MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM

HYDROLOGY
&
HYDRAULICS
REPORT

For the Replacement of:

Lower Sandy Bridge
Over
Sandy Stream
In
Lexington TWP, Maine

Br #5128

WIN 019291.00
AC-BR-1929(100)X

(Excerpted from PDR)

MAINEDOT
Maine Department of Transportation

The Lower Sandy Bridge (#5128) on Long Falls Dam Road in Lexington TWP spans over the Sandy Stream. The Sandy Stream originates up in Carrying Place Township in Otter Pond at around elevation 1262 feet (NGVD). The stream then flows southerly through Middle Carry Pond and on through the mountains in Highland Plantation picking up gradient. Once the stream comes into Lexington TWP, it flattens out to a low lying swampy area named Lexington Flats. Throughout the flats and under the bridge, the Sandy becomes a meandering stream, with multiple overlapping floodplains. The topography of this area suggests that during flooding, once the stream banks are overtopped, the whole area becomes inundated. Just downstream of the bridge, the Sandy Stream terminates as it flows into Gilman Pond at a water surface elevation of about 372 feet (NGVD). It's important to note that a small tributary of the Sandy Stream, or possibly even another branch, meets the main channel just upstream of the northeast wingwall at the bridge site (see photographs in Appendix A).

Very little information exists on the flow data for the Sandy Stream through Lexington TWP. There are no FEMA flood insurance studies or rate maps. The drainage basin characteristics for the bridge were provided by the Maine Department of Transportation Environmental Office - Hydrology Section. The flows were computed using the 1999 USGS full regression equation. Additional information was also available from the Bartlett Bridge PDR completed in 2006 (PIN 012635.00). Bartlett Bridge is located just downstream of Gilman Pond in New Portland. There is an inconsistency between the peak flow data reported in the Bartlett Bridge PDR and the peak flow data provided for the Lower Sandy Bridge. The provided flows for the Lower Sandy Bridge are higher than those reported in the Bartlett Bridge PDR, even though the Lower Sandy Bridge is further upstream. This inconsistency is due to an error in the drainage basin analysis for the Bartlett Bridge.

Additional flood history of the Bartlett Bridge, specifically that of the 1987 flood, was provided by the dam owner of the private dam immediately downstream of Bartlett Bridge. This data was helpful in determining flood elevations at the Lower Sandy Bridge. The flows used in design are therefore those calculated using the 1999 USGS full regression equation and these flows were qualitatively compared against the 1987 flood information available for the Bartlett Bridge.

Summary

Drainage Area = 62 mi^2

Ordinary High Water ($Q_{1.1}$) = 897.4 cfs

10 Year Flood (Q_{10}) = 3,294.8 cfs

25 Year Flood (Q_{25}) = 4,157.8 cfs

Design Discharge (Q_{50}) = 4,829.5 cfs

Check Discharge (Q_{100}) = 5,540.5 cfs

Q_{500} = 7,292.1 cfs

Input from the public on flood history was obtained at the preliminary public hearing and through several phone calls. Basically, what was learned from the hearing is that during normal flooding, the bridge is the high point of the road and the approaches on either end of the bridge experience overtopping. One audience member reported that the highest he's seen the water at the bridge was 8 to 10 inches below the steel. Buster Pinkham, owner of the Happy Horseshoe Campground for 31 years, was asked specifically about the 1987 flood. He recalls the water level at the bridge coming close to the elevation of the road surface, but not quite overtopping the bridge. Cliff Marchant, who owns a camp just north of the bridge on Long Falls Dam Road, was also asked about flood history. He remembers only one time when major flooding occurred. It sounded like from his timeline that it was in fact the 1987 flood. During this event, he recalls the bridge actually being overtopped. Finally, it is clear from past inspection photos and underwater reports that debris can be an issue on site.

Reported By: Richard E. Myers, P.E.
Date: July 2, 2012

Note: Relevant data and reports are provided in the appendix of this Preliminary Design Report.

Hydraulic Modeling

The existing bridge and proposed structure were analyzed using HEC-RAS, version 4.0, the river analysis software developed by the U.S. Army Corps of Engineers. The HEC-RAS model for this project represents a 214 foot long segment of Sandy Stream, including the bridge at the stream crossing on Long Falls Dam Road. The small tributary coming off of the upstream northern wingwall was not modeled for simplicity. Quantitative effects of the small tributary would be negligible. Its presence is more important from a qualitative perspective for layout of the proposed bridge and treatment of the northern approach. Two cross-sections upstream of the bridge and two downstream make up the geometry of the stream. The existing bridge was modeled and run with peak flows and varying boundary conditions until the results correlated well with the information provided by the public and the data in the Bartlett Bridge PDR.

As described in the hydrology report, the Sandy Stream throughout Lexington TWP is located in a low lying swampy area named Lexington Flats. Throughout this reach, the Sandy is a meandering stream with multiple overlapping floodplains. The topography of the area suggests that during flooding, once the stream banks are overtopped, the whole area becomes inundated. Furthermore, since the area is very flat, any water in the floodplain will mostly be standing water and not contribute to streamflow. This type of stream environment is very difficult to model accurately once the flow enters the floodplain. Therefore, the intent of the hydraulic analysis for this project is to determine approximate stream velocities at peak flows for scour computations and sizing of riprap. Headwater surface elevations and freeboard will be assessed qualitatively. Since according to the public, the existing bridge has had few flooding issues other than what sounds like one major event, the goal will be to at least maintain existing freeboard. Following is a discussion of each of the parameters used in the final analysis as well as other assumptions made for both the existing and proposed models.

- Steady flow
- Manning's "n"
    - Channel = 0.026
    - Overbanks = 0.12

  Manning's "n" values for both the channel and floodplain were determined using guidance provided in a U.S.G.S. document titled "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains." The channel roughness value reflects a fine streambed material ($n_b = 0.02$), somewhat eroded banks with exposed tree roots ($n_1 = 0.002$), and some debris in the stream ($n_3 = 0.004$) such as logs. The floodplain roughness value was determined qualitatively using photographs of floodplains where roughness coefficients have been verified. The photograph in figure 13 of the document most closely represents the floodplain of the Sandy Stream in the vicinity of the project.

- Contraction/Expansion coefficients

  The default values of 0.1 for contraction and 0.3 for expansion were used.

- Ineffective flow areas

  The existing bridge creates a constriction on the channel and at high flows when the banks are overtopped, there will be ineffective flow in the floodplain. The ineffective flow areas were determined by drawing a line parallel to the existing wingwalls and where that line intersected each stream cross section is where the cutoff was assumed for ineffective flow. The elevation for ineffective flow was set at the road elevation because once the road overtops, the water can flow over the road. For the proposed structure, a similar approach was used except that the constriction on the floodplain was assumed to be at 1:1 off of the proposed wingwall.

- Subcritical flow regime

- Boundary conditions

  Since there is no hard flood elevation data at the bridge, normal depth was used as the boundary condition and different magnitudes of stream slope were input and run until resulting water surface elevations approximately matched what the public had witnessed. It would have been difficult to get an accurate average stream slope from just the survey data because of the short length of survey and the variability of the streambed elevations in that length. The resulting stream slope used for the normal depth boundary condition is 0.0002 ft/ft.

Once the model was calibrated, peak flows were run through the existing structure and proposed structure for comparison. Since the existing structure is a through-girder style bridge, the proposed vertical alignment raises the grade to allow for a traditional slab-on-girder bridge while maintaining a bottom-of-superstructure elevation. Following is a list of assumptions made for the proposed structure:

- 108’-0” single span
- Superstructure with 3” of pavement, ¼” membrane, 8” slab, and 43” steel girders
- Integral abutments with embankments sloped down to the existing abutments cutoff near Q1.1.

Headwater elevations and stream velocities are reported in the table below for peak flows for the existing and proposed structures. The overall open area has increased between existing and proposed, so the headwater elevations and velocities should decrease as is portrayed by the results. Even though the headwater elevations decreased slightly, there is no freeboard at Q50 for the proposed structure because the superstructure depth is slightly deeper than existing. The difference however is very little between existing and proposed and due to the accuracy of the model, the actual Q50 elevation could be above or below the bottom of the proposed superstructure.

SUMMARY

		Existing Structure	Proposed Bridge
Unit		71'-0" Span	108'-0" Span
	Area of Opening (s.f.)	1,036	1,208
	Headwater El. Q1.1	372.86	372.86
	Headwater El. Q10	378.71	378.69
ft	Headwater El. Q25	380.14	380.10
	Headwater El. Q50	381.15	381.10
	Headwater El. Q100	382.18	382.11
	Headwater El. Q500	384.57	384.41
	Vertical Clearance		
	@ Q50	0.16	None
	@ Q100	None	None
	Discharge Vel. Q1.1	1.88	1.69
	Discharge Vel. Q10	3.84	3.27
ft/s	Discharge Vel. Q25	4.38	3.69
	Discharge Vel. Q50	4.76	4.08
	Discharge Vel. Q100	5.35	4.68
	Discharge Vel. Q500	6.95	6.13

*Elevations based on NAVD

According to the Bridge Design Guide, for other riverine bridges, two feet minimum vertical clearance between the bottom of the superstructure and the Q50 backwater elevation is preferred where there has been no history of ice jams. In addition, the bridge should be capable of passing the Q100 or the flood of record, whichever is greater, without any serious harm to the structure, roadway, or adjacent property. When possible, it is preferred to have one foot of freeboard at Q100. These criteria are not satisfied. Since conditions are improving by increasing the span length and there have been no known issues with ice jams, it was determined that this hydraulic opening is satisfactory for the priority of road. In addition, for a pile-supported integral abutment style bridge, even if the superstructure experienced some load from water, ice, or debris, the overall structure should stay stable.

Scour countermeasures will be in place and the piles are anticipated to be around 120 feet long. Stream velocities are relatively low and the approach embankments will be protected by plain riprap.

Scour

As discussed in the Summary of Preliminary Design, scour has already been an issue at this bridge site. The existing bridge causes a constriction on the stream resulting in debris buildup and extensive scour. In 1996, grout bags were placed under and in front of the existing abutments to prevent scour from getting worse and causing a potential bridge failure. The scour holes under the abutments were at most about 2 feet deep, 8 feet back, exposing some of the timber piles. According to an underwater report in 2000, free-flowed grout in front of the grout bags has begun to undermine. The current topography of the stream shows that there is still a large scour hole under the bridge. If the streambed was approximated to be at a constant slope through the bridge using the survey information up and downstream, the maximum depth of scour from this theoretical streambed that has already occurred is 10.5 feet.

Future scour susceptibility and magnitude for the proposed bridge structure was analyzed using the procedures and guidelines provided in Hydraulic Engineering Circular (HEC) 18 - Evaluating Scour at Bridges, 5th edition, published by FHWA/NHI in April of 2012. Total scour is separated into three components:

- Long-term aggradation and degradation
- General scour
- Local scour

Aggradation and degradation are long-term streambed elevation changes due to natural or man-made causes. Aggradation consists of the deposition of streambed material at the bridge due to erosion upstream of the bridge in the watershed. Degradation involves the lowering or scouring of the streambed at the bridge due to erosion and an imbalance of material carried away from the bridge and material carried in. In order to assess past and future aggradation and degradation, two sources of information were considered. First, the streambed profile from the existing plans was compared to the streambed profile from present day survey. As described in the calculations (refer to the appendix of this report), neglecting the presence of the existing scour hole, the current streambed would be at approximately the same elevation as it was in 1928. The other source of potential degradation at this site is the possibility of a dam removal in New Portland, just downstream of the Bartlett Bridge. Upon reviewing the Bartlett Bridge plans, it was found that bedrock is fairly shallow under Gilman Stream in New Portland at the bridge. Combine this information with the fact that Gilman Pond lies between Sandy Stream and Gilman Stream, it was determined that degradation of the streambed will likely not stretch back as far as the Lower Sandy Bridge. Therefore, the amount of scour due to aggradation and/or degradation under the proposed structure is recommended to be zero.

The most common type of general scour is contraction scour. Other types include erosion due to planform characteristics (such as dunes), river bends, variable downstream control, or other changes to the river that decrease streambed elevation at the bridge. Contraction scour is the only type anticipated to occur at the proposed bridge. The river reach is fairly straight through the bridge and at high flows, the streambed will likely be fairly flat. In its simplest terms,

contraction scour is essentially the result of the river cross section under the bridge trying to achieve equilibrium when the area is constricted as compared to the upstream cross sections. Contraction scour was evaluated using equations provided in HEC 18. Water depth, width, flow, and streambed grain-size are all contributing factors to the magnitude. The geotechnical team provided a grain-size distribution for two samples of the streambed. The samples were about equal with D_{50} equal to 0.14 mm for one sample and 0.13 mm for the other. All calculations are based on D_{50} equal to 0.13 mm. The latest version of HEC 18 also includes guidance on predicting additional scour when a bridge opening is flowing full under a pressure flow situation. This type of scour is called vertical contraction scour. Since the hydraulic analysis for the proposed structure predicts that the bridge will be flowing full during the Q100 and Q500 events, vertical contraction scour was included as part of the total scour analysis. Both Q100 and Q500 flood events were analyzed and the results are shown below. Supporting calculations are included in the appendix.

Contraction Scour

	Scour Depth from Existing Streambed	Resulting Thalweg Elevation
Q_{100}	4.4 ft	356.0 ft
Q_{500}	7.9 ft	352.5 ft

Local scour involves the removal of material around piers and abutments due to acceleration of flow and the resulting vortices induced by obstruction to the flow. Local scour is highly dependent on structure geometry and shape as well as flow angle of attack, velocity, and depth. Local scour can also be increased by debris lodged on the substructure unit. For this project, local scour was not computed. The proposed abutments are to be built behind the existing with plain riprap slope embankments. The riprap will be placed up to elevation 382 feet which is between Q50 and Q100. Considering the fact that the proposed abutments will be pile supported, protected by riprap, and built behind another pile-supported concrete structure with grout bags underneath, local scour should be negligible. The proposed pile foundation will be designed for conditions associated with the maximum contraction scour depth and no other sources of scour.

Reported by: Richard E. Myers, P.E.
Date: July 2, 2012

Note: Supporting documentation is provided in the appendix of this Preliminary Design Report.