The Maine - Kennebec Bridge (#2506) over the Kennebec River on State Route 197 connects the towns of Richmond and Dresden, Maine. The Kennebec River drainage basin is located entirely in the state of Maine and drains approximately one-fifth of the state. Beginning at the outlet of Moosehead Lake, the Kennebec River flows south for approximately 145 miles to Abagadasset Point in Merrymeeting Bay where it joins with the Androscoggin River and four smaller rivers before traveling another 20.5 miles to its mouth in the Atlantic Ocean. The Maine - Kennebec Bridge is located approximately 26.4 miles upstream of the mouth of the river. The tide range at the bridge is approximately five feet. The width of the river during normal stages at the bridge is approximately 1,010 feet at low tide and 1,280 feet at high tide. The existing bridge is aligned well with the river; the bridge itself is perpendicular to flow on a straight tangent and the piers are aligned parallel to the flow.

Flooding of the Kennebec River generally occurs in the spring as a result of heavy rainfall combined with snowmelt. In addition, ice jams on the Kennebec are an annual concern. The most notable recorded floods in the area of Richmond and Dresden were in March of 1936 and April of 1987. The flood in 1987 had a peak discharge of 232,000 cubic feet per second at the U.S.G.S. gage in Sidney (recurrence interval of over 100 years) which is the highest on record there. According to [4], the estimated peak discharge during the 1936 flood in Augusta was 160,000 cubic feet per second; however, due to the presence of ice jams, the flood resulted in the highest flood elevations on record. At the bridge, the 1936 flood had a peak elevation of approximately 18.8 feet (NAVD) and the ice actually took out the recently completed structure (see figures 1 and 2). The flood of 1987 had a peak elevation of approximately 13.8 feet (NAVD) at the bridge, but unlike the flood of 1936, it was a free flow event.

Figure 1: 1936 ice jam event at Maine - Kennebec Bridge

Figure 2: Existing bridge span carried downstream

The hydrologic analysis for the Maine - Kennebec Bridge (#2506) over the Kennebec River on State Route 197 can be separated into two categories: river flow and tidal flow. The drainage basin characteristics of the Kennebec River were provided by the Maine Department of Transportation Environmental Office - Hydrology Section in partnership with the Bridge Program. The drainage area was determined by typical methods and compared to multiple reference documents. All of the resources were in good agreement. The peak flows were determined by a combination of methods. First, peak flow data from a U.S.G.S. gaging station on the Kennebec River near Waterville, Maine, was used to estimate peak flows at the bridge site by weighting the drainage area difference between the two sites. Then, these computed peak flows at the bridge site were calibrated to historical records and the Flood Insurance Studies of the two towns. This calibration essentially comprised of increasing the peak flows computed from the weighting method by adjusting the amount of wetlands until Q100 matched what was used in the Flood Insurance Studies. The resulting Q100 correlates well with the amount of flow recorded during the 1987 flood at the gaging station in Waterville and also one in North Sidney, Maine.

Summary

Drainage Area = 5841 mi^2

$Q_{1.1}$	=	61,028	cfs
Q_{10}	=	159,473	cfs
Q_{25}	=	188,743	cfs
Q_{50}	=	210,703	cfs
Q_{100}	=	233,000	cfs
Q_{500}	=	286,768	cfs

As mentioned previously, the Kennebec River at the bridge site does experience tidal flow. Tidal information was obtained through two different sources:

- Tide data available through the National Oceanic and Atmospheric Administration (NOAA) website
- Gages installed by Maine DOT just upstream of the existing bridge

The tidal datums on the NOAA website are based on data collected at a benchmark in Richmond and referenced to the 1983 - 2001 tidal epoch. The gage installed by Maine DOT was located just upstream of the existing bridge near the ledge outcrop on the Dresden side of the river. The gage collected data from November 15, 2011 to December 28, 2011. In comparing the data from the two sources, it was found that the mean lower low water elevation reported by NOAA correlated well with the minimum low tide elevation found by Maine DOT. On the contrary, the mean higher high water elevation reported by NOAA was significantly lower than the maximum high tide elevation found by Maine DOT by about two feet. The difference likely stems from a difference between the averages of high tides versus a single maximum high tide. Therefore, for the purposes of reporting tidal datums, the information from the NOAA website will be referenced. For the purposes of hydraulic modeling, the actual data collected by Maine DOT will be used because it represents an actual maximum and minimum tide elevation. Finally, the new bridge design should account for the expectation that mean sea level will rise one to two feet over the next 100 years. It is recommended for this project to assume a mean sea level rise of 1.6 feet for future tidal datums. Shown below are two tables; the first reports the current tidal datums and the second reports the adjusted tidal datums for a 1.6 foot rise in sea level. All elevations reported below are referenced to the NAVD of 1988.

<u>Current tide datums</u>

Mean Higher High Water (MHHW)	=	3.92	ft	
Mean High Water (MHW)	=	3.60	ft	
Mean Low Water (MLW)	=	-1.69	ft	
Mean Lower Low Water (MLLW)	=	-1.87	ft	
Highest Predicted Astronomical Tide (HAT)	=	5.11	ft	
Lowest Predicted Astronomical Tide (LAT)	=	-2.71	ft	
Maximum High Tide (MaineDOT gage)	=	5.88	ft	November 24, 2011
Minimum Low Tide (MaineDOT gage)	=	-1.83	ft	November 21, 2011

<u>Future tide datums</u>

Mean Higher High Water (MHHW)	=	5.52	ft
Mean High Water (MHW)	=	5.20	ft
Mean Low Water (MLW)	=	-0.09	ft
Mean Lower Low Water (MLLW)	=	-0.27	ft
Highest Predicted Astronomical Tide (HAT)	=	6.71	ft
Lowest Predicted Astronomical Tide (LAT)	=	-1.11	ft
Maximum High Tide (MaineDOT gage)	=	7.48	ft
Minimum Low Tide (MaineDOT gage)	=	-0.23	ft

Additional information was gathered by speaking to the bridge operator, Elwin Page, and reviewing historical photos. The existing bridge has not been overtopped, but the Dresden

approach has been. Ice is a serious concern here. As mentioned previously, the 1936 flood and ice jam event took out the existing bridge. Elwin Page reported that he has seen ice clear across the entire river and probably as thick as 10 feet. Also note that according to the latest inspection report, the current scour rating of the existing bridge is T, meaning tidal, and it is considered low risk.

Reported By: Richard E. Myers, P.E.
Date: March 12, 2012

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

References

1. Flood Insurance Study for the town of Richmond issued in 1998

2. Flood Insurance Study for the town of Dresden issued in 1998

3. "Kennebec River - 1936 Flood Profile & Low Flow Profile - Richmond to Bingham" issued by the U.S. Army Corps of Engineers

4. "Special Flood Hazard Information Report - Kennebec River" prepared by the Department of the Army, New England Division, Corps of Engineers in 1975

5. "Drainage Areas of Surface Water Bodies of the Kennebec River Basin in Southwestern Maine" issued by the U.S.G.S. in 1980

6. "Flood of April 1987 in Maine" issued by the U.S.G.S. in 1991

7. "Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals" issued by the U.S.G.S. in 1999

Hydraulic Modeling

The existing and proposed bridge structures 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 2,692 foot long segment of the Kennebec River, including the bridge carrying Route 197. Fifteen total river cross-sections were developed plus an upstream and downstream bridge cross-section. Note that the existing fenders were not modeled because they are directly in-line with pier 5 and are approximately the same width. The furthest cross-section downstream of the bridge (hereinafter referred to as section 361 in HEC-RAS) is a hybrid between input data provided by U.S.G.S. of the Dresden Flood Insurance Study (FIS) hydraulic model and digital topography created by the Department's survey team. All of the other cross-sections are based on the aforementioned digital topography. The following assumptions are consistent throughout each of the models:

- Steady flow
- Manning's "n"
    - Main channel = 0.025
    - Weedy high tide areas of channel = 0.027
    - Wooded areas with thick brush = 0.16
    - Thin wooded areas with little brush = 0.11
    - Light brush = 0.055
    - Lawn grass = 0.03
    - Paved areas = 0.015

In order to calibrate and verify that the hydraulic model was reasonable, the existing bridge was modeled and run with a Q100 flood event and checked against WSPRO output data provided by U.S.G.S. of the Kennebec River model for the Dresden FIS. WSPRO, short for Water Surface Profile, is essentially an older style of HEC-RAS. It's important to note that the U.S.G.S. model incorporated the entire stretch of the Kennebec River in the town of Dresden, which is close to a 45,000 foot reach of the river, and is part of a much larger network of modeling on the river for neighboring towns. The link between the U.S.G.S. WSPRO model and the HEC-RAS model was section 361 and 2435 in HEC-RAS. Section 361 in the HEC-RAS model corresponds to section G of the FIS. This cross-section is located at the limits of the survey for the project, close to the head of Swan Island. This cross-section coincidentally is the only cross-section in the U.S.G.S. model between the existing bridge and Swan Island, so it was an ideal candidate for the downstream boundary condition. Section 2435 corresponds to section I in the U.S.G.S. model and is the only upstream cross-section in both the survey and the U.S.G.S. model. Verification of the HEC-RAS model consisted of inputting the water surface elevation computed for section 361 at Q100 in the U.S.G.S. model as the downstream boundary condition and comparing the computed upstream water surface elevation at section 2435. The results were almost identical.

Once the existing model was verified, boundary conditions had to be determined for the remaining flood profiles. These boundary conditions had to be broken into two categories: affected by tide and unaffected by tide. The lower flow profiles, including Q1.1, Q2, Q5, and Q10 were all examined for tidal influences. If the profile was to be affected by the tide, a known water surface elevation boundary condition was input at section 361. Through trial and error, the

boundary condition was changed until the water surface elevation closest to the tide gage (see Hydrology Report) matched the recorded minimum or maximum tide elevation. Low tide only affected Q1.1. High tide affected Q1.1, Q2, and Q5. Note that the tide gage was located closest to section 1415 in HEC-RAS.

Since water surface elevations are unknown for the remaining flood profiles (Q10, Q25, Q50, and Q500), another type of boundary condition was required. For subcritical flow, HEC-RAS gives the option of using normal depth as a downstream boundary condition which is a calculated uniform depth based on an inputted slope of the river. In order to determine the appropriate slope for input, different slopes were tried for the Q100 flood profile as a boundary condition until the results matched those obtained previously during verification with a known water surface elevation. This resulting slope was then used for the remaining profiles. The following is a list of the existing bridge model runs and corresponding boundary conditions:

Profile	**B.C. Type**	**B.C. Value**	**Notes**
$Q_{1.1}$	Known water surface elevation	Elev. = -1.99	Low tide
$Q_{1.1}$	Known water surface elevation	Elev. = 5.84	High tide
Q_2	Normal depth	Slope = 0.000135	Low tide
Q_2	Known water surface elevation	Elev. = 5.77	High tide
Q_5	Normal depth	Slope = 0.000135	Low tide
Q_5	Known water surface elevation	Elev. = 5.68	High tide
Q_{10}	Normal depth	Slope = 0.000135	
Q_{25}	Normal depth	Slope = 0.000135	
Q_{50}	Normal depth	Slope = 0.000135	
Q_{100}	Known water surface elevation	Elev. = 12.57	
Q_{500}	Normal depth	Slope = 0.000135	

The proposed bridge was modeled in HEC-RAS based on geometry and pier location given by the design team. The bridge was analyzed with present day and future tide levels. For the present day model, the boundary conditions listed above for the existing bridge were used. For the future model, the boundary conditions listed above for the existing bridge were modified to account for an increase in tide of 1.6 feet (as discussed in the Hydrology Report). These modifications are shown in the table below:

Profile	**B.C. Type**	**B.C. Value**	**Notes**
$Q_{1.1}$	Known water surface elevation	Elev. = -0.39	Low tide
$Q_{1.1}$	Known water surface elevation	Elev. = 7.44	High tide
Q_2	Known water surface elevation	Elev. = 7.37	High tide
Q_5	Known water surface elevation	Elev. = 7.28	High tide

Maximum headwater elevations and river velocities are reported in the table below for peak flows for the existing bridge and the proposed bridge with current and future tide levels. In the table below, the tide only affected Q1.1. As expected, the maximum headwater elevation at Q1.1 occurs at high tide whereas the maximum velocity under the bridge at Q1.1 occurs at low

tide. The area of opening shown below, although dramatically different between existing and proposed, had little effect on the models. The key to the decrease in headwater elevation and velocity between existing and proposed is the change in width. The proposed bridge has fewer piers than the existing bridge and the proposed Dresden abutment embankment will be set back outside the channel rather than projecting into it. Note that Lincoln Road overtops at about a Q25 flood event in both existing and proposed models. The low point on Lincoln Road is at an approximate elevation of nine feet. It's also important to note that all elevations and velocities reported below are for free-flow conditions in the Kennebec. Ice jam effects were not incorporated in the hydraulic model due to the uncertainty involved in modeling ice in HEC-RAS. It is recommended that the 1936 flood elevation on record at the existing bridge be used as a worse case headwater elevation for the proposed bridge under an ice jam condition. This elevation is reported below. Velocities during an ice jam event will likely be less than free-flow conditions.

		Existing Bridge	**Proposed Bridge**	**Proposed Bridge**
Unit			Present Day	Future
	Area of Opening (s.f.)	39,600	104,400	
	Headwater El. Q1.1	5.88	5.88	7.47
	Headwater El. Q10	7.45	7.43	7.43
ft	Headwater El. Q25	9.79	9.76	9.76
	Headwater El. Q50	11.38	11.34	11.34
	Headwater El. Q100	12.84	12.80	12.80
	Headwater El. Q500	16.42	16.37	16.37
	1936 Flood El.	18.82		
	Velocity Q1.1	3.95	3.70	3.37
	Velocity Q10	6.31	5.93	5.93
ft/s	Velocity Q25	6.78	6.38	6.38
	Velocity Q50	7.12	6.71	6.71
	Velocity Q100	7.46	7.04	7.04
	Velocity Q500	8.11	7.69	7.69

*Elevations based on NAVD

Scour

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, 4th edition, published by FHWA/NHI in May of 2001. 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. With the exception of the appearance of a few localized scour holes, the two profiles are very close in elevation to each other. Note that the localized scour holes correspond to the location of existing piers, numbers 4 and 6, and a large mass of debris picked up by the consultant hydrographer, Substructure. Past underwater inspection reports were also reviewed for any indication of aggradation or degradation and again, nothing noted other than a few localized scour holes. 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 at proposed piers 4 and 6 (the only two piers potentially being founded on pile rather than directly on bedrock). Water depth, width, flow, and streambed grain-size are all contributing factors to the magnitude. The geotechnical team provided grain-size distributions for piers 3 through 6. At pier 4, D_{50} throughout the contraction scour depth is 0.57 mm and at pier 6, it's 0.16 mm. Both Q100 and Q500 flood events were analyzed and the results are shown below.

Contraction Scour

	Pier 4	Pier 6
Q_{100}	1.0 ft	2.4 ft
Q_{500}	1.5 ft	3.4 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. Local pier scour was evaluated for proposed piers 4 and 6. There are two methods provided in HEC 18 for calculating local pier scour, simple and complex. The simple method essentially assumes a constant pier width throughout the entire depth of scour whereas the complex method determines the progression of scour depths due to varying widths throughout the height of the pier and scour hole. The complex method was developed for piers consisting of a shaft, pile cap, and piles, all within the scour depth. For the purposes of the Preliminary Design Report, the simple method was employed for estimating local pier scour depth. During final design, when the pier geometry is set, local scour may be reevaluated using the complex method. The advantage to reevaluating is that the complex method may result in shallower scour depths.

Preliminary calculations by the design team give a pier shaft width of 8 feet. Since this is the width of the pier most exposed to the flow, this will be the base width used for preliminary scour computations. In order to account for debris lodged on a pier, it is common to increase the pier width used in the calculations. For this project, it is assumed that the pier width is increased 2 feet due to the presence of debris, thereby giving 10 feet as a total width for scour computations. Similar to contraction scour, Q100 and Q500 were analyzed for local pier scour and the results are shown below. Also shown are other assumptions related to the calculation of local pier scour. Note that the angle of attack for pier 6 is assumed to be 11 degrees. This is an approximation considering overbank and contraction influence between pier 6 and the Dresden overbank area.

Local Pier Scour

	Pier 4	Pier 6	Notes
Q_{100}	16.6 ft	15.9 ft	
Q_{500}	17.6 ft	17.9 ft	
D_{50}	0.57 mm	0.16 mm	used mainly for contraction scour
K_1	1.0	1.0	round nose shaft
K_2	1.0	1.39	approx. 11° angle of attack on pier 6
K_3	1.1	1.1	plane streambed (no dunes)
K_4	1.0	1.0	$D_{50} < 2$ mm
a	10 ft	10 ft	constant pier width plus debris

As can be seen above, local scour is by far the largest contributor to total scour. Note that local scour was not computed for the abutments. The Richmond abutment is anticipated to be a full-height cantilever type abutment founded directly on bedrock and the Dresden abutment is anticipated to be a stub abutment on piles founded on bedrock; both will be located well away from the channel. Traditionally, riprap is placed up to the Q50 elevation to protect abutment embankments from scour and erosion. In this case, riprap extents shall be up to the Bridge Engineer. It is recommended in this report to provide, at a minimum, plain riprap along the slopes of the rebuilt portions of Lincoln Road adjacent to the river. As mentioned previously, it is expected that Lincoln Road will overtop at Q25 and above. Plain riprap, according to the MaineDOT Bridge Design Guide, should be adequate up to average water velocities of 12 feet per second. The river velocity out near Lincoln road was computed to be about 6 feet per second.

Below is a summary table of total scour. All contraction and local scour computations are available upon request.

Total Scour

	Pier 4	Pier 6
Q_{100}	17.6 ft	18.3 ft
Q_{500}	19.1 ft	21.3 ft

Reported by: Richard E. Myers, P.E.
Date: March 12, 2012

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