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SCOUR EVALUATIONS OF SELECTED BRIDGES IN ALABAMA

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ABSTRACT

This report presents the results of a study of foundation scour at 18 bridge sites in Alabama which have experienced large floods with recurrence intervals of 100 years or larger. These sites are located throughout the state. Seven of the sites are in the north, two are located in the central east, three are located in the central west, and the last six are located in the south east. These locations provided a variety of soil types from sand in the south to marl in the central west to rock foundations in the north. The purpose of the study was to perform a comparative evaluation of actual pier foundation depths and calculated scour depths during actual past floods based on available scour formulas. The results of the report show that the maximum scour depths computed by using the available scour equations are reasonable for foundations consisting of sand, but the formulas, which have been developed for noncohesive foundation materials, are not suitable for determining the scour depths for foundations with cohesive soils.

ACKNOWLEDGMENTS

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I. Introduction

Background:

In a recent review of Alabama highway research needs, one of the topics identified was titled *Evaluation of the Effects of Scour Models on Required Pile Lengths* (HRC, 1998). It was concluded in that review that "more research needs to be conducted on foundation requirements," and that "current scour models require minimum pile tip elevations that are often beyond reasonably obtained limits." Similar observations are also made in the Federal Highway Administration's (FHWA) *Hydraulic Engineering Circular No. 18* (HEC-18), *Evaluating Scour at Bridges* (Federal Highway Administration, 1995). In particular, it is noted in that publication that "adequate consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for scour. The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a prudent design." It is further stated that such data should include "performance of existing structures during past floods."

Purpose and Scope:

In view of a definite need for a better understanding of the utility of available scour formulas, this project was undertaken to collect data on 18 designated bridge sites throughout Alabama and perform scour evaluations for known (100-year, 200-year or 500-year) flood events which occurred at these sites. Plots were developed to compare field measurements obtained during yearly site inspections, ground elevations taken during flood measurements where available, and the computed scour from the estimated and measured flood events. A critical evaluation of each site presents an indication of how and why the scour calculations may compute excessive scour depths.

METHODS OF STUDY:

In order to study a bridge site in depth, all of the supporting information must be collected. This collection includes a history of hydrologic information, hydraulic information, structural information, and geotechnical information. The Alabama Department of Transportation (ALDOT) houses structural and geotechnical information on all bridges that they maintain; usually in the form of construction plans, and through the use of its bridge information management system. The United States Geological Survey (USGS) stores a history of hydrologic and hydraulic information on selected bridges throughout Alabama in the form of a hardcopy database and through websites depending on the information required. All of the data needed and that was available was collected from these two organizations and compiled in a format to facilitate a scour evaluation. Scour calculations were performed using methodologies described in HEC-18 by Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer program. This information is presented in the report.

ORGANIZATION OF THE REPORT:

Compilations of the bridge sites, which have been evaluated, are presented in Section II. A discussion of the type and extent of the available data and how it was collected is in Section III. A presentation and discussion of the model development and HEC-18 scour calculations are presented in Section IV followed by critical evaluations of the collected and analyzed data in Section V. The main conclusions are presented in Section VI. There are two appendices printed separately from the main report. Appendix

A contains detailed information about the sites and Appendix B contains the hydraulic scour calculations. Both the main report and the appendices are also available separately as Adobe Portable Document Format (PDF) files.

II. Selected Bridge Sites

The bridge sites, which have been studied, are listed in Table 1. The table gives the U.S. Geological Survey gage station number for each site, followed by station name, the construction date (year) of the main bridge at the site, the date (year) of the most extreme flood experienced by the bridge, an approximate lower-bound estimate of the recurrence interval of the flood (as 100, 200, or 500 years) as determined with the flood frequency (regression equation) method of Atkins (1996), and the bridge identification numbers of the bridges at the site. The locations of the selected bridge sites, along with their U.S. Geological Survey Station numbers, are indicated in Figure 1.

There are 18 sites identified in Table 1, which have all experienced an extreme flood of recurrence interval of at least 100 years, and 6 of these sites have experienced a flood of a recurrence interval of 500 years or more. A few of the bridges, most notably southeastern Alabama bridges with soils similar to sand, required countermeasures after the flood events. All bridges having soils characterized with cohesive properties had little scour and few or no scour countermeasures in place.

Table 1. Selected bridge sites in Alabama

Gage Station	STATION NAME	*Built Date	Flood Date	Recurrence Interval	Main Channel Bridges BIN	Relief Bridges BIN
3576250	Limestone Creek near Athens	1952	1973	200	463; 6544	
3574500	Paint Rock River near Woodville	1985	1990	100	13564; 11960	
2367500	Lightwood Knot Creek at Babbie	1982	1990	100	12968	12969
2361000	Choctawhatchee River near Newton	1976	1990	500	11857	
2363000	Pea River near Ariton	1970	1990	100	10355	10354; 5572; 5573
2450000	Mulberry Fork near Garden City	1960	1990	200	6914	
2364000	Pea River at Elba	1959	1990	100	6434	
3575830	Indian Creek near Madison	1965	1973	100	5509; 8735	
2438000	Buttahatchee River below Hamilton	1953	1973	500	4202	13690
2467500	Sucarnoochee River at Livingston	1946	1979	200	2911	2912
2439000	Buttahatchee River near Sulligent	1940	1973	500	2561	2560; 2559; 2558; 2557
2343275	Abbie Creek near Abbevile	1941	1953	100	2540	
2427700	Turkey Creek at Kimbrough	1940	1961	200	2264	2265; 2266
2360000	West Fork Choctawhatchee River at Blue Springs	1938	1990	500	1731	
2451750	Vest Creek near Baldwin	1934	1964	200	1127	
2469500	Tuckabum Creek near Butler	1934	1964	200	1095	1096; 1097; 970
2340750	Osanippa Creek near Fairfax	1929	1961	500	615	
2406000	Talladega Creek near Talladega	1928	1951	500	4470	

*some relief bridges may have been built at different years

Figure 1. Location of selected bridge sites in Alabama

III. Available Information

There were three primary sources for information: the Alabama Department of Transportation (ALDOT), the United States Geological Survey (USGS) and site field inspections conducted by the investigators. In general, ALDOT provided information on the structural details of the bridge and a history of the ground line at the bridge. The USGS provided a record of detailed hydrologic data and some hydraulic data for each site. The site inspections provided soil samples and observations of what scour had occurred at each site.

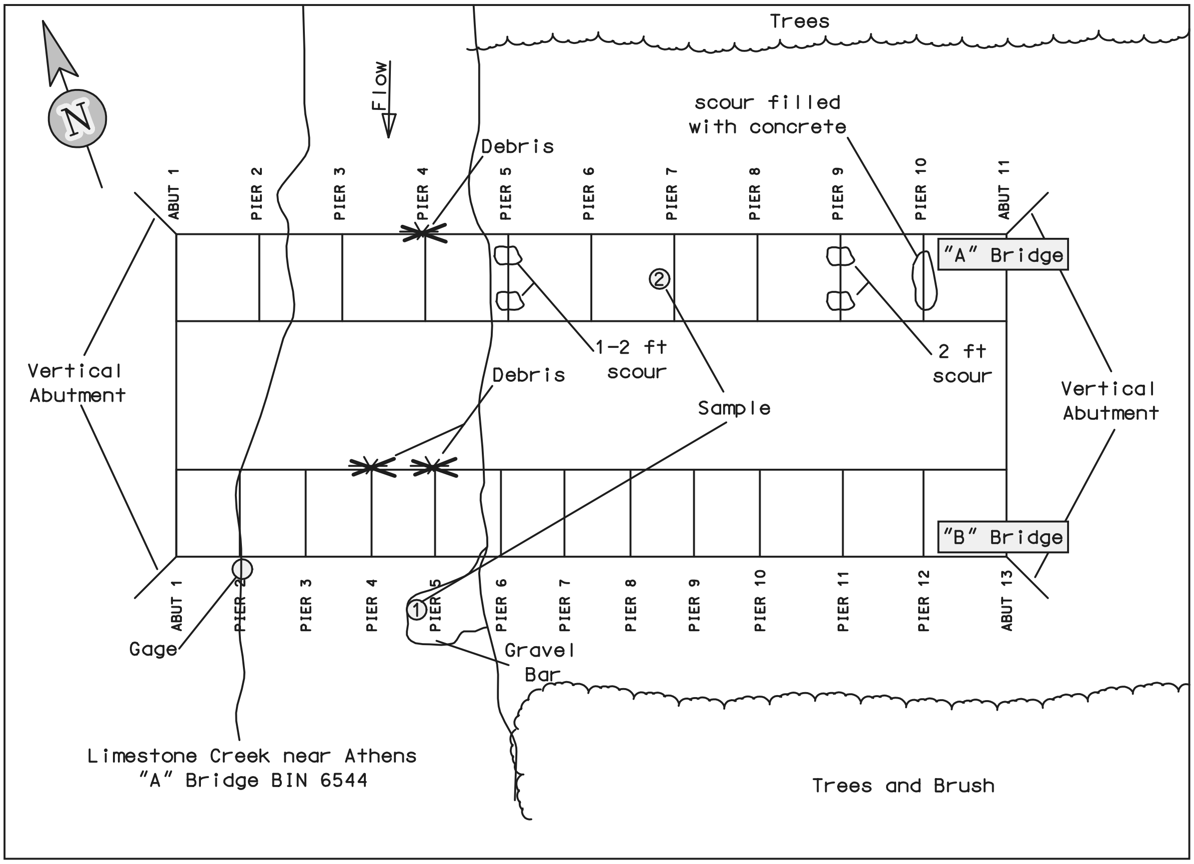
ALDOT information included the Alabama Bridge Information Management System (ABIMS) information, plan information, and bridge card information. The data base of the Alabama Bridge Information Management System (ABIMS) contains information about Alabama's bridges which includes profiles of the bridge structures, the original ground level and subsequent periodic sounding profiles, bridge footing and piling types, and elevations and structural appraisal information derived from historic data and visual inspections by ALDOT bridge inspectors. Along with ABIMS information, historical structural information exists in hardcopy form. Some of this information is used to supply the ABIMS database. Construction plans consisting of a cover sheet, plan/profile sheets, general elevation views, and core borings were available in full or partially for the bridge sites. A bridge card was also available for most bridges, which was used to supply historic information about a structure. ALDOT has created bridge cards for every bridge in the ALDOT inventory. The data includes superstructure information (floor depth, railing description, stringers – beams or girders, trusses, etc.), substructure information (length of piles, foundation material, etc.), and general information (name, construction dates, overall length, number of spans, original cost, minimum design rating, pavement width, shoulder width, sight distance, etc.). The majority of this ALDOT information was obtained from ALDOT's main office in Montgomery, but some inspection reports and plans came from ALDOT Division offices and County Engineers.

The USGS hydrologic data and hydraulic data were obtained from the Alabama USGS offices in Montgomery and in Tuscaloosa and from their website. Each bridge site in the study had at present or previously a gage in place at or near the bridge.

Historically, gaging station data have been used for management of national land and water resources. Many of the gages were used to record stages over periods of time. During large storm events and when possible the USGS measures velocities and areas to calculate discharges per stage heights to develop rating curves. The rating curves and measured stages are then used for determining discharges for daily flows. This data is used for determining low flows, average flows, and peak flows. This information can be used for several applications. For example, low flows and average flows can be used in determining water supply constraints while peak discharges may be used in the design of bridges and culverts.

The data compiled in a recent report by Atkins (1996) for annual storm events in Alabama aided in the choosing of the sites for this study. All of the sites have experienced a storm event of a recurrence interval of at least 100-years. The majority of the sites were located at or near state structures to ensure sufficient data existed for analysis. All available field measurements of velocities, ground lines, and calculated discharges were obtained for all of the bridge sites that were selected.

Field inspections were conducted during the fall months of 2001 when vegetation was sparse. The field crews consisted of three men. Each site bridge was videoed, photographed, and reviewed based on visual inspections. Clues to scour including old mud lines, timber piles originally cut off to the ground, in-place riprap countermeasures, and obvious contraction, pier, and abutment scour were identified and labeled on site sketches found in Appendix A. An example of one of the site sketches found in Appendix A can be seen in Figure 2. Soil samples were taken from the channel and from the overbank for the main channel bridges, and from representative areas reflecting the entire reach of the relief bridges. Each sample was analyzed to determine the diameter of the soil for which 50% of the sizes were smaller (D_{50}), the diameter of the soil for which 90% of the sizes were smaller (D_{90}), the liquid limit (LL), the plastic limit (PL), and the plasticity index (PI) at the Auburn University Civil Engineering Soil Mechanics Laboratory. All data were logged and recorded including the soil samples, video, photographs, site inspection notes, aerial photos, quadrangle location maps, structure history, plan information, ABIMS profiles and structure appraisal history, and USGS hydrologic and hydraulic information. The available data for each site are presented in detail in a separate appendix, Appendix A.



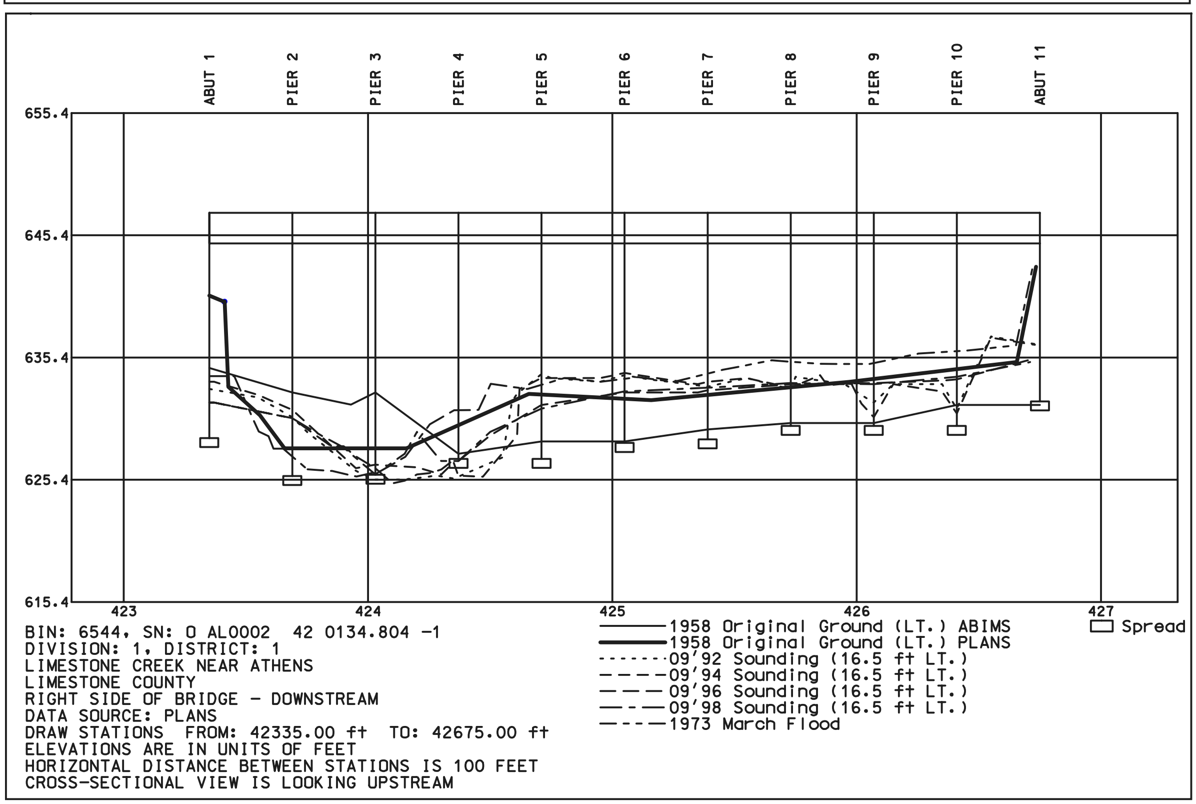


Figure 2. Site inspection sketch and elevation view for the Limestone Creek bridge near Athens

IV. Model Development and Scour Calculations

Model Development

Each bridge site was modeled using the Hydrologic Engineering Center River Analysis System (HEC-RAS) software (Brunner, 2001a, b). 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. 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 and stage measurements. Discharges and starting downstream water surface elevations were two boundary conditions needed in modeling the sites. The discharges were taken from USGS records or from field measurements. The starting downstream water surface elevations were either taken from USGS records and field measurements or were calculated based on assuming uniform flow using the average channel slope and conveyance at the downstream section.

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 calculation results are presented in detail in a separate appendix, Appendix B. The following section describes how scour is 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).

$$Vc = 10.95 y_1^{1/6} D_{50}^{1/3} \qquad (1)$$

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.

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

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

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

V_*/ω	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).

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

$$y_s = y_2 - y_0 \qquad (5)$$

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).

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

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 given below:

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:

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):

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

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 below:

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

Local Scour at Abutments

Local scour occurs at abutments when the abutment obstructs the flow. The obstruction of the flow forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and forms a vertical wake vortex at the downstream end of the abutment.

HEC No. 18 report recommends two equations for the computation of live-bed abutment scour. When the wetted embankment length (L) divided by the approach flow depth (y_1) is greater than 25, the HEC No. 18 report suggests using the Highways in the River Environment (HIRE) equation (Richardson et al., 1990, p.V-101). When the wetted embankment length divided by the approach depth is less than or equal to 25, the HEC No. 18 report suggests using an equation by Froelich (Froelich, 1989, pp.13-18).

HIRE Equation

The HIRE equation is based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE). The HIRE equation presented in HEC-18 (Fourth Edition, FHWA, 2001, p. 7.9) is:

$$y_s = 4y_1\left(\frac{K_1}{0.55}\right)K_2 Fr^{0.33} \qquad (12)$$

Where:

y_s =Scour depth in ft

y_1 =Depth of flow at the tow of the abutment on the overbank or in the main channel, ft

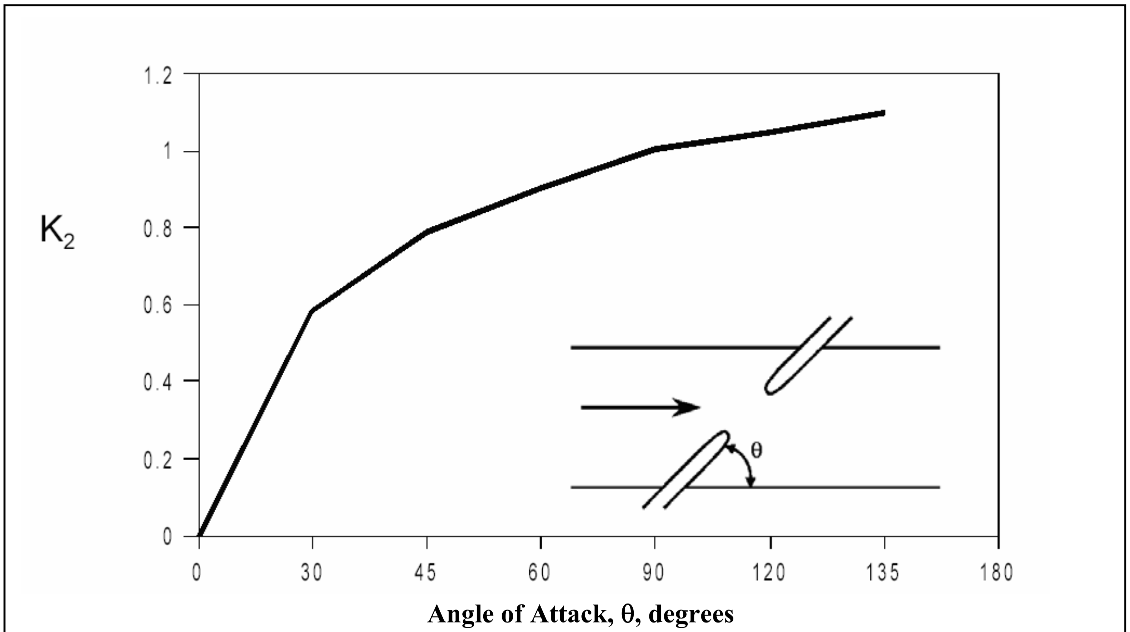
K_1 =Correction factor for abutment shape

K_2 =Correction factor for angle of attack (θ) of flow with abutment.

Fr =Froude number based on velocity and depth adjacent and just upstream of the abutment toe

Description	**K_1**
Vertical-wall Abutment	1.00
Vertical-wall Abutment with wingwalls	0.82
Spill-through Abutment	0.55

The correction factor, K_2, for angle of attack can be taken from below



Correction factor for abutment skew, K_2

Froelich's Equation

Froelich analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation presented in HEC-18 (Fourth Edition, FHWA, 2001, p. 7.8):

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{L'}{y_a} \right)^{0.43} Fr^{0.61} + 1 \qquad (13)$$

Where:

y_s	= Scour depth in ft
K_1	= Correction factor for abutment shape
K_2	= Correction factor for angle of attack (θ) of flow with abutment.

L'	= Length of abutment (embankment) projected normal to flow, ft
y_a	= Average depth of flow on the floodplain at the approach section, ft
Fr	= Froude number of the floodplain flow at the approach section
	$= \dfrac{V_e}{(gy_a)^{1/2}}$
V_e	= Average velocity of the approach flow, ft/s
	$= \dfrac{Q_e}{A_e}$, ft/s
Q_e	= Flow obstructed by the abutment and embankment at the approach section, cfs
A_e	= Flow area of the approach section obstructed by the abutment and embankment, ft^2

V. Critical Evaluations

Results and Discussion

Detailed site data and calculations are presented in Appendix A and Appendix B. Here, a critical evaluation of each site is presented using plots developed from the measured soundings from ALDOT, original groundline and soil boring profiles from the plans, and soundings taken during the measured flood when available. Scoured bed profiles were added to the plots based on conventional calculations with an assumed fine sand D_{50} equal to 0.16 mm (0.00050 ft). (Such assumptions are usually made by some engineers in the absence of direct soils information from a site). Calculations were also made using a D_{50} from soils sampled during the site inspections. Conventional calculation of scour depths does not account for cohesion in the bed material. Cohesive soils are less erodible than noncohesive soils of similar grain size. Cohesive bonds in the bed material increase the critical bed shear stress, which delays the onset of scour. Also, if the critical bed shear stress is exceeded, cohesion in the bed material reduces the rate of scour (see, e.g., Briaud et al., 1999, 2001a, 2001b; Güven et al., 2001; Güven et al., 2002, TRB Paper No. 02-2127, accepted for publication). Conventional scour calculations do not account for consolidated soil or rock bed material. General site characteristics are pointed out and opinions are expressed based on site comparisons. The evaluations are summarized in this chapter.

LIMESTONE CREEK NEAR ATHENS

This site is located in Limestone County. There are two parallel main channel bridges. Two views of the site are shown in Figures 3 and 4. The detailed site information is contained in Appendix A. In 1973 the bridge experienced an approximate 200-year flood. The bridges were analyzed for this event together. Scour calculations found in Appendix B were performed as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was a loose sand and gravel. The soils were analyzed at the Auburn University Soils Laboratory and the results were D_{50} = 0.022 mm (0.000070 ft) for the overbank soil and D_{50} = 2.7 mm (0.0089 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as a well graded sand with gravel. Figure 5 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, a recent sounding, and a sounding taken during the measured flood. The calculated scour with the assumed grain size indicates 4 to 25 feet of combined contraction and pier scour and 34 to 47 feet of contraction and abutment scour. The recent sounding and site inspection determined these depths were not reached in the overbank and it is highly unlikely the channel reached those depths as well. The calculated scour with the sampled grain size indicates 4 to 47 feet of combined contraction and pier scour and 34 to 69 feet of combined contraction and abutment scour. Scour observed in the field consisted mainly

Figure 3. Limestone Creek near Athens (BIN# 6544);
view looking west toward abutment 1.

Figure 4. Limestone Creek near Athens (BIN# 6544);
view from the upstream side looking
at debris around pier 4.

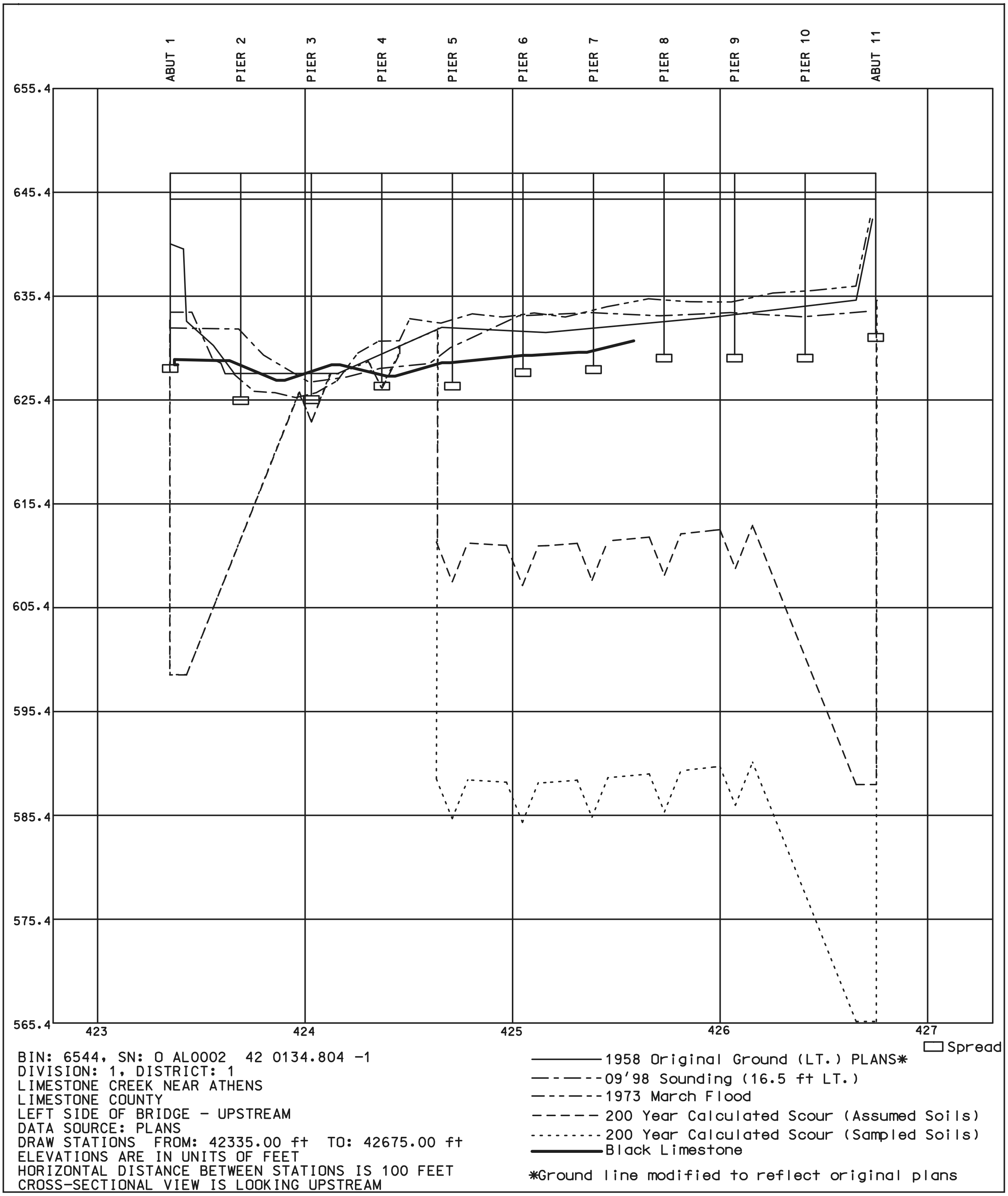


Figure 5. Sounding profiles and scour evaluations for the main channel bridge at the Limestone Creek site near Athens

of 2 to 3 feet deep local scour at piers #5, #9, and #10 on the upstream bridge and almost no scour at these piers on the downstream bridge. The sounding also indicates very little if any long-term degradation.

The reason these calculated scour depths were not reached at this site is due to the cohesive nature of the soil. Even if the top cohesive layer were scoured away, a hard limestone rock is present which would provide even more resistance to scour than the cohesive material. Calculated scour depths do not account for cohesion on the limestone rock.

INDIAN CREEK NEAR MADISON

The site is located in Madison County. There are two parallel main channel bridges. Two views of the site are shown in Figures 6 and 7. The detailed site information is located in Appendix A. In 1973 the bridge experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of soil in the overbank and in the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was loose sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.020 mm (0.000070 ft) for the overbank soil and D_{50} = 1.28 mm (0.00420 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as a well graded sand. Figure 8 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline

Figure 6. Indian Creek near Madison (BIN# 5509);
view looking north at gage on pier 8.

Figure 7. Indian Creek near Madison (BIN# 5509);
view of scour hole under Bridge A at pier 3.

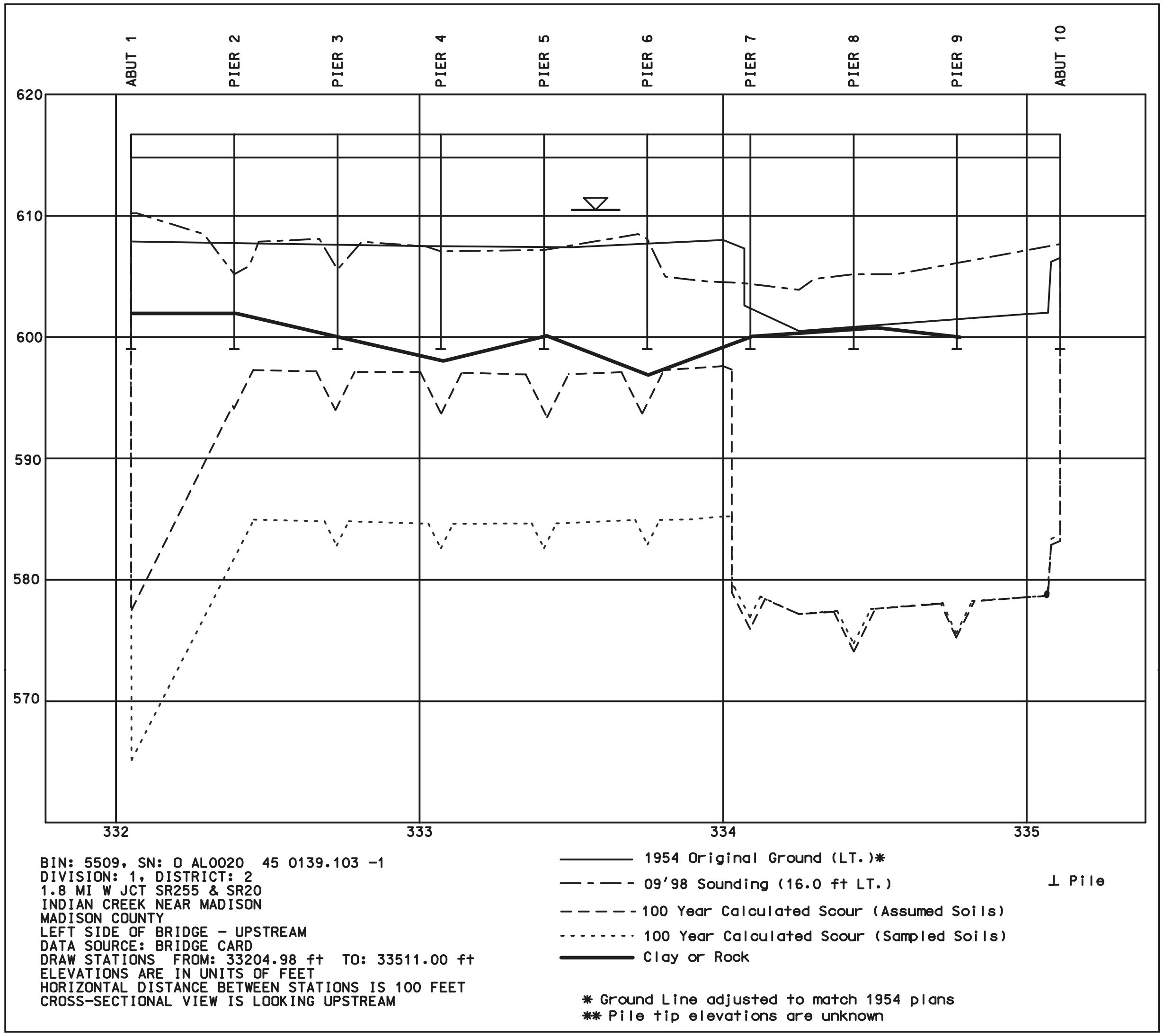


Figure 8. Sounding profiles and scour evaluations for the main channel bridge at the Indian Creek site near Madison

and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain size indicates 14 to 27 feet of combined contraction and pier scour and 23 to 30 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 25 to 27 feet of combined contraction and pier scour and 23 to 43 feet of combined contraction and abutment scour. The recent sounding indicates the main channel has aggraded and there is 2 to 3 feet of local scour at pier #2 and pier #3. Scour observed in the field consisted of a 3 to 4 feet deep scour hole around pier #2 of bridge A, and several piles had been re-encased with concrete on piers #2 and #3. Another 3 to 4 feet scour hole was found around pier #9 of bridge B, with several piles being re-encased there as well.

The reason these calculated scour depths were not found on the overbank and main channel is due to the cohesive nature of the soil. Even if the top sand layer were scoured away in the main channel a clay layer is present which would provide resistance to scour.

PAINT ROCK RIVER NEAR WOODVILLE

The site is located in Jackson County. There are two parallel main channel bridges. Two views of the site are shown in Figures 9 and 10. The detailed site information is located in Appendix A. In 1990 the bridge experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be compact sand and

Figure 9. Paint Rock River near Woodville (BIN# 13564);
view looking west toward abutment 1.

Figure 10. Paint Rock River near Woodville (BIN# 13564);
view under Bridge A from pier 13
looking northwest toward abutment 1.

the channel was loose sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.0310 mm (0.00010 ft) for the overbank soil and D_{50} = 0.65 mm (0.00210 ft) for the main channel soil. The soil in the overbank was classified as silt and the soil in the main channel was classified as a well graded sand. Figure 11 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain size indicates 25 to 33 feet of combined contraction and pier scour and 35 to 38 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 31 to 48 feet of combined contraction and pier scour and 55 to 56 feet of combined contraction and abutment scour. The recent sounding indicates the main channel has aggraded slightly and there is 3 to 4 feet of degradation at piers #4 and #5. The bridges were plated with class 2 riprap indicating scour had occurred throughout the bridge reach. The exact amount of scour damage was unknown, but it is obvious that the calculated depths were not reached, since this would have caused a bridge failure.

The reason these calculated scour depths were not reached on the overbank and main channel is due to the clay layer below the silt and sand layer in the overbank and main channel. This cohesive layer would provide resistance to scour.

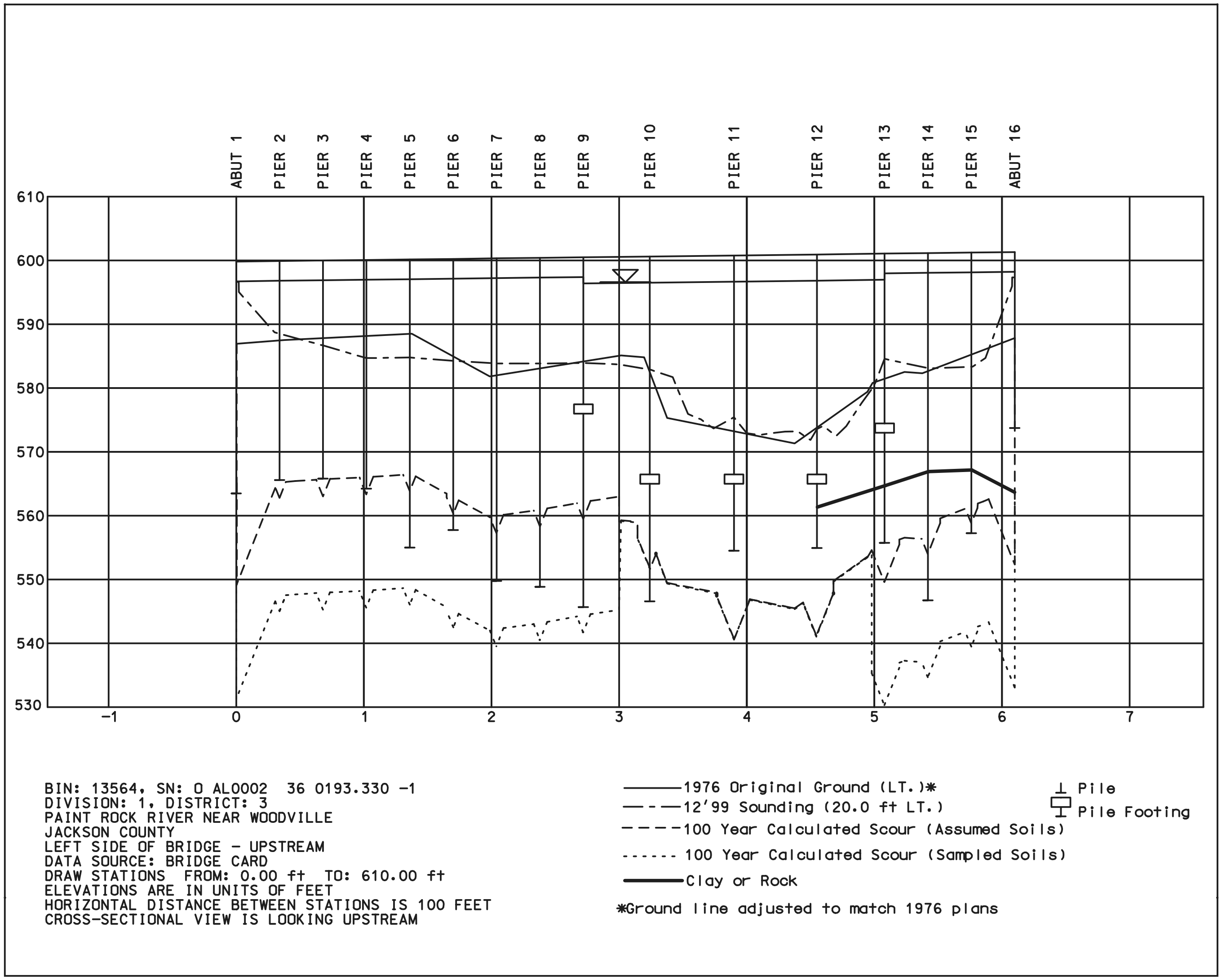


Figure 11. Sounding profiles and scour evaluations for the main channel bridge at the Paint Rock River site near Woodville

VEST CREEK NEAR BALDWIN

The site is located in Cullman County. There is one main channel bridge. Two views of the site are shown in Figures 12 and 13. The detailed site information is located in Appendix A. In 1964 the bridge experienced an approximate 200-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Visual observations indicated rock was present throughout this site therefore no soil samples were taken. Figure 14 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain size indicates 3 to 8 feet of combined contraction and pier scour and 0 to 3 feet of contraction and abutment scour. The recent sounding indicates the channel has aggraded in the channel approximately 2 to 3 feet. The site inspection revealed local erosion on the east abutment due to local runoff. Rock outcroppings can be found throughout the channel. No scour was present in the channel. The basis for the calculated scour depths not being reached throughout the site is due to a rock foundation.

Figure 12. Vest Creek near Baldwin (BIN# 1127); view from the upstream side looking southwest toward abutment 4.

Figure 13. Vest Creek near Baldwin (BIN# 1127); view from the upstream side looking west toward abutment 4.

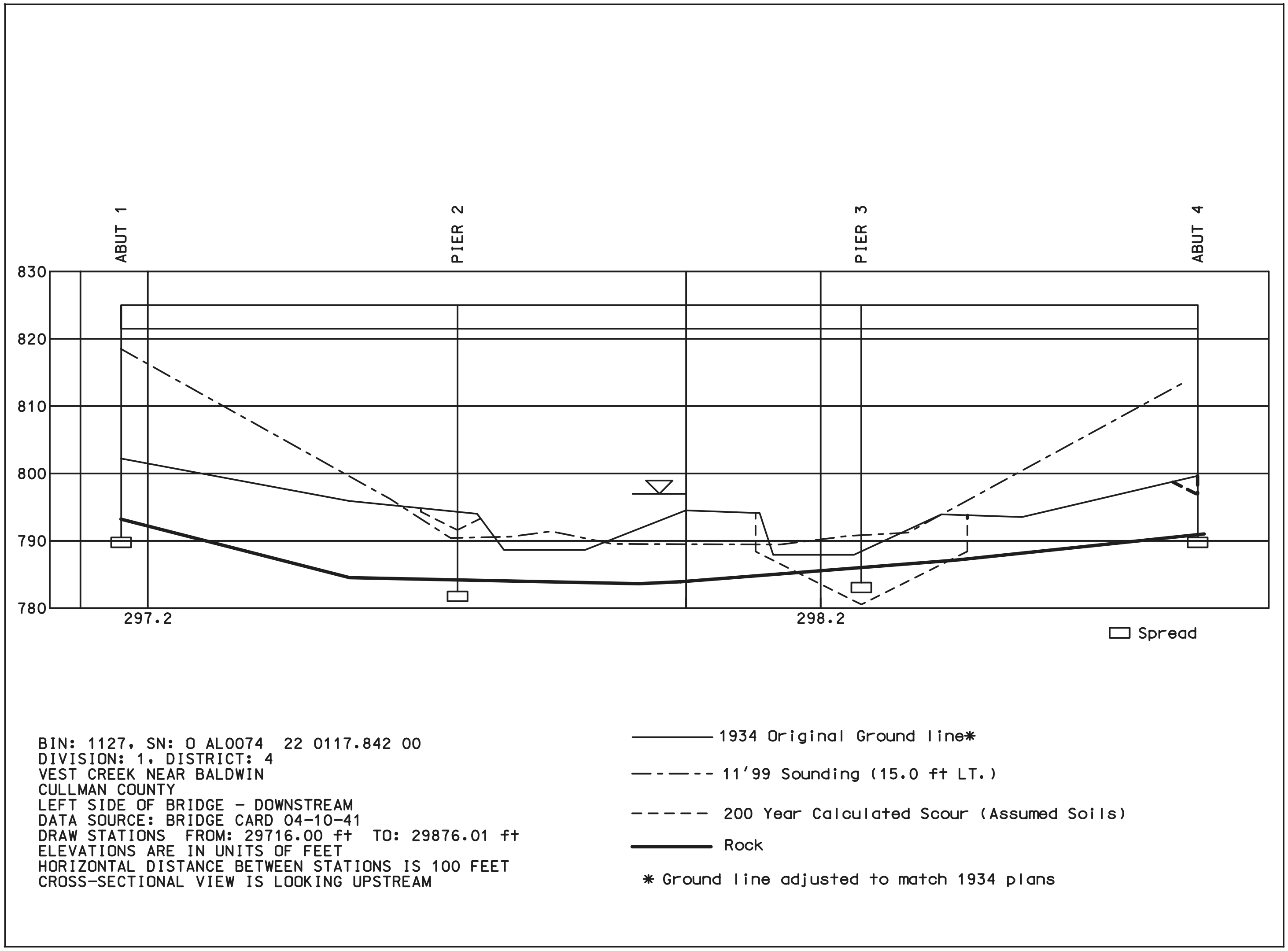


Figure 14. Sounding profiles and scour evaluations for the main channel bridge at the Vest Creek site near Baldwin

BUTTAHATCHEE RIVER BELOW HAMILTON

The site is located in Marion County. There are two bridges, a main channel bridge and relief bridge. Two views of the site are shown in Figures 15 and 16. The detailed site information is located in Appendix A. In 1973 the bridges experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the main channel bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The relief bridge was replaced in 1985 and was not included in the analysis. The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was loose sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.055 mm (0.00018 ft) for the overbank soil and D_{50} = 0.27 mm (0.00089 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as a poorly graded sand. Figure 17 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, a recent sounding, and a sounding taken during the measured flood. The calculated scour with the assumed grain size indicates 12 to 31 feet of combined contraction and pier scour and 12 to 30 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 20 to 30 feet of combined contraction and pier scour and 12 to 38 feet of combined contraction and abutment scour.

Figure 15. Buttahatchee River below Hamilton (BIN# 4202); view from the downstream side looking north toward abutment 10.

Figure 16. Buttahatchee River below Hamilton (BIN# 4202); view from pier 6 looking south toward abutment 1.

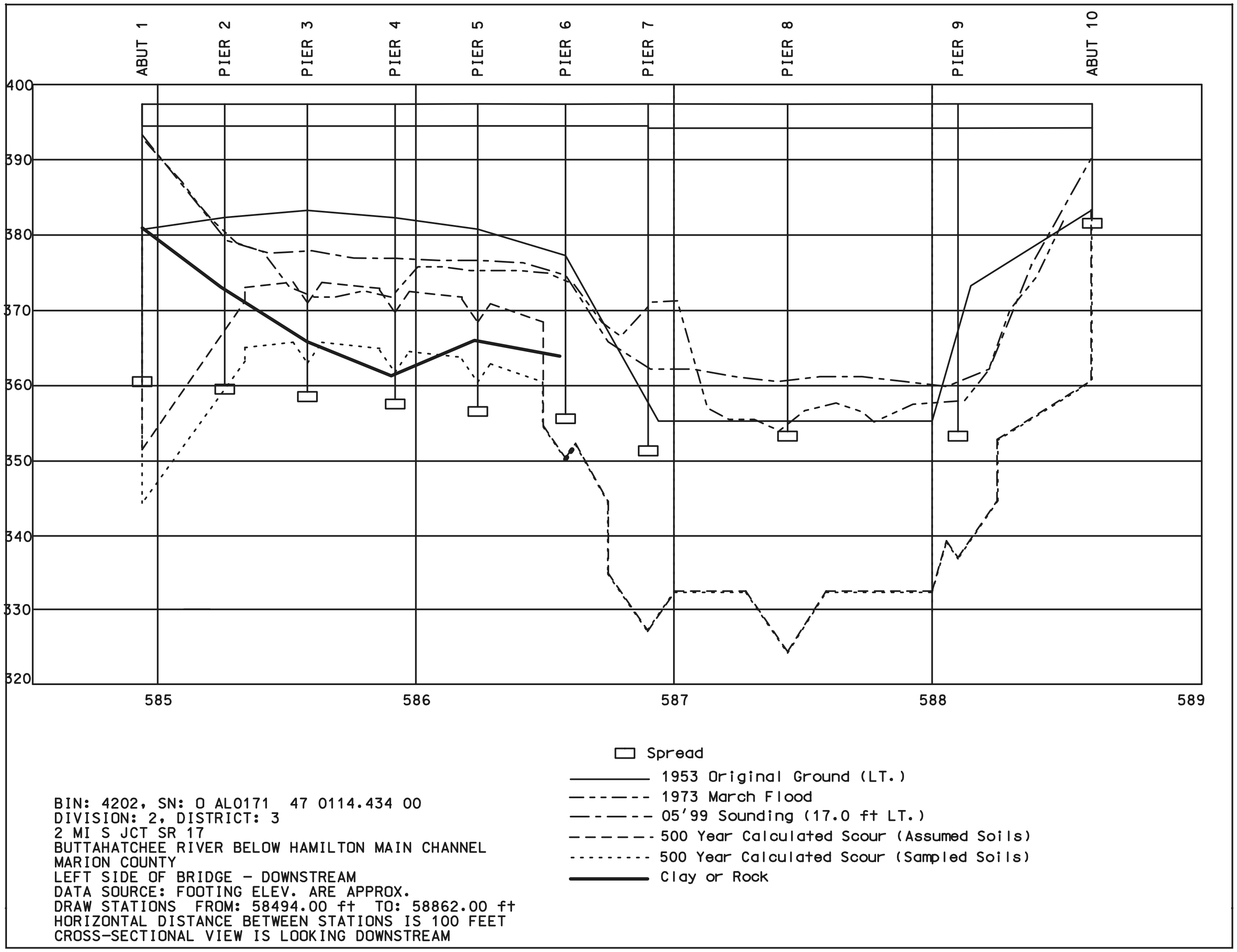


Figure 17. Sounding profiles and scour evaluations for the main channel bridge at the Buttahatchee River site below Hamilton

The 1973 flood sounding indicated that the calculated scour depths with the assumed fine sand D_{50} values were reached on the left overbank, but the calculated scour depths with the sampled soil size were not reached. The core borings and analyzed sampled soil indicate a silty sand and clay is present. The site inspection suggested possible long-term degradation in the overbank. The calculated scour depths were not observed anywhere else at the bridge site due to the cohesive nature of the soil. Below the clay layer is a rock layer that would provide even more resistance to scour.

BUTTAHATCHEE RIVER NEAR SULLIGENT

The site is located in Lamar County. There is one main channel bridge and four relief bridges. Two views of the site are shown in Figures 18 and 19. The detailed site information is located in Appendix A. In 1973 the bridges experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the bridges as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was silt. Sand was present in the overbank of the main channel as well. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.020 mm (0.000070 ft) for the overbank soil and D_{50} = 0.33 mm (0.0011 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as poorly graded sand. Figures 20, 21, 22, 23, and 24 show the results of the calculated scour depths plotted on the ABIMS profiles for the main channel and all four relief bridges. The plots

Figure 18. Buttahatchee River near Sulligent, Relief Bridge 4 (BIN# 2557); view from the downstream side looking south toward abutment 1.

Figure 19. Buttahatchee River near Sulligent, Relief Bridge 4 (BIN# 2557); view from the downstream side looking southeast at ponded water near abutment 1.

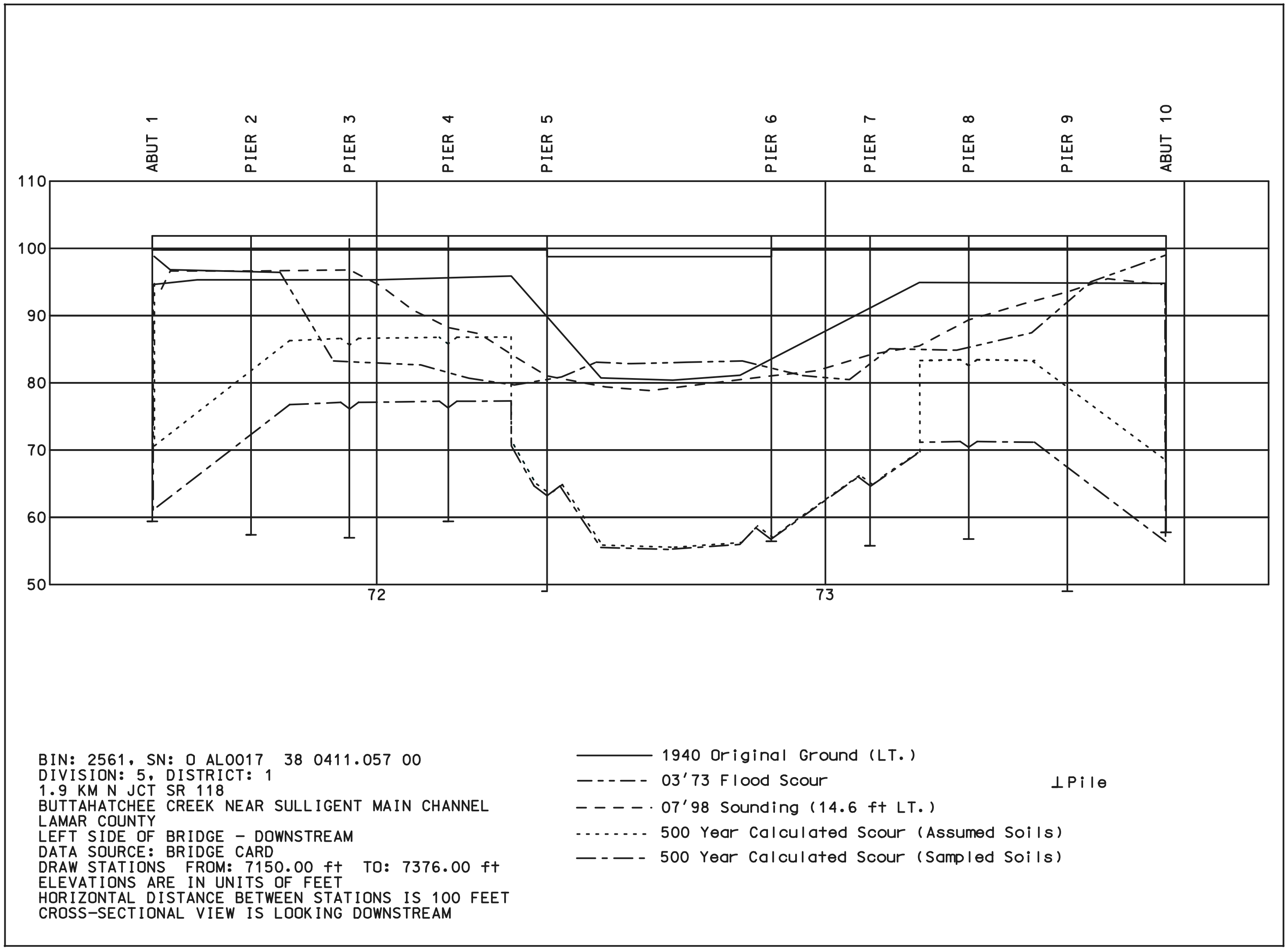


Figure 20. Sounding profiles and scour evaluations for the main channel bridge at the Buttahatchee River site near Sulligent

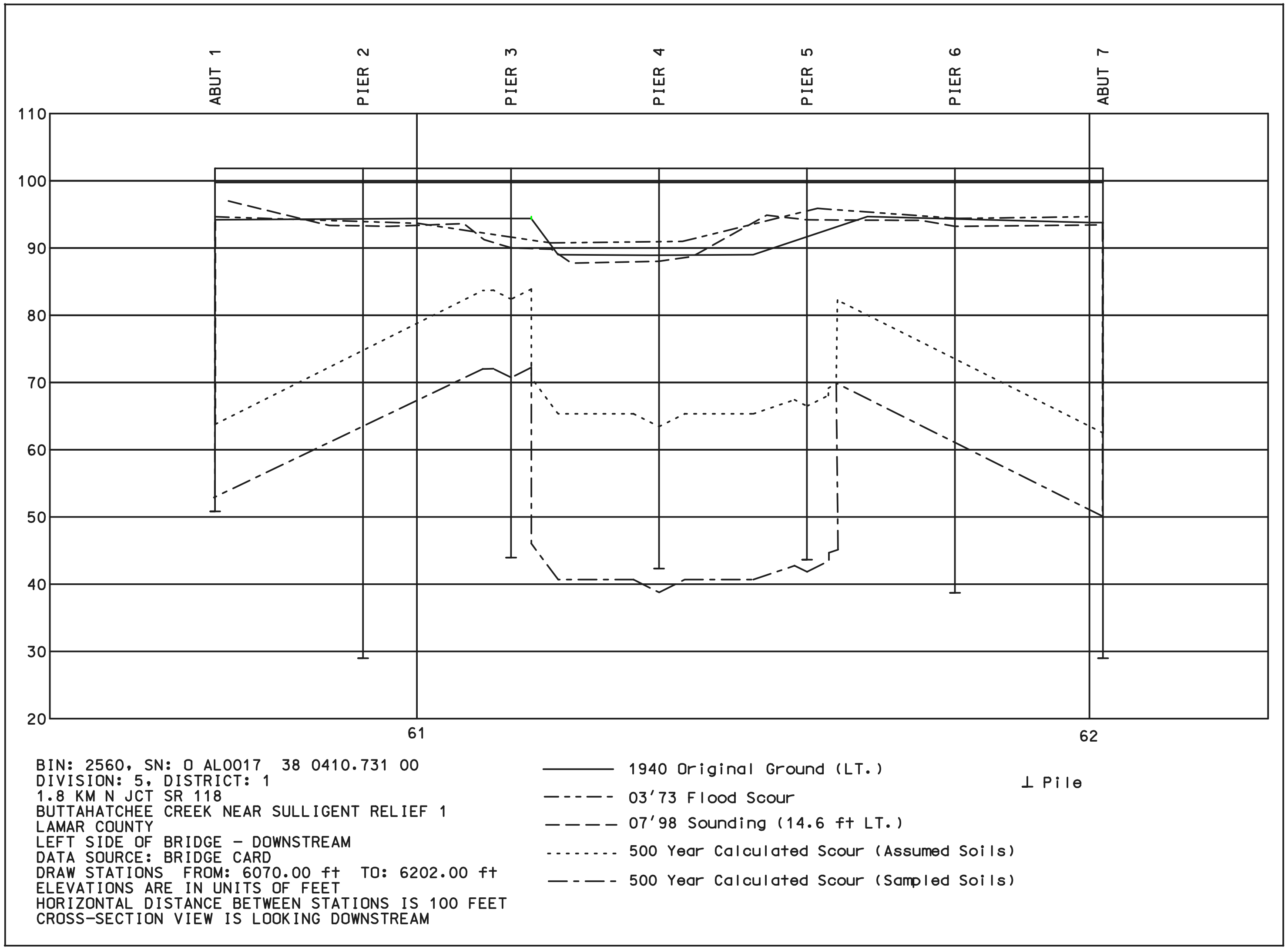


Figure 21. Sounding profiles and scour eveluations for relief bridge 1
at the Buttahathee River site near Sulligent

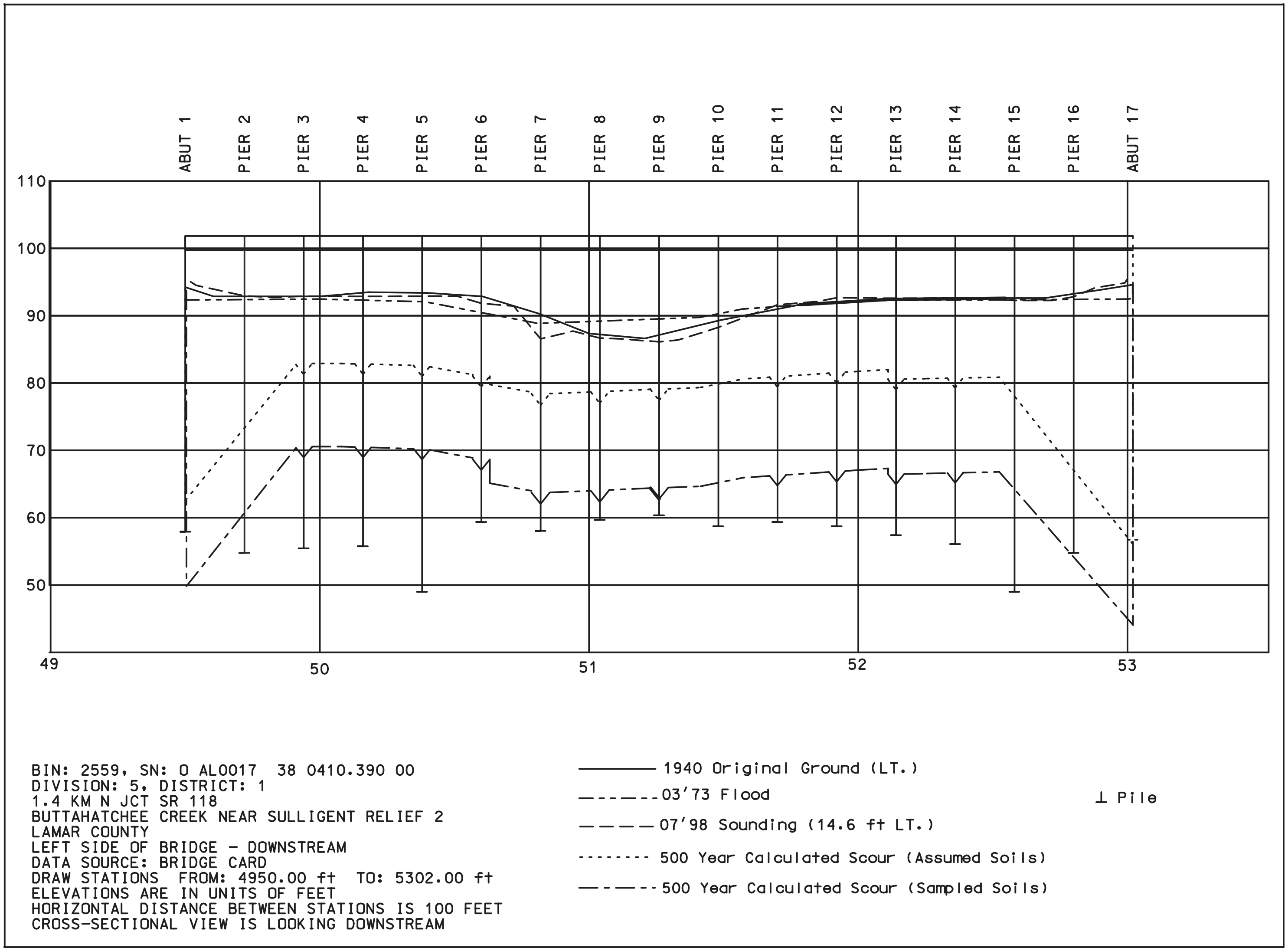


Figure 22. Sounding profiles and scour evaluations for relief bridge 2 at the Buttahatchee River site near Sulligent

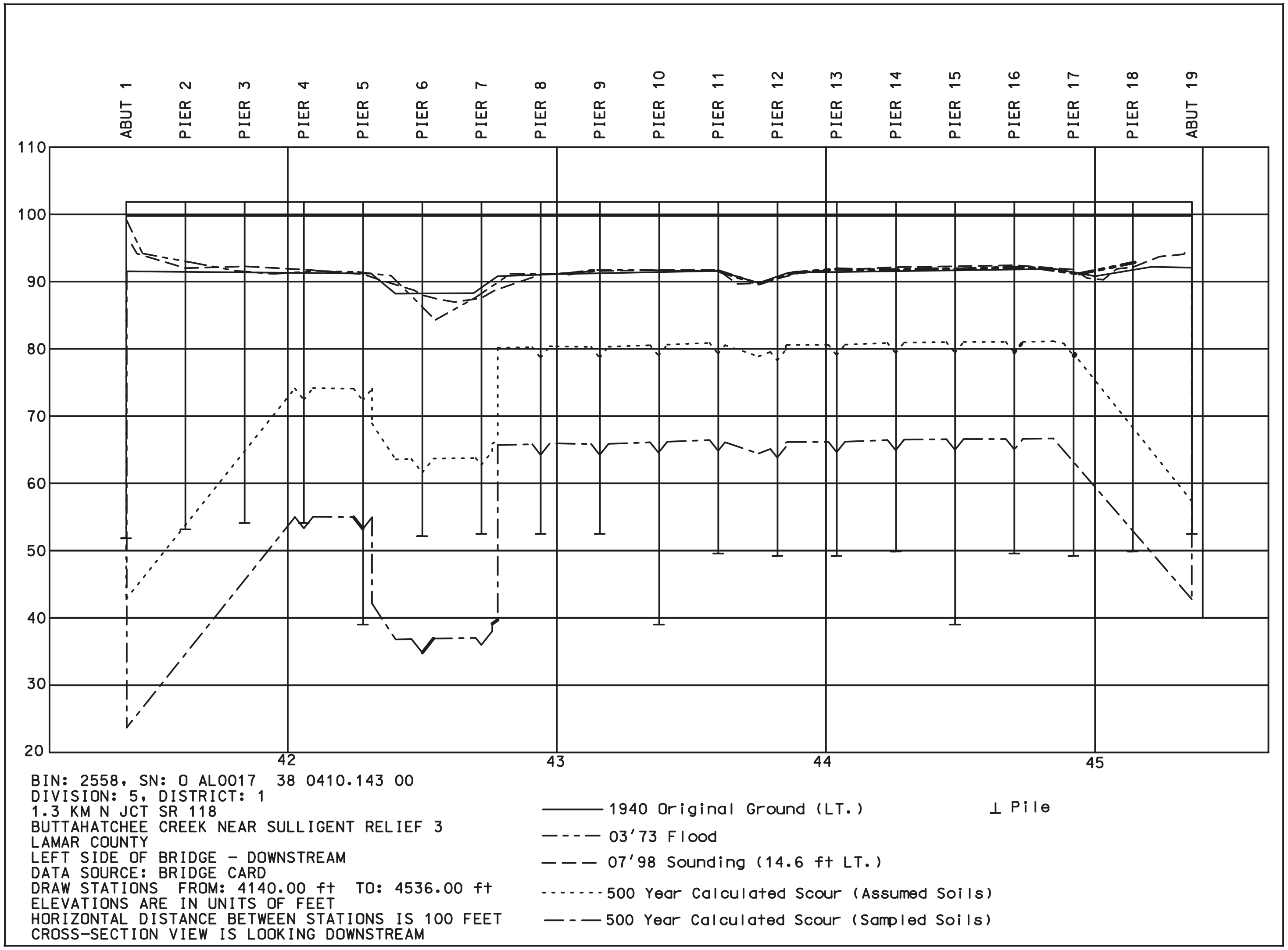


Figure 23. Sounding profiles and scour evaluations for relief bridge 3 at the Buttahatchee River site near Sulligent

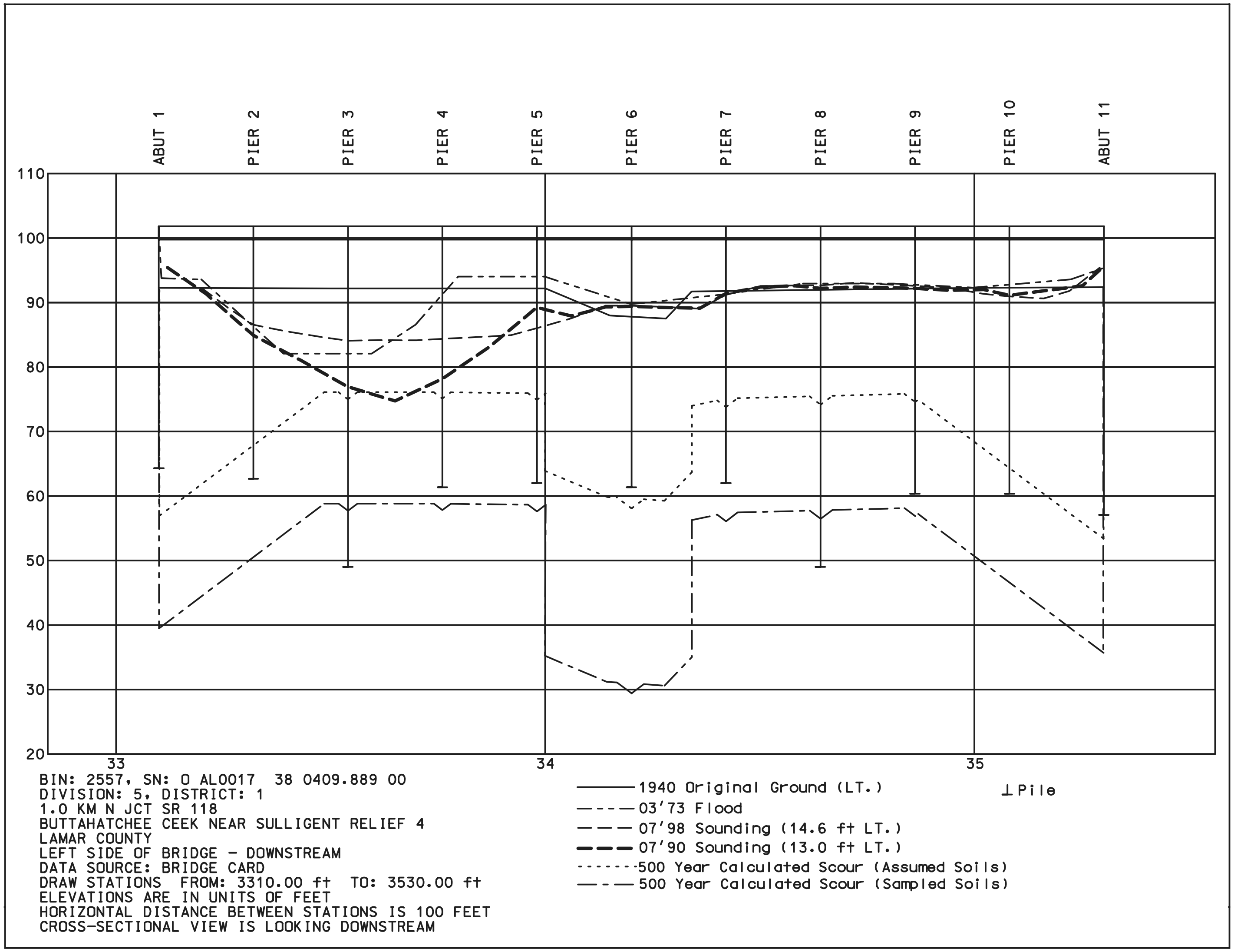


Figure 24. Sounding profiles and scour evaluations for relief bridge 4 at the Buttahatchee River site near Sulligent

consist of original groundline from the plans, a recent sounding, a sounding from the 1973 flood, and calculated scour with the assumed grain size and with the measured grain size for the measured event.

The calculated scour at the main channel with the assumed grain size indicates 10 to 27 feet of combined contraction and pier scour and 24 to 26 feet of contraction and abutment scour. The recent sounding and site inspection determined these depths were reached for the assumed soil on the left overbank and not reached on the right overbank or in the channel. Scour observed in the field consisted of a 2 foot scour hole at pier #2. The sounding indicates a widening of the channel. The calculated scour depths for the sampled soils were not reached. The reason these calculated scour depths were not reached is due to the cohesive nature of the soil.

The calculated scour at relief #1 for the assumed grain size indicates 12 to 26 feet of combined contraction and pier scour and 30 to 31 feet combined contraction and abutment scour. The calculated scour at relief #1 for the sampled grain size indicates 24 to 50 feet of combined contraction and pier scour and 42 to 44 feet of combined contraction and abutment scour. Scour observed in the field consisted mainly of 3 feet deep local scour at pier #4. Riprap has been added between piers #5 and #6. The sounding indicates 3 to 4 feet deep scour between piers #4 and pier #5 and at the abutments. The calculated scour depths were not reached for either soils condition due to the cohesive nature of the soil.

The calculated scour at relief #2 for the assumed grain size indicates 11 to13 feet of combined contraction and pier scour and 31 to 36 feet combined contraction and abutment scour. The calculated scour at relief #2 for the sampled grain size indicates 23 to 27 feet of combined contraction and pier scour and 44 to 51 feet of combined contraction and abutment scour. Scour observed in the field consisted of 2 feet deep local pier scour located from pier #11 to pier #16. The sounding indicates the channel has shifted. The calculated scour depths were not reached for either soils condition due to the cohesive nature of the soil.

The calculated scour at relief #3 for the assumed grain size indicates 12 to 27 feet of combined contraction and pier scour and 35 to 49 feet combined contraction and abutment scour. The calculated scour at relief #3 for the sampled grain size indicates 27 to 53 feet of combined contraction and pier scour and 49 to 68 feet of combined contraction and abutment scour. Riprap was in place from abutment #1 to pier #3 indicating scour may have occurred in the past. The soundings indicate the channel has scoured between pier #6 and pier #7. The calculated scour depths were not reached for either the assumed or the sampled soil size due to the cohesive nature of the soil and riprap placement.

The calculated scour at relief #4 for the assumed grain size indicates 17 to 30 feet of combined contraction and pier scour and 35 to 39 feet combined contraction and abutment scour. The calculated scour at relief #4 for the sampled grain size indicates 34 to 58 feet of combined contraction and pier scour and 53 to 57 feet of combined contraction and abutment scour. The entire bridge reach had ponded water around it. The soundings indicate the channel has scoured 20 feet between pier #1 and pier #5 and was filled sometime after 1990. The calculated scour depths were exceeded at piers #3 and #4 for the assumed grain size, but were not exceeded for the sampled soil size. The reason these calculated scour depths were not reached for the sampled soils at this site is due to the cohesive nature of the soil.

MULBERRY FORK NEAR GARDEN CITY

The site is located in Blount County. There is one main channel bridge. Two views of the site are shown in Figures 25 and 26. The detailed site information is located in Appendix A. In 1990 the bridge experienced an approximate 200-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank and in the channel to be sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.25 mm (0.00082 ft) for the overbank soil and D_{50} = 0.90 mm (0.0030 ft) for the main channel soil. The soil in the overbank was classified as a poorly graded sand and the soil in the main channel was classified as a well graded sand with some gravel. Figure 27 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain sizes and with the sampled grain sizes indicate 7 to 14 feet of combined contraction and pier scour and 0 to 7 feet of contraction and abutment scour for both. The recent sounding indicates the left overbank has aggraded by 5 to 6 feet between piers #3 and #4. The site inspection revealed sand overbanks and rock outcropping in the channel. Scour observed in the field consisted of 5 feet of pier scour at pier #3.

Figure 25. Mulberry Fork near Garden City (BIN# 6914); view from the downstream side looking north toward abutment 8.

Figure 26. Mulberry Fork near Garden City (BIN# 6914); view from the upstream side looking north toward abutment 8.

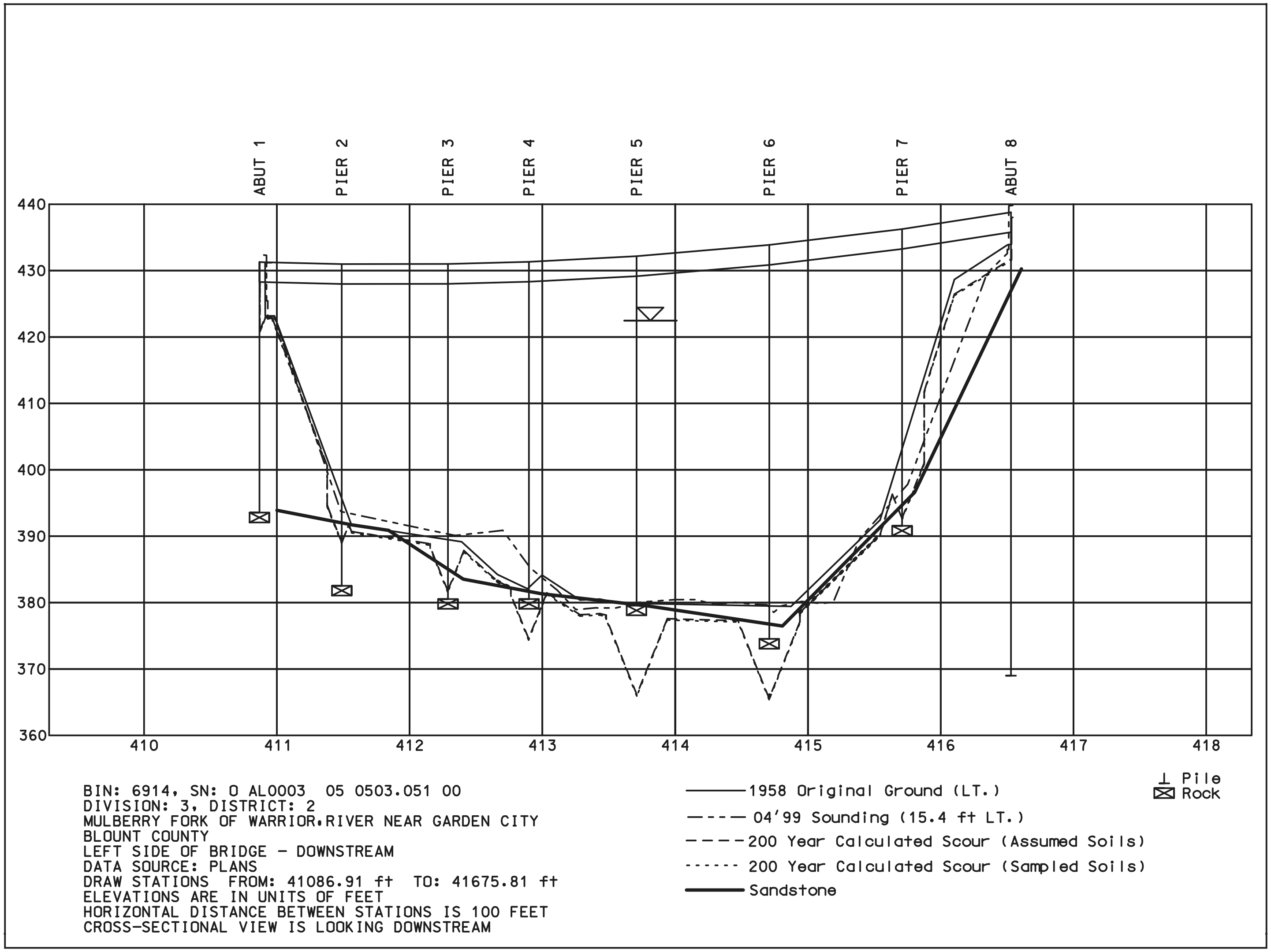


Figure 27. Sounding profiles and scour evaluations for the main channel bridge at the Mulberry Fork site near Garden City

The calculated scour depths in the left overbank were consistent with the scour observed in the field. This is due to the noncohesive sandy material. The scour depths were not reached throughout the rest of the channel due to the rock foundation.

TALLADEGA CREEK NEAR TALLADEGA

The site is located in Talladega County. There is one main channel bridge. Two views of the site are shown in Figures 28 and 29. The detailed site information is located in Appendix A. In 1951 the bridge experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank and main channel to be a consolidated clay. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.019 mm (0.000062 ft) for the overbank soil and D_{50} = 0.02 mm (0.000066 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as a silty clay. Figure 30 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain size indicates 6 to 9 feet of combined contraction and pier scour and 18 to 21 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 9 to 28 feet of combined contraction and pier scour and 26 to 40 feet of combined contraction and abutment scour. The recent sounding indicates the channel has

Figure 28. Talladega Creek near Talladega (BIN# 4470); view from the upstream side looking west toward abutment 1.

Figure 29. Talladega Creek near Talladega (BIN# 4470); view from the upstream side looking northeast toward abutment 8.

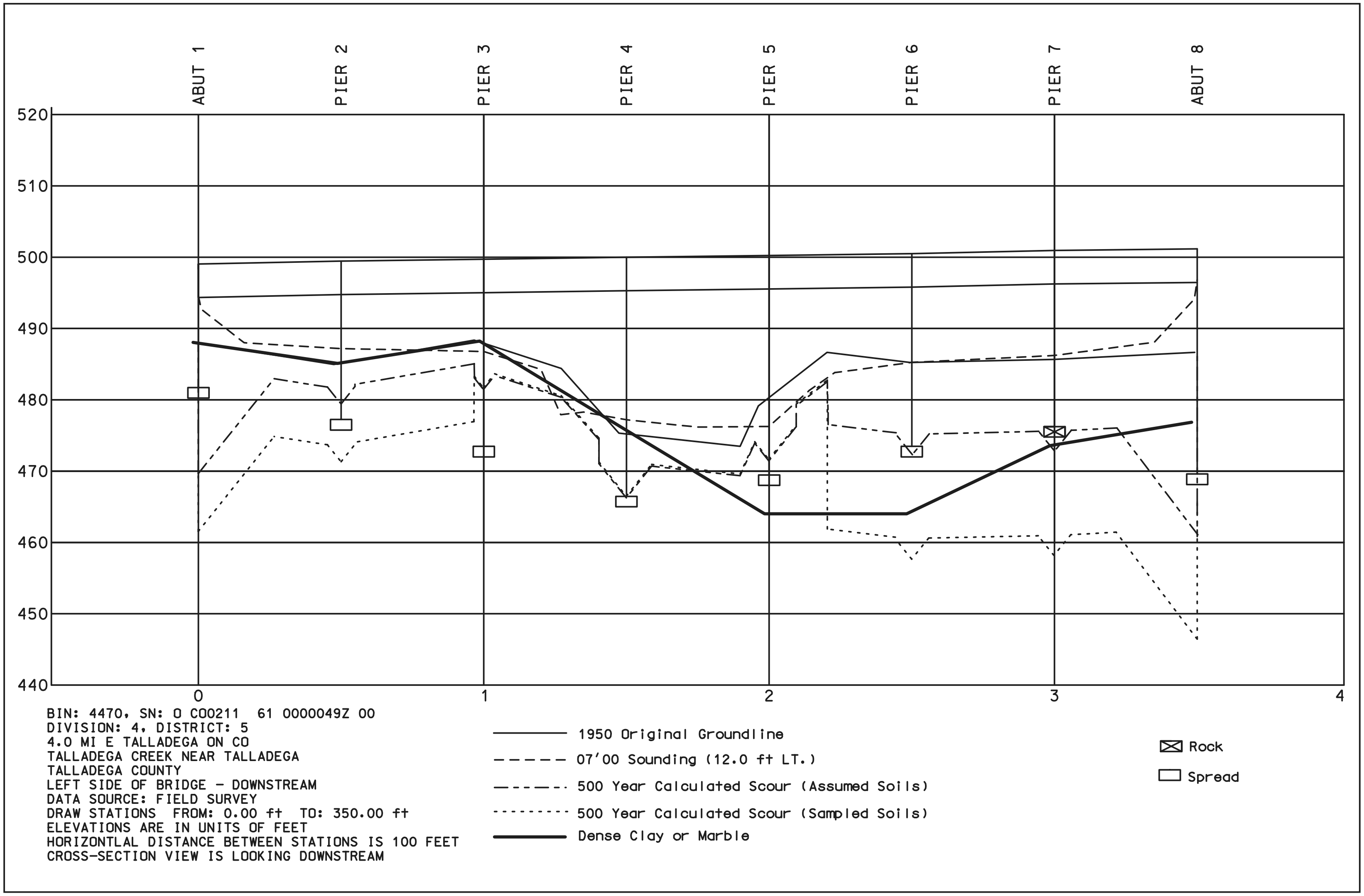


Figure 30. Sounding profiles and scour evaluations for the main channel bridge at the Talladega Creek site near Talladega

widened slightly and has aggraded 2 to 3 feet in the channel. No scour was observed at the site but signs of erosion existed between piers #5 and #6 and had been deterred by riprap.

The reason these calculated scour depths were not reached anywhere at the site is due to the cohesive nature of the soil. Even if the top cohesive layer were scoured away, a marl is present which would provide even more resistance to scour than the cohesive material.

OSANIPPA CREEK NEAR FAIRFAX

The site is located in Chambers County. There is one main channel bridge. Two views of the site are shown in Figures 31 and 32. The detailed site information is located in Appendix A. In 1961 the bridge experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was loose sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.052 mm (0.00017 ft) for the overbank soil and D_{50} = 1.1 mm (0.0036 ft) for the main channel soil. The soil in the overbank was classified as lean clay and the soil in the main channel was classified as a poorly graded sand. Figure 33 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour

Figure 31. Osanippa Creek near Fairfax (BIN# 615); view from the upstream side looking south toward abutment 1.

Figure 32. Osanippa Creek near Fairfax (BIN# 615); view from the upstream side looking east toward abutment 9.

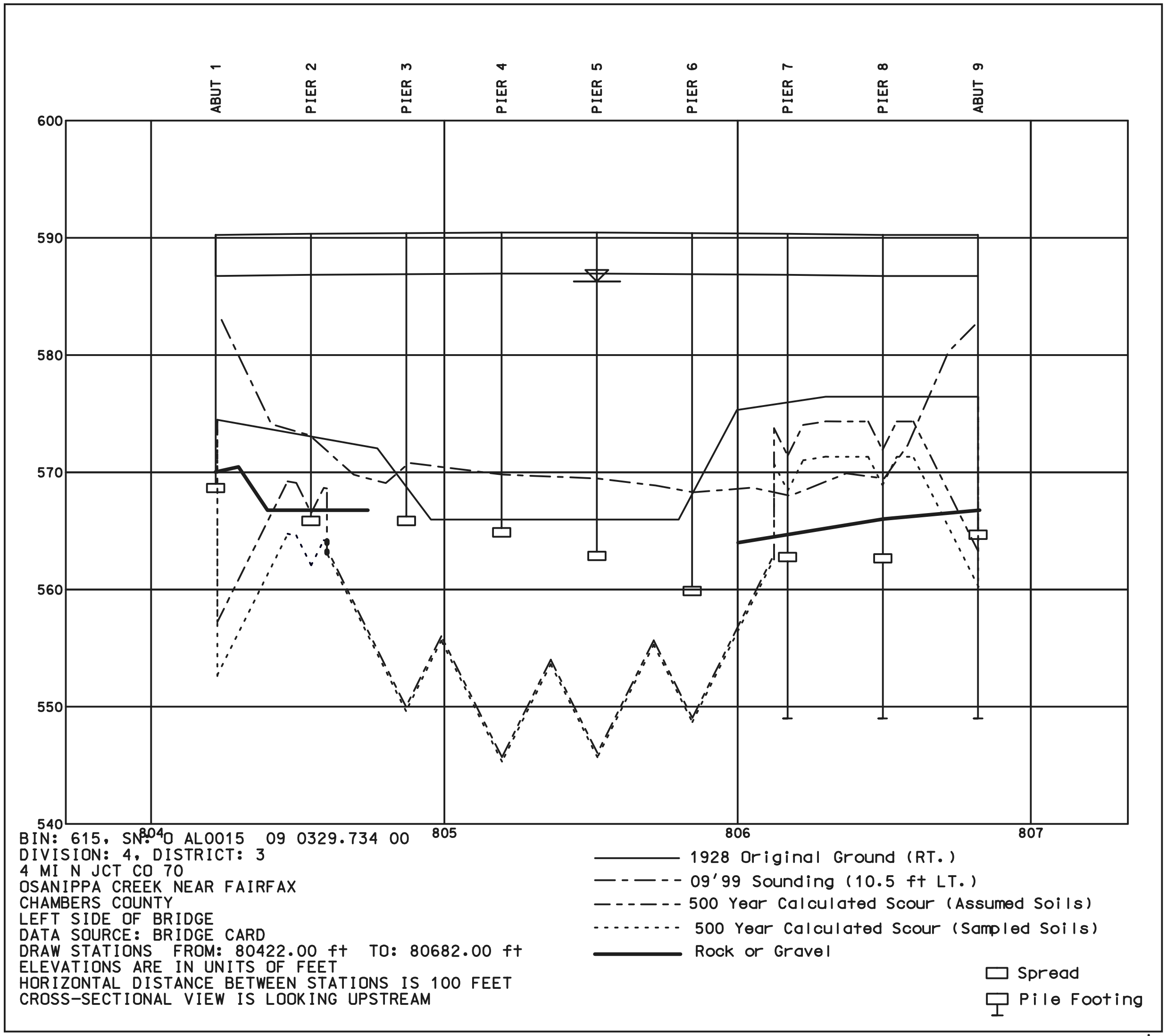


Figure 33. Sounding profiles and scour evaluations for the main channel bridge at the Osanippa Creek site near Fairfax

with the assumed grain size indicates 4 to 21 feet of combined contraction and pier scour and 13 to 17 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 7 to 21 feet of combined contraction and pier scour and 16 to 22 feet of combined contraction and abutment scour. The recent sounding indicates the channel has widened significantly and has degraded on the left and right overbanks approximately 4 to 8 feet and has aggraded in the channel approximately 4 feet. The site inspection revealed the channel is skewed greatly under the bridge almost running parallel causing the soundings to indicate the main channel to have widened. Scour observed in the field consisted of 2 to 3 feet deep combined local scour and contraction scour in the main channel.

The reason these calculated scour depths were exceeded on the left overbank is due to the shift in the channel. The basis for the scour depths not being reached throughout the rest of the site is due to the cohesive nature of the soil. Even if the top cohesive layer were scoured away, rock is present which would provide even more resistance to scour than the cohesive material.

SUCARNOOCHEE RIVER AT LIVINGSTON

The site is located in Sumter County. There is one main channel bridge and one relief bridge. Two views of the site are shown in Figures 34 and 35. The detailed site information is located in Appendix A. In 1979 the bridges experienced an approximate 200-year flood. Scour calculations found in Appendix B were performed for this event on the bridges as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the

Figure 34. Sucarnoochee River at Livingston (BIN# 2911);
view from pier 22 looking south toward abutment 1.

Figure 35. Sucarnoochee River at Livingston (BIN# 2911);
view from the upstream side looking south toward
abutment 1.

channel. Visual observations indicated the soil in the overbank and main channel to be clay. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.060 mm (0.00020 ft) for the overbank soil and D_{50} = 0.055 mm (0.00018 ft) for the main channel soil. The soil in the overbank was classified as silt and the soil in the main channel was classified as a lean clay. Figures 36 and 37 show the results of the calculated scour depths plotted on the ABIMS profiles for the main channel and the relief bridges. The plots consist of the original groundline from the plans, core boring depths of a marl layer, a recent sounding, a sounding from the measured flood, and calculated scour with the assumed grain size and with the measured grain size for the measured event.

The calculated scour at the main channel with the assumed grain size indicates 6 to 11 feet of combined contraction and pier scour and 24 to 26 feet of contraction and abutment scour. The calculated scour at the main channel with the sampled soil sizes indicates 6 to 21 feet of combined contraction and pier scour and 12 to 36 feet of combined abutment and contraction scour. The recent sounding indicates these depths were not reached for the assumed and sampled soils calculations. Scour observed in the field consisted of erosion just upstream of pier #6 in the channel bank. The reason these calculated scour depths were not reached for the sampled soil is due to the cohesive nature of the soil.

The calculated scour at relief #1 for the assumed grain size indicates 24 feet of combined contraction and pier scour and 38 to 40 feet combined contraction and abutment scour. The calculated scour at relief #1 for the sampled grain size indicates 33 feet of combined contraction and pier scour and 47 to 49 feet of combined contraction

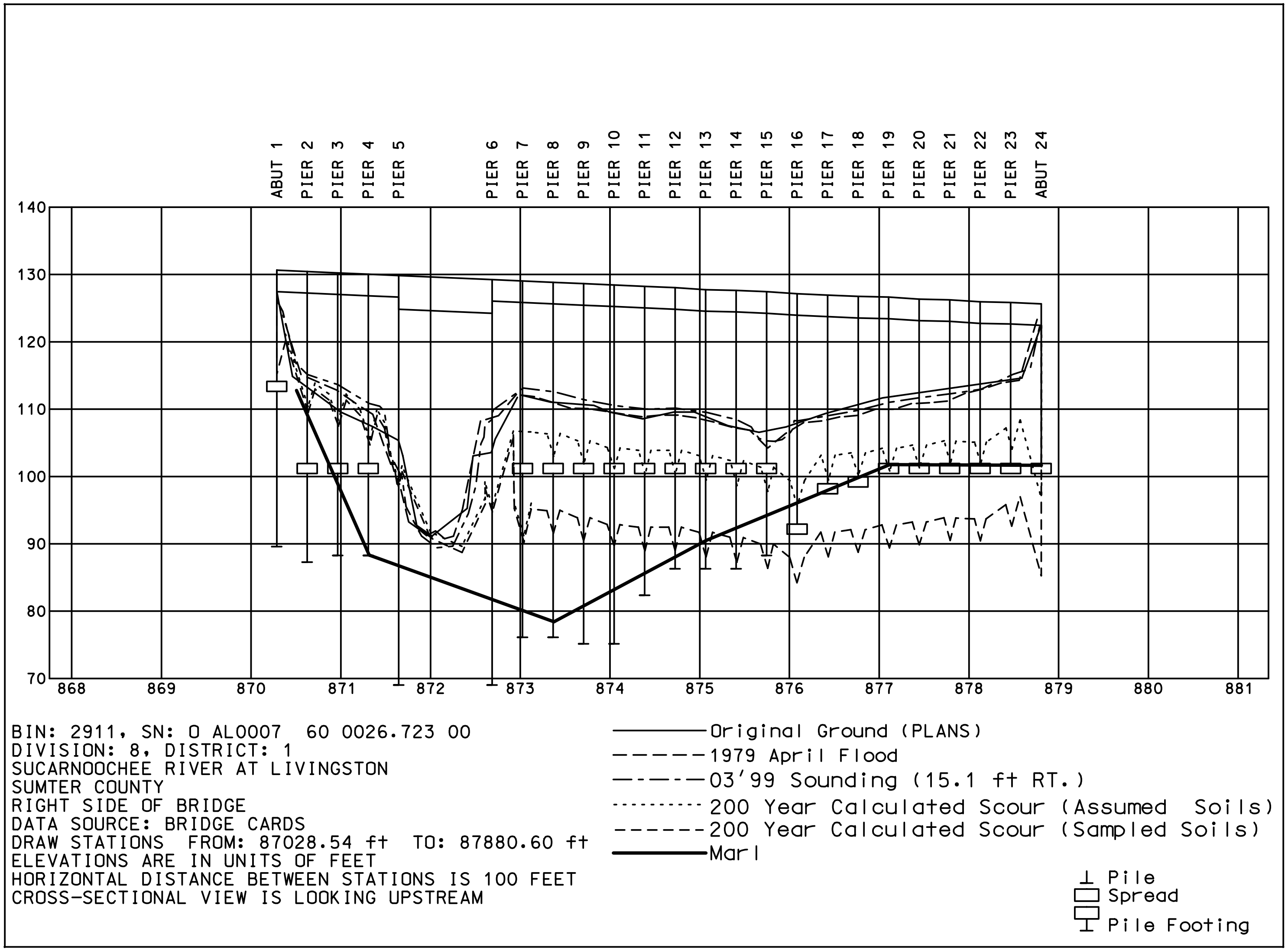


Figure 36. Sounding profiles and scour evaluations for the main channel bridge at the Sucarnoochee River site at Livingston

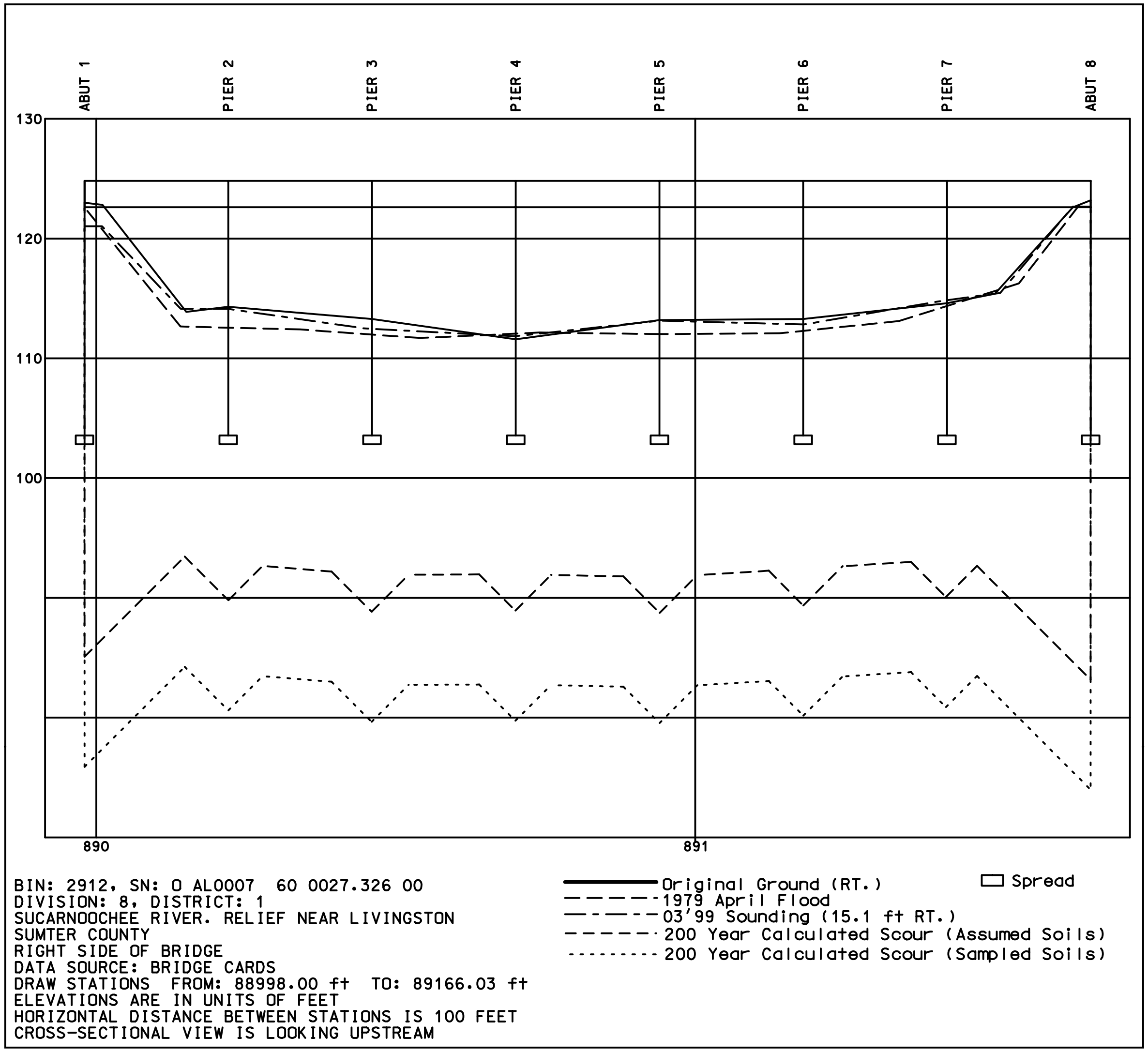


Figure 37. Sounding profiles and scour evaluations for relief bridge 1 at the Sucarnoochee River site at Livingston

and abutment scour. Scour observed in the field consisted mainly of 1 to 2 feet deep local scour at pier #2 through #5. The calculated scour depths were not reached for either soils condition due to the cohesive nature of the soil.

TUCKABUM CREEK NEAR BUTLER

The site is located in Choctaw County. There is one main channel bridge and three relief bridges. Two views of the site are shown in Figures 38 and 39. The detailed site information is located in Appendix A. In 1964 the bridges experienced an approximate 200-year flood. Scour calculations found in Appendix B were performed for this event on the main channel bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The relief bridges have been replaced with new structures. The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel of the main channel bridge. Visual observations indicated the soil in the overbank to be a silty sandy clay and the main channel to be a sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.049 mm (0.00016 ft) for the overbank soil and D_{50} = 0.29 mm (0.00094 ft) for the main channel soil. The soil in the overbank was classified as a lean clay and the soil in the main channel was classified as a poorly graded sand. Figure 40 shows the results of the calculated scour depths plotted on the ABIMS profiles for the main channel. The plots consist of original groundline from the plans, core boring depths of a marl layer, a recent sounding, and calculated scour with the assumed grain size and with the measured grain size for the measured event.

Figure 38. Tuckabum Creek near Butler (BIN# 1095); view from the downstream side looking north toward abutment 32.

Figure 39. Tuckabum Creek near Butler (BIN# 1095); facing south toward abutment 1 view of scour at pier 2.

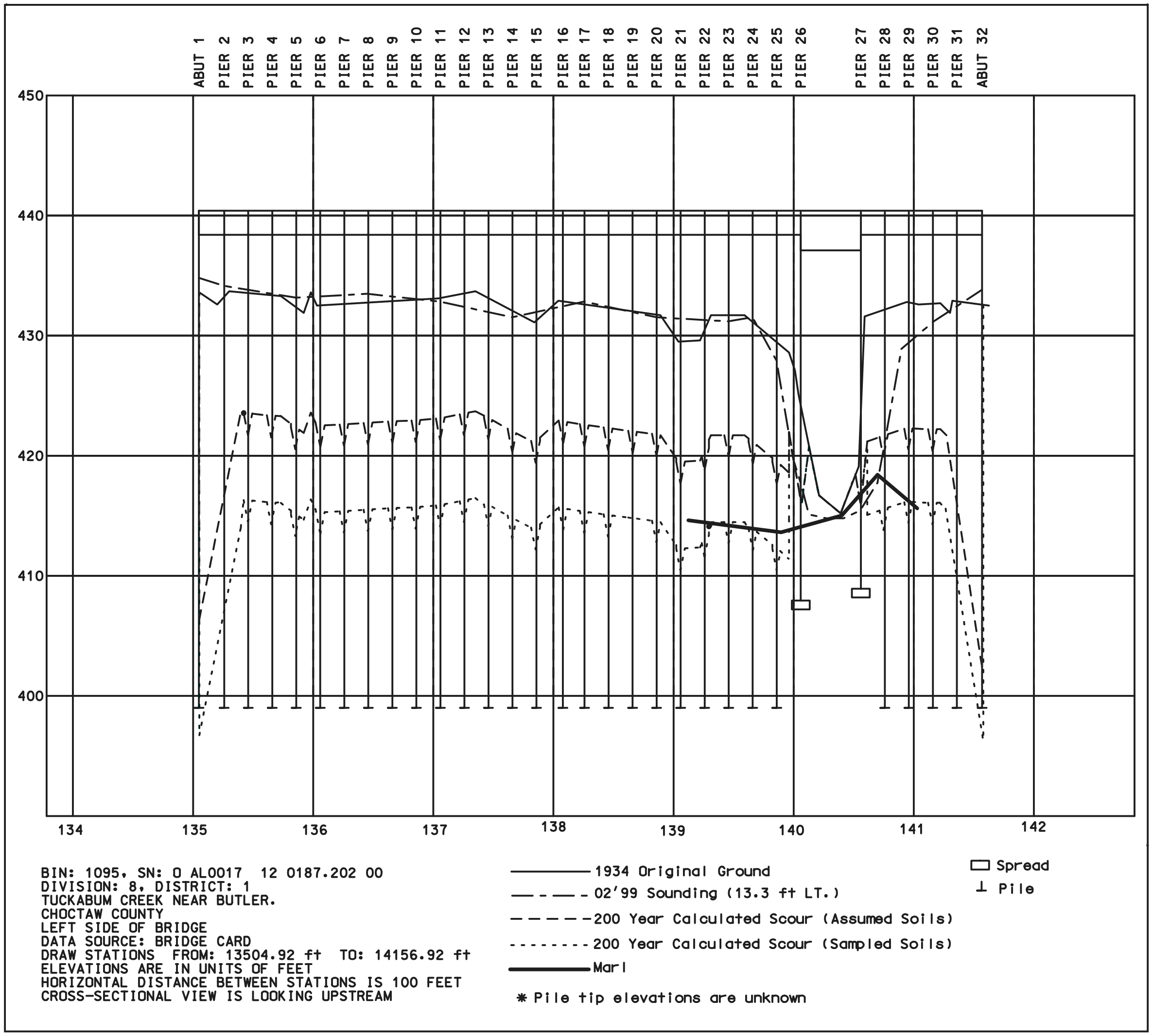


Figure 40. Sounding profiles and scour evaluations for the main channel bridge at the Tuckabum Creek site near Butler

The calculated scour at the main channel with the assumed grain size indicates 7 to 22 feet of combined contraction and pier scour and 36 to 37 feet of contraction and abutment scour. The calculated scour at the main channel with the sampled soil sizes indicates 7 to 15 feet of combined contraction and pier scour and 30 feet of combined abutment and contraction scour. The recent sounding indicates these depths were not reached for the assumed and sampled soils calculations. Scour observed in the field consisted of 1 to 2 feet of scour from piers #2 to #25. The reason these calculated scour depths were not reached for the sampled soil is due to the cohesive nature of the soil.

TURKEY CREEK AT KIMBROUGH

The site is located in Wilcox County. There is one main channel bridge and two relief bridges. Two views of the site are shown in Figures 41 and 42. The detailed site information is located in Appendix A. In 1962 the bridge experienced an approximate 200-year flood. Scour calculations found in Appendix B were performed for this event on the bridges as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be consolidated clay while the soil from the channel was silt. Sand was present in the overbank of the main channel as well. The soils were analyzed at the Auburn University Soils Laboratory and the results were D_{50} = 0.036 mm (0.00012 ft) for the overbank soil and D_{50} = 0.067 mm (0.00022 ft) for the main channel soil. The soil in the overbank was classified as silty clay and the soil in the main channel was classified as a silt. Figures 43, 44, and 45 show the results of the calculated scour depths plotted on the ABIMS profiles for the main

Figure 41. Turkey Creek at Kimbrough, Relief Bridge 2 (BIN# 2266); view from pier 10 looking south toward abutment 1.

Figure 42. Turkey Creek at Kimbrough, Relief Bridge 2 (BIN# 2266); view from the downstream side looking southwest at abutment 1.

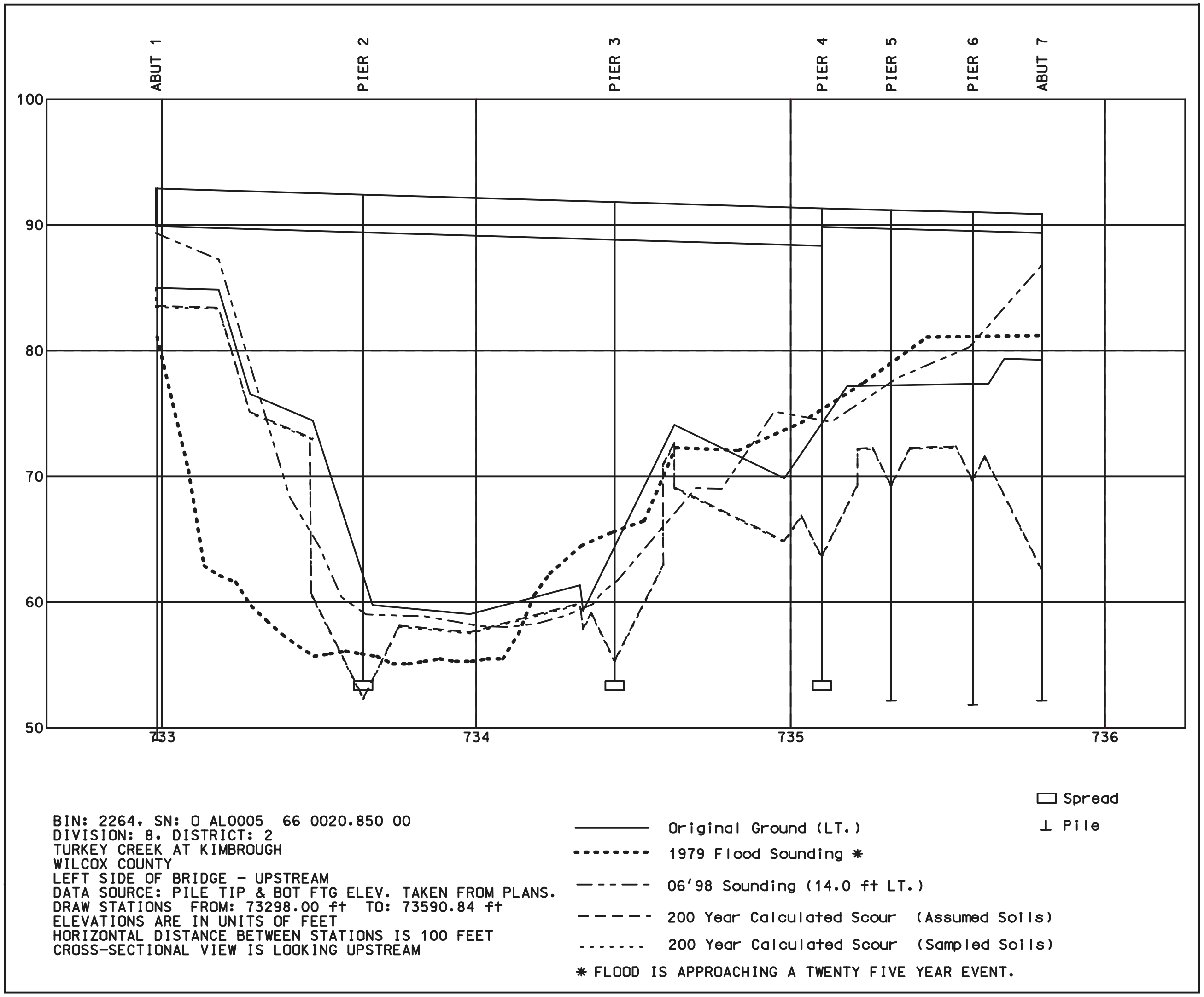


Figure 43. Sounding profiles and scour evaluations for the main channel bridge at the Turkey Creek site at Kimbrough

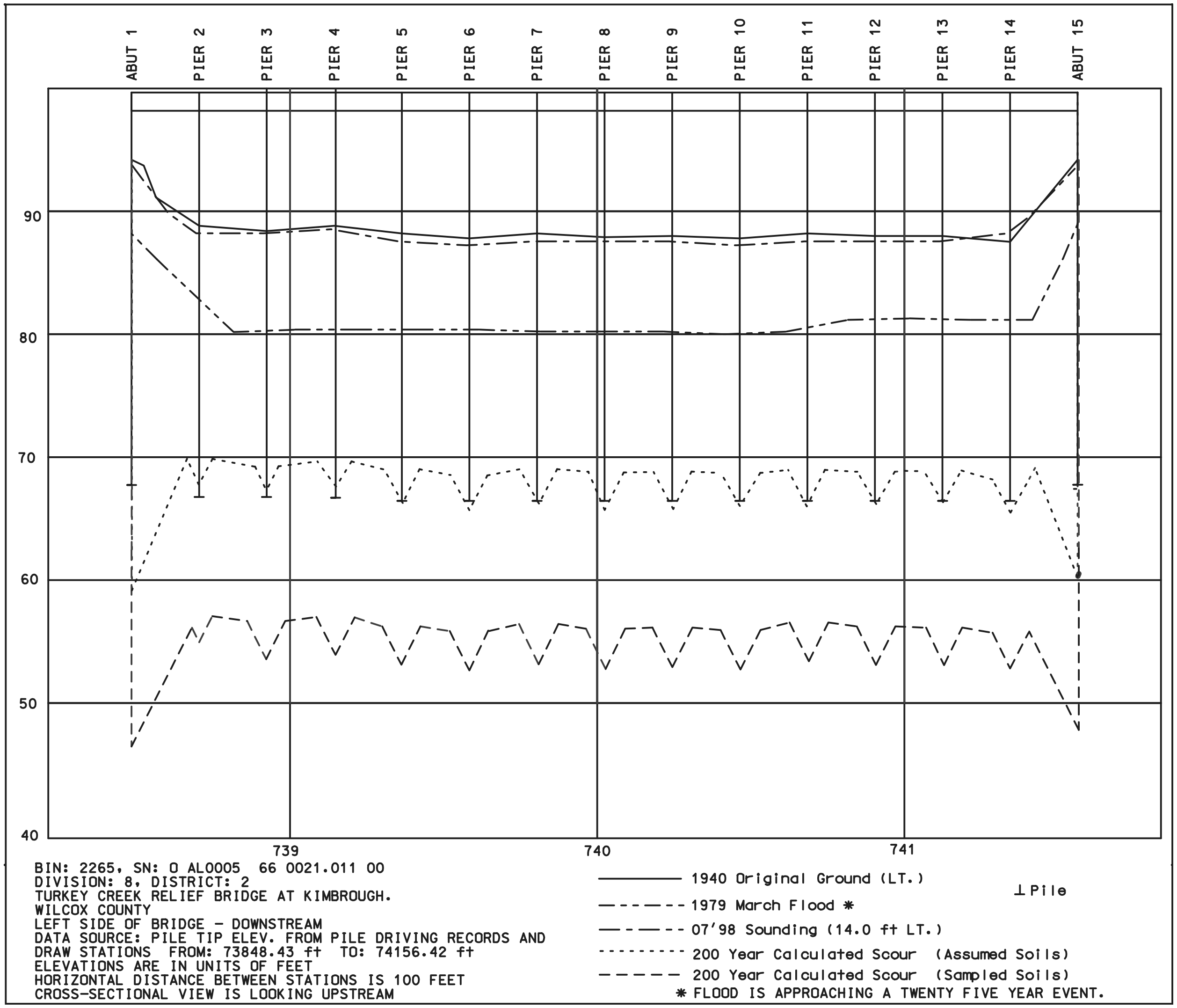


Figure 44. Sounding profiles and scour evaluations for relief bridge 1 at the Turkey Creek site at Kimbrough

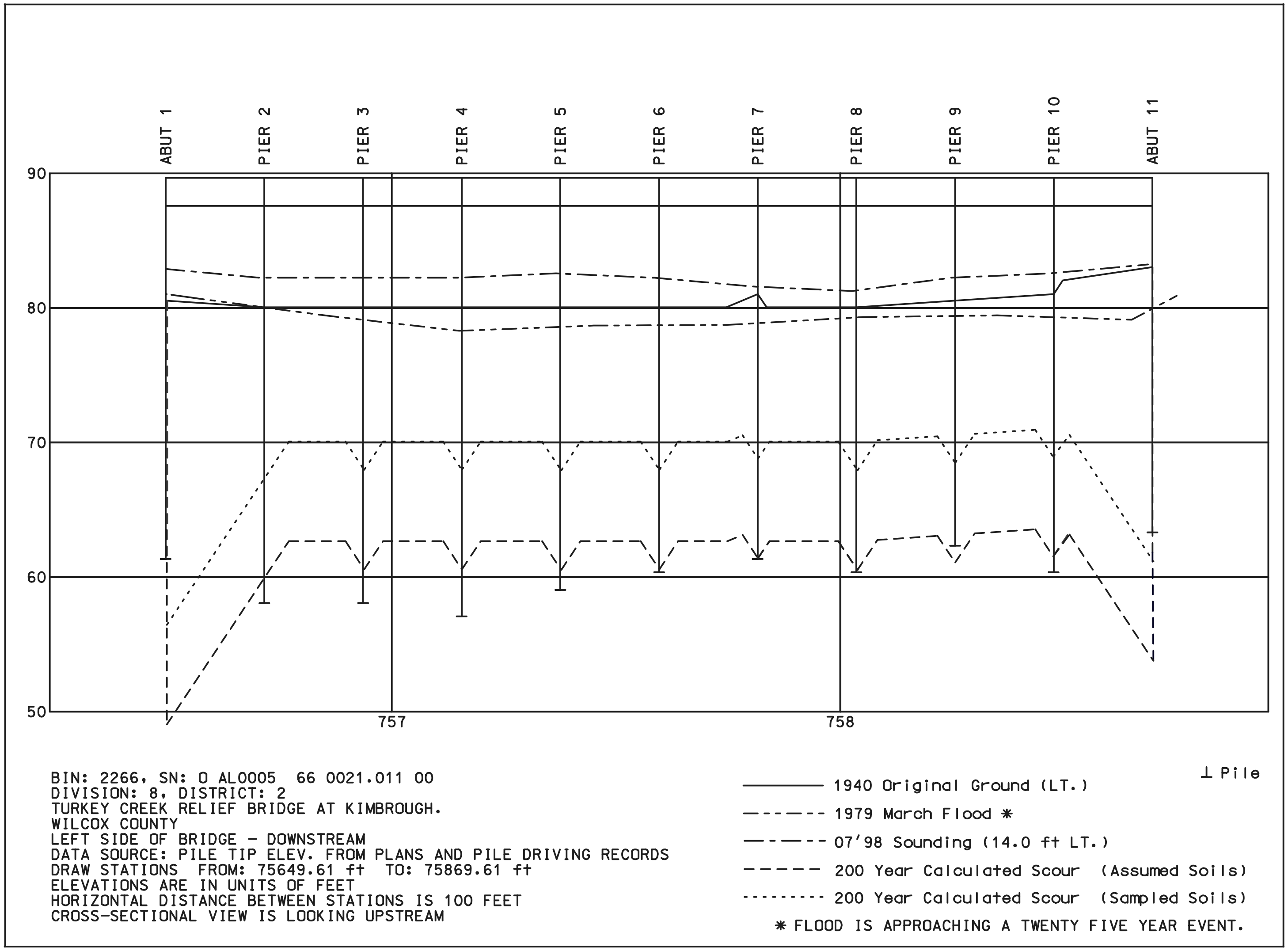


Figure 45. Sounding profiles and scour evaluations for relief bridge 2 at the Turkey Creek site at Kimbrough

channel and both relief bridges. The plots consist of the original groundline from the plans, a recent sounding, a flood measurement from 1979, and calculated scour with the assumed grain size and with the measured grain size for the measured event.

The calculated scour at the main channel with the assumed grain size indicates 8 to 11 feet of combined contraction and pier scour and 0 to 17 feet of contraction and abutment scour. The recent sounding and site inspection determined these depths were not reached in the overbank or in the channel. Due to livebed conditions in the overbank and main channel, there was no difference in scour depths for the assumed and sampled soils. Scour observed in the field consisted of erosion at abutment #1 embankment and a scour hole at pier #3. The flood measurement sounding also indicates a widening of the channel at abutment #1 during the flood, but seems to have been filled by riprap.

The calculated scour at relief #1 for the assumed grain size indicates 22 feet of combined contraction and pier scour and 34 to 35 feet combined contraction and abutment scour. The calculated scour at relief #1 for the sampled grain size indicates 35 feet of combined contraction and pier scour and 47 to 48 feet of combined contraction and abutment scour. Scour observed in the field consisted mainly of 1 to 2 feet deep local scour at pier #2 and 1 foot of scour extending from pier #4 to pier #12. The sounding also indicates very little long-term degradation.

The calculated scour at relief #2 for the assumed grain size indicates 12 feet of combined contraction and pier scour and 21 to 24 feet combined contraction and abutment scour. The calculated scour at relief #2 for the sampled grain size indicates 19 feet of combined contraction and pier scour and 29 to 31 feet of combined contraction and abutment scour. Scour observed in the field consisted of a large scour hole 2 to 3 feet deep located between abutment #1 and pier #2 (Figure 42). Another 1 foot deep scour hole was located at pier #8. The sounding indicates aggradation throughout the bridge. The reason these calculated scour depths were not reached at this site is due to the cohesive nature of the soil.

West Fork Choctawhatchee River at Blue Springs

The site is located in Barbour County. There is one main channel bridge. Two views of the site are shown in Figures 46 and 47. The detailed site information is located in Appendix A. In 1990 the bridge experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on an assumed fine sand. Visual observations indicated the soil in the overbank and in the channel to be a brown sandy clay. Figure 48 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour indicates 6 to 7 feet of combined contraction and pier scour and 3 to 11 feet of contraction and abutment scour. The recent sounding indicates the right overbank has degraded 2 to 3 feet throughout the bridge site. The site inspection revealed the bridge had been replaced. The date on the barrier rail was 2000.

The calculated scour depth was reached at pier #4 due to a shift in the main channel. The calculated scour depths were not reached throughout the rest of the channel due to the cohesive nature of the soil.

Figure 46. West Fork Choctawhatchee River at Blue Springs (BIN# 16717); view from the downstream side looking northwest toward abutment 1.

Figure 47. West Fork Choctawhatchee River at Blue Springs (BIN# 16717); view looking south at pier 3.

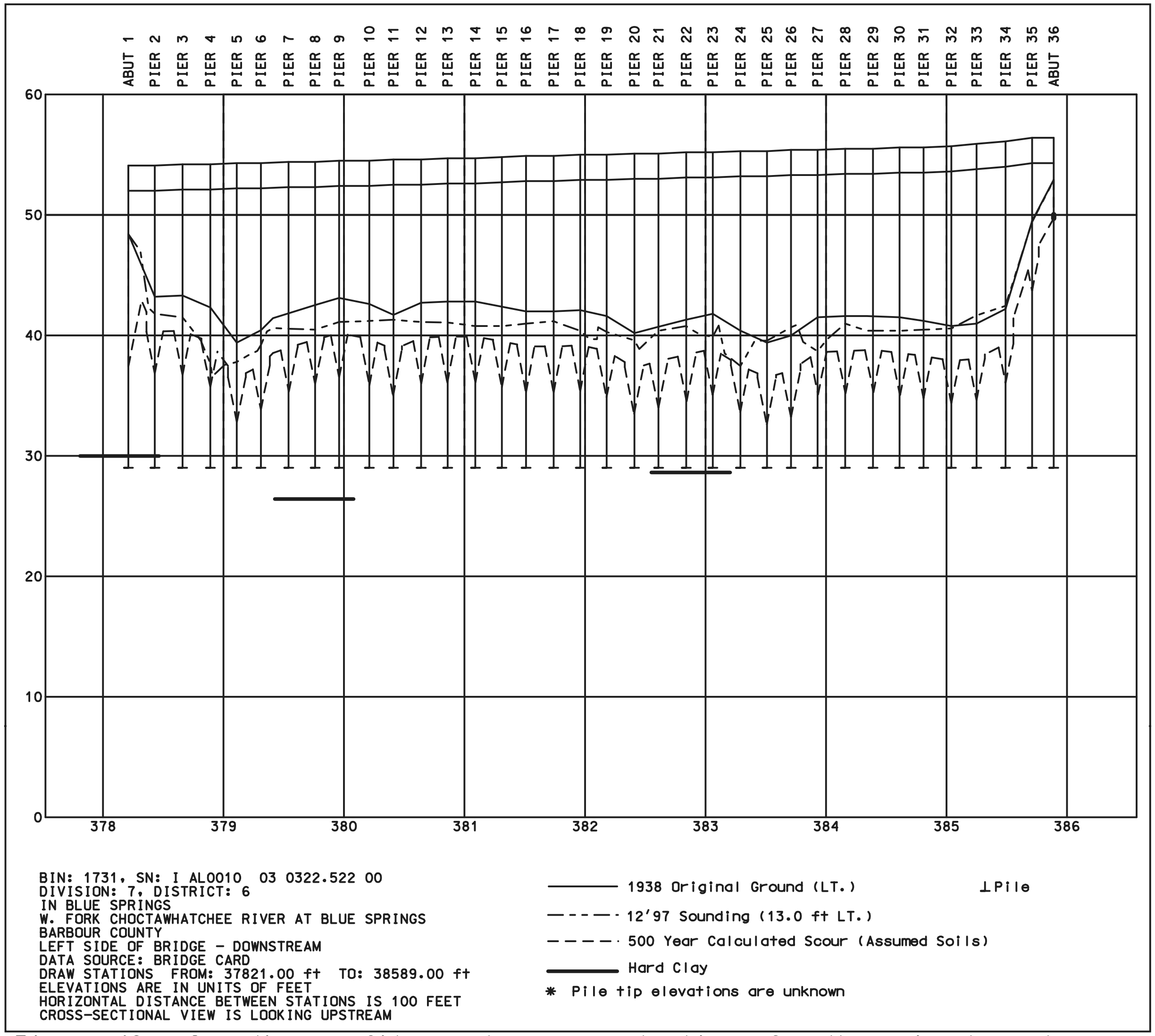


Figure 48. Sounding profiles and scour evaluations for the main channel bridge at the West Fork Choctawhatchee River at Blue Springs

ABBIE CREEK NEAR ABBEVILLE

The site is located in Henry County. There is one main channel bridge. Two views of the site are shown in Figures 49 and 50. The detailed site information is located in Appendix A. In 1953 the bridge experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. A soil sample was obtained from the top layer of the soil on the overbank. Visual observations indicated the soil in the overbank and in the channel to be a brown sandy material. The soil was analyzed at the Auburn University Soils Laboratory and the results were D_{50} = 0.037 mm (0.00012 ft). The soil was classified as a silt. Figures 51a and 51b show the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a recent sounding. The calculated scour with the assumed grain sizes indicates 6 to 10 feet of combined contraction and pier scour and 25 to 34 feet of contraction and abutment scour. The calculated scour with the sampled grain sizes indicates 6 to 17 feet of combined contraction and pier scour and 31 to 41 feet of contraction and abutment scour. The recent sounding does not indicate any appreciable scour. The site inspection revealed the bridge had scour countermeasures in-place (Figures 49 and 50). Riprap was located at abutment #1 and from pier #4 to pier #8 and from pier #22 to pier #26. A large scour hole 2 to 3 feet deep was located under the bridge from pier #2 to pier #4. One to 2 feet of pier scour was also present from pier #14 to pier #22 and evidence of an old scour hole that had been filled was located at abutment #26.

Figure 49. Abbie Creek near Abbeville (BIN# 2540);
view from the downstream side looking west
toward abutment 1.

Figure 50. Abbie Creek near Abbeville (BIN# 2540);
view from the upstream side looking west
toward abutment 1.

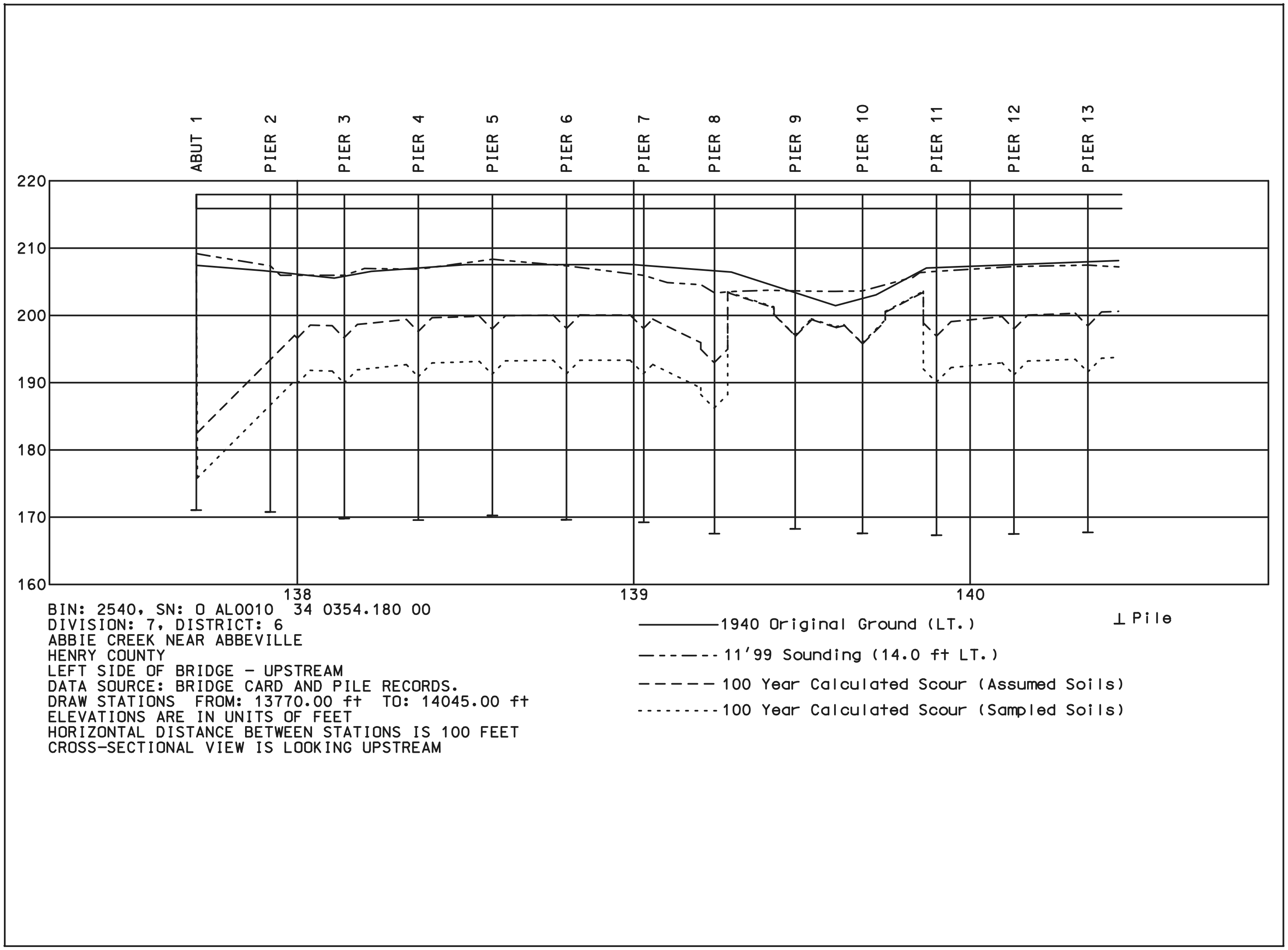


Figure 51a. Sounding profiles and scour evaluations for the main channel bridge at the Abbie Creek site near Abbeville (left side of bridge)

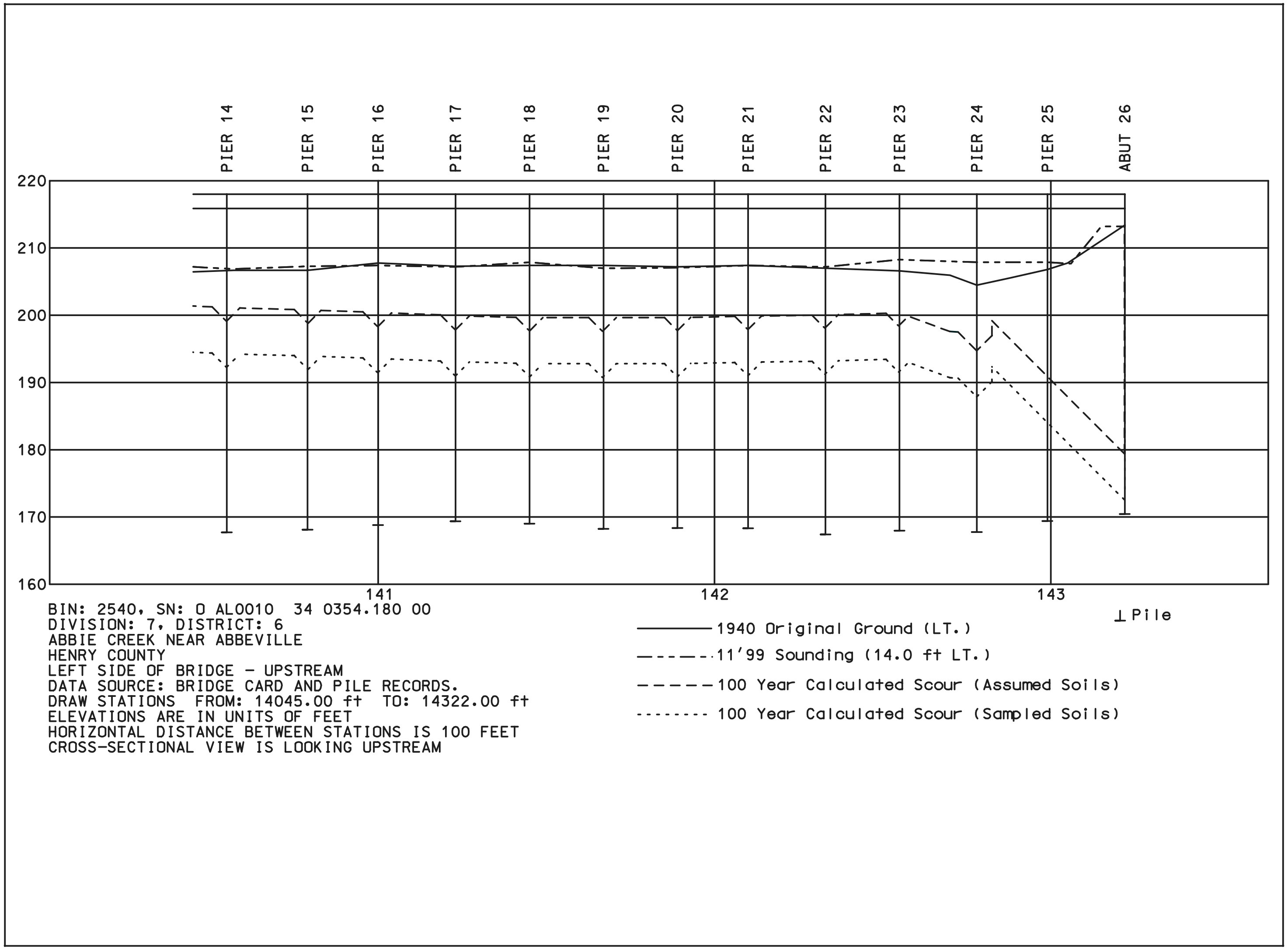


Figure 51b. Sounding profiles and scour evaluations for the main channel bridge at the Abbie Creek site near Abbeville (right side of bridge)

It is not known if the calculated scour depths were reached throughout the bridge, but it is apparent that the bridge has experienced scour damage. The soil, a silty material, is noncohesive which may be the reason for the excessive scour damage.

PEA RIVER NEAR ARITON

The site is located in Dale County. There are two parallel main channel bridges and two parallel relief bridges. Two views of the site are shown in Figures 52 and 53. The detailed site information is located in Appendix A. In 1990 the bridges experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridges as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel of the main channel bridges and from the overbank of the relief bridges. Visual observations indicated the soils to be sand. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.18 mm (0.00059 ft) for the overbank soil and D_{50} = 0.19 mm (0.00062 ft) for the main channel soil at the main channel bridges and D_{50} = 0.031 mm (0.0001 ft) for the overbank soil at the relief bridges. The soil in the overbank and main channel was classified as a poorly graded sand from the main channel bridges and classified as a lean clay from the relief bridges. Figures 54 and 55 show the results of the calculated scour depths plotted on the ABIMS profiles for the main channel and the relief bridges. The plots consist of the original groundlines from the plans, core boring depths of a marl layer, a recent sounding, calculated scour with the assumed grain size and with the measured grain size for the measured event, and a sounding from the 1975 flood.

Figure 52. Pea River near Ariton (BIN# 5573); view from the upstream side of Bridge B looking northeast toward historic structure.

Figure 53. Pea River near Ariton (BIN# 5573); view from the upstream side of historic structure looking south.

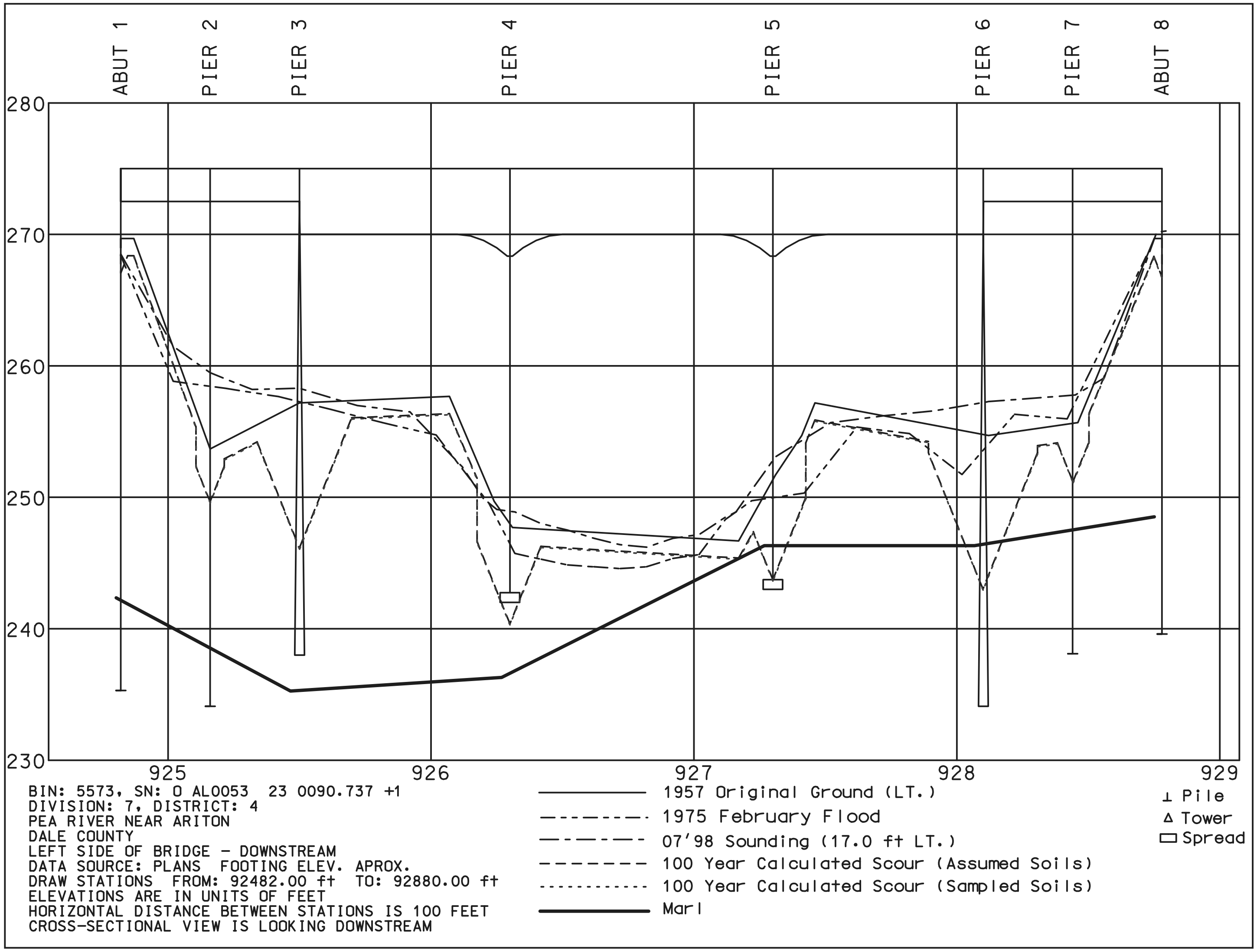


Figure 54. Sounding profiles and scour evaluations for the main channel bridge at the Pea River site near Ariton

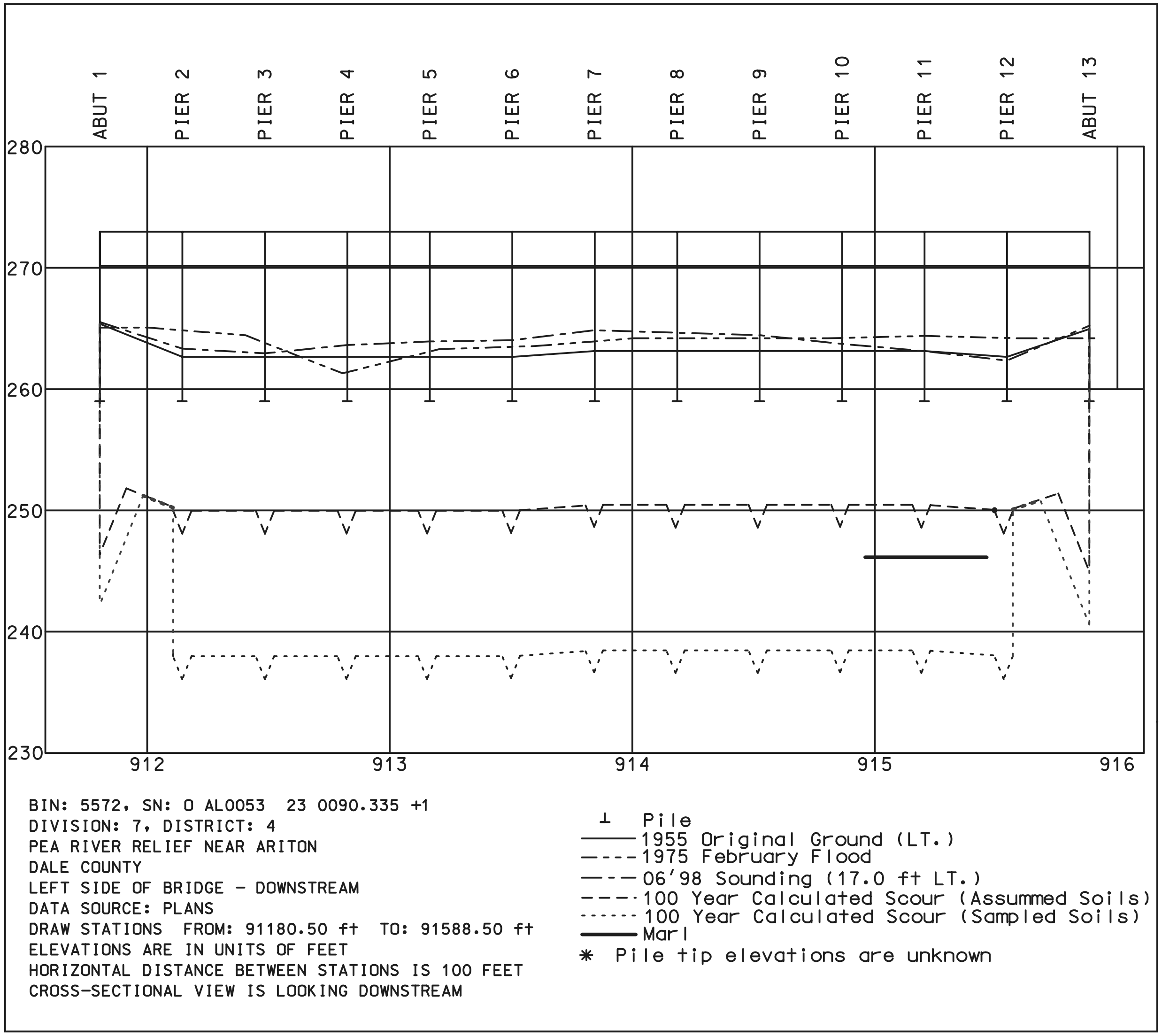


Figure 55. Sounding profiles and scour evaluations for relief bridge 1 at the Pea River site near Ariton

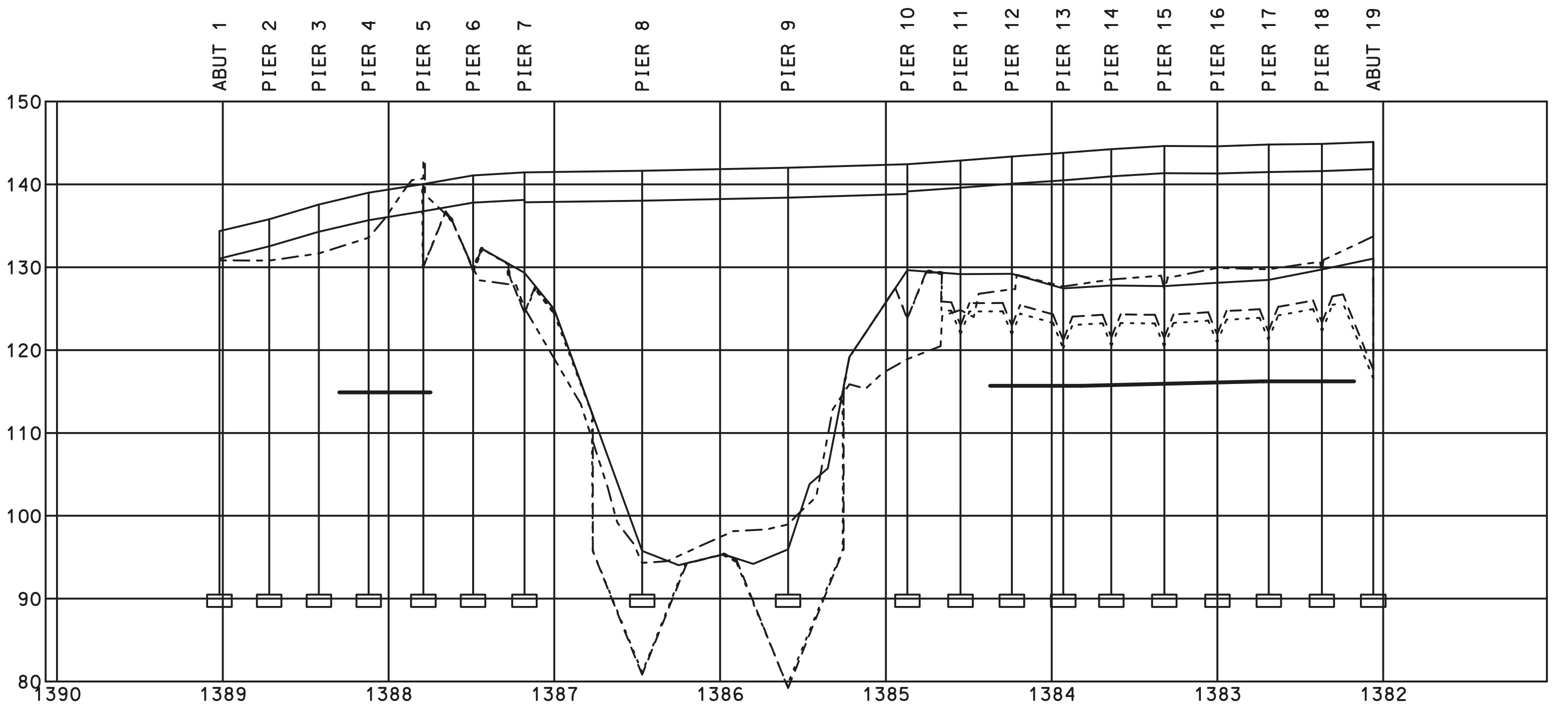
The calculated scour at the main channel bridges with the assumed grain size and with the sampled grain size indicates 4 to 12 feet of combined contraction and pier scour and 3 feet of contraction and abutment scour. The calculated scour at the relief bridges with the assumed grain soil sizes indicates 15 feet of combined contraction and pier scour and 19 to 20 feet of combined abutment and contraction scour. The recent sounding indicates these scour depths were not reached for the assumed and sampled soils calculations. No significant scour was observed in the field although riprap was in-place showing evidence that scour has occurred in the past at both the main channel bridges and throughout the relief bridges. It is unknown to what extent the scour was at this site.

PEA RIVER AT ELBA

The site is located in Coffee County. There is one main channel bridge. Two views of the site are shown in Figures 56 and 57. The detailed site information is located in Appendix A. In 1990 the bridge experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be a sand and in the channel to be a marl. The soils were analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.46 mm (0.0015 ft) for the overbank soil and D_{50} = 0.041 mm (0.00014 ft) for the main channel soil. The soil in the overbank was classified as a poorly graded sand and the soil in the main channel was classified as a silt. Figure 58 shows the results of the calculated scour depths plotted on the Alabama Bridge

Figure 56. Pea River at Elba (BIN# 6434); view from the downstream side looking east toward abutment 19.

Figure 57. Pea River at Elba (BIN# 6434); view from the upstream side looking southwest toward abutment 1.



BIN: 6434, SN: 0 AL0012 16 0265.405 00
DIVISION: 7, DISTRICT: 2
PEA RIVER AT ELBA
COFFEE COUNTY
LEFT SIDE OF BRIDGE - UPSTREAM
DATA SOURCE: PLANS F-91(4) ASSUMED ELEV.
DRAW STATIONS FROM: 138206.00 ft TO: 138902.00 ft
ELEVATIONS ARE IN UNITS OF FEET
HORIZONTAL DISTANCE BETWEEN STATIONS IS 100 FEET
CROSS-SECTIONAL VIEW IS LOOKING UPSTREAM

1930 Original Ground (LT.)
10'98 Sounding (17.0 ft LT.)
100 Year Calculated Scour (Assumed)
100 Year Calculated Scour (Sampled)
Marl
Spread

* Dike at Pier #5 added after 1930

** The footing elevations are unknown

Figure 58. Sounding profiles and scour evaluations for the main channel bridge at the Pea River site at Elba

Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, and a recent sounding. The calculated scour with the assumed grain size indicates 3 to 17 feet of combined contraction and pier scour and 10 to 13 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 3 to 17 feet of combined contraction and pier scour and 10 to 14 feet of contraction and abutment scour. The recent sounding indicates the left overbank has degraded 10 feet at pier #10. This exceeds the calculated scour at that point. The downstream sounding does not reflect the erosion. The site inspection revealed the left overbank had experienced erosion and had been filled.

The left overbank had shown signs of scour although it had been filled. The extent of the scour depths is unknown but the erosion is probably due to the noncohesive sandy material. The scour depths were not reached throughout the rest of the channel due to the marl foundation.

CHOCTAWHATCHEE RIVER NEAR NEWTON

The site is located in Dale County. There is one main channel bridge. Two views of the site are shown in Figures 59 and 60. The detailed site information is located in Appendix A. In 1990 the bridge experienced an approximate 500-year flood. Scour calculations found in Appendix B were performed for this event on the bridge as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. Soil samples were obtained from the top layer of the soil on the overbank and from the channel. Visual observations indicated the soil in the overbank to be a sand and in the channel to be a marl. The soils were analyzed at the Auburn University Civil Engineering Soils

Figure 59. Choctawhatchee River near Newton (BIN# 11857); view from the upstream side looking northwest toward abutment 11.

Figure 60. Choctawhatchee River near Newton (BIN# 11857); view from the downstream side looking southeast toward abutment 1.

Laboratory and the results were D_{50} = 0.36 mm (0.0012 ft) for the overbank soil and D_{50} = 0.031 mm (0.00010 ft) for the main channel soil. The soil in the overbank was classified as a poorly graded sand and the soil in the main channel was classified as an elastic silt. Figure 61 shows the results of the calculated scour depths plotted on the Alabama Bridge Information System (ABIMS) profile with the original groundline and a soils boring profile from the plans, a recent sounding, and a sounding from a known flood event. The calculated scour with the assumed grain size indicates 14 to 31 feet of combined contraction and pier scour and 9 to 35 feet of contraction and abutment scour. The calculated scour with the sampled grain size indicates 11 to 22 feet of combined contraction and pier scour and 6 to 27 feet of contraction and abutment scour. The recent sounding indicates the left overbank has degraded 3 to 4 feet at pier #3 and 2 to 3 feet between piers #6 and #7. The site inspection revealed riprap in-place on the upstream right bank extending to the end of the guide bank. The riprap added was a scour countermeasure placed after a large scour hole had formed during the 1990 event.

The scour is probably due to the upper layer of noncohesive sandy material. The calculated scour depths were not reached throughout the rest of the channel due to the hard silt foundation, which provides resistance to scour.

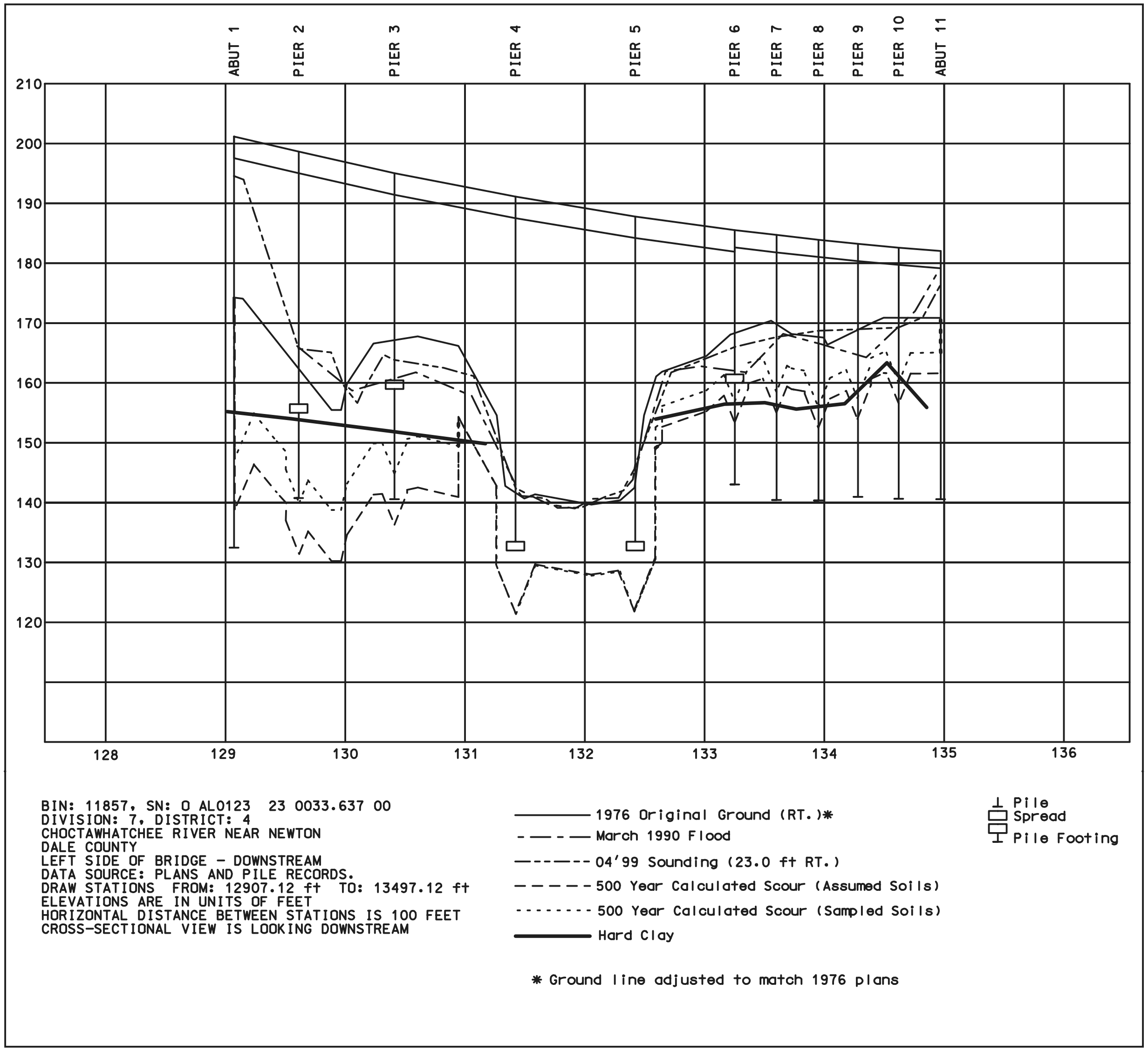


Figure 61. Sounding profiles and scour evaluations for the main channel bridge at the Choctawhatchee River site near Newton

LIGHTWOOD KNOT CREEK AT BABBIE

The site is located in Covington County. There is one main channel bridge and one relief bridge. Two views of the site are shown in Figures 62 and 63. The site information is located in Appendix A. In 1990 the bridges experienced an approximate 100-year flood. Scour calculations found in Appendix B were performed for this event on the bridges as outlined in Hydraulic Engineering Circular 18 (HEC-18). The scour analysis was performed based on both an assumed fine sand and on field soil samples. A soil sample was obtained from the top layer of the soil from the main channel bridge. Visual observations indicated the soils to be sand. The soil was analyzed at the Auburn University Civil Engineering Soils Laboratory and the results were D_{50} = 0.30 mm (0.00098 ft) for the main channel bridge and assumed to be the same for the relief bridge soil. The soil was classified as a poorly graded sand. Figures 64 and 65 show the results of the calculated scour depths plotted on the ABIMS profiles for the main channel bridge and the relief bridge. The plots consist of original groundline from the plans, core boring depths of a sandy clay and sandrock layer, a recent sounding, and calculated scour with the assumed grain size and with the measured grain size for the measured event.

The calculated scour at the main channel bridge with the assumed grain size indicates 3 to 23 feet of combined contraction and pier scour and 40 to 43 feet of contraction and abutment scour. The calculated scour at the main channel bridge with the sampled grain size indicates 3 to 19 feet of combined contraction and pier scour and 34 to 38 feet of contraction and abutment scour. The calculated scour at the relief bridge with the assumed grain soil sizes indicates 18 feet of combined contraction and pier scour and 30 to 32 feet of combined abutment and contraction scour. The calculated scour at the

Figure 62. Lightwood Knot Creek at Babbie (BIN# 12968); view from the downstream side looking west toward abutment 1.

Figure 63. Lightwood Knot Creek at Babbie (BIN# 12968); view from the upstream side looking west toward abutment 1.

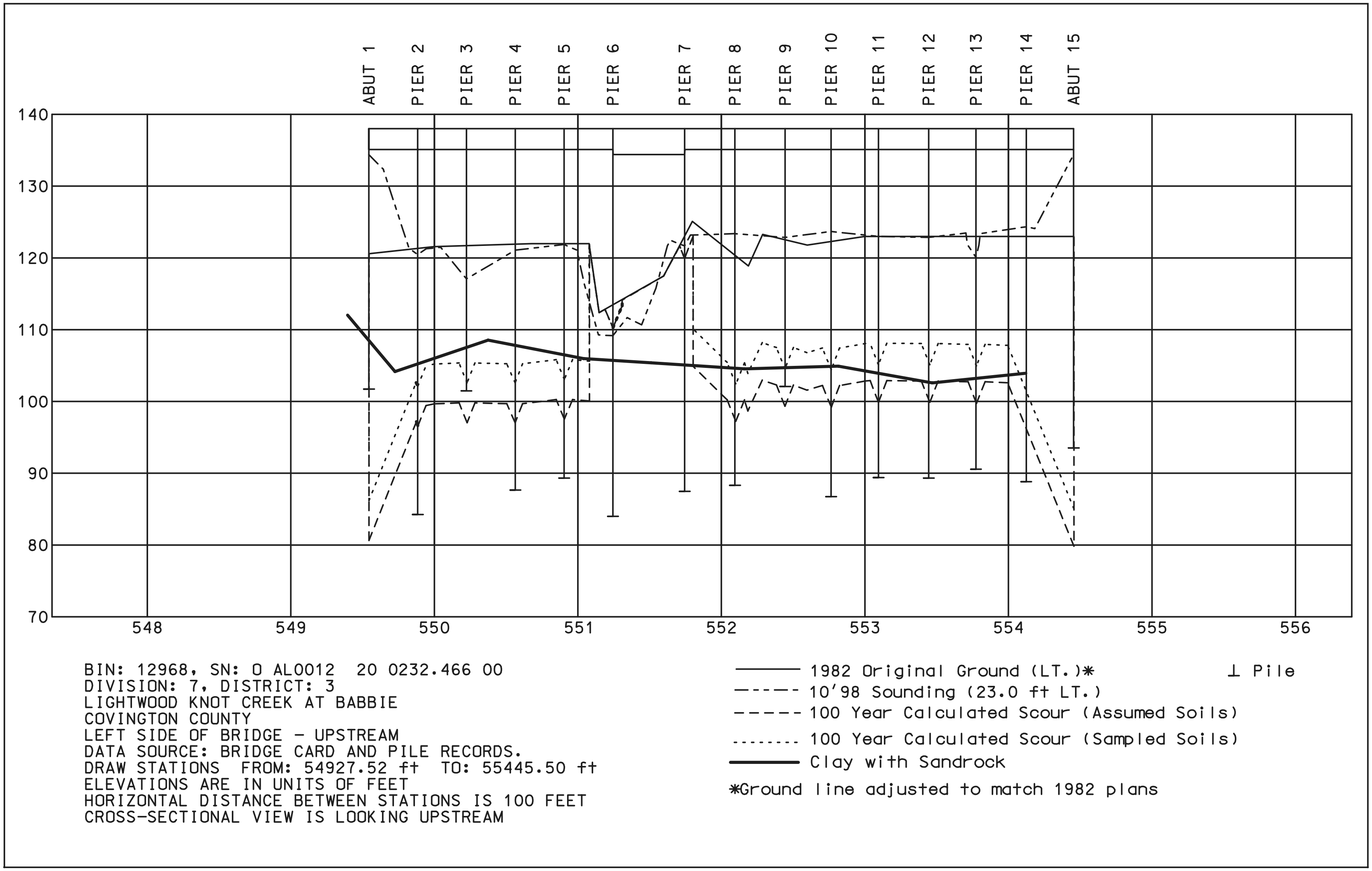


Figure 64. Sounding profiles and scour evaluations for the main channel bridge at the Lightwood Knot Creek site at Babbie

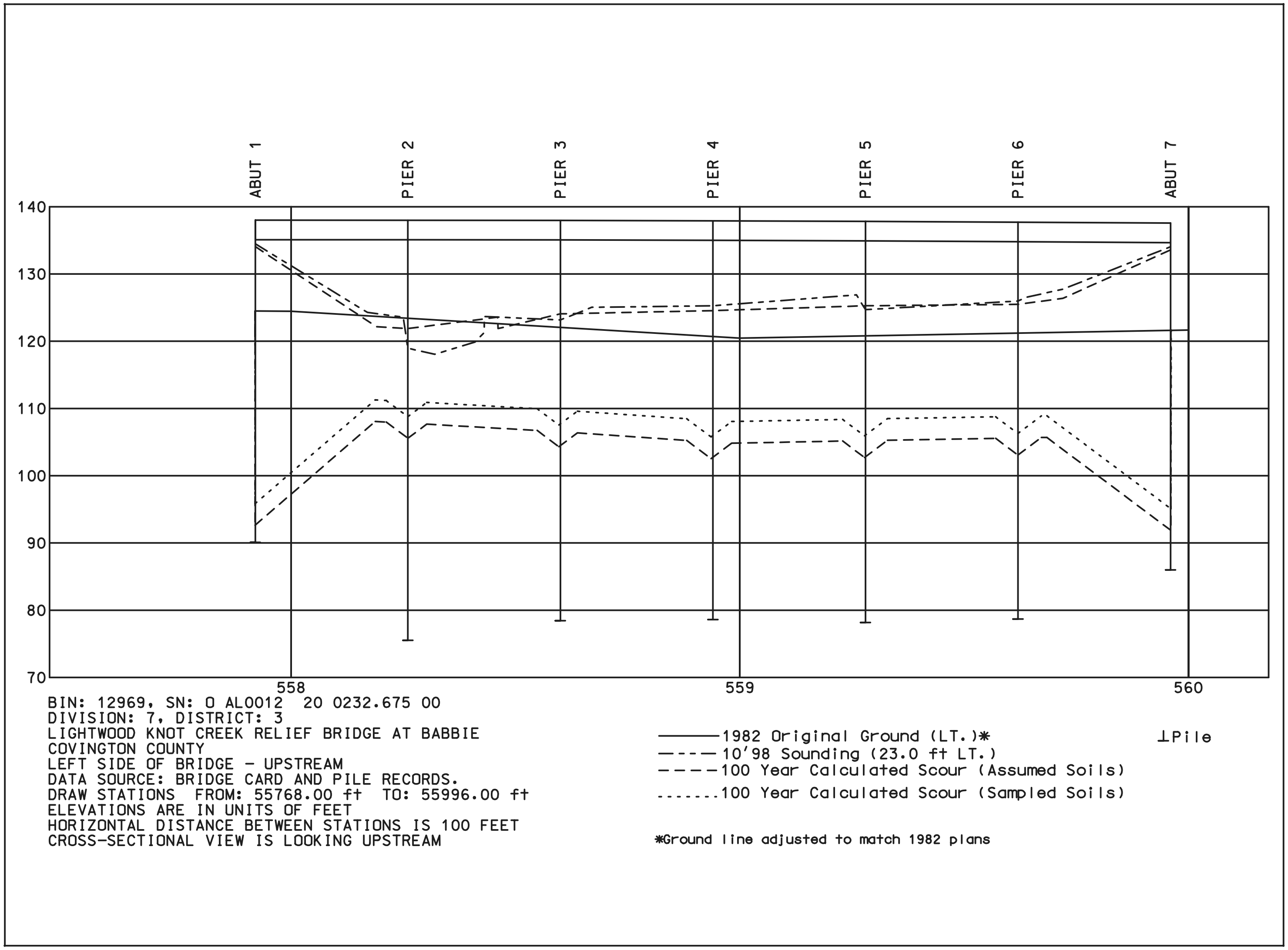


Figure 65. Sounding profiles and scour evaluations for relief bridge 1 at the Lightwood Knot Creek site at Babbie

relief bridge with the sampled grain soil sizes indicates 15 feet of combined contraction and pier scour and 27 to 29 feet of combined abutment and contraction scour. The recent sounding indicates the main channel deepened past the calculated scour depth. There was 3 to 4 feet of scour around pier #3 at the main channel bridge and at pier #2 at the relief bridge. The site inspection in 2001 revealed riprap was in-place from abutment to abutment at the main channel bridge and at the relief bridge. This indicates that scour has occurred in the past at both the main channel bridge and throughout the relief bridge. Local scour was indicated on the 1990 inspection report to be around all of the piers and approximately 4 feet deep around piers #3 and #4 per the ALDOT bridge inspector. The reason for the scour is probably due to the noncohesive soil.

VI. CONCLUSION

Scour evaluations were performed for 18 selected bridge sites in Alabama that had experienced an event equal to or greater than a 100-year flood event. Plots were derived for comparing field measurements made during yearly site inspections, ground elevations taken during floods where available, core boring information, and the computed scour from the estimated and measured flood events. Critical evaluation of each site shows differences between calculated and evident scour at the sites. It was found that scour evaluations performed on bridges with sand foundations experienced scour problems consistent with scour depths calculated from the HEC-18 equations. Scour evaluations performed on sites with cohesive soils showed that calculated scour depths were not consistent with observed scour at the site. For most cases, the reason for this discrepancy is due to higher critical shear stress in soils with cohesive forces. Also, even if the critical shear stress is exceeded, the rate of scour is less for cohesive soil. Thus, the flow of water over this soil would have to generate a higher shear stress over a long period of time to reach the calculated scour depths from HEC-18. In contrast, for noncohesive sandy soils, it is assumed that maximum scour depths occur instantaneously for a given discharge. The equations in HEC-18 do not consider cohesive forces to determine scour depths. Beyond the input for the HEC-18 equations, additional site-specific information and improved scour calculation methods are needed for design and for analyzing existing bridges at sites as discussed by Briaud et al. (1999, 2001a, 2001b), Güven et al. (2001), and Güven et al. (2002, TRB Paper No. 02-2127, accepted for publication).

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