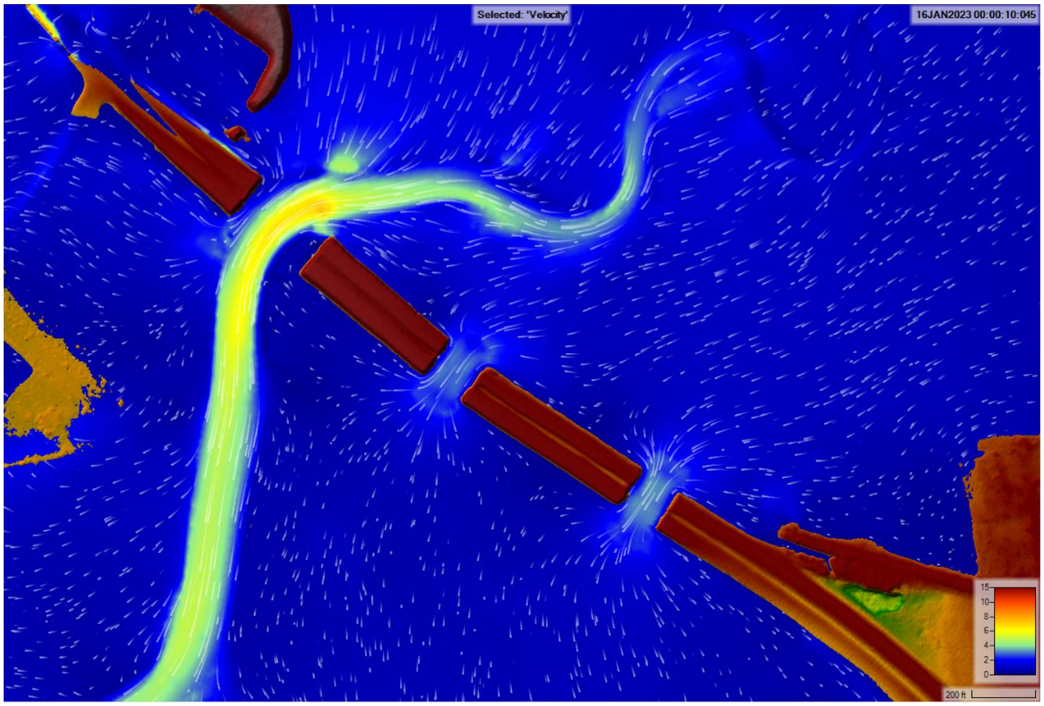


Figure 3.1.2.2A Bridges #1-3 100-Year Velocity

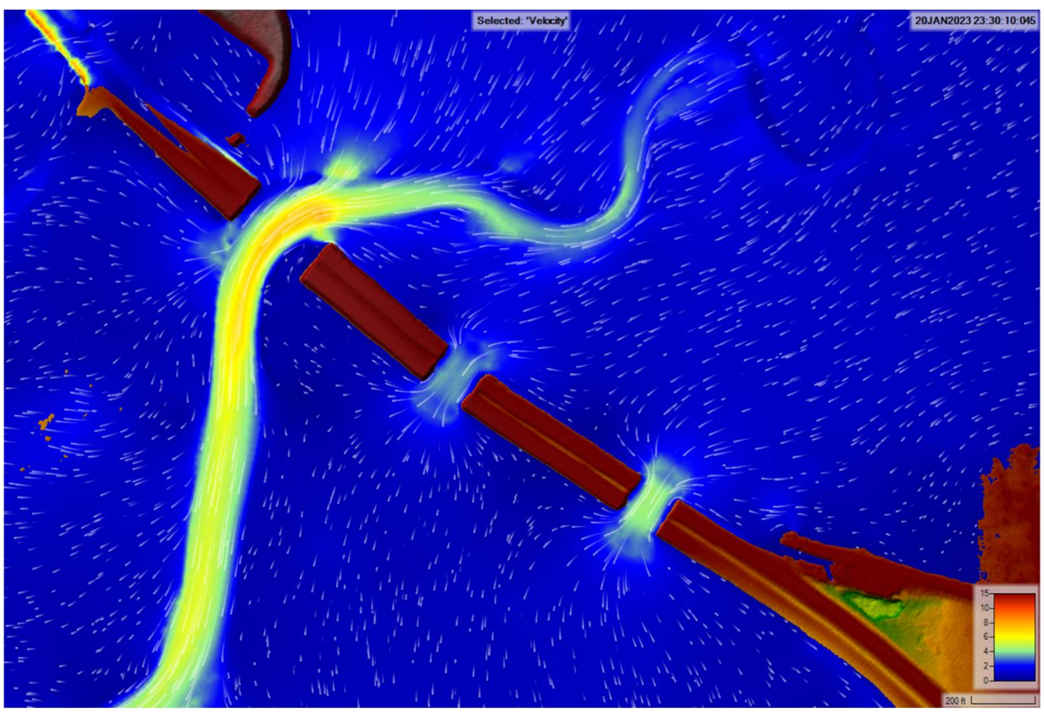


Figure 3.1.2.2B Bridges #1-3 500-Year Velocity

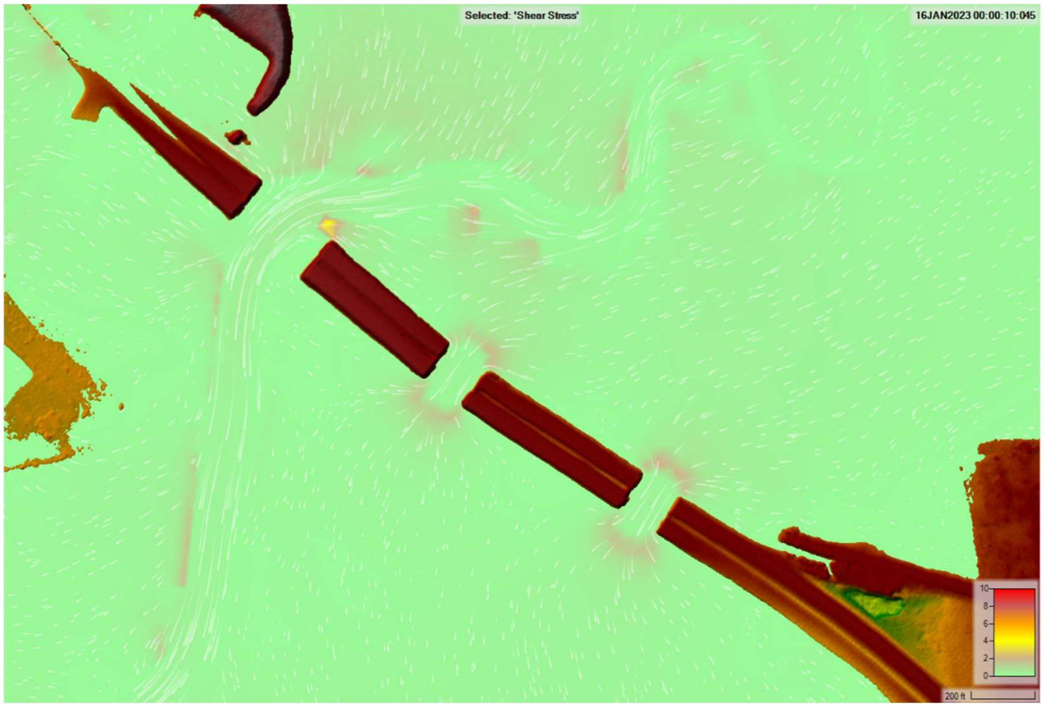


Figure 3.1.2.2C Bridges #1-3 100-Year Shear Stress

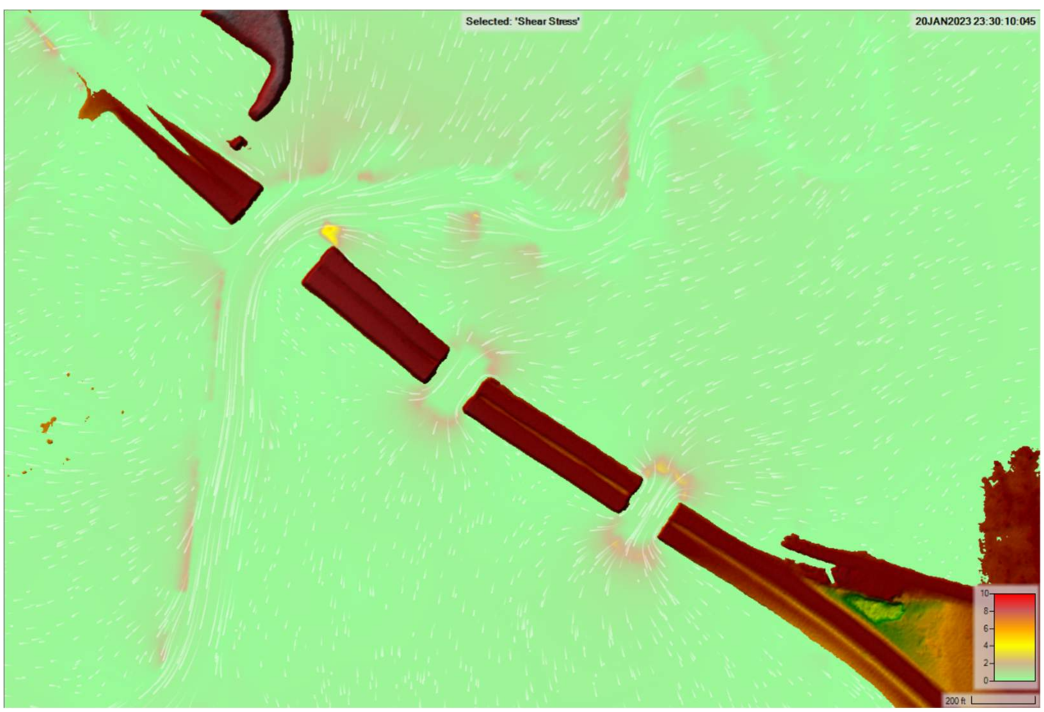


Figure 3.1.2.2D Bridges #1-3 500-Year Shear Stress

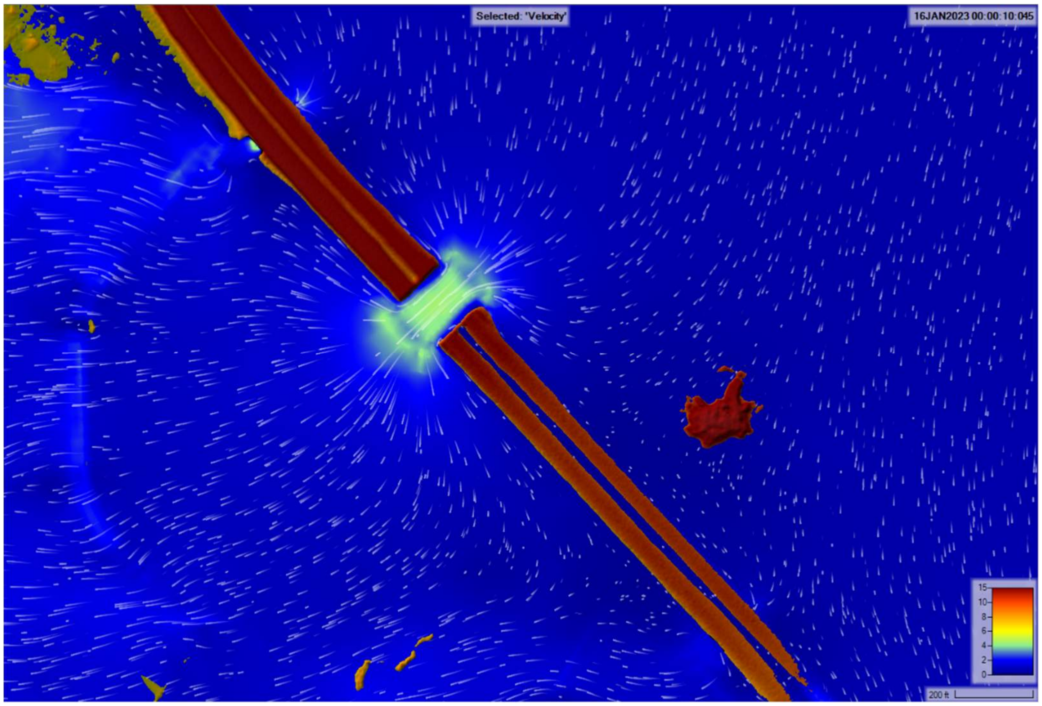


Figure 3.1.2.2E Bridge #4 & Culvert #4 100-Year Velocity

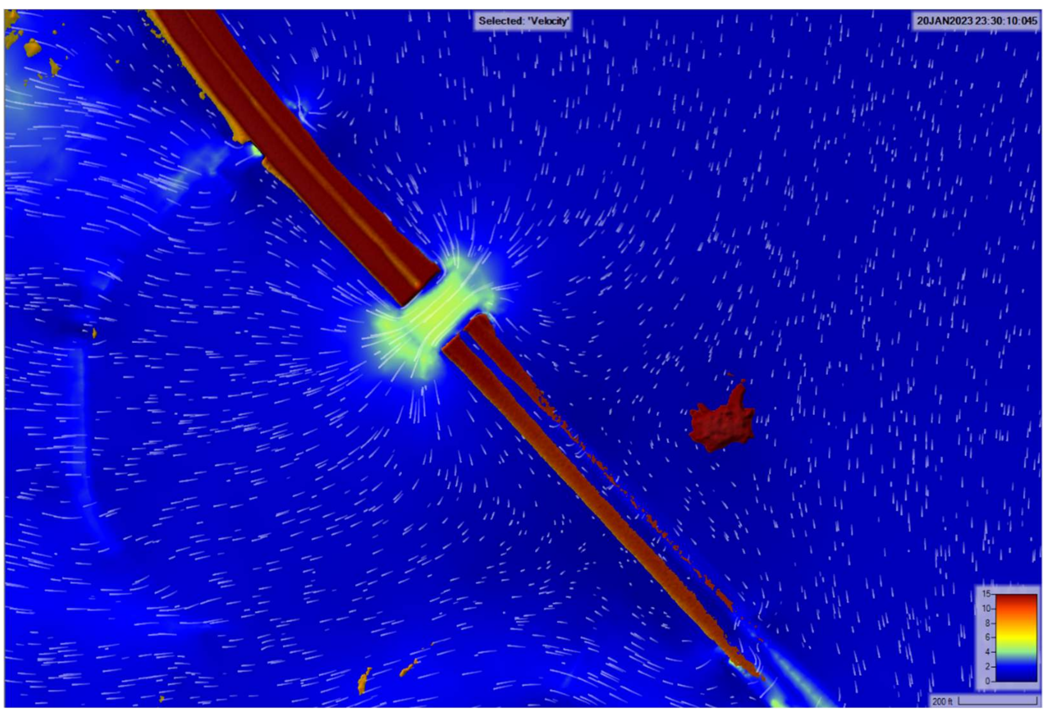


Figure 3.1.2.2F Bridge #4 & Culvert #4 500-Year Velocity

Figure 3.1.2.2G Bridge #4 & Culvert #4 100-Year Shear Stress

Figure 3.1.2.2H Bridge #4 & Culvert #4 500-Year Shear Stress

3.1.3 HECRAS 2D Model vs 1D FEMA

As stated in Section 2.1 FEMA Floodplain, the Lumber River Crossing is in a Zone AE floodplain with a regulatory floodway. RK&K reviewed the available FEMA model and compared the model to the values developed in the approved FEMA 1D Model. Figure 3.1.3 shows the 1D cross sections overlaid on the results of the HEC-RAS 2D model. At cross section 217017, the 100-year Elevation and 500-Year elevation is 82.74ft and 83.84ft, respectively. These elevations do not show the Lumber River crossing as overtopping, which contradicts both the HEC-RAS 2D model and flooding seen in past events. This highlights the presence of inherent inaccuracies in the 1-D modeling process, which are heavily dependent on the frequency of cross sections along a reach and the variability of the floodplain between reaches. The 2D modeling process has a significant advantage in that the full terrain of the floodplain is available to the program, facilitating the collection of flood information at any one point within the 2D mesh. Therefore, HEC-RAS 2D can model the flow across the actual road surface, rather than interpolating between sections using bridge geometry inputs. This results in a far more accurate picture of the actual conditions in a floodplain and a better idea of how changes to a crossing will affect the conveyance through it.

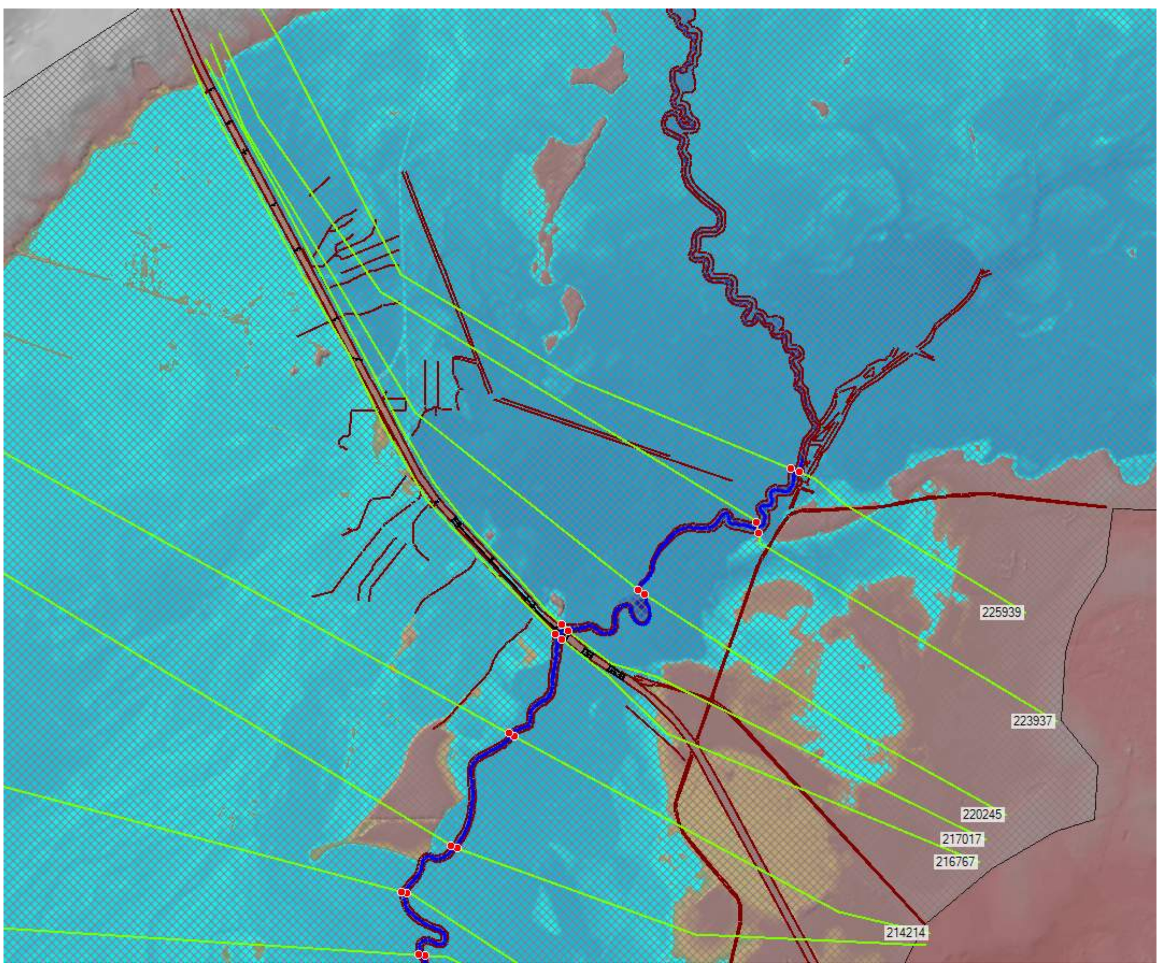


Figure 3.1.3 FEMA 1D Cross Sections

4 PROPOSED ALTERNATIVES

Based on historical damage and results of the existing model described above, RK&K has developed three mitigation alternatives, which vary in goal and complexity. These alternatives include reinforcement of the existing shoulder to limit damage from overtopping, installation of guide banks at bridges to protect against abutment scour, and raising grade and adding a structure to prevent overtopping and increase conveyance. NCDOT's main concerns are maintaining traffic flow and minimizing repair costs along the crossing. The following sections describe these three alternatives in more detail and how they improve the existing issues.

4.1 Shoulder Strengthening

The first alternative explored attempts to rectify shoulder damage due to overtopping while maintaining the grade of the existing roadway. Shoulder strengthening entails incorporation of either rock or a commercially-available erosion control product into the existing shoulder to anchor the soil and resist shear stresses from overtopping flow, which tend to promote and propagate scour across unprotected surfaces. Erosion control product types considered include Articulated Concrete Block Mattresses (ACBM) and Turf Reinforcement Matting (TRM). Given the significant amount of overtopping flow and the variability of contributing factors along the crossing, preference is given towards High Performance Turf Reinforcement Matting (HPTRM) over standard TRM. The 100-year maximum shear stress (4.7 lb/ft^2) and velocity (10.0 fps) values seen in the existing model were used as the minimum criteria for commercially-available erosion control products considered. A list of products was compiled and re-evaluated against the 500-year maximum shear stress (6.3 lb/ft^2) and velocity (10.3 fps). All products on the list, with the exception of Enkamat 7010, were found to meet or exceed these higher limits. Note that maximum velocity and sheer stress exceed design limits for Class II riprap, but it has been included for comparison. Aside from Class II riprap, the options should not present a hazard to motorists. Figures 4.1A and 4.1B show examples of shoulder reinforcement using ACBM and HPTRM, respectively. NCDOT has recommended use of Roadway Standard Drawing 275.01 (Rock Plating) as a starting point for development of a detail for shoulder strengthening. This standard drawing is included in Appendix C.

TABLE 4A: SUMMARY OF REINFORCEMENT OPTIONS

Option	Type	T_{max}	V_{max}
Class II Riprap	Rock	4.8	8
Flexamat	ACBM	24	30
Earthlok	ACBM	24	19
ShoreFlex	ACBM	18	30
Pyramat 75	HPTRM	16	25
RollMax TMax	HPTRM	16	25
T-RECS	HPTRM	15	25
Enkamat 7010	TRM	6	14

Figure 4.1A. Flexamat ACBM Installation Along Roadway Shoulder Prone to Overtopping
Credit: Flexamat/Coleman Moore Company

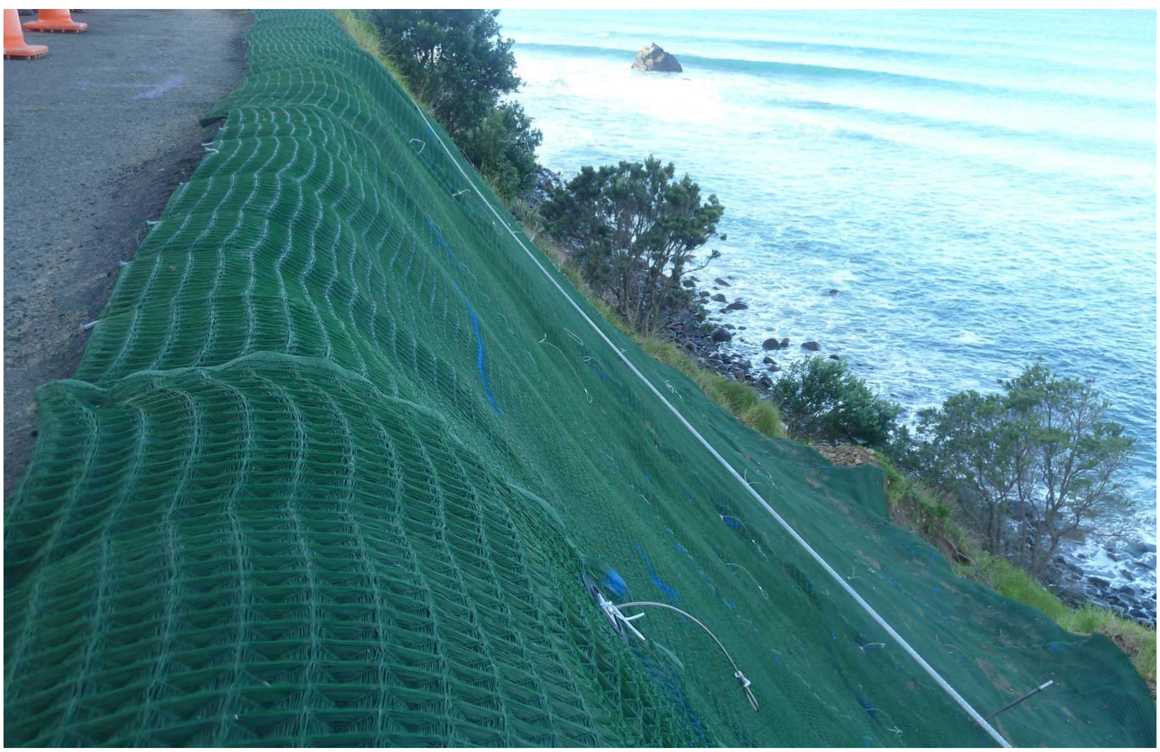


Figure 4.1B. Pyramat HPTRM Shoulder/Embankment Installation
Credit: Cirtex Civil

4.2 Guide Banks

An additional 2D plan was created to assess the potential benefits of guide banks to mitigate scour on bridge abutments and along the upstream embankment. This plan was generated from the Existing model, with terrain modifications and n-value overrides used to represent guide banks on the upstream side of all bridge crossings. A set was also modeled on the downstream side of Bridge #4, as the record of past abutment scour and velocities/flow patterns from the existing model suggest would be beneficial. Guide banks were designed based off guidance from HEC-23 and feature the following characteristics: 10ft top width, 2:1 side slopes, and constructed of Class II riprap (n-value 0.065). Top elevation was set at 86.5ft, approximately 2ft above the existing 100-yr WSEL at the main channel bridge (Bridge #3). The guide banks extend 50ft upstream/downstream from the bridge, and 20ft laterally away from the channel/bridge opening.

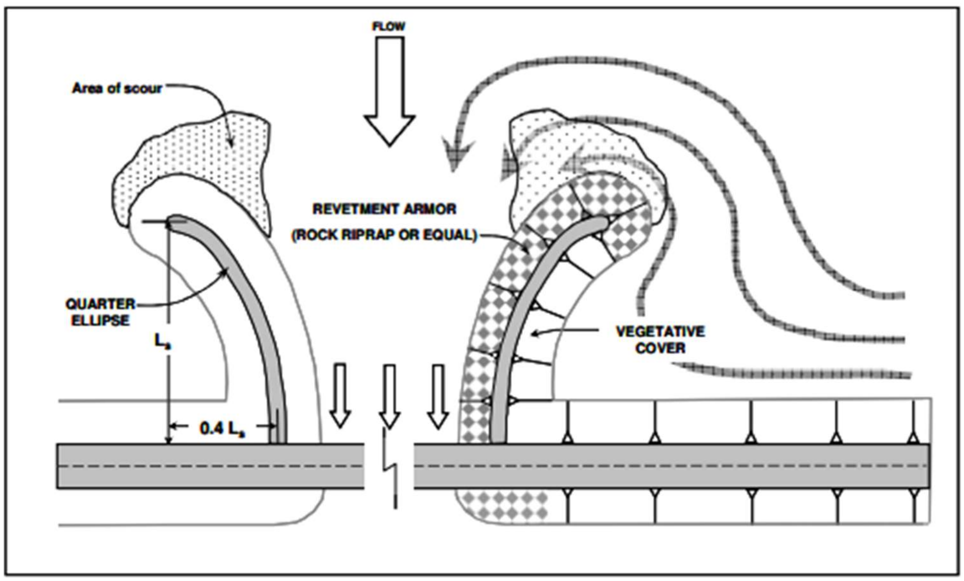


Figure 4.2A. Typical Guide Bank Example (HEC 23)

Based on velocities and flow patterns seen with the 100-yr and 500-yr discharges, guide banks could be effective at preventing scour at abutments and adjacent road embankments at the upstream side of all bridges along the crossing. As discussed in section 3.1.1.2, the entire upstream embankment between Boardman and NC-72 experiences significant lateral flow, primarily towards the main channel crossing (Bridge #3). This behavior can be seen in Figures 4.2B and 4.2D, especially at the upstream right abutment/bridge corner, where lateral flow is producing increased velocities along the embankment. As this accelerated flow reaches the bridge corner, it tends to cut into and scour out the abutment. The curvature of the main channel also promotes sustained higher velocities at the upstream left abutment/bridge corner, as seen in the velocity color ramp. Figures 4.2C and 4.2E show how upstream guide banks force this erosive flow off of the bridge corners and into the channel proper, concentrating flow and shear stress at the end of the guide bank rather than along the embankment, reducing the potential for scour to propagate along the bridge abutments.

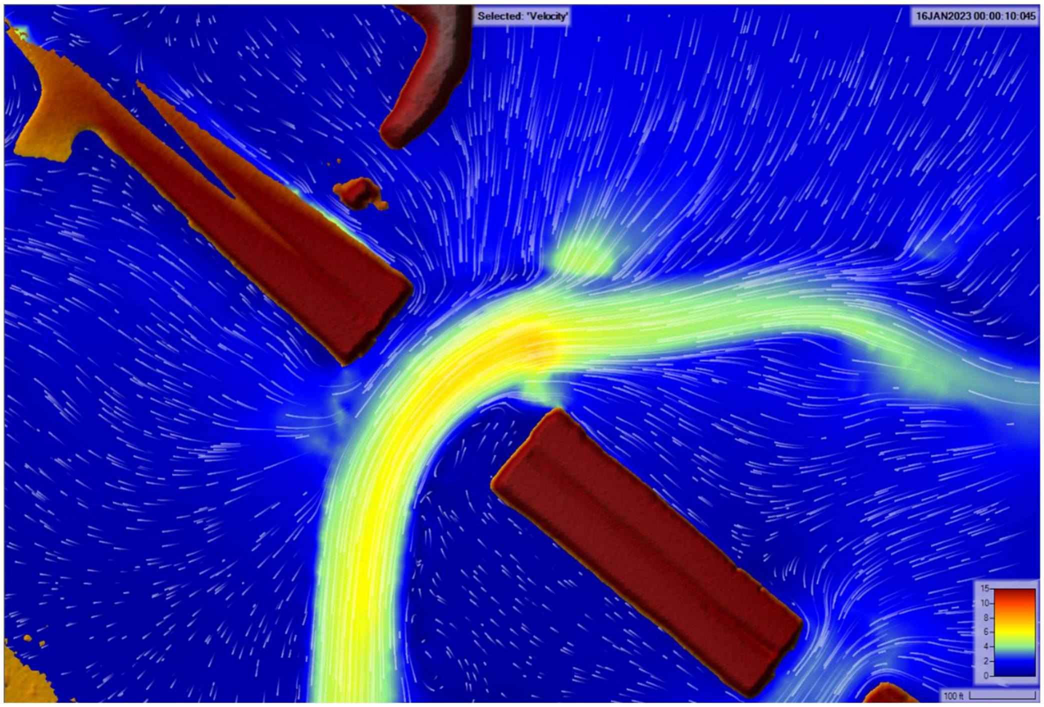


Figure 4.2B. Bridge #3 Velocities and Flow Patterns (100-Yr), Existing Conditions

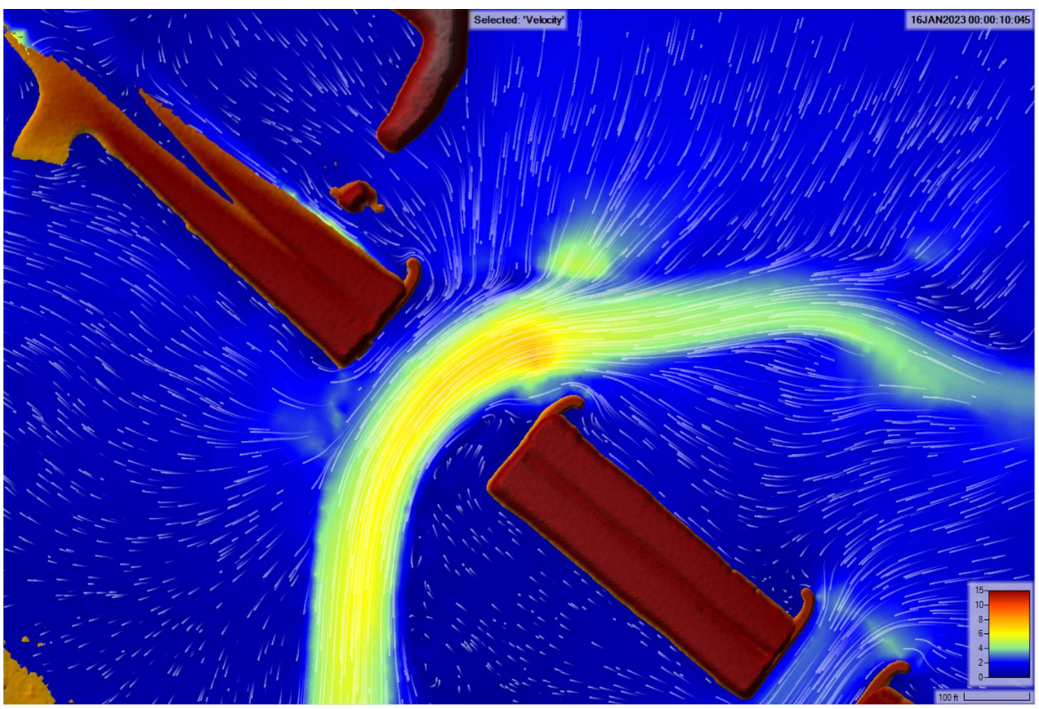


Figure 4.2C. Bridge #3 Velocities and Flow Patterns (100-Yr) with Guide Banks

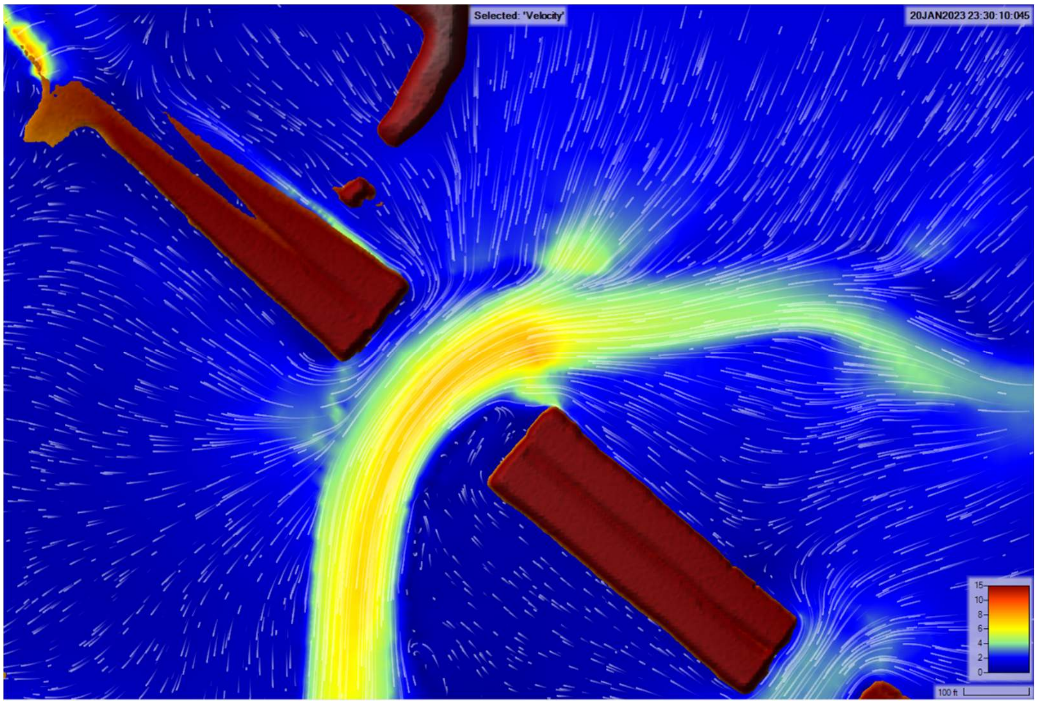


Figure 4.2D. Bridge #3 Velocities and Flow Patterns (500-Yr), Existing Conditions

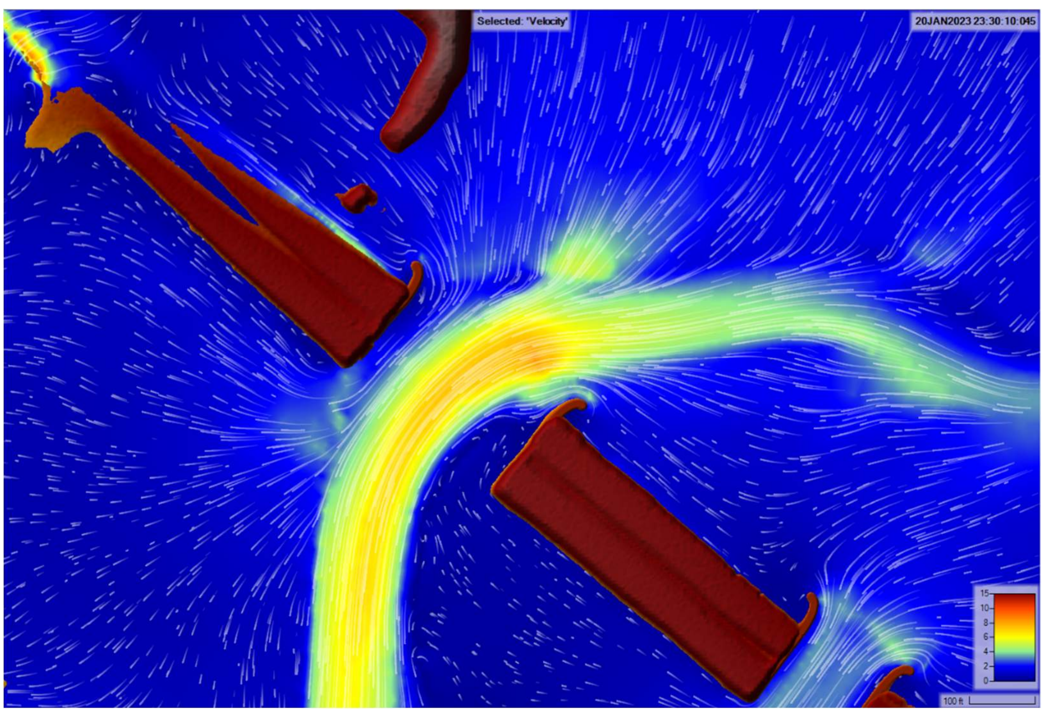


Figure 4.2E. Bridge #3 Velocities and Flow Patterns (500-Yr) with Guide Banks

Guide banks can also be useful on the downstream side of a bridge to protect the abutments from expanding flow. A notable example is Bridge #4, where greater velocities can be seen on the downstream side, as shown in Figures 4.2F and 4.2J. Discharge from this bridge in particular spreads quickly in the downstream floodplain, forcing flow against the embankment to either side. This generates increased shear stress at the bridge corners, as seen in Figures 4.2H and 4.2L, which can undermine the abutments. Guide banks here would effectively perform the same function as on the upstream side, but for expanding flow rather than contracting. As can be seen in Figures 4.2G and 4.2K, they contain the flow and limit expansion until well beyond the abutments and embankment. Erosive flow is concentrated against the stable riprap of the guide banks rather than the susceptible bridge corners (Figures 4.2I and 4.2M). Table 4B below compares velocities along all bridge abutments in existing conditions and with incorporation of guide banks. Some minor increases can be attributed to slight constriction of the bridge opening with guide banks. Significant decreases in velocity can be seen along the abutments for Bridge #3 and Bridge #4.

Table 4B: Comparison of Velocities on Bridge Abutments, Existing vs. Guide Banks (fps)

Plan	Event	Bridge #1		Bridge #2		Bridge #3		Bridge #4	
		LT	RT	LT	RT	LT	RT	LT	RT
Existing	100	2.40	2.78	2.43	2.43	3.60	2.10	3.12	3.22
	500	3.02	3.39	3.18	2.77	4.35	2.86	3.40	3.57
Guide Banks	100	2.85	2.68	2.39	2.24	1.51	2.08	2.04	2.16
	500	3.61	3.49	2.76	2.67	1.85	2.76	2.46	2.60

Figures 4.2N through 4.2Q show the 500-year shear stress along the left and right abutments of Bridge #4 in existing conditions and with the incorporation of guide banks. The existing roadway falls roughly between 50ft and 200ft for each figure. These results show that guide banks are very effective at reducing shear stress at the bridge corners to practically zero and pushing erosive flow away from the bridge abutments. Further improvement and reduction in shear stress could be achieved by grading a smoother transition between the excavation under the bridge and natural ground on the upstream and downstream sides.

Additional results for this alternative at Bridges #1 and #2 can be found in Appendix 3.

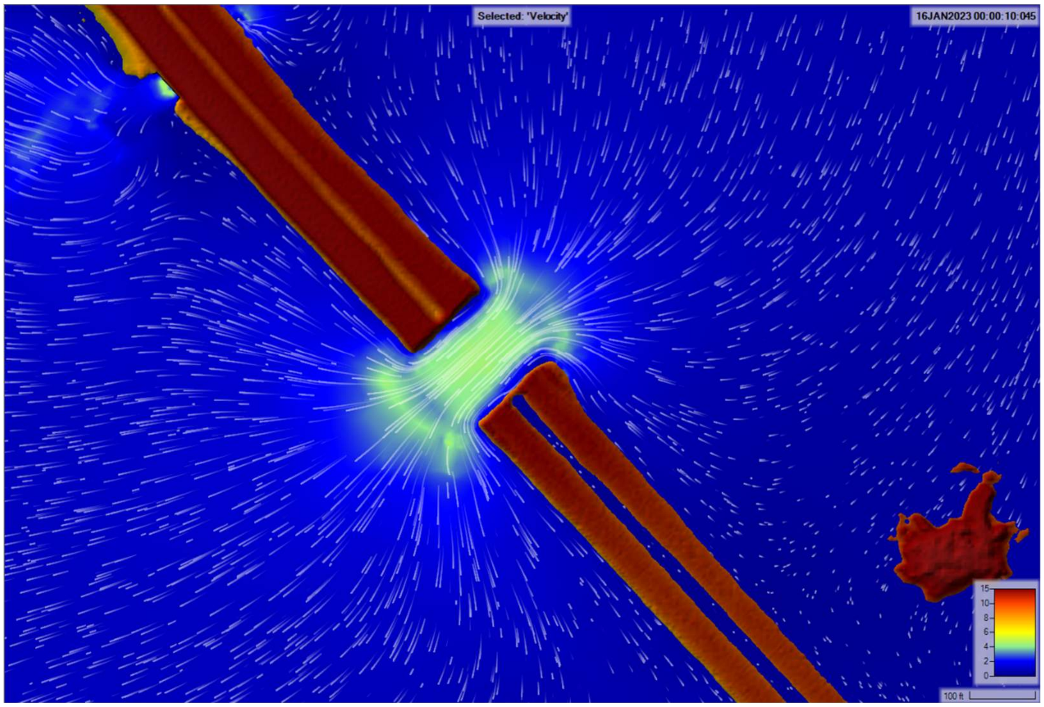


Figure 4.2F. Bridge #4 Velocities and Flow Patterns (100-Yr), Existing Conditions

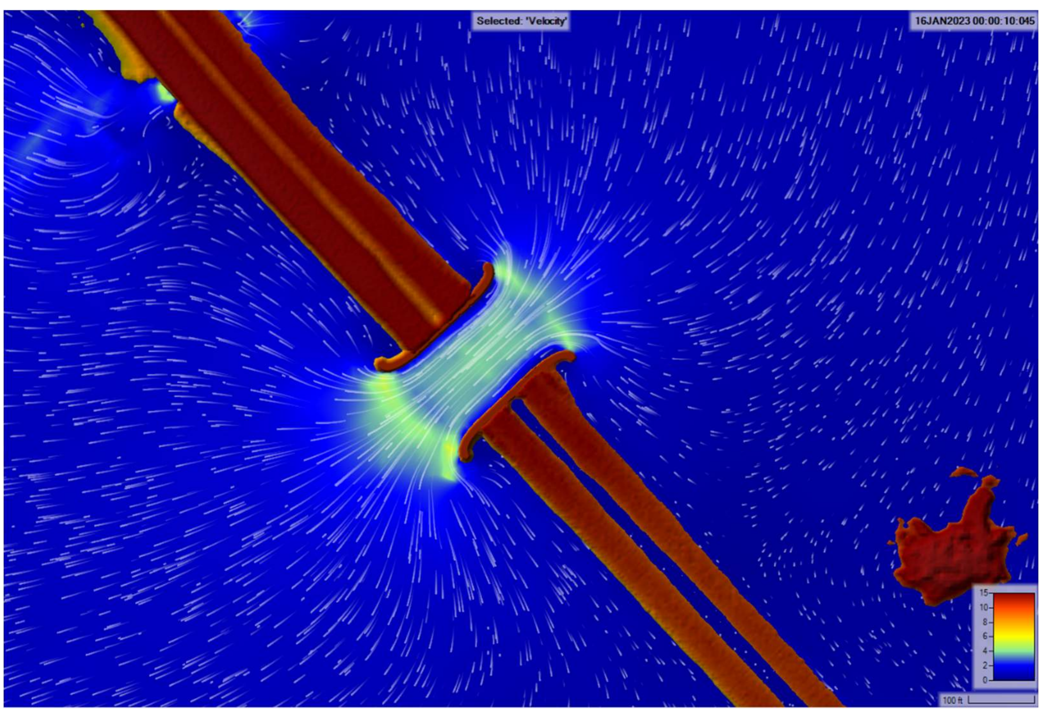


Figure 4.2G. Bridge #4 Velocities and Flow Patterns (100-Yr), Guide Banks

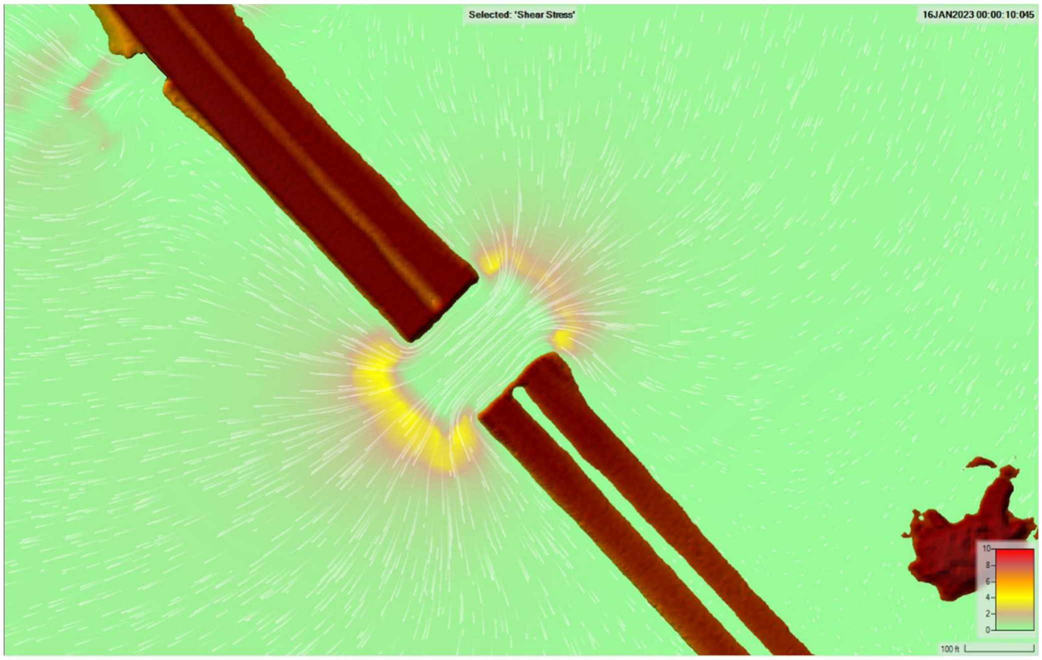


Figure 4.2H. Bridge #4 Shear Stress (100-Yr), Existing Conditions

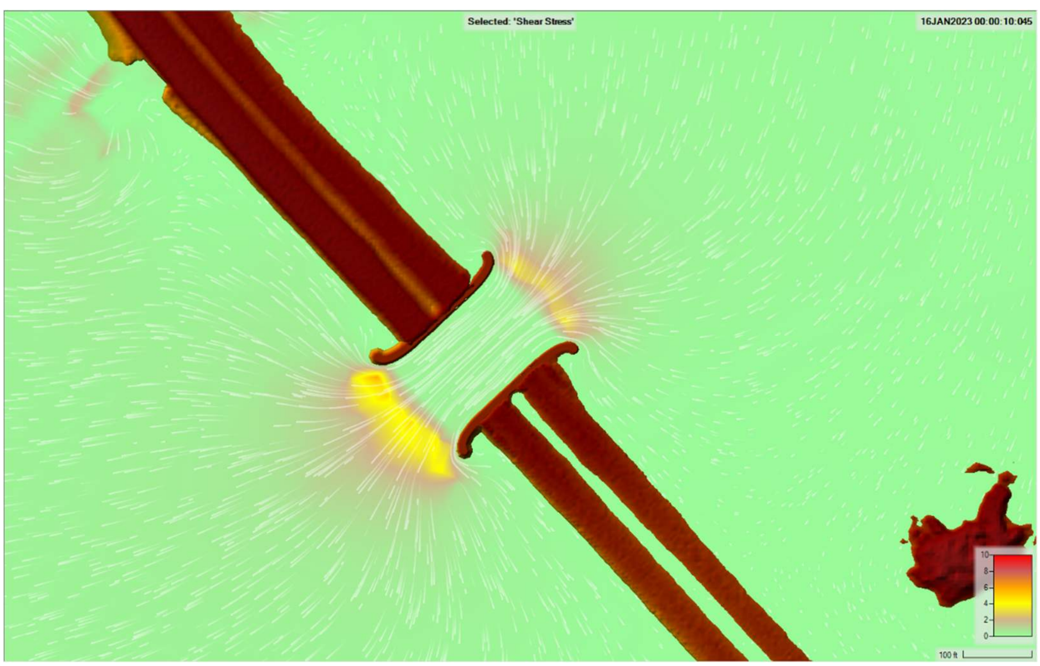


Figure 4.2I. Bridge #4 Shear Stress (100-Yr), Guide Banks

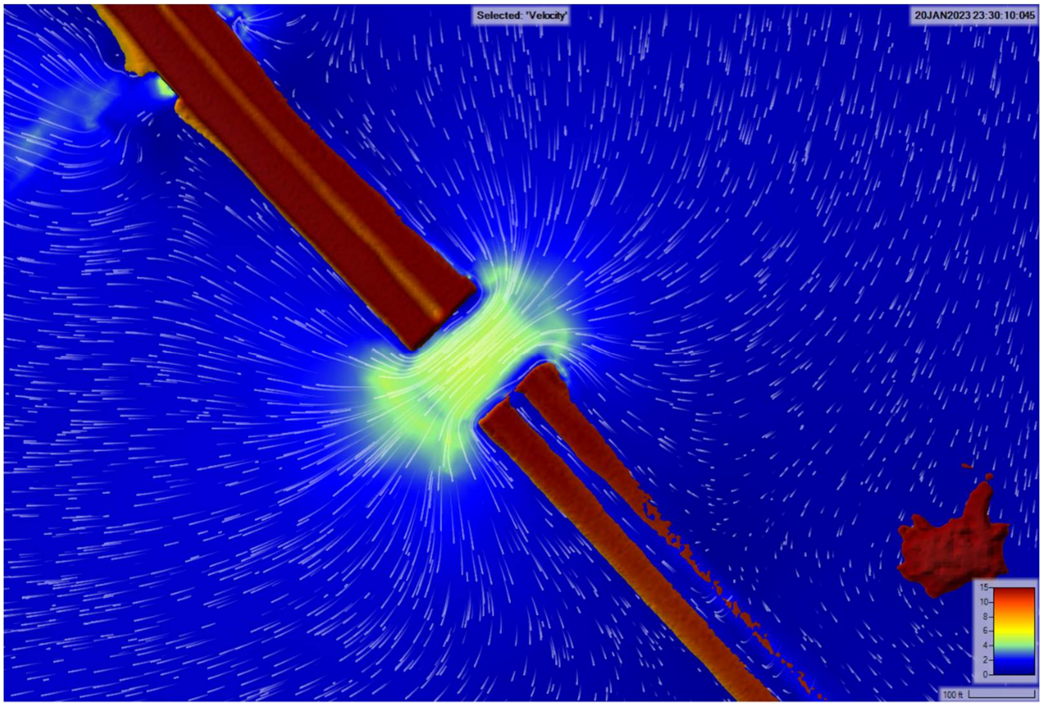


Figure 4.2J. Bridge #4 Velocities and Flow Patterns (500-Yr), Existing Conditions

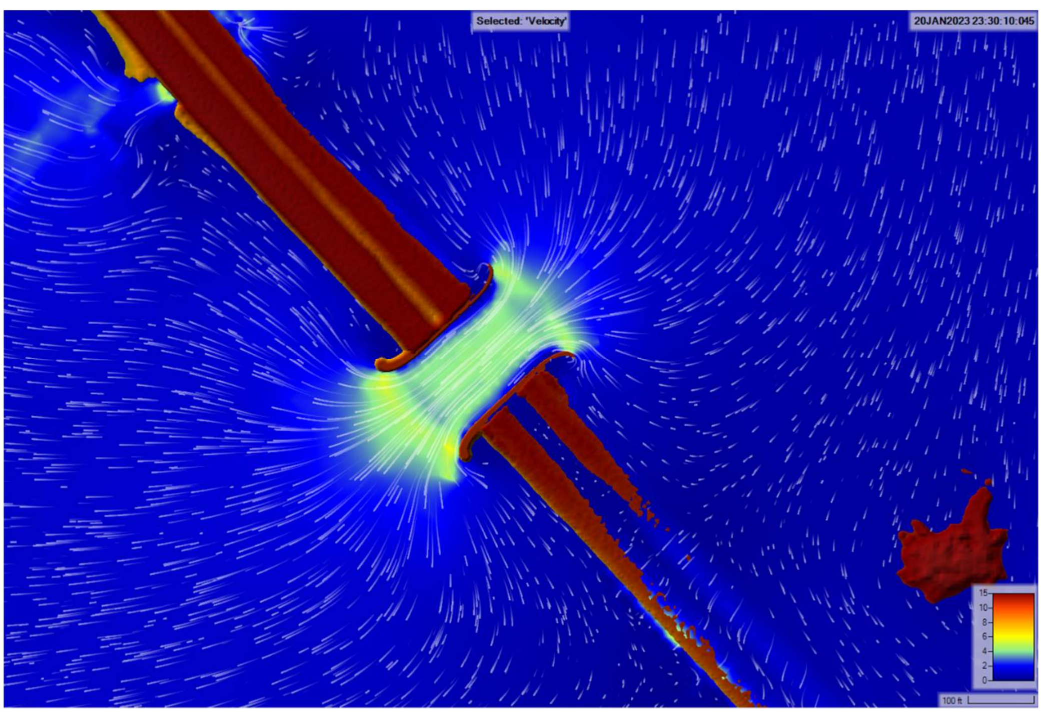


Figure 4.2K. Bridge #4 Velocities and Flow Patterns (500-Yr), Guide Banks

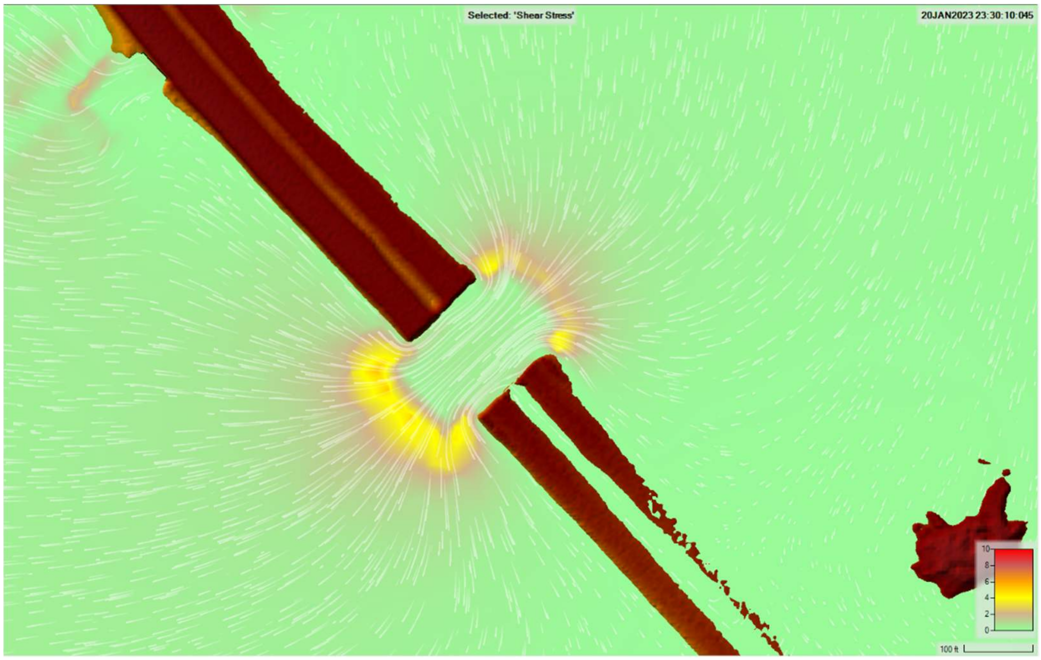


Figure 4.2L. Bridge #4 Shear Stress (500-Yr), Existing Conditions

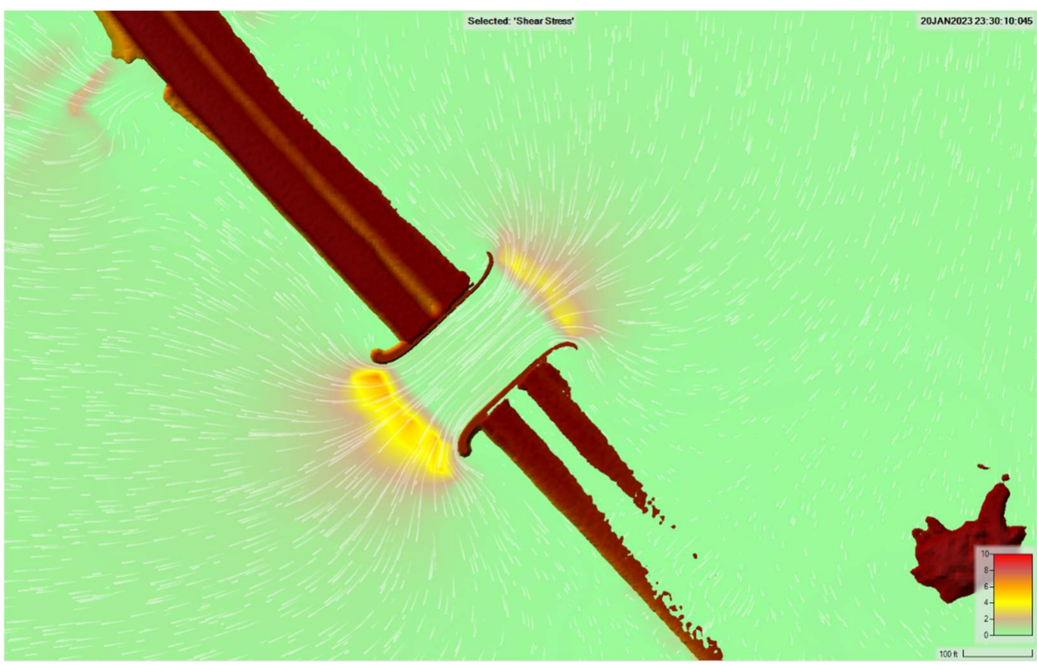


Figure 4.2M. Bridge #4 Shear Stress (500-Yr), Guide Banks

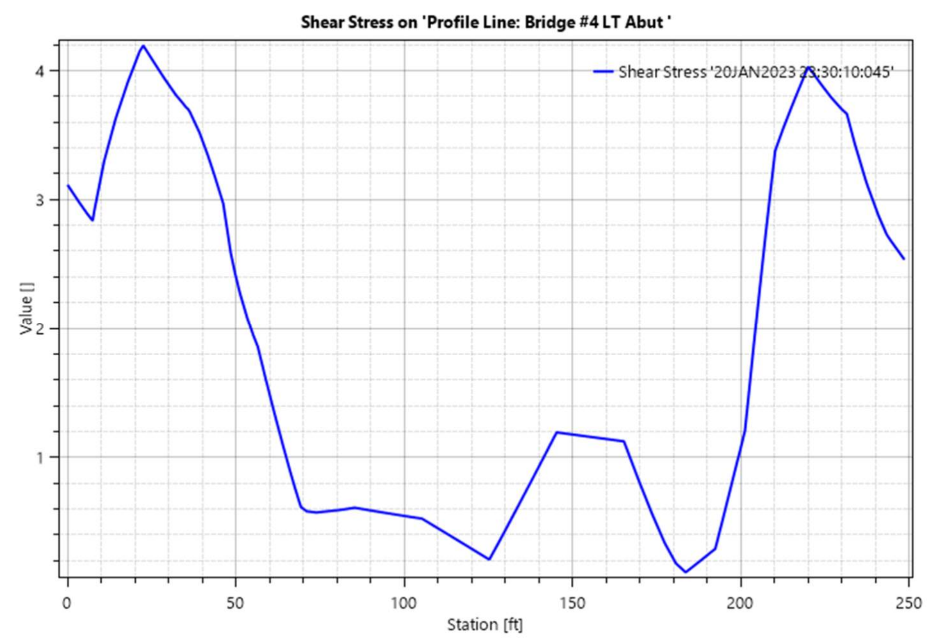


Figure 4.2N. Bridge #4 Left Abutment Shear Stress (500-Yr), Existing Conditions

Figure 4.2O. Bridge #4 Left Abutment Shear Stress (500-Yr), Guide Banks

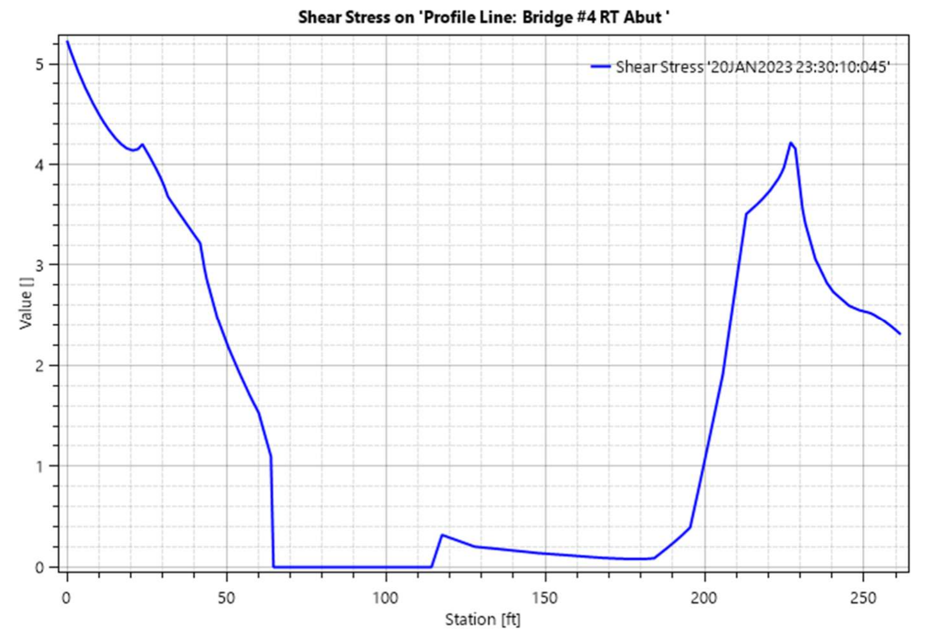


Figure 4.2P. Bridge #4 Right Abutment Shear Stress (500-Yr), Existing Conditions

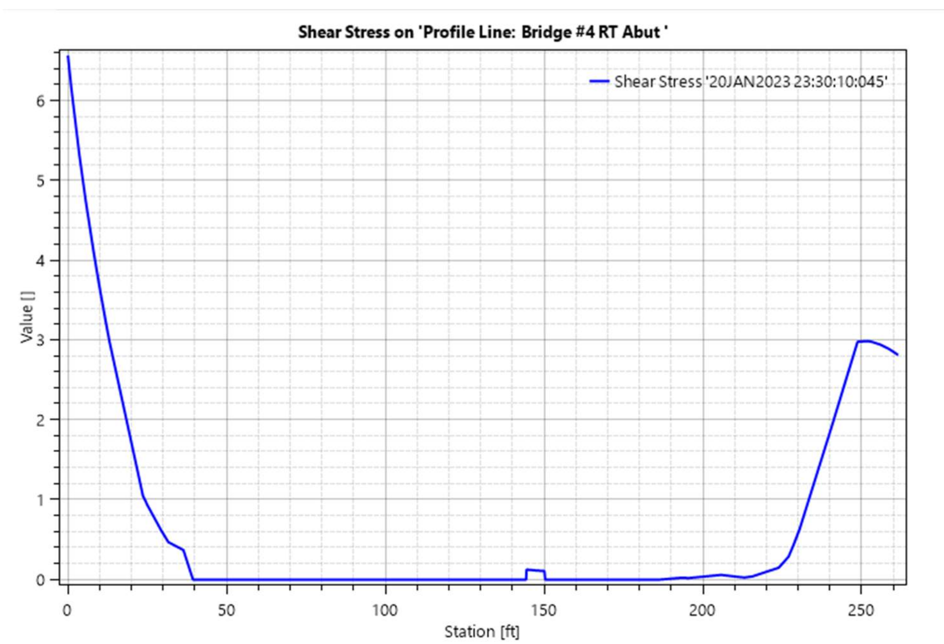


Figure 4.2Q. Bridge #4 Right Abutment Shear Stress (500-Yr), Guide Banks

4.3 Raised Grade & Proposed Bridge

A third HEC-RAS 2D plan was developed to investigate elimination of roadway overtopping in the 100-yr flood via raising the grade of the roadway. This was modeled using a terrain modification set to 88.0ft or higher along the roadway between Bridge #3 and Bridge #4. In order to offset the loss of conveyance from overtopping, a new floodplain bridge and channel is proposed on the right side of the floodplain adjacent to the intersection of NC-130 with US-74. This location was chosen to provide relief to flow that is being trapped by US-74 and NC-72 and forced laterally towards the river channel. Instead, the flow will be routed to a low area in the right floodplain, which is currently cut off and underutilized for conveyance.

Figure 4.3A. Proposed Floodplain Bridge and Channel in RAS Mapper

Figure 4.3B. Aerial of Proposed Floodplain Bridge/Channel Location & Noted Properties

The proposed excavated channel, modeled as a terrain modification at elevation 78.5ft or lower, is necessary to facilitate positive drainage through this elevated agricultural area. The channel, roughly 300ft wide and 3500ft long, is flanked by berms with a top elevation of 88.0ft or higher, set 2.0ft higher than the existing 100-yr WSEL upstream of the proposed bridge. These berms are intended to prevent increasing WSELs on residential properties in the vicinity of the channel. The bridge was sized at 400ft overall length with a span arrangement of 4@100ft to maximize conveyance while avoiding major impacts to adjacent properties. Deck elevation was set at 93.0ft to provide at least a foot of clearance between top of berms and low chord. Figures 4.3C and 4.3D depict the 100-year and 500-year velocities and flow patterns, respectively, along the flood channel.

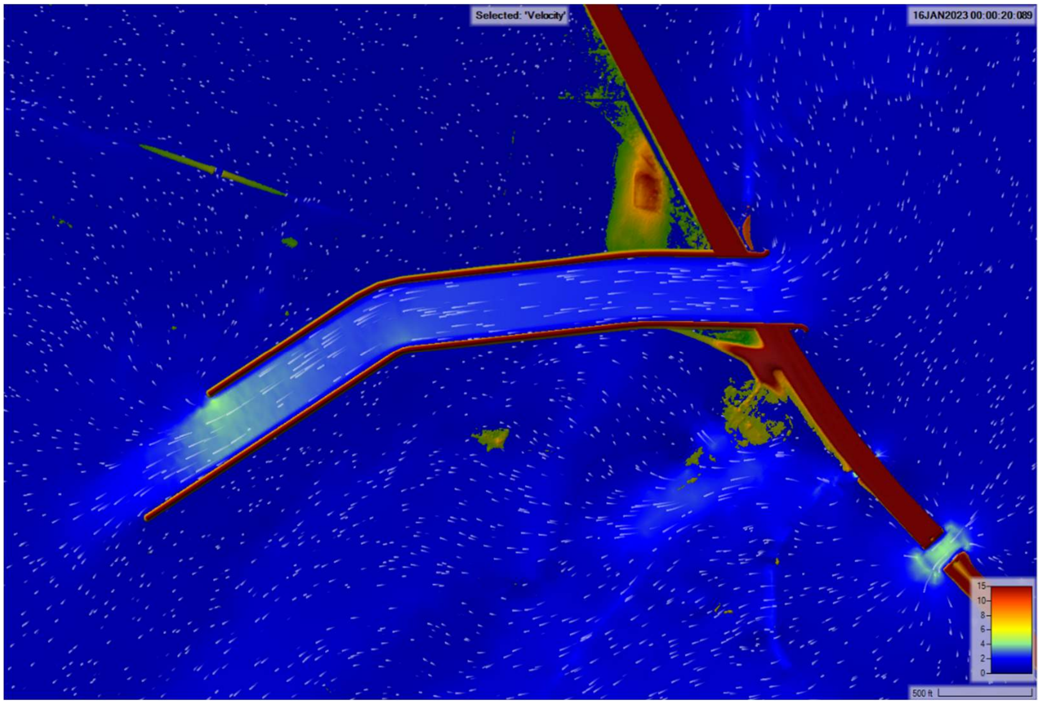


Figure 4.3C. 100-yr Velocities and Flow Patterns for Proposed Floodplain Bridge and Channel

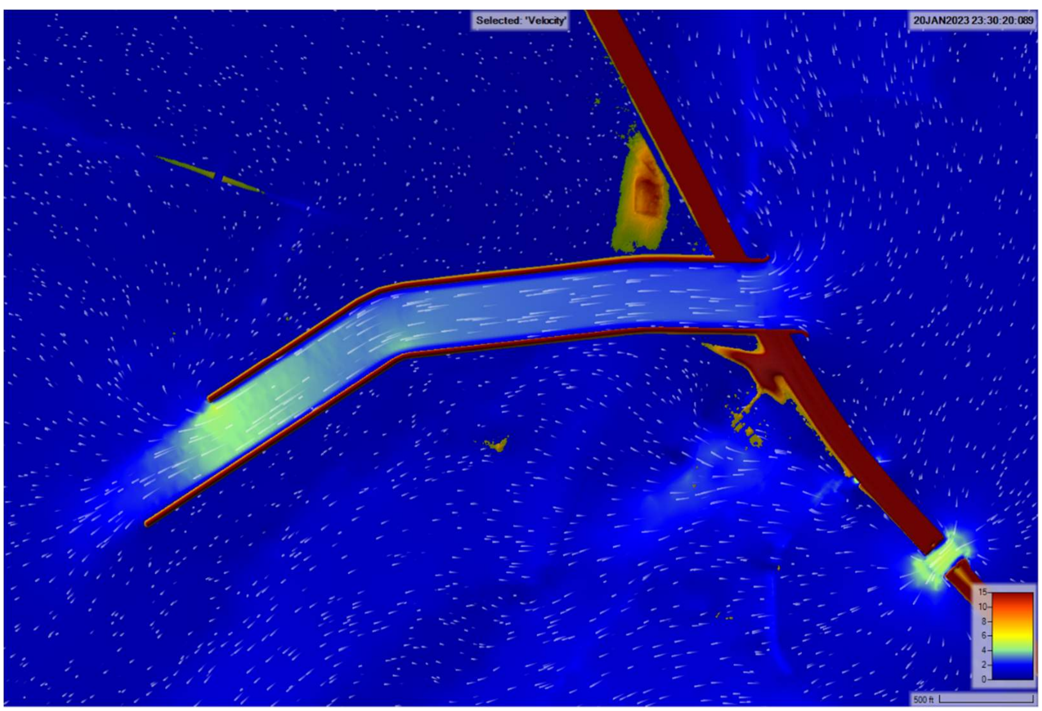


Figure 4.3D. 500-yr Velocities and Flow Patterns for Proposed Floodplain Bridge and Channel

NC-130 in its current alignment represents an obstacle to this plan, but Robeson County parcel data, shown within RK&K's AGOL map on Figure 4.3E, indicates NCDOT has acquired right-of-way at the current intersection of NC-72 and US-74 for a proposed interchange as part of the upgrade to freeway status, which would see NC-130 realigned. A single residence stands in the path of the channel at 20636 NC-130, which would necessitate removal, but all other nearby homes would retain access to NC-130 or US-74. Stephens Community Cemetery, which is immediately north of the proposed channel but well above the 500-yr WSEL, would require a new driveway to tie to the old NC-130 alignment west of the channel.

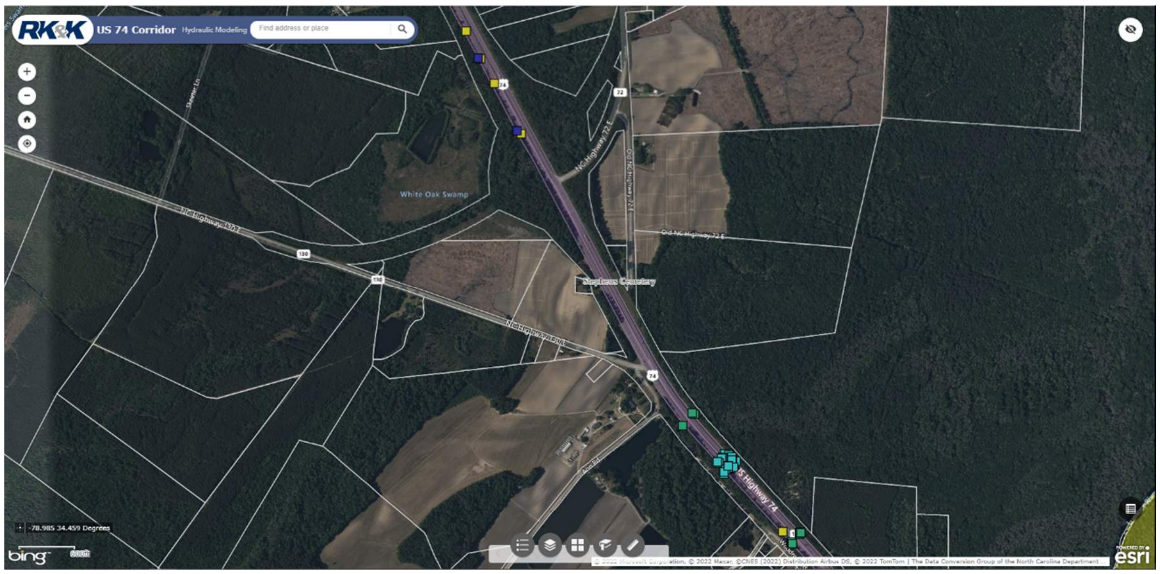


Figure 4.3E. RK&K AGOL Map - NCDOT ROW for Future US-74/NC-72/NC-130 Interchange

Figure 4.3F compares the 100-year headwater elevation between Bridge #3 and the old alignment of NC-72 under existing conditions and with the incorporation of the flood channel and bridge. It shows that the channel and bridge, located near station 4750, is sufficient to convey the 100-year overtopping discharge and lower the water surface on the upstream side of US-74 roughly 0.5ft. The 500-year water surface comparison (Figure 4.3G) shows that the Alternative 2 water surface does just exceed the Existing water surface on the southeast side of the profile, near Bridge #3, but is significantly reduced, up to 1.0ft, along most of the profile. The suitability of the channel and bridge is further confirmed via comparison of overtopping flow under existing conditions (Figure 4.3H) and discharge through the bridge for the Alternative 2 plan (Figure 4.3I). It is clear that the channel and bridge have ample capacity to convey the overtopping flow through the 100-year event. The 500-year comparison is much closer, but suggests that the channel and bridge can convey just under 6000 cfs, vs a total overtopping discharge approaching 5600 cfs.

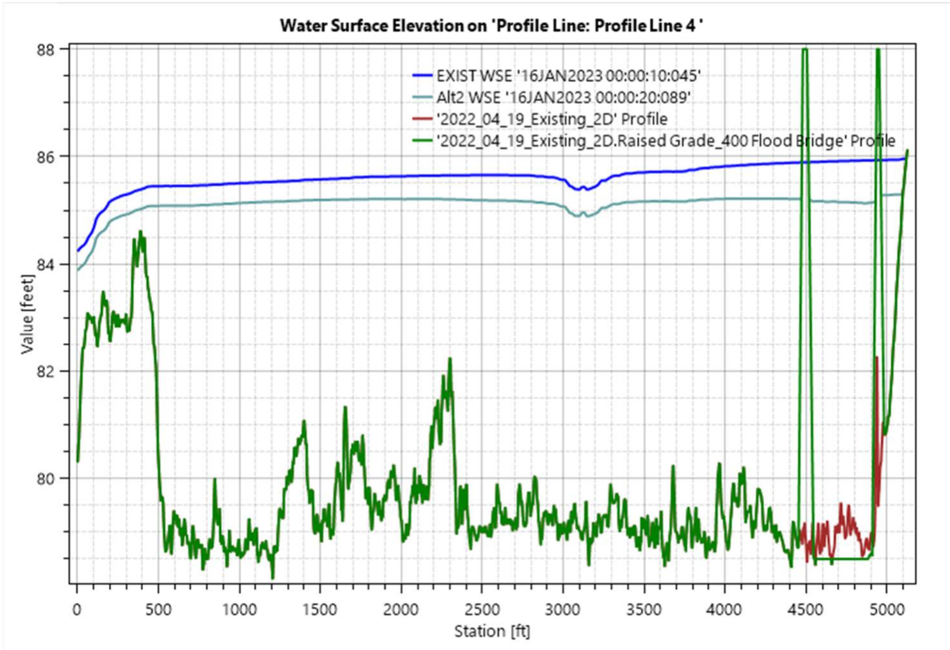


Figure 4.3F. 100-Yr Water Surface Elevation Comparison Along Upstream Embankment

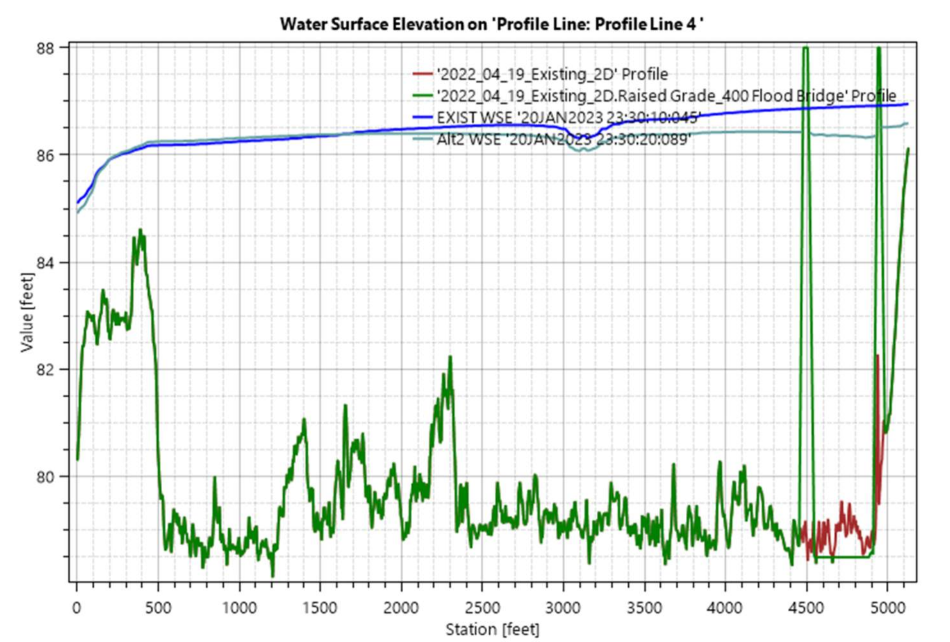


Figure 4.3G. 500-Yr Water Surface Elevation Comparison Along Upstream Embankment

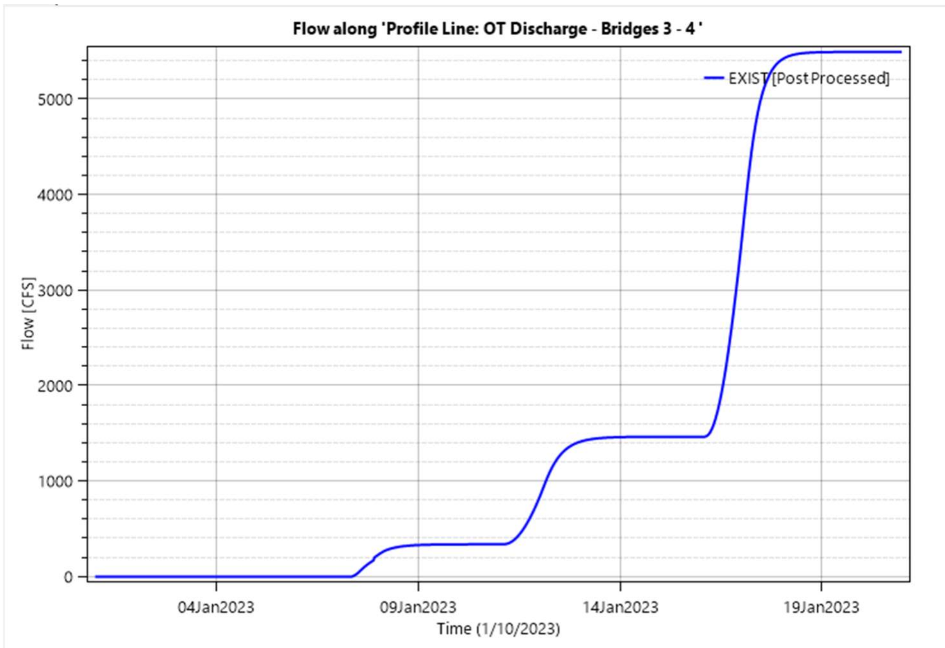


Figure 4.3H. Overtopping Flow Between Bridges #3 & #4 (Existing)

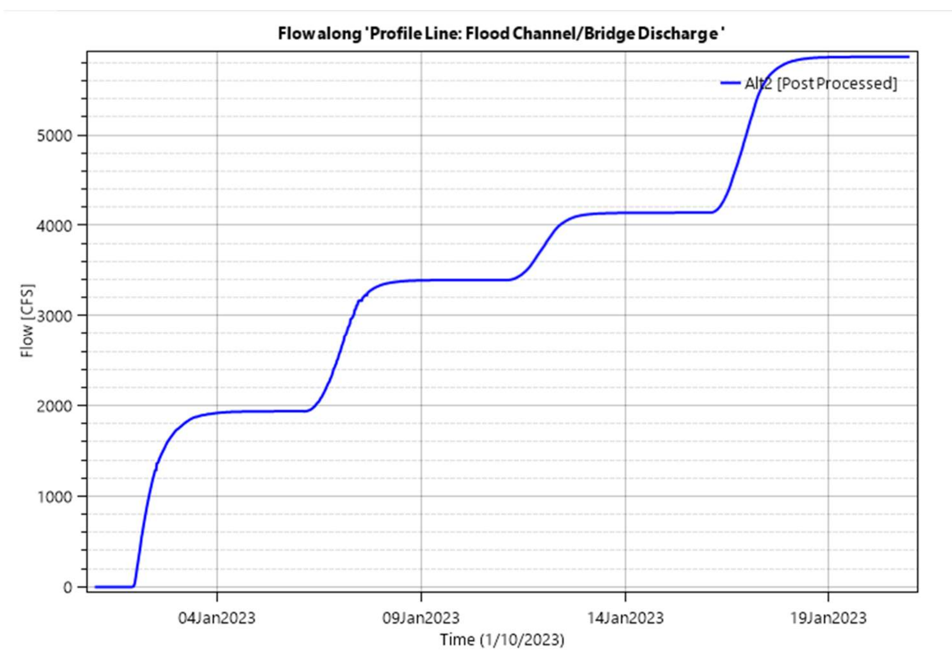


Figure 4.3I. Flow Through Flood Bridge & Channel (Alt 2)

The most notable impact from this alternative would be seen at the existing residence at 20296 NC-130, which sits at an elevation of 79ft. The channel and berms were designed to carry flow past the parcel before discharging to a lower area of the floodplain. However, due to land use and associated n-values (mainly the open water of an existing pond), a portion of the discharge is flowing around the end of the berm and back north towards the parcel (Figures 4.3K and 4.3L). This is producing a rise of approximately 0.15ft along the boundary of the parcel (Figure 4.3M). This rise practically disappears in the 500-year event (Figure 4.3N), as both water surface elevations sit at roughly 81.8ft.

Figure 4.3J. 20296 NC-130
Credit: Google Streetview

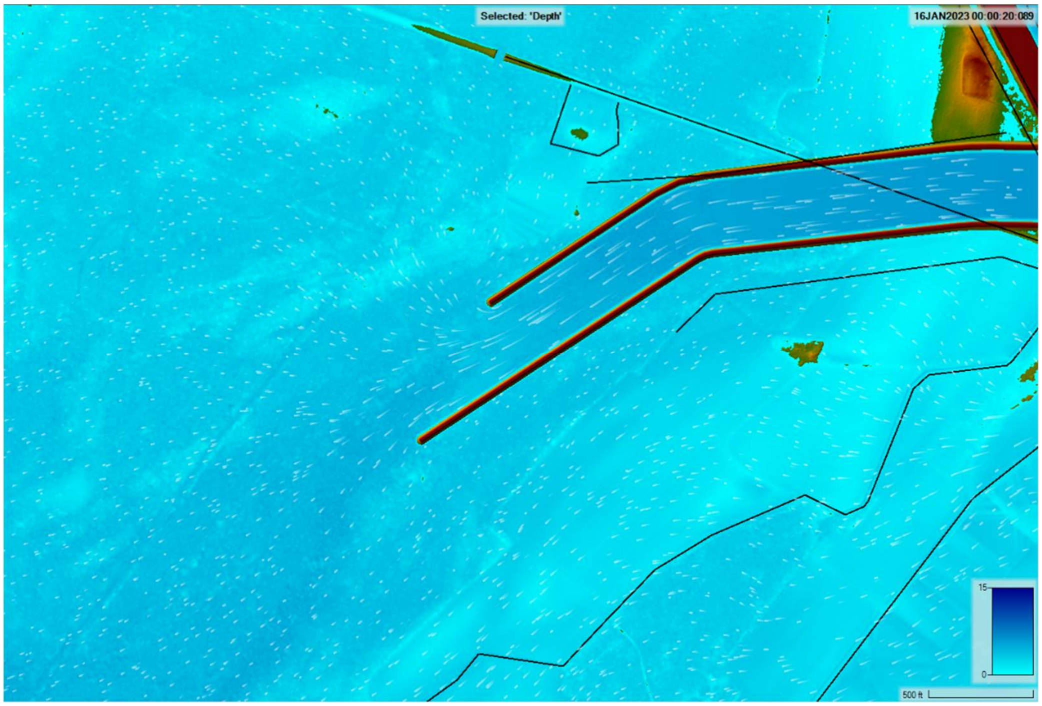


Figure 4.3K. 100-Yr Depth and Flow Patterns at 20296 NC-130 with Flood Channel

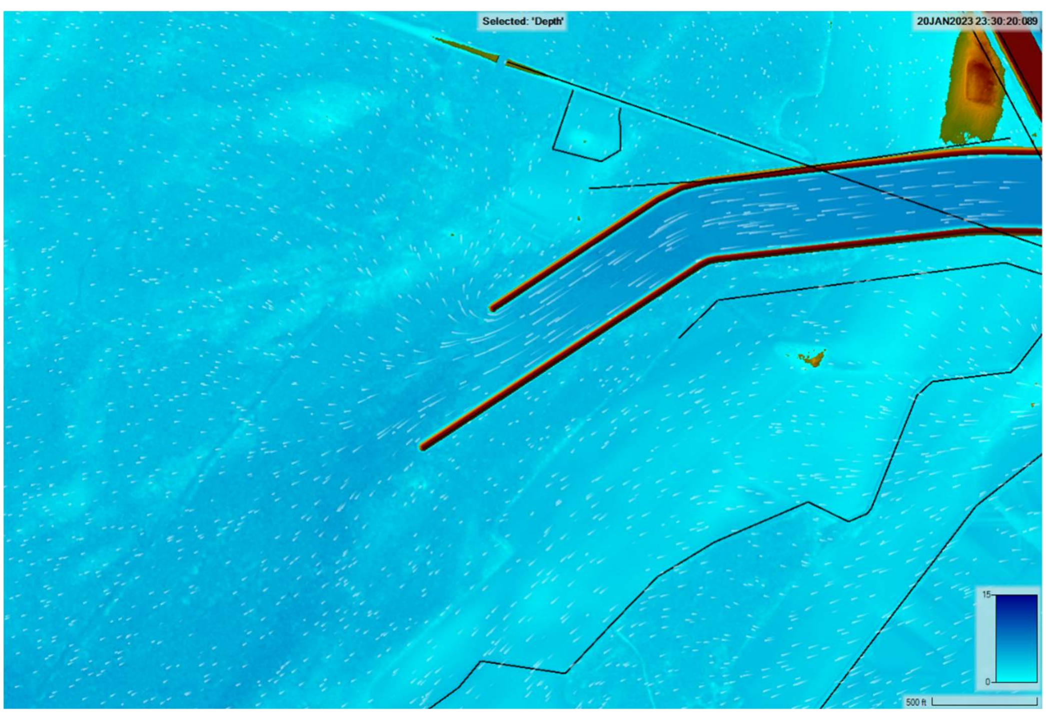


Figure 4.3L. 500-Yr Depth and Flow Patterns at 20296 NC-130 with Flood Channel

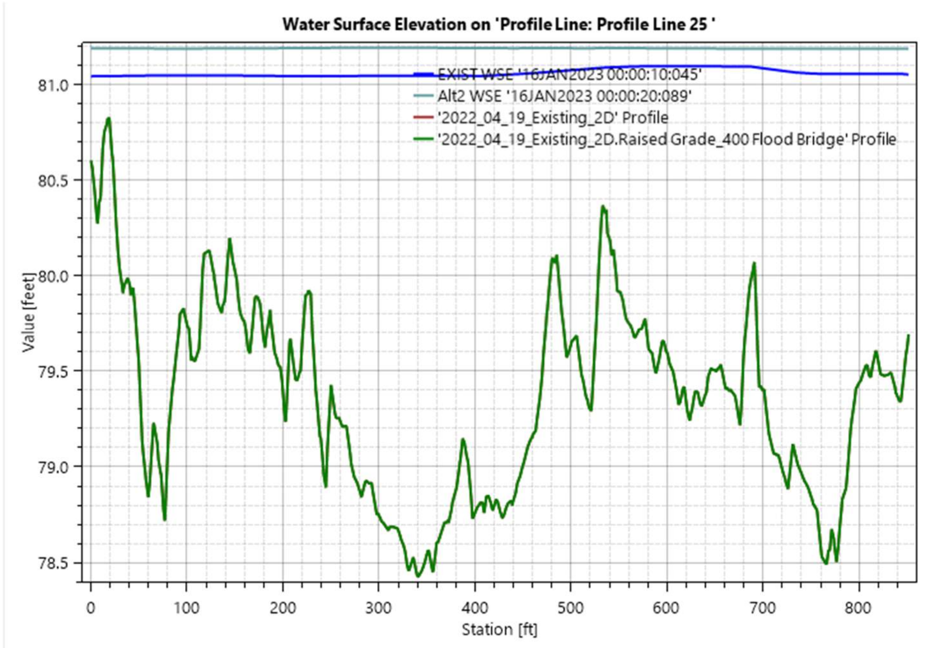


Figure 4.3M. 100-Yr Water Surface Elevation Comparison at 20296 NC-130

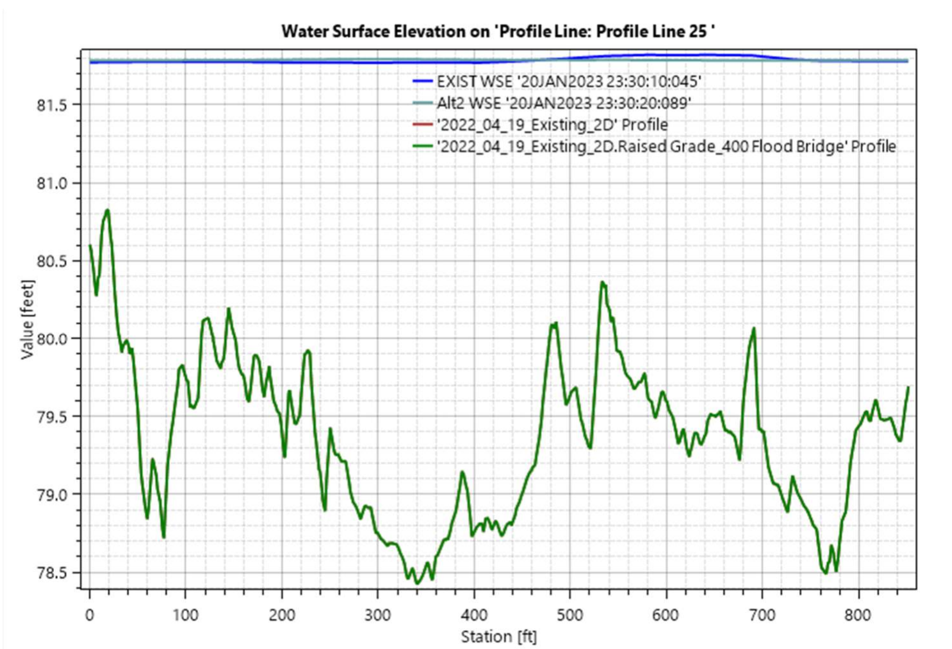


Figure 4.3N. 500-Yr Water Surface Elevation Comparison at 20296 NC-130

5 CONCLUSION

NCDOT requested RK&K to perform a hydraulic assessment of the US-74 Lumber River crossing as part of an effort to minimize future maintenance costs due to large storm events. RK&K has carried out this assessment using HEC-RAS 2D to determine the potential for damages in future storm events and to identify suitable mitigation options for resiliency of the crossing. It was determined that the Lumber River Crossing experiences overtopping for both the 100-year and 500-year storm events, which produces strong erosive velocities on the shoulder of the Eastbound lanes. In addition, the current velocities and flow patterns through the four sets of bridges promotes scour and erosion along abutments and adjacent embankment. Three potential alternatives have been identified to mitigate damage in low frequency storms, which vary in intent, effectiveness, and complexity. Table 5 summaries the impacts each alternative would have to NCDOT's resiliency concerns. RK&K presents these alternatives to NCDOT for consideration and evaluation. Cost estimates for each alternative are to be included in future submittals.

TABLE 5: SUMMARY OF ALTERNATIVES

Concern	Mitigation Options		
	Alternative 1: Shoulder Hardening	**Alternative 2: Guide Banks**	**Alternative 3: Raised Grade and Proposed Bridge**
Road Closure (Overtopping)	-	-	**X**
Damage to Roadway Surface	**X**	-	**X**
Scour to Structures (Bridge Abutments)	-	**X**	**-**

APPENDIX A
HYDROLOGY INFORMATION

Lumber River - Willoughby Rd (SR-2121)

Region ID:	NC
Workspace ID:	NC20220603201922157000
Clicked Point (Latitude, Longitude):	34.50394, -78.94451
Time:	2022-06-03 16:19:44 -0400

Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	772	square miles
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	7.49	inches
LC06DEV	Percentage of land-use from NLCD 2006 classes 21-24	10.7	percent
LC06IMP	Percentage of impervious area determined from NLCD 2006 impervious dataset	1.98	percent
PCTREG1	Percentage of drainage area located in Region 1 - Piedmont / Ridge and Valley	0	percent
PCTREG2	Percentage of drainage area located in Region 2 - Blue Ridge	0	percent
PCTREG3	Percentage of drainage area located in Region 3 - Sandhills	41.9	percent
PCTREG4	Percentage of drainage area located in Region 4 - Coastal Plains	58.1	percent
PCTREG5	Percentage of drainage area located in Region 5 - Lower Tifton Uplands	0	percent

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Peak Southeast US over 1 sqmi 2009 5158]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	772	square miles	1	9000
PCTREG1	Percent Area in Region 1	0	percent	0	100
PCTREG2	Percent Area in Region 2	0	percent	0	100
PCTREG3	Percent Area in Region 3	41.9	percent	0	100
PCTREG4	Percent Area in Region 4	58.1	percent	0	100
PCTREG5	Percent Area in Region 5	0	percent	0	100

Peak-Flow Statistics Flow Report [Peak Southeast US over 1 sqmi 2009 5158]

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	4270	ft^3/s	2450	7440	34.5
20-percent AEP flood	7210	ft^3/s	4170	12500	34
10-percent AEP flood	9410	ft^3/s	5350	16500	35.1
4-percent AEP flood	12200	ft^3/s	6700	22200	37.5
2-percent AEP flood	14600	ft^3/s	7760	27500	39.6
1-percent AEP flood	16900	ft^3/s	8680	32900	41.9
0.5-percent AEP flood	19300	ft^3/s	9570	38900	44.3
0.2-percent AEP flood	22500	ft^3/s	10600	47700	47.7

Peak-Flow Statistics Citations

Weaver, J.C., Feaster, T.D., and Gotvald, A.J.,2009, Magnitude and frequency of rural floods in the Southeastern United States, through 2006—Volume 2, North Carolina: U.S. Geological Survey Scientific Investigations Report 2009–5158, 111 p. (http://pubs.usgs.gov/sir/2009/5158/)

Application Version: 4.9.0
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.0

Big Swamp - Old Whiteville Rd. (SR-1002)

Region ID:	NC
Workspace ID:	NC20220603183448903000
Clicked Point (Latitude, Longitude):	34.47990, -78.87825
Time:	2022-06-03 14:35:32 -0400

Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	422	square miles
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	7.9	inches
LC06IMP	Percentage of impervious area determined from NLCD 2006 impervious dataset	0.79	percent
PCTREG1	Percentage of drainage area located in Region 1 - Piedmont / Ridge and Valley	0	percent
PCTREG2	Percentage of drainage area located in Region 2 - Blue Ridge	0	percent
PCTREG3	Percentage of drainage area located in Region 3 - Sandhills	0	percent
PCTREG4	Percentage of drainage area located in Region 4 - Coastal Plains	100	percent
PCTREG5	Percentage of drainage area located in Region 5 - Lower Tifton Uplands	0	percent

General Disclaimers

This watershed has been edited, computed flows and basin characteristics may not apply. For more information, submit a support request from the 'Help' button in the upper-right of the screen, attach a pdf of this report and request assistance from your local streamstats regional representative.

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Peak Southeast US over 1 sqmi 2009 5158]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	422	square miles	1	9000
PCTREG1	Percent Area in Region 1	0	percent	0	100
PCTREG2	Percent Area in Region 2	0	percent	0	100
PCTREG3	Percent Area in Region 3	0	percent	0	100
PCTREG4	Percent Area in Region 4	100	percent	0	100
PCTREG5	Percent Area in Region 5	0	percent	0	100

Peak-Flow Statistics Flow Report [Peak Southeast US over 1 sqmi 2009 5158]

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

Statistic	Value	Unit	PIl	PIu	ASEp
50-percent AEP flood	3050	ft^3/s	1760	5290	34.5
20-percent AEP flood	5450	ft^3/s	3170	9380	34
10-percent AEP flood	7240	ft^3/s	4140	12700	35.1
4-percent AEP flood	9570	ft^3/s	5280	17300	37.5
2-percent AEP flood	11600	ft^3/s	6200	21700	39.6
1-percent AEP flood	13800	ft^3/s	7130	26700	41.9
0.5-percent AEP flood	15700	ft^3/s	7830	31500	44.3
0.2-percent AEP flood	18600	ft^3/s	8840	39100	47.7

Peak-Flow Statistics Citations

Weaver, J.C., Feaster, T.D., and Gotvald, A.J.,2009, Magnitude and frequency of rural floods in the Southeastern United States, through 2006--Volume 2, North Carolina: U.S. Geological Survey Scientific Investigations Report 2009-5158, 111 p. (http://pubs.usgs.gov/sir/2009/5158/)

Application Version: 4.9.0
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.0

APPENDIX B
HISTORICAL DATA

Damage & Repairs - Hurricane Matthew

Damage & Repairs - Hurricane Matthew (cont.)

Damage & Repairs - Hurricane Matthew (cont.)

Damage & Repairs - Hurricane Matthew (cont.)

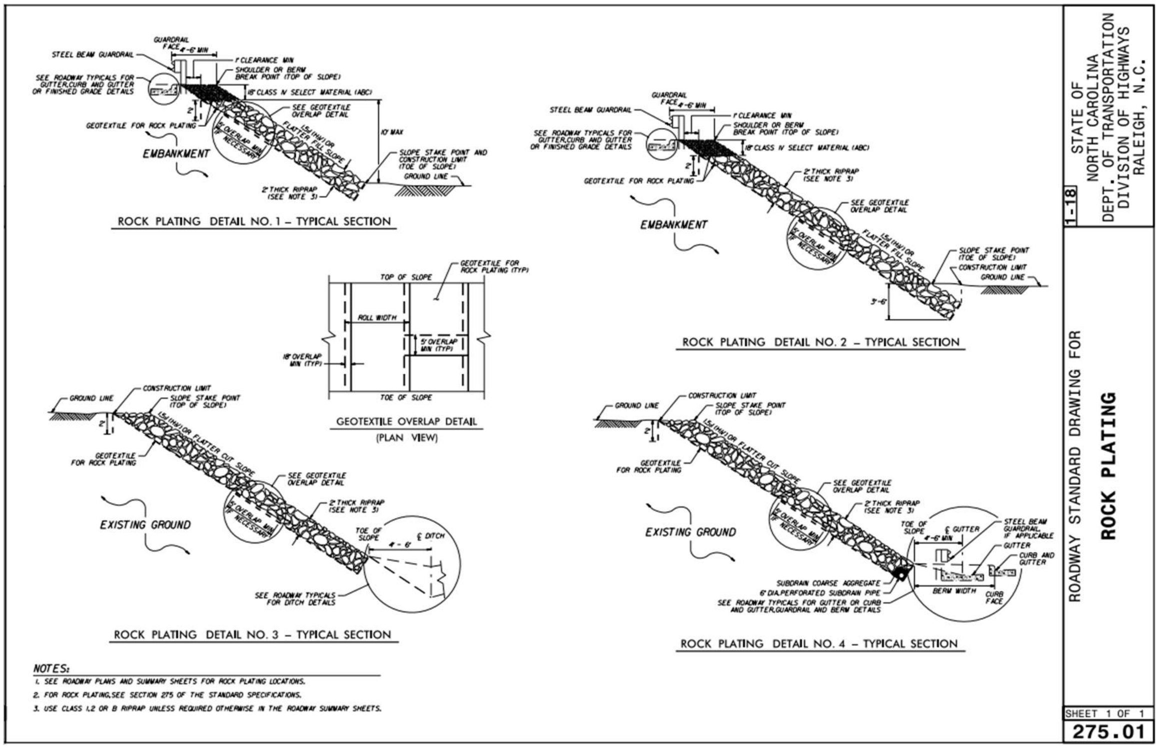
Damage & Repairs - Hurricane Florence

Damage & Repairs - Hurricane Florence (cont.)

Damage & Repairs - Culvert #4 (March 2020)

APPENDIX C
ALTERNATIVES DATA

Rock Plating Detail



NCDOT Roadway Standard Drawing 275.01: Rock Plating

Design Limits for Erosion Control Products

Flexamat® Design Parameters

Test	Tested Value	Bed Slope	Soil Classification	Limiting Value
ASTM 6460	Shear Stress	30%	Sandy Loam (USDA)	24 + PSF
ASTM 6460	Velocity	20%	Loam (USDA)	30 + st/sec

Flexamat ACBM Parameters

General Composition of Materials	
Blocks	Concrete 50 MPA
Interlocking Geogrid	Flexible High Strength 60/60 kN Geogrid
Greenstar Rating	3 Stars
Underlay Options	Vegetative (biodegradable) Non Vegetative (non-biodegradable)
Concrete Colour	Can be coloured as requested
Manufacturing Values	
Roll Width	1.2m, 2.5m, 5.0m, Custom
Roll Length	15m, 20m, 25m, Custom
Material Weight	50kgs per Square Metre
Block Size	165mm x 165mm x 60mm
Performance Design Criteria	
Expected Design Life	50+ years Non Vegetative 100+ years Vegetative
Percentage Open Area	20%
Ultimate Tensile Strength	≥60 kN/m
Critical Flow Shear Stress for initiation of scour	≥1.149 kpa
Maximum Flow Velocity	≥5.79 m/s
Manning's n	0.05

Earthlok ACBM Parameters

Design Limits for Erosion Control Products (cont.)

A. ShoreFlex® will resist erosion and scour due to hydraulic forces. ShoreFlex® will meet the requirements listed in Table 2 when tested with a backing material on a non-vegetated surface.

Table 2: Limiting shear stress testing, ASTM D 6460

Test	Tested value	Bed Slope	Limiting Value
ASTM 6460	Shear Stress	10% & 20%	18 lb./ft.^{2}
ASTM 6460	Velocity	10% & 20%	30 ft./sec

ShoreFlex ACBM Parameters

PYRAMAT® WOVEN PRODUCT LINE CARD[1]

	Property	Test Method	Value	Unit	PYRAMAT® 25	PYRAMAT® 50	PYRAMAT® 75
PHYSICAL	Mass/Unit Area	ASTM D-6566	MARV	oz/yd^2 g/m^2	8.0 271	11.0 373	14.0 475
	Thickness	ASTM D-6525	MARV	in mm	0.25 6.35	0.30 7.62	0.40 10.16
	Light Penetration	ASTM D-6567	MARV	% Passing	35%	25%	10%
	Color	Visual	-	-	Green or Tan	Green or Tan	Green or Tan
MECHANICAL	Grab Tensile Strength	ASTM D-6818	MARV	lb/ft kN/m	2000 x 1800 29.2 x 26.3	3200 x 3000 46.7 x 43.8	4000 x 3000 58.4 x 43.8
	Grab Elongation	ASTM D-6818	MARV	%	20 x 20	30 x 30	40 x 35
	Resiliency	ASTM D-6524	MARV	%	70%	70%	80%
	Flexibility	ASTM D-6575	MARV	in-lb mg-cm	0.195 225,000	0.195 225,000	0.534 616,154
ENDURANCE	UV Resistance	ASTM D-4355	MARV	% Retained @ 1,000 hrs	90%	-	-
				% Retained @ 3,000 hrs	90%	90%	90%
				% Retained @ 6,000 hrs	-	90%	90%
PERFORMANCE	Velocity (Vegetated)	Large Scale	MARV	ft/sec m/sec	20 6.1	22 6.7	25 7.6
	Shear Stress (Vegetated)	Large Scale	MARV	lb/ft^2 Pa	12 575	14 670	16 766
	Manning's "n" (Unvegetated)	Calculated	MARV	N/A	0.028	0.028	0.028
	Seedling Emergence	ASTM D-7322	Typical	%	255%	-	296%
	Roll Sizes	Measured	Typical		8.5 ft x 120 ft	8.5 ft x 120 ft 15.0 ft x 120 ft	8.5 ft x 120 ft 15.0 ft x 120 ft

NOTES:

1. The property values listed above are effective 03/09/2018 and are subject to change without notice. Values represent testing at time of manufacture.

PyraMat HPTRM Parameters

Design Limits for Erosion Control Products (cont.)

ROLLMAX™
ROLLED EROSION CONTROL

Specification Sheet

TMax™ High-Performance Turf Reinforcement Mat

DESCRIPTION

The TMax™ high-performance turf reinforcement mat (HP-TRM) shall be a machine-produced mat of 100% UV-stabilized, high denier polypropylene monofilament yarns woven into permanent, high-strength, three-dimensional turf reinforcement matting. Available in either a green/black or a tan/black coloring, the mat shall be composed of polypropylene yarns woven into a uniform configuration of resilient, pyramid-like projections. The mat provides sufficient thickness, optimum open area, and three-dimensionality for effective erosion control and vegetation reinforcement against high flow induced shear forces. The mat has high tensile strength for excellent damage resistance and for increasing the bearing capacity of vegetated soils subject to heavy loads from maintenance equipment and other vehicular traffic. The material has very high interlock and reinforcement capacities with both soil and root systems, and is designed for erosion control applications on steep slopes and vegetated waterways.

Material Content		
Woven Structure	100% UV stable Polypropylene Monofilament yarns	Black/Green or Black/Tan

Index Property	Test Method	Typical
Thickness	ASTM D6525	0.4 in (10 mm)
Resiliency	ASTM D6524	75%
Mass/Unit Area	ASTM D6566	11.3 oz/yd² (382 g/m²)
Tensile Strength - MD	ASTM D6818	4,400 lbs/ft (64 kN/m)
Elongation - MD	ASTM D6818	35%
Tensile Strength - TD	ASTM D6818	3,300 lbs/ft (48.2 kN/m)
Elongation - TD	ASTM D6818	30%
Light Penetration	ASTM D6567	75% coverage
UV Stability	ASTM D4355	>90% @ 3000 hr

Design Permissible Shear Stress*	
Vegetated Shear	16 psf (766 Pa)
Vegetated Velocity	25 fps (7.6 m/s)

+ Minimum Average Roll VAlue
*Design values extrapolated from large scale ASTM D6460 testing

RollMax TMax HPTRM Parameters

T-RECS®

Turf Reinforcement Erosion Control Solution®

The high-performance T-Recs® Turf Reinforcement Mat is a permanent three dimensional, woven polypropylene geotextile designed for steep slopes up to 0.5:1 and is an ideal non-hard armoring solution for high velocity channels.

T-Recs® is manufactured with a patented process of cross directional monofilament fibers woven into multiple dimensions featuring the T-Recs® Technology with dome characteristics. This unique process and feature aids in the performance of the product and gives additional support to the vegetation. The product provides reinforcing capabilities and interlocking root system, while assisting the vegetation establishment. Product can be either surface applied or soiled filled to maximize performance.

The T-Recs® meets Type 5.A, 5.B, 5.C, 5.D, 5.E specification requirements established by the Erosion Control Technology Council (ECTC) and Federal Highway Administration's (FHWA) FP-03 Section 713.18.

INDEX VALUE PROPERTIES

PROPERTY	TEST METHOD[2]	ENGLISH	METRIC
Mass Per Unit Area[3]	ASTM D 6566	8.50 oz/yd²	288.2 g/m²
Thickness	ASTM D 6525	.45 inch	11.4 mm
Tensile Strength-MD[3]	ASTM D 6818	3000 lb/ft	44 kN/m
Elongation – MD	ASTM D 6818	41%	
Tensile Strength-TD[3]	ASTM D 6818	3000 lb/ft	44 kN/m
Elongation-TD	ASTM D 6818	17%	
Light Penetration	ASTM D 6567	34%	
Germination/Seedling Emergence	ECTC Method 4	636% Improvement	
UV Resistance (6,000 hours)	ASTM D4355	91%	
Resiliency	ASTM D 6524	70%	
Flexibility	ASTM D 6575	0.3 in-lb	346,154 mg-cm
Color	Observed	Green/Blue or Tan/Blue	
Roll Size	Measured	12.0 ft x 75.0 ft	3.7 m x 22.9 m

PERFORMANCE DESIGN VALUES

PROPERTY	TEST METHOD[1]	ENGLISH[2]	METRIC[2]
Vegetated Shear Stress	ASTM D 6460	15 lb/ft²	718 Pa
Vegetated Velocity	ASTM D 6460	25 ft/sec	7.6 m/sec
Manning's N	ASTM D 6460	0.028	

T-RECS HPTRM Parameters

Design Limits for Erosion Control Products (cont.)

Enkamat® 7010 is a 3-dimensional turf reinforcement mat (TRM) made of continuous monofilaments fused at their intersections. Ninety-five (95%) percent of the Enkamat is open and available for soil, mulch and root interaction, creating the most effective root reinforcement mat (R2M) available. Enkamat is manufactured from nylon to eliminate the buoyancy factor associated with submerged conditions and provides permanent TRM protection in vegetated channels as well as on slopes.

- Permanent erosion control for vegetated channels with expected shear stresses ≤ 8 psf.
- Permanent erosion control for slight to moderate slopes (≤1H:1V).
- Support and enhance hydraulic and agronomic performance of ecosystem plants.
- Excellent substrate for hydraulically applied mulches for applications where calculated hydraulic Forces exceed the threshold of the mulch by itself and/or unreniforced vegetation.
- Meets requirements for FHWA FP-03 Type 5B TRM

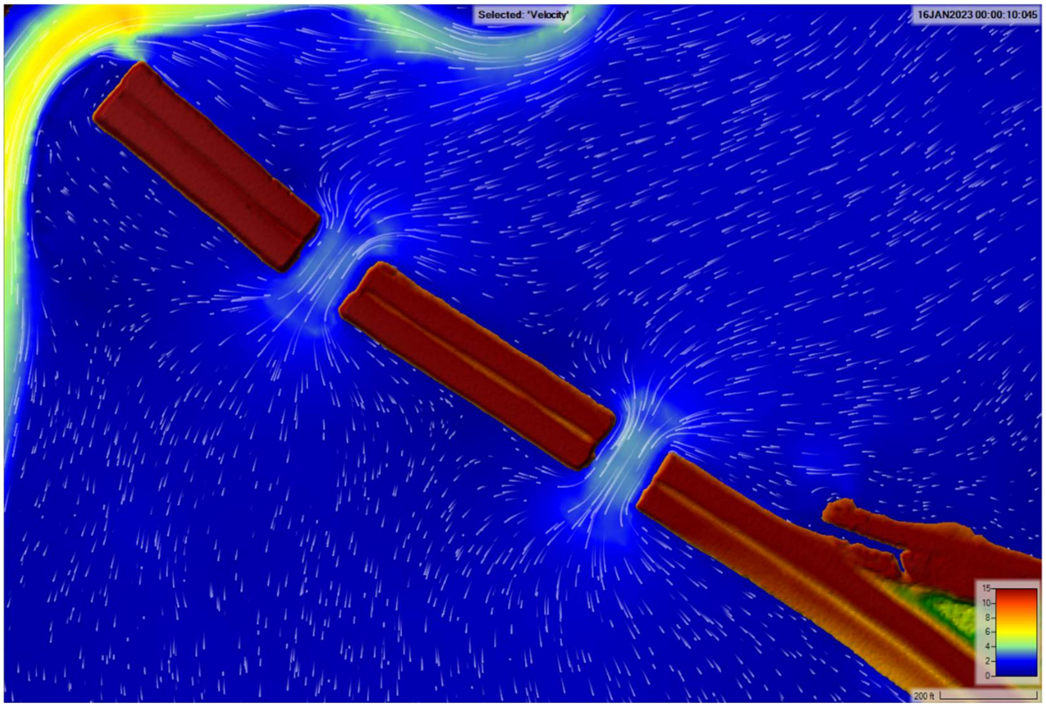
Mechanical Properties	Test Method	Units	MARV Roll Value
Tensile Strength	ASTM D 6818	kN/m (lbs/ft)	2.2 (150)
Thickness	ASTM D 6525	mm (in)	7.5 (0.3)
Mass/Unit Area	ASTM D 6566	g/m^2 (oz/yd^2)	220 (6.5)
Resiliency	ASTM D 6524	%	>80
UV Stability	ASTM D 4355	% strength retained	80 @ 2000 hr

Performance Properties	Test Method	Units	Typical Roll Value
Permissible Velocity			
30 minute, vegetated	Flume test[1]	m/s (ft/s)	5.8 (19)
50 hour, vegetated	Flume test[1]	m/s (ft/s)	4.2 (14)
Permissible Shear Stress			
30 minute, vegetated	Flume test[1]	kN/m^2 (lbs/ft^2)	0.38 (8.0)
50 hour, vegetated	Flume test[1]	kN/m^2 (lbs/ft^2)	0.29 (6.0)
Manning's n Range[2]	Flume test[1]		0.022—0.042

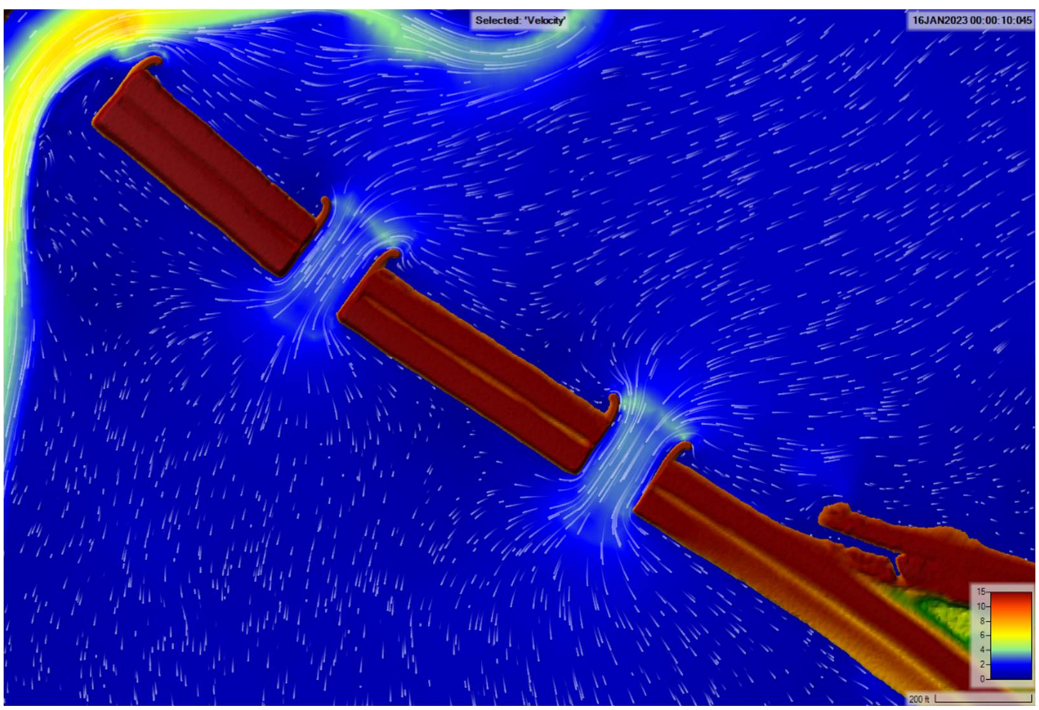
1. Flume test performed at independent laboratory—data and details available upon request.
2. Depending on vegetation type and height, use engineering field experience and examine a range of Manning's n values during design.

Enkamat 7010 TRM Parameters

Bridges #1 & #2 Velocity Comparison

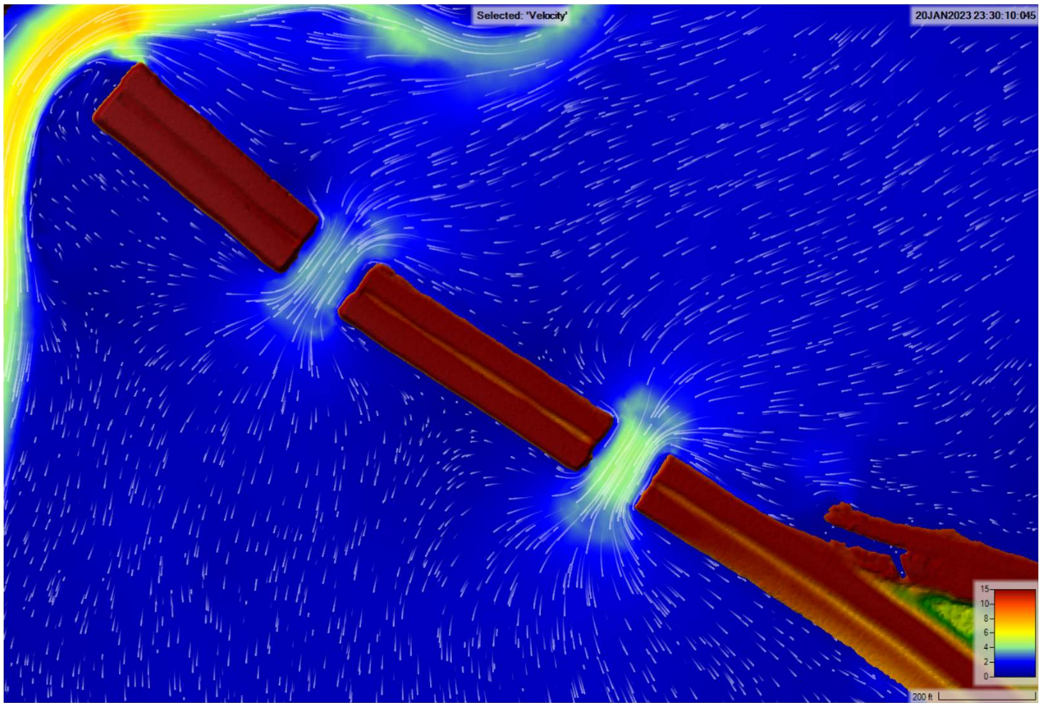


Bridges #1 & #2 100-year Velocity, Existing Conditions

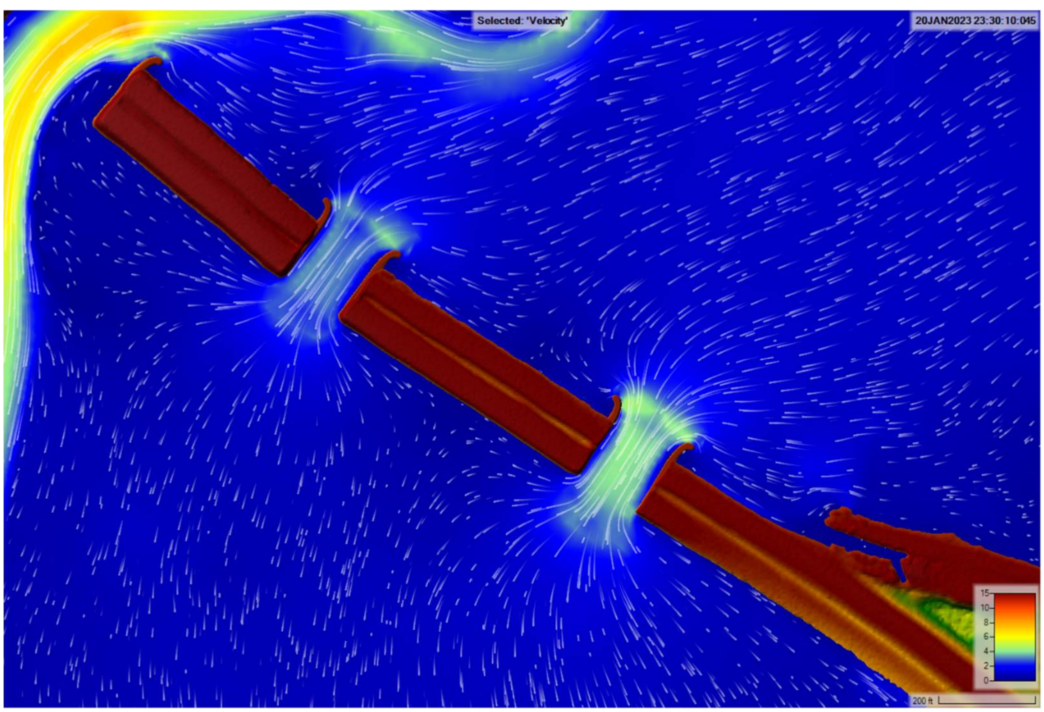


Bridges #1 & #2 100-year Velocity, Guide Banks

Bridges #1 & #2 Velocity Comparison (cont.)



Bridges #1 & #2 500-year Velocity, Existing Conditions



Bridges #1 & #2 500-year Velocity, Guide Banks