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Field Study of Scour Critical Bridges in Rhode Island

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FIELD STUDY OF SCOUR CRITICAL BRIDGES IN
RHODE ISLAND

BY
WENDY K. LAURENT

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
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OF
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ABSTRACT

All bridges in the United States are required to complete a detailed scour analysis in accordance with the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular Number 18 (HEC-18) standards using predetermined design flood events. Proper scour depth predictions are essential, as over conservative estimates may lead to a bridge being classified as scour critical, leading to a required plan of action and possible costly remediation efforts. Conversely, under conservative estimates could lead to reduced performance of a bridge or even complete failure (Arneson et al. 2012).

In Rhode Island there are 127 bridges that are classified as scour critical, requiring the Rhode Island Department of Transportation (RIDOT) to create a detailed plan of action for each bridge and regularly monitor them. Following historic floods in 2010, an evaluation of the scour critical bridges throughout the state was performed, and this study suggested that the current HEC-18 methodology to evaluate scour is over conservative (AECOM 2013). The experiments used to develop the current scour equations found in HEC-18 do not include detailed site characteristics such as complex hydrology, vegetation, cobbles, or soil cohesion which are all common features in Rhode Island.

The objective of this study is to evaluate the current scour methodology on selected scour critical bridges in Rhode Island. Four bridges were selected: Three riverine bridges and one bridge crossing a tidal inlet. To estimate information such as flow, velocity, and depth, each bridge site must be modeled using the US Army Corps of Engineer's Hydraulic Engineering Center's River Analysis System (HEC-RAS). Conditions from the 2010 flooding event were used to model the riverine bridges and

Hurricane Sandy in 2012 was used to model the marine bridge. Scour at each site was predicted using the equations presented in HEC-18 and compared with both past and present scour observations. Finally, the sensitivity of multiple parameters within the HEC-18 equations were evaluated to create an upper and lower bound for the scour prediction during the event of interest.

To support the modeling effort, a detailed field testing program was conducted. At each bridge, the local bathymetry and topography was obtained using a combination of interferometric sonar, real-time kinematic, and total station surveying. Information about the bed conditions and existing scour features was obtained from analysis of grab samples, side scan sonar images, and CHIRP sub-bottom profiler. From this data, any existing scour features were noted. Minor scour was present at one of the riverine bridges. Prominent scour features were observed at the tidal bridge. Both features observed in this study have occurred in the past and have been slightly in-filled according to documentation.

Cross sections were created from the survey information to accurately model the bridge sites. Steady, one-dimensional HEC-RAS models of the 2010 flooding event were created at the three riverine sites and the associated scour was estimated. For the tidal site, complex unsteady one-dimensional models were created for Hurricane Sandy and the resulting scour was predicted. Predicted and observed values of scour were compared and it was concluded that the scour prediction equations over estimated scour at all four sites where scour analyses were completed.

A sensitivity analysis identified variations in the predicted scour depths, providing an upper and lower limit with regards to grain size, angle of attack, Froehlich's length of active flow, and the Manning friction coefficient. It was determined that the selected median grain size plays a significant role on the prediction of scour, varying the estimates by about five feet. The angle of attack significantly affected pier scour, but had minimal effects on abutment scour. Froehlich's length of active flow had an average change of 2.4 feet on abutment scour at the four bridge sites. Finally, the Manning friction coefficient had the smallest changes with an average change in scour of 1.1 feet. It was easily seen that the scour predictions over-predicted the depth of scour as compared to the observed, indicating that the scour prediction equations over-estimate scour features in Rhode Island.

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-Wendy K. Laurent

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CHAPTER 1

INTRODUCTION

All bridges across the United States are required to complete a detailed scour analysis in accordance to the FHWA's HEC-18 standards using predetermined design flood events. Scour is the erosion of sediment due to water flowing across it. It is dependent on soil parameters, local geometries, and flow parameters. Each bridge must be analyzed using a risk based approach in which the significance of the bridge is analyzed and compared to the economic consequences of failure. It is suggested that when designing for scour, a design flood frequency higher than the 100-year flood frequency should be used. This means that in one year, there is a one percent probability that the 100-year flood would be exceeded.

Scour is a difficult process to study due to its cyclical nature; a scour depression that occurred at the height of a flood may be in-filled with sediment from upstream as flood waters recede. Total scour is comprised of three types of scour: general scour, contraction scour, and local scour. General scour is the natural or man-induced aggradation or degradation of the river bed over time. Contraction scour occurs due to the narrowing of a channel at the bridge. This constriction increases local velocities of the stream, lowering the depth of the bed. Local scour at a bridge is the removal of sediment near the piers and abutments. These structures are obstacles which cause an acceleration of the flow and the formation of erosive vortices. Together, these scour processes combine to reduce the resistance of a bridge against scour.

The current methods for estimating the different types of scour are based on the results of small-scale, idealized laboratory flume experiments with limited field verification (Arneson et al. 2012). For example, field measurements of pier scour are the most abundant, however the pier scour predictions are still over conservative as shown in Figure 1. The experiments do not account for detailed site characteristics such as complex hydrology, vegetation, cobbles, or soil cohesion which are all common aspects of bridges throughout Rhode Island. Proper scour depth predictions are essential as over conservative estimates may lead to a bridge being classified as scour critical, leading to a detailed plan of action to be created and possible remediation efforts to be

Figure 1: Predicted vs. observed pier scour using HEC-18 pier scour equation (Muller and Wagner 2005).

required. A plan of action is a document in which guidance is presented for the scour critical bridge; it lays out a strategy to be implemented before, during, and after a flood incorporating the type and frequency of inspections to be held and a schedule for the design of scour countermeasures. Meanwhile, under conservative estimates could lead to either reduced performance of a bridge or even complete failure (Arneson et al. 2012).

The National Bridge Inspection Standards requires an Item 113 rating, or a scour rating, of zero to nine to describe a bridge. This classification is shown in Table 1. A rating between 0 and 3 qualifies a bridge as being scour critical. If a bridge is listed as

scour critical, then the FHWA requires the bridge owner to provide a plan of action and for additional monitoring of the bridge to occur. (Arneson et al. 2012). Currently, in Rhode Island there are 127 bridges that are classified as scour critical, requiring the Rhode Island Department of Transportation (RIDOT) to create a detailed plan of action and regularly monitor these bridges.

Table 1: Item 113- Scour Critical Bridges (FHWA 2001)

Code	Description
N	Bridge not over waterway.
U	Bridge with "unknown" foundation that has not been evaluated for scour. Until risk can be determined, plan of action should be developed and implemented to reduce the risk to users from a bridge failure during and immediately after a flood event.
T	Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections until an evaluation is performed ("Unknown" foundations in "tidal" waters should be coded U.)
9	Bridge foundations (including piles) on dry land well above flood water elevations.
8	Bridge foundations determined to be stable for the assessed or calculated scour conditions. Scour is determined to be above top of footing by assessment, by calculation or by installation of properly designed countermeasures.
7	Countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a plan of action have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event.
6	Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for assessed or calculated scour conditions. Scour is determined to be within the limits of footing or piles by assessment, by calculations or by installation of properly designed countermeasures.
4	Bridge foundations determined to be stable for assessed or calculated scour conditions; field review indicates action is required to protect exposed foundations.
3	Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions: Scour within limits of footing or piles, scour below spread-footing base or pile tips.
2	Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations, which are determined to be unstable by: A comparison of calculated scour and observed scour during the bridge

	inspection, or an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.
1	Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic. Failure is imminent based on: A comparison of calculated and observed scour during the bridge inspection, or an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.
0	Bridge is scour critical. Bridge has failed and is closed to traffic.

Beginning in February 2010 and extending into April 2010, historic rainfall occurred in the state of Rhode Island. Due to higher than average temperatures the ground was not frozen and the increased rainfall allowed for the grounds across the state to become saturated and for rivers to run at higher than average depths. At the end of March, another large rainfall event hit Rhode Island, dropping six to ten inches of rain on the already saturated state. Rainfall exceeded all historical observations as seen in Figure 2. Subsequently, extreme flooding occurred. A portion of Interstate-95 was closed due to flood waters and the Warwick and Rhode Island Malls were completely inundated by the Pawtuxet River (NWS ND). Twelve of twenty-one United States Geological Survey (USGS) Gauge's recorded this event above a 500-year recurrence interval, with three more classifying the rainfall between a 100 and 500-year event.

Figure 2: Statewide Precipitation in Rhode Island for March 1895-2010 (NOAA N.D.a).

In 2010, the consulting firm AECOM was contracted to evaluate the scour critical bridges throughout the state and create detailed plans of action for the bridges. However, after the 2010 floods occurred, the plans of action were delayed for scour reviews to occur based on field data collected during the 2010 floods. Each bridge's foundation, scour features, and response to the 2010 flood was analyzed. It was easily observed that many bridges that were calculated to be scour critical did not respond as expected to the 2010 flood. This study suggested that the fourth edition (2001) of the FHWA methodology to evaluate scour is over conservative (AECOM 2013). Since the completion of the AECOM study, no detailed field investigations have been conducted in Rhode Island to test the current, fifth edition HEC-18 methodology.

Given the possible over conservative nature of the HEC-18 approach and the large number of bridges in Rhode Island that are classified as scour critical, the objective of

this research is to evaluate the current scour methodology on selected scour critical bridges in Rhode Island. This is accomplished through a detailed field and numerical study of four bridges in Rhode Island: Kenyons Bridge (RI Bridge No. 020601) in Charlestown, the First Barberville Bridge (RI Bridge No. 040101) in Hopkinton, the Esmond Street Bridge (RI Bridge No. 094801) in Smithfield and Weekapaug Bridge (RI Bridge No. 099701) in Westerly. The bridge's locations can be seen in Figure 3. A fifth bridge site, Nannaquaket Bridge (RI Bridge No. 012601) in Tiverton, was investigated however, a detailed model could not be created due to incomplete storm information at this site making modeling efforts outside of the scope of this study and a scour analysis infeasible. The field results of Nannaquaket Bridge will be presented in Appendix A. Three of the modeled bridges are fresh water sites over rivers while the other modeled bridge is a marine site and is tidally influenced. At each bridge, the local bathymetry and topography was obtained using a combination of interferometric sonar, real-time kinematic, and total station surveying. Information about the bed conditions and existing scour features was obtained from analysis of grab samples, side scan sonar images, and CHIRP sub-bottom profiler.

Figure 3: Location of five selected scour critical bridge sites in RI.

In order to estimate information such as flow, velocity, and depth, each bridge site was modeled using the U.S. Army Corps of Engineer's HEC-RAS. Conditions from the 2010 flooding event were used to model the riverine bridges and Hurricane Sandy in 2012 was used to model the marine bridge. Input flow conditions for the riverine bridges were obtained from the USGS stream gauges and translated to the bridge while Marissa Torres's ADvanced CIRCulation (ADCIRC) hydrodynamic model of the study area was used to simulate water levels at the marine sites. Scour at each site was predicted using the equations presented in HEC-18 and compared with both past and present scour observations. The sensitivity of multiple parameters within the HEC-18 equations is also evaluated upon the completion of modeling efforts.

This thesis is organized in nine chapters. A literature review is presented in Chapter two on the fundamentals of scour, the HEC-18 methodology, and published studies that are relevant to this study. In Chapter three, the methods used in the study are described in detail including all equipment and programs used. Chapters four through seven present the analysis and results for each bridge site, including the bridge history, survey methods, largest flooding event, scour predictions, and the observed scour. Chapter eight presents the results of a sensitivity analysis of the scour predictions for the four selected bridges to understand the uncertainty in the results. Finally, Chapter nine summarizes the results and makes recommendations for further studies.

CHAPTER 2

REVIEW OF LITERATURE

The literature review will first review fundamental concepts regarding scour. Next, the equations used in this study will be presented. Finally, any similar studies that have been completed will be presented in this section.

2.1 SCOUR BACKGROUND

Scour is the erosion of soil as water flows across it. Sediment will naturally erode or accrete over time; this is known as general scour. General scour is bed aggradation or degradation which can occur in the short-term, due to a single event, or in the long-term (Rocker et al. 2011). Contraction scour is similar to general scour but is due to the contraction of a channel, decreasing the flow area, and thus increasing the local velocity. Local scour occurs as water moves around an obstacle, such as a bridge abutment, a bridge pier, a spur, or an embankment. As the water flows, it must change its path, increasing turbidity, either eroding or accreting sediments near the obstacle. If a large amount of erosion occurs in the proximity of the structure, then its' stability could be diminished. Total scour is defined as the combination of these scour phenomenon: General scour, contraction scour, and local scour, which includes pier and abutment scour (Arneson et al. 2012).

The rate of scour is highly dependent on the bed material. For example, a loose sand will scour faster than a cohesive clay or a well vegetated gravel bottom (Arneson et al.

2012). The threshold of erosion is the critical shear stress or critical velocity of the water at which erosion begins to occur. This threshold is strongly dependent on the soil properties. For coarse grained soils such as sands, the relationship can be described by the mean grain size of the particles. However for fine grained soils, such as silts and clays, no simple relationship exists between the mean grain size and the critical shear stress or critical velocity as depicted by Figure 4. This scattered data suggests that erosion is not simply controlled by grain size. Various correlations, such as undrained shear strength and plasticity index, have been investigated to find agreement but none have been successful, indicating that erosion is a phenomena based on multiple soil parameters (Rocker et al. 2011).

Figure 4: Erosion threshold as a function of the critical velocity or critical shear stress versus the mean grain size (Rocker et al. 2011).

There are two sediment-velocity relationships under which scour occurs: Clear-water or live-bed. When the velocity of the water exceeds the critical velocity, the soil particles will become suspended, and begin to erode. If the approach velocity is less than the critical velocity but the local velocity at an obstacle is greater than the critical velocity, then clear-water scour will begin to occur. During clear-water scour conditions, the soil particles that are suspended in the water column are very few to

none. Live-bed scour occurs when both the approach velocity and the local velocity are greater than the critical velocity and soil particles are suspended throughout the water column. It has been found that live-bed scour conditions typically produce smaller scour holes as sediment falls out of the water column and deposits as the flow progresses (Rocker et al. 2011).

The effects of scour must be taken into account when designing any piece of infrastructure located on a channel-bed or sea-bed; it should then continue to be analyzed throughout the lifetime of the structure. There have been a multitude of methods to predict the depth of scour. The majority of the equations used to predict scour are based on laboratory flume experiments producing idealized approximations for which there have been few field verifications. It is difficult to study scour in the field. A scour hole that is produced during a flood event may be in-filled with sediment shortly after, making it unnoticeable when waters recede. To complete an in-depth study of scour in the field, detailed sub-surface investigations must take place (Arneson et al. 2012).

2.2 HYDRAULIC ENGINEERING CIRCULAR NO. 18 (HEC-18) SCOUR EQUATIONS

The fifth edition of HEC-18 was released in April, 2012. During this time, the consulting firm AECOM was completing their scour analysis from the 2010 floods on Rhode Island's scour critical bridges (see Chapter 1), but did not re-evaluate the bridges to reflect the changes published in the fifth edition of HEC-18. The AECOM report stated that "the procedures outlined in HEC-18 are very conservative." AECOM

indicated that some of the bridges that were listed as scour critical did not respond to the floods as predicted, and a list of these scour critical bridges was suggested for scour recalculation using the fifth edition of HEC-18 (AECOM 2013). Changes between the fourth and fifth edition of HEC-18 that relate to the prediction of scour include an improved discussion of scour at tidal bridges to reflect material from Highways in the Coastal Environment (HEC-25), approaches to determine contraction and pier scour in cohesive soils, a revised abutment scour section, and novel approaches for calculating pier scour which includes the estimation of pier scour in coarse materials (FHWA Hydraulics 2012).

The HEC-18 scour prediction equations will be presented in this section. Prediction equations will be organized into three sections based on their driving mechanism. First the contraction scour equations will be presented, followed by pier scour, then abutment scour. Any revised or novel approaches presented in the fifth edition of HEC-18 will be noted with each equation.

2.2.1 CONTRACTION SCOUR

Contraction scour occurs due to a reduction in the flow area. This contraction can be due to either the channel area naturally decreasing or a bridge reducing the size of the channel. The principle of contraction scour is based on continuity, or the conservation of sediment. In the problem, the channel width is constrained and depth increases until the live-bed or clear-water limiting conditions are met. Live-bed contraction scour occurs when sediment from upstream is being transported to the bridge. The depth beneath the bridge will continue to increase until equilibrium is

reached and the sediment transported into the bridge area is equal to the sediment transported out. There are two cases of clear-water contraction scour; the first is where no sediment transport occurs from upstream, or the small amounts of material being transported stay suspended in the flow. During the process of contraction scour, scour will continue to occur until the velocity of the flow decreases to the critical velocity or likewise, the shear stress becomes less than the critical shear stress of the bed materials allowing for no additional sediment transport to occur (Arneson et al. 2012).

The first step in predicting contraction scour is calculating if the upstream will be under live-bed or clear-water conditions. This approach is completed by calculating the critical velocity (V_c) at which the grain size of the bed material (D) will be suspended in the upstream reach (Equation 1).

Equation 1: Critical Velocity (Arneson et al. 2012).

$$V_c = K_u y^{\frac{1}{6}} D^{\frac{1}{3}}$$

where:

V_c = Critical velocity (ft/s);

K_u = 11.17 (English units);

y = Average depth of flow upstream of the bridge (ft); and

D = Particle size for V_c (ft).

To find the live-bed or clear-water conditions the D_{50} , or the median grain size, of approximately the upper foot of bed material upstream of the bridge is used. If the critical velocity of the bed material is less than the upstream velocity, then live-bed conditions will be present. If the critical velocity of the bed material is greater than the upstream velocity, then clear-water conditions will apply. Some limitations to this methodology include if either cohesive materials are present or if the bed is armored.

When live-bed contraction scour conditions are met and there is armoring of bed material or the sediment is transported into the bridges reach, both live-bed contraction scour and clear-water contraction scour should be calculated and the smaller of the two should be taken.

For live-bed scour over a long contraction, Laursen created an equation that solved for the depth of flow in the upstream and contracted areas through the principle of continuity for discharge and sediment transport equations (Laursen 1960). A modified version of Laursen's 1960 equation is suggested in HEC-18:

Equation 2: Live-bed Contraction Scour (Arneson et al. 2012).

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1} \right)^{\frac{6}{7}} \left(\frac{W_1}{W_2} \right)^{k_1}$$

$$y_s = y_2 - y_o$$

where:

y_2 = Average depth in the contracted section (ft);

y_1 = Average depth upstream (ft);

Q_2 = Flow in the contracted channel (ft³/s);

Q_1 = Flow upstream (ft³/s);

W_1 = Bottom width of the upstream channel (ft);

W_2 = Bottom width of the main channel (ft);

k_1 = Exponent based on mode of bed material transport;

y_s = Average scour depth (ft); and

y_o = Average starting depth of contracted channel (ft).

This approach assumes that sediment transport is occurring upstream of the bridge. If the bottom widths of the channel are not well defined, then the top width of the

channel can be used. In addition, the width of the piers must be subtracted from the channel width at the bridge. It is mentioned in the literature that Laursen's equation may overestimate scour if either the bridge is at the upstream end of a natural contraction or if the contraction is from the bridge itself (Arneson et al. 2012).

Laursen suggested the development of a clear-water scour equation in 1963, leading to the current clear-water scour equation. Equation 3 is based on Shields relationship for critical shear for non-cohesive materials which are well correlated.

Equation 3: Clear-water Contraction Scour (Arneson et al. 2012).

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7}$$

$$y_s = y_2 - y_o$$

where:

y_2 = Average depth in the contracted section after scour (ft);

K_u = 0.0077 (English units);

Q = Discharge through the bridge (ft³/s);

D_m = Diameter of the smallest non-transportable particle in the contracted section or $1.25D_{50}$ (ft);

W = Bottom width of the contracted section (ft);

y_s = Average scour depth (ft); and

y_o = Average starting depth of contracted channel (ft).

With this equation, a lower limit of 0.2mm is applied to D_{50} due to the effects of cohesion. If the bed material is stratified, then the equation can be used with successive D_m of the bed materials (Arneson et al. 2012). A comparison of this equation to

laboratory measurements suggests that this is a suitable method, producing comparable results (Laursen 1963).

For cohesive materials, material testing must occur as scour in fines is a slower processes and depends on soil properties, as the critical velocity is not easily defined by the median grain size in cohesive soils (Briaud et al. 2011). From the necessary testing, the critical shear can be determined in addition to the relation between the erosion rate and shear. When the shear stress decreases to the critical shear, scour has reached equilibrium and the process concludes (Arneson et al. 2012). Li and Oh of Texas A&M found that the normalized contraction scour in cohesive sediment is independent of the contraction length and shape, but is instead linearly dependent on differences in the Froude number indicating contraction scour is velocity dependent (Briaud et al. 2011). This concept was taken, allowing for the ultimate contraction scour of a cohesive material to be expressed by the SRICOS-EFA method developed by Briaud and colleagues (2011),

Equation 4: Ultimate Contraction Scour for Cohesive Soils (Arneson et al. 2012).

$$y_{s-ult} = 0.94y_1 \left(\frac{1.83V_2}{\sqrt{gy_1}} \frac{K_u \sqrt{\frac{\tau_c}{\rho_w}}}{gny_1^{1/3}} \right)$$

where:

y_1 = Upstream average flow depth (ft);

V_2 = Average flow velocity in the contracted section (ft/s);

K_u = 1.486 (English units);

τ_c = Critical shear stress (lb/ft²); and

n = Manning n.

This method is based on laboratory data completed at Texas A&M University. In the equation, it is assumed that the depth of the upstream flow is equal to the depth in the constricted area. The equation predicts the scour at the centerline, downstream of the contraction entrance and the scour seen at the entrance can be up to 35 percent greater than that downstream. This equation was not published in the fourth edition of HEC-18 (Arneson et al. 2012).

2.2.2 PIER SCOUR

Pier scour occurs due to flow being interrupted by an obstacle (i.e. the pier), leading to scour around the feature. The intensity of the scour is dependent on bed characteristics, flow pattern, and the pier/footing configuration. Although studied greatly in the laboratory, not much field data has been collected with regard to this type of scour. In addition, most of the laboratory studies are comprised of simple piers with no variation in variables such as shape, depth, or the angle of attack (Arneson et al. 2012).

Based on the CSU equation, the HEC-18 equation is suggested for both live-bed and clear-water pier scour predictions,

Equation 5: HEC-18 Pier Scour (Arneson et al. 2012).

$$\frac{y_s}{y_1} = 2 K_1 K_2 K_3 \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43}$$

where:

y_s = Depth of pier scour (ft);

y_1 = Upstream flow depth (ft);

K_1 = Pier nose shape correction factor;

K_2 = Flow angle of attack correction factor;

K_3 = Bed condition correction factor;

a = Pier width (ft);

Fr_1 = Froude number directly upstream of the pier, $Fr = \frac{V_1}{(gy_1)^{\frac{1}{2}}}$;

V_1 = Mean upstream velocity (ft/s); and

g = Acceleration due to gravity (ft/s²).

Although designed for more simplistic pier configurations, the equation is easily adapted for more complex scenarios that are discussed in alternate sections of the HEC-18 manual. It is suggested by the manual that the depth of scour is to remain below three times the width of the pier for round piers. If the angle of attack is greater than five degrees, then K_1 should be set equal to one as K_2 will govern. However, K_2 should only be used when the entirety of the pier is subjected to the flow's angle of attack. If K_2 is improperly used, then over prediction of pier scour will occur (Arneson et al. 2012). While it rarely under predicts, the equation was found to often over predict the intensity scour (Briaud et al. 2011). In prior editions of HEC-18, bed material size was included, however the pier scour in coarse bed material equation was added in place of a fourth correction factor for armored beds. Improvements to this equation would be the inclusion of bed material size and increased consideration of the flow field at the bridge (Arneson et al. 2012).

The Florida Department of Transportation (FDOT) created methodology to predict pier scour based on results found in a multiple National Cooperative Highway Research Programs (NCHRP). This equation is novel to the HEC-18 publication. The

methodology combines the concepts of an adapted Sheppard and Miller equation with the NCHRP's equation and is presented in Equation 6 (Arneson et al. 2012).

Equation 6: Florida DOT Pier Scour (Arneson et al. 2012).

$$\frac{y_s}{a^*} = 2.5 f_1 f_2 f_3 \quad \text{for } 0.4 \leq \frac{V_1}{V_c} < 1.0$$

$$\frac{y_s}{a^*} = f_1 \left[2.2 \left(\frac{\frac{V_1}{V_c} - 1}{\frac{V_{lp}}{V_c} - 1} \right) + 2.5 f_3 \left(\frac{\frac{V_{lp}}{V_c} - \frac{V_1}{V_c}}{\frac{V_{lp}}{V_c} - 1} \right) \right] \quad \text{for } 1.0 \leq \frac{V_1}{V_c} \leq \frac{V_{lp}}{V_c}$$

$$\frac{y_s}{a^*} = 2.2 f_1 \quad \text{for } \frac{V_1}{V_c} > \frac{V_{lp}}{V_c}$$

where:

y_s = Depth of pier scour (ft);

a^* = Effective pier width, $a^* = K_{sf} a_{proj}$ (ft);

$K_{sf} = 1.0$ for circular or round nosed piers;

$K_{sf} = 0.86 + 0.97 \left(\left| \frac{\pi\theta}{180} - \frac{\pi}{4} \right| \right)^4$ for square nosed piers;

a_{proj} = Projected pier width in direction of the flow,

$$a_{proj} = a \cos \theta + L \sin \theta \text{ (ft);}$$

θ = Angle of attack of flow (degrees);

a = Pier width (ft);

L = Pier length (ft);

$$f_1 = \tanh \left[\left(\frac{y_1}{a^*} \right)^{0.4} \right];$$

$$f_2 = \left\{ 1 - 1.2 \left[\ln \left(\frac{V_1}{V_c} \right) \right]^2 \right\};$$

$$f_3 = \left[\frac{\left(\frac{a^*}{D_{50}} \right)^{1.13}}{10.6 + 0.4 \left(\frac{a^*}{D_{50}} \right)^{1.33}} \right];$$

V_1 = Mean upstream velocity (ft/s);

V_c = Critical velocity, $V_c = 5.75 u_c^* \log \left(5.53 \frac{y_1}{D_{50}} \right)$ (ft/s);

$u_c^* = K_u (0.0377 + 0.041 D_{50}^{1.4})$ for $0.1mm < D_{50} < 1mm$;

$u_c^* = K_u \left(0.1 D_{50}^{0.5} - \frac{0.0213}{D_{50}} \right)$ for $1mm < D_{50} < 100mm$;

$K_u = 1.0$ (English units);

V_{lp} = Live-bed peak scour velocity,

$V_{lp} = 5V_c$ OR $0.6\sqrt{gy_1}$ (whichever is greater, ft/s); and

D_{50} = Median bed particle size (ft).

Detailed step by step instructions can be found in HEC-18 or on the FDOT website. Having a wide range of applications, the FDOT equation is best suited for wide piers in fine bed materials with shallow flows (Arneson et al. 2012).

A new pier scour equation for pier scour in coarse bed materials was created in place of a correction factor for coarse-bed armoring (Equation 7).

Equation 7: Pier Scour in Coarse Bed Materials (Arneson et al. 2012).

$$y_s = 1.1 K_1 K_2 a^{0.62} y_1^{0.38} \tanh \left(\frac{H^2}{1.97 \sigma^{1.5}} \right)$$

where:

y_s = Depth of pier scour (ft);

K_1 = Pier nose shape correction factor;

K_2 = Flow angle of attack correction factor;

a = Pier width (ft);

y_1 = Upstream flow depth (ft);

H = Densimetric particle Froude number, $H = \frac{V_1}{\sqrt{g(S_g - 1)D_{50}}}$;

S_g = Sediment specific gravity;

g = Acceleration due to gravity (ft/s^2);

D_{50} = Median grain size of the bed material (ft); and

σ = Sediment gradation coefficient, $\sigma = \frac{D_{84}}{D_{50}}$.

This equation is applicable for clear-water conditions with an approach velocity less than the critical velocity. It is suggested for median grain sizes greater than 20mm and sediment gradation greater than 1.5 (Arneson et al. 2012). Based on both laboratory and field data, this equation was refined to reduce over prediction of scour while yet, remain conservative (Guo et al. 2012).

Pier scour in cohesive materials is dependent on the soil properties and progresses at slower rates. Like contraction scour in cohesive materials, the critical velocity and shear stress must be found through material testing in addition to the erosion rate (Arneson et al. 2012). This methodology is based on laboratory data completed at Texas A&M University by Gudavalli and Briaud. Gudavalli ran 43 flume experiments on two sands and three clays, while Briaud and coworkers completed flume experiments on more complex piers in clay (Briaud et al. 2011). Combining these studies, Briaud and his colleagues presented Equation 8,

Equation 8: Pier Scour in Cohesive Materials (Arneson et al. 2012).

$$y_s = 2.2 K_1 K_2 a^{0.65} \left(\frac{2.6V_1 - V_c}{\sqrt{g}} \right)^{0.7}$$

where:

y_s = Depth of pier scour (ft);

K_1 = Pier nose shape correction factor;

K_2 = Flow angle of attack correction factor;

a = Pier width (ft);

V_1 = Mean upstream velocity (ft/s);

V_c = Critical velocity (ft/s); and

g = Acceleration due to gravity (ft/s²).

This equation predicts the maximum scour that the pier would sustain over a long period of time. For cohesive materials scour is time-dependent and should be taken into account during design (Arneson et al. 2012).

2.2.3 ABUTMENT SCOUR

Abutment scour occurs due to the obstruction of flow by an abutment and roadway embankment. Multiple instances of abutment failure have been documented as the abutments are the most vulnerable to damage. Abutment scour occurs due to the formation of a vortices at the upstream end of the abutment. The vortices travel along the length of the abutment due to the obstructed flow leading to erosion at the toe. This process depends on the flow through the main channel, the obstructed flow, the abutment geometry and alignment, and the bed material properties (Arneson et al. 2012).

Froehlich's abutment scour equation is based on 170 laboratory flume experiments with live-bed conditions but can be used for both live-bed and clear-water conditions (Arneson et al. 2012).

Equation 9: Froehlich's Abutment Scour Equation (Arneson et al. 2012).

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{L'}{y_a} \right)^{0.43} Fr^{0.61} + 1$$

where:

y_s = Depth of abutment scour (ft);

y_a = Average depth of flow on the floodplain (ft);

K_1 = Abutment shape coefficient;

K_2 = Angle of embankment to the flow coefficient, $K_2 = \left(\frac{\theta}{90}\right)^{0.13}$;

$\theta < 90$ if embankment points downstream;

$\theta > 90$ if embankment points upstream;

L' = Length of active flow obstructed by the embankment (ft);

Fr = Froude number of upstream flow, $Fr = \frac{V_e}{(g y_a)^{1/2}}$;

$V_e = \frac{Q_e}{A_e}$ (ft/s);

Q_e = Flow obstructed by the abutment (ft³/s); and

A_e = Approach cross section flow area obstructed by embankment (ft²).

Froehlich performed a regression analysis on his laboratory experiments to derive Equation 9. The length of active flow blocked by an embankment, L' , is often difficult to estimate, and a MATLAB script that was used to estimate this term can be found in Appendix B. It must be found by an analysis of conveyance tubes versus distance at a cross section upstream of the bridge abutment. Sometimes, this value may be estimated to be zero, resulting a depth of scour of zero; for this reason, the addition of one was added to modify this equation (Arneson et al. 2012)

Based on field data from Mississippi River spurs, the HIRE equation is:

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} K_2$$

where:

y_s = Depth of abutment scour (ft);

y_1 = Depth of flow at the abutment (ft);

Fr = Froude number of upstream flow, $Fr = \frac{V_e}{(g y_1)^{1/2}}$;

$V_e = \frac{Q_e}{A_e}$ (ft/s);

Q_e = Flow obstructed by the abutment (ft³/s);

A_e = Approach cross section flow area obstructed by embankment (ft²);

K_1 = Abutment shape coefficient; and

K_2 = Skew angle of the abutment to the flow coefficient.

This version of the equation was modified from E.V. Richardson and his colleague's studies on the Army Corps of Engineers data to include a correction factor for the skew angle of the abutment (Richardson et al. 2001). Although developed from scour data on Mississippi rock dikes, the scour is related to length, comparable to laboratory studies in which the scour is related to abutment length. These in-situ measurements of scour allow for a more accurate extrapolation of the experimental data (Richardson et al. 1990). The HIRE equation is best suited for situations where the ratio of the projected abutment length to the flow depth is greater than 25 (Arneson et al. 2012).

New to the fifth edition, the NCHRP 24-20 abutment scour approach was derived beginning with the influences of non-uniform flow due to contraction and applying a factor to account for the increased turbulence developed at the abutment. This equation was based on laboratory data and verified by field observations (Ettema et al. 2010). When the abutment is in proximity or set back from the main channel, as in many cases,

Equation 10: NCHRP 24-20 Abutment Scour (Arneson et al. 2012).

$$y_s = y_{max} - y_o$$

$$y_{max} = \alpha_A y_c \quad \text{OR} \quad y_{max} = \alpha_B y_c$$

where:

y_s = Depth of abutment scour (ft);

$y_{\max}$ = Maximum flow depth due to scour (ft);

y_o = Original flow depth (ft);

α_A = Live-bed amplification factor;

y_c = Flow depth with the inclusion of contraction scour (ft);

$$y_c = y_1 \left(\frac{q_{2c}}{q_1} \right)^{\frac{6}{7}} \quad (\text{live-bed});$$

y_1 = Upstream flow depth (ft);

q_{2c} = Unit discharge at the bridge reach (ft²/s);

q_1 = Unit discharge upstream (ft²/s);

$$y_c = \left(\frac{q_{2f}}{K_u D_{50}^{\frac{3}{2}}} \right)^{\frac{6}{7}} \quad \text{OR} \quad y_c = \left(\frac{\gamma}{\tau_c} \right)^{\frac{3}{7}} \left(\frac{n q_{2f}}{K_u} \right)^{\frac{6}{7}} \quad (\text{clear-water});$$

q_{2f} = Unit discharge at the bridge reach (ft²/s);

K_u = 11.17 (English units);

D_{50} = Median grain size (ft);

γ = Unit weight of water (lb/ft³);

τ_c = Critical shear stress of the floodplain sediment (lb/ft²);

n = Manning n value of the bed; and

α_B = Clear-water amplification factor.

This method is deemed advantageous due to not having to determine the effective embankment length, which is not clearly defined at many sites. In addition, the calculations include the effects of contraction scour in the analysis. Uncertainty may arise due to calculating the unit discharge at the abutment and due to determining the

grain size or critical shear stress of the floodplain sediments. This method is not applicable when the embankment is breached and the abutment is essentially a pier (Arneson et al. 2012).

2.3 SIMILAR STUDIES

The US FHWA published a report entitled “Field Observations and Evaluations of Streambed Scour at Bridges” in 2005 suggesting methodology for scour analysis and prediction at bridges. As previously mentioned, the majority of the current scour equations are based on laboratory flume experiments. In the laboratory, measurements are easily obtained and parameters can be selected and adjusted; however, in the field accurate measurements are not easily obtained and conditions are not idealized. Within the report, the scour data base is discussed. The data base has valuable information in regards to scour, but is incomplete as not all of the factors that contribute to scour are entirely incorporated. Complete, reliable detailed data sets are not only sought out by researchers trying to develop new methodology for predicting scour, but also desired to further develop and complete the data base as conditions vary over geomorphic regions. A wide range of conditions are seen in the field; however, no equations have been able to accurately predict the depth of scour over this large array (Mueller and Wagner 2005).

Similar studies to this study have been completed by Maine and South Carolina. Both of which completed field investigations and scour predictions, finding that theoretical equations are not suitable for estimating scour as they under and over predict scour hole depths.

The study performed by the USGS in Maine, published in 2008, compared field data to scour predictions on bridge abutments at 50 bridge sites. Field data was collected both on land and by boat. A theodolite was used to find local elevations, bridge geometry, and cross sections both upstream and downstream from the bridge site. Ground penetrating radar (GPR) was used to detect the depth of in-filled scour holes and checked with the resistance of a metal rod indicating a change in bed materials. Bed material properties were found with grid sampling of 100 samples at the bridge site. Scour was then predicted using the HEC-18 Froehlich/Hire, Sturm, and Maryland methodologies in addition to the Melville method and envelope curves. No correlations were found between their observed and calculated scour depths. The equations both under predicted and over predicted the scour holes sometimes by one to two orders of magnitude (Lombard and Hodgkins 2008).

The USGS and South Carolina DOT conducted a scour study from 1996-1999, investigating 144 bridges throughout the state using theoretical equations, hydraulic models, and field data collection. Field data consisted of a complete site description including photographs and sketches, material samples from the bed in both in and out of scoured areas, in addition to measurements of scour depths and the approximate infill of scour holes. The methodology behind their field study consisted of a fathometer to locate scour holes, a total station survey to complete scour contour plots, and a range pole to penetrate and estimate the sediment infill of scour holes. Sediment samples were taken with a drive-tube sampler. The study concluded that the HEC-18 theoretical equations often exceeded the observed scour depths. They found that controlling variables in the lab such as the flow duration and depth, the sediment gradation and size,

the abutment shape and skew, and the channel geometry have little influence on scour in the field. With further investigation, it was found that variables such as the embankment length, the geometric contraction ratio, the flow velocity, and soil cohesion have a greater influence on the depth of scour in the field. From this data, the South Carolina team was able to create envelope curves using the geometric contraction ratio and embankment length to better describe scour in South Carolina providing reasonable ranges for the depth of scour at a bridge site. The data and findings of this study have been compiled and are available on an online scour data base, however the findings are specific to that regional geomorphic area (Benedict 2003).

An additional study was completed in 2006 by the USGS and FHWA in South Carolina with the data located in the scour data base. This study further investigated the correlation of field evidence at the 144 sites to the HEC-18 equations, specifically the modified Froehlich, Strum, Maryland, HIRE, and Young equations for the flood conditions of a 100 year flood event from historic records or hydraulic models. It was found that the equations often over predict, but sometimes under predict the depth of scour and are not reasonable for estimation purposes within the state of South Carolina (Benedict et al. 2006).

Specifically, in Rhode Island, AECOM was contracted to create the scour plan of action for 116 scour critical bridges in Rhode Island which included a summary of the scour characteristics and actions to be taken before, during, and after a flooding event. These plan of actions were put on hold due to the significant flooding that occurred during March 2010, exceeding the 500-year flood in some areas, causing for a reevaluation to be performed based on the field data collected after the floods. Many

bridges responded differently than expected during the March 2010 flood event, and AECOM suggested this could be due to a few different reasons: HEC-18's procedures being over conservative especially for abutment scour, cohesive soils, and areas with vegetation, equations from the previous analysis not being updated with the most recent version of HEC-18, hydraulic complexities and real life conditions, and other countermeasures such as vegetation, cobbles, and other bed materials not considered in HEC-18 procedures. Bridges with high performance, exceeding the hydraulic design criteria, during the 2010 floods were recommended to be taken off of the scour critical list (AECOM 2013).

CHAPTER 3

METHODOLOGY

This chapter describes the methodology used for this study and is divided into sections based on data collection and processing. First, a brief outline of the complete scour analysis process will be presented in this introduction. The the data collection and processing methodology will then be described. This section includes a number of sub sections describing how data collection took place, the equipment used, and the programs used to process the data.

With a desired outcome of comparing the predicted scour during the bridge's largest historical flood to the largest recorded scour at the bridge, a detailed method was created. It includes detailed data collection, data processing, and flow modeling. First a desktop study of each bridge was performed. Each study looked into the bridge design, past bridge inspections, the bridge's plan of action, and scour summary documents all of these significant documents were provided by RIDOT. Next, a field survey of each bridge was completed. These surveys included pictures to describe bottom conditions, soil samples, topography of a site, bathymetry of the river or inlet bottom, and sub-bottom profiles. Once this data was collected and processed, it was modeled in the HEC-RAS to find the corresponding water levels and flow conditions at the bridge during the flood of interest. The information from the bridge's design documents, surveys, and model were then used to complete the scour prediction equations presented in HEC-18.

These predictions were compared to both the historical scour and the current scour recorded at each bridge.

3.1 DATA COLLECTION AND PROCESSING

To properly collect information about bed conditions, topography, and bathymetry, a variety of equipment and methods were used. At the tidal sites a pontoon boat, shown in Figure 5, was used to collect field data. The riverine sites, however, were very shallow and data was collected while wading in the river or through the use of a kayak. Background information regarding the systems used and the procedures will be discussed in this section. The specific methods used for each bridge will be discussed in the analysis section due to variations in site conditions which determined what methods were employed.

Figure 5: Pontoon boat setup at Weekapaug in Westerly, RI.

3.1.1 BED CONDITIONS

In order to characterize the bed conditions at each site, pictures and sediment samples were collected. A Nikon Coolpix AW100 waterproof camera was used to collect pictures of bottom conditions at each shallow water bridge site. Pictures were

taken upstream, at the bridges reach, and downstream of the bridge documenting the bed material and vegetation.

To collect images of the bottom conditions at sites where wading was not permitted, an EdgeTech 6205 bathymetry and side scan sonar system operated by the URI Coastal Mapping Laboratory was used, as shown in Figure 7. This specific system runs at a frequency of 550 kHz and 1600 kHz; the side scan images are collected at both the high and low frequency (EdgeTech 2015). Side scan sonar transmits sound energy through the water column; this energy is then reflected and returned to the transducers. The intensity of the return signal is used to create an image of the channel bed. Strong intensity return signals indicate a hard object such as an object jutting from the sea floor and are represented by light areas, while the softer return signals appear darker. Shadows are cast in areas where there is little to no return, depicted by Figure 7 (NOAA 2016). The side scan images were processed in the software CleanSweep by Oceanic Imaging Consultants.

Figure 7: Edgetech 6205 bathymetry and side scan sonar bow mounted on the pontoon boat.

Figure 7: Side scan sonar shadows (NOAA 2016).

A series of push and grab samples were taken at each site in order for grain size analysis to be completed. For the boat based bridges, the VanVeen grab sampler was used (Figure 8). At riverine sites, samples were taken with a shovel and bucket while wading. Although this method may lose some of the fine materials, it was concluded to be the best method based on height restrictions and rocky conditions at many of the riverine bridges. For each site, visual descriptions and grain size analysis of the sediments were performed. Any vegetation or riprap was noted as these features may affect the scour process and should be taken into account when analyzing results.

Figure 8: The VanVeen grab sampler being deployed on the pontoon boat.

3.1.2 TOPOGRAPHY

For topographic surveys, a TopCon electronic total station and Trimble Real Time Kinematic (RTK) system were used. The TopCon GTS 212 electronic total station, shown in Figure 9 was obtained from URI's Civil Engineering Department. The total station survey was used to find topography surrounding the bridge and the bathymetry beneath bridge. A total station allows for the surveyor to find the horizontal and vertical distances as well as the slope from the instrument to the reflector rod (Grahl 2013).

Figure 9: Topcon GTS212 electronic total station used to find topography and bathymetry.

A Trimble R10 Global Navigation Satellite System (GNSS) Receiver was used with a Trimble TSC3 Controller and the virtual reference station (VRS) network to complete RTK surveys at the bridge sites (Figure 10). This equipment was borrowed from the Rhode Island Coastal Resource Management Council. The RTK was used to take surveys of the bridge, its surrounding topography, and the river channel. To begin a survey, communication between the R10, TSC3, and VRS network must first be setup.

This allows for the VRS network to communicate with satellites and act as a base station to which the R10 communicates with, allowing for it to find its positioning within 0.59 inches (Trimble 2014). The TSC3 acts as a middleman between the R10 and VRS systems, displaying and recording the data (Trimble 2016).

Figure 10: Trimble RTK system used for topography.

To geo-reference the total station data, a set of temporary bench marks were set up and marked at each site. These temporary benchmarks were then surveyed with the RTK system allowing for the location and elevation of the benchmarks to be found and the total station data to be tied into a datum. The data sets were imported into the ArcGIS ArcMap software. This software is a powerful mapping tool which allows for data to be managed, processed, and analyzed (ArcGIS 2016). Once the data points were properly imported, each set of survey points was checked for any incorrect data points. The data

points were then combined and a triangular irregular network, also known as a TIN, was created to envision the survey area as a surface.

The survey data was also merged with the Rhode Island State Digital Elevation Model (DEM), obtained from the Rhode Island Geographic Information System (RIGIS), to create a larger area of interest around the bridge. The DEM was taken by airborne LiDAR and is accurate within 3.3 feet in the state of Rhode Island (RIGIS 2011). LiDAR stands for Light Detection and Ranging; it is a remote sensing system that collects data using a laser, scanner, and GPS system. The laser pulses, sending light beams to the surface below. These beams are then read by the scanner providing a distance to the surface, allowing for an array of precise data points to be quickly and efficiently collected (NOAA 2015). ArcMap was used to clip the DEM, being sure to exclude the river surface data as the LiDAR system used cannot penetrate through water. The DEM data was then combined with the total station and RTK survey data to make an accurate TIN of the bathymetry and topography at the bridge site. From this combined surface, cross sections were cut using the HEC-GeoRAS tool. These cross sections were imported into HEC-RAS for hydrographic modeling of the river.

3.1.3 BATHYMETRY

To collect bathymetry, the EdgeTech 6205 bathymetry and side scan sonar was used again (Figure 7). The bathymetry uses the 550 kHz frequency providing the user with a depth range of 1.5 to 165 feet (EdgeTech 2015). This dual system functions on the fundamentals of Phase Differencing Bathymetric Sonar, also known as interferometry, allowing for larger, more accurate swaths than a Multibeam Echo Sounder to be

collected in shallow water (Brisson et al. 2014). The system sends out an acoustic signal; then at multiple receivers, the arrival phase is recorded, allowing for the angle of the seabed off which the signal originated to be calculated. This angle is combined with travel time data to create information for a discrete seafloor location. These locations are then combined to create a profile and 3D model of the channels bathymetry (NOAA N.D.b). Its' wide swath allows for shallow water surveys to be completed faster as more data can be collected in one pass and with more accuracy, making it ideal for this study (Brisson et al. 2014).

The data was once again processed in the software CleanSweep. Once processed, any possible contraction scour is easily characterized at a bridge site. In addition, it is hopeful that possible local scour holes were present in the bathymetric maps, however, if either the holes have been in-filled or if the signal is blocked, the EdgeTech 6205 will not be able to locate the scour features. In the study completed by South Carolina, a fathometer system was used to create local bathymetry (Benedict et al. 2006). The Maine scour study found channel bathymetry cross sections using a total theodolite station (Lombard and Hodgkins 2008). No similar studies were found that used advanced sonar technologies to find the channel bathymetry.

3.1.4 SUB-BOTTOM

To detect in-filled scour holes a CHIRP sub-bottom system was used at the tidal sites. A Teledyne Benthos CHIRP III system was used to collect sub-bottom profiles as seen in Figure 12. This system relays acoustic energy to create a sub-surface image of the sea floor. Changes in acoustic impedances are collected, which can be interpreted as

sedimentary layers (WHOI 2016). The equipment was towed either off the side or stern of the pontoon boat (Figure 12) in a catamaran type setup with two large floats to which its source and transducers were connected. This model has a 2-7kHz frequency sweep and is equipped with transceiver box DSP664. Data acquisition and processing for the CHIRP system took place in SonarWiz 5. The CHIRP system will allow for any changes in stratigraphy due to an in-filled scour hole to be noted at the tidally influenced sites.

Figure 12: CHIRP sub-bottom system in catamaran float configuration.

Figure 12: CHIRP sub-bottom system, indicated by red arrow, being towed behind the stern of the pontoon boat. The system was also cinched to the side of the boat for increased control while passing under the bridge.

3.2 MODELING IN HEC-RAS

HEC-RAS is the U.S. Army Corps of Engineers River Analysis system which was developed by the Hydraulic Engineering Center. The program is able to solve one-dimensional steady flow analysis in addition to one and two-dimensional unsteady flow analysis; a one-dimensional steady flow analysis was used for the riverine bridges and a one-dimensional unsteady flow analysis was used for the tidal bridges. The steady flow computation is based on the solution of the energy equation where friction and contraction/expansion of the modeled river section provide energy losses. When the water surface profile is rapidly varied, such as at bridges, the momentum equation is balanced. For the unsteady flow computations, the conservation of mass and momentum are used to model the flow conditions within HEC-RAS (Brunner and CEIWR-HEC 2016).

It can be noted that one-dimensional sediment transport is easily calculated in HEC-RAS allowing for computation of scour. However, these scour predictions will not be used as the equations are based on the fourth edition of HEC-18 which was published in 2001 and is now considered a predecessor as the fifth edition was published in 2012 (Brunner and CEIWR-HEC 2016).

To complete a model, cross sections of the bridge surroundings were extracted from the tinned surfaces that combine topography and bathymetry in ArcGIS. To do so, the HEC-GeoRAS tool was used. These cross sections were carefully created to be perpendicular to the defined flow lines and river centerline and are shown in Figure 13. Each cross section was scrutinized, and any inaccurate points that were not representative of the area were deleted. They were then imported into HEC-RAS

defining the topography, river bank locations, centerline of the channel, flow paths, ineffective flow areas, and the location of the bridge. The bridge geometry was added next, specifying the deck, abutments, and piers. Manning coefficients were varied horizontally across each cross section using knowledge obtained from surveying to describe the local terrain and its frictional properties. These Manning coefficient values were checked against published ranges for the river of interest. Flow data corresponding to the flood of interest was added next, in addition to the flow boundary conditions. The flow data corresponding to the largest flood was obtained from the closest USGS flow gauge and translated either up or down stream to the bridge using the same methodology presented in the bridge's Plan of Action. Finally, the model's steady or unsteady flow analysis were run and the necessary parameters were examined.

Figure 13: HEC-GeoRAS tool in ArcMap used to create hydrographic features and cut cross sections at bridge sites.

Parameters used in the scour prediction analysis include the depth, velocity, and flow both upstream and at the bridge's reach. The length of active flow projected to the abutment included in Froehlich's abutment scour equation (Equation 9) was found by producing equal conveyance tubes along the abutment and embankment. To do this, the locations of the left and right banks were varied and conveyance across the banks was recorded. These values were then put into a MATLAB program that was created in order to find tubes of equal conveyance and decide at which point the flow transitioned from active flow where it contributes to scour, to inactive flow where it has little effect on the flow passing along the abutment.

For high flow scenarios, such as when the bridge is overtopped, the energy based calculations and the pressure and weir flow calculations were performed and compared. If the waterline reaches the lower chord of the bridge deck, pressure flow begins and the bridge is treated like an orifice. If overtopped, weir calculations are computed, treating the bridge like a weir and water can travel over the bridge (Brunner 2016). Weir coefficients were calculated using the Hydraulics of Bridge Waterways document (Bradley 1978).

Due to small study areas and no high flow scenarios taking place during this study, the modeled bridges could not be validated with measured velocities and water levels. Instead to validate the HEC-RAS model for each study area, the Manning coefficients were increased and decreased based on Federal Emergency Mapping Agency (FEMA) and USGS reports in order to observe changes in velocity and water level. The model results were compared to the values used in the study and any used to calculate scour values. This method explored the sensitivity of the model to frictional changes. Other coefficients that could be varied include the geometry and flow conditions. However, geometry was held constant throughout all of the study even though changes may occur to the river, the measurements taken were assumed to be precise and representative of the river system. The input flow was a known value, and was held constant, representing the highest flood scenario seen by the bridge. Changes to the flow boundary conditions could be made, however normal boundary conditions were used throughout and were seen to have little effect. The complete analysis of the Manning coefficients can be found in Chapter 9, section 2.4. Validation of the tidal model was completed through a tidal analysis and the comparison of the velocity during a tidal cycle at the bridge to the

Coastal Engineering Manual's Inlet Hydrodynamics average cross-sectional velocity (USACE 2002). A complete overview of this validation can be found in Chapter 8, section 3. Due to the complexity of each site, every model was produced differently and will be discussed in greater detail within the results and analysis section.

CHAPTER 4

KENYONS BRIDGE (NO. 020601), CHARLESTOWN, RI

This chapter presents the results of the field testing program, modeling of flow, and scour for Kenyons Bridge.

Kenyons Bridge or Bridge 020601 is located in Charlestown, RI and crosses the Pawcatuck River, as shown in Figure 15 and Figure 3. The bridge was constructed in 1926, and was reconstructed in 1984 to expand its width. As shown in Figure 15, the bridge is a single span arch made with concrete that carries two lanes of traffic on Route 91. Kenyons Bridge is about 51 feet long and 44.6 feet wide with a maximum of approximately 5 feet of clearance during normal flow conditions. Bed conditions indicate cobbles and boulders with sand and silts infilling voids (Fura and Mahmutoglu 2010a).

Figure 15: Location of Kenyons Bridge crossing the Pawcatuck River in Charlestown, RI (Google).

Figure 15: View of Kenyons Bridge, standing on the southwest embankment looking upstream (north).

Scour studies from the 2010 Plan of Action estimated 15.3 feet of scour occurring on the east abutment and 6.6 feet of scour on the western abutment with an additional 1.2 feet of contraction scour for a 100-year event (Table 2). Although previous scour reports note flow favoring the west side of the bridge, it is seen that increased scour is predicted for the east abutment (Fura and Mahmutoglu 2010a). This increase in predicted scour on the east abutment could be due to decreased depths at that abutment, however this is not certain as complete scour prediction calculations were not included in the plan of action report.

Table 2: Summary of Scour Predictions for Kenyons Bridge Using HEC-18 Fourth Edition (Fura and Mahmutoglu 2010a).

	10-year flow	50-year flow	100-year flow	500-year flow
Contraction Scour (ft)	0.1	0.5	1.2	2.0
Left (West) Abutment Scour (ft)	3.9	6.0	6.6	7.9
Right (East) Abutment Scour (ft)	12.4	15.0	15.3	13.5

In the past, scour holes have been found along both abutments with a large two feet deep by 30 feet long scour hole along the west abutment in 2010 (Fura and Mahmutoglu 2010a). Since 2010, the western abutment has experienced increased scour at the joint between the original and expanded bridge; increasing the hole in size to be about 33 feet long by two feet deep in 2013 (Pechillo 2013a). The eastern abutment experienced only three inches of scour over a four-foot span noted in the 2010 inspection (Fura and Mahmutoglu 2010a).

4.1 FIELD TESTING PROGRAM

Field data was collected using a kayak, a 15-foot Jon Boat (R/V Fred Pease), and by wading in the shallow portions of the river. Throughout the survey many data collection methods were used; the EdgeTech 6205 system was used to collect side scan and bathymetry data, a total station was used to survey bathymetry beneath the bridge, the Trimble RTK collected topography and geometry of the bridge, grab samples were collected for soil classification, and a Nikon Coolpix was used to collect underwater photographs.

The side scan images and bathymetry were successfully collected however difficulty occurred while trying to link them to the POS-MV system for GPS coordinates due to the low lying bridge and significant tree cover at the site. With each pass under the bridge the GPS system lost its signal and the data could not be correctly geo-referenced causing processing issues specifically in bathymetry data. Screen grabs of the waterfall image in CleanSweep could be taken, allowing for side scan images to be analyzed. Possible scour features were visible in the side scan images occurring on the western abutment as indicated by the blue arrows in Figure 16. Other noticeable features include submerged piles at the southern end of the east abutment and the steel sheeting located at the southern end of the west abutment.

Figure 16: Side Scan Image from Kenyons Bridge on 7/7/2016, blue dashed lines indicate bridge abutments and arrows indicate possible scour features.

The bathymetry collected did not cover the entirety of the channel as the nadir was very large and a mosaic was not able to be created due to the lost GPS signal. When analyzing the bathymetry, no data was recorded at the two possible scour features highlighted in the side scan image. This lack of data indicates that either a feature is

blocking the signal or a hole at which the signal cannot reach is at this location. In order to collect bathymetry data to make a surface of the channel, a kayak was used in tandem with the RTK and the total station. RTK shots were taken upstream and downstream of the bridge while the total station was used beneath the bridge where signal was lost. The combined topography and bathymetry is shown in Figure 17.

Figure 17: Combined topography and bathymetry at Kenyons Bridge in Charlestown, RI referenced to RI SPC, NAD 1983 in feet..

Finally, the bed conditions were captured with underwater photographs and grab samples. The Nikon camera successfully portrayed the bottom conditions which consisted of some rocks with vegetation upstream and downstream (Figure 18). Approximate locations of the grab samples are indicated by red stars in Figure 19. The grain size distributions of the grab samples are shown in Figure 20. It was found that the median grain size ranged from 0.0722 feet (22 millimeters) to 0.00125 feet (0.38 millimeters). The average of the four samples was taken and the representative median grain size at Kenyons Bridge was calculated to be 0.0387 feet (11.8 millimeters). Also

shown in Figure 20 is the grain size distribution of a sample taken from this bridge during the 2009 bridge inspection (Fura and Mahmutoglu 2010a).

Figure 18: Bottom conditions captured with the waterproof Nikon camera downstream of Kenyons Bridge in Charlestown, RI.

Figure 19: Approximate location of grab samples taken at Kenyons Bridge indicated by red stars.

Figure 20: Grain size distribution for Kenyons Bridge in Charlestown, RI.

4.2 CURRENT SCOUR FEATURES

Upon visiting the bridge no scour was observed while wading along the abutments; in fact the area near each abutment was much shallower than the middle of the channel. It is believed that rip-rap or stones must have been placed at some point on the west abutment as it was armored at the time of the visit. Meanwhile, the eastern abutment consisted of a much softer material with the occasional stone. The surface created in GIS from survey data (Figure 21) indicated deeper depths along the western side of the channel, but no specific scour features could be indicated.

Figure 21: Bathymetric surface at Kenyons Bridge in Charlestown, RI referenced to RI SPC, NAD 1983 in feet.

4.3 HEC-RAS MODELING

With processed surveys completed, it was now possible to create cross sections and define the river features using HEC-GeoRAS as shown in Figure 22. Fifteen cross sections were created, perpendicular to the Pawcatuck River, to define the area of interest around Kenyons Bridge.

Figure 22: Cross sections at Kenyons Bridge in Charlestown, RI. Numbers refer to cross section station numbers in feet from the southern end of the river centerline.

To evaluate any upstream values in the scour calculations, the bridge width was used to find the approximate location of an acceptable upstream cross section to characterize the upstream conditions. Kenyons Bridge is nearly 45 feet wide, and its upstream fascia is located at cross section 249.5 (Figure 26). Therefore, cross section 287.5 (Figure 24) was chosen to be the upstream cross section used in scour calculations. Below, the models' starting cross section, cross section used for upstream calculations, cross section at the bridge, and ending cross section are presented in Figure 24, Figure 24,

Figure 26, and Figure 26 respectively. The remaining cross sections can be found in Appendix C. Each figure depicts the velocity distribution and water level across the cross section for the 2010 flood. The manning bottom friction coefficients selected to represent the channel and its surroundings are located at the top of each figure and vary horizontally across each cross section; they ranged from 0.075 to 0.033 based on the FEMA and USGS reports (Zarriello et al. 2014a).

Figure 24: Kenyons Bridge cross section 360.39, located at the start of the model upstream of the bridge.

Figure 24: Kenyons Bridge cross section 287.54, used for scour calculations as the upstream cross section.

Figure 26: Kenyons Bridge cross section 249.55, the upstream bridge cross section. Note the bridge is an arc but does not appear so do to scale.

Figure 26: Kenyons Bridge cross section 39.97, located at the end of the model downstream of the bridge.

To model Kenyons Bridge a rating curve from USGS flow gauge 01117500 was used as the downstream boundary condition. The location of this gauge is ideal as it is located just downstream of the bridge as shown in Figure 27. Maximum flow associated with the gauge was found to be 3,490 cubic feet per second occurring on March 31, 2010 and was used as the flow for steady flow calculations (USGS 2016a).

Figure 27: Location of Gauge 01117500 with respect to Kenyons Bridge (USGS 2016a).

With all geometry inputted and flow characterized, the model could successfully be run for the 2010 floods. It is easily observed in the above cross sections (Figure 24 through Figure 26) that the bridge was overtopped in 2010 and the surrounding area was inundated. These results appear to be in agreement with photos captured during the flood. Figure 29 was taken at the bridge on March 30, 2010; it appears that flooding has begun and the banks have been breached. On March 31, 2010 the bridge was not accessible, the road was completely inundated as depicted in Figure 29.

Figure 29: West of Kenyons Bridge on March 30, 2010. The river has been overtopped, inundating the area surrounding the bridge (Fox 2010a).

Figure 29: East of Kenyons Bridge on March 31, 2010. The area surrounding the bridge is completely inundated and the bridge can not be reached (Aube 2010).

The average depth of the channel was 13.7 feet while at the bridge an average depth of approximately 15 feet was seen. Due to bridge overtopping, the bridge was treated as a weir and the pressure flow solutions were used by HEC-RAS. Velocities ranged from 1.1 feet per second to 3.3 feet per second with an average of 2.2 feet per second and 2.4 feet per second at the bridge. Input parameters for the scour calculations will be included in the next section. In addition, a full summary of model outputs can be found in Appendix C.

4.4 SCOUR CALCULATIONS

To calculate contraction scour, the critical velocity was first found using the upstream cross section 287 for flow input variables, Table 3. It was found that clear-water conditions would occur for this flood scenario.

Table 3: Kenyons Bridge Contraction Scour Predictions for 2010 Flood

y (ft)	D ₅₀ (ft)	K _u	V _c	V	Clear-water or live-bed?
13.58	0.0387	11.17	5.83	2.41	Clear-water

Laursen's modified 1963 equation (Equation 3) was used to predict clear-water scour, Table 4. A negative scour value was calculated, indicating that no contraction scour would occur and accretion of sediment may occur. To be conservative, a value of zero will be used for any negative scour.

Table 4: Clear-Water Contraction Scour for Kenyons Bridge

K_u	Q (ft ³ /s)	D_m (ft)	W (ft)	y_2 (ft)	y_0 (ft)	y_s (ft)
0.0077	3490	0.048	52.20	11.70	16.35	0

Froehlich's abutment scour equation (Equation 9) and the NCHRP 24-20 equation (Equation 10) were applicable to calculate abutment scour at Kenyons Bridge. In order to calculate the length of active flow across each abutment for Froehlich's equation, MATLAB was used to plot the conveyance and the associated curves for each abutment, depicted in Figure 30. Due to the complete inundation at this site, conveyance tubes were found beyond the length of the abutments in order to calculate if there was live flow across the embankment, beyond the abutment. Five equal conveyance tubes were found and are depicted in Figure 31. For the left abutment, a length of 64.3 feet was used, encompassing the first three tubes. On the right abutment, a length of 121.2 feet was used, also encompassing the first three conveyance tubes.

Figure 30: Conveyance at the abutments of Kenyons Bridge.

Figure 31: Conveyance tubes for Kenyons Bridge.

It was calculated that 5.03 feet of abutment scour would occur at the left abutment, while 10.38 feet of scour would occur at the right abutment, Table 5.

Table 5: Froehlich's Abutment Scour Prediction for Kenyons Bridge During the 2010 Flood

Abutment	K_1	K_2	L (ft)	L' (ft)	A_c (ft ²)	Q_c (ft ³ /s)	V_c (ft ² /s)	y_a (ft)	Fr	y_s (ft)
Right (East)	0.82	0.98	204	121	765	682	0.89	3.76	0.08	10.38
Left (West)	0.82	0.98	270	64	393	302	0.77	1.46	0.11	5.03

The NCHRP 24-20 equation (Equation 10) predicted 11.77 feet of contraction and abutment scour on the left side of the channel and 12.83 feet of scour on the right side of the channel. A full summary of values can be found in Table 6.

Table 6: NCHRP 24-20 Abutment Scour Prediction for Kenyons Bridge During the 2010 Flood

Abutment	q_{2f} (ft ² /s)	K_u	D_{50} (ft)	y_c (ft)	α_B	y_o (ft)	y_{max} (ft)	y_s (ft)
Right (East)	66.85	11.17	0.038	12.68	1.97	10.29	23.12	12.83
Left (West)	66.85	11.17	0.038	12.68	1.97	11.35	23.12	11.77

4.5 DISCUSSION

The largest scour depression seen at Kenyons Bridge was 30 feet in length and two feet deep. It was located along the western abutment in a scour report from July, 2010. Although the hole may have been in-filled or repaired in the months since the flood, an eight-foot difference remains between the predicted value and the observed, Table 7. It must be considered that the current bathymetry is being used for the comparison to observed scour; since the creation of the scour hole in 2010 the bathymetry at the site may have changed significantly.

Table 7: Kenyons Bridge Summary of Scour

	Predicted Scour based on 2010 flow	Largest observed scour and associated year
Contraction Scour (ft)	0	-
Right (East) Abutment Scour (ft)	10.38	0.25, 2010
Left (West) Abutment Scour (ft)	5.03	2 , 2013

CHAPTER 5

FIRST BARBERVILLE BRIDGE (NO. 040101), HOPKINTON, RI

This chapter presents the results of the study on the First Baberville Bridge. A description of the bridge history, the field testing program, and the modeling of the flow in HEC-RAS will be presented, followed by a discussion of the scour estimates.

The First Baberville Bridge, or Bridge number 004101 was built in 1925 in Hopkinton, RI and crosses over the Wood River, as shown in Figure 33 and Figure 3. This concrete bridge carries two lanes of traffic over a span of 48 feet. Downstream from the bridge lies USGS flow station 01118000 which was used to find the peak flows at the bridge. A small dam is located approximately 35 feet upstream of the bridge (Figure 33), which controls the hydrodynamics at this site. The angle of attack at the bridge ranges from 5 to 20 degrees. According to a sieve analysis completed in 1994, bed materials at the First Baberville Bridge were recorded to be boulders, cobbles, gravel, and sand with the median grain size (D_{50}) of the armor layer being 120mm and the soil beneath the armor layer having a median grain size of 9mm according to a sieve analysis completed in 1994 (Fura and Mahmutoglu 2010b).

Figure 33: Location of the First Barberville Bridge crossing the Wood River in Hopkinton, RI (Google).

Figure 33: View of the First Barberville Bridge, standing on the northeast bank looking southwest.

During a routine inspection in 1994, 0.5 feet of scour was noted at the center pier. The scour calculations completed with this inspection reveal that the bridge qualified as scour critical for the 10, 50, 100, and 500-year events, with complete undermining of the abutments and pier for all events. During the 500-year event, a predicted 16.9 feet of scour would occur at the eastern abutment, 14.5 feet of scour at the center pier, and 17.2 feet of scour at the western abutment as summarized by Table 8.

Table 8: Summary of Scour Predictions for the First Barberville Bridge Using HEC-18 Fourth Edition (Whitman & Howard 2009).

	10-year flow	50-year flow	100-year flow	500-year flow
Contraction Scour (ft)	0.0	1.1	1.6	2.2
Pier Scour (ft)	10.4	11.3	11.8	12.3
Left (West) Abutment Scour (ft)	9.3	11.0	12.0	14.7
Right (East) Abutment Scour (ft)	10.1	11.9	12.7	15.0

In 2008, the First Baberville Bridge was revisited for inspection. No scour was observed and no counter measures were in place. The bridge was given an item 113 rating of 3A, or scour critical, moderate. However, during the 2010 inspection, the bridge sustained an item 113 rating of 2—scour critical, extensive scour, in need of

immediate repairs due to a 25 feet long, 20 feet wide, 4 feet deep scour depression encompassing the entirety of the eastern span and exposing the abutment foundation (Fura and Mahmutoglu 2010b). This large depression resulted in a depth variation of about 5 feet between the eastern and western spans. In addition, the footing of the pier was exposed (Pechillo 2010).

Repairs took place, including the placement of four to six-inch rip-rap in three areas: along the east abutment, beneath the eastern span, and in front of the span creating a berm as shown in Figure 34 (Collins Engineers N.D.). During the most recent inspection in 2015 scour was observed upstream of the eastern bridge span extending 25 feet to a depth of 4 feet, with a hole of 3 feet diameter by 1-foot-deep located at the nose of the pier (Sauco, LaPlante, and Paull 2015).

Figure 34: Berm (indicated by red arrow) located in front of the eastern span at the First Barberville Bridge. Total station setup for bathymetry collection indicated by blue arrow.

5.1 FIELD TESTING PROGRAM

Field testing was preformed in July and August, 2016. Due to dry weather conditions and low water levels, it was safe to conduct bathymetric surveys by wading. The Topcon total station was used to survey the bathymetry beneath the bridge, while the Trimble RTK was used to collect channel bathymetry in the upstream and downstream reaches. Data points could not be taken just downstream of the waterfall due to large depths posing a safety hazard. The Trimble RTK was also used to generate topography at the bridge site and characterize the benchmarks used for the total station survey. Figure 35

depicts the surveyed points combined with the DEM, creating the topography and bathymetry at the First Barberville Bridge.

Figure 35: Combined topography and bathymetry at the First Barberville Bridge referenced to RI SPC, NAD 1983 in feet located in Hopkinton, RI.

Finally, bottom conditions were noted and photographed using the Nikon Coolpix and grab samples were collected. The rip-rap characterized can be seen in Figure 36. A soil sample was collected at only one location due to the complete coverage of rip-rap at the bridge. The approximate location is shown in Figure 37 at about the center of the western abutment. The median grain size of this sample was found to be 0.0034 feet (1 millimeter) excluding the rip-rap or 0.26 feet (80 millimeters) with the inclusion of the riprap (Figure 38). Also shown in Figure 38 are two grain size distribution curves that were sampled in 1994 and included in the plan of action (Fura and Mahmutoglu 2010b).

Figure 36: Rocky bottom conditions with minimal growth at the First Barberville Bridge in Hopkinton, RI.

Figure 37: Approximate location of soil grab sample taken at the First Barberville Bridge.

Figure 38: Grain size analysis for the sample taken at the First Barberville Bridge.

5.2 CURRENT SCOUR FEATURES

Due to the rip-rap placed in front of the eastern span, depths are similar between the eastern and western spans according to surveys as shown in Figure 39. The deepest depths at the bridge site are located surrounding the pier. However, no distinct scour features were observed during site visits. The largest area of concern would be on the eastern span directly behind the rip-rap berm and along the eastern side of the pier.

Figure 39: Bathymetry beneath the First Barberville Bridge referenced to RI SPC, NAD 1983 in feet.

5.3 HEC-RAS MODELING

Using the surfaces created by combining the DEM and surveys, cross sections were created and the Wood river was defined in ArcMap using HEC-GeoRAS (Figure 40). Due to large depths, survey data was limited upstream. Meanwhile, downstream depths decreased allowing increased access, but poor GPS signals due to tree coverage limited data collection.

Figure 40: Cross sections at the First Barberville Bridge. Numbers refer to cross section station numbers in feet from the southern end of the river centerline.

Cross section 146 was used as the upstream cross section for scour calculations. Although this cross section is closer than the bridge's width, it is not strongly influenced by the waterfall located upstream. The model's starting cross section, upstream cross section 146, bridge cross section, and ending cross section can be seen in Figure 42, Figure 42, Figure 44, and Figure 44 below. All remaining cross sections can be found in Appendix D. The velocity distribution and water level associated with the 2010 flood are depicted on each cross section.

Figure 42: The First Barberville Bridge cross section 176.82, located at the start of the model.

Figure 42: The First Barberville Bridge cross section 146.44, used as the upstream cross sections for scour calculations.

Figure 44: The First Barberville Bridge cross section 124.85, the upstream bridge cross section.

Figure 44: The First Barberville Bridge cross section 9.79, located at the end of the model.

USGS gauge 01118000, which dates back to 1941, was used to model the flow at the First Barberville Bridge. The largest flood since 1941 at this site occurred on March 30, 2010, discharging 5,470cfs (USGS 2016b). Using the same translation equation as presented in the plan of action, this discharge was translated to the bridge by:

Discharge at Bridge No. 004101

$$= \text{USGS gauge flow} * \left(\frac{\text{Bridge drainage area}}{\text{USGS gauge drainage area}} \right)^{0.72}$$

where the bridge drainage area is 53.4 square miles and the USGS gauge drainage area is 72.4 square miles (Fura and Mahmutoglu 2010b). According to this equation, the flow at the bridge was calculated to be 4,380 cubic feet per second, exceeding the 500-year reoccurrence interval for this site.

Manning coefficients ranged from 0.15 in the overbanks to 0.045 in the channel and were adjusted accordingly across each cross section. These values were chosen based on the FEMA and USGS report by Zarriello, Straub, and Smith in 2014. Upstream, a critical depth boundary condition was employed and downstream a normal depth of 0.0027 was used. This slope was calculated from the DEM in ArcGIS extending approximately 4.5 miles downstream of the river.

For the 2010 flood scenario, the model predicts that the bridge would be overtopped, and therefore a pressure and weir analysis was used within HEC-RAS. Images from the 2010 floods indicate extreme flooding in the area and slight overtopping of the bridge as predicted by the model (Figure 46 and Figure 46).

Figure 46: Standing on the northwest bank of the Wood River, looking toward the First Barberville Bridge on March 30, 2010 (Fox 2010b).

Figure 46: The First Barberville Bridge was slightly overtopped due to extreme flooding on March 30, 2010 (Fox 2010c).

The average depth of the channel was 8.4 feet. While at the bridge, an average depth of about 11.5 feet occurred, causing 0.75 feet of overtopping. Velocities reached speeds of 8.3 feet per second within the channel, averaging at about 7 feet per second at the bridge span. All input parameters for scour calculations will be included in the next section, and a full list of outputs can be found in Appendix D.

5.4 SCOUR CALCULATIONS

To complete scour calculations for the First Barberville Bridge two analysis were completed. The first using the median grain size of just the sub-surface material (1 millimeter or 0.0034 feet). The second analysis was completed with the inclusion of the rip-rap later where the median grain size was 0.26 feet (80 millimeters).

5.4.1 SCOUR CALCULATIONS WITH OUT TO RIP-RAP

Using upstream cross section 146, the critical velocity was found to be 2.59 feet per second, much less than the velocity of the flood plain, initiating live-bed scour (Table 9).

Table 9: The First Barberville Bridge Contraction Scour Predictions for 2010 Flood

y (ft)	D ₅₀ (ft)	K _u	V _c	V	Clear-water or live-bed?
13.15	0.0034	11.17	2.59	4.66	Live-bed

To analyze contraction scour for live-bed conditions, Laursen's 1960 modified equation (Equation 2) was used. It was determined that 11.29 feet of contraction scour would occur at the bridge as shown in Table 10. This amount of scour would undermine the structure without considering the effects of local scour.

Table 10: Live-Bed Contraction Scour for the First Barberville Bridge

y_1 (ft)	y_o (ft)	Q_1 (ft ³ /s)	Q_2 (ft ³ /s)	W_1 (ft)	W_2 (ft)	k_1	y_2 (ft)	y_s (ft)
13.15	11.16	3213.63	4380	52.47	35.5	0.69	22.45	11.29

To analyze pier scour, two methods were used. First the HEC-18 pier scour equation (Equation 5) was used. It is estimated that 8.95 feet of pier scour would occur (Table 110). The FDOT methodology was used next, estimating that 7.80 feet of pier scour would occur for the 2010 flood scenario with the median grain size of the sub-surface layer, shown in Table 12.

Table 11: HEC-18 Pier Scour for the First Barberville Bridge

y_1 (ft)	K_1	a (ft)	L (ft)	K_2	K_3	V_1 (ft/s)	Fr	y_s (ft)
11.47	1.1	3.75	22	1.0	1.1	7.49	0.39	8.95

Table 12: FDOT Pier Scour for the First Barberville Bridge

D_{50} (ft)	uc^*	y_1 (ft)	V_c (ft/s)	V_{lp} (ft/s)	K_{sf}	a^* (ft)	f_1	f_3	y_{s-c} (ft)	y_{s-lp} (ft)	y_s (ft)
0.0034	0.08	11.47	2.02	22	1.23	4.61	0.89	0.59	6.09	9.06	7.80

Next, local scour in the form of abutment scour was analyzed using Froehlich's abutment scour equation (Equation 9) and the NCHRP 24-20 equation (Equation 10). To calculate the live length of flow across the abutments, the MATLAB script found in Appendix B was used. The associated curves that represent conveyance across the abutment are seen in Figure 47. The conveyance tubes, which are used to find the effective length, are depicted in Figure 48.

Figure 47: Conveyance across the First Barberville Bridge abutments.

Figure 48: Conveyance tubes for the First Barberville Bridge.

Using three conveyance tubes along each abutment, the live length of flow was determined to be 25.85 feet for the right abutment and 44.15 feet for the left abutment. Scour was then calculated to be 8.98 feet for the right abutment and 7.51 feet for the left abutment using Froehlich’s methodology as presented in Table 13.

Table 13: Froehlich's Abutment Scour Prediction for the First Barberville Bridge During the 2010 Flood

Abutment	K_1	K_2	L (ft)	L' (ft)	A_c (ft ²)	Q_c (ft ³ /s)	V_c (ft ² /s)	y_a (ft)	Fr	y_s (ft)
Right (West)	0.82	1.0	86	25.9	280	603	2.16	3.26	0.21	8.98
Left (East)	0.82	1.0	127	44.2	243	439	1.81	1.91	0.23	7.51

Next, the NCHRP 24-20 equation (Equation 10) was used to estimate scour at the First Barberville Bridge using the sub-surface grain size. The results of this methodology are presented in Table 14. It was predicted that over 17.5 feet of scour would occur on each abutment.

Table 14: NCHRP 24-20 Abutment Scour Prediction for the First Barberville Bridge During the 2010 Flood

Abutment	q_{2f} (ft ² /s)	q_1 (ft ² /s)	y_1 (ft)	D_{50} (ft)	y_c (ft)	α_A	y_o (ft)	y_{max} (ft)	y_s (ft)
Right (West)	123.38	61.25	13.15	0.0034	23.97	1.25	10.28	29.96	19.68
Left (East)	123.38	61.25	13.15	0.0034	23.97	1.25	12.38	29.96	17.58

5.4.2 SCOUR CALCULATIONS INCLUDING RIP-RAP

Next, the rip-rap was included in the grain size analysis, making the median grain size 0.263 feet or 80 millimeters. The critical velocity was found to be 10.99 feet per second, making the conditions clear-water conditions, depicted in Table 15.

Table 15: The First Barberville Bridge Contraction Scour Predictions for 2010 Flood

y (ft)	D_{50} (ft)	K_u	V_c	V	Clear-water or live-bed?
13.15	0.263	11.17	10.99	4.66	Clear-water

Contraction scour was then calculated to be zero, Table 16, based on the Laursen's 1963 modified clear-water contraction scour equation (Equation 3). A negative scour value was calculated, however to be conservative, a value of zero will be used.

Table 16: Clear-Water Contraction Scour for the First Barberville Bridge

K_u	Q (ft ³ /s)	D_m (ft)	W (ft)	y_2 (ft)	y_0 (ft)	y_s (ft)
0.0077	4380	0.328	35.5	10.59	11.16	0

Pier scour was recalculated with the inclusion of the armor layer, allowing for the pier scour in coarse bed materials equation (Equation 7) to be used in addition to the HEC-18 equation (Equation 5) and the FDOT equation (Equation 6). Due to no inclusion of grain size, the pier scour based on the HEC-18 equation remained the same as the previous analysis, equating to 8.95 feet, refer to Table 11. The scour predicted by the FDOT equation decreased to be 6.49 feet of scour at the pier, Table 17.

Table 17: FDOT Pier Scour for the First Barberville Bridge

D_{50} (ft)	uc^*	y_1 (ft)	V_c (ft/s)	V_{lp} (ft/s)	K_{sf}	a^* (ft)	f_1	f_3	y_{s-c} (ft)	y_{s-lp} (ft)	y_s (ft)
0.263	0.89	11.47	12.25	61.27	1.23	4.61	0.89	0.89	9.15	9.06	6.49

The pier scour in coarse bed material methodology decreased the scour estimation significantly to 0.24 feet, Table 18.

Table 18: Pier Scour in Coarse Bed Materials at the First Barberville Bridge

D_{84} (ft)	σ	K_1	K_2	a (ft)	y_1 (ft)	S_g	D_{50} (ft)	V_1 (ft/s)	H_{fr}	y_s (ft)
4	15.24	1.1	1.0	3.75	11.47	2.65	0.263	7.49	2.01	0.24

Finally, abutment scour was calculated for the First Barberville Bridge. The estimation based on Froehlich's methodology did not change as it does not involve median grain size, Table 13 can be referenced. The results using the NCHRP 24-20 prediction did change, decreasing significantly. With the inclusion of the rip-rap layer, approximately 10 to 12 feet of scour was predicted to occur on each abutment, Table 19.

Table 19: NCHRP 24-20 Abutment Scour Prediction for the First Barberville Bridge During the 2010 Flood

Abutment	q_{2f} (ft ² /s)	K_u	D_{50} (ft)	y_c (ft)	α_B	y_o (ft)	y_{max} (ft)	y_s (ft)
Right (West)	123.38	11.17	0.263	11.49	1.96	10.28	29.96	12.23
Left (East)	123.38	11.17	0.263	11.49	1.96	12.38	29.96	10.13

5.5 DISCUSSION

After the 2010 floods, a large scour depression was observed over the entire eastern span, causing depth differences of about four feet in comparison to the western span. Pier scour at this site has varied in the past, reaching up to one foot. Since 2010, rip-rap has been placed, creating an armor layer at the bridge. The predicted scour in Table 20 below is based on the calculations with the current bathymetry and grain sizes with and without the armor layer. The pier scour calculations are based on FDOT methodology and the pier scour in coarse bed material methodology. The abutment scour predictions are those from Froehlich's abutment scour equation (Equation 9) which yielded a smaller approximation than the NCHRP equation (Equation 10).

Table 20: First Barberville Bridge Summary of Scour

	Predicted Scour based on 2010 flow		Largest observed scour and associated year
	$D_{50}=0.0034$ ft	$D_{50}=0.263$ ft	
Contraction Scour (ft)	11.29	0	-
Pier Scour (ft)	7.80	0.24	1, 2015
Left (East) Abutment Scour (ft)	8.98	8.98	-
Right (West) Abutment Scour (ft)	7.51	7.51	4, 2010

The First Barberville Bridge should be reevaluated with the inclusion of the armor layer as it was characteristic of the site. In addition, flow conditions have changed due to the berm placed northeast of the bridge. It would be beneficial if there was an equation

for abutment scour for coarse bed materials similar to the new pier scour equation. To further improve this analysis, more sediment samples should be taken at the site to better characterize the underlying soil though the armor layer will still control the grain size distribution.

CHAPTER 6

ESMOND STREET BRIDGE (NO. 094801), SMITHFIELD, RI

This chapter presents the results of the analysis of the Esmond Street Bridge. The history of the bridge will be discussed in addition to the field testing program, flow modeling, and the scour calculations.

Located in Smithfield, RI (Figure 3) Bridge number 094801, the Esmond Street Bridge, was constructed in 1979. Crossing the Woonasquatucket River, the bridge spans 48 feet carrying two lanes of traffic (Figure 50 and Figure 50). The Esmond Street Bridge is constructed of reinforced concrete on spread footings. It lies approximately 300 feet downstream of a small dam with another dam an additional 500 feet downstream. USGS station 01114500 is used for flow quantities, it is located nearly 9,000 feet downstream. The flow angle of attack has been recorded by previous inspections to be 0 degrees (Fura and Mahmutoglu 2010c).

Figure 50: Location of the Esmond Street Bridge crossing the Woonasquatucket River in Smithfield, RI (Google).

Figure 50: Looking south, downstream at the Esmond Street Bridge.

A scour inspection completed in 1995 indicated an armor layer was present and the bed was comprised of cobbles, gravel, and sand. The median grain size (D_{50}) of the armor layer was recorded to be 112mm with a median grain size of 1.6mm for the deeper sediments. Rip-rap had been placed on the abutments, however the rip-rap located on the east abutment developed a scour hole approximately 20 feet long and 3 feet deep, exposing the footing of Bridge 948. The bridge was then deemed to be scour critical for the 10, 50, 100, and 500-year events, with overtopping occurring during the 500-year event. For the 500-year event it was calculated that a total of 16.4 feet of scour would occur on the left abutment and 11.4 feet of scour on the right abutment of the bridge, presented in Table 21. These estimates yielded an item 113 rating of 3B for the Esmond Street Bridge (Fura and Mahmutoglu 2010c).

Table 21: Summary of Scour Predictions for the Esmond Street Bridge Using HEC-18 Fourth Edition (Fura and Mahmutoglu 2010c)

	10-year flow	50-year flow	100-year flow	500-year flow
Contraction Scour (ft)	1.7	2.5	3.5	4.8
Left (East) Abutment Scour (ft)	8.3	11.0	11.6	11.6
Right (West) Abutment Scour (ft)	6.0	6.3	6.4	6.6

In 2007, Bridge 948 was revisited and no scour was observed. However, the stone rip-rap that had once been present on the abutments had been washed away. The bridge was then given an item 113 rating of 3C, scour critical high. It was suggested that rip-rap be placed and a scour critical elevation plaque be posted to note water levels at which the bridge should be monitored (Fura and Mahmutoglu 2010c).

6.1 FIELD TESTING PROGRAM

Due to low flow conditions at the Esmond Street Bridge, all surveys were completed by wading in the river. At this bridge the Topcon total station was used to record bathymetry under the bridge using a temporary bench mark. The bridge was then revisited with the Trimble RTK and bathymetry and topography surrounding the bridge was collected in addition to the location of the temporary bench mark. Both data sets were combined with the DEM to create a surface from which cross sections were cut as seen in Figure 51.

Figure 51: Combined topography and bathymetry at the Esmond Street Bridge in Smithfield, RI referenced to RI SPC, NAD 1983 in feet.

Bottom conditions were recorded and photographed using the Nikon Coolpix AW100 and grab samples of the sediment at the bridge were also collected. The bottom consisted of vegetation upstream with scattered rocks throughout the reach, depicted in Figure 52. Five grab samples were taken under the bridge, the approximate location of

these samples can be seen in Figure 53. The median grain size ranged from 0.005 feet (1.5 millimeters) to 0.075 (23 millimeters) with an average of 0.025 feet (7.7 millimeters) (Figure 540. Sample one was not used for this analysis due to a large amount of organic material and very little sediment. The average of samples two, three, and four were used for the scour calculations presented in this section, yielding a median grain size of 0.0082 feet (2.5 millimeters) at the bridge. It should be noted that rocks were not taken into account when calculating the grain size at the bridge, this will generate a more conservative result similar to sample two taken in 1995 which was included in the Bridge's plan of action (Fura and Mahmutoglu 2010c).

Figure 52: Bottom conditions captured with the Nikon camera at the Esmond Street Bridge. Vegetation is present in the river with rocks throughout.

Figure 53: Location of samples at the Esmond Street Bridge.

Figure 54: Grain size analysis for the Esmond Street Bridge. Sample one was not included due to a large amount of leaves and little sediment.

6.2 CURRENT SCOUR FEATURES

While visiting the bridge it was noticed that the depths at the eastern abutment were deeper than those along the western abutment, as seen in Figure 55. There were many areas where large rocks were concentrated and behind them slight scour depressions existed, such as the red area seen just southeast of the center in Figure 55. which exhibited a depression nearly two feet deep. At the northeast abutment corner a slight scour feature was present with a change in depth of about 1.25 feet.

Figure 55: Bathymetry under the Esmond Street Bridge collected with the Topcon total station referenced to RI SPC, NAD 1983 in feet.

6.3 HEC-RAS MODELING

After all of the data had been combined and processed in ArcGIS, cross sections were cut with the HEC-GeoRAS tool. A total of 13 cross sections were extracted as seen in Figure 56. Manning bottom friction coefficients were selected based on the FEMA Simulated and Observed 2010 Flood Water Elevation report and ranged from 0.07 to 0.037 (Zarriello et al. 2014b).

Figure 56: Cross sections at the Esmond Street Bridge in Smithfield, RI.

For the scour calculations, upstream cross section 202 was used due to the bridges width of 48 feet and this being the nearest upstream cross section within that distance. The first, upstream, bridge, and end cross sections can be seen below in Figure 57, Figure 58, Figure 59, and Figure 60. All remaining cross sections are presented in Appendix E. Across each cross section the velocity distribution and associated water level for the 2010 flood at the Esmond Street Bridge are seen.

Figure 57: Esmond Street Bridge cross section 304, located upstream at the start of the model.

Figure 58: Esmond Street Bridge cross section 202, the upstream cross section used in scour calculations.

Figure 59: Esmond Street Bridge cross section 155, located at the upstream side of the bridge.

Figure 60: Esmond street bridge cross section 32, located at the downstream boundary of the model.

USGS gauge station 01114500 was used to find the maximum flow at the Esmond Street Bridge. The maximum flow occurred on March 30, 2010, discharging 1,810 cubic feet per second (USGS 2016c). This flow was translated to the bridge through:

Discharge at Bridge No. 094801

$$= USGS \text{ gauge flow} * \left(\frac{\text{Bridge drainage area}}{\text{USGS gauge drainage area}} \right)^{0.70}$$

where the bridge drainage area is 38.85 square miles and the USGS gauge drainage area is 35.16 square miles (Fura and Mahmutoglu 2010c). Therefore, the calculated flow that occurred at the bridge during the 2010 flood was 1,688 cubic feet per second. This flow was used as the input into HEC-RAS for completing the steady flow calculations with a downstream boundary condition of 0.0065 normal depth.

With the above inputs, the model was successfully run. The average depth was found to be 4.91 feet, with a depth of 6.93 feet at the bridge. Velocities ranged from 4.11 to 7.22 feet per second with an average of 5.50 feet per second. In the following section, input parameters for the scour equations will be discussed. A full summary of the model's outputs can be found in Appendix E.

6.4 SCOUR CALCULATIONS

The contraction scour was calculated using the upstream cross section 202 as a representative cross section for the upstream input variables. It was first found that live-bed conditions would exist at the bridge site under the flow conditions of the 2010 floods, Table 22.

Table 22: Esmond Street Bridge Contraction Scour Predictions for 2010 Flood

y (ft)	D ₅₀ (ft)	K _u	V _c	V	Clear-water or live-bed?
5.44	0.0082	11.17	2.99	5.04	Live-bed

Using Laursen's modified 1960 contraction scour equation (Equation 2) for live-bed conditions, it was predicted that 0.31 feet of contraction scour would occur at the Esmond Street Bridge, Table 23.

Table 23: Live-Bed Contraction Scour for the Esmond Street Bridge

y_1 (ft)	y_o (ft)	Q_1 (ft ³ /s)	Q_2 (ft ³ /s)	W_1 (ft)	W_2 (ft)	k_1	y_2 (ft)	y_s (ft)
5.44	6.9	1666.52	1688.00	71.64	48.0	0.69	7.25	0.31

To calculate local scour conditions, Froehlich's abutment scour equation (Equation 9) and the NCHRP 24-20 equation (Equation 10) were used. First, in order to calculate the love flow across each abutment for Froehlich's equation, a MATLAB script was used to find equal conveyance tubes. The best fit line equations were found in excel and are plotted in Figure 61. The first two conveyance tubes seen in Figure 62 were used to characterize both abutments. A length of 2.05 feet was used to characterize the left abutment and 1.5 feet was used for the right abutment.

Figure 61: Conveyance at the abutments of the Esmond Street Bridge.

Figure 62: Conveyance tubes for the Esmond Street Bridge.

Approximatley two and a half feet of abutment scour was predicted to occur on each abutment at the Esmond Street Bridge due to the 2010 flood flow. The results are presented in Table 24.

Table 24: Froehlich's Abutment Scour Prediction for the Esmond Street Bridge During the 2010 Flood

Abutment	K_1	K_2	L (ft)	L' (ft)	A_e (ft ²)	Q_e (ft ³ /s)	V_e (ft ² /s)	y_a (ft)	Fr	y_s (ft)
Right (East)	0.82	0.88	5	1.5	23.22	90.48	3.90	4.64	0.32	6.98
Left (West)	0.82	0.99	6.24	2.05	11.95	22.71	1.90	1.92	0.24	3.45

Using the NCHRP 24-20 equation (Equation 10), 4.14 feet of scour was predicted to occur on the east abutment and 7.69 feet on the west abutment as summarized in Table 25.

Table 25: NCHRP 24-20 Abutment Scour Prediction for the Esmond Street Bridge During the 2010 Flood

Abutment	q_{2f} (ft ² /s)	q_1 (ft ² /s)	y_1 (ft)	D_{50} (ft)	y_c (ft)	α_A	y_o (ft)	y_{max} (ft)	y_s (ft)
Right (East)	35.17	23.26	5.44	0.0082	7.75	1.46	7.18	11.32	4.14
Left (West)	35.17	23.26	5.44	0.0082	7.75	1.46	3.63	11.32	7.69

6.5 DISCUSSION

While visiting the bridge site, a slight scour depression was noticed at the start of the east abutment. This feature existed in the past, extending the length of the abutment with a depth of about three feet. There was a broken wire with entangled debris that diverted flow away from the abutment and allowed for accretion of sediment along the southern portion of the east abutment. No inspection reports following the 2010 floods indicated any evidence of scour, Table 26. It can be said that the eastern abutment favors flow and often exhibits scour features along its length.

Table 26: Esmond Street Bridge Summary of Scour

	Predicted Scour based on 2010 flow	Largest observed scour and associated year
Contraction Scour (ft)	0.31	-
Right (East) Abutment Scour (ft)	6.98	3, 1995
Left (West) Abutment Scour (ft)	3.45	-

Small scour predictions were calculated for the Esmond Street Bridge during the 2010 flood flow. These small predictions could be due to the intensity of the flood incident being small in the northern part of Rhode Island, with only up to a 25-year recurrence interval occurring at the downstream stream gauge (AECOM 2013).

CHAPTER 7

WEEKAPAug BRIDGE (NO. 099701), WESTERLY, RI

Located in Westerly, RI, Bridge 099701, the Weekapaug Bridge seen in Figure 64 and Figure 3, crosses the Weekapaug Breachway, connecting the Winnapaug Pond to Block Island Sound. Originally built in 1936, Bridge 997 was rebuilt in 2011. The bridge is constructed of reinforced concrete and masonry, carrying two lanes of traffic 124 feet across the waterway (Figure 64). The primary bed material in the inlet is sand with a median grain size of 9.5 millimeters. The bed also consists of rip-rap, gravel, and shells. There is a strong tidal influence at the site and any flow into the pond is negligible in comparison (Fura and Mahmutoglu 2010d).

Figure 64: Location of Bridge 997, Weekapaug Bridge in Westerly, RI crossing the Weekapaug Breachway (Google).

Figure 64: Looking north at Weekapaug Bridge in Westerly, RI.

In the 1996 scour investigation, DYNLET was used to model the flow through the breachway under various storm surge scenarios. The maximum scour predicted was during the flood tide for the piers and the ebb tide for the abutments. The ebb tide controlled for the abutments because north of the bridge the breachway widens slightly,

while on the southern side, the width remains constant. For a 100 year storm surge event, 34 feet of scour would be seen on the west abutment, 31.1 feet of scour would be seen on the east abutment, and 21.4 to 21.6 feet of scour would be seen on the piers. An item 113, 3B rating was given to the bridge as it was scour critical for the 10, 50, 100, and 500-year storm surge scenarios (Fura and Mahmutoglu 2010d).

Table 27: Summary of Scour Predictions for the Weekapaug Bridge Using HEC-18 Fourth Edition (Fura and Mahmutoglu 2010d)

	10-year flow	50-year flow	100-year flow	500-year flow
Contraction Scour (ft)	2.9	5.3	7.2	6.4
Left (West) Pier Scour (ft)	13.0	13.8	14.2	14.6
Right (East) Pier Scour (ft)	13.1	13.9	14.4	14.8
Left (West) Abutment Scour (ft)	25.9	26.3	23.9	25.6
Right (East) Abutment Scour (ft)	23.1	23.5	26.8	22.4

After being rebuilt in 2011, scour was observed at the bridge in both 2012 and 2013. In 2012, there was a scour hole north of the western pier that was on the order of a 10-foot diameter 2 to 4 feet deep. During the same visit it was also noted that the depths were much greater in the center of the channel (Pechillo 2012). In the 2013 survey, evidence of scour appeared on the west abutment, just beyond the placed rip-rap. Where there was no rip-rap, scour as deep as 1.2 feet was recorded. On the north nose of the west pier, the 10-foot diameter, scour depression remained, exposing the footing of the pier for a length of 7 feet on the northeast corner. At this time, the bridge was rated 3, unstable (Pechillo 2013b).

7.1 FIELD TESTING PROGRAM

Due to large depths and high flow, majority of surveying was carried out using the research pontoon boat seen in Figure 66. To collect side scan images and bathymetry data, the EdgeTech 6205 sonar system was used off the pontoon boat. Only a small portion of data was surveyed on the northeast side of the breachway using the Trimble RTK and waders (Figure 66). A surface of the combined bathymetry datasets is seen below in Figure 67.

Figure 66: Setup of the pontoon boat for the Weekapaug Bridge survey.

Figure 66: Use of the Trimble RTK system to wade the northeast portion of the Breachway.

Figure 67: Bathymetry data at Weekapaug Breachway referenced to RI SPC, NAD 1983 in feet.

To include topography at the site, the DEM was used to create a larger surface of the breachway and the surrounding area, this surface is seen in Figure 68. Data points were populated with an increased density closer to the river to produce a surface with accuracy from which cross sections could be produced.

Figure 68: Combined topography and bathymetry at Weekapaug Breachway in Westerly, RI referenced to RI SPC, NAD 1983 in feet.

Sub-bottom information was successfully collected using the CHIRP system side mounted to the pontoon boat. The EdgeTech 6205 side scan images were used to characterize the bottom conditions at Weekapaug Breachway. It was found that the bottom consisted of sporadic boulders, with a boulder field to the northeast of the bridge. A side scan image beneath the center pass of the bridge, shown in Figure 69, depicts the two piers, the rock embankment surrounding the inlet, and the beginning of the mentioned boulder field to the northeast of the bridge. In addition, two large shadows are cast near the piers, indicating possible scour features.

Figure 69: Side scan image at Weekapaug Bridge highlighting the piers, embankments, a boulder field, and possible scour features.

The VanVeen grab sampler was used to collect grab samples of the sediment on the north side of the breachway. Samples were not collected on the southern side of the breachway due to the incoming tide and clearance under the bridge. Approximate sample locations can be seen in Figure 70. The grain size distribution was similar for samples one and two; the average of these median grain sizes was 0.001 feet (0.32

millimeters) and was used for the scour analysis. The grain size distributions can be seen in Figure 71. The samples collected within this study do not correlate to those collected in the plan of action's 1994 analysis, one possible explanation of this is due to Hurricane Sandy pushing sand from the local beach into the inlet (Fura and Mahmutoglu 2010d).

Figure 70: Location of samples collected at Weekapaug Breachway.

Figure 71: Grain size distribution at Weekapaug Breachway.

7.2 CURRENT SCOUR FEATURES

During the Weekapaug survey, the side scan allowed for instantaneous viewing of the bottom conditions at the bridge. Although side scan imagery must be interpreted by the user, shadows indicate possible scour depressions. One possible scour feature was noticed in the side scan instantaneous waterfall image at the northern side of the west pier as highlighted in Figure 69. This feature was reoccurring with each pass, indicating that it indeed was a depression and not a shadow from a rock. After processing the bathymetry data, seen in Figure 72, a deeper depth was at the north nose of the west pier with a maximum depth difference of approximately one foot over a width of eight feet.

Figure 72: Bathymetry at Weekapaug Bridge, referenced to RI SPC, NAD 1983 in feet.

A pass with the CHIRP sub-bottom profiler was done close to this location, it depicted a depression beginning at the location of the pier and extending north approximately five feet (Figure 73). Possible infilling is seen at the southern end of the depression as indicated by a softer return, however this is just an approximation.

Figure 73: Sub-bottom profile on the western span of Weekapaug Bridge.

On the eastern abutment, two areas of increased depths existed, one to the north and one to the south. Differences in depth of approximately 1.75 to 1.5 feet occur throughout this span. There are also many rocks and boulders along the eastern side of the bridge which create scour depressions.

To the north, a possible scour feature had been previously indicated in the instantaneous side scan imagery (Figure 69), however this was ruled out by the bathymetry indicating shallow depths at the end of the east pier in addition to the comparison of side scan images in Figure 74. In the pass on the left, the vessel was traveling beneath the center span while on the right the vessel was traveling beneath the eastern span. The left image displays a large shadow at the nose of the pier which indicates a possible scour feature. However, when passing the feature from the other side, no shadow is displayed, making one unsure of the feature. It was confirmed with the bathymetry that there is most likely a boulder at the north nose of the east pier, not a scour feature. This boulder is also seen in the sub-bottom profile in Figure 75. It is possible that these boulders could be the cause of the scour features and large depth differences.

Figure 74: Side scan images traveling under the center span (left) and the eastern span (right), indicating a rock pile at the nose of the east pier due to lack of shadow in the pass under the eastern span.

Figure 75: Sub-bottom profile on the eastern span of Weekapaug Bridge.

7.3 HEC-RAS MODELING

In order to properly model the Weekapaug Breachway an unsteady analysis had to be performed because it is tidally influenced. Similar to the steady flow analysis, cross sections were first input into HEC-RAS from ArcGIS. A total of 25 cross sections were created extending beyond the breachway as depicted in Figure 76.

Figure 76: Cross sections for Weekapaug Bridge in Westerly, RI.

Next, the bridge geometry was input into the model. Cross section 1700 was used as the upstream cross section for analyzing scour at the bridge. This cross section was chosen based on the bridge's width of 32 feet and a flood tide controlling the maximum flow during Hurricane Sandy. The starting cross section 2392, upstream cross section 1700, bridge cross section 1655, and final cross section 100 are pictured below in Figure 77, Figure 78, Figure 79, and Figure 80. It is easily seen that the channel does not change greatly from north to south. A full summary of cross sections can be found in Appendix F. Flooding is depicted in the area just west of the breachway; this is valid considering that during Hurricane Sandy, much of Misquamicut was closed due to extreme flooding. The dunes along Atlantic Avenue were breached, allowing for the surge to inundate the main strip alongside the breachway (Hanrahan 2012). In addition, the northern cross

sections of the model extend west into Winnapaug Pond's estuary, which contains grass and reeds at the southern portion.

Figure 77: Weekapaug cross section 2392 at the entrance of Weekapaug Breachway.

Figure 78: Weekapaug cross section 1700 used as the upstream cross section for scour calculations.

Figure 79: Weekapaug cross section 1655 indicating that the Weekapaug Bridge was not overtopped.

Figure 80: Weekapaug cross section 100, located at the north side of the breachway. This cross section has extreme flooding on both sides of the breachway, however this is expected as the surrounding area is estuarine.

In order to run an unsteady model to examine the flow conditions during Hurricane Sandy, two boundary conditions are needed. At the northern end of the pond a temporary USGS Sensor was deployed during Hurricane Sandy. This gauge was a pressure transducer, allowing for the barometric pressure and water level to be recorded over the course of the storm (USGS 2016d). On the other end of the breachway, a time series was taken from an ADCIRC+SWAN model (Marissa Torres, personal communication, October 2016) which simulated the winds, water levels, and waves for the duration of Hurricane Sandy in Rhode Island. A comparison of the two boundary signals to the tidal signal without storm surge can be seen in Figure 81. Both signals were input as stage hydrographs in HEC-RAS and drove the flow at each end of the model.

Figure 81: Comparison of the boundary conditions to the tidal signal at Weekapaug Breachway during Hurricane Sandy.

To check the accuracy of the model, the maximum velocities obtained in HEC-RAS modeling a tidal signal were compared to the Coastal Engineering Manual's (CEM) Inlet Hydrodynamics for the maximum cross-sectional averaged velocity (USACE 2002). A maximum velocity of 2.87 feet per second occurred in the HEC-RAS Weekapaug Model during a tidal model at the breachway. The two tidal signals seen in Figure 82 were used as the boundary conditions of the model. Using the CEM's methodology, the maximum velocity was approximated to be approximately 2.93 feet per second. A summary of the CEM methodology and inputs can be found in Appendix F.

Figure 82: Tidal signal used to validate Weekapaug Breachway velocities.

7.4 SCOUR CALCULATIONS

To complete the scour analysis, a level two tidal scour analysis was done, meaning the hydraulic variables obtained in the HEC-RAS unsteady analysis of Weekapaug Breachway were used with the riverine scour equations presented in HEC-18, see Appendix F. First, the maximum flow condition was found in the data set, as due to its unsteady nature, a series of time sets must be analyzed over the duration of the event. The maximum flow occurred on October 30, 2012 at 12:00, this time step was used to calculate scour at the Weekapaug Bridge.

With the maximum flow of Hurricane Sandy known, the critical velocity was calculated to observe if clear-water or live-bed conditions were at the site during the storm. Using cross section 1700, live-bed conditions were predicted to occur during Hurricane Sandy as presented in Table 28.

Table 28: Weekapaug Bridge Contraction Scour Predictions for Hurricane Sandy

y (ft)	D ₅₀ (ft)	K _u	V _c	V	Clear-water or live-bed?
9.76	0.001	11.17	1.66	6.55	Live-bed

To analyze the contraction scour, Laursen's 1960 modified equation (Equation 2) was used. It predicted that 3.27 feet of contraction scour would occur at the Weekapaug Bridge during Hurricane Sandy. A summary of values can be found in Table 29.

Table 29: Live-Bed Contraction Scour for Weekapaug Bridge

y ₁ (ft)	y _o (ft)	Q ₁ (ft ³ /s)	Q ₂ (ft ³ /s)	W ₁ (ft)	W ₂ (ft)	k ₁	y ₂ (ft)	y _s (ft)
9.76	9.7	8265.02	8239.57	169.41	112.4	0.69	10.03	3.27

In order to analyze pier scour, the HEC-18 and FDOT pier scour equations (Equation 5 and Equation 6) were used for both piers. The HEC-18 equation predicted 8.48 feet of scour to occur on the east pier and 8.43 feet of scour to occur on the west pier. The Florida DOT equation predicted 6.74 and 6.76 feet of scour to occur on the east and west piers respectively. The summary of these analysis can be found in Table 30 and

Table 31.

Table 30: HEC-18 Pier Scour for Weekapaug Bridge

Pier	y ₁ (ft)	K ₁	a (ft)	L (ft)	K ₂	K ₃	V ₁ (ft/s)	Fr	y _s (ft)
Right (East)	10.14	1.0	4	34.83	1.0	1.1	7.78	0.43	8.48
Left (West)	9.68	1.0	4	34.83	1.0	1.1	7.78	0.44	8.43

Table 31: FDOT Pier Scour for Weekapaug Bridge

Pier	uc*	y ₁ (ft)	V _c (ft/s)	V _{lp} (ft/s)	K _{sf}	a* (ft)	f ₁	f ₃	y _{s-c} (ft)	y _{s-lp} (ft)	y _s (ft)
Right (East)	0.05	10.14	1.25	10.85	1	4	0.896	0.480	4.30	7.88	6.74
Left (West)	0.05	9.68	1.25	10.59	1	4	0.890	0.480	4.28	7.84	6.76

To predict abutment scour with Froehlich's equation (Equation 9), conveyance was plotted along the east and west abutments as seen in Figure 83. Equal conveyance tubes

were found within MATLAB (Figure 84) allowing for the length of live flow across each abutment to be determined.

Figure 83: Conveyance curves across abutments at Weekapaug Bridge.

Figure 84: Conveyance tubes across the abutments for Weekapaug Bridge

Four tubes were used for both abutments to describe the length of active flow. The left abutment length was determined to be 5.8 feet and the right abutment length was

determined to be 9.7 feet. Scour was calculated with Froehlich's abutment scour to be 6.91 feet on the right abutment and 5.63 feet on the east abutment. The values used in this calculation can be found in Table 32.

Table 32: Froehlich's Abutment Scour Prediction for Weekapaug Bridge During Hurricane Sandy

Abutment	K_1	K_2	L (ft)	L' (ft)	A_e (ft ²)	Q_e (ft ³ /s)	V_e (ft ² /s)	y_a (ft)	Fr	y_s (ft)
Right (East)	1.0	1.0	21.56	9.7	53.69	122.99	2.29	2.49	0.26	6.91
Left (West)	1.0	1.0	16.83	5.8	38.77	82.9	2.14	2.30	0.25	5.63

Abutment scour was also calculated using the NCHRP 24-20 equation (Equation 10). Using this method and the values in Table 33, scour was estimated to be 7.14 feet on the east abutment and 6.98 feet on the west abutment.

Table 33: NCHRP 24-20 Abutment Scour Prediction for Weekapaug Bridge During Hurricane Sandy

Abutment	q_{2f} (ft ² /s)	q_1 (ft ² /s)	y_1 (ft)	D_{50} (ft)	y_c (ft)	α_A	y_o (ft)	y_{max} (ft)	y_s (ft)
Right (East)	73.31	48.79	5.09	0.0010	7.22	1.7	5.13	12.27	7.14
Left (West)	73.31	48.79	5.29	0.0010	7.22	1.7	5.29	11.62	6.98

7.5 DISCUSSION

After Hurricane Sandy in 2012, a large scour depression developed at the northern side of the west pier. Extending ten feet in diameter and four feet deep, the footing of the pier was exposed. Although in-filled, this feature still exists with a diameter of approximately eight feet and a depth of one foot. The scour previously seen on the west abutment was not seen on the abutment at this visit, however possible scour depressions were noted on the east abutment. A summary of the maximum scour seen at Weekapaug Bridge and the associated scour predictions is presented in Table 34 below.

Table 34: Weekapaug Bridge Summary of Scour

	Predicted Scour based on Hurricane Sandy flow	Largest observed scour and associated year
Contraction Scour (ft)	3.27	-
Left (West) Pier Scour (ft)	6.74	4, 2012
Right (East) Pier Scour (ft)	6.76	-
Left (West) Abutment Scour (ft)	6.91	1.2, 2013
Right (East) Abutment Scour (ft)	5.63	~1.75, 2016

It would have been beneficial to collect additional data on the eastern span of the bridge as only one pass was completed due to clearance restrictions with the rising tide. This was not prioritized as scour features had not been seen on this abutment in the past, also side scan data did not indicate any scour features while surveying. The data pass under the eastern span had some discrepancies and did not line up in side scan images seamlessly, this would call for additional data to be collected. In addition, samples could be collected beneath the bridge and on the southern side of the bridge to better understand the grain size distribution at Weekapaug Bridge.

CHAPTER 8

DISCUSSION OF RESULTS AND SENSITIVITY ANALYSIS

This chapter will present a summary of the scour analyses for the 2010 floods and Hurricane Sandy along with a sensitivity analysis of certain parameters within the scour analysis. The parameters that will be studied include the median grain size, the angle of attack, Froehlich's length of active flow term, and the Manning bottom friction coefficients.

8.1 DISCUSSION OF RESULTS

In order to judge the performance of the scour prediction equations, the scour predictions of the maximum flow event were compared to the present and historical scour observations. The HEC-RAS models created for the 2010 floods and Hurricane Sandy provided values of flow that would occur during the flood at each bridge site. These flow conditions were then used in the HEC-18 scour prediction equations to provide accurate scour predictions for the flood scenario. It is arguable that selecting one value to represent a physical condition at a bridge is not realistic. For example, the sediment at each bridge is solely expressed in the calculations as the median grain size which often does not account for all the bottom characteristics at a bridge. If a range of values was realistic for an input variable, then the associated scour was calculated with this range. These additional analyses for ranging variables will be discussed in the sensitivity analysis.

Figure 85 depicts a comparison of the predicted values to the values of scour observed at each bridge site. Estimations of contraction scour, pier scour, and abutment scour are presented based on their equation.

Figure 85: Comparison of the predicted scour to the largest observed scour at each scour critical bridge studied.

For approximately half of the features, no scour was observed to have existed. These values may or may not be true, as it is possible that a scour hole once existed at these features but was in-filled without being noticed. If paleo scour features were found for a specific bridge feature, the data points associated with that feature would shift toward the line of equity. Observed values of contraction scour are seen to be zero throughout. Contraction scour is hard to measure without a complete survey surface of the site to estimate the difference in the bed elevations. During the 2016 surveys, there was no evidence of contraction scour at the bridges. A trend was noted that the Florida DOT equation (Equation 6) produced smaller scour predictions, as seen in Figure 85. Also, for abutment scour, the Froehlich equation (Equation 9) produced results less than that

of the NCHRP equation (Equation 10) however, this is due to the NCHRP equation's inclusion of contraction scour in its estimation. Overall, for the bridges studied the predicted scour exceeds the observed scour by over two feet for the majority of the bridge features.

It should be noted that the methods used in this study may not be appropriate when designing for scour, design analyses must be conservative and often including factors of safety. However, based on this study, the scour prediction equations prove to overestimate the severity of scour at a bridge due to a large flooding event; this in combination with conservative assumptions by a design engineer could create extremely high scour predictions that may or may not be realistic for the bridge of interest. Due to the conservative nature of design practices, bridges that are listed as scour critical based solely on the scour equations should be evaluated on a case by case basis with their scour history to determine if they are indeed scour critical.

8.2 SENSITIVITY ANALYSIS

This section will be subdivided based on the variable studied: median grain size, angle of attack, Froehlich's length of active flow, or Manning bottom friction coefficient. Each section will look into which equation it impacts and the influence of the variable on the equation. Acceptable ranges were chosen based on past reports, knowledge of the conditions at the bridge, or the maximum feasible value. This was done with the idea that these conditions may have been present during the time of flooding and thus represent the possible variation in scour predictions.

8.2.1 MEDIAN GRAIN SIZE

For each bridge the past and present grain size distributions, presented in Chapters four through seven, were examined to select appropriate grain size bounds seen in Table 35. Based on knowledge of soil conditions at each bridge, the appropriate samples were chosen to represent the upper and lower limit, notes regarding the grain size distribution and the representative sample can be found in Table 35. The equations effected by the change in grain size include the critical velocity equation (Equation 1), the contraction scour equations (Equation 2 and Equation 3), the FDOT pier scour equation (Equation 6), the pier scour in coarse bed materials equation (Equation 7), and the NCHRP 24-20 abutment scour equation (Equation 10).

Table 35: Summary of Median Grain Size's Used in Sensitivity Analysis

	Lower Limit (LL)	Analysis	Upper Limit (UL)	Notes
Kenyons Bridge	0.0012ft (0.38mm)	0.039ft (11.8mm)	0.072ft (22mm)	Figure 20 LL:S3 UL:S1
First Barberville Bridge	0.0034ft (1.05mm)	See upper and lower limits	0.263ft (80mm)	Figure 38 LL:S1 UL:S1 rip-rap
Esmond Street Bridge	0.0049ft (1.5mm)	0.0082ft (2.5mm)	0.076ft (23mm)	Figure 54 LL:S4 UL:S5
Weekapaug Bridge	0.001ft (0.29mm)	0.001ft (0.32mm)	0.292ft (89mm)	Figure 71 LL:W1 UL:1994-S1

Median Grain Size (ft)	1	0.1	0.01	0.001
Kenyons Bridge				
First Barberville Bridge				
Esmond Street Bridge				
Weekapaug Bridge				

The critical velocity determines if live-bed or clear-water conditions are at the bridge site. For each bridge, the lower median grain size produced a live-bed condition, while the larger median grain size produced clear-water conditions. High critical velocities represent the clear-water conditions in Figure 86.

Figure 86: Comparison of changes in the critical velocity and contraction scour based on median grain size variation.

For the large grain sizes with clear-water conditions, zero or close to zero contraction scour was predicted. Meanwhile for the live-bed conditions, large amounts of scour were predicted at Kenyons Bridge and the First Barberville Bridge (Figure 86). This could be due to the increased contraction between the upstream cross sections and the bridge and the associated flow within the channel at these sites. Based on the lower limit grain size at Weekapaug Bridge, it would be expected that an increased spread in contraction scour would have occurred however, the upstream depth, flow, and channel width was very similar to the values at the bridge, decreasing the influence of the median grain size on the contraction scour.

Next, the effects of median grain size on pier scour were explored. Both the FDOT and pier scour in coarse bed materials equations (Equation 6 and Equation 7) were

analyzed (Figure 87). Only two of the bridge sites have pier features: The First Barberville Bridge has one center pier while Weekapaug Bridge has a left and a right pier. Weekapaug's left pier is depicted by blue and the right pier is depicted by red.

Figure 87: Changes in pier scour based on variations in the median grain size. Green represents a center pier, blue represents a left pier, and red represents a right pier.

Very minute changes were seen in pier scour predictions for the left and right piers. The FDOT equation (Equation 6) produced changes of about 1.3 feet for the First Barberville Bridge, while changes of 0.02 feet were seen for Weekapaug Bridge. Only the upper bounds allowed for the pier scour in coarse bed material equation (Equation 7) to be used, but the use of this equation decreased the pier scour by over six feet to values less than half a foot.

The NCHRP 24-20 equation (Equation 10) was evaluated next, and results are seen in Figure 88. This equation is dependent on if live-bed or clear-water conditions exist at the bridge. When clear-water conditions exist the median grain size is used to find the depth to which the scour will erode to, meanwhile if live-bed conditions are present only one value of scour is calculated.

Figure 88: Effects of grain size on the NCHRP abutment scour equation.

The NCHRP 24-20 abutment scour equation (Equation 10) is greatly affected by the variation in median grain size. With the exception of the Esmond Street Bridge where flow conditions were moderately low, changes were seen to be two-fold nearly doubling between the minimum and maximum median grain sizes selected, as depicted by Figure 88. The largest differences arose at Kenyons Bridge and the First Barberville Bridge and seem to have arose due to large unit flow differences in the upstream cross sections versus at the bridge.

In this study a large range of grain sizes were seen at each bridge site which can have great influence on many of the scour prediction equations. A summary of the average differences in scour for a particular equation is presented in Table 36. The critical velocity and contraction scour at a bridge site is highly dependent on the grain size to determine if live-bed or clear-water conditions exist. Beyond this, channel geometry and flow conditions often control. For pier scour, massive differences are seen when the pier scour in coarse bed materials equation (Equation 7) can be employed. Similar to contraction scour, the largest differences in abutment scour were seen at the

bridges with the highest differences between the upstream flow and the flow at the bridge.

Table 36: Summary of average change in scour based on variation in grain size.

	Contraction Scour	FDOT Pier Scour	NCHRP 24-20 Abutment Scour
Average change in scour (ft)	5.75	0.87	5.90

8.2.2 ANGLE OF ATTACK

The angle of attack is the angle at which the flow hits the piers or abutments of a bridge. This parameter has effects on the HEC-18 pier scour equation (Equation 5), the FDOT pier scour equation (Equation 6), and the Froehlich abutment scour equation (Equation 9). Angles were varied from negative 20 degrees to 20 degrees in 10 degree increments. Due to symmetry in piers, only the three positive values were needed for the analysis of the pier scour equations. For Froehlich's abutment scour equation, all five angles were used, the negative values indicate an abutment pointing downstream with the flow.

First, the angle of attack was analyzed for the HEC-18 and FDOT pier scour equations (Equation 5 and Equation 6). The results of this analysis can be seen in Figure 89.

Figure 89: The effect of angle of attack on pier scour. Green represents a center pier, blue a left pier, and red a right pier.

The angle of attack had a greater variation in values for the FDOT equation (Equation 6) as compared to the HEC-18 equation (Equation 5). This is due to the HEC-18 equation including the angle in a correction factor, while the FDOT equation incorporates it based on the area effected. As angle of attack increases so does pier scour. In this variation of angle of attack, scour doubled for all piers with an angle of 20 degrees. The scour became nearly five times the original prediction for the FDOT equation at the Weekapaug Bridge piers.

The angle of the flow was also varied to analyze Froehlich's abutment scour equation (Equation 9).

Figure 90: Variation of angle of attack on Froehlich's abutment scour equation.

Almost no changes were seen in predicted scour amounts using Froehlich's equation (Equation 9) as seen in Figure 90. An average difference of 0.24 feet of scour was seen between -20 and 20 degrees for all of the bridges, making the angle of attack on abutments for Froehlich's equation negligible.

To summarize, the angle of attack has greatest influence on the FDOT pier scour equation (Equation 6) and very little influence on Froehlich's abutment scour equation (Equation 9) as presented in Table 37. It is very difficult to measure this value in the field and predict its direction under high flow conditions without complex two-dimensional modeling. If analyzing a pier, caution should be taken when choosing the angle of attack.

Table 37: Summary of average change in scour based on variation in angle of attack.

	HEC 18 Pier Scour	FDOT Pier Scour	Froehlich Abutment Scour
Average change in scour (ft)	12.01	20.21	0.24

8.2.3 FROEHLICH'S LENGTH OF ACTIVE FLOW

Next, the length of active flow in Froehlich's equation (Equation 9) was evaluated. In this study, conveyance at each bridge was analyzed in MATLAB in order to find the length of active flow. To be conservative, many consultants use the full length of flow across an abutment or across the embankment when employing Froehlich's abutment scour equation. The variation of the length can be seen in Figure 91, these values represent the value established in the conveyance analysis and the more conservative total length of flow along the embankment.

Figure 91: Variation of active length and its impacts on Froehlich's abutment scour equation.

The smallest change that occurred was 0.94 feet of additional scour due to an increase in length of 4.2 feet at the Esmond Street Bridge, this is most likely due to the small velocities present at this site. Meanwhile, the largest variation in scour was seen at Kenyons Bridge; here, there was an increase in length of 329 feet due to complete inundation. Scour thus increased by 4.2 feet. Overall, an average change of 2.4 feet was observed, Table 38. Although this equation is also reliant on flow conditions at the

abutment, the length of flow across an abutment is a key factor in the magnitude of scour predicted by Froehlich's abutment scour equation (Equation 9).

Table 38: Summary of average change in scour based on variation in Froehlich's L' .

	Froehlich Abutment Scour
Average change in scour (ft)	2.40

8.2.4 MANNING BOTTOM FRICTION COEFFICIENT

The Manning bottom friction coefficient is an assumption made in the HEC-RAS model. It is based on bottom conditions at each site. For a clean, straight channel with some stones and weeds, values range from 0.03 to 0.04. If the river is winding and there are increased stones, values range from 0.045 to 0.065. Likewise, for a flood plain with scattered brush and heavy weeds, values range from 0.035 to 0.07, but if there is dense brush the values increase to 0.16 (Brunner 2016). Because this parameter lies within the model and effects the flow, any adjustments of this parameter will cause fluctuations in contraction scour, pier scour, and abutment scour.

The Manning coefficient values were selected based on local knowledge of each site and confirmed with published FEMA values for the three riverine bridges. To pick realistic variations in the Manning coefficients the FEMA reports were used to create upper and lower bounds based on their ranges for the river of interest, as seen in Table 39. For Weekapaug Bridge, no FEMA report was published so a standard 0.01 was added or subtracted from the Manning coefficients used in the original analysis. These adjustments were applied uniformly across each model.

Table 39: Summary of Manning bottom friction coefficient variations

	Kenyons Bridge	First Barberville Bridge	Esmond Street Bridge	Weekapaug Bridge
Original Range	0.075-0.033	0.14-0.035	0.07-0.037	0.05-0.03
Lower Limit	-0.005	-0.01	-0.012	-0.01
Upper Limit	+0.007	+0.02	+0.005	+0.01
Source	(Zarriello et al. 2014a)	(Zarriello et al. 2014a)	(Zarriello et al. 2014b)	

Manning Bottom Friction Coefficients

Bridge	n value (approximate range)
Kenyons Bridge	0.028 - 0.040
First Barberville Bridge	0.025 - 0.055
Esmond Street Bridge	0.025 - 0.042
Weekapaug Bridge	0.020 - 0.040

First, the changes that the Manning coefficient had on the critical velocity and contraction scour were investigated. Figure 92 depicts the results.

Figure 92: Variation of critical velocity and contraction scour to variation in Manning coefficients

It is easily seen that the Manning coefficient has insignificant variation on the critical velocity with changes less than 0.1 feet per second. For contraction scour, the Manning

coefficient played a larger role, with changes up to 1.4 feet at the First Barberville Bridge.

Next the effects of the manning coefficient on pier scour at the First Barberville Bridge and Weekapaug Bridge were investigated. The effects of the Manning coefficient on pier scour can be seen in Figure 93. At Weekapaug Bridge the Manning coefficient had a range of 0.02, while the First Barberville bridge had a range of 0.03.

Figure 93: Comparison of the variation of pier scour based on a shifting Manning coefficient. Green represents a center pier, blue represents a left pier, and red represents a right pier.

At the First Barberville Bridge an average variation of 0.52 feet of pier scour was calculated. Meanwhile, at Weekapaug Bridge an average of 2.06 feet of increased scour was found. This is most likely due to either the smaller grain size at Weekapaug Bridge, or the effects of the bottom friction on the unsteady model and its maximum stages. Larger fluctuations, approximately 0.5 feet, were seen in the HEC-18 pier scour equation (Equation 5) as compared to the FDOT equation (Equation 6).

Finally, the effects of the Manning coefficient on abutment scour was studied, seen in Figure 94.

Figure 94: Abutment scour comparison with the variation of the Manning bottom friction coefficient.

The variations seen in Froehlich equation (Equation 9) were slightly larger than those seen in the NCHRP 24-20 equation (Equation 10), as shown in Figure 94. Most variations were under two feet with the exception of the Esmond Street Bridge where a difference of 4.6 feet of scour was predicted to occur on the right abutment with Froehlich's equation and an average difference of just 3.2 feet was predicted for both abutments using the NCHRP equation. Smaller variations in scour were seen at the other three bridges due to the flow conditions being larger and dominating the scour predictions

In conclusion, the Manning coefficient had the largest impact on the HEC-18 pier scour equation (Equation 5) and the Froehlich abutment scour equation (Equation 9), Table 40. Meanwhile, very little variations were seen in the critical velocity (Equation 1) and the associated contraction scour (Equation 2 or Equation 3) values while changing the Manning bottom friction coefficients. This analysis determines the accuracy of the three riverine HEC-RAS models as the Manning coefficient's were the only unknown inputs into the model.

Table 40: Summary of average change in scour based on variation in Manning coefficient.

	Contraction Scour	HEC- 18 Pier Scour	FDOT Pier Scour	Froehlich Abutment Scour	NCHRP 24-20 Abutment Scour
Average change in scour (ft)	0.57	1.48	1.10	1.59	0.94

CHAPTER 9

CONCLUSION

The Federal Highway Administration requires that a detailed scour analysis be performed using the HEC-18 standards on bridges over water. As a result of this analysis, Rhode Island has 127 bridges listed as scour critical, which means that a detailed plan of action must be developed for each of these bridges, and RIDOT must monitor and maintain each bridge. It was suggested by a report following historic floods in 2010 that the methodology presented in the fourth edition of HEC-18 to predict scour is often over conservative throughout the state of Rhode Island and that scour should be reevaluated with the recently released fifth edition of HEC-18 (AECOM 2013).

Given this problem, the objective of this study was to evaluate the current scour methodology for the following scour critical bridges in Rhode Island:

- Kenyon's Bridge (No. 020601) in Charlestown, RI
- First Barberville Bridge (No. 040101) in Hopkinton, RI
- Esmond Street Bridge (No. 094801) in Smithfield, RI
- Weekapaug Bridge (No. 099701) in Westerly, RI
- Nannaquacket Bridge (No. 012601) in Tiverton, RI.

To estimate information such as flow, velocity, and depth, each bridge site was modeled using HEC-RAS. Riverine sites were modeled using conditions from the 2010 flooding event, and Hurricane Sandy was used to model the marine bridge. Scour at

each of the sites was predicted using the equations presented in the fifth edition of HEC-18. The predictions were then compared to both the past and present scour observations. Finally, the sensitivity of the median grain size, the angle of attack, the length of active flow, and the Manning coefficient on the HEC-18 equations was evaluated and upper and lower limits of the predictions were examined.

The field testing program utilized various technologies to collect data on bridges throughout the state. Bottom conditions were characterized with a Nikon Coolpix AW100 waterproof camera and an EdgeTech 6205 side scan sonar system. Grab samples were collected at each site in order to properly characterize the median grain size. The bathymetry was also collected by the EdgeTech 6205 sonar system and used to identify scour features and to map the local channel for the creation of detailed cross sections in HEC-RAS. For the riverine bridges which were not accessible by boat, bathymetry was collected with the Trimble RTK system and a Topcon total station. The Trimble and Topcon systems were also used to collect local topography and bridge geometry. The bathymetric and topographic surveys were combined with the RI DEM to provide complete coverage of the topography surrounding the bridge from which cross sections were cut for hydraulic modeling. From the detailed field work, scour features were found at the Esmond Street Bridge and Weekapaug Bridge. These features had been observed in the past and have been in-filled since their last measurements.

Steady one-dimensional HEC-RAS models of the 2010 event were created and analyzed at the three riverine sites. For the tidal site, a more complex unsteady one-dimensional model was created for Hurricane Sandy. The scour analyses were

completed based on the HEC-RAS models and it was found that scour was over predicted at each bridge feature. The sensitivity analysis revealed the greatest sensitivity in the angle of attack on a pier, displaying an maximum increase of 19 feet of scour over 20 degrees; however, the angle of attack had little effect on abutment scour at the selected bridge sites with an average change of 0.24 feet. The smallest average change across all features amongst all the bridges was seen in the sensitivity of the Manning coefficient with an average change of 1.1 feet as depicted in Table 41.

Table 41: Summary of the average variation of scour based on the sensitivity analysis.

	Grain Size	Angle of Attack	Froehlich's Length of Active Flow	Manning's Coefficient
Average scour change (ft)	5.3	10.8	2.4	1.1

Moving forward, this study has generated a number of paths for future research.

- Provide sub-bottom profiles at riverine bridges in order to characterize the maximum scour depth. The scour observations in this report are based on physical observations on a particular date. To know the complete scour history, paleo scour features must be analyzed with technologies such as ground penetrating radar.
- Improve modeling techniques to those beyond one-dimensional modeling.
- For tidal bridge sites, reevaluate scour using the methodology presented in HEC-25, "Highways in the Coastal Environment."
- Develop better methods for characterizing grain size distributions. It would be beneficial to include a range to describe the sediment at a bridge site. In addition, there are limitations to grab sampling as that is only the sediment

present at one discrete location to a certain depth which may or may not be characteristic to the entirety of the sediment at the bridge site of interest. It is also common that larger gravels and rocks may lie atop or beneath the sediment from which grain size analysis were completed. These larger features armor the bed, thus decreasing scour beneath its layer.

- Perform a more comprehensive study of the 2010 floods in Rhode Island. In particular, it would be interesting to study the relationship between flooding, observed scour, and bridge performance. This study can then be used in combination with the HEC-18 risk based approach for determining if a bridge's scour critical rating should be removed.
- Persistent modeling and monitoring of a characteristic riverine and tidal bridge is suggested. Based on the bridges chosen for this study, Kenyons Bridge and Weekapaug Bridge would be recommended.

From these future research matters, suggestions on how to effectively predict scour in the state of Rhode Island may be more transparent. Currently, great variations are easily obtained based on the parameters provided in the HEC-18 scour prediction equations, allowing for increasingly over conservative prediction values for bridges throughout the state.

APPENDICES

APPENDIX A: NANNAQUAKET BRIDGE (NO. 012601), TIVERTON, RI

This Appendix provides an overview of the field testing program and its results at Nannaquacket Bridge.

Bridge 126, the Nannaquacket Bridge crosses the Quaket River connecting the Nannaquacket pond with the Sakonnet River in Tiverton, RI, seen in Figure 95 and Figure 3. The bridge was constructed in 1958 and carries two lanes of traffic over a 61-foot span (Figure 96); it was designed to have the piers and abutments aligned with the flow, giving an angle of attack of zero. The flow at this location is primarily tidal with flood tides moving southerly into the pond and ebb tides moving north toward the Sakonnet River. Local velocities at the bridge are higher than one may expect due to the extreme contraction that occurs at the mouth of the pond, with the worst flow scenarios occurring due to storm surge up the Sakonnet River (Fura and Mahmutoglu 2010e).

Figure 95: Location of the Nannaquacket Bridge crossing the Quaket Inlet in Tiverton, RI (GOOGLE)

Figure 96: The Nannaquacket Bridge in Tiverton, RI.

A 1995 investigation found the median grain size of 3.2mm on the north side of the bridge and 2.6mm on the southern side of the bridge. The divers report indicated

boulders, gravel, and sand at the site with the boulders ranging from six to twelve inches. No scour was recorded to be seen in 1995. FastTABS, a wetland hydrodynamic modeling program, was used to model the storm surge that would create the highest flow scenario at the bridge as the normal river flow is minimal in comparison to the storm surge. It was estimated that 31.3 feet of scour would occur on the east abutment with 8.4 feet on the east pier and 30.3 feet on the west abutment with 7.5 feet on the west pier for a 500-year event. In the report, it was mentioned that the scour may be over estimated due to the tidal flow, however it is hard to remove it from the scour critical list due to the magnitude of these values of scour. Bridge 126 was given an item 113 rating of 3A, scour critical moderate, at this time as it was only scour critical for the 100 and 500-year events (Fura and Mahmutoglu 2010e).

The 2010 plan of action indicated that the bridge was scour critical for the 10, 50, 100, and 500-year storms. For an 100-year event, 27.5 feet of scour would be seen on the west abutment, 26.7 feet of scour on the right abutment, 6.8 feet of scour on the west pier, and 7.6 feet of scour on the east pier was predicted, Table 42. An inspection in 2009 noted three to four feet of scour on the western side and southern nose of the east pier, exposing the footing (Fura and Mahmutoglu 2010e). In the 2013 inspection it was noted that the scour seen in 2009 was still present, the hole on the west side of the east pier was approximately 20 feet long and 2.5 feet deep, exposing the footing at this location. At this time, the bridge was given a 113 rating of 3, or scour critical, unstable (Pechillo 2013c). Following the 2016 inspection, the scour remained and the bridge remained at an item 113 rating of 3 (Wadsworth and Benn 2016).

Table 42: Summary of Scour Predictions for the Nannaquaket Bridge Using HEC-18 Fourth Edition (Fura and Mahmutoglu 2010e)

	10-year flow	50-year flow	100-year flow	500-year flow
Contraction Scour (ft)	0.0	0.0	1.1	1.2
Left (East) Pier Scour (ft)	6.0	6.1	6.5	7.2
Right (West) Pier Scour (ft)	5.4	5.4	5.7	6.3
Left (East) Abutment Scour (ft)	23.3	24.0	25.6	30.3
Right (West) Abutment Scour (ft)	24.0	24.8	26.4	31.3

A.1 FIELD TESTING PROGRAM

The pontoon boat was used to complete surveys at the Nannaquaket Bridge in Tiverton, RI due to the large depths present at the site. From the pontoon boat, the EdgeTech system was used. The side scan sonar allowed for bottom conditions to be classified and for any noticeable scour holes to be identified as seen in Figure 97. The EdgeTech system also allowed for bathymetry data throughout the channel to be collected and combined with the DEM to create a surface at Nannaquaket, shown in Figure 98.

Figure 97: Side scan image at Nannaquaket Bridge highlighting the piers, boulders, and a possible scour feature.

Figure 98: Collected bathymetry combined with the DEM to create a full surface in Tiverton referenced to WGS 1984, UTM zone 19N in feet.

The bathymetry at the bridge was extracted to make a smaller surface from which any scour features could be identified (Figure 99). What had appeared to be a scour hole in the side scan imagery, did not agree with the bathymetry. At the southern end of the east pier, an area of increased elevation exists. Based on the bathymetry, depth differences of half a foot exist along the north-east portion of the east pier, this would be the only scour observed with the bathymetry data.

Figure 99: Bathymetry at Nannaquaket Bridge referenced to WGS 1984, UTM zone 19N in feet..

Sub-bottom information was collected by the CHIRP system side mounted on the pontoon boat. A pass beneath the center span of the bridge along the east pier is presented in Figure 100. A large bump is seen at the surface of the sea floor just beyond the bridge. In this area, the hard bottom return seen throughout the images is not as prominent, indicating a change in sediment. This could indicate an in-filled scour hole, however it is not clearly indicated in the sub-bottom image and can not be verified. This feature corresponds to the bathymetry at the southern end of the east pier.

Figure 100: Sub-bottom profile along the western side of the east pier.

Grab samples were collected with the VanVeen grab sampler inside Nannaquaket pond, beneath the bridge, and in the channel. Approximate locations of the samples can be seen in Figure 101. The samples were sorted by hand to remove shells from the sediment. The grain size distribution curves can be viewed in Figure 102. A median grain size of 0.081 feet (24.8 millimeters) was found by taking the average of samples two and three. This median grain size is quite different from the median grain size in 1995 which was 0.0095 feet (2.9 millimeters).

Figure 101: Locations of the sediment samples taken at Nannaquaket Bridge.

Figure 102: Grain size distribution at Nannaquaket Bridge.

Modeling efforts were not completed at this bridge due to unavailable boundary conditions. The ADCIRC+SWAN model for Hurricane Sandy in Rhode Island does not resolve the inlet into Nannaquaket Pond (Marissa Torres, personal communication, October 2016). Methodology was suggested on how to create this model but was not employed due to the approximations that had to be made. If this model were created, additional cross sections would have to be created for the extent of Nannaquaket Pond based on bathymetric charts of the pond since the bathymetric survey taken in this study did not cover the entirety of the pond. A time history of water levels northwest of the inlet would have been extracted from Torres's Hurricane Sandy model (Marissa Torres, personal communication, October 2016), input as a stage hydrograph, and act as the first boundary condition at the north end of the model. Meanwhile at the southern end, flow hydrograph comprised of zeroes would have been placed, acting as a reflective boundary.

APPENDIX B: MATLAB FUNCTION TO FIND THE LIVE LENGTH OF FLOW IN FROEHLICH'S ABUTMENT SCOUR EQUATION

In this Appendix, the MATLAB script to find equal conveyance tubes is presented. Data from Kenyons Bridge is used in the script to provide a complete example. Inputs of the script include conveyance, distance across the abutment, and a representative equation. To find the conveyance across the embankment in HEC-RAS, the bank location was iteratively changed allowing for conveyance values of the overbank to be noted. The best fit equation was created using a polynomial best fit of conveyance versus distance in Microsoft Excel. The best fit equations are reproduced in the script as seen in Figure 103.

Figure 103: MATLAB output plots of conveyance equations for Kenyon's Bridge and equal conveyance tubes.

Slices of equal conveyance developed from the best fit lines were used to find the live length of flow in Froehlich's abutment scour equation as seen in Equation 9 and below:

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{L'}{y_a} \right)^{0.43} Fr^{0.61} + 1 .$$

The slices of equal conveyance (Figure 103) are found and presented by this script:

```

%Wendy Laurent
%Used to find equal conveyance tubes across a bridge abutment
%Bridge 206, Kenyons Bridge
clear all; clc; close all;
%%
%INPUT: Distance across the abutment and conveyance
L_dist=[269.6 259.6 249.6 239.6 229.6 219.6 209.6 189.6 169.6 129.6
99.6 59.6 29.6 6.6 0];
R_dist=[203.83 193.83 183.83 173.83 163.83 153.83 143.83 123.83 83.83 23.83
0.83 0];

L_conv=[21654 11448 8010.4 7077.4 6401.6 5820.6 5275.3 4704 3899.7
2604.1 2045.7 908.4 644.4 170.4 0];
R_conv=[50288.3 40786.1 35322.6 31989.2 29749.2 27772.5 25592.5 21604.6
14994.6 4360.3 53.9 0];

numchan = 5; % Number of channels to be considered

L_max= max(L_dist);
for x= 1:L_max
    %INPUT: best fit equations found in excel
    L_C(x)= (0.0000000006*(x^6)) - (0.000004185*(x^5)) + (0.0011413*(x^4)) -
(0.142356*(x^3)) + (7.948145*(x^2)) + (- 138.9434*x) + 372.4087;
    L_d(x)= x;
end

R_max= max(R_dist);
for x= 1:R_max
    %INPUT: best fit equations found in excel
    R_C(x)=(0.0000000027*(x.^6))+(- 0.000015878*(x.^5)) + (0.003492682*(x.^4)) +
(-0.353514661*(x.^3)) + (15.603573810*(x.^2)) + (- 31.798133425.*x)+
33.907797588;
    R_d(x)= x;
end

%%
%Plot conveyance and distance points
figure (1)
subplot (2, 2, 1)
plot(L_dist, L_conv, 'o-')
title('Left Abutment Conveyance');
xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');
subplot (2, 2, 3)
plot(R_dist, R_conv, 'o-');
title('Right Abutment Conveyance');

```

```

xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');

%Plot conveyance vs distance from equations
% figure (2)
subplot (2, 2, 2)
plot (L_d, L_C)
title('Left Abutment Conveyance');
xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');
subplot (2, 2, 4)
plot (R_d, R_C)
title('Right Abutment Conveyance');
xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');

%%
%Find equal conveyance tubes

L_tot_Conv= trapz(L_C);
R_tot_Conv= trapz (R_C);

spacing=.05;
L_C1 = L_C;
L_C_points=[0:spacing: L_max];
x= L_C_points;
%INPUT: best fit equations from excel
L_C=(0.000000006*(x.^6)) - (0.000004185*(x.^5)) + (0.0011413*(x.^4)) -
(0.142356*(x.^3)) + (7.948145*(x.^2)) + (- 138.9434.*x) + 372.4087;
L_C= fliplr(L_C)*spacing;

L_ch = L_tot_Conv/numchan; % Defines flow channel
%find equal slices
for i2= 2:length(L_C);
    i = 1;
    Flow1 = trapz(L_C(i:i2));
    if Flow1 >= L_ch; % When flow = 1 flow tube
        L1(i2) = i2;
    end
    Flow2 = Flow1-L_ch; % When total = flow - 1 flow tube
    if Flow2 >= L_ch;
        L2(i2) = i2;
    end
    Flow3 = Flow2-L_ch; % When flow = flow - 2 flow tubes
    if Flow3 >= L_ch;
        L3(i2) =i2;
    end
end

```

```

end
Flow4 = Flow3-L_ch; % When total = flow - 1 flow tube
if Flow4 >= L_ch;
    L4(i2) = i2;
end
Flow5 = Flow4-L_ch; % When flow = flow - 2 flow tubes
if Flow5 >= L_ch;
    L5(i2) = i2;
end
end
end

R_tot_Conv= trapz(R_C);
R_C1 = R_C;
R_C_points= [0:spacing: R_max];
x= R_C_points;
%INPUT: best fit equations from excel
R_C=(0.000000027*(x.^6))+(- 0.000015878*(x.^5)) + (0.003492682*(x.^4)) + (-
0.353514661*(x.^3)) + (15.603573810*(x.^2)) + (- 31.798133425.*x)+
33.907797588;
R_C= fliplr(R_C)*spacing;

R_ch = R_tot_Conv/numchan; % Defines flow channel
%find equal slices
for i2= 2:length(R_C);
    i = 1;
    Flow1 = trapz(R_C(i:i2));
    if Flow1 >= R_ch; % When flow = 1 flow tube
        R1(i2) = i2;
    end
    Flow2 = Flow1-R_ch; % When total = flow - 1 flow tube
    if Flow2 >= R_ch;
        R2(i2) = i2;
    end
    Flow3 = Flow2-R_ch; % When flow = flow - 2 flow tubes
    if Flow3 >= R_ch;
        R3(i2) = i2;
    end
    Flow4 = Flow3-R_ch; % When total = flow - 1 flow tube
    if Flow4 >= R_ch;
        R4(i2) = i2;
    end
    Flow5 = Flow4-R_ch; % When flow = flow - 2 flow tubes
    if Flow5 >= R_ch;
        R5(i2) = i2;
    end
end
end

```



```

end

%%
%assign slices
L_len_idx = find(L1~=0, 1, 'first');
L1= L1(L_len_idx);

L_len_idx = find(L2~=0, 1, 'first');
L2= L2(L_len_idx);

L_len_idx = find(L3~=0, 1, 'first');
L3= L3(L_len_idx);

L_len_idx = find(L4~=0, 1, 'first');
L4= L4(L_len_idx);

L_len_idx = find(L5~=0, 1, 'first');
L5= L5(L_len_idx);

R_len_idx = find(R1~=0, 1, 'first');
R1= R1(R_len_idx);

R_len_idx = find(R2~=0, 1, 'first');
R2= R2(R_len_idx);

R_len_idx = find(R3~=0, 1, 'first');
R3= R3(R_len_idx);

R_len_idx = find(R4~=0, 1, 'first');
R4= R4(R_len_idx);

R_len_idx = find(R5~=0, 1, 'first');
R5= R5(R_len_idx);

%Find the Area of the slice
L_A1= trapz(L_C(1:L1));
L_A2= trapz(L_C(L1:L2));
L_A3= trapz(L_C(L2:L3));
L_A4= trapz(L_C(L3:L4));
L_A5= trapz(L_C(L4:L5));
L_convs=[ L_A1, L_A2, L_A3, L_A4, L_A5];

R_A1= trapz(R_C(1:R1));
R_A2= trapz(R_C(R1:R2));
R_A3= trapz(R_C(R2:R3));
R_A4= trapz(R_C(R3:R4));

```

```

R_A5= trapz(R_C(R4:R5));
R_convs=[ R_A1, R_A2, R_A3, R_A4, R_A5];

%%
%x values
L_x=[0, L_C_points(L1), L_C_points(L2),L_C_points(L3), L_C_points(L4),
L_C_points(L5), L_max];
R_x=[0, R_C_points(R1), R_C_points(R2),R_C_points(R3), R_C_points(R4),
R_C_points(R5), R_max];

%find y values from conveyance/length
L_Y1= L_A1/L_x(2);
L_Y2= L_A2/L_x(3);
L_Y3= L_A3/L_x(4);
L_Y4= L_A4/L_x(5);
L_Y5= L_A5/L_x(6);
L_y= [L_Y1, L_Y1, L_Y2, L_Y3, L_Y4, L_Y5, 0];
R_Y1= R_A1/R_x(2);
R_Y2= R_A2/R_x(3);
R_Y3= R_A3/R_x(4);
R_Y4= R_A4/R_x(5);
R_Y5= R_A5/R_x(6);
R_y= [R_Y1, R_Y1, R_Y2, R_Y3, R_Y4, R_Y5,0];

%%
%Plot the conveyance tubes for each abutment
figure(3)
p(1)=subplot(2, 1, 1);
stairs (L_x, L_y);
title('Left Abutment Equal Conveyance Tubes');
xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');
ylim([0 120000]);

p(2)=subplot(2, 1, 2);
stairs (R_x, R_y);
title('Right Abutment Equal Conveyance Tubes');
xlabel('Distance across the abutment (ft)');
ylabel('Conveyance (cfs)');

```

APPENDIX C: KENYONS BRIDGE (NO. 020601), CHARLESTOWN, RI

C.1 KENYONS BRIDGE CROSS SECTIONS

The cross sections for Kenyons Bridge in Charlestown, RI are found in Figure 104 through Figure 118 beginning upstream and working downstream.

Figure 108: Kenyons Bridge cross section 266.

Figure 109: Kenyons Bridge cross section 263.

Figure 110: Kenyons Bridge cross section 251.

Figure 111: Kenyons Bridge cross section 249, bridge upstream.

Figure 112: Kenyons Bridge cross section 249, bridge downstream.

Figure 113: Kenyons Bridge cross section 196.

Figure 114: Kenyons Bridge cross section 185.

Figure 115: Kenyons Bridge cross section 163.

Figure 116: Kenyons Bridge cross section 89.

Figure 117: Kenyons Bridge cross section 52.

Figure 118: Kenyons Bridge cross section 39.

C.2 KENYONS BRIDGE HEC-RAS OUTPUTS

Found below are three tables summarizing the HEC-RAS outputs used in the the scour analysis for the 2010 flood at Kenyons Bridge, seen in Figure 119 through Figure 121.

Plan: n3 Pawcatuck Pawcatuck RS: 249.5497 Profile: March 2010

E.G. US. (ft)	56.23	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	56.11	E.G. Elev (ft)	56.23	56.15
Q Total (cfs)	3490.00	W.S. Elev (ft)	56.11	56.08
Q Bridge (cfs)	831.27	Crit W.S. (ft)	46.35	46.82
Q Weir (cfs)	2658.73	Max Chl Dpth (ft)	16.59	14.95
Weir Sta Lft (ft)	0.00	Vel Total (ft/s)	2.78	2.01
Weir Sta Rgt (ft)	524.43	Flow Area (sq ft)	1256.34	1737.30
Weir Submerg	0.94	Froude # Chl	0.16	0.11
Weir Max Depth (ft)	3.99	Specif Force (cu ft)	5498.75	5560.36
Min El Weir Flow (ft)	52.25	Hydr Depth (ft)	2.77	2.87
Min El Prs (ft)	51.30	W.P. Total (ft)	570.43	722.63
Delta EG (ft)	0.08	Conv. Total (cfs)		
Delta WS (ft)	0.03	Top Width (ft)	453.23	606.16
BR Open Area (sq ft)	331.87	Frctn Loss (ft)		
BR Open Vel (ft/s)	2.50	C & E Loss (ft)		
BR Sluice Coef		Shear Total (lb/sq ft)		
BR Sel Method	Press/Weir	Power Total (lb/ft s)		

Figure 119: Kenyons Bridge HEC-RAS bridge output table.

HEC-RAS Plan: n3 River: Pawcatuck Reach: Pawcatuck Profile: March 2010

Reach	River Sta	Profile	Vel Total (ft/s)	Vel Chnl (ft/s)	Hydr Depth (ft)	Hydr Depth C (ft)	Q Channel (cfs)	Q Left (cfs)
Pawcatuck	360.3901	March 2010	2.13	2.57	9.83	13.55	2414.02	231.65
Pawcatuck	351.4639	March 2010	1.91	2.41	9.68	13.63	2391.14	289.27
Pawcatuck	333.911	March 2010	1.76	2.40	6.48	14.33	2058.39	357.66
Pawcatuck	312.2062	March 2010	1.58	2.25	4.14	12.92	2219.73	165.81
Pawcatuck	287.5418	March 2010	1.47	2.41	4.12	13.48	1938.01	214.20
Pawcatuck	266.08	March 2010	1.38	2.30	4.88	15.17	2231.03	263.20
Pawcatuck	263.1974	March 2010	1.56	2.80	3.76	13.80	2393.31	287.40
Pawcatuck	251.7345	March 2010	1.81	3.26	3.67	14.72	2505.73	302.24
Pawcatuck	249.5497		Bridge					
Pawcatuck	196.1379	March 2010	1.45	2.60	3.55	13.15	2279.78	322.91
Pawcatuck	185.3493	March 2010	0.93	1.76	5.51	12.96	1584.16	504.06
Pawcatuck	163.0821	March 2010	0.63	1.15	7.78	13.48	1015.35	941.45
Pawcatuck	129.2619	March 2010	0.69	1.10	7.41	13.60	884.16	1068.38
Pawcatuck	89.77869	March 2010	1.05	1.69	7.39	12.45	1299.12	1006.65
Pawcatuck	52.77306	March 2010	1.24	1.77	9.35	14.20	1775.07	634.83
Pawcatuck	39.97197	March 2010	1.66	2.15	9.49	13.90	1367.71	641.85

Figure 120: Kenyons Bridge HEC-RAS velocity, depth, and flow output table.

HEC-RAS Plan: n3 River: Pawcatuck Reach: Pawcatuck Profile: March 2010

Reach	River Sta	Profile	Q Right (cfs)	Flow Area (sq ft)	Area Left (sq ft)	Area Right (sq ft)	Conv. Chnl (cfs)	Conv. Left (cfs)	Conv. Right (cfs)	Top Width (ft)	Top W Chnl (ft)
Pawcatuck	360.3901	March 2010	844.33	1641.53	196.18	506.40	175878.6	16877.5	61515.5	167.03	69.30
Pawcatuck	351.4639	March 2010	809.59	1825.80	212.62	619.55	222126.0	26871.9	75206.8	188.56	72.89
Pawcatuck	333.911	March 2010	1073.95	1979.45	295.93	827.57	195057.1	33892.6	101769.1	305.60	59.72
Pawcatuck	312.2062	March 2010	1104.47	2213.55	285.16	943.93	204976.7	15311.3	101990.1	535.31	76.18
Pawcatuck	287.5418	March 2010	1337.79	2371.73	452.45	1115.39	166556.1	18409.0	114972.0	575.12	59.62
Pawcatuck	266.08	March 2010	995.78	2520.21	483.98	1064.25	221677.3	26151.4	98941.4	516.48	64.07
Pawcatuck	263.1974	March 2010	809.29	2234.29	495.61	882.70	191724.1	23023.6	64830.7	594.90	62.02
Pawcatuck	251.7345	March 2010	682.03	1927.07	393.11	765.35	179801.0	21687.8	48939.3	524.43	52.20
Pawcatuck	249.5497										
Pawcatuck	196.1379	March 2010	887.31	2401.35	545.76	977.79	180548.3	25572.9	70271.3	675.66	66.76
Pawcatuck	185.3493	March 2010	1401.78	3758.80	966.87	1894.15	199180.5	63377.1	176250.4	681.74	69.25
Pawcatuck	163.0821	March 2010	1533.20	5498.45	1920.47	2696.16	201417.9	186758.0	304143.3	706.49	65.44
Pawcatuck	129.2619	March 2010	1537.46	5088.82	1916.03	2370.11	161684.0	195370.1	281149.7	686.84	59.00
Pawcatuck	89.77869	March 2010	1184.23	3319.30	1321.81	1229.00	152565.0	118218.2	139072.3	449.07	61.72
Pawcatuck	52.77306	March 2010	1080.10	2805.89	741.32	1064.52	226478.5	80996.7	137808.3	300.10	70.45
Pawcatuck	39.97197	March 2010	1480.45	2106.47	460.40	1009.75	144661.6	67887.7	156585.9	222.01	45.79

Figure 121: Kenyons Bridge HEC-RAS flow, area, conveyance, and width output table.

APPENDIX D: FIRST BARBERVILLE BRIDGE (NO. 004101), HOPKINTON, RI

D.1 FIRST BARBERVILLE BRIDGE CROSS SECTIONS

See below the cross sections in Figure 122 through Figure 143 from the First Barberville Bridge in Hopkinton, RI beginning upstream and working downstream.

Figure 126: First Barberville Bridge cross section 138.

Figure 127: First Barberville Bridge cross section 133.

Figure 128: First Barberville Bridge cross section 129.

Figure 129: First Barberville Bridge cross section 128, bridge upstream.

Figure 130: First Barberville Bridge cross section 128, bridge downstream.

Figure 131: First Barberville Bridge cross section 103.

Figure 132: First Barberville Bridge cross section 101.

Figure 133: First Barberville Bridge cross section 95.

Figure 134: First Barberville Bridge cross section 88.

Figure 135: First Barberville Bridge cross section 81.

Figure 136: First Barberville Bridge cross section 76.

Figure 137: First Barberville Bridge cross section 66.

Figure 138: First Barberville Bridge cross section 59.

Figure 139: First Barberville Bridge cross section 54.

Figure 140: First Barberville Bridge cross section 43.

Figure 141: First Barberville Bridge cross section 30.

Figure 142: First Barberville Bridge cross section 23.

Figure 143: First Barberville Bridge cross section 9.

D.2 FIRST BARBERVILLE BRIDGE HEC-RAS OUTPUTS

Found below in Figure 144 through Figure 146 are three tables summarizing the HEC-RAS outputs used in the the scour analysis for the 2010 flood at the First Barberville Bridge.

Plan: 2 Wood WOOD RS: 128.8488 Profile: 2010 Flood

E.G. US. (ft)	115.27	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	114.55	E.G. Elev (ft)	115.26	115.26
Q Total (cfs)	4380.00	W.S. Elev (ft)	114.55	114.55
Q Bridge (cfs)	3111.04	Crit W.S. (ft)	115.39	110.37
Q Weir (cfs)	1268.96	Max Chl Dpth (ft)	12.38	12.52
Weir Sta Lft (ft)	0.00	Vel Total (ft/s)	7.02	6.78
Weir Sta Rgt (ft)	254.61	Flow Area (sq ft)	623.86	645.70
Weir Submerg	0.00	Froude # Chl	0.63	0.58
Weir Max Depth (ft)	1.46	Specif Force (cu ft)	3718.63	3413.76
Min El Weir Flow (ft)	113.81	Hydr Depth (ft)	2.50	2.55
Min El Prs (ft)	111.00	W.P. Total (ft)	356.38	365.83
Delta EG (ft)	0.47	Conv. Total (cfs)		
Delta WS (ft)	2.47	Top Width (ft)	249.57	253.33
BR Open Area (sq ft)	271.12	Frctn Loss (ft)		
BR Open Vel (ft/s)	11.47	C & E Loss (ft)		
BR Sluice Coef		Shear Total (lb/sq ft)		
BR Sel Method	Press/Weir	Power Total (lb/ft s)		

Figure 144: First Barberville Bridge HEC-RAS bridge output table.

HEC-RAS Plan: 2 River: Wood Reach: WOOD Profile: 2010 Flood

Reach	River Sta	Profile	Vel Total (ft/s)	Vel Chnl (ft/s)	Hydr Depth (ft)	Hydr Depth C (ft)	Q Channel (cfs)	Q Left (cfs)
WOOD	176.8232	2010 Flood	5.38	6.27	3.70	6.69	4083.09	225.40
WOOD	164.0323	2010 Flood	5.76	7.57	3.25	5.37	3531.42	763.65
WOOD	154.8884	2010 Flood	3.43	4.41	7.62	12.35	3402.37	266.97
WOOD	146.4409	2010 Flood	3.17	4.66	5.89	13.15	3213.63	361.46
WOOD	138.4343	2010 Flood	3.29	4.66	5.30	11.97	3356.70	419.79
WOOD	133.1752	2010 Flood	3.66	5.60	4.77	11.71	3375.12	400.51
WOOD	129.3487	2010 Flood	4.58	7.71	3.83	11.14	3338.29	438.72
WOOD	128.8488	Bridge						
WOOD	103.6655	2010 Flood	8.01	8.36	6.47	8.19	4280.27	82.38
WOOD	101.1652	2010 Flood	8.24	9.29	6.12	8.62	3778.89	199.31
WOOD	95.10232	2010 Flood	8.03	9.07	4.47	7.96	3841.13	199.20
WOOD	88.23674	2010 Flood	8.34	9.67	4.28	7.17	3953.77	155.39
WOOD	81.80484	2010 Flood	7.61	9.39	3.75	7.21	3631.59	270.07
WOOD	78.16148	2010 Flood	6.99	8.71	3.32	6.81	3683.46	139.99
WOOD	66.90012	2010 Flood	7.13	8.68	3.39	6.56	3762.87	147.86
WOOD	59.80856	2010 Flood	6.34	7.69	3.47	6.95	3484.90	187.37
WOOD	54.59997	2010 Flood	5.49	6.72	3.88	7.43	3499.00	232.15
WOOD	43.39098	2010 Flood	5.25	6.37	3.84	7.04	3731.71	144.77
WOOD	30.77439	2010 Flood	5.01	6.17	3.91	7.48	3648.86	219.78
WOOD	23.21589	2010 Flood	4.92	6.01	4.19	7.65	3537.28	383.13
WOOD	9.788291	2010 Flood	5.45	6.30	4.28	7.16	3584.25	475.46

Figure 145: First Barberville Bridge HEC-RAS velocity, depth, and flow output table.

HEC-RAS Plan: 2 River: Wood Reach: WOOD Profile: 2010 Flood

Reach	River Sta	Profile	Q Right (cfs)	Flow Area (sq ft)	Area Left (sq ft)	Area Right (sq ft)	Conv. Chnl (cfs)	Conv. Left (cfs)	Conv. Right (cfs)	Top Width (ft)	Top W Chnl (ft)
WOOD	176.8232	2010 Flood	71.51	814.35	129.84	32.83	75619.8	4174.6	1324.4	220.26	97.47
WOOD	164.0323	2010 Flood	84.94	761.04	264.99	29.37	45429.5	9823.8	1092.6	234.23	86.93
WOOD	154.8684	2010 Flood	710.86	1277.62	240.18	266.08	131290.8	10302.0	27422.8	167.70	62.48
WOOD	146.4409	2010 Flood	804.91	1379.89	346.86	343.19	121028.9	13613.0	30314.0	234.46	62.47
WOOD	138.4943	2010 Flood	603.51	1331.73	341.36	299.02	105312.7	13170.5	18634.4	251.44	57.78
WOOD	133.1752	2010 Flood	604.37	1197.87	300.88	294.28	91413.7	10847.7	16369.1	250.88	51.47
WOOD	129.3487	2010 Flood	602.98	955.32	242.88	276.59	66852.9	9180.2	12617.3	249.57	38.87
WOOD	128.8488										
WOOD	103.6655	2010 Flood	17.35	546.51	21.99	12.72	59483.0	1144.9	241.1	84.41	62.47
WOOD	101.1652	2010 Flood	401.80	531.52	42.90	81.83	54803.5	2879.9	5805.9	86.84	47.21
WOOD	95.10232	2010 Flood	339.67	545.47	43.19	76.79	54319.6	2817.0	4903.4	121.90	53.17
WOOD	88.23674	2010 Flood	270.84	525.42	39.03	77.42	49559.6	1947.7	3394.9	122.78	57.00
WOOD	81.80484	2010 Flood	478.35	575.39	64.05	124.47	46417.8	3451.9	6114.1	153.62	53.63
WOOD	76.16148	2010 Flood	556.56	626.37	61.23	142.29	48426.4	1840.4	7317.1	183.59	62.08
WOOD	66.90012	2010 Flood	469.27	614.12	44.82	135.75	49804.3	1917.7	6086.4	181.20	66.08
WOOD	59.80856	2010 Flood	707.73	691.20	52.67	185.43	49113.2	2640.7	9974.1	199.11	65.24
WOOD	54.56997	2010 Flood	648.85	797.12	71.27	204.90	57295.0	3801.4	10624.8	205.19	70.08
WOOD	43.36096	2010 Flood	503.52	834.70	68.57	179.94	70213.7	2724.0	9473.9	217.35	83.32
WOOD	30.77439	2010 Flood	511.36	873.86	100.89	181.80	73755.1	4442.4	10336.2	223.58	79.07
WOOD	23.21589	2010 Flood	459.59	890.95	139.47	163.22	72703.8	7874.6	9446.3	212.73	76.88
WOOD	9.788291	2010 Flood	320.28	804.26	130.82	104.47	68956.7	9147.6	6162.1	187.76	76.44

Figure 146: First Barberville Bridge HEC-RAS flow, area, conveyance, and width output table.

APPENDIX E: ESMOND STREET BRIDGE (NO. 094801), SMITHFIELD, RI

E.1 ESMOND STREET BRIDGE CROSS SECTIONS

See below in Figure 147 to Figure 161 are the cross sections from the Esmond Street Bridge (No. 094801) in Smithfield, RI beginning upstream and working downstream.

Figure 147: Esmond Street Bridge cross section 304.

Figure 148: Esmond Street Bridge cross section 270.

Figure 149: Esmond Street Bridge cross section 238.

Figure 150: Esmond Street Bridge cross section 220.

Figure 151: Esmond Street Bridge cross section 202.

Figure 152: Esmond Street Bridge cross section 187.

Figure 153: Esmond Street Bridge cross section 162.

Figure 154: Esmond Street Bridge cross section 158.

Figure 155: Esmond Street Bridge cross section 155, bridge upstream.

Figure 156: Esmond Street Bridge cross section 155, bridge downstream.

Figure 157: Esmond Street Bridge cross section 106.

Figure 158: Esmond Street Bridge cross section 90.

Figure 159: Esmond Street Bridge cross section 76.

Figure 160: Esmond Street Bridge cross section 49.

Figure 161: Esmond Street Bridge cross section 32.

E.2 ESMOND STREET BRIDGE HEC-RAS OUTPUTS

Found below in Figure 162 through Figure 164 are three tables summarizing the HEC-RAS outputs used in the the scour analysis for the 2010 flood at the Esmond Street Bridge.

Plan: 1.65 Woonasquatucket Woonasquatucket RS: 155.8238 Profile: 2010				
E.G. US. (ft)	119.60	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	119.24	E.G. Elev (ft)	119.59	119.48
Q Total (cfs)	1688.00	W.S. Elev (ft)	119.13	118.90
Q Bridge (cfs)	1688.00	Crit W.S. (ft)	116.07	115.24
Q Weir (cfs)		Max Chl Dpth (ft)	7.42	9.48
Weir Sta Lft (ft)		Vel Total (ft/s)	5.47	6.13
Weir Sta Rgt (ft)		Flow Area (sq ft)	308.66	275.45
Weir Submerg		Froude # Chl	0.38	0.40
Weir Max Depth (ft)		Specif Force (cu ft)	1287.47	1423.41
Min El Weir Flow (ft)	121.41	Hydr Depth (ft)	6.43	7.44
Min El Prs (ft)	120.00	W.P. Total (ft)	61.29	46.63
Delta EG (ft)	0.16	Conv. Total (cfs)	35934.3	32241.7
Delta WS (ft)	0.36	Top Width (ft)	48.00	37.02
BR Open Area (sq ft)	316.74	Frctn Loss (ft)	0.10	0.03
BR Open Vel (ft/s)	6.13	C & E Loss (ft)	0.01	0.01
BR Sluice Coef		Shear Total (lb/sq ft)	0.69	1.01
BR Sel Method	Energy only	Power Total (lb/ft s)	3.79	6.20

Figure 162: Esmond Street Bridge HEC-RAS bridge output table.

Reach	River Sta	Profile	Vel Total (ft/s)	Vel Chnl (ft/s)	Hydr Depth (ft)	Hydr Depth C (ft)	Q Channel (cfs)	Q Left (cfs)	Q Right (cfs)
Woonasquatucket	304.1207	2010	4.01	4.11	3.62	4.53	1668.39	4.09	15.52
Woonasquatucket	270.0903	2010	4.32	4.52	4.15	5.01	1625.64	23.14	39.22
Woonasquatucket	238.3388	2010	4.63	5.22	4.10	5.86	1393.69	254.89	39.42
Woonasquatucket	220.4412	2010	4.60	5.08	4.69	6.29	1470.67	167.72	49.61
Woonasquatucket	202.5116	2010	5.04	5.16	4.67	5.44	1666.52	4.13	17.35
Woonasquatucket	187.2762	2010	6.27	6.27	4.38	4.38	1688.00		
Woonasquatucket	162.9418	2010	4.81	4.81	5.01	5.01	1688.00		
Woonasquatucket	158.0248	2010	4.83	4.83	5.87	5.87	1688.00		
Woonasquatucket	155.8238		Bridge						
Woonasquatucket	104.7143	2010	6.00	6.00	7.02	7.02	1688.00		
Woonasquatucket	90.39623	2010	5.99	5.99	5.38	5.38	1688.00		
Woonasquatucket	76.96196	2010	5.12	5.12	5.88	5.88	1688.00		
Woonasquatucket	49.08693	2010	7.22	7.22	4.41	4.41	1688.00		
Woonasquatucket	32.32348	2010	7.20	7.20	4.25	4.25	1688.00		

Figure 163: Esmond Street Bridge HEC-RAS velocity, depth, and flow output table.

Reach	River Sta	Profile	Flow Area (sq ft)	Area Left (sq ft)	Area Right (sq ft)	Conv. Chnl (cfs)	Conv. Left (cfs)	Conv. Right (cfs)	Top Width (ft)	Top W Chnl (ft)
Woonasquatucket	304.1207	2010	421.39	5.36	9.65	32425.6	79.4	301.7	116.30	89.66
Woonasquatucket	270.0903	2010	390.51	13.43	17.61	41766.3	594.6	1007.7	94.19	71.74
Woonasquatucket	238.3388	2010	364.71	79.36	18.16	34025.0	6222.8	962.5	88.86	45.58
Woonasquatucket	220.4412	2010	366.58	57.36	19.69	38271.2	4364.6	1291.0	78.18	46.02
Woonasquatucket	202.5116	2010	334.78	3.13	8.67	38355.7	95.0	399.4	71.64	59.41
Woonasquatucket	187.2762	2010	269.35			27026.2			61.47	61.47
Woonasquatucket	162.9418	2010	350.79			37475.5			69.96	69.96
Woonasquatucket	158.0248	2010	349.53			40440.7			59.51	59.51
Woonasquatucket	155.8238									
Woonasquatucket	104.7143	2010	281.38			32794.4			40.11	40.11
Woonasquatucket	90.39623	2010	281.84			30837.4			52.35	52.35
Woonasquatucket	76.96196	2010	329.72			33288.4			56.08	56.08
Woonasquatucket	49.08693	2010	233.72			20875.6			52.95	52.95
Woonasquatucket	32.32348	2010	234.35			20936.6			55.11	55.11

Figure 164: Esmond Street Bridge HEC-RAS area, conveyance, and width output table.

APPENDIX F: WEEKAPAUG BRIDGE (NO. 099701), WESTERLY, RI

F.1 COASTAL ENGINEERING MANUAL'S INLET HYDRODYNAMICS

Found below is the methodology used to find the maximum cross-sectional averaged velocity at Weekapaug Breachway using the CEM's chapter on Inlet Hydrodynamics (USACE 2002). First the inlet cross sectional area must be found,

$$A_c = Bd$$

where:

A_c = Inlet cross sectional area (ft²);

B = Inlet width (ft); and

d = Inlet depth (ft).

It was calculated to be 1615 square feet due to a width of 170 feet and an approximate depth of 9.5 feet. Next, the hydraulic radius of the cross sections is found using,

$$R = \frac{A_c}{(B + 2d)}$$

where:

R = Inlet hydraulic radius (ft).

This was found to be 8.5 feet. The variables F , K_1 , and K_2 are found next:

$$F = k_{en} + k_{ex} + \frac{fL}{4R}$$

$$K_1 = \frac{a_o A_b F}{2 L A_c}$$

$$K_2 = \frac{2\pi}{T} \sqrt{\frac{L A_b}{g A_c}}$$

where:

k_{en} = Entrance energy loss coefficient;

k_{ex} = Exit energy loss coefficient;

f = Darcy-Weisbach friction term;

L = Inlet length(ft);

a_o = Ocean tide amplitude (ft);

A_b = Surface area of the bay (ft²);

T = Tidal period (s); and

g = Acceleration due to gravity (32.2 ft/s²).

F equated to 4.1. A k_{en} value of 0.15 due to the the inlet being lined by two jetties with a turn at the entrance. The k_{ex} value was chosen to be 1.0 as suggested by the CEM. An ocean amplitude of 1.85 feet was used based on the tidal signal at the inlet. K_1 and K_2 were found to be 14.1 and 0.15 respectively and were used to find the values of the bay to sea amplitude and the dimensionless maximum velocity. Finally, the maximum cross-sectionally averaged velocity during a tidal period can be found,

$$V_m = \frac{V_m' 2\pi a_s A_b}{A_c T}$$

where:

V_m' = Dimensionless maximum velocity; and

a_s = Tidal amplitude of the sea (ft).

The maximum cross-sectionally average velocity for the tidal period was found to be 2.93 feet per second which is in agreement with the 2.87 feet per second obtained in the HEC-RAS model, validating the Weekapaug Breachway model.

F.2 WEEKAPAUG BRIDGE CROSS SECTIONS

See below the cross sections in Figure 165 to Figure 191 of Weekapaug Bridge (No. 099701) in Westerly, RI beginning at the south end of the breachway and working north.

Figure 165: Weekapaug Bridge cross section 2392.

Figure 166: Weekapaug Bridge cross section 2300.

Figure 167: Weekapaug Bridge cross section 2200.

Figure 168: Weekapaug Bridge cross section 2100.

Figure 169: Weekapaug Bridge cross section 2000.

Figure 170: Weekapaug Bridge cross section 1900.

Figure 171: Weekapaug Bridge cross section 1800.

Figure 172: Weekapaug Bridge cross section 1700.

Figure 173: Weekapaug Bridge cross section 1660.

Figure 174: Weekapaug Bridge cross section 1655, bridge upstream.

Figure 175: Weekapaug Bridge cross section 1655, bridge downstream.

Figure 176: Weekapaug Bridge cross section 1610.

Figure 177: Weekapaug Bridge cross section 1500.

Figure 178: Weekapaug Bridge cross section 1400.

Figure 179: Weekapaug Bridge cross section 1300.

Figure 180: Weekapaug Bridge cross section 1200.

Figure 181: Weekapaug Bridge cross section 1100.

Figure 182: Weekapaug Bridge cross section 1000.

Figure 183: Weekapaug Bridge cross section 900.

Figure 184: Weekapaug Bridge cross section 800.

Figure 185: Weekapaug Bridge cross section 700.

Figure 186: Weekapaug Bridge cross section 600.

Figure 187: Weekapaug Bridge cross section 500.

Figure 188: Weekapaug Bridge cross section 400.

Figure 189: Weekapaug Bridge cross section 300.

Figure 190: Weekapaug Bridge cross section 200.

Figure 191: Weekapaug Bridge cross section 100.

F.3 WEEKAPAUG BRIDGE HEC-RAS OUTPUTS

In Figure 192 through Figure 194 are three tables summarizing the HEC-RAS outputs used in the the scour analysis for the Hurricane Sandy at Weekapaug Bridge.

Plan: Plan 08 Weekapaug		W. Flood RS: 1655.879	Profile: 30OCT2012 1200	
E.G. US. (ft)	3.36	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	2.70	E.G. Elev (ft)	3.46	3.37
Q Total (cfs)	8264.73	W.S. Elev (ft)	2.52	2.50
Q Bridge (cfs)	8239.57	Crit W.S. (ft)	-1.41	-1.83
Q Weir (cfs)	25.16	Max Chl Dpth (ft)	10.76	11.65
Weir Sta Lft (ft)	287.42	Vel Total (ft/s)	7.78	7.47
Weir Sta Rgt (ft)	414.29	Flow Area (sq ft)	1061.86	1106.84
Weir Submerg	0.00	Froude # Chl	0.45	0.42
Weir Max Depth (ft)	0.39	Specif Force (cu ft)	7113.76	7418.87
Min El Weir Flow (ft)	3.11	Hydr Depth (ft)	9.45	9.85
Min El Prs (ft)	7.50	W.P. Total (ft)	164.92	168.48
Delta EG (ft)	0.25	Conv. Total (cfs)	172230.5	192300.0
Delta WS (ft)	0.12	Top Width (ft)	112.40	112.40
BR Open Area (sq ft)	1622.14	Frctn Loss (ft)		
BR Open Vel (ft/s)	7.76	C & E Loss (ft)		
BR Sluice Coef		Shear Total (lb/sq ft)	0.93	0.76
BR Sel Method	Energy/Weir	Power Total (lb/ft s)	7.20	5.66

Figure 192: Weekapaug Bridge HEC-RAS bridge output table.

Reach	River Sta	Profile	Vel Total (ft/s)	Vel Chnl (ft/s)	Hydr Depth (ft)	Hydr Depth C (ft)	Q Channel (cfs)	Q Left (cfs)	Q Right (cfs)
W. Flood	2392.013	30OCT2012 1200	3.71	4.33	4.57	9.87	7441.42	830.52	
W. Flood	2300	30OCT2012 1200	5.11	5.11	10.63	10.63	8270.01		
W. Flood	2200	30OCT2012 1200	5.87	5.87	9.13	9.13	8269.14		
W. Flood	2100	30OCT2012 1200	5.41	5.41	9.43	9.43	8268.28		
W. Flood	2000	30OCT2012 1200	5.98	5.98	9.33	9.33	8267.48		
W. Flood	1900	30OCT2012 1200	5.31	5.31	8.12	8.12	8266.63		
W. Flood	1800	30OCT2012 1200	5.77	5.77	9.27	9.27	8265.78		
W. Flood	1700	30OCT2012 1200	5.00	5.00	9.76	9.76	8265.02		
W. Flood	1660.907	30OCT2012 1200	6.55	6.55	7.88	7.88	8264.73		
W. Flood	1655.879	Bridge							
W. Flood	1610.619	30OCT2012 1200	5.78	5.78	8.26	8.26	8264.73		
W. Flood	1500	30OCT2012 1200	4.39	4.39	7.72	8.09	8263.58	0.03	
W. Flood	1400	30OCT2012 1200	4.13	4.49	3.16	6.91	8153.51	108.16	
W. Flood	1300	30OCT2012 1200	3.82	4.39	3.15	6.66	8010.48	248.63	
W. Flood	1200	30OCT2012 1200	3.15	3.93	2.70	7.21	7794.99	461.06	
W. Flood	1100	30OCT2012 1200	2.78	3.60	2.58	7.11	7679.45	572.25	0.01
W. Flood	1000	30OCT2012 1200	2.60	3.76	2.41	7.32	7349.49	868.44	28.50
W. Flood	900	30OCT2012 1200	2.33	3.64	2.22	6.62	6846.42	1340.00	53.69
W. Flood	800	30OCT2012 1200	2.20	3.32	2.41	6.09	6983.28	1115.72	134.09
W. Flood	700	30OCT2012 1200	2.08	3.27	2.30	6.05	6864.08	1148.91	213.03
W. Flood	600	30OCT2012 1200	2.21	3.69	1.94	6.13	6698.25	1261.61	259.13
W. Flood	500	30OCT2012 1200	2.28	3.82	1.85	6.11	6234.92	1502.72	473.47
W. Flood	400	30OCT2012 1200	2.93	4.94	1.88	6.38	6821.75	1169.12	212.18
W. Flood	300	30OCT2012 1200	2.83	5.01	1.96	6.24	6512.90	1376.50	306.97
W. Flood	200	30OCT2012 1200	3.02	5.24	1.89	6.42	6231.57	1637.61	320.68
W. Flood	100	30OCT2012 1200	3.44	5.94	1.74	5.70	6744.43	1351.91	87.01

Figure 193: Weekapaug Bridge HEC-RAS velocity, depth, and flow outputs.

Reach	River Sta	Profile	Flow Area	Area Left	Area Right	Conv. Chnl	Conv. Left	Conv. Right	Top Width	Top W Chnl
			(sq ft)	(sq ft)	(sq ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
W. Flood	2392.013	30OCT2012 1200	2229.38	509.94		331832.0	37035.1		487.57	174.21
W. Flood	2300	30OCT2012 1200	1619.70			331779.8			152.43	152.43
W. Flood	2200	30OCT2012 1200	1408.35			265837.4			154.30	154.30
W. Flood	2100	30OCT2012 1200	1529.38			297184.4			162.24	162.24
W. Flood	2000	30OCT2012 1200	1381.61			263695.7			148.05	148.05
W. Flood	1900	30OCT2012 1200	1556.38			264579.4			191.64	191.64
W. Flood	1800	30OCT2012 1200	1433.49			285756.6			154.68	154.68
W. Flood	1700	30OCT2012 1200	1654.08			320192.5			169.41	169.41
W. Flood	1660.907	30OCT2012 1200	1261.36			217435.4			160.10	160.10
W. Flood	1655.879									
W. Flood	1610.619	30OCT2012 1200	1430.81			284904.7			173.23	173.23
W. Flood	1500	30OCT2012 1200	1881.56	0.34		361343.8	1.1		243.75	232.45
W. Flood	1400	30OCT2012 1200	1998.11	184.07		313094.3	4153.2		631.83	262.56
W. Flood	1300	30OCT2012 1200	2159.80	336.83		310437.8	9635.4		685.41	273.55
W. Flood	1200	30OCT2012 1200	2617.31	636.14		342051.9	20231.9		970.23	274.89
W. Flood	1100	30OCT2012 1200	2968.00	833.14	0.17	369718.3	27550.4	0.7	1149.47	300.04
W. Flood	1000	30OCT2012 1200	3172.47	1155.67	64.38	357967.8	42298.8	1388.0	1314.75	266.86
W. Flood	900	30OCT2012 1200	3543.08	1556.96	106.64	315246.8	61701.1	2472.3	1592.87	284.04
W. Flood	800	30OCT2012 1200	3739.21	1429.64	204.75	369941.6	59105.4	7103.4	1551.74	345.41
W. Flood	700	30OCT2012 1200	3948.03	1502.82	347.15	373591.5	62532.0	11594.3	1717.98	346.59
W. Flood	600	30OCT2012 1200	3726.80	1494.46	414.89	319032.8	60089.6	12342.1	1921.49	296.70
W. Flood	500	30OCT2012 1200	3595.47	1438.20	524.04	233708.8	56327.7	17747.5	1944.22	267.39
W. Flood	400	30OCT2012 1200	2797.79	1150.89	265.52	247912.5	42487.5	7711.1	1491.74	216.53
W. Flood	300	30OCT2012 1200	2892.94	1277.08	315.09	233850.2	49424.2	11022.1	1472.77	208.31
W. Flood	200	30OCT2012 1200	2714.98	1246.42	279.39	176752.4	46449.3	9095.8	1432.77	185.25
W. Flood	100	30OCT2012 1200	2381.19	1148.11	97.52	196140.7	39316.1	2530.5	1366.69	199.32

Figure 194: Weekapaug Bridge HEC-RAS flow area, conveyance, and width outputs.

BIBLIOGRAPHY

- AECOM (2013). "Scour Critical Bridge Reviews for March 2010 Floods." *Rhode Island Department of Transportation (RIDOT)*, 1, Providence, RI.
- ArcGIS. (2016). "ArcMap." *ArcGIS for Desktop*, <
<http://desktop.arcgis.com/en/arcmap/>>. (Sept. 25, 2016).
- Arneson, L., Zevenbergen, L., Lagasse, P., and Clopper, P. (2012). "Evaluating Scour at Bridges." 5th ed., *Federal Highway Administration (FHWA) Hydraulic Engineering Circular (HEC) 18*, Fort Collins, CO.
- Aube, D. R. (2010). "Route 91 Fishing Access." *Wood-Pawcatuck Watershed Association*. <
<http://www.wpwa.org/galleryPhoto.php?galleryID=18&photoID=145>> (Oct. 10, 2016).
- Benedict, S. T. (2003). "Clear-Water Abutment and Contraction Scour in the Coastal Plain and Piedmont Provinces of South Carolina, 1996-99." *United States Geological Survey (USGS)*, Columbia, SC.
- Benedict, S. T., Deshpande, N., Aziz, N. M., and Conrads, P. A. (2006). "Trends of Abutment Scour Prediction Equations Applied to 144 Field Sites in South Carolina." *United States Geological Survey (USGS)*, Reston, VA.
- Bradley, J. N. (1978). *Hydraulics of Bridge Waterways*. Federal Highway Administration Bridge Division.
- Briaud, J. L., Chen H. C., Chang, K. A., Oh, S. J., Chen, S., Wang J., Li, Y., Kwak, K., Nartjano, P., Gudaralli, R., Wei, W., Pergu, S., Cad, Y. W., and Ting, F.

- (2011). “The SRICOS-EFA Method.” *Texas A&M University*, College Station, TX.
- Brisson, L. N., Wolfe, D. A., and Staley, M. (2014). “Interferometric Swath Bathymetry for Large Scale Shallow Water Hydrographic Surveys.” *Canadian Hydrographic Conference*, Saint Johns, NL, Canada.
- Brunner, G. W., and CEIWR-HEC. (2016). *HEC-RAS River Analysis System User’s Manual*, U.S. Army Corps of Engineers Institute for Water Resources Hydraulic Engineering Center (HEC), Davis, CA.
- Brunner, G. W. (2016). *HEC-RAS River Analysis System Hydraulic Reference Manual Version 5.0*, U.S. Army Corps of Engineers Institute for Water Resources Hydraulic Engineering Center (HEC), Davis, CA.
- EdgeTech. (2015). “6205 Bathymetry & Side Scan System User Manual.” *EdgeTech*, <http://www.edgetech.com/wp-content/uploads/2015/07/6205-Hardware-Users-Manual-0014877_Rev_D.pdf>. (Jun. 20, 2016).
- Ettema, R., Nakato, T., and Muste, M. (2010). “Estimation of Scour Depth at Bridge Abutments.” *National Cooperative Highway Research Program (NCHRP)*, Washington, D.C.
- FHWA, (2001). “ACTION: Revision of Coding Guide, Item 113 - Scour Critical Bridges.” *U.S. Department of Transportation Federal Highway Administration (FHWA)*. <<https://www.fhwa.dot.gov/engineering/hydraulics/policymemo/revguide.cfm>> . (Nov. 15, 2016)

- FHWA Hydraulics. (2012). “Update HEC 18 and HEC 20 and NHI Course 135046.”
U.S. Department of Transportation, Federal Highway Administration,
Hydraulics Engineering,
<<https://content.govdelivery.com/accounts/USDOTFHWAHYD/bulletins/439516>> (Sept. 25, 2016).
- Fox, C. (2010a). “Jay Cronan Fishing Access, Pawcatuck River at Rt. 91.” *Wood-Pawcatuck Watershed Association.* <
<http://www.wpwa.org/galleryPhoto.php?galleryID=15&photoID=103>> (Oct.
10, 2016).
- Fox, C. (2010b). “Barberville Dam, Wood River.” *Wood-Pawcatuck Watershed Association.*
<<http://www.wpwa.org/galleryPhoto.php?galleryID=15&photoID=120>> (Oct.
10, 2016).
- Fox, C. (2010c). “Barberville Dam, Arcadia Rd Bridge, Wood River.” *Wood-Pawcatuck Watershed Association.*
<<http://www.wpwa.org/galleryPhoto.php?galleryID=15&photoID=120>> (Oct.
10, 2016).
- Fura, R. and Mahmutoglu, S. (2010a). *Scour Critical Bridge—Plan of Action 020601.*
Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Fura, R. and Mahmutoglu, S. (2010b). *Scour Critical Bridge—Plan of Action 004110.*
Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Fura, R. and Mahmutoglu, S. (2010c). *Scour Critical Bridge—Plan of Action 094801.*
Rhode Island Department of Transportation (RIDOT), Providence, RI.

- Fura, R. and Mahmutoglu, S. (2010d). *Scour Critical Bridge—Plan of Action 099701*. Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Fura, R. and Mahmutoglu, S. (2010e). *Scour Critical Bridge—Plan of Action 012601*. Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Google (2016). *Google Inc.* <<https://www.google.com/maps>>. (Aug. 15, 2016).
- Grahl, C. (2013). “Four Basic Total Station Functions.” *Allen Precision Equipment*, <<http://www.allenprecision.com/page/4-basic-total-station-functions/>>. (Sept. 25, 2016).
- Guo, J., Suaznabar, O., Shan, H., and Shen, J. (2012). “Pier Scour in Clear-Water Conditions with Non-Uniform Bed Materials.” *Federal Highway Administration (FHWA)*, McLean, VA.
- Hanrahan, R. (2012). “Misquamicut Unrecognizable After Sandy.” *NBC Connecticut*, <<http://www.nbcconnecticut.com/news/local/Misquamicut-Unrecognizable-After-Sandy-177367451.html>> (Nov. 16, 2016).
- NOAA, (N.D.a) “Rhode Island Statewide Precipitation.” *National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center, National Environmental Satellite, Data, and Information Service (NESDIS)* <<https://www.ncdc.noaa.gov/sotc/service/national/timeseries01/037-00/201003-201003.gif>>. (Nov. 16, 2016).
- NOAA. (N.D.b) “Phase Differencing Bathymetric Sonar.” *NOAA Office of Coast Survey*, <<http://www.nauticalcharts.noaa.gov/csdl/PDBS.html>>. (Jun. 20, 2016)

- NOAA. (2015). “What is LIDAR.” *National Ocean Service National Oceanic and Atmospheric Administration*, < <http://oceanservice.noaa.gov/facts/lidar.html>>. (Sept. 25, 2016).
- NOAA (2016). “Side Scan Sonar.” *NOAA Office of Coast Survey*, <<http://www.nauticalcharts.noaa.gov/hsd/SSS.html>>. (Sept. 25, 2016).
- NWS, (N.D.). “Flooding in Rhode Island.” *National Oceanic and Atmospheric Administration (NOAA), National Weather Service (NWS)*. <<http://www.floodsafety.noaa.gov/states/ri-flood.shtml>>. (Nov. 16, 2016).
- Laursen, E. M. (1960). “Scour at Bridge Crossings.” *Journal of the Hydraulics Division*, 86 (2), 39-54.
- Laursen, E. M. (1963). “An Analysis of Relief Bridge Scour.” *Journal of the Hydraulics Division*, 89 (3), 93-118.
- Lombard, P. J., and Hodgkins G. A. (2008). “Comparison of Observed and Predicted Abutment Scour at Selected Bridges in Maine.” *United States Geological Survey (USGS)*, Reston, VA.
- Mueller, D. S. and Wagner C. R. (2005). “Field Observations and Evaluations of Streambed Scour at Bridges.” *Federal Highway Administration (FHWA)*, McLean, VA.
- Pechillo, T. H. J., (2010). *Supplemental Scour Inspection of BIN 004101*. Collins Engineers Inc, Rocky Hill, CT.
- Pechillo, T. H. J., (2012). *Underwater Inspection of Weekapaug Bridge*. Collins Engineers Inc, Rocky Hill, CT.
- Pechillo, T. H. J., (2013a). *Underwater Inspection of Bridge No. 020601*. Collins

- Engineers Inc, Rocky Hill, CT.
- Pechillo, T. H. J., (2013b). *Underwater Inspection of Weekapaug Bridge*. Collins Engineers Inc, Rocky Hill, CT.
- Pechillo, T. H. J., (2013c). *Underwater Inspection of Bridge No. 012601*. Collins Engineers Inc, Rocky Hill, CT.
- Richardson, E. V., Simons, D. B., and Julien, P. Y. (1990). “Highways in the River Environment Participant Notebook.” *Federal Highway Administration (FHWA)*, McLean, VA.
- Richardson, E. V., Simons, D. B., and Lagasse, P. F. (2001). “River Engineering for Highway Encroachments Highways in the River Environment.” *Federal Highway Administration (FHWA)*, Springfield, VA.
- RIGIS. (2011). “Spring 2011 Rhode Island Statewide LiDAR Data – Details.” *Rhode Island Geographical Information System (RIGIS)*,
<http://www.rigis.org/data/topo/2011/Spring_LiDAR_Project_Details>. (Sept. 25, 2016).
- Rocker, K., Jr., Thompson, D., Jung, B., Briaud, J. L., and Lin, S. (2011). “Handbook for Marine Geotechnical Engineering.” *NAVFAC Naval Facilities Engineering Command*, Port Hueneme, CA.
- Sauco, P., LaPlante, M., and Paull, N. (2015). *Scour Critical Bridge—Bridge Inspection Report 004101*. Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Trimble. (2014). “Trimble R10 GNSS System.” *Trimble*,
<http://trl.trimble.com/docushare/dsweb/Get/Document-625158/022543->

- 544E_TrimbleR10_DS_1014_LR.pdf>.(Sept. 25, 2016).
- Trimble. (2016). “VRS Now.” *Trimble*, < http://www.trimble.com/positioning-services/vrs-now.aspx?tab=How_it_Works>. (Sept. 25, 2016).
- USACE. (2002). *Coastal Engineering Manual (CEM)*, United States Army Corps of Engineers (USACE), Washington, DC.
- USGS. (2016a). “USGS 01117500 PAWCATUCK RIVER AT WOOD RIVER JUNCTION, RI.” *U.S. Geological Survey (USGS)*.
<http://waterdata.usgs.gov/ri/nwis/inventory/?site_no=01117500&agency_cd=USGS> (Oct. 10, 2016).
- USGS. (2016b). “USGS 01118000 WOOD RIVER AT HOPE VALLEY, RI.” *U.S. Geological Survey (USGS)*,
<http://waterdata.usgs.gov/ri/nwis/uv?site_no=01118000> (Sept. 14, 2016).
- USGS. (2016c). “USGS 01114500 WOONASQUATUCKET RIVER AT CENTERDALE, RI.” *U.S. Geological Survey (USGS)*.
<http://nwis.waterdata.usgs.gov/ri/nwis/peak/?site_no=01114500&agency_cd=USGS> (Oct. 10, 2016).
- USGS, (2016d). “Hurricane Sandy Data Viewer, Storm Tide Sensor 2351.” *U.S. Geological Survey (USGS)*. <<http://stn.wim.usgs.gov/sandy/>> (Oct. 15, 2016).
- Wadsworth, J., and Benn, D. (2016). *Bridge Inspection Report 012601*. Rhode Island Department of Transportation (RIDOT), Providence, RI.
- Webb, D. J., Anderson, N. L., Newton, T., Cardimona, S., and Ismail, A. (N.D.).
Ground Penetrating Radar (GPR): A Tool for Monitoring Bridge Scour, Rolla, MO.

- WHOI. (2016). “Chirp Sub-Bottom Profiler.” *Woods Hole Oceanographic Institute Coastal Systems Group*, <<http://web.whoi.edu/coastal-group/about/how-we-work/field-methods/chirp-sub-bottom-profiler/>>. (Sept. 25, 2016).
- Zarriello, P.J., Straub, D.E., and Smith, T.E. (2014a). “Simulated and Observed 2010 Floodwater Elevations in the Pawcatuck and Wood Rivers, Rhode Island.” *Scientific Investigations Report 2013–5193, U.S. Department of Homeland Security-Federal Emergency Management Agency and U.S. Geological Survey*, Reston, VA.
- Zarriello, P.J., Straub, D.E., and Westenbroek, S.M. (2014b). “Simulated and Observed 2010 Flood-Water Elevations in Selected River Reaches in the Moshassuck and Woonasquatucket River Basins, Rhode Island.” *Scientific Investigations Report 2013–5191, U.S. Department of Homeland Security-Federal Emergency Management Agency and U.S. Geological Survey*, Reston, VA.