HYDROLOGY REPORT

Spaulding Brook Bridge (BR #2790) carries State Route 17 (Eastern Avenue) over northerly-flowing Spaulding Brook in Augusta, Maine. As seen in Figure 1, approximately 1/10 mile west of the project is Thorne Brook Bridge (BR #2850) over Thorne (Riggs) Brook. Thorne Brook flows north-easterly and reaches a junction with Spaulding Brook located approximately 850-ft downstream from Spaulding Brook Bridge. There is a private road/driveway stream crossing (Sightly Ridge Road) approximately 300-ft downstream from the existing Spaulding Brook structure. Aerial imagery appears to show additional primitive crossings further downstream. The next brook crossing with a public road is approximately 1.25 miles downstream at South Belfast Avenue where the brook passes through two large culverts.

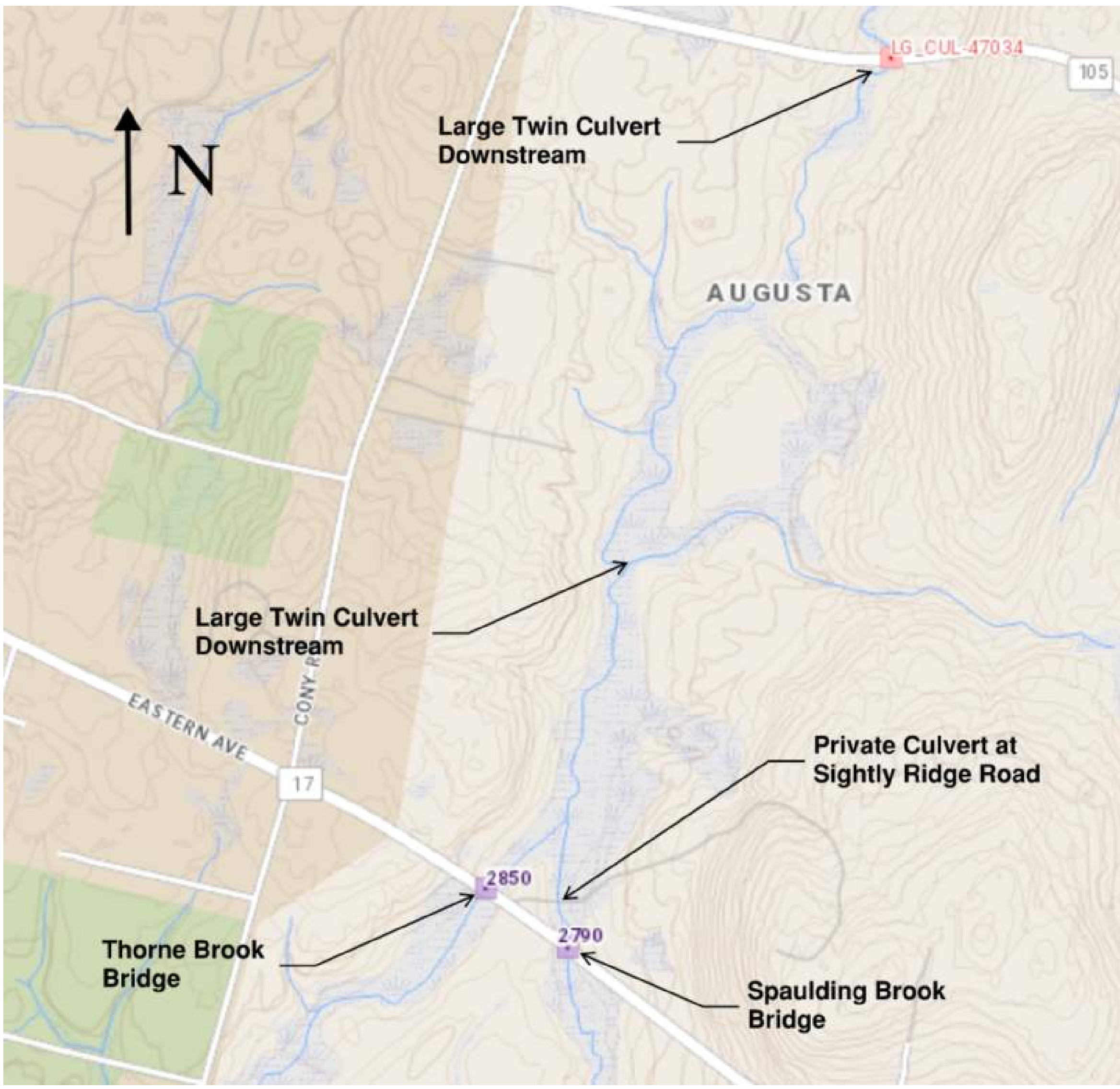


Figure 1 – Site Location

The drainage basin characteristics for Spaulding Brook was provided by the Maine DOT Environmental Group, Hydrology Section. The watershed area for Spaulding Brook at Spaulding Brook Bridge is 1.77 square miles with 0.4 square miles of wetlands. The watershed is rural with private residences/businesses located primarily to the south and east of Spaulding Brook Bridge. Flowrates are shown in Table 1 below.

Table 1: Design flow rates

Return Period (yr.)	Annual Exceedance Probability	Spaulding Brook (ft^3/s)
1.1	91%	20
2	50%	45
5	20%	70
10	10%	85
25	4%	110
50	2%	130
100	1%	150
200	0.5%	175
500	0.2%	210

HYDRAULIC REPORT

FLOOD INFORMATION

The FEMA Flood Insurance Study (FIS) that covers the Town of Augusta, Maine was published June 16, 2011, and is a comprehensive report covering many communities in Kennebec County. FEMA and the Maine Flood Hazard Map indicate that Spaulding Brook Bridge is in a Zone A floodplain, which is a "Special Flood Hazard Area" without Base Flood Elevation (BFE). The FIS states that the Zone A limits were developed for a 1-percent-annual-chance floodplain using approximate methods. See Figure 2.

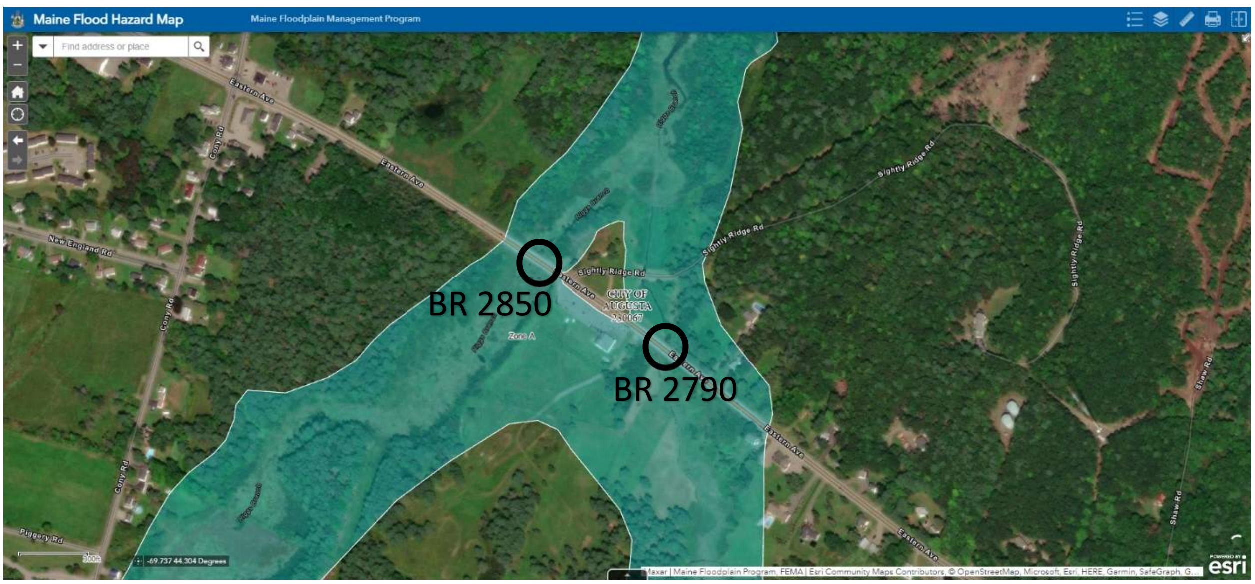


Figure 2: Maine Flood Hazard Map showing Bridge 2790 in Zone A Floodplain

No information from abutters or town officials regarding flooding at the project site has been received. There is no known documentation regarding whether any maintenance has been required in the past to clear obstructions or debris from the existing culvert.

HYDRAULIC MODELING

Hydraulic models for the existing conditions and the proposed sliplining of the culverts were developed using HY-8 Version 8.0.0 (May 29, 2024). The hydraulic models excluded the culverts at the private downstream driveway as well as the junction with Thorne (Riggs) Brook, both located downstream of the project site.

<u>Existing Conditions Model:</u>

The stream channel assumptions for modeling the tailwater included an approximation of the streambed as well as the overbank extents. The streambed and banks were estimated using ground survey data. Due to the relatively small size of the channel, during storm events the brook extends onto large overbank areas on either side of the brook as shown in Figure 3. The width of the overbank area for modeling purposes was estimated based on the surrounding field elevation determined from a combination of sources including Google Earth, contours from the MaineDOT Public Map Viewer, and site photos.

The slope of the channel used for modeling was determined by plotting thalweg points from ground survey over the length of the surveyed reach and applying a best-fit line through the points. A stream slope of 0.03% was determined from the thalweg data.

Manning's Coefficients were developed for the brook channel and overbank areas using Cowan's Method as outlined in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* by George J. Arcement Jr. and Verne R. Schneider. The brook and floodplain contain large amounts of herbaceous vegetation that contribute to higher values for Manning's n. The Manning's coefficients determined for the brook and overbank areas were 0.074 and 0.107 respectively.

The existing steel plate pipe arch culvert was modeled in HY-8 using a 139.4" span and 89.1" rise culvert. These dimensions align with both the existing plans and field measurements. A Manning's "n" of 0.030 was used for the existing culvert.

Existing Conditions Model Calibration

The existing conditions model was calibrated against available data which included:

1. FEMA flood map
2. Hydraulic elevations provided in the 1988 existing bridge plans
3. Photograph during 5/1/2023 storm
4. 12/13/2022 site visit measurement of water surface

The FEMA flood map appears unreliable to use at this location because it shows a low part of Rte. 17 between the Thorne Brook Bridge and the Spaulding Brook Bridge as being above the Zone A floodplain, and it shows higher portions of Rte. 17 near the Thorne Brook Bridge being inundated. This is not consistent with the topography. Therefore, consider other data for model calibration.

Headwater elevations for the 50-year and 100-year storm recurrence intervals are provided in the 1988 bridge plans. The 1988 plans were assumed to use the NGVD 29 datum (since the NAVD 88 datum didn't become an official vertical datum until 1993 per the NOAA National Geodetic Survey). Therefore, the headwater elevations were adjusted to account for a datum shift of -0.70 ft between NGVD 29 and NAVD 88 datums, based on the latitude and longitude of the bridge.

Table 2 – Headwater Elevations Based on 1988 Bridge Plans

Recurrence Interval	1988 Plan Elevation (ft) (NGVD 29 assumed)	Datum Adjusted Headwater Elevation (ft) (NAVD 88)
50 year	179.2	178.5
100 year	179.7	179.0

Figure 3: Downstream of Spaulding Brook Bridge during storm (5/1/2023)

Figure 4: Close-up of posts and rope / chain at private road / driveway during storm (5/1/2023)

The May 1, 2023 storm in central Maine caused significant flooding in the floodplains surrounding the bridge. Figures 3 & 4 show flooding downstream of the bridge with inundation of the private road/driveway and its culvert. Per ground survey, the elevations at the tops of private stream crossing culvert pipes are El 177.65 (west pipe inlet) and 177.68 (east pipe outlet), so the water surface is at an elevation higher than these elevations. Comparing the height of the rope / chain spanning across the private driveway between the posts in Figures 4 & 5, it appears that the depth of water flowing across the driveway is very shallow, maybe a couple of inches.

The elevation of the private road/driveway and thickness of cover over the private driveway culverts shown in Figure 5 are unknown, but the culverts don’t appear to have much cover. Based on ground survey, the culvert pipes are approximately 4-ft in diameter. Since the driveway doesn't appear to be much higher than the tops of the culvert pipes, a minimal cover of 6 inches was assumed. Therefore, the water surface elevation for the May 1 storm is estimated to be around Elevation 178.4 +/-.

Figure 5: Close-up of posts and rope / chain at private road / driveway from Google Streetview

The Figure 3 photo (taken at 7:50am on May 1, 2023) provides an illustration of how widespread the floodwater surface is for a moderate storm event. Precipitation amounts from NOAA Online Weather Data:

1. At Augusta Landfill (1.8 miles from bridge)
    a. 0.15" on 4/30/2023
    b. 4.17" on 5/1/2023
2. At Augusta Airport (3.25 miles from bridge)
    a. 3.00" on 4/30/2023
    b. 1.69" on 5/1/2023

The hydrology report indicates:

- 25-year storm with 5.16” of precipitation in 24 hours.
- 10-year storm with 4.31" in 24 hours.
- 5-year storm with 3.69” in 24 hours.
- 2-year storm with 2.94" in 24 hours.

Therefore, based on the hydrology data, the May 1 storm may correlate to an approximate recurrence interval of approximately Q10, although rainfall in the watershed likely varied from that recorded at the two nearby weather data sites. Therefore, this data can be used to add credibility to the hydraulic model but is assumed to not be accurate enough to allow for precise calibration.

During a 12/13/2022 site visit, measurements were obtained to determine an approximate elevation for the Ordinary High Water Mark (OHWM) as indicated by rust staining on the culvert. The OHWM was estimated to be at Elevation 177.6 +/-.

Existing Conditions Hydraulic Results:

Table 3 – Existing Conditions Results from HY-8

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.16	1.11	2.739	0.37	7-A2t	-1.00	0.77	2.58	3.04	0.80	0.34
Q2	45.00	45.00	177.01	1.71	3.590	0.48	7-A2t	-1.00	1.14	3.39	3.85	1.31	0.38
Q5	70.00	70.00	177.60	2.17	4.176	0.56	7-A2t	-1.00	1.43	3.93	4.39	1.74	0.40
Q10	85.00	85.00	177.89	2.42	4.470	0.60	7-A2t	-1.00	1.58	4.20	4.66	1.97	0.42
Q25	110.00	110.00	178.32	2.80	4.903	0.66	7-A2t	-1.00	1.81	4.58	5.04	2.34	0.44
Q50	130.00	130.00	178.63	3.08	5.214	0.70	7-A2t	-1.00	1.98	4.84	5.30	2.62	0.45
Q100	150.00	150.00	178.92	3.35	5.504	0.74	7-A2t	-1.00	2.14	5.09	5.55	2.89	0.46
Q200	175.00	175.00	179.26	3.66	5.843	0.79	7-A2t	-1.00	2.33	5.36	5.82	3.22	0.48
Q500	210.00	210.00	179.71	4.07	6.288	0.85	7-A2t	-1.00	2.57	5.70	6.16	3.66	0.50
Overtopping	560.00	484.58	183.30	6.79	9.878	1.33	4-FFf	-1.00	4.14	7.42	8.30	7.21	0.67

The results indicate that the culvert is outlet controlled due to tailwater conditions at all evaluated recurrence intervals. The headwater to depth (HW/D) ratios are 0.70 at Q50 and 0.74 at Q100. It also indicates that no overtopping of the roadway is expected for any of the considered recurrence intervals.

The results from the Existing Conditions model match well with the datum adjusted headwater elevations from the 1988 plans:

- Q50 headwater elevation of 178.63 is slightly more than the 1988 headwater elevation of 178.5.
- Q100 headwater elevation of 178.92 is slightly less than the 1988 headwater elevation of 179.0.

The Q10 model result of 177.89 is approximately 0.5 feet lower than the estimated May 1, 2023 flood elevation of 178.4 +/-. The Q25 model result of 178.32 is within 0.1 feet of the estimated flood elevation.

The Q5 model result of 177.60 matches the site visit measured OHWM estimate of 177.6 +/-. OHW is often associated with Q1.1 to Q2 but can be associated with higher elevations. This result may indicate that the actual tailwater cross section may be more bowl or U-shaped than the linear gain in elevation assumed between the top of bank elevation and the assumed edge of floodplain elevation.

Sensitivity:

The Existing Conditions model is sensitive to many of the design parameters in the model due to tailwater conditions and it is very sensitive to the tailwater slope. A comparison of results from the Existing Conditions model (slope = 0.03%) with a sensitivity test slope of 0.01% indicates that the test slope yields an increase in the Q100 headwater elevation of nearly 1 foot. This sensitivity is expected for tailwater controlled conditions with nearly flat slopes.

The model is also sensitive to Manning's n assumptions for the tailwater. A reduction in the Manning's n from the Existing Conditions model assumptions of 0.074 channel / 0.107 floodplain to sensitivity test values of 0.06 channel / 0.08 floodplain yields a reduction in the Q100 headwater elevation of nearly 0.4 feet.

Based on sensitivity testing for the nearby and similar Thorne Brook Bridge, the model is also sensitive to the tailwater elevations assumed for the channel and floodplain. Since the channel elevations at the Spaulding Brook bridge were developed based on ground survey data, no sensitivity analysis was performed for the tailwater elevations at this bridge.

Proposed Alternatives and Results:

To rehabilitate the existing culvert, two slipline alternatives were modeled to determine the hydraulic effects. Two models were created, one for a 9'-4" x 6'-3" pipe arch, and one for an 8'-10" x 6'-1" pipe arch. The proposed invert elevations of the liners are 6" above the existing culvert invert elevations for both alternatives (the existing culvert and proposed liner are on an adverse slope).

The existing culvert has a hydraulic opening of 67 square feet. The proposed sliplining alternatives have openings of approximately 46 square feet and 43 square feet, respectively. These represent a reduction in area of 31% and 36% respectively.

Table 4 – HY-8 Results for 9'-4" x 6'-3" Sliplining Alternative

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	HW/D	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.20	1.20	2.28	0.37	7-A2t	NA	0.81	2.08	3.04	1.25	0.34
Q2	45.00	45.00	177.10	1.87	3.18	0.51	7-A2t	NA	1.23	2.89	3.85	1.91	0.38
Q5	70.00	70.00	177.73	2.39	3.81	0.61	7-A2t	NA	1.55	3.43	4.39	2.47	0.40
Q10	85.00	85.00	178.05	2.68	4.13	0.66	7-A2t	NA	1.72	3.70	4.66	2.78	0.42
Q25	110.00	110.00	178.54	3.11	4.62	0.74	7-A2t	NA	1.98	4.08	5.04	3.27	0.44
Q50	130.00	130.00	178.90	3.42	4.98	0.80	7-A2t	NA	2.17	4.34	5.30	3.64	0.45
Q100	150.00	150.00	179.24	3.71	5.32	0.85	7-A2t	NA	2.35	4.59	5.55	4.00	0.46
Q200	175.00	175.00	179.65	4.04	5.73	0.92	7-A2t	NA	2.56	4.86	5.82	4.44	0.48
Q500	210.00	210.00	180.22	4.49	6.30	1.01	7-A2t	NA	2.84	5.20	6.16	5.05	0.50

Table 5 – HY-8 Results for 8'-10" x 6'-1" Sliplining Alternative

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	HW/D	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.21	1.20	2.29	0.37	7-A2t	NA	0.77	2.08	3.04	1.26	0.34
Q2	45.00	45.00	177.11	1.89	3.19	0.52	7-A2t	NA	1.20	2.89	3.85	1.97	0.38
Q5	70.00	70.00	177.75	2.44	3.83	0.63	7-A2t	NA	1.53	3.43	4.39	2.55	0.40
Q10	85.00	85.00	178.08	2.74	4.16	0.68	7-A2t	NA	1.70	3.70	4.66	2.88	0.42
Q25	110.00	110.00	178.57	3.20	4.65	0.76	7-A2t	NA	1.97	4.08	5.04	3.39	0.44
Q50	130.00	130.00	178.94	3.51	5.02	0.82	7-A2t	NA	2.17	4.34	5.30	3.78	0.45
Q100	150.00	150.00	179.30	3.80	5.38	0.88	7-A2t	NA	2.35	4.59	5.55	4.16	0.46
Q200	175.00	175.00	179.73	4.15	5.81	0.95	7-A2t	NA	2.58	4.86	5.82	4.63	0.48
Q500	210.00	210.00	180.33	4.63	6.41	1.05	7-A2t	NA	2.86	5.20	6.16	5.28	0.50

The headwater elevations for the proposed alternatives are not significantly higher than the existing conditions elevations, even though the proposed hydraulic openings are significantly smaller. The Q1.1 headwater elevation for the 8'-10" x 6'-1" liner is only 0.05 feet higher than the existing conditions. At Q100, the deviation is greater at 0.38 feet. The relatively small difference in headwater elevations is expected given that the stream is strongly controlled by the tailwater conditions at the site.

The results indicate that both sliplining alternatives do not result in the overtopping of the roadway at Q500 and that outlet control occurs at all design discharges. The outlet velocities of the slipline alternatives do increase. For Q1.1, the design discharge increases 58% from 0.80 ft/s to 1.26 ft/s for the 8'-10" x 6'-1" liner. At Q100, the difference is 44% as the outlet velocity increases from 2.89 ft/s to 4.16 ft/s. These higher velocities may result in the outlet scour hole becoming larger.

Due to the change in the low chord and the increased headwater depths, the headwater to depth ratios of the culverts increase for the sliplining alternatives.

- Q50 HW/D ratio increase from 0.70 for the existing culvert to 0.80 and 0.82 for the 9'-4" x 6'-3" and 8'-10" x 6'-1" liner alternatives, respectively.
- Q100 HW/D ratio increase from 0.74 to 0.85 and 0.88, respectively.

The HW/D ratio for Q50 is below the 0.90 limit indicated in the Bridge Design Guide for culvert-type structures and passes the Q100 without flowing full.

For additional hydraulic information, see Appendix E.

SCOUR ANALYSIS

A scour analysis was not performed on the existing culvert or for the sliplining alternatives. However, the existing plans indicate 2 foot thick plain riprap aprons are located at each end of the existing culvert.

Appendix E

Hydraulics Data

This data was originally prepared for the replacement of the existing culvert with a precast concrete box culvert. Later, the decision was made to rehabilitate the culvert by sliplining it. Sliplining calculations have been appended to the end of these calculations.

WIN:	23126.00		**Project Name:**	
Town:	Augusta		**Stream Name:**	Spaulding Brook
Route No.	ME17 - Eastern Ave.		**Bridge Name:**	Spaulding Brook
Asset ID:	2790		**Analysis by:**	ajs / csh
Lat:	44.30089	**Long:** -69.73559	**Date:**	6/10/2022

Peak Flow Calculations by USGS Regression Equations (Lombard/Hodgkins, 2021; Hodgkins, 1999 & Lombard/Hodgkins, 2015)

Enter data in blue cells only!

ver. 2021 Jan 01

	km^2	mi^2	ac	*Enter data in [mi^2]*
A	4.58	1.77	1132.8	Watershed Area *DRNAREA*
W	0.97	0.4	239.7	Wetlands area (by NWI)
P_c	441460	4903960		watershed centroid (E, N; UTM 19N; meters)
County	Kennebec			*choose county from drop-down menu*

Worksheet prepared by:
Charles S. Hebson, PE
Environmental Office
Maine Dept. Transportation
Augusta, ME 04333-0016
207-557-1052
Charles.Hebson@maine.gov

Watershed Characteristics from StreamStats

STORNWI	21.16	NWI Wetlands %
SANDGRAVF	0.00	sand & gravel aquifer as decimal fraction of watershed A
ELEV	236.3	mean basin elevation (ft)
BSLDEM10M	6.93	mean basin slope (%)
COASTDIST	58.00	distance from the coast (mi)
ELEVMAX	401.9	maximum basin elevation (ft)
LC06WATER	0.55	percent of drainage basin land cover as open water
PRECIP	42.8	mean annual precipitation
STATSGOA	0.52	mean basin percentage of hydrological soil group A

References:

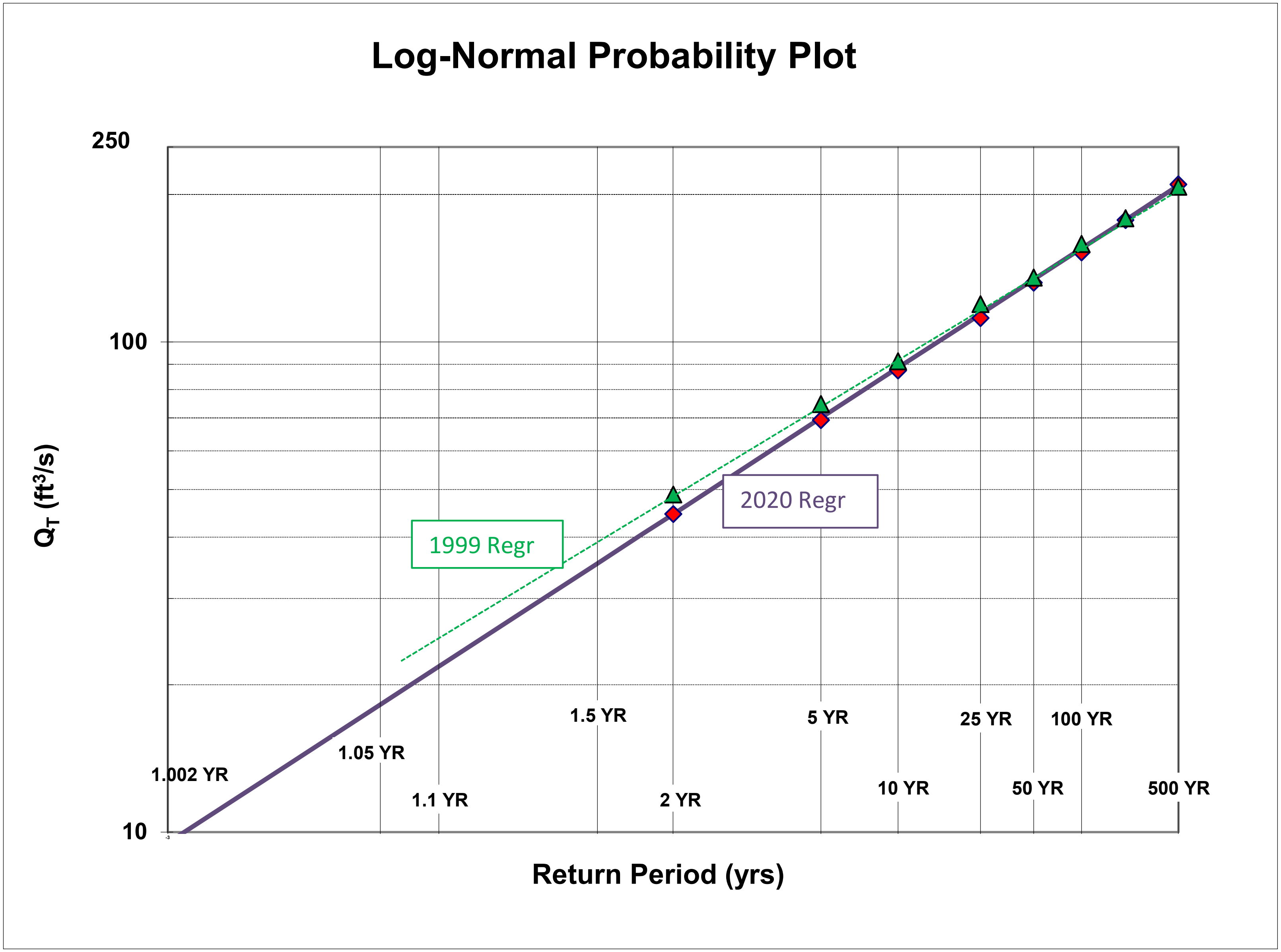
Hodgkins, G.A., 1999.
Estimating the magnitude of peak flows for streams in Maine
for Selected Recurrence Intervals
WRIR 99-4008, USGS Augusta, ME

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

Lombard, P.J. & G.A. Hodgkins, 2021.
Estimating Flood Magnitude and Frequency on Gaged and
Ungaged Streams in Maine
SIR 2020-5092, USGS, Augusta, ME.

Ret Pd T (yr)	**I24**	**Q_T (ft^3/s) *1999 / 2015***	**Q_T (ft^3/s) *2021***	**Q_T (ft^3/s) *Design***
1.1			22	***20***
2	2.94	49	45	***45***
5	3.69	75	69	***70***
10	4.31	91	87	***85***
25	5.16	119	112	***110***
50	5.80	135	132	***130***
100	6.48	158	152	***150***
200	7.26	179	177	***175***
500	8.45	207	210	***210***

Instructions:
Enter values in blue cells only, watershed data from StreamStats
Copy I24 values from Stream Stats
Use results under "Design"
Check against gage data and FEMA studies if available
Questions? Check with ENV / Hydrology Section



Log-Normal Probability Plot
250
100
10
Q_T (ft³/s)
1999 Regr
2020 Regr
1.002 YR
1.05 YR
1.1 YR
1.5 YR
2 YR
5 YR
10 YR
25 YR
50 YR
100 YR
500 YR
Return Period (yrs)

WIN:	23126.00	**Project Name:**	0
Town:	Augusta	**Stream Name:**	Spaulding Brook
Route No.	ME17 - Eastern Ave.	**Bridge Name:**	Spaulding Brook
Asset ID:	2790	**Analysis by:**	ajs / csh
Lat:	44.30089 **Long:** -69.73559	**Date:**	6/10/2022

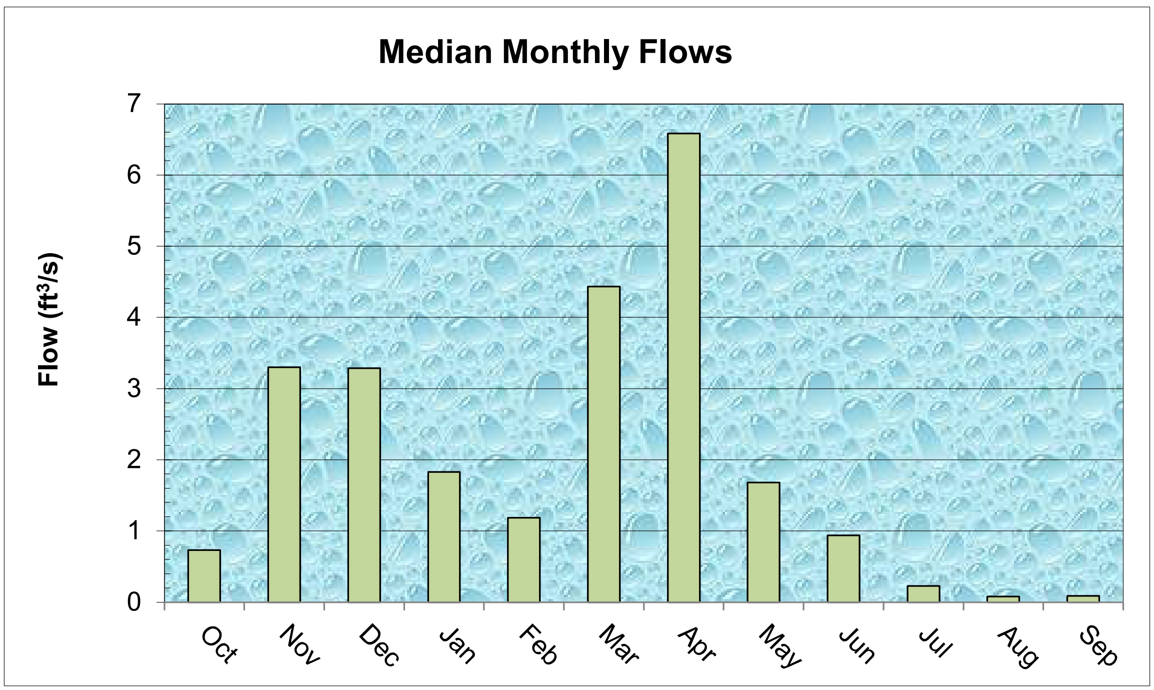
DO NOT ENTER ANY DATA ON THIS PAGE; EVERYTHING IS CALCULATED

MAINE MONTHLY MEDIAN FLOWS and HYDRAULIC GEOMETRY BY USGS REGRESSION EQUATIONS (2004, 2013, 2015)

	Value	Variable	Explanation
	1.77	*A*	Area (mi^2)
441460	4903960	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	57.85	*DIST*	Distance from Coastal reference line (mi)
	42.8	*pptA*	Mean Annual Precipitation (inches)
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median} (ft3/s)	(m^3/s)
Jan	**1.83**	0.0519
Feb	**1.19**	0.0336
Mar	**4.43**	0.1256
Apr	**6.58**	0.1866
May	**1.68**	0.0477
Jun	**0.94**	0.0266
Jul	**0.23**	0.0065
Aug	**0.08**	0.0023
Sep	**0.09**	0.0025
Oct	**0.73**	0.0207
Nov	**3.30**	0.0935
Dec	**3.28**	0.0931

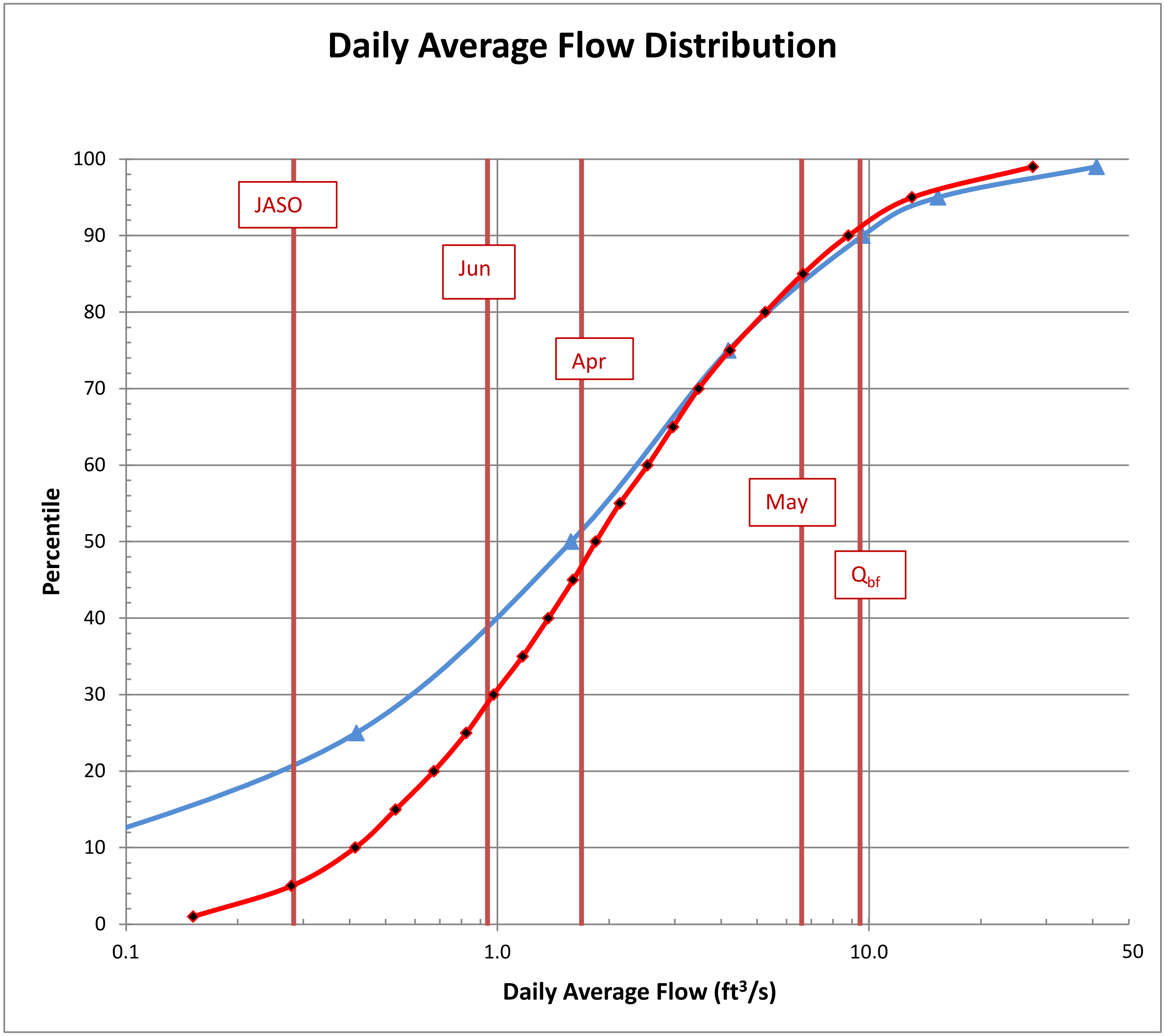
Q_{bf}	**9.5**	
ann avg	**3.9**	
ann med	**1.6**	
$Q_{1.002}$	**9.5**	
$Q_{1.01}$	**12.8**	
$Q_{1.05}$	**18.2**	
Q_{bf}	**39.0**	assume v = 4ft/s



W_{bf}	**13.5**	estimated bankfull width (ft)
d_{bf}	**0.7**	estimated bankfull depth (ft)
A_{bf}	**7.4**	estimated bankfull flow area (ft^2)

References

Dudley, 2013. FY2013 Progress Report - Phase 1 …, USFWS QRP Project
Dudley, 2004. Estimating Monthly Streamflows … , SIR 2004-5026
Dudley, 2015. Regression Equations for Monthly & Annual Mean..., USGS SIR 2015-5151



Daily Avg Flow Dist

A_{ws} = (mi^2)		1.8
	Q (ft^3/s)	
Pctl	**Median**	**84th pctl**
1.00E-06	0.00	0.00
1	0.15	0.27
5	0.28	0.45
10	0.41	0.62
15	0.53	0.78
20	0.67	0.94
25	0.82	1.10
30	0.98	1.26
35	1.17	1.44
40	1.37	1.65
45	1.60	1.87
50	1.84	2.21
55	2.14	2.57
60	2.54	3.02
65	2.97	3.51
70	3.48	4.10
75	4.22	4.93
80	5.26	5.88
85	6.64	7.54
90	8.79	10.13
95	13.05	15.75
99	27.60	36.33

Q_{bf}	9.5
$Q_{1.002}$	9.5
$Q_{1.1}$	21.8
Q_2	44.5

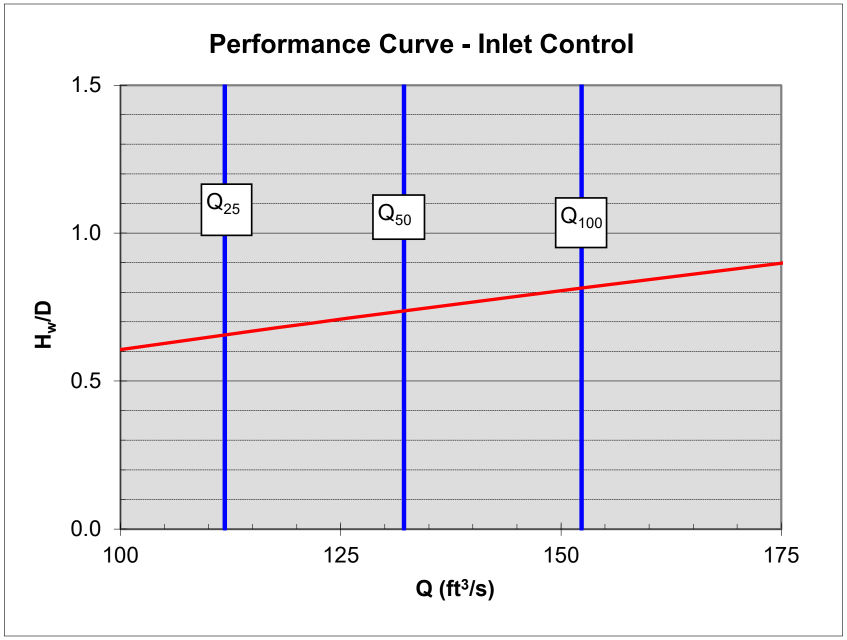
NOTE: ***This page is for preliminary sizing only.***
Final design should be done with HY8 or HDS-5

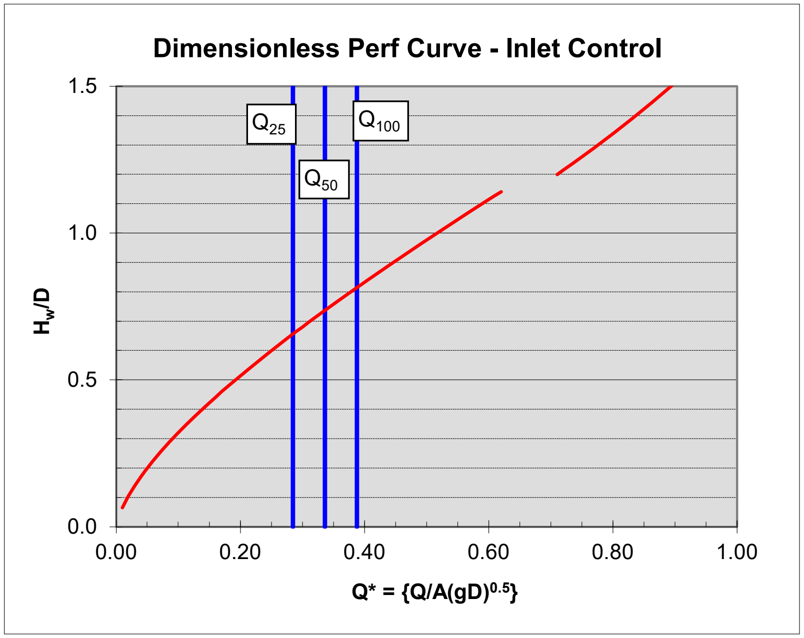
Note:
culvert dimensions are for open flow area; adjust for lost capacity due to embedding / backfilling (min {2' / 25% rise} embedment)
Finish analysis with HY-8

Preliminary Culvert Sizing - Round & Box Culverts

Choose shape and inlet type by pull-down menu in green cells

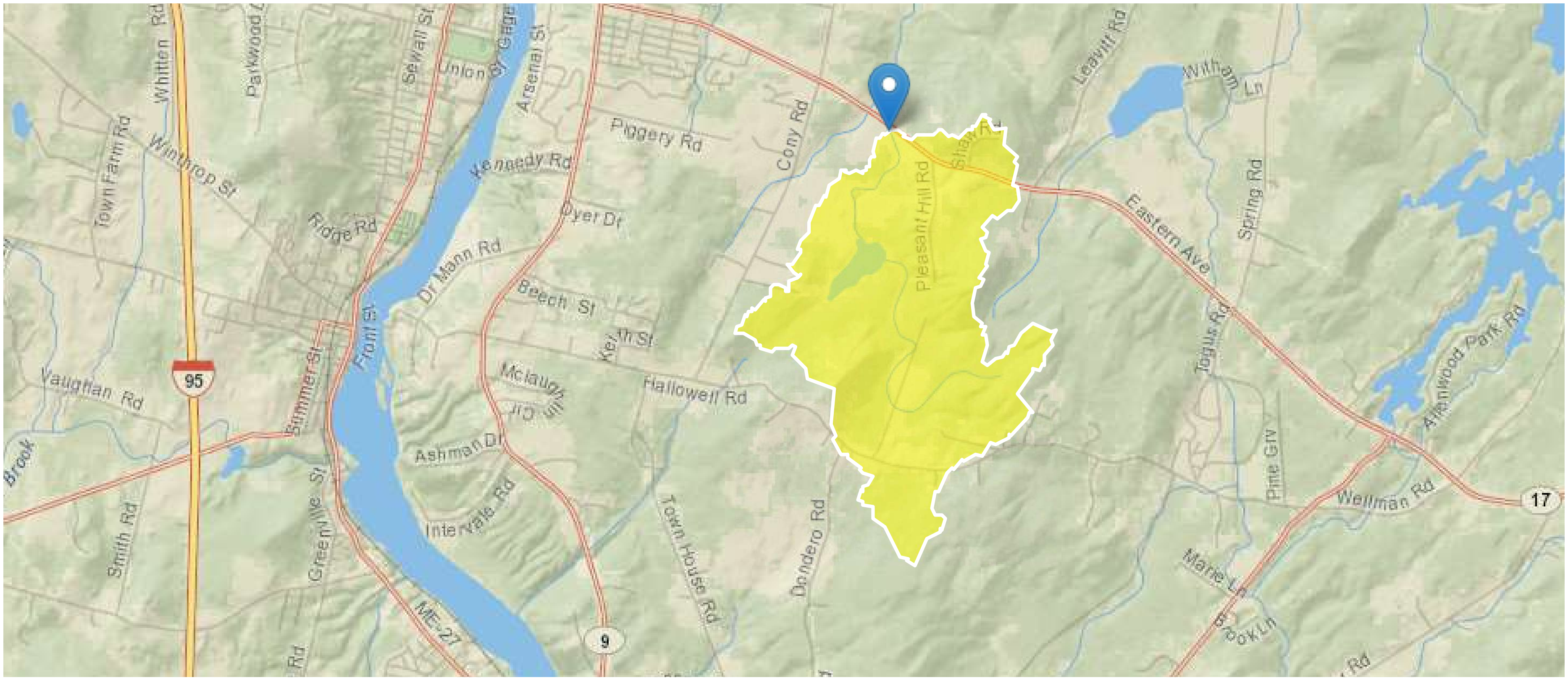
Shape:	Round					
Inlet Type:	Circ RCP Proj		Q_{25}	111.8		
D or R (ft)	6	*diam / rise*	Q_{50}	132.2	trial D / R =	5.5
w (ft)	*0*	*box span*	Q_{100}	152.3	trial w: BFW =	13.5
Slope (ft/ft)	0.01					
A (ft^2)	28.27					
g (ft/s^2)	32.2					





SS-Rpt-23126-Augusta

Region ID: ME
Workspace ID: ME20220610124606428000
Clicked Point (Latitude, Longitude): 44.30017, -69.73555
Time: 2022-06-10 08:46:35 -0400



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
BSLDEM10M	Mean basin slope computed from 10 m DEM	6.93	percent
CENTROIDX	Basin centroid horizontal (x) location in state plane coordinates	441460.17	meters
CENTROIDY	Basin centroid vertical (y) location in state plane units	4903960.38	meters
COASTDIST	Shortest distance from the coastline to the basin centroid	58	miles
DRNAREA	Area that drains to a point on a stream	1.77	square miles
ELEV	Mean Basin Elevation	236.3	feet
ELEVMAX	Maximum basin elevation	401.9	feet
I24H100Y	Maximum 24-hour precipitation that occurs on average once in 100 years	6.48	inches
I24H10Y	Maximum 24-hour precipitation that occurs on average once in 10 years	4.31	inches
I24H200Y	Maximum 24-hour precipitation that occurs on average once in 200 years	7.26	inches
I24H25Y	Maximum 24-hour precipitation that occurs on average once in 25 years	5.16	inches
I24H2Y	Maximum 24-hour precipitation that occurs on average once in 2 years - Equivalent to precipitation intensity index	2.94	inches
I24H500Y	Maximum 24-hour precipitation that occurs on average once in 500 years	8.45	inches
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	5.8	inches
I24H5Y	Maximum 24-hour precipitation that occurs on average once in 5 years	3.69	inches
LC06WATER	Percent of open water, class 11, from NLCD 2006	0.55	percent
LC11DEV	Percentage of developed (urban) land from NLCD 2011 classes 21-24	6.41	percent
LC11IMP	Average percentage of impervious area determined from NLCD 2011 impervious dataset	1.2	percent
PRDECFEB90	Basin average mean precipitation for December to February from PRISM 1961-1990	9.94	inches
PRECIP	Mean Annual Precipitation	42.8	inches

Parameter Code	Parameter Description	Value	Unit
SANDGRAVAF	Fraction of land surface underlain by sand and gravel aquifers	0	dimensionless
SANDGRAVAP	Percentage of land surface underlain by sand and gravel aquifers	0	percent
STATSGOA	Percentage of area of Hydrologic Soil Type A from STATSGO	0.52	percent
STORAGE	Percentage of area of storage (lakes ponds reservoirs wetlands)	22.849	percent
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	21.16	percent

Peak-Flow Statistics

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

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	0.26	5680
I24H2Y	24 Hour 2 Year Precipitation	2.94	inches	1.92	4.17
STORAGE	Percent Storage	22.849	percent	0	29.4
I24H5Y	24 Hour 5 Year Precipitation	3.69	inches	2.48	5.38
I24H10Y	24 Hour 10 Year Precipitation	4.31	inches	2.84	6.38
I24H25Y	24 Hour 25 Year Precipitation	5.16	inches	3.3	7.75
I24H50Y	24 Hour 50 Year Precipitation	5.8	inches	3.65	8.79
I24H100Y	24 Hour 100 Year Precipitation	6.48	inches	3.99	9.88
I24H200Y	24 Hour 200 YearPrecipitation	7.26	inches	5.26	11.1
I24H500Y	24 Hour 500 Year Precipitation	8.45	inches	5.95	13.1

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

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

Statistic	Value	Unit	PII	PIu	ASEp
50-percent AEP flood	41	ft^3/s	21.7	77.4	39.1
20-percent AEP flood	63.4	ft^3/s	34.1	118	38.1
10-percent AEP flood	80	ft^3/s	42.4	151	38.9
4-percent AEP flood	102	ft^3/s	53.3	195	39.9
2-percent AEP flood	120	ft^3/s	61.7	234	39.7
1-percent AEP flood	139	ft^3/s	71.8	269	40.7
0.5-percent AEP flood	162	ft^3/s	80.8	325	42.8
0.2-percent AEP flood	191	ft^3/s	93.9	389	43.8

Peak-Flow Statistics Citations

Lombard, P.J., and Hodgkins, G.A.,2020, Estimating flood magnitude and frequency on gaged and ungaged streams in Maine: U.S. Geological Survey Scientific Investigations Report 2020–5092, 56 p. (https://doi.org/10.3133/sir20205092)

Flow-Duration Statistics

Flow-Duration Statistics Parameters [Statewide Annual SIR 2015 5151]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	236.3	feet	239	2120

Flow-Duration Statistics Disclaimers [Statewide Annual SIR 2015 5151]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Flow-Duration Statistics Flow Report [Statewide Annual SIR 2015 5151]

Statistic	Value	Unit
1 Percent Duration	0.00278	ft^3/s
5 Percent Duration	0.0214	ft^3/s
10 Percent Duration	0.0673	ft^3/s
25 Percent Duration	0.417	ft^3/s
50 Percent Duration	1.58	ft^3/s
75 Percent Duration	4.18	ft^3/s
90 Percent Duration	9.6	ft^3/s
95 Percent Duration	15.3	ft^3/s
99 Percent Duration	41	ft^3/s

Flow-Duration Statistics Citations

Dudley, R.W.,2015, Regression equations for monthly and annual mean and selected percentile streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2015–5151, 35 p. (http://dx.doi.org/10.3133/sir20155151)

Annual Flow Statistics

Annual Flow Statistics Parameters [Statewide Annual SIR 2015 5151]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	236.3	feet	239	2120

Annual Flow Statistics Disclaimers [Statewide Annual SIR 2015 5151]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Annual Flow Statistics Flow Report [Statewide Annual SIR 2015 5151]

Statistic	Value	Unit
Mean Annual Flow	3.95	ft^3/s

Annual Flow Statistics Citations

Dudley, R.W.,2015, Regression equations for monthly and annual mean and selected percentile streamflows for ungaged rivers in Maine: U.S. Geological Survey Scientific Investigations Report 2015–5151, 35 p. (http://dx.doi.org/10.3133/sir20155151)

Bankfull Statistics

Bankfull Statistics Parameters [Central and Coastal Bankfull 2004 5042]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	2.92	298

Bankfull Statistics Parameters [Appalachian Highlands D Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	0.07722	940.1535

Bankfull Statistics Parameters [New England P Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	3.799224	138.999861

Bankfull Statistics Parameters [USA Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	1.77	square miles	0.07722	59927.7393

Bankfull Statistics Disclaimers [Central and Coastal Bankfull 2004 5042]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Bankfull Statistics Flow Report [Central and Coastal Bankfull 2004 5042]

Statistic	Value	Unit
Bankfull Streamflow	9.45	ft^3/s
Bankfull Width	10.3	ft
Bankfull Depth	0.721	ft
Bankfull Area	7.43	ft^2

Bankfull Statistics Flow Report [Appalachian Highlands D Bieger 2015]

Statistic	Value	Unit

Statistic	Value	Unit
Bieger_D_channel_width	19.3	ft
Bieger_D_channel_depth	1.32	ft
Bieger_D_channel_cross_sectional_area	25.8	ft^2

Bankfull Statistics Disclaimers [New England P Bieger 2015]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Bankfull Statistics Flow Report [New England P Bieger 2015]

Statistic	Value	Unit
Bieger_P_channel_width	29.6	ft
Bieger_P_channel_depth	1.56	ft
Bieger_P_channel_cross_sectional_area	46.3	ft^2

Bankfull Statistics Flow Report [USA Bieger 2015]

Statistic	Value	Unit
Bieger_USA_channel_width	15.1	ft
Bieger_USA_channel_depth	1.36	ft
Bieger_USA_channel_cross_sectional_area	23.3	ft^2

Bankfull Statistics Flow Report [Area-Averaged]

Statistic	Value	Unit
Bankfull Streamflow	9.45	ft^3/s

Statistic	Value	Unit
Bankfull Width	10.3	ft
Bankfull Depth	0.721	ft
Bankfull Area	7.43	ft^2
Bieger_D_channel_width	19.3	ft
Bieger_D_channel_depth	1.32	ft
Bieger_D_channel_cross_sectional_area	25.8	ft^2
Bieger_P_channel_width	29.6	ft
Bieger_P_channel_depth	1.56	ft
Bieger_P_channel_cross_sectional_area	46.3	ft^2
Bieger_USA_channel_width	15.1	ft
Bieger_USA_channel_depth	1.36	ft
Bieger_USA_channel_cross_sectional_area	23.3	ft^2

Bankfull Statistics Citations

Dudley, R.W.,2004, Hydraulic-Geometry Relations for Rivers in Coastal and Central Maine: U.S. Geological Survey Scientific Investigations Report 2004-5042, 30 p (http://pubs.usgs.gov/sir/2004/5042/pdf/sir2004-5042.pdf)

Bieger, Katrin; Rathjens, Hendrik; Allen, Peter M.; and Arnold, Jeffrey G.,2015, Development and Evaluation of Bankfull Hydraulic Geometry Relationships for the Physiographic Regions of the United States, Publications from USDA-ARS / UNL Faculty, 17p. (https://digitalcommons.unl.edu/usdaarsfacpub/1515?utm_source=digitalcommons.unl.edu%2Fusdaarsfacpub%2F1515&utm_medium=PDF&utm_campaign=PDFCoverPages)

Application Version: 4.9.0

StreamStats Services Version: 1.2.22

NSS Services Version: 2.2.0

Tailwater Channel Slope

The slope of the tailwater channel was selected by considering the slope of the natural surveyed streambed and slope of the surveyed surface waterway downstream of the structure. The slope of the waterway over a larger extent using information collected from USGS StreamStats and Google Earth was also considered. A brief summary of the slopes determined from each method is shown below.

Estimated Existing Tailwater Channel Slope Based on Surveyed Streambed

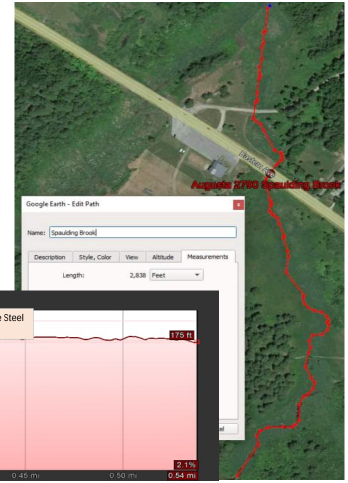
	Station (ft)	Elevation (ft)	Survey Point #
Downstream Point	34.94	172.90	1506
Upstream Point	657.50	173.11	1615
Channel Slope	0.0003		

Tailwater Slope Based on Interpreted Water Surface

	Station (ft)	Elevation (ft)	Survey Point #
Location 1	17.26	173.97	1511
Location 2	308.78	174.34	1435
Slope	0.0013	See longitudinal profile below	

Tailwater Slope Based on Google Earth Path

Net Elevation Change	2.8	ft
Distance of Path	2838	ft
Calculated Slope	0.0010	ft/ft



Tailwater Channel Slope Based on StreamStats Elevation Profile Tool

	Distance (mi)	Elevation (ft)
Downstream Point	1.17	174.59
Upstream Point	0.00	178.81
Channel Slope	0.0007	ft/ft

Latitude	Longitude	Elevation (ft)	Distance (mi)
44.29334	-69.73894	178.81	0.00
44.30687	-69.73428	174.59	1.17

Since the culvert is outlet controlled at all recurrence intervals, the flattest slope will results in the greatest tailwater and headwater elevations. The shallowest slope should be used as it will be the controlling condition. Note that all the values considered for the channel slope are relatively flat and the deviation between the steepest and shallowest slope is approximately 1 ft over a 1,000 ft distance.

Channel Slope Used for HY-8 Modeling	0.0003	ft/ft

Purpose:

The purpose of this calculation is to determine the inputs used in HY-8 modeling for both the proposed and existing culverts. HY-8 Version 8.0.0 (May 29, 2024) was used for the analysis.

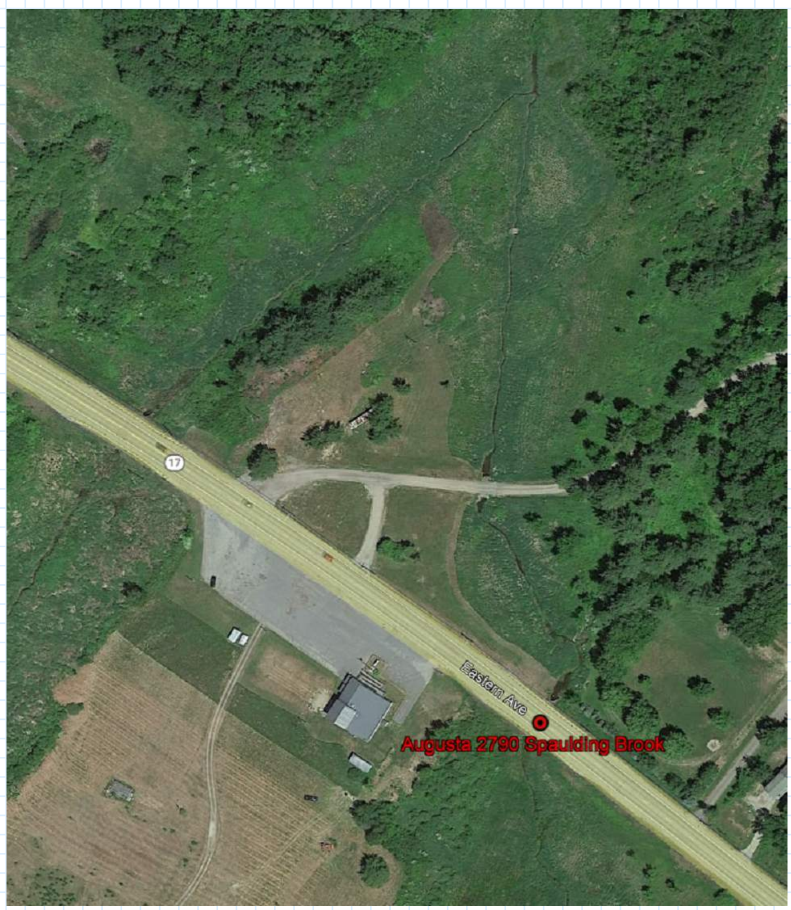
References:

- WSP Project Scoping Document *MaineDOT Structural GCA Contract - WIN 023126.00 - Augusta, Spaulding Brook Bridge (BR. # 2790)* dated January 10, 2023
- MaineDOT Hydrology *Augusta-23126-BR2790-Spaulding-Hydro.PDF* dated 6/10/2022
- George J. Arcement, Jr. and Verne R. Schneider *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains,* USGS Water Supply Paper 2339, Department of the Interior, 1989
- WSP, *Spaulding Brook Longitudinal Thalweg Profile*, calculation dated May 2023
- *Thorne Bridge & Spaulding Bridge* Record Plans dated January 1988

Site and Assumptions:

Spaulding Brook flows from South to North. Spaulding Brook Bridge is identified in the photo. Downstream of the structure is an additional brook crossing for a private driveway as well as a junction with the nearby Thorne (also known as Riggs) Brook. Spaulding Brook is relatively narrow and shallow that rises to a wide floodplain during storm events.

Per Task 4 of the WSP project scope titled *MaineDOT Structural GCA Contract - WIN 023126.00 - Augusta, Spaulding Brook Bridge (BR. # 2790)* dated January 10, 2023, the private driveway/culverts as well as the confluence of the two brooks is **NOT** included in the hydraulic analysis of this project.



Discharge Data:

User-defined discharge data was provided by MaineDOT in the hydrology file "Augusta-23126-BR2790-Spaulding-Hydro.PDF". All recurrence flow intervals provided by MaineDOT were included in the analysis.

Ret Pd T (yr)	I24	Q_T (ft^3/s) 1999 / 2015	Q_T (ft^3/s) 2021	Q_T (ft^3/s) Design
1.1			22	20
2	2.94	49	45	45
5	3.69	75	69	70
10	4.31	91	87	85
25	5.16	119	112	110
50	5.80	135	132	130
100	6.48	158	152	150
200	7.26	179	177	175
500	8.45	207	210	210

From MaineDOT Hydrology Report

Tailwater Data:

Spaulding Brook in the location around the culvert consists of a relatively narrow and shallow stream with a large floodplain. As such, a user-defined irregular channel was selected to model the effects of the tailwater when stream levels rise above the narrow-channel limits.

Brook Dimensions

A representative location for the tailwater dimensions was selected from survey information in the CAD files. The section selected corresponds to a portion of the reach that is downstream of the existing culvert, away from any localized effects of the existing culvert, and on a relatively straight portion of the reach. Approximate dimensions are shown.

Approximate thalweg elevation: $Elev_{thalweg} := 173.1 \text{ ft}$ Use approximate streambed elevation determined in WSP Thalweg Profile *WSP Thalweg Profile*

Check this elevation versus the invert elevation from the existing plans. The existing plans indicate that the flow line was set level at El. 173.8. The plans are dated January 1988, but do not indicate what datum was used. Since the North American Vertical Datum of 1988 (NAVD 88) didn't become an official vertical datum until 1993 (NOAA National Geodetic Survey), it is assumed that the plan elevations may be based on the NGVD 29 datum. Using the NGS Coordinate Conversion and Transformation Tool (NCAT) with the latitude and longitude for the bridge, the vertical datum offset was determined to be -0.70 ft (rounded) with a lower elevation in the NAVD 88 datum.

Vertical datum offset: $Datum := -0.70 \text{ ft}$

Plan flowline elevation (NGVD 29 assumed): $Elev_{flow.NGVD29} := 173.8 \text{ ft}$

Plan flowline elevation (NAVD 88): $Elev_{flow.NAVD88} := Elev_{flow.NGVD29} + Datum = 173.1 \text{ ft}$

Note that the thalweg elevation from the streambed profile and the flowline elevation from the 1988 plans (adjusted for datum shift) match.

The streambed is assumed to slope linearly up from the thalweg to the waterway edges indicated in the survey.

Representative streambank nodes used. $Node_{stream1} := 1482$ $Node_{stream2} := 1480$

Approximate distance between nodes: $b_{stream} := 6.0\ ft$

Approximate elevation of waterway edges: $Elev_{stream} := 174.0\ ft$

Similarly, the streambed is assumed to slope linearly from the waterway edges to the bank lines captured in the survey.

Representative bank line nodes: $Node_{bank1} := 1483$ $Node_{bank2} := 1479$

Approximate distance between nodes: $b_{bank} := 10.5\ ft$

Approximate elevation of bank lines: $Elev_{bank} := 174.75\ ft$

Floodplain Dimensions

The floodplain surrounding the brook is large relative to the stream and flat. This can be seen in the provided Google image below along with a photo taken during a recent storm by a MaineDOT employee.

Survey information is limited in this area but measuring in Google Earth indicates that the floodplain, bounded by the private drive on the left and neighboring property on the right, is in excess of 200' when measuring between points of 180' elevation. The 180' floodplain elevation was selected as it provides a reasonable width and a vertical buffer to the existing edge of pavement on Route 17 which is at approximately 182.5'.

Floodplain width for modeling: $b_{flood} := 200\ ft$

The elevation at the edge of the floodplain will be modeled as 180'. Additionally, two points will be located directly above the floodplain limits to account for flows that exceed the 180' elevation.

Assumed high elevation of floodplain: $Elev_{flood} := 180\ ft$

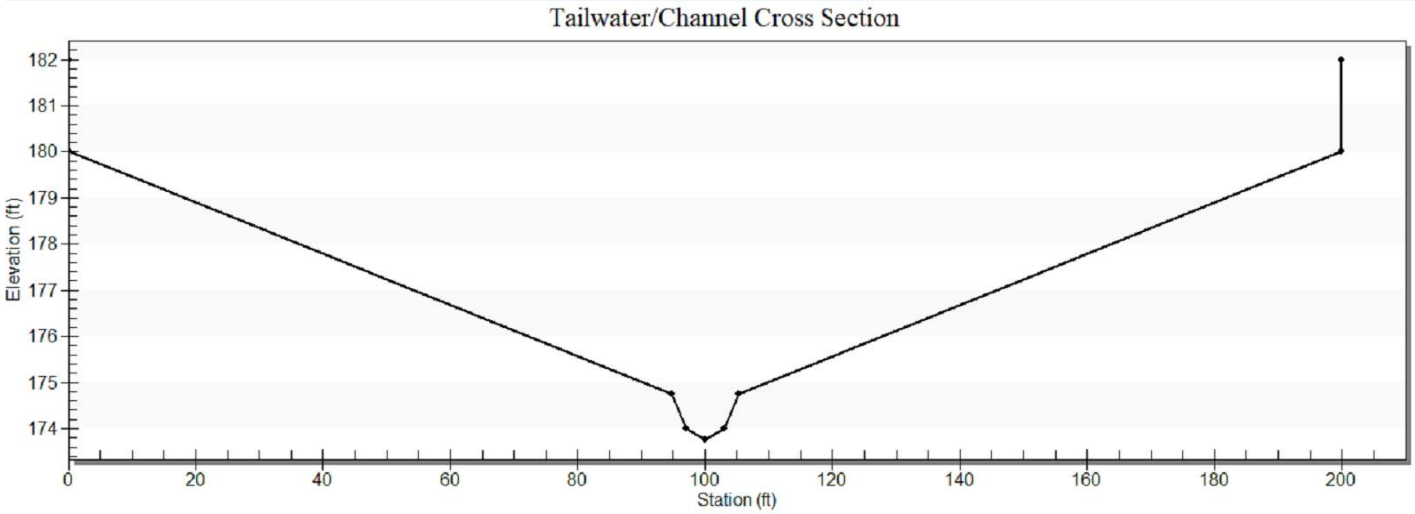
Assumed elevation for floods exceeding 180' elevation: $Elev_{excess} := 182\ ft$

Downstream floodplain:

May 1, 2023 Storm - Flooding Downstream of Spaulding Brook Bridge:

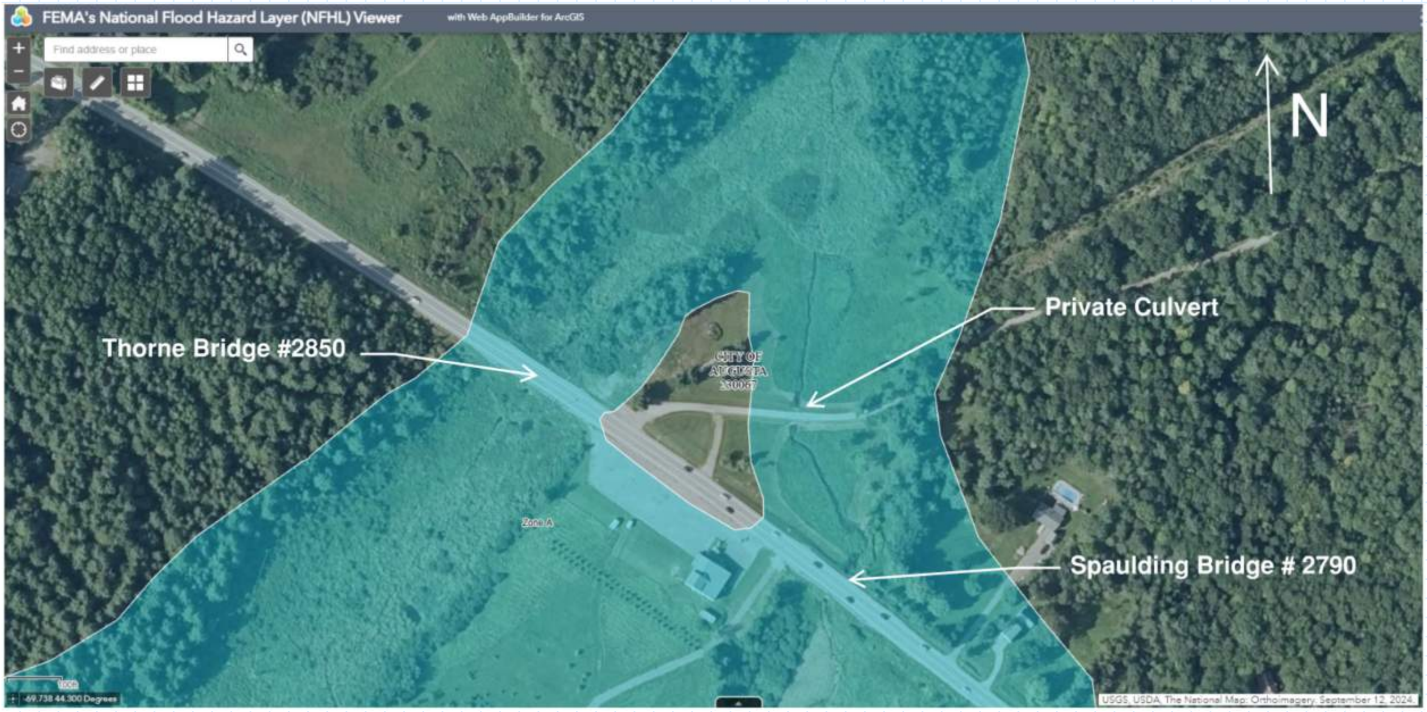
Coordinates for Tailwater Table

$$Coords := \begin{bmatrix} 0\ \text{ft} & Elev_{excess} \\ 0\ \text{ft} & Elev_{flood} \\ \frac{b_{flood}}{2} - \frac{b_{bank}}{2} & Elev_{bank} \\ \frac{b_{flood}}{2} - \frac{b_{stream}}{2} & Elev_{stream} \\ \frac{b_{flood}}{2} & Elev_{thalweg} \\ \frac{b_{flood}}{2} + \frac{b_{stream}}{2} & Elev_{stream} \\ \frac{b_{flood}}{2} + \frac{b_{bank}}{2} & Elev_{bank} \\ b_{flood} & Elev_{flood} \\ b_{flood} & Elev_{excess} \end{bmatrix} = \begin{bmatrix} 0.00 & 182.00 \\ 0.00 & 180.00 \\ 94.75 & 174.75 \\ 97.00 & 174.00 \\ 100.00 & 173.10 \\ 103.00 & 174.00 \\ 105.25 & 174.75 \\ 200.00 & 180.00 \\ 200.00 & 182.00 \end{bmatrix} \text{ft}$$



Google Satellite Imagery: Note that light brown areas are likely marsh grasses and dark brown areas are likely deciduous trees and shrubs that have dropped their foliage. Coniferous (evergreen) trees are green.

FEMA Flood Map: Note that Thorne Brook channel is not visible due to vegetative growth.



Manning's n

Manning's n was estimated using the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains, by George J. Arcement, Jr. and Verne R. Schneider, USGS Water Supply Paper 2339, Department of the Interior, 1989*

Channel n Value:

Determine the Manning's Coefficient of the channel using Cowan's Method described on page 2 of the referenced manual.

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad (3)$$

where

n_b = a base value of n for a straight, uniform, smooth channel in natural materials,
n_1 = a correction factor for the effect of surface irregularities,
n_2 = a value for variations in shape and size of the channel cross section,
n_3 = a value for obstructions,
n_4 = a value for vegetation and flow conditions, and
m = a correction factor for meandering of the channel.

Downstream outlet pool and channel with underwater vegetation visible in the channel:

Vegetation at culvert inlet:

Base value for Manning's n:	$n_b := 0.022$	Grain size at site unknown. Assume fine gravel and median sand sizes of D50 = 0.5 mm.	*USGS Table 1*
Surface irregularity factor:	$n_1 := 0.01$	Based on 0.006 to 0.010 for moderate irregularity	*USGS Table 2 (see next page)*
Channel variation factor:	$n_2 := 0.002$	Based on occasional variation	*USGS Table 2 (see next page)*
Obstruction factor:	$n_3 := 0.005$	Minor effects from downstream obstructions in channel such as debris deposits indicated in longitudinal stream profile and/or other features	*USGS Table 2 (see next page)*
Vegetation factor:	$n_4 := 0.03$	0.025 to 0.050 for large amount of vegetation based on turf grass growing in channel where average depth of flow is about equal to height of vegetation. Note that grass and vegetation in this channel appears much coarser (rougher) than turf grass.	*USGS Table 2 (see next page)*

Determine the meander ratio by comparing the straight-line distance the stream travels compared to the thalweg length between that same distance.

Straight-line distance: $length_{straight} := 277.9\ \text{ft}$ Between downstream invert to thalweg just upstream of private drive.

Thalweg length: $length_{thalweg} := 333.5\ \text{ft}$ Between downstream invert to thalweg just upstream of private drive

Meander ratio: $meander_{ratio} := \dfrac{length_{thalweg}}{length_{straight}} = 1.20$

Meander factor: $m := \dfrac{1.00 + 1.15}{2} = 1.075$ Meander on the edge of minor and appreciable. Use the average of the meander factors. *USGS Table 2 (see next page)*

Manning's coefficient for channel: $n_{channel} := \left(n_b + n_1 + n_2 + n_3 + n_4\right) \cdot m = 0.074$

Table 2. Adjustment values for factors that affect the roughness of a channel
[Modified from Aldridge and Garrett, 1973, table 2]

Channel conditions		*n* value adjustment[1]	Example
Degree of irregularity (n_1)	Smooth	0.000	Compares to the smoothest channel attainable in a given bed material.
	Minor	0.001–0.005	Compares to carefully dredged channels in good condition but having slightly eroded or scoured side slopes.
	Moderate	0.006–0.010	Compares to dredged channels having moderate to considerable bed roughness and moderately sloughed or eroded side slopes.
	Severe	0.011–0.020	Badly sloughed or scalloped banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of channels in rock.
Variation in channel cross section (n_2)	Gradual	0.000	Size and shape of channel cross sections change gradually.
	Alternating occasionally	0.001–0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-sectional shape.
	Alternating frequently	0.010–0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-sectional shape.
Effect of obstruction (n_3)	Negligible	0.000–0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
	Minor	0.005–0.015	Obstructions occupy less than 15 percent of the cross-sectional area, and the spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp-edged angular objects.
	Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause the effects of several obstructions to be additive, thereby blocking an equivalent part of a cross section.
	Severe	0.040–0.050	Obstructions occupy more than 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause turbulence across most of the cross section.
Amount of vegetation (n_4)	Small	0.002–0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.010–0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season, growing along the banks, and no significant vegetation is evident along the channel bottoms where the hydraulic radius exceeds 2 ft.
	Large	0.025–0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft; bushy willows about 1 year old intergrown with some weeds along side slopes (all vegetation in full foliage), and no significant vegetation exists along channel bottoms where the hydraulic radius is greater than 2 ft.
	Very large	0.050–0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; bushy willow trees about 1 year old intergrown with weeds along side slopes (all vegetation in full foliage), or dense cattails growing along channel bottom; trees intergrown with weeds and brush (all vegetation in full foliage).
Degree of meandering[2] (m)	Minor	1.00	Ratio of the channel length to valley length is 1.0 to 1.2.
	Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
	Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

Flood-Plain / Overbank n Value:

Downstream:

Determine the Manning's Coefficient of the floodplain using Cowan's Method described on page 8 of the referenced manual.

$$n=(n_b+n_1+n_2+n_3+n_4)m \qquad (6)$$

where

n_b = a base value of n for the flood plain's natural bare soil surface,
n_1 = a correction factor for the effect of surface irregularities on the flood plain,
n_2 = a value for variations in shape and size of the flood-plain cross section, assumed to equal 0.0,
n_3 = a value for obstructions on the flood plain,
n_4 = a value for vegetation on the flood plain, and
m = a correction factor for sinuosity of the flood plain, equal to 1.0.

Note that the floodplain characteristics are generally similar upstream and downstream.

Base value for Manning's n:	$n_{b.fp} := 0.022$	Assume floodplain material is similar to channel material	*USGS Table 1*
Surface irregularity factor:	$n_{1.fp} := 0.005$	Based on 0.001 to 0.005 for minor irregularity	*USGS Table 3 (see next page)*
Variation factor:	$n_{2.fp} := 0.000$	Not applicable	*USGS Table 3 (see next page)*
Obstruction factor:	$n_{3.fp} := 0.02$	Based on obstructions occupying from 15 to 50% of the cross-sectional area. Assumed due to presence of downstream private driveway crossing the flood plain.	*USGS Table 3 (see next page)*
Vegetation factor:	$n_{4.fp} := 0.06$	Based on 0.05 to 0.100 for very large amount of vegetation based on mature field crops where depth of flow is less than the height of the vegetation. The dense marsh grasses are assumed to be similar in roughness to mature field crops. Average depth of flow appears subjective. However, tops of marsh grasses are visible in May 1, 2023 storm photos.	*USGS Table 3 (see next page)*
Meander factor:	$m_{fp} := 1.00$	Meander not applicable for floodplain	*USGS Table 3 (see next page)*

Manning's coefficient for floodplain:

$$n_{fp} := \left(n_{b.fp} + n_{1.fp} + n_{2.fp} + n_{3.fp} + n_{4.fp}\right) \cdot m_{fp} = 0.107$$

Table 3. Adjustment values for factors that affect roughness of flood plains
[Modified from Aldridge and Garrett, 1973, table 2]

Flood-plain conditions		n value adjustment	Example
Degree of irregularity (n_1)	Smooth	0.000	Compares to the smoothest, flattest flood plain attainable in a given bed material.
	Minor	0.001–0.005	Is a flood plain slightly irregular in shape. A few rises and dips or sloughs may be visible on the flood plain.
	Moderate	0.006–0.010	Has more rises and dips. Sloughs and hummocks may occur.
	Severe	0.011–0.020	Flood plain very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pastureland and furrows perpendicular to the flow are also included.
Variation of flood-plain cross section (n_2)		0.0	Not applicable.
Effect of obstructions (n_3)	Negligible	0.000–0.004	Few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, or isolated boulders, occupy less than 5 percent of the cross-sectional area.
	Minor	0.005–0.019	Obstructions occupy less than 15 percent of the cross-sectional area.
	Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-sectional area.
Amount of vegetation (n_4)	Small	0.001–0.010	Dense growth of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation, or supple tree seedlings such as willow, cottonwood, arrowweed, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.011–0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation, or moderately dense stemmy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees in the dormant season.
	Large	0.025–0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation, or 8- to 10-year-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft, or mature row crops such as small vegetables, or mature field crops where depth of flow is at least twice the height of the vegetation.
	Very large	0.050–0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation, or moderate to dense brush, or heavy stand of timber with few down trees and little undergrowth where depth of flow is below branches, or mature field crops where depth of flow is less than the height of the vegetation.
	Extreme	0.100–0.200	Dense bushy willow, mesquite, and saltcedar (all vegetation in full foliage), or heavy stand of timber, few down trees, depth of flow reaching branches.
Degree of meander (m)		1.0	Not applicable.

Slope of Channel:

The slope of the channel was determined in a separate document by WSP: Spaulding Brook Longitudinal Thalweg Profile. This is attached to the end of this document.

The brook's slope through this section of reach is approximately flat.

$slope := 0.0003$ *WSP Thalweg Profile*

Existing Culvert Data:

Existing culvert data is indicated below. The existing culvert is a structural steel plate pipe arch.

Existing culvert span: $Span_{exist} := 11\ \text{ft} + 7\ \text{in} = 139\ \text{in}$

Existing culvert rise: $Rise_{exist} := 7\ \text{ft} + 5\ \text{in} = 89\ \text{in}$ Agrees with field measurement

HY-8 shape library for a Pipe Arch with a Steel Structural Plate material provides an option for a 139.4" span and 89.1" rise culvert. Assume this shape is what is used at Spaulding Brook.

The existing pipe is not embedded into the streambed.

Manning's coefficient for existing culvert: $n_{exist} := 0.030$ Based on HEC-RAS published data and various sources assuming 2" deep corrugations

The existing culvert is straight with mitered ends and no inlet depression. Stream channel does have materials accumulated just upstream of the inlet as indicated in the longitudinal profile (longitudinal profile attached to the end of this document).

Existing inlet elevation: $inlet_{exist} := 173.42\ \text{ft}$ *Ground survey*

Existing culvert length: $Length_{exist} := 72\ \text{ft}$ *1988 Record Plans*

Existing outlet elevation: $outlet_{exist} := 173.56\ \text{ft}$ *Ground survey*

Existing Roadway Elevation:

Approximate minimum roadway crown elevation near culvert is based on survey points 1136 and 1155 with an elevation of 182.90.

Calibration of Existing Conditions Model:

The existing conditions model was calibrated against all available data which included:

1. Hydraulic elevations provided in the 1988 existing bridge plans
2. Photograph during 5/1/2023 storm event
3. 12/13/2022 site visit measurement of water surface
4. FEMA flood map

The FEMA flood map appears unreliable to use at this location because it shows the lowest part of Rte. 17 from the Thorne Brook Bridge to the Spaulding Brook Bridge as being above the Zone A floodplain and it shows Rte. 17 as being under water along its higher locations away from the two bridges. This defies the reality of the topography. Therefore, consider other data for model calibration.

1988 Plan Data:

Recall datum offset: $Datum = -0.7\ \text{ft}$

Q100 from 1988 existing bridge plans: $Q_{100.exist.29} := 179.7\ \text{ft}$ Assumed to be in NGVD 29 datum

Q50 from 1988 existing bridge plans: $Q_{50.exist.29} := 179.2\ \text{ft}$ Assumed to be in NGVD 29 datum

Adjusted Q100 from 1988 existing bridge plans: $Q_{100.exist.88} := Q_{100.exist.29} + Datum = 179.0 \text{ ft}$ Adjusted to NAVD 88 datum

Adjusted Q50 from 1988 existing bridge plans: $Q_{50.exist.88} := Q_{50.exist.29} + Datum = 178.5 \text{ ft}$ Adjusted to NAVD 88 datum

May 1, 2023 Storm Data:

The May 1, 2023 storm made the news:
https://www.centralmaine.com/2023/05/01/weekend-storm-brings-power-outages-and-flooding-to-central-maine/

Downstream flooding with private driveway and culvert submerged:

Close-up of flooding at private driveway culvert - note disturbance in water surface at driveway:

Per ground survey, tops of private downstream culvert pipes are at El 177.65 (west pipe inlet) and 177.68 (east pipe outlet), so the water surface is at an elevation higher than these elevations.

Comparing the height of the rope / chain spanning across the private driveway between the posts in the photos above and below, it appears that the depth of water flowing across the driveway is very shallow, maybe a couple of inches.

Private driveway culvert from Google Streetview:

The thickness of cover over the private driveway culvert is unknown, but it doesn't appear very thick. Based on ground survey, the culvert pipes are approximately 4-ft in diameter. While this image isn't the best for estimating thickness of soil cover, it is the best available information. The driveway doesn't appear to be much higher than the tops of the culvert pipes, so assume minimal cover of 6 inches.

Approximate top of driveway elevation: $Elev_{drive} := 177.68\ \text{ft} + 0.5\ \text{ft} = 178.18\ \text{ft}$

Approximate May 1, 2023 water surface elevation: $Elev_{May1.flood} := Elev_{drive} + 2\ \text{in} = 178.35\ \text{ft}$ Say 178.4 +/-

The May 1, 2023 photo (taken at 7:50am) from MaineDOT provides an illustration of how widespread the floodwater surface is for a moderate storm event. Precipitation amounts from NOAA Online Weather Data:

1. 0.15" on 4/30/2023 & 4.17" on 5/1/2023 at the Augusta Landfill (1.8 miles from bridge site)
2. 3.00" recorded on 4/30/2023 and 1.69" on 5/1/2023 at the Augusta Airport (3.25 miles from bridge site)

The hydrology report indicates a 10-year storm has 4.31" of precipitation in 24 hours and a 2-year storm has 2.94" in 24 hours. Therefore, the May 1 storm event is assumed to correlate to an estimated recurrence interval of at least Q2 to Q10.

Site Visit Data:

During 12/13/2022 site visit, culvert rise was verified to be approximately 7'-6" and the water surface was measured to be approximately 5'-6 1/2" below the crown of the culvert. The Ordinary High Water (OHW) rust staining on the culvert is 26" higher than the water surface at the outlet. Due to tailwater controlled conditions, the water surface is assumed to be level throughout the culvert length.

Measured water depth at culvert inlet: $d_{site} := (7\ \text{ft} + 6\ \text{in}) - (5\ \text{ft} + 6.5\ \text{in}) = 1.96\ \text{ft}$

Estimated water surface elevation in culvert: $Elev_{site} := d_{site} + inlet_{exist} = 175.38\ \text{ft}$ Say Elevation 175.4 +/-

Estimated OHW water surface elevation in culvert: $Elev_{OHW} := Elev_{site} + 26\ \text{in} = 177.55\ \text{ft}$ Say Elevation 177.6 +/-

Models:

Because the existing culvert has a negative slope and the 1988 plans indicate that the culvert was to be constructed level, two existing conditions models were developed.

The first model is the existing conditions Calibration Model. It is intended to simulate the as-designed conditions to be comparable with the water surface elevations on the 1988 plans. Due to differences in the invert elevations between the 1988 plans and the surveyed invert elevations, an Existing Conditions Model was created from the Calibration Model with adjustments to use the surveyed invert elevations instead of the invert elevation from the 1988 plans.

Calibration Model Results:

The model was developed with the intent to correlate as well as possible with the 1988 headwater elevations, adjusted for datum shift, the May 1, 2023 storm observations and site visit measurements. As discussed in the Sensitivity Analysis section below, assumptions for the tailwater slope and channel elevations, as well as the tailwater Manning's N values, greatly affect the model results.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.15	1.12	3.052	0.41	7-H2t	-1.00	0.77	3.04	3.04	0.66	0.34
Q2	45.00	45.00	177.00	1.72	3.896	0.52	7-H2t	-1.00	1.14	3.85	3.85	1.14	0.38
Q5	70.00	70.00	177.58	2.18	4.476	0.60	7-H2t	-1.00	1.43	4.39	4.39	1.55	0.40
Q10	85.00	85.00	177.87	2.43	4.767	0.64	7-H2t	-1.00	1.58	4.66	4.66	1.78	0.42
Q25	110.00	110.00	178.30	2.81	5.195	0.70	7-H2t	-1.00	1.81	5.04	5.04	2.14	0.44
Q50	130.00	130.00	178.60	3.09	5.503	0.74	7-H2t	-1.00	1.98	5.30	5.30	2.41	0.45
Q100	150.00	150.00	178.89	3.35	5.789	0.78	7-H2t	-1.00	2.14	5.55	5.55	2.68	0.46
Q200	175.00	175.00	179.22	3.67	6.125	0.82	7-H2t	-1.00	2.33	5.82	5.82	3.00	0.48
Q500	210.00	210.00	179.67	4.08	6.566	0.88	7-H2t	-1.00	2.57	6.16	6.16	3.44	0.50
Overtopping	560.00	484.58	183.30	6.80	10.198	1.37	4-FFf	-1.00	4.14	7.42	8.30	7.21	0.67

Based on the calibration model results, the headwater elevations in the table above are consistent with the 1988 plan elevations (Q50 & Q100) adjusted for datum shift.

Q100 = 178.89 is just below the datum adjusted 1988 plan elevation of 179.0.
Q50 = 178.60 is just above the datum adjusted 1988 plan elevation of 178.5.

The Q10 result of El. 177.87 is within 0.5 feet of the May 1, 2023 estimated flood elevation of 178.4 +/-.

The Q5 elevation of 177.58 appears to match the site visit OHW estimate of 177.6 +/-. OHW is often assumed to be correlated with Q1.1 or Q2 for natural streams, but a 5-year recurrence interval appears reasonable for this very flat marshy floodplain due to tailwater controlled conditions.

Existing Conditions Model Results:

This Existing Conditions model was created from the Calibration Model but adjusted to use the surveyed invert elevations instead of the invert elevation from the 1988 plans. As can be seen when comparing the headwater elevations of the Calibration Model vs the Existing Conditions Model, there are very minor differences between the results.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.16	1.11	2.739	0.37	7-A2t	-1.00	0.77	2.58	3.04	0.80	0.34
Q2	45.00	45.00	177.01	1.71	3.590	0.48	7-A2t	-1.00	1.14	3.39	3.85	1.31	0.38
Q5	70.00	70.00	177.60	2.17	4.176	0.56	7-A2t	-1.00	1.43	3.93	4.39	1.74	0.40
Q10	85.00	85.00	177.89	2.42	4.470	0.60	7-A2t	-1.00	1.58	4.20	4.66	1.97	0.42
Q25	110.00	110.00	178.32	2.80	4.903	0.66	7-A2t	-1.00	1.81	4.58	5.04	2.34	0.44
Q50	130.00	130.00	178.63	3.08	5.214	0.70	7-A2t	-1.00	1.98	4.84	5.30	2.62	0.45
Q100	150.00	150.00	178.92	3.35	5.504	0.74	7-A2t	-1.00	2.14	5.09	5.55	2.89	0.46
Q200	175.00	175.00	179.26	3.66	5.843	0.79	7-A2t	-1.00	2.33	5.36	5.82	3.22	0.48
Q500	210.00	210.00	179.71	4.07	6.288	0.85	7-A2t	-1.00	2.57	5.70	6.16	3.66	0.50
Overtopping	560.00	484.58	183.30	6.79	9.878	1.33	4-FFf	-1.00	4.14	7.42	8.30	7.21	0.67

Sensitivity Analysis:

Determine sensitivity of existing conditions model for input data.

Tailwater Slope:

Assume tailwater slope of 0.01. This lowers Q100 by 1.62 ft relative to the Existing Conditions model.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	175.04	1.11	1.624	0.22	7-A2t	-1.00	0.77	1.33	1.79	1.85	1.87
Q2	45.00	45.00	175.68	1.71	2.264	0.30	7-A2t	-1.00	1.14	1.84	2.30	2.71	1.93
Q5	70.00	70.00	176.15	2.17	2.727	0.37	7-A2t	-1.00	1.43	2.16	2.62	3.44	1.94
Q10	85.00	85.00	176.39	2.42	2.970	0.40	7-A2t	-1.00	1.58	2.32	2.78	3.84	1.95
Q25	110.00	110.00	176.76	2.80	3.341	0.45	7-A2t	-1.00	1.81	2.53	2.99	4.47	1.98
Q50	130.00	130.00	177.04	3.08	3.617	0.49	7-A2t	-1.00	1.98	2.68	3.14	4.94	2.01
Q100	150.00	150.00	177.30	3.35	3.881	0.52	7-A2t	-1.00	2.14	2.82	3.28	5.38	2.04
Q200	175.00	175.00	177.62	3.66	4.197	0.57	7-A2t	-1.00	2.33	2.97	3.43	5.91	2.07
Q500	210.00	210.00	178.04	4.07	4.623	0.62	7-A2t	-1.00	2.57	3.16	3.62	6.62	2.12
Overtopping	723.33	666.48	183.23	9.12	9.810	1.32	7-A2c	-1.00	4.97	4.97	5.24	13.11	2.59

Assume tailwater slope of 0.0001. This increases Q100 by 0.98 ft relative to the Existing Conditions model.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.68	1.11	3.258	0.44	7-A2t	-1.00	0.77	3.10	3.56	0.64	0.21
Q2	45.00	45.00	177.68	1.71	4.256	0.57	7-A2t	-1.00	1.14	4.08	4.54	1.08	0.24
Q5	70.00	70.00	178.36	2.17	4.942	0.67	7-A2t	-1.00	1.43	4.73	5.19	1.44	0.26
Q10	85.00	85.00	178.70	2.42	5.285	0.71	7-A2t	-1.00	1.58	5.05	5.51	1.65	0.27
Q25	110.00	110.00	179.21	2.80	5.787	0.78	7-A2t	-1.00	1.81	5.52	5.98	1.97	0.28
Q50	130.00	130.00	179.57	3.08	6.146	0.83	7-A2t	-1.00	1.98	5.84	6.30	2.22	0.29
Q100	150.00	150.00	179.90	3.35	6.479	0.87	7-A2t	-1.00	2.14	6.13	6.59	2.47	0.30
Q200	175.00	175.00	180.28	3.66	6.864	0.92	7-A2t	-1.00	2.33	6.46	6.92	2.78	0.31
Q500	210.00	210.00	180.74	4.07	7.317	0.99	7-A2t	-1.00	2.57	6.82	7.28	3.23	0.33
Overtopping	723.33	666.44	183.23	9.12	9.810	1.32	7-A2c	-1.00	4.97	4.97	0.00	13.11	0.00

As can be seen, the model is sensitive to the slope input data. This is expected with tailwater control. The slope assumed for the existing conditions analysis is based on over 600 feet of channel thalweg survey data. This should provide enough data to generate a reasonable estimate. Google Earth and the StreamStats Elevation Profile Tool were used to develop slope estimates as well, and these were slightly steeper than the slope from the survey data.

Manning's N for Tailwater:

Decrease Manning's N to 0.06 for channel and 0.08 for flood plain. This lowers Q100 by 0.38 feet.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	175.97	1.11	2.546	0.34	7-A2t	-1.00	0.77	2.38	2.84	0.88	0.43
Q2	45.00	45.00	176.75	1.71	3.333	0.45	7-A2t	-1.00	1.14	3.12	3.58	1.44	0.47
Q5	70.00	70.00	177.30	2.17	3.876	0.52	7-A2t	-1.00	1.43	3.61	4.07	1.90	0.50
Q10	85.00	85.00	177.57	2.42	4.150	0.56	7-A2t	-1.00	1.58	3.85	4.31	2.16	0.52
Q25	110.00	110.00	177.98	2.80	4.556	0.61	7-A2t	-1.00	1.81	4.19	4.65	2.55	0.54
Q50	130.00	130.00	178.27	3.08	4.848	0.65	7-A2t	-1.00	1.98	4.43	4.89	2.85	0.56
Q100	150.00	150.00	178.54	3.35	5.122	0.69	7-A2t	-1.00	2.14	4.65	5.11	3.14	0.57
Q200	175.00	175.00	178.86	3.66	5.443	0.73	7-A2t	-1.00	2.33	4.90	5.36	3.49	0.59
Q500	210.00	210.00	179.29	4.07	5.866	0.79	7-A2t	-1.00	2.57	5.21	5.67	3.96	0.61
Overtopping	630.00	539.31	183.35	7.42	9.930	1.34	4-FFf	-1.00	4.40	7.42	7.89	8.02	0.83

As can be seen, the above parameters have noticeable effects on the Existing Conditions model results. The tailwater slope has a potentially greater influence than the Manning's N assumptions. In combination, these variables greatly affect the model and its results. This is largely because the bridge is located in a very flat densely vegetated floodplain that is tailwater controlled.

Proposed Culvert Data:

Proposed culvert data is indicated below. The replacement culvert is anticipated to be a reinforced concrete box.

Culvert Span

The culvert span will be based on 1.0 times the estimated bankfull width provided in the MaineDOT hydrology report. The span will be increased to the nearest whole foot. Not critical fish habitat.

Estimated bankfull width: $b_{bankfull} := 13.5\ \text{ft}$ *MaineDOT hydrology report*

Culvert span: $Span := 14\ \text{ft}$

Culvert rise: $Rise := 8\ \text{ft}$

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Culvert depth of streambed fill: $d_{stream.fill} := 24 \text{ in}$ HY-8 Embedment *WSP Thalweg Profile*

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Manning's Coefficient for Culvert

Manning's coefficient, walls and top: $n_{culvert1} := 0.012$ finished concrete

Manning's coefficient, bottom: $n_{culvert2} := \left(n_b + n_1 + n_2 + n_3\right) = 0.039$ Channel Manning's n, determined earlier, without the vegetation and meander factor.

Misc. Culvert Configuration

The culvert will be straight through the crossing without an inlet depression. Additionally, the inlet configuration will have the box sides extended with square edge at crown (last option in figure below from HDS-5 Table C.2. Entrance Loss Coefficients).

HDS-5 Table C.2. Entrance Loss Coefficients:

- Box, Reinforced Concrete

Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Site Data:

Culvert invert elevation: $Elev_{invert} := 171.10 \text{ ft}$ *WSP Thalweg Profile*

Streambed inlet elevation: $Elev_{inlet} := 173.10 \text{ ft}$ *WSP Thalweg Profile*

Assumed culvert length: $L_{culvert} := 78 \text{ ft}$

Streambed outlet elevation: $Elev_{outlet} := Elev_{inlet} = 173.1 \text{ ft}$ Culvert placed level

Proposed Roadway Profile Data:

Roadway profile data has been included to verify appropriate freeboard, overtopping, and fill over the top of the proposed culvert.

Minimum Fill:

Determine the minimum fill over the culvert which will be at the outside, east edge of the culvert as the roadway profile elevation increases in the westward direction.

Roadway profile is on a sag vertical curve.

Vertical Curve Length: $VCL := 500 \text{ ft}$

PVC Station: $Sta_{PVC} := 10515.00 \text{ ft}$

PVC Elevation: $El_{PVC} := 184.87 \text{ ft}$

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

PVI Station: $Sta_{PVI} := Sta_{PVC} + \frac{VCL}{2} = 10765 \text{ ft}$

PVI Elevation: $El_{PVI} := 182.34 \text{ ft}$

PVT Station: $Sta_{PVT} := Sta_{PVC} + VCL = 11015 \text{ ft}$

PVT Elevation: $El_{PVT} := 182.94 \text{ ft}$

Grade 1 Slope: $G1 := \frac{El_{PVI} - El_{PVC}}{Sta_{PVI} - Sta_{PVC}} = -1.01\%$

Grade 2 Slope: $G2 := \frac{El_{PVT} - El_{PVI}}{Sta_{PVT} - Sta_{PVI}} = 0.24\%$ Matches plan callout of 0.24%, OK

Check Station 108+50 vs PDR plans: $x_{10850} := 10850 \text{ ft} - Sta_{PVC} = 335 \text{ ft}$

Roadway crown elevation at Sta 108+50: $El_{10850} := El_{PVC} + G1 \cdot x_{10850} + \frac{(G2 - G1) \cdot x_{10850}^{2}}{2 \cdot VCL} = 182.88 \text{ ft}$ Plan callout of 182.88, OK

Roadway crown elevation at outside wall of box culvert at Station 108+68.66 RT:

$x_{box} := 10868.66 \text{ ft} - Sta_{PVC} = 353.66 \text{ ft}$

$El_{crown.box} := El_{PVC} + G1 \cdot x_{box} + \frac{(G2 - G1) \cdot x_{box}^{2}}{2 \cdot VCL} = 182.86 \text{ ft}$

This is location where edge of box culvert intersects face of proposed rail

Travel way width: $b_{travel} := 12 \text{ ft}$

Travel way cross-slope: $x_{travel} := 0.02 \frac{\text{ft}}{\text{ft}}$

Width to face of rail: $b_{shoulder} := 7 \text{ ft}$

Shoulder cross-slope: $x_{shoulder} := 0.04 \frac{\text{ft}}{\text{ft}}$

Minimum shoulder elevation over culvert: $Elev_{shoulder.min} := El_{crown.box} - b_{travel} \cdot x_{travel} - b_{shoulder} \cdot x_{shoulder} = 182.34 \text{ ft}$

Approximate thickness of culvert top slab: $t_{slab.top} := 14 \text{ in}$ To be determined by fabricator, comparable to similar projects

Minimum fill/pavement over culvert: $t_{fill.min} := 24 \text{ in}$

Maximum elevation for top of culvert rise: $Elev_{max.rise} := Elev_{shoulder.min} - t_{fill.min} - t_{slab.top} = 179.17\ ft$

Maximum low chord to accommodate $t_{fill.min}$

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Elevation at top of rise: $Elev_{rise} := Elev_{invert} + Rise = 179.1\ ft$

Check minimum fill: $Check_{fill} := \mathbf{if}\left(Elev_{max.rise} > Elev_{rise}, \text{“Okay”}, \text{“Consider Revising”}\right) = \text{“Okay”}$

Buffer: $Buffer := Elev_{max.rise} - Elev_{rise} = 0.07\ ft$

Additional fill beyond 2-ft minimum over culvert

Approximate roadway top width: $b_{road} := 44\ ft$

two 12' lanes and two 7' shoulders with 3' behind guardrail each side

Proposed Model Results:

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.14	0.64	3.044	0.51	7-H2t	-1.00	0.40	3.04	3.04	0.47	0.34
Q2	45.00	45.00	176.97	1.16	3.875	0.65	7-H2t	-1.00	0.68	3.85	3.85	0.84	0.38
Q5	70.00	70.00	177.54	1.56	4.437	0.74	7-H2t	-1.00	0.92	4.39	4.39	1.14	0.40
Q10	85.00	85.00	177.82	1.78	4.716	0.79	7-H2t	-1.00	1.05	4.66	4.66	1.30	0.42
Q25	110.00	110.00	178.22	2.11	5.120	0.85	7-H2t	-1.00	1.24	5.04	5.04	1.56	0.44
Q50	130.00	130.00	178.51	2.36	5.406	0.90	7-H2t	-1.00	1.39	5.30	5.30	1.75	0.45
Q100	150.00	150.00	178.77	2.60	5.668	0.94	7-H2t	-1.00	1.53	5.55	5.55	1.93	0.46
Q200	175.00	175.00	179.07	2.88	5.968	0.99	7-H2t	-1.00	1.69	5.82	5.82	2.15	0.48
Q500	210.00	210.00	179.47	3.23	6.372	1.06	4-FFf	-1.00	1.91	6.00	6.16	2.50	0.50
Overtopping	653.33	560.28	183.32	6.31	10.217	1.70	4-FFf	-1.00	3.68	6.00	8.73	6.67	0.70

Due to tailwater controlled water surface elevations, the proposed elevations are similar and slightly lower than the existing conditions. The outlet velocities are also slightly lower.

The culverts, in both the existing and proposed conditions, are outlet controlled under all recurrence intervals. Both culverts have similar cross sectional openings. Per the existing plans, the existing culvert has a cross section of 67 square feet. The proposed culvert, with a 14'-0" span and 6'-0" above the thalweg / flowline (8'-0" rise minus 2'-0" minimum embedment) results in a 84 square foot modeled opening.

Both the existing and proposed culverts do not indicate roadway overtopping for any of the modeled recurrence intervals. The culvert is flowing full for the 500-year recurrence interval.

HW/D for Q50:

Per the 2003 MaineDOT Bridge Design Guide with 2018 updates, section 2.3.10.2.A, the headwater-to-depth ratio of the culvert at Q50 should be equal to or less than 0.9.

Q50 headwater elevation per HY-8 analysis: $HW_{Q50} := 178.51\ ft$

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Headwater depth at Q50: $HW_{d.Q50} := HW_{Q50} - Elev_{inlet} = 5.41\ \text{ft}$

Headwater Depth ratio, Q50: $HW.D_{Q50} := \dfrac{HW_{d.Q50}}{Rise - d_{stream.fill}} = 0.90$ Matches 0.90 in table above, Ok

Check: Culvert HW/D ratio is at the limit prescribed in the Bridge Design Guide. Ok.

Note that if the proposed culvert rise were increased to lower the HW/D ratio, it would cause substantial impacts to the surrounding area due to raising the roadbed since it is a relatively flat roadway on both approaches to the bridge. This would increase the construction costs, project extents, environmental impacts, utility impacts, and potentially ROW impacts.

According to the Bridge Inspection Report, there is a slight chance of overtopping the bridge, but there is no known documentation of this occurring.

Freeboard at Q100:

Per the MaineDOT Bridge Design Guide section 2.3.10.2.A, outlet controlled culverts should have a minimum of 1 foot freeboard, **from the edge of the pavement**, at Q100. Additional guidance from the "Maine DOT Hydraulic Capacity Standard & Guidance" indicates the culvert should not exceed "100% full" at Q100. Since this second statement is more restrictive at this site use that as the limiting elevation.

Elevation at top of culvert rise: $Elev_{rise} = 179.1\ \text{ft}$

Q100 headwater elevation: $HW_{Q100} := 178.77\ \text{ft}$

Check: $Check_{HW.D.Q100} := \textbf{if}\left(HW_{Q100} < Elev_{rise}, \text{“Okay”}, \text{“Revise”}\right) = \text{“Okay”}$

Scour Considerations:

The proposed bridge is a four-sided box culvert, with a constructed stream channel contained within the box. The culvert is therefore not susceptible to scour. Riprap aprons and toe walls are also included at culvert end to further resist undermining.

Outlet velocities for the culvert should be checked to confirm that plain riprap is sufficient to resist being carried by stream flows. Evaluating Scour at Bridges, Fifth Edition (HEC-18) is used to determine the acceptability of plain riprap.

Minimum average riprap stone size: $D_{riprap} := 5\ \text{in} = 0.127\ \text{m}$ *MaineDOT Standard Specification 703.26*

Depth of flow at Q500: $y := 6.372\ \text{ft}$

Shields equation: $V_c := 6.19 \cdot \left(\dfrac{y}{\text{m}}\right)^{\frac{1}{6}} \cdot \left(\dfrac{D_{riprap}}{\text{m}}\right)^{\frac{1}{3}} \cdot \dfrac{\text{m}}{\text{s}} = 11.402\ \dfrac{\text{ft}}{\text{s}}$ *HEC-18 Eq. C.12*

Q500 outlet velocity: $V_{Q500} := 2.50\ \dfrac{\text{ft}}{\text{s}}$

Check: $Check_{riprap} := \textbf{if}\left(V_{Q500} < V_c, \text{“Plain Riprap is Acceptable”}, \text{“Revise”}\right) = \text{“Plain Riprap is Acceptable”}$

Project Notes

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Project Units: U.S. Customary Units
Outlet Control Option: Profiles
Exit Loss Option: Standard Method

Crossing Notes

Crossing Input: Proposed Conditions Model

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	User-Defined	
Discharge List	Define...	
TAILWATER DATA		
Channel Type	Irregular Channel	
Irregular Channel	Define...	
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	182.860	ft
Roadway Surface	Paved	
Top Width	44.000	ft

Culvert Input: Proposed Conditions Model

Parameter	Value	Units
CULVERT DATA		
Name	Proposed	
Shape	Concrete Box	
Material	Concrete	
Span	14.000	ft
Rise	8.000	ft
Embedment Depth	24.000	in
Manning's n (Top/Sides)	0.012	
Manning's n (Bottom)	0.039	
Culvert Type	Straight	
Inlet Configuration	Square Edge (0º flare) Wingwall (Ke=0.7)	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	171.100	ft
Outlet Station	78.000	ft
Outlet Elevation	171.100	ft

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Table 2 - Summary of Culvert Flows at crossing: Proposed Conditions Model

Headwater Elevation (ft)	Discharge Names	Total Discharge (cfs)	Proposed Discharge (cfs)	Roadway Discharge (cfs)	Iterations
176.14	Q1.1	20.00	20.00	0.00	1
176.97	Q2	45.00	45.00	0.00	1
177.54	Q5	70.00	70.00	0.00	1
177.82	Q10	85.00	85.00	0.00	1
178.22	Q25	110.00	110.00	0.00	1
178.51	Q50	130.00	130.00	0.00	1
178.77	Q100	150.00	150.00	0.00	1
179.07	Q200	175.00	175.00	0.00	1
179.47	Q500	210.00	210.00	0.00	1
182.86	Overtopping	557.75	557.75	0.00	Overtopping

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Table 3 - Culvert Summary Table: Proposed

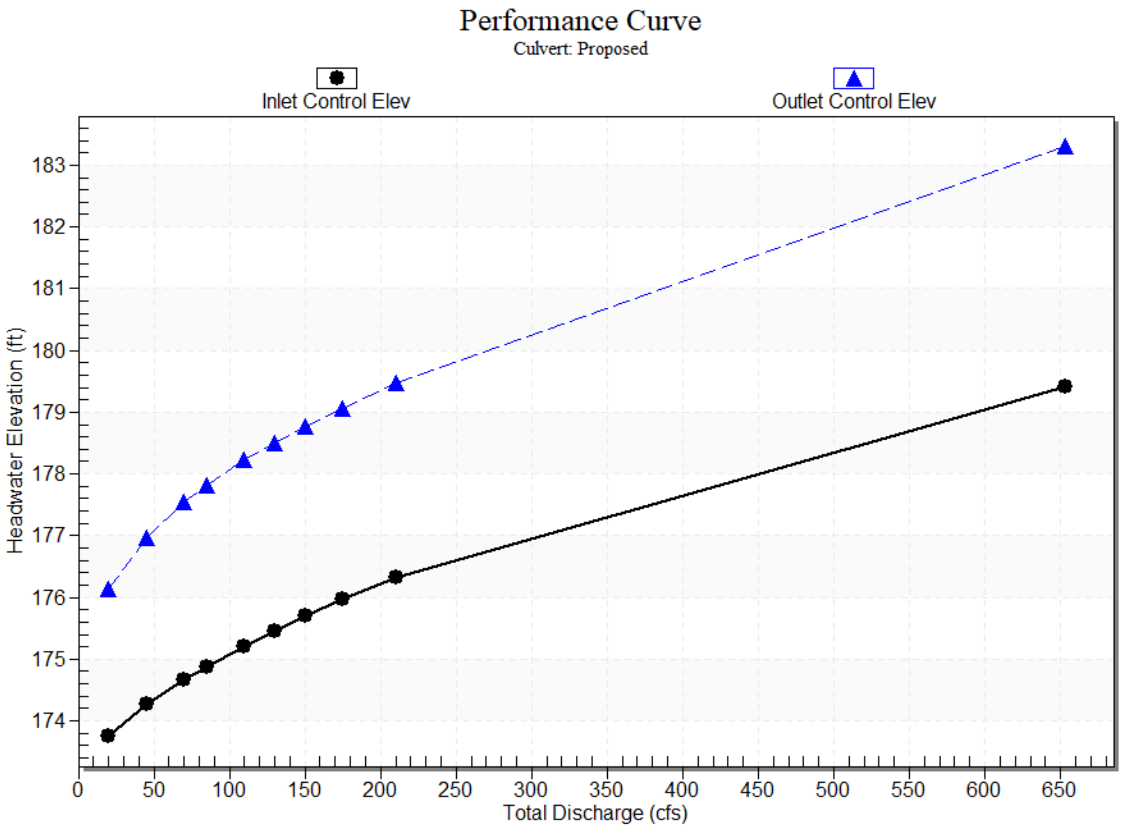
Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.14	0.64	3.044	0.51	7-H2t	-1.00	0.40	3.04	3.04	0.47	0.34
Q2	45.00	45.00	176.97	1.16	3.875	0.65	7-H2t	-1.00	0.68	3.85	3.85	0.84	0.38
Q5	70.00	70.00	177.54	1.56	4.437	0.74	7-H2t	-1.00	0.92	4.39	4.39	1.14	0.40
Q10	85.00	85.00	177.82	1.78	4.716	0.79	7-H2t	-1.00	1.05	4.66	4.66	1.30	0.42
Q25	110.00	110.00	178.22	2.11	5.120	0.85	7-H2t	-1.00	1.24	5.04	5.04	1.56	0.44
Q50	130.00	130.00	178.51	2.36	5.406	0.90	7-H2t	-1.00	1.39	5.30	5.30	1.75	0.45
Q100	150.00	150.00	178.77	2.60	5.668	0.94	7-H2t	-1.00	1.53	5.55	5.55	1.93	0.46
Q200	175.00	175.00	179.07	2.88	5.968	0.99	7-H2t	-1.00	1.69	5.82	5.82	2.15	0.48
Q500	210.00	210.00	179.47	3.23	6.372	1.06	4-FFf	-1.00	1.91	6.00	6.16	2.50	0.50
Overtopping	653.33	560.28	183.32	6.31	10.217	1.70	4-FFf	-1.00	3.68	6.00	8.73	6.67	0.70

Culvert Barrel Data

Culvert Barrel Type: Straight Culvert
Inlet Elevation(invert): 173.10 ft
Outlet Elevation (invert): 173.10 ft
Culvert Length: 78.00 ft
Culvert Slope: 0.00 ft/ft

Proposed Data here is not applicable to the sliplining rehabilitation of the existing culvert. See "Sliplining Hydraulics Analysis" calculation at the end of Appendix E.

Culvert Performance Curve Plot: Proposed



Project Notes

Project Units: U.S. Customary Units
Outlet Control Option: Profiles
Exit Loss Option: Standard Method

Crossing Notes

Crossing Input: Existing

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	User-Defined	
Discharge List	Define...	
TAILWATER DATA		
Channel Type	Irregular Channel	
Irregular Channel	Define...	
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	182.900	ft
Roadway Surface	Paved	
Top Width	43.770	ft

Culvert Input: Existing

Parameter	Value	Units
CULVERT DATA		
Name	Existing	
Shape	Pipe Arch	
Material	Steel Structural Plate	
Size	Define...	
Span	139.400	in
Rise	89.100	in
Embedment Depth	0.000	in
Manning's n	0.030	
Culvert Type	Straight	
Inlet Configuration	Mitered (Ke=0.7)	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	173.420	ft
Outlet Station	72.000	ft
Outlet Elevation	173.560	ft
Number of Barrels	1	

Table 2 - Summary of Culvert Flows at crossing: Existing

Headwater Elevation (ft)	Discharge Names	Total Discharge (cfs)	Existing Discharge (cfs)	Roadway Discharge (cfs)	Iterations
176.16	Q1.1	20.00	20.00	0.00	1
177.01	Q2	45.00	45.00	0.00	1
177.60	Q5	70.00	70.00	0.00	1
177.89	Q10	85.00	85.00	0.00	1
178.32	Q25	110.00	110.00	0.00	1
178.63	Q50	130.00	130.00	0.00	1
178.92	Q100	150.00	150.00	0.00	1
179.26	Q200	175.00	175.00	0.00	1
179.71	Q500	210.00	210.00	0.00	1
182.90	Overtopping	482.50	482.50	0.00	Overtopping

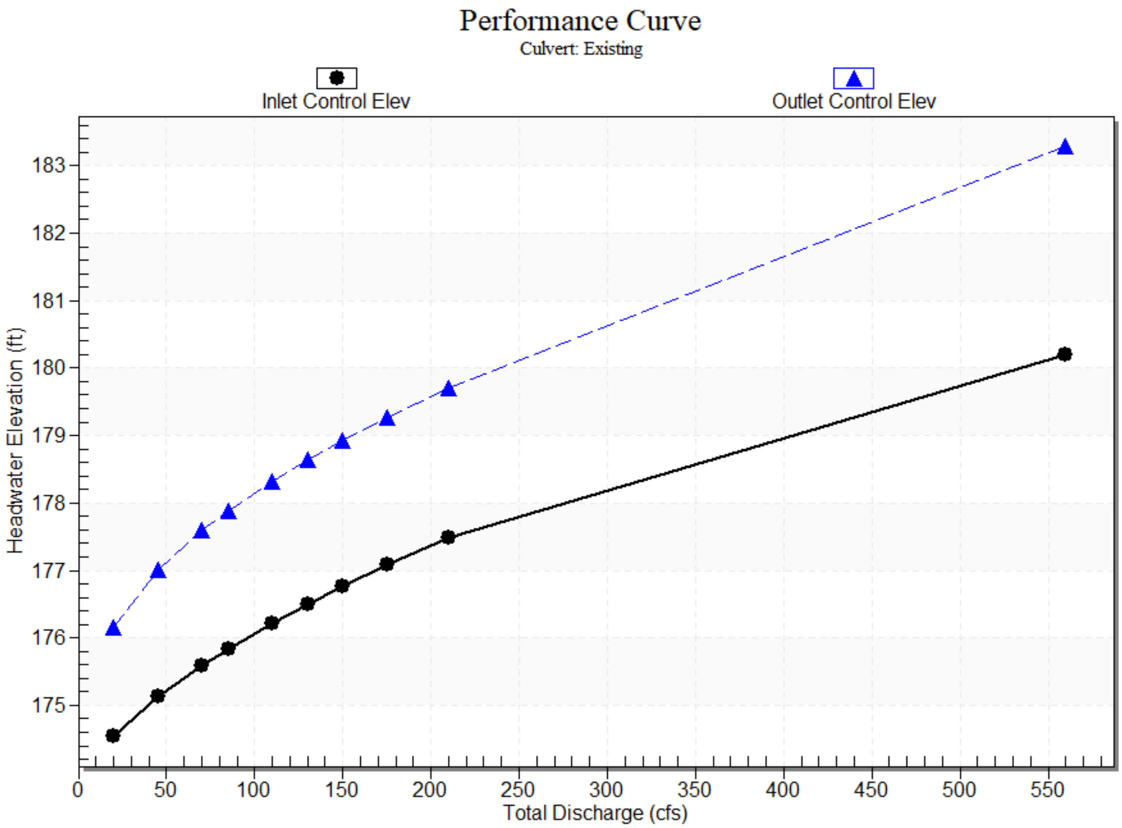
Table 3 - Culvert Summary Table: Existing

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.16	1.11	2.739	0.37	7-A2t	-1.00	0.77	2.58	3.04	0.80	0.34
Q2	45.00	45.00	177.01	1.71	3.590	0.48	7-A2t	-1.00	1.14	3.39	3.85	1.31	0.38
Q5	70.00	70.00	177.60	2.17	4.176	0.56	7-A2t	-1.00	1.43	3.93	4.39	1.74	0.40
Q10	85.00	85.00	177.89	2.42	4.470	0.60	7-A2t	-1.00	1.58	4.20	4.66	1.97	0.42
Q25	110.00	110.00	178.32	2.80	4.903	0.66	7-A2t	-1.00	1.81	4.58	5.04	2.34	0.44
Q50	130.00	130.00	178.63	3.08	5.214	0.70	7-A2t	-1.00	1.98	4.84	5.30	2.62	0.45
Q100	150.00	150.00	178.92	3.35	5.504	0.74	7-A2t	-1.00	2.14	5.09	5.55	2.89	0.46
Q200	175.00	175.00	179.26	3.66	5.843	0.79	7-A2t	-1.00	2.33	5.36	5.82	3.22	0.48
Q500	210.00	210.00	179.71	4.07	6.288	0.85	7-A2t	-1.00	2.57	5.70	6.16	3.66	0.50
Overtopping	560.00	484.58	183.30	6.79	9.878	1.33	4-FFf	-1.00	4.14	7.42	8.30	7.21	0.67

Culvert Barrel Data

Culvert Barrel Type: Straight Culvert
Inlet Elevation(invert): 173.42 ft
Outlet Elevation (invert): 173.56 ft
Culvert Length: 72.00 ft
Culvert Slope: -0.00 ft/ft

Culvert Performance Curve Plot: Existing



10 Al Paul Lane, Suite 103
Merrimack, NH 03054
wsp.com

Project: Spaulding Brook Bridge #2790

Task: ***Slipline Hydraulic Analysis***

Client: MaineDOT

Calculated by: E. Caron

Date: 3/19/2026

Checked by: K. James

Date: 3/30/2026

Design Intent:

Determine the effects of a 9'-4" x 6'-3" slipline and a 8'-10" x 6'-1" slipline with a steel plate pipe arch at Spaulding Brook using HY-8.

Design Information:

The hydraulic inputs used in HY-8 are based on the existing stream and culvert geometry submitted previously as *Preliminary Design Report Spaulding Book Bridge #2790 Over Spaulding Brook* by WSP for WIN 023126.00. Changes in proposed hydraulic modeling are summarized below.

Proposed Lining Geometry:

The new inverts will be placed 6" above the existing invert elevations and liners will be steel plate pipe arches.

Proposed invert elevation change: $\Delta_{invert} := 6 \text{ in}$

Existing inlet invert elevation: $EL_{inlet.exist} := 173.42 \text{ ft}$

Proposed inlet invert elevation: $EL_{inlet.propose} := EL_{inlet.exist} + \Delta_{invert} = 173.92 \text{ ft}$

Existing outlet invert elevation: $EL_{outlet.exist} := 173.56 \text{ ft}$

Proposed outlet invert elevation: $EL_{outlet.propose} := EL_{outlet.exist} + \Delta_{invert} = 174.06 \text{ ft}$

Existing Condition Results:

The existing condition hydraulic model results are shown here for comparison.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	HW/D	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.16	1.11	2.74	0.37	7-A2t	NA	0.77	2.58	3.04	0.80	0.34
Q2	45.00	45.00	177.01	1.71	3.59	0.48	7-A2t	NA	1.14	3.39	3.85	1.31	0.38
Q5	70.00	70.00	177.60	2.17	4.18	0.56	7-A2t	NA	1.43	3.93	4.39	1.74	0.40
Q10	85.00	85.00	177.89	2.42	4.47	0.60	7-A2t	NA	1.58	4.20	4.66	1.97	0.42
Q25	110.00	110.00	178.32	2.80	4.90	0.66	7-A2t	NA	1.81	4.58	5.04	2.34	0.44
Q50	130.00	130.00	178.63	3.08	5.21	0.70	7-A2t	NA	1.98	4.84	5.30	2.62	0.45
Q100	150.00	150.00	178.92	3.35	5.50	0.74	7-A2t	NA	2.14	5.09	5.55	2.89	0.46
Q200	175.00	175.00	179.26	3.66	5.84	0.79	7-A2t	NA	2.33	5.36	5.82	3.22	0.48
Q500	210.00	210.00	179.71	4.07	6.29	0.85	7-A2t	NA	2.57	5.70	6.16	3.66	0.50

The inlet configuration is assumed to be mitered to match the existing pipe arch and Manning's n is based on pipe arch size.

9'-4"x6'-3" Steel Plate Pipe Arch Slipline Results:

Note that the existing culvert and proposed sliplining are set on an adverse slope.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	HW/D	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.20	1.20	2.28	0.37	7-A2t	NA	0.81	2.08	3.04	1.25	0.34
Q2	45.00	45.00	177.10	1.87	3.18	0.51	7-A2t	NA	1.23	2.89	3.85	1.91	0.38
Q5	70.00	70.00	177.73	2.39	3.81	0.61	7-A2t	NA	1.55	3.43	4.39	2.47	0.40
Q10	85.00	85.00	178.05	2.68	4.13	0.66	7-A2t	NA	1.72	3.70	4.66	2.78	0.42
Q25	110.00	110.00	178.54	3.11	4.62	0.74	7-A2t	NA	1.98	4.08	5.04	3.27	0.44
Q50	130.00	130.00	178.90	3.42	4.98	0.80	7-A2t	NA	2.17	4.34	5.30	3.64	0.45
Q100	150.00	150.00	179.24	3.71	5.32	0.85	7-A2t	NA	2.35	4.59	5.55	4.00	0.46
Q200	175.00	175.00	179.65	4.04	5.73	0.92	7-A2t	NA	2.56	4.86	5.82	4.44	0.48
Q500	210.00	210.00	180.22	4.49	6.30	1.01	7-A2t	NA	2.84	5.20	6.16	5.05	0.50

10 Al Paul Lane, Suite 103
Merrimack, NH 03054
wsp.com

8'-10"x6'-1" Steel Plate Pipe Arch Slipline Results:

Note that the existing culvert and proposed sliplining are set on an adverse slope.

Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(ft)	HW/D	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	20.00	20.00	176.21	1.20	2.29	0.37	7-A2t	NA	0.77	2.08	3.04	1.26	0.34
Q2	45.00	45.00	177.11	1.89	3.19	0.52	7-A2t	NA	1.20	2.89	3.85	1.97	0.38
Q5	70.00	70.00	177.75	2.44	3.83	0.63	7-A2t	NA	1.53	3.43	4.39	2.55	0.40
Q10	85.00	85.00	178.08	2.74	4.16	0.68	7-A2t	NA	1.70	3.70	4.66	2.88	0.42
Q25	110.00	110.00	178.57	3.20	4.65	0.76	7-A2t	NA	1.97	4.08	5.04	3.39	0.44
Q50	130.00	130.00	178.94	3.51	5.02	0.82	7-A2t	NA	2.17	4.34	5.30	3.78	0.45
Q100	150.00	150.00	179.30	3.80	5.38	0.88	7-A2t	NA	2.35	4.59	5.55	4.16	0.46
Q200	175.00	175.00	179.73	4.15	5.81	0.95	7-A2t	NA	2.58	4.86	5.82	4.63	0.48
Q500	210.00	210.00	180.33	4.63	6.41	1.05	7-A2t	NA	2.86	5.20	6.16	5.28	0.50