PRELIMINARY DESIGN REPORT

PENOBSCOT RIVER BRIDGE (#2660)
OVER
PENOBSCOT RIVER

Howland-Enfield, Maine

Federal PIN: # BH-1670(500)X

WIN: # 16705.00

July 2013

Submitted To:
Maine Department of Transportation

Engineer:
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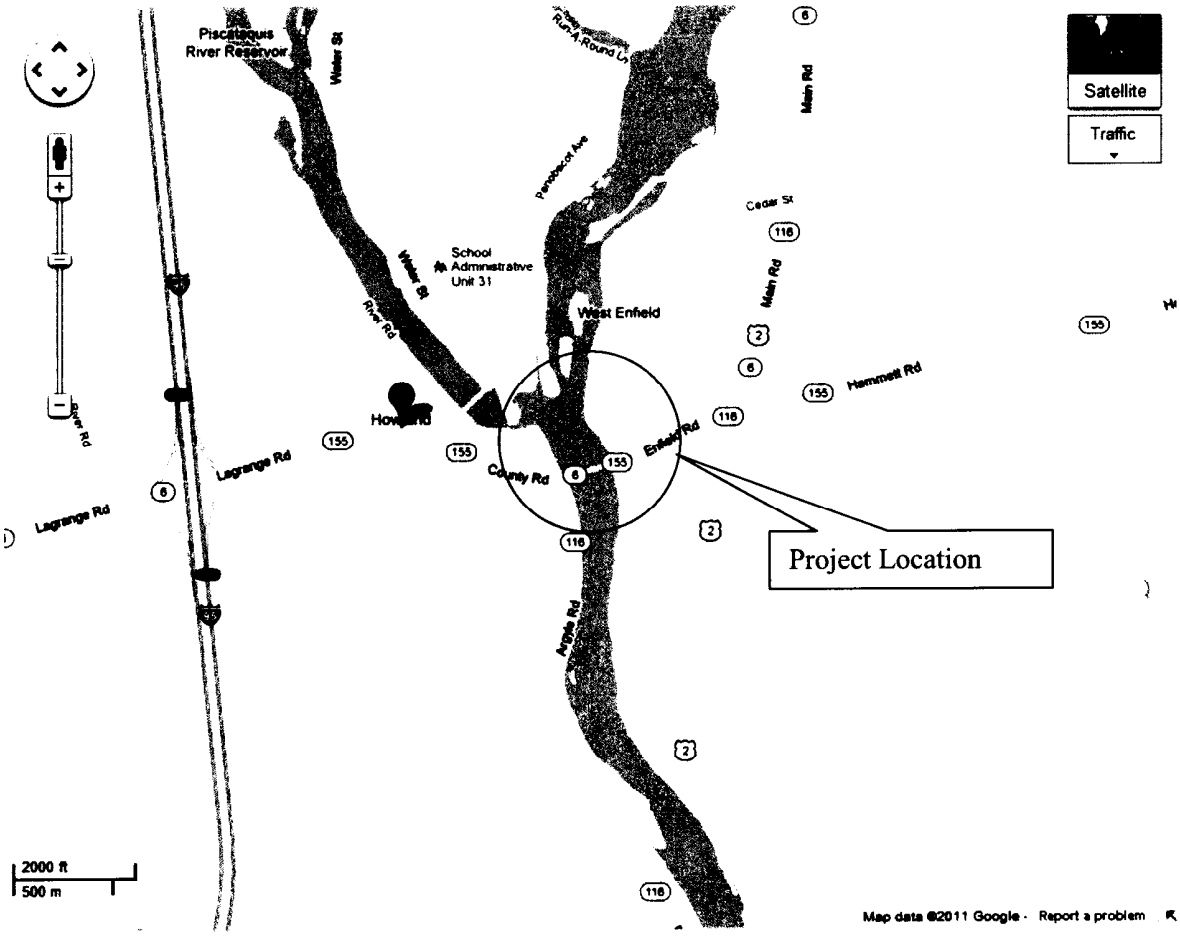
12 Northbrook Road
Building A, Suite One
Howland - Enfield, Maine 04105

Contact Person:
Rick Hebert, P.E.

Penobscot River Bridge #2660
Howland - Enfield, Maine

D. LOCATION MAP

Howland – Enfield, Penobscot River Bridge #2660, WIN 16705.00
Route 155/6 over the Penobscot River



Latitude: 45° 14' 11" N, Longitude: 68° 39' 06" W

Penobscot River Bridge #2660
Howland - Enfield, Maine

I. HYDROLOGY / HYDRAULICS / SCOUR REPORT

Hydrology:
Design flows for the Q500, Q100, Q50, and Q10 discharges were taken from the 1996 Federal Emergency Management Agency Flood Insurance Study for the Town of Howland as reported for site location (at State Routes 6 and 155). Design flows for the Q25 and Q1.1 discharges (derived using peak flow calculations by USGS Regression Equations) were provided by the Hydraulics Section of the Maine DOT:

Design Discharges

$Q_{1.1}$ = 46,216 ft^3/s
Q_{10} = 98,400 ft^3/s
Q_{25} = 122,900 ft^3/s
Q_{50} = 135,300 ft^3/s
Q_{100} = 148,800 ft^3/s
Q_{500} = 186,800 ft^3/s

Watershed Area = 6671 mi^2
Wetlands Area = 810 mi^2

The design discharges were compared to the flow-frequency statistical analysis at USGS gage at West Enfield (same location), as published in the USGS publication Estimating the Magnitude and Frequency of Peak Flows for Streams in Maine for Selected Recurrence Intervals, WRI99-4008.

Q_{10} = 99,864 ft^3/s
Q_{25} = 119,272 ft^3/s
Q_{50} = 135,740 ft^3/s
Q_{100} = 148,208 ft^3/s
Q_{500} = 182,084 ft^3/s

The reported discharges compared favorably and were not significantly different.

Hydraulics:
The water surface profile was developed for steady flow at the peak discharge for the following design flows using HEC-RAS:

$Q_{1.1}$, ordinary high water
Q_{10}
Q_{25}
Q_{50}, design discharge
Q_{100}, check discharge
Q_{500}, super flood discharge used to evaluate scour

Specific river sections were requested by T.Y. Lin International and surveyed and provided by the Maine DOT Survey Section. Geometric data for the existing bridge was taken from the existing bridge plans (see Section O). All elevations were referenced to the project datum (NAVD 1988). Geometric data for the proposed bridge was taken from the preliminary design and plans (see Section J). Other input data was obtained through field observations and research conducted for the preliminary design.

Reach boundary conditions were established using water surface profile data from the 1996 FEMA Flood Insurance Study (FIS). The model for the existing bridge was checked for convergence through comparison with the flood profile developed for the FEMA FIS through the modeled reach.

The proposed bridge will not increase backwater depth in the river. The existing and proposed bridges are located at different locations. The existing bridge intersects the channel at modeled River Sta 100+00. The proposed bridge (Alternate 5) intersects the channel at River Sta 99+10, approximately 90 feet downstream of the existing bridge. To assess the difference in backwater influences for the two bridges, the headwater elevations for the river sections located nearest the upstream of the two bridges (River Sta 100+20 and River Sta 99+80) respectively were compared as follows (assuming the existing bridge is removed in its entirety):

Headwater Elevation At Existing Bridge RS 100+20:

Discharge	Existing	Proposed
$Q_{1.1}$	133.5 ft	133.5 ft
Q_{10}	143.8 ft	143.8 ft
Q_{25}	146.1 ft	146.2 ft
Q_{50}	147.7 ft	147.8 ft
Q_{100}	149.2 ft	149.2 ft
Q_{500}	152.7 ft	152.8 ft

Headwater Elevation At Proposed Bridge RS 99+80 (Alternate 5):

Discharge	Existing	Proposed
$Q_{1.1}$	133.5ft	133.4ft
Q_{10}	143.7 ft	143.7 ft
Q_{25}	146.1 ft	146.1 ft
Q_{50}	147.7 ft	147.7 ft
Q_{100}	149.1 ft	149.1 ft
Q_{500}	152.7 ft	152.7 ft

The differences in backwater depth for all discharges at both locations are not significant.

Discharge velocities at the proposed bridge RS 99+35 are as follows:

Discharge	Proposed
$Q_{1.1}$	5.6 ft/s
Q_{25}	5.7 ft/s
Q_{50}	6.4 ft/s
Q_{50}	6.4 ft/s
Q_{100}	6.8 ft/s
Q_{500}	7.4 ft/s

Discharge velocities at the proposed bridge location are similar and slightly lower than at the existing bridge location.

The low chords for both the existing and proposed bridge are sloped. The lowest low chord elevation of the existing bridge (Elev. 153.1) occurs at the west abutment end of the bridge, and at this location the lowest low chord provides 5.4 feet of clearance above the Q_{50} elevation. The average low chord elevation of the existing bridge (Elev. 157.7) provides an average of 10.0 ft of clearance above the Q_{50} elevation. The low chord of the proposed bridge is both sloped and haunched. The lowest low chord elevation of the proposed bridge (Elev. 151.7) occurs at Abutment 1, and at this location the lowest low chord is 4.0 feet above the Q_{50} elevation, 2.5 feet above the Q_{100} elevation, and 1.3 feet above the 1923 Flood of Record (Elev. 150.4). The average low chord elevation of the proposed bridge provides an average of approximately 7.5 feet of clearance above the Q_{50} elevation. The existing bridge meets the desired minimum freeboard requirements specified in the MaineDOT Bridge Design Guide. The proposed bridge also meets the desired minimum freeboard requirements specified in the MaineDOT Bridge Design Guide (low chord clearance over 4 feet at the Q_{50} design discharge and low chord above the Q_{100} and Flood of Record 1923 discharge elevations).

The hydraulic conditions for the proposed new bridge (Alternate 5) will be improved in comparison to the existing bridge. The obstruction of flow caused by the existing bridge is greater than the proposed bridge due to the difference in the number of piers. The existing bridge has four piers located in the channel, and the proposed bridge will have three piers located in the channel.

Refer to Section P for additional detailed hydraulic data for the existing bridge and for the replacement bridge (Alternate 5).

Scour:
The proposed bridge will be designed for scour in accordance with the requirements of the Maine DOT Bridge Design Guide, AASHTO LRFD Bridge Design Specifications, Hydraulic Engineering Circular No. 18 (HEC-18), Evaluating Scour at Bridges (Fifth Edition), Hydraulic Engineering Circular No. 20 (HEC-20), Stream Stability at Highway Structures (Third Edition), and Hydraulic Engineering Circular No. 23 (HEC-23), Bridge Scour and Stream Instability Countermeasures (Third Edition).

The bridge is located on a near straight reach of the Penobscot River approximately 1000 ft downstream of the confluence of the Piscataquis River. Upstream of the confluence, the Penobscot River bends moderately to the northeast. A dam is located on the Piscataquis River immediately upstream of the confluence with the Penobscot River, and another dam is located on the Penobscot River approximately 1.5 miles upstream of the bridge. The channel is straight, non-braided, and equiwidth. The Penobscot River flows through a low relief valley and has a narrow floodplain (less than 10 times the channel width). The channel is probably incised with semi-alluvial channel boundaries. The banks are high and generally well vegetated and stable. Geomorphic factors indicate the Penobscot River is laterally stable within the bridge reach with low potential for lateral migration.

The scour evaluations include the influences of aggradation and degradation, contraction scour, local abutment scour and local pier scour. The evaluation also considered the influence of debris accumulation. The removal of the upstream dams on the Penobscot and Piscataquis Rivers has not been included in the hydraulics and scour analysis. Although the dams may

serve to mitigate flow, storage capacity is not significant and at high flows run of river conditions are anticipated and the dams are located upstream of the bridge reach influence.

Influences from aggradation or degradation need not be considered. This conclusion is based on observed site conditions, a review of the existing bridge plans, bridge maintenance and inspection records, and the fact that activities or conditions that could influence aggradation or degradation are not present within the reach of influence for the river.

Flow velocity and flow distribution information for the scour evaluations were obtained from the hydraulic analysis HEC-RAS model. Geotechnical parameters used in the scour evaluations were obtained from review of the Geotechnical Data Report prepared by Maine DOT dated December 30, 2010 (see Section J). This subsurface data was obtained near the existing bridge crossing location, but conditions were assumed to be similar at the proposed downstream crossing location. The three borings taken within the river channel indicate the upper layer of the channel bed consist of gravel with a thickness ranging from 4' to 17.5' below the channel bed. The D_{50} grain size for this upper layer material ranged from ~5 – 10 mm. Below the gravel substrate is a layer of sand with variable thickness ranging from 18' to 23.5. The D_{50} grain size for this deeper layer material ranged from ~0.5 – 1.1 mm. Below the sand within the east half of the channel there is a variable depth layer of silt. All materials down to bedrock are considered highly scourable. A D50 grain size of 5mm was used for the contraction scour computations, and a D_{84} grain size of 7mm was used for the evaluation of bed roughness (k_s) for the complex pier scour computations.

Scour evaluations were prepared for the Q_{100} and Q_{500} design flows. Contraction, local abutment, and local pier scour computations were performed in accordance with guidelines provided in HEC-18. Influences of debris accumulation were not considered significant because these is not a history of significant debris accumulation at the existing bridge, spans are long and the bridge does not significantly restrict the river channel flow, the piers will be well aligned with the flow, and because the historic performance of the proposed pier type with a sharp angled upstream nose in similar applications.

The crossing is only slightly susceptible to contraction scour. The following factors contribute to the crossing's vulnerability to contraction scour:

- Relative restriction of channel flow caused by approach embankments, abutments, and piers.
- The fineness of the bed material.

Contraction scour values are noted in the Scour Evaluation table below. The preliminary design of the substructures and foundations considers the influence of contraction scour. Proposed riprap in front of abutments will be embedded a minimum of 1 foot below the streambed elevation including contraction scour. The top of the pier pile caps will be located below the contraction scour surface elevation.

Stub abutments founded on deep pile foundations to bedrock are proposed for the recommended bridge replacement (Alternate 5). Disturbed slopes below the Q_{50} elevation will be riprapped in accordance with the requirements of the Maine DOT Bridge Design Guide and HEC-23.

The following factors contribute significantly to the vulnerability of the evaluated abutments to scour:

- Foundation type (piles to bedrock).
- Encroachment of the abutment and approach slopes within the river channel.

Abutment scour values are noted in the Scour Evaluation table below. Abutment scour computations are computed in accordance with the NCHRP Method described in HEC-18 and include the contraction scour component. Abutment scour computations are generally conservative, but for the given site conditions these values should be considered reasonable. A three foot thick plain riprap apron is recommended to protect the abutment slopes. Upstream of the bridge the approach riprap will extend as required to protect the embankment from unimpeded flow along the approach embankments. Downstream of the bridge, riprap will extend a minimum of 5 feet beyond the top of bank. Following the current recommendations of HEC-23, the riprap slope will extend 25 ft along the downstream side approach embankment slope, and the riprap apron will extend 25 ft from the toe of the abutment embankment slope. The edge of the riprap will be embedded a minimum of 1 foot below the channel contraction scour depth in accordance with MaineDOT Standard Details. The top of the riprap slope protection will be above the Q_{50} elevation.

The proposed piers for the replacement bridge alternate are evaluated for two local pier scour conditions. The first condition (known as simple local pier scour) assumes that the top of the pile cap will be located below the combined contraction and local pier scour depth. The second condition assumes that the top of the pile cap will be located below the contraction scour depth. The second condition will allow local pier scour to expose the pile cap and cofferdam seal. This condition is known as "complex local pier scour", and is evaluated using the "Superposition of Scour Components Method" outlined in HEC 18. For the complex scour condition, the bottom of the seal will be located a minimum of 2 feet below the scour depth.

Pier scour evaluations consider combined contraction and complex local pier scour. The top of the pile cap will be located 3 ft below the riverbed to keep it below the 100-yr event contraction scour depth and to allow the footing to be buried below the bottom of the 3 foot thick plain riprap used to backfill the top portion of the cofferdam. The bottom of the seal will need to be located a minimum of 7 ft below the riverbed to maintain at least 2 feet of cover over the bottom of the seal under the Q_{100} event. Considering the 5 ft thick footing depth, the minimum required seal depth for scour is 2 ft.

The seal depth requirements also consider the construction requirements for the cofferdam, and the hydrostatic buoyancy force governs the thickness for the seal. The minimum seal thickness required for the Q_{25} event is 24.5 ft. The minimum seal thickness required for the Q_{10} event is 23 ft. These thicknesses were judged to be too costly. A minimum seal thickness of 17 ft is selected for the development of the estimated construction cost, and this thicknesses provides adequate embedment below the Q_{100} combined contraction and local complex pier scour depth and provides for construction of the top of the cofferdam 2.5 ft above the $Q_{1.1}$ event. The bottom of the seal considering the combined contraction and complex local pier scour will not expose the bottom of the seal for the Q_{100} and Q_{500} discharges. A modified seal size may be considered during final design if a higher cofferdam overtopping elevation is desired, but the proposed 2.5 ft above the $Q_{1.1}$ event is considered

reasonable for the development of comparative costs for the alternates considered for this study.

Results of the Scour Evaluation for Bridge Replacement Alternate 5 for the Q_{100} flow are as follows:

Scour Components:	Abutment 1	Pier 1	Pier 2	Abutment 2
Contraction Scour (ft)	0.2	0.2	0.2	0.2
Abutment Scour (ft)*	12.1	-	-	12.1
Local Pier Scour (ft)	-	4.8 **	4.8**	-
Total Scour (ft)	12.1	5.0	5.0	12.1
Ground Elevation (ft)	-	~123	~115.5	-
Scour Elevation (ft)	***	118.0	110.5	***

* Computed abutment scour includes pier scour component.
** "Complex local pier scour"
*** Abutments will be designed assuming no ground support (lateral or vertical) as a result of soil loss due to contraction scour and properly designed riprap revetment will be provided to protect abutments in accordance with HEC-18 Section 7.6. Design of the foundations with the abutment scour prism removed is not required.

Results of the Scour Evaluation for Bridge Replacement Alternate 5 for the Q_{500} flow are as follows:

Scour Components:	Abutment 1*	Pier 1	Pier 2	Abutment 2*
Contraction Scour (ft)	0.3	0.3	0.3	0.3
Local Abutment Scour (ft)	14.0	-	-	14.0
Local Pier Scour (ft)	-	4.9 **	4.9**	-
Total Scour (ft)	12.1	5.2	5.2	12.1
Ground Elevation (ft)	-	~123	~115.5	-
Scour Elevation (ft)	***	117.8	110.3	***

* Computed abutment scour includes pier scour component.
** "Complex local pier scour"
*** Abutments will be designed assuming no ground support (lateral or vertical) as a result of soil loss due to contraction scour and properly designed riprap revetment will be provided to protect abutments in accordance with HEC-18 Section 7.6. Design of the foundations with the abutment scour prism removed is not required.

All three piers will be supported on a pile cap and founded on steel H-pile driven to refusal. A cofferdam and concrete seal will be required to construct the piers. The inside of the cofferdam will be backfilled with granular borrow and the top 3 feet will be backfilled with stone fill / plain riprap to match the elevation of the existing surrounding streambed.

Stub abutments founded on deep pile foundations to bedrock with 1.75H:1V plain riprap slopes will be used. Disturbed slopes below the Q_{50} elevation will be riprapped in accordance with the recommendations of the Maine DOT Bridge Design Guide and HEC-23.

Additional geotechnical explorations will be required during the final design phase to confirm assumed subsurface and streambed conditions at the recommended bridge crossing

location. Foundation types and designs and scour computations should be updated during final design based on the findings of the supplemental subsurface investigations.

Refer to Section P for additional detailed scour data for the replacement bridge (Alternate 5).

J. PRELIMINARY PLANS

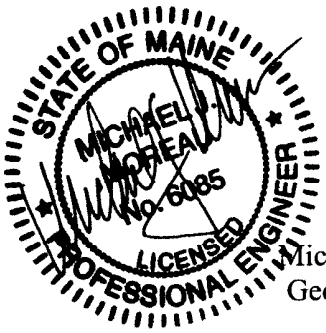
Preliminary Plans (13sheets) are provided for the recommended Alternate 5 - Bridge Replacement, Four Span (940') Haunched Steel Girder Bridge on Parallel Downstream Tangent Alignment C with Curved End Spans Located 100 ft Downstream of Existing.

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DATA REPORT

For:

PENOBSCOT RIVER BRIDGE
ROUTES 6 AND 155 OVER PENOBSCOT RIVER
HOWLAND-ENFIELD, MAINE



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Penobscot County
PIN 16705.00

Soils Report 2010-37
Bridge No. 2660

Fed No. BH-1670(500)X
December 30, 2010

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1.0 INTRODUCTION

We show the Penobscot River Bridge that carries Routes 6 and 155 over the Penobscot River between Howland and Enfield, Maine, on Sheet 1, Site Location Map found at the end of this data report. The purpose of our subsurface investigation is to provide subsurface condition data along the existing bridge alignment to help assess feasibility and alternatives for bridge replacement along that alignment.

The existing simply supported five-span steel through truss was originally built in 1896, re-built in 1934, and widened and replaced in 1941. The original granite masonry abutments were founded on steep sloping bedrock at the banks. The 1941 bridge replacement plans indicate that the original abutments were underpinned and the concrete abutment footing extensions were founded on bedrock. The piers are founded on timber crib footings bearing on native soil. The original stone piers were extended at both ends with concrete founded on timber cribbing in 1941. Riprap was placed in front of the abutments and around the pier widening.

A 1995 scour assessment noted scattered riprap and partial footing exposure at piers 1 and 2, and missing riprap at piers 3 and 4. MaineDOT performed an underwater inspection in 2001 and found partial foundation exposure at piers 1, 2, and 3. Scour studies have indicated that pier 2 may be totally undermined during a 100 year storm event.

2.0 GEOLOGIC SETTING

The Maine Geologic Survey "Surficial Geology of Passadumkeag Quadrangle, Maine, Open-file No. 81-4" (1981) indicates that surficial soils in the vicinity of the Penobscot River Bridge consist primarily of stream alluvium deposits with nearby glacial-marine and glacial till soil unit contacts. The stream alluvium unit is typically made up of sand, gravel and silt. Glacial-marine deposits generally consist of silt, clay, and sand, commonly a clayey silt, but sand is abundant at the surface in some places. The glacial till is typically a heterogeneous mixture of sand, silt, clay, and stones.

According to the Maine Geologic Survey Bedrock Geologic Map of Maine (1985), the bedrock at the Penobscot River Bridge site consists of Silurian-Ordovician, calcareous sandstone, interbedded sandstone and impure limestone. Locally the bedrock has been identified as phyllite and metasiltstone and is part of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION PROGRAM

We investigated subsurface conditions in the vicinity of the existing bridge by drilling five test borings, BB-HEPR-101 through BB-HEPR-105. The approximate boring locations and coordinates are shown on Sheet 2 and Sheet 3, Boring Location Plan, found at the end of this report. We terminated all of the borings with bedrock cores. The Maine Department of Transportation (MaineDOT) drill rig and crew and the contract drill crew, Northern Test

Borings of Gorham, Maine, conducted the borings on August 16 through August 19, 2010. We present the details and sampling methods used, field data obtained, and soil and groundwater conditions encountered in the boring logs in Appendix A provided at the end of this report.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. A consultant inspector or a MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions in the borings. The MaineDOT survey crew determined the boring location coordinates in the field after the borings were completed. The survey coordinates are based on the NAVD 88 datum.

The borings were drilled using solid stem auger and cased wash boring techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.84 and 0.678 to the raw field N-values obtained with the MaineDOT and Northern Test Boring drill rigs, respectively. Bedrock was cored using an NQ-2 core barrel producing a 2.0-inch diameter rock core.

4.0 LABORATORY TESTING PROGRAM

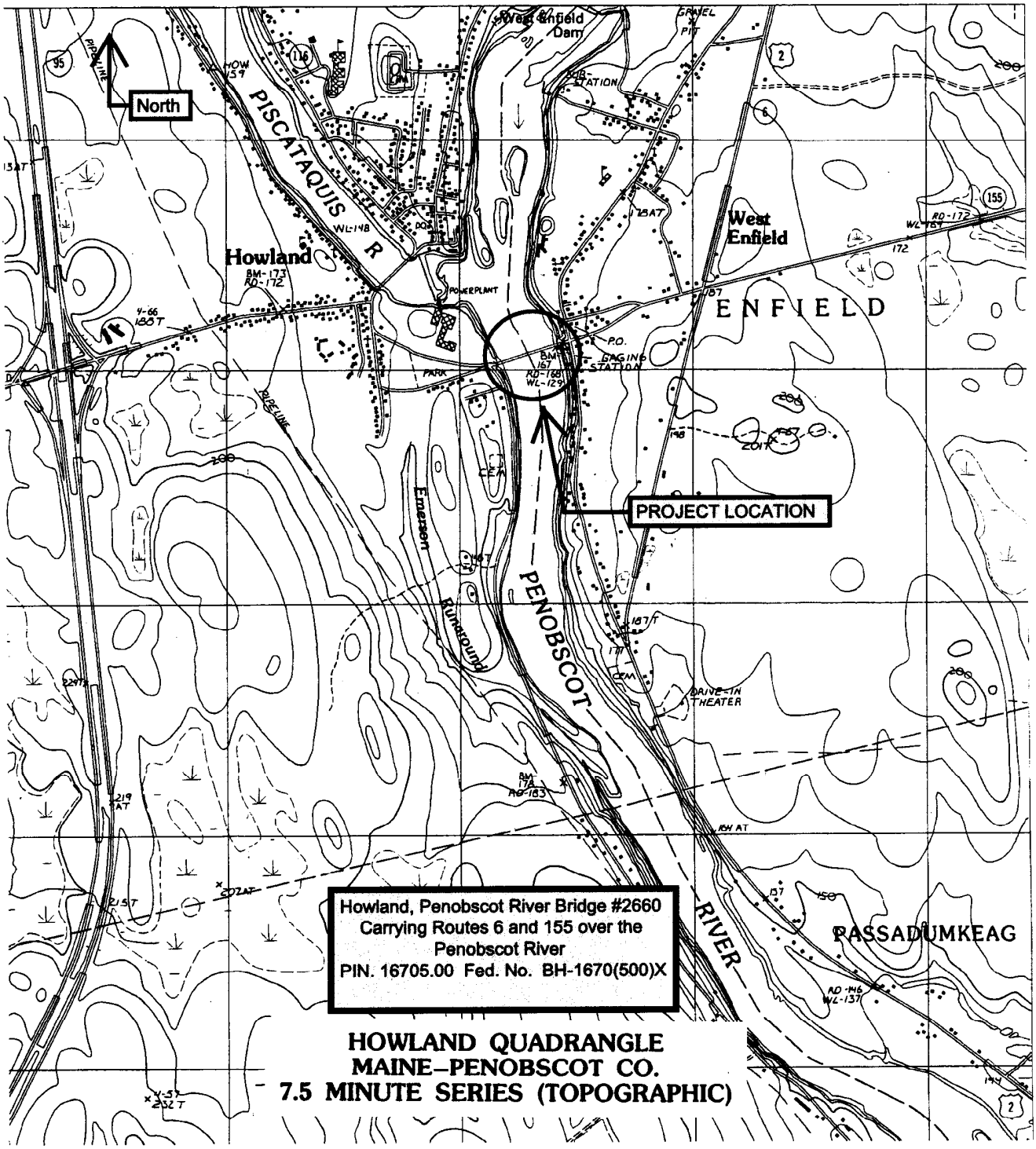
We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification and soil properties. We performed the soil laboratory testing at the AASHTO accredited MaineDOT Soils Laboratory in Bangor, Maine. Laboratory testing consisted of 21 standard grain size analyses with natural water content, one with hydrometer analysis, and one Atterberg Limits test. We present the results of the laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classification and water content data are also presented on the boring logs in Appendix A.

5.0 CLOSURE

This Geotechnical Data Report has been prepared for use by the MaineDOT Bridge Program for specific application to the replacement feasibility study for the Penobscot River Bridge carrying Routes 6 and 155 over the Penobscot River in Howland-Enfield, Maine. We have prepared the report in accordance with generally accepted soil and foundation engineering practices. No other intended use or warranty is expressed or implied.

The Department conducted a limited number of soil explorations at discrete locations near the existing bridge and a limited number of laboratory soil tests. The Department shall not be responsible for any bidder's or contractor's interpretations of, or estimates or conclusions drawn from, the geotechnical information. Data provided may not be representative of the subsurface conditions between the boring locations.

Sheets



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KILOMETERS 1 .5 0 1 2

METERS 1000 0 1000 2000

MILES 1 .5 0 1

FEET 1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10 000

CONTOUR INTERVAL 10 FEET

PROJ. MANAGER		BY	DATE
DESIGN-DETAILED	M.MORIAU	T.WHITE	DEC 2010
CHECKED-REVIEWED			
DESIGN-DETAILED2			
DESIGN-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1670(500)X

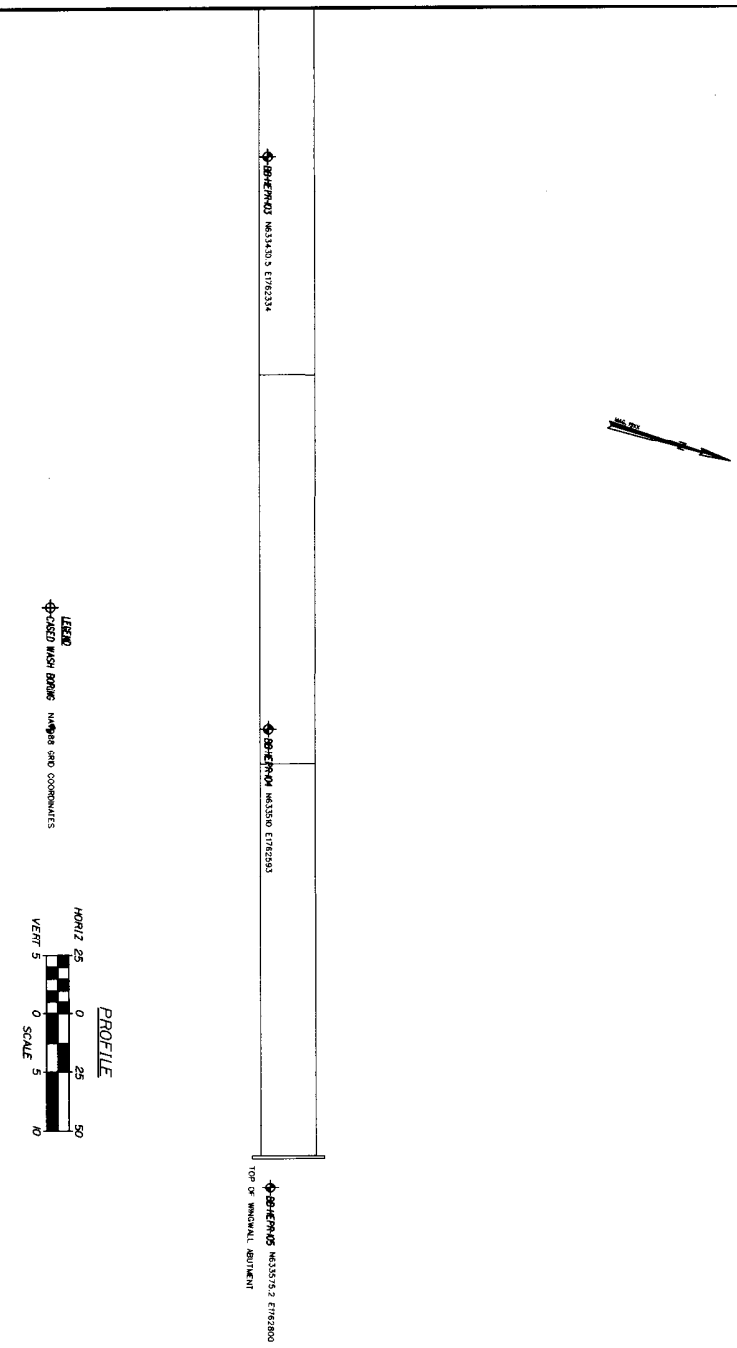
BRIDGE NO. 2600 PIN 16705.00 BRIDGE PLANS

PENOBSCOT RIVER BRIDGE
PENOBSCOT RIVER
HOWLAND PENOBSCOT COUNTY

BORING LOCATION PLAN

SHEET NUMBER 2 OF 3

Filename: ...\00\geotech\msta\006_BLPlan Division: GEOTECH Username: terry.white Date:12/21/2010



PENOBSCOT RIVER BRIDGE PENOBSCOT RIVER HOWLAND PENOBSCOT COUNTY	PROJ MANAGER		BY	DATE
	DESIGN-DETAILED	M.MOREAU	T.WHITE	DEC 2010
	CHECKED-REVIEWED			
	DESIGN2-DETAILED2			
	DESIGN3-DETAILED3			
	REVISIONS 1			
	REVISIONS 2			
	REVISIONS 3			
	REVISIONS 4			
	FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1670(500)X

BRIDGE NO. 2660 PIN 16706.00 BRIDGE PLANS

BORING LOCATION PLAN

SHEET NUMBER 3 OF 3

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Howland-Enfield Project Number: 16705.00

Boring & Sample Identification Number	Northing	Easting	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	Classification AASHTO	Classification Frost
	NAVD88										
BB-HEPR-101, 1D	633293	1761886	1.0-3.0	237550	1	3.9			SW-SM	A-1-b	0
BB-HEPR-101, 4D	633293	1761886	15.0-17.0	207064	1	21.8			SM	A-2-4	II
BB-HEPR-101, 6D	633293	1761886	25.0-25.8	207066	1	9.7			SM	A-2-4	II
BB-HEPR-102, 1D	633348.9	1762069	0.0-2.0	207067	1	10.2			GP	A-1-a	0
BB-HEPR-102, 4D	633348.9	1762069	16.0-18.0	207068	1	11.7			SW-SM	A-1-b	0
BB-HEPR-103, 1D	633430.5	1762334	0.0-2.0	207069	2	9.9			GW	A-1-a	0
BB-HEPR-103, 2D	633430.5	1762334	3.5-5.5	207070	2	7.7			GW	A-1-a	0
BB-HEPR-103, 5D	633430.5	1762334	18.5-20.5	207071	2	11.5			SW-SM	A-1-b	0
BB-HEPR-103, 7D	633430.5	1762334	28.5-30.0	207072	2	10.1			SW	A-1-a	0
BB-HEPR-103, 10D	633430.5	1762334	43.5-44.8	207073	2	7.9			SM	A-4	IV
BB-HEPR-104, 1D	633510	1762593	0.0-2.0	207074	3	10.1			GP	A-1-a	0
BB-HEPR-104, 2D	633510	1762593	7.0-8.33	207075	3	8.3			SM	A-1-b	II
BB-HEPR-104, 3D	633510	1762593	12.0-14.0	239827	3	15.1			SP-SM	A-1-b	0
BB-HEPR-104, 5D	633510	1762593	22.0-24.0	239828	3	10.3			SW-SM	A-1-a	0
BB-HEPR-104, 7D	633510	1762593	32.0-34.0	239829	3	11.6			ML	A-4	IV
BB-HEPR-104, 10D	633510	1762593	47.0-48.8	239830	3	8.4			SM	A-4	IV
BB-HEPR-105, 1D	633575.2	1762800	1.0-3.0	239831	4	2.7			SW-SM	A-1-b	0
BB-HEPR-105, 3D	633575.2	1762800	10.0-12.0	239832	4	7.5			SW-SM	A-1-b	0
BB-HEPR-105, 4D/B	633575.2	1762800	15.0-17.0	239833	4	24.1	33	12	CL	A-6	III
BB-HEPR-105, 5D	633575.2	1762800	20.0-22.0	239834	4	18.5			SM	A-2-4	II
BB-HEPR-105, 8D	633575.2	1762800	35.0-37.0	239835	4	9.1			SM	A-1-b	II

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

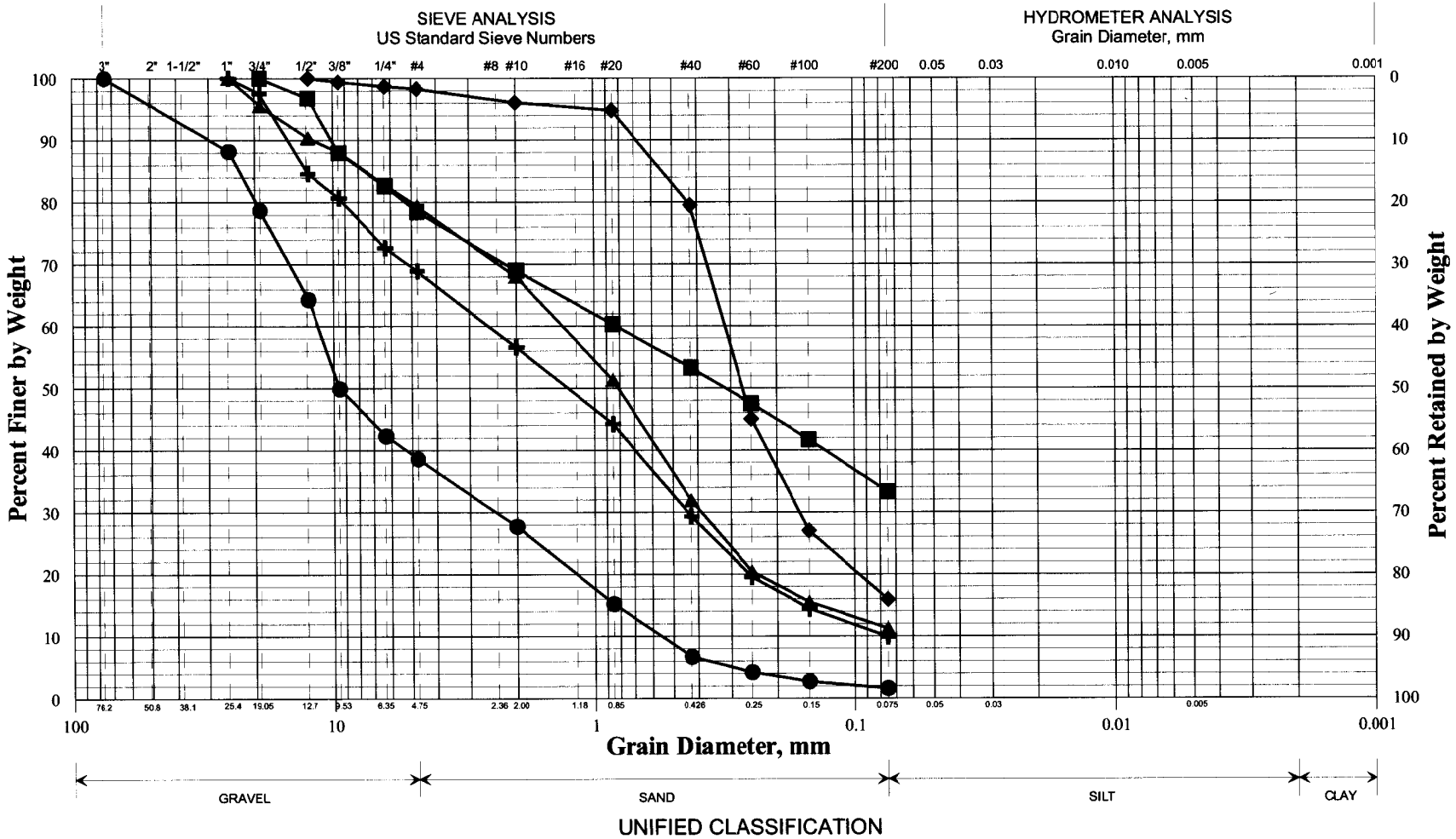
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

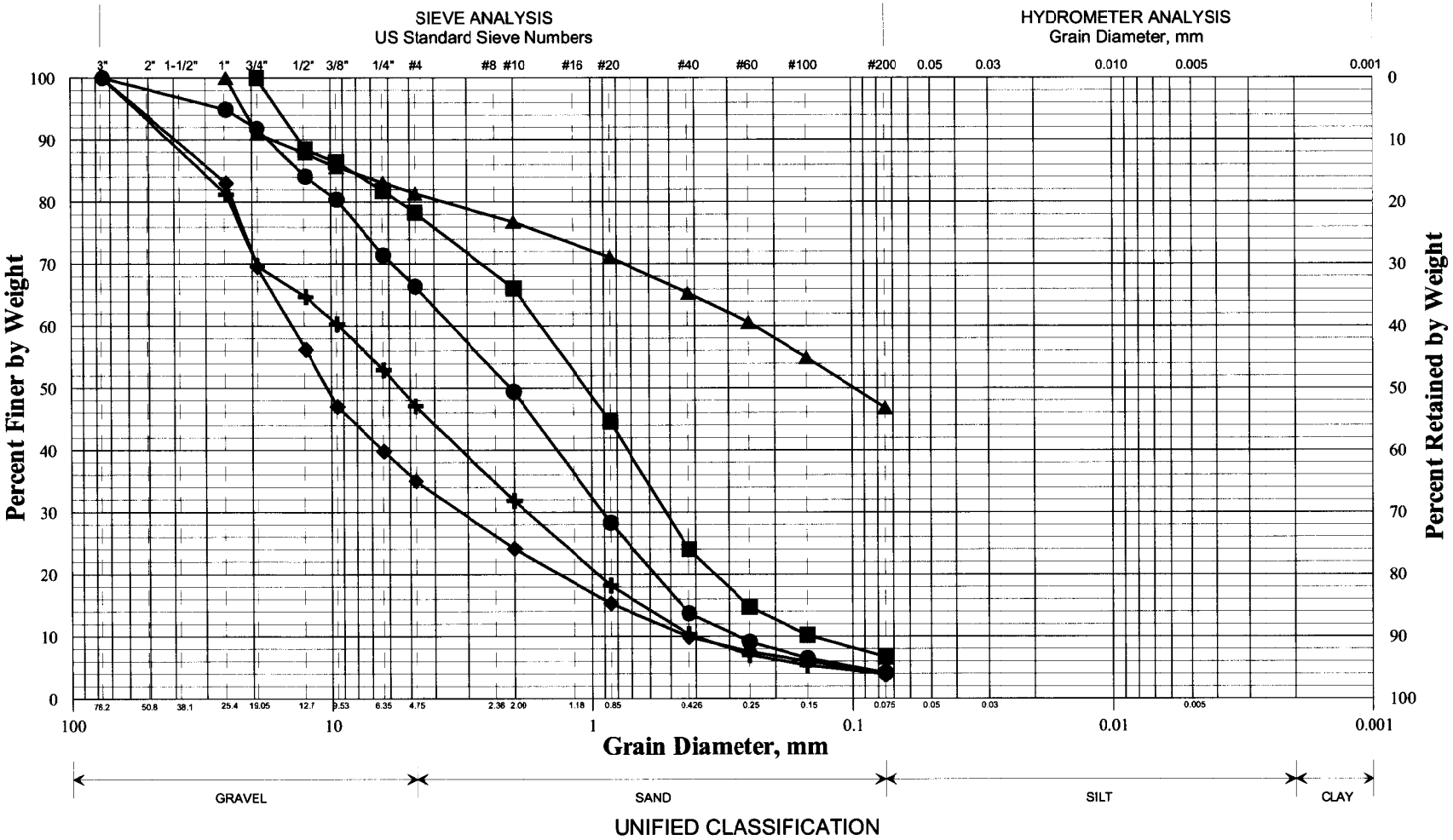


	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-101/1D	633293	1761886	1.0-3.0	SAND, some gravel, trace silt.	3.9			
◆	BB-HEPR-101/4D	633293	1761886	15.0-17.0	SAND, little silt, trace gravel.	21.8			
■	BB-HEPR-101/6D	633293	1761886	25.0-25.8	SAND, some silt, some gravel.	9.7			
●	BB-HEPR-102/1D	633348.9	1762069	0.0-2.0	Sandy GRAVEL, trace silt.	10.2			
▲	BB-HEPR-102/4D	633348.9	1762069	16.0-18.0	SAND, some gravel, little silt.	11.7			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



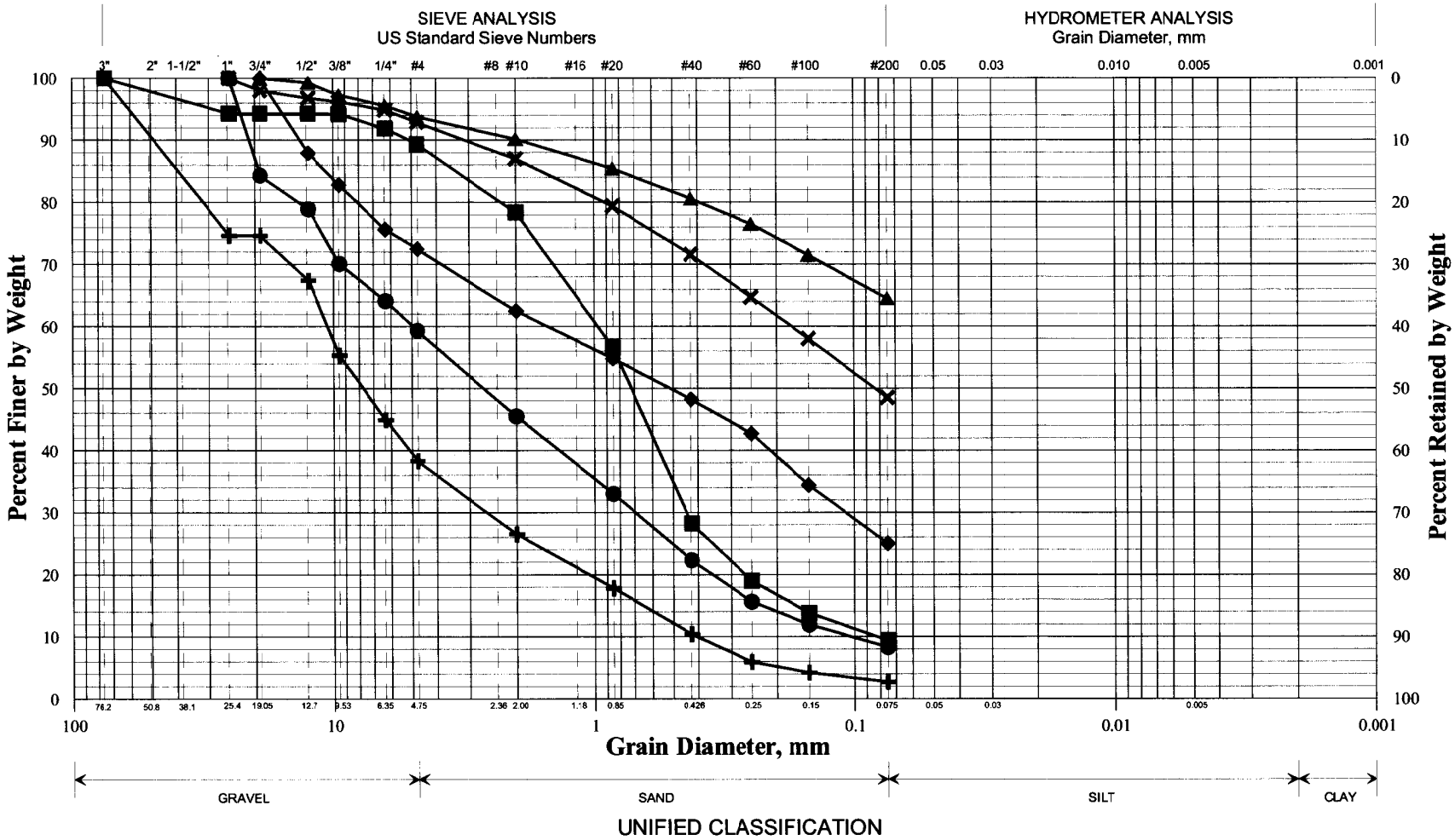
UNIFIED CLASSIFICATION

	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-103/1D	633430.5	1762334	0.0-2.0	Sandy GRAVEL, trace silt.	9.9			
◆	BB-HEPR-103/2D	633430.5	1762334	3.5-5.5	GRAVEL, some sand, trace silt.	7.7			
■	BB-HEPR-103/5D	633430.5	1762334	18.5-20.5	SAND, some gravel, trace silt.	11.5			
●	BB-HEPR-103/7D	633430.5	1762334	28.5-30.0	SAND, some gravel, trace silt.	10.1			
▲	BB-HEPR-103/10D	633430.5	1762334	43.5-44.8	SILT, some sand, little gravel.	7.9			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



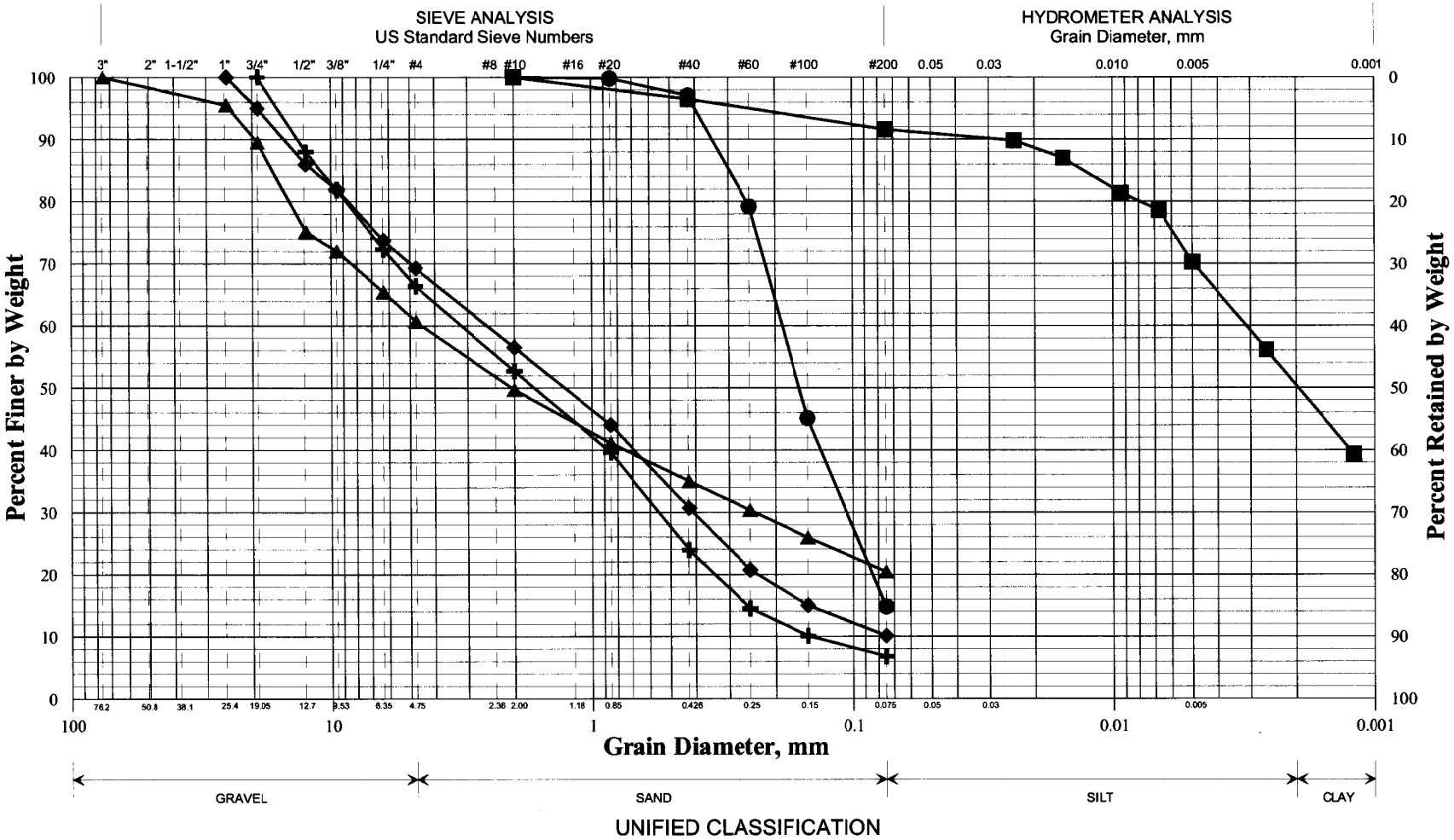
UNIFIED CLASSIFICATION

	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-104/1D	633510	1762593	0.0-2.0	GRAVEL, some sand, trace silt.	10.1			
◆	BB-HEPR-104/2D	633510	1762593	7.0-8.3	SAND, some gravel, some silt.	8.3			
■	BB-HEPR-104/3D	633510	1762593	12.0-14.0	SAND, little gravel, trace silt.	15.1			
●	BB-HEPR-104/5D	633510	1762593	22.0-24.0	Gravelly SAND, trace silt.	10.3			
▲	BB-HEPR-104/7D	633510	1762593	32.0-34.0	SILT, some sand, trace gravel.	11.6			
✕	BB-HEPR-104/10D	633510	1762593	47.0-48.8	Sandy SILT, trace gravel.	8.4			

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

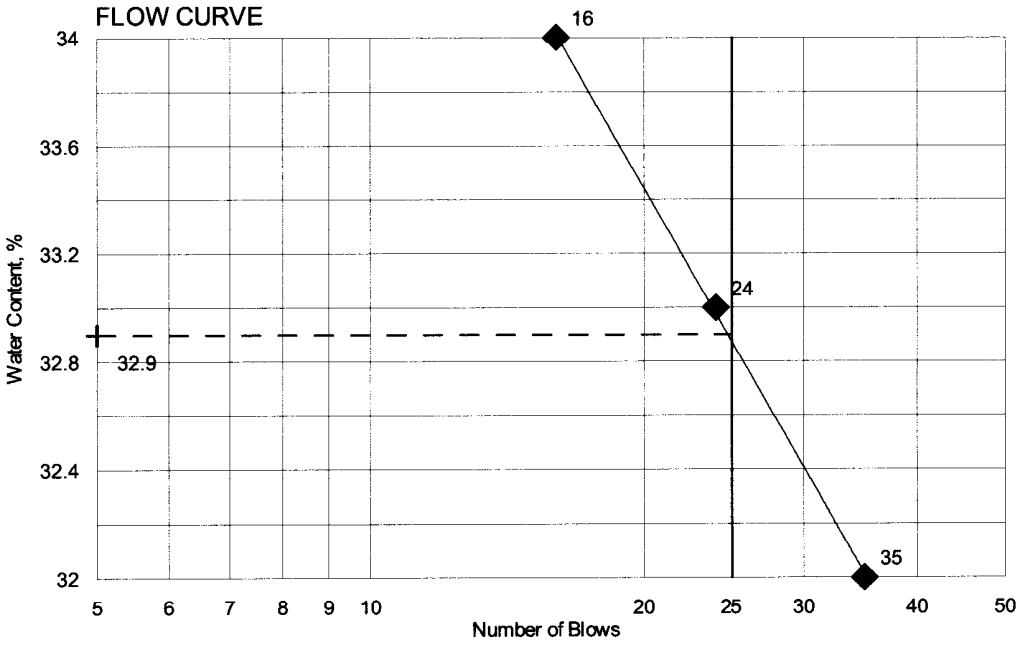


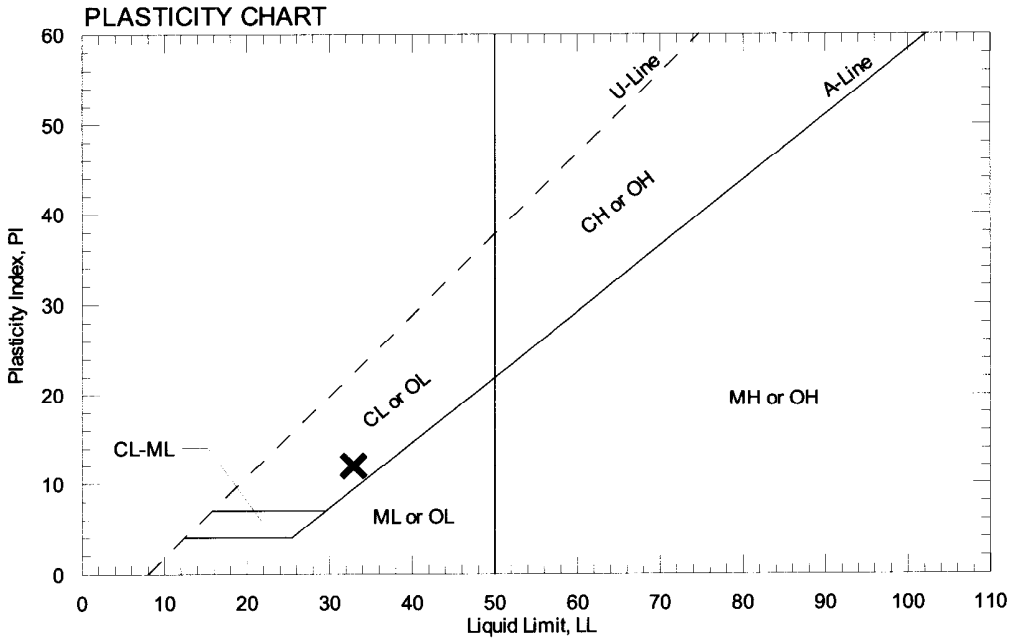
	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-105/1D	633575.2	1762800	1.0-3.0	SAND, some gravel, trace silt.	2.7			
◆	BB-HEPR-105/3D	633575.2	1762800	10.0-12.0	SAND, some gravel, trace silt.	7.5			
■	BB-HEPR-105/4DB	633575.2	1762800	15.2-17.0	CLAY-SILT, trace sand.	24.1	33	21	12
●	BB-HEPR-105/5D	633575.2	1762800	20.0-22.0	SAND, little silt.	18.5			
▲	BB-HEPR-105/8D	633575.2	1762800	35.0-37.0	Gravelly SAND, little silt.	9.1			
✖									

PIN
016705.00
Town
Howland,Enfield
Reported by/Date
WHITE, TERRY A 10/20/2010

SHEET 4

TOWN	Howland,Enfield	Reference No.	239833 92
PIN	016705.00	Water Content, %	24.1
Sampled	8/19/2010	Plastic Limit	21
Boring No./Sample No.	BB-HEPR-105/4DB	Liquid Limit	33
Station		Plasticity Index	12
Depth	15.2-17.0	Tested By	BBURR





O. SURVEY PLANS OF EXISTING BRIDGE

The following 1896 pre-existing bridge plans are provided:

- Details of Foundations for Piers 1-4
- Plan of Piers and Abutments

The following 1941 bridge widening and superstructure replacement plans are provided:

- Survey Plan Sheet 2 of 14
- Survey Plan Sheet 3 of 14
- Abutment 1 Sheet 6 of 14
- Abutment 1 Sheet 7 of 14
- Piers 1 & 2 Sheet 8 of 14
- Piers 3 & 4 Sheet 9 of 14
- Truss Details Sheet 10 of 14
- Truss Details Sheet 12 of 14
- Roadway Slab and Reinforcing Steel Sheet 14 of 14

Penobscot River Bridge #2660
Howland - Enfield, Maine

P. HYDROLOGY / HYDRAULIC / SCOUR DATA

Hydrology:

- Design flows derived using peak flow as reported in the 1996 Federal Emergency Management Agency Flood Insurance Study for the Town of Howland.
- Design flows derived using peak flow and calculations by USGS Regression Equations provided by the Hydraulics Section of the Maine DOT.

River near Mattawamkeag; and No. 01034500 on the Penobscot River at West Enfield (References 9 and 10). The results from those analyses were updated for this study. The revised flood flows were within the 90 percent confidence interval of discharges determined for the floodplain management study, the results of which were, therefore, used for this FIS.

Flood discharges on the Piscataquis River were determined from a log-Pearson Type III analysis of USGS stream gage No. 01034000 at Medford (Reference 10).

Discharges at sites in Howland were computed using the following equation:

$$Q_s = Q_g \ (A_s \ / \ A_g)^a$$

where Q_s is the discharge at the site, Q_g is the discharge at the gage, A_s is the drainage area of the site, A_g is the drainage area of the gage, and the exponent, "a," equals 0.8 for all recurrence intervals.

A summary of the drainage area-peak discharge relationships for a portion of 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
PENOBSCOT RIVER					
At State Routes 6 and 155	6,671.0	98,400	133,300	148,800	186,800
At Stanford Dam	5,217.1	64,000	86,500	96,500	121,000
At upstream corporate limits	5,199.4	68,800	86,300	96,200	120,700
PISCATAQUIS RIVER					
At State Route 116	1,452.9	48,400	74,800	87,900	123,500
At upstream corporate limits	1,267.8	43,400	67,100	78,800	110,700

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals.

Cross sections for the flooding sources studied by detailed methods were obtained from aerial photographs at a scale of 1:9,600, field surveys, and from USGS topographic maps (References 11 and 12). All bridges, dams, and culverts were field surveyed to obtain elevation data and structural geometry.

Project Name:	Howland Penob Rvr Bridge	**PIN:**	12675
Stream Name:	Penonscot River	**Town:**	Howland
Bridge Name:		**Bridge No.**	2660
Route No.	SR 155	**USGS Quad:**	
Analysis by:	CSH	**Date:**	6/22/2010

Peak Flow Calculations by USGS Regression Equations (Hodgkins, 1999)

Note:

Enter data in blue cells only!

	km^2	mi^2	ac	
A	17277.89	6671.000	4269440.0	***Enter data in [mi^2]*** Watershed Area
W	2097.90	810.000	518400.0	Wetlands area (by NWI)
P_c	510000	5057000	*(approx)*	watershed centroid (E, N; UTM 19N; meters)
County	Penobscot N			*choose county from drop-down menu*
pptA	41.5			mean annual precipitation (inches; by look-up)
SG	0.00			sand & gravel aquifer as decimal fraction of watershed A

A (km^2)	17277.89	**Conf Lvl**	0.67
W (%)	12.14		

Worksheet prepared by:
Charles S. Hebson, PE
Chief Hydrologist
Maine Dept. Transportation
Augusta, ME 04333-0016
207-287-1105
Charles.Hebson@maine.gov

Ret Pd	**Peak Flow Estimate**			
T (yr)	**Lower**	**Q_T (m^3/s)**	**Upper**	**Q_T (ft^3/s)**
1.1		1308.88		46216.4
2	1422.86	2003.64	2821.50	70748.7
5	1843.29	2603.33	3676.75	91923.6
10	2108.95	3008.26	4291.06	106221.7
25	2407.01	3480.61	5033.08	122900.4
50	2616.83	3827.65	5598.73	135154.4
100	2827.45	4185.08	6194.59	147775.2
500	3283.62	5016.77	7664.69	177142.1

Reference:

Hodgkins, G., 1999.
Estimating the magnitude of peak flows for streams in Maine for selected recurrence intervals
Water-Resources Investigations Report 99-4008
US Geological Survey, Augusta, Maine

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

Penobscot River Bridge #2660
Howland - Enfield, Maine

Hydraulics:

- **Existing Bridge / Rehabilitation Alternate 1:**
  Hydraulic analysis and data for Existing Bridge and Bridge Rehabilitation Alternate 1 includes the following HEC-RAS model output:
  - Standard Table 1
  - Standard Table 2
  - Six XS Bridge Table
  - Cross Section Tables
  - Cross Section Plots
  - Profile Plot

Howland, Penobscot River Bridge - Existing Bridge & Alternate 1

Standard Table 1

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1

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)	
1	10450	Q 1.1	46216.00	118.80	134.07		134.51	0.000702	5.31	8709.72	846.39	0.29
1	10450	Q 10	98400.00	118.80	143.96		144.45	0.000346	5.64	17476.27	927.68	0.23
1	10450	Q 25	122900.00	118.80	146.34		146.95	0.000365	6.26	19616.60	1043.60	0.24
1	10450	Q 50	133300.00	118.80	147.94		148.55	0.000338	6.31	21547.76	1128.37	0.23
1	10450	Q 100	148800.00	118.80	149.36		150.04	0.000344	6.63	23206.10	1203.99	0.24
1	10450	Q 500	186800.00	118.80	152.95		153.75	0.000338	7.20	27879.05	1361.44	0.24
1	10050	Q 1.1	46216.00	121.70	133.64		134.16	0.001018	5.80	7970.44	897.78	0.34
1	10050	Q 10	98400.00	121.70	143.81		144.31	0.000367	5.65	17479.44	975.76	0.23
1	10050	Q 25	122900.00	121.70	146.19		146.80	0.000383	6.24	19635.50	1001.68	0.24
1	10050	Q 50	133300.00	121.70	147.80		148.41	0.000351	6.28	21465.97	1025.93	0.23
1	10050	Q 100	148800.00	121.70	149.22		149.90	0.000356	6.59	22958.26	1066.22	0.24
1	10050	Q 500	186800.00	121.70	152.82		153.61	0.000346	7.14	27610.14	1467.29	0.24
1	10020	Q 1.1	46216.00	120.59	133.53	129.25	134.10	0.001196	6.08	7604.07	895.66	0.37
1	10020	Q 10	98400.00	120.59	143.76	132.19	144.28	0.000386	5.82	16915.28	913.09	0.24
1	10020	Q 25	122900.00	120.59	146.13	133.37	146.77	0.000404	6.45	19081.43	914.00	0.25
1	10020	Q 50	133300.00	120.59	147.74	133.86	148.39	0.000371	6.50	20550.56	914.61	0.24
1	10020	Q 100	148800.00	120.59	149.15	134.54	149.87	0.000378	6.82	21842.83	915.15	0.25
1	10020	Q 500	186800.00	120.59	152.73	136.09	153.58	0.000373	7.44	25914.77	1366.59	0.25
1	10000		Bridge									
1	9980	Q 1.1	46216.00	120.84	133.46		134.03	0.001186	6.07	7616.76	894.73	0.37
1	9980	Q 10	98400.00	120.84	143.73		144.25	0.000387	5.79	17000.21	916.73	0.24
1	9980	Q 25	122900.00	120.84	146.10		146.74	0.000406	6.41	19171.23	916.98	0.25
1	9980	Q 50	133300.00	120.84	147.71		148.35	0.000374	6.46	20646.29	917.14	0.24
1	9980	Q 100	148800.00	120.84	149.12		149.83	0.000381	6.78	21940.36	917.29	0.24
1	9980	Q 500	186800.00	120.84	152.69		153.54	0.000378	7.39	26006.86	1370.13	0.25
1	9900	Q 1.1	46216.00	121.30	133.46		133.92	0.000847	5.47	8441.83	902.19	0.32
1	9900	Q 10	98400.00	121.30	143.74		144.20	0.000327	5.45	18073.59	963.09	0.22
1	9900	Q 25	122900.00	121.30	146.12		146.69	0.000345	6.05	20370.54	971.81	0.23
1	9900	Q 50	133300.00	121.30	147.73		148.31	0.000318	6.10	21941.24	989.31	0.22
1	9900	Q 100	148800.00	121.30	149.14		149.78	0.000324	6.41	23399.13	1057.65	0.23
1	9900	Q 500	186800.00	121.30	152.73		153.49	0.000322	7.00	27481.53	1230.17	0.23
1	9184	Q 1.1	46216.00	118.20	132.78		133.30	0.000861	5.84	7916.57	777.63	0.32
1	9184	Q 10	98400.00	118.20	143.39		143.95	0.000367	5.97	16587.35	854.62	0.24
1	9184	Q 25	122900.00	118.20	145.72		146.41	0.000402	6.67	18583.65	903.48	0.25
1	9184	Q 50	133300.00	118.20	147.34		148.05	0.000375	6.73	20159.29	1001.69	0.24
1	9184	Q 100	148800.00	118.20	148.74		149.51	0.000385	7.09	21569.64	1021.82	0.25
1	9184	Q 500	186800.00	118.20	152.29		153.22	0.000386	7.75	25293.70	1073.16	0.26
1	8600	Q 1.1	46216.00	116.20	132.10		132.77	0.000932	6.57	7062.39	628.86	0.34
1	8600	Q 10	98400.00	116.20	142.92		143.69	0.000455	7.07	14271.70	697.46	0.27
1	8600	Q 25	122900.00	116.20	145.13		146.11	0.000513	8.00	15825.54	709.41	0.29
1	8600	Q 50	133300.00	116.20	146.75		147.77	0.000484	8.12	16985.78	718.20	0.28
1	8600	Q 100	148800.00	116.20	148.08		149.22	0.000509	8.61	17945.31	727.43	0.29
1	8600	Q 500	186800.00	116.20	151.49		152.90	0.000538	9.58	20602.90	806.44	0.31
1	8380	Q 1.1	46216.00	115.40	131.75		132.53	0.001191	7.07	6549.46	622.86	0.38
1	8380	Q 10	98400.00	115.40	142.75		143.58	0.000501	7.31	13711.43	674.69	0.28
1	8380	Q 25	122900.00	115.40	144.93		145.99	0.000565	8.28	15190.48	683.12	0.30
1	8380	Q 50	133300.00	115.40	146.56		147.65	0.000530	8.39	16311.13	689.45	0.29
1	8380	Q 100	148800.00	115.40	147.87		149.09	0.000557	8.90	17224.10	705.52	0.30
1	8380	Q 500	186800.00	115.40	151.27		152.77	0.000585	9.88	19737.68	763.05	0.32
1	8100	Q 1.1	46216.00	113.20	131.53		132.23	0.000825	6.75	7012.73	572.67	0.33
1	8100	Q 10	98400.00	113.20	142.54		143.43	0.000472	7.67	13671.02	636.94	0.28
1	8100	Q 25	122900.00	113.20	144.65		145.82	0.000553	8.79	15027.95	649.26	0.30
1	8100	Q 50	133300.00	113.20	146.27		147.49	0.000530	8.95	16090.72	658.75	0.30
1	8100	Q 100	148800.00	113.20	147.54		148.92	0.000567	9.54	16933.92	668.28	0.31
1	8100	Q 500	186800.00	113.20	150.85		152.58	0.000620	10.72	19220.14	716.02	0.33
1	7250	Q 1.1	46216.00	115.70	130.94	124.37	131.53	0.000750	6.17	7956.81	771.15	0.31
1	7250	Q 10	98400.00	115.70	142.40	128.11	143.01	0.000340	6.46	18441.59	1058.96	0.23
1	7250	Q 25	122900.00	115.70	144.53	129.65	145.31	0.000390	7.32	20757.12	1112.52	0.25
1	7250	Q 50	133300.00	115.70	146.20	130.26	146.99	0.000367	7.40	22646.97	1154.40	0.25
1	7250	Q 100	148800.00	115.70	147.50	131.13	148.38	0.000387	7.84	24168.93	1187.06	0.26
1	7250	Q 500	186800.00	115.70	150.90	133.14	151.95	0.000407	8.66	28350.09	1272.46	0.27

Howland, Penobscot River Bridge - Existing Bridge & Alternate 4
Standard Table 2

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width
			(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)
1	10450	Q 1.1	134.51	134.07	0.44	0.34	0.01		46216.00		846.39
1	10450	Q 10	144.45	143.96	0.49	0.14	0.00	18.87	98381.13		927.68
1	10450	Q 25	146.95	146.34	0.61	0.15	0.00	70.03	122788.70	41.22	1043.60
1	10450	Q 50	148.55	147.94	0.62	0.14	0.00	123.53	133001.70	174.79	1128.37
1	10450	Q 100	150.04	149.36	0.68	0.14	0.00	194.40	148188.90	416.74	1203.99
1	10450	Q 500	153.75	152.95	0.80	0.14	0.00	462.37	184520.60	1816.99	1361.44
1	10050	Q 1.1	134.16	133.64	0.52	0.06	0.01		46216.00		897.78
1	10050	Q 10	144.31	143.81	0.50	0.02	0.00		98351.10	48.90	975.76
1	10050	Q 25	146.80	146.19	0.60	0.02	0.00	0.00	122758.70	141.34	1001.68
1	10050	Q 50	148.41	147.80	0.61	0.02	0.00	1.30	133069.00	229.73	1025.93
1	10050	Q 100	149.90	149.22	0.67	0.02	0.01	6.15	148452.00	341.88	1066.22
1	10050	Q 500	153.61	152.82	0.79	0.02	0.01	44.82	185381.80	1373.41	1467.29
1	10020	Q 1.1	134.10	133.53	0.57	0.00	0.00		46216.00		895.66
1	10020	Q 10	144.28	143.76	0.53	0.00	0.00	0.74	98393.48	5.79	913.09
1	10020	Q 25	146.77	146.13	0.65	0.00	0.00	1.41	122887.90	10.73	914.00
1	10020	Q 50	148.39	147.74	0.66	0.00	0.00	1.93	133283.60	14.43	914.61
1	10020	Q 100	149.87	149.15	0.72	0.00	0.00	2.57	148778.40	18.99	915.15
1	10020	Q 500	153.58	152.73	0.86	0.00	0.00	4.63	186345.00	450.36	1366.59
1	10000		Bridge								
1	9980	Q 1.1	134.03	133.46	0.57	0.08	0.03		46216.00		894.73
1	9980	Q 10	144.25	143.73	0.52	0.03	0.02	0.31	98399.68		916.73
1	9980	Q 25	146.74	146.10	0.64	0.03	0.02	0.60	122899.40		916.98
1	9980	Q 50	148.35	147.71	0.65	0.03	0.02	0.82	133299.20		917.14
1	9980	Q 100	149.83	149.12	0.71	0.03	0.02	1.10	148798.90		917.29
1	9980	Q 500	153.54	152.69	0.85	0.03	0.03	1.98	186388.50	409.50	1370.13
1	9900	Q 1.1	133.92	133.46	0.47	0.61	0.01		46216.00		902.19
1	9900	Q 10	144.20	143.74	0.46	0.25	0.01	2.80	98388.98	8.21	963.09
1	9900	Q 25	146.69	146.12	0.57	0.27	0.01	13.40	122857.00	29.62	971.81
1	9900	Q 50	148.31	147.73	0.58	0.25	0.01	25.77	133222.00	52.19	989.31
1	9900	Q 100	149.78	149.14	0.64	0.25	0.01	42.45	148652.80	104.79	1057.65
1	9900	Q 500	153.49	152.73	0.76	0.25	0.02	108.66	186131.60	559.69	1230.17
1	9184	Q 1.1	133.30	132.78	0.53	0.52	0.01		46216.00		777.63
1	9184	Q 10	143.95	143.39	0.55	0.24	0.02	74.49	98325.52		854.62
1	9184	Q 25	146.41	145.72	0.69	0.26	0.03	182.30	122716.30	1.44	903.48
1	9184	Q 50	148.05	147.34	0.70	0.25	0.03	280.82	132957.40	61.78	1001.69
1	9184	Q 100	149.51	148.74	0.78	0.26	0.04	400.94	148193.10	205.92	1021.82
1	9184	Q 500	153.22	152.29	0.92	0.26	0.05	821.05	185102.30	876.60	1073.16
1	8600	Q 1.1	132.77	132.10	0.67	0.23	0.01		46189.96	26.04	628.86
1	8600	Q 10	143.69	142.92	0.77	0.10	0.01	38.54	97791.10	570.36	697.46
1	8600	Q 25	146.11	145.13	0.99	0.12	0.01	85.73	121938.60	875.66	709.41
1	8600	Q 50	147.77	146.75	1.02	0.11	0.01	128.96	132087.50	1083.49	718.20
1	8600	Q 100	149.22	148.08	1.14	0.12	0.01	180.42	147307.50	1312.10	727.43
1	8600	Q 500	152.90	151.49	1.41	0.12	0.01	360.22	184598.90	1840.92	806.44
1	8380	Q 1.1	132.53	131.75	0.78	0.28	0.02	6.95	46209.05		622.86
1	8380	Q 10	143.58	142.75	0.83	0.14	0.01	386.35	97958.20	55.46	674.69
1	8380	Q 25	145.99	144.93	1.06	0.16	0.01	592.87	122190.50	116.67	683.12
1	8380	Q 50	147.65	146.56	1.09	0.15	0.01	731.01	132397.30	171.72	689.45
1	8380	Q 100	149.09	147.87	1.22	0.16	0.02	894.02	147704.80	201.17	705.52
1	8380	Q 500	152.77	151.27	1.50	0.17	0.02	1369.31	184933.70	496.96	763.05
1	8100	Q 1.1	132.23	131.53	0.70	0.67	0.04	239.49	45944.18	32.33	572.67
1	8100	Q 10	143.43	142.54	0.90	0.34	0.09	1467.68	96412.04	520.29	636.94
1	8100	Q 25	145.82	144.65	1.17	0.39	0.12	2050.44	120068.00	781.60	649.26
1	8100	Q 50	147.49	146.27	1.22	0.37	0.13	2403.45	129936.10	960.44	658.75
1	8100	Q 100	148.92	147.54	1.38	0.39	0.15	2839.02	144814.70	1146.33	668.28
1	8100	Q 500	152.58	150.85	1.73	0.42	0.20	4069.41	181148.80	1581.80	716.02
1	7250	Q 1.1	131.53	130.94	0.58			512.95	45518.17	184.88	771.15
1	7250	Q 10	143.01	142.40	0.61			3201.72	92087.09	3111.20	1058.96
1	7250	Q 25	145.31	144.53	0.77			4407.28	113842.60	4650.15	1112.52

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1 (Continued)

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width
			(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)
1	7250	Q 50	146.99	146.20	0.79			5113.56	122486.00	5700.46	1154.40
1	7250	Q 100	148.38	147.50	0.88			5990.32	135871.30	6938.40	1187.06
1	7250	Q 500	151.95	150.90	1.05			8409.67	167786.00	10604.35	1272.46

Howland, Penobscot River Bridge - Existing Bridge & Alternate 1

Bridge Cross Section Tables

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 1.1

E.G. US. (ft)	134.10	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	133.53	E.G. Elev (ft)	134.10	134.05
Q Total (cfs)	46216.00	W.S. Elev (ft)	133.49	133.44
Q Bridge (cfs)	46216.00	Crit W.S. (ft)	129.30	129.20
Q Weir (cfs)		Max Chl Dpth (ft)	12.90	12.60
Weir Sta Lft (ft)		Vel Total (ft/s)	6.27	6.23
Weir Sta Rgt (ft)		Flow Area (sq ft)	7373.09	7417.02
Weir Submerg		Froude # Chl	0.38	0.38
Weir Max Depth (ft)		Specif Force (cu ft)	43147.27	43524.28
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	8.51	8.57
Min El Prs (ft)	162.30	W.P. Total (ft)	919.24	915.43
Delta EG (ft)	0.07	Conv. Total (cfs)	1254265.0	1270259.0
Delta WS (ft)	0.07	Top Width (ft)	866.42	865.74
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.05	0.00
BR Open Vel (ft/s)	6.27	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.68	0.67
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 10

E.G. US. (ft)	144.28	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	143.76	E.G. Elev (ft)	144.28	144.26
Q Total (cfs)	98400.00	W.S. Elev (ft)	143.72	143.71
Q Bridge (cfs)	98400.00	Crit W.S. (ft)	132.30	132.22
Q Weir (cfs)		Max Chl Dpth (ft)	23.13	22.87
Weir Sta Lft (ft)		Vel Total (ft/s)	5.99	5.96
Weir Sta Rgt (ft)		Flow Area (sq ft)	16426.40	16499.59
Weir Submerg		Froude # Chl	0.22	0.22
Weir Max Depth (ft)		Specif Force (cu ft)	174152.10	175423.10
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	18.47	18.54
Min El Prs (ft)	162.30	W.P. Total (ft)	1034.44	1033.00
Delta EG (ft)	0.03	Conv. Total (cfs)	4454775.0	4467416.0
Delta WS (ft)	0.03	Top Width (ft)	889.50	890.16
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	5.99	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.48	0.48
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 25

E.G. US. (ft)	146.77	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	146.13	E.G. Elev (ft)	146.77	146.75
Q Total (cfs)	122900.00	W.S. Elev (ft)	146.09	146.07
Q Bridge (cfs)	122900.00	Crit W.S. (ft)	133.53	133.44
Q Weir (cfs)		Max Chl Dpth (ft)	25.50	25.23
Weir Sta Lft (ft)		Vel Total (ft/s)	6.63	6.61
Weir Sta Rgt (ft)		Flow Area (sq ft)	18530.85	18604.77
Weir Submerg		Froude # Chl	0.23	0.23
Weir Max Depth (ft)		Specif Force (cu ft)	222479.60	223886.30
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	20.80	20.87
Min El Prs (ft)	162.30	W.P. Total (ft)	1058.15	1056.70
Delta EG (ft)	0.04	Conv. Total (cfs)	5378234.0	5382274.0
Delta WS (ft)	0.03	Top Width (ft)	890.96	891.54
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	6.63	C & E Loss (ft)	0.00	0.01

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 25 (Continued)

Coef of Q		Shear Total (lb/sq ft)	0.57	0.57
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 50

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	148.39			
W.S. US. (ft)	147.74	E.G. Elev (ft)	148.39	148.37
Q Total (cfs)	133300.00	W.S. Elev (ft)	147.69	147.68
Q Bridge (cfs)	133300.00	Crit W.S. (ft)	134.02	133.92
Q Weir (cfs)		Max Chl Dpth (ft)	27.10	26.84
Weir Sta Lft (ft)		Vel Total (ft/s)	6.68	6.65
Weir Sta Rgt (ft)		Flow Area (sq ft)	19962.66	20038.49
Weir Submerg		Froude # Chl	0.23	0.23
Weir Max Depth (ft)		Specif Force (cu ft)	255740.70	257283.90
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	22.40	22.48
Min El Prs (ft)	162.30	W.P. Total (ft)	1074.24	1072.79
Delta EG (ft)	0.04	Conv. Total (cfs)	6037920.0	6035177.0
Delta WS (ft)	0.03	Top Width (ft)	891.04	891.57
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	6.68	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.57	0.57
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 100

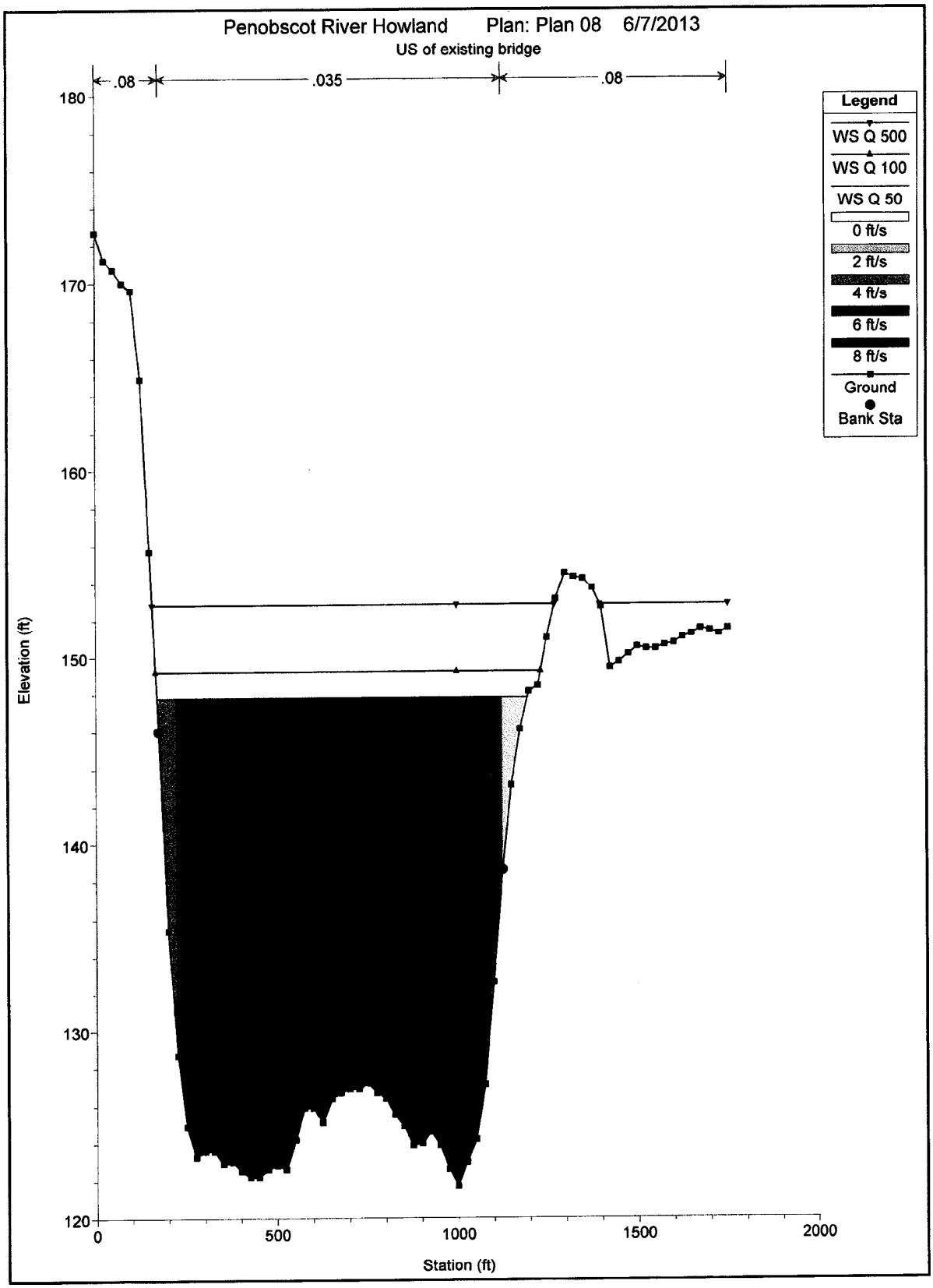
		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	149.87			
W.S. US. (ft)	149.15	E.G. Elev (ft)	149.87	149.85
Q Total (cfs)	148800.00	W.S. Elev (ft)	149.10	149.09
Q Bridge (cfs)	148800.00	Crit W.S. (ft)	134.74	134.62
Q Weir (cfs)		Max Chl Dpth (ft)	28.51	28.25
Weir Sta Lft (ft)		Vel Total (ft/s)	7.01	6.99
Weir Sta Rgt (ft)		Flow Area (sq ft)	21217.62	21293.86
Weir Submerg		Froude # Chl	0.23	0.23
Weir Max Depth (ft)		Specif Force (cu ft)	289509.90	291138.60
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	23.81	23.88
Min El Prs (ft)	162.30	W.P. Total (ft)	1088.33	1086.87
Delta EG (ft)	0.04	Conv. Total (cfs)	6635618.0	6625284.0
Delta WS (ft)	0.03	Top Width (ft)	891.08	891.55
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	7.01	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.61	0.62
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 500

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	153.58			
W.S. US. (ft)	152.73	E.G. Elev (ft)	153.58	153.56
Q Total (cfs)	186800.00	W.S. Elev (ft)	152.67	152.66
Q Bridge (cfs)	186333.80	Crit W.S. (ft)	136.32	136.23
Q Weir (cfs)		Max Chl Dpth (ft)	32.08	31.82
Weir Sta Lft (ft)		Vel Total (ft/s)	7.42	7.40
Weir Sta Rgt (ft)		Flow Area (sq ft)	25170.52	25241.07
Weir Submerg		Froude # Chl	0.24	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	383472.00	385330.30
Min El Weir Flow (ft)	150.15	Hydr Depth (ft)	18.81	18.87
Min El Prs (ft)	162.30	W.P. Total (ft)	1573.99	1571.99

Plan: Alternate 4 Penobscot 1 RS: 10000 Profile: Q 500 (Continued)

Delta EG (ft)	0.04	Conv. Total (cfs)	8243096.0	8208929.0
Delta WS (ft)	0.03	Top Width (ft)	1338.14	1337.60
BR Open Area (sq ft)	29193.05	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	7.64	C & E Loss (ft)	0.00	0.02
Coef of Q		Shear Total (lb/sq ft)	0.51	0.52
Br Sel Method	Energy only	Power Total (lb/ft s)	-34.00	-34.00


Penobscot River Howland Plan: Plan 08 6/7/2013
US of existing bridge
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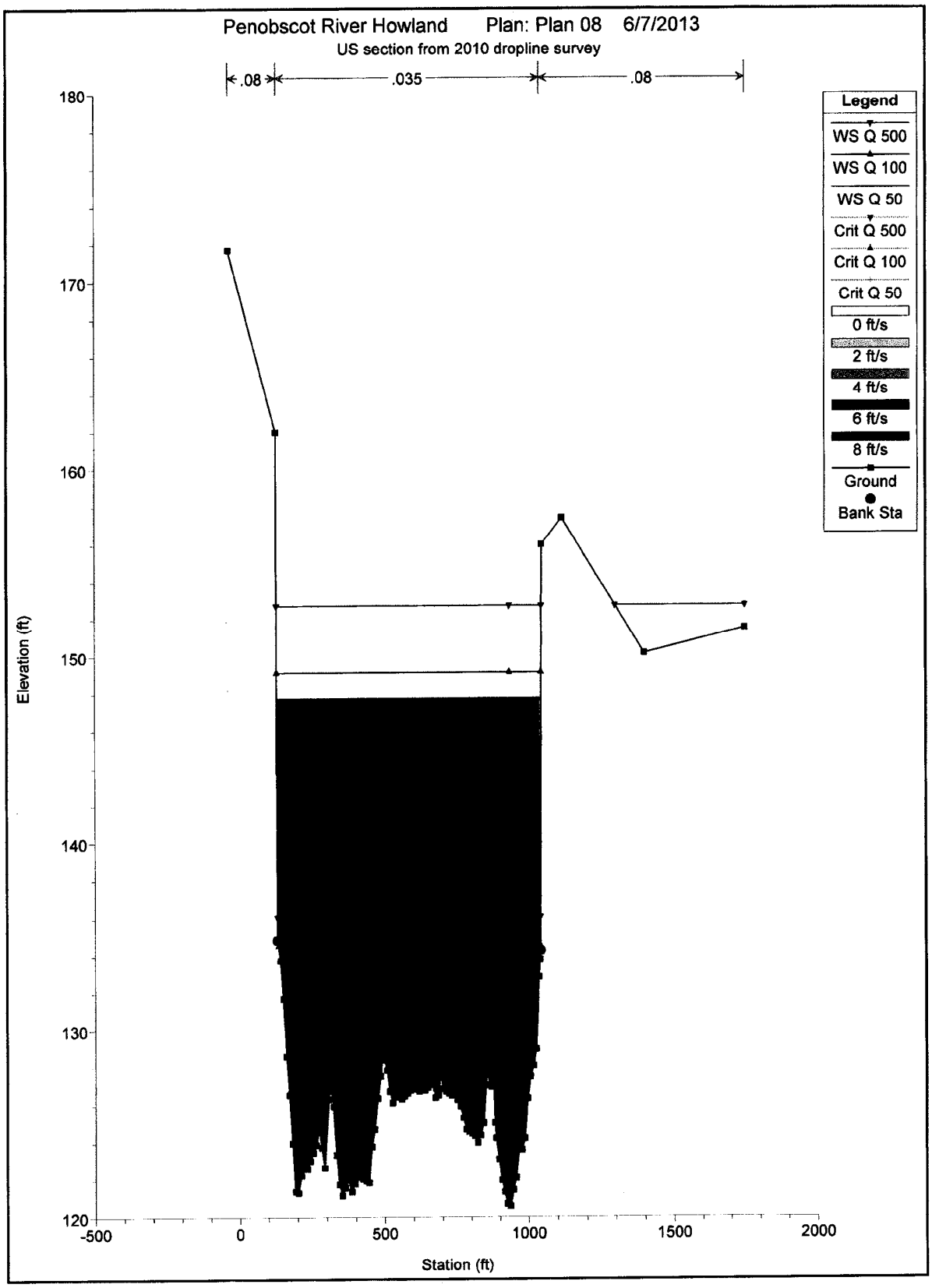
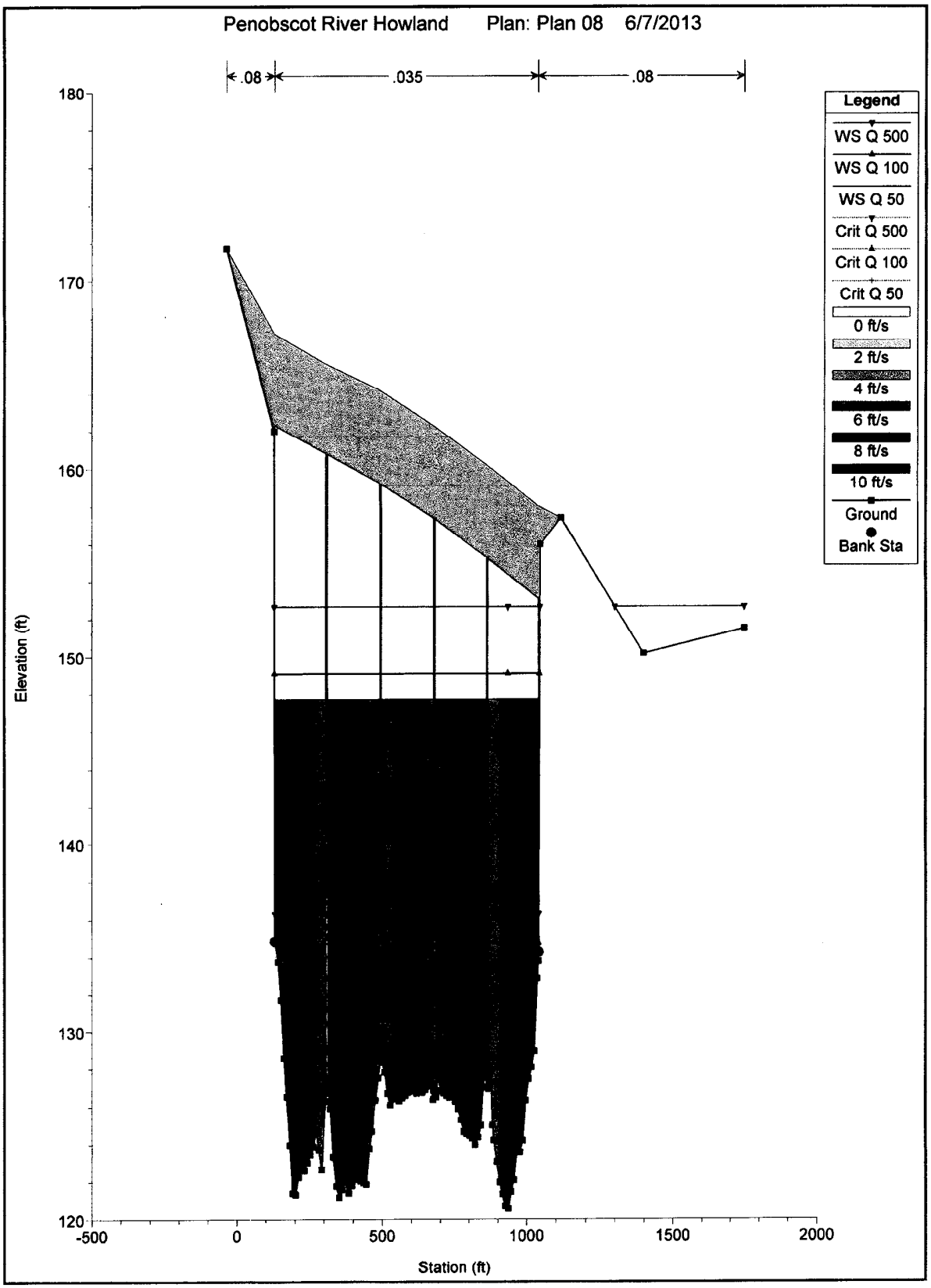
.08
Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
0
500
1000
1500
2000
Station (ft)


Penobscot River Howland Plan: Plan 08 6/7/2013
US section from 2010 dropline survey
.08
.035
.08
Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
-500
0
500
1000
1500
2000
Station (ft)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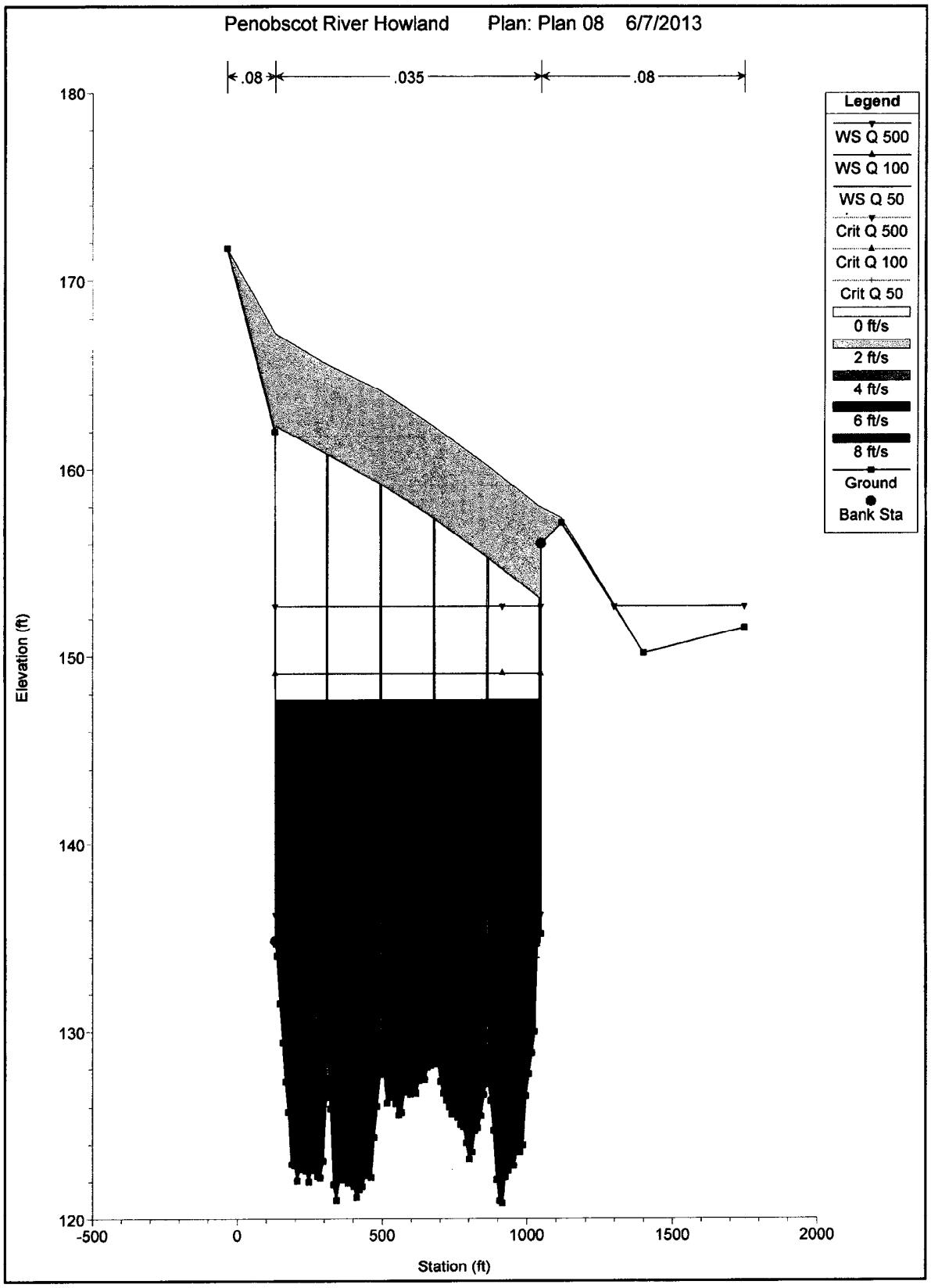
.08
.035
.08
Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
10 ft/s
Ground
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
-500
0
500
1000
1500
2000
Station (ft)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.08
.035
.08
Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
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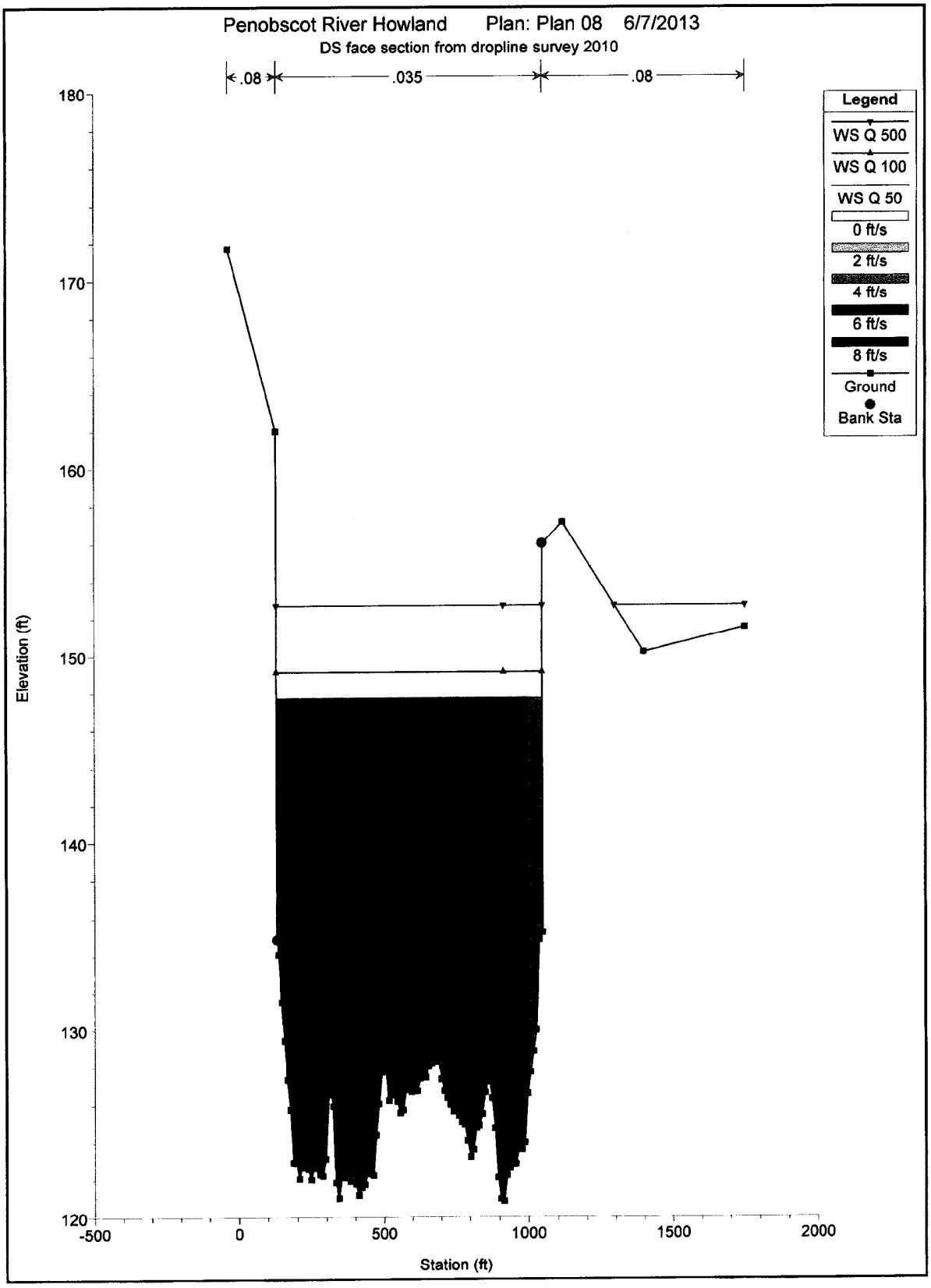
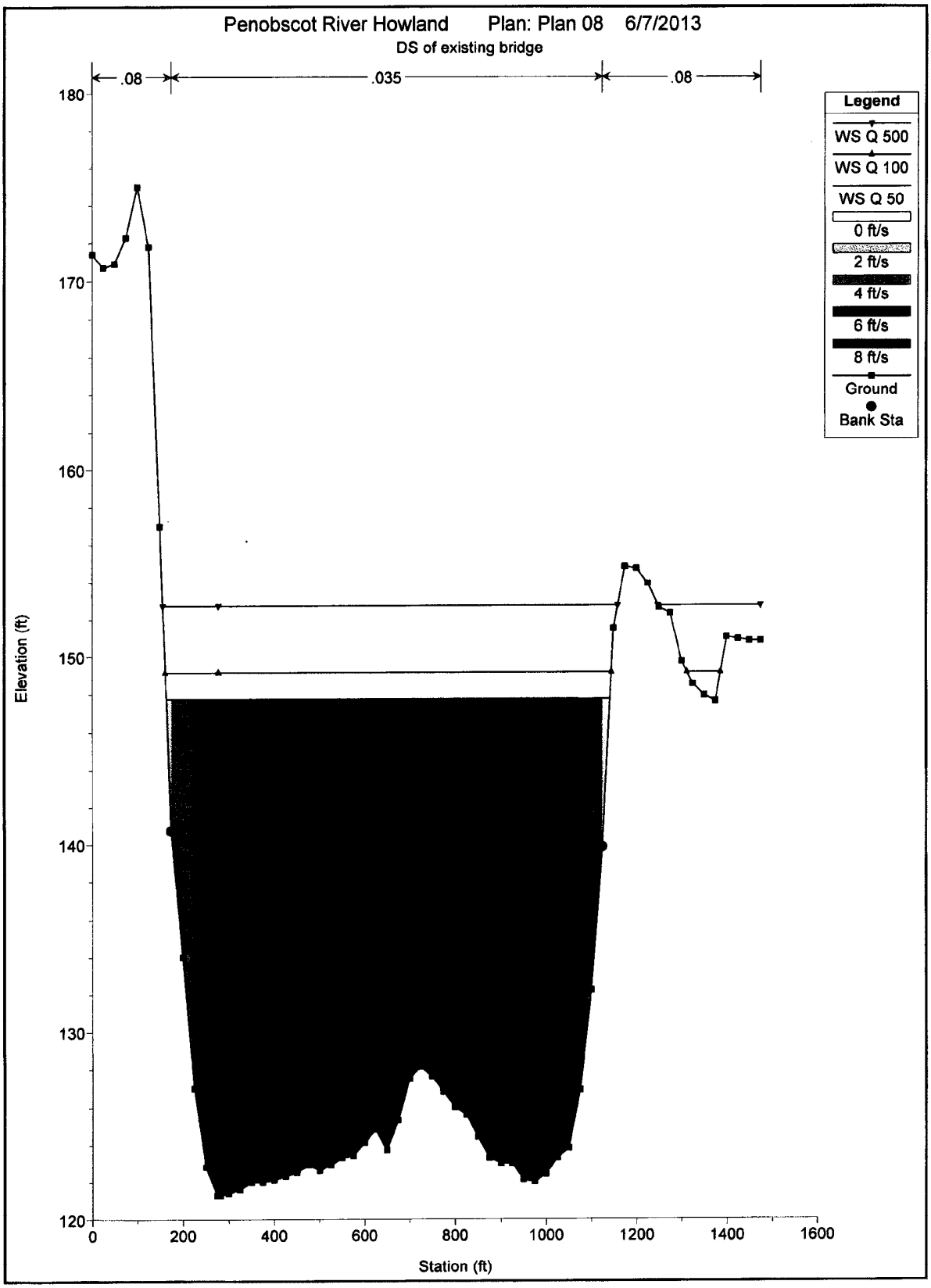
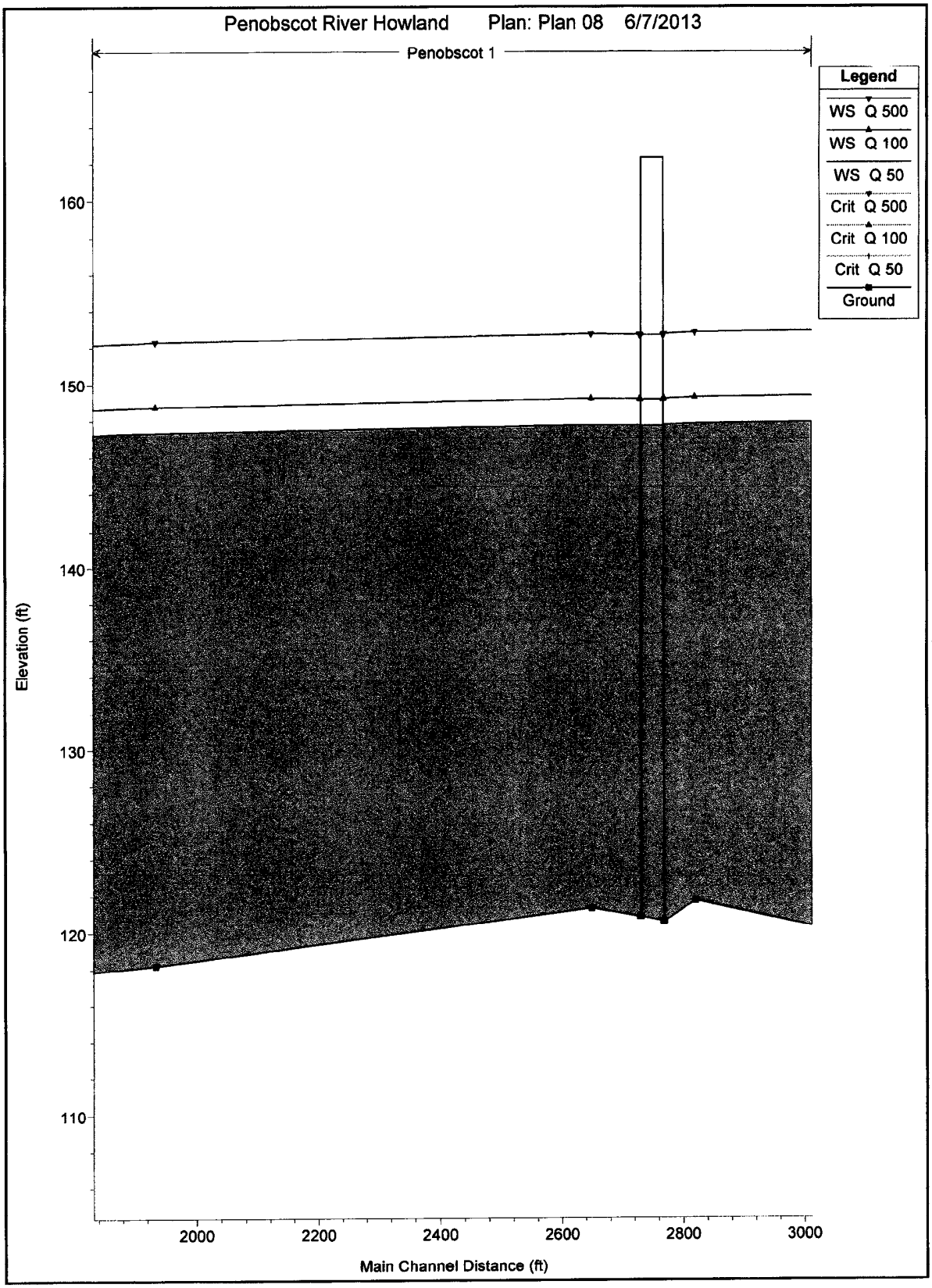
170
160
150
140
130
120
-500
0
500
1000
1500
2000
Station (ft)


Penobscot River Howland Plan: Plan 08 6/7/2013
DS face section from dropline survey 2010
.08
.035
.08
Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
-500
0
500
1000
1500
2000
Station (ft)


Penobscot River Howland Plan: Plan 08 6/7/2013
DS of existing bridge
.08
.035
.08
Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
0
200
400
600
800
1000
1200
1400
1600
Station (ft)


Penobscot River Howland Plan: Plan 08 6/7/2013
Penobscot 1
Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
Ground
Elevation (ft)
160
150
140
130
120
110
2000
2200
2400
2600
2800
3000
Main Channel Distance (ft)

Penobscot River Bridge #2660
Howland - Enfield, Maine

Hydraulics:

- **Proposed Bridge Replacement Alternate 3:**
  Hydraulic analysis and data for Bridge Replacement Alternate 3includes the following HEC-RAS model output:
  - Standard Table 1
  - Standard Table 2
  - Six XS Bridge Table
  - Cross Section Tables
  - Cross Section Plots
  - Profile Plot

Howland, Penobscot River Bridge - Alternate 3
Standard Table 1

HEC-RAS Plan: Plan 07 River: Penobscot Reach: 1

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	10450	PF 1	46216.00	118.80	134.07		134.51	0.000702	5.31	8709.28	846.39	0.29
1	10450	PF 2	106222.00	118.80	144.23		144.79	0.000386	6.00	17727.24	931.07	0.24
1	10450	PF 3	122900.00	118.80	146.38		146.98	0.000364	6.25	19851.96	1045.40	0.24
1	10450	PF 4	135154.00	118.80	148.02		148.65	0.000343	6.38	21645.32	1132.96	0.23
1	10450	PF 5	147775.00	118.80	149.38		150.05	0.000336	6.58	23229.42	1205.02	0.23
1	10450	PF 6	177142.00	118.80	152.83		153.55	0.000309	6.86	27706.90	1360.48	0.23
1	10050	PF 1	46216.00	121.70	133.64		134.16	0.001018	5.80	7969.89	897.77	0.34
1	10050	PF 2	106222.00	121.70	144.06		144.63	0.000409	6.02	17726.92	978.47	0.25
1	10050	PF 3	122900.00	121.70	146.23		146.83	0.000381	6.23	19870.32	1002.20	0.24
1	10050	PF 4	135154.00	121.70	147.89		148.51	0.000356	6.34	21552.46	1027.20	0.24
1	10050	PF 5	147775.00	121.70	149.24		149.91	0.000350	6.53	22981.41	1066.48	0.24
1	10050	PF 6	177142.00	121.70	152.71		153.42	0.000316	6.80	27442.11	1462.76	0.23
1	10020	PF 1	46216.00	120.59	133.53		134.10	0.001186	6.06	7603.50	895.65	0.37
1	10020	PF 2	106222.00	120.59	144.00		144.60	0.000431	6.20	17139.73	913.18	0.25
1	10020	PF 3	122900.00	120.59	146.16		146.81	0.000401	6.44	19113.39	914.01	0.25
1	10020	PF 4	135154.00	120.59	147.82		148.49	0.000377	6.56	20626.29	914.64	0.24
1	10020	PF 5	147775.00	120.59	149.17		149.88	0.000371	6.77	21863.79	915.16	0.24
1	10020	PF 6	177142.00	120.59	152.62		153.40	0.000340	7.08	25771.48	1362.48	0.24
1	9980	PF 1	46216.00	120.84	133.49		134.05	0.001176	6.05	7637.60	894.87	0.37
1	9980	PF 2	106222.00	120.84	143.99		144.58	0.000430	6.16	17240.29	916.76	0.25
1	9980	PF 3	122900.00	120.84	146.15		146.79	0.000402	6.39	19222.54	916.98	0.25
1	9980	PF 4	135154.00	120.84	147.81		148.47	0.000378	6.52	20741.13	917.15	0.24
1	9980	PF 5	147775.00	120.84	149.16		149.87	0.000373	6.72	21982.35	917.29	0.24
1	9980	PF 6	177142.00	120.84	152.61		153.38	0.000343	7.03	25899.02	1366.96	0.24
1	9900	PF 1	46216.00	121.30	133.48		133.94	0.000840	5.48	8462.92	902.35	0.31
1	9900	PF 2	106222.00	121.30	144.01		144.53	0.000364	5.80	18328.04	964.06	0.23
1	9900	PF 3	122900.00	121.30	146.17		146.74	0.000342	6.03	20424.87	972.01	0.23
1	9900	PF 4	135154.00	121.30	147.83		148.42	0.000322	6.16	22044.59	999.11	0.23
1	9900	PF 5	147775.00	121.30	149.19		149.82	0.000318	6.36	23447.13	1059.09	0.23
1	9900	PF 6	177142.00	121.30	152.64		153.33	0.000293	6.66	27380.58	1227.84	0.22
1	9184	PF 1	46216.00	118.20	132.80		133.33	0.000853	5.82	7939.49	777.82	0.32
1	9184	PF 2	106222.00	118.20	143.61		144.24	0.000413	6.38	16752.98	856.29	0.25
1	9184	PF 3	122900.00	118.20	145.78		146.46	0.000399	6.66	18637.03	905.88	0.25
1	9184	PF 4	135154.00	118.20	147.44		148.16	0.000381	6.80	20257.22	1003.10	0.25
1	9184	PF 5	147775.00	118.20	148.79		149.55	0.000377	7.02	21624.60	1022.60	0.25
1	9184	PF 6	177142.00	118.20	152.25		153.09	0.000348	7.36	25252.84	1072.61	0.24
1	8600	PF 1	46216.00	116.20	132.13		132.80	0.000922	6.55	7084.85	629.14	0.34
1	8600	PF 2	106222.00	116.20	143.06		143.95	0.000518	7.58	14372.15	698.24	0.28
1	8600	PF 3	122900.00	116.20	145.19		146.17	0.000509	7.98	15870.23	709.75	0.29
1	8600	PF 4	135154.00	116.20	146.84		147.87	0.000492	8.21	17047.10	718.66	0.28
1	8600	PF 5	147775.00	116.20	148.15		149.27	0.000498	8.53	17994.30	727.97	0.29
1	8600	PF 6	177142.00	116.20	151.54		152.81	0.000481	9.07	20643.09	807.38	0.29
1	8405	PF 1	46216.00	115.40	131.82	126.47	132.59	0.001179	7.01	6594.65	623.27	0.38
1	8405	PF 2	106222.00	115.40	142.91	130.86	143.83	0.000572	7.73	13816.68	675.29	0.30
1	8405	PF 3	122900.00	115.40	145.04	131.88	146.06	0.000556	8.12	15264.04	683.54	0.30
1	8405	PF 4	135154.00	115.40	146.69	132.58	147.76	0.000534	8.33	16398.64	689.94	0.29
1	8405	PF 5	147775.00	115.40	148.00	133.33	149.16	0.000538	8.65	17310.05	707.52	0.30
1	8405	PF 6	177142.00	115.40	151.40	134.90	152.70	0.000513	9.16	19838.22	764.25	0.30
1	8380		Bridge									
1	8355	PF 1	46216.00	115.40	131.73		132.50	0.001214	7.07	6534.85	622.72	0.38
1	8355	PF 2	106222.00	115.40	142.82		143.76	0.000580	7.76	13760.84	674.97	0.30
1	8355	PF 3	122900.00	115.40	144.95		145.98	0.000563	8.15	15201.76	683.19	0.30
1	8355	PF 4	135154.00	115.40	146.59		147.68	0.000541	8.36	16332.43	689.57	0.30
1	8355	PF 5	147775.00	115.40	147.89		149.06	0.000545	8.69	17236.70	705.81	0.30
1	8355	PF 6	177142.00	115.40	151.26		152.57	0.000521	9.21	19729.50	762.95	0.30
1	8100	PF 1	46216.00	113.20	131.53		132.23	0.000825	6.75	7012.72	572.67	0.33
1	8100	PF 2	106222.00	113.20	142.56		143.60	0.000549	8.28	13685.91	637.08	0.30
1	8100	PF 3	122900.00	113.20	144.65		145.82	0.000553	8.79	15027.96	649.26	0.30
1	8100	PF 4	135154.00	113.20	146.27		147.52	0.000545	9.08	16092.10	658.76	0.30
1	8100	PF 5	147775.00	113.20	147.54		148.90	0.000559	9.48	16933.51	668.27	0.31
1	8100	PF 6	177142.00	113.20	150.85		152.41	0.000557	10.16	19224.21	716.10	0.31
1	7250	PF 1	46216.00	115.70	130.94	124.37	131.53	0.000750	6.17	7956.81	771.15	0.31
1	7250	PF 2	106222.00	115.70	142.40	128.64	143.11	0.000396	6.97	18441.59	1058.96	0.25
1	7250	PF 3	122900.00	115.70	144.53	129.65	145.31	0.000390	7.32	20757.12	1112.52	0.25
1	7250	PF 4	135154.00	115.70	146.20	130.36	147.01	0.000377	7.50	22646.97	1154.40	0.25
1	7250	PF 5	147775.00	115.70	147.50	131.08	148.36	0.000382	7.78	24168.93	1187.06	0.26
1	7250	PF 6	177142.00	115.70	150.90	132.65	151.85	0.000366	8.21	28350.09	1272.46	0.25

Howland, Penobscot River Bridge - Alternate 3
standard Table 2

HEC-RAS Plan: Plan 07 River: Penobscot Reach: 1

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Vel Head (ft)	Frctn Loss (ft)	C & E Loss (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Top Width (ft)
1	10450	PF 1	134.51	134.07	0.44	0.34	0.01		46216.00		846.39
1	10450	PF 2	144.79	144.23	0.56	0.16	0.00	23.85	106198.10	0.00	931.07
1	10450	PF 3	146.98	146.38	0.61	0.15	0.00	70.86	122786.30	42.87	1045.40
1	10450	PF 4	148.65	148.02	0.63	0.14	0.00	128.15	134838.70	187.16	1132.96
1	10450	PF 5	150.05	149.38	0.67	0.14	0.00	193.88	147163.70	417.47	1205.02
1	10450	PF 6	153.55	152.83	0.72	0.13	0.00	430.44	175048.90	1662.70	1360.48
1	10050	PF 1	134.16	133.64	0.52	0.06	0.01		46216.00		897.77
1	10050	PF 2	144.63	144.06	0.56	0.02	0.00		106163.50	58.48	978.47
1	10050	PF 3	146.83	146.23	0.60	0.02	0.00	0.01	122757.40	142.57	1002.20
1	10050	PF 4	148.51	147.89	0.62	0.02	0.00	1.48	134914.80	237.78	1027.20
1	10050	PF 5	149.91	149.24	0.66	0.02	0.01	6.21	147427.00	341.83	1066.48
1	10050	PF 6	153.42	152.71	0.71	0.02	0.01	40.92	175846.60	1254.51	1462.76
1	10020	PF 1	134.10	133.53	0.57	0.05	0.00		46216.00		895.65
1	10020	PF 2	144.60	144.00	0.60	0.02	0.00	0.84	106214.60	6.55	913.18
1	10020	PF 3	146.81	146.16	0.64	0.02	0.00	1.42	122887.80	10.78	914.01
1	10020	PF 4	148.49	147.82	0.67	0.02	0.00	1.98	135137.20	14.78	914.64
1	10020	PF 5	149.88	149.17	0.71	0.01	0.00	2.56	147753.50	18.91	915.16
1	10020	PF 6	153.40	152.62	0.78	0.01	0.00	4.35	176744.40	393.22	1362.48
1	9980	PF 1	134.05	133.49	0.57	0.08	0.03		46216.00		894.87
1	9980	PF 2	144.58	143.99	0.59	0.03	0.02	0.36	106221.60		916.76
1	9980	PF 3	146.79	146.15	0.63	0.03	0.02	0.61	122899.40		916.98
1	9980	PF 4	148.47	147.81	0.66	0.03	0.02	0.85	135153.20		917.15
1	9980	PF 5	149.87	149.16	0.70	0.03	0.02	1.09	147773.90		917.29
1	9980	PF 6	153.38	152.61	0.77	0.03	0.02	1.87	176777.10	363.01	1366.96
1	9900	PF 1	133.94	133.48	0.46	0.61	0.01		46216.00		902.35
1	9900	PF 2	144.53	144.01	0.52	0.28	0.01	3.69	106208.00	10.30	964.06
1	9900	PF 3	146.74	146.17	0.57	0.26	0.01	13.71	122856.10	30.19	972.01
1	9900	PF 4	148.42	147.83	0.59	0.25	0.01	26.96	135072.50	54.52	999.11
1	9900	PF 5	149.82	149.19	0.63	0.25	0.01	42.63	147626.00	106.37	1059.09
1	9900	PF 6	153.33	152.64	0.69	0.23	0.01	101.68	176526.20	514.14	1227.84
1	9184	PF 1	133.33	132.80	0.53	0.52	0.01		46216.00		777.82
1	9184	PF 2	144.24	143.61	0.63	0.27	0.03	86.61	106135.40		856.29
1	9184	PF 3	146.46	145.78	0.69	0.26	0.03	184.91	122713.20	1.85	905.88
1	9184	PF 4	148.16	147.44	0.72	0.25	0.03	290.06	134794.10	69.85	1003.10
1	9184	PF 5	149.55	148.79	0.76	0.25	0.04	401.68	147162.70	210.61	1022.60
1	9184	PF 6	153.09	152.25	0.83	0.24	0.04	775.15	175542.70	824.18	1072.61
1	8600	PF 1	132.80	132.13	0.67	0.20	0.01		46189.48	26.52	629.14
1	8600	PF 2	143.95	143.06	0.89	0.11	0.00	43.47	105553.80	624.75	698.24
1	8600	PF 3	146.17	145.19	0.98	0.11	0.00	86.94	121932.70	880.40	709.75
1	8600	PF 4	147.87	146.84	1.04	0.10	0.00	132.80	133915.50	1105.74	718.66
1	8600	PF 5	149.27	148.15	1.12	0.11	0.00	181.10	146286.20	1307.75	727.97
1	8600	PF 6	152.81	151.54	1.26	0.10	0.00	343.58	175045.40	1753.05	807.38
1	8405	PF 1	132.59	131.82	0.76				46216.00		623.27
1	8405	PF 2	143.83	142.91	0.93	0.01	0.01	17.98	106141.40	62.65	675.29
1	8405	PF 3	146.06	145.04	1.02	0.01	0.01	44.22	122736.50	119.24	683.54
1	8405	PF 4	147.76	146.69	1.08	0.01	0.01	74.51	134901.90	177.54	689.94
1	8405	PF 5	149.16	148.00	1.16	0.01	0.02	107.43	147466.80	200.77	707.52
1	8405	PF 6	152.70	151.40	1.30	0.01	0.02	224.71	176432.90	484.35	764.25
1	8380		Bridge								
1	8355	PF 1	132.50	131.73	0.78	0.25	0.02		46216.00		622.72
1	8355	PF 2	143.76	142.82	0.93	0.14	0.01	17.34	106143.40	61.25	674.97
1	8355	PF 3	145.98	144.95	1.03	0.14	0.01	43.06	122739.90	117.05	683.19
1	8355	PF 4	147.68	146.59	1.08	0.14	0.02	72.88	134906.40	174.71	689.57
1	8355	PF 5	149.06	147.89	1.17	0.14	0.02	105.27	147470.30	199.39	705.81
1	8355	PF 6	152.57	151.26	1.31	0.14	0.02	220.32	176454.30	467.37	762.95
1	8100	PF 1	132.23	131.53	0.70	0.67	0.04	239.49	45944.18	32.33	572.67
1	8100	PF 2	143.60	142.56	1.04	0.39	0.10	1586.44	104072.70	562.89	637.08
1	8100	PF 3	145.82	144.65	1.17	0.39	0.12	2050.44	120068.00	781.60	649.26

HEC-RAS Plan: Plan 07 River: Penobscot Reach: 1 (Continued)

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Vel Head (ft)	Frctn Loss (ft)	C & E Loss (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Top Width (ft)
1	8100	PF 4	147.52	146.27	1.25	0.38	0.13	2437.11	131742.90	973.95	658.76
1	8100	PF 5	148.90	147.54	1.36	0.39	0.15	2819.39	143817.20	1138.45	668.27
1	8100	PF 6	152.41	150.85	1.56	0.38	0.18	3859.83	171781.60	1500.52	716.10
1	7250	PF 1	131.53	130.94	0.58			512.95	45518.17	184.88	771.15
1	7250	PF 2	143.11	142.40	0.71			3456.23	99407.26	3358.52	1058.96
1	7250	PF 3	145.31	144.53	0.77			4407.28	113842.60	4650.15	1112.52
1	7250	PF 4	147.01	146.20	0.81			5184.68	124189.60	5779.75	1154.40
1	7250	PF 5	148.36	147.50	0.86			5949.06	134935.30	6890.60	1187.06
1	7250	PF 6	151.85	150.90	0.95			7974.87	159111.00	10056.08	1272.46

Howland, Penobscot River Bridge - Alternate 3

Six XS Bridge Table

HEC-RAS Plan: Plan 07 River: Penobscot Reach: 1

Reach	River Sta	Profile	E.G. Elev (ft)	W.S. Elev (ft)	Crit W.S. (ft)	Frctn Loss (ft)	C & E Loss (ft)	Top Width (ft)	Q Left (cfs)	Q Channel (cfs)	Q Right (cfs)	Vel Chnl (ft/s)
1	8600	PF 1	132.80	132.13		0.20	0.01	629.14		46189.48	26.52	6.55
1	8600	PF 2	143.95	143.06		0.11	0.00	698.24	43.47	105553.80	624.75	7.58
1	8600	PF 3	146.17	145.19		0.11	0.00	709.75	86.94	121932.70	880.40	7.98
1	8600	PF 4	147.87	146.84		0.10	0.00	718.66	132.80	133915.50	1105.74	8.21
1	8600	PF 5	149.27	148.15		0.11	0.00	727.97	181.10	146286.20	1307.75	8.53
1	8600	PF 6	152.81	151.54		0.10	0.00	807.38	343.58	175045.40	1753.05	9.07
1	8405	PF 1	132.59	131.82	126.47			623.27		46216.00		7.01
1	8405	PF 2	143.83	142.91	130.86	0.01	0.01	675.29	17.98	106141.40	62.65	7.73
1	8405	PF 3	146.06	145.04	131.88	0.01	0.01	683.54	44.22	122736.50	119.24	8.12
1	8405	PF 4	147.76	146.69	132.58	0.01	0.01	689.94	74.51	134901.90	177.54	8.33
1	8405	PF 5	149.16	148.00	133.33	0.01	0.02	707.52	107.43	147466.80	200.77	8.65
1	8405	PF 6	152.70	151.40	134.90	0.01	0.02	764.25	224.71	176432.90	484.35	9.16
1	8380 BR U	PF 1	132.56	131.75	126.58			601.50		46216.00		7.20
1	8380 BR U	PF 2	143.82	142.79	130.95	0.02	0.00	601.58		106222.00		8.14
1	8380 BR U	PF 3	146.04	144.90	132.00	0.02	0.00	601.60		122900.00		8.58
1	8380 BR U	PF 4	147.74	146.53	132.70	0.02	0.00	601.61		135154.00		8.83
1	8380 BR U	PF 5	149.13	147.82	133.41	0.02	0.00	601.62		147775.00		9.19
1	8380 BR U	PF 6	152.67	151.14	134.99	0.03	0.01	416.87		177142.00		9.92
1	8380 BR D	PF 1	132.52	131.70	126.58			601.36		46216.00		7.24
1	8380 BR D	PF 2	143.79	142.76	130.95	0.01	0.03	601.58		106222.00		8.15
1	8380 BR D	PF 3	146.02	144.87	132.00	0.01	0.03	601.60		122900.00		8.59
1	8380 BR D	PF 4	147.72	146.51	132.70	0.01	0.04	601.61		135154.00		8.84
1	8380 BR D	PF 5	149.11	147.80	133.42	0.01	0.04	601.62		147775.00		9.20
1	8380 BR D	PF 6	152.63	151.14	134.99	0.01	0.05	601.65		177142.00		9.80
1	8355	PF 1	132.50	131.73		0.25	0.02	622.72		46216.00		7.07
1	8355	PF 2	143.76	142.82		0.14	0.01	674.97	17.34	106143.40	61.25	7.76
1	8355	PF 3	145.98	144.95		0.14	0.01	683.19	43.06	122739.90	117.05	8.15
1	8355	PF 4	147.68	146.59		0.14	0.02	689.57	72.88	134906.40	174.71	8.36
1	8355	PF 5	149.06	147.89		0.14	0.02	705.81	105.27	147470.30	199.39	8.69
1	8355	PF 6	152.57	151.26		0.14	0.02	762.95	220.32	176454.30	467.37	9.21
1	8100	PF 1	132.23	131.53		0.67	0.04	572.67	239.49	45944.18	32.33	6.75
1	8100	PF 2	143.60	142.56		0.39	0.10	637.08	1586.44	104072.70	562.89	8.28
1	8100	PF 3	145.82	144.65		0.39	0.12	649.26	2050.44	120068.00	781.60	8.79
1	8100	PF 4	147.52	146.27		0.38	0.13	658.76	2437.11	131742.90	973.95	9.08
1	8100	PF 5	148.90	147.54		0.39	0.15	668.27	2819.39	143817.20	1138.45	9.48
1	8100	PF 6	152.41	150.85		0.38	0.18	716.10	3859.83	171781.60	1500.52	10.16

Howland, Penobscot River Bridge - Alternate 3

Bridge XS Tables, Q1.1, Q50, Q100/Q500

Plan: Plan 07 Penobscot 1 RS: 8380 Profile: PF 1

E.G. US. (ft)	132.59	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	131.82	E.G. Elev (ft)	132.56	132.52
Q Total (cfs)	46216.00	W.S. Elev (ft)	131.75	131.70
Q Bridge (cfs)	46216.00	Crit W.S. (ft)	126.58	126.58
Q Weir (cfs)		Max Chl Dpth (ft)	16.35	16.30
Weir Sta Lft (ft)		Vel Total (ft/s)	7.20	7.24
Weir Sta Rgt (ft)		Flow Area (sq ft)	6416.06	6386.40
Weir Submerg		Froude # Chl	0.39	0.39
Weir Max Depth (ft)		Specif Force (cu ft)	48923.13	48655.46
Min El Weir Flow (ft)	158.01	Hydr Depth (ft)	10.67	10.62
Min El Prs (ft)	156.32	W.P. Total (ft)	659.08	658.69
Delta EG (ft)	0.08	Conv. Total (cfs)	1241885.0	1232817.0
Delta WS (ft)	0.10	Top Width (ft)	601.50	601.36
BR Open Area (sq ft)	18940.25	Frctn Loss (ft)		
BR Open Vel (ft/s)	7.24	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)	0.84	0.85
Br Sel Method	Momentum	Power Total (lb/ft s)	6.06	6.16

Plan: Plan 07 Penobscot 1 RS: 8380 Profile: PF 4

E.G. US. (ft)	147.76	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	146.69	E.G. Elev (ft)	147.74	147.72
Q Total (cfs)	135154.00	W.S. Elev (ft)	146.53	146.51
Q Bridge (cfs)	135154.00	Crit W.S. (ft)	132.70	132.70
Q Weir (cfs)		Max Chl Dpth (ft)	31.13	31.11
Weir Sta Lft (ft)		Vel Total (ft/s)	8.83	8.84
Weir Sta Rgt (ft)		Flow Area (sq ft)	15307.78	15292.60
Weir Submerg		Froude # Chl	0.31	0.31
Weir Max Depth (ft)		Specif Force (cu ft)	236193.60	235844.20
Min El Weir Flow (ft)	158.01	Hydr Depth (ft)	25.44	25.42
Min El Prs (ft)	156.32	W.P. Total (ft)	747.78	747.62
Delta EG (ft)	0.09	Conv. Total (cfs)	4863320.0	4855942.0
Delta WS (ft)	0.10	Top Width (ft)	601.61	601.61
BR Open Area (sq ft)	18940.25	Frctn Loss (ft)	0.02	0.01
BR Open Vel (ft/s)	8.84	C & E Loss (ft)	0.00	0.04
Coef of Q		Shear Total (lb/sq ft)	0.99	0.99
Br Sel Method	Energy only	Power Total (lb/ft s)	8.71	8.74

Plan: Plan 07 Penobscot 1 RS: 8380 Profile: PF 5

E.G. US. (ft)	149.16	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	148.00	E.G. Elev (ft)	149.13	149.11
Q Total (cfs)	147775.00	W.S. Elev (ft)	147.82	147.80
Q Bridge (cfs)	147775.00	Crit W.S. (ft)	133.41	133.42
Q Weir (cfs)		Max Chl Dpth (ft)	32.42	32.40
Weir Sta Lft (ft)		Vel Total (ft/s)	9.19	9.20
Weir Sta Rgt (ft)		Flow Area (sq ft)	16083.61	16067.99
Weir Submerg		Froude # Chl	0.31	0.31
Weir Max Depth (ft)		Specif Force (cu ft)	261541.80	261165.30
Min El Weir Flow (ft)	158.01	Hydr Depth (ft)	26.73	26.71
Min El Prs (ft)	156.32	W.P. Total (ft)	755.51	755.36
Delta EG (ft)	0.09	Conv. Total (cfs)	5244908.0	5237144.0
Delta WS (ft)	0.10	Top Width (ft)	601.62	601.62
BR Open Area (sq ft)	18940.25	Frctn Loss (ft)	0.02	0.01
BR Open Vel (ft/s)	9.20	C & E Loss (ft)	0.00	0.04

Plan: Plan 07 Penobscot 1 RS: 8380 Profile: PF 5 (Continued)

Coef of Q		Shear Total (lb/sq ft)	1.06	1.06
Br Sel Method	Energy only	Power Total (lb/ft s)	9.69	9.72

Plan: Plan 07 Penobscot 1 RS: 8380 Profile: PF 6

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	152.70	E.G. Elev (ft)	152.67	152.63
W.S. US. (ft)	151.40	W.S. Elev (ft)	151.14	151.14
Q Total (cfs)	177142.00	Crit W.S. (ft)	134.99	134.99
Q Bridge (cfs)	177142.00	Max Chl Dpth (ft)	35.74	35.74
Q Weir (cfs)		Vel Total (ft/s)	9.92	9.80
Weir Sta Lft (ft)		Flow Area (sq ft)	17862.52	18079.21
Weir Sta Rgt (ft)		Froude # Chl	0.29	0.31
Weir Submerg		Specif Force (cu ft)	330471.10	329936.40
Weir Max Depth (ft)		Hydr Depth (ft)	42.85	30.05
Min El Weir Flow (ft)	158.01	W.P. Total (ft)	957.42	775.42
Min El Prs (ft)	156.32	Conv. Total (cfs)	5334537.0	6264263.0
Delta EG (ft)	0.13	Top Width (ft)	416.87	601.65
Delta WS (ft)	0.14	Frctn Loss (ft)	0.03	0.01
BR Open Area (sq ft)	18940.25	C & E Loss (ft)	0.01	0.05
BR Open Vel (ft/s)	9.92	Shear Total (lb/sq ft)	1.28	1.16
Coef of Q		Power Total (lb/ft s)	12.74	11.40
Br Sel Method	Energy only			

Howland, Penobscot River Bridge - Alternate 3

Cross Section Tables - Q100 & Q500

Plan: Plan 07 Penobscot 1 RS: 8600 Profile: PF 5 U/S Approach Section (4)

E.G. Elev (ft)	149.27	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.12	Wt. n-Val.	0.080	0.035	0.080
W.S. Elev (ft)	148.15	Reach Len. (ft)	195.00	195.00	1985.00
Crit W.S. (ft)		Flow Area (sq ft)	142.10	17144.65	707.55
E.G. Slope (ft/ft)	0.000498	Area (sq ft)	142.10	17144.65	707.55
Q Total (cfs)	147775.00	Flow (cfs)	181.10	146286.20	1307.75
Top Width (ft)	727.97	Top Width (ft)	23.40	632.20	72.37
Vel Total (ft/s)	8.21	Avg. Vel. (ft/s)	1.27	8.53	1.85
Max Chl Dpth (ft)	31.95	Hydr. Depth (ft)	6.07	27.12	9.78
Conv. Total (cfs)	6621050.0	Conv. (cfs)	8114.1	6554343.0	58593.6
Length Wtd. (ft)	204.14	Wetted Per. (ft)	26.36	634.48	75.16
Min Ch El (ft)	116.20	Shear (lb/sq ft)	0.17	0.84	0.29
Alpha	1.07	Stream Power (lb/ft s)	0.21	7.17	0.54
Frctn Loss (ft)	0.11	Cum Volume (acre-ft)	43.79	506.49	66.71
C & E Loss (ft)	0.00	Cum SA (acres)	3.32	17.97	7.07

Plan: Plan 07 Penobscot 1 RS: 8600 Profile: PF 6

E.G. Elev (ft)	152.81	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.26	Wt. n-Val.	0.080	0.035	0.080
W.S. Elev (ft)	151.54	Reach Len. (ft)	195.00	195.00	1985.00
Crit W.S. (ft)		Flow Area (sq ft)	232.70	19292.19	1118.20
E.G. Slope (ft/ft)	0.000481	Area (sq ft)	232.70	19292.19	1118.20
Q Total (cfs)	177142.00	Flow (cfs)	343.58	175045.40	1753.05
Top Width (ft)	807.38	Top Width (ft)	29.94	632.20	145.24
Vel Total (ft/s)	8.58	Avg. Vel. (ft/s)	1.48	9.07	1.57
Max Chl Dpth (ft)	35.34	Hydr. Depth (ft)	7.77	30.52	7.70
Conv. Total (cfs)	8074613.0	Conv. (cfs)	15661.3	7979043.0	79908.9
Length Wtd. (ft)	206.30	Wetted Per. (ft)	33.74	634.48	148.17
Min Ch El (ft)	116.20	Shear (lb/sq ft)	0.21	0.91	0.23
Alpha	1.11	Stream Power (lb/ft s)	0.31	8.29	0.36
Frctn Loss (ft)	0.10	Cum Volume (acre-ft)	55.68	566.79	99.00
C & E Loss (ft)	0.00	Cum SA (acres)	3.74	17.88	11.11

Plan: Plan 07 Penobscot 1 RS: 8405 Profile: PF 5 Immediate U/S contracted section (3)

E.G. Elev (ft)	149.16	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.16	Wt. n-Val.	0.080	0.035	0.080
W.S. Elev (ft)	148.00	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	133.33	Flow Area (sq ft)	92.02	17039.60	178.44
E.G. Slope (ft/ft)	0.000538	Area (sq ft)	92.02	17039.60	178.44
Q Total (cfs)	147775.00	Flow (cfs)	107.43	147466.80	200.77
Top Width (ft)	707.52	Top Width (ft)	17.87	650.00	39.64
Vel Total (ft/s)	8.54	Avg. Vel. (ft/s)	1.17	8.65	1.13
Max Chl Dpth (ft)	32.60	Hydr. Depth (ft)	5.15	26.21	4.50
Conv. Total (cfs)	6370551.0	Conv. (cfs)	4631.4	6357264.0	8655.3
Length Wtd. (ft)	10.00	Wetted Per. (ft)	20.63	654.08	42.28
Min Ch El (ft)	115.40	Shear (lb/sq ft)	0.15	0.88	0.14
Alpha	1.03	Stream Power (lb/ft s)	0.17	7.57	0.16
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	43.27	429.98	46.52
C & E Loss (ft)	0.02	Cum SA (acres)	3.23	15.10	4.51

Plan: Plan 07 Penobscot 1 RS: 8405 Profile: PF 6

E.G. Elev (ft)	152.70	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.30	Wt. n-Val.	0.080	0.035	0.080
W.S. Elev (ft)	151.40	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	134.90	Flow Area (sq ft)	162.98	19253.82	421.42
E.G. Slope (ft/ft)	0.000513	Area (sq ft)	162.98	19253.82	421.42
Q Total (cfs)	177142.00	Flow (cfs)	224.71	176432.90	484.35
Top Width (ft)	764.25	Top Width (ft)	23.79	650.00	90.46
Vel Total (ft/s)	8.93	Avg. Vel. (ft/s)	1.38	9.16	1.15
Max Chl Dpth (ft)	36.00	Hydr. Depth (ft)	6.85	29.62	4.66
Conv. Total (cfs)	7824226.0	Conv. (cfs)	9925.1	7792908.0	21393.2
Length Wtd. (ft)	10.00	Wetted Per. (ft)	27.45	654.08	93.27
Min Ch El (ft)	115.40	Shear (lb/sq ft)	0.19	0.94	0.14
Alpha	1.05	Stream Power (lb/ft s)	0.26	8.63	0.17
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	54.79	480.52	63.92
C & E Loss (ft)	0.02	Cum SA (acres)	3.62	15.01	5.74

Plan: Plan 07 Penobscot 1 RS: 8380 BR U Profile: PF 5 U/S Bridge Section

E.G. Elev (ft)	149.13	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.31	Wt. n-Val.		0.035	
W.S. Elev (ft)	147.82	Reach Len. (ft)	30.00	30.00	30.00
Crit W.S. (ft)	133.41	Flow Area (sq ft)		16083.61	
E.G. Slope (ft/ft)	0.000794	Area (sq ft)		16083.61	
Q Total (cfs)	147775.00	Flow (cfs)		147775.00	
Top Width (ft)	601.62	Top Width (ft)		601.62	
Vel Total (ft/s)	9.19	Avg. Vel. (ft/s)		9.19	
Max Chl Dpth (ft)	32.42	Hydr. Depth (ft)		26.73	
Conv. Total (cfs)	5244908.0	Conv. (cfs)		5244908.0	
Length Wtd. (ft)	30.00	Wetted Per. (ft)		755.51	
Min Ch El (ft)	115.40	Shear (lb/sq ft)		1.06	
Alpha	1.00	Stream Power (lb/ft s)		9.69	
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	43.26	426.18	46.50
C & E Loss (ft)	0.00	Cum SA (acres)	3.22	14.96	4.51

Plan: Plan 07 Penobscot 1 RS: 8380 BR U Profile: PF 6

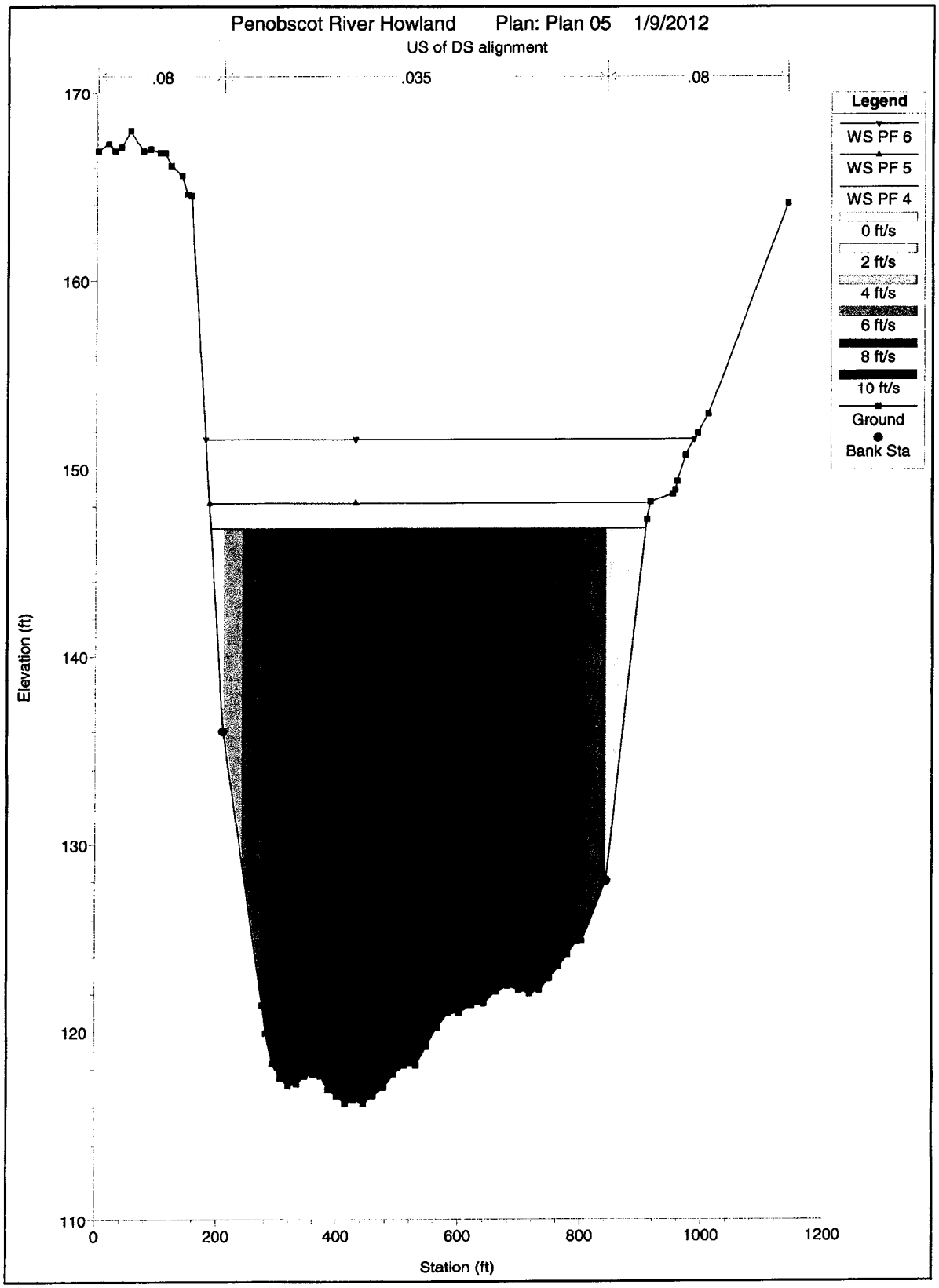
E.G. Elev (ft)	152.67	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.53	Wt. n-Val.		0.035	
W.S. Elev (ft)	151.14	Reach Len. (ft)	30.00	30.00	30.00
Crit W.S. (ft)	134.99	Flow Area (sq ft)		17862.52	
E.G. Slope (ft/ft)	0.001103	Area (sq ft)		17862.52	
Q Total (cfs)	177142.00	Flow (cfs)		177142.00	
Top Width (ft)	416.87	Top Width (ft)		416.87	
Vel Total (ft/s)	9.92	Avg. Vel. (ft/s)		9.92	
Max Chl Dpth (ft)	35.74	Hydr. Depth (ft)		42.85	
Conv. Total (cfs)	5334537.0	Conv. (cfs)		5334537.0	
Length Wtd. (ft)	30.00	Wetted Per. (ft)		957.42	
Min Ch El (ft)	115.40	Shear (lb/sq ft)		1.28	
Alpha	1.00	Stream Power (lb/ft s)		12.74	
Frctn Loss (ft)	0.03	Cum Volume (acre-ft)	54.77	476.26	63.87
C & E Loss (ft)	0.01	Cum SA (acres)	3.61	14.89	5.73

Plan: Plan 07 Penobscot 1 RS: 8380 BR D Profile: PF 5 D/S Bridge Section

E.G. Elev (ft)	149.11	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.31	Wt. n-Val.		0.035	
W.S. Elev (ft)	147.80	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	133.42	Flow Area (sq ft)		16067.99	
E.G. Slope (ft/ft)	0.000796	Area (sq ft)		16067.99	
Q Total (cfs)	147775.00	Flow (cfs)		147775.00	
Top Width (ft)	601.62	Top Width (ft)		601.62	
Vel Total (ft/s)	9.20	Avg. Vel. (ft/s)		9.20	
Max Chl Dpth (ft)	32.40	Hydr. Depth (ft)		26.71	
Conv. Total (cfs)	5237144.0	Conv. (cfs)		5237144.0	
Length Wtd. (ft)	10.00	Wetted Per. (ft)		755.36	
Min Ch El (ft)	115.40	Shear (lb/sq ft)		1.06	
Alpha	1.00	Stream Power (lb/ft s)		9.72	
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	43.26	415.10	46.50
C & E Loss (ft)	0.04	Cum SA (acres)	3.22	14.54	4.51

Plan: Plan 07 Penobscot 1 RS: 8380 BR D Profile: PF 6

E.G. Elev (ft)	152.63	Element	Left OB	Channel	Right OB
Vel Head (ft)	1.49	Wt. n-Val.		0.035	
W.S. Elev (ft)	151.14	Reach Len. (ft)	10.00	10.00	10.00
Crit W.S. (ft)	134.99	Flow Area (sq ft)		18079.21	
E.G. Slope (ft/ft)	0.000800	Area (sq ft)		18079.21	
Q Total (cfs)	177142.00	Flow (cfs)		177142.00	
Top Width (ft)	601.65	Top Width (ft)		601.65	
Vel Total (ft/s)	9.80	Avg. Vel. (ft/s)		9.80	
Max Chl Dpth (ft)	35.74	Hydr. Depth (ft)		30.05	
Conv. Total (cfs)	6264263.0	Conv. (cfs)		6264263.0	
Length Wtd. (ft)	10.00	Wetted Per. (ft)		775.42	
Min Ch El (ft)	115.40	Shear (lb/sq ft)		1.16	
Alpha	1.00	Stream Power (lb/ft s)		11.40	
Frctn Loss (ft)	0.01	Cum Volume (acre-ft)	54.77	463.88	63.87
C & E Loss (ft)	0.05	Cum SA (acres)	3.61	14.54	5.73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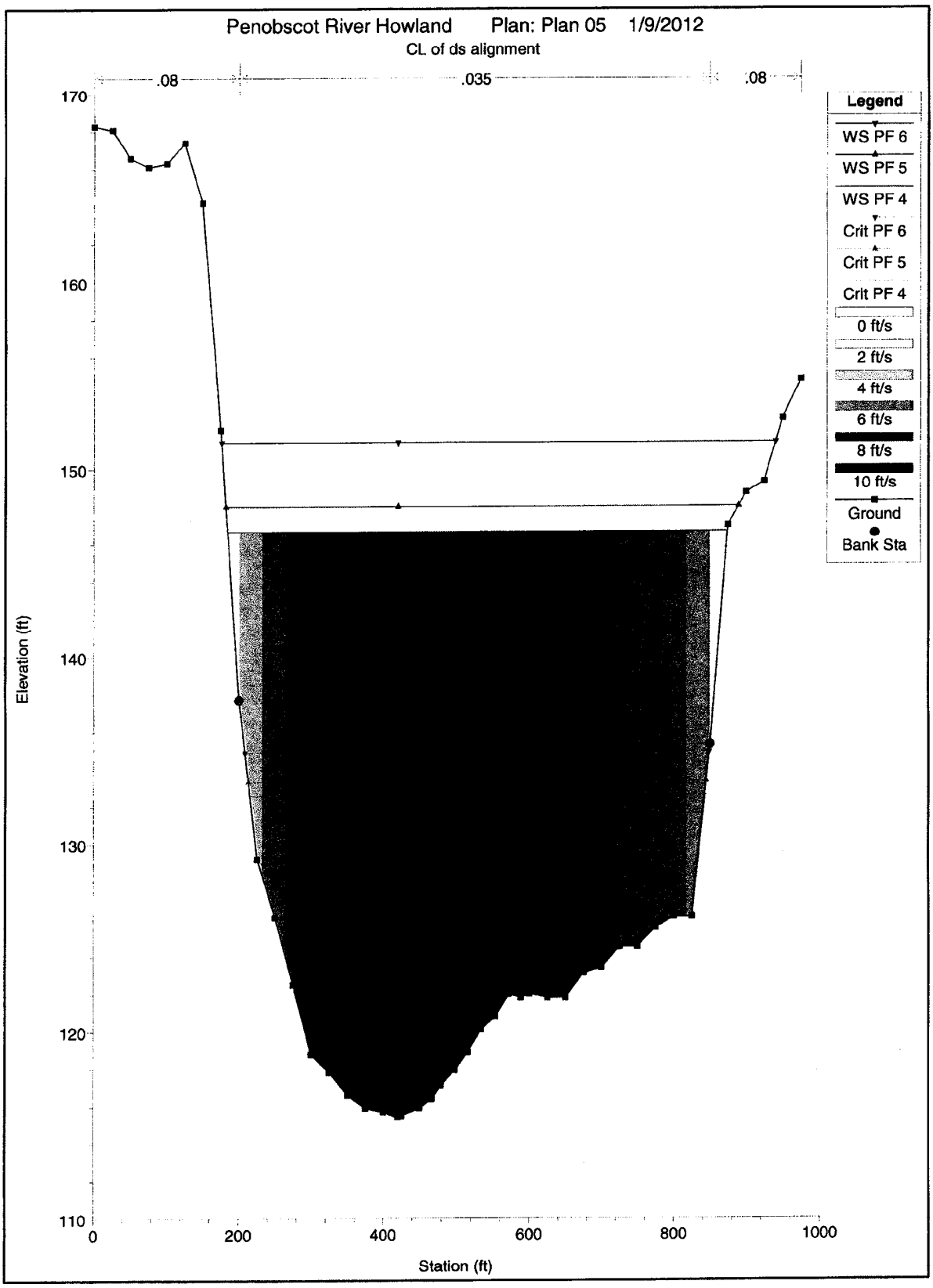
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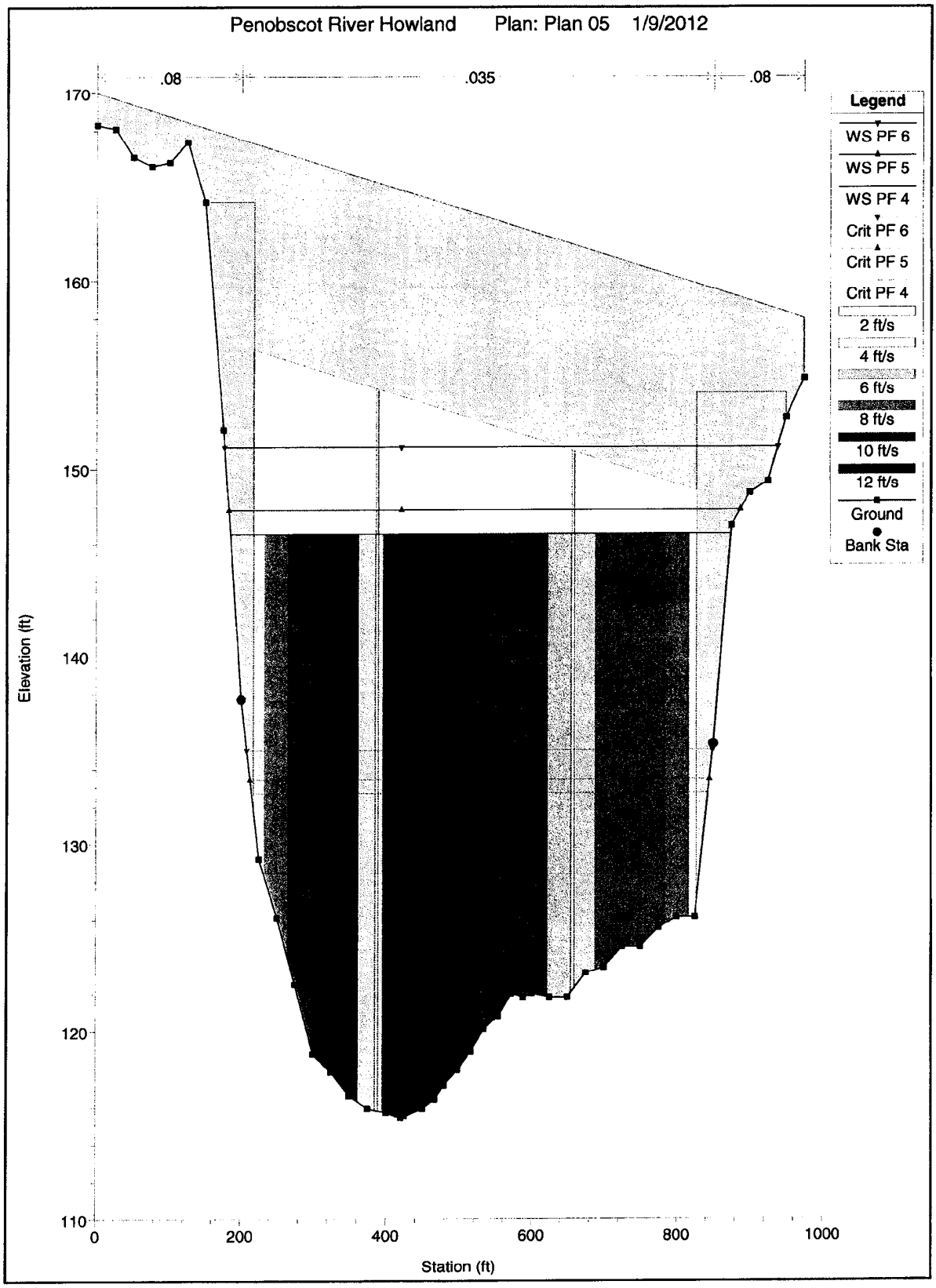
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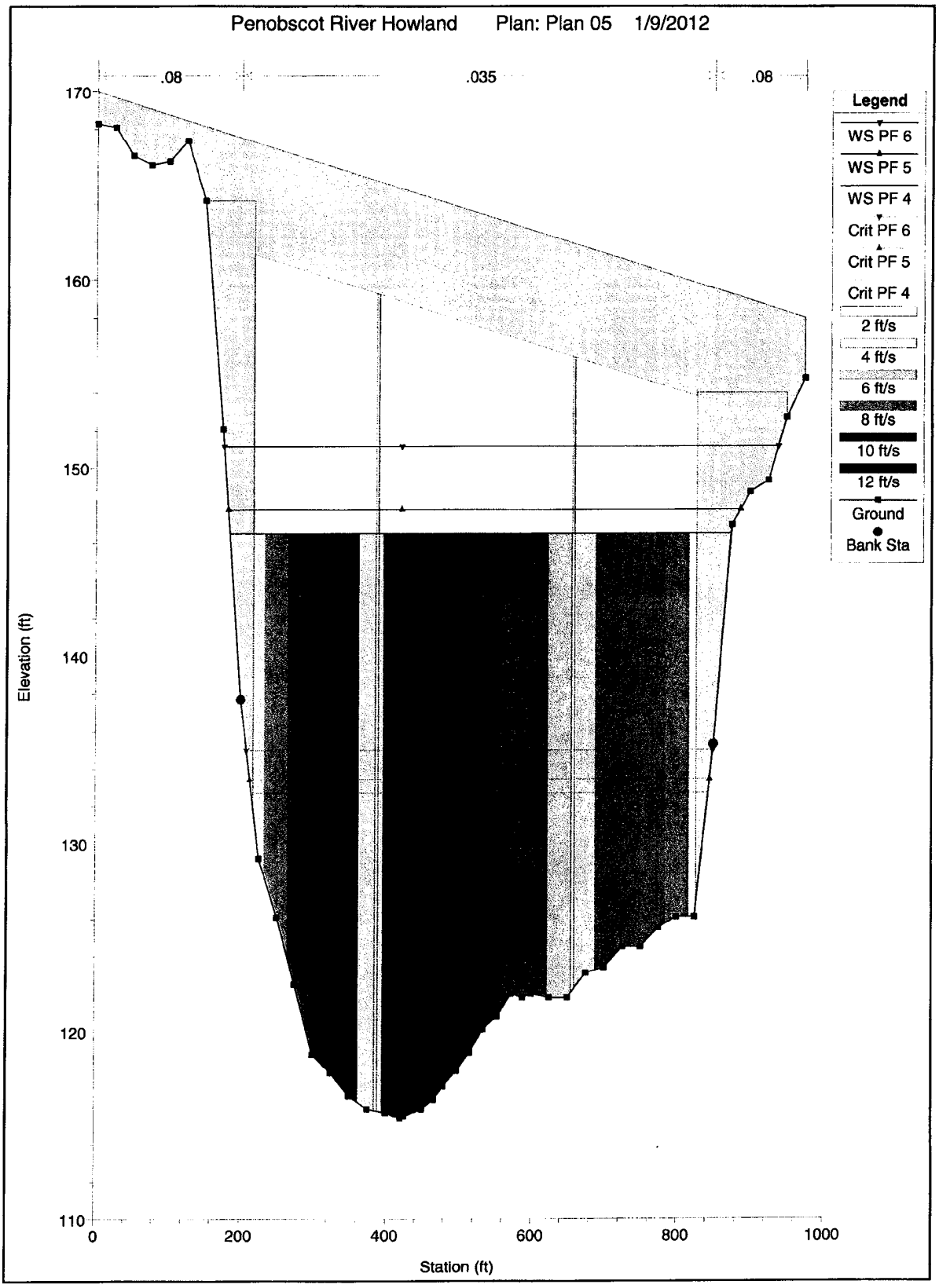
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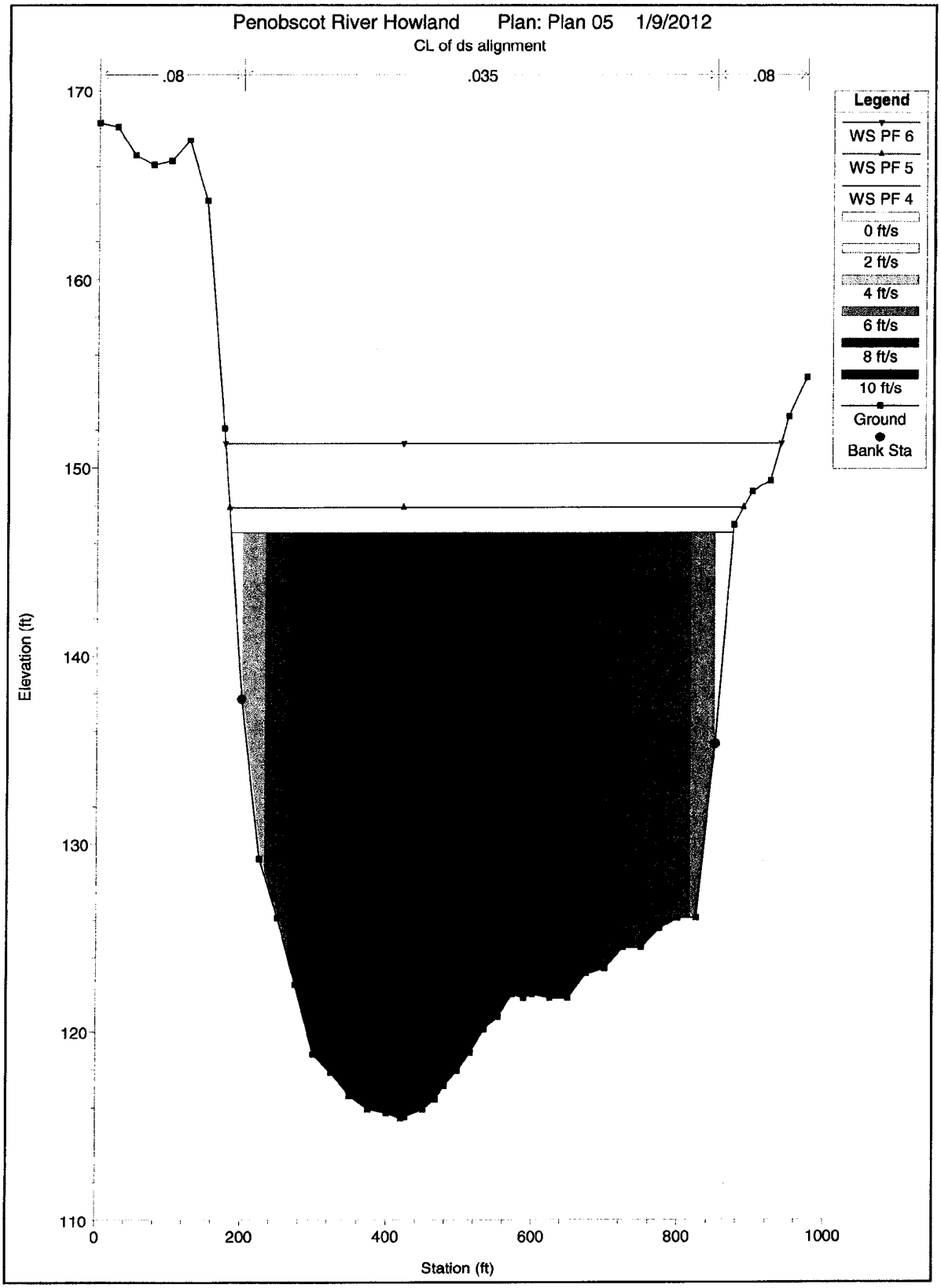
Station (ft)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Legend
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WS PF 5
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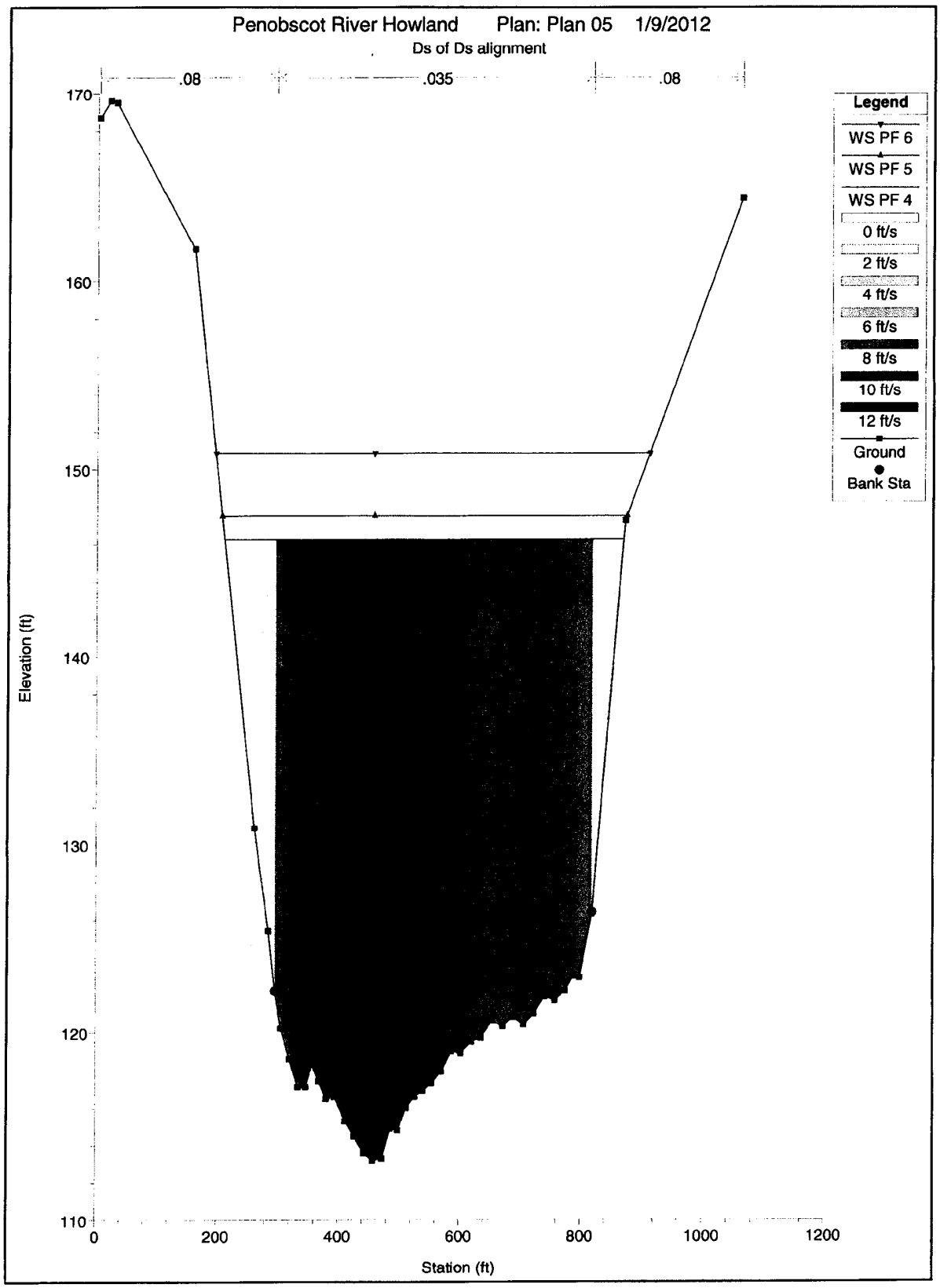
Station (ft)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Legend
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WS PF 5
WS PF 4
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6 ft/s
8 ft/s
10 ft/s
12 ft/s
Ground
Bank Sta
Elevation (ft)
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130
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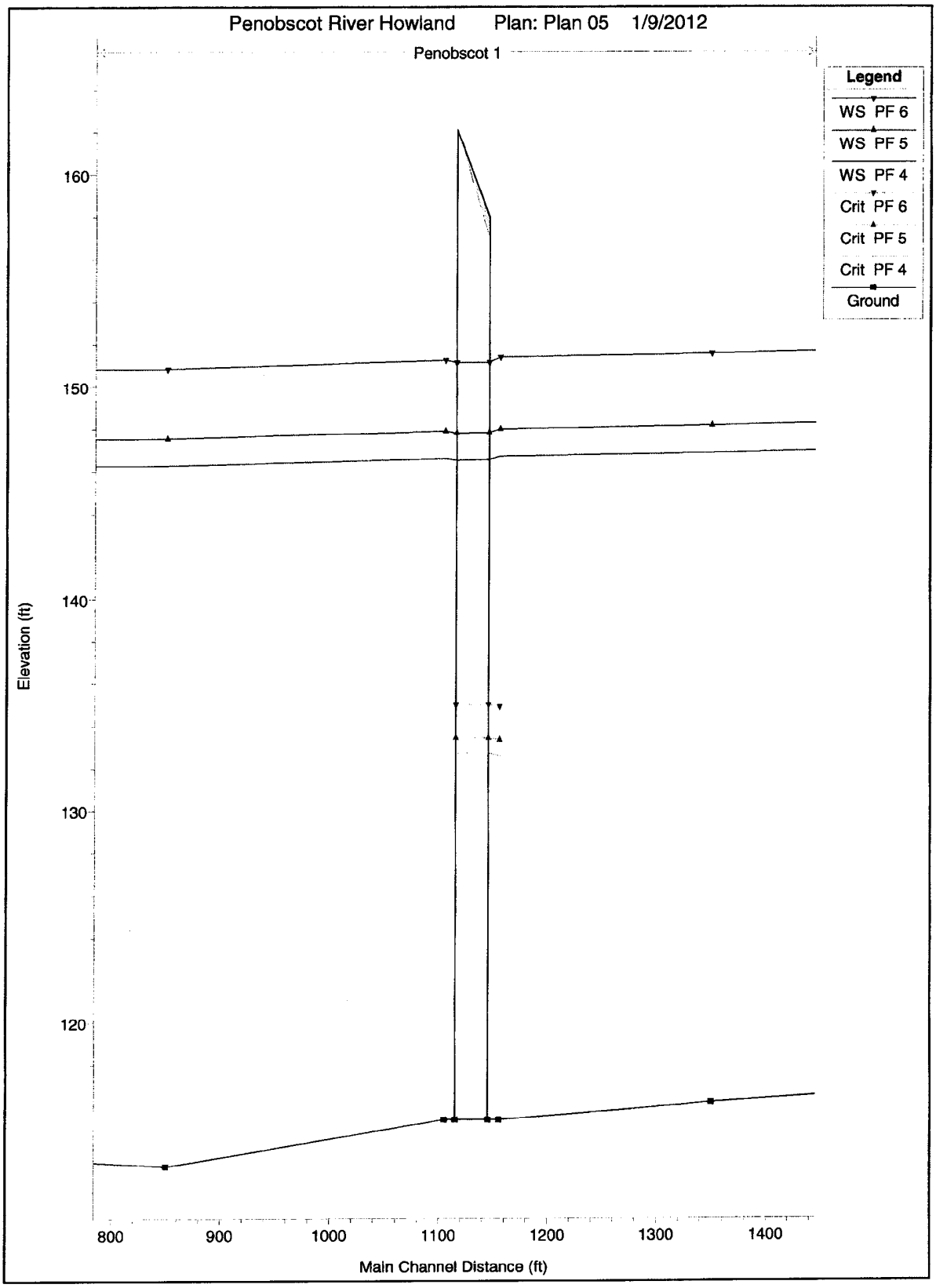
1000
1200
Station (ft)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Legend
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WS PF 5
WS PF 4
Crit PF 6
Crit PF 5
Crit PF 4
Ground
Elevation (ft)
160
150
140
130
120
800
900
1000
1100
1200
1300
1400
Main Channel Distance (ft)

Penobscot River Bridge #2660
Howland - Enfield, Maine

Hydraulics:

- **Proposed Bridge Replacement Alternate 5:**
  Hydraulic analysis and data for Bridge Replacement Alternate 5 includes the following HEC-RAS model output:
  - Standard Table 1
  - Standard Table 2
  - Six XS Bridge Table
  - Cross Section Tables
  - Cross Section Plots
  - Profile Plot

  Specific hydraulic analyses were not conducted for Alternates 2 and 4, because differences in waterway geometry and span arrangement are not significantly different than Alternate 5. Profile modifications were incorporated into Alternate 5 following the hydraulic analysis to raise the low chord to 4 ft above the Q_{50} elevation. As modeled, the Q_{500} elevation is near the modeled low chord on the far west end of the bridge. Revisions to the modeled results will not be significant for the raised low chord at this location.

Howland, Penobscot River Bridge - Alternate 5
Standard Table 1

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1

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)	
1	10450	Q 1.1	46216.00	118.80	134.04		134.49	0.000692	5.43	8681.46	846.12	0.29
1	10450	Q 10	98400.00	118.80	143.93		144.48	0.000349	5.96	17458.59	927.50	0.23
1	10450	Q 25	122900.00	118.80	146.35		147.03	0.000376	6.67	19825.54	1044.06	0.24
1	10450	Q 50	133300.00	118.80	147.91		148.61	0.000354	6.76	21522.10	1127.16	0.24
1	10450	Q 100	148800.00	118.80	149.34		150.11	0.000364	7.11	23179.78	1202.83	0.25
1	10450	Q 500	186800.00	118.80	152.95		153.86	0.000365	7.76	27845.03	1361.44	0.25
1	10050	Q 1.1	46216.00	121.70	133.61		134.15	0.001007	5.90	7945.58	897.56	0.34
1	10050	Q 10	98400.00	121.70	143.80		144.33	0.000373	5.91	17464.74	975.60	0.24
1	10050	Q 25	122900.00	121.70	146.21		146.87	0.000392	6.55	19853.94	1001.96	0.25
1	10050	Q 50	133300.00	121.70	147.79		148.46	0.000364	6.62	21451.50	1025.71	0.24
1	10050	Q 100	148800.00	121.70	149.21		149.95	0.000372	6.96	22944.13	1066.06	0.25
1	10050	Q 500	186800.00	121.70	152.83		153.70	0.000369	7.59	26885.35	1467.57	0.25
1	10020	Q 1.1	46216.00	120.59	133.51		134.09	0.001178	6.14	7586.34	895.48	0.37
1	10020	Q 10	98400.00	120.59	143.76		144.31	0.000395	5.97	16918.54	913.09	0.24
1	10020	Q 25	122900.00	120.59	146.17		146.85	0.000412	6.61	19122.85	914.01	0.25
1	10020	Q 50	133300.00	120.59	147.75		148.44	0.000382	6.67	20564.66	914.62	0.25
1	10020	Q 100	148800.00	120.59	149.17		149.93	0.000390	7.01	21863.29	915.16	0.25
1	10020	Q 500	186800.00	120.59	152.78		153.68	0.000387	7.67	25165.93	1368.61	0.26
1	9980	Q 1.1	46216.00	120.84	133.45		134.05	0.001179	6.20	7608.31	894.68	0.37
1	9980	Q 10	98400.00	120.84	143.73		144.29	0.000400	6.05	17003.49	916.73	0.24
1	9980	Q 25	122900.00	120.84	146.14		146.83	0.000420	6.70	19211.73	916.98	0.25
1	9980	Q 50	133300.00	120.84	147.72		148.42	0.000390	6.77	20658.30	917.14	0.25
1	9980	Q 100	148800.00	120.84	149.14		149.91	0.000398	7.12	21957.84	917.29	0.25
1	9980	Q 500	186800.00	120.84	152.74		153.67	0.000397	7.79	25261.71	1371.93	0.26
1	9935	Q 1.1	46216.00	121.30	133.45	128.25	133.93	0.000831	5.57	8439.14	902.17	0.31
1	9935	Q 10	98400.00	121.30	143.75	131.21	144.25	0.000333	5.71	18077.55	983.11	0.22
1	9935	Q 25	122900.00	121.30	146.16	132.39	146.78	0.000354	6.35	20412.97	971.97	0.24
1	9935	Q 50	133300.00	121.30	147.74	132.88	148.37	0.000331	6.43	21952.40	990.45	0.23
1	9935	Q 100	148800.00	121.30	149.16	133.58	149.86	0.000340	6.77	23344.87	1058.21	0.24
1	9935	Q 500	186800.00	121.30	152.77	135.18	153.61	0.000343	7.42	26920.51	1231.38	0.24
1	9910		Bridge									
1	9885	Q 1.1	46216.00	121.30	133.39		133.87	0.000851	5.61	8378.45	901.71	0.32
1	9885	Q 10	98400.00	121.30	143.71		144.21	0.000335	5.72	18042.19	962.97	0.23
1	9885	Q 25	122900.00	121.30	146.12		146.74	0.000357	6.36	20371.14	971.61	0.24
1	9885	Q 50	133300.00	121.30	147.70		148.33	0.000333	6.44	21911.34	986.49	0.23
1	9885	Q 100	148800.00	121.30	149.11		149.82	0.000342	6.78	23300.21	1056.76	0.24
1	9885	Q 500	186800.00	121.30	152.70		153.54	0.000346	7.44	26851.31	1229.42	0.24
1	9184	Q 1.1	46216.00	118.20	132.73		133.27	0.000849	5.91	7881.78	777.33	0.32
1	9184	Q 10	98400.00	118.20	143.37		143.96	0.000366	6.19	16544.31	854.41	0.24
1	9184	Q 25	122900.00	118.20	145.72		146.46	0.000400	6.95	18586.18	903.59	0.25
1	9184	Q 50	133300.00	118.20	147.31		148.07	0.000378	7.06	20120.83	1001.14	0.25
1	9184	Q 100	148800.00	118.20	148.70		149.55	0.000391	7.44	21527.73	1021.23	0.26
1	9184	Q 500	186800.00	118.20	152.25		153.27	0.000400	8.18	25244.42	1072.50	0.26
1	8600	Q 1.1	46216.00	116.20	132.06		132.74	0.000918	6.64	7038.22	628.55	0.34
1	8600	Q 10	98400.00	116.20	142.88		143.70	0.000462	7.28	14248.00	697.28	0.27
1	8600	Q 25	122900.00	116.20	145.12		146.16	0.000524	8.25	15822.67	709.39	0.29
1	8600	Q 50	133300.00	116.20	146.71		147.79	0.000500	8.40	16954.37	717.96	0.29
1	8600	Q 100	148800.00	116.20	148.03		149.25	0.000527	8.92	17909.96	727.04	0.30
1	8600	Q 500	186800.00	116.20	151.44		152.94	0.000562	9.95	20556.09	805.34	0.31
1	8380	Q 1.1	46216.00	115.40	131.72		132.50	0.001189	7.09	6532.60	622.70	0.38
1	8380	Q 10	98400.00	115.40	142.74		143.59	0.000502	7.39	13706.94	674.66	0.28
1	8380	Q 25	122900.00	115.40	144.96		146.04	0.000565	8.36	15213.80	683.26	0.30
1	8380	Q 50	133300.00	115.40	146.55		147.67	0.000535	8.49	16308.13	689.43	0.30
1	8380	Q 100	148800.00	115.40	147.87		149.12	0.000563	9.02	17221.08	705.45	0.31
1	8380	Q 500	186800.00	115.40	151.27		152.81	0.000594	10.02	19735.56	763.02	0.32
1	8100	Q 1.1	46216.00	113.20	131.50		132.21	0.000833	6.77	6996.30	572.50	0.33
1	8100	Q 10	98400.00	113.20	142.54		143.44	0.000476	7.71	13670.79	636.94	0.28
1	8100	Q 25	122900.00	113.20	144.69		145.87	0.000555	8.81	15055.98	649.52	0.30
1	8100	Q 50	133300.00	113.20	146.27		147.51	0.000535	9.00	16092.97	658.77	0.30
1	8100	Q 100	148800.00	113.20	147.55		148.95	0.000573	9.59	16937.44	668.35	0.31
1	8100	Q 500	186800.00	113.20	150.86		152.62	0.000627	10.78	19228.02	716.17	0.33
1	7250	Q 1.1	46216.00	115.70	130.92	124.36	131.50	0.000750	6.12	7938.39	770.55	0.31
1	7250	Q 10	98400.00	115.70	142.40	128.05	143.02	0.000341	6.44	18441.59	1058.96	0.23
1	7250	Q 25	122900.00	115.70	144.58	129.54	145.36	0.000390	7.30	20805.03	1113.61	0.25
1	7250	Q 50	133300.00	115.70	146.20	130.17	147.00	0.000370	7.41	22646.97	1154.40	0.25
1	7250	Q 100	148800.00	115.70	147.50	131.02	148.40	0.000391	7.85	24168.93	1187.06	0.26
1	7250	Q 500	186800.00	115.70	150.90	133.02	151.99	0.000413	8.70	28350.09	1272.46	0.27

Howland, Penobscot River Bridge - Alternate 5
Standard Table 2

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width
			(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)
1	10450	Q 1.1	134.49	134.04	0.46	0.33	0.01	80.25	46030.13	105.63	846.12
1	10450	Q 10	144.48	143.93	0.54	0.14	0.00	684.34	97065.56	650.10	927.50
1	10450	Q 25	147.03	146.35	0.68	0.15	0.01	1027.89	121191.90	680.17	1044.06
1	10450	Q 50	148.61	147.91	0.70	0.14	0.01	1236.17	131170.20	893.64	1127.16
1	10450	Q 100	150.11	149.34	0.77	0.15	0.01	1502.97	146037.70	1259.33	1202.83
1	10450	Q 500	153.86	152.95	0.91	0.15	0.01	2274.02	181516.20	3009.81	1361.44
1	10050	Q 1.1	134.15	133.61	0.54	0.05	0.00	37.80	46079.35	98.85	897.56
1	10050	Q 10	144.33	143.80	0.54	0.02	0.00	430.81	97269.70	699.50	975.60
1	10050	Q 25	146.87	146.21	0.66	0.02	0.00	634.06	121249.30	1016.62	1001.96
1	10050	Q 50	148.46	147.79	0.67	0.02	0.00	752.29	131354.90	1192.85	1025.71
1	10050	Q 100	149.95	149.21	0.74	0.02	0.00	905.04	146523.30	1371.68	1066.06
1	10050	Q 500	153.70	152.83	0.88	0.02	0.00	1340.82	183090.10	2369.08	1467.57
1	10020	Q 1.1	134.09	133.51	0.58	0.05	0.00	35.22	46158.94	21.84	895.48
1	10020	Q 10	144.31	143.76	0.55	0.02	0.00	430.00	97742.88	227.12	913.09
1	10020	Q 25	146.85	146.17	0.67	0.02	0.00	585.75	122007.00	307.26	914.01
1	10020	Q 50	148.44	147.75	0.69	0.02	0.00	661.78	132291.40	346.83	914.62
1	10020	Q 100	149.93	149.17	0.76	0.02	0.00	760.56	147640.50	398.98	915.16
1	10020	Q 500	153.68	152.78	0.91	0.02	0.00	1005.11	185263.50	531.43	1368.61
1	9980	Q 1.1	134.05	133.45	0.59	0.08	0.03	96.36	45990.84	128.80	894.68
1	9980	Q 10	144.29	143.73	0.56	0.03	0.02	611.71	97084.73	703.56	916.73
1	9980	Q 25	146.83	146.14	0.69	0.03	0.02	811.01	121147.60	941.41	916.98
1	9980	Q 50	148.42	147.72	0.70	0.03	0.02	904.33	131340.80	1054.88	917.14
1	9980	Q 100	149.91	149.14	0.78	0.03	0.02	1029.17	146565.70	1205.10	917.29
1	9980	Q 500	153.67	152.74	0.93	0.03	0.03	1334.51	183890.40	1575.10	1371.93
1	9935	Q 1.1	133.93	133.45	0.48	0.01	0.00	67.85	46051.99	96.16	902.17
1	9935	Q 10	144.25	143.75	0.50	0.00	0.00	570.72	97152.74	676.54	963.11
1	9935	Q 25	146.78	146.16	0.62	0.00	0.00	838.32	121094.90	966.74	971.97
1	9935	Q 50	148.37	147.74	0.63	0.00	0.00	990.25	131180.30	1129.47	990.45
1	9935	Q 100	149.86	149.16 ✓	0.70	0.00	0.01	1181.64	146279.70	1338.69	1058.21
1	9935	Q 500	153.61	152.77	0.84	0.00	0.01	1705.42	183265.10	1829.48	1231.38
1	9910		Bridge								
1	9885	Q 1.1	133.87	133.39	0.49	0.60	0.01	66.75	46054.62	94.64	901.71
1	9885	Q 10	144.21	143.71	0.50	0.25	0.01	569.09	97155.91	674.99	962.97
1	9885	Q 25	146.74	146.12	0.62	0.26	0.01	836.21	121099.10	964.66	971.81
1	9885	Q 50	148.33	147.70	0.63	0.25	0.01	988.17	131184.50	1127.36	986.49
1	9885	Q 100	149.82	149.11	0.70	0.26	0.01	1179.27	146284.50	1336.25	1056.76
1	9885	Q 500	153.54	152.70	0.84	0.26	0.02	1701.41	183269.60	1828.99	1229.42
1	9184	Q 1.1	133.27	132.73	0.54	0.52	0.01	29.53	46163.41	23.06	777.33
1	9184	Q 10	143.96	143.37	0.59	0.24	0.02	502.20	97573.02	324.78	854.41
1	9184	Q 25	146.46	145.72	0.74	0.27	0.03	786.73	121637.50	475.77	903.59
1	9184	Q 50	148.07	147.31	0.76	0.25	0.03	976.24	131884.90	438.86	1001.14
1	9184	Q 100	149.55	148.70	0.85	0.26	0.04	1210.95	146862.70	726.33	1021.23
1	9184	Q 500	153.27	152.25	1.02	0.27	0.05	1918.98	183140.40	1740.61	1072.50
1	8600	Q 1.1	132.74	132.06	0.68	0.23	0.01	45.74	46150.09	20.17	628.55
1	8600	Q 10	143.70	142.88	0.81	0.11	0.00	692.35	97250.34	457.30	697.28
1	8600	Q 25	146.16	145.12	1.04	0.12	0.00	1008.79	121183.90	707.29	709.39
1	8600	Q 50	147.79	146.71	1.08	0.11	0.00	1199.13	131226.00	874.80	717.96
1	8600	Q 100	149.25	148.03	1.21	0.12	0.00	1433.02	146304.70	1062.26	727.04
1	8600	Q 500	152.94	151.44	1.51	0.13	0.00	2090.03	183216.80	1493.12	805.34
1	8380	Q 1.1	132.50	131.72	0.78	0.28	0.02	5.40	46208.13	2.47	622.70
1	8380	Q 10	143.59	142.74	0.84	0.14	0.01	309.12	97849.05	241.82	674.66
1	8380	Q 25	146.04	144.96	1.08	0.16	0.01	476.97	122040.10	382.91	683.26
1	8380	Q 50	147.67	146.56	1.11	0.15	0.01	586.92	132233.00	480.06	689.43
1	8380	Q 100	149.12	147.87	1.25	0.16	0.01	718.62	147553.00	528.37	705.45
1	8380	Q 500	152.81	151.27	1.54	0.17	0.02	1103.51	184836.80	859.64	763.02
1	8100	Q 1.1	132.21	131.50	0.71	0.67	0.04	190.83	45999.56	25.60	572.50
1	8100	Q 10	143.44	142.54	0.91	0.34	0.09	1178.88	96803.22	417.90	636.94
1	8100	Q 25	145.87	144.69	1.18	0.39	0.12	1651.52	120618.10	630.38	649.52

Q100 Elev.

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1 (Continued)

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Vel Head	Frctn Loss	C & E Loss	Q Left	Q Channel	Q Right	Top Width
			(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)
1	8100	Q 50	147.51	146.27	1.23	0.37	0.13	1932.81	130594.70	772.44	658.77
1	8100	Q 100	148.95	147.55	1.40	0.40	0.15	2283.98	145594.10	921.92	668.35
1	8100	Q 500	152.62	150.86	1.76	0.43	0.20	3276.69	182249.30	1273.98	716.17
1	7250	Q 1.1	131.50	130.92	0.58			240.43	45829.46	146.11	770.55
1	7250	Q 10	143.02	142.40	0.62			2034.29	93873.39	2492.33	1058.96
1	7250	Q 25	145.36	144.58	0.79			2864.92	116291.50	3743.54	1113.61
1	7250	Q 50	147.00	146.20	0.80			3363.22	125357.40	4579.42	1154.40
1	7250	Q 100	148.40	147.50	0.90			3978.69	139241.80	5579.55	1187.06
1	7250	Q 500	151.99	150.90	1.09			5711.09	172538.30	8550.67	1272.46

Howland, Penobscot River Bridge - Alternate 5
Six XS Bridge Table

HEC-RAS Plan: Alternate 4 River: Penobscot Reach: 1

Reach	River Sta	Profile	E.G. Elev	W.S. Elev	Crit W.S.	Frctn Loss	C & E Loss	Top Width	Q Left	Q Channel	Q Right	Vel Chnl
			(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft/s)
1	9980	Q 1.1	134.05	133.45		0.08	0.03	894.68	96.36	45990.84	128.80	6.20
1	9980	Q 10	144.29	143.73		0.03	0.02	916.73	611.71	97084.73	703.56	6.05
1	9980	Q 25	146.83	146.14		0.03	0.02	916.98	811.01	121147.60	941.41	6.70
1	9980	Q 50	148.42	147.72		0.03	0.02	917.14	904.33	131340.80	1054.88	6.77
1	9980	Q 100	149.91	149.14		0.03	0.02	917.29	1029.17	146565.70	1205.10	7.12
1	9980	Q 500	153.67	152.74		0.03	0.03	1371.93	1334.51	183890.40	1575.10	7.79
1	9935	Q 1.1	133.93	133.45	128.25	0.01	0.00	902.17	67.85	46051.99	96.16	5.57
1	9935	Q 10	144.25	143.75	131.21	0.00	0.00	963.11	570.72	97162.74	676.54	5.71
1	9935	Q 25	146.78	146.16	132.39	0.00	0.00	971.97	838.32	121094.90	966.74	6.35
1	9935	Q 50	148.37	147.74	132.88	0.00	0.00	990.45	990.25	131180.30	1129.47	6.43
1	9935	Q 100	149.86	149.16	133.58	0.00	0.01	1058.21	1181.64	146279.70	1338.69	6.77
1	9935	Q 500	153.61	152.77	135.18	0.00	0.01	1231.38	1705.42	183265.10	1829.48	7.42
1	9910 BR U	Q 1.1	133.92	133.41	128.32	0.03	0.00	847.61		46190.31	25.69	5.74
1	9910 BR U	Q 10	144.24	143.70	131.34	0.01	0.00	885.07	70.67	98063.58	265.74	5.88
1	9910 BR U	Q 25	146.77	146.11	132.55	0.02	0.00	893.82	132.02	122365.70	402.26	6.55
1	9910 BR U	Q 50	148.36	147.68	133.05	0.01	0.00	899.56	177.63	132634.70	487.65	6.63
1	9910 BR U	Q 100	149.85	149.10	133.75	0.02	0.00	904.70	234.86	147968.50	596.58	6.98
1	9910 BR U	Q 500	153.60	152.66	135.38	0.02	0.00	669.61	483.66	185815.10	501.24	7.77
1	9910 BR D	Q 1.1	133.89	133.37	128.32	0.01	0.01	847.48		46190.59	25.40	5.77
1	9910 BR D	Q 10	144.23	143.69	131.34	0.00	0.01	885.02	70.47	98064.14	265.40	5.89
1	9910 BR D	Q 25	146.76	146.09	132.55	0.00	0.01	893.76	131.70	122366.50	401.77	6.56
1	9910 BR D	Q 50	148.35	147.67	133.05	0.00	0.01	899.50	177.28	132635.60	487.14	6.64
1	9910 BR D	Q 100	149.84	149.08	133.75	0.00	0.02	904.64	234.43	147969.60	595.98	6.99
1	9910 BR D	Q 500	153.58	152.64	135.38	0.00	0.03	671.28	482.16	185816.60	501.25	7.78
1	9885	Q 1.1	133.87	133.39		0.60	0.01	901.71	66.75	46054.62	94.64	5.61
1	9885	Q 10	144.21	143.71		0.25	0.01	962.97	569.09	97155.91	674.99	5.72
1	9885	Q 25	146.74	146.12		0.26	0.01	971.81	836.21	121099.10	964.66	6.36
1	9885	Q 50	148.33	147.70		0.25	0.01	986.49	988.17	131184.50	1127.36	6.44
1	9885	Q 100	149.82	149.11		0.26	0.01	1056.76	1179.27	146284.50	1336.25	6.78
1	9885	Q 500	153.54	152.70		0.26	0.02	1229.42	1701.41	183269.60	1828.99	7.44
1	9184	Q 1.1	133.27	132.73		0.52	0.01	777.33	29.53	46163.41	23.06	5.91
1	9184	Q 10	143.96	143.37		0.24	0.02	854.41	502.20	97573.02	324.78	6.19
1	9184	Q 25	146.46	145.72		0.27	0.03	903.59	786.73	121637.50	475.77	6.95
1	9184	Q 50	148.07	147.31		0.25	0.03	1001.14	976.24	131884.90	438.86	7.06
1	9184	Q 100	149.55	148.70		0.26	0.04	1021.23	1210.95	146862.70	726.33	7.44
1	9184	Q 500	153.27	152.25		0.27	0.05	1072.50	1918.98	183140.40	1740.61	8.18

Howland, Penobscot River Bridge - Alternate 5
Bridge XS Tables

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 1.1

E.G. US. (ft)	133.93	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	133.45	E.G. Elev (ft)	133.92	133.89
Q Total (cfs)	46216.00	W.S. Elev (ft)	133.41	133.37
Q Bridge (cfs)	46216.00	Crit W.S. (ft)	128.32	128.32
Q Weir (cfs)		Max Chl Dpth (ft)	12.11	12.07
Weir Sta Lft (ft)		Vel Total (ft/s)	5.73	5.75
Weir Sta Rgt (ft)		Flow Area (sq ft)	8070.79	8039.54
Weir Submerg		Froude # Chl	0.33	0.33
Weir Max Depth (ft)		Specif Force (cu ft)	48570.84	48305.72
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	9.52	9.49
Min El Prs (ft)	161.66	W.P. Total (ft)	915.21	914.83
Delta EG (ft)	0.06	Conv. Total (cfs)	1467123.0	1458061.0
Delta WS (ft)	0.07	Top Width (ft)	847.61	847.48
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.03	0.01
BR Open Vel (ft/s)	5.75	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.55	0.55
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 10

E.G. US. (ft)	144.25	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	143.75	E.G. Elev (ft)	144.24	144.23
Q Total (cfs)	98400.00	W.S. Elev (ft)	143.70	143.69
Q Bridge (cfs)	98400.00	Crit W.S. (ft)	131.34	131.34
Q Weir (cfs)		Max Chl Dpth (ft)	22.40	22.39
Weir Sta Lft (ft)		Vel Total (ft/s)	5.79	5.80
Weir Sta Rgt (ft)		Flow Area (sq ft)	16988.54	16975.16
Weir Submerg		Froude # Chl	0.23	0.23
Weir Max Depth (ft)		Specif Force (cu ft)	186903.90	186661.20
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	19.19	19.18
Min El Prs (ft)	161.66	W.P. Total (ft)	1019.71	1019.56
Delta EG (ft)	0.03	Conv. Total (cfs)	4740681.0	4734948.0
Delta WS (ft)	0.04	Top Width (ft)	885.07	885.02
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.01	0.00
BR Open Vel (ft/s)	5.80	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.45	0.45
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 25

E.G. US. (ft)	146.78	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	146.16	E.G. Elev (ft)	146.77	146.76
Q Total (cfs)	122900.00	W.S. Elev (ft)	146.11	146.09
Q Bridge (cfs)	122900.00	Crit W.S. (ft)	132.55	132.55
Q Weir (cfs)		Max Chl Dpth (ft)	24.81	24.79
Weir Sta Lft (ft)		Vel Total (ft/s)	6.43	6.43
Weir Sta Rgt (ft)		Flow Area (sq ft)	19125.18	19110.46
Weir Submerg		Froude # Chl	0.24	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	237260.30	236964.30
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	21.40	21.38
Min El Prs (ft)	161.66	W.P. Total (ft)	1044.10	1043.94
Delta EG (ft)	0.04	Conv. Total (cfs)	5681895.0	5675245.0
Delta WS (ft)	0.04	Top Width (ft)	893.82	893.76
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	6.43	C & E Loss (ft)	0.00	0.01

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 25 (Continued)

Coef of Q		Shear Total (lb/sq ft)	0.54	0.54
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 50

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	148.37			
W.S. US. (ft)	147.74	E.G. Elev (ft)	148.36	148.35
Q Total (cfs)	133300.00	W.S. Elev (ft)	147.68	147.67
Q Bridge (cfs)	133300.00	Crit W.S. (ft)	133.05	133.05
Q Weir (cfs)		Max Chl Dpth (ft)	26.38	26.37
Weir Sta Lft (ft)		Vel Total (ft/s)	6.49	6.49
Weir Sta Rgt (ft)		Flow Area (sq ft)	20539.59	20525.59
Weir Submerg		Froude # Chl	0.24	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	270968.70	270667.30
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	22.83	22.82
Min El Prs (ft)	161.66	W.P. Total (ft)	1060.12	1059.96
Delta EG (ft)	0.04	Conv. Total (cfs)	6331133.0	6324610.0
Delta WS (ft)	0.04	Top Width (ft)	899.56	899.50
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.01	0.00
BR Open Vel (ft/s)	6.49	C & E Loss (ft)	0.00	0.01
Coef of Q		Shear Total (lb/sq ft)	0.54	0.54
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 100

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	149.86			
W.S. US. (ft)	149.16	E.G. Elev (ft)	149.85	149.84
Q Total (cfs)	148800.00	W.S. Elev (ft)	149.10	149.08
Q Bridge (cfs)	148800.00	Crit W.S. (ft)	133.75	133.75
Q Weir (cfs)		Max Chl Dpth (ft)	27.80	27.78
Weir Sta Lft (ft)		Vel Total (ft/s)	6.82	6.83
Weir Sta Rgt (ft)		Flow Area (sq ft)	21814.86	21800.20
Weir Submerg		Froude # Chl	0.24	0.25
Weir Max Depth (ft)		Specif Force (cu ft)	305692.90	305360.50
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	24.11	24.10
Min El Prs (ft)	161.66	W.P. Total (ft)	1074.47	1074.30
Delta EG (ft)	0.04	Conv. Total (cfs)	6932971.0	6925969.0
Delta WS (ft)	0.05	Top Width (ft)	904.70	904.64
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	6.83	C & E Loss (ft)	0.00	0.02
Coef of Q		Shear Total (lb/sq ft)	0.58	0.58
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 500

		Element	Inside BR US	Inside BR DS
E.G. US. (ft)	153.61			
W.S. US. (ft)	152.77	E.G. Elev (ft)	153.60	153.58
Q Total (cfs)	186800.00	W.S. Elev (ft)	152.66	152.64
Q Bridge (cfs)	186800.00	Crit W.S. (ft)	135.38	135.38
Q Weir (cfs)		Max Chl Dpth (ft)	31.36	31.34
Weir Sta Lft (ft)		Vel Total (ft/s)	7.57	7.58
Weir Sta Rgt (ft)		Flow Area (sq ft)	24663.06	24648.33
Weir Submerg		Froude # Chl	0.24	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	401582.90	401067.80
Min El Weir Flow (ft)	156.27	Hydr Depth (ft)	36.83	36.72
Min El Prs (ft)	161.66	W.P. Total (ft)	1345.00	1343.17

Plan: Alternate 4 Penobscot 1 RS: 9910 Profile: Q 500 (Continued)

Delta EG (ft)	0.06	Conv. Total (cfs)	7409707.0	7409637.0
Delta WS (ft)	0.07	Top Width (ft)	669.61	671.26
BR Open Area (sq ft)	27674.90	Frctn Loss (ft)	0.02	0.00
BR Open Vel (ft/s)	7.58	C & E Loss (ft)	0.00	0.03
Coef of Q		Shear Total (lb/sq ft)	0.73	0.73
Br Sel Method	Energy only	Power Total (lb/ft s)	0.00	0.00

Howland, Penobscot River Bridge - Alternate 5
Cross Section Tables

Plan: Alternate 4 Penobscot 1 RS: 10050 Profile: Q 100

E.G. Elev (ft)	149.95	Element	Left OB	Channel	Right OB	
Vel Head (ft)	0.74	Wt. n-Val.	0.100	0.035	0.100	
W.S. Elev (ft)	149.21	Reach Len. (ft)	50.00	50.00	50.00	
Crit W.S. (ft)		Flow Area (sq ft)	655.01	21060.76	1228.36	
E.G. Slope (ft/ft)	0.000372	Area (sq ft)	655.01	21060.76	1228.36	S_1
Q Total (cfs)	148800.00	Flow (cfs)	905.04	146523.30	1371.68	Q_1
Top Width (ft)	1066.06	Top Width (ft)	58.27	850.00	157.79	W_1
Vel Total (ft/s)	6.49	Avg. Vel. (ft/s)	1.38	6.96	1.12	V_{ms}
Max Chl Dpth (ft)	27.51	Hydr. Depth (ft)	11.24	24.78	7.78	y_1
Conv. Total (cfs)	7712082.0	Conv. (cfs)	46907.0	7594082.0	71092.3	
Length Wtd. (ft)	50.00	Wetted Per. (ft)	61.91	850.87	159.80	
Min Ch El (ft)	121.70	Shear (lb/sq ft)	0.25	0.58	0.18	
Alpha	1.13	Stream Power (lb/ft s)	1750.00	0.00	0.00	
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	72.57	1186.32	79.15	
C & E Loss (ft)	0.00	Cum SA (acres)	6.00	43.90	9.53	

Plan: Alternate 4 Penobscot 1 RS: 10050 Profile: Q 500

E.G. Elev (ft)	153.70	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.88	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	152.83	Reach Len. (ft)	50.00	50.00	50.00
Crit W.S. (ft)		Flow Area (sq ft)	882.75	24136.52	1866.09
E.G. Slope (ft/ft)	0.000369	Area (sq ft)	882.75	24136.52	2601.18
Q Total (cfs)	186800.00	Flow (cfs)	1340.82	183090.10	2369.08
Top Width (ft)	1467.57	Top Width (ft)	67.60	850.00	549.97
Vel Total (ft/s)	6.95	Avg. Vel. (ft/s)	1.52	7.59	1.27
Max Chl Dpth (ft)	31.13	Hydr. Depth (ft)	13.06	28.40	9.48
Conv. Total (cfs)	9724226.0	Conv. (cfs)	69798.8	9531100.0	123326.8
Length Wtd. (ft)	50.00	Wetted Per. (ft)	71.91	850.87	198.95
Min Ch El (ft)	121.70	Shear (lb/sq ft)	0.28	0.65	0.22
Alpha	1.17	Stream Power (lb/ft s)	1750.00	0.00	0.00
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	94.61	1338.49	121.81
C & E Loss (ft)	0.00	Cum SA (acres)	6.81	43.71	14.86

Plan: Alternate 4 Penobscot 1 RS: 10020 Profile: Q 100

E.G. Elev (ft)	149.93	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.76	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	149.17	Reach Len. (ft)	40.00	40.00	40.00
Crit W.S. (ft)		Flow Area (sq ft)	512.89	21046.51	303.89
E.G. Slope (ft/ft)	0.000390	Area (sq ft)	512.89	21046.51	303.89
Q Total (cfs)	148800.00	Flow (cfs)	760.56	147640.50	398.98
Top Width (ft)	915.16	Top Width (ft)	31.32	863.83	20.01
Vel Total (ft/s)	6.81	Avg. Vel. (ft/s)	1.48	7.01	1.31
Max Chl Dpth (ft)	28.58	Hydr. Depth (ft)	16.38	24.36	15.19
Conv. Total (cfs)	7536391.0	Conv. (cfs)	38520.7	7477664.0	20207.3
Length Wtd. (ft)	40.00	Wetted Per. (ft)	45.13	869.34	32.10
Min Ch El (ft)	120.59	Shear (lb/sq ft)	0.28	0.59	0.23
Alpha	1.05	Stream Power (lb/ft s)	1750.00	0.00	0.00
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	71.89	1162.15	78.27
C & E Loss (ft)	0.00	Cum SA (acres)	5.95	42.92	9.43

Plan: Alternate 4 Penobscot 1 RS: 10020 Profile: Q 500

E.G. Elev (ft)	153.68	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.91	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	152.78	Reach Len. (ft)	40.00	40.00	40.00
Crit W.S. (ft)		Flow Area (sq ft)	626.43	24161.55	377.95
E.G. Slope (ft/ft)	0.000387	Area (sq ft)	626.43	24161.55	1197.43
Q Total (cfs)	186800.00	Flow (cfs)	1005.11	185263.50	531.43
Top Width (ft)	1368.61	Top Width (ft)	31.65	863.83	473.12
Vel Total (ft/s)	7.42	Avg. Vel. (ft/s)	1.60	7.67	1.41
Max Chl Dpth (ft)	32.19	Hydr. Depth (ft)	19.79	27.97	17.94
Conv. Total (cfs)	9489885.0	Conv. (cfs)	51061.8	9411825.0	26997.9
Length Wtd. (ft)	40.00	Wetted Per. (ft)	48.76	869.34	35.86
Min Ch El (ft)	120.59	Shear (lb/sq ft)	0.31	0.67	0.25
Alpha	1.06	Stream Power (lb/ft s)	1750.00	0.00	0.00
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	93.75	1310.78	119.63
C & E Loss (ft)	0.00	Cum SA (acres)	6.75	42.73	14.28

Plan: Alternate 4 Penobscot 1 RS: 9910 BR U Profile: Q 100

E.G. Elev (ft)	149.85	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.75	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	149.10	Reach Len. (ft)	33.25	33.25	33.25
Crit W.S. (ft)	133.75	Flow Area (sq ft)	208.28	21186.32	420.26
E.G. Slope (ft/ft)	0.000461	Area (sq ft)	208.28	21186.32	420.26
Q Total (cfs)	148800.00	Flow (cfs)	234.86	147968.50	596.58
Top Width (ft)	904.70	Top Width (ft)	27.38	838.00	39.32
Vel Total (ft/s)	6.82	Avg. Vel. (ft/s)	1.13	6.98	1.42
Max Chl Dpth (ft)	27.80	Hydr. Depth (ft)	7.61	25.28	10.69
Conv. Total (cfs)	6932971.0	Conv. (cfs)	10942.8	6894232.0	27796.2
Length Wtd. (ft)	33.25	Wetted Per. (ft)	31.33	998.39	44.75
Min Ch El (ft)	121.30	Shear (lb/sq ft)	0.19	0.61	0.27
Alpha	1.04	Stream Power (lb/ft s)	1475.00	0.00	0.00
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	69.93	1100.15	76.08
C & E Loss (ft)	0.00	Cum SA (acres)	5.82	40.42	9.21

$A_{BR} = \Sigma A_{Area} = 21,814.86$
Q_2, Q_{BR}
W_2 (Deduct Pier widths)
y_0, y_{BR}

Plan: Alternate 4 Penobscot 1 RS: 9910 BR U Profile: Q 500

E.G. Elev (ft)	153.60	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.93	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	152.66	Reach Len. (ft)	33.25	33.25	33.25
Crit W.S. (ft)	135.38	Flow Area (sq ft)	317.34	23900.33	445.39
E.G. Slope (ft/ft)	0.000636	Area (sq ft)	317.34	23900.33	445.39
Q Total (cfs)	186800.00	Flow (cfs)	483.66	185815.10	501.24
Top Width (ft)	669.61	Top Width (ft)	33.80	635.80	
Vel Total (ft/s)	7.57	Avg. Vel. (ft/s)	1.52	7.77	1.13
Max Chl Dpth (ft)	31.36	Hydr. Depth (ft)	9.39	37.59	
Conv. Total (cfs)	7409707.0	Conv. (cfs)	19185.3	7370639.0	19882.6
Length Wtd. (ft)	33.25	Wetted Per. (ft)	38.67	1220.80	85.53
Min Ch El (ft)	121.30	Shear (lb/sq ft)	0.33	0.78	0.21
Alpha	1.05	Stream Power (lb/ft s)	1475.00	0.00	0.00
Frctn Loss (ft)	0.02	Cum Volume (acre-ft)	91.30	1239.81	114.89
C & E Loss (ft)	0.00	Cum SA (acres)	6.61	40.25	13.06

$\Sigma A = 24,663.06$
W_2
Bottom width of the main channel in the contracted section same as Q_{100}, lower value of top width here accounts for WSE above top flange.

Hydraulic Depth $= \frac{23900.33}{826}$
$= 28.93'$

Plan: Alternate 4 Penobscot 1 RS: 9910 BR D Profile: Q 100

E.G. Elev (ft)	149.84	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.75	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	149.08	Reach Len. (ft)	8.35	8.35	8.35
Crit W.S. (ft)	133.75	Flow Area (sq ft)	207.83	21172.74	419.63
E.G. Slope (ft/ft)	0.000462	Area (sq ft)	207.83	21172.74	419.63
Q Total (cfs)	148800.00	Flow (cfs)	234.43	147969.60	595.98
Top Width (ft)	904.64	Top Width (ft)	27.36	838.00	39.29
Vel Total (ft/s)	6.83	Avg. Vel. (ft/s)	1.13	6.99	1.42
Max Chl Dpth (ft)	27.78	Hydr. Depth (ft)	7.60	25.27	10.68
Conv. Total (cfs)	6925969.0	Conv. (cfs)	10911.8	6887317.0	27740.1
Length Wtd. (ft)	8.35	Wetted Per. (ft)	31.29	998.29	44.72
Min Ch El (ft)	121.30	Shear (lb/sq ft)	0.19	0.61	0.27
Alpha	1.04	Stream Power (lb/ft s)	1475.00	0.00	0.00
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	69.77	1083.99	75.76
C & E Loss (ft)	0.02	Cum SA (acres)	5.80	39.78	9.18

Plan: Alternate 4 Penobscot 1 RS: 9910 BR D Profile: Q 500

E.G. Elev (ft)	153.58	Element	Left OB	Channel	Right OB
Vel Head (ft)	0.93	Wt. n-Val.	0.100	0.035	0.100
W.S. Elev (ft)	152.64	Reach Len. (ft)	8.35	8.35	8.35
Crit W.S. (ft)	135.38	Flow Area (sq ft)	316.59	23886.34	445.39
E.G. Slope (ft/ft)	0.000636	Area (sq ft)	316.59	23886.34	445.39
Q Total (cfs)	186800.00	Flow (cfs)	482.16	185816.60	501.25
Top Width (ft)	671.26	Top Width (ft)	33.76	637.50	
Vel Total (ft/s)	7.58	Avg. Vel. (ft/s)	1.52	7.78	1.13
Max Chl Dpth (ft)	31.34	Hydr. Depth (ft)	9.38	37.47	
Conv. Total (cfs)	7409637.0	Conv. (cfs)	19125.5	7370629.0	19882.6
Length Wtd. (ft)	8.35	Wetted Per. (ft)	38.62	1219.01	85.53
Min Ch El (ft)	121.30	Shear (lb/sq ft)	0.33	0.78	0.21
Alpha	1.05	Stream Power (lb/ft s)	1475.00	0.00	0.00
Frctn Loss (ft)	0.00	Cum Volume (acre-ft)	91.06	1221.57	114.55
C & E Loss (ft)	0.03	Cum SA (acres)	6.59	39.76	13.06

Howland, Penobscot River Bridge - Alternate 5
Flow Distribution Cross Section Tables

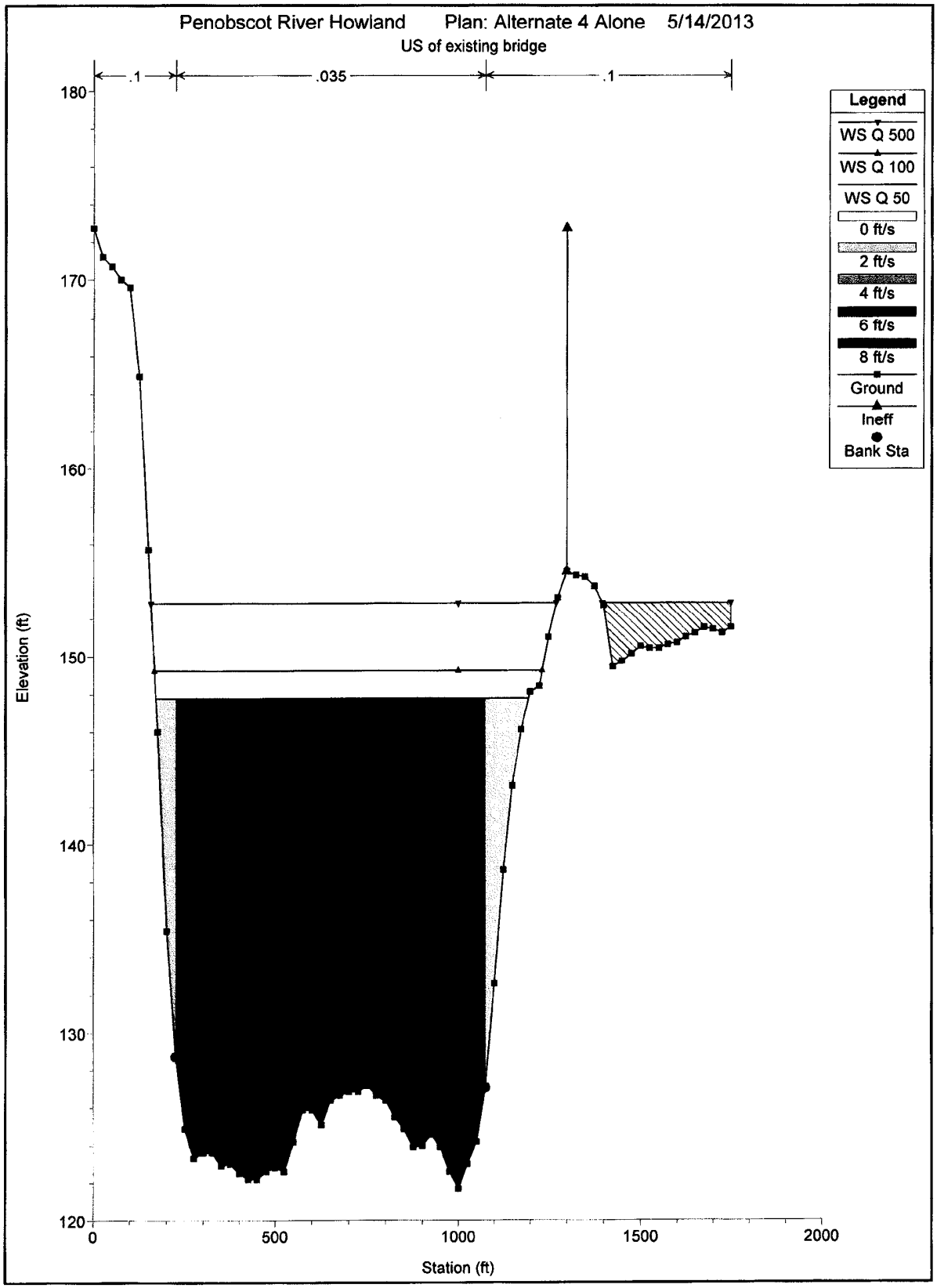
Plan: Alternate 4 Penobscot 1 RS: 9935 Profile: Q 100

	Pos	Left Sta	Right Sta	Flow	Area	W.P.	Percent	Hydr	Velocity	Shear	Power
		(ft)	(ft)	(cfs)	(sq ft)	(ft)	Conv	Depth(ft)	(ft/s)	(lb/sq ft)	(lb/ft s)
1	LOB	135.00	180.00	72.41	100.54	20.67	0.05	5.59	0.72	0.10	0.07
2	LOB	180.00	225.00	1109.23	716.11	46.67	0.75	15.91	1.55	0.33	0.50
3	Chan	225.00	267.50	7214.22	1077.00	42.88	4.85	25.34	6.70	0.53	3.57
4	Chan	267.50	310.00	8449.73	1180.08	42.51	5.68	27.77	7.16	0.59	4.22
5	Chan	310.00	352.50	8286.24	1166.21	42.50	5.57	27.44	7.11	0.58	4.14
6	Chan	352.50	395.00	8136.84	1153.51	42.50	5.47	27.14	7.05	0.58	4.07
7	Chan	395.00	437.50	8030.88	1144.49	42.50	5.40	26.93	7.02	0.57	4.01
8	Chan	437.50	480.00	7825.19	1126.84	42.50	5.26	26.51	6.94	0.56	3.91
9	Chan	480.00	522.50	7785.26	1123.37	42.50	5.23	26.43	6.93	0.56	3.89
10	Chan	522.50	565.00	7570.13	1104.65	42.50	5.09	25.99	6.85	0.55	3.78
11	Chan	565.00	607.50	7292.16	1080.22	42.51	4.90	25.42	6.75	0.54	3.64
12	Chan	607.50	650.00	6990.30	1053.35	42.53	4.70	24.78	6.64	0.53	3.49
13	Chan	650.00	692.50	6622.60	1020.59	42.62	4.45	24.01	6.49	0.51	3.30
14	Chan	692.50	735.00	5482.63	910.54	42.54	3.68	21.42	6.02	0.45	2.74
15	Chan	735.00	777.50	5642.20	926.16	42.52	3.79	21.79	6.09	0.46	2.82
16	Chan	777.50	820.00	6193.46	979.41	42.51	4.16	23.04	6.32	0.49	3.09
17	Chan	820.00	862.50	6782.49	1034.55	42.54	4.56	24.34	6.56	0.52	3.39
18	Chan	862.50	905.00	7525.84	1100.87	42.51	5.06	25.90	6.84	0.55	3.76
19	Chan	905.00	947.50	7755.59	1120.92	42.51	5.21	26.37	6.92	0.56	3.88
20	Chan	947.50	990.00	8105.69	1150.90	42.50	5.45	27.08	7.04	0.58	4.05
21	Chan	990.00	1032.50	7739.20	1119.55	42.52	5.20	26.34	6.91	0.56	3.87
22	Chan	1032.50	1075.00	6849.00	1042.12	42.69	4.60	24.52	6.57	0.52	3.41
23	ROB	1075.00	1175.00	1338.69	912.87	73.77	0.90	13.04	1.47	0.26	0.39
24	ROB	1175.00	1250.00							0.00	0.00
25	ROB	1250.00	1325.00	0.00	4.54	13.77	0.00	0.33	0.00	0.01	0.00
26	ROB	1325.00	1400.00	0.00	68.21	61.59	0.00	1.11	0.00	0.02	0.00

y_1 pier, V_1

Plan: Alternate 4 Penobscot 1 RS: 9935 Profile: Q 500

	Pos	Left Sta	Right Sta	Flow	Area	W.P.	Percent	Hydr	Velocity	Shear	Power
		(ft)	(ft)	(cfs)	(sq ft)	(ft)	Conv	Depth(ft)	(ft/s)	(lb/sq ft)	(lb/ft s)
1	LOB	135.00	180.00	151.60	175.40	27.27	0.08	7.46	0.86	0.14	0.12
2	LOB	180.00	225.00	1553.82	878.51	46.67	0.83	19.52	1.77	0.40	0.71
3	Chan	225.00	267.50	9050.45	1230.37	42.88	4.84	28.95	7.36	0.61	4.52
4	Chan	267.50	310.00	10408.75	1333.45	42.51	5.57	31.38	7.81	0.67	5.25
5	Chan	310.00	352.50	10230.64	1319.59	42.50	5.48	31.05	7.75	0.67	5.16
6	Chan	352.50	395.00	10067.61	1306.89	42.50	5.39	30.75	7.70	0.66	5.08
7	Chan	395.00	437.50	9951.84	1297.86	42.50	5.33	30.54	7.67	0.65	5.02
8	Chan	437.50	480.00	9726.89	1280.21	42.50	5.21	30.12	7.60	0.65	4.90
9	Chan	480.00	522.50	9683.22	1276.75	42.50	5.18	30.04	7.58	0.64	4.88
10	Chan	522.50	565.00	9447.62	1258.02	42.50	5.06	29.60	7.51	0.63	4.76
11	Chan	565.00	607.50	9142.55	1233.60	42.51	4.89	29.03	7.41	0.62	4.61
12	Chan	607.50	650.00	8810.48	1206.72	42.53	4.72	28.39	7.30	0.61	4.44
13	Chan	650.00	692.50	8403.88	1173.96	42.62	4.50	27.62	7.16	0.59	4.22
14	Chan	692.50	735.00	7141.35	1063.91	42.54	3.82	25.03	6.71	0.54	3.60
15	Chan	735.00	777.50	7319.43	1079.54	42.52	3.92	25.40	6.78	0.54	3.69
16	Chan	777.50	820.00	7931.38	1132.79	42.51	4.25	26.65	7.00	0.57	4.00
17	Chan	820.00	862.50	8581.49	1187.92	42.54	4.59	27.95	7.22	0.60	4.32
18	Chan	862.50	905.00	9398.88	1254.25	42.51	5.03	29.51	7.49	0.63	4.74
19	Chan	905.00	947.50	9650.52	1274.30	42.51	5.17	29.98	7.57	0.64	4.86
20	Chan	947.50	990.00	10033.53	1304.27	42.50	5.37	30.69	7.69	0.66	5.06
21	Chan	990.00	1032.50	9632.51	1272.92	42.52	5.16	29.95	7.57	0.64	4.85
22	Chan	1032.50	1075.00	8652.10	1195.50	42.69	4.63	28.13	7.24	0.60	4.34
23	ROB	1075.00	1175.00	1829.48	1183.78	88.99	0.98	13.99	1.55	0.29	0.44
24	ROB	1175.00	1250.00	0.00	0.27	3.26	0.00	0.08	0.00	0.00	0.00
25	ROB	1250.00	1325.00	0.00	143.93	75.17	0.00	1.92	0.00	0.04	0.00
26	ROB	1325.00	1400.00	0.00	326.43	75.24	0.00	4.35	0.00	0.09	0.00
27	ROB	1400.00	1475.00	0.00	142.68	76.97	0.00	1.90	0.00	0.04	0.00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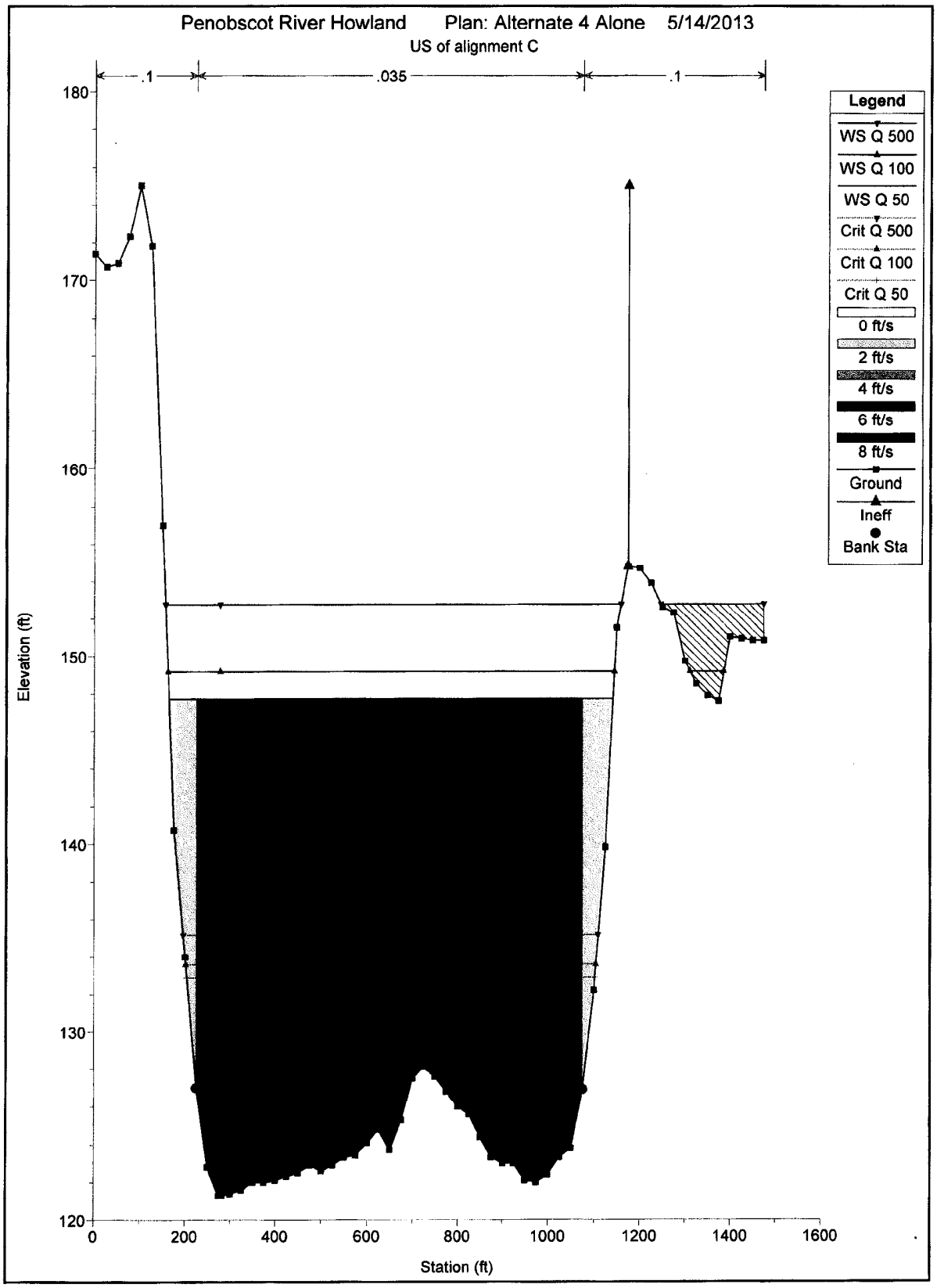
US of existing bridge
.1
.035
.1
Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Ineff
Bank Sta
Elevation (ft)
180
170
160
150
140
130
120
0
500
1000
1500
2000
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
US of alignment C
.1
.035
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Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Ineff
Bank Sta
Elevation (ft)
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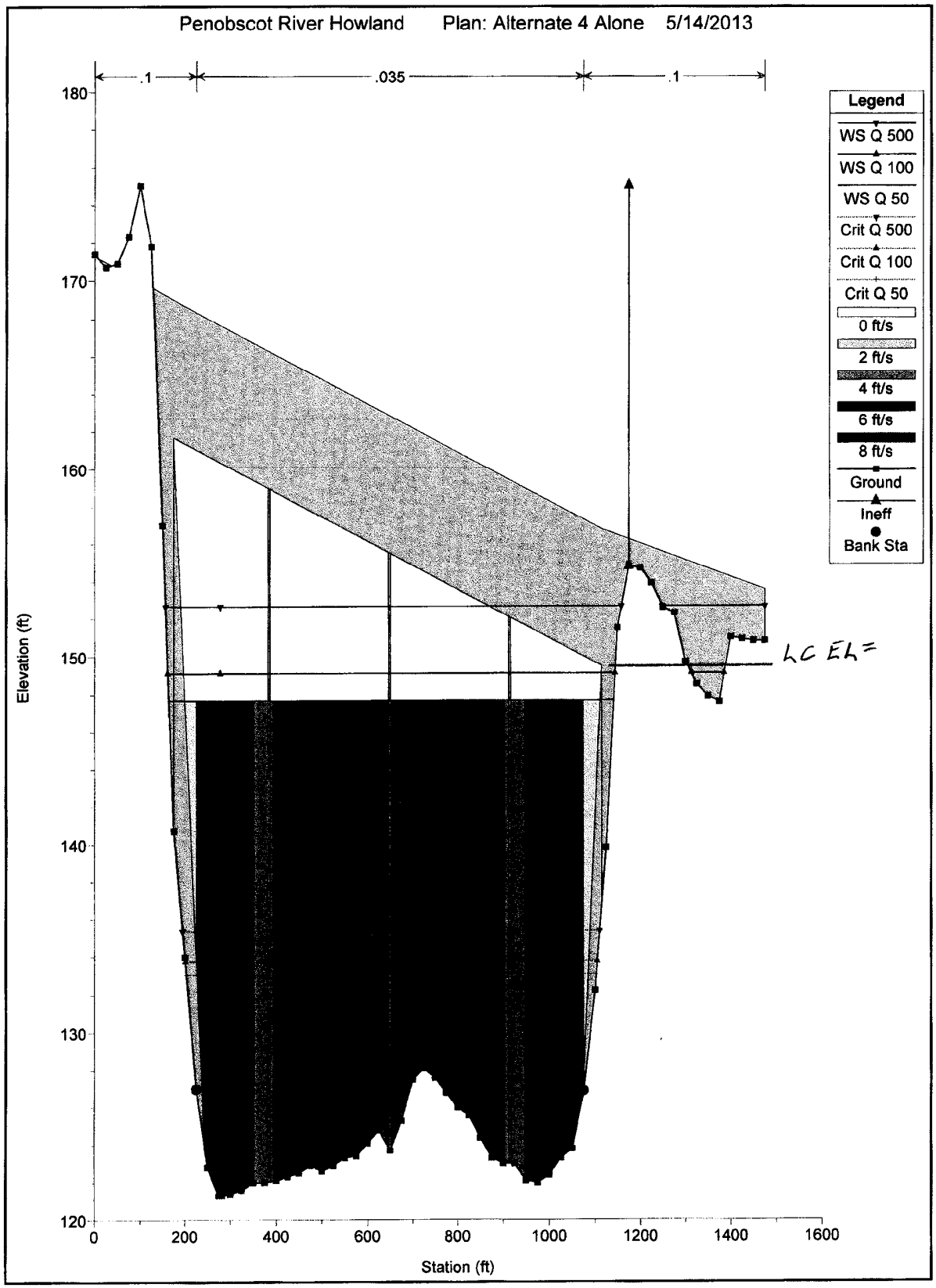
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
.1
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Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Ineff
Bank Sta
LC EL=
Elevation (ft)
180
170
160
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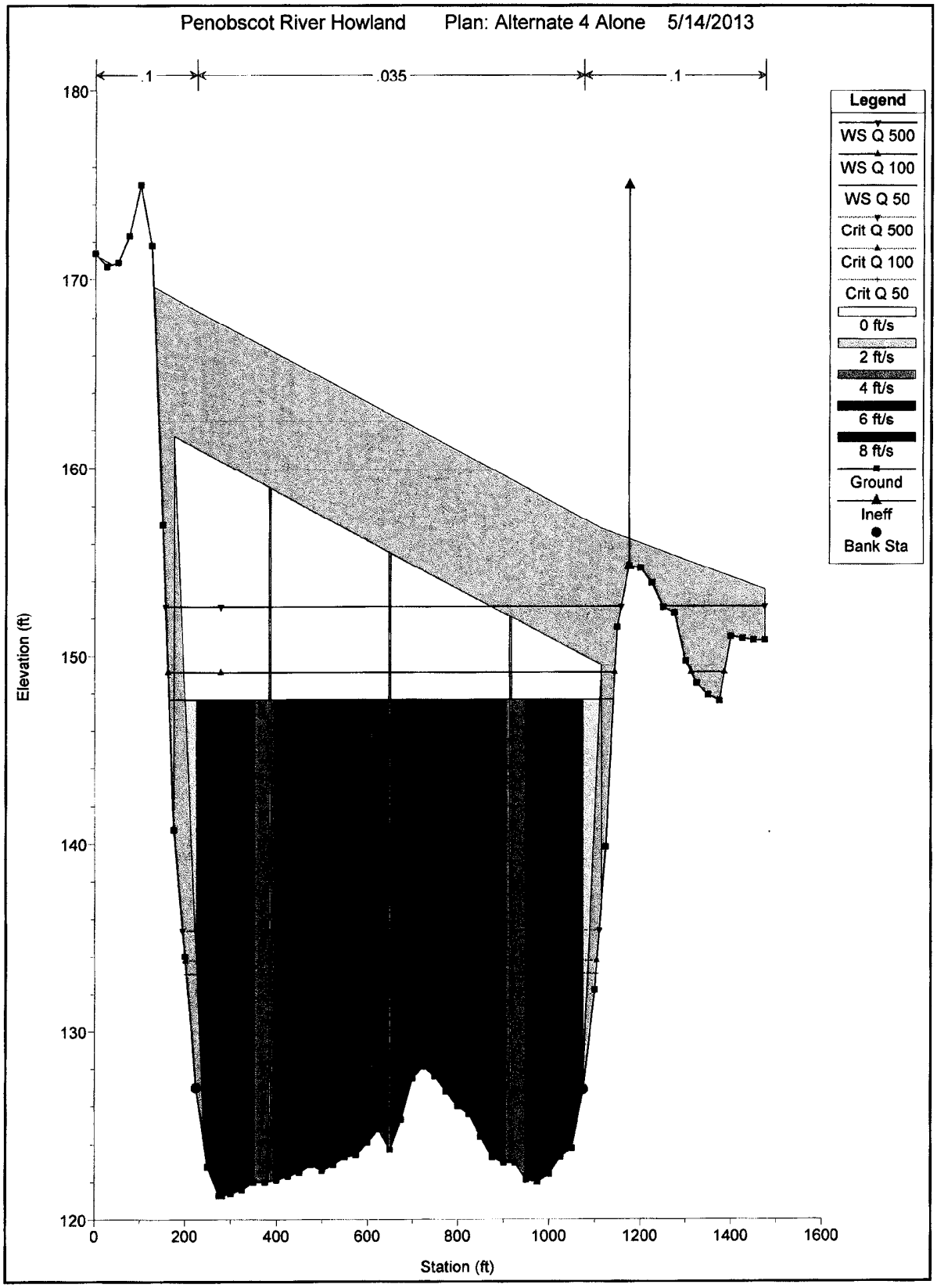
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
.1
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Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Ineff
Bank Sta
Elevation (ft)
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140
130
120
0
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400
600
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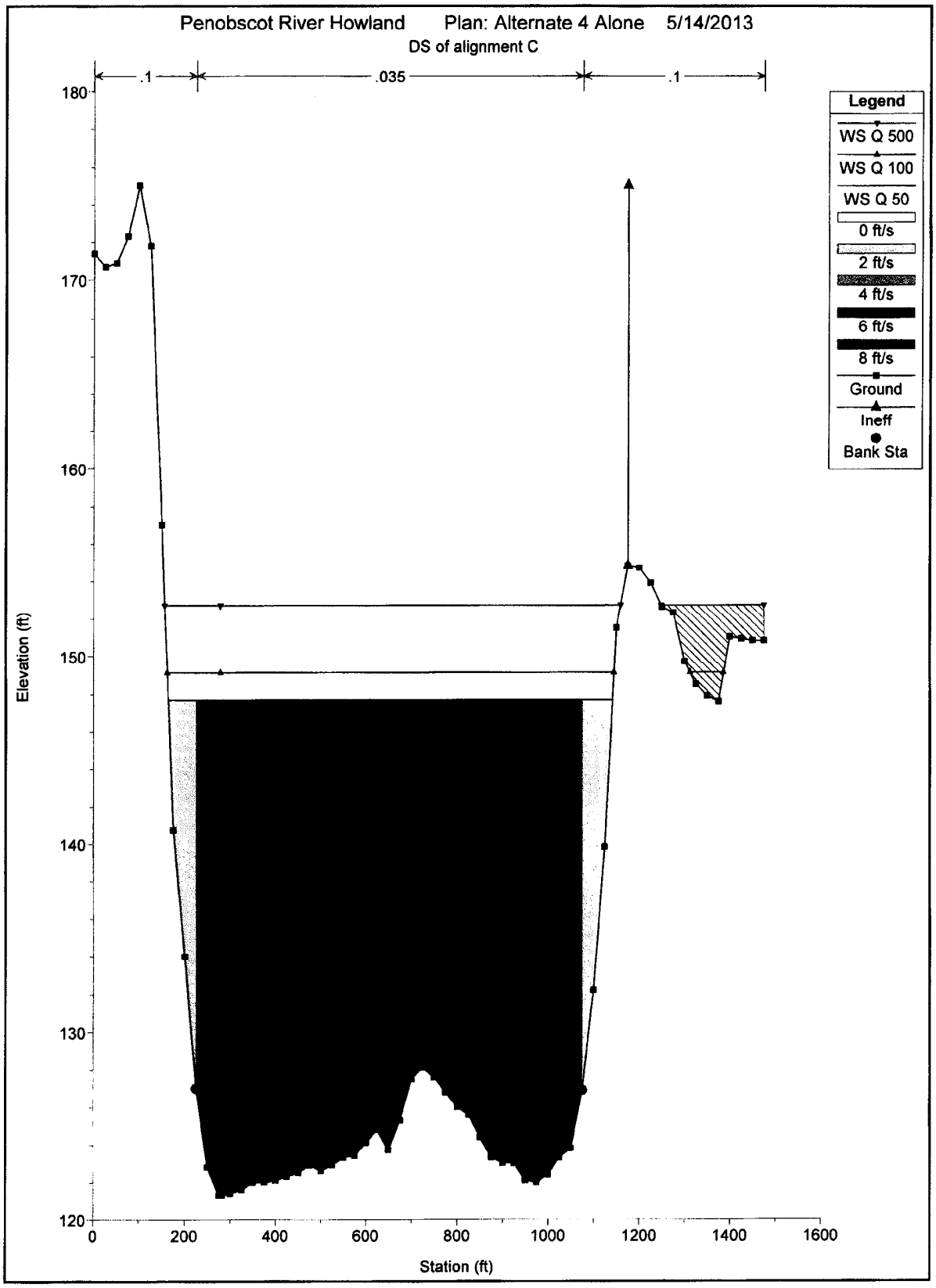
1000
1200
1400
1600
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
DS of alignment C
.1
.035
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Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Ineff
Bank Sta
Elevation (ft)
180
170
160
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0
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600
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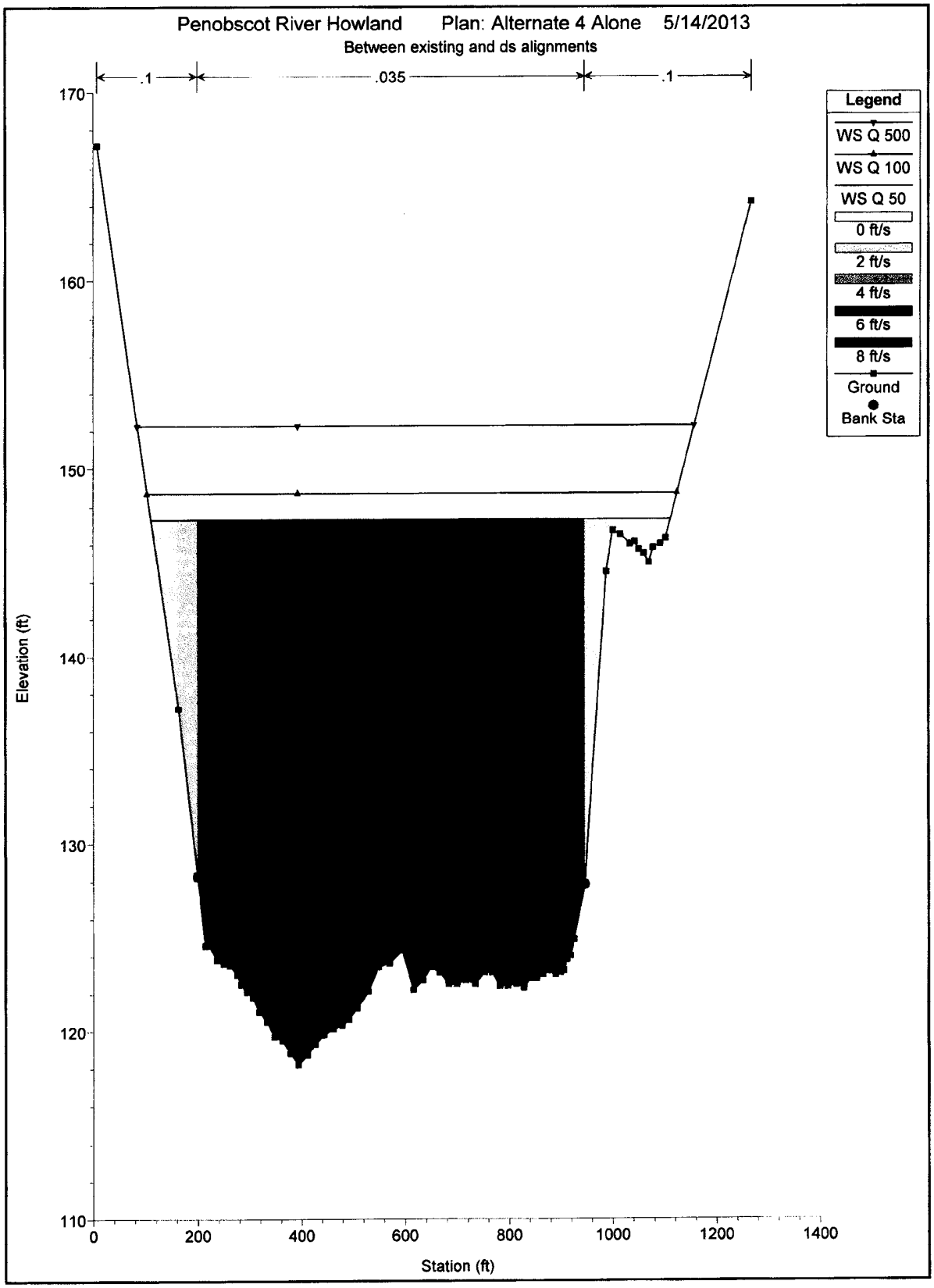
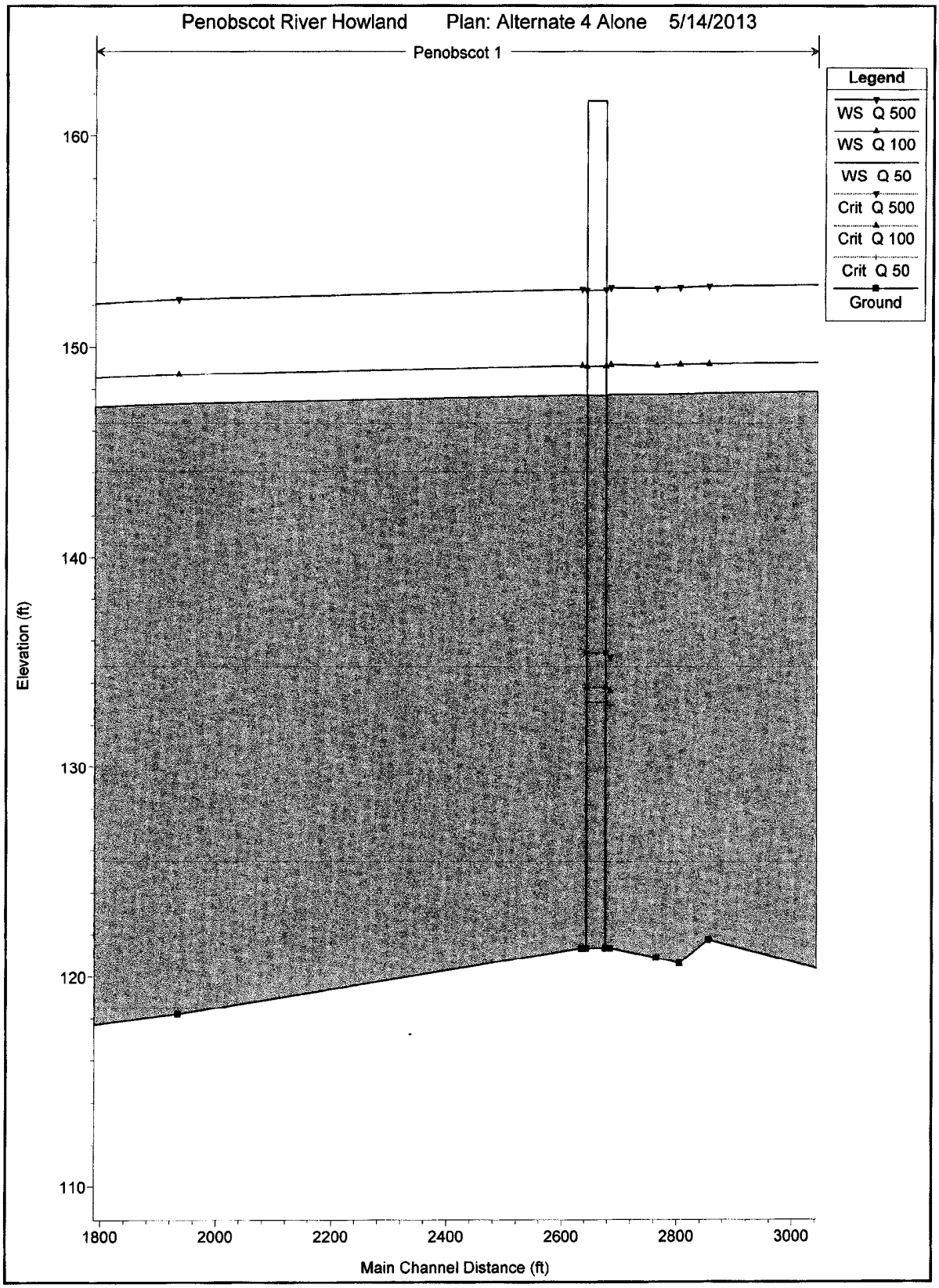
1400
1600
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
Between existing and ds alignments
.1
.035
.1
Legend
WS Q 500
WS Q 100
WS Q 50
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
170
160
150
140
130
120
110
0
200
400
600
800
1000
1200
1400
Station (ft)


Penobscot River Howland Plan: Alternate 4 Alone 5/14/2013
Penobscot 1
Legend
WS Q 500
WS Q 100
WS Q 50
Crit Q 500
Crit Q 100
Crit Q 50
Ground
Elevation (ft)
160
150
140
130
120
110
1800
2000
2200
2400
2600
2800
3000
Main Channel Distance (ft)

Penobscot River Bridge #2660
Howland - Enfield, Maine

Scour:

Scour analysis and data for Bridge Replacement Alternate 5 includes the following scour analysis computations for the Q_{100} and Q_{500} year events:

- Contraction Scour
- Local Pier Scour (Simple)
- Abutment Scour
- Local Pier Scour (Complex)

Complex analysis of local pier scour was conducted in accordance with HEC-18 guidelines. Complex analysis of local pier scour is required since local scour of the pier stem is anticipated to expose the wider pile cap and cofferdam seal under the design Q_{100} and Q_{500} events.

Bridge Replacement Alternate 5 will be founded on piles to bedrock, and abutment slopes will be protected using riprap. The final design will need to consider abutment scour conditions and revetment needs in accordance with the MaineDOT Bridge Design Guide.

Additional geotechnical explorations will be required during the final design phase to confirm assumed subsurface and streambed conditions. Scour computations will need to be updated during final design for the proposed Bridge Replacement Alternate 5 location based on the findings of the supplemental subsurface investigations.

Similar results would be anticipated for Bridge Replacement Alternates 2 and 4, and properly designed revetment is required to protect the existing piers and approach slopes for Bridge Rehabilitation Alternate 1.

A copy of the Scour Critical Bridge Plan of Action (POA) Report dated November 2009 is included at the end of this Section.

Project: Howland, Penobscot River Bridge Job No. 41167.05.00 Sheet: 1 of
Item: Scour Comps - PDR Designer: MMd Date: 1/8/12
Checker: DSM Date: 1/12
Grid: 1/8"

Geotechnical Boring Data - Bed Material Properties

Boring/Sample No.	Depth	D50	D84
BB-HERP-102/1D	0-2	9.5mm	23mm
BB-HERP-102/4D	16-18	.82mm	7 mm
BB-HERP-103/1D	0-2	5.5mm	30mm
BB-HERP-103/2D	3.5-5.5	10.1 mm	27mm
BB-HERP-103/5D	18.5-20.5	1.1mm	7.6mm
BB-HERP-104/1D	0-2	7.7mm	38mm
BB-HERP-104/2D	7-8.3	.8mm	10mm
BB-HERP-104/3D	12-14	.73mm	3.1mm

Avg D50 = 9.5+.82+5.5+10.1+1.1+7.7+0.8+0.73 / 8 = 3.5 mm

Avg D84 = 23+7+30+27+7.6+38+10+3.1 / 8 = 18.2 mm

0-2' Depth Avg D50 = 9.5+5.5+7.7/3 = 7.6 mm (Relevant to Contraction Scour)

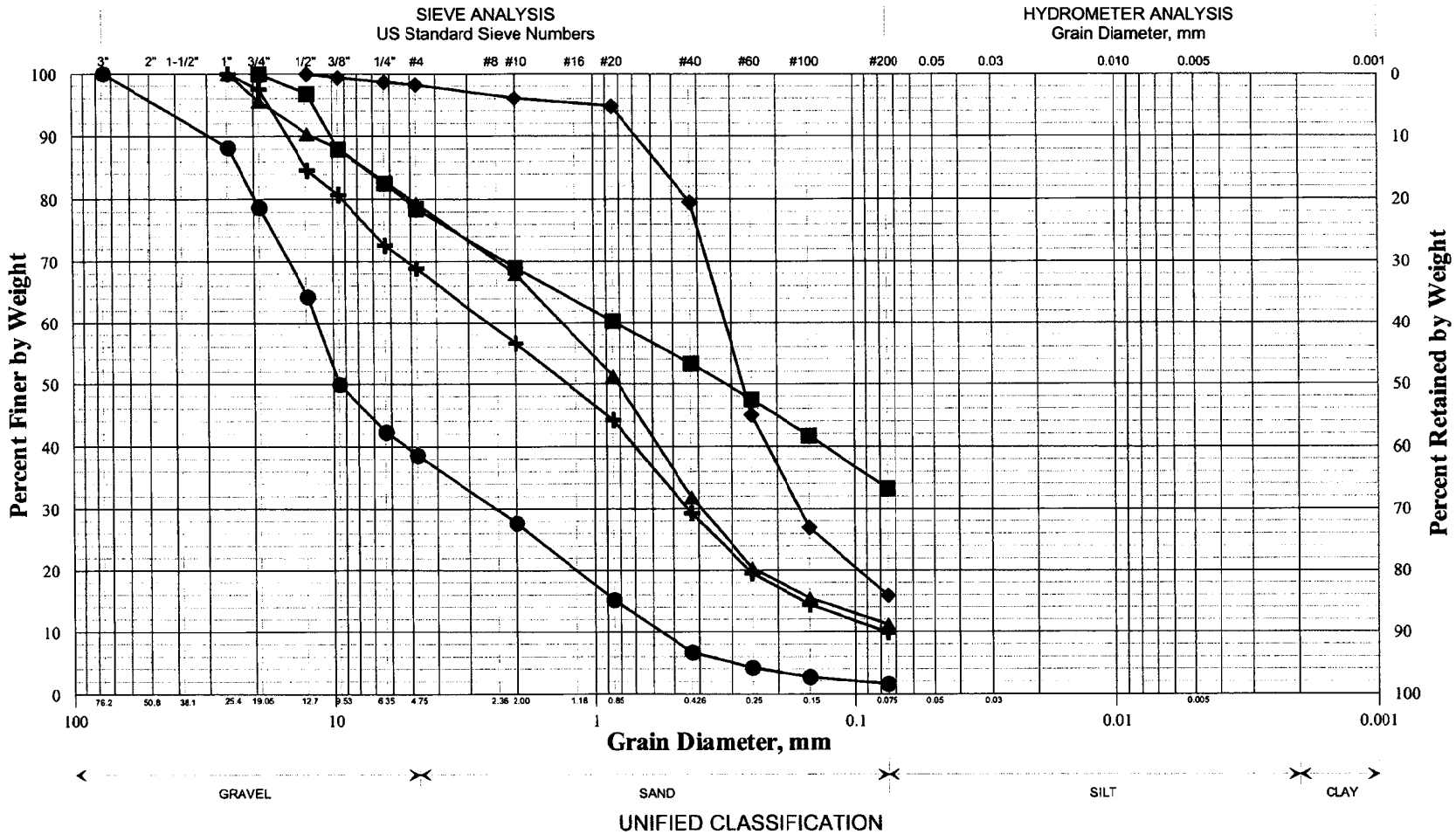
Neglect 0-2' Depth & 103/2D 10.1mm outlier Avg D50 = 0.82+1.1+0.8+0.73/4 = 0.86mm (Relevant to Contraction Scour > 2 ft)

Neglect 0-2' Depth & 103/2D 27mm outlier Avg D84 = 7+7.6+10+3.1/4 = 6.9mm (Relevant to Complex Pier Scour)

D50 = 5mm for Contraction Scour if ≲ 2'-4' computed
= 0.8mm for Contraction scour if w/ 5mm computed ≳ 2'-4'
& for Pier Scour Consideration for armoring potential K4 = 1.0

D84 = 7mm for Complex Pier Scour Ks = 7mm = 0.023'

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



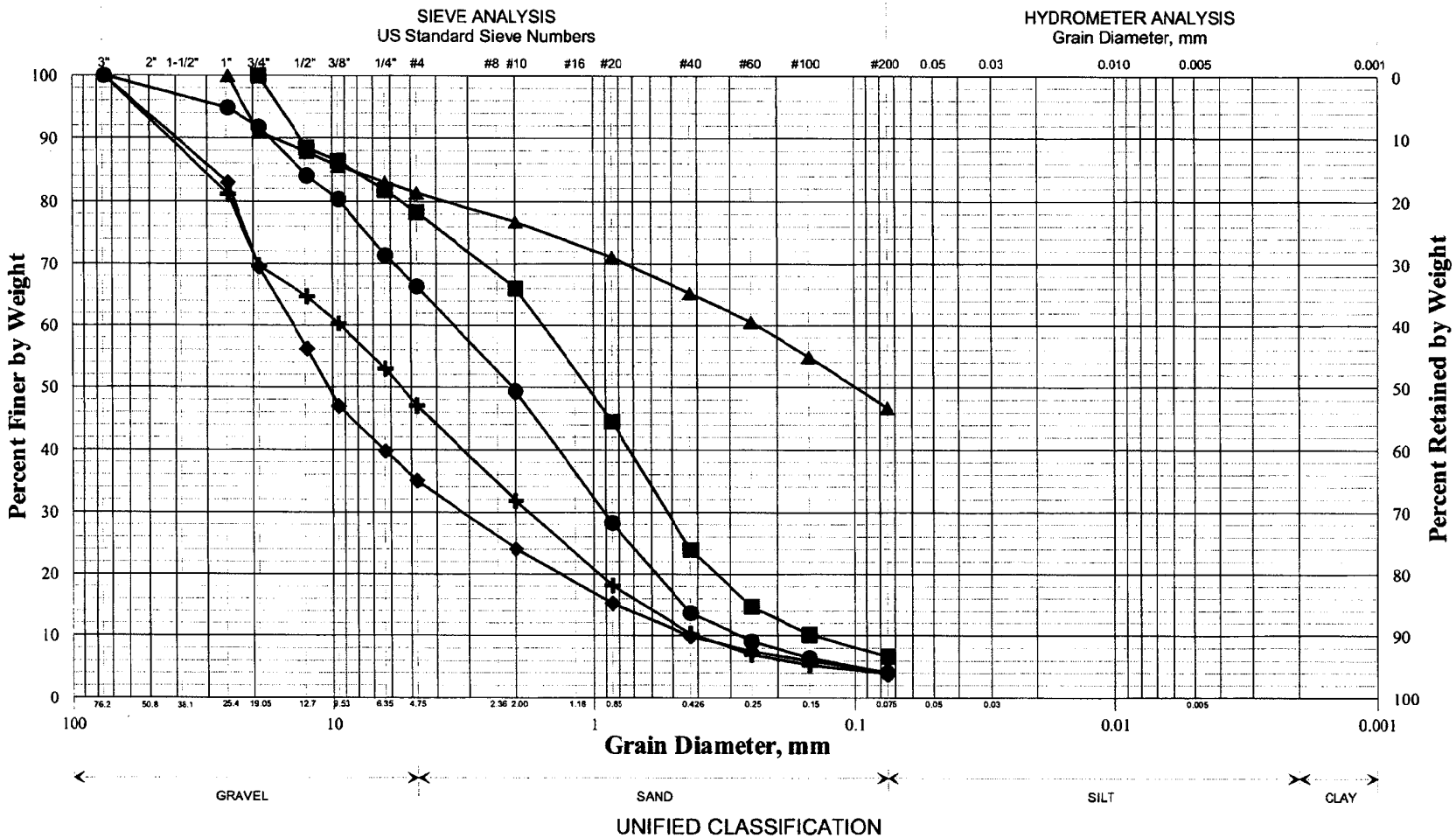
	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-101/1D	633293	1761886	1.0-3.0	SAND, some gravel, trace silt.	3.9			
◆	BB-HEPR-101/4D	633293	1761886	15.0-17.0	SAND, little silt, trace gravel.	21.8			
■	BB-HEPR-101/6D	633293	1761886	25.0-25.8	SAND, some silt, some gravel.	9.7			
●	BB-HEPR-102/1D	633348.9	1762069	0.0-2.0	Sandy GRAVEL, trace silt.	10.2			
▲	BB-HEPR-102/4D	633348.9	1762069	16.0-18.0	SAND, some gravel, little silt.	11.7			
✕									

PIN
016705.00
Town
Howland, Enfield
Reported by/Date
WHITE, TERRY A 9/29/2010

161

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-103/1D	633430.5	1762334	0.0-2.0	Sandy GRAVEL, trace silt.	9.9			
◆	BB-HEPR-103/2D	633430.5	1762334	3.5-5.5	GRAVEL, some sand, trace silt.	7.7			
■	BB-HEPR-103/5D	633430.5	1762334	18.5-20.5	SAND, some gravel, trace silt.	11.5			
●	BB-HEPR-103/7D	633430.5	1762334	28.5-30.0	SAND, some gravel, trace silt.	10.1			
▲	BB-HEPR-103/10D	633430.5	1762334	43.5-44.8	SILT, some sand, little gravel.	7.9			
✕									

PIN
016705.00
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Howland,Enfield 162
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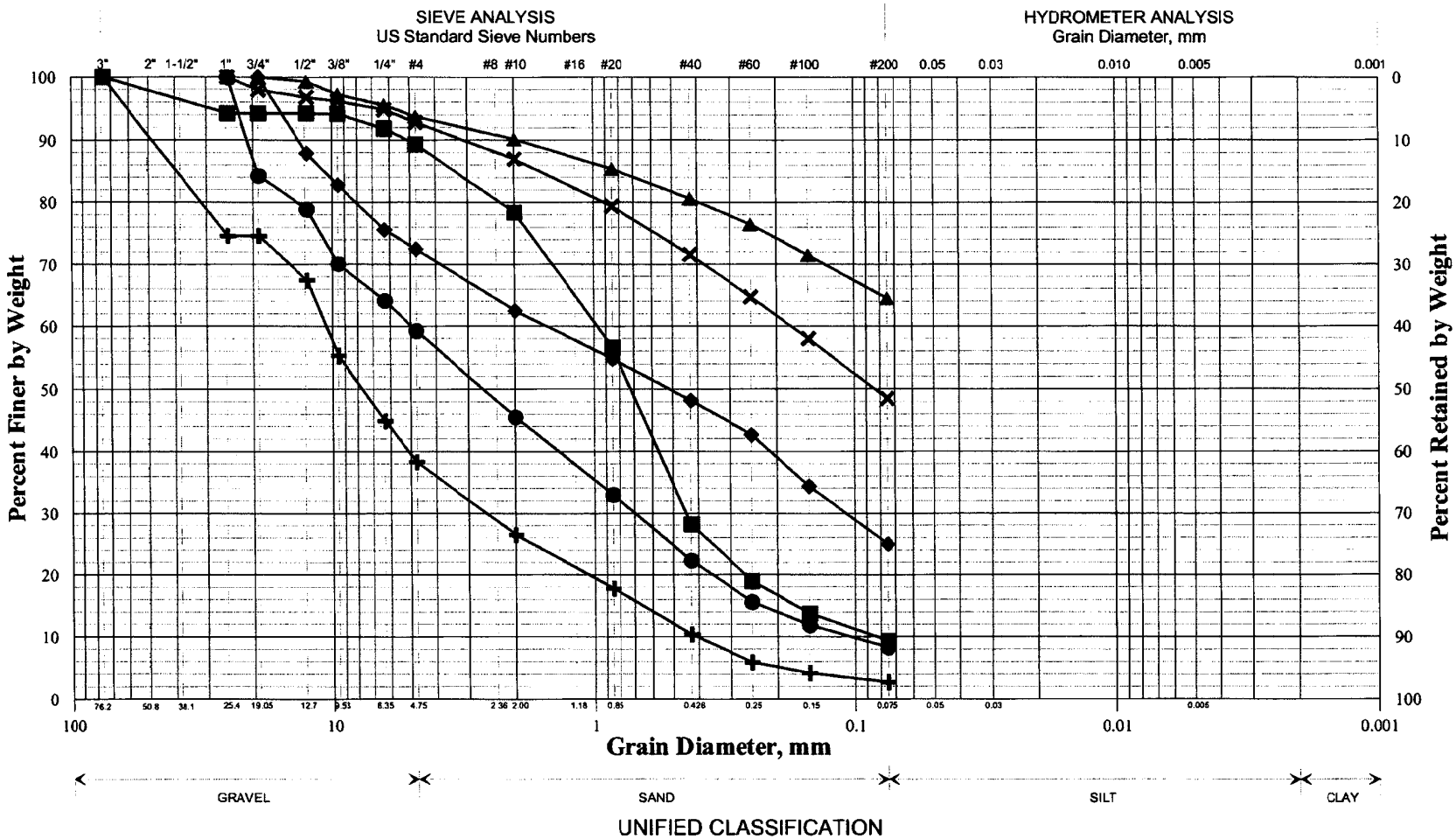
WHITE, TERRY A 9/29/2010

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Northing	Easting	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-HEPR-104/1D	633510	1762593	0.0-2.0	GRAVEL, some sand, trace silt.	10.1			
◆	BB-HEPR-104/2D	633510	1762593	7.0-8.3	SAND, some gravel, some silt.	8.3			
■	BB-HEPR-104/3D	633510	1762593	12.0-14.0	SAND, little gravel, trace silt.	15.1			
●	BB-HEPR-104/5D	633510	1762593	22.0-24.0	Gravelly SAND, trace silt.	10.3			
▲	BB-HEPR-104/7D	633510	1762593	32.0-34.0	SILT, some sand, trace gravel.	11.6			
✕	BB-HEPR-104/10D	633510	1762593	47.0-48.8	Sandy SILT, trace gravel.	8.4			

PIN
016705.00
Town
Howland, Enfield 163
Reported by/Date
WHITE, TERRY A 9/29/2010

T·Y·LIN INTERNATIONAL
engineers | planners | scientists

PROJECT:	Howland	**JOB NO:**	411697.00
TASK:	Scour Analysis of Proposed Structure		
DESIGN:	DSM	**DATE:**	2/4/2013
CHECK:	RMH	**DATE:**	6/3/2013

Howland Alt 5: Scour Analysis - Q_{100}

References:

1. *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges, 5th Edition*
2. *Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures, 3rd Edition*

Methodology:

Check each type of scour in turn, using HEC-18 equations.

1. *Contraction scour*
   - *Clear Water Scour*
   - *Live Bed Scour*
2. *Abutment Scour*
3. *Simple Pier Scour*
4. *Complex Pier Scour*

Design revetment using HEC-23 equations

Assumptions:

- *Checking Q100 with this sheet*
- *Upstream Section is River Sta 10050 in HEC-RAS*
- *D50 is 5 mm for top 2-4' of bed material, 0.8 mm below that*
- *D84 is 7mm for Complex Pier Scour*
- *For pier scour, use Easternmost pier, in deepest water*

Legend:

Text in this font is a direct quotation of the HEC manual

Text in this font is this sheet's descriptions and discussion

Equations with this formatting are manual inputs

Equations with this formatting are important results

Contraction Scour (HEC-18 6):

Critical Velocity Calculation (HEC-18 6.2.1)

To calculate the critical velocity above which bed material of size D and smaller will be transported, use the following equation:

$$V_c = K_u \cdot y^{\frac{1}{6}} \cdot D^{\frac{1}{3}} \qquad \text{(Equation 6.1)}$$

Where:
- V_c = Critical velocity above which bed material of size D and smaller will be transported
- y = Average depth of flow upstream of the bridge
- D = Particle size for V_c
- D_{50} = Particle size in a mixture of which 50 percent are smaller
- K_u = 11.17 for English units

D_{50} of Bed Material $D_{50} := 5mm$

Average Depth of Flow Upstream for Given Condition $y := 24.78ft$

Coefficient for Critical Velocity Equation $K_u := 11.17$

Critical Velocity Calculation

$$V_c := K_u \cdot \left(\frac{y}{ft}\right)^{\left(\frac{1}{6}\right)} \cdot \left(\frac{D_{50}}{ft}\right)^{\left(\frac{1}{3}\right)} \cdot \frac{ft}{s} = 4.85 \cdot \frac{ft}{s}$$

Live Bed Contraction Scour (HEC-18 6.3)

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\left(\frac{6}{7}\right)} \cdot \left(\frac{W_1}{W_2}\right)^{k_1}$$

$$y_s = y_2 - y_0$$

Where:
- y_1 = Average depth in the upstream main channel
- y_2 = Average depth in the contracted section
- y_0 = Existing depth in the contracted section before scour
- y_s = Average scour depth
- Q_1 = Flow in the upstream channel transporting sediment, not including overbank flows
- Q_2 = Flow in the contracted channel
- W_1 = Bottom width of the upstream main channel that is transporting bed material
- W_2 = Bottom width of the main channel in the contracted section less pier width(s)
- k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

- V_* = $(\tau_0/\rho)^{1/2} = (gy_1S_1)^{1/2}$, shear velocity in the upstream section
- ω = Fall velocity of bed material based on the D_{50} (Figure 6.8)
- g = Acceleration of gravity
- S_1 = Slope of energy grade line of main channel

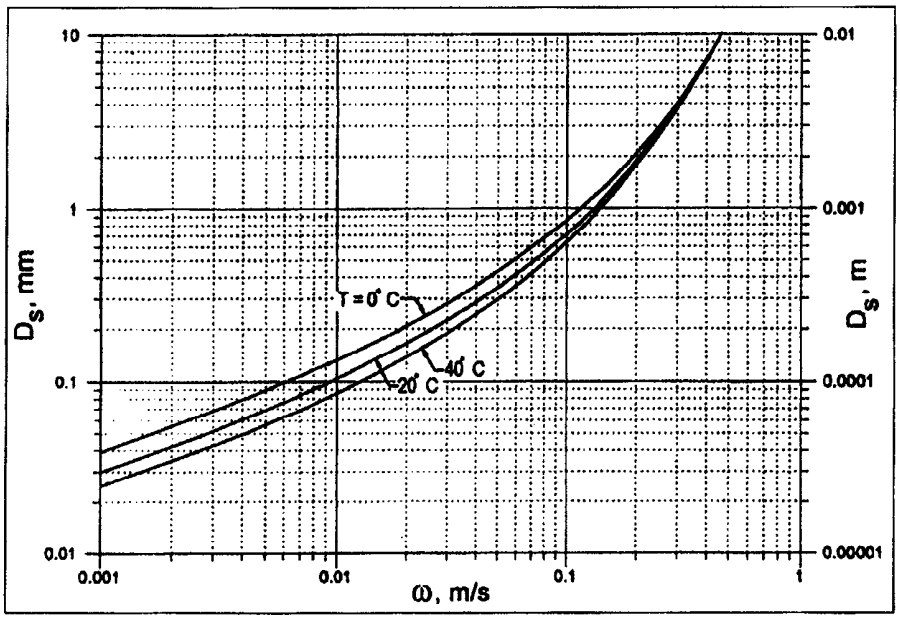


Figure 6.8

Live Bed Contraction Scour, cont.

D_{50} of Bed Material $D_{50} = 5 \cdot mm$

Fall velocity of bed material based on the D_{50} (From Figure 6.8, above) $\omega := 0.34 \frac{m}{s} = 1.12 \cdot \frac{ft}{s}$

Acceleration of Gravity $g = 32.17 \cdot \frac{ft}{s^2}$

Average Depth in Upstream Main Channel $y_1 := 24.78ft$

Slope of Energy Grade Line of Main Channel $S_1 := .0004$

Shear Velocity in Upstream Section $V_{star} := \sqrt{g \cdot y_1 \cdot S_1} = 0.56 \cdot \frac{ft}{s}$

Live Bed Contraction Scour Exponent

$$k_1 := \begin{cases} 0.59 & \text{if } (V_{star} \div \omega) < 0.50 \\ 0.64 & \text{if } (V_{star} \div \omega) \geq 0.50 \wedge (V_{star} \div \omega) \leq 2.0 \\ 0.69 & \text{if } (V_{star} \div \omega) > 2.0 \end{cases} = 0.64$$

Flow in the Upstream Main Channel Transporting Sediment $Q_1 := 146523.3 \frac{ft^3}{s}$

Flow in the Contracted Channel $Q_2 := 147968.50 \frac{ft^3}{s}$

Bottom Width of the Upstream Main Channel $W_1 := 850ft$

Bottom Width of the Main Channel in the Contracted Section $W_2 := 838ft - 3 \cdot 4ft = 826 \cdot ft$

Average Depth in the Contracted Section

$$y_{2.live} := y_1 \cdot \left[\left(\frac{Q_2}{Q_1} \right)^{\left(\frac{6}{7}\right)} \cdot \left(\frac{W_1}{W_2} \right)^{k_1} \right] = 25.45 \cdot ft$$

Existing Depth in the Contracted Section (Before Scour) $y_0 := 25.28ft$

Live-Bed Contraction Scour Depth $y_{s.live} := y_{2.live} - y_0 = 0.17 \cdot ft$

Clear-Water Contraction Scour (HEC-18 6.4)

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (presented in the Appendix C)

$$y_2 = \left[\frac{K_u \cdot Q^2}{D_m^{\left(\frac{2}{3}\right)} \cdot W^2}\right]^{\left(\frac{3}{7}\right)}$$

$$y_s = y_2 - y_0$$

Where:
- y_2 = Average equilibrium depth in the contracted section after contraction scour
- y_0 = Average existing depth in the contracted section
- y_s = Average contraction scour depth
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W
- D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section
- D_{50} = Median diameter of bed material
- W = Bottom width of the contracted section less pier width(s)
- K_u = 0.025 for SI units, 0.0077 for English units

Coefficient for Calculating Clear-Water Contraction Scour $K_u := 0.0077$

Discharge Through the Bridge $Q := Q_2 = 147968.5 \cdot \frac{ft^3}{s}$

D_{50} of Bed Material $D_{50} = 0.20 \cdot in$

Diameter of Smallest Nontransportable Particle in Bed Material $D_m := 1.25 \cdot D_{50} = 0.25\ in$

Bottom Width of the Main Channel in the Contracted Section $W := W_2 = 826 \cdot ft$

Average equilibrium depth in the contracted section after contraction scour

$$y_{2.Clr} := \left[\frac{K_u \cdot \left(Q \div \frac{ft^3}{s}\right)^2}{\left(D_m \div ft\right)^{\left(\frac{2}{3}\right)} \cdot (W \div ft)^2}\right]^{\left(\frac{3}{7}\right)} \cdot ft = 32.2 \cdot ft$$

Existing Depth in the Contracted Section (Before Scour) $y_0 = 25.28 \cdot ft$

Clear-Water Contraction Scour Depth $y_{s.Clr} := y_{2.Clr} - y_0 = 6.92 \cdot ft$

Contraction Scour Summary

Upstream Section Velocity in Channel $V_{us} := 6.96 \frac{ft}{s}$

Critical Velocity Above Which Bed Material Will be Transported $V_c = 4.85 \cdot \frac{ft}{s}$

Contraction Scour Type

$$Contraction_{Scour.Type} := \begin{cases} \text{"Live-Bed"} & \text{if } V_{us} \geq V_c \\ \text{"Clear-Water"} & \text{otherwise} \end{cases}$$

$$\boxed{Contraction_{Scour.Type} = \text{"Live-Bed"}}$$

Contraction Scour Depth

$$y_{s.contraction} := \begin{cases} y_{s.live} & \text{if } V_{us} \geq V_c \\ y_{s.Clr} & \text{otherwise} \end{cases}$$

$$\boxed{y_{s.contraction} = 0.17 \cdot ft}$$

Local Scour at Abutments (HEC-18 8):

NCHRP 24-20 Abutment Scour Approach (HEC-18 8.6.3)

The three scour conditions illustrated in Figure 8.7 are (a) scour occurring when the abutment is in or close to the main channel, (b) scour occurring when the abutment is set back from the main channel, and (c) scour occurring when the embankment breaches and the abutment foundation acts as a pier. As illustrated in Figure 8.8, the NCHRP study also concluded that there is a limiting depth of abutment scour when the geotechnical stability of the embankment or channel bank is reached. The abutment scour computed from the NCHRP approach is total scour at the abutment; it is not added to contraction scour because it already includes contraction scour. The advantages of using the NCHRP abutment scour equations include (1) not using the effective embankment length, L', which is difficult to determine in many situations, (2) the equations are more physically representative of the abutment scour process, and (3) the equations predict total scour at the abutment rather than the abutment scour component that is then added to contraction scour. The scour equations for conditions (a) and (b) are:

$$y_{max} = \alpha_A \cdot y_c \quad \text{or} \quad y_{max} = \alpha_B \cdot y_c \qquad (8.3)$$

$$y_s = y_{max} - y_0 \qquad (8.4)$$

Where:
- y_{max} = Maximum flow depth resulting from abutment scour, ft (m)
- y_c = Flow depth including live-bed or clear-water contraction scour, ft (m)
- α_A = Amplification factor for live-bed conditions
- α_B = Amplification factor for clear-water conditions
- y_s = Abutment scour depth, ft (m)
- y_0 = Flow depth prior to scour, ft (m)

Based on the NCHRP (2010b) study, if the projected length of the embankment, L, is 75 percent or greater than the width of the floodplain (B_f), scour condition (a) in Figure 8.7 occurs and the **contraction scour calculation is performed using a live-bed scour calculation**. The contraction scour equation is a simplified version of the live-bed contraction scour equation (see Chapter 6). The equation combines the discharge and width ratios due to the similarity of the exponents because other uncertainties are more significant. By combining the discharge and width, the live-bed contraction scour equation simplifies to the ratio of two unit discharges. Unit discharge (q) can be estimated either by discharge divided by width or by the product of velocity and depth. The contraction scour equation is:

$$y_c = y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\left(\frac{6}{7}\right)} \qquad (8.5)$$

Where:
- y_c = Flow depth including live-bed contraction scour, ft (m)
- y_1 = Upstream flow depth, ft (m)
- q_1 = Upstream unit discharge, ft^2/s (m^2/s)
- q_{2c} = Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s (m^2/s)

NCHRP 24-20 Abutment Scour Approach, cont.

The value of q_{2c} can be estimated as the total discharge in the bridge opening divided by the width of the bridge opening. The value of y_c is then used in Equation 8.3 to compute the total flow depth at the abutment. The value of α_A is selected from Figure 8.9 for spill through abutments and Figure 8.10 for wingwall abutments. The solid curves should be used for design. The dashed curves represent theoretical conditions that have yet to be proven experimentally. For low values of q_2/q_1, contraction scour is small, but the amplification factor is large because flow separation and turbulence dominate the abutment scour process. For large values of q_2/q_1, contraction scour dominates the abutment scour process and the amplification factor is small.

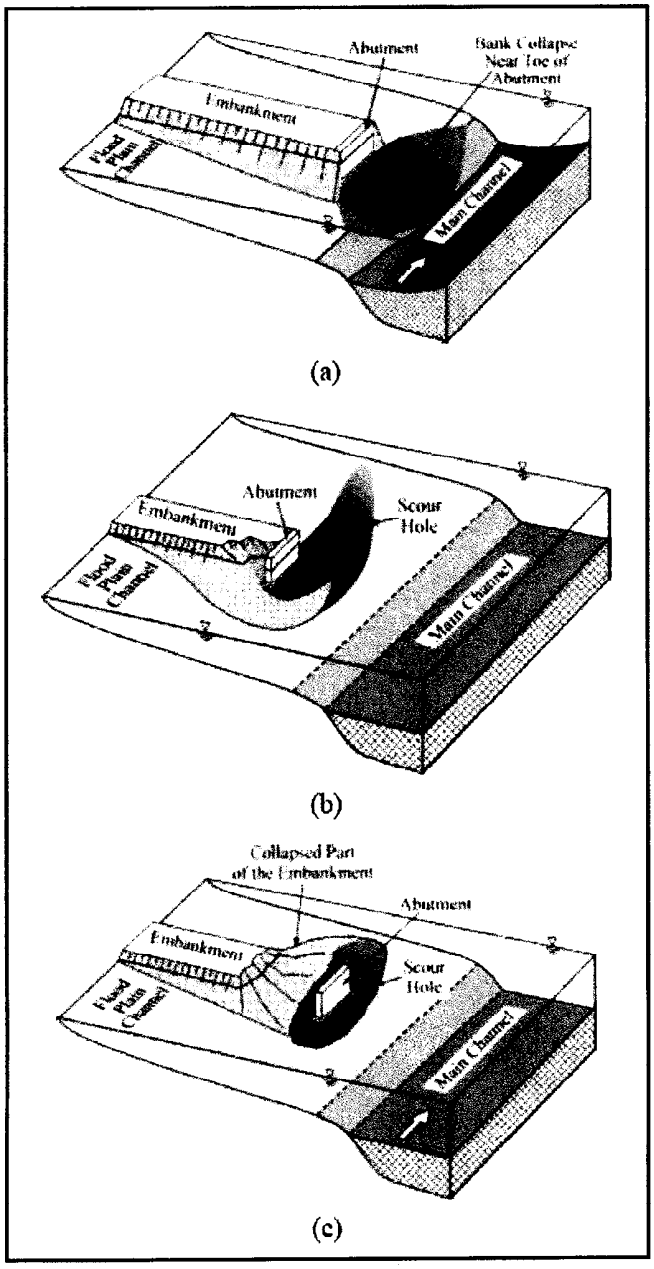


Figure 8.7. Abutment scour conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

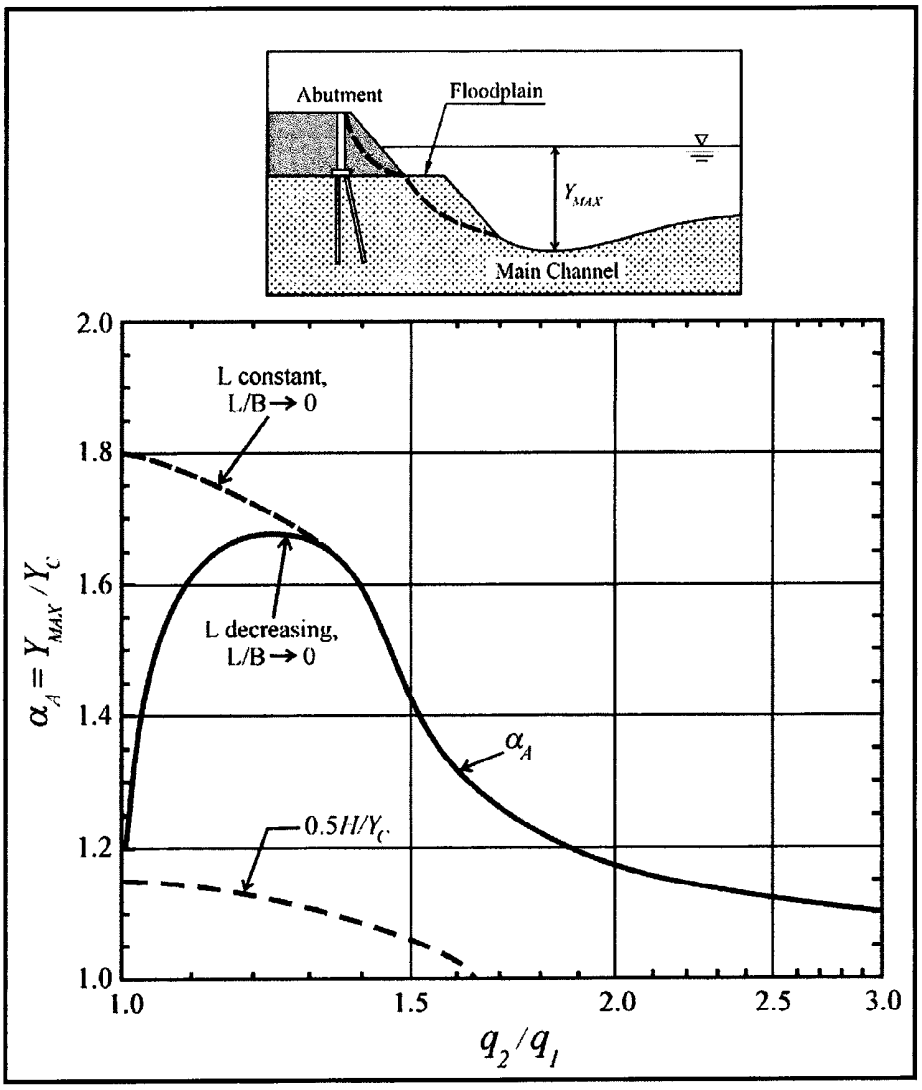


Figure 8.9. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

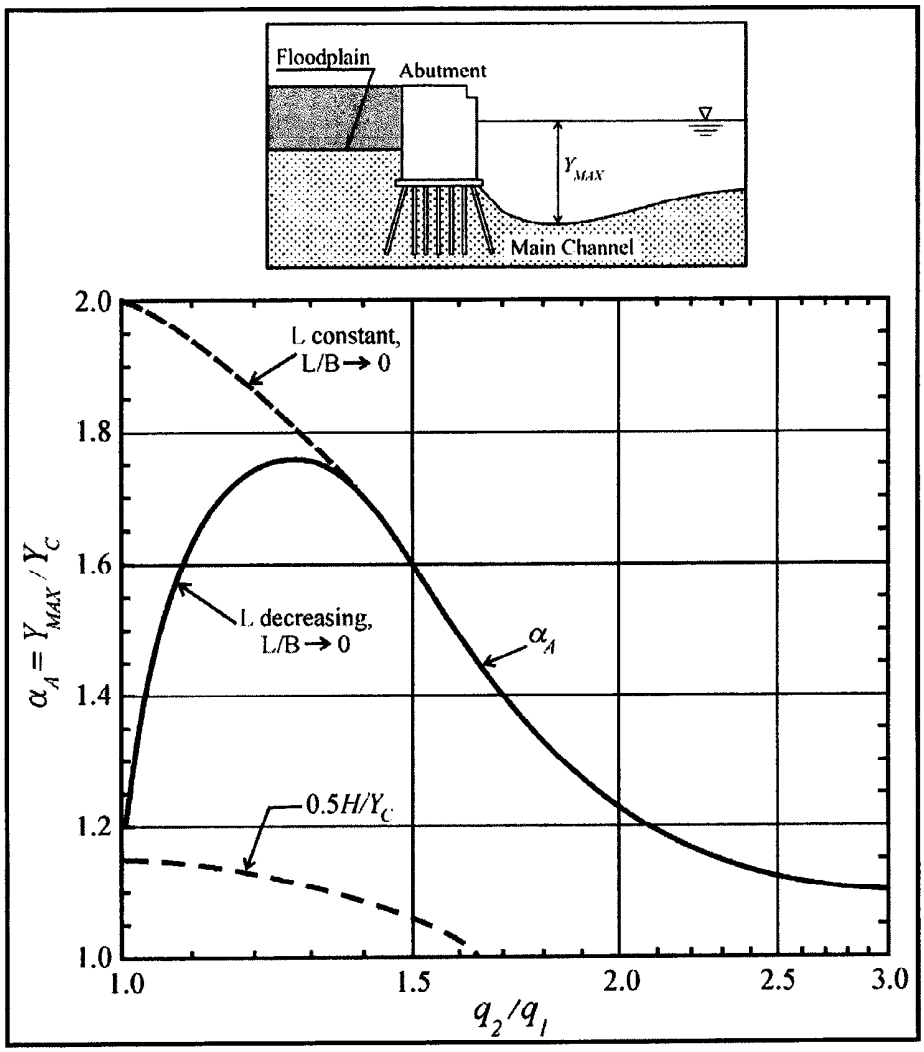


Figure 8.10. Scour amplification factor for wingwall abutments and live-bed conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

NOTE: this analysis only includes the equations for live-bed abutment scour, since that is what is occuring for this abutment case.

Total Upstream Channel Flow $Q_1 = 146523.3 \cdot \frac{ft^3}{s}$

Flow through bridge opening $Q_{2c} := Q_2 = 147968.5 \cdot \frac{ft^3}{s}$

Width of the Upstream Main Channel $W_1 = 850 \cdot ft$

Width of flow through Bridge Opening $W_{2c} := W_2 = 826 \cdot ft$

Upstream unit discharge $q_1 := \frac{Q_1}{W_1} = 172.38 \cdot \frac{ft^2}{s}$

Unit discharge in the constricted opening accounting for non-uniform flow distribution $q_{2c} := \frac{Q_{2c}}{W_{2c}} = 179.14 \cdot \frac{ft^2}{s}$

Ratio of q_2 to q_1 $\frac{q_{2c}}{q_1} = 1.04$

Amplification factor for live-bed conditions $\alpha_A := 1.46$ *From chart above*

Upstream flow depth $y_1 = 24.78 \cdot ft$

Flow depth including live-bed contraction scour $y_{c.abut} := y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\left(\frac{6}{7}\right)} = 25.61 \cdot ft$

Maximum flow depth resulting from abutment scour $y_{max} := y_{c.abut} \cdot \alpha_A = 37.39 \cdot ft$

Existing Depth in the Contracted Section (Before Scour) $y_0 = 25.28 \cdot ft$

Abutment Scour Depth $y_{s.abut} := y_{max} - y_0 = 12.11 \cdot ft$

Local Scour at Piers (HEC-18 7):

Local Pier Scour Equation (HEC-18 7.2)

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water scour. The equation predicts maximum pier scour depths. The equation is:

$$\frac{y_s}{y_1} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_1}\right)^{0.65} \cdot Fr_1^{0.43} \qquad (7.1)$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$$y_s \le 2.4 \cdot a \quad \text{if } Fr \le 0.8$$
$$y_s \le 3.0 \cdot a \quad \text{if } Fr > 0.8 \qquad (7.2)$$

Where:
- y_s = Scour depth
- y_1 = Flow depth directly upstream of the pier
- K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
- K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
- K_3 = Correction factor for bed condition from Table 7.3
- a = Pier width
- L = Length of pier
- Fr_1 = Froude Number directly upstream of the pier $= \frac{V_1}{\sqrt{g \cdot y_1}}$
- V_1 = Mean velocity of flow directly upstream of the pier
- g = Acceleration of gravity

The correction factor, K_2, for angle of attack of flow, θ, is calculated using the following equation:

$$K_2 = \left(\cos(\theta) + \frac{L}{a \cdot \sin(\theta)}\right)^{0.65} \qquad (7.4)$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 7.4 and Table 7.2.

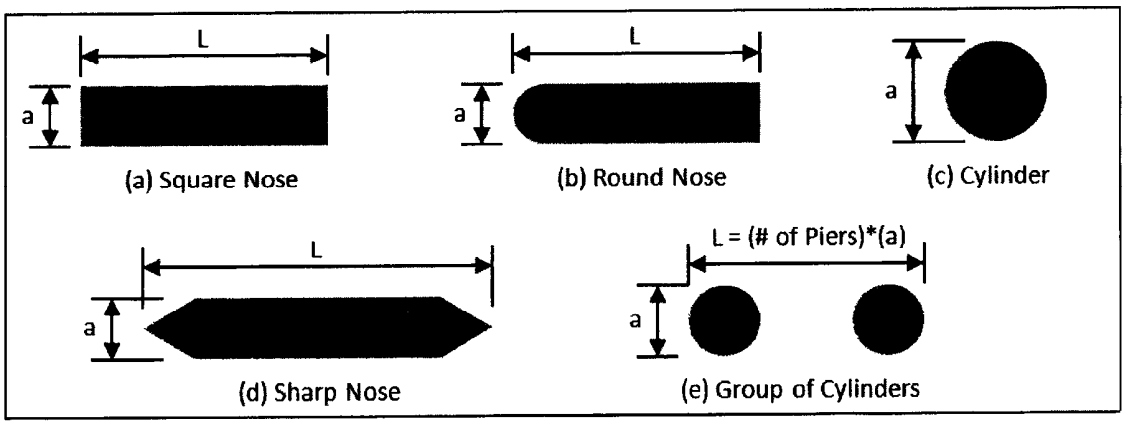


Figure 7.3

Local Pier Scour, cont.

Table 7.1. Correction Factor, K_1, for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.2. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier			

Table 7.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.		
Bed Condition	Dune Height ft	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

Pier Width $a := 4ft$

Length of Pier $L := 43.5ft$

Length to Width Ratio (for chart) $\frac{L}{a} = 10.88$

Flow Depth Directly Upstream of Pier $y_{1.pier} := 27.77ft$ *Section STA 9935*

Angle of Attack of Flow $\theta := 0deg$

Mean Velocity of Flow Directly Upstream of Pier $V_1 := 7.16\frac{ft}{s}$ *Section STA 9935*

Correction Factor for Pier Nose Shape $K_1 := 0.9$

Correction Factor for Angle of Attack of Flow $K_2 := \left(\cos(\theta) + \frac{L}{a}\cdot\sin(\theta)\right)^{0.65} = 1$

Correction Factor for Bed Condition $K_3 := 1.1$

Local Pier Scour, cont.

Froude Number Directly Upstream of Pier

$$Fr_1 := \frac{V_1}{\sqrt{g \cdot y_{1.pier}}} = 0.24$$

Ratio of Scour Depth to Flow Depth

$$Ratio_{y.s_y.1} := 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_1}\right)^{0.65} \cdot Fr_1^{\,0.43} = 0.33$$

Max Scour Depth

$$y_{s.pier.MAX} := \text{if}\left(Fr_1 \le 0.8, 2.4 \cdot a, 3.0 \cdot a\right) = 9.6 \cdot ft$$

Pier Scour Depth

$$y_{s.pier} := \min\left(y_{s.pier.MAX}, Ratio_{y.s_y.1} \cdot y_{1.pier}\right) = 9.09 \cdot ft$$

This is below the top of the pile cap (which is located 3 feet below grade) so complex pier scour must be evaluated

Scour for Complex Pier Foundations (HEC-18 7.5)

The components of a complex pier are illustrated in Figure 7.5 (Jones and Sheppard 2000). This is followed by a definition of the variables. Note that the pile cap can be above the water surface, at the water surface, in the water or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, the methods of analysis given in this manual may give smaller or larger scour depths.

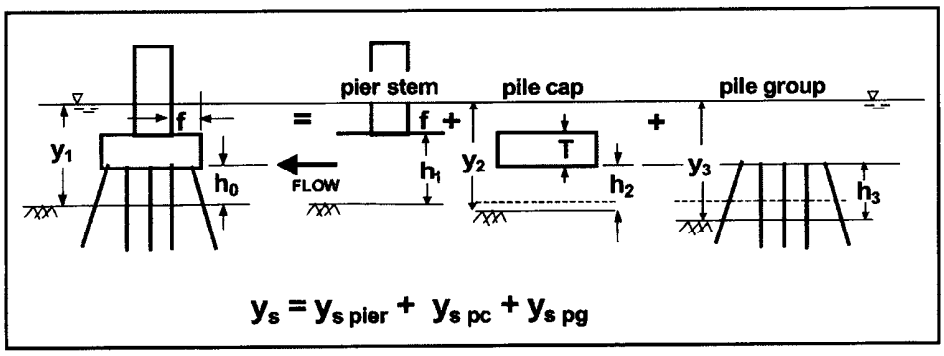


Figure 7.5. Definition sketch for scour components for a complex pier.

Scour for Complex Pier Foundations, cont.

The variables illustrated in Figure 7.5 and others used in computations are as follows:

f = Distance between front edge of pile cap or footing and pier, ft (m)
h_o = Height of the pile cap above bed at beginning of computation, ft (m)
$h_1 = h_o + T$ = height of the pier stem above the bed before scour, ft (m)
$h_2 = h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, ft (m)
$h_3 = h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after the pier stem and pile cap scour components have been computed, ft (m)
S = Spacing between columns of piles, pile center to pile center, ft (m)
T = Thickness of pile cap or footing, ft (m)
y_1 = Approach flow depth at the beginning of computations, ft (m)
$y_2 = y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations ft (m)
$y_3 = y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, ft (m)
V_1 = Approach velocity used at the beginning of computations, ft/sec (m/sec)
$V_2 = V_1(y_1/y_2)$ = adjusted velocity for pile cap computations, ft/sec (m/sec)
$V_3 = V_1(y_1/y_3)$ = adjusted velocity for pile group computations, ft/sec (m/sec)

Preliminary Inputs:

Distance between front edge of pile cap or footing and pier	$f := 7.5ft$	*Conservatively combine seal and footing and using seal dimensions*
Height of the pile cap above bed at beginning of computation	$h_0 := -8ft$	
Thickness of pile cap	$T := 5ft$	
Height of the pier stem above the bed before PIER scour	$h_1 := h_0 + T + y_{s.contraction} = -2.83 \cdot ft$	
Approach flow depth at the beginning of computations	$y_{1.pier} = 27.77 \cdot ft$	
Approach velocity used at the beginning of computations	$V_1 = 7.16 \cdot \frac{ft}{s}$	

Scour for Complex Pier Foundations, cont.

Total scour from superposition of components is given by:

$$y_s = y_{s_pier} + y_{s_pc} + y_{s_pg} \qquad (7.22)$$

where:

y_s = Total scour depth, ft (m)
$y_{s\ pier}$ = Scour component for the pier stem in the flow, ft (m)
$y_{s\ pc}$ = Scour component for the pier cap or footing in the flow, ft (m)
$y_{s\ pg}$ = Scour component for the piles exposed to the flow, ft (m)

Each of the scour components is computed from the basic pier scour Equation 7.1 using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 7.5, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections guidance for calculating each of the components is given.

Determination of the Pier Stem Scour Depth component (HEC-18 7.5.3)

The first computation is the scour estimate, $y_{s\ pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation (Equation 7.1). In Equation 7.1, a_{pier} is the pier width and other variables in the equation are as defined previously. This base scour estimate is multiplied by $K_{h\ pier}$, given in Figure 7.6 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component as follows:

$$\frac{y_{s_pier}}{y_1} = K_{h_pier}\left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a_{pier}}{y_1}\right)^{0.65} \cdot \left(\frac{V_1}{\sqrt{g \cdot y_{1\ pier}}}\right)^{0.43}\right] \qquad (7.23)$$

where:

$K_{h\ pier}$ = Coefficient to account for height of pier stem above bed and shielding effect by pile cap overhang distance "f" in front of pier stem (from Figure 7.6)

Pier Width	$a_{pier} := a = 4 \cdot ft$	
Length of Pier	$L = 43.5 \cdot ft$	
Flow Depth Directly Upstream of Pier	$y_{1.pier} = 27.77 \cdot ft$	*Section STA 9935*
Angle of Attack of Flow	$\theta = 0 \cdot deg$	
Mean Velocity of Flow Directly Upstream of Pier	$V_1 = 7.16 \cdot \frac{ft}{s}$	*Section STA 9935*
Correction Factor for Pier Nose Shape	$K_1 = 0.9$	
Correction Factor for Angle of Attack of Flow	$K_2 = 1$	
Correction Factor for Bed Condition	$K_3 = 1.1$	

Pier Stem Scour Depth component, cont.

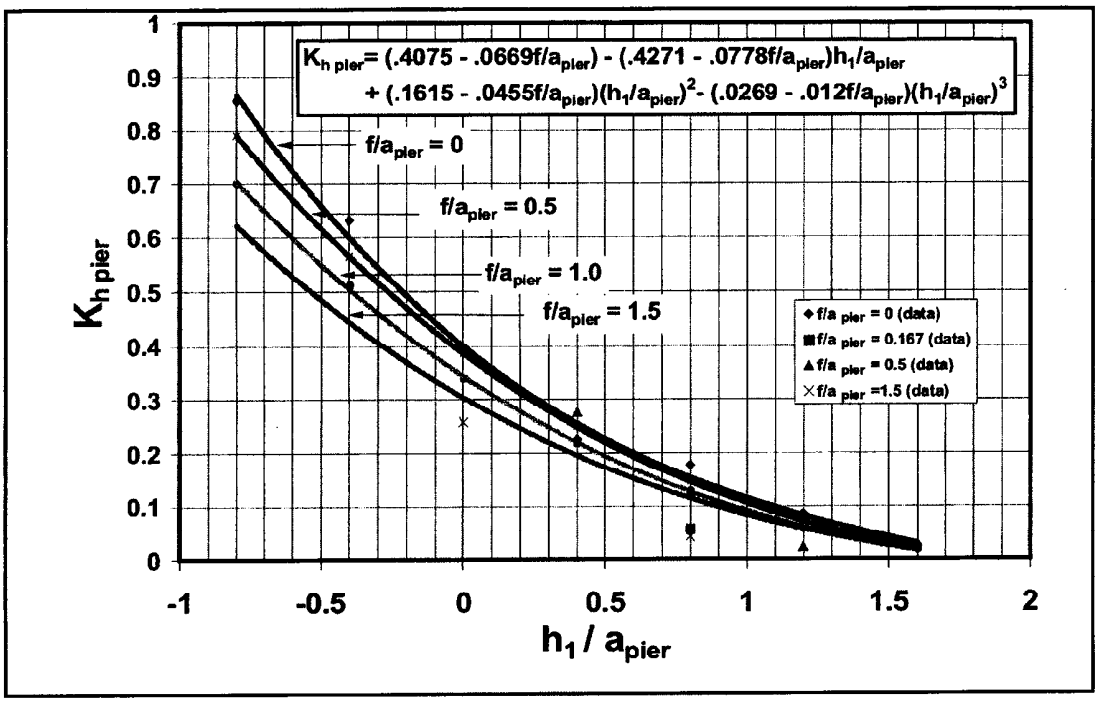


Figure 7.6. Suspended pier scour ratio (Jones and Sheppard 2000).

Pile Cap Overhang Distance $f = 7.5 \cdot ft$

Ratio f/a$_{pier}$ $\frac{f}{a_{pier}} = 1.88$

For use in above chart

Ratio h$_1$/a$_{pier}$ $\frac{h_1}{a_{pier}} = -0.71$

Coefficient to account for height of pier stem above bed and shielding effect by pile cap overhang distance "f" in front of pier stem (From Figure 7.6) $K_{h_pier} := 0.57$

Ratio of Scour Depth to Flow Depth

$$Ratio_{y.s_pier_y.1} := K_{h_pier} \cdot \left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a_{pier}}{y_{1.pier}} \right)^{0.65} \cdot \left(\frac{V_1}{\sqrt{g \cdot y_{1.pier}}} \right)^{0.43} \right] = 0.17$$

Pier Stem Scour Depth $y_{s.pier} := Ratio_{y.s_pier_y.1} \cdot y_{1.pier} = 4.81 \cdot ft$

Determination of the Pile Cap (Footing) Scour Depth Component (HEC-18 7.5.4)

Note: only Case 2 is represented here, where the tremie seal acts as a very tall footing, and the piles are never exposed to flow. For exposed piles, please refer to HEC 18 Section 7.5.4

Case 2. Bottom of the Pile Cap (Footing) Located On or Below the Bed.

One limitation of the procedure described above [Case 1] is that the design chart in Figure 7.7 has not been developed for the case of the bottom of the pile cap or footing being below the bed (i.e., negative values of h2). In this case, use a modification of the exposed footing procedure that has been described in previous editions of HEC-18. The previous procedure was developed from experiments in which the footing was never undermined by scour and tended to be an over predictor if the footing is undermined.

As for Case 1:

$$y_2 = y_1 + \frac{y_{s_pier}}{2}$$

$$V_2 = V_1 \cdot \left(\frac{y_1}{y_2}\right)$$

The average velocity of flow at the exposed footing (V_f) is determined using the following equation:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93 \cdot \frac{y_f}{k_s} + 1\right)}{\ln\left(10.93 \cdot \frac{y_2}{k_s} + 1\right)} \qquad (7.25)$$

where:

V_f = Average velocity in the flow zone below the top of the footing, ft/s (m/s)
V_2 = Average adjusted velocity in vertical of flow approaching the pier, ft/s (m/s)
ln = Natural log to the base e
y_f = $h_1 + y_{s\ pier}/2$ = distance from the bed (after degradation, contraction scour, and pier stem scour) to the top of the footing, ft (m)
k_s = Grain roughness of the bed (normally taken as the D_{84} for sand size bed material and 3.5 D_{84} for gravel and coarser bed material), ft (m)
y_2 = Adjusted depth of flow upstream of the pier, including degradation, contraction scour and half the pier stem scour, ft (m)

See Figure 7.8 for an illustration of variables.

Compute the pile cap scour depth component, $y_{s\ pc}$ from Equation 7.1 using the full pile cap width, a_{pc}, y_f, V_f as the width, flow depth, and velocity parameters, respectively. The wide pier factor K_w in Section 7.4 should be used in this computation if (1) the total depth $y_2 < 0.8\ a_{pc}$, (2) the Froude Number $V_2/(gy_2)^{1/2} < 1$, and (3) $a_{pc} > 50\ D_{50}$. Use y_2/a_{pc} to compute the K_w factor if it is applicable. The scour component equation for the case 2 pile cap or footing can then be written:

$$\frac{y_{spc}}{y_f} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_w \cdot \left(\frac{a_{pc}}{y_f}\right)^{0.65} \cdot \left(\frac{V_f}{\sqrt{g \cdot y_f}}\right)^{0.43} \qquad (7.26)$$

Determination of the Pile Cap (Footing) Scour Depth Component, cont.

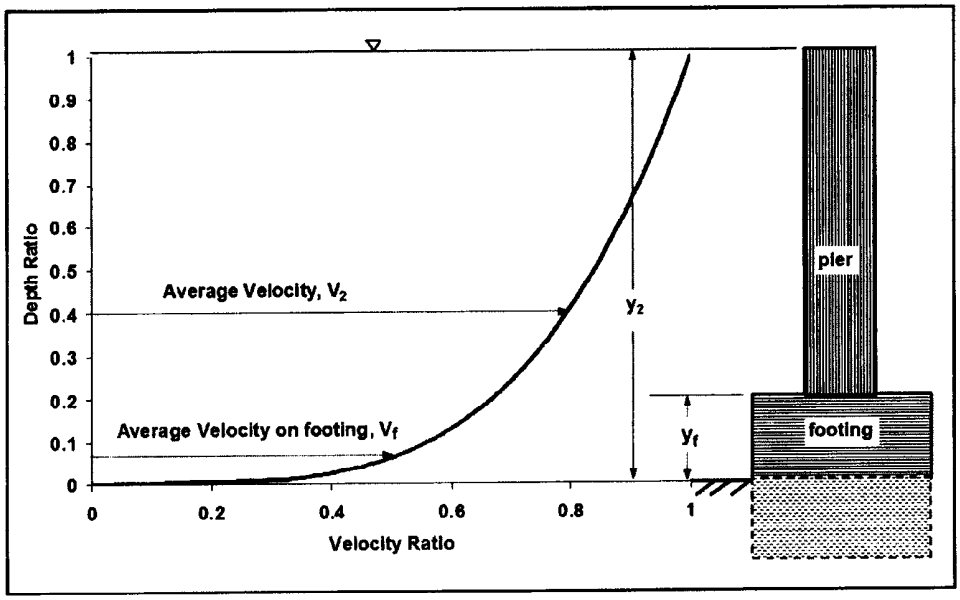


Figure 7.8. Definition sketch for velocity and depth on exposed footing.

Pier Scour at Wide Piers (Section 7.4)

$$K_w = 2.58 \cdot \left(\frac{y}{a}\right)^{0.34} \cdot Fr_1^{\,0.65} \quad \text{for} \quad \frac{V}{V_c} < 1 \qquad (7.20)$$

$$K_w = 1.0 \cdot \left(\frac{y}{a}\right)^{0.13} \cdot Fr_1^{\,0.25} \quad \text{for} \quad \frac{V}{V_c} \geq 1 \qquad (7.21)$$

Engineering judgment should be used in applying K_w because it is based on limited data from flume experiments. Engineering judgment should take into consideration the volume of traffic, the importance of the highway, cost of a failure (potential loss of lives and dollars) and the change in cost that would occur if the K_w factor is used.

Correction Factor for Pier Nose Shape $K_1 := 1.1$ *Footing is rectangular*

Correction Factor for Angle of Attack of Flow $K_2 = 1$

Correction Factor for Bed Condition $K_3 = 1.1$

Adjusted Flow Depth $y_{2.pier} := y_{1.pier} + \frac{y_{s.pier}}{2} = 30.18 \cdot ft$

Adjusted Flow Velocity $V_2 := V_1 \cdot \left(\frac{y_{1.pier}}{y_{2.pier}}\right) = 6.59 \cdot \frac{ft}{s}$

Average Distance from the bed to top of footing: $y_f := h_1 + \frac{y_{s.pier}}{2} = -0.42 \cdot ft$ *No Footing Scour!!*

Determination of the Pile Cap (Footing) Scour Depth Component, cont.

Width of footing $a_{pc} := 23.5\text{ft}$ *(Conservatively assume equal to tremie seal width below)*

Use K_w Factor?

$$UseK_w := \text{if}\left[\left[\left(y_{2.pier} < 0.8\cdot a_{pc}\right) \wedge \left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}} < 1\right) \wedge \left(a_{pc} > 50\cdot D_{50}\right)\right], \text{"Yes"}, \text{"No"}\right]$$

$$UseK_w = \text{"No"}$$

Wide Pier Correction Factor

$$K_w := \begin{cases} 1 & \text{if } UseK_w = \text{"No"} \\ \left[2.58\cdot\left(\frac{y_{2.pier}}{a_{pc}}\right)^{0.34}\cdot\left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}}\right)^{0.65}\right] & \text{if } \left(UseK_w = \text{"Yes"}\right) \wedge \left(\frac{V_2}{V_c} < 1\right) \\ \left[2.58\cdot\left(\frac{y_{2.pier}}{a_{pc}}\right)^{0.34}\cdot\left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}}\right)^{0.65}\right] & \text{if } \left(UseK_w = \text{"Yes"}\right) \wedge \left(\frac{V_2}{V_c} \geq 1\right) \end{cases}$$

$$K_w = 1$$

Grain roughness of the bed (Normall D_{84}) $k_s := 7\text{mm}$

Average velocity in the flow zone below the top of footing

$$V_f := V_2\cdot\frac{\ln\left(10.93\cdot\frac{y_f}{k_s} + 1\right)}{\ln\left(10.93\cdot\frac{y_{2.pier}}{k_s} + 1\right)} = (3.65 + 2.16i)\cdot\frac{\text{ft}}{\text{s}}$$

There is no flow below top of footing, so no further scour, and the velocity calc does not make sense.

Ratio of Scour Depth to Adjusted Flow Depth

$$Ratio_{y.spc_y.f} := 2.0\cdot K_1\cdot K_2\cdot K_3\cdot K_w\cdot\left(\frac{a_{pc}}{y_f}\right)^{0.65}\cdot\left(\frac{V_f}{\sqrt{g\cdot y_f}}\right)^{0.43} = -0.91 + 35i$$

Pile Cap (Footing) Scour Depth

$$y_{spc} := \text{if}\left(y_f \leq 0\text{ft}, 0\text{ft}, Ratio_{y.spc_y.f}\cdot y_f\right) = 0\cdot\text{ft}$$

Total Complex Pier Scour:

Total Complex Pier Scour

$$y_{s.total.pier} := y_{s.pier} + y_{spc} = 4.81\cdot\text{ft}$$

Rock Riprap at Bridge Piers (HEC-23 DG11):

Sizing Rock Riprap at Piers (HEC-23 DG11.3)

To determine the required size of stone for riprap at bridge piers, NCHRP Project 24-23 recommends using the rearranged Isbash equation from the Federal Highway Administration's Hydraulic Engineering Circular No. 23 (Second Edition) (Lagasse et al. 2001) to solve for the median stone diameter:

$$d_{50} = \frac{0.692 \cdot (V_{des})^2}{(S_g - 1) \cdot 2 \cdot g} \qquad (11.1)$$

Where:
- d_{50} = Particle size for which 50% is finer by weight
- V_{des} = Design velocity for local conditions at the pier
- S_g = Specific gravity of rock riprap
- g = Acceleration due to gravity

It is important that the velocity used in Equation 1.1 is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. If the cross-section or channel average velocity, V_{avg}, is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 \cdot K_2 \cdot V_{avg} \qquad (11.2)$$

If a velocity distribution is available from stream tube or flow distribution output of a 1-D model or directly from a 2-D model, then only the pier shape coefficient should be used. The maximum velocity in the active channel Vmax is often used since the channel could shift and the highest velocity could impact any pier.

$$V_{des} = K_1 \cdot V_{max} \qquad (11.3)$$

Where:
- V_{des} = Local velocity at pier
- K_1 = Shape factor equal to 1.5 for round-nose piers or 1.7 for square-faced piers
- K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)
- V_{avg} = Channel average velocity at the bridge
- V_{max} = Maximum velocity in the active channel

Sizing Rock Riprap at Piers, cont.

Specific Gravity of Riprap $S_g := 2.75$

Mean Velocity of Flow Directly Upstream of Pier $V_{avg} := V_1 = 7.16 \cdot \frac{ft}{s}$ *Section STA 9935*

Correction Factor for Pier Nose Shape $K_1 := 1.5$

Correction Factor for Velocity Distribution $K_2 := 1.0$

Design Velocity $V_{des} := K_1 \cdot K_2 \cdot V_{avg} = 10.74 \cdot \frac{ft}{s}$

Median Riprap Stone Diameter $d_{50.pier} := \frac{0.692 \cdot (V_{des})^2}{(S_g - 1) \cdot 2 \cdot g} = 8.51\ in$

$d_{50.pier} = 0.71 \cdot ft$

Riprap Thickness $t_{riprap.pier} := \max(d_{50.pier} \cdot 3.0, 1ft) = 2.13 \cdot ft$

Riprap Extents (HEC-23 DG11.6)

The optimum performance of riprap as a pier scour countermeasure was obtained when the riprap extended a distance of 2 times the pier width in all directions around the pier (Lagasse et al. 2007).

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential cased by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α, which is a function of the width (a) and the length (L) of the pier (or pile bents) and the skew angle α as given below (Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cdot \cos(\alpha) + L \cdot \sin(\alpha)}{a}\right)^{0.65} \qquad (11.3)$$

A filter layer is typically required for riprap at bridge piers. The filter should ***not*** be extended fully beneath the riprap; instead, it should be terminated 2/3 of the distance from the pier to the edge of the riprap.

Pier Width $a = 48\ in$

Length of Pier $L = 43.5 \cdot ft$

Angle of Attack of Flow $\alpha := \theta = 0 \cdot deg$

Adjustment Factor $K_\alpha := \left(\frac{a \cdot \cos(\alpha) + L \cdot \sin(\alpha)}{a}\right)^{0.65} = 1$

Extent of Riprap Layer at Pier $L_{riprap.pier} := 2 \cdot a \cdot K_\alpha = 8 \cdot ft$

Rock Riprap at Bridge Abutments (HEC-23 DG14):

Sizing Rock Riprap at Abutments (HEC-23 DG14.3)

For Froude Numbers < 0.80, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y}\right) \qquad (14.1)$$

Where:
- D_{50} = Median stone diameter, ft
- V = Characteristic average velocity in the contracted section
- S_s = Specific gravity of rock riprap
- g = Gravitational acceleration
- y = Depth of flow in the contracted bridge opening
- K = 0.89 for a spill-through abutment
  1.02 for a vertical wall abutment

For Froude Numbers > 0.80, Equation 14.2 is recommended:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y}\right)^{0.14} \qquad (14.2)$$

Where: K = 0.61 for a spill-through abutment
0.69 for a vertical wall abutment

In both equations, the coefficient K is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure.

If the Set Back Ratio (SBR) of the abutment (Set-back length/average channel flow depth) is less than 5.0, compute characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the roadway.

Flowrate Through Bridge Section $Q_{br} := 148800 \frac{ft^3}{s}$

Flow Area Through Bridge Section $A_{br} := 21814.86 ft^2$

Characteristic Average Velocity in Contracted Section $V := \frac{Q_{br}}{A_{br}} = 6.82 \cdot \frac{ft}{s}$

Depth of Flow in Contracted Bridge Section $y_{br} := 25.28 ft$

Froude Number in Bridge Section $Fr_{br} := \frac{V}{\sqrt{g \cdot y_{br}}} = 0.24$

Specific Gravity of Rock Riprap $S_s := 2.75$

Coefficient K (Check Froude Number and Abutment Type) $K := 0.89$

Sizing Rock Riprap at Abutments, cont.

Median Riprap Stone Diameter

$$D_{50.br} := \begin{cases} y_{br} \cdot \left[\frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y_{br}} \right) \right] & \text{if } Fr_{br} \le 0.8 \\ \left[y_{br} \cdot \left[\frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y_{br}} \right)^{0.14} \right] \right] & \text{otherwise} \end{cases} = 8.83\,\text{in}$$

$$D_{50.br} = 0.74 \cdot ft$$

Riprap Thickness

$$t_{riprap.br} := \max(D_{50.br} \cdot 1.5, 1ft) = 1.1 \cdot ft$$

Riprap Extents

The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment but need not exceed 25ft.

$$Apron_{br} := \min(2 \cdot y_{br}, 25ft) = 25 \cdot ft$$

Spill-through abutment slopes should be protected with the rock riprap size computed from Equations 14.1 or 14.2 to an elevation 2ft above expected high water elevation for the design flood.

Design Flood Elevation $Elev_{Q.100} := 149.16ft$

Elevation of Top of Riprap $Elev_{riprap.br} := Elev_{Q.100} + 2ft = 151.16 \cdot ft$

T·Y·LIN INTERNATIONAL
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PROJECT:	Howland	**JOB NO:**	411697.00
TASK:	Scour Analysis of Proposed Structure		
DESIGN:	DSM	**DATE:**	2/4/2013
CHECK:	RMH	**DATE:**	6/3/13

Howland Alt 5: Scour Analysis - Q_{500}

References:

1. *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges, 5th Edition*
2. *Hydraulic Engineering Circular No. 23: Bridge Scour and Stream Instability Countermeasures, 3rd Edition*

Methodology:

Check each type of scour in turn, using HEC-18 equations.

1. *Contraction scour*
   - *Clear Water Scour*
   - *Live Bed Scour*
2. *Abutment Scour*
3. *Simple Pier Scour*
4. *Complex Pier Scour*

Design revetment using HEC-23 equations

Assumptions:

- *Checking Q500 with this sheet*
- *Upstream Section is River Sta 10050 in HEC-RAS*
- *D50 is 5 mm for top 2-4' of bed material, 0.8 mm below that*
- *D84 is 7mm for Complex Pier Scour*
- *For pier scour, use Easternmost pier, in deepest water*

Legend:

Text in this font is a direct quotation of the HEC manual

Text in this font is this sheet's descriptions and discussion

Equations with this formatting are manual inputs

Equations with this formatting are important results

Contraction Scour (HEC-18 6):

Critical Velocity Calculation (HEC-18 6.2.1)

To calculate the critical velocity above which bed material of size D and smaller will be transported, use the following equation:

$$V_c = K_u \cdot y^{\frac{1}{6}} \cdot D^{\frac{1}{3}} \qquad \text{(Equation 6.1)}$$

Where:
- V_c = Critical velocity above which bed material of size D and smaller will be transported
- y = Average depth of flow upstream of the bridge
- D = Particle size for V_c
- D_{50} = Particle size in a mixture of which 50 percent are smaller
- K_u = 11.17 for English units

D_{50} of Bed Material $D_{50} := 5mm$

Average Depth of Flow Upstream for Given Condition $y := 28.40ft$

Coefficient for Critical Velocity Equation $K_u := 11.17$

Critical Velocity Calculation

$$V_c := K_u \cdot \left(\frac{y}{ft}\right)^{\left(\frac{1}{6}\right)} \cdot \left(\frac{D_{50}}{ft}\right)^{\left(\frac{1}{3}\right)} \cdot \frac{ft}{s} = 4.96 \cdot \frac{ft}{s}$$

Live Bed Contraction Scour (HEC-18 6.3)

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\left(\frac{6}{7}\right)} \cdot \left(\frac{W_1}{W_2}\right)^{k_1}$$

$$y_s = y_2 - y_0$$

Where:
- y_1 = Average depth in the upstream main channel
- y_2 = Average depth in the contracted section
- y_0 = Existing depth in the contracted section before scour
- y_s = Average scour depth
- Q_1 = Flow in the upstream channel transporting sediment, not including overbank flows
- Q_2 = Flow in the contracted channel
- W_1 = Bottom width of the upstream main channel that is transporting bed material
- W_2 = Bottom width of the main channel in the contracted section less pier width(s)
- k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

- V_* = $(\tau_0/\rho)^{1/2} = (gy_1S_1)^{1/2}$, shear velocity in the upstream section
- ω = Fall velocity of bed material based on the D_{50} (Figure 6.8)
- g = Acceleration of gravity
- S_1 = Slope of energy grade line of main channel

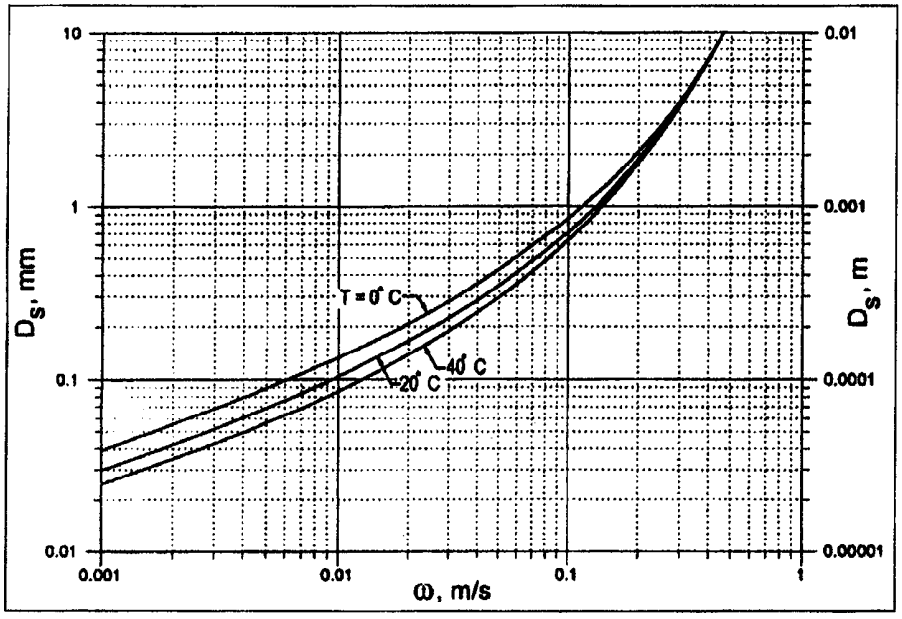

Figure 6.8

Live Bed Contraction Scour, cont.

D_{50} of Bed Material $D_{50} = 5 \cdot mm$

Fall velocity of bed material based on the D_{50} (From Figure 6.8, above) $\omega := 0.34 \frac{m}{s} = 1.12 \cdot \frac{ft}{s}$

Acceleration of Gravity $g = 32.17 \cdot \frac{ft}{s^2}$

Average Depth in Upstream Main Channel $y_1 := 28.40ft$

Slope of Energy Grade Line of Main Channel $S_1 := .00037$

Shear Velocity in Upstream Section $V_{star} := \sqrt{g \cdot y_1 \cdot S_1} = 0.58 \cdot \frac{ft}{s}$

Live Bed Contraction Scour Exponent

$$k_1 := \begin{cases} 0.59 & \text{if } (V_{star} \div \omega) < 0.50 \\ 0.64 & \text{if } (V_{star} \div \omega) \geq 0.50 \wedge (V_{star} \div \omega) \leq 2.0 \\ 0.69 & \text{if } (V_{star} \div \omega) > 2.0 \end{cases} = 0.64$$

Flow in the Upstream Main Channel Transporting Sediment $Q_1 := 183090.1 \frac{ft^3}{s}$

Flow in the Contracted Channel $Q_2 := 185815.1 \frac{ft^3}{s}$

Bottom Width of the Upstream Main Channel $W_1 := 850ft$

Bottom Width of the Main Channel in the Contracted Section $W_2 := 838ft - 3 \cdot 4ft = 826 \cdot ft$

Average Depth in the Contracted Section

$$y_{2.live} := y_1 \cdot \left[\left(\frac{Q_2}{Q_1} \right)^{\left(\frac{6}{7}\right)} \cdot \left(\frac{W_1}{W_2} \right)^{k_1} \right] = 29.29 \cdot ft$$

Existing Depth in the Contracted Section (Before Scour) $y_0 := 28.93ft$

Live-Bed Contraction Scour Depth $y_{s.live} := y_{2.live} - y_0 = 0.36 \cdot ft$

Clear-Water Contraction Scour (HEC-18 6.4)

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (presented in the Appendix C)

$$y_2 = \left[\frac{K_u \cdot Q^2}{D_m^{\left(\frac{2}{3}\right)} \cdot W^2}\right]^{\left(\frac{3}{7}\right)}$$

$$y_s = y_2 - y_0$$

Where:
- y_2 = Average equilibrium depth in the contracted section after contraction scour
- y_0 = Average existing depth in the contracted section
- y_s = Average contraction scour depth
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W
- D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section
- D_{50} = Median diameter of bed material
- W = Bottom width of the contracted section less pier width(s)
- K_u = 0.025 for SI units, 0.0077 for English units

Coefficient for Calculating Clear-Water Contraction Scour $K_u := 0.0077$

Discharge Through the Bridge $Q := Q_2 = 185815.1 \cdot \frac{ft^3}{s}$

D_{50} of Bed Material $D_{50} = 0.20 \cdot in$

Diameter of Smallest Nontransportable Particle in Bed Material $D_m := 1.25 \cdot D_{50} = 0.25\ in$

Bottom Width of the Main Channel in the Contracted Section $W := W_2 = 826 \cdot ft$

Average equilibrium depth in the contracted section after contraction scour

$$y_{2.Clr} := \left[\frac{K_u \cdot \left(Q \div \frac{ft^3}{s}\right)^2}{\left(D_m \div ft\right)^{\left(\frac{2}{3}\right)} \cdot (W \div ft)^2}\right]^{\left(\frac{3}{7}\right)} \cdot ft = 39.14 \cdot ft$$

Existing Depth in the Contracted Section (Before Scour) $y_0 = 28.93 \cdot ft$

Clear-Water Contraction Scour Depth $y_{s.Clr} := y_{2.Clr} - y_0 = 10.21 \cdot ft$

Contraction Scour Summary

Upstream Section Velocity in Channel $V_{us} := 7.59 \frac{ft}{s}$

Critical Velocity Above Which Bed Material Will be Transported $V_c = 4.96 \cdot \frac{ft}{s}$

Contraction Scour Type

$$Contraction_{Scour.Type} := \begin{cases} \text{"Live-Bed"} & \text{if } V_{us} \geq V_c \\ \text{"Clear-Water"} & \text{otherwise} \end{cases}$$

$$\boxed{Contraction_{Scour.Type} = \text{"Live-Bed"}}$$

Contraction Scour Depth

$$y_{s.contraction} := \begin{cases} y_{s.live} & \text{if } V_{us} \geq V_c \\ y_{s.Clr} & \text{otherwise} \end{cases}$$

$$\boxed{y_{s.contraction} = 0.36 \cdot ft}$$

Local Scour at Abutments (HEC-18 8):

NCHRP 24-20 Abutment Scour Approach (HEC-18 8.6.3)

The three scour conditions illustrated in Figure 8.7 are (a) scour occurring when the abutment is in or close to the main channel, (b) scour occurring when the abutment is set back from the main channel, and (c) scour occurring when the embankment breaches and the abutment foundation acts as a pier. As illustrated in Figure 8.8, the NCHRP study also concluded that there is a limiting depth of abutment scour when the geotechnical stability of the embankment or channel bank is reached. The abutment scour computed from the NCHRP approach is total scour at the abutment; it is not added to contraction scour because it already includes contraction scour. The advantages of using the NCHRP abutment scour equations include (1) not using the effective embankment length, L', which is difficult to determine in many situations, (2) the equations are more physically representative of the abutment scour process, and (3) the equations predict total scour at the abutment rather than the abutment scour component that is then added to contraction scour. The scour equations for conditions (a) and (b) are:

$$y_{max} = \alpha_A \cdot y_c \quad \text{or} \quad y_{max} = \alpha_B \cdot y_c \qquad (8.3)$$

$$y_s = y_{max} - y_0 \qquad (8.4)$$

Where:
- y_{max} = Maximum flow depth resulting from abutment scour, ft (m)
- y_c = Flow depth including live-bed or clear-water contraction scour, ft (m)
- α_A = Amplification factor for live-bed conditions
- α_B = Amplification factor for clear-water conditions
- y_s = Abutment scour depth, ft (m)
- y_0 = Flow depth prior to scour, ft (m)

Based on the NCHRP (2010b) study, if the projected length of the embankment, L, is 75 percent or greater than the width of the floodplain (B_f), scour condition (a) in Figure 8.7 occurs and the **contraction scour calculation is performed using a live-bed scour calculation**. The contraction scour equation is a simplified version of the live-bed contraction scour equation (see Chapter 6). The equation combines the discharge and width ratios due to the similarity of the exponents because other uncertainties are more significant. By combining the discharge and width, the live-bed contraction scour equation simplifies to the ratio of two unit discharges. Unit discharge (q) can be estimated either by discharge divided by width or by the product of velocity and depth. The contraction scour equation is:

$$y_c = y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\left(\frac{6}{7}\right)} \qquad (8.5)$$

Where:
- y_c = Flow depth including live-bed contraction scour, ft (m)
- y_1 = Upstream flow depth, ft (m)
- q_1 = Upstream unit discharge, ft^2/s (m^2/s)
- q_{2c} = Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s (m^2/s)

NCHRP 24-20 Abutment Scour Approach, cont.

The value of q_{2c} can be estimated as the total discharge in the bridge opening divided by the width of the bridge opening. The value of y_c is then used in Equation 8.3 to compute the total flow depth at the abutment. The value of α_A is selected from Figure 8.9 for spill through abutments and Figure 8.10 for wingwall abutments. The solid curves should be used for design. The dashed curves represent theoretical conditions that have yet to be proven experimentally. For low values of q_2/q_1, contraction scour is small, but the amplification factor is large because flow separation and turbulence dominate the abutment scour process. For large values of q_2/q_1, contraction scour dominates the abutment scour process and the amplification factor is small.

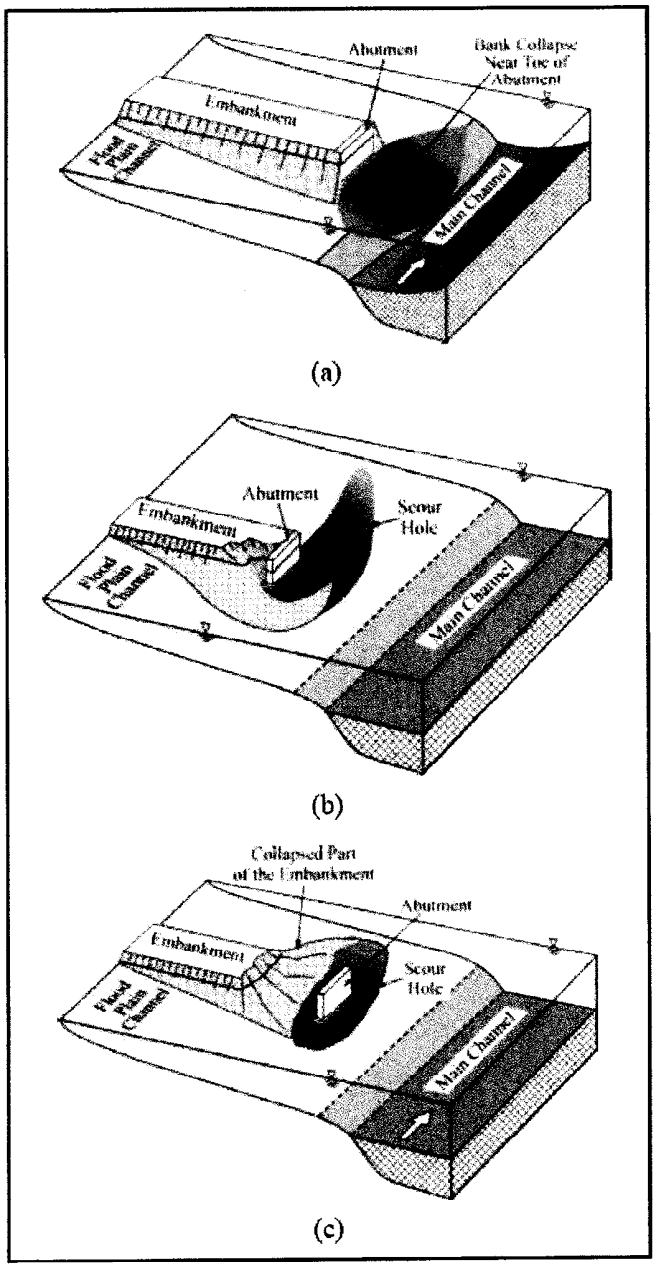

Figure 8.7. Abutment scour conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

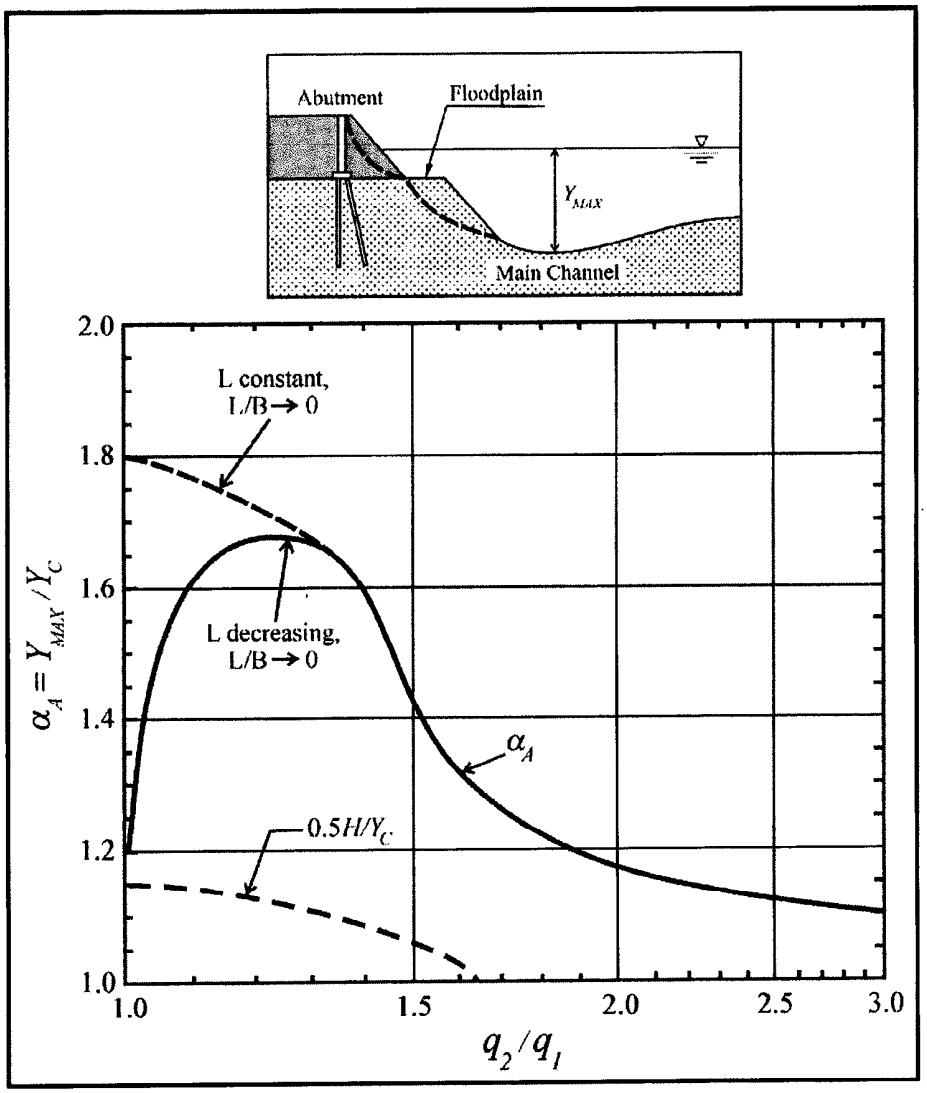

Figure 8.9. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

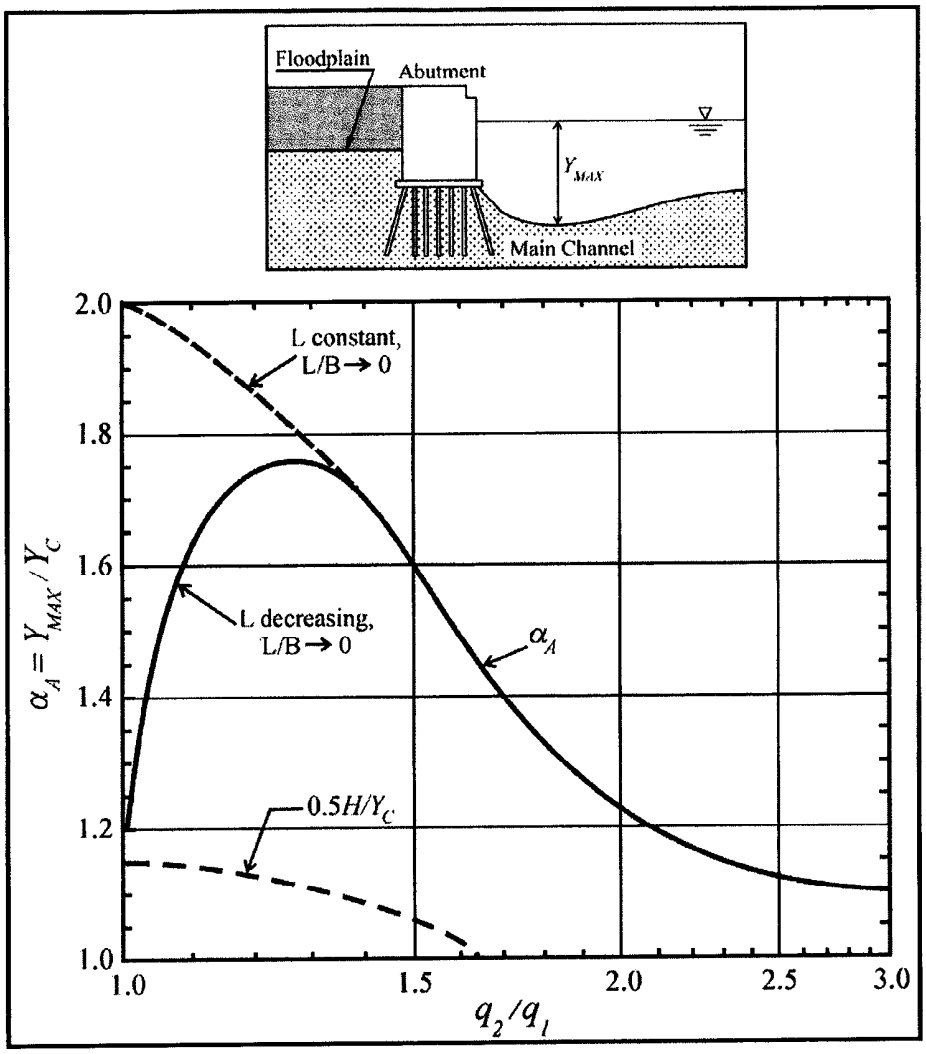

Figure 8.10. Scour amplification factor for wingwall abutments and live-bed conditions (NCHRP 2010b).

NCHRP 24-20 Abutment Scour Approach, cont.

NOTE: this analysis only includes the equations for live-bed abutment scour, since that is what is occuring for this abutment case.

Total Upstream Channel Flow $Q_1 = 183090.1 \cdot \frac{ft^3}{s}$

Flow through bridge opening $Q_{2c} := Q_2 = 185815.1 \cdot \frac{ft^3}{s}$

Width of the Upstream Main Channel $W_1 = 850 \cdot ft$

Width of flow through Bridge Opening $W_{2c} := W_2 = 826 \cdot ft$

Upstream unit discharge $q_1 := \frac{Q_1}{W_1} = 215.4 \cdot \frac{ft^2}{s}$

Unit discharge in the constricted opening accounting for non-uniform flow distribution $q_{2c} := \frac{Q_{2c}}{W_{2c}} = 224.96 \cdot \frac{ft^2}{s}$

Ratio of q_2 to q_1 $\frac{q_{2c}}{q_1} = 1.04$

Amplification factor for live-bed conditions $\alpha_A := 1.46$ *From chart above*

Upstream flow depth $y_1 = 28.4 \cdot ft$

Flow depth including live-bed contraction scour $y_{c.abut} := y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\left(\frac{6}{7}\right)} = 29.48 \cdot ft$

Maximum flow depth resulting from abutment scour $y_{max} := y_{c.abut} \cdot \alpha_A = 43.04 \cdot ft$

Existing Depth in the Contracted Section (Before Scour) $y_0 = 28.93 \cdot ft$

Abutment Scour Depth $y_{s.abut} := y_{max} - y_0 = 14.11 \cdot ft$

Local Scour at Piers (HEC-18 7):

Local Pier Scour Equation (HEC-18 7.2)

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water scour. The equation predicts maximum pier scour depths. The equation is:

$$\frac{y_s}{y_1} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_1}\right)^{0.65} \cdot Fr_1^{0.43} \qquad (7.1)$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$$y_s \le 2.4 \cdot a \quad \text{if } Fr \le 0.8$$
$$y_s \le 3.0 \cdot a \quad \text{if } Fr > 0.8 \qquad (7.2)$$

Where:
- y_s = Scour depth
- y_1 = Flow depth directly upstream of the pier
- K_1 = Correction factor for pier nose shape from Figure 7.3 and Table 7.1
- K_2 = Correction factor for angle of attack of flow from Table 7.2 or Equation 7.4
- K_3 = Correction factor for bed condition from Table 7.3
- a = Pier width
- L = Length of pier
- Fr_1 = Froude Number directly upstream of the pier = $\frac{V_1}{\sqrt{g \cdot y_1}}$
- V_1 = Mean velocity of flow directly upstream of the pier
- g = Acceleration of gravity

The correction factor, K_2, for angle of attack of flow, θ, is calculated using the following equation:

$$K_2 = \left(\cos(\theta) + \frac{L}{a \cdot \sin(\theta)}\right)^{0.65} \qquad (7.4)$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 7.4 and Table 7.2.

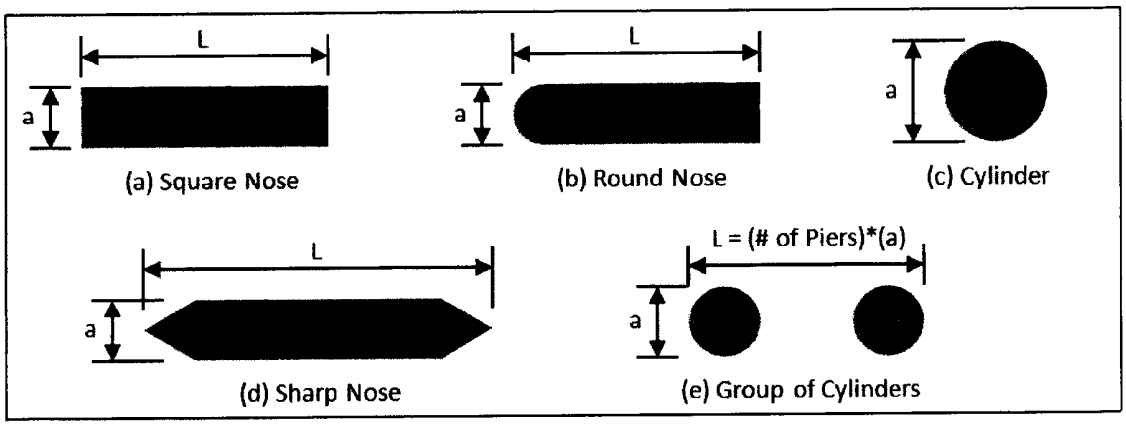

Figure 7.3

Local Pier Scour, cont.

Table 7.1. Correction Factor, K_1, for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.2. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier			

Table 7.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.		
Bed Condition	Dune Height ft	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

Pier Width $a := 4ft$

Length of Pier $L := 43.5ft$

Length to Width Ratio (for chart) $\frac{L}{a} = 10.88$

Flow Depth Directly Upstream of Pier $y_{1.pier} := 31.38ft$ *Section STA 9935*

Angle of Attack of Flow $\theta := 0deg$

Mean Velocity of Flow Directly Upstream of Pier $V_1 := 7.81 \frac{ft}{s}$ *Section STA 9935*

Correction Factor for Pier Nose Shape $K_1 := 0.9$

Correction Factor for Angle of Attack of Flow $K_2 := \left(\cos(\theta) + \frac{L}{a} \cdot \sin(\theta)\right)^{0.65} = 1$

Correction Factor for Bed Condition $K_3 := 1.1$

Local Pier Scour, cont.

Froude Number Directly Upstream of Pier

$$Fr_1 := \frac{V_1}{\sqrt{g \cdot y_{1.pier}}} = 0.25$$

Ratio of Scour Depth to Flow Depth

$$Ratio_{y.s_y.1} := 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_1}\right)^{0.65} \cdot Fr_1^{0.43} = 0.3$$

Max Scour Depth

$$y_{s.pier.MAX} := if\left(Fr_1 \le 0.8, 2.4 \cdot a, 3.0 \cdot a\right) = 9.6 \cdot ft$$

Pier Scour Depth

$$y_{s.pier} := min\left(y_{s.pier.MAX}, Ratio_{y.s_y.1} \cdot y_{1.pier}\right) = 9.5 \cdot ft$$

This is below the top of the pile cap (which is located 3 feet below grade) so complex pier scour must be evaluated

Scour for Complex Pier Foundations (HEC-18 7.5)

The components of a complex pier are illustrated in Figure 7.5 (Jones and Sheppard 2000). This is followed by a definition of the variables. Note that the pile cap can be above the water surface, at the water surface, in the water or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, the methods of analysis given in this manual may give smaller or larger scour depths.

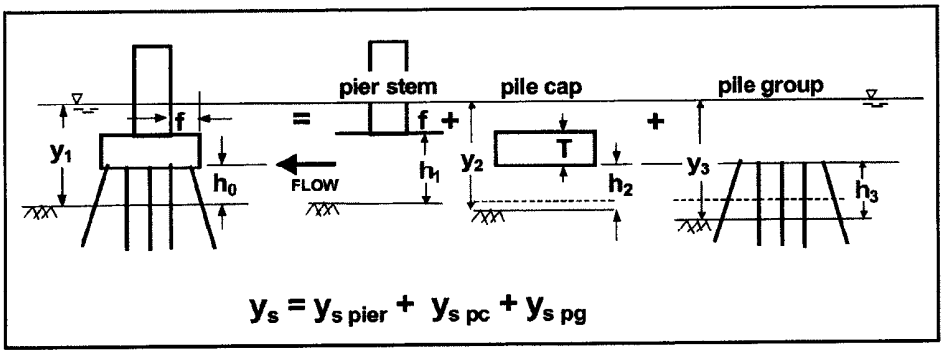

Figure 7.5. Definition sketch for scour components for a complex pier.

Scour for Complex Pier Foundations, cont.

The variables illustrated in Figure 7.5 and others used in computations are as follows:

f = Distance between front edge of pile cap or footing and pier, ft (m)
h_o = Height of the pile cap above bed at beginning of computation, ft (m)
$h_1 = h_o + T$ = height of the pier stem above the bed before scour, ft (m)
$h_2 = h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, ft (m)
$h_3 = h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after the pier stem and pile cap scour components have been computed, ft (m)
S = Spacing between columns of piles, pile center to pile center, ft (m)
T = Thickness of pile cap or footing, ft (m)
y_1 = Approach flow depth at the beginning of computations, ft (m)
$y_2 = y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations ft (m)
$y_3 = y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, ft (m)
V_1 = Approach velocity used at the beginning of computations, ft/sec (m/sec)
$V_2 = V_1(y_1/y_2)$ = adjusted velocity for pile cap computations, ft/sec (m/sec)
$V_3 = V_1(y_1/y_3)$ = adjusted velocity for pile group computations, ft/sec (m/sec)

Preliminary Inputs:

Distance between front edge of pile cap or footing and pier	$f := 7.5ft$	*Conservatively combine seal and footing and using seal dimensions*
Height of the pile cap above bed at beginning of computation	$h_0 := -8ft$	
Thickness of pile cap	$T := 5ft$	
Height of the pier stem above the bed before PIER scour	$h_1 := h_0 + T + y_{s.contraction} = -2.64 \cdot ft$	
Approach flow depth at the beginning of computations	$y_{1.pier} = 31.38 \cdot ft$	
Approach velocity used at the beginning of computations	$V_1 = 7.81 \cdot \frac{ft}{s}$	

Scour for Complex Pier Foundations, cont.

Total scour from superposition of components is given by:

$$y_s = y_{s_pier} + y_{s_pc} + y_{s_pg} \qquad (7.22)$$

where:

y_s = Total scour depth, ft (m)
$y_{s\ pier}$ = Scour component for the pier stem in the flow, ft (m)
$y_{s\ pc}$ = Scour component for the pier cap or footing in the flow, ft (m)
$y_{s\ pg}$ = Scour component for the piles exposed to the flow, ft (m)

Each of the scour components is computed from the basic pier scour Equation 7.1 using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 7.5, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections guidance for calculating each of the components is given.

Determination of the Pier Stem Scour Depth component (HEC-18 7.5.3)

The first computation is the scour estimate, $y_{s\ pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation (Equation 7.1). In Equation 7.1, a_{pier} is the pier width and other variables in the equation are as defined previously. This base scour estimate is multiplied by $K_{h\ pier}$, given in Figure 7.6 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component as follows:

$$\frac{y_{s_pier}}{y_1} = K_{h_pier}\left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a_{pier}}{y_1}\right)^{0.65} \cdot \left(\frac{V_1}{\sqrt{g \cdot y_{1.pier}}}\right)^{0.43}\right] \qquad (7.23)$$

where:

$K_{h\ pier}$ = Coefficient to account for height of pier stem above bed and shielding effect by pile cap overhang distance "f" in front of pier stem (from Figure 7.6)

Pier Width	$a_{pier} := a = 4 \cdot ft$	
Length of Pier	$L = 43.5 \cdot ft$	
Flow Depth Directly Upstream of Pier	$y_{1.pier} = 31.38 \cdot ft$	*Section STA 9935*
Angle of Attack of Flow	$\theta = 0 \cdot deg$	
Mean Velocity of Flow Directly Upstream of Pier	$V_1 = 7.81 \cdot \frac{ft}{s}$	*Section STA 9935*
Correction Factor for Pier Nose Shape	$K_1 = 0.9$	
Correction Factor for Angle of Attack of Flow	$K_2 = 1$	
Correction Factor for Bed Condition	$K_3 = 1.1$	

Pier Stem Scour Depth component, cont.

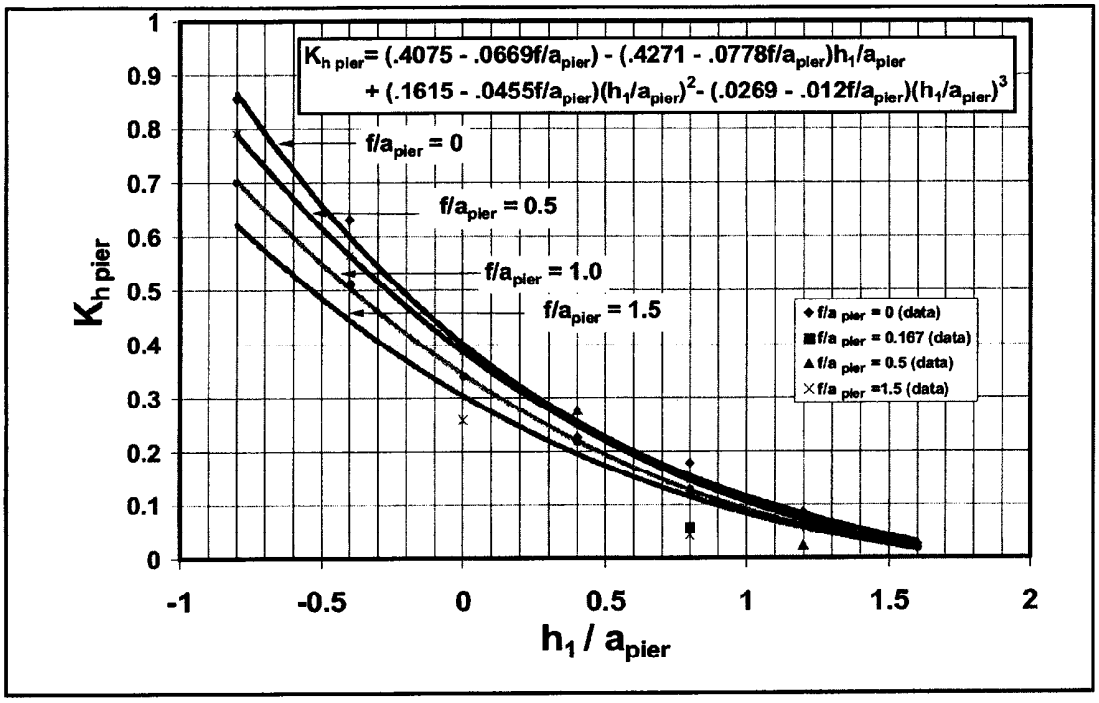

Figure 7.6. Suspended pier scour ratio (Jones and Sheppard 2000).

Pile Cap Overhang Distance $f = 7.5 \cdot ft$

Ratio f/a_{pier} $\frac{f}{a_{pier}} = 1.88$ *For use in above chart*

Ratio h_1/a_{pier} $\frac{h_1}{a_{pier}} = -0.66$

Coefficient to account for height of pier stem above bed and shielding effect by pile cap overhang distance "f" in front of pier stem (From Figure 7.6) $K_{h_pier} := 0.56$

Ratio of Scour Depth to Flow Depth

$$Ratio_{y.s_pier_y.1} := K_{h_pier} \cdot \left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a_{pier}}{y_{1.pier}} \right)^{0.65} \cdot \left(\frac{V_1}{\sqrt{g \cdot y_{1.pier}}} \right)^{0.43} \right] = 0.16$$

Pier Stem Scour Depth $y_{s.pier} := Ratio_{y.s_pier_y.1} \cdot y_{1.pier} = 4.99 \cdot ft$

Determination of the Pile Cap (Footing) Scour Depth Component (HEC-18 7.5.4)

Note: only Case 2 is represented here, where the tremie seal acts as a very tall footing, and the piles are never exposed to flow. For exposed piles, please refer to HEC 18 Section 7.5.4

Case 2. Bottom of the Pile Cap (Footing) Located On or Below the Bed.

One limitation of the procedure described above [Case 1] is that the design chart in Figure 7.7 has not been developed for the case of the bottom of the pile cap or footing being below the bed (i.e., negative values of h2). In this case, use a modification of the exposed footing procedure that has been described in previous editions of HEC-18. The previous procedure was developed from experiments in which the footing was never undermined by scour and tended to be an over predictor if the footing is undermined.

As for Case 1:

$$y_2 = y_1 + \frac{y_{s_pier}}{2}$$

$$V_2 = V_1 \cdot \left(\frac{y_1}{y_2}\right)$$

The average velocity of flow at the exposed footing (V_f) is determined using the following equation:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93 \cdot \frac{y_f}{k_s} + 1\right)}{\ln\left(10.93 \cdot \frac{y_2}{k_s} + 1\right)} \qquad (7.25)$$

where:

V_f = Average velocity in the flow zone below the top of the footing, ft/s (m/s)
V_2 = Average adjusted velocity in vertical of flow approaching the pier, ft/s (m/s)
ln = Natural log to the base e
$y_f = h_1 + y_{s\ pier}/2$ = distance from the bed (after degradation, contraction scour, and pier stem scour) to the top of the footing, ft (m)
k_s = Grain roughness of the bed (normally taken as the D_{84} for sand size bed material and 3.5 D_{84} for gravel and coarser bed material), ft (m)
y_2 = Adjusted depth of flow upstream of the pier, including degradation, contraction scour and half the pier stem scour, ft (m)

See Figure 7.8 for an illustration of variables.

Compute the pile cap scour depth component, $y_{s\ pc}$ from Equation 7.1 using the full pile cap width, a_{pc}, y_f, V_f as the width, flow depth, and velocity parameters, respectively. The wide pier factor K_w in Section 7.4 should be used in this computation if (1) the total depth $y_2 < 0.8\ a_{pc}$, (2) the Froude Number $V_2/(gy_2)^{1/2} < 1$, and (3) $a_{pc} > 50\ D_{50}$. Use y_2/a_{pc} to compute the K_w factor if it is applicable. The scour component equation for the case 2 pile cap or footing can then be written:

$$\frac{y_{spc}}{y_f} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_w \cdot \left(\frac{a_{pc}}{y_f}\right)^{0.65} \cdot \left(\frac{V_f}{\sqrt{g \cdot y_f}}\right)^{0.43} \qquad (7.26)$$

Determination of the Pile Cap (Footing) Scour Depth Component, cont.

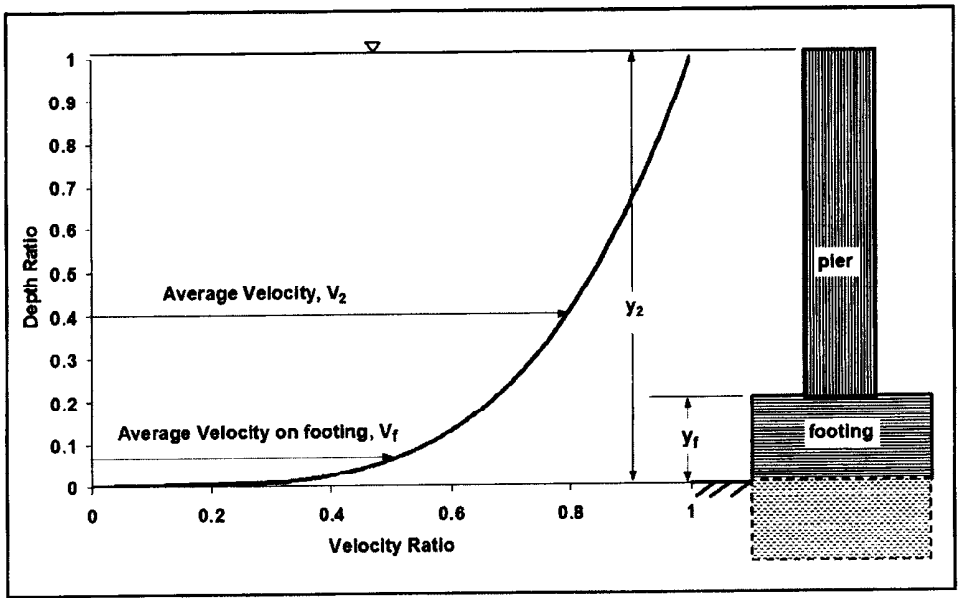

Figure 7.8. Definition sketch for velocity and depth on exposed footing.

Pier Scour at Wide Piers (Section 7.4)

$$K_w = 2.58 \cdot \left(\frac{y}{a}\right)^{0.34} \cdot Fr_1^{\,0.65} \quad \text{for} \quad \frac{V}{V_c} < 1 \qquad (7.20)$$

$$K_w = 1.0 \cdot \left(\frac{y}{a}\right)^{0.13} \cdot Fr_1^{\,0.25} \quad \text{for} \quad \frac{V}{V_c} \geq 1 \qquad (7.21)$$

Engineering judgment should be used in applying K_w because it is based on limited data from flume experiments. Engineering judgment should take into consideration the volume of traffic, the importance of the highway, cost of a failure (potential loss of lives and dollars) and the change in cost that would occur if the K_w factor is used.

Correction Factor for Pier Nose Shape $K_1 := 1.1$ *Footing is rectangular*

Correction Factor for Angle of Attack of Flow $K_2 = 1$

Correction Factor for Bed Condition $K_3 = 1.1$

Adjusted Flow Depth $y_{2.pier} := y_{1.pier} + \frac{y_{s.pier}}{2} = 33.87 \cdot ft$

Adjusted Flow Velocity $V_2 := V_1 \cdot \left(\frac{y_{1.pier}}{y_{2.pier}}\right) = 7.23 \cdot \frac{ft}{s}$

Average Distance from the bed to top of footing: $y_f := h_1 + \frac{y_{s.pier}}{2} = -0.14 \cdot ft$ *No Footing Scour!!*

Determination of the Pile Cap (Footing) Scour Depth Component, cont.

Width of footing $a_{pc} := 23.5\text{ft}$ *(Conservatively assume equal to tremie seal width below)*

Use K_w Factor?

$$\text{UseK}_w := \text{if}\left[\left[\left(y_{2.pier} < 0.8\cdot a_{pc}\right) \wedge \left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}} < 1\right) \wedge \left(a_{pc} > 50\cdot D_{50}\right)\right], \text{"Yes"}, \text{"No"}\right]$$

$$\text{UseK}_w = \text{"No"}$$

Wide Pier Correction Factor

$$K_w := \begin{cases} 1 & \text{if } \text{UseK}_w = \text{"No"} \\ \left[2.58\cdot\left(\frac{y_{2.pier}}{a_{pc}}\right)^{0.34}\cdot\left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}}\right)^{0.65}\right] & \text{if } \left(\text{UseK}_w = \text{"Yes"}\right) \wedge \left(\frac{V_2}{V_c} < 1\right) \\ \left[2.58\cdot\left(\frac{y_{2.pier}}{a_{pc}}\right)^{0.34}\cdot\left(\frac{V_2}{\sqrt{g\cdot y_{2.pier}}}\right)^{0.65}\right] & \text{if } \left(\text{UseK}_w = \text{"Yes"}\right) \wedge \left(\frac{V_2}{V_c} \geq 1\right) \end{cases}$$

$$K_w = 1$$

Grain roughness of the bed (Normall D_{84}) $k_s := 7\text{mm}$

Average velocity in the flow zone below the top of footing

$$V_f := V_2\cdot\frac{\ln\left(10.93\cdot\frac{y_f}{k_s} + 1\right)}{\ln\left(10.93\cdot\frac{y_{2.pier}}{k_s} + 1\right)} = (3.13 + 2.35i)\cdot\frac{\text{ft}}{\text{s}}$$

There is no flow below top of footing, so no further scour, and the velocity calc does not make sense.

Ratio of Scour Depth to Adjusted Flow Depth

$$\text{Ratio}_{y.spc_y.f} := 2.0\cdot K_1\cdot K_2\cdot K_3\cdot K_w\cdot\left(\frac{a_{pc}}{y_f}\right)^{0.65}\cdot\left(\frac{V_f}{\sqrt{g\cdot y_f}}\right)^{0.43} = -6.27 + 86.8i$$

Pile Cap (Footing) Scour Depth

$$y_{spc} := \text{if}\left(y_f \leq 0\text{ft}, 0\text{ft}, \text{Ratio}_{y.spc_y.f}\cdot y_f\right) = 0\cdot\text{ft}$$

Total Complex Pier Scour:

Total Complex Pier Scour

$$y_{s.total.pier} := y_{s.pier} + y_{spc} = 4.99\cdot\text{ft}$$

Rock Riprap at Bridge Piers (HEC-23 DG11):

Sizing Rock Riprap at Piers (HEC-23 DG11.3)

To determine the required size of stone for riprap at bridge piers, NCHRP Project 24-23 recommends using the rearranged Isbash equation from the Federal Highway Administration's Hydraulic Engineering Circular No. 23 (Second Edition) (Lagasse et al. 2001) to solve for the median stone diameter:

$$d_{50} = \frac{0.692 \cdot (V_{des})^2}{(S_g - 1) \cdot 2 \cdot g} \qquad (11.1)$$

Where:
- d_{50} = Particle size for which 50% is finer by weight
- V_{des} = Design velocity for local conditions at the pier
- S_g = Specific gravity of rock riprap
- g = Acceleration due to gravity

It is important that the velocity used in Equation 1.1 is representative of conditions in the immediate vicinity of the bridge pier including the constriction caused by the bridge. If the cross-section or channel average velocity, V_{avg}, is used, then it must be multiplied by factors that are a function of the shape of the pier and its location in the channel:

$$V_{des} = K_1 \cdot K_2 \cdot V_{avg} \qquad (11.2)$$

If a velocity distribution is available from stream tube or flow distribution output of a 1-D model or directly from a 2-D model, then only the pier shape coefficient should be used. The maximum velocity in the active channel Vmax is often used since the channel could shift and the highest velocity could impact any pier.

$$V_{des} = K_1 \cdot V_{max} \qquad (11.3)$$

Where:
- V_{des} = Local velocity at pier
- K_1 = Shape factor equal to 1.5 for round-nose piers or 1.7 for square-faced piers
- K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)
- V_{avg} = Channel average velocity at the bridge
- V_{max} = Maximum velocity in the active channel

Sizing Rock Riprap at Piers, cont.

Specific Gravity of Riprap $S_g := 2.75$

Mean Velocity of Flow Directly Upstream of Pier $V_{avg} := V_1 = 7.81 \cdot \frac{ft}{s}$ *Section STA 9935*

Correction Factor for Pier Nose Shape $K_1 := 1.5$

Correction Factor for Velocity Distribution $K_2 := 1.0$

Design Velocity $V_{des} := K_1 \cdot K_2 \cdot V_{avg} = 11.72 \cdot \frac{ft}{s}$

Median Riprap Stone Diameter $d_{50.pier} := \frac{0.692 \cdot (V_{des})^2}{(S_g - 1) \cdot 2 \cdot g} = 10.12\,in$

$d_{50.pier} = 0.84 \cdot ft$

Riprap Thickness $t_{riprap.pier} := \max(d_{50.pier} \cdot 3.0, 1ft) = 2.53 \cdot ft$

Riprap Extents (HEC-23 DG11.6)

The optimum performance of riprap as a pier scour countermeasure was obtained when the riprap extended a distance of 2 times the pier width in all directions around the pier (Lagasse et al. 2007).

In the case of wall piers or pile bents consisting of multiple columns where the axis of the structure is skewed to the flow direction, the lateral extent of the protection should be increased in proportion to the additional scour potential cased by the skew. While there is no definitive guidance for pier scour countermeasures, it is recommended that the extent of the armor layer should be multiplied by a factor K_α, which is a function of the width (a) and the length (L) of the pier (or pile bents) and the skew angle α as given below (Richardson and Davis 2001):

$$K_\alpha = \left(\frac{a \cdot \cos(\alpha) + L \cdot \sin(\alpha)}{a}\right)^{0.65} \qquad (11.3)$$

A filter layer is typically required for riprap at bridge piers. The filter should ***not*** be extended fully beneath the riprap; instead, it should be terminated 2/3 of the distance from the pier to the edge of the riprap.

Pier Width $a = 48\,in$

Length of Pier $L = 43.5 \cdot ft$

Angle of Attack of Flow $\alpha := \theta = 0 \cdot deg$

Adjustment Factor $K_\alpha := \left(\frac{a \cdot \cos(\alpha) + L \cdot \sin(\alpha)}{a}\right)^{0.65} = 1$

Extent of Riprap Layer at Pier $L_{riprap.pier} := 2 \cdot a \cdot K_\alpha = 8 \cdot ft$

Rock Riprap at Bridge Abutments (HEC-23 DG14):

Sizing Rock Riprap at Abutments (HEC-23 DG14.3)

For Froude Numbers < 0.80, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y}\right) \qquad (14.1)$$

Where:
- D_{50} = Median stone diameter, ft
- V = Characteristic average velocity in the contracted section
- S_s = Specific gravity of rock riprap
- g = Gravitational acceleration
- y = Depth of flow in the contracted bridge opening
- K = 0.89 for a spill-through abutment
  1.02 for a vertical wall abutment

For Froude Numbers > 0.80, Equation 14.2 is recommended:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y}\right)^{0.14} \qquad (14.2)$$

Where:
- K = 0.61 for a spill-through abutment
  0.69 for a vertical wall abutment

In both equations, the coefficient K is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure.

If the Set Back Ratio (SBR) of the abutment (Set-back length/average channel flow depth) is less than 5.0, compute characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the roadway.

Flowrate Through Bridge Section $Q_{br} := 186800 \frac{ft^3}{s}$

Flow Area Through Bridge Section $A_{br} := 24663.06 ft^2$

Characteristic Average Velocity in Contracted Section $V := \frac{Q_{br}}{A_{br}} = 7.57 \cdot \frac{ft}{s}$

Depth of Flow in Contracted Bridge Section $y_{br} := 28.93 ft$

Froude Number in Bridge Section $Fr_{br} := \frac{V}{\sqrt{g \cdot y_{br}}} = 0.25$

Specific Gravity of Rock Riprap $S_s := 2.75$

Coefficient K (Check Froude Number and Abutment Type) $K := 0.89$

Sizing Rock Riprap at Abutments, cont.

Median Riprap Stone Diameter

$$D_{50.br} := \begin{cases} y_{br} \cdot \left[\frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y_{br}} \right) \right] & \text{if } Fr_{br} \le 0.8 \\ \left[y_{br} \cdot \left[\frac{K}{(S_s - 1)} \cdot \left(\frac{V^2}{g \cdot y_{br}} \right)^{0.14} \right] \right] & \text{otherwise} \end{cases} = 10.88\,in$$

$$D_{50.br} = 0.91 \cdot ft$$

Riprap Thickness

$$t_{riprap.br} := \max(D_{50.br} \cdot 1.5, 1ft) = 1.36 \cdot ft$$

Riprap Extents

The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment but need not exceed 25ft.

$$Apron_{br} := \min(2 \cdot y_{br}, 25ft) = 25 \cdot ft$$

Spill-through abutment slopes should be protected with the rock riprap size computed from Equations 14.1 or 14.2 to an elevation 2ft above expected high water elevation for the design flood.

Design Flood Elevation $Elev_{Q.100} := 149.16ft$

Elevation of Top of Riprap $Elev_{riprap.br} := Elev_{Q.100} + 2ft = 151.16 \cdot ft$