MAINE DOT - ROUTE 1 (CAPE NEDDICK BRIDGE) OVER CAPE NEDDICK RIVER, YORK COUNTY, MAINE

HYDROLOGIC AND HYDRAULIC REPORT

June 2018

PREPARED FOR

MAINE DEPARTMENT OF TRANSPORTATION

PREPARED BY

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The following is a report of the hydrologic and hydraulic analysis of the existing and proposed bridges at Route 1 (Cape Neddick Bridge) over Cape Neddick River in York County, ME.

1.0 Introduction

Cape Neddick Bridge carries Route 1 over Cape Neddick River approximately 1.06 miles upstream from Cape Neddick Harbor.

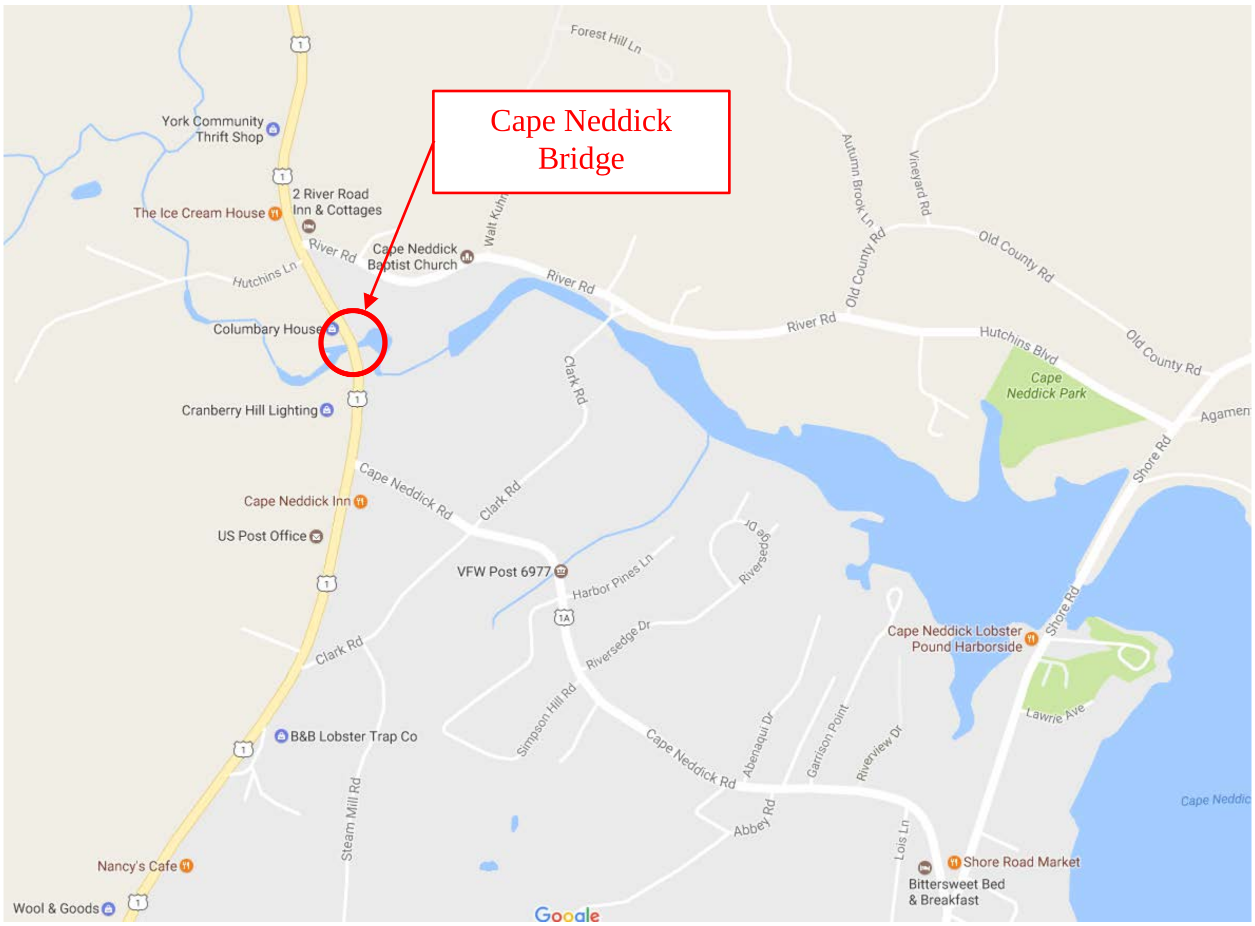

Figure 1 – Project Location Map

Cape Neddick Bridge is approximately 0.25 miles downstream of the Hutchins Lane Bridge. Cape Neddick Bridge is approximately 0.33 miles upstream from Clarks Bridge and 0.94 miles upstream from Shore Road Bridge, both crossing over the Cape Neddick River.

Figure 2 – Aerial image showing project site

2.0 Existing Data Review

The following information was reviewed as part of the hydraulic analysis:

- FEMA Flood Insurance Rate Map. York County, ME. June 17, 2002. The FEMA flood insurance rate map shows the project in Zone AE. Zone AE means that base flood elevations have been determined. The base flood elevation shown on the FIRM is an elevation of 20 downstream and 21 upstream of the project bridge. The flood profile provided in the FEMA Flood Insurance Study indicated that the water surface elevation of the 100-year event was approximately 20.9 upstream of the bridge and 20.0 downstream of the bridge.

- USGS Scientific Investigations Report 2008-5047, "Flood of May 2006 in York County, Maine. This report discusses the base flood elevations and flows from the FEMA FIRM information and compares the information to the flood information gathered from the May 2006 flood in York County. The May 2006 flood event, colloquially referred to as "The Mother's Day Flood", is considered the flood of record. Flows were estimated at twice the 500-year event at the Cape Neddick River project area, approximately 2000 cfs.

- Historic Flood Data: During the initial public meeting, several residents came forward with anecdotal information regarding the May 2006 flood otherwise known as "The Mother's Day Flood". The residents provided photos of the event shortly after peak flows and provided descriptions of when maximum water elevations occurred during the storm.

- Existing and proposed preliminary plans, site survey and site photos, including data from MaineDOT Bridge files. The existing bridge is single span slab bridge. The existing abutments are cantilever vertical wall abutments aligned with the flow. The proposed bridge is a single span buried arch structure.

- On August 12, 2016, MaineDOT conducted a site visit at the bridge. Photographs are provided in **Appendix H**.

- USGS gage information is not available for this bridge location.

Figure 3 – Existing Structure Opening

3.0 Hydrology

The peak flows for the Cape Neddick River at the location of the bridge replacement were calculated in accordance with USGS regression equations. The drainage area at the location of the bridge replacement project is 8.36 square miles. Peak flows can be found in **Table 1**. The hydrology report can be found in **Appendix B**.

In addition to the calculated flows provided, flows were also provided in the FEMA Flood Insurance Study (FIS). The flows were transposed based on the relationship of the drainage area to the bridge and the drainage area to the stream gage downstream. The flows from the FEMA FIS are also provided in **Table 1**. The FEMA FIS flows are reported for the Cape Neddick River at the confluence with Cape Neddick Harbor.

Table 1: Flood Information
(For calculations see **Appendix B**)

Year Storm	**USGS Regression Equations at Bridge (cfs)**	FEMA FIS Flows (cfs)
Drainage Area	**8.36 sq. mi.**	9.53 sq. mi
$Q_{1.1}$	**94.7**	---
Q_2	**190.0**	---
Q_5	**295.4**	----
Q_{10}	**365.1**	479
Q_{25}	**479.9**	---
Q_{50}	**549.6**	785
Q_{100}	**644.1**	950
Q_{500}	**852.7**	1420
Flood of Record	**-**	2250

The hydraulic analysis was completed using the peak flows calculated from the USGS regression equations. The flows shown in **bold** were used in the hydraulic analysis at Cape Neddick Bridge over Cape Neddick River. The flood of record flow used in the hydraulic analysis is 2000 cfs, this value is reduced from the flow reported in FEMA to be consistent with the USGS regression equation flows that are all lower than FEMA values for this location. These flows were compared to what has been observed at the project location to validate which flow data. As can be seen in the photographs provided in **Appendix H,** there is no clear high water mark present at the site. However, the flood of record had maximum elevations at or above the roadway elevations adjacent to the bridge however never overtopped the roadway. The hydraulic model shows that the existing structure has a large hydraulic drop through the structure during the flood of record with minor clearance at the storm event. The hydraulic drop can be seen in photographs located in **Appendix H.** Therefore, these flows were accepted as valid for the analysis.

4.0 Hydraulic Analysis

Hydraulic calculations for the existing and proposed conditions along Cape Neddick River were performed using the U.S. Army Corps of Engineers' software HEC-RAS, version 4.1. HEC-RAS supports one-dimensional, steady flow, water surface profiles calculations. Cross-sections were cut from survey gathered for this project.

The existing bridge was constructed in 1928. The bridge is approximately 14 feet from abutment to abutment. The low chord of the existing bridge is 27.5 feet. The existing structure has a hydraulic opening of approximately 183 square feet.

The MaineDOT Bridge Design Guide states that bridges that are not major riverine bridges shall have a depth of 2 feet minimum of freeboard over the 50-year storm event or 1 foot minimum of freeboard over the 100-year storm event. The existing structure currently meets this requirement however does not meet the requirement for bank width. The existing bank width is 23 feet and the existing structure is 15 feet clear span. This is causing the existing structure to constrict the waterway. The proposed bridge is approximately 29 feet from face of abutment to face of abutment and the footings and seals are approximately 23 feet providing full bank width. The low chord of the proposed structure has been raised from existing conditions to approximately 25.1 feet. The proposed structure offers approximately 233 square feet of hydraulic opening.

The HEC-RAS model was run assuming a downstream water surface elevation. The Cape Neddick River is tidal approximately 0.2 miles from the project location and the tidal region was not modeled as part of this project as the model was able to converge prior to the tidal zone. The downstream water surface elevations for the 10-, 50-, 100-, and 500-year events were found using the FEMA FIS and the flood profile. The water surface elevations for the other events were estimated from USGS mapping and the FEMA FIS.

The model was run using "subcritical" flow due to the Froude numbers at all cross-sections below 1.0. The model includes approximately 530 feet of Cape Neddick River with approximately 400 feet downstream of the bridge and 140 feet upstream of the bridge. Cross-sections were set approximately 25 feet apart from each other beginning at the downstream end of the model and moving upstream.

Manning's n numbers were found in the FEMA FIS for Cape Neddick River. The n-value for the channel was 0.030 and the n-value for the overbanks was 0.05. Ineffective flow areas were set upstream of the bridge based on contraction and expansion.

The analysis found that the bridge is not overtopped by any storm event. All storm events are forced through the opening within the structure including the flood of record. The hydraulic model was validated using anecdotal information, specifically regarding "The Mother's Day Flood." There were several reports the flood water at its height was very close to contacting the low chord of the bridge and falling short of overtopping the Route 1 roadway. Additionally, an adjacent property reported that flood waters for this same event was within several feet of a structure on the property. Using this information and the existing survey, the floodwater elevation was determined to be approximately elevation 29 feet. The existing model was able to replicate this condition with upstream cross-sections reaching elevations of 29.3 feet for the flood of record. The proposed structure is able to reduce this upstream water elevation to 24.3 feet providing a 5-foot reduction in water surface elevation.

The proposed project increases the hydraulic opening of the structure however the low chord of the bridge was lowered and therefore the clearance between the water surface and the low chord of the structure remained within 0.3 of the existing structure clearance. The proposed structure is not under pressure-flow and offers 8.1 feet of clearance between the 50-year water surface elevation and the low chord of the bridge. As a result of the proposed bridge replacement, the water surface elevation for all storm events has been decreased. In addition, the velocities have decreased as well.

Figure 4 – Downstream side of bridge one day after "Mother's Day Flood"

Table 2 provides a summary of the hydraulic analysis of the existing and proposed conditions at the Cape Neddick River.

Table 2: Hydraulic Analysis Summary

Summary of Hydraulic Data – Route 1 over Cape Neddick River	Existing Bridge	Proposed Bridge	Delta
Low Chord Upstream face of Bridge	27.5	25.1	-2.4
Bridge Opening Area, ft^2	183	233	50
Headwater at Upstream face of bridge, Q1.1, ft.	15.08	15.03	-0.05
Headwater at Upstream face of bridge, Q10, ft.	17.79	16.39	-1.40
Headwater at Upstream face of bridge, Q25, ft.	18.75	16.81	-1.94
Headwater at Upstream face of bridge, Q50, ft.	19.29	17.02	-2.27
Headwater at Upstream face of bridge, Q100, ft.	19.97	17.28	-2.69
Headwater at Upstream face of bridge, Q500, ft.	21.32	17.84	-3.48
Discharge Velocity at Q1.1, fps	5.39	1.57	-3.82
Discharge Velocity at Q10, fps	8.43	3.70	-4.73
Discharge Velocity at Q25, fps	9.15	4.42	-4.73
Discharge Velocity at Q50, fps	9.52	4.70	-4.82
Discharge Velocity at Q100, fps	10.01	5.05	-4.96
Discharge Velocity at Q500, fps	10.95	5.53	-5.42
Headwater at Upstream face of bridge, Flood of Record, ft.	26.89	20.35	-6.54
Discharge Velocity at Flood of Record, fps	13.87	10.09	-3.78
Clearance at Q50, ft.	8.21	8.08	-0.13
Clearance at Q100, ft.	7.53	7.82	0.29

The HEC-RAS model was reviewed for errors, warnings, and notes. There were several notes produced by HEC-RAS for the existing and proposed models about multiple critical depths found at several cross-sections. While there were no errors produced, some warnings stated there might be need for more cross-sections. The number of cross-sections were reviewed and were deemed acceptable for this analysis.

5.0 Scour Analysis

A scour analysis was performed based on equations from FHWA publication HEC-18 (Fifth Edition). The 100-year and 500-year events were analyzed for scour at the proposed Route 1 crossing. The D_{50} of the streambed material was found during geotechnical testing. The D_{50} of the material was found to be 2.889 mm. This number was used to determine whether clear water or live bed scour analysis was to be performed. At Route 1, clear-water scour is required to be calculated for contraction scour. In addition, local scour was calculated per HEC-18 for the abutments.

Boring logs indicated that bedrock was found approximately one to two feet below the streambed elevation. Calculations indicate scour depths of 6 feet or greater. However, the scour calculations do not take the presence of bedrock into account. Since bedrock is shallow, the bridge foundations will be constructed directly on top of bedrock. The foundations are concrete footings constructed on seal concrete. Scour is not an issue at the river crossing.

The total scour depths can be found in Table 8 and the scour analysis can be found in the **Appendix G**.

Table 8: Scour Depths

	100 - year storm	
	Abutment 1	Abutment 2
Aggradation/ Degradation (ft.)	0.00	0.00
Contraction/Expansion Scour (ft.) *	2.78	2.78
Local Scour (ft.)	0.00	0.00
TOTAL SCOUR (ft.)	***2.78***	***2.78***

	500-year storm	
	Abutment 1	Abutment 2
Aggradation/ Degradation (ft.)	0.00	0.00
Contraction/Expansion Scour (ft.) *	3.93	3.93
Local Scour (ft.)	2.05	2.70
TOTAL SCOUR (ft.)	***5.98***	***6.63***

6.0 Summary

In summary, a preliminary hydrology, hydraulic and scour evaluation was completed for the Cape Neddick Bridge Replacement. The hydrology for the Cape Neddick River was calculated using USGS regression equations. Additionally, flow data from a FEMA flood insurance study (FIS) and a USGS scientific investigations report on the flood of May 2006 in York County was used in the evaluation.

The existing bridge at Route 1 over Cape Neddick River in York County is proposed to be replaced. The low chord of the existing structure is at 27.5 feet. The existing structure offers approximately 183 square feet of hydraulic opening.

The proposed hydraulic opening is being increased from existing conditions to provide full bank width of the waterway. The low chord of the proposed structure has been lowered to 25.1 feet to ensure the structure is outside of the roadway template and underground utilities can be installed in the roadway shoulder. Lengthening the span of the structure has increased the hydraulic opening to approximately 233 square feet.

Hydraulic calculations for the existing and proposed conditions along Cape Neddick River were performed using the U.S. Army Corps of Engineers' software HEC-RAS, version 4.1. HEC-

RAS supports one-dimensional, steady flow, water surface profile calculations. Cross-sections were cut from survey data gathered for this project. This evaluation concluded that the existing structure provides 8.21 feet of clearance at the Q100 storm event and 0.61 feet for the storm of record.

The proposed structure will decrease the water surface elevations for all storm events. In addition, the increased hydraulic opening allows for approximately 8.08 feet of clearance between the low chord of the structure and the 50-year water surface elevation and

Although scour depths reach 6 feet and greater, scour is not a concern for this location because bedrock is located at or near the streambed and new foundations will be constructed directly on top of the bedrock.

Appendix Contents

APPENDIX A

FEMA FIRM

NOTES TO USERS

To obtain more detailed information in areas where Base Flood Elevations (BFEs) and/or floodways have been determined, users are encouraged to consult the Flood Profiles and Floodway Data tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations and therefore may not exactly reflect the flood elevation data presented in the FIS report. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in the FIS report in conjunction with the data shown on this FIRM.

Elevation Reference Mark (ERM) elevations listed on this map were obtained and/or developed to establish vertical control for determination of flood elevations and floodplain boundaries portrayed on this map. Users should be aware that these ERM elevations may have changed since the publication of this map. To obtain up-to-date elevation information on National Geodetic Survey (NGS) ERMs shown on this map, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit their website at www.ngs.noaa.gov. Map users should seek verification of non-NGS ERM monument elevations when using these elevations for construction or floodplain management purposes.

DIGITAL DATA AVAILABILITY: Digital files containing the thematic floodplain information shown on this map can be made available on CD-ROM by request. The files are currently archived in MicroStation design (DGN) file format referenced to the Universal Transverse Mercator (UTM) projection and the North American Datum of 1927 (NAD27). To obtain the digital files, send a written request to: Flood Insurance Information Specialist, 2977 Prosperity Avenue, Fairfax, Virginia 22031. Telephone (703) 876-0148, Fax (703) 876-0073.

ELEVATION REFERENCE MARKS

REFERENCE MARK	ELEVATION IN FT. (NGVD)†	DESCRIPTION OF LOCATION
RM 23	19.11	Standard USC&GS disk, stamped 1943, on northwest wing wall of Shore Road bridge over the Cape Neddick River.

†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, but possible.

UNDEVELOPED COASTAL BARRIERS*

Identified 1983 | Identified 1990 or Later | Otherwise Protected Areas Identified 1991 or Later

* 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**
- (A)—(A) Cross Section Line
- (23)- - -(23) Transect Line
- (EL 987) Base Flood Elevation in Feet Where Uniform Within Zone**
- RM7 × Elevation Reference Mark
- • M1.5 River Mile

**Referenced to the National Geodetic Vertical Datum of 1929

NOTES TO USERS

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' National Geodetic Vertical Datum of 1929 (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.

NOTE: The coordinate system used for the production of this Flood Insurance Rate Map (FIRM) is Universal Transverse Mercator (UTM), North American Datum of 1927 (NAD27), Clarke 1866 spheroid. Differences in the datum and spheroid used in the production of FIRMs for adjacent communities may result in slight positional differences in map features at the community boundaries. These differences do not affect the accuracy of the information shown on the FIRM.

ATTENTION: Flood elevations on this map are referenced to the National Geodetic Vertical Datum of 1929. These flood elevations must be compared to structure and ground elevations referenced to the same datum. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, contact the National Geodetic Survey at the following address:

Vertical Network Branch, N/CG13
National Geodetic Survey, NOAA
Silver Spring Metro Center 3
1315 East-West Highway
Silver Spring, Maryland 20910
(301) 713-3191

BASE MAP SOURCE: For the stream lines planimetric base map files were provided in digital format by the State of Maine, Office of GIS. These files were compiled at a scale of 1:24,000 from U.S. Geological Survey 7.5-Minute Series Topographic Maps. Additional information may have been derived from other sources. Users of this FIRM should be aware that minor adjustments may have been made to specific base map features.

MAP REPOSITORY

York Town Planner's Office, 186 York Street, York, Maine 03909 (Maps available for reference only, not for distribution.)

INITIAL IDENTIFICATION:
JUNE 21, 1974
FLOOD HAZARD BOUNDARY MAP REVISIONS:
OCTOBER 22, 1976
FLOOD INSURANCE RATE MAP EFFECTIVE:
DECEMBER 15, 1983
FLOOD INSURANCE RATE MAP REVISIONS:

July 15, 1992 – to add undeveloped coastal barriers.

June 17, 2002 – to change base flood elevations, special flood hazard areas, and zone designations; to update map format, and the effects of wave action; to add roads and road names; to incorporate Primary Frontal Dune analysis; and to reflect revised shoreline.

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
500 0 500 FEET

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

TOWN OF
YORK,
MAINE
YORK COUNTY

PANEL 26 OF 32

(SEE MAP INDEX FOR PANELS NOT PRINTED)

COMMUNITY - PANEL NUMBER
230159 0026 D

MAP REVISED:
JUNE 17, 2002

Federal Emergency Management Agency

APPENDIX B

Hydrology

WIN:	21709.00	**Project Name:**	Cape Neddick Bridge
Town:	York	**Stream Name:**	Cape Neddick River
Route No.	US 1	**Bridge Name:**	Cape Neddick Bridge
Asset ID:	2127	**Analysis by:**	CSH
Lat:	43.1925 **Long:** -70.6198	**Date:**	1/6/2017

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

Enter data in blue cells only!

	km^2	mi^2	ac	*Enter data in [mi^2]*
A	21.65	8.36	5350.4	Watershed Area *DRNAREA*
W	3.88	1.5	959.3	Wetlands area (by NWI)
P_c	365450	4784235		watershed centroid (E, N; UTM 19N; meters)
County	York			*choose county from drop-down menu*
pptA	46.7			mean annual precipitation (inches; by look-up)
SG	0.00			sand & gravel aquifer as decimal fraction of watershed A

A (km^2)	21.65	**Conf Lvl**	0.67
W (%)	17.93		NWI Wetlands % *STORNWI*

Worksheet prepared by:
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207-557-1052
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ver. 2016 Feb 05

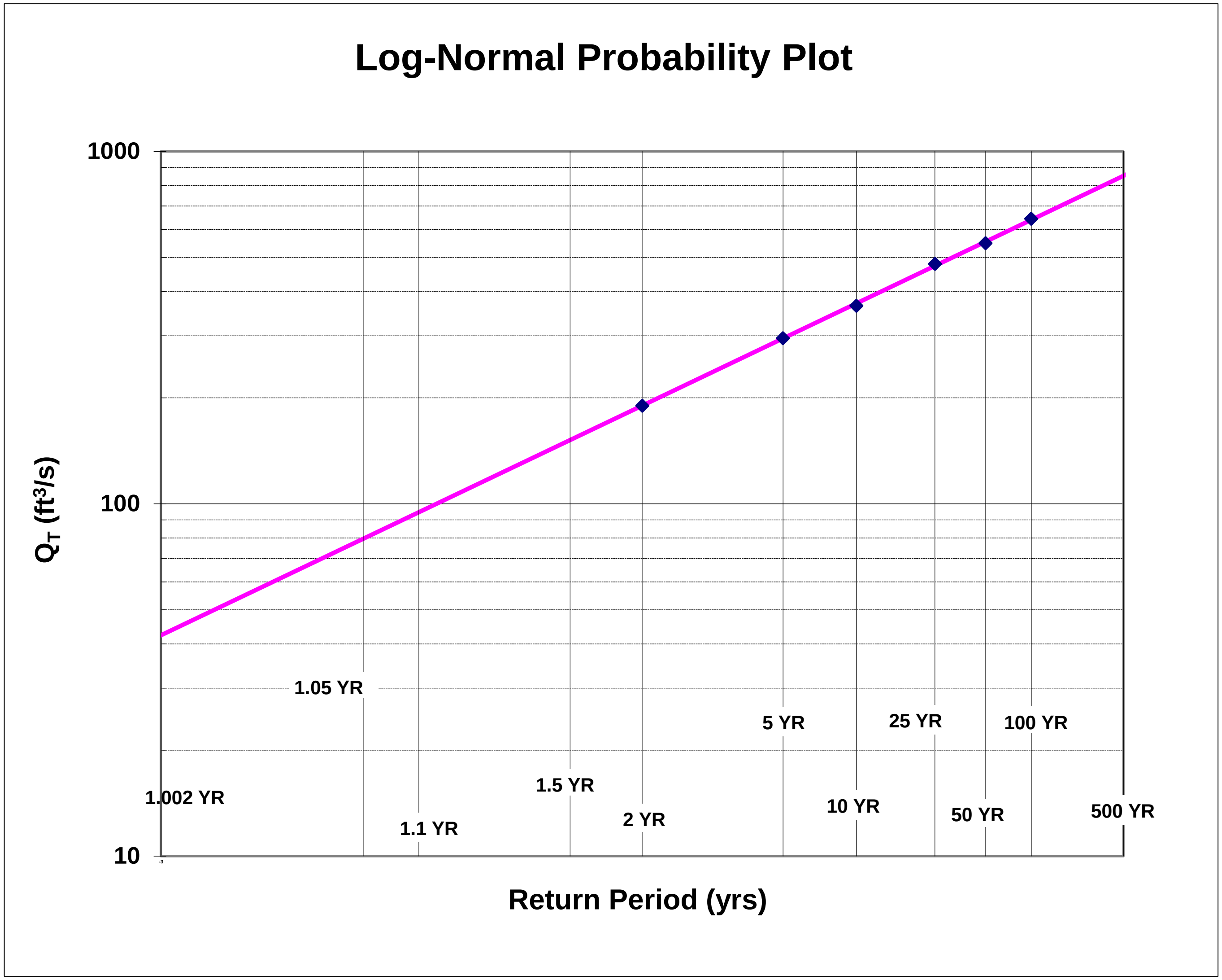
References:

Hodgkins, G.A., 1999.
Estimating the magnitude of peak flows for streams in Maine for selected recurrence intervals
WRIR 99-4008, USGS Augusta, ME

Lombard, P.J. & G.A. Hodgkins, 2015.
Peak flow regression equations for small, ungaged streams in Maine - Comparing map-based to field-based variables
SIR 2015-4059, USGS, Augusta, ME

Ret Pd T (yr)	Peak Flow Estimate Lower	Q_T (m^3/s)	Upper	Q_T (ft^3/s)
1.1		2.68		94.7
2		5.38		190.0
5		8.37		295.4
10		10.34		365.1
25		13.59		479.9
50		15.57		549.6
100		18.24		644.1
500		24.15		852.7

$Q_T = b \times A^a \times 10^{-wW}$



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

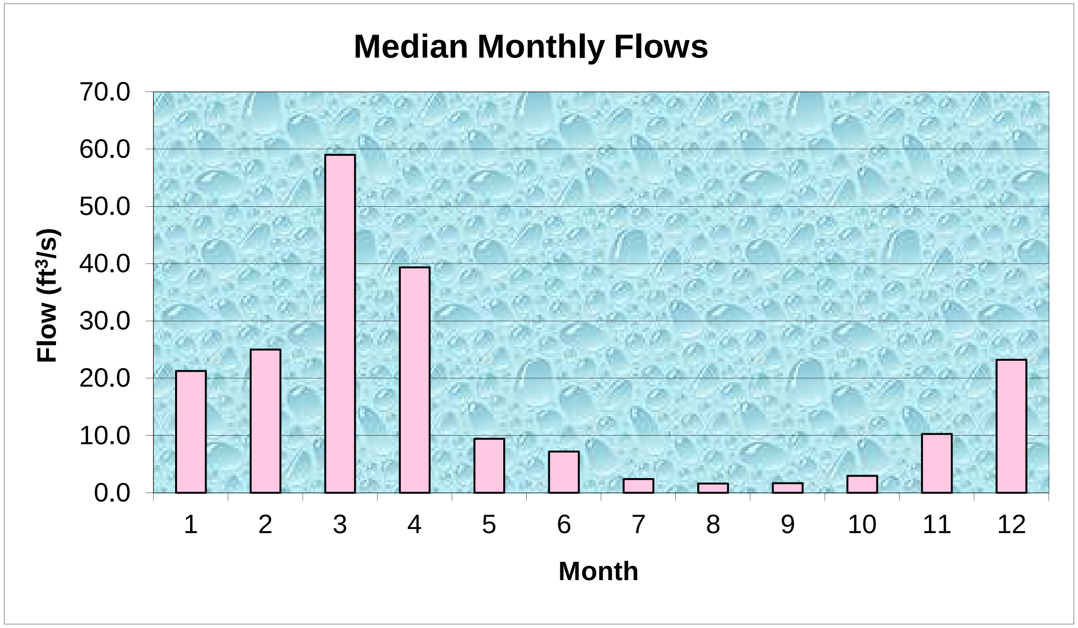
WIN:	21709.00	**Project Name:**	Cape Neddick Bridge
Town:	York	**Stream Name:**	Cape Neddick River
Route No.	US 1	**Bridge Name:**	Cape Neddick Bridge
Asset ID:	2127	**Analysis by:**	CSH
Lat:	43.19250 **Long:** -70.6198	**Date:**	1/6/2017

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

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

	Value	**Variable**	**Explanation**
	8.36	*A*	Area (mi^2)
365450	4784235	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	17.33	*DIST*	Distance from Coastal reference line (mi)
	46.7	*pptA*	Mean Annual Precipitation (inches)
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median} (ft3/s)	(m^3/s)
Jan	**21.27**	0.6027
Feb	**25.02**	0.7089
Mar	**58.99**	1.6718
Apr	**39.36**	1.1154
May	**9.46**	0.2682
Jun	**7.21**	0.2043
Jul	**2.41**	0.0683
Aug	**1.64**	0.0465
Sep	**1.72**	0.0488
Oct	**3.00**	0.0851
Nov	**10.30**	0.2920
Dec	**23.22**	0.6581



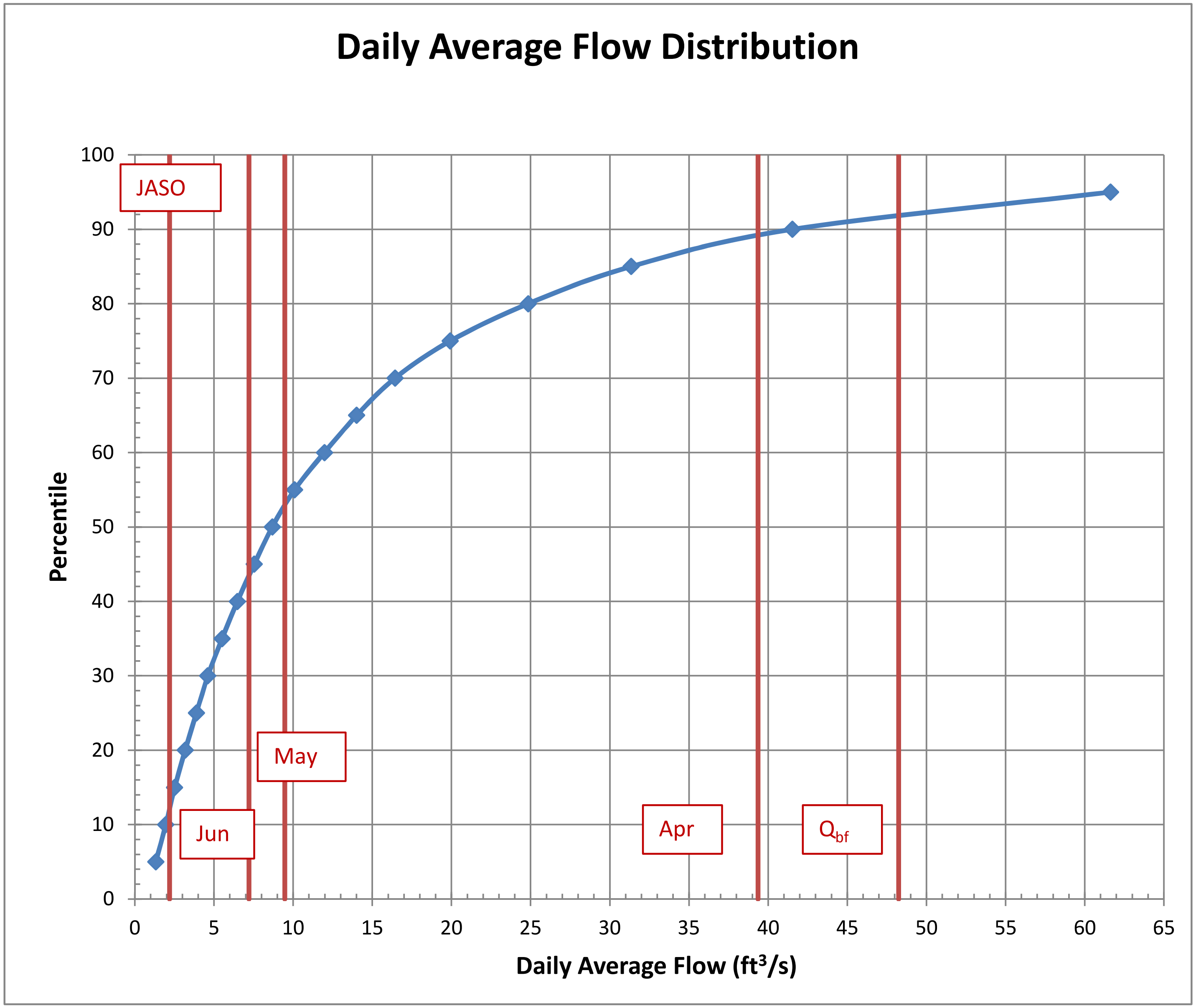
Q_{bf}	**48.2**	
ann avg	**17.5**	
ann med	**9.4**	
$Q_{1.002}$	**42.3**	
$Q_{1.01}$	**56.3**	
$Q_{1.05}$	**79.6**	
Q_{bf}	**113.2**	assume v = 4ft/s

W_{bf}	**23.1**	estimated bankfull width (ft)
d_{bf}	**1.2**	estimated bankfull depth (ft)
A_{bf}	**28.3**	estimated bankfull flow area (ft^2)

References

Dudley, R.W., 2004. Hydraulic Geometry Relations ..., SIR 2004-5042

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



Daily Avg Flow Dist

A_{ws} = (mi²) 8.4

Q (ft³/s)

Pctl	Median	84th pctl
5	1.32	2.12
10	1.95	2.94
15	2.51	3.67
20	3.18	4.45
25	3.89	5.22
30	4.61	5.94
35	5.52	6.79
40	6.47	7.81
45	7.54	8.83
50	8.68	10.42
55	10.08	12.13
60	11.98	14.24
65	14.01	16.59
70	16.44	19.36
75	19.92	23.28
80	24.85	27.79
85	31.35	35.62
90	41.53	47.83
95	61.64	74.38

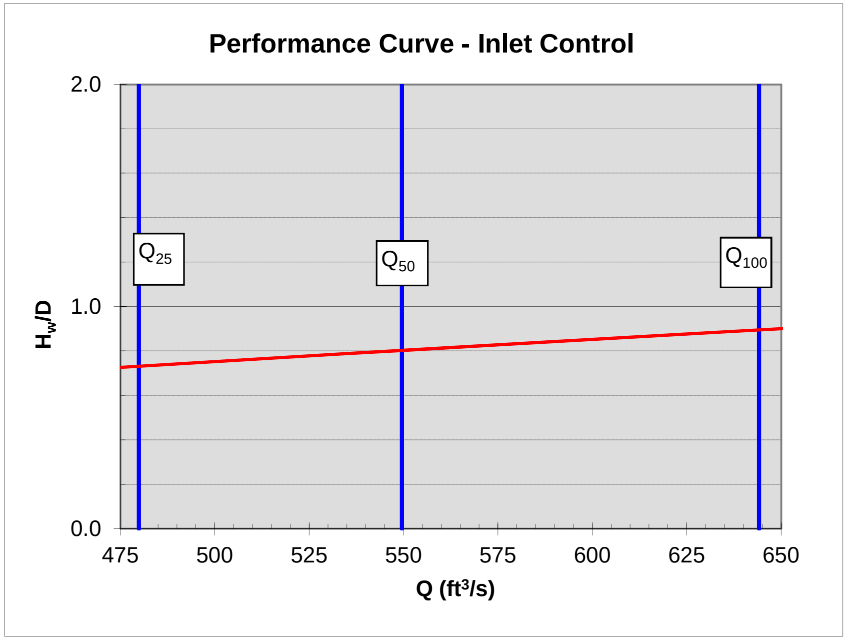
Q_{bf}	48.2
$Q_{1.002}$	42.3
$Q_{1.1}$	94.7
Q_2	190.0

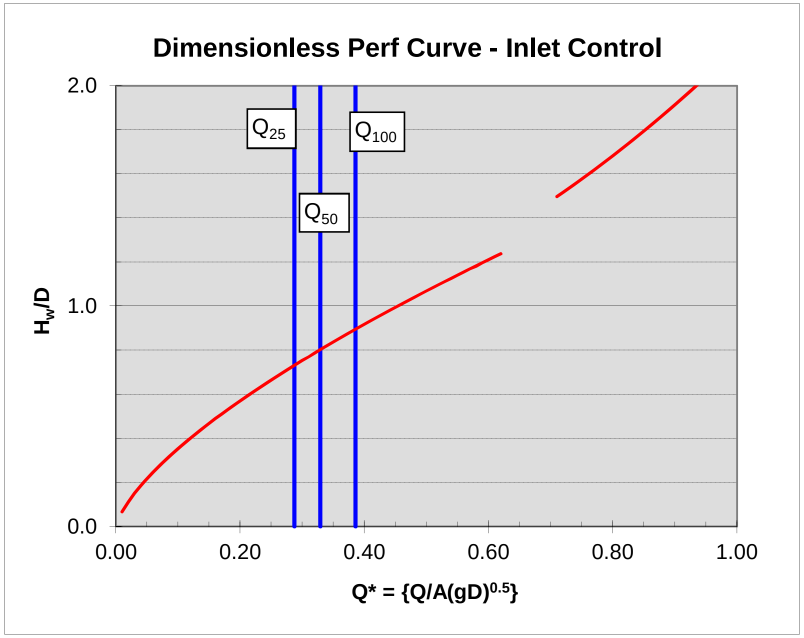
NOTE: ***This page is for preliminary sizing only.***
Final design should be done with HY8 or HDS-5

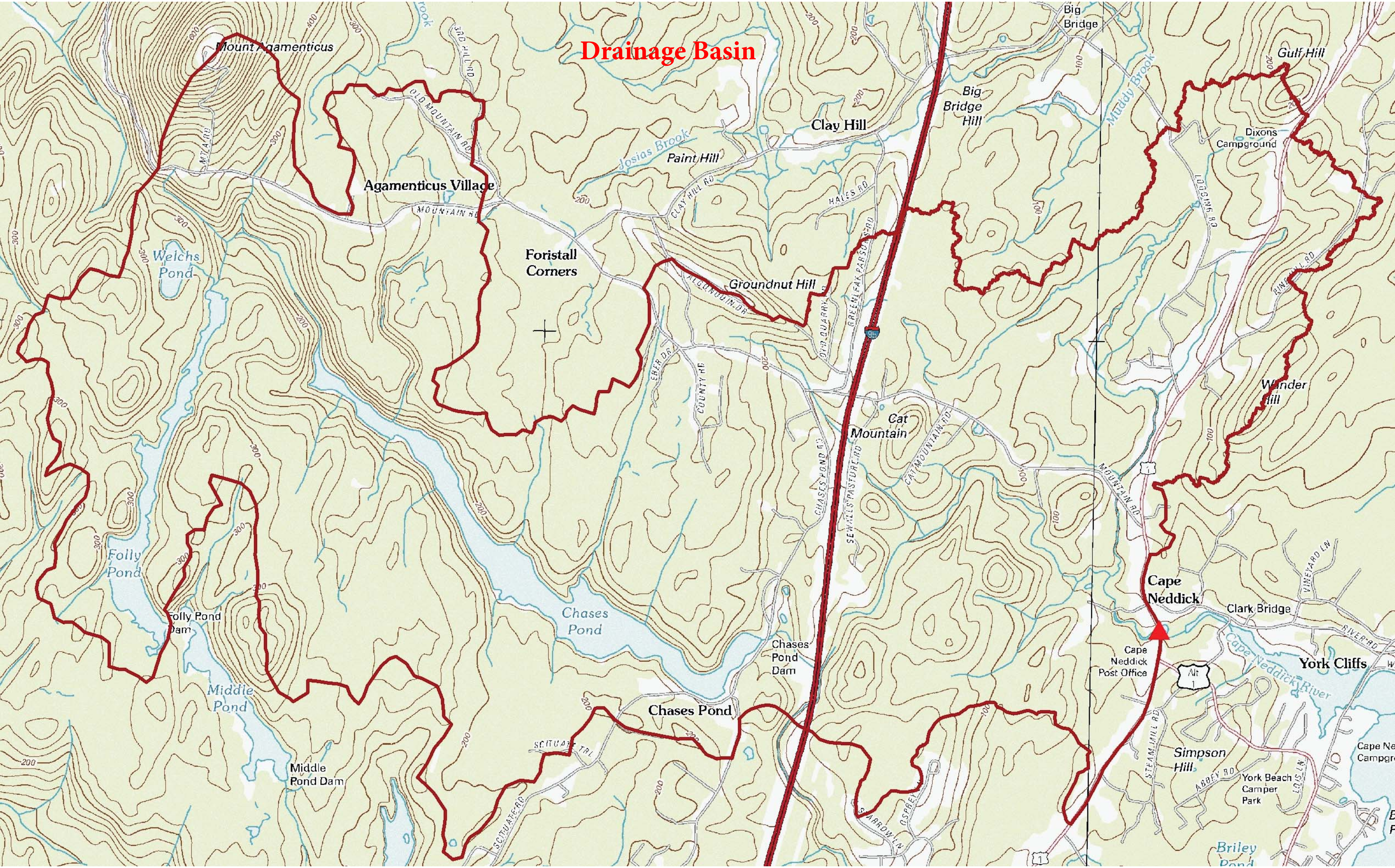
Preliminary Culvert Sizing - Round & Box Culverts

Shape:	Box					
Type:	Box 0 ww		Q_{25}	479.9		
D or R (ft)	6		Q_{50}	549.6	trial D / R =	10.3
w (ft)	*20*	*box width*	Q_{100}	644.1	trial w: BFW =	23.1
Slope (ft/ft)	0.02					
A (ft^2)	120.00					
g (ft/s^2)	32.2					

Note: culvert dimensions are for open flow area; adjust for lost capacity due to embedding / backfilling (min {2' / 25% rise} embedment). This is hydraulic sizing only; check for other requirements.





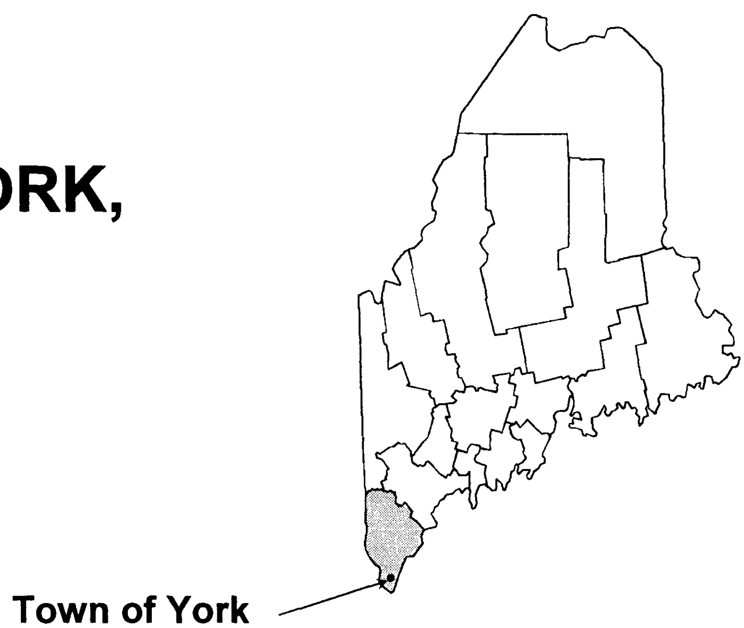

Drainage Basin
Mount Agamenticus
Agamenticus Village
Foristall Corners
Clay Hill
Paint Hill
Josias Brook
Groundnut Hill
Big Bridge
Big Bridge Hill
Muddy Brook
Gulf Hill
Dixons Campground
Welchs Pond
Cat Mountain
Wonder Hill
Folly Pond
Folly Pond Dam
Middle Pond
Middle Pond Dam
Chases Pond
Chases Pond Dam
Cape Neddick
Clark Bridge
Cape Neddick Post Office
Cape Neddick River
York Cliffs
Simpson Hill
York Beach Camper Park
Briley Pond
OLD MOUNTAIN RD
MOUNTAIN RD
CLAY HILL RD
HALES RD
ALGONQUIN DR
OLD QUARRY
GREENLEAF PARSONS RD
COUNTY RD
CHASES POND RD
SEWALLS PASTURE RD
CAT MOUNTAIN RD
LODGING RD
SCITUATE TRL
SCITUATE RD
STEAM MILL RD
ABBEY RD
VINEYARD LN
RIVER RD
SPARROW LN
OSPREY LN

APPENDIX C

FEMA Information/USGS Report

FLOOD INSURANCE STUDY

TOWN OF YORK, MAINE
YORK COUNTY



REVISED:
JUNE 17, 2002

Federal Emergency Management Agency

COMMUNITY NUMBER - 230159

NOTICE TO FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

FIS Effective Date: September 15, 1983 (Flood Insurance Study report);
December 15, 1983 (Flood Insurance Rate Map)

Revised FIS Dates: July 15, 1992 (Flood Insurance Rate Map only)
June 17, 2002

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FLOOD INSURANCE STUDY
TOWN OF YORK, YORK COUNTY, MAINE

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study (FIS) revises and updates a previous FIS/Flood Insurance Rate Map (FIRM) for the Town of York, York County, Maine. This information will be used by the community to update existing floodplain regulations as part of the Regular Phase of the National Flood Insurance Program (NFIP). The information will also be used by local and regional planners to further promote sound land use and floodplain development.

In some States or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence and the State (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for this FIS are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

For the original, September 15, 1983, FIS report and December 15, 1983, FIRM (hereinafter referred to as the 1983 FIS), the hydrologic and hydraulic analyses were prepared by Stone and Webster Engineering Corporation for the Federal Emergency Management Agency (FEMA), under Contract No. H-4092. The stillwater flooding portion for that study was completed in October 1978. The wave runup and wave height analyses were completed in May 1982.

For this revision, the hydrologic and hydraulic analyses for the Atlantic Ocean were prepared by ENSR for FEMA, under Contract No. EMW-95-C-4783. This work was completed in September 2, 1998.

The digital base map files for the streamlines were provided by the State of Maine, Department of Administrative and Financial Services, Office of GIS, 125 State House Station, Augusta, Maine, 04333-0125, under a Memorandum of Agreement between the Office of GIS and FEMA. These files were complied at a scale of 1:24,000 from U.S. Geological Survey (USGS) 7.5-Minute Series Topographic Maps. In addition, the files were modified to match the previously compiled Flood Insurance Study of the Town of York. The digital base map files for the roads were provided by the Town of York and were compiled from 1998 aerial photographs.

The digital FIRMs were produced in Universal Transverse Mercator coordinates referenced to the North American Datum of 1927 and the Clarke 1866 spheroid.

1.3 Coordination

The purpose of an initial Consultation Coordination Officer's (CCO) meeting is to discuss the scope of the FIS. A final CCO meeting is held to review the results of the study.

For the 1983 FIS, an initial CCO meeting was held on August 3, 1976, and a final CCO meeting was held on January 20, 1983. Both of these meetings were attended by representatives of the Town of York, Stone and Webster Engineering Corporation, and FEMA.

For this revision, initial CCO meetings were held on September 9, 1994, and February 15, 1996, and were attended by representatives of the Town of York, ENSR, the State of Maine, and FEMA. A final CCO meeting was held on October 5, 2000.

2.0 AREA STUDIED

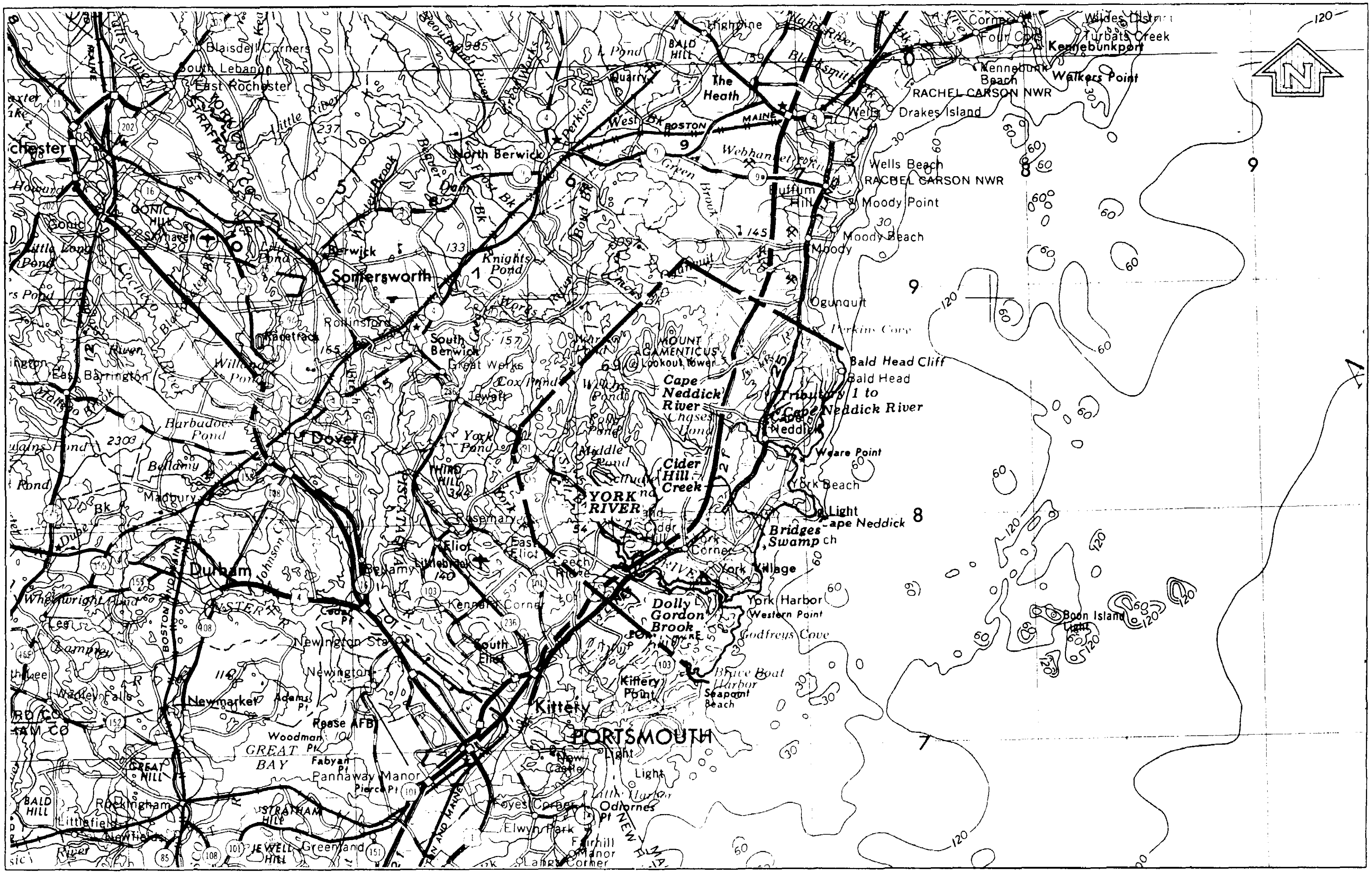
2.1 Scope of Study

This FIS covers the incorporated area of the Town of York, York County, Maine. The area of study is shown on the Vicinity Map (Figure 1).

For the 1983 FIS, the following streams were studied by detailed methods: the Cape Neddick River, for approximately 1.4 miles upstream of Shore Road; Tributary 1 to Cape Neddick River, for a distance of approximately 0.6 mile upstream from its confluence with the Cape Neddick River; Dolly Gordon Brook, for approximately 1.6 miles upstream of its confluence with the York River; Cider Hill Creek, for a distance of approximately 1.9 miles upstream of its confluence with the York River; and Bridges Swamp, for approximately 0.7 mile upstream of Long Beach Avenue. The tidally controlled York River was studied by detailed methods from its confluence with the Atlantic Ocean to a point approximately 1.5 miles upstream of Scotland Bridge Road. The entire coastline of the Atlantic Ocean within the Town of York and all the estuaries within the town were studied by detailed methods. Areas of shallow flooding located at Cow Point Beach, Long Branch, Cape Neddick Harbor, and along a portion of Long Beach were also studied by detailed methods.

For this revision, the entire Atlantic Ocean coastline within the Town of York was restudied by detailed methods.

Limits of detailed study are indicated on the Flood Profiles (Exhibit 1) and/or on the FIRM (Exhibit 2). The areas studied by detailed methods were selected with priority given to all known flood hazard areas and areas of projected development and proposed construction.



FEDERAL EMERGENCY MANAGEMENT AGENCY	APPROXIMATE SCALE
TOWN OF YORK, ME (YORK CO.)	4 0 4 8 12 MILES
	VICINITY MAP

FIGURE 1

The remaining portions of the Cape Neddick River, Dolly Gordon Brook, Cider Hill Creek, Bridges Swamp, and the York River were studied by approximate methods. Bell Marsh, Boulter Pond, Braceys Swamp, Chases Pond, Chicks Brook, Clay Hill Brook, Cutts Ridge Brook, Folly Pond, Garden Brook, Indian Pond, Johnson Brook, the Josias River, Lake Carolyn, Libby Brook, South Branch Libby Brook, West Branch Libby Brook, the Little River, Middle Pond, Moulton Brook, Muddy Brook, Rogers Brook, Rush Swamp, Scituate Pond, Smelt Brook, Southside Brook, Welchs Pond, Whippoorwill Swamp and several unnamed areas were studied by approximate methods. Approximate analyses were used to study those areas having low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon by, FEMA and the Town of York.

2.2 Community Description

The Town of York is a coastal community in York County in southwest Maine, approximately 40 miles south of Portland along U.S. Route 1. York is bordered by the Village of Ogunquit and the Town of Wells to the north, by the Towns of North and South Berwick to the northwest, by the Town of Eliot to the southwest, and by the Atlantic Ocean to the east. York has a land area of 56 square miles and a year-round population of approximately 10,452 (U.S. Census, 1998).

The Town of York provides services for tourism, recreation boating, and sport fishing and is noted for its beaches and anchorages along the Atlantic Ocean. Private property extends to the natural rocky shoreline along much of the northern and southern ends of York. Residential and commercial development is located primarily along U.S. Route 1, the shoreline and the areas of York Harbor, York Village, and Cape Neddick. Development in the floodplains consists of privately owned summer and year-round homes, commercial establishments, and some historic landmarks. Since York is one of the major resort communities in the area, the summer population is three to four times greater than the winter population (U.S. Army Corps of Engineers [USACE], New England Division, 1975).

The coastline is characterized by crescent-shaped beaches, rocky headlands, and tidal flats. Inland of the immediate coast, the topography is gently rolling, rising to low hills with elevations ranging from 50 to 300 feet. Unlike other coastal areas, several ponds are located in York.

Cape Neddick Harbor is located at the mouth of the Cape Neddick River, approximately 3.5 miles northeast of York Harbor. One mile southeast of the harbor is Cape Neddick, a prominent headland. The cape is almost completely developed with homes, guesthouses, hotels, motels, and restaurants. To the north and south of the cape is the Village of York Beach, a popular summer resort. At the mouth of the river, approximately 2.5 miles southwest of Cape Neddick, is the summer resort of York Harbor (New England River Basins Commission [NERBC], 1971). The Harbor is a popular overnight stop for transient recreational boats and is used extensively for fishing, boating, and lobstering (USACE, 1971).

The Town of York is drained by a number of separate basins. The York and Cape Neddick watersheds are of primary importance to the community. The York River and its major tributaries, Smelt, Dolly Gordon, and Cider Hill Brooks, are located in the southern portion of the town and have a combined drainage area of approximately 30 square miles. Many of these streams originate in the numerous ponds in York. The York River, within the Town of York, is influenced by tides from the Atlantic Ocean.

The Cape Neddick River drains an area of approximately 10 square miles in central York. The river originates in Chases Pond and is tidally controlled to a point approximately 300 feet east of U.S. Route 1. Much of the coastal area of York is drained by numerous small streams, such as Bridges Swamp, the Little River, and Rush Swamp Brook, which flow directly into the ocean.

The northern part of the town is drained by the Josias River and its tributary, Clay Hill Brook. Several small mountain streams flow from the northwest part of York into South Berwick.

The climate of the Town of York is characteristic of coastal Maine. During the winter, York experiences approximately two periods of inclement weather a week resulting from the frequent passage of storms. The average annual precipitation is approximately 43 inches with slight geographical variation. The winter months are normally the wettest with precipitation from northeasters occurring in the form of rain or wet snow. The average annual snowfall in York is 70 inches (National Oceanic and Atmospheric Administration [NOAA], 1976). The average wind velocities over the coastal areas are usually greater during the winter, due to the increase in cyclonic activity. The average annual air temperature is 45 degrees Fahrenheit (°F) with daily mean temperatures ranging from 25°F to 68°F.

2.3 Principal Flood Problems

The low-lying coastal areas of York are subject to periodic flooding and wave attack accompanying coastal storms such as northeasters and hurricanes. The majority of these storms cause damage to low coastal highways, boats, beaches, and seawalls. Occasionally, however, a major storm accompanied by strong onshore winds and high tides results in surge and wave activity that causes extensive property damage and erosion. Some of the more significant storms in the York area include the storms of November 1945, 1963, and 1968; February 1972 and 1978; and October 1991. These storms have damaged harbors, marinas, and residential and commercial developments in the flood-prone coastal areas. Continuing erosion associated with severe storms also acts to reduce beach and dune width to below protective and recreational use requirements (USACE, 1971). Present and future demands associated with the seasonal tourist industry will further intensify the pressure for development of flood-prone coastal land.

Some riverine flooding has occurred in the non-tidal portions of streams within York; however, most of the floodplains are relatively undeveloped, thus limiting flood damage. Riverine flooding has not been a serious problem in the community (USACE, New England Division, 1975).

2.4 Flood Protection Measures

In 1945, the USACE constructed a seawall at York Harbor. The seawall has been washed out and consequently improvements made (Town Clerk, 1978).

The State of Maine provides concrete seawalls and stone revetments to protect coastal highways. Other protective structures were constructed and are maintained by private property owners to satisfy their individual requirements. These structures include such backshore protection as timber and steel sheetpiles, bulkheads, stone revetments, concrete seawalls, and pre-cast concrete units (USACE, 1971). Seawalls delineated on the FIRM for this revision have been identified as flood protection structures that reduce wave effects during the base flood. There are no flood protection structures on the streams in York.

3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the community, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this study. Flood events of a magnitude which are expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood which equals or exceeds the 100-year flood (1-percent chance of annual exceedence) in any 50-year period is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency and peak elevation-frequency relationships for each flooding source studied in detail affecting the community.

For the 1983 FIS, the hydrologic analysis for the Town of York involved the study of two types of flooding sources: inland flooding of those areas affected by riverine flooding and coastal flooding affected by coastal storm surge and wave action. Combinations of both of the flooding types were considered for some areas.

Since the streams studied by detailed riverine methods are ungaged, the 10-, 50-, 100-, and 500-year discharges were computed based on the Maine flood magnitude and frequency formulas developed by the USGS (USGS, 1975, and Merli, 1978). Stone and Webster Engineering Corporation has performed an independent evaluation of these formulas and has found them to be applicable to the York area. The USGS formulas predict discharges based on the parameters of watershed drainage area, main channel slope, and percentage of the area of lakes and ponds.

A summary of the drainage area-peak discharge relationships for the streams studied by detailed methods is shown in Table 1, "Summary of Discharges."

TABLE 1 - SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
CAPE NEDDICK RIVER					
At confluence with Cape Neddick Harbor	9.53	479	785	950	1,420
TRIBUTARY 1 TO CAPE NEDDICK RIVER					
Above private road	2.01	138	226	274	409
DOLLY GORDON BROOK					
Above confluence with York River	3.42	196	329	400	613
Above Maine Turnpike	1.25	88	147	179	274
CIDER HILL CREEK					
Above confluence with York River	4.99	285	477	582	886
BRIDGES SWAMP					
At mouth	1.09	110	198	249	403

In New England, flooding of low-lying coastal areas is caused primarily by storm surges generated by extratropical coastal storms called northeasters. Hurricanes occasionally produce significant storm surges in New England, but not with the frequency of northeasters.

The effects of wave action are also considered in the determination of flood hazard areas. Coastal structures that are located above stillwater flood elevations can still be severely damaged by wave runup, wave-induced erosion, and wave-borne debris. For example, during the northeasters of January and February 1978, considerable damage along the Maine coast was caused by wave activity, even though most of the damaged structures were above the high-water level. The extent of wave runup past stillwater levels depend greatly on the wave conditions and local topography.

For this revision, coastal hydrologic analyses were carried out to estimate the 100-year storm characteristics. Published values in the Tidal Flood Survey (USACE, 1988) and National Ocean Service tidal benchmark data (NOAA, 1988; 1987; and 1984) were used to estimate the stillwater elevation for a 100-year flood.

The stillwater elevations have been determined for the 10-, 50-, and 100-year floods for the flooding sources studied by detailed methods and are summarized in Table 2, "Summary of Stillwater Elevations."

TABLE 2 - SUMMARY OF STILLWATER ELEVATIONS

	ELEVATION (feet NGVD)			
FLOODING SOURCE AND LOCATION	10-YEAR	50-YEAR	100-YEAR	500-YEAR
ATLANTIC OCEAN Entire open coast shoreline within the Town of York.	8.5	9.3	12.2[1]	*
ATLANTIC OCEAN Northwest of Long Beach Avenue between Beacon Street and Nubble Road	8.5	9.3	9.6	*
ATLANTIC OCEAN/ CAPE NEDDICK RIVER North of Algonic west of Shore Road	8.5	9.3	9.6	*
ATLANTIC OCEAN/ CAPE NEDDICK HARBOR Along Bay Haven Road southeast of Main Street	8.5	9.3	9.6	*
ATLANTIC OCEAN/ PHILLIPS COVE Northwest of Shore Road north of Bayberry Lane	8.5	9.3	9.6	*

[1]Includes wave setup of 2.6 feet
*Data not available

TABLE 2 - SUMMARY OF STILLWATER ELEVATIONS - continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NGVD) 10-YEAR	50-YEAR	100-YEAR	500-YEAR
ATLANTIC OCEAN/ LITTLE RIVER				
West of York Street south of Bayview Road	8.5	9.3	9.6	*
ATLANTIC OCEAN/ GODFREY'S COVE				
Northwest of Lickla Kings Road	8.5	9.3	9.6	*
ATLANTIC OCEAN/ EASTERN POINT				
East of Millbury Lane south of Eastern Point Road	8.5	9.3	9.6	*
ATLANTIC OCEAN/ YORK HARBOR				
West of Stage Neck Road between York Street and Western Point Boulevard	8.5	9.3	9.6	*
ATLANTIC OCEAN/ YORK RIVER				
Approximately 700 feet west of Brave Boat Harbor Road	7.9	8.4	8.7	9.2
ATLANTIC OCEAN/ BRAVE BOAT HARBOR				
Southwest of Raynes Neck Road north of York corporate limits	8.5	9.3	9.6	*

*Data not available

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of the flooding sources studied in detailed were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each of these flooding sources. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

Riverine Hydraulics

Cross sections for the flooding sources studied by detailed riverine methods were obtained from photogrammetric mapping while below-water sections were obtained by field survey (James W. Sewall Co., 1977). All bridges, dams, and culverts were field surveyed to obtain elevation data and structural geometry.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross-section locations are also shown on the FIRM (Exhibit 2).

Water-surface elevations of floods of the selected recurrence intervals were computed using 2 computer models. The tidal portions of the streams in York were analyzed using a one-dimensional storm surge model for coastal rivers (New England Coastal Engineers, 1977). The riverine portions of the streams studied by detailed methods utilized the USACE HEC-2 step-backwater computer program (USACE, 1973). For the streams studied by detailed riverine methods, with the exception of Bridges Swamp, the starting water-surface elevations were taken at the mean spring tide level of 5.4 feet. For Bridges Swamp, the starting water-surface elevations were computed using the slope/area method.

Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

Roughness factors (Manning's "n") used in the hydraulic computations were chosen by field inspection. The following tabulation shows the channel and overbank "n" values for the streams studied by detailed methods:

Stream	Channel "n"	Overbank "n"
Cape Neddick River	0.013-0.050	0.090
Tributary 1 to Cape Neddick River	0.013-0.050	0.090
Dolly Gordon Brook	0.013-0.050	0.050-0.090
Cider Hill Creek	0.020-0.050	0.050-0.020
Bridges Swamp	0.050	0.090

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

Coastal Hydraulics

Hydraulic analyses of the shoreline characteristics of the Atlantic Ocean were carried out to provide estimates of wave heights, corresponding wave crest elevations, and wave runup of floods of the selected recurrence intervals along the shoreline.

As of 1989, FEMA defines a "coastal high hazard area" as an area of special flood hazards extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high-velocity wave action (i.e., wave heights greater than or equal to 3 feet) from storms or seismic sources. The "primary frontal dune" is defined as a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope.

Wave height and runup calculations used in this study follow the methodologies described in FEMA's Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping (FEMA, 1995). WHAFIS 3.0 was used to predict wave heights. RUNUP 2.0 was used to predict wave runup on natural shores. Calculations based on the Shore Protection Manual (USACE, 1984) were used to predict wave runup on seawall barriers.

Wave heights and wave runup were computed along transects which were located perpendicular to the average shoreline. The transects were located with consideration given to the physical and cultural characteristics of the land so that they would closely represent conditions in their locality. Transects were spaced close together in areas of complex topography and dense development. In areas having more uniform characteristics, the transects were spaced at larger intervals. It was also necessary to locate transects in areas where unique flooding existed and in areas where computed wave heights varied significantly between adjacent transects. Figure 2 illustrates the location of the transects for the community.

Thirty-four transects were surveyed along the York coastline by Chas. H. Sells, Inc., for this revision. The surveyed transects were straightened by mathematically projecting each surveyed point onto a straight line connecting the first and last surveyed points. Bathymetric data from the U.S. Geological Survey topographic maps (USGS, 1973) was used to extend the transects offshore. Coastal processes that may affect the transect profile, such as dune erosion and seawall scour, were estimated following the Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping (FEMA, 1995). One transect was revised by Dewberry & Davis to reflect storm-induced erosion (dune removal) in the wave analysis.

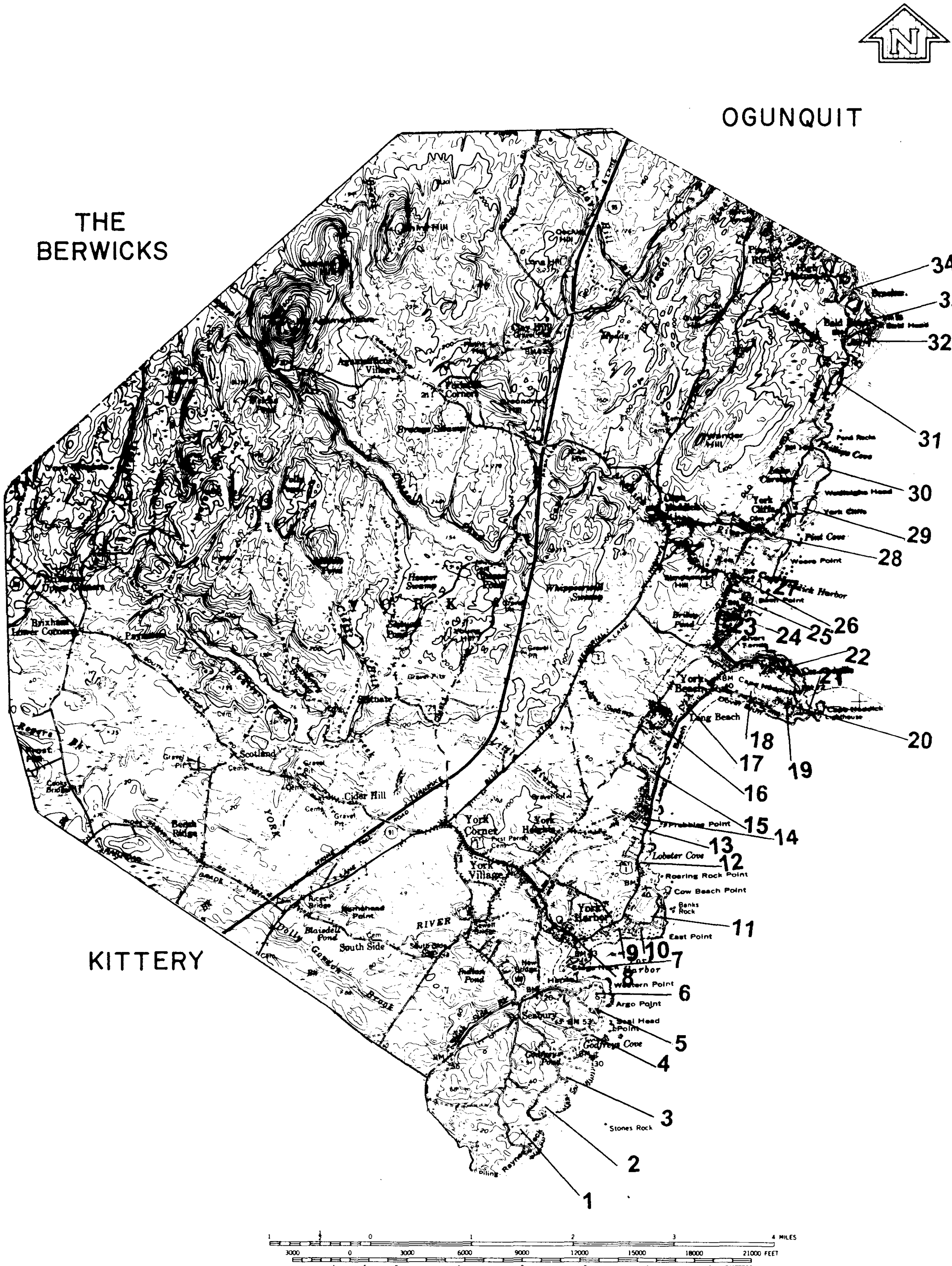
Table 3, "Transect Descriptions," provides a listing of the location of each transect and the 100-year stillwater elevations, as well as the maximum wave crest elevations at the shoreline.

TABLE 3 – TRANSECT DESCRIPTIONS

		ELEVATION (feet NGVD)	
TRANSECT	LOCATION	100-YEAR STILLWATER	MAXIMUM 100-YEAR WAVE CREST[2]
1	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 0.79 mile southeast of the intersection of Raynes Neck Road and Jungle Road	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.

[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.



TRANSECT LOCATION MAP

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME.
(YORK COUNTY)

FIGURE 3

TABLE 3 – TRANSECT DESCRIPTIONS - continued

TRANSECT	LOCATION	ELEVATION (feet NGVD) 100-YEAR STILLWATER	MAXIMUM 100-YEAR WAVE CREST[2]
2	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 0.52 mile southwest of Godfreys Cove Road and Kings Road	12.2[1]	18.7
3	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 2,000 feet southeast of the intersection of Godfreys Cove Road and Kings Road	12.2[1]	18.7
4	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,700 feet northeast of the intersection of Godfreys Cove Road and Kings Road	12.2[1]	18.7
5	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,800 feet southeast of the intersection of Western Road and Western Point Road	12.2[1]	18.7
6	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,800 feet east of the intersection of Western Road and Western Point Road	9.6	19.2[3]
7	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,200 feet southeast of the intersection of York Street and Orchard Lane	12.2[1]	18.7
8	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 650 feet southeast of the intersection of York Street and Orchard Lane	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[3]Maximum 100-year wave runup elevation.

TABLE 3 – TRANSECT DESCRIPTIONS - continued

TRANSECT	LOCATION	ELEVATION (feet NGVD) 100-YEAR STILLWATER	MAXIMUM 100-YEAR WAVE CREST[2]
9	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,000 feet southeast of the intersection of York Street and Aldis Lane	12.2[1]	18.7
10	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 900 feet southeast of the intersection of Eastern Point Road and Millbury Lane	12.2[1]	18.7
11	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 850 feet southeast of the intersection of Roaring Rock Road and Wavecrest Drive	12.2[1]	18.7
12	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 750 feet northeast of the intersection of York Street and Eureka Avenue	12.2[1]	18.7
13	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 850 feet southeast of the intersection of Hiram Street and Willard Street	9.6	21.6[3]
14	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 850 feet southeast of the intersection of Parker Street and Edwards Street	12.2[1]	18.7
15	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 300 feet southeast of the intersection of Long Beach Avenue and Webber Road	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[3]Maximum 100-year wave runup elevation.

TABLE 3 – TRANSECT DESCRIPTIONS - continued

TRANSECT	LOCATION	ELEVATION (feet NGVD) 100-YEAR STILLWATER	ELEVATION (feet NGVD) MAXIMUM 100-YEAR WAVE CREST[2]
16	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 400 feet southeast of the intersection of Long Beach Avenue and Beacon Street	12.2[1]	18.7
17	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 700 feet southeast of the intersection of Church Street and Midnight Drive	9.6	19.0[3]
18	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 625 feet southwest of the intersection of 6th Avenue and Nubble Road	12.2[1]	18.7
19	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 350 feet southwest of the intersection of Nubble Road and Shelton Avenue	9.6	20.2[3]
20	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 800 feet southeast of the intersection of Nubble Street and Cycad Avenue	12.2[1]	18.7
21	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 500 feet northeast of the intersection of Nubble Road and Cycad	9.5	19.8[3]
22	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 600 feet northeast of the intersection of Broadway Road and Kendall Road	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[3]Maximum 100-year wave runup elevation.

TABLE 3 – TRANSECT DESCRIPTIONS - continued

TRANSECT	LOCATION	ELEVATION (feet NGVD) 100-YEAR STILLWATER	ELEVATION (feet NGVD) MAXIMUM 100-YEAR WAVE CREST[2]
23	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 650 feet southeast of Main Street and Beach Street	12.2[1]	18.7
24	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 600 feet northeast of Freeman Street and Newport Road	12.2[1]	18.7
25	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 900 feet southeast of Freeman Street and Bay Haven Road	12.2[1]	18.7
26	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 900 feet northeast of the intersection of Shore Road and Bay Haven Road	12.2[1]	18.7
27	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,800 feet southeast of the intersection of Shore Road and River Road	12.2[1]	18.7
28	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 2,000 feet southeast of the intersection of Shore Road and Old Country Road	12.2[1]	18.7
29	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 3,500 feet northeast of the intersection of Shore Road and Old Country Road	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 3 – TRANSECT DESCRIPTIONS - continued

TRANSECT	LOCATION	ELEVATION (feet NGVD) 100-YEAR STILLWATER	MAXIMUM 100-YEAR WAVE CREST[2]
30	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 450 feet southeast of the intersection of Drifting Gull Road and Bayberry Lane	12.2[1]	18.7
31	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 2,650 feet south of the intersection of Shore Road and Bald Head Cliff Road	12.2[1]	18.7
32	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 1,600 feet southeast of the intersection of Shore Road and Bald Head Cliff Road	12.2[1]	18.7
33	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 2,200 feet northeast of the intersection of Shore Road and Bald Head Cliff Road	12.2[1]	18.7
34	At the shoreline of the Atlantic Ocean, in the Town of York, approximately 300 feet northeast of the intersection of Bay Road and Circuit Road	12.2[1]	18.7

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

Along each transect, wave envelopes were computed considering the combined effects of changes in ground elevation, vegetation, and physical features.

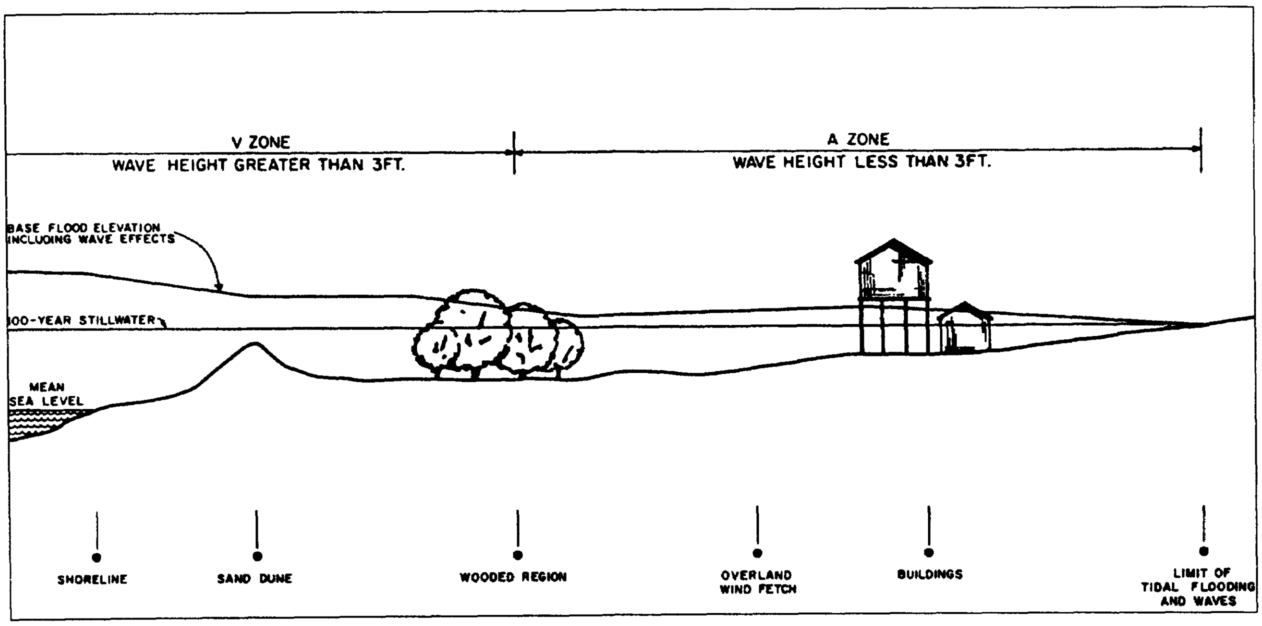
The Wave Information Study (WIS) Report 33 (USACE, 1995) was used to assess wave characteristics. WIS station 98 is the most relevant for York, and provided the 20 year maximun observed significant wave height and period values. Mean wave characteristics were determined as specified in Guidelines and Specificaitons for Wave Elevation Determination and V Zone Mapping (FEMA, 1995):

$$H_{bar} = (H_s)(0.626)$$
$$T_{bar} = (T_s)(0.85)$$

Where H_{bar} is the average wave height of all waves, H_s is the significant wave height or the average over the highest one-third of waves, T_{bar} is the average wave period, and T_s is the significant wave associated with the significant wave height.

Wave setup was calculated using the procedures detailed in the Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping (FEMA, 1995). Because much of the York coastline has experienced historical flooding and damage above predicted surge and runup elevations, setup was assumed to be an important component of the analyses and was applied to the entire exposed coastline in the Town of York. Areas of shallow flooding, designated AO zones, are shown along portions of the shoreline. These areas are the result of wave runup overtopping and ponding behind seawalls and berms for average depths of 1 to 2 feet. The results of the calculations are accurate until local topography, vegetation, or cultural development within the community undergo major changes.

Figure 3 represents a sample transect that illustrates the relationship between the stillwater elevation, the wave crest elevation, the ground profile, and the location of the A/V boundary.



TRANSECT SCHEMATIC Figure 3

Table 4, "Transect Data," the flood hazard zone and base flood elevations for each transect flooding source is provided, along with the 100-year stillwater elevation for the respective flooding source.

TABLE 4 – TRANSECT DATA

FLOODING SOURCE	STILLWATER ELEVATION (feet NGVD) 10-YEAR	50-YEAR	100-YEAR	500-YEAR	ZONE	BASE FLOOD ELEVATION (feet NGVD)[2]
ATLANTIC OCEAN						
Transect 1	8.5	9.3	12.2[1]	*	VE	15-19
	8.5	9.3	9.6	*	VE	15
					AE	15
Transect 2	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	13
					AE	13
Transect 3	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
					AO	Depth 1 foot
	8.5	9.3	9.6	*	AE	10
Transect 4	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
	8.5	9.3	9.6	*	AE	10-12
Transect 5	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	14
					AE	14
Transect 6	8.5	9.3	9.6	*	VE	19
					AE	19
Transect 7	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
Transect 8	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
	8.5	9.3	9.6	*	AE	10
Transect 9	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*Data not available.

TABLE 4 – TRANSECT DATA - continued

FLOODING SOURCE	STILLWATER ELEVATION (feet NGVD) 10-YEAR	50-YEAR	100-YEAR	500-YEAR	ZONE	BASE FLOOD ELEVATION (feet NGVD)[2]
ATLANTIC OCEAN						
Transect 10	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	14
					AE	14
Transect 11	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
					AO	Depth 2 feet
Transect 12	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
Transect 13	8.5	9.3	9.6	*	VE	22
					AE	22
					AO	Depth 1 foot
					AE	10
Transect 14	8.5	9.3	12.2[1]	*	VE	18-19
	8.5	9.3	9.6	*	VE	18
					AE	18
					AO	Depth 1 foot
Transect 15	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
Transect 16	8.5	9.3	12.2[1]	*	VE	17-19
	8.5	9.3	9.6	*	VE	17
					AE	17
					AO	Depth 1 foot
					AE	10
Transect 17	8.5	9.3	9.6	*	VE	19
					AE	19
					AO	Depth 1 foot
					AE	10
Transect 18	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*Data not available.

TABLE 4 – TRANSECT DATA - continued

FLOODING SOURCE	STILLWATER ELEVATION (feet NGVD)				ZONE	BASE FLOOD ELEVATION (feet NGVD)[2]
	10-YEAR	50-YEAR	100-YEAR	500-YEAR		
ATLANTIC OCEAN						
Transect 19	8.5	9.3	9.6	*	VE	20
					AE	20
Transect 20	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
Transect 21	8.5	9.3	9.6	*	VE	20
					AE	20
Transect 22	8.5	9.3	12.2[1]	*	VE	17-19
	8.5	9.3	9.6	*	VE	17
					AE	17
Transect 23	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
					AO	Depth 2 feet
Transect 24	8.5	9.3	12.2[1]	*	VE	15-19
	8.5	9.3	9.6	*	VE	15
					AE	15
Transect 25	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	13
					AE	13
Transect 26	8.5	9.3	12.2[1]	*	VE	14-19
					AE	12-14
Transect 27	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	14
					AE	14
Transect 28	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.6	9.6	*	VE	14
					AE	14
					AO	Depth 1 foot
					AE	10

[1]Includes wave setup of 2.6 feet.
[2]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*Data not available.

TABLE 4 – TRANSECT DATA - continued

FLOODING SOURCE	STILLWATER ELEVATION (feet NGVD) 10-YEAR	50-YEAR	100-YEAR	500-YEAR	ZONE	BASE FLOOD ELEVATION (feet NGVD)[2]
ATLANTIC OCEAN						
Transect 29	8.5	9.3	12.2[1]	*	VE	16-19
	8.5	9.3	9.6	*	VE	16
					AE	16
Transect 30-31	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	13
					AE	13
Transect 32	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	14
					AE	14
Transect 33	8.5	9.3	12.2[1]	*	VE	15-19
	8.5	9.3	9.6	*	VE	15
					AE	15
Transect 34	8.5	9.3	12.2[1]	*	VE	14-19
	8.5	9.3	9.6	*	VE	14
					AE	14

After analyzing the wave heights along each transect, wave elevations were interpolated between transects. Various source data were used for interpolation, including topographic maps and engineering judgment. Controlling features affecting the elevations were identified and considered in relation to their position at a particular transect and their variation between transects.

All elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). Elevation reference marks (ERMs) used in this study, and their descriptions, are shown on the FIRM. ERMs shown on the FIRM represent those used during the preparation of this and previous FISs. The elevations associated with each ERM were obtained and/or developed during FIS production to establish vertical control for determination of flood elevations and floodplain boundaries shown on the FIRM. Users should be aware that these ERM elevations may have changed since the publication of this FIS. To obtain up-to-date elevation information on National Geodetic Survey (NGS) ERMs shown on this map, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit their website at www.ngs.noaa.gov. Map users should seek verification of non-NGS ERM monument elevations when using these elevations for construction or floodplain management purposes.

3.3 Vertical Datum

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FISs and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD29). With the finalization of the North American Vertical Datum of 1988 (NAVD88), many FIS reports and FIRMs are being prepared using NAVD88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to NGVD29. Structure and ground elevations in the community must, therefore, be referenced to NGVD29. It is important to note that adjacent communities may be referenced to NAVD88. This may result in differences in base flood elevations across the corporate limits between the communities.

For more information on NAVD of 1988, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992, or contact the Vertical Network Branch, National Geodetic Survey, Coast and Geodetic Survey, National Oceanic and Atmospheric Administration, Rockville, Maryland 20910 (Internet address http://www.ngs.noaa.gov).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 100-year floodplain data, which may include a combination of the following: 10-, 50-, 100-, and 500-year flood elevations; delineations of the 100-year and 500-year floodplains; and 100-year floodway. This information is presented on the FIRM and in many components of the FIS, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent annual chance (100-year) flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent annual chance (500-year) flood is employed to indicate additional areas of flood risk in the community. For the streams studied in detail, the 100- and 500-year floodplains have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:2,400 with a contour interval of 5 feet (James W. Sewall Co., 1977).

For this revision, the boundaries were interpolated between coastal transects using aerial photogrammetric topographic maps at a scale of 1"=400' with a contour interval of 4 feet (Chas. H. Sells, Inc., July 1996).

For the areas studied by approximate methods, the boundaries of the 100-year floodplain were delineated using Flood Hazard Boundary Map for the Town of York (U.S. Department of Housing and Urban Development, 1976).

The 100- and 500-year floodplain boundaries are shown on the FIRM (Exhibit 2). On this map, the 100-year floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones AO, VE, A, and AE), and the 500-year floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 100- and 500-year floodplain boundaries are close together, only the 100-year floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 100-year floodplain boundaries are shown on the FIRM (Exhibit 2).

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 100-year floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 100-year flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as a minimum standard that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this study were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (Table 5). The computed floodways are shown on the revised FIRM (Exhibit 2). Since it was determined that the 100-year tidal flood elevation on the York River is greater than the elevation produced by riverine flooding for the length studied by detailed methods, no HEC-2 analysis was done for the York River. Therefore, the York River floodway was delineated as coincident with channel banks based on engineering judgement. In cases where the floodway and 100-year floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION (FEET NGVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Dolly Gordon Brook								
A	1.33[1]	20	97	1.8	9.4	9.4	9.4	0.0
B	1.42[1]	52	213	0.8	9.5	9.5	9.6	0.1
C	1.55[1]	10	23	7.8	11.1	11.1	11.6	0.5
D	1.57[1]	18	53	3.5	13.1	13.1	14.1	1.0
E	1.61[1]	18	131	1.4	18.4	18.4	18.4	0.0
Cider Hill Creek								
A	1.23[1]	35	252	2.3	8.8	8.8	8.8	0.0
B	1.30[1]	30	145	4.0	9.0	9.0	9.0	0.0
C	1.89[1]	102	402	1.4	11.8	11.8	12.8	1.0
Bridges Swamp								
A	0.018[2]	26	62	4.0	9.6	9.6	10.6	1.0
B	0.068[2]	250	901	0.3	10.0	10.0	11.0	1.0
C	0.158[2]	10	27	9.3	11.7	11.7	12.2	0.5
D	0.218[2]	38	96	2.6	16.6	16.6	17.5	0.9
E	0.258[2]	8	18	13.7	21.2	21.2	21.8	0.6
F	0.278[2]	17	34	7.3	30.7	30.7	31.7	1.0
G	0.338[2]	27	63	4.0	35.1	35.1	35.3	0.2
H	0.368[2]	27	338	0.7	46.6	46.6	46.7	0.1
I	0.418[2]	10	133	1.9	46.6	46.6	46.7	0.1
J	0.518[2]	7	86	2.9	46.6	46.6	46.8	0.2
K	0.677[2]	199	520	0.5	46.6	46.6	47.5	0.9

[1]Miles above confluence with York River
[2]Miles above Long Beach Avenue

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

FLOODWAY DATA

DOLLY GORDON BROOK – CIDER HILL CREEK – BRIDGES SWAMP

TABLE 5

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION (FEET NGVD)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Cape Neddick River								
A	0.820[1]	30	203	4.7	9.6[3]	8.6	9.4	0.8
B	0.860[1]	20	83	11.5	12.4	12.4	12.7	0.3
C	0.900[1]	43	143	6.7	16.3	16.3	17.2	0.9
D	0.940[1]	23	131	7.3	19.6	19.6	19.6	0.0
E	0.970[1]	68	345	2.8	20.9	20.9	21.2	0.3
F	1.117[1]	17	115	8.2	23.0	23.0	23.6	0.6
G	1.197[1]	44	328	2.9	24.7	24.7	25.5	0.8
H	1.218[1]	44	402	2.4	27.3	27.3	27.3	0.0
I	1.317[1]	120	596	1.6	27.5	27.5	27.6	0.1
Tributary 1 to Cape Neddick River								
A	0.012[2]	14	74	3.7	27.7	27.7	28.2	0.5
B	0.082[2]	61	146	1.9	28.5	28.5	29.4	0.9
C	0.142[2]	88	71	3.9	30.4	30.4	30.6	0.2
D	0.172[2]	10	28	9.6	36.7	36.7	36.7	0.0
E	0.202[2]	10	31	8.9	41.9	41.9	42.2	0.3
F	0.312[2]	55	108	2.5	47.9	47.9	48.2	0.3
G	0.341[2]	55	340	0.8	52.4	52.4	52.4	0.0
H	0.596[2]	9	28	9.9	54.2	54.2	54.6	0.4

[1]Miles above Shore Road
[2]Miles above confluence with Cape Neddick River
[3]Tidal flooding from the Atlantic Ocean

TABLE 5

FEDERAL EMERGENCY MANAGEMENT AGENCY

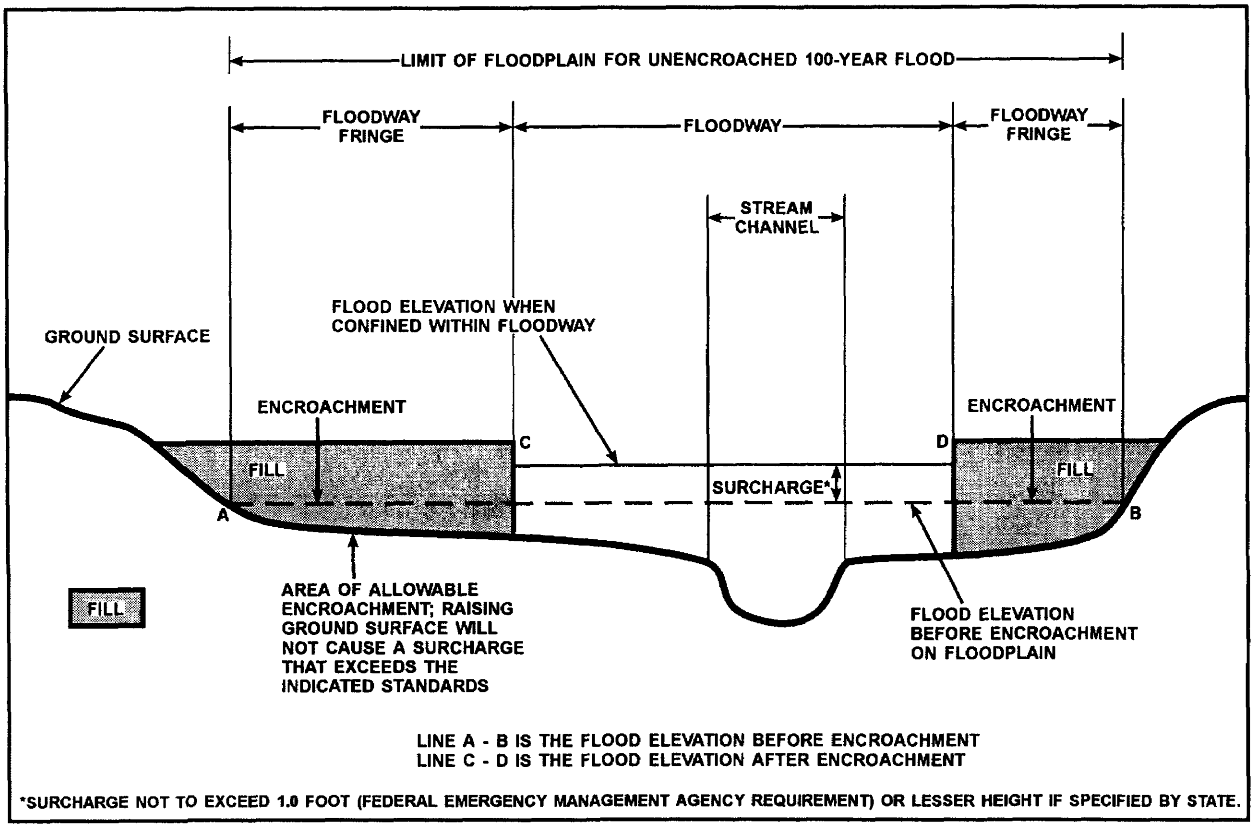
TOWN OF YORK, ME
(YORK CO.)

FLOODWAY DATA

CAPE NEDDICK RIVER – TRIBUTARY 1 TO CAPE NEDDICK RIVER

Encroachment into areas subject to inundation by floodwaters having hazardous velocities aggravates the risk of flood damage, and heightens potential flood hazards by further increasing velocities. A listing of stream velocities at selected cross sections is provided in Table 5, "Floodway Data." To reduce the risk of property damage in areas where the stream velocities are high, the community may wish to restrict development in areas outside the floodway.

The area between the floodway and 100-year floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 100-year flood by more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 4, "Floodway Schematic."



FLOODWAY SCHEMATIC Figure 4

Near the mouths of streams studied in detail, floodway computations are made without regard to flood elevations on the receiving water body. Therefore, "Without Floodway" elevations presented in Table 5 for certain downstream cross sections of the Cape Neddick River are lower than the regulatory flood elevations in that area, which must take into account the 100-year flooding due to backwater from other sources.

5.0 INSURANCE APPLICATIONS

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. The zones are as follows:

Zone A

Zone A is the flood insurance rate zone that corresponds to the 100-year floodplains that are determined in the FIS by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no base flood elevations or depths are shown within this zone.

Zone AE

Zone AE is the flood insurance rate zone that corresponds to the 100-year floodplains that are determined in the FIS by detailed methods. In most instances, whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AH

Zone AH is the flood insurance rate zone that corresponds to the areas of 100-year shallow flooding (usually areas of ponding) where average depths are between 1 and 3 feet. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AO

Zone AO is the flood insurance rate zone that corresponds to the areas of 100-year shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot depths derived from the detailed hydraulic analyses are shown within this zone.

Zone A99

Zone A99 is the flood insurance rate zone that corresponds to areas of the 100-year floodplain that will be protected by a Federal flood protection system where construction has reached specified statutory milestones. No base flood elevations or depths are shown within this zone.

Zone V

Zone V is the flood insurance rate zone that corresponds to the 100-year coastal floodplains that have additional hazards associated with storm waves. Because approximate hydraulic analyses are performed for such areas, no base flood elevations are shown within this zone.

Zone VE

Zone VE is the flood insurance rate zone that corresponds to the 100-year coastal floodplains that have additional hazards associated with storm waves. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone X

Zone X is the flood insurance rate zone that corresponds to areas outside the 500-year floodplain, areas within the 500-year floodplain, and areas of 100-year flooding where average depths are less than 1 foot, areas of 100-year flooding where the contributing drainage area is less than 1 square mile, and areas protected from the 100-year flood by levees. No base flood elevations or depths are shown within this zone.

Zone D

Zone D is the flood insurance rate zone that corresponds to unstudied areas where flood hazards are undetermined, but possible.

6.0 FLOOD INSURANCE RATE MAP

The FIRM is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in Section 5.0 and, in the 100-year floodplains that were studied by detailed methods, shows selected whole-foot base flood elevations or average depths. Insurance agents use the zones and base flood elevations in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 100- and 500-year floodplains. Floodways and the locations of selected cross sections used in the hydraulic analyses and floodway computations are shown where applicable. This FIRM includes flood hazard information that was presented separately on the Flood Boundary and Floodway Map in the previously printed FIS for the Town of York.

7.0 OTHER STUDIES

FISs have been prepared for the Towns of Eliot, Kittery, North Berwick, South Berwick, and Wells, and the Village of Ogunquit (FEMA, June 1989; FEMA, July 1986; FEMA, August 1984; FEMA, December 1984; FEMA, Town of Wells, January 1983; and FEMA, Village of Ogunquit, January 1983).

Because it is based on more up-to-date analyses, this FIS supersedes the previously printed FIS for the Town of York (FEMA, September 1983). This FIS also supersedes the Flood Boundary and Floodway Map for the Town of York, which was published as part of the previously printed FIS. The information on the Flood Boundary and Floodway Map has been added to the FIRM accompanying this FIS.

8.0 LOCATION OF DATA

Information concerning the pertinent data used in preparation of this study can be obtained by contacting FEMA, Federal Insurance and Mitigation Administration, J.W. McCormack Post Office and Courthouse Building, Room 462, Boston, Massachusetts 02109.

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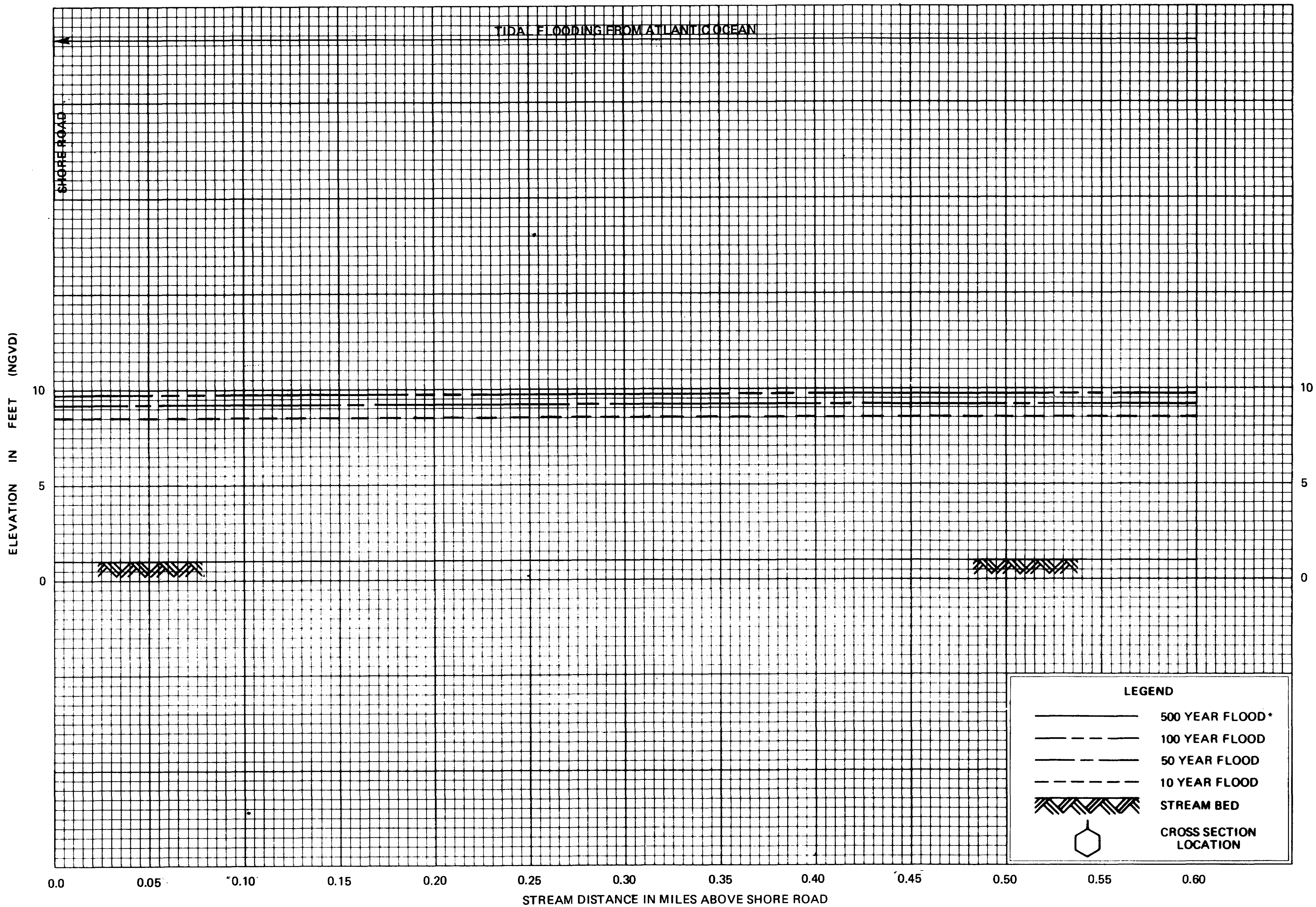


*DATA NOT AVAILABLE

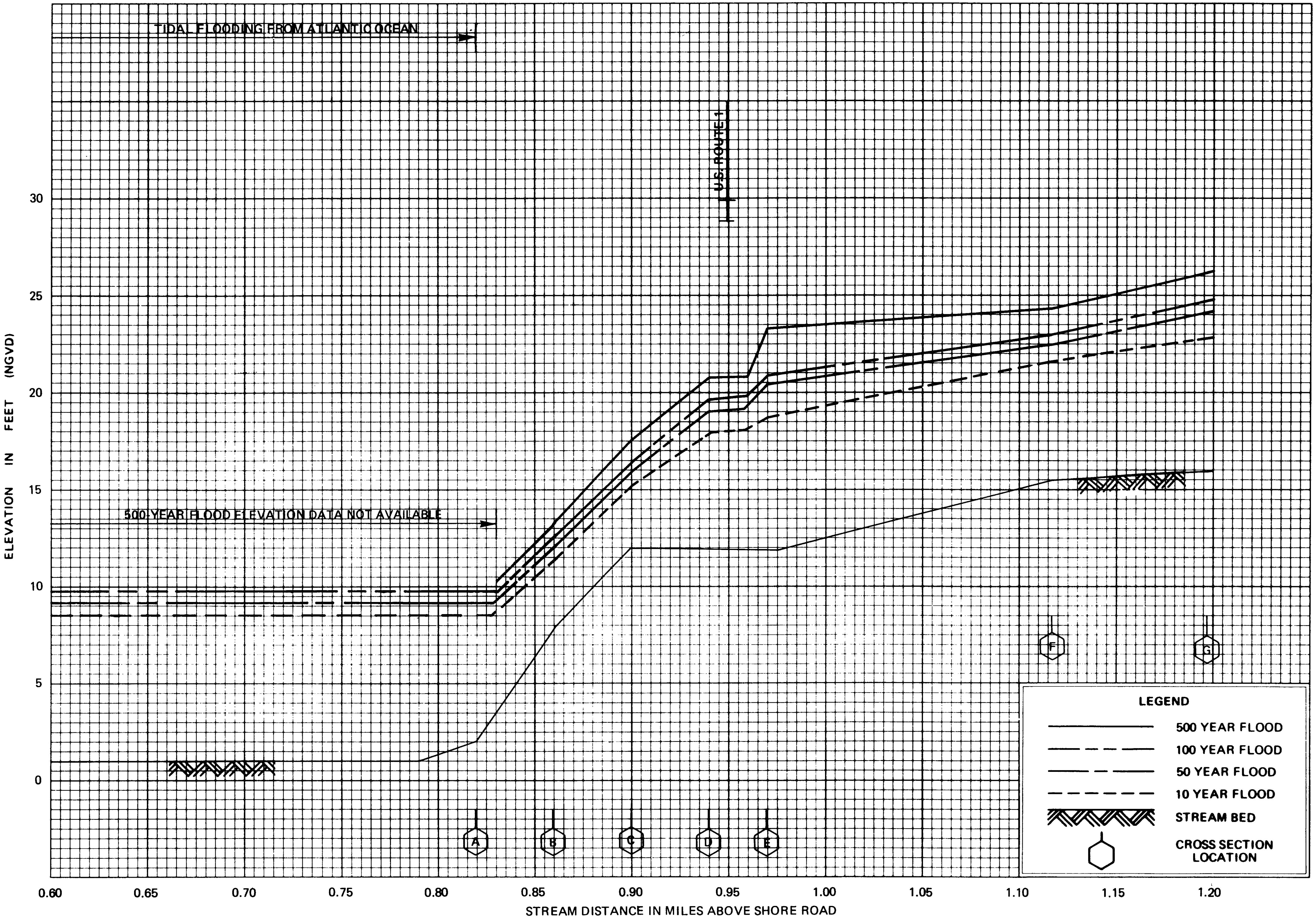
FLOOD PROFILES

CAPE NEDDICK RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

01P



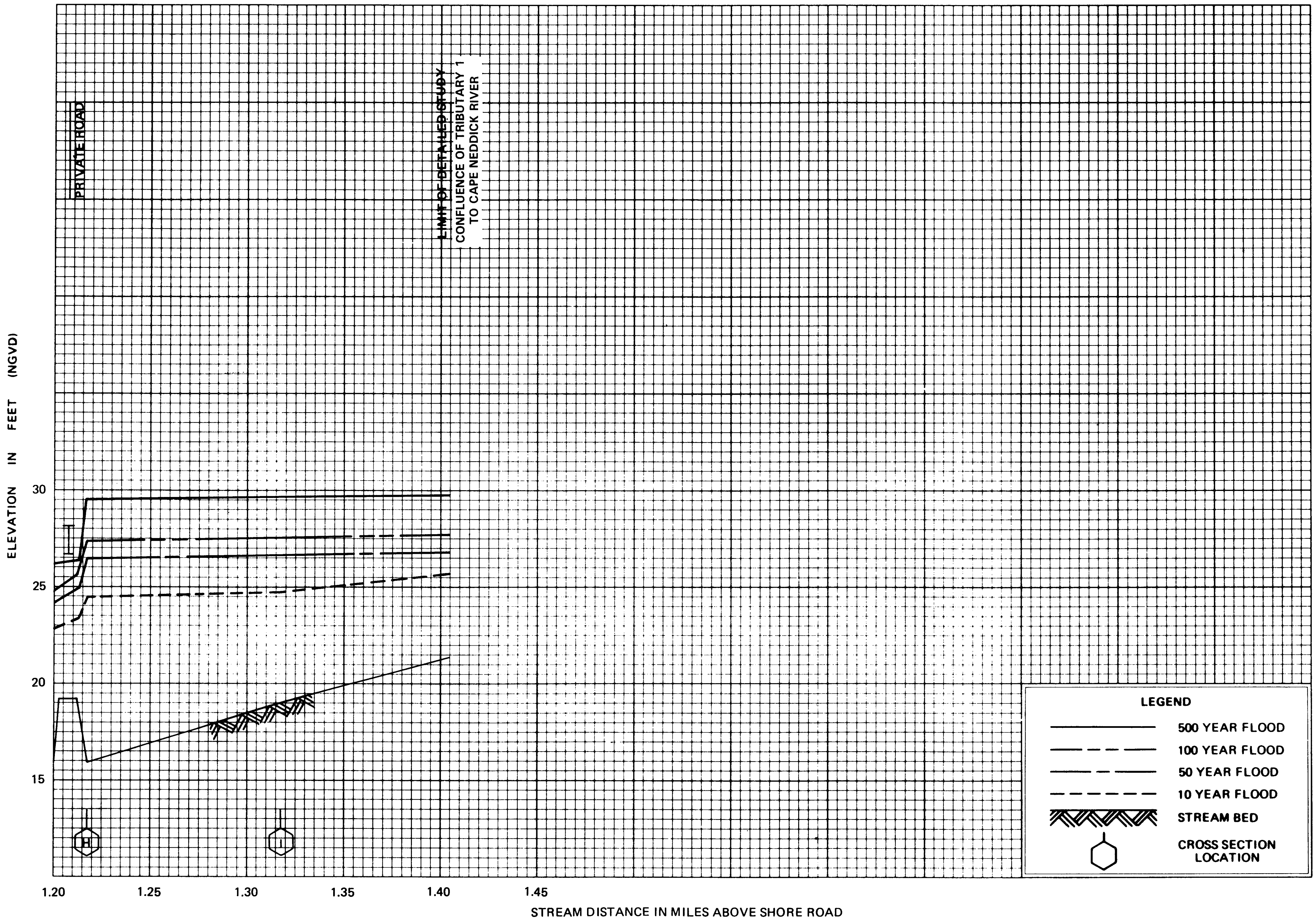
FLOOD PROFILES

CAPE NEDDICK RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

02P



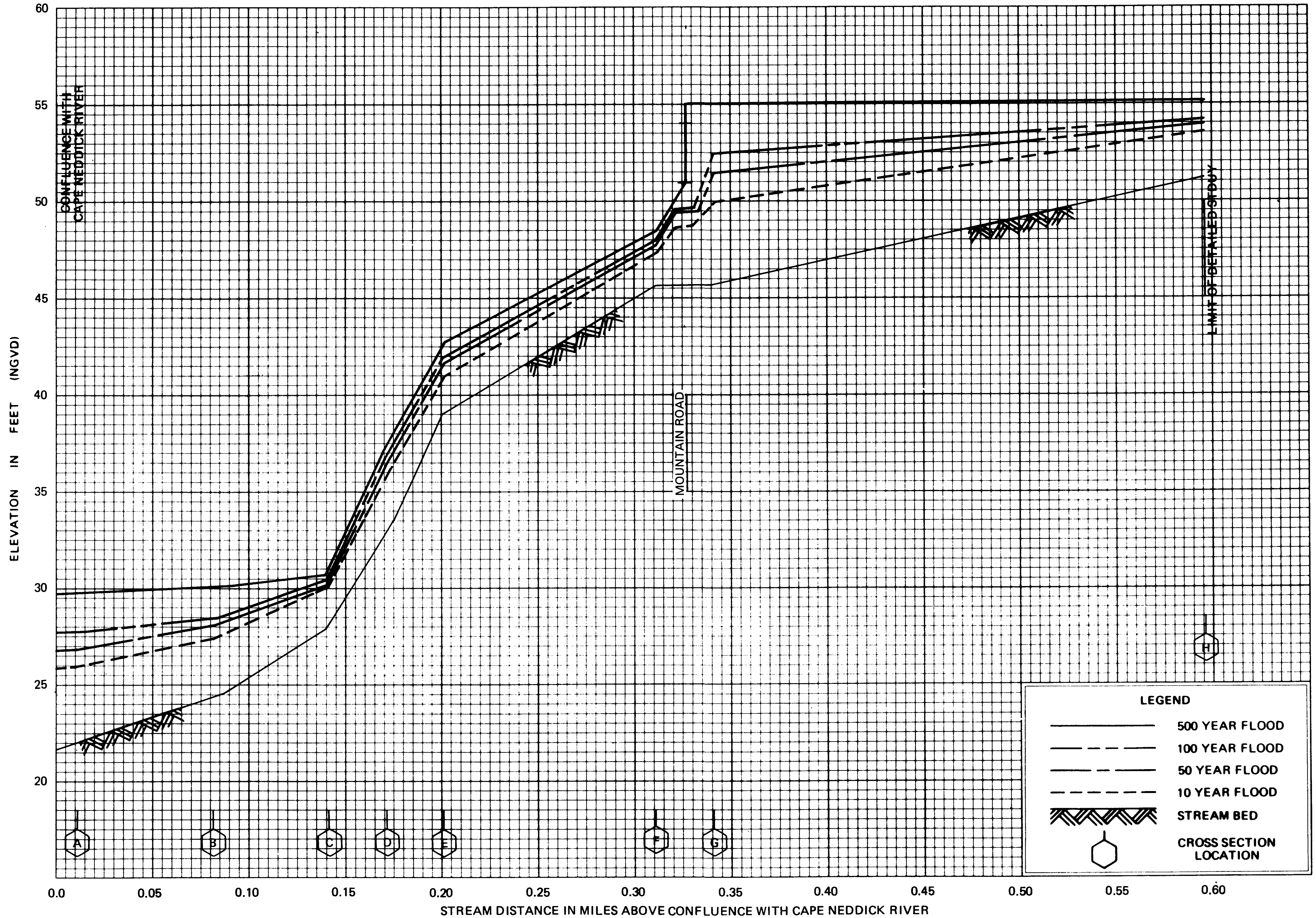
FLOOD PROFILES

CAPE NEDDICK RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

03P



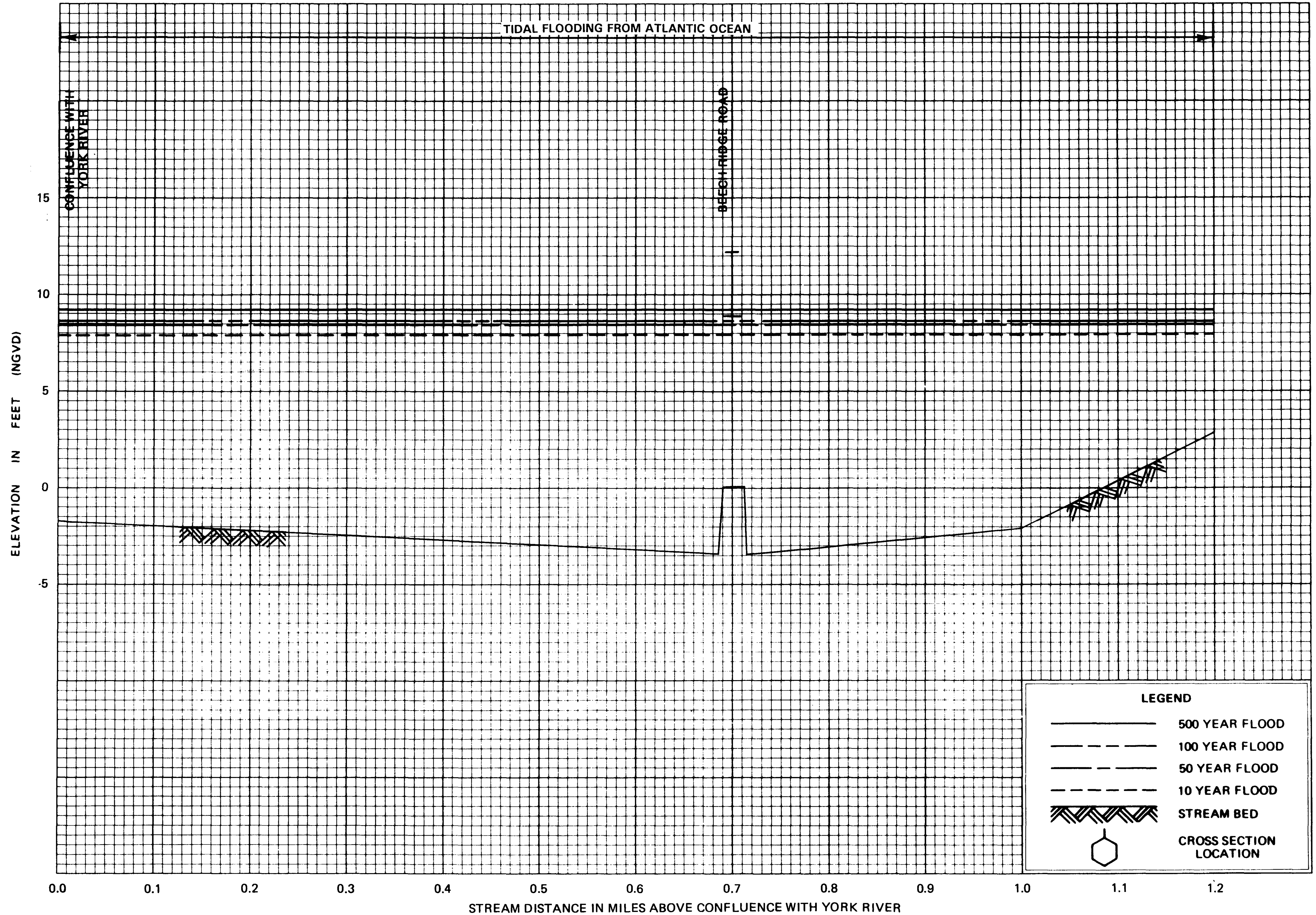
FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

FLOOD PROFILES

TRIBUTARY 1 TO CAPE NEDDICK RIVER

04P



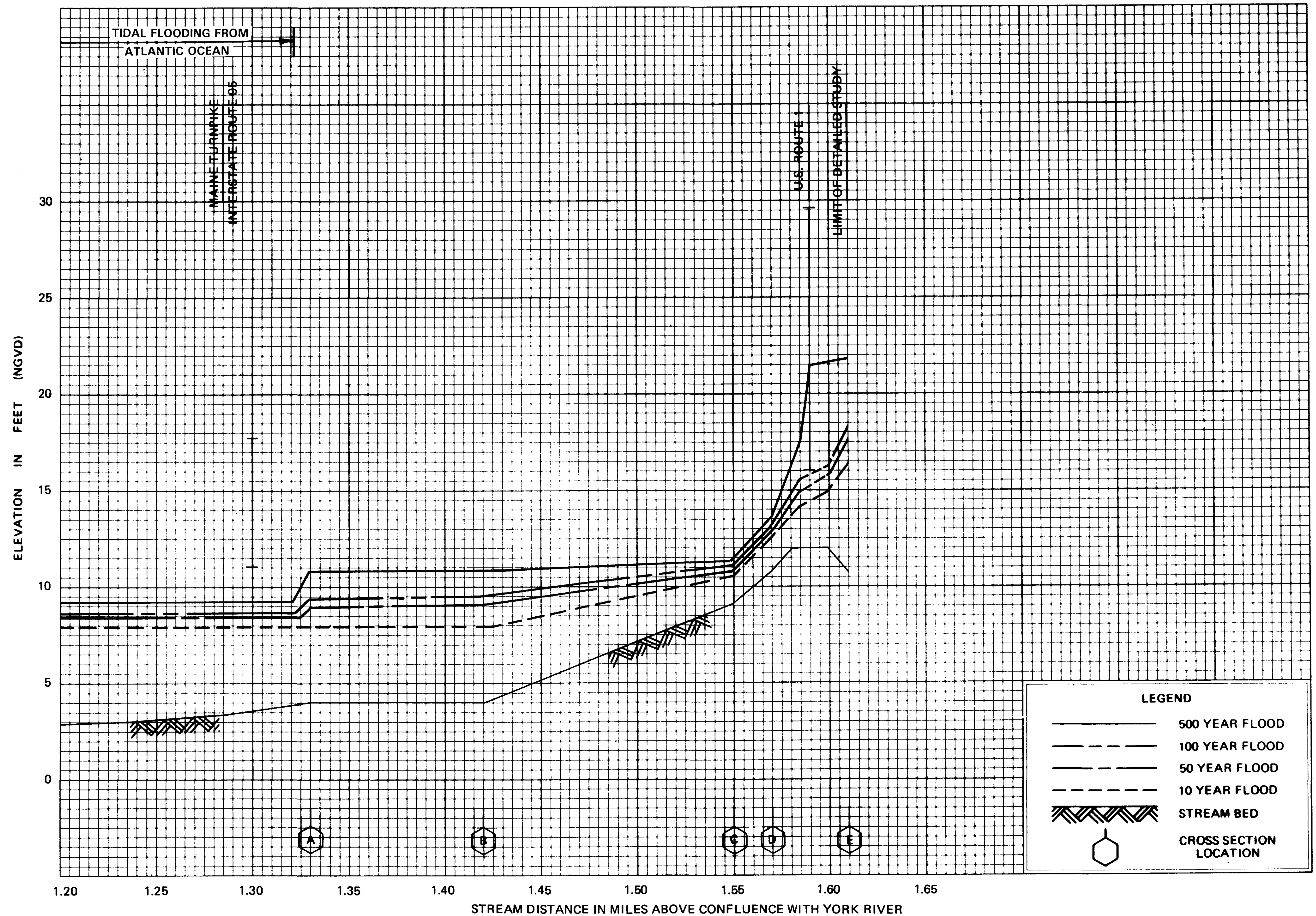
FLOOD PROFILES

DOLLY GORDON BROOK

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

05P



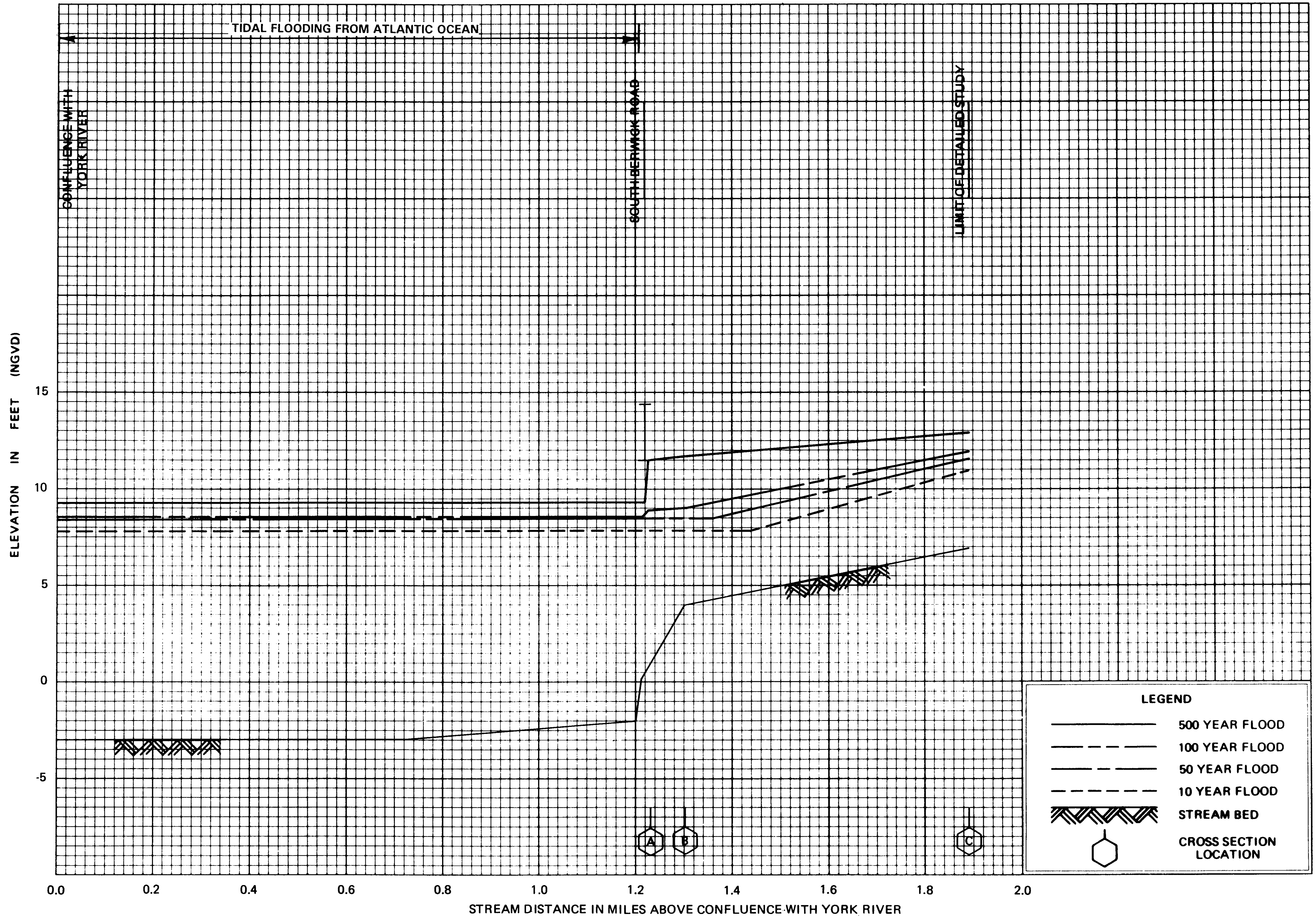
FLOOD PROFILES

DOLLY GORDON BROOK

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

06P

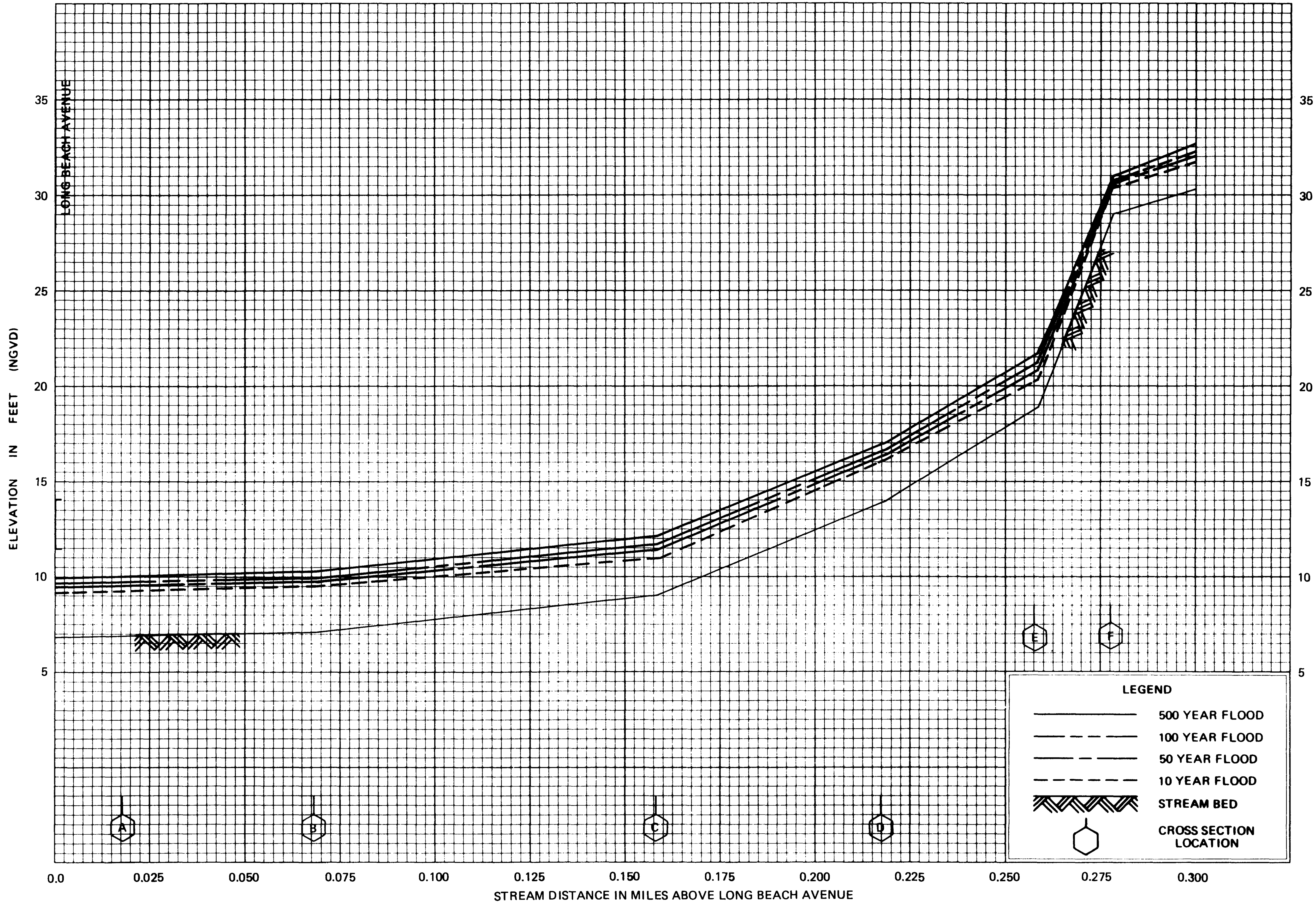


FLOOD PROFILES

CIDER HILL CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)



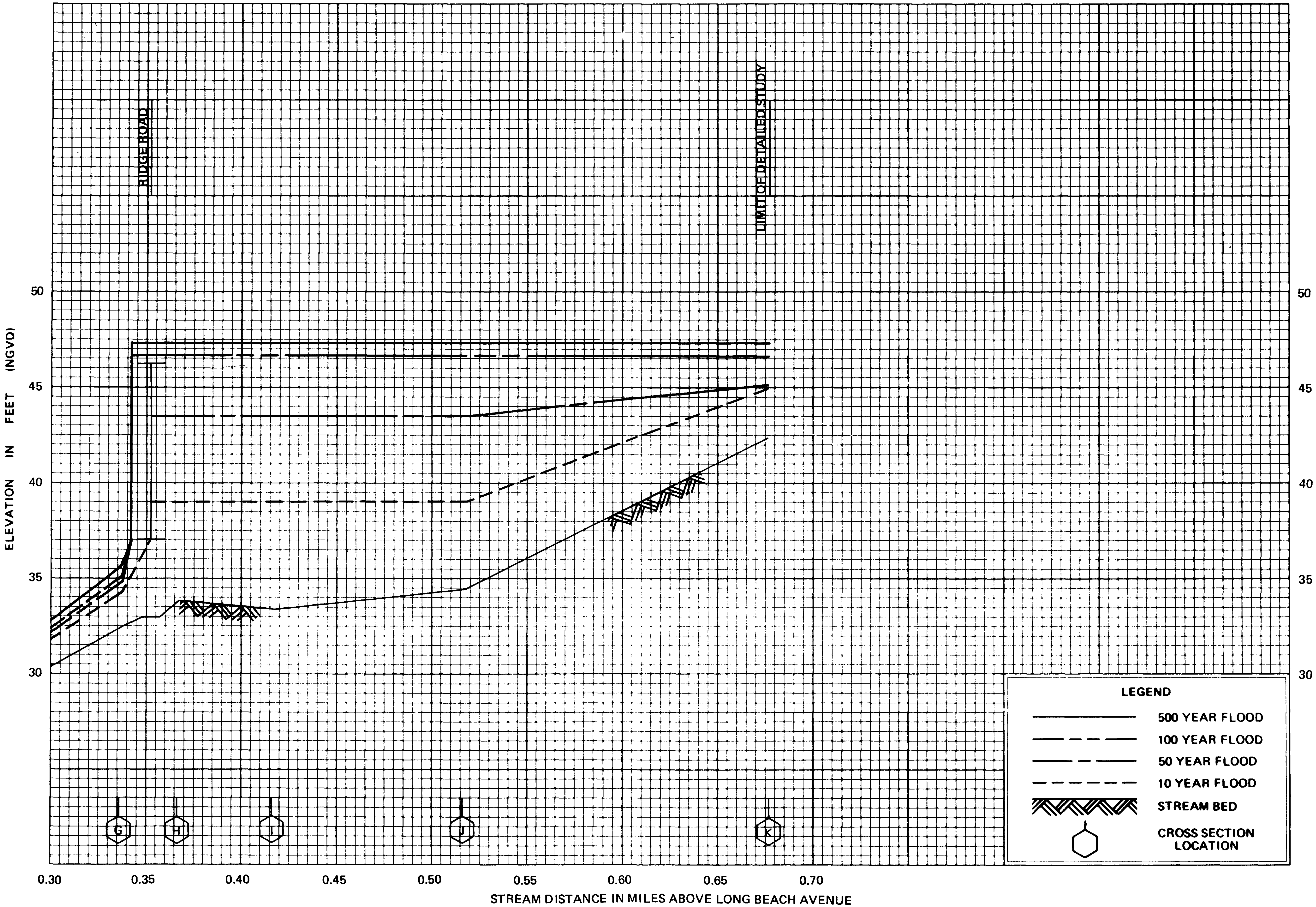
FLOOD PROFILES

BRIDGES SWAMP

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME
(YORK CO.)

08P



FLOOD PROFILES

BRIDGES SWAMP

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF YORK, ME

(YORK CO.)

09P

USGS
science for a changing world

Prepared in cooperation with the Federal Emergency Management Agency

Flood of May 2006 in York County, Maine

Scientific Investigations Report 2008–5047

U.S. Department of the Interior
U.S. Geological Survey

Cover Photo: Bridge failure on Shore Road over Cape Neddick River, York, Maine.

Flood of May 2006 in York County, Maine

By Gregory J. Stewart and Joshua P. Kempf

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U.S. Department of the Interior
U.S. Geological Survey

U.S. Department of the Interior
DIRK KEMPTHORNE, Secretary

U.S. Geological Survey
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Contents

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Plate

Conversion Factors

Multiply	By	To obtain
	Length	
inch (in.)	2.54	centimeter (cm)
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
	Area	
square mile (mi^2)	259.0	hectare (ha)
square mile (mi^2)	2.590	square kilometer (km^2)
	Volume	
cubic foot (ft^3)	0.02832	cubic meter (m^3)
	Flow rate	
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

Flood of May 2006 in York County, Maine

By Gregory J. Stewart and Joshua P. Kempf

Abstract

A stalled low-pressure system over coastal New England on Mother's Day weekend, May 13–15, 2006, released rainfall in excess of 15 inches. This flood (sometimes referred to as the "Mother's Day flood") caused widespread damage to homes, businesses, roads, and structures in southern Maine. The damage to public property in York County was estimated to be $7.5 million. As a result of these damages, a presidential disaster declaration was enacted on May 25, 2006, for York County, Maine. Peak-flow recurrence intervals for eight of the nine streams studied were calculated to be greater than 500 years. The peak-flow recurrence interval of the remaining stream was calculated to be between a 100-year and a 500-year interval.

This report provides a detailed description of the May 2006 flood in York County, Maine. Information is presented on peak streamflows and peak-flow recurrence intervals on nine streams, peak water-surface elevations for 80 high-water marks at 25 sites, hydrologic conditions before and after the flood, comparisons with published Flood Insurance Studies, and places the May 2006 flood in context with historical floods in York County.

At sites on several streams, differences were observed between peak flows published in the Flood Insurance Studies and those calculated for this study. The differences in the peak flows from the published Flood Insurance Studies and the flows calculated for this report are within an acceptable range for flows calculated at ungaged locations, with the exception of those for the Great Works River and Merriland River. For sites on the Mousam River, Blacksmith Brook, Ogunquit River, and Cape Neddick River, water-surface elevations from Flood Insurance Studies differed with documented water-surface elevations from the 2006 flood.

Introduction

Typically, rainfall is the most important factor contributing to extreme floodflows on streams in Maine. Total rainfall amounts, rainfall intensity, and spatial distribution of rainfall in a drainage basin are all important in determining the magnitude of floodflows. Antecedent hydrologic conditions (such as snowpack water equivalent, streamflow, stream-ice thickness, and soil moisture) can contribute to the intensity of a particular flood. The flood of May 10–17, 2006, was solely the result of extraordinary rainfall caused by a stalled low-pressure system that continually circulated moisture over southern Maine for more than 5 days. Many bridges and culverts were compromised in the municipalities of Cape Neddick, Wells, and Ogunquit as widespread flooding occurred in southern Maine (fig. 1). Federal disaster aid for York County was authorized under a disaster declaration issued by the President of the United States, George W. Bush.

Flood-related data are useful for many purposes. The Federal Emergency Management Agency (FEMA) and the Maine Emergency Management Agency (MEMA) require timely information on the magnitude and recurrence intervals of flooding to facilitate the mitigation processes for flood damage. Peak-flow magnitudes and recurrence intervals are used for land-use planning, including flood-plain boundary delineation and the design of bridges, culverts, and structures in the floodplain. Peak flows, peak-flow recurrence intervals, and peak water-surface elevations (associated with the peak flows) are used to establish, assess, and verify the accuracy of FEMA values for 100-year and 500-year floodplain limits and flood profiles. These values, published by FEMA in Flood Insurance Studies, are used to promote sound floodplain management by local, State, and Federal officials. Flood data also are used for other scientific purposes, including the study of the effects of climate change and land use on hydrologic regimes and ecosystems.

Documented flood data are needed for future planning and assessment, especially in the protection of life and property. During 2006–07, the U.S. Geological Survey (USGS), in cooperation with FEMA, documented and quantified a large amount of data associated with the Mother's Day flood. This report provides a detailed description of the May 2006 flood in York County, Maine. The meteorological characteristics of the May 10 to 17, 2006, storm are summarized, including rainfall amounts and their associated recurrence intervals. Hydrologic conditions (streamflow and rainfall) in the area prior to the storm are described. This report gives peak water-surface elevations for 80 high-water marks at 25 sites, and peak streamflows and associated recurrence intervals for 9 sites during the May 2006 flood. Peak flows and peak water-surface elevations in previous FEMA Flood Insurance Studies are compared to values obtained for the May 2006 flood.

Hydrologic Conditions Prior to Flooding

The hydrologic conditions in a watershed prior to a storm can be important in determining the severity of flooding. For example, elevated streamflows prior to extreme rainfall would result in higher floodflows than would occur if streamflows had been low. Only minimal data are available on conditions prior to the extreme rainfall on May 10–17, 2006. Available data on rainfall and streamflows are presented in this section.

Rainfall

The National Weather Service operates recording and observer-operated precipitation gages throughout York County, Maine. Daily rainfall totals at these gages were compared to one another, as well as to the monthly departure from normal at individual gages. The rainfall during the month of April 2006 was less than normal (Tom Hawley, National Weather Service, oral commun., 2006).

Streamflows

Currently (2006), there are no active continuous-record streamflow-gaging stations in York County, Maine. The three closest USGS streamflow gaging stations were chosen to represent general streamflow conditions before the flood in southern York County. The station at Stony Brook at East Sebago, Maine (Cumberland County), had an April 2006 monthly mean flow of 1.98 ft^3/s, which is less than the long-term April monthly mean flow of 5.1 ft^3/s, based on 11 years of record. The Winnicut River at Greenland, near Portsmouth, New Hampshire, had an April 2006 monthly mean flow of 29.4 ft^3/s, which is less than the April mean monthly flow of 60 ft^3/s, based on 4 years of record. The Cocheco River near Rochester, New Hampshire, had an April 2006 monthly mean flow of 150 ft^3/s, below the April mean monthly flow of 316 ft^3/s, based on 12 years of record. These three streamflow gages indicate that overall antecedent flow conditions were below normal, based on 4 to 12 years of record and the flow calculations for April 2006.

May 2006 Rainfall and Rainfall Recurrence Intervals

Higher than normal rainfall occurring May 10 to 17 (fig. 2)—most of which was received from the Mother's Day storm on May 13 to 15 (fig. 1)—was the primary contributor to the flood. It was the wettest month of May recorded in Portland, Cumberland County, Maine (which borders York County to the northeast) since 1871. A record precipitation amount of 12.34 in. was measured for the month, breaking the 1984 monthly record of 9.64 in. Portland had 8.52 in. more rain than normal during the month of May 2006 (National Oceanic and Atmospheric Administration, 1996). May 2006 monthly totals for Cape Neddick, Eliot, Sanford, Kennebunkport, Hollis, and W. Buxton in York County were 20.26 in., 17.86 in., 15.54 in., 14.43 in., 14.27 in., 12.63 in., respectively.

Rainfall totals from May 13 to 15 (3-day) and May 10 to 17 (8-day) are shown as lines of equal rainfall (isolines) in figures 1 and 2. These isolines were computed by the National Weather Service (NWS) using the monitoring sites shown in figure 1 and 2. Although Portland received a total of 7.04 in. of rain during May 10–17, Cape Neddick received 15.96 in. because it was closer to the centroid of greatest rainfall (Tom Hawley, National Weather Service, oral commun., 2006).

Before rainfall started on May 10, 2006, a blocking weather pattern formed over the eastern United States. As this pattern fully developed, it enabled the continuous circulation of moisture over much of New England. Eight-day rainfalls of 12 to 14 in. were common throughout York County, Maine (Tom Hawley, National Weather Service, written commun., 2006).

Although most of the intense rainfall was received during May 13–15, the ground became saturated because of rainfall that began as early as May 9th. After the intense rain began to taper off, flooding in rivers and streams was prolonged because lesser amounts of rainfall continued for days afterward (Tom Hawley, National Weather Service, written commun., 2006).

The Atlas of Precipitation Extremes for the Northeastern United States and Southeastern Canada (Wilks and Cember, 1993) contains maps with recurrence-interval isolines for 1-, 2-, 4-, 7-, and 10-day rainfall totals. The T-year (where T equals 100, 50, 25, 10, 5, and 2) rainfall recurrence intervals for southern Maine were interpolated from these maps. The recurrence interval is the average period of time between rainfalls that are greater than, or equal to, a specified magnitude. As an example of recurrence interval, the 100-year rainfall is the rainfall that, on long-term average, would be equaled or exceeded once every 100 years. This means that there is a 1.0 percent chance every year that a rainfall of this magnitude will be equaled or exceeded. The 100-year rainfall totals for 2-day, 4-day, 7-day, and 10-day periods in southern Maine are 8 in., 9 in., 11 in., and 12 in., respectively (Wilks and Cember, 1993). The average rainfall, based on the National Weather Service precipitation records, received during the Mother's Day flood of May 2006 exceeded the 100-year recurrence interval for 2-day, 4-day, 7-day, and 10-day rainfall totals at many locations throughout southern York County (especially Cape Neddick, Eliot, and Sanford).

Figure 1. Location of study area and total precipitation for 3-day period from May 13 through May 15, 2006, York County, Maine.

Figure 2. Total precipitation for 8-day period from May 9 through May 15, 2006, York County, Maine.

May 2006 Peak Stream Elevations and Flows

The rain that fell in York County, Maine, during May 10–17, 2006, resulted in extreme water-surface peak elevations and peak streamflows, which are described in the following sections. The photographs in figures 3 and 4 show examples of the damages from the flooding.

Peak Water-Surface Elevations

Peak water-surface elevations were determined for 80 points at 25 sites in York County, Maine, following the May 2006 flood (table 1). The 25 site locations and 80 points are shown on plate 1. Most of the sites are at bridges or culverts, which typically have a large effect on peak water-surface elevations. For example, bridges and culverts often cause constrictions in a stream during floodflows that result in higher peak water-surface elevations on the upstream side of the structure than on the downstream side. Because of this, both upstream and downstream peak water-surface elevations were determined, when possible. Peak water-surface elevations are based on high-water marks (fig. 1 and 2) which were identified using the techniques of Benson and Dalrymple (1967). These marks are referenced to the North American Vertical Datum of 1988 (NAVD 88). All final marks in this report were confirmed either with additional marks or corroborating evidence. The corroborating evidence generally consisted of wash lines on the bank (areas below wash lines have been scoured of leaves and other normal debris), debris in tree branches, and observations by local residents. The corroborating evidence is critical in evaluating the accuracy of a high-water mark but difficult to document. Any other available information was used in the process of evaluating the high-water marks. The locations of all the high-water marks are shown on plate 1. All of the high-water marks are located near 1 of 25 different structures. Each structure is shown on plate 1 as a inset image

Figure 3. Logging road culvert failure over the Josias River, York County, Maine.

Figure 4. Bridge failure over Cape Neddick River, York County, Maine.

and labeled as an area with the corresponding site number. Three of the 22 inset images show 2 individual sites on the same image.

For an accurate evaluation of flood-peak elevation data, an understanding of the methods used to obtain them and their relation to stream profiles is needed. For example, flood elevations can be affected by water pileup on the upstream side of hydraulic structures (for example, bridges, and culverts) and by drawdown on the downstream side of these structures (Fontaine and Nielsen, 1994). To accurately interpolate peak water-surface elevations between the sites listed in table 1, further investigation and a knowledge of the hydraulics of specific streams is needed. The exact time of the peak elevation or the peak flow from the May 2006 York County, Maine, flood is not known for any of the points. Some anecdotal information was documented from observations, but further investigation and additional confirmation would be required to produce the exact dates and times of the peak flows during the Mother's Day flood of 2006.

Calculation of Peak Flow

Peak flow calculations can be made using a variety of methods. The most reliable method is a calibrated and recently confirmed relation between flows and water-surface elevations (commonly called a rating curve). If the observed water-surface elevation is higher than any previously measured for the rating curve, it may be possible to extend the curve. If a rating curve is not available, or is unreliable for extremely high flows, the peak flow can be calculated using indirect methods. Indirect methods of determining peak flow are based on hydraulic relations between flow and the geometry of the channel. A field survey is made after the flood to determine the location and elevation of high-water marks and the characteristics of the channel (Benson, 1967). Flow during the May 2006 flood was calculated for nine streams using a rating extension or indirect methods.

Rating Extension

To compute the peak flow by rating extension for the Mousam River near West Kennebunk and at Branch Brook near Kennebunk, the rating curve was analyzed by applying arithmetic processes, as well as basic concepts of open-channel flow, to available streamflow measurements. The relation of stage to flow is usually controlled by a cross section or a reach of a channel referred to as the control. Section controls exist for both the Mousam River near West Kennebunk and at Branch Brook near Kennebunk. The rating curve can be graphically represented as a straight line when plotted in logarithmic space. The graphing scale is altered by using a gage-height scale offset until the rating can be represented as a straight line. The rating can then be extrapolated beyond its calibrated range as a straight line (Rantz, 1982). This is believed to be a reasonable approach based on the stability and type of the river control for high flows at both locations.

To determine the validity of the assumption that a straight line rating extension is reasonable, the water-surface elevation scale offset was adjusted above and below the final value. The gross adjustments to the offset created changes in the calculated flow ranging from +/- 20 to 30 percent. The adjustments can't be quantified because the graphical nature of the extension, but the interpretation was applied using hydraulic principles and the basic channel shape.

Indirect Method

One particular type of indirect method was used to determine peak flows for all streams for the May 2006 flood—the contracted-opening method. The contraction of a stream channel by a roadway crossing creates an abrupt drop in water-surface elevation between an approach section and the contracted section under the bridge. The contracted section is, in a sense, a flow meter which can be utilized to compute floodflows by use of high-water marks upstream and downstream from the contraction. The geometry of the channel and bridge are defined by field surveys.

The contracted-opening method of indirect measurements requires surveys of four cross sections and additional information on the constricting structure. The approach section is the natural, unconstricted channel upstream from a bridge (or culvert). The upstream bridge face section defines the minimum flow area of the contraction. Generally, this section is located between the bridge abutments on the upstream side of the bridge. The downstream bridge face section defines the downstream constricted conditions. The exit section is located downstream from the constriction and is used to define natural, unconstricted-channel downstream conditions. The bridge structure is defined to identify

Table 1. Peak water-surface elevations for May 2006 flood in York County, Maine.

[Northings and Eastings referenced to Maine State Plane West, North American Datum 1983; Elevations referenced to North American Vertical Datum of 1988; Ave, avenue; BR, bridge; DS, downstream; HWM, high-water mark; I-95, Interstate 95; LE, left edge of water (when facing downstream); Num., number; R., ID, identification; river; Rd., road; RE, right edge of water; RR, railroad; Rt., route; St. street; US, upstream; WS, water-surface elevation at time of survey, not a high-water mark]

Point ID	Site number	Northing	Easting	Elevation	Waterbody	Location, description
1	4	183333.8	2773638.6	249.88	Little River	Diamond Hill Rd., US LE BR Rail
2	4	183325.8	2773624.1	249.70	Little River	Diamond Hill Rd., DS LE BR Rail
3	3	177650.7	2767100.9	231.71	Little River	Long Swamp Rd., US LE
4	3	177709.0	2767040.6	231.63	Little River	Long Swamp Rd., US RE
5	3	177536.4	2766881.0	229.57	Little River	Long Swamp Rd., DS LE
6	3	177605.4	2766858.8	229.57	Little River	Long Swamp Rd., DS RE
7	2	175493.3	2762968.2	194.98	Little River	Ridlon Rd., US LE 1
8	2	175466.9	2762881.6	194.24	Little River	Ridlon Rd., US LE 2
9	2	175556.0	2762847.4	195.00	Little River	Ridlon Rd., US RE
10	2	175416.3	2762887.2	194.22	Little River	Ridlon Rd., DS LE
11	2	175465.2	2762765.2	194.10	Little River	Ridlon Rd., DS RE
12	1	172466.8	2758068.3	175.23	Little River	Hubbard Rd., US RE 1
13	1	172472.9	2758064.8	174.05	Little River	Hubbard Rd., US RE 2
14	1	172637.4	2758065.6	175.00	Little River	Hubbard Rd., US RE 3
15	1	172435.9	2757937.0	175.12	Little River	Hubbard Rd., DS RE 1
16	1	172439.0	2757937.4	175.16	Little River	Hubbard Rd., DS RE 2
17	1	172454.5	2758036.6	175.24	Little River	Hubbard Rd., DS RE 3
18	5	182182.8	2760265.7	206.74	Keay Brook	Ridlon Rd., US LE
19	5	182014.4	2760215.9	203.82	Keay Brook	Ridlon Rd., DS LE
20	5	182225.6	2760177.4	207.79	Keay Brook	Ridlon Rd., US RE
21	6	177044.8	2756695.3	189.31	Keay Brook	Hubbard Rd., US LE
22	6	177133.0	2756700.4	189.44	Keay Brook	Hubbard Rd., US RE
23	6	177004.3	2756460.0	187.97	Keay Brook	Hubbard Rd., DS LE
24	9	213650.3	2800903.8	233.89	Mousam River	Rt. 4, DS RE 1
25	9	213637.9	2800906.7	233.93	Mousam River	Rt. 4, DS RE 2
26	9	213636.4	2800907.0	233.98	Mousam River	Rt. 4, DS RE 3
27	8	213475.2	2822153.2	170.66	Mousam River	Whicher’s Mill Rd., US RE 1
28	8	213481.4	2822123.2	170.78	Mousam River	Whicher’s Mill Rd., US RE 2
29	8	213479.0	2822117.9	170.87	Mousam River	Whicher’s Mill Rd., US RE 3
30	8	213479.0	2822174.0	170.59	Mousam River	Whicher’s Mill Rd., US RE 4
31	8	213476.7	2822203.2	170.83	Mousam River	Whicher’s Mill Rd., US RE 5
32	7	208181.7	2841425.2	87.01	Mousam River	Mill St.(near Rt. 99), DS LE 1
33	7	208171.4	2841453.3	87.16	Mousam River	Mill St.(near Rt. 99), DS LE 2
34	7	208163.7	2841476.2	87.03	Mousam River	Mill St.(near Rt. 99), DS LE 3
35	7	208168.0	2841477.9	87.04	Mousam River	Mill St.(near Rt. 99), DS LE 4
36	24	201296.5	2852892.8	26.62	Mousam River	Rt. 1, DS LE
37	25	182424.5	2845129.7	46.86	Blacksmith Brook	Rt. 1, US LE
38	25	182362.6	2845258.8	42.13	Blacksmith Brook	Rt. 1, DS LE
39	23	199159.7	2842027.0	51.17	BranchBrook	Rt. 9A (old gage), DS LE 1
40	23	199160.9	2842035.5	51.50	BranchBrook	Rt. 9A (old gage), DS LE 2
41	23	199268.7	2841936.2	53.43	BranchBrook	Rt. 9A (old gage), US RE 3
42	23	199288.0	2841932.6	53.33	BranchBrook	Rt. 9A (old gage), US RE 4

Table 1. Peak water-surface elevations for May 2006 flood in York County, Maine.—Continued

[Northings and Eastings referenced to Maine State Plane West, North American Datum 1983; Elevations referenced to North American Vertical Datum of 1988; Ave, avenue; BR, bridge; DS, downstream; HWM, high-water mark; I-95, Interstate 95; LE, left edge of water (when facing downstream); Num., number; R., ID, identification; river; Rd., road; RE, right edge of water; RR, railroad; Rt., route; St. street; US, upstream; WS, water-surface elevation at time of survey, not a high-water mark]

Point ID	Site number	Northing	Easting	Elevation	Waterbody	Location, description
43	11	172981.7	2800865.6	124.88	Great Works River	Canal St. Dam above Rt. 9, US LE 1
44	11	172970.2	2800843.3	124.65	Great Works River	Canal St. Dam above Rt. 9, US LE 2
45	10	171109.8	2800922.4	110.34	Great Works River	Madison St., DS LE
46	12	151299.4	2803352.1	97.16	Great Works River	“Emery’s Bridge” @Hooper’s Rd., US RE
47	12	151294.6	2803321.1	97.22	Great Works River	“Emery’s Bridge” @Hooper’s Rd., DS RE 1
48	12	151303.7	2803304.3	94.23	Great Works River	“Emery’s Bridge” @Hooper’s Rd., DS RE 2
49	12	151236.1	2803405.7	97.09	Great Works River	“Emery’s Bridge” @Hooper’s Rd., US LE 3
50	13	140094.7	2786233.9	83.10	Great Works River	Rt. 236, US LE
51	13	140056.7	2786140.1	83.33	Great Works River	Rt. 236, DS LE
52	14	141059.1	2784972.7	81.66	Great Works River	Brattle St.Dam, US RE
53	15	157611.5	2829183.5	91.42	Ogunquit River	N. Village Rd., DS RE
54	15	157694.3	2829182.4	92.67	Ogunquit River	N. Village Rd., US RE
55	16	158406.5	2832456.1	72.64	Ogunquit River	I-95, DS LE 1
56	16	158409.7	2832453.4	72.69	Ogunquit River	I-95, DS LE 2
57	16	158239.2	2832400.7	72.81	Ogunquit River	I-95, DS RE 1
58	16	158272.3	2832433.6	72.77	Ogunquit River	I-95, DS RE 2
59	16	158287.6	2832447.3	72.65	Ogunquit River	I-95, DS RE 3
60	17	159768.0	2837834.9	43.65	Ogunquit River	Rt.1, US RE
61	17	159806.6	2837991.9	34.91	Ogunquit River	Rt.1, DS LE
62	21	186318.4	2847377.3	24.03	Merriland River	RR bridge US of Rt.1, DS RE 1
63	21	186299.2	2847370.8	24.19	Merriland River	RR bridge US of Rt.1, DS RE 2
64	21	186282.3	2847407.4	24.15	Merriland River	RR bridge US of Rt.1, DS RE 3
65	21	186372.1	2847278.7	25.18	Merriland River	RR bridge US of Rt.1, US RE 1
66	21	186375.3	2847265.3	25.53	Merriland River	RR bridge US of Rt.1, US RE 2
67	21	186366.9	2847292.0	25.17	Merriland River	RR bridge US of Rt.1, US RE 3
68	22	186119.3	2847692.3	22.28	Merriland River	Rt. 1 Trailer park,DS RE 1
69	22	186113.6	2847944.8	22.20	Merriland River	Rt. 1 Trailer park,DS RE 2
70	18	131233.7	2831982.0	20.22	Cape Neddick River	Rt. 1, DS RE 1
71	18	131229.5	2831969.3	20.59	Cape Neddick River	Rt. 1, DS RE 2
72	18	131225.4	2831959.3	20.28	Cape Neddick River	Rt. 1, DS RE 3
73	18	131222.9	2831949.3	20.94	Cape Neddick River	Rt. 1, DS RE 4
74	18	131209.5	2831889.2	26.39	Cape Neddick River	Rt. 1, US RE 1
75	18	131225.4	2831882.5	26.16	Cape Neddick River	Rt. 1, US LE 1
76	18	131220.4	2831874.2	26.35	Cape Neddick River	Rt. 1, US LE 2
77	19	124664.8	2834168.4	11.83	Briley Brook	Bay St.(650’ West of York’s Short Sand’s Beach), HWM 1
78	19	124654.0	2834152.9	11.87	Briley Brook	Bay St.(650’ West of York’s Short Sand’s Beach), HWM 2
79	20	115737.6	2830119.4	12.98	Little River	Bayview Ave. off Long Sand’s Rd.(145’ West of Rt. 1) HWM 1
80	20	115748.2	2830131.8	13.00	Little River	Bayview Ave. off Long Sand’s Rd.(145’ West of Rt. 1) HWM 2

the type of bridge. The embankment slopes for the entrance and exit are measured, and the location, type, and size of any guide banks, wing walls, and abutments are measured. In addition to cross-section surveys and collection of bridge structure information, the downstream river slope is computed, and channel and floodplain roughness values are estimated (Matthai, 1967).

The geometric survey data were entered in the U.S. Army Corp of Engineers step-backwater computer program HEC-RAS. The one-dimensional steady flow water-surface profile computation component of HEC-RAS was used in the analysis. A range of flows, 2-year to 500-year recurrence interval flows, and a flow 50 percent greater than the 500-year flow were entered into the model. Each flow generated an independent hydraulic profile. The profiles provided guidance to calibrate the model. After calibration, flows were selected and input into the model so that the resulting water-surface-elevation output from HEC-RAS matched the high-water marks that were observed for this study (table 1) at the downstream exit cross sections and the upstream approach cross sections (U.S. Army Corps of Engineers, 2004).

A sensitivity analysis was performed on the final calibrated models. The boundary condition of slope was set, and energy-loss characteristics (including channel roughness, and contraction and expansion coefficients) were modified to determine the effects of these changes on the resulting water-surface elevations and the final computed flows (Chow, 1959). In addition to the computed 2006 floodflow, the hydraulic profiles (with recurrence intervals from 2 years to greater than 500 years) were reviewed to determine the stability of the model.

Peak Flows

Peak flows were calculated for nine different locations in York County for the May 2006 flood (table 2). Seven of the flows were calculated using an indirect method and two flows were calculated using a rating extension technique. The location of the sites with peak-flow determinations and their associated recurrence intervals are shown in plate 1. Recurrence intervals are discussed in the Peak-Flow Recurrence Intervals section of this report.

Table 2. Peak flows calculated for the May 2006 flood and peak flows for 10-, 50-, 100-, and 500-year flood recurrence intervals in York County, Maine.

[>, greater than; mi^2, square mile; ft^3/s, cubic feet per second]

Site Number	Stream and location	Peak flow (ft^3/s) for given recurrence interval[a]				Peak flow calculated during May 2006 flood[b]		Drainage area (mi^2)
		10-year	50-year	100-year	500-year	Peak flow (ft^3/s)	Recurrence interval	
3	Little River at Long Swamp Road, Berwick	1,630	2,310	2,630	3,380	3,390	> 500-year	51.3
6	Keay Brook at Hubbard Road, Berwick	457	673	775	1,020	1,120	> 500-year	11.3
8	Mousam River near West Kennebunk (USGS streamflow gaging station number 01069500)	2,470[c]	3,600[c]	4,100[c]	5,470[c]	6,100	> 500-year	98.9
12	Great Works River at Emery's Bridge on Hooper's Sand Road, South Berwick	2,130	3,020	3,430	4,420	6,460	> 500-year	64
17	Ogunquit River at North Village Road, Ogunquit	644	964	1,120	1,490	3,110	> 500-year	12.1
18	Cape Neddick River at State Route 1, York	439	653	755	1,000	2,250	> 500-year	9.47
21	Merriland River at RR bridge above State Route 1, Kennebunk	385	547	623	800	2,240	> 500-year	15.4
23	Branch Brook near Kennebunk (USGS streamflow gaging station number 01069700)	600[c]	978[c]	1,160[c]	1,640[c]	1,500	100- to 500-year	10.3
25	Blacksmith Brook at State Route 1, Wells	116	178	208	280	473	> 500-year	2.2

[a]Computed using regression equations estimates published in Hodgkins (1999).

[b]Computed using indirect methods.

[c]Flows computed using Log Pearson type III estimates published in Hodgkins (1999).

Peak-Flow Recurrence Intervals

The recurrence interval is the average period of time between peak flows that are greater than or equal to a specified magnitude. For example, the 50-year peak flow is the flow that would be exceeded or equaled, on long-term average, once in 50 years. This does not imply that flooding will happen at regular intervals. Two 50-year peak flows could occur in 2 consecutive years or even the same year. In contrast, a 50-year peak flow might not occur for 100 years. The reciprocal of the recurrence interval is called the annual exceedance probability; that is, the probability that a given peak flow will be exceeded or equaled in any given year. For example, the annual exceedance probability of the 50-year peak flow is 0.02. In other words, there is a 2 percent chance that the 50-year peak flow will be exceeded or equaled in any given year.

The 500-, 100-, 50-, and 10-year recurrence-interval peak flows for sites in table 2 were determined as explained below. The peak flows from the May 2006 flood were then compared to these recurrence-interval peak flows to determine the recurrence intervals of the May 2006 peak flows. All recurrence interval estimates have an uncertainty associated with them. The uncertainty generally increases as the recurrence interval increases. Recurrence intervals that are greater than 100 years have a large amount of uncertainty associated with them (Interagency Advisory Committee on Water Data, 1982).

The recurrence-interval peak flows for seven sites in this study without historical peak-flow data were calculated using the regression equations presented in Hodgkins (1999). These regression equations were developed using generalized least squares regression procedures based on data from 70 USGS streamflow gaging stations in Maine and eastern New Hampshire. The final explanatory variables used in the equations were drainage area and percentage of basin wetlands. The equations were generated using drainage areas ranging from 0.9 to 1,650 mi^2 with basin wetlands ranging from 0.7 percent to 26.7 percent of the drainage area. The recurrence-interval peak flows for Branch Brook near Kennebunk and Mousam River near West Kennebunk, which have historical peak-flow data, were published previously by Hodgkins (1999). Additional peak-flow data have not been collected at these sites since the Hodgkins (1999) report was published, and thus the recurrence-interval peak flows recorded in Hodgkins (1999) were used for the analysis in this study. Peak-flow recurrence intervals for eight of the nine streams studied were calculated to be greater than 500-years. The peak-flow recurrence interval of the remaining stream was calculated to be between a 100-year and 500-year interval.

Historical Perspective on Flood

Accurate flood-damage information is difficult to obtain. In most cases, the total flood damage is never known. As a result of calculated damages to specific types of property, a presidential disaster declaration was made on May 25, 2006, for York County, Maine. Public flooding damages for York County are estimated to be $7.5 million. (Maine Emergency Management Agency, written commun., 1996). In retrospect, flood-damage data can be used as a measure of the effectiveness of attempts to mitigate flood hazards and to place an individual flood into historical perspective. Documented data on peak flows and high water marks from the May 2006 event can be used in the future for assessing the flood from a historical perspective.

Historical Peak Flows

Minimal historical peak-flow data are available for much of southern Maine, especially York County. The USGS operated a continuous-record streamflow gaging station at the Mousam River at Wicher's Mill Road near West Kennebunk, Maine, from 1939 to 1984. The highest peak flow recorded during that time was 4,030 ft^3/s in March 1983. A peak flow of 3,600 ft^3/s during the flood of October 1996 was documented outside of the period of continuous record (Hodgkins and Stewart, 1997). On the basis of 45 years of continuous record, assuming all large floods from 1985 to 2005 were documented, the May 2006 flow of 6,100 ft^3/s is the highest documented flow at this site from 1939 to 2006. The USGS operated a streamflow gaging station on Branch Brook near Kennebunk, Maine, from 1964 to 1974 that was designed to record all annual peak flows. The highest peak recorded during that time was 723 ft^3/s in 1972. From 1975 to 2005, the highest documented peak flow was 1,020 ft^3/s during the flood of October 1996. On the basis of 11 years of continuous peak-flow record, assuming all large floods from 1975 to 2005 have been documented, the May 2006 flow of 1,500 ft^3/s is the highest documented flow at this site from 1964 to 2006.

For the other seven locations where flows were determined by indirect methods, historical streamflow records are not available. A general comparison to theoretical maximum floods is possible however. Crippen and Bue (1977) determined envelope curves relating empirical maximum peak flows to drainage area to provide a guide for estimating the magnitude of maximum floodflows that can be expected at a given site on a stream; the curves are plotted in log space with the drainage area plotted on the x-axis and the flow on the y-axis. The curves were developed by analyzing thousands of sites with recorded floodflows, then using the sites with the most extreme flows to draw the envelope curves for 17 different regions in the United States. Maine is in region 1 (region 1 comprises Maine, Vermont, most of New Hampshire, most of Connecticut, western Massachusetts, and a small part of Rhode Island).

In general, all of the peak flows computed for this study plotted a similar distance below the region 1 envelope curve. The peak flow that plotted closest to the envelope curve occurred at the Ogunquit River. The peak flow that plotted furthest from the envelope curve occurred at Keay Brook. None of the floodflows for the Mother's Day flood reached or exceeded the potential maximum floodflow as defined by the Crippen and Bue (1977) envelope curves. The plotting positions provide evidence that the peak flow calculations for this study are reasonable and that all nine sites experienced similar amounts of runoff per unit drainage area (ft^3/mi^2).

Historical Peak Water-Surface Elevations

Minimal historical peak water-surface-elevation data are available for much of York County, Maine. Peak elevations can be used for comparison with historical elevations at a specific location if the river channel or hydraulic structure has not changed substantially between floods. Data are available for comparison of elevations during the flood of 1996 (Hodgkins and Stewart, 1997) with those during the flood of 2006; for the site discussed below it is believed that the structure and river channel may have changed between these floods. Blacksmith Brook at Rt. 1, Wells, Maine recorded an upstream peak water-surface elevation (NAVD88) of 46.72 ft and a downstream peak water-surface elevation of 41.42 ft for the 1996 flood. The peak water-surface elevation recorded during the May 2006 event, upstream from the site, was 46.86 ft (NAVD88), and the downstream peak water-surface elevation was 42.13 ft. The upstream peak water-surface elevations were very similar at a difference of 0.14 ft, but the downstream elevations varied by 0.71 ft. This difference may indicate that the hydraulic relation has changed sometime during the period from October 1996 to May 2006. Both sites discussed below are believed to have stable river channels and hydraulic structures between the floods. At Branch Brook near Kennebunk, Maine, the upstream peak water-surface elevation (NAVD88) was 49.92 ft, and the downstream peak water-surface elevation was 49.32 ft for the 1996 flood; during the May 2006 flood, the upstream peak water-surface elevation (NAVD88) was 53.33 ft, and the downstream peak water-surface elevation was 51.50 ft. At Mousam River near West Kennebunk, Maine, the peak water-surface elevation (NAVD88) was 170.41 ft for the 1996 flood, which is lower than the 2006 flood elevation of 170.66 ft (Hodgkins and Stewart, 1997).

Comparison with Existing Flood Insurance Studies

Comparisons were made between existing Flood Insurance Study data and the May 2006 flood data. In York County, detailed Flood Insurance Studies are available for the communities of Berwick, North Berwick, Kennebunk, Kennebunkport, Ogunquit, Sanford, South Berwick, Wells, and York. The original publication dates of the studies range from 1982 to 1984; parts of many of the studies were revised during 1998 to 2003. In a detailed Flood Insurance Study, the peak water-surface elevations for selected recurrence intervals (particularly the 100-year elevations) are calculated using hydrologic and hydraulic analyses. The hydrologic analysis involves defining the flow or range of peak flows that can be expected for selected recurrence intervals. The hydraulic analysis involves computing flood elevations on the basis of physical features of a stream, such as channel geometry, bed material, and structure geometry.

Two types of comparisons were made between the Flood Insurance Studies and the May 2006 flood. At sites where the peak flow was determined for the May 2006 flood, independent analyses of hydrology and hydraulics were completed. For the May 2006 sites with a calculated peak flow, this flow was compared to the flow published in the existing Flood Insurance Study. At sites for which only peak surface-water elevations are available for the May 2006 event, only elevation comparisons were made. It was not feasible to determine the accuracy of the hydraulics or the hydrology independent of each other in the existing Flood Insurance Studies.

In a detailed Flood Insurance Study, peak flows for selected recurrence intervals (the hydrology) are input into a step-backwater hydraulic model containing the geometric data for the stream reach and other data. The model generates peak water-surface elevations at selected cross sections in the reach. The resulting elevations from many cross sections in the model, including cross sections at and near bridges, are plotted and labeled as flood profiles in the Flood Insurance Study. The 100-year and 500-year flood elevations were selected from the flood profile graph for comparison with the elevations for the May 2006 flood. For example, if the May 2006 flood elevation was measured for the point 100 feet upstream from a given structure, the elevation selected from the flood-profile plot would be 100 feet upstream from that structure.

The elevation comparison sites are on the same stream as the flow comparison sites, but at different locations. At sites where the May 2006 peak flow was not calculated, only the elevations were available for analysis. The May 2006 peak flow could be determined at many of the elevation comparison sites by a simple drainage-area correction. The peak flow also could be calculated directly from regression equations. The drainage-area adjustment calculations would yield the same results as the regression equations because the same calculations would be done on the May 2006 flow and the calculated flows for each recurrence interval. The calculation of hydrology peak flows for selected recurrence intervals for the 2006 flood using the regression equations was not done because biases from a drainage-area adjustment for the May 2006 flow would produce similar peak flows and complicate the analysis of the floodflows.

Hydrologic Analysis

The peak flows published in the Flood Insurance Study for Sanford and South Berwick (Federal Emergency Management Agency, 1998 and 1984, respectively) were computed using the Natural Resources Conservation Services computer program TR-20 (U.S. Department of Agriculture, 1990). The 24-hour duration storm and normal antecedent moisture conditions were used in both studies to compute the range in flows for the 10-, 50-, 100- and 500-year recurrence intervals. The remaining published peak flows for Kennebunk, Ogunquit, York, and Wells (Federal Emergency Management Agency, 1982, 1983, 2003, and 2003, respectively) were computed using the equations published in Morrill (1975). These equations were used to predict flows based on drainage area, main-channel slope, and percentage of area of lakes and ponds. The equations used by Morrill have been superseded by equations in a more recent publication (Hodgkins, 1999). The floodflows calculated for this report used the updated regression equations in Hodgkins (1999), which were based on substantially more peak-flow data from Maine streams than used by Morrill and were generated using more advanced statistical techniques.

Typically, the 10-, 50-, 100-, and 500-year floodflows are published in detailed Flood Insurance Studies. FEMA regulates flood insurance policies to the 100-year floodflow and corresponding flood elevation, but the additional recurrence interval data are useful for other regulatory purposes. Only the 100-year and 500-year recurrence intervals were used in this analysis. Comparison to 10-year and 50-year

Table 3. Peak flows calculated for the May 2006 flood in York County, Maine, and peak flows from published Flood Insurance Studies for the 100- and 500-year recurrence interval events.

[NS, location not identified in study; ft^3/s, cubic feet per second; mi^2, square mile]

Site number	Stream and location	Peak flow (ft^3/s) for given recurrence interval				Community	Drainage area (mi^2)
		Calculated[a]		Published flood insurance study			
		100-year	500-year	100-year	500-year		
8	Mousam River near West Kennebunk (USGS streamflow gaging station number 01069500)	4,100	5,470	4,000	[b]	Sanford	99.0[c]
NS	Mousam River at confluence with the Atlantic Ocean, Kennebunk	- -	- -	3,508	4,505	Kennebunk	119
NS	Great Works River at Old South Berwick Road	- -	- -	3,640	5,180	South Berwick	46.1
12	Great Works River at Emery's Bridge on Hooper's Sand Road, South Berwick	3,430	4,420	- -	- -	South Berwick	64.0
NS	Great Works River at Emery's Bridge Road	- -	- -	7,640	10,500	South Berwick	82.1
17	Ogunquit River at North Village Road, Ogunquit	1,120	1,490	1,602	2,422	Ogunquit[d]	12.1[e]
18	Cape Neddick River at US Route 1, York	755	1,000	950	1,420	York	9.47[f]
21	Merriland River at RR bridge above State Route 1, Kennebunk	623	800	- -	- -	Kennebunk	15.4
NS	Merriland River at Lord's Road, Kennebunk	- -	- -	1,846	2,757	Kennebunk	16.8
25	Blacksmith Brook at US Route 1, Wells	208	280	385	607	Wells	2.20[g]

[a]Flow computed using Log Pearson type III estimates published in Hodgkins (1999).

[b]500-year flow was not published in the Sanford Flood Insurance Study.

[c]Drainage area from Sanford Flood Insurance Study is 102.0 mi^2.

[d]The flows in the Wells and Ogunquit Flood Insurance Studies did not match. Ogunquit flows were used.

[e]Drainage area from Ogunquit Flood Insurance Study is 11.94 mi^2.

[f]Drainage area from Cape Neddick Flood Insurance Study is 9.53 mi^2.

[g]Drainage area from Blacksmith Flood Insurance Study is 2.25 mi^2.

recurrence intervals were not made to limit repetition. The peak flows computed for this study and those in published Flood Insurance Studies are presented in table 3 for six sites.

On the Mousam River a direct comparison could be made only for a 100-year event using the Sanford Flood Insurance Study (Federal Emergency Management Agency, 1998). The 500-year flow was not published in the effective (existing) Flood Insurance Study for this community. The 100-year floodflow calculated for this study was almost identical to the flows published in the Flood Insurance Study. For the site at the mouth of the Mousam River (drainage area of 119 mi^2), the 100-year and 500-year floodflows were published in the Kennebunk Flood Insurance Study (Federal Emergency Management Agency, 1982); the flows in the Flood Insurance Study are lower than those calculated for 100-year and 500-year flows in this study. The flows published in the Flood Insurance Study were calculated using regression equations from Morrill (1975). An alternate method of calculating the 100-year and 500-year floodflows would entail the use of the historical peak-streamflow record for the Mousam River near West Kennebunk (drainage area of 99 mi^2) along with a drainage-area adjustment. This calculation was done using Hodgkins equations (1999), which produced a flows similar to the peaks generated by analyzing the historical peak-streamflow record.

A direct comparison of peak streamflows could not be made for the Great Works River at Emery's Bridge (site 12). Flows published in the Flood Insurance Study were calculated for points upstream and downstream from the site used for this study on the Great Works River (table 3). For comparison, a drainage-area adjustment (Hodgkins, 1999) was made to the calculated 100-year and 500-year floodflows at site 12. The adjusted flows at site 12 were 38 percent (100-year) and 43 percent (500-year) lower than the flows upstream from this site that were published in the South Berwick Flood Insurance Study (Federal Emergency Management Agency, 1984), which were calculated using the TR-20 method. A basic drainage-area adjustment equation is

$$Q_A = Q_B\,(A_A / A_B)^c,$$

where Q_A and Q_B equal flow at site A and B, respectively, and A_A and A_B equal drainage area at site A and B, respectively. The drainage-area adjustment values (c) for the Great Works River (site 12) 100-year and 500-year floodflows were calculated as 1.28 and 1.22, respectively. In Hodgkins (1999), the recommended values for (c) are 0.748 for the 100-year and 0.729 for the 500-year floodflows.

For the Ogunquit River, Cape Neddick River, Merriland River, and Blacksmith Brook (sites 17, 18, 21, and 25, respectively; table 3), a direct comparison was made between the peak flows calculated for this study using Hodgkins (1999) and the respective peak flows published in the Flood Insurance Studies. The Merriland River peak flows were adjusted using a drainage area adjustment to make the direct comparison. At all four locations, the peak flows calculated for this study were lower than the Flood Insurance Study peak flows; the 100-year floodflows were 21 percent to 66 percent lower and the 500-year floodflows were 30 percent to 71 percent lower.

Hydraulic Analysis

For site-specific hydraulic analyses, the peak water-surface elevations from the May 2006 flood were compared directly with the elevations published in the corresponding Flood Insurance Studies. Sixteen of the 25 sites had detailed data that could be used for analysis.

There are two major problems with selecting the elevation from Flood Insurance Study flood-profile plots. First, the elevations used on all of the flood profiles are referenced to National Geodetic Vertical Datum of 1929 (NGVD29) and the May 2006 peak elevations are referenced to the North American Vertical Datum 1988 (NAVD88). This problem was resolved by using the latitude and longitude to determine the correction and applying the correction to the NGVD29 elevations. The correction can be done using an on-line utility VERTCON (National Geodetic Survey, 1999). The second problem is the error in determining an elevation from a profile plot. Generally, changes to the water-surface profile around structures can be large. Choosing water-surface elevations at specified distances upstream or downstream from a bridge is difficult, owing to the plot scale and cross-section locations. Water-surface profiles are linear interpolations between known values at each cross section. These problems potentially introduce error into the overall comparison. The error varies depending on the rate of the change in elevation near the structure. The error can not be corrected, but knowledge of the magnitude of the possible error was taken into account when reviewing the results from this comparison.

The peak elevations selected from appropriate Flood Insurance Studies for the 100-year and 500-year flood profiles and the peak elevations for the 2006 flood are listed in table 4. The 2006 flood-peak water-surface elevations can be from a single high-water mark, or an average of two, three, or four different high-water marks for the upstream and (or) downstream side of each structure (upstream marks are averaged together and downstream marks are averaged together). The average distance upstream or downstream from a given structure was used to determine the distance used for the selected elevation from the Flood Insurance Study water-surface profiles. The differences between the Flood Insurance Study flood profile 500-year elevations and the 2006 elevations are shown in table 4. Positive differences indicate the May 2006 elevations are higher than the 500-year elevations published in the Flood Insurance Studies. Because the peak flows for all but one of the sites with calculated flows from the May 2006 flood are associated with greater than 500-year recurrence intervals, the May 2006 peak water-surface elevations were expected to be greater than the 500-year Flood Insurance Study peak water-surface elevations. When the elevation differences are not consistent with expected results, there could be a problem with the hydraulic model. Inaccurate values for energy losses through the structure would result in inconsistent elevation differences. These potential errors in the hydraulic model could be corrected by collecting additional information, entering the information into the model, and calibrating the model to the May 2006 flood elevations.

Data for the 2006 flood are available for five points at three sites on the Mousam River. At two of the points, the 500-year Flood Insurance Study elevations are not available for comparison. At one of the remaining three points, the differences were 0.4 ft which indicated agreement between elevations from the 2006 flood event and the 500-year peak water-surface elevation published in the Flood Insurance Study. At the remaining two points, differences were negative, -3.8 ft and -2.3 ft (the May 2006 peak water-surface elevations were lower than the 500-year peak water-surface elevation published in the Flood Insurance Study). For the Mousam River, the 500-year peak flows from the current study are similar to those published in the Flood Insurance Study. Because the 500-year flows are similar, the negative elevation difference likely was due to excessive energy losses or errors in the geometric data in the published Flood Insurance Study hydraulic model.

For Blacksmith Brook, only two points at one site are available for analysis. The upstream difference between the 500-year peak water-surface elevation published in the Flood Insurance Study and the May 2006 peak water-surface elevation is 3.4 ft, and the downstream difference is -0.6 ft. This indicates that insufficient energy losses may have been used in the published Flood Insurance Study hydraulic model.

Seven points at five sites on the Great Works River in two communities are available for comparison. For the two most-upstream points in North Berwick (sites 10 and 11), elevation differences are 0.5 ft and -0.9 ft. Both elevation differences are reasonable, given the uncertainty of hydraulic models and hydrologic methods. The remaining five points are all in South Berwick (sites 12, 13, and 14), and differences range from -1.0 ft to -6.6 ft. The water-surface elevation drops through the hydraulic structures and the consistency between marks appear reasonable, indicating the published hydraulic models correctly simulate the observed differences in water-surface elevation from the May 2006 flood. The South Berwick Flood Insurance Study used substantially higher 500-year recurrence interval flows than the 2006 floodflows calculated using Hodgkins (1999). The higher Flood Insurance Study flows likely are the cause of the large negative differences in water-surface elevation.

Data from five points at three sites on the Ogunquit River are available for analysis. The differences between the May 2006 elevations and the elevations in the Flood Insurance Study range from 3.3 ft to 6.4 ft (table 4). For the U.S. Route 1 bridge crossing (site 17), the upstream difference is 6.4 ft and the downstream difference is 3.3 ft. The hydrology comparison indicated that the 500-year peak flows in the Wells Flood Insurance Study (Federal Emergency Management Agency, 2003) are higher than the 500-year peak flows calculated for this study, but the Flood Insurance Study 500-year peak flow is substantially smaller than the actual peak flow for the May 2006 flood. When comparing the differences between the 100-year and 500-year profiles in the Wells Flood Insurance Study, a consistent water-surface elevation drop was observed between the upstream and downstream side of the bridge. The elevation difference at the Route 1 crossing (site 17) may indicate a problem with the losses through the structure or the 2006 floodflow was substantially larger than the modeled flows. The water-surface elevations on the flood profiles increase on the upstream side faster than the downstream side compared to the May 2006 water-surface elevations. This discrepancy indicates it is likely that insufficient energy loses were simulated in the hydraulic model in the Flood Insurance Study.

Data are available from three locations at two different sites on the Merriland River. The hydrology comparison for the Merriland River showed the largest difference between the Flood Insurance Study 500-year flow of 2,757 ft^3/s at one point (table 3) and the 2006 500-year flow of 800 ft^3/s at another point calculated using Hodgkins (1999). The drainage areas are similar, and the 500-year floodflows should be similar. The difference in hydrology is important; if the flow in the Wells Flood Insurance Study is too great, the resulting 100-year flood elevation and mapped floodplain would be based on a flow that is too high. The elevation differences range from -0.4 ft to 0.5 ft upstream and downstream from Route 1, indicating that reasonable results were obtained from the hydraulic analysis in the Flood Insurance Study.

At one site on the Cape Neddick River at a bridge, both upstream and downstream elevations are available for comparison. The upstream difference is 6.2 ft, and the downstream difference is 0.5 ft. The hydrology comparison indicates that the York Flood Insurance Study 500-year peak flow is higher than the May 2006 500-year peak flow calculated using Hodgkins (1999), but the calculated peak flow for the May 2006 flood (table 2) is substantially larger than the 500-year peak flow in the Flood Insurance Study. This indicates it is likely insufficient energy loses were simulated in the hydraulic model in the Flood Insurance Study.

Summary and Conclusions

The flood of May 10–17, 2006 (sometimes referred to as the “Mother’s Day flood”), caused widespread damage to numerous roads, structures, and property in York County, Maine. During 2006-07, the U.S. Geological Survey, in cooperation with the Federal Emergency Management Agency, collected and analyzed an extensive amount of flood data from York County to document the flood. Eighty peak water-surface elevations were marked and surveyed following the May 2006 flood. Peak flows from the flood and their associated recurrence intervals were calculated for nine streams. Eight of these calculated flows had recurrence intervals greater

Table 4. Peak water-surface elevations during the May 2006 flood and the 100-year and 500-year elevations from published Flood Insurance Studies for York County, Maine.

[Elevations, in feet, referenced to North American Vertical Datum of 1988 (NAVD88); Ave, avenue; BR, bridge; DS, downstream; ID, Identification number; I-95, Interstate 95; LE, left edge of water (when facing downstream); Num., number; Rd., road; RE, right edge of water; RR, railroad; Rt., route; St. street; US, upstream]

Site number	Peak flood elevations		2006 Peak stream elevation (ft)	Difference between 500-year and 2006 elevation (ft)	Stream	Location, description
	100-year (ft)	500-year (ft)				
25	42.5	43.5	46.9	3.4	Blacksmith Brook	Rt. 1, US LE
25	41.8	42.8	42.1	-0.6	Blacksmith Brook	Rt. 1, DS LE
18	19.0	20.1	26.3	6.2	Cape Neddick River	Rt. 1, US RE
18	18.9	20.0	20.5	0.5	Cape Neddick River	Rt. 1, DS RE
11	122.3	124.3	124.8	0.5	Great Works River	Canal St. Dam above Rt. 9, US LE
10	108.3	111.3	110.3	-0.9	Great Works River	Madison St., DS LE
12	96.7	98.4	97.1	-1.3	Great Works River	“Emery’s Bridge” at Hooper’s Rd., US RE
12	96.1	98.2	97.2	-1.0	Great Works River	“Emery’s Bridge” at Hooper’s Rd., DS RE
13	86.5	89.7	83.1	-6.6	Great Works River	Rt. 236, US LE
13	85.8	87.6	83.3	-4.2	Great Works River	Rt. 236, DS LE
14	84.3	86.1	81.7	-4.4	Great Works River	Brattle St.Dam, US RE
21	23.0	25.7	25.3	-0.4	Merriland River	RR bridge US of Rt.1, US RE
21	22.5	24.7	24.1	-0.5	Merriland River	RR bridge US of Rt.1, DS RE
22	20.1	22.1	22.2	0.2	Merriland River	Rt. 1 Trailer park,DS RE
9	236.3	237.7	233.9	-3.8	Mousam River	Rt. 4, DS RE
8	167.6	- -	170.8	- -	Mousam River	Wicher’s Mill Rd., US RE
8	164.3	- -	170.7	- -	Mousam River	Wicher’s Mill Rd., DS RE
7	88.5	89.4	87.1	-2.3	Mousam River	Mill St.(near Rt. 99), DS LE
24	25.3	26.3	26.6	0.4	Mousam River	Rt. 1, DS LE
15	85.5	88.3	92.7	4.4	Ogunquit River	N. Village Rd., US RE
15	85.3	87.9	91.4	3.6	Ogunquit River	N. Village Rd., DS RE
16	68.1	69.5	72.7	3.3	Ogunquit River	I-95, DS LE
17	34.0	37.3	43.6	6.4	Ogunquit River	Rt.1, US RE
17	29.9	31.6	34.9	3.3	Ogunquit River	Rt.1, DS LE

than 500 years, whereas one had a recurrence interval of 100 to 500 years. Peak-flow recurrence intervals greater than 100 years are associated with a large amount of uncertainty due to the limited number of years of record.

Floodflows can result from the interrelation of rainfall, rainfall intensity, antecedent drainage-basin conditions, physical drainage-basin characteristics, and other factors. The flood of May 2006 was a direct result of extreme rainfall; 10 to greater than 15 inches of rain were recorded in York County during May 10–17, 2006. The average rainfall received in York County during this storm exceeded the 100-year recurrence interval for 2-day, 4-day, 7-day, and 10-day rainfall totals at many locations throughout southern York County (especially Cape Neddick, Eliot, and Sanford).

The hydrologic (computation of peak flows for selected recurrence intervals) and hydraulic (computation of water-surface elevations based on selected peak flows using hydraulic models) results from published Flood Insurance Studies were compared with values calculated for this study. At sites on several streams, differences were observed between peak flows published in Flood Insurance Studies and those calculated for this study. Multiple methods were used for the Flood Insurance Study hydrology calculations. Revised regression equations for estimating peak flows of selected recurrence intervals have been generated using additional data collected since the publication of many of the Flood Insurance Studies. Use of these revised equations in this study may account for many of the differences in values between this study and the Flood Insurance Studies. The differences in the peak flows between the published Flood Insurance Studies and the flows calculated for this report are within an acceptable range for flows calculated at ungaged locations, with the exception of those for the Great Works River and Merriland River. For sites on the Mousam River, Blacksmith Brook, Ogunquit River, and Cape Neddick River, water-surface elevations from Flood Insurance Studies differed with documented water-surface elevations from the 2006 flood. A suggestion for future work is to revise selected Flood Insurance Studies for York County to incorporate additional data collected since their publication.

Acknowledgments

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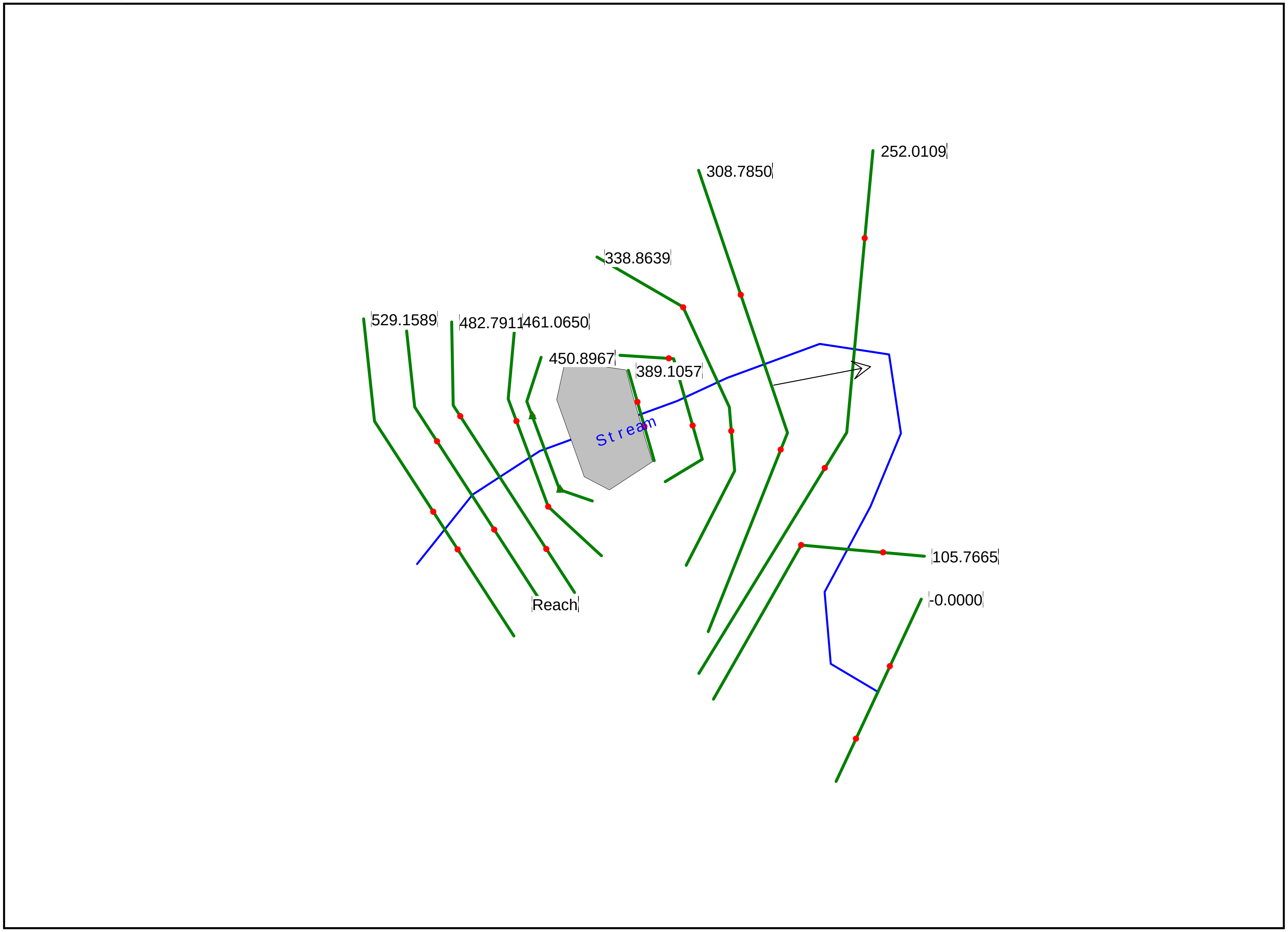
USGS

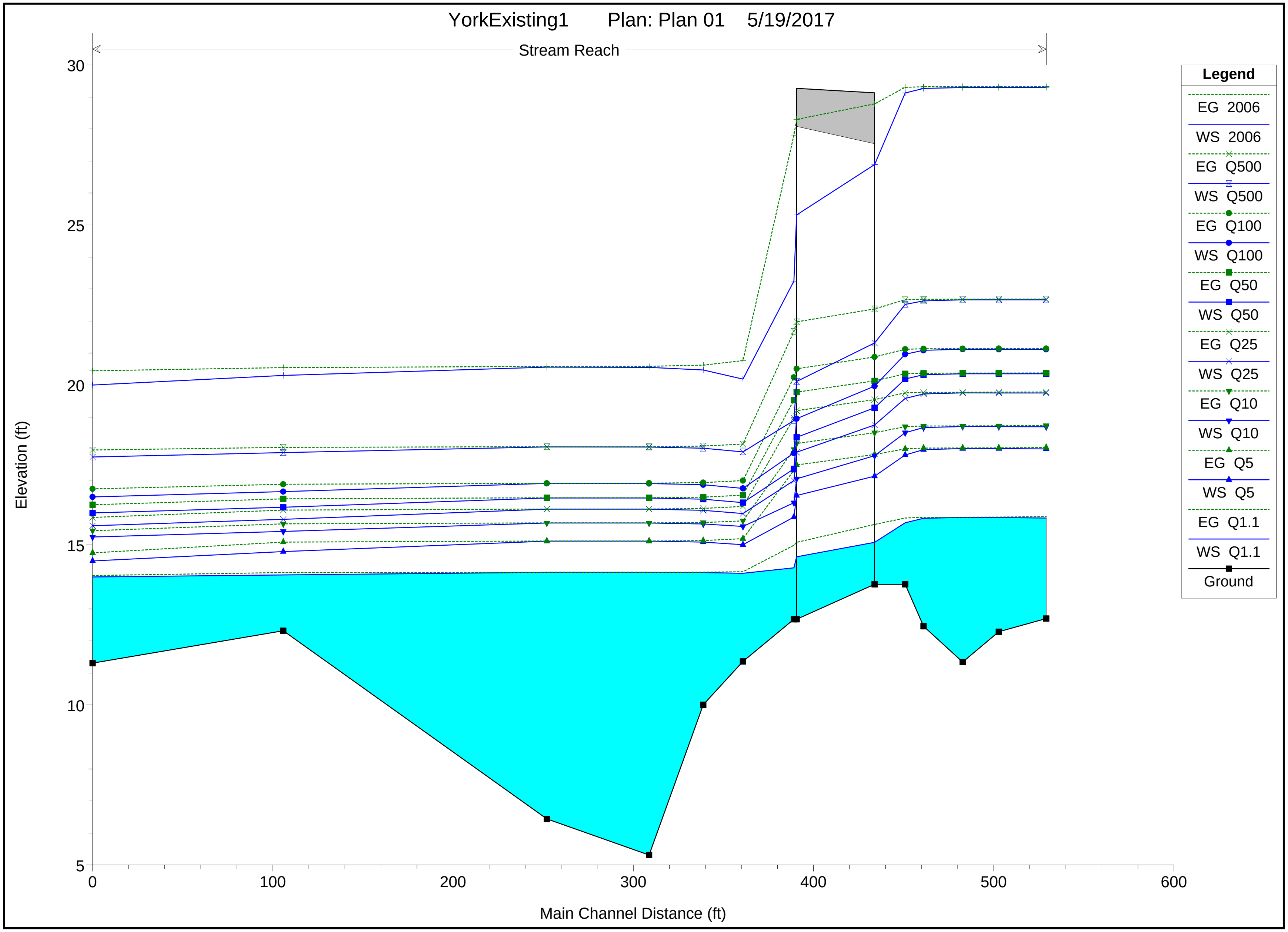
Stewart and others—**Flood of May 2006 in York County, Maine**—Scientific Investigations Report 2008–5047

APPENDIX D

Existing HEC-RAS Analysis

Existing Cross-section Layout





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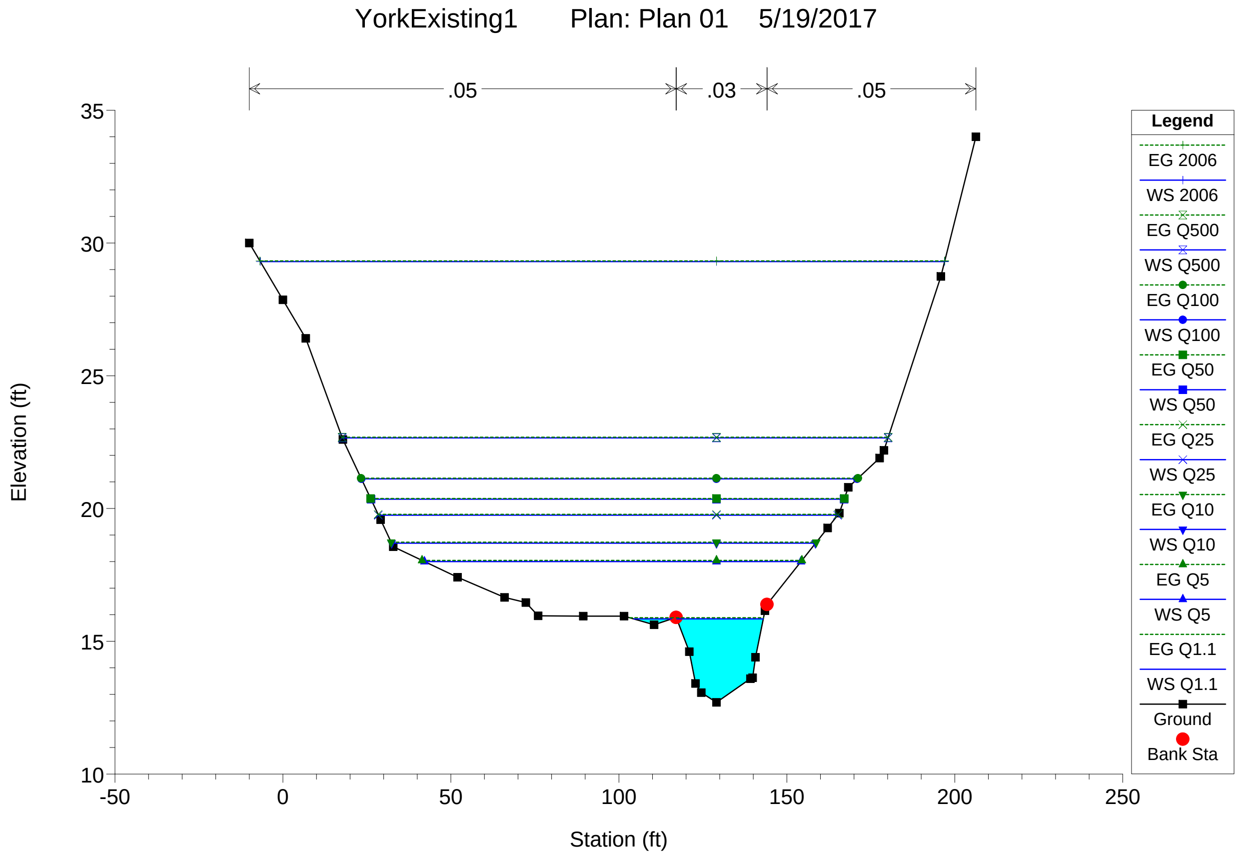
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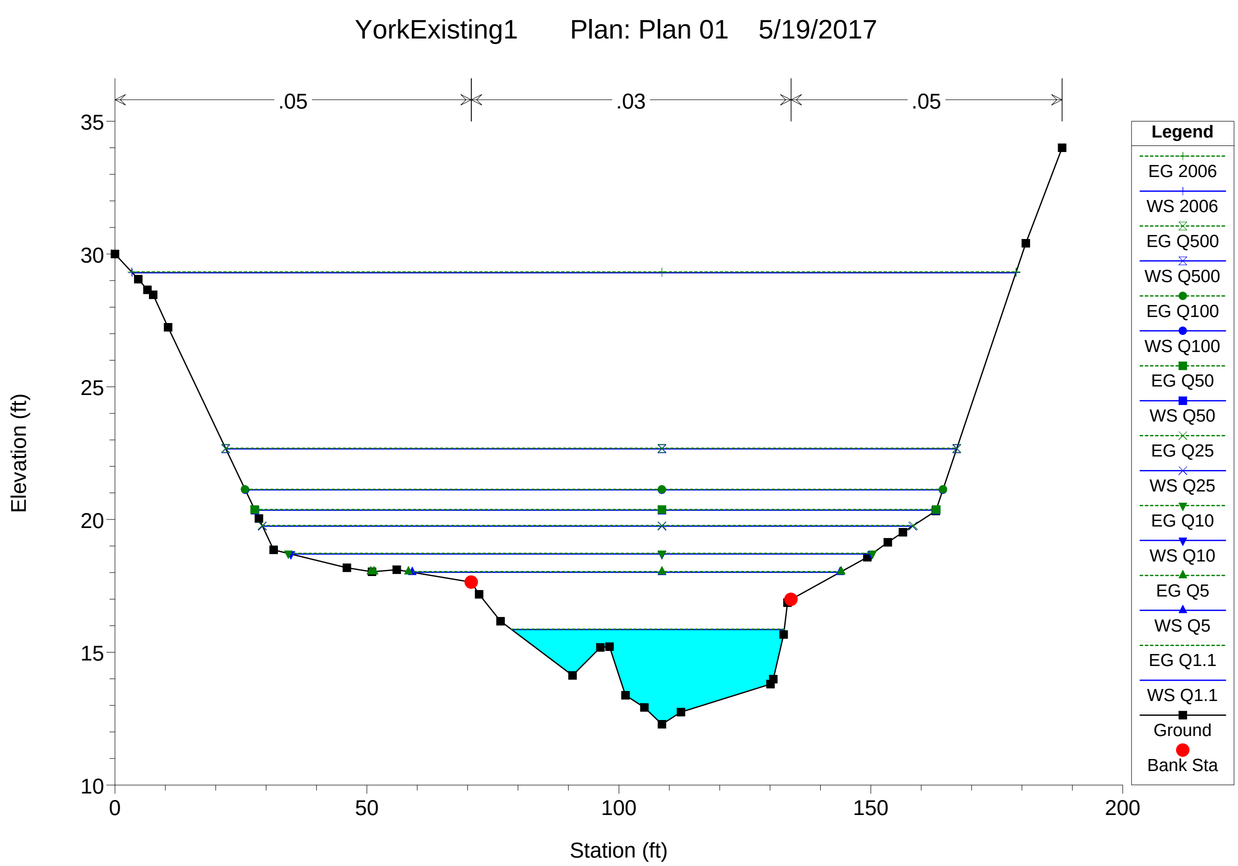
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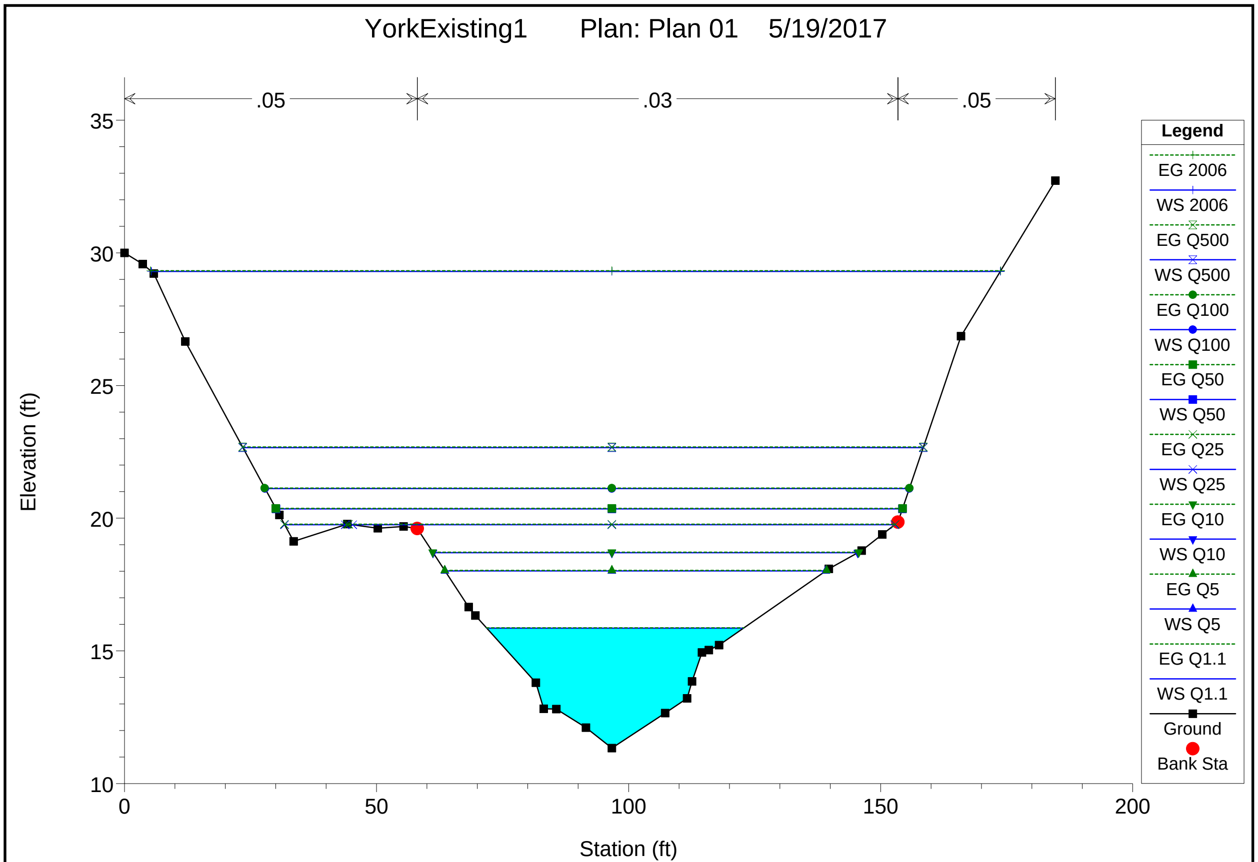
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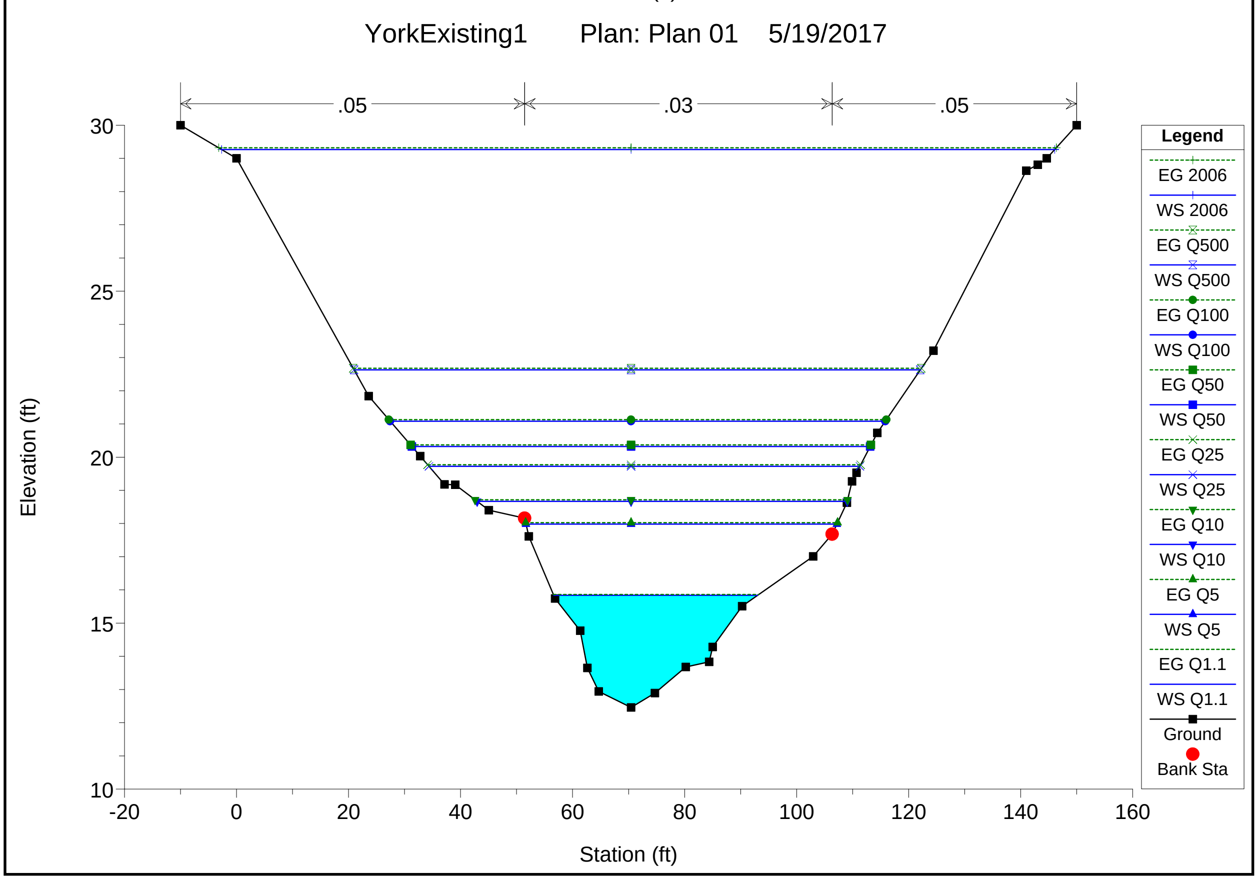
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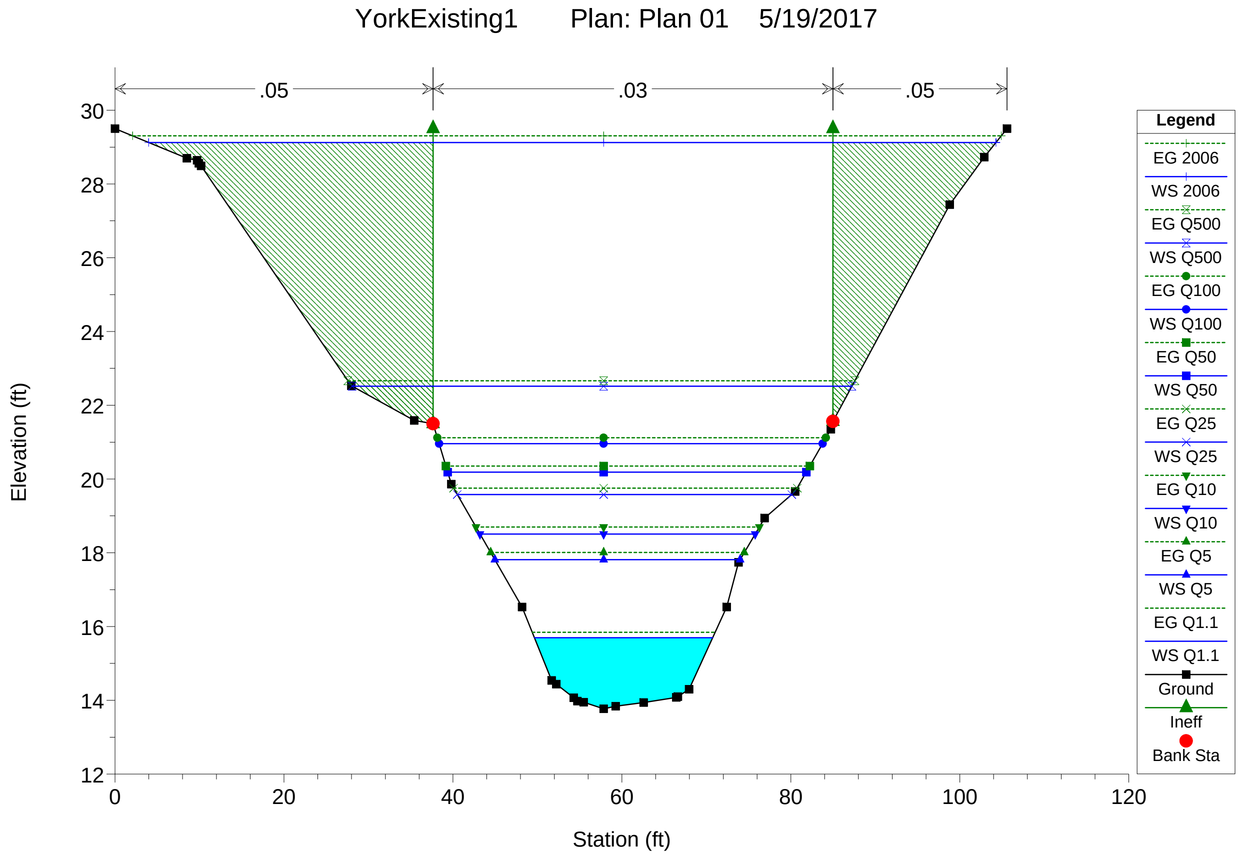
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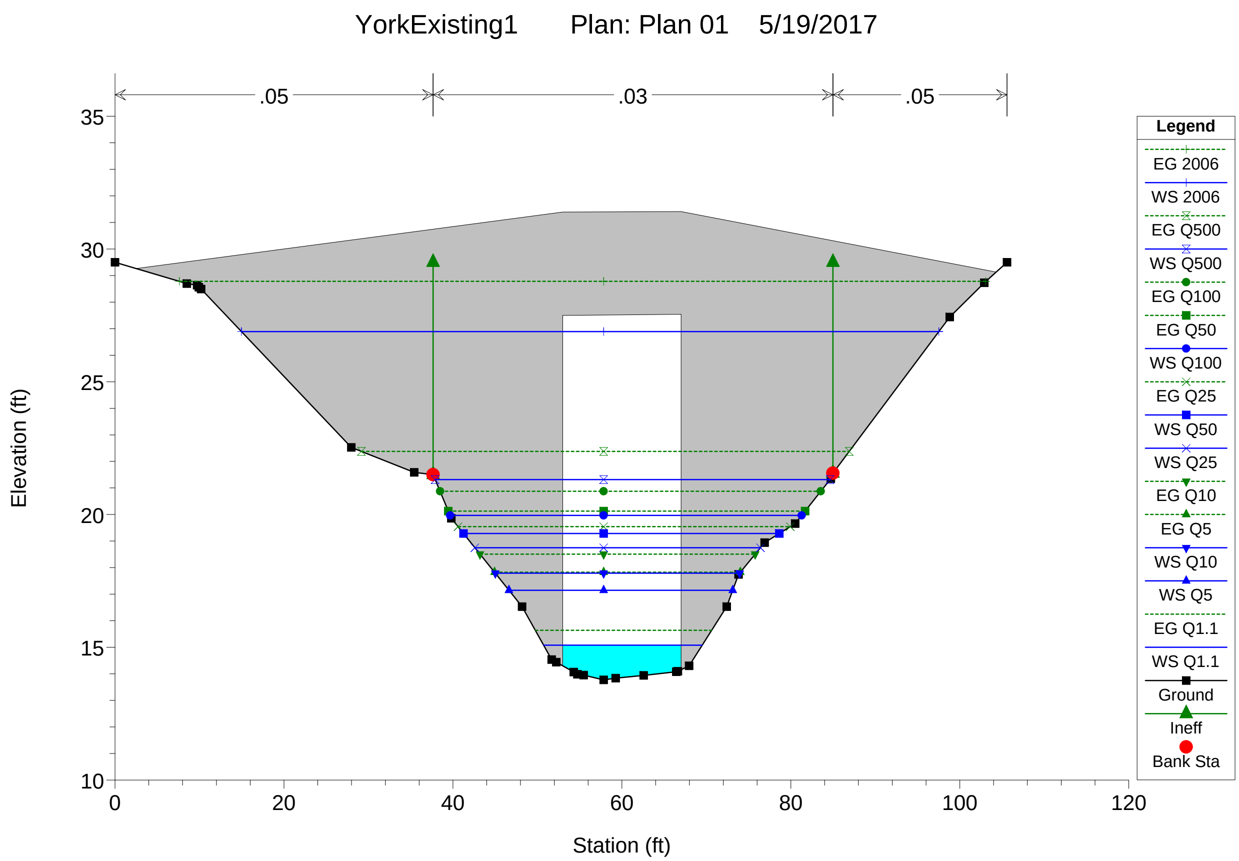
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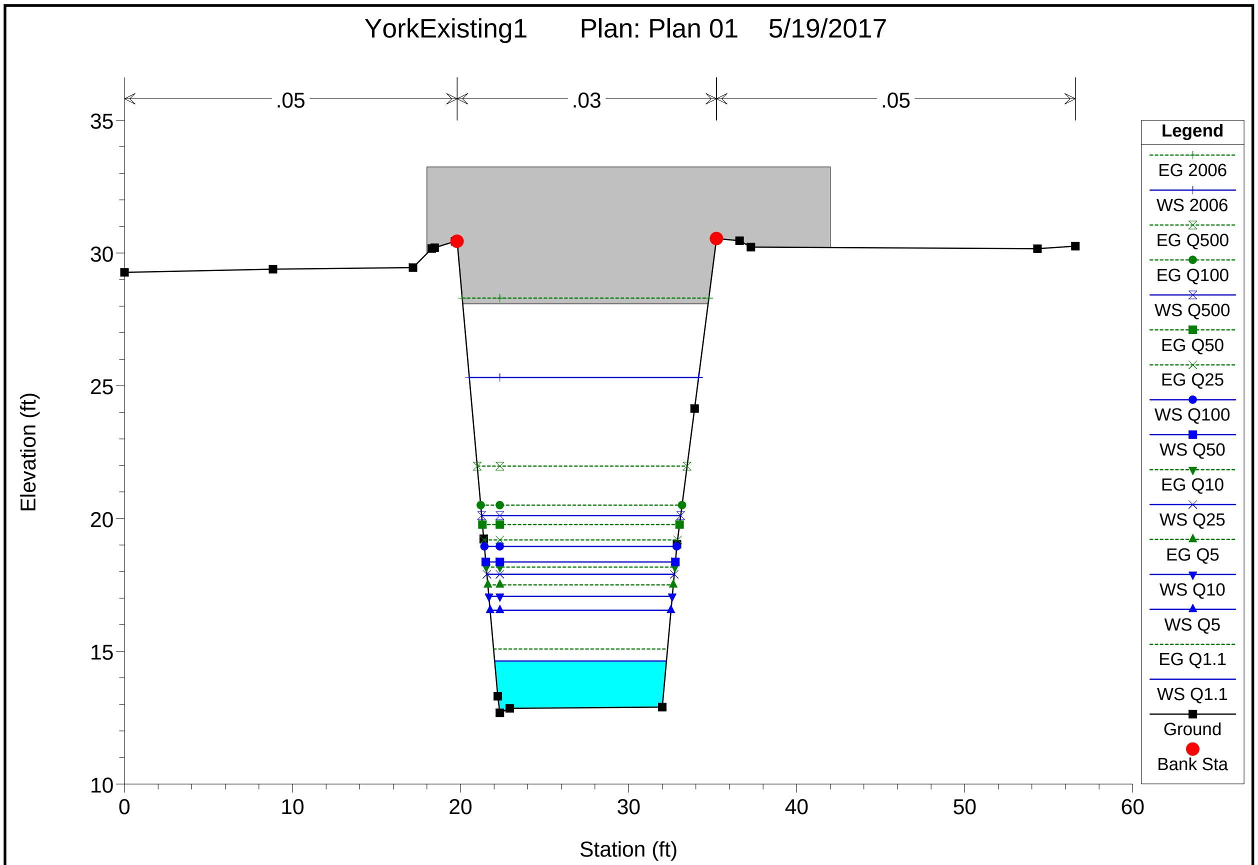
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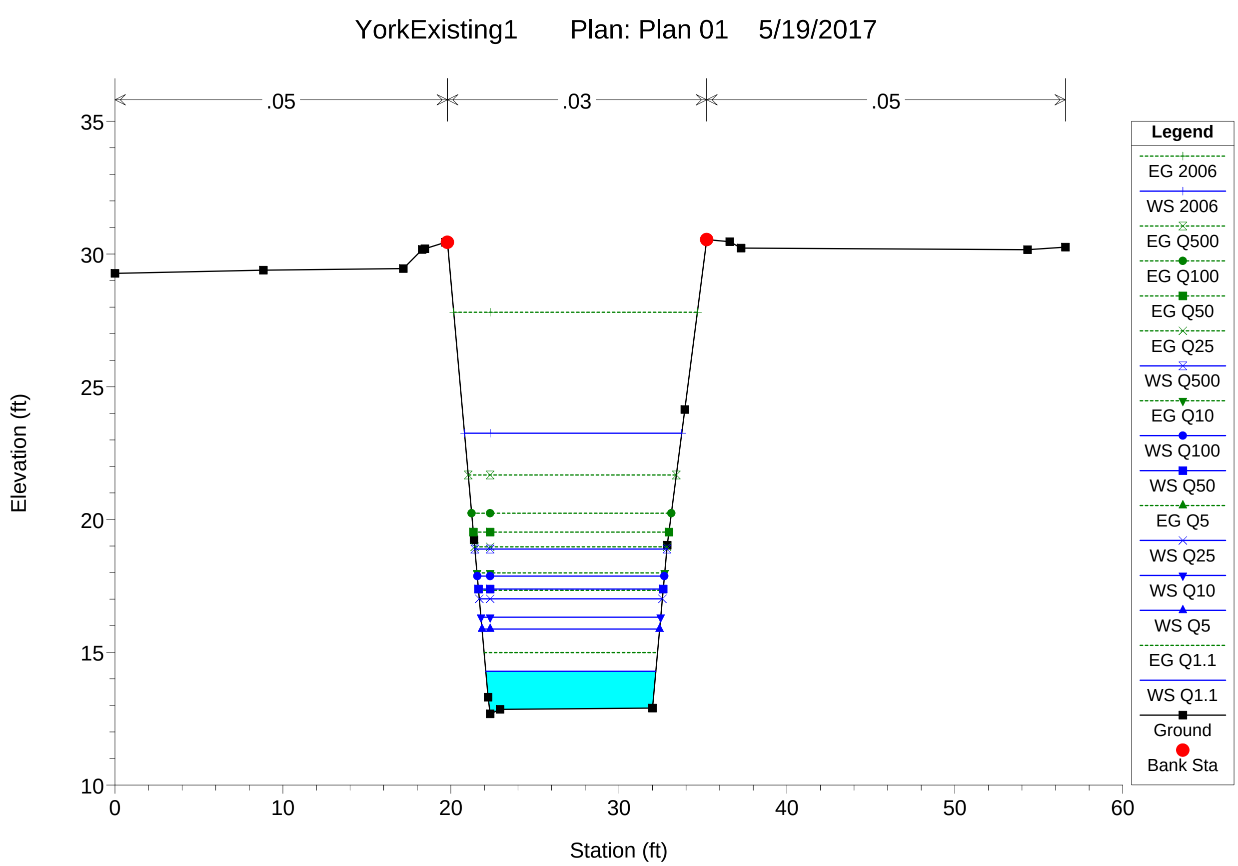
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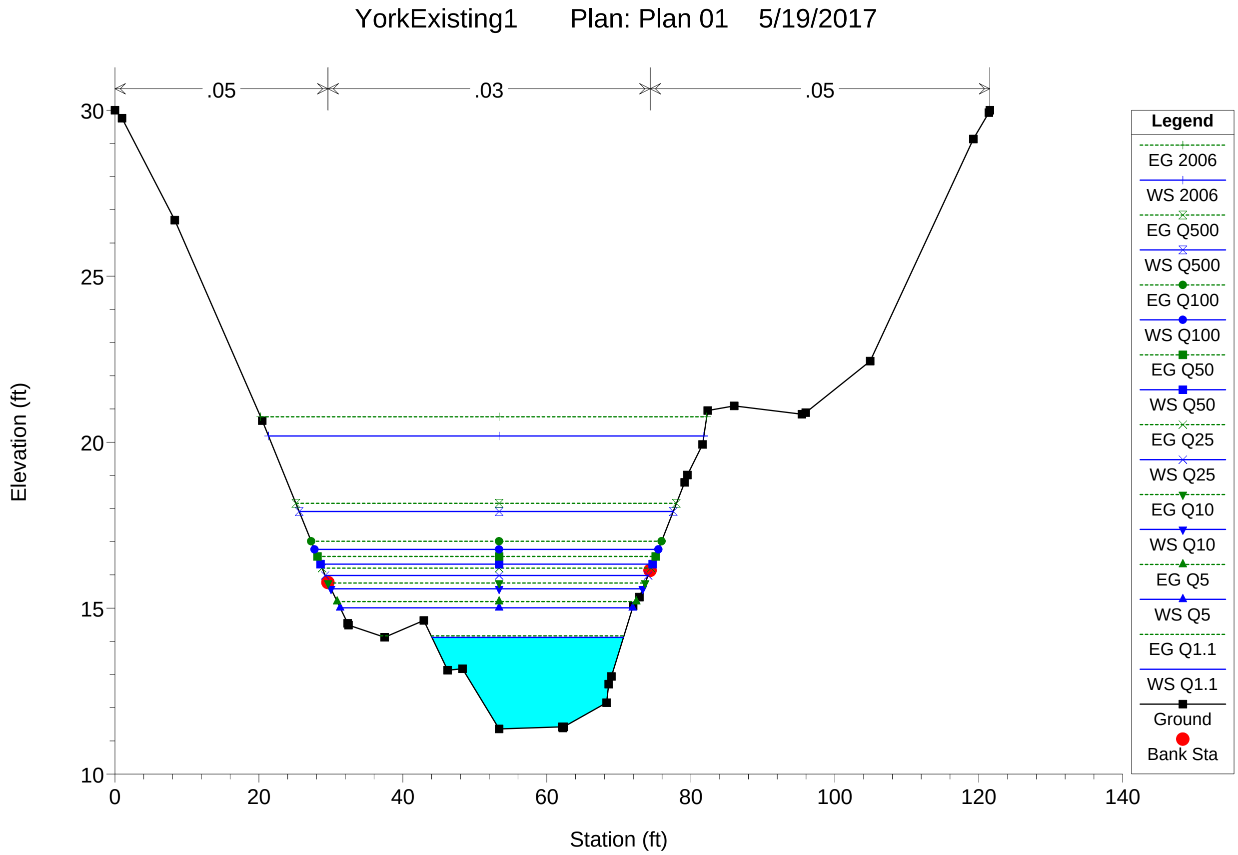
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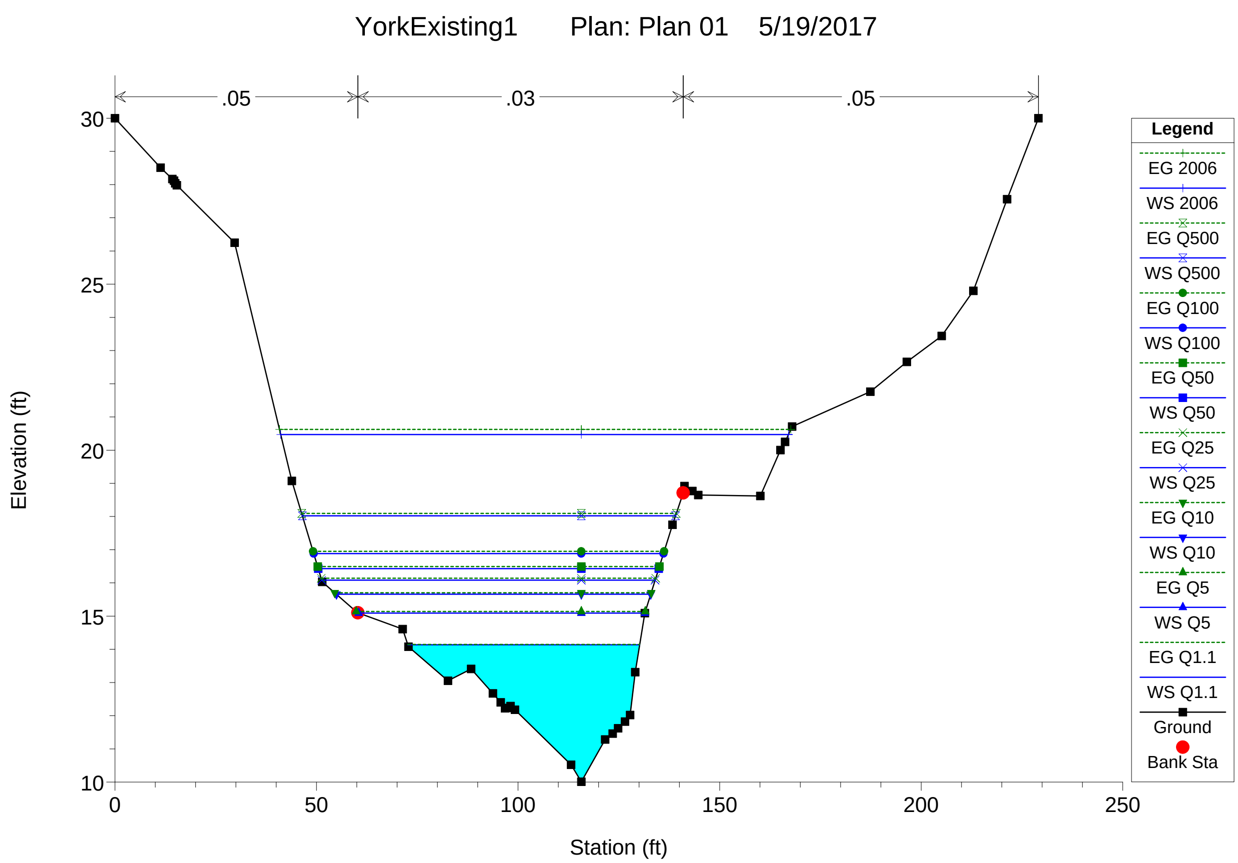
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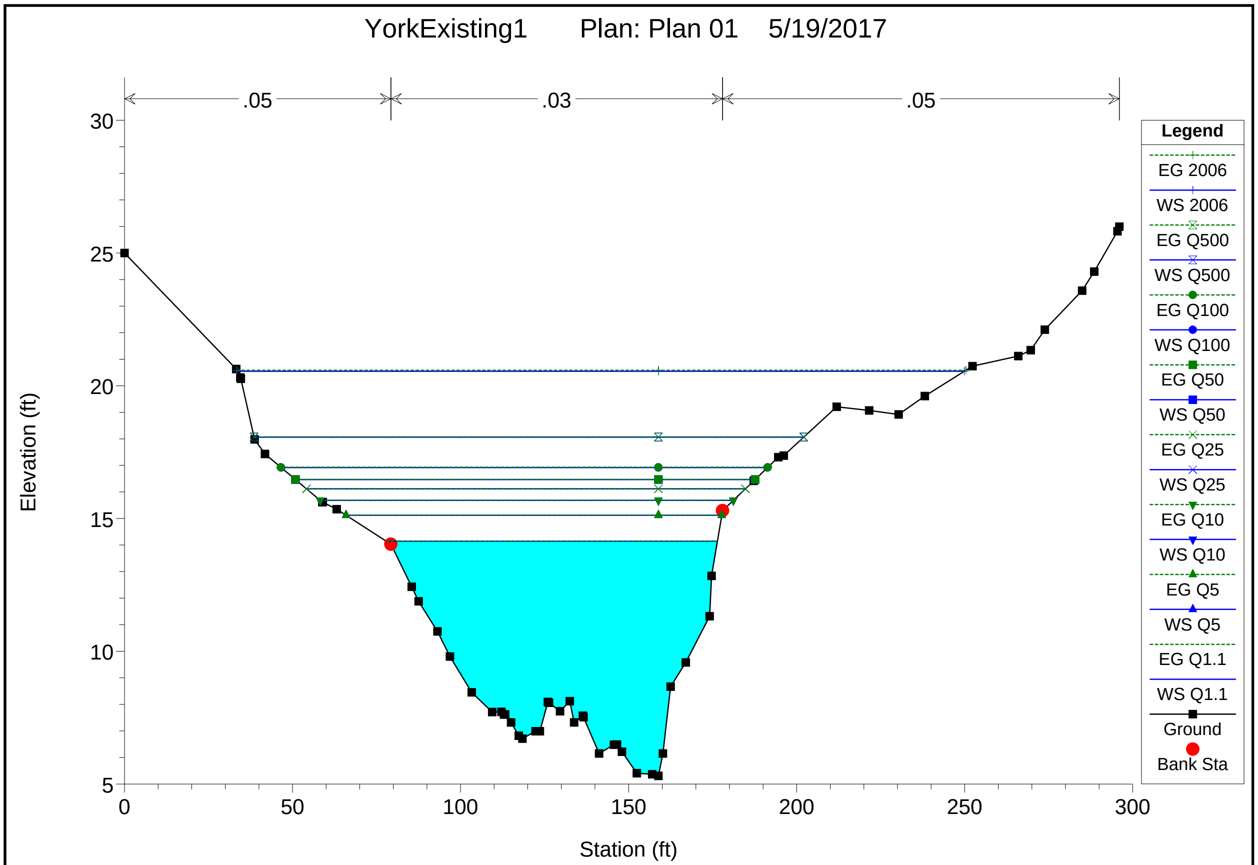

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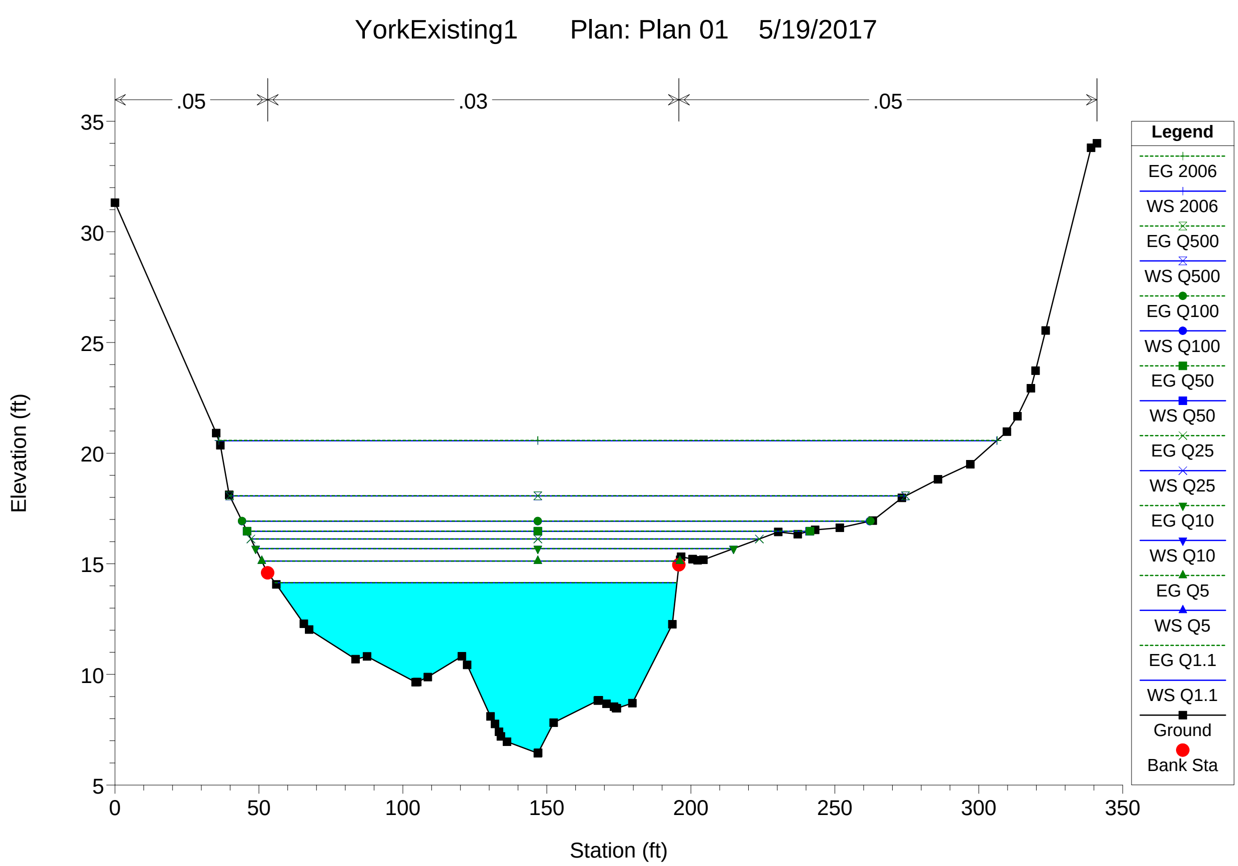

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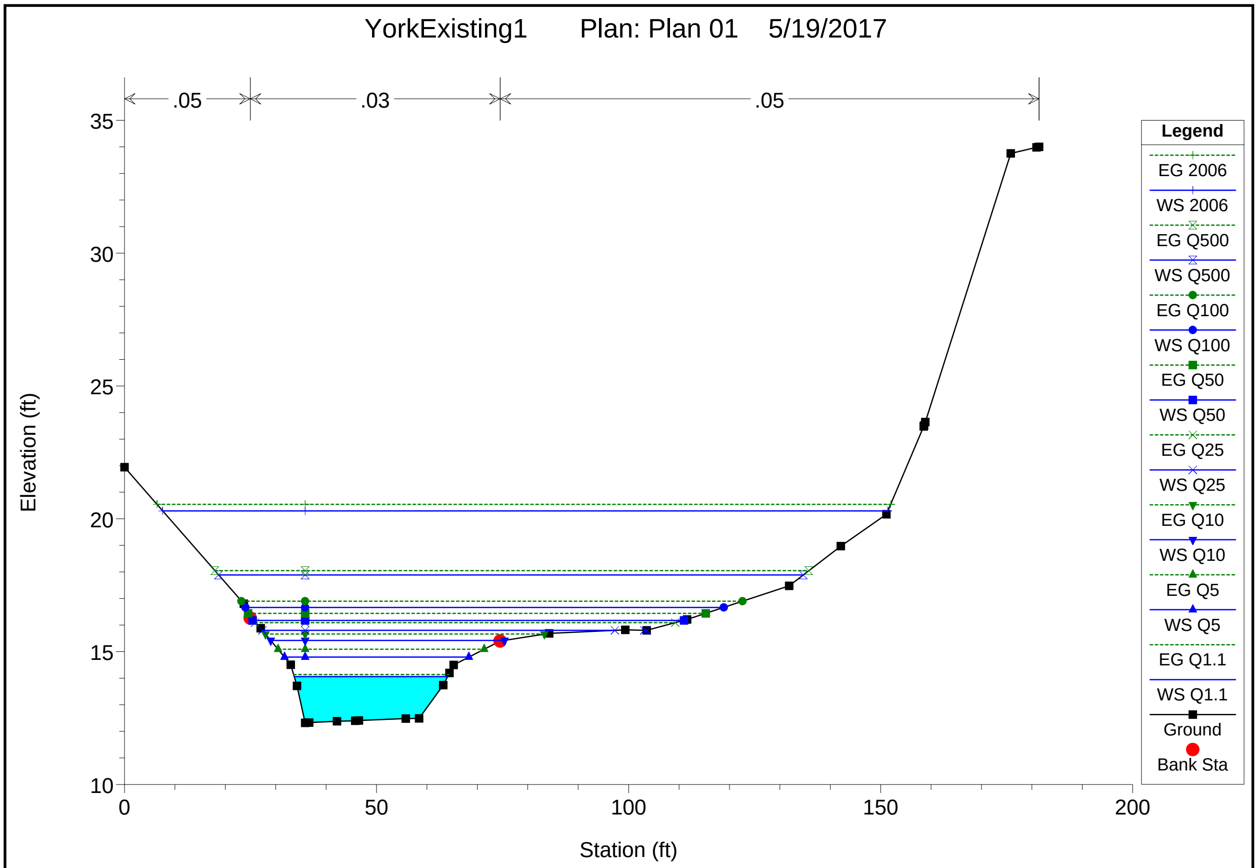

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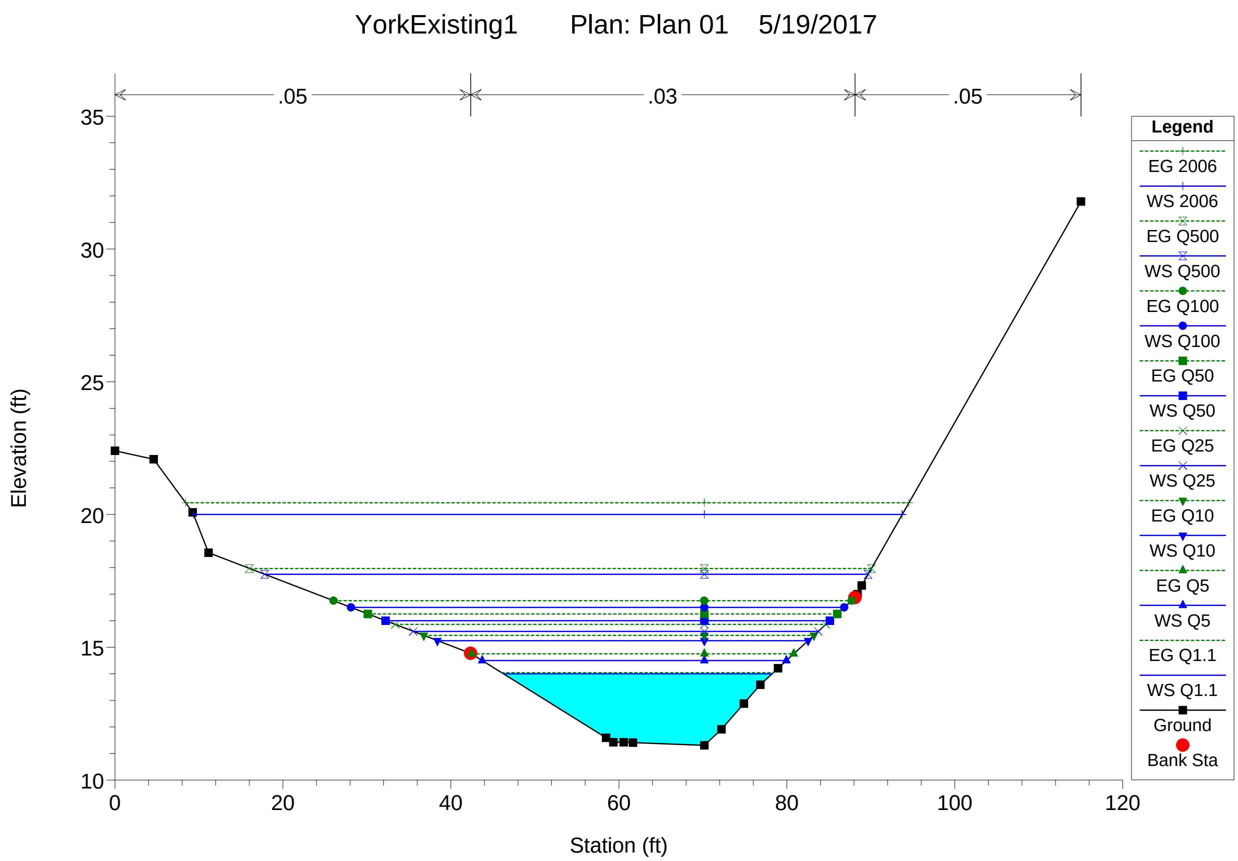

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Ground
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HEC-RAS Plan: Plan 01 River: Stream Reach: Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	529.1589	Q1.1	94.70	12.70	15.84		15.89	0.000454	1.70	56.78	36.96	0.20
Reach	529.1589	Q5	295.40	12.70	18.00		18.05	0.000237	1.92	241.13	112.11	0.16
Reach	529.1589	Q10	365.10	12.70	18.69		18.73	0.000180	1.85	323.87	126.18	0.15
Reach	529.1589	Q25	479.90	12.70	19.75		19.78	0.000123	1.74	462.74	136.69	0.13
Reach	529.1589	Q50	549.60	12.70	20.35		20.38	0.000104	1.70	545.94	140.84	0.12
Reach	529.1589	Q100	644.10	12.70	21.11		21.14	0.000086	1.67	655.92	147.60	0.11
Reach	529.1589	Q500	852.70	12.70	22.66		22.68	0.000065	1.65	897.39	162.46	0.10
Reach	529.1589	2006	2000.00	12.70	29.30		29.32	0.000033	1.70	2104.60	203.71	0.08
Reach	502.8161	Q1.1	94.70	12.29	15.86		15.87	0.000136	0.89	106.49	54.12	0.11
Reach	502.8161	Q5	295.40	12.29	18.01		18.04	0.000116	1.25	243.02	84.91	0.11
Reach	502.8161	Q10	365.10	12.29	18.70		18.72	0.000097	1.28	314.45	115.25	0.11
Reach	502.8161	Q25	479.90	12.29	19.75		19.78	0.000074	1.29	444.26	129.01	0.10
Reach	502.8161	Q50	549.60	12.29	20.35		20.37	0.000065	1.30	523.31	135.18	0.09
Reach	502.8161	Q100	644.10	12.29	21.11		21.14	0.000056	1.30	627.93	138.45	0.09
Reach	502.8161	Q500	852.70	12.29	22.66		22.68	0.000044	1.32	847.01	145.05	0.08
Reach	502.8161	2006	2000.00	12.29	29.30		29.32	0.000026	1.49	1904.57	175.39	0.07
Reach	482.7911	Q1.1	94.70	11.34	15.86		15.87	0.000072	0.75	125.74	50.94	0.08
Reach	482.7911	Q5	295.40	11.34	18.01		18.03	0.000101	1.12	262.64	75.56	0.11
Reach	482.7911	Q10	365.10	11.34	18.70		18.72	0.000095	1.15	317.47	84.26	0.10
Reach	482.7911	Q25	479.90	11.34	19.75		19.77	0.000080	1.16	416.81	119.54	0.10
Reach	482.7911	Q50	549.60	11.34	20.35		20.37	0.000068	1.16	490.26	124.27	0.09
Reach	482.7911	Q100	644.10	11.34	21.12		21.14	0.000056	1.17	586.64	127.82	0.09
Reach	482.7911	Q500	852.70	11.34	22.66		22.68	0.000042	1.18	789.70	134.99	0.08
Reach	482.7911	2006	2000.00	11.34	29.30		29.32	0.000023	1.34	1790.37	168.40	0.06
Reach	461.0650	Q1.1	94.70	12.46	15.84		15.86	0.000314	1.34	70.66	36.38	0.17
Reach	461.0650	Q5	295.40	12.46	17.98		18.03	0.000271	1.72	172.22	55.50	0.17
Reach	461.0650	Q10	365.10	12.46	18.67		18.72	0.000214	1.74	213.92	66.07	0.16
Reach	461.0650	Q25	479.90	12.46	19.72		19.77	0.000160	1.76	289.36	76.92	0.14
Reach	461.0650	Q50	549.60	12.46	20.32		20.37	0.000139	1.78	336.81	81.82	0.13
Reach	461.0650	Q100	644.10	12.46	21.08		21.13	0.000120	1.80	401.80	88.40	0.13
Reach	461.0650	Q500	852.70	12.46	22.63		22.68	0.000093	1.84	548.73	101.09	0.12
Reach	461.0650	2006	2000.00	12.46	29.27		29.32	0.000050	2.03	1364.70	148.75	0.09
Reach	450.8967	Q1.1	94.70	13.77	15.70	15.02	15.84	0.002492	3.10	30.57	21.09	0.45
Reach	450.8967	Q5	295.40	13.77	17.81	16.10	18.01	0.001349	3.53	83.60	29.03	0.37

HEC-RAS Plan: Plan 01 River: Stream Reach: Reach (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	450.8967	Q10	365.10	13.77	18.51	16.39	18.70	0.001124	3.48	105.05	32.58	0.34
Reach	450.8967	Q25	479.90	13.77	19.58	16.81	19.75	0.000892	3.35	143.29	39.59	0.31
Reach	450.8967	Q50	549.60	13.77	20.19	17.05	20.35	0.000752	3.26	168.35	42.45	0.29
Reach	450.8967	Q100	644.10	13.77	20.96	17.35	21.12	0.000615	3.18	202.35	45.39	0.27
Reach	450.8967	Q500	852.70	13.77	22.52	17.95	22.67	0.000410	3.10	275.49	59.17	0.23
Reach	450.8967	2006	2000.00	13.77	29.13	20.34	29.30	0.000180	3.40	588.26	100.29	0.17
Reach	412.5		Bridge									
Reach	389.1057	Q1.1	94.70	12.68	14.29	14.29	14.99	0.016167	6.74	14.06	10.12	1.01
Reach	389.1057	Q5	295.40	12.68	15.87	15.87	17.33	0.016116	9.69	30.47	10.57	1.01
Reach	389.1057	Q10	365.10	12.68	16.32	16.32	17.99	0.016326	10.36	35.24	10.70	1.01
Reach	389.1057	Q25	479.90	12.68	17.01	17.01	18.97	0.016538	11.24	42.70	10.89	1.00
Reach	389.1057	Q50	549.60	12.68	17.38	17.38	19.53	0.016937	11.76	46.73	11.00	1.01
Reach	389.1057	Q100	644.10	12.68	17.87	17.87	20.24	0.017254	12.35	52.17	11.14	1.01
Reach	389.1057	Q500	852.70	12.68	18.89	18.89	21.68	0.017748	13.39	63.66	11.43	1.00
Reach	389.1057	2006	2000.00	12.68	23.25	23.25	27.81	0.020301	17.13	116.74	12.93	1.00
Reach	360.8755	Q1.1	94.70	11.36	14.11		14.16	0.000563	1.80	52.75	26.60	0.22
Reach	360.8755	Q5	295.40	11.36	15.01		15.20	0.001945	3.47	85.14	40.64	0.42
Reach	360.8755	Q10	365.10	11.36	15.58		15.75	0.001419	3.35	109.12	43.31	0.37
Reach	360.8755	Q25	479.90	11.36	15.98		16.20	0.001543	3.78	126.86	44.88	0.39
Reach	360.8755	Q50	549.60	11.36	16.32		16.56	0.001396	3.87	142.46	46.15	0.38
Reach	360.8755	Q100	644.10	11.36	16.77		17.01	0.001236	3.97	163.36	47.78	0.37
Reach	360.8755	Q500	852.70	11.36	17.91		18.15	0.000860	3.97	220.27	51.97	0.32
Reach	360.8755	2006	2000.00	11.36	20.18		20.76	0.001237	6.18	348.18	60.48	0.41
Reach	338.8639	Q1.1	94.70	10.01	14.14		14.15	0.000117	0.83	113.69	57.50	0.10
Reach	338.8639	Q5	295.40	10.01	15.09		15.14	0.000376	1.71	172.76	70.99	0.19
Reach	338.8639	Q10	365.10	10.01	15.66		15.70	0.000293	1.71	214.93	77.97	0.18
Reach	338.8639	Q25	479.90	10.01	16.08		16.14	0.000326	1.96	249.13	82.69	0.19
Reach	338.8639	Q50	549.60	10.01	16.43		16.49	0.000310	2.02	278.20	84.46	0.19
Reach	338.8639	Q100	644.10	10.01	16.88		16.95	0.000290	2.09	316.85	86.75	0.18
Reach	338.8639	Q500	852.70	10.01	18.02		18.09	0.000225	2.13	419.20	92.59	0.17
Reach	338.8639	2006	2000.00	10.01	20.47		20.62	0.000309	3.21	697.44	125.96	0.21
Reach	308.7850	Q1.1	94.70	5.31	14.14		14.14	0.000001	0.17	549.36	98.57	0.01
Reach	308.7850	Q5	295.40	5.31	15.12		15.12	0.000007	0.46	652.08	111.80	0.03

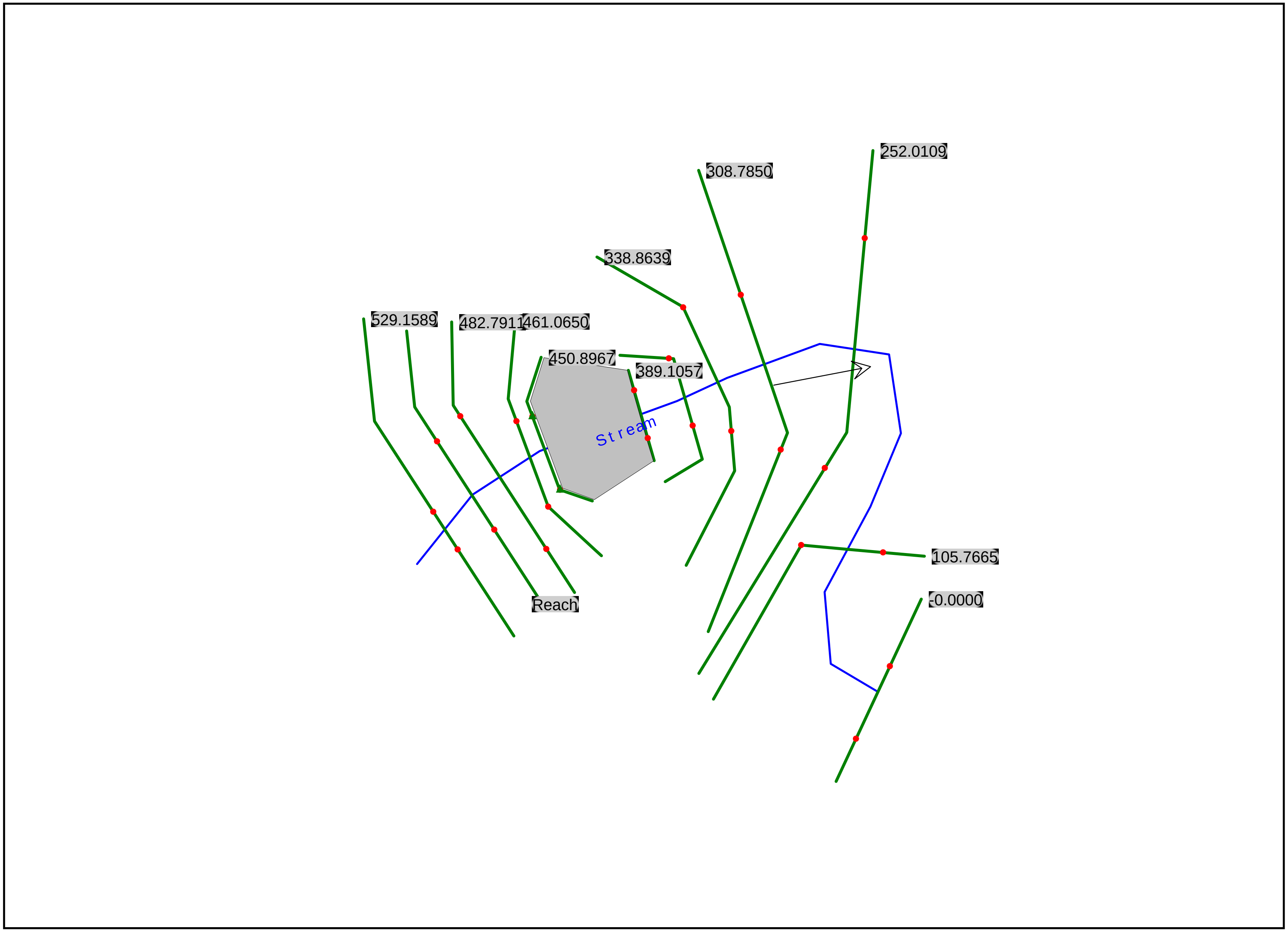
HEC-RAS Plan: Plan 01   River: Stream   Reach: Reach (Continued)

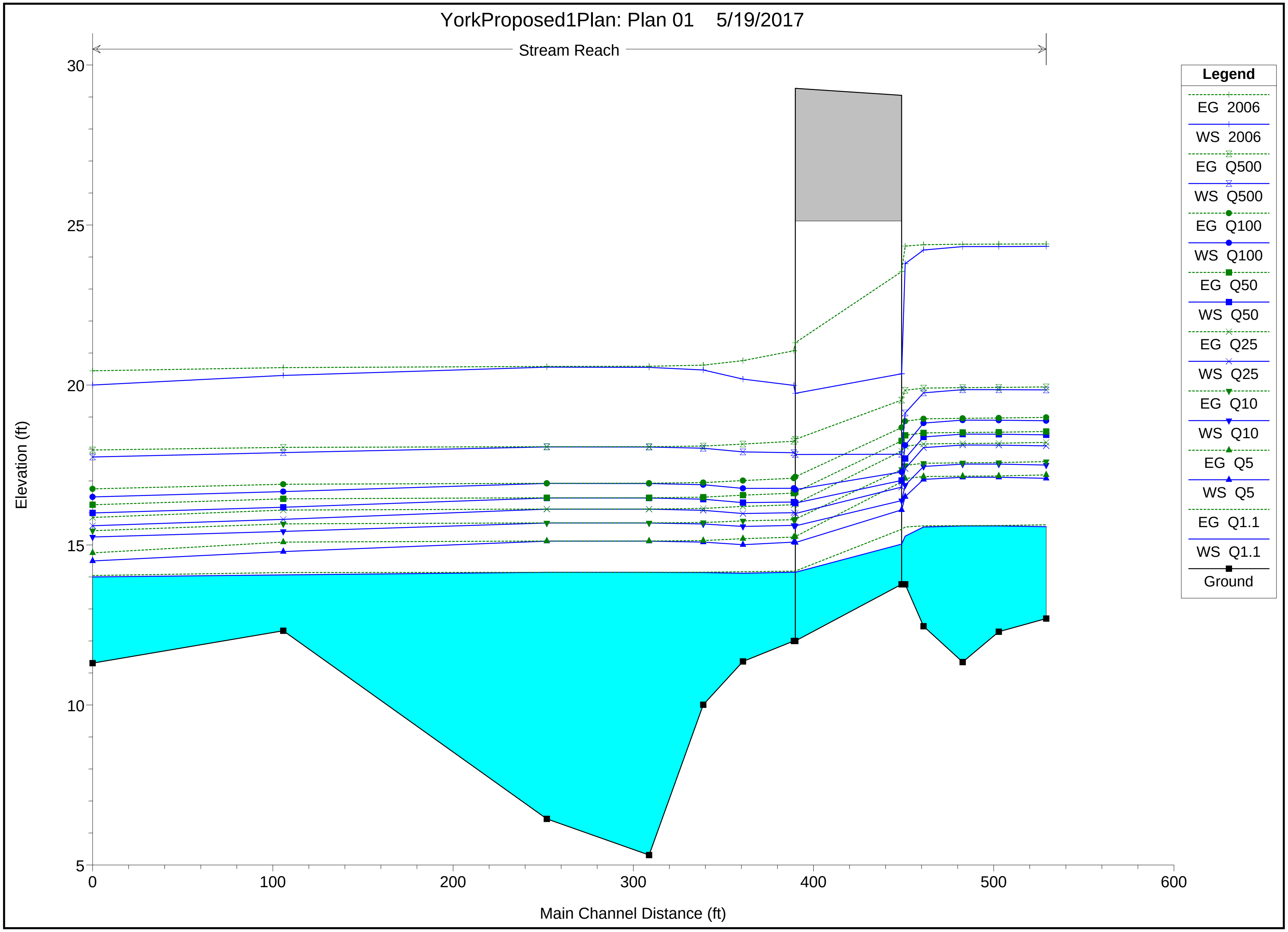
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	308.7850	Q10	365.10	5.31	15.69		15.69	0.000009	0.52	718.07	122.92	0.03
Reach	308.7850	Q25	479.90	5.31	16.12		16.12	0.000012	0.64	772.93	130.57	0.04
Reach	308.7850	Q50	549.60	5.31	16.47		16.48	0.000014	0.70	819.65	136.76	0.04
Reach	308.7850	Q100	644.10	5.31	16.92		16.93	0.000015	0.77	883.47	144.76	0.05
Reach	308.7850	Q500	852.70	5.31	18.06		18.07	0.000017	0.89	1060.18	163.51	0.05
Reach	308.7850	2006	2000.00	5.31	20.55		20.58	0.000039	1.57	1534.05	216.37	0.08
Reach	252.0109	Q1.1	94.70	6.44	14.14		14.14	0.000001	0.15	611.25	139.57	0.01
Reach	252.0109	Q5	295.40	6.44	15.12		15.12	0.000007	0.39	750.48	145.19	0.03
Reach	252.0109	Q10	365.10	6.44	15.69		15.69	0.000008	0.44	839.53	165.95	0.03
Reach	252.0109	Q25	479.90	6.44	16.12		16.12	0.000010	0.54	913.70	176.53	0.04
Reach	252.0109	Q50	549.60	6.44	16.47		16.47	0.000011	0.58	977.73	195.33	0.04
Reach	252.0109	Q100	644.10	6.44	16.92		16.93	0.000012	0.63	1072.04	218.02	0.04
Reach	252.0109	Q500	852.70	6.44	18.06		18.07	0.000013	0.71	1330.60	234.68	0.04
Reach	252.0109	2006	2000.00	6.44	20.56		20.58	0.000026	1.21	1964.64	270.12	0.07
Reach	105.7665	Q1.1	94.70	12.32	14.06		14.14	0.001217	2.16	43.79	30.43	0.32
Reach	105.7665	Q5	295.40	12.32	14.79		15.09	0.003575	4.37	67.56	36.56	0.57
Reach	105.7665	Q10	365.10	12.32	15.43		15.66	0.002459	3.90	93.58	46.38	0.48
Reach	105.7665	Q25	479.90	12.32	15.80		16.09	0.002503	4.31	114.29	70.44	0.50
Reach	105.7665	Q50	549.60	12.32	16.18		16.44	0.002017	4.16	144.69	85.60	0.45
Reach	105.7665	Q100	644.10	12.32	16.67		16.90	0.001478	3.97	188.85	94.91	0.40
Reach	105.7665	Q500	852.70	12.32	17.89		18.05	0.000714	3.45	318.92	115.99	0.29
Reach	105.7665	2006	2000.00	12.32	20.30		20.54	0.000668	4.48	632.54	143.89	0.30
Reach	-0.0000	Q1.1	94.70	11.31	14.00	12.51	14.04	0.000559	1.68	56.27	31.99	0.22
Reach	-0.0000	Q5	295.40	11.31	14.50	13.53	14.75	0.002663	4.03	73.32	36.24	0.50
Reach	-0.0000	Q10	365.10	11.31	15.25	13.79	15.45	0.001537	3.56	103.26	44.14	0.39
Reach	-0.0000	Q25	479.90	11.31	15.60	14.17	15.86	0.001777	4.10	119.43	48.23	0.43
Reach	-0.0000	Q50	549.60	11.31	16.00	14.37	16.26	0.001538	4.08	139.65	52.90	0.41
Reach	-0.0000	Q100	644.10	11.31	16.50	14.63	16.75	0.001320	4.07	167.56	58.74	0.38
Reach	-0.0000	Q500	852.70	11.31	17.75	15.11	17.97	0.000799	3.83	249.47	71.86	0.31
Reach	-0.0000	2006	2000.00	11.31	20.00	17.04	20.44	0.001022	5.63	429.32	84.38	0.38

APPENDIX E

Proposed HEC-RAS Analysis

Proposed Cross-section Layout





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Stream Reach
Legend
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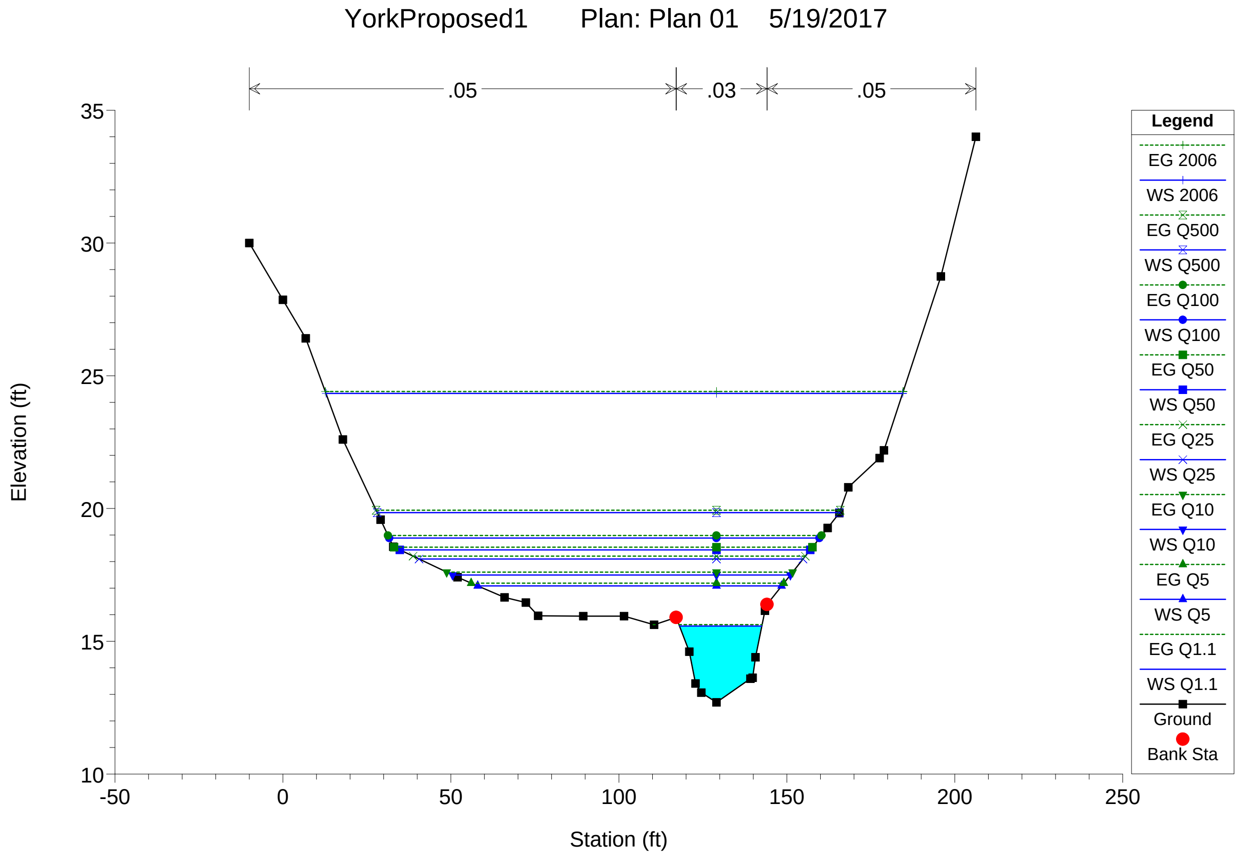
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Elevation (ft)
Main Channel Distance (ft)
30
25
20
15
10
5
0
100
200
300
400
500
600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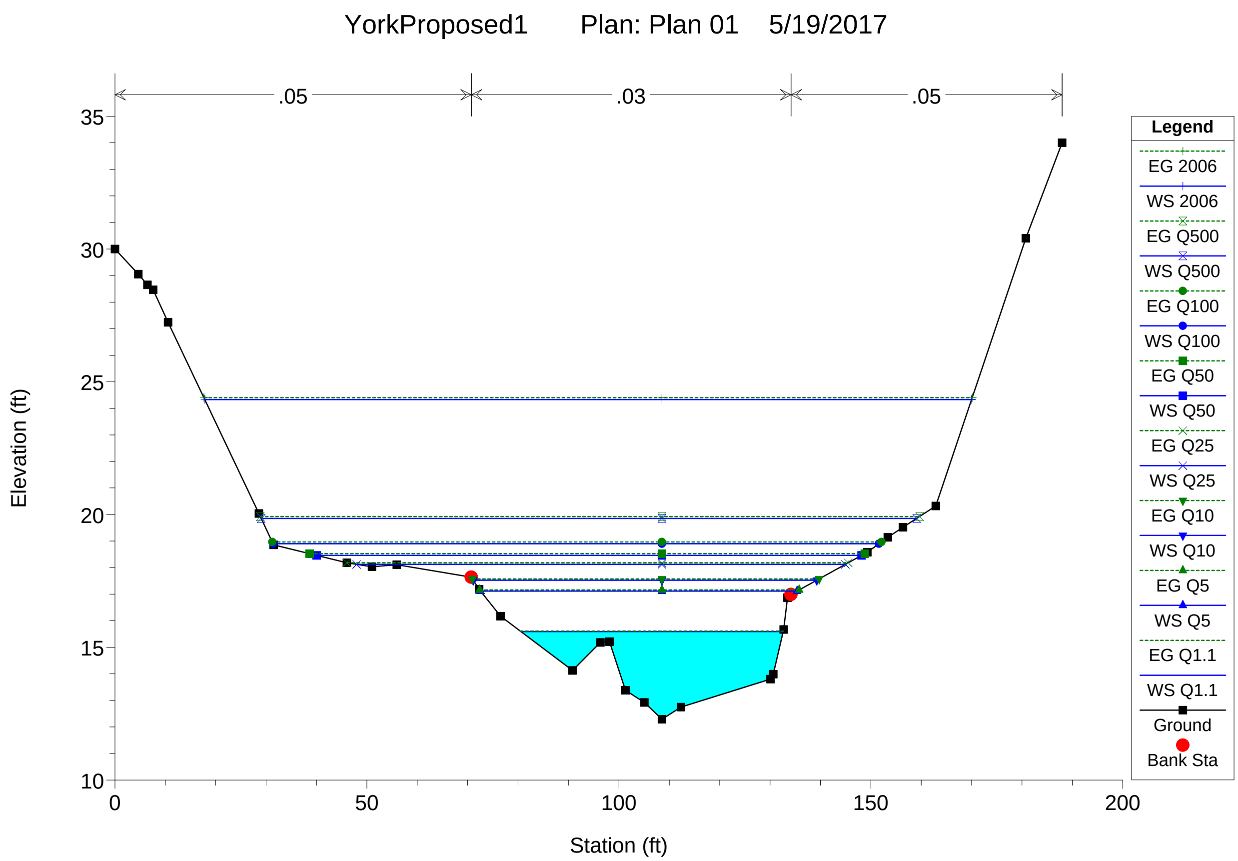
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta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.03
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Elevation (ft)
Station (ft)
Legend
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WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
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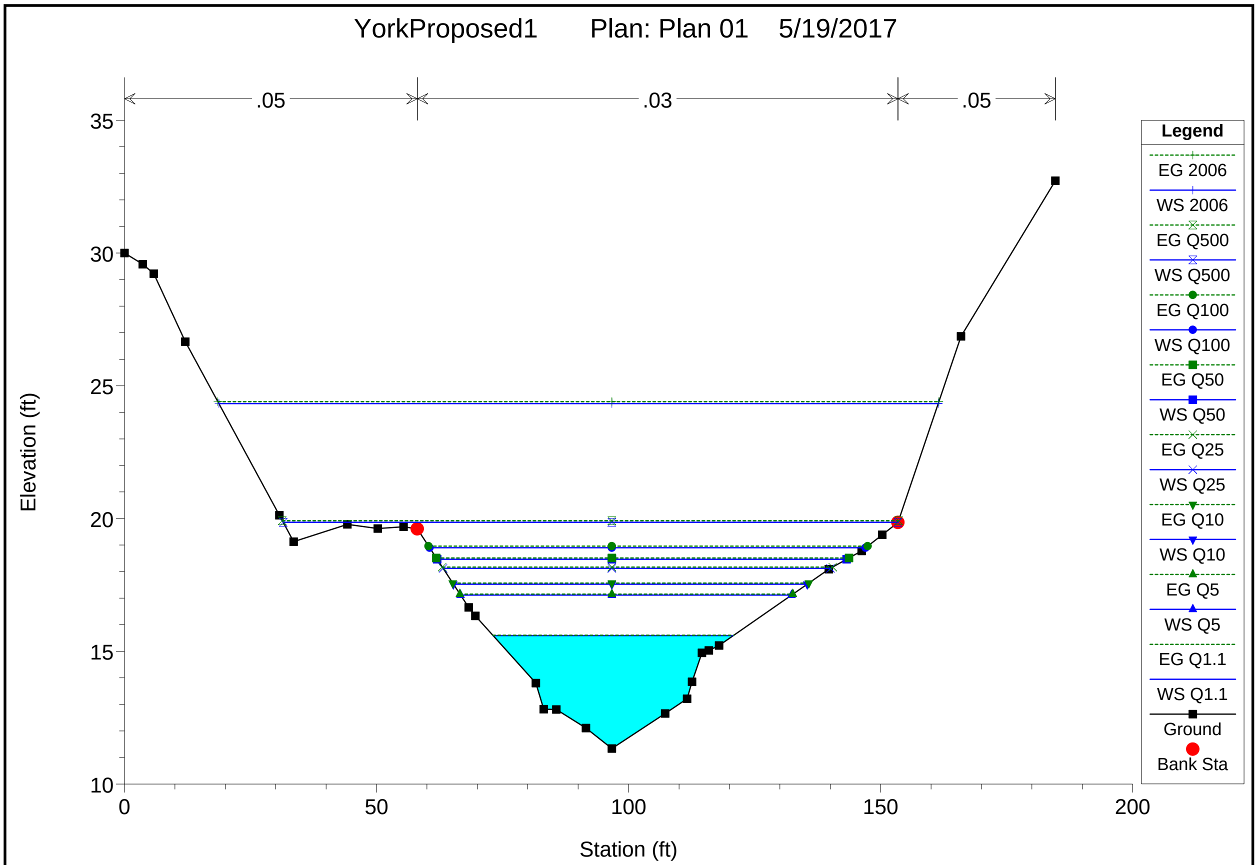
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta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.05
Elevation (ft)
Station (ft)
Legend
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WS 2006
EG Q500
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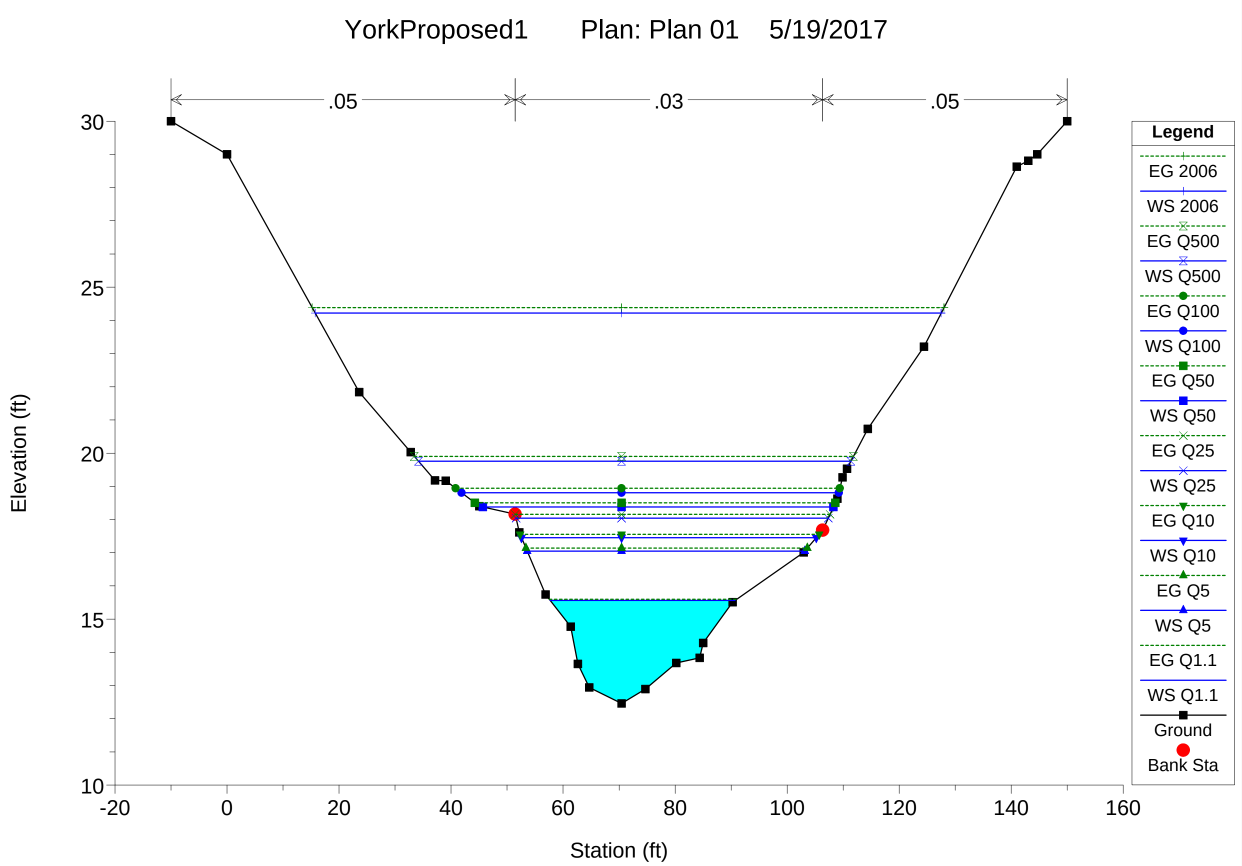
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta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.05
.03
.05
Elevation (ft)
Station (ft)
Legend
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WS 2006
EG Q500
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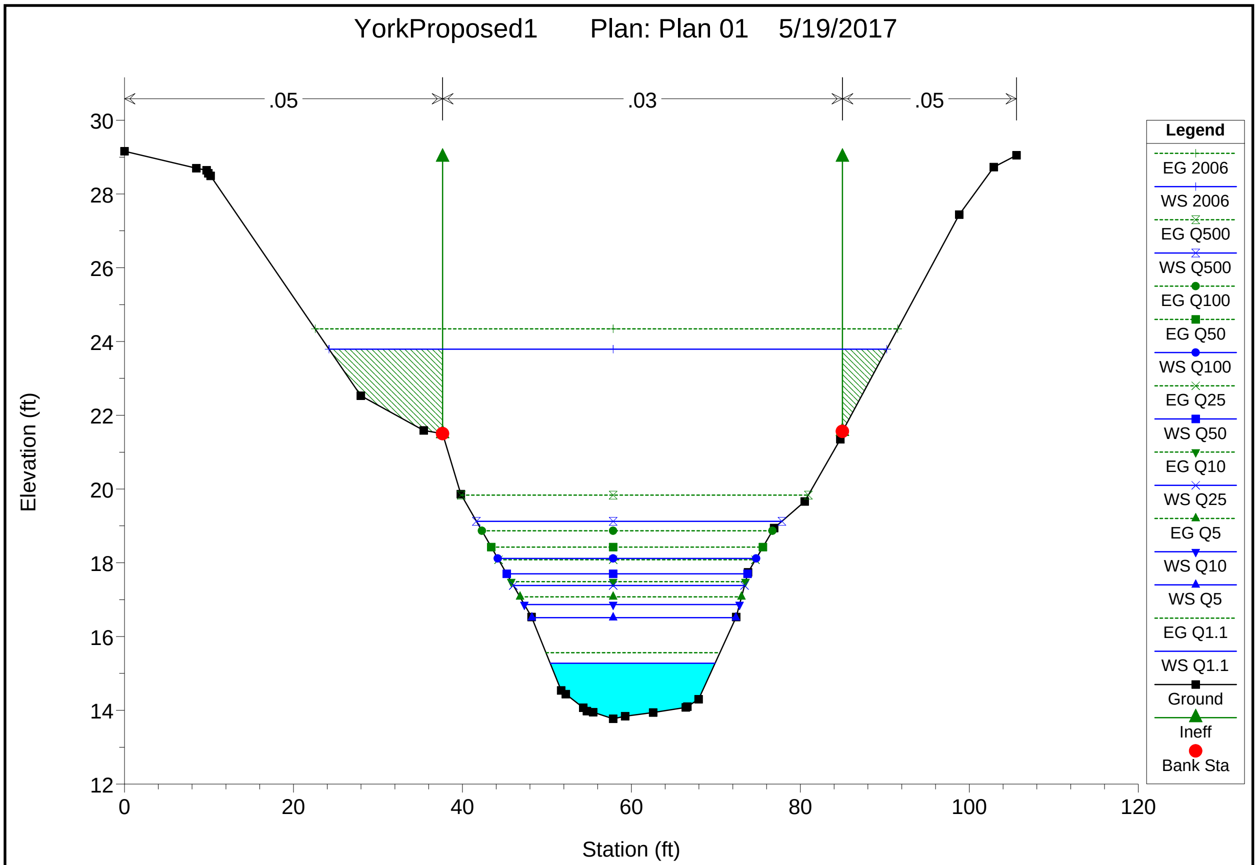
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta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.03
.05
Legend
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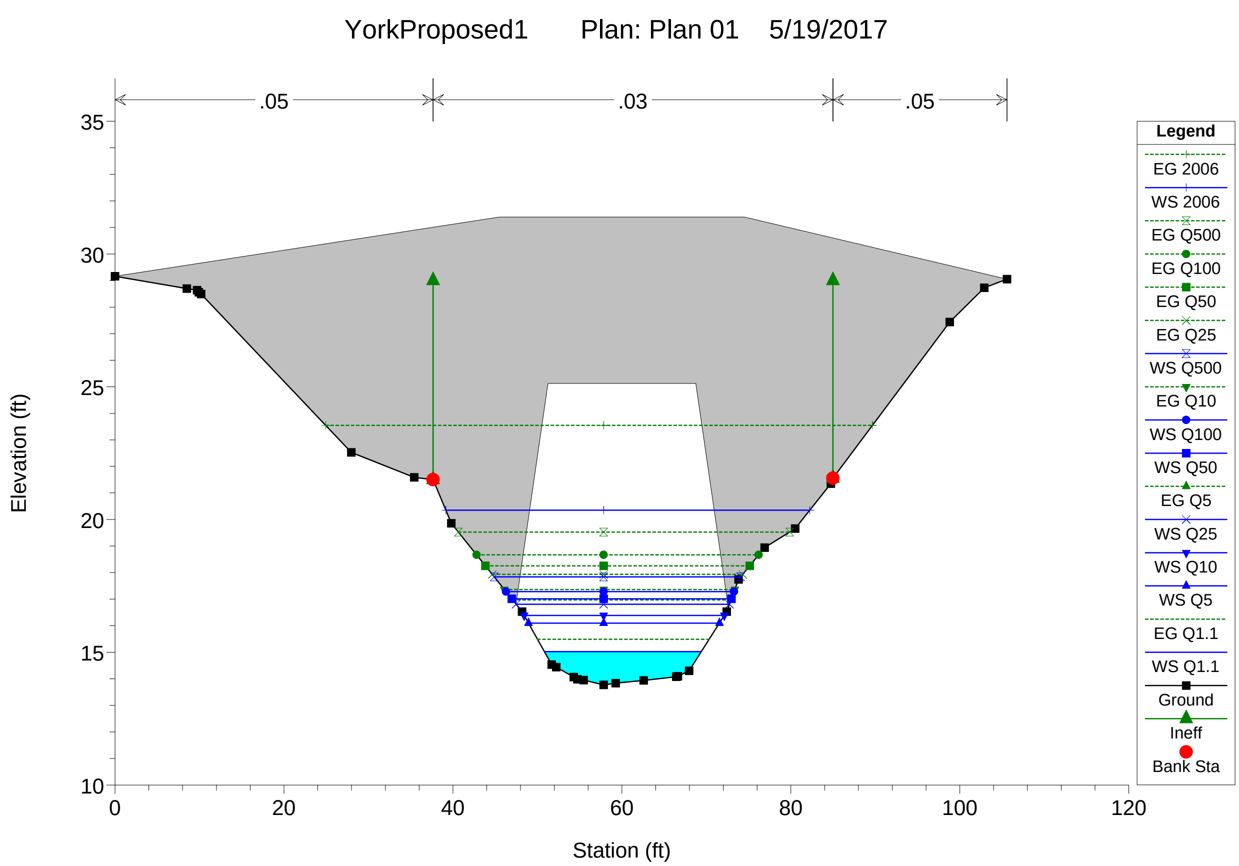
WS 2006
EG Q500
WS Q500
EG Q100
EG Q50
WS Q100
EG Q25
WS Q50
EG Q10
WS Q25
EG Q5
WS Q10
WS Q5
EG Q1.1
WS Q1.1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)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.03
.05
Legend
EG 2006
WS 2006
EG Q500
EG Q100
EG Q50
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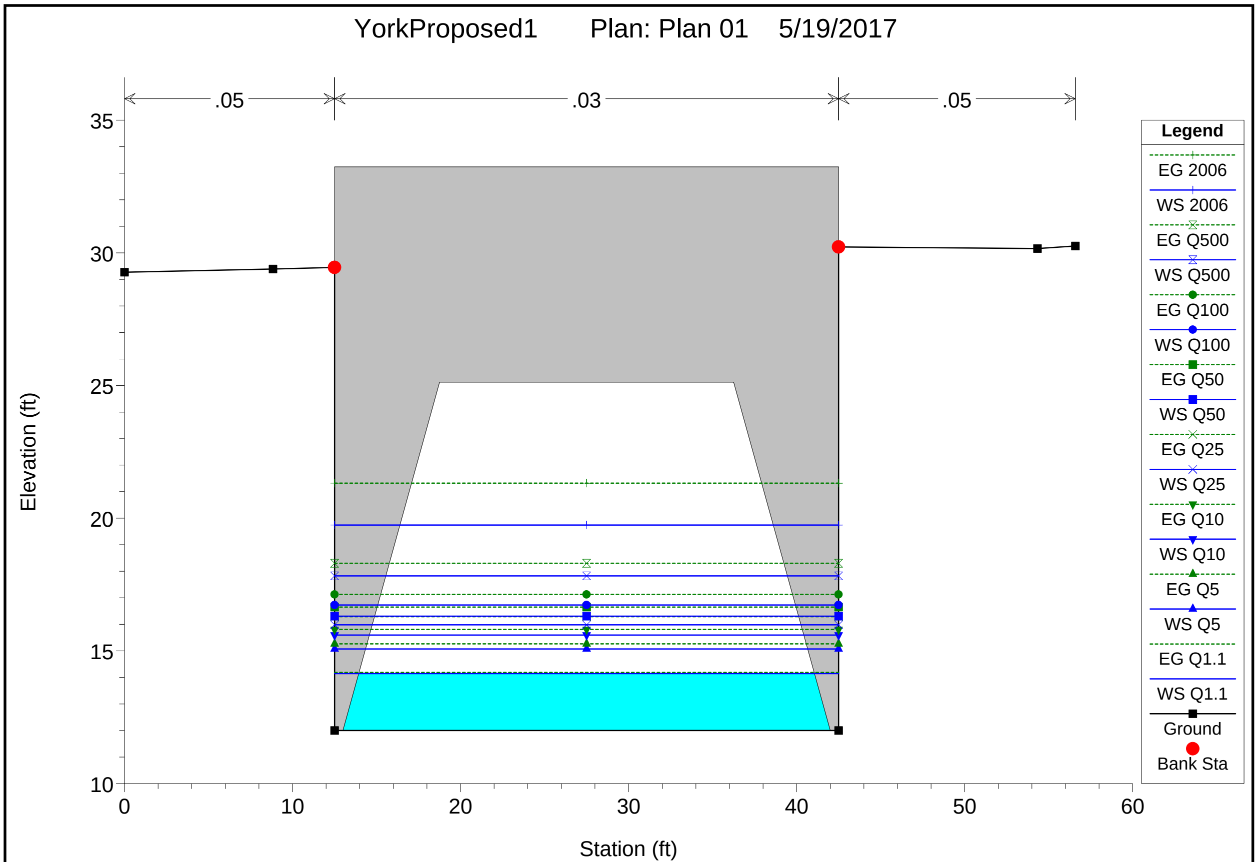
WS Q500
EG Q10
WS Q100
WS Q50
EG Q5
WS Q25
WS Q10
WS Q5
EG Q1.1
WS Q1.1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


YorkProposed1 Plan: Plan 01 5/19/2017
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.03
.05
Elevation (ft)
Station (ft)
Legend
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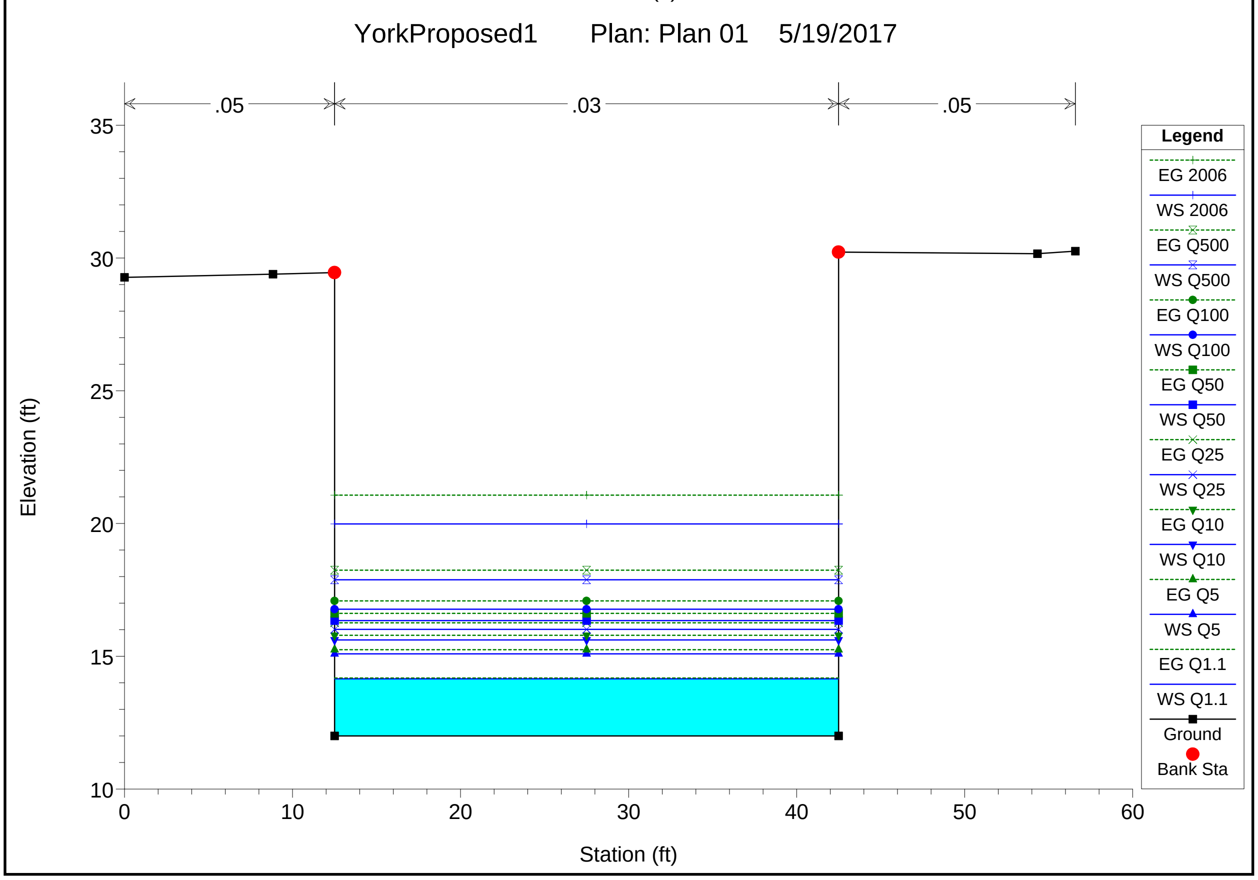
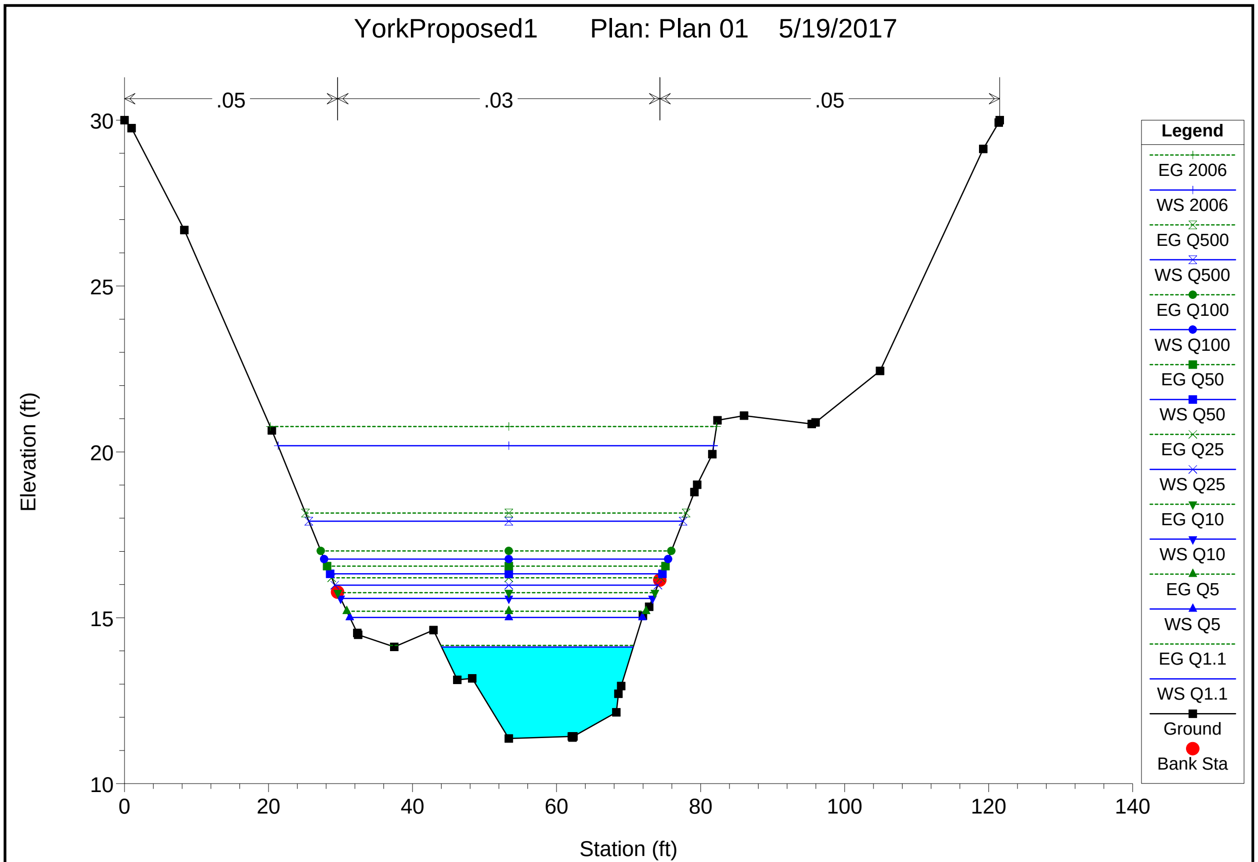
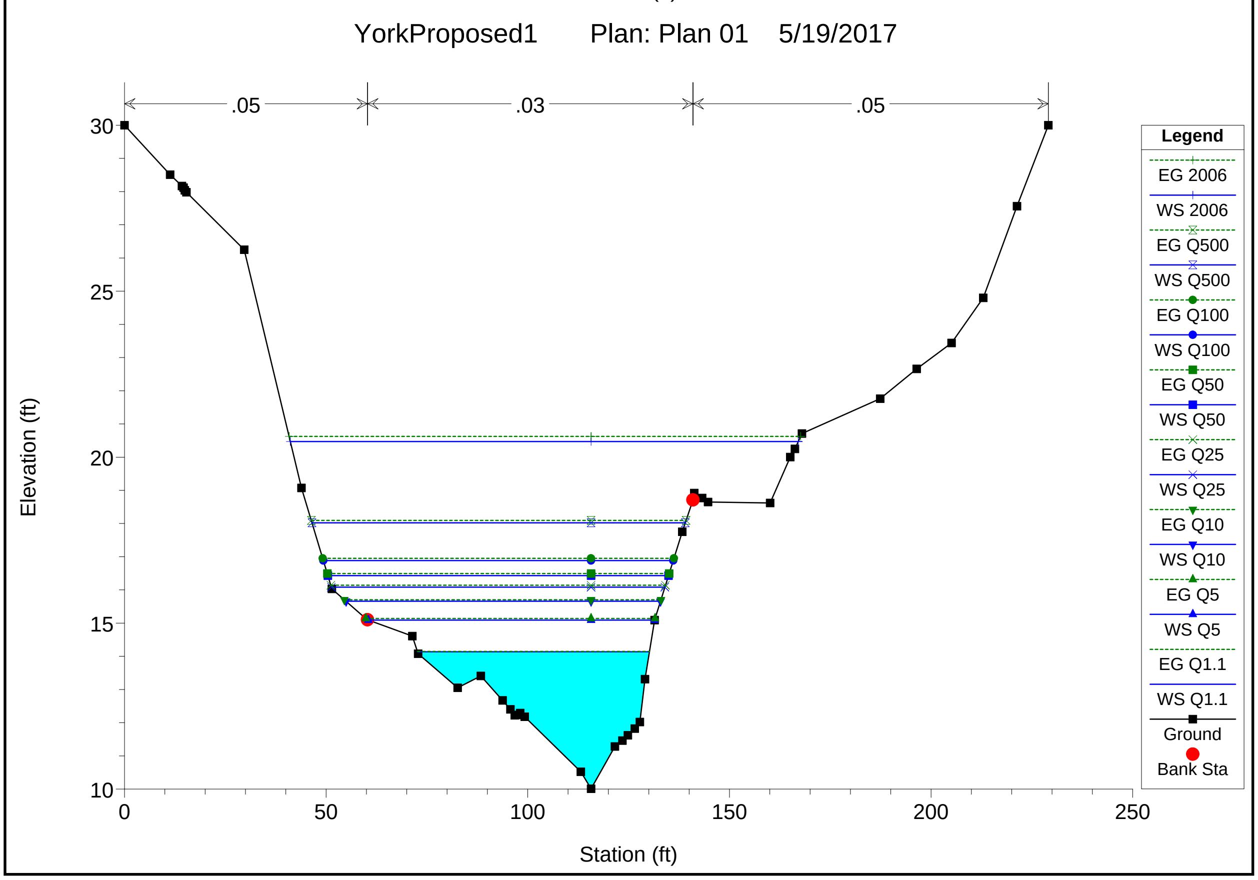
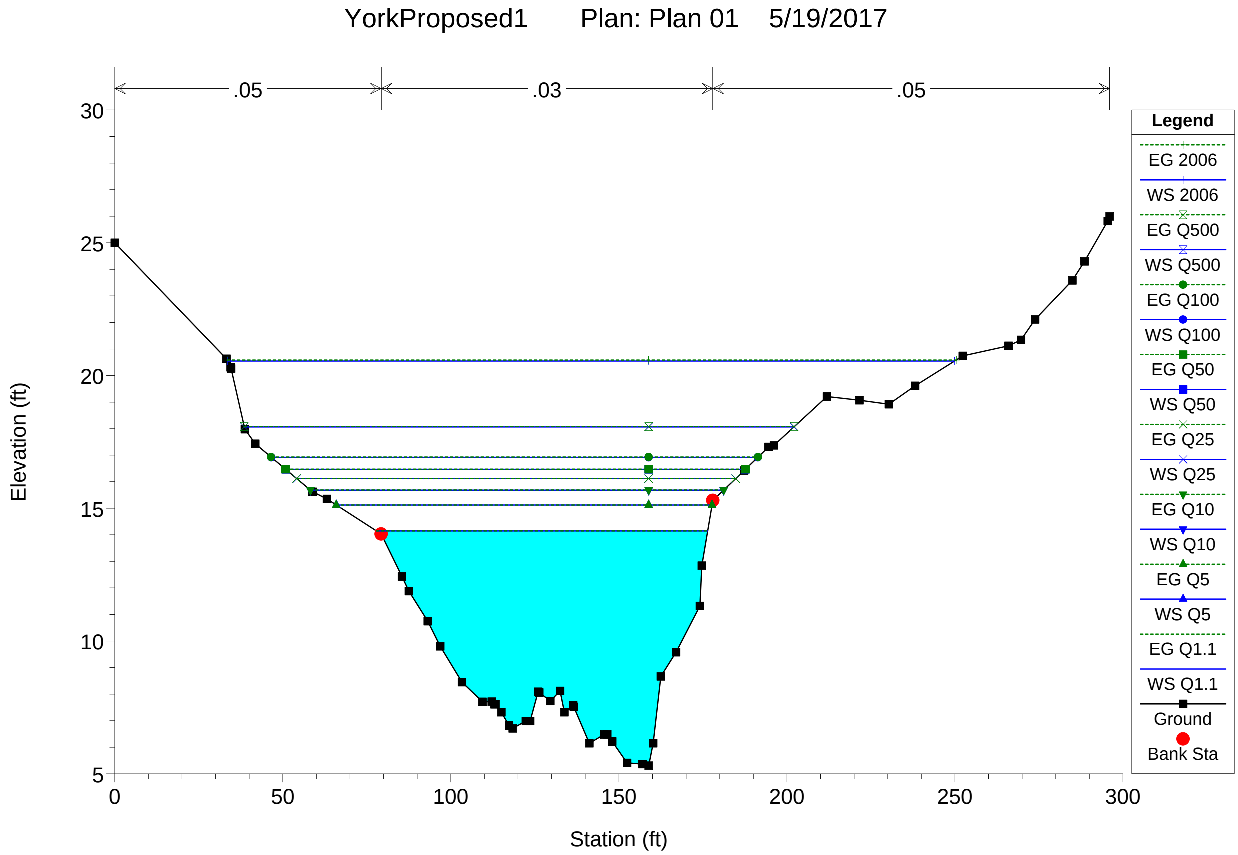
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta

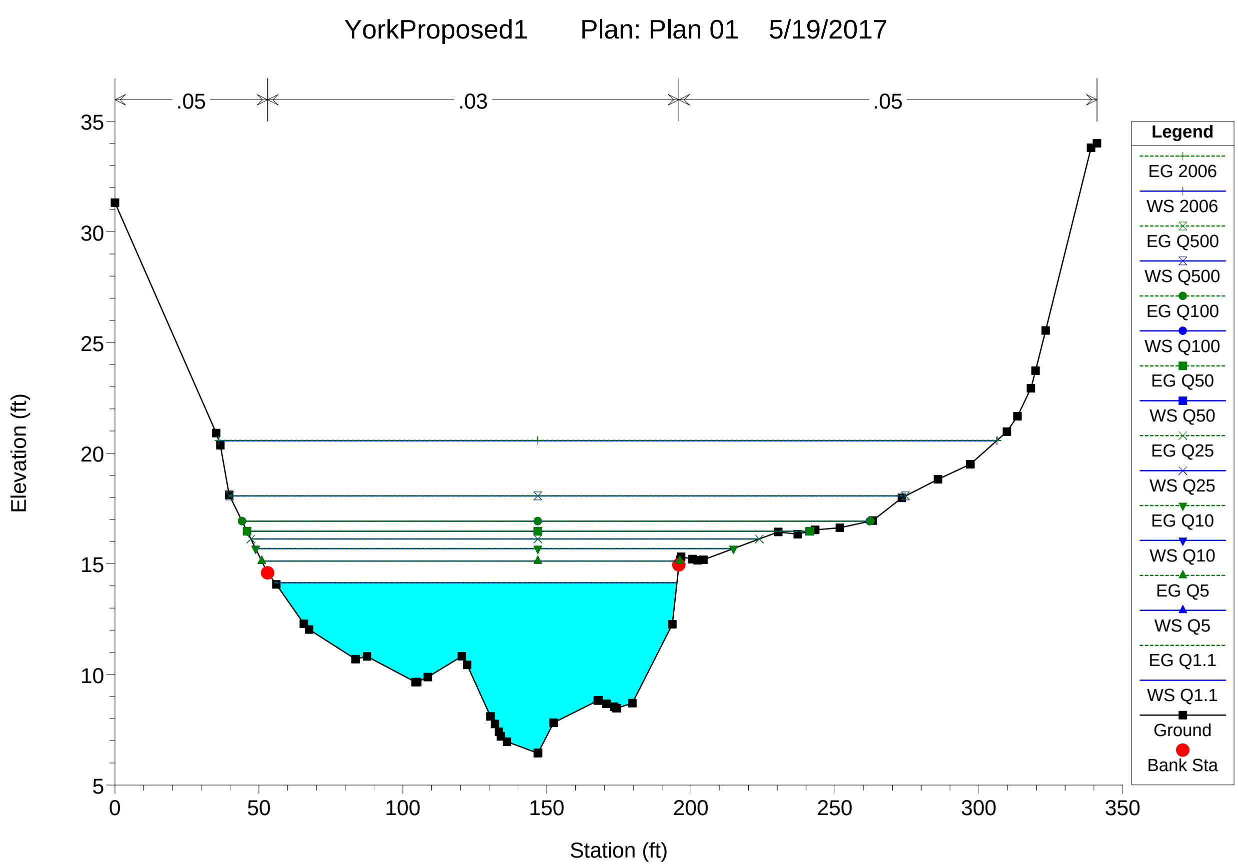

YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta


YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta

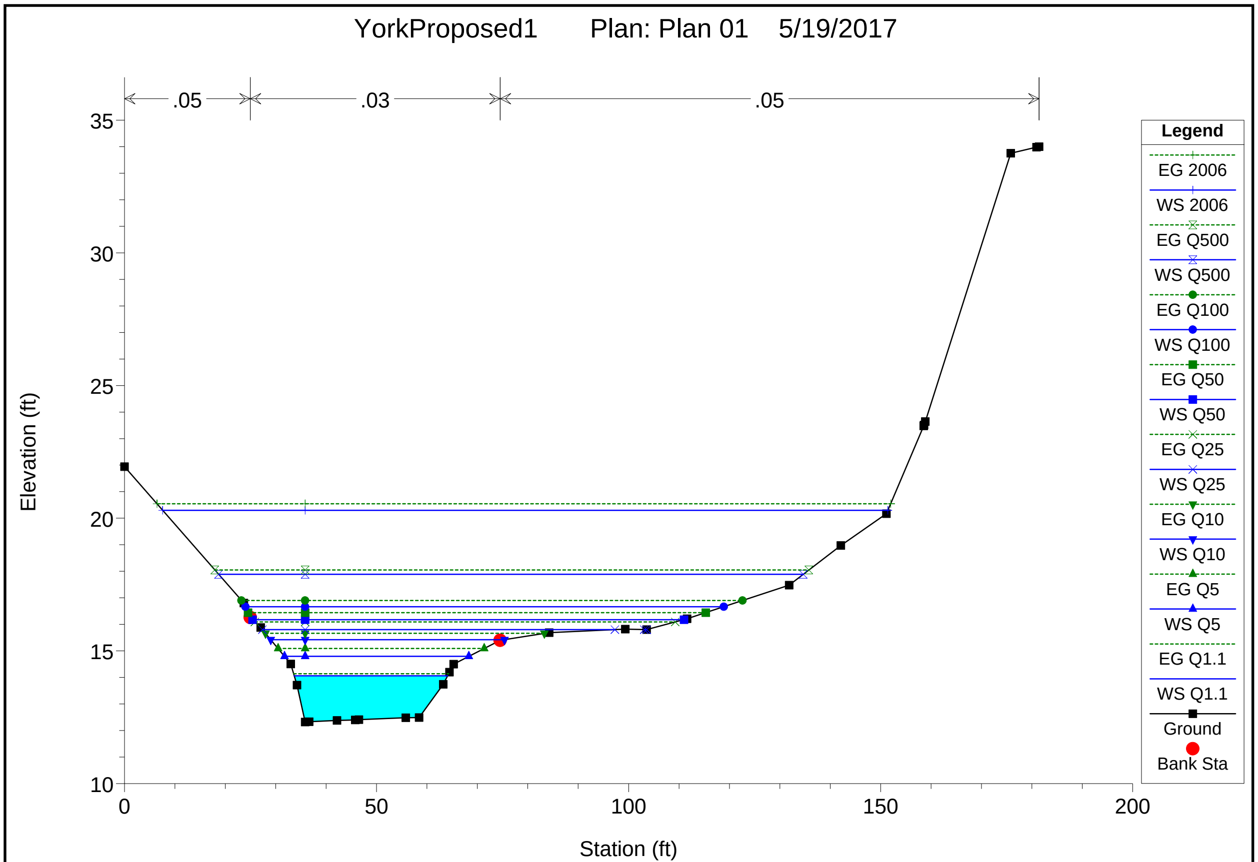

YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta

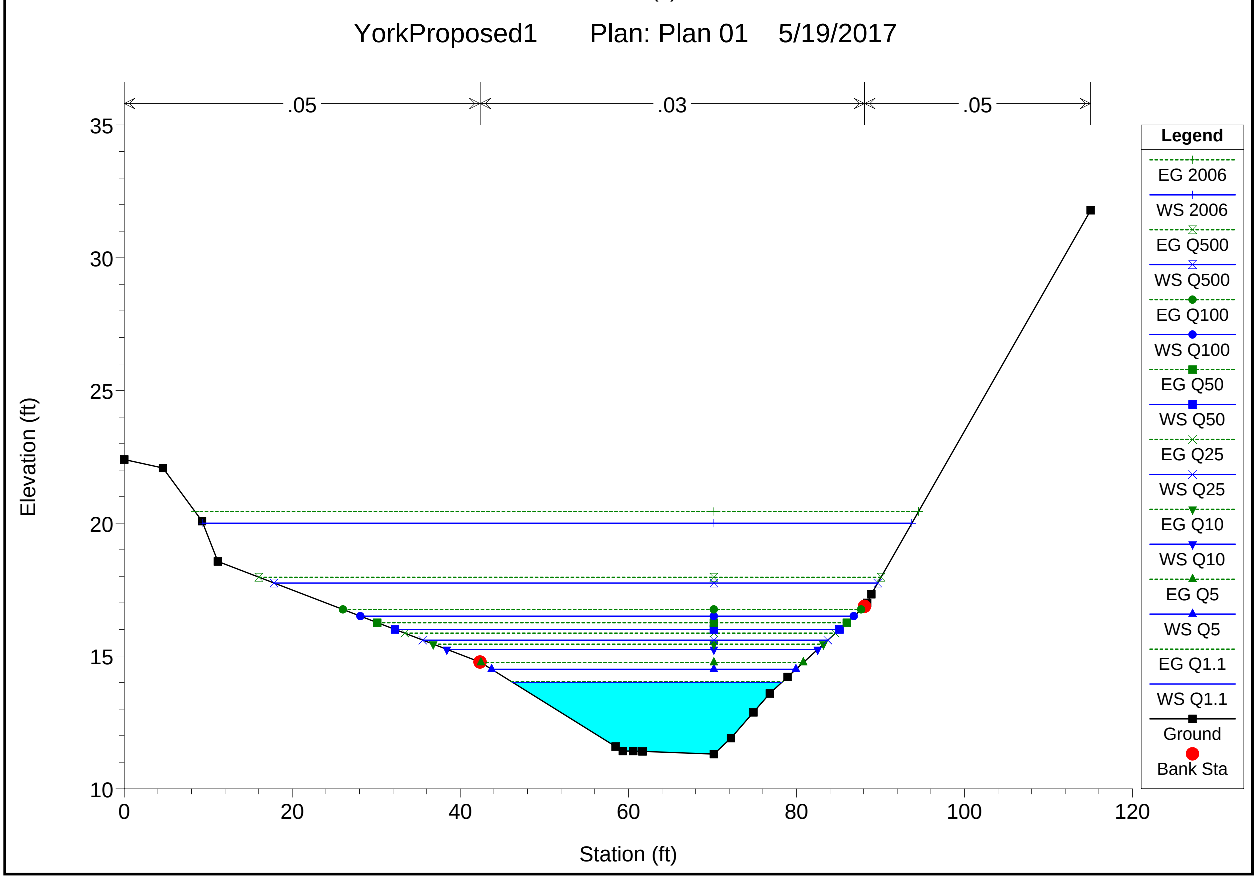


YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta



YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta


YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
0
50
100
150
200
10
15
20
25
30
35
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta


YorkProposed1 Plan: Plan 01 5/19/2017
.05
.03
.05
Elevation (ft)
Station (ft)
0
20
40
60
80
100
120
10
15
20
25
30
35
Legend
EG 2006
WS 2006
EG Q500
WS Q500
EG Q100
WS Q100
EG Q50
WS Q50
EG Q25
WS Q25
EG Q10
WS Q10
EG Q5
WS Q5
EG Q1.1
WS Q1.1
Ground
Bank Sta

HEC-RAS Plan: Plan 01   River: Stream   Reach: Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	529.1589	Q1.1	94.70	12.70	15.57		15.63	0.000656	1.94	48.76	24.51	0.24
Reach	529.1589	Q5	295.40	12.70	17.08		17.19	0.000713	2.82	147.78	90.47	0.27
Reach	529.1589	Q10	365.10	12.70	17.50		17.60	0.000644	2.90	187.41	100.54	0.27
Reach	529.1589	Q25	479.90	12.70	18.10		18.21	0.000565	3.00	252.01	114.31	0.26
Reach	529.1589	Q50	549.60	12.70	18.44		18.55	0.000523	3.04	292.59	122.16	0.25
Reach	529.1589	Q100	644.10	12.70	18.88		18.99	0.000467	3.05	348.18	128.09	0.24
Reach	529.1589	Q500	852.70	12.70	19.85		19.94	0.000361	3.01	476.12	137.59	0.22
Reach	529.1589	2006	2000.00	12.70	24.33		24.41	0.000166	2.96	1176.71	171.63	0.16
Reach	502.8161	Q1.1	94.70	12.29	15.59		15.61	0.000207	1.03	92.38	52.04	0.14
Reach	502.8161	Q5	295.40	12.29	17.12		17.16	0.000279	1.65	179.55	62.82	0.17
Reach	502.8161	Q10	365.10	12.29	17.53		17.58	0.000281	1.78	206.43	68.18	0.17
Reach	502.8161	Q25	479.90	12.29	18.12		18.18	0.000276	1.97	252.90	96.98	0.18
Reach	502.8161	Q50	549.60	12.29	18.46		18.53	0.000268	2.05	287.83	108.17	0.18
Reach	502.8161	Q100	644.10	12.29	18.90		18.97	0.000256	2.14	337.91	120.22	0.18
Reach	502.8161	Q500	852.70	12.29	19.85		19.93	0.000219	2.24	457.33	130.08	0.17
Reach	502.8161	2006	2000.00	12.29	24.33		24.40	0.000119	2.46	1094.54	152.17	0.14
Reach	482.7911	Q1.1	94.70	11.34	15.59		15.60	0.000095	0.84	112.66	47.66	0.10
Reach	482.7911	Q5	295.40	11.34	17.12		17.15	0.000210	1.48	199.35	65.68	0.15
Reach	482.7911	Q10	365.10	11.34	17.53		17.57	0.000226	1.61	227.26	70.21	0.16
Reach	482.7911	Q25	479.90	11.34	18.12		18.17	0.000245	1.77	271.05	76.84	0.17
Reach	482.7911	Q50	549.60	11.34	18.46		18.52	0.000252	1.84	297.92	81.22	0.17
Reach	482.7911	Q100	644.10	11.34	18.90		18.96	0.000256	1.92	334.62	86.52	0.17
Reach	482.7911	Q500	852.70	11.34	19.86		19.92	0.000235	2.02	429.23	121.97	0.17
Reach	482.7911	2006	2000.00	11.34	24.33		24.40	0.000111	2.21	1021.05	142.71	0.13
Reach	461.0650	Q1.1	94.70	12.46	15.56		15.60	0.000448	1.55	61.13	33.00	0.20
Reach	461.0650	Q5	295.40	12.46	17.05		17.14	0.000728	2.40	122.86	49.56	0.27
Reach	461.0650	Q10	365.10	12.46	17.45		17.55	0.000718	2.54	143.57	52.64	0.27
Reach	461.0650	Q25	479.90	12.46	18.04		18.16	0.000673	2.74	175.56	55.75	0.27
Reach	461.0650	Q50	549.60	12.46	18.38		18.50	0.000635	2.83	195.07	62.60	0.27
Reach	461.0650	Q100	644.10	12.46	18.81		18.94	0.000590	2.95	223.22	67.36	0.26
Reach	461.0650	Q500	852.70	12.46	19.75		19.90	0.000493	3.11	291.96	77.20	0.25
Reach	461.0650	2006	2000.00	12.46	24.22		24.39	0.000256	3.45	718.26	111.76	0.20
Reach	450.8967	Q1.1	94.70	13.77	15.28	15.02	15.56	0.006604	4.29	22.06	19.51	0.71
Reach	450.8967	Q5	295.40	13.77	16.51	16.10	17.07	0.006128	6.03	49.00	24.15	0.75

HEC-RAS Plan: Plan 01   River: Stream   Reach: Reach (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	450.8967	Q10	365.10	13.77	16.87	16.39	17.49	0.005794	6.30	57.93	25.47	0.74
Reach	450.8967	Q25	479.90	13.77	17.39	16.81	18.08	0.005490	6.70	71.57	27.36	0.73
Reach	450.8967	Q50	549.60	13.77	17.70	17.05	18.43	0.005197	6.84	80.35	28.52	0.72
Reach	450.8967	Q100	644.10	13.77	18.12	17.35	18.87	0.004878	6.95	92.67	30.58	0.70
Reach	450.8967	Q500	852.70	13.77	19.13	17.95	19.84	0.003829	6.76	126.12	36.17	0.64
Reach	450.8967	2006	2000.00	13.77	23.79	20.34	24.34	0.001166	5.96	335.76	66.02	0.39
Reach	412.5		Bridge									
Reach	389.1057	Q1.1	94.70	12.00	14.15		14.18	0.000379	1.47	64.47	30.00	0.18
Reach	389.1057	Q5	295.40	12.00	15.09		15.25	0.001181	3.19	92.69	30.00	0.32
Reach	389.1057	Q10	365.10	12.00	15.61		15.79	0.001111	3.37	108.42	30.00	0.31
Reach	389.1057	Q25	479.90	12.00	16.01		16.26	0.001395	3.99	120.33	30.00	0.35
Reach	389.1057	Q50	549.60	12.00	16.34		16.62	0.001438	4.22	130.26	30.00	0.36
Reach	389.1057	Q100	644.10	12.00	16.77		17.09	0.001483	4.50	143.19	30.00	0.36
Reach	389.1057	Q500	852.70	12.00	17.88		18.24	0.001394	4.83	176.45	30.00	0.35
Reach	389.1057	2006	2000.00	12.00	19.99		21.07	0.003144	8.35	239.59	30.00	0.52
Reach	360.8755	Q1.1	94.70	11.36	14.11		14.16	0.000563	1.80	52.75	26.60	0.22
Reach	360.8755	Q5	295.40	11.36	15.01		15.20	0.001945	3.47	85.14	40.64	0.42
Reach	360.8755	Q10	365.10	11.36	15.58		15.75	0.001419	3.35	109.12	43.31	0.37
Reach	360.8755	Q25	479.90	11.36	15.98		16.20	0.001543	3.78	126.86	44.88	0.39
Reach	360.8755	Q50	549.60	11.36	16.32		16.56	0.001396	3.87	142.46	46.15	0.38
Reach	360.8755	Q100	644.10	11.36	16.77		17.01	0.001236	3.97	163.36	47.78	0.37
Reach	360.8755	Q500	852.70	11.36	17.91		18.15	0.000860	3.97	220.27	51.97	0.32
Reach	360.8755	2006	2000.00	11.36	20.18		20.76	0.001237	6.18	348.18	60.48	0.41
Reach	338.8639	Q1.1	94.70	10.01	14.14		14.15	0.000117	0.83	113.69	57.50	0.10
Reach	338.8639	Q5	295.40	10.01	15.09		15.14	0.000376	1.71	172.76	70.99	0.19
Reach	338.8639	Q10	365.10	10.01	15.66		15.70	0.000293	1.71	214.93	77.97	0.18
Reach	338.8639	Q25	479.90	10.01	16.08		16.14	0.000326	1.96	249.13	82.69	0.19
Reach	338.8639	Q50	549.60	10.01	16.43		16.49	0.000310	2.02	278.20	84.46	0.19
Reach	338.8639	Q100	644.10	10.01	16.88		16.95	0.000290	2.09	316.85	86.75	0.18
Reach	338.8639	Q500	852.70	10.01	18.02		18.09	0.000225	2.13	419.20	92.59	0.17
Reach	338.8639	2006	2000.00	10.01	20.47		20.62	0.000309	3.21	697.44	125.96	0.21
Reach	308.7850	Q1.1	94.70	5.31	14.14		14.14	0.000001	0.17	549.36	98.57	0.01
Reach	308.7850	Q5	295.40	5.31	15.12		15.12	0.000007	0.46	652.08	111.80	0.03

HEC-RAS Plan: Plan 01 River: Stream Reach: Reach (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach	308.7850	Q10	365.10	5.31	15.69		15.69	0.000009	0.52	718.07	122.92	0.03
Reach	308.7850	Q25	479.90	5.31	16.12		16.12	0.000012	0.64	772.93	130.57	0.04
Reach	308.7850	Q50	549.60	5.31	16.47		16.48	0.000014	0.70	819.65	136.76	0.04
Reach	308.7850	Q100	644.10	5.31	16.92		16.93	0.000015	0.77	883.47	144.76	0.05
Reach	308.7850	Q500	852.70	5.31	18.06		18.07	0.000017	0.89	1060.18	163.51	0.05
Reach	308.7850	2006	2000.00	5.31	20.55		20.58	0.000039	1.57	1534.05	216.37	0.08
Reach	252.0109	Q1.1	94.70	6.44	14.14		14.14	0.000001	0.15	611.25	139.57	0.01
Reach	252.0109	Q5	295.40	6.44	15.12		15.12	0.000007	0.39	750.48	145.19	0.03
Reach	252.0109	Q10	365.10	6.44	15.69		15.69	0.000008	0.44	839.53	165.95	0.03
Reach	252.0109	Q25	479.90	6.44	16.12		16.12	0.000010	0.54	913.70	176.53	0.04
Reach	252.0109	Q50	549.60	6.44	16.47		16.47	0.000011	0.58	977.73	195.33	0.04
Reach	252.0109	Q100	644.10	6.44	16.92		16.93	0.000012	0.63	1072.04	218.02	0.04
Reach	252.0109	Q500	852.70	6.44	18.06		18.07	0.000013	0.71	1330.60	234.68	0.04
Reach	252.0109	2006	2000.00	6.44	20.56		20.58	0.000026	1.21	1964.64	270.12	0.07
Reach	105.7665	Q1.1	94.70	12.32	14.06		14.14	0.001217	2.16	43.79	30.43	0.32
Reach	105.7665	Q5	295.40	12.32	14.79		15.09	0.003575	4.37	67.56	36.56	0.57
Reach	105.7665	Q10	365.10	12.32	15.43		15.66	0.002459	3.90	93.58	46.38	0.48
Reach	105.7665	Q25	479.90	12.32	15.80		16.09	0.002503	4.31	114.29	70.44	0.50
Reach	105.7665	Q50	549.60	12.32	16.18		16.44	0.002017	4.16	144.69	85.60	0.45
Reach	105.7665	Q100	644.10	12.32	16.67		16.90	0.001478	3.97	188.85	94.91	0.40
Reach	105.7665	Q500	852.70	12.32	17.89		18.05	0.000714	3.45	318.92	115.99	0.29
Reach	105.7665	2006	2000.00	12.32	20.30		20.54	0.000668	4.48	632.54	143.89	0.30
Reach	-0.0000	Q1.1	94.70	11.31	14.00	12.51	14.04	0.000559	1.68	56.27	31.99	0.22
Reach	-0.0000	Q5	295.40	11.31	14.50	13.53	14.75	0.002663	4.03	73.32	36.24	0.50
Reach	-0.0000	Q10	365.10	11.31	15.25	13.79	15.45	0.001537	3.56	103.26	44.14	0.39
Reach	-0.0000	Q25	479.90	11.31	15.60	14.17	15.86	0.001777	4.10	119.43	48.23	0.43
Reach	-0.0000	Q50	549.60	11.31	16.00	14.37	16.26	0.001538	4.08	139.65	52.90	0.41
Reach	-0.0000	Q100	644.10	11.31	16.50	14.63	16.75	0.001320	4.07	167.56	58.74	0.38
Reach	-0.0000	Q500	852.70	11.31	17.75	15.11	17.97	0.000799	3.83	249.47	71.86	0.31
Reach	-0.0000	2006	2000.00	11.31	20.00	17.04	20.44	0.001022	5.63	429.32	84.38	0.38

APPENDIX F

Preliminary Plans

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

SPECIFICATIONS

Design: Load and Resistance Factor Design per AASHTO LRFD Bridge Design Specifications, Eighth Edition 2017.

DESIGN LOADING

Live Load	HL - 93 Modified for Strength 1

TRAFFIC DATA

Current (2018) AADT	14,180
Future (2038) AADT	14,890
DHV - % of AADT	11%
Design Hour Volume	164
Heavy Trucks (% of AADT)	4%
Heavy Trucks (% of DHV)	3%
Directional Distribution (% of DHV)	54%
18 kip Equivalent P 2.0	281
18 kip Equivalent P 2.5	268
Design Speed (mph)	35

HYDROLOGIC DATA

Drainage Area	8.36 sq mi
Discharge (Q25)	479.9 cfs
Design Discharge (Q50)	549.6 cfs
Check Discharge (Q100)	644.1 cfs
Headwater Elevation (Q1.1)	15.03 ft
Headwater Elevation (Q25)	16.81 ft
Headwater Elevation (Q50)	17.02 ft
Headwater Elevation (Q100)	17.28 ft
Discharge Velocity (Q1.1)	1.57 fps
Discharge Velocity (Q25)	4.42 fps
Discharge Velocity (Q50)	4.70 fps
Discharge Velocity (Q100)	5.05 fps

MATERIALS

Concrete:	
Precast	Class "P"
All Other	Class "A"
Reinforcing Steel	ASTM A 615, Grade 60

BASIC DESIGN STRESSES

Concrete, Class "A"	f 'c = 4,000 psi
Concrete, Class "P"	f 'c = 6,000 psi
Reinforcing Steel	f y = 60,000 psi

YORK
YORK COUNTY
CAPE NEDDICK BRIDGE
OVER
CAPE NEDDICK RIVER

U.S. ROUTE 1
FEDERAL AID PROJECT NO. STP-2170(900)
PROJECT LENGTH 0.145 mi.
BRIDGE NO. 2127

LIST OF DRAWINGS

UTILITIES

Central Maine Power
Consolidated Communications
Spectrum Cable
York Water District

MAINTENANCE OF TRAFFIC

Maintain alternating one-way traffic controlled by temporary traffic signals during phase 1 and maintain two lanes of traffic during phase 2.

PROJECT LOCATION:	US Route 1 over Cape Neddick River located 0.12 mile north of Route 1A, Latitude 43°11'33" N, Longitude 70°37'11" W
PROGRAM AREA:	Bridge
OUTLINE OF WORK:	Bridge Replacement with 500' of Approach Work

HNTB

STATE OF MAINE DEPARTMENT OF TRANSPORTATION
APPROVED | DATE
COMMISSIONER:
CHIEF ENGINEER:

STATE OF MAINE ROLAND A. LAVALLEE No. 6452 LICENSED PROFESSIONAL ENGINEER

SIGNATURE
6452
P.E. NUMBER
06/29/18
DATE

PROJECT INFORMATION

PROGRAM	BRIDGE
PROJECT MANAGER	DEVAN EATON, P.E.
DESIGNER	TIMOTHY COTE, P.E.
CONSULTANT	HNTB
PROJECT RESIDENT	
CONTRACTOR	
PROJECT COMPLETION DATE	

WIN 21709.00

STP-2170(900)X

YORK
CAPE NEDDICK BRIDGE
TITLE SHEET

SHEET NUMBER
1
OF 29

Date:6/29/2018
Username:
Division:
Filename:001_Title.dgn

ITEM NO.	DESCRIPTION	QUANTITY	UNIT
202.19	Removing Existing Bridge (Concrete = 530 CY)	1	LS
202.202	Removing Pavement Surface	660	SY
203.20	Common Excavation	1,250	CY
203.24	Common Borrow	50	CY
304.10	Aggregate Subbase Course - Gravel	1,300	CY
403.2081	Hot Mix Asphalt, 12.5 mm Nominal Maximum Size (Polymer Modified)	210	TON
403.209	Hot Mix Asphalt, 9.5 mm Nominal Maximum Size (Sidewalks, Drives, & Incidentals)	6	TON
403.213	Hot Mix Asphalt, 12.5 mm Nominal Maximum Size (Base and Intermediate Base course)	180	TON
403.2131	Hot Mix Asphalt, 12.5 mm Nominal Maximum Size (Base and Intermediate course, Polymer Modified)	150	TON
409.15	Bituminous Tack Coat, Applied	140	GAL
411.09	Untreated Aggregate Surface Course	2.5	CY
461.131	Temporary Pavement	32	TON
514.06	Curing Box for Concrete Cylinders	1	EA
524.301	Temporary Structural Support	1	LS
526.305	Temporary Concrete Barrier, Braced Type I (640 LF)	1	LS
527.34	Work Zone Crash Cushions	3	UN
531.51	Bridge Structure Detail-Build	1	LS
606.1301	31" W-Beam Guardrail - Mid-Way Splice (Steel Post, 8" Offset Blocks, Single Faced)	325	LF
606.1304	31" W-Beam Guardrail - Mid-Way Splice (Steel Post, 8" Offset Blocks, Over 15' Radius)	25	LF
606.1306	31" W-Beam Guardrail - Mid-Way Splice Tangent Terminal (31" Height)	3	EA
606.2591	Anchorage Assembly for Driveway Radius	1	EA
606.353	Reflectorized Flexible Guardrail Marker	7	EA
609.31	Curb Type 3	380	LF
610.08	Plain Riprap	10	CY
610.18	Stone Ditch Protection	40	CY
613.319	Erosion Control Blanket	32	SY
615.07	Loam	46	CY
618.14	Seeding Method Number 2	4	UN
619.12	Mulch	4	UN
619.14	Erosion Control Mix	46	CY
620.58	Erosion Control Geotextile	130	SY
627.733	4" White or Yellow Painted Pavement Marking Line	2,600	LF
627.77	Removing Existing Pavement Marking	1,010	SF
627.78	Temporary 4" Painted Pavement Marking Line, White or Yellow	3,450	LF
629.05	Hand Labor, Straight Time	35	HR
631.12	All Purpose Excavator (including operator)	10	HR
631.172	Truck-large (including operator)	10	HR
637.071	Dust Control	1	LS
639.19	Field Office, Type B	1	EA
643.72	Temporary Traffic Signal: Route 1	1	LS
652.312	Type III Barricades	2	EA
652.33	Drum	25	EA
652.34	Cone	25	EA
652.35	Construction Signs	250	SF
652.361	Maintenance of Traffic Control Devices	1	LS
652.38	Flaggers	240	HR
652.41	Portable Changeable Message Sign	2	EA
656.75	Temporary Soil Erosion and Water Pollution Control	1	LS
659.10	Mobilization	1	LS
830.10	Water Main Bridge Crossing, Install Only	1	LS
822.3701	16" Ductile Iron Water Main, Install Only	200	LF

Filename: 002_Estimated Quantites.dgn Division: Username: Date:6/29/2018

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

BRIDGE NO. 2127 WIN 21709.00 BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Forgacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-		-
REVISIONS 2	-		-
REVISIONS 3	-		-
REVISIONS 4	-		-
FIELD CHANGES	-		-

SIGNATURE

P.E. NUMBER

DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK YORK COUNTY

ESTIMATED QUANTITIES

HNTB

SHEET NUMBER
2
OF 29

GENERAL NOTES

1. The clearing limits shall be 5' beyond and parallel to the construction slope lines or as shown on the plans unless otherwise authorized by the Resident. The exact limits will be established in the field by the Resident. Payment for clearing will be considered incidental to Contract Items. Single tree and stump removal shall be considered clearing.

2. All utility facilities shall be adjusted by the respective utilities unless otherwise noted.

3. Place loam 2 inches deep on all new or reconstructed side slopes or as directed by the Resident.

4. A MASH compliant guardrail end treatment shall be installed concurrently with the placement of each section of end beam guardrail.

5. Project info referred to below may be accessed at the following MaineDOT web address: http://www.maine.gov/contractors/

6. The existing bridge plans may be accessed at the MaineDOT web address. The plans are reproductions of the original drawings as prepared for the construction of the bridge. It is very unlikely that the plans will show any construction field changes or any alterations which may have been made to the bridge during its life span.

7. The Project Geotechnical Report titled: "Geotechnical Design Report Replacement of Cape Neddick Bridge," may be accessed at the MaineDOT web address.

8. Geotechnical information furnished or referred to in this plan set for the use of Bidders and the Contractor. No assurance is given that the information or interpretations will be representative of actual subsurface conditions at the construction site. MaineDOT will not be responsible for the Bidder's or Contractor's interpretations of, or conclusions drawn from, the geotechnical information. The boring logs contained in the plan set present factual and interpretive subsurface information collected at discrete locations. Data provided may not be representative of the subsurface conditions between boring locations.

9. Quantities included for pay items measured and paid for by Lump Sum are estimated quantities and are provided by MaineDOT for informational purposes only. Lump Sum pay items will be paid for at the Contract Bid amount, with no addition or reduction in payment to the Contractor if the actual final quantities are different from the MaineDOT provided estimated quantities, except as follows:

   a. If a Lump Sum pay item is eliminated, the requirements of Standard Specifications Section 109.2, Elimination of Items, will take precedence.

   b. If other Contract Documents specifically allow a change in payment for a Lump Sum pay item, those requirements will be followed.

   c. If a design change results in changes to estimated quantities for Lump Sum pay items, price adjustments will be made in accordance with Standard Specifications Section 109.7, Equitable Adjustments to Compensation and Time.

10. The Contractor shall submit a Bridge Removal Plan to the Resident at least 10 business days prior to the start of demolition work. The plan shall outline the methods and equipment to be used to remove and dispose of all materials included in the existing bridge. No work related to the removal of the bridge shall be undertaken by the Contractor until MaineDOT has reviewed the Bridge Removal Plan for appropriateness and completeness. Payment for all work necessary for developing, submitting and finalizing the demolition Plan will be considered incidental to the bridge removal pay item.

11. For easements, construction limits and right of way lines, refer to the Right of Way Map.

12. All embankment material, except as otherwise shown, placed below El. 15.03 shall be Granular Borrow meeting the requirements of Subsection 703.19, Material for Underwater Backfill.

13. The hydrologic report of the bridge site may be accessed at the MaineDOT web address. The hydrologic report is based on the designers interpretation of the information obtained for the subject site. No assurance is given that the information or the conclusions of the report will be representative of actual conditions at the time of construction.

14. Location of utilities shown are approximate and should be verified in the field by the Contractor.

15. All waste material not used on the project shall be disposed of off the project in a waste area. The Contractor shall have a written agreement with the waste area owner, a copy of which shall be provided to the Department. All work associated with this shall be considered incidental to related Contract items.

16. No existing drainage shall be abandoned, removed or plugged without prior approval of the Resident. Existing drainage called to be abandoned, removed, or plugged shall be incidental to related Contract items.

17. Extended Use Erosion Control Blanket, seeded gutters, riprap downspouts and other gutters lined with Stone Ditch Protection shall be constructed after paving and shoulder work is completed where it is apparent that runoff will cause continual erosion. Payment will be made under the appropriate Contract items.

18. Erosion Control Mix may be substituted in those area normally receiving loam and seed as directed by the Resident. Placement shall be in accordance with Standard Specification Section 619, Mulch. Payment will be made under Item No. 619.14, Erosion Control Mix.

19. Where called for on the Plans, where new pavement joins existing pavement, the existing pavement shall be sawcut along a smooth line to a neat, even, vertical joint, as directed by the Resident. Broken or raveled edges will not be permitted. All work necessary for the preparation of this joint will be considered incidental to the related Contract items.

20. Do not excavate for Aggregate Subbase Course where existing material is suitable as determined by the Resident.

21. In areas where the Resident directs the Contractor not to excavate to the subgrade line shown on the Plans, payment for removing existing pavement, grubbing, shaping, ditching, and compacting the existing subbase and layers of new subbase 6 inches or less thick will be made under appropriate equipment rental items.

22. Place a 24-in wide strip of Temporary Erosion Control Blanket on the sideslopes along the top of riprap and behind the wingwalls.

23. Unpaved entrances shall be constructed with 14" Aggregate Subbase Course Gravel or 11" Aggregate Subbase Course Gravel and 3" Untreated Aggregate Surface Course unless otherwise noted on the Plans or directed by the Resident.

24. Residential paved entrances shall be constructed with: 2" hot mix asphalt and 12" Aggregate Subbase Course Gravel.

25. A 3' paved lip shall be placed at all gravel entrances unless otherwise noted on the Plans or directed by the Resident.

26. Existing signs within the Project Limits shall be removed and reset as directed by the Resident. Payment for removal and reinstallation of existing signs will be considered incidental to the Contract. No separate payment will be made.

27. Protective Coating for Concrete Surfaces shall be applied to the following areas:

   On all concrete headwalls and concrete wall surfaces that are exposed and to limit lines, one foot beyond intersections of concrete surfaces with the ground.

28. Bidders and Contractors should anticipate encountering boulders and cobbles in the excavations for the proposed detailed-build structure and temporary structural support. Any additional work shall be incidental to the related Contract pay items.

HNTB

Filename: 003_GenNotes.dgn
Division:
Username:
Date:6/29/2018

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2170(900)X
BRIDGE NO. 2127
WIN 21709.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Fargacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

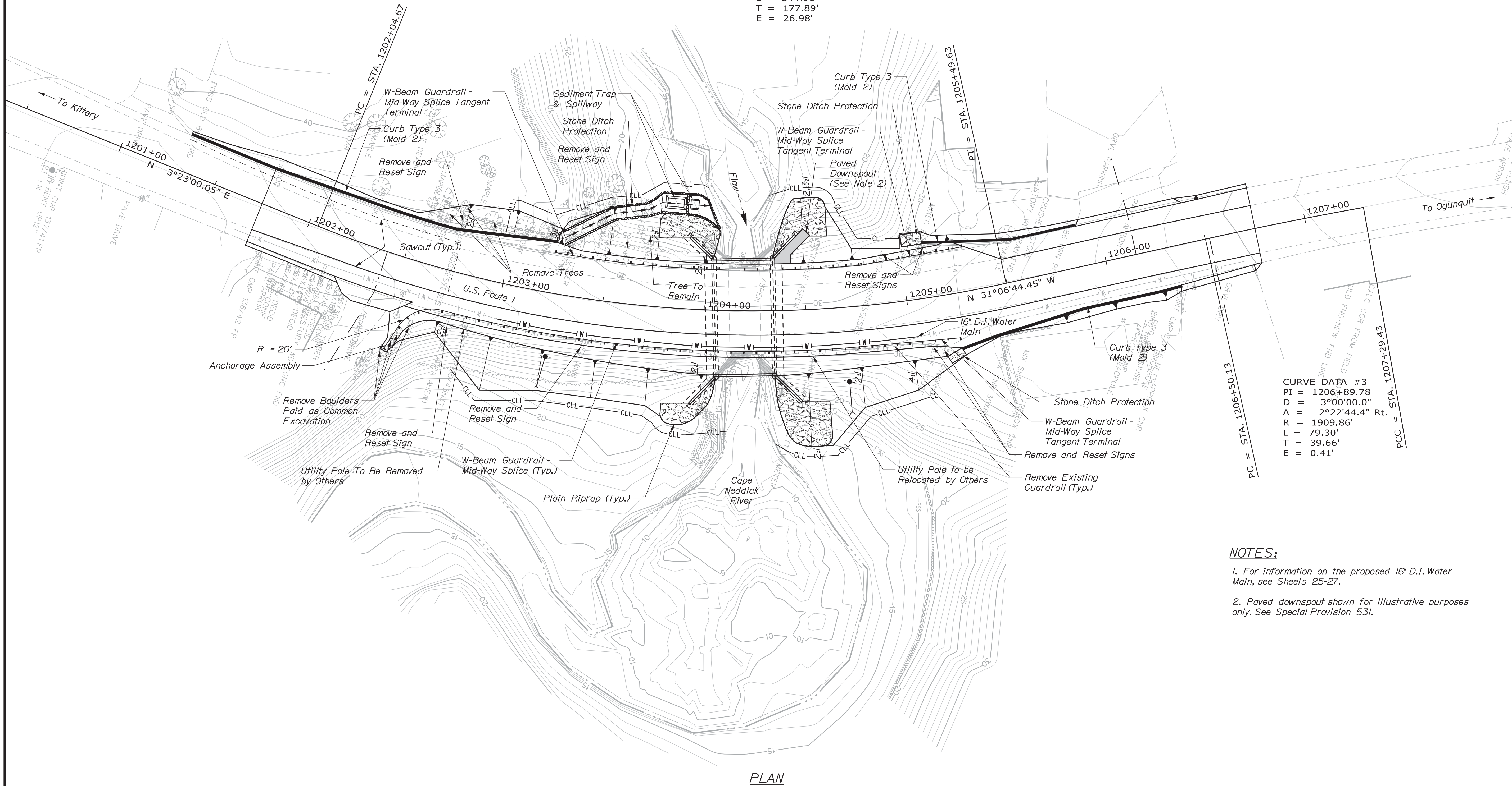
DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK
YORK COUNTY

GENERAL NOTES

SHEET NUMBER
3
OF 29

CURVE DATA #2
PI = 1203+82.55
D = 9°59'59.9"
Δ = 34°29'44.5" Lt.
R = 572.96'
L = 344.96'
T = 177.89'
E = 26.98'



CURVE DATA #3
PI = 1206+89.78
D = 3°00'00.0"
Δ = 2°22'44.4" Rt.
R = 1909.86'
L = 79.30'
T = 39.66'
E = 0.41'

NOTES:

1. For information on the proposed 16" D.I. Water Main, see Sheets 25-27.

2. Paved downspout shown for illustrative purposes only. See Special Provision 531.

PLAN

25 0 25 50
Scale of Feet

HNTB

Date:6/29/2018
Username:
Division:
Filename: 004_Plan.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

WIN 21709.00

BRIDGE NO. 2127

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	Devon Eaton	
DESIGN-DETAILED	E. Fargacs	6\18
CHECKED-REVIEWED	L. Driscoll	6\18
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1	C. Helmick	6\18
REVISIONS 2	R. Hanf	6\18
REVISIONS 3	-	-
REVISIONS 4	-	-
FIELD CHANGES	-	-

SIGNATURE

P.E. NUMBER

DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK YORK COUNTY

GENERAL PLAN

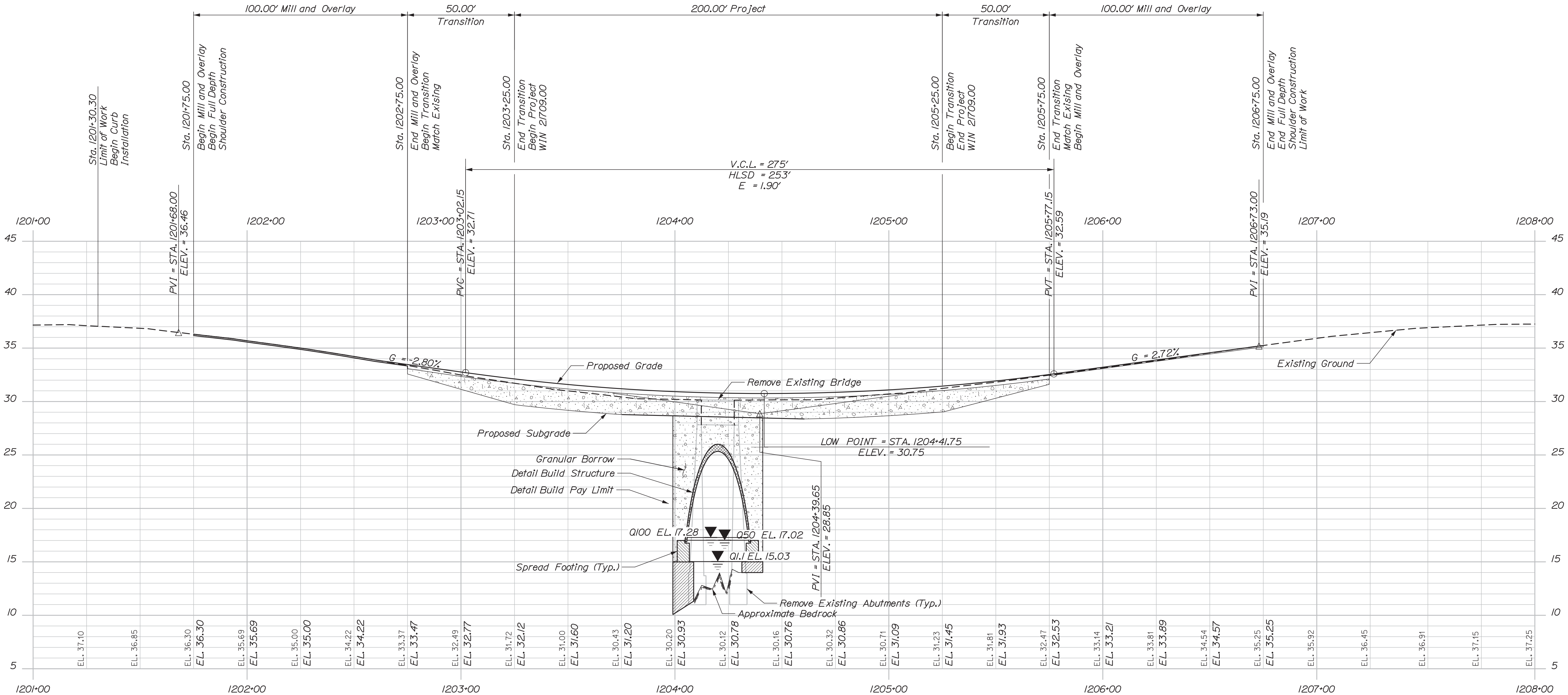
SHEET NUMBER
4
OF 29

Date:6/29/2018

Username:

Division:

Filename: 005_Profile.dgn



PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

Scale of Feet

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

BRIDGE PLANS

WIN 21709.00

BRIDGE NO. 2127

	DATE	BY	
PROJ. MANAGER		Devon Eaton	
DESIGN-DETAILED	6\18	C. Helmick	E. Fargacs
CHECKED-REVIEWED	6\18	R. Hanf	L. Driscoll
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

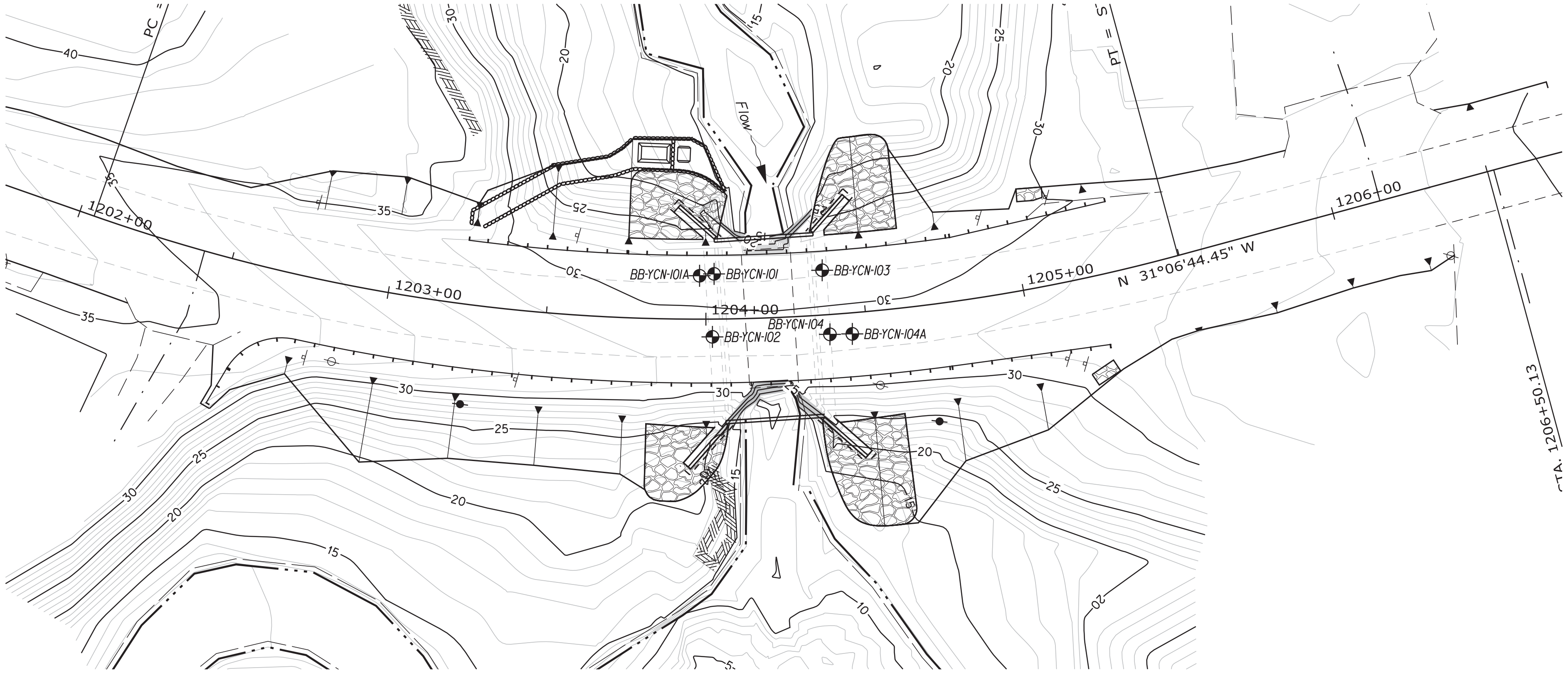
CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

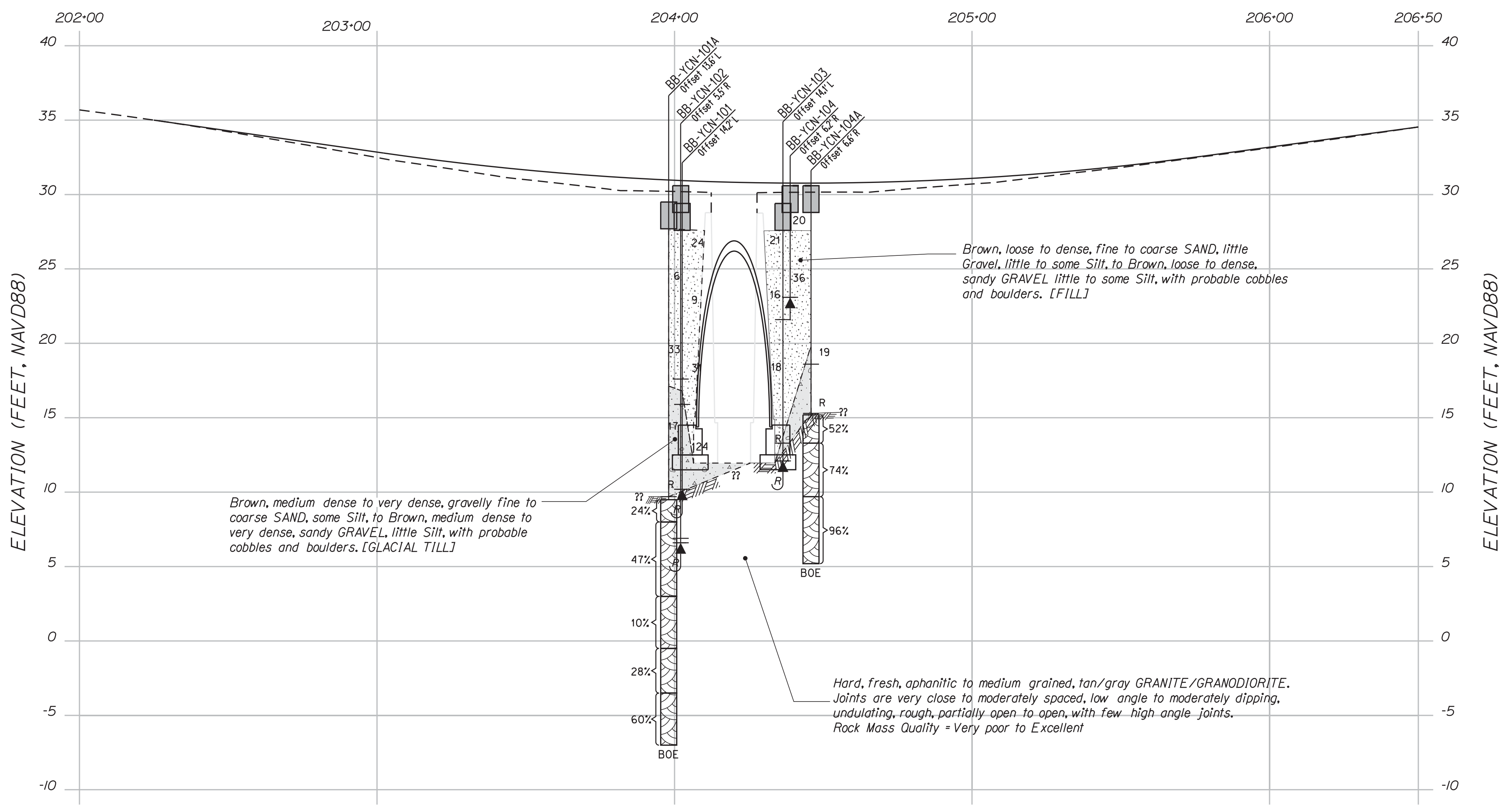
PROFILE

SHEET NUMBER

5

OF 29





NOTES

1) Base map developed from electronic files provided by MaineDOT on March 15, 2017 (Files included BDPLAN.dgn, Contours.dgn, Points.dgn, RWPLAN.dgn, Text.dgn, topo.dgn and wetlands.dgn).

2) Profile developed from electronic files provided by HNTB on March 17, 2016 (Files included Profile.dgn and z_Bridge Elevation.dgn).

3) The as-drilled locations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA except for BB-YCN-104 which was determined using offset from BB-YCN-104A.

4) BB-YCN-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between March 6 and March 8, 2017.

5) Interpreted top of rock considers general trend of ledge lines between borings BB-YCN-101A and -104A.

6) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

BORING LOCATION PLAN LEGEND

BB-YCN-104 — Location and designation of cased wash boring

INTERPRETIVE SUBSURFACE PROFILE LEGEND

- Interpreted Top of Bedrock (See Note 5)
- Boring No., Offset, if shown
- Pavement Thickness if applicable
- 6 — Energy-Corrected SPT N60 Value (blows/foot)
- WOH — Indicates weight of hammer
- R — Split Spoon Refusal (>50 blows for 1" penetration)
- Strata Interface
- RQD= Rock Quality Designation for Rock Core Sample
- BOE — Bottom of Exploration
- Bottom of Exploration, Refusal
- Bottom of Exploration, No Refusal

PLAN

25 0 25 50
Scale

PROFILE

Horiz. 25 0 25 50
Vert. 5 0 5 10
SCALE

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRIDGE NO. 2127 | WIN 21709.00 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	BMC	APR 2017
CHECKED-REVIEWED	CLS / ARB	APR 2017
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

CAPE NEDDICK BRIDGE
ROUTE 1 OVER CAPE NEDDICK RIVER
YORK — YORK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
6
OF 29

Filename: ...\Geotech\MSTA\BLP\002_BLP.dgn Division: HIGHWAY Username: common Date:6/29/2018

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-101
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	29.4	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/7/17 - 3/7/17	Drilling Method:	SSA/Drive & Wash	Core Barrel:	-
Boring Location:	N130976.6, E918950.7	Casing ID/OD:	4"/3"	Water Level *:	13.5'
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		-ASPHALT-	
								28.1	1.3 -CONCRETE-	
	1D	24/3	2.0 - 4.0	16-16-8-6	24	24		27.6	1.8 Brown, dry, medium dense, fine to coarse SAND, some Gravel, little Silt, with probable cobbles and boulders. -FILL- (SW-SM)	
5	2D	24/1	5.0 - 7.0	14-3-6-4	9	9			Brown, dry, loose, fine to coarse SAND, some Gravel, some Silt, with probable cobbles and boulders. -FILL- (SM)	
10	3D	24/9	10.0 - 12.0	9-14-17-21	31	31	89		Brown, moist, dense, fine to coarse SAND, some Gravel, some Silt, with probable cobbles and boulders. -FILL- (SM)	G*1 A-2-4, SM wc=14.6
							61			
							70			
							58	15.9	13.5	
							50			
15	4D	24/7	15.0 - 17.0	10-99-25-18	124	124	128		Brown, wet, very dense, sandy GRAVEL, little Silt, with probable cobbles and boulders. -GLACIAL TILL- (GM)	G*2 A-1-a, GM wc=14.3
							117			
							136			
							66			
							22/2"	10.2	Broke NW casing. Moved to BB-YCN-101A. 19.2 Bottom of Exploration at 19.20 feet below ground surface.	
20										
25										

Remarks:

1. Water level measured 15 minutes after completion of drilling.
2. Cobbles and boulders throughout fill, based on casing and roller cone advancement.
3. 4" casing met refusal at 19.2' bgs; then 3" casing broke while setting up to core. Bottom 10.0' of casing remained in borehole.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 2
Boring No.: BB-YCN-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-101A
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	29.5	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/8/17 - 3/8/17	Drilling Method:	SSA/Spin & Wash	Core Barrel:	NX2
Boring Location:	N130972.4, E918952.5	Casing ID/OD:	3"	Water Level *:	13.2'
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPUN		Refer to Boring BB-YCN-101. Moved approximately 4.5' south of boring BB-YCN-101 and spun 3" casing to rock. No sampling in overburden materials.	
5										
10										
15										
								9.7	19.8	
20	R1	18/17	20.0 - 21.5	RQD = 24%			NX2		Spun casing to 19.8' bgs then roller cone to 20.0' bgs and set up to core. R1: Hard, fresh, fine to medium grained, gray/tan, GRANITE/GRANODIORITE. Joints are very close to close, low angle, rough, undulating, fresh, partially open to open. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 2.75, 2.5/6" Recovery = 94%	
	R2	60/58	21.5 - 26.5	RQD = 47%					R2: Hard, fresh, fine to medium grained, gray/tan, GRANITE/GRANODIORITE. Primary joints are very close to close, low angle, rough, undulating, fresh, partially open to open. Secondary joints are moderately spaced, moderate to high angle, rough, undulating, fresh to discolored, partially open to open. Rock Mass Quality = Poor Rock Core Times (min/ft): 3.5, 2.75, 3.5, 3.0, 3.75 Recovery = 97%	
25										
	R3	42/42	26.5 - 30.0	RQD = 10%					R3: Hard, fresh, aphanitic, gray, GRANODIORITE with calcite stringers. Joints are extremely close to close, low angle, rough, undulating, fresh to discolored, partially open to open. Two high angle joints. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 3.0, 3.25, 3.0, 2.75/6" Recovery = 100%	
30	R4	36/36	30.0 - 33.0	RQD = 28%					R4: Hard, fresh, fine to medium grained, tan/gray, GRANITE/GRANODIORITE. Joints are very close to close, low angle, rough, undulating, fresh to discolored, tight to open. Rock Mass Quality = Poor Rock Core Times (min/ft): 1.5, 2.5, 3.0 Recovery = 100%	
	R5	42/42	33.0 - 36.5	RQD = 60%					R5: Hard, fresh, coarse grained, tan/gray, GRANITE/GRANODIORITE. Joints are very close to moderately spaced, low angle, rough, undulating, fresh to discolored, partially open to open. Rock Mass Quality = Fair Rock Core Times (min/ft): 3.0, 2.75, 2.0, 1.25/6" Recovery = 100%	
35								-7.0	36.5 Bottom of Exploration at 36.50 feet below ground surface.	
40										
45										
50										

Remarks:

1. 3" casing to 20' bgs.
2. Water level measured 20 minutes after completion of drilling.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 2
Boring No.: BB-YCN-101A

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-102
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	30.6	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/6/17 - 3/6/17	Drilling Method:	SSA/Drive & Wash	Core Barrel:	-
Boring Location:	N130981.5, E918969.8	Casing ID/OD:	3"	Water Level *:	15.5'
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		-ASPHALT-	
								29.3	1.3 -CONCRETE-	
	1D	10/10	2.0 - 2.8	19-100/4"				28.8	1.8 Brown/gray, dry, sandy GRAVEL, little Silt, with probable cobbles and boulders. -FILL- (GP-GM)	
5	2D	24/10	5.0 - 7.0	2-4-2-4	6	6			Brown, dry, loose, fine to coarse SAND, some Silt, little Gravel, with probable cobbles and boulders. -FILL- (SM)	G*3 A-2-4, SM wc=10.4
10	3D	24/3	10.0 - 12.0	3-20-13-8	33	33			Brown, dry, dense, fine to coarse SAND, some Silt, little Gravel, with probable cobbles and boulders. -FILL- (SM)	
							33			
							47	17.6	13.0	
							24			
							20			
15	4D	24/7	15.0 - 17.0	5-4-13-15	17	17	14		Brown, wet, medium dense, gravelly fine to coarse SAND, little Silt, with probable cobbles and boulders. -GLACIAL TILL- (SM)	
							33			
							37			
							110			
							79	10.4	No recovery.	
20	5D	2/0	20.0 - 20.2	60/2"	R		RC		20.2 -POSSIBLE BOULDER-	
								8.6	22.0 -POSSIBLE GLACIAL TILL-	
								7.9	22.7 -POSSIBLE BEDROCK-	
								6.6	24.0 Bottom of Exploration at 24.00 feet below ground surface.	
25										

Remarks:

1. 3" casing to 20' bgs.
2. Water level measured 20 minutes after completion of drilling.
3. Cobbles and boulders throughout Fill, based on casing and roller cone advancement.
4. Split spoon refusal at 20.2' bgs, roller cone from 20.2'-24.0' with approximately 1000 psi pressure 750 rpm, intermediate resistance from 20.2'-22.0' bgs, then minimal resistance from 22.0'-22.7' bgs, then increased resistance (approximately 5 min/ft) from 22.7'-24.0' bgs.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-YCN-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-103
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	29.4	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/7/17 - 3/7/17	Drilling Method:	SSA/Spin & Wash	Core Barrel:	-
Boring Location:	N131008.8, E918940.3	Casing ID/OD:	3"	Water Level *:	12.7'
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA		-ASPHALT-	
								27.9	1.5 -CONCRETE-	
	1D	24/8	2.0 - 4.0	12-10-11-27	21	21		27.6	1.8 Brown, moist, medium dense, gravelly fine to coarse SAND, little Silt, with probable cobbles and boulders. -FILL- (SM)	
5	2D	24/0	5.0 - 7.0	14-8-8-8	16	16	SPUN		No recovery.	
							/ RC			
								21.6	7.8	
10	3D	24/6	10.0 - 12.0	4-7-11-17	18	18			Brown, moist, medium dense, gravelly fine to coarse SAND, little Silt, with probable cobbles and boulders. -GLACIAL TILL- (SM)	G*4 A-1-a, SM wc=13.8
15								13.3		
	4D	1/0	16.0 - 16.1	50/1"	R		RC		Split spoon refusal at 16.1' bgs. No recovery. 16.1 -POSSIBLE BEDROCK-	
								12.1	17.3 Bottom of Exploration at 17.30 feet below ground surface.	
20										
25										

Remarks:

1. From 2.0'-6.0' bgs encountered probable cobbles throughout, based on casing and roller cone advancement.
2. Roller cone advancement with approximately 1,000 psi 750 RPM, from 16.1'-17.3' bgs, advancement greater than 5 min/ft.
3. Water level measured 30 minutes after completion of drilling.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-YCN-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2170(900)X
WIN 21709.00
BRIDGE PLANS
BRIDGE NO. 2127

	BY	DATE	
PROJ. MANAGER	Devon Eaton		SIGNATURE
DESIGN-DETAILED	E. Fargacs / C. Helmick	6/18	
CHECKED-REVIEWED	L. Driscoll / R. Hanf	6/18	P.E. NUMBER
DESIGN2-DETAILED2	-	-	
DESIGN3-DETAILED3	-	-	DATE
REVISIONS 1	-	-	
REVISIONS 2	-	-	
REVISIONS 3	-	-	
REVISIONS 4	-	-	
FIELD CHANGES	-	-	

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

BORING LOGS 1

SHEET NUMBER
7
OF 29

Date:6/29/2018

Username:

Division:

Filename: 008_Boring Logs 2.dgn

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-104
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	30.6	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/6/17 - 3/6/17	Drilling Method:	SSA/Drive & Wash	Core Barrel:	NX2
Boring Location:	N131016.7, E918958.9	Casing ID/OD:	4"	Water Level *:	-
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			-ASPHALT-	
								29.4		1.2	
	1D	24/10	1.8 - 3.8	21-12-8-9	20	20		28.8		-CONCRETE- 1.8	
										Brown/gray, dry, medium dense, GRAVEL, some fine to coarse Sand, trace Silt, with probable cobbles and boulders. -FILL- (GP-GM)	G*5 A-1-a, GP-GM wc=2.8
5	2D	24/9	5.0 - 7.0	25-21-15-14	36	36				Dark brown, moist, dense, fine to coarse SAND, some Gravel, little Silt, with probable cobbles and boulders,. -FILL- (SM)	
								23.1		No refusal; SSA kicked and could not advance casing, probable boulder; pulled, and moved approximately 7.0' north. 7.5	
										Bottom of Exploration at 7.50 feet below ground surface.	
10											
15											
20											
25											

Remarks:

1. Boulders/cobbles throughout Fill, based on casing and roller cone advancement.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 2

Boring No.: BB-YCN-104

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project:	Cape Neddick Bridge 09.0025930.00	Location:	Route 1 Over Cape Neddick River York, Maine	Boring No.:	BB-YCN-104A
				PIN:	21709.00
Driller:	New England Boring	Elevation (ft.)	30.6	Auger ID/OD:	4.25" OD
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	Blaine Cardali	Rig Type:	Truck - Mobile Drill	Hammer Wt./Fall:	140/30
Date Start/Finish:	3/6/17 - 3/6/17	Drilling Method:	SSA/Drive & Wash	Core Barrel:	NX2
Boring Location:	N131023.4, E918956.9	Casing ID/OD:	4"/3"	Water Level *:	10.6'
Hammer Efficiency Factor:	0.6	Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			-ASPHALT-	
								29.4		1.2	
								28.8		-CONCRETE- 1.8	
										Refer to Boring BB-YCN-104 for soil description. Moved approximately 7.0' north, advanced boring to 10.0' and started sampling.	
5											
								23.1		7.5	
10	3D	24/12	10.0 - 12.0	21-11-8-34	19	19	65			Brown, moist, medium dense, fine to coarse SAND, some Silt, trace Gravel, with probable cobbles and boulders. -FILL- (SM)	G*6 A-2-4, SM wc=15.2
							47				
							309	18.6		12.0	
							118				
							48				
15	4D R1	4/1 23/20	15.0 - 15.3 15.4 - 17.3	100/4" RQD = 52%	R		RC NX2	15.3		Brown, wet, fine to medium SAND, little Silt, with probable cobbles and boulders. -GLACIAL TILL- (SM) 15.3	
										Split spoon refusal at 15.3' bgs, roller cone to 15.4' and set up to core. R1: Hard, fresh, fine to medium grained, gray, GRANODIORITE/GRANITE. Joints are close to moderately spaced, low angle, rough, undulating, fresh to discolored, partially open to open. Rock Mass Quality = Fair Rock Core Times (min/ft): 1.5, 1.75/.9' Recovery = 87%	q_p=4774 ksf
	R2	43/40	17.3 - 20.9	RQD = 74%						R2: Hard, fresh, fine to medium grained, gray, GRANODIORITE/GRANITE. Joints are close to moderately spaced, low angle, rough, undulating, fresh to discolored, partially open to open. One high angle joint. Rock Mass Quality = Fair Rock Core Times (min/ft): 2.5, 2.0, 2.5, 1.5/0.6' Recovery = 93%	q_p=4912 ksf
20	R3	54/60	20.9 - 25.4	RQD = 96%						R3: Hard, fresh, fine to coarse grained, gray, GRANODIORITE/GRANITE. Joints are close to moderately spaced, low angle, rough, undulating, fresh to discolored, partially open to open. Rock Mass Quality = Excellent Rock Core Times (min/ft): 2.0, 3.5, 3.0, 2.75, 2.5/0.5' Recovery = 111%	
25								5.2		25.4	
										Bottom of Exploration at 25.40 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. 4" casing to 15.0' bgs then 3" to 15.4' bgs.
2. Water level taken approximately 15 minutes after completion of drilling.
3. Boulders/cobbles throughout Fill, based on casing and roller cone advancement.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 2

Boring No.: BB-YCN-104A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

WIN 21709.00

BRIDGE PLANS

BRIDGE NO. 2127

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Fargocs	C. Helmick	6/18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6/18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

BORING LOGS 2

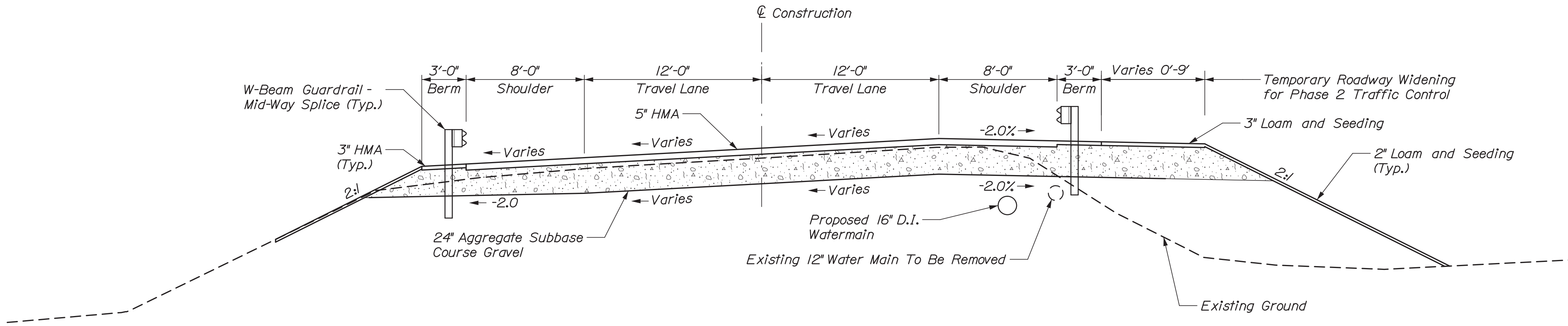
SHEET NUMBER

8

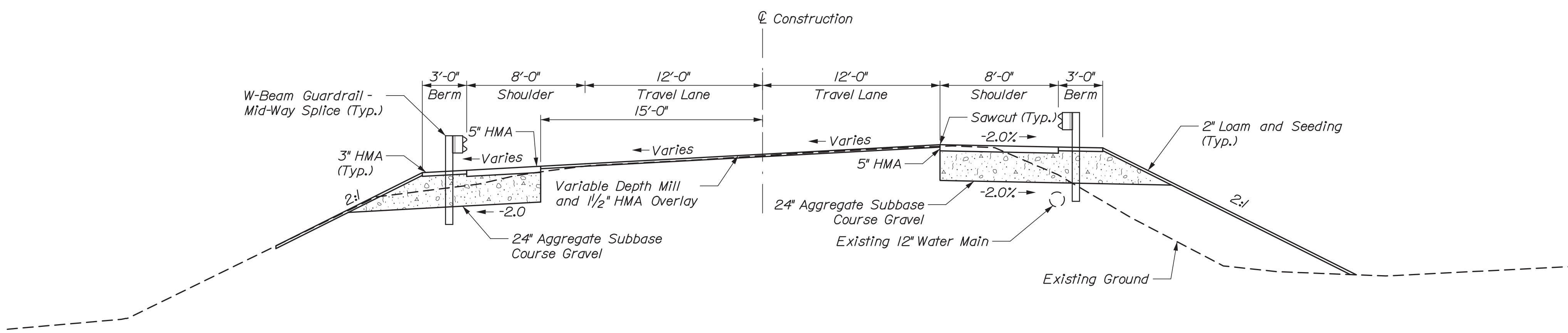
OF 29

NOTES:

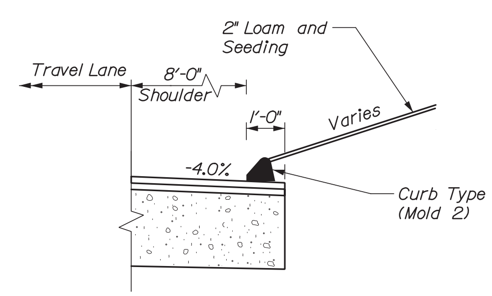
1. The pavement, base and subbase depths as shown on the plans are intended to be nominal.
2. When superelevation of the travelway exceeds the slope of the low side shoulder, the low side shoulder pavement shall have the same cross slope as the travelway.
3. Crowns for both normal and superelevation sections for all courses of subbase and pavement shall be straight.



FULL CONSTRUCTION
(Superelevated)



MILL & FILL
FULL DEPTH SHOULDER CONSTRUCTION
(Superelevated)



CURB TYPE 3 DETAIL

Route 1 Super (e)				
Left Shldr. %	Left Lane %	Station	Right Lane %	Right Shldr. %
Match Existing		1201+75	Match Existing	
-4.0	-4.0	1202+00	+4.0	0.0
-4.7	-4.7	1202+25	+4.7	-1.0
-5.4	-5.4	1202+50	+5.4	-2.0
		To		
-5.4	-5.4	1205+00	+5.4	-2.0
-4.5	-4.5	1205+25	+4.5	-2.0
-4.0	-3.6	1205+50	+3.6	-2.0
-4.0	-2.7	1205+75	+2.7	-2.0
-4.0	-1.8	1206+00	+1.8	-2.0
-4.0	-0.9	1206+25	+0.9	-2.0
-3.3	+0.1	1206+50	-0.1	-2.0
Match Existing		1206+75	Match Existing	

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

WIN 21709.00

BRIDGE NO. 2127

BRIDGE PLANS

PROJ. MANAGER	Devon Eaton	BY	DATE
DESIGN-DETAILED	E. Fargacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

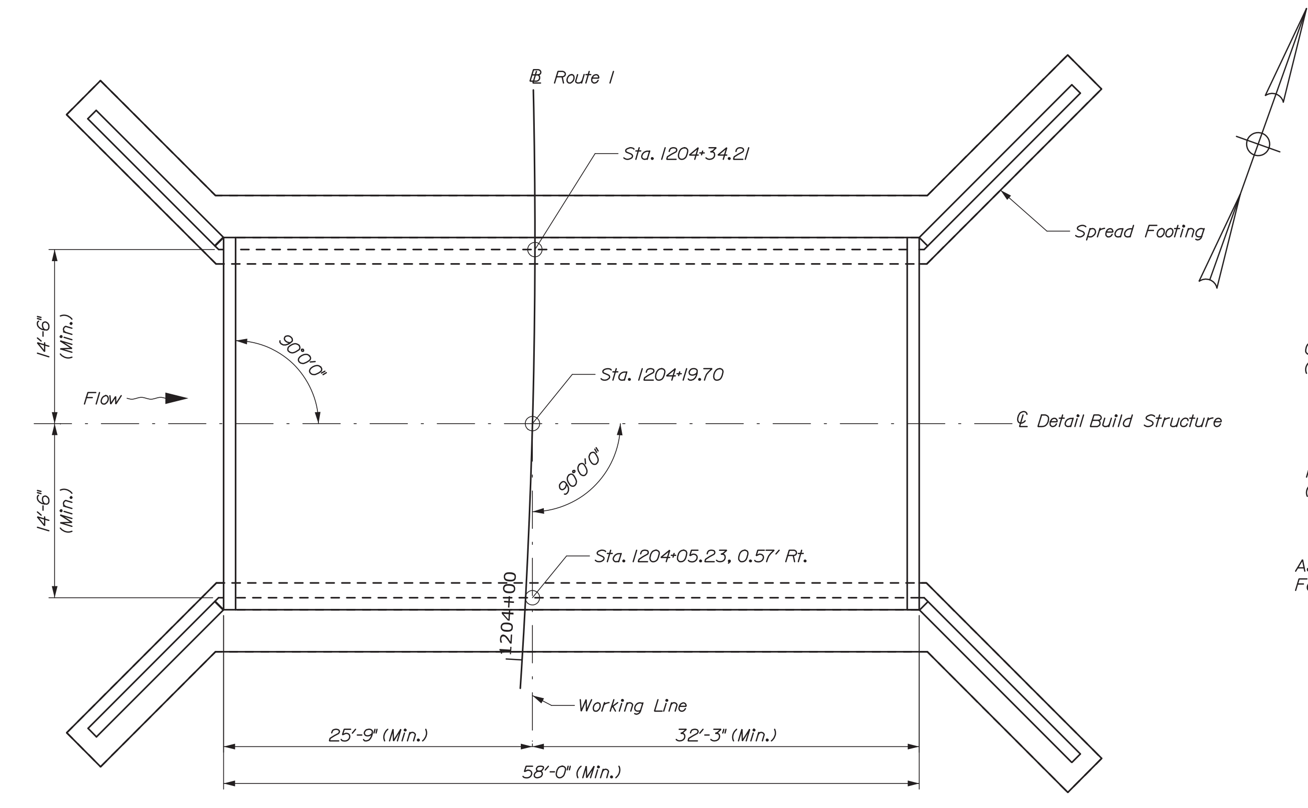
DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

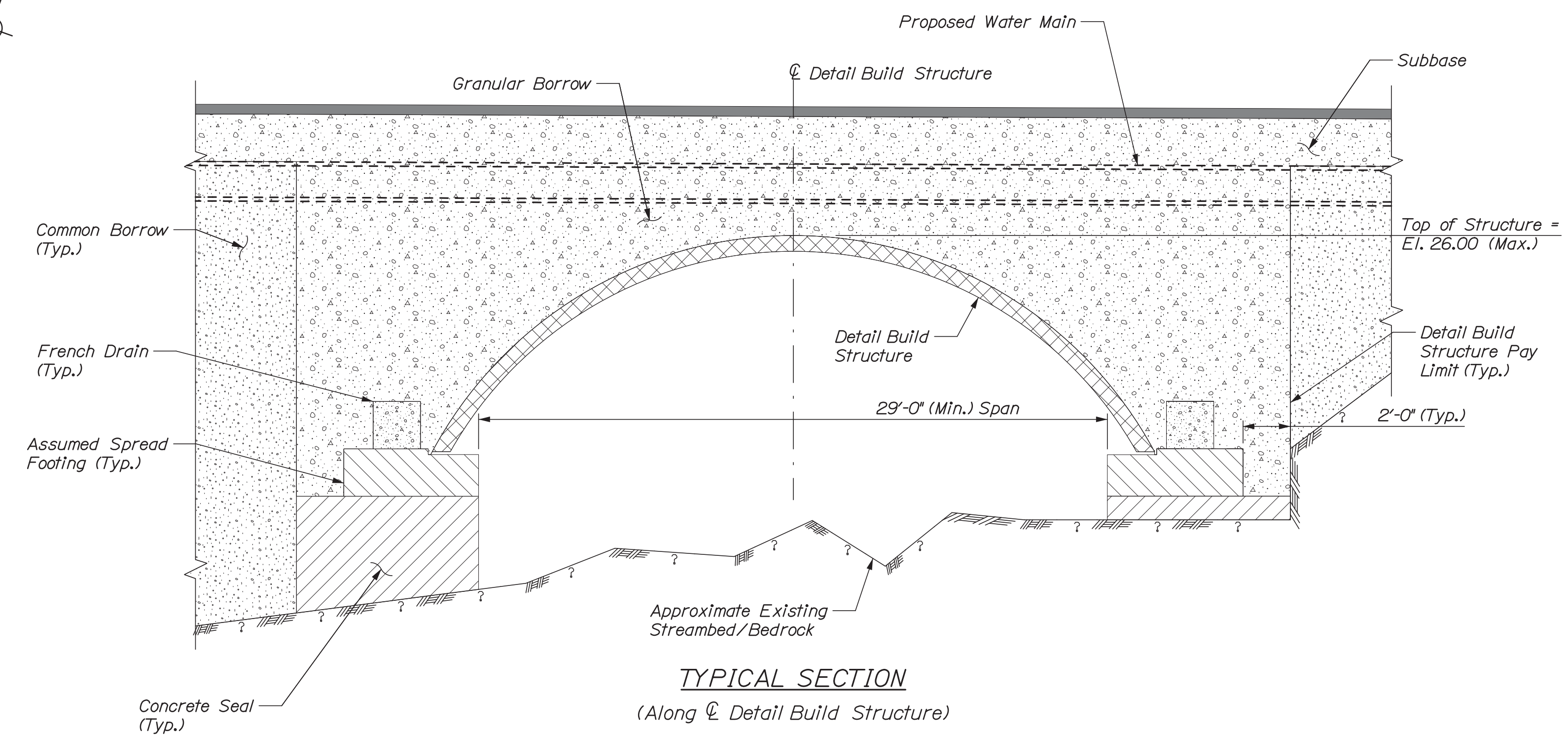
TYPICAL SECTIONS

SHEET NUMBER
9
OF 29

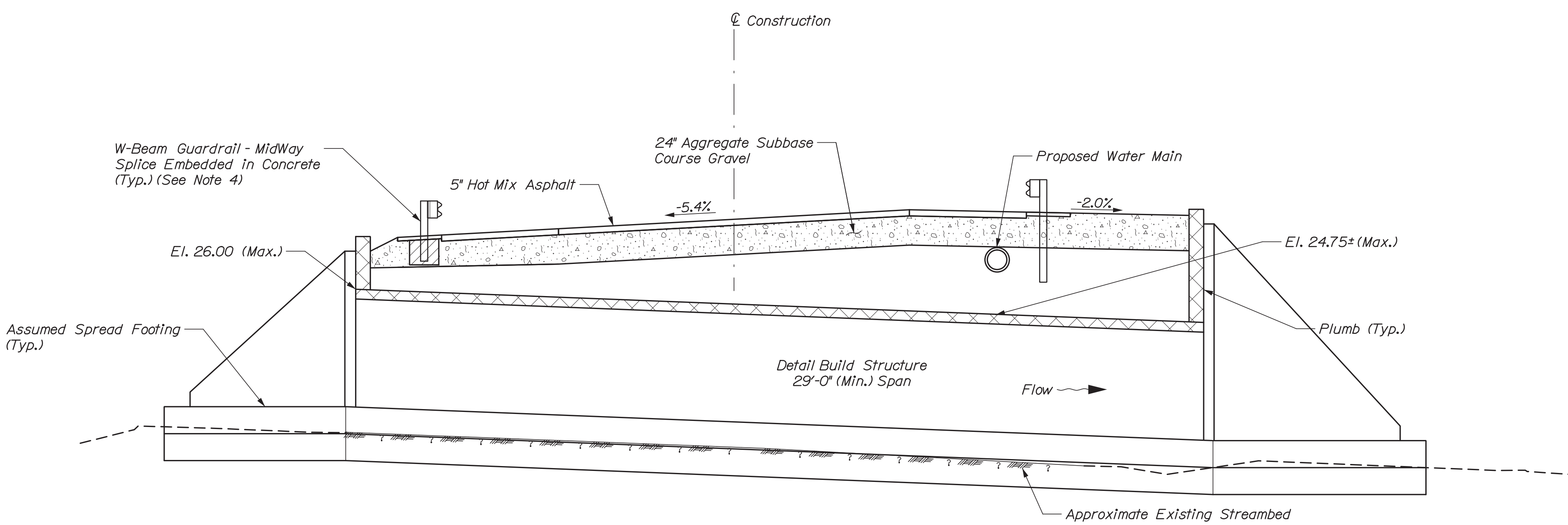
Filename: 009_TypicalSection.dgn
Division:
Username:
Date:6/29/2018



BRIDGE STRUCTURE DETAIL BUILD PLAN



TYPICAL SECTION
(Along ℄ Detail Build Structure)



LONGITUDINAL CROSS SECTION

DETAIL BUILD BRIDGE STRUCTURE NOTES:

1. Structure and foundation are shown for illustrative purposes only. See Special Provision 531.

2. Construct French Drains behind each base of the Structure Detail Build and wingwalls in accordance with Standard Specification Section 512, French Drains. Daylight french drains through weepholes in the abutment and wingwalls. Coordinate daylight locations with Resident in field.

3. Foundations for detail build structure are shown for illustrative purposes only. Actual dimensions will vary based on the Contractor's proposed design concept. See Special Provision 531 and Project Geotechnical Report for additional information and design requirements.

4. For additional information see Standard Detail 606(20).

5. Foundation concrete shall be placed on bedrock, cleaned of all loose rock or soil. The bedrock subgrade shall be confirmed to be relatively level. Where the bedrock slope exceeds 4H:1V, the bedrock shall be benched to make level steps or made completely level. When prepared bedrock surface is below the bottom of the footing, concrete fill may be placed to fill the void.

6. Concrete seal not shown in plan view for clarity.

7. Concrete seal shall be incidental to the Detail Build Structure Pay Item.

8. The Detail Build Structure shall have a minimum hydraulic opening of 233 square feet.

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

WIN 21709.00

BRIDGE PLANS

BRIDGE NO. 2127

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Forgacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

YORK

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY

BRIDGE PLAN AND DETAILS

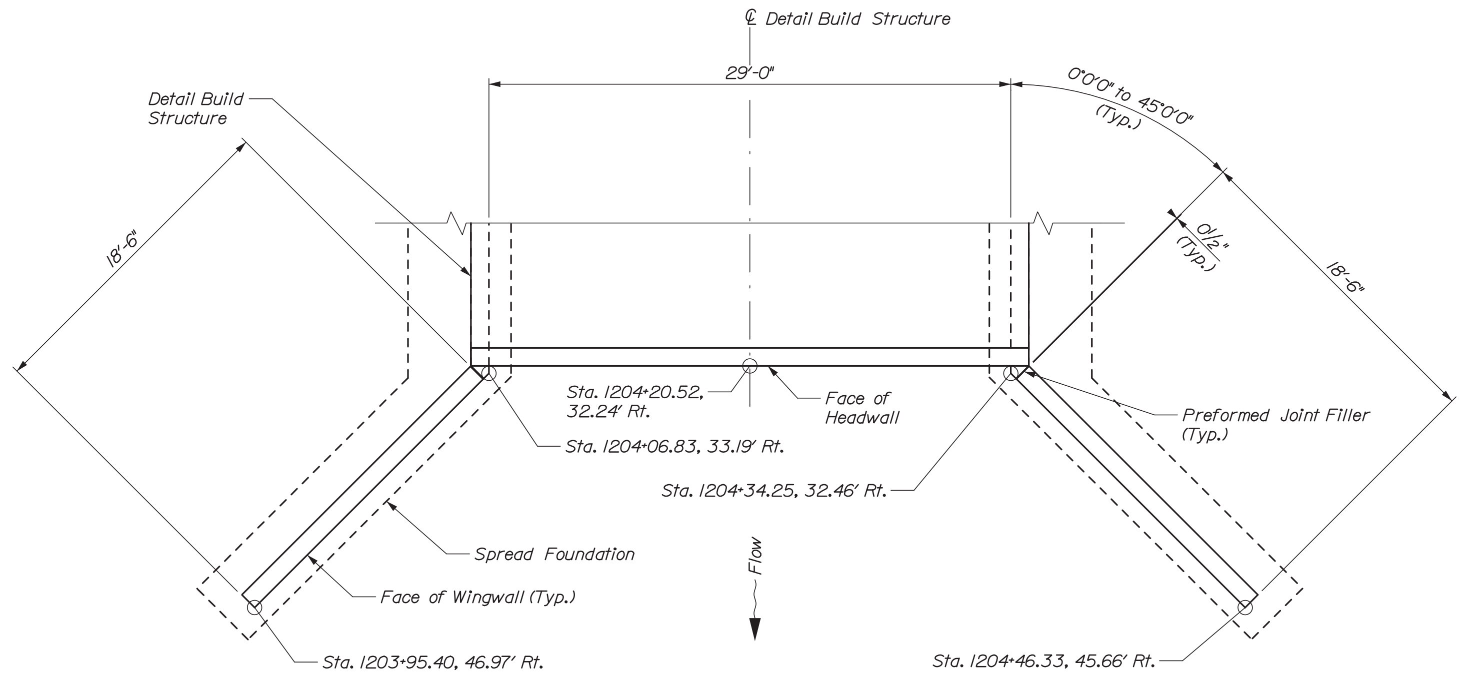
SHEET NUMBER
23
OF 29

Date:6/29/2018

Username:

Division:

Filename: 023_Typ.dgn



PLAN

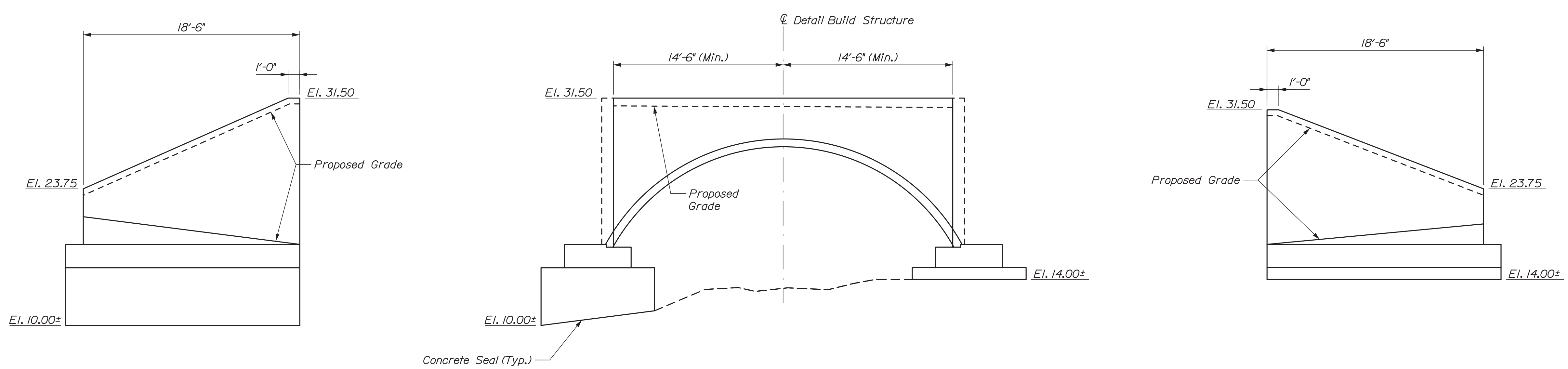
DETAIL BUILD BRIDGE HEADWALL AND WINGWALL NOTES:

1. Bottom of concrete seal is approximate. Top of bedrock is variable and shall be field verified.

2. Detail build headwall and wingwall structure shown is for illustrative purposes only. See Special Provision 531 and Project Geotechnical Report.

3. Wingwalls and their footings shall be backfilled with Granular Borrow. Backfill will not be measured for payment, but shall be included in the Detail Build Structure Pay Item.

4. Refer to the Project Geotechnical Report for detail build wingwall geotechnical design requirements.



SOUTHEAST WINGWALL

EAST HEADWALL

NORTHEAST WINGWALL

ELEVATION

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

BRIDGE NO. 2127

WIN 21709.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Fargacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

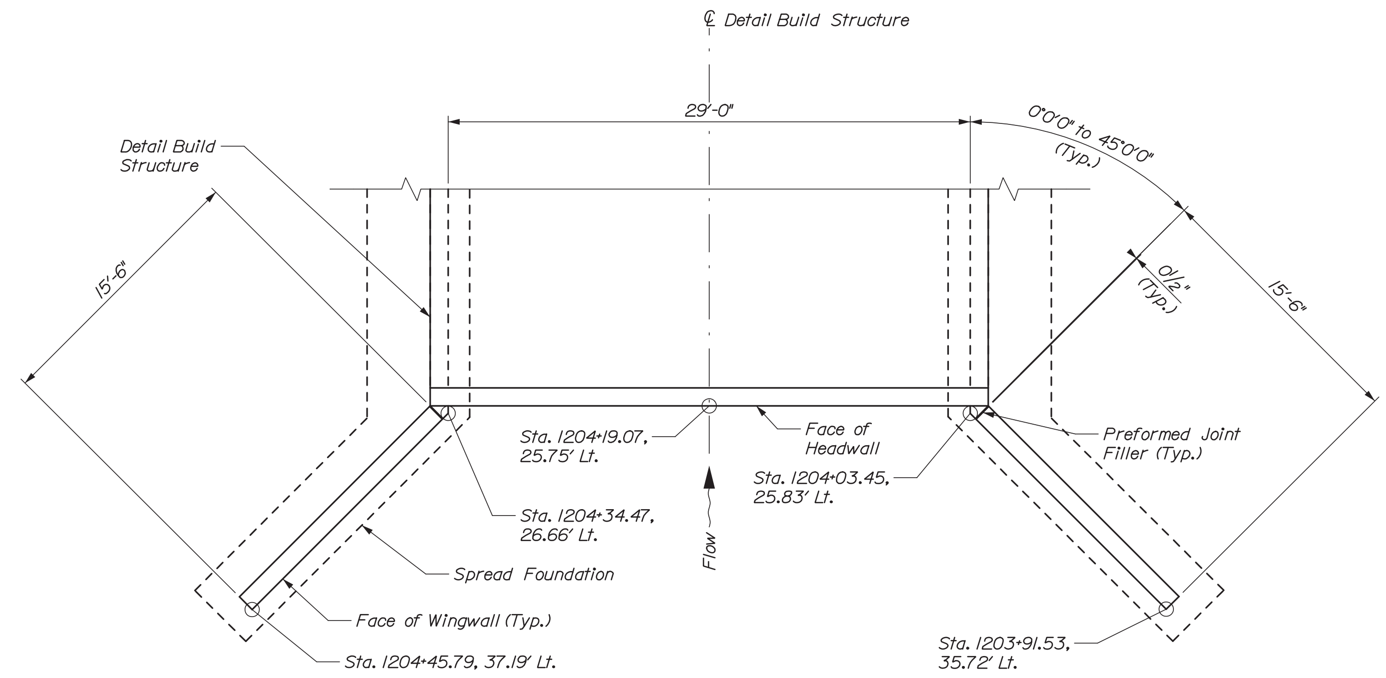
DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

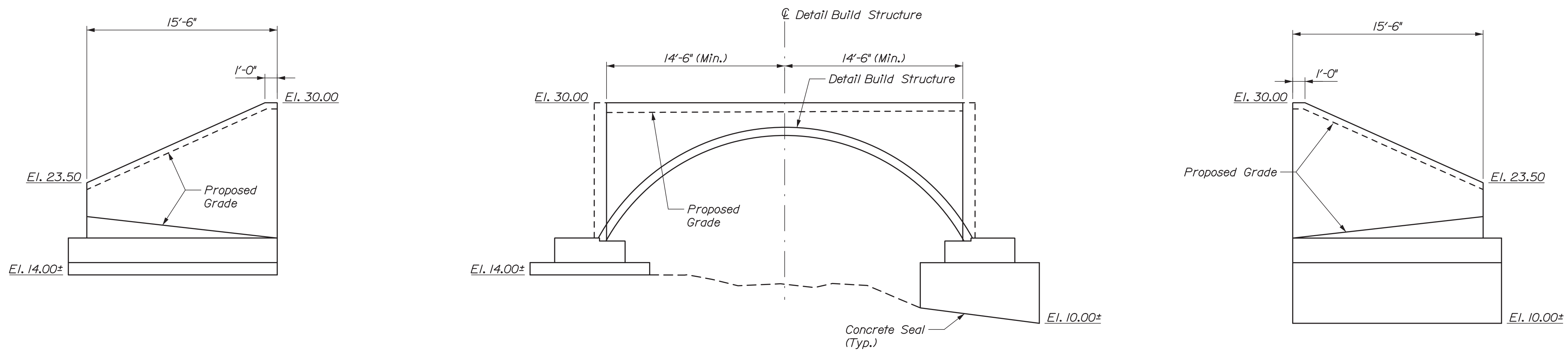
EAST HEADWALL
AND WINGWALLS

SHEET NUMBER
24
OF 29

Filename: 024_Headwall 1.dgn
Division:
Username:
Date:6/29/2018



PLAN



ELEVATION

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(900)X

WIN 21709.00

BRIDGE NO. 2127

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	Devon Eaton		
DESIGN-DETAILED	E. Fargacs	C. Helmick	6\18
CHECKED-REVIEWED	L. Driscoll	R. Hanf	6\18
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

CAPE NEDDICK BRIDGE
CAPE NEDDICK RIVER
YORK COUNTY
YORK

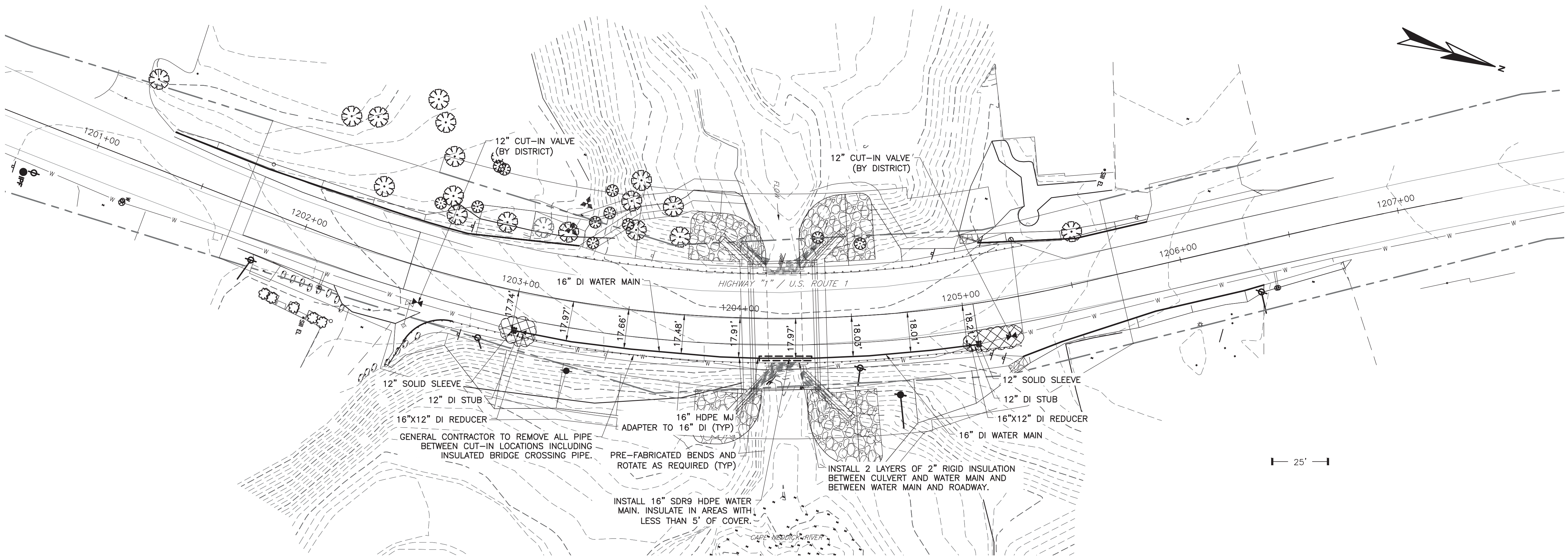
WEST HEADWALL
AND WINGWALLS

SHEET NUMBER
25
OF 29

Filename: 025_Headwall 2.dgn
Division:
Username:
Date:6/29/2018

NOTES:

1. EXISTING CONDITION INFORMATION SHOWN ON THIS PLAN IS BASED ON INFORMATION FROM THE STATE OF MAINE DEPARTMENT OF TRANSPORTATION (MDOT) BRIDGE DESIGN PLAN SET TITLED, "CAPE NEDDICK BRIDGE OVER CAPE NEDDICK RIVER – U.S. ROUTE 1 – FEDERAL AID PROJECT NO. STP–2170(900)X" DATED JANUARY 18, 2018, CREATED BY HNTB.
2. CONTRACTOR TO DIG TEST PIT TO CONFIRM LOCATION OF EXISTING 12" WATER MAIN AND EXISTING UTILITIES.
3. INSERTION VALVES TO BE INSTALLED BY YORK WATER DISTRICT.



NO.	SUBMISSIONS/REVISIONS	APP'D	DATE

DRAWN BY
CHECKED BY WTE
DATE
APPROVED BY
DATE
BOOK NO.
PROJECT NO.
SCALE 13970A

STATE OF MAINE – JEFFREY P. MUSICH – No. 6058 – LICENSED – PROFESSIONAL ENGINEER

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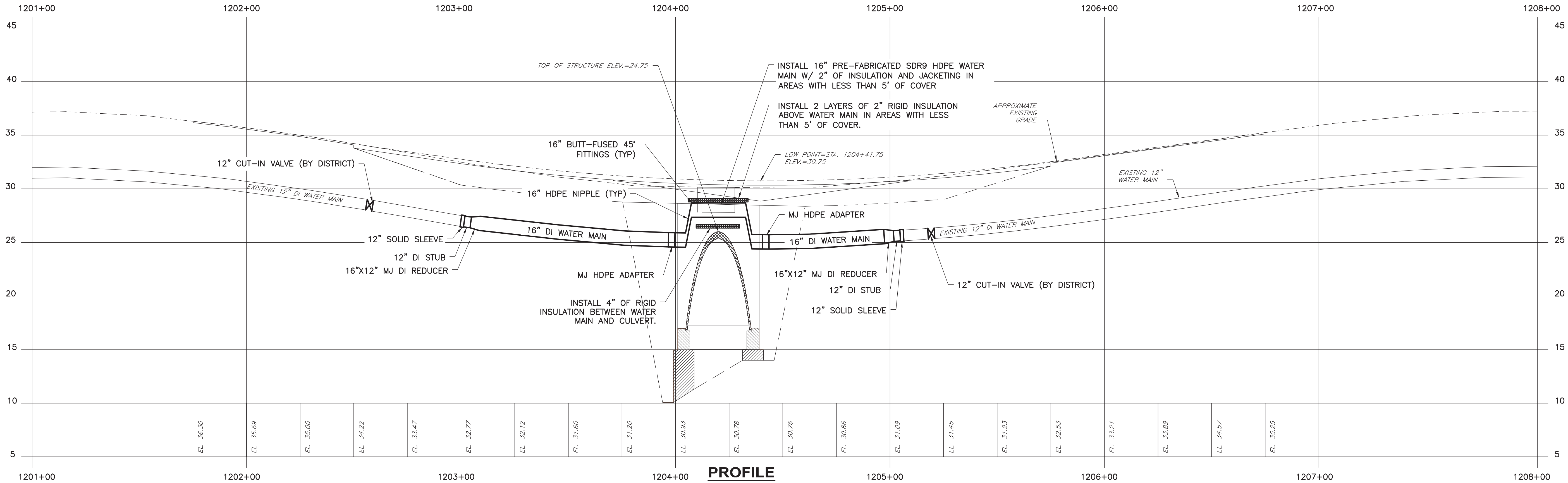
CAPE NEDDICK CROSSING
CAPE NEDDICK RIVER
YORK — YORK COUNTY

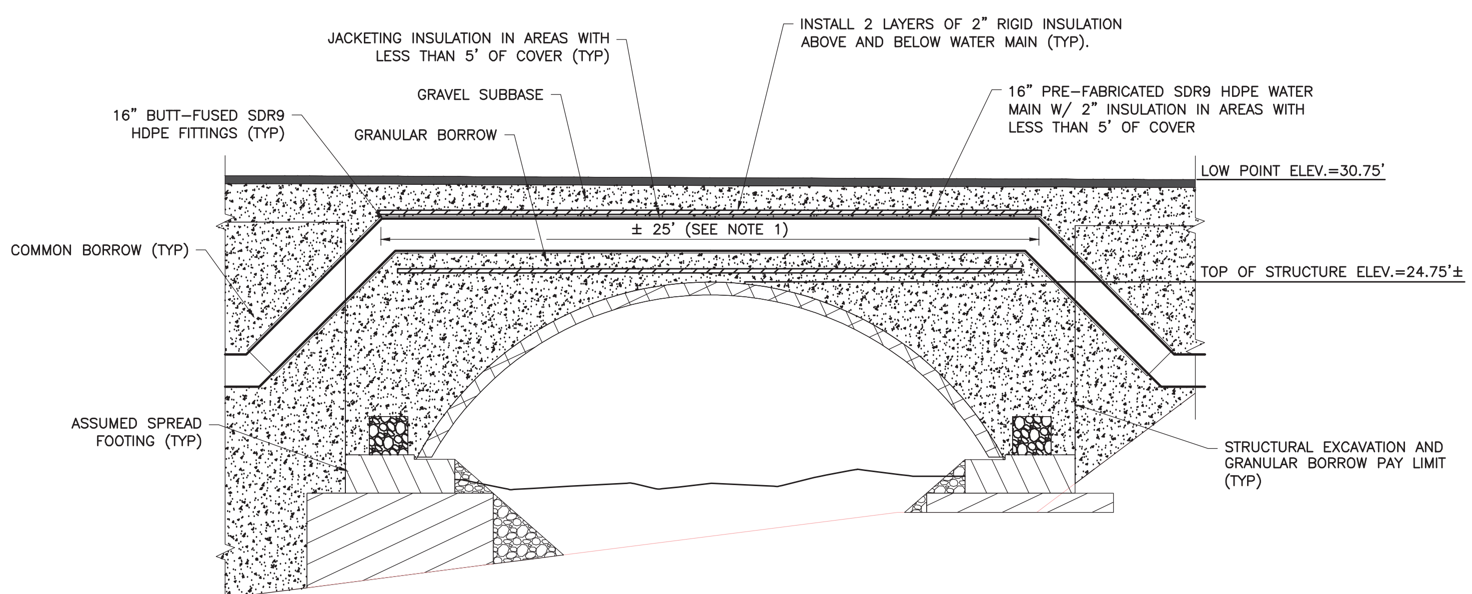
WATER MAIN REPLACEMENT – SITE PLAN

DRAWING
26



PROFILE

SCALE
VERT=1"-5'
HORIZ=1"-25'

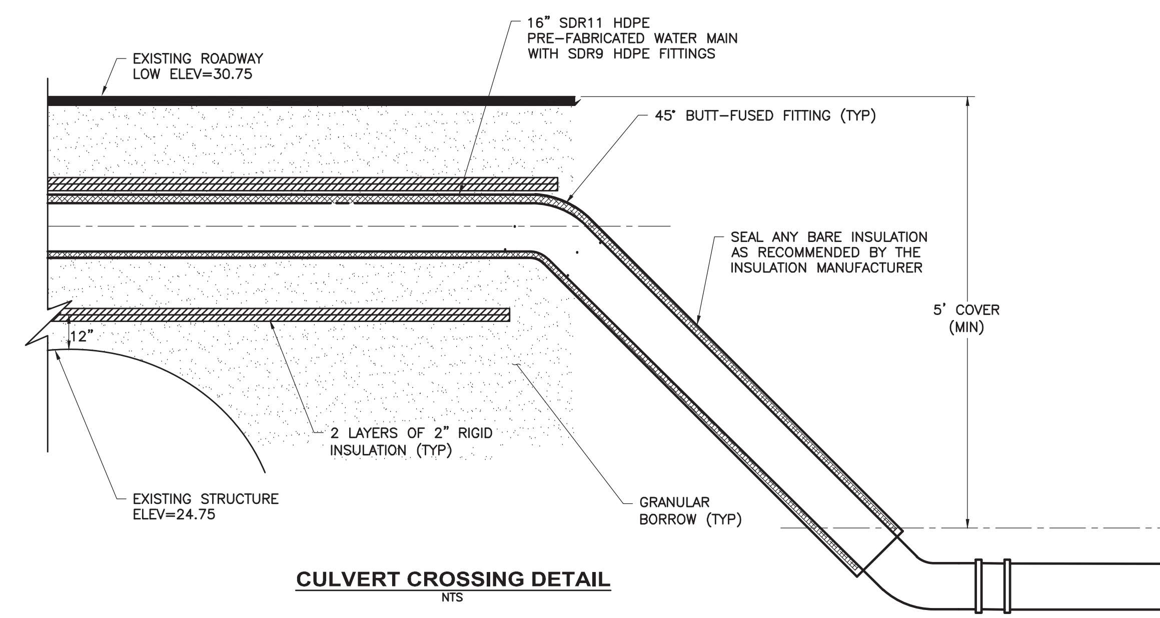


TYPICAL SECTION - CULVERT CROSSING

NTS

NOTES:

1. EXACT DIMENSIONS WILL BE CONFIRMED AFTER CULVERT DESIGN IS APPROVED DURING THE CONSTRUCTION PHASE.
2. DISTRICT WILL FURNISH ENTIRE HDPE PIPE SEGMENT, PRE-FABRICATED AND FUSED. DISTRICT WILL ALSO FURNISH ALL DI PIPE AND FITTINGS.
3. ALL HDPE SHALL HAVE 2-INCHES OF PIPE INSULATED JACKETING (TO BE FURNISHED WITH PIPE BY THE YORK WATER DISTRICT.



CULVERT CROSSING DETAIL

NTS

1. CONTRACTOR TO NOTE THIS DETAIL IS TYPICAL FOR BOTH SIDES OF BRIDGE.
2. INSTALL 16" SDR9 HDPE PRE-FABRICATED WATER MAIN OVER THE CULVERT. ALL BENDS AND WATER MAIN SHALL BE INSULATED WITH 2" INSULATION AND JACKET IN AREAS WITH LESS THAN 5' OF COVER.
3. TWO LAYERS OF 2" RIGID POLYURETHANE INSULATION SHALL BE PLACED ABOVE AND BELOW WATER MAIN IN AREAS WITH LESS THAN 5' OF COVER.
4. ALL BELOW GRADE BENDS SHALL BE RESTRAINED MECHANICAL JOINT AND BE PROVIDED WITH THRUST BLOCKS. SEE THRUST BLOCK DETAIL, THIS SHEET.
5. ALL UNINSULATED BELOW GRADE PIPING SHALL BE ENCASED WITH A TUBE TYPE POLYETHYLENE ENCASEMENT WRAP. FOR ADDITIONAL INFORMATION, REFER TO SPECIFICATION.

LAST SAVED BY: WILLIAM.EDGAR 6/26/2018 9:41 AM

G:\DWGS\ME\YORKWD\13970-CAPENEDDICKBRIDGE\13970-FD\CIV\13970A-PROFILEPLAN.DWG | ProfilePlan | 1:4.56 | ---- | 6/26/2018 9:52:46 AM | WILLIAM.EDGAR

SUBMISSIONS/REVISIONS | NO | APP'D | DATE

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BOOK NO.
PROJECT NO.
SCALE 13970A

STATE OF MAINE — JEFFREY P MUSICH No. 6658 — LICENSED PROFESSIONAL ENGINEER

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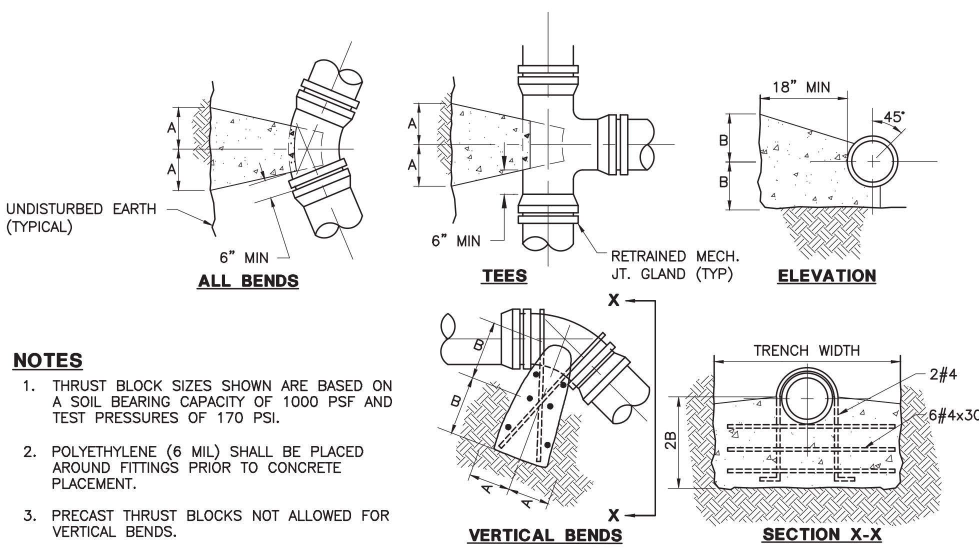
CAPE NEDDICK CROSSING
CAPE NEDDICK RIVER
YORK YORK COUNTY

WATER MAIN CROSSING - PROFILE VIEW

DRAWING
27

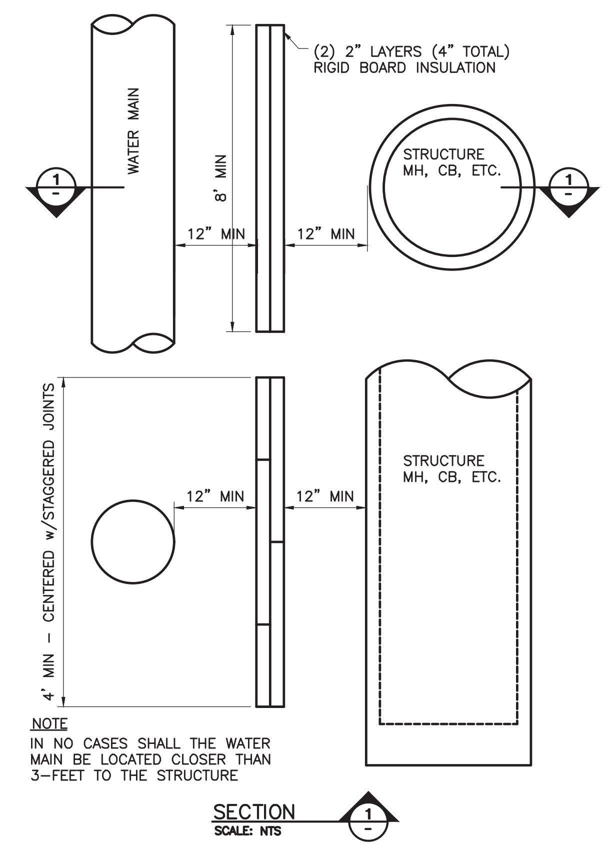


NOTES

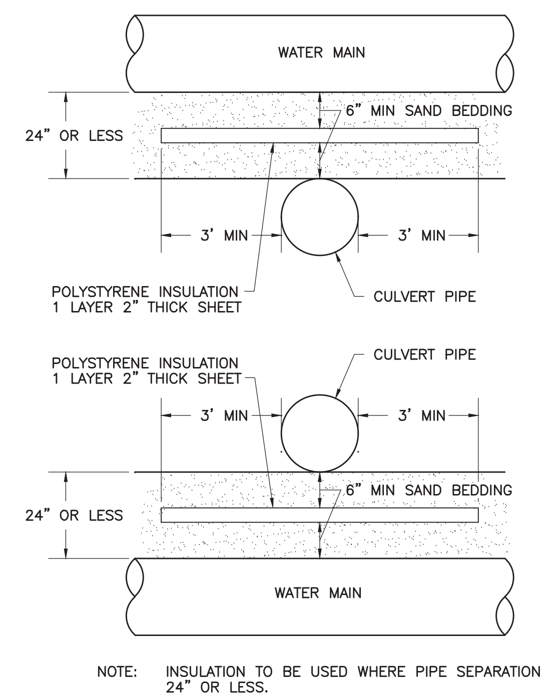
1. THRUST BLOCK SIZES SHOWN ARE BASED ON A SOIL BEARING CAPACITY OF 1000 PSF AND TEST PRESSURES OF 170 PSI.
2. POLYETHYLENE (6 MIL) SHALL BE PLACED AROUND FITTINGS PRIOR TO CONCRETE PLACEMENT.
3. PRECAST THRUST BLOCKS NOT ALLOWED FOR VERTICAL BENDS.

PIPE SIZE	90° BEND		45° BEND		22 1/2° BEND		11 1/4° BEND		TEE		VERTICAL BEND (DOWN)		DEAD END	
	A	B	A	B	A	B	A	B	A	B	A	B	A	B
6"	15"	12"	12"	9"	9"	6"	6"	6"	12"	12"	24"	21"	–	–
8"	20"	15"	14"	12"	9"	9"	9"	6"	18"	12"	33"	24"	–	–
10"	21"	21"	18"	15"	15"	9"	9"	9"	20"	18"	40"	27"	–	–
12"	27"	24"	23"	15"	15"	12"	12"	9"	25"	18"	48"	30"	–	–
16"	37"	30"	30"	21"	21"	15"	13"	12"	32"	24"	57"	36"	–	–
20"/24"	36"	36"	30"	24"	24"	24"	24"	24"	32"	30"	32"	30"	32"	30"

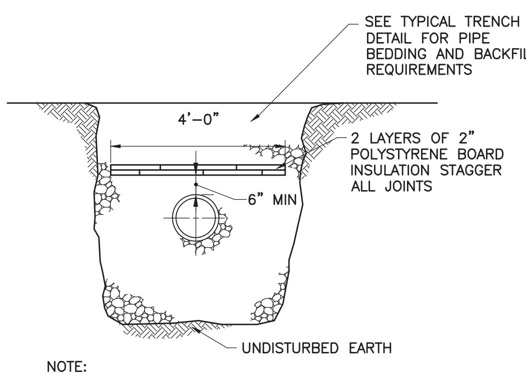
THRUST BLOCK DETAIL
NTS



WATER MAIN-STRUCTURE INSULATION DETAIL
NTS



CULVERT CROSSING DETAIL
NTS

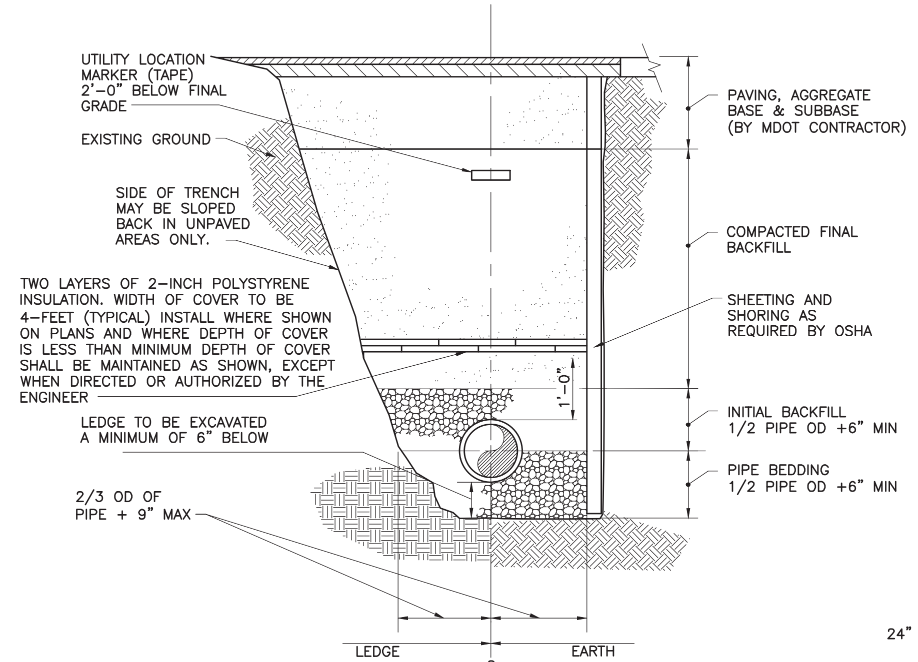


NOTE:
1. TRENCH PIPE INSULATION TO BE USED WHERE DEPTH OF COVER IS LESS THAN 5'-0" FEET OR AS DIRECTED BY THE ENGINEER.

TRENCH PIPE INSULATION DETAIL
NTS

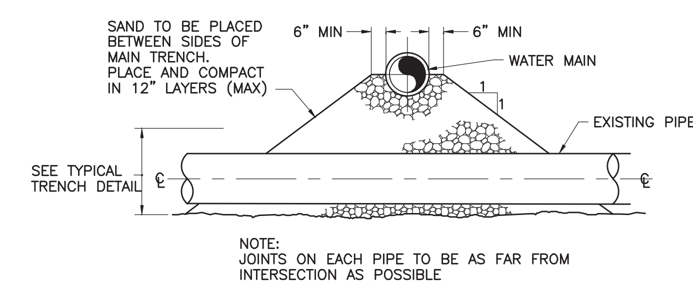
NOTES:

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE LAYOUT OF THE NEW WATER MAIN. LAYOUT SHALL BE REVIEWED AND ACCEPTED BY YORK WATER DISTRICT.
2. MINIMUM DEPTH OF COVER FOR WATER MAINS SHALL BE 5'-0" AND MAXIMUM DEPTH OF COVER SHALL BE 7'-0" UNLESS SHOWN OTHERWISE ON THE DRAWINGS. THE NEW WATER MAINS SHALL GENERALLY FOLLOW THE GROUND CONTOUR.
3. NEW MAINS SHALL BE DUCTILE IRON PIPE WITH PUSH-ON JOINTS UNLESS SHOWN OTHERWISE ON THE DRAWINGS. ALL FITTINGS SHALL BE DUCTILE IRON MECHANICAL JOINT, UNLESS OTHERWISE REQUIRED FOR JOINT RESTRAINT OR SHOWN ON THE DRAWINGS.
4. ALL BENDS, TEES, REDUCERS, HYDRANTS AND PLUGS SHALL BE RESTRAINED BY USING PRECAST CONCRETE THRUST BLOCKS AND MECHANICAL JOINT RESTRAINTS, OR OTHER METHODS AS SHOWN ON THE DRAWINGS. ANCHOR TEES SHALL BE USED FOR ALL HYDRANT BRANCHES EXCEPT IN LOCATIONS WHERE GATE VALVES CAN NOT BE CONNECTED DIRECTLY TO TEE, IN WHICH CASE RODS SHALL BE USED.
5. TEST PRESSURE FOR THE COMBINED PRESSURE AND LEAKAGE TEST SHALL BE AS NOTED IN SPECIFICATION SECTION 611.3.12.
6. TRENCHES SHALL NOT BE LEFT OPEN DURING NON-WORKING HOURS. ALL OPEN PIPES SHALL BE SECURED WITH A WATER-TIGHT PLUG WHEN THE PIPE IS TEMPORARILY BACKFILLED AND WHEN PIPE LAYING IS NOT IN PROGRESS.
7. WHERE A NEW WATER MAIN PASSES BELOW OR ABOVE A SANITARY OR STORM SEWER, A FULL LENGTH OF WATER MAIN SHALL BE CENTERED ABOVE/BELOW THE SEWER. MAINTAIN A MINIMUM SEPARATION DISTANCE OF 18 INCHES.
8. CONNECTIONS TO EXISTING WATER MAIN SHALL BE COORDINATED WITH THE YORK WATER DISTRICT.
9. WHERE NEW PIPING IS TO BE CONNECTED TO EXISTING PIPING, THE CONTRACTOR SHALL FURNISH AND INSTALL ALL ADAPTERS, FITTINGS, AND ADDITIONAL PIPE AS REQUIRED TO COMPLETE THE CONNECTION. CONTRACTOR SHALL VERIFY LOCATION, ELEVATION, ORIENTATION AND MATERIAL OF CONSTRUCTION. TEST PITS SHALL BE USED AS REQUIRED.
10. PROVIDE 4" RIGID INSULATION OVER WATER LINES WHEN COVER IS ALLOWED, BY THE ENGINEER, TO BE LESS THAN 5' OVER MAINS AND SERVICES. STAGGER ALL JOINTS ON RIGID INSULATION BOARD. INSULATION BOARD TO BE SUPPLIED BY DISTRICT.
11. WATER SERVICE INTERRUPTIONS SHALL BE 4 HOURS OR LESS IN DURATION WITH NOTIFICATION TO ALL WATER CUSTOMERS AND THE YORK WATER DISTRICT A MINIMUM OF 72 HOURS PRIOR TO ANY INTERRUPTION IN SERVICE.
12. DO NOT SCALE DRAWINGS UNLESS OTHERWISE NOTED.
13. CONTRACTOR SHALL PROVIDE, FOR REVIEW AND APPROVAL BY THE ENGINEER AND THE YORK WATER DISTRICT, DETAILED CONSTRUCTION SCHEDULES OUTLING PROJECT TIMING, EXPECTED SHUTDOWNS AND ANTICIPATED DURATION.
14. ALL ABANDONED PIPES SHALL BE REMOVED AND LOADED ONTO YORK WATER DISTRICT TRUCKS. THE YORK WATER DISTRICT WILL DISPOSE OF ALL ABANDONED PIPES AND MATERIALS.
15. ALL PIPE MATERIALS AND APPURTENANCES TO BE FURNISHED BY THE YORK WATER DISTRICT.



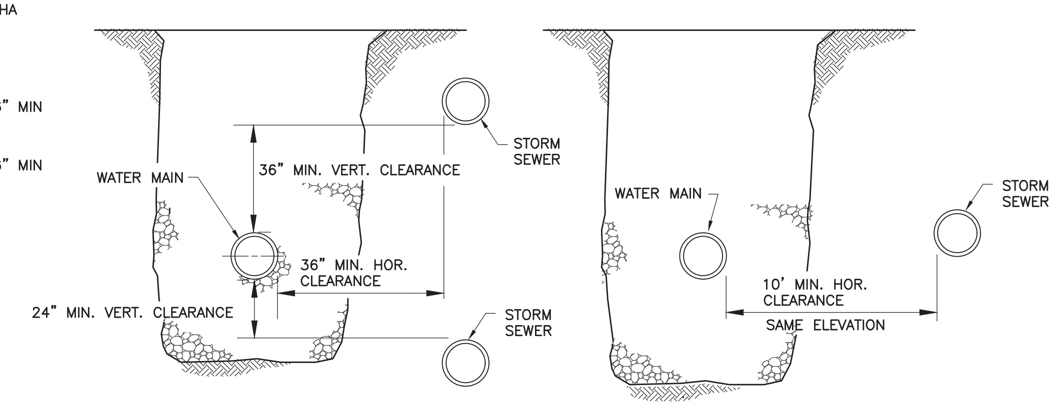
NOTES:
1. ALL EXCAVATION MUST MEET OSHA STANDARDS
2. INSTALL 3 FOOT LONG IMPERVIOUS MATERIAL DAM IN BEDDING/INITIAL BACKFILL MATERIAL EVERY 100 FEET TO PREVENT TRENCH GROUND WATER FROM BEING CHANNELED ALONG BEDDING/INITIAL BACKFILL.

PIPE TRENCH DETAIL
NTS



NOTE:
JOINTS ON EACH PIPE TO BE AS FAR FROM INTERSECTION AS POSSIBLE

PIPE CROSSING DETAIL
NTS



TRENCH PIPE CLEARANCE DETAIL
NTS

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YORK COUNTY
WATER MAIN DEAILS I

DRAWING
28

APPENDIX G

Scour Analysis

SCOUR EVALUATION

Purpose:

Calculation of abutment scour depths at buried structure.

References:

- AASHTO LRFD Bridge Design Specifications, 7th edition, 2014
- HEC-18 Evaluation Scour at Bridges, Fifth Edition

Notes & Assumptions:

- Calculation for scour for Design Scour Q100 and check scour Q500 event.

Q100 DESIGN SCOUR EVENT:

CONTRACTION SCOUR

Determine Type of Scour:

Average Depth of Flow Upstream: $y := 6.35ft$ From x-section 461.07

Particle Size for Vc: $D_{50} := 2.889mm$ From geotechnical report

Unit Factor: $K_u := 11.17$ For English units

Critical Velocity: $V_c := K_u \cdot \left(\frac{y}{ft}\right)^{\frac{1}{6}} \cdot \left(\frac{D_{50}}{ft}\right)^{\frac{1}{3}} \cdot \frac{ft}{sec} = 3.22 \cdot \frac{ft}{sec}$

Design Flow Velocity: $V := 2.95 \frac{ft}{sec}$

Scour Evaluation Type:

$$Scour_{type} := \begin{vmatrix} \text{"Clear-water"} \text{ if } V_c > V \\ \text{"Live-bed"} \text{ if } V > V_c \end{vmatrix} = \text{"Clear-water"}$$

Clear-Water Contraction Scour:

Unit Factor: $K_u := 0.0077$ For English units

Particle Size: $D_m := 1.25 \cdot D_{50} = 3.611 \cdot mm$

Bottom Width of Contracted Section: $W := 29ft$

Flow in Contracted Channel: $Q_2 := 644.10 \frac{ft^3}{sec}$

Average Depth in Contracted Section:

$$y_2 := \left[\frac{K_u \cdot \left(\frac{Q_2}{\frac{ft^3}{sec}} \right)^2}{\left(\frac{D_m}{ft} \right)^{\frac{2}{3}} \cdot \left(\frac{W}{ft} \right)^2} \right]^{\frac{3}{7}} \cdot ft = 6.29 \cdot ft$$

Contraction Scour Depth: $y_{sc100} := y_2 - 3.51ft = 2.78 \cdot ft$

LOCAL SCOUR AT ABUTMENT 1 AND 2

Local scour is not present for Q100 at abutment 1 and 2 due to no flow in the overbanks.

TOTAL SCOUR AT ABUTMENT 1 AND 2

Since local scour is not present at Q100, total scour is the contraction scour.

Q500 SCOUR EVENT:

CONTRACTION SCOUR

Determine Type of Scour:

Average Depth of Flow Upstream: $y := 7.26ft$ From x-section 461.06

Particle Size for Vc: $D_{50} = 2.889 \cdot mm$ From geotechnical report

Unit Factor: $K_u := 11.17$ For English units

Critical Velocity: $V_c := K_u \cdot \left(\frac{y}{ft}\right)^{\frac{1}{6}} \cdot \left(\frac{D_{50}}{ft}\right)^{\frac{1}{3}} \cdot \frac{ft}{sec} = 3.29 \cdot \frac{ft}{sec}$

Design Flow Velocity: $V := 3.11 \frac{ft}{sec}$

Scour Evaluation Type: $Scour_{type} := \begin{array}{|l} \text{"Clear-water" if } V_c > V \\ \text{"Live-bed" if } V > V_c \end{array} = \text{"Clear-water"}$

Clear-Water Contraction Scour:

Unit Factor: $K_u := 0.0077$ For English units

Particle Size: $D_m := 1.25 \cdot D_{50} = 3.611 \cdot mm$

Bottom Width of Contracted Section: $W := 29ft$

Flow in Contracted Channel: $Q_2 := 852.70 \frac{ft^3}{sec}$

Average Depth in Contracted Section:

$$y_2 := \left[\frac{K_u \cdot \left(\frac{Q_2}{\frac{ft^3}{sec}}\right)^2}{\left(\frac{D_m}{ft}\right)^{\frac{2}{3}} \cdot \left(\frac{W}{ft}\right)^2} \right]^{\frac{3}{7}} \cdot ft = 8 \cdot ft$$

Contraction Scour Depth: $y_{sc500} := y_2 - 4.07ft = 3.93 \cdot ft$

LOCAL SCOUR AT ABUTMENT 1

Froehlich's Live-Bed Abutment Scour *HEC-18: 8.6.1*

Abutment Shape Factor: $K_1 := 0.82$ Vertical Wall with Flared Wingwalls

Abutment Angle to Flow: $\theta := 90$

Angle of Embankment to Flow Factor: $K_2 := \left(\frac{\theta}{90}\right)^{0.13} = 1.00$

Length of Active Flow Obstruction: $L_i := 5.03ft$ from x-section 461.06

Flow Area of Embankment: $A_e := 5.28ft^2$ from x-section 461.06

Flow Obstructed by Embankment: $Q_e := 3.40 \frac{ft^3}{sec}$ from x-section 461.06

Velocity at Obstructed Flow: $V_e := 0.64 \frac{ft}{sec}$ from x-section 461.06

Length of Embankment normal to flow: $L := 5.03ft$

Average Flow Depth in Floodplain: $y_a := \frac{A_e}{L} = 1.05 \cdot ft$

Froude Number: $Fr := \frac{V_e}{(g \cdot y_a)^{0.5}} = 0.11$

Gravity: $g = 32.2 \cdot \frac{ft}{sec^2}$

Scour Depth: $y_{s\_FR} := \left[2.27 \cdot K_1 \cdot K_2 \cdot \left(\frac{L_i}{y_a}\right)^{0.43} \cdot Fr^{0.61} + 1\right] \cdot y_a = 2.05 \cdot ft$

HIRE Live-Bed Abutment Scour

HEC-18: 8.6.2

Abutment Shape Factor: $K_1 = 0.82$ Vertical Wall with Flared Wingwalls

Angle of Embankment to Flow Factor: $K_2 = 1.00$

Length of Embankment normal to flow: $L = 5.03 \cdot ft$

Average Flow Depth in Floodplain: $y_1 := 1.05ft$ from HEC-RAS x-section 461.06

Froude Number: $Fr := \frac{V_e}{(g \cdot y_1)^{0.5}} = 0.11$

Gravity: $g = 32.2 \cdot \frac{ft}{sec^2}$

Scour Depth: $y_{s\_HIRE} := \left(4 \cdot Fr^{0.33} \cdot \frac{K_1}{0.55} \cdot K_2\right) \cdot y_1 = 3.02 \cdot ft$

Abutment Scour

Determine Equation:

$$y_{sL5001} := \begin{cases} y_{s\_FR} & \text{if } \frac{L}{y_a} \le 25 \\ y_{s\_HIRE} & \text{otherwise} \end{cases} = 2.05 \cdot ft$$

LOCAL SCOUR AT ABUTMENT 2

Froehlich's Live-Bed Abutment Scour *HEC-18: 8.6.1*

Abutment Shape Factor: $K_1 := 0.82$ Vertical Wall with Flared Wingwalls

Abutment Angle to Flow: $\theta := 90$

Angle of Embankment to Flow Factor: $K_2 := \left(\frac{\theta}{90}\right)^{0.13} = 1.00$

Length of Active Flow Obstruction: $L_i := 17.27\,ft$ from x-section 461.06

Flow Area of Embankment: $A_e := 17.23\,ft^2$ from x-section 461.06

Flow Obstructed by Embankment: $Q_e := 11.30\,\frac{ft^3}{sec}$ from x-section 461.06

Velocity at Obstructed Flow: $V_e := 0.66\,\frac{ft}{sec}$ from x-section 461.06

Length of Embankment normal to flow: $L := 17.27\,ft$

Average Flow Depth in Floodplain: $y_a := \frac{A_e}{L} = 1.00 \cdot ft$

Froude Number: $Fr := \frac{V_e}{(g \cdot y_a)^{0.5}} = 0.12$

Gravity: $g = 32.2 \cdot \frac{ft}{sec^2}$

Scour Depth: $y_{s\_FR} := \left[2.27 \cdot K_1 \cdot K_2 \cdot \left(\frac{L_i}{y_a}\right)^{0.43} \cdot Fr^{0.61} + 1\right] \cdot y_a = 2.70 \cdot ft$

HIRE Live-Bed Abutment Scour *HEC-18: 8.6.2*

Abutment Shape Factor: $K_1 = 0.82$ Vertical Wall with Flared Wingwalls

Angle of Embankment to Flow Factor: $K_2 = 1.00$

Length of Embankment normal to flow: $L = 17.27 \cdot ft$

Average Flow Depth in Floodplain: $y_1 := 1.00ft$ from HEC-RAS x-section 461.06

Froude Number: $Fr := \frac{V_e}{(g \cdot y_1)^{0.5}} = 0.12$

Gravity: $g = 32.2 \cdot \frac{ft}{sec^2}$

Scour Depth: $y_{s\_HIRE} := \left(4 \cdot Fr^{0.33} \cdot \frac{K_1}{0.55} \cdot K_2\right) \cdot y_1 = 2.93 \cdot ft$

Abutment Scour

Determine Equation:

$$y_{sL5002} := \begin{cases} y_{s\_FR} & \text{if } \frac{L}{y_a} \le 25 \\ y_{s\_HIRE} & \text{otherwise} \end{cases} = 2.70 \cdot ft$$

<u>TOTAL SCOUR AT ABUTMENT 1 AND 2</u>

Abutment 1: $y_{s1} := y_{sc500} + y_{sL5001} = 5.98 \cdot ft$

Abutment 2: $y_{s2} := y_{sc500} + y_{sL5002} = 6.64 \cdot ft$

APPENDIX H

Site Photographs

Photo 1 – Cape Neddick River – Looking Downstream

Photo 2 – Cape Neddick River – Looking Upstream

Photo 3 – Route 1 – South Approach

Photo 4 – Route 1 – North Approach

Photo 5 – Cape Neddick Bridge – Downstream Face of Bridge

Photo 6 – Cape Neddick Bridge – Looking Downstream

Photo 5 – Cape Neddick Bridge – Downstream Face of Bridge after FOR

Photo 6 – Cape Neddick Bridge – Downstream Face of Bridge after FOR