

HYDROLOGY REPORT

The Pearce Brook Bridge Number 6 (#5019) over Pearce Brook carries Green Street in the town of Houlton in Aroostook County, Maine. The Pearce Brook watershed extends to the south and southeast from the bridge location. The watershed extends for approximately 4.3 miles south for a total area of 9.1 square miles. Pearce Brook ends at its confluence with Meduxnekeag River 0.35 miles downstream of the bridge.

The 2008 flood in northern Maine was a Q47 event for the Meduxnekeag River, which Pearce Brook is part of its basin. The water surface elevation was 341.11 foot (NAVD) at the USGS gage# 1018000 located on the Meduxnekeag River south of Houlton.

Flood records for the town of Houlton were found in a Flood Insurance Study published by Federal Emergency Management Agency in 1992. The FIS stated that notable on the Meduxnekeag River floods were recorded during 1954, 1958, 1961, 1973, 1976, and 1979.

An abutter provided well-documented records for floods on Pearce Brook. According to these records notable on the Pearce Brook floods were recorded during February 1970, February 2005, January 2006, December 2010, March 2011, and June 2012. These records from the abutter and the FIS overlap for 21 years, but not a single flood recorded at both locations at the same time. Based on the records from an abutter and the FIS the Pearce Brook floods don't seem to correlate with floods of the Meduxnekeag River.

According to the abutter's records, ice jams amplify flooding issues at the bridge location. The records mention that in the March 2011 flood, chunks of ice were blocking the opening of the crossing and describe the water to be high and fast. After that flood, a slab of ice was found on the flood plain measuring up to 4' by 6'. The ice jams occurrence could be a result of a combination of the small opening and the poor alignment of the existing bridge.

The drainage basin characteristics for this bridge were provided by the Maine Department of Transportation Environmental Office, Hydrology Section. Peak flows were calculated with techniques described in the United States Geological Survey Regression Equations (Hodgkins, 1999 & Lombard/Hodgkins, 2015). In the table below are the discharges for the respective storm provided by the MaineDOT Environmental Office along with discharges described in the 1992 FIS. The values provided by the FIS are more conservative, so they were used in hydraulic design.

SUMMARY

	USGS	FEMA	
Drainage Area	9.1	8.0	mi ²
Q1.1	150		ft ³ /s
Q10	650	740	ft ³ /s
Q25	850		ft ³ /s
Q50	1000	1180	ft ³ /s
Q100	1180	1410	ft ³ /s
Q500	1610	1830	ft ³ /s

Reported by: Shkara, Ahmed R
Date: May 24, 2018

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.



HYDRAULIC REPORT

The Flood Insurance Study from 1992 showed a profile of Pearce Brook with Pearce Brook #6 Bridge, along with structures upstream and downstream. In the profile, a relatively small water surface drop indicates a relatively small flow constriction. The profile showed that there is relatively small water surface drop at Pearce Brook #6, while a much greater water surface drops at each of the structures upstream and downstream, which means that Pearce Brook #6 is not the main flood issue at the bridge site and the water is backing up from the upstream and downstream crossings. While the bridges up stream were replaced since the study and they may not present an obstruction any more, the structures downstream are still in place as they were at the time of the study, so Pearce Brook #6 can be modeled as tailwater controlled. The water elevations downstream of the bridge can be assumed to have stayed the same for the sake of modeling the hydraulics of the stream.

The floods recorded by the abutter have water elevations approaching 350 ft NAVD at the upstream and might be lower at downstream as at that point the bridge is overtopped. There are no gages on Pearce Brook to determine the discharges for these storm peak flows. These floods are assumed roughly to be around Q10 events based on the high frequency of occurring and the fact that they didn't occur simultaneously with a major flooding event in the general region. These elevations can be compared to the water elevations of Q10 and Q50 resulting from the HEC-Ras model to check the model's accuracy based on this assumption.

The Court Street Bridge is located on Pearce Brook upstream from the Green Street Crossing and it was replaced in the early 1990s. According to the Court Street Bridge PDR from 1992, the previous structure had a span less than 10' and it was replaced by the new frame which has a hydraulic opening of 22' span by 7' clearance from the streambed = 153 sf. According to the PDR referenced above, the Court Street Bridge has similar discharge flows to Pearce Brook #6 Bridge and Court Street Bridge flows 90% full at Q50 (1,000 cfs as calculated then) and slightly more than full at Q100 (1,200 cfs as calculated then). Achieving an equal or a larger hydraulic opening at Pearce Brook #6 would potentially ensure that water would not be backing up at Pearce Brook #6, as it would be at least capable of conveying flows that passed through Court Street Bridge.

Both the existing and the proposed bridges were analyzed using HEC-RAS 5.0.1 (Hydrologic Engineering Center's River Analysis System), a one-dimensional hydraulics software program developed by the United States Army Corps of Engineers. A 213' long portion of Pearce Brook intersecting Pearce Brook Bridge #6 was modeled. Four cross sections upstream and seven cross sections downstream of the bridge were utilized to model the stream. The

stream flows from south to north. The HEC-RAS models were based on the following assumptions:

- Steady flow
- Manning's Roughness Coefficients
 - o Channel Bed: $n = 0.04$
 - o Bridge Abutment: $n = 0.015$
 - o Flood Plain: $n = 0.06$
- Default expansion (0.3) and contraction (0.1) values
- Flow Regime: Subcritical
- Bridge Modeling Approach: Default settings
- Boundary condition: two models used the following two boundary conditions
 - o Bed slope (0.006)
 - o Known water surface elevations from the 1992 FIS
- The structural elements were inputted manually for both the existing bridge and the proposed alternative.
- Assumed areas of inactive flow that occurs during floods were inputted manually

Both the replacement bridge and the existing bridge were modeled twice; first, using the slope of the channel bed as the boundary condition; second, using estimated water surface elevations from the Flood Insurance Study as the boundary condition. The FIS included a water surface profile for Pearce Brook. The profile shows water surface elevations for different peak flow discharges, the streambed, and the existing crossing. The water surface elevations for the corresponding peak flow storms are presented in the table below:

Storm	FEMA Water Surface Elevation (ft) NAVD 88
Q10	347.86
Q50	352.88
Q100	353.37
Q500	353.85

The Q1.1 water surface elevation was assumed to be 341' as there was no record of it found. The assumption was based on adding the estimated bankfull depth to the proposed fill elevation taking into consideration the constriction at the bridge. The MaineDOT Environmental office estimated the bankfull depth to be 1.3', while the proposed special fill elevation is a little over 339.5' and the constriction would slightly increase the elevation, bringing it to approximately 341'.

The two different boundary conditions are needed to predict the bridge's performance in both the current condition and the future condition when structures downstream are improved and don't present a constriction to the brook.

The first model (with channel bed slope as boundary condition) showed that the existing bridge overtops at some flood smaller than Q50 and it presents a substantial constriction. There is a considerable water surface drop at Q10 and higher, but the water surface does not get as high as the Flood Insurance Study shows.

The second model for the existing bridge with the water surface elevations from the Flood Insurance Study show the crossing overtopped with a small water surface drop at Q10 and no water surface drop at Q50, Q100, and Q500 at the existing bridge. In this model, the small water surface drop at Q10 and the absence of water surface drops at the high flow means that the flow is backing up from the constrictions downstream and maybe the confluence with the Meduxnekeag River.

The results from the HEC-RAS model for the existing crossing that was based on the water surface elevations seem to agree with the records from the abutter. The bridge overtops at some storm peak flow of Q10 and larger, which what was the recorded storms are assumed to be.

The performance of the proposed frame was significantly better than the existing bridge; it showed little obstruction when modeled with channel bed slope as boundary condition, and does not overtop. However, the model still is overtopped when modeled with water surface elevations as boundary condition. In the second model (with water surface elevations as boundary condition) the bridge is overtopped at the same storm that the existing bridge is overtopped.

The results of all the models show that while the existing bridge presents a constriction to flow, the flood issue is mainly caused by the structures downstream and/or the confluence with the Meduxnekeag River, which agrees with the assumption based on the FIS water surface elevations. Improving the hydraulics of Pearce Brook Bridge #6 only does not alleviate the flooding issue at storms exceeding Q10 at the bridge location. Improving the hydraulics of downstream bridges is essential.

The results presented in the table below are for the future conditions when the crossings downstream are improved (using the channel bed slope as a boundary condition). In the table below, it can be seen that the velocity of the outlet flow is lower at the existing bridge at high storms, because the water then overtops the existing bridge which provides pressure relief. This does not happen with the proposed bridge, hence the higher flow velocity. If the

downstream conditions didn't control, the proposed structure offers a significant improvement to the hydraulics at the site by lowering the water surface elevation; the crossing would likely not overtop for storms less than a Q100 event.

SUMMARY FOR ANALYSIS USING THE CHANNEL BED SLOPE AS A BOUNDARY CONDITION

		Existing Structure	Recommended Structure
		10' Span X 7' Rise Frame	22' Span X 6.8' Rise Frame
Total Area of Waterway Opening	ft ²	91.0	160.0
Headwater elevation @ Q _{1.1}	ft	342.7	342.2
Headwater elevation @ Q ₁₀	ft	347.4	344.8
Headwater elevation @ Q ₅₀	ft	350.9	346.1
Headwater elevation @ Q ₁₀₀	ft	351.1	346.8
Headwater elevation @ Q ₅₀₀	ft	352.4	348.1
Freeboard @ Q ₅₀	ft	-0.9	3.9
Freeboard @ Q ₁₀₀	ft	-1.1	3.2
Outlet Velocity @ Q _{1.1}	ft/s	4.9	3.3
Outlet Velocity @ Q ₁₀	ft/s	10.8	9.2
Outlet Velocity @ Q ₅₀	ft/s	3.7	11.7
Outlet Velocity @ Q ₁₀₀	ft/s	3.0	12.5
Outlet Velocity @ Q ₅₀₀	ft/s	5.0	13.2

If the downstream conditions remain as a control, the proposed structure offers an improvement to the hydraulics at the site by only lowering the flow velocities, because of the increased hydraulic opening. The crossing will still overtop at high flows at the site as long the downstream condition remains the same. If the downstream conditions are improved in the future, the site will experience lower water surface elevation during high flows (higher than Q10) and thus the flood issue will be mitigated. The results from the model that used water surface elevations as the boundary condition are presented in the table below,

SUMMARY FOR ANALYSIS USING ESTIMATED WATER SURFACE ELEVATIONS AS BOUNDARY CONDITION

		Existing Structure	Recommended Structure
		10' Span X 7' Rise Frame	22' Span X 6.8' Rise Frame
Total Area of Waterway Opening	ft ²	91.0	160.0
Headwater elevation @ Q _{1.1}	ft	343.2	342.2
Headwater elevation @ Q ₁₀	ft	349.8	347.9
Headwater elevation @ Q ₂₅	ft	351.5	351.5
Headwater elevation @ Q ₅₀	ft	352.9	352.9
Headwater elevation @ Q ₁₀₀	ft	353.4	353.4
Headwater elevation @ Q ₅₀₀	ft	353.8	353.8
Freeboard @ Q ₅₀	ft	-2.9	-2.9
Freeboard @ Q ₁₀₀	ft	-3.4	-3.4
Outlet Velocity @ Q _{1.1}	ft/s	3.30	3.30
Outlet Velocity @ Q ₁₀	ft/s	4.10	4.10
Outlet Velocity @ Q ₅₀	ft/s	2.13	2.13
Outlet Velocity @ Q ₁₀₀	ft/s	2.27	2.27
Outlet Velocity @ Q ₅₀₀	ft/s	2.65	2.65

Reported by: Shkara, Ahmed R
Date: May 24, 2018

For more information regarding the output from HEC-RAS, please see Appendix E.

Appendix E

Hydraulics Data

Project Name: Houlton - Pearce Brook
 Stream Name: Pearce Brook
 Bridge Name:
 Route No. Green Street
 Analysis by: CSH

PIN: 21694
 Town: Houlton
 Bridge No. 5019
 USGS Quad:
 Date: 10/6/2016

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

Enter data in blue cells only!

	km ²	mi ²	ac
A	23.57	9.10	5824.0
W	1.18	0.5	291.2

P _c	591085	5104477
County	Aroostook S	
pptA	39.0	
SG	0.00	

A (km ²)	11.93
W (%)	5.00

Conf Lvl 0.67

Enter data in [mi²]
 Watershed Area *DRNAREA*
 Wetlands area (by NWI)

watershed centroid (E, N; UTM 19N; meters)
choose county from drop-down menu
 mean annual precipitation (inches; by look-up)
 sand & gravel aquifer as decimal fraction of watershed A

Worksheet prepared by:

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ver. 2016 Feb 05

References:

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

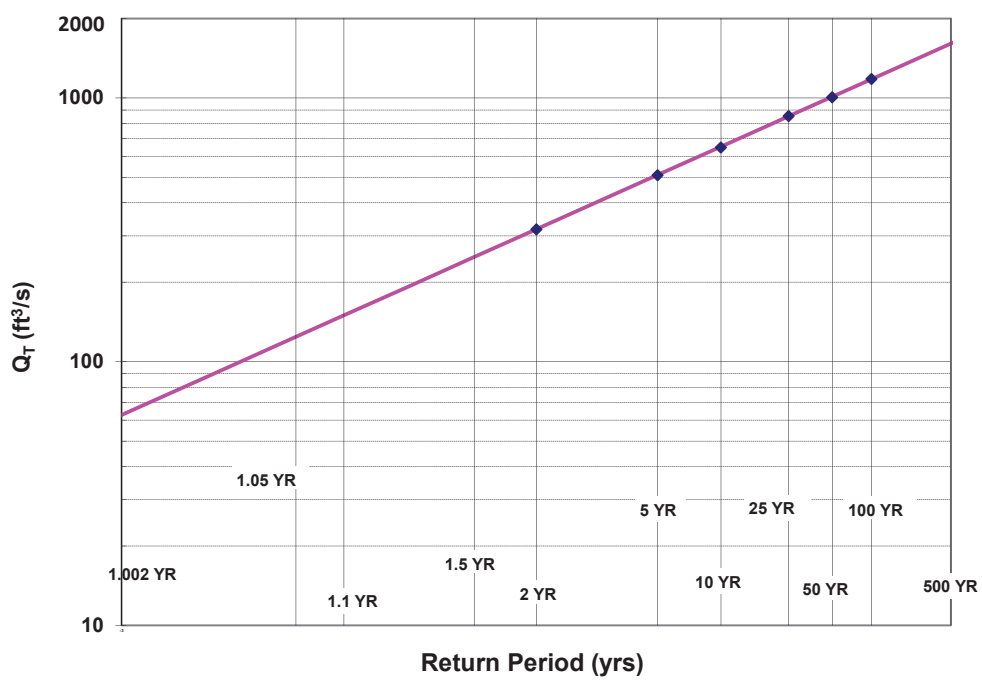
Lombard, P.J. & G.A. Hodgkins, 2015.
 Peak flow regression equations for small, ungaged streams in
 Maine - Comparing map-based to field-based variables
SIR 2015-4059, USGS, Augusta, ME

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

Ret Pd	Peak Flow Estimate		
T (yr)	Lower	Q _T (m ³ /s)	Upper
1.1		4.24	
2		9.01	
5		14.43	
10		18.37	
25		24.16	
50		28.51	
100		33.41	
500		45.58	

Q _T (ft ³ /s)
149.8
318.0
509.5
648.7
853.0
1006.6
1179.8
1609.5

Log-Normal Probability Plot



Project Name:	Houlton - Pearce Brook	PIN:	21694
Stream Name:	Pearce Brook	Town:	Houlton
Bridge Name:	0	Bridge No.	5019
Route No.	Green Street	USGS Quad:	
Analysis by:	CSH	Date:	10/6/2016

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

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

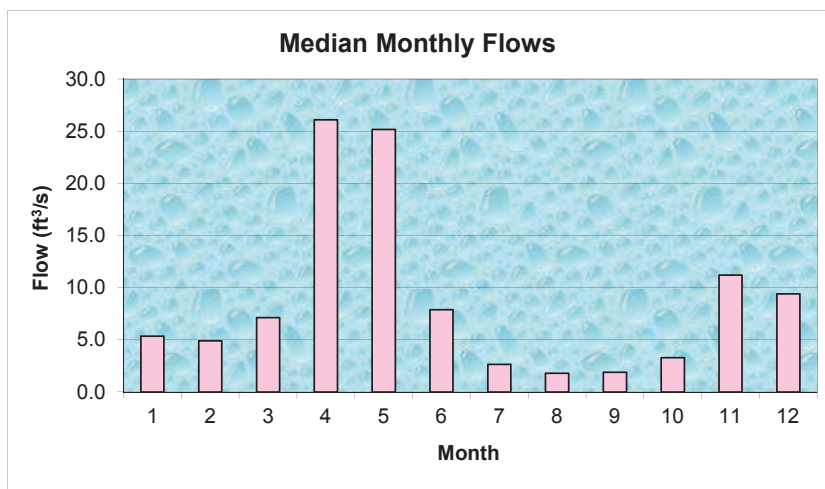
	Value	Variable	Explanation
	9.10	A	Area (mi ²)
591085	5104477	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	118.82	$DIST$	Distance from Coastal reference line (mi)
	39.0	$pptA$	Mean Annual Precipitation (inches)
	0.00	SG	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median}	
	(ft ³ /s)	(m ³ /s)
Jan	5.35	0.1517
Feb	4.91	0.1393
Mar	7.12	0.2019
Apr	26.10	0.7397
May	25.18	0.7135
Jun	7.90	0.2239
Jul	2.66	0.0753
Aug	1.80	0.0511
Sep	1.89	0.0535
Oct	3.29	0.0932
Nov	11.22	0.3179
Dec	9.41	0.2668

Q_{bf}	52.7
ann avg	17.2
ann med	7.9
$Q_{1.002}$	62.8
$Q_{1.01}$	85.5
$Q_{1.05}$	124.1
Q_{bf}	121.7

assume $v = 4$ ft/s

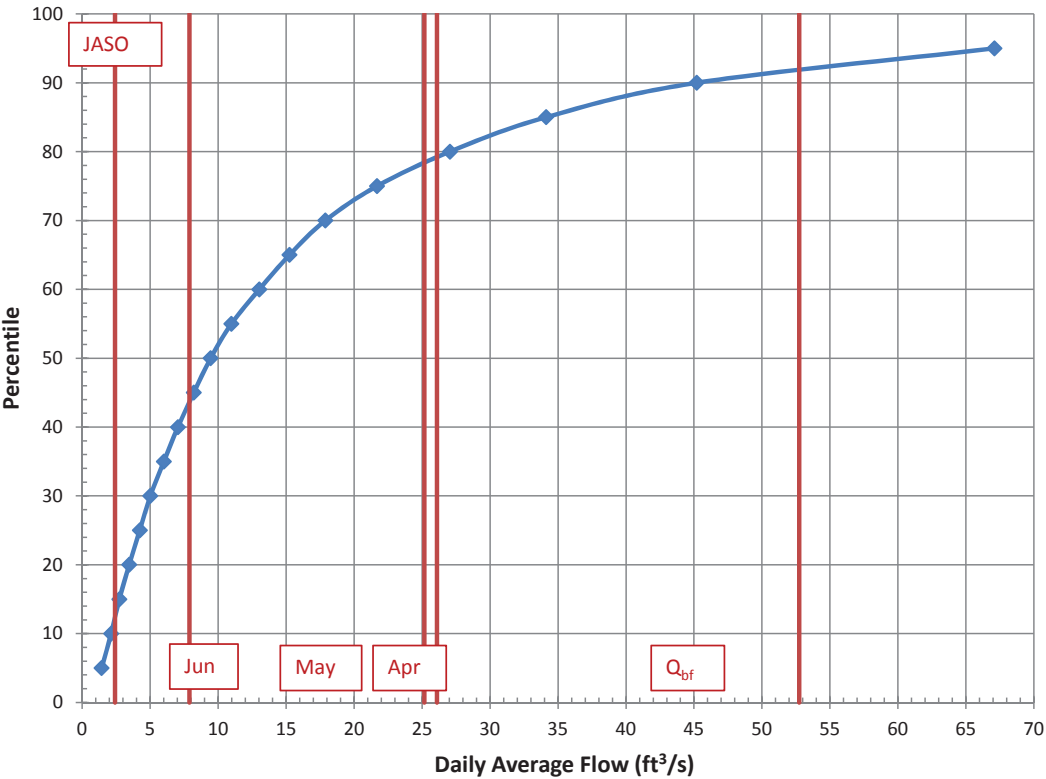
W_{bf}	24.2	estimated bankfull width (ft)
d_{bf}	1.3	estimated bankfull depth (ft)
A_{bf}	30.4	estimated bankfull flow area (ft ²)



References

Dudley, R.W., 2004. Hydraulic Geometry Relations ..., SIR 2004-5042
Dudley, R.W., 2004. Estimating Monthly Streamflows ..., SIR 2004-5026

Daily Average Flow Distribution



Daily Avg Flow Dist

A_{ws} = (mi²) 9.1

Q (ft³/s)

Pctl	Median	84 th pctl
5	1.43	2.31
10	2.13	3.20
15	2.73	3.99
20	3.46	4.84
25	4.24	5.68
30	5.01	6.47
35	6.01	7.39
40	7.04	8.50
45	8.21	9.61
50	9.45	11.35
55	10.98	13.21
60	13.04	15.50
65	15.25	18.06
70	17.89	21.07
75	21.69	25.34
80	27.05	30.25
85	34.13	38.77
90	45.21	52.06
95	67.09	80.96

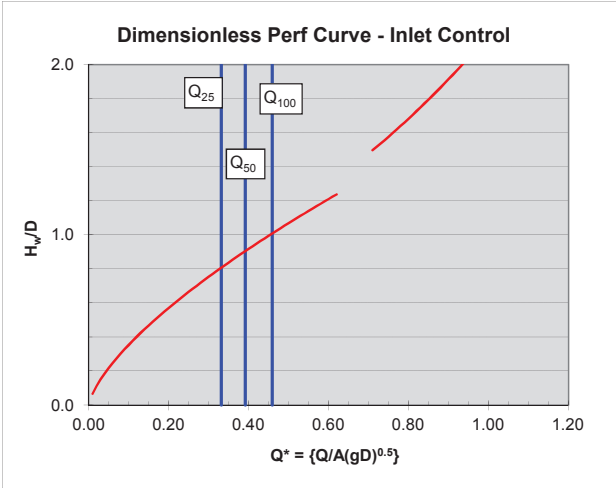
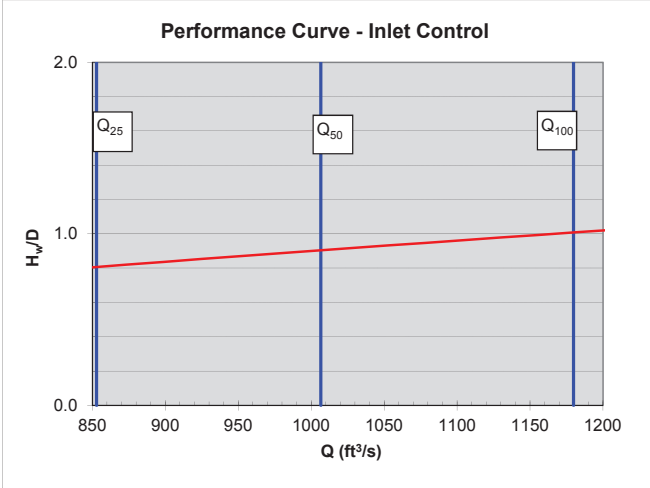
Q _{bf}	52.7
Q _{1.002}	62.8
Q _{1.1}	149.8
Q ₂	318.0

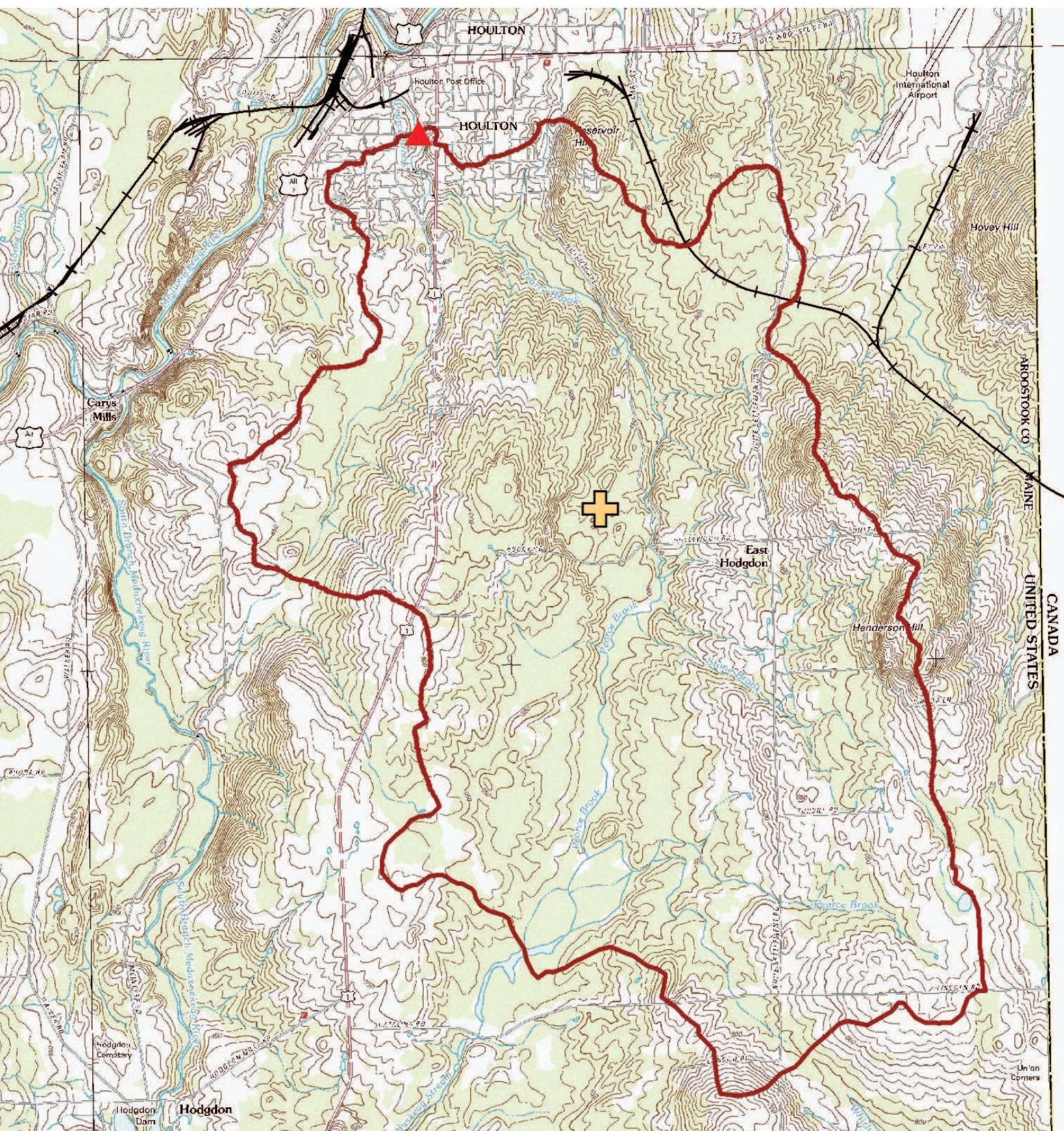
NOTE: This page is for preliminary sizing only.
Final design should be done with HY8 or HDS-5

Preliminary Culvert Sizing - Round & Box Culverts

Shape:	Box			
Type:	Box 0 ww			
D or R (ft)	8	Q ₂₅	853.0	
w (ft)	20 box width	Q ₅₀	1006.6	trial D / R = 13.2
Slope (ft/ft)	0.02	Q ₁₀₀	1179.8	trial w: BFW = 24.2
A (ft ²)	160.00			
g (ft/s ²)	32.2			

Note: culvert dimensions are for open flow area; adjust for lost capacity due to embedding / backfilling (min {2' / 25% rise} embedment)





3.1 Hydrologic Analyses

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

The hydrologic analyses for the Town of Houlton were taken from an SCS Flood Plain Management Study for the Town of Houlton (Reference 6).

Flood discharges on the Meduxnekeag River were computed from a log-Pearson Type III analysis of former USGS gage no. 01018000 near Houlton, which has period of record 1940 to 1982 (References 5 and 7). Discharges at sites in the Town of Houlton 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 0.080.

Discharges on B Stream, Pearce Brook and Brown Brook were generated from the SCS TR-20 hydrologic evaluation model and checked against the USGS regression equation for Maine (References 8 and 9).

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

TABLE 1 - SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
MEDUXNEKEAG RIVER					
At Lowery Road	256.26	7,770	10,570	11,760	14,540
At North Street, U. S. Routes 1 & 2	231.00	7,160	9,730	10,830	13,390
At Carys Mills Road	106.42	3,850	5,230	5,820	7,200
B STREAM					
At County Road, U. S. Route 2	45.49	3,460	5,780	6,930	9,350
At Ludlow Road	44.75	3,410	5,700	6,830	9,220
At B Road	40.92	3,180	5,300	6,340	8,540
PEARCE BROOK					
At Bangor Street	8.00	740	1,180	1,410	1,830
At Court Street	6.99	690	1,110	1,320	1,700
At Hollywood Road	5.91	660	1,010	1,220	1,570

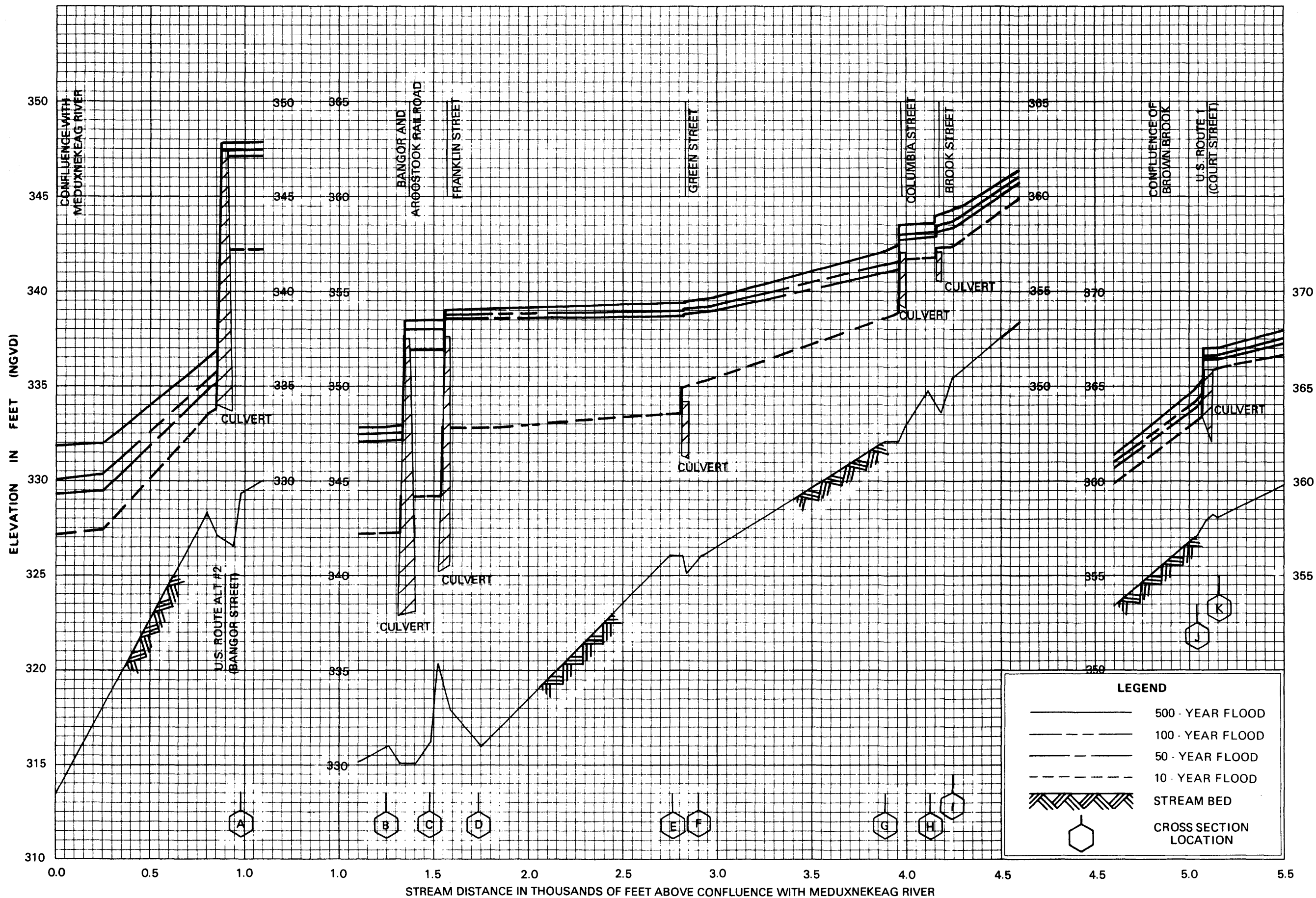
Memo

To: Andrew Lathe
From: Charles Hebson
CC:
Date: 28 October 2016
Re: 21694 Houlton – Pearce Brook – Design Peak Flow Hydrology

Our standard hydrology report is attached, with flows calculated by the USGS regression equations. The US Geological Survey watershed tool StreamStats was unable to delineate the watershed, so we digitized the watershed “manually” in ArcMAP. We checked the result flow values against the 1991 FEMA Flood Insurance Study, as reported in Table 1 below. Our calculated design flows are slightly (16%) less than the values obtained in the FIS, even though our estimated watershed area is somewhat (14%) larger. In order to maintain consistency with the FIS, I recommend we adopt the FIS values.

Table 1. Design Peak Flow Summary (flows in ft^3/s)

Return Period	USGS Regression Pearce Brook @ Green St $A_{ws} = 9.1 \text{ mi}^2$	<i>FEMA FIS (1991)</i> <i>Pearce Brook @ Bangor St</i> $A_{ws} = 8.0 \text{ mi}^2$ <i>(recommended)</i>
1.1	150	
2	320	
5	510	
10	650	740
25	850	
50	1,000	1,180
100	1,180	1,410
500	1,610	1,830



FLOOD PROFILES

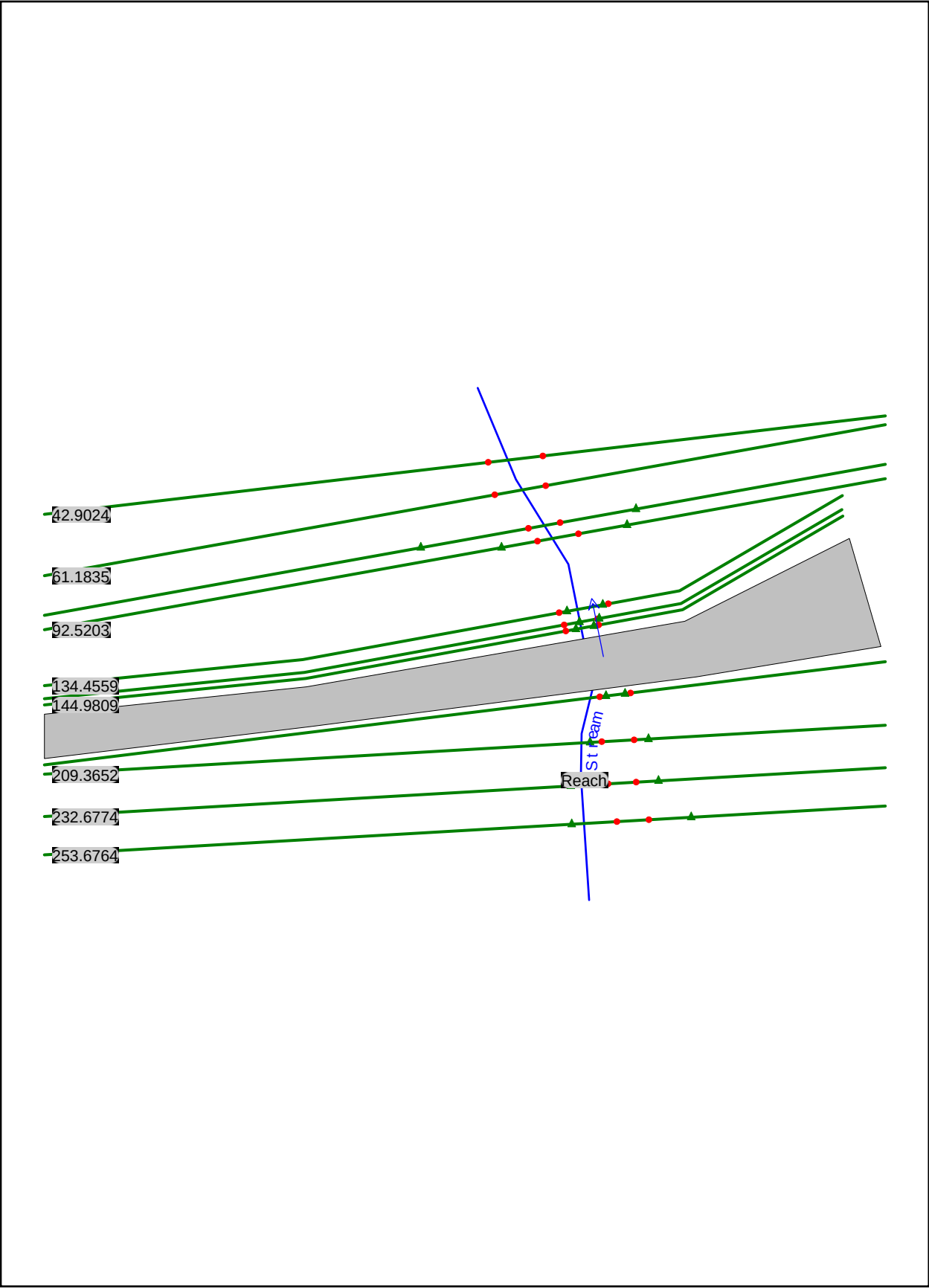
PEARCE BROOK

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF HOULTON, ME

(AROOSTOOK CO.)

HEC-RAS





ROB

RTOB

LTOB

LOB

3+00.05

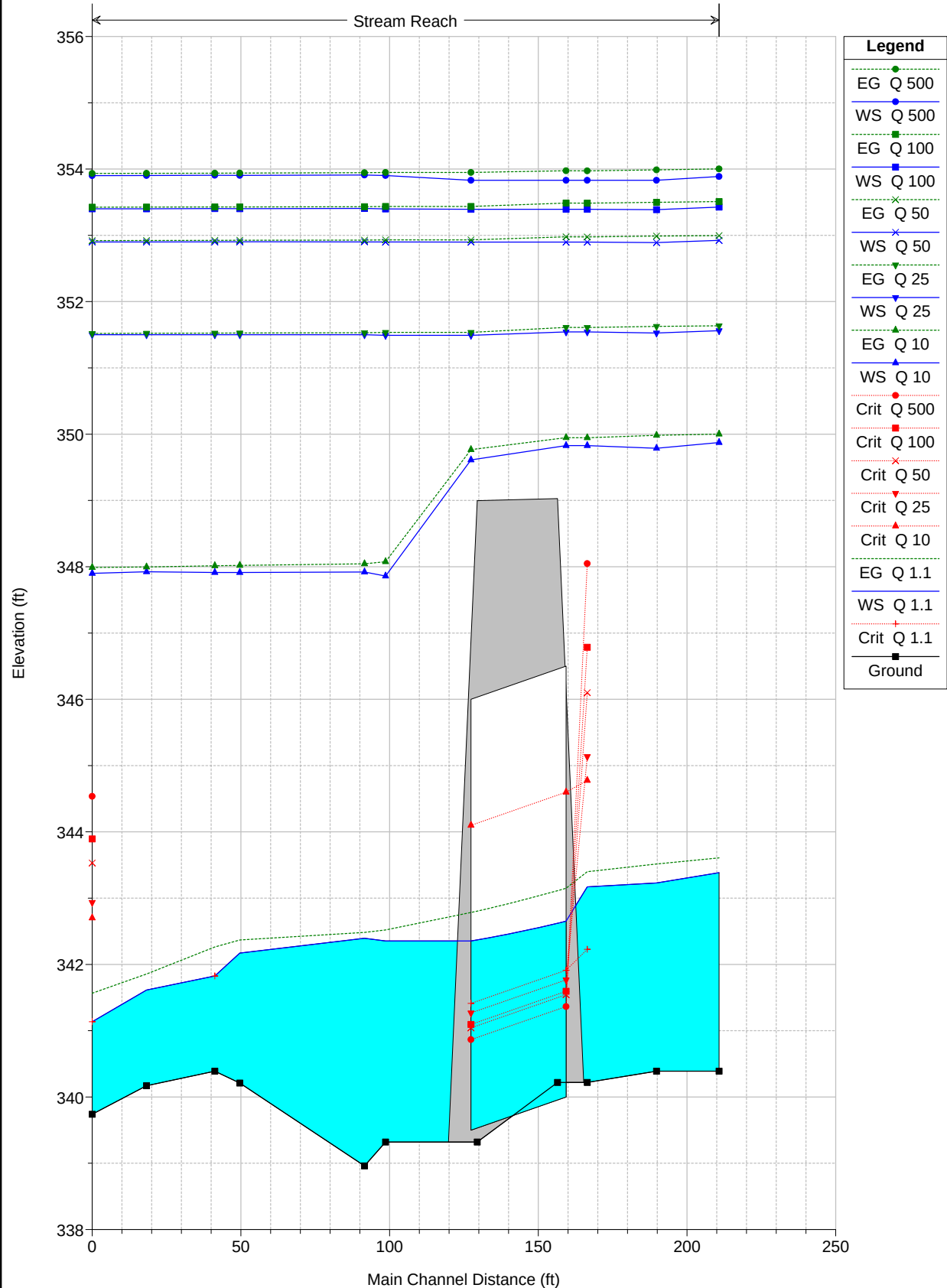
2+00

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Existing Bridge, Profile

TEST 4 Plan: Plan 01 2/6/2019



Propose Frame, Profile

TEST 4 Plan: Plan 01 2/6/2019

