Hydraulic Report

Purpose

The purpose of the hydraulic analysis is to determine flood flow stages, and water velocities through the channel directly upstream and downstream of the Kennebunk Clayhill Bridge. The water stages for the different return periods determine the adequacy of the bridge opening, finished set grade, and superstructure depth and type. The velocities determine the possibility of erosion, and or scour of the channel. The scour results are used to make decisions on the type of substructure and depth of foundations, as well as, the need for embankment and substructure protection.

Procedure

The hydraulic analysis for the Mousam River was conducted for the existing bridge, and the new proposed bridge waterways. The existing channel was modeled using HEC-RAS analysis program provided by the Army Corps of Engineers. The program applies pre-determined flood flows to the existing channel geometry. By using In-Roads program capabilities, cross sections for the channel upstream and downstream of the bridge were obtained from recent survey and topographic information. The cross sections data were input into the program to obtain a geometric model for the existing channel. The resulting model was used for both the existing and proposed bridge analysis. The program utilizes principles of open channel hydraulics to determine important hydraulic parameters such as water surface elevations for flood flows, water velocities at or near the bridge, and scour depth at abutments and piers locations. Since the flows in the channel are combined tidal and riverene, the analysis was based on steady flow analysis by applying riverine flood flows supplied by USGS, and setting the tail water at MLLW and MHHW tidal elevations. Therefore, the maximum and minimum points in the entire tide cycle are checked. The Q50 flow, with tail water set at MHHW determines the size of the bridge opening and bottom of beam elevation. The Q50 flow, with tail water set at MLLW typically results in the highest velocities, and determines required scour and erosion protection measures. Additionally, since the riverene flows for this bridge are substantial, a steady analysis with the tail water set at normal water elevation must be checked as riverene flows may control over tidal.

A) Existing Bridge

The existing bridge superstructure and substructure were modeled into the HEC Program, which has capabilities of modeling the top and bottom chord locations of the superstructure, and determining the hydraulic properties with respect to the flow stage. For example, when the program recognizes that the flow stage obtained is above the bottom chord elevation of the superstructure, it switches from open channel flow to weir flow equations in its analysis. The program also models the substructure abutments, piers, and obstructions in the flow path, and turns out an analysis based on the model created, and its effects on the hydraulic properties. Both steady and unsteady flow analysis were considered to determine which of the two should be ruled out based on inconsistencies of results.

I- USGS Regression flow data

i) Steady flow analysis

Three runs of steady flow analysis were performed for USGS data flows, one with tail-water set at tidal MLLW, the other with tail-water set at tidal MHHW, and a third with tail-water set at normal water level determined by the natural slope of the stream bed. The results obtained are as follows:

1- Tail-water set at MLLW; MLLW Elevation @ - 4.97' below NAVD datum

Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	-4.04	-4.60	5.98
Q_{10}	3226	-1.79	-2.65	7.61
Q_{25}	4458	-0.10	-1.88	7.19
Q_{50}	5091	0.34	-1.55	7.58
Q_{100}	5762	0.77	-1.22	7.99
Q_{500}	7365	1.71	-0.46	8.87

Freeboard @ Q_{50} = 5.90 ft

2- Tail-water at MHHW; MHHW Elevation @ 5.11' above NAVD datum

Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	5.12	-4.60	0.93
Q_{10}	3226	5.10	-2.65	2.63
Q_{25}	4458	5.09	-1.88	3.65
Q_{50}	5091	5.10	-1.55	4.18
Q_{100}	5762	5.02	-1.22	4.74
Q_{500}	7365	4.96	- 0.47	6.10

Freeboard @ Q_{50} = 1.19 ft

3- Tail-water determined by the natural slope of the stream bed

Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	-1.50	-4.60	2.49
Q_{10}	3226	2.46	-2.65	3.52
Q_{25}	4458	4.00	-1.88	4.07
Q_{50}	5091	4.62	-1.55	4.36
Q_{100}	5762	5.20	-1.22	4.63
Q_{500}	7365	6.44	- 0.47	5.15

Freeboard @ Q_{50} = 1.62 ft

iv) Discussion on freeboard of the existing bridge

A freeboard of 1.19 ft is obtained at Q50 at MHHW. According to the BDG, 2 ft of free board at Q50 is the required minimum. It is thus clear that the finished grade of the existing bridge and the current lowest bottom chord elevation do not meet requirements. A minimum of additional freeboard of 0.81 ft at Q50 is needed. If an additional 0.75 ft of forecasted future rise of sea level for the next hundred years is considered, the total rise required would be 1.56 ft; such that the lowest bottom chord elevation set at 7.80 ft. On the other hand, and based on FEMA data, the predicted flood stage at Q10 including consideration of wave heights is 7.45 ft, and the bottom chord elevation should be set at 9.45ft. This will meet minimum requirements for clearance for tidal bridges. Additionally, the BDG requires that the top of Rip Rap shelf should be placed at least 2 ft above MHHW of 5.05 ft. Allowing 1.5 ft of clearance above that level, means that the bottom chord elevation should be set at 8.55’. The existing bridge bottom chord elevation is currently at 6.24 ft, and is too low for all cases.

v) Discussion on outlet flow velocities and scour conditions

The analysis results of the existing bridge indicate that maximum outlet velocity at MLLW at Q500 is 8.87 ft/s. To reduce velocities, the flow area must be increased by either increasing the bridge span or clearance, or a combination of both. The results of the scour analysis may prove that the existing bridge is hydraulically deficient in counteracting scour effects, and may well explain stream and embankments erosion that has occurred over the years in the vicinity of the bridge structure.

vi) Observed Scour

The observed scour on the site is a result of comparison of stream bed cross sections directly upstream and down stream of the bridge. A dive inspection report in 1990 has concluded that the stream bed at the east side of the bridge has experienced an average of 5 to 6 ft of erosion, while the west side experienced aggrading of stream bed material in the range of 2 to 3 ft. Most recent survey has indicated that the stream has not rebounded much since 1990, and the average current observed scour for each of the pile bents at the east side is nearly 5 ft. There is also evidence of stream lateral shifting on the east side; as the upstream east embankment has experienced moderate erosion of its banks concurrent with deposition of sediment material at the west side. This active erosion agrees with current understanding of stream behavior under a set of geographic and morphologic conditions associated with this bridge.

vii) Estimated Scour Analysis

Scour was estimated using hydraulic models and empirical equations at the 50,100, and 500 year flood events. The purpose was to compare the results with the observed scour, and to determine the type of event that has taken place. The results of the scour analysis for the existing channel using a HEC-RAS program were as follows:

a- 50 Year flood event

Pier + Contraction Scour = 10.88 ft
Left Abutment + Contraction Scour = 6.88 ft
Right Abutment + Contraction scour = 8.04 ft

b- 100 Year flood event

Pier + Contraction Scour = 11.78 ft
Left Abutment + Contraction Scour = 7.53 ft
Right Abutment + Contraction scour = 9.73 ft

c- 500 Year flood event

Pier + Contraction Scour = 13.16 ft
Left Abutment + Contraction Scour = 9.55 ft
Right Abutment + Contraction scour = 12.21 ft

viii) Comparison of Scour Values

The results of the analysis indicate that the maximum current observed scour of 6 ft at the right side of the channel is less than that estimated at any event. It must be noted however that the scour may have reached the depth indicated in the analysis during or immediately after the event, and that deposition of sediment may have taken place thereafter. This type of cycle is characteristic to alluvial systems. It is therefore prudent to consider the estimated scour values for evaluation of the existing bridge substructure during the flood event; when the high probability of substructure failure is more likely to take place.

B) Proposed Bridge

It is evident from the hydraulic results of the existing bridge that the overall profile of the proposed bridge has to be raised in combination with providing a relatively shallow superstructure. The proposed lowest bottom chord elevation and the finished grade must be set taking into consideration cost, tolerable environmental, right of way, and construction impacts. A separate hydraulic analysis will be carried out for the proposed bridge. The results of the analysis will be checked to determine conformance with the requirements, and the vertical profile will be set to generate the least impacts on the surroundings.

I- USGS Regression flow data

i) Steady flow analysis

1- Tail-water set at MLLW; MLLW Elevation @ - 4.97' below NAVD datum

Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	-4.16	-4.59	6.34
Q_{10}	3226	-1.95	-2.65	7.96
Q_{25}	4458	-1.06	-1.88	8.72
Q_{50}	5091	0.17	-1.52	7.73
Q_{100}	5762	0.61	-1.18	8.10
Q_{500}	7365	1.58	-0.44	8.90

Freeboard @ Q_{50} = 8.38 ft

<u>2- Tail-water at MHHW; MHHW Elevation @ 5.11' above NAVD datum</u>

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Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	5.11	-4.59	0.91
Q_{10}	3226	5.08	-2.65	2.59
Q_{25}	4458	5.06	-1.88	3.58
Q_{50}	5091	5.04	-1.52	4.10
Q_{100}	5762	5.02	-1.18	4.65
Q_{500}	7365	4.96	- 0.45	5.97

Freeboard @ Q_{50} = 3.51 ft

<u>3- Tail-water at normal water elevation</u>

Type	Flow (cfs)	Headwater Elev (ft)	Critical WS (ft)	Velocity(ft/s)
$Q_{1.1}$	1141	-1.50	-4.59	2.49
Q_{10}	3226	2.46	-2.65	3.46
Q_{25}	4458	4.00	-1.88	3.99
Q_{50}	5091	4.62	-1.52	4.27
Q_{100}	5762	5.20	-1.18	4.56
Q_{500}	7365	6.40	- 0.45	5.24

Freeboard @ Q_{50} = 3.93 ft

<u>ii) Unsteady flow analysis</u>

For accurate analysis of unsteady flow, a large survey which covers many miles upstream and downstream from the bridge site is required. Due to limited financial resources, the required extended survey is not available, and additional survey is costly. Historically, the unsteady flow has not controlled in many cases. Therefore, justification for performing unsteady flow has no support. Since the steady flow analysis checks the entire tide cycle at its maximum and minimum in combination with riverine flows, its results are deemed quite satisfactory for design.

<u>iii) Discussion on steady flow results</u>

The results of the steady flow analysis for both the existing and the proposed bridge turned out a maximum surface water elevation at Q10 for the MHHW point, and maximum water velocity at Q500 for the MLLW point. The MHHW results show that the

USGS riverene flows at Q10 and Q50 does not control, and much larger riverene flows at these flood events will be required to raise the headwater elevation above that set by tidal MHHW. However, the USGS flows at Q100 and Q500 result in water stages above the tidal elevations. Also, since the resulting headwater elevations are above critical water surface elevations for all return periods, it indicates subcritical flow. Based on maintenance reports, moderate ice damage is evident in the existing wooden pier piles, but it is not clear whether significant ice jams that have the capabilities of raising the flow stage have occurred in the past.

iv) Discussion on freeboard of the proposed bridge

A freeboard of 3.51 ft is obtained at Q50 at MHHW which satisfies BDG requirements of minimum 2 ft. If forecasted future rise of sea level of 0.75 ft for the next hundred years is considered, the available freeboard would be 2.76 ft. On the other hand, and based on FEMA data, the predicted flood stage at Q10 including wave heights is 7.45 ft. With the bottom chord set at 8.55 ft, it will provide a freeboard of 3.51 ft.

v) Discussion on outlet flow velocities and scour conditions

The analysis results of the proposed bridge indicate that maximum outlet velocity at MLLW at Q500 is 8.90 ft/sec. Since this is almost the same value as that for the existing bridge, scour conditions should be evaluated to determine maximum possible scour and its effect on the proposed bridge substructure.

vii) Estimated Scour of the Proposed Bridge

Scour was estimated using HEC-RAS at the 50,100, and 500 year flood events. The purpose was to estimate the maximum possible scour that can occur, and determine its effect on the proposed bridge substructure during flood events. The results of the analysis could dictate preliminary decisions as to the geometry, clearances, type of substructure, span length, and alignment of the proposed bridge. The results of the analysis are as follows:

a- 50 Year flood event

Pier + Contraction Scour = 11.47 ft
Left Abutment + Contraction Scour = 7.57 ft
Right Abutment + Contraction scour = 7.90 ft

b- 100 Year flood event

Pier + Contraction Scour = 12.43 ft
Left Abutment + Contraction Scour = 7.93 ft
Right Abutment + Contraction scour = 9.34 ft

c- 500 Year flood event

Pier + Contraction Scour = 13.33 ft
Left Abutment + Contraction Scour = 10.44 ft
Right Abutment + Contraction scour = 12.38 ft

Comparison of Scour between Existing and Proposed Bridge

The results of scour for the proposed bridge show little or no change from those of the existing bridge. The proposed bridge span was longer and the hydraulic opening larger. In this site however, the dominating factor for scour is the type of soil the stream bed is made of. The results of either analysis show consistent scour value of 13’ at the 500 yr flood event. Based on preliminary geotechnical data, there is nearly 33’ of depth between top of stream bed and bedrock. Therefore, with a loss of 13’ to scour, 20’ remain for embedment of piles. This information is very important in the assessment of piles foundations at the intermediate piers. This assessment will determine pier capacity, the ability of the piles to resist the applied loads, and the number of piles required.

Summary

The summary for the results of the hydraulic study are in the following table:

Type	Flow (cfs)	HighestWater Elevation (ft) Proposed	Highest Water Elevation (ft) Existing	Highest Velocity (ft/s) Proposed	Highest Velocity (ft/s) Existing
$Q_{1.1}$	1141	5.11	5.12	6.34	5.98
Q_{10}	3226	5.08	5.10	7.96	7.61
Q_{25}	4458	5.06	5.09	8.72	7.19
Q_{50}	5091	5.05	5.10	7.73	7.58
Q_{100}	5762	5.22	5.20	8.10	7.99
Q_{500}	7365	6.40	6.44	8.90	8.87

From the above results, it can be noted that water elevation and velocities between existing and proposed bridge openings does not change much due to the dominance of tidal flows, whose stage levels and velocities are independent of the bridge opening area provided.

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