HYDROLOGY REPORT

Route 150, a corridor priority 3 roadway, with AADT of 1460, crosses the East Branch Wesserunsett Stream, at a skew angle of 54.633 degrees. The steel girder bridge has a clearance of 11-12 feet to the stream bed below. The bridge is programmed to be rehabilitated.

East Branch Wesserunsett Stream flows generally in a southwesterly direction north of the bridge and approaches the bridge from the north immediately upstream. The vector of the stream flow appears to intersect with the southerly edge of the east abutment (a 6' deep scour hole exists at that location). Southerly of the bridge, the stream bends in a south-south westerly direction as it diverges from the Route 150 alignment. The stream reach on which the bridge is located slopes about 10 feet in 835 feet, or 0.012 feet/foot slope. See the illustration below. Soil borings indicate that the soils at the elevation of the stream bed are gravelly sands with a little silt. A reasonable D50 value for this type of soil would range between 0.8 mm to 2 mm in the stream bed, based on evaluation of soils data by Maine DOT geotechnical engineers.

The bank full stream width value for this reach of the stream has been determined by the Maine DOT environmental group to be 22'. Based on the survey plan prepared by the Maine DOT survey group, the 22' width corresponds to about elevation 498 just upstream and downstream of the bridge. Design bridge openings will be designed around the 1.2 bank full width, or 26.4' at elevation 498, as a minimum width through the bridge opening. A comparison of record drawings from 1962 and the current topographic survey, the stream elevation through the bridge has lowered from about 498 to elevation 495 in the intervening 54 years.

Environmental issues of concern include Atlantic Salmon critical habitat, brook floaters, and possibly mussels.

Measure the distance between multiple points on the ground

Length: 834.93 Feet

☐ Show Elevation Profile

☑ Mouse Navigation

The tributary watershed for this bridge is 18.3 square miles and is long and narrow, following the alignment of the East Branch of the Wesserunsett Stream. The shape of the watershed would lead one to believe that concentration times are short and intense storms will yield relatively rapid response from the watershed. The mean annual rainfall is 39.5 inches. Attenuation provided by ponds and wetlands is estimated to be only 1.3 square miles including Smith Pond near the top of the watershed. The stream flows through a 0.2 square mile wetland about 1.5 miles upstream from the bridge. The watershed is about 11 miles long with an average width of about 1.6 miles. The high end of the watershed above Smith Pond is at elevation 1250. The watershed drops some 750 feet over 60,000 feet for an average slope of 0.0125 feet per foot. There are no upstream or downstream dams that would regulate or create backwaters affecting the hydraulic calculations.

SUMMARY OF PEAK FLOWS

Drainage Area	18.3	mi^2
Q1.1	312.6	ft^3/s
Q10	1286.5	ft^3/s
Q25	1658.3	ft^3/s
Q50	1952.1	ft^3/s
Q100	2266.1	ft^3/s
Q500	3051.1	ft^3/s

Hydrology Reported by: Charles Hebson, PE Date: February 5, 2016

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

HYDRAULIC REPORT

CONTEXT

Gilman Bridge in Athens carries State Route 150 over a southerly flowing reach of the East Branch of the Wesserunsett Stream. The bridge crosses the stream at a significant angle causing a skew of over 54 degrees in the bridge. The existing bridge has been classified as scour critical with a supporting report prepared in July of 2011 by T.Y. Lin International. The existing single span rolled girder bridge spans 56 feet but the channel width at the top of footing is 28 feet as a result of the skew angle. The bridge is founded on spread footings set at elevation 494 on a gravelly sand soil. The footings have been undermined in one location by more than 6 feet with the top of footing exposed along the entire length of both abutments. The scour in this location is likely abutment scour caused by the high water flow direction impacting the easterly abutment at this location as the stream changes direction to follow its alignment. The stream channel under the bridge is generally about 2' lower in elevation than the stream channel immediately upstream and downstream of the bridge. Existing bridge plans confirm that erosive forces have lowered the elevation of the stream bed under the bridge over the past 53 years. 1962 record drawings show the stream bed at approximately elevation 499 feet at the face of the abutment. The most recent topographic survey records elevations nearer 496 feet.

ISSUES INVESTIGATED

Both scour and hydraulic capacity have been investigated. The existing condition, a proposed 75' span steel girder replacement bridge and a 90' span steel girder replacement bridge alternative have been considered.

METHODOLOGY

HEC-RAS has been used to predict water surface elevations and average flow velocities. Seven hydraulic cross sections were developed from the topographic survey provided by the Department, extrapolating the outside section boundaries where survey information did not exist. Because of the large skew, the bridge was modelled as part of the stream without using the bridge algorithm contained in the HEC-RAS software. Four of those seven sections have all or part of their cross section in portions of the barrel of the bridge. Section 248 has been selected as representative of headwater elevation and average flow velocity for reporting and scour calculations.

HEC-RAS' steady flow, mixed flow regime algorithm was utilized. The upstream and downstream boundary condition selected was normal depth with a slope of 0.012 ft/ft upstream and 0.012 ft/ft downstream. This slope was selected from the contours contained in the Department's MapViewer software. Roughness coefficients adopted for the analysis were; concrete wall – 0.015, riprap – 0.035, stream bed 0.035, overbank meadow 0.03, overbank brush 0.06.

HYDRAULIC ANALYSIS RESULTS

		Bridge Repair/Scour Countermeasures			**Replacement Bridge 75' Span**			**Replacement Bridge 90' Span**		
Storm	Peak Flow cfs	Water Surface Elevation	Flow Area SF	Channel Velocity fps	Water Surface Elevation	Flow Area SF	Channel Velocity fps	Water Surface Elevation	Flow Area SF	Channel Velocity fps
Q1.1	312.6	499.9	129	2.4	500.0	126	2.5	499.9	120	2.6
Q10	1286.5	503.1	222	5.8	503.1	255	5.2	503.1	245	5.4
Q25	1658.3	504.0	246	6.8	504.0	291	5.9	504.0	285	6.0
Q50	1952.1	504.5	261	7.5	504.5	315	6.4	504.5	312	6.5
Q100	2266.1	505.1	276	8.3	505.1	341	6.9	505.1	340	6.9
Q500	3051.1	506.4	312	9.9	506.4	402	8.0	506.4	408	7.8

A review of the results in the chart above shows that Q50 and Q100 water surface elevations remain about the same with a replacement bridge when compared with the existing conditions. Velocities through the bridge are reduced with both replacement bridges. The lowering of the velocity for the large storms will also reduce the risk of stream bed scour and undermining of the armored abutment slopes.

The width of the existing bridge is wider than the upstream and downstream stream channels. The vertical sides of the existing bridge have a smooth surface resulting in a relatively efficient hydraulic opening, but with high flow velocities. The replacement bridges have larger hydraulic openings but higher friction losses at lower elevations resulting in lower velocities but little change in predicted water surface elevation for the design storms.

The following chart shows the predicted roadway elevations, structural depths and clearance under the girder at the lower end of the lowest girder flange. All three options feature a prediction of more than 2’ of clearance over the Q50 headwater elevation. The existing bridge has a structural depth (vertical distance between the centerline finished grade and the bottom flange of the lowest girder at any given station) of 3.88’. At the low end of the bridge, the clearance over predicted Q50 headwater elevation predicted at HEC-RAS section 248 is 2.4’. The proposed vertical alignment raises the finished grade by about 5” at the downstream abutment for both the 75’ span option and the 90’ span option. The structural depth of the 75’ span steel grider bridge with 30” plate girders is 3.97 feet. The Q50 clearance for this option is predicted to be 2.6 feet. The 90’ span bridge will adopt a 36” deep plate girder with a structural depth of 4.47 feet. This increase in depth along with an increased hydraulic opening at higher flows predicts 2.1 feet of clearance over the Q50 headwater elevation at section 248.

The hydraulic capacity of the existing bridge is adequate; although the higher velocities exhibited through the bridge opening have contributed to the lowering of the stream elevation and the deep scour hole on the south side of the east abutment. The hydraulic capacity of the 75’ span and 90’ span options is actually lower for smaller flows as the proposed channel shape is trapezoidal rather than the rectangular shape that currently exists. The hydraulic capacities of the 75’ span option and the 90’ span option are very similar due to the fact that their respective cross sections are identical below elevation 500.5.

The chart below shows the assumptions and calculations undertaken to calculate the predicted clearance for the Q50 and Q100 design storms under all three alternatives considered.

Location	Station	Exist Centerline Elevation	Proposed Centerline Elevation	Slope	Structural Depth	Q50 Headwater Elevation **Clearance Feet**	Q100 Headwater Elevation **Clearance Feet**
PVI	587.00		512.08				
PVI	845.80		510.31	-0.0068			
Exist Roadway	625.00	511.56					
Existing Roadway	750.00	510.51		-0.0084			
Exist Abut #1	664.50	511.23					
Exist Abut #2	720.50	510.76			3.88	504.5	505.1
				Elevation Difference		**2.4**	**1.8**
Abutment #1 75' span	655.00	511.31	511.61	0.31			
Abutment #2 75' span	730.00	510.68	511.10	0.42	3.97	504.5	505.1
						2.6	**2.0**
Abutment #1 90' span	647.50	511.37	511.67	0.30			
Abutment #2 90' span	737.50	510.62	511.05	0.44	4.47	504.5	505.1
						2.1	**1.5**

SCOUR

The Maine DOT Bridge Design Guide, section 2.3.11, requires that scour be considered for both the Q100 storm and the Q500 storm. From the charts above and more detailed information contained in Appendix E, average channel velocities for these storms are shown for both design storms and for all three alternatives.

The scour analysis prepared by T.Y. Lin predicts total scour depths of over 28’ below the existing footing for the Q500 storm and over 5’ for the Q10 design storm. The T.Y. Lin Scour Report (July 2011) used the HEC 18 based algorithm embedded in HEC-RAS version used at the time. The D50 value of soil particle size used to predict critical velocity was 4 mm. Their analysis predicted zero contraction scour for the 10, 100 and 500 year design storms, but predicted very large scour depths due to abutment scour. The HEC algorithm used both the HIRE and Froehlich equations for abutment scour, both of which utilize the length of the intercepting embankment transverse to stream flow as a significant input parameter. The current version of HEC-23 Evaluating Scour at Bridges, April 2012, casts doubt on the validity of both of these equations based on the NCHRP (2010b) study, which maintains that values obtained by the use of the HIRE and Froehlich equations greatly overestimate abutment scour.

The approach recommended by this NCHRP study is as follows, quoted from page 8.4 of HEC-18.

> *Contraction scour should be viewed as the reference scour depth for calculating abutment scour. Abutment scour should be taken as the product of the contraction scour caused by flow acceleration through the constricted opening multiplied by a factor accounting for large-scale turbulence. This approach would replace the current approach for adding contraction scour to a separately computed abutment scour.*
>
> *Abutments should be designed to have a minimum setback distance from the channel bank of the main channel with riprap protection of the embankment and a riprap apron to protect against scour. The setback distance should accommodate the apron width recommended in HEC-23 (FHWA 2009).*

Contraction scour has been calculated for the Q100 and Q500 storms for all three options. HEC 18 chapter 6, equations 6.1, 6.2, 6.3, 6.4 and 6.5 are used to predict contraction scour. Input variables are obtained from HEC-RAS output as follows:

y	Average Depth of Flow Upstream of Bridge	Section 297 flow area/top width	
y0	Existing depth of water in contracted channel before scour	Section 248 W.S. Elev - Min Ch El	
W1	Bottom Width of upstream channel	65% of bank full width (assumed)	14.3 feet
W2	Bottom width of contracted channel	width at elevation 497 (assumed)	varies
K1	factor varying between 0.59 and 0.69	use 0.64 (this parameter is not sensitive)	
Q	peak flow through the bridge	design storm flows for Q100 and Q500 storms	
Q1	peak flow in channel (not in overbank) upstream	from HEC-RAS output Section 297	
Q2	peak flow rate in constricted channel	from HEC-RAS output Section 248	

D50 soil values for the stream bed materials were established by Maine DOT geotechnical engineers to range between 0.8 mm and 2 mm.

See the appendix for calculation of contraction scour. No contraction scour was predicted for either the Q100 or Q500 storm for the existing bridge; for the 75’ span steel girder option or for the 90’ span steel girder option. For the range of D50 values given, scour falls into the live bed category. The equations in HEC18 chapter 6 predict negative values for scour, therefore, no scour is predicted.

As discussed in the paragraphs above, HEC 18 chapter 8, Evaluating Local Scour at Abutments, recommends an alternate approach, described in paragraph 8.6.3 and following. Local abutment scour is thought to be better predicted as a factor applied to the theoretical depth of water to the bottom of the contraction scour hole predicted by the contraction scour equations. Even though contraction scour is predicted to be zero (or negative), when the factor is applied to the theoretical contraction scour water depth, abutment scour yields a positive value when this factored value is reduced by the pre-event water depth.

The calculations contained in appendix E, more fully illustrate the methodology described in HEC 18. The following chart shows the predicted depth of combined abutment scour and contraction scour for each of the three cases and both the Q100 and Q500 design storms.

	Contraction plus Abutment Scour Depth - feet	
Option	Q100	Q500
Existing Rehabilitation Option	0.0	0.0
75' Span Steel Girder Bridge Option	1.8	3.9
90' Span Steel Girder Bridge Option	1.8	4.6

The relatively wide bottom dimension in the existing condition helps the existing condition escape scour predictions for either contraction or abutment scour. The similarity in stream cross section geometry for lower flows explains the similarity in scour depths predicted for the Q100 storm. The additional scour depth for the 90’ span option over the 75’ span option for the Q500 storm is counter-intuitive.

The results of the scour calculations should be viewed with some judgement. The results are highly dependent on the magnitude of the input parameters and which hydraulic section the parameters are taken from. Past behavior of the stream interacting with the existing bridge should be given much more weight than the HEC 18 calculation results.

Over the life of this bridge, the stream elevation has lowered under the bridge. There is also a deep localized scour hole (6 feet deep) next to the easterly abutment footing at the downstream quarter point. It is clear that erosive forces over the service live of this bridge have undermined the footing in one localized area about 10’ long and threaten the remaining footings. Regardless of the results of the HEC 18 calculations, measures should be taken to protect the existing footings under the repair option.

Even though velocities diminish somewhat with the larger hydraulic openings afforded by the two steel girder options, sound judgement would make provision for long-term scour, especially localized abutment scour along the toe of the riprap placed in front of the southerly end of the east abutment.

SCOUR COUNTERMEASURES

Provisions to protect the replacement bridge from scour include the following. A well-founded riprap apron should be installed, stretching from the front face of the abutment to the stream bed. The piling supporting the abutment should be socketed into the supporting bedrock to provide the necessary structural stability and strength to support the bridge if the soil surrounding the abutment were to wash out in a large storm event. The toe of the riprap apron (at least on the southerly end of the easterly abutment) should be protected with a shallow steel sheet wall or with a similar gabion wall capable of resisting horizontal soil forces behind the exposed scour face and resistant to the erosive effects of the high velocity water flow.

Scour countermeasure options to be applied to the repair option include:

1. Articulating concrete block revetment material placed over stone underlain with a soil filter.
2. Partially grouted riprap
3. Full depth riprap
4. Grout bags

Collaboration and discussion with environmental regulators to arrive at the least impactful approach to scour mitigation should be undertaken prior to choosing a scour countermeasure approach in the event that the repair option is chosen.