HYDROLOGY REPORT

Wells Buffam Bridge – Wells, York County, BR #2107

General Information and Scope

This study is prepared to provide hydrologic information from water resources agencies; on the crossing at the Buffam Bridge in Wells, Maine. The hydrology data will be used in the hydraulic evaluation of the existing and proposed bridge openings. The existing bridge spans the Webhannet River at the route 1 crossing and carries riverine flow from west to east toward the river mouth at the ocean front. The existing opening is approximately 18 feet wide at the top near the bottom of the superstructure and 16 feet wide at the footings level. The flow runs between the breast walls of the existing concrete mass abutments.

The Maine DOT Hydrology department used the 1999 Hodgkin's USGS regression analysis to estimate peak flows at the bridge opening inlet. The design flows are those determined by Maine DOT as follows:

SUMMARY

Drainage Area	5.38	mi^2
Q1.1	42.8	ft^3/s
Q10	144.0	ft^3/s
Q25	189.0	ft^3/s
Q50	209.1	ft^3/s
Q100	244.9	ft^3/s
Q500	313.1	ft^3/s

Tidal Flow

Tidal flow taking place downstream was considered to determine if it controls the flow at the bridge. The following tidal elevations data for the Wells station was obtained from the National Oceanic and Atmospheric administration (NOAA), and is tabulated as follows:

Type of Tidal Flow	Elevation (ft)
Mean Higher High Water (MHHW)	4.42
Mean High Water (MHW)	3.99
Mean Tide Level (MTL)	-0.41
Mean Low Water (MLW)	-4.80
Mean Lower Low Water (MLLW)	--5.14

Wells Buffam Bridge – Wells, York County, BR #2107

Existing Bridge and channel Analysis

A hydraulic analysis was performed for the existing bridge channel to determine its hydraulic properties. The existing channel upstream and downstream of the bridge was cut into many sections at close intervals to capture the truest bathymetry of the channel possible. The sections were cut wide enough to contain the entire flood plain geometry, such that the entire flow is contained within its boundaries. Sections were also cut through the waterfall over 100 feet upstream of the Buffam Bridge. Added to the model were the Buffam Bridge, and a foot bridge upstream between the Buffam Bridge and the waterfall. The model was calibrated by varying the boundary conditions and by comparing stage elevation results. Both the supercritical flow and the mixed flow conditions were investigated for riverine type flow, and subcritical flow condition for the tidal flow. The water surface profiles were checked for possible hydraulic jumps using known water surface elevations, critical depth, and normal depth boundary conditions. The results of the analysis indicated hydraulic jumps occur at the bottom of the waterfall and directly upstream of the pedestrian foot bridge. The results are tabulated as follows

Buffam Bridge - Supercritical Flow Regime   Maximum Stage elevations and velocities

	Flow (cfs)	Headwater Elevation (ft)	Channel Velocity (ft/s)	Froude#
$Q_{1.1}$	42.8	2.11	4.58	1.01
Q_{10}	144.0	2.88	6.49	1.00
Q_{25}	189.0	3.14	7.10	1.00
Q_{50}	209.1	3.24	7.40	1.01
Q_{100}	244.9	3.44	7.73	1.00
Q_{500}	313.1	3.75	8.44	1.01

Freeboard @ Q_{50} = 10.66 ft
Freeboard@Q_{100} = 10.64 ft
Existing Opening = 211.0 sq.ft.

With the existing lowest bottom chord elevation at 13.90 ft, there is plenty of freeboard for the existing Buffam bridge at flood flows Q50, and Q100. This indicates that the existing bridge opening is satisfactory for passing all flood flows, and there is no risk of overflow or overtopping. Also, the velocities are not too high indicating critical flow conditions with Froude #'s = 1.

Buffam Bridge - Mixed Flow Regime   Maximum Stage elevations and velocities

	Flow (cfs)	Headwater Elevation (ft)	Channel Velocity (ft/s)	Froude#
$Q_{1.1}$	42.8	3.51	1.30	0.17

Q_{10}	144.0	4.52	2.86	0.30
Q_{25}	189.0	4.84	3.39	0.33
Q_{50}	209.1	4.96	3.61	0.35
Q_{100}	244.9	5.17	3.98	0.37
Q_{500}	313.1	5.53	4.62	0.41

Freeboard @ Q_{50} = 8.94 ft
Freeboard@Q_{100} = 8.73 ft
Existing Opening = 211.0 sq.ft.

The results indicate that there is satisfactory freeboard for the existing lowest bridge bottom chord at flood flows Q50, and Q100. However, the velocities and Froude #'s are much lower than those obtained in the supercritical flow regime, indicating subcritical flows with Froude #'s < 1. In this mixed flow regime, the downstream conditions of the subcritical flow controlled the flow thru the bridge.

Buffam Bridge - Subcritical Flow Regime

	Flow (cfs)	Headwater Elevation (ft)	Channel Velocity (ft/s)
$Q_{1.1}$	42.8	4.42	4.17
Q_{10}	144.0	4.54	4.59
Q_{25}	189.0	4.90	4.57
Q_{50}	209.1	5.06	4.56
Q_{100}	244.9	5.31	4.58
Q_{500}	313.1	5.81	4.54

Freeboard @ Q_{50} = 8.84 ft
Freeboard@Q_{100} = 8.59 ft
Existing Opening = 211.0 sq.ft

In this subcritical flow regime, the elevation of the highest and lowest tides were used as boundary conditions to capture the variability of the flow elevations at the bridge due to tidal fluctuations. The results of the analysis indicate that tidal effects govern the 1.1 year recurrence interval and riverine flow generally controls less frequent events. The subcritical flow with the tail water set at MHHW level cause the highest flow stage elevations at the Buffam bridge, but still have satisfactory freeboard for the existing lowest bridge bottom chord at flood flows Q50, and Q100.

Conclusion The results of the analysis of the existing bridges indicate that both bridges are hydraulically satisfactory to pass the expected flood and tidal flows with little or no risk of overtopping; after accounting for Hydraulic Jumps and backwater conditions that takes place due to the variable ground slopes along the upstream part of the channel.

Existing Bridge Scour Analysis

Scour was estimated for the Buffam Bridge by using the mixed flow hydraulic model, which results in the worst-case condition in terms of flow velocities at the 500-year flood event. Empirical equations based on the HEC-18 Scour Evaluation Manual were used to determine scour values. Using the D_{50} particle size obtained from particle distribution chart for the site, and Froehlich formula for computing Abutment scour, the HEC-RAS Hydraulic Design Functions section computed contraction and abutment scours. The software results were hand checked and verified to be the following:

Contraction Scour = 2.24 ft

Total Scour

Left Abutment + Contraction Scour = 5.07 ft

Right Abutment + Contraction scour = 3.75 ft

Scour Countermeasures The results indicate 5'+ of anticipated scour at the left abutment. Geotechnical analysis concluded that the abutments and wingwalls of the Buffam bridge are partially founded on shallow bedrock and partially on gravel with a shallow overburden to bedrock, but not on wooden piles. However, with proper scour countermeasures such as dry grout cement bags, that can be used to fill scoured areas around abutments and wing-walls, or protect against anticipated scour action, the abutments will be reused. This is one of the important factors that was taken into consideration in the decision of reusing the existing abutments. However, geotechnical and environmental aspects, right of way, and structural considerations also played an important role in making that decision.

SUMMARY

		Existing Structure	Recommended Structure
		18 ft span CIP Concrete Slab	18 ft span Precast Superstucture
Total Area of Waterway Opening	ft^2	211	211
Headwater elevation @ $Q_{1.1}$	ft	4.42	4.42
Headwater elevation @ Q_{10}	ft	4.54	4.54
Headwater elevation @ Q_{25}	ft	4.90	4.90
Headwater elevation @ Q_{50}	ft	5.06	5.06
Headwater elevation @ Q_{100}	ft	5.31	5.31
Headwater elevation @ Q_{500}	ft	5.81	5.81
Freeboard @ Q_{50}	ft	8.84	8.84
Freeboard @ Q_{100}	ft	8.59	8.59
Outlet Velocity @ $Q_{1.1}$	ft/s	4.17	4.17
Outlet Velocity @ Q_{10}	ft/s	4.59	4.59
Outlet Velocity @ Q_{25}	ft/s	4.57	4.57
Outlet Velocity @ Q_{50}	ft/s	4.56	4.56
Outlet Velocity @ Q_{100}	ft/s	4.58	4.58
Outlet Velocity @ Q_{500}	ft/s	4.54	4.54

PRELIMINARY HYDROLOGY REPORT

Wells Branch Bridge – Wells, York County, BR #3091

General Information and Scope

This study is prepared to provide hydrologic information from water resources agencies; on the water crossing at the Branch Bridge in Wells, Maine. The data obtained will be used in the hydraulic evaluation of the existing opening for capacity with two barrels open to flow, and with only one barrel open with the scope of the project being Bridge Rehabilitation. In that case one barrel will need to be dry to perform rehab work on it, while the full flow gets diverted to the other barrel. The existing barrel openings has 8 ft. span and 8 ft. rise each. The flow at the inlet is contained within flared wing walls and an inlet apron that extends to the extremities of the wing walls. The flow runs North-West to South-East through the barrels and under the roadway to the outlet which also has flared wingwalls and a downstream apron with the same shape and size as that of the inlet. This geometrical information about the culvert will be used to perform a hydraulic analysis of the existing bridge opening to determine its adequacy in accommodating flood flows through the bridge, and throughout the rehabilitation period; expected to be scheduled in the summer at low flows.

Hydrologic information and flow data are sought from three different sources:

1- Flood Emergency Management Agency (FEMA)
2- United States Geological Survey (USGS)
3- Maine DOT Hydrology

Flood Emergency Management Agency (FEMA)

A 1992 FEMA study for flood insurance was performed for the Wells area as a revision to an earlier 1983 study. The study included the bridge crossing and its channel at the wells-Kennebunk line. FEMA developed a flood map and the results indicated that the 100-year flood stays mostly within the channel boundaries with minimum overflow to the adjacent flood plain. This indicates that the Branch Brook channel upstream and downstream from the bridge is quite satisfactory in containing and passing the 100-year flood flow with minimum overflow of its banks. However, a base flood elevation was not established, and flood hazard factors were not determined in the study.

United States Geological Survey

The USGS maintains gaging station # 01069700 right at the Branch Brook Bridge and collects data as to stage elevation and flow. The peak flow statistics are provided for a drainage basin area of 10.7 sq-mi with 11.3% wetlands as follows:

Type	Flow (cfs)
Q_2	262.0
Q_5	452.0
Q_{10}	600.0
Q_{25}	809.0
Q_{50}	978.0
Q_{100}	1160.0
Q_{500}	1640.0

Streamstats Report of USGS indicates a drainage area of 9.1 sq.mi and 9.84% of storage of combined waterbodies and wetlands. The peak flow data are computed using rain gage data and regression methods and predicted as follows:

Type	Flow (cfs)
$Q_{1.1}$	77.2
Q_2	269.0
Q_5	426.0
Q_{10}	536.0
Q_{25}	705.0
Q_{50}	823.0
Q_{100}	965.0
Q_{500}	1300.0

The BFW has been determined to be 24.2 ft with a bank-full stream flow of 52.7 cfs. With a Q1.1 of 77.2 cfs as predicted in Streamstats, the Q1.1 flow would clearly overflow the banks. This contradicts the FEMA data, which indicates somewhat satisfactory channel at a much larger Q100 flood flow.

Maine DOT Hydrology Data

The Maine DOT Hydrology department used rainfall data and the 1999 Hodgkin's USGS regression analysis to estimate peak flows at the bridge opening inlet. The process involved determining the configuration area of the contributing watershed, and the percentage of wetland area within the watershed area. The results are tabulated as follows:

Watershed area: A = 9.0 mi^2, Wetland Area = 0.9 mi^2, W (%) = 9.84

Type	Flow (cfs)
$Q_{1.1}$	129.4
Q_2	269.0
Q_5	426.3
Q_{10}	536.8
Q_{25}	705.8
Q_{50}	823.6
Q_{100}	965.4
Q_{500}	1302.3

<u>Data Comparison and Choice</u>

From the above, it is evident that the Maine DOT data and USGS Streamstats data are almost identical. Therefore, the Maine DOT data has a level of reliability that can be used in the hydraulic analysis of the existing culvert bridge. The stream gage data is most accurate however because it is based on actual measurement. Then both data will be tested in the hydraulic analysis, and the most reasonable data will depend on the analysis outcome in comparison with current observations and conditions at the bridge. For example, if Q500 of either the Main DOT or USGS gage data indicate overtopping, and it becomes known from historical evidence and user information that there has never been an overtopping event, the amount will be dropped to a lower value where overtopping do not occur.

Reported by: Roger M. Naous, P.E.
Date: January 27, 2020

PRELIMINARY HYDRAULIC REPORT

Wells Buffam Bridge – Wells, York County, BR #2107

Existing Bridge Culvert Analysis

A hydraulic analysis using Maine DOT flow data was performed for the existing 2-barrel bridge culvert to determine its hydraulic properties and its ability to pass the flood flows in its current condition. The hydraulic analysis was performed using Hy-8 software for culvert design and checked with hand calculations. With no plans present for the current culvert, the culvert geometry was configured using recent survey, maintenance reports, and in person site measurements. The hydraulic analysis should prove that the culvert, which is being rehabilitated, is capable to pass the flood flows with minimum adverse effects before and after the rehabilitation work is completed. The type of rehab may reduce the hydraulic opening which may increase velocities and worsen the scour conditions; already present at the downstream apron with evidence of undermining and localized loss of support. Below are the Hy-8 results of the hydraulic analysis for the existing culvert:

Culvert Summary Table - Culvert (Existing Condition)

Discharge Names	Total Discharge	Culvert Discharge	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(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	129.40	129.40	45.34	1.94	**2.14**	2-M2c	1.30	1.27	1.27	0.80	6.39	8.55
Q2	269.00	269.00	46.68	3.16	**3.48**	2-M2c	2.15	2.06	2.06	1.26	8.15	11.26
Q5	426.30	426.30	47.92	4.31	**4.72**	2-M2c	2.99	2.80	2.80	1.68	9.50	13.33
Q10	536.80	536.80	48.69	5.04	**5.49**	2-M2c	3.54	3.27	3.27	1.95	10.26	14.47
Q25	705.80	705.80	49.79	6.06	**6.59**	2-M2c	4.34	3.92	3.92	2.33	11.24	15.94
Q50	823.60	823.60	50.51	6.72	**7.31**	2-M2c	4.89	4.35	4.35	2.58	11.83	16.82
Q100	965.40	965.40	51.32	7.51	**8.12**	7-M2c	5.53	4.84	4.84	2.86	12.48	17.75
Q500	1302.30	1302.30	53.11	9.45	**9.91**	7-M2c	7.02	5.90	5.90	3.49	13.79	19.62

The results indicate that the existing culvert is undersized for flows of Q100 and more as the culvert flows 100% full with no available freeboard. At Q50 the culvert flows 86% full. However, there isn't a possibility of overtopping due to the high fill above the culvert, even at the Q500 event. The outlet velocity is nearly 12.5 ft/sec at Q100. With relatively flat culvert slope, the outlet velocities are quite high causing possible scour downstream directly below the apron slab and near the flared wingwalls. This scour and undermining potential are noted by over a 6 inch drop off at the end of the downstream apron.

Existing Bridge Culvert Analysis with one Barrel open, Stage I

The analysis was performed for only one barrel open, while the other barrel gets worked on by the rehabilitation crew. Therefore, the flow will be diverted to only one barrel using cofferdams or dikes, and it is sought to determine the maximum flow that the culvert can handle and the required cofferdam height such that overflow will not occur and pour into the other barrel during construction. This time the results of the analysis weren't as favorable:

Culvert Summary Table - Culvert Stage I , one barrel open

Discharge Names	Total Discharge	Culvert Discharge	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(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	129.40	129.40	46.59	3.08	**3.39**	2-M2c	2.09	2.01	2.01	0.80	8.05	8.55
Q2	269.00	269.00	48.70	5.05	**5.50**	2-M2c	3.54	3.27	3.27	1.26	10.27	11.26
Q5	426.30	426.30	50.68	6.88	**7.48**	2-M2c	5.02	4.45	4.45	1.68	11.97	13.33
Q10	536.80	536.80	51.92	8.11	**8.72**	7-M2c	6.01	5.19	5.19	1.95	12.93	14.47
Q25	705.80	705.80	53.66	10.13	**10.46**	7-M2c	7.49	6.23	6.23	2.33	14.16	15.94
Q50	823.60	823.60	54.96	**11.76~**	11.59	7-M2c	8.00	6.90	6.90	2.58	14.91	16.82
Q100	965.40	965.40	57.25	**14.05~**	13.21	7-M2c	8.00	7.68	7.68	2.86	15.72	17.75
Q500	1302.30	1302.30	64.29	**21.09**	17.71	6-FFc	8.00	8.00	51.20	3.49	20.35	19.62

With the top of culvert at 51.20 ft., the culvert will be flowing 93% full at Q5 and 100% full at Q10. However, with the top of the road being at 67.00 ft., overtopping will still not occur at Q500 with a calculated stage elevation of 64.29 ft. The truth is that when the flow exceeds the cofferdam height, it will spill to the other barrel. Based on the results, a 3.5 ft. tall cofferdam will be required at Q1.1. The rehabilitation work should therefore be done at low flow periods with flows of Q1.1 and less. From the above results, flows at Q1.1, Q2, and Q5 are outlet controlled with Hw < D and with the tail water at critical depth. All other flows are outlet controlled with Hw > D and with tail water at critical depth. At Q100, the flow velocity approaches 16 ft/s which is expected to cause much scour damage downstream. None of the high flows are expected to be reached before the culvert gets rehabbed and with both barrels open.

Rehabbed Culvert Analysis with one Barrel open, Stage II

When rehab work is completed in one barrel, the full flow is switched to it while work commences on the other barrel. The rehab work being performed, which is described to be a 2 ft. tall and 6-inch-thick concrete wall jacket from the culvert bottom slab, reduces the flow area is by 2 ft.^{2} in each barrel. This reduction occurs in the bottom part of the culvert at flows less than Q1.1 with the bottom hydraulic width reduced from 8ft. to 7ft. for the first 2 ft. of depth in the barrel. The result is a rise in stage elevation that must be accounted for to determine the required depth of the cofferdam. After adjusting the culvert geometry and running the analysis again, the following results are obtained:

Discharge Names	Total Discharge	Culvert Discharge	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(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	129.40	129.40	47.29	3.19	**4.09**	2-M2c	2.38	2.26	2.26	0.80	8.06	8.55
Q2	269.00	269.00	49.64	5.24	**6.44**	2-M2c	3.84	3.53	3.53	1.26	10.25	11.26
Q5	426.30	426.30	51.85	7.50	**8.65**	7-M2c	5.33	4.70	4.70	1.68	11.98	13.33
Q10	536.80	536.80	53.23	9.02	**10.03**	7-M2c	6.33	5.43	5.43	1.95	12.94	14.47
Q25	705.80	705.80	55.18	11.43	**11.98**	7-M2c	8.00	6.47	6.47	2.33	14.18	15.94
Q50	823.60	823.60	56.89	**13.69~**	13.24	7-M2c	8.00	7.15	7.15	2.58	14.92	16.82
Q100	965.40	965.40	60.13	**16.93~**	15.33	7-M2c	8.00	7.92	7.92	2.86	15.74	17.75
Q500	1302.30	1225.14	67.40	**24.20**	19.93	6-FFc	8.00	8.00	51.20	3.49	19.76	19.62

This time the culvert will be flowing 100% full at Q5 and the water level near the top of the headwall at Q10. Compared with stage one analysis, the narrowing of the bottom width of the culvert due to rehab causes a rise in stage elevations of 0.7’ at Q1.1, and 1.17’ at Q5. Based on the results, a 4.3 ft. tall cofferdam will be required at Q1.1. From the above results, flows of Q5 and larger are outlet controlled with the head water HW > D and with tail-water at critical depth. At Q5 the outlet velocity is nearly 12 ft./s and it increases thereafter to nearly 20 ft./s at Q500. Moreover, flows at Q5 or more are orifice type flows with large downstream velocities that will cause enormous scour damage and possible formation of scour pool unless armoring of the downstream channel takes place.

Rehabbed Culvert Analysis with two Barrels open, Stage III, Final stage

In this stage, the rehab work is completed, and the flow is released to both barrels. The effect of the full rehab work on the hydraulic conditions of the culvert is assessed and the results of the analysis are as follows:

Culvert Summary Table - Culvert Stage III, two barrels open

Discharge Names	Total Discharge	Culvert Discharge	Headwater Elevation (ft)	Inlet Control Depth(ft)	Outlet Control Depth(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	129.40	129.40	45.85	2.03	**2.65**	2-M2c	1.45	1.37	1.37	0.80	6.72	8.55
Q2	269.00	269.00	47.38	3.27	**4.18**	2-M2c	2.43	2.32	2.32	1.26	8.14	11.26
Q5	426.30	426.30	48.76	4.40	**5.56**	2-M2c	3.28	3.06	3.06	1.68	9.48	13.33
Q10	536.80	536.80	49.63	5.23	**6.43**	2-M2c	3.84	3.53	3.53	1.95	10.24	14.47
Q25	705.80	705.80	50.86	6.50	**7.66**	2-M2c	4.65	4.17	4.17	2.33	11.27	15.94
Q50	823.60	823.60	51.66	7.31	**8.46**	7-M2c	5.20	4.59	4.59	2.58	11.86	16.82
Q100	965.40	965.40	52.57	8.28	**9.37**	7-M2c	5.85	5.08	5.08	2.86	12.50	17.75
Q500	1302.30	1302.30	54.57	10.64	**11.37**	7-M2c	7.35	6.15	6.15	3.49	13.80	19.62

Compared with initial stage without the rehab, the Q1.1 flow rises by 0.51', and the flow at Q50 rises by 1.15'. Overtopping will still not occur in both conditions, and the outlet velocity remain almost the same. From the above comparison, it can be concluded that the change of the hydraulic conditions in the culvert due to the rehab work remain acceptable and within tolerable limits. In conclusion, the finished rehabbed culvert does not cause much more adverse hydraulic effects than the existing culvert conditions.

		Existing Structure	Rehabbed Structure
		17 ft span CIP Concrete Culvert	17 ft span CIP Concrete Culvert
Total Area of Waterway Opening	ft^2	128	124
Headwater elevation @ $Q_{1.1}$	ft	45.34	45.85
Headwater elevation @ Q_{10}	ft	48.69	49.63
Headwater elevation @ Q_{25}	ft	49.79	50.68
Headwater elevation @ Q_{50}	ft	50.51	51.66
Headwater elevation @ Q_{100}	ft	51.32	52.57
Headwater elevation @ Q_{500}	ft	53.11	54.57
Freeboard @ Q_{50}	ft	0.69	0
Freeboard @ Q_{100}	ft	0	0
Outlet Velocity @ $Q_{1.1}$	ft/s	6.39	6.72
Outlet Velocity @ Q_{10}	ft/s	10.26	10.24
Outlet Velocity @ Q_{25}	ft/s	11.24	11.27
Outlet Velocity @ Q_{50}	ft/s	11.83	11.86
Outlet Velocity @ Q_{100}	ft/s	12.48	12.50
Outlet Velocity @ Q_{500}	ft/s	13.79	13.80

Reported by: Roger M. Naous, P.E.

Date: May 7, 2020