HYDROLOGY REPORT

Thorne Brook Bridge (BR #2850) carries State Route 17 (Eastern Avenue) over northerly-flowing Thorne (Riggs) Brook in Augusta, Maine. As seen in Figure 1, approximately 1/10 mile east of the project is Spaulding Brook Bridge (BR #2790) over Spaulding Brook. Spaulding Brook flows northerly and reaches a junction with Thorne Brook located approximately 850-ft downstream from Spaulding Brook Bridge. There is a beaver dam located approximately 75' downstream from the existing structure. The nearest public roadway crossing is approximately 1.25 miles downstream at South Belfast Avenue where the brook passes through two large culverts.

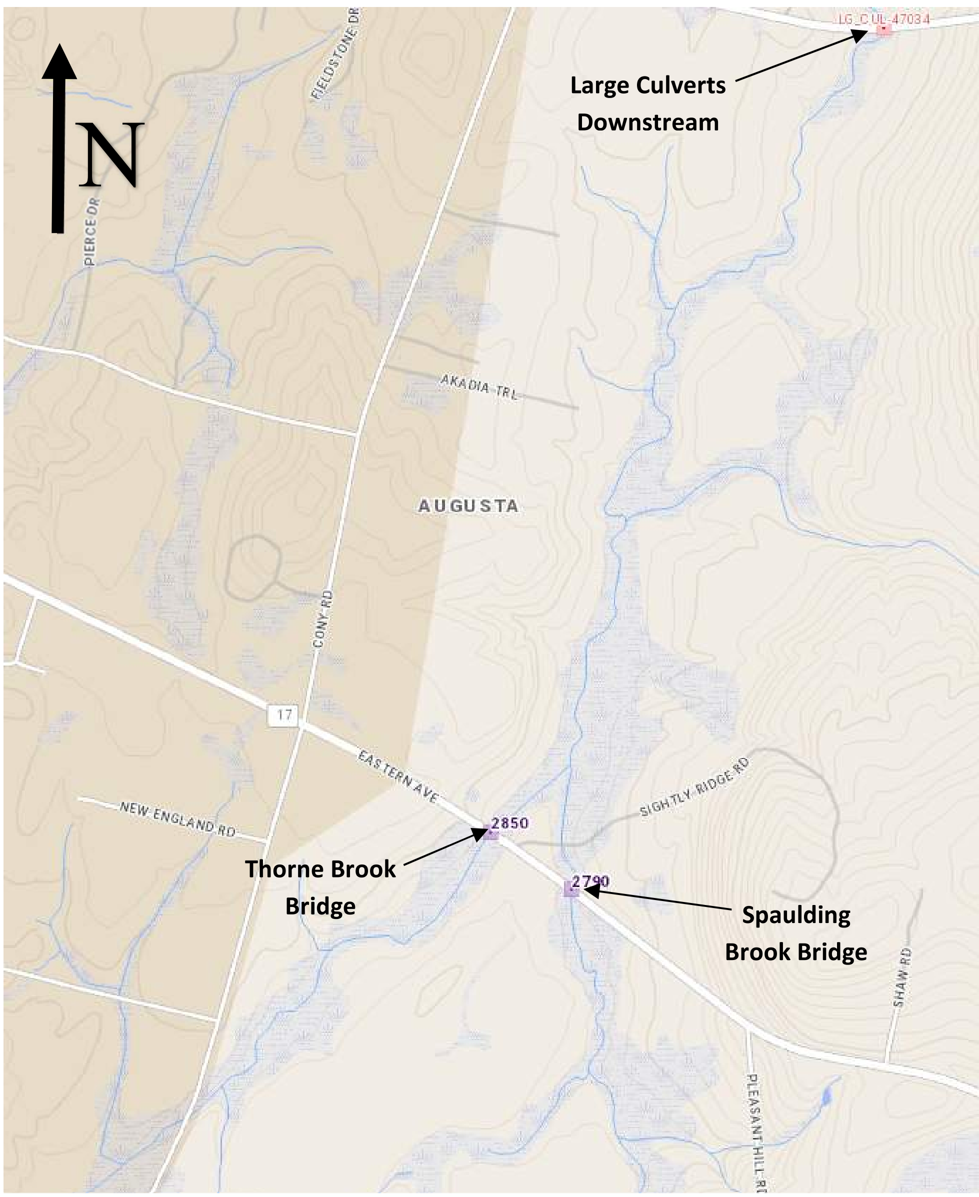


Figure 1: Site location

The drainage basin characteristics for Thorne Brook was provided by the Maine DOT Environmental Group, Hydrology Section. The watershed area for Thorne Brook at Thorne Brook Bridge is 0.95 square miles with 0.2 square miles of wetlands. The watershed is rural with a few private residences/businesses located primarily to the south and west of Thorne Brook Bridge. Flowrates are shown in Table 1 below.

Table 1: Design flow rates

Return Period (yr.)	Annual Exceedance Probability	Thorne Brook (ft^3/s)
1.1	91%	15
2	50%	30
5	20%	50
10	10%	60
25	4%	80
50	2%	95
100	1%	110
200	0.5%	130
500	0.2%	155

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 Thorne 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.

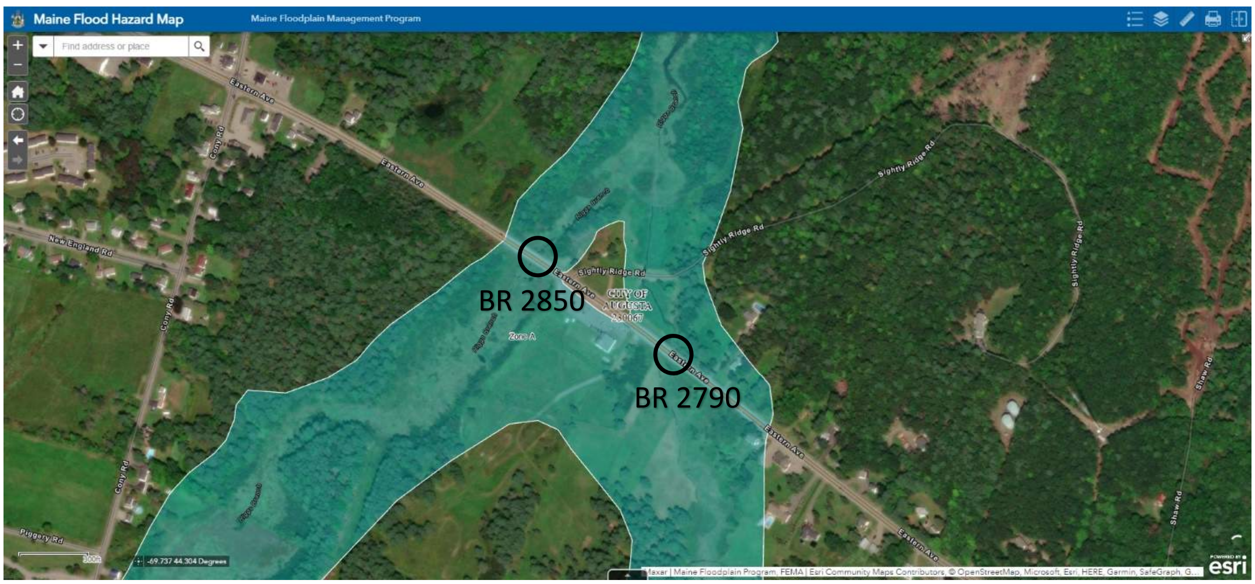


Figure 2: Maine Flood Hazard Map showing Bridge 2790 in Zone A Floodplain

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. However, there is a beaver dam located downstream of the structure which has impounded water at the 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 downstream beaver dam and the downstream junction with Spaulding Brook.

<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 based on the surveyed culvert inverts and the flowline elevation from the 1988 bridge plans since no ground survey has been obtained yet for the channel. Note: Subsequent to the 10/04/2024

Final PDR submission, additional survey was obtained in February 2025. A comparison was made between the original Existing Conditions model and the Existing Conditions model updated with the February 2025 tailwater channel and culvert invert elevations. The resulting differences in headwater elevations and velocities are not considered significant, with more conservative results for the original elevations. Given that the original results are more conservative, do not change the scope of the proposed work, and significant rework would be required, the calculations and hydraulic report were not updated with the February 2025 elevations.

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.

Due to similarities in tailwater areas, and lack of thalweg/stream survey downstream of the structure, the slope of the stream for hydraulic modeling was assumed to be the same slope as nearby Spaulding Brook. Since both structures are located in the same floodplain, which is relatively flat, the assumption that both tailwaters have the same slope is reasonable for this hydraulic analysis. This can be revisited if/when additional ground survey is obtained.

Figure 3: Downstream of Thorne Brook Bridge after a storm (5/1/2023)

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 very large amounts of herbaceous vegetation (marsh grasses, cattails, dense brush and mature trees) that contribute to higher values for Manning's n. The Manning's coefficients determined for the brook and overbank areas were 0.117 and 0.177 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	179.6	178.9
100 year	179.8	179.1

A May 1, 2023 storm in central Maine caused significant flooding in the floodplains surrounding the bridge. 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 5.14” of precipitation in 24 hours.
- 10-year storm with 4.29" in 24 hours.
- 5-year storm with 3.67” in 24 hours.
- 2-year storm with 2.93" in 24 hours.

An estimated flood elevation of Elevation 178.4 +/- was determined in the Spaulding Brook Bridge #2790 hydraulic analysis – see Spaulding Brook Bridge hydraulic analysis for more information. Given the very flat terrain in this area, this elevation is considered reasonable to use as an estimate at this bridge.

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, no measurements were obtained to determine an approximate elevation for the Ordinary High Water Mark (OHWM). Rust staining on the culvert could not be measured because the staining was submerged due to backwater from a downstream beaver dam. The water surface elevation was estimated to be at approximately Elevation 178, so OHW is assumed to be lower than this elevation.

<u>Existing Conditions Hydraulic Results:</u>

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	15.00	15.00	176.49	0.96	2.558	0.34	7-A2t	-1.00	0.66	1.91	2.36	0.86	0.19
Q2	30.00	30.00	177.19	1.37	3.262	0.44	7-A2t	-1.00	0.94	2.60	3.05	1.18	0.22
Q5	50.00	50.00	177.85	1.79	3.921	0.53	7-A2t	-1.00	1.21	3.23	3.68	1.54	0.24
Q10	60.00	60.00	178.12	1.98	4.192	0.56	7-A2t	-1.00	1.32	3.48	3.93	1.69	0.25
Q25	80.00	80.00	178.60	2.32	4.665	0.63	7-A2t	-1.00	1.53	3.92	4.37	1.99	0.27
Q50	95.00	95.00	178.91	2.55	4.979	0.67	7-A2t	-1.00	1.68	4.21	4.66	2.20	0.28
Q100	110.00	110.00	179.20	2.77	5.266	0.71	7-A2t	-1.00	1.81	4.47	4.92	2.40	0.29
Q200	130.00	130.00	179.55	3.05	5.620	0.76	7-A2t	-1.00	1.98	4.78	5.23	2.65	0.30
Q500	155.00	155.00	179.96	3.38	6.027	0.81	7-A2t	-1.00	2.18	5.14	5.59	2.96	0.31
Overtopping	792.22	761.03	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

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 0.67 at Q50 and

0.71 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 178.91 is slightly more than the 1988 headwater elevation of 178.9.
- Q100 headwater elevation of 179.20 is slightly higher than the 1988 headwater elevation of 179.1.

The Q10 model result of 178.12 is approximately 0.3 feet lower than the estimated May 1, 2023 flood elevation of 178.4 +/- and the Q25 model result of 178.60 is 0.2 feet higher than the estimated flood elevation.

Headwater elevations for the Q1.1, Q2, and Q5 recurrence intervals are all below Elevation 178 +/- (measured water surface elevation from the site visit) indicating general consistency with the OHWM.

<u>Sensitivity:</u>

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 over 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 0.117 channel / 0.177 floodplain to sensitivity test values of 0.08 channel / 0.14 floodplain yields a reduction in the Q100 headwater elevation of over 0.4 feet.

Sensitivity testing for the tailwater channel elevation indicates that the model is quite sensitive to this parameter. If the channel thalweg elevation is increased or decreased by 1-ft, the headwater elevation results for Q100 are increased or decreased by nearly 0.7-ft, respectively. This is one of the reasons why additional survey to obtain channel thalweg and bank elevations has been requested.

<u>Proposed Alternatives and Results:</u>

To rehabilitate the existing culvert, two slipline alternatives were modeled to determine 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	15.00	15.00	176.55	1.02	2.12	0.34	7-A2t	NA	0.70	1.41	2.36	1.51	0.19
Q2	30.00	30.00	177.27	1.48	2.84	0.45	7-A2t	NA	0.99	2.10	3.05	1.84	0.22
Q5	50.00	50.00	177.95	1.96	3.52	0.56	7-A2t	NA	1.30	2.73	3.68	2.27	0.24
Q10	60.00	60.00	178.24	2.17	3.81	0.61	7-A2t	NA	1.43	2.98	3.93	2.46	0.25
Q25	80.00	80.00	178.74	2.56	4.31	0.69	7-A2t	NA	1.67	3.42	4.37	2.83	0.27
Q50	95.00	95.00	179.08	2.83	4.65	0.74	7-A2t	NA	1.83	3.71	4.66	3.10	0.28
Q100	110.00	110.00	179.40	3.08	4.97	0.79	7-A2t	NA	1.98	3.97	4.92	3.35	0.29
Q200	130.00	130.00	179.80	3.39	5.37	0.86	7-A2t	NA	2.17	4.28	5.23	3.69	0.30
Q500	155.00	155.00	180.26	3.74	5.83	0.93	7-A2t	NA	2.39	4.64	5.59	4.09	0.31

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	15.00	15.00	176.55	1.01	2.12	0.35	7-A2t	NA	0.67	1.41	2.36	1.50	0.19
Q2	30.00	30.00	177.27	1.49	2.84	0.47	7-A2t	NA	0.96	2.10	3.05	1.87	0.22
Q5	50.00	50.00	177.96	1.99	3.53	0.58	7-A2t	NA	1.27	2.73	3.68	2.32	0.24
Q10	60.00	60.00	178.25	2.21	3.82	0.63	7-A2t	NA	1.40	2.98	3.93	2.53	0.25
Q25	80.00	80.00	178.77	2.62	4.34	0.71	7-A2t	NA	1.65	3.42	4.37	2.93	0.27
Q50	95.00	95.00	179.11	2.90	4.68	0.77	7-A2t	NA	1.81	3.71	4.66	3.21	0.28
Q100	110.00	110.00	179.43	3.17	5.00	0.82	7-A2t	NA	1.97	3.97	4.92	3.48	0.29
Q200	130.00	130.00	179.84	3.48	5.41	0.89	7-A2t	NA	2.17	4.28	5.23	3.83	0.30
Q500	155.00	155.00	180.32	3.84	5.89	0.96	7-A2t	NA	2.40	4.64	5.59	4.26	0.31

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 0.06 feet higher than the existing conditions. At Q100, the deviation is greater at 0.23 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 sliplining alternatives do increase. For Q1.1, the design discharge increases 75% for both alternatives. At Q100, the difference is 45% as the outlet velocity increases from 2.40 ft/s to 3.48 ft/s for the 8’-10” x 6’-1” liner. 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 0.67 to 0.74 and 0.77 for the 9’-4” x 6’-3” and 8’-10” x 6’-1” liner alternatives respectively.

- Q100 HW/D ratio increase from 0.71 to 0.79 and 0.82, 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:	26444.00	**Project Name:**	
Town:	Augusta	**Stream Name:**	Thorne Brook
Route No.	ME17 - Eastern Ave.	**Bridge Name:**	Thorne Brook
Asset ID:	2850	**Analysis by:**	csh
Lat:	44.30192 **Long:** -69.73770	**Date:**	9/23/2022

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	2.46	0.95	608.0	Watershed Area *DRNAREA*
W	0.43	0.2	105.7	Wetlands area (by NWI)
P_c	440167	4904916		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

STORAGE	18.32	
STORNWI	17.38	NWI Wetlands %
SANDGRAVF	0.00	sand & gravel aquifer as decimal fraction of watershed A
ELEV	213	mean basin elevation (ft)
BSLDEM10M	5.21	mean basin slope (%)
COASTDIST	59.00	distance from the coast (mi)
ELEVMAX	311	maximum basin elevation (ft)
LC06WATER	0	percent of drainage basin land cover as open water
PRECIP	42.8	mean annual precipitation
STATSGOA	1.69	mean basin percentage of hydrological soil group A

References:

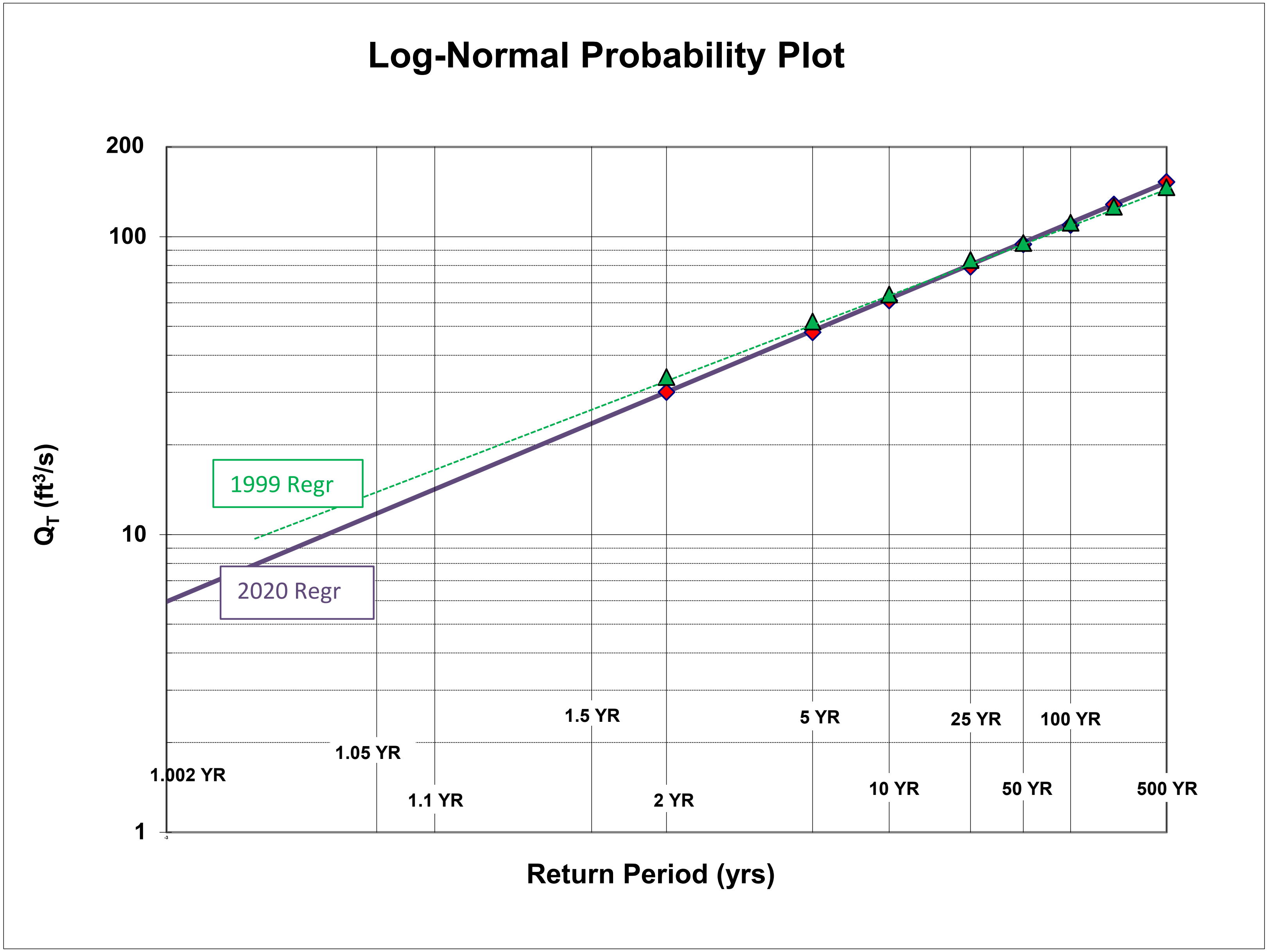
Hodgkins, G.A., 1999.
Estimating the magnitude of peak flows for streams in Maine for Selected Recurrence Intervals
WRIR 99-4008, USGS Augusta, ME

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			14	***15***
2	2.93	34	30	***30***
5	3.67	52	48	***50***
10	4.29	64	61	***60***
25	5.14	83	79	***80***
50	5.78	95	94	***95***
100	6.46	111	110	***110***
200	7.24	126	128	***130***
500	8.42	147	153	***155***

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
200
100
10
1
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:	26444.00	**Project Name:**	0
Town:	Augusta	**Stream Name:**	Thorne Brook
Route No.	ME17 - Eastern Ave.	**Bridge Name:**	Thorne Brook
Asset ID:	2850	**Analysis by:**	csh
Lat:	44.30192 **Long:** -69.73770	**Date:**	9/23/2022

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

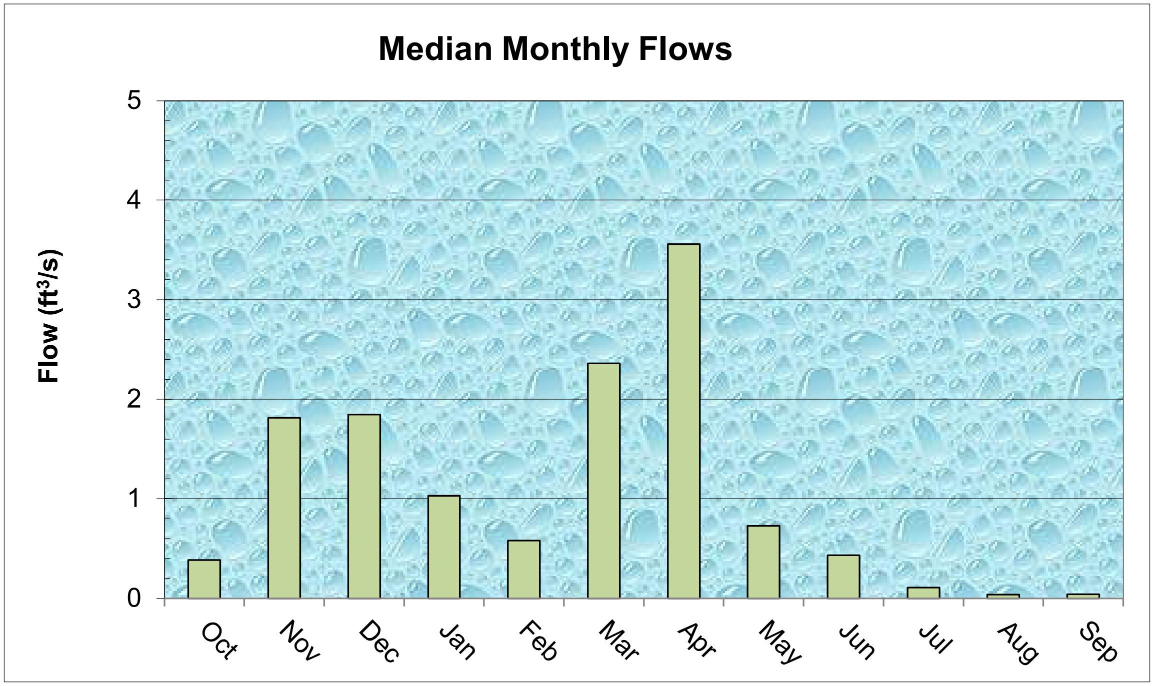
MAINE MONTHLY MEDIAN FLOWS and HYDRAULIC GEOMETRY BY USGS REGRESSION EQUATIONS (2004, 2013, 2015)

	Value	Variable	Explanation
	0.95	*A*	Area (mi^2)
440167	4904916	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	58.76	*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	**1.03**	0.0292
Feb	**0.58**	0.0164
Mar	**2.36**	0.0668
Apr	**3.56**	0.1008
May	**0.73**	0.0207
Jun	**0.43**	0.0123
Jul	**0.11**	0.0031
Aug	**0.04**	0.0010
Sep	**0.04**	0.0012
Oct	**0.38**	0.0109
Nov	**1.81**	0.0514
Dec	**1.85**	0.0523

Q_{bf}	**4.9**	
ann avg	**2.2**	
ann med	**0.8**	
$Q_{1.002}$	**5.9**	
$Q_{1.01}$	**8.1**	
$Q_{1.05}$	**11.8**	
Q_{bf}	**24.2**	assume v = 4ft/s

W_{bf}	**10.3**	estimated bankfull width (ft)
d_{bf}	**0.6**	estimated bankfull depth (ft)
A_{bf}	**4.4**	estimated bankfull flow area (ft^2)

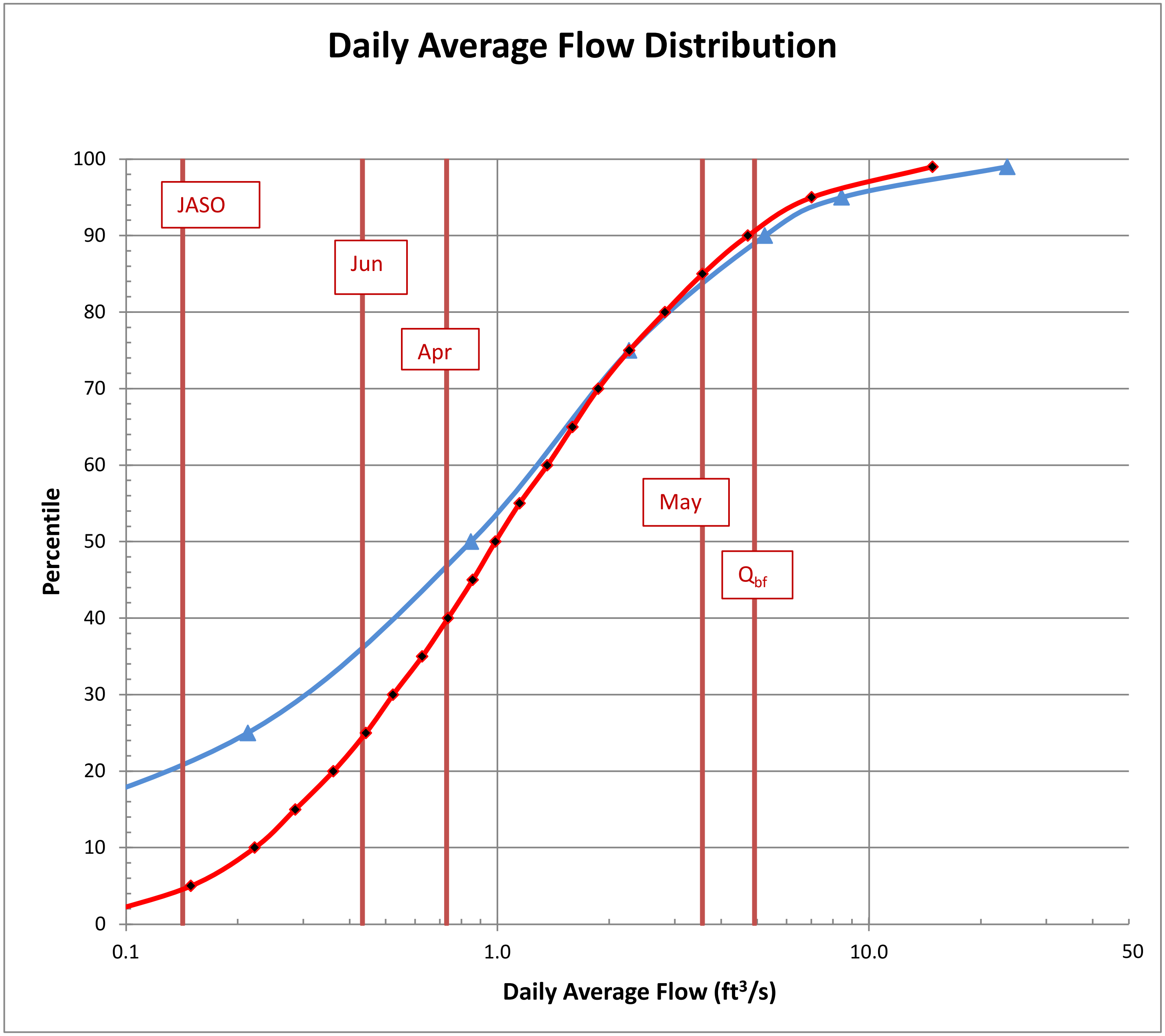


References

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)		1.0
	Q (ft^3/s)	
Pctl	**Median**	**84^{th} pctl**
1.00E-06	0.00	0.00
1	0.08	0.14
5	0.15	0.24
10	0.22	0.33
15	0.29	0.42
20	0.36	0.51
25	0.44	0.59
30	0.52	0.68
35	0.63	0.77
40	0.74	0.89
45	0.86	1.00
50	0.99	1.18
55	1.15	1.38
60	1.36	1.62
65	1.59	1.89
70	1.87	2.20
75	2.26	2.65
80	2.82	3.16
85	3.56	4.05
90	4.72	5.43
95	7.00	8.45
99	14.81	19.50

Q_{bf}	4.9
$Q_{1.002}$	5.9
$Q_{1.1}$	14.2
Q_2	30.1

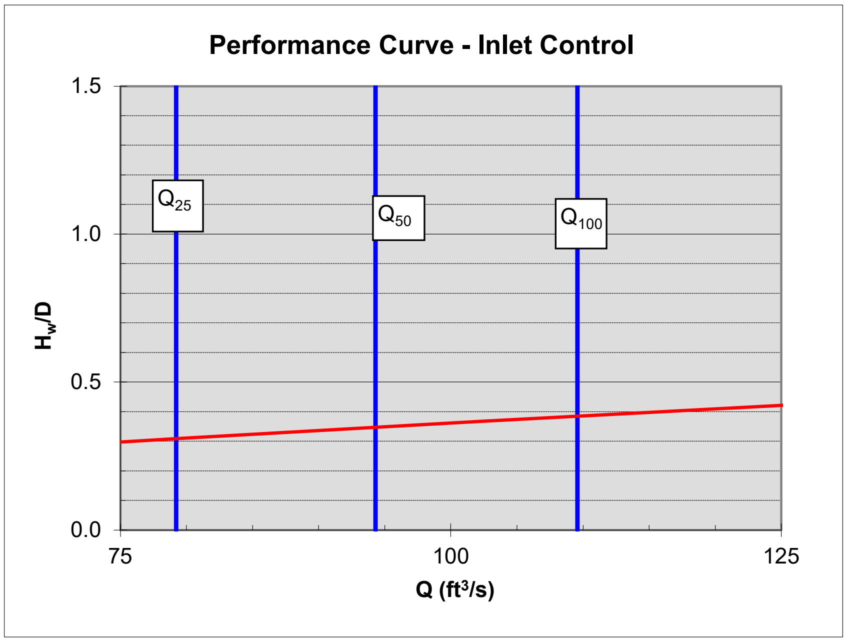
NOTE: ***This page is for preliminary sizing only.***
Final design should be done with HY8 or HDS-5

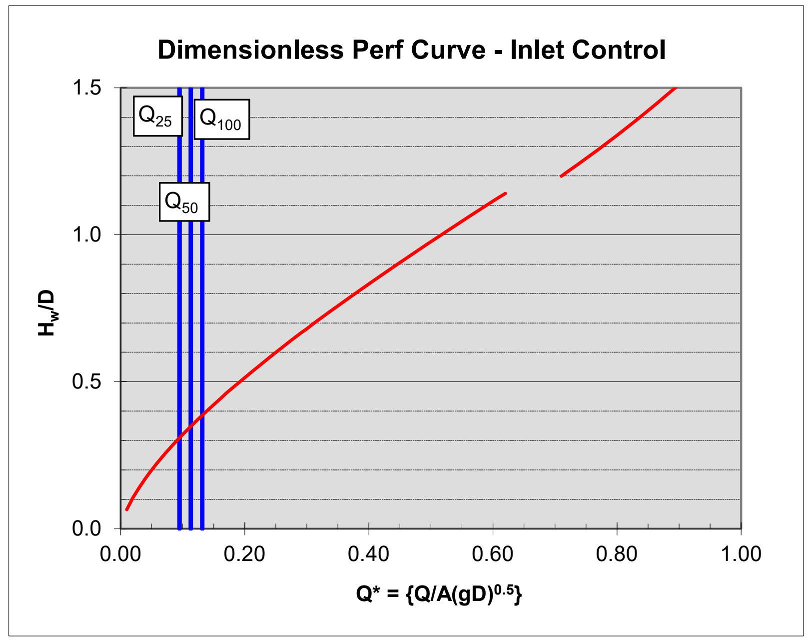
Note:
culvert dimensions are for open flow area; adjust for lost capacity due to embedding / backfilling (min {2' / 25% rise} embedment)
Finish analysis with HY-8

Preliminary Culvert Sizing - Round & Box Culverts

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

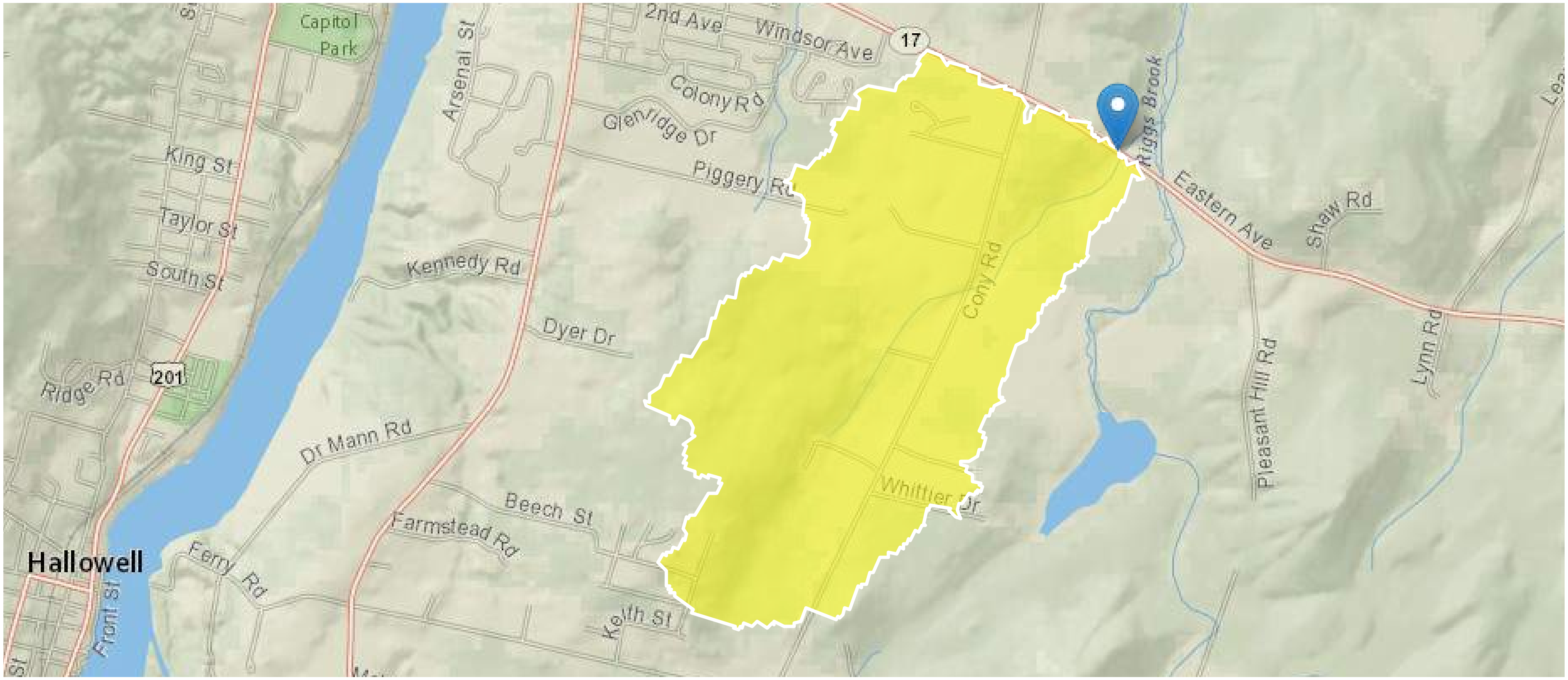
Shape:	Box					
Inlet Type:	Circ RCP Proj		Q_{25}	79.2		
D or R (ft)	6	*diam / rise*	Q_{50}	94.3	trial D / R =	4.8
w (ft)	*10*	*box span*	Q_{100}	109.6	trial w: BFW =	10.3
Slope (ft/ft)	0.01					
A (ft^2)	60.00					
g (ft/s^2)	32.2					





Augusta 26444 Thorne Brook Br #2850

Region ID: ME
Workspace ID: ME20220923163949853000
Clicked Point (Latitude, Longitude): 44.30191, -69.73780
Time: 2022-09-23 12:40:12 -0400



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
BSLDEM10M	Mean basin slope computed from 10 m DEM	5.21	percent
CENTROIDX	Basin centroid horizontal (x) location in state plane coordinates	440167.43	meters
CENTROIDY	Basin centroid vertical (y) location in state plane units	4904916.33	meters
COASTDIST	Shortest distance from the coastline to the basin centroid	59	miles
DRNAREA	Area that drains to a point on a stream	0.95	square miles
ELEV	Mean Basin Elevation	213.2	feet
ELEVMAX	Maximum basin elevation	311.1	feet
I24H100Y	Maximum 24-hour precipitation that occurs on average once in 100 years	6.46	inches
I24H10Y	Maximum 24-hour precipitation that occurs on average once in 10 years	4.29	inches
I24H200Y	Maximum 24-hour precipitation that occurs on average once in 200 years	7.24	inches
I24H25Y	Maximum 24-hour precipitation that occurs on average once in 25 years	5.14	inches
I24H2Y	Maximum 24-hour precipitation that occurs on average once in 2 years - Equivalent to precipitation intensity index	2.93	inches
I24H500Y	Maximum 24-hour precipitation that occurs on average once in 500 years	8.42	inches
I24H50Y	Maximum 24-hour precipitation that occurs on average once in 50 years	5.78	inches
I24H5Y	Maximum 24-hour precipitation that occurs on average once in 5 years	3.67	inches
JULAVPRE	Mean July Precipitation	3.37	inches
LC06WATER	Percent of open water, class 11, from NLCD 2006	0	percent
LC11DEV	Percentage of developed (urban) land from NLCD 2011 classes 21-24	32.1	percent
LC11IMP	Average percentage of impervious area determined from NLCD 2011 impervious dataset	8.73	percent
PCTSNDGRV	Percentage of land surface underlain by sand and gravel deposits	0	percent

Parameter Code	Parameter Description	Value	Unit
PRDECFEB90	Basin average mean precipitation for December to February from PRISM 1961-1990	9.91	inches
PRECIP	Mean Annual Precipitation	42.8	inches
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	1.69	percent
STORAGE	Percentage of area of storage (lakes ponds reservoirs wetlands)	18.32	percent
STORNWI	Percentage of strorage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	17.38	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	0.95	square miles	0.26	5680
I24H2Y	24 Hour 2 Year Precipitation	2.93	inches	1.92	4.17
STORAGE	Percent Storage	18.32	percent	0	29.4
I24H5Y	24 Hour 5 Year Precipitation	3.67	inches	2.48	5.38
I24H10Y	24 Hour 10 Year Precipitation	4.29	inches	2.84	6.38
I24H25Y	24 Hour 25 Year Precipitation	5.14	inches	3.3	7.75
I24H50Y	24 Hour 50 Year Precipitation	5.78	inches	3.65	8.79

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
I24H100Y	24 Hour 100 Year Precipitation	6.46	inches	3.99	9.88
I24H200Y	24 Hour 200 YearPrecipitation	7.24	inches	5.26	11.1
I24H500Y	24 Hour 500 Year Precipitation	8.42	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	30.1	ft^3/s	16	56.6	39.1
20-percent AEP flood	48	ft^3/s	25.9	89	38.1
10-percent AEP flood	61.4	ft^3/s	32.6	115	38.9
4-percent AEP flood	79.8	ft^3/s	41.8	152	39.9
2-percent AEP flood	94.4	ft^3/s	48.7	183	39.7
1-percent AEP flood	110	ft^3/s	57	212	40.7
0.5-percent AEP flood	129	ft^3/s	64.6	258	42.8
0.2-percent AEP flood	154	ft^3/s	76	312	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	0.95	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	213.2	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	0.00104	ft^3/s
5 Percent Duration	0.00907	ft^3/s
10 Percent Duration	0.0307	ft^3/s
25 Percent Duration	0.213	ft^3/s
50 Percent Duration	0.848	ft^3/s
75 Percent Duration	2.26	ft^3/s
90 Percent Duration	5.24	ft^3/s
95 Percent Duration	8.43	ft^3/s
99 Percent Duration	23.6	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	0.95	square miles	14.9	1419
SANDGRAVAF	Fraction of Sand and Gravel Aquifers	0	dimensionless	0	0.212
ELEV	Mean Basin Elevation	213.2	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	2.16	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)

Application Version: 4.10.1
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.1

Purpose:

The purpose of this calculation is to determine the inputs used in HY-8 modeling for both the proposed and existing culverts as well as determine if the resulting hydraulic data meets design requirements. HY-8 Version 8.0.0 (May 29, 2024) was used for the analysis.

References:

- WSP Project Scoping Document *MaineDOT Structural GCA Contract - WIN 026444.00 - Augusta, Thorne Brook Bridge (BR. # 2850)* dated January 10, 2023
- MaineDOT Hydrology *Augusta-26444-Thorne_Br-2850Hydro_Rpt.PDF* dated 9/23/2022
- 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
- *Thorne Bridge & Spaulding Bridge* Record Plans dated January 1988

Site and Assumptions:

Thorne Brook, also known as Riggs Brook, flows from south to north. Thorne Brook Bridge is identified in the image on the right. Downstream of the structure is a beaver dam as well as a junction with the nearby Spaulding Brook. Thorne Brook appears relatively narrow and shallow in aerial imagery but site photographs and Google Street View imagery from various dates indicates that water regularly overbanks the stream and onto the floodplain or wetlands area. The overbanking may be the typical condition due to the beaver dam downstream.

Per Task 4 of the WSP project scope titled *MaineDOT Structural GCA Contract - WIN 026444.00 - Augusta, Thorne Brook Bridge (BR. # 2850)* dated January 10, 2023, the confluence of the two brooks is **NOT** included in the hydraulic analysis of this project. Additionally, the effect of the beaver dam is also not considered in the hydraulic analysis of the project.

Discharge Data:

User-defined discharge data was provided by MaineDOT in the hydrology file "Augusta-26444-Thorne_Br-2850Hydro_Rpt.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	2021	Q_T (ft^3/s) Design
1.1			14	15
2	2.93	34	30	30
5	3.67	52	48	50
10	4.29	64	61	60
25	5.14	83	79	80
50	5.78	95	94	95
100	6.46	111	110	110
200	7.24	126	128	130
500	8.42	147	153	155

From MaineDOT Hydrology Report

Tailwater Data:

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. No stream bank and thalweg survey information for the waterway has been collected due to the flooding at the site which is likely caused by the beaver dam.

Channel Thalweg

Until ground survey of the existing channel is obtained, the channel thalweg will be assumed to be the same elevation as the proposed culvert invert. The proposed culvert invert will be set equal to the lesser of the existing culvert invert elevations. The existing invert elevations at the culvert ends are in error. Additional invert elevations were obtained at locations near the start of the mitered culvert ends and these are consistent with the culvert rise. These points will be used to extrapolate the elevations at the ends of the culvert.

Upstream culvert invert elevations near start of miter: $Elev_{exist.inv.upstream.initial} := 173.98\ \text{ft}$

Downstream culvert invert elevations near start of miter: $Elev_{exist.inv.downstream.initial} := 174.49\ \text{ft}$

Distance between surveyed invert elevations: $d_{surveyed.invert} := 54.98\ \text{ft}$

Existing culvert slope:

$$slope_{exist} := \frac{Elev_{exist.inv.upstream.initial} - Elev_{exist.inv.downstream.initial}}{d_{surveyed.invert}} = -0.009$$

Negative indicates culvert slopes upward in the downstream direction.

Distance between surveyed culvert end and invert elevation (upstream): $d_{survey.upstream} := 5.85\ \text{ft}$

Distance between surveyed culvert end and invert elevation (downstream): $d_{survey.downstream} := 6.80\ \text{ft}$

Approximate surveyed existing culvert length:

$$l_{culvert.exist.survey} := d_{surveyed.invert} + d_{survey.upstream} + d_{survey.downstream} = 67.63\ \text{ft}$$

The actual culvert length differs from that indicated in the existing plans. This may have been reduced due to site limitations.

Extrapolated upstream culvert elevation: $Elev_{exist.inv.upstream} := Elev_{exist.inv.upstream.initial} + slope_{exist} \cdot d_{survey.upstream} = 173.93\ \text{ft}$

Extrapolated downstream culvert elevation: $Elev_{exist.inv.downstream} := Elev_{exist.inv.downstream.initial} - slope_{exist} \cdot d_{survey.downstream} = 174.55\ \text{ft}$

Check these elevations versus the invert elevation from the existing plans. The existing plans indicate that the flow line was set level at El. 174.8. 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): $Elev_{flow.NGVD29} := 174.8\ \text{ft}$

Plan flowline elevation (NAVD 88): $Elev_{flow.NAVD88} := Elev_{flow.NGVD29} + Datum = 174.1\ \text{ft}$

The NAVD 88 adjusted plan elevation is between the ground survey invert elevations, therefore it is reasonably consistent with them. The differing invert elevations indicate either differential settlement or incorrect installation. Since the existing conditions hydraulic model should be calibrated against known data, the flowline elevation from the 1988 plans will be used to calibrate the model. However, to be conservative for grading, the thalweg will be assumed to be the lower of the surveyed inverts.

Thalweg elevation for proposed project plans: $Elev_{thalweg.plans} := \min\left(Elev_{exist.inv.upstream}, Elev_{exist.inv.downstream}\right) = 173.93\ \text{ft}$

Approximate thalweg elevation for hydraulic model: $Elev_{thalweg} := Elev_{flow.NAVD88} = 174.10\ \text{ft}$

Brook Dimensions

Since survey data regarding the brook dimensions is not available, the channel dimensions will be approximated using the estimated bankfull width and depth provided in the hydrology report. The channel section will be considered triangular with a top width equal to bankfull width and a center-of-channel depth equal to bankfull depth. This will produce a slightly smaller bankfull flow area than that shown in the hydrology report.

The streambed is assumed to slope linearly up from the thalweg to bankfull width.

W_{bf}	10.3	estimated bankfull width (ft)
d_{bf}	0.6	estimated bankfull depth (ft)
A_{bf}	4.4	estimated bankfull flow area (ft^2)

MaineDOT Hydrology Report

Estimated bankfull width: $b_{bank} := 10.3\ \text{ft}$

Estimated bankfull depth: $d_{bank} := 0.6\ \text{ft}$

Approximate elevation stream banks: $Elev_{bank} := Elev_{thalweg} + d_{bank} = 174.7\ \text{ft}$

Floodplain Dimensions

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

Survey information downstream of the structure in the area of the beaver dam was used to measure the width of the floodplain. The width of the floodplain is in excess of 200' just downstream of the structure when measuring between 180' contours. 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 183'.

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

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. The excess elevations are still lower than the existing edge of pavement elevations.

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

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

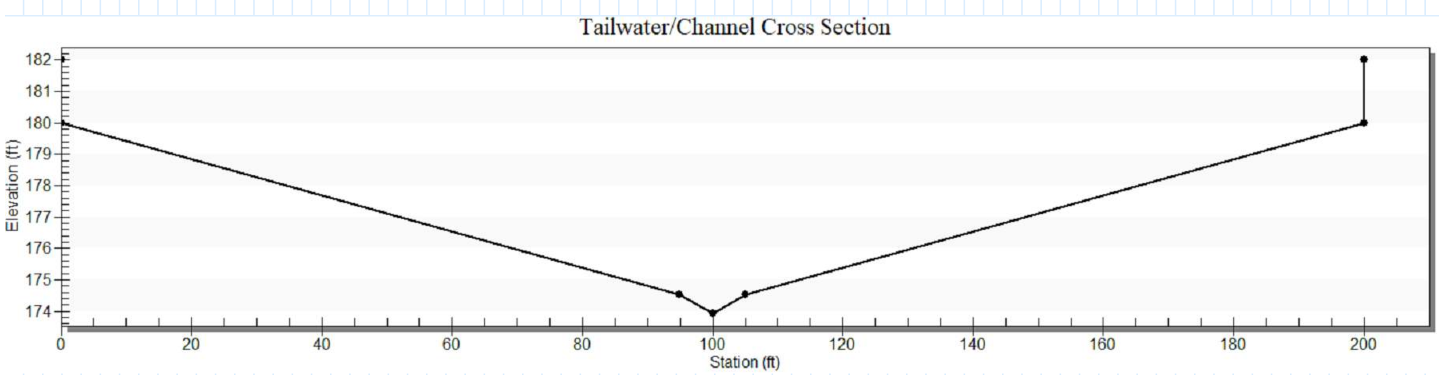
Google Street View May 2019 - Downstream, No Beaver Dam

Google Street View May 2023 - Downstream, With Beaver Dam

May 1, 2023 Storm - MaineDOT Photo - Downstream, Beaver dam submerged

Coordinates for Tailwater Table

$$Coords := \begin{bmatrix} 0\ \text{ft} & Elev_{excess} \\ 0\ \text{ft} & Elev_{flood} \\ \dfrac{b_{flood}}{2} - \dfrac{b_{bank}}{2} & Elev_{bank} \\ \dfrac{b_{flood}}{2} & Elev_{thalweg} \\ \dfrac{b_{flood}}{2} + \dfrac{b_{bank}}{2} & Elev_{bank} \\ b_{flood} & Elev_{flood} \\ b_{flood} & Elev_{excess} \end{bmatrix} = \begin{bmatrix} 0.00 & 182.00 \\ 0.00 & 180.00 \\ 94.85 & 174.70 \\ 100.00 & 174.10 \\ 105.15 & 174.70 \\ 200.00 & 180.00 \\ 200.00 & 182.00 \end{bmatrix} \text{ft}$$



Google Satellite Imagery:

FEMA Flood Map:

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*

Because the depth of flow in a flat wide wetland area is shallow in relation to the size of the roughness elements, n values can be large. (Page 6, Adjustment Factors for Channel n Values)

Channel n Value:

Downstream at Outlet:

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

428 Dow Highway
Eliot, ME 03903
wsp.com

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

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.

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 an assumed moderate irregularity — *USGS Table 2 (see next page)*

Channel variation factor: $n_2 := 0.005$ — Based on occasional variation — *USGS Table 2 (see next page)*

Obstruction factor: $n_3 := 0.030$ — Assume appreciable obstruction as there are likely remnant beaver building materials throughout stream. Note this does not account for the beaver dam itself which is excluded from the analysis. — *USGS Table 2 (see next page)*

Vegetation factor: $n_4 := 0.05$ — Very large amount of vegetation with weeds and cattails. Channel is not visible in FEMA map due to vegetation. — *USGS Table 2 (see next page)*

In Google satellite imagery it can be seen that the channel downstream of the structure has minor meandering.

Meander factor: $m_{ch} := 1.00$ — minor meander — *USGS Table 2 (see next page)*

Manning's coefficient for channel: $n_{channel} := \left(n_b + n_1 + n_2 + n_3 + n_4\right) \cdot m_{ch} = 0.117$

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:

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 minor irregularity with hummocks (high points) indicated by bushes and trees	*USGS Table 3 (see next page)*
Variation factor:	$n_{2.fp} := 0.000$	Not applicable	*USGS Table 3 (see next page)*
Obstruction factor:	$n_{3.fp} := 0.030$	Similar to channel, account for the potential of material in the flood plain.	*USGS Table 3 (see next page)*
Vegetation factor:	$n_{4.fp} := 0.12$	Extreme amount of vegetation. Flood-plain with marsh grasses, cattails, dense brush and large trees.	*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:

$$n_{fp} := \left(n_{b.fp} + n_{1.fp} + n_{2.fp} + n_{3.fp} + n_{4.fp}\right) \cdot m_{fp} = 0.177$$

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 downstream of the structure is assumed to be similar to that of Spaulding Brook Bridge #2790 as they are located in the same drainage area. Survey of the thalweg was not obtained for Thorne Brook. Since this project has similar topography, is close to and shares a nearby junction with Spaulding Brook, and most importantly because the water surface elevations are tailwater controlled, the channel slope determined for Spaulding Brook will be used here. This slope is very shallow and essentially flat.

Slope of channel: $slope := 0.0003$

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 Thorne Brook.

The existing pipe is assumed 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 known inlet depression due to insufficient ground survey.

Existing inlet elevation: $inlet_{exist} := Elev_{exist.inv.upstream} = 173.93\ \text{ft}$

Existing culvert length: $Length_{exist} := 67\ \text{ft} + 8\ \text{in}$ Approximate based on survey data. Existing plans indicate 72' but it appears the culvert was reduced at Thorne Brook

Existing outlet elevation: $outlet_{exist} := Elev_{exist.inv.downstream} = 174.55\ \text{ft}$

Note that the surveyed outlet invert is at a greater elevation than the inlet. This means there is a negative slope through the culvert.

Existing Roadway Elevation:

Approximate minimum roadway crown elevation near culvert is based on survey point 1227 with an elevation of 184.60.

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: $Q_{100.exist.29} := 179.8\ \text{ft}$ Assumed to be in NGVD 29 datum

Q50 from 1988 existing bridge plans: $Q_{50.exist.29} := 179.6\ \text{ft}$ Assumed to be in NGVD 29 datum

Adjusted Q100 from 1988 existing bridge plans: $Q_{100.exist.88} := Q_{100.exist.29} + Datum = 179.1\ \text{ft}$ Adjusted to NAVD 88 datum

Adjusted Q50 from 1988 existing bridge plans: $Q_{50.exist.88} := Q_{50.exist.29} + Datum = 178.9\ \text{ft}$ Adjusted to NAVD 88 datum

May 1, 2023 Storm Data:

The May 1, 2023 photo 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. 4.17" recorded on 5/1/2023 at the Augusta Landfill (1.8 miles from bridge site)
2. 3.00" recorded on 4/30/2023 at the Augusta Airport (3.25 miles from bridge site)

The hydrology report indicates a 10-year storm has 4.29" of precipitation in 24 hours and a 2-year storm has 2.93" in 24 hours. Therefore, the May 1 storm event is assumed to correlate to an estimated recurrence interval of at least Q2 to Q10.

The May 1, 2023 estimated flood elevation is 178.4 +/- from the Spaulding Brook Bridge #2790 hydraulics analysis. See Spaulding Brook Bridge #2790 hydraulic analysis for additional information on how this elevation was estimated.

Site Visit Data:

During 12/13/2022 site visit, culvert rise was verified to be approximately 7'-4" and the water surface was measured to be approximately 3'-2 1/2" below the crown of the culvert. The water surface was observed to be higher than the Ordinary High Water (OHW) rust staining on the culvert.

Measured water depth at culvert inlet: $d_{site} := (7\ \text{ft} + 4\ \text{in}) - (3\ \text{ft} + 2.5\ \text{in}) = 4.13\ \text{ft}$

Estimated water surface elevation in culvert: $Elev_{site} := d_{site} + inlet_{exist} = 178.05\ \text{ft}$ Say Elevation 178 +/-

Note that a beaver dam is reportedly causing backwater at this location. Therefore, the measured water surface elevation provides an understanding that OHW is below elevation 178.05, but is not very useful for calibrating this model. Use the 1988 plan data and the May 1, 2023 storm data for model calibration.

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, and the May 1, 2023 storm observations. 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	15.00	15.00	176.48	0.97	2.382	0.32	7-H2t	-1.00	0.66	2.36	2.36	0.66	0.19
Q2	30.00	30.00	177.18	1.39	3.084	0.42	7-H2t	-1.00	0.94	3.05	3.05	0.98	0.22
Q5	50.00	50.00	177.84	1.81	3.741	0.50	7-H2t	-1.00	1.21	3.68	3.68	1.33	0.24
Q10	60.00	60.00	178.11	2.00	4.012	0.54	7-H2t	-1.00	1.32	3.93	3.93	1.49	0.25
Q25	80.00	80.00	178.58	2.35	4.483	0.60	7-H2t	-1.00	1.53	4.37	4.37	1.78	0.27
Q50	95.00	95.00	178.90	2.58	4.796	0.65	7-H2t	-1.00	1.68	4.66	4.66	1.99	0.28
Q100	110.00	110.00	179.18	2.81	5.082	0.68	7-H2t	-1.00	1.81	4.92	4.92	2.18	0.29
Q200	130.00	130.00	179.53	3.09	5.435	0.73	7-H2t	-1.00	1.98	5.23	5.23	2.44	0.30
Q500	155.00	155.00	179.94	3.42	5.840	0.79	7-H2t	-1.00	2.18	5.59	5.59	2.75	0.31
Overtopping	775.00	742.39	184.75	10.31	10.646	1.43	7-H2c	-1.00	5.28	5.28	0.00	13.82	0.00

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 = 179.18 is just above the datum adjusted 1988 plan elevation of 179.1.
Q50 = 178.90 is the same as the datum adjusted 1988 plan elevation of 178.9.

The Q10 result of El. 178.11 is also reasonably consistent with the May 1, 2023 estimated flood elevation of 178.4 +/-.

The Q1.1 and Q2 elevations are both lower than the site measured estimate of 178 +/-, so this is consistent with the model.

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	15.00	15.00	176.49	0.96	2.558	0.34	7-A2t	-1.00	0.66	1.91	2.36	0.86	0.19
Q2	30.00	30.00	177.19	1.37	3.262	0.44	7-A2t	-1.00	0.94	2.60	3.05	1.18	0.22
Q5	50.00	50.00	177.85	1.79	3.921	0.53	7-A2t	-1.00	1.21	3.23	3.68	1.54	0.24
Q10	60.00	60.00	178.12	1.98	4.192	0.56	7-A2t	-1.00	1.32	3.48	3.93	1.69	0.25
Q25	80.00	80.00	178.60	2.32	4.665	0.63	7-A2t	-1.00	1.53	3.92	4.37	1.99	0.27
Q50	95.00	95.00	178.91	2.55	4.979	0.67	7-A2t	-1.00	1.68	4.21	4.66	2.20	0.28
Q100	110.00	110.00	179.20	2.77	5.266	0.71	7-A2t	-1.00	1.81	4.47	4.92	2.40	0.29
Q200	130.00	130.00	179.55	3.05	5.620	0.76	7-A2t	-1.00	1.98	4.78	5.23	2.65	0.30
Q500	155.00	155.00	179.96	3.38	6.027	0.81	7-A2t	-1.00	2.18	5.14	5.59	2.96	0.31
Overtopping	792.22	761.03	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

Sensitivity Analysis:

Determine sensitivity of existing conditions model for input data.

Tailwater Slope:

Assume tailwater slope of 0.01. This lowers Q100 by 1.71 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	15.00	15.00	175.60	0.96	1.668	0.22	7-A2t	-1.00	0.66	0.80	1.25	2.86	0.86
Q2	30.00	30.00	176.04	1.37	2.105	0.28	7-A2t	-1.00	0.94	1.16	1.61	3.36	0.95
Q5	50.00	50.00	176.48	1.79	2.550	0.34	7-A2t	-1.00	1.21	1.49	1.94	3.98	1.03
Q10	60.00	60.00	176.67	1.98	2.742	0.37	7-A2t	-1.00	1.32	1.62	2.07	4.27	1.06
Q25	80.00	80.00	177.02	2.32	3.091	0.42	7-A2t	-1.00	1.53	1.85	2.30	4.80	1.11
Q50	95.00	95.00	177.26	2.55	3.331	0.45	7-A2t	-1.00	1.68	2.00	2.45	5.16	1.14
Q100	110.00	110.00	177.49	2.77	3.558	0.48	7-A2t	-1.00	1.81	2.13	2.58	5.51	1.17
Q200	130.00	130.00	177.77	3.05	3.844	0.52	7-A2t	-1.00	1.98	2.29	2.74	5.95	1.21
Q500	155.00	155.00	178.11	3.38	4.183	0.56	7-A2t	-1.00	2.18	2.48	2.93	6.46	1.25
Overtopping	792.22	761.07	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	5.34	14.00	1.75

Assume tailwater slope of 0.0001. This increases Q100 by 1.03 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	15.00	15.00	177.00	0.96	3.074	0.41	7-A2t	-1.00	0.66	2.44	2.89	0.64	0.12
Q2	30.00	30.00	177.86	1.37	3.926	0.53	7-A2t	-1.00	0.94	3.28	3.73	0.91	0.14
Q5	50.00	50.00	178.65	1.79	4.719	0.64	7-A2t	-1.00	1.21	4.05	4.50	1.20	0.16
Q10	60.00	60.00	178.97	1.98	5.043	0.68	7-A2t	-1.00	1.32	4.37	4.82	1.34	0.16
Q25	80.00	80.00	179.54	2.32	5.608	0.76	7-A2t	-1.00	1.53	4.91	5.36	1.59	0.17
Q50	95.00	95.00	179.91	2.55	5.979	0.81	7-A2t	-1.00	1.68	5.26	5.71	1.77	0.18
Q100	110.00	110.00	180.23	2.77	6.299	0.85	7-A2t	-1.00	1.81	5.56	6.01	1.96	0.19
Q200	130.00	130.00	180.59	3.05	6.664	0.90	7-A2t	-1.00	1.98	5.89	6.34	2.21	0.20
Q500	155.00	155.00	181.03	3.38	7.098	0.96	7-A2t	-1.00	2.18	6.27	6.72	2.51	0.21
Overtopping	792.22	761.03	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	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 believed to be reasonable, but additional ground survey data along the channel is required to improve its accuracy.

Tailwater Channel Elevations:

Increase tailwater channel section elevations by 1 foot. This increases the Q100 by 0.68 feet 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	15.00	15.00	177.38	0.96	3.454	0.47	7-A2t	-1.00	0.66	2.82	2.27	0.54	0.18
Q2	30.00	30.00	178.03	1.37	4.103	0.55	7-A2t	-1.00	0.94	3.46	2.91	0.85	0.21
Q5	50.00	50.00	178.64	1.79	4.709	0.63	7-A2t	-1.00	1.21	4.04	3.49	1.21	0.23
Q10	60.00	60.00	178.89	1.98	4.959	0.67	7-A2t	-1.00	1.32	4.28	3.73	1.36	0.24
Q25	80.00	80.00	179.33	2.32	5.395	0.73	7-A2t	-1.00	1.53	4.69	4.14	1.66	0.25
Q50	95.00	95.00	179.61	2.55	5.684	0.77	7-A2t	-1.00	1.68	4.96	4.41	1.87	0.26
Q100	110.00	110.00	179.88	2.77	5.950	0.80	7-A2t	-1.00	1.81	5.20	4.65	2.08	0.27
Q200	130.00	130.00	180.20	3.05	6.272	0.84	7-A2t	-1.00	1.98	5.48	4.93	2.34	0.28
Q500	155.00	155.00	180.53	3.38	6.603	0.89	7-A2t	-1.00	2.18	5.76	5.21	2.68	0.30
Overtopping	792.22	761.02	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

Decrease tailwater channel section elevations by 1 foot. This lowers the Q100 by 0.69 feet 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	15.00	15.00	175.68	0.96	1.754	0.24	7-A2t	-1.00	0.66	0.99	2.44	2.10	0.21
Q2	30.00	30.00	176.39	1.37	2.458	0.33	7-A2t	-1.00	0.94	1.71	3.16	1.98	0.23
Q5	50.00	50.00	177.08	1.79	3.150	0.42	7-A2t	-1.00	1.21	2.38	3.83	2.18	0.26
Q10	60.00	60.00	177.37	1.98	3.438	0.46	7-A2t	-1.00	1.32	2.66	4.11	2.30	0.27
Q25	80.00	80.00	177.87	2.32	3.942	0.53	7-A2t	-1.00	1.53	3.12	4.57	2.55	0.28
Q50	95.00	95.00	178.21	2.55	4.275	0.58	7-A2t	-1.00	1.68	3.43	4.88	2.73	0.29
Q100	110.00	110.00	178.51	2.77	4.582	0.62	7-A2t	-1.00	1.81	3.71	5.16	2.91	0.30
Q200	130.00	130.00	178.89	3.05	4.958	0.67	7-A2t	-1.00	1.98	4.04	5.49	3.14	0.31
Q500	155.00	155.00	179.32	3.38	5.390	0.73	7-A2t	-1.00	2.18	4.42	5.87	3.42	0.33
Overtopping	792.22	761.00	184.74	10.57	10.810	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

The model is sensitive to the channel elevation assumptions. Assuming a tailwater elevation at the elevation indicated in the 1988 bridge plans, adjusted for datum shift, appears reasonable given the limited data available.

Manning's N for Tailwater:

Decrease Manning's N to 0.08 for channel and 0.14 for flood plain. This lowers Q100 by 0.44 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	15.00	15.00	176.24	0.96	2.311	0.31	7-A2t	-1.00	0.66	1.66	2.11	1.04	0.25
Q2	30.00	30.00	176.89	1.37	2.958	0.40	7-A2t	-1.00	0.94	2.28	2.73	1.38	0.28
Q5	50.00	50.00	177.50	1.79	3.569	0.48	7-A2t	-1.00	1.21	2.85	3.30	1.77	0.31
Q10	60.00	60.00	177.75	1.98	3.822	0.51	7-A2t	-1.00	1.32	3.09	3.54	1.94	0.32
Q25	80.00	80.00	178.19	2.32	4.264	0.57	7-A2t	-1.00	1.53	3.49	3.94	2.26	0.34
Q50	95.00	95.00	178.49	2.55	4.558	0.61	7-A2t	-1.00	1.68	3.75	4.20	2.48	0.35
Q100	110.00	110.00	178.76	2.77	4.829	0.65	7-A2t	-1.00	1.81	3.99	4.44	2.69	0.36
Q200	130.00	130.00	179.09	3.05	5.162	0.70	7-A2t	-1.00	1.98	4.28	4.73	2.96	0.37
Q500	155.00	155.00	179.48	3.38	5.546	0.75	7-A2t	-1.00	2.18	4.60	5.05	3.28	0.38
Overtopping	792.22	760.99	184.74	10.57	10.810	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

As can be seen, all of the above parameters have noticeable effects on the Existing Conditions model results. The tailwater slope and channel elevations have a potentially greater influence than the Manning's N assumptions. In combination, these variables greatly affect the model and its results.

Assumptions for each of the parameters have been made based on limited data. If new data becomes available, these assumptions can be revisited.

Proposed Culvert Data:

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

Culvert Span

The culvert span will be based on the estimated bankfull width provided in the MaineDOT hydrology report. The span will be increased to the nearest even whole foot.

Estimated bankfull width: $b_{bank} = 10.3\ \text{ft}$ *MaineDOT hydrology report*

Culvert span: $Span := 12\ \text{ft}$ This site is being designed to 1.0 x bankfull width and not 1.2x bankfull width since it is not a critical fish habitat.

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 Rise

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

Culvert depth of streambed fill: $d_{stream.fill} := 24\ \text{in}$ Culvert embedment

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: $n_{culvert2} := \left(n_b + n_1 + n_2 + n_3\right) = 0.067$ Channel Manning's n, determined earlier. Vegetation factor and meander factor omitted through the culvert.

Misc. Culvert Configuration

The culvert will be straight and assumed to not have a significant 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 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 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: $Elev_{invert} := Elev_{thalweg.plans} - d_{stream.fill} = 171.93\ \text{ft}$ Culvert placed level

Streambed inlet elevation: $Elev_{inlet} := Elev_{thalweg.plans} = 173.93\ \text{ft}$

Culvert length: $L_{culvert} := 58\ \text{ft}$

Streambed outlet elevation: $Elev_{outlet} := Elev_{thalweg.plans} = 173.93\ \text{ft}$ 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.

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.

Vertical Curve Length: $VCL := 250 \text{ ft}$

PVC Station: $Sta_{PVC} := 11448.00 \text{ ft}$

PVC Elevation: $El_{PVC} := 183.98 \text{ ft}$

PVI Station: $Sta_{PVI} := Sta_{PVC} + \frac{VCL}{2} = 11573 \text{ ft}$

PVI Elevation: $El_{PVI} := 184.28 \text{ ft}$

PVT Station: $Sta_{PVT} := Sta_{PVC} + VCL = 11698 \text{ ft}$

PVT Elevation: $El_{PVT} := 192.98 \text{ ft}$

Grade 1 Slope: $G1 := \frac{El_{PVI} - El_{PVC}}{Sta_{PVI} - Sta_{PVC}} = 0.24\%$ Matches plan callout of 0.24%, OK

Grade 2 Slope: $G2 := \frac{El_{PVT} - El_{PVI}}{Sta_{PVT} - Sta_{PVI}} = 6.96\%$ Matches plan callout of 6.96%, OK

Check Station 115+25 vs PDR plans: $x_{11525} := 11525 \text{ ft} - Sta_{PVC} = 77 \text{ ft}$

Roadway crown elevation at Sta 115+25: $El_{11525} := El_{PVC} + G1 \cdot x_{11525} + \frac{(G2 - G1) \cdot {x_{11525}}^2}{2 \cdot VCL} = 184.96 \text{ ft}$ Plan callout of 184.96, OK

Roadway crown elevation at outside wall of box culvert at Station 115+19.79:

$x_{box} := 11519.79 \text{ ft} - Sta_{PVC} = 71.79 \text{ ft}$

$El_{crown.box} := El_{PVC} + G1 \cdot x_{box} + \frac{(G2 - G1) \cdot {x_{box}}^2}{2 \cdot VCL} = 184.84 \text{ ft}$

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

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

Width to hinge-point: $b_{shoulder} := 7 \text{ ft} + 3 \text{ ft} = 10 \text{ ft}$

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

Minimum shoulder elevation over culvert: $Elev_{shoulder.min} := El_{crown.box} - b_{travel} \cdot x_{travel} - b_{shoulder} \cdot x_{shoulder} = 184.20 \text{ ft}$

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

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.

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

Maximum elevation for top of culvert rise: $Elev_{max.rise} := Elev_{shoulder.min} - t_{fill.min} - t_{slab.top} = 181.04 \text{ ft}$ Maximum low chord to accommodate $t_{fill.min}$

Elevation at top of rise: $Elev_{rise} := Elev_{invert} + Rise = 179.93 \text{ ft}$

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

Buffer: $Buffer := Elev_{max.rise} - Elev_{rise} = 1.113 \text{ ft}$ Additional fill beyond 2-ft minimum over culvert

Miscellaneous Roadway Information:

Roadway top width: $b_{road} := 44 \text{ ft}$ 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	15.00	15.00	176.48	0.58	2.550	0.42	7-H2t	-1.00	0.36	2.53	2.36	0.49	0.19
Q2	30.00	30.00	177.18	0.98	3.250	0.54	7-H2t	-1.00	0.58	3.22	3.05	0.78	0.22
Q5	50.00	50.00	177.83	1.38	3.903	0.65	7-H2t	-1.00	0.81	3.85	3.68	1.08	0.24
Q10	60.00	60.00	178.10	1.56	4.171	0.70	7-H2t	-1.00	0.92	4.10	3.93	1.22	0.25
Q25	80.00	80.00	178.57	1.89	4.637	0.77	7-H2t	-1.00	1.11	4.54	4.37	1.47	0.27
Q50	95.00	95.00	178.87	2.12	4.943	0.82	7-H2t	-1.00	1.25	4.83	4.66	1.64	0.28
Q100	110.00	110.00	179.15	2.34	5.222	0.87	7-H2t	-1.00	1.38	5.09	4.92	1.80	0.29
Q200	130.00	130.00	179.49	2.62	5.563	0.93	7-H2t	-1.00	1.54	5.40	5.23	2.00	0.30
Q500	155.00	155.00	179.88	2.94	5.950	0.99	7-H2t	-1.00	1.73	5.76	5.59	2.24	0.31
Overtopping	878.33	855.45	184.95	11.02	9.729	1.84	7-H2t	-1.00	5.40	5.40	0.00	13.19	0.00

Due to tailwater controlled water surface elevations, the proposed elevations are very similar to existing, and they are very slightly lower. 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 12'-0" span and 6'-0" above the thalweg / flowline (8'-0" rise minus 2'-0" minimum embedment) results in a 72 square foot modeled opening.

Both the existing and proposed culverts do not indicate roadway overtopping for any of the modeled recurrence intervals.

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.

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.

Q50 headwater elevation per HY-8 analysis: $HW_{Q50} := 178.87\ \text{ft}$

Headwater depth at Q50: $HW_{d.Q50} := HW_{Q50} - Elev_{inlet} = 4.94\ \text{ft}$

Headwater Depth ratio, Q50: $HW.D_{Q50} := \dfrac{HW_{d.Q50}}{Rise - d_{stream.fill}} = 0.824$ Matches 0.82 in table above, Ok

Check: $Check_{HW.D.Q50} := \textbf{if}\left(HW.D_{Q50} \le 0.90, \text{“Okay”}, \text{“Revise”}\right) = \text{“Okay”}$

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: $Elev_{rise} = 179.93\ \text{ft}$

Q100 headwater elevation: $HW_{Q100} := 179.15\ \text{ft}$

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: $y := 5.95\ \text{ft}$

Shields equation: $V_c := 6.19 \cdot \left(\dfrac{y}{\text{m}}\right)^{\frac{1}{6}} \cdot \left(\dfrac{D_{riprap}}{\text{m}}\right)^{\frac{1}{3}} \cdot \dfrac{\text{m}}{\text{s}} = 11.273\ \dfrac{\text{ft}}{\text{s}}$ *HEC-18 Eq. C.12*

Q500 outlet velocity: $V_{Q500} := 2.24\ \dfrac{\text{ft}}{\text{s}}$

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

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

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.

Crossing Notes

Crossing Input: Thorne Brook - Proposed

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	200.000	ft
Crest Elevation	184.840	ft
Roadway Surface	Paved	
Top Width	44.000	ft

Culvert Input: Thorne Brook - Proposed

Parameter	Value	Units
CULVERT DATA		
Name	Proposed	
Shape	Concrete Box	
Material	Concrete	
Span	12.000	ft
Rise	8.000	ft
Embedment Depth	24.000	in
Manning's n (Top/Sides)	0.012	
Manning's n (Bottom)	0.067	
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	171.930	ft
Outlet Station	58.000	ft
Outlet Elevation	171.930	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: Thorne Brook - Proposed

Headwater Elevation (ft)	Discharge Names	Total Discharge (cfs)	Proposed Discharge (cfs)	Roadway Discharge (cfs)	Iterations
176.48	Q1.1	15.00	15.00	0.00	1
177.18	Q2	30.00	30.00	0.00	1
177.83	Q5	50.00	50.00	0.00	1
178.10	Q10	60.00	60.00	0.00	1
178.57	Q25	80.00	80.00	0.00	1
178.87	Q50	95.00	95.00	0.00	1
179.15	Q100	110.00	110.00	0.00	1
179.49	Q200	130.00	130.00	0.00	1
179.88	Q500	155.00	155.00	0.00	1
184.84	Overtopping	848.00	848.00	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

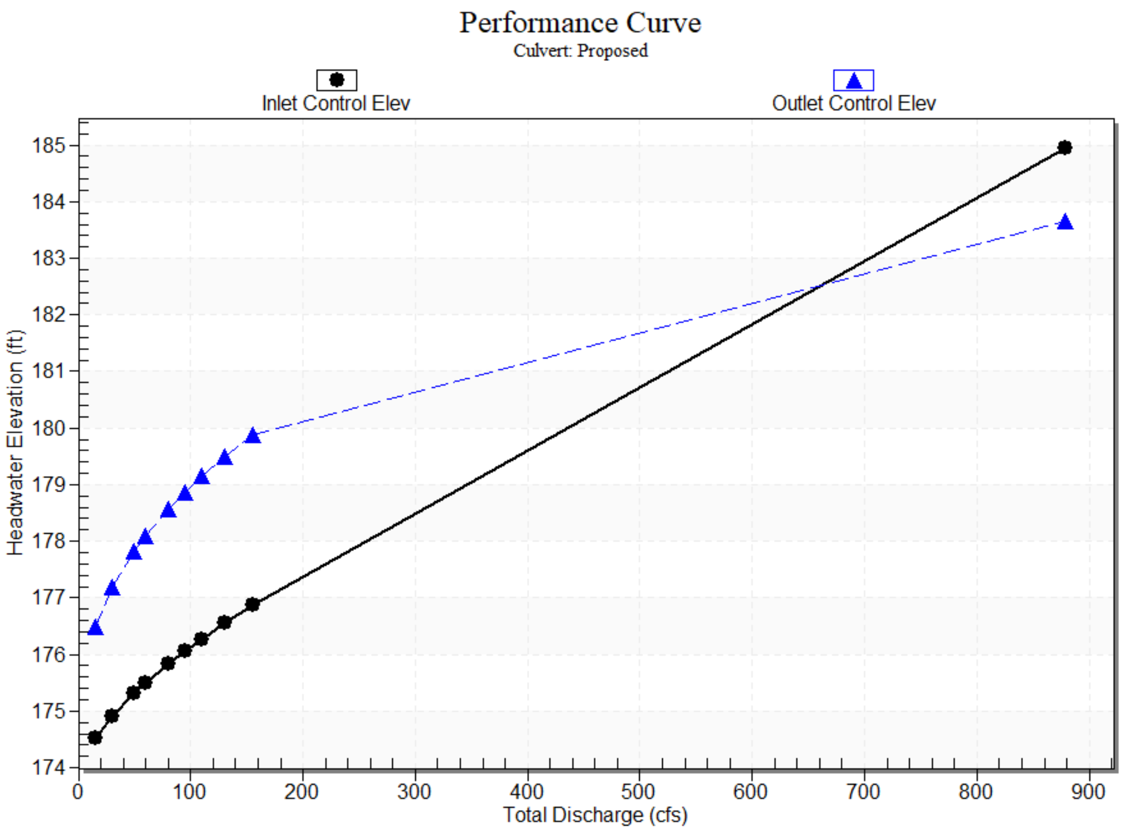
Discharge Names	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
Q1.1	15.00	15.00	176.48	0.58	2.550	0.42	7-H2t	-1.00	0.36	2.53	2.36	0.49	0.19
Q2	30.00	30.00	177.18	0.98	3.250	0.54	7-H2t	-1.00	0.58	3.22	3.05	0.78	0.22
Q5	50.00	50.00	177.83	1.38	3.903	0.65	7-H2t	-1.00	0.81	3.85	3.68	1.08	0.24
Q10	60.00	60.00	178.10	1.56	4.171	0.70	7-H2t	-1.00	0.92	4.10	3.93	1.22	0.25
Q25	80.00	80.00	178.57	1.89	4.637	0.77	7-H2t	-1.00	1.11	4.54	4.37	1.47	0.27
Q50	95.00	95.00	178.87	2.12	4.943	0.82	7-H2t	-1.00	1.25	4.83	4.66	1.64	0.28
Q100	110.00	110.00	179.15	2.34	5.222	0.87	7-H2t	-1.00	1.38	5.09	4.92	1.80	0.29
Q200	130.00	130.00	179.49	2.62	5.563	0.93	7-H2t	-1.00	1.54	5.40	5.23	2.00	0.30
Q500	155.00	155.00	179.88	2.94	5.950	0.99	7-H2t	-1.00	1.73	5.76	5.59	2.24	0.31
Overtopping	878.33	855.45	184.95	11.02	9.729	1.84	7-H2t	-1.00	5.40	5.40	0.00	13.19	0.00

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 Barrel Data

Culvert Barrel Type: Straight Culvert
Inlet Elevation(invert): 173.93 ft
Outlet Elevation (invert): 173.93 ft
Culvert Length: 58.00 ft
Culvert Slope: 0.00 ft/ft

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: Thorne Brook -Existing Conditions

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	200.000	ft
Crest Elevation	184.600	ft
Roadway Surface	Paved	
Top Width	42.500	ft

Culvert Input: Thorne Brook -Existing Conditions

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	173.930	ft
Outlet Station	67.670	ft
Outlet Elevation	174.550	ft
Number of Barrels	1	

Table 2 - Summary of Culvert Flows at crossing: Thorne Brook -Existing Conditions

Headwater Elevation (ft)	Discharge Names	Total Discharge (cfs)	Existing Discharge (cfs)	Roadway Discharge (cfs)	Iterations
176.49	Q1.1	15.00	15.00	0.00	1
177.19	Q2	30.00	30.00	0.00	1
177.85	Q5	50.00	50.00	0.00	1
178.12	Q10	60.00	60.00	0.00	1
178.60	Q25	80.00	80.00	0.00	1
178.91	Q50	95.00	95.00	0.00	1
179.20	Q100	110.00	110.00	0.00	1
179.55	Q200	130.00	130.00	0.00	1
179.96	Q500	155.00	155.00	0.00	1
184.60	Overtopping	745.16	745.16	0.00	Overtopping

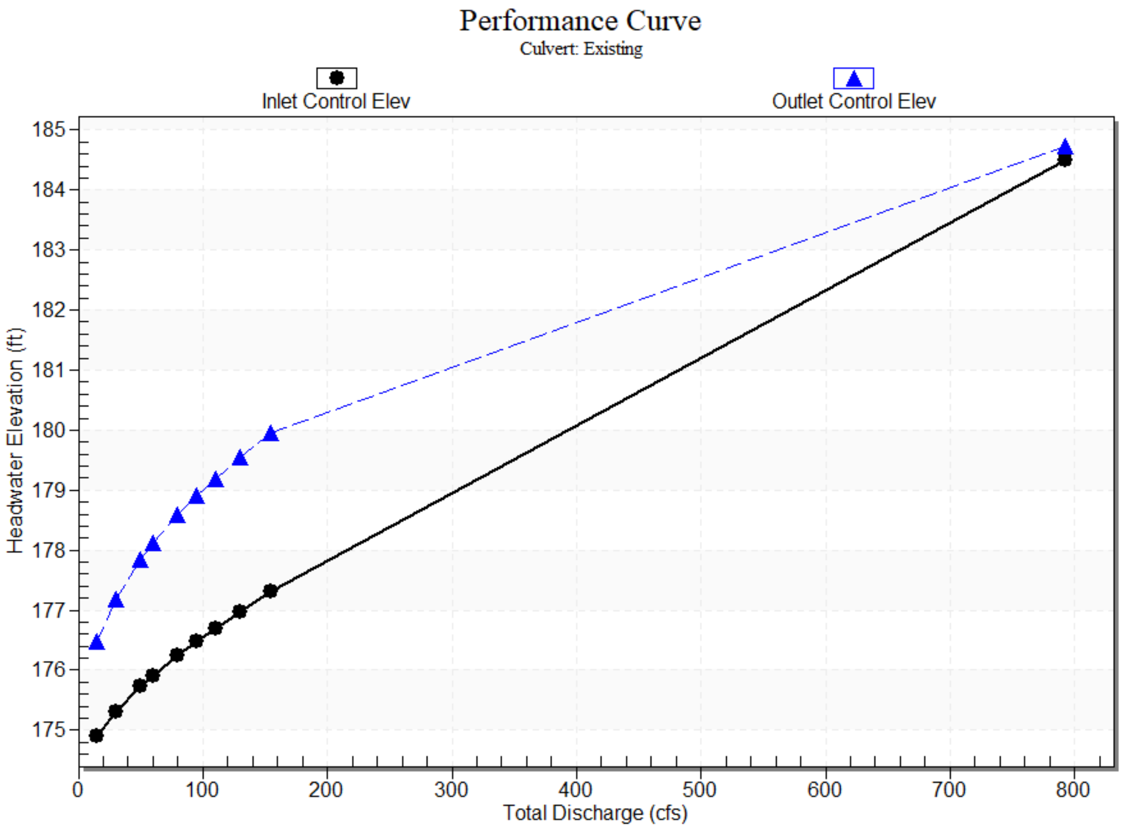
Table 3 - Culvert Summary Table: Existing

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	15.00	15.00	176.49	0.96	2.558	0.34	7-A2t	-1.00	0.66	1.91	2.36	0.86	0.19
Q2	30.00	30.00	177.19	1.37	3.262	0.44	7-A2t	-1.00	0.94	2.60	3.05	1.18	0.22
Q5	50.00	50.00	177.85	1.79	3.921	0.53	7-A2t	-1.00	1.21	3.23	3.68	1.54	0.24
Q10	60.00	60.00	178.12	1.98	4.192	0.56	7-A2t	-1.00	1.32	3.48	3.93	1.69	0.25
Q25	80.00	80.00	178.60	2.32	4.665	0.63	7-A2t	-1.00	1.53	3.92	4.37	1.99	0.27
Q50	95.00	95.00	178.91	2.55	4.979	0.67	7-A2t	-1.00	1.68	4.21	4.66	2.20	0.28
Q100	110.00	110.00	179.20	2.77	5.266	0.71	7-A2t	-1.00	1.81	4.47	4.92	2.40	0.29
Q200	130.00	130.00	179.55	3.05	5.620	0.76	7-A2t	-1.00	1.98	4.78	5.23	2.65	0.30
Q500	155.00	155.00	179.96	3.38	6.027	0.81	7-A2t	-1.00	2.18	5.14	5.59	2.96	0.31
Overtopping	792.22	761.03	184.74	10.57	10.811	1.46	7-A2c	-1.00	5.35	5.35	0.00	14.00	0.00

Culvert Barrel Data

Culvert Barrel Type: Straight Culvert
Inlet Elevation(invert): 173.93 ft
Outlet Elevation (invert): 174.55 ft
Culvert Length: 67.67 ft
Culvert Slope: -0.01 ft/ft

Culvert Performance Curve Plot: Existing



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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 Thorne 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 Thorne Book Bridge #2850 Over Thorne Brook* by WSP for WIN 026444.00. 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: $EL_{inlet.exist} := 173.93\ \text{ft}$

Proposed inlet invert elevation: $EL_{inlet.propose} := EL_{inlet.exist} + \Delta_{invert} = 174.43\ \text{ft}$

Existing outlet invert elevation: $EL_{outlet.exist} := 174.55\ \text{ft}$

Proposed outlet invert elevation: $EL_{outlet.propose} := EL_{outlet.exist} + \Delta_{invert} = 175.05\ \text{ft}$

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 (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	15.00	15.00	176.49	0.96	2.558	0.34	7-A2t	-1.00	0.66	1.91	2.36	0.86	0.19
Q2	30.00	30.00	177.19	1.37	3.262	0.44	7-A2t	-1.00	0.94	2.60	3.05	1.18	0.22
Q5	50.00	50.00	177.85	1.79	3.921	0.53	7-A2t	-1.00	1.21	3.23	3.68	1.54	0.24
Q10	60.00	60.00	178.12	1.98	4.192	0.56	7-A2t	-1.00	1.32	3.48	3.93	1.69	0.25
Q25	80.00	80.00	178.60	2.32	4.665	0.63	7-A2t	-1.00	1.53	3.92	4.37	1.99	0.27
Q50	95.00	95.00	178.91	2.55	4.979	0.67	7-A2t	-1.00	1.68	4.21	4.66	2.20	0.28
Q100	110.00	110.00	179.20	2.77	5.266	0.71	7-A2t	-1.00	1.81	4.47	4.92	2.40	0.29
Q200	130.00	130.00	179.55	3.05	5.620	0.76	7-A2t	-1.00	1.98	4.78	5.23	2.65	0.30
Q500	155.00	155.00	179.96	3.38	6.027	0.81	7-A2t	-1.00	2.18	5.14	5.59	2.96	0.31

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	15.00	15.00	176.55	1.02	2.12	0.34	7-A2t	NA	0.70	1.41	2.36	1.51	0.19
Q2	30.00	30.00	177.27	1.48	2.84	0.45	7-A2t	NA	0.99	2.10	3.05	1.84	0.22
Q5	50.00	50.00	177.95	1.96	3.52	0.56	7-A2t	NA	1.30	2.73	3.68	2.27	0.24
Q10	60.00	60.00	178.24	2.17	3.81	0.61	7-A2t	NA	1.43	2.98	3.93	2.46	0.25
Q25	80.00	80.00	178.74	2.56	4.31	0.69	7-A2t	NA	1.67	3.42	4.37	2.83	0.27
Q50	95.00	95.00	179.08	2.83	4.65	0.74	7-A2t	NA	1.83	3.71	4.66	3.10	0.28
Q100	110.00	110.00	179.40	3.08	4.97	0.79	7-A2t	NA	1.98	3.97	4.92	3.35	0.29
Q200	130.00	130.00	179.80	3.39	5.37	0.86	7-A2t	NA	2.17	4.28	5.23	3.69	0.30
Q500	155.00	155.00	180.26	3.74	5.83	0.93	7-A2t	NA	2.39	4.64	5.59	4.09	0.31

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Project: Thorne Brook Bridge #2850

Task: ***Slipline Hydraulic Analysis***

Client: MaineDOT

Calculated by: E. Caron

Date: 3/19/2026

Checked by: K. James

Date: 3/30/2026

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	15.00	15.00	176.55	1.01	2.12	0.35	7-A2t	NA	0.67	1.41	2.36	1.50	0.19
Q2	30.00	30.00	177.27	1.49	2.84	0.47	7-A2t	NA	0.96	2.10	3.05	1.87	0.22
Q5	50.00	50.00	177.96	1.99	3.53	0.58	7-A2t	NA	1.27	2.73	3.68	2.32	0.24
Q10	60.00	60.00	178.25	2.21	3.82	0.63	7-A2t	NA	1.40	2.98	3.93	2.53	0.25
Q25	80.00	80.00	178.77	2.62	4.34	0.71	7-A2t	NA	1.65	3.42	4.37	2.93	0.27
Q50	95.00	95.00	179.11	2.90	4.68	0.77	7-A2t	NA	1.81	3.71	4.66	3.21	0.28
Q100	110.00	110.00	179.43	3.17	5.00	0.82	7-A2t	NA	1.97	3.97	4.92	3.48	0.29
Q200	130.00	130.00	179.84	3.48	5.41	0.89	7-A2t	NA	2.17	4.28	5.23	3.83	0.30
Q500	155.00	155.00	180.32	3.84	5.89	0.96	7-A2t	NA	2.40	4.64	5.59	4.26	0.31