Evaluation of Foundation Scour at the US 82 Eufaula Bridge

ALDOT Research Project 930-490

Prepared by

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ABSTRACT

A bridge site located at Eufaula over the Chattahoochee River on the border of the Alabama-Georgia state line has experienced significant scour at a specific pier as determined from underwater site inspections and soundings. The Alabama Department of Transportation requested a study to determine the reason for the excessive scour that has occurred. A finite element model was used to analyze the flow through the bridge for the large flood event of 1964. The model used in the analysis was the Finite Element Surface Water Modeling System (FESWMS) computer program. The flow depth and velocity distributions obtained with the flow model were used to estimate maximum scour depths based on equations suggested by Güven et al. (2002) for contraction scour and by HEC-18 (Federal Highway Administration, 2001) for complex pier scour. The computations show that excessive scour depths are indeed possible in the vicinity of the pier in question. Contraction scour and pier scour aggravated by the alignment of the flow are believed to be primary factors for the excessive scour.

ACKNOWLEDGMENTS

This project was supported by the Alabama Department of Transportation (Research Project No. 930-490) and administered by the Highway Research Center of Auburn University. The authors thank Dr. Frazier Parker, Professor of Civil Engineering and Director of the Highway Research Center, for his support of the project. The authors also thank the Bridge Scour Section of the Maintenance Bureau, the Materials and Tests Bureau of the Alabama Department of Transportation, and the Army Corp of Engineers for their help in data collection for the site presented in the report. Thanks are also given to Mrs. Lori Veal who aided in figure preparation of the report and Ms. Priscilla Clark who helped with the hardcopy and web publication of the report.

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I. INTRODUCTION

BACKGROUND:

A bridge site located at Eufaula over the Chattahoochee River has experienced significant scour at a specific pier as determined from underwater site inspections and soundings. The Alabama Department of Transportation requested a study to determine the reason for the excessive scour that has occurred. A finite element model study was conducted by Auburn University for the Alabama Department of Transportation (ALDOT). The study investigated the cause for excessive erosion at a specific pier at the bridge over the Chattahoochee River on U.S. 82. Model development and results of the study are discussed. And, a new method to calculate clear-water contraction scour using a two-dimensional finite element model method is presented.

The bridge site is located to the east of Eufaula over the Chattahoochee River connecting Eufaula, Alabama to Georgetown, Georgia (Figure 1). The river is approximately one mile wide at the site. There is a power dam located approximately 19.2 miles downstream of the site. It is controlled by the Army Corp of Engineers. The headwater of the dam forms the Walter F. George Reservoir (Figure 2). The bridge was constructed in 1962 on a new alignment (Figure 3). A smaller tributary named Chewalla Creek joins the river just upstream from the bridge. The drainage area at the site for the Chattahoochee River is 6,730 square miles. The drainage area at the site for Chewalla Creek is 26.7 square miles. Average daily inflow and outflow discharge measurements and stage records exist for the dam since its construction in 1963 to the present.

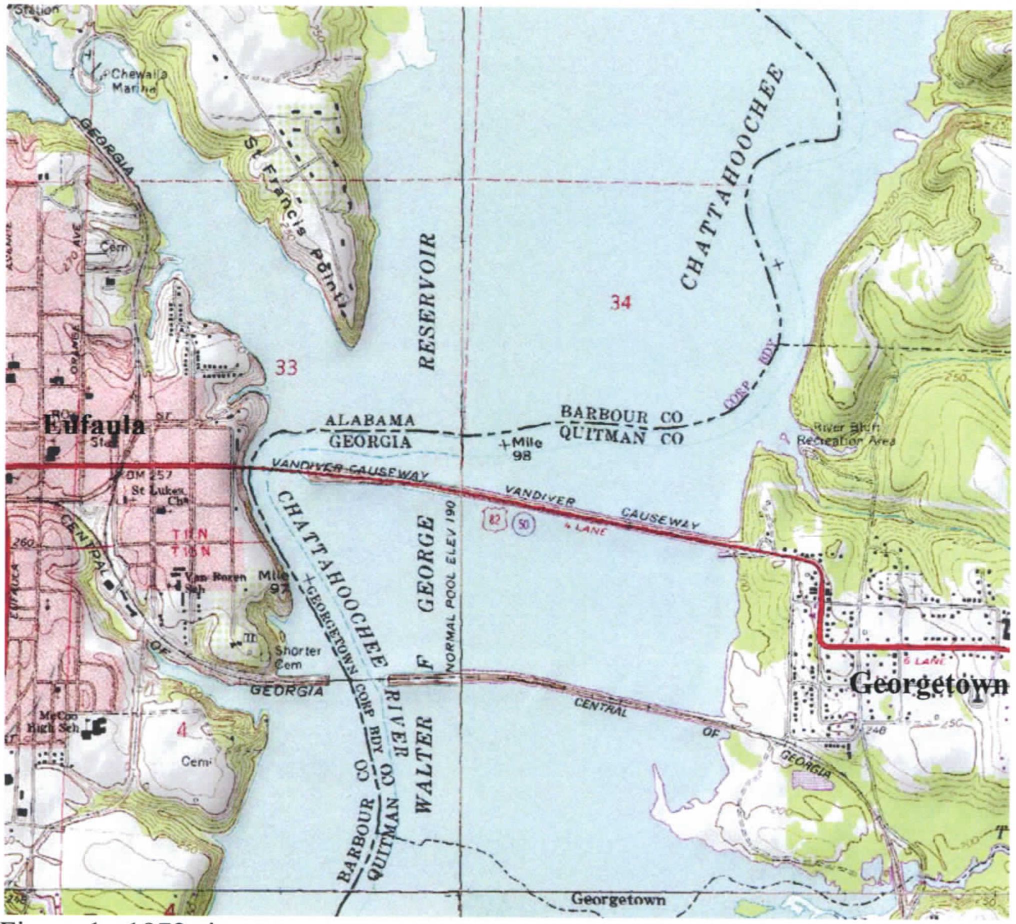

Figure 1. 1972 site map.

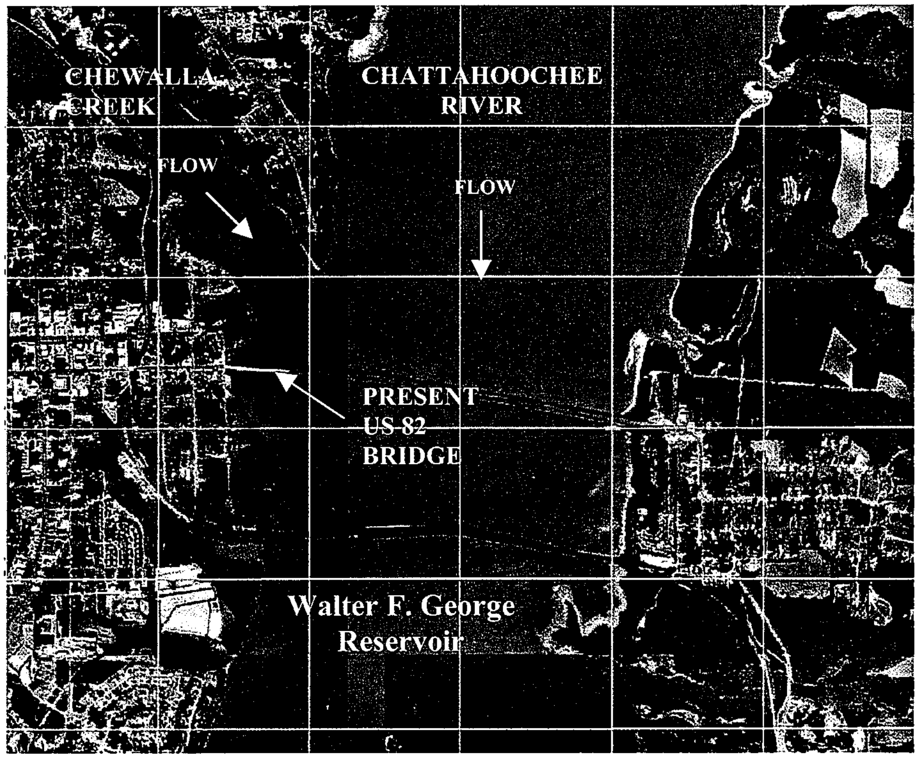

Figure 2. 2000 aerial photo existing conditions.

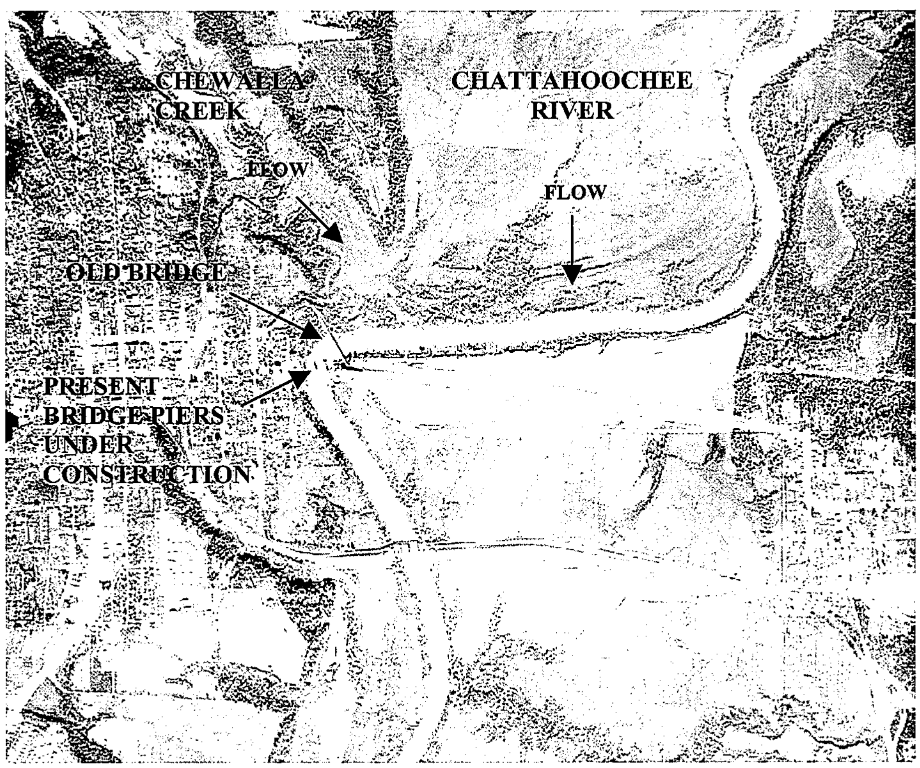

Figure 3. 1962 aerial photo with old alignment and existing bridge under construction.

The present US 82 bridge is 1,132 feet long with nine spans and eight piers (Figure 4). The core borings at the site describe the original ground at the pier in question to be a silty sand on top followed by a clayey silty sand below (Figure 5). Underwater inspection reports from 1971 indicated excessive scour at pier 6. This can be seen in Figure 6, which shows the original groundline dated 1964 (approximate ground elevation is 170 ft for this measurement at pier 6) and the 1971 sounding (approximate ground elevation at 30 feet downstream of the centerline of the bridge by pier 6 is 154 ft). Also, shown in Figure 6 is the measured depth at pier 6 observed by the divers who conducted the underwater inspection in 1971 (approximate elevation is 145 ft). Hydrologic data including discharges and water surface elevations were collected from the Army Corp of Engineers. The discharges were plotted from 1963 to 1971 and are shown in Figure 7. The plot indicates that the largest discharge for that time period occurred in 1964 (about 113,000 cfs, stage 189.3 ft). Assuming that the scour occurred very quickly then this flood would have produced the greatest scour for that time period. Riprap was added to protect the exposed piling and to try and arrest the scour. The riprap can be seen in the current soundings in Figure 8. The largest event of record was in 1990 (discharge 164,000 cfs, stage 194.4 ft), which corresponds approximately to a 100-year flood. The riprap provided protection against the higher stresses from the increased velocities of this flood.

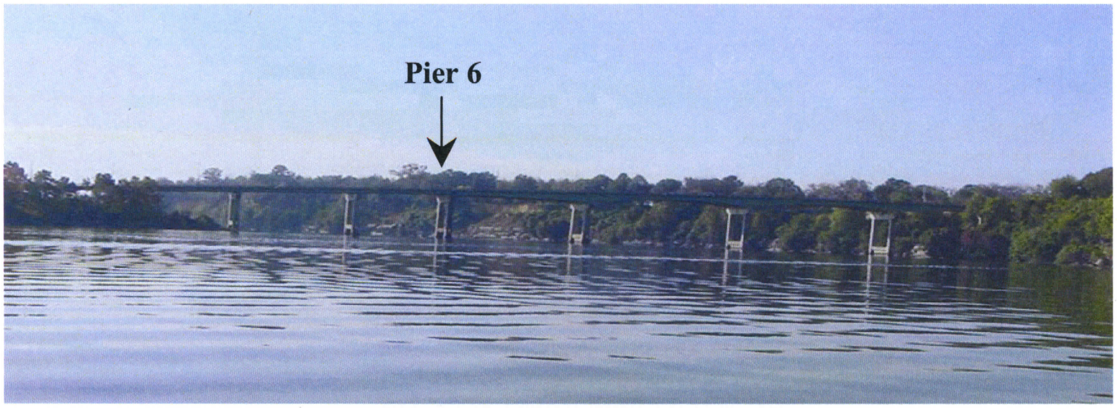

Figure 4. Elevation view of US 82 bridge looking downstream.

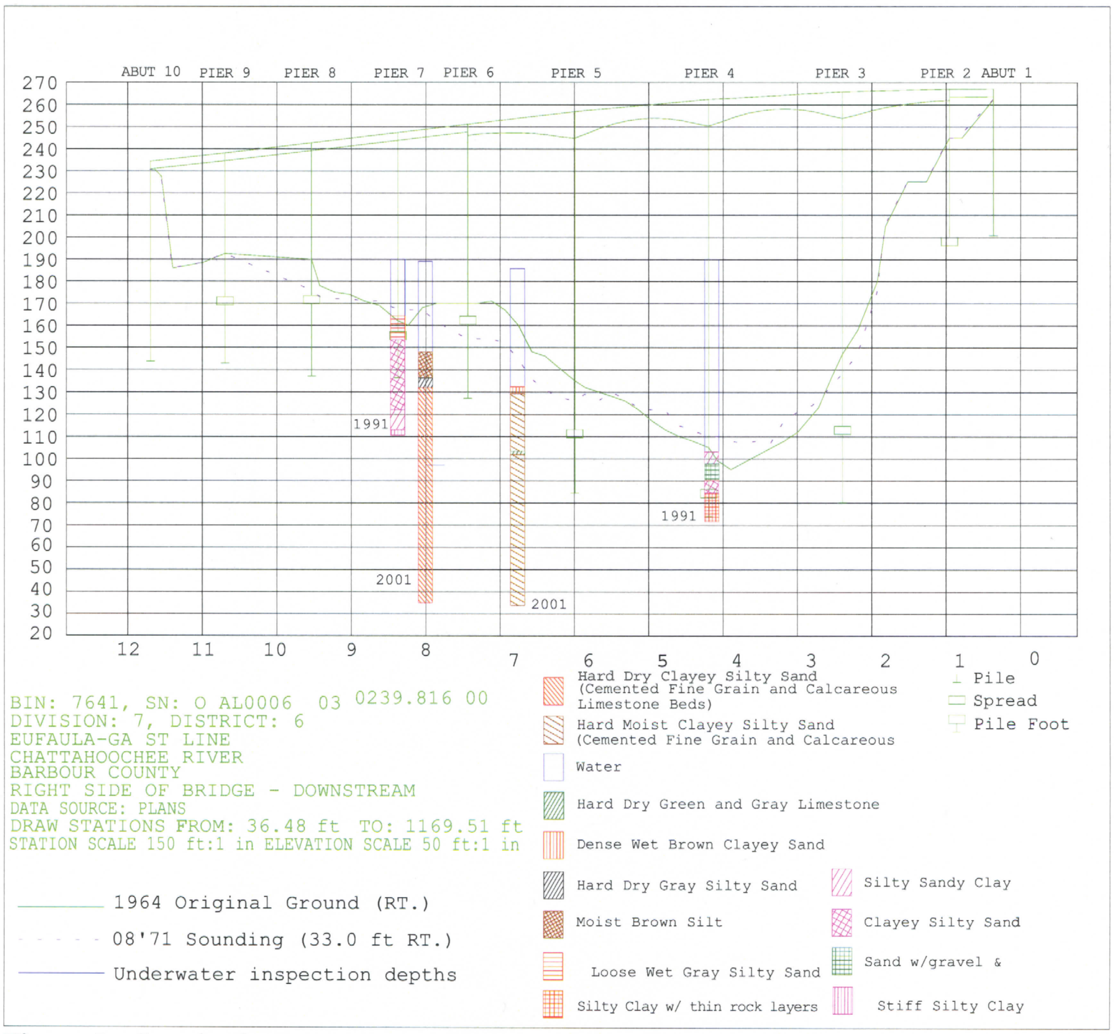

Figure 5. Core borings.

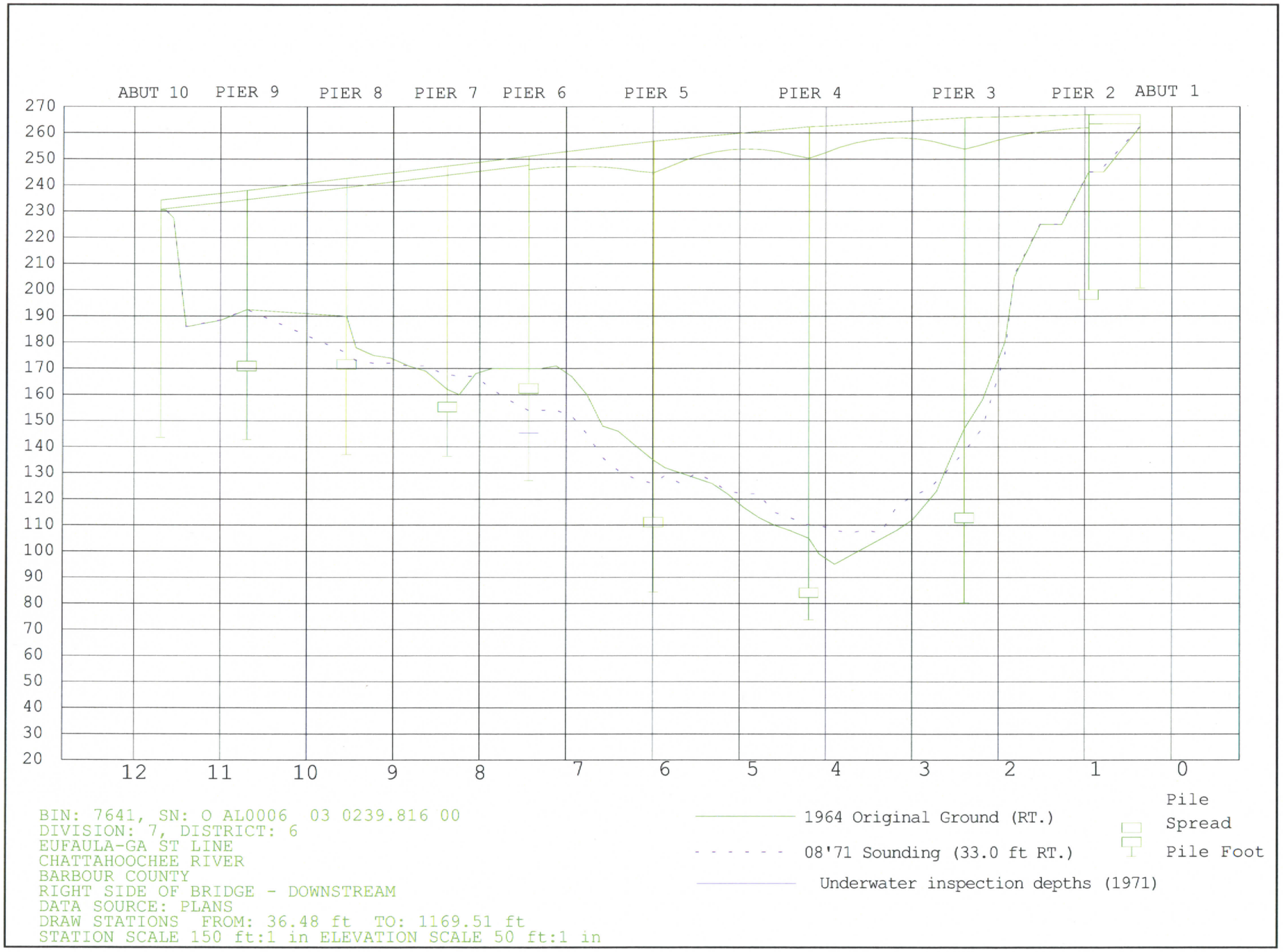

Figure 6. ABIMS plot with original groundline and 1971 and underwater inspection data (view is looking downstream).

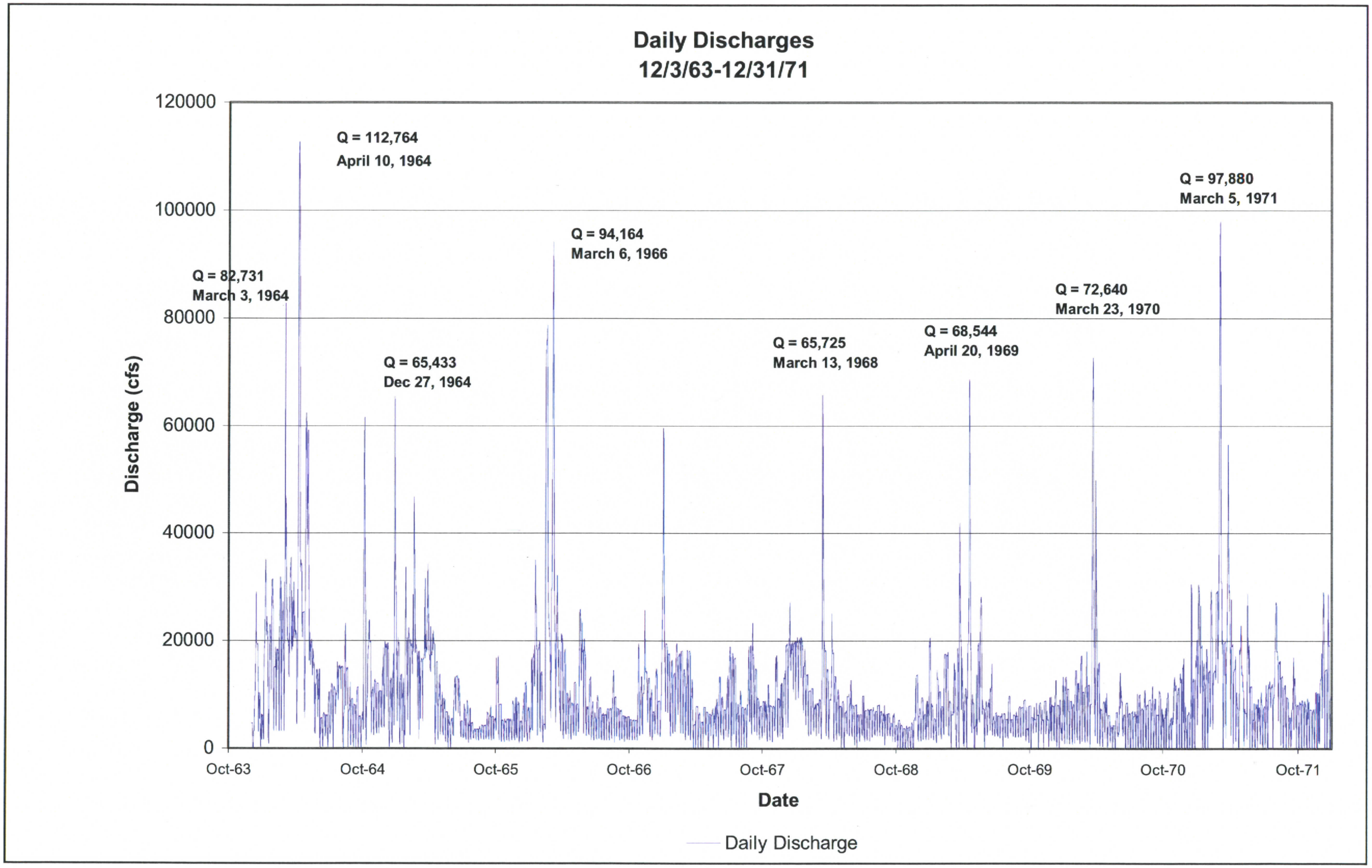

Figure 7. Daily discharges from 1963 to 1971.

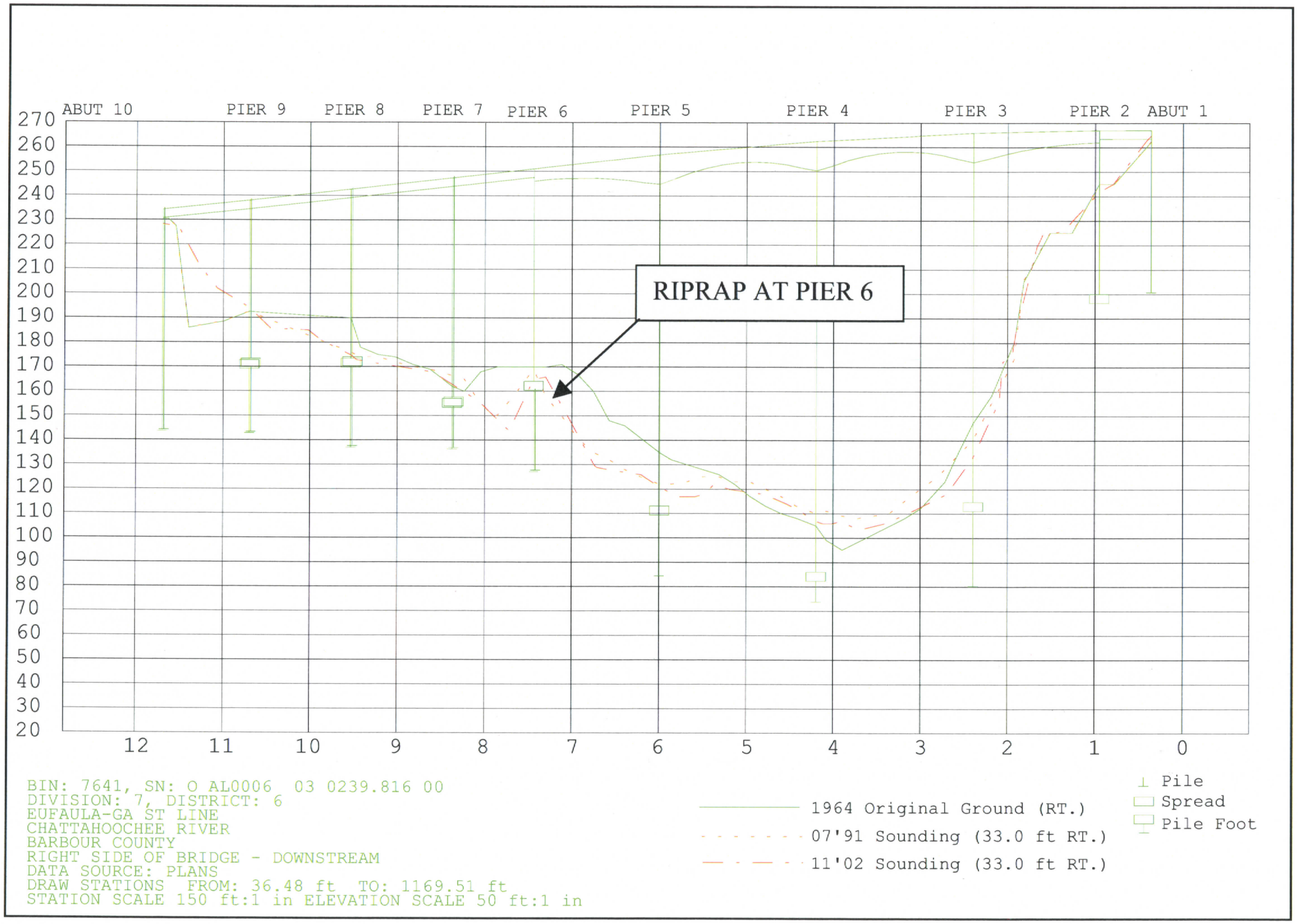

Figure 8. Riprap placed at pier 6 (view looking downstream).

II. Model Development and Scour Calculations

A detailed study was conducted on this site to investigate the reasons behind the excessive scour at pier 6. A 2-Dimensional **F**inite **E**lement **S**urface **W**ater **M**odeling **S**ystem (FESWMS) was used to model the flow at the site, along with a graphical user interface software, **S**urface Water **M**odeling **S**ystem (SMS). Advances in computers and software in the last fifteen years have made finite element modeling of surface water not only feasible, but also practical. Complex flow patterns, wide floodplains, meandering channels, and multiple bridge openings are just a few problems well suited for this type of modeling. The model was developed using information gathered from several sources, which included the Alabama Department of Transportation, the Army Corp of Engineers, the United States Geological Survey, and the Auburn University Library.

A hydrographic survey was conducted by ALDOT using a global positioning system interfaced with a fathometer. The survey was processed and compared to an early quadrangle map to ensure accuracy. A partial contour of the bridge site was taken from the plans and was geo-referenced, digitized and combined with the hydrographic survey to determine the bathymetry. Digital quadrangle maps and aerial photos were collected and geo-referenced to the same coordinate system of the survey. Roadway and bridge plans of the site were collected and used for the roadway profile and pier locations in the model. Foundations were determined from the bridge plans, which were made available by ALDOT.

First a mesh was developed (Figure 9) defining the domain of the model of the site. The bathymetry was then interpolated to the mesh (Figure 10) defining the geometry of the model. Manning's "n" roughness values were assigned to the elements using the geo-referenced aerial photo. Initial boundary conditions, discharge, and stage were determined based on the largest flood (namely, April 1964 flood) recorded during the life of the bridge before 1971. The velocity distribution obtained from the model is shown in Figure 11.

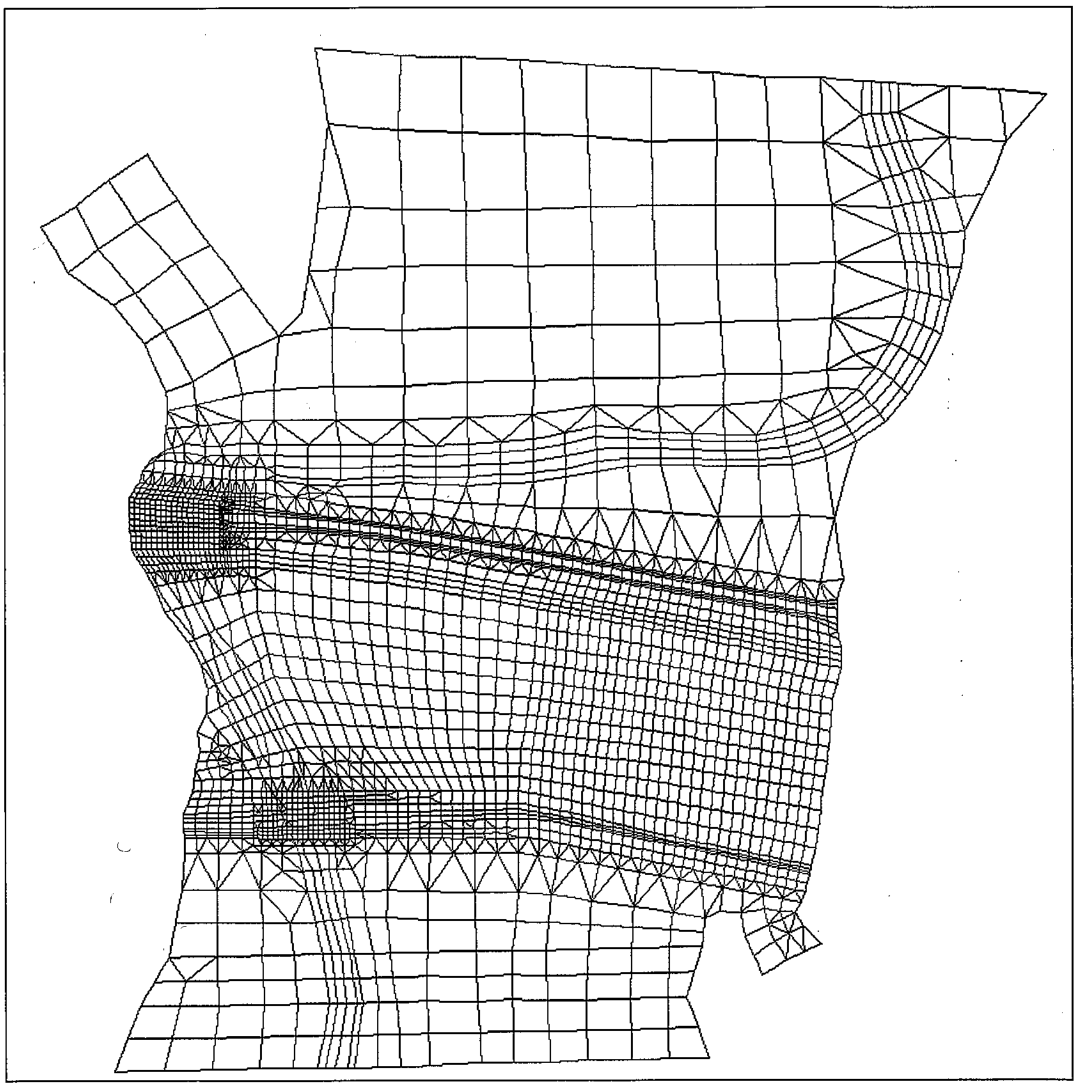

Figure 9. Finite element mesh.

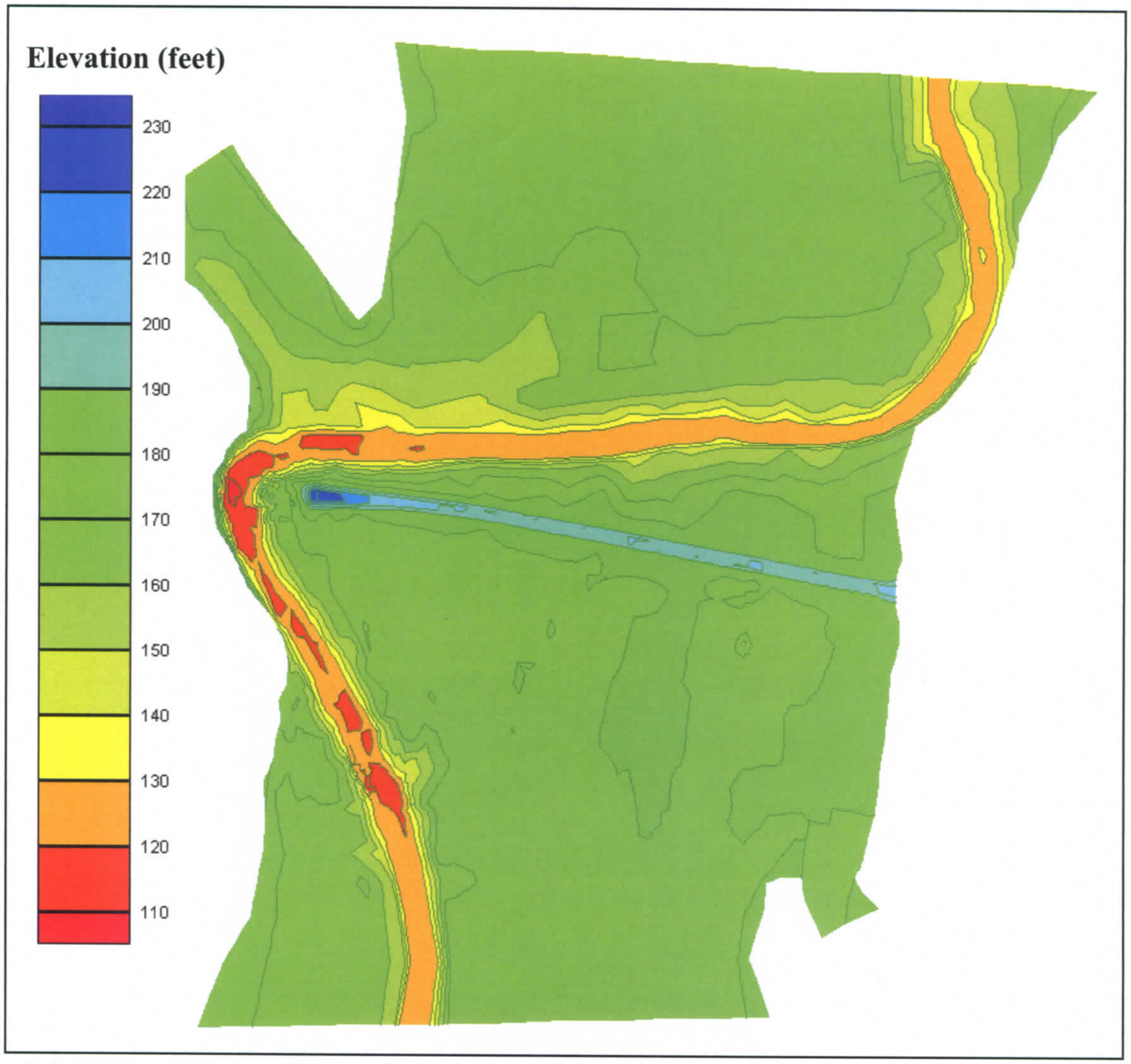

Figure 10. Elevation contour map of site.

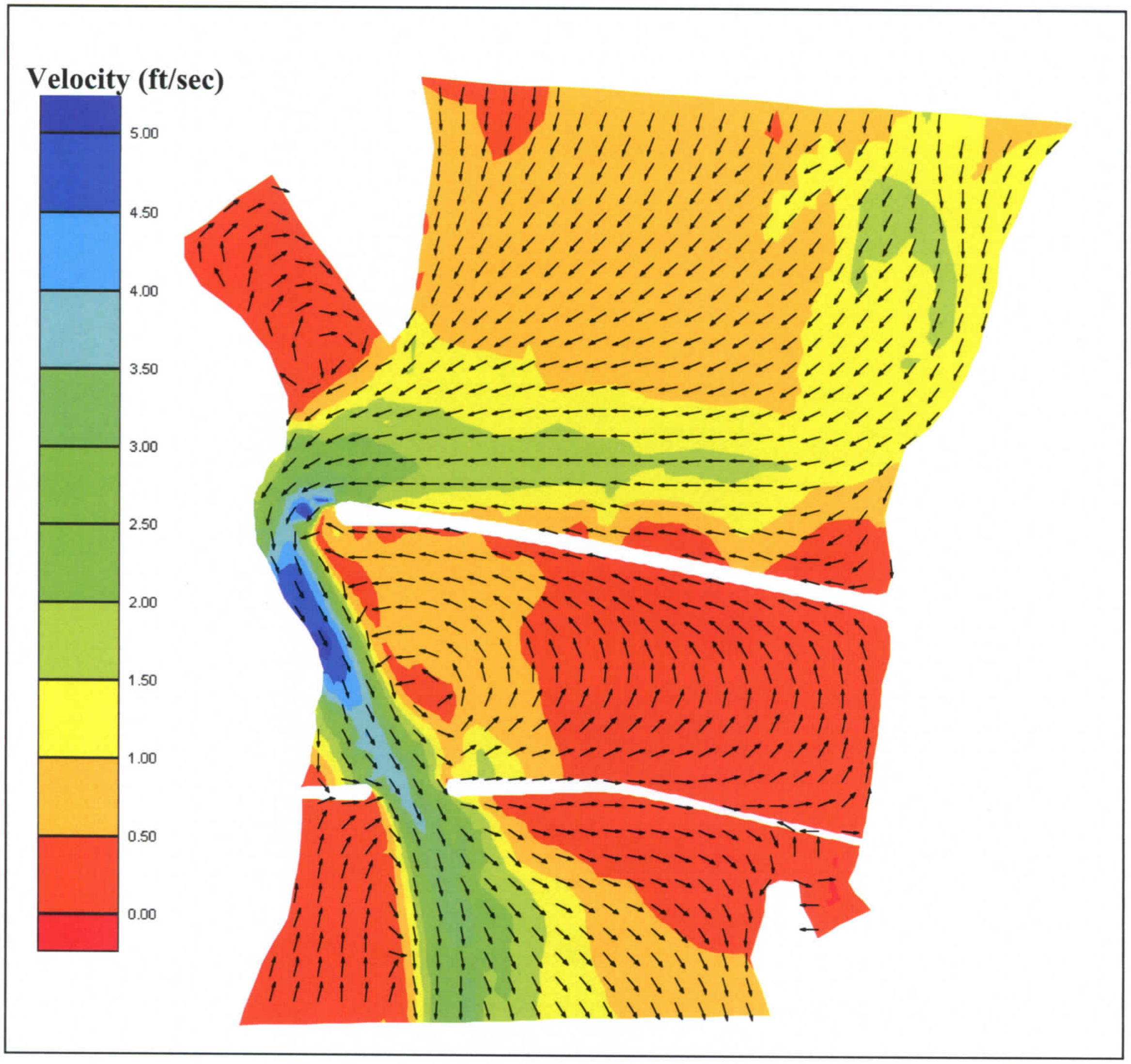

Figure 11. Velocity contours and flow directions of the 1964 flood.

Soil borings indicated the top layer of soil adjacent to the scoured layer at pier 6 was a loose wet gray silty sand. Below this layer was a clayey silty sand followed by a sandy clay. Scour calculations were performed using the equations suggested by Güven et al. (2002) for contraction scour and by Hydraulic Engineering Circular 18, HEC-18 (Fourth Edition, FHWA, 2001) for complex pier scour. The methods described below were taken directly from these sources.

Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction of the stream channel or by a bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction. If there is no bed material transported into the contraction or if the material transported is mostly in suspension the type of scour is called clear-water scour (HEC-18, Fourth Edition, FHWA, 2001, p. 5.1). Due to the very low flow velocities in the reservoir upstream of the contraction it is assumed in this study that the scour in the contraction is clear-water scour.

According to Güven et al.(2002), under clear-water conditions the maximum depth of flow is related to the critical shear stress and the friction factor through the following equation.

$$y_{\max}^2 = \frac{\rho q^2}{8\tau_c} f\left(\text{Re}, k_s, y_{\max}\right) \qquad (1)$$

where:

y_{max}	=	Maximum flow depth, ft
ρ	=	Fluid density, slugs/ft^3
q	=	Unit discharge, ft^2/s
τ_c	=	Critical shear stress, lb/ft^2
f	=	friction factor $f(\mathrm{Re}, k_s, y_{\max})$

Güven et al. (2002) used the following formula to find the friction factor:

$$f(\mathrm{Re}, k_s, y_{\max}) = \frac{0.25}{\left[\log\left(\frac{\frac{k_s}{3.7}}{4y_{\max}} + \frac{5.74}{\mathrm{Re}^{0.9}}\right)\right]^2} \tag{2}$$

where:

Re	=	Reynolds number
k_s	=	Effective roughness height

Due to the uncertainty associated with the roughness of the bed materials and partly due to the uncertainty associated with the friction formula itself, in this study instead of using this equation it was simply assumed that the friction factor was 0.008. (This value of the friction factor (0.008) is the smallest value displayed in the Moody diagram (see, e.g., Roberson and Crowe, 1993, p. 428). For the range of bed roughness and high Reynolds numbers of this flood the friction factor derived from equation 2 is probably within 15% of this value. Hence, the maximum flow depths which would be obtained based on equation 1 would be within 7% of the maximum flow depths obtained in this study with $f = 0.008$). If the flow conditions in the contraction are such that the initial value of the flow depth y_i is greater than the maximum flow depth y_{max}, which corresponds to the critical shear stress τ_c, this means that the bed shear stress τ_i, which corresponds to the depth y_i, is smaller than the critical shear stress ($\tau_i < \tau_c$) and there is no scour in the contraction. If, however $y_i < y_{max}$, this means that $\tau_i > \tau_c$ and scour occurs in the contraction until the maximum flow depth y_{max} is reached.

If the velocity head is neglected in the analysis, the corresponding maximum scour depth, s_{1max}, may be obtained using

$$s_{1\max} = y_{\max} - y_i \quad (4)$$

Pier Scour

In this study the pier scour was calculated using the procedure described in HEC-18 (Fourth Edition, FHWA, 2001). This procedure presented below very closely follows the description presented in HEC-18 (Fourth Edition, FHWA, 2001).

Superposition of Scour Components Method of Analysis

The components of a complex pier are illustrated in Figure 12. Note that the pile cap can be above the water surface, at the water surface, in the water, or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group.

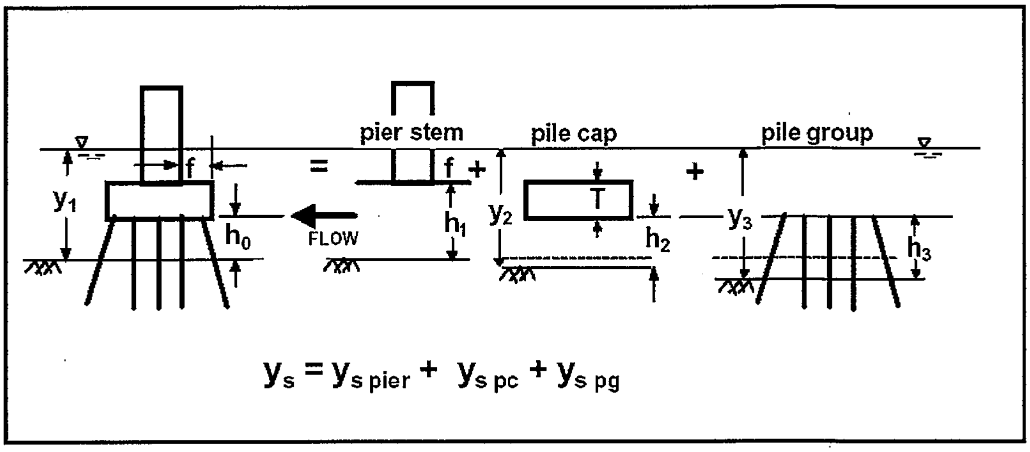

Figure 12. Definition sketch for scour components for a complex pier.

where:

f	=	Distance between front edge of pile cap or footing and pier, (ft)
h_o	=	Height of the pile cap above bed at beginning of computation, (ft)
h_1	=	$h_o + T$ = height of the pier stem above the bed before scour, (ft)
h_2	=	$h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, (ft)
h_3	=	$h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after the pier stem and pile cap scour components have been computed, (ft)
S	=	Spacing between columns of piles, pile center to pile center, (ft)
T	=	Thickness of pile cap or footing, (ft)
y_1	=	Approach flow depth at the beginning of computations, (ft)
y_2	=	$y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations (ft)
y_3	=	$y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, (ft)
V_1	=	Approach velocity used at the beginning of computations, (ft/sec)
V_2	=	$V_1(y_1/y_2)$ =adjusted velocity for pile cap computations, (ft/sec)
V_3	=	$V_1(y_1/y_3)$ =adjusted velocity for pile group computations, (ft/sec)

Total scour from superposition of components is given by:

$$y_s = y_{s\ pier} + y_{s\ pc} + y_{s\ pg} \qquad (5)$$

where:

y_s	=	Total scour depth, (ft)
$y_{s\ pier}$	=	Scour component for the pier stem in the flow, (ft)
$y_{s\ pc}$	=	Scour component for the pier cap or footing in the flow, (ft)
$y_{s\ pg}$	=	Scour component for the piles exposed to the flow, (ft)

Each of the scour components is computed from the basic pier scour equation using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 12, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections guidance for calculating each of the components is given. The height adjustment is included in the equivalent pier size for the pile cap.

There are three components to calculating complex pier scour. Pier stem scour is calculated first followed by the pile cap scour and then finally the pile group scour. The three pier scour components are summed and added to the contraction scour to obtain the total scour. Scour depths will be increased or decreased depending on certain parameters including, shape of the pier, angle of attack, width of the pier, velocity, and depth.

Pier Stem Scour

The need to compute the pier stem scour depth component occurs when the pier cap or the footing is in the flow and the pier stem is subjected to sufficient flow depth and velocity as to cause scour. The first computation is the scour estimate, $y_{s\ pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation (Equation 6).

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43} \qquad (6)$$

where:

y_s = Depth of scour in feet
K_1 = Correction factor for pier nose shape
K_2 = Correction factor for angle of attack of flow
K_3 = Correction factor for bed condition
K_4 = Correction factor for armoring of bed material
a = Pier width in feet
y_1 = Flow depth directly upstream of the pier in feet
Fr_1 = Froude Number directly upstream of the pier

For round nose piers aligned with the flow, the maximum scour depth is limited as follows:

$y_s \leq 2.4$ times the pier width (a) for $Fr_1 \leq 0.8$
$y_s \leq 3.0$ times the pier width (a) for $Fr_1 > 0.8$

The correction factor for pier nose shape, K_1, is shown in Table 1.

Table 1. Correction factor for pier nose shape, K_1.

Shape of Pier Nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinder	1.0
Group of cylinders	1.0
Sharp nose (triangular)	0.9

The correction factor for the attack of the flow, K_2, is calculated using the equation shown in HEC-18 (Fourth Edition, FHWA, 2001, p. 6.4):

$$K_2 = \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65} \qquad (7)$$

where:

L = Length of the pier along the flow line, ft.
θ = Angle of attack of the flow, with respect to the pier.

or can be determined from Figure 13.

Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, m			

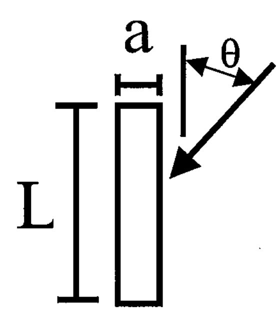

Figure 13. Correction factor for angle of attack on pier, K_2.

The correction factor for bed condition, K_3, is shown in Table 2.

Table 2. Correction factor for bed condition, K_3.

Bed Condition	**Dune Height H feet**	K_3
Clear-Water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.1 to 1.2
Large Dunes	H 30	1.3

The correction factor K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 0.20 feet. The correction factor results from recent research, which showed that when velocity (V_1) is less than the critical velocity (V_{c90}) of the D_{90} size of the bed material, and there is a gradation in sizes in the bed material, the D_{90} will limit the scour depth. The equations are presented in HEC-18 (Third Edition, FHWA, 1993, pp. 37-38):

$$K_4 = \left[1 - 0.89(1 - V_R)^2\right]^{0.5} \qquad (8)$$

where:

$$V_R = \left[\frac{V_1 - V_i}{V_{c90} - V_i}\right] \quad (9)$$

$$V_i = 0.645\left[\frac{D_{50}}{a}\right]^{0.053} V_{c50} \quad (10)$$

V_R = Velocity ratio

V_1 = Average velocity in the main channel or overbank area at the cross section just upstream of the bridge, ft/s

V_i = Velocity when particles at a pier begin to move, ft/s

V_{c90} = Critical velocity for D_{90} bed material size, ft/s

V_{c50} = Critical velocity for D_{50} bed material size, ft/s

a = Pier width, ft

$$V_c = 10.95 Y^{1/6} D_c^{1/3} \quad (11)$$

where:

Y = The depth of water just upstream of the pier, ft

D_c = Critical particle size for critical velocity V_c, ft

Limiting K_4 values and bed material size are given in Table 3.

Table 3. Limits for bed material size and K_4 values.

Factor	**Minimum Bed Material Size**	**Minimum K_4 Value**	**V_R>1.0**
K_4	$D_{50} \geq 0.2$ ft	0.7	1.0

The basic scour estimate given by equation 6 is multiplied by a factor, $K_{h\ pier}$, given in Figure 15 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component as follows:

$$\frac{y_{spier}}{y_1} = K_{hpier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right] \qquad (12)$$

where:

K_{hpier} = Coefficient to account for the height of the pier stem above the bed and the shielding effect by the pile cap overhang distance "f" in front of the pier stem (from Figure 14)

a_{pier} = Width of the pier stem, ft

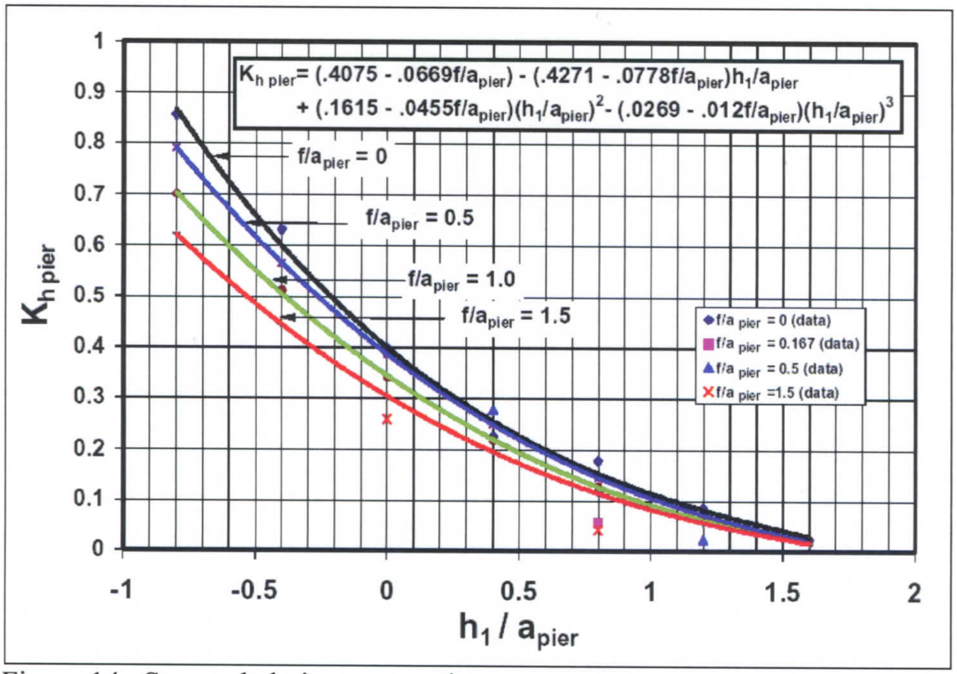

Figure 14. Suspended pier scour ratio.

The quantity in the square brackets in Equation 12 is the basic pier scour ratio as if the pier stem were full depth and extended below the scour.

Pile Cap (Footing) Scour

The need to compute the pile cap or footing scour depth component occurs when the pile cap is in the flow by design, or as the result of long-term degradation, contraction scour, and/or by local scour attributed to the pier stem above it. As described below, there are two cases to consider in estimating the scour caused by the pile cap (or footing). Equation 6 is used to estimate the scour component in both cases, but the conceptual strategy for determining the variables to be used in the equation is different (partly due to limitations in the research that has been done to date). In both cases the wide pier factor, K_w, may be applicable for this computation. Only case 1 will be discussed here.

Case 1: The bottom of the pile cap is above the bed and in the flow either by design or after the bed has been lowered by scour caused by the pier stem component. The strategy is to reduce the pile cap width, a_{pc}, to an equivalent full depth solid pier width, a^*_{pc}, using Figure 15.

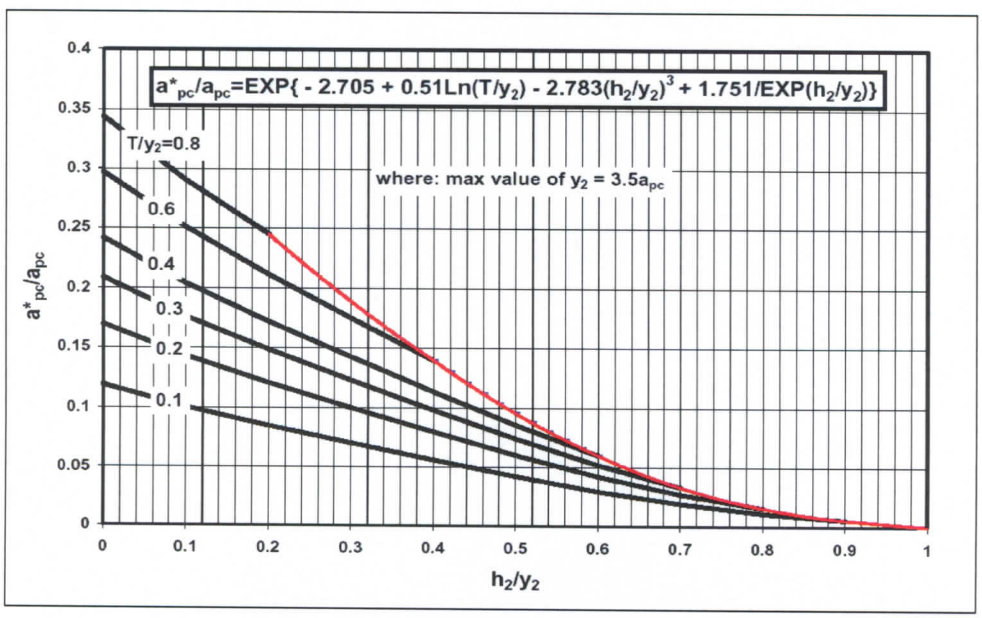

Figure 15. Pile cap (footing) equivalent width.

The equivalent pier width, an adjusted flow depth, y_2, and an adjusted flow velocity, V_2, are then used in Equation 6 to estimate the scour component.

Case1. Bottom of the pile cap (footing) in the flow above the bed

T	=	Thickness of the pile cap exposed to the flow, (ft)
h_2	=	$h_o+y_{spier}/2$, (ft)
y_2	=	$y_1+y_{spier}/2$, = adjusted flow depth, (ft)
V_2	=	$V_1(y_1/y_2)$, = adjusted flow velocity, (ft/s)

where:

h_o	=	Original height of the pile cap above the bed, (ft)
y_1	=	Original flow depth at the beginning of the computation, (ft)
y_{spier}	=	Pier stem scour depth component, (ft)
V_1	=	Original approach velocity at the beginning of the computation, (ft)

Determine a^*_{pc}/a_{pc} from Figure 16 as a function of h_2/y_2 and T/y_2 (note that maximum value of $y_2 = 3.5a_{pc}$). Compute $a^*_{pc} = (a^*_{pc}/a_{pc})\ a_{pc}$; where a^*_{pc} is the width of the equivalent pier to be used in Equation 6 and a_{pc} is the width of the original pile cap. Compute the pile cap scour component, $y_{s\ pc}$ from Equation 6 using a^*_{pc}, y_2, and V_2 as the pier width, flow depth, and velocity parameters, respectively. The rationale for using the adjusted velocity for this computation is that the near bottom velocities are the primary currents that produce scour and they tend to be reduced in the local scour hole from the overlying component. For skewed flow use the L/a for the original pile cap as the L/a for the equivalent pier to determine K_2. Apply the wide pier correction factor, K_w, if (1) the total depth, $y_2 < 0.8\ a^*_{pc}$, (2) the Froude Number $V_2/(g\ y_2)^{1/2} < 1$, and (3) $a^*_{pc} > 50\ D_{50}$. The scour component equation for the case 1 pile cap can then be written:

$$\frac{y_{spc}}{y_2} = \left[2.0K_1K_2K_3K_4\left(\frac{a^*_{pc}}{y_2}\right)^{0.65}\left(\frac{V_2}{\sqrt{gy_2}}\right)^{0.43}\right] \tag{13}$$

Pile Group Scour

The following method is used for determining pile group scour depth by taking into consideration the spacing between piles, the number of pile rows and a height factor to account for the pile length exposed to the flow. Guidelines are given for analyzing the following typical cases:

- Special case of piles aligned with each other and with the flow. No angle of attack.
- General case of the pile group skewed to the flow, with an angle of attack, or pile groups with staggered rows of piles.

The strategy for estimating the pile group scour component is the same for both cases, but the technique for determining the projected width of piles is simpler for the special case of aligned piles. The strategy is as follows:

- Project the width of the piles onto a plane normal to the flow.
- Determine the effective width of an equivalent pier that would produce the same scour if the pile group penetrated the water surface.
- Adjust the flow depth, velocity, and exposed height of the pile group to account for the pier stem and pile cap scour components previously calculated.
- Determine the pile group height factor based on the exposed height of the pile group above the bed.
- Compute the pile group scour component using a modified version of Equation 6.

Projected width of piles

For the special case of aligned piles, the projected width, a_{proj}, onto a plane normal to the flow is simply the width of the collapsed pile group as illustrated in Figure 16.

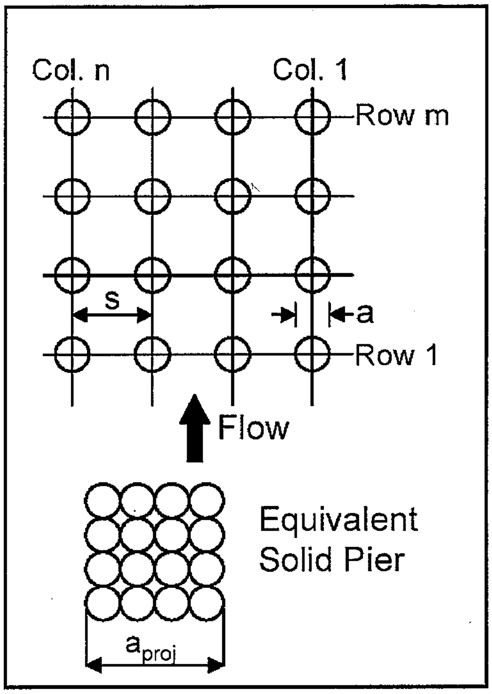

Figure 16. Projected width of piles for the special case of aligned flow.

For the general case, HEC-18 indicates that a pile group could be represented by an equivalent solid pier that has an effective width, a^*_{pg}, equal to a spacing factor multiplied by the sum of the non-overlapping projected widths of the piles onto a plane normal to the flow direction. The aligned pile group is a special case in which the sum of the non-overlapping projected widths happens to be the same as the width of the collapsed pile group. The procedure for the general case is the same as the procedure for the aligned pile groups except for the determination of the width of the equivalent solid which is a more tedious process for the general case. The sum of the projected widths can be determined by sketching the pile group to scale and projecting the outside edges of each pile onto the projection plane as illustrated in Figure 17 or by systematically calculating coordinates of the edges of each pile along the projection plane. The coordinates are sorted in ascending order to facilitate inspection to eliminate double counting of overlapping areas. Additional experiments are being conducted at the Federal Highway Administration (FHWA) hydraulics laboratory to test simpler techniques for estimating the effective width, but currently this summation technique is a logical choice. A reasonable method is to exclude piles other than the two rows and one column closest to the plane of projection as illustrated by the bold outlines in Figure 17.

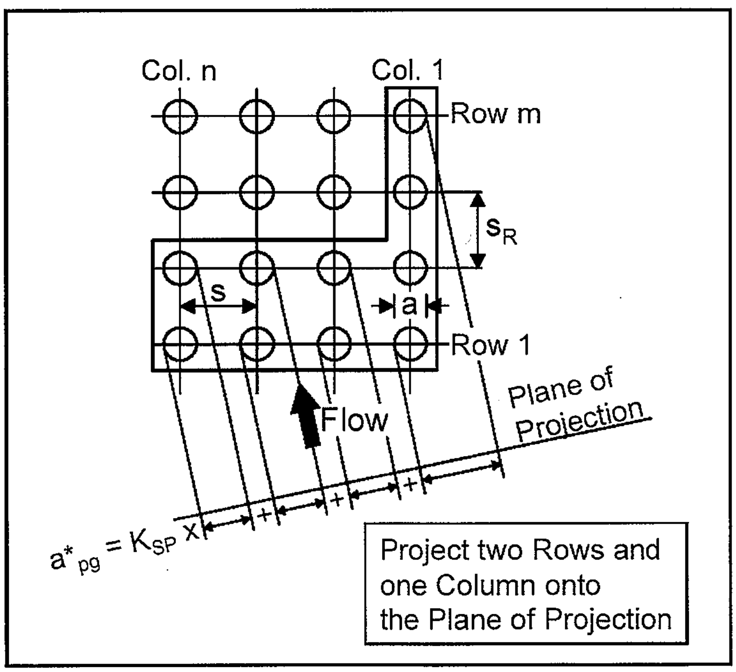

Figure 17. Projected width of piles for the general case of skew flow.

The effective width of an equivalent full depth pier is the product of the projected width of piles multiplied by a spacing factor and a number of aligned rows factor (used for the special case of aligned piles only).

$$a^*_{pg} = a_{proj} K_{sp} K_m \tag{14}$$

where:

a_{proj}	=	sum of non-overlapping projected widths of piles, ft
K_{sp}	=	Coefficient for pile spacing (Figure 18)
K_m	=	Coefficient for number of aligned rows, m (Figure 19 – note that K_m is constant for all S/a values when there are more than 6 rows of piles)
K_m	=	1.0 for skewed or staggered pile groups

The number of rows factor, K_m, is 1.0 for the general case of skewed or staggered rows of piles because the projection technique for skewed flow accounts for the number of rows and is already conservative for staggered rows.

Adjusted flow depth and velocity

The adjusted flow depth and velocity to be used in the pier scour equation are as follows:

$$y_3 = y_1 + y_{spier}/2 + y_{spc}/2, \text{ (ft)}$$

$$V_3 = V_1(y_1/y_3), \text{ (ft/s)}$$

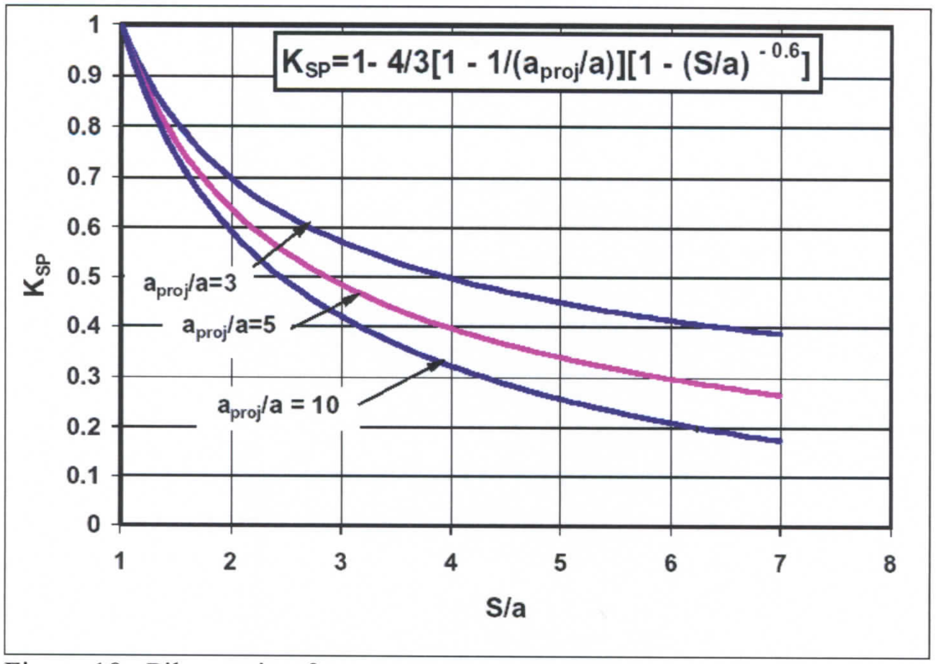

Figure 18. Pile spacing factor.

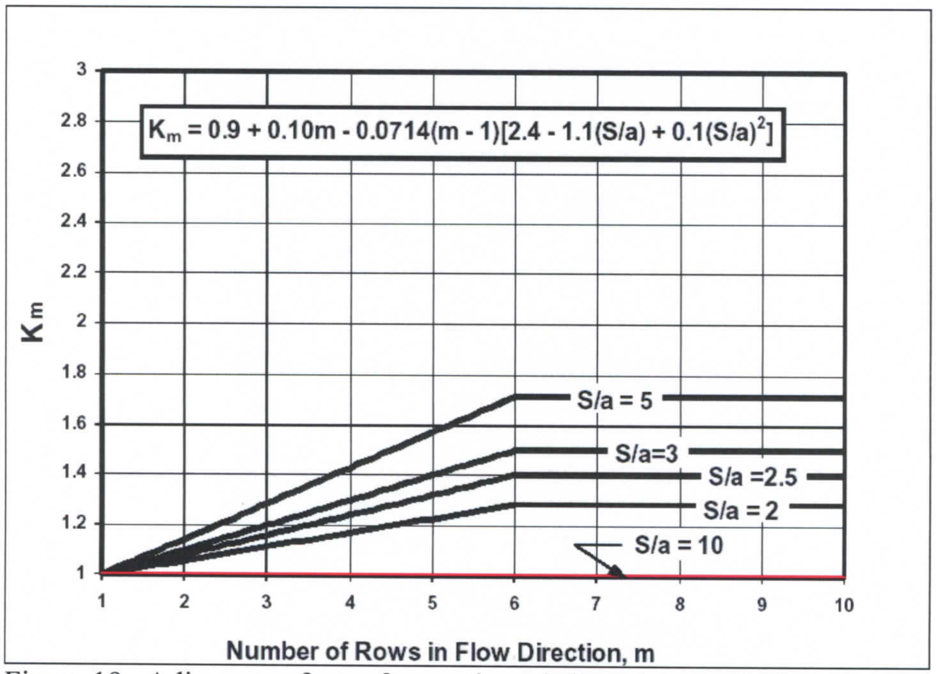

Figure 19. Adjustment factor for number of aligned rows of piles.

The scour equation for a pile group can then be written as follows:

$$\frac{y_{spg}}{y_3} = K_{hpg}\left[2.0K_1K_3K_4\left(\frac{a^*_{pg}}{y_3}\right)^{0.65}\left(\frac{V_3}{\sqrt{gy_3}}\right)^{0.43}\right] \tag{15}$$

where:

$K_{h\,pg}$ = Pile group height factor given in Figure 20 as a function of h_3/y_3 (note that the maximum value of $y_3 = 3.5\ a^*_{pg}$)

h_3 = $h_0 + y_{s\,pier}/2 + y_{s\,pc}/2$ = height of pile group above the lowered stream bed after pier and pile cap scour components have been computed, (ft)

K_2 from Equation 6 has been omitted because pile widths are projected onto a plane that is normal to the flow. The quantity in the square brackets is the scour ratio for a solid pier of width, a^*_{pg}, if it extended to the water surface. This is the scour ratio for a full depth pile group.

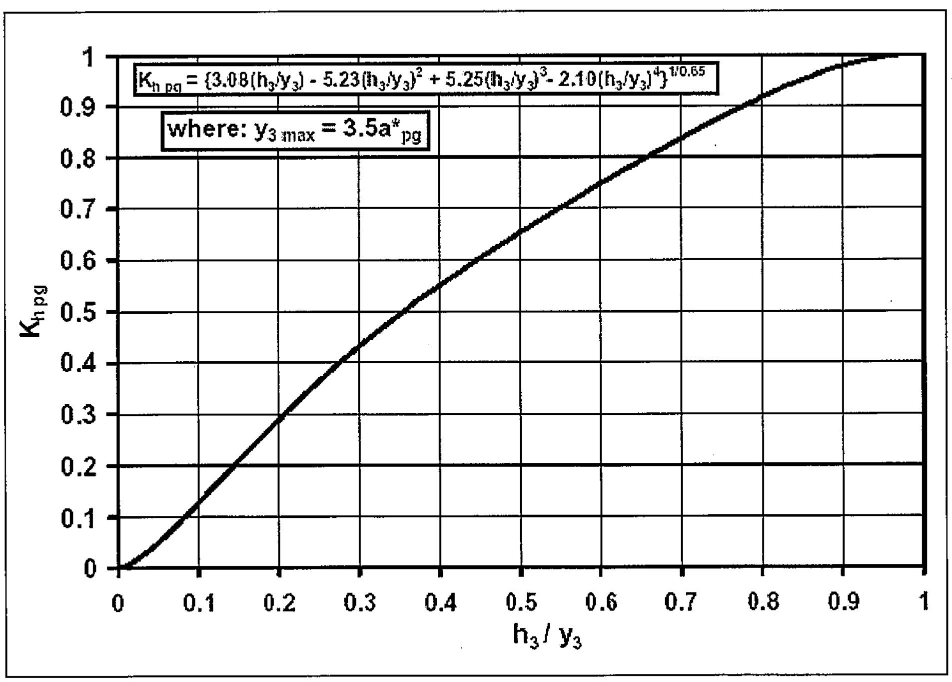

Figure 20. Pile group height adjustment factor.

Total Scour Depth for the Complex Pier

The total scour for the complex pier from Equation (16) is:

$$ys = y_{spier} + y_{spc} + y_{spg} \qquad (16)$$

Scour Calculations and Assumptions

First, contraction scour was calculated using the equations by Güven et al. (2002). Due to the lack of erosion function data for the scoured soil, scour was calculated using a range of critical shear stresses. The critical shear stress was assumed first to be equivalent of a fine sand (0.028 lb/ft^2). The next critical shear stress was assumed to be equivalent of a very fine sand and silt (0.005 lb/ft^2). In all cases in this study the friction factor (f) was assumed to be 0.008. This is the lowest value of the friction factor that is displayed in the classical Moody diagram shown in Engineering Fluid Mechanics (Fifth Edition, Roberson and Crowe, 1993, p. 428). The actual friction factor could be different from this, but a more detailed calculation was avoided due to the uncertainties associated with the available formulas for very high Reynolds numbers ($Re > 10^7$) as encountered in this study. Each critical shear stress and friction factor was applied to each node in the model. The model results showed scour in areas where scour did not occur in the field. But, also the model did indicate scour in the area where excessive scour depths did occur in the field. The results of the two calculations indicated maximum clear-water contraction scour depths of 8 feet (Figure 21) for the fine sand assumption and 40 feet (Figure 22) for the very fine sand and silt.

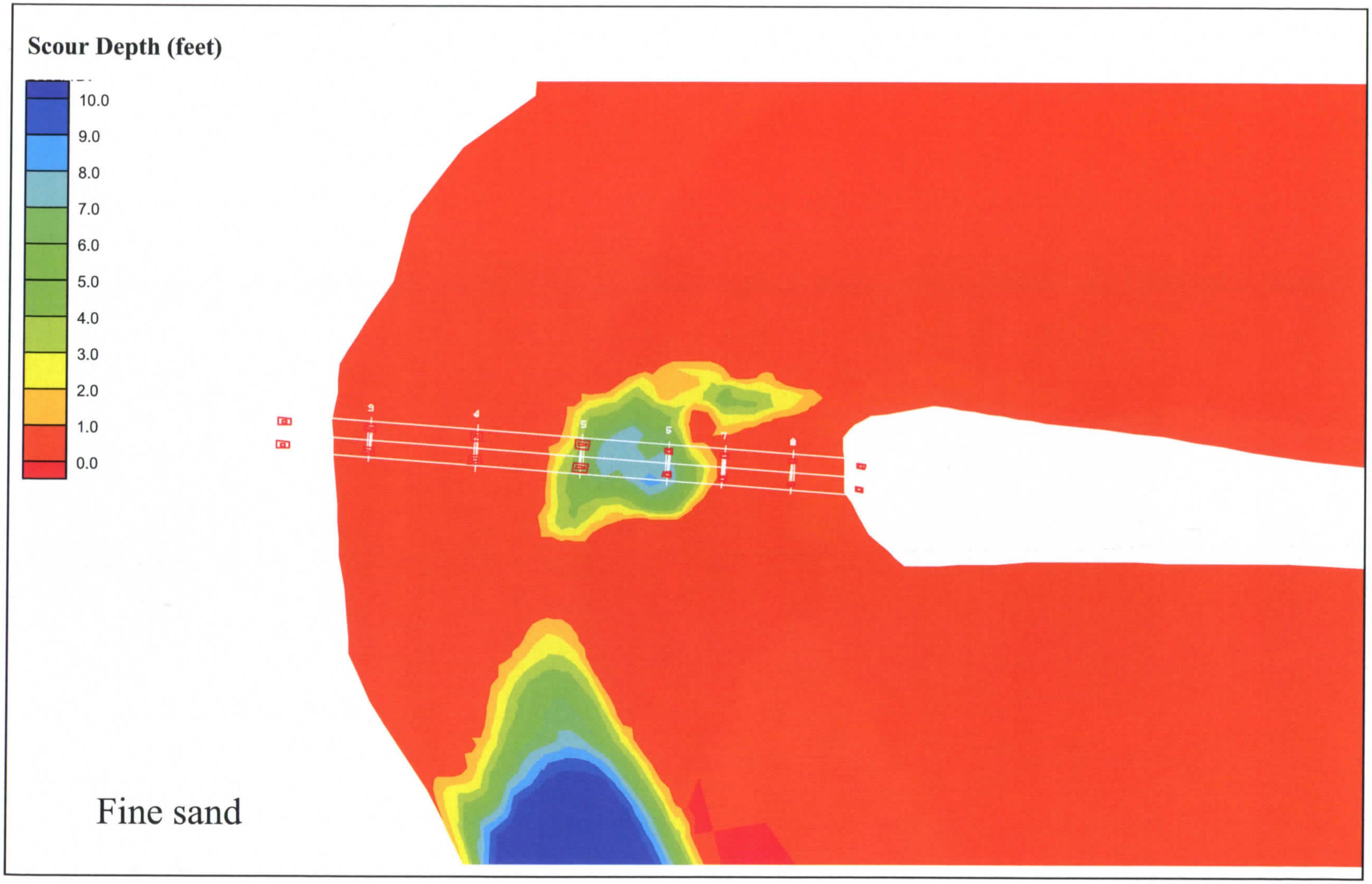

Figure 21. Maximum scour depth relative to the original ground calculated using a critical shear stress (τ_c) of 0.028 lb/ft^2 and a friction factor (f) of 0.008.

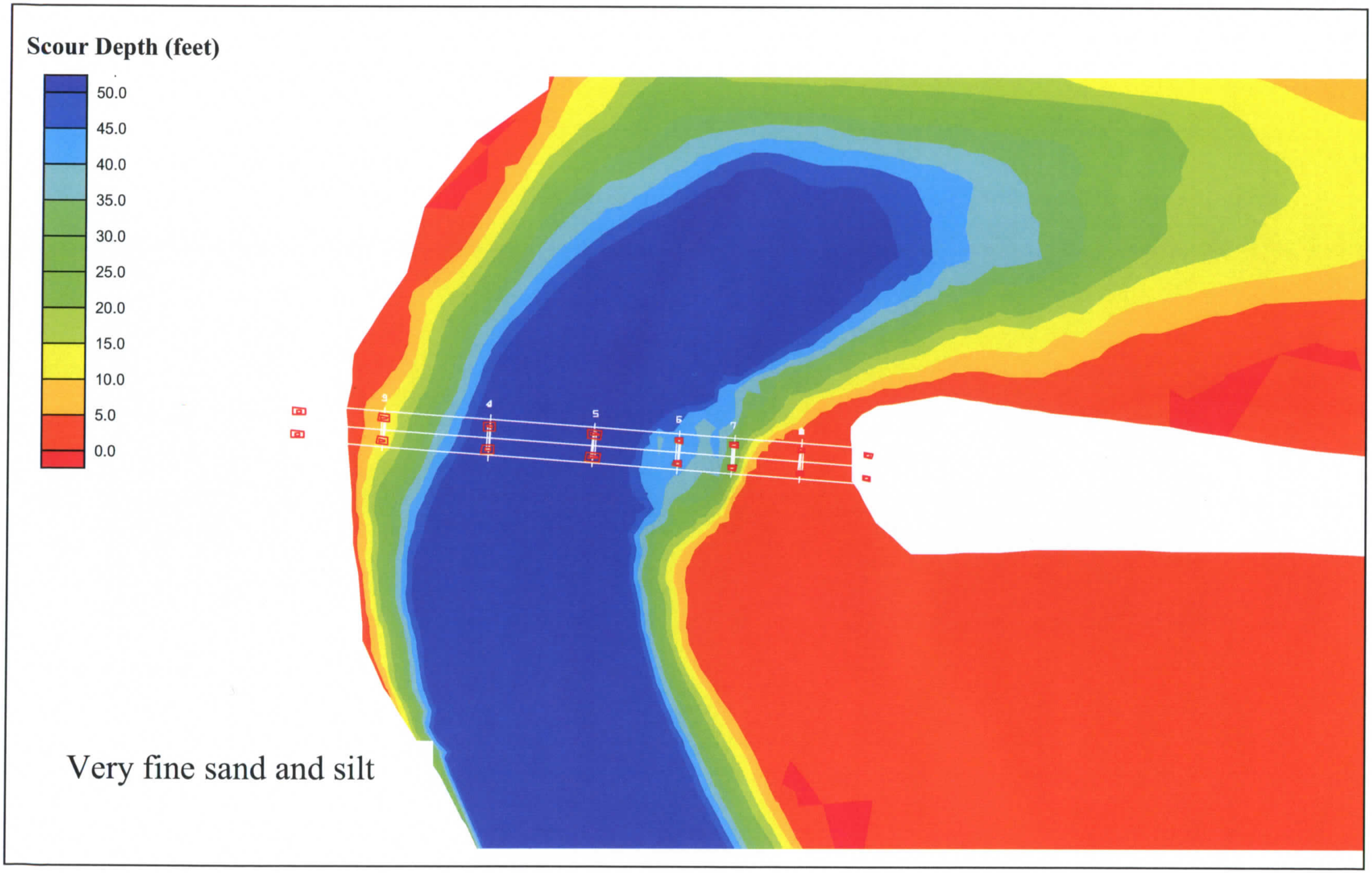

Figure 22. Maximum scour depth relative to the original ground calculated using a critical shear stress (τ_c) of 0.005 lb/ft^2 and a friction factor (f) of 0.008.

Next, pier scour was computed for Pier 6 using the complex pier scour methods from HEC-18. Pier 6 is a pile footing. Complex pier scour calculations were performed. Pier size and shape were obtained from the plans and the velocity, depth and flow angle were obtained from the 2-D model (Figure 23).

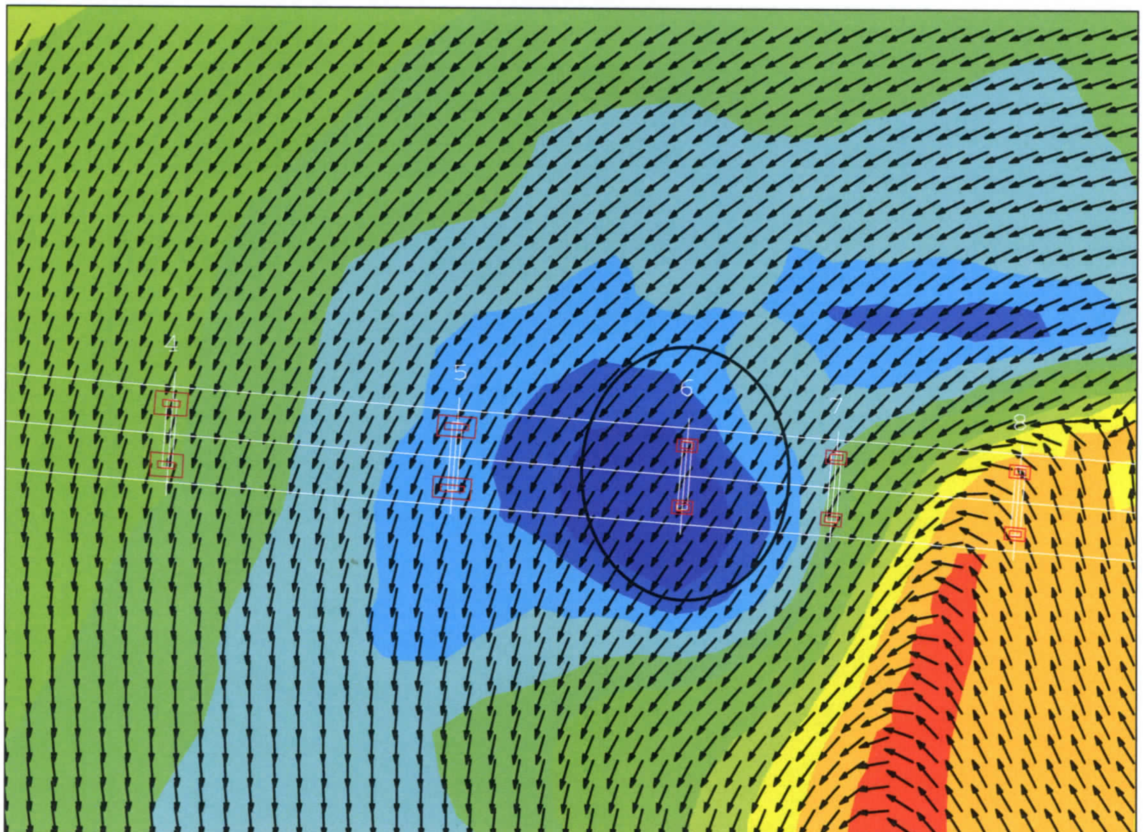

Figure 23. Thirty degree angle of attack at pier 6.

The pier scour calculations were based on the flow depths corresponding to the scoured bottom after the contraction scour is taken into account (that is with the flow depth equal to the original flow depth plus the contraction scour). As for the flow velocity, the velocity calculated for the original ground was used in the pier scour calculation although it is expected that this velocity would be reduced somewhat due to an increase in depth as a result of the scour. The information used in the calculations can be seen in Table 4. These calculated pier scour depths were then added to the contraction scour to obtain a range of the total scour. Table 5 shows that pier scour ranges from 12 feet to 13 feet and contraction scour ranges from 8 feet to 40 feet. The total calculated scour ranges from 20 to 53 feet. The actual scour was 35 to 40 feet. If we had used a lower velocity it would have given a slightly lower pier scour depth. Although there is some uncertainty the calculations do indicate that such scour depths are possible as seen in this study. The total scour was compared to the actual scour depths. The actual scour depths were determined by subtracting the existing hydrographic survey elevations from the modified survey elevations based on the partial contours of the original ground at the bridge. The actual scour depths may be seen in Figure 24. The scour hole is not seen as deep around pier 6 due to the riprap countermeasures in place.

Table 4. Complex pier calculations.

Stem Scour	**Fine Sand**	**Very Fine Sand and Silt**
Initial Scour (Contraction)	**8**	**40**
V_1 (fps)	4.5	4.5
y_1 (ft)	27	59
y_{scontr} (ft)	0	0
V_{contr} (ft)	4.5	4.5
y_{contr} (ft)	27	59
a_{pier} (ft)	7.5	7.5
K_1	1	1
K_2	1.24	1.24
K_3	1.1	1.1
K_4	1	1
K_h	0.23	0.02
Ysstem (ft)	3.24	0.32

Table 4. Complex pier calculations (Continued).

Footing Scour	**Fine Sand**	**Very Fine Sand and Silt**
f (ft)	2.54	2.54
ho (ft)	-1.5	30.5
T (ft)	5	5
K_w	1.00	1.00
a_f (ft)	13.00	13.00
K_1	1	1
K_2	1.3	1.3
K_3	1.1	1.1
K_4	1	1
h1 (ft)	3.5	35.5
y_2 (ft)	28.62	59.16
y_2 (plot) (ft)	28.62	45.50
V_2 (fps)	4.24	4.49
h_2 (ft)	0.12	30.66
a*/a	0.157	0.023
$a^*_{footing}$ (ft)	2.04	0.29
Ysfooting (ft)	6.31	2.02

Pile Group Scour	**Fine Sand**	**Very Fine Sand and Silt**
h_3 (ft)	3.28	31.67
y_3 (ft)	31.78	60.17
y_3 (plot) (ft)	18.47	18.47
V_3 (ft)	3.82	4.41
a_{proj} (ft)	10.81	10.81
k_{sp}	0.49	0.49
s (ft)	2.5	2.5
a_{piles} (ft)	1	1
km (skew)	1	1
Km	1.16	1.16
M	3	3
a^* (ft)	5.29	5.28
K_h	0.25	1
K_1	1.1	1.1
K_2	1	1
K_3	1.1	1.1
K_4	1	1
Ysgrouping (ft)	2.43	11.13
YSTOTAL (ft)	**12**	**13**

Table 5 Total scour calculated and actual scour surveyed.

Scour Calculation Results	Fine Sand	Very Fine Sand and Silt
Contraction Scour (ft)	8	40
Pier Scour (ft)	12	13
Total Scour (ft)	20	53
Actual Scour Depths	**From Surveys**	
Total Scour (ft)	35-40	

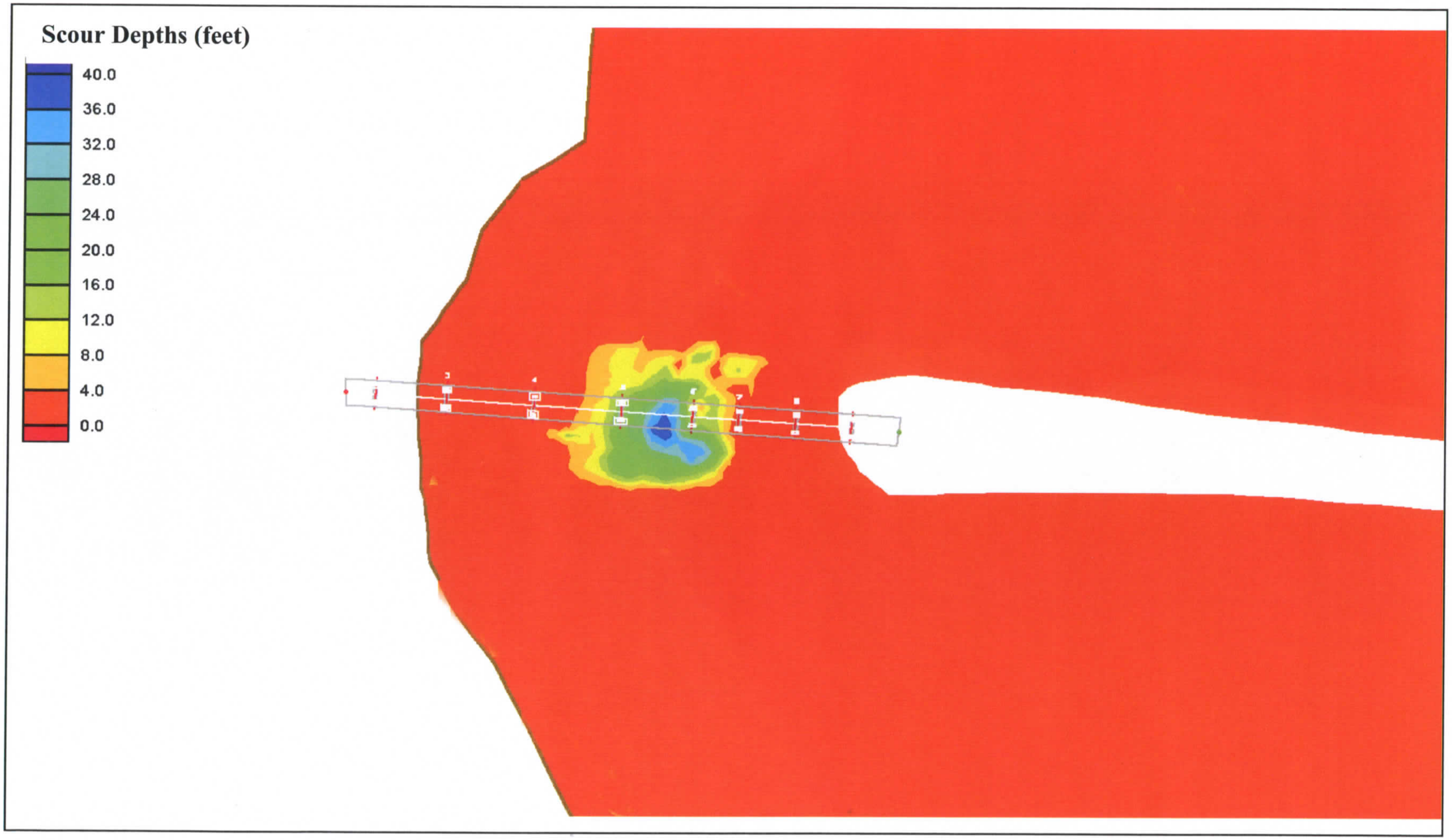

Figure 24. Maximum scour depth relative to the original ground calculated using the bottom topography based on the original 1961 survey elevations and the 2001 survey elevations.

III. CONCLUSIONS

The contraction of the floodplain from the roadway encroachment increased the velocities and shear stresses at Pier 6. This coupled with the thirty degree flow angle aggravated the erosion conditions at the pier. A large flood occurred in April of 1964, which probably caused most of the scour. Scour depths were most likely worsened by intermediate floods leading up to 1971 when the scour was first documented. Riprap was placed around the pier for protection after it was discovered. The riprap increased the bed critical shear stress and probably prevented more scour local to the pier. The largest event of record was the 1990 event, which the bridge survived.

The model predicted scour in the vicinity of the scoured area at the bridge. The extent of the scour depths is variable depending on the type of soil. Changes in elevations can mean changes in soil types as well. Although the critical shear stresses of the scoured soils were not available, we were able to make assumptions based on the soils descriptions from existing boring information. The maximum actual scour depths were within the range of the total calculated scour depths. The reason for the scour not showing up as deep at the pier is due to the amount of riprap protection added. This added riprap also may have had a part in increased velocity, shear stresses, and scour depths between Pier 5 and Pier 6. If the present calculation methods had existed when the bridge was built it appears that some of the problems could have been predicted.

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