HYDROLOGY REPORT

Hydrology Description

Hydrology was developed for the flood of record, which occurred on April 1, 1987, and for a range of annual recurrence interval flows that include the 1.1, 10, 25, 50, 100, and 500-year events.

A comparison of flows at the Dover Bridge, based on the FEMA FIS, USCS gage 0103500, and flood flow frequency analysis following USGS Bulletin 17C was performed. The analysis indicated that the FEMA FIS (1993) produced the highest flows at the Dover Bridge. The additional recurrence interval flows were interpolated to develop the range of flows included in the summary below.

Refer to Appendix E for Hydrology information provided by MaineDOT's Hydraulics Section.

Table 1. FEMA Summary of Discharges

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
PISCATAQUIS RIVER					
At downstream corporate limits	368.4	20,900	30,700	34,500	43,100
At East Dover Road	358.7	20,500	30,000	33,600	41,900
At Essex Street	352.2	20,200	29,500	33,200	41,300
At State Route 15	344.1	19,900	29,100	32,700	40,800
At upstream corporate limits	340.8	19,600	28,800	32,400	40,400

Table 2A. Summary of Hydrology Information

SUMMARY		
Drainage Area	352.2	mi^2
Q1.1	6,800	ft^3/s
Q10	20,200	ft^3/s
Q25	25,500	ft^3/s
Q50	29,500	ft^3/s
Q100	33,200	ft^3/s
Q500	41,300	ft^3/s
Flood of Record	42,290	ft^3/s

Reported by: Lissa Robinson, P.E.
Date: February 28, 2023

Notes:

1. All elevations based on North American Vertical Datum (NAVD) of 1988.
2. Drainage area based on the FEMA FIS 1993.
3. Annual recurrence interval flows provided by MaineDOT and based on FEMA FIS 1993.
4. Flood of Record occurred on April 1, 1987. Method of derivation: T> 500 by 17C EMA; T = 120 by simple plotting position, a = 0.

Table 2B. Comparison of Hydrology of Peak Flow Data

T (yrs)	Q (ft3/s) – Scaled Gage, by Bull 17C EMA	Q (ft3/s) – 1993 FIS
1.1	5,435	6,800
2	9,925	12,000
5	14,685	17,300
10	18,000	20,200
25	22,370	25,500
50	25,725	29,500
100	29,165	33,200
500	37,575	41,300
Flood of Record 04/01/1987	42,290 (T > 500 by 17C EMA; T = 120 by simple plotting position, a = 0)	

HYDRAULIC REPORT

A hydraulic analysis was performed to estimate peak water surface elevation and velocity at the Dover Bridge for a range of flows and for the existing and replacement bridge configurations. The hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) version 6.1.0 software (USACE, 2021). The model was used to simulate 2-dimensional (2D) flows with unsteady analysis, the diffusion wave equation set, and a 1-second fixed computation interval.

The model simulates flow over an area of approximately 0.85 square miles and along a 1.7-mile reach of the Piscataquis River. The flow area extends approximately 0.9 miles upstream and 0.8 miles downstream of the Dover Bridge and includes two hydraulic structures at Upper and Lower Dams. The purpose of the 2D model was to capture flow conditions including shallow flow and split flows across the reach of interest. The model grid size was set to 50 ft by 50 ft to optimize model accuracy, stability, and run time. HEC-RAS 2D creates an elevation-volume relationship for each cell based on the details of the underlying terrain, an attribute not common to other 2D models of which some assume a single elevation and a flat bottom for each cell in the model.

The digital elevation model (DEM) (i.e., terrain) for the model was compiled from three data sources: a survey of the channel bottom at the bridge with 1 ft contours, a 1-meter LiDAR survey prepared in 2015 and distributed by USGS (2020), and FEMA cross-sections of the river channel. The DEM elevations were referenced to North American Vertical Datum of 1988 (NAVD 88). Breaklines were input to align the cell faces of the mesh to linear features such as the dams and bridge.

The 2D flow area included spatially varied Manning's n-values based on the 2016 National Land Cover Database (NLCD) for the Conterminous United States (MRLC, 2019). Manning's n-values were assigned to land cover groups based on Chow (1959), USGS (2015), Jarrett (1984), Trieste and Jarrett (1987), Costa and Jarrett (2008), Grant (1997), Reid and Hickin (2008), and Wahl (1994).

The existing bridge has five piers with a width of about 6 ft at the base and 4 ft near the low chord of the superstructure. Based existing plans, the existing bridge was input as a total opening width of 252.9 ft near the low chord and an opening of 250.4 ft near the base of the abutments. The proposed bridge was input to the hydraulic model with vertical abutments, an opening width of 257 ft, and two 5-ft-wide, vertical piers. Ultimately, a two span structure was proposed, but given the insignificant changes in water elevations between the existing 6-span structure and the 3-span replacement structure, the hydraulic analysis was not refined for the the proposed 2-span alternative.

Table 3 below summarizes the water surface elevations computed by the hydraulic model for the range of flows evaluated as part of the analysis.

Table 3. Flow and Water Surface Elevations at the Dover Bridge (#5118)

		Existing Structure	Proposed Structure*
Total Area of Waterway Opening	ft^2	4,426	4,705
Headwater elevation at Q1.1	ft	331.0	331.0
Headwater elevation at Q25	ft	334.8	334.7
Headwater elevation at Q50	ft	335.6	335.5
Headwater elevation at Q100	ft	336.4	336.2
Headwater elevation at Q500	ft	337.8	337.6
Flood of Record (April 1987)	ft	338.0	337.8
Low chord elevation	ft	338.7	338.7
Freeboard at Q50	ft	3.1	3.2
Freeboard at Q100	ft	2.3	2.5
Velocity at Q1.1	ft/s	3.2	2.8
Velocity at Q25	ft/s	8.9	7.8
Velocity at Q50	ft/s	9.8	8.6
Velocity at Q100	ft/s	10.5	9.3
Velocity at Q500	ft/s	12.0	10.6
Flood of Record (April 1987)	ft/s	12.1	10.7

Notes:

1. Headwater elevations in Table 3 are based on hydraulic model results for maximum water surface elevation along profile line upstream of Dover Bridge.
2. Velocity values in Table 3 are based on hydraulic model results for maximum velocity along profile line at center of Dover Bridge.
3. Annual recurrence interval flows based on FEMA FIS 1993.

4. Flood of Record occurred on April 1, 1987. Method of derivation: T > 500 by 17C EMA; T = 120 by simple
plotting position, a=0.
5. Elevations based on North American Vertical Datum of 1988 (NAVD 88). Conversion: NAVD88 = NGVD29
- 0.56 ft (NOAA, 2021).
6. Replacement bridge configuration based on proposed design at the time of this preliminary hydraulic analysis. Other configurations are being evaluated.
7. Dover Bridge low chord assumed to be at EL. 338.7 ft for freeboard calculations.

*The proposed structure was changed to a two-span alternative after the hydraulic analysis was completed. It is anticipated that the changes in water elevations between the three-span and two-span alternatives will be negligible and within margins of error expected due to modeling variables, and therefore, further hydraulic analysis was not performed.

The 2D hydraulic model results indicated a peak water surface at elevation 336.4 ft at the profile line just upstream of Dover Bridge for the 100-year event. These results compare well with the FEMA elevation of 336.4 ft for the 100-year event developed in 1991 with the previous WSP2 model. However, despite the match in water surface elevations, a difference in results would not be unexpected since there are likely differences in the hydraulic treatment of the dam located about 70 ft downstream of the bridge as well as technical advances in modeling since 1991 including 2D HEC-RAS modeling software, terrain development, and other current modeling and routing methods.

Scour Analysis

Scour analysis was performed at the proposed Dover Bridge for potential pier and contraction scour. Because the Dover Bridge will be replaced, the existing bridge was not analyzed for scour. Bridge inspection data indicates the channel and scour conditions at the existing dam as "Bank protection needs minor repairs" and "Stable for scour conditions".


Photo 1- Upstream Side of Abutments Looking Southwest

During geotechnical field work, scour was observed at the piers just below the waterline (Photo 1). Significant erosion or scour was not observed at the bridge abutments at the time of the field visit.

The scour estimates were developed based on Hydraulic Circular No. 18 (HEC-18) under the U.S. Department of Transportation, Federal Highway Administration (FHWA, 2012).

During boring exploration under the preliminary geotechnical investigation, which is summarized in a separate report, the driller was not able to recover material from the channel bed but was able to retrieve samples from the abutments. Borings at the location of the proposed abutments and lab analysis for grain size indicated a D50, the median particle diameter, of 2.5 mm at a depth of 15 to 17 ft below the road surface on the east bank and a D50 of 0.2 mm at a depth of 10 to 12 ft below the road surface on the west bank. Shallower, finer-grained samples at the abutments were not evaluated for scour since they appeared to be fill and unrepresentative of the channel bottom or banks.

Pier and contraction scour were estimated using a D50 of 2.5 mm. Calculations are included in Appendix C. Abutment scour was not evaluated because the proposed abutment layout had not been developed at the time of this study. It is recommended that abutment scour be estimated when final design information becomes available. Contraction and pier scour for the proposed Dover Bridge are summarized in Table 4.

Table 4. Scour Analysis

Calculation	100-year Event	500-year Event
Critical Velocity (ft/s)	12.3	12.5
Mean Velocity (ft/s) at Dover Bridge	8.5	9.7
Mean Velocity (ft/s) upstream Dover Bridge	4.3	5.0
D50 (mm)	2.5	2.5
Clear Water Contraction Scour (ft)	20.3	25.9
Pier Scour (ft)	8.9	9.5

Notes:

1. Scour depths estimated based on D50 of 2.5 mm and do not account for shallower bedrock.
2. The potential for rock scour was not evaluated as part of this study.

Of note, and as indicated by Hodgkins and Lombard (USGS, 2002) the "HEC-18 pier-scour equations are intended to be envelope equations, ideally never underpredicting scour depths and not appreciably overpredicting them. The 2002 study of pier scour in Maine considered the HEC-18 equations to perform well in Maine where twenty-two out of twenty-three pier-scour depths were overpredicted by depths of 0.7 ft to 18.3 ft, with the study indicating one underprediction by 4.5 ft." Hodgkins and Lombard (USGS, 2008) also developed a study in 2008 titled “Comparison of Observed and Predicted Scour at Selected Bridges in Maine,” which the design team recommends be consulted when the abutment scour estimates are developed.

- The hydraulic characteristics of the river in the vicinity of the Dover Bridge indicate that water levels are strongly influenced by the upstream and downstream dams.
- Assuming the proposed low chord for the replacement bridge matches the existing low chord elevation of 338.7 ft, the proposed clearance would be about 3.2 ft for the 50-year event and 2.5 ft for the 100-year event, based on proposed bridge design information provided by the design team and based on the hydraulic model results.
- The MaineDOT Bridge Design Guide (2003), Structure Capacity (Riverine) indicates that “Major riverine bridges" must be designed with a freeboard depth of 4 feet for the 50-year event (Q50). All bridge-type structures should be capable of passing the Q100 or the flood of record, whichever is greater. When possible, there should be 1 foot of freeboard for the 100-year event (Q100).
    - At Dover Bridge, the Q50 appears to be the limiting event for setting the low chord elevation.
    - The model results indicate the proposed bridge configuration can pass the flood of record, which is greater than the Q100 and has a peak water surface at El. 337.8 ft for the proposed bridge configuration.
- Potential contraction scour depth for the proposed bridge design was estimated to be 20.3 feet below the streambed for the 100-year event, and 25.9 feet below the streambed for the 500-year event, assuming material capable of being scoured along the entire column. The pier scour depth was estimated to be about 8.9 feet for the 100-year event, and 9.5 feet for the 500-year event. These scour depths were based on a D50 of 2.5 mm. Results will be different for different D50 values. In boring BB-DFPR-102 performed through the river near the middle of the existing bridge, bedrock was encountered 3.5 feet below top of river sediment (29 feet below top of existing bridge deck). At abutment borings BB-DFPR-101 and BB-DFPR-103, bedrock was encountered at 19 feet and 20 feet below top of road, respectively. These data indicate that bedrock is likely the limiting factor for pier and contraction scour at the boring locations. Scour can be mitigated through installation of designed scour protection, such as heavy riprap or articulated mats.
- At the time of preparation of this study, a preliminary geotechnical design report was also being performed.
- Abutment scour should be estimated once abutment designs become available. Designs should be developed to address potential scour, as applicable.
- Scour protection should be developed during the final design phase.

The recommendations in this report are based in part on the data obtained from the borings. It was not possible to evaluate sediments in the main channel due to lack of material retrieval during boring exploration. Furthermore, 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.

Appendix E

Hydraulics Data

GEI Consultants

Consulting
Engineers and
Scientists

Preliminary Design Hydrologic and Hydraulic Report

Dover Bridge #5118
Essex Street over Piscataquis River, WIN 23120.00

Dover-Foxcroft, Maine

Submitted to:
Thornton Tomasetti
14 York Street
Portland, ME 04101

Submitted by:
GEI Consultants, Inc.
5 Milk Street
Portland, ME 04101
207-797-8901

Revised February 2023
January 2023
Project 2103596

Lissa Robinson

Lissa Robinson, P.E.
Senior Civil Engineer

Gillian Williams, P.E.
Project Manager

Table of Contents

Tables

Figures

Appendices

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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 the Dover Bridge (MaineDOT Bridge #5118), which carries Essex Street over the Piscataquis River in Dover-Foxcroft, Piscataquis County, Maine (Fig. 1).

The existing bridge spans the Piscataquis River and is situated about 3,190 ft downstream of Upper Dam (also referred to as Moosehead Dam) and 74 ft upstream of Lower Dam (also referred to as Browns Mill Dam). Built in 1930, the concrete bridge is 259 ft long with 6 main spans and a road width of 21 ft (Fig. 2). Annual average daily traffic (AADT) on the bridge is indicated as 2,256 vehicles per day in the 2020 MaineDOT Bridge Maintenance records and 2,087 vehicles per day on the MaineDOT Public Map Viewer (MaineDOT, 2022). Dover Bridge is currently listed as "poor condition (advanced deterioration)" for both the deck and superstructure, and "fair condition (minor section loss)" for the substructure. The channel and scour conditions are indicated as "Bank protection needs minor repairs" and "Stable for scour conditions" (MaineDOT, 2021).

Photo 1- Upstream View of Dover Bridge

The Dover Bridge design team is also working on a preliminary geotechnical design study. The preliminary geotechnical design included subsurface investigations, geotechnical laboratory testing including grain size analysis, and the preparation of a preliminary geotechnical design report. This hydrology and hydraulic study relied on the field observations and grain size analyses conducted as part of the preliminary geotechnical study.

2. Existing Data Review

This study relied on several readily available data sources to establish the basis for the hydrologic and hydraulic analysis including the Federal Emergency Management agency (FEMA) Flood Insurance Study (FIS) (1993), FEMA Flood Maps (FEMA, 2021), United States Geological Survey (USGS) streamflow gage data, USGS StreamStats, and information on Upper Dam (i.e., Moosehead Dam) and Lower Dam (i.e., Browns Mill Dam) provided by Kruger Energy by email on September 20, 2021. Reference materials also included a FEMA request for hydraulic modeling for the Piscataquis River along the reach that includes Dover Bridge.

The FEMA FIS (1993) and FEMA Flood Maps include information on the extent and depth of flooding for the 100-year annual recurrence interval flows. The FEMA FIRMette and a flood profile from the FIS are included in Appendix A. Mapping by FEMA also indicates that, during the 100-year event, flows in the Piscataquis River would split just upstream of Upper Dam and East Main Street with minor flows separating from the main channel at the right bank, continuing south of the town of Dover-Foxcroft, and reconnecting with the river south of Dover Bridge and Lower Dam. The FEMA FIS report indicates peak flow at the Dover Bridge of 33,200 cfs for the 100-year annual recurrence interval event. The FEMA FIS estimated this flow would result in a peak water surface elevation of 336.4 ft North American Vertical Datum of 1988 (NAVD88) (Fig. 5). Under the National Geodetic Vertical Datum of 1929 (NGVD29), the elevation at Dover Bridge for the 100-year event would be 337.0 ft. NAVD88 is the reference datum for elevations in this report. Based on the NOAA National Geodetic Survey (NGS) online Coordinate Conversion and Transformation Tool (NCAT) NAVD88 = NGVD29 - 0.56 ft (NOAA, 2021).

To inform hydraulic model development, a Flood Insurance Study Data Request was submitted to FEMA for available hydrologic and hydraulic models of the Piscataquis River. The information was provided in the form of a 49-page pdf of input and output data developed in 1991 using the Natural Resources Conservation Service's Computer Program for Water Surface Profiles (WSP). The WSP2 model data provided an understanding of the channel geometry developed by FEMA and helped refine the channel in the digital elevation model (i.e., terrain) used for the hydraulic analysis.

Historic streamflow data was reviewed for USGS Gage Number 01031500, Piscataquis River near Dover-Foxcroft. This gage has a period of record that spans from 1903 to present for the 298 square-mile watershed (USGS, 2021a) and is located about 5.2 miles upstream of Dover Bridge. The USGS gage data were evaluated using the USGS flood frequency software PeakFQ (USGS, 2019) and compared with the FEMA peak flows to develop an understanding of annual recurrence interval flows at Dover Bridge.

The online USGS program StreamStats (USGS, 2021b) was also accessed to review flows and watershed area for Dover Bridge (Fig. 6). StreamStats indicated a watershed for Dover Bridge of approximately 354 square miles. The PeakFQ output and StreamStats Report are included in Appendix A.

Kruger Energy provided several studies and drawings, which were used to guide setup of the dam geometry in the hydraulic model. Information included a dam breach analysis study prepared for Moosehead Dam (Upper Dam) by Kleinschmidt Associates (1993), a headwater rating curve plot for Browns Mill Dam (Kruger Energy, 2021), a 1995 Site Plan for Browns Mills Dam prepared by Rivers Engineering Corporation, and 1993 Topographic Survey of Browns Mills Dam prepared by Gregory W. Crispell Co., Inc. The 1993 dam breach study was used to understand the spillway crest elevation and width at Upper Dam. The headwater rating curve was input to the hydraulic model as a user defined curve to establish the stage discharge relationship at Lower Dam (Browns Mill Dam). The plan and survey were used to evaluate elevations at Browns Mills Dam.

3. Hydrology

The design team developed hydrology for the flood of record, which occurred on April 1, 1987, and for a range of annual recurrence interval flows that included the 1.1, 10, 25, 50, 100, 500-year events.

MaineDOT prepared a comparison of flows at the Dover Bridge based on the FEMA FIS, USGS gage 0103500, and flood flow frequency analysis following USGS Bulletin 17C. The analysis indicated that the FEMA FIS (1993) produced the highest flows at Dover Bridge. The 1993 FIS flows, shown in FEMA Table 1 below, were confirmed by GEI for use in this study. MaineDOT interpolated the additional recurrence interval flows to develop the range of flows included in Table 2, Hydrology.

Table 1. FEMA Summary of Discharges

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
PISCATAQUIS RIVER					
At downstream corporate limits	368.4	20,900	30,700	34,500	43,100
At East Dover Road	358.7	20,500	30,000	33,600	41,900
At Essex Street	352.2	20,200	29,500	33,200	41,300
At State Route 15	344.1	19,900	29,100	32,700	40,800
At upstream corporate limits	340.8	19,600	28,800	32,400	40,400

Table 2. Hydrology

SUMMARY	
Drainage Area	352.2 mi^2
Q1.1	6,800 ft^3/s
Q10	20,200 ft^3/s
Q25	25,500 ft^3/s
Q50	29,500 ft^3/s
Q100	33,200 ft^3/s
Q500	41,300 ft^3/s
Flood of Record	42,290 ft^3/s

Notes:
1. Drainage area based on FEMA FIS (1993).
2. Annual recurrence interval flows provided by MaineDOT and based on FEMA FIS 1993.
3. Flood of Record occurred on April 1, 1987. Method of derivation: T > 500 by 17C EMA; T = 120 by simple plotting position, a=0.

4. Hydraulic Analysis

This study included hydraulic analysis to estimate peak water surface elevation and velocity at Dover Bridge for a range of flows and for the existing and replacement bridge configurations. The hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center River Analysis System (HEC-RAS) version 6.1.0 software (USACE, 2021). The model was used to simulate 2-dimensional (2D) flows with unsteady analysis, the diffusion wave equation set, and a 1-second fixed computation interval.

The model simulates flow over an area of approximately 0.85 square miles and along a 1.7-mile reach of the Piscataquis River. The flow area extends approximately 0.9 miles upstream and 0.8 miles downstream of the Dover Bridge and includes two hydraulic structures at Upper and Lower Dams. The purpose of the 2D model was to capture flow conditions including shallow flow and split flows across the reach of interest. The model grid size was set to 50 ft by 50 ft to optimize model accuracy, stability, and run time. HEC-RAS 2D creates an elevation-volume relationship for each cell based on the details of the underlying terrain, an attribute not common to other 2D models of which some assume a single elevation and a flat bottom for each cell in the model.

The digital elevation model (DEM) (i.e., terrain) for the model was compiled from three data sources: a survey of the channel bottom at the bridge with 1 ft contours, a 1-meter LiDAR survey prepared in 2015 and distributed by USGS (2020), and FEMA cross-sections of the river channel. The DEM elevations were referenced to North American Vertical Datum of 1988 (NAVD 88). Breaklines were input to align the cell faces of the mesh to linear features such as the dams and bridge.

The 2D flow area included spatially varied Manning's n-values based on the 2016 National Land Cover Database (NLCD) for the Conterminous United States (MRLC, 2019). Manning's n-values were assigned to land cover groups based on Chow (1959), USGS (2015), Jarrett (1984), Trieste and Jarrett (1987), Costa and Jarrett (2008), Grant (1997), Reid and Hickin (2008), and Wahl (1994).

The existing 6-span bridge has five piers with a width of about 6 ft at the base and 4 ft near the low chord of the deck. Based on information provided by the design team, the existing bridge was input as a total opening width of 252.9 ft near the low chord and an opening of 250.4 ft near the base of the abutments. The proposed 3-span bridge was input to the hydraulic model with vertical abutments, an opening width of 257 ft, and two 5-ft-wide, vertical piers. The existing and proposed bridges used in the modeling are included in Appendix B.

Table 3 below summarizes the water surface elevations computed by the hydraulic model for the range of flows evaluated as part of the analysis.

Table 3. Flow and Water Surface Elevations at Dover Bridge

		Existing Structure 6-span	Proposed Structure 3-span
Total Area of Waterway Opening	ft^2	4,426	4,705
Headwater elevation at $Q_{1.1}$	ft	331.0	331.0
Headwater elevation at Q_{25}	ft	334.8	334.7
Headwater elevation at Q_{50}	ft	335.6	335.5
Headwater elevation at Q_{100}	ft	336.4	336.2
Headwater elevation at Q_{500}	ft	337.8	337.6
Flood of Record (April 1987)	ft	338.0	337.8
Low chord elevation	ft	338.7	338.7
Freeboard at Q_{50}	ft	3.1	3.2
Freeboard at Q_{100}	ft	2.3	2.5
Velocity at $Q_{1.1}$	ft/s	3.2	2.8
Velocity at Q_{25}	ft/s	8.9	7.8
Velocity at Q_{50}	ft/s	9.8	8.6
Velocity at Q_{100}	ft/s	10.5	9.3
Velocity at Q_{500}	ft/s	12.0	10.6
Flood of Record (April 1987)	ft/s	12.1	10.7

Notes:
1. Headwater elevations in Table 3 are based on hydraulic model results for maximum water surface elevation along profile line upstream of Dover Bridge.
2. Velocity values in Table 3 are based on hydraulic model results for maximum velocity along profile line at center of Dover Bridge.
3. Annual recurrence interval flows based on FEMA FIS 1993.
4. Flood of Record occurred on April 1, 1987. Method of derivation: T > 500 by 17C EMA; T = 120 by simple plotting position, a=0.
5. Elevations based on North American Vertical Datum of 1988 (NAVD 88). Conversion: NAVD88 = NGVD29 - 0.56 ft (NOAA, 2021).
6. Replacement bridge configuration based on proposed design.
7. Dover Bridge low chord assumed to be at EL. 338.7 ft for freeboard calculations.

The 2D hydraulic model results indicated a peak water surface at elevation 336.4 ft at the profile line just upstream of Dover Bridge for the 100-year event. These results compare well with the FEMA elevation of 336.4 ft for the 100-year event developed in 1991 with the previous WSP2 model. However, despite the match in water surface elevations, a difference in results would not be unexpected since there are likely differences in the hydraulic treatment of the dam located about 70 ft downstream of the bridge as well as technical advances in modeling since 1991 including 2D HEC-RAS modeling software, terrain development, and other current modeling and routing methods.

The structures evaluated for this work included the existing 6-span structure and a proposed 3-span structure. We understand that a proposed 2-span structure is also under consideration. The results of the analyses for the existing 6-span and proposed 3-span bridges indicated an approximately 0.2 ft or less increase in freeboard due to the reduction in number of piers from five for the 6-span structure to two for the 3-span structure. While a proposed 2-span structure was not evaluated as part of this work, we anticipate that results for a 2-span structure would not significantly differ from the 3-span results, with expected increases in freeboard of about 0.1 ft (assuming the same low chord elevation for all configurations). Similarly, the velocities for a 2-span structure would likely be slightly less than the 3-span velocities. If the 2-span structure is the preferred bridge configuration, the water elevations and velocities should be confirmed with a hydraulic model during final design.

5. Scour Analysis

Scour analysis was performed at the proposed Dover Bridge for potential pier and contraction scour for the proposed 3-span bridge. Because the Dover Bridge will be replaced, the existing bridge was not analyzed for scour. Online bridge data maintained by MaineDOT indicate the channel and scour conditions at the existing dam as "Bank protection needs minor repairs" and "Stable for scour conditions" (MaineDOT, 2021).

During geotechnical field work, scour was observed at the piers just below the waterline (Photo 2). Significant erosion or scour was not observed at the bridge abutments at the time of the field visit.

Photo 2- Upstream Side of Abutments Looking Southwest

The scour estimates were developed based on Hydraulic Circular No. 18 (HEC-18) under the U.S. Department of Transportation, Federal Highway Administration (FHWA, 2012).

During boring exploration under the preliminary geotechnical investigation, which is summarized in a separate report, the driller was not able to recover material from the channel bed but was able to retrieve samples from the abutments. Borings at the location of the proposed abutments and lab analysis for grain size indicated a D_{50}, the median particle diameter, of 2.5 mm at a depth of 15 to 17 ft below the road surface on the east bank and a D_{50} of 0.2 mm at a depth of 10 to 12 ft below the road surface on the west bank. Shallower, finer-grained samples at the abutments were not evaluated for scour since they appeared to be fill and unrepresentative of the channel bottom or banks.

Pier and contraction scour were estimated using a D_{50} of 2.5 mm. Calculations are included in Appendix C. Abutment scour was not evaluated because the proposed abutment layout had not been developed at the time of this study. It is recommended that abutment scour be estimated when final design information becomes available. Contraction and pier scour for the proposed 3-span Dover Bridge are summarized in Table 4.

Table 4. Scour Analysis

Calculation	100-year Event	500-year Event
Critical Velocity (ft/s)	12.3	12.5
Mean Velocity (ft/s) at Dover Bridge	8.5	9.7
Mean Velocity (ft/s) upstream Dover Bridge	4.3	5.0
D_{50} (mm)	2.5	2.5
Clear Water Contraction Scour (ft)	20.3	25.9
Pier Scour (ft)	8.9	9.5

Notes:
1. Scour depths estimated based on D_{50} of 2.5 mm and do not account for shallower bedrock.
2. The potential for rock scour was not evaluated as part of this study.

Of note, and as indicated by Hodgkins and Lombard (USGS, 2002) the "HEC-18 pier-scour equations are intended to be envelope equations, ideally never underpredicting scour depths and not appreciably overpredicting them. The 2002 study of pier scour in Maine considered the HEC-18 equations to perform well in Maine where twenty-two out of twenty-three pier-scour depths were overpredicted by depths of 0.7 ft to 18.3 ft, with the study indicating one underprediction by 4.5 ft." Hodgkins and Lombard (USGS, 2008) also developed a study in 2008 titled "Comparison of Observed and Predicted Scour at Selected Bridges in Maine," which the design team recommends be consulted when the abutment scour estimates are developed.

6. Summary of Findings and Recommendations

- The hydraulic characteristics of the river in the vicinity of the Dover Bridge indicate that water levels are strongly influenced by the upstream and downstream dams.

- Assuming the proposed low chord for the replacement 3-span bridge matches the existing low chord elevation of 338.7 ft, the proposed clearance would be about 3.2 ft for the 50-year event and 2.5 ft for the 100-year event, based on the proposed 3-span bridge design information provided by the design team and the hydraulic model results.

- The MaineDOT Bridge Design Guide (2003), Structure Capacity (Riverine) indicates that “Major riverine bridges" must be designed with a freeboard depth of 4 ft for the 50-year event (Q50). All bridge-type structures should be capable of passing the Q100 or the flood of record, whichever is greater. When possible, there should be 1 ft of freeboard for the 100-year event (Q100).

    - At Dover Bridge, the Q50 appears to be the limiting event for setting the low chord elevation.

    - The model results indicate the proposed 3-span bridge configuration can pass the flood of record, which is greater than the Q100 and has a peak water surface at El. 337.8 ft.

- Potential contraction scour depth for the proposed 3-span bridge design was estimated to be 20.3 ft below the streambed for the 100-year event, and 25.9 ft below the streambed for the 500-year event, assuming material capable of being scoured along the entire column. The pier scour depth was estimated to be about 8.9 ft for the 100-year event, and 9.5 ft for the 500-year event. These scour depths were based on a D_{50} of 2.5 mm. Results will be different for different D_{50} values. In boring BB-DFPR-102 performed through the river near the middle of the existing bridge, bedrock was encountered 3.5 ft below top of river sediment (29 ft below top of existing bridge deck). At abutment borings BB-DFPR-101 and BB-DFPR-103, bedrock was encountered at 19 ft and 20 ft below top of road, respectively. These data indicate that bedrock is likely the limiting factor for pier and contraction scour at the boring locations. Scour can be mitigated through installation of designed scour protection, such as heavy riprap or articulated mats.

- At the time of preparation of this study, GEI was also working on preliminary geotechnical design report under a separate task from this hydrology and hydraulic study.

- Abutment scour should be estimated once abutment designs become available. Designs should be developed to address potential scour, as applicable.

- Scour protection should be developed during the final design phase.

- While a proposed 2-span structure was not evaluated as part of this work, the results of the hydraulic analysis for the existing 6-span and proposed 3-span structures indicate that a proposed 2-span structure would likely result in slightly lower headwater elevation, greater freeboard, and lower velocity than the proposed 3-span and existing 6-span structure, assuming the same low chord elevation.

7. Limitations

This report presents the results of hydrologic and hydraulic analysis for the proposed Dover Bridge replacement. The results are based on readily available online information and the proposed bridge design information provided by the design team at the time of this report and may require modification if there are any changes in the nature, design, and/or location of the data or proposed design. It is recommended 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 borings. It was not possible to evaluate sediments in the main channel due to lack of material retrieval during boring exploration. Furthermore, 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 other purposes, in part or in whole, is at the sole risk of the user.

8. References

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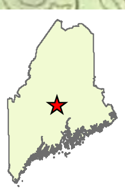
Figures



Dover Bridge Replacement Project Dover-Foxcroft, Maine WIN 23120.00	GEI Consultants	SITE LOCATION MAP
Thornton Tomasetti Portland, Maine	Project 2103596	January 2023 Fig. 1

\\geiconsultants.com\data\Data_Storage\Working\THORNTON TOMASETTI\2103596 Dover Bridge

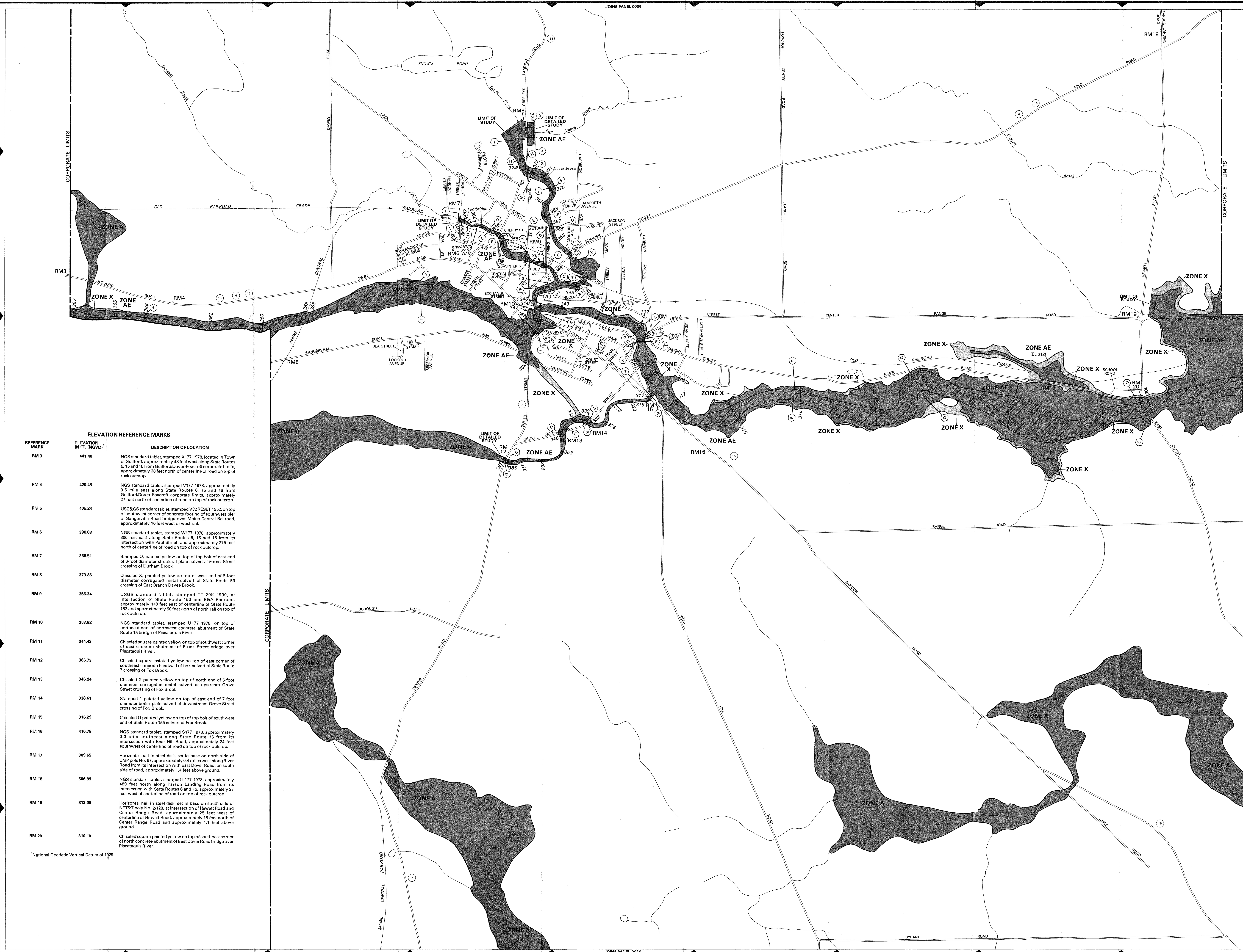
Dover Bridge Replacement Project Dover-Foxcroft, Maine WIN 23120.00	GEI Consultants	PROJECT SITE MAP
Thornton Tomasetti Portland, Maine	Project 2103596	January 2023 Fig. 2

\\bos1v-FS02\Data_Storage\Working\THORNTON TOMASETTI\2103596 Dover Bridge

Appendix A

Hydrologic Data

A.1 FEMA Data



ELEVATION REFERENCE MARKS

REFERENCE MARK	ELEVATION IN FT. (NGVD)[1]	DESCRIPTION OF LOCATION
RM 3	441.40	NGS standard tablet, stamped X177 1978, located in Town of Guilford, approximately 48 feet west along State Routes 6, 15 and 16 from Guilford/Dover-Foxcroft corporate limits, approximately 28 feet north of centerline of road on top of rock outcrop.
RM 4	420.45	NGS standard tablet, stamped V177 1978, approximately 0.5 mile east along State Routes 6, 15 and 16 from Guilford/Dover-Foxcroft corporate limits, approximately 27 feet north of centerline of road on top of rock outcrop.
RM 5	405.24	USC&GS standard tablet, stamped V32 RESET 1952, on top of southwest corner of concrete footing of southwest pier of Sangerville Road bridge over Maine Central Railroad, approximately 10 feet west of west rail.
RM 6	398.03	NGS standard tablet, stampd W177 1978, approximately 300 feet east along State Routes 6, 15 and 16 from its intersection with Paul Street, and approximately 275 feet north of centerline of road on top of rock outcrop.
RM 7	368.51	Stamped O, painted yellow on top of top bolt of east end of 6-foot diameter structural plate culvert at Forest Street crossing of Durham Brook.
RM 8	373.86	Chiseled X, painted yellow on top of west end of 5-foot diameter corrugated metal culvert at State Route 53 crossing of East Branch Davee Brook.
RM 9	356.34	USGS standard tablet, stamped TT 20K 1930, at intersection of State Route 153 and B&A Railroad, approximately 140 feet east of centerline of State Route 153 and approximately 50 feet north of north rail on top of rock outcrop.
RM 10	353.82	NGS standard tablet, stamped U177 1978, on top of northeast end of northwest concrete abutment of State Route 15 bridge of Piscataquis River.
RM 11	344.43	Chiseled square painted yellow on top of southwest corner of east concrete abutment of Essex Street bridge over Piscataquis River.
RM 12	386.73	Chiseled square painted yellow on top of east corner of southeast concrete headwall of box culvert at State Route 7 crossing of Fox Brook.
RM 13	346.94	Chiseled X painted yellow on top of north end of 5-foot diameter corrugated metal culvert at upstream Grove Street crossing of Fox Brook.
RM 14	338.61	Stamped 1 painted yellow on top of east end of 7-foot diameter boiler plate culvert at downstream Grove Street crossing of Fox Brook.
RM 15	316.29	Chiseled O painted yellow on top of top bolt of southwest end of State Route 155 culvert at Fox Brook.
RM 16	410.78	NGS standard tablet, stamped S177 1978, approximately 0.3 mile southeast along State Route 15 from its intersection with Bear Hill Road, approximately 24 feet southwest of centerline of road on top of rock outcrop.
RM 17	309.65	Horizontal nail in steel disk, set in base on north side of CMP pole No. 67, approximately 0.4 miles west along River Road from its intersection with East Dover Road, on south side of road, approximately 1.4 feet above ground.
RM 18	506.89	NGS standard tablet, stamped L177 1978, approximately 480 feet north along Parson Landing Road from its intersection with State Routes 6 and 16, approximately 27 feet west of centerline of road on top of rock outcrop.
RM 19	313.09	Horizontal nail in steel disk, set in base on south side of NET&T pole No. 2/128, at intersection of Hewett Road and Center Range Road, approximately 25 feet west of centerline of Hewett Road, approximately 18 feet north of Center Range Road and approximately 1.1 feet above ground.
RM 20	310.10	Chiseled square painted yellow on top of southeast corner of north concrete abutment of East Dover Road bridge over Piscataquis River.

[1]National Geodetic Vertical Datum of 1929.

LEGEND

SPECIAL FLOOD HAZARD AREAS INUNDATED BY 100-YEAR FLOOD

- **ZONE A** No base flood elevations determined.
- **ZONE AE** Base flood elevations determined.
- **ZONE AH** Flood depths of 1 to 3 feet (usually areas of ponding); base flood elevations determined.
- **ZONE AO** Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding; velocities also determined.
- **ZONE A99** To be protected from 100-year flood by Federal flood protection system under construction; no base flood elevations determined.
- **ZONE V** Coastal flood with velocity hazard (wave action); no base flood elevations determined.
- **ZONE VE** Coastal flood with velocity hazard (wave action); base flood elevations determined.

FLOODWAY AREAS IN ZONE AE

OTHER FLOOD AREAS

- **ZONE X** Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.

OTHER AREAS

- **ZONE X** Areas determined to be outside 500-year floodplain.
- **ZONE D** Areas in which flood hazards are undetermined.

UNDEVELOPED COASTAL BARRIERS†

Identified 1983 — Identified 1990 — Otherwise Protected Areas

†Coastal barrier areas are normally located within or adjacent to special flood hazard areas.

- Floodplain Boundary
- Floodway Boundary
- Zone D Boundary
- Boundary Dividing Special Flood Hazard Zones, and Boundary Dividing Areas of Different Coastal Base Flood Elevations Within Special Flood Hazard Zones.
- 513 — Base Flood Elevation Line; Elevation in Feet*
- D — D — Cross Section Line
- (EL 987) — Base Flood Elevation in Feet Where Uniform Within Zone*
- RM 7x — Elevation Reference Mark
- •M1.5 — River Mile

*Referenced to the National Geodetic Vertical Datum of 1929

NOTES

This map is for use in administering the National Flood Insurance Program; it does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size, or all planimetric features outside Special Flood Hazard Areas. The community map repository should be consulted for possible updated flood hazard information prior to use of this map for property purchase or construction purposes.

Coastal base flood elevations apply only landward of 0.0 NGVD, and include the effects of wave action; these elevations may also differ significantly from those developed by the National Weather Service for hurricane evacuation planning.

Areas of special flood hazard (100-year flood) include Zones A, AE, AH, AO, A99, V, and VE.

Certain areas not in Special Flood Hazard Areas may be protected by flood control structures.

Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the Federal Emergency Management Agency.

Floodway widths in some areas may be too narrow to show to scale. Floodway widths are provided in the Flood Insurance Study Report.

For adjoining map panels see separately printed Map Index.

MAP REPOSITORY
Town Office, 34 East Main Street, Dover-Foxcroft, Maine 04426 (Maps available for reference only, not for distribution).

INITIAL IDENTIFICATION:
DECEMBER 17, 1976

FLOOD HAZARD BOUNDARY MAP REVISIONS:
NONE

FLOOD INSURANCE RATE MAP EFFECTIVE:
JULY 2, 1979

FLOOD INSURANCE RATE MAP REVISIONS:
April 2, 1993-to change base flood elevations, to add base flood elevations, to add special flood hazard areas, to change special flood hazard areas, to change zone designations, to update map format, and to reflect updated topographic information.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at (800) 638-6620.

APPROXIMATE SCALE
1000 0 1000 FEET

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

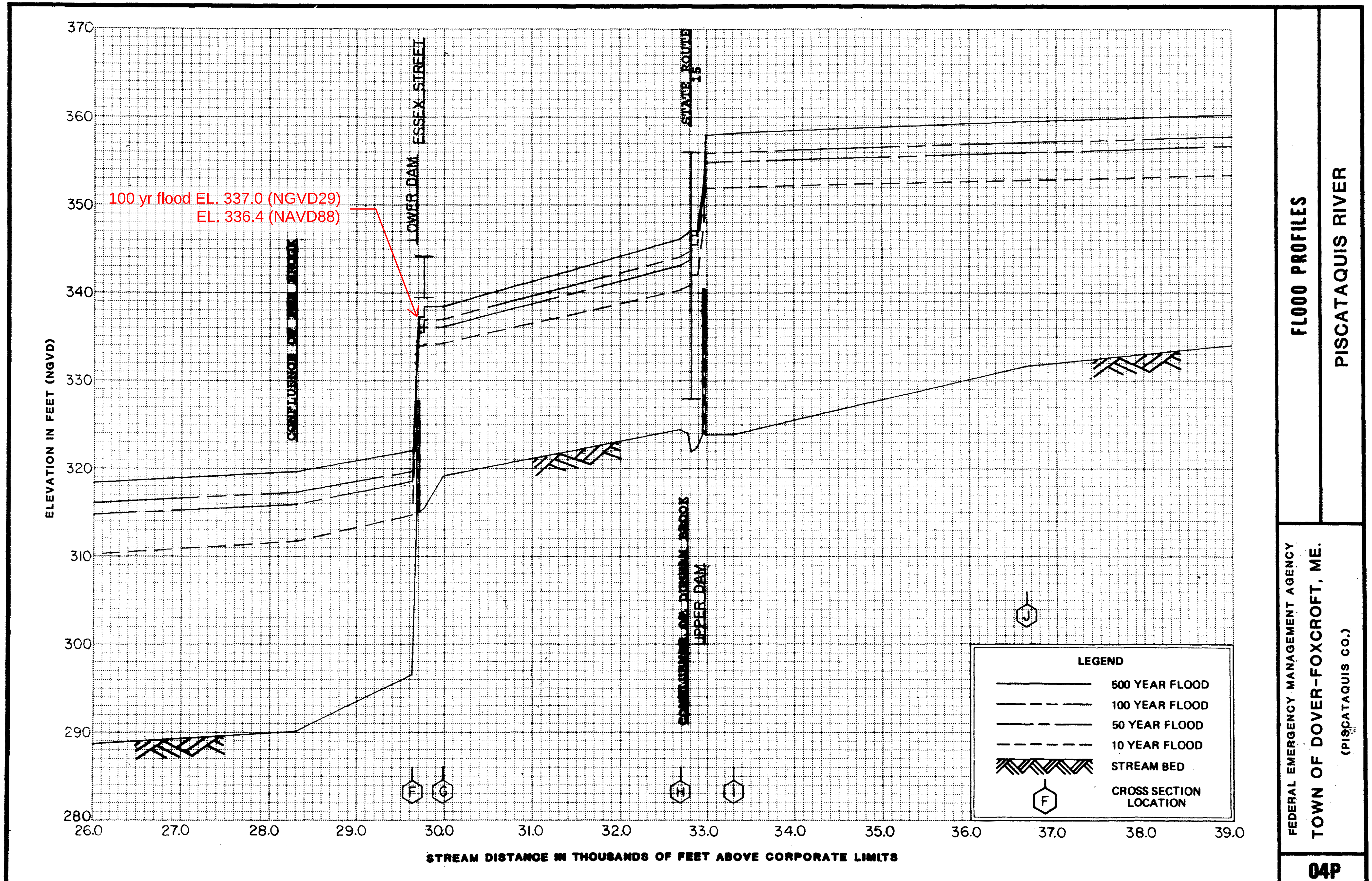
TOWN OF
DOVER-FOXCROFT, MAINE
PISCATAQUIS COUNTY

PANEL 10 OF 25
(SEE MAP INDEX FOR PANELS NOT PRINTED)

COMMUNITY-PANEL NUMBER
230116 0010 B

MAP REVISED:
APRIL 2, 1993

Federal Emergency Management Agency


100 yr flood EL. 337.0 (NGVD29)
EL. 336.4 (NAVD88)
ELEVATION IN FEET (NGVD)
370
360
350
340
330
320
310
300
290
280
LOWER DAM
ESSEX STREET
STATE ROUTE 15
UPPER DAM
F
G
H
I
J
26.0
27.0
28.0
29.0
30.0
31.0
32.0
33.0
34.0
35.0
36.0
37.0
38.0
39.0
STREAM DISTANCE IN THOUSANDS OF FEET ABOVE CORPORATE LIMITS
LEGEND
500 YEAR FLOOD
100 YEAR FLOOD
50 YEAR FLOOD
10 YEAR FLOOD
STREAM BED
CROSS SECTION LOCATION
FLOOD PROFILES
PISCATAQUIS RIVER
FEDERAL EMERGENCY MANAGEMENT AGENCY
TOWN OF DOVER-FOXCROFT, ME.
(PISCATAQUIS CO.)
04P

A.2 USGS Peak FQ

```
1
  Program PeakFq           U. S. GEOLOGICAL SURVEY            Seq.002.000
  Version 7.3         Annual peak flow frequency analysis      Run Date / Time
  10/25/2019                                                    09/20/2021 17:14

                         --- PROCESSING OPTIONS ---

                      Plot option         = Graphics device
                      Basin char output   = None
                      Print option        = Yes
                      Debug print         = No
                      Input peaks listing = Long
                      Input peaks format  = WATSTORE peak file

                      Input files used:
                         peaks (ascii)  - C:\Users\lrobinson\OneDrive - GEI
Consultants, Inc\Documents\PEAK.TXT

                         specifications - C:\Users\lrobinson\OneDrive - GEI
Consultants, Inc\Documents\PKFQWPSF.TMP

                      Output file(s):
                         main - C:\Users\lrobinson\OneDrive - GEI Consultants,
Inc\Documents\PEAK.PRT


  ***  User responsible for assessment and interpretation of the following analysis
 ***

1


  Program PeakFq           U. S. GEOLOGICAL SURVEY            Seq.001.001
  Version 7.3         Annual peak flow frequency analysis      Run Date / Time
  10/25/2019                                                    09/20/2021 17:14

        Station - 01031500  Piscataquis River near Dover-Foxcroft, Maine


                     TABLE 1 - INPUT DATA SUMMARY

                Number of peaks in record            =      117
                Peaks not used in analysis           =        0
                Gaged peaks in analysis              =      117
                Historic peaks in analysis           =        0
                Beginning Year                       =     1903
                Ending Year                          =     2019
                Historical Period Length             =      117
                Skew option                          = STATION SKEW
                Regional skew                        =   --
```

```
          Standard error                    =     --
          Mean Square error                 =     --
     Gage base discharge                    =      0.0
     User supplied high outlier threshold =   --
     User supplied PILF (LO) criterion    =   --
     Plotting position parameter          =     0.00
     Type of analysis                              EMA
     PILF (LO) Test Method                        MGBT
     Perceptible Ranges:
         Start Year  End Year  Lower Bound  Upper Bound
               1903      2019          0.0          INF    DEFAULT

     Interval Data                     =   None Specified
```

TABLE 2 - DIAGNOSTIC MESSAGE AND PILF RESULTS

```
*WCF107I-ACCEPTED GEN SKEW OUTSIDE MAP LIMITS.-999.000  -0.400   0.800
*WCF151I-17B WEIGHTED SKEW REPLACED BY USER OPTION.  -136.165    -0.036  -1
 EMA002W-CONFIDENCE INTERVALS ARE NOT EXACT IF HISTORIC PERIOD > 0
```

```
MULTIPLE GRUBBS-BECK TEST RESULTS
MULTIPLE GRUBBS-BECK PILF THRESHOLD     N/A
NUMBER OF PILFS IDENTIFIED                0
```

Kendall's Tau Parameters

	TAU	P-VALUE	MEDIAN SLOPE	No. of PEAKS
GAGED PEAKS	0.176	0.005	28.750	117

1

Station - 01031500 Piscataquis River near Dover-Foxcroft, Maine

TABLE 3 - ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

LOGARITHMIC

	MEAN	STANDARD DEVIATION	SKEW
EMA WITHOUT REG SKEW	3.9345	0.2050	-0.036
EMA WITH REG SKEW	3.9345	0.2050	-0.036

EMA ESTIMATE OF MSE OF SKEW WITHOUT REG SKEW 0.0477
EMA ESTIMATE OF MSE OF SKEW W/GAGED PEAKS ONLY (AT-SITE) 0.0477

TABLE 4 - ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

ANNUAL EXCEEDANCE PROBABILITY	<- EMA ESTIMATE -> WITH REG SKEW	WITHOUT REG SKEW	<- FOR EMA ESTIMATE WITH REG SKEW -> LOG VARIANCE OF EST.	<-CONFIDENCE LIMITS-> 5.0% LOWER	95.0% UPPER
0.9950	2510.	2510.	0.0030	1874.0	3009.0
0.9900	2833.	2833.	0.0022	2215.0	3313.0
0.9500	3938.	3938.	0.0009	3398.0	4384.0
0.9000	4688.	4688.	0.0006	4186.0	5141.0
0.8000	5786.	5786.	0.0005	5288.0	6279.0
0.6667	7034.	7034.	0.0004	6486.0	7605.0
0.5000	8624.	8624.	0.0004	7978.0	9322.0
0.4292	9381.	9381.	0.0004	8682.0	10150.0
0.2000	12800.	12800.	0.0005	11810.0	13980.0
0.1000	15720.	15720.	0.0007	14370.0	17530.0
0.0400	19530.	19530.	0.0012	17510.0	22750.0
0.0200	22460.	22460.	0.0018	19750.0	27180.0
0.0100	25460.	25460.	0.0026	21890.0	32090.0
0.0050	28550.	28550.	0.0036	23950.0	37520.0
0.0020	32770.	32770.	0.0052	26570.0	45630.0

*Note: If Station Skew option is selected then EMA ESTIMATE WITH REG SKEW will display values for and be equal to EMA ESTIMATE WITHOUT REG SKEW.

1

Station - 01031500 Piscataquis River near Dover-Foxcroft, Maine

TABLE 5 - INPUT DATA LISTING

WATER YEAR	PEAK VALUE	PEAKFQ CODES	FLOW INTERVALS (WHERE LOWER BOUND NOT = UPPER BOUND) LOWER BOUND	UPPER BOUND	REMARKS
1903	5140.0				
1904	7420.0				
1905	2410.0				
1906	10400.0				
1907	8040.0				
1908	10100.0				
1909	17400.0				
1910	4010.0				
1911	4110.0				
1912	7380.0				
1913	7130.0				
1914	6930.0				
1915	6100.0				
1916	6200.0				
1917	14600.0				
1918	5960.0				
1919	4710.0				
1920	8650.0				
1921	7600.0				
1922	8350.0				
1923	21500.0				
1924	8690.0				
1925	4570.0				
1926	8040.0				
1927	7780.0				
1928	10400.0				
1929	9600.0				
1930	8040.0				
1931	6870.0				
1932	12900.0				
1933	5960.0				
1934	8040.0				
1935	5590.0				
1936	19300.0				
1937	6750.0				
1938	8110.0				
1939	6240.0				
1940	13700.0				
1941	4010.0				
1942	6970.0				
1943	4680.0				
1944	13500.0				
1945	7190.0				
1946	5300.0				
1947	11600.0				
1948	9640.0				
1949	3100.0				
1950	11300.0				

1951	17400.0
1952	9310.0
1953	15200.0
1954	13200.0
1955	9560.0
1956	5150.0
1957	2990.0
1958	13300.0
1959	5250.0
1960	7190.0
1961	4920.0
1962	5420.0
1963	6810.0
1964	14000.0
1965	5250.0
1966	5050.0
1967	22800.0
1968	8990.0
1969	7460.0
1970	12800.0
1971	13400.0
1972	6950.0
1973	10500.0
1974	19200.0
1975	6830.0
1976	10200.0
1977	7190.0
1978	10800.0
1979	19300.0
1980	5960.0
1981	10300.0
1982	9220.0
1983	18800.0
1984	12700.0
1985	3290.0
1986	8860.0
1987	37300.0
1988	7010.0
1989	7990.0
1990	7140.0
1991	8970.0
1992	7470.0
1993	12100.0
1994	13100.0
1995	4020.0
1996	11200.0
1997	14300.0
1998	9880.0
1999	8320.0
2000	12600.0

2001	8820.0
2002	4600.0
2003	3280.0
2004	11000.0
2005	12600.0
2006	13900.0
2007	13000.0
2008	15200.0
2009	10300.0
2010	9380.0
2011	13900.0
2012	7650.0
2013	12500.0
2014	14000.0
2015	8400.0
2016	10900.0
2017	8460.0
2018	7780.0
2019	9100.0

Explanation of peak discharge qualification codes

PeakFQ CODE	NWIS CODE	DEFINITION
D	3	Dam failure, non-recurrent flow anomaly
G	8	Discharge greater than stated value
X	3+8	Both of the above
L	4	Discharge less than stated value
K	6 OR C	Known effect of regulation or urbanization
O	O	Opportunistic peak
H	7	Historic peak

- Minus-flagged discharge -- Not used in computation
  -8888.0 -- No discharge value given
- Minus-flagged water year -- Historic peak used in computation

1

Station - 01031500 Piscataquis River near Dover-Foxcroft, Maine

TABLE 6 - EMPIRICAL FREQUENCY CURVES -- HIRSCH-STEDINGER PLOTTING POSITIONS

WATER YEAR	RANKED DISCHARGE	EMA ESTIMATE	FLOW INTERVALS (WHERE LOWER BOUND NOT = UPPER BOUND) LOWER BOUND	UPPER BOUND
1987	37300.0	0.0085		
1967	22800.0	0.0169		
1923	21500.0	0.0254		
1936	19300.0	0.0424		
1979	19300.0	0.0339		
1974	19200.0	0.0508		
1983	18800.0	0.0593		
1909	17400.0	0.0763		
1951	17400.0	0.0678		
1953	15200.0	0.0932		
2008	15200.0	0.0847		
1917	14600.0	0.1017		
1997	14300.0	0.1102		
1964	14000.0	0.1271		
2014	14000.0	0.1186		
2006	13900.0	0.1441		
2011	13900.0	0.1356		
1940	13700.0	0.1525		
1944	13500.0	0.1610		
1971	13400.0	0.1695		
1958	13300.0	0.1780		
1954	13200.0	0.1864		
1994	13100.0	0.1949		
2007	13000.0	0.2034		
1932	12900.0	0.2119		
1970	12800.0	0.2203		
1984	12700.0	0.2288		
2000	12600.0	0.2458		
2005	12600.0	0.2373		
2013	12500.0	0.2542		
1993	12100.0	0.2627		
1947	11600.0	0.2712		
1950	11300.0	0.2797		
1996	11200.0	0.2881		
2004	11000.0	0.2966		
2016	10900.0	0.3051		
1978	10800.0	0.3136		
1973	10500.0	0.3220		
1906	10400.0	0.3390		
1928	10400.0	0.3305		
1981	10300.0	0.3559		
2009	10300.0	0.3475		
1976	10200.0	0.3644		
1908	10100.0	0.3729		
1998	9880.0	0.3814		

1948	9640.0	0.3898
1929	9600.0	0.3983
1955	9560.0	0.4068
2010	9380.0	0.4153
1952	9310.0	0.4237
1982	9220.0	0.4322
2019	9100.0	0.4407
1968	8990.0	0.4492
1991	8970.0	0.4576
1986	8860.0	0.4661
2001	8820.0	0.4746
1924	8690.0	0.4831
1920	8650.0	0.4915
2017	8460.0	0.5000
2015	8400.0	0.5085
1922	8350.0	0.5169
1999	8320.0	0.5254
1938	8110.0	0.5339
1907	8040.0	0.5678
1926	8040.0	0.5593
1930	8040.0	0.5508
1934	8040.0	0.5424
1989	7990.0	0.5763
1927	7780.0	0.5932
2018	7780.0	0.5847
2012	7650.0	0.6017
1921	7600.0	0.6102
1992	7470.0	0.6186
1969	7460.0	0.6271
1904	7420.0	0.6356
1912	7380.0	0.6441
1945	7190.0	0.6695
1960	7190.0	0.6610
1977	7190.0	0.6525
1990	7140.0	0.6780
1913	7130.0	0.6864
1988	7010.0	0.6949
1942	6970.0	0.7034
1972	6950.0	0.7119
1914	6930.0	0.7203
1931	6870.0	0.7288
1975	6830.0	0.7373
1963	6810.0	0.7458
1937	6750.0	0.7542
1939	6240.0	0.7627
1916	6200.0	0.7712
1915	6100.0	0.7797
1918	5960.0	0.8051
1933	5960.0	0.7966
1980	5960.0	0.7881

1935	5590.0	0.8136
1962	5420.0	0.8220
1946	5300.0	0.8305
1959	5250.0	0.8475
1965	5250.0	0.8390
1956	5150.0	0.8559
1903	5140.0	0.8644
1966	5050.0	0.8729
1961	4920.0	0.8814
1919	4710.0	0.8898
1943	4680.0	0.8983
2002	4600.0	0.9068
1925	4570.0	0.9153
1911	4110.0	0.9237
1995	4020.0	0.9322
1910	4010.0	0.9492
1941	4010.0	0.9407
1985	3290.0	0.9576
2003	3280.0	0.9661
1949	3100.0	0.9746
1957	2990.0	0.9831
1905	2410.0	0.9915

1

Station - 01031500 Piscataquis River near Dover-Foxcroft, Maine

TABLE 7 - EMA REPRESENTATION OF DATA

WATER	<----- OBSERVED ---->		<-------- EMA ------->		<---- USER-ENTERED PERCEPTIBLE RANGES ->		<-------- FINAL PERCEPTIBLE RANGES ->	
YEAR	Q_LOWER	Q_UPPER	Q_LOWER	Q_UPPER	LOWER	UPPER	LOWER	UPPER
1903	5140.0	5140.0	5140.0	5140.0	0.0	INF	0.0	INF
1904	7420.0	7420.0	7420.0	7420.0	0.0	INF	0.0	INF
1905	2410.0	2410.0	2410.0	2410.0	0.0	INF	0.0	INF
1906	10400.0	10400.0	10400.0	10400.0	0.0	INF	0.0	INF
1907	8040.0	8040.0	8040.0	8040.0	0.0	INF	0.0	INF

```
   1908    10100.0    10100.0    10100.0    10100.0          0.0         INF
0.0        INF
   1909    17400.0    17400.0    17400.0    17400.0          0.0         INF
0.0        INF
   1910     4010.0     4010.0     4010.0     4010.0          0.0         INF
0.0        INF
   1911     4110.0     4110.0     4110.0     4110.0          0.0         INF
0.0        INF
   1912     7380.0     7380.0     7380.0     7380.0          0.0         INF
0.0        INF
   1913     7130.0     7130.0     7130.0     7130.0          0.0         INF
0.0        INF
   1914     6930.0     6930.0     6930.0     6930.0          0.0         INF
0.0        INF
   1915     6100.0     6100.0     6100.0     6100.0          0.0         INF
0.0        INF
   1916     6200.0     6200.0     6200.0     6200.0          0.0         INF
0.0        INF
   1917    14600.0    14600.0    14600.0    14600.0          0.0         INF
0.0        INF
   1918     5960.0     5960.0     5960.0     5960.0          0.0         INF
0.0        INF
   1919     4710.0     4710.0     4710.0     4710.0          0.0         INF
0.0        INF
   1920     8650.0     8650.0     8650.0     8650.0          0.0         INF
0.0        INF
   1921     7600.0     7600.0     7600.0     7600.0          0.0         INF
0.0        INF
   1922     8350.0     8350.0     8350.0     8350.0          0.0         INF
0.0        INF
   1923    21500.0    21500.0    21500.0    21500.0          0.0         INF
0.0        INF
   1924     8690.0     8690.0     8690.0     8690.0          0.0         INF
0.0        INF
   1925     4570.0     4570.0     4570.0     4570.0          0.0         INF
0.0        INF
   1926     8040.0     8040.0     8040.0     8040.0          0.0         INF
0.0        INF
   1927     7780.0     7780.0     7780.0     7780.0          0.0         INF
0.0        INF
   1928    10400.0    10400.0    10400.0    10400.0          0.0         INF
0.0        INF
   1929     9600.0     9600.0     9600.0     9600.0          0.0         INF
0.0        INF
   1930     8040.0     8040.0     8040.0     8040.0          0.0         INF
0.0        INF
   1931     6870.0     6870.0     6870.0     6870.0          0.0         INF
0.0        INF
   1932    12900.0    12900.0    12900.0    12900.0          0.0         INF
0.0        INF
```

1933	5960.0	5960.0	5960.0	5960.0	0.0	INF	0.0	INF
1934	8040.0	8040.0	8040.0	8040.0	0.0	INF	0.0	INF
1935	5590.0	5590.0	5590.0	5590.0	0.0	INF	0.0	INF
1936	19300.0	19300.0	19300.0	19300.0	0.0	INF	0.0	INF
1937	6750.0	6750.0	6750.0	6750.0	0.0	INF	0.0	INF
1938	8110.0	8110.0	8110.0	8110.0	0.0	INF	0.0	INF
1939	6240.0	6240.0	6240.0	6240.0	0.0	INF	0.0	INF
1940	13700.0	13700.0	13700.0	13700.0	0.0	INF	0.0	INF
1941	4010.0	4010.0	4010.0	4010.0	0.0	INF	0.0	INF
1942	6970.0	6970.0	6970.0	6970.0	0.0	INF	0.0	INF
1943	4680.0	4680.0	4680.0	4680.0	0.0	INF	0.0	INF
1944	13500.0	13500.0	13500.0	13500.0	0.0	INF	0.0	INF
1945	7190.0	7190.0	7190.0	7190.0	0.0	INF	0.0	INF
1946	5300.0	5300.0	5300.0	5300.0	0.0	INF	0.0	INF
1947	11600.0	11600.0	11600.0	11600.0	0.0	INF	0.0	INF
1948	9640.0	9640.0	9640.0	9640.0	0.0	INF	0.0	INF
1949	3100.0	3100.0	3100.0	3100.0	0.0	INF	0.0	INF
1950	11300.0	11300.0	11300.0	11300.0	0.0	INF	0.0	INF
1951	17400.0	17400.0	17400.0	17400.0	0.0	INF	0.0	INF
1952	9310.0	9310.0	9310.0	9310.0	0.0	INF	0.0	INF
1953	15200.0	15200.0	15200.0	15200.0	0.0	INF	0.0	INF
1954	13200.0	13200.0	13200.0	13200.0	0.0	INF	0.0	INF
1955	9560.0	9560.0	9560.0	9560.0	0.0	INF	0.0	INF
1956	5150.0	5150.0	5150.0	5150.0	0.0	INF	0.0	INF
1957	2990.0	2990.0	2990.0	2990.0	0.0	INF	0.0	INF

1958	13300.0	13300.0	13300.0	13300.0	0.0	INF	0.0	INF
1959	5250.0	5250.0	5250.0	5250.0	0.0	INF	0.0	INF
1960	7190.0	7190.0	7190.0	7190.0	0.0	INF	0.0	INF
1961	4920.0	4920.0	4920.0	4920.0	0.0	INF	0.0	INF
1962	5420.0	5420.0	5420.0	5420.0	0.0	INF	0.0	INF
1963	6810.0	6810.0	6810.0	6810.0	0.0	INF	0.0	INF
1964	14000.0	14000.0	14000.0	14000.0	0.0	INF	0.0	INF
1965	5250.0	5250.0	5250.0	5250.0	0.0	INF	0.0	INF
1966	5050.0	5050.0	5050.0	5050.0	0.0	INF	0.0	INF
1967	22800.0	22800.0	22800.0	22800.0	0.0	INF	0.0	INF
1968	8990.0	8990.0	8990.0	8990.0	0.0	INF	0.0	INF
1969	7460.0	7460.0	7460.0	7460.0	0.0	INF	0.0	INF
1970	12800.0	12800.0	12800.0	12800.0	0.0	INF	0.0	INF
1971	13400.0	13400.0	13400.0	13400.0	0.0	INF	0.0	INF
1972	6950.0	6950.0	6950.0	6950.0	0.0	INF	0.0	INF
1973	10500.0	10500.0	10500.0	10500.0	0.0	INF	0.0	INF
1974	19200.0	19200.0	19200.0	19200.0	0.0	INF	0.0	INF
1975	6830.0	6830.0	6830.0	6830.0	0.0	INF	0.0	INF
1976	10200.0	10200.0	10200.0	10200.0	0.0	INF	0.0	INF
1977	7190.0	7190.0	7190.0	7190.0	0.0	INF	0.0	INF
1978	10800.0	10800.0	10800.0	10800.0	0.0	INF	0.0	INF
1979	19300.0	19300.0	19300.0	19300.0	0.0	INF	0.0	INF
1980	5960.0	5960.0	5960.0	5960.0	0.0	INF	0.0	INF
1981	10300.0	10300.0	10300.0	10300.0	0.0	INF	0.0	INF
1982	9220.0	9220.0	9220.0	9220.0	0.0	INF	0.0	INF

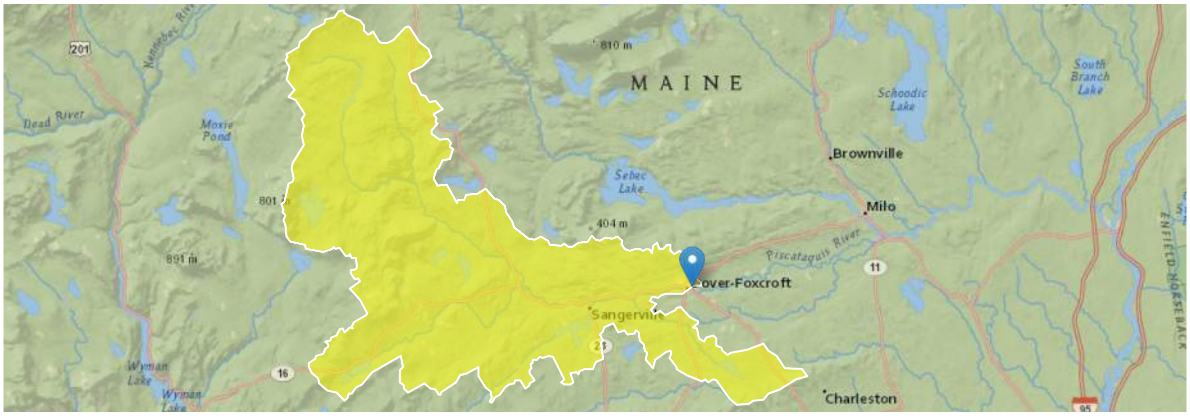
1983	18800.0	18800.0	18800.0	18800.0	0.0	INF	0.0	INF
1984	12700.0	12700.0	12700.0	12700.0	0.0	INF	0.0	INF
1985	3290.0	3290.0	3290.0	3290.0	0.0	INF	0.0	INF
1986	8860.0	8860.0	8860.0	8860.0	0.0	INF	0.0	INF
1987	37300.0	37300.0	37300.0	37300.0	0.0	INF	0.0	INF
1988	7010.0	7010.0	7010.0	7010.0	0.0	INF	0.0	INF
1989	7990.0	7990.0	7990.0	7990.0	0.0	INF	0.0	INF
1990	7140.0	7140.0	7140.0	7140.0	0.0	INF	0.0	INF
1991	8970.0	8970.0	8970.0	8970.0	0.0	INF	0.0	INF
1992	7470.0	7470.0	7470.0	7470.0	0.0	INF	0.0	INF
1993	12100.0	12100.0	12100.0	12100.0	0.0	INF	0.0	INF
1994	13100.0	13100.0	13100.0	13100.0	0.0	INF	0.0	INF
1995	4020.0	4020.0	4020.0	4020.0	0.0	INF	0.0	INF
1996	11200.0	11200.0	11200.0	11200.0	0.0	INF	0.0	INF
1997	14300.0	14300.0	14300.0	14300.0	0.0	INF	0.0	INF
1998	9880.0	9880.0	9880.0	9880.0	0.0	INF	0.0	INF
1999	8320.0	8320.0	8320.0	8320.0	0.0	INF	0.0	INF
2000	12600.0	12600.0	12600.0	12600.0	0.0	INF	0.0	INF
2001	8820.0	8820.0	8820.0	8820.0	0.0	INF	0.0	INF
2002	4600.0	4600.0	4600.0	4600.0	0.0	INF	0.0	INF
2003	3280.0	3280.0	3280.0	3280.0	0.0	INF	0.0	INF
2004	11000.0	11000.0	11000.0	11000.0	0.0	INF	0.0	INF
2005	12600.0	12600.0	12600.0	12600.0	0.0	INF	0.0	INF
2006	13900.0	13900.0	13900.0	13900.0	0.0	INF	0.0	INF
2007	13000.0	13000.0	13000.0	13000.0	0.0	INF	0.0	INF

```
   2008    15200.0    15200.0    15200.0    15200.0         0.0        INF
0.0        INF
   2009    10300.0    10300.0    10300.0    10300.0         0.0        INF
0.0        INF
   2010     9380.0     9380.0     9380.0     9380.0         0.0        INF
0.0        INF
   2011    13900.0    13900.0    13900.0    13900.0         0.0        INF
0.0        INF
   2012     7650.0     7650.0     7650.0     7650.0         0.0        INF
0.0        INF
   2013    12500.0    12500.0    12500.0    12500.0         0.0        INF
0.0        INF
   2014    14000.0    14000.0    14000.0    14000.0         0.0        INF
0.0        INF
   2015     8400.0     8400.0     8400.0     8400.0         0.0        INF
0.0        INF
   2016    10900.0    10900.0    10900.0    10900.0         0.0        INF
0.0        INF
   2017     8460.0     8460.0     8460.0     8460.0         0.0        INF
0.0        INF
   2018     7780.0     7780.0     7780.0     7780.0         0.0        INF
0.0        INF
   2019     9100.0     9100.0     9100.0     9100.0         0.0        INF
0.0        INF
1


 End PeakFQ analysis.
   Stations processed :       1
   Number of errors   :       0
   Stations skipped   :       0
   Station years      :     117


Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4,  or *.)
(2, 4, and * records are ignored.)

 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:  01031500       USGS Piscataquis River near Dover-


 For the station below, the following records were ignored:

 FINISHED PROCESSING STATION:
```

A.3 StreamStats

StreamStats Report

Region ID: ME
Workspace ID: ME20220118180104477000
Clicked Point (Latitude, Longitude): 45.18387, -69.21908
Time: 2022-01-18 13:01:32 -0500



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	353.81	square miles
I24H2Y	Maximum 24-hour precipitation that occurs on average once in 2 years - Equivalent to precipitation intensity index	2.96	inches
STORAGE	Percentage of area of storage (lakes ponds reservoirs wetlands)	10.206	percent
I24H5Y	Maximum 24-hour precipitation that occurs on average once in 5 years	3.63	inches
I24H10Y	Maximum 24-hour precipitation that occurs on average once in 10 years	4.19	inches
I24H25Y	Maximum 24-hour precipitation that occurs on average once in 25 years	4.95	inches
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	5.54	inches
I24H100Y	Maximum 24-hour precipitation that occurs on average once in 100 years	6.13	inches
I24H200Y	Maximum 24-hour precipitation that occurs on average once in 200 years	6.77	inches
I24H500Y	Maximum 24-hour precipitation that occurs on average once in 500 years	7.68	inches

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

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	353.81	square miles	0.26	5680
I24H2Y	24 Hour 2 Year Precipitation	2.96	inches	1.92	4.17
STORAGE	Percent Storage	10.206	percent	0	29.4
I24H5Y	24 Hour 5 Year Precipitation	3.63	inches	2.48	5.38
I24H10Y	24 Hour 10 Year Precipitation	4.19	inches	2.84	6.38
I24H25Y	24 Hour 25 Year Precipitation	4.95	inches	3.3	7.75
I24H50Y	24 Hour 50 Year Precipitation	5.54	inches	3.65	8.79
I24H100Y	24 Hour 100 Year Precipitation	6.13	inches	3.99	9.88
I24H200Y	24 Hour 200 YearPrecipitation	6.77	inches	5.26	11.1
I24H500Y	24 Hour 500 Year Precipitation	7.68	inches	5.95	13.1

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

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

Statistic	Value	Unit	PII	Plu	ASEp
50-percent AEP flood	7520	ft^3/s	4060	13900	39.1
20-percent AEP flood	10300	ft^3/s	5650	18800	38.1
10-percent AEP flood	12300	ft^3/s	6660	22700	38.9
4-percent AEP flood	14600	ft^3/s	7810	27300	39.9
2-percent AEP flood	16500	ft^3/s	8700	31300	39.7
1-percent AEP flood	18300	ft^3/s	9710	34500	40.7
0.5-percent AEP flood	19700	ft^3/s	10100	38300	42.8
0.2-percent AEP flood	21800	ft^3/s	11100	42900	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)

Application Version: 4.6.2
StreamStats Services Version: 1.2.22
NSS Services Version: 2.1.2

A.4 MaineDOT Hydrology Email

Robinson, Lissa

From: Hebson, Charles <Charles.Hebson@maine.gov>
Sent: Wednesday, October 27, 2021 2:25 PM
To: Stetson, Jason B
Cc: Robinson, Lissa
Subject: [EXT] Dover-Foxcroft 23120 Dover Br #5118 - Prelim Hydrology
Attachments: FIS-Dover-Foxcroft-1993.pdf

EXTERNAL EMAIL

Jason,

Here is preliminary hydrology suitable for starting the analysis. This is based on scaling the USGS gage 01031500 17C/EMA results upstream of the project. These results are larger than simple regression calculations. I will be sending a final report later. Numbers may be revised, but not significantly. The current FEMA Flood Insurance Study is dated 1993.

I recommend starting with the FEMA FIS values. The underlying hydrologic analysis is somewhat dated, but the estimates aren't much larger than Bull 17C results and the FIS is an established document. I would rather not design according to hydrology less than the governing FIS if we can avoid it. Let's see where this gets us; if this presents design problems, we can discuss.

Charlie

T (yrs)	Q (ft3/s) – Scaled Gage, by Bull 17C EMA	Q (ft3/s) – 1993 FIS
1.1	5,435	6,800
2	9,925	12,000
5	14,685	17,300
10	18,000	20,200
25	22,370	25,500
50	25,725	29,500
100	29,165	33,200
500	37,575	41,300
Flood of Record 04/01/1987	42,290 (T > 500 by 17C EMA; T = 120 by simple plotting position, a = 0)	

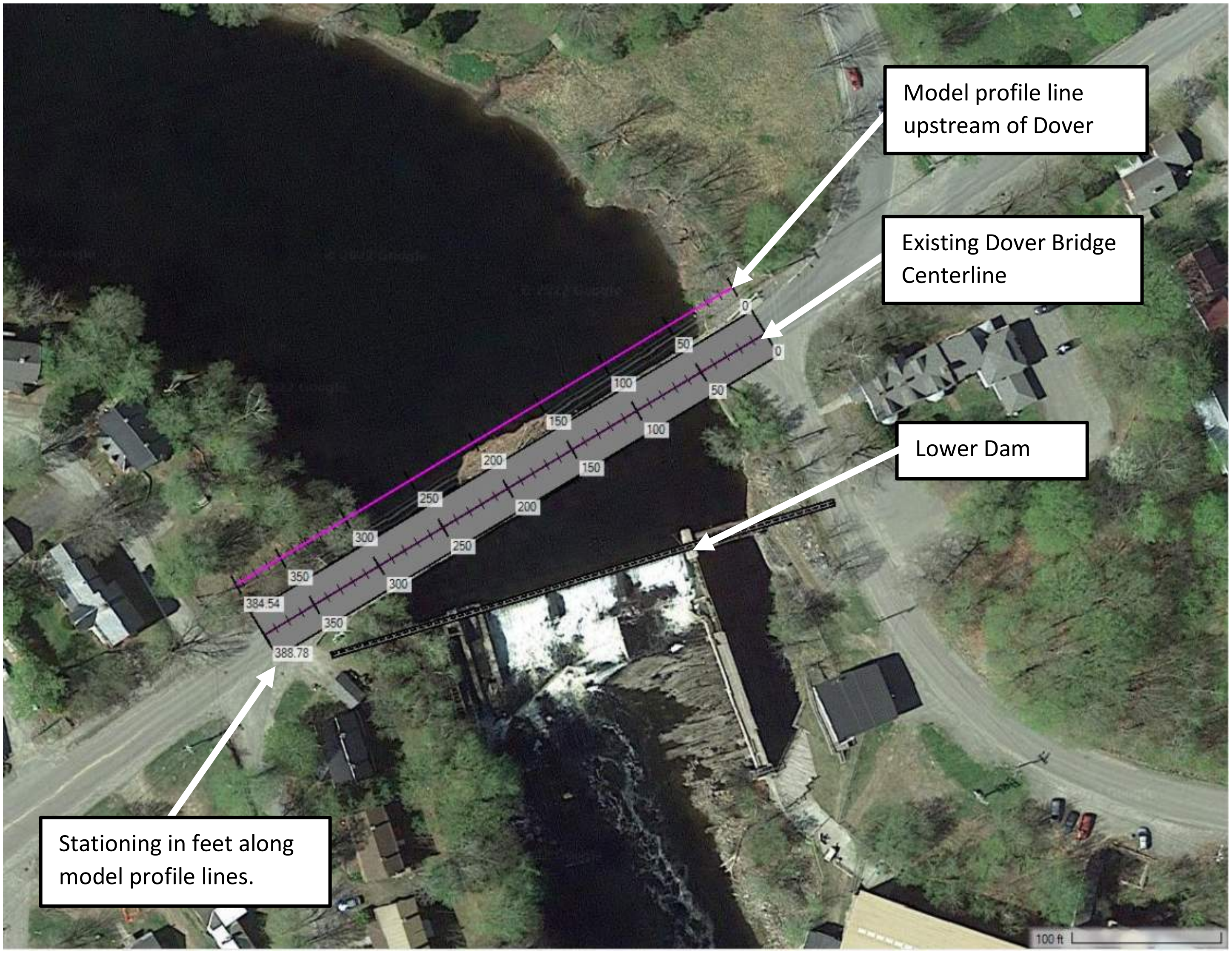
Charles Hebson, P.E.

MaineDOT / Environmental Office
State House Station 16
Augusta ME 04333-0016
207.557.1052

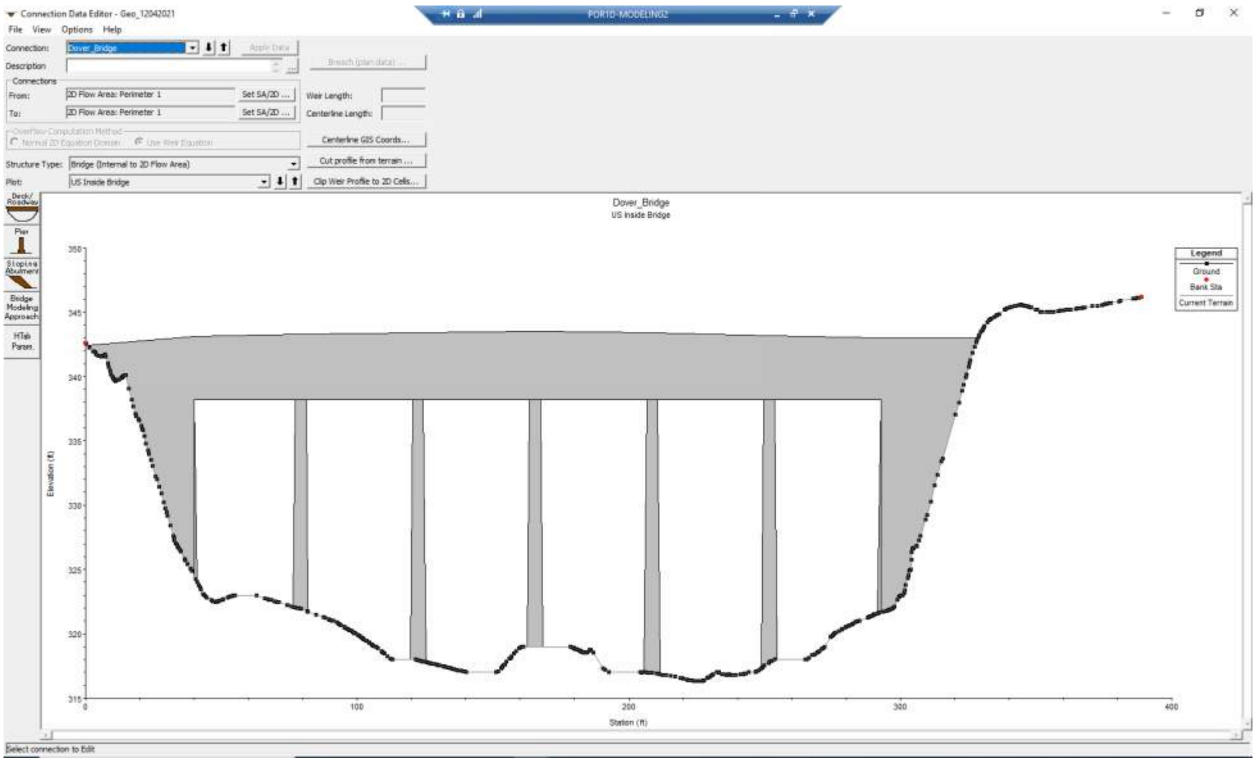
Appendix B

HEC-RAS Plan View and Water Surface Elevations at Dover Bridge

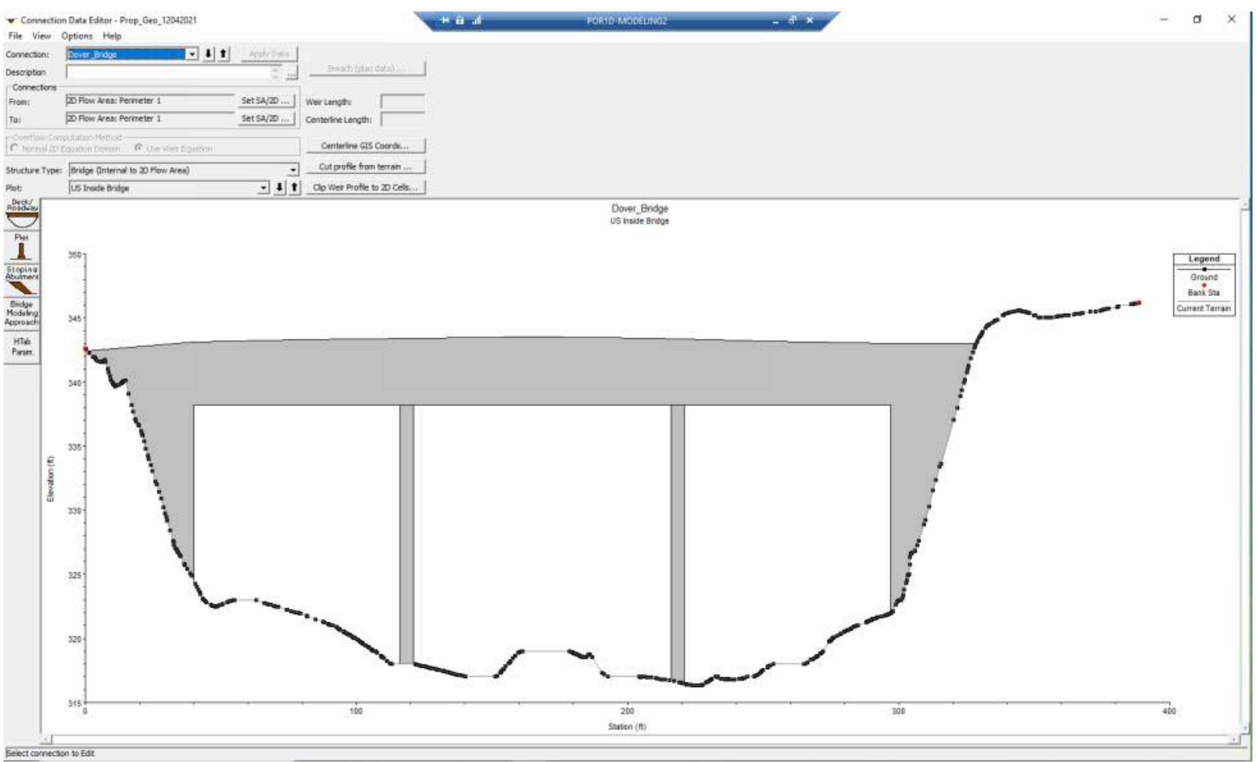
EXISTING DOVER BRIDGE - PLAN VIEW



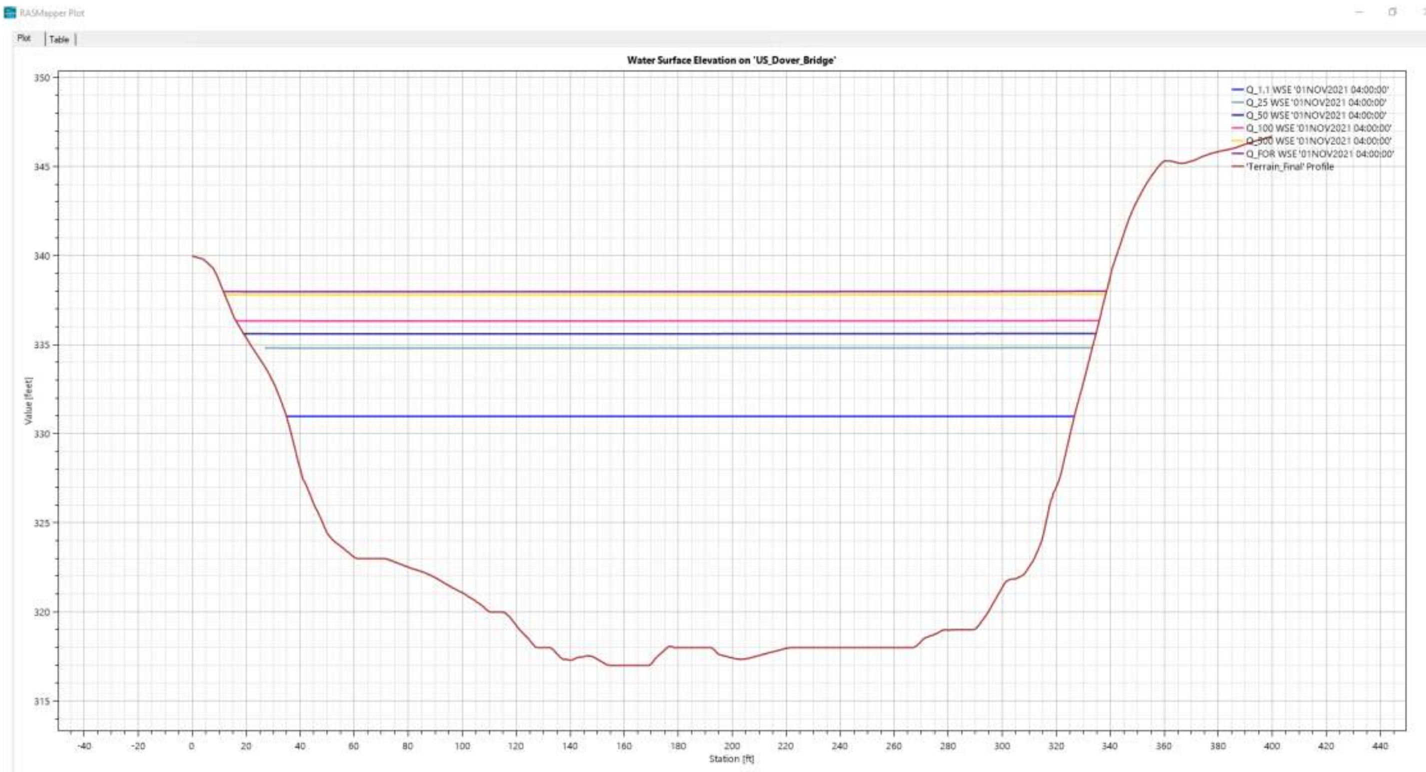
DOVER BRIDGE- EXISTING CONDITION



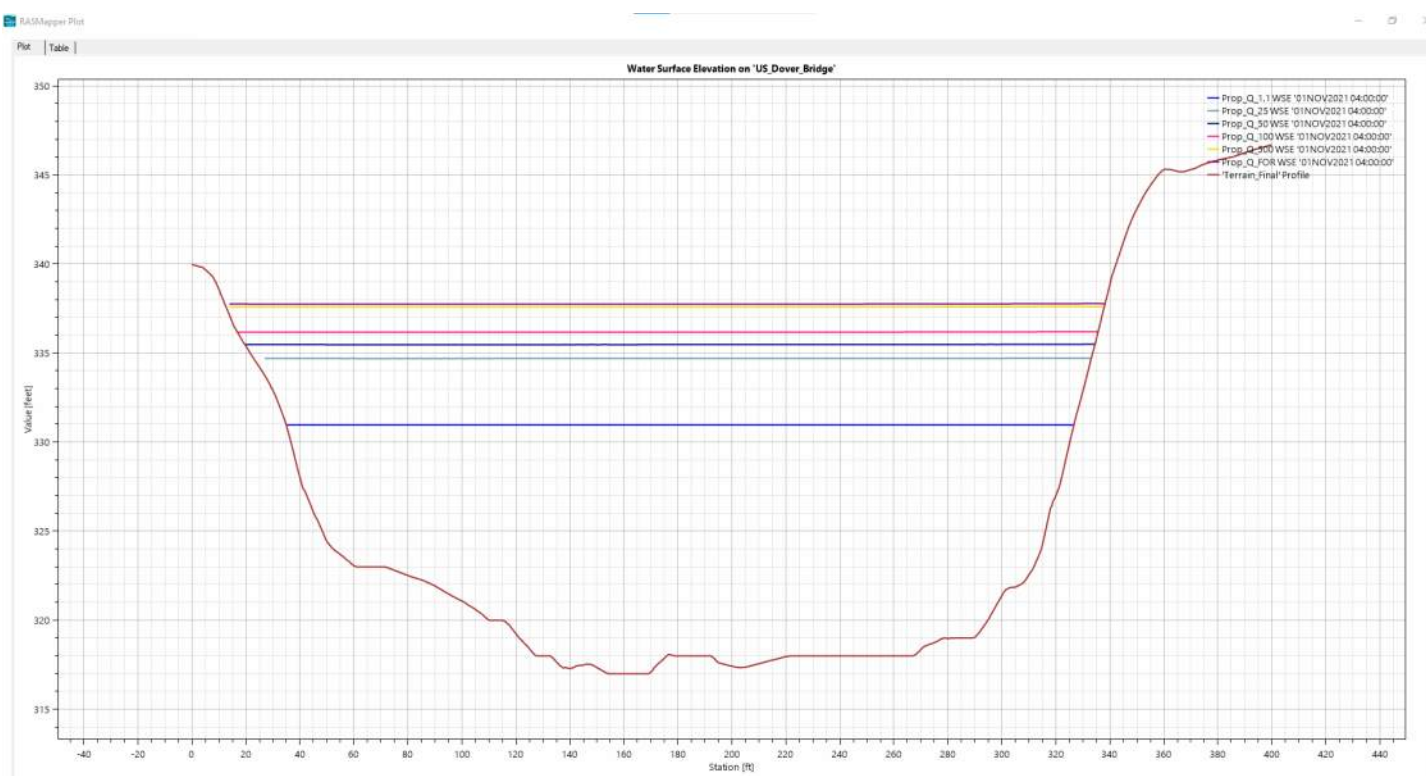
DOVER BRIDGE- PROPOSED CONDITION



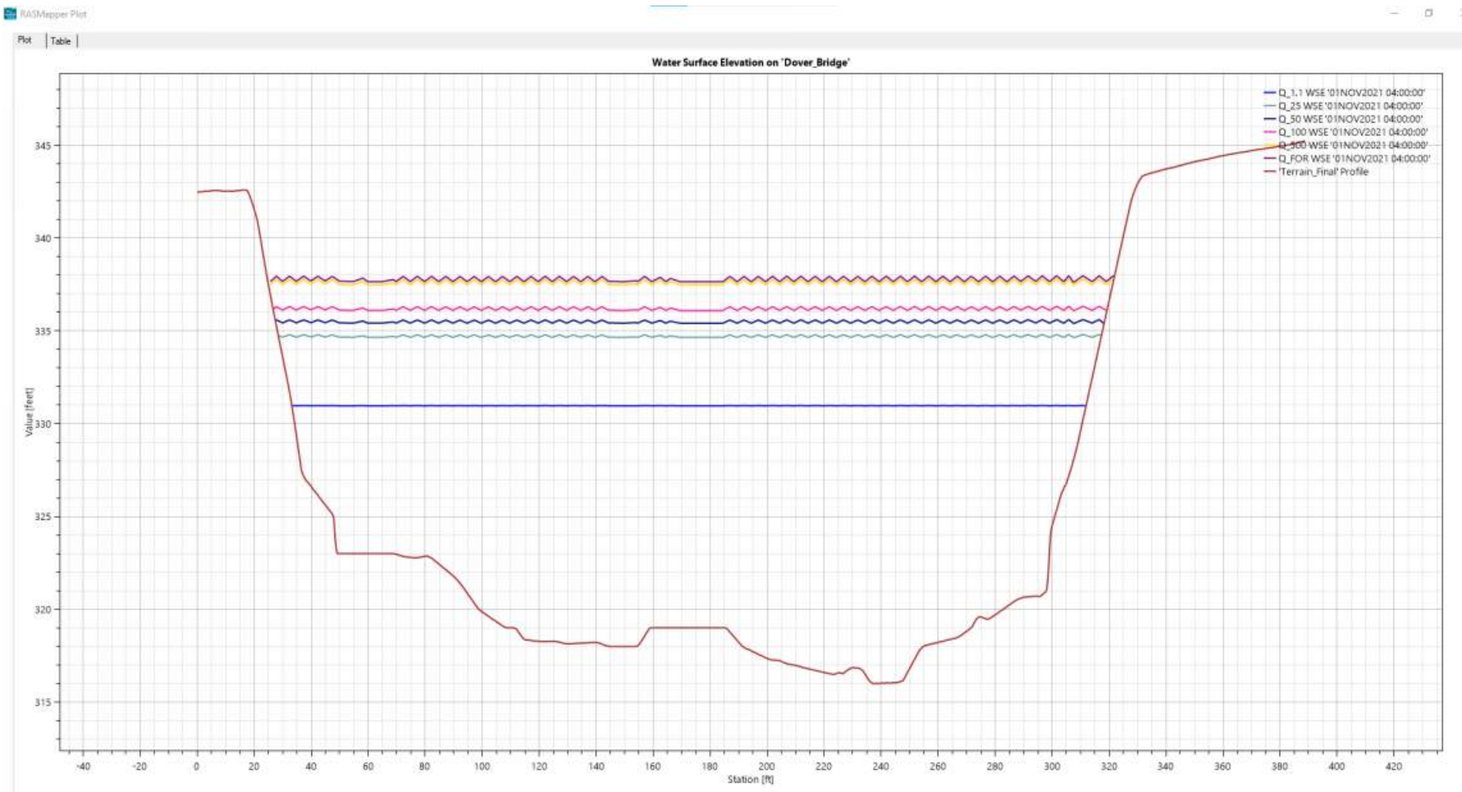
WATER SURFACE ELEVATIONS AT PROFILE LINE UPSTREAM OF DOVER BRIDGE - EXISTING CONDITION

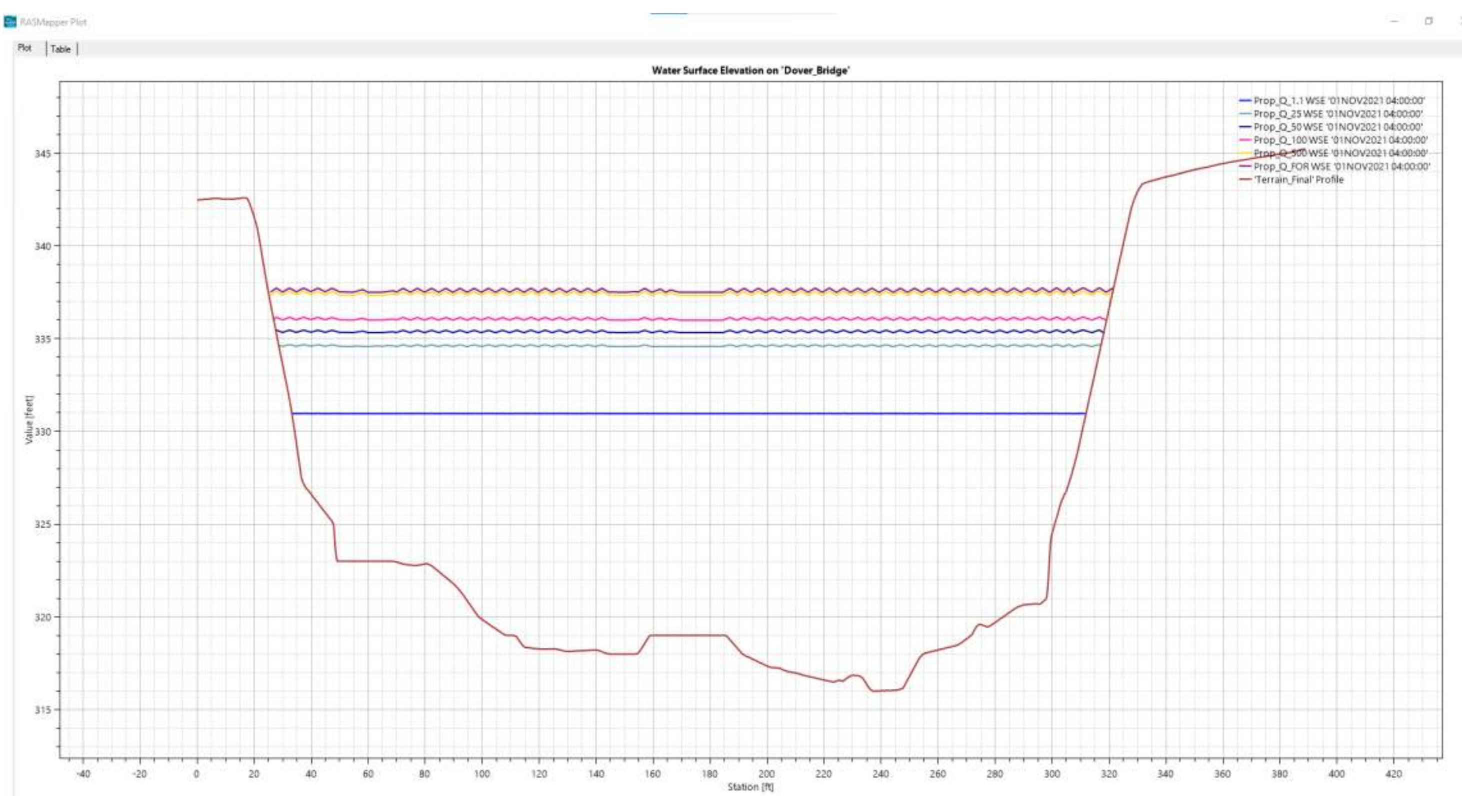


WATER SURFACE ELEVATIONS AT PROFILE LINE UPSTREAM OF DOVER BRIDGE- PROPOSED CONDITION

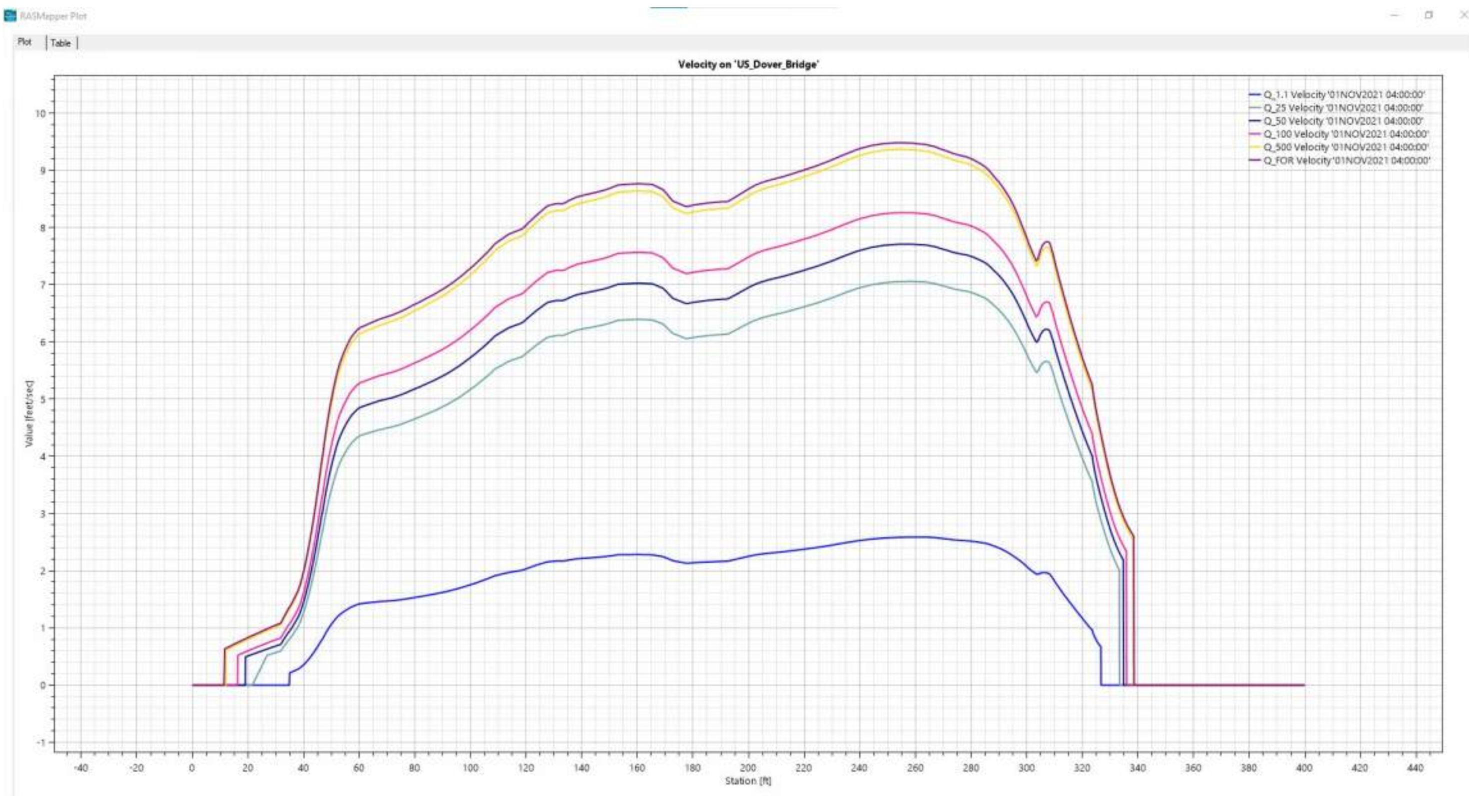


WATER SURFACE ELEVATIONS AT CENTERLINE OF DOVER BRIDGE - EXISTING CONDITION



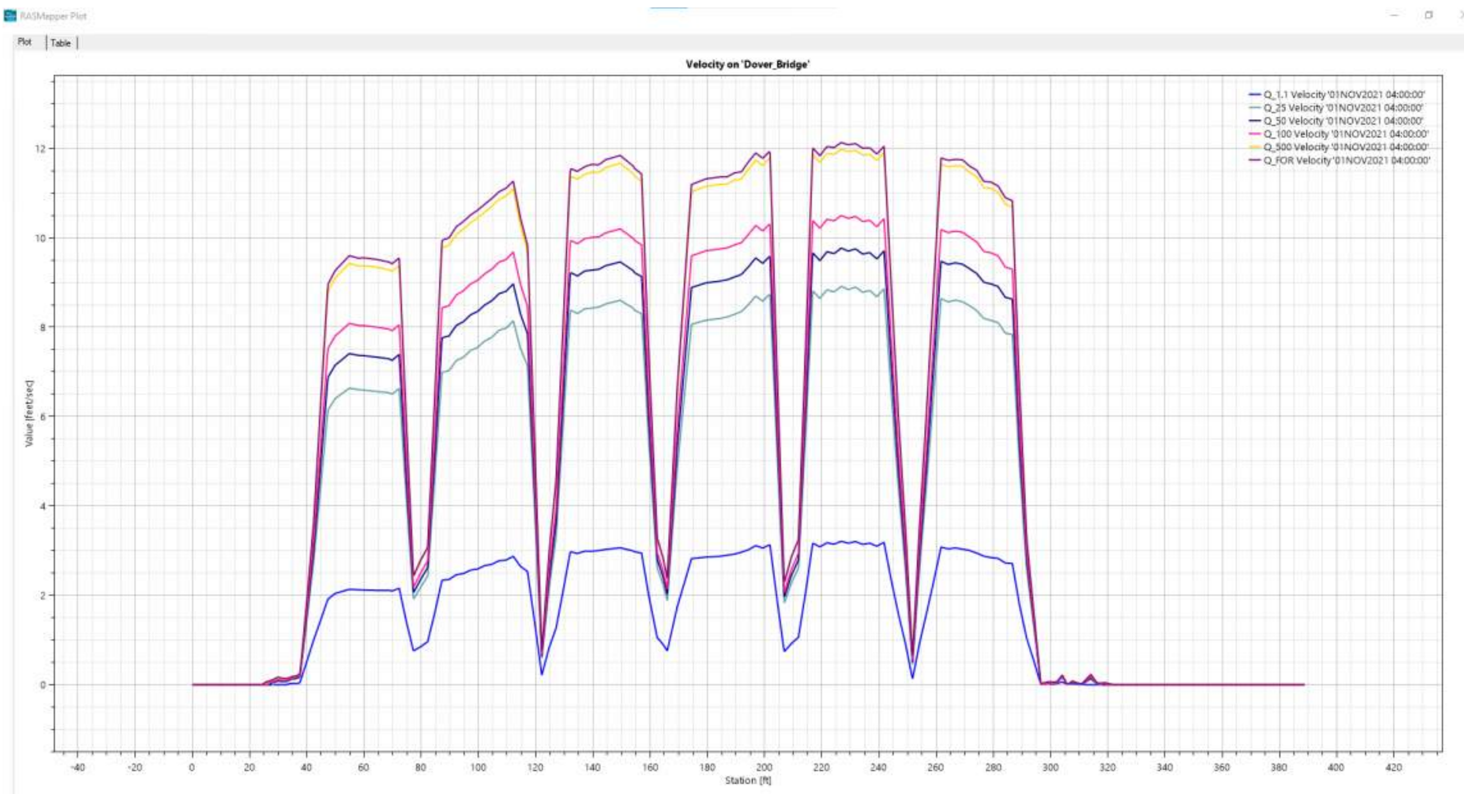


VELOCITY AT PROFILE LINE UPSTREAM OF DOVER BRIDGE - EXISTING CONDITION

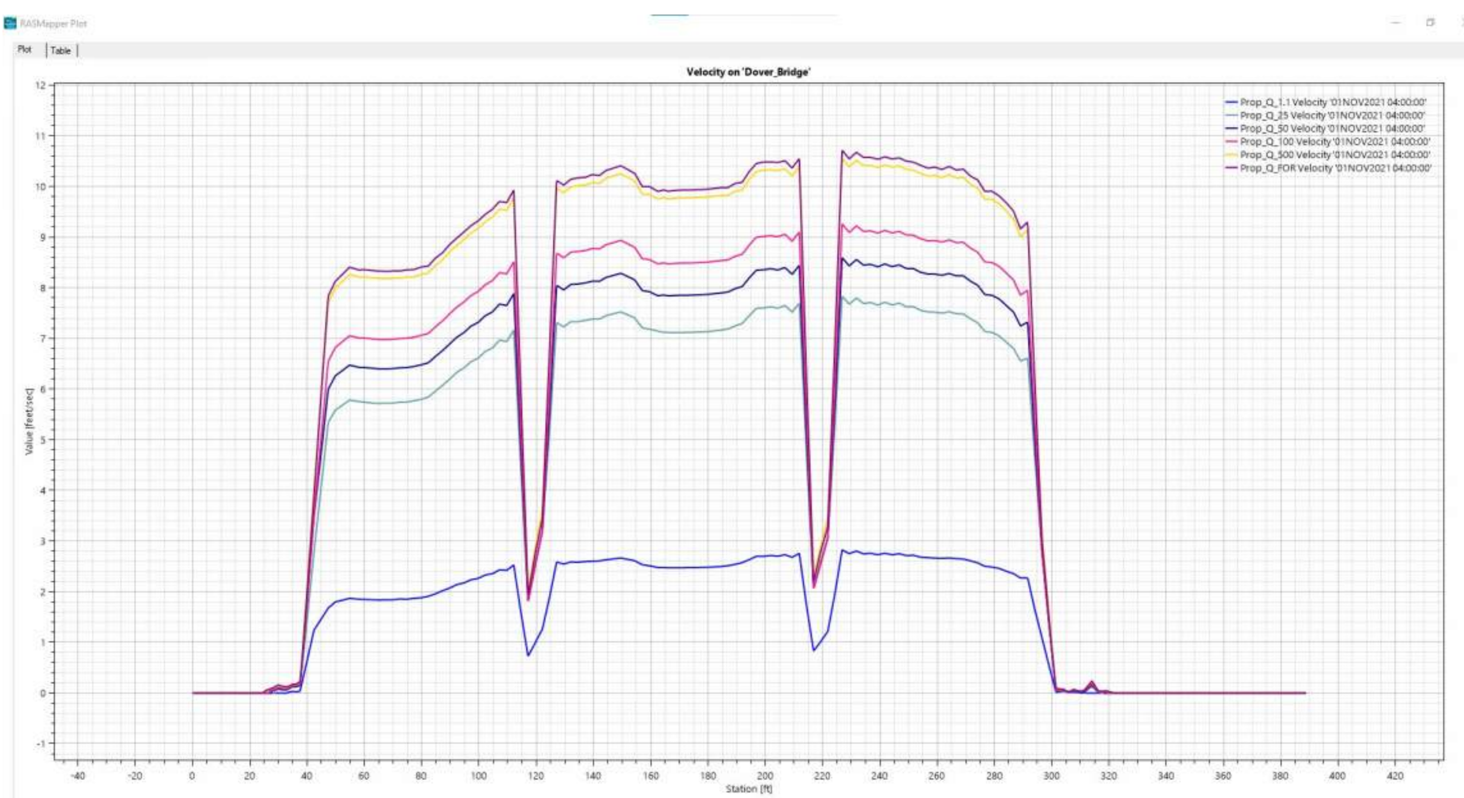


VELOCITY AT PROFILE LINE UPSTREAM OF DOVER BRIDGE- PROPOSED CONDITION

VELOCITY AT CENTERLINE OF DOVER BRIDGE - EXISTING CONDITION



VELOCITY AT CENTERLINE OF DOVER BRIDGE- PROPOSED CONDITION



Appendix C

Scour Calculations

CRITICAL VELOCITY

Boring ID	Depth (ft)	D50 (mm)[4]	Q100 Critical Velocity (ft/s)[1,2]	Q500 Critical Velocity (ft/s)[1,3]	Mean Velocity Q100 (ft/s)	Mean Velocity Q500 (ft/s)
BB-DFPR-101	5 to 6.7	0.5	6.9	7.0	8.5	9.7
BB-DFPR-101	10 to 12	0.2	5.5	5.6		
BB-DFPR-103	5 to 7	4.1	14.4	14.7		
BB-DFPR-103	15 to 17	2.5	12.3	12.5		

Notes:

1. $V_c = K_U Y^{1/6} D^{1/3}$, HEC 18 Critical Velocity equation 6.1

2. Y_{Q100}= 9.7 ft, average depth of flow upstream of bridge

3. Y_{Q500}= 10.7 ft, average depth of flow upstream of bridge

4. Grain size samples obtained from a 50.8 mm (2 inch) outer diameter and 35 mm (1.375 inch) inner diameter split spoon.

CLEAR-WATER CONTRACTION SCOUR

Variable	100 yr	500 yr	Unit	Notes
Q	33,935	42,175	ft^3/s	Flow at dover bridge profile line
D_m	0.010	0.010	ft	Calculated D_m=1.25*D_{50}
D_{50}	2.5	2.5	mm	From Grain size analysis
D_{50}	0.008	0.008	ft	Converted to ft
W	240.4	240.4	ft	Width at Dover Bridge profile line minus piers
Ku	0.0077	0.0077		English Units
Y_0	11.7	12.7	ft	HEC-RAS Linterp Dover Bridge profile line
Solve for Y_2	32.0	38.6	ft	Calculated Eq. 6.4
Solve for Ys	20.3	25.9	ft	Calculated Eq. 6.5

HEC-18 PIER SCOUR EQUATION

Variable	100 yr	500 yr	Unit	Notes
Y_1	9.7	10.7	ft	HEC-RAS Linterp US dover bridge profile line
Pier Shape	Sharp Nose	Sharp Nose		
K_1	0.9	0.9		From Table 7.1, assumed sharp nosed pier
K_2	1.0	1.0		From Table 7.2, assumed angle of attack is 0
K_3	1.1	1.1		From Table 7.3
a	5	5	ft	Provided by Thornton Tomesetti
L	34.4	34.4	ft	Estimated based on bridge deck width, used in model
Fr_1	0.5	0.5		Calculated $Fr_1=V_1/(gy_1)^{1/2}$
V_1	8.0	9.0	ft/s	Velocity at US dover bridge profile line
g	32.2	32.2	ft/s^2	Gravity
Solve for Y_s/a	1.8	1.9	ft	Calculated Eq. 7.3
Solve for Ys	8.9	9.5	ft	Calculated Eq. 7.3
Solve for Y_s/Y_1	0.9	0.9	ft	Calculated Eq. 7.1
Solve for Ys	8.9	9.5	ft	Calculated Eq. 7.1