Scour Evaluations of Two Bridge Sites in Alabama with Cohesive Soils

ALDOT Research Project 930-490R

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ABSTRACT

Curry et al. (2002) conducted a scour evaluation study on bridges in Alabama that had experienced significant flood events (100 year event or greater). It was reported that current methods developed for noncohesive soils described in HEC-18 (1995 Third Edition) were inadequate for computing scour in cohesive soils. This report provides further evaluations of scour at two sites in Alabama with cohesive soils using methods developed recently by Briaud et al. (1999, 2001a, 2001b, 2003) and Güven et al. (2002a, 2002b). Comparisons of calculated scour obtained with the new methods for cohesive soils indicate better agreement with field observations.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

PAGE

LIST OF TABLES

PAGE

LIST OF FIGURES

PAGE

I. INTRODUCTION

Curry et al. (2002) conducted a scour evaluation study on bridges that had experienced significant flood events (100 year event or greater). The conclusion from the report was that current methods for calculating scour were inadequate for computing scour in cohesive soils. Due to the need for a better method of computing scour in cohesive soils new methods have been suggested. Briaud et al. (1999, 2001a, 2001b, 2003) presented two methods for computing contraction and pier scour in cohesive soils called Simple SRICOS (ScouR In COhisive Soils) and Extended SRICOS. Güven et al. (2001, 2002a, 2002b) presented a one-dimensional approach for modeling time-dependent clear-water contraction scour in cohesive soils, which may be called the "DASICOS" method (Differential Analysis of Scour In COhesive Soils) for present purposes. The methods rely on a new erosion function apparatus (EFA), described by Briaud et al. (1999), which allows the measurement of the critical shear stress of a sample of bed soil and the erosion rate of the soil sample as a function of the bed shear stress imposed by the flowing stream. Two sites in Alabama were selected to do a detailed scour analysis using the new methods for cohesive soils. The two sites were Pea River at Elba and Choctawhatchee River near Newton. Data were gathered for each site including bridge plans, location maps, aerial photos, soundings, hydrologic data, rating curves, soil samples, and core borings. The bridge information was used to create one-dimensional models of each site using the Army Corp of Engineer's Hydrologic Engineering Center River Analysis System (HEC-RAS) software (Brunner, 2001a, b). The models were used for determining flow distributions, discharges per unit width and hydraulic depths in the overbanks and main channels for each site. The hydrologic data provided a history of flood events that the sites had experienced. The core borings and soil samples were used to identify the foundations and determine critical shear stresses and erosion functions for each site. Using this information scour calculations were made with the new methods. Computer programs used to perform these calculations are presented in Appendix A. Plots were generated and comparisons were made between the new methods, the existing HEC-18 methods, and the actual field conditions.

II. BACKGROUND FOR THE TWO SITES

The data gathered for each site included bridge plans, location maps, aerial photos, soundings, hydrologic data, rating curves, soil samples, and core borings. The data for each site are summarized below.

The first bridge site is located on U.S. 84 on the east side of Elba, Alabama over the Pea River (Figure 1 and 2). The bridge was constructed in 1930 and widened in 1959. The present bridge is approximately 688 feet long with 18 spans and 17 piers. The drainage area for the site is 959 square miles. The main channel (Figure 3) is approximately 200 feet wide with banks 30 to 35 feet high. The floodplain looking downstream is 300 feet wide on the left overbank (Figure 4) and 50 feet wide on the right overbank. Core borings at the site describe the soils to have sandy clay on top with a hard silt with clay (marl) below (Figure 5). A history of groundlines in the form of soundings for the left and right side of the bridge were obtained that dated back to the original groundline of 1930 (Figure 6 and 7). These provide a picture of what scour occurred. Pictures taken after the 1998 flood indicated some contraction and local scour shown in Figure 8.

Peak discharges were obtained from the USGS website (www.usgs.gov) since the construction of the bridge (Figure 9 and Table 1). Peak discharges and daily discharges did not exist from 1956 to 1971, and data before 1974 and during the 1980s was unavailable. Only stage information was available for the 1990 event (largest event of record) from the 1990 USGS Water Resources Data report. The stages were used to find the discharges for the events using the current USGS rating curve developed for the site.

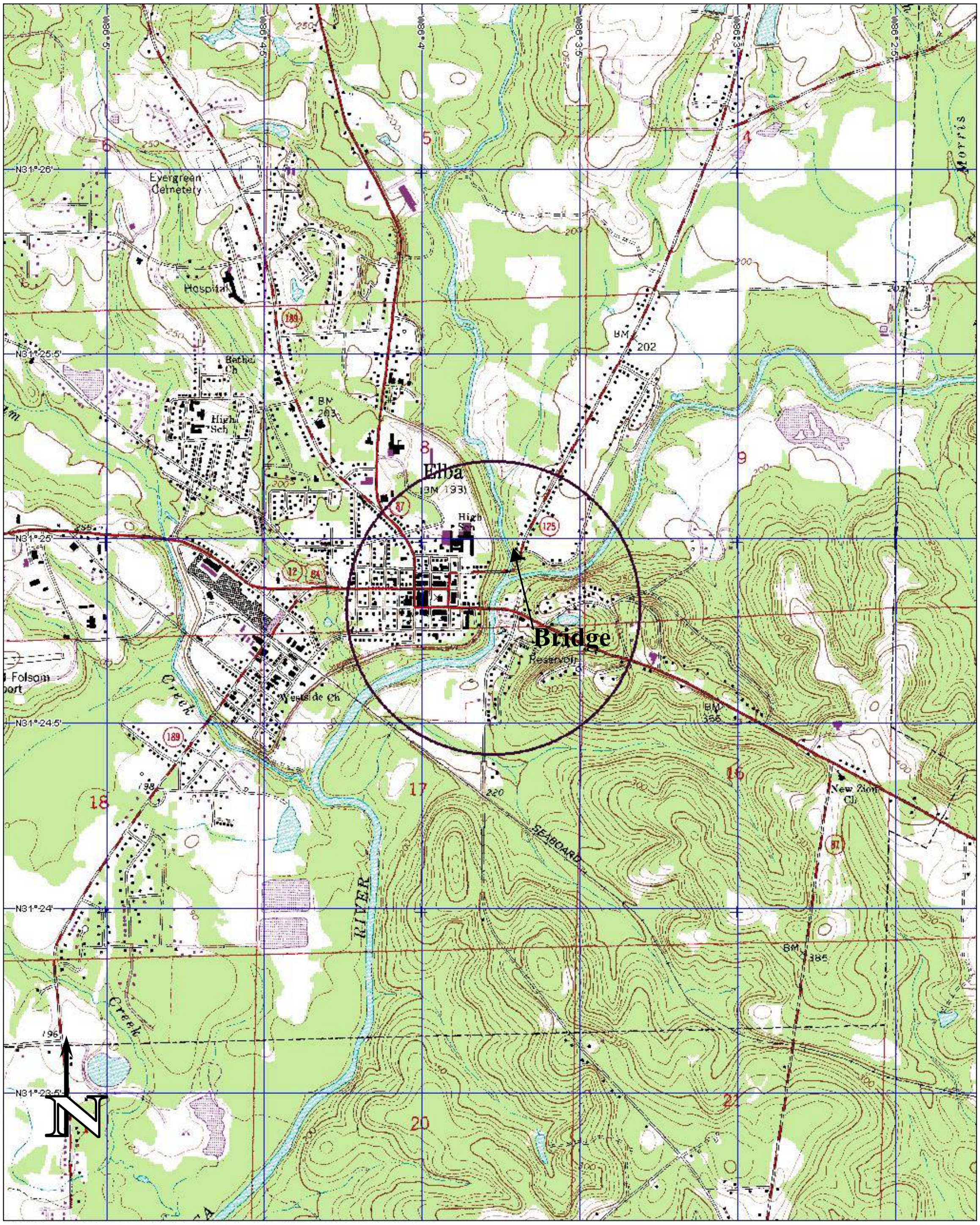


Figure 1. USGS quadrangle map of the Pea River site at Elba, Alabama 07/01/1973

Figure 2. Aerial photo of Pea River at Elba

Figure 3. View of main channel looking west on the upstream side at Pea River at Elba

Figure 4. View of left overbank looking east on the downstream side at Pea River at Elba

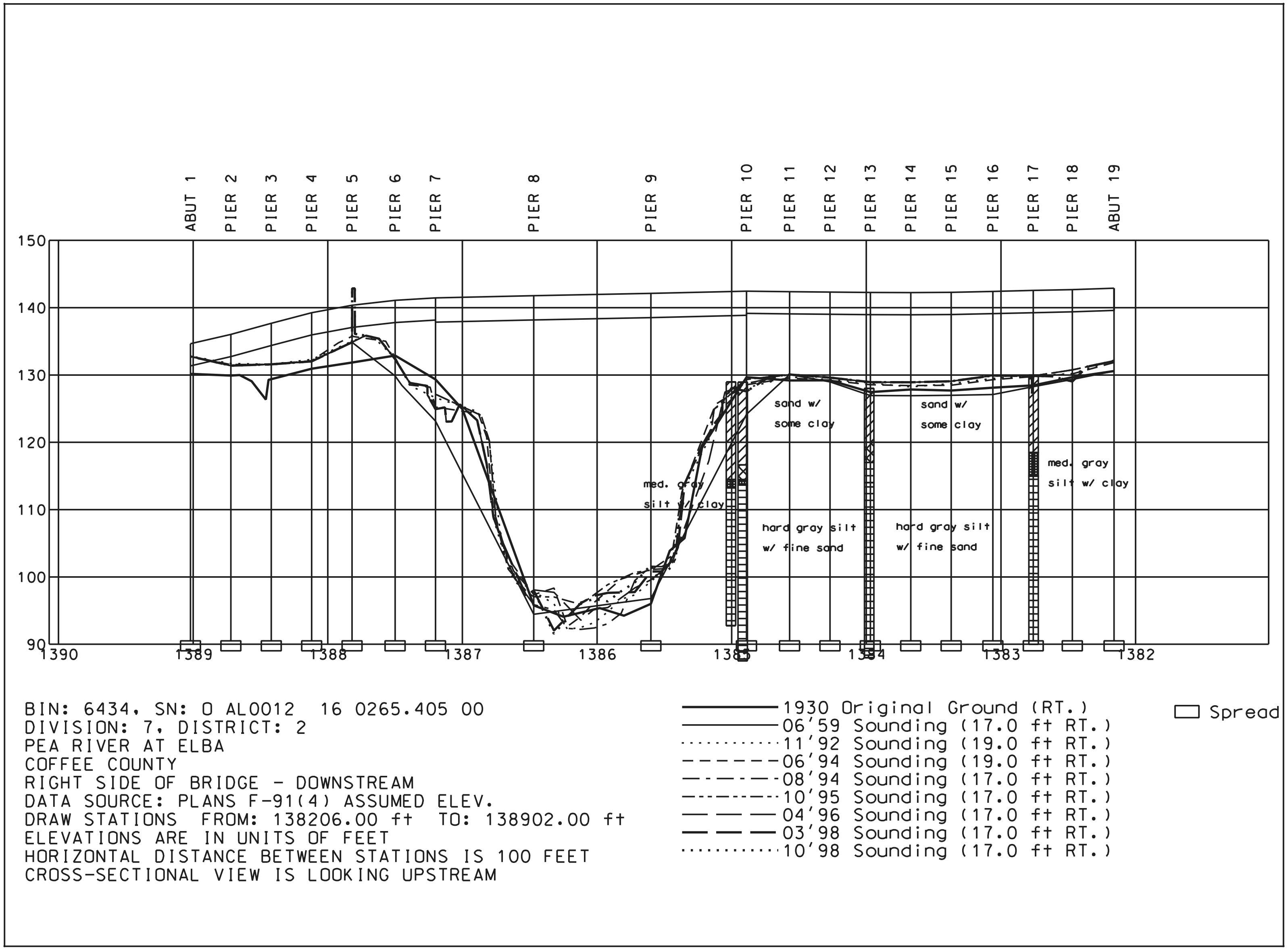


Figure 5. Core borings for the Pea River site at Elba

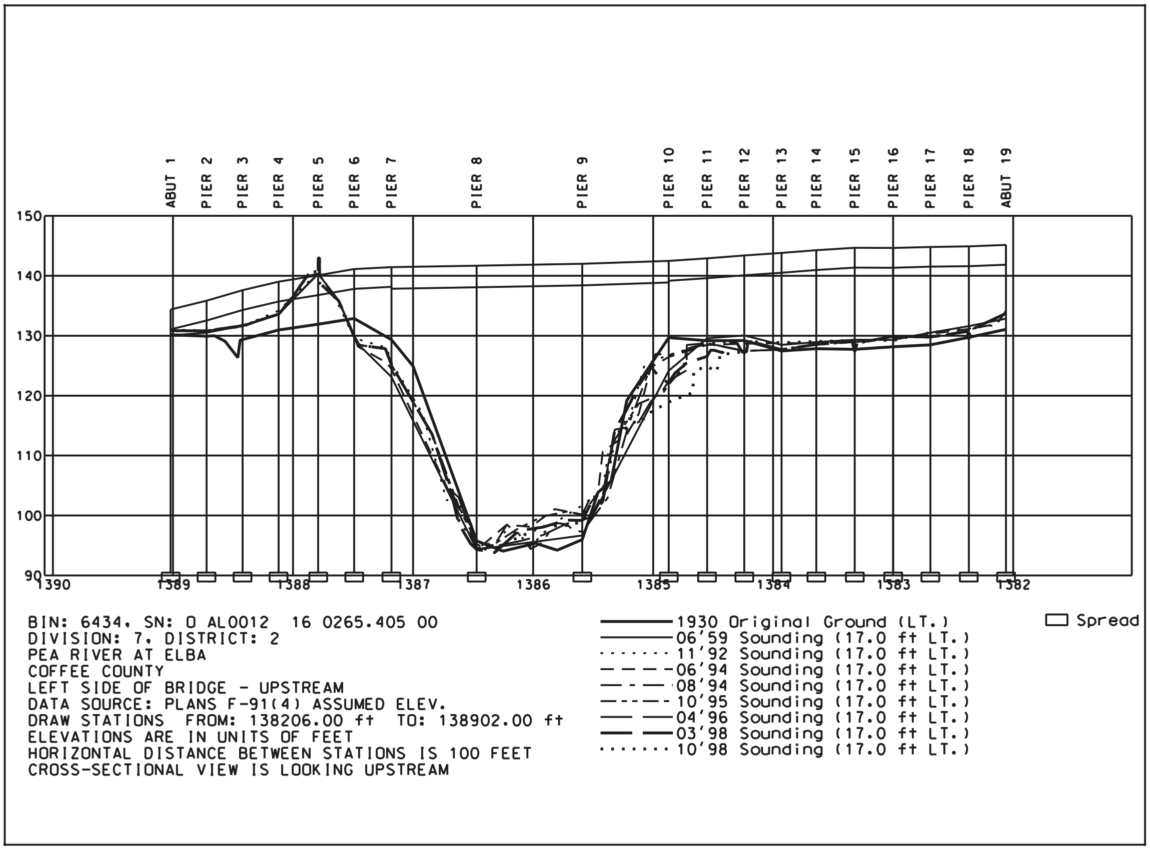


Figure 6. Upstream side of soundings of Pea River at Elba

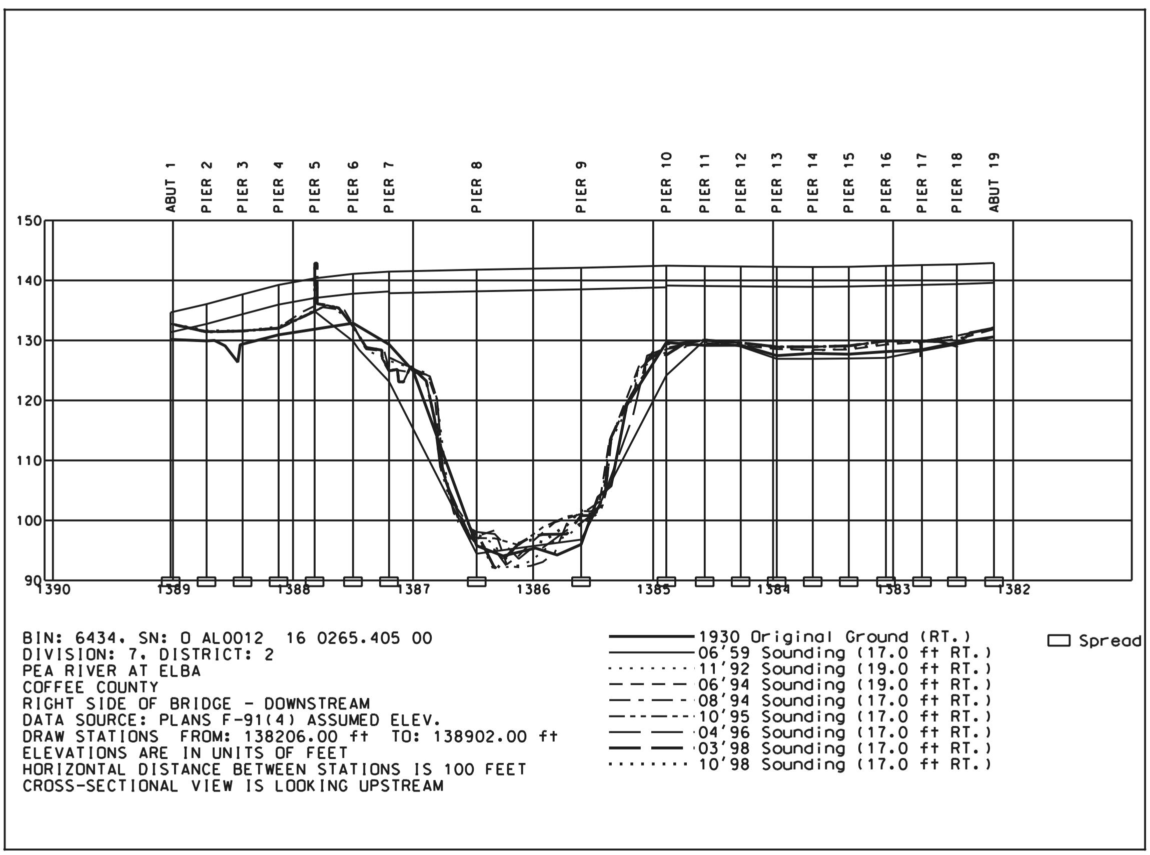


Figure 7. Downstream side of soundings of Pea River at Elba

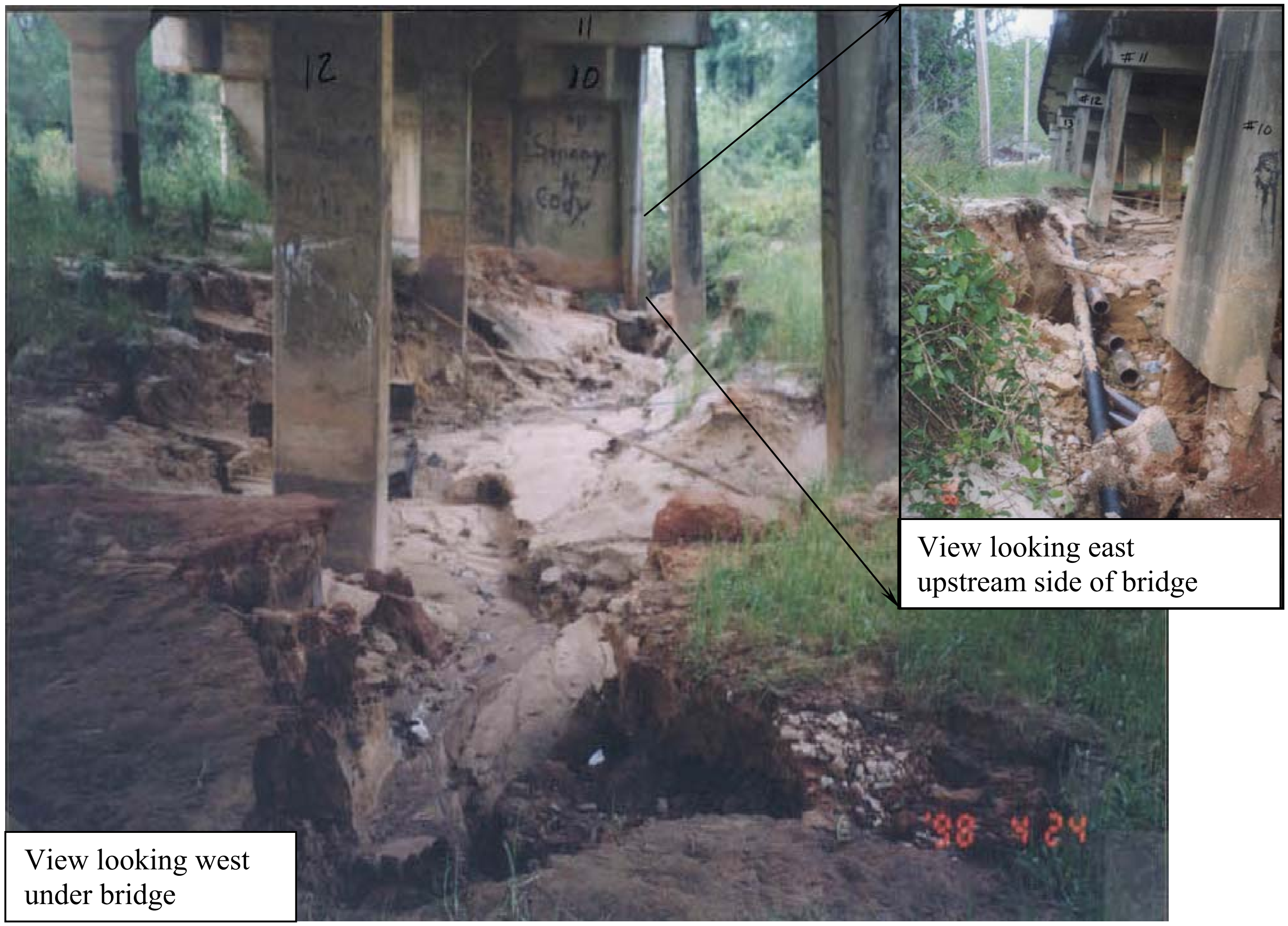


Figure 8. Local scour under the bridge after the 1998 flood

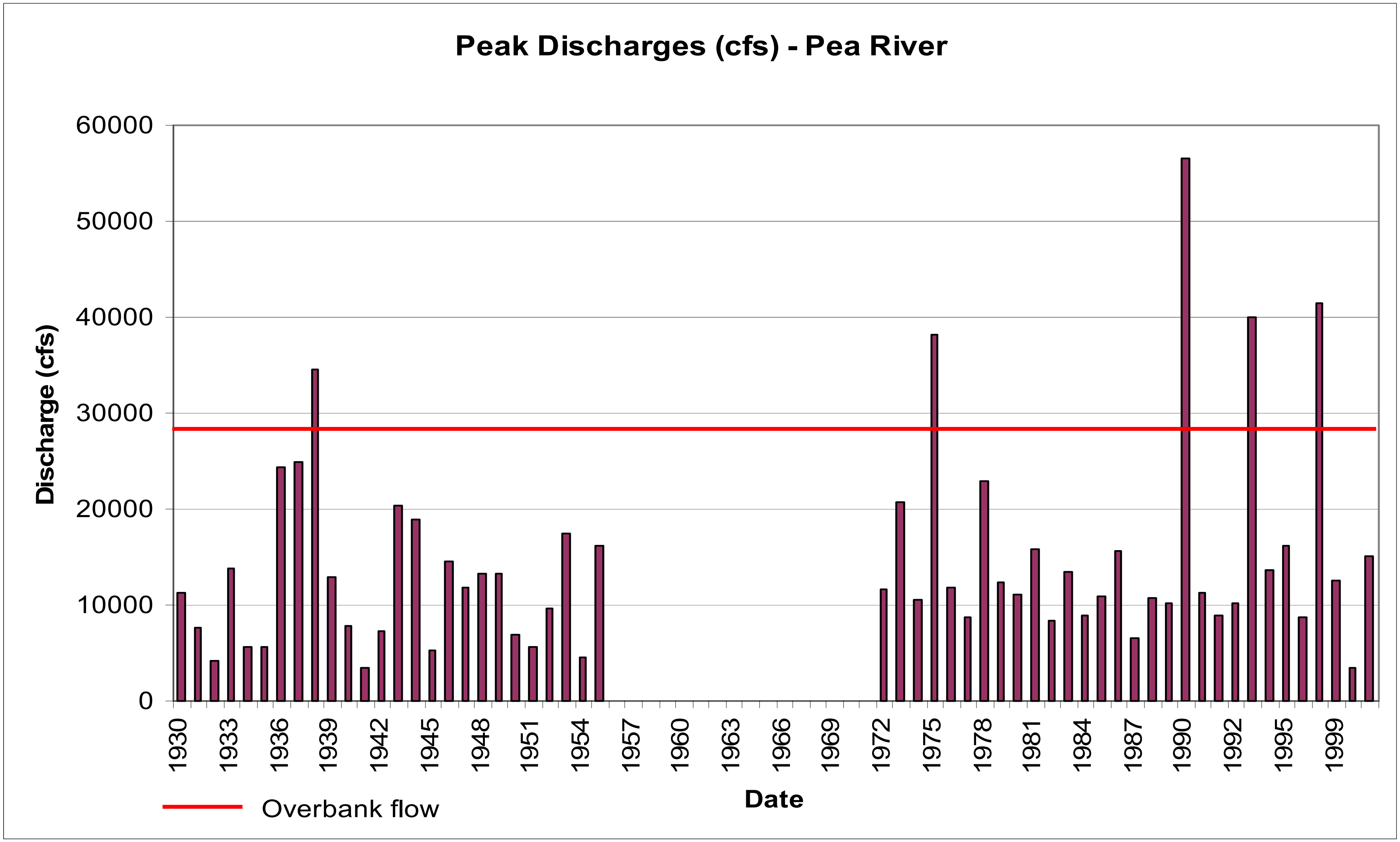


Figure 9. Peak discharges for Pea River at Elba

Table 1 Peak discharges for Pea River at Elba

Water Year	Date	Gage Height (feet)	Stream-flow (cfs)
1929	Mar. 1929	43.50	90,000[2,7]
1930	1930		11,200[2]
1931	1931		7,600[2]
1932	1932		4,200[2]
1933	1933		13,800[2]
1934	1934		5,700[2]
1935	1935		5,600[2]
1936	Jan. 21, 1936	29.55	24,300
1937	Apr. 06, 1937	30.00	25,000
1938	Mar. 17, 1938	35.00	34,500
1939	Feb. 28, 1939	20.80	12,900
1940	Feb. 18, 1940	16.00	7,900
1941	Mar. 07, 1941	10.23	3,520
1942	Apr. 10, 1942	15.40	7,350
1943	Jan. 19, 1943	26.80	20,400
1944	Mar. 24, 1944	25.80	19,000
1945	Apr. 29, 1945	12.80	5,270
1946	May 21, 1946	22.30	14,600
1947	Apr. 03, 1947	19.80	11,800
1948	Mar. 07, 1948	21.20	13,300
1949	Nov. 30, 1948	21.10	13,200
1950	Sep. 01, 1950	15.00	7,000
1951	Mar. 29, 1951	13.40	5,720
1952	Mar. 27, 1952	17.80	9,650
1953	Dec. 04, 1952	24.60	17,500
1954	Jan. 01, 1954	11.79	4,550
1955	Apr. 14, 1955	23.60	16,200
1972	Mar. 03, 1972	19.70	11,700
1973	Mar. 12, 1973	27.00	20,700
1974	Jan. 01, 1974	18.60	10,500
1975	Feb. 19, 1975	37.26	38,200
1976	May 15, 1976	19.90	11,900
1977	Nov. 29, 1976	16.90	8,750
1978	Jan. 26, 1978	28.60	22,900
1979	Mar. 04, 1979	20.40	12,400
1980	Mar. 13, 1980	19.15	11,100
1981	Feb. 12, 1981	23.40	15,900
1982	Feb. 04, 1982	16.55	8,420
1983	May 20, 1983	21.35	13,500
1984	Mar. 25, 1984	17.05	8,900
1985	Feb. 06, 1985	19.00	10,900
1986	Mar. 15, 1986	23.15	15,600
1987	Feb. 28, 1987	14.55	6,630
1988	Mar. 04, 1988	18.85	10,700
1989	Jun. 17, 1989	18.30	10,200
1990	Mar. 17, 1990	43.28	56,600
1991	Jan. 31, 1991	19.30	11,200
1992	Jan. 14, 1992	17.10	8,950
1993	Nov. 26, 1992	18.33	10,200
1994	Jul. 07, 1994	38.33	40,000
1995	Feb. 12, 1995	21.40	13,600
1996	Oct. 05, 1995	23.62	16,200
1997	Feb. 15, 1997	16.91	8,760
1998	Mar. 06, 1998	39.23	41,500
1999	Mar. 14, 1999	20.51	12,600
2000	Mar. 20, 2000	10.07	3,430
2001	Mar. 04, 2001	22.70	15,100

Soil samples from core borings (Figure 10) were obtained from specific locations at the bridge site (Figure 11). The samples were tested in an erosion function apparatus (EFA) to determine the critical shear stress and the erosion function for each layer. More details of the results of the tests will be discussed later and details of how the tests are performed can be found in "Erosion Functions of Cohesive Soils," by Crim (2003).

Figure 10. Drilling crew collecting samples at Pea River site at Elba

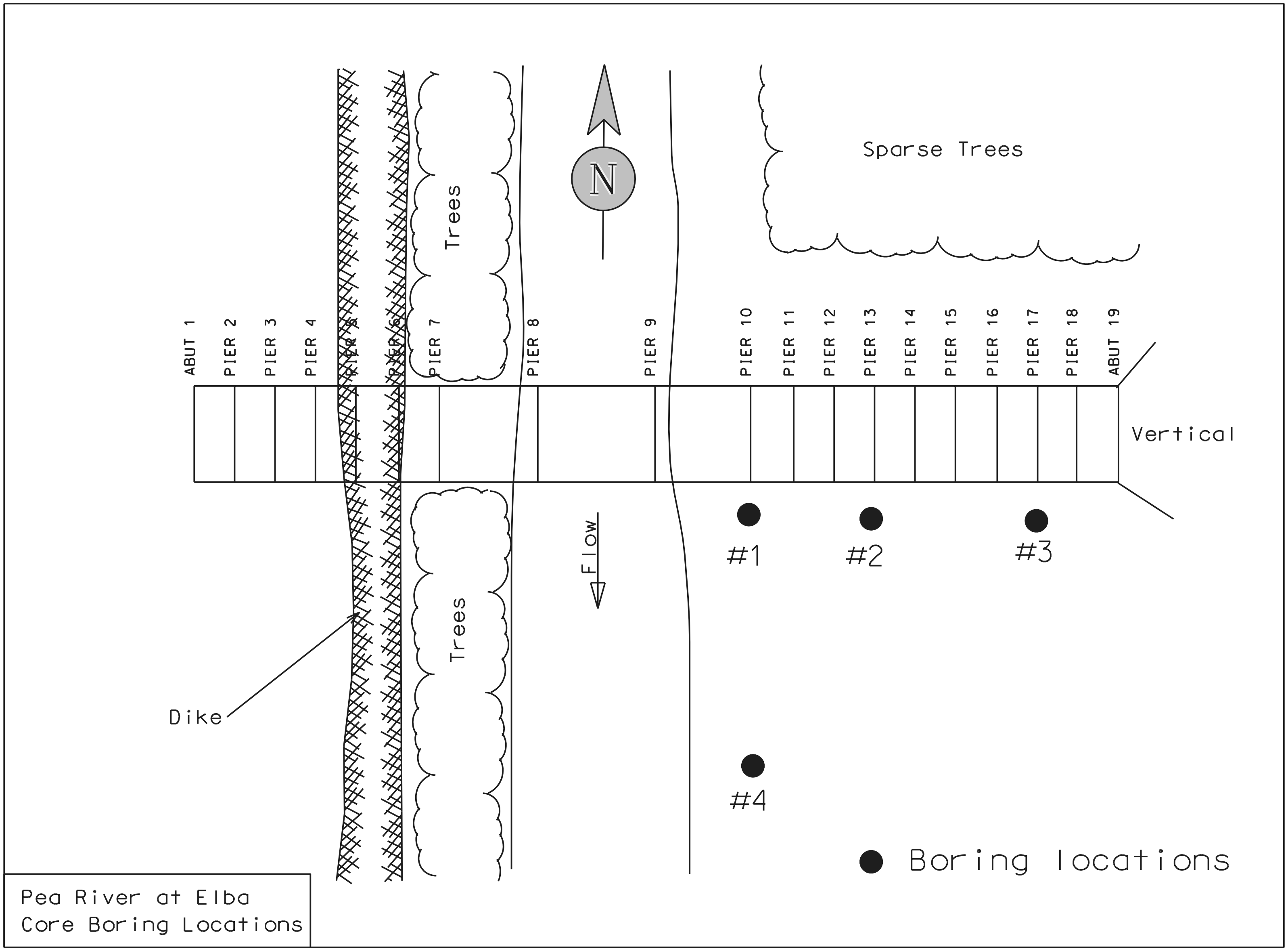


Figure 11. Boring locations for the Pea River site at Elba

The second bridge site considered in this study is located on State Route 123 near Newton, Alabama over the Choctawhatchee River (Figure 12 and 13). The bridge was constructed in 1976. The bridge is 584 feet long with 10 spans and 9 piers. The drainage area for the site is 686 square miles. The main channel (Figure 14) is approximately 150 feet wide with banks 20 to 25 feet high. The floodplain looking downstream is roughly 150 feet wide on the left overbank (Figure 15) and 225 feet wide on the right overbank. Core borings at the site describe the soil on the right overbank to have sand on top with a clay beneath and the left overbank to have a sandy clay on top with a clay beneath (Figure 16). The main channel is a hard clay. A history of the groundlines in the form of soundings for the left and right side of the bridge were obtained that dated back to 1976 (Figure 17 and 18).

Peak discharges were obtained since the construction of the bridge (Figure 19 and Table 2). A history of daily discharges was also obtained from the USGS website dating back to the construction date of the bridge.

Soil samples from core borings were obtained from specific locations at the bridge site (Figure 20). The samples were tested in an erosion function apparatus (EFA).

After the collection of the preliminary data, one-dimensional hydraulic models were constructed for both sites using HEC-RAS. Each model was analyzed with specific discharges and stages taken incrementally from the rating curves. These runs provided discharges per unit width, q, (q = Flowrate/TopWidth) and hydraulic depths, Y_h, (Y_h = Area/TopWidth) for the main channel and for the overbanks for varying discharges and stages. This data was used for interpolating q's for the measured stages and discharges in the main channel and in the overbanks.

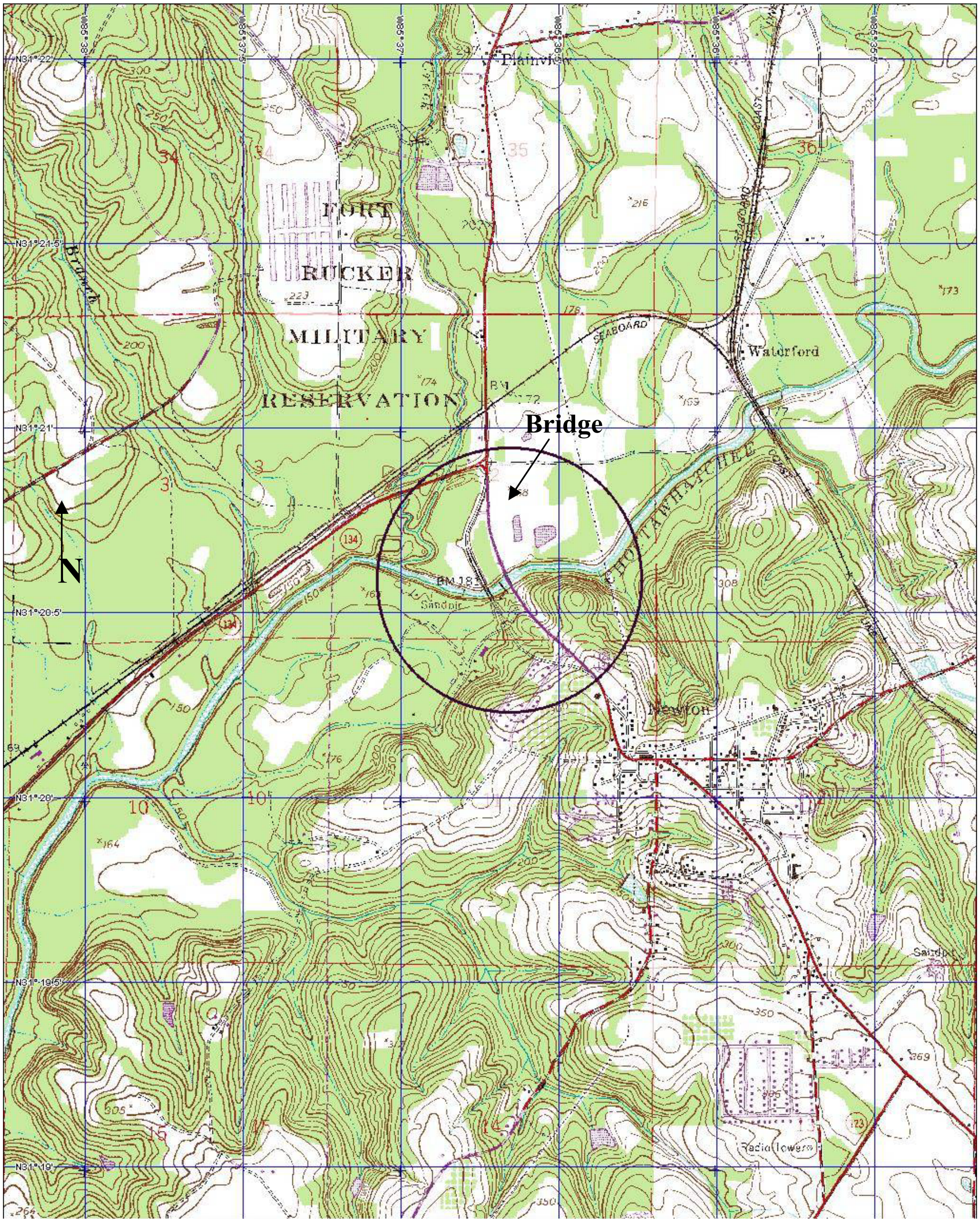


Figure 12. USGS quadrangle map of the Choctawhatchee River site near Newton, Alabama 07/01/1973

Figure 13. Aerial photo of Choctawhatchee River near Newton

Figure 14. View of main channel looking south on the downstream side at Choctawhatchee site near Newton

Figure 15. View of right overbank looking north on the upstream side at Choctawhatchee River site near Newton

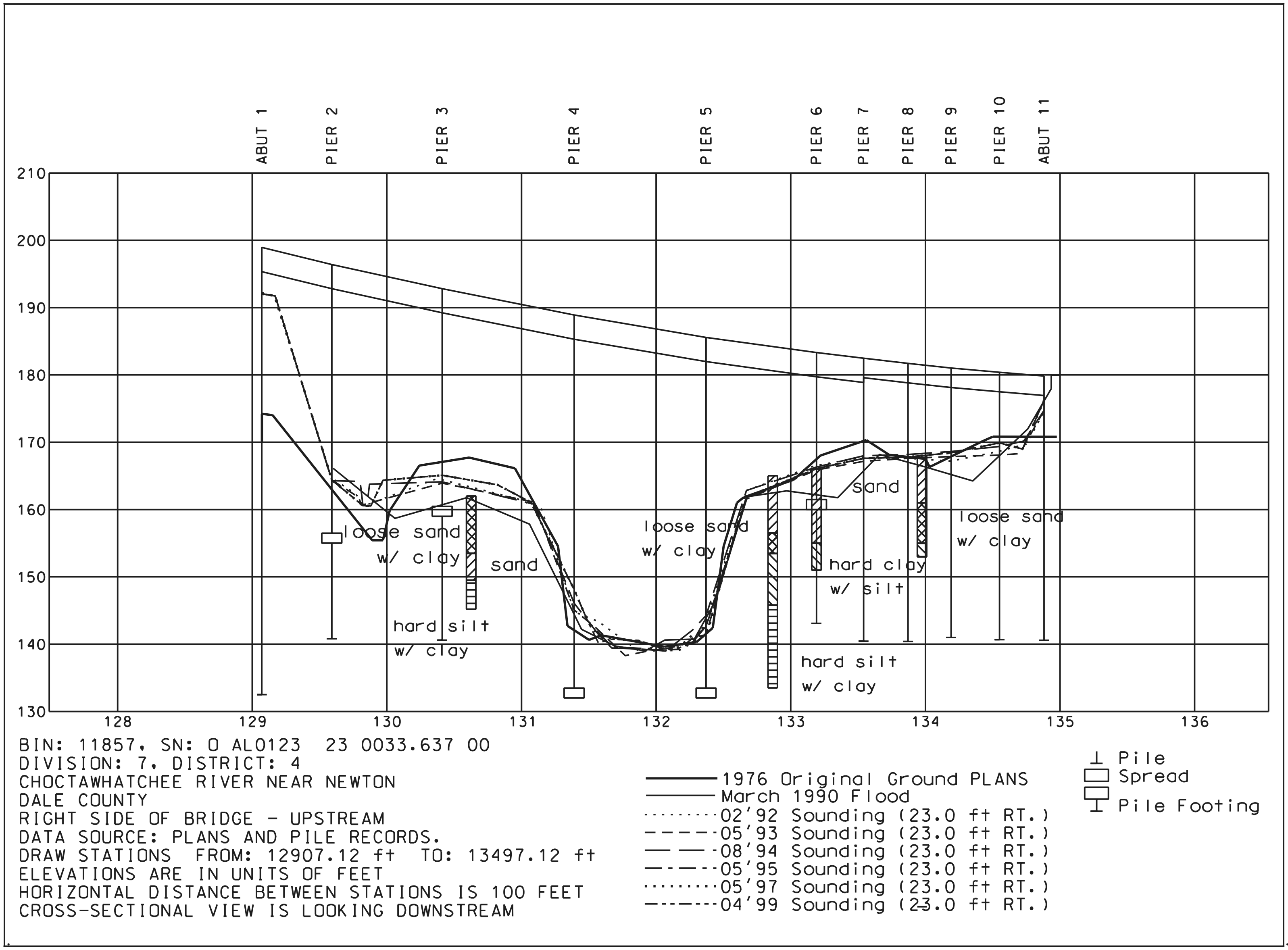


Figure 16. Core borings for the Choctawhatchee River site near Newton

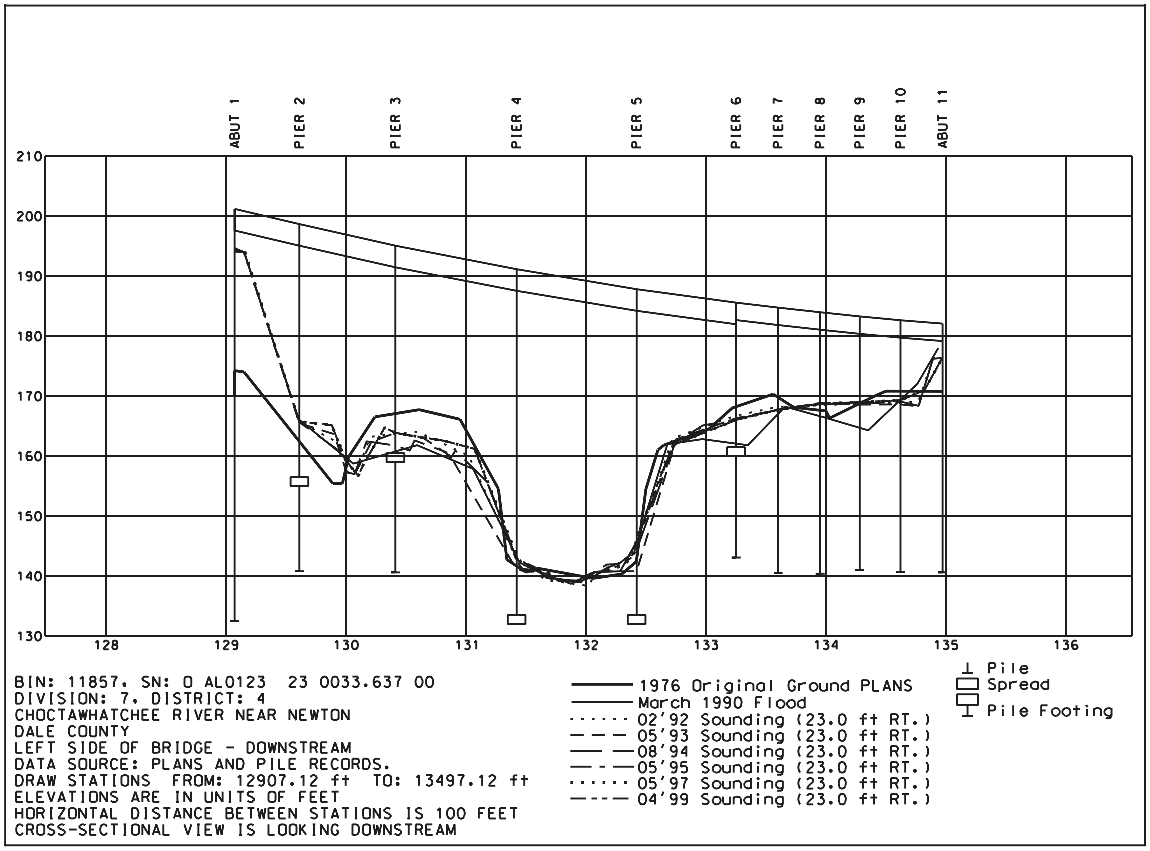


Figure 17. Downstream side of soundings of Choctawhatchee River near Newton

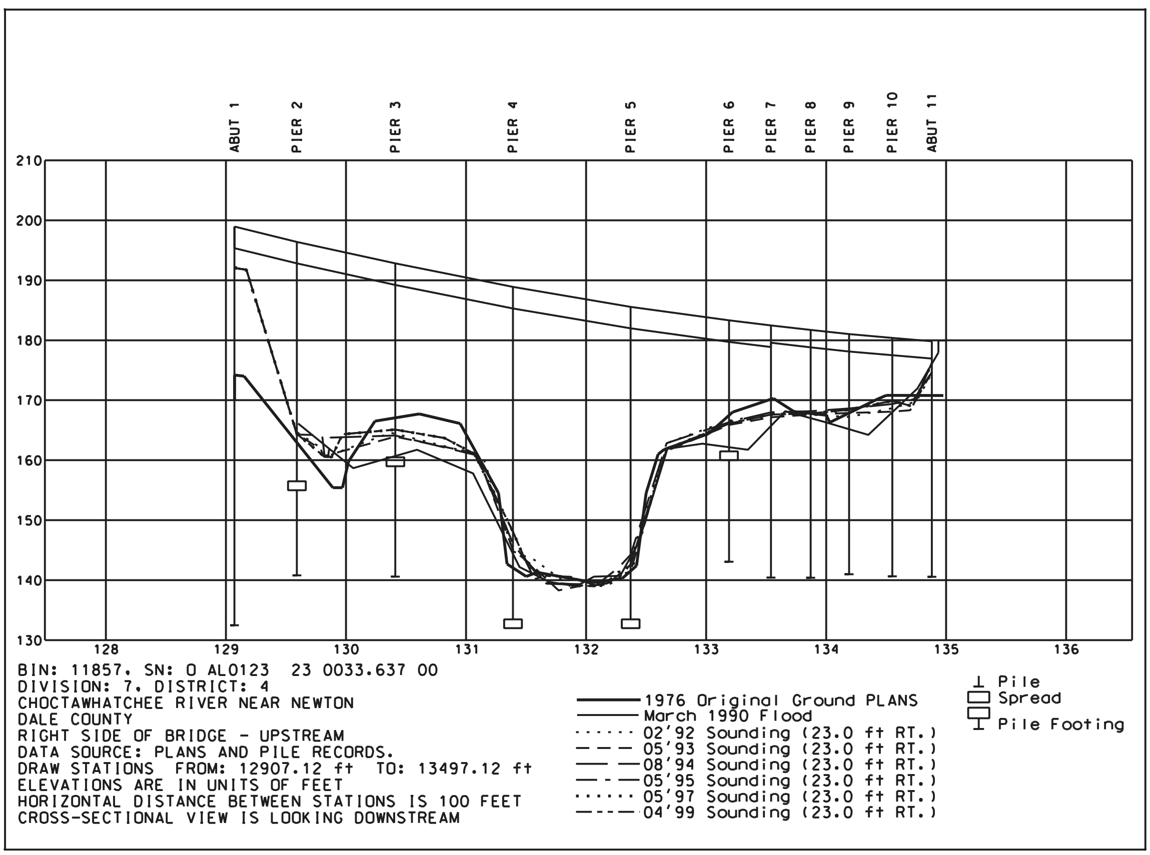


Figure 18. Upstream side of soundings of Choctawhatchee River near Newton

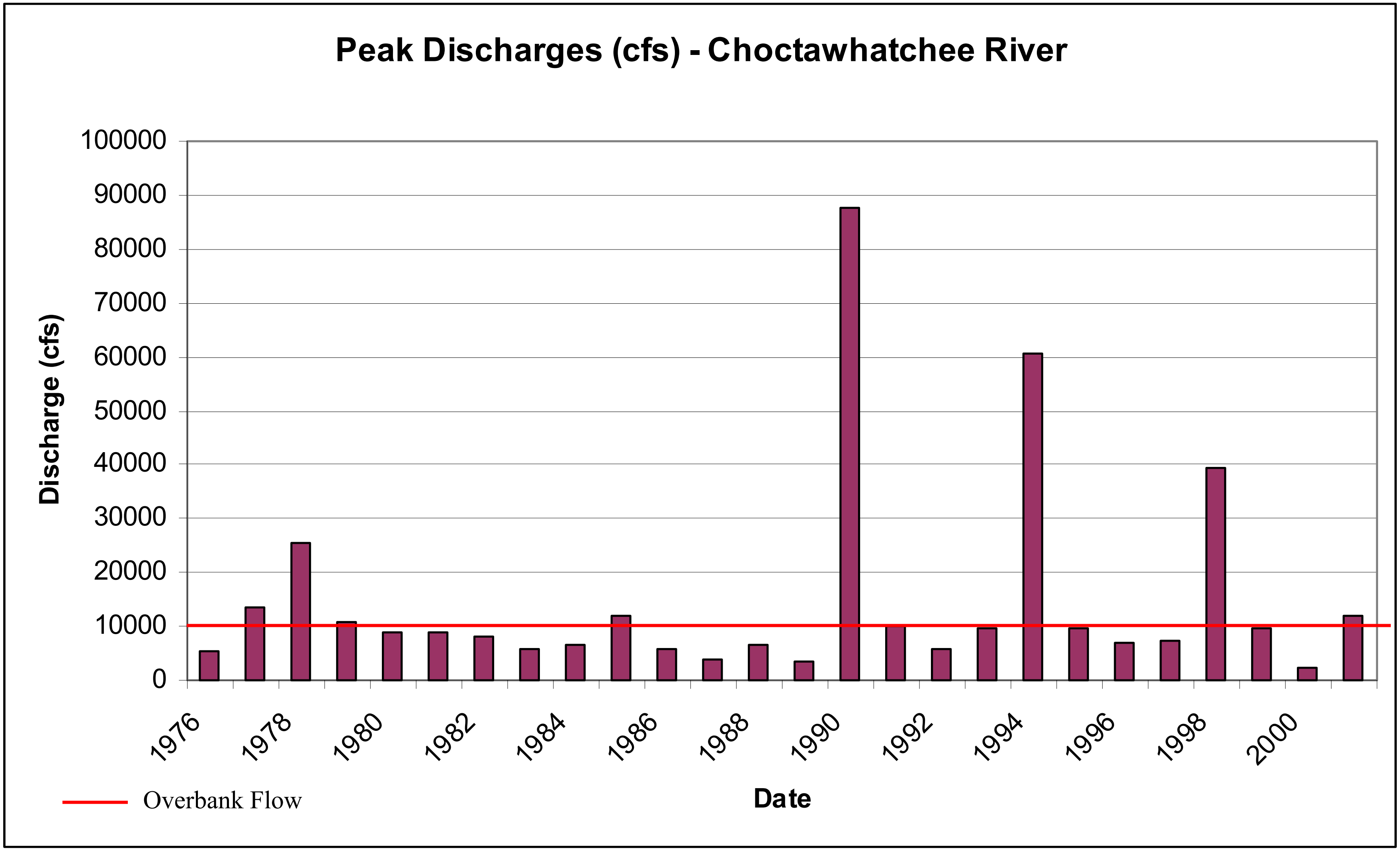


Figure 19. Peak discharges for Choctawhatchee River near Newton

Table 2 Peak discharges Choctawhatchee River near Newton

Water Year	Date	Gage Height (feet)	Stream-flow (cfs)
1976	Jan. 28, 1976	14.26	5,490
1977	Nov. 30, 1976	25.72	13,700
1978	Jan. 27, 1978	31.26	25,300
1979	Feb. 25, 1979	23.17	11,000
1980	Mar. 13, 1980	20.74	8,940
1981	Feb. 11, 1981	20.90	9,070
1982	Feb. 04, 1982	19.41	7,940
1983	Mar. 27, 1983	15.97	5,800
1984	May 04, 1984	17.23	6,490
1985	Feb. 07, 1985	24.21	12,000
1986	Mar. 15, 1986	15.46	5,730
1987	Mar. 30, 1987	11.58	3,690
1988	Mar. 05, 1988	17.03	6,620
1989	Jun. 16, 1989	11.00	3,410
1990	Mar. 18, 1990	40.30	87,500
1991	Feb. 01, 1991	22.10	10,100
1992	Mar. 07, 1992	15.58	5,790
1993	Nov. 27, 1992	21.46	9,520
1994	Jul. 07, 1994	37.78	60,800
1995	Feb. 12, 1995	21.78	9,790
1996	Mar. 19, 1996	17.86	7,110
1997	Jan. 09, 1997	18.38	7,430
1998	Mar. 09, 1998	34.58	39,200
1999	Oct. 01, 1998	21.51	9,560
2000	Feb. 14, 2000	8.70	2,350
2001	Mar. 05, 2001	24.02	11,800

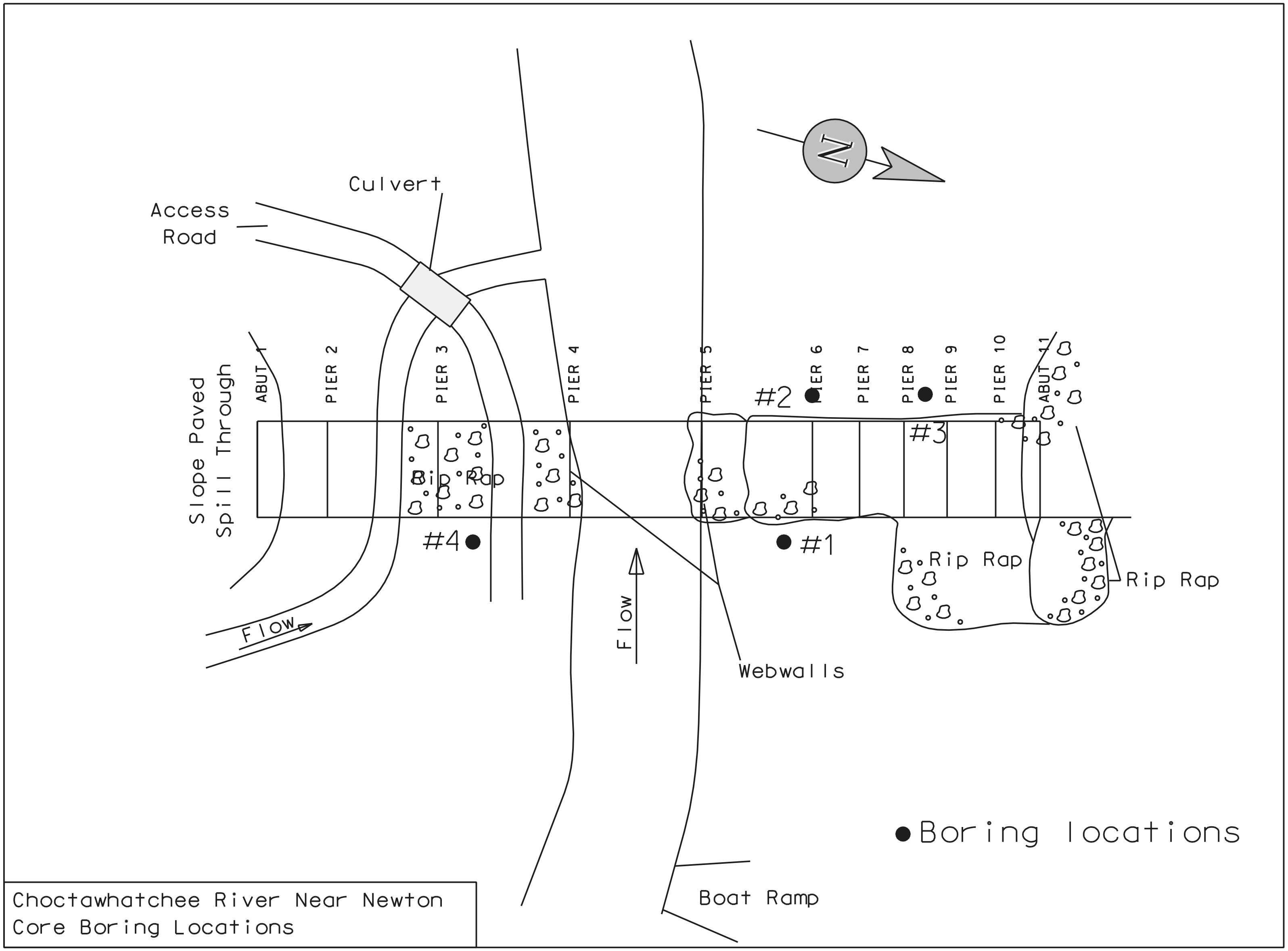


Figure 20. Boring locations for the Choctawhatchee River site near Newton

III. PRESENT METHODOLOGY AND SCOUR CALCULATIONS

Pier, contraction, and abutment scour was previously calculated based on methods from Hydraulic Engineering Circular 18 (HEC-18) for the two sites in a report by Curry et al. (2002). The method assumes scour occurs very quickly and that maximum scour is reached for each given discharge and stage. The calculated scour was plotted and can be seen in Figures 21 and 22.

Each bridge site was modeled using HEC-RAS. The models are defined in HEC-RAS as a series of cross-sections and associated parameters. Stream cross-sections were obtained from the plan/profile sheet of the construction plans and from a sounding taken by the USGS. Each cross-section was propagated approximately a bridge length upstream and downstream of the site using the average slope of the channel estimated from a USGS quadrangle map.

Several parameters were required to define the HEC-RAS model such as Manning's n-values and boundary conditions. Manning's n-values were estimated for the channel and overbank areas based on engineering judgment of the site. The n-values were adjusted in some cases to calibrate the models to known depth averaged velocity measurements. Discharges and starting downstream water surface elevations were two boundary conditions needed in modeling the sites. The discharges and starting downstream water surface elevations were taken from USGS records.

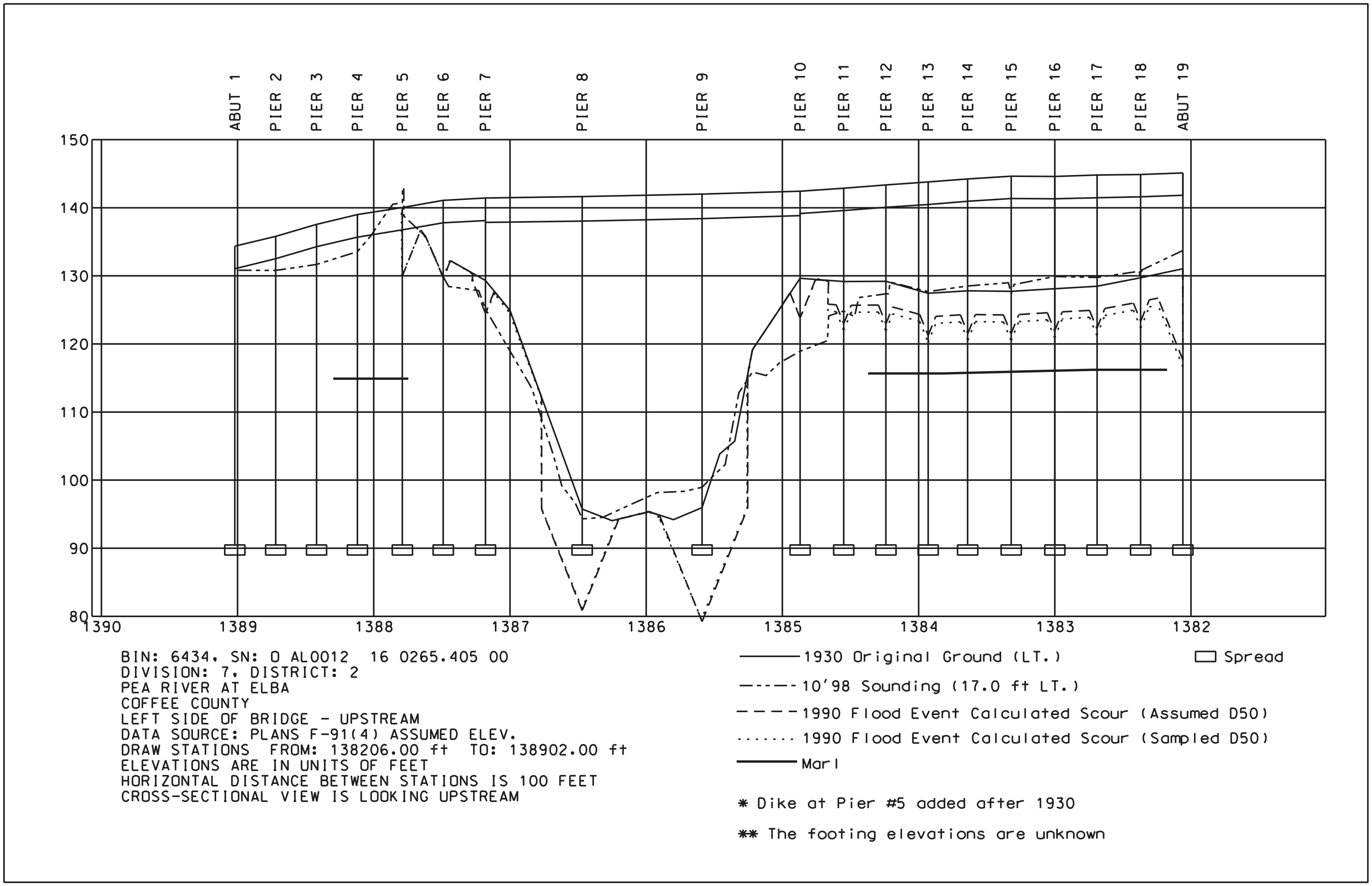


Figure 21. HEC-18 Scour calculations for Pea River at Elba

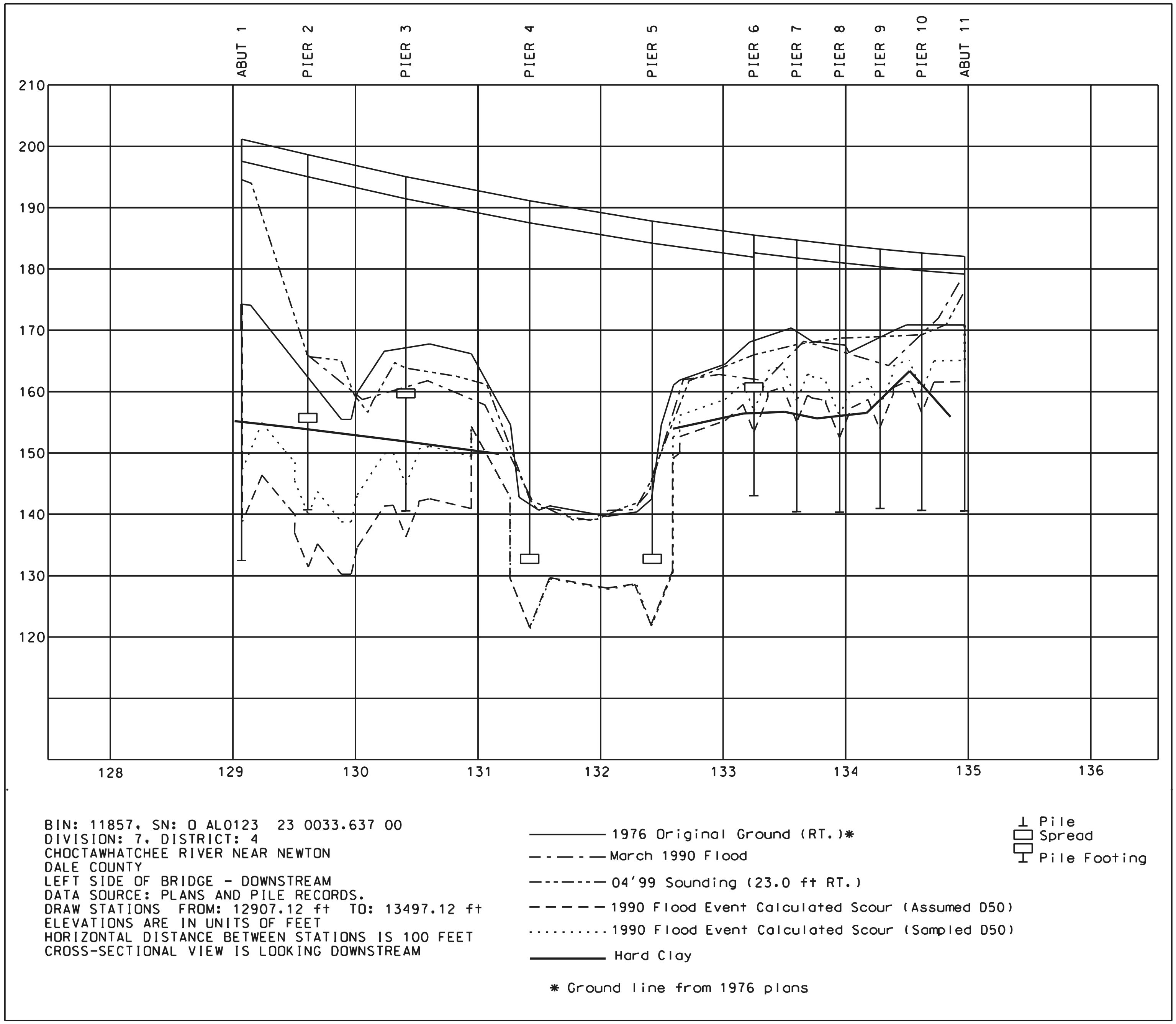


Figure 22. HEC-18 scour calculations for Choctawhatchee River near Newton

HEC-18 Scour Calculations

The hydraulic variables of the output of HEC-RAS were used to calculate scour depths. HEC-RAS has built in routines for calculating scour based on methods described in Hydraulic Engineering Circular No. 18 (HEC No. 18, FHWA, 1995). Contraction scour and local scour (pier scour and abutment scour) were computed for each site. As stated previously, all scour calculations were based on methods described in HEC-18. The following section describes how scour was calculated with excerpts taken directly from HEC-18 (FHWA, 1993, 1995, 2001) and the HEC-RAS Hydraulic Reference Manual (Brunner, 2001b).

Contraction Scour

As presented in HEC-18 and HEC-RAS, contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or a bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. The contraction of flow due to a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This flow from the floodplain can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

There are two conditions for contraction scour: clear-water and live-bed scour. Clear-water scour occurs when the bed material sediment transport in the uncontracted approach section is negligible or material transported through the contracted section is mostly in suspension. Live-bed scour occurs when there is transport of bed material from the upstream reach into the crossing.

Four conditions of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river;
b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment; or
c. Abutments are set back from the stream channel.

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river.

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to case 1).

D_{50} values can be used to determine the velocity associated with the initiation of motion, which in turn can be used as an indicator for clear-water or live-bed scour conditions. If the mean velocity (V) in the upstream reach is equal to or less than the critical velocity (Vc) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water scour. Also, if the ratio of the shear velocity of the flow to the fall velocity of the D_{50} of the bed material (V^*/ω) is greater than 3, contraction and local scour may be clear-water. If the mean velocity is greater than the critical velocity of the median bed material size, live-bed scour will occur.

The following equation is used by HEC-RAS to calculate the critical velocity. The derivation of the equation can be seen in HEC-18 (Second Edition, FHWA, 1993, p. 12).

$$Vc = 10.95 y_1^{1/6} D_{50}^{1/3} \tag{1}$$

Where:

Vc = Critical velocity above which bed material of size D_{50} and smaller will be transported, ft/s

y_1 = Average depth of flow in the main channel or overbank area at the approach section, ft

D_{50} = Bed material particle size in a mixture of which 50% are smaller, ft

Live-Bed Contraction Scour

Live-bed contraction scour was calculated in HEC-RAS using a modified version of Laursen's 1960 equation (HEC-18 Fourth Edition, FHWA, 2001 p. 5.10) for live-bed scour at a long contraction. The modification is to eliminate the ratio of Manning's n. The equation assumes that bed material is being transported in the upstream section.

$$y_2 = y_1 \left[\frac{Q_2}{Q_1}\right]^{6/7} \left[\frac{W_1}{W_2}\right]^{k_1} \qquad (2)$$

$$y_s = y_2 - y_0 \qquad (3)$$

Where:

y_s = Average depth of contraction scour, ft.

y_1 = Average depth in the upstream main channel, ft.

y_2 = Average depth after scour in the contracted section, ft. This is taken as the section inside the bridge at the upstream end in HEC-RAS.

y_0 = Average depth before scour in the main channel or floodplain at the contracted section, ft.

Q_1 = Flow in the main channel or floodplain at the approach section, which is transporting sediment, cfs.

Q_2 = Flow in the main channel or floodplain at the contracted section, which is transporting sediment, cfs.

W_1 = Bottom width in the main channel or floodplain at the approach section, feet. This is approximated as the top width of the active area in HEC-RAS.

W_2 = Bottom width of the main channel or floodplain at the contracted section less pier widths, feet. This is approximated as the top width of the active flow area.

k_1 = Exponent for mode of bed material transport

Table 3. Determining values for $\mathbf{k_1}$

V*/ω	$\mathbf{k_1}$	**Mode of Bed Material Transport**
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

$V_* = (\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, ft/s
ω = Fall velocity of bed material based on the D_{50}, ft/s
g = Acceleration of gravity, ft/s^2
S_1 = Slope of energy grade line of main channel, ft/ft

Clear-Water Contraction Scour

The following equation is used by HEC-RAS to calculate clear-water contraction scour. The derivation of the equation can be seen in HEC-18 (Second Edition, FHWA, 1993, p. 12).

$$y_2 = \left[\frac{Q_2^2}{C D_m^{2/3} W_2^2} \right]^{3/7} \tag{4}$$

$$y_s = y_2 - y_0 \tag{5}$$

Where:

Dm	= Diameter of the smallest non-transportable particle in the bed material ($1.25D_{50}$) in the contracted section, ft.
D_{50}	= Median diameter of the bed material, ft
C	= 120 for English units
W_2	= Bottom width of the bridge less pier widths, or overbank width (set back distance), ft

LOCAL SCOUR

Local Scour at Piers

Pier scour occurs due to acceleration of flow around the pier and the formation of flow vortices (known as the horseshoe vortex). The horseshoe vortex removes material from the base of the pier, creating a scour hole. The factors that affect the depth of local scour at a pier are: velocity of the flow just upstream of the pier, depth of flow, width of the pier, length of the pier if skewed to the flow, size and gradation of bed material, angle of attack of approach flow, shape of pier, bed configuration, and the formation of ice jams and debris.

HEC-RAS uses the Colorado State University (CSU) equation to calculate pier scour under both live-bed and clear-water conditions. The equation is presented in HEC-18 (Fourth Edition, FHWA, 2001, p. 6.4).

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \tag{6}$$

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 given in Table 4:

Table 4. Correction factor for pier nose shape

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.

If L/a is larger than 12, the program uses L/a = 12 as a maximum. If the angle of attack is greater than 5 degrees, K_2 dominates and K_1 should be set to 1.0.

The correction factor for bed condition, K_3, is shown in the table below:

Table 5. Correction factor for bed condition

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 by A. Molinas at CSU, 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):

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

where:

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

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

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

$$V_c = 10.95 Y^{1/6} D_c^{1/3} \tag{11}$$

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 below:

Table 6. 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

IV. CALCULATION OF SCOUR IN COHESIVE SOILS

SRICOS METHOD FOR PIER SCOUR

The Scour Rate in Cohesive Soils (SRICOS) method was introduced by Briaud et al. (1999) for a constant approach velocity and a circular pier. The SRICOS method for pier scour was later extended by Briaud et al. (2001b) for multiflood and multilayer situations. In NCHRP report 24-15 by Briaud et al. (2003) the SRICOS method for pier scour is further extended for use with complex piers.

The SRICOS method first consists of obtaining Shelby tube samples from the bridge site and performing EFA tests on them to find the erosion function of the soil. Then the maximum shear stress around a circular pier on a flat bottom is found with the following equation,

$$\tau_{\max} = 0.094\rho V^2 \left(\frac{1}{\log \mathrm{Re}} - \frac{1}{10} \right) \tag{12}$$

where ρ is the density of water (1000 kg/m^3), V is the velocity of the water, and Re is the Reynolds Number defined as VB/ν, where B is the pier diameter and ν is the kinematic viscosity of water (10^{-6} m^2/s at 20°C).

Once τ_{max} is found the corresponding initial scour rate ($\dot{z}_i$) can be found from the erosion function. The erosion function is the scour rate ($\dot{z}$) versus shear stress (τ) curve that is found by doing soil tests using the EFA.

The maximum pier scour depth is the scour depth attained after a long time of exposure to flood conditions. This depth may be better termed as the "ultimate" scour depth (McLean et al., 2003 a, b). The equation for this maximum pier scour depth is given by Briaud et al. (1999) as

$$z_{max} = 0.18mm\left(Re\right)^{0.635}. \tag{13}$$

The time variation of scour depth can now be calculated with the following equation,

$$z = \frac{t}{\dfrac{1}{\dot{z}_i} + \dfrac{t}{z_{max}}} \tag{14}$$

where t is the length of time since the beginning of the flood starting with a flat bottom around the pier. This function satisfies the condition z = 0 when t = 0, $dz/dt \rightarrow \dot{z}_i$ when $t \rightarrow 0$, and $z \rightarrow z_{max}$ when $t \rightarrow \infty$.

E-SRICOS for Multiflood Conditions

In this method the time dependent hydrograph is broken into consecutive segments of time intervals Δt each with a constant approach velocity. For each time interval, first an equivalent elapsed time t* is calculated as

$$t^* = \frac{\dfrac{z_1}{\dot{z}_i}}{\left(1 - \dfrac{z_1}{z_{max}}\right)} \tag{15}$$

where z_1 is the cumulative scour depth at the beginning of time interval Δt, $\dot{z}_i$ is the initial scour rate for τ_{max} obtained with a flat bottom corresponding to the approach velocity during a time interval Δt, and z_{max} is the maximum scour corresponding to the approach velocity for time interval Δt. The present equation 15 corresponds to equation 7 of Briaud et al. (2001b), but with a different, simpler, form.

Once this t* is found for multifloods the cumulative scour depth at the end of the time interval Δt can be calculated as

$$z_2 = z(t^*+\Delta t) = \frac{t^*+\Delta t}{\frac{1}{\dot{z}_i} + \frac{t^*+\Delta t}{z_{max}}} \tag{16}$$

where z_2 is the cumulative scour depth at the end of time interval Δt. The time variation of cumulative scour is calculated in this manner using daily flows so the time interval Δt is one day.

S-SRICOS with Equivalent Time

The Simple SRICOS method was developed to do quick hand calculations to predict the scour that would occur for a variable flow hydrograph of a long duration. To do this an equivalent time (t_e) has to be found to substitute for t in equation 14 along with the values of $\dot{z}_i$ and z_{max} corresponding to maximum approach velocity of record. The equation given by Briaud et al. (2001b) for calculating the equivalent time is

$$t_e = 73\left(t_{hydro}\right)^{0.126}\left(V_{max}\right)^{1.706}\left(\dot{z}_i\right)^{-0.20} \tag{17}$$

where t_{hydro} (yrs) is the number of years that the bridge has been built, V_{max} is the maximum velocity of record, and $\dot{z}_i$ is the initial scour rate corresponding to τ_{max} from equation 12.

Square Piers

The equations above are all given for circular piers. To do the calculations for square piers there are some shape factors from Briaud et al. (2003) that must be taken into account. A shape factor must be multiplied into both τ_{max} and z_{max}. The shape factor that is multiplied into τ_{max} (equation 12) is given as

$$k_{sh} = 1.15 + 7e^{-4\left(\frac{L}{B}\right)} \tag{18}$$

where L and B are the dimensions of the piers. In our case we only dealt with square piers, so L = B. The shape factor that is given to be multiplied into z_{max} (equation 2) is $K_{sh} = 1.1$. These shape factors enable us to use the equations for circular piers for square piers by multiplying the equations by the appropriate shape factor.

Before access to the recent report by Briaud et al. (2003) we used another approach to define an effective diameter for a square pier. This effective diameter was calculated as

$$B = a\sqrt{2} \tag{19}$$

where a is the width of the square pier.

SRICOS Method for Contraction Scour

The SRICOS method for contraction scour is outlined by Briaud et al. (2003). The calculations that were done for the SRICOS method in this report followed the same procedures except for a few modifications.

In Briaud et at. (2003) the Manning n is used to get the bottom shear stress, while we are using the Darcy-Weisbach friction factor to get bottom shear stress. The maximum shear stress at the bottom is calculated using the equation,

$$\tau_{max} = \frac{1}{8}\rho f V_{hec}^2 \tag{20}$$

where ρ is the density of water (1000 kg/m^3), V_{hec} is the velocity that comes from HEC-RAS, and f = f (Re_{con}) is the friction factor assuming a smooth boundary, where Re_{con} is the Reynolds number in the contraction defined as $Re_{con} = 4q/\nu$ where q is the flow per unit width and ν in the kinematic viscosity of water (10^{-6} m^2/s at 20°C).

Once τ_{max} is found the corresponding initial scour rate ($\dot{z}_i$) can be found from the erosion function. The erosion function is the scour rate ($\dot{z}$) versus shear stress (τ) curve that is found by doing soil tests using the EFA.

Now the ultimate value of the maximum scour depth in the contraction can be found. In order to do this the critical Froude number and the Froude number corresponding to the velocity in the contraction must be found. The critical Froude number is calculated as

$$Fr_c = \sqrt{\frac{8\tau_c}{\rho g f y_h}} \tag{21}$$

and the following equation is used to calculate the Froude number corresponding to the velocity in the contraction,

$$Fr_{hec} = \frac{V_{hec}}{\sqrt{g y_h}} \tag{22}$$

where τ_c is the critical shear stress of the soil, ρ is the density of water (1000 kg/m^3), g is acceleration due to gravity (9.81 m^2/s), f is the friction factor, and y_h is the water depth in the contraction, and V_{hec} is the velocity that comes from HEC-RAS. Now the maximum scour depth in the contraction can be calculated as

$$z_{max}(\text{Cont}) = 1.90\left[1.49 Fr_{hec} - Fr_c\right] y_h \tag{23}$$

Briaud et al. (2003) also defined z_{unif} (Cont), but we were only concerned with z_{max} (Cont). This equation is equivalent to equation 7.9 in Briaud et al. (2003).

The time variation of scour depth can now be calculated with the following equation,

$$z = \frac{t}{\frac{1}{z_i} + \frac{t}{z_{max}(\text{Cont})}} \tag{24}$$

where t is the length of time of the flood that scour is being calculated for.

E-SRICOS for Multiflood Conditions

The Extended SRICOS method for multiflood conditions had to be used in order to calculate the cumulative contraction scour for the entire period of record as in the case of pier scour, first an equivalent elapsed time, t* is calculated using the following equation at each time step,

$$t^* = \frac{\frac{z_1}{z_i}}{\left(1 - \frac{z_1}{z_{max}(\text{Cont})}\right)} \tag{25}$$

where z_1 is the cumulative scour depth at the beginning of time interval Δt, $\dot{z}_i$ is the initial scour rate for τ_{max} corresponding to the approach velocity during the time interval Δt, and z_{max} is the ultimate scour depth for Re_{con} corresponding to the approach velocity for time interval Δt.

Once this t* is found for multifloods the time variation of scour depth can then be calculated as

$$z_2 = z(t^{*}+\Delta t) = \frac{t^{*}+\Delta t}{\dfrac{1}{\dot{z}_i} + \dfrac{t^{*}+\Delta t}{z_{max}(\mathrm{Cont})}} \tag{26}$$

where z_2 is the cumulative scour depth at the end of time interval Δt.

DASICOS Method

The development of scour with time may also be calculated based on the rate of scour given by the erosion function depending on the local value of the bed shear stress. Because of its differential nature this approach may be called the DASICOS method (Differential Analysis of Scour In Cohesive Soils) for the present purposes. Examples of this approach have been presented in the recent studies of Güven et al. (2002a, b), Chen (2002), and McLean et al. (2003a, b). Güven et al. (2002a, b) uses a one-dimensional flow analysis while Chen (2002) uses a three-dimensional flow model. McLean et al. (2003a, b) flow model is two-dimensional. In the present study an approach similar to Güven et al. one-dimensional analysis is used.

In the DASICOS method the shear stress is calculated based on local conditions with the following equation,

$$\tau = \frac{\rho f q^2}{8y^2} \tag{27}$$

where ρ is the density of water (1000 kg/m^3), q is the flow per unit width, y is the depth, and f = f (Re_{con}) is the friction factor assuming a smooth boundary. The Reynolds number in the contraction is calculated as $Re_{con} = 4q/\nu$ where q is the flow per unit width and ν in the kinematic viscosity of water (10^{-6} m^2/s at 20°C). The depth that is used in the above equation is the scour depth added to the initial depth. The following equation is used for this at each time step,

$$y(t) = y_h + z(t) \tag{28}$$

where y is the new depth, y_h is the initial depth, and z is the scour for that time step.

The scour rate is defined as dz/dt and this is equal to R (τ) at any location. R is the erosion function which gives the scour rate corresponding to the shear stress.

$$\frac{dz}{dt} = R(\tau) \tag{29}$$

The cumulative scour in the contraction is calculated by integrating equation 29 using Euler's method with the following equation,

$$z_2 = z_1 + R(\tau_1)\Delta t \tag{30}$$

where z_2 is the cumulative scour depth at the end of time interval Δt, z_1 is the cumulative scour depth at the beginning of time interval Δt, τ_1 is the shear stress corresponding to the initial depth at time t_1 and the average velocity during time interval Δt, and R is the scour rate corresponding to τ_1 on the erosion function.

Ultimate Scour Depth

The ultimate scour depth can be found for any flood over an infinite time by first calculating the ultimate water depth (y_{ult}) and subtracting the initial depth corresponding to the flood (y_h) from it,

$$z_{ult} = y_{ult} - y_h \tag{31}$$

where y_{ult} is calculated from the following equation,

$$y_{ult} = \sqrt{\frac{\rho f q^2}{8\tau_c}} \tag{32}$$

where ρ is the density of water (1000 kg/m^3), q is the flow per unit width, τ_c is critical shear stress from the erosion function, and f = f (Re_{con}) is the friction factor assuming a smooth boundary. The Reynolds number in the contraction is calculated as $Re_{con} = 4q/\nu$ where q is the flow per unit width and ν in the kinematic viscosity of water (10^{-6} m^2/s at 20°C).

Programs

Programs were written in MATLAB to perform the above calculations. The code for different methods can be found in Appendix A.

V. RATING FUNCTIONS FOR THE SITES

Stage-discharge rating curves were obtained from the USGS for both Pea River at Elba (Figure 23) and Choctawhatchee River near Newton (Figure 24). Daily discharges since the construction of the bridge were obtained from the USGS for the Choctawhatchee River site only (Figure 25). We only used data through the 1990 flood due to riprap being added immediately after it. A full set of hydrologic data did not exist for Pea River at Elba. Hydraulic models were constructed for the sites. The cross-sections were broken up into a left overbank, a right overbank, and a main channel (Figure 26 and 27) due to the change in geometry across the sections. Incremental discharges and stages were taken from the rating curves for determining flow distributions for the overbanks and main channel. Top widths were taken for the overbanks and main channel from the model to determine unit discharges, q (Discharge/TopWidth). Hydraulic depths, y_h (Area/TopWidth) were taken from the model as well. Using discharge versus q data and discharge versus y_h data, plots (found in the Appendix B) were developed of stage versus q and stage versus y_h. The relationships were used to interpolate the q's and the y_h's using the daily discharge records. The entire record of the daily discharges for Choctawhatchee River near Newton was filtered to discard the discharges less than 800 cfs in order to reduce the amount of calculations. The critical shear stress is not exceeded until near 3000 cfs. The q and y_h data were used as input for the time dependent cohesive scour calculations.

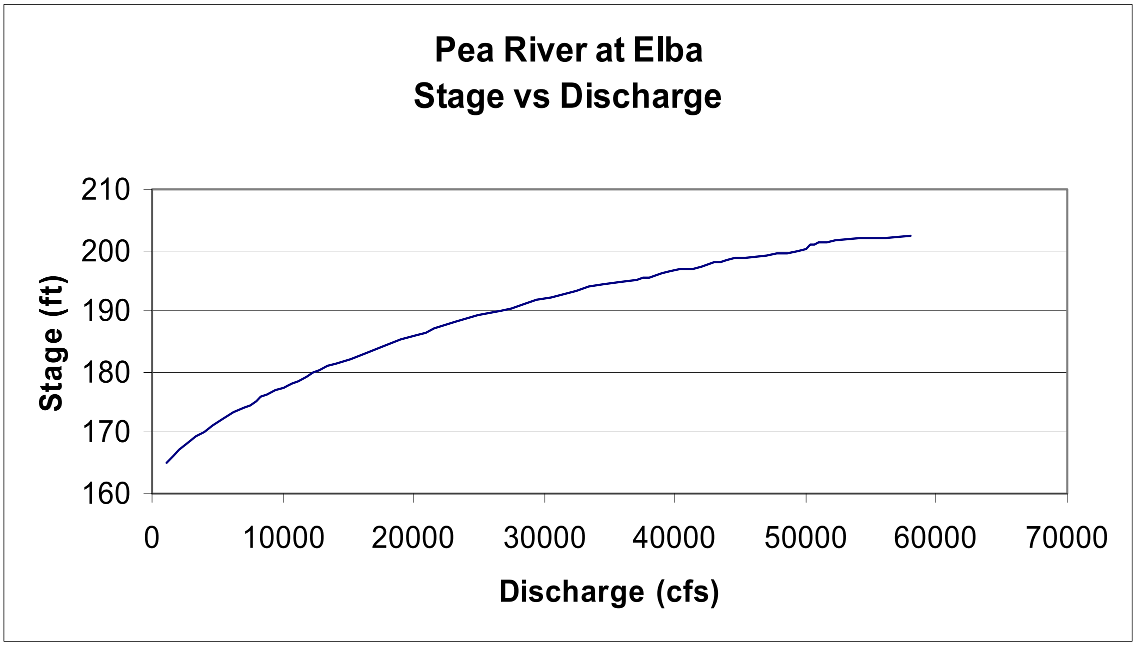


Figure 23 Stage vs discharge for Pea River at Elba

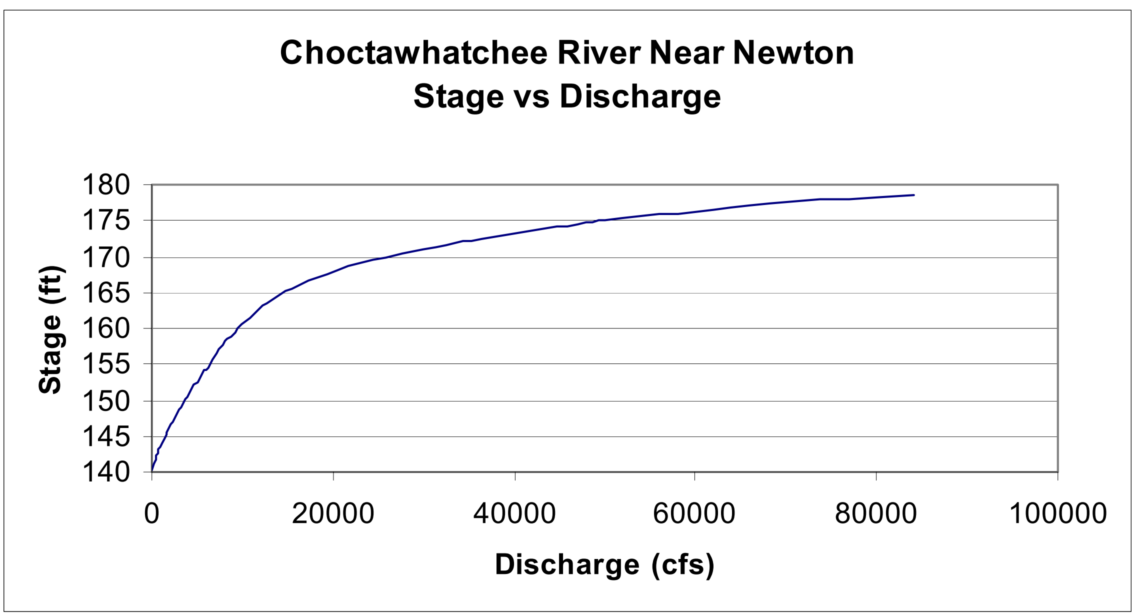


Figure 24. Stage vs discharge for Choctawhatchee River near Newton

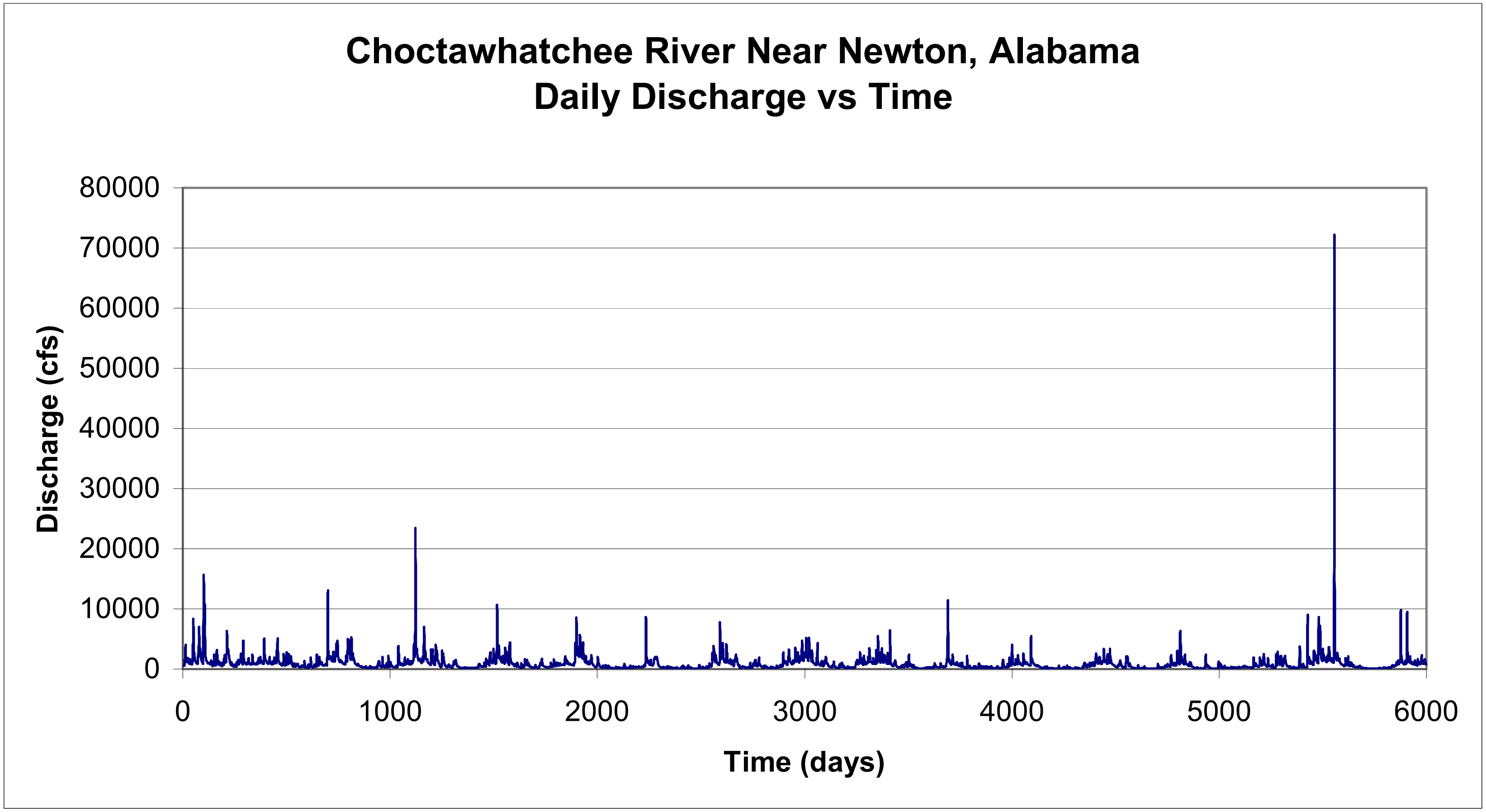


Figure 25. Daily discharges from 1975 to 1990

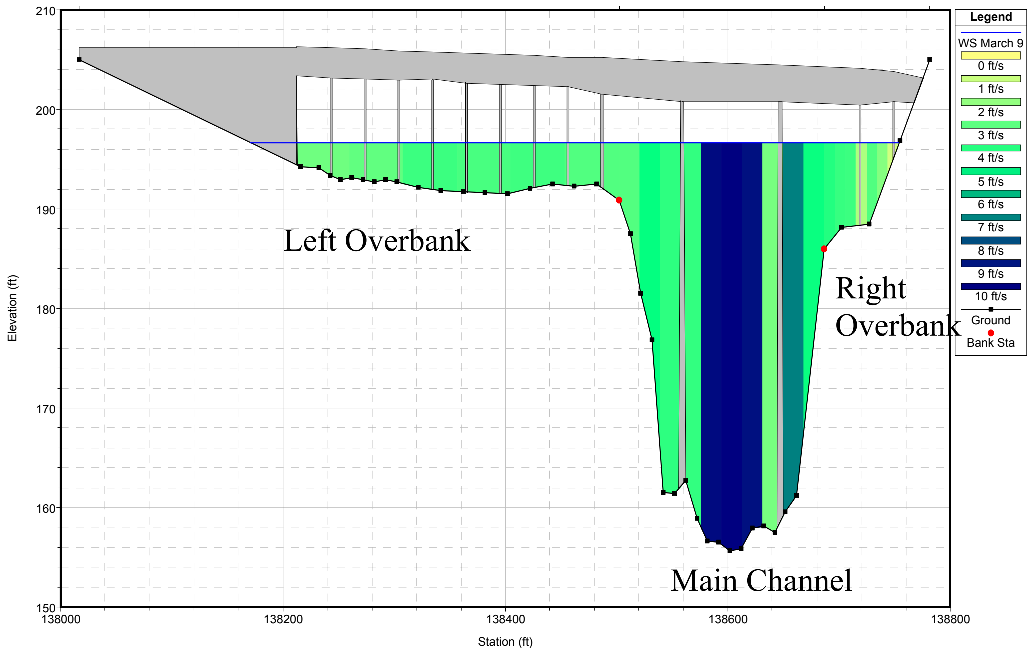


Figure 26. Cross-section of Pea River at Elba

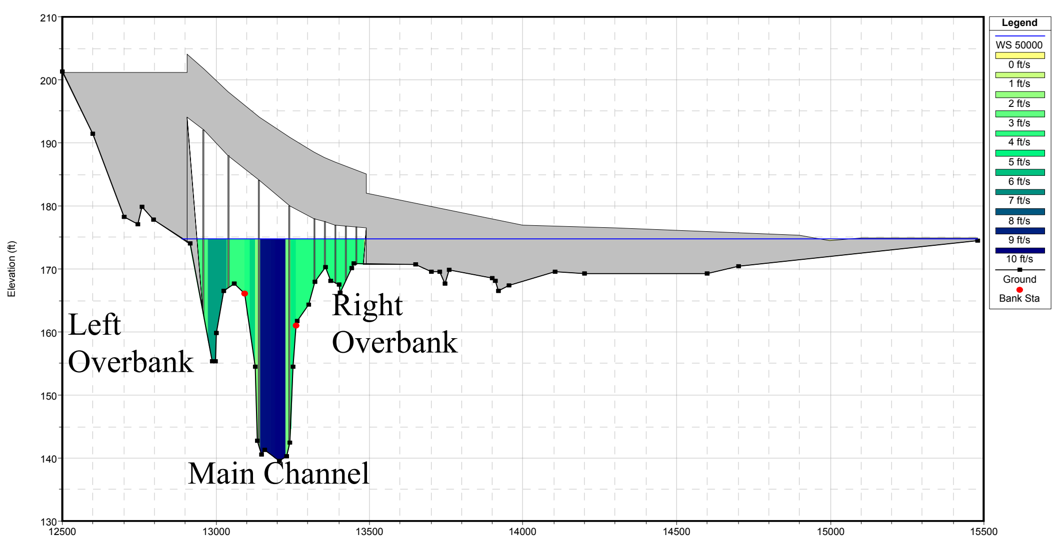


Figure 27. Cross-section of Choctawhatchee River Near Newton

VI SOIL EXPLORATION AND EFA DATA FOR THE TWO SITES

The Alabama Department of Transportation (ALDOT) supplied the soil samples used in this study. The samples were obtained in the field by pushing or driving an ASTM standard Shelby tube with an outside diameter of 76.2 mm into the ground (ASTM-D1587). ALDOT also supplied the boring logs with the samples. These logs gave valuable information that was recorded during the actual sampling process. This information included the depth at which the sample was taken, soil descriptions, and blow counts (N). The blow counts were determined with standard penetration tests (ASTM-D1586). Figure 28 shows the site sketch with core boring locations and Figure 29 shows the core borings for Pea River at Elba. Figure 30 shows the site sketch with core boring locations and Figure 31 shows the core borings for Choctawhatchee River near Newton.

The Samples were tested from the Pea River site at Elba and from the Choctawhatchee River site near Newton. These samples are listed in TABLE 7 with depths and soil descriptions taken from boring logs.

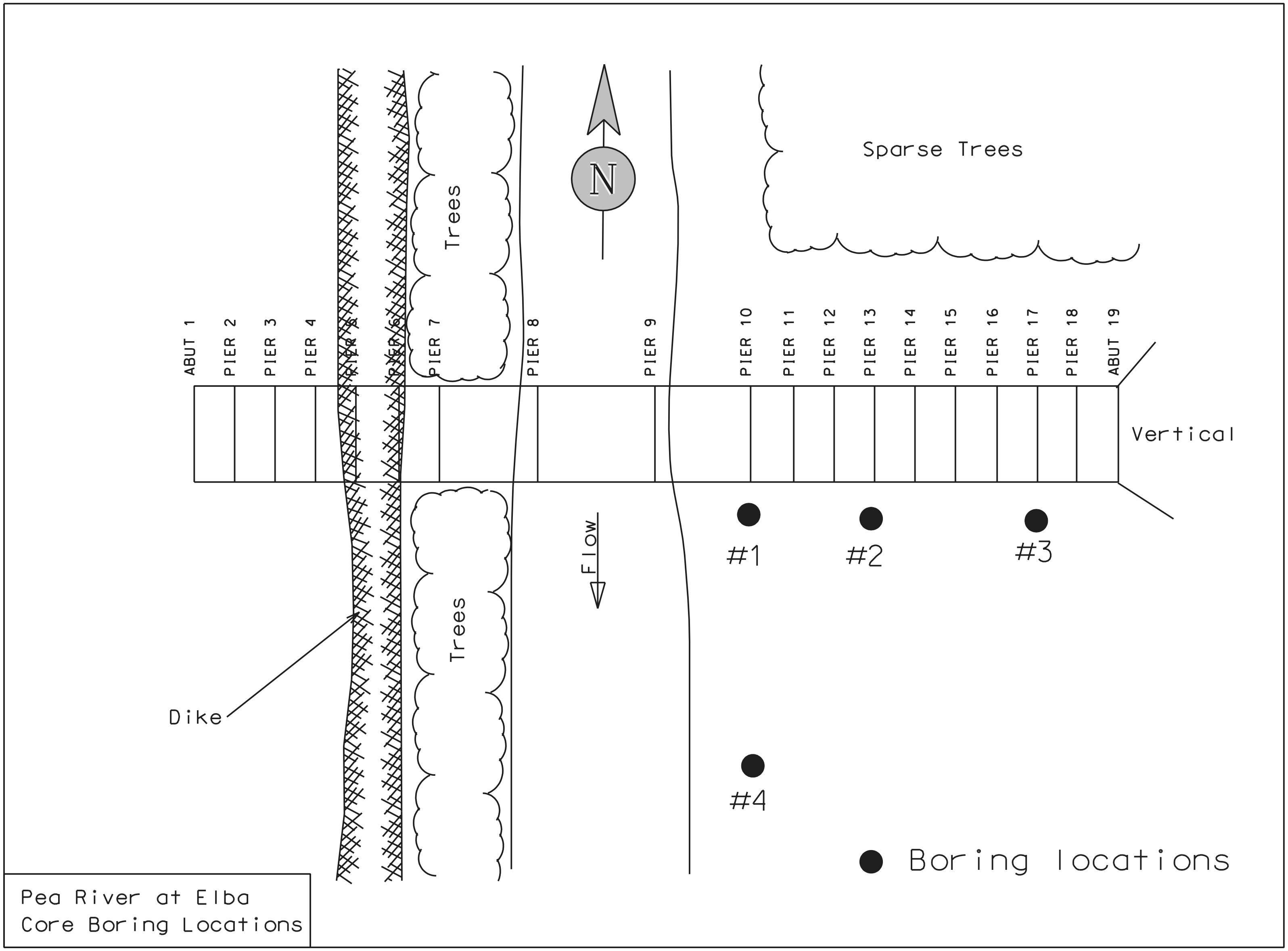


Figure 28. Site sketch of Pea River site at Elba with boring locations

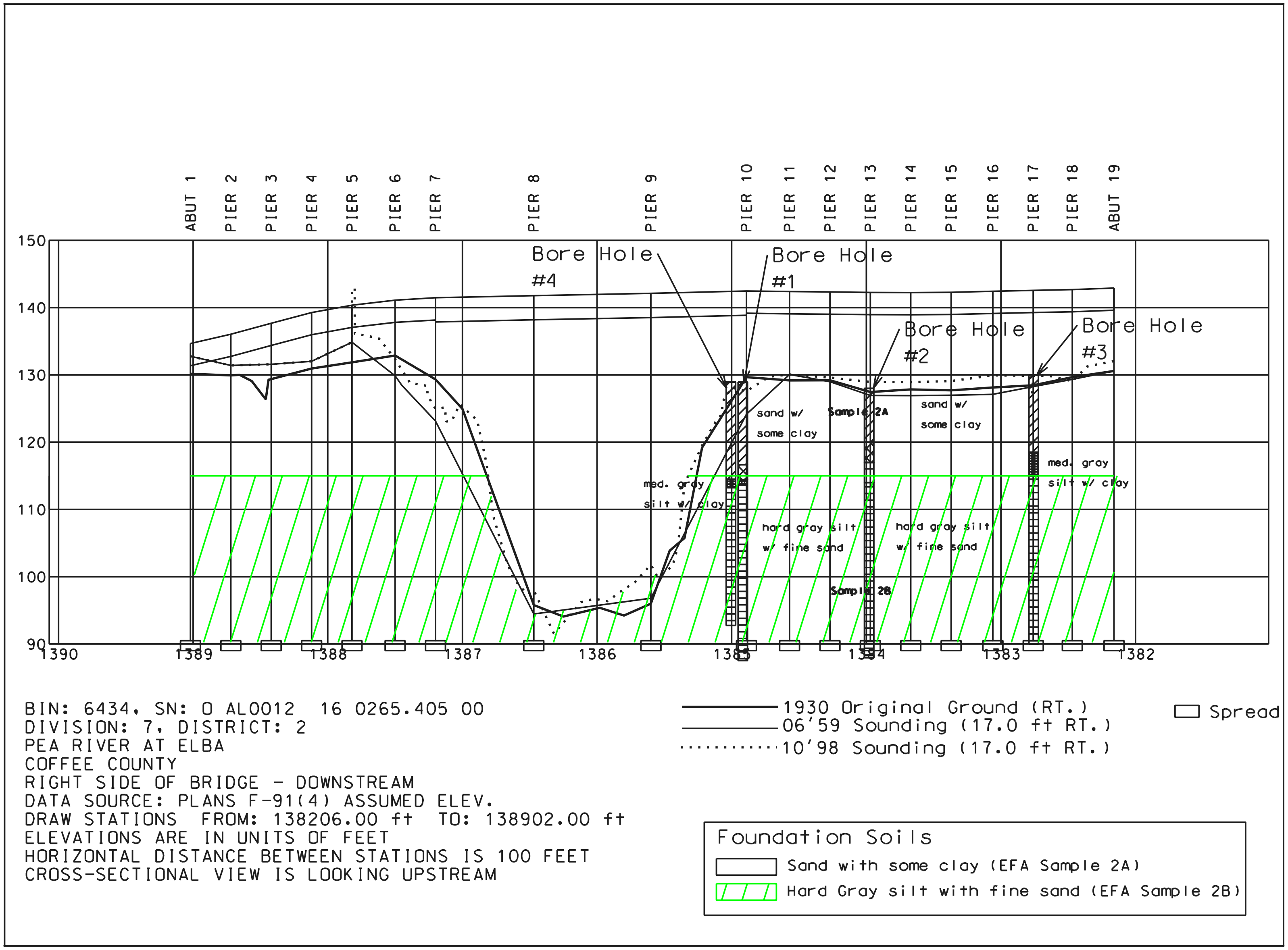


Figure 29. Core borings and foundation soils for the Pea River site at Elba

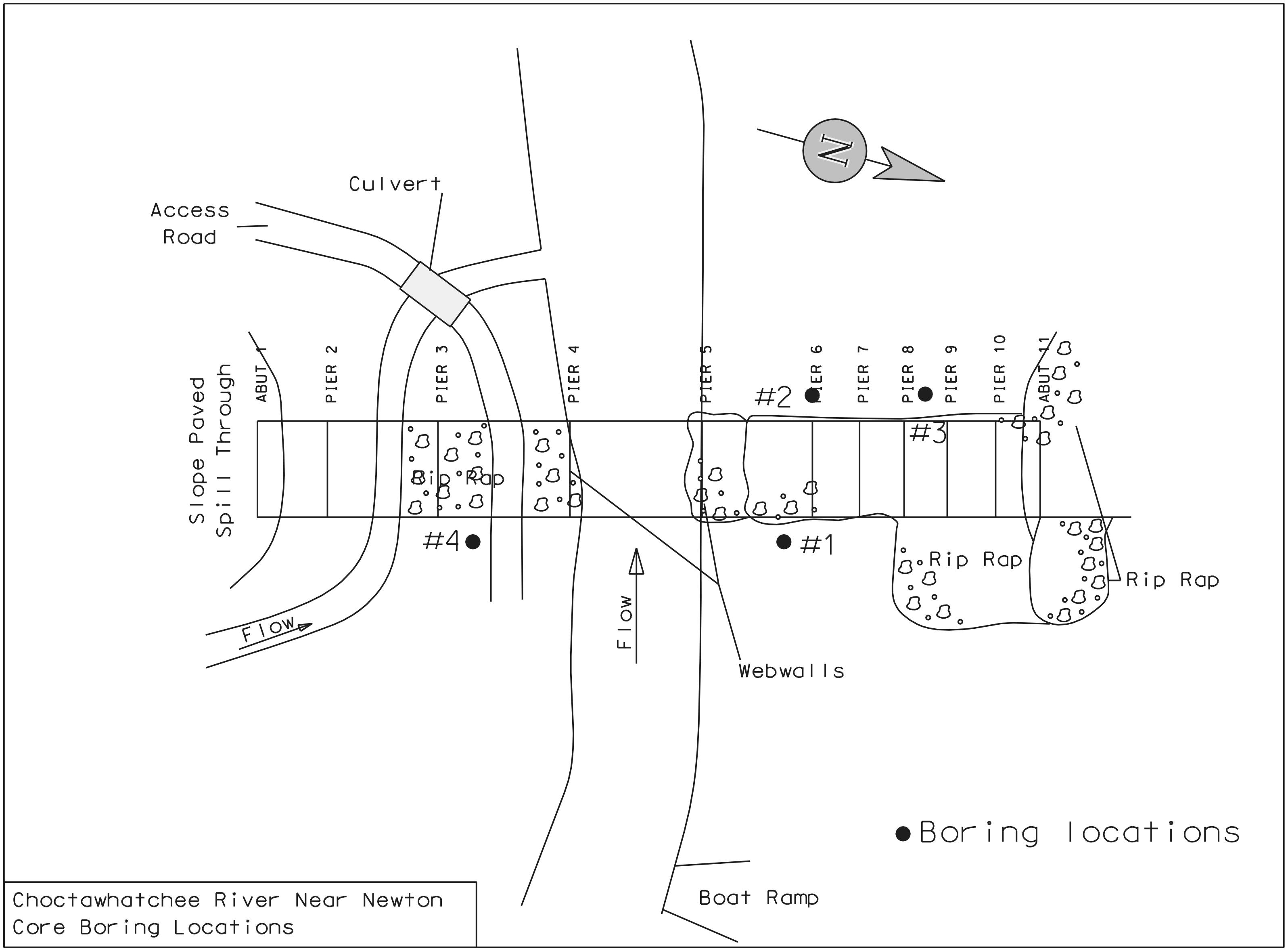


Figure 30. Site sketch of the Choctawhatchee River site near Newton with boring locations

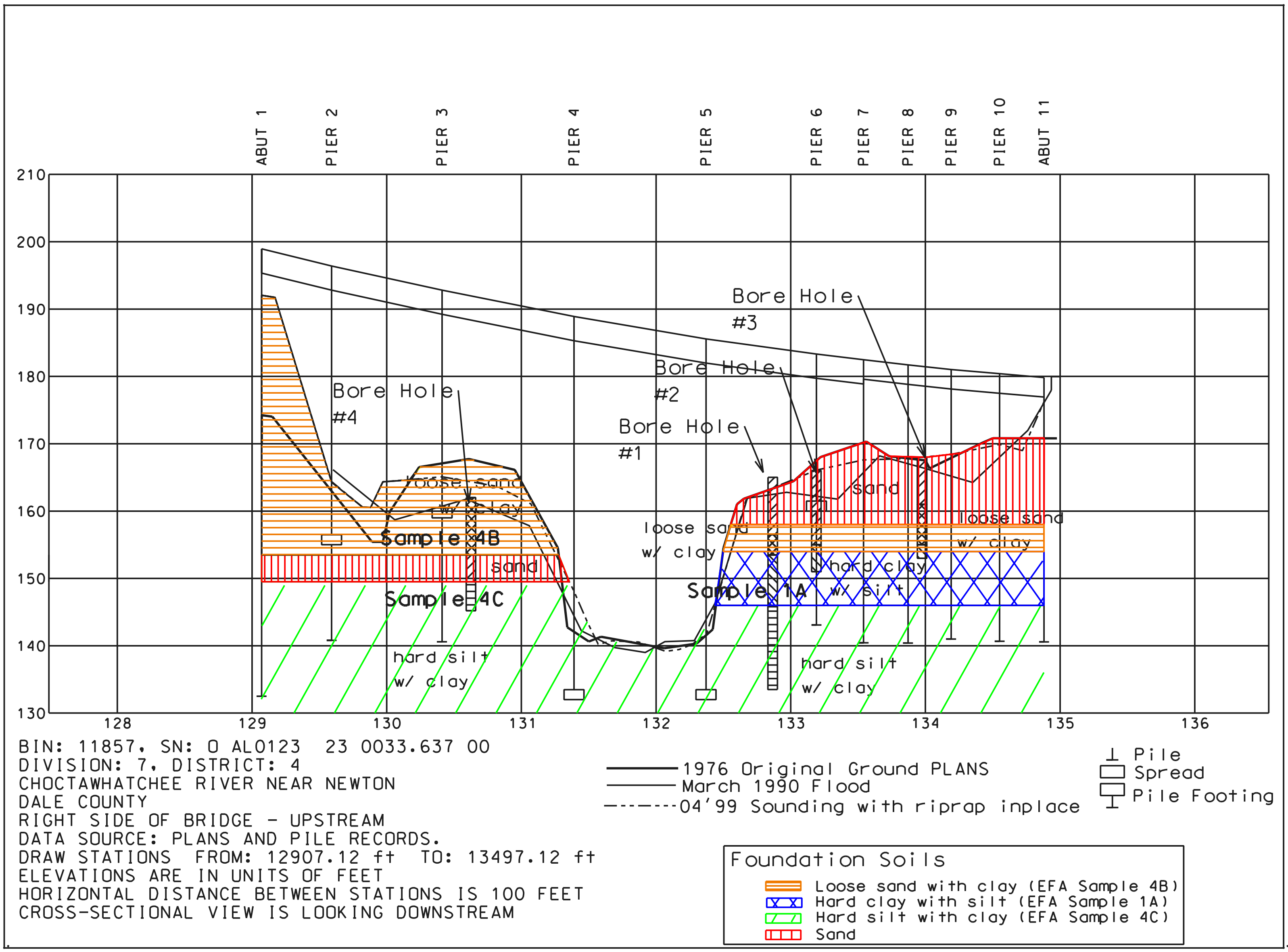


Figure 31. Core borings and foundation soils for the Choctawhatchee River site near Newton

EFA TESTING

The EFA can be used for any type of soil which can be sampled with a standard Shelby tube. It has been used for both coarse grained soils such as sands and for fine grained soils such as clays. The EFA is used to find the erosion function of a soil. The erosion function is the relation between the scour rate ($\dot{z}$) and the shear stress (τ) as shown in Figure 32. The critical shear stress (τ_c) is the shear stress below which no scour takes place. The initial erodibility (S_i) indicates how fast the soil scours at the critical shear stress and is the slope of a straight line tangent to the erosion function at the critical shear stress.

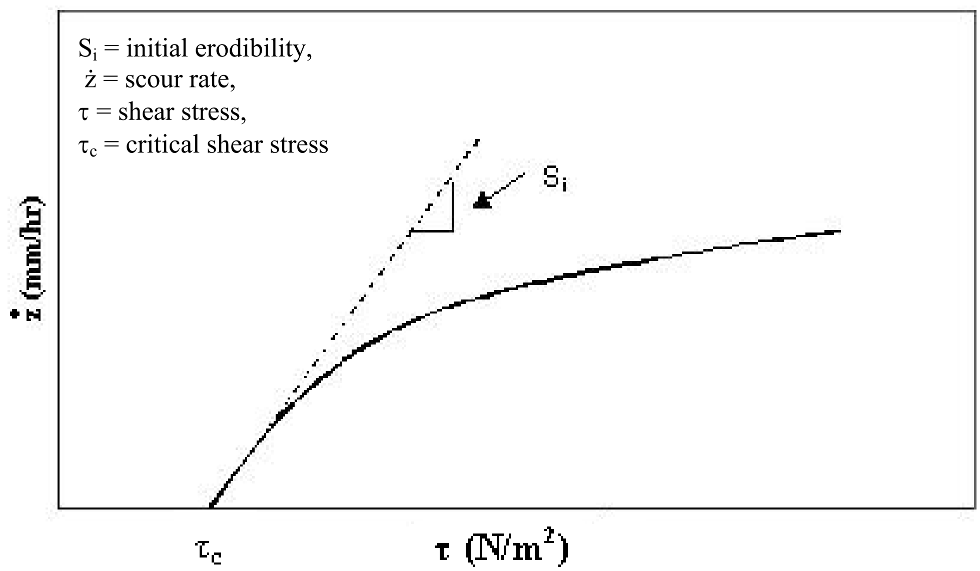


Figure 32. Erosion function obtained from running an EFA test

Briaud et al. (1999, 2001a, 2001b) developed the EFA and the basic operating procedures can be found in Briaud et al. (2001a). The EFA for this study (Figure 33) was essentially the same as the EFA described by Briaud et al. (2001a), but there were some differences that made the operating procedure a little different. A detailed description of the EFA testing procedure can be found in Crim (2003). Figure 34 shows a sketch of the important parts of the EFA.

FIGURE 33. Auburn University's Erosion Function Apparatus (EFA)

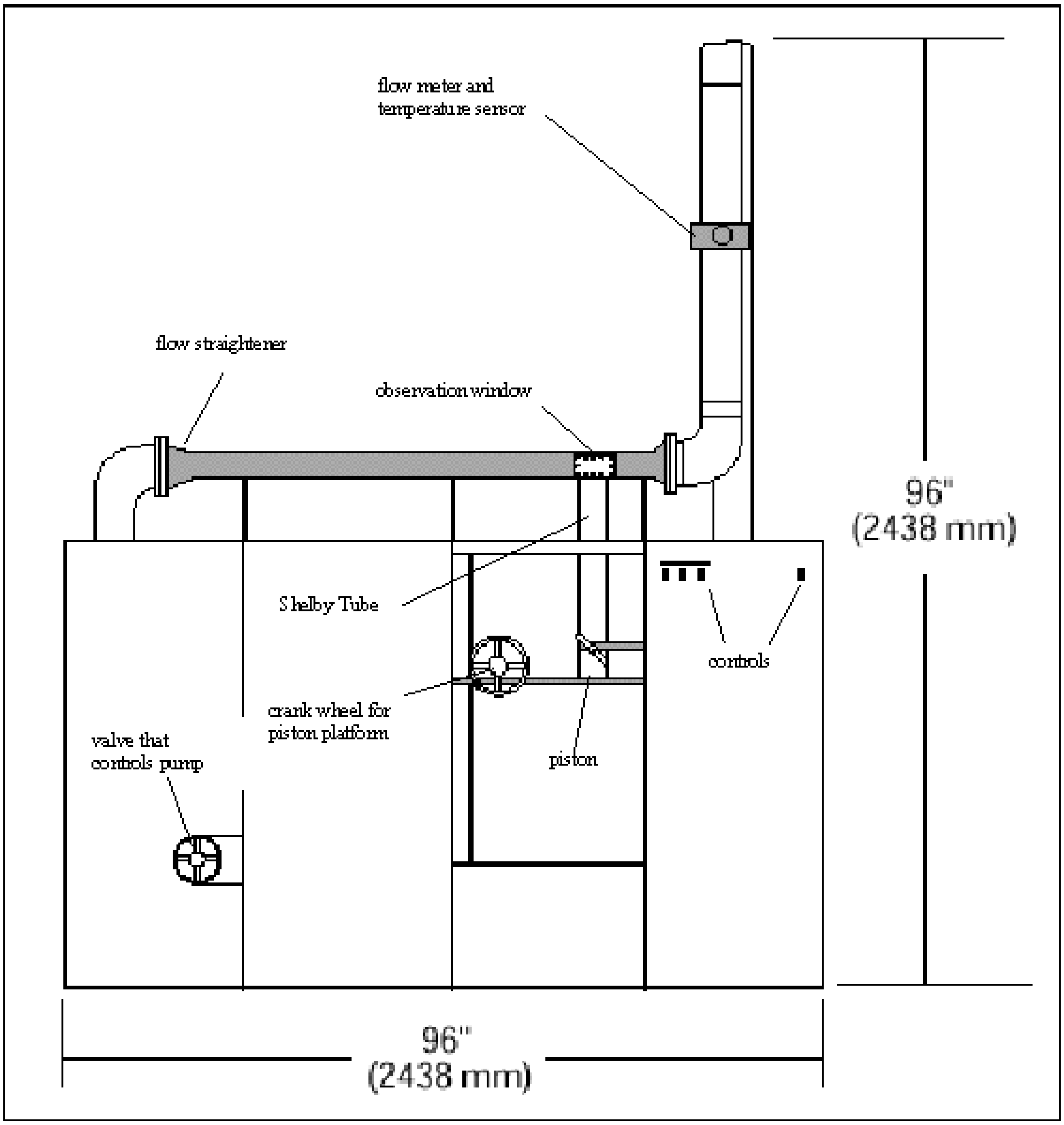


FIGURE 34. Schematic showing the important parts of the EFA

To begin an EFA test the tank is filled with water and the prepared sample installed in the EFA. The Shelby tube is held vertically over the piston and slowly pushed down over the piston. Once the Shelby tube is in place over the piston it is secured by tightening a clamp.

After the Shelby tube is securely in place the soil sample is brought to the top of the tube by pushing the piston control on the EFA in the up position until the sample comes out of the top of the Shelby tube. Once this is done the soil is trimmed evenly with the top of the sampling tube.

The sample is then inserted into the rectangular conduit opening. The sample is raised into the opening by using the crank wheel, aligned flush with the bottom of the conduit, and the two screws on the platform tightened so that the Shelby tube cannot move during testing (Figure 35).

FIGURE 35. Raising the Shelby tube into the conduit opening and placing it flush with the bottom using the crank wheel

The pump is turned on and the valve to regulate water velocity is opened allowing flow through the conduit. The flow rate is measured by means of a propeller type flow meter. The flow rate is combined with the cross sectional area of the pipe to compute the average velocity of the flow.

After the velocity is set the soil is raised into the flow 1 mm in 0.5 mm increments, which is controlled by the EFA computer. The computer records time, average velocity, temperature, the soil sample advance that is pushed, and elapsed time.

The flow is maintained until 1 mm of soil is completely scoured away. The scour is usually not uniform and the surface of the soil sample usually becomes uneven through the duration of a test. Some of the exposed sample surface may have scoured more than 1 mm while some of it may have scoured less. When this happens the operator subjectively decides when the scour is "on average" about 1 mm.

At the end of a test the pump is turned off and the water drains from the conduit. The soil sample can then be lowered out of the conduit opening and prepared for the next test. This is done by pushing some of the soil through the sampling tube and trimming it even with the top of the tube. The sample is again raised into the opening and the test at the next velocity is run. The test is repeated for between 5 and 8 velocities. By doing the test at several velocities, the scour rate (mm/hr) vs. velocity (m/s) data is obtained. This data is evaluated to give a scour rate (mm/hr) vs. shear stress (N/m^2) relationship, which is defined as the erosion function of the soil.

EFA TEST DATA REDUCTION AND PRESENTATION

The scour rate ($\dot{z}$) is the measure of how fast a particular soil erodes over time. The scour rate for a particular soil with a set water velocity flowing over it can be calculated from an EFA test. This scour rate is

$$\dot{z} = \Delta h / \Delta t \qquad (33)$$

where Δh is the length of soil eroded in a time Δt. The length Δh that is eroded during an individual test at a specified velocity is 1 mm. The time Δt is how long it takes for the 1 mm of soil sample to be eroded.

The shear stress applied by the water to the soil at the soil water interface is generally considered to be the major parameter causing erosion (Briaud et al., 2001a). The EFA does not directly give the shear stress applied to the soil. It does however give velocity (V), which is related to the shear stress that the water imposes on the soil sample.

According to Briaud et al. (2001a) the best way to determine the shear stresses for the EFA is by using the Moody chart which gives the relation between the pipe friction factor f, the Reynolds Number Re, and the relative roughness ε/d where ε = roughness height and d = pipe diameter. The Reynolds number is calculated as

$$\mathrm{Re} = \frac{\mathrm{VD}}{\upsilon} \qquad (34)$$

where V = average velocity in the pipe, D = $4R_h$ = hydraulic diameter of the pipe (R_h = hydraulic radius), and υ = kinematic viscosity of water (10^{-6} m^2/s at 20˚C).

After the Reynolds number is calculated the friction factor can be determined. For the cohesive and fine-grained soils that were tested in this study the roughness was considered to be smooth in the EFA. The equation that is used to calculate the friction factor for smooth conditions is

$$\frac{1}{\sqrt{f}} = 2.0 \log \left(\mathrm{Re}\sqrt{f}\right) - 0.8 . \tag{35}$$

This equation was used for smooth conditions in Moody's original paper (Moody, 1944). An approximation of equation 35, called the Blasius equation (Henderson, 1966), that was used for $\mathrm{Re} < 10^5$ is

$$f = \frac{0.316}{\mathrm{Re}^{1/4}} . \tag{36}$$

This equation simplifies some of the calculations by making it possible to calculate f directly if the Reynolds number is known. If $\mathrm{Re} > 10^5$ then equation 35 is used to find the friction factor. These are the equations for the smooth line on the Moody diagram and make it possible to calculate the friction factor for a smooth surface without having to go to the actual Moody diagram (Henderson, 1966).

The shear stress in the EFA is calculated as

$$\tau = \frac{\rho f \mathrm{V}^2}{8} \tag{37}$$

where τ = shear stress, f = friction factor from the Moody chart, ρ = mass density of water (1000 kg/m^3), and V = average velocity in the conduit.

Scour rate and shear stress are plotted to develop erosion functions for soils. From these erosion functions, critical shear stress (τ_c), and initial erodibility (S_i) are determined. Best fit curves are visually determined for shear stresses greater than τ_c. The erosion functions for the tested soils can be found in Appendix C.

SOIL CLASSIFICATION TESTING

Soils were tested to determine particle size and plasticity. Particle size analysis was done using procedures in ASTM-D422 to determine D_{50}. Plastic limits, liquid limits, and plasticity indices (PI) of soils were determined using procedures in ASTM-D4318. Results from these tests are contained in TABLE 7. Based on EFA tests, soil erosion properties, τ_c, and S_i , for the soils are also shown in TABLE 7.

TABLE 7. Soil properties for the tested soils.

Sample	**Soil Description**	**Depth (ft)**	**D_{50} (mm)**	**PI (%)**	**τ_c (N/m^2)**	**S_i (mm/hr)/(N/m^2)**
Choctawhatchee (1A)	Gray Clay	10.0 - 12.0	0.027	24	2.5	0.96
Choctawhatchee (3B)	Sand w/ Clay	5.0 - 7.0	0.15	6	0.65	9.5
Choctawhatchee (4B)	Sand w/ Clay	6.8 - 8.0	0.32	NP*	0.46	10.4
Choctawhatchee (4C)	Gray Silt w/ Clay	11.0 - 13.0	0.029	14	1.25	1.2
Pea (2A)	Tan Clay	10.0 - 12.0	0.041	13	2.7	0.7
Pea (2B)	Gray Silt	13.5 - 15.5	0.032	11	1.4	-
Pea (3A)	Gray Silt	10 - 11.5	0.034	NP*	1.5	3.7

* NP = Non-Plastic

VII. SCOUR EVALUATION AND RESULTS FOR PEA RIVER AT ELBA

Peak discharge and daily discharge data do not exist from 1956 to 1971; also, data before 1974 and during the 1980s was unavailable. Only stage information was available for the 1990 flood (largest event of record). The 1990 flood data came from the USGS Water Resources Data reports. The stages were used to find the discharges for the events using the current USGS rating curve developed for the site. The EFA results and overbank's unit discharges from the largest event of record indicated that no scour occurs in the overbank because the critical shear stress is not exceeded. The soundings showed that no scour occurred in the overbank on the left side of the bridge and minor scour occurred on the right side of the bridge. The 1998 upstream sounding shows scour right at the beginning of the left overbank near the main channel, but is probably due to the exposed utility pipe that shows up in Figure 36. The downstream sounding shows no scour. The 1998 flood which was lower than the 1990 flood showed 2-3 feet of local scour between piers #10 and #12. A two-dimensional numerical model was constructed of the site and shows high velocity zones in the shape of and in the area of the scoured area (Figure 36). Blue indicates the higher velocities and red indicates the lower velocities. Due to the lack of data, time dependent scour was not computed for this site. Pier scour was computed for the main channel piers using the Simple SRICOS method and can be seen in Figure 37. Field observations and soundings indicated no scour in the main channel.

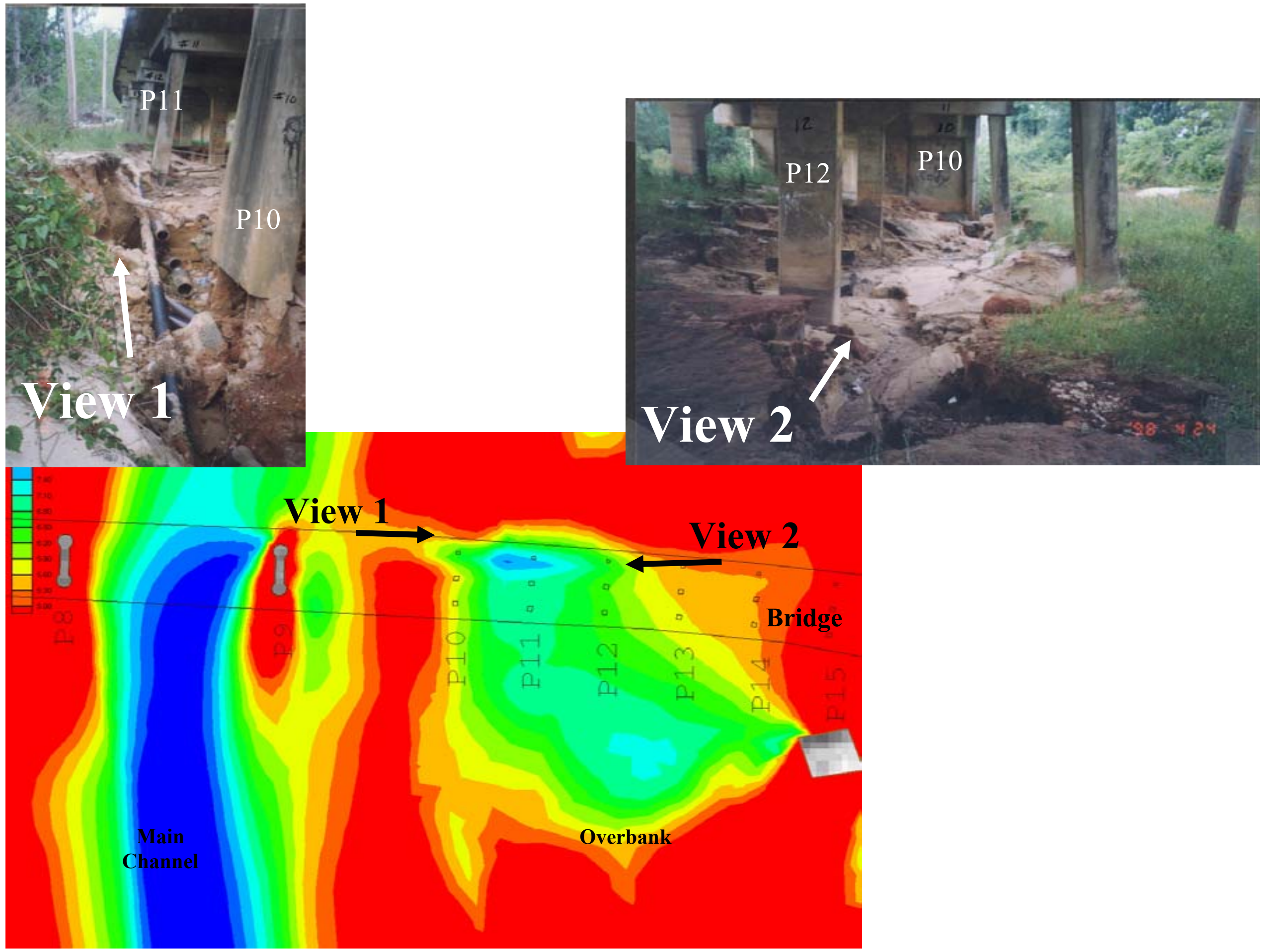


Figure 36. High velocity zones for the Pea River site at Elba and pictures after the 1998 flood event

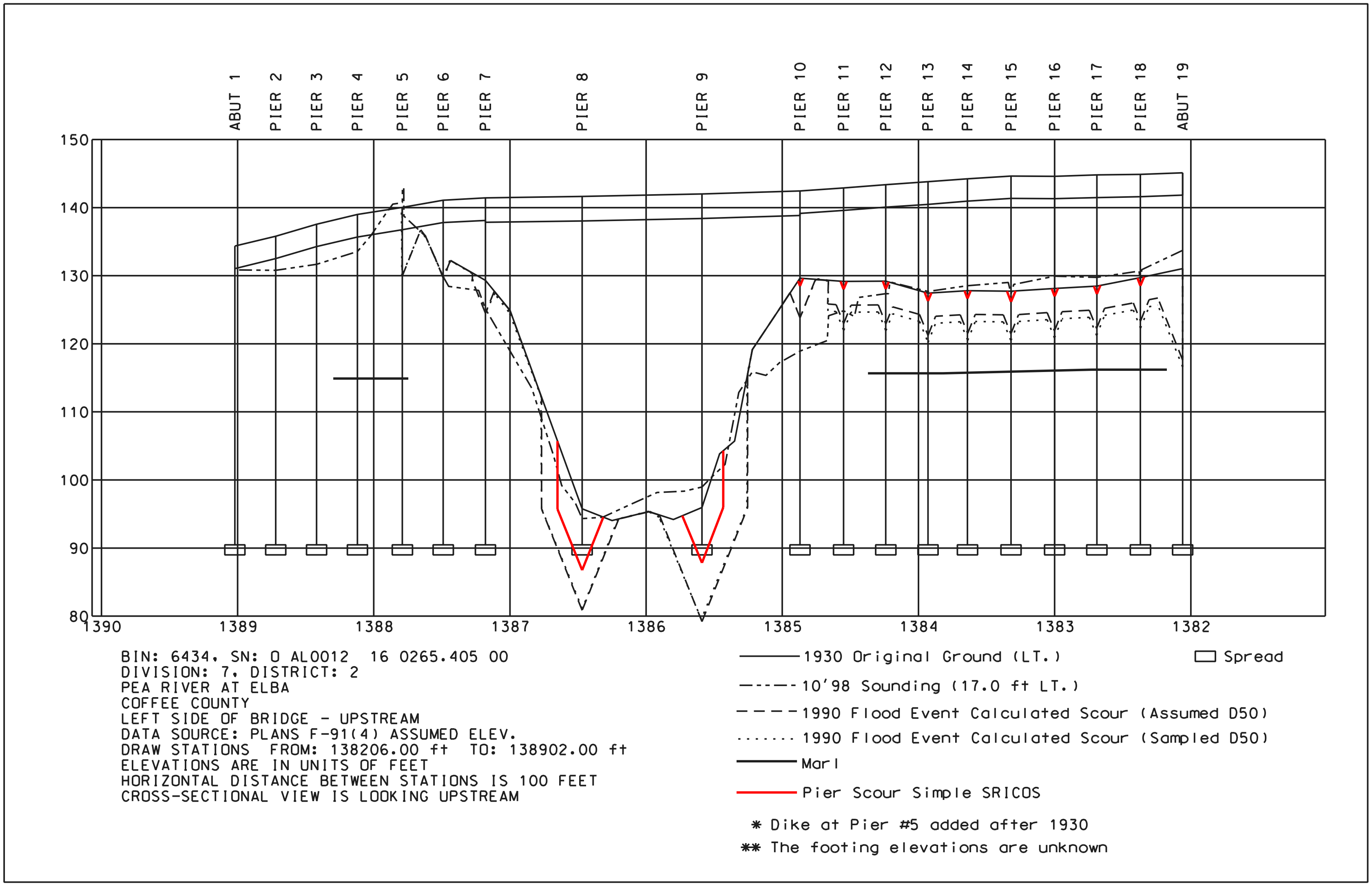


Figure 37. Scour calculations for Pea River at Elba

VIII. SCOUR EVALUATION AND RESULTS FOR CHOCTAWHATCHEE RIVER NEAR NEWTON

Several methods for computing scour were performed for this site. Daily stream flow data existed since construction. There was 6000 days of data from 1975 to 1990. After the 1990 flood riprap was placed in the overbanks. Calculations were performed for the filtered 2298 days of discharges from 1975 to 1990. The DASICOS method and Extended SRICOS methods were used to calculate contraction scour in the left overbank and in the main channel. Figures 38 and 39 show cumulative scour plots for the main channel and Figures 40 and 41 show cumulative scour plots for the left overbank. The plots show that DASICOS and E-SRICOS give similar results. The right overbank was comprised of sand, which was previously calculated using HEC-18 methods.

The Simple SRICOS method and Extended SRICOS method for pier scour were calculated for the two main channel piers and one overbank pier using an adaptation for square piers and with the NCHRP Report 24-15 method Briaud et al. (2003). The results of the Extended and Simple SRICOS pier scour calculations for the left overbank and the main channel using both the adaptation for square piers and the NCHRP Report 24-15 method are shown in Figures 42 and 43. The adaptation for square piers, as described earlier, gives similar results to the methods for complex piers described by Briaud et al. (2003), which involve shape factors. This gives us more confidence that the shape factors are reasonable. The added contraction and pier scour depths were plotted on the bridge profile and are shown in Figure 44.

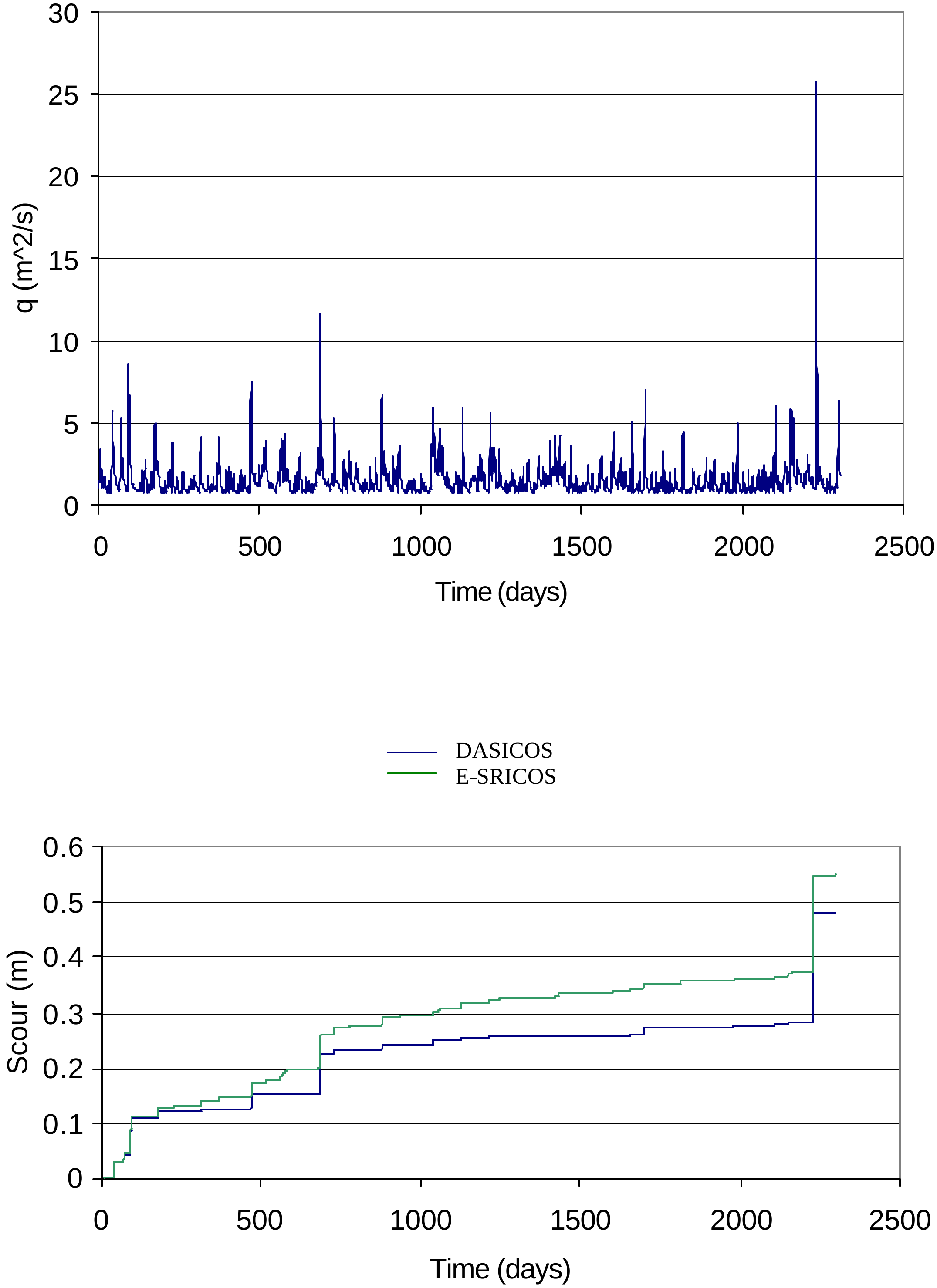
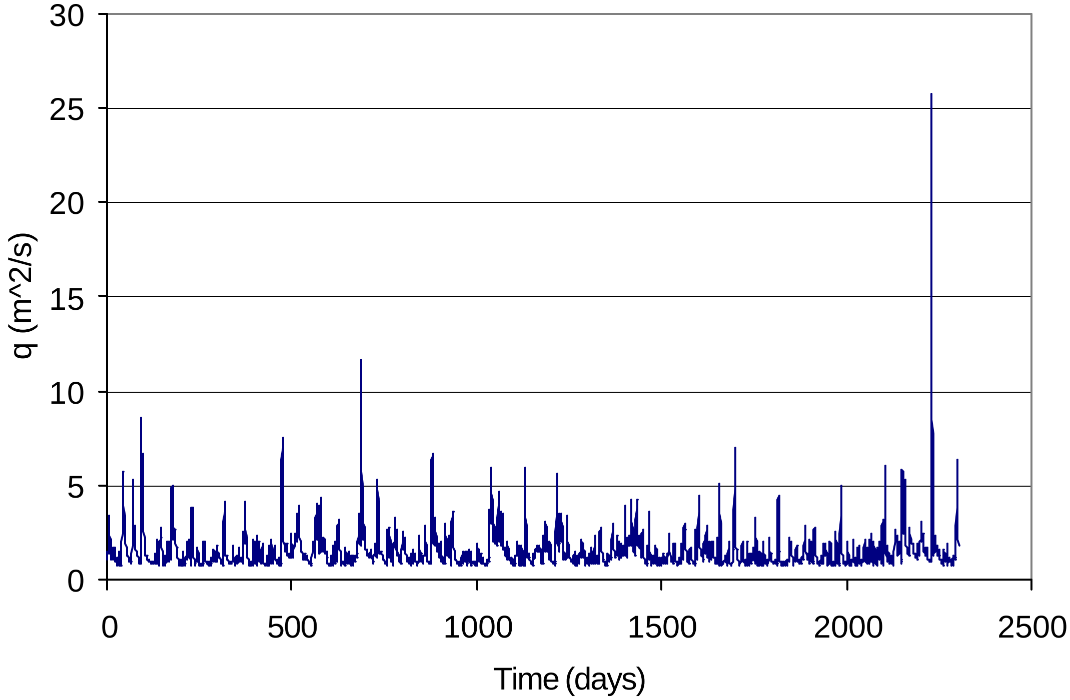


Figure 38. Choctawhatchee River near Newton contraction scour for the main channel.



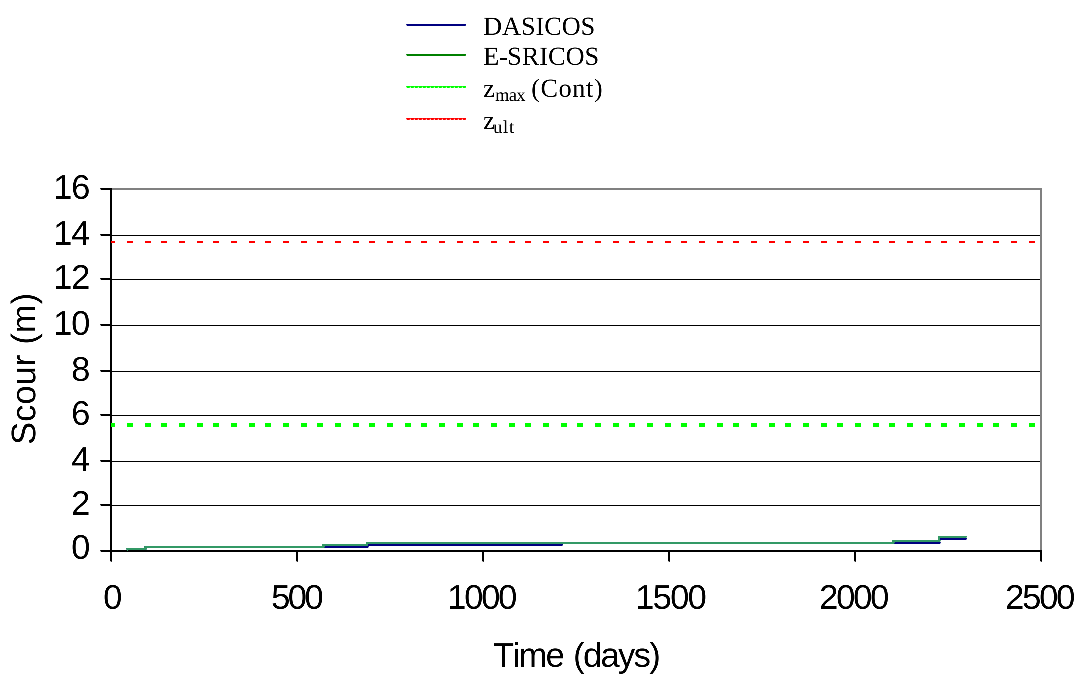


Figure 39. Choctawhatchee River near Newton contraction scour for the main channel with ultimate scour depths plotted.

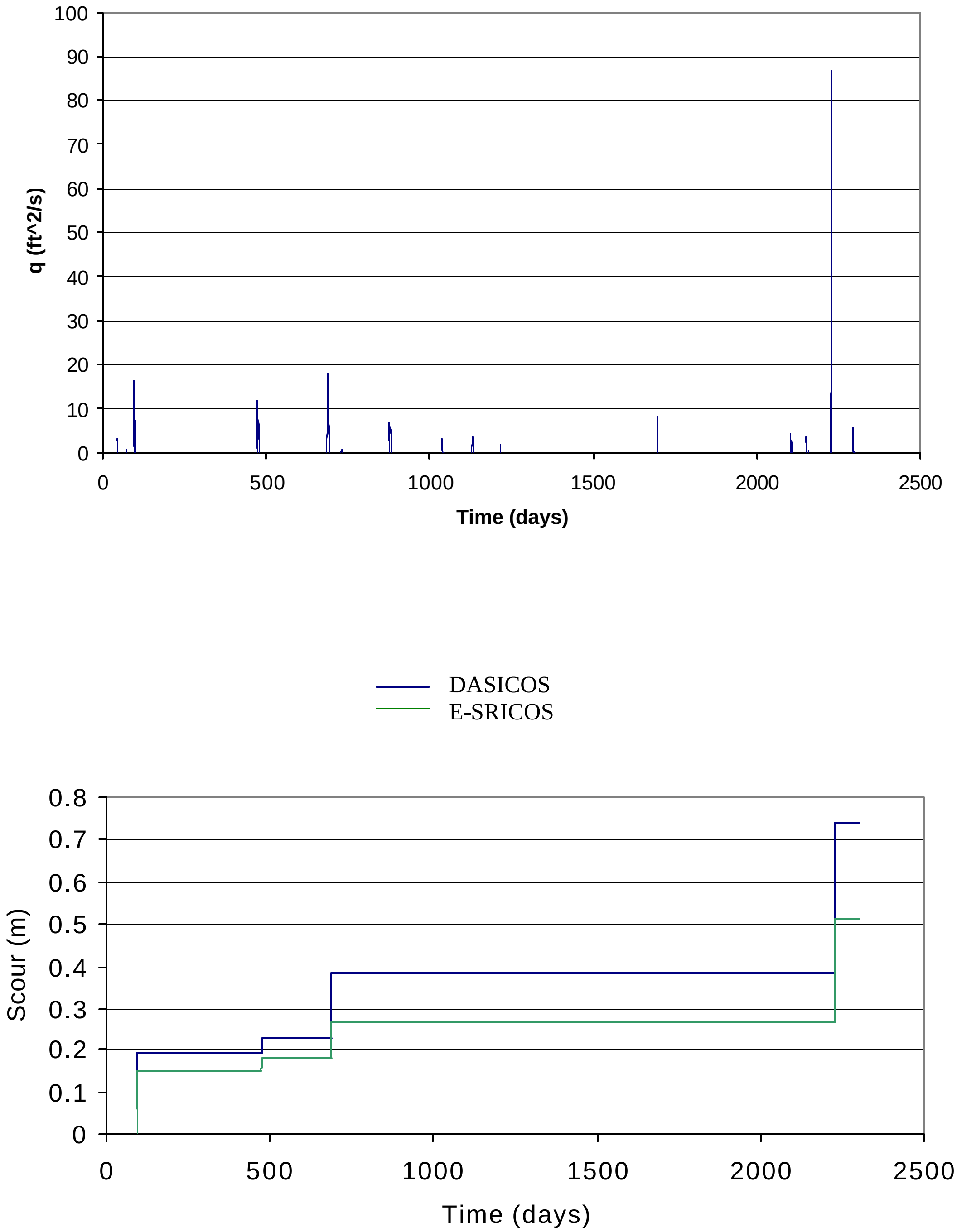


Figure 40. Choctawhatchee River near Newton contraction scour for the left overbank.

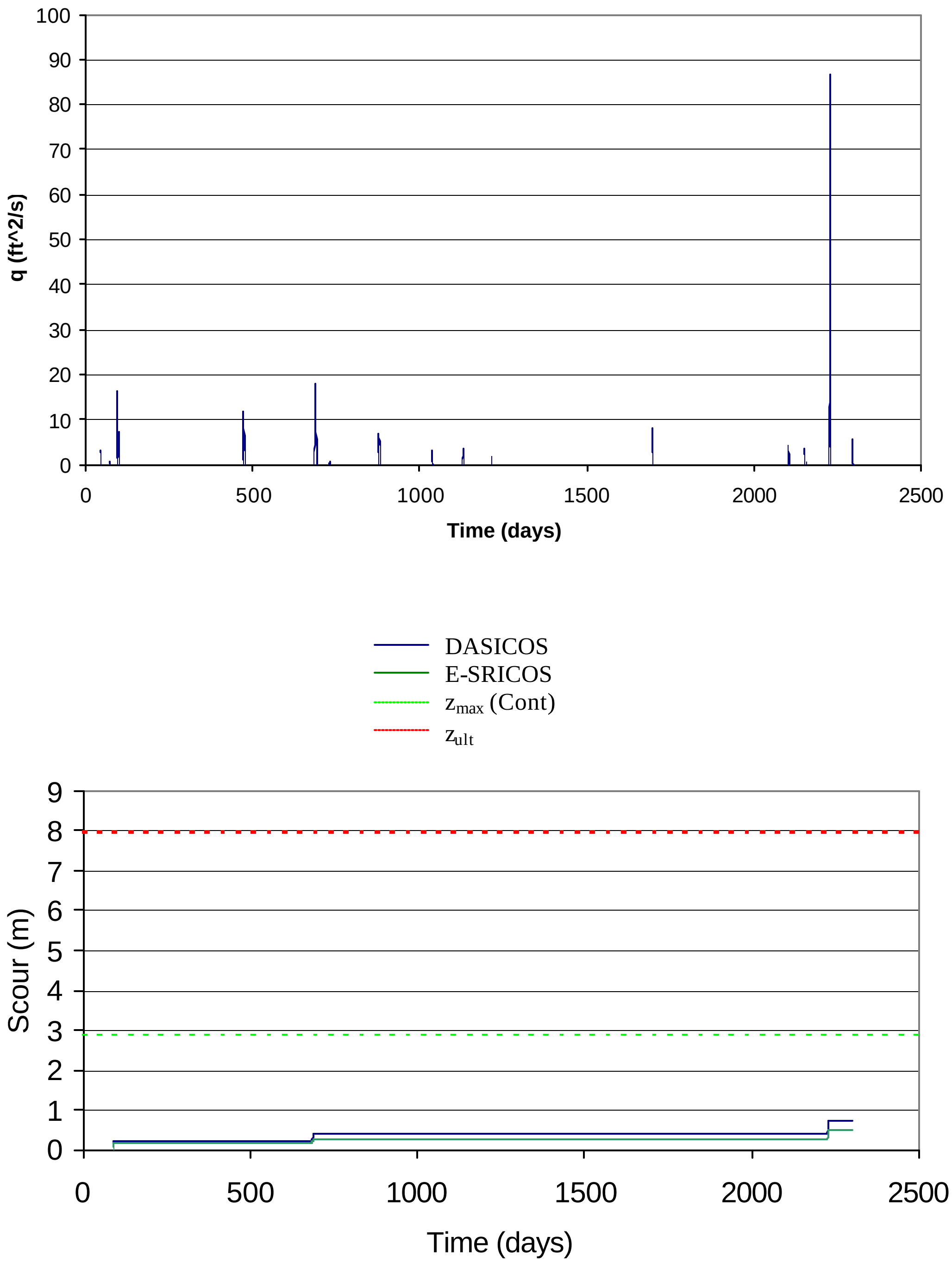


Figure 41. Choctawhatchee River near Newton contraction scour for the left overbank with ultimate scour depths plotted.

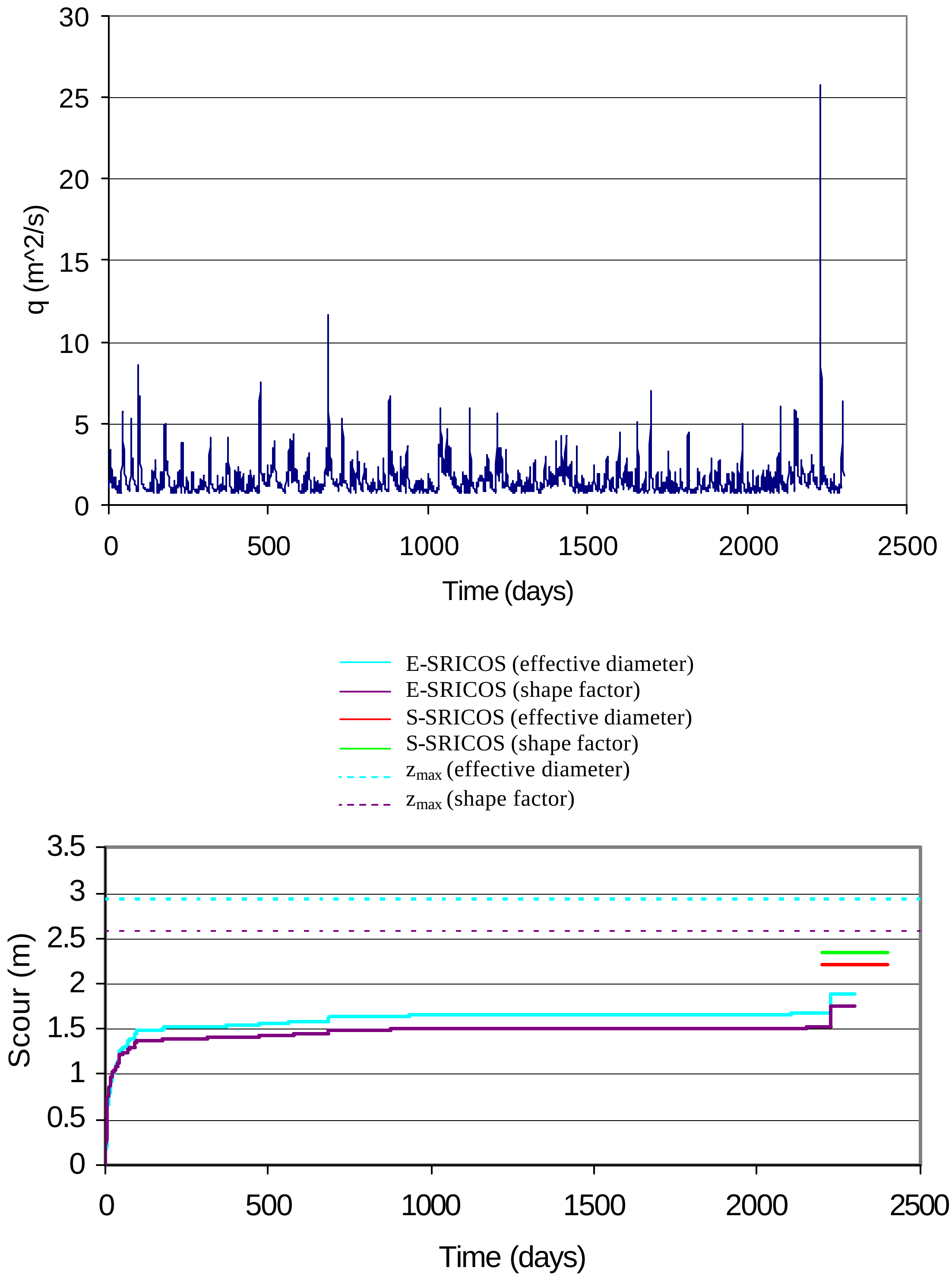


Figure 42. Choctawhatchee River near Newton pier scour for the main channel with ultimate scour depths plotted.

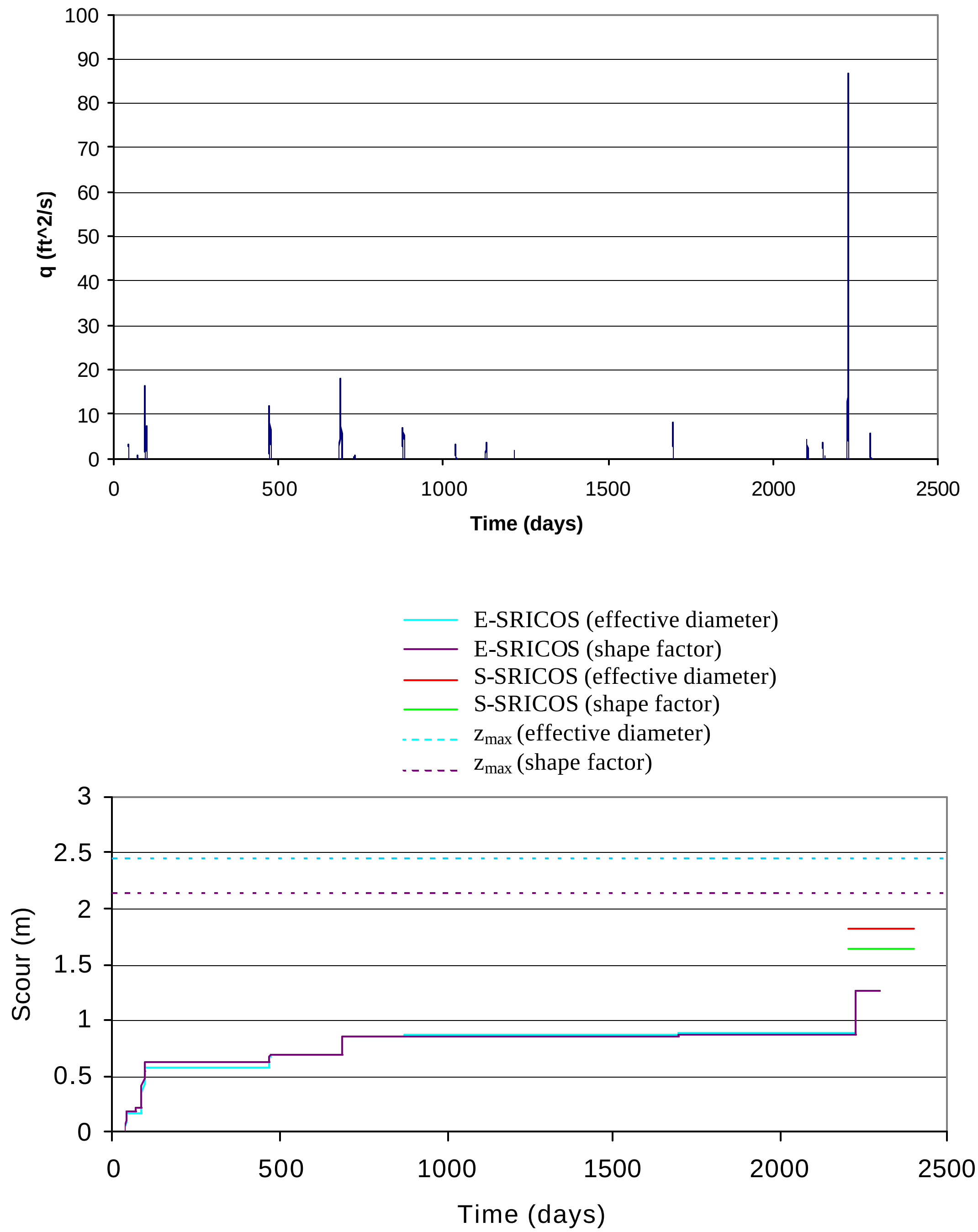


Figure 43. Choctawhatchee River near Newton pier scour for the left overbank with ultimate scour depths plotted.

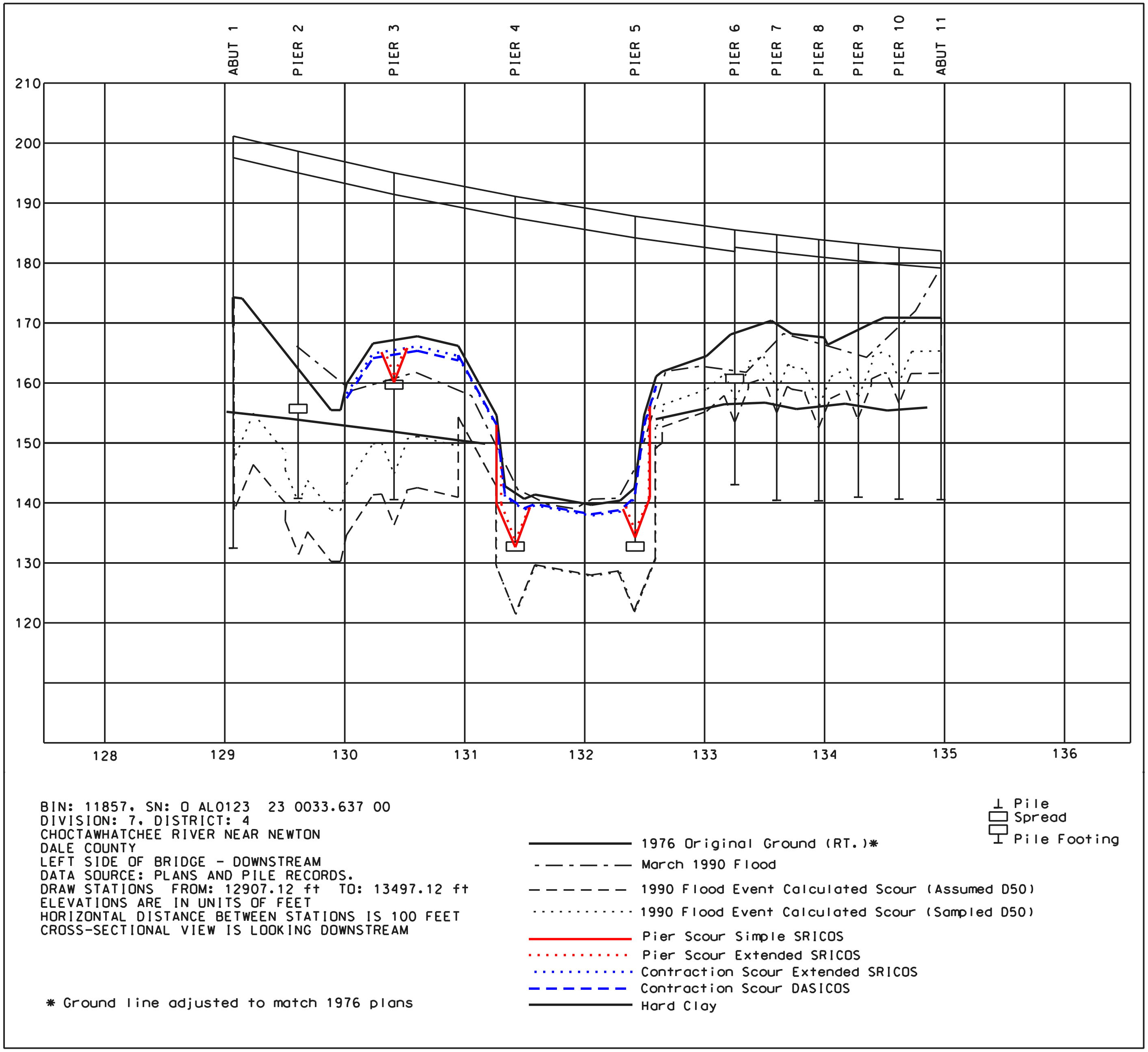


Figure 44. Scour calculations for Choctawhatchee River near Newton

IX. CONCLUSIONS

The use of the EFA data and computational methods based on the scour rate for cohesive soils improved the prediction results considerably. The calculated values of scour for cohesive soils were in better agreement with the observed values compared with the calculated values of scour using HEC-18 methods for noncohesive soils. This is especially true for the left and right overbank of the Pea River site and the left overbank and main channel of the Choctawhatchee River site. For the Pea River site the SRICOS method shows very little pier scour and no contraction scour in the left overbank in agreement with observations. Both the Simple SRICOS method and the HEC-18 method show considerable scour around the main channel piers while observations show no scour. The differences between the prediction and the observations in this case is most likely due to incomplete information about the soil characteristics of the main channel. A direct sample from the main channel could not be obtained, but a sample from a similar depth to the main channel bed elevation was obtained from the overbank.

There is still some uncertainty about these results presented here due to the 3-D nature of the actual flows and the variability of the soil properties at the sites. Additional work with multidimensional numerical models and comparisons with more field data and laboratory physical model experiments are needed.

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Appendix A

MATLAB Computer Programs

All input and output of the programs are in metric.

MATLAB PROGRAM FOR DASICOS CONTRACTION SCOUR

```
% DASICOS Method for contraction scour
% Program reads q(t)and y(t) stage hydrograph from Excel file
% Calculates transient scour (simple Euler method)

format compact
clear all

% Enter excel finle name and sheet name
H=xlsread('ChoctawLOB','Dataforrun');

% number of daily average flow measurements (q), dt = 1 day
% yh is the downstream stage (depth) reading from exel file
dt=1;
D=2300;
m=D/dt;
for j=1:m
   t(j)=H(j,1);
   yh(j)=H(j,2);
   q(j)=H(j,3);
end

subplot(3,1,1)
plot(t,q)
hold
grid
ylabel('q (m^2/sec)')
xlabel('Time (days)')

subplot(3,1,2)
plot(t,yh)
hold
grid
xlabel('Time (days)')
ylabel('Contraction Depth (m)')

%   Soil characteristics
Si= 0.2496; Tc= 0.46; rho= 1000; g= 9.81;
So= 0.04224; To= 0.75;
D=0.00032; Ks=D/2; v = 10^-6;
```

```
% z(j)= cumulative scour depth. No scour as an initial condition
z(1)=0;

for j=1:m
  if q(j)>0

    Y(j)=yh(j)+z(j);

  % Reynolds number
    Re= 4*q(j)/v;

  % Kr
    Kr= Ks/(4*Y(j));

  % use Swamee-Jain Equation to find intial friction factor
    fr(1) = 0.25/((log10((Kr/3.7)+(5.74/Re^0.9)))^2);

  % use Henderson Equation to find friction factor
    for k=1:5
      fr(k+1) = 0.25/((log10((Kr/3)+(2.5/(Re*fr(k)^.5))))^2);
      f(j)=fr(k+1);
    end

  % calculate the shear stress
    T(j)=(f(j)*rho*q(j).^2)/(8*Y(j)^2);

  % calculate the scour rate
    if T(j)>To
      R= Si*(To-Tc) + So*(T(j)-To);
      elseif T(j)>Tc
        R= Si*(T(j)-Tc);
      else
        R=0;
    end


  % Euler approximation of derivative
    z(j+1)=z(j)+R*dt;

  else
    z(j+1)=z(j);
  end

    t(j+1)=t(j)+dt;
end
```

```
%   write scour output to file that can be read with Excell
dlmwrite('LOB.out',z',' ')

[t;z]';
subplot(3,1,3)
plot(t,z)
hold
plot(0,0)
grid
xlabel('Time (days)')
ylabel('Scour Depth (m)')
refresh
```

MATLAB Program for E-SRICOS Contraction Scour

```
%   E-SRICOS Method to calculate contraction scour

format compact
clear all

% Enter Excel file name and sheet name
H=xlsread('ChoctawLOB','Dataforrun');

% number of daily average flow measurements (q), dt = 1 day
% yh is the downstream stage (depth) reading from Excel file
dt=1;
D=2300;
m=D/dt;
for J=1:m
    t(J)=H(J,1);
    yh(J)=H(J,2);
    q(J)=H(J,3);
end

subplot(3,1,1)
plot(t,q)
hold
grid
ylabel('q (m^2/sec)')
xlabel('Time (days)')

subplot(3,1,2)
plot(t,yh)
hold
grid
xlabel('Time (days)')
ylabel('Contraction Depth (m)')

%   Soil characteristics
Si= 0.2496; Tc= 0.46; rho= 1000; g= 9.81;
So= 0.04224; To= 0.75;
D=0.00032; Ks=D/2; v = 10^-6; g=9.81;
```

```
%   z(j)= cumulative scour depth. No scour as an initial condition
z(1)=0;

for J=1:m

    if q(J)>0

    %   calculate velocity
        V(J)=q(J)./yh(J);

    %   Reynolds number
        Re(J)= 4*q(J)/v;

    %   Kr
        Kr(J)= Ks/(4*yh(J));

    %   use Swamee-Jain Equation to find intial friction factor
        fr(1) = 0.25/((log10((Kr(J)/3.7)+(5.74/Re(J)^0.9)))^2);

    %   use Henderson Equation to find friction factor
        for k=1:5
            fr(k+1) = 0.25/((log10((Kr(J)/3)+(2.5/(Re(J)*fr(k)^.5))))^2);
            f(J)=fr(k+1);
        end

    %   calculate shear stress
        T(J)=(f(J)*rho*V(J).^2)/8;

    %   calculate the critical Froude number
        Frc(J)=((8*Tc)/(rho*g*f(J)*yh(J)))^0.5;

    %   calculate Froude number from velocity
        Fr(J)=V(J)/((g*yh(J))^0.5);

    else
        V(J)=0;
        Re(J)=0;
        T(J)=0;
        Frc(J)=0;
        Fr(J)=0;
    end
```

```
   %   calculate scour rate
      if T(J)>To
         R(J)= Si*(To-Tc) + So*(T(J)-To);
         else if T(J)>Tc
         R(J)=Si.*(T(J)-Tc);
            else
         R(J)=0;
         end
      end

   if R(J)>0
   %   calculate max scour depth for flow condition J
      zm(J)=yh(J)*1.9*((1.46*Fr(J))-Frc(J));

   %   calculate scour depth
      if z(J)>=zm(J)
         z(J+1)=z(J);
      else
         ts(J)=(z(J)/R(J))/(1-(z(J)/zm(J)));
         tss(J)=ts(J)+dt;
         z(J+1)=tss(J)/((1/R(J))+(tss(J)/zm(J)));
      end

   else
      z(J+1)=z(J);
   end

   t(J+1)=t(J)+dt;
end

%   write scour output to file that can be read in Excell
dlmwrite('LOBSRICOScontraction.out',z',' ')

[t;z]'
subplot(3,1,3)
plot(t,z)
grid on
xlabel('Time (days)')
ylabel('Scour Depth (m)')
refresh
```

MATLAB Program for E-SRICOS Pier Scour

```
%   E-SRICOS Method to calculate pier scour

format compact
clear all

% Enter Excel file name and sheet name
H=xlsread('ChoctawLOB','Dataforrun');

% number of daily average flow measurements (q), dt = 1 day
% yh is the downstream stage (depth) reading from Excel file
dt=1;
D=2300;
m=D/dt;
for J=1:m
    t(J)=H(J,1);
    yh(J)=H(J,2);
    q(J)=H(J,3);
end

subplot(3,1,1)
plot(t,q)
hold
grid
ylabel('q (m^2/sec)')
xlabel('Time (days)')

subplot(3,1,2)
plot(t,yh)
hold
grid
xlabel('Time (days)')
ylabel('Contraction Depth (m)')

%   Pier characteristics for square in meters
B= 1.1;
L=B;

%   calculate shape factor for T (shear stress)
ksh=1.15+(7*(exp(-4*(L/B))));

%   shape factor for zm (max scour)
Ksh=1.1;
```

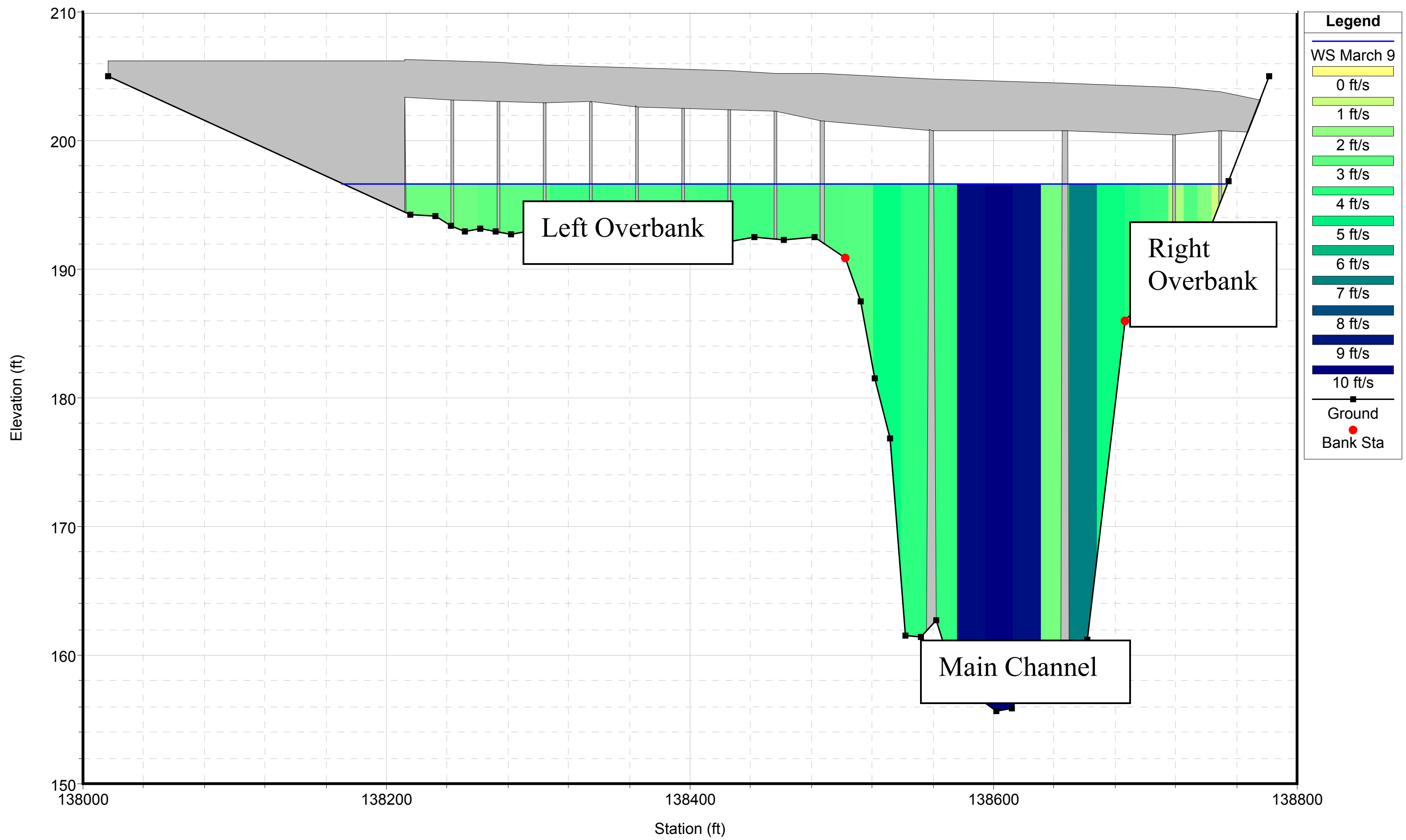
```
%   Soil characteristics
Si= 0.2496; Tc= 0.46; rho= 1000; g= 9.81;
So= 0.04224; To= 0.75;
D=0.00032; Ks=D/2; v = 10^-6;


%   z(j)= cumulative scour depth. No scour as an initial condition
z(1)=0;

for J=1:m

   if q(J)>0

   %   calculate velocity
      V(J)=q(J)./yh(J);

   %   Reynolds number
      Re(J)= B*V(J)/v;

   %   calculate shear stress
      T(J)=ksh*0.094*rho.*V(J).^2*((1/log10(Re(J)))-(1/10));

   else
      V(J)=0;
      Re(J)=0;
      T(J)=0;
   end


   %   calculate scour rate
      if T(J)>To
         R(J)= Si*(To-Tc) + So*(T(J)-To);
         else if T(J)>Tc
         R(J)=Si.*(T(J)-Tc);
            else
         R(J)=0;
         end
      end
```

```
  if R(J)>0
  %   calculate max scour depth for flow condition J
     zm(J)=(Ksh*0.18*(Re(J)^0.635))/1000;

  %   calculate scour depth
     if z(J)>=zm(J)
        z(J+1)=z(J);
     else
        ts(J)=(z(J)/R(J))/(1-(z(J)/zm(J)));
        tss(J)=ts(J)+dt;
        z(J+1)=tss(J)/((1/R(J))+(tss(J)/zm(J)));
     end

  else
     z(J+1)=z(J);
  end

  t(J+1)=t(J)+dt;
end

% write scour output to file that can be read with Excel
dlmwrite('LOBpierk.out',z',' ')

[t;z]'
subplot(3,1,3)
plot(t,z)
grid on
xlabel('Time (days)')
ylabel('Scour Depth (m)')
refresh
```

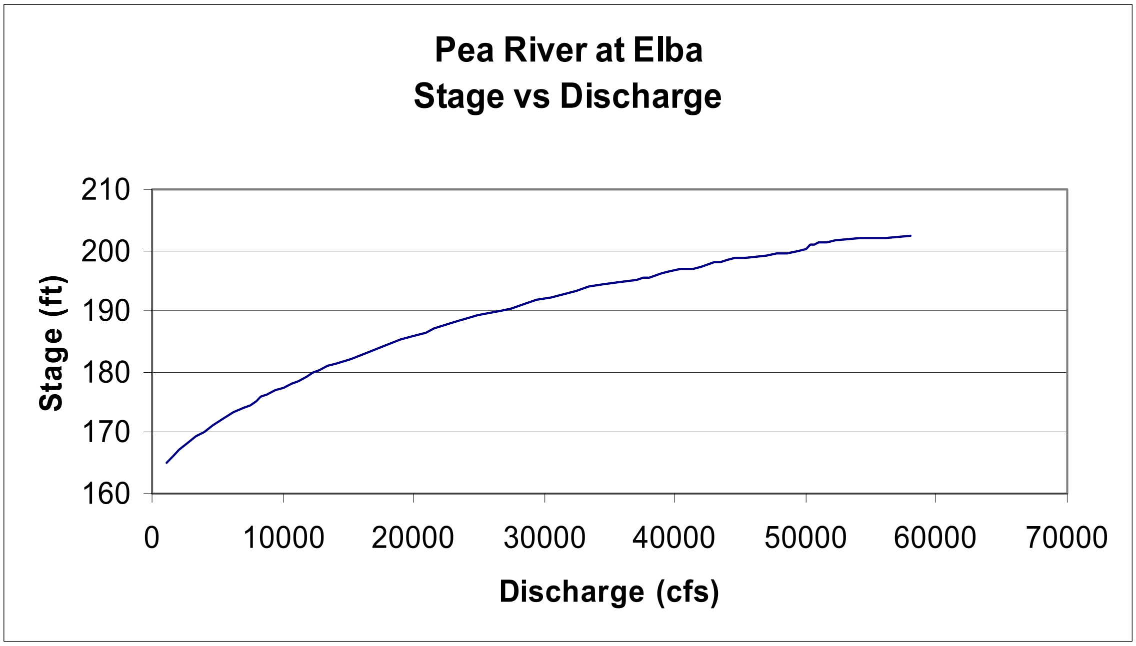
Appendix B

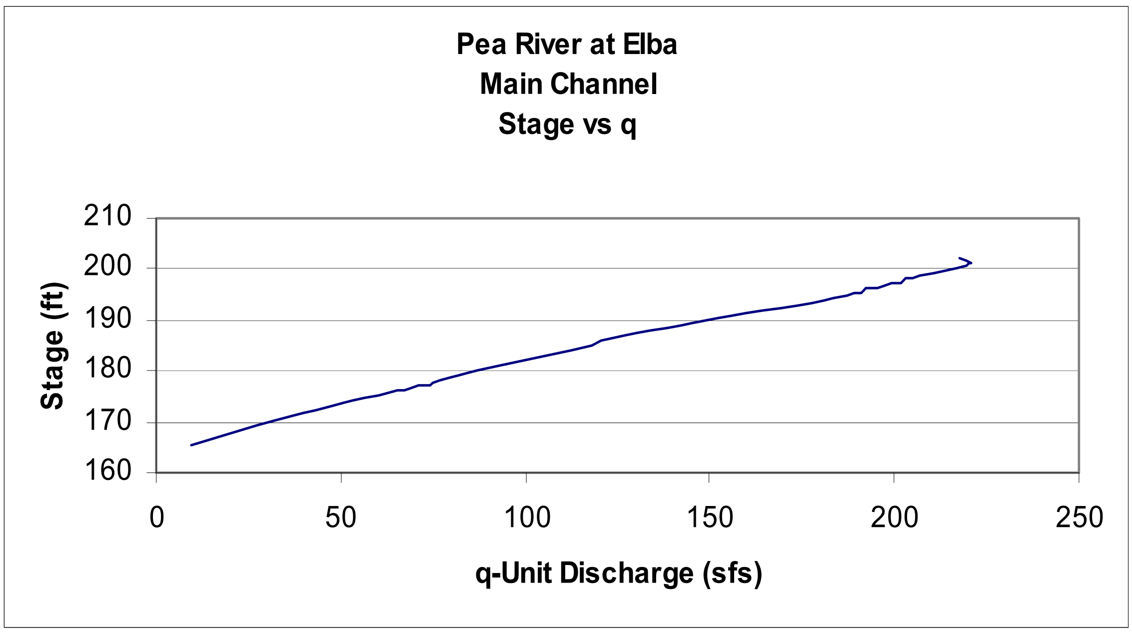
Stream Flow Rating Functions

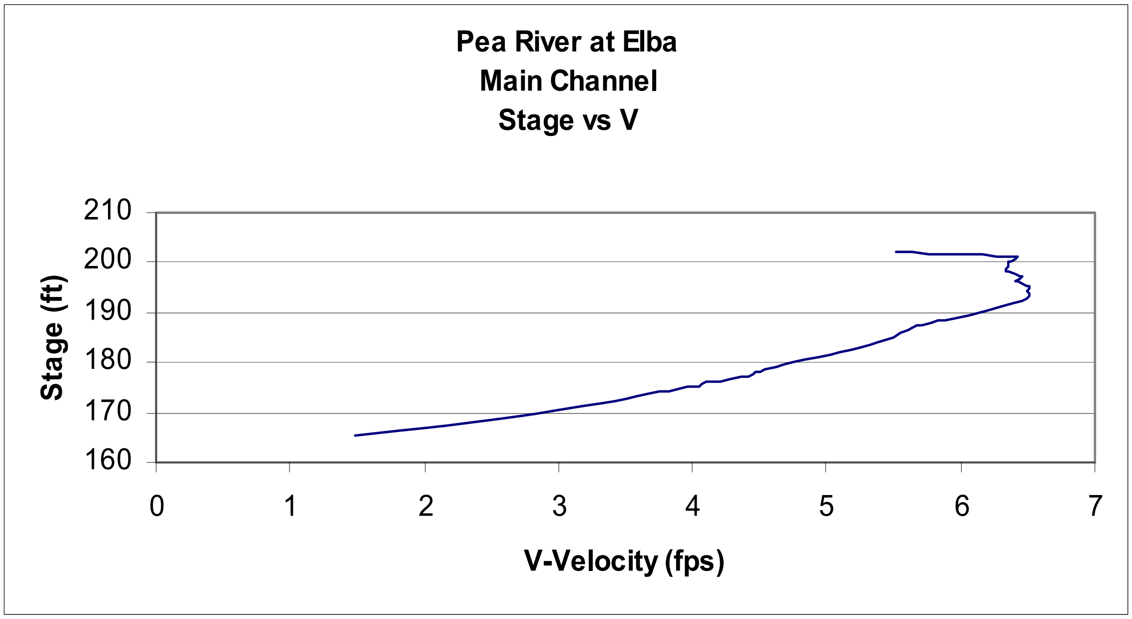
The cross-sections were broken up into a left overbank, a right overbank, and a main channel due to the change in geometry across the sections. Incremental discharges (Q) and stages were taken from the rating curve for determining flow distributions for the overbanks and main channel. Top widths were taken for the overbanks and main channel from the model to determine unit discharges, q (discharge/TopWidth). Hydraulic depths, Yh (Area/top width) were taken from the model as well. Using discharge vs q and discharge vs Yh, plots were developed of stage vs q, stage vs Yh, and stage vs V for the overbanks and main channels. The anomalies in the curves come from changes in flow distributions due to stages rising above the overbanks and coming in contact with the underside of the bridges.

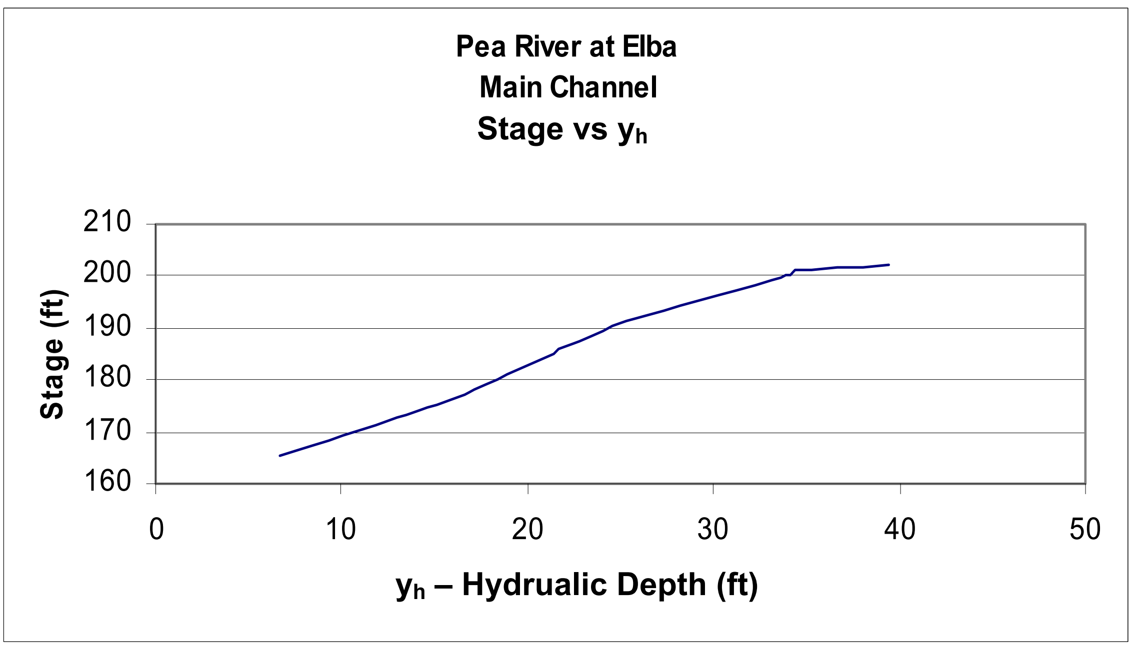


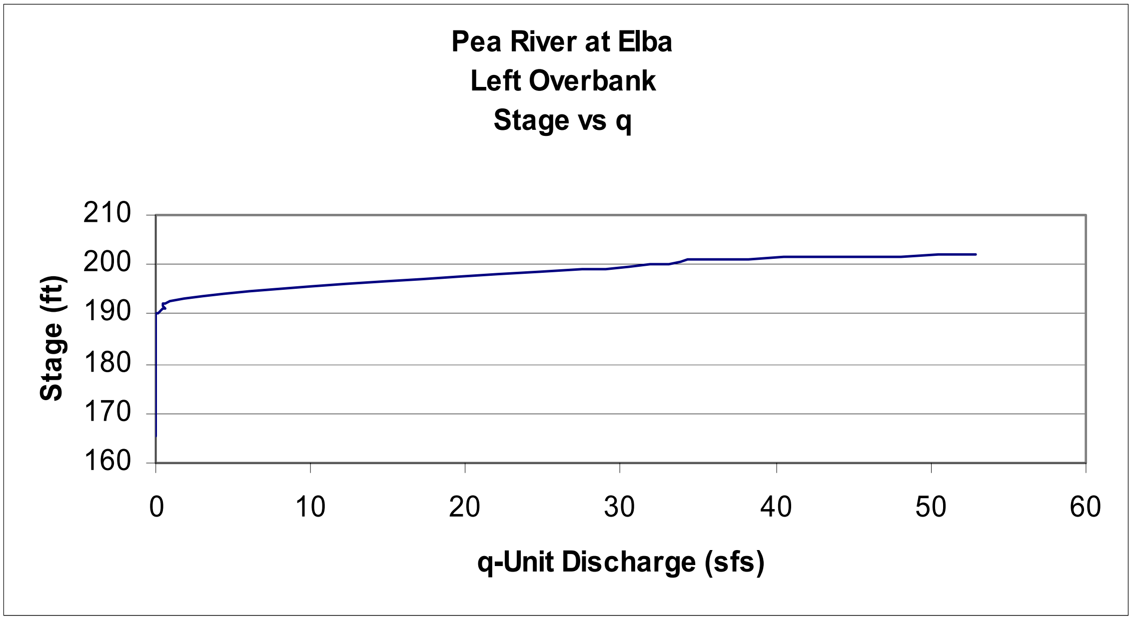
Cross-section and velocity distribution for one discharge and stage for Pea River at Elba

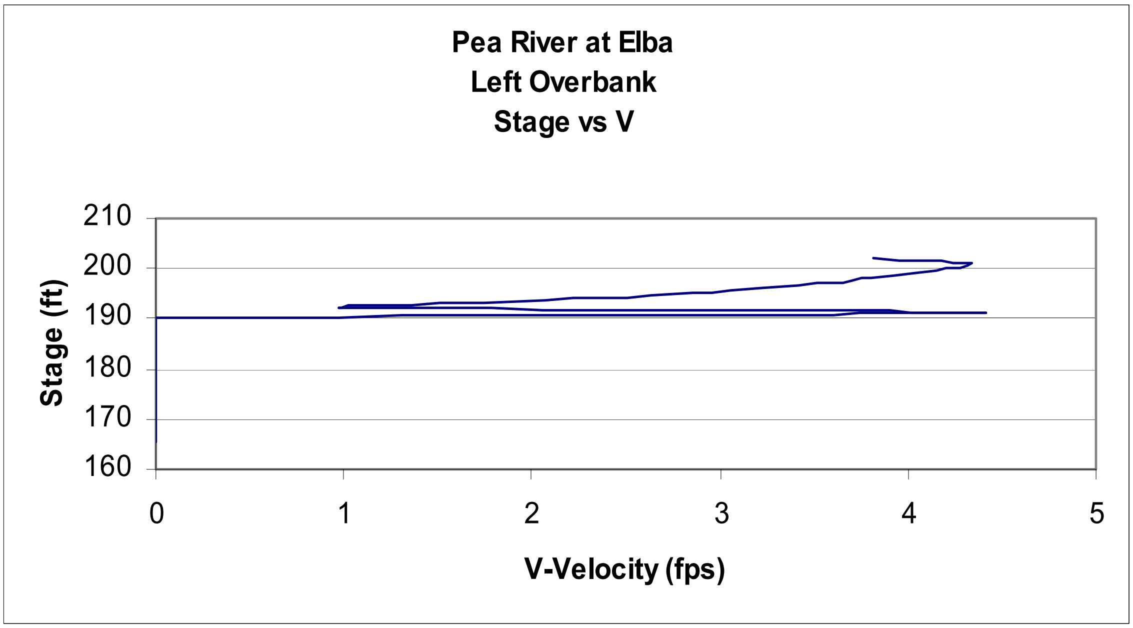

Pea River at Elba
Stage vs Discharge
Stage (ft)
210
200
190
180
170
160
0
10000
20000
30000
40000
50000
60000
70000
Discharge (cfs)

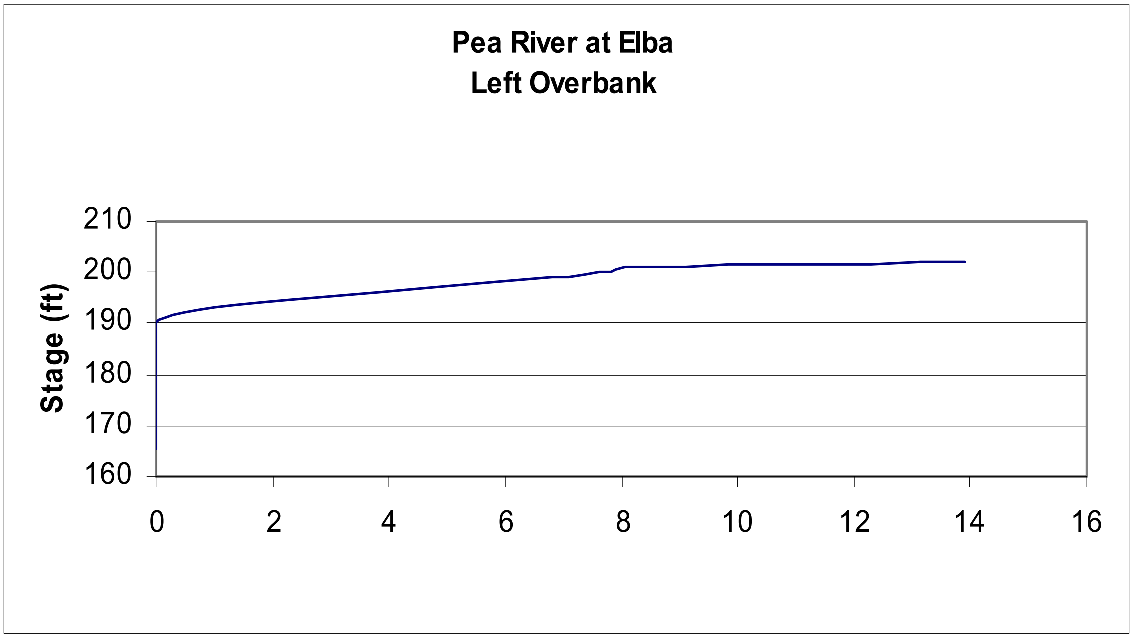

Pea River at Elba
Main Channel
Stage vs q
Stage (ft)
210
200
190
180
170
160
0
50
100
150
200
250
q-Unit Discharge (sfs)

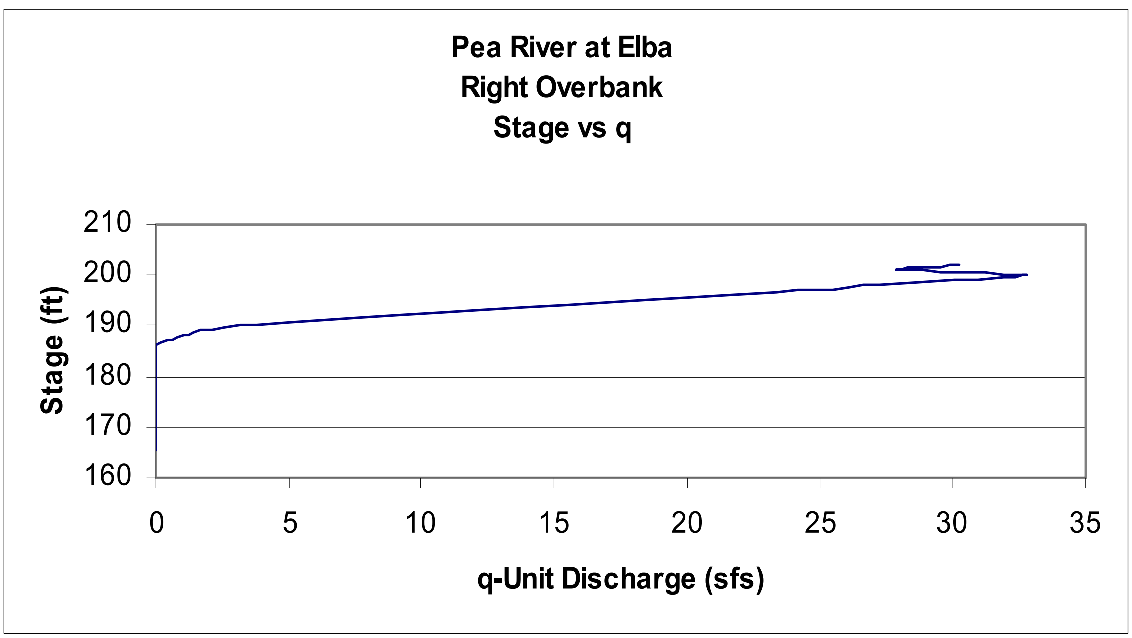

Pea River at Elba
Main Channel
Stage vs V
210
200
190
180
170
160
Stage (ft)
0
1
2
3
4
5
6
7
V-Velocity (fps)

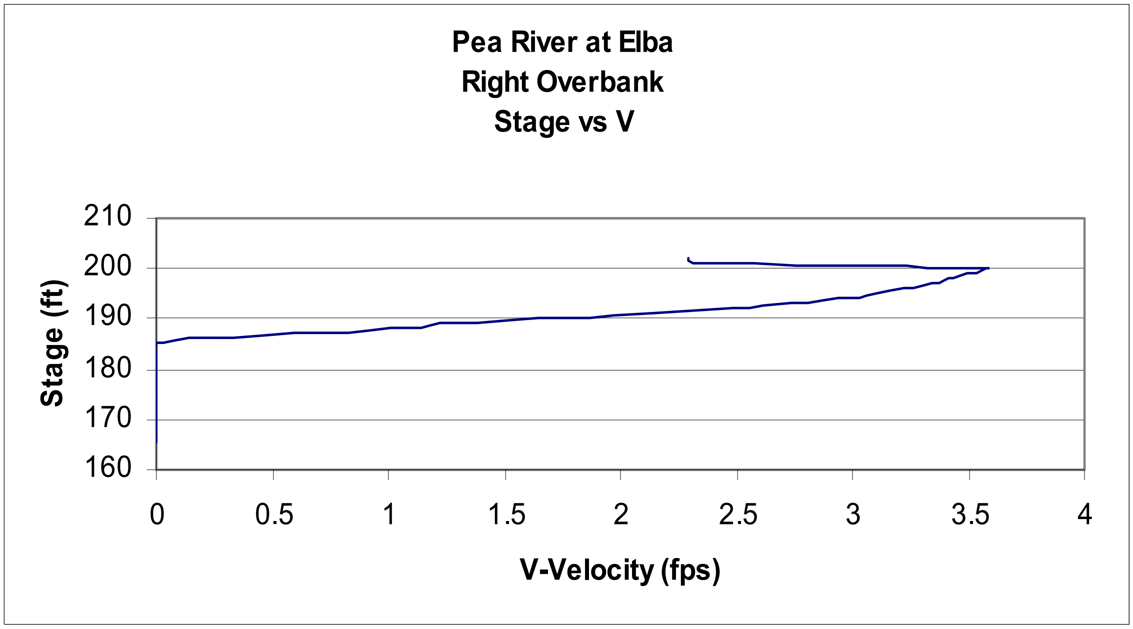

Pea River at Elba
Main Channel
Stage vs y_h
210
200
190
180
170
160
Stage (ft)
0
10
20
30
40
50
y_h – Hydrualic Depth (ft)

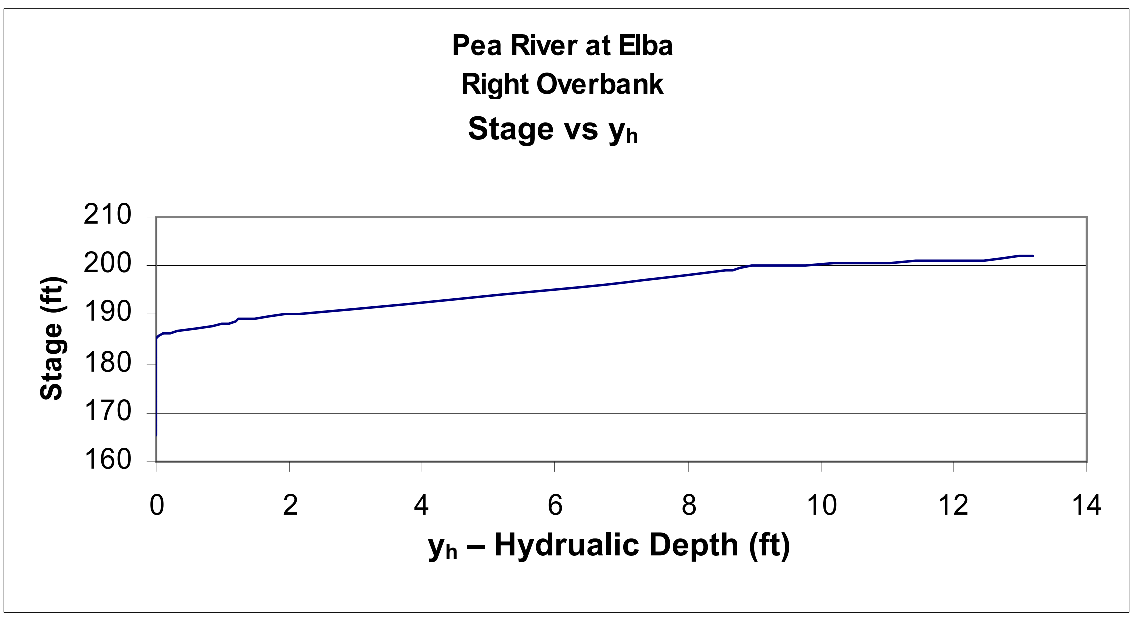

Pea River at Elba
Left Overbank
Stage vs q
Stage (ft)
210
200
190
180
170
160
0
10
20
30
40
50
60
q-Unit Discharge (sfs)

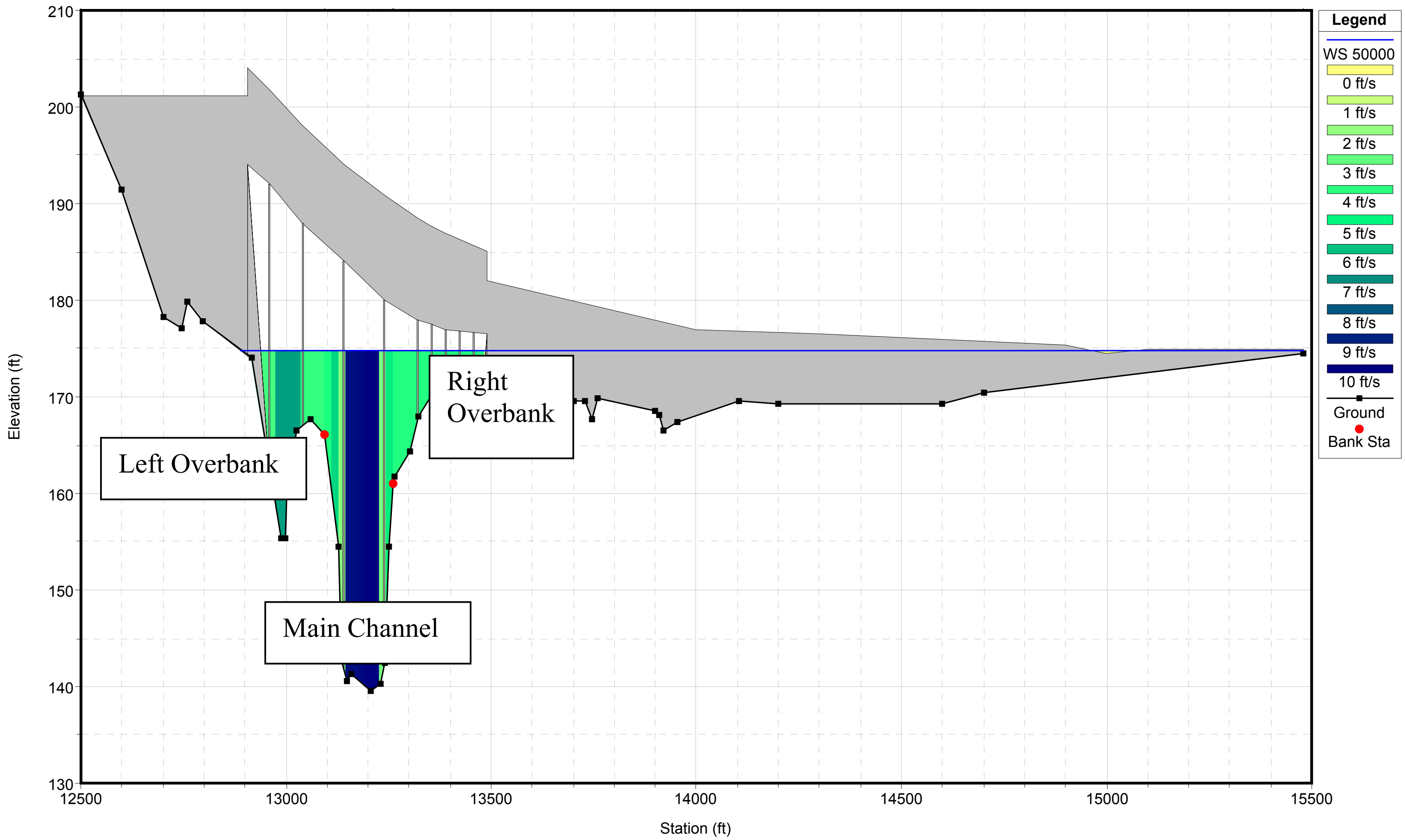

Pea River at Elba
Left Overbank
Stage vs V
Stage (ft)
210
200
190
180
170
160
0
1
2
3
4
5
V-Velocity (fps)


Pea River at Elba
Left Overbank
Stage (ft)
210
200
190
180
170
160
0
2
4
6
8
10
12
14
16

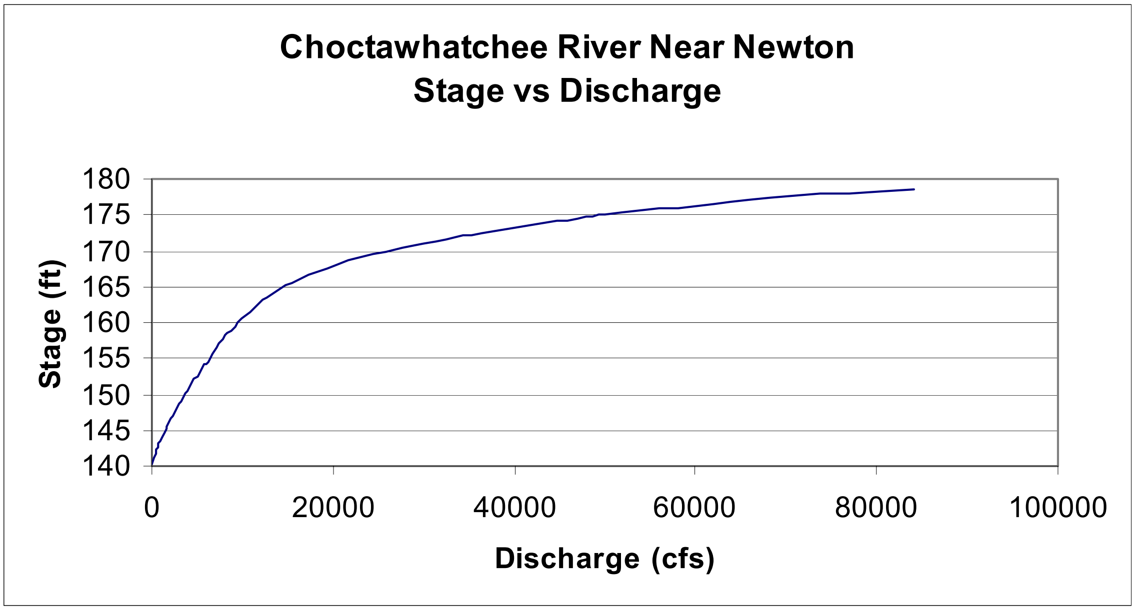

Pea River at Elba
Right Overbank
Stage vs q
Stage (ft)
210
200
190
180
170
160
0
5
10
15
20
25
30
35
q-Unit Discharge (sfs)

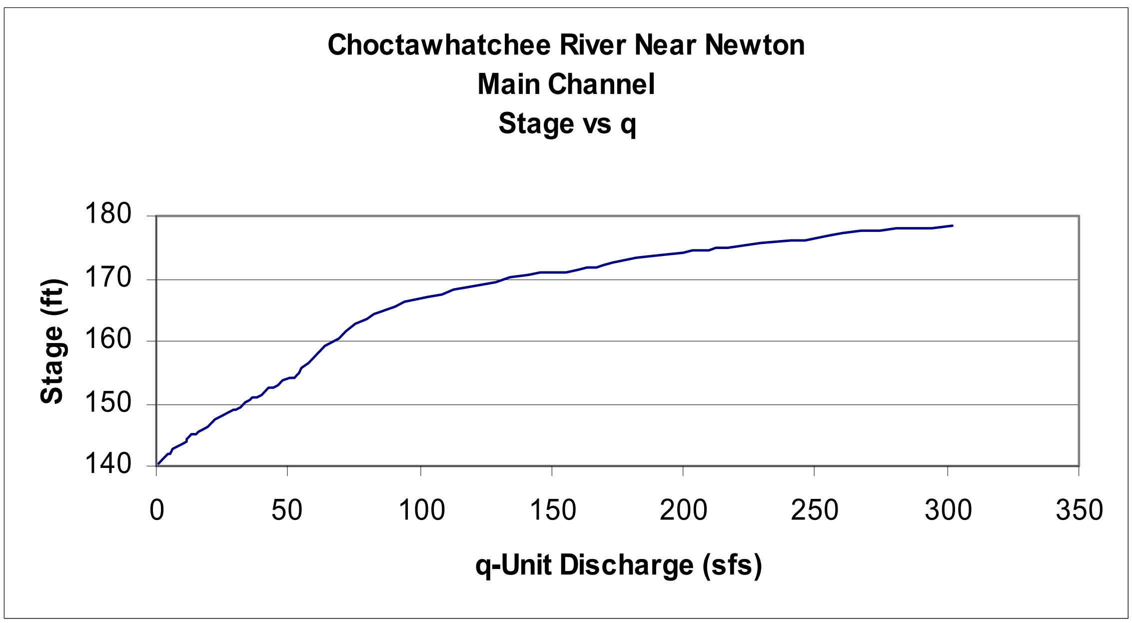

Pea River at Elba
Right Overbank
Stage vs V
Stage (ft)
210
200
190
180
170
160
0
0.5
1
1.5
2
2.5
3
3.5
4
V-Velocity (fps)

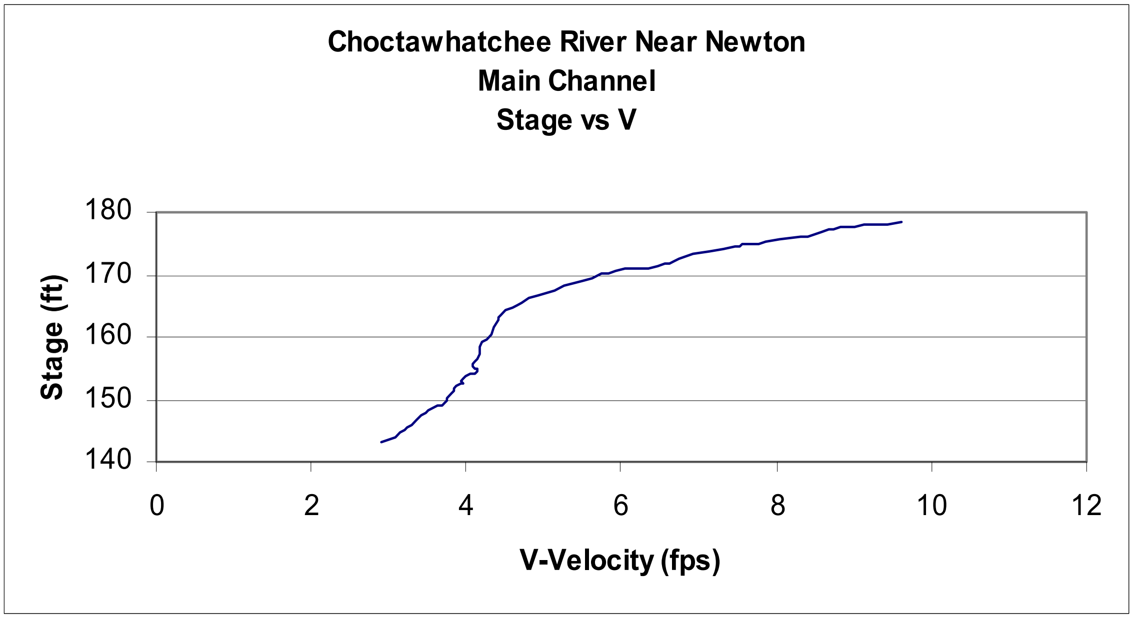

Pea River at Elba
Right Overbank
Stage vs y_h
Stage (ft)
210
200
190
180
170
160
0
2
4
6
8
10
12
14
y_h – Hydrualic Depth (ft)

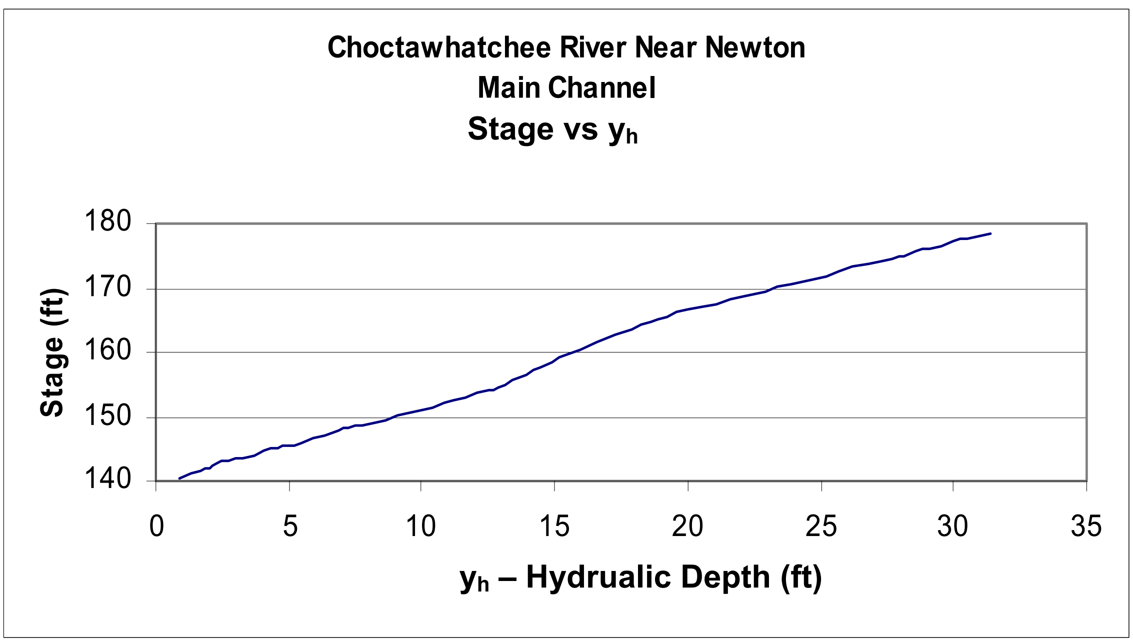


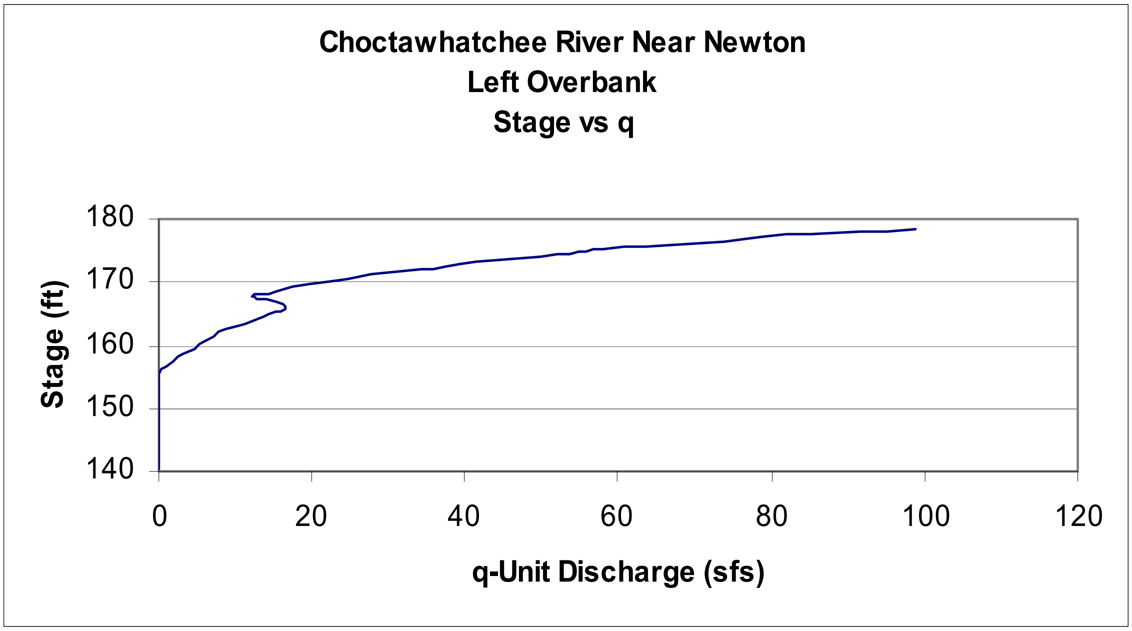
Cross-section and velocity distribution for one discharge and stage for Choctawhatchee River near Newton

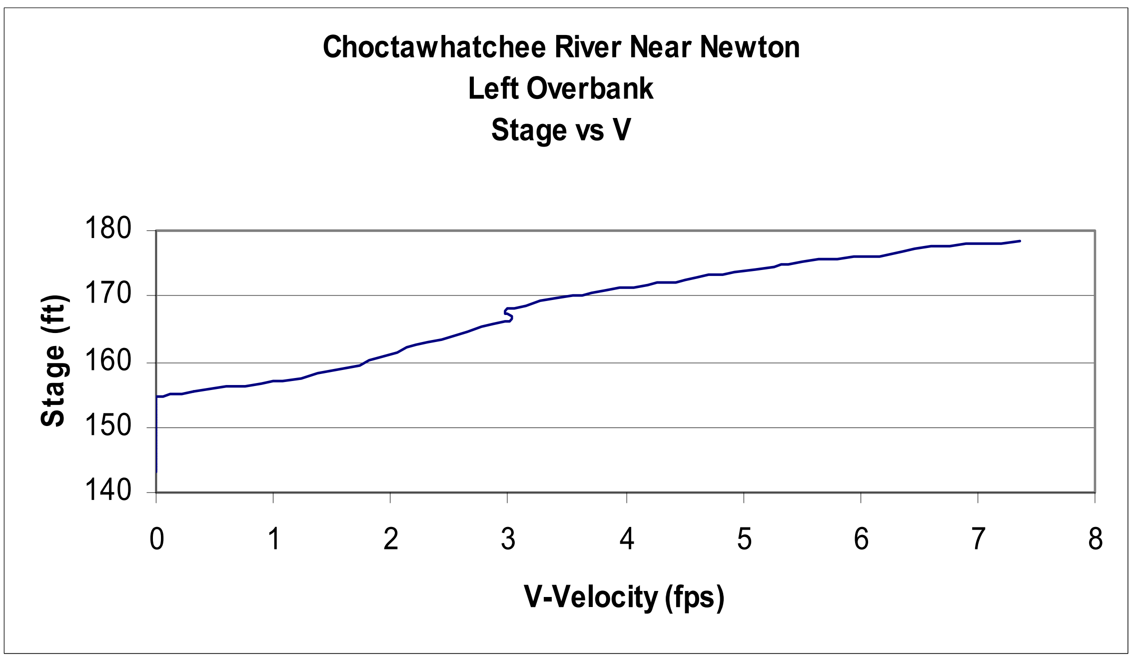

Choctawhatchee River Near Newton
Stage vs Discharge
Stage (ft)
180
175
170
165
160
155
150
145
140
0
20000
40000
60000
80000
100000
Discharge (cfs)

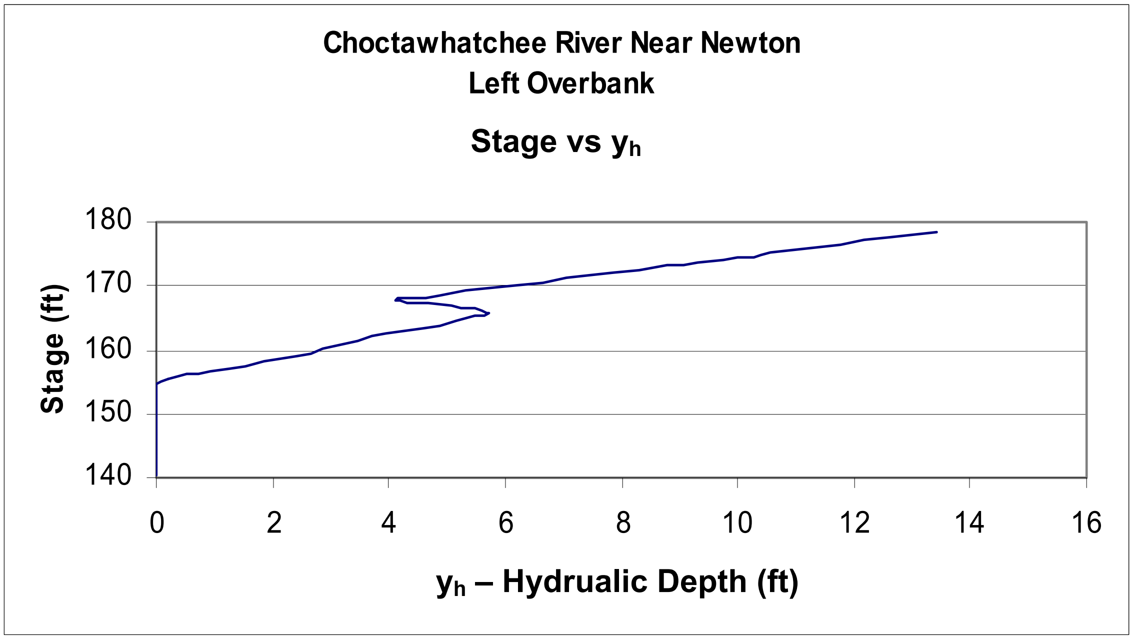

Choctawhatchee River Near Newton
Main Channel
Stage vs q
Stage (ft)
180
170
160
150
140
0
50
100
150
200
250
300
350
q-Unit Discharge (sfs)


Choctawhatchee River Near Newton
Main Channel
Stage vs V
Stage (ft)
180
170
160
150
140
0
2
4
6
8
10
12
V-Velocity (fps)


Choctawhatchee River Near Newton
Main Channel
Stage vs yh
Stage (ft)
180
170
160
150
140
0
5
10
15
20
25
30
35
yh – Hydrualic Depth (ft)


Choctawhatchee River Near Newton
Left Overbank
Stage vs q
Stage (ft)
180
170
160
150
140
0
20
40
60
80
100
120
q-Unit Discharge (sfs)

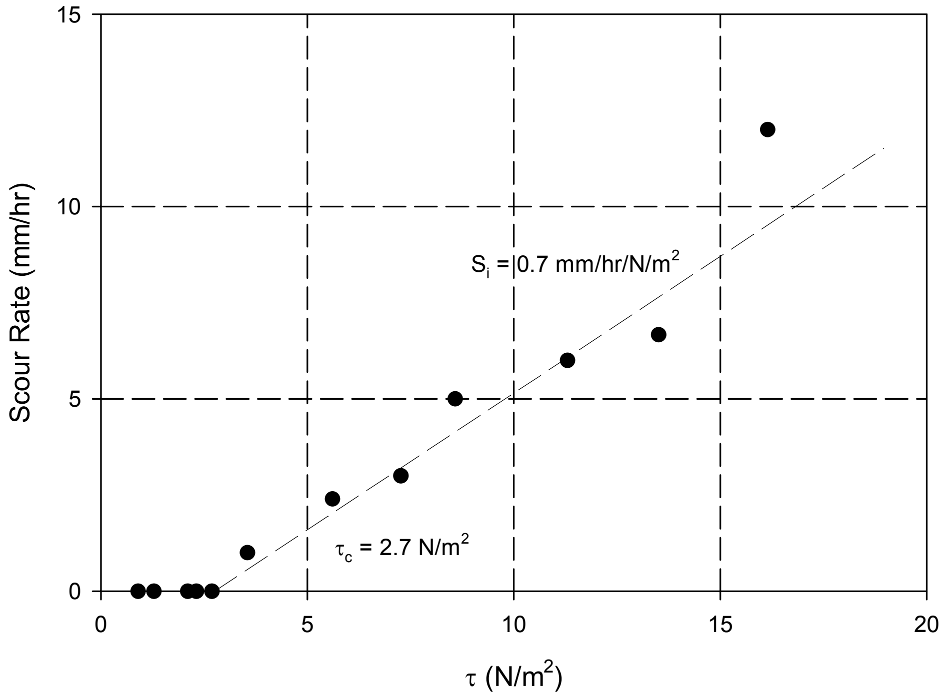

Choctawhatchee River Near Newton
Left Overbank
Stage vs V
Stage (ft)
180
170
160
150
140
0
1
2
3
4
5
6
7
8
V-Velocity (fps)



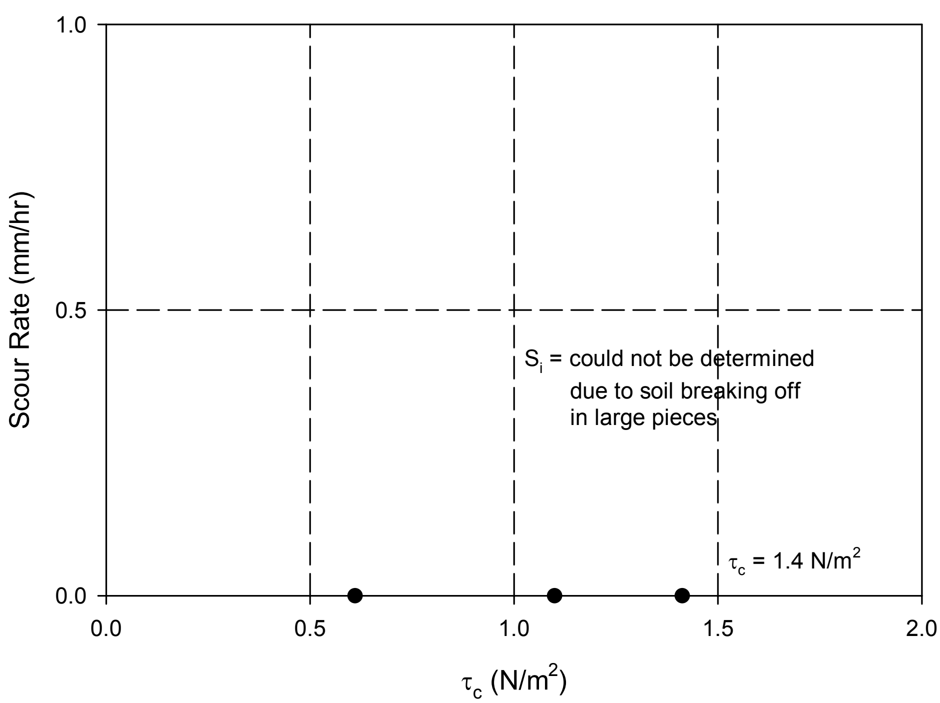
Choctawhatchee River Near Newton
Left Overbank
Stage vs yh
180
170
160
150
140
Stage (ft)
0
2
4
6
8
10
12
14
16
yh – Hydrualic Depth (ft)

Appendix C

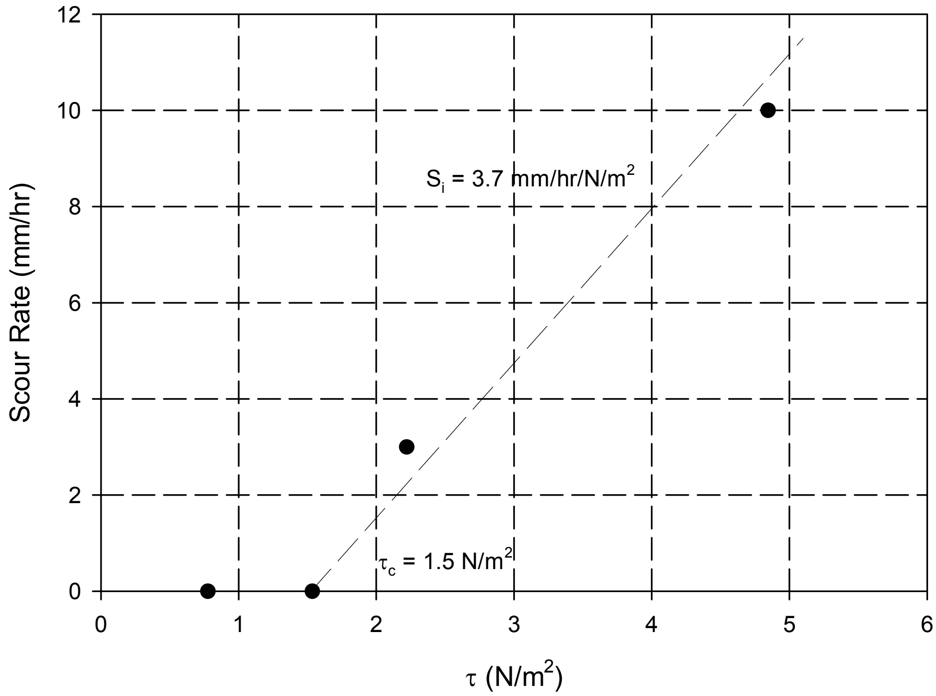
Erosion Functions



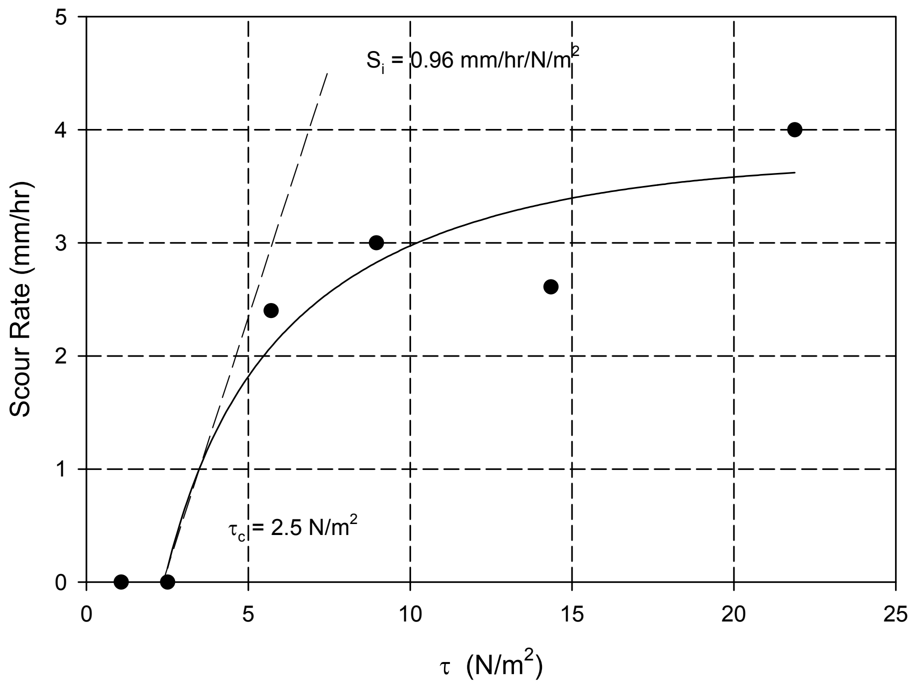
Pea River at Elba Sample 2A



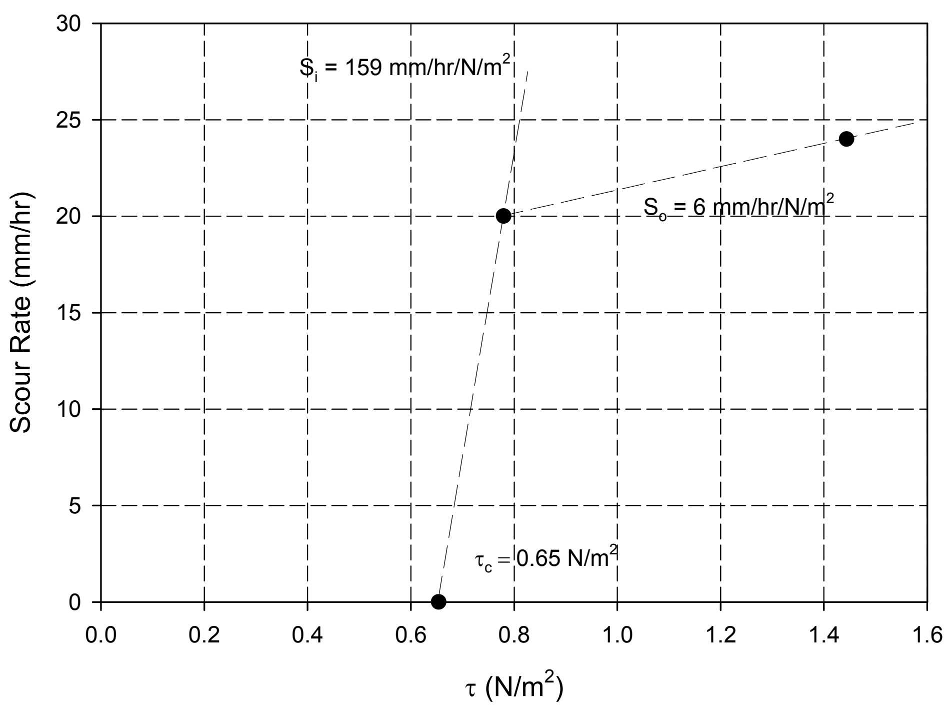
Pea River at Elba Sample 2B



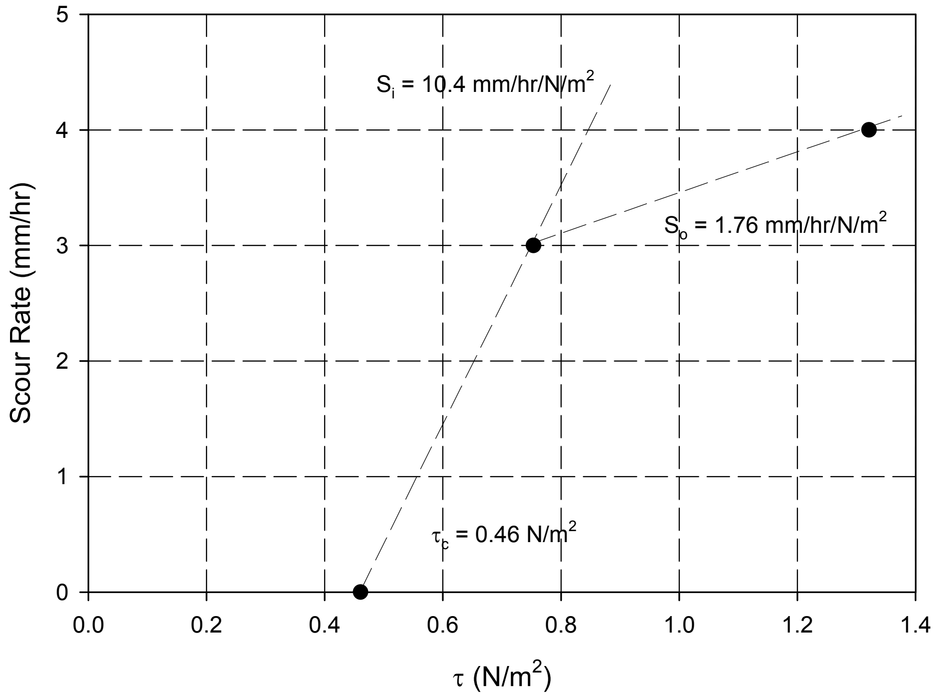
Pea River at Elba Sample 3A



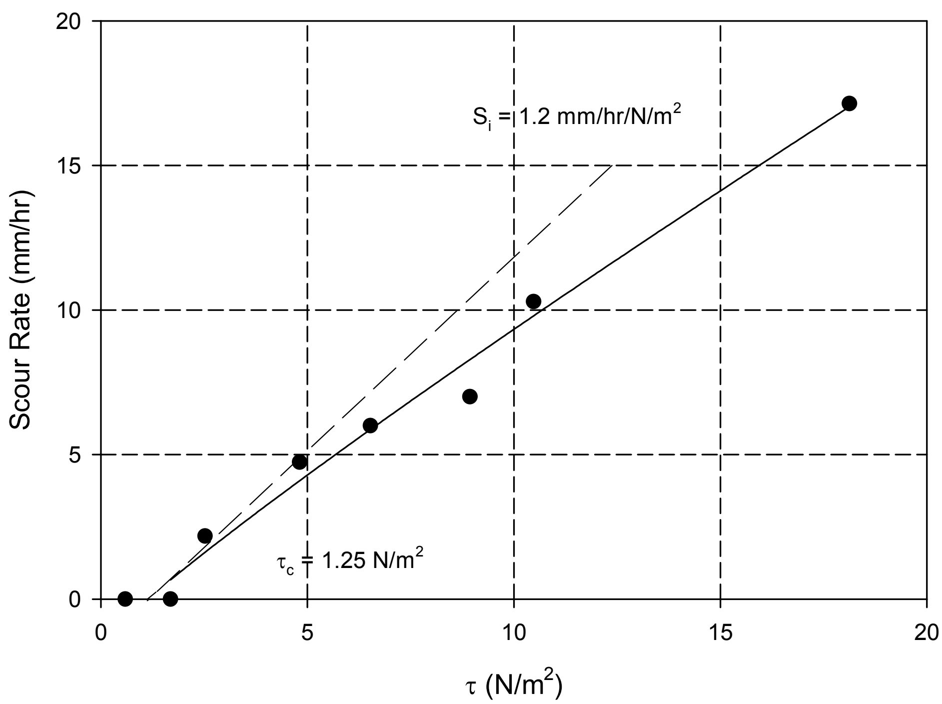
Choctawhatchee River near Newton Sample 1 A



Choctawhatchee River near Newton Sample 3B



Choctawhatchee River Near Newton Sample 4B



Choctawhatchee River Near Newton Sample 4C