HYDROLOGY REPORT

Stimson Bridge carries State Route 5 over the Little Ossipee River, connecting the towns of Waterboro and Limerick, Maine. The Little Ossipee River begins in Acton, Maine at Balch Pond Dam and flows northeast for approximately 30 miles to its confluence with the Saco River in the town of Limington, Maine. The Little Ossipee River is approximately 120 feet wide at the location of Stimson Bridge, which is located approximately 14.3 miles upstream of its confluence with the Saco River.

The drainage basin characteristics for the bridge are based on the comparison of projected peak flow rates from three sources. The first source is a 2007 report on the estimated magnitudes and recurrence intervals of peak flows on the Mousam and Little Ossipee Rivers for the flood of April 2007, with the estimated peak flows generated using the Expected Moments Algorithm (EMA) (Cohn and others, 1997; Cohn and others, 2001; Griffs and others, 2004) and the flows from the April 2007 flood. The second source is the Maine Department of Transportation (MaineDOT) Environmental Office – Hydrology Section which provided peak flows computed using the 1999 U.S.G.S. full regression equation. The third and final source is the Federal Emergency Management Agency (FEMA) flood insurance study for York County Maine, which generates peak discharges from a county wide analysis using drainage area relationships for the entire County. The three sets of peak flow data were further examined in cooperation with Charles Hebson from the MaineDOT Environmental Office - Hydrology Section, where it was determined that the peak flows from the 2007 report using the EMA estimates were the most appropriate as it is the most up to date analysis of the Little Ossipee River. The MaineDOT flows are the result from information collected prior to the April 2007 flood, and the FEMA flood insurance study values are larger scale estimates resulting in overly conservative flows, making both sets of data less accurate than the 2007 report flows (see Appendix E - Hydrology & Hydraulic Information for more information). The 2007 report based on the April 2007 flood does not estimate a $Q_{1.1}$ flow rate so this value was determined by projecting Q1.1 on a log-normal probability plot based on the other data points; see Figures 1 & 2 for more information.

The Little Ossipee River typically floods in the spring due to the combination of heavy rainfall, snowmelt and ice jams. The flood of record at the Route 5 crossing was the March 1936 flood with a peak discharge of 8,530 cubic feet per second at the Little Ossipee Flowage Dam located near the bridge in Limerick and Waterboro according to the FEMA flood insurance study. This event exceeded the 100 year recurrence interval for the river at the bridge location with a peak water elevation of 314.34 ft (NAVD 88) when the peak discharge is modeled using a HEC-RAS model. Similarly, the April 2007 flood had a recorded peak discharge of 8,220 cubic

feet per second for the Little Ossipee River, which also exceeds the 100 year recurrence interval for the river at the bridge location, with a peak water elevation of 314.11 ft (NAVD 88), when modeled using HEC-RAS.

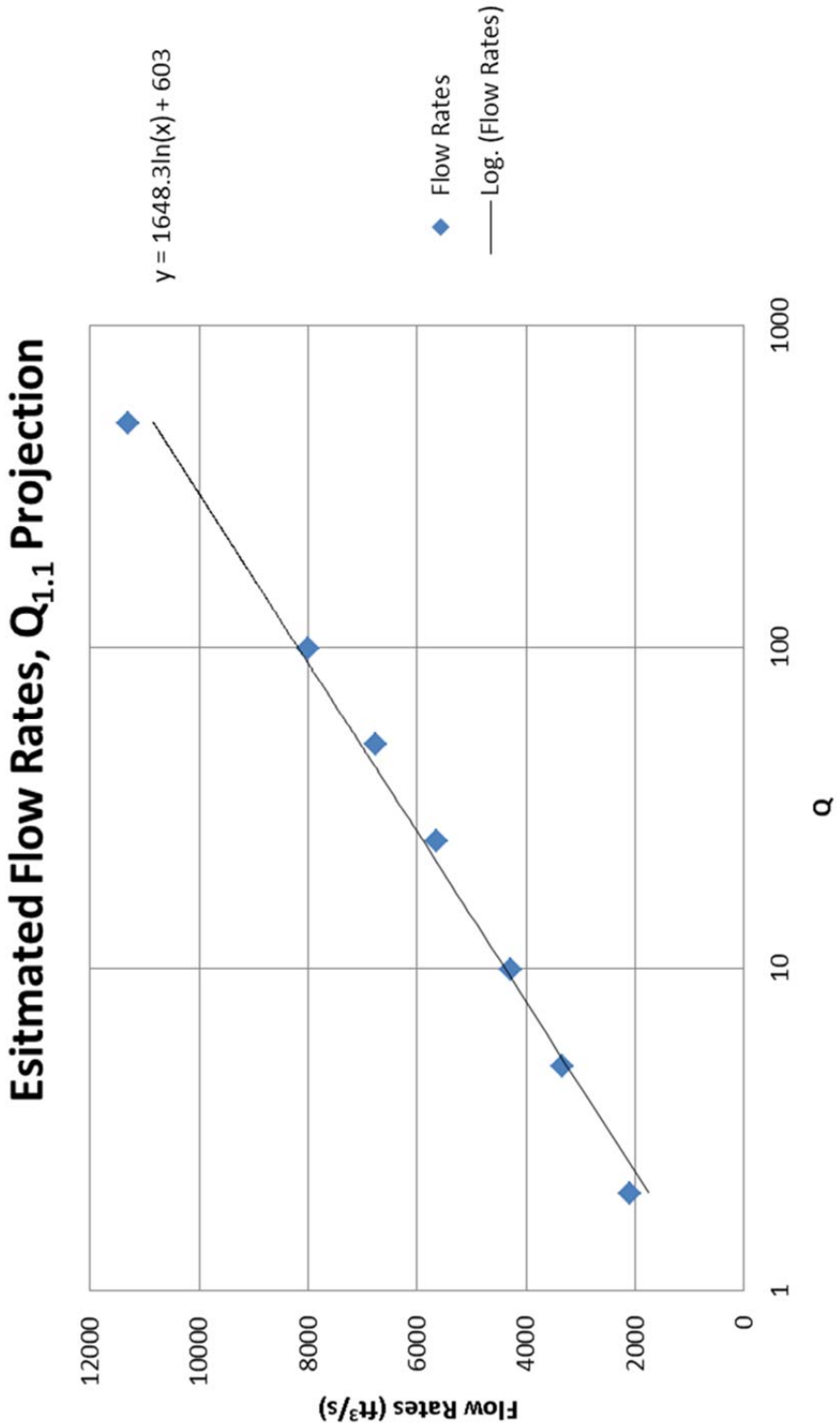

Figure 1: Estimated flow rates, $Q_{1.1}$ projection

Drainage Area	113	mi^2
$Q_{1.1}$	760	ft^3/s
Q_{10}	4,280	ft^3/s
Q_{25}	5,640	ft^3/s
Q_{50}	6,770	ft^3/s
Q_{100}	7,990	ft^3/s
Q_{500}	11,300	ft^3/s

Figure 2: Hydrology Summary

Reported by: Devan Eaton, P.E.
Date: September 30, 2015

HYDRAULIC REPORT

Hydraulic Modeling

The existing bridge and proposed structure were analyzed using HEC-RAS, version 4.1.0, the river analysis software developed by the U.S. Army Corps of Engineers. The HEC-RAS model for this project represents a 500 foot long segment of the Little Ossipee River including the bridge at the river crossing on State Route 5 (Sokokis Trail). Three cross-sections upstream of the bridge and four downstream make up the geometry of the stream.

The 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 = 0.036
    - Overbanks = 0.08
- Contraction/Expansion coefficients

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

- Ineffective flow areas

  The existing bridge approaches are fill slopes 20 plus feet in height, resulting in a constriction of the channel; therefore at high flows, there will be ineffective flow upstream and downstream of the existing bridge in the overbanks due to the available width of the flood plain and the magnitude of contraction and expansion. The ineffective flow areas were determined separately for the upstream and downstream cross-sections. Upstream of the existing structure, on both the north and south sides of the river, a combination of old stack stone walls and the topography of the surrounding wetland area creates banks that contain the river flow; these banks were defined as the ineffective flow boundaries. At the existing bridge location, on both the north and south sides of the river, the existing approach fill slopes create a contraction in the river. On the north side of the river, the approximate toe of slope was used as the ineffective flow boundary as the fill slope is steep and only approximately 40 feet long, so the water would not have much space to pool until it has overtopped the fill slope. On the south side of the river, the toes of slopes were used as the ineffective flow boundary as the fill slope is approximately 200 feet long, so water would pool at the fill slope during periods

of extremely high flow and slowly be funneled under the existing/proposed bridge. Downstream of the bridge, the toes of slopes in the area on the south side of the river between the fill slopes of the existing and the old alignments was used as the ineffective flow boundary.

- Subcritical flow regime
- Boundary condition
    - Normal depth: S = 0.0056 ft/ft

In order to check the validity of the model and boundary conditions, the estimated peak flows from the FEMA flood insurance study were used as there are documented estimated water elevations for the flow rates at the varying recurrence intervals. According to the FEMA estimations, which were calculated using the USDA NRCS WSP-2 computer program, the Q100 flows pass under the existing superstructure, and the Q500 flows hit the superstructure, but still pass under without overtopping the roadway. The HEC-RAS model, when using the FEMA flows, showed a similar interaction for the superstructure and the Q100 and Q500 flows, however the water elevations were approximately 2 feet lower than those estimated in the FEMA profile. This difference is likely due to differences in Manning's "n" values and normal depth stream slope estimations. The FEMA study defines ranges of Manning's "n" values for both the stream and overbanks for the entire length of the Little Ossipee River with undefined field determined values used for analysis. Since the field values were not explicitly known, the Manning's "n" used in the HEC-RAS model were based off of U.S.G.S examples of similar site conditions, with the chosen "n" value falling within the defined FEMA range. The stream slope for the HEC-RAS model only takes approximately 500 feet of the stream around the existing bridge into account to calculate stream slope, whereas the FEMA stream slopes likely take the entire river profile into account. With similar interactions between the headwater elevation and the superstructure occurring, it was determined that the model was appropriately representing the project site.

Once the model was calibrated, the previously defined peak flows were run through the existing and proposed structures for comparison (see Hydrology section for estimated peak flows). Since the existing tee-beam superstructure is relatively flat, the proposed vertical alignment raises the grade to provide a minimum 0.5% grade across the structure. The existing structure is comprised of three spans with two in-water piers. The proposed structure is comprised of two spans, requiring only one pier, which eliminates one in-water obstruction. The integral abutments will have 1.75:1 slopes in front of the breast wall, which will encroach on the river from both the Waterboro and Limerick sides. The combination of a proposed grade increase and only requiring one pier will result in additional hydraulic capacity under the

proposed structure; however this additional capacity is only slightly larger than existing due to the encroachment of the integral abutment slopes into the river.

The following is a list of assumptions made for the proposed structure:

- 168’-0” two span structure comprised of two 84'-0” spans
- A superstructure comprised of 40” steel girders, 1 ½” blocking, 8” concrete deck and a 3” bituminous wearing surface with a ¼” high performance waterproofing membrane.
- Two integral abutments supported on H-pile driven to ledge and one cast-in-place reinforced concrete wall pier with spread footings on bedrock.

Headwater elevations and stream velocities from peak flows for the existing and proposed structures are reported in the table below. The overall hydraulic opening has increased between the existing and proposed structures, mostly due to the removal of one pier and the raise in grade, however the headwater elevations and velocities only slightly vary as shown in the results, due to the presence of the integral abutment slopes which encroach on the river and slightly constrict the channel.

SUMMARY

		Existing Structure	Recommended Structure
		135' 3-Span Concrete Tee-beams[1]	168' 2-Span Steel girders[2]
Total Area of Waterway Opening	ft^2	1490	1642
Headwater elevation @ $Q_{1.1}$	ft	307.24	307.31
Headwater elevation @ Q_{10}	ft	310.75	310.62
Headwater elevation @ Q_{25}	ft	312.12	311.99
Headwater elevation @ Q_{50}	ft	313.10	312.97
Headwater elevation @ Q_{100}	ft	314.05	313.93
Headwater elevation @ Q_{500}	ft	316.02	315.89
Freeboard @ Q_{50}	ft	3.89	4.47
Freeboard @ Q_{100}	ft	2.94	3.51
Flood Of Record (March 1936) Elevation 314.34 ft			
Velocity @ $Q_{1.1}$	ft/s	7.15	7.00
Velocity @ Q_{10}	ft/s	10.13	11.08
Velocity @ Q_{25}	ft/s	9.84	10.88
Velocity @ Q_{50}	ft/s	9.93	11.02
Velocity @ Q_{100}	ft/s	10.13	11.23
Velocity @ Q_{500}	ft/s	11.19	12.41

1. Includes two 5'-0" wide mass concrete piers
2. Includes one 6'-0" wide pier with 10'-0" exposed footing

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

The Little Ossipee River is not defined as a major riverine as it is located within the Saco River drainage basin. The bridge has a history of ice jams according to local residents. These ice jams have not caused the river to overtop the roadway, but have reportedly resulted in increases in the headwater elevations to within two feet of the bottom of the superstructure. According to the BDG, 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; however, since there is a history of ice jams at the bridge location, more than two feet of minimum vertical clearance is preferred. 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 properties. When possible, it is preferred to have one foot of freeboard at Q100. The proposed structure meets the minimum requirements as defined in the BDG, with a low chord elevation of 317.44 ft.

Scour

Scour is present at the bridge site, as portions of the footing of the existing pier closest to the Waterboro abutment are exposed. This scour is considered minor as no undermining is present at any of the existing substructure locations. Though recent inspection reports show no increase to the scour on the existing structure, the proposed structure will also be susceptible to scour. Maintaining the existing bridge alignment with similar abutment locations set in close proximity to the natural channel will continue to result in a constriction on the stream due to the approach roadway embankments and long overbank areas, particularly on the southwest corner of the project. This contraction will continue to increase the susceptibility of the substructure units to local scour.

Future scour susceptibility and magnitude for the proposed bridge 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 is the depositing of streambed material at the bridge due to erosion of the streambed upstream of the site. Degradation is the removal/scouring of the streambed at the bridge due to erosion and an imbalance of material carried away from the site and the material carried in. The streambed at this bridge location has a relatively flat slope, and has no man-made obstructions such as dams, and as a result the aggradation and degradation for this site is assumed to be zero.

As previously mentioned, this site will experience contraction scour, which is the most common type of general scour and the only type assumed to occur at this bridge. Contraction

scour for this site was evaluated using equations from HEC -18, specifically those for cohesionless streambed material. A grain-size distribution was provided by the geotechnical team for the soils from a boring taken in the river channel at the project site. The soil in the channel is a gravel-sand mixture with a D_{50} of 7.3mm. According to HEC-18, for a D_{50} greater than 0.2mm, live-bed and clear-water scour equations are the most practical and result in the most accurate scour estimates. The live-bed and clear-water scour equations are based on the ratio of the flow data located just upstream of the bridge and at the bridge constriction. The analysis performed for this project site uses the flow data from the estimated 100 year flood event, as defined above. Using the proposed conditions with this flow data, the contraction scour was calculated to be 5.76 feet, (see Appendix F for more information).

Local scour is the result of material being removed around piers and abutments due to the acceleration of flow caused by obstructions to the flow. Local scour is largely affected by variances in structure geometry and shape as well as the flow angle of attack, velocity and depth of the flow. The methods for determining the local scour vary for abutments and piers, and are calculated separately as a result. Similar to the procedure used for contraction scour, the flow data for the estimated 100 year flood event was used in all local scour computations. The projected local scour depths vary across the width of the river channel, ranging from calculated depths of 7.76 feet at abutment 1 (Waterboro), 22.9 feet at the proposed pier, and 7.76 feet at abutment 2 (Limerick). Three borings taken near the proposed substructure locations confirm that ledge does vary across the river channel as previously mentioned in the available soils information section above (see bridge recommendation form), however not as drastically as the predicted local scour depths. The predicted abutment scour depths were calculated using the NCHRP methods from HEC-18, which produces a total scour depth (contraction and local scour) at the abutment locations. These calculated total scour depths correlate relatively well with the actual measured depths to bedrock from the borings as the measured bedrock depths are within 4-5 feet of the scour depths, which are expected to be conservative as stated in HEC-18. The predicted value at the pier does not correlate well with the proposed pier boring as the predicted local scour depth is over five times the measured depth to bedrock. This value is extremely conservative on its own and does not take any debris collection into account, which has been an issue at the bridge in the past and would increase the calculated scour depth if added. This large difference will not be an issue for this site as the pier will be founded on bedrock and will not be subject to local scour. For this reason, only the abutment local scour numbers will serve as a guide during the final design process for the substructures.

The existing structure has been in service for over eighty years with little to no measureable scour damage. The proposed structure eliminates a pier, has rip-rap slopes providing protection for the integral abutments, and has increased vertical clearances over all flood water levels, which should improve the hydraulic performance.

Reported by: Devan Eaton, P.E.
Date: September 30, 2015

APPENDIX F

Hydrology & Hydraulic Documentation

Summary from a conversation with Charlie Hebson on 7/6/15:

In relation to the flows to be used for the hydraulic modeling of the Little Ossipee River at the location of the Stimson Bridge; use the recurrence intervals of Estimated Peak Flows from Table 2 from the USGS study on the Flood of April 2007. The flows are as follows:

Table 2. Estimated peak flows for selected recurrence intervals, Little Ossipee River near South Limington (USGS station number 01066500).

[USGS, U.S. Geological Survey; ft^3/s, cubic feet per second; confidence intervals are 95-percent confidence intervals]

Recurrence interval (years)	Estimated peak flow (ft^3/s)	Lower confidence interval (ft^3/s)	Upper confidence interval (ft^3/s)
2	2,090	1,760	2,480
5	3,330	2,770	4,080
10	4,280	3,490	5,450
25	5,640	4,430	7,700
50	6,770	5,160	9,850
100	7,990	5,910	12,500
500	11,300	7,690	21,400

As noted by: Devan Eaton, PE
Date: 7/6/15

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Prepared in cooperation with the Federal Emergency Management Agency

Estimated Magnitudes and Recurrence Intervals of Peak Flows on the Mousam and Little Ossipee Rivers for the Flood of April 2007 in Southern Maine

Open-File Report 2007–1146

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U.S. Department of the Interior
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Estimated Magnitudes and Recurrence Intervals of Peak Flows on the Mousam and Little Ossipee Rivers for the Flood of April 2007 in Southern Maine

By Glenn A. Hodgkins, Gregory J. Stewart, Timothy A. Cohn, and Robert W. Dudley

Abstract

Large amounts of rain fell on southern Maine from the afternoon of April 15, 2007, to the afternoon of April 16, 2007, causing substantial damage to houses, roads, and culverts. This report provides an estimate of the peak flows on two rivers in southern Maine—the Mousam River and the Little Ossipee River—because of their severe flooding. The April 2007 estimated peak flow of 9,230 ft^3/s at the Mousam River near West Kennebunk had a recurrence interval between 100 and 500 years; 95-percent confidence limits for this flow ranged from 25 years to greater than 500 years. The April 2007 estimated peak flow of 8,220 ft^3/s at the Little Ossipee River near South Limington had a recurrence interval between 100 and 500 years; 95-percent confidence limits for this flow ranged from 50 years to greater than 500 years.

Introduction

Flood-related data is useful for many purposes. For example, the Federal Emergency Management Agency (FEMA) and Maine Emergency Management Agency (MEMA) need timely information on the magnitudes and recurrence intervals of floods to help respond to flood damage. Peak-flow magnitudes are used to help delineate floodplain boundaries for land-use planning, and to design bridges, culverts, and other structures in the floodplain. Flood data are also used for scientific purposes, including the study of the effects of climate changes and land-use changes on hydrologic regimes, which in turn affect ecosystems.

On April 9, 2007, a thin, wet snowpack covered the ground in the Mousam and Little Ossipee River Basins. Snowpack was measured at four locations in or near the basins on April 9 (West Kennebunk, Newfield, Hollis, Cornish, fig. 1) as part of the Maine Cooperative Snow Survey program. Snowpack depths ranged from 2.4 to 5.3 in. and had an equivalent water content of 0.8 to 1.5 in. There was no snowpack at the sites in West Kennebunk, Newfield, and Hollis on April 19 (no data available for Cornish) (Maine Cooperative Snow Survey, 2007). A storm on April 12 and 13 may have changed the equivalent water content in the snowpack prior to the April 2007 flood, either by adding to the snowpack or by melting and then replacing the snowpack (Thomas Hawley, National Weather Service, written commun., 2007).

Most of the rainfall that caused the April 2007 flood occurred from the afternoon of April 15, 2007, to the afternoon of April 16, 2007. Maximum 24-hour total precipitation at continuous-record precipitation gages in Sanford, Hollis, and Cornish (fig. 1) were 7.58 in., 4.97 in., (Thomas Hawley, National Weather Service, Gray, Maine) and 4.60 in., respectively (Cornish data from USGS continuous-rainfall gage).

This report presents results of a preliminary study conducted by the U.S. Geological Survey (USGS) to collect, compute, and compile flood-related data for the Mousam and Little Ossipee Rivers, which were substantially impacted by the storm. Estimates are presented of the magnitudes and associated recurrence intervals of peak flows for the Mousam River and the Little Ossipee River, for the flood of April 2007. Although the Mousam River Basin has reservoirs with sufficient storage capacity to potentially affect the magnitude of peak flows, the actual effect of reservoirs on April 2007 peak flows was not analyzed in this report.

Methods of Estimating the Magnitudes and Recurrence Intervals of Peak Flows

Estimation of Magnitudes

The 2007 peak flow at the Mousam River near West Kennebunk was estimated from surveyed April 2007 high watermarks by extension of the historical rating curve (the relation between river height and flow). This is believed to be a reasonable approach based on the stability and type of the river height/flow control for high flows at this location.

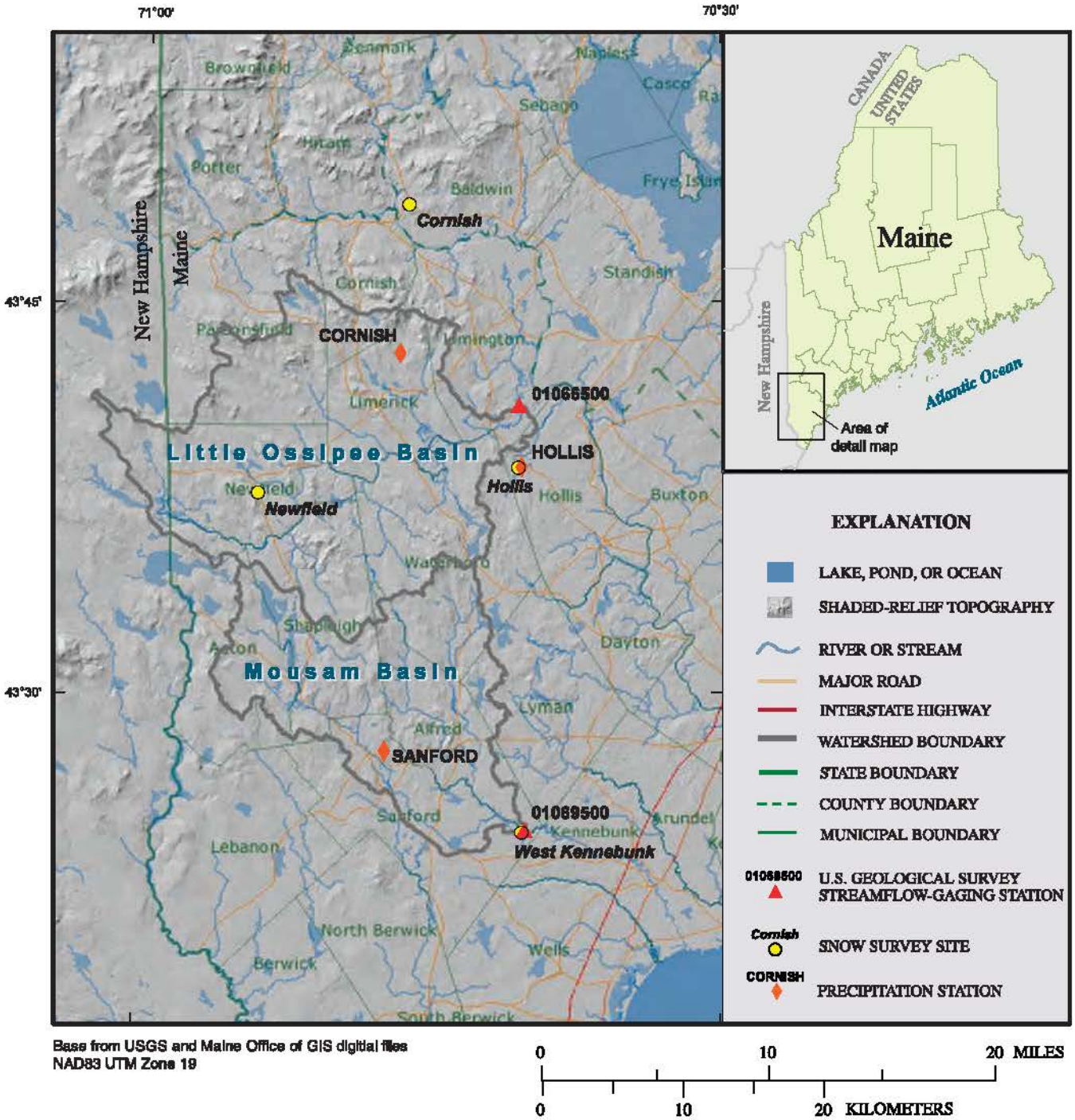

Figure 1. Mousam River and Little Ossipee River Basins in southern Maine and data-collection locations.

The 2007 peak flow at the Little Ossipee River near South Limington was estimated using surveyed April 2007 high watermarks and hydraulic principles for contracted openings (Matthai, 1967) at the Sand Pond Road bridge. The geometry of the channel and bridge was defined by field surveys. The geometric data were entered in the U.S. Army Corps of Engineers (USACE) step-backwater computer program HEC-RAS (U.S. Army Corps of Engineers, 2004). The one-dimensional steady-flow water-surface profile computation component of HEC-RAS was used in the analysis. Boundary conditions and energy-loss parameters, including channel roughness and contraction and expansion coefficients, were measured and(or) estimated in the field and further adjusted during calibration of the model. The model was fully calibrated to match 2007 flood high-watermark elevations at the upstream approach and downstream exit sections of the bridge to within ±0.01 ft of the highest-quality high-watermark elevations. In addition to the calibrated 2007 flood flow, a range of flows (with recurrence intervals from 2 years to 500 years) were modeled as part of the calibration to test that model functionality was well conditioned.

Estimation of Recurrence Intervals

The recurrence interval is the statistically computed average period of time between peak flows that are greater than, or equal to, a specified magnitude. For example, the 50-year peak flow is the flow that would be exceeded or equaled, on long-term average, once in 50 years. This does not imply that flooding will happen at regular intervals. Two 50-year peak flows could be experienced in 2 consecutive years. Conversely, a 50-year peak flow might not be experienced for 100 years. The reciprocal of the recurrence interval is the annual exceedance probability, which is the probability that a given peak flow will be exceeded or equaled in any given year. For example, the annual exceedance probability of the 50-year peak flow is 0.02. In other words, there is a 2-percent chance that the 50-year peak flow will be exceeded or equaled in any given year.

The U.S. Geological Survey (USGS) collected continuous streamflow data on the Mousam River near West Kennebunk (USGS station number 01069500) from October 1939 to September 1984 and on the Little Ossipee River near South Limington (station number 01066500) from August 1940 to October 1982 (fig. 1). Annual peak-flow data for these stations were obtained from the USGS National Water Information System (U.S. Geological Survey, 2007).

For the Mousam River, the largest peak flow recorded from 1939 to 1984 was 4,020 ft^3/s on March 20, 1983. Known peak flows occurred outside of the period of continuous record; on October 22, 1996, the peak flow was 3,600 ft^3/s and on May 15, 2006, the peak flow was 6,100 ft^3/s. For the Little Ossipee River, the largest peak flow recorded from 1940 to 1982 was 5,760 ft^3/s on March 15, 1977; on March 19, 1936, the peak flow was 8,530 ft^3/s; and on October 22, 1996, the peak flow was 5,800 ft^3/s.

Peak flows for selected recurrence intervals were estimated for this report by use of the Expected Moments Algorithm (EMA) (Cohn and others, 1997; Cohn and others, 2001; Griffis and others, 2004). EMA is a generalization of the procedures in Bulletin 17B (Interagency Advisory Committee on Water Data, 1982) and was designed to better accommodate historical peak-flow data—known peak flows outside the period of continuous streamflow data collection. Although the procedures in Bulletin 17B can employ historical data, EMA makes more efficient use of historical data and the EMA confidence intervals are more accurate than the ones given in Bulletin 17B. Peak flows from the April 2007 flood were used in the calculations.

Regional information was not used to estimate the peak flows for the Mousam River because of its substantial historical flow regulation. Reservoirs (Square Pond, Mousam Lake, Estes Lake) above the USGS streamflow-gaging station near West Kennebunk were believed in 1984 to have a combined capacity of about 700,000,000 ft^3 (Haskell and others, 1985). Flow-storage capacity above the station could affect recorded peak flows by more than 10 percent (Hodgkins, 1999; Benson, 1962).

Regional information was used to compute a weighted skew for the Little Ossipee River following the procedures in Griffis and others (2004). The generalized skew used at this site was 0.029 with a standard error of 0.297 (Hodgkins, 1999). Regional regression equations were not used to weight the at-site data; Hodgkins (1999) found that the weighted estimates for the 2-year to 500-year peak flows differed by less than 3 percent from station estimates for the Little Ossipee River.

Estimated Magnitudes and Recurrence Intervals of Peak Flows for the Flood of April 2007

The estimated April 2007 peak flow for the Mousam River near West Kennebunk was 9,230 ft^3/s. The largest previously known peak flow from 1939 to 1984, 1996, and 2006 was 6,100 ft^3/s on May 15, 2006. The computed April 2007 peak flow for the Little Ossipee River near South Limington was 8,220 ft^3/s. The largest known peak flow from 1936, 1940 to 1982, and 1996 was 8,530 ft^3/s on March 19, 1936.

Estimated peak flows for the Mousam River and Little Ossipee River and their associated recurrence intervals are shown in tables 1 and 2, along with 95-percent confidence intervals. The April 2007 flood at the Mousam River near West Kennebunk had a recurrence interval between 100 and 500 years; 95-percent confidence limits ranged from 25 years to greater than 500 years. The April 2007 flood at the Little Ossipee River near South Limington had a recurrence interval between 100 and 500 years; 95-percent confidence limits ranged from 50 years to greater than 500 years (table 3).

Table 1. Estimated peak flows for selected recurrence intervals, Mousam River near West Kennebunk (USGS station number 01069500).

[USGS, U.S. Geological Survey; ft³/s, cubic feet per second; confidence intervals are 95-percent confidence intervals]

Recurrence interval (years)	Estimated peak flow (ft³/s)	Lower confidence interval (ft³/s)	Upper confidence interval (ft³/s)
2	1,390	1,120	1,650
5	2,230	1,750	2,890
10	3,010	2,370	4,400
25	4,340	3,210	9,830
50	5,630	3,930	18,300
100	7,240	4,730	32,500
500	12,700	6,990	126,000

Table 2. Estimated peak flows for selected recurrence intervals, Little Ossipee River near South Limington (USGS station number 01066500).

[USGS, U.S. Geological Survey; ft³/s, cubic feet per second; confidence intervals are 95-percent confidence intervals]

Recurrence interval (years)	Estimated peak flow (ft³/s)	Lower confidence interval (ft³/s)	Upper confidence interval (ft³/s)
2	2,090	1,760	2,480
5	3,330	2,770	4,080
10	4,280	3,490	5,450
25	5,640	4,430	7,700
50	6,770	5,160	9,850
100	7,990	5,910	12,500
500	11,300	7,690	21,400

Table 3. Estimated peak flows and recurrence intervals for flood of April 2007.

[USGS, U.S. Geological Survey; mi², square miles; ft³/s, cubic feet per second; confidence intervals are 95-percent confidence intervals]

River name	USGS station number	Drainage area (mi²)	Estimated peak flow (ft³/s)	Recurrence interval (years)	Confidence intervals (years)
Mousam River near West Kennebunk, Maine	01069500	99.0	9,230	100 to 500	25 to greater than 500
Little Ossipee River near South Limington, Maine	01066500	168	8,220	100 to 500	50 to greater than 500

Summary

The Federal Emergency Management Agency (FEMA) and Maine Emergency Management Agency (MEMA) need timely information on the magnitude and recurrence intervals of floods to help respond to flood damage. Large amounts of rain fell on southern Maine in April 2007, causing substantial damage to houses, roads, and culverts. Most of the rainfall occurred from the afternoon of April 15, 2007, to the afternoon of April 16, 2007; maximum 24-hour total precipitation at continuous-record precipitation gages in Sanford, Hollis, and Cornish was 7.58 in., 4.97 in., and 4.60 in., respectively.

The resulting April 2007 peak flow at the Mousam River near West Kennebunk was estimated from surveyed flood marks by extension of the historical rating curve (the relation between river height and flow). The peak flow at the Little Ossipee River near South Limington was estimated using surveyed flood marks and hydraulic principles for contracted openings.

The estimated April 2007 peak flow for the Mousam River near West Kennebunk was 9,230 ft^3/s. The largest known peak flow from 1939 to 1984, 1996, and 2006 was 6,100 ft^3/s on May 15, 2006. The estimated April 2007 peak flow for the Little Ossipee River near South Limington was 8,220 ft^3/s. The largest known peak flow from 1936, 1940 to 1982, and 1996 was 8,530 ft^3/s on March 19, 1936.

Peak flows for selected recurrence intervals were estimated for this report by use of the Expected Moments Algorithm (EMA). Recurrence intervals are the statistically computed long-term-average period of time between peak flows that are greater than, or equal to, a specified magnitude.

The April 2007 estimated peak flow at the Mousam River near West Kennebunk had a recurrence interval between 100 and 500 years; 95-percent confidence limits ranged from 25 years to greater than 500 years. The April 2007 flood at the Little Ossipee River near South Limington had a recurrence interval between 100 and 500 years; 95-percent confidence limits ranged from 50 years to greater than 500 years.

Acknowledgments

Thomas Hawley of the National Weather Service in Gray, Maine, provided most of the meteorological data analyzed in this report. Joshua Kempf and Nicholas Stasulis of the USGS Maine Water Science Center collected a substantial amount of field data that were required for this report.

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USGS

Hodgkins, G.A. and others—Estimated Magnitudes and Recurrence Intervals of Peak Flows on the Mousam and Little Ossipee Rivers for the Flood of April 2007 in Southern Maine—Open-File Report 2007–1146

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HEC-RAS Plan

MAG. 19XX
N
POE = STA. 105+00.00
105+00
104+00
CURVE DATA #2
PI = 152+50.77
D = 5°40'00.0"
Δ = 35°24'00.0" Lt.
R = 1011.10'
L = 624.71'
T = 322.68'
E = 50.24'
PC = STA. 149+28.09
STA. 102+50.00
103+00
147+00
N8°33'50.97"E
148+00
149+00
150+00
151+00
152+00
153+00
154+00
102+00
101+00
Flow
100+00
POB = STA. 100+00.00
10+00
PC = STA. 10+15.00
PT = STA. 10+62.87
11+00
11+50
POE = STA. 11+50.00

Project Name:	Stinson Br	**PIN:**	20476
Stream Name:	Little Ossipee River	**Town:**	Waterboro-Limerick
Bridge Name:	Stinson	**Bridge No.**	2807
Route No.	ME 5	**USGS Quad:**	
Analysis by:	CSH	**Date:**	3/10/2014

Peak Flow Calculations by USGS Regression Equations (Hodgkins, 1999)

With Gage Adjustment

Enter data in blue cells only!

	km^2	mi^2	ac	*Enter data in [mi^2]*
A	292.67	113.00	72320.0	Watershed Area
W	41.90	16.18	10353.7	Wetlands area (by NWI)
P_c	348950.6	4833564		watershed centroid (E, N; UTM 19N; meters)
County	York			*choose county from drop-down menu*
pptA	46.7			mean annual precipitation (inches; by look-up)
SG	0.00			sand & gravel aquifer as decimal fraction of watershed A

A (km^2)	292.67	**Conf Lvl**	0.67
W (%)	14.32		

Worksheet prepared by:
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Maine Dept. Transportation
Augusta, ME 04333-0016
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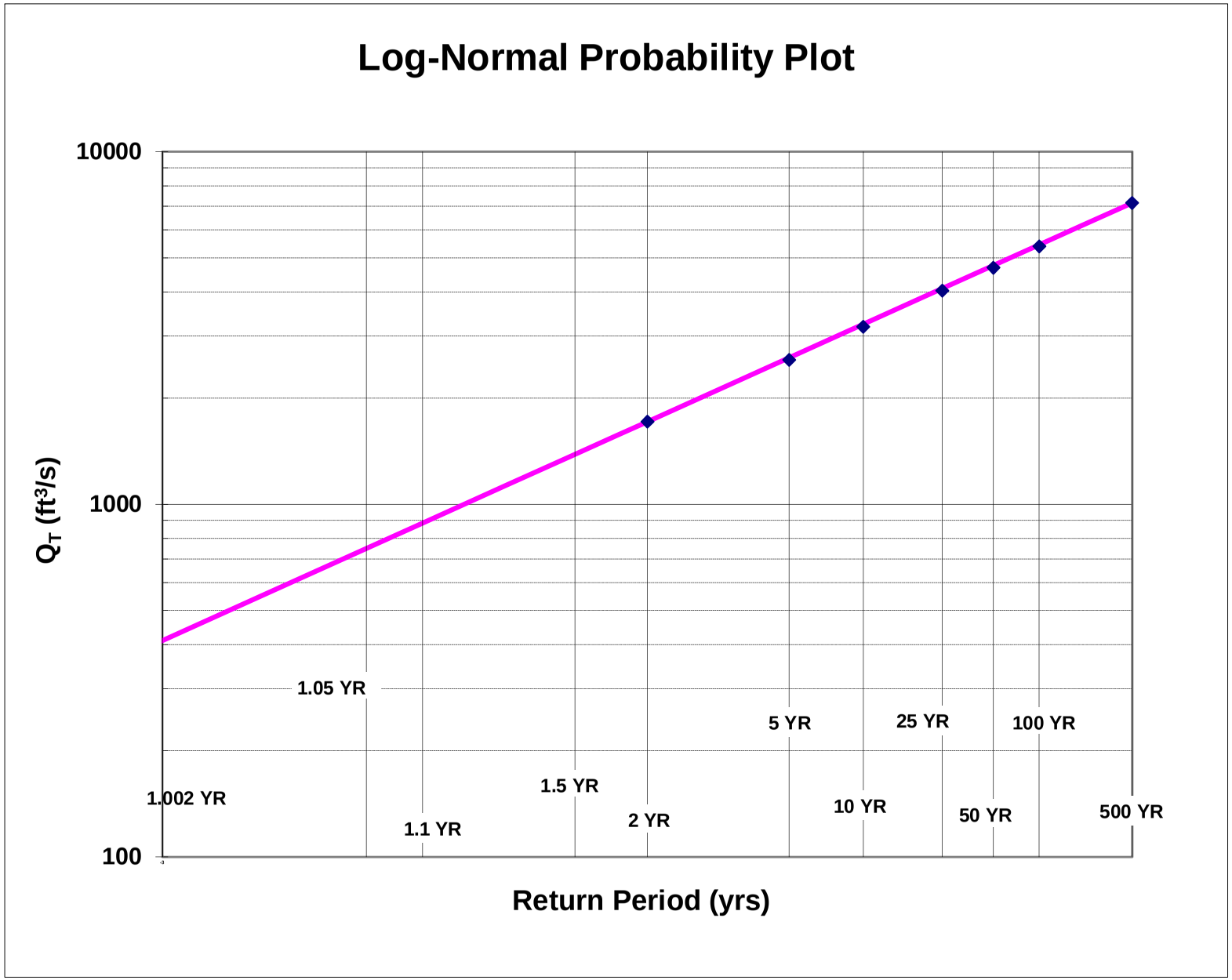
Ret Pd T (yr)	Peak Flow Estimate Lower	Q_T (m^3/s)	Upper	Q_T (ft^3/s)	*Use Gage Adjusted*
1.1		25.06		884.8	***779***
2	39.70	55.22	76.79	1949.7	***1716***
5	57.07	79.54	110.86	2808.6	***2569***
10	68.72	96.73	136.16	3415.6	***3189***
25	83.28	118.75	169.33	4193.1	***4032***
50	93.99	135.48	195.30	4783.9	***4690***
100	105.10	153.21	223.35	5409.9	***5393***
500	130.09	195.47	293.70	6902.0	***7158***

Reference:

Hodgkins, G., 1999.
Estimating the magnitude of peak flows for streams in Maine for selected recurrence intervals
Water-Resources Investigations Report 99-4008
US Geological Survey, Augusta, Maine

$Q_T = b \times A^a \times 10^{-WW}$

USGS Gage 01066500
Little Ossipee River near South Limington, ME


Log-Normal Probability Plot
10000
1000
100
Q_T (ft³/s)
1.002 YR
1.05 YR
1.1 YR
1.5 YR
2 YR
5 YR
10 YR
25 YR
50 YR
100 YR
500 YR
Return Period (yrs)

Project Name:	Stinson Br	**PIN:**	20476
Stream Name:	Little Ossipee River	**Town:**	Waterboro-Limerick
Bridge Name:	Stinson	**Bridge No.**	3987
Route No.	ME 5	**USGS Quad:**	
Analysis by:	CSH	**Date:**	12/1/2008

DO NOT ENTER ANY DATA ON THIS PAGE; EVERYTHING IS CALCULATED

MAINE MONTHLY MEDIAN FLOWS BY USGS REGRESSION EQUATIONS (2004)

Worksheet prepared by:
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Augusta, ME 04333-0016
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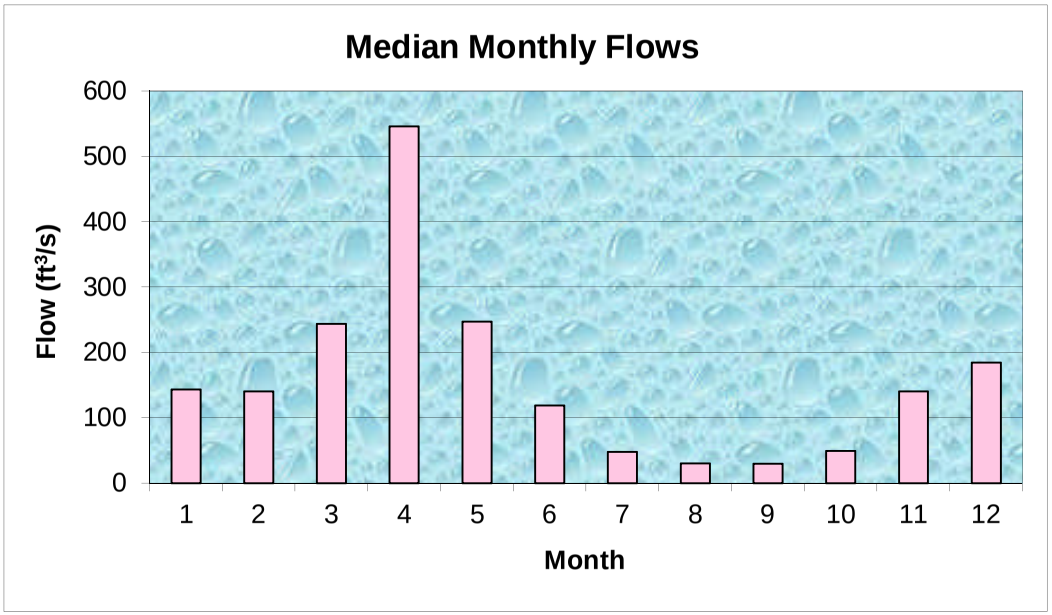
	Value	Variable	Explanation
	113.000	*A*	Area (mi^2)
348950.6	4833564	P_c	Watershed centroid (E,N; UTM; Zone 19; meters)
	48.79	*DIST*	Distance from Coastal reference line (mi)
	46.7	*pptA*	Mean Annual Precipitation (inches)
	0.00	*SG*	Sand & Gravel Aquifer (decimal fraction of watershed area)

Month	Q_{median} (ft3/s)	(m^3/s)
Jan	**143.44**	4.0650
Feb	**140.66**	3.9860
Mar	**243.83**	6.9099
Apr	**546.05**	15.4744
May	**247.40**	7.0110
Jun	**118.80**	3.3666
Jul	**48.00**	1.3601
Aug	**30.29**	0.8584
Sep	**29.64**	0.8400
Oct	**49.22**	1.3948
Nov	**140.72**	3.9879
Dec	**184.57**	5.2305

Q_{bf}	**742.8**
ann avg	**231.3**
ann med	**128.6**
$Q_{1.002}$	**411.0**
$Q_{1.01}$	539.7
$Q_{1.05}$	749.5

W_{bf}	89.0	
d_{bf}	6.9	
Q_{bf}	2471.7	assume v = 4ft/s



Lebanon, Town of - continued:	The hydrologic and hydraulic analyses for the July 3, 2002 study for Salmon Falls River were taken from the work performed for the City of Rochester and the Town of Milton as described above. The hydrologic analyses for Little River, Bog Brook, and Great Brook were prepared by USGS for FEMA under Inter-Agency Agreement No. EMW-98-IA-0175, Project Order No. 1. This work was completed in February 2000.
Limerick, Town of:	The hydrologic and hydraulic analyses for the August 1, 1984 study for the Little Ossipee River were performed by the USDA NRCS during the course of the Flood Hazard Analyses for the Little Ossipee River in the towns of Limerick and Waterboro. This work was completed in September 1977.
Limington, Town of:	The hydrologic and hydraulic analyses for the October 1, 1981 study were prepared by the Water Resources Division of USGS for FEMA under Inter-Agency Agreement No. IAA-H-14-78, Project Order No. 10. This work was completed in December 1979.
Lyman, Town of:	The hydrologic and hydraulic analyses for the May 15, 1991 study were prepared by USGS for FEMA under Inter-Agency Agreement No. EMW-85-E-1823, Project Order No. 20. This work was completed in March 1989.
Newfield, Town of:	The hydrologic and hydraulic analyses for the December 5, 1984 study were performed by the USDA NRCS during the course of the Flood Hazard Analyses for the Little Ossipee River and Balch Pond in the towns of Newfield, Acton, and Shapleigh. This work was completed in September 1977.
North Berwick, Town of:	The hydrologic and hydraulic analyses for the August 1, 1984 study were performed by USDA NRCS during the course of the Flood Hazard Analyses for the Great Works River in the towns of North Berwick and Sanford. This work was completed in September 1977.

Waterboro, Town of:	The hydrologic and hydraulic analyses for the August 1, 1984 study for the Little Ossipee River from the downstream corporate limits to the Waterboro-Limington-Limerick town boundary were performed by USGS during the course of the FIS for the Town of Limington. This work was completed in December 1979. The hydrologic and hydraulic analyses for the Little Ossipee River from the Waterboro-Limington-Limerick town boundary to the upstream corporate limits and for Little Ossipee Lake were performed by the USDA NRCS during the course of the Flood Hazard Analyses for the Little Ossipee River in Waterboro and Limerick. This work was completed in September 1977.
Wells, Town of:	The hydrologic and hydraulic analyses for the January 5, 1983 FIS report and July 5, 1983 FIRM (herein after referred to as the 1983 FIS) were prepared by Stone & Webster Engineering Corporation for FEMA under Contract No. H-4092. This work was completed in September 1978. The wave height and runup analysis for that study was completed in February 1982. For the January 16, 2003 revision, the hydrologic and hydraulic analyses for the Atlantic Ocean were prepared by ENSR for FEMA under Contract No. EMW-95-C-4783. This work was completed in September 1998.
York, Town of:	The hydrologic and hydraulic analyses for the September 15, 1983 FIS and December 15, 1983 FIRM (herein after referred to as the 1983 FIS) were prepared by Stone & Webster Engineering Corporation for FEMA under Contract No. H-4092. The stillwater flooding portion for that study was completed in October 1978. The wave runup and wave height analyses were completed in May 1982. The hydrologic and hydraulic analyses for the June 17, 2002 revision for the Atlantic Ocean were prepared by ENSR for FEMA under Contract No. EMW-95-C-4783. This work was completed September 2, 1998.

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – continued

Flooding Source Name	Description of Study Reaches
Kennebunk River	From approximately 500 feet upstream of its confluence with the Atlantic Ocean to approximately 2,750 feet upstream of U.S. Route 1 in Arundel
Little Ossipee Lake	For the entire shoreline within the Town of Waterboro, including the outlet stream to its confluence with the Little Ossipee River
Little Ossipee River	From its confluence with Saco River to the confluence with Balch Pond Dam
Little River (Town of Berwick)*	From its confluence with Salmon Falls River to the upstream corporate limit (approximately 18,300 feet upstream of Diamond Hill Road)
Little River (Town of Cornish)	From its confluence with the Ossipee River to State Routes 5 and 117
Little River (Town of Kennebunk)	From approximately 1,000 feet upstream of its confluence with the Atlantic Ocean to its confluence with Branch Brook
Little River	From approximately 650 feet upstream of its confluence with the Atlantic Ocean to approximately 2,380 feet upstream of Oak Ridge Road in the City of Biddeford
Littlefield River	From Estes Lake Dam to approximately 1.8 miles upstream of U.S. Route 202 and State Route 4 in the Town of Alfred
Merriland River (Lower Reach)	From its confluence with the Little River to approximately 2,900 feet upstream of U.S. Route 1
Merriland River (Upper Reach)	From the dam just downstream of Hobbs Farm Road to a point approximately 2.1 miles upstream
Middle Branch Mousam River	From approximately 3,000 feet upstream of confluence with Estes Lake to State Route 4/State Route 202
Mill Brook	For the entire length within the community of Old Orchard Beach from Ross Road to approximately 1.2 miles upstream
Moors Brook	From Meetinghouse Road in Biddeford to a point approximately 1.55 miles upstream

*Studied by USGS-ME

The towns of Acton, Eliot, Newfield, South Berwick, and Shapleigh are experiencing an increase in urbanization, and developmental pressures are expected to continue increasing in these towns. Urbanization within the watershed increases the flood hazard by increasing the rate and amount of runoff. Encroachment into the floodplain by land filling and other developments constricts the flow and reduces the storage of floodwaters. This in turn increases flood depths and the area flooded upstream.

Flooding occurs almost annually along the Great Works River, affecting the towns of North Berwick, South Berwick, and Sanford. Most of the floods are caused by rapid thawing of snow and ice in late winter and early spring, and the flooding is often accelerated by rainfall and ice jams. Less often, flooding occurs later in the year as a result of hurricanes. Minor flooding occurs almost annually. The flood of March 1936 is generally considered to be the largest in recent years in the Town of South Berwick. This flood also involved the Salmon Falls River. It was generated from approximately 6 inches of rain over a 10-day period and further complicated by high antecedent moisture conditions, snow cover, and ice laden streams. The recurrence interval of this flood was estimated at approximately 1-percent-annual-chance. There are no available records of any significant flood damage in the study area (Reference 16). Notable floods also occurred in this region in 2006 and 2007.

In the Town of Sanford, the flood of March 1983 produced maximum flows of 4,020 cubic feet per second (cfs) at the USGS gage on the Mousam River near West Kennebunk and had a recurrence interval of approximately 85 years (Reference 17). No records of flood damages to the town are available.

Minor flooding occurs almost annually in the towns of Acton, Limerick, Limington, Newfield, Shapleigh, and Waterboro due to snowmelt and ice jams on the Little Ossipee River. Areas flooded include wooded and open lowlands, roads, and bridges. The flood of record occurred in March 1936 and had a peak discharge of 8,530 cfs at the Little Ossipee Flowage Dam in Limerick and Waterboro. This flood was generated from approximately 4 inches of rain over a 2- day period and further complicated by high antecedent moisture conditions, snow cover, and ice laden streams. The recurrence interval of the flood was estimated to be slightly in excess of 1-percent-annual-chance. There are no available records of any flood damage in the study area (Reference 16).

A USGS gaging station is located on the Little Ossipee River near South Limington and had a period of record of 42 years. The highest recorded discharge (5,760 cfs) at the gage occurred in March 1977. The USGS gaging station was discontinued in September 1982 (Reference 17).

Numerous low-lying areas along the Little Ossipee River are subject to frequent flooding. Flooding of these areas can cause damage to dwellings, existing woodlands, roads, and bridges. A 1-percent-annual-chance flood in Waterboro would inundate approximately 1,300 acres; the 0.2-percent-annual-chance flood would inundate approximately 1,500 acres In Limerick, a 1-percent-annual-chance flood would inundate approximately 900 acres; the 0.2-percent-annual-chance flood would inundate approximately 1,100 acres (Reference 18).

In the towns of Waterboro and Limerick, an estimated 60 building lots are located within the 1-percent-annual-chance floodplain adjacent to the Little Ossipee Flowage.

Approximately 15 of these lots have been recently developed with either seasonal or year-round homes. Due to this development pressure on floodplain lands, significant flood damage potential exists if development is allowed to continue uncontrolled (Reference 18).

Small areas of localized flooding occur along the southern portion of the Town of Limington. The Little Ossipee River flows approximately 10 miles through a relatively uninhabited section of the town. The river drops a total of 58 feet in Limington, at a fairly constant rate, except for two sets of falls. There has been very little development in Limington on the floodplains of the river.

Low-lying areas of Cornish and Parsonsfield are subject to periodic flooding caused by the overflow of the Ossipee River. Cornish also is subject to periodic flooding from the Little River. The most severe flooding occurs in the early spring as a result of snowmelt and heavy rain in conjunction with ice jams. Additional floods, generally lower in magnitude, also occur in late summer as a result of hurricanes and tropical storms. The two major floods occurring in Cornish and Parsonsfield from the Ossipee River were in 1936 and 1953, and the frequencies and peak discharges of these floods were recorded at the two USGS gages at Cornish. The peak discharge of the 1936 flood was 17,200 cfs, and its recurrence interval of approximately 0.3-percent-annual-chance. The 1953 flood had a peak discharge of 13,800 cfs, and a recurrence interval of 1-percent-annual-chance (Reference 16). In the Town of Parsonsfield, the 1936 flood caused flooding of the woolen mill located in Kezar Falls Village and the area near South River. Damages were slight due to the relatively steep banks and sparse development in the floodplain.

Major floods on the Saco River have occurred in the spring, and are usually the result of heavy rainfall combined with snowmelt. Although there has been flooding during other months, 10 of the 14 greatest floods have occurred during March, April, or May. Heavy rainfall associated with hurricanes moving up the coast of Maine has caused flooding in the fall. This flooding occurs on the riverine and estuarine reaches of the Saco River and affects the towns of Buxton, Dayton, Hollis, Limington, Cornish, and the cities of Biddeford and Saco.

The greatest recorded flood in the lower Saco River basin occurred at West Buxton in March 1936, with a peak flow of 58,200 cfs (Reference 16). In the vicinity of the cities of Biddeford and Saco, damage from the 1936 flood was centered in the more intensely developed residential, commercial, and industrial areas (Reference 19). The March 1953 flood, the most severe recorded for the New Hampshire portion of the Saco River, caused a peak flow of 50,000 cfs at West Buxton. The 1953 flood caused about $1.8 million in flood damage (Reference 20). The March 1936 and March 1953 floods have recurrence intervals of 0.25- and 0.625-percent-annual-chance, respectively. Small areas of localized flooding have been noted along the small brooks studied by approximate methods. Road overflow is not uncommon on these brooks, especially when the culverts at road crossings are clogged with ice.

In the Town of Cornish, the peak discharge of the March 1936 flood was 45,000 cfs, and its recurrence interval is 0.5-percent-annual-chance. The March 1953 flood had a peak discharge of 42,400 cfs and a recurrence interval of 0.58-percent-annual-chance. The frequencies and peak discharges of these floods were recorded at the two USGS gages at Cornish (Reference 16).

building codes providing for the flood-proofing of existing structures in such areas (Reference 19).

In Buxton, Dayton, and Hollis, the Saco River is a wide, smoothly flowing river, which is affected by backwater from the three power dams owned by the Central Maine Power Company. Skelton Station is a large hydro-power dam located at Union Falls in Dayton. This composite dam is about 1,965 feet long and was completed in 1949. The station runs at full capacity during high water. Any excess water is passed through two grated, concrete spillway sections that have a capacity of approximately 132,000 cfs. The reservoir, which extends about 3 miles upstream from this dam, has a normal pool elevation of 126.3 feet.

The Clarks Mill Dam in Dayton is town-operated to maintain a reservoir for fire protection. The stop logs in the sluice are seldom changed. It is a low head dam, and the spillway is adequate to pass the peak flows of this study. The dam near Dennett Road is abandoned and in very poor shape. It is a low head rock crib dam, and the gate section has been completely washed out. Downstream of this dam, the stream drops rapidly to join the Saco River.

At Bar Mills, there is another hydro-power dam on the Saco River just north of U.S. Highway 202. The dam, which is about 1,245 feet long, was completed in 1888. The power station runs at full capacity during high water. Excess water during high flows is passed over a 62.6 foot long spillway section. The dam creates a reservoir that extends about 4.5 miles upstream, almost to West Buxton. The normal full pond elevation is 146.5 feet.

Proceeding upstream, the next dam on the Saco River is the West Buxton Power Station, built in 1906. It is a concrete gravity dam about 585 feet long and 30 feet high. The dam has two overflow sections, with a total crest length of 333 feet, a gated section with a 20-foot wide vertical lift gate, and two 40-foot wide stanchion sections. The power station could pass a 0.2-percent-annual-chance discharge with no retention of floodwaters. At its normal elevation of 177.1 feet, the pond extends about 1.5 miles upstream, almost to the Bonny Eagle Dam.

The hydro-power station at Bonny Eagle was built in 1910. It is a concrete gravity dam approximately 1,120 feet long with a 350-foot spillway section. This station has a maximum operating head of 38 feet.

Flood elevations downstream from the confluence of the Saco and Little Ossipee Rivers are affected by the dam at the Bonny Eagle Power Station, located about 1.5 miles downstream. The Little Ossipee River and its network of small streams are controlled by the operation of dams located upstream of the Town of Limington at the outlets of Little Ossipee Lake and Balch and Ledgemere Ponds, which have a combined storage capacity of 581 million cubic feet.

None of these dams significantly alters streamflow or controls flooding.

The Saco River Corridor Commission has zoned all the shoreland of the Saco River in Dayton, Hollis, and Limington, all the shoreland of the Little Ossipee River in Limington, and all of the shoreland along the Ossipee River from the New Hampshire border to its confluence with the Saco River. This protected area is known as the "Saco

River Corridor" and includes the land areas within 500 feet of the normal high water mark of the Saco River. In areas where the floodplain exceeds 500 feet in width, the corridor is extended to include the floodplain area up to a maximum of 1,000 feet wide. In Dayton, Hollis, and Limington, the Saco River corridor is zoned either Resource Protection or Limited Residential, which severely restricts encroachment and development in the floodplain (Reference 29). In 1967, the United States Army Corps of Engineers (USACE) made a study of the need for flood control structures in the Saco River basin-use regulations in the floodplain could best control and protect development (Reference 30). The Saco River Environmental Advisory Committee has mapped the Saco River floodplain based on the 1936 flood data. The maps have been provided to all Saco River Corridor communities for their use in planning and zoning in the floodplain. Results of updated pre-countywide studies were used to revise the delineation of the corridor.

Buxton's Town Land Use Plan (1976) provides flood protection by limiting development within 300 horizontal feet of the normal high-water mark of Bonny Eagle and Duck Ponds, The Bog (Groveville), the Little and Stroudwater Rivers, and McKenney Brook (Reference 29).

In the Town of Acton, a 10-foot high earthfill dam with a steel and concrete spillway is located at the outlet of Balch Pond. If adequately maintained, the dam provides safe passage of flood flows.

Estes Lake on the Mousam River has a capacity of 210 million cubic feet of storage. This affords flood protection for towns downstream from the Town of Alfred but not for the Town of Alfred itself because the lake is located close to the downstream corporate limits. Shaker Pond has a natural outlet with no dam.

The greatest flood protection measures afforded the Town of Cornish along the Ossipee and Saco Rivers are the relatively steep banks, which tend to contain the flood flows. In the headwaters of the Ossipee River, Ossipee Lake acts as a natural flood-retarding basin, reducing the peak discharge in the Town of Cornish. The bog areas upstream of Hiram Falls Dam on the Saco River also act as a flood storage basin and reduce the peak discharge in Cornish. Non-structural flood protection measures are also being utilized to aid in the prevention of future flood damage. These are land-use regulations adopted from the Saco River Corridor Commission, established in 1973. These land-use regulations control buildings within areas that have a high risk of flooding.

Flood protection measures in the Town of Eliot along the Piscataqua River are limited to those afforded by local zoning ordinances and those privately constructed by local residents.

The dam on Cooks Brook at Clarks Mills is operated by the Town of Hollis to maintain a reservoir for fire prevention. The stop logs in the sluice are changed infrequently. It is a low head dam, and the spillway is adequate to pass the peak flows utilized in this study. The dam near Dennett Road is abandoned and in poor condition. It is a low head, rock crib dam, with the gate section completely washed out. None of these dams significantly alter high stream flow or control flooding in the Town of Hollis.

In the Town of Limerick, the Little Ossipee Flowage Dam outlet on the Little Ossipee River has 4-foot high flashboards used for temporary flood protection.

Several dams are located on streams within the Town of Kennebunk, but these do not provide significant flood protection. Stone breakwater jetties have been constructed at the mouth of the Kennebunk River by the federal government in cooperation with local interests.

Three dams are located along the Great Works River in the Town of North Berwick. These dams have not been designed for flood control in the community, but they do serve to reduce peak flows downstream.

A 10-foot high earthfill dam with a steel and concrete spillway is located at the outlet of Balch Pond in the Town of Newfield. If adequately maintained, the dam provides safe passage of flood flows.

The USDA NRCS in Orono, Maine, maintains a flood control dike on Ogunquit Beach in the Town of Ogunquit. A stabilization project calls for the re-establishment of the primary dune height of 22 feet. The project affects a 6,300-foot length of beach and has a 5:1 fore-and-back slope with a 25-foot top width (Reference 31). Several dams are located on streams within Ogunquit, but these do not provide significant flood protection.

The steep banks of the Ossipee River, which contain the riverflows, provide the greatest flood protection to the Town of Parsonsfield. In the headwaters of the Ossipee River, Ossipee Lake reduces the peak flow discharges in the Town of Parsonsfield. There are no structural flood protection measures scheduled for construction in the future.

A masonry dam approximately 25 feet high is located at the outlet of Leigh's Mill Pond in South Berwick. Brattle Dam No. 1, a masonry structure approximately 10 feet high, crosses the Great Works River at Brattle Street. Brattle Dam No. 2, a stone and timber dam approximately 15 feet high, is located some 300 feet downstream. A masonry dam approximately 9 feet high is located at the outlet of Shapleigh Pond in the Town of Shapleigh. If adequately maintained, the dams provide safe passage of flood flows.

The flow of the Mousam River is regulated by Square Pond and Mousam and Estes Lakes, with a combined capacity of approximately 700 million cubic feet, and by power plants upstream. Seven dams on the Mousam River in the Town of Sanford provide negligible flood protection

In the Town of Waterboro, the Little Ossipee Flowage Outlet Dam on the Little Ossipee River has 4-foot high flashboards used for temporary flood protection. The Little Ossipee Lake Dam on Little Ossipee Lake provides safe passage of flood flows at its present discharge capacity.

Concrete seawall barriers exist along much of the coastline in the Town of Wells, and a jetty extends out from the entrance to Wells Harbor. Seawalls delineated on the FIRM for the January 16, 2003 Wells FIS revision have been identified as flood protection structures that reduce wave effects during the base flood.

In 1945, the USACE constructed a seawall at York Harbor in the Town of York. The seawall has been washed out and consequently improvements made (Reference 32). Seawalls delineated on the FIRM for the July 15, 2002 York FIS revision have been identified as flood protection structures that reduce wave effects during the base flood. There are no flood protection structures on the streams in York.

For the Little Ossipee River, flood flows for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods were computed using the USDA NRCS TR-20 computer program (Reference 33). A 24-hour duration storm and normal antecedent moisture conditions were used. The USGS gage (No. 01066500) located on the Little Ossipee River near South Limington has a period of record of 36 years. The computed discharges correlated favorably with a log-Pearson Type III analysis of the gage data (Reference 34).

For the Littlefield River and Mousam River (Lower Reach) for the Town of Alfred, the primary source of peak-flow data used to determine flood discharges for the flooding sources studied by detailed methods was USGS gaging station No. 01069500 on the Mousam River near West Kennebunk. The West Kennebunk gage is located approximately 0.5 mile downstream of Estes Lake Dam and has a drainage area of approximately 99 square miles. Records of flood peaks were available at this gage from 1940 to 1984. The 1-percent-annual-chance discharge at the gage was based on a log-Pearson Type III analysis of annual peak flow data (Reference 35).

Peak discharges upstream from the USGS gage were established by adjusting the discharge computed at the gage from differences in drainage area between the upstream site and the gage using the following formula:

$$Q = Q_g(A/A_g)^b$$

where Q is the desired 1-percent-annual-chance flood discharge at the upstream site, Q_g is the 1-percent-annual-chance flood discharge at the USGS gage, and A and A_g, are drainage areas at the respective sites. The value of the exponent b is 0.8.

Since the gage is located downstream of all regulation affecting Sanford, the peak flow data already take into account the effects of any regulation by upstream lakes and reservoirs. The flood discharges computed at the gage were modified on the basis of drainage area relationships to compute the adopted discharges in Sanford (Reference 34). The discharges for the Mousam River (Lower Reach) were derived from discharges calculated for Estes Lake. According to USGS Water-Supply Paper 1580-B, a useable storage capacity of less than 4.5 million cubic feet per square mile, in general, affects peak discharges by less than 10 percent (Reference 36). The useable storage capacity of Estes Lake is less than this limit; therefore, it was not considered in the computation of upstream flood discharges. The 1-percent-annual-chance flood flow for Tributary to Middle Branch Mousam River was determined using the USGS regional regression equation for the region (Reference 34).

Two USGS gages on the Ossipee River were used to establish the peak discharge-frequency relationships. The gage located at Effingham Falls, New Hampshire, has 34 years of record, and the gage located at Cornish, Maine, has 60 years of record (References 37 and 38). Values of the 10-, 2-, 1-, 0.2-percent-annual-chance peak discharges were obtained from a log-Pearson Type III distribution of annual peak flow data in accordance with the U.S. Water Resources Council Bulletin 17 (Reference 39). Peak flows for other locations on the Ossipee River were computed by use of the drainage area proration method mentioned above, $Q = Q_g(A/A_g)^b$ (Reference 40). The value of b applied to the Ossipee River was 0.8. This value was based on the analysis of peak discharges at the two gages listed above.

In the pre-countywide July 4, 1988 FIS for the Town of Kennebunkport, the 1-percent-annual-chance flood elevation at the Lake of the Woods area was also determined by the

TABLE 5 – SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE
KEAY BROOK[1] – continued					
Upstream from confluence with Murdock Lake tributary, about 500 feet downstream from Ridlon Road	9.20	421	626	723	960
Upstream from confluence with Unnamed Tributary, about 3,100 feet upstream from Ridlon Road	8.40	403	601	695	925
KENNEBUNK POND					
At outlet (west)	1.14	*	*	69	*
KENNEBUNK RIVER					
At mouth	52.88	2,317	3,644	4,335	6,262
LITTLE OSSIPEE RIVER					
At the mouth	187.00	4,630	7,890	9,640	14,800
At USGS gage No. 01066500	168.00	4,250	7,240	8,850	13,600
At the Limington-Limerick-Waterboro town boundary	157.00	4,030	6,860	8,380	12,900
At Little Ossipee Flowage Outlet Dam	155.20	2,900	6,400	7,800	12,000
At State Route 5	112.40	4,000	7,400	9,000	13,400
At a point approximately 2,500 feet below the upstream corporate limits	66.50	1,700	3,300	4,000	6,000

*Data not computed
[1]Updated calculations for this countywide study

For the Salmon Falls River in Lebanon, the computer model Quick-2, a simple step-backwater modeling program (Reference 78) was used to analyze Bog Brook, Great Brook, and Little River. The model requires cross section data, Manning's "n" value, slope, and flow data as input. The input values were determined from field reconnaissance. All of the structures and representative cross sections were modeled for each flooding source. The flood line was produced using engineering judgment. Cross sections for the backwater analyses of the Salmon Falls River were obtained from aerial photographs flown in May 1980 at a scale of 1.0 inch equals 800 feet. The below-water sections were obtained by field measurement. Water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step- backwater computer program (Reference 79). Starting water-surface elevations for the Salmon Falls River were taken from known elevations in the FIS for the City of Rochester, New Hampshire (Reference 48).

Cross section data for the Little Ossipee River in Limerick were obtained from 21 surveyed valley and bridge cross sections and from USGS topographic maps (Reference 57). Water-surface elevations of floods of the selected recurrence intervals were computed using the USDA NRCS WSP-2 computer program (Reference 59).

In Limington, cross section data for the backwater analyses of the Saco and Little Ossipee Rivers were obtained from aerial photographs (Reference 80). The below-water sections were obtained by field measurements. For the streams studied by approximate methods, the 1-percent-annual-chance flood elevations were estimated by a method developed by USGS hydrologists at the Augusta, Maine, office (Reference 73). The regional stage-frequency relationship indicates that the inundation limit of the 1-percent-annual-chance flood is an estimated 10 feet higher than the stream elevation as mapped on USGS topographic maps (References 57 and 58). The Town of Lebanon used this data for their starting water-surface elevation data for the Little Ossipee River.

For the Bunganut, Kennebunk, Roberts-Wadley, and Swan Ponds in Lyman, cross sections were extended into overbank areas using topographic maps (Reference 60). The 1-percent-annual-chance flood elevation for Bunganut Pond was determined by rating the outlet overflow weir and culvert pipe. The outlet weir was rated by applying the USGS step-backwater computer program (Reference 81). Traditional flow-over-weir equations were not applicable in this case. Flow through the culvert was determined by applying appropriate formulas published by USGS (Reference 82). The 1-percent-annual-chance flood elevation for Swan Pond was determined by rating the culvert at the outlet of the pond. Flow through the culvert was calculated by applying standard USGS practices (Reference 82). Flow over the roadway at the culvert was computed using the USGS step-backwater computer program (Reference 81). The 1-percent-annual-chance flood elevation for Kennebunk Pond was determined by rating both the east and west outlets of the pond. The constricted opening at the west outlet was rated by using the USGS step-backwater computer program (Reference 81). The culvert at the east outlet was rated using standard USGS practices (Reference 82).

The 1-percent-annual-chance flood elevation for Roberts-Wadley Pond in Lyman was calculated using the USGS step-backwater computer program (Reference 81). The dam at the outlet of the pond is irregular and has two small breached sections. Traditional weir equations were not appropriate. The step-backwater program was used to compute critical flow over the dam and then to adjust for the effects of velocity head to compute the pond's 1-percent-annual-chance flood elevation. Starting elevations used in step-

For the Great Works River in South Berwick, topographic data were obtained from 32 surveyed valley and bridge cross sections and from USGS topographic maps (References 60 and 83). Field surveys were obtained in 1975. Only those features in the floodplain at the time the surveys were made were considered in the computations. Water-surface elevations of floods of the selected recurrence intervals were computed using the USDA NRCS WSP-2 computer program (Reference 59). Starting water-surface elevations for the Great Works River were calculated by the slope/area method.

Valley and bridge cross sections for Little Ossipee Lake and Little Ossipee River in Waterboro were obtained from USGS 15-minute series topographic maps (Reference 57). Water-surface elevations of floods of the selected recurrence intervals for the Little Ossipee River from the downstream corporate limits to the Waterboro-Limington-Limerick town boundary were computed using the USGS step-backwater computer program (Reference 92). Water-surface elevations for the Little Ossipee River from the Waterboro-Limington-Limerick town boundary to the upstream corporate limits were computed using the USDA NRCS WSP-2 computer program (Reference 59). Starting water-surface elevations for the Little Ossipee River were obtained from the FIS for Limington (described above).

Water-surface elevations for Blacksmith Brook, Depot Brook, Green Brook, Little River (Town of Kennebunk), Merriland River (Lower and Upper Reach), Ogunquit River, South Branch of West Brook, Stevens Brook, Tributary 1 to Green Brook, Webhannet River, and West Brook in Wells were computed using USACE HEC-2 step-backwater model (Reference 65). Cross sectional data for both computer models were obtained from photogrammetric mapping, while below-water cross sections were obtained by field survey.

Also in Wells, the Little River (Town of Kennebunk) is an estuary where water-surface elevations are not a function of discharge alone. For this reason, the HEC-2 step-backwater computer program was not used in analyzing the flooding on the Little River (Town of Kennebunk) (Reference 65). Starting water-surface elevations for the Ogunquit, Webhannet, and Meniland Rivers and Blacksmith Brook were taken at critical depth. Starting water-surface elevations for Depot Brook, Green Brook, Stevens Brook, West Brook, and South Branch of West Brook were obtained using the slope/area method. For Tributary 1 to Green Brook, the starting water-surface elevations were taken from the flood profile of Green Brook at their confluence. It was determined that flooding from Branch Brook and Little River is controlled by the Atlantic Ocean for their detailed study lengths within Wells. Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

In York, cross sections for the Cape Neddick River, Cider Hill Creek, Dolly Gordon Brook, Tributary 1 to Cape Neddick River and the York River were obtained from photogrammetric mapping while below-water sections were obtained by field survey (Reference 68). Water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (Reference 65). For the streams studied by detailed riverine methods, with the exception of Bridges Swamp, the starting water-surface elevations were taken at the mean spring tide level of 5.4 feet. For Bridges Swamp, the starting water-surface elevations were computed using the slope/area method.

River, Saco River, Salmon Falls River (floodway data tables [FDT] only), Smith Brook, Spinney Creek, Spruce Creek, Stevens Brook, Webhannet River, and the York river (FIRM only) were revised for backwater elevations.

Roughness factors (Manning's "n") used in the hydraulic computations for this updated countywide study were determined from field observations guided by U.S. Geological Water Supply Publications (References 96 and 97).

Table 6, "Manning's "n" Values," shows the channel and overbank "n" values for the streams studied by detailed methods in both the pre-countywide and countywide studies. Values calculated for this countywide study have been noted in the table:

TABLE 6 – MANNING'S "n" VALUES

Flooding Source	Channel "n"	Overbanks
Batson River	0.020-0.050	0.050-0.090
Blacksmith Brook	0.050	0.090
Bridges Swamp	0.050	0.090
Bunganut Pond (at outlet)	0.030-0.045	0.055-0.110
Cape Neddick River	0.013-0.050	0.090
Cider Hill Creek	0.020-0.050	0.050-0.020
Coffin Brook[1]	0.045-0.05	0.09-0.11
Coffin Brook Tributary 1[1]	0.05-0.06	0.1-0.11
Cooks Brook	0.030-0.055	0.040-0.110
Day Brook	0.013-0.050	0.050-0.090
Depot Brook	0.013-0.050	0.050-0.090
Dolly Gordon Brook	0.013-0.050	0.050-0.090
Driscoll Brook[1]	0.04-0.05	0.045-0.1
Ferguson Brook[1]	0.045-0.06	0.05-0.12
Goosefare Brook	0.013-0.050	0.050-0.090
Great Works River	0.030-0.060	0.035-0.110
Green Brook	0.030-0.050	0.030-0.090
Kennebunk Pond (at west outlet)	0.030-0.045	0.055-0.110
Kennebunk River	0.020-0.050	0.090
Keay Brook[1]	0.045-0.06	0.085-0.11
Little Ossipee River	0.015-0.057	0.060-0.100
Little Ossipee River (Limington)	0.020-0.060	0.045-0.125
Little River	0.020-0.050	0.050-0.090
Little River (Cornish)	0.035-0.05	0.08-0.10
Little River(Berwick)[1]	0.045-0.075	0.05-0.15
Littlefield River	0.030-0.045	0.055-0.110
Merriland River	0.013-0.050	0.050-0.090
Mill Brook	0.013-0.050	0.050-0.090
Moors Brook	0.020-0.050	0.050-0.090
Mousam River (Alfred)	0.030-0.070	0.070-0.080
Mousam River (Kennebunk)[1]	0.038-0.045	0.09-0.11
Mousam River (Lower Reach)	0.03-0.07	0.07-0.08
Mulloy Brook[1]	0.045-0.06	0.09

[1]Updated calculations for this countywide study

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages state and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS report provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations; delineations of the 1- and 0.2-percent-annual-chance floodplains; and a 1-percent-annual-chance floodway. This information is presented on the FIRM and in many components of the FIS report, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS report as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community.

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM. On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, AO, AH, and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For unrevised streams in York County, data were taken from previously printed FISs for each individual community and are compiled below.

For each stream studied by detailed methods, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:62,500, with a contour interval of 20 feet (Reference 57); at a scale of 1:62,500, with a contour interval of 10 feet (Reference 83); at a scale of 1:24,000, with contour intervals of 10 and 20 feet (References 114 and 115); at a scale of 1:24,000, with a contour interval of 20 feet (References 58); at a scale of 1:24,000, with a contour interval of 10 feet (Reference 60); at a scale of 1:4,800, with a contour interval of 4 feet (References 70, 75, 76, 80, and 116); at a scale of 1:4,800, with a contour interval of 5 feet (Reference 74); at a scale of 1:2,400, with a contour interval of 5 feet (Reference 68); and at a scale of 1:480, with a contour interval of 2 feet (Reference 117).

For the towns of Acton, Limerick, North Berwick, Newfield, Sanford, Shapleigh South Berwick, and Waterboro, the boundaries were interpolated between cross sections using aerial photographs at a scale of 1:20,000 in addition to using topographic maps (Reference 118).

In addition to topographic maps, the floodplain boundaries of the Town of Berwick were delineated using the FISs for the cities of Somersworth and Rochester, New Hampshire.

For the Town of Eliot, portions of the flood boundaries for Spinney Creek were taken from the FIS for the Town of Kittery (Reference 43).

For unrevised flooding sources studied by approximate methods, the boundaries of the 1-percent-annual-chance floodplain were delineated using the Flood Hazard Boundary Maps (FHBM) for the Town of Acton (Reference 119); Town of Alfred (Reference 120); Town of Arundel, also using USGS topographic maps at a scale of 1:24,000 with contour intervals of 10 and 20 feet (References 58, 60, and 121); City of Biddeford (Reference 122); Town of Cornish (Reference 123); Town of Eliot (Reference 124); Town of Kennebunk, also using information from town officials and residents (Reference 125); Town of Kennebunkport, also using information from town officials and residents (Reference 126); Town of Kittery (Reference 127); Town of Lyman (Reference 128); Town of North Berwick (Reference 129); Town of Newfield (Reference 130); Town of Ogunquit (Reference 131); Town of Old Orchard beach (Reference 114); Town of Shapleigh (Reference 132); Town of South Berwick (Reference 133); Town of Wells (Reference 134); and the Town of York (Reference 135).

For the streams studied by approximate methods in the Town of Berwick, the boundary of the 1-percent-annual-chance flood was determined using a regional equation developed by the United States Geological Survey (Reference 34) in conjunction with the Corps of Engineers HEC-2 backwater model (Reference 67) and the FHBM for the Town of Berwick (Reference 136).

The approximate 1-percent-annual-chance flood boundaries in Buxton and Limington were delineated using 7.5- and 15-minute USGS topographic maps enlarged to a scale of 1"=1,000' (References 57, 58, and 137).

The approximate 1-percent-annual-chance flood boundaries for the five small streams studied in Dayton and the approximate studies of the streams in Hollis were delineated on 15-minute USGS topographic maps enlarged to a scale of 1:12,000 (References 57 and 138).

For Parsonsfield, the approximate study area flood boundaries were delineated using topographic maps at scales of 1:24,000 and 1:62,500, with a contour interval of 20 feet (References 57 and 58). Approximate flood boundaries in some portions of the study area were taken from the FHBM (Reference 86).

For the streams studied by approximate methods in the City of Saco, the 1-percent-annual-chance floodplain boundaries were delineated using the previously printed FHBM of Saco (Reference 139). The portion of Deep Brook south of Route 5 was redelineated using updated topographic maps (Reference 3).

For the Town of Sanford, the approximate study area flood boundaries were transferred from the previously published FIS for Sanford (Reference 115).

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM.

For this countywide revision, for the stream studied in detail, the 10-, 2-, 1-, and 0.2-percent-annual-chance floodplains have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using LiDAR data, with a contour interval of 2 feet.

For this countywide revision, for the coastal areas and riverine backwater effects studied in detail, the 10-, 2-, 1-, and 0.2-percent-annual-chance floodplains have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using LiDAR data, with a contour interval of 2 feet (Reference 13).

In Kennebunkport, for the revised areas studied by approximate methods, the 1-percent-annual-chance floodplain was delineated using LiDAR data with a contour interval of 2 feet (Reference 13).

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the base flood can be carried without substantial increases in flood heights. Minimum federal standards limit such increases to 1 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways used for this study were computed for certain stream segments on the basis of equal-conveyance reduction from each side of the floodplain. In areas of tidal flooding, floodways are generally not applicable; however, since the estuarine portion of streams can experience flooding that occurs independently from tidal flooding, floodways have been determined for those downstream portions of the Saco River, Little River, Moors Brook, and Thatcher Brook in Biddeford; of the Kennebunk, Mousam and Little Rivers in Kennebunk; of Spinney Creek, Spruce Creek, and the Piscataqua River in Kittery; of Goosefare and Mill Brooks in Old Orchard Beach; of the Saco River and Goosefare Brook in Saco; and of the York River in York.

The coastal study impacted the limit of backwater effects on some of the Floodway Data Tables and Flood Profiles by revising the annual 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations at the confluence of rivers and the coastal flooding sources. Affected Floodway Data Tables and Flood Profiles were updated for Blacksmith Brook, Bridges Swamp, Cape Neddick River, Cider Hill Creek, Depot Brook, Dolly Gordon Brook, Goosefare Brook, Josias River, Kennebunk River, Little River, Merriland River (Lower Reach), Mill Brook, Mousam River (Town of Kennebunk), Ogunquit River, Salmon Falls River (FDT only), Spinney Creek, Spruce Creek, Stevens Brook, Webhannet River, and the York river (FIRM only).

The results of the floodway computations are tabulated for selected cross sections (see Table 11, “Floodway Data”). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

In some instances, no HEC-2 step-backwater analysis has been performed for the downstream portions of the flooding source. In these cases, their floodways were determined using engineering judgment. The floodways for the Saco River, Little River, Moors Brook, and Thatcher Brook in Biddeford; the Little River in Kennebunk; the Piscataqua River in Kittery; Goosefare and Mill Brooks in Old Orchard Beach; the Saco River in Saco; and the York River in York are entirely coincident with the channel banks.

Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (see Table 11, "Floodway Data"). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

Portions of the computed floodway widths extend beyond the corporate limits for Cooks Brook in Dayton and Hollis; for Goosefare Brook in Old Orchard Beach and Saco; for the Great Works River in North Berwick and Sanford; for Green Brook in Wells; for the Kennebunk River in Arundel, Kennebunk, and Kennebunkport; for the Little River in Kennebunkport and Wells; for the Little Ossipee River in Acton, Limerick, Limington, Newfield, Shapleigh, and Waterboro; for the Mousam River in Alfred; for the Mousam River (Lower Reach) in Sanford; for the Ogunquit River in Ogunquit and Wells; for the Ossipee River in Cornish and Parsonsfield; for the Saco River in Biddeford, Buxton, Cornish, Dayton, Hollis, Limington, and Saco; for the Saco River-Right Channel in Buxton; for the Salmon Falls River in Berwick and Lebanon; and for Spinney Creek in Eliot and Kittery.

For this countywide study, for the revised flooding sources in the Town of Berwick, including Coffin Brook, Unnamed Tributary to Coffin Brook (Coffin Brook Tributary 1), Driscoll Brook, Ferguson Brook, Keay Brook, Little River, Mulloy Brook, Worster Brook, and Unnamed Tributary to Worster Brook (Worster Brook Tributary 3), no floodway data were computed. Profiles were created for these flooding sources and are included in this countywide report (Exhibit 1).

No floodways were computed for the Littlefield River and Tributary to Middle Branch Mousam River in Alfred, for Skelton Station Pond in Dayton, and for the Town of Lyman. A floodway was computed for Sawyer Brook in Saco; however, it is not shown in this FIS due to extremely narrow floodway widths.

Encroachment into areas subject to inundation by floodwaters having hazardous velocities aggravates the risk of flood damage and heightens potential flood hazards by further increasing velocities. A listing of stream velocities at selected cross sections is provided in Table 11, "Floodway Data." To reduce the risk of property damage in areas where the stream velocities are high, the community may wish to restrict development in areas outside the floodway.

Near the mouths of streams studied in detail, floodway computations are made without regard to flood elevations on the receiving water body. Therefore, "Without Floodway" elevations presented in Table 11 for certain downstream cross sections of Thatcher Brook in Biddeford, of the Ossipee River in Cornish, of the Kennebunk and Little Rivers in Kennebunkport, of the Little Ossipee River in Limington, of the Ogunquit River Tributary in Ogunquit, of Stevens Brook in Wells, and of the Cape Neddick River in

York are lower than the regulatory flood elevations in that area, which must take into account the 1-percent-annual-chance flooding due to backwater from other sources.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation (WSEL) of the base flood more than 1 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 3, "Floodway Schematic."

Under the State of Maine Revised Statutes Annotated (M.R.S.A.) Title 38 § 439-A, 7C where the floodway is not designated on the Flood Insurance Rate Map, the floodway is considered to be the channel of a river or other water course and the adjacent land areas to a distance of one-half the width of the floodplain, as measured from the normal high water mark to the upland limit of the floodplain, unless a technical evaluation certified by a registered professional engineer is provided demonstrating the actual floodway based upon approved FEMA modeling methods.

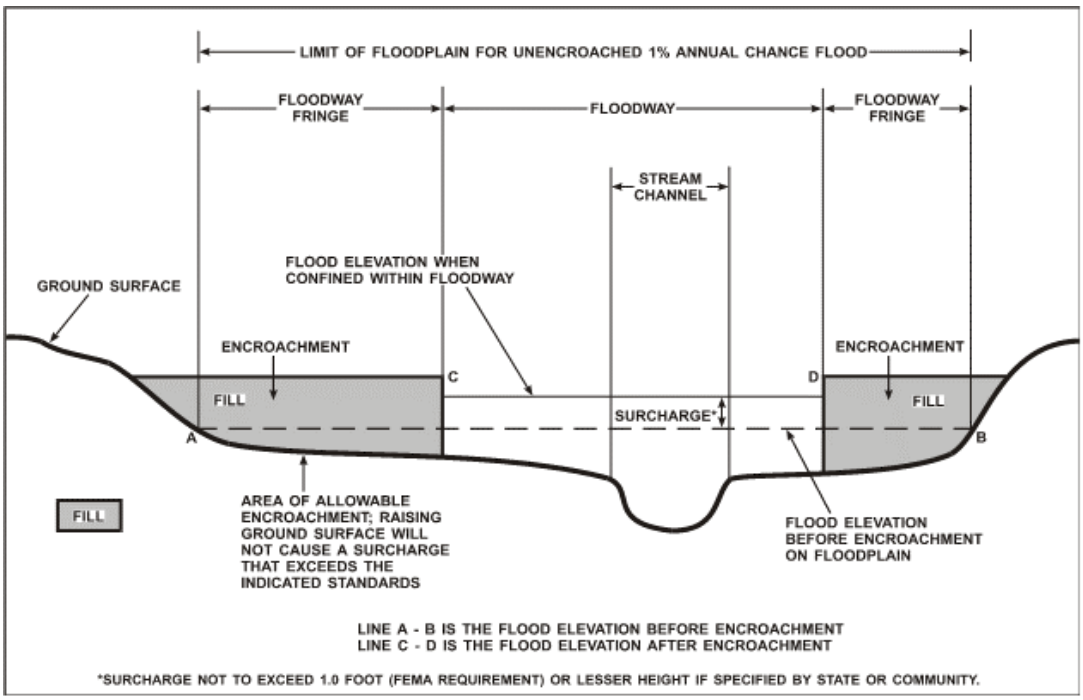
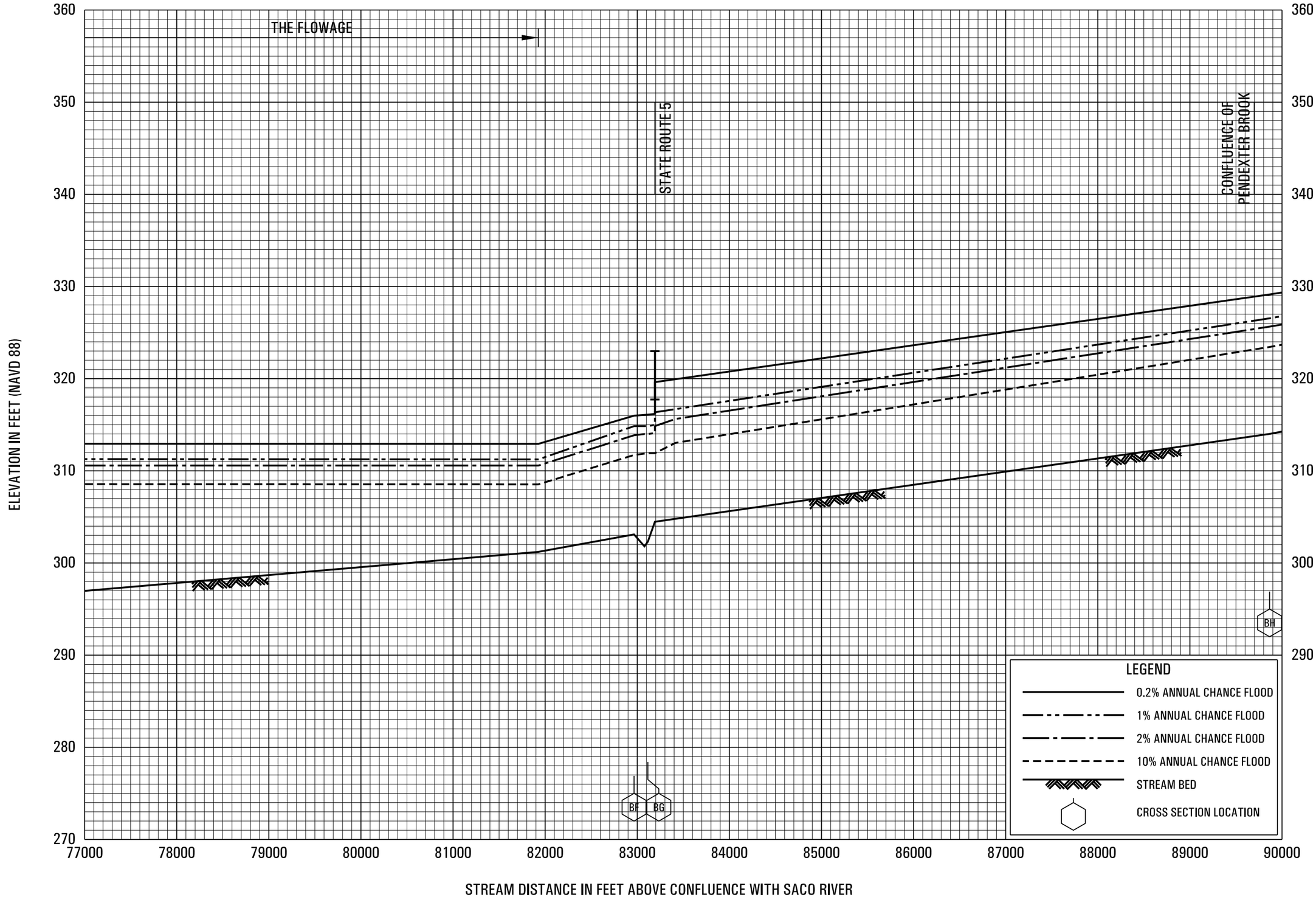


Figure 3. Floodway Schematic

Table 12, "1-Percent-Annual-Chance Flood Data," shows the 1-percent-annual-chance flood data for Littlefield River. No floodway data were computed for Littlefield River.



FLOOD PROFILES

LITTLE OSSIPEE RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

YORK COUNTY, ME

(ALL JURISDICTIONS)

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION (FEET NAVD 88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	1,050	220	963	10.0	226.3	221.3 [2]	222.3	1.0
B	1,150	130	894	10.8	226.3	224.6 [2]	225.2	0.6
C	1,250	150	756	12.8	230.3	230.3	230.3	0.0
D	1,380	100	1,170	8.2	234.1	234.1	234.1	0.0
E	1,460	100	1,360	7.1	235.6	235.6	235.6	0.0
F	1,860	130	1,680	5.7	236.7	236.7	236.8	0.1
G	2,210	140	1,260	7.7	239.5	239.5	239.5	0.0
H	2,740	220	2,800	3.4	243.1	243.1	243.1	0.0
I	3,530	350	4,970	1.9	243.9	243.9	244.1	0.2
J	4,190	460	6,060	1.6	244.1	244.1	244.4	0.3
K	5,320	530	6,780	1.4	244.4	244.4	244.8	0.4
L	6,750	700	8,810	1.0	244.6	244.6	245.0	0.4
M	8,060	500	5,590	1.6	244.7	244.7	245.2	0.5
N	8,820	600	7,680	1.2	244.9	244.9	245.4	0.5
O	9,810	1,220	13,300	0.7	244.9	244.9	245.5	0.6
P	12,710	430	4,300	2.1	245.5	245.5	246.2	0.7
Q	14,920	280	2,880	3.1	247.8	247.8	248.4	0.6
R	15,960	450	6,400	1.4	248.5	248.5	249.1	0.6
S	16,630	310	4,960	1.8	248.7	248.7	249.3	0.6
T	18,940	750	12,100	0.7	249.1	249.1	249.7	0.6
U	20,140	750	11,400	0.8	249.1	249.1	249.7	0.6
V	24,330	700	6,890	1.3	249.5	249.5	250.3	0.8
W	28,840	800	11,300	0.8	250.0	250.0	250.9	0.9
X	31,140	870	9,170	1.0	250.2	250.2	251.1	0.9
Y	31,670	850	8,240	1.1	250.3	250.3	251.2	0.9
Z	32,120	400	3,120	2.8	250.5	250.5	251.5	1.0
AA	32,660	430	896	9.9	259.5	259.5	259.5	0.0

[1] FEET ABOVE CONFLUENCE WITH SACO RIVER

[2] ELEVATIONS COMPUTED WITHOUT CONSIDERING BACKWATER EFFECTS FROM SACO RIVER

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

YORK COUNTY, ME

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE OSSIPEE RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION (FEET NAVD 88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AB	32,900	350	1,790	5.0	266.2	266.2	266.8	0.6
AC	33,010	280	1,250	7.1	267.4	267.4	268.3	0.9
AD	33,090	260	1,550	5.7	269.0	269.0	270.0	1.0
AE	33,270	260	862	10.3	272.7	272.7	272.7	0.0
AF	33,690	150	953	9.3	279.4	279.4	279.7	0.3
AG	33,860	120	977	9.1	280.8	280.8	281.6	0.8
AH	33,970	90	897	9.9	283.9	283.9	283.9	0.0
AI	34,240	280	2,380	3.7	284.9	284.9	285.7	0.8
AJ	34,340	320	2,600	3.4	285.3	285.3	286.1	0.8
AK	34,620	230	2,480	3.6	286.0	286.0	286.9	0.9
AL	34,980	430	4,990	1.8	286.5	286.5	287.4	0.9
AM	35,850	1,090	10,600	0.8	286.7	286.7	287.6	0.9
AN	39,160	735	6,210	1.4	287.4	287.4	288.4	1.0
AO	39,910	810	4,800	1.8	287.8	287.8	288.7	0.9
AP	41,660	930	9,020	1.0	288.3	288.3	289.3	1.0
AQ	44,040	800	7,320	1.2	288.6	288.6	289.6	1.0
AR	45,460	490	4,360	2.0	289.0	289.0	290.0	1.0
AS	45,980	330	2,560	3.5	289.5	289.5	290.4	0.9
AT	46,110	690	2,420	3.7	290.3	290.3	290.7	0.4
AU	47,060	570	5,760	1.5	290.8	290.8	291.8	1.0
AV	48,070	810	6,980	1.3	291.2	291.2	292.2	1.0
AW	48,740	680	5,900	1.5	291.4	291.4	292.4	1.0
AX	49,570	570	5,040	1.7	291.8	291.8	292.8	1.0
AY	50,530	720	5,990	1.4	292.3	292.3	293.3	1.0
AZ	51,760	300	3,290	2.6	293.2	293.2	294.1	0.9
BA	53,190	680	5,860	1.4	294.2	294.2	295.1	0.9
BB	54,020	500	5,060	1.7	294.7	294.7	295.6	0.9

[1] FEET ABOVE CONFLUENCE WITH SACO RIVER

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

YORK COUNTY, ME

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE OSSIPEE RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION (FEET NAVD 88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
BC	54,415	155	1,997	3.9	295.2	295.2	296.2	1.0
BD	54,565	205	2,436	3.2	295.2	295.2	296.2	1.0
BE	54,765	140	2,210	3.5	295.5	295.5	296.5	1.0
BF	82,965	287	1,948	4.6	314.8	314.8	315.8	1.0
BG	83,115	88	985	9.1	314.8	314.8	315.8	1.0
BH	89,865	190	1,696	5.2	326.5	326.5	327.5	1.0
BI	99,765	244	1,849	3.5	344.9	344.9	345.9	1.0
BJ	99,965	318	1,437	4.5	346.0	346.0	347.0	1.0
BK	105,065	110	1,037	3.9	354.5	354.5	355.5	1.0
BL	110,765	171	1,278	3.1	362.7	362.7	363.7	1.0
BM	115,800	85	644	4.6	383.6	383.6	384.6	1.0
BN	131,000	186	864	2.7	420.1	420.1	421.1	1.0
BO	137,300	130	764	2.8	437.8	437.8	438.8	1.0
BP	142,950	61	253	3.3	473.7	473.7	474.7	1.0
BQ	143,160	53	621	1.3	486.0	486.0	487.0	1.0
BR	143,235	66	472	1.8	487.8	487.8	488.8	1.0
BS	144,275	64	283	2.9	495.9	495.9	496.9	1.0
BT	145,350	75	290	2.8	504.7	504.7	505.7	1.0
BU	145,450	300	3,157	0.3	515.7	515.7	516.7	1.0
BV	147,650	500	6,449	0.1	515.7	515.7	516.7	1.0
BW	157,300	60	233	2.7	530.2	530.2	531.2	1.0
BX	157,450	69	148	4.3	532.0	532.0	533.0	1.0
BY	162,250	53	178	3.1	549.0	549.0	550.0	1.0
BZ	162,350	25	112	4.9	550.5	550.5	551.5	1.0
CA	162,600	145	1,120	0.5	552.3	552.3	553.3	1.0

[1] FEET ABOVE CONFLUENCE WITH SACO RIVER

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

YORK COUNTY, ME

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE OSSIPEE RIVER

Scour Analysis:

Scour analysis is done per Chapter 2 of the BDG, using the FHWA document: HEC-18 Evaluating Scour At Bridges, Fifth Edition, April 2012.

Variables:

$D_{50} := 7.3mm = 0.024\,ft$	Particle size in a mixture of which 50% are smaller (ft), from Geotech report
$D_{84} := 35mm = 0.115\,ft$	Particle size in a mixture of which 84% are smaller (ft) from Geotech report
$soil_type := "gravel"$	Soil type based on D50, see grain size distribution char in Geotech report
$D_m := 1.25 \cdot D_{50} = 0.03\,ft$	Diameter of the smallest nontransportable particle in the bed material in the contracted section
$g := 32.2\,\frac{ft}{s^2}$	Acceleration of gravity
$n := 0.036$	Mannings n value, used in HEC-RAS
$Q_{total} := 7990\,\frac{ft^3}{s}$	Total flow @ Q100
$Q_{up} := 6126.5\,\frac{ft^3}{s}$	Discharge in the channel upstream of the bridge contraction, RS: 10475.31
$Q_{contract} := 7674.88\,\frac{ft^3}{s}$	Discharge in the channel at the contraction, RS:10252.17BR D
$S_1 := 0.0056\,\frac{ft}{ft}$	Slope of energy grade line of main channel, measured from Microstation river section
$V := 6.05\,\frac{ft}{s}$	Average velocity upstream of bridge, from HEC-RAS model, RS: 10293.06
$V_2 := 7.28\,\frac{ft}{s}$	Average velocity through the structure, RS: 10252.17BR D
$W := 135ft - 5ft = 130\,ft$	Bottom width of the contracted section less pier width(s) (ft), @ Q100, RS: 10252.17BR U
$W_{up} := 172ft$	Bottom width of the upstream main channel @ Q 100, RS: 10475.31
$W_{top} := (145.37ft - 5ft) = 140.37\,ft$	Top width of the channel, at the contracted section less pier width(s) (ft), @ Q100, RS: 10252.17BR U
$W_{up_top} := 116.68ft$	Top width of the upstream main channel from HEC-RAS output table @ Q100, RS: 10475.31
$y := 8.68ft$	Average depth of flow upstream of the bridge (ft), from HEC-RAS model @ Q100, RS:10475.31
$y_{contract} := 6.66ft$	Depth of flow at the contraction, from HEC-RAS model @ Q100, RS: 10252.17BR U (for type = bridges)

$\gamma_w := 62.4 \frac{lb}{ft^3}$ Density of water

Contraction Scour:

This project meets the conditions of Case 1b of the Contraction Scour cases, according to 6.2.2 of HEC-18 Evaluation Scour at Bridges, Fifth Edition 2012.

$D := D_{50} = 0.024 \cdot ft$ Particle size for V_c (ft)

$K_u := 11.17$ Correction factor (english units), from HEC-18

$V_c := K_u \cdot \left(\frac{y}{ft}\right)^{\frac{1}{6}} \cdot \left(\frac{D}{ft}\right)^{\frac{1}{3}} \cdot \frac{ft}{s} = 4.616 \cdot \frac{ft}{s}$ HEC-18 Eq. 6.1 - Critical velocity above which bed material of size D and smaller will be transported (ft/s)

$$cohesion := \begin{cases} \text{"cohesionless"} & \text{if } D_{50} > 0.2mm \\ \text{"cohesive"} & \text{otherwise} \end{cases} = \text{"cohesionless"}$$

Check for cohesive soil, HEC-18 pg 6.2, and section 6.7

$$controlling_contraction_scour := \begin{cases} \begin{cases} \text{"clear water eq"} & \text{if } V_c > V \\ \text{"live bed eq"} & \text{if } V_c < V \end{cases} & \text{if cohesion} = \text{"cohesionless"} \\ \text{"ultimate scour eq"} & \text{if cohesion} = \text{"cohesive"} \end{cases} = \text{"live bed eq"}$$

HEC-18, 6.2.1

Live-Bed Contraction Scour, HEC-18, 6.3

$Q_{1_livebed} := Q_{up} = 6126.5 \cdot \frac{ft^3}{s}$ Flow in the upstream channel transporting sediment

$Q_{2_livebed} := Q_{contract} = 7674.88 \cdot \frac{ft^3}{s}$ Flow in the contracted channel

$W_{1_livebed} := W_{up} = 172\ ft$ Bottom width of the upstream main channel that is transporting bed material

$W_{2_livebed} := W = 130\ ft$ Bottom width of the main channel in contracted section less pier width(s)

$y_o := y_{contract} = 6.66\ ft$ Existing depth in the contracted section before scour

$y_{1_livebed} := y = 8.68\ ft$ Average depth in the upstream main channel

$V_{star} := \left(g \cdot y_{1_livebed} \cdot S_1\right)^{\frac{1}{2}} = 1.251 \frac{ft}{s}$ Shear velocity in the upstream section

$\omega := 0.9\frac{m}{s} = 2.953\frac{ft}{s}$

Fall velocity of bed material based on the D50, using HEC-18 figure 6.8.

$$k_1 := \begin{cases} 0.59 & \text{if } \frac{V_{star}}{\omega} < 0.5 \\ 0.64 & \text{if } 0.5 < \frac{V_{star}}{\omega} < 2.0 \\ 0.69 & \text{otherwise} \end{cases} = 0.59$$

Exponent

$$y_{2_livebed} := \left(\frac{Q_{2_livebed}}{Q_{1_livebed}}\right)^{\frac{6}{7}} \cdot \left(\frac{W_{1_livebed}}{W_{2_livebed}}\right)^{k_1} \cdot y_{1_livebed} = 12.42\,ft$$

$y_{s_livebed} := y_{2_livebed} - y_o = 5.76\,ft$

Clear Water Contraction Scour, HEC-18, 6.4

$K_{u_clearwater} := 0.0077$

Correction factor for clearwater contraction scour

$Q_{clearwater} := Q_{contract} = 7674.88 \cdot \frac{ft^3}{s}$

Discharge through the bridge or on the set-back overbank area at the bridge associated with the width, W

$W_{clearwater} := W = 130\,ft$

Bottom width of the contracted section less pier width(s)

$$y_{2_clearwater} := \left[\frac{K_{u_clearwater} \cdot \left[\frac{Q_{clearwater}}{\left(ft^3 \div s\right)}\right]^2}{\left(\frac{D_m}{ft}\right)^{0.667} \cdot \left(\frac{W_{clearwater}}{ft}\right)^2}\right]^{\frac{3}{7}} \cdot ft = 11.166\,ft$$

HEC-18 Eq. 6.4

$y_{s_clearwater} := y_{2_clearwater} - y_o = 4.506\,ft$

HEC-18 Eq. 6.5 - average contraction scour depth

Contraction Scour in Cohesive Materials, HEC-18, 6.7

$K_{u_ult} := 1.486$

coefficient for ultimate scour, HEC-18, 6.7.1

$\tau_c := 0.05 \cdot \left(\frac{D_{50}}{ft}\right)^{-0.4} \cdot \frac{lb}{ft^2} = 0.222\frac{lb}{ft^2}$

Critical shear stress, from lower bound of HEC-18, Figure 6.9 for mean grain size

$V_2 = 7.28\frac{ft}{s}$

Average velocity of flow through contraction

$y_{1_ult} := y = 8.68\,ft$

Average depth in the upstream main channel

$$y_{s_ult} := 0.94 \cdot \frac{y_{1_ult}}{ft} \cdot \left[\frac{1.83 \cdot \frac{V_2}{\frac{ft}{s}}}{\sqrt{\frac{g}{\frac{ft}{s^2}} \cdot \frac{y_{1_ult}}{ft}}} - \frac{K_{u_ult} \cdot \sqrt{\frac{\left(\frac{\tau_c}{\frac{lb}{ft^2}}\right)}{\frac{\gamma_w}{\frac{lb}{ft^3}}}}}{\frac{g}{\frac{ft}{s^2}} \cdot n \cdot \left(\frac{y_{1_ult}}{ft}\right)^{\frac{1}{3}}} \right] \cdot ft = 6.198\ ft$$

Ultimate scour

Controlling Contraction Scour:

controlling_contraction_scour = "live bed eq"

$$y_{s_general} := \begin{cases} y_{s_livebed} & \text{if controlling_contraction_scour} = \text{"live bed eq"} \\ y_{s_clearwater} & \text{if controlling_contraction_scour} = \text{"clear water eq"} \\ y_{s_ult} & \text{otherwise} \end{cases} = 5.76 \cdot ft$$

Local Scour at Abutment 1 (Waterboro):

Flow is obstructed by both the abutment and the embankments, both in the channel and overbank area. Use conveyance to determine how much flow is considered effective and therefore L`

Froehlich Equation:

$K_1 := 0.55$ — Coefficient for abutment shape, HEC-18, Table 8.1 for spill-through (integral) abutment

$\theta := 114.17deg$ — Orientation of embankment angle to the flow, measured from microstation

$L := 227ft$ — Length of embankment projected normal to the flow, assuming overbank as elev. 320

$L := L \cdot \cos(90deg - \theta) = 207.1\ ft$ — Length adjusted for skew of embankment

$y_a := 5.75ft$ — Average depth of flow on the floodplain, from HEC-RAS model

$Q_e := 1115.95 \frac{ft^3}{s}$ — Flow obstructed by the abutment and approach embankment (right bank), RS: 10252.17BR U

$K_2 := \left(\frac{\theta}{90}\right)^{0.13} = 0.609$ — Coefficient for angle of embankment to flow

$A_e := y_a \cdot L = 1191\,ft^2$ — Flow area of the approach cross section obstructed by the embankment

$V_{tube} := 6.05\,\frac{ft}{s}$ — Velocity in the channel just upstream of the abutment, from HEC-RAS model, RS: 10293.06

$y_{tube} := 8.15ft$ — Water depth just upstream of the abutment, from HEC-RAS model, RS: 10293.06

$q_{tube} := V_{tube} \cdot y_{tube} = 49.308 \cdot \frac{\frac{ft^3}{s}}{ft}$ — See HEC-18 example, pg 8.21

$L' := \frac{Q_e}{q_{tube}} = 22.632\,ft$ — Length of active flow obstructed by the embankment

$V_e := \frac{Q_e}{A_e} = 0.937\,\frac{ft}{s}$ — Velocity of obstructed flow

$Fr := \frac{V_e}{(g \cdot y_a)^{\frac{1}{2}}} = 0.069$ — Froude Number of approach flow upstream of the abutment

$y_{s_A1_froehlich} := \left[\frac{y_a}{ft} \cdot 2.27 K_1 \cdot K_2 \cdot \left(\frac{\frac{L'}{ft}}{\frac{y_a}{ft}} \right)^{0.43} \cdot Fr^{0.61} + 1 \right] \cdot ft = 2.542\,ft$ — Froehlich's Abutment scour equation, HEC-18 Eq 8.1

NCHRP 24-20 Abutment Scour Approach

$y_0 := y_{contract} = 6.66\,ft$ — Flow depth prior to scour

$q_1 := \frac{Q_{up}}{W_{up_top}} = 52.5 \cdot \frac{ft^2}{s}$ — Upstream unit discharge

$q_{2c} := \frac{Q_{total}}{W_{top}} = 56.921 \cdot \frac{ft^2}{s}$ — Unit discharge in the constricted opening, accounting for a non-uniform flow distribution

$y_1 := y = 8.68\,ft$ — Flow depth including live-bed contraction scour

$L := 198.92ft$ — Length of the embankment, measured from HEC-RAS

$B_f := 235.26ft$ — Length of the flood plain, measured from HEC-RAS

$$controlling_contraction_scour_LS := \begin{cases} \text{"live bed"} & \text{if } \frac{L}{B_f} \geq 0.75 \\ \text{"clear water"} & \text{otherwise} \end{cases} = \text{"live bed"}$$

$$y_{c_lb} := y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\frac{6}{7}} = 9.302\ ft$$

HEC-18 Eq. 8.5, Flow depth including live-bed contraction scour

$K_u := 11.17$

HEC-18, pg 8.13

$$y_{c_cw} := \left[\frac{\frac{q_{2c}}{ft^2 \div s}}{K_u \cdot \left(\frac{D_{50}}{ft}\right)^{\frac{1}{3}}}\right]^{\frac{6}{7}} \cdot ft = 11.729\ ft$$

HEC-18 Eq 8.6, flow depth including clear-water contraction scour

$$y_c := \begin{cases} y_{c_lb} & \text{if controlling_contraction_scour_LS} = \text{"live bed"} \\ y_{c_cw} & \text{otherwise} \end{cases} = 9.302 \cdot ft$$

Controlling y_c

$$\frac{q_{2c}}{q_1} = 1.084$$

$\alpha_A := 1.55$

Amplification factor for live-bed conditions, from HEC-18 figures 8.9 - 8.10

$\alpha_B := 2.55$

Amplification factor for clear-water conditions, from HEC-18 figures 8.11 - 8.12

$$\alpha := \begin{cases} \alpha_A & \text{if controlling_contraction_scour_LS} = \text{"live bed"} \\ \alpha_B & \text{otherwise} \end{cases} = 1.55$$

$y_{max} := \alpha \cdot y_c = 14.418\ ft$

HEC-18 Eq 8.3, maximum flow depth resulting from abutment scour

$y_{s_A1_NCHRP} := y_{max} - y_0 = 7.758\ ft$

HEC-18 Eq. 8.4, abutment scour depth

Controlling A1 Scour:

$$y_{s_A1} := \max\left(y_{s_A1_froehlich}, y_{s_A1_NCHRP}\right) = 7.758\ ft$$

Pier Scour for Complex Pier Foundations, from HEC-18 section 7.5

Pier is defined as a Complex Pier Foundation due to having a stem and a footing

General Pier Variables:

$a := 6ft$ — Pier width

$a_{pc} := a + 2ft + 2ft = 10\,ft$ — Pile cap/footing width

$t_{footing} := 3ft$ — Thickness of the pier footing/pile cap

$h_o := 0ft$ — Height of the footing/pile cap above the bed before scour

$y_{1_pier} := y = 8.68\,ft$ — Flow depth directly upstream of the pier

$L_{pier} := 40ft$ — Length of the pier

$\theta_{pier} := 15deg$ — Skew angle of flow

$f := 2ft$ — Distance between front edge of footing/pile cap and pier

$V_1 := 6.55\,\frac{ft}{s}$ — Mean velocity of flow directly upstream of the pier, from HEC-RAS model, RS: 10293.06

$$F_{r1} := \frac{V_1}{\left(g \cdot y_{1_pier}\right)^{0.5}} = 0.392$$

Froude Number directly upstream of pier HEC-18 pg. 7.3

Correction Factors:

nose_shape := "sharp"

$$K_1 := \begin{cases} 1.1 & \text{if nose_shape = "square"} \\ 1.0 & \text{if nose_shape = "round"} \\ 1.0 & \text{if nose_shape = "circular cylinder"} \\ 1.0 & \text{if nose_shape = "group of cylinders"} \\ 0.9 & \text{otherwise} \end{cases} = 0.9$$

Correction factor for pier nose shape, from HEC-18 Figure 7.3 and Table 7.1

$$K_2 := \left(\cos\left(\theta_{pier}\right) + \frac{L_{pier}}{a} \cdot \sin\left(\theta_{pier}\right)\right)^{0.65} = 1.903$$

Correction factor for angle of attack of flow from HEC-18 Table 7.2 or Equation 7.4

$Dunes := \text{"no"}$ — Are there dunes present during flooding?

$H_{dune} := 0ft$ — Height of dunes, if no dunes enter 0

$$K_3 := \begin{cases} 1.1 & \text{if } H_{dune} = 0 \\ 1.1 & \text{if } 2ft \le H_{dune} \le 10ft \\ 1.2 & \text{if } 10ft \le H_{dune} \le 30ft \\ 1.3 & \text{otherwise} \end{cases} \quad = 1.1$$

Correction factor for bed condition from HEC-18 Table 7.3

Pier Stem Scour Depth Component, from HEC-18 section 7.5.3:

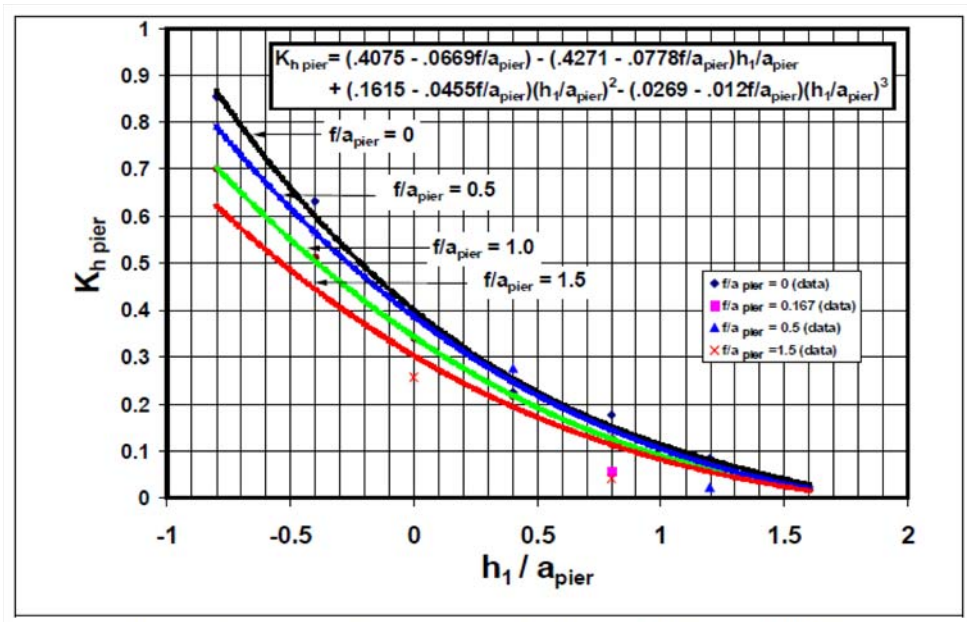


Figure 7.6. Suspended pier scour ratio (Jones and Sheppard 2000).

$h_1 := h_o + t_{footing} = 3\ ft$ — Height of the pier stem above the bed before scour

$\frac{h_1}{a} = 0.5$ — Value to determine K_{h_pier} from HEC-18 Figure 7.5

$\frac{f}{a} = 0.333$ — Value to determine K_{h_pier} from HEC-18 Figure 7.5

$$K_{h_pier} := \left(0.4075 - 0.0669 \cdot \frac{f}{a}\right) - \left(0.4271 - 0.0778 \cdot \frac{f}{a}\right) \cdot \left(\frac{h_1}{a}\right) \ldots + \left(0.1615 - 0.0455 \cdot \frac{f}{a}\right) \cdot \left(\frac{h_1}{a}\right)^2 - \left(0.0269 - 0.012 \cdot \frac{f}{a}\right) \cdot \left(\frac{h_1}{a}\right)^3 \quad = 0.218$$

From HEC-18 Table 7.5

$$y_{s_stem} := y_{1_pier} \cdot K_{h_pier} \cdot \left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_{1_pier}} \right)^{0.65} \cdot F_{r1}^{\ 0.43} \right] = 3.755\ ft$$

Pier Stem Scour Depth Component HEC-18 Eq 7.23

Pier Pile Cap/Footing Scour Depth Component, from HEC-18 section 7.5.4:

There are two cases considered in estimating the scour caused by the pile cap/footing:

Case 1: the bottom of the pile cap/footing is above the bed and in the flow either by design or after the bed has been lowered by scour cased by the pier stem component.

Case 2: the bottom of the pile cap/footing is on or below the bed.

controlling_case := "case 2"

Controlling case from the description above

$$h_2 := h_o + \frac{y_{s_stem}}{2} = 1.877\ ft$$

$$y_{2_pier} := y_{1_pier} + \frac{y_{s_stem}}{2} = 10.557\ ft$$

$$V_2 := V_1 \cdot \left(\frac{y_{1_pier}}{y_{2_pier}} \right) = 5.385\ \frac{ft}{s}$$

Case 1:

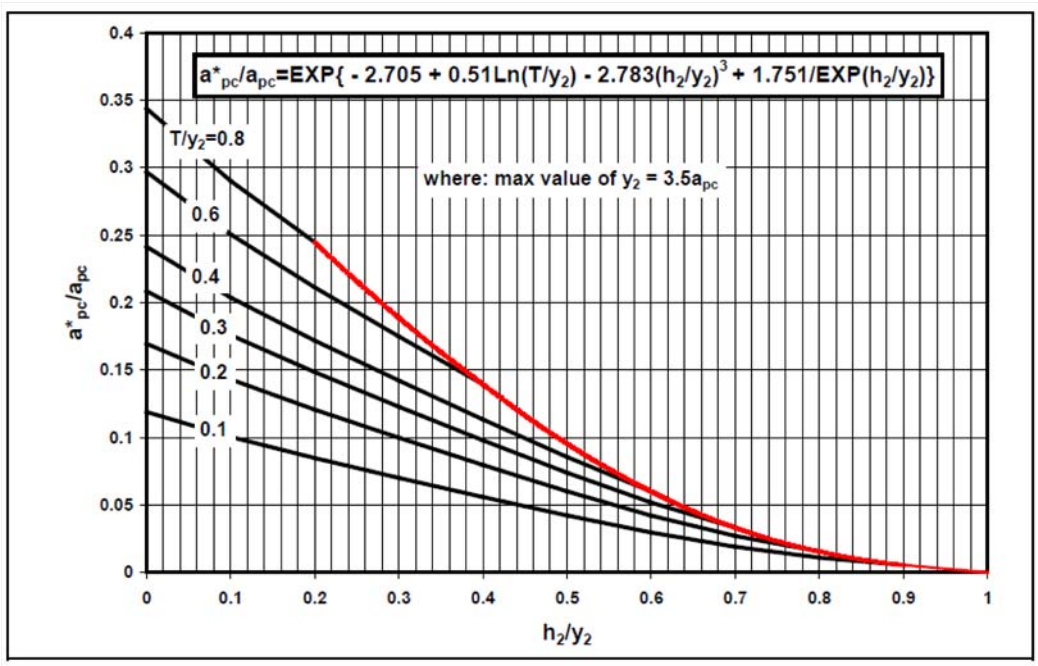


Figure 7.7. Pile cap (footing) equivalent width (Jones and Sheppard 2000).

$$\frac{t_{footing}}{y_{2_pier}} = 0.284$$

$$y_{2_pier} := \min\left(y_{2_pier}, 3.5 \cdot a_{pc}\right) = 10.557\ ft$$

HEC-18 pg 7.15

$$a'_{pc} := a_{pc} \cdot \left[e^{\left[-2.705 + 0.51 \cdot \ln\left(\frac{t_{footing}}{y_{2_pier}}\right) - 2.783 \cdot \left(\frac{h_2}{y_{2_pier}}\right)^3 + \frac{1.751}{\left(e^{\frac{h_2}{y_{2_pier}}}\right)} \right]} \right] = 1.501\ ft$$

HEC-18 Figure 7.7

$$F_{r2} := \frac{V_2}{\left(g \cdot y_{2_pier}\right)^{0.5}} = 0.292 \qquad check_F_{r2} := \frac{V_2}{\left(g \cdot y_{2_pier}\right)^{0.5}} < 1 = 1$$

$$y_{2_pier} \le 0.8 \cdot a'_{pc} = 0 \qquad check_y_{2_pier} := y_{2_pier} \le 0.8 \cdot a'_{pc} = 0$$

$$a'_{pc} \ge 50 \cdot D_{50} = 1 \qquad check_a'_{pc} := a'_{pc} \ge 50 \cdot D_{50} = 1$$

$$check_Kw_necessary := \begin{cases} \text{"yes"} & \text{if } \left(check_F_{r2} + check_y_{2_pier} + check_a'_{pc}\right) = 3 \\ \text{"no"} & \text{otherwise} \end{cases} = \text{"no"}$$

$$K_{w_c1} := \begin{cases} \left[2.58 \cdot \left(\frac{y_{2_pier}}{a'_{pc}}\right)^{0.34} \cdot F_{r2}^{\,0.65}\right] & \text{if } \frac{V}{V_c} < 1 \\ \left[1.0 \cdot \left(\frac{y_{2_pier}}{a'_{pc}}\right)^{0.13} \cdot F_{r2}^{\,0.25}\right] & \text{otherwise} \end{cases} = 0.947$$

HEC-18 equations 7.20 & 7.21

$$K_{w_c1} := \begin{cases} K_{w_c1} & \text{if } check_Kw_necessary = \text{"yes"} \\ 1.0 & \text{otherwise} \end{cases} = 1$$

$$y_{s_pc_c1} := y_{2_pier} \cdot \left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_{w_c1} \cdot \left(\frac{a'_{pc}}{y_{2_pier}}\right)^{0.65} \cdot \left(\frac{V_2}{\sqrt{g \cdot y_{2_pier}}}\right)^{0.43}\right] = 6.594\ ft$$

HEC-18 Eq. 7.24

Case 2:

$$k_s := \begin{cases} 3.5 \cdot D_{84} & \text{if } soil_type = \text{"gravel"} \\ D_{84} & \text{otherwise} \end{cases} = 0.402 \cdot ft$$

Grain roughness of the bed (normally taken as the D_{84} for sand size bed material and 3.5*D_{84} for gravel and coarser bed material

$$y_f := h_1 + \frac{y_{s_stem}}{2} = 4.877\ ft$$

Distance from the bed (after degredation, contractrion scour and pier stem scour) to the top of the footing

$$V_f := V_2 \cdot \left(\frac{\ln\left(10.93 \cdot \frac{y_f}{k_s} + 1\right)}{\ln\left(10.93 \cdot \frac{y_{2_pier}}{k_s} + 1\right)}\right) = 4.655\ \frac{ft}{s}$$

Average velocity in the flow zone below the top of the footing

$$F_{r2} := \frac{V_2}{\left(g \cdot y_{2_pier}\right)^{0.5}} = 0.292 \qquad check_F_{r2} := \frac{V_2}{\left(g \cdot y_{2_pier}\right)^{0.5}} < 1 = 1$$

$$y_{2_pier} \le 0.8 \cdot a_{pc} = 0 \qquad check_y_{2_pier} := y_{2_pier} \le 0.8 \cdot a_{pc} = 0$$

$a_{pc} \geq 50 \cdot D_{50} = 1$ $\qquad$ $check_a_{pc} := a_{pc} \geq 50 \cdot D_{50} = 1$

$$check_Kw_necessary := \begin{cases} \text{"yes"} & \text{if } (check_F_{r2} + check_y_{2_pier} + check_a_{pc}) = 3 \\ \text{"no"} & \text{otherwise} \end{cases} = \text{"no"}$$

$$K_{w_c2} := \begin{cases} \left[2.58 \cdot \left(\frac{y_{2_pier}}{a_{pc}}\right)^{0.34} \cdot F_{r2}^{0.65}\right] & \text{if } \frac{V}{V_c} < 1 \\ \left[1.0 \cdot \left(\frac{y_{2_pier}}{a_{pc}}\right)^{0.13} \cdot F_{r2}^{0.25}\right] & \text{otherwise} \end{cases} = 0.74$$

HEC-18 equations 7.20 & 7.21

$$K_{w_c2} := \begin{cases} K_{w_c2} & \text{if } check_Kw_necessary = \text{"yes"} \\ 1.0 & \text{otherwise} \end{cases} = 1$$

$$y_{s_pc_c2} := y_f \cdot \left[2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot K_{w_c2} \cdot \left(\frac{a_{pc}}{y_f}\right)^{0.65} \cdot \left(\frac{V_f}{\sqrt{g \cdot y_f}}\right)^{0.43}\right] = 19.145\,ft$$

HEC-18 Eq. 7.26

Controlling Pile cap/footing scour:

$$y_{s_pc} := \begin{cases} y_{s_pc_c1} & \text{if } controlling_case = \text{"case 1"} \\ y_{s_pc_c2} & \text{otherwise} \end{cases} = 19.145 \cdot ft$$

Pier Pile Group Scour Depth Component, from HEC-18 section 7.5.5:

There are no piles used in the pier for this project.

$y_{s_pg} := 0ft$

Total Scour for Complex Piers:

$y_{s_P} := y_{s_stem} + y_{s_pc} + y_{s_pg} = 22.9\,ft$ $\qquad$ Total scour for complex pier foundations

Local Scour at Abutment 2 (Limerick):

Flow is obstructed by both the abutment and the embankments, both in the channel and overbank area. Use conveyance to determine how much flow is considered effective and therefore L`

Froehlich Equation:

$K_1 := 0.55$ $\qquad$ Coefficient for abutment shape, HEC-18, Table 8.1 for spill-through (integral) abutment

$\theta := 62.5deg$ $\qquad$ Orientation of embankment angle to the flow, measured from microstation

$L := 62ft$ $\qquad$ Length of embankment projected normal to the flow, assuming overbank as elev. 320

$L := L \cdot \cos(90deg - \theta) = 54.995\,ft$ $\qquad$ Length adjusted for skew of embankment

$y_a := 4.31ft$ $\qquad$ Average depth of flow on the floodplain, from HEC-RAS model, RS: 10252.17BR U

$Q_e := 446.08 \frac{ft^3}{s}$ — Flow obstructed by the abutment and approach embankment (left bank), RS: 10252.17BR U

$K_2 := \left(\frac{\theta}{90}\right)^{0.13} = 0.563$ — Coefficient for angle of embankment to flow

$A_e := y_a \cdot L = 237\, ft^2$ — Flow area of the approach cross section obstructed by the embankment

$V_{tube} := 6.55 \frac{ft}{s}$ — Velocity in the channel just upstream of the abutment, from HEC-RAS model RS: 10293.06

$y_{tube} := 9.58 ft$ — Water depth just upstream of the abutment, from HEC-RAS model, RS: 10293.06

$q_{tube} := V_{tube} \cdot y_{tube} = 62.749 \cdot \frac{\frac{ft^3}{s}}{ft}$ — See HEC-18 example, pg 8.21

$L' := \frac{Q_e}{q_{tube}} = 7.109\, ft$ — Length of active flow obstructed by the embankment

$V_e := \frac{Q_e}{A_e} = 1.882 \frac{ft}{s}$ — Velocity of obstructed flow

$Fr := \frac{V_e}{(g \cdot y_a)^{\frac{1}{2}}} = 0.16$ — Froude Number of approach flow upstream of the abutment

$y_{s_A2_froehlich} := \left[\frac{y_a}{ft} \cdot 2.27 K_1 \cdot K_2 \cdot \left(\frac{\frac{L'}{ft}}{\frac{y_a}{ft}}\right)^{0.43} \cdot Fr^{0.61} + 1\right] \cdot ft = 2.228\, ft$ — Froehlich's Abutment scour equation, HEC-18 Eq 8.1

NCHRP 24-20 Abutment Scour Approach

$y_0 := y_{contract} = 6.66\, ft$ — Flow depth prior to scour

$q_1 := \frac{Q_{up}}{W_{up_top}} = 52.5 \cdot \frac{ft^2}{s}$ — Upstream unit discharge

$q_{2c} := \frac{Q_{total}}{W_{top}} = 56.921 \cdot \frac{ft^2}{s}$ — Unit discharge in the constricted opening, accounting for a non-uniform flow distribution

$y_1 := y = 8.68\, ft$ — Flow depth including live-bed contraction scour

$L := 25.12 ft$ — Length of the embankment, measured from HEC-RAS

$B_f := 25.12 ft$ — Length of the flood plain, measured from HEC-RAS

$$controlling_contraction_scour_LS := \begin{cases} \text{"live bed"} & \text{if } \frac{L}{B_f} \geq 0.75 \\ \text{"clear water"} & \text{otherwise} \end{cases} = \text{"live bed"}$$

$$y_{c_lb} := y_1 \cdot \left(\frac{q_{2c}}{q_1}\right)^{\frac{6}{7}} = 9.302\ ft$$

HEC-18 Eq. 8.5, Flow depth including live-bed contraction scour

$$K_u := 11.17$$

HEC-18, pg 8.13

$$y_{c_cw} := \left[\frac{\frac{q_{2c}}{ft^2 \div s}}{K_u \cdot \left(\frac{D_{50}}{ft}\right)^{\frac{1}{3}}}\right]^{\frac{6}{7}} \cdot ft = 11.729\ ft$$

HEC-18 Eq 8.6, flow depth including clear-water contraction scour

$$y_c := \begin{cases} y_{c_lb} & \text{if } controlling_contraction_scour_LS = \text{"live bed"} \\ y_{c_cw} & \text{otherwise} \end{cases} = 9.302 \cdot ft$$

Controlling y_c

$$\frac{q_{2c}}{q_1} = 1.084$$

$$\alpha_A := 1.55$$

Amplification factor for live-bed conditions, from HEC-18 figures 8.9 - 8.10

$$\alpha_B := 2.55$$

Amplification factor for clear-water conditions, from HEC-18 figures 8.11 - 8.12

$$\alpha := \begin{cases} \alpha_A & \text{if } controlling_contraction_scour_LS = \text{"live bed"} \\ \alpha_B & \text{otherwise} \end{cases} = 1.55$$

$$y_{max} := \alpha \cdot y_c = 14.418\ ft$$

HEC-18 Eq 8.3, maximum flow depth resulting from abutment scour

$$y_{s_A2_NCHRP} := y_{max} - y_0 = 7.758\ ft$$

HEC-18 Eq. 8.4, abutment scour depth

Controlling A2 Scour:

$$y_{s_A2} := \max\left(y_{s_A2_froehlich}, y_{s_A2_NCHRP}\right) = 7.758\ ft$$

Scour Summary:

$y_{s_general} = 5.76\ ft$ General (contractrion) scour

Abutment 1:

$y_{s_A1_froehlich} = 2.542\ ft$ Local scour based on Froehlich Eq.

$y_{s_A1_total} := y_{s_general} + y_{s_A1_froehlich} = 8.302\ ft$ Total general & local scour

$y_{s_A1_NCHRP} = 7.758\ ft$ Total scour based on NCHRP methods

Pier:

$y_{s_P} = 22.9\ ft$ Scour @ Pier

Abutment 2:

$y_{s_A2_froehlich} = 2.228\ ft$ Local scour based on Froehlich Eq.

$y_{s_A2_total} := y_{s_general} + y_{s_A1_froehlich} = 8.302\ ft$ Total general & local scour

$y_{s_A2_NCHRP} = 7.758\ ft$ Total scour based on NCHRP methods