Technical Report HL-94-1
May 1995

**US Army Corps of Engineers**
Waterways Experiment Station

# Demonstration Erosion Control Project Monitoring Program

# Fiscal Year 1993 Report

## Volume I: Main Text

*by Nolan K. Raphelt, William A. Thomas, Bobby J. Brown, David D. Abraham, David L. Derrick, Billy E. Johnson, Brenda L. Martin, Lisa C. Hubbard, Michael J. Trawle, WES*

*Chester C. Watson, Steven R. Abt, Colorado State University*

*Colin R. Thorne, University of Nottingham*

DTIC
ELECTE
JUN 15 1995
S D
B

Approved For Public Release; Distribution Is Unlimited

19950613 100

DTIC QUALITY INSPECTED 3

Prepared for U.S. Army Engineer District, Vicksburg

The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.

PRINTED ON RECYCLED PAPER

Technical Report HL-94-1
May 1995

# Demonstration Erosion Control Project Monitoring Program

# Fiscal Year 1993 Report

## Volume I: Main Text

by Nolan K. Raphelt, William A. Thomas,
Bobby J. Brown, David D. Abraham,
David L. Derrick, Billy E. Johnson,
Brenda L. Martin, Lisa C. Hubbard,
Michael J. Trawle

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Chester C. Watson, Steven R. Abt

Civil Engineering Department
Engineering Research Center
Colorado State University
Fort Collins, CO 80523

Colin R. Thorne

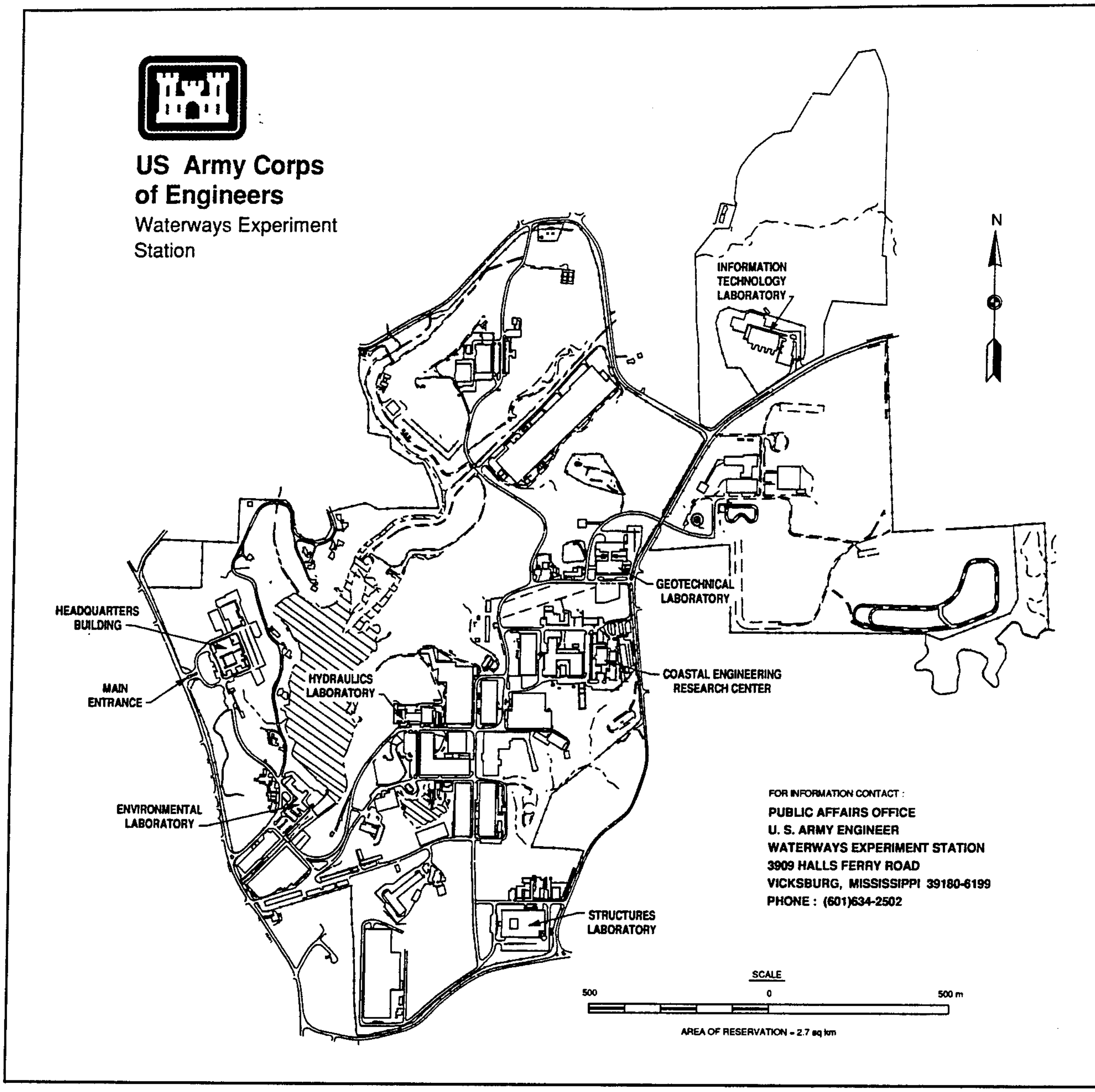
University of Nottingham
Department of Geography
University Park
Nottingham, England NG7 2RD

Final report

Approved for public release; distribution is unlimited

DTIC QUALITY INSPECTED 3

Prepared for U.S. Army Engineer District, Vicksburg
Vicksburg, MS 39180-5191




**Waterways Experiment Station Cataloging-in-Publication Data**

Demonstration Erosion Control Project Monitoring Program : fiscal year 1993 report. Volume I, Main text / by Nolan K. Raphelt ... [et al.] ; prepared for U.S. Army Engineer District, Vicksburg.
258 p. : ill. ; 28 cm. — (Technical report ; HL-94-1 v.1)
Includes bibliographical references.
1. Watershed management — Mississippi. 2. Flood control — Mississippi. 3. Erosion — Mississippi. 4. Hydraulic structures — Mississippi — Models. I. Raphelt. Nolan K. II. United States. Army. Corps of Engineers. Vicksburg District. III. U.S. Army Engineer Waterways Experiment Station. IV. Hydraulics Laboratory (U.S. Army Engineer Waterways Experiment Station) V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-94-1 v.1.
TA7 W34 no.HL-94-1 v.1

# Contents

[1] A limited number of copies of Appendices A-E were published under separate cover. Copies are available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

Accession For
NTIS GRA&I
DTIC TAB
Unannounced
Justification
By
Distribution/
Availability Codes
Dist | Avail and/or Special
A-1

[1] A limited number of copies of Appendices A-E were published under separate cover. Copies are available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

# Preface

This report discusses work performed during Fiscal Year 1993 by the Hydraulics Laboratory (HL) of the U.S. Army Engineer Waterways Experiment Station (WES) requested and sponsored by the U.S. Army Engineer District (USAED), Vicksburg.

The report was prepared by personnel of the Waterways Division (WD) and Hydraulic Structures Division (HSD), HL, and by the Civil Engineering Department of Colorado State University (CSU), Fort Collins, CO. Appendixes A, B, C, D, and E, prepared by HL personnel, are published as separate volumes.

WES acknowledges with appreciation the assistance and direction of Messrs. Franklin E. Hudson, Life Cycle Program Manager (LCPM), USAED, Vicksburg; Phil G. Combs, Acting Chief, River Stabilization Branch, Engineering Division, USAED, Vicksburg; and Charles D. Little, Hydraulics Section, Hydraulics Branch, Engineering Division, USAED, Vicksburg. Messrs. James Ross, Jasper Lummus, and Phil Combs assisted in all phases of the bendway weir/willow post study in Chapter 8.

The report was prepared under the direct supervision of Messrs. Michael J. Trawle, Chief, Math Modeling Branch (MMB), WD; and Thomas J. Pokrefke, Chief, River Engineering Branch, WD; and under the general supervision of Dr. Larry L. Daggett, Chief, WD; and Messrs. Glenn A. Pickering, Chief, HSD; R. A. Sager, Assistant Director, HL; and Frank A. Herrmann, Director, HL. This report was prepared by Messrs. Nolan K. Raphelt, William A. Thomas, David D. Abraham, David L. Derrick, Billy E. Johnson, Michael J. Trawle, and Mmes. Brenda L. Martin and Lisa C. Hubbard, and Dr. Bobby J. Brown, HL: Drs. Chester C. Watson and Steven R. Abt, CSU; and Dr. Colin R. Thorne, University of Nottingham, Nottingham, England, under contract to CSU. Messrs. Abraham and Derrick observed several DEC sites for the bendway weir study in Chapter 8. Mr. Derrick was either the principal investigator or a consultant on all bendway weir tests on all of these models.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

*The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.*

# Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| acres | 4,046.856 | square meters |
| cubic feet | 0.02831685 | cubic meters |
| degrees (angle) | 0.01745329 | radians |
| feet | 0.3048 | meters |
| inches | 2.54 | centimeters |
| miles (U.S. statute) | 1.609344 | kilometers |
| pounds (mass) | 0.4535924 | kilograms |
| square miles | 2.589988 | square kilometers |
| tons (2,000 lb, mass) | 907.1847 | kilograms |

# Summary

The authorized plan for the Demonstration Erosion Control (DEC) Project in the Yazoo Basin, Mississippi, provides for the development of a system for control of sediment, erosion, and flooding in the foothills area of the basin. The area's 15 watersheds are Abiaca Creek, Batupan Bogue, Black Creek, Burney Branch, Cane-Mussacuna Creek, Coldwater River, Hickahala-Senatobia Creek, Hotophia Creek, Hurricane-Wolf Creek, Long Creek, Otoucalofa Creek, Pelucia Creek, Sherman Creek, Toby Tubby Creek, and Town Creek (Charleston).

Public Law 98-8, the Emergency Jobs Appropriation Act of 1982, provided for the initial authorization of the DEC Project as a cooperative effort through the U.S. Department of Agriculture (USDA) Soil Conservation Service. Public Law 98-50, the Energy and Water Development Appropriation Act of Fiscal Year (FY) 1984, further directed joint effort by the U.S. Army Corps of Engineers and Soil Conservation Service for the foothills area of the Yazoo Basin. Public Law 99-662, the Water Resources Development Act of 1986, specified that the DEC Project was authorized by Public Law 98-8, and further directed that the DEC Project was exempt from the cost-sharing requirements of Public Law 99-662.

To assist in the evaluation of the performance of erosion control features installed as part of the DEC Project, the Hydraulics Laboratory (HL) of the U.S. Army Engineer Waterways Experiment Station (WES) initiated a comprehensive monitoring program in July 1991. The WES portion of the DEC monitoring program is designed as a multiyear program planned through FY 1997. The components of the monitoring program, including the design and implementation of an engineering database, development of evaluation procedures and design tools, and all field data collected through September 1993, are presented in detail in this report.

The field data collected through September 1993 for hydraulic structures and channel response monitoring included stage measurements at 31 continuous recording gauges and 37 crest gauges, located in 9 DEC watersheds (Black River, Abiaca Creek, Coldwater River, Hickahala-Senatobia Creek, Burney Branch, Hotophia Creek, Otoucalofa Creek, Batupan Bogue, and Long Creek). Also, during FY 1993, detailed channel geometries were collected at 22 sites in the same 9 DEC watersheds. These surveys, initiated in December

1991 and conducted semiannually, are designed to evaluate long-term channel response to changes in hydrologic and hydraulic regime. A comparative analysis of the 1992 and 1993 surveys at the 22 monitoring sites is presented in this report.

The engineering database/Geographic Information System (GIS) being used in the DEC monitoring program to manage the large amount of data being assembled is based on Intergraph hardware and software. As of September 1993, the database includes Soil Conservation Service curve numbers, land use, and soil type, each on 1-acre grids, for five of the DEC watersheds (Coldwater, Hickahala-Senatobia, Hurricane-Wolf, Cane-Mussacuna, and Long). The database contains U.S. Geological Survey digital elevation models and quad maps for all 15 DEC watersheds. The database also includes Spot-View satellite photography at 10-m resolution for all 15 watersheds. The database also includes the locations and design parameters for all U.S. Army Corps of Engineers construction existing as of September 1993 for riser pipe, low-drop, and high-drop structures; bank stabilization features; and box culvert grade control structures. Locations of proposed and constructed levees, floodwater retarding structures, and channel improvement features are also in the database. The database also contains all major tributaries and highways for the 15 watersheds.

Detailed geomorphic studies were conducted on two streams (Otoucalofa and Hotophia Creeks) using survey data from 1985 and 1992. The surveys consisted of channel profiles and cross sections made at half-mile intervals. The surveys were used to assess channel changes from 1985 to 1992. Channel profiles were compared to determine zones of aggradation or degradation and channel cross sections to determine width and depth changes.

During FY 1993 the hydrology effort focused on the testing of a two-dimensional hydrologic model (CASC2D) for applicability to DEC watersheds. The Goodwin Creek watershed, a highly gaged watershed used by the Agricultural Research Service as a field laboratory, was the test site for the application of both one-dimensional (HEC-1) and two-dimensional (CASC2D) hydrologic models. The results indicated that for largely ungaged watersheds such as those in the DEC Project, the two-dimensional approach can outperform the one-dimensional approach in the predictive mode. The results of this investigation are given in this report.

An Intergraph-base procedure (design tool) that takes advantage of the engineering database/GIS, initially developed during FY 1992 to support the U.S. Army Engineer District, Vicksburg, hydraulic design of riser pipes, was expanded to include five DEC watersheds (Coldwater, Hickahala-Senatobia, Hurricane-Wolf, Cane-Mussacuna, and Long). The procedure automates a number of the steps previously done manually, resulting in significant reduction in the time required to conduct the hydraulic design of riser pipes.

A design procedure for spacing grade control structures (design tool), based on the computer program "Hydraulic Design for Channels," SAM, was developed and tested on a DEC stream (Byhalia Creek). The test application consisted of evaluating the spacing of low-drop structures required to provide a stable stream without further degradation that could cause failure of grade control structures. The proposed procedure has merit in assisting the engineer in designing structural solutions that have the potential for long-term beneficial impact in reducing channel degradation and streambank erosion. The procedure and test application are discussed in detail in this report.

During FY 1992, high-drop structures on Hotophia Creek (1) and Burney Branch (1) and low-drop structures on Worsham Creek (3) and Long Creek (1) were instrumented to collect stage data just upstream and downstream of the structures. During early FY 1993, two additional low-drop structures (Hickahala and James Wolf Creeks) were instrumented, for a total of eight structures instrumented as of September 1993. Discharge rating curves have been developed for these structures using the stage measurements and physical-model-based discharge coefficients. Also during FY 1993, field inspection of the high-drop and low-drop grade control structures in the DEC Project, including an evaluation form for each structure, was accomplished.

The use of bendway weirs as bank protection is being tested at the Harland Creek test site. This site encompasses a channel reach of Harland Creek that includes 14 bends over a distance of approximately 12,000 ft. Bendway weirs were constructed in selected bends during FY 1993, and the reach is being monitored. In addition, the use of willow posts as bank protection is being tested at the same Harland Creek test site. During FY 1994, over 9,000 willow posts will be planted in selected bends. Also as part of the bank stability effort, aerial reconnaissance videos were made again on all 15 watersheds and a broad-based assessment conducted using these videos.

The physical model study initiated in FY 1992 to investigate the feasibility of a sheet-pile grade control structure with a 10-ft drop was completed. Current design criteria for a sheet-pile grade control structure limit the drop height to 6 ft. Model results and recommendations regarding the riprap stability of 10-ft sheet-pile drop structures are given in this report.

The results and conclusions of each part of the monitoring program for FY 1993 are described in this report.

# 1 Introduction

## Background

The Demonstration Erosion Control (DEC) Project provides for the development of a system for control of sediment, erosion, and flooding in the foothills area of the Yazoo Basin, Mississippi (Figure 1). Structural features used in developing rehabilitation plans for the DEC watersheds include high-drop grade control structures similar to the U.S. Department of Agriculture Soil Conservation Service (SCS) Type C structure; low-drop grade control structures similar to the Agricultural Research Service (ARS) low-drop structure; pipe drop structures; bank stabilization, which includes stone longitudinal toe and transverse dikes, and full bank paving; and a combination of retention and detention reservoirs. In addition, other features such as levees, pumping plants, land treatments, and developing technologies may also be used.

Evaluation of the performance of these erosion control features can contribute to the improvement and development of design guidance. Most of the previous Yazoo Basin evaluation has been limited to single-visit data collection, with no comprehensive monitoring of the structure or the effect of the structure on channel stability. The portion of the DEC Monitoring Program being conducted by the Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station (WES), is a multiyear program initiated in late Fiscal Year (FY) 1991 and planned through FY 97. To fully document the impacts of the DEC Project will require more than 6 years. A monitoring plan for the DEC Project after FY 97 will be provided at the appropriate time.

## Objective

The purpose of monitoring is to evaluate and document watershed response to the implemented DEC Project. Documentation of watershed response to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds.

The objective of this report is to document the WES monitoring activities during the period from June 1992 through September 1993.

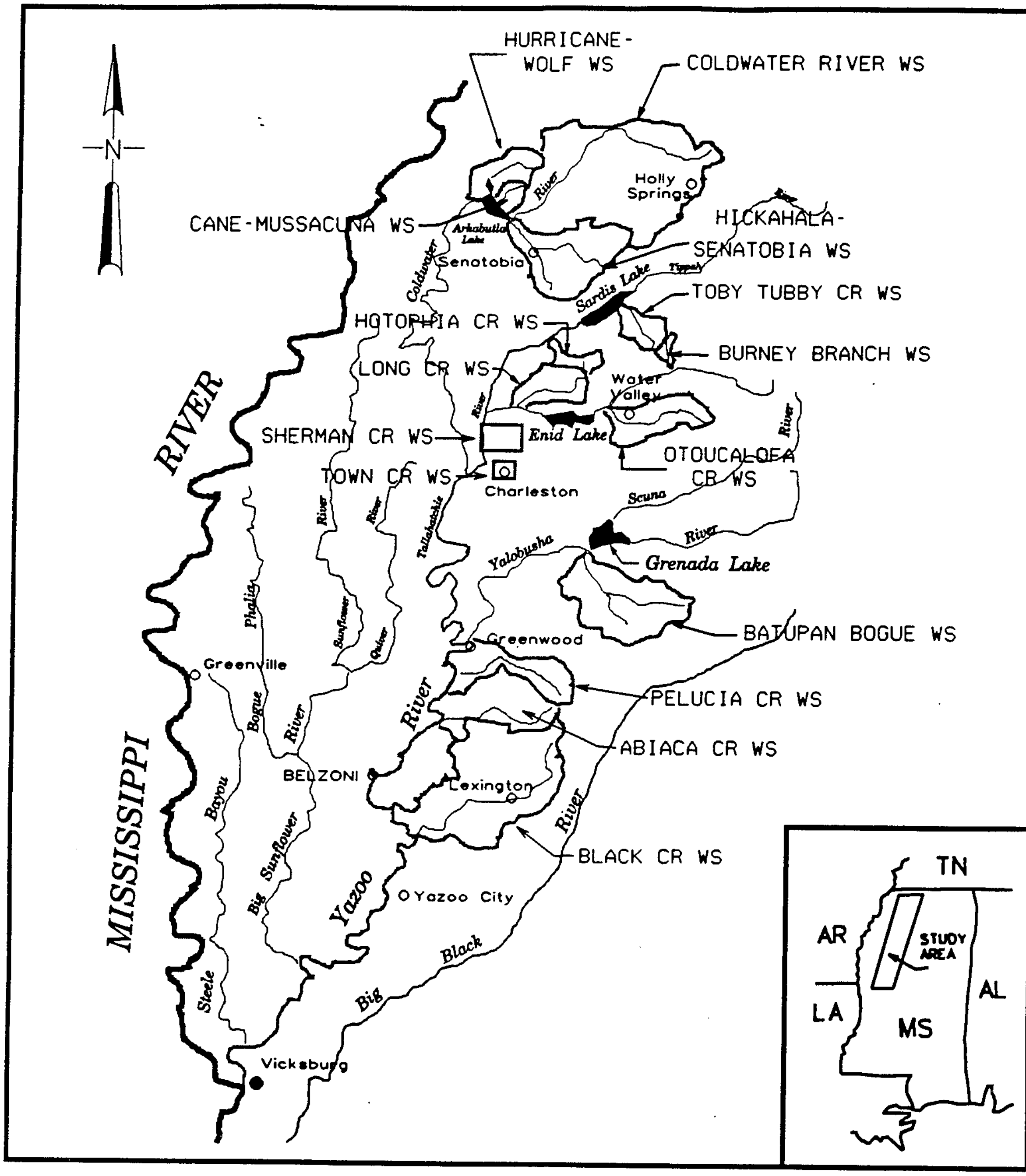



Figure 1. Vicinity map of DEC watersheds

## Approach

To provide the information necessary for the effective evaluation of the DEC Project, the DEC Monitoring Program includes eleven technical areas that address the major physical processes of erosion, sedimentation, and flooding:

*a.* Stream gauging.

*b.* Data collection and data management.

*c.* Hydraulic performance of structures.

*d.* Channel response.

*e.* Hydrology.

*f.* Upland watersheds.

*g.* Reservoir sedimentation.

*h.* Environmental aspects.

*i.* Streambank stability.

*j.* Design tools.

*k.* Technology transfer.

The WES portion of the monitoring program has primary responsibility for all technical areas except stream gauging and environmental aspects. The primary responsibility for these technical areas rests with the U.S. Geological Survey (USGS) and ARS, respectively.

## Technical Area Descriptions

The following is a general description of the work being performed by WES in the nine technical areas.

### Data collection and data management

The purpose of the data collection and data management technical area is to assemble, to the extent possible, all data that have been accumulated to date in the DEC Project, and develop an engineering database that will be periodically updated as new monitoring data are collected and analyzed. The database resides on an Intergraph workstation, and access to the database is made user-friendly with Intergraph software. The database is available to all participants in the monitoring program to provide for analysis and evaluation of the various elements of the DEC Project. In addition to the extensive hydraulic and sedimentation data being collected in the monitoring program, the database contains aerial photography, USGS digital elevation grids, USGS quadrangle maps, and project feature locations and information.

### Hydraulic performance of structures

Six grade control structures were selected for detailed data collection to evaluate hydraulic performance. The structures were selected on the basis of special features, including high drop, low drop, significant upstream flow constriction, limited upstream flow constriction, free flow, and submerged flow. The structures were instrumented to collect data to evaluate discharge coefficients, energy dissipation, flow velocity distribution, and effects of submergence on performance. All riprap bank stabilization measures in each watershed will be visually monitored and problem areas identified. A minimum of three riprap bank stabilization installations including riprap blanket revetment, riprap toe protection, and riprap dikes were selected to evaluate toe and end section scour. Data are being collected during runoff events to measure magnitude and location of maximum scour and the corresponding hydraulic parameters. This technical area also included the construction of a physical model of a low-drop structure. The model was used to determine if modifications can be made to the low-drop structure design that either maintain or enhance performance characteristics at a reduction in cost.

### Channel response

The channel response monitoring focusses on two major areas: channel sedimentation and channel-forming discharge. Monitoring for channel sedimentation includes an annual geomorphic update of selected watersheds. In addition to the geomorphic update, 22 sites where structures exist or are anticipated were selected for intensive monitoring over the life of the program. Channels upstream and downstream of the selected structures are being monitored for cross-section changes, thalweg changes, berm formation, bank failure, and vegetation development. Five additional sites where no structures are planned are also being monitored. These five sites serve as a control group and assist in the evaluation of channel response to structures. Photo documentation of structures and channels is being conducted and included in the database. A subset of these structures and channels is being instrumented for stage, discharge, suspended sediment concentration, and bed-load material measurements. The numerical sediment transport model HEC-6 (U.S. Army Engineer Hydrologic Engineering Center (USAEHEC) 1993) and the new computer program SAM (Thomas et al., in preparation) are being used to predict the stability of channels monitored by this work effort. Also, the DEC watersheds are providing data that will be used to test design procedures and techniques for the channel-forming discharge concept. Successful development of such channel-forming discharge methodology could result in significant design cost savings for the DEC Project.

### Hydrology

Rainfall provides the energy to sustain erosional processes. The ability to measure rainfall and compute runoff accurately is crucial in the design of stable flood-control channels. Accurate flow rates are needed to design functional project features properly and maintain stability in the channel system as well as monitor the project. CASC2D hydrologic models of a selected number of watersheds are being developed. Hydrologic modeling and hydraulic structures monitoring are being coordinated so that hydrologic parameters used in CASC2D can be determined at locations in the watersheds where USGS gauging stations do not exist.

### Upland watersheds

ARS has been given the primary responsibility for this technical area. WES was not active in this area during FY 93. The two items related to the upland watersheds to be monitored by ARS are system sediment loading (sediment yield) and sediment production from gully formation. Stabilization measures being installed to reduce upland erosion will be monitored by ARS over the next 4 years to determine if a measurable change in the quantity of sediment being transported from watersheds occurs. Data already collected by USGS and ARS over the past 6 years will be analyzed and interpreted by ARS to serve as the base for future comparisons. The numerical modeling of sediment runoff from watersheds by WES is planned as part of the analysis and interpretation process. Also, sediment production from two or three active gullies will be analyzed by ARS by comparing surveys made prior to the design of drop pipes and the survey made just prior to construction of the drop pipes.

### Reservoir sedimentation

The major sources of reservoir deposition are upland erosion, erosion of the channel banks, and erosion of the channel bed. The reduction of the inflowing sediment load is being addressed in the channel response, bank stability, and upland watershed technical areas. Starting in FY 94, WES will use the results of the analysis performed in these areas to determine the effects of the project on reservoir sedimentation.

### Streambank stability

Streambank stability depends on hydraulic parameters related to flow conditions and the characteristics of the materials in the banks. All channels will be visually monitored periodically to determine reaches that are experiencing severe bank stability problems. In addition to the overall visual monitoring, five sites where aggradation is occurring and five sites where bank caving is occurring were selected for detailed monitoring. At the selected sites, surveys of closely spaced sections will be made semiannually to document changes. After sufficient data have been collected, appropriate numerical models will be applied to determine if existing numerical techniques can be adapted to predict bank stability and/or bank failures accurately.

## Design tools

The procedures and techniques used in the design of the different features of the DEC Project have the potential for national and international applications. Effective application of these design procedures and techniques may require development of computer-based packages and the validation of numerical models such as CASC2D, HEC-6, and SAM. In conjunction with ongoing research, WES is developing design tools specifically targeted for the planning and design of stable flood-control projects.

## Technology transfer

Technology transfer is an important part of the DEC Project and will be given high priority at WES during the life of the monitoring program. When appropriate, WES personnel will present results at national and international technical conferences and symposiums. When appropriate, WES personnel will host workshops and training classes for both Corps and non-Corps personnel. WES will annually report on the DEC monitoring program using several different formats. For FY 93, these include the following:

*a.* A video report on channel degradation processes.

*b.* An updated engineering database on the Intergraph system including aerial photos, surveys (channel and structural), results of numerical studies, etc.

*c.* A short executive summary report.

*d.* A detailed WES technical report on monitoring, data collection, data analysis, and project evaluation.

# 2 Data Collection and Data Management

As in the FY 92 work, the FY 93 WES data collection effort is in direct support of the other DEC monitoring functions. Data being collected consist of water surface elevations and flow rates obtained from the various streams and rivers in the DEC watersheds. The primary use is as input to hydraulic and hydrologic models. A secondary use is in the analysis of the performance of hydraulic structures.

The raw data are recorded in feet of water relative to an arbitrary reference point. Depending on the type of instrumentation used, the data must be added to or subtracted from a known datum to represent the true water surface elevation. In the case of the flow rate measurements, the data are recorded as velocities associated with known cross-sectional areas. From these, a flow rate is calculated for a given cross section.

The instrumentation being used is the same as in FY 92. For a complete description see Raphelt et al. (1993).

The data collection effort for FY 93 involved the following activities:

*a.* Deployment of new instruments.

*b.* Continuing operation of existing instruments.

*c.* Stage data processing.

Each of these activities is described in the following paragraphs.

## Deployment of New Instruments

Ultrasonic level measuring devices using Sutron 8200 data loggers were installed at Harland Creek site 030120 on 6 October 1992, at Otoucalofa Creek site 091420 on 7 October 1992, at Lee Creek site 051020 on 8 October 1992, and at Sykes Creek site 021720 on 1 December 1992. No new pressure transducers were installed. New crest gauges were installed at Nolehoe Creek sites 050728 and 050750, and at Perry Creek sites 021630 and 021660.

## Continuing Operation of Existing Instruments

Even though both types of electronic data logging instruments can store in excess of 2 months of data at 10-min logging intervals, in general, an attempt is made to visit each site at least once a month. During such a visit several functions are attended to. Data are uploaded from the data logger to a personal computer (PC) via an RS-232 communication cable. A cursory scan of the data is made to determine if the collected data appear reasonable with respect to quantitative and temporal values. If the data look reasonable, normal maintenance is then carried out. For example, wiring is checked, batteries are checked and replaced if needed, instrument sensors and housings are checked and cleaned if required, and locks are oiled. If the data do not look reasonable, then most of these maintenance procedures are carried out, and in addition, the cause for the malfunction is searched out. This might include replacing internal fuses, checking connector strip connections, and determining if the sensor transducers are functioning correctly by visual, aural, and electrical return signal inspection. If the fault is found and can be corrected in the field, that course of action is taken and the instrument is recalibrated. If the fault cannot be found, the malfunctioning components are removed and brought back to WES for repair and/or replacement.

During FY 93 the amount of downtime due to the need for repair or replacement of electronic instruments was tabulated. For the Stevens pressure transducer assemblies, the total number of days of downtime was 601. The 15 pressure transducers were deployed for the full 365 days of the year, representing a total of 5,475 possible days of operation. This represents a 10.9 percent downtime.

The amount of downtime for the Sutron/Lundahl ultrasonic assemblies was 380 days. Not all assemblies were installed for the full year. Taking that into account, the total number of days the instruments were deployed was 5,758. This represents a 6.6 percent downtime.

An attempt was made to calibrate each instrument at least once during the year. It must be emphasized that this calibration was not meant to be an overall correction for data throughout the year, but rather a check on the possibility of instrument drift, and to assure that the instrument was still functioning over its full range. For the ultrasonic instruments the calibration consisted of "taping down" to the water surface and checking the actual distance to the instrument reading. A water pressure calibration tube was used for checking the accuracy of the pressure transducers. The dates on which the different instruments were checked are shown in Table 1.

## Stage Data Processing

Using vendor-supplied software, the data are downloaded from the laptop PC to the office PC on which the data processing software are resident. The data from the Stevens recorders are already in ASCI form, while a special program supplied by Sutron Corporation must be used to convert the data recorded by the Sutron logger from binary to ASCI form. This raw data file must be edited so that the headers and other information from both types of recorders are put into a similar format. Several batch files and macros were written to perform this function. However, up to this point no data have been altered.

When small amounts of data were initially graphed, many fluctuations due to temperature and large spikes of uncertain origin were noticed. For the Stevens transducers the spikes were generally not more than 1 to 1-1/2 ft,[1] while for the ultrasonic sensors, they varied from 0.5 ft to the instrument's maximum range. The cause for the anomalous readings from the pressure transducers has not been ascertained. The causes for the anomalous readings from the ultrasonic instruments have been found to be due to any of various circumstances, some of which are no water in the creek, insect nests in the wave guide, birds flying under the instrument, and damaged or malfunctioning pulse emitters. An example of data from a pressure transducer assembly that produced unwanted and unidentified spikes is shown in Figure 2. Data from an ultrasonic data collection assembly that includes various spikes and spurious data are shown in Figure 3.

It can be seen by viewing the raw data shown in Figures 2 and 3 that, in general, the desired hydrographs of 1-ft rise and larger can be visually picked out. However, the "noise" can be distracting and also takes up much data storage space. For most purposes in the DEC monitoring project, events of less than 1 ft are not of significant interest. Therefore a spike-removal/data-averaging program was written that uses these raw data files to produce files that are much clearer and smaller. The program works by checking for changes in stage values greater than some amount that is observed to be possible in the data logging interval (10 min). The greatest observed change in stage over 10 min that was found was approximately 1.2 ft. Therefore a conservative value of 1.5 ft was selected. Thus, whenever a change in stage of more than 1.5 ft in a 10-min interval occurs, the program considers it a spike, and deletes it from the data. The inherent danger in this is that a real change greater than 1.5 ft in the interval could possibly happen. However, then a hydrograph with a missing peak would be evident. The raw data files are saved, and thus the data could be retrieved and the peak reconstructed manually.

---

[1] A table of factors for converting non-SI units of measurement to SI units is found on page viii.

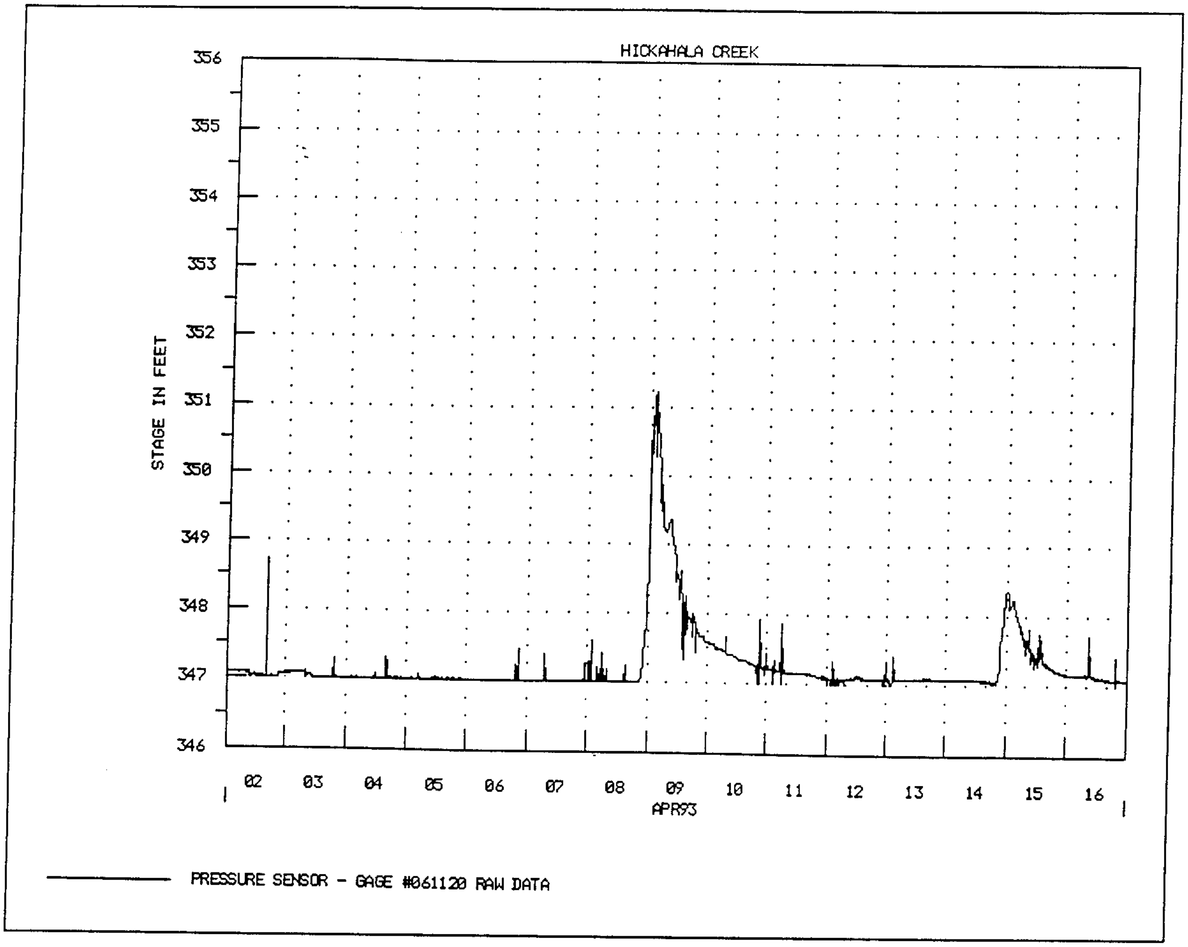



Figure 2. Pressure transducer data with spikes

Besides removing the spikes, the program averages daily data. If there is no change greater than a user-specified input value for the day, then all data points for that day are averaged and replaced by one data point, centered in time. The user-specified input value is usually between 0.4 and 1 ft. It can be seen that this will greatly reduce the number of data points in a file. For example, if a file contains 30 days of records with 10-min data intervals, then a total of 4,320 data points are logged. If during this time an event greater than 1 ft occurred for 2 days, then the program will save all data points related to the 2-day event, but replace the other days with one data point each. So the 30-day data file will be reduced from 4,320 to 316 data points. It should be noted that in the process of removing spikes and reducing the length of the data files, the integrity of the events in terms of magnitude and time is preserved. This is clearly shown by a comparison of Figure 2 to Figure 4 and Figure 3 to Figure 5. For purposes of flow estimation at times other than the peak events, it appears that the line graphed by the average values should approximate the true water surface elevation. An actual 48-hr field study is planned to verify this assumption.

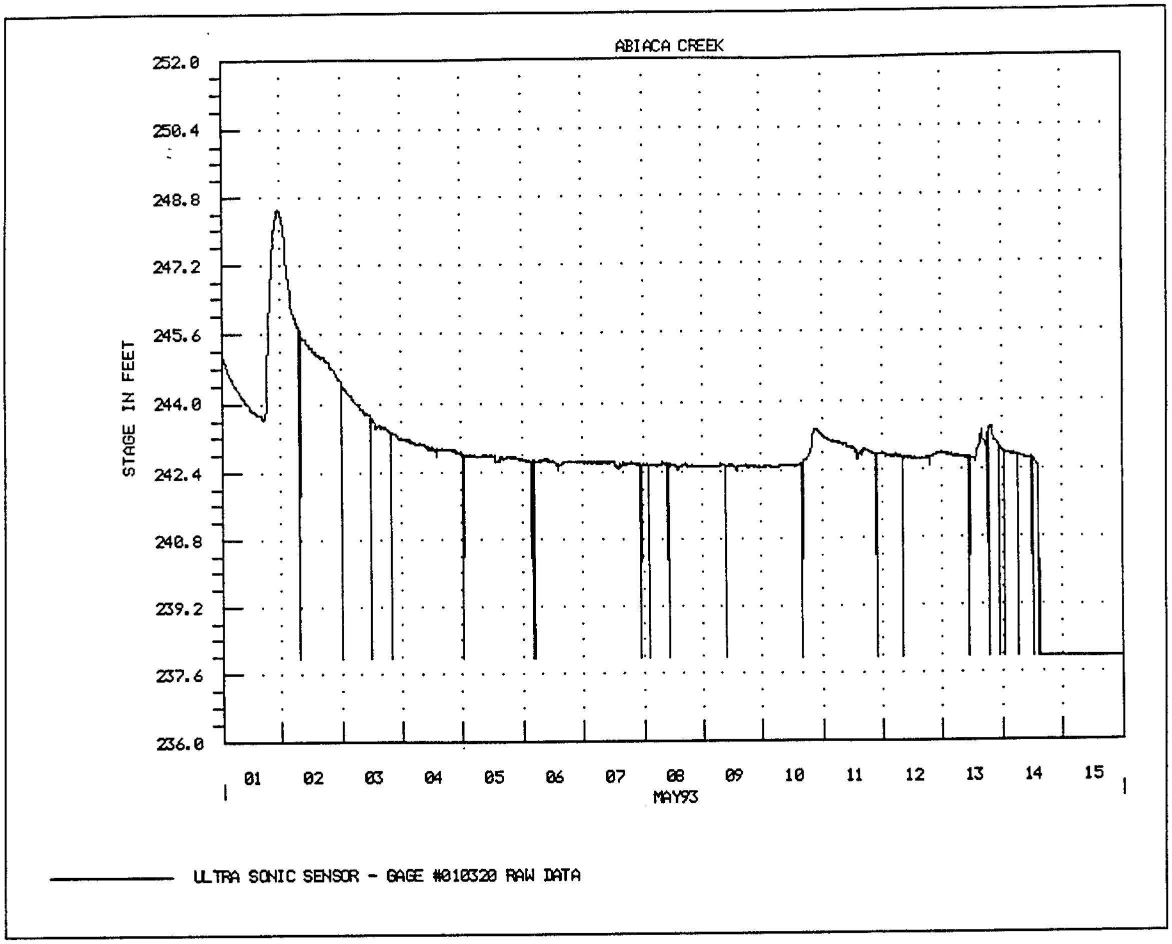



Figure 3. Ultrasonic sensor data with spikes

The spike removal and averaging program also rewrites the event and raw data files so that they can be used in the Corps of Engineers hydrologic data processing program called DSS (Data Storage System). The graphics routines of the DSS can be used to graph large amounts of data. Therefore, the stage data, once placed into the database, will be accessible for any type of graphing manipulation normally accomplished using DSS. The processing of the stage data into flow rates is covered in Chapter 6.

Once the event data files are produced, it is necessary to review them for completeness, datums, calibration adjustments, and accuracy. Checking for completeness involves determining where valid data gaps exist, for example, when the instrument was not deployed or when it malfunctioned. When such times are identified from written and electronic records, a valid data gap exists. Otherwise the data could be considered lost or missing.

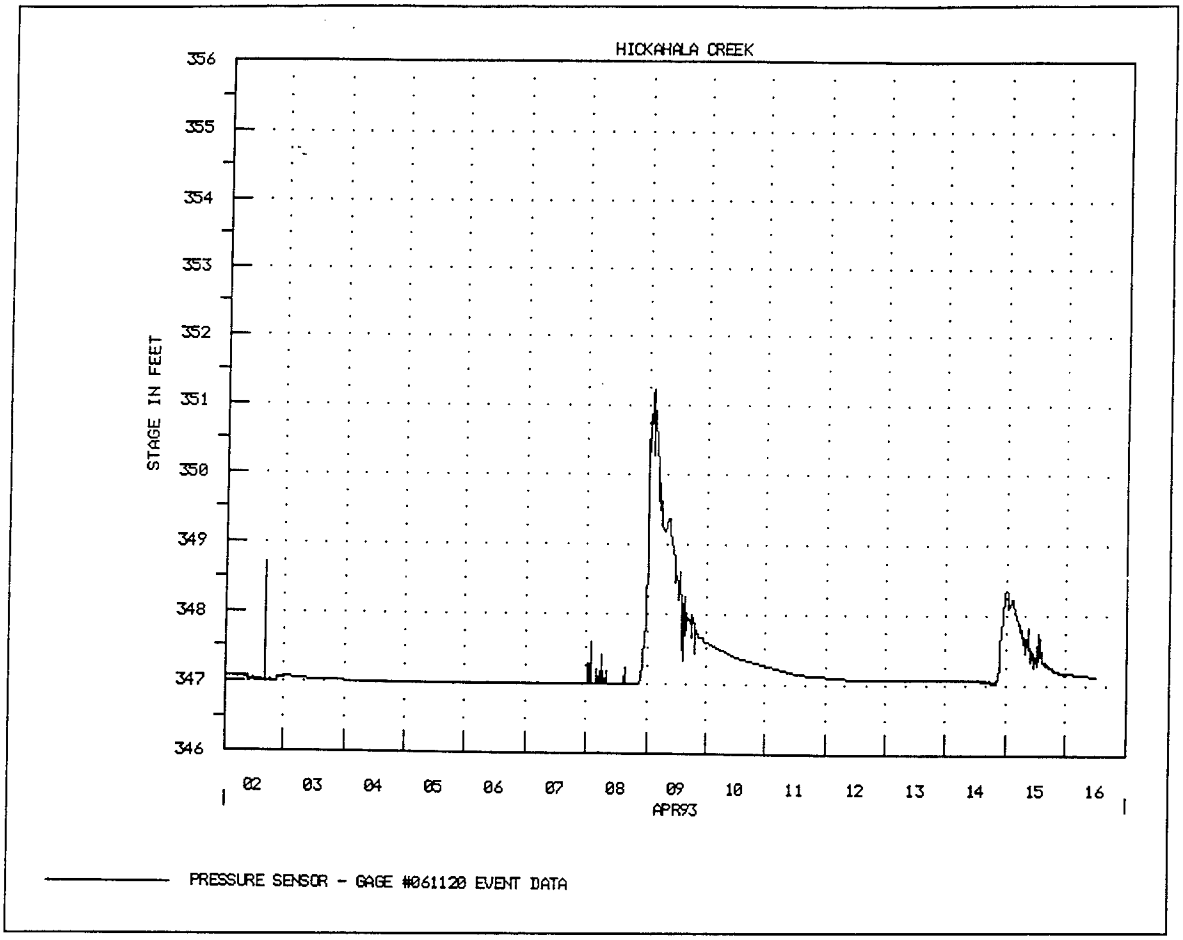



Figure 4. Reduced pressure transducer data

Establishing true mean sea level (msl) water surface elevations required that each instrument be surveyed in. The surveys were conducted by WES and Colorado State University (CSU), Fort Collins, CO. Benchmarks near each site had to be located, identified, and traced back to their originator. These sources were the Corps of Engineers District, state and county road divisions, and SCS.

During normal data retrieval trips, instruments were calibrated as described in the section, "Continuing Operation of Existing Instruments," in this chapter. When the data were finally graphed and reviewed, any adjustments to the data as a result of the calibration were made. Due to the number of instruments and the large area over which they are deployed, only one planned calibration was completed. This is not sufficient to develop an error correction curve that is useful for the entire year of data. Therefore the correction was normally used only for the data within that data retrieval period.

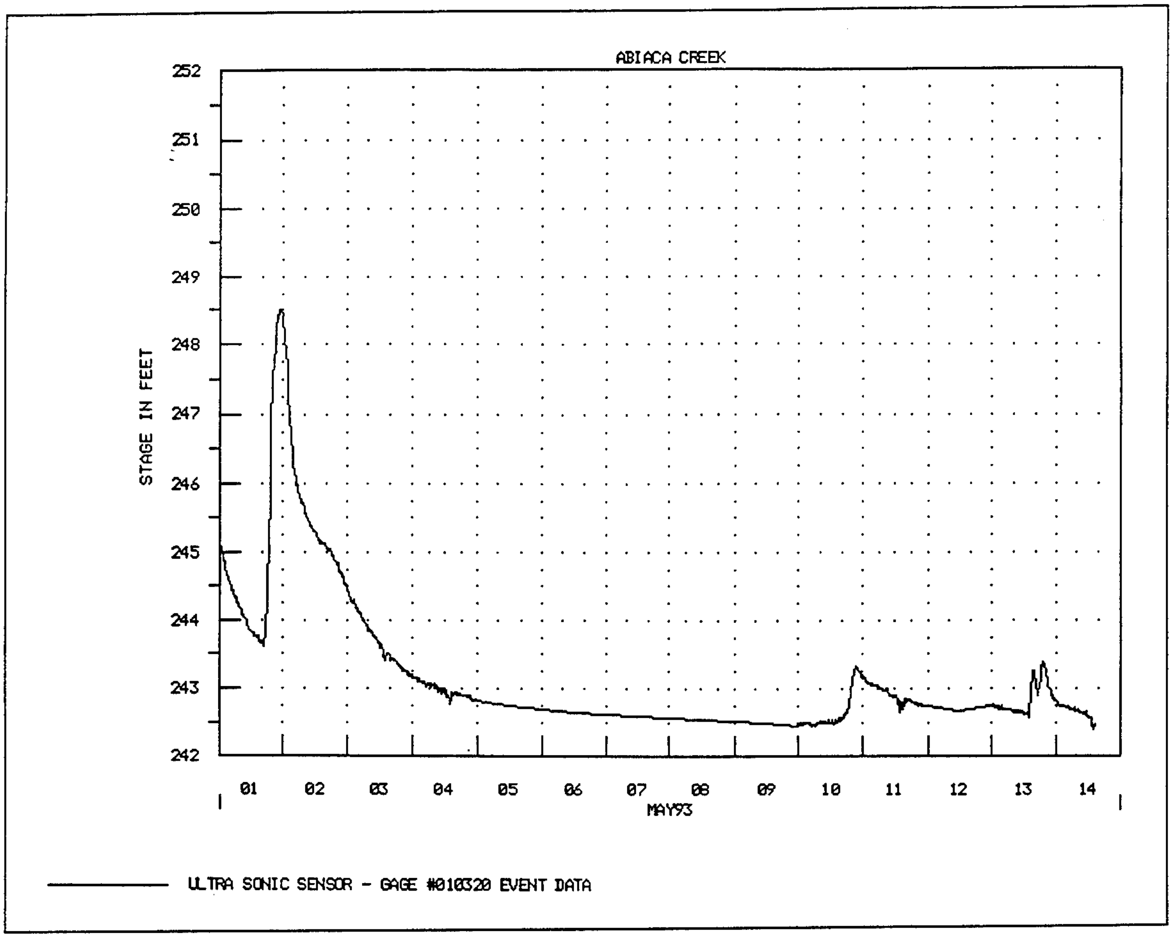



Figure 5. Reduced ultrasonic sensor data

The final check for the data was assessing the accuracy. In terms of absolute accuracy, a test is planned for the ultrasonic instruments and pressure transducers in early FY 94. This assessment is envisioned to involve a statistical analysis of recorded, averaged, and measured data from a single field site over a 48-hr period. In the absence of such data, and based on cursory field observations, the ultrasonic data can show a range of variation of $\pm 0.15$ ft. This appears to be due largely to temperature variations and wind. Other than the assessment of this elusive absolute accuracy, the instruments were checked for relative accuracy, i.e., transducer variation. That is, they were compared one against the other for duplication of event magnitudes and times. Also, where possible, they were compared to crest gauges. The crest gauge readings and the date on which each one was read are shown in graphs like those of Figure 6.

Finally, several of the larger events for each site were graphed as a profile. One of these graphs can be seen in Figure 7. This figure is intended not just to show the profile, but also to help the viewer visualize the location and identification number of each instrument at the site.

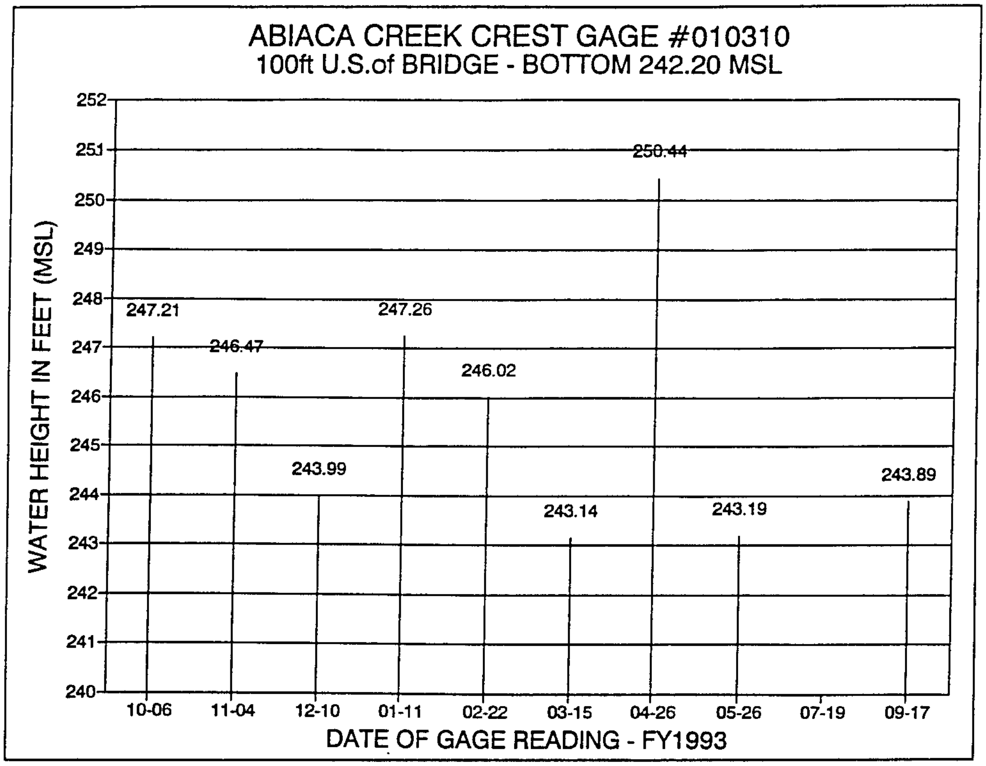



Figure 6. Example crest gauge readings

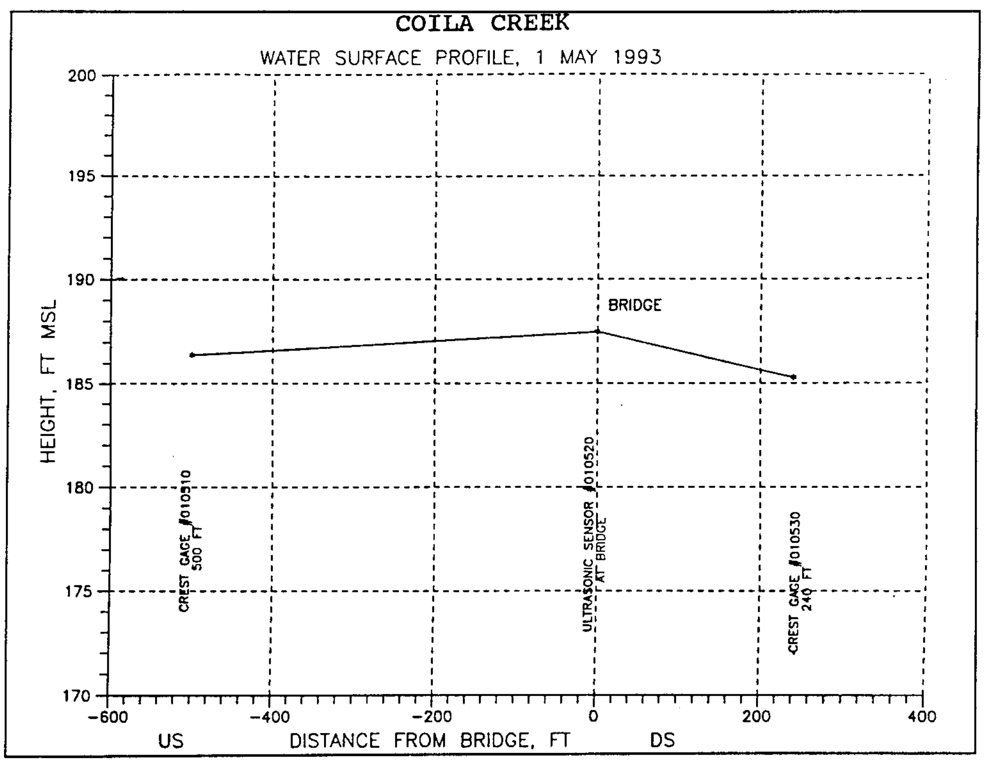



Figure 7. Example water surface profile

## Summary

The data in Appendix A of this report are retrieved from 22 sites during FY 93. Each site contains from one to seven instruments. The graphs in Appendix A show the stage hydrographs, the creek name, dates, and the instrument ID number. The last two digits of the six-digit ID number generally increase by tens from upstream to downstream. In general, the peak stages of the three largest events in the FY were tabulated as shown in Figure 8. These tables precede the stage hydrographs in the main body of data in Appendix A.

Readings, ft msl, for Selected Peak Event Data, Sarter Creek FY93

| Gage No. | Distance Upstream (-) Downstream (+) of Bridge | Readings, ft msl | | | | |
|---|---|---|---|---|---|---|
| | | 20Dec92 | 16Feb93 | 09Apr93 | 03Aug93 | |
| Crest 091510 | -540 ft | 344.3 | 345.2 | 346.9 | 346.6 | |
| Ultra-sonic 091520 | On Bridge | 342.8 | 343.1 | 345.2 | 344.1 | |
| Crest 091550 | +640 ft | 341.8 | 342.3 | 341.0 | 343.1 | |
| | | | | | | |
| | | | | | | |

Figure 8. Example of selected peak event data

# Engineering Database/GIS

### Approach

The purpose of the engineering database/Geographic Information System (GIS) is to serve as a repository for all design, analysis, and monitoring data collected on the DEC Project. The engineering database/GIS concept was chosen for the DEC Project because it allows for the storage, retrieval, analysis, and graphical display of all data. When completed, it is anticipated that the database will contain design data for all project features such as low- and high-drop structures, bank stabilization structures, floodwater-retarding structures, channel improvements, levees, riser pipes, and box culverts. Every effort will be made to include data from all participating agencies in the DEC Project.

The database will contain an index of all studies, analyses, and published reports for the DEC Project. Important or significant reports from the index list will be incorporated as documents into the database. The database will be tied to the GIS system for graphical display of the data. The Informix relational database is being used to store the data, which allows analysis of project features. In addition to the Informix relational database, the USAEHEC's data storage system, HECDSS, will be embedded in the engineering data-base/ GIS. The HECDSS database will contain stage, discharge, and cross-section data and will serve as a base for running numerical models. It is anticipated that HEC-1 (USAEHEC 1990), HEC-2 (USAEHEC 1982), and later in the project, two-dimensional hydrology and three-dimensional hydraulic models will run from data stored in the database.

### Computer hardware and software

The engineering database/GIS is being developed in the Intergraph 6040 workstation. The engineering database/GIS uses a number of MGE products. MGE is the umbrella under which Intergraph's GIS and database management software run. The system uses the Microstation software package. Microstation capabilities include computer-aided drafting and design (CADD), editing and placement of project features, editing and drawing on project features, and design and development of new design files. Also under MGE are Imager for imaging processing, IVEC for vectorization of scanned data, and Grid Analysis. Grid Analysis is used to develop grids for soil type, land use, slope, and elevation. Imager is used for image processing. Imager is also used with Grid Analysis for the hydrologic studies. MGE Terrain Modeler and a number of MGE translator programs translate Digital Line Graph (DLG) and Digital Elevation Model (DEM) data into the Intergraph format. It is anticipated that two additional Intergraph pieces of software will become important in the database. The DBS software will be used for document storage and retrieval, and the Inroads program will be used to store terrain model data and survey data, develop HEC decks for two- and three-dimensional models, and monitor surveys and changes in cross sections and survey areas. The HEC database will be used to store stage discharge and cross-section data.

### Status

As of 30 September 1993, the database contained SCS curve numbers on a 1-acre grid for five of the DEC watersheds (Cane-Mussacuna, Coldwater, Hickahala-Senatobia, Hurricane-Wolf, and Long). The 1:24000 digital quadrangle maps and DEM'S have been incorporated into the engineering database for all the DEC watersheds. Initially, streams and roads from the 1:100000 USGS DLG's were incorporated into the database. As the 1:24000 DLG data become available, they will be added to the database. Spot-View satellite photography has been incorporated into the database and is used as a visual reference for all project features. Satellite photography at 10-m resolution is in the database for all DEC watersheds. The engineering database consists of the locations and design parameters for all construction existing in FY 93 by the U.S. Army Corps of Engineers for riser pipe, low-drop, and high-drop structures; bank stabilization; and box-culvert grade control structures. Locations of proposed and constructed levees, floodwater-retarding structures, and channel improvement and box control structures are also in the database. Land use and soil type data for Coldwater, Hickahala-Senatobia, Long, Hurricane-Wolf, and Cane-Mussacuna basins are in the database on a 1-acre grid. Elevation and slope data for Cane-Mussacuna, Coldwater, Hickahala, and Hurricane-Wolf watersheds are in the database on a 30-m grid. The database contains all major tributaries and highways for the 15 DEC watersheds. The 1:100,000 digital DLG files are the source of the stream and highway data. A summary of the data contained in the database at the end of FY 93 is given in Table 2.

# 3 Channel Response, Semiannual Survey of 22 Long-Term Sites

## Introduction

In December 1991, field monitoring of 20 DEC stability sites was begun, and was repeated again with field surveys in June 1992, December 1992, and January 1993. Two additional sites were added beginning with the December 1992 survey, for a total of 22 monitoring sites. The locations of the watersheds containing the 22 study sites are shown in Figure 9. This chapter is a summary of the progress made in FY 1993.

## Objective

The objective of the field monitoring program and related analyses is to continue to monitor, document, and interpret the response of DEC channels to changes in the hydrologic and hydraulic regime, to monitor structure conditions, and to analyze the changes in bank stability. The primary objective of the work is to assist in developing improved design guidance for the DEC Project. The database will include survey and other data for 22 sites. Several areas of interest are being addressed in the program: (a) development of the basic understanding of the physical principles involved in assessing channel bank stability as the stream channel aggrades, (b) defining the effective discharge and channel-forming or dominant discharge in channel stabilization, (c) determining the effect of grade control on channel planform, (d) determining the temporal and spatial effectiveness of grade control, and (e) determining the effect of channel rehabilitation on flood-wave attenuation. The sites include locations of drop structures, bank stabilization, reaches affected by reservoirs, channelization, sediment traps, and sites that vary in the degree of active erosion.

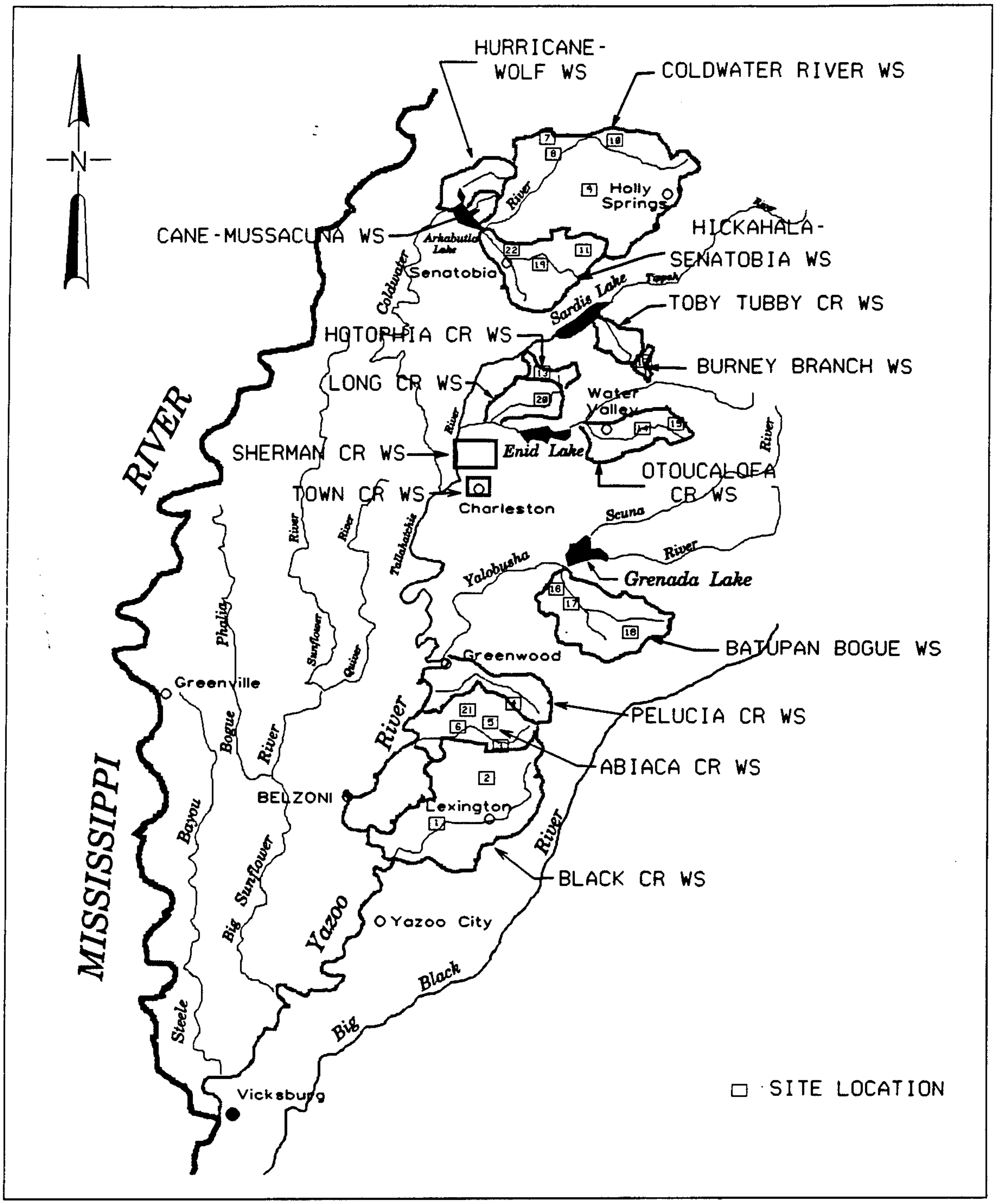



Figure 9. Site location map

Data are being analyzed and tabulated for use by other investigators at WES. In addition, graduate students at CSU will be conducting research on a topic related to DEC channel response.

## Monitoring Sites

The selected sites include approximately 15 existing low-drop structures, 3 existing high-drop structures, an anticipated channelization project, 20 anticipated low-drop sites, 2 anticipated high-drop sites, chevron dikes, bank stabilization, a sediment basin, and 6 control reaches in approximately 32 miles of study reach at 22 different locations. These sites have been selected to represent many of the different DEC watersheds, types of channel planform and sediment gradation, particular causes of instability, types of channel rehabilitation, and locations of special interest. The location of the 22 monitoring sites for 1993 are shown in Figure 9. Each site will be briefly discussed in the following sections.

### Harland Creek

Site 1 is located on Harland Creek in the Black Creek watershed. The site is near Eulogy, MS, and can be found on the Lexington quadrangle map in T14N, R1E, Sections 22 and 27. Harland Creek is a mixed sand and gravel bed stream, exhibiting some of the original meandering tendency shown on the map in Figure 10. The study reach is approximately 4,000 ft in length, 2,000 ft upstream and downstream of the county road bridge. The stream is unstable, with bank erosion and significant channel widening. Several areas of massive bank failures were identified, and these failure sites, along with bed and bank erosion, provide a high sediment yield to the downstream.

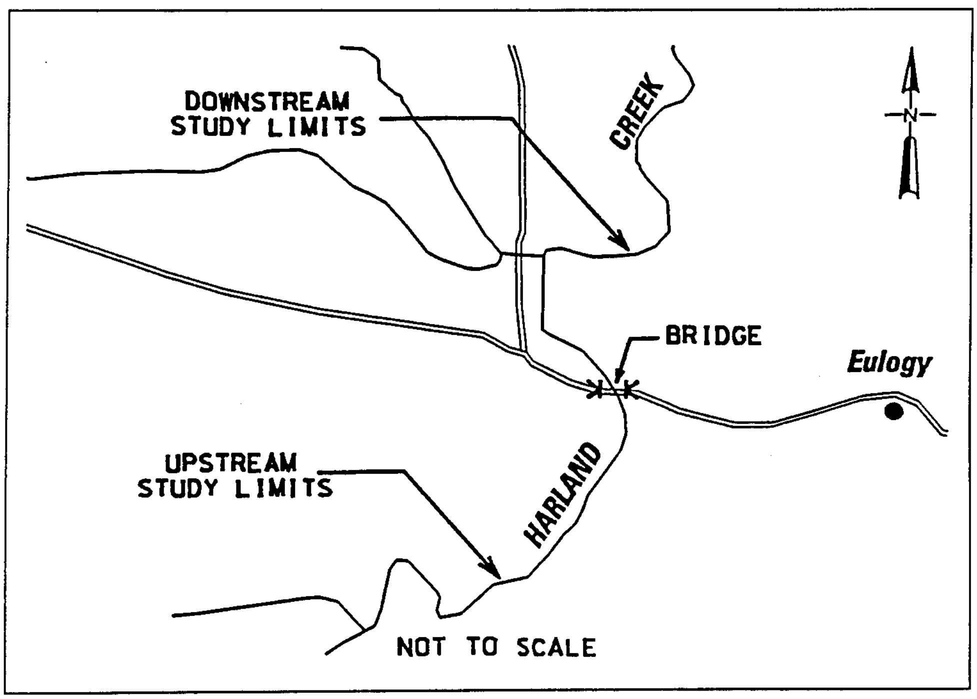



Figure 10. Harland Creek, Site 1

The site was chosen because of the mixed bed load, the fact that surveys were made before and after riprap stabilization measures were constructed in the reach, and a major reservoir planned immediately upstream of the site. Presently, stream gauging in the reach is installed. The watershed area at the site is approximately 27 square miles. HEC-1 hydrology and HEC-2 hydraulics were developed by Northwest Hydraulic Consultants, Inc. (1989). Portions of the study reach were surveyed during 1991 for bank stabilization construction planning. The 1992 field data will allow a comparison of the existing conditions with the previous contractor analyses, and provide a baseline of field information for comparison with the 1993 surveys, which were made after the channel stabilization was constructed.

### Fannegusha Creek

Site 2 is located on Fannegusha Creek, also in the Black Creek watershed, and can be found on the Coila quadrangle map in T16N, R3E, Sections 1 and 2. As shown in Figure 11, the study reach is approximately 4,000 ft in length, 2,000 ft upstream and downstream of a county road bridge. Two low-drop structures are planned for the site, one immediately downstream of the bridge and the other approximately 2,000 ft downstream of the bridge. The stream is presently unstable, and it has been reported that the county bridge has been closed since January 1992 due to channel widening. Initial

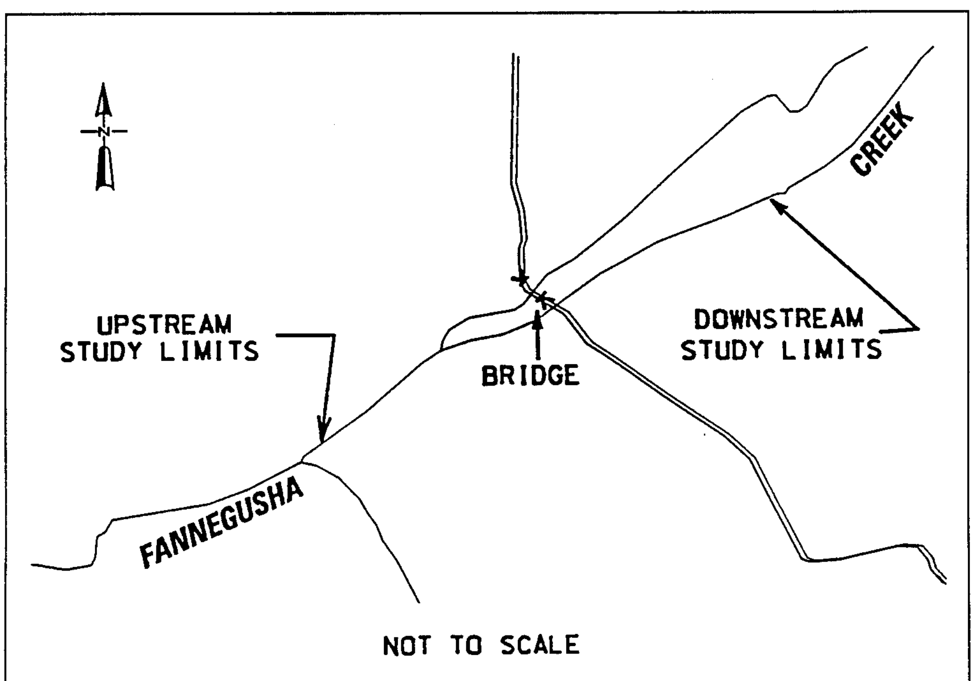



Figure 11. Fannegusha Creek, Site 2

observations indicate that the channel will continue to widen without stabilization measures due to a downstream oversteepened reach.

Watershed area at the site is approximately 18 square miles. HEC-1 hydrology and HEC-2 hydraulics were developed by Northwest Hydraulic Consultants, Inc. (1988). This reach was chosen as representing a very unstable sand bed channel. The 1992 and 1993 field data collection will begin to establish baseline data from which evaluation of the effects of the two proposed low-drop structures can be made.

### Abiaca Creek

Five sites have been selected in the Abiaca Creek watershed, and these sites can be found on the Seven Pines quadrangle map. Water Engineering and Technology, Inc. (WET), (1989b) prepared HEC-1 hydrology and HEC-2 hydraulics based on a survey provided by the U.S. Army Engineer District, Vicksburg. WES (Freeman et al. 1992) completed a HEC-6 analysis of Abiaca Creek (1992). The drainage area of the watershed is about 100 square miles, and SCS reservoirs control approximately 60 percent of the watershed. Coila Creek is the principal tributary to Abiaca Creek, and this watershed is approximately 76 percent controlled. Upstream of the Coila Creek confluence, Abiaca Creek is about 49 percent controlled. This watershed supplies a downstream wildlife area, but has been severely affected by sand and gravel mining.

Site 3 is shown in Figure 12, and is located in T17N, R3E, Section 20, at the Highway 17 crossing of Abiaca Creek. The approximate watershed area at this site is 26.5 square miles. This site was selected because of the relative stability of the channel at this location, particularly in comparison with the downstream sites that have been severely impacted by gravel mining. The streambed at Site 3 is primarily a sand bed with minor amounts of gravel, and the banks are generally well vegetated with mature vegetation down to the low-water surface. However, erosion of the outside bank of the bendways was noted.

Site 4 is on Abiaca Creek and extends approximately 4,000 ft upstream from the confluence of Coila Creek as shown in Figure 13. This site is located in T17N, R2E, Section 4, and has a watershed area of approximately 44 square miles. This site is also located approximately 1.8 miles downstream of a major sand and gravel processing operation that can be associated with increased supply of suspended and bed material load. Stream banks in this reach are relatively stable, and the bed gives the appearance of an aggraded reach.

Site 5 is located on Coila Creek, a tributary to Abiaca Creek in T17N, R2E, Section 4. The site extends upstream approximately 4,000 ft from the confluence with Abiaca Creek as shown in Figure 13. The site has a watershed area of approximately 42 square miles, very similar to Site 4, and allows

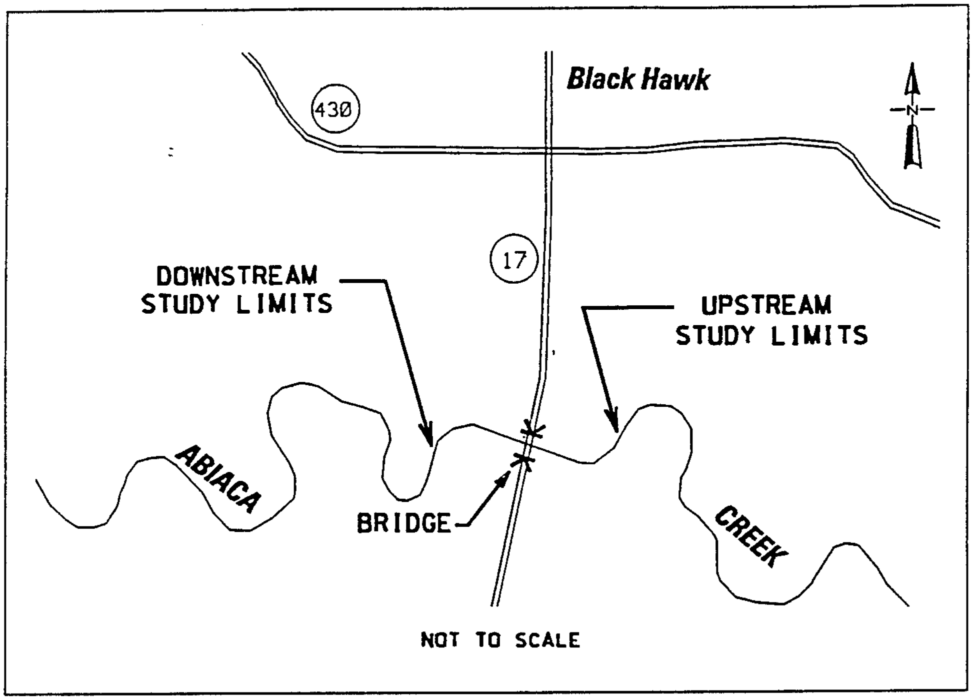



Figure 12. Abiaca Creek, Site 3

the comparison of two almost equal size drainage basins. A high proportion of the Coila Creek basin is controlled by SCS reservoirs, and the gravel mines on Coila Creek are not as active as the Abiaca Creek sites.

Site 6 is located on Abiaca Creek as the stream emerges from the hill line into the flatter Yazoo Delta in T17N, R1E, Sections 13 and 14, as shown in Figure 14. Drainage area at this location is approximately 99 square miles. This is the site of the Pine Bluff gauging station with records from 1963 to 1980. This station has recently been reactivated and includes a pumped sediment sampler. The study reach extends approximately 4,000 ft downstream of the Pine Bluff gauging station.

Site 21 is in T17N, R1E, Section 18, near the mouth of Abiaca Creek at Highway 49 as the stream enters the wildlife area (Figure 15). The Vicksburg District has designed a sediment trap basin at this location by setting the levees back and allowing frequent overflow of the stream. The reach is approximately 4,000 ft in length.

Channelization of the lower basin during the early 1920's set in motion a complex cycle of channel incision, and continuing mining of the watershed complicates rehabilitation of the watershed. The District is presently designing sediment trapping immediately upstream of the wildlife area. The complexity and importance of the watershed emphasize the purpose of these four study sites. The District has suggested an additional study site at the downstream extent of the sediment trapping facility for future years.

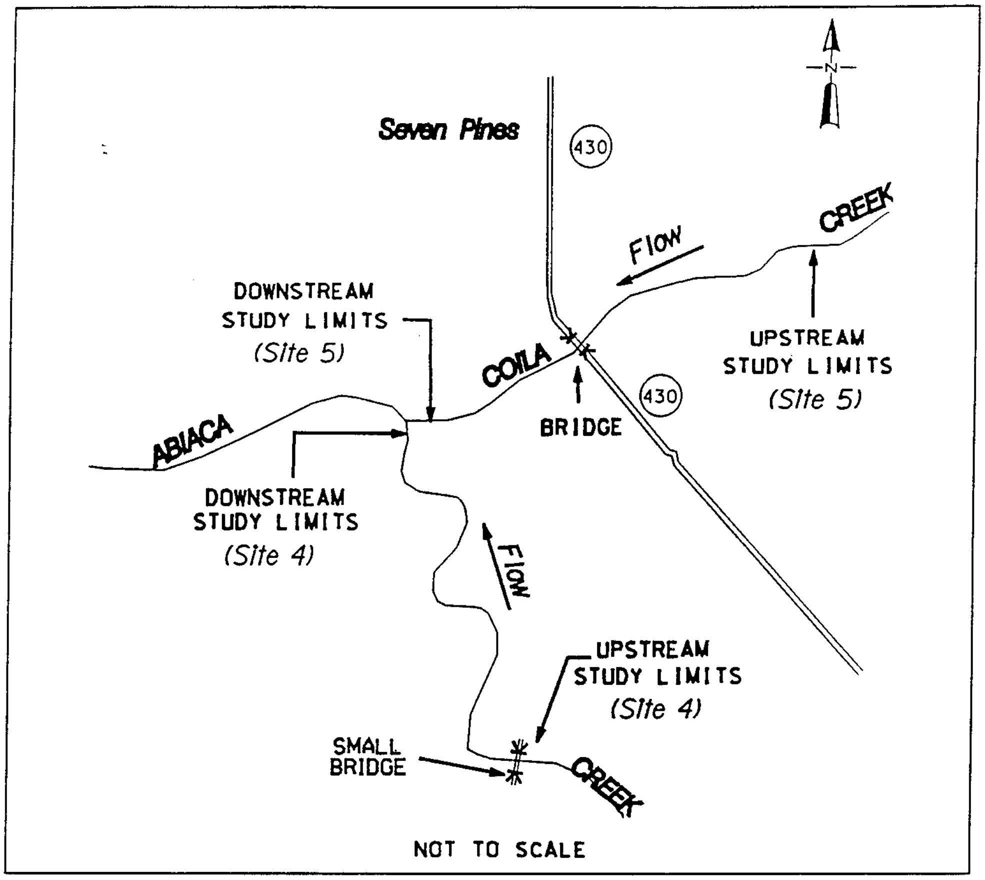



Figure 13. Abiaca Creek, Site 4, and Coila Creek, Site 5

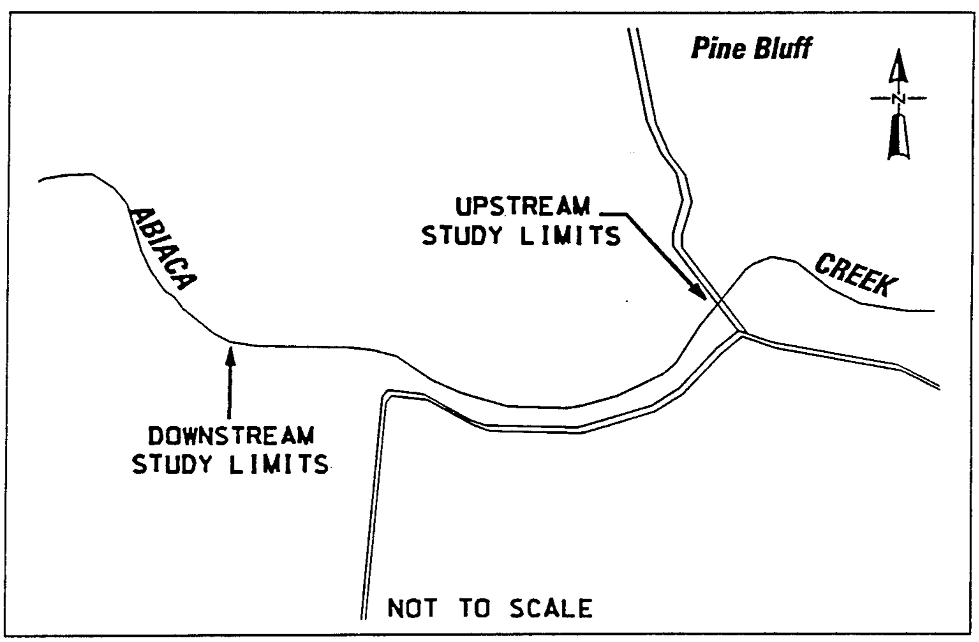



Figure 14. Abiaca Creek, Site 6

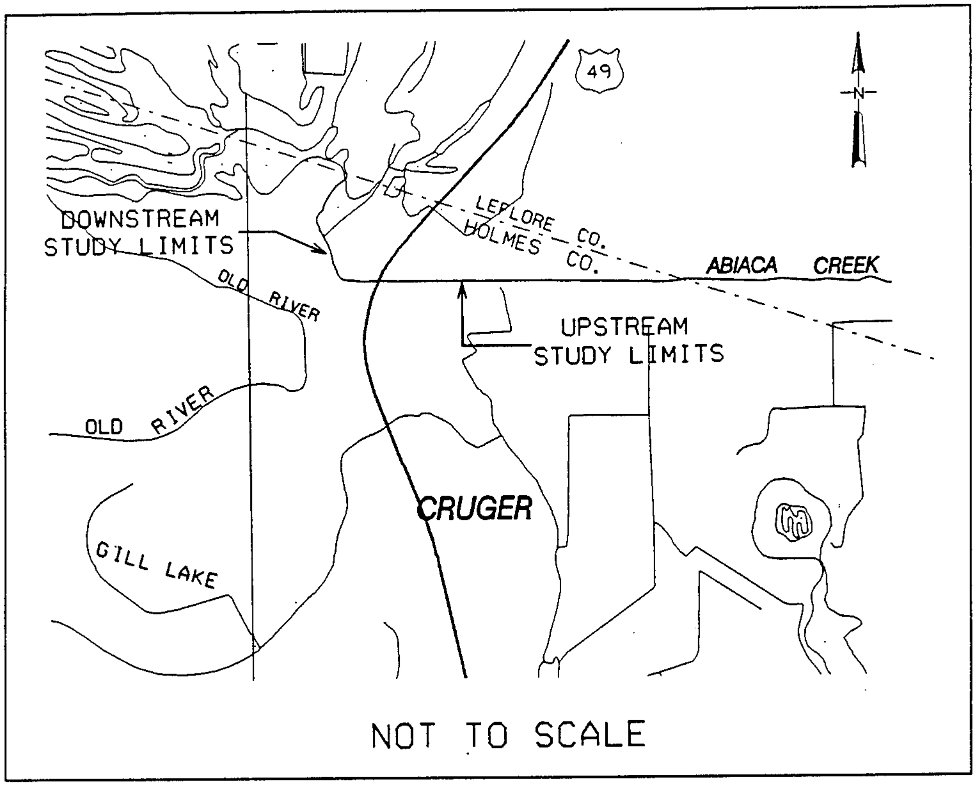



Figure 15. Abiaca Creek, Site 21

## Coldwater River Basin

The hydrology of the Coldwater River basin was developed by Lenzotti and Fullerton Consulting Engineers, Inc. (1990) using HEC-1. Surveys of the channels were completed in 1991 by the District, and HEC-2 hydraulics has subsequently been developed.

Site 7 is located on Nolehoe Creek in the Coldwater River basin near the community of Olive Branch, MS. The site is located on the Hernando quadrangle map, T1S, R7W, Section 35, and has a drainage area of approximately 3.7 square miles. The study reach, shown in Figure 16, is approximately 4,000 ft in length, extending downstream from a box culvert. The channel is extremely unstable and is deeply incised. Bed material load ranges from sand to particles in excess of 30 mm. Two low-drop structures are planned for the reach; however, permission to construct the structures has not been received from the landowner. Stream stage recording stations have been recently installed by WES at the downstream roadway culvert.

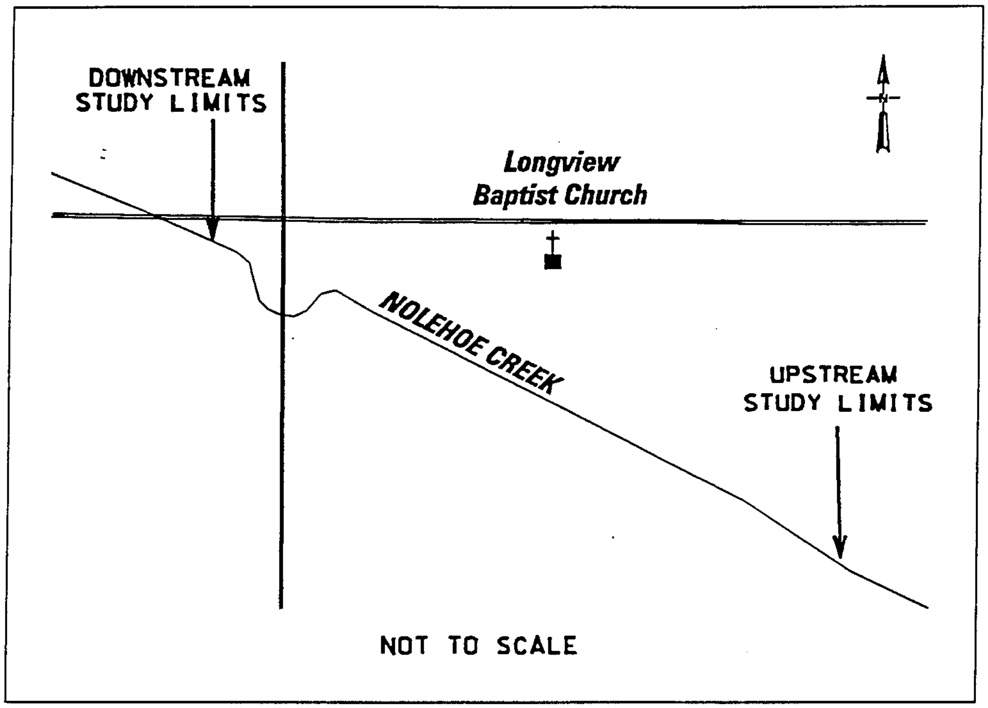



Figure 16. Nolehoe Creek, Site 7

This incising reach is between upstream and downstream box culverts, and the reach is representative of suburban development in the metro-Memphis area. An interview with a local landowner confirmed that a major cutoff of the channel had been made in the last 10 years. These conditions are typical of the result of ill-planned local development improvements, and the documentation of the resulting problems may be of value in assisting future local drainage planning.

Site 8 is on Lick Creek in the Coldwater River basin, approximately 2 miles south of Olive Branch, MS, at the site of an anticipated high-drop structure, which is planned to protect the Highway 305 bridge. Shown in Figure 17, the study reach is approximately 4,000 ft in length, 2,000 ft upstream and downstream of the bridge, in T2S, R6W, Section 3. This site is also on the Hernando quadrangle map. Watershed area is approximately 8.5 square miles. Stream gauging is planned for the future at this site; however, no stream gauging is presently available.

This site was selected to monitor the effects of a planned high-drop structure. Lick Creek is actively degrading downstream of the bridge, and incision has begun upstream of the bridge.

Site 9 is located on Red Banks Creek in the Coldwater River basin. As shown in Figure 18, the study reach extends approximately 2.5 miles upstream from the bridge on the county road between the communities of Warsaw and Watson, MS. This site can be located on the Byhalia quadrangle map, T3S, R5W, Section 24, and R4W, Sections 19 and 20, and has a

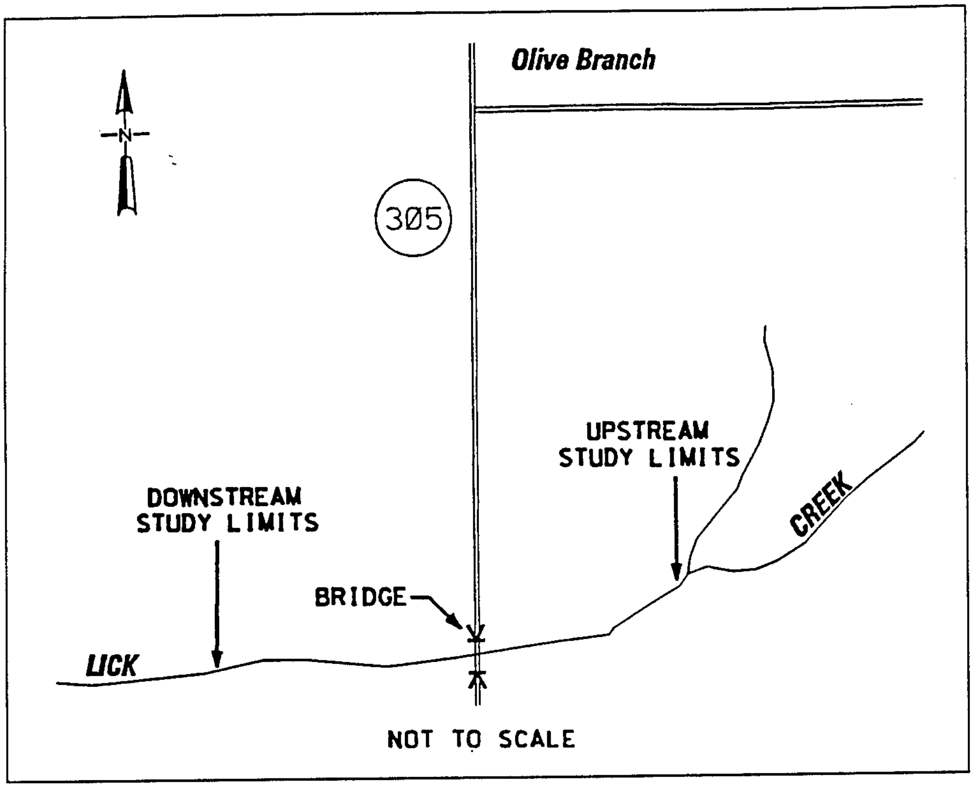



Figure 17. Lick Creek, Site 8

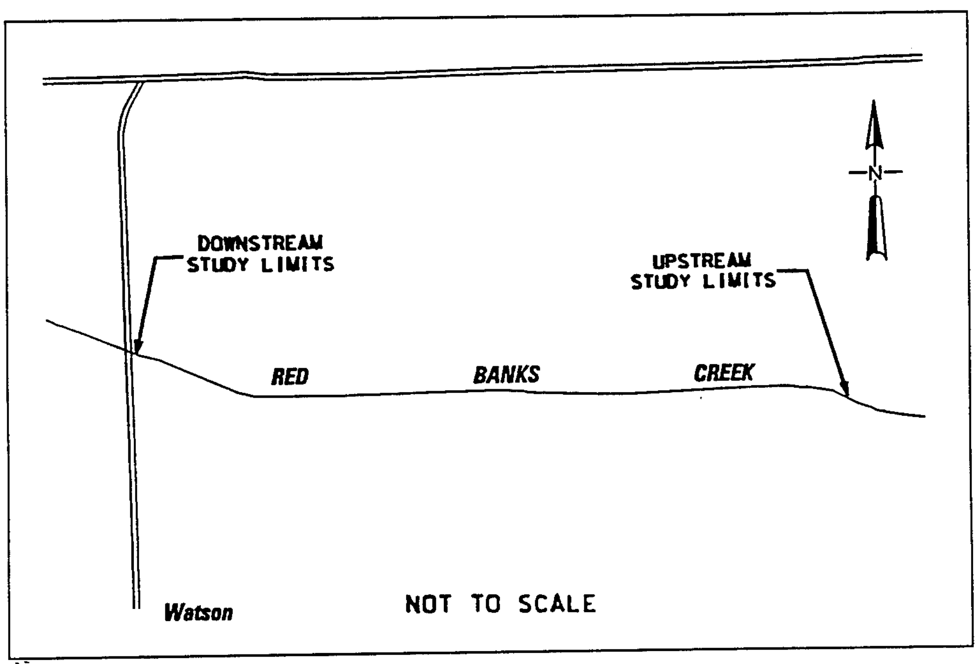



Figure 18. Red Banks Creek, Site 9

watershed area of approximately 28 square miles. The bed sediment load is sand, and the stream flows in a deeply incised and widened, straight channel resulting from earlier channelization. Site 9 is unique in that it is the only DEC monitoring site using chevron dikes and longitudinal dikes for channel stabilization.

Site 10 is on Lee Creek in the Coldwater River basin, approximately 6 miles north of Victoria, MS. The site can be located on the Byhalia quadrangle map in T2S, R4W, Sections 9 and 10. As shown in Figure 19, the study reach extends approximately 2,000 ft upstream and downstream of the highway bridge. The channel is relatively stable and is transporting minor amounts of gravel in a sand bed. Upstream of the bridge, the channel exhibits some meandering and apparently has not been channelized in this reach. Downstream of the bridge, the channel is stable with mature, 14-in.-diameter trees near the low-water surface. The remnants of spoil piles indicate that the lower channel has been channelized. This reach provides an excellent opportunity to document a stable, channelized, sand bed stream.

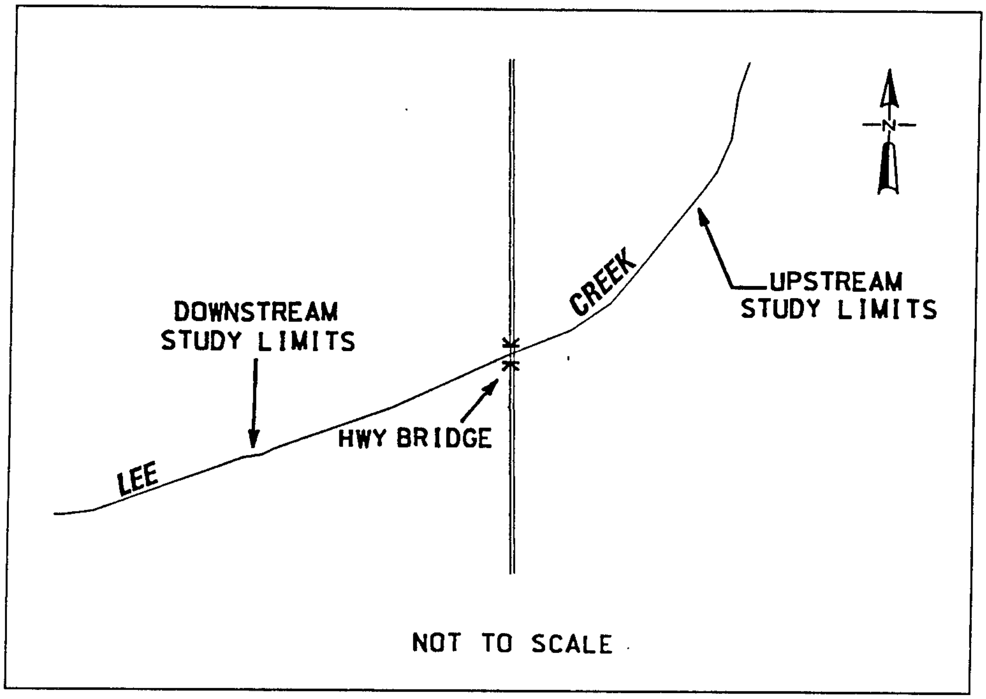



Figure 19. Lee Creek, Site 10

### Hickahala Creek

Hickahala Creek is a major tributary to the Coldwater River with a drainage area of approximately 230 square miles at the confluence with the Coldwater. Simons, Li and Associates (SLA) (1987) conducted field reconnaissance, developed HEC-1 hydrology and HEC-2 hydraulics, and conducted sediment transport analyses for the Vicksburg District in 1987. The hydraulic computations were prepared based on channel geometry from 1968 and 1985 surveys. Additional surveys have been made in selected areas to assess the effects of stabilization measures on James Wolf Creek, and construction-related surveys have been conducted on James Wolf and upper Hickahala Creeks. USGS stream gauge records are available near the mouth of the watershed.

Site 11 is located in the upper watershed of Hickahala Creek, a watershed area of approximately 9 square miles. The site is located on the Tyro quadrangle map in T5S, R5W, Sections 2 and 3, a portion of which is shown in Figure 20. The site begins at a county road bridge and extends downstream to the confluence with the South Fork, and continues downstream on Hickahala Creek for approximately 1.25 miles. The total study reach is approximately 2 miles in length and includes three existing structures. The lower portion of the study reach is actively incising into a clay, cohesive bed. The upstream portion of the study reach is relatively stable with a sand bed. The reach was selected to monitor the response of the complex of structures.

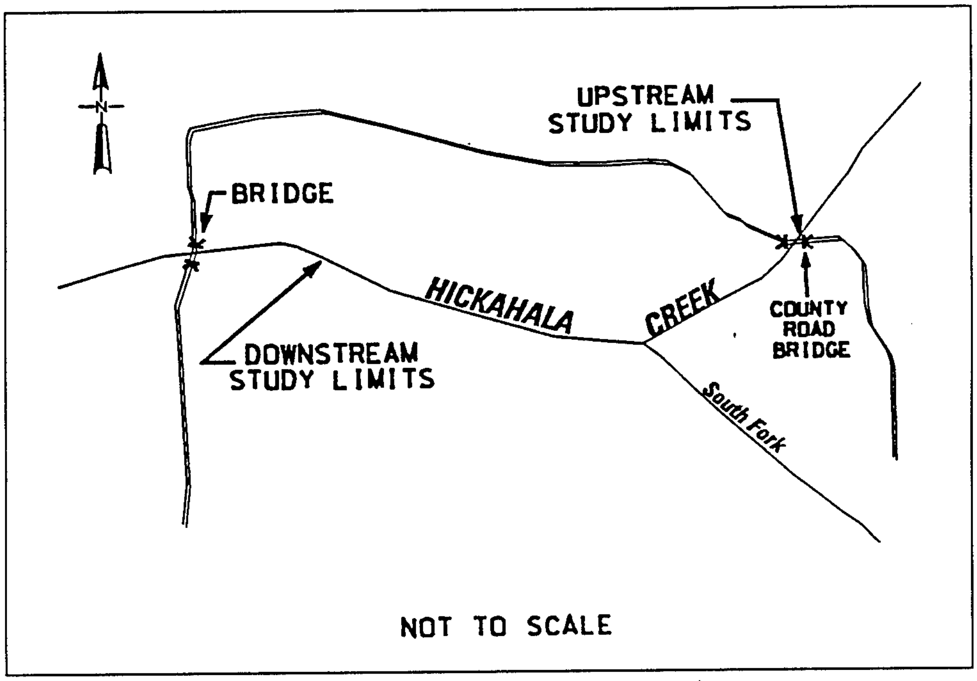



Figure 20. Hickahala Creek, Site 11

Site 22 (Figure 21) is located at the mouth of Hickahala Creek near Senatobia, MS. The present channel is severely choked by debris and a logjam. Channelization of the reach to relieve flooding is planned.

Site 19 is located in the Hickahala Creek watershed on James Wolf Creek. At this location, James Wolf has a drainage area of approximately 11 square miles; however, it is extremely deep and wide. The site is located on the Tyro quadrangle map in T5S, R5W, Section 28. The study reach, shown in Figure 22, extends downstream of the east-west county road for a distance of approximately 4,000 ft encompassing a low-drop structure. This low-drop

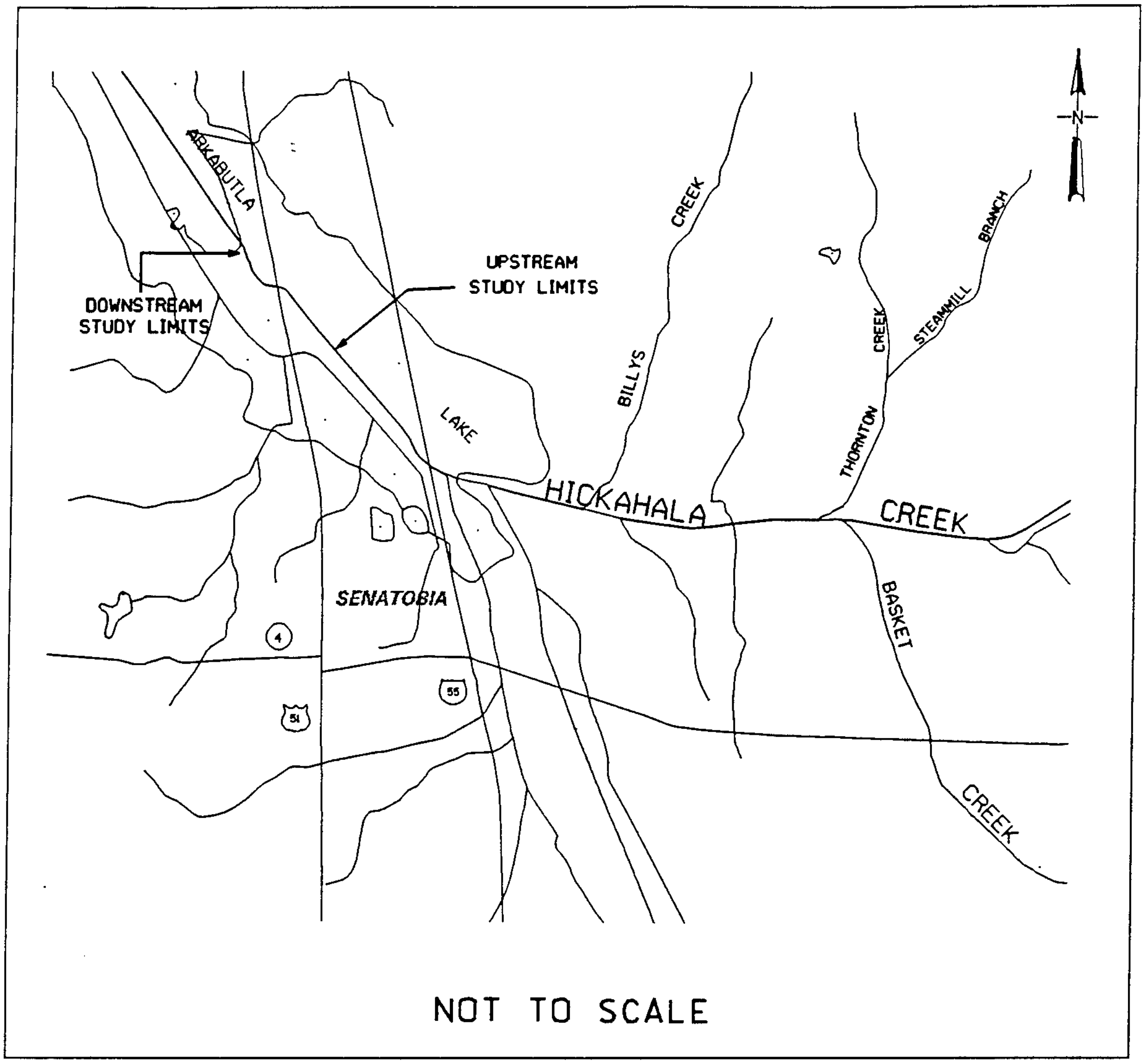



Figure 21. Hickahala Creek, Site 22

structure appears to be stabilizing the bed of the stream; however, the banks remain unstable due to the significant depth. The stream is sand bed, and at low-flow conditions, the channel may be nearly dry. The drop structure on James Wolf Creek monitoring reach has required significant repair since construction and is presently in need of additional repair. Two additional drop structures were constructed on James Wolf Creek downstream of the monitoring reach during 1993.

### Burney Branch

Site 12 (Figure 23) is located on Burney Branch near Oxford, MS. The study reach begins at the Highway 7 crossing of Burney Branch and extends downstream for a distance of approximately one mile through a reach containing two SCS high-drop structures. The drainage area of Burney Branch at this

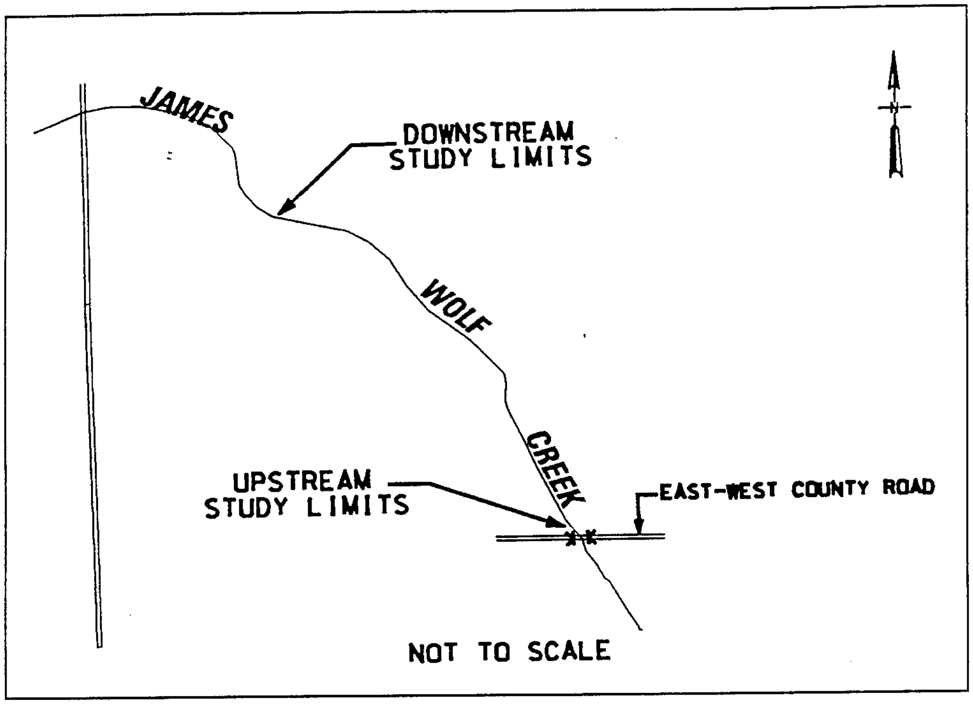



Figure 22. James Wolf Creek, Site 19

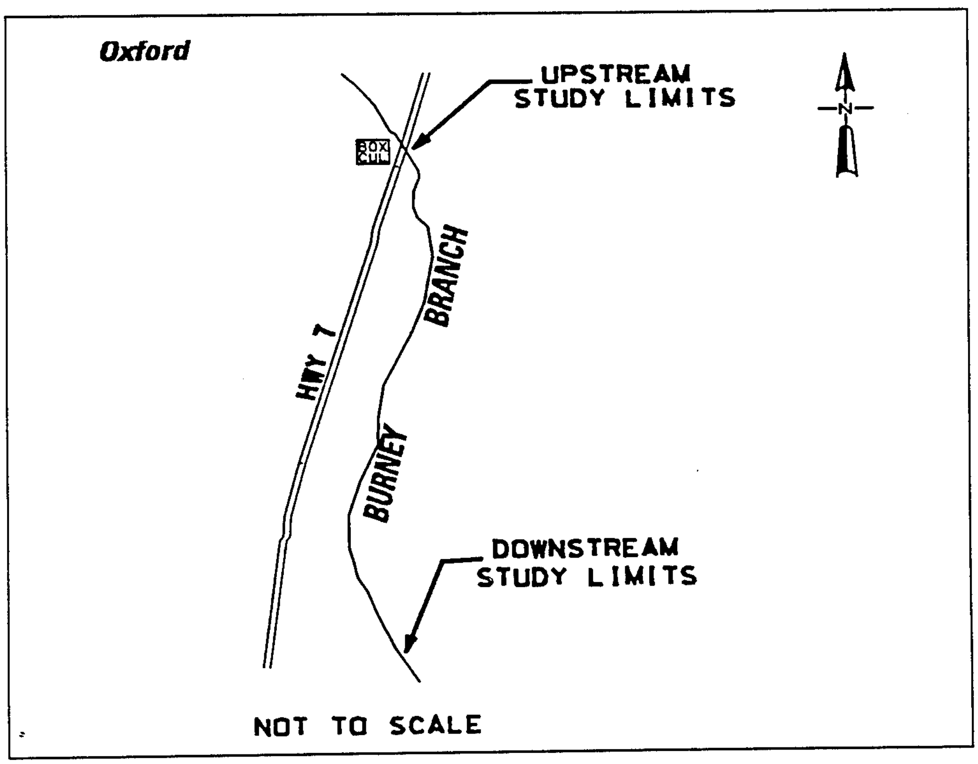



Figure 23. Burney Branch, Site 12

location is approximately 10 square miles. The site can be located on the Oxford quadrangle map, T9S, R3W, Sections 4 and 9.

The two high-drop structures have been very successful in rehabilitating this reach of Burney Branch. Both structures were constructed in 1982 by SCS, and the effects of the structures on the channel were surveyed and analyzed by Schumm, Harvey, and Watson (1984). These structures were designed to contain the 100-year discharge and include the provision for floodplain storage using valley dams in conjunction with each structure. The original design of the structures provided for a bed slope of 0.0008 between structures, based on Lane's tractive stress analysis. The 1984 surveyed bed slope was 0.0012, indicating that the upstream sediment yield was greater than planned. Since 1984, several major channel stabilization projects have been constructed upstream. The survey made in January 1992 will document the effects of changes since 1984 and will provide data with which to evaluate channel change as sediment supply is reduced. Channel stabilization under conditions of reducing sediment supply is a situation that will be faced as the success of the DEC programs is realized. Potentially, upstream stabilization can cause stability problems downstream.

### Hotophia Creek

Site 13 is located on Hotophia Creek, west of Oxford, MS. As shown in Figure 24, the site encompasses approximately 2 miles of Hotophia and Marcum Creeks and is located on the Sardis quadrangle map T9S, R6W, Sections 1 and 2, and in T9S, R5W, Section 6. The watershed area at the site on Hotophia Creek is approximately 17 square miles. A USGS gauging station is located at the Highway 315 bridge crossing of the creek. The study reach includes the confluences of Marcum Creek and Deer Creek. A low-drop on Hotophia Creek is at the downstream extent, two low-drops are on Deer Creek, a high-drop is located on Hotophia Creek immediately downstream of the Marcum Creek confluence, and a low-drop is located on Marcum Creek. Two additional high-drops, one within the reach and one upstream of the reach, were under construction at the time of the 1993 surveys. WES-installed stream gauging is available at the first high-drop.

Hotophia Creek was channelized in 1961, and was surveyed by the District in 1985. WET conducted field reconnaissance in 1986 and prepared HEC-1 hydrology and HEC-2 hydraulics (WET 1987b). This site is important because of the complexity of the various constructed elements, and the need to document channel response to the high-drop grade control. In addition, data from Burney Branch and Hotophia Creek provide the opportunity for a comparison of data from adjacent watersheds.

### Otoucalofa Creek

Site 14 is on Otoucalofa Creek, east of Water Valley, MS. The study

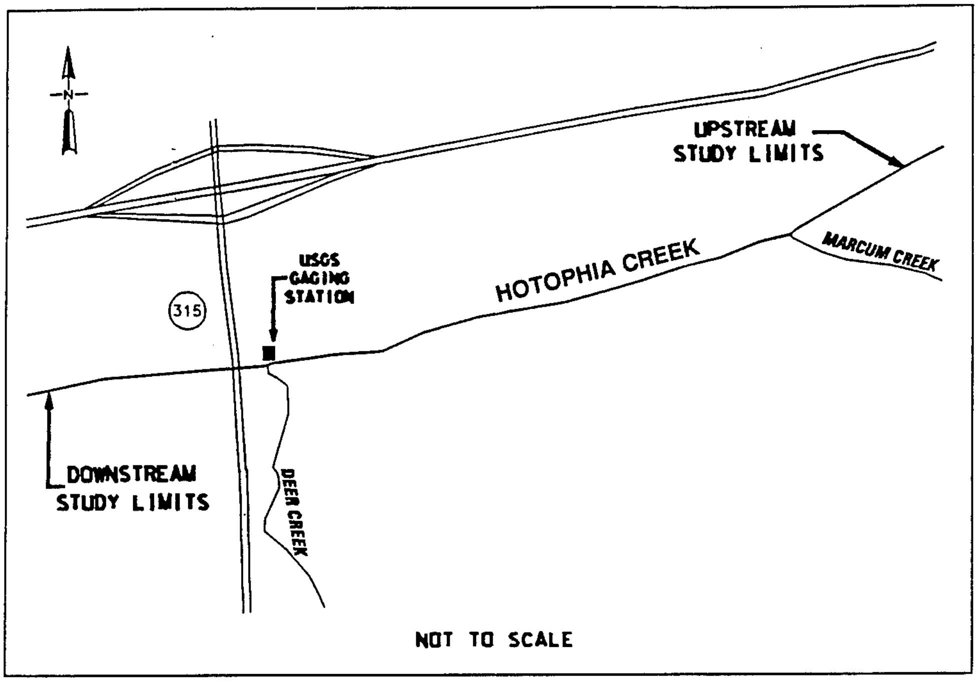



Figure 24. Hotophia Creek, Site 13

reach is 4,000 ft in length, 2,000 ft upstream and downstream of the Mt. Liberty Church Road bridge, in T11S, R3W, Sections 4 and 5, of the Water Valley quadrangle map as shown in Figure 25. Watershed area at the site is approximately 41 square miles. No stream gauging is presently available; however, this site will be gauged at the bridge in the future.

A low-drop structure is proposed for the future, and presently riprap dikes and longitudinal dikes are constructed throughout the reach. The reach was observed to be actively incising, and this incision is occurring at an elevation below the recently placed stone. This site provides a unique opportunity to observe the stone subjected to severe degradation.

Site 15 is on Sarter Creek, which is a tributary of Otoucalofa Creek upstream of Site 14. Sarter Creek is located on the Paris quadrangle map and has a watershed area of approximately 6.4 square miles. The study reach is 4,000 ft in length and is almost completely straight as a result of previous channelization, as shown in Figure 26. This site extends downstream of the Highway 315 bridge. The site is unusual in that it had remained relatively unchanged since channelization; however, it is apparent that headcutting affected the reach in 1993.

### Batupan Bogue

Batupan Bogue watershed contains three study sites: Perry Creek, Sykes Creek, and Worsham Creek. A USGS stream gauge is located at the mouth of Batupan Bogue with a drainage area of approximately 245 square miles. In

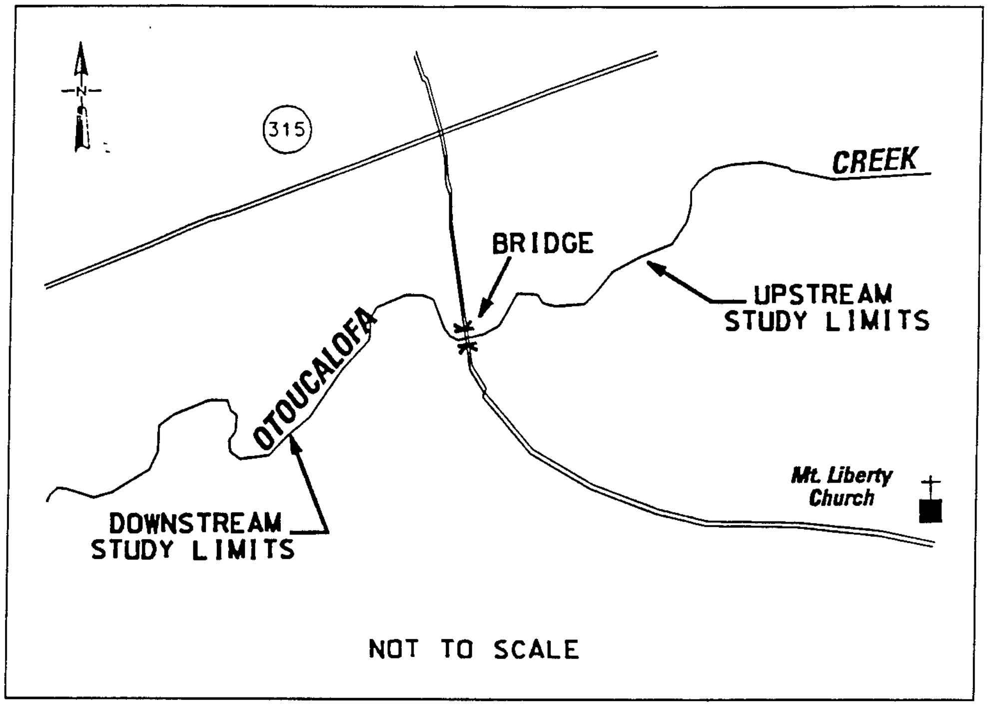



Figure 25. Otoucalofa Creek, Site 14

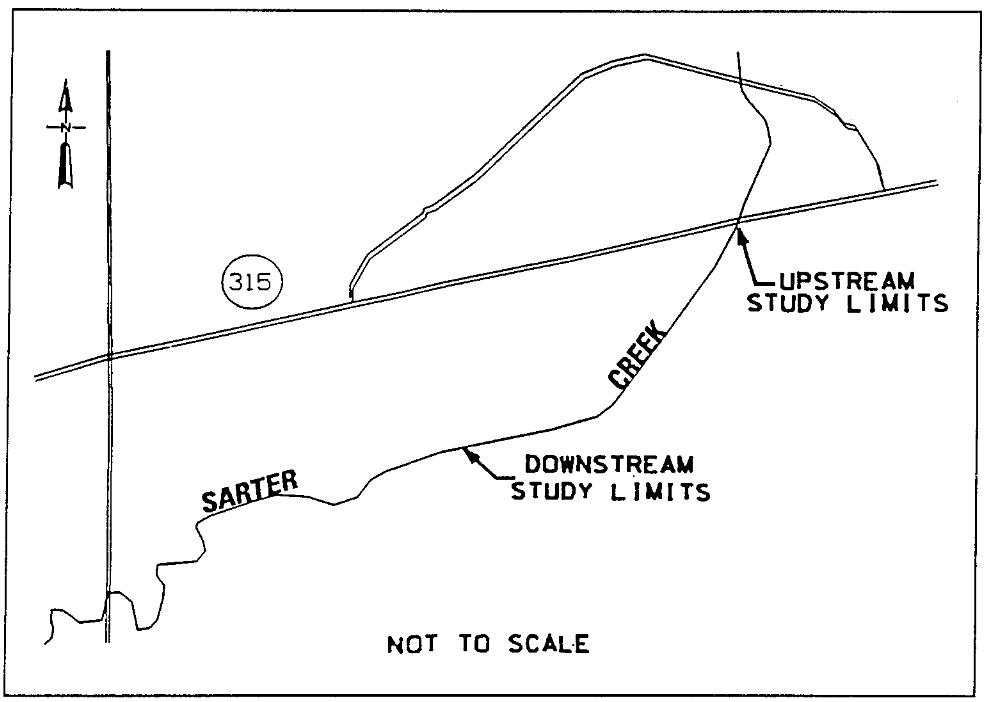



Figure 26. Sarter Creek, Site 15

1987 WET (1987a) prepared HEC-1 hydrology to match then-existing Federal Emergency Management Agency hydrology and HEC-2 hydraulics based on 1987 surveyed cross sections. Numerous stabilization structures have been constructed since 1988, and surveys have been conducted in association with planning for those structures.

Site 16 is located on Perry Creek as shown in Figure 27. The study reach begins approximately at the northern line of T21N, R4E, Section 1, and continues upstream through Sections 2 and 11. The study reach is located on the McCarley quadrangle map. The entire study reach length is approximately 2 miles, as shown in Figure 27. Four low-drop structures were under construction during the 1993 surveys. This site will allow the investigation of the effects of four structures in series. The site is unique because within the study reach the channel moves from a deeply incised stream to a stream that might have existed prior to channelization. Plans are to gauge the stream at the I-55 box culvert downstream of the study reach.

Site 17 is located on Sykes Creek as shown in Figure 28. The study reach extends 2,000 ft upstream and downstream of the county road bridge across Sykes Creek located in T21N, R5E, Sections 27, 33, and 34. This site is found on the McCarley quadrangle map. Gauging is presently available for the approximately 12.3-square-mile watershed area at the county road bridge.

Site 18 is a study reach encompassing portions of Worsham Creek, West Fork, and Middle Fork as shown in Figure 29. The site is located on the Duck Hill quadrangle map in T20N, R6E, Sections 14, 15, 16, 21, 22, and 23. Total stream length is approximately 3.5 miles, and the watershed area at the confluence is approximately 19 square miles. The streams are deeply incised and active. Ten low-drop structures are constructed in this study reach.

### Long Creek

Site 20 is located on Long Creek, T10S, R6W, Sections 4, 5, and 8, as shown in Figure 30. The site can be found on the Oakland quadrangle map and has a watershed area of about 11 square miles. Three low-drop structures existed prior to 1991, and the fourth was constructed in 1993 as the downstream limit of the monitoring reach. A fifth structure was constructed in 1993 downstream of the reach. The study reach is approximately 2 miles in length, extending downstream from the eastern boundary of Section 4. The site also includes a reach that has been monitored by the Vicksburg District and includes the bank stability sites reported by Zevenbergen et al. (1990). Portions of this reach are very unstable and are presently incising. The reach downstream of the existing structures has a clay bed that is slowly incising. This clay bed has a very narrow, deeply incised channel along some reaches and was inundated by the structures built in 1993.

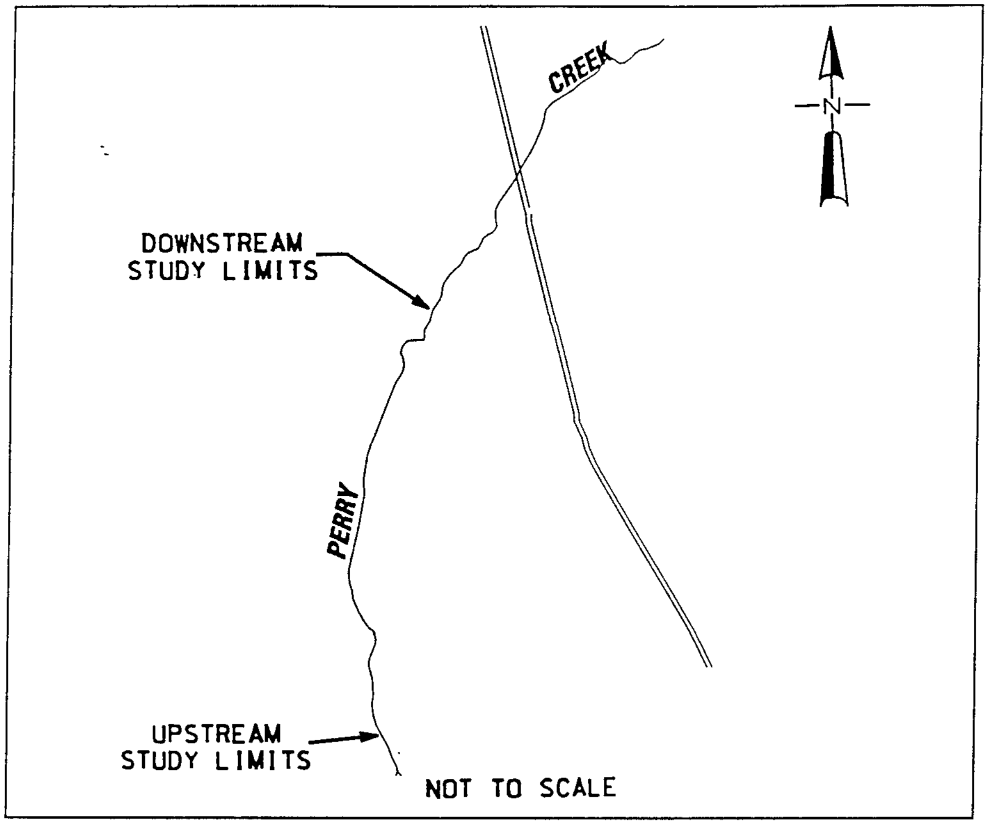



Figure 27. Perry Creek, Site 16

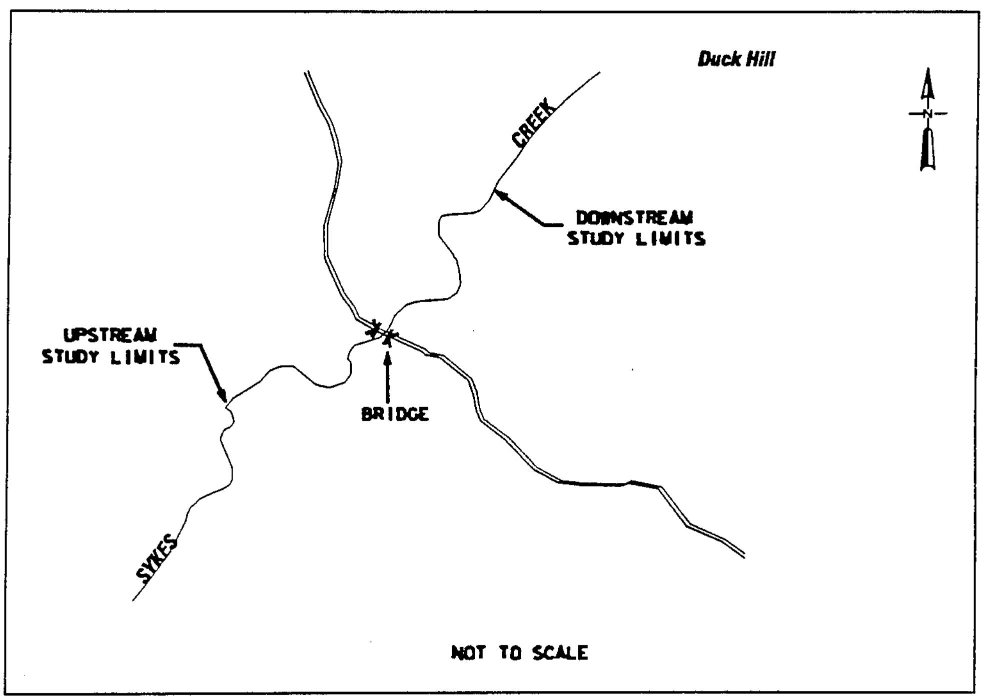



Figure 28. Sykes Creek, Site 17

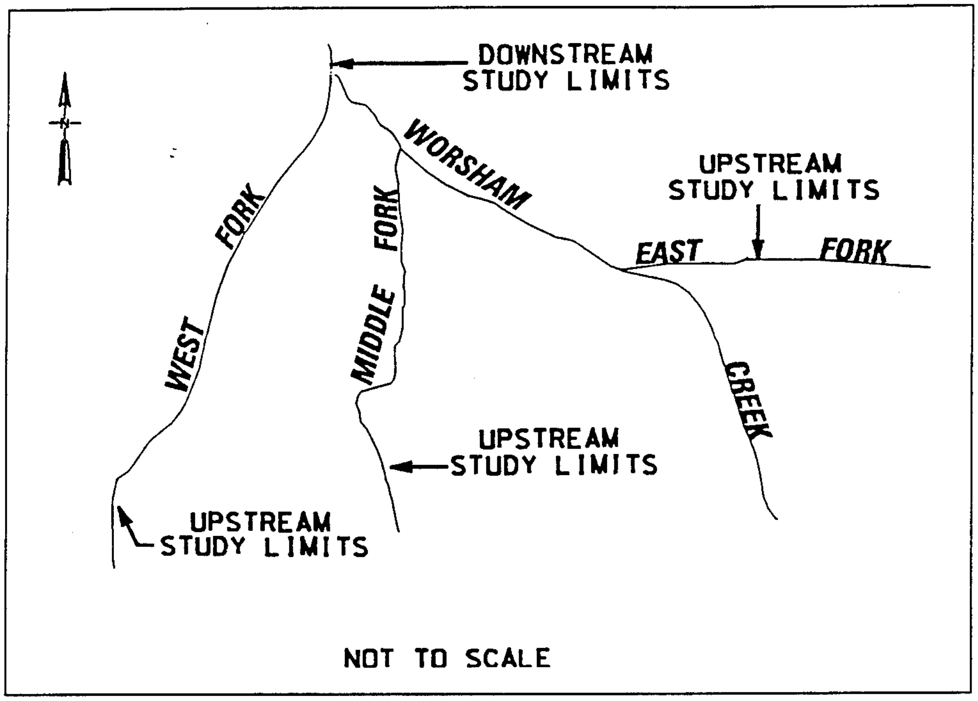



Figure 29. Worsham Creek, Site 18

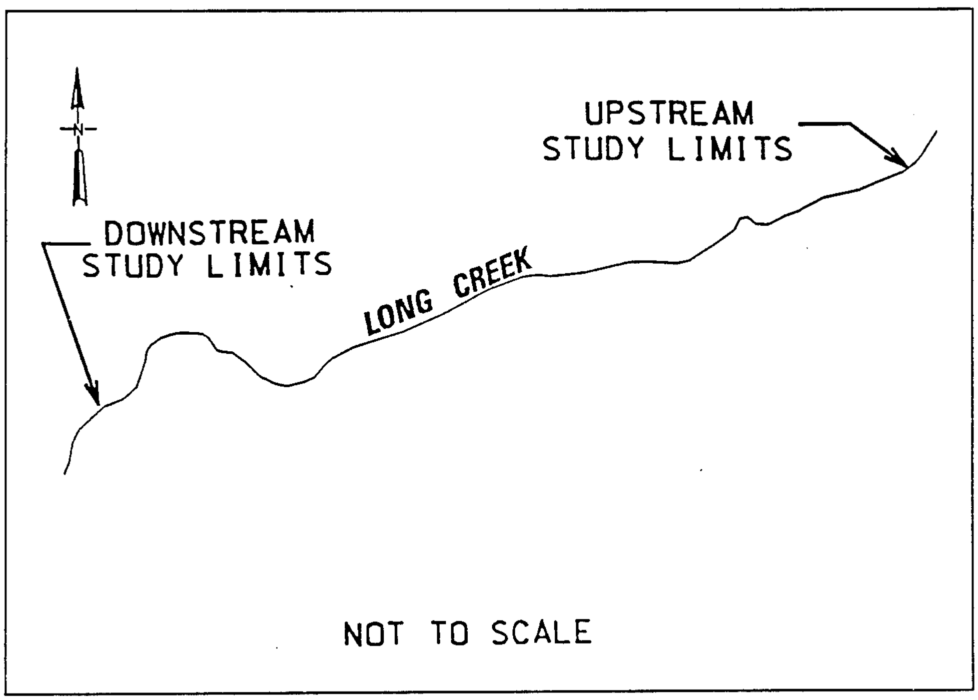



Figure 30. Long Creek, Site 20

# Monitoring and Analysis of Incised Streams

The purpose of this section is to present the proposed approach to monitoring and analysis of incised DEC streams. This approach provides a basis for monitoring the effects of channel stabilization and rehabilitation measures constructed in the DEC watersheds of the Yazoo Basin. The approach is not a solution to the watershed problems, but it does provide guidance for the CSU monitoring program and may serve as a tool with which to plan watershed rehabilitation and to evaluate the effects of the construction. The approach has been developed over a period of time in cooperation with previous work with the Vicksburg District (WET 1989a), WES, SCS, ARS and other investigators.

It is important that the watershed problems be placed in the context of the entire watershed and that a preliminary assessment of these interrelationships be established. The watershed problems can be identified in a number of ways. Interviews with local government officials and local landowners and field reconnaissance generally will pinpoint the major problems in a watershed. Most of the watersheds will have all or some of the following types of problems:

*a.* Watershed erosion.

*b.* Channel erosion.

*c.* Agricultural and urban flooding.

*d.* Sedimentation of reservoirs and agricultural land.

*e.* Damage to infrastructure (bridges, pipelines, etc.).

*f.* Environmental problems.

### Data sources

Current data represent a single point in time. The addition of historical data enables trends to be identified and provides further information on rates of change in the watershed. Data from watersheds that may be affected by or influence the project watershed also should be collected.

In previous investigations of Yazoo Basin watersheds, the following agencies have been sources of historical information: Vicksburg District, WES, U.S. Army Engineer Division, Lower Mississippi Valley, SCS, ARS, USGS, Mississippi State Highway Department, State Archives, State Geological Survey, State Land Office, county offices, city and municipality offices, state and local historical societies, newspapers, and local drainage and levee districts.

The following types of information can be obtained from these sources: channel and reservoir surveys, flood history, watershed work plans, bridge plans and subsequent surveys, watershed erosion information, geological data, drainage district records, land use records, previously installed erosion mitigation measures, land ownership information, historical sediment yield data, General Land Office Survey plats, and aerial photographic coverage. Current aerial photography and channel profile and cross-section surveys are essential for every project. This is not an exhaustive list of available data, but it does provide a guide to the types of information that may be available for a specific watershed.

### Field investigation

A field investigation should attempt to place the channel reach of interest in a watershed context. Aerial reconnaissance, or at least a drive to accessible points within the watershed prior to ground level investigation, provides a much broader view of the watershed problems. Field observations can be recorded on aerial photographs in the field, and features along the channel should be located with the aid of a hip chain or some other measuring device at a measured distance from a mapped point such as a bridge or structure. This will allow field observations to be related to subsequent analyses.

Field work should be carried out with two criteria: (a) collect as much information as possible when in the field on the assumption that returning to the given site will not be possible, and (b) record everything using photographs, notes, or sketches. Recalling detail after leaving the field is difficult if not impossible. At the end of a day in the field, it is important to try to summarize and write down the daily observations. In many cases, the significance of individual observations becomes apparent only when a sufficient amount of information has been obtained. While in the field, it is important to generate working hypotheses and to test them. It should be apparent that the primary objective of doing field work is to obtain an understanding of the system being investigated, and not to just collect data (WET 1989b).

One of the primary purposes of the project is to monitor the performance of the grade control and bank stabilization features in the streams. Observations to be recorded include photographs, movement of the riprap since construction, exposed filter material, erosion upstream and downstream of the structure, and general effectiveness of the structure. Other parameters that describe channel and bank stability such as various survey and sediment data provide information with which to monitor the effects of these structures on the channels.

Planform is a description of the stream geometry as viewed from above. A reach may be straight, braided, or meandering. The influence of man-made or geologic factors on the channel planform should be noted.

In the DEC watershed, geologic information is concerned with both the alluvial valley fill and the underlying Tertiary-age formations in the incised channels of the Yazoo Basin. The sediments that compose the banks of the channel have variable resistance to fluvial erosion, and these sediments are susceptible to gravity-induced failure. The types of materials that compose the streambanks should be noted. Many of the Tertiary-age formations beneath the more recent alluvial valley fill provide very little resistance to erosion. However, these sands are interbedded frequently with more competent shales. The shales are more resistant to erosion and frequently provide local base level control in the bed of the channel. Work by Grissinger and Murphey (1982) demonstrated considerable relief on the surface of the shales; therefore, caution should be exercised about the lateral continuity of shale outcrops if these outcrops are being considered as grade control elements. Lateral continuity must be established by drilling. Most of the shale units will erode only slowly, and there is reasonable evidence that knickpoints formed in the resistant units are relic features that have been exposed by the present cycle of erosion. Evidence for erosion of the shales generally can be found downstream if the outcrop is eroding. The evidence will be the presence of small pieces of shale in the bed of the channel some distance downstream of the outcrop. These small pieces should not be confused with the presence of large blocks of shale and blocks of iron-cemented sandstone that may be located immediately downstream of the outcrop. These larger blocks generally are relic features exposed with the outcrop that document erosion of the shales under a very different discharge regime than exists at the present (WET 1989b).

Sediment data were collected from the bed, berms, and banks of the stream. Data collection includes obtaining samples of material from the bed and banks for laboratory determination of grain size distributions and probing in the bed of the channel to determine the depth of sediment accumulation. In general, the bed material sample should be collected from the thalweg at the channel crossing and from midbar to obtain a representative material sample. The type of sediment located in the toe of the bank will provide information to determine if the bank will be undercut due to fluvial detachment of the noncohesive sediments.

The depth of sediment in the bed of the channel should be determined by probing in the thalweg at the surveyed cross sections and between the cross sections. Probing for sediment depth is important for two reasons: (a) the depth of sediment is correlated with the evolutionary stage of the channel, and (b) probing may reveal the presence of coarser sediments, outcrop, or clays below the surface.

Sediment samples of the materials in the tributary mouth bars will be collected to determine the sizes of sediment being introduced to the main stem. Observation of the tributary confluences has been useful to determine if the tributary and main channel bed elevations are equal. This can provide information on the recent aggradational or degradational history of that reach of the main channel. A perched tributary is generally evidence for recent activity in the main stem.

Channel morphologic data are obtained primarily from the channel surveys. These surveys include the shape of the cross section, the form of the longitudinal profile of the channel bed and top banks, the presence of terraces and longitudinal berms, and the presence of bed forms. Many of the historic surveys have biased the selection of cross section to where access to the channel is easiest. This can result in the selection of cross sections that are not very representative of the reach as a whole.

Knickpoints or knickzones in the bed of the channel are indicators of bed instability; therefore, the locations of these features are being surveyed. Terraces are former floodplains and, therefore, provide evidence of past degradation of the channel. Berms are aggradational features that are formed when the shear stress in the bed of the channel at the channel margins has been reduced to a level that allows the sandy bed material to be stabilized by mud drapes (Harvey and Watson 1988). The berms then are colonized by vegetation and may define the margins of the dominant discharge channel. The presence of berms is indicative of dynamic equilibrium in the channel. It is important to be able to document the continuity of berms and terraces along a channel. The continuity of mapped terrace and berm surfaces can later be confirmed by the use of HEC-2 water surface profiles (WET 1989a).

Field data will provide information for an empirical approach to determining the bank stability criteria in the channel. Therefore, the following types of information must be collected at eroding sites: type of failure—slab or circular arc, cantilever bank height, bank angle, types of materials involved in the failure, and presence of tension cracks in the upper bank materials. In many cases, the type of failure is masked by subsequent erosion of the failed material. The recent flood history of the channel should be known, at least in general terms, before the field work is conducted. The absence of recent flood events can provide a very misleading picture of channel and bank stability. Many banks may appear to be stable when dry, but these same banks will fail if saturated (Thorne, Biedenharn, and Combs 1988; Harvey 1984). Kudzu on many banks may provide an illusion of bank stability. Most bank failure takes place during flood recession, and as a result, the failed materials may be present in the bed of the channel if a significant flood flow has not occurred since the bank failed. Bank erosion can take place as a result of gullying of the upper bank. Concentrated flows that spill into the deeply incised channel from the floodplain cause the erosion.

The presence or absence of vegetation within an incised channel is often indicative of the degree of activity within the channel (Simon and Hupp 1987). Riparian vegetation requires a stable substrate to persist (Harvey and Watson 1988). The presence of an annual crop of seedlings should not be interpreted to mean that the channel is stable. The seedlings may be removed by the next flood flow; therefore, the size or estimate of the vegetation should be recorded in the field investigation. The spatial distribution of the in-channel vegetation will be recorded. Substantial in-channel vegetation may be associated with the presence of longitudinal berms that are indicators of lateral stability in the channel. Vegetation affects the hydraulic roughness. Photographs should be taken to document roughness changes. Elevations of continuous debris lines have been surveyed to provide data from which hydraulic roughness can be developed. These data are duplicated by crest gauges and recording stations at some locations. Instrumentation has been installed by WES and the USGS.

### Bank stability

Bank stability data collection and analysis are important components of the monitoring and analysis of incised streams projects. In the incised streams of the Yazoo Basin, bank instability generally is initiated by degradation of the channel.[1] Bank failure can be due to local effects such as those related to the concave bank of a meander bend, or can be a systemwide occurrence due to channel incision. Bank stability can be expressed in terms of the height of the bank, the bank angle, and the geotechnical properties of the materials that compose the bank.

Geotechnical borings and testing of bank stability are not within the scope of the present project; however, previous investigations by Thorne, Murphey, and Little (1981) and as a result of the ongoing structure design projects by the District provide data. A modified Culman analysis (Thorne, Murphey, and Little 1981; Little and Murphey 1982) can be used to determine the stability of the bank (Figure 31).

Long-term monitoring of bank stability is essential. Surveyed cross sections should be analyzed using Osman and Thorne (1988), and procedures to predict response will be investigated.

### Geomorphic analysis

The purpose of the geomorphic analyses is to relate the field observations and measurements to the survey and historical data. This allows the morphologic and dynamic aspects of the watershed system, including channel planform and cross-sectional dimensions, to be quantified and placed in a spatial context. If historical data are available, the channel morphology can also be placed into a temporal context as well.

The planform of the channel may reflect the reasons for the current erosional status. Many of the channels of the Yazoo Basin were channelized for flood control. Channelization of the streams lowered the base level.

---

[1] C. C. Watson, M. D. Harvey, S. A. Schumm, and D. I. Gregory. (1986). "Performance of Burney Branch and Muddy Creek channel stabilization measures," unpublished report, Department of Earth Resources, Colorado State University, Fort Collins.

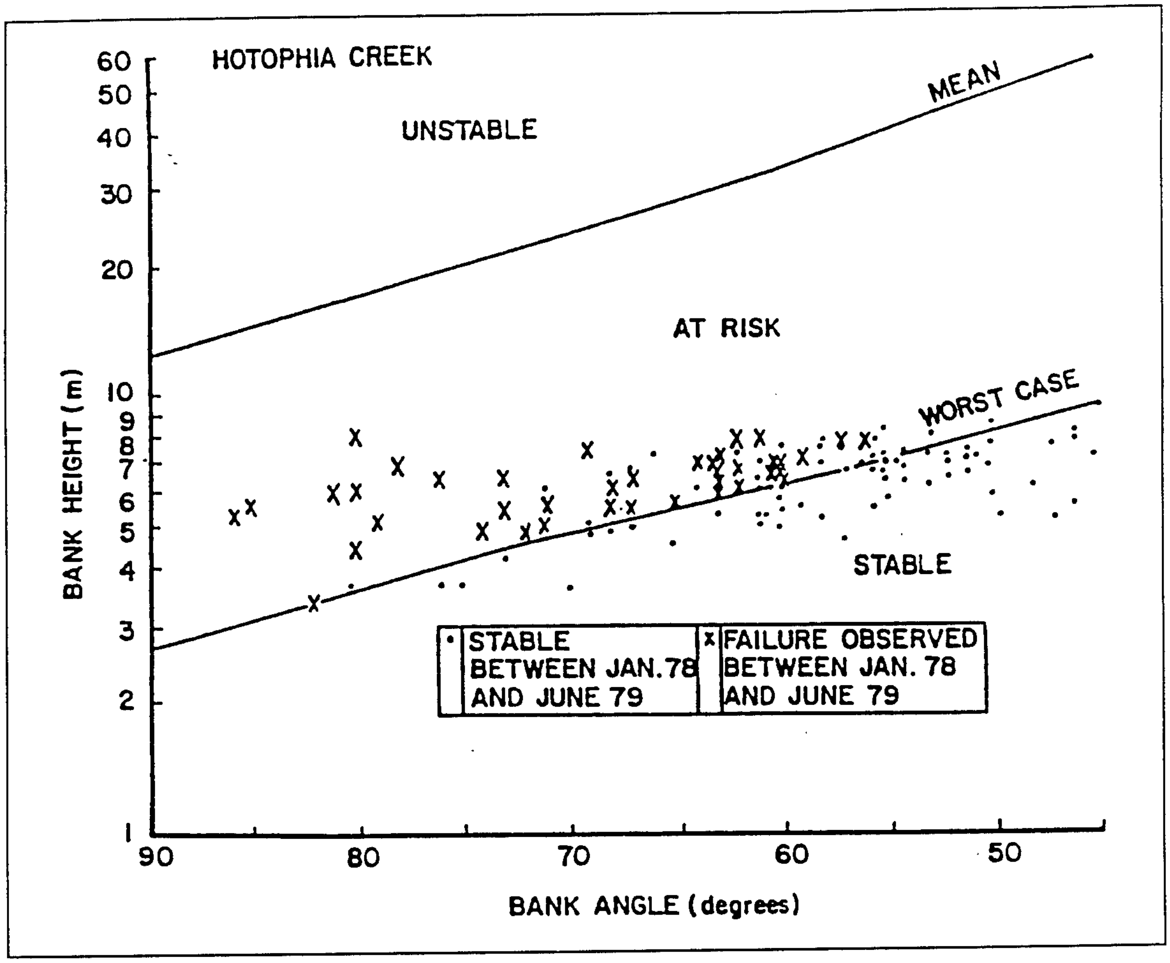



Figure 31. Bank stability graph for Hotophia Creek (Thorne, Murphey, and Little 1981)

Channelization may have involved straightening, enlarging the cross section of an existing channel, clearing and snagging, or combinations of these techniques. The channelization history of the streams plays an important role in interpreting the existing planform. Many of the sinuous tributaries are incised as a result of channelization and incision of the main channel. These tributaries are presently incised meandering channels. The first step in the geomorphic analysis is to determine channel planform.

The basic data for an analysis of the cross-section characteristics of the channel are being obtained from the surveyed cross sections. Once the cross sections have been coded into a HEC-2 format, the cross-section parameters at any discharge can be easily obtained. These include channel width, channel depth, cross-section area, and width-depth ratio. The morphometric parameters will be plotted against channel station, and comparison of historic data will be made.

The cross section, thalweg survey data, and field investigations are used to develop longitudinal profiles of the bed of the channel, the valley floor, top bank profile, and terraces and berms if these are present within the channel.

The construction of longitudinal profiles of the channel bed and the top bank profile provides useful information on the channel dynamics. The bed profile can be used primarily to identify oversteepened reaches of the channel. Oversteepened reaches of the channel bed can result from two situations. First, the oversteepening may be due to the presence of resistant materials in the bed, which may form knickpoints or knickzones. Field inspection and comparison of available profiles will determine the rate of degradation of these reaches. Second, the oversteepened reach may be due to aggradation (Harvey and Watson 1988). This aggradation indicates that sediment supply to the reach is greater than the sediment transport capacity of the reach, and the bed slope is aggrading to transport the supplied material. The excessive supply should be identified in the field and from aerial photos.

The presence of oversteepened reaches of the valley floor profile is generally indicative of historical sediment deposition on the floor of the valley (Happ, Rittenhouse, and Dobson 1940; Schumm, Harvey, and Watson 1984). Special attention should be given to these reaches during field and geotechnical investigations.

Historical data on channel bed slopes during the evolution of a given channel are rarely available; therefore, a comparison of terrace, valley floor, and berm profiles provides an indirect way of determining slope adjustment through time. Since the terraces represent former floodplain elevations, the terrace slopes provide an indirect measure of the sediment load and hydrologic regime of the channel at the time the terrace was the active floodplain of the channel. However, it should be recognized that the planform of the channel when the terrace was the active floodplain may have been different. For example, if the terrace slope is steeper than the current bed slope in a dynamic equilibrium reach of the channel, then it can be inferred that the sediment load of the channel in the past was higher than that of the present channel.

Laboratory analysis of the sediment samples collected during the field investigation provides the grain size distribution parameters $d_{84}$, $d_{50}$, and $d_{16}$ and the sorting coefficient $(d_{84}/d_{16})^{0.5}$. These grain size parameters for the bed material then can be plotted against station to determine the spatial distribution of the sediment sizes. The samples should be carefully analyzed to identify the full range of sediment size, due to the importance of cohesive materials and the existence of bimodal (sand-gravel) sediment distributions in the Yazoo Basin streams.

Geomorphic analysis of a channel involves the use of three types of data: (a) field observations and measurements, (b) survey data, and (c) historical data. It is important to use the data from the three sources to cross-check any results and conclusions drawn from any one of the sources. Data gathered at any point in time represent an instantaneous view of the system; however, every channel has a history, and the determination of that history is important in providing a context. Caution should always be used in the determination of rates of change if data points are widely spaced in time. Similarly, the flood history of the channel should be investigated if any confidence is to be placed in the rate of change information. The primary objective of the geomorphic analysis is to define the dynamics of the system, but this understanding can be achieved only by integrating all of the data and evidence.

### Hydrologic and hydraulic analyses

The USGS gauging stations located on the DEC monitoring sites are listed in Table 3. Many of these locations monitor larger watershed areas than the study site locations, and, in addition, several of the USGS gauged sites are of relatively short duration. The Corps of Engineers HEC-1 computer model can be used to define the magnitude of the various recurrence interval events; however, data for calibration have been limited. The USGS regionalized method (USGS 1991), which has been recently updated, has previously been used to develop Yazoo Basin project hydrology. Hydrology used for analysis of the DEC monitoring sites has been taken from Vicksburg District data, and was developed primarily by the District or contractors using HEC-1.

WES has committed to an aggressive schedule of gauge installation and stream gauging. Many of the gauges were installed during 1992. Until enough records can be developed for the sites using the WES gauges, flow duration data will be developed using USGS gauging station data. In FY 94 flow duration relationships for the sites will be prepared by transferring existing USGS data on the basis of a ratio of drainage area or other statistical techniques.

Hydraulic analyses are being conducted using the Corps of Engineers HEC-2 water surface profile model. The HEC-2 output is used to (a) define the hydraulic conditions and bank-full discharge; (b) define hydraulic parameters for sediment transport and related analyses; and (c) define water surface elevations for flows of given recurrence intervals. The channel cross sections used in the hydraulic investigation are those from the field survey. Proper use and calibration of the HEC-2 model are enhanced by the field observations.

Sediment transport modeling has three primary functions. First, the model is used to predict the locations of aggradation and degradation along the channel. Second, the model is used to determine the effective discharge or range of effective discharges for the channel. The effective discharge or range of discharges are those that transport the majority of sediment and, therefore, do most of the geomorphic work in the channel (Biedenharn, Little, and Thorne 1987; Watson and Harvey 1988). Third, the model is used to determine the sediment yield from the watershed. Ideally, the sediment yield should be divided into channel and nonchannel sources. In most of the Yazoo Basin watersheds, channel erosion produces the bulk of the sediment yield.[1] The

---

[1] C. C. Watson, M. D. Harvey, S. A. Schumm, and D. I. Gregory. (1986). "Performance of Burney Branch and Muddy Creek channel stabilization measures," unpublished report, Department of Earth Resources, Colorado State University, Fort Collins.

model also can be used to determine the reduction in sediment yield or the aggradation or degradation effects of any remedial measures.

In the incised channels of the Yazoo Basin, a fixed lateral boundary for the channel is a simplification because considerable channel widening takes place as a result of degradation. Bank failure provides considerable sediment inflow to the channel. This adjustment should be considered in developing future models for analysis, and modification of previous bank stability models will be made in FY 94 to attempt this adjustment.

SAM, a computer program developed by WES (Thomas et al., in preparation), has been used in the computation of sediment discharge, sediment yield, effective discharge, and a range of channel morphology that will satisfy water and sediment continuity for the sites. The sediment discharge for each segment was determined using the Brownlie sediment transport function. Sediment yield was computed only for the USGS gauging site on Hotophia Creek. The DEC monitoring sites were compared using the smaller of the 2-year or bank-full discharge, the Brownlie equation for sediment discharge, and a composite cross section representing the segment morphology. A composite cross section was developed for each segment using a SAM subroutine, which averages width, depth, velocity, and roughness, and computes energy gradient. The SAM stable channel computation for each segment was computed using the average bank angle for the segment, an estimated bank roughness, sediment size data, and the smaller of the 2-year or bank-full discharge. Comparisons were made between the composite cross section and the stable channel computations for each segment.

## Results

The results of the second year of monitoring, 1993, provide the first opportunity for comparison of two DEC monitoring data sets. This section will include a general overview of the 1993 data, and subsequent sections will provide results of each of the 22 monitoring sites and a sediment yield calculation for the mouth of Hotophia Creek.

### Abiaca Creek and Coila Creek

Four DEC monitoring sites, 3, 4, 6, and 21, are located on Abiaca Creek. Coila Creek Site 5 is confluent with Abiaca Creek at Site 4 and is included in this subsection. Descriptions of the four sites are given in the section, "Monitoring Sites," in this chapter.

### Abiaca Creek Site 3

Site 3 extends approximately 2,000 ft upstream and downstream of

Highway 17 in the upper portion of the basin with a drainage area of 26.5 square miles. Comparative thalweg profiles for Site 3 are shown in Figure 32. The surveyed thalweg slope for 1992 was 0.0009, and was surveyed as 0.0011 in 1993. Only minor differences in the thalweg profiles are shown except that the downstream 1,000 ft of the 1993 profile indicates approximately 2 ft of incision. Comparison of survey cross sections indicated an increase in channel volume, substantiating that some incision is occurring.

In general, this reach appears to be near quasi-equilibrium conditions. The bed of the stream is primarily fine sand ($D_{50}$ = 0.25 mm) with one minor gravel riffle existing in the upper portion of the site. Loose-bed sediment is generally 2 to 3 ft in depth. Some bank instability exists in the outside of the meander bends and at locations of fallen trees. Numerous trees apparently have been blown down in the reach, perhaps in 1991 prior to the monitoring surveys. At the upstream end of the monitoring site, a sharp bendway presently exists, which may cut off in the near future and could cause the several upstream beaver dams to fail. A cutoff would result in a steeper approach and may result in an increase of sediment supply.

The 2-year discharge of the reach is 3,339 cfs, and the bank-full capacity is approximately 1,700 cfs or 50 percent of the 2-year discharge. The composite cross section for the site was 55.9 ft in width, 6.5 ft in depth, with an energy slope of 0.00098 in 1992, and was 58.8 ft in width, 6.4 ft in depth, with an energy slope of 0.00094 in 1993. Regime dimensions for the bank-full discharge are 86.6 ft in width, 8.26 ft in depth, with a slope of 0.00019. Using the Brownlie sediment transport function, a bank roughness of $n$ = 0.035, and the average bank angle, the sediment transport for the bank-full discharge was 7,602 tons/day in 1992 and 6,043 tons/day in 1993, a 20 percent decrease. This equated to 136 and 103 tons/day/foot of width, respectively.

Figure 33 is a comparison of stable channel computations for the site. The two data points represent the 1992 and 1993 composite cross sections developed from HEC-2 runs of the bank-full discharge at a total channel roughness of 0.035. The 1992 stable channel curve indicates a minimum slope of 0.0006 at a width of 76 ft, while the 1993 curve has a minimum slope of 0.00057 at 66 ft. The primary change between the two curves is that the 1993 curve shifts from upper regime (smoother) to lower regime (rougher) at about 70 ft in width. Comparing the composite data points and the stable channel computation curves presents a noticeable difference. This difference is due to the difference in estimated roughness in the HEC-2 runs and the estimated bank roughness and computed bed roughness of the SAM runs. The data suggest that the width would increase approximately another 10 ft to attain the minimum energy slope if the sediment supply remains constant. Erosion resistance of the toe of the bank may resist widening, which will result in a steeper energy gradient than 0.0006.

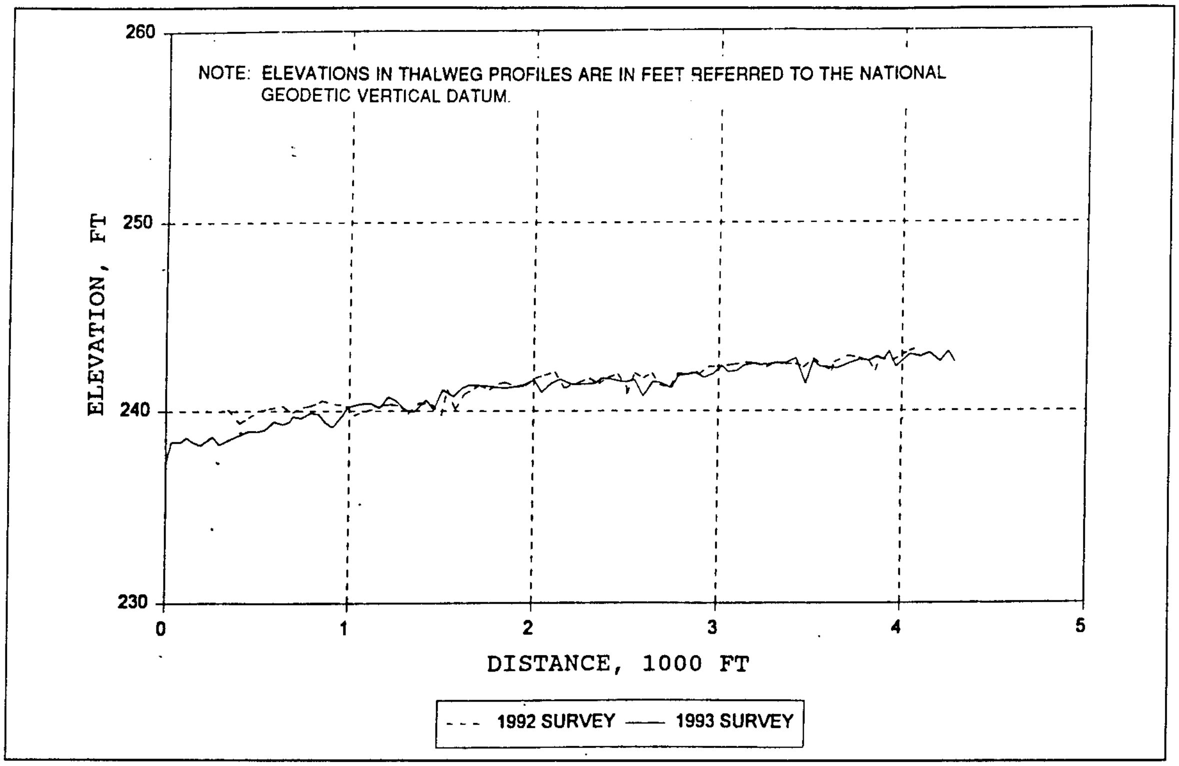



Figure 32. Comparison of 1992 and 1993 thalweg profiles for Abiaca Creek, Site 3

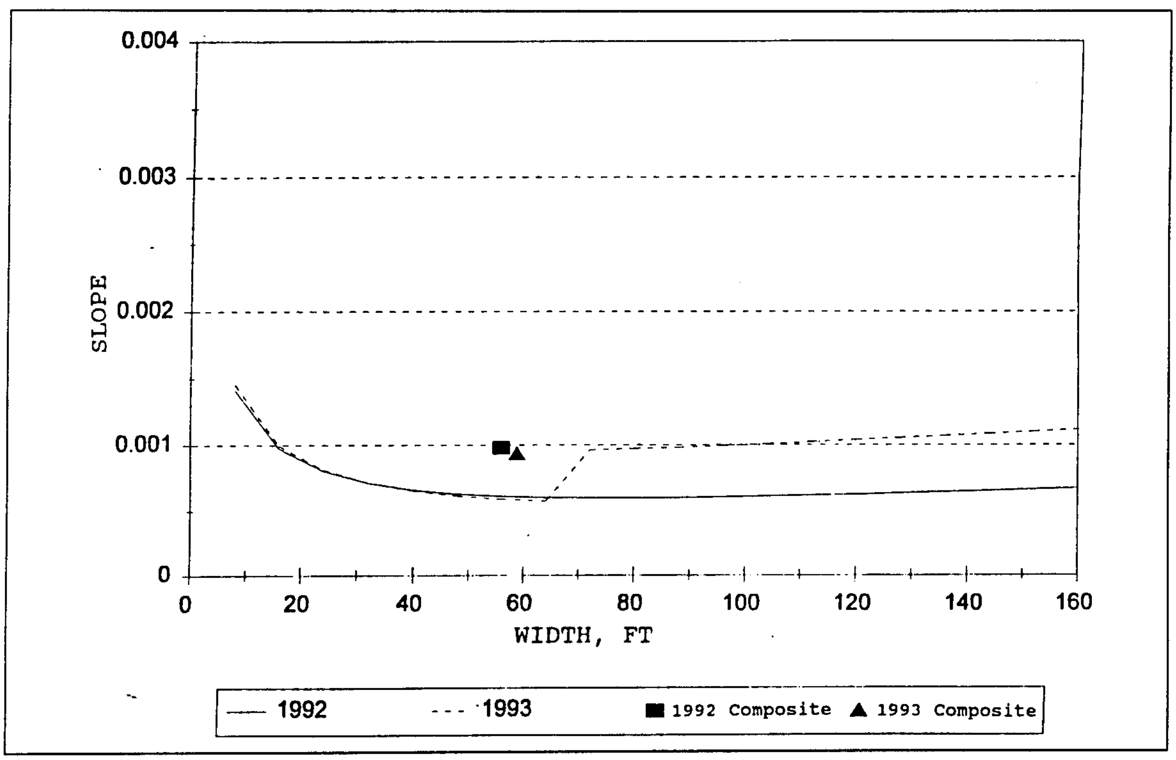



Figure 33. Comparison of 1992 and 1993 stable channel computations for Abiaca Creek, Site 3

## Abiaca Creek, Site 4

Site 4 extends from the confluence of Abiaca Creek and Coila Creek for approximately 4,000 ft. The upstream extent of the reach is approximately 500 ft upstream of a low timber bridge, which was apparently constructed by the local landowner or gravel miner. Depending on the amount of debris accumulation, some backwater and aggradation can exist at the bridge. Comparative thalweg profiles for Site 4 are shown in Figure 34. The surveyed thalweg slope for 1992 was 0.0017 and was 0.0016 in 1993. The 1992 thalweg indicates a break in slope approximately 3,200 ft upstream of the downstream limit, which may have been caused by debris accumulating on the low timber bridge. The 1993 thalweg indicates no break and has 2-3 ft of aggradation from 1,000 ft to 3,200 ft upstream of the beginning of the site.

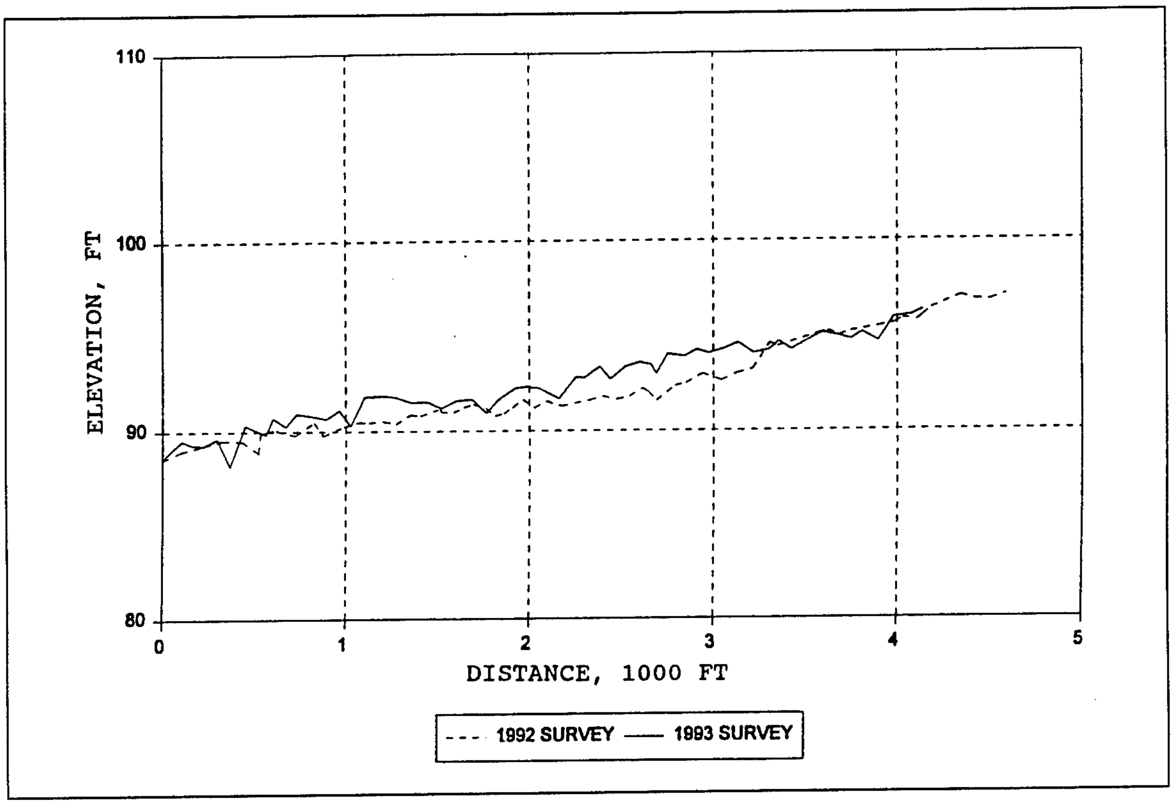



Figure 34. Comparison of 1992 and 1993 thalweg profiles for Abiaca Creek, Site 4

The morphology of Site 4 is characterized by a meandering, sand and gravel bed stream with generally low bank heights. Mature birch trees provide significant shade along the banks and indicate relative planform stability along the reach. Evidence exists of recent gravel mining of a large point bar near the timber bridge. No significant trees are found on the point bar. The bed material is characterized as medium sand ($D_{50}$ = 0.49 mm), and loose bed material is at least 3 ft in depth. A significant amount of gravel is in the bed material and in the alternate bars. Approximately 1,500 ft upstream of the confluence, the right bank is formed by a high bluff in excess of 50 ft in height while the left bank is low with evidence of a chute cutoff that is active during higher flows. Erosion of the high-bluff banks of Abiaca Creek is a major natural source of sediment, and this site is typical of sites that could be protected to reduce local sediment sources.

The 2-year discharge of the reach is 3,780 cfs, and the bank-full capacity is approximately 900 cfs, less than 25 percent of the 2-year discharge. The dimensions of the composite cross section for the site were 70 ft in width, 3.2 ft in depth, with an energy slope of 0.0019 in 1992, and 74 ft in width, 3.4 ft in depth, with an energy slope of 0.0014 in 1993. Regime dimensions for the bank-full discharge are 63 ft in width, 5.3 ft in depth, with a slope of 0.00033, narrower, deeper, and flatter than observed. Using the Brownlie sediment transport function, a bank roughness of $n = 0.035$, and the average bank angle, the sediment transport for the bank-full discharge was 3,221 tons/day in 1992 and 1,925 tons/day in 1993, a 40 percent decrease. This equated to 46 and 26 tons/day/foot of width, respectively. Therefore, Site 4 sediment transport capacity at bank-full is approximately 30 percent of the upstream Site 3 capacity. This is due primarily to the low bank-full discharge at Site 4 and large sediment size.

Figure 35 is a comparison of stable channel computations for the site. The two data points are representative of the 1992 and 1993 composite cross sections developed from HEC-2 runs of bank-full discharge at a total channel roughness of 0.035. The 1992 stable channel curve indicates a minimum slope of 0.0015 at a width of only 23 ft, due primarily to the shift in roughness regime that is shown. The 1993 stable channel curve remains in upper regime throughout the full range in width, with the minimum slope occurring at 0.0014, which is almost the same as the 1992 slope, at a width of 51 ft. The data shown in Figure 35 indicate that the reach could be stable at less width and greater depth, which could result in a more desirable habitat for fish.

### Coila Creek, Site 5

Coila Creek Site 5 extends upstream from the confluence with Abiaca Creek for a distance of approximately 4,000 ft. The study reach extends approximately 2,000 ft upstream of highway bridge 430, as shown in Figure 13. The approximately 42.1-square-mile watershed upstream of the reach had been heavily mined previously, and a high proportion of the present watershed is controlled by SCS reservoirs. Figure 36 is a comparison of the 1992 and 1993 thalweg profiles for Coila Creek. The surveyed thalweg slope for 1992 was 0.0018 and was 0.0019 in 1993. Only minor change is evident in comparison of thalwegs, with minor degradation along the channel from 1992 to 1993.

The lower 2,000 ft of Site 5 is characterized by a sand and gravel meandering channel with relatively high point bars, primarily of sand. The

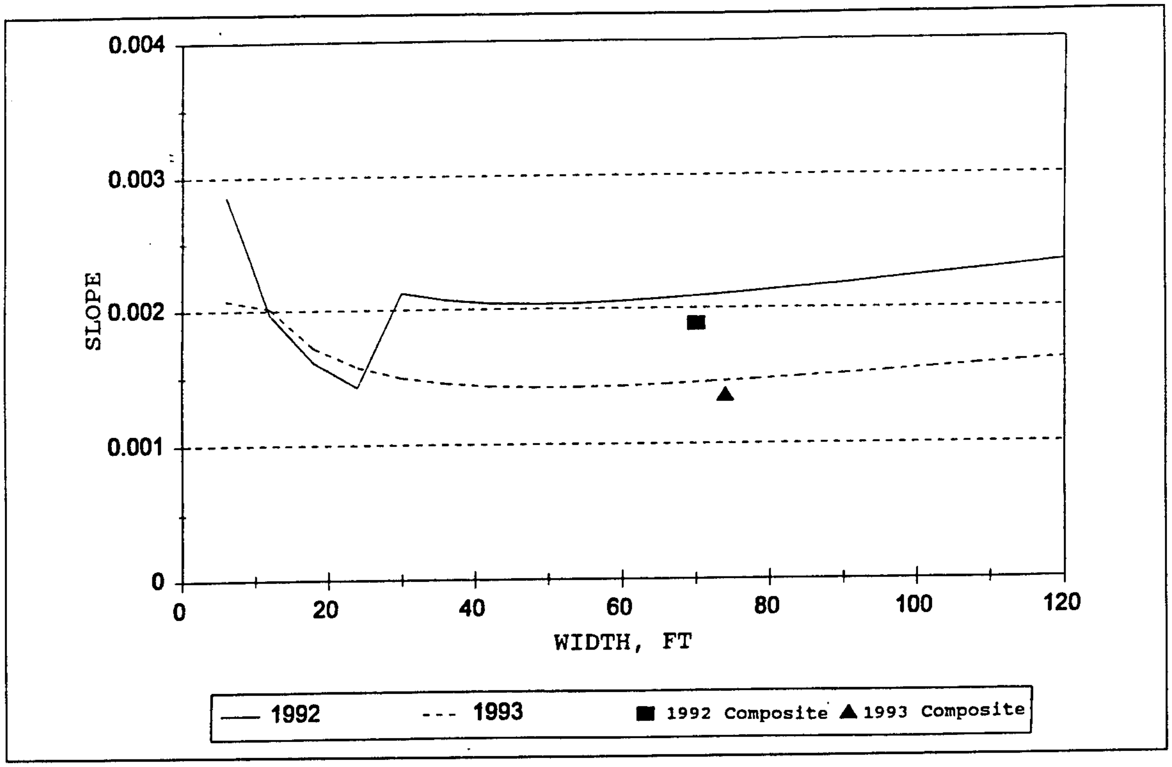



Figure 35. Comparison of 1992 and 1993 stable channel computations for Abiaca Creek, Site 4

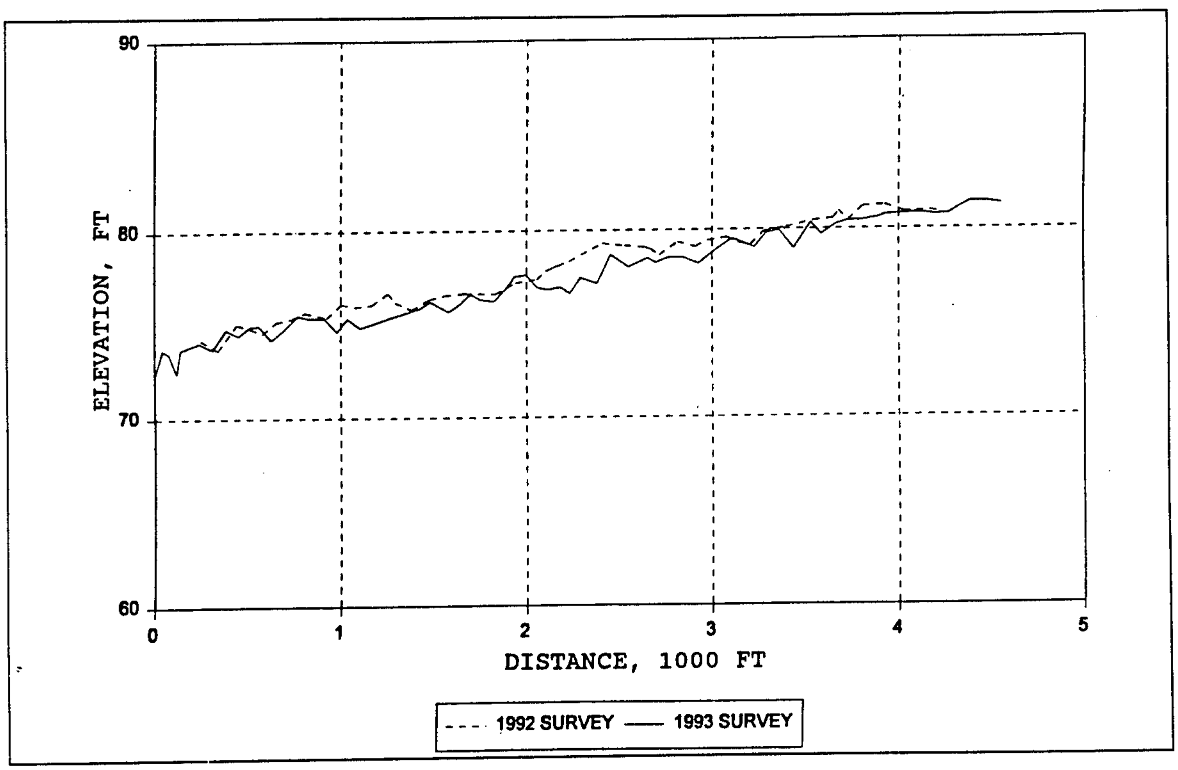



Figure 36. Comparison of 1992 and 1993 thalweg profiles for Coila Creek, Site 5

left bank is high and is adjacent to the high bluff described in the previous section on Abiaca Creek, Site 4. The right bank is lower, and evidence suggests that multiple channels are present at discharges greater than bank-full. Vegetation is moderate to heavy with birch and willow trees. Heavy grass growth is present in some areas. Upstream of the highway bridge, the banks are much lower and are predominantly grass. The low, grassy banks are frequently undercut. Several low, heavily grassed islands were noted, which appear to have been formed by chute cutoffs. The bed of the stream is medium gravel, $D_{50}$ = 10 mm. The combination of upstream watershed control, low bank height, and a relatively coarse bed sediment gives the appearance of relative stability in the portion of the reach upstream of the bridge. Near the confluence, the banks are actively eroding and survey temporary benchmarks (TBM's) were lost due to erosion from 1992 to 1993.

The 2-year discharge of the reach is 4,780 cfs, and the bank-full capacity is approximately 730 cfs, about 15 percent of the 2-year discharge. The dimensions of the composite cross section for the site were 57 ft in width, 3.2 ft in depth, with an energy slope of 0.0018 in 1992, and 65.2 ft in width, 3.0 ft in depth, with an energy slope of 0.0017 in 1993. Regime dimensions for the bank-full discharge are 56.7 ft in width, 4.8 ft in depth, with a slope of 0.0007. Therefore, the 1993 composite channel is wider, shallower, and steeper than the regime prediction. Using the Brownlie sediment transport function, a bank roughness of n = 0.035, and the average bank angle, the sediment transport for the bank-full discharge was 56 tons/day in 1992 and 34 tons/day in 1993, a 39 percent decrease. This significant change in transport capacity is the result of using the same discharge and doubling the composite channel width. However, at any reasonable estimate of bank-full discharge, the sediment transport capacity would be low.

Figure 37 is a comparison of the 1992 and 1993 stable channel computations for Coila Creek. The two data points represent the 1992 and 1993 composite cross sections developed from HEC-2 runs of bank-full discharge at a total channel roughness of 0.035. The 1992 stable channel curve indicates a minimum slope of 0.0021 at a width of only 23 ft. The 1993 stable channel curve indicates a similar width of 29 ft at a minimum slope of 0.0013. The predicted channel widths are three to five times less than observed widths. The variance between predicted and observed widths may be due to the lack of calibration of the Brownlie relationships for roughness and sediment transport in the gravel-size ranges of bed sediment.

### Abiaca Creek, Site 6

Abiaca Creek Site 6 extends approximately 4,500 ft from the hill line into the flatter Yazoo River floodplain. The USGS gauging station at the bluff line is approximately 700 ft downstream of the upstream limit. The drainage area is approximately 99.1 square miles. Figure 38 is a comparison of the 1992 and 1993 thalwegs, which indicates little change between 1992 and 1993. The surveyed slope was 0.0009 in 1992 and was 0.0008 in 1993.

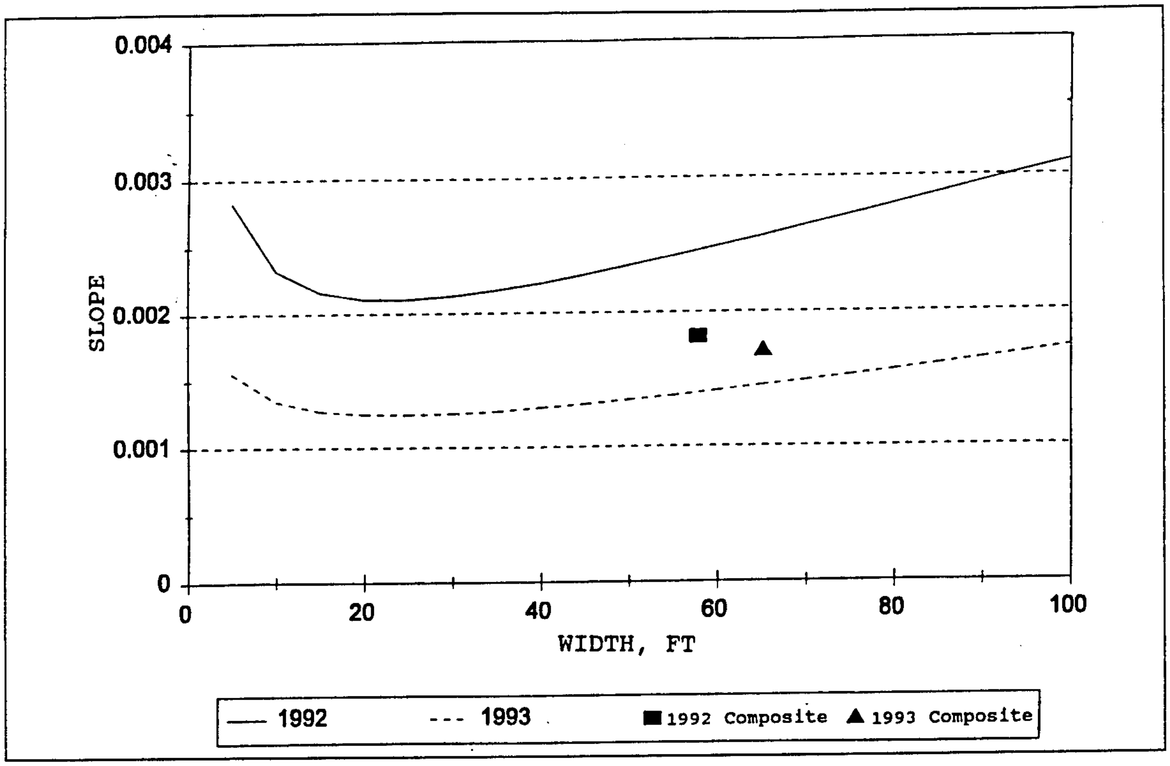



Figure 37. Comparison of 1992 and 1993 stable channel computations for Coila Creek, Site 5

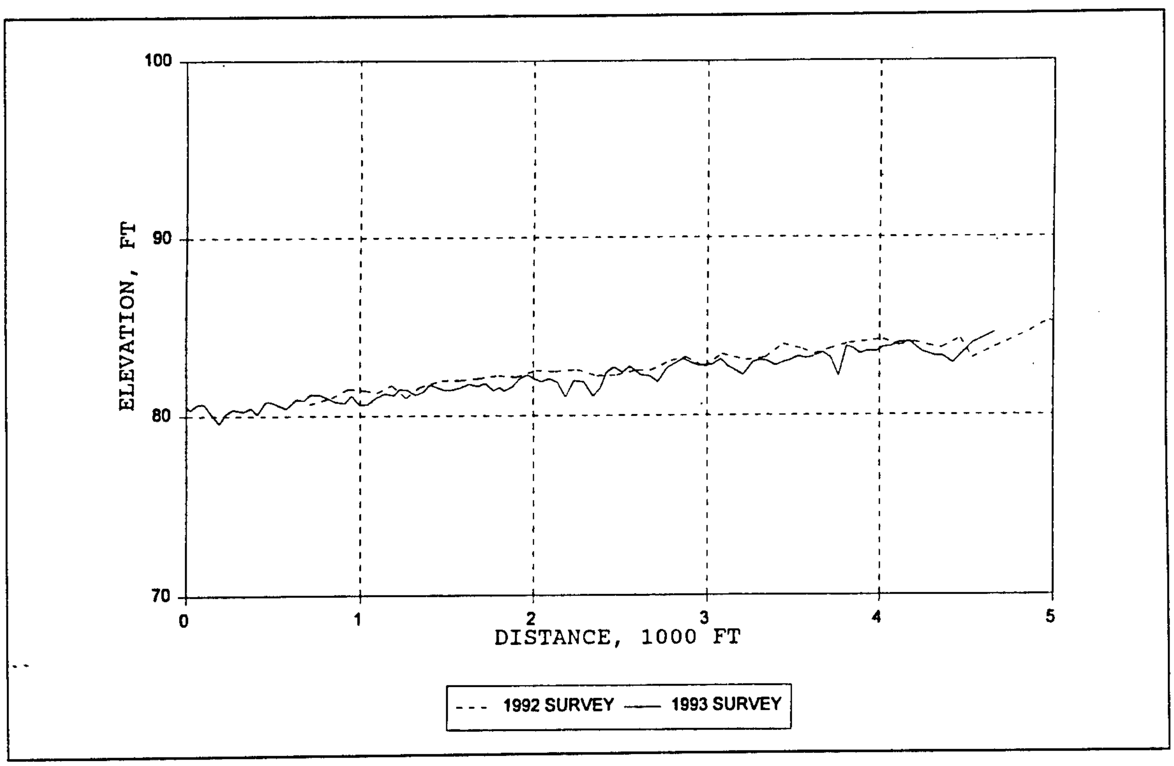



Figure 38. Comparison of 1992 and 1993 thalweg profiles for Abiaca Creek, Site 6

With the exception of the upstream extent of the reach, the channel morphology is that of a straight, excavated channel. The upstream extent impinges on a high bank in a natural bendway, and that bend is the only significant bank failure along the reach. Typically, the banks are heavily vegetated with vines and trees. Some agricultural debris was noted along the channel. The excavated channel has some minor changes in direction, and does have significant alternate bars within the channel. Bed sediments are characterized as medium sand ($D_{50}$ = 0.37 mm), and the bed comprises loose sediment to a depth of at least 3 ft. No significant bank instability was noted downstream of the bluff line bridge.

The 2-year discharge of the reach is 7,095 cfs, and the bank-full capacity is approximately 2,020 cfs, about 28 percent of the 2-year discharge. The dimensions of the composite cross section for the site were 104.0 ft in width, 5.7 ft in depth, with an energy slope of 0.00062 in 1993. Cross-section survey data were not available from 1992. Regime dimensions for the bank-full discharge are 94.4 ft in width, 6.9 ft in depth, with a slope of 0.00029. Therefore, the 1993 composite channel is wider, shallower, and steeper than the regime prediction. Using the Brownlie sediment transport function, a bank roughness of $n = 0.035$, and the average bank angle, the sediment transport for the bank-full discharge was 1,831 tons/day in 1993, or about 18 tons/day/foot of width.

Figure 39 shows close agreement between the 1993 composite cross section computed using HEC-2 runs of the bank-full discharge and the stable channel computations. The minimum slope was computed at 0.00069 as compared with the composite slope of 0.00062. The computed width at the minimum slope was 69 ft compared with the composite 104 ft, only 66 percent of the composite; however, the stable channel curve is very flat for the reach. Very little adjustment of the site is expected without significant change in the sediment supply.

## Abiaca Creek, Site 21

Abiaca Creek Site 21 is downstream of Site 5, and is at the downstream extent of the channelized reach of Abiaca Creek at the Highway 49 and railroad bridges. This site was added in time for the 1993 surveys; no data are available for 1992. Figure 40, the thalweg profile for 1993, indicates the depth increases in the downstream portion of the reach, in the vicinity of the highway and railroad bridges. The regime width is 159 ft, the depth is 9.9 ft, and the slope is 0.00024. Time composite 1993 dimensions are 77 percent of regime width (122 ft), 91 percent of regime depth (9.0 ft), and 213 percent of regime slope (0.00051). Compared with other sites, Abiaca Site 21 is

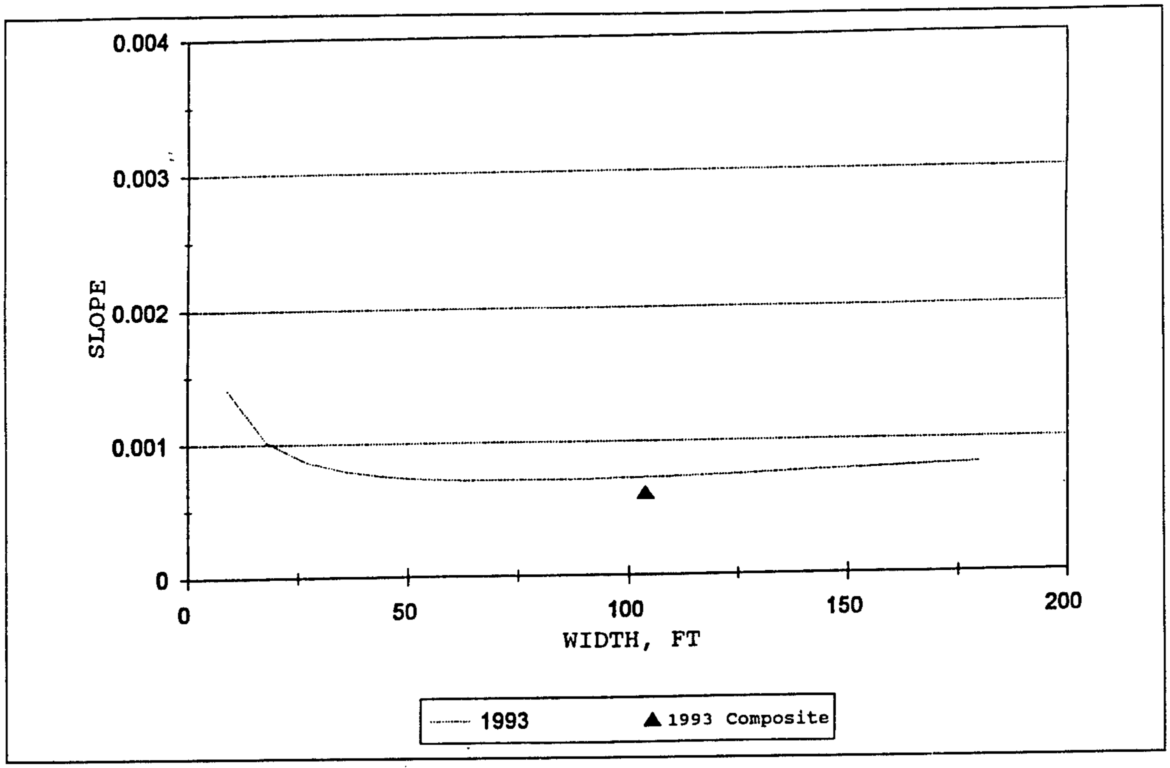



Figure 39. Stable channel computation for Abiaca Creek, Site 6, 1993

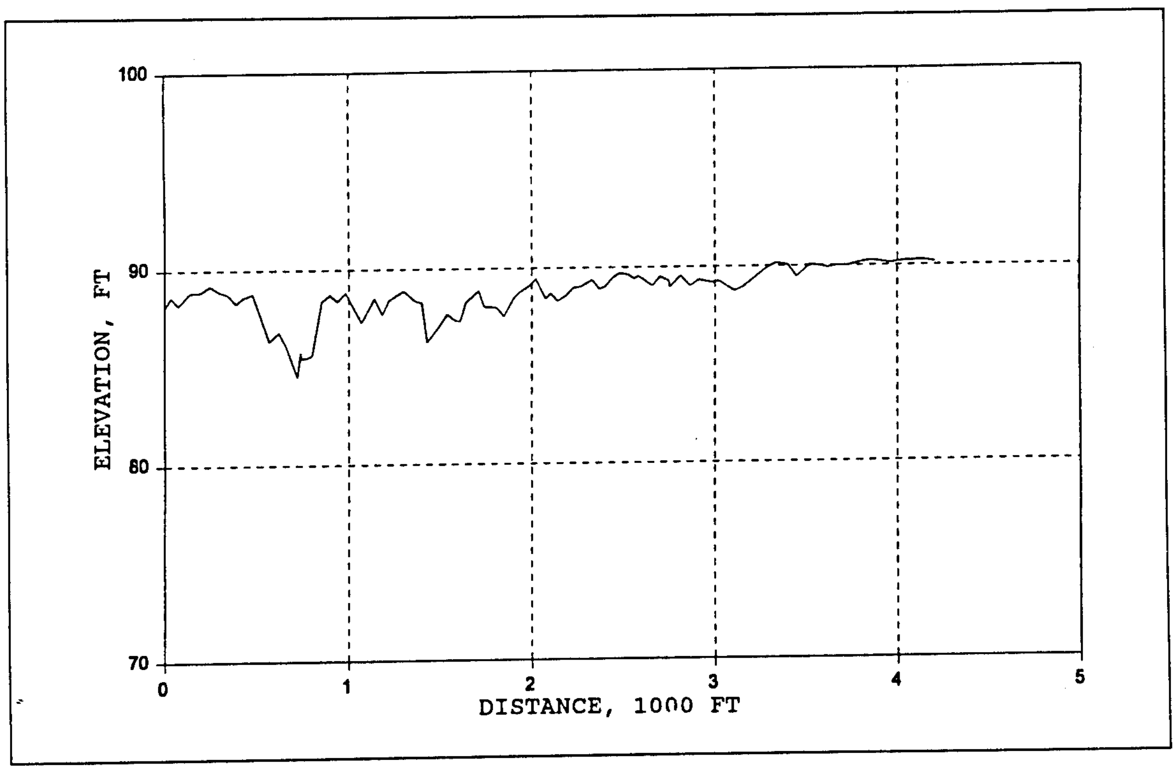



Figure 40. Thalweg profile of Abiaca Creek, Site 21, for 1993 data

relatively near regime condition. Figure 41 is a comparison of computed stable channel dimensions and the 1993 composite condition. The 2-year sediment discharge is 6,414 tons/day, or 53 tons/ft/day. With the relatively low unit width sediment discharge, the close agreement of the computed stable channel curve and composite conditions is significant. The site appears to be stable for the eroding condition sediment supply at the 2-year discharge.

### Burney Branch

Burney Branch is a tributary of the Yocona River in Lafayette County, near Oxford, MS. Extreme floods in 1973 accelerated channel incision, which endangered a box culvert beneath Highway 7 and the sewage treatment facilities of the city of Oxford. The SCS constructed six Type C high-drop structures to remove 59.4 ft of gradient to establish a channel gradient of 0.0008. Watson et al. reported that the bed slopes between the first two structures was approximately 0.0012.[1]

The DEC monitoring site 12 is located on Burney Branch and extends downstream from the Highway 7 box culvert through the first high-drop structure to the second structure, a distance of approximately 5,700 ft. Figure 42 is a comparison of the thalweg profiles surveyed in 1992 and 1993. The site has been divided into two segments. Segment 1 is downstream between the two drop structures, and segment two is between the upstream drop structure and the box culvert. As shown in Figure 42, no significant change occurred in the thalweg profile.

The lower half of segment 1, immediately upstream of drop structure 2, is an excellent example of channel evolution accelerated by drop structure emplacement. The deep, wide incised channel that existed prior to construction of the drop structure has been filled, and a new inner channel has resulted with berms forming along the toe of the banks of the incised channel. Willow, sycamore, and birch trees are colonizing the deposited berms. Upstream of this reach, a nearby hospital manicures the sloped banks, even through a reach of Kelner jacks along the right bank. Upstream of the hospital property the banks have become heavily vegetated. Upstream of the first drop structure, much of the channel is riprapped or heavily vegetated. The streambed is medium sand ($D_{50}$ = 0.36 mm). Loose sand exists to a depth of at least 3 ft.

Table 4 lists the results of data analyses and computations for the Burney Branch site.

The data shown in Table 4 indicate that the composite section width and depth are similar to regime dimensions, and that the energy gradient of the

---

[1] C. C. Watson, M. D. Harvey, S. A. Schumm, and D. I. Gregory. (1986). "Performance of Burney Branch and Muddy Creek channel stabilization measures," unpublished report, Department of Earth Resources, Colorado State University, Fort Collins.

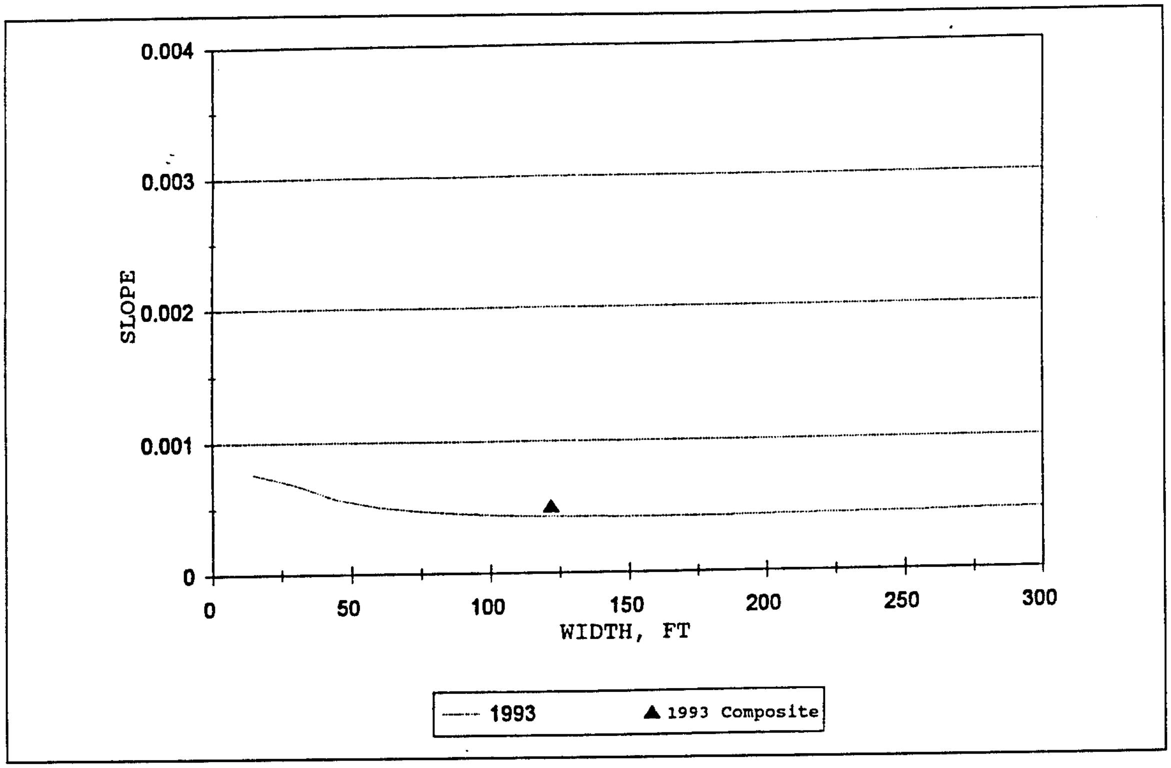



Figure 41. Comparison of computed stable channel computations for Abiaca Creek, Site 21

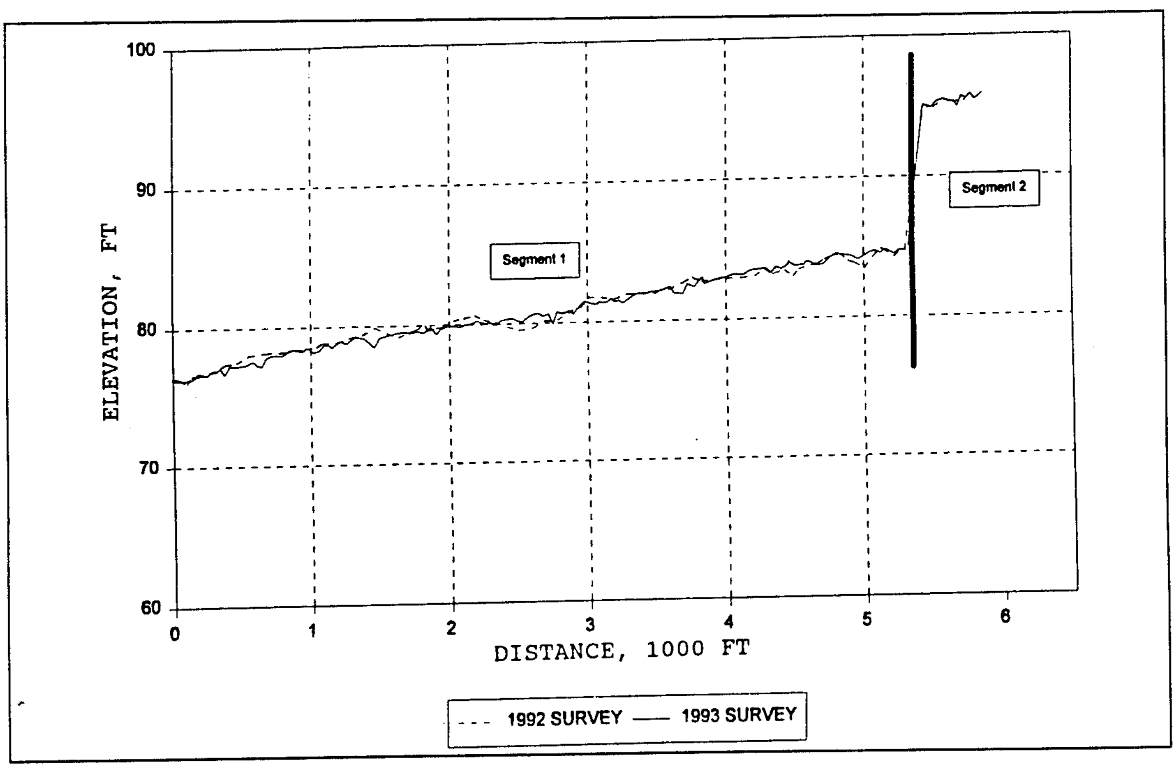



Figure 42. Comparison of 1992 and 1993 thalweg profiles for Burney Branch, Site 12

composite section is two to three times the predicted regime relationship. In addition, the bed slope is three to four times the energy gradient. Comparison of HEC-2 backwater runs for this site confirms that the energy gradient decreases as the discharge increases. One explanation for the difference between the bed slope and the 2-year energy gradient is that the bed slope is being formed by a discharge less than the 2-year discharge. This discharge is the effective discharge. This would also imply that most of the sediment is being transported by a discharge less than the 2-year discharge. Additional investigation of these data will be made in 1994.

Figure 43 compares the stable channel computations for segment 1, and Figure 44 compares the stable channel computations for segment 2. These data indicate close agreement for the composite section width and slope with the minimum slope and width from the stable channel computations.

## Harland Creek, Site 1

Site 1 extends approximately 2,000 ft upstream and downstream of the county road bridge near Eulogy, MS. The downstream portion of the site was riprapped following the 1992 surveys and prior to the 1993 surveys. The surveyed channel thalweg slope is the same for 1992 and 1993, as shown in Figure 45. No significant difference is apparent following the riprap emplacement; however, response may require several years.

This site is a meandering sand and gravel mixed-bed stream. The bed sediment is characterized as medium sand ($D_{50}$ = 0.50 mm). However, the upstream portion of the point bars is gravel, and the remainder of the bars are sand and gravel mixed. The depth of loose sediment in the reach is in excess of 3 ft. Prior to the riprap treatment, the outside of the bendway was eroding. Debris along the new riprap indicates that the stone has been overtopped and deposition of the sediment and accumulation of sediment from failing banks have filled the void between the stone and the bank. Bank erosion is continuing upstream of the bridge where no stone was placed, and a small tributary near the downstream extent of the reach indicates a significant sediment load is moving into Harland Creek.

The 2-year discharge for the reach is 3,739 cfs, and the bank-full discharge was found to be 1,970 cfs, about 53 percent of the 2-year. Table 5 summarizes the analyses developed for the reach. Although the bed slope of the reach has not changed, the 1993 channel composite section was wider, slightly deeper, and about 20 percent flatter than the 1992 composite section. The 1993 section is approaching regime width and depth. The lower portion of the reach may respond to the bank stabilization by becoming deeper and narrower; however, with continued sediment supply from upstream and tributaries, the response may be masked. Computations using the Brownlie sediment transport function indicate that the bank-full sediment transport capacity decreased approximately 30 percent between the surveys. Figure 46 compares the stable channel computation results and the composite cross-section data.

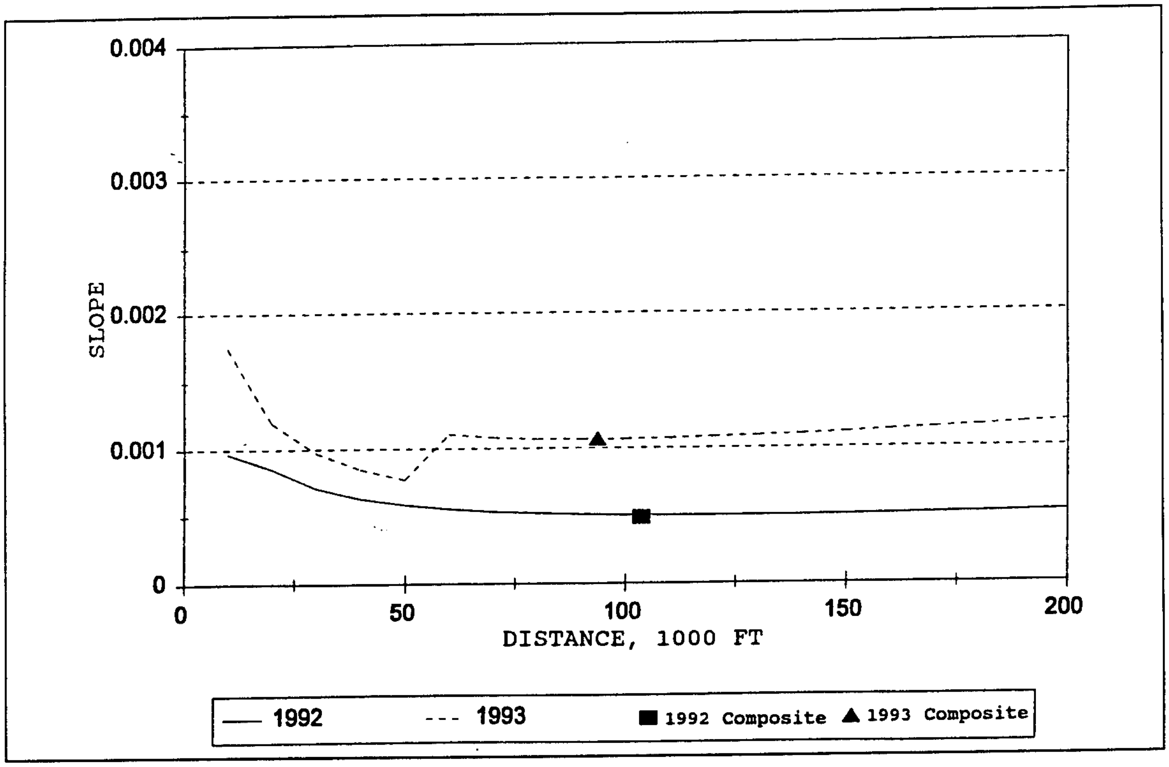



Figure 43. Comparison of 1992 and 1993 stable channel computations for Burney Branch, Site 12, segment 1

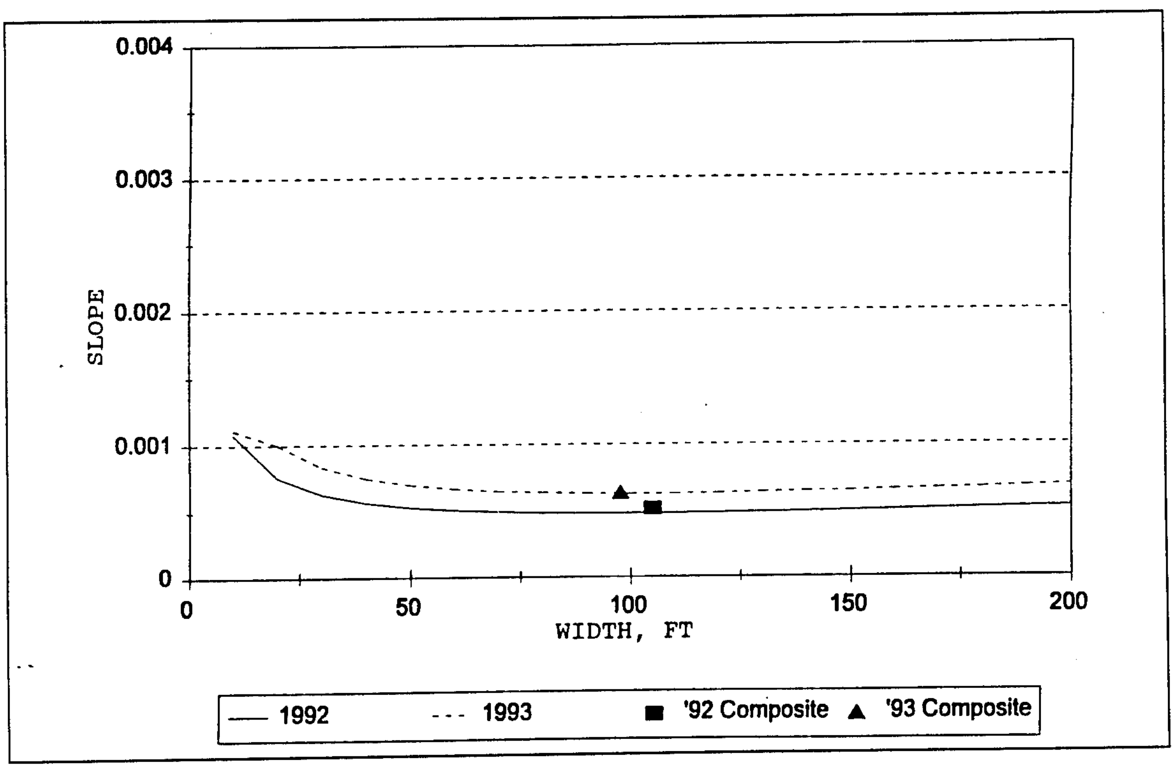



Figure 44. Comparison of 1992 and 1993 stable channel computations for Burney Branch, Site 12, segment 2

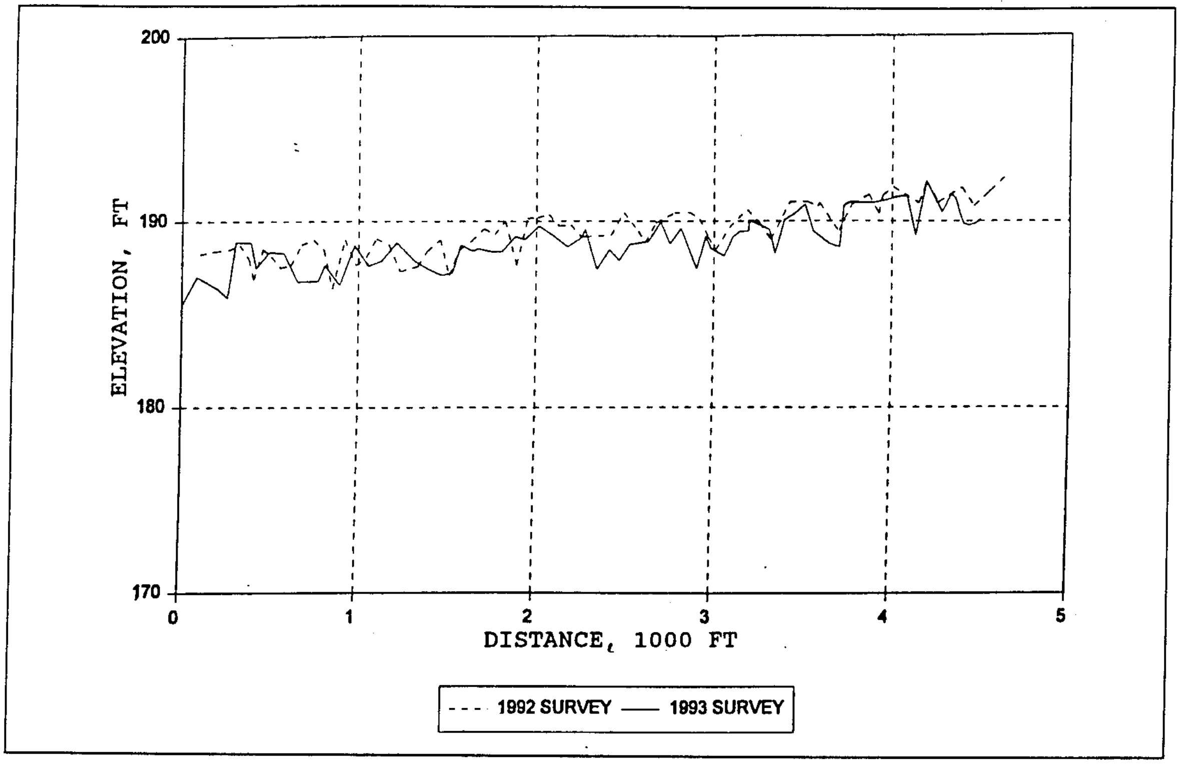



Figure 45. Comparison of 1992 and 1993 thalweg profiles for Harland Creek, Site 1

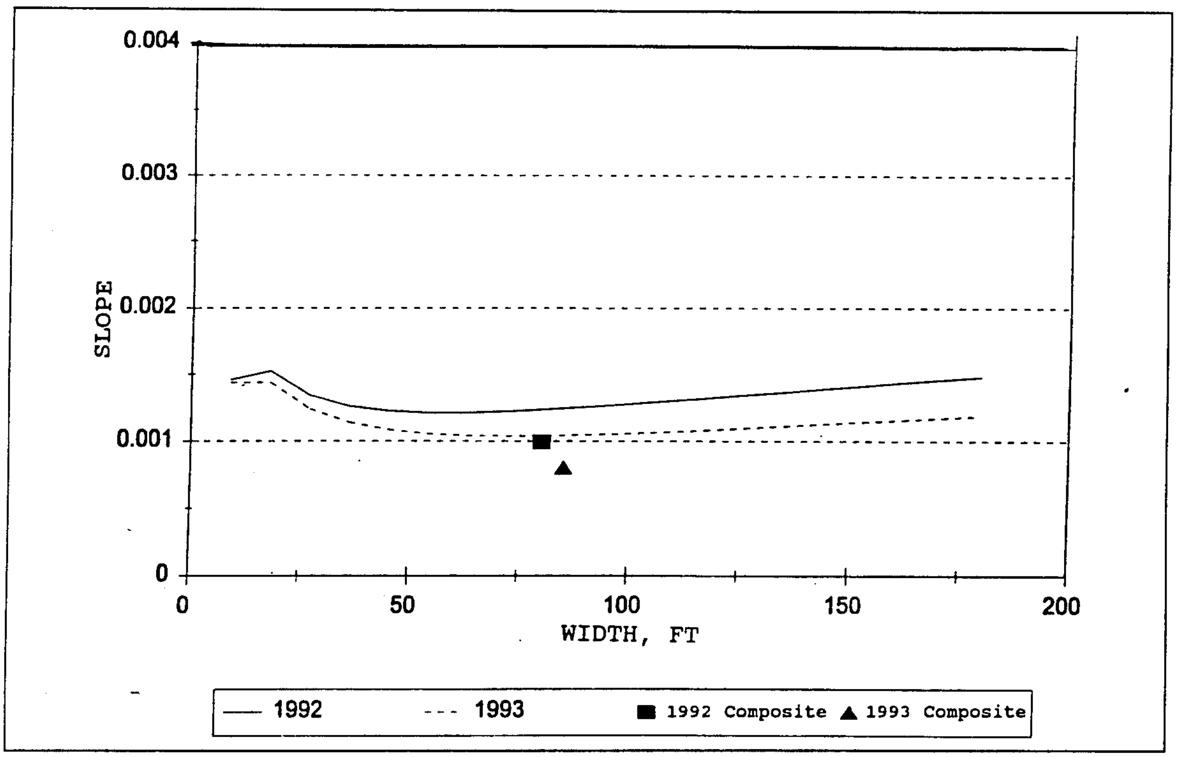



Figure 46. Comparison of SAM stable channel computations and the 1992 and 1993 composite cross section for Harland Creek, Site 1

The relationship of the two data sources indicates that the reach is mildly aggradational; however, the hydraulic roughness used in the computations could not be calibrated. The composite width is greater than the computed stable channel width.

### Hickahala Creek, Site 11

Hickahala Creek, Site 11, extends for approximately 4,000 ft, encompassing two low-drop structures, the lower structure being completed in 1992. The 2-year discharge is 2,158 cfs, and the bank-full discharge is 1,290 cfs, about 60 percent of the 2-year. Three reaches are defined by the two structures as shown in Figure 47, a comparison of 1992 and 1993 thalweg profiles. In addition to the two structures shown, a third structure was recently completed on South Fork, Hickahala Creek, which is a tributary to the study reach in segment 2. Segment 3 has been relatively stable with some bank erosion at the outside of the bank in the approach to the upstream drop structure. Portions of segment 1 and segment 2 exhibit incision and bank failure and a wide variance in channel depth. Segment 2 has not yet completely responded to the structure construction, both on the main channel and the tributary.

The segment 3 survey was extended for approximately 1,500 ft below the 1992 downstream extent. This portion of the study reach is steep and the channel is formed in a resistant fine-grained material. As shown in Table 6, extension of the survey distorts the segment 3 data for the composite slope, which would have been steeper in 1992 if the same length of reach had been surveyed. The segment 2 bed slope decreased and composite section energy slope increased, along with a narrowing of the composite section. Increase in the segment 2 composite slope resulted in a significant increase in the sediment transport capacity; however, the segment should respond to decreased transport capacity next year. Segment 1 bed slope and energy gradient are approximately the same for 1993, and channel width and depth increased over 1992 dimensions. Sediment transport capacity also increased from 1992 to 1993.

Figures 48, 49, and 50 are comparisons of 1992 and 1993 stable channel computations and composite sections for segments 1, 2, and 3, respectively. Although the composite channel width in segment 1 is much less than the regime width, good agreement exists for the 1993 stable channel computation and the composite section. Figure 49 indicates that segment 2 is predicted to be degradational for the 1993 data, and was observed to be unstable. By comparison, the 1993 data for segment 3 is much more unstable. The channel is responding dramatically to preconstruction incision and to the structure construction. Particular attention will be paid to the segment 3 response to ascertain re-incision tendency.

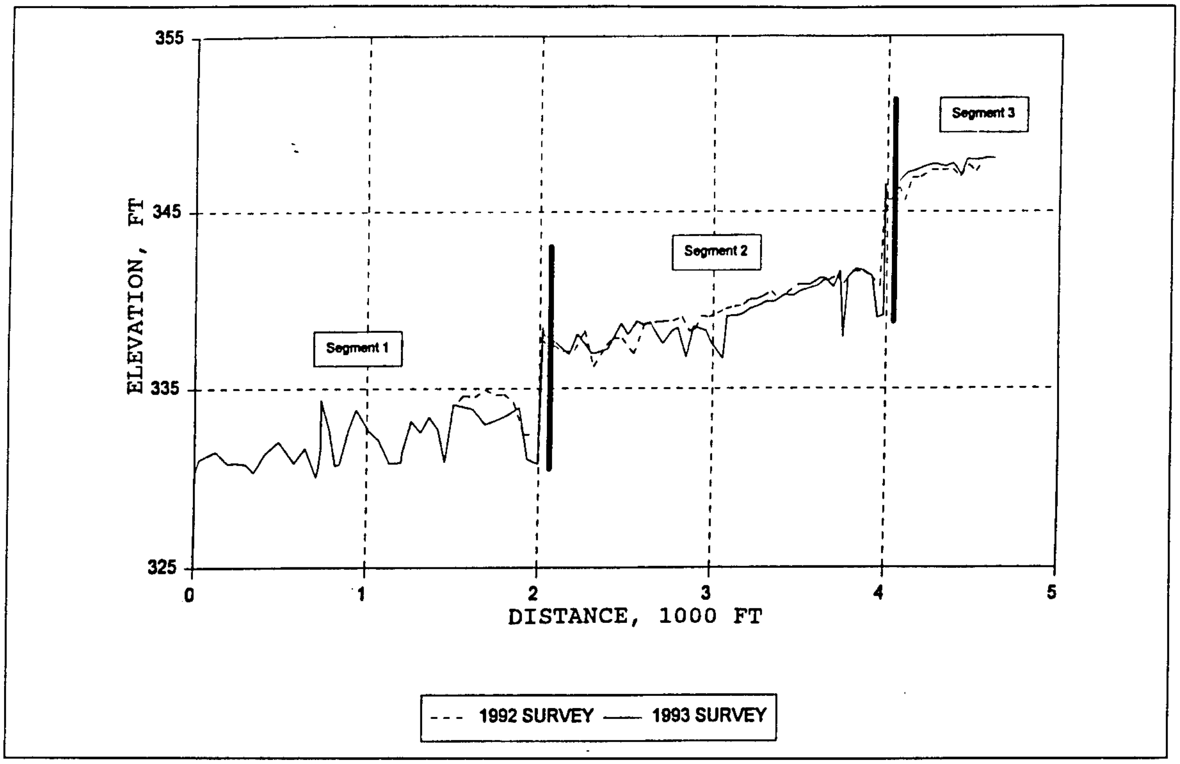



Figure 47. Comparison of 1992 and 1993 thalweg profiles for Hickahala Creek, Site 11

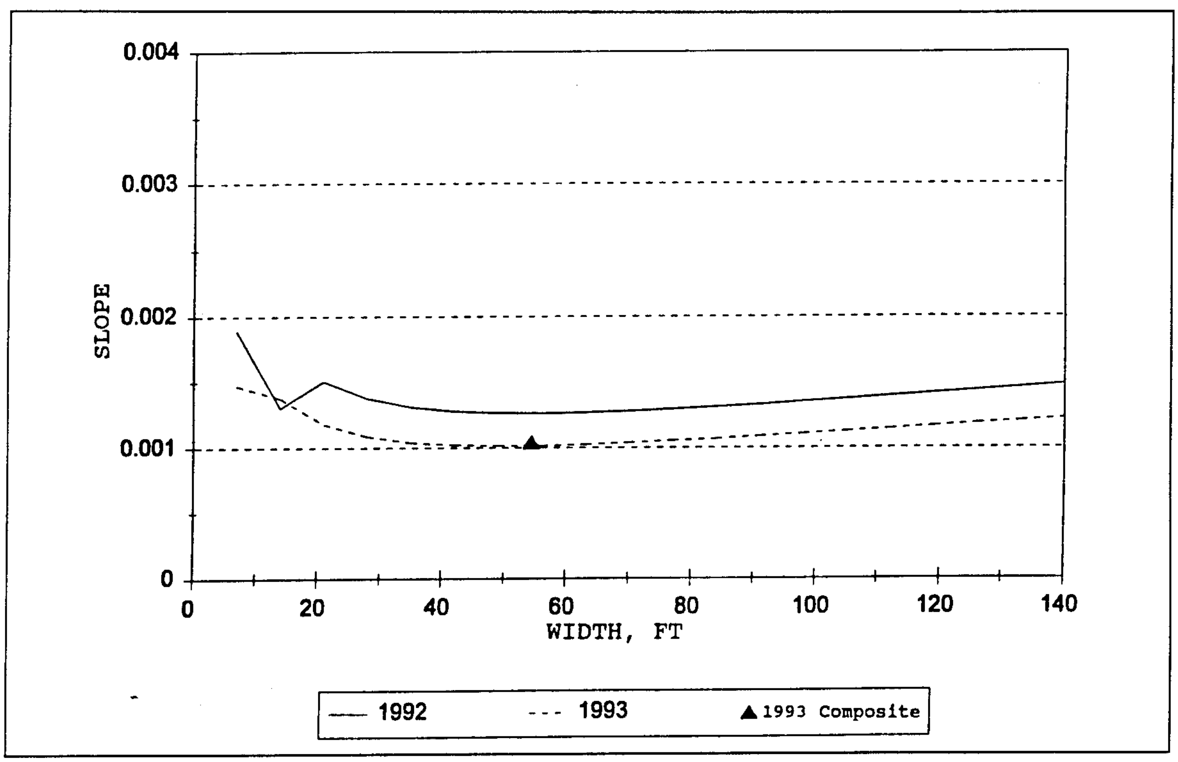



Figure 48. Hickahala Creek (Site 11), segment 1, stable channel and composite section comparison

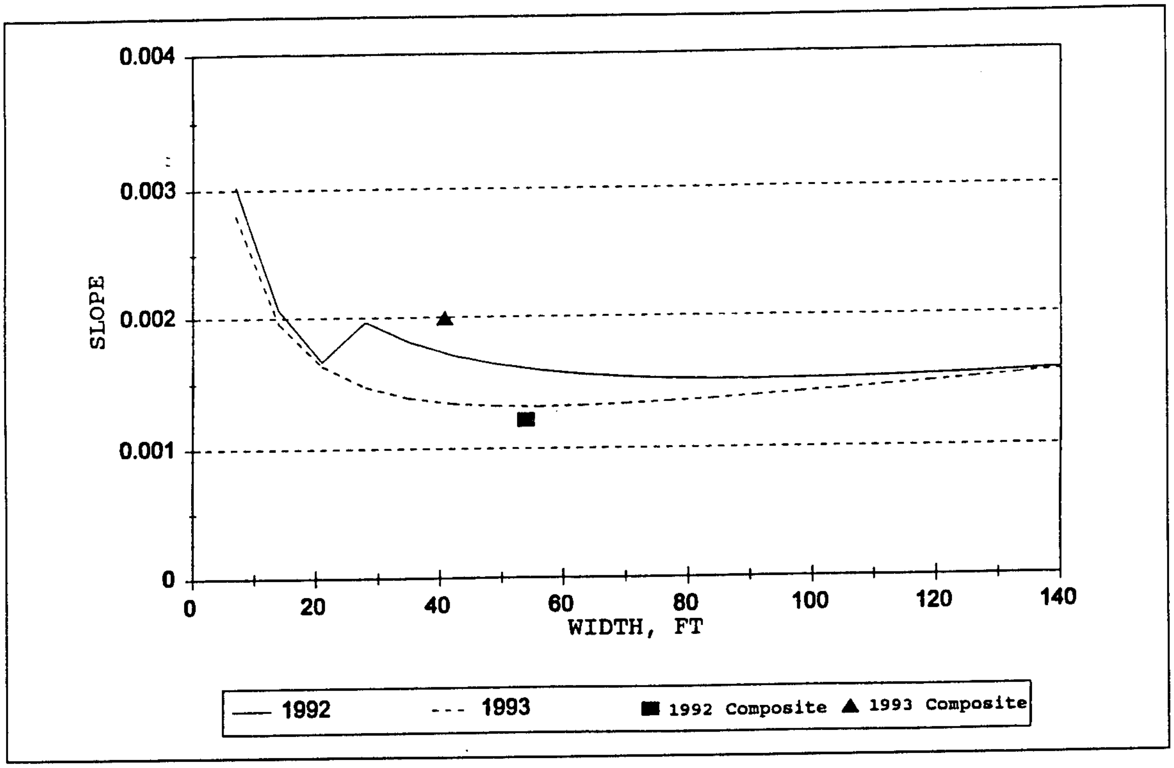



Figure 49. Hickahala Creek (Site 11), segment 2, stable channel and composite section comparison

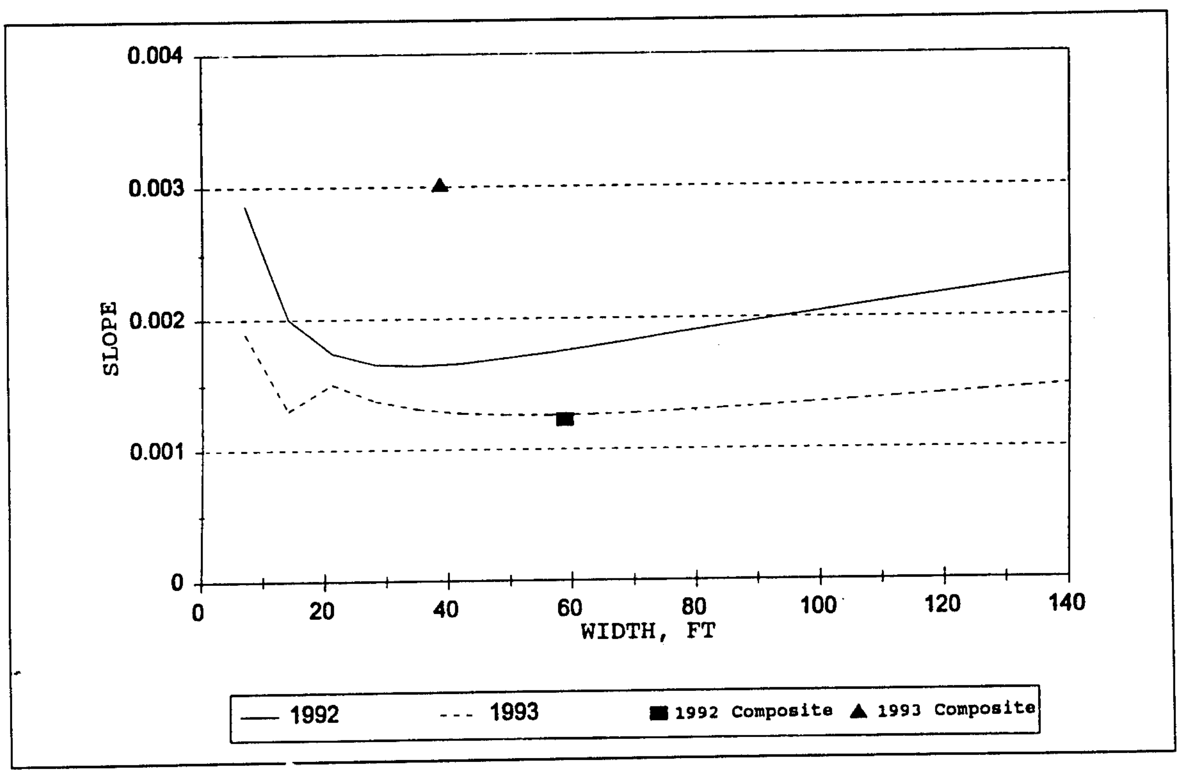



Figure 50. Hickahala Creek (Site 11), segment 3, stable channel and composite section comparison

### Red Banks Creek, Site 9

Red Banks Creek, Site 9, is located in the Coldwater River Basin in Watson, MS. The site extends for approximately 14,000 ft upstream of the Watson-to-Moscow Highway and is divided into two segments as shown in Figure 51. Segment 1, the downstream segment, extends from the highway bridge upstream for a distance of 5,000 ft to the most downstream of four chevron weirs constructed by the Vicksburg District. Segment 2, the upstream segment, encompasses the upstream chevron weirs and is about 9,000 ft in length. The chevron weirs are unique structures among the DEC monitoring sites, and appear to be effective in raising the bed of Red Banks Creek by aggradation. The weirs are shown in Figure 51 at locations of 5,000, 7,000, 9,000, and 13,000 ft upstream of the bridge. Most of segment 2 has a riprap toe emplaced along both banks. Segment 1 has some bank stabilization and does not have any grade control. At the bridge, a combination of low stone dikes connected at the riverward extent has worked effectively in aligning the stream with the bridge right of way and narrowing the channel. The drainage at the study site is 27.8 square miles, and the 2-year discharge is 3,951 cfs, which flows within bank. The bed slope in 1993 for segment 1 was surveyed to be 0.0009, and segment 2 was surveyed to be 0.0018. The bed material is characterized as medium sand ($D_{50}$ = 0.5 mm). The channel bed slope is relatively flat; loose sediment depth is greater than

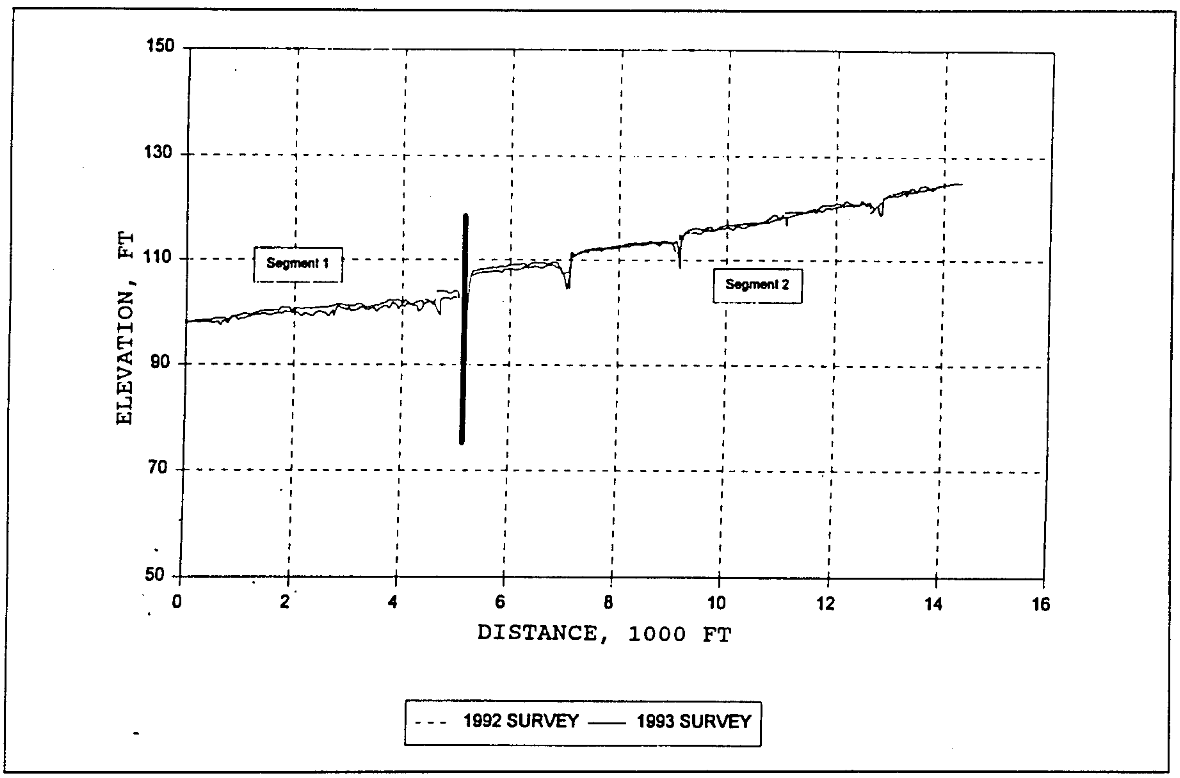



Figure 51. Comparison of 1992 and 1993 thalweg profiles for Red Banks Creek, Site 9

3 ft; and some reaches have berms colonized by birch and willow trees. However, at other locations the banks are kudzu covered and steep, and have no colonization by woody vegetation. Close attention should be paid to the possible causal relationship between steep bank and kudzu, which may be causing the lack of woody vegetation.

Table 7 summarizes data analysis of Red Banks, Site 9. The data show that the 2-year composite energy slope in 1993 is greater than bed slope, and that the energy slope in segment 2 is greater than in segment 1. This implies that emplacement of the chevron weirs has helped to create a steeper hydraulic gradient as aggregation has occurred. Sediment transport capacity for the composite section as calculated by the Brownlie transport function is relatively high, which implies that the sediment supply is also high. As stabilization measures are emplaced in the watershed to reduce the sediment supply, the slope of the bed will flatten and the chevron weirs may be subjected to hydraulic forces, which will be more severe than under present conditions. Additional analyses, perhaps HEC-6, are needed, to determine future structures or loading conditions.

Figures 52 and 53 compare the stable channel computations and the composite cross sections for segment 1 and segment 2, respectively, which emphasize the possibility of chevron weir instability. If the estimates of the composite hydraulic roughness are correct, Figure 52 indicates that segment 1 conditions are near the stable channel criteria. Figure 54 is an expanded segment 2 profile.

Figure 53 indicates that the channel may be degradational. Relatively high composite slopes, such as 0.0002, suggest that this segment should be monitored closely and that the effect of upstream stabilization efforts should be carefully considered.

### Lee Creek, Site 10

Lee Creek, Site 10, is a relatively small tributary of the Coldwater River with a drainage area of approximately 7.5 square miles that was originally chosen because no construction was planned for the watershed. The reach extends approximately 2,000 ft upstream and downstream of the highway bridge for a total of 4,000 ft. The upstream land use is rural and cotton fields, and the reach appears to receive a significant sediment load from a combination of bank erosion and fine-grained sediment from the cotton fields. The bed sediment is characterized as medium sand ($D_{50}$ = 0.40 mm), and the depth of loose sediment in the channel is generally greater than 3 ft. The 2-year discharge for the reach is 1,377 cfs, and the bank-full discharge is approximately 900 cfs, about 65 percent of the 2-year discharge. No head-cutting was observed in the study reach; however, as shown in Figure 55, the thalweg comparison for 1992 and 1993 indicates significant degradation in the upper portion of the study reach. An observation concerning the bank erosion is that in the cotton field portion of the reach, little woody vegetation exists

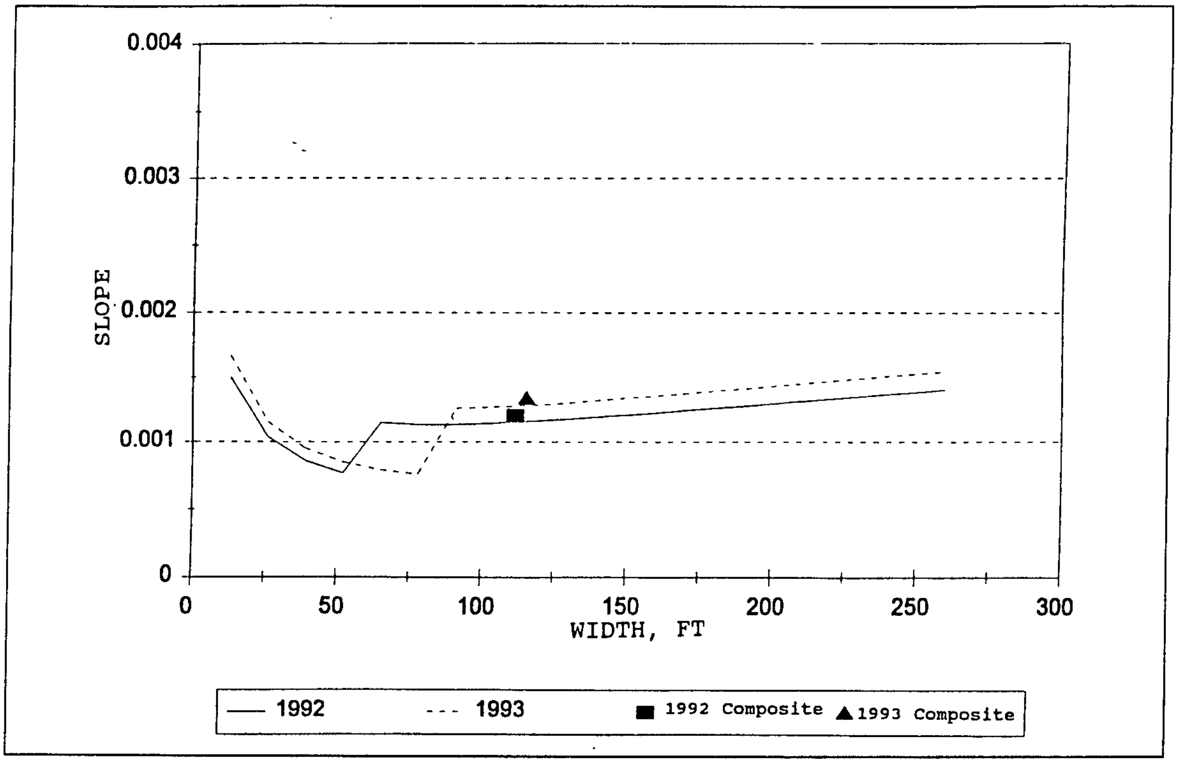



Figure 52. Comparison of 1992 and 1993 stable channel computations and composite cross sections for Red Banks, Site 9, segment 1

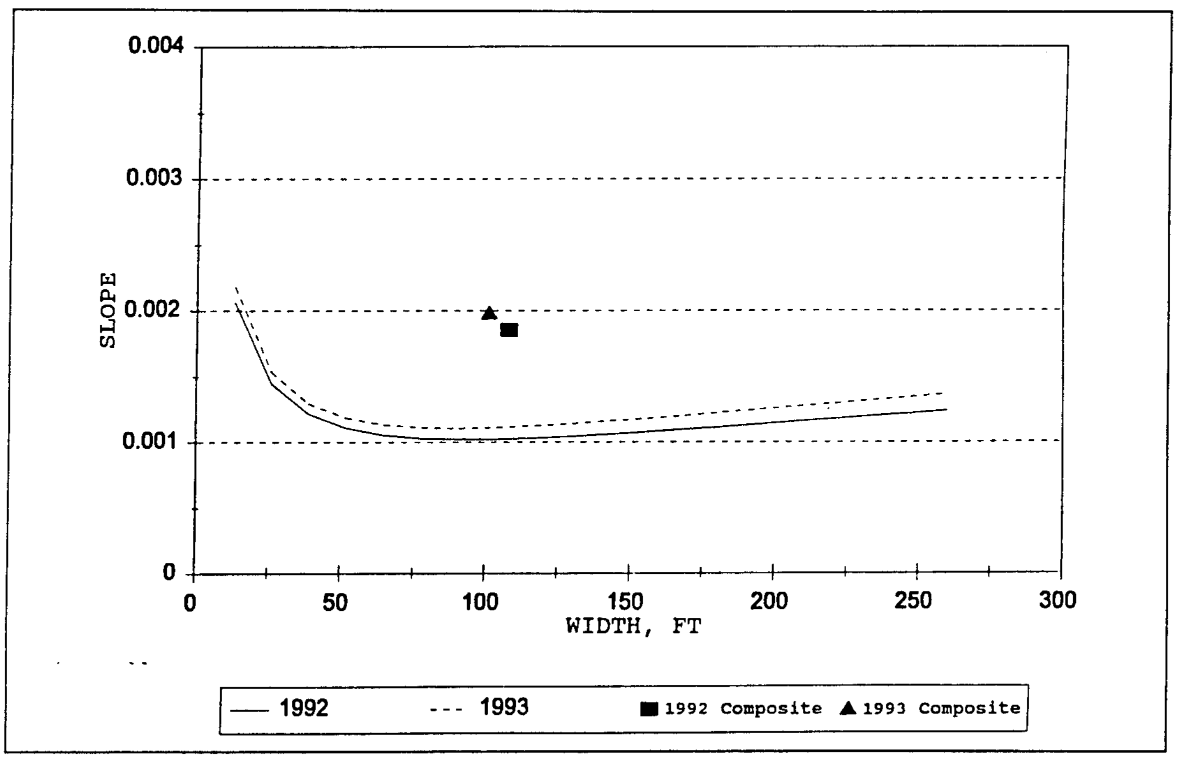



Figure 53. Comparison of 1992 and 1993 stable channel computations and composite cross sections for Red Banks, Site 9, segment 2

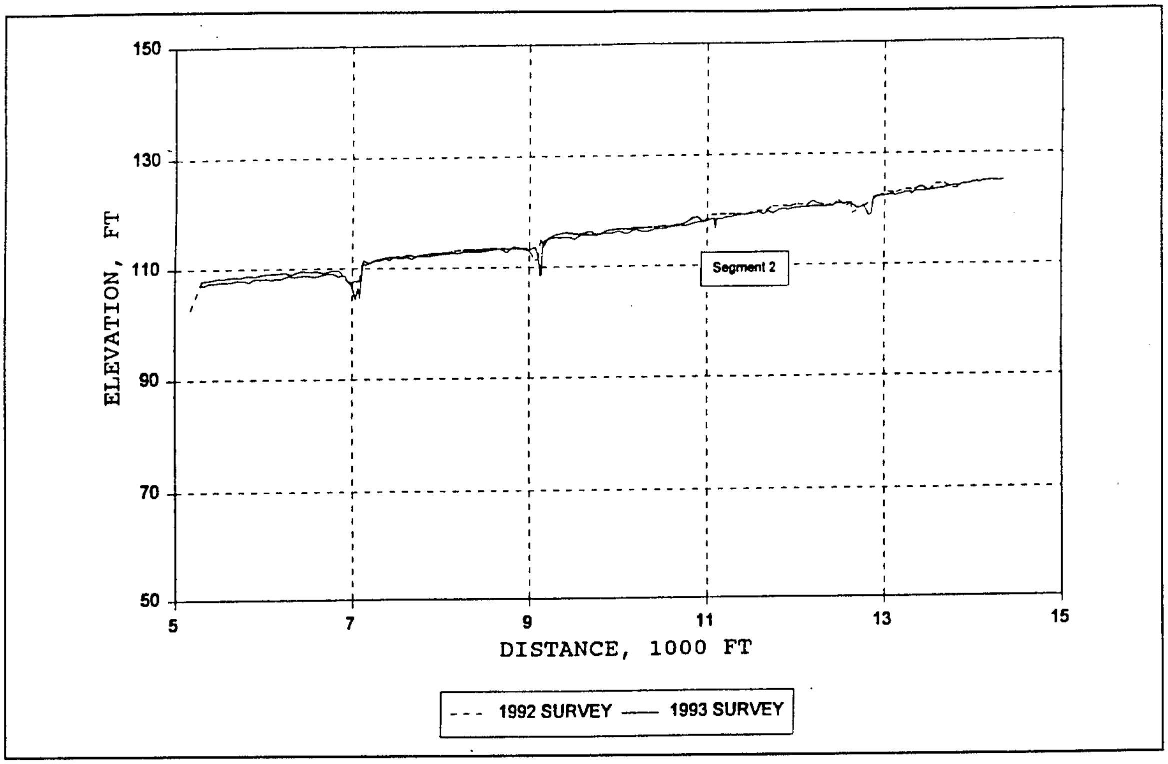



Figure 54. Expanded segment 2 thalweg profile for Red Banks, Site 9

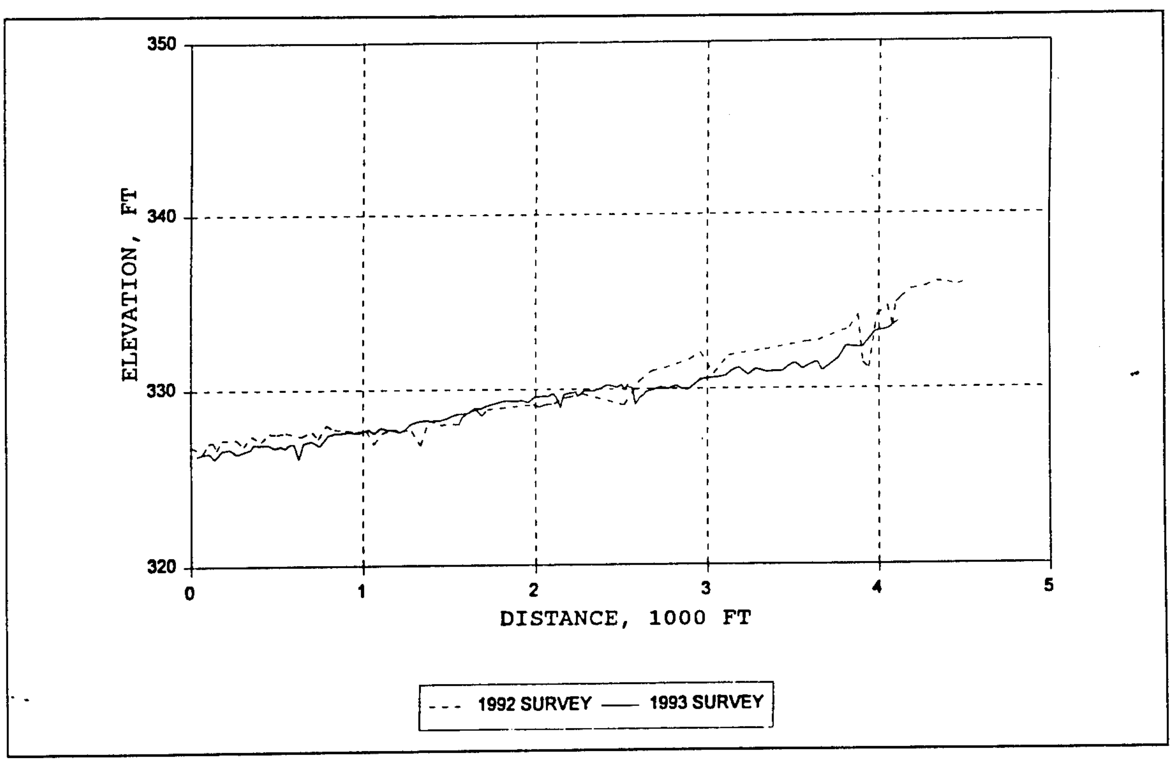



Figure 55. Comparison of 1992 and 1993 thalweg profiles for Lee Creek, Site 10

along the toe of the bank slope, apparently having been suppressed by kudzu. Birch trees growing on an island are causing erosion of the opposite banks. The downstream portion of the study reach is in pasture, and birch trees are growing on the top bank and down to the toe of the slope. Banks are relatively stable and shaded, and with the exception of lack of low-water depth, the downstream reach appears to be a candidate for conservation as a good habitat.

Table 8 summarizes the data and analyses for Lee Creek, Site 10. The bed slope survey and the composite energy gradient both indicate that the gradient is decreasing. Along with the gradient change, the width is decreasing and the depth is increasing. Compared to regime dimensions, the composite channel is narrower, shallower, and much steeper.

Figure 56 compares stable channel computations and composite cross-section characteristics for Lee Creek. The computations indicate that a flow regime shift occurs at the dimensions of the composite cross section. If estimates of the hydraulic roughness are correct, then the 1993 channel appears degradational. Based on the observed degradation (Figure 55) and the results of the stable channel computations, consideration should be given to construction of bed stabilization in the study reach and upstream of the reach, perhaps using chevron weirs.

## Hotophia and Marcum Creeks, Site 13

Hotophia and Marcum Creeks, Site 13, extend from an early low-drop structure downstream of Highway 315 on Hotophia Creek upstream for approximately 8,500 ft, and up Marcum Creek, a tributary to Hotophia Creek, for a distance of approximately 2,800 ft. Deer Creek is also a tributary of Hotophia Creek, but is not surveyed as part of the site. For the 1992 survey, a low-drop existed at the downstream extent, a high-drop about 6,000 ft upstream was under construction, and a low-drop existed at the upstream extent of the Marcum Creek reach. Presently, the Hotophia Creek portion of the site is divided into two segments. Segment 1 is downstream of the existing high-drop structure, and Segment 2 is upstream of the existing high-drop structure. Marcum Creek is considered as a separate segment. The high-drop under construction during the 1993 survey was in segment 1.

Figure 57 is a comparison of the 1992 and 1993 thalweg profiles for Hotophia Creek, and Figure 58 is a similar graph for Marcum Creek. These two graphs indicate change in the thalweg elevations. Some filling has begun upstream of the high-drop in segment 2, and minor degradation has occurred downstream of the high-drop in segment 1. Marcum Creek appears to have degraded in some locations approximately 3 ft, and degradation has moved upstream and is approaching the Marcum Creek low-drop. Downstream of the high-drop, the banks are failing and incision is continuing. Completion of the downstream high-drop should cause dramatic changes in this reach. Upstream of the existing high-drop, the bank appears to be relatively stable

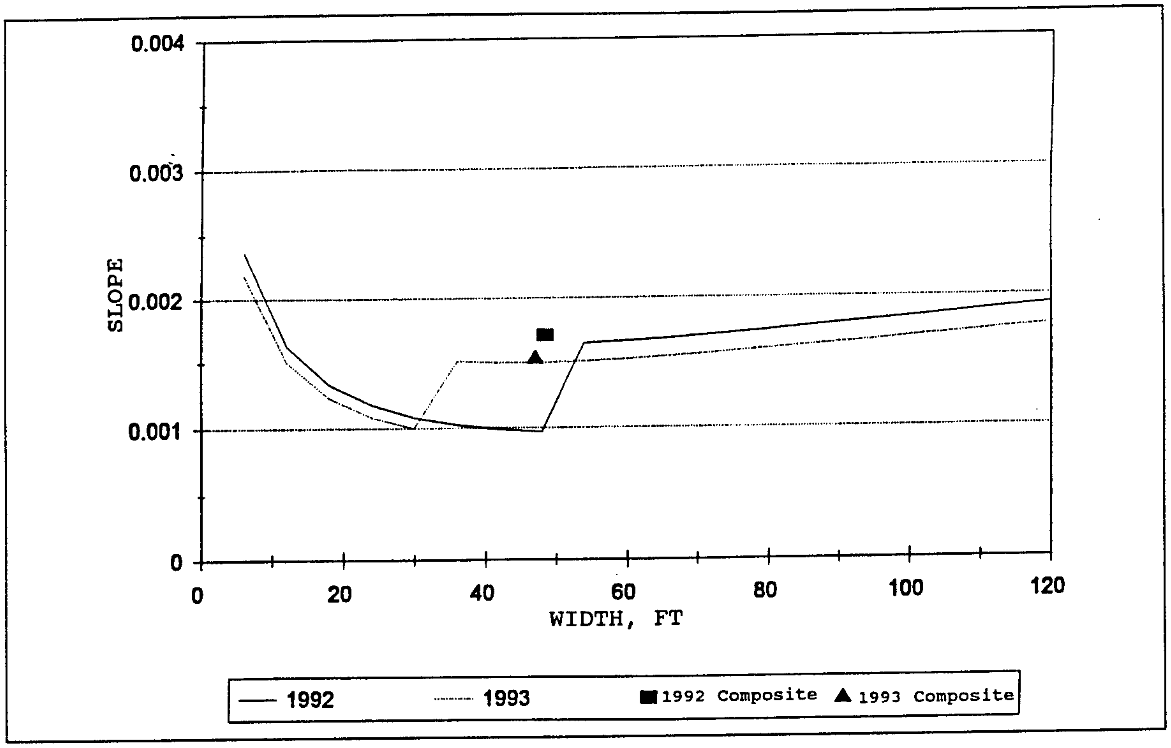



Figure 56. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for Lee Creek, Site 10

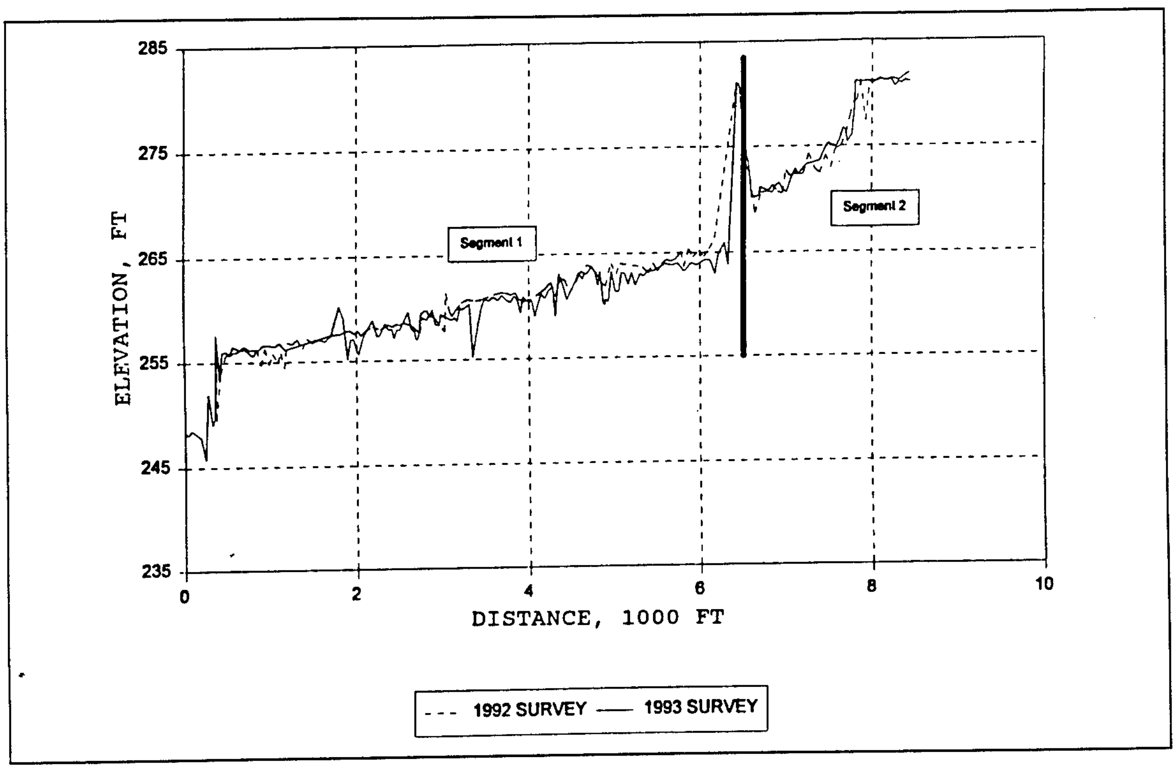



Figure 57. Comparison of 1992 and 1993 thalweg profiles for Hotophia Creek, segments 1 and 2

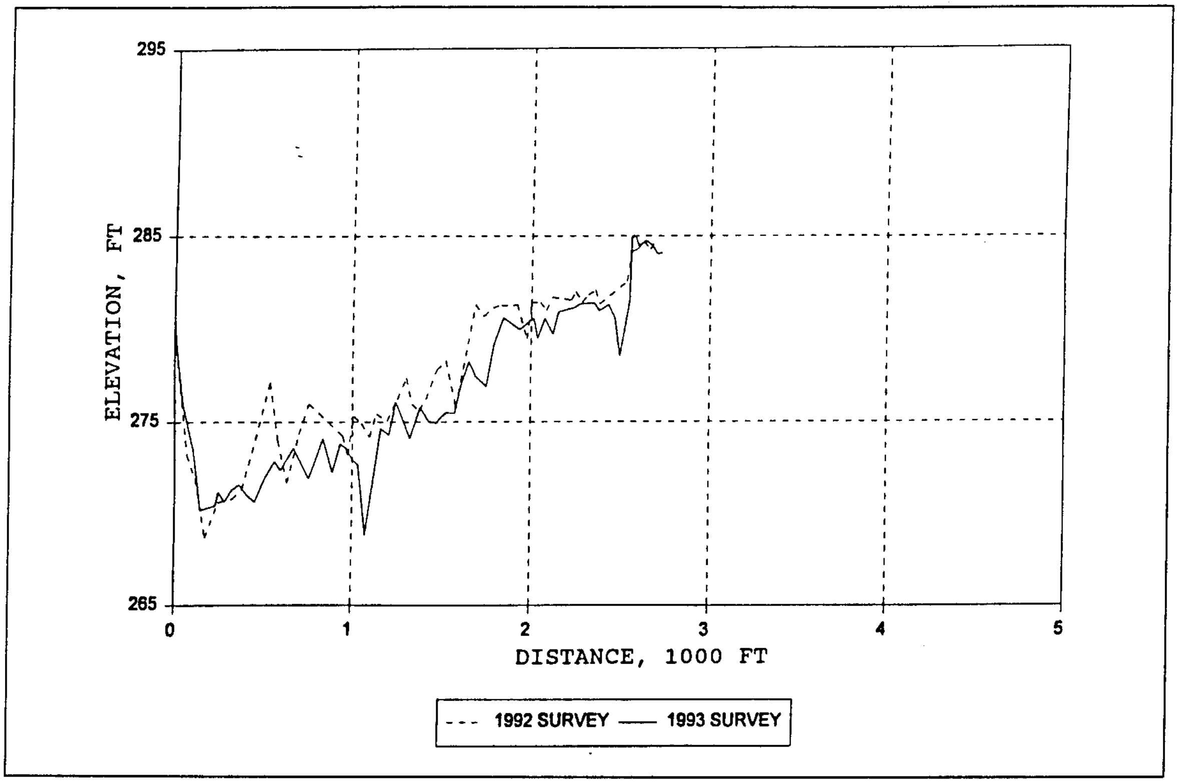



Figure 58. Comparison of 1992 and 1993 thalweg profiles for Marcum Creek

although some deep-seated tension cracking is evident. On the left bank upstream of the existing high-drop, cattle are causing some erosion. The upstream sediment wedge appears to be slowly moving downstream, and filling of the Marcum Creek confluence should influence that tributary. The drainage area at the downstream extent of segment 1 is 17.1 square miles with a 2-year discharge of 3,386 cfs. Segment 2 drainage area is 5.4 square miles with a 2-year discharge of 1,180 cfs. Marcum Creek has a drainage area of 4.7 square miles with a 2-year discharge of 1,190 cfs. The 2-year discharge is contained within bank.

Table 9 is a summary of the data analysis for these segments. The effect of the existing high-drop structure is shown by the reduction of the energy gradient upstream of the structure from 0.0047 in 1992 to 0.00058 in 1993, by the reduction in slope from downstream to upstream, and by the reduction of surveyed channel slope and energy gradient on Marcum Creek and on Hotophia Creek upstream of the structure. Sediment transport capacity was reduced to only 3.4 percent of the prestructure amount for segment 2. This can be expected to increase as the pool fills, but a long-duration reduction can be expected. Completion of the upstream and downstream high-drops will have dramatic effects on the site, and additional sediment reduction and storage will be created. Figures 59, 60, and 61 compare the stable channel computations and the composite cross-section characteristics for segments 1 and 2 and Marcum Creek. Lower Hotophia appears to be degradational and will be affected by high-drop construction. The relationship between the 1992

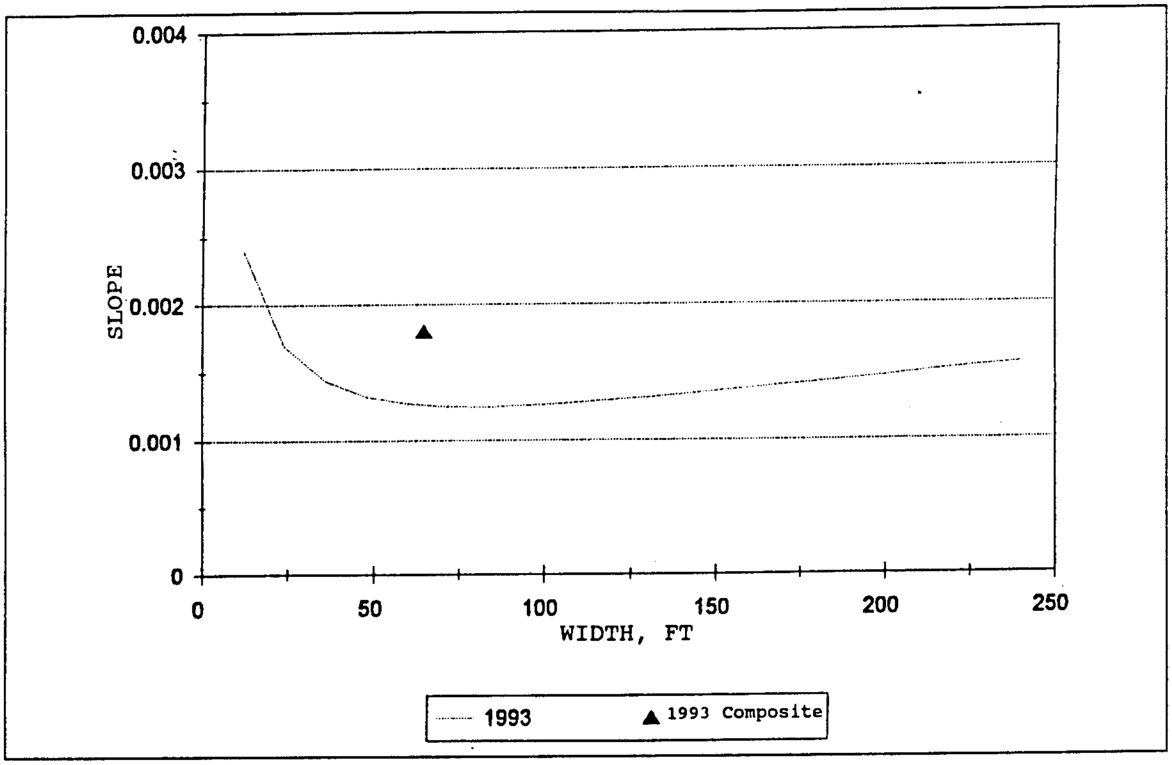



Figure 59. Comparison of the 1993 stable channel computations with the composite cross section for Hotophia Creek, segment 1

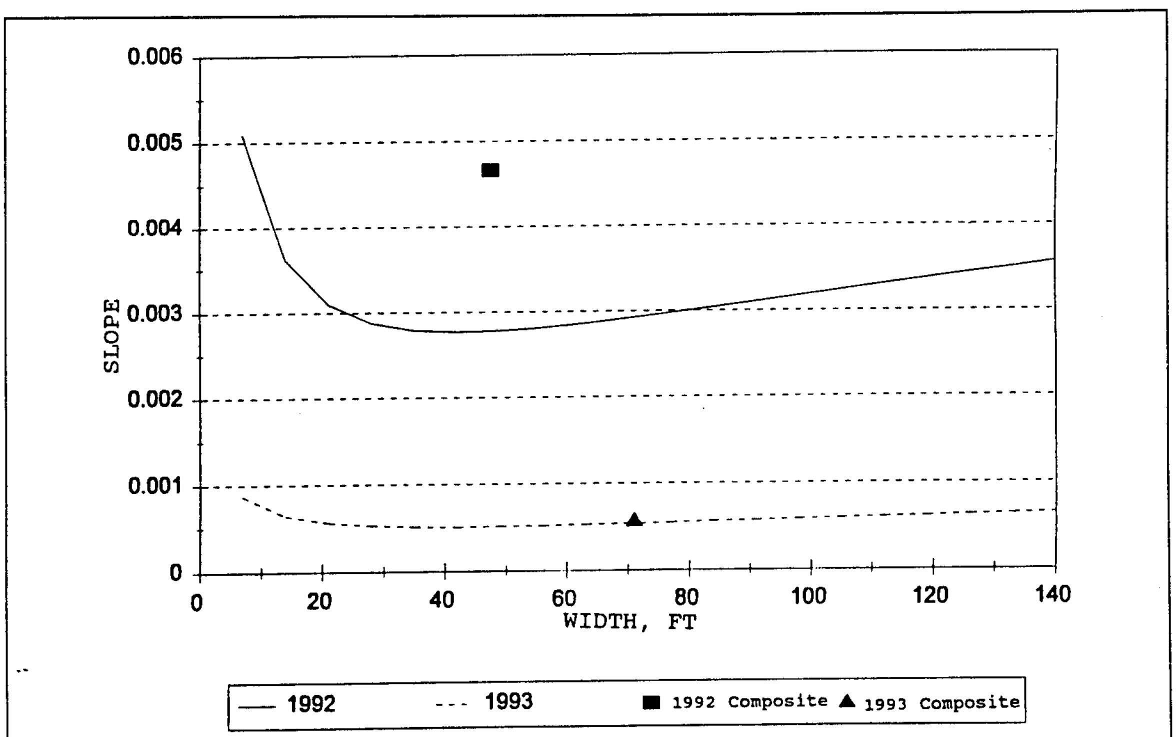



Figure 60. Comparison of 1992 and 1993 stable channel computations with composite cross section for Hotophia Creek, se?ment 2

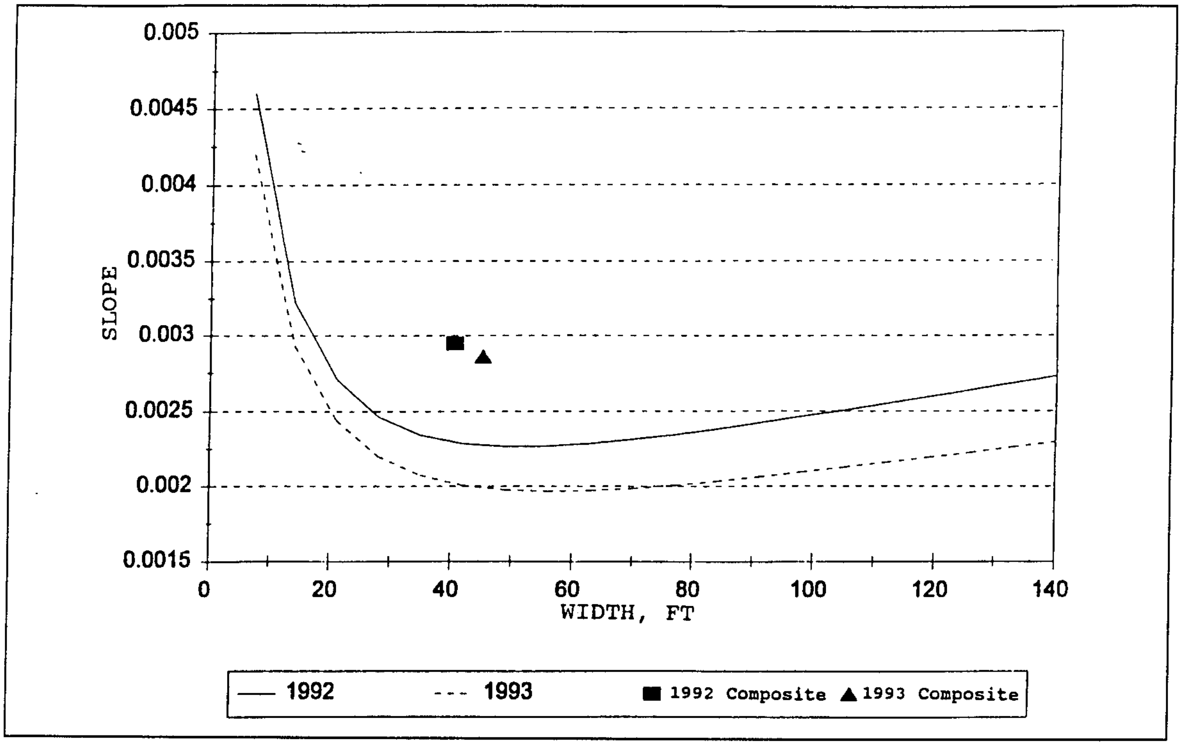



Figure 61. Comparison of 1992 and 1993 stable channel computations with composite cross section for Marcum Creek

and 1993 data in Figure 60 is spectacular in the amount of slope reduction, and in the relationship between the composite data point and the stable channel curve for the 2 years. The 1992 data indicate a strongly degradational reach, and the 1993 data indicate that the composite section is near equilibrium. The gradient of the reach has decreased in Marcum Creek, but the reach is continuing to be degradational. Figure 61 confirms observations made from the thalweg profiles.

### Nolehoe Creek, Site 7

Nolehoe Creek, Site 7, is a small tributary of the Coldwater River Basin. Drainage area is only 3.7 square miles and is in an urbanizing watershed of metropolitan Memphis. The 2-year discharge is 978 cfs and is contained within the channel banks. The study reach was initially channelized. Incision is severe and continuing upstream. Grade control has been planned for the reach; however, construction access has been denied. Much of the reach has steep banks and a clay bed with relatively little accumulated sediment. The reach generally gives the appearance of extreme instability. Samples of the noncohesive bed material collected in the study reach are characterized as medium gravel ($D_{50}$ = 10 mm). These samples are representative of the material remaining in the reach and have been used for the stable channel analyses; however, the samples may not be representative of the material moving through the reach. With the channel bed slope of 0.0037, sand may be carried through as wash load, whereas if the bed slope and energy gradient were reduced, the character of the deposits and bed sediments may be changed.

Table 10 summarizes the data analyses for Nolehoe Creek, Site 7. The data indicate that segment 1 bed slope is about the same and the composite energy slope is decreasing. Figure 62 indicates minor degradation of the upstream portion of segment 1. Segment 2 is degrading as shown in Figure 62, and the data show that the bed slope is increasing. Figures 63 and 64 are comparisons of the stable channel computations and the composite cross-section characteristics. These figures generally indicate that the site is aggrading, which is not supported by other data. As discussed in the previous paragraph, this discrepancy was probably caused by use of a sediment size not representative of the total sediment transport.

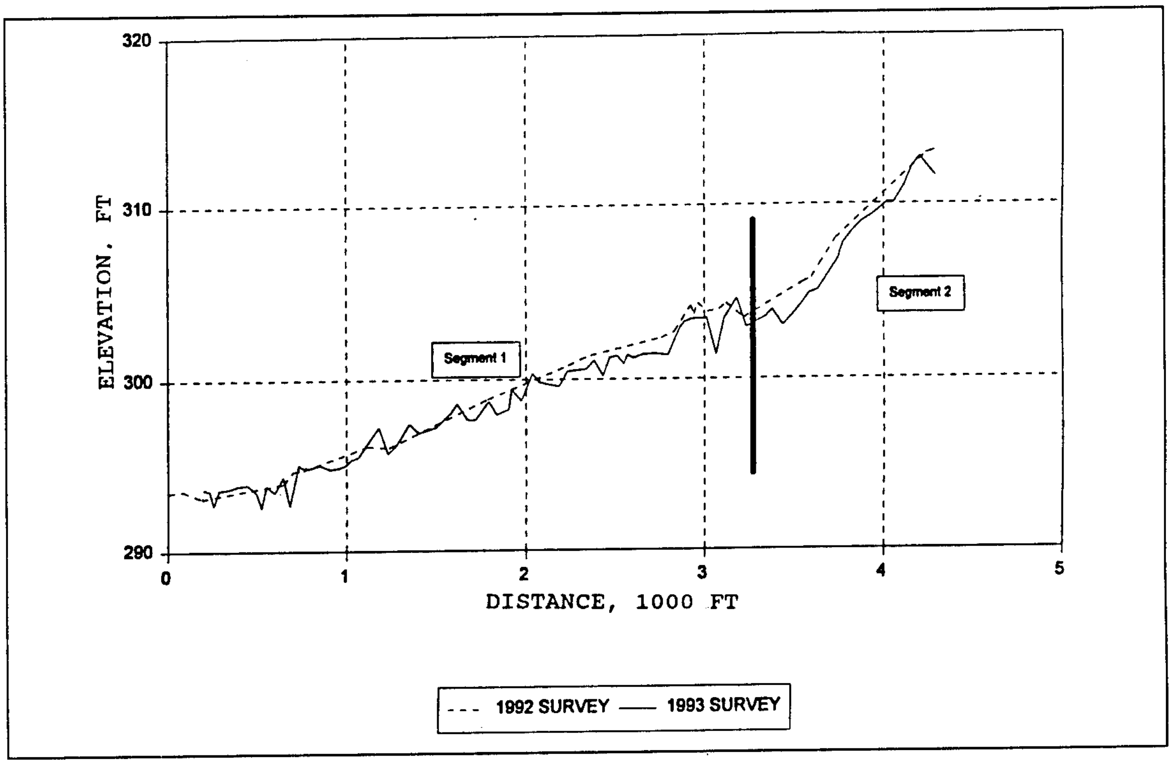



Figure 62. Comparison of 1992 and 1993 thalweg profiles for Nolehoe Creek, segments 1 and 2

## Sarter Creek, Site 15

Sarter Creek, Site 15, is a tributary of Otoucalofa Creek with a drainage area of approximately 6.4 square miles. The 2-year discharge is 1,391 cfs, and the bank-full discharge is approximately 1,010 cfs, about 73 percent of the 2-year discharge. The bed material is characterized as a medium sand ($D_{50}$ = 0.37 mm). Sarter Creek was originally chosen because the channelized stream had maintained the straight alignment and little evidence of

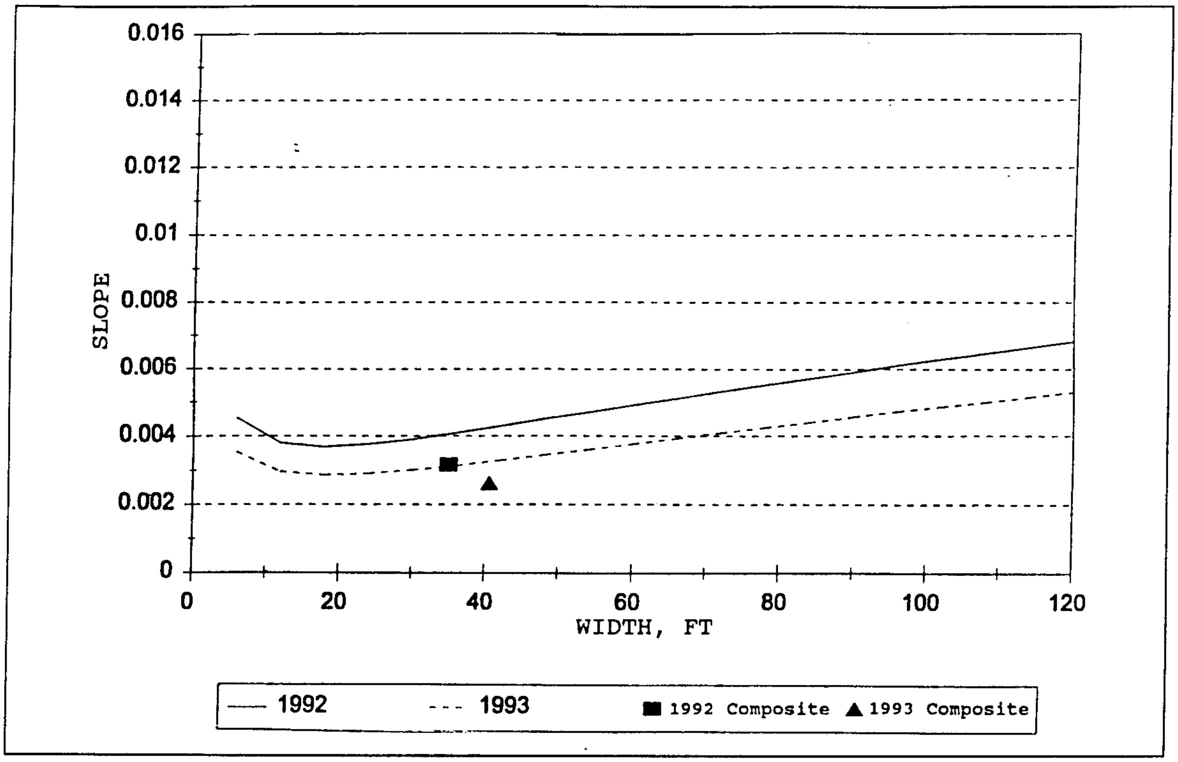



Figure 63. Comparison of 1992 and 1993 stable channel computations and composite cross-section characteristics for Nolehoe Creek, segment 1

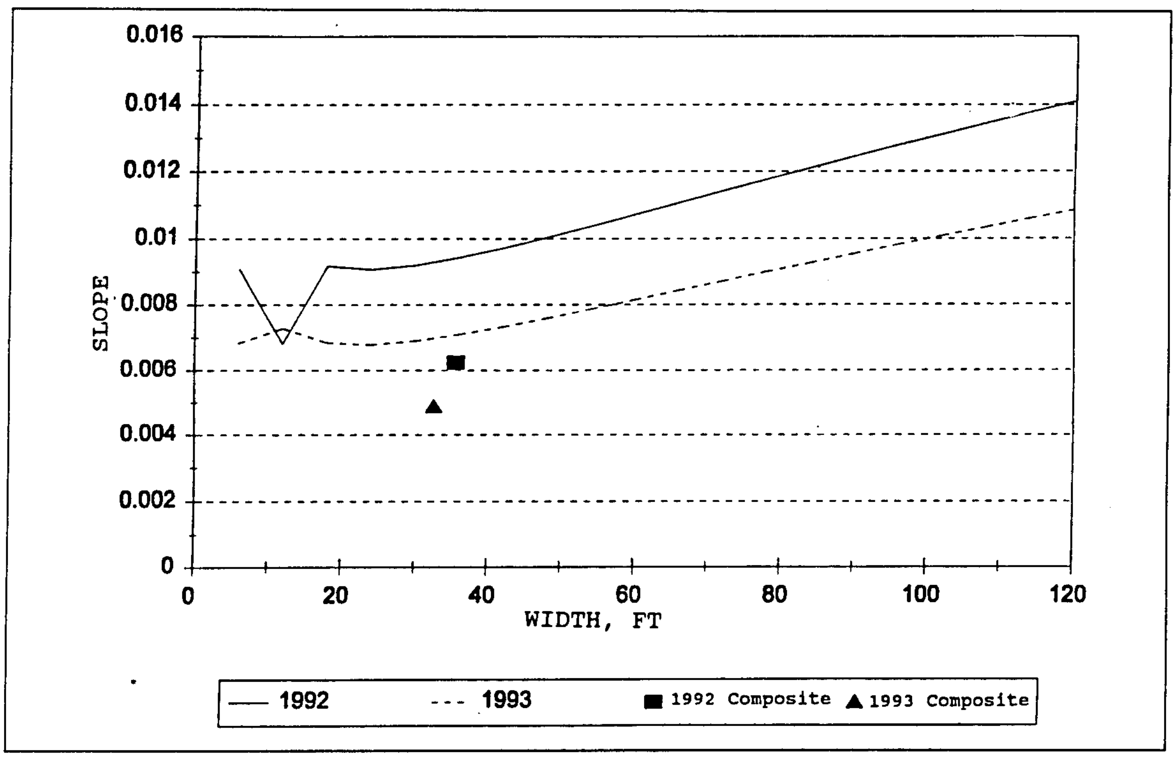



Figure 64. Comparison of 1992 and 1993 stable channel computations and composite cross-section characteristics for Nolehoe Creek, segment 2

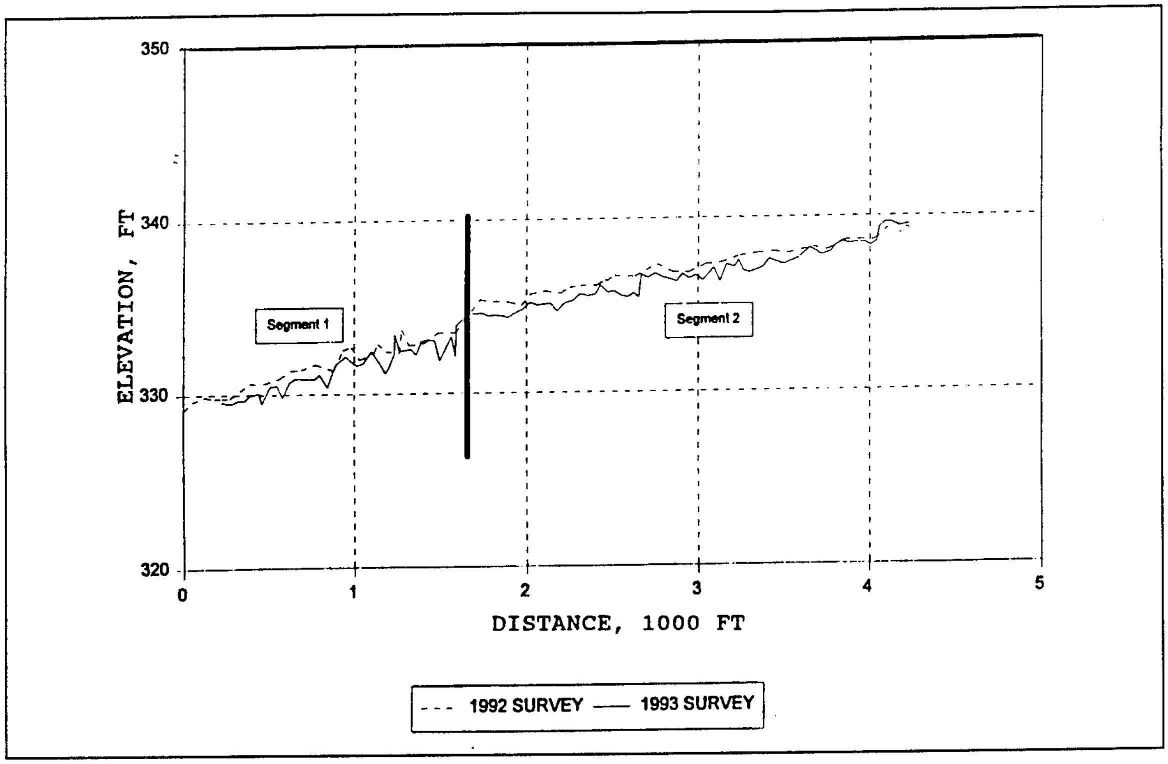



Figure 65. Comparison of 1992 and 1993 thalweg profiles for Sarter Creek

incision and widening was apparent. As shown in Figure 65, the site is divided into two segments at a headcut, which exists about 1,700 ft upstream of the downstream extent of the site. Both segments experienced degradation during the 1992-1993 period.

Table 11 summarizes the data analyses for Sarter Creek, Site 15. The bed slope and the depth increased slightly, which may be explained by recent incision for which sufficient time has not passed to accomplish widening. Lack of widening could explain the resulting decrease in the composite channel energy gradient and reduction of sediment transport. If this assumption is correct, widening will occur. Segment 2 apparently has not experienced the most recent headcut migration and deep incision. Channel slope, composite cross-section energy gradient, and sediment transport increased with little change in width or depth.

Figures 66 and 67 are the stable channel computations and composite cross-section comparisons for Sarter Creek, segment 1 and segment 2, respectively. Figure 66 indicates that segment 1 is degradational. Figure 67 indicates that segment 2 is in relative equilibrium with the sediment supply. Unfortunately, this reach will be destabilized by the headcut migrating upstream unless bed stabilization measures are implemented.

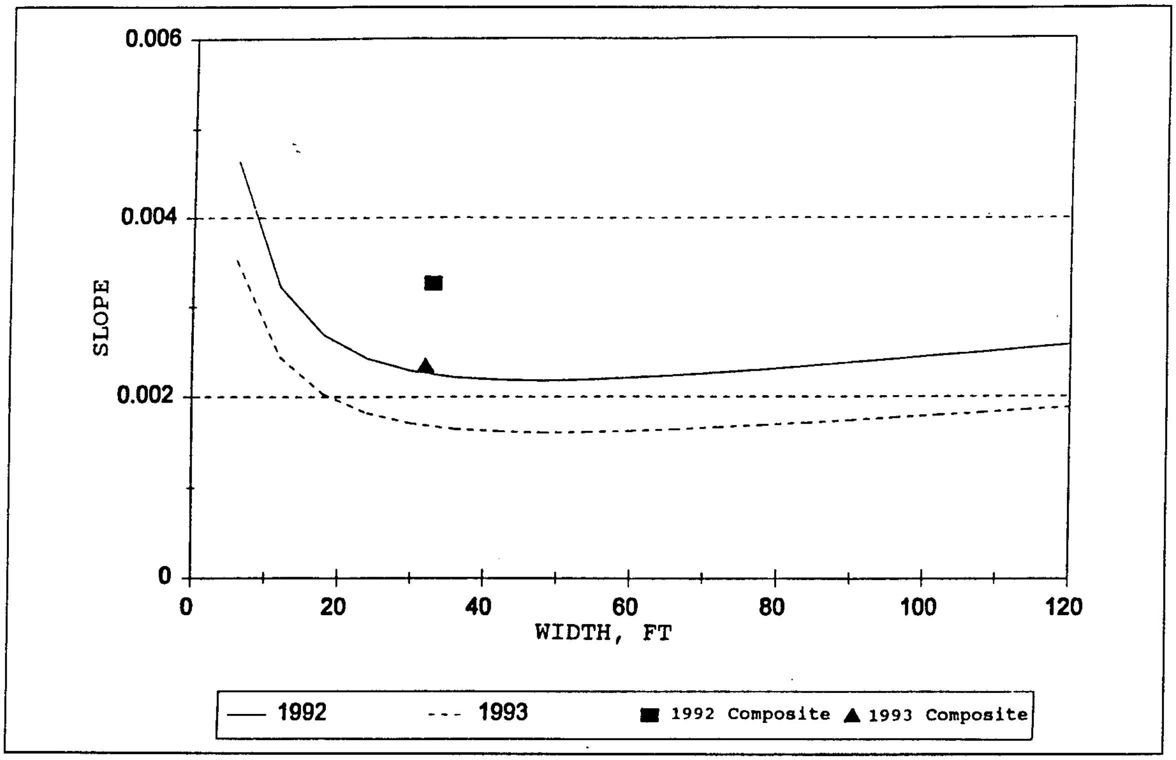



Figure 66. Comparison of 1992 and 1993 stable channel computations and composite cross-section characteristics for Sarter Creek, segment 1

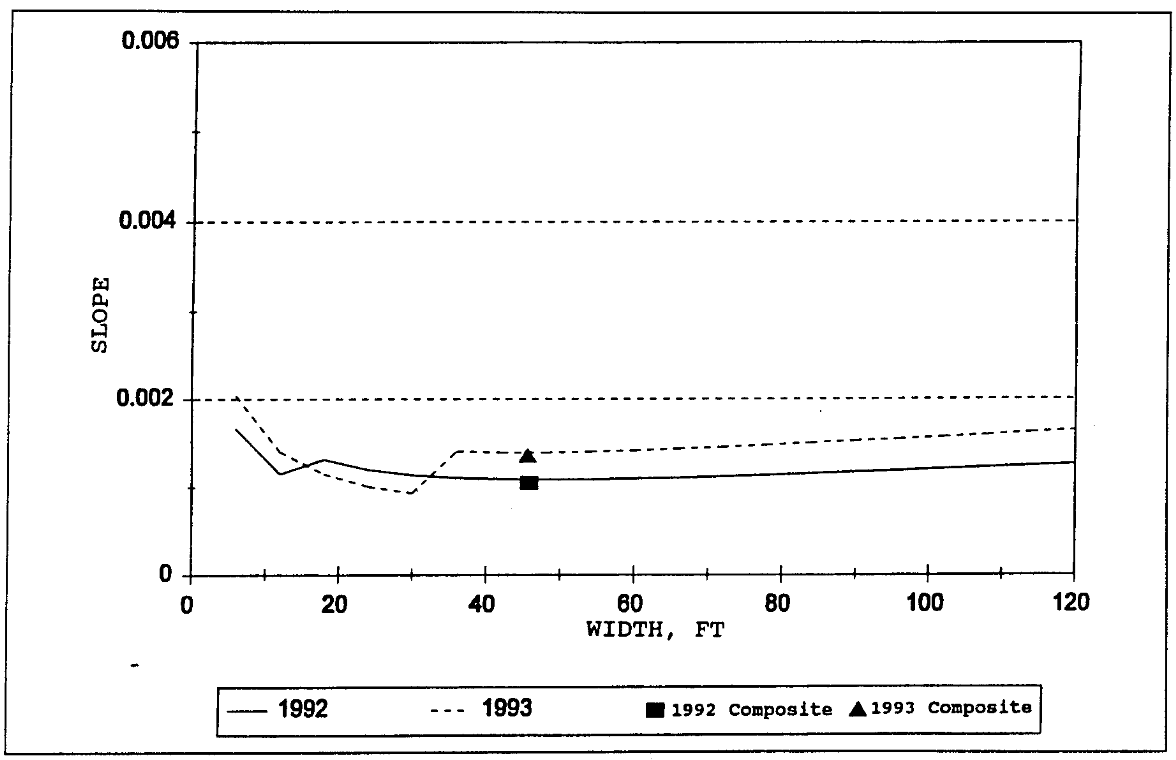



Figure 67. Comparison of 1992 and 1993 stable channel computations and composite cross-section characteristics for Sarter Creek, segment 2

## James Wolf Creek, Site 19

James Wolf Creek, Site 19, is a tributary of Hickahala Creek with a drainage area of 10.8 square miles. The bed material is characterized as medium sand ($D_{50}$ = 0.35 mm), and loose sand depth through the site is in excess of 3 ft. The 2-year discharge of James Wolf Creek is 2,189 cfs and is completely contained within the channel. A low-drop grade control structure exists approximately 2,200 ft upstream from the downstream limit defining segment 1, and the site extends approximately 3,000 ft upstream to define segment 2. The banks are generally steep and 20 to 25 ft in height. Figure 68 is a comparison of the thalweg profiles for 1992 and 1993. Minor degradation occurred in the upper portion of segment 2, and minor aggradation occurred in the upper portion of segment 1 and lower portion of segment 2. Although little change is evident from the thalweg comparison, the amount of encroachment in segment 2 by willow trees at the bank toe is significant. Encroachment by the high-roughness vegetation may be the cause of the apparent incision in segment 2. No roughness data exist for the site.

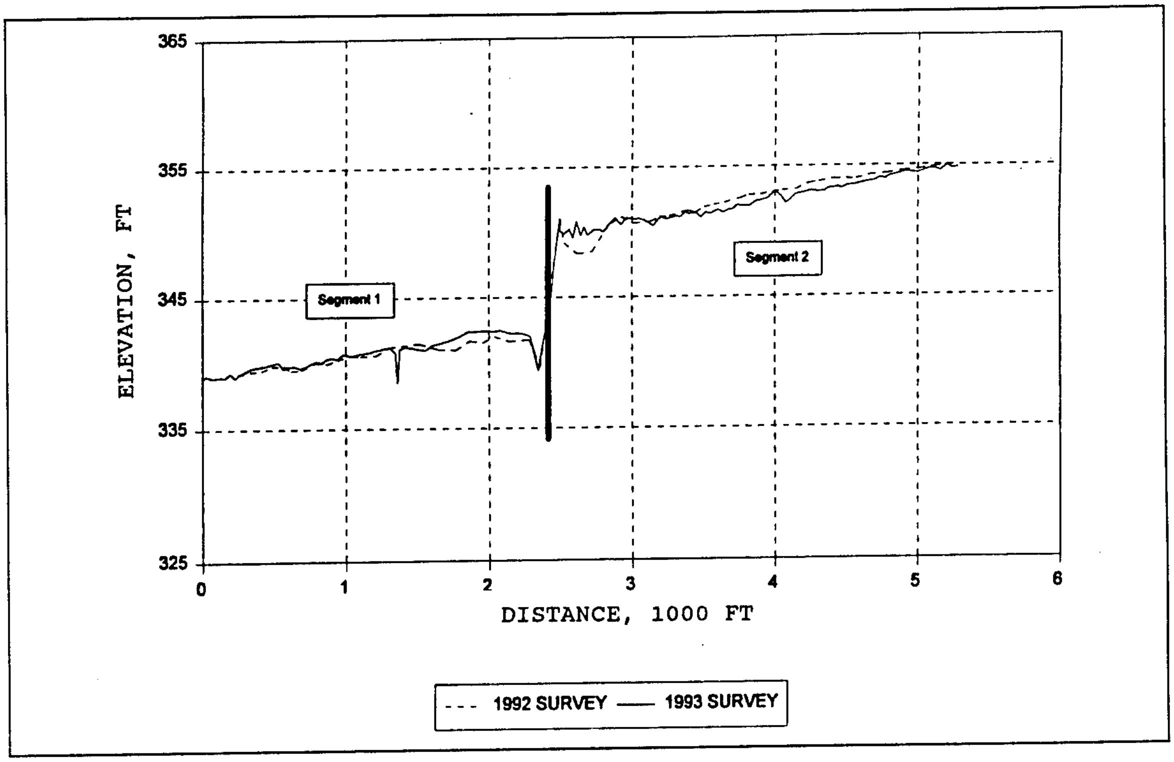



Figure 68. Comparison of 1992 and 1993 thalweg profiles for James Wolf Creek, Site 19

Table 12 summarizes the data and analyses for the site. Very little change is evident in segment 2, with minor increases in the relatively low unit sediment transport capacity. Change in the energy gradient results in reduction in the sediment transport; however, the rate remains high. Construction of two low-drops downstream to be completed after the surveys in 1993 may have some effect at the site, and evidence may be available in the 1994 surveys.

Figures 69 and 70 compare the stable channel computations and the composite cross-section data for James Wolf Creek. If the estimates of the roughness are correct, both figures indicate that the channels are degradational. In general, all the data and analyses indicate relatively small change occurred at the site between 1992 and 1993. The stability of the existing low-drop and the effect of the two downstream drops under construction will be of interest in 1994.

### Long Creek, Site 20

Long Creek, Site 20, extends approximately 10,500 ft upstream of a low-drop structure completed in 1993. Including the downstream structure, four low-drop structures are located within the site, and as shown in Figure 71, the structures define four segments. Comparison of the surveyed thalwegs indicates that aggradation of several locally deep portions of the site occurred. Erosion of the secondary scour hole downstream of the second grade control increased, which may place additional stress on that structure. The drainage area for Site 20 is 11.1 square miles with a 2-year discharge of 2,209 cfs and a bank-full discharge of 960 cfs. The bed sediment is characterized as medium sand ($D_{50}$ = 0.38 mm).

Table 13 summarizes the data and analyses for Long Creek. The data indicates that the bed slope changed little and the energy gradient decreased for segments 1, 2, and 3. The data from segment 4 may be undependable due to the short length of the reach and due to the variation in channel morphology at the site. Additional cross sections and a greater length will be surveyed for 1994. The effects of the low-drop grade control structures in reducing energy gradient in segments 1, 2, and 3 have resulted in low rates of unit sediment transport. Channel composite width is increasing significantly, probably due to raising of the cross section in the incised channel. Figures 72, 73, and 74 are the comparisons of stable channel computations and the composite cross-section data. Each of the segments is indicated to be aggradational.

Continued monitoring of this site in 1994 will focus on segment 4 to determine if the sediment transport is as high as predicted by the existing data, and on the response of the channel to the drop structures, particularly the downstream, newer structure. The response of the channel upstream of the second structure has been of special interest as a bank stability study and because of the cyclic nature of fill and scour in the reach. This reach will be addressed in a separate report following the 1994 survey.

### Worsham Creek, Site 18

Worsham Creek, Site 18, consists of four streams: Worsham Creek, East Fork Worsham Creek, Middle Fork Worsham Creek, and West Fork Worsham Creek. Table 14 summarizes watershed data for Worsham Creek,

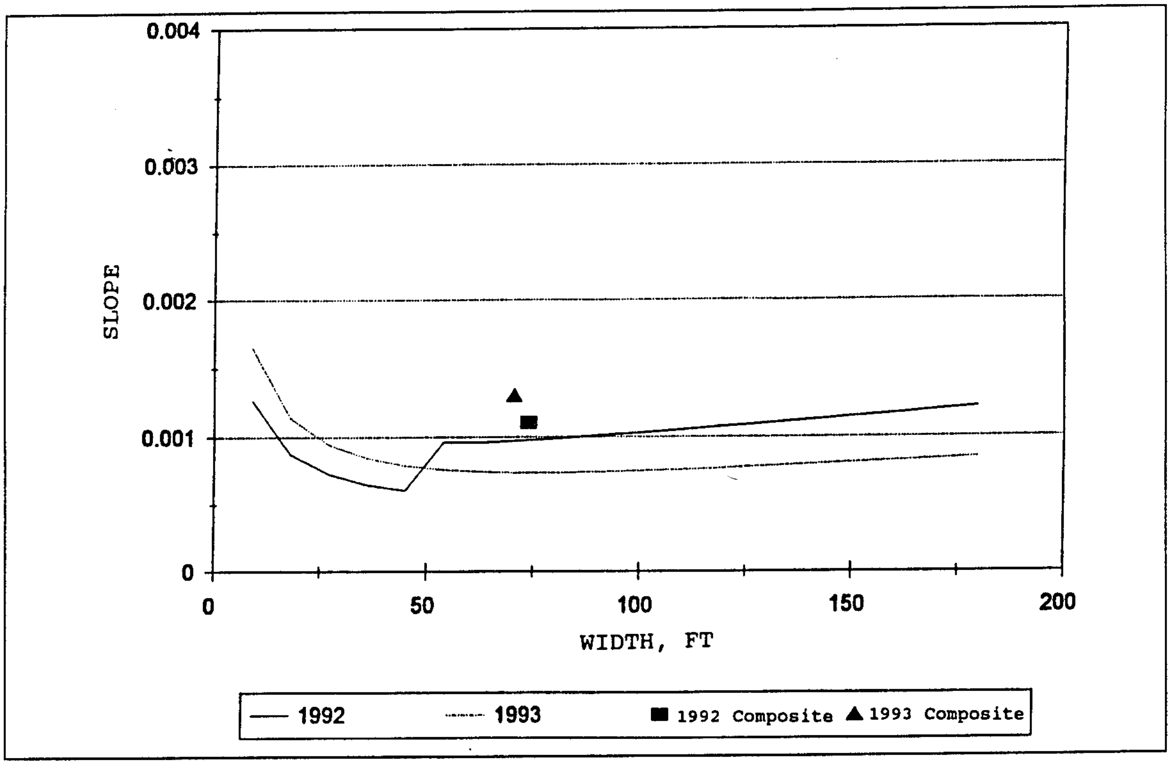



Figure 69. Comparison of 1992 and 1993 stable channel computations and composite cross-section characteristics for James Wolf Creek, Site 19, segment 1

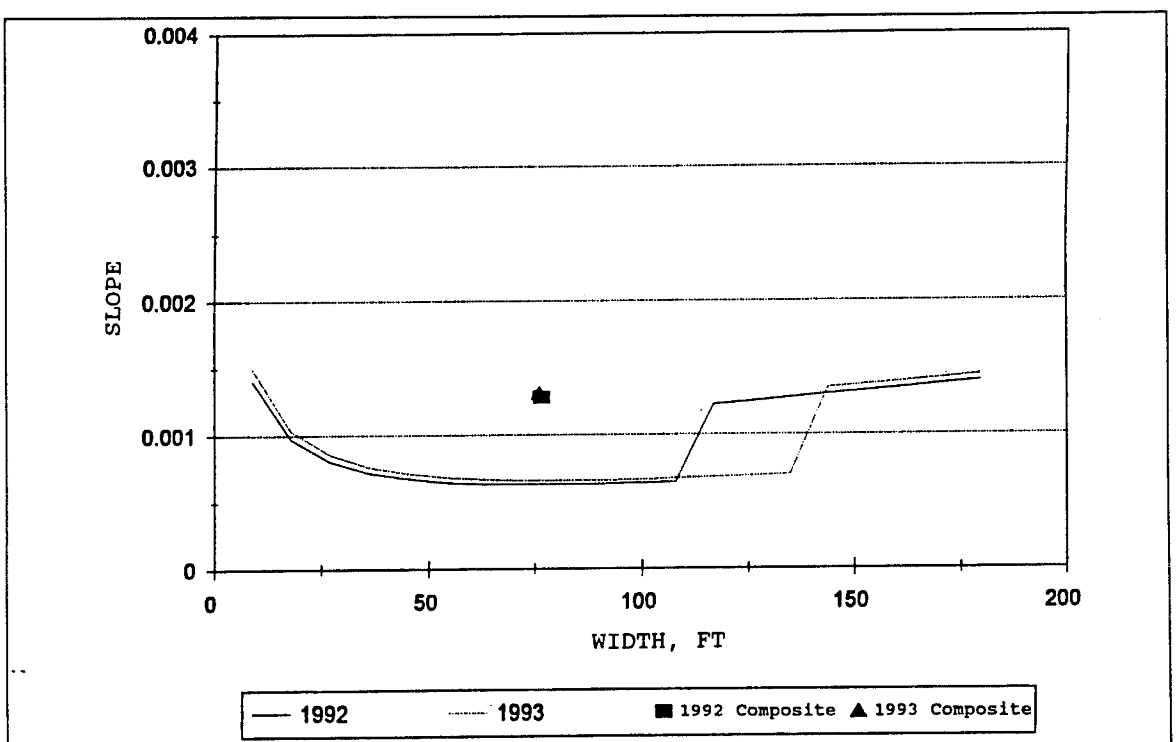



Figure 70. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for James Wolf Creek, Site 19, segment 2

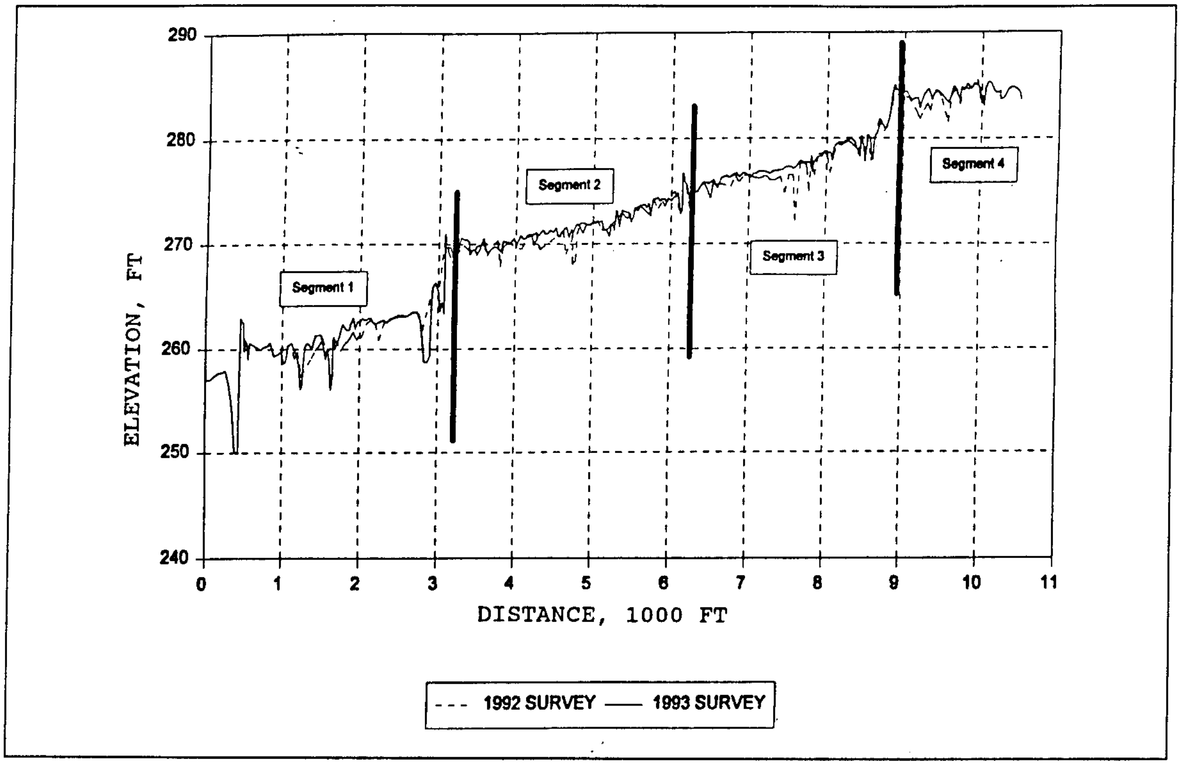



Figure 71. Comparison of 1992 and 1993 thalweg profiles for Long Creek

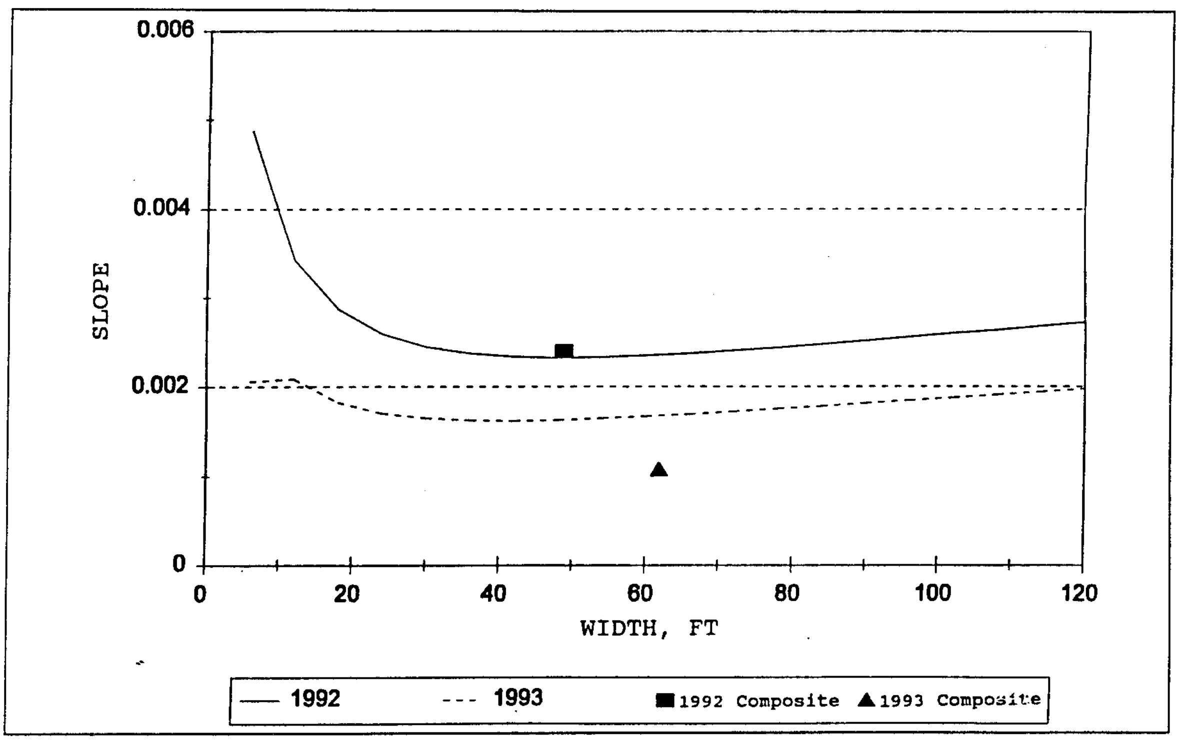



Figure 72. Comparison of 1992 and 1993 stable channel computations and composite cross sections for Long Creek, segment 1

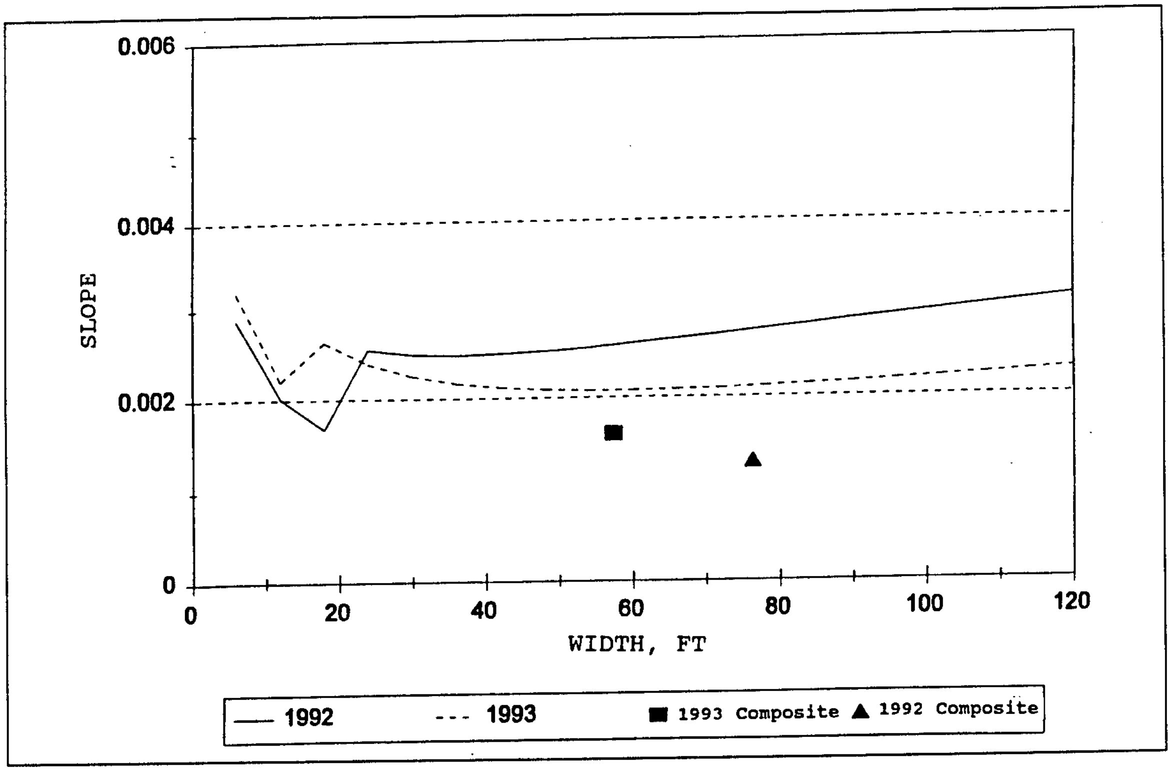



Figure 73. Comparison of 1992 and 1993 stable channel computations and composite cross sections for Long Creek, segment 2

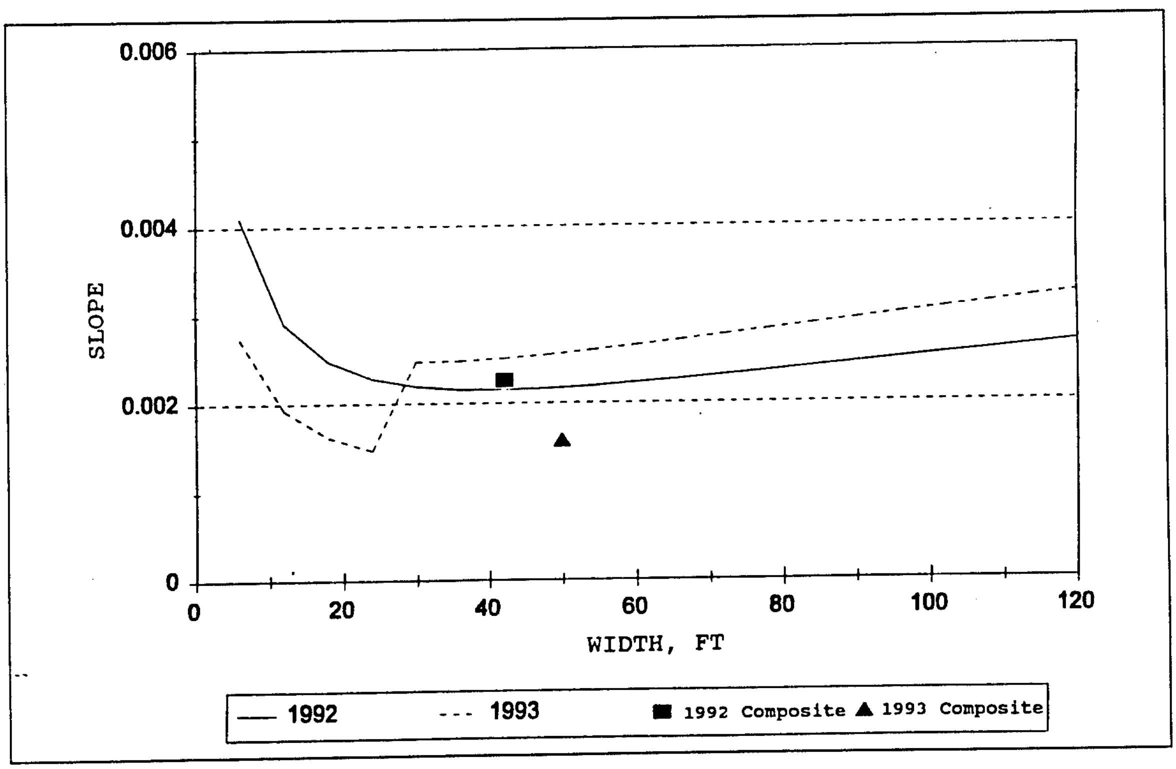



Figure 74. Comparison of 1992 and 1993 stable channel computations and composite cross sections for Long Creek, segment 3

Site 18. As shown in the table, the bed sediments are characterized as fine and medium sands, and the channels can contain the 2-year discharge. All of the channels are relatively straight and deeply incised with unstable banks along reaches of the channels.

Figure 75 is a comparison of the thalweg profiles for 1992 and 1993 on Worsham Creek beginning at the confluence of Middle Fork and continuing upstream on East Fork. Segment 1 begins at the confluence of Middle Fork and terminates at an oversteepened reach of erosion-resistant clay. Segment 2 terminates at the drop structure immediately downstream of the highway. Minor aggradation occurred between 1992 and 1993 for the lower two segments, and relatively little movement of the oversteepened reach occurred. Segment 3 includes the highway bridge and terminates at the newly constructed (1992) drop structure. The profile indicates that the reach is degrading. Segment 3 channel banks are eroding and the reach continues to widen. Segment 4 extends upstream of the new structure to the upstream structure on East Fork. A backwater from the new structure extends upstream of the Worsham Creek confluence; however, upstream of the backwater the resistant clay forms the bed up to the upstream East Fork structure, which is the steep portion of the profile shown in Figure 75.

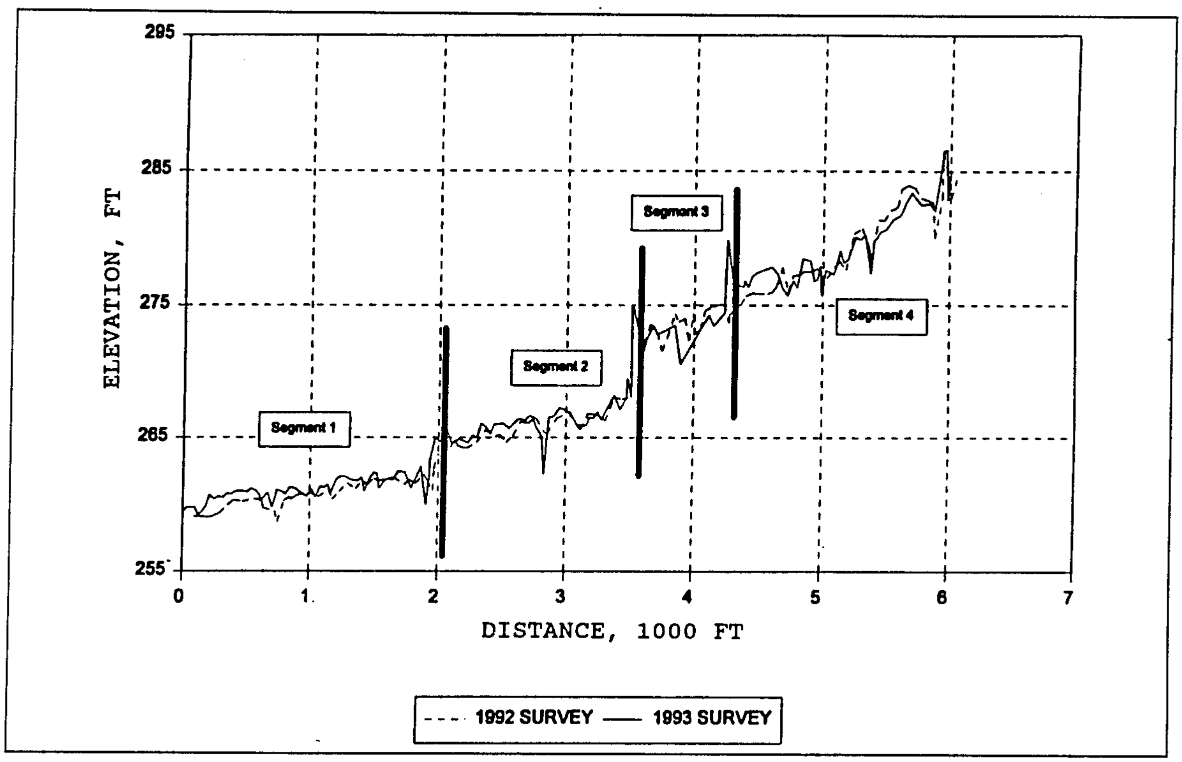



Figure 75. Comparison of 1992 and 1993 thalweg profiles for East Fork of Worsham Creek, Site 18

Figures 76 through 79 and Table 15 summarize the computed and composite data and analyses. Unfortunately, the 1992 survey data were not adequate upstream of segment 1 for analyses. The bed slope in segment 1 decreased significantly due to aggradation downstream. Beginning with segment 1 and moving upstream, the bed slopes increased to a maximum of 0.0042 in segment 4. The composite hydraulic energy gradient for each section was the greatest in segment 2, the result of the oversteepened reach downstream. Comparison of the sediment transport (tons/day) for the four segments indicates that segment 3 should be aggrading and that segment 2 should be degrading. The supply reach, segment 4, may have more transport capacity than is supplied to the reach, as supported by field evidence. This would make the downstream segments have a greater tendency to degrade.

Continuing monitoring will document any scour that may affect the grade control structures. Consideration should be given to placing a riprap toe with chevron weirs in segment 3 to limit channel widening and to enhance aggradation.

Figure 80 is a comparison of 1992 and 1993 thalweg profiles for Middle Fork of Worsham Creek. Each of the four segments is separated by a low-drop grade control structure. The 1992 survey of segment 3 was not obtained due to construction at the site during the field period. Comparison of the survey for the other three segments indicates that degradation was active during the period. As shown in Table 16, the bed slope decreased from 1992 to 1993, as expected during degradation. The energy gradient in segments 1 and 2 increased, probably due to channel widening, and energy gradients decreased in segments 3 and 4. Generally, the sediment transport rates, as shown in Table 16, are less than in East Fork. As in East Fork, Middle Fork composite channel width is approximately 50 percent of the regime width.

Figures 81 through 84 are comparisons of the stable channel computations and the composite cross section data for degments 1 through 4 on Middle Fork Worsham Creek. Segment 1 and 2 data indicate that the slope of the stable channel has increased from 1992 to 1993, and that the composite section is degradational compared with the stable channel computation. More emphasis could be placed on the data if better roughness information was available. The segment 3 data are in agreement; however, the upstream portion of the segment will probably widen due to the channel evolution sequence phase of the channel. The segment 4 stable channel computation curve and the composite section have both decreased in slope, although the data indicate that the segment will continue to degrade. At 287 tons/day/foot of width, a reduction in sediment yield is desirable.

The response of segments 1 and 2 may be the result of reduction in sediment supply from upstream. Additional grade control downstream and within the segments should be considered. Segment 3 is responding to recent construction. Segment 4 is producing significant sediment, and the county bridge upstream of the most upstream structure failed during the monitoring due to

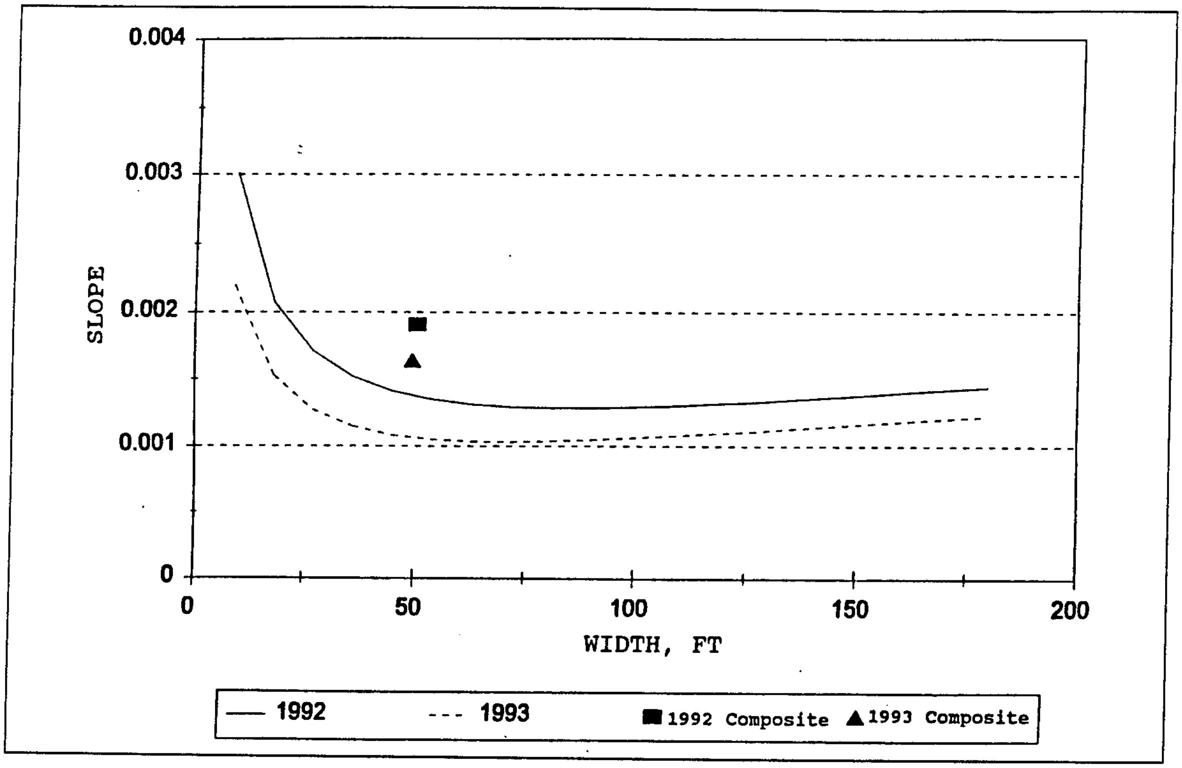



Figure 76. Comparison of stable channel computations and composite cross sections for East Worsham, segment 1

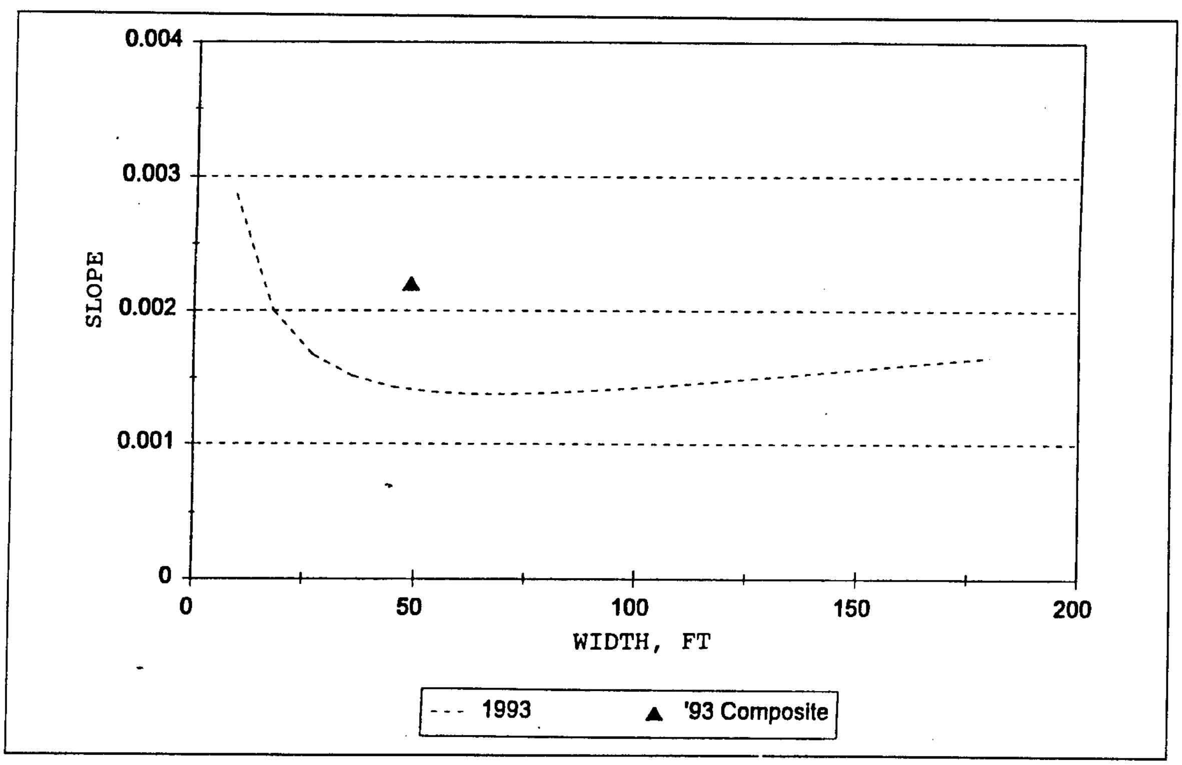



Figure 77. Comparison of stable channel computations and composite cross sections for East Worsham, segment 2

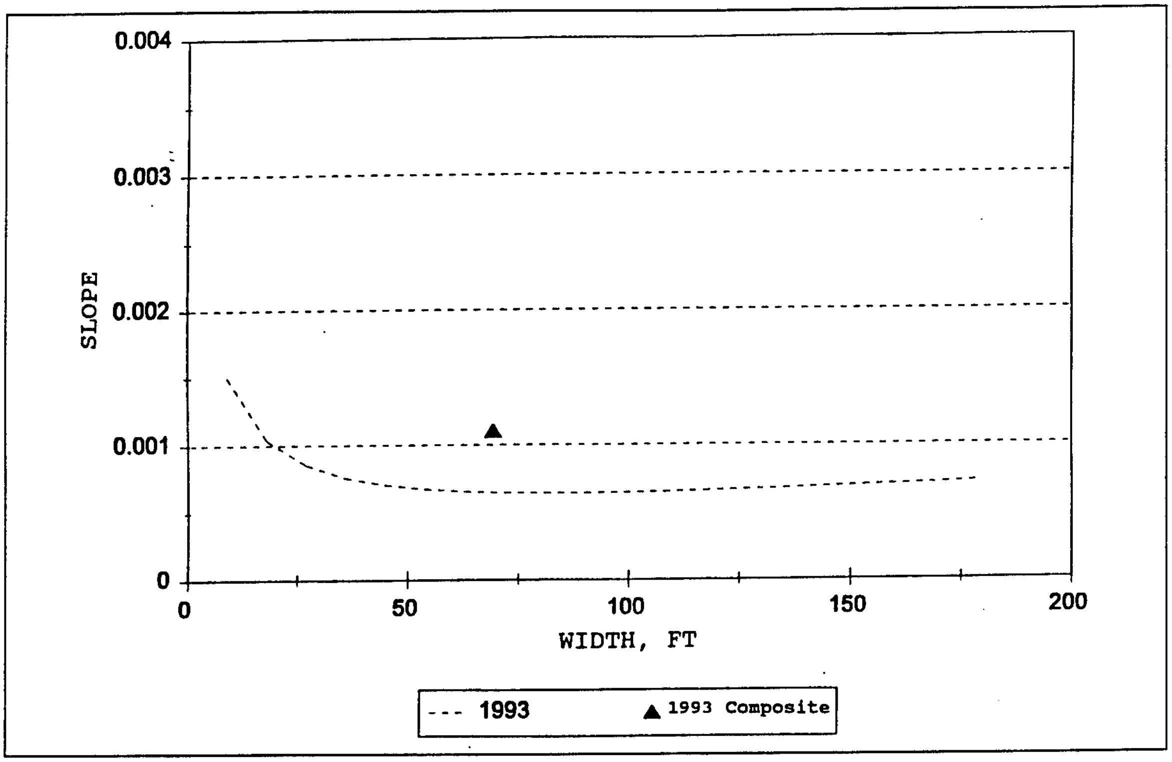



Figure 78. Comparison of stable channel computations and composite cross sections for East Worsham, segment 3

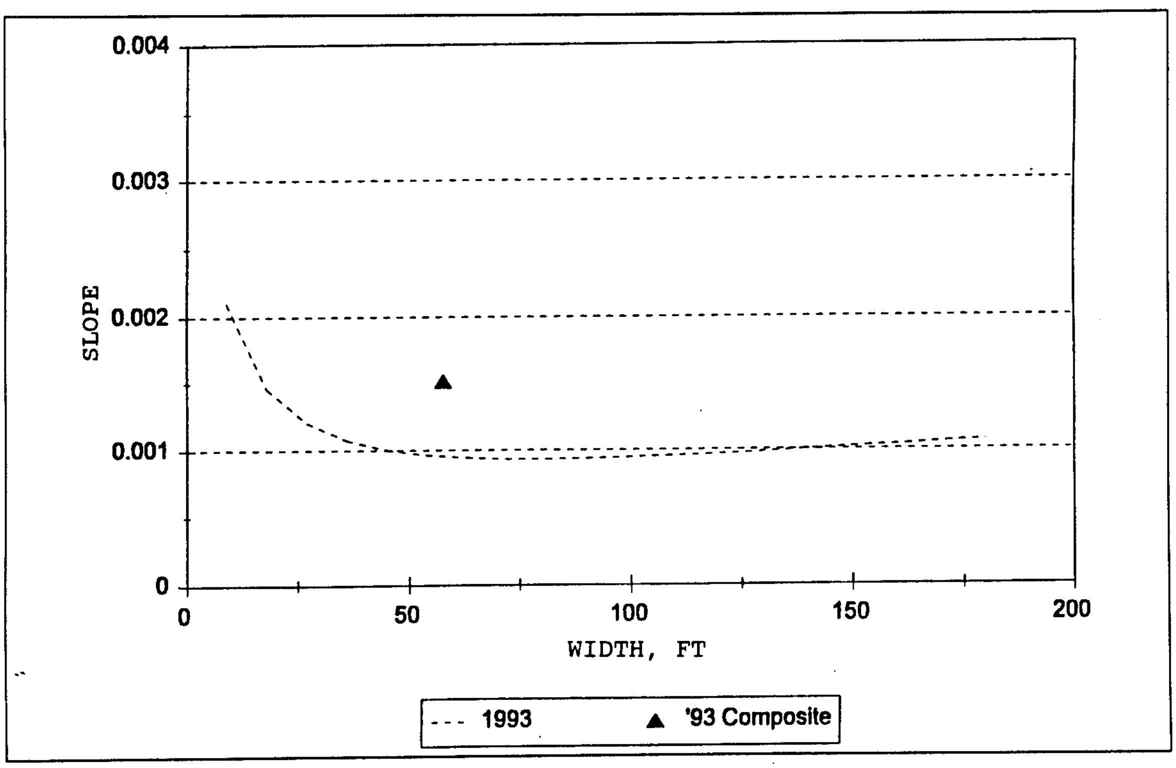



Figure 79. Comparison of stable channel computations and composite cross sections for East Worsham, segment 4

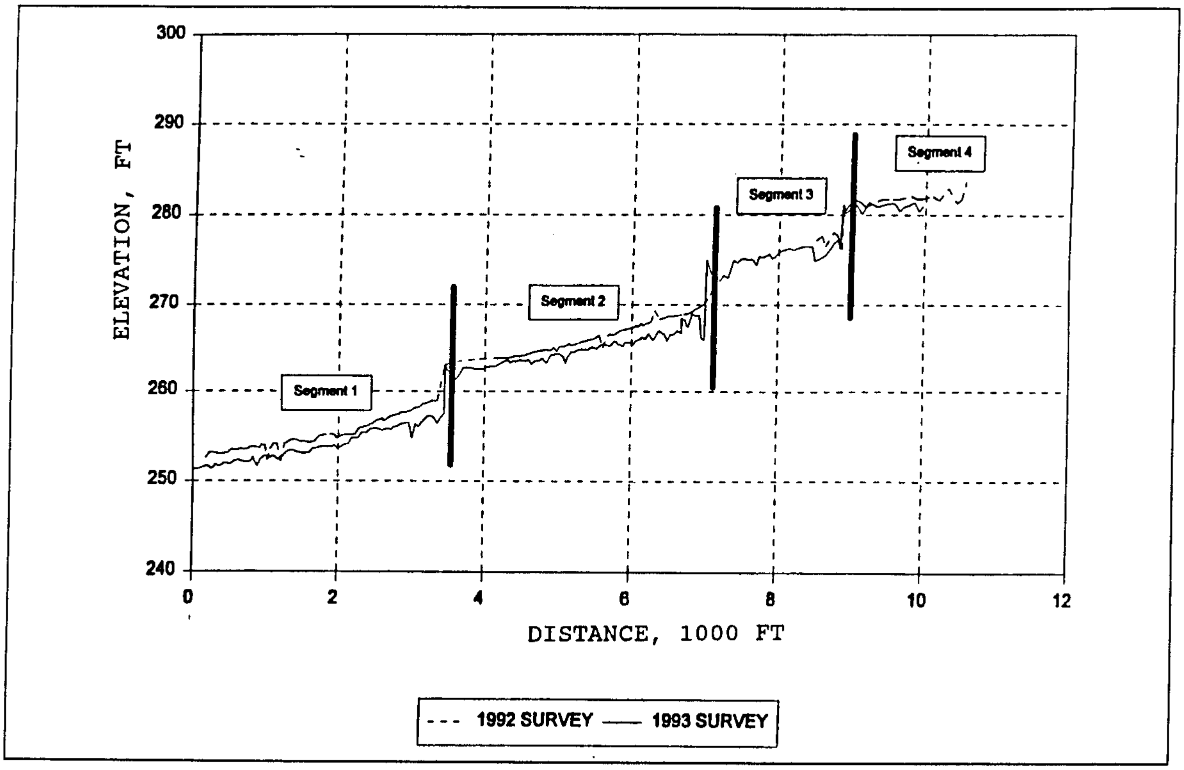



Figure 80. Comparison of 1992 and 1993 thalweg profiles for Middle Fork Worsham Creek, Site 18

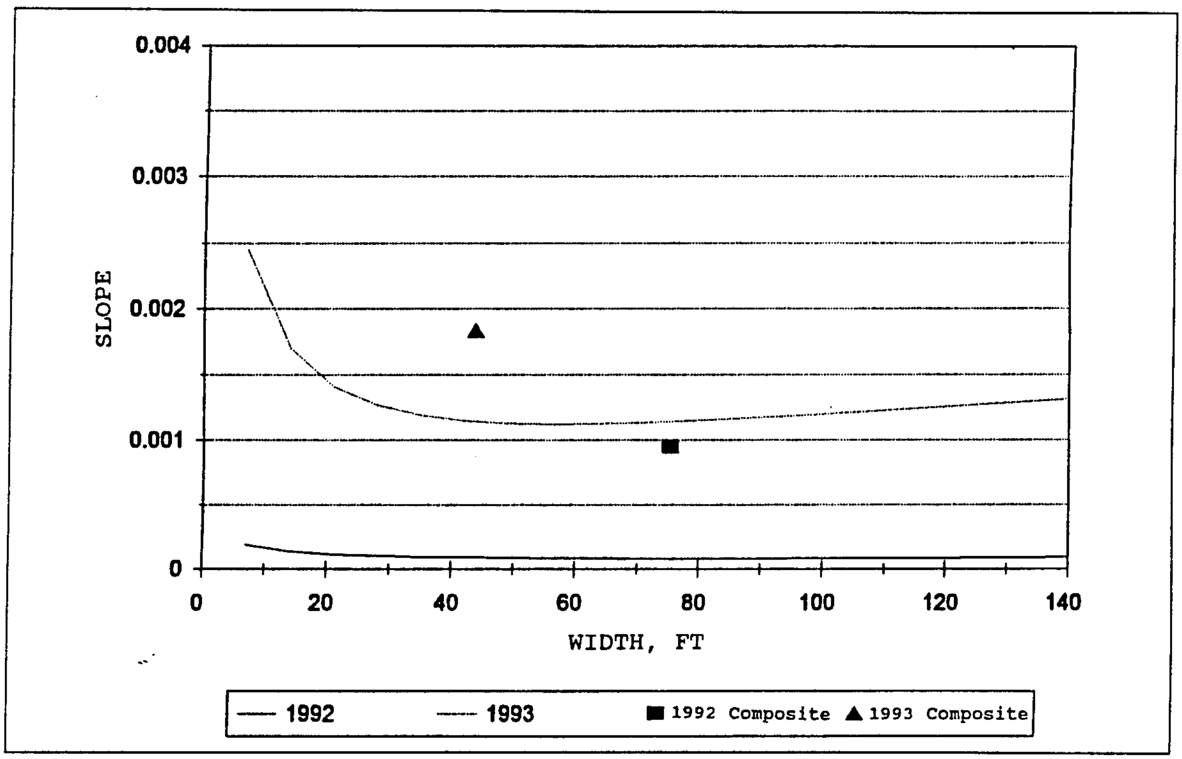



Figure 81. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for segment 1, Middle Fork Worsham

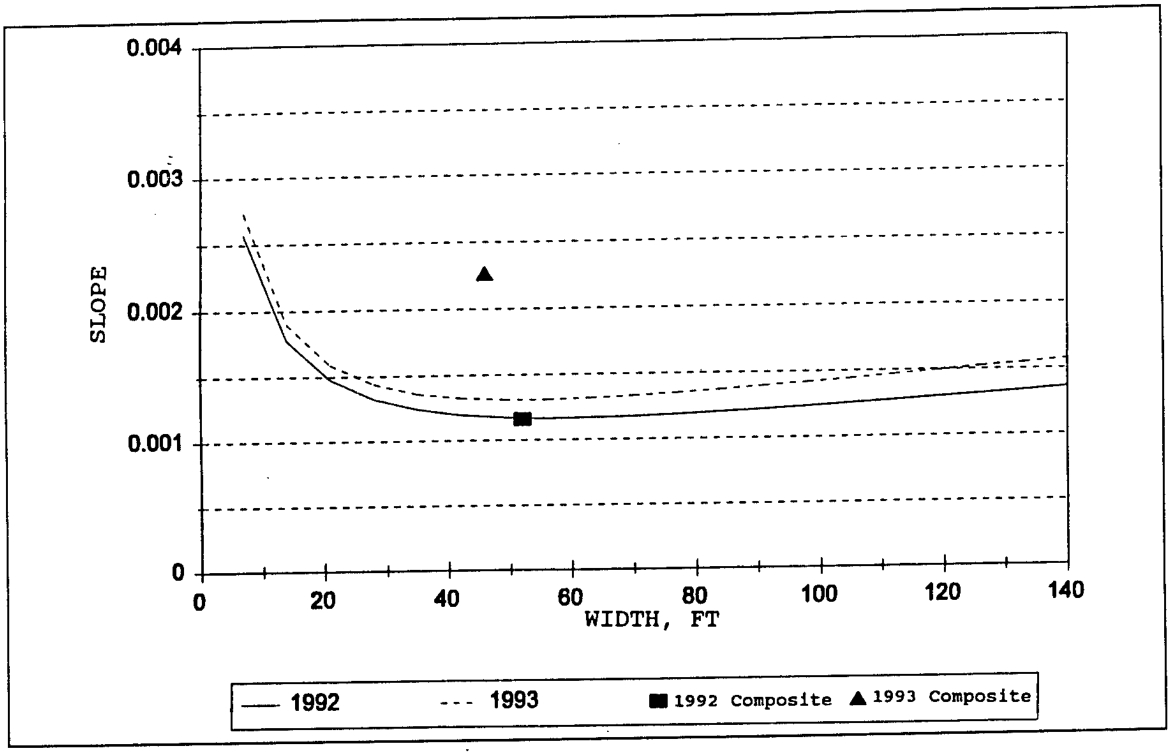



Figure 82. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for segment 2, Middle Fork Worsham

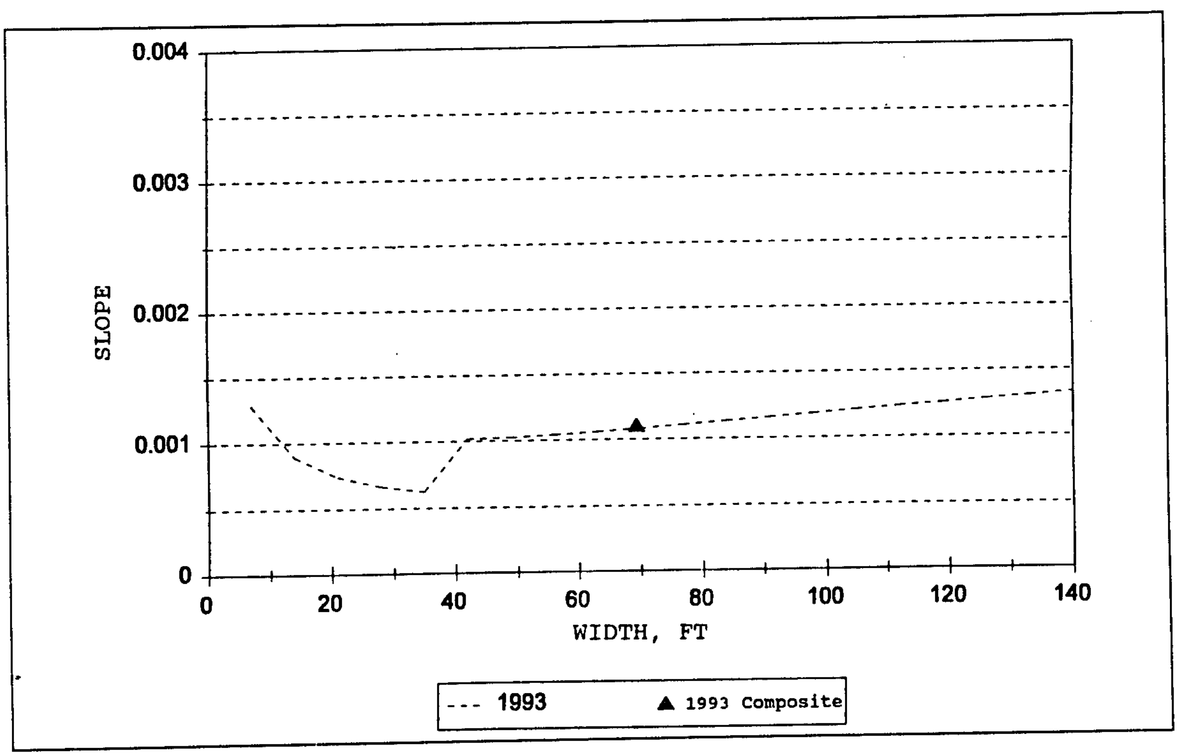



Figure 83. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for segment 3, Middle Fork Worsham

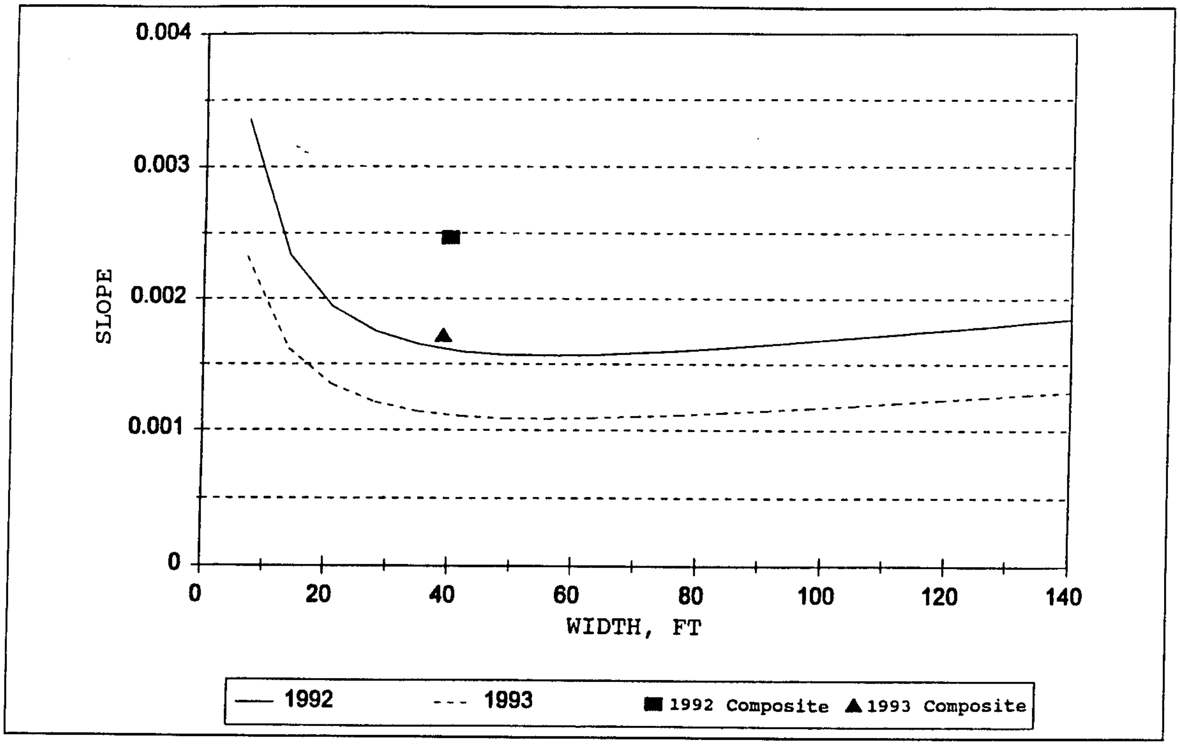



Figure 84. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for segment 4, Middle Fork Worsham

channel widening. Additional stabilization and grade control should be considered.

Figure 85 compares 1992 and 1993 thalweg profiles for West Fork Worsham Creek. Each of the four segments is separated by a lowdrop grade control structure. Relatively little change occurred in segments 1 and 4. Segment 3 aggraded in response to the downstream new grade control, while the upper portion of segment 2 degraded in response to the reduced sediment supply. Table 17 documents that the bed slopes decreased for each segment, and that the energy gradient decreased for segments 1, 2, and 3. The segment 4 energy gradient increased. This is confirmed in Figures 86 through 89, which compare stable channel computations and the composite cross section. For each of the first three segments the slope of the computed stable channel and the composite section decreased, although the position of the composite cross section implies that the channel will continue to degrade. Figure 89 indicates that the slopes are increasing and the creek is becoming more degradational.

Consideration should be given to placing several chevron weirs in segment 4 and continuing upstream. Additional grade control may be required downstream in segment 1.

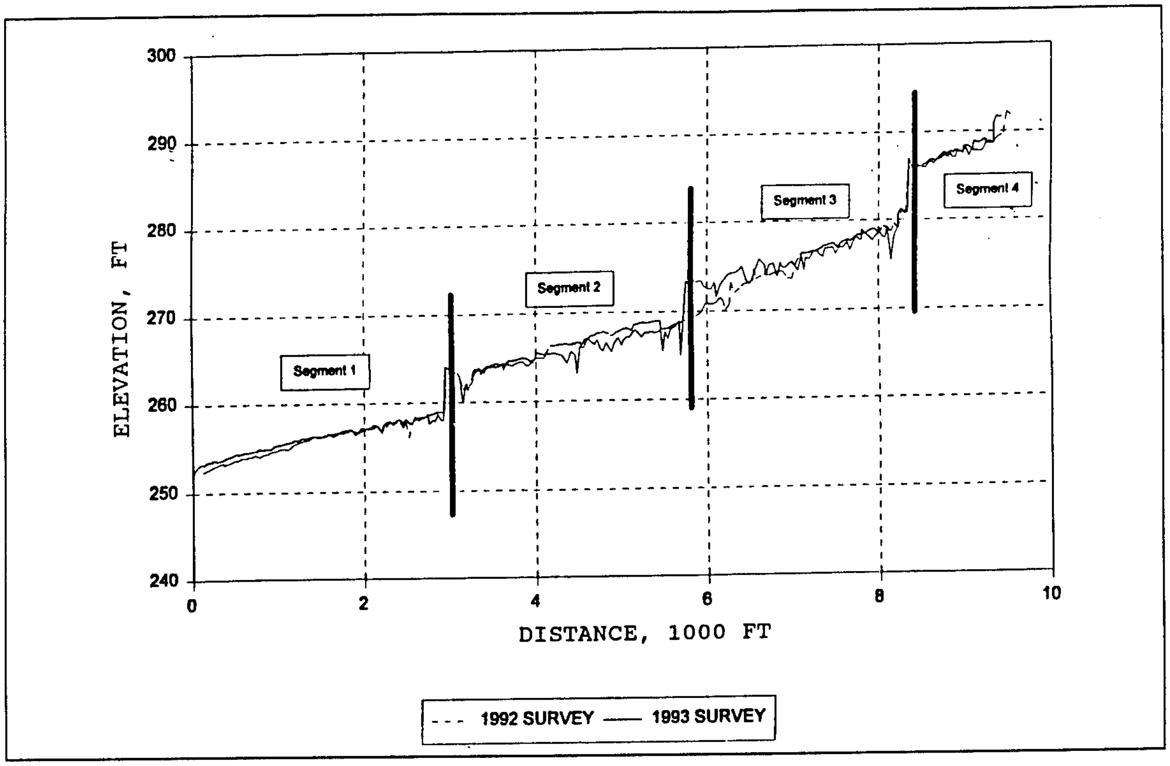



Figure 85. Comparison of 1992 and 1993 thalweg profiles for West Fork Worsham, Site 18

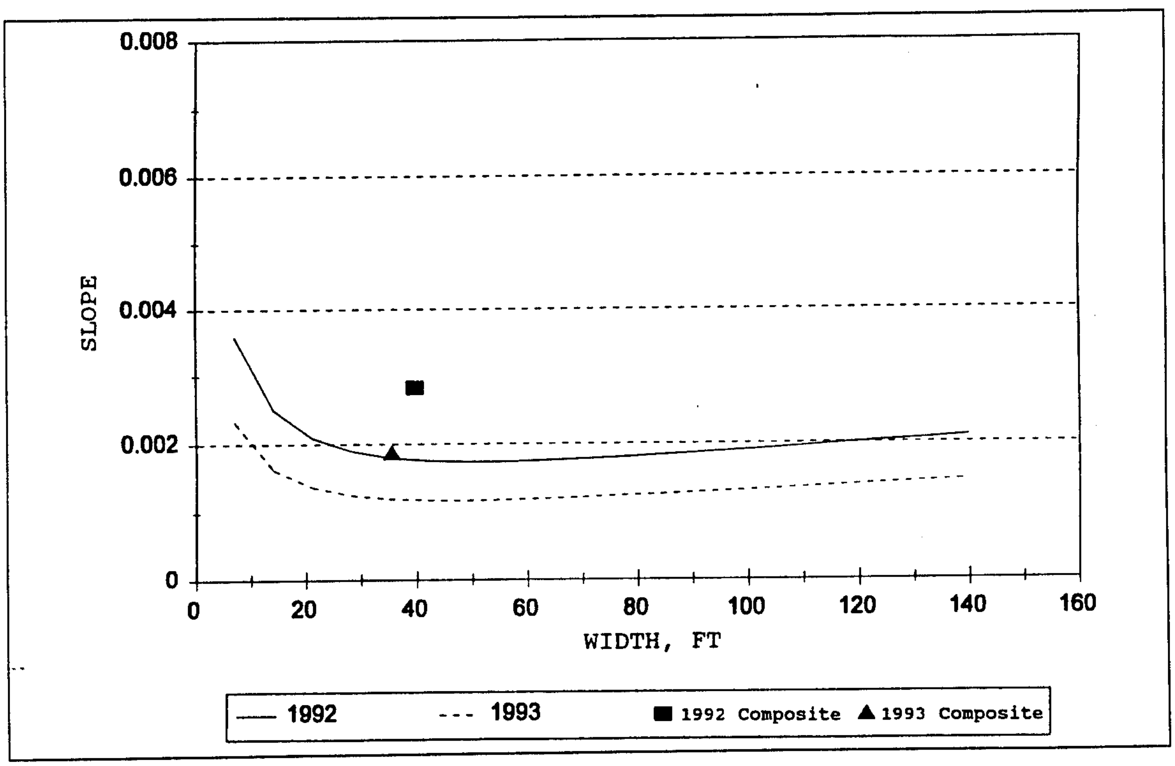



Figure 86. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for West Fork Worsham Creek, segment 1

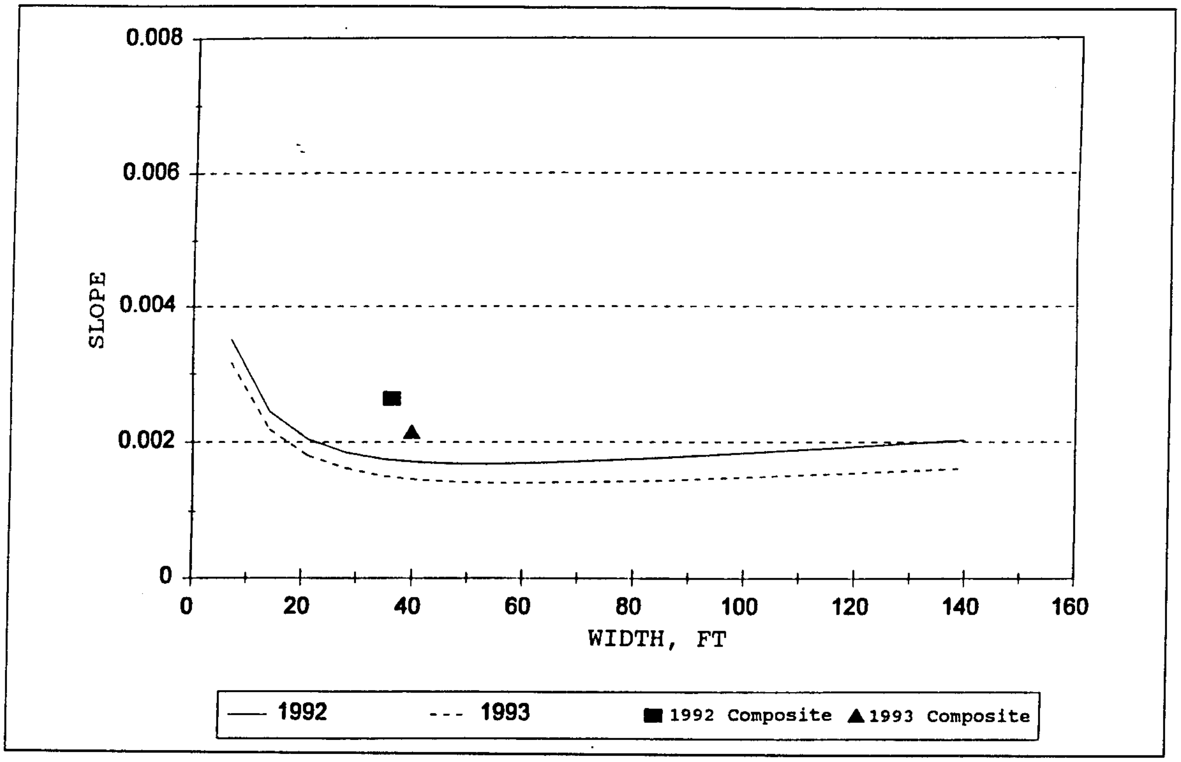



Figure 87. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for West Fork Worsham Creek, segment 2

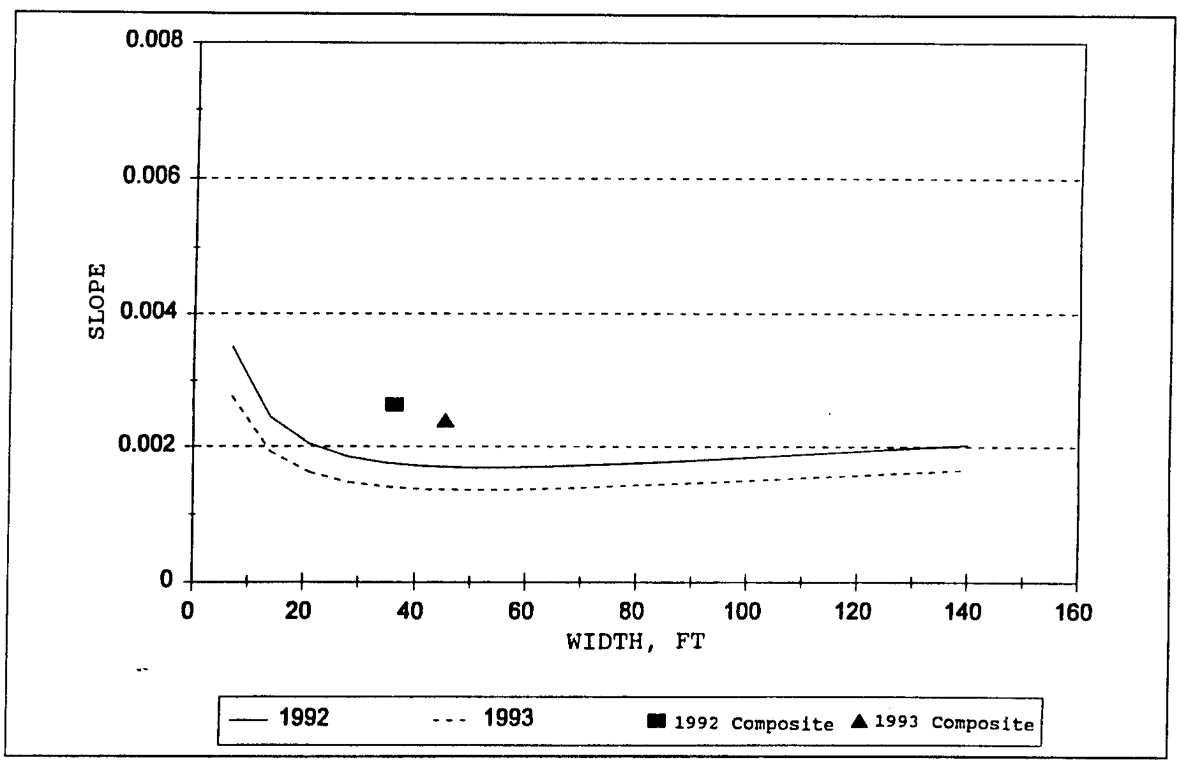



Figure 88. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for West Fork Worsham Creek, segment 3

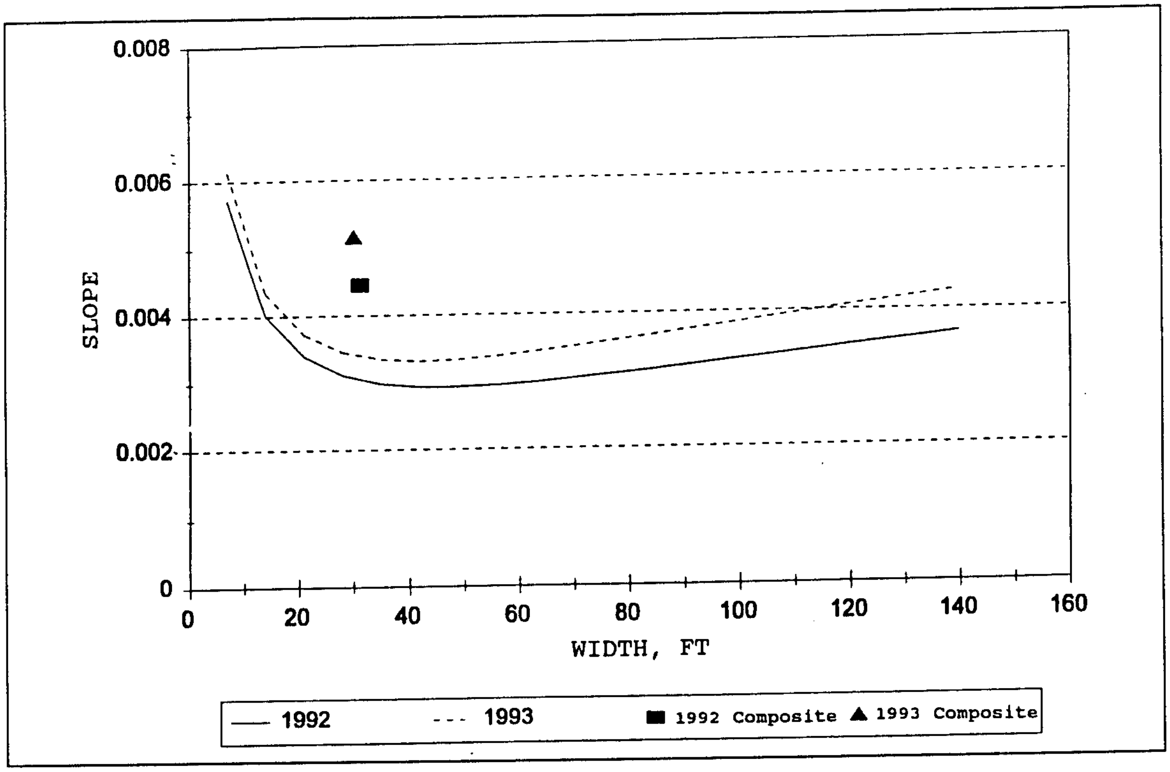



Figure 89. Comparison of 1992 and 1993 stable channel computations and composite cross-section data for West Fork Worsham Creek, segment 4

## Perry Creek, Site 16

In 1992, Perry Creek was an example of a single stream in which all phases of the channel evolution sequence were present from an evolved type 5 at the lower limit of the site, to a relatively undisturbed type 1 at the upstream extent. During 1992 and by the June 1993 surveys, two new grade control structures had been completed and two more were under construction. At the upstream extent of the site, the drainage area is 8.1 square miles and the 2-year discharge is 1,790 cfs. The bed material is characterized as medium sand ($D_{50}$ = 0.32 mm).

Figure 90 compares thalwegs for Perry Creek. Segment 1 begins downstream of the first newly constructed drop structure, and segment 2 begins at the first new structure. Segment 3 begins at the second structure, and segment 4 begins at the third structure and encompasses the fourth structure, which was under construction at the time of the survey. Comparison of the surveys indicates aggradation in segments 2 and 3, and minor degradation in segment 1. The change in segment 4 is due to continuing construction.

Table 18 summarizes the data and analyses for the site. Cross-section surveys were not available in 1992 due to survey error. Figures 91 through 94 compare composite cross-section data with stable channel computations. Sediment transport is very low for segments 1 and 2, and the energy gradient for segment 1 is less than regime. Channel response in segment 3 may be to

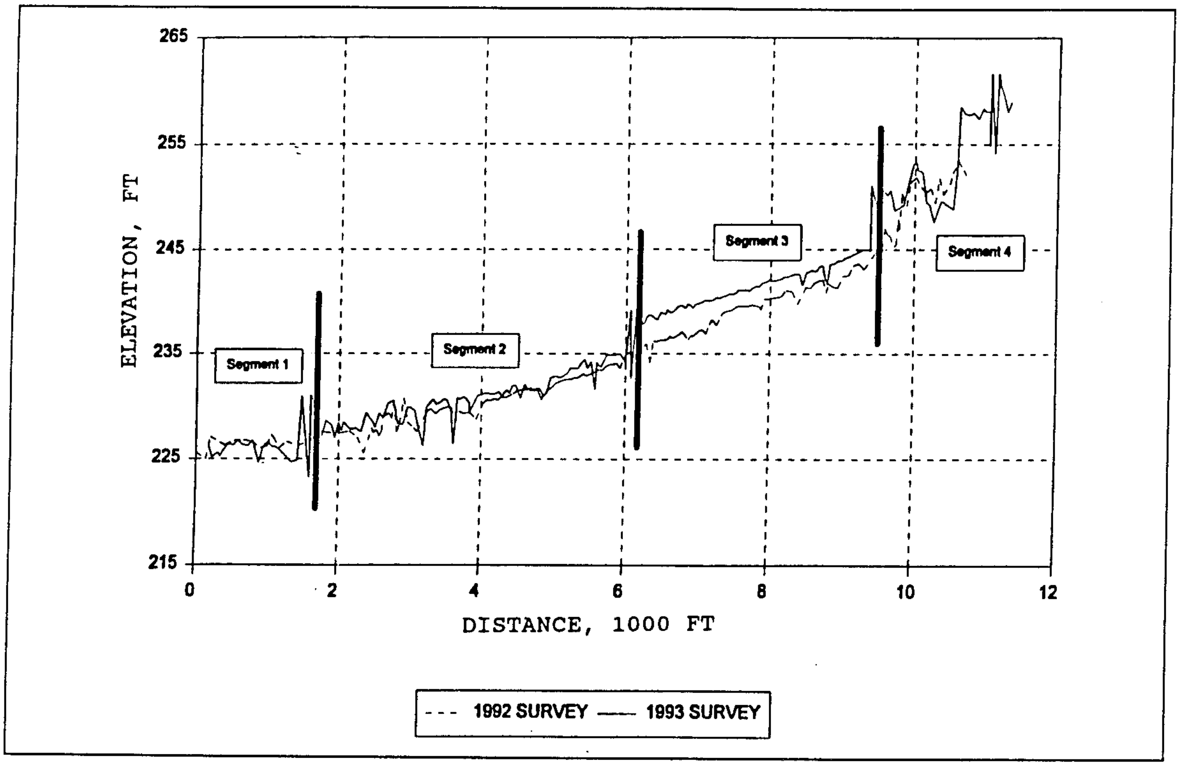



Figure 90. Comparison of 1992 and 1993 thalweg profiles for Perry Creek

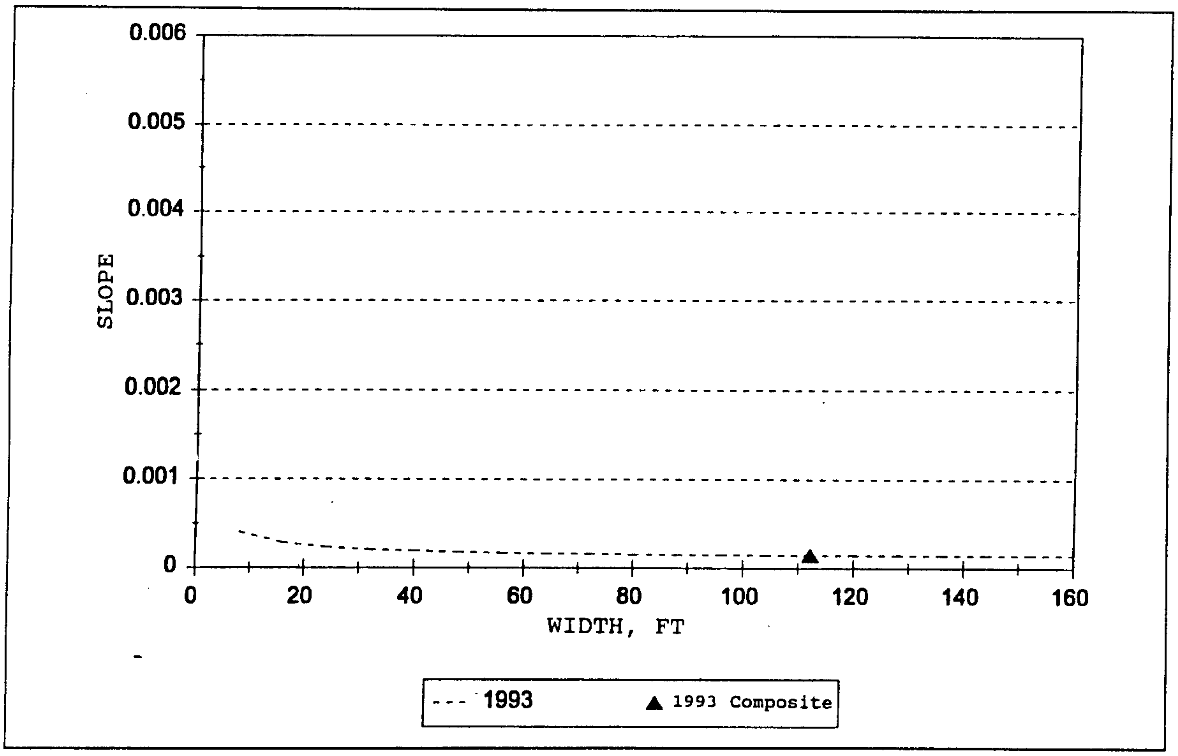



Figure 91. Comparison of 1993 stable channel computations and composite cross-section data for Perry Creek, segment 1

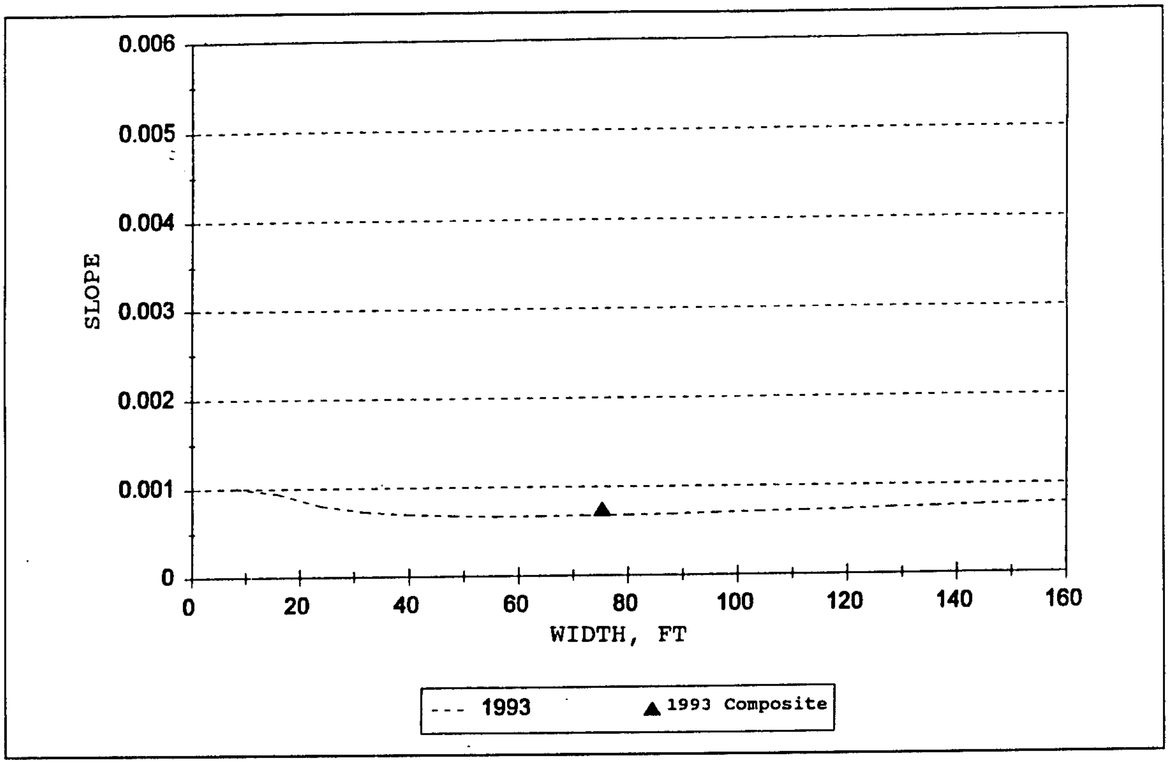



Figure 92. Comparison of 1993 stable channel computations and composite cross-section data for Perry Creek, segment 2

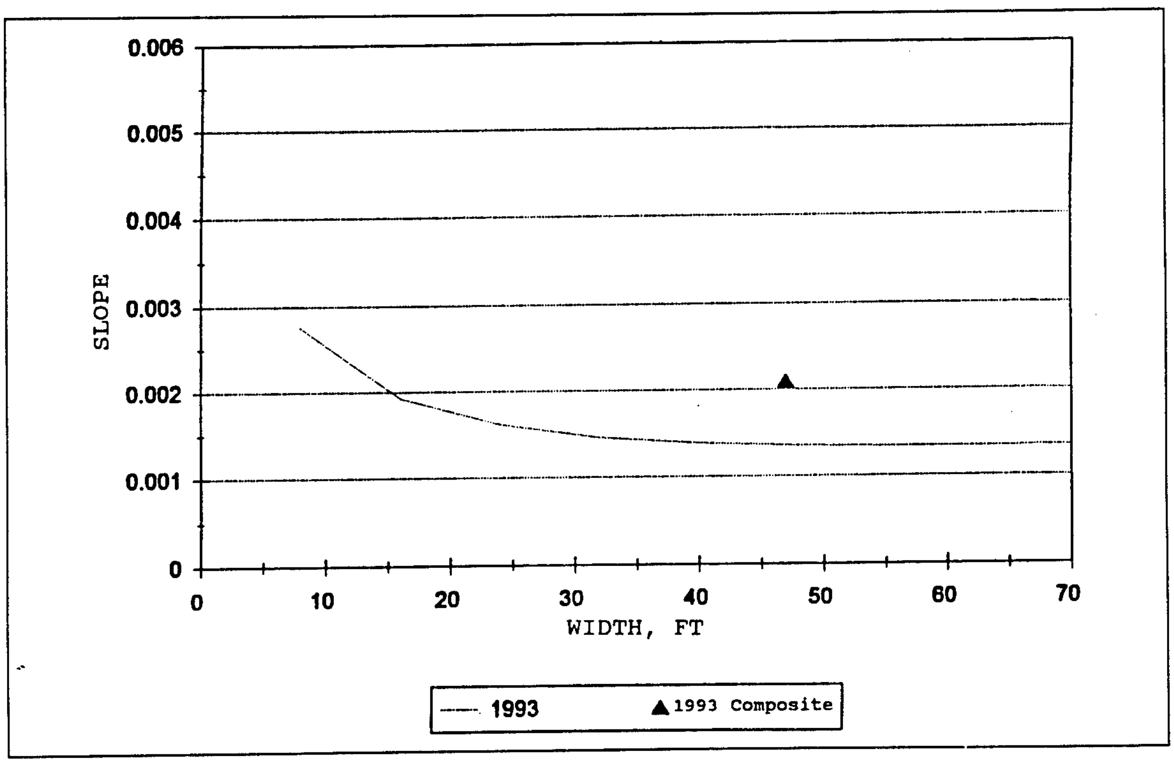



Figure 93. Comparison of 1993 stable channel computations and composite cross-section data for Perry Creek, segment 3

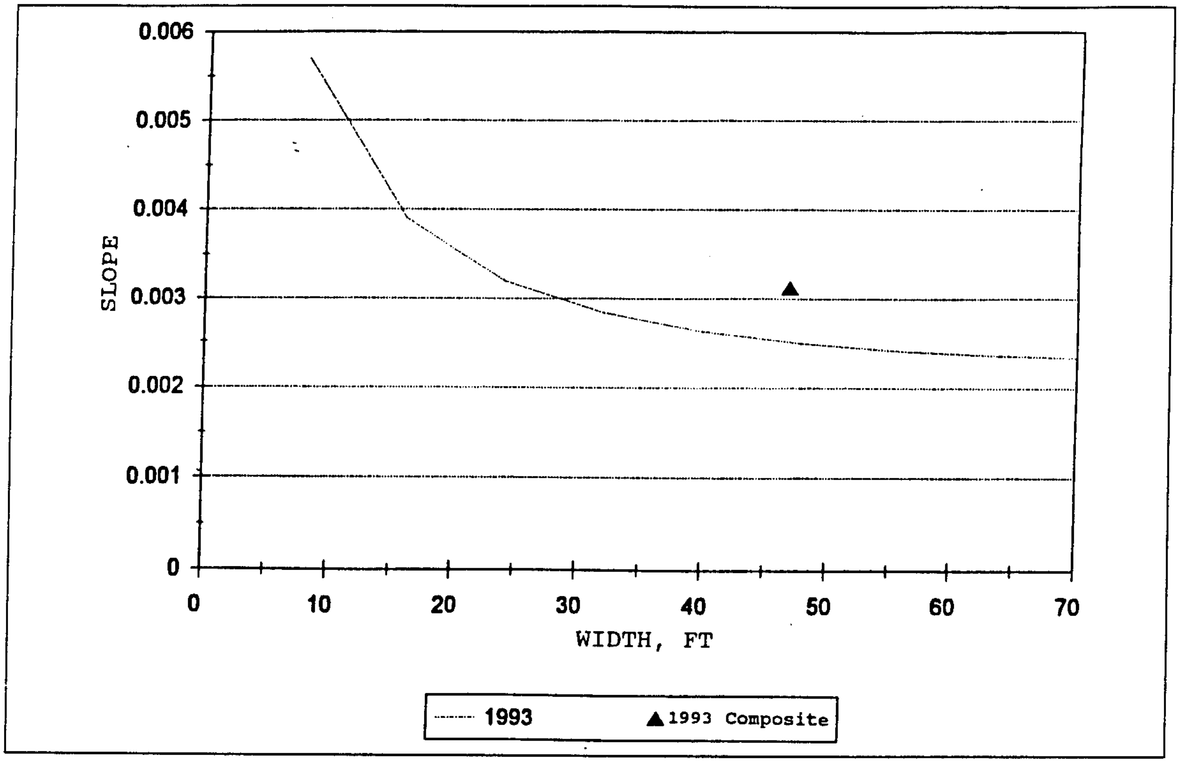



Figure 94. Comparison of 1993 stable channel computations and composite cross-section data for Perry Creek, segment 4

widen and result in a lower composite energy gradient. However, a portion of segment 4 bed material prior to construction was large (4- to 6-in.) ironstone plates, which may be affecting the bed and energy gradient. Segment 4 transport is extremely high and is not realistic due to the ongoing construction. Four drop structures constructed in series with the improvements in structure and with the improvements in system configuration to affect hydraulic control should bring rapid and significant change to Perry Creek.

## Otoucalofa Creek, Site 14

Otoucalofa Creek, Site 14, extends approximately 2,000 ft upstream and downstream of the Mt. Liberty Church Road bridge. The channel meanders downstream of the bridge and is relatively straight upstream. Most of the study reach length has been treated using toe riprap or short riprap dike bank stabilization. The watershed area is approximately 41.1 square miles with a 2-year discharge of 4,617 cfs. The bed material is characterized as medium sand ($D_{50}$ = 0.40 mm). Figure 95 compares 1992 and 1993 thalweg profiles, which shows that the stream is continuing to degrade, particularly for the lower 2,500 ft. The upper 1,500 ft has a bed of erosion-resistant fine-grained clay, and upstream of the site the bed is a steep knickzone.

Table 19 summarizes data and analyses for Otoucalofa Creek. The data indicate that the bed slope and the energy gradient are the same, and the

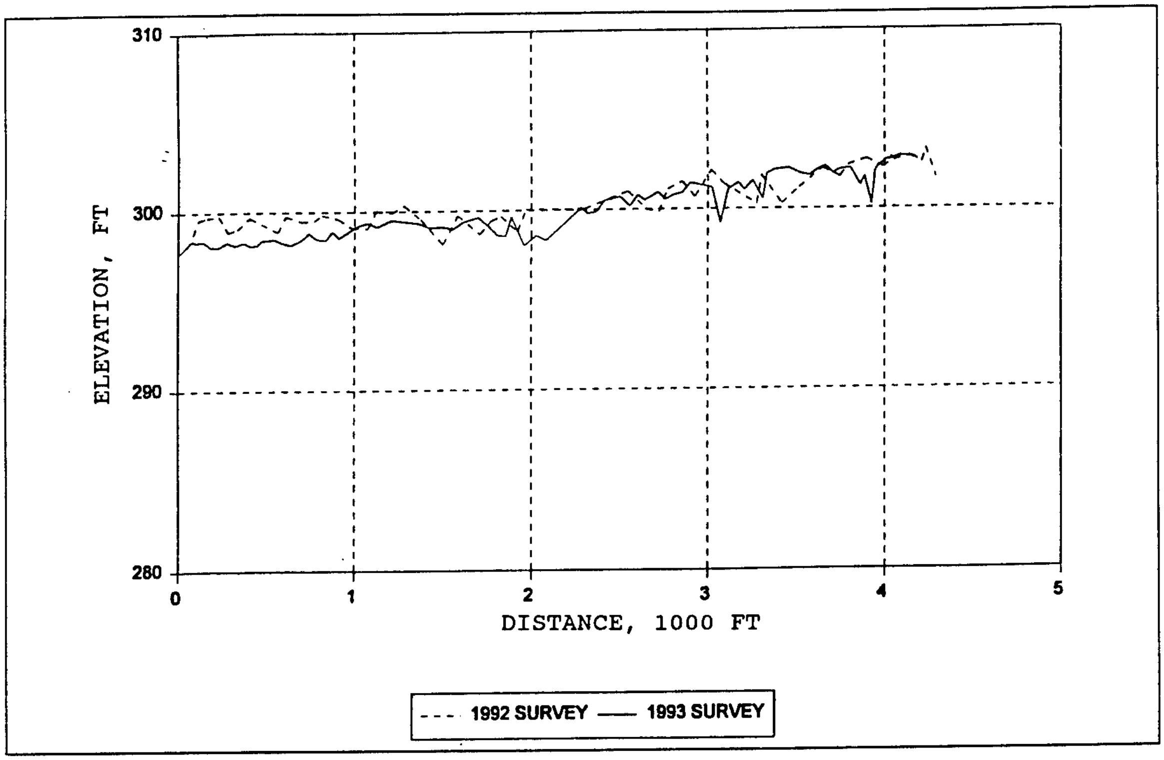



Figure 95. Comparison of 1992 and 1993 thalweg profiles for Otoucalofa Creek, Site 14

composite width is approaching the regime width. Sediment transport capacity has increased by 150 percent. Figure 96 compares the 1992 and 1993 stable channel computations and the composite section data, which indicate that the slope of the composite section has increased and that at the existing width the channel shifts from upper regime to lower regime. If sediment supply is to be reduced, the energy gradient must be reduced to establish stability. Consideration should be given to construction of a grade control structure or structures beginning downstream of the bridge to establish a lower energy gradient. Field evidence suggests that the channel will overbank at the existing condition and the grade control structures will decrease flood-control capacity.

## Sykes Creek, Site 17

Sykes Creek, Site 17, is a meandering, medium sand-bed ($D_{50}$ = 0.34 mm) stream with a drainage area of 12.2 square miles. The 2-year discharge is 2,542 cfs, about 50 percent of the bank-full discharge. The site was originally chosen because no work was anticipated on the reach and initial inspection indicated that the channel was relatively stable. Field inspection indicated that permeable dikes and fencing had been used earlier in an effort to stabilize eroding bends. All of these features have been flanked or truncated, and bend erosion is continuing. Figure 97 compares the thalweg profiles, which show that the upper portion of the site has degraded 1 to 2 ft during the 1992-1993 period. Table 20 summarizes data and analyses, which

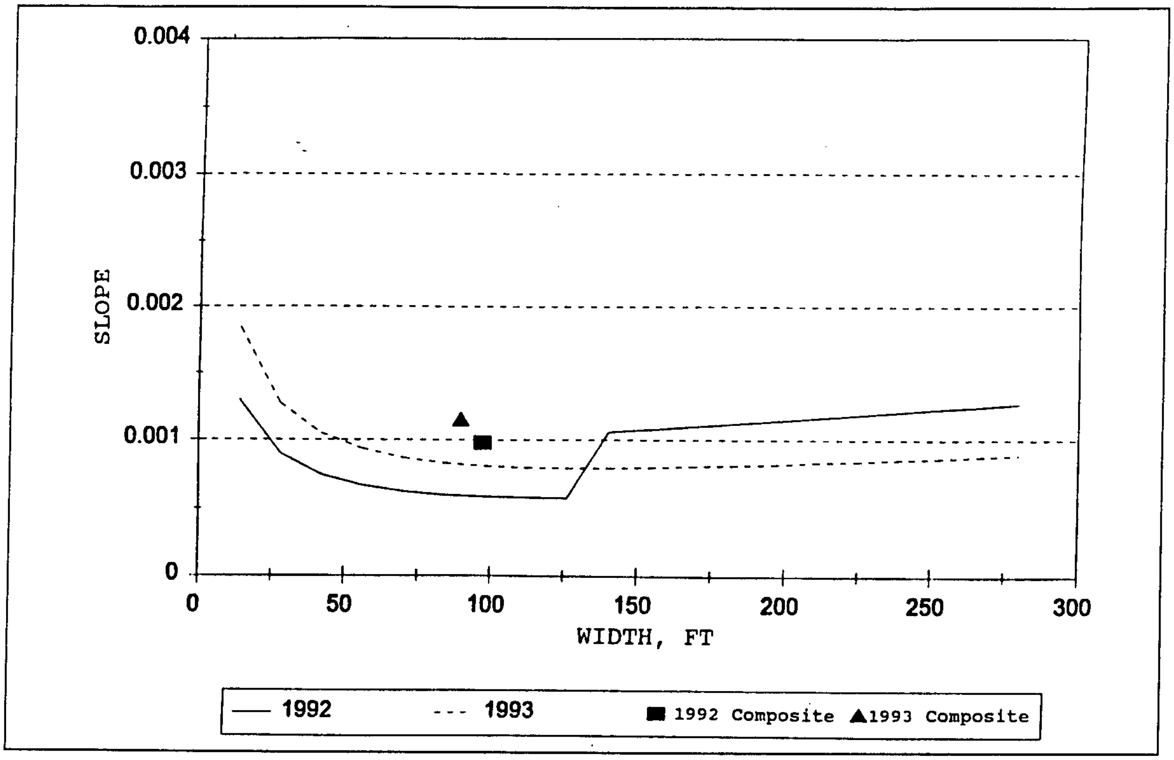



Figure 96. Comparison of 1992 and 1993 stable channel computations and the composite cross-section data for Otoucalofa Creek, Site 14

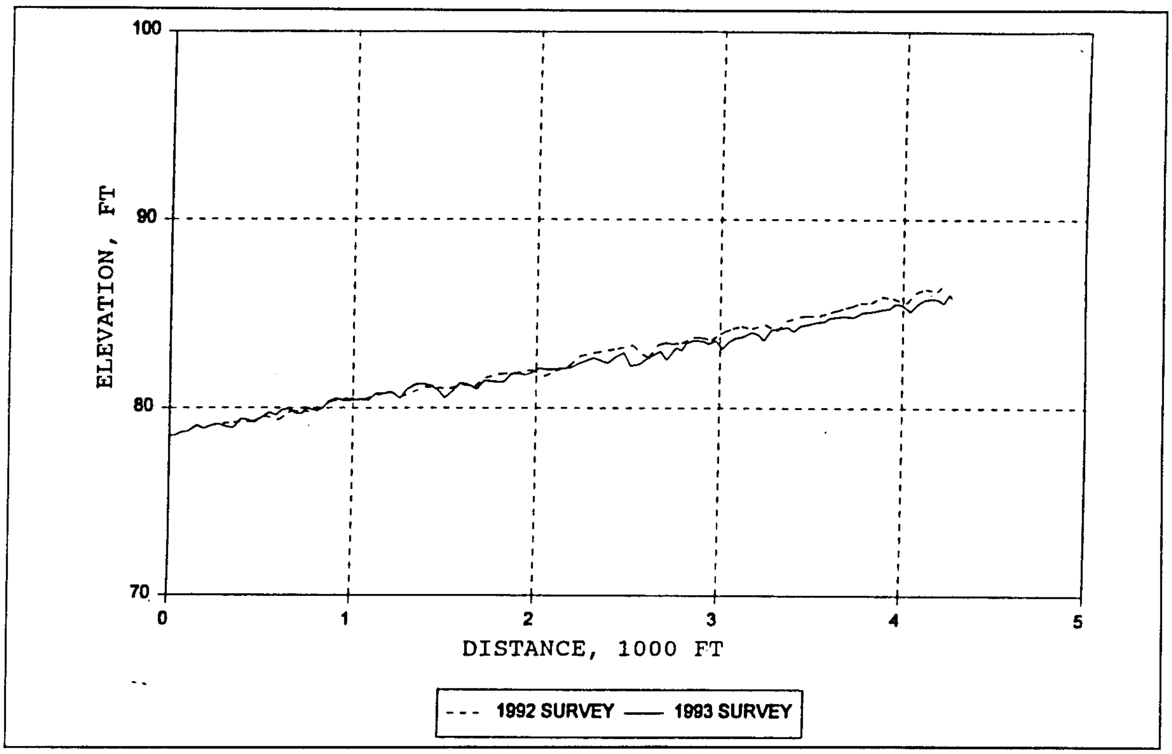



Figure 97. Comparison of 1992 and 1993 thalweg profiles for Sykes Creek, Site 17

indicate that although the bed has flattened, the composite energy gradient has increased, resulting in a 25 percent increase in sediment transport capacity. Figure 98 compares the stable channel computations and the composite cross-section data, which indicate that the site will continue to be degradational.

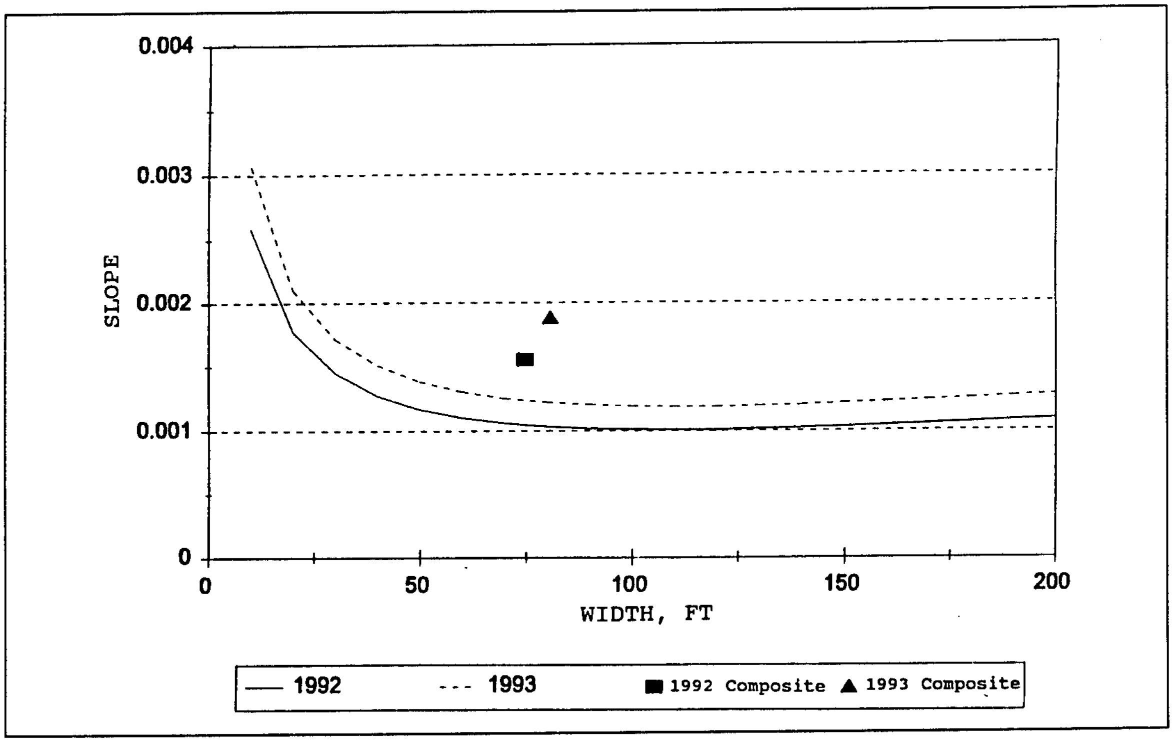



Figure 98. Comparison of 1992 and 1993 stable channel computations and the composite cross-section data for Sykes Creek, Site 17

Erosion in the study reach was observed only at the outer bends, the loose sand depth was greater than 3 ft, and no headcutting or knickzones were observed. However, the apparent stability may be due to the high sediment supply, and efforts should be made to reduce the supply of sediment. The WES thalweg profile of Sykes (Waller and Hubbard 1993) indicates degradation continues up to Highway I-55. Consideration of bendway stabilization, perhaps using bioengineering, and grade control using chevron weirs is suggested.

## Fannegusha Creek, Site 2

Fannegusha Creek, Site 2, is in the Black Creek-Fannegusha Creek watershed and has a drainage area of approximately 18 square miles, with a 2-year discharge of 3,325 cfs. The bed material is medium sand ($D_{50}$ = 0.31 mm), and the depth of loose bed material varies from zero to greater than 3 ft within the site. Figure 99 compares the 1992 and 1993 thalweg profiles, which indicate a very unstable, complex morphology. The upstream portion of the site is very steep, and the bed is formed by a series of hard,

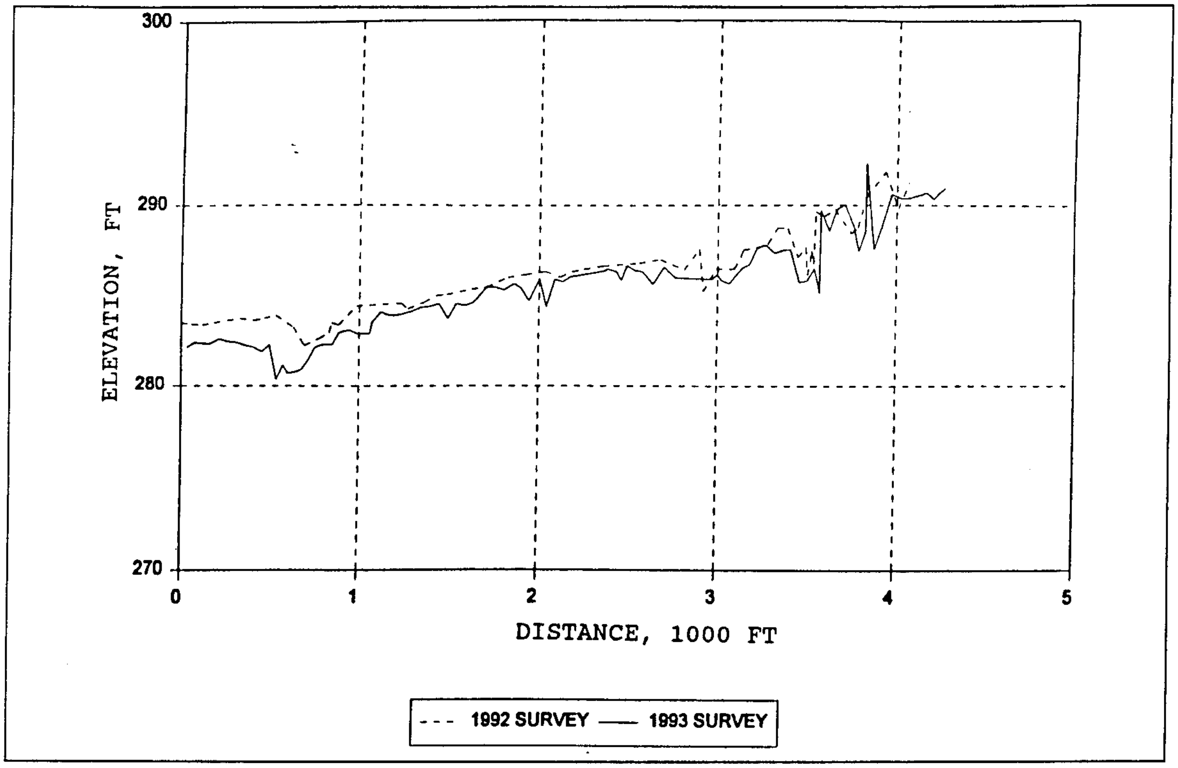



Figure 99. Comparison of 1992 and 1993 thalweg profiles for Fannegusha Creek, Site 2

fine-grained-material headcuts, which have moved little during the year. The remainder of the reach has degraded. A convexity was formed as the sand deposited upstream of debris collected upstream of a county road bridge, and by outcrops of resistant material downstream. The extreme lower portion of the site is a U-shaped channel with a center incision flowing in resistant material. Table 21 includes data that indicate that the channel is decreasing in sediment transport capacity and is widening. Figure 100 compares the stable channel computations and the composite cross-section data. As shown, the channel remains degradational. The reach is very unstable and grade control should be strongly considered. Presently, the county road bridge is unsafe.

## Lick Creek, Site 8

Lick Creek, Site 8, was originally selected because a high-drop structure is planned for the site. The structure has not been constructed, and a temporary riprap fill has been emplaced. The site extends approximately 2,000 ft upstream and downstream of the highway bridge. Downstream the channel is deeply incised and has widened. Upstream the channel is relatively narrow and the channel is headcutting at the extreme upstream portion of the site. An emerging wetland area is located along the upstream left bank, which at the time of the 1993 field inspection was flooded at a shallow depth. Continued channel incision and gully formation along the channel may drain the emerging wetland area.

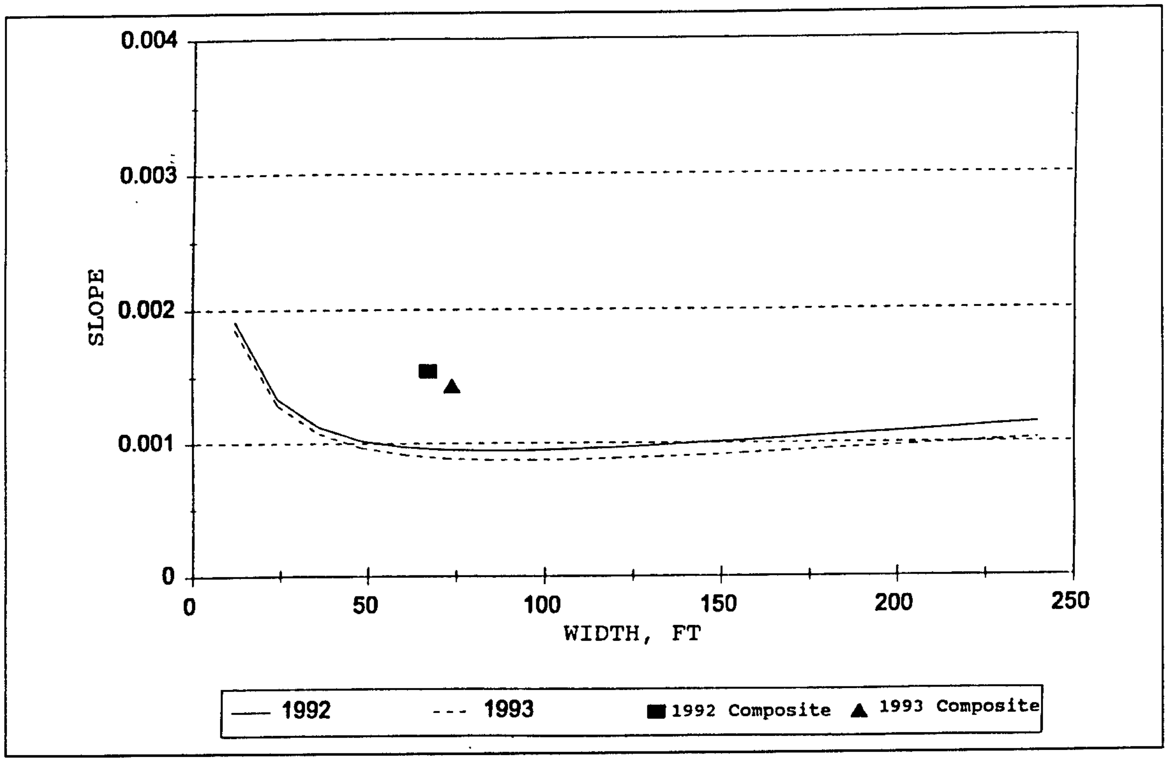



Figure 100. Comparison of 1992 and 1993 stable channel computations and the composite cross-section data for Fannegusha Creek, Site 2

Figure 101 compares the 1992 and 1993 thalweg profiles, which show continuing degradation. The bed slopes range from 0.0025 to 0.003. However, the composite cross-section energy gradient indicates that the gradient is decreasing and that sediment transport is decreasing (Table 22). This is also supported by Figures 102 and 103, which indicate lower stable channel slopes for the upstream segment 2. Unfortunately, segment 1 remains degradational, and the degradation can be expected to move upstream. These data suggest that the temporary riprap fill may be resisting the channel incision. Construction of the planned high-drop and review of gully formation in the upstream flooded reach are recommended.

### Sediment yield of Hotophia Creek

A special study was conducted to determine the sediment yield at the mouth of Hotophia Creek. No DEC monitoring site exists at this location; however, the Vicksburg District was particularly interested in the performance of the watershed. Survey data were available for 1977, 1985, and 1992, and a USGS gauging station was established at Highway 315 near the mouth of Hotophia Creek. Figure 104 compares the survey data, which show the degradation that has occurred since 1977. Analysis of this data set provided the opportunity to establish a flow-duration relationship, compute a sediment rating curve for a series of discharges for the different channel surveys, compute a sediment rating curve for a series of discharges relative to a series of

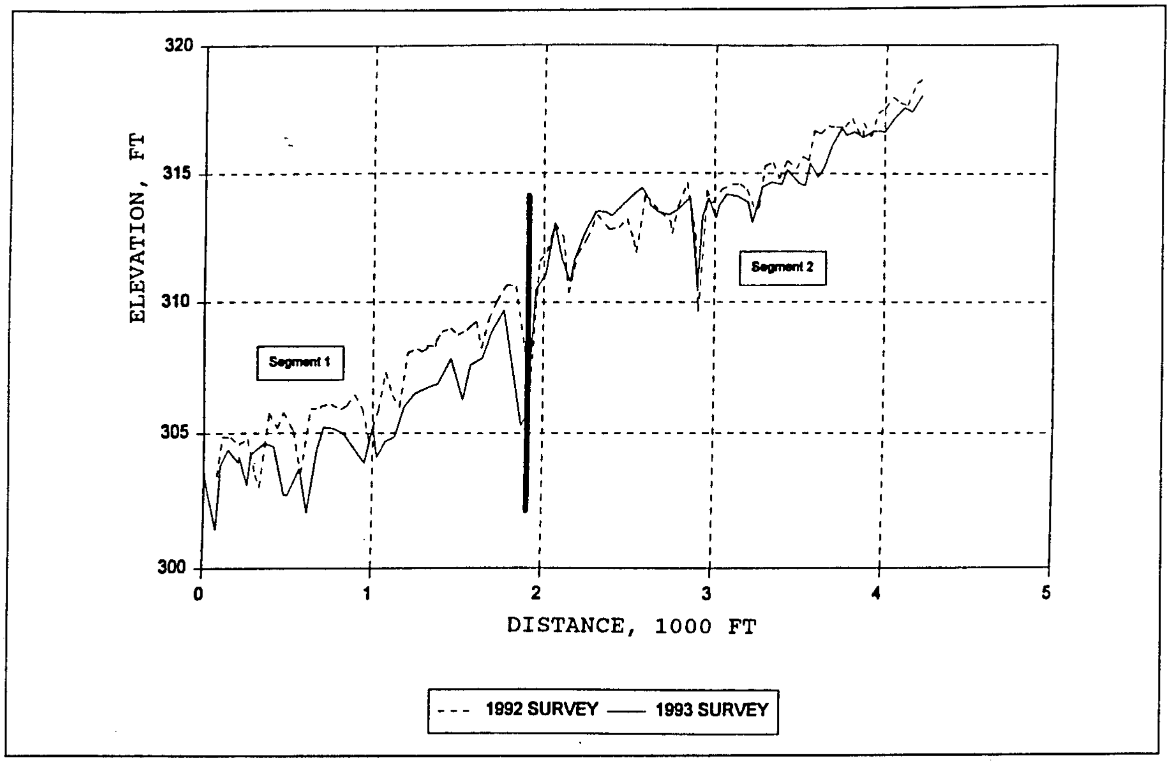



Figure 101. Comparison of 1992 and 1993 thalweg profiles for Lick Creek, Site 8

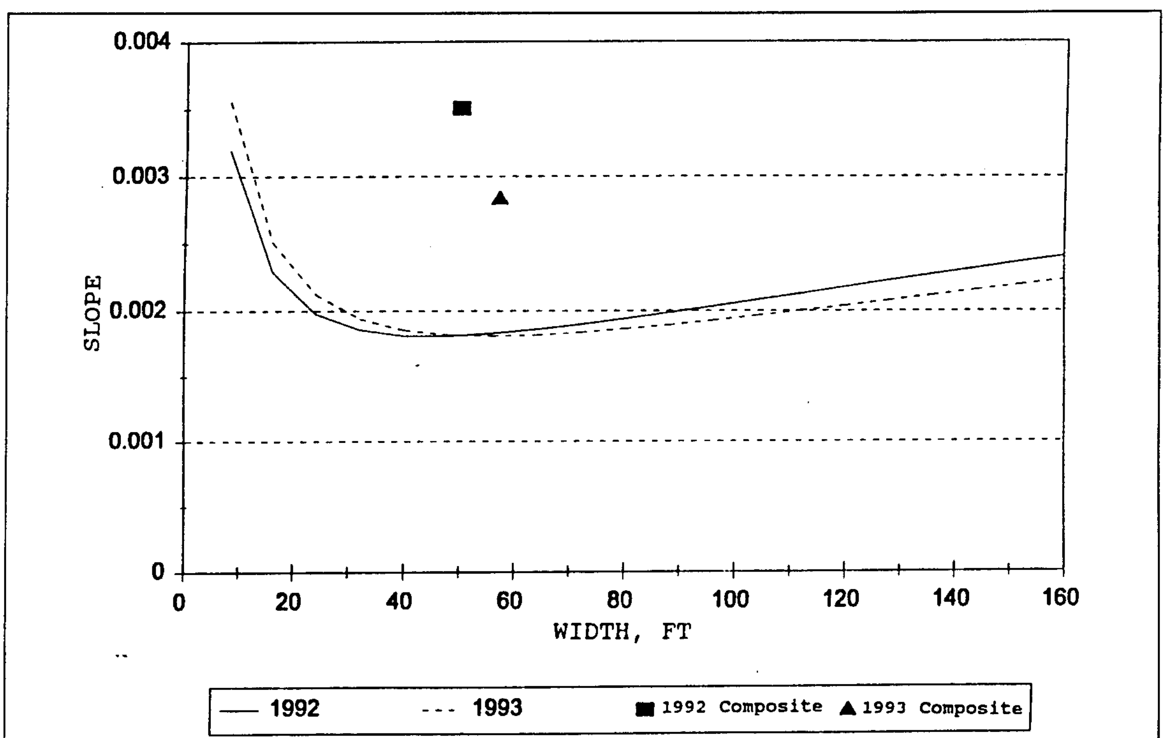



Figure 102. Comparison of 1992 and 1993 stable channel computations and the composite cross-section data for Lick Creek, segment 1

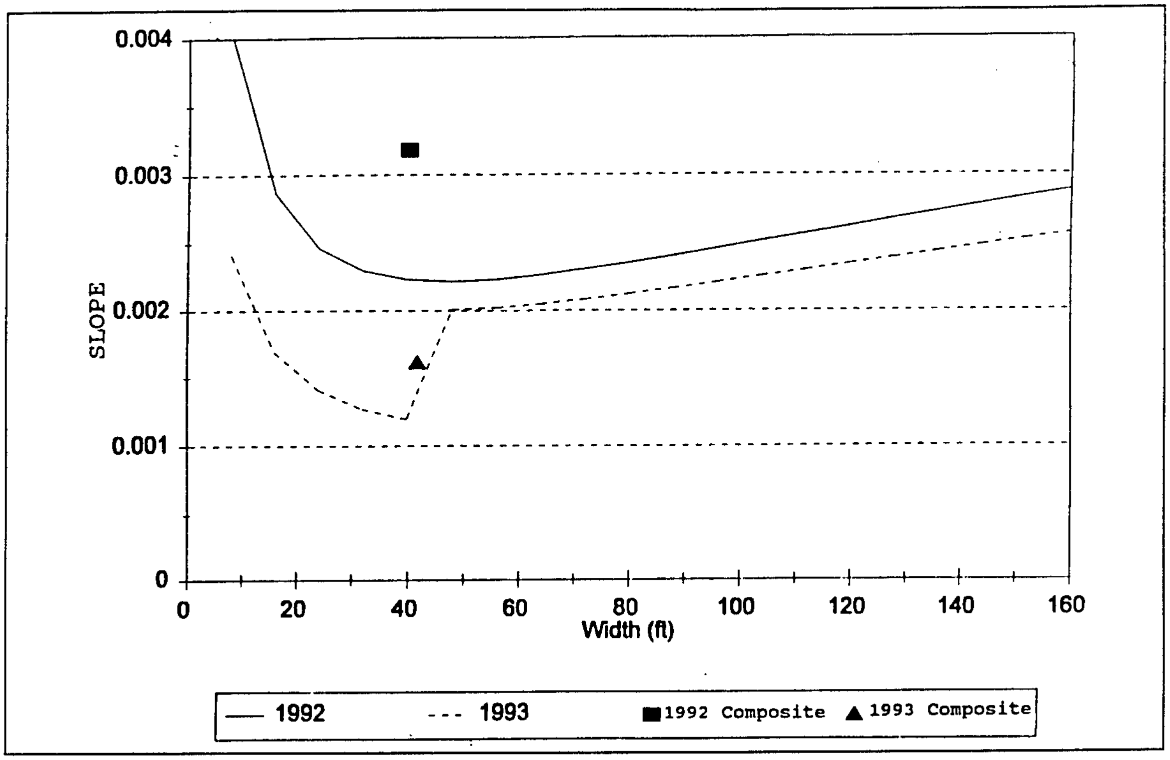



Figure 103. Comparison of 1992 and 1993 stable channel computations and the composite cross-section data for Lick Creek, Segment 2

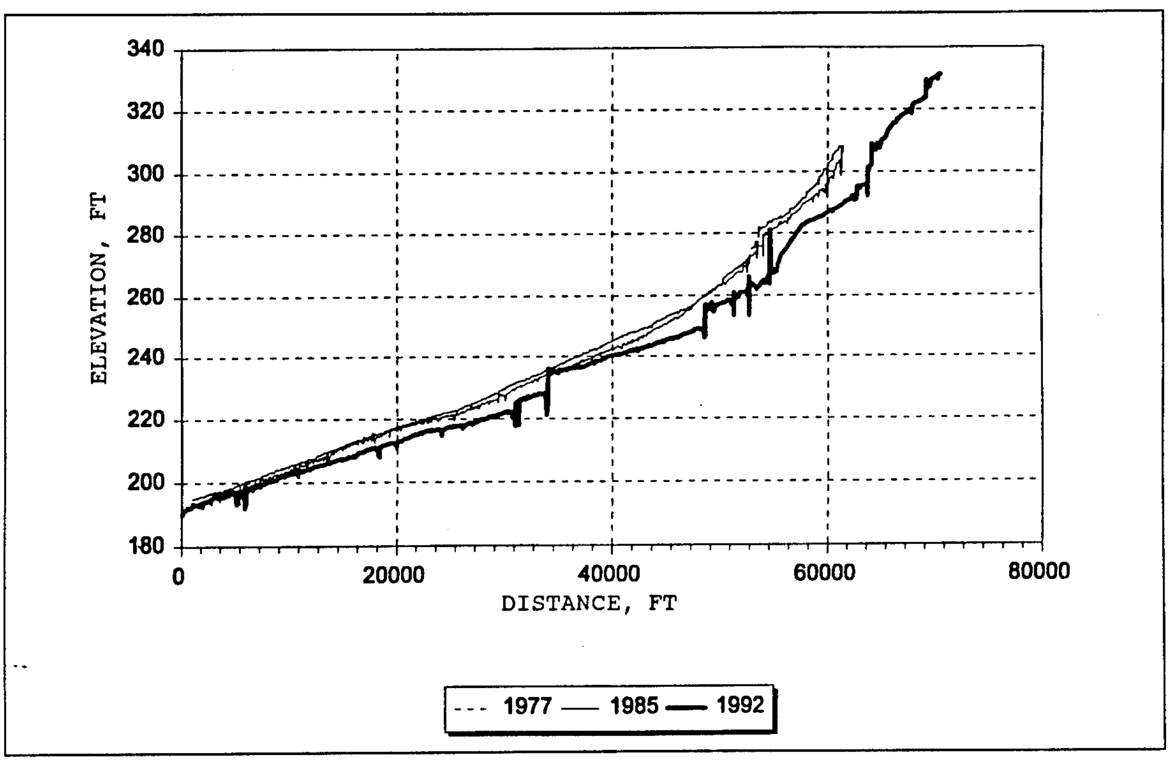



Figure 104. Comparison of 1977, 1985, and 1992 thalweg profile comparisons for Hotophia Creek

channel surveys, and compute sediment yield, not just the sediment discharge for the 2-year discharge as has been computed for all the study sites. The primary difference between this location and the DEC monitoring sites is that the USGS gauging station provides hydrology data.

Composite cross-section data were developed using the SAM program for each of the three surveys provided by the Vicksburg District. Each survey was used in HEC-2 to develop a backwater profile. The slope, velocity, width, and depth of each cross section were carefully reviewed. Data from cross sections that were indicative of a constricted location were omitted, usually identified by an energy gradient at least an order of magnitude greater than the average gradient. This resulted in data from no more than three sections being omitted. A sediment rating curve was then developed for the composite section for each of the three surveys.

Flow-duration curves are generally developed using mean daily discharge. Because the DEC streams have hydrographs that respond quickly, perhaps rising from average conditions to peak discharge in only a few hours, the flow-duration relationship for Hotophia Creek was developed using discharge data measured every 15 minutes. Although the number of data points was greatly increased, causing some computational problems, the result was worthwhile. Figure 105 compares the flow-duration relationships developed using mean daily and 15-minute discharges. The relationships show that the mean

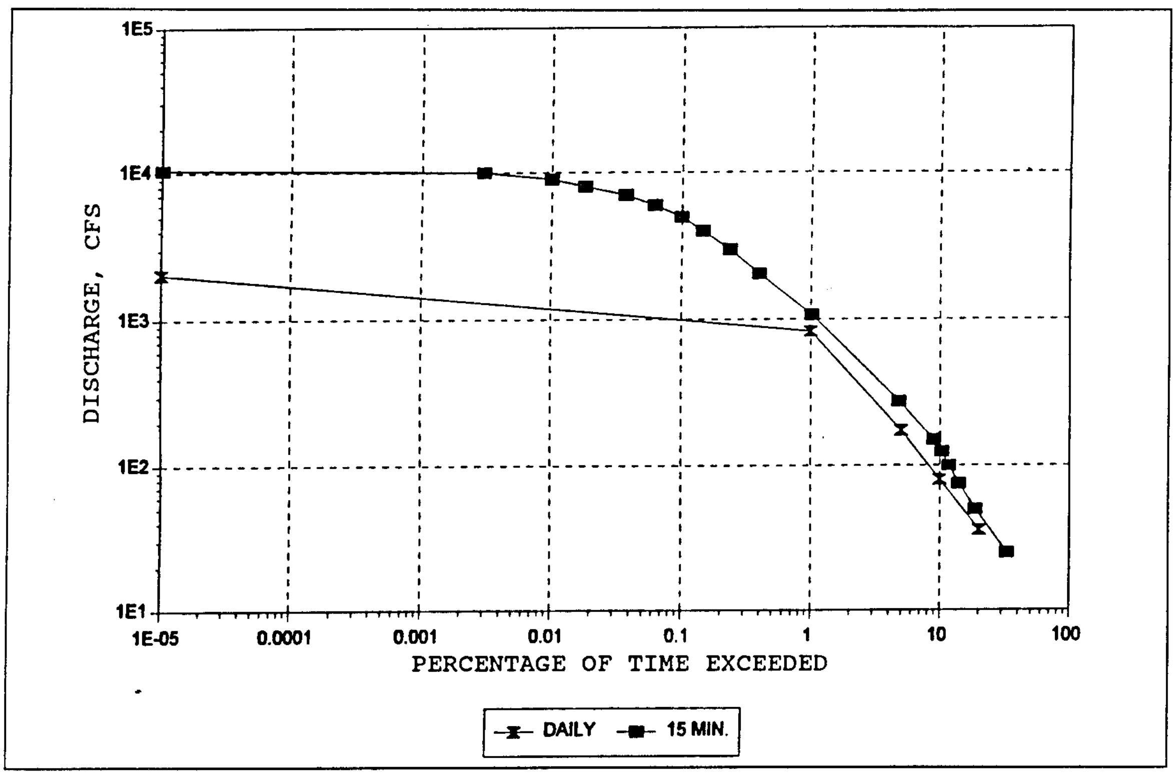



Figure 105. Comparison of flow-duration curves developed from mean daily discharge and 15-minute discharge sampling interval

daily relationship underpredicts the higher discharges, which can result in underprediction of the sediment yield and the effective discharge.

The sediment yield calculation was made using the 15-minute discharge data, which was computed as 116,800 tons per year. This yield is approximately 13 percent less than the computed 1985 sediment yield. By comparison, the sediment discharge of the 2-year discharge was computed to be 19,220 tons per day or 7,015,300 tons per year; however, the 2-year discharge occurs only 0.12 percent of the time or about 10 hours per year. The 2-year sediment discharge was found to increase approximately 11 percent over the 1985 value. Therefore, monitoring the performance of stabilization measures using only the 2-year discharge is shown to be in error, and in this case, indicates that the upstream measures are not reducing sediment yield. This example underscores the necessity of developing a dependable watershed hydrology.

### Summary of design methods

Three methods of predicting channel morphology for design purposes have been followed in the preceding sections. These methods are the regime procedure, the Vicksburg District slope-area procedure, and the SAM stable channel procedure. Regime predictions of the width, depth, and slope are based on the smaller of the bank-full discharge or the 2-year discharge, and slope and depth predictions including consideration of bed material size. The regime width prediction also includes consideration of the erodibility of the bank material. The slope prediction of the Vicksburg District procedure is based solely on the drainage area. SAM predictions are based on sediment and water discharge continuity, the bank angle, and the hydraulic roughness.

Comparisons of the 1992 and 1993 composite section average width, depth, and slope with the regime average predictions indicate that the regime procedure yields reasonable values for width and depth. The composite averages for 1992 and 1993 width are 91 percent and 89 percent of the regime prediction, respectively. Similarly, the composite depth averages for 1992 and 1993 are 81 percent and 82 percent of the regime prediction, respectively. Composite slope values are 467 percent and 391 percent of the regime predictions for 1992 and 1993, respectively. The relationship for width is depicted in Figure 106, and the relationship for depth is depicted in Figure 107. The regime depth prediction appears to be a practical upper envelope value, while the width prediction line has a more even distribution. Figure 108 depicts the 1993 composite slope values, the regime slope prediction, and a line representing the Vicksburg District slope-area relationship. Clearly, the slope-area relationship is a better predictor than the regime method for slope using this data set. The slope-area curve was developed for the basin in which the 1993 slope values were computed. The 1993 composite values represent a diversity of stable and unstable channels, and no conclusions should be drawn concerning the stability of the streams based on Figures 106, 107, or 108. With only discharge and drainage area a reasonable

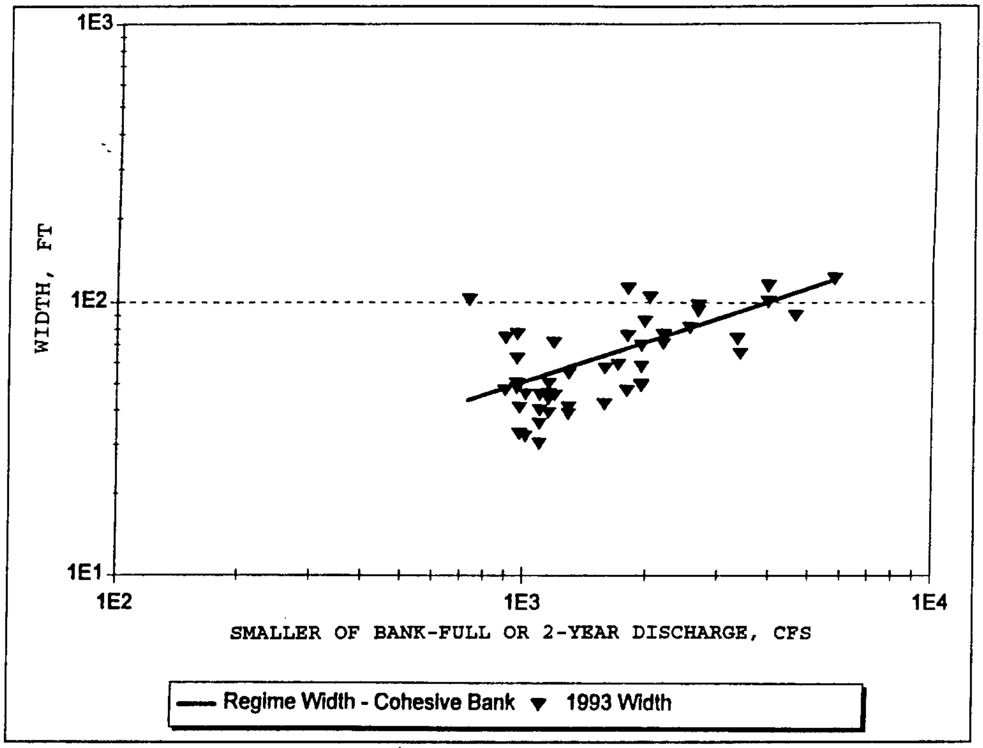



Figure 106. Comparison of regime width and composite width for Hotophia Creek

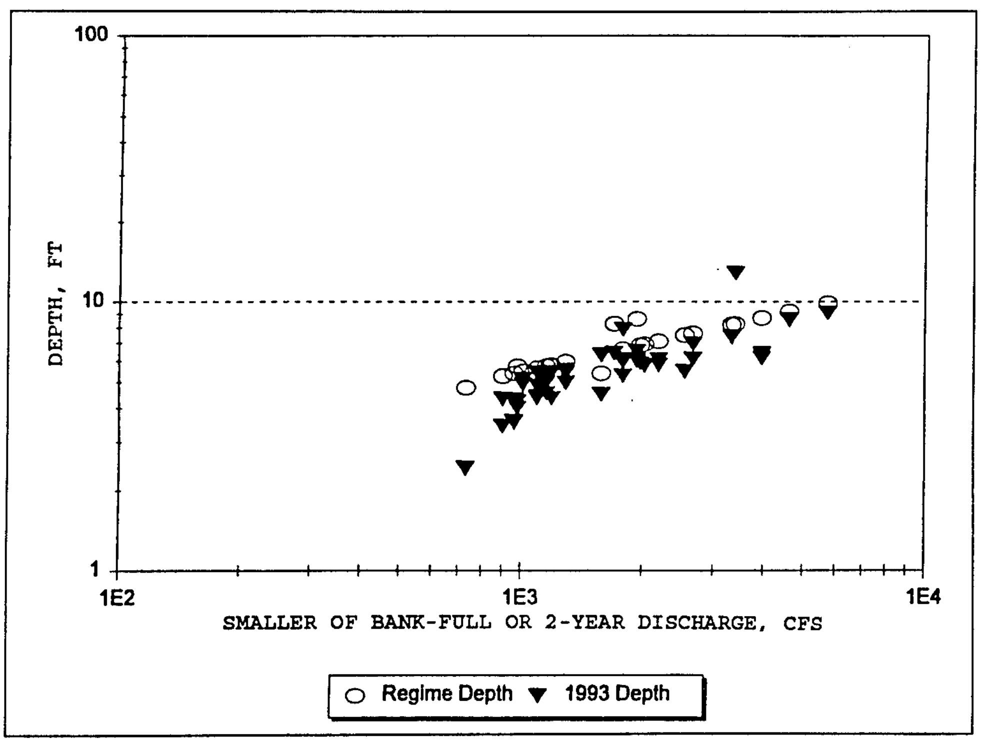



Figure 107. Comparison of regime depth and 1993 composite depth for Hotophia Creek

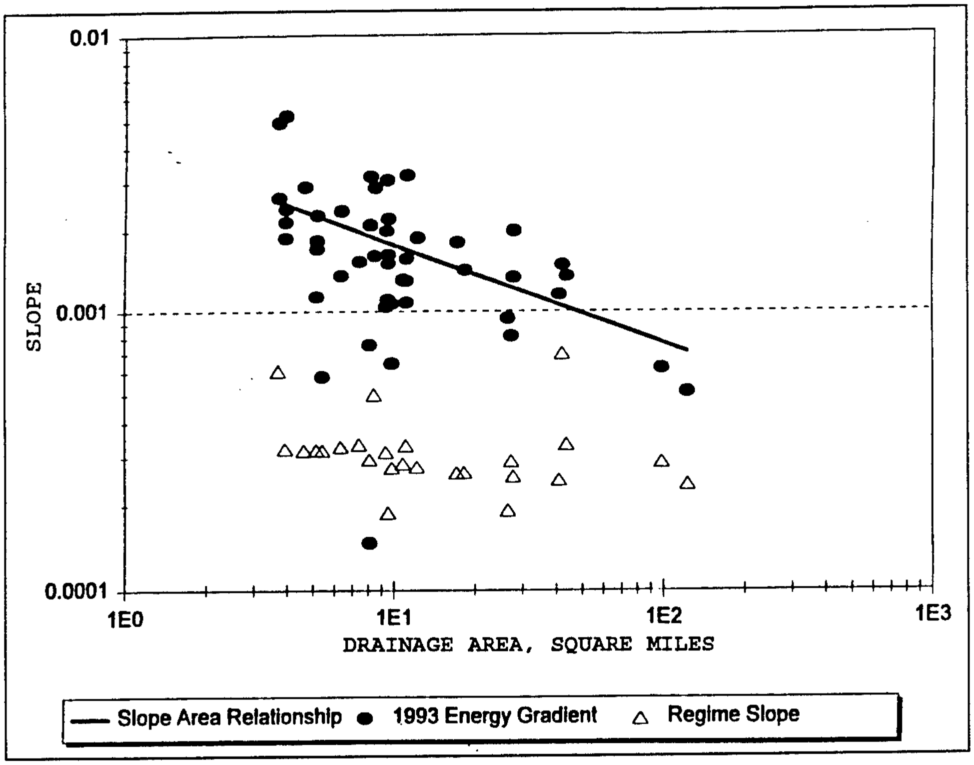



Figure 108. Comparison of slope for Hotophia Creek

estimate of the width, depth, and slope of the DEC monitoring sites could be made using the regime method for width and depth and the slope-area method for slope. These estimates do not denote that the cross section or slope is stable or, more importantly, desirable. A satisfactory design procedure should have the flexibility to allow the designer to manipulate sediment size, channel morphology, hydraulic roughness, and discharge to develop a channel that is in equilibrium for a selected water and sediment yield.

Figure 109 illustrates the relationship between the energy gradient and sediment discharge computed for each of the monitoring segments using the smaller of the bank-full or 2-year discharge. Sediment discharge increases with increasing gradient. Some of the scatter can also be attributed to the size of the sediment available for transport. For example, the three points that are generally lower and to the right of most of the data are for segments for which the bed sediment of $D_{50}$ = 10 mm was used. The remaining points are computed using sediment in the sand ranges.

The previous discussion of Hotophia Creek indicated that sediment yield could be reduced to approximately 5.2 tons/acre/year. For Hotophia Creek this converted to approximately 30 tons/ft/day at the bank-full discharge. As shown in Figure 109, 30 tons/ft/day would represent a significant, attainable improvement for most of the monitoring segments. Only 2-year discharges are available for most of the segments; therefore, an example that achieves 30 tons/ft/day sediment discharge for segment 1 of Red Banks Creek is presented in Figures 110 and 111. Figure 110 is the comparison of the 1992 and

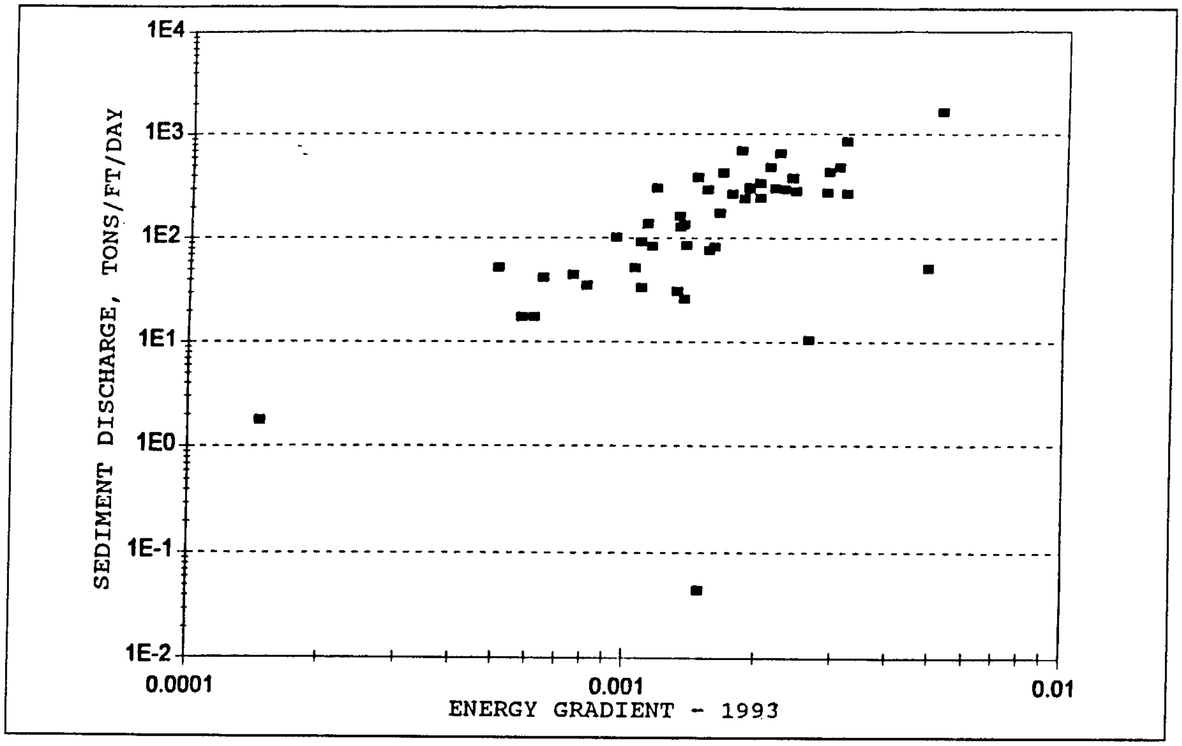



Figure 109. Unit sediment discharge for the smaller of the bank-full or 2-year discharge plotted as a function of energy gradient for Hotophia Creek

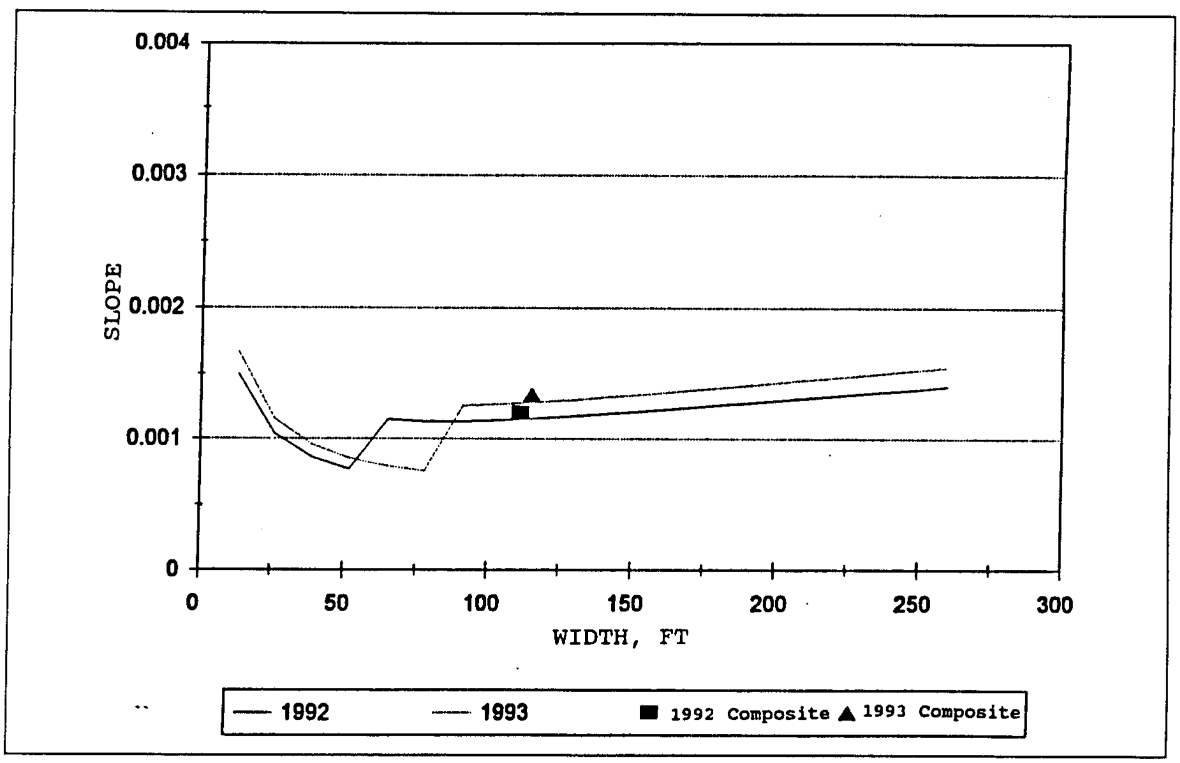



Figure 110. Comparison of stable channel computation and composite cross-section data for existing condition sediment yield of 124 tons/ft/day for 1992 and 137 tons/ft/day for 1993, Red Banks Creek, segment 1

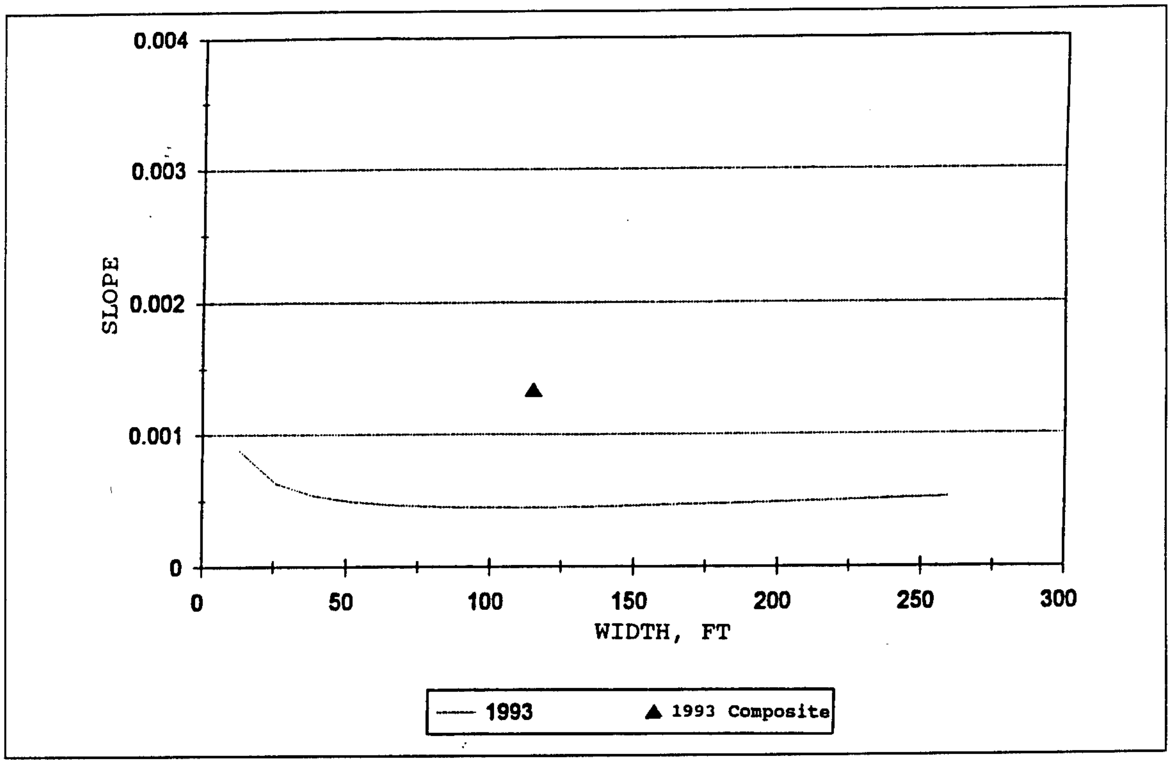



Figure 111. Comparison of composite cross-section data for 1993 with a stable channel computation for a hypothetical sediment yield of 30 tons/ft/day, Red Banks Creek, segment 1 (adjusted)

1993 composite cross-section data and the stable channel computations for the existing conditions of 124 tons/ft/day and 137 tons/ft/day, respectively. Figure 110 indicates that the channel is relatively stable; i.e., the composite section data is on the stable channel curve. Figure 111 is a comparison of the 1993 composite cross-section data and the stable channel computation for an estimated sediment discharge of 30 tons/ft/day. Recommended design practice could require the design of the channel at a selected annual sediment yield using the effective discharge. Of course, 30 tons/ft/day is an arbitrary choice of sediment discharge. Another approach to watershed sediment yield control could be to assess the sediment discharge for a large number of sites and to develop alternatives for reaching a target yield downstream using a combination of practices. For example, the cost of improving a prolific sediment supplier could be too great and effort could be concentrated on less costly streams to achieve an overall improvement of a 35 percent sediment reduction in the total watershed.

SAM has the flexibility to achieve this design. The previous example requires that a hypothetical supply reach be used. A supply reach that has a desired sediment yield may not actually exist along the stream. Another requirement of the hypothetical supply reach is that the sediment size of the supply reach must represent the bed material size of the stable reach. An example of using a hypothetical sediment size is shown in Figures 112 and 113 using Nolehoe Creek, segment 1 morphology. Figure 112 is the

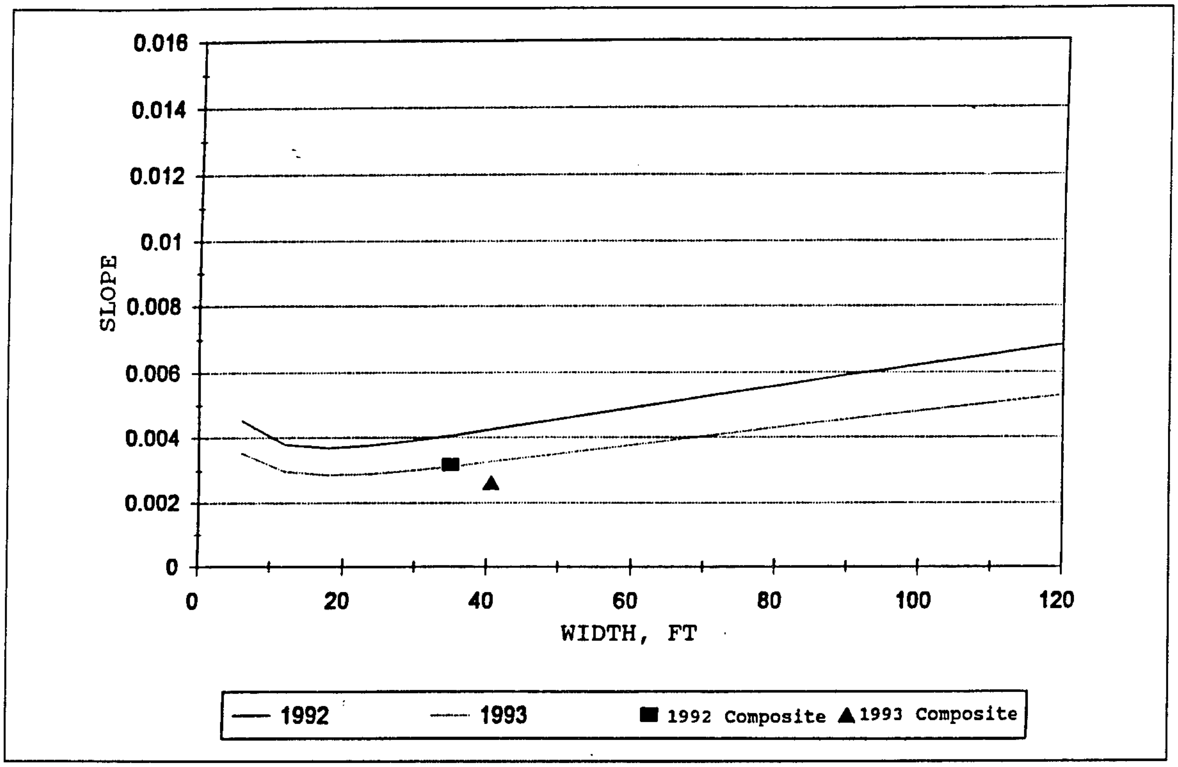



Figure 112. Comparison of 1992 and 1993 stable channel computation with composite cross-section data for the observed bed sediment, $D_{50}$ = 10 mm, Nolehoe Creek, segment 1

comparison of 1992 and 1993 stable channel computations with composite cross-section data for the observed bed sediment, which was found to be scattered, thin deposits of gravel ($D_{50}$ = 10 mm) over an erosion-resistant clay. Figure 112 indicates that the segment is aggradational. Figure 113 is a comparison of the 1993 composite section data and a stable channel computation for Nolehoe Creek using a mixed sand and gravel bed sediment measured in Harland Creek ($D_{50}$ = 0.50 mm). Using the smaller sediment size, the stable channel slopes reduce significantly. In the situation for Nolehoe Creek, sediment should be obtained from upstream and downstream sites that the designer judges to be representative of the bed sediment that will result as the segment becomes stable.

Therefore, a satisfactory design procedure to achieve a desired sediment yield should be based on sediment yield using a sediment size that will exist as the channel becomes stable. In drastically disturbed watersheds, this may require using a hypothetical supply reach.

### Segment sediment discharge

Each DEC monitoring site is divided into from one to four segments. Table 23 lists each segment for the sites and includes the type of grade control for each segment if one is located at the downstream segment limit.

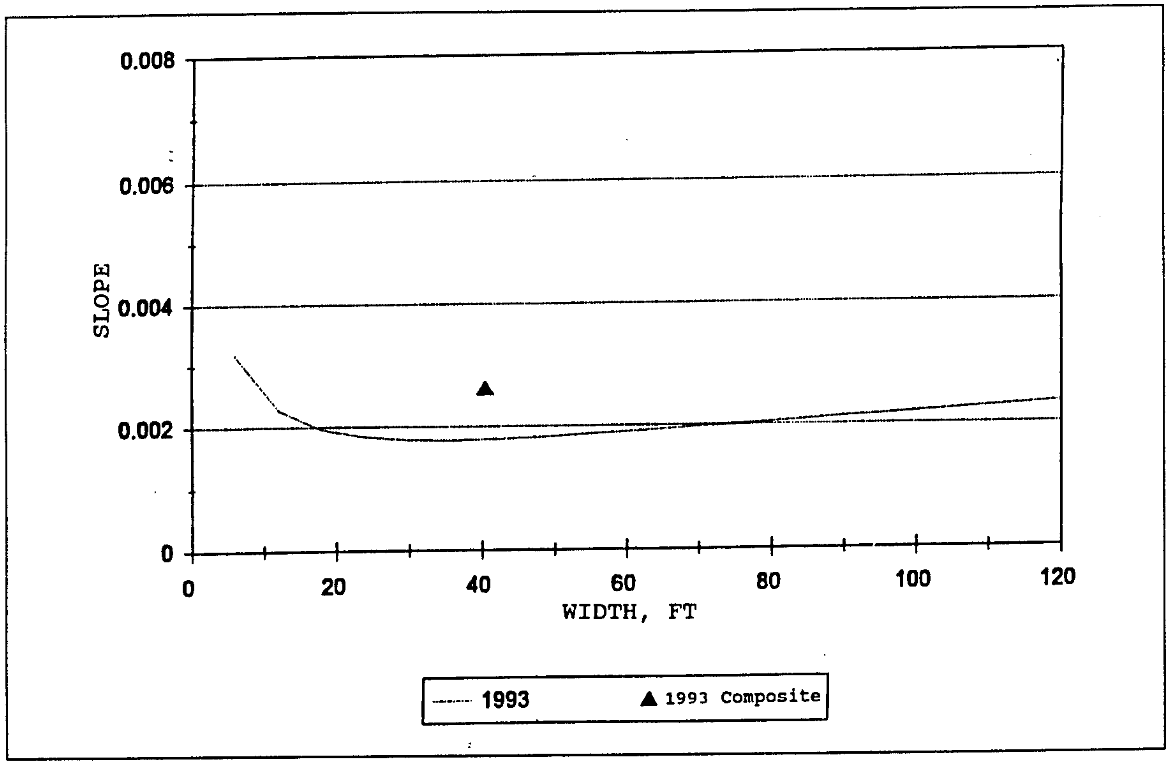



Figure 113. Comparison with the 1993 composite cross-section data with the stable channel computation for bed sediment of $D_{50}$ = 0.50 mm, Nolehoe Creek (using sediment characterized from Harland Creek)

Figure 114 is a bar chart depicting the 1993 sediment discharge for each segment. Sediment concentration for 1993 was reduced by approximately 15 percent from 1992 levels based on the average of all segments included in both the 1992 and 1993 data set.

The top ten sediment discharge rate segments are above 20,000 tons/day and are as follows (given by site and segment number): West Worsham 18W-4; Lower Hotophia 13-1; Perry 16-4; Red Banks 9-1; East Worsham 18E-2; Fannegusha 2-1; Otoucalofa 14-1; Sykes 17-1; Perry 16-3; and East Worsham 18E-1. Perry Creek sites should be ignored because construction was underway during the 1993 survey. The sites on Otoucalofa, Sykes, Fannegusha, and East Worsham have no structural downstream control. Of the top ten, the remaining sites are Red Banks Creek with chevron weirs, and Lower Hotophia and West Worsham Creeks, with low-drop structures. The apparent stability of the chevron weirs on Red Banks may be due in part to the high sediment supply to the reach. A previous recommendation was made for analyses of the reach. Lower Hotophia Creek may become a critical situation because three high-drop structures are now in place upstream of the reach. The reduction in sediment supply caused by these structures may cause scour of the reach, and careful monitoring of the reach is vital. Consideration for additional stabilization measures is recommended for the portion upstream of grade control on West Worsham.

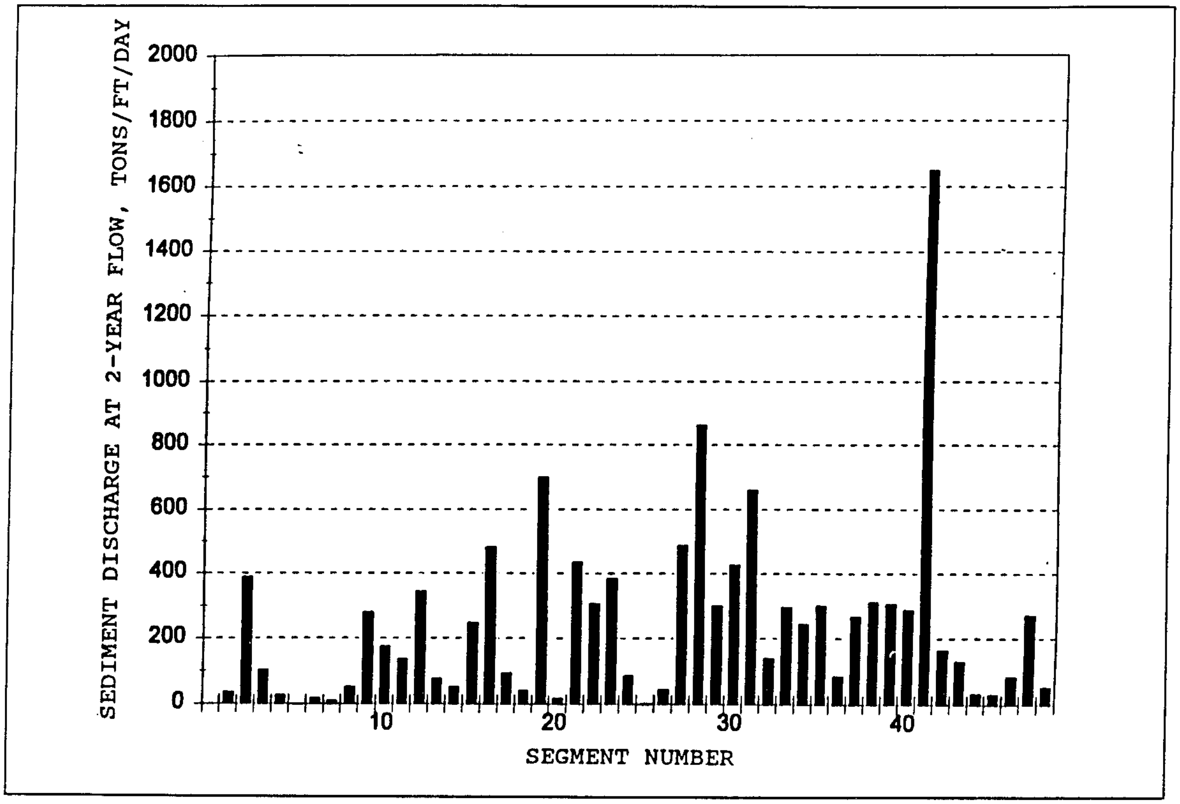



Figure 114. Segment sediment discharge

By comparison, the sediment discharge for high-drop versus low-drop structures indicates that the high-drop structures are performing much better than the low-drop structures. The better performance may be because low-drop structures generally lack hydraulic control for effective discharges or the 2-year discharge. Additional monitoring may confirm this initial observation. Many of the segments with no control have less sediment discharge than those segments with structures. However, many of the reaches chosen were relatively stable and needed no structures, and comparison of segments is inappropriate.

## Conclusions and Recommendations

The following conclusions can be drawn from the data and analyses included in this chapter:

*a.* Approximately 122,000 ft of stream channel have been surveyed twice in 1993, which includes cross-section surveys in January and thalweg surveys in June. The 1993 surveys are the second data set for the DEC monitoring sites, and comparison of the 1992 and 1993 data has provided a basis for establishing trends in channel response and structure performance. Comparison of the 1992 and 1993 average sediment discharge concentration indicates a reduction of 15 percent. Continued monitoring will be required to establish long-term trends.

*b.* Comparison of sediment discharge for the smaller of the bank-full or the 2-year discharge indicates that the sediment discharge per unit width variation is extreme, from less than 1 ton/ft/day to approximately 1,500 tons/ft/day. The sediment yield at the mouth of Hotophia Creek is approximately 5.2 tons/acre/year, which is approximately 30 tons/ft/day. Evaluation of channel stability by observation of channel morphology and change in thalweg profile may not be a reliable method of assessing sediment yield.

*c.* Working with the various monitoring sites and discussing the history of the design for some of the sites with Vicksburg District personnel have led to the conclusion that two primary design goals could be the focus of stabilization design: (a) arrest headcut migration and induce channel stability for the prevailing sediment supply; and (b) control sediment yield and induce channel stability for the desired sediment supply. Design for a new desired sediment yield introduces an added dimension to the geomorphic model of channel evolution and to empirical relationships such as the slope-area relationship.

*d.* Prior empirical stability criteria have not included sediment discharge or sediment yield directly, and have been based on the observation of channel morphology, vegetation, and change in thalweg elevation or the water surface elevation of a specific discharge. While geomorphic stability can be inferred from these observations, it is most readily evaluated by the direct comparison of sediment supply to sediment yield. Quantification of sediment yield and the relationship between channel morphology and sediment discharge must be included in the design of channel stabilization measures for the control of sediment transport to downstream reaches.

*e.* Although not conclusive, the preliminary results available indicate that the high-drop structures result in lower sediment yield than the low-drop structures. Median unit sediment discharge for high-drop structures is 58 tons/ft/day, and median unit sediment discharge for low-drop structures is 290 tons/ft/day. High-drop structures provide a greater degree of hydraulic control than low-drop structures. In addition, many of the early low-drop structures were emplaced only to arrest advancing headcuts and no consideration was made of reducing the energy gradient.

*f.* SAM is a flexible, multifaceted tool that can be used to develop a practical design procedure for channel stabilization projects requiring limitation of sediment yield. Stable channel computations have been compared with composite cross-section data for each segment of 21 monitoring sites, and for 1992 and 1993. SAM has been demonstrated to be practical and useful. Data that can be used to better monitor and assess the performance of DEC measures are (a) hydraulic roughness for the reaches, including allocation of roughness between the bank roughness and the bed roughness; and (b) sufficient hydrology to develop flow-duration relationships based on a discharge sampling interval that can define the hydrograph. Both of these data needs are being addressed by CSU and WES personnel in FY94.

*g*. The supply reach concept is common to most sediment transport models, and is necessary to produce reliable results. Sediment supply rate, size, and distribution are required at the upstream model boundary as input to the model. In drastically disturbed channels such as in the DEC monitoring sites, the rate of sediment supply is too great to be acceptable as a design input, and the size of the sediment being sampled in the reach of interest may not be representative of sediment that will comprise the future stable channel. An example has been given in this chapter of the significance of these choices.

The following specific recommendations are given:

*a*. Develop sufficient hydrology to define reliable flow-duration relationships for any site in the DEC.

*b*. Develop a design procedure for stabilization measures incorporating a selected project sediment yield goal or a sediment yield reduction goal.

*c*. Concentrate efforts to assess channel hydraulic roughness data. Improve data collection accuracy if initial assessment indicates improvement is required. Hydraulic roughness may be impacted more by ecologically desirable bioengineering measures than by riprap placement. Roughness assessment of alternative stabilization measures is essential.

*d*. The capability of SAM to predict sediment transport in gravel or mixed-bed channels should be appraised for streams such as Harland Creek and Abiaca Creek. Modification of the program is recommended to incorporate the full range of sediment sizes encountered in the DEC.

Work is continuing on bank stability and the relationship between the channel conveyance before and after bank failure. Adequate sampling methods and the quantity of sediment necessary for a statistically valid quantification of sediment size and distribution are continuing to be assessed; field work in January 1994 will test a recently developed method. A second Harland Creek site incorporating innovative stabilization measures has been added for January 1994. The collection of a third data set during 1994 will significantly add to the monitoring value.

# 4 Channel Response, Detailed Geomorphic Assessments

Detailed geomorphic assessments were conducted on the two watersheds that were resurveyed in 1992. These watersheds were Otoucalofa Creek and Hotophia Creek. Both the 1985 and 1992 surveys consisted of channel profiles (thalweg) and cross sections made at approximately half-mile intervals. The surveys were used to determine channel changes from 1985 to 1992. The 1985 surveys had been used by the Vicksburg District in various analyses of the channel systems. The 1992 surveys were used to determine channel changes since 1985. Channel profiles were compared to determine zones of aggradation and degradation. Channel cross sections were plotted for use in determining width and depth changes. The complete sets of channel profile and cross-section plots of Hotophia Creek and Otoucalofa Creek watersheds are contained in Appendixes B and C of this report, respectively. A general description of the channel assessments follows.

## Channel Profiles

The channel profiles from 1985 and 1992 were digitized. Channel stationing began at the mouth of each channel and increased in the upstream direction along the channel thalweg. No survey baseline was used on either of the two surveys, and channel stationing was dependent on the measured distance along the thalweg. Since the thalweg tends to shift over time, the measured distances were often inconsistent between the two surveys. Locations of bridges, culverts, grade control structures, tributary intersections, and other channel features noted on the surveys were used to fit the stationing from the 1992 survey to that from the 1985 survey. The two channel profiles were then plotted to the 1985 stationing. These plots are included in Appendixes B and C of this report. Areas of significant channel aggradation or degradation can be located using these plots.

## Channel Cross Sections

Channel cross sections from 1992 were plotted against the same cross sections from 1985. Whenever possible, the 1992 cross sections were surveyed at the same location as the 1985 cross sections. Additional cross sections were surveyed in 1992, and several cross sections in the Otoucalofa Creek watershed were surveyed at different locations from those of 1985. Cross sections from 1985 were then plotted against 1992 cross sections. The cross-section stationing was determined from the corresponding channel profile; therefore stationing between 1985 and 1992 cross sections may be different even though the cross sections have the same locations.

## Watersheds

### Hotophia Creek Watershed

**Hotophia Creek.** For evaluation purposes, Hotophia Creek was divided into five reaches as follows (referenced to 1985 stationing):

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 197+00 |
| 2 | 197+00 to 329+00 |
| 3 | 329+00 to 473+00 |
| 4 | 473+00 to 549+00 |
| 5 | 549+00 to 613+00 |

In reach 1 (lowermost), the channel appeared stable between station 0+00 and 56+00 from 1985 to 1992. From 56+00 to 105+00, the channel degradation averaged about 1 ft. From 105+00 to the upper end of reach 1 at 197+00, channel degradation increased from about 1 ft (at 105+00) to about 4 ft (at 197+00). In reach 2, channel degradation increased from 4 ft at the lower end to about 5 ft at the upper end. From the lower end of reach 3 (329+00) to the grade control structure (340+00), degradation averaged about 6 ft. From the grade control structure to 358+00, the channel appeared stable between 1985 and 1992. From 358+00 to the upper end of reach 3 (473+00), the channel was degradational, increasing from about 1 ft (358+00) to about 7 ft (473+00). In reach 4, the channel was highly degradational, averaging about 8 ft. In reach 5, the channel was highly degradational, averaging almost 10 ft with a maximum change of about 13 ft at the upper end (613+00).

**Harris Creek.** No 1985 survey data were available for comparison with the 1992 data.

**Mill Creek.** No 1985 survey data were available for comparison with the 1992 data.

**Deer Creek.** This creek was divided into two reaches as follows:

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 15+00 |
| 2 | 15+00 to 114+00 |

In reach 1, the channel ranged from stable at the lower end (0+00) to about 2 ft degradational in the upper half of the reach. In reach 2, the channel was slightly degradational, averaging about 1 ft through the entire reach.

**Marcum Creek.** This creek was divided into two reaches as follows.

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 13+00 |
| 2 | 13+00 to 95+00 |

In the lower portion of reach 1 (0+00 to 10+00), the channel was degradational between 1985 and 1992, ranging from 2 ft at station 0+00 to stable at station 10+00. The upper portion of reach 1 (10+00 to 13+00) was aggradational, ranging from stable at station 10+00 to 1 ft at station 13+00. In the lower portion of reach 2 (13+00 to 50+00) the channel was consistently degradational, averaging between 1 and 2 ft. From station 50+00 to station 76+00, the channel was relatively stable. In the upper portion of reach 2 (76+00 to 95+00) the channel was again degradational, averaging over 1 ft.

### Otoucalofa Creek Watershed

**Otoucalofa Creek.** For evaluation purposes, Otoucalofa Creek was divided into 16 reaches as listed in the following tabulation (referenced to 1985 stationing). In reach 1 (lowermost reach), sufficent data were not available for comparison. In reaches 2 through 7, no significant aggradation or degradation trends were observed between 1985 and 1992. In reach 8, a degradational trend was noted, with maximum degradation of about 5 ft occurring between 1985 and 1992. Reaches 9 and 10 showed degradation averaging about 2 ft, with a 3-ft maximum. Reaches 11 through 13 showed no significant aggradation or degradation. Reaches 14 through 16 indicated degradation averaging about 4 ft, with maximum degradation of about 6 ft.

**Susie Perry Creek.** No 1985 survey data were available for comparison with the 1992 data.

| Reach | Stationing (Otoucalofa Creek) |
|---|---|
| 1 | 0+00 to 228+00 |
| 2 | 228+00 to 365+00 |
| 3 | 365+00 to 397+00 |
| 4 | 397+00 to 436+00 |
| 5 | 436+00 to 461+00 |
| 6 | 461+00 to 554+00 |
| 7 | 554+00 to 584+00 |
| 8 | 584+00 to 738+00 |
| 9 | 738+00 to 775+00 |
| 10 | 775+00 to 838+00 |
| 11 | 838+00 to 930+00 |
| 12 | 930+00 to 1037+00 |
| 13 | 1037+00 to 1082+00 |
| 14 | 1082+00 to 1132+00 |
| 15 | 1132+00 to 1251+00 |
| 16 | 1251+00 to 1305+00 |

**West Johnson Creek.** No 1992 survey data were available for comparison with the 1985 data.

**Johnson Creek.** The creek is divided into four reaches as follows:

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 106+00 |
| 2 | 106+00 to 122+00 |
| 3 | 122+00 to 158+00 |
| 4 | 158+00 to 184+00 |

In reach 1 (lowermost reach), no significant aggradation or degradation trends were observed between 1985 and 1992. Reaches 2 through 4 indicate a slight degradational trend, averaging almost 2 ft, with a maximum of about 3 ft.

**Town Creek.** No 1985 survey data were available for comparison with the 1992 data.

**Tributary 16A.** No 1992 survey data were available for comparison with the 1985 data.

**Tributary 15C.** No 1992 survey data were available for comparison with the 1985 data.

**Simmons Creek.** From station 0+00 to 30+00, the creek showed a slight degradational trend of about 1 ft from 1985 to 1992. From 30+00 to 33+00, the creek was stable. From 33+00 to 65+00, the creek was highly degradational, ranging from 0 ft at 33+00 to a maximum of 12 ft at 65+00.

**Hitchcock Creek.** No 1992 data were available for comparison with the 1985 data.

**Greasy Creek.** The creek was divided into four reaches as follows:

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 95+00 |
| 2 | 95+00 to 111+00 |
| 3 | 111+00 to 138+00 |
| 4 | 138+00 to 158+00 |

In reach 1 (lowermost reach), a very slight degradation trend, averaging about 1/2 ft, was observed. In reaches 2 through 4, no 1992 data were available for comparison with the 1985 data.

**Moore Creek.** From station 20+00 to 64+00, a degradation trend was observed, averaging almost 3 ft with a maximum change of 5 ft. Below station 20+00, insufficient data from 1985 made meaningful comparison impossible.

**Gordon Creek.** The creek was divided into three reaches as follows (referenced to 1985 stationing):

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 39+00 |
| 2 | 39+00 to 75+00 |
| 3 | 75+00 to 90+00 |

In reach 1 (lowermost), no aggradational or degradational trends were observed. In reachs 2 and 3, 1985 data were not available for comparison with the 1992 data.

**Main East Spring.** No 1992 data were available for comparison with the 1985 data.

**South Spring Creek.** No 1992 data were available for comparison with the 1985 data.

**Mill Creek.** Only limited 1985 data were available. However the comparison with the 1992 data shows that (a) about 2 ft of aggradation occurred at station 0+00, (b) between 21+00 and 26+00 the channel was stable, and (c) between 46+00 and 56+00 about 1 ft of degradation occurred.

**Smith Creek.** No 1992 data were available for comparison with the 1985 data.

**Hanna Creek.** No 1992 data were available for comparison with the 1985 data.

**Tributary 12A.** No 1992 data were available for comparison with the 1985 data.

**Sarter Creek.** The creek was divided into three reaches as follows:

| Reach | Stationing |
|---|---|
| 1 | 0+00 to 77+00 |
| 2 | 77+00 to 163+00 |
| 3 | 163+00 to 292+00 |

In reach 1 (lowermost) and the lower portion of reach 2, a degradational trend averaging about 3 ft was observed. In the upper portion of reach 2 and reach 3, the channel appeared stable.

**West Sarter Creek.** No 1992 data were available for comparison with the 1985 data.

**Paris Town Creek.** Comparison of 1985 and 1992 data showed the channel to be stable from station 0+00 to 85+00, with no significant aggradational or degradation trends during that period.

**Dickey Creek.** No 1992 data were available for comparison with the 1985 data.

**Shippy Creek.** From station 0+00 to 140+00, the comparison of 1985 and 1992 data showed an aggradational trend, averaging about 1 ft with a maximum change of 3 ft.

**Tributary 2A.** No 1992 data were available for comparison with the 1985 data.

**Tributary 1B.** No 1992 data were available for comparison with the 1985 data.

# 5 Hydrology

## Introduction

In preparation for the hydrologic analyses that are to take place in the DEC watersheds, a history of the hydrologic modeling efforts for north Mississippi was prepared. As a result of this analysis, it was determined that since there were a number of ungauged watersheds within the project area and there was some concern about the lumped model approach, a case study was needed to compare a distributed model with two presently used lumped model techniques. The distributed model uses spatially varied data such as land use, soil type, elevations, and rainfall. The focus of the study was to see if more detailed spatial data could offset the lack of gauge data in calibrating the hydrology models. The following is a discussion of the history analysis, the distributed versus lumped model study, the conclusions drawn from the study, and recommendations for improving the distributed hydrology model as it pertains to the DEC project.

## Hydrologic Modeling Efforts in North Mississippi

In the past, ARS, the Vicksburg District, and SCS have worked to stabilize streambank erosion in north Mississippi. In this effort, many structures have been built that required the estimation of a design flow. In the design of these structures, various hydrologic models have been used. In some cases, different methods or models were used for the same watershed. In those cases, a significant difference in computed design flows usually resulted. The question then is, "Which method computes the flow that is closest to the correct flow?" Two examples of studies where different methods have been applied are presented in the following paragraphs. The discussion focuses on a description of the watersheds, the procedures used on each watershed, and the results from each procedure. The primary purpose of the methods employed was to generate a peak design flow for streambank erosion and/or grade control structures.

## Long Creek Watershed

The Long Creek watershed is located in the southwestern part of Panola County in north-central Mississippi (FTN Associates, Ltd., 1987). The watershed covers an area of approximately 86 square miles (55,074 acres) and is rectangular in shape, approximately 13 miles long and 8 miles wide. The Long Creek basin drains into the Yocona River downstream of Enid Reservoir and is a part of the Yazoo River Basin. The relief of the watershed is 314 ft with the lowest point at el 170.5 ft and the highest point at el 485 ft referenced to the National Geodetic Vertical Datum (NGVD). Long Creek watershed lies in a subtropical region characterized by mild, humid winters and long, hot, and humid summers. The weather in the region is controlled by its proximity to the Gulf of Mexico and prevailing southerly winds. The wet seasons are winter and spring with prolonged, low-intensity rains. During the summer and fall, rain falls mostly as thunderstorms with intense rainfall, short duration, and limited aerial coverage. Hurricane-force winds do not affect the region, but heavy rainfall from tropical storms does occur occasionally in the summer and fall months.

There are three National Weather Service stations in the vicinity of the Long Creek watershed: Batesville, Enid Dam, and Water Valley. Average annual precipitation for these three stations is given in Table 24. This study was conducted in 1987 and as such the data presented here are accurate only up to that point in time. The driest year of record was 1981 with the wettest year being 1973. In 1981, the rainfall amounts (in inches) at the three stations were Batesville, 38.83; Enid Dam, 34.61; and Water Valley, 34.62. In 1973, the rainfall amounts (in inches) were Batesville, 75.35; Enid Dam, 73.96; and Water Valley, 80.89.

In the Long Creek watershed, 12 streams were used in the hydrologic analysis. Table 25 lists the streams and the drainage area of each.

For these streams, three different hydrologic analyses were performed. In 1966, the Vicksburg District used the generalized peak flow frequency analysis procedure for the Yazoo Hill area (FTN Associates, Ltd., 1987). This procedure was developed by taking numerous observed discharge readings within the Yazoo Hill area and using least squares regression analysis to determine a relationship for peak flow as a function of the physiographic watershed parameters. When the basin area, slope, and stream length are known, the peak flow for a specific frequency can be calculated.

In 1976, an analysis of the flood frequency of Mississippi streams was done (Colson and Hudson 1976). This method involved essentially the same regression analysis as the Yazoo Hill area procedure except that data were taken over the whole state instead of just the Yazoo Hill area. Again, when the basin area, slope, and stream length are known, the USGS equations can be used to calculate a peak flow for a given frequency.

In 1987, FTN Associates, Ltd., Little Rock, AR, performed an HEC-1 study for the Vicksburg District, using Snyder's unit hydrograph method for overland flow and the Muskingum channel routing routine (FTN Associates, Ltd., 1987). The Muskingum method requires three parameters for each channel reach: the Muskingum *K* coefficient, the Muskingum *X* coefficient, and the number of routing subreaches within the channel reach. These parameters are best determined from recorded inflow and outflow data for the reach in question (Simons, Li and Associates 1987). However, since streamflow data are limited for the watersheds being modeled, other methods were used to estimate the parameters.

The Muskingum *K* coefficient is known as the storage coefficient and is the ratio of storage to discharge. It has the dimensions of time and can be estimated as the travel time of the flood wave through the reach. The only data available for estimating travel time were the channel length and slope determined from topographic maps and estimates of channel size and roughness based on site visit observations. These data were used to estimate the velocity and hence the travel time through each reach (Simons, Li and Associates 1987).

The Muskingum coefficient *X* has theoretical limits between 0.0 and 0.5 with a mean value near 0.2. For this study (FTN Associates, Ltd., 1987), the coefficient was set equal to 0.10 for all channel reaches to reflect the expected storage effects characteristic of this type of watershed.

Results from the three peak flow frequency methods are presented in Table 26.

## Hickahala-Senatobia Creek watershed

The Hickahala-Senatobia watershed is located approximately 30 miles south of Memphis, TN, in northwestern Mississippi. Hickahala Creek is a tributary to the Coldwater River just upstream of Arkabutla Reservoir. The Hickahala Creek watershed is located in portions of Tate, Panola, and Marshall Counties and encompasses approximately 230 square miles. The largest urban area of the watershed, the city of Senatobia, is located near the confluence of Senatobia and Hickahala Creeks, approximately 6 miles upstream of the confluence of Hickahala Creek and the Coldwater River (Simons, Li and Associates 1987).

As with the Long Creek watershed, three different hydrologic methods were used to estimate design flows for the streambank erosion and grade control structures in the watershed: a calibrated HEC-1 model using Synder's unit hydrograph method for overland flows and the Muskingum channel routing method, the USGS flood frequency method for Mississippi streams, and the Vicksburg District generalized peak flow frequency method for Yazoo Hill area.

The parameters used in the Muskingum routing, $K$ and $X$, were obtained in a similar manner as in the Long Creek study. However for this watershed, $X$ was estimated to be 0.15. Results are presented in Table 27.

From an inspection of Tables 26 and 27, a significant difference in the design flows computed by each method is observed. From Table 26, the percent variance (i.e., 100 × (Maximum $Q$/Minimum $Q$)) for the 2-year-frequency storm had a minimum of 171.1, a maximum of 355.9, and an average of 247.8. The variance for the 100-year-frequency storm had a minimum of 115.8, a maximum of 238.3, and an average of 170.6. From Table 27, the percent variance for the 2-year-frequency storm had a minimum of 136.9, a maximum of 443.1, and an average of 220.9. The variance for the 100-year-frequency storm had a minimum of 122.8, a maximum of 231.4, and an average of 173.4.

## Purpose and Scope of Distributed Versus Lumped Model Study

Since design flows must often be computed for ungauged watersheds, the discrepancy observed in these peak flows causes concern about the relative accuracy of hydrologic methods/models currently used to simulate rainfall events. This concern was the stimulus for conducting the study reported herein. In particular, the commonly employed SCS and Snyder's unit hydrograph methods of the HEC-1 computer program (one-dimensional lumped models) have been compared to a recently developed two-dimensional distributed model from CSU, CASC2D, to determine if this model, which contains more spatial data, can produce more reliable results when applied to ungauged basins.

Traditionally, the Snyder and SCS unit hydrograph methods are used to estimate the peak discharge for the purpose of designing channels, structures, etc. In using these methods, it is necessary to calibrate the lag time and infiltration parameters for each of the methods. Typically in using the Snyder method, initial and uniform loss rates are calibrated for different storm events. When using the SCS method, an SCS curve number relating land use and soil type to loss rates is estimated.

The watershed chosen for this study was the Goodwin Creek watershed, which is a subwatershed of the Long Creek watershed. The ARS has been active for a number of years in gauging Goodwin Creek. These data provide an excellent opportunity to apply these hydrologic models and to assess their performance on gauged and ungauged watershed scenarios, and to gain insight in choosing infiltration and other loss parameters for their application to north Mississippi streams.

## Description of Goodwin Creek Watershed Data

Goodwin Creek watershed contains approximately 8.4 square miles and is located within the Long Creek watershed. There are 17 rainfall gauges and 14 discharge gauges located within the boundary of the watershed. For this analysis, five main stem discharge gauges and one tributary gauge were used. These six discharge gauges are spread uniformly over the watershed. Since the main goal was to evaluate how the models performed in an ungauged watershed scenario, these gauges provided enough information to draw conclusions and to make recommendations. Fifty-seven channel cross sections on the main stem and the tributaries provided the necessary data for constructing the channel geometric database. The period of record for the rainfall and flow data is approximately 7 years, 1981 to 1988. As a part of the DEC Project, a GIS database has been created for Goodwin Creek watershed. The GIS contains such data as land use grids, soil type grids, elevation grids, SCS curve number grids, slope grids, USGS digital line graphics, and aerial photography. The grid cell resolution for all the grids used in this study was 416 ft by 416 ft.[1] This resolution was adequate for the Goodwin Creek watershed.

The land use for this watershed varies from forest, to cropland, to pasture, and to small ponds. Over the past 10 to 20 years, this area has experienced streambank instability and sedimentation problems due to changes in land use. There are three primary soil types in the watershed: loam, sandy loam, and silt loam, with silt loam being the predominant soil type. The maximum elevation is el 412.9 ft NGVD and the minimum elevation is el 236.8 ft NGVD.

In setting up the models, the grid cell data (i.e., land use, soil type, elevation) had to be extracted from the GIS and manipulated into the proper format for CASC2D. The lumped models used the GIS to compute average values (i.e., roughness coefficients and soil types) for each subarea. {start here}The channel cross sections were averaged for each routing reach by plotting the cross sections for each reach and estimating the average cross section for each reach.

For the past 12 years, ARS has been extensively gauging the Goodwin Creek watershed. The data currently being gathered are rainfall, discharge, suspended sediment, and bed load. ARS is also generating land use grids from various time periods for this watershed. The field sampling and measurement stations are located at grade control structures built in the late 1970's. This effort was a joint project conducted by the ARS and the Vicksburg District.

[1] Personal Communication, 17 August 1992, Billy E. Johnson, from Dr. Bahram Saghafian, Construction Engineering Research Laboratory, U.S. Army Corps of Engineers, and Dr. Fred Ogden, University of Iowa.

## Application of Models and Methods

There were two parts to the Goodwin Creek analysis. In the first part, five rainfall events were simulated using 17 rainfall gauges and 6 discharge gauges. Two HEC-1 models, SCS unit hydrograph and Snyder unit hydrograph, and CASC2D were used to simulate rainfall runoff for the purpose of comparing them to observed flow records. In this analysis, all models used the Green and Ampt infiltration routine. The HEC-1 models used the Muskingum-Cunge channel routing routine while CASC2D used a one-dimensional diffusive wave routine. The peak flow, time to peak, volume of runoff, and hydrograph variance parameters were summarized for all three models from the output.

In the second part of this analysis, two hypothetical storm events were simulated using data from one rainfall gauge (Gage 54) considered to be representative of the average observed storm conditions. This analysis was performed using only one lumped model (Snyder) and the distributive model (CASC2D). The reason for comparing only one lumped model (Snyder) instead of two lumped models (Snyder and SCS), as in the first part of this study, is that there are only minor differences in the methodologies (i.e., peak flow equations). Based upon the results from the first part of this study, only minor differences were noted between the two lumped models; therefore, little would have been gained by running both models for the second part. The reason for this simulation scenario was to compare the models assuming a temporally varied rainfall event uniformly distributed spatially over the entire watershed.

## Modeling Approach used for Comparison Study

As stated previously, in the first part of this study, each of the selected hydrologic models was applied to five observed storm events, using 17 rainfall gauges, with the predicted discharge hydrographs compared to observed hydrographs at six stream gauge locations. Each HEC-1 model was calibrated (optimizing initial loss and soil moisture content) using data at Gage 1 (mouth of Goodwin Creek). Storms 1 and 3 were also calibrated at Gages 5 and 8. This was done to evaluate the relative accuracy of the lumped models when sufficient sub-basin gauge data were available. Storms 2, 4, and 5 were calibrated only at the mouth of Goodwin Creek. This was done to assess the relative accuracy of the lumped models using limited gauge information. CASC2D was calibrated (optimizing initial soil moisture content) only at the mouth for all five storm events. The reason for this was to evaluate the response of a distributed model using limited gauge information. Initial runs were made with no calibration at all; however, this proved unsuccessful due to a lack of knowledge about how the initial antecedent moisture and ground cover conditions changed from storm to storm.

The parameters considered in the calibration and simulation comparisons were peak flow, time to peak, total runoff volume, and four hydrograph variance values (i.e., standard error, objective function, average absolute error, and average percent absolute error). The results of final simulation computer runs are presented in Appendix D.

In the second part of this study, rainfall Gage 54 was used to simulate the hypothetical uniform rainfall events over the watershed. All of the discharge gauges were used to calibrate the HEC-1 lumped model, however only Gage 1 was used to calibrate CASC2D. Storm events 1 and 3 were chosen, because storm 1 was a slow-rising and -falling storm while storm 3 was a fast-rising and -falling storm. The same infiltration function, overland routing routine, and channel routing routine described in the first part of this study were used in the second part for both the Snyder HEC-1 and CASC2D models. The results of final computer runs are presented in Appendix D.

## Summary

In setting up the models in this comparison study, the options in the HEC-1 model that most closely represented the components used in the CASC2D model were selected. However, the manner or solution techniques in which the equations or functions are applied in HEC-1 and CASC2D are slightly different. For example, in the Green-Ampt infiltration function component, the HEC-1 version allows for an initial loss parameter to be input for each sub-basin area. This is not available in the CASC2D version, but could be indirectly simulated by defining a depression storage value for each grid cell. Another example is the distribution of rainfall over the watershed. The CASC2D model uses an interpolation scheme based on the inverse distance squared from the cell to the rain gauges, while the HEC-1 model uses a weighting factor for each rain gauge based upon applying a Theissen-Polygon method to the sub-basin area. Also, the representation of cross sections for the channel routing component is different between the models. HEC-1 models the average cross section for a reach with an eight-point station-elevation scheme that includes both overbanks and the main channel and allows a different roughness value for each of the three sections of the total cross section. In the CASC2D model, the channel cross section is assumed to be rectangular and it lies in the middle of a square channel element. Any given channel path, identified by a series of elements through which it passes, must have a constant width, depth, and roughness. For each channel element, there can be channel flow restricted to the channel width and an overland (i.e., overbank or floodplain) flow when overflow from the channel occurs.

Because of the inherent differences in the solution techniques used by the two models for solving the equations for the infiltration and channel routing components, slight differences in simulation results are to be expected. However, the fact that the methodologies (i.e., lumped versus distributed) used for solving the overland flow routing component were totally different was the principal reason for making this comparison study. Major differences in the results predicted using the two selected models are thought to be due primarily to the overland flow routing components.

The channel routing solution technique is thought to be the greatest limitation of the CASC2D model for two reasons. First, as noted in the discussion of the simulation results (Appendix D), the times to peak values predicted by the model were consistently too early as compared to observed data. The use of a rectangular cross-section shape equivalent to bank-full channel size causes higher values of hydraulic radius for depths less than bank-full stage, thus giving higher velocities and quicker travel times. Second, the overland flows and channel routing diffusive wave equations are solved using an explicit numerical solution technique causing the time-step to be restricted to ensure stability. Generally speaking, the more intense the rainfall, the steeper the watershed, and the smaller the grid size, the shorter the time-step. For too long a time-step, negative depth and/or friction slopes may be computed causing an error message to be printed and simulation to stop. Smaller time-steps are also required as the depth increases in the channel to ensure stability. This becomes a severe limitation for simulating high-intensity, short-duration storm events. The model could not be used for a third part of this study planned to simulate a synthetic design storm of 10-year frequency and 6-hour duration equivalent to 6 in. of total rainfall using a Huff First Quartile Rainfall Distribution. During simulation, flow rates quickly reached values greater than bank-full (i.e., approximately 3,500 cfs) and stability restrictions caused the model to quit even with a small time-step of 5 seconds. Therefore, the third part, simulation of synthetic design storms, could not be completed for comparison purposes using the current version of the CASC2D model.

In 1987, HEC-1 was used to calibrate Snyder's unit hydrograph coefficients for the Goodwin Creek watershed (FTN Associates, Ltd., 1987). They used a total of 10 sub-basins and 13 storms in their analysis. Five of the storms that had nearly uniform rainfall over the watershed were selected for calibration purposes and the other storms were used for verification. These values of Snyder's coefficients, $C_p$=0.843 and $C_t$=0.90, were also used in this study and appear to work reasonably well. Computed lag time values were adjusted to the selected time-step (duration) of 2 minutes for simulation purposes. This value of lag time was used with the SCS unit hydrograph method in HEC-1.

Since the beginning of this study, new research and development of the CASC2D model are underway. One version of CASC2D contains a soil moisture accounting routing and is interfaced with the GRASS GIS to make it a continuous simulation model.[1] Another investigation is working on a version using a Holly-Priessman implicit numerical technique for the channel

---

[1] Personal Communication from Dr. Bahram Saghafian, 7 June 1993, Construction Engineering Research Laboratory, Champaign, IL.

routing component.[1] Coordination is ongoing by WES to have these new versions tested and verified and then combined into a working comprehensive model that will handle a variety of hydrologic modeling problems.

Future development will also include the addition of upland sediment yield and overland and channel sediment transport routines. This will allow the user to estimate sediment loads from various upland land use changes to assist in the design of sediment and erosion control structures. CASC2D has the ability to use rainfall data from a weather radar system. As better radar rainfall data become available, they will enhance the desirability and use of this model in the future.

## Conclusions

Based upon the results of the observed and hypothetical storm events simulated for the Goodwin Creek watershed, the following conclusions can be made:

*a.* In the case where there is accurate spatial data representation of the watershed variability in soils and land use, a distributed model will simulate the true shape, rate of rise, and volume of the streamflow runoff hydrograph more closely than the lumped unit hydrograph methods.

*b.* In the case where sufficient sub-basin stream gauge data are available for calibration purposes, the lumped unit hydrograph models such as HEC-1 can reproduce the observed hydrograph reasonably well.

*c.* The lumped models rely heavily on sub-basin stream gauge data to adequately simulate the observed hydrograph; however, CASC2D can simulate adequately as long as accurate spatial data are available. If accurate spatial data and sub-basin stream gauge data are both lacking, then both models (i.e., lumped or distributed) may produce questionable results.

*d.* Since the distributive model CASC2D consistently produced more realistic results in terms of hydrograph shape and volume of runoff, it offers more flexibility, when performing sediment studies, than the lumped unit hydrograph models. This will be especially true when evaluating the effects of specific land use changes or best agricultural management practices on erosion and sediment control within the watershed.

---

[1] Personal Communication from Dr. Fred Ogden, 11 June 1993, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City.

*e.* In this study, a GIS database had already been developed. In the case where a GIS database does not exist, a decision will have to be made as to whether an intensive stream gauging operation is more cost effective than developing data in a GIS. As time goes by, more GIS information will be available for a low cost. This should help to facilitate the development of a specific watershed GIS database and thus help to reduce the amount of stream gauge data needed. Once a GIS database is developed, a distributed model will be no more difficult to set up than a lumped model. In the event that a lumped model is still desired, the GIS data will help estimate the unit hydrograph and infiltration parameters with more accuracy than traditional methods.

## Recommendations

The principal objective of this study was to evaluate the watershed hydrology model, CASC2D, for purposes of application to ungauged watersheds. The simulation results from this study show that the CASC2D model will produce adequate results for design purposes with a limited amount of gauge data. GIS databases are currently being developed for most of the watersheds located in north Mississippi that will be part of the DEC Project. Because less sub-basin stream gauge data need to be collected for use with a distributive model than a lumped model, it is recommended that the CASC2D model be used as an aid in the design and evaluation of streambank erosion and grade control structures in the future.

It is recommended that the channel routing component of the CASC2D model be revised as soon as possible to more realistically represent the channel cross sections to improve the timing of the simulated runoff hydrographs. It is also recommended that the channel routing component be uncoupled or separated from the overbank routing component for modeling overbank flows. This would allow other numerical channel routing techniques to be evaluated and perhaps eliminate the stability problems caused by time-steps that are too long. For design purposes of the erosion control measures, the model must be able to handle high-intensity, short-duration storm events. It is also recommended in the near future that the CASC2D model be enhanced by adding sediment yield and transport subroutines for both the overland flow and channel routing components. This will allow evaluation of planned watershed best management practices and erosion or sediment control structures.

For future use of the HEC-1 model with Snyder's unit hydrograph method on streams located in north Mississippi that have similar watershed characteristics to Goodwin Creek, it is recommended that the values used in this study will be good starting approximations for calibrations or simulations.

# 6 Performance of Hydraulic Structures

## Hydraulic Structures Monitoring

### Purpose and scope

The purpose of this work area is to collect field data on selected structures including riprap bank stabilization structures to evaluate hydraulic performance. The grade control structures would be selected on basis of special features to include high-drop, low-drop, significant upstream flow constriction, limited upstream flow constriction, free flow, and submerged flow. The bank stabilization structures would be selected jointly with Vicksburg District personnel. The grade control structures would be instrumented to collect data to evaluate discharge coefficients, energy dissipation, flow velocity distribution, and effects of submergence on performance. All grade control structures and riprap bank stabilization measures in each watershed would be visually monitored and problem areas identified. Visual monitoring would consist of aerial videoing and ground inspections.

### Description of work for FY 93

A low-drop structure on Long Creek and a high-drop on Hotophia Creek were selected to instrument in FY 92 to include water surface elevation recorders upstream and downstream of the weir and a cable way for measuring flow velocities in the upstream approach. Also during this period three low-drop structures on Worsham Creek and one high-drop on Burney Branch Creek were instrumented with recording water surface gauges placed upstream and downstream of the weir. Four additional low-drop structures were instrumented early in FY 93 for a total of eight instrumented structures. Therefore 8 of the 22 long-term monitoring sites that CSU is surveying and monitoring for channel response contain an instrumented structure. Instrumentation of riprap bank stabilization installations planned for FY 93 was not accomplished because the Vicksburg District decided that visual inspection of selected bank stabilization sites would meet their requirements. The visual inspections will be conducted beginning in FY 94. Aerial videos of the main channel and major tributaries were made, and the general observations from these videos on the existing condition of grade control and bank stabilization structures are reported in Chapter 8. A visual inspection of 55 grade control structures was conducted by CSU, and only a summary will be presented in this report since the detailed results are reported in a separate contract report (Watson, Abt, and Hogan 1993).

### Status and conclusions

**FY 93 progress.** The work effort during this reporting period for this task was directed at analyzing model studies data (WET 1990; Abt et al. 1991) and using the results to develop discharge ratings at the structures using field stage data. Effective evaluation of channel response for the monitoring sites is contingent on defining the hydrology that occurs during the evaluation period. Therefore, at least initially, discharge ratings will be estimated by measuring the stages at those sites that have structures and estimating the discharge using model discharge coefficients.

**Long Creek low-drop.** The low-drop structure on Long Creek was instrumented in FY 92 for the purpose of obtaining field data to correlate with model data, particularly with regard to discharge coefficients. Two physical model studies were conducted at CSU to evaluate the performance of the structure under flow conditions not investigated by Little and Murphey (1982), to determine if cost reduction modifications to the structure were feasible (WET 1990), and to develop riprap sizing criteria for the ARS-type low-drop structures (Abt et al. 1991).

Data analysis of the two CSU physical model studies (WET 1990; Abt et al. 1991) indicated that the discharge coefficient was constant up to a submergence of 0.80. Submergence is defined as the ratio of the difference between the tailwater elevation and the weir crest elevation $t'$ and critical depth $Y_c$, i.e., $t'/Y_c$. See Figure 115 for a definition sketch. The discharge coefficient $C_d$ is defined by:

$$C_d = \frac{Q}{(B + zH_1)H_t^{3/2}} \qquad (1)$$

where:

$Q$ = discharge, cfs

$B$ = width of weir, ft

$z$ = lateral side slope of weir, $z/1$ ft vertical

$H_t$ = upstream flow depth, ft

$H_t$ = total upstream head = $H_1 + V^2/2g$

$V$ = upstream average flow velocity, ft/sec

$g$ = acceleration of gravity, ft/sec$^2$

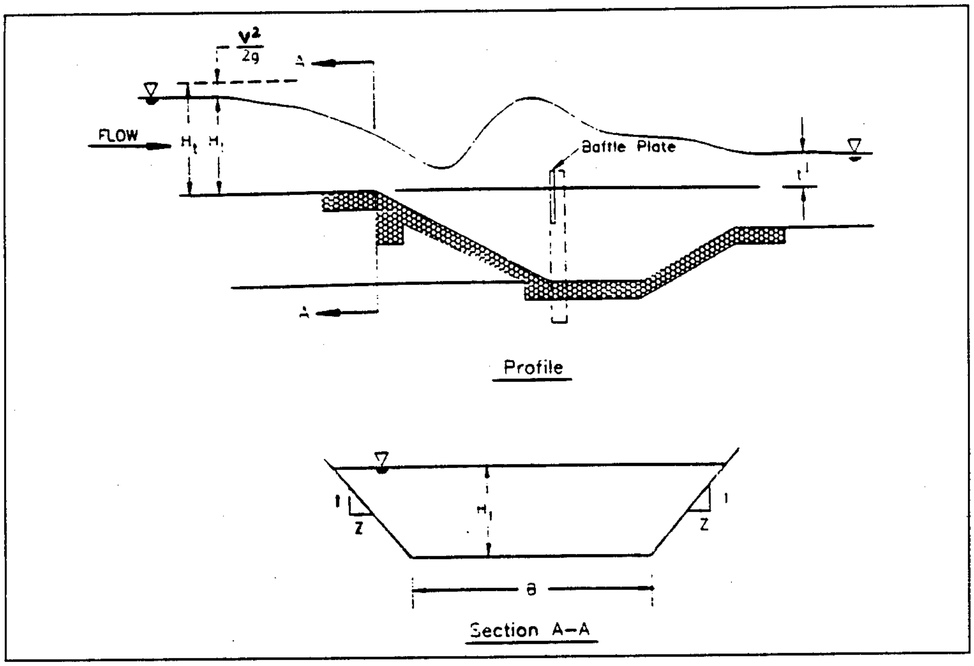



Figure 115. Definition sketch

Figure 116 shows the relationship between the discharge coefficient and submergence for the CSU data. Linear regression curve fitting techniques were applied to the data with the following results:

$$C_d = 2.5 \qquad 0 < \frac{t'}{Y_c} < 0.8 \tag{2}$$

$$C_d = 2.354\left[\frac{t'}{Y_c}\right]^{-0.328} \qquad 0.8 < \frac{t'}{Y_c} < 1.4 \tag{3}$$

Using these discharge coefficients with the geometry of the cross section at the upstream and downstream gauge locations, a stage-discharge curve was developed for the head-and tailwater cross sections. Linear regression was applied to the stage-discharge curves to develop equations that were used to translate the stage hydrographs recorded at the site to discharge hydrographs (Figures 117-119).

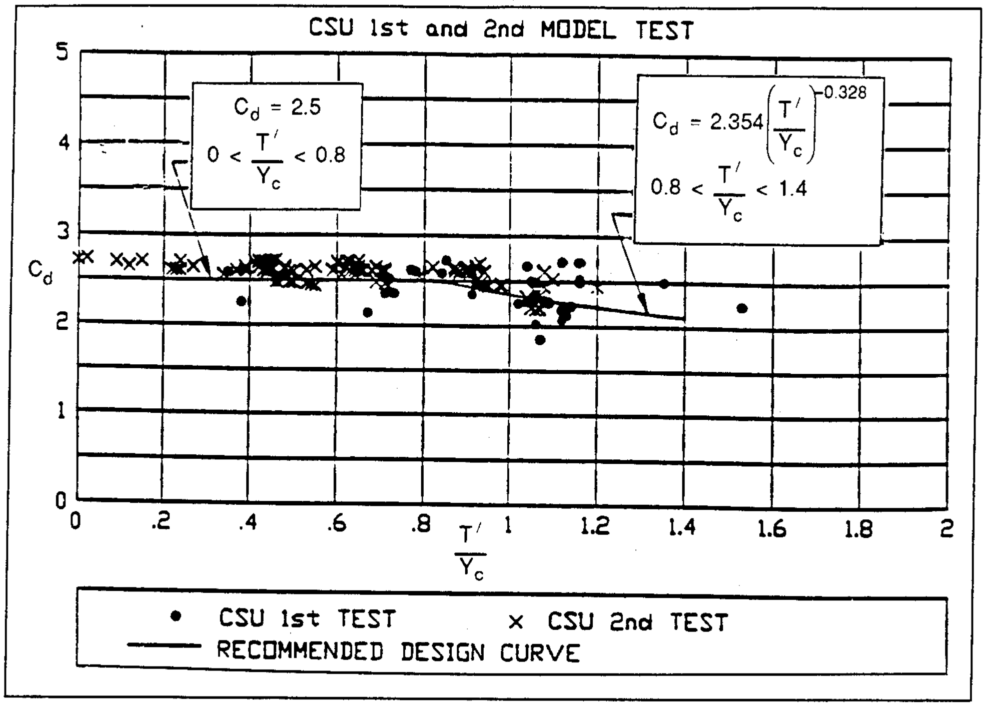



Figure 116. Discharge coefficients model data

**Hotophia Creek high-drop.** Design guidance for high-drop structures in the DEC project is given in the SCS *Engineer Handbook*, Section 11, "Drop Spillways" (SCS, no date), and is referred to as a Type C high-drop structure. The discharge coefficient $C_d$ is defined by Equation 1 with the weir side slope set to zero because the walls are vertical on this type of structure. Therefore, the equation for $C_d$ becomes:

$$C_d = \frac{Q}{BH_t^{3/2}} \tag{4}$$

SCS recommends a design value of $C_d = 3.1$ for free-flow conditions (unsubmerged) with the flow approaching the weir subcritical, i.e., the depth of flow is greater than critical depth. SCS states in the Handbook that the recommended value is based on very little data but they believe it "is sufficiently conservative to have made allowance for possible end contractions and other indeterminate factors that effect the discharge capacity."

The SCS Handbook provides a procedure for computing the discharge for submerged flow conditions. The procedure was developed from test results of submerged flow over several types of weirs and earth embankments and not from experimental data on submerged flow over the Type C structure. The Handbook states " ... precise results should not be expected from submergence computations." The ratio of the submerged discharge $Q_s$ to the free discharge $Q_f$ as a function of the submergence $H_2/H_1$ is shown in Figure 120.

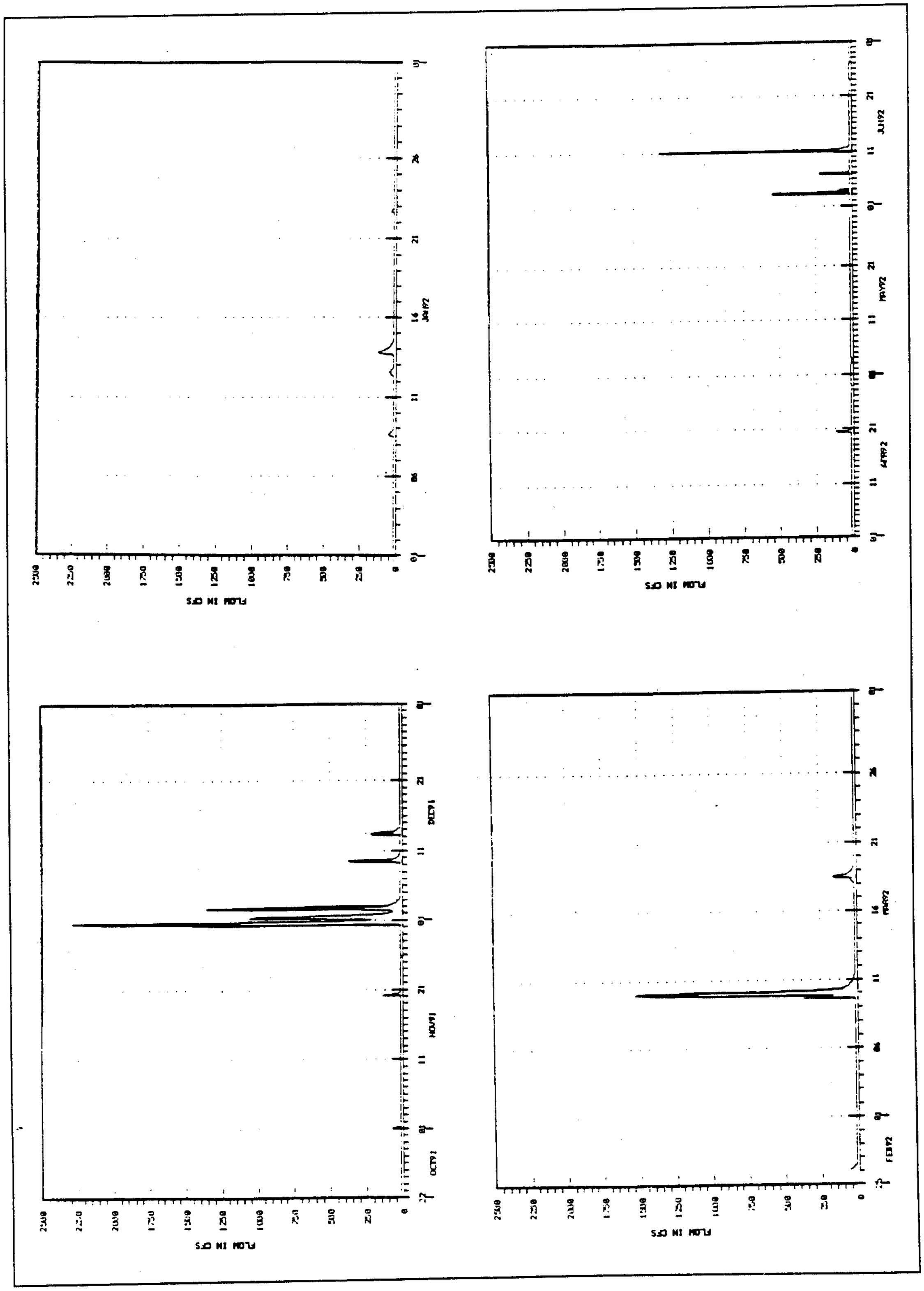


Figure 117. Discharge hydrograph, Long Creek (October 1991-June 1992)

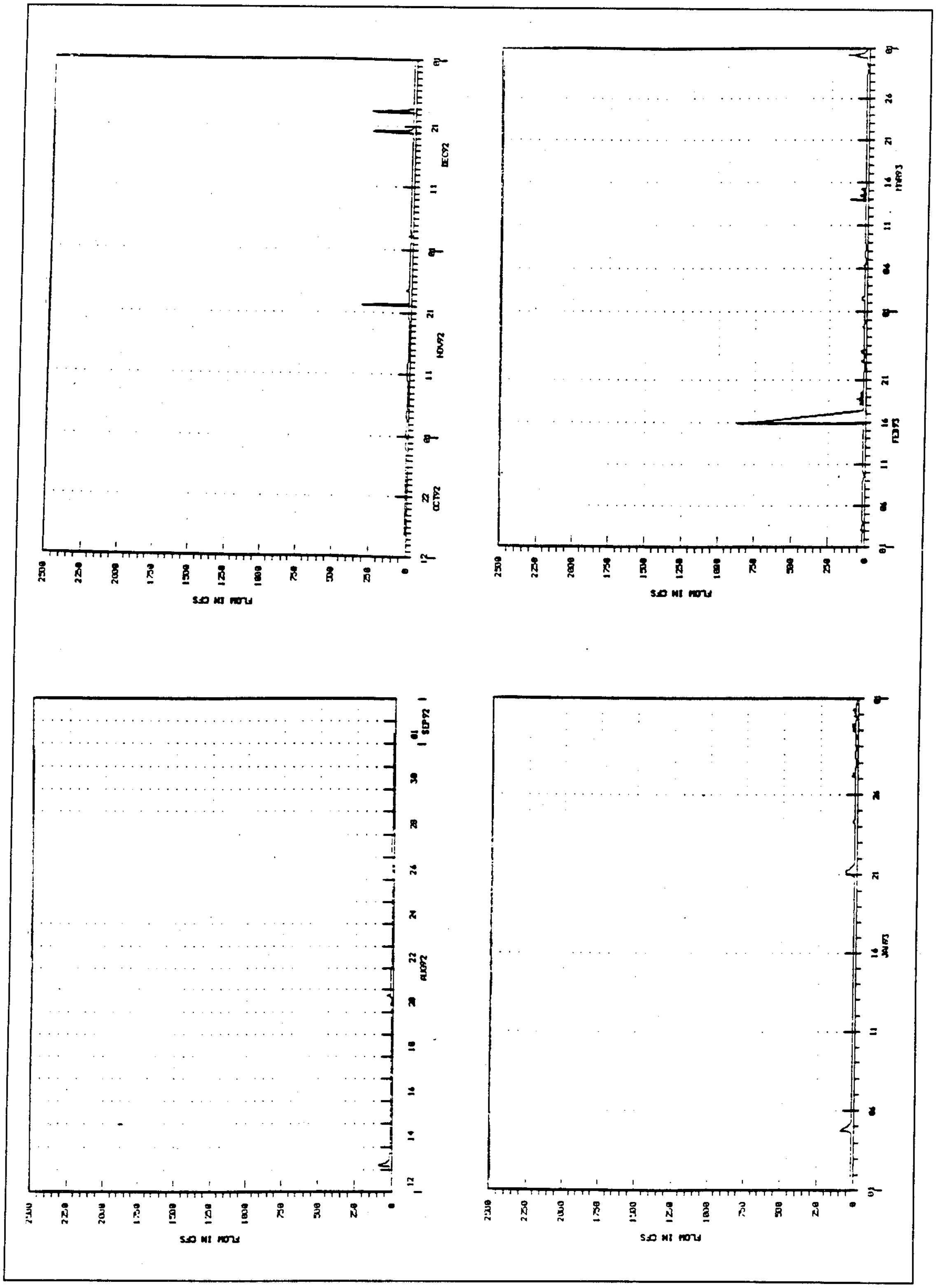


Figure 118. Discharge hydrograph, Long Creek (August 1992-March 1993)

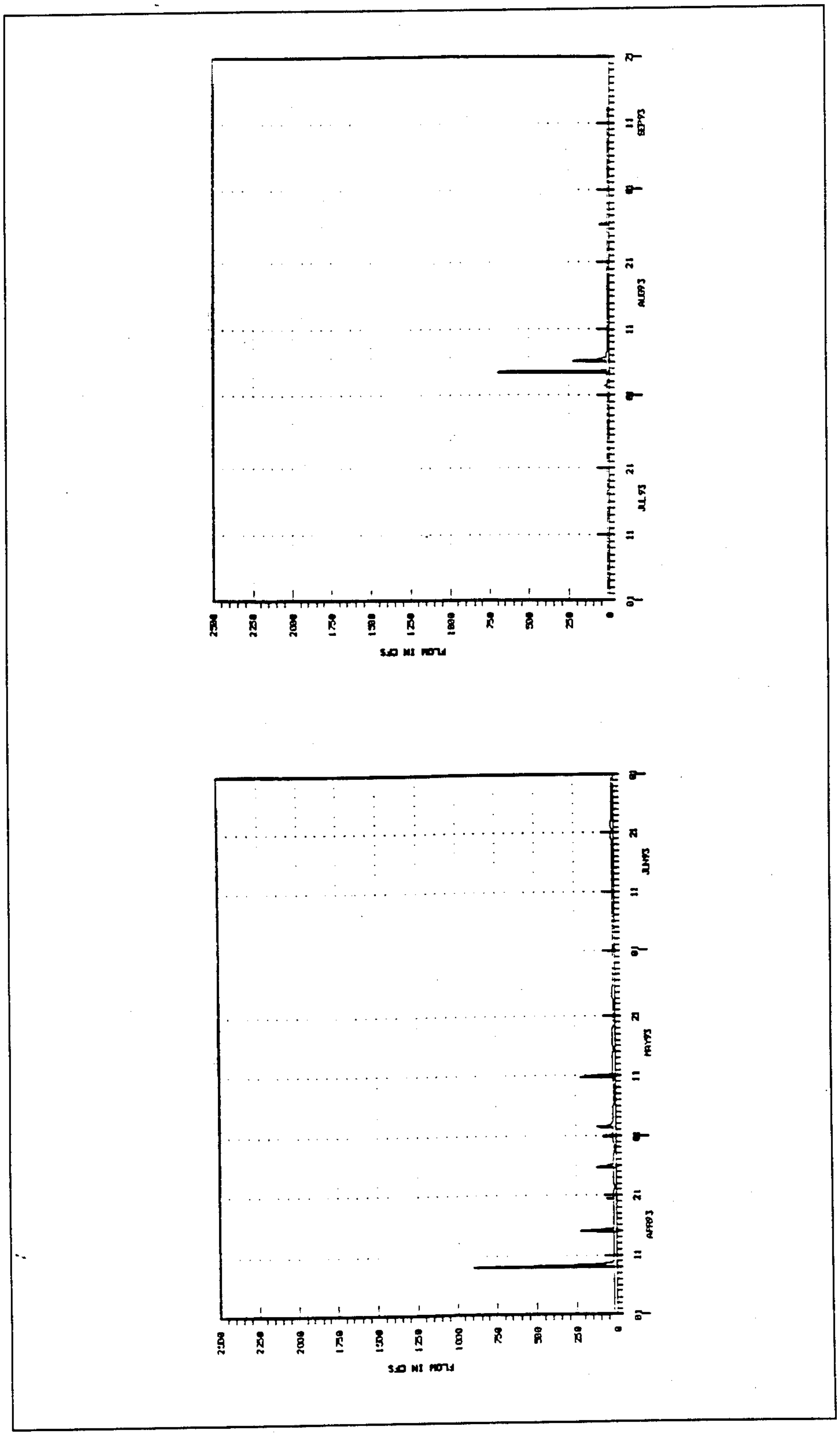


Figure 119. Discharge hydrograph, Long Creek (April 1993-September 1993)

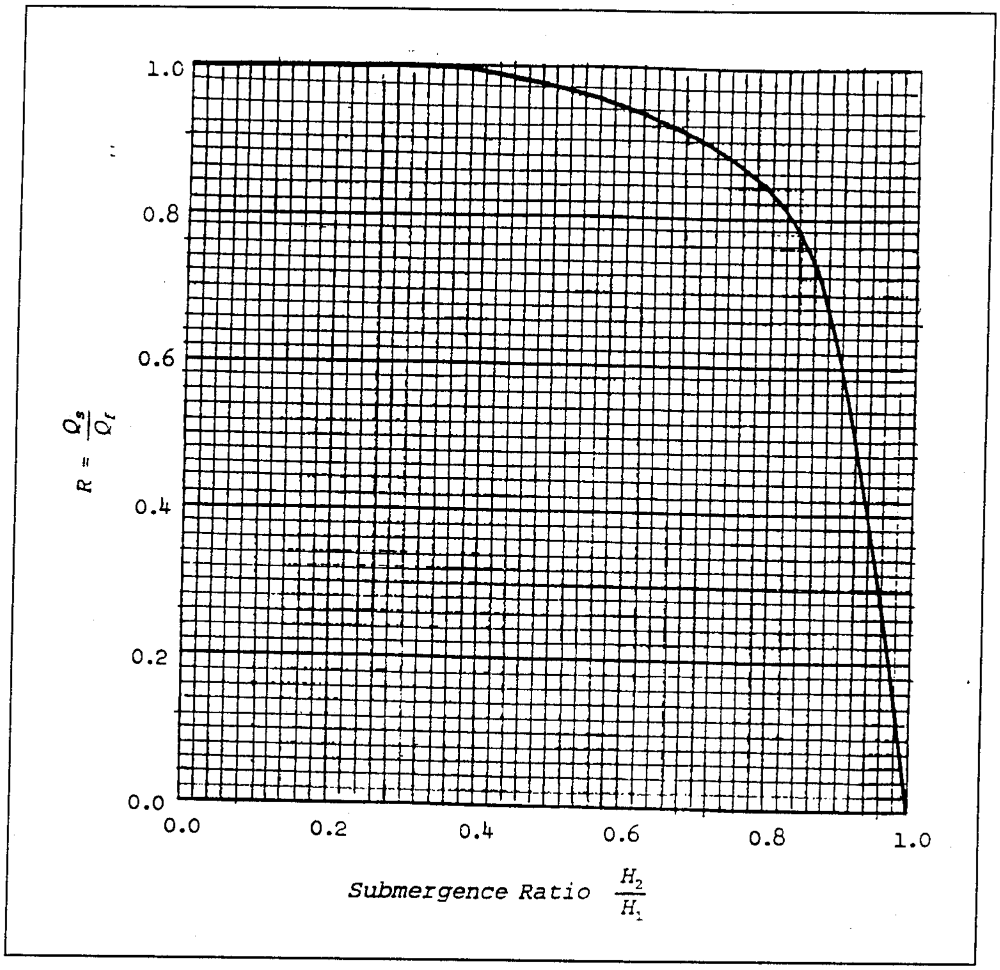



Figure 120. Hotophia Creek high-drop, effects of submergence

In a manner similar to that for the Long Creek low-drop structure, relationships were developed that were used to translate the stage hydrographs recorded at the site to discharge hydrographs (Figures 121 and 122).

## Visual Inspection of DEC Drop Structures

Field inspections were made in June 1993 of the high-drop and low-drop grade control structures in the DEC project by CSU personnel. A structure evaluation form for each structure was prepared and is presented in Watson, Abt, and Hogan (1993). The report contains a series of photographic slides of each structure and approximately 5 hours of narrated and annotated video tape showing the 1993 field conditions. The inspection report is summarized in this section for convenience.

The six most common problems observed are as follows. The percentage shown for each problem denotes the percentage of total structures in which the problem occurred:

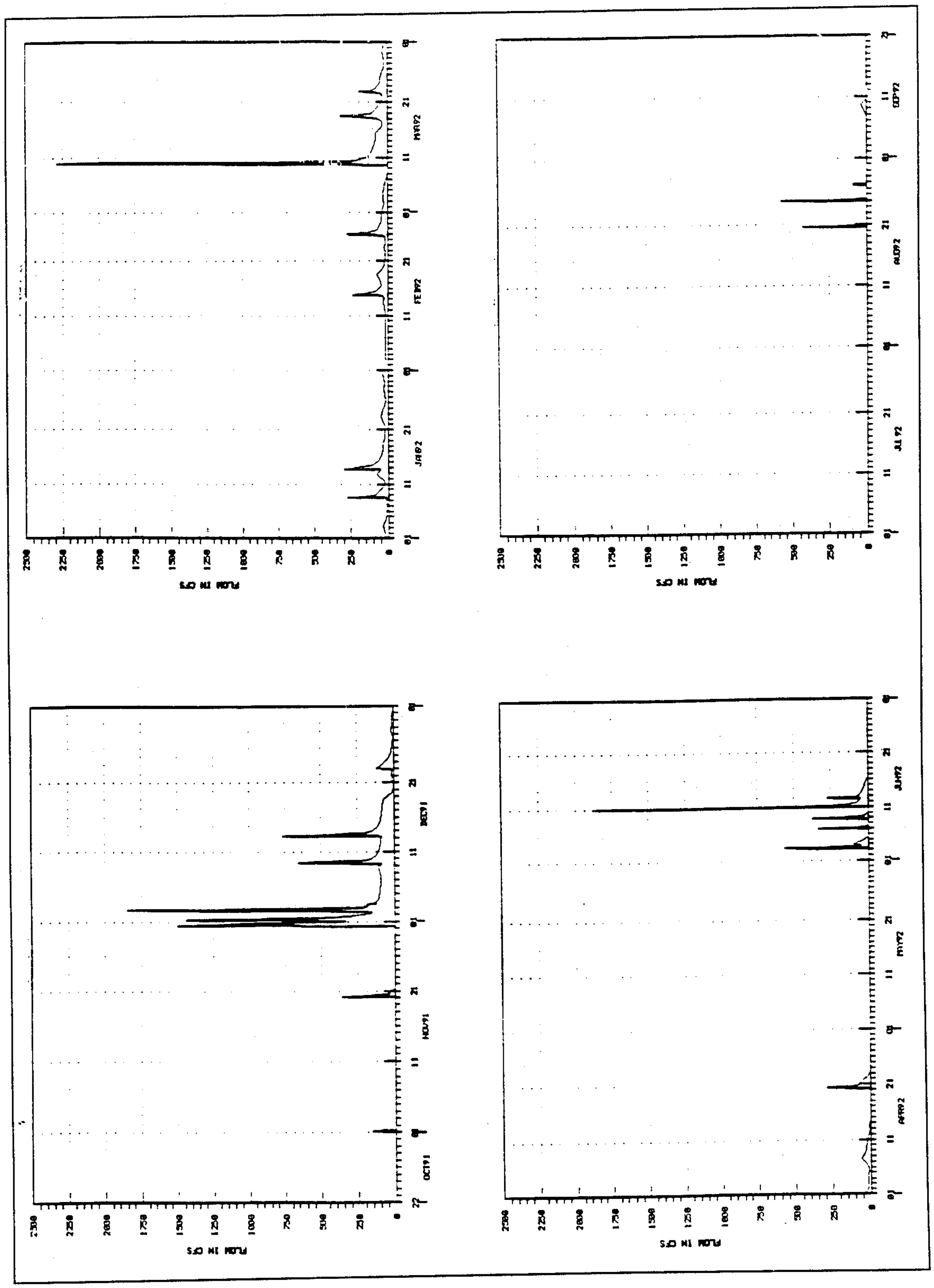


Figure 121. Discharge hydrographs, Hotophia Creek (October 1991-September 1992)

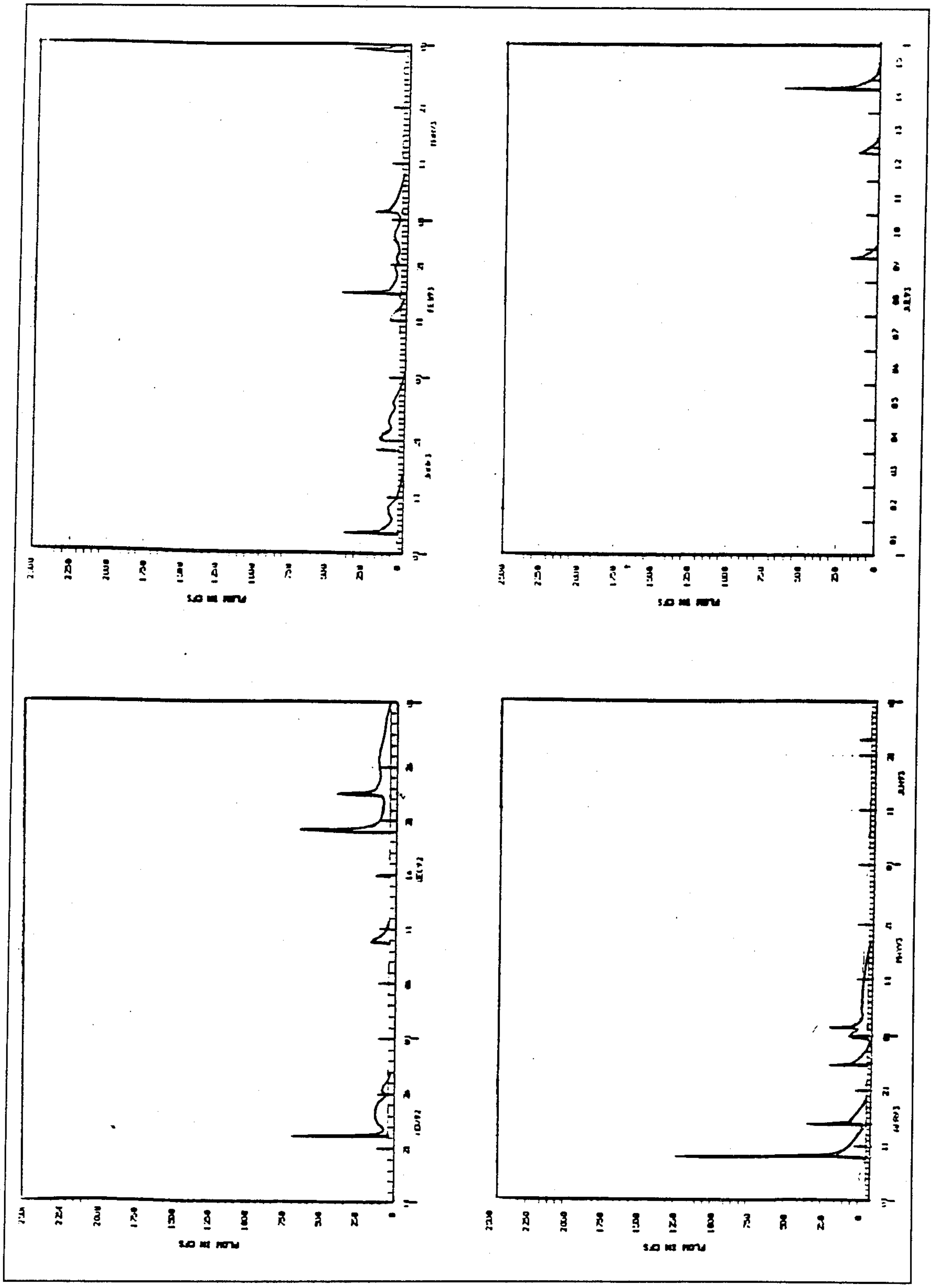


Figure 122. Discharge hydrographs, Hotophia Creek (November 1992-July 1993)

*a.* Riprap is displaced from the face of the weir (41 percent).

*b.* The channel bank upstream or downstream of the structure is failing (37 percent).

*c.* Bank erosion or piping beneath the riprap is occurring caused by overbank drainage (24 percent).

*d.* Riprap is launching at the upstream or downstream apron (28 percent).

*e.* Severe headcutting is migrating into the basin (17 percent).

*f.* Woody vegetation has become established in the upstream or downstream apron, and is impairing the conveyance or the weir unit discharge of the structure (19 percent).

In addition to identifying the types of problems and making recommendations for resolving the problems, CSU assigned a priority to each structure as follows:

*a.* Category 1 structures are under an imminent threat of loss of function.

*b.* Category 2 structures have problems that should be resolved.

*c.* Category 3 structures have no significant problems.

Table 28 summarizes the types of problems (problem types correspond to subparagraphs *a-f*) and the category for each structure evaluated. Four structures are in category 1, 32 in category 2, and 19 in category 3.

## Low-Drop Structure Model Testing

### Background

Low-drop grade control structures have been used to arrest erosion in incising channels. The concept of the drop structure was originally developed based on an equivalent energy approach. Numerous variations and types of these structures have been constructed both in model studies and in prototype locations.

Sheet-pile grade control structures have been used in the DEC Project to arrest erosion due to headcutting. These structures consist of an upstream approach transition section from the natural channel to the sheet-pile weir, a vertical drop into a riprap stilling basin to dissipate the energy, and a downstream transition. The sheet-pile and riprap approach to low-drop design is an economical alternative to a concrete structure and apron.

## Purpose and approach

Current design criteria for a sheet-pile grade control structure limit the drop height to 6 ft. The limits are partially based on hydraulic limitations and partially on structural design limitations of the vertical placement of the sheet-pile cutoff. Due to the potential for cost savings with a sheet-pile structure as opposed to a concrete drop structure, a reevaluation of structural design components by the Vicksburg District verified the constructability of a higher drop (10 ft). However, the hydraulic performance and riprap design criteria were not heretofore tested for the ARS-type drop structure nor design criteria developed for sheet-pile riprap drops greater than 6 ft.

Drop structures have typically been classified either as low- or high-drops according to a ratio of drop height $H$ to critical depth $Y_c$. Low-drops are classified as those with an $H/Y_c$ less than or equal to 1. The proposed drop height of 10 ft would change the classification of drop structure for the same design discharge and critical depth of 6 ft by exceeding 1. Therefore, based on the differences between the actual drop classification and the proposed design criteria, it was necessary to study the hydraulic performance of this structure.

The purpose of this model study was to modify and/or develop guidance regarding both the hydraulic design and the stable riprap design to accommodate a 10-ft drop structure with an $H/Y_c$ greater than 1. The objective of the study was to determine the feasibility of using a higher drop and develop design guidance pertaining to the higher drop. A 1:12-scale physical model was used to investigate the proposed sheet-pile grade control structure with a 10-ft drop. Details of the study are given in Appendix E of this report.

## Test and results

The same stilling basin design, type 1, was used throughout testing. Two different weir designs were used: a trapezoidal weir, type 1 design weir, and a rectangular weir, type 2 design weir. The small stone remained constant in the described locations throughout testing. In the areas containing large stone, however, two gradations of stone, R1500 and R2200, were tested. In one series of tests, the R1500 gradation was grouted to test stability.

The small section of riprap placed immediately upstream of the weir was grouted due to riprap failure occurring for low tailwater conditions. During testing it was found that velocities in that area exceeded 16 fps with a discharge of 4,000 cfs. Riprap failure was defined as the condition where sufficient stone displacement occurred to expose the underlying filter cloth.

A total of 93 tests were conducted for this study. A detailed discussion of test results is given in Appendix E of this report.

# Conclusions and Recommendations

## Hydraulic conditions

In general, velocities in the approach channel increased with increasing discharges. Velocities in the exit channel increased with decreasing submergence. To ensure stability of the approach and exit channels, more consideration should be given to the velocities in these areas.

Tests performed using the type 2 weir seemed to indicate that energy dissipation was more confined to the stilling basin than during the trapezoidal weir tests. While this led to an earlier failure of the stone below the weir, velocities were lower in the exit channel. Furthermore, upstream of the weir, the water was pooled behind the weir causing lower velocities in the approach. Optimum hydraulic performance would dictate the transition to the weir should not be an expansion, but a more controlled contraction of the channel. However, due to the limitations regarding stone placement on a 2.5V:1H slope in the upstream approach this may not be feasible. For detailed discussion of findings, see Appendix E of this report.

## Riprap stability

In areas where loose stone was grouted in the model, both upstream of the weir and in the stilling basin, no failure of the stone occurred. Since there is a risk of mass failure due to uplift, consideration of this option should be based on field success.

Due to the instability of the small stone at the higher discharge during the grouted basin tests, consideration should be given to either extending the grouted section of rock or increasing the small stone size. These tests also verified the need to study the effects of riprap stability with discharges greater than the design.

A properly designed granular filter of some type should be provided beneath the graded riprap to prevent piping through voids in the rock.

In summary, the objective of this study was accomplished by providing an equation for design of stone in a 10-ft drop structure. However, some caution should be exhibited in the implementation of the structure in a movable bed channel, especially as regards the upstream approach and the downstream exit channel. A detailed discussion of conclusions and recommendations is given in Appendix E of this report.

# 7 Design Tools, Design Procedure for Grade Control Structures, Byhalia Creek Case Study

## Data Requirements

The six parameters that the hydraulic design engineer deals with are (a) width, (b) depth, (c) slope, (d) hydraulic roughness, (e) bank line migration, and (f) planform. The first four are the focus of this chapter. They are referred to as channel dimensions. The case study presented here demonstrates the application of a design procedure that is presently being developed in the Flood Control Channels Research Program. It is proposed here for testing and evaluation of Byhalia Creek in the DEC. The calculations that are required have been packaged in the computer program "Hydraulic Design of Channels," SAM (Thomas et al., in preparation). The data required to conduct the study are

*a.* Thalwag profile data.

*b.* HEC-2 input data file(s).

*c.* Hydrologic data.

*d.* Bed gradation data.

## Summary

This procedure follows the same tasks that were presented in Chapter 10 of the Fiscal Year 1992 Report (Raphelt et al. 1993). Specific points are outlined below, and the tasks are presented in detail in the following paragraphs. The data files shown as examples are the actual data files used in the study.

a. A drainage basin map and network of stream channels was developed as shown in Figure 123.

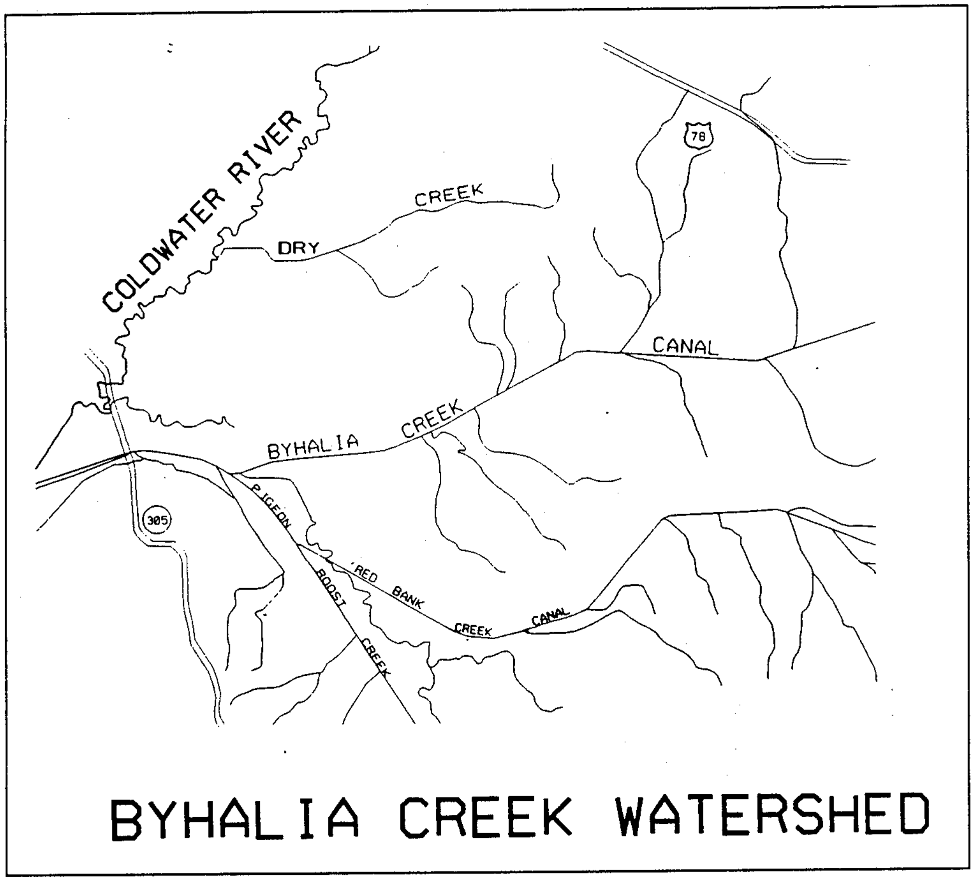



Figure 123. Drainage basin map with network of stream channels

b. The thalweg profile (elevation versus station) was plotted, and a thalweg slope was calculated for each natural break in that gradient. That process identified four reaches (Figure 124).

c. Design channel cross section type B (Figure 45, page 81, of the Fiscal Year 1992 Report) was selected for this creek.

d. HEC-2 was run for existing conditions using the 2-year flood for the channel-forming discharge.

e. A reference reach was selected using output from the HEC-2 run and information from a site inspection of Byhalia Creek.

f. A representative section in the reference reach was chosen and normal depth calculations were run with SAM to get composite width, depth, and velocity values.

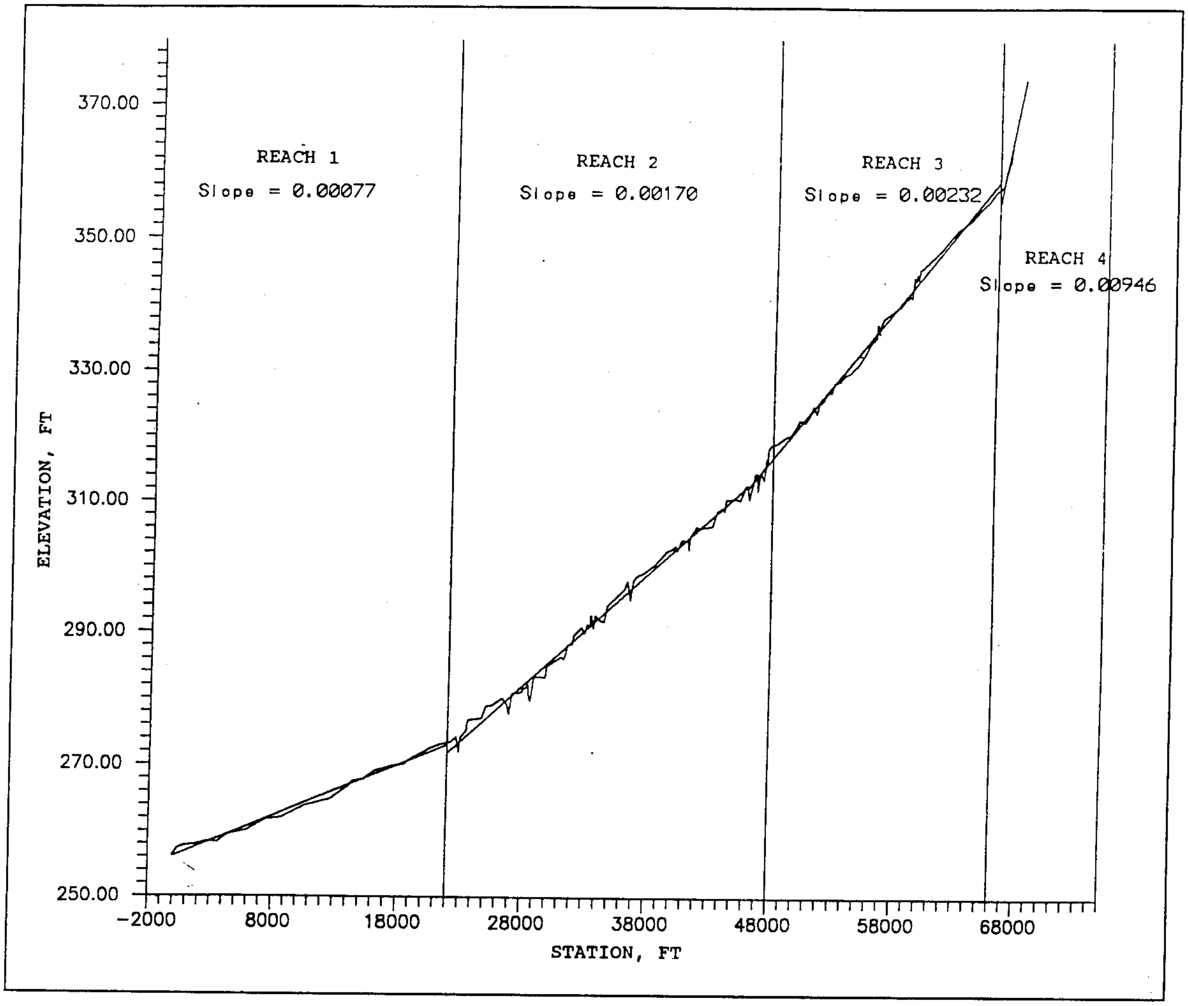



Figure 124. Average thalweg slopes by reach

*g*. SAM.sed was run to get the bed material concentration for the channel-forming discharge.

*h*. A stable width-slope relationship was calculated using the stable channel analytical method in SAM.

*i*. The bed material grain size and concentration from this reference reach were transferred to upstream reaches while reducing the flood peak discharge. The Copeland stable channel dimensions method within SAM was used to obtain a design graph of width versus slope for each reach.

*j*. For each of these reaches the normal depth option within SAM was used to calculate the effective width of the existing channel. This was used to get the stable design slope off the Copeland graphs.

*k*. The design was tested with HEC-2.

## Approach

The first task was to assemble data. HEC-2 data files were furnished by Vicksburg District for existing conditions. These data were used without modification except in the vicinity of the proposed drop structures. It was necessary to add three cross sections at each drop structure. These were inserted from the nearest measured cross section in the data set.

Hydrologic data were also furnished by Vicksburg District. The future runoff peaks are expected to be the same as historical rates. Consequently, the same frequencies were used for future as for historical conditions.

Sediment data were collected during the site reconnaissance visit. These data consist of several samples of the bed sediment.

The thalweg profile data from the Byhalia Creek survey were read into Grapher and the profile shown in Figure 124 was plotted. Breaks in the natural gradient are obvious. These data were fitted with four straight-line segments, and reach limits were set at each break point.

HEC-2 was run in the existing geometric model, and plots of the water surface profile, bed elevation, and bank heights were made with Grapher (Figure 125). Some data manipulation is necessary to bring the data files into Grapher, but the procedure is manageable. Moreover, there may be other graphical packages that can be used more directly. Grapher is mentioned because this analysis was done using that software package.

As a sensitivity test, three grade control structures were installed to see what effect they would have on the hydraulic parameters. The grade control structures were designed as sharp-crested weirs, with a crest length equal to the bottom width of the channel and a side slope of 1V:3H. SUMPO was used to get output variables for comparison, especially section number, depth, top width, slope, distance, velocity, shear, water surface slope, bed evaluation, and left and right bank heights. In this case it appeared that the design slope and grade control structures did not have much effect on the water surface slope.

In the existing conditions HEC-2 model, the 2-year flood just filled the channel in reach 1. Therefore, that reach was selected to be the reference reach, and the 2-year flood was selected to be the design discharge.

SAM.hyd was used to calculate the hydraulic parameters in the reference reach. These parameters are the velocity, width, depth, and slope for the 2-year flood. The input data file for SAM.hyd uses the X1 and GR records from cross-section 25 in the existing HEC-2 model. There were a number of cross sections in reach 1, but the velocity, width, and depth at cross-section 25 matched the average values for reach 1 as calculated in SAM.m95. Therefore, that cross section was used to calculate the sediment transport parameters for the entire range of water discharges in the sediment discharge rating table.

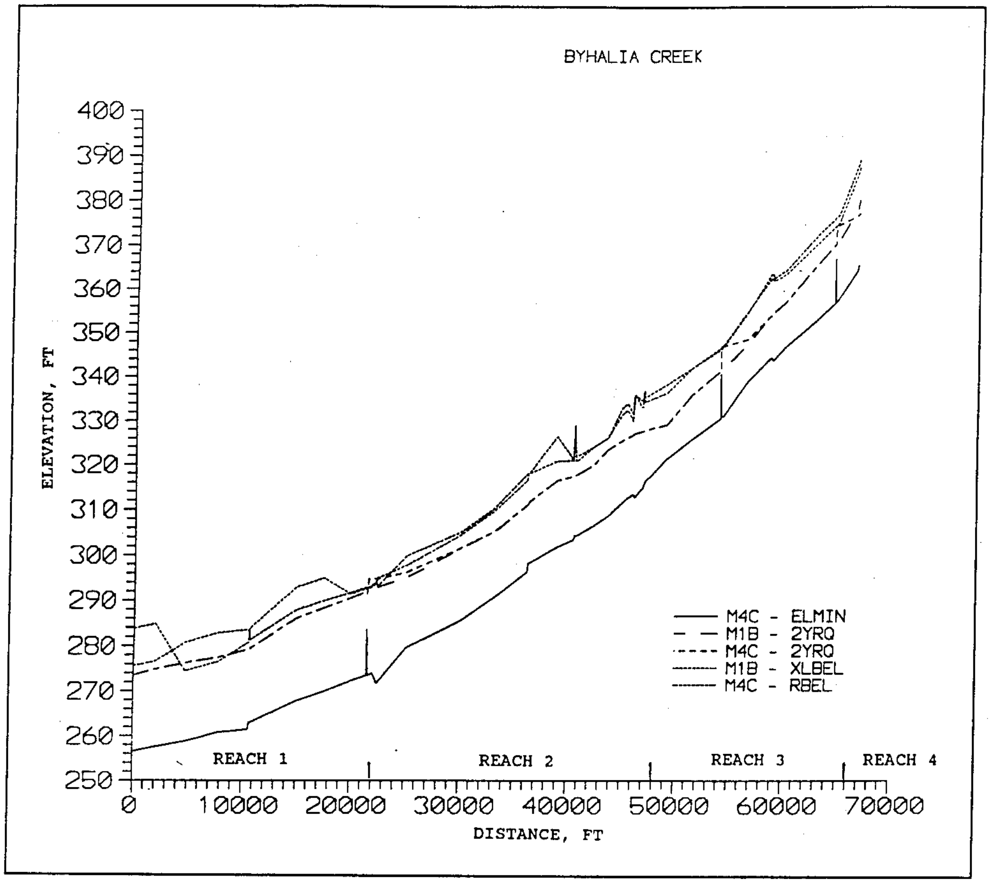



Figure 125. Water surface profile, bed and bank elevation plots

Except for the X1-GR records, the data records in SAM.hyd differ from HEC-2. The NE record prescribes Manning's equation for hydraulic roughness for all panels in the cross section, and the KN record prescribes the n value in each panel. Bed gradation for the channel is coded on PF and PFC records. The water discharges, energy slope, and water temperatures are coded on QW, ES, and WT records, respectively. The ZW record prescribes the DSS path name. The F# record defines 8-column fields, which aids in coding data.

This SAM calculation produces two output files: HYDRAULICS.OUT and SEDIMENT.IN. The SEDIMENT.IN file contains the parameters needed for the sediment transport calculations.

This task was designed to confirm a sediment transport function for the stable channel calculations. SAM.sed provides 13 sediment transport functions. They can be turned on or off by the YES/NO option on each record. Selection of the most appropriate function for this creek was aided by the SAM.aid code based on the water velocity, depth, width, slope, and the $D_{50}$ of the bed material in this reach.

SAM.aid predicted Yang, Yang $D_{50}$, and Laursen-Madden. None of these are available in the stable channel analytical method. That method is presently built around the Brownlie $D_{50}$ sediment transport function. Therefore, both the Yang and the Brownlie functions were made active. Results are shown in Figure 126. These results are sufficiently close together to adopt Brownlie for the calculations.

The channel-forming discharge is assumed to be the 2-year flood, based on work reported in FY 1992. The 2-year flood is 7,500 cfs along this reach.

The calculated bed material concentration for the channel-forming discharge is 743 mg/l. The $D_{50}$ size of that mixture is 0.46 mm. This concentration was adopted as the concentration of the bed material load for the 2-year flood peak.

The next task was to establish the hydrologic parameters for the design. The 2-year discharge of the reference reach, 7,500 cfs, was decreased as the drainage area decreased in reaches 2, 3, and 4. These values, shown in Figure 127, were furnished by Vicksburg District. Locations are identified by channel station.

The next task is to apply the Copeland method to calculate the stable channel dimensions. The concept is to design a channel that will transport a sediment concentration less than or equal to that concentration in the reference reach.

The GC record of the input data contains the design water discharge, the design bed material concentration in the water column, the valley slope, the slope of the left bank and the right bank of the channel, respectively, and the n value for the left and right banks of the channel, respectively. Water temperature is coded on the WT record. The PF/PFC records contain percent finer data for the bed surface in the reference reach. The gradation data are from the reference reach. It is important not to mix gradation curves since this method is based on the equilibrium condition.

The HYD.OUT file lists the physical and water properties and a table of stable channel width-depth-slopes for the given water discharge, sediment concentration, and particle size. DSS produced the graphs shown in Figure 128. The legend shows the water discharge and the corresponding channel station. The existing effective widths are plotted on the curves to show what the design slope should be to be in balance with the existing channel width. In reaches 2, 3 and 4, that slope is considerably less than the existing slope. However, this procedure resulted in very similar slopes from reach to reach on Byhalia Creek. An average value, 0.001 ft/ft, was selected except in reach 4 where the value was increased to 0.0015 ft/ft. The calculated slope in reach 4 is about 0.001, also. However, this is a short reach, and engineering judgement suggested the width can be reduced from existing conditions by placing some bank stabilization as required.

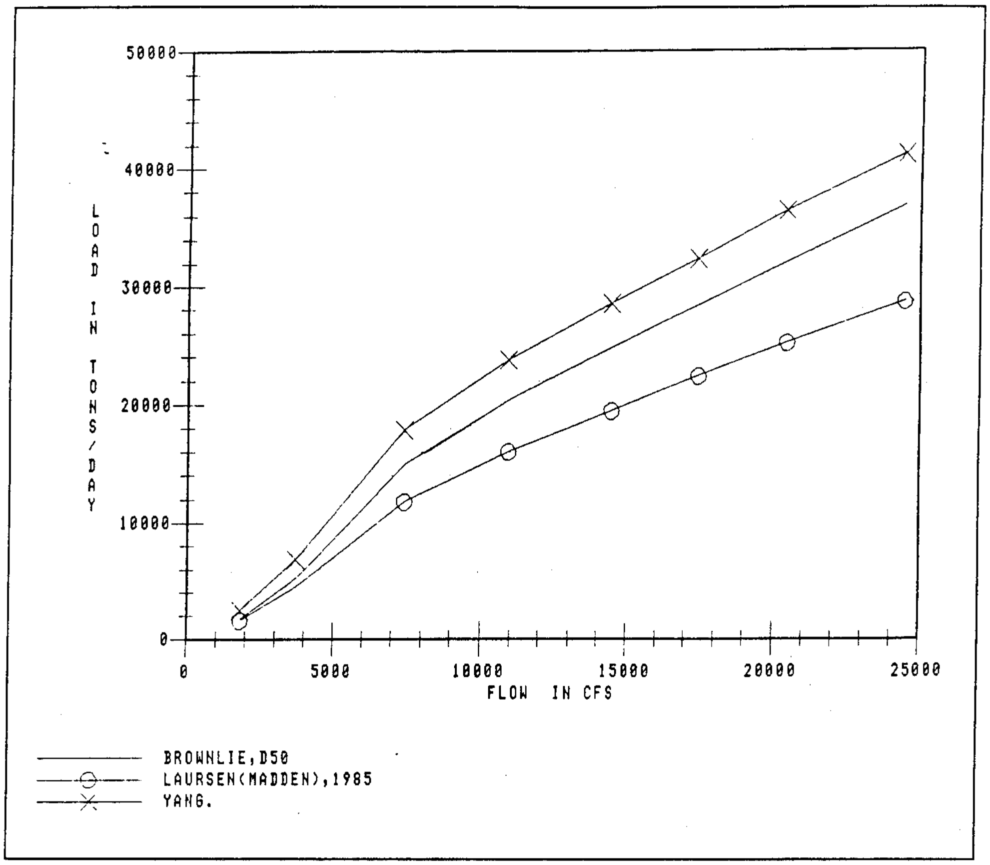



Figure 126. Transport function results

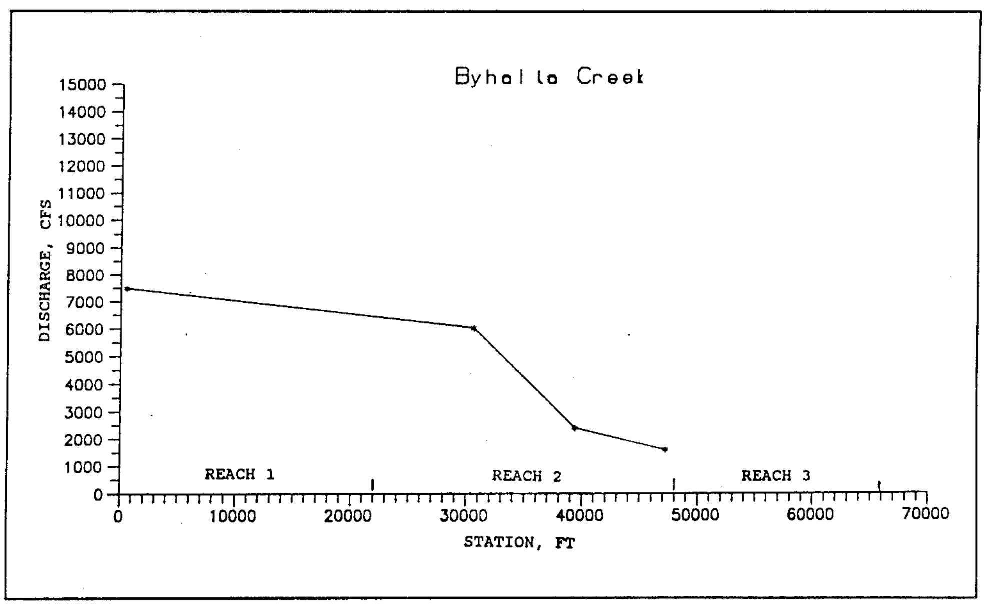



Figure 127. The 2-year discharge along Byhalia Creek

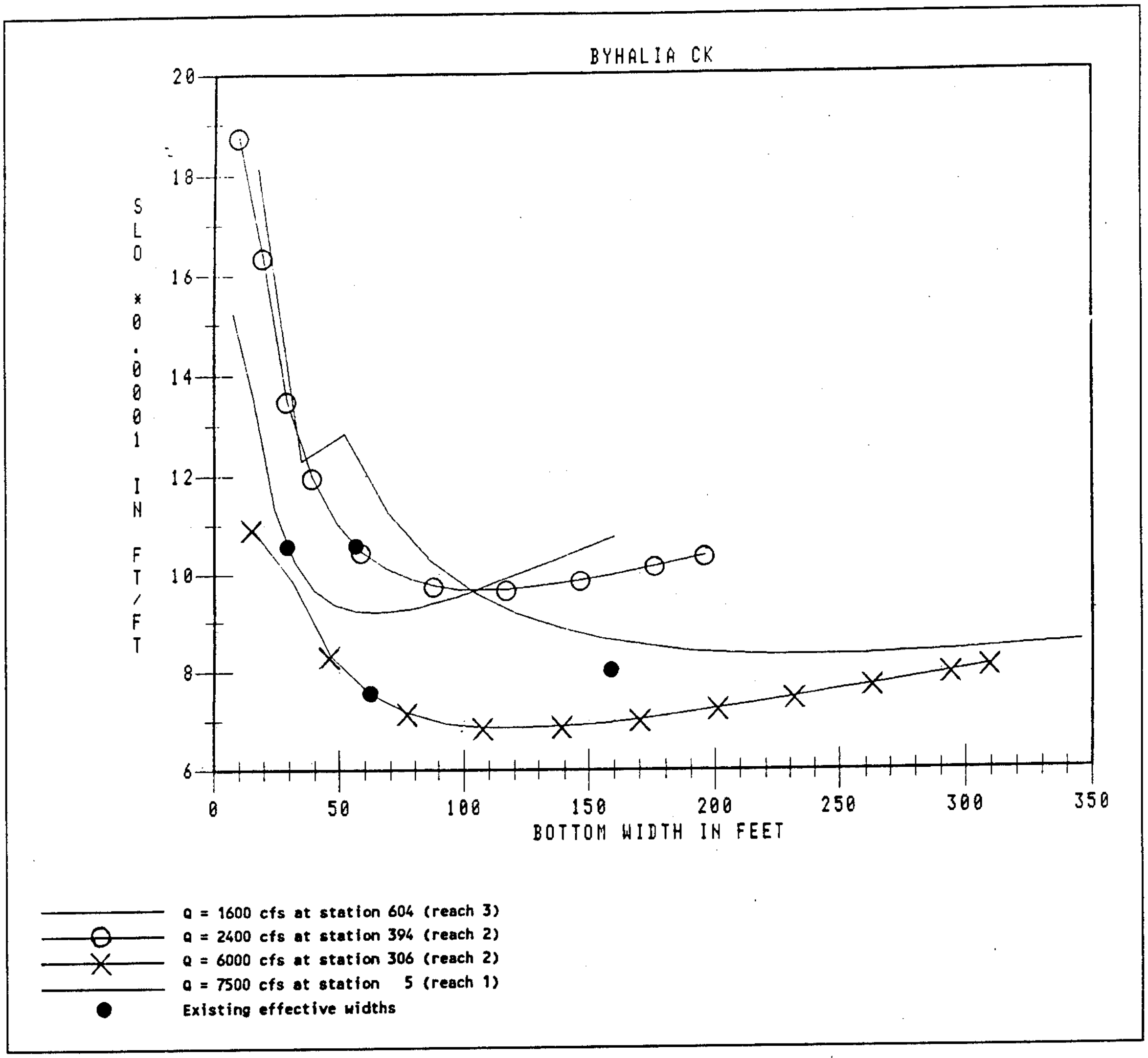



Figure 128. Stable channel calculations by reach

The slope of reach 2 is of considerable interest since it plots lower than reach 1. This is attributed to the relatively steep banks in reach 2 as compared with those on reach 1. The irregular shape of the width-slope curve in reach 1 is due to a change in the bed regime between widths of 30 and 50 ft.

The next task was to space the grade control structures along the creek. Starting in reach 1, the 0.001 slope was projected along the thalweg profile until the existing thalweg profile began to exceed the stable slope. At that point a 10-ft-high weir was placed and the stable slope was projected off the weir crest. Where that slope intersected the thalweg profile another 10-ft-high weir was placed. The procedure was repeated through reaches 1, 2 and 3. In reach 4 the slope was increased to 0.0015.

The grade control structures were designed to a height of 10 ft because that is the maximum height permitted for the "low-drop" design. The spacing is shown in Figure 129. In general, the first structure was located at the upstream end of the reference reach. The next structure was placed where the calculated stable slope from the first weir intersected the channel bed profile. That process was repeated to the most upstream structure. The width of the structure was made equal to the effective width of the existing channel as calculated by SAM so it was balanced with the adopted slopes.

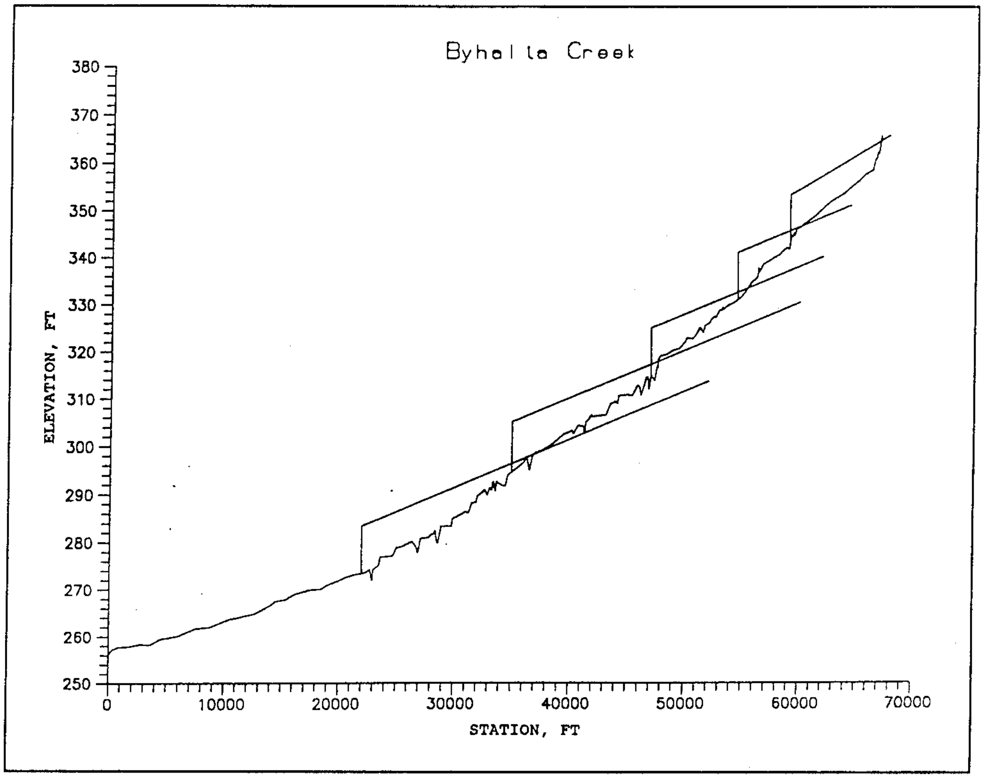



Figure 129. Grade control structure spacing

The design was tested using a head loss criteria. That is, the head loss across the structure was made less than or equal to 1 ft at the 10-year discharge. That discharge is about the bank-full condition in reaches 2, 3 and 4. The normal depth calculations in SAM.hyd were used for that task. The HYD.IN files were developed for a cross section upstream from each structure. The first runthrough was without structures in place so that the base level of the water surface elevation for the 10-year discharge could be obtained. The normal depth option was then run again using the proposed slope and structure width. The objective was to limit head loss across the structure for those flows near bank-full conditions so the structure would not be flanked by erosion during the flood. Also, water elevations would not be increased for those discharges exceeding bank-full. This procedure resulted in two more drop structures than the Vicksburg District's slope-area procedure for siting drop structures in the DEC.

The final design should be tested using HEC-2. The water surface profile should show definite weir control up to the channel-forming discharge. Head loss across the weir should not exceed a foot at top of bank, and the weir should continue to drown out as the discharge increases.

## Problems Encountered

Problems encountered during the conduct of this case study are as follows:

*a.* HEC-2 deck has to be detailed to be acceptable. Problems were experienced because cross sections were spaced too far apart in the basic HEC-2 deck. Additional cross sections were developed by repeating the cross section nearest to the weir locations.

*b.* The Copeland stable channel analytical method requires more detailed roughness than is required by HEC-2. That is, the channel roughness has to be distributed across the section to include an n value for each bank of the channel plus one for the channel bed. The bed value is not prescribed. It is calculated by the analytical procedure based on the bed sediment gradation curve.

*c.* The gradation data were not detailed. Averaging had to be carried out.

*d.* The selection of the grade control heights by the two-criteria method was time-consuming.

# 8 Bank Stability

## Aerial Inspection

### Purpose and scope

The purpose of the aerial inspection task is to identify from aerial reconnaissance the channels in the various watersheds that appear to be the most active with regard to bed/bank stability. The channels were flown in the spring of 1993, and aerial videos were made on the main channel and major tributaries in each watershed from a fixed-wing aircraft flying at an altitude of 2,400 ft above the ground surface. A second flight was made over the 22 long-term monitoring sites at the same altitude but with the camera lens set to maximum magnification to get better resolution on the pictures. The general description of channel conditions as observed from the videos is the subject of this section of the report.

### Description of work

The ARS Sedimentation Laboratory in a cooperative agreement with WES assumed the responsibility for obtaining aerial videos of the watersheds. The ARS used Super VHS (SVHS) video equipment that records frames in digital format that can be easily read into the computer database. The camera was mounted vertically to a fixed-winged Cessna 181 aircraft to provide a view of the ground similar to traditional aerial photography. The flight lines were flown at an altitude of approximately 2,400 ft above the ground surface, and the zoom lens on the camera was set at minimum magnification. The longitudinal distance on each frame is approximately 2,000 ft and the lateral distance approximately 1,400 ft. This altitude was selected because at lower altitudes, the more sinuous channels were impossible to track with the vertically mounted camera, since the aircraft must be maintained in a level position. Even at this altitude, taping was possible only for short reaches, and the flight line would have to be broken, the aircraft would circle, and taping would resume on a new line. A small TV monitor was mounted in the cockpit to help the pilot anticipate turns, which greatly aided in reducing the flight line breaks on some channels. Approximately 50 hours of flying time was required to complete the job.

### Status

Eighty-two creeks were videotaped by ARS personnel during the spring of 1993, and the results are on five tapes. ARS prepared a log for each tape describing significant landmarks such as tributaries, highways, railroad crossings, etc., referenced to the elapsed time from start of tape. The time is shown on the tape for easy reference. Table 29 lists creeks that were taped arranged from major watershed to subwatersheds.

### Observations

The ARS log sheets for each tape were adapted to note observations in viewing the tapes. The major features of streambed, streambank, riparian vegetation, floodplain use, condition of structures, and general comments were listed and characterized to the extent possible from the tape viewing. The scale of each video frame was too small to ascertain anything more than general characteristics. The results from viewing the video tapes are shown in Tables 30-34.

## Harland Creek Test Site

### Introduction

This work unit was originally designed to apply state-of-the art bendway weir technology to the realm of small-stream bank protection. The work was to consist of reviewing and analyzing available prototype stream surveys, selecting a single suitable bend, and designing an effective field of bendway weirs to protect the outer bank of that bend from further erosion. The design would be of sufficient detail so that construction plans and specifications could be accurately and easily generated. After construction, the study reach would be monitored to ascertain the effectiveness of the bank protection plan.

### Scope of work

As stated in the previous paragraph, the original plan was to design, construct, and monitor one set of bendway weirs in one unrevetted bend in a small stream; however, this plan was soon expanded to include design and construction of bendway weirs in a reach of stream with 14 distinct bends. The plan was further modified to include willow post planting where applicable. The willow posts could be used as either stand-alone bank protection structures or in combination with bendway weirs. After additional analysis, it was decided that where conditions warranted, traditional longitudinal peaked stone toe dikes (with tiebacks) would supplement the bendway weir and willow post bank protection structures. A constraint was placed on the designers whereby all bank protection measures used should not fail or require extensive maintenance. This constraint required that all bank protection structures be tailored specifically for the reach in which they were located. No attempt was made to establish a universal set of design guidelines.

Stream corridor habitat enhancement/restoration was also a top priority of the designers. While not compromising the principal goal of bank protection, many habitat enhancing features were identified and where possible incorporated into the study. An informal agreement was reached with the ARS National Sedimentation Laboratory (NSL) in Oxford, MS, to assist in this endeavor.

### Description of the prototype

Vicksburg District personnel were tasked to pick a suitable reach of stream in which to conduct this demonstration project. A section of Harland Creek in Holmes County, Mississippi (Figure 130), was chosen for the following reasons: it is a fairly stable (not actively degrading) meandering stream, is not deeply incised, has a number of bends experiencing active bank erosion, is of a suitable size, is located near WES and the Vicksburg District (field trips could be completed in one working day), and appears to have the appropriate flow characteristics with which to test bendway weir and willow post bank protection structures. Harland Creek is part of the Black Creek watershed, which is contained within the Yazoo River Basin. The test reach is approximately 12,000 ft long and averages 75 to 150 ft in width (measured from top bank to top bank). Bed material ranges from silts and clays to gravel. The stream is of a "flashy" nature (i.e., stages rise and fall quickly during rainfall events). Most surrounding land is either under cultivation or forested.

### Description of prototype problems

The main problem encountered in this reach of stream is bank erosion. The outer banks (concave banks) of the bends are fairly steep and are actively eroding. Bank height in the bends ranges from 5 to 44 ft, with most banks in the 10- to 20-ft range. The vegetative cover on the banks ranges from bare soil to fairly dense vegetation (grasses, shrubs, small trees, etc.). Clay lenses, topsoil, silts, sands, and gravel were observed in eroded areas of the banks.

### Introduction to environmental considerations

From the very beginning of this work unit, environmental considerations were an important concern of the designers. It was felt that the types of bank protection planned for this study would present a unique opportunity to demonstrate new channel stability methods while at the same time enhancing aquatic and terrestrial stream corridor habitat.

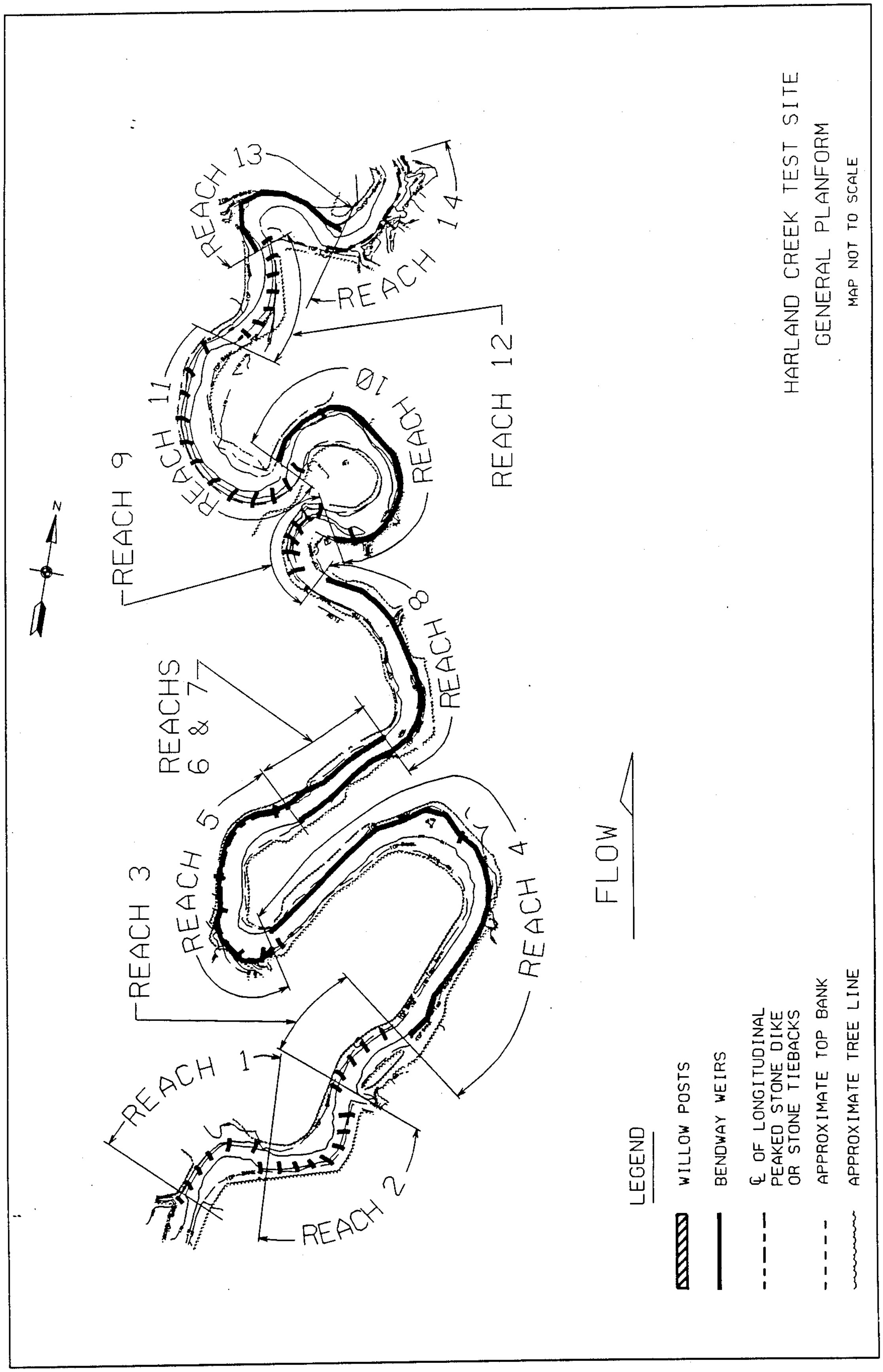



Figure 130. Harland Creek test site

Toward this end an informal agreement was reached with Dr. Doug Shields of the USDA-ARS NSL to assist WES personnel on the stream characteristics needed for habitat restoration in incised, unstable small streams. The Harland Creek bank protection plan will integrate as many environmental characteristics as possible into the final design while keeping within the stated goal of successfully adopting bendway weir and willow post methodology to the realm of small streambank protection.

On two occasions WES personnel traveled to Oxford, MS, to observe several DEC stream habitat restoration sites. Habitat improvement structures, including willow trees and longitudinal stone dikes on point bars, upstream- and downstream-angled (45-deg) short spur dike extensions on existing hard points, willow posts both in rows and bunches, perpendicular short spur dikes attached to existing longitudinal peaked stone toe dikes, and upstream angled V-shaped stone chevrons were all examined and discussed.

### Stream features needed for habitat improvement

The following features, according to Dr. Shields, are needed and/or desirable for successful stream habitat rehabilitation/improvement:

*a.* Occasional deep pools.

*b.* Scour holes. Should be stable, not prone to filling, and at least twice as deep as the average stream depth.

*c.* Stable habitat (critical).

*d.* A diversity of habitat (pool and riffle regime) (highly desirable).

*e.* Solid substrate for invertebrates (i.e., stone) (beneficial).

*f.* A wide stone size gradation (i.e., quarry run) best for benthic macroinvertibrates.

*g.* Canopy cover. Shade canopy cools the immediate area, provides protection from predators, introduces insects and leafy matter into the stream, and provides a source of woody debris.

*h.* Woody debris. This is an extremely important component of stream ecosystems. In one study only 4 percent of the stream bed and banks was made up of woody debris but approximately 60 percent of the biomass lived there. Debris provides good cover and protection for fishes and in this project will be left in place whenever possible.

*i.* Dr. Scott Knight (research ecologist at ARS-NSL) states that stone dikes (or weirs in this case) are generally better for fish habitat than other types of bank protection structures used in the DEC program.[1] They provide diversity of habitat, stable substrate, scour holes, and cover and protection for small fishes.

*j.* Dr. Shields believes that a low-water channel meandering within the weir field would be the planform of choice.

### Introduction and history of bendway weirs

A bendway weir in a navigable river is loosely defined as a rock structure built of Graded A Stone (5,000-lb maximum weight), located in the navigation channel of a bend and angled from 10 to 30 deg upstream of a line drawn perpendicular to the bank line at the bank end of the weir. In cases where the outer bank of the bend does not have a constant radius, one is usually calculated and employed when laying out the position of the weirs (this allows the bendway weirs within a field to act as a coherent unit). The bendway weir is level-crested at an elevation low enough to allow normal river traffic to pass over the weir unimpeded. This elevation gives reasonable assurance that the weirs will be completely submerged at all times during all river stages. Since they cannot be seen, the natural scenic beauty of the river is not disturbed. The bendway weir must be of adequate height and length to intercept a large enough percentage of flow at the river cross section where the weir is located to produce the following seven hydraulic improvements:

*a.* A wider navigation channel through the bend.

*b.* Deposition at the toe of the revetment on the outside of the bend.

*c.* Relocation of the channel thalweg from the toe of the outer bank revetment to a position along a line connecting the river ends of the weirs.

*d.* Surface water currents that do not concentrate on the outer bank of the bend.

*e.* More uniform flow velocities across the bend cross section.

*f.* An improved navigation channel in the crossing downstream of the bend.

*g.* An improved alignment of the navigation channel throughout the bend and downstream crossing.

During the last five years (1989-1993) bendway weirs had been tested in eight physical movable-bed models and one physical fixed-bed model at WES. These models were of navigable rivers with revetted outer banks in the bends

[1] Personal correspondence between Dr. Knight and Mr. Derrick on 9 September 1994.

where the weirs were located. These models were focused primarily on solving severe navigation problems; however, one model also dealt extensively with the environmental effects of bendway weirs on aquatic and terrestrial habitat. The effects on the least tern (a federally protected endangered species) and aquatic life in the vicinity of the weirs were studied in detail.

Thirty-nine bendway weirs have been constructed at four locations on the middle Mississippi River (between St. Louis, MO, and Cairo, IL). Results have been encouraging with prototype results meeting or exceeding the predictions of the model tests. An intensive monitoring program has been in place since the first prototype weir was built in 1990. Monitoring will continue for several years and, when complete, should give a highly detailed and comprehensive account of prototype bendway weir effects and performance from both the hydraulic and environmental points of view.

The remaining WES effort involved a limited series of tests (two) with bendway weirs in a single unrevetted bendway of a sand-bed (DEC) model of a small stream. Information on these tests can be found in Pokrefke (1993). Mr. David Derrick was either the principal investigator or a consultant on all bendway weir tests on all of these models.

A significant amount of knowledge on navigable river bendway weir design, performance, sedimentation patterns, and flow distribution has been learned from the models and prototype installations. However, knowledge of small stream/unrevetted bank bendway weir performance is limited to the two test runs on the DEC model.

### Bendway weir design guidelines for the Harland Creek test reach

The bendway weirs in this study were designed to reduce erosion on the outer banks of the bends by reducing near-bank velocities, reducing the concentration of currents on the outer bank of the bend, and producing a better current alignment through the bends and crossings. A number of complex factors went into determining the design parameters and layout of the bendway weir bank protection structures. While not an all-encompassing list, the main factors behind the bendway weir final design were design and modeling experience of the WES personnel, extensive small stream design experience of the Vicksburg District personnel, environmental considerations, stream geometry, stream features, engineering judgement, and project costs. The following bendway weir design parameters were used for the 54 weirs employed in this study:

*a.* All sets (fields) of bendway weirs were built in an upstream to downstream progression.

*b.* All weir roots (or keys) were constructed to the same specifications as traditional Vicksburg District transverse stone dike (hard point) keyways. An existing proven keyway design was deemed most prudent. Additionally, the inspectors, and possibly the contractor, would be familiar with this design.

*c.* Weir spacing was determined by the radius of the bend, the geometry and features of the outer (concave) bank of the bend, and the small-stream design experience of Vicksburg District personnel. A maximum spacing of 100 ft was agreed upon. In most cases an even more conservative spacing of 75 ft was used.

*d.* A smoothed curve was used to approximate the top bank geometry. All weir angles were referenced to this curve. The maximum bendway weir angle was set at 20 deg. This was based both on knowledge gained from the movable-bed model studies (Pokrefke 1993) and the geometry (small-radius bends) of the test reach. Bend radii ranged from 150 to 635 ft while degree of curvature ranged from 30 to 178 deg. In most cases the angles of the weirs at the upper and lower ends of the bends were reduced so that flow would be well aligned when entering or exiting a bend. Some model tests on navigation projects have shown that bendway weirs can have a pronounced effect up to 3 miles downstream of a weir field. With the tight bends and very short crossings of this study, extreme care was exercised in determining the weir angles near the exits of the bends so that flow would properly enter the downstream crossing and bend.

*e.* Weir lengths were determined as follows: each weir was located in the field; the anticipated relocation of the thalweg was determined; the end of each weir was then centered on the relocated thalweg; the weir lengths were drawn on the stream survey maps and the top bank curve used to determine weir angle was used to connect the stream ends of the weirs; and weir lengths were then adjusted to fit this curve.

*f.* All weirs were sloped (except for the solitary weir in reach 4, which is level-crested). The heights of the weirs were calculated as follows: in each bend after the weirs were laid out on the stream survey maps, the highest stream-end weir elevation was ascertained; 2 ft was added to that elevation and the result was used as the stream-end height for all weirs in the bend; and the bank end elevation for all weirs in the bend was set 2 ft higher than the stream-end elevation. The reach 4 weir was built level-crested at an elevation approximately 4 ft above the stream bed.

*g.* All bendway weirs and weir roots were constructed of R-650 stone (650-lb maximum weight stone). Vicksburg District personnel determined that this stone size would be adequate for the anticipated velocities and flow rates of the stream and would also be well suited to meet the minimum height requirement (2 ft) of the weirs. Field calculations indicated that many stones measured between 17 and 24 in. along the major axis. The largest stone measured was 38 by 39 in.

*h*. Crest width of all bendway weirs was specified as 2 ft.

*i*. Side slopes were specified as the natural angle of repose (1V on 1.5 H).

### Anticipated effects of bendway weirs on the Harland Creek test reach

The weirs used in this study are radically different from the weirs found in a navigable river. Navigable river bendway weirs are level-crested, very long (500 to 1,400 ft), and submerged at all times with a high (15 to 60 ft or more) column of water overtopping the crest. The small-stream bendway weirs in this study are sloped, fairly short (17 to 60 ft), and emergent except during very high flows. Therefore, this portion of the report will be divided into two sections: high-water effects and low-water effects of small-stream bendway weirs.

**Anticipated high-water effects of bendway weirs.** The weirs in Harland Creek will act more like navigable river bendway weirs during high-water events than during low or base flow. During high water when the weirs are totally submerged, flow overtopping the weir should be redirected at an angle approximately perpendicular to the crest of the weir (this assumption was used in setting the weir angles so that flow would be correctly aligned through the bends and crossings). When the weirs are overtopped, flow in the bends should be nearly parallel to the banks with no concentration of currents on the outer bank of the bend. Surface water velocities should be more uniform across any bend cross section with the highest velocities found near the stream ends of the weirs. These high velocities should cause relatively deep scour holes to form at the stream ends of the weirs. During a long-duration high-water event the scour holes should connect, effectively moving the channel thalweg from the toe of the outer bank to the area near the stream ends of the weirs. Flow through the bend and downstream crossing should be better aligned. Sediment redistribution will probably occur within the study reach, but the effects of this are hard to predict at this time. This scenario (currents reduced near and aligned parallel to the outer bank and improved current alignment through the bends and crossings) should result in excellent bank protection.

**Anticipated low-water effects of bendway weirs.** At this time, the effects of bendway weirs during low or base flow are a matter of conjecture. The effects of low flows on streambank erosion are not nearly as critical as high-flow forces. While the possible channel configurations are endless, the designers feel that one of two low-water channel configurations is probable. During base and low flows the entire weir is emergent. When the weirs were observed just days after being built, flow filtered both through the entire length of and around the stream end of all weirs. This created a series of riffles (at the end of the weirs) and pools (the old deep stream channel located at the toe of the bank between weirs). If this configuration stabilizes and is dominant, the channel would meander throughout the weir field. However, higher flows should form scour holes off the stream ends of the weirs. If the high-flow channel proves dominant, then the thalweg would follow a route connecting the stream ends of the weirs. From a hydraulic point of view either of these bed configurations would be acceptable as flow velocities would be lessened or moved away from the eroded outer bank.

## Introduction to willow post planting

In 1992 Mr. David Abraham, WES, attended a workshop at which Mr. Don Roseboom, Illinois State Water Survey, lectured on the willow post method for small stream bank protection. Mr. Abraham pursued this topic, and as a result Messrs. Abraham and David Derrick attended a 1-day willow post planting workshop in Peoria, IL.[1] This workshop included a morning of instruction, a small-scale (table model) movable-bed model demonstration of the willow post method, and discussions with experienced contractors. The afternoon session consisted of field site observations of an in-progress willow post planting operation and a walk-through of two completed willow post projects. Successes and failures of different methods and materials were discussed. Mr. Roseboom gave Messrs. Derrick and Abraham an extended tour of the oldest (installed in 1988) willow post demonstration project and a "Lunker" aquatic habitat improvement project. Eighteen willow post projects have been completed under the direction of Mr. Roseboom.

## Advantages of the willow post method

Guidance for application of willow posts was taken from the Illinois State Water Survey Miscellaneous Publication Number 130.[2]

The willow post method is a means of controlling streambank erosion through the systematic installation of large native willow cuttings to stabilize eroding streambanks. The stabilization process is twofold. Willow foliage lowers floodwater velocities on and near the eroding bank, and the root system of the willow binds the soil. Roseboom states that flow rates measured during a 1,100-cfs flood event were 4.2, 2.6, and 1.6 fps at the outside of the bend, at the streamward row of willows, and at the second row of willows, respectively. One study reach sustained damage to only 13 willows (out of 620) during a high-water ice flow event.

The advantages of the willow post method are as follows:

*a.* Materials and installation costs are low.

*b.* The method is environmentally sound and acceptable.

---

[1] Streambank Stabilization Workshop, 18 March 1993, sponsored and conducted by Illinois State Water Survey and University of Illinois Co-operative Extension Service, Peoria, IL.

[2] Illinois State Water Survey. (1992). "The willow-post method for streambank stabilization," Illinois State Water Survey Miscellaneous Publication 130, Peoria, IL.

*c.* Ongoing maintenance costs are low.

*d.* The method has been tested and proven effective in Illinois.

*e.* Bank protection is long-term.

*f.* The willows do not grow into the stream or above top bank of the stream.

*g.* More valuable trees will grow after the willows are established.

*h.* The matured willows provide canopy cover.

*i.* The matured willows provide terrestrial habitat.

### Willow post planting design guidelines

The guidelines used in this study are a slightly modified version of the design developed by Roseboom and presented at the Streambank Stabilization Workshop:

*a.* Spacing will be on a 3-ft grid, i.e., 3 ft between rows and 3 ft between willow posts in each row. This is tighter than the 5- by 5-ft grid espoused by Roseboom. It was felt that the closer spacing would allow the butt ends of the willows in the rows higher up the bank to still be within the water table.

*b.* Post diameter should be a minimum of 3 in. at the butt end.

*c.* Minimum willow post length should be 10 ft.

*d.* Posts should be planted at least 8 ft deep. Again, this will help to insure that the butt ends of all willows will be in the water table. Also, the row of willows at water's edge should be stable even in the presence of outer bank scour.

*e.* Not more than 4 ft of the post should be above the ground.

*f.* Native willows in good condition should be used.

*g.* Posts should be cut, soaked in water, and planted within 48 hr.

*h.* At the time of cutting, the tops of the posts should be marked. The willow posts must be planted in the "up" position. Marking the tops will assure that the posts are not inadvertently planted upside down.

*i.* Posts must be planted when dormant (usually between 1 December and 1 March, dependent on weather). Dormancy is defined as after the leaves have dropped and before the leaf buds have appeared. Experts state that the willows must be planted when dormant to assure a high rate of survival.

*j.* Willows grow best in silt and clay. When planted in sand, growth is slower.

*k.* The willow post holes should be formed by an 8-in.-diam metal ram or auger with a flat (not pointed) end.

*l.* Willow posts will be planted for the entire length of the outer bank of each bend.

*m.* The butt ends of the willows should make contact with the bottom of the hole

*n.* Willows should not be planted above top bank.

*o.* Grass should be planted between the willows.

*p.* Approximately 9,383 willow posts (estimate provided by the Vicksburg District) are scheduled to be planted in this study.

### Willow post planting

At the time this report was written the willow posts had not been planted. Tentative plans call for a meeting in January 1994 at which the exact positions of all willow posts will be determined and marked. Planting is scheduled to begin the week after this meeting, and barring unforeseen events, should be completed well before the end of the dormant period.

After the willows are established and have stabilized their immediate area, it is expected that "volunteer" willows will introduce themselves. Other species of trees (typically river birch, cottonwood, and sycamore) would also be expected to grow after the willows are established.

### Introduction to the final design

During a field trip in December 1992, Vicksburg District personnel laid out traditional transverse and longitudinal dike bank protection structures for each reach of the study. A rough preliminary design incorporating weirs, willow posts, and traditional bank protection structures was laid out by WES personnel using USGS quad sheets. Using aerial photographs, the design was revised at the request of the Vicksburg District so that the amount of materials to be specified in the contract could be estimated to within plus or minus 10 percent. This design was then submitted for review by the U.S. Army

Engineer Division, Lower Mississippi Valley, and the final design was formulated in July 1993.

### The final design

The following list will reference the reach by number, specify the figure number that contains the reach, describe the reach geometry, state the type (or types) of bank protection employed, list the specifications of the bank protection, and in some cases, the reasons used in choosing that particular bank protection. Each reach contains only one bend. However, each reach was broken down into as many subsections as needed to adequately describe the geometry of the reach. Please note that all weir specifications (lengths, angles, spacing) were listed in an upstream to downstream progression and were obtained from the construction drawings and are not "as built" measurements. Also, please note that the number of willow posts specified for each reach are approximate figures generated by the Vicksburg District.

### Reach 1

Reach geometry (straight, then curved) (Figure 131): Section 1, straight for 175 ft; Section 2, curved, 200-ft radius, 83 deg of curvature, top bank length 290 ft; top bank (concave bank) length for the entire reach 465 ft. Bank protection consists of six bendway weirs with lengths of 22, 30, 28, 38, 40, and 30 ft, and angled 10, 20, 20, 15, 15, and 10 deg upstream. Spacing was specified as 75 ft between weirs 1 and 2, 100 ft between weirs 2 and 3, 75 ft between weirs 3 and 4, and 100 ft between weirs 4 and 5, and 5 and 6. For all weirs in this reach the stream end weir heights were set at el 172 ft NGVD and the bank end heights were set at el 174 ft. The uneven weir spacing in this reach was due to an irregular outer bank.

### Reach 2

Reach geometry (straight, then curved) (Figure 131): Section 1, straight for 160 ft; Section 2, curved, 230-ft radius, 94 deg, top bank length 375 ft; total length of top bank of reach 535 ft. Bank protection consists of eight bendway weirs, with lengths of 35, 38, 35, 30, 48, 35, 35, and 46 ft, spaced evenly at intervals of 75 ft, with all weirs angled 20 deg upstream, except for weirs 1 and 8, which are angled at 10 and 15 deg, respectively.

### Reach 3

Reach geometry (curved) (Figure 131): 435-ft radius, 64 deg, top bank length 485 ft. Bank protection consists of five bendway weirs, lengths of 35, 35, 36, 32, and 35 ft, spaced evenly at intervals of 75 ft, angled upstream at 10, 15, 20, 10, and 0 deg, respectively. Weir heights were set at el 170 ft for

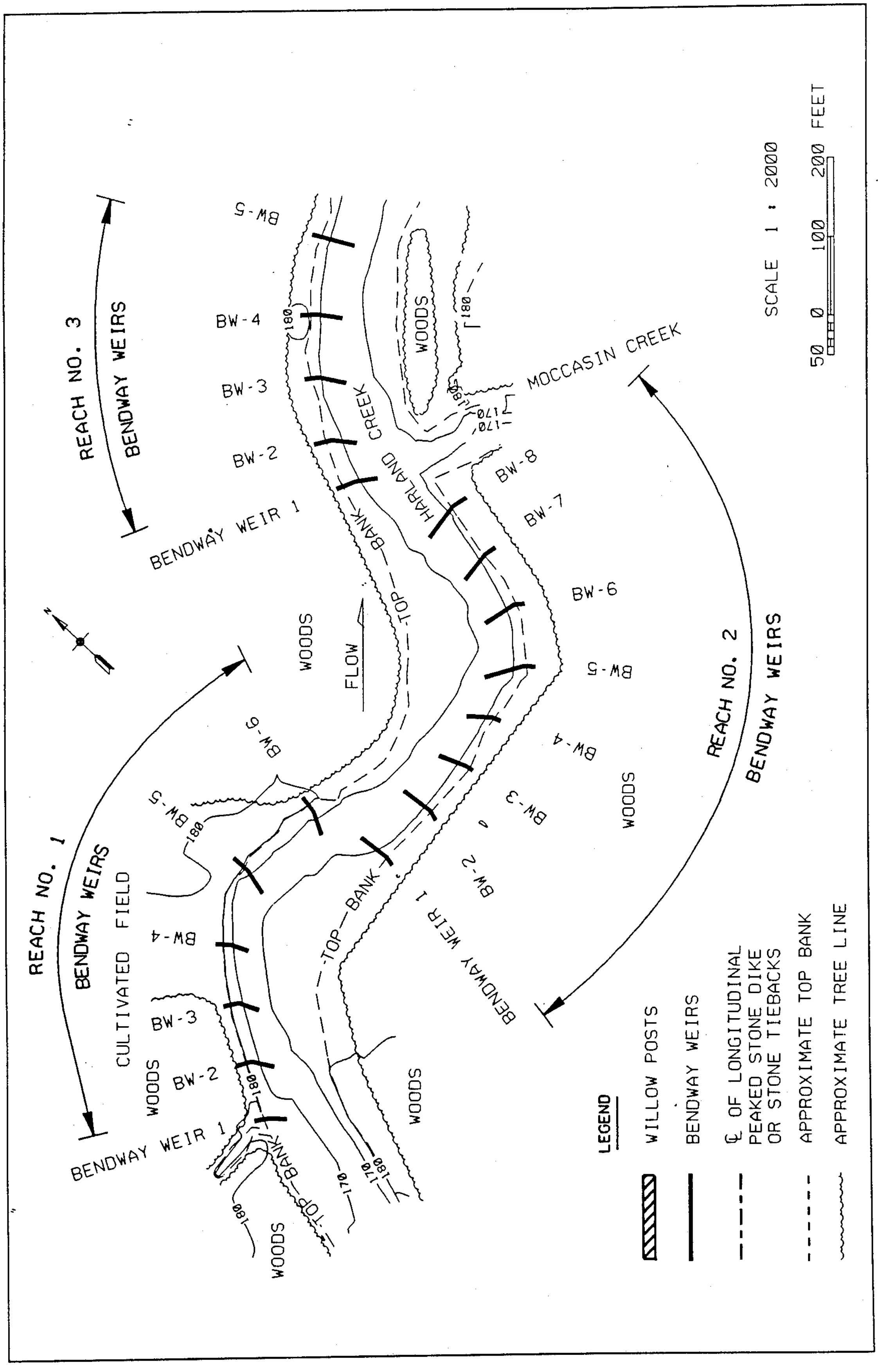



Figure 131. Harland Creek test site, reaches 1, 2, and 3

the stream ends and el 172 ft for the bank ends. This bend has a fairly smooth radius outer bank and appeared well suited for bendway weirs. The upstream crossing is very complex with Moccasin Creek entering from the right about midway through the crossing.

### Reach 4

Reach geometry (straight, curved, then straight) (Figure 132): Section 1, straight for 565 ft; Section 2, curved, 330-ft radius, 162 deg, top bank length 930 ft; Section 3, straight for 575 ft; total top bank length of reach 2,070 ft. Bank protection consists of a combination of willow posts and one bendway weir. Willow posts in the straight sections (sections 1 and 3, total of 1,178 willows) will consist of three rows, 3 ft between rows, willows spaced 3 ft apart, first row located at water's edge. In the bend itself (Section 2) bank protection will consist of five rows of willows (approximately 1,405 willows), 3 ft between rows, willows spaced 3 ft apart, with the first row located at water's edge. A total of 2,583 willows will be used in reach 4. The single bendway weir was placed immediately upstream of a rather large bank blow-out in the middle of Section 2 on the right bank. The stream had eroded the outer bank approximately 75 ft, forming an island to the right of the original channel with the current thalweg in a channel immediately to the right of the island. This bendway weir is designed to redirect high-water currents away from the blown-out area and into the original channel. The weir is level-crested at an elevation approximately 4 ft above the stream bed (el 167 ft), angled 10 deg upstream, and built to a length of 27 ft.

### Reach 5

Reach geometry (multiple-radii bend) (Figure 133): Section 1, curved, 170-ft radius, 155 deg, top bank length 460 ft; Section 2, curved, 555-ft radius, 22 deg, top bank length 220 ft; Section 3, curved, 225-ft radius, 82 deg, top bank length 320 ft; total top bank length of reach 1,000 ft. Bank protection consists of a combination of a longitudinal peaked stone dike, six bendway weirs, and willow posts. This bend has for at least a portion of its length an extremely high outer bank (measured maximum height of 44 ft). A tight radius bend near this high bank prompted the designers to be very conservative with the bank protection design for this reach. A 1-ton/linear-foot longitudinal peaked stone toe dike (constructed of 200-lb maximum weight stone) will follow the toe of the outer bank for the entire length of the reach. Keyed into this are a series of bendway weirs, all angled 20 deg upstream (except for weir 6, which was angled 10 deg <u>downstream</u>). Weirs 1-3 were evenly spaced at 100-ft intervals, and weirs 4 and 5 were evenly spaced 200 ft apart. Weir 6 was located 50 ft upstream of the downstream end of the longitudinal peaked stone toe dike. Vicksburg District personnel had observed scour holes at the downstream end of many longitudinal peaked stone toe dikes at several locations throughout the District. In some cases the scour holes have caused failure of the stone toe dike in that location. In response to

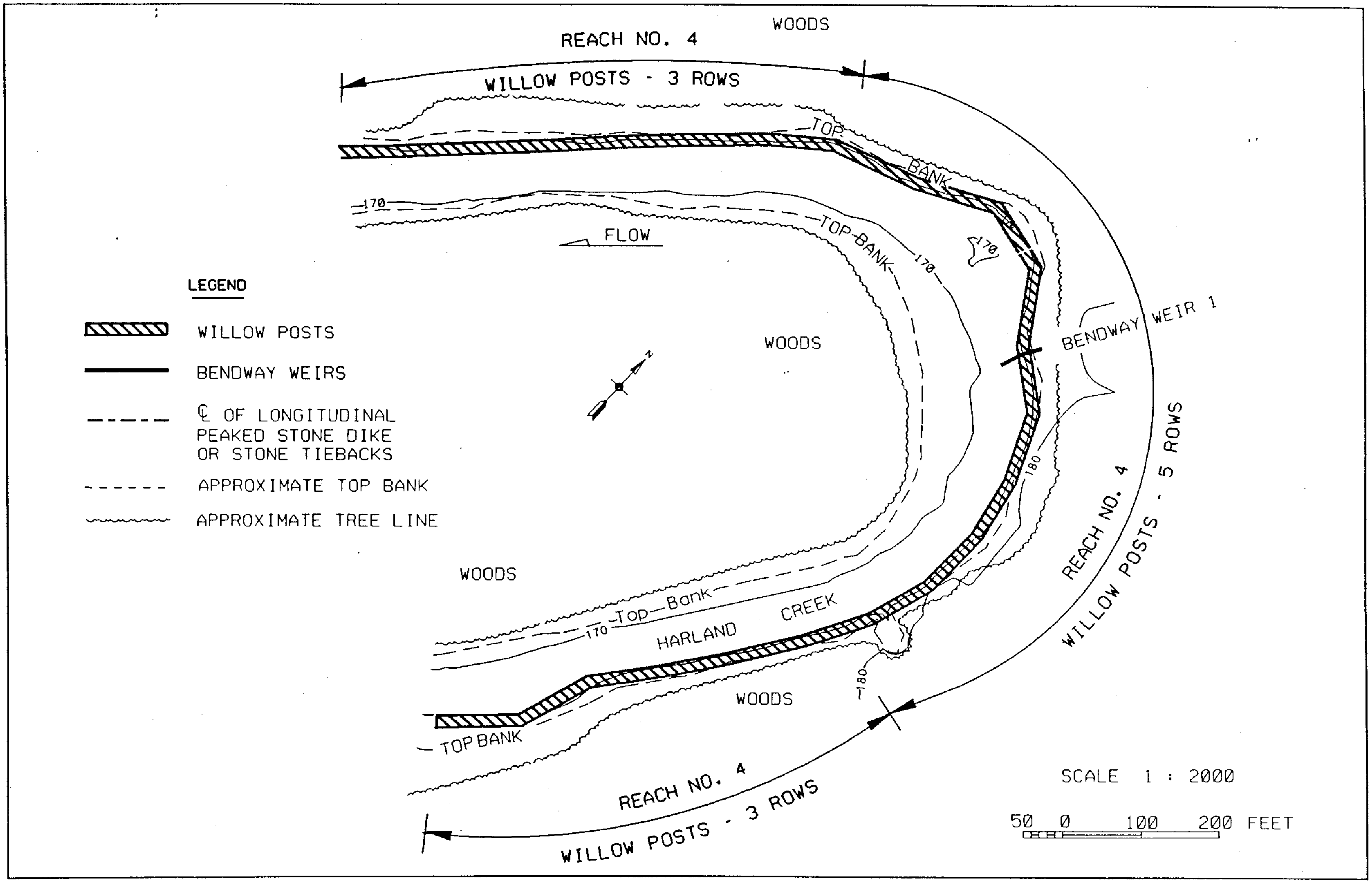



Figure 132. Harland Creek test site, reach 4

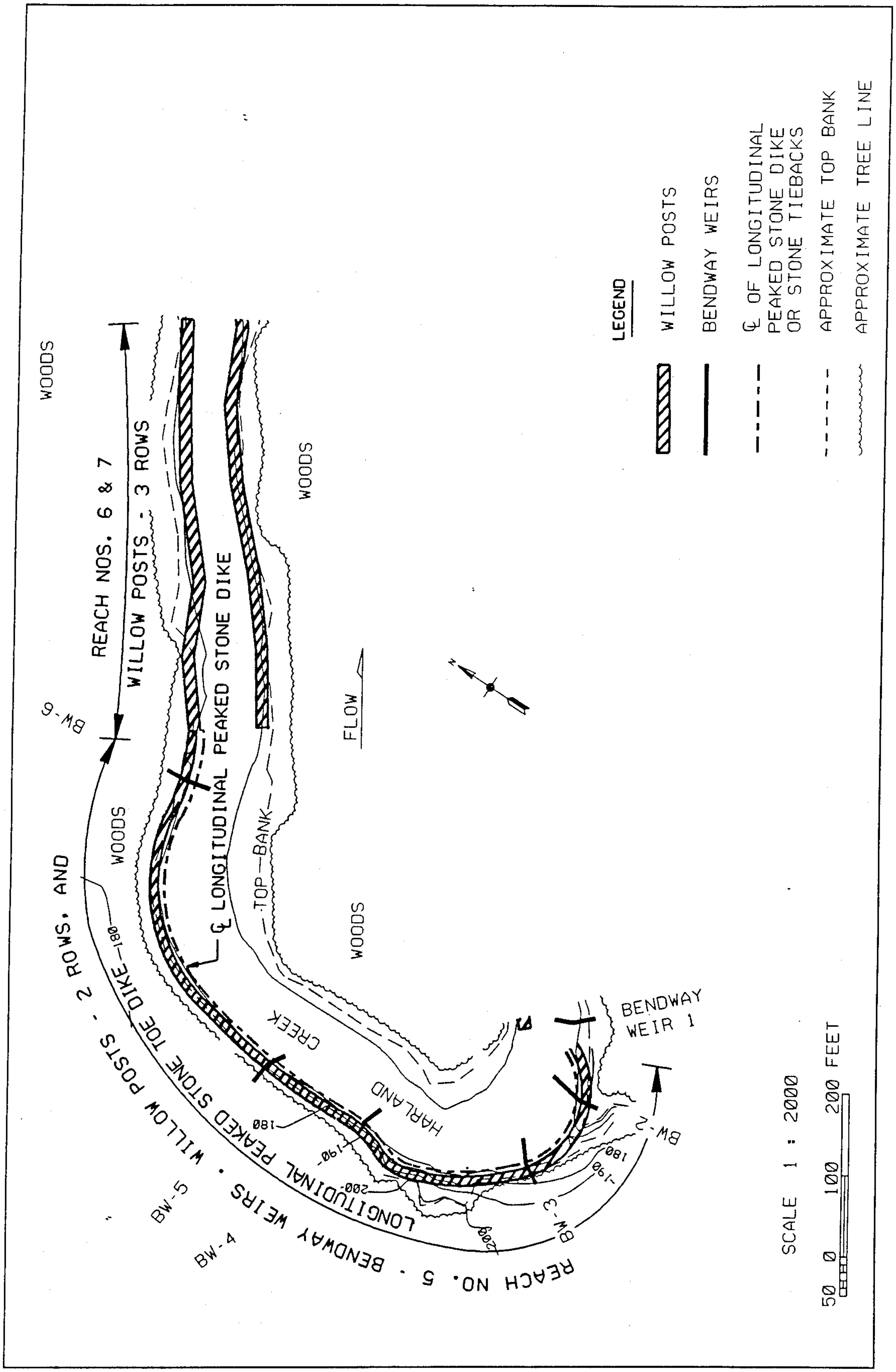



Figure 133. Harland Creek test site, reaches 5, 6, and 7

this, weir 6 was designed to direct flow away from the longitudinal peaked stone toe dike, thereby, it was hoped, eliminating the damaging scour hole. Weir lengths for Reach 5 are 21, 25, 25, 27, 27, and 27 ft. Weir heights were set at el 166 ft for all stream ends and el 168 ft for all bank ends. Two rows of willow posts will also be incorporated into this design, with the first row of willows planted at the bank-side foot of the longitudinal peaked stone toe dike and the second row 3 ft bankward of the first row. In each row, willow posts will be planted on 3-ft centers. Approximately 685 willows will be used in the reach.

### Reaches 6 and 7

Reach 6 geometry (curved) (Figure 133): 635-ft radius, 30 deg, concave bank length 335 ft, total length both banks of the reach 555 ft; Reach 7 geometry (curved): 525-ft radius (medium), 38 deg, concave bank length 350 ft, total length both banks of reach 640 ft. Bank protection consists of three rows of willow posts (on 3-ft centers) on both sides of the stream for the entire length of both reaches. The first row will be positioned at water's edge with the other rows spaced 3 ft apart. Reaches 6 and 7 are fairly straight, and WES personnel feel it would be best to stabilize these reaches in the existing location. Also, the bendway weirs in the upstream bend (Reach 5) could have some effect on flow direction through Reaches 6 and 7 and could possibly change the position of some areas of bank attack. For these reasons the decision was made to protect both banks. Approximately 1,206 willow posts will be used for Reaches 6 and 7.

### Reach 8

Reach geometry (curved, straight, then curved) (Figure 134): Section 1, curved, 275-ft radius (large), 86 deg, top bank length 410 ft; Section 2, straight for 190 ft; Section 3, curved, 425-ft radius, 55 deg, top bank length 410 ft; total top bank length of reach 1,010 ft. Bank protection consists solely of willow posts, five rows, 3 ft between rows, willows on 3-ft centers, first row positioned at water's edge. Approximately 1,630 willows will be used.

### Reach 9

Reach geometry (curved) (Figure 134): 175-ft radius, 178 deg, top bank length 540 ft. Seven bendway weirs, all angled 20 deg upstream, lengths of 32, 29, 17, 30, 37, 20, and 37 ft, and spaced evenly at intervals of 75 ft. All weir heights were set at el 164 ft NGVD for the stream ends and el 166 ft NGVD for the bank ends.

Reaches 9 and 13 are the only small-radius/long-duration bends in the study reach. The designers wished to use willow posts in one and bendway weirs in the other. Reach 13 was deemed unsuitable for bendway weirs due

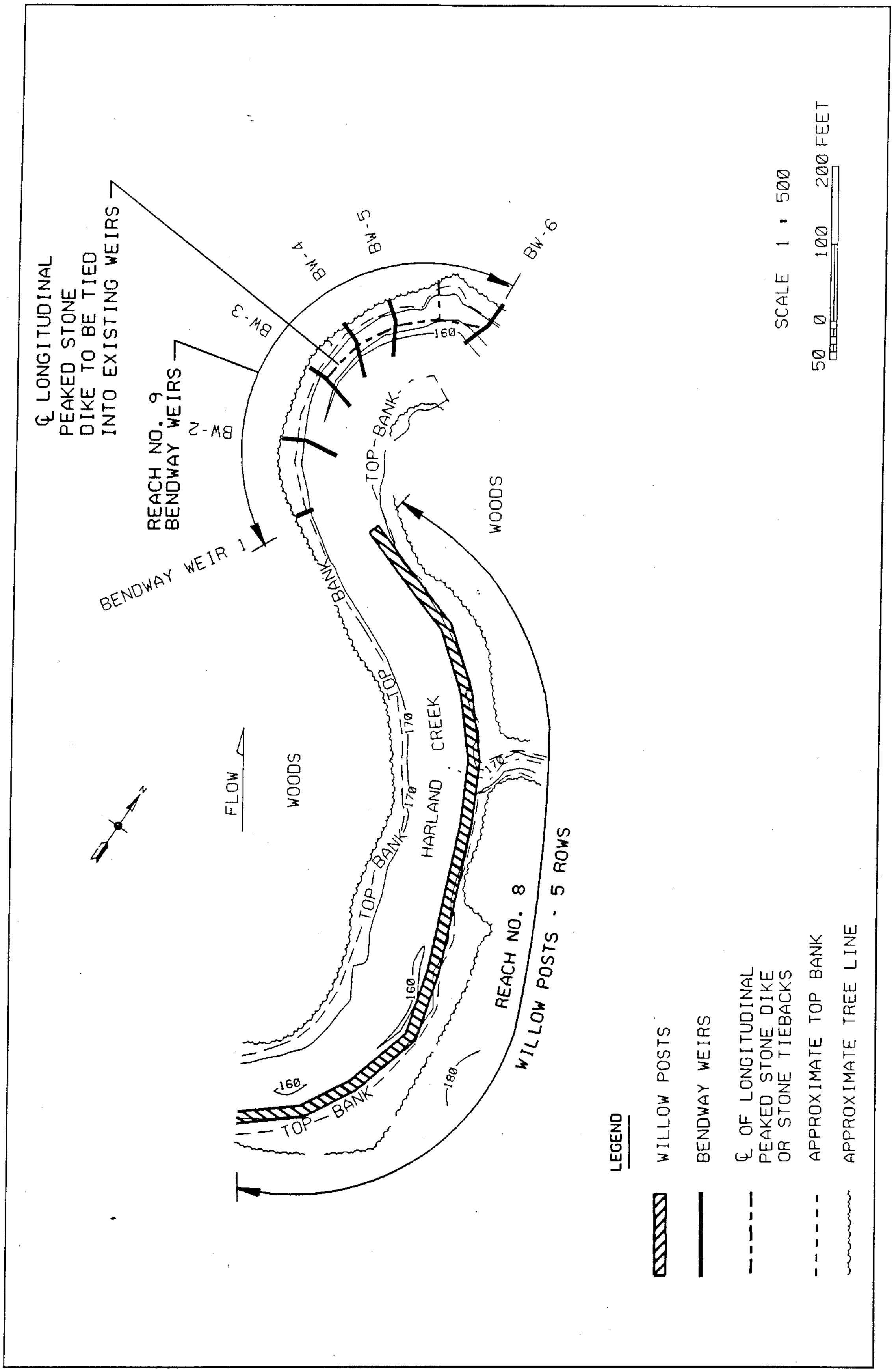



Figure 134. Harland Creek test site, reaches 8 and 9

to the extremely flat outer bank (approximately 230 ft long) in the upstream section of the bend. Therefore, bendway weirs were placed in Reach 9 and willows in Reach 13. Reach 13 will give additional data since willows will be planted on two extremely different bank types (very flat and steep).

### Reach 10

Reach geometry (curved, straight, then multi-radius curve) (Figure 135): Section 1, curved, 245-ft radius, 146 deg, top bank length 620 ft; Section 2, straight for 180 ft; Section 3, 150-ft radius, 71 deg, top bank length 185 ft; Section 4, 345-ft radius, 50 deg, top bank length 300 ft; total top bank length of reach 1,285 ft. Bank protection consists of a combination of willow posts and two bendway weirs. The willow posts will be planted in five rows, spaced 3 ft between rows, with willows spaced 3 ft apart, with the first row located at the water's edge. Approximately 1,805 willows will be planted in this reach. The two bendway weirs were positioned approximately 85 and 990 ft from the upstream end of the bend, with lengths of 60 and 30 ft, angled 15 and 10 deg upstream, with stream ends at el 164 ft NGVD and bank-end elevations of el 166 ft NGVD. Geometrically speaking the outer bank of this bend is extremely irregular. Bendway weirs would not likely be effective due to the irregular flow patterns. However, the two bendway weirs specified in this plan should help in reducing pressure on the outer bank and realigning currents in the immediate area downstream of the weirs. The upstream weir should also help in aligning the currents entering Reach 10.

### Reach 11

Reach geometry (multi-radius bend) (Figure 135): Section 1, curved, 220-ft radius, 108 deg, top bank length 415 ft; Section 2, curved, 415-ft radius, 108 deg, top bank length 780 ft; total top bank length of reach 1,195 ft. Twelve bendway weirs, all weirs spaced evenly at intervals of 100 ft, all angled 20 deg upstream (except for weir 12 which is angled 10 deg upstream), and all built to a length of 40 ft except for weirs 6, 8, and 11, which are 38, 46, and 46 ft, respectively. All weir heights were set at el 162 ft NGVD for the stream end and el 164 ft NGVD for the bank end.

The outer bank of this bend is relatively free of anomalies, averages 15 to 20 ft in height, and is evenly eroded along its length, thus appearing to be well suited for bendway weirs.

### Reach 12

Reach geometry (curved) (Figure 136): 350-ft radius, 90 deg, top bank length 550 ft. Seven bendway weirs, all angled 20 deg upstream (except for weirs 1 and 7, which are both angled 10 deg upstream), all weirs spaced evenly at intervals of 75 ft and built to lengths of 30 ft. All weir heights were

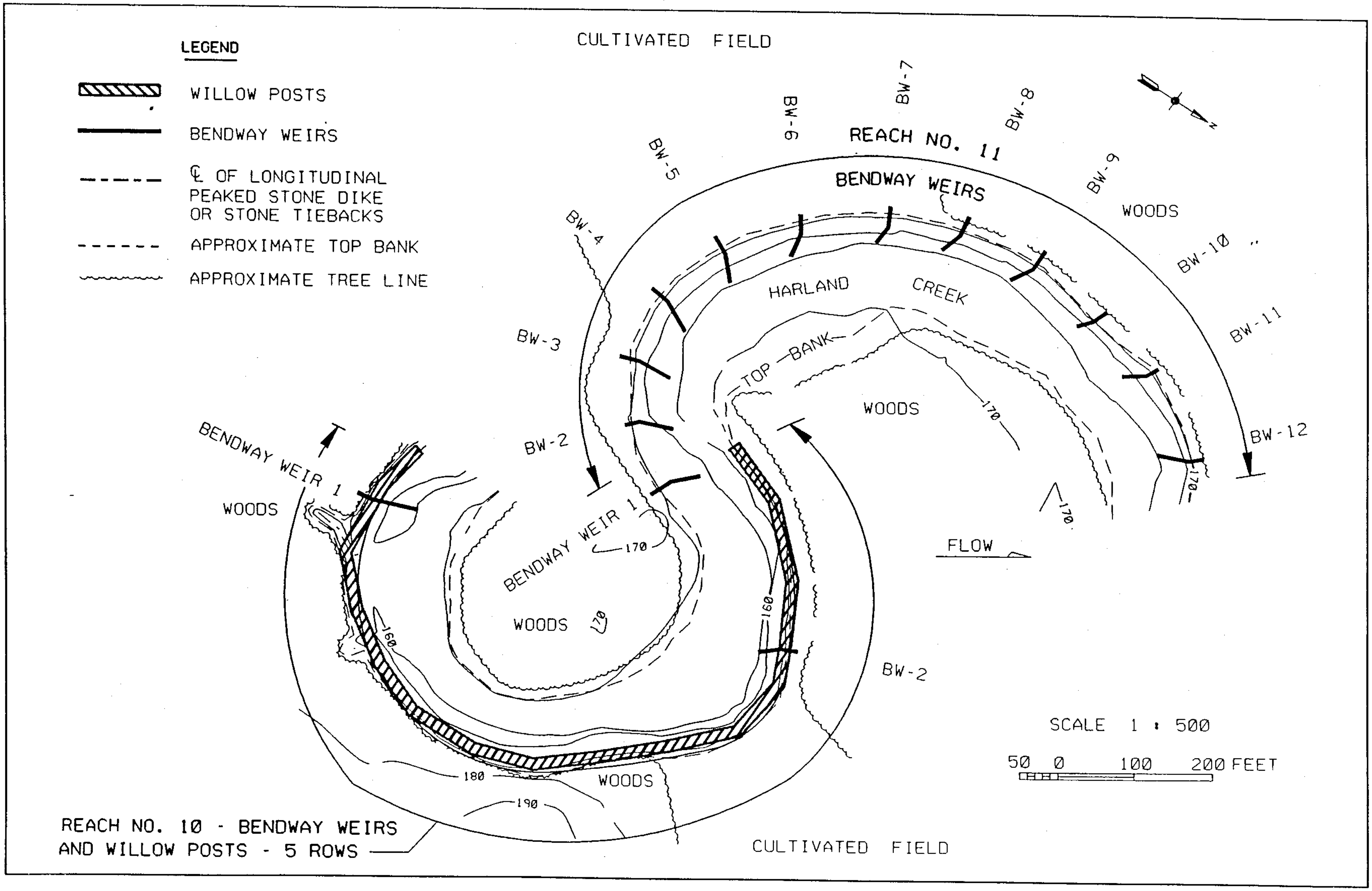



Figure 135. Harland Creek test site, reaches 10 and 11

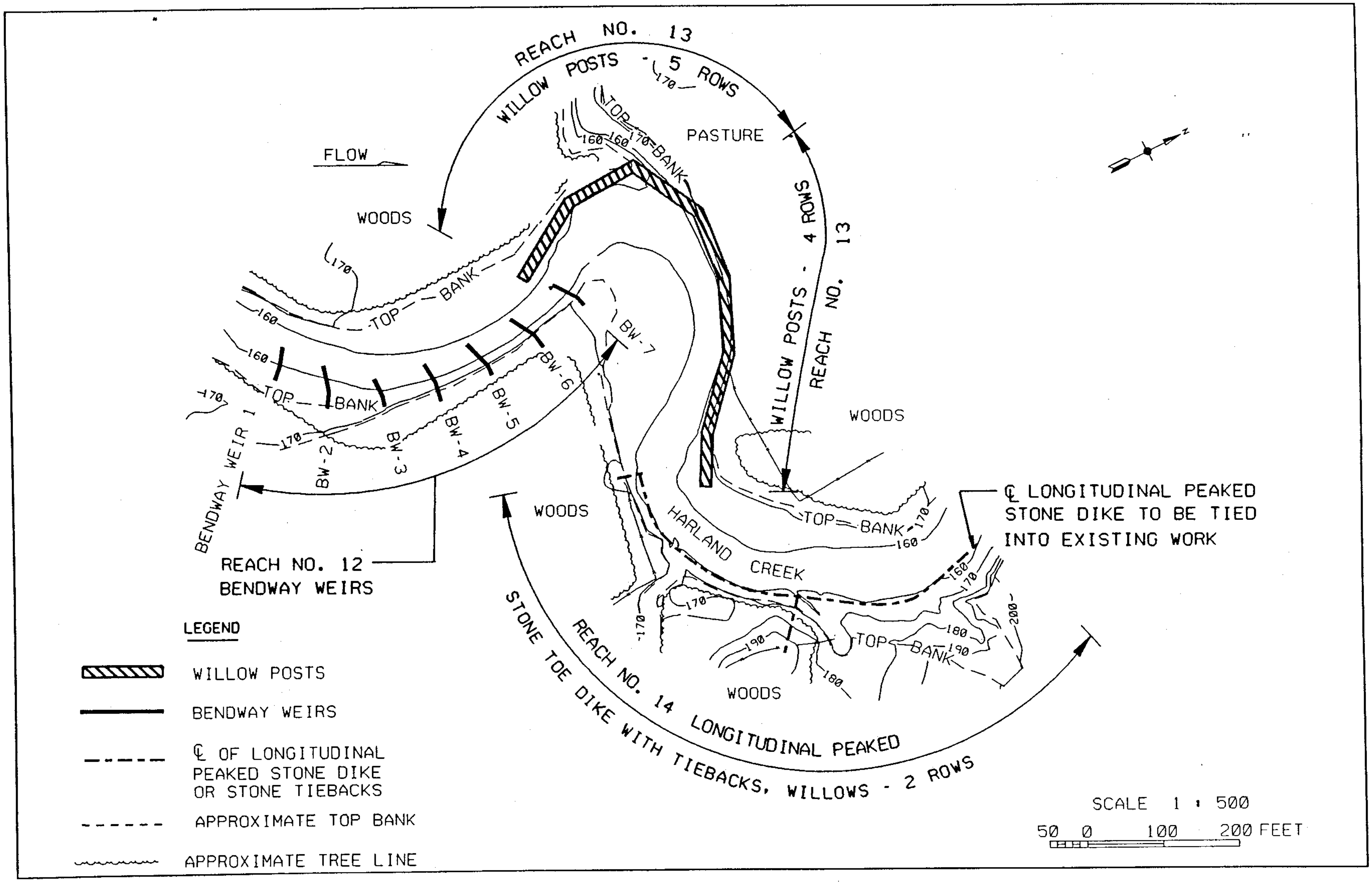



Figure 136. Harland Creek test site, reaches 12, 13, and 14

set at el 162 ft NGVD for the stream end and el 164 ft NGVD for the bank end.

### Reach 13

Reach geometry (multi-radius bend, then straight) (Figure 136): Section 1, curved, 185-ft radius, 72 deg, top bank length 230 ft; Section 2, curved, 375-ft radius, 62 deg, top bank length 405 ft; Section 3, straight for 55 ft; total top bank length of reach 690 ft. Bank protection will consist solely of willow posts. The upstream 230 ft of the bend will have five rows of willows, first row starting 5 ft from the water's edge, spaced 3 ft on center. The remaining 460 linear feet of bank will consist of four rows of willows, 3 ft between rows, willows on 3-ft centers, first row positioned at water's edge. Approximately 1,005 willows will be used in this reach. Section 1 of this bend has a very flat bank. The stream is on a 15- to 20-ft bluff for the remainder of the reach. Bendway weirs were not used in this reach since there is no bank to key into in Section 1 and with the sharp transition angle between Sections 1 and 2 it would be difficult to correctly angle the bendway weirs in this area.

### Reach 14

Reach geometry (curved) (Figure 136): 295-ft radius, 133 deg, top bank length 685 ft. Bank protection consists of a Vicksburg District designed traditional 1-ton/linear-foot longitudinal peaked stone toe dike (with two tiebacks) and willow posts. The peaked stone dike follows the toe of the outer bank for most of the length of the reach and then ties into some existing bank paving near the lower end of the bend. The dike and tiebacks were constructed of R-200 stone (200-lb maximum weight stone). The first row of willows will be planted at the bank-side toe of the peaked stone dike and the second row 3 ft bankward. In each row willow posts will be spaced 3 ft apart. Approximately 469 willows will be planted in reach 14.

### The construction contract

Contract specification modifications were delivered by WES personnel to Vicksburg District personnel on 25 March 1993. Bids for the construction contract were opened on 8 July 1993. The contract was awarded to Procon, Inc., of Brandon, MS, in the amount of $303,660.00. Procon's contract bid was broken down as follows:

*a.* $75,000.00 for willow post planting.

*b.* $1,100.00 for mobilization and demobilization.

*c.* $9,000.00 for debris removal.

*d.* $5,000.00 for erosion control.

*e.* $100.00 each for the first 35, $100.00 each over 35. Tieback and weir root excavation and backfill.

*f.* $24.00 per ton for R-200 stone (used for longitudinal dikes and tiebacks). The contract estimates 1,860 tons will be needed. Any amount over 1,860 tons will also cost $24.00 per ton.

*g.* $24.00 per ton for R-650 stone (used for bendway weir construction). The contract estimates 4,220 tons will be needed. Any quantity over this amount will also cost $24.00 per ton.

Stone work commenced in late August. By 10 September 1993 all stone work was complete. Willow post planting should begin the week of 17-21 January 1994. Planting should be completed well before the 1 March 1994 contract completion date.

### Bank protection costs

The per-linear-foot cost for the Harland Creek study reach was $25.94 (contract cost of $303,660.00 divided by the total length of protected bank, 11,705 ft).

Relying on figures provided by the Vicksburg District, the costs of the bendway weirs (evenly spaced 100 ft apart) compared to other typical stone bank protection structures reveal that the weirs compare favorably with a 1-ton/linear-foot longitudinal peaked stone toe dike, cost half as much as a 2-ton/linear-foot longitudinal dike, and are more costly than transverse hard points. A typical bendway weir requires approximately 120 tons of R-650 stone. A typical 1-ton/linear-foot longitudinal dike with tiebacks requires a like amount of R-200 stone (stone costs are usually the same for R-200 and R-650 stone). The Vicksburg District feels that transverse hard points are the most cost- effective stone bank protection structures currently in use. Approximately 80 tons of R-200 stone are used for each transverse hard point. If the same spacing is employed for both, this would make the bendway weirs approximately one and a half times as expensive as the hard points. This disparity would be reduced if weir spacing were increased.

The construction contract line item cost for willow post planting was $75,000.00. Dividing this figure by the number of willow posts planted (approximately 9,383) gives a per-willow cost of $7.99. This figure includes procurement and transportation of the willows, bank grading, augering, willow placement, and backfilling the hole. This is a fair price considering the specifications cited and the fact that area contractors were unfamiliar with this type of work. This cost is hard to compare with other willow post projects due to the different construction specifications and procurement procedures used.

### First impressions of the completed stone work

An inspection trip by Messrs. Derrick and Abraham on 10 September 1993 revealed all stone bank stabilization structures to be complete and of high quality. All bendway weirs on the right descending bank appeared to be located properly. However, due to contractor error, all but two weirs on the left descending bank were located improperly. Five of these weirs were completely relocated and eleven weirs were reangled. A comprehensive discussion of this issue will be included in the 1994 DEC report.

The first impression of the weirs is that they appear large relative to the stream size. This could be misleading as some settling and loss of stone can be expected. The new channel shape, location, and geometry should indicate how well the weirs are "sized" to the stream.

### Long-term monitoring of the study reach

Current plans to monitor and document the Harland Creek demonstration site include the use of occasional field observations and notes, 35mm photographs, videotape, bed surveys, and (possibly) aerial photographs.
Dr. Shields will make an informal evaluation of the environmental effects of the project. A report on the results of the monitoring of the study reach will be assembled and published in 1995. This study reach was added to the long-term monitoring sites and will be evaluated as part of that effort.

### Technology transfer

Progress, accomplishments, and results of this work unit were reported to the Vicksburg District through meetings, telephone conversations, photographic documentation, field reconnaissance trips, a series of monthly progress reports, and the DEC quarterly review meetings. Using photographs and videotape, study results are also prominently displayed via the portable DEC multimedia display center.

This report documents all work unit accomplishments through September 1993. A future report will document all work and monitoring of the study site through September 1994.

### Future test ideas

This "demonstration project" was designed with the understanding that all bank protection measures should be successful i.e., no flanking, catastrophic failures, or excessive maintenance allowed. This would be a difficult mandate to satisfy when using conventional bank protection structures. Since the willow posts and bendway weirs were basically untried for bendway protection in a small southern stream, the designers were understandably cautious on the designs used in this study. The authors suggest that if the current project succeeds, the following parameters should be investigated to fully maximize the design potential of the bank protection structures:

*a.* Bendway weirs spaced further apart, 120- and 150-ft spacings suggested.

*b.* Bendway weirs built level-crested, a 2-ft minimum section suggested.

*c.* Don't key all bendway weirs into the bank. Possibly only the first and last weirs of a field or every other weir in a field could be keyed in.

*d.* Bendway weirs not keyed into the bank. Keyways replaced with bank paving around the weir root.

*e.* Willow posts be planted further apart, 5- or 6-ft spacings suggested.

*f.* Willow rows be spaced further apart, 4- or 5-ft spacings suggested. Caution should be exercised here. In some areas an increased distance between rows could put the last rows quite high on the bank. If the bottoms of the willow posts are not in the water table, slower growth, or a higher failure rate, or both, could result.

# 9 FY 94 Work Plan

This work plan presents the work areas, funding requirements, and reporting activities for the proposed DEC Project Monitoring Program to be conducted by the Hydraulics Laboratory at WES.

The purpose of the DEC Project Monitoring Program is to evaluate and document watershed response to the implemented project features. One major goal of the DEC Project is to reduce sediment yield to the Yazoo River. Therefore, a major objective of the monitoring program is to determine the effectiveness of DEC Project features in reducing sediment yield. Documentation of watershed responses to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds.

This work plan proposes 11 technical areas for the DEC monitoring program that will effectively monitor the major physical processes of erosion. The following areas are to be monitored and/or addressed:

*a.* Stream gauging.

*b.* Data collection and data management.

*c.* Hydraulic performance of structures.

*d.* Channel response.

*e.* Hydrology.

*f.* Upland watersheds.

*g.* Reservoir sedimentation.

*h.* Environmental aspects.

*i.* Bank stability.

*j.* Design tools.

*k*. Technology transfer.

WES is proposing significant activities in all technical areas except stream gauging and environmental aspects. A major focus of the FY 94 effort will be in watershed sediment yield analysis. Detailed investigations of procedures that require sediment yield analysis are proposed in three technical areas (channel response, upland watersheds, and design tools). These efforts should complement each other, and lead to results that will quantify the amount of sediment reduction achieved by the DEC Project.

The following is a general description of the work to be performed in the nine technical areas by WES. The specific work tasks discussed in each work area should be viewed as a starting point for planning the FY 94 monitoring program. It is anticipated that the monitoring program will need to be modified as data are collected and analyzed and as new and different areas of concern develop. To accomplish this, the Hydraulics Laboratory will work closely with Vicksburg District personnel and will schedule quarterly review sessions with the Vicksburg District. This will allow the monitoring program to be adjusted as necessary to meet the needs of the DEC Project.

## Data Collection and Data Management

The purpose of the data collection and data management work area is to assemble, to the extent possible, all the data that have been collected to date in the DEC project, and to develop an engineering database/GIS that is continually updated as new data are collected and analyzed. The database resides on an Intergraph computer platform, and access to the database is currently through Intergraph workstations. During FY 94, after planned modifications, the database will become accessible to all participants in the DEC project. The database now contains aerial and satellite photography, watershed maps with DEC structure reports, USGS digital elevation grids, USGS quadrangle maps, soil type grids, and land use grids. The database now contains the information needed to use the small watershed hydrology design tool delivered during FY 92 for the hydrologic design of riser pipes. The hydraulic, sedimentation, and geometric survey data being collected in the monitoring program will be added to the database, along with various project feature designs and specifications, trip reports and field observations, and an index of study reports by others. As data are placed into the database, it will be used by WES to effectively conduct tasks in the monitoring program such as channel response evaluations and sediment yield reduction studies and should also become increasingly useful to the Vicksburg District for engineering activities related to watershed erosion control in the DEC Project.

For FY 94, the placement of collected data into the engineering database will continue, including WES, CSU, Vicksburg District, ARS, and SCS data. Historical data will continue to be added as necessary. Another task will be to provide access to the database for GRASS users (ARS and SCS). This will be done by adding translator software that will interface between Intergraph and

GRASS formats. Database maintenance, software updates, and user support will be conducted as necessary. Emphasis will be placed on ensuring that Vicksburg District personnel have complete and easy access to all data and features in the database. The major FY 94 tasks are as follows:

*a.* Continue stage data collection at established gauging stations.

*b.* Continue discharge measurements at established stations.

*c.* Continue quality control processing of stage data.

*d.* Develop stage-discharge rating curve for each established gauging station.

*e.* Continue routing database maintenance, updates, and support.

*f.* Continue to build engineering database as new data become available.

*g.* Add translator software to database for GRASS users (ARS and SCS).

## Hydraulic Performance of Structures

Two grade-control structures have been selected for detailed data collection to evaluate hydraulic performance. Data collection and data analysis will be a continuing effort in FY 94 for these selected structures. An annual visual inspection program for all grade control structures in the DEC Project has been established and the first year inspection has been completed and reported. An annual visual inspection to include selected bank stabilization structures will be initiated in FY 94.

The FY 94 focus in this technical area will be to continue data collection and analysis for the Long and Hotophia Creeks grade control structures. Model discharge coefficients will be correlated with field measurements, and discharge ratings will be established for these structures. The effectiveness of using grade control structures for stream gauging will be evaluated. The annual visual inspection of grade control structures will be repeated and expanded to include selected bank stabilization structures. The major FY 94 tasks are as follows:

*a.* Correlate model and prototype hydraulic performance of the Long Creek low-drop structure and the Hotophia Creek high-drop structure.

*b.* Conduct and document visual inspection of selected grade control structures in the DEC watersheds.

*c.* Conduct and document visual inspection of selected bank stabilization structures in the DEC watersheds.

*d.* Develop discharge rating curves for the long-term monitoring sites using stage, discharge gauging, and survey data.

## Channel Response

The channel response monitoring is being directed toward channel sedimentation. Monitoring for channel sedimentation includes a geomorphic update of selected watersheds annually. In addition to the geomorphic update, 22 sites where structures exist or are anticipated will continue to be intensively monitored. Channels upstream and downstream of the structures will continue to be monitored for changes in cross section, thalweg changes, berm formation, bank failure, and vegetation development. Selected sites where no structures are planned will also continue to be monitored. These five sites serve as control and will assist in the evaluation of the channel response to structures. Structures and channels at selected long-term monitoring sites have been instrumented for stage and discharge by WES. Suspended sediment concentrations are being measured at other locations by USGS. Bed-material samples at the 23 long-term sites are being collected by CSU. HEC-6 and the computer program SAM will be used to predict the stability of channels monitored by this work effort.

For FY 94, the channel response technical area accomplishments will be the continued data collection and analysis at the 23 long-term monitoring sites. Detailed geomorphic studies for selected watersheds (Burney Branch and Abiaca Creek) will be performed. The Hotophia Creek sediment reduction study will be conducted as a team effort with the Vicksburg District, ARS, and USGS. The approach will be two-phased, with Phase 1 using available data and the Universal Soil Loss Equation (USLE), and Phase 2 using the HEC-6 model. The objective will be to quantify the reduction in sediment yield as a result of the DEC Project. The following are major FY 94 tasks:

*a.* Conduct long-term monitoring at 23 selected sites (CSU Contract).

*b.* Conduct Hotophia Creek sediment reduction study.

*c.* Conduct detailed geomorphic studies (Burney Branch and Abiaca Creek).

## Hydrology

Rainfall provides the energy to sustain erosional processes. The ability to accurately measure rainfall and compute runoff is crucial in the design of stable flood control channels. Accurate flow rates are needed to properly design functional project features and maintain stability in the channel system.

Work conducted during FY 93 comparing the applicability of HEC-1 and CASC2D models at the Goodwin Creek watershed has indicated that CASC2D provides much better overall performance than HEC-1 for small ungaged watersheds. Based on the Goodwin Creek results, hydrologic models (CASC2D) of a minimum of six DEC watersheds will be developed in FY 94. The hydrologic modeling and the hydraulic structures monitoring are being coordinated so that the hydrologic parameters used in CASC2D can be verified at locations in the watersheds where USGS gauging stations do not exist. The CASC2D model is well suited to the database/GIS methodology being implemented in the DEC monitoring program, since it is a two-dimensional, cell-based model. Once the watershed models are developed, they will also be used at the long-term monitoring sites, using observed hydrology and assisting in the development of stage-discharge rating curves as needed. Accurate flow calculations will increase the usefulness and utility of studies being performed in the channel response technical area.

During FY 94, two versions of CASC2D will be combined into one comprehensive program. The first version was developed by Dr. Fred Ogden at the University of Iowa. This version has an improved channel routing routine that handles irregular shaped cross sections and hydraulic structures (i.e., culverts, weirs, low-drops, high-drops, and bridges). The second version of CASC2D was developed by Dr. Bahram Saghafian at the Construction Engineering Research Laboratory (CERL). This version has been linked with the GRASS GIS and contains a soil moisture accounting routine that allows single events or continuous simulations. The GRASS GIS is a U.S. Army Corps of Engineers product developed at CERL. Another feature to be added, watershed erosion, will allow the model to be used for sediment yield calculations. The addition of a sediment capability is being done in cooperation with the ARS Sedimentation Laboratory under the direction of Dr. Carlos Alonzo.

The application of CASC2D, with improved channel routing, continuous simulations, GRASS GIS linkage, and watershed erosion, will allow basin-wide simulations over a long period of time (i.e., 20 years). With CASC2D linked to a GIS, different types or numbers of hydraulic structures can be tested for sediment yield reduction. Also, predicted changes in land use can be tested for future impact on sediment yield. The following are major FY 94 tasks:

*a.* Prepare CASC2D hydrology model for application to DEC watersheds.

*b.* Construct CASC2D models for a minimum of six watersheds in support of sediment yield studies.

## Upland Watersheds

Two areas related to the upland watershed that require monitoring are (a) system sediment loading (sediment yield) and (2) sediment production from gully formation. Stabilization measures are being installed in the DEC project watersheds to reduce erosion. The purpose of the upland watershed technical area is to determine if there is a measurable change in the quantity of sediment being transported from each watershed for the next 5 years. Data that have already been collected by the USGS and ARS for the past 5 years will be analyzed to serve as the basis for future comparisons. Numerical modeling of the sediment runoff from the watersheds will be incorporated into the data analysis and interpretation process. Sediment production from two or three active gullies will be analyzed by comparing surveys made prior to the design of drop pipes and the survey made just prior to construction of the drop pipes.

For FY 94, WES will test the applicability of the sediment yield model, GISSRM, on the Goodwin Creek watershed. The model will be calibrated and pre- and post-DEC comparisons of sediment yield will be made. If the approach is successful, the model should have applicability to other DEC watersheds. Major FY 94 tasks include the following:

*a.* Separate sediment yield into basic sediment sources (land surface, gully, bank, and bed erosion).

*b.* Validate Sediment Yield Model (GISSRM) to Goodwin Creek.

*c.* Test selected scenarios for sediment yield.

## Reservoir Sedimentation

The major sources of reservoir sediment deposits are upland erosion, erosion of the channel banks, and erosion of the channel bed. The reduction of the inflowing sediment load is being addressed in the channel response, bank stability, and upland watershed technical areas. The results of the analysis performed in these areas will be used to determine the effects of the project on reservoir sedimentation.

During FY 94, Phase 1 of the study to evaluate the impact of the Hickahala/Senatobia Creek watershed on sedimentation in Arkabutla Reservoir will be completed. Major FY 94 tasks include the following:

*a.* Evaluate historical deposition in the Arkabutla Reservoir.

*b.* Evaluate historical land use in Hickahala/Senatobia Creek watershed.

*c.* Evaluate gauge data from Hickahala Creek (sediment and water).

*d.* Validate sediment yield model (GISSRM) using historical records from Hickahala Creek.

*e.* Test "no DEC" and "ultimate DEC" scenarios for impact on Arkabutla Reservoir sedimentation.

## Bank Stability

Channel bank stability depends on hydraulic parameters related to flow conditions and the characteristics of the materials in the banks. During FY 94, an evaluation of bioengineering techniques for bank stabilization, initiated in FY 93, will be continued at the Harland Creek test reach. The test reach also includes a bendway weir site, which will be monitored during the year. In addition, five additional bioengineering bank stabilization sites will be designed (CSU contract) in cooperation with ARS. Also during FY 94, the "March" slope stability analysis procedure will be added to SAM for application to DEC channels. Finally, the aerial visual inspection will be conducted for all 15 watersheds in cooperation with ARS. Major FY 94 tasks include the following:

*a.* Conduct monitoring of Harland Creek bank stabilization test reach (bioengineering techniques and bendway weirs).

*b.* Evaluate performance of bioengineering techniques and bendway weirs at Harland Creek test reach.

*c.* Add "March" bank stability analysis procedures to SAM.

*d.* Design bioengineering bank stabilization features for five locations.

*e.* Conduct aerial visual inspection of 15 DEC watersheds for bank stability.

## Design Tools

The design procedures and techniques used in the design of the different features of the DEC project have the potential for national and international applications. To demonstrate their applicability, the procedures need to be organized into a systematic method, documented, and guidance prepared to illustrate their use.

As reported in Chapter 10 of Raphelt et al. (1993), refinement of the DEC traditional hydraulic design techniques is possible by incorporating the latest technology from the Flood Control Channels Research Program into the procedures. Of particular interest is the method for designing the height, spacing, and sequence of construction of grade control structures. The design tools proposed in this work will formalize those procedures, document their application, and train Vicksburg District hydraulic engineers in their use. The following are FY 94 tasks:

*a.* Organize procedures into a systematic method.

*b.* Validate the method on a DEC watershed.

*c.* Prepare guidance that will illustrate their application to a DEC watershed.

## Technology Transfer

Technology transfer is an important part of the DEC program and will be given high priority at WES during the monitoring program. With Vicksburg District approval, WES personnel will present results at national and international technical conferences and symposiums. When appropriate, WES will host workshops and training classes for both Corps and non-Corps personnel. WES will annually report on the DEC monitoring program using several different formats. For FY 94, the following activities in technology transfer will be accomplished:

*a.* A detailed WES technical report, including appendixes under separate cover, on monitoring, data collection, data analysis, and project evaluation.

*b.* An updated engineering database on the Intergraph system including aerial photos, surveys (channel and structural), and results of numerical studies to be provided to the Vicksburg District.

*c.* A short executive summary report (5 pages or less).

# 10 General Assessments After 2 Years

As the result of FY 93 activities, the following assessments are given:

*a.* Preliminary analysis of channel surveys from the 22 long-term monitoring site and field observations have shown the following:

(1) Approximately 122,000 ft of stream channel have been surveyed at the 22 sites.

(2) The 1993 surveys are the second data set for the sites, and comparison of the 1992 and 1993 data have provided a basis for establishing trends in channel response and structure performance.

(3) Calculation of sediment discharge at the monitoring sites indicates that the sediment discharge per unit width variation is extreme.

(4) Although not conclusive, the preliminary analysis indicates that the high-drop structures result in lower sediment yield than the low-drop structures.

*b.* The collection of continuous stage measurements at 31 sites continues with minimal downtime and acceptable accuracy.

*c.* The inspection of drop structures indicated six common problems:

(1) Riprap was displaced from the face of the weir (on 41 percent of the structures).

(2) The channel bank upstream or downstream of the structure is failing (24 percent).

(3) Bank erosion or piping beneath the riprap is occurring caused by overbank drainage (24 percent).

(4) Riprap is launching at the upstream or downstream apron (28 percent).

(5) Severe headcutting is migrating into the basin (17 percent).

(6) Woody vegetation has become established in the upstream or downstream apron, and is impairing the conveyance of the weir unit discharge of the structure (19 percent).

*d.* The two-dimensional hydrology model, CASC2D, outperformed the one-dimensional model, HEC-1, on the Goodwin Creek watershed application. CASC2D was shown to have significant advantages to HEC-1 in largely ungauged watersheds such as the DEC watersheds. The CASC2D should prove useful in providing the hydrology needed to use design tools under development.

*e.* The applicability of the SAM-based design procedure for grade control spacing was demonstrated for Byhalia Creek.

# References


Abt, Steven R., Watson, Chester C., Johns, Derek D., Hamilton, Glenn B., Garton, Andrew D., Florentin, C. Bradley, and Thornton, Christopher I. (1991). "Riprap sizing criteria for ARS-type drop structures," prepared for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, under Contract No. DACW39-90-K-0027, by Colorado State University, Fort Collins, CO.

Biedenharn, D. S., Little, C. D., and Thorne. C. R. (1987). "Magnitude and frequency analysis in large rivers." *Hydraulic engineering:* Proceedings of the 1987 National Conference on Hydraulic Engineering, Williamsburg, VA, August 3-7, 1987. Robert M. Ragan, ed., ASCE, New York, 782-787.

Colson, B. E., and Hudson, J. W. (1976). "Flood frequency of Mississippi streams," prepared for the Mississippi State Highway Department, Jackson, MS, by the U.S. Geological Survey, Washington, DC.

Freeman, Gary E., Benn, Lisa W., Raphelt, Nolan K., and Thomas, William A. (1992). "Abiaca Creek sedimentation study," Miscellaneous Paper HL-92-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

FTN Associates, Ltd. (1987). "Data compilation and preliminary assessment of the hydrologic and hydraulic characteristics of Long Creek Watershed," prepared for the U.S. Army Engineer District, Vicksburg.

Grissinger, E. H., and Murphey, J. B. (1982). "Present channel stability and Late-Quaternary valley deposits in northern Mississippi," Special Publication No. 6, International Association of Sedimentology, 241-250.

Happ, S. C., Rittenhouse, and Dobson, G. C. (1940). "Some principles of accelerated stream and valley sedimentation," Technical Bulletin 695, U.S. Department of Agriculture, Washington, D.C.

Harvey, M. D. (1984). "A geomorphic evaluation of a grade-control structure in a meandering channel." *River Meandering: Proceedings of the*

*Conference Rivers '83,* Charles M. Elliott, ed., ASCE, New York, 284-294.

Harvey, M. D., and Watson, C. C. (1988). "Channel response to grade-control structures on Muddy Creek, MS," *Regulated Rivers: Research and Management* 2, 79-92.

Headquarters, U.S. Army Corps of Engineers. (1990 (17 Sep)). "Stability of flood control channels," EC 1110-8-1(FR), expired 30 September 1991, U.S. Government Printing Office, Washington, DC.

Lenzotti and Fullerton Consulting Engineers, Inc. (1990). "Hydrologic analysis of the Coldwater River watershed," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Little, W. C., and Murphey, J. B. (1982). "Model study of low drop grade control structures," *Journal of Hydraulics Division, ASCE,* 108(HY10), 1132-1146.

Northwest Hydraulic Consultants, Inc. (1988). "Black and Fannegusha Creek hydrologic and hydraulic analysis," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Northwest Hydraulic Consultants, Inc. (1989). "Long Creek watershed field investigation and geomorphic analyses," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Osman, A. M., and Thorne, C. R. (1988). "River bank stability analysis," *Journal of Hydraulic Engineering,* ASCE, 114(2), 134-150.

Pokrefke, Thomas J. (1993). "Demonstration Erosion Control Project Monitoring Program; Fiscal Year 1992 report; Volume VII; Appendix F, Model study of bendway weirs as bank protection," Technical Report HL-93-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Raphelt, Nolan K., Waller, Terry N., Abraham, David D., Brown, Bobby J., Johnson, Billy E., Martin, Sandra K., Thomas, William A., Hubbard, Lisa C., Watson, Chester C., Abt, Steven R., Thorne, Colin R. (1993). "Demonstration Erosion Control Project Monitoring Program, Fiscal Year 1992 report; Volume I, Main text," Technical Report HL-93-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 59-65.

Schumm, S. A., Harvey, M. D., and Watson, C. C. (1984). "Incised channels: Morphology, dynamics and control," Water Resources Publications, Littleton, CO, 200 p.

Scott, Stephen H. (1992). "Applying ultrasonic surface detectors to hopper dredge production monitoring," Technical Note DRP-3-06, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Simon, A., and Hupp, C. R. (1987). "Geomorphic and vegetative recovery processes along modified Tennessee streams: an interdisciplinary approach to distributed fluvial systems," IAHS-AISH Publ. No. 167, 251-262.

Simons, Li and Associates. (1987). "Preliminary data collection, hydrologic, hydraulic and geomorphic analysis for the Hickahala/Senatobia Creek watershed; Volume 1, Main report," Fort Collins, Co.

Soil Conservation Service. (n.d.) "Drop spillways," Section 11, *National Engineering Handbook,* U.S. Department of Agriculture, Washington, DC.

Thomas, William A., Copeland, Ronald R., Raphelt, Nolan K., and McComas, Dinah N. "Hydraulic design of channels (SAM)" (in preparation), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Thorne, C. R., Biedenharn, D. S., and Combs, P. G. (1988). "Riverbank instability due to bed degradation." *Hydraulic Engineering; Proceedings of the 1988 National Conference,* Colorado Springs, CO, August 8-12, 1988, Steven R. Abt and Johannes Gessler, ed., Hydraulics Division, American Soceity of Civil Engineers, New York, 132-137.

Thorne, C. R., Murphey, J. B., and Little, W. C. (1981). "Stream channel stability; Appendix D, Bank stability and bank material properties in the Bluffline streams of northwest Mississippi," U.S. Department of Agriculture Sedimentation Laboratory, Oxford, MS.

U.S. Army Engineer Hydrologic Engineering Center. (1982). "HEC-2: Water surface profiles; users manual," Davis, CA.

U.S. Army Engineer Hydrologic Engineering Center. (1990). "HEC-1: Flood hydrograph package; user's manual," Report CPD-1A, Davis, CA.

U.S. Army Engineer Hydrologic Engineering Center. (1993). "HEC-6: Scour and deposition in rivers and reservoirs; user's manual," Report CPD-6, Davis, CA.

U.S. Geological Survey. (1991 (Dec)). *Instrument News,* Hydrologic Instrumentation Facility, 20.

Waller, Terry N., and Hubbard, Lisa C. (1993). "Demonstration Erosion Control Project Monitoring Program; Fiscal Year 1992 report; Volume IV; Appendix C, Batupan Bogue watershed profiles and cross sections," Technical Report HL-93-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Water Engineering and Technology, Inc. (1986). "Preliminary basin investigations, Hotophia Creek and Batupan Bogue," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Water Engineering and Technology, Inc. (1987a). "Investigation of erosion and flood control alternatives for Batupan Bogue watershed," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Water Engineering and Technology, Inc. (1987b). "Recommended improvements for stabilization of Hotophia Creek watershed," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Water Engineering and Technology, Inc. (1989a). "Development of alternatives for flood and erosion control in the Abiaca Creek watershed," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Water Engineering and Technology, Inc. (1989b). "Field investigation and geomorphic analysis of Abiaca Creek watershed," Report to U.S. Army Engineer District, Vicksburg, Vicksburg, MS.

Water Engineering and Technology, Inc. (1990). "Value engineering of ARS-type low drop structures," Prepared under Contract No. DACW39-89-D-0001 for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Watson, Chester C., and Harvey, Michael D. (1988). "Channel response to SCS Type-C grade-control structures on Burney Branch, Mississippi." *Hydraulic Engineering; Proceedings of the 1988 National Conference.* Colorado Springs, CO, August 8-12, 1988, Steven R. Abt and Johannes Gessler, ed., Hydraulics Division, American Society of Civil Engineers, New York, 776-781.

Watson, Chester C., Abt, Steven R., and Hogan, Scott. (1993). "Monitoring of DEC drop structures," prepared by Colorado State University, Fort Collins, CO, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Zevenbergen, L. W., Spitz, W. J., Watson, C. C., and Biedenharn, D. S. (1990). "Evaluation of streambank erosion control demonstration projects." *Hydraulic engineering: proceedings of the 1990 national conference,* San Diego, CA, July 30-August 3, 1990. Howard H. Chang and Joseph C. Hill, ed., American Society of Civil Engineers, New York, Vol 2, 832-837.

## Table 1
## Instrument Accuracy/Calibration Check

| Site No. | Site Name | Sutron ID No. | Sutron Date | Stevens ID No. | Stevens Date |
|---|---|---|---|---|---|
| 1 | HARLAND | 030120 | 04-19-93 | | |
| 2 | FANNEGUSHA | 030220 | 04-26-93 | 030210 | 05-26-93 |
| 3 | ABIACA | 010320 | 04-26-93 | | |
| 5 | COILA | 010520 | 05-10-93 | | |
| 7 | NOLEHOE | 050730 | 04-21-93 | | |
| | | 050740 | 04-21-93 | | |
| 8 | LICK | 050820 | 03-29-93 | | |
| 9 | RED BANKS | | | 050910 | 06-08-93 |
| 10 | LEE | 051020 | 04-21-93 | | |
| 11 | HICKAHALA | | | 061120 | 06-11-93 |
| | | | | 061130 | 05-10-93 |
| 12 | BURNEY BRANCH | | | 041220 | 06-21-93 |
| | | | | 041230 | 06-15-93 |
| 13 | HOTOPHIA | | | 071330 | 06-15-93 |
| | | | | 071340 | 06-15-93 |
| 14 | OTOUCALOFA | 091420 | 10-07-92 | | |
| 15 | SARTER | 091520 | 05-05-93 | | |
| 16 | PERRY | 021640 | 08-20-92 | | |
| | | 021650 | 08-20-92 | | |
| 17 | SYKES | 021720 | 04-22-93 | | |
| 18 | WORSHAM WEST | | | 021811 | 05-11-93 |
| 18 | WORSHAM MIDDLE | 021821 | 04-28-93 | 021823 | 05-27-93 |
| 18 | WORSHAM EAST | 021831 | 04-28-93 | 021833 | 05-27-93 |
| 19 | JAMES WOLF | | | 061920 | 06-21-93 |
| | | | | 061930 | 06-21-93 |
| 20 | LONG | 082020 | 04-20-93 | 082030 | 06-16-93 |
| | | | | 082050 | 06-16-93 |

## Table 2
## Information Available on Engineering Database

| Watershed | El[1] | Soil Type[2] | Land Use[2] | Slope[1] | Curve Number[2] | Hydraulic Structures | Quad Maps | Spot Photos[3] |
|---|---|---|---|---|---|---|---|---|
| Abiaca | | | | | | X | X | X |
| Batupan Bogue | | | | | | | X | X |
| Black-Fannegusha | | | | | | | X | X |
| Burney Branch | | | | | | | X | X |
| Cane-Mussacuna | X | X | X | X | X | X | X | X |
| Coldwater | X | X | X | X | X | X | X | X |
| Hickahala-Senatobia | X | X | X | X | X | X | X | X |
| Hotophia | | | | | | | X | X |
| Hurricane-Wolf | X | X | X | X | X | X | X | X |
| Long | X | X | X | X | X | X | X | X |
| Otoucalofa | | | | | | | X | X |
| Pelucia | | | | | | | X | X |
| Toby Tubby | | | | | | | X | X |

[1]Grid size = 30 m × 30 m
[2]Grid size = 1 acre
[3]Satellite photography at 10-m resolution

Table 3
USGS Gauge Sites

| Site No. | Site | Gauge No. | Area (sq mile) | Length (miles) | Slope (ft/mile) | Q2 (cfs) | Q5 (cfs) | Q10 (cfs) | Q25 (cfs) | Q50 (cfs) | Q100 (cfs) | Q200 (cfs) | Q500 (cfs) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1) | Hotopha Creek near Batesville | 07273100 | 35.10 | 14.8 | 12.6 | 4104 | 6297 | 7703 | 9572 | 10732 | 12550 | 13589 | 15424 |
| 2) | Otoucalofa Creek Canal near Water Valley | 07274252 | 97.1 | 20.1 | 8.3 | 7852 | 12523 | 15734 | 20017 | 22722 | 26763 | 29224 | 33308 |
| 3) | Peters (Long) Creek near Pope | 07275530 | 79.2 | 13 | 15.3 | 9406 | 15110 | 18966 | 24213 | 27332 | 31859 | 34365 | 38928 |
| 4) | Hickahala Creek near Senatobia | 07277700 | 122 | 19.6 | 9.9 | 10531 | 17006 | 21458 | 27425 | 31108 | 36568 | 39789 | 45270 |
| 5) | Senatobia Creek near Senatobia | 07277730 | 82 | 15.8 | 10.3 | 7757 | 12386 | 15566 | 19855 | 22512 | 26395 | 28697 | 32637 |
| 6) | Skuna River at Bruce | 07283000 | 254 | 31.2 | 3.6 | 11389 | 18726 | 24194 | 31513 | 36405 | 43431 | 48222 | 55422 |
| 7) | Harland Creek near Howard | 07287404 | 62.1 | 16.3 | 11 | 6260 | 9845 | 12238 | 15436 | 17419 | 20417 | 22168 | 25195 |

| Table 4<br>Burney Branch, Site 12 | | |
|---|---|---|
| Parameter | 1992 | 1993 |
| **Segment 1** | | |
| Bed slope | 0.0015 | 0.0016 |
| Composite section width, ft | 103.6 | 93.7 |
| Composite section depth, ft | 7.3 | 6.11 |
| Composite section slope | 0.00049 | 0.0011 |
| Sediment transport, tons/day | 2,570 | 8,727 |
| Sediment transport, tons/day/ft | 25 | 93 |
| **Segment 2** | | |
| Bed slope | 0.0020 | 0.0018 |
| Composite section width, ft | 105.0 | 97.7 |
| Composite section depth, ft | 7.1 | 6.9 |
| Composite section slope | 0.00052 | 0.00064 |
| Sediment transport, tons/day | 2,836 | 4,057 |
| Sediment transport, tons/day/ft | 27 | 42 |

| Table 5<br>Harland Creek, Site 1 | | |
|---|---|---|
| Parameter | 1992 | 1993 |
| Bed slope | 0.0009 | 0.0009 |
| Composite section width, ft | 80.2 | 85.2 |
| Composite section depth, ft | 5.72 | 5.88 |
| Composite section slope | 0.0009975 | 0.0008091 |
| Sediment transport, tons/day | 4,217.98 | 2,989.02 |
| Sediment transport, tons/day/ft | 52.59 | 35.08 |

**Table 6**
**Hickahala Creek, Site 11**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | -- | 0.0011 |
| Composite section width, ft | 53.9 | 54.4 |
| Composite section depth, ft | 5.3 | 5.5 |
| Composite section slope | 0.0012 | 0.0010 |
| Sediment transport, tons/day | 1,040 | 2,819 |
| Sediment transport, tons/day/ft | 19 | 52 |
| **Segment 2** | | |
| Bed slope | 0.0026 | 0.0016 |
| Composite section width, ft | 53.9 | 40.7 |
| Composite section depth, ft | 5.3 | 5.4 |
| Composite section slope | 0.0012 | 0.0020 |
| Sediment transport, tons/day | 1,040 | 10,024 |
| Sediment transport, tons/day/ft | 19 | 246 |
| **Segment 3** | | |
| Bed slope | 0.0034 | 0.0024 |
| Composite section width, ft | 58.6 | 38.5 |
| Composite section depth, ft | 5.0 | 5.0 |
| Composite section slope | 0.0012 | 0.0030 |
| Sediment transport, tons/day | 989 | 18,524 |
| Sediment transport, tons/day/ft | 17 | 481 |

**Table 7**
**Red Banks Creek, Site 9**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0009 | 0.0009 |
| Composite section width, ft | 111.3 | 115.2 |
| Composite section depth, ft | 6.8 | 6.4 |
| Composite section slope | 0.0012 | 0.0013 |
| Sediment transport, tons/day | 13,817 | 15,775 |
| Sediment transport, tons/day/ft | 124 | 137 |
| **Segment 2** | | |
| Bed slope | 0.0020 | 0.0018 |
| Composite section width, ft | 107.8 | 100.9 |
| Composite section depth, ft | 6.0 | 6.2 |
| Composite section slope | 0.0018 | 0.0020 |
| Sediment transport, tons/day | 29,946 | 34,596 |
| Sediment transport, tons/day/ft | 278 | 343 |

**Table 8**
**Lee Creek, Site 10**

| Parameter | 1992 | 1993 |
|---|---|---|
| Bed slope | 0.0020 | 0.0016 |
| Composite section width, ft | 48.4 | 47.0 |
| Composite section depth, ft | 4.12 | 4.3 |
| Composite section slope | 0.0017 | 0.0015 |
| Sediment transport, tons/day | 4,104 | 3,628 |
| Sediment transport, tons/day/ft | 85 | 77 |

**Table 9**
**Lower Hotophia Creek, Site 13**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0018 | 0.002 |
| Composite section width, ft | | 64.7 |
| Composite section depth, ft | | 12.7 |
| Composite section slope | | 0.0018 |
| Sediment transport, tons/day | | 45,161 |
| Sediment transport, tons/day/ft | | 698 |
| **Segment 2** | | |
| Bed slope | 0.0075 | 0.0069 |
| Composite section width, ft | 47.4 | 71 |
| Composite section depth, ft | 3.63 | 5.3 |
| Composite section slope | 0.00467 | 0.00058 |
| Sediment transport, tons/day | 36,738 | 1,249 |
| Sediment transport, tons/day/ft | 775 | 18 |
| **Segment 3** | | |
| Bed slope | 0.0053 | 0.0052 |
| Composite section width, ft | 40.40 | 45.10 |
| Composite section depth, ft | 4.3 | 4.4 |
| Composite section slope | 0.0030 | 0.0029 |
| Sediment transport, tons/day | 25,225 | 19,589 |
| Sediment transport, tons/day/ft | 624 | 434 |

**Table 10**
**Nolehoe Creek, Site 7**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0038 | 0.0037 |
| Composite section width, ft | 35 | 40.6 |
| Composite section depth, ft | 4.4 | 4.2 |
| Composite section slope | 0.0032 | 0.0026 |
| Sediment transport, tons/day | 726 | 428 |
| Sediment transport, tons/day/ft | 21 | 11 |
| **Segment 2** | | |
| Bed slope | 0.0098 | 0.0103 |
| Composite section width, ft | 35.6 | 32.7 |
| Composite section depth, ft | 3.5 | 4.0 |
| Composite section slope | 0.0063 | 0.0049 |
| Sediment transport, tons/day | 2,635 | 1,679 |
| Sediment transport, tons/day/ft | 74 | 51 |

**Table 11**
**Sarter Creek, Site 15**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0028 | 0.0030 |
| Composite section width, ft | 32.9 | 32.0 |
| Composite section depth, ft | 4.6 | 5.1 |
| Composite section slope | 0.0033 | 0.0024 |
| Sediment transport, tons/day | 19,274 | 12,294 |
| Sediment transport, tons/day/ft | 586 | 384 |
| **Segment 2** | | |
| Bed slope | 0.0016 | 0.0019 |
| Composite section width, ft | 45.8 | 45.6 |
| Composite section depth, ft | 5.3 | 4.9 |
| Composite section slope | 0.0010 | 0.0014 |
| Sediment transport, tons/day | 2,696 | 3,928 |
| Sediment transport, tons/day/ft | 59 | 86 |

**Table 12**
**James Wolf Creek, Site 19**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0013 | 0.0016 |
| Composite section width, ft | 73.9 | 70.4 |
| Composite section depth, ft | 6.2 | 6.1 |
| Composite section slope | 0.0011 | 0.0013 |
| Sediment transport, tons/day | 7,785 | 11,577 |
| Sediment transport, tons/day/ft | 105 | 164 |
| **Segment 2** | | |
| Bed slope | 0.0020 | 0.0019 |
| Composite section width, ft | 76.7 | 76.1 |
| Composite section depth, ft | 5.8 | 5.8 |
| Composite section slope | 0.0013 | 0.0013 |
| Sediment transport, tons/day | 9,404 | 9,794 |
| Sediment transport, tons/day/ft | 123 | 129 |

**Table 13**
**Long Creek, Site 20**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0028 | 0.0029 |
| Composite section width, ft | 46.8 | 61.9 |
| Composite section depth, ft | 3.8 | 4.2 |
| Composite section slope | 0.0024 | 0.0011 |
| Sediment transport, tons/day | 8,768 | 2,062 |
| Sediment transport, tons/day/ft | 180 | 33 |
| **Segment 2** | | |
| Bed slope | 0.0018 | 0.0019 |
| Composite section width, ft | 57.3 | 76.3 |
| Composite section depth, ft | 3.9 | 3.5 |
| Composite section slope | 0.0016 | 0.0013 |
| Sediment transport, tons/day | 3,889 | 2,337 |
| Sediment transport, tons/day/ft | 68 | 31 |
| **Segment 3** | | |
| Bed slope | 0.0025 | 0.0024 |
| Composite section width, ft | 42.1 | 49.9 |
| Composite section depth, ft | 4.3 | 4.31 |
| Composite section slope | 0.0023 | 0.0016 |
| Sediment transport, tons/day | 8,783 | 4,145 |
| Sediment Transport, tons/day/ft | 209 | 83 |
| **Segment 4** | | |
| Bed slope | 0.0015 | 0.0007 |
| Composite section width, ft | 75 | 48 |
| Composite section depth, ft | 4.0 | 3.6 |
| Composite section slope | 0.00083 | 0.0032 |
| Sediment transport, tons/day | 1,332 | 12,986 |
| Sediment transport, tons/day/ft | 18 | 271 |

**Table 14**
**Summary of Watershed Data for Worsham Creek, Site 18**

| Creek | Segment | Drainage Area square miles | Bank-full Discharge cfs | 2-Year Discharge cfs | Sediment $D_{50}$ mm | Sediment Characterization |
|---|---|---|---|---|---|---|
| East Worsham | 1<br>2<br>3<br>4 | 9.5 | 1,935 | 1,935 | 0.25 | Fine sand |
| Middle Worsham | 1<br>2<br>3<br>4 | 5.2 | 1,153 | 1,153 | 0.30 | Medium sand |
| West Worsham | 1<br>2<br>3<br>4 | 4.0 | 1,096 | 1,096 | 0.31 | Medium sand |

**Table 15**
**East Worsham Creek, Site 18**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0027 | 0.0012 |
| Composite section width, ft | 50.5 | 49.5 |
| Composite section depth, ft | 6.2 | 6.5 |
| Composite section slope | 0.0019 | 0.0016 |
| Sediment transport, tons/day | | 21,145 |
| Sediment Transport, tons/day/ft | | 427 |
| **Segment 2** | | |
| Bed slope | 0.0027 | 0.0016 |
| Composite section width, ft | | 49.9 |
| Composite section depth, ft | | 6.0 |
| Composite section slope | | 0.0022 |
| Sediment transport, tons/day | | 32,223 |
| Sediment transport, tons/day/ft | | 659 |
| **Segment 3** | | |
| Bed slope | 0.0050 | 0.0025 |
| Composite section width, ft | | 69.4 |
| Composite section depth, ft | | 6.0 |
| Composite section slope | | 0.0011 |
| Sediment transport, tons/day | | 9,631 |
| Sediment transport, tons/day/ft | | 139 |
| **Segment 4** | | |
| Bed slope | 0.0050 | 0.0042 |
| Composite section width, ft | | 57.9 |
| Composite section depth, ft | | 6.1 |
| Composite section slope | | 0.0015 |
| Sediment transport, tons/day | | 17,024 |
| Sediment transport, tons/day/ft | | 294 |

## Table 16
## Middle Worsham Creek, Site 18

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0019 | 0.0014 |
| Composite section width, ft | 75.40 | 43.7 |
| Composite section depth, ft | 8.7 | 5.0 |
| Composite section slope | 0.009 | 0.0018 |
| Sediment transport, tons/day | 59 | 10,582 |
| Sediment transport, tons/day/ft | 1 | 242 |
| **Segment 2** | | |
| Bed slope | 0.0021 | 0.0017 |
| Composite section width, ft | 52 | 45.9 |
| Composite section depth, ft | 5.1 | 4.5 |
| Composite section slope | 0.0012 | 0.0023 |
| Sediment transport, tons/day | 4,237 | 13,762 |
| Sediment transport, tons/day/ft | 81 | 300 |
| **Segment 3** | | |
| Bed slope | -- | 0.0017 |
| Composite section width, ft | 52.0 | 49.9 |
| Composite section depth, ft | 5.1 | 5.3 |
| Composite section slope | 0.0012 | 0.0011 |
| Sediment transport, tons/day | 4,237 | 4,231 |
| Sediment transport, tons/day/ft | 81 | 85 |
| **Segment 4** | | |
| Bed slope | 0.007 | 0.0003 |
| Composite section width, ft | 39.8 | 38.9 |
| Composite section depth, ft | 4.8 | 5.4 |
| Composite section slope | 0.0025 | 0.0017 |
| Sediment transport, tons/day | 17,034 | 10,379 |
| Sediment transport, tons/day/ft | 428 | 287 |

## Table 17
## West Worsham Creek, Site 18

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0021 | 0.0018 |
| Composite section width, ft | 39.7 | 35.5 |
| Composite section depth, ft | 4.5 | 5.4 |
| Composite section slope | 0.0028 | 0.0019 |
| Sediment transport, tons/day | 18.43 | 11,014 |
| Sediment transport, tons/day/ft | 454 | 310 |
| **Segment 2** | | |
| Bed slope | 0.0033 | 0.0020 |
| Composite section width, ft | 36.2 | 39.4 |
| Composite section depth, ft | 4.8 | 4.9 |
| Composite section slope | 0.0026 | 0.0021 |
| Sediment transport, tons/day | 17,131 | 12,185 |
| Sediment transport, tons/day/ft | 473 | 305 |
| **Segment 3** | | |
| Bed slope | 0.0033 | 0.0025 |
| Composite section width, ft | 36.2 | 45.4 |
| Composite section depth, ft | 4.8 | 4.4 |
| Composite section slope | 0.0026 | 0.0024 |
| Sediment transport, tons/day | 17,131 | 13,000 |
| Sediment transport, tons/day/ft | 473 | 286 |
| **Segment 4** | | |
| Bed slope | 0.0042 | 0.0026 |
| Composite section width, ft | 31.2 | 30.1 |
| Composite section depth, ft | 4.5 | 4.4 |
| Composite section slope | 0.0045 | 0.0052 |
| Sediment transport, tons/day | 39,450 | 496.31 |
| Sediment transport, tons/day/ft | 1,246 | 1,649 |

**Table 18**
**Perry Creek, Site 16**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | | -0.0005 |
| Composite section width, ft | | 112.2 |
| Composite section depth, ft | | 7.85 |
| Composite section slope | | 0.0001468 |
| Sediment transport, tons/day | | 197.151 |
| Sediment transport, tons/day/ft | | 1.78 |
| **Segment 2** | | |
| Bed slope | | 0.0017 |
| Composite section width, ft | | 75.3 |
| Composite section depth, ft | | 6.09 |
| Composite section slope | | 0.0007516 |
| Sediment transport, tons/day | | 3,337.50 |
| Sediment transport, tons/day/ft | | 44.32 |
| **Segment 3** | | |
| Bed slope | | 0.0020 |
| Composite section width, ft | | 47.0 |
| Composite section depth, ft | | 5.96 |
| Composite section slope | | 0.0020929 |
| Sediment transport, tons/day | | 488.13 |
| Sediment transport, tons/day/ft | | 10.39 |
| **Segment 4** | | |
| Bed slope | | 0.0063 |
| Composite section width, ft | | 46.90 |
| Composite section depth, ft | | 5.29 |
| Composite section slope | | 0.0031240 |
| Sediment transport, tons/day | | 40,329.90 |
| Sediment transport, tons/day/ft | | 856.91 |

**Table 19**
**Otoucalofa Creek, Site 14, Segment 1**

| Parameter | 1992 | 1993 |
|---|---|---|
| Bed slope | 0.0009 | 0.0011 |
| Composite section width, ft | 96.8 | 89.4 |
| Composite section depth, ft | 8.56 | 8.55 |
| Composite section slope | 0.0009832 | 0.0011554 |
| Sediment transport, tons/day | 17,912.70 | 27,278.6 |
| Sediment transport, tons/day/ft | 185.05 | 305.13 |

**Table 20**
**Sykes Creek, Site 17, Segment 1**

| Parameter | 1992 | 1993 |
|---|---|---|
| Bed slope | 0.0019 | 0.0015 |
| Composite section width, ft | 74.9 | 80.8 |
| Composite section depth, ft | 6.09 | 5.49 |
| Composite section slope | 0.0015539 | 0.0018826 |
| Sediment transport, tons/day | 19,397.70 | 24,197.8 |
| Sediment transport, tons/day/ft | 258.98 | 299.48 |

**Table 21**
**Fannegusha Creek, Site 2, Segment 1**

| Parameter | 1992 | 1993 |
|---|---|---|
| Bed slope | 0.0017 | 0.0020 |
| Composite section width, ft | 67.0 | 73.8 |
| Composite section depth, ft | 7.66 | 7.39 |
| Composite section slope | 0.0015430 | 0.0014304 |
| Sediment transport, tons/day | 33,888.70 | 28,640.3 |
| Sediment transport, tons/day/ft | 505.80 | 388.08 |

**Table 22**
**Lick Creek, Site 8**

| Parameter | 1992 | 1993 |
|---|---|---|
| **Segment 1** | | |
| Bed slope | 0.0037 | 0.0030 |
| Composite section width, ft | 50.7 | 57.0 |
| Composite section depth, ft | 4.65 | 4.50 |
| Composite section slope | 0.0032 | 0.0028 |
| Sediment transport, tons/day | 20,572.10 | 15,827.80 |
| Sediment transport, tons/day/ft | 405.76 | 277.68 |
| **Segment 2** | | |
| Bed slope | 0.0030 | 0.0025 |
| Composite section width, ft | 37.60 | 41.90 |
| Composite section depth, ft | 5.75 | 6.32 |
| Composite section slope | 0.0028 | 0.0016 |
| Sediment transport, tons/day | 21,290.10 | 7,306.66 |
| Sediment transport, tons/day/ft | 566.23 | 174.38 |

## Table 23
## Segment List, DEC Monitoring Sites

| Site No. | Stream Name | Segment No. | Drop Structure Type[1] |
|---|---|---|---|
| 1 | Harland Creek | 1 | 0 |
| 2 | Fannegusha Creek | 1 | 0 |
| 3 | Abiaca Creek | 1 | 0 |
| 4 | | 1 | 0 |
| 5 | Coila Creek | 1 | 0 |
| 6 | Abiaca Creek | 1 | 0 |
| 7 | Nolehoe Creek | 2 | 0 |
| | | 1 | 0 |
| 8 | Lick Creek | 2 | 0 |
| | | 1 | 0 |
| 9 | Red Banks Creek | 2 | 0 |
| | | 1 | 3 |
| 10 | Lee Creek | 1 | 0 |
| 11 | Hickahala Creek | 1 | 0 |
| | | 2 | 1 |
| | | 3 | 1 |
| 12 | Burney Branch | 1 | 2 |
| | | 2 | 2 |
| 13 | Lower Hotophia Creek | 1 | 1 |
| 13 | Upper Hotophia Creek | 1 | 2 |
| 13 | Marcum Creek | 1 | 2 |
| 14 | Otoucalofa Creek | 1 | 0 |
| 15 | Sarter Creek | 1 | 0 |
| | | 2 | 0 |
| 16 | Perry Creek | 1 | 0 |
| | | 2 | 1 |
| | | 3 | 1 |
| | | 4 | 1 |
| 17 | Sykes Creek | 1 | 0 |

(Continued)

[1]0 = no drop structure
1 = low-drop structure
2 = high-drop structure
3 = chevron weir

## Table 23 (Concluded)

| Site No. | Stream Name | Segment No. | Drop Structure Type |
|---|---|---|---|
| 18E | East Worsham Creek | 1 | 0 |
| | | 2 | 0 |
| | | 3 | 1 |
| | | 4 | 1 |
| 18M | Middle Worsham Creek | 1 | 0 |
| | | 2 | 1 |
| | | 3 | 1 |
| | | 4 | 1 |
| 18W | West Worsham Creek | 1 | 0 |
| | | 2 | 1 |
| | | 3 | 1 |
| | | 4 | 1 |
| 19 | James Wolf Creek | 1 | 0 |
| | | 2 | 1 |
| 20 | Long Creek | 1 | 1 |
| | | 2 | 1 |
| | | 3 | 1 |
| | | 4 | 1 |
| 21 | Abiaca Creek | 1 | 0 |
| 22 | Hickahala Creek | 1 | 0 |
| 23 | Harland Creek | 1 | 0 |

## Table 24
## Average Annual Precipitation at Stations in the Long Creek Watershed Vicinity

| Station | Average Annual Precipitation in. | Period of Record |
|---|---|---|
| Batesville | 53.53 | 1949 - 1986 |
| Enid Dam | 51.79 | 1949 - 1986 |
| Water Valley | 54.25 | 1949 - 1986 |

## Table 25
## Drainage Areas for the Streams Studied in the Long Creek Watershed

| Stream Name | Drainage Area | |
|---|---|---|
| | Acres | Square Miles |
| Peters Creek | 55,074 | 86.05 |
| Bobo Bayou | 3,940 | 6.16 |
| Pope Tributary | 1,916 | 2.99 |
| Long Creek | 39,265 | 61.35 |
| Johnson Creek | 13,257 | 20.71 |
| Hurt Creek | 4,486 | 7.01 |
| Goodwin Creek | 5,489 | 8.58 |
| Goodwin Creek Tributary No. 2 | 436 | 0.68 |
| Goodwin Creek Tributary No. 3 | 878 | 1.37 |
| Goodwin Creek Tributary No. 4 | 997 | 1.56 |
| Goodwin Creek Tributary No. 4E | 279 | 0.44 |
| Caney Creek | 9,221 | 14.41 |

**Table 26**
**Listing of Results from the Flood Frequency Analyses for Long Creek Watershed**

| Stream Location | Return Period years | USGS cfs[1] | Vicksburg District cfs[2] | HEC-1 cfs[2] |
|---|---|---|---|---|
| Peters Creek at Yocona River Drainage Area = 86.05 square miles | 2 | 6,266 | 9,200 | 22,300 |
| | 5 | 10,975 | 13,200 | 29,000 |
| | 10 | 14,314 | 16,700 | 33,900 |
| | 25 | 20,137 | 23,500 | 39,400 |
| | 50 | 24,326 | 28,500 | 44,700 |
| | 100 | 31,759 | 35,000 | 49,900 |
| Bobo Bayou at Peters Creek Drainage Area = 6.16 square miles | 2 | 1,547 | 1,780 | 2,970 |
| | 5 | 2,554 | 2,500 | 3,820 |
| | 10 | 3,273 | 3,200 | 4,390 |
| | 25 | 4,163 | 4,500 | 5,060 |
| | 50 | 4,983 | 5,500 | 5,700 |
| | 100 | 5,554 | 6,800 | 6,440 |
| Pope Tributary at Peters Creek Drainage Area = 2.99 square miles | 2 | 823 | 1,100 | 1,670 |
| | 5 | 1,312 | 1,580 | 2,140 |
| | 10 | 1,659 | 2,000 | 2,460 |
| | 25 | 2,090 | 2,800 | 2,840 |
| | 50 | 2,484 | 3,400 | 3,170 |
| | 100 | 2,758 | 4,250 | 3,580 |
| Long Creek at Peters Creek Drainage Area = 61.35 square miles | 2 | 7,393 | 7,400 | 14,000 |
| | 5 | 13,334 | 10,500 | 18,100 |
| | 10 | 17,828 | 13,500 | 21,100 |
| | 25 | 23,657 | 19,000 | 24,400 |
| | 50 | 29,157 | 23,000 | 27,600 |
| | 100 | 32,935 | 28,500 | 31,000 |
| Johnson Creek at Long Creek Drainage Area = 20.72 square miles | 2 | 2,239 | 3,750 | 7,170 |
| | 5 | 3,761 | 5,300 | 9,270 |
| | 10 | 4,892 | 6,800 | 10,700 |
| | 25 | 6,410 | 9,500 | 12,400 |
| | 50 | 7,828 | 11,500 | 14,000 |
| | 100 | 8,776 | 14,300 | 16,000 |

(Sheet 1 of 3)

Note: Data from FTN Associates, Ltd. (1987).
[1]Colson and Hudson 1976.
[2]FTN Associates, Ltd., 1987.

## Table 26 (Continued)

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| Hurt Creek at Johnson Creek Drainage Area = 7.01 square miles | 2 | 1,150 | 1,880 | 3,260 |
| | 5 | 1,872 | 2,580 | 4,180 |
| | 10 | 2,393 | 3,400 | 4,840 |
| | 25 | 3,123 | 4,800 | 5,580 |
| | 50 | 3,743 | 5,800 | 6,250 |
| | 100 | 4,310 | 7,200 | 7,060 |
| Goodwin Creek at Long Creek Drainage Area = 8.58 square miles | 2 | 1,356 | 2,150 | 3,510 |
| | 5 | 2,224 | 3,050 | 4,500 |
| | 10 | 2,851 | 3,900 | 5,140 |
| | 25 | 3,740 | 5,500 | 5,910 |
| | 50 | 4,484 | 6,000 | 6,630 |
| | 100 | 5,200 | 8,200 | 7,560 |
| Goodwin Creek Tributary No. 2 at Goodwin Creek Drainage Area = 0.68 square mile | 2 | 244 | 430 | 558 |
| | 5 | 357 | 620 | 709 |
| | 10 | 431 | 790 | 808 |
| | 25 | 535 | 1,100 | 926 |
| | 50 | 617 | 1,350 | 1,030 |
| | 100 | 705 | 1,680 | 1,190 |
| Goodwin Creek Tributary No. 3 at Goodwin Creek Drainage Area = 1.37 square miles | 2 | 346 | 670 | 916 |
| | 5 | 523 | 960 | 1,160 |
| | 10 | 645 | 1,220 | 1,340 |
| | 25 | 822 | 1,720 | 1,540 |
| | 50 | 961 | 2,100 | 1,710 |
| | 100 | 1,114 | 2,600 | 1,950 |
| Goodwin Creek Tributary No. 4 at Goodwin Creek Drainage Area = 2.00 square miles | 2 | 457 | 740 | 1,320 |
| | 5 | 704 | 1,050 | 1,680 |
| | 10 | 877 | 1,320 | 1,930 |
| | 25 | 1,125 | 1,850 | 2,220 |
| | 50 | 1,323 | 2,250 | 2,460 |
| | 100 | 1,530 | 2,800 | 2,810 |

(Sheet 2 of 3)

**Table 26 (Concluded)**

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| Goodwin Creek Tributary No. 4E at Goodwin Creek Tributary No. 4 Drainage Area = 0.44 square mile | 2 | 218 | 320 | 373 |
| | 5 | 315 | 440 | 474 |
| | 10 | 379 | 580 | 538 |
| | 25 | 455 | 810 | 617 |
| | 50 | 527 | 980 | 687 |
| | 100 | 571 | 1,220 | 800 |
| Caney Creek at Long Creek Drainage Area = 14.40 square miles | 2 | 2,607 | 3,000 | 5,670 |
| | 5 | 4,457 | 4,200 | 7,290 |
| | 10 | 5,826 | 5,400 | 8,470 |
| | 25 | 7,537 | 7,600 | 9,790 |
| | 50 | 9,159 | 9,200 | 10,980 |
| | 100 | 10,202 | 11,500 | 12,410 |

(Sheet 3 of 3)

## Table 27
## Peak Discharge Estimates within the Hickahala-Senatobia Creek Watershed

| Stream Location | Return Period years | USGS cfs[1] | Vicksburg District cfs[2] | HEC-1 cfs[3] |
|---|---|---|---|---|
| Hickahala Creek Upstream of Cathey Creek Drainage Area = 19.24 square miles | 2 | 2,630 | 3,600 | 2,646 |
| | 5 | 4,484 | 5,100 | 4,073 |
| | 10 | 5,860 | 6,500 | 5,301 |
| | 25 | 7,685 | 9,100 | 6,725 |
| | 50 | 9,360 | 11,100 | 8,263 |
| | 100 | 10,577 | 13,800 | 9,988 |
| Hickahala Creek Downstream of Beards Creek and Cathey Creek Drainage Area = 36.82 square miles | 2 | 3,721 | 5,100 | 5,260 |
| | 5 | 6,420 | 7,200 | 7,835 |
| | 10 | 8,420 | 9,300 | 10,193 |
| | 25 | 11,231 | 13,000 | 12,734 |
| | 50 | 13,715 | 15,800 | 15,597 |
| | 100 | 15,803 | 19,400 | 18,789 |
| Hickahala Creek Downstream of Whites Creek Drainage Area = 50.00 square miles | 2 | 4,050 | 6,500 | 5,881 |
| | 5 | 6,964 | 9,300 | 8,460 |
| | 10 | 9,121 | 11,900 | 10,850 |
| | 25 | 12,156 | 16,500 | 13,564 |
| | 50 | 14,895 | 20,000 | 16,575 |
| | 100 | 17,015 | 25,000 | 19,928 |
| Hickahala Creek Downstream of Lick Creek Drainage Area = 98.47 square miles | 2 | 6,580 | 10,000 | 11,129 |
| | 5 | 11,545 | 14,500 | 15,311 |
| | 10 | 15,222 | 18,000 | 19,359 |
| | 25 | 20,400 | 25,800 | 23,849 |
| | 50 | 25,117 | 31,000 | 28,980 |
| | 100 | 28,679 | 38,500 | 34,687 |
| Hickahala Creek Downstream of Basket Creek and Thornton Creek Drainage Area = 119.49 square miles | 2 | 6,942 | 11,300 | 11,931 |
| | 5 | 12,130 | 16,000 | 16,131 |
| | 10 | 15,942 | 20,600 | 20,266 |
| | 25 | 21,358 | 28,800 | 24,879 |
| | 50 | 26,286 | 35,000 | 30,165 |
| | 100 | 29,972 | 43,000 | 36,057 |

(Sheet 1 of 5)

Note: Data from Simons, Li and Associates (1987).
[1]Colson and Hudson 1976.
[2]FTN Associates, Ltd., 1987.

## Table 27 (Continued)

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| Hickahala Creek Upstream of Senatobia Creek Drainage Area = 125.97 square miles | 2 | 6,672 | 11,500 | 11,996 |
| | 5 | 11,573 | 16,600 | 16,208 |
| | 10 | 15,142 | 21,100 | 20,330 |
| | 25 | 20,274 | 29,800 | 24,951 |
| | 50 | 24,909 | 36,000 | 30,243 |
| | 100 | 28,418 | 44,500 | 36,133 |
| Hickahala Creek at Coldwater River Drainage Area = 229.51 square miles | 2 | 9,115 | 17,000 | 28,310 |
| | 5 | 15,844 | 24,000 | 36,069 |
| | 10 | 20,650 | 31,000 | 43,774 |
| | 25 | 27,741 | 43,000 | 51,992 |
| | 50 | 34,031 | 52,500 | 60,637 |
| | 100 | 39,989 | 64,000 | 70,405 |
| Cathey Creek at Hickahala Creek Drainage Area = 4.03 square miles | 2 | 700 | 1,300 | 933 |
| | 5 | 1,108 | 1,900 | 1,352 |
| | 10 | 1,397 | 2,400 | 1,741 |
| | 25 | 1,820 | 3,400 | 2,141 |
| | 50 | 2,163 | 4,050 | 2,582 |
| | 100 | 2,515 | 5,000 | 3,068 |
| Beards Creek at Hickahala Creek Drainage Area = 10.94 square miles | 2 | 1,337 | 2,500 | 1,963 |
| | 5 | 2,176 | 3,500 | 2,935 |
| | 10 | 2,775 | 4,000 | 3,815 |
| | 25 | 3,723 | 6,400 | 4,745 |
| | 50 | 4,440 | 7,700 | 5,783 |
| | 100 | 5,366 | 9,300 | 6,934 |
| Whites Creek at Hickahala Creek Drainage Area = 5.38 square miles | 2 | 856 | 1,600 | 1,120 |
| | 5 | 1,362 | 2,250 | 1,750 |
| | 10 | 1,719 | 2,800 | 2,246 |
| | 25 | 2,270 | 4,000 | 2,792 |
| | 50 | 2,688 | 4,800 | 3,398 |
| | 100 | 3,210 | 6,000 | 4,070 |
| James Wolf Creek Downstream of Martin Dale Creek; Drainage Area = 25.40 square miles | 2 | 2,779 | 4,250 | 3,638 |
| | 5 | 4,728 | 6,000 | 5,153 |

## Table 27 (Continued)

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| James Wolf Creek (Cont) | 10 | 6,178 | 7,700 | 6,537 |
| | 25 | 8,137 | 10,800 | 8,049 |
| | 50 | 9,944 | 13,200 | 9,728 |
| | 100 | 11,229 | 16,200 | 11,589 |
| James Wolf Creek at Hickahala Creek Drainage Area = 38.90 square miles | 2 | 3,005 | 5,800 | 4,877 |
| | 5 | 5,069 | 8,000 | 6,526 |
| | 10 | 6,577 | 10,500 | 8,178 |
| | 25 | 8,808 | 14,500 | 9,968 |
| | 50 | 10,724 | 17,500 | 12,055 |
| | 100 | 12,458 | 21,500 | 14,385 |
| Martin Dale Creek at James Wolf Creek Drainage Area = 4.94 square miles | 2 | 846 | 1,550 | 1,256 |
| | 5 | 1,351 | 2,200 | 1,721 |
| | 10 | 1,713 | 2,750 | 2,147 |
| | 25 | 2,233 | 3,800 | 2,584 |
| | 50 | 2,661 | 4,700 | 3,069 |
| | 100 | 3,085 | 5,400 | 3,535 |
| Lick Creek at Hickahala Creek Drainage Area = 3.11 square miles | 2 | 594 | 1,100 | 912 |
| | 5 | 925 | 1,600 | 1,440 |
| | 10 | 1,155 | 2,000 | 1,847 |
| | 25 | 1,509 | 2,750 | 2,301 |
| | 50 | 1,773 | 3,400 | 2,735 |
| | 100 | 2,112 | 4,300 | 3,210 |
| Basket Creek at Hickahala Creek Drainage Area = 9.71 square miles | 2 | 1,182 | 2,350 | 1,503 |
| | 5 | 1,914 | 3,300 | 2,159 |
| | 10 | 2,439 | 4,300 | 2,763 |
| | 25 | 3,250 | 5,900 | 3,432 |
| | 50 | 3,887 | 7,200 | 4,185 |
| | 100 | 4,625 | 9,000 | 5,020 |
| Thornton Creek at Hickahala Creek Drainage Area = 4.65 square miles | 2 | 964 | 1,400 | 1,200 |
| | 5 | 1,547 | 1,950 | 1,783 |
| | 10 | 1,957 | 2,500 | 2,268 |
| | 25 | 2,535 | 3,500 | 2,812 |

(Sheet 3 of 5)

## Table 27 (Continued)

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| Thornton Creek (Cont) | 50 | 3,006 | 4,300 | 3,393 |
| | 100 | 3,487 | 5,200 | 4,034 |
| Steammill Branch at Thornton Creek Drainage Area = 2.39 square miles | 2 | 549 | 960 | 692 |
| | 5 | 852 | 1,350 | 1,028 |
| | 10 | 1,062 | 1,750 | 1,341 |
| | 25 | 1,365 | 2,400 | 1,675 |
| | 50 | 1,606 | 2,900 | 2,024 |
| | 100 | 1,865 | 3,700 | 2,409 |
| Billys Creek at Hickahala Creek Drainage Area = 2.93 square miles | 2 | 594 | 1,051 | 759 |
| | 5 | 928 | 1,550 | 1,198 |
| | 10 | 1,163 | 1,950 | 1,591 |
| | 25 | 1,505 | 2,650 | 1,992 |
| | 50 | 1,777 | 3,300 | 2,394 |
| | 100 | 2,072 | 4,200 | 2,836 |
| Senatobia Creek Downstream of Mattic Creek Drainage Area = 55.14 square miles | 2 | 4,622 | 7,900 | 16,961 |
| | 5 | 8,023 | 9,900 | 19,264 |
| | 10 | 10,545 | 12,700 | 22,750 |
| | 25 | 14,113 | 17,700 | 26,202 |
| | 50 | 17,297 | 21,500 | 27,894 |
| | 100 | 19,892 | 26,500 | 31,255 |
| Senatobia Creek at Hickahala Creek Drainage Area = 64.68 square miles | 2 | 3,770 | 7,650 | 16,706 |
| | 5 | 6,365 | 10,900 | 19,305 |
| | 10 | 8,231 | 13,900 | 22,755 |
| | 25 | 11,170 | 19,800 | 26,274 |
| | 50 | 13,567 | 23,800 | 27,839 |
| | 100 | 16,100 | 29,000 | 31,159 |
| Tolbert Jones Creek at Senatobia Creek Drainage Area = 4.64 square miles | 2 | 757 | 1,500 | 1,797 |
| | 5 | 1,200 | 2,100 | 2,179 |
| | 10 | 1,514 | 2,700 | 2,572 |
| | 25 | 1,987 | 3,700 | 3,044 |
| | 50 | 2,358 | 4,600 | 3,476 |
| | 100 | 2,779 | 5,300 | 4,012 |

## Table 27 (Concluded)

| Stream Location | Return Period years | USGS cfs | Vicksburg District cfs | HEC-1 cfs |
|---|---|---|---|---|
| Mattic Creek at Senatobia Creek Drainage Area = 27.88 square miles | 2 | 2,798 | 4,100 | 9,422 |
| | 5 | 4,741 | 5,900 | 11,148 |
| | 10 | 6,159 | 7,500 | 13,171 |
| | 25 | 8,288 | 10,500 | 15,103 |
| | 50 | 10,033 | 13,000 | 16,121 |
| | 100 | 11,875 | 15,500 | 18,080 |
| Nelson Creek at Mattic Creek Drainage Area = 14.23 square miles | 2 | 1,486 | 2,800 | 4,808 |
| | 5 | 2,433 | 4,200 | 5,679 |
| | 10 | 3,123 | 5,200 | 6,679 |
| | 25 | 4,136 | 7,600 | 7,860 |
| | 50 | 4,997 | 9,200 | 8,132 |
| | 100 | 5,786 | 11,500 | 9,158 |
| Gravel Springs Creek at Senatobia Creek Drainage Area = 2.84 square miles | 2 | 511 | 1,100 | 1,224 |
| | 5 | 790 | 1,550 | 1,523 |
| | 10 | 982 | 2,000 | 1,635 |
| | 25 | 1,288 | 2,750 | 1,899 |
| | 50 | 1,512 | 3,600 | 2,060 |
| | 100 | 1,815 | 4,200 | 2,637 |
| West Ditch at Hickahala Creek Drainage Area = 16.84 square miles | 2 | 1,515 | 3,250 | 5,083 |
| | 5 | 2,469 | 4,600 | 5,824 |
| | 10 | 3,150 | 6,950 | 6,801 |
| | 25 | 4,267 | 8,250 | 7,872 |
| | 50 | 5,111 | 10,000 | 8,412 |
| | 100 | 6,203 | 12,000 | 9,518 |

(Sheet 5 of 5)

## Table 28
## 1993 Drop Structure Evaluation Summary

| Label | Stream | Category[1] | Problem Type[2] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | a | b | c | d | e | f |
| HD-1 | Hotophia | 2 | | | c | | | |
| LD-2 | Beard's | 3 | | | | | | |
| LD-1 | Black Creek | 3 | | | | | | |
| LD-1 | Caffe Branch | 2 | a | | | d | | f |
| LD-1 | Campbell | 3 | | | | | | |
| LD-1 | Caney | 2 | a | b | | d | | |
| LD-2 | Caney | 2 | a | b | | | | |
| LD-3 | Caney | 2 | | b | | | | |
| LD-1 | Crowder | 2 | | | | d | e | f |
| LD-2 | Crowder Creek | 2 | | | c | | | |
| LD-1 | Deer | 1 | a | | c | d | e | |
| LD-2 | Deer | 2 | | b | c | | | f |
| LD-1 | East Fork Worsham | 2 | a | b | c | d | | f |
| LD-1 | Eskridge | 2 | | | | | | f |
| LD-2 | Eskridge | 2 | a | | | | | |
| LD-1 | Hickahala | 2 | a | b | | | | |
| LD-2 | Hickahala | 3 | | | | | | |
| LD-3 | Hickahala | 2 | a | b | | | | |
| LD-4 | Hickahala | 2 | a | | | | | |
| LD-6 | Hickahala | 3 | | | | | | |
| LD-7 | Hickahala | 3 | | | | | | |
| LD7-7 | Hotophia | 2 | a | b | | d | e | f |
| LD7-8 | Hotophia | 2 | a | b | | d | e | |
| LD-1 | James Wolf | 1 | | | | d | e | |
| LD-9B-1 | Johnson Creek | 2 | a | | | d | | |
| LD-1 | Little Bogue | 1 | a | b | c | d | e | |
| LD-1 | Little Mouse | 3 | | | | | | |
| LD-2 | Little Mouse | 3 | | | | | | |
| LD-1 | Long | 2 | a | b | c | d | | |
| LD-2 | Long | 2 | a | | | | | f |

(Continued)

[1]Defined on page 139.
[2]Defined beginning on page 136.

## Table 28 (Concluded)

| Label | Stream | Category | Problem Type a | b | c | d | e | f |
|---|---|---|---|---|---|---|---|---|
| LD-3 | Long | 2 | | b | | | | f |
| LD-4 | Long | 3 | | | | | | |
| LD-5 | Long | 3 | | | | | | |
| LD-1 | Middle Fork Worsham | 2 | a | b | c | d | e | f |
| LD-2 | Middle Fork Worsham | 2 | a | b | c | | | |
| LD-3 | Middle Fork Worsham | 2 | | | c | | | |
| LD-1 | Marcum | 2 | a | b | | d | e | |
| LD-1 | Martin Dale | 3 | | | | | | |
| LD-1 | Mill | 2 | a | | | | | f |
| LD-2 | Mill | 3 | | | | | | |
| LD-3 | Perry Creek | 3 | | | | | | |
| LD-4 | Perry Creek | 3 | | | | | | |
| LD-1 | South Fork Hickahala | 2 | | | c | | | |
| LD-2 | South Fork Hickahala | 3 | | | | | | |
| LD-3 | South Fork Hickahala | 3 | | | | | | |
| LD-1 | Senatobia | 3 | | | | | | |
| LD-1 | Tarrey Creek | 3 | | | | | | |
| LD-2 | Tarrey Creek | 3 | | | | | | |
| LD-1 | West Fork Worsham | 2 | a | | | d | | |
| LD-3 | West Fork Worsham | 2 | a | b | c | | | |
| LD-4 | West Fork Worsham | 1 | | | c | | | |
| LD-1 | White's | 2 | | b | | | | |
| LD-1 | White's | 2 | | b | | | | |
| LD-1 | Worsham | 2 | a | | c | d | e | |
| LD-2 | Worsham | 2 | | b | | | | |

## Table 29
## Aerial Videotapes of DEC Watersheds USDA-ARS-NSL Flights Spring 1992

| Main Stem (Fourth-Order Tributary) | Third-Order Tributary | Second-Order Tributary | First-Order Tributary |
|---|---|---|---|
| Hotophia Creek (Tributary to Little Tallahatchie River) | Harris Creek<br>Mill Creek<br>Deer Creek<br>Marcum Creek | | |
| Long Creek (Tributary to Yocona River) | Bobo Bayou<br>Johnson Creek<br>Hurt Creek<br>Goodwin Creek<br>Caney Creek | | |
| Toby Tubby Creek (Tributary to Little Tallahatchie River) | East Goose Creek<br>West Goose Creek | | |
| Burney Branch (Tributary to Yocona River) | Burney West #1<br>Burney West #2 | | |
| Coldwater (Tributary to Tallahatchie River) | Hickahala Creek | Hickahala N. Fork<br>Hickahala S. Fork<br>Cathey Creek<br>James Wolf Creek<br>Senatobia Creek | |
| | Hurricane Creek | Wolf Creek<br>Panther Creek | |
| | Mussacuna Creek | | |
| | Cane Creek | Secret Creek | |
| | Beartail Creek<br>Grays Creek<br>Camp Creek<br>Pigeon Creek | Cuffawa Creek<br>Byhalia Creek<br>Red Banks Creek | |
| Otoucalofa Creek (Tributary to Yocona River) | Susie Perry Creek<br>Johnson Creek<br>Town Creek<br>Greasy Creek<br>Moore Creek<br>Gordon Creek<br>Otoucalofa S.#1 | Spring Creek | |
| | Mill Creek<br>Smith South<br>Sarter Creek<br>Hanna Creek<br>Smith Creek<br>Shippy Creek | | |

(Continued)

**Table 29 (Concluded)**

| Main Stream (Fourth-Order Tributary) | Third-Order Tributary | Second-Order Tributary | First-Order Tributary |
|---|---|---|---|
| Batupan Bogue (Tributary to Yalobusha River) | Big Bogue Creek | Eskridge Creek<br>Jackson Creek<br>Wilkens Creek<br>Sykes Creek | |
| | Jack Creek<br>Perry Creek<br>Little Bogue Creek | Caffe Branch<br>Crowder Creek<br>Epison Creek<br>Campbell Creek<br>Powell Creek<br>Mouse Creek | |
| Black Creek (Tributary to Yazoo River) | Harland Creek | Moccasin Creek<br>Williams Creek | Butterworth Creek |
| | Fannegusha Creek | Bophumpa Creek<br>Tchula Lake | Millstone Bayou<br>Spring Branch<br>Chicopa Creek |
| | Abiaca Creek | Coila Creek | |
| Palucia Creek (Tributary to Yazoo River) | Ashly Creek | | |

## Table 30
## Log of Aerial Videotape 1

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2-10 | Black Creek | 0063 | 0:00:44 | Black Cr.-Sec.30-R.R. Bridge (Parker Bayou) T1N-R1W | | Can't Tell | Mostly stable some local erosion | Moderately wide woody veg buffer zone | Cultivated with a little woodland | Can't Tell Looks O.K. | Sinuous stream with some straight reaches seems mostly stable |
| | | 0160 | 0:01:48 | Bridge-Hwy 40E.-Sec.29 (Mouth Tipton Bayou) | | Some side bars deposits | Appears to be moderate erosion -local bank failures | Narrow woody veg buffer zone | " | Can't Tell | Straighter narrower reach |
| | | 0322 | 0:03:39 | Bear Lake entrance-Right-Sec.23W (1000 ft. South of Howard) | | Alluvial fan type deposits visible from tributary | Bank erosion visible around fan otherwise mostly stable | Woodland & shrubs | Mostly woodland some cultivated | N/A | Early straight reach appears less stable around tributary |
| | | 1002 | 0:13:43 | Harland Cr.-Left | 24,300 | Low sinuosity some side bar deposits | Mostly stable | Some wood shrubs | Cultivated | N/A | No apparent problem |
| | | 1567 | 0:20:58 | Owens Branch-Sec.34-Right | 74,100 | Sand-bed w/ dunes meandering | Evidence of some failures, especially on outer bank | Mixed cultivated & wood | Cultivated with wood | Some riprap failure? | Meandering reach-some bank erosion problems |
| | | 1626 | 0:21:56 | Bridge-Hwy 7-Lexington | 85,000 | Moderate sinuous Can't Tell | Mostly stable | Narrow woody buffer | Suburban | No Problem | No apparent problem |
| | | 1695 | 0:23:04 | Tarrey Cr.-Left | 95,000 | Sand bed deposits Pt. Bars | Outer bank erosion | Mixed wood & cropland | Cultivated with wood | N/A | Localized bendway erosion & sedimentation |
| | | 1719 | 0:23:28 | Bridge-Hwy 12 | 95,900 | Sand-Bed | Outer bank erosion scars | Crops & woody | Suburban, cultivated woodland | OK | Nearby bank erosion |
| | | 1854 | 0:25:46 | Bridge-Grvl.Rd. Sec.21 | 115,600 | Sand-bed. Some side bars | Mostly stable local erosion | Good woody buffer zone | Cultivated | OK | Minor deposition near bridge |
| | | 1919 | 0:26:53 | Bridge-Drt.Rd. | 129,100 | Sand-bed w/dunes some side deposits | Unstable banks | Narrow woody buffer | Mostly cultivated | Bank erosion at bridge | Threat from bank erosion? |
| | | 1962 | 0:27:38 | Bridge-Pvd.Rd.Sec.11 | 141,500 | Sand bed meandering river w/ point bars | Bank eroding some deep-seated failures | Woody | Cultivated & wood | Presently OK but may be threatened | Channel actively migrating |
| 2-24 | Black Creek | 2102 | 0:30:07 | Bridge-Pvd.Rd.Sec.11 | 141,500 | Sand-bed reach with tortuous meanders active point bar deposition | Active outer bank erosion | Woody | Mainly cultivated | Seem OK | Structures are ok but may be threatened by channel migration |
| | | 2146 | 0:30:56 | Structure | | | | | | | |
| | | 2175 | 0:31:27 | Bridge-Pvd.Rd. | 159,200 | | | | | | |
| | | 2306 | 0:33:54 | End Upper end of Black Cr. | | | | | | | |

## Table 30 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2-24 | Harland Creek | 2325 | 0:34:15 | Mouth Harland Cr. | 0 | Sand-bed with dunes actively meandering dissected bars | Active bank migration & erosion | Woody buffer strip | Cultivated & woody | Structures OK | Clear evidence of deposition + channel filling. Possible meandering/braided transition? |
| | | 2344 | 0:34:36 | Bridge-Drt.Rd. | 7,800 | | | | | | |
| | | 2374 | 0:35:10 | Williams Cr.-Right | 18,000 | | | | | | |
| | | 2398 | 0:35:38 | Bridge-Grvl.Rd.Sec.11 | 25,400 | | | | | | |
| | | 2423 | 0:36:07 | Moccasin Creek | 35,200 | | | | | | |
| | | 2462 | 0:36:52 | Bridge-Grvl.Rd.Sec.14&15 | 38,800 | As Above | | | | Bridge in poor condition | " |
| | | 2497 | 0:37:33 | Bridge-Grvl.Rd.Sec.27 (Tolarville/Eulogy Rd.) | 53,100 | As Above | | | | | |
| | | 2574 | 0:31:27 | South Fk.Harland-Left Bridge-on South Fk Harland | 70,000 | As Above | | | | OK | Structures all seem ok but may be under threat sediment choked reach |
| | | 2595 | 0:39:28 | Bridge-Pvd.Rd.Sec.4 on South Fk. Harland | | | | | | | |
| | | 2777 | 0:43:09 | Bridge-Grvl.Rd.Sec.11 | | Can't Tell - Flickering image | | | | | |
| | | 2777 | 0:43:09 | End of South Fork Harland Cr. | | | | | | | |
| 2-24 | South Fork Harland Creek | 2789 | 0:36:46 | South Fk. Harland joining Harland Cr. | 70,000 | Sand-bed meandering with pt. bars | Outer bank erosion Channel actively migrating | Narrow woody buffer | Mainly cultivated | OK | Active tending to unstable |
| | | 2804 | 0:43:42 | Bridge-Grvl.Rd.Sec.4 | 74,900 | | | | | OK | |
| | | 2826 | 0:44:08 | Bridge-Grvl.Rd.Sec.3 | 80,000 | | | | | | |
| | | 2861 | 0:44:52 | Bridge-Pvd.Rd.Sec.2 | | | | | | | |
| | | 2889 | 0:45:28 | End of N.Fork & Harland Bridge on Hwy.433 Sec.1 | | As Above | | | | OK | |
| | | 2889 | 0:45:28 | End Harland Cr. | | | | | | | |
| 2-24 | Williams Creek | 2905 | 0:45:47 | Mouth Williams Cr. on Harland Cr. | | Sand-bed w/dunes + point bars | Steep banks Outer bank erosion | Woody | Mostly woody | Mostly OK Some bank erosion threat | |
| | | 2982 | 0:47:25 | Bridge-Pvd.Rd.Sec.7 | | | | | | | |
| | | 3003 | 0:47:52 | Pipeline Sec.8 | | | | | | | |
| | | 3048 | 0:48:50 | End of Williams Cr. Pvd.Rd. & Bridge Sec.3 | | | | | | | |
| 2-24 | Moccasin Creek | 3091 | 0:49:45 | Mouth of Moccasin on Harland Creek | 0<br>35,000 | Sand-bed. Extensive bar deposits Meandering | Steep banks tending to unstable | Mainly woody or crops | Cultivated & woodland | OK at present | Unstable meandering reach |
| | | 3110 | 0:50:10 | Bridge-Grvl.Rd.Sec.13 | | | | | | | |
| | | 3229 | 0:52:46 | End of Moccasin | | | | | | | |
| 2-24 | Fannegusha Creek | 3279 | 0:53:53 | Beginning of channel-Sec.32 | | Bed-obscured Straight reach Low sinuosity | Localized failures mostly stable | Narrow woody buffer | Mainly cultivated | | Seems OK no app problem |
| | | 3335 | 0:55:08 | Old R.R. Grade | | | | | | | |
| | | 3348 | 0:55:26 | Bridge-Pvd.Rd.Sec.28 | | | | | | | |

## Table 30 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2-24 | Fannegusha Creek (Continued) | 3445 | 0:57:38 | Bridge-Hwy 12-Sec.14 | | Sand-major bed bar deposits | Extensive outer bank erosion | Woody buffer zone | Mainly cultivated | Looks OK | Unstable actively migrating |
| | | 3529 | 0:59:35 | Bridge-Drt.Rd.Sec.6 | | Sand-bed Extensive bar deposits | Tending to unstable banks | Narrow woody buffer | Cultivated & wood | Looks OK | Actively deposition + migration |
| | | 3578 | 1:00:42 | Ford-Sec.32 | | | | | | | |
| | | 3593 | 1:01:04 | White Cr.-Sec.33-Right | | Sand-bed Meandering Extensive pt bar deposits | Outer bank erosion | Wide woody buffer | Wood & cultivated | N/A | Actively migrating tending to unstable |
| | | 3655 | 1:02:31 | Bophumpa Cr.-Left | | | | | | | |
| | | 3703 | 1:03:39 | Bridge-Hwy 17-Sec.25 | | Sand-bed Low sinuosity | Incipient outer bank erosion | Medium woody buffer | Mainly cultivated | Look ok at present | Tending to unstable |
| | | 3758 | 1:04:58 | Little Fannegusha-Left | | | | | | | |
| | | 3791 | 1:05:47 | Bridge-Grvl.Rd.Sec.8 | | Sand-bed w/point bars Meandering | Active outer bank erosion | Narrow woody buffer | Cultivated | Looks OK | Tending to unstable |
| | | 3891 | 1:08:12 | Bridge-Grvl.Rd.Sec.1 | | Narrow, incised reach | Can't Tell | As Above | Cultivated & woody | Can't Tell | |
| | | 3919 | 1:08:54 | Bridge-Grvl.Rd.Sec.31,36,1&6 | | Straight reach Sand-bed | Steep, but mostly stable | Narrow band of shrubs | Cultivated, some wood | Some failed rip-rap | |
| | | 3919 | 1:08:54 | End Fannegusha Cr. | | | | | | | |
| 2-24 | Bophumpa Creek | 4062 | 1:12:27 | Mouth Bophumpa Cr. on Fannegusha Cr. | | Moderately sinuous reach | Mostly stable | Woody | Woods, some cultivated | OK as far as can tell | |
| | | 4093 | 1:13:13 | Bridge-Hwy 17-Sec.36 | | Flight too high to tell much detail | | | | | |
| | | 4116 | 1:13:49 | Bridge-Grvl.Rd.Sec.31 | | | | | | | |
| | | 4194 | 1:15:48 | End Bophumpa Cr. | | | | | | | |
| 2-24 | Millstone Bayou | 4241 | 1:17:01 | Mouth Millstone Bayou-Sec.29&30 on Tchula Lake | | Low sinuosity Bed details obscured by flow | Mostly stable - local erosion | Woody | Woods | OK | No apparent problem |
| | | 4248 | 1:17:11 | Bridge-R.R.&Hwy 49-Sec.29 | | | | | | | |
| | | 4306 | 1:18:41 | End Millstone Bayou | | | | | | | |
| 2-24 | Spring Branch | 4306 | 1:18:41 | Mouth Spring Branch-Sec.15 on Millstone Bayou | | Too high & too woody to tell any detail. Structures look OK | | | | | |
| | | 4353 | 1:19:18 | Bridge-Grvl.Rd.Sec.7 | | | | | | | |
| | | 4353 | 1:19:18 | End Spring Branch | | | | | | | |
| 2-24 | Chicopa Creek | 4419 | 1:21:38 | Beginning of Chicopa Cr. | | Sand-bed with point bars Meandering | Mostly stable, some outer bank erosion | Woody | Woodland | OK | No major apparent problem |
| | | 4452 | 1:22:31 | Gravel Rd. Sec.7 | | | | | | | |
| | | 4470 | 1:23:00 | End Chicopa Creek | | | | | | | |
| 2-24 | Abiaca Creek | 4518 | 1:24:17 | Beginning of Abiaca Cr. | | Sand-bed w/dunes Sinuous | Mostly steep. Some failure | Shrubs, trees, grass | Cultivated | OK | No apparent problem |
| | | 4541 | 1:24:52 | Bridge-R.R. | | As Above | | | | | |
| | | 4568 | 1:25:37 | Bridge-Hwy 49-Sec.18 | | | | | | | |
| | | 4670 | 1:28:22 | Bridge-Grvl.Rd.Sec.13 | | Sand bed w/dunes and points bars Sinuous | Steep banks tending to unstable | Wide woody buffer | Mainly cultivated | Bank erosion at site | Lateral migration problem at bridge? |
| | | 4670 | 1:28:22 | End of Site #6 | | | | | | | |
| | | 4940 | 1:35:53 | End of Abiaca Creek | | | | | | | |

# Table 30 (Concluded)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2-24 | Coila Creek | 4773 | 1:31:12 | Right mouth of Coila | | Sand bed. Major depositional reach | Outer banks migration | Some wood & shrubs | Cultivated | N/A | Severe sedimentation |
| | | 4803 | 1:32:03 | Site #4 | | | | | | | |
| | | 4947 | 1:36:05 | Mouth of Coila | | | | | | | |
| | | 4953 | 1:36:16 | Bridge-Pvd.Rd.Sec.4 Site #5 | | Sand-bed with bar deposition | Eroding banks | Woody | Woody & cultivated | Can't Tell | Tending to unstable |
| | | 5018 | 1:38:07 | Bridge-Pvd.Rd.Sec.36 (Matthews Cem.) | | Sand-bed Active deposition | Can't Tell | Woody | Mainly woody | Can't Tell | Tending to unstable |
| | | 5023 | 1:38:15 | End of Coila Creek went on to Dry Creek | | | | | | | |
| 2-24 | Pelucia Creek | 5101 | 1:40:29 | Mouth Pelucia Cr. on Yazoo River | | Too high to tell | Appears mostly stable | Wood buffer zone | Industrial | Looks OK | |
| | | 5105 | 1:40:37 | Bridge-R.R.-Sec.32 | | Low sinuosity reach | | | | | |
| | | 5117 | 1:40:57 | Bridge-Hwy 49-Sec.32 | | As Above | | | | | |
| | | 5155 | 1:42:03 | Bridge-Pvd.Rd.Sec.34 (under construction) | | Can't Tell | | | | | |
| | | 5203 | 1:43:26 | Bridge-Pvd.Rd.Sec.31 | | Straight reach Sand | Mostly stable | Grass | | OK | No app problem |
| | | 5309 | 1:46:35 | Bridge-Pvd.Rd.Sec.23 (Airport Rd.) | | Sand-bed w/dunes Straight | Steep banks, some erosion | Wood & shrubs | Lagoons, woods | OK | " |
| | | 5362 | 1:48:08 | Bridge-Grvl.Rd.Sec.30 (Ferguson gravel pit) | | Can't Tell | | | | | |
| | | 5438 | 1:50:26 | Bridge-Drt.Rd.Sec.29&32 | | Can't Tell | | | | | |
| | | 5505 | 1:52:27 | Bridge-Hwy 17-Sec.35 | | Bed obscured Straight reach | Mostly stable | Narrow woody buffer Can't Tell | Cultivated | OK | |
| | | 5613 | 1:55:45 | Bridge Sec. 18 | | | | | | | |
| | | 5613 | 1:55:45 | End Pelucia Cr. | | | | | | | |

# Table 31
## Log of Aerial Videotape 2

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Long Creek | 0100 | 0:01:01 | Mouth Long Cr. on Yocona River | 0 | Sand-bed w/dunes + pt. bars | Steep eroding banks | Narrow woody buffer | Mainly cultivated some trees | Looks OK | Tending to infill? |
| | | 0233 | 0:02:28 | Bridge-Pvd.Rd.Sec.18 | 16,000 | | | | | | |
| | | 0251 | 0:02:40 | Bobo Bayou-Right | 18,000 | | | | | | |
| | | 0334 | 0:03:36 | Bridge-R.R. | 28,200 | Sand-bed Straight | Steep unstable banks | Narrow woody buffer | Mainly cultivated | Looks OK | Bank erosion may be problem |
| | | 0349 | 0:03:47 | Bridge-Hwy 51-Sec.9 | 30,100 | | | | | | |
| | | 0384 | 0:04:11 | Bridge-I 55 | 34,500 | Sand-bed w/extensive bar deposits | Steep banks some mass failures | Shrubs | Mainly cultivated | Structures look OK at present-bank erosion | Bank erosion a problem but sediment infill suggests regaining stability |
| | | 0390 | 0:04:15 | Johnson Cr.-Left | 35,000 | | | | | | |
| | | 0406 | 0:04:26 | Goodwin Cr.-Left | 37,400 | | | | | | |
| | | 0420 | 0:04:36 | Bridge-Pvd.Rd.Sec.11 | 39,800 | | | | | | |
| | | 0495 | 0:05:29 | Caney Cr.-Left | 49,000 | Sand-bed w/point bars. Less extensive deposition | Steep banks Unstable | Shrubs & trees | Mainly cultivated | Can't Tell | |
| | | 0512 | 0:05:42 | Bridge-Grvl.Rd.Sec.12 | 51,800 | | | | | | |
| | | 0568 | 0:06:23 | Bridge-Pvd.Rd.Sec 7&8 | 58,900 | As Above | As Above | As Above | | | |
| | | 0644 | 0:07:20 | New Structure Sec. 8 | 61,000 | Sand-bed w/well developed bars Sinuous | Steep banks Eroding unstable | Shrubs, Grass | Mainly cultivated | Good | Unstable banks |
| | | 0663 | 0:07:34 | New Structure Sec. 8 | 63,700 | | | | | | |
| | | 0680 | 0:07:47 | Structure | 66,000 | | | | | | |
| | | 0681 | 0:07:48 | Bridge-Pvd.Rd.Sec.4&5 | 66,100 | | | | | | |
| | | 0722 | 0:08:19 | Structure | 68,000 | Sand-bed Sinuous Point bars | Steep banks some failures | Shrubs & trees | Cultivated & wood | Structures OK | No apparent problem |
| | | 0745 | 0:08:37 | Structure | 71,500 | | | | | | |
| | | 0753 | 0:08:43 | Bridge-Grvl.Rd.Sec.3&4 | 72,500 | | | | | | |
| | | 0901 | 0:10:41 | Bridge-Grvl.Rd.Sec.25,26 33&36 | 88,500 | | | | | | |
| | | 0901 | 0:10:41 | End Long Cr. | 88,500 | | | | | | |
| 3-11 | Bobo Bayou | 1128 | 0:13:49 | Mouth Bobo Bayou Cr. on Long Cr. | 0 (18,000) | Sand-bed some pt. bars | Incised Steep unstable banks | Narrow woody buffer | Mainly cultivated | Can't Tell | Tending to unstable |
| | | 1160 | 0:14:17 | Bridge-Grvl.Rd.Sec.7 | 5,100 | | | | | | |
| | | 1212 | 0:15:02 | Bridge-Grvl.Rd.Sec.6 & 31 | 14,000 | | | | | | |
| | | 1248 | 0:15:34 | Bridge-Grvl.Rd.Sec.31 & 32 | 19,400 | As Above | | | | | |
| | | 1248 | 0:15:34 | End Bobo Bayou | 19,400 | | | | | | |
| 3-11 | Johnson Creek | 1274 | 0:15:57 | Mouth Johnson Cr. on Long Cr. | 0 (35,000) | Sand-bed Moderately sinuous Well developed point bars | Steep banks some erosion | Narrow woody buffer | Mainly cultivated | Looks OK | Potentially unstable banks, looks OK |
| | | 1300 | 0:16:20 | Bridge-Grvl.Rd.Sec.3 | 4,100 | | | | | | |
| | | 1309 | 0:16:28 | Hurt Cr.-Right | 6,100 | | | | | | |

# Table 31 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Johnson Creek | 1348 | 0:17:03 | Bridge-Pvd.Rd.Sec.35 | 13,200 | Sand-bed | Steep banks | Shrubs | Cultivated, some | OK | Sedimentation is severe |
| | (Continued) | | | | | Extensive bar | | | wood | | |
| | | | | | | deposits | | | | | |
| | | 1421 | 0:18:10 | Bridge-Grvl.Rd.Sec.25&30 | 25,000 | Sand-bed w/dunes | Steep, unstable | Shrubs & | Cultivated | Structures | |
| | | 1447 | 0:18:34 | Structure-Sec.19 | | & pt. bars | banks | grass | | Look OK | |
| | | 1460 | 0:18:46 | Bridge-Pvd.Rd.Sec.19&20 | 32,400 | Moderate | | | | | |
| | | 1467 | 0:18:52 | Structure-Sec.20 | | sinuosity | | | | | |
| | | 1477 | 0:18:46 | Structure-Sec.20 | 36,100 | As Above | | | | | |
| | | 1509 | 0:19:32 | Bridge-Grvl.Rd.Sec.21&28 | 43,000 | | | Can't Tell - Creek is too small | | | |
| | | 1584 | 0:20:43 | Bridge-Grvl.Rd.Sec.23 | 54,300 | | | | | | |
| | | 1617 | 0:21:15 | Hwy 315 | | | | | | | |
| | | 1617 | 0:21:15 | End Johnson Cr. | | | | | | | |
| 3-11 | Hurt Creek | 1667 | 0:22:03 | Mouth Hurt Cr. | 0 | Sand-bed | Steep banks | Shrubs | Mainly cultivated | Bank erosion | |
| | | | | on Johnson Cr. | (6,100) | Extensive bar | tending to | | | at bridge | |
| | | 1713 | 0:22:48 | Bridge-Pvd.Rd.Sec.27&34 | 7,900 | deposits | unstable | | | | |
| | | 1740 | 0:23:15 | Bridge-Pvd.Rd.Sec.27 | 12,500 | | | | | | |
| | | | | (Eureka Rd.) | | | | | | | |
| | | 1779 | 0:23:54 | Bridge-Grvl.Rd.Sec.22 | 17,200 | | | Can't Tell | | | |
| | | 1835 | 0:24:50 | End Hurt Cr. | | | | | | | |
| 3-11 | Goodwin Creek | 1857 | 0:25:11 | Mouth Goodwin Cr. | 0 | Sand-bed. | Steep, unstable | Grass/shrub | Mainly cultivated | Structures | Unstable |
| | | | | on Long Cr. | (37,500) | Sinuous | eroding banks | | | Look OK | |
| | | 1862 | 0:25:17 | Station 1 | | w/extensive bar | | | | | |
| | | 1873 | 0:25:28 | Bridge-Grvl.Rd.Sec.2&11 | 3,800 | deposits | | | | | |
| | | 1911 | 0:26:07 | Sta.2&Eureka Rd.Sec.2&35 | 11,700 | As Above | | | | | |
| | | 2015 | 0:27:55 | Station 3&4 Sec.31 | 21,000 | Sand-bed | Some bank | Narrow woody | Mainly cultivated | OK | |
| | | | | Bridge-Grvl.Rd. | | Moderately | erosion | buffer | | | |
| | | 2018 | 0:27:58 | Left Goodwin to Sta.6 | 24,800 | sinuous | | | | | |
| | | | | | | Bar deposits | | | | | |
| | | 2049 | 0:28:31 | Sta.6 - Sec.30 | | Can't Tell | | | | | |
| | | 2081 | 0:29:05 | Bridge-Pvd.Rd.Sec.31&32 | | Sand-bed | Unstable banks | Grasses | Cultivated | Can't Tell | Unstable banks |
| | | 2104 | 0:29:28 | Sta.5 | 24,000 | Sinuous | | | | | |
| | | 2168 | 0:30:37 | Sta.8 & 9 - Sec.29 | 29,300 | | | | | | |
| | | 2190 | 0:31:02 | Sta. 12 | 39,500 | | | | | | |
| | | 2190 | 0:31:02 | End Goodwin Cr. | | | | | | | |
| 3-11 | Caney Creek | 0932 | 0:11:05 | Mouth Caney Cr. | 0 | | | | | | |
| | | | | on Long Cr. | (49,000) | | | | | | |
| | | 0950 | 0:11:20 | Grvl.Rd.Sec.13-Bridge | 4,500 | | | | | | |
| | | 0956 | 0:11:25 | Structure-Sec.18 | | | | | | | |
| | | 0978 | 0:11:43 | Structure-Sec.18 | | | | | | | |
| | | 1000 | 0:12:01 | Structure-Sec.17 | 12,500 | | | | | | |

## Table 31 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Caney Creek (Continued) | 1000 | 0:12:01 | Bridge-Pvd.Rd. | | | | | | | |
| | | 1104 | 0:13:29 | Bridge-Grvl.Rd.Sec.21&22 | 24,500 | | | | | | |
| | | 1104 | 0:13:29 | End Caney Cr. | | | | | | | |
| 3-11 | Hotophia Creek | 2408 | 0:36:00 | Mouth Hotophia Cr. On Tallhatchie River | 0 | Sand bed Meandering Extensive bar deposits | Steep, unstable | Shrubs & grass some trees | Mainly cultivated some wood | Mostly OK | Actively migrating & widening channel |
| | | 2485 | 0:36:31 | Bridge - Hwy 35 | 7,100 | | | | | | |
| | | 2580 | 0:38:21 | Mouth Harris Cr. on Hotophia Cr. | 27,000 | | | | | | |
| | | 2622 | 0:39:11 | Bridge - Hwy 6 | 30,300 | | | As Above | | | |
| | | 2627 | 0:39:17 | Bridge-Grvl.Rd.Sec.9 | 31,900 | | | | | | |
| | | 2633 | 0:39:24 | Structure | 32,300 | | | | | | |
| | | 2644 | 0:39:37 | Mouth Hill Cr. on Hotophia Cr. | 35,000 | | | | | | |
| | | 2680 | 0:40:19 | Bridge-Grvl.Rd.Sec.10&11 | 42,800 | | | | | | |
| | | 2701 | 0:40:45 | Structure | 47,300 | | | | | | |
| | | 2705 | 0:40:49 | Bridge-Hwy315 | 48,000 | | | As Above | | | |
| | | 2708 | 0:40:53 | Mouth Deer Cr. on Hotophia Cr. | 48,100 | | | | | | |
| | | 2731 | 0:41:20 | Structure | 53,700 | | | | | | |
| | | 2734 | 0:41:25 | Mouth Marcum Cr. on Hotophia Cr. | 53,900 | | | | | | |
| | | 2763 | 0:42:00 | New Structure | 57,300 | Sand bed | Steep unstable banks | Shrubs & grasses | Cultivated & woody | Seem OK | Headcuts |
| | | 2799 | 0:42:43 | Bridge-Pvd.Rd.Sec.32&5 | 62,400 | | | | | | |
| | | 2860 | 0:43:58 | End Hotophia Cr. | 71,000 | | | | | | |
| 3-11 | Harris Creek | 2869 | 0:44:08 | Mouth Harris Cr. on Hotophia Cr. | 0 (27,000) | Can't Tell | Steep, eroding banks | Trees & shrubs | Cultivated & woody | Can't Tell | Appears incised & laterally unstable |
| | | 2871 | 0:44:11 | 2 Structures | 1,000 | | | | | | |
| | | 2879 | 0:44:21 | Bridge-Hwy 6 | 2,800 | | | | | | |
| | | 2885 | 0:44:28 | Bridge-Pvd.Rd.Sec.7&8 | 4,700 | | | | | | |
| | | 2919 | 0:45:11 | End Harris Cr. | | | | | | | |
| 3-11 | Hill Creek | 2936 | 0:45:32 | Mouth Hill Cr. on Hotophia | 0 (35,000) | Sand-bed Highly sinuous point bars | Very steep, eroding outer banks | Shrubs, trees | Cultivated & woody | OK | Planform instability |
| | | 2941 | 0:45:39 | Structure | 1,500 | | | | | | |
| | | 2949 | 0:45:49 | Structure | 3,800 | | | | | | |
| | | 2950 | 0:45:50 | Bridge-Grvl.Rd.Sec.9&16 | 3,900 | Can't Tell | | | | | |
| | | 2964 | 0:46:06 | Bridge-Grvl.Rd.Sec.15 | 7,000 | | | | | | |
| | | 2964 | 0:46:06 | End Hill Cr. | | | | | | | |
| 3-11 | Deer Creek | 2977 | 0:46:23 | Mouth Deer Cr. on Hotophia | 0 (48,100) | Can't Tell | | | | | |
| | | 2983 | 0:46:30 | Structure | 1,500 | | | | | | |
| | | 2985 | 0:46:33 | Structure | 2,000 | | | | | | |

# Table 31 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Deer Creek (Continued) | 2991 | 0:46:41 | Bridge-Grvl.Rd.Sec.12 | 3,900 | Can't Tell | | | | | |
| | | 3040 | 0:47:43 | Deer Cr. Res. | 10,000 | | | | | | |
| | | 3063 | 0:48:13 | End Deer Cr. | | | | | | | |
| 3-11 | Marcum Creek | 3074 | 0:48:27 | Mouth Marcum Cr. on Hotophia | 0 (53,800) | Sand-bed Moderately sinuous | Steep banks Some local erosion | Very narrow buffer of trees & shrubs | Cultivated | Seems OK | Tending to unstable |
| | | 3083 | 0:48:37 | Structure | 3,000 | | | | | | |
| | | 3094 | 0:48:52 | Bridge-Grvl.Rd.Sec.6&7 | 4,600 | | | | | | |
| | | 3113 | 0:49:16 | Res. | | | | | | | |
| | | 3154 | 0:50:09 | End Marcum Cr. | | | | | | | |
| 3-11 | Otoucalofa Creek | 3155 | 0:50:10 | Mouth Otoucalofa Cr. on Yocona River-Sec.34 | 0 | Sand-bed w/dunes Straight Some alternate bar deposits | Appears stable | Shrubs thin woody buffer | Cultivated some woodland | OK | No apparent problem |
| | | 3221 | 0:51:36 | Susie Perry Cr.-Left | 18,000 | | | | | | |
| | | 3227 | 0:51:44 | Pwr. Line-Sec.7 | 21,500 | | | | | | |
| | | 3241 | 0:52:02 | Bridge-Hwy 7-Sec.7 | 23,600 | | | As Above | | | |
| | | 3243 | 0:52:05 | Johnson Cr.-Left | 24,000 | | | | | | |
| | | 3252 | 0:52:17 | Town Cr.-Right | 26,500 | Sand-bed w/dunes & alternate bars Moderately sinuous Possible nickpoints | Step eroding unstable | Shrubs & grass isolated trees | Cultivated | Looks OK | Tending to unstable |
| | | 3260 | 0:52:27 | Bridge-Old Hwy 7-Sec.8 | 28,600 | | | | | | |
| | | 3263 | 0:52:31 | Bridge-Old R.R.-Sec.8 | 29,500 | | | | | | |
| | | 3280 | 0:52:54 | Bridge-Pvd.St.St.Sec.16&9 | 33,800 | " | " | Narrow woody buffer | " | " | " |
| | | 3317 | 0:53:42 | Bridge-Pvd.Rd.Sec.14 | 43,200 | Sand-bed w/dunes Sinuous more extensive pt. bars | Active outer bank erosion | Narrow woody buffer | Cultivated | OK | Active migrating channel |
| | | 3337 | 0:54:10 | GreasyCr.-Right-Sec.11&12 | 47,800 | | | | | | |
| | | 3359 | 0:54:38 | Moore Cr.-Right-Sec.6&7 | 57,500 | Torturous meanders w/point bars | Unstable outer banks | Shrubs & trees | Cultivated some woodland | N/A | More stable, but migrating laterally |
| | | 3364 | 0:54:46 | Gordon Branch | 59,500 | | | | | | |
| | | 3390 | 0:55:20 | Otoucalofa South#1-Left | 63,600 | Sand-bed | | | | | |
| | | 3444 | 0:56:33 | Bridge-Pvd.Rd.Sec.5 | 72,000 | As Above | As Above | As Above | As Above | Seems OK | |
| | | 3459 | 0:56:54 | Mills Cr.-Sec.33&34 | 75,700 | | | | | | |
| | | 3472 | 0:57:11 | Smith South-Left-Sec.33&34 | 79,800 | Less sinuous | As Above | " | " | N/A | No apparent problem |
| | | 3493 | 0:57:41 | Hanna Cr.-Right | 80,000 | Sand-bed some point bars | | | | | |
| | | 3514 | 0:58:09 | Ford | | Sand bed Straight | Local bank erosion | " | " | OK | " |
| | | 3526 | 0:58:26 | Starter Cr.-Right-Sec.34 | 86,400 | Sand-bed Sinuous w/well developed bars | Active outer bank erosion | " | " | N/A | Planform instability |
| | | 3582 | 0:59:43 | Bridge-Grvl.Rd.Sec.31&36 | 106,800 | | Can't Tell - Too high | | | | |

# Table 31 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Otoucalofa Creek | 3592 | 0:59:56 | Dickey Cr.-Right | 109,500 | | | Creek too small | | | |
| | (Continued) | 3666 | 1:01:40 | Shippy Cr.-Right | 115,000 | | | & obscured by some trees | | | |
| | | 3675 | 1:01:53 | Bridge-Drivers flat-Sec.6 | 117,300 | | | | | | |
| | | 3696 | 1:02:23 | Farm Rd.-Sec.8 | 121,900 | | | | | | |
| | | 3709 | 1:02:40 | Bridge-Driveway-Sec.8 | 125,500 | | | | | | |
| | | 3722 | 1:02:58 | Bridge-Grvl.Rd.Sec.9 | 129,200 | | | | | | |
| | | 3738 | 1:03:22 | Pipeline | 134,000 | | | | | | |
| | | 3746 | 1:03:33 | Bridge-Hwy 9W-Sec.15 | 135,900 | | | | | | |
| | | 3746 | 1:03:33 | End Otoucalofa Cr. | | | | | | | |
| 3-11 | Susie Perry Creek | 3810 | 1:05:05 | Mouth Susie Perry Cr. | 0 | Can't Tell | Steep banks some | Shrubs & trees | Mainly cultivated | Can't Tell | Incised? Tending to |
| | | | | on Otoucalofa Cr. | (18,000) | Composition of | local instability | | | | unstable |
| | | 3846 | 1:05:27 | Bridge-Hwy 32-Sec.12 | 6,500 | bed fairly | | | | | |
| | | | | | | straight | | | | | |
| | | 3862 | 1:06:18 | Bridge-Pvd.Rd.Sec.13 | 11,500 | As Above | Active bank | " | " | Threat from | |
| | | | | | | | erosion | | | erosion of | |
| | | | | | | | | | | banks? | |
| | | 3880 | 1:06:45 | Bridge-Pvd.Rd.Sec.24 | 17,000 | Can't Tell | | | | | |
| | | 3880 | 1:06:45 | End Susie Perry Cr. | | | | | | | |
| 3-11 | Johnson Creek | 3888 | 1:06:56 | Mouth Johnson Cr. | 0 | Sand-bed | Steep, unstable | Shrubs | Mainly cultivated | Bank erosion | Width instability |
| | | | | on Otoucalofa Cr. | (24,000) | Fairly straight | banks | | | close to | |
| | | 3901 | 1:07:15 | Bridge-Hwy 32-Sec.1788 | 4,000 | | | | | bridge | |
| | | 3914 | 1:07:34 | Bridge-Old Hwy 7-Sec.17 | 8,100 | Sand-bed | Steep, unstable | As Above | As Above | Can't Tell | Tending to unstable |
| | | 3918 | 1:07:40 | Old R.R. | 9,600 | Straight | banks | | | | |
| | | 3970 | 1:08:57 | End Johnson Cr. | | Bar deposition | | | | | |
| 3-11 | Town Creek | 3978 | 1:08:57 | Mouth Town Cr. | 0 | Can't Tell | | | | | |
| | | | | on Otoucalofa Cr. | (26,500) | | | | | | |
| | | 3991 | 1:09:28 | Bridge-Old Hwy 7-Sec.8 | 3,900 | | | | | | |
| | | 3996 | 1:09:35 | Bridge-Hwy 315-Sec.4 | 5,100 | | | | | | |
| | | 4011 | 1:09:47 | Bridge-Pvd.St.Sec.4 | 10,000 | | | | | | |
| | | 4034 | 1:10:31 | End Town Cr. | | | | | | | |
| 3-11 | Greasy Creek | 4052 | 1:10:58 | Mouth Greasy Cr. | 0 | Bed material | Steep, unstable | Grass & shrubs | Cultivated | Can't Tell | Tending to unstable |
| | | | | on Otoucalofa Cr. | (47,800) | unknown | banks | | | | |
| | | 4070 | 1:11:24 | Bridge-Grvl.Rd.Sec.1 | 5,000 | Sinuous | | | | | |
| | | 4084 | 1:11:45 | Bridge-Grvl.Rd.Sec.1 | 9,000 | " | " | " | " | " | " |
| | | 4123 | 1:12:45 | Bridge-Pvd.Rd.Sec.25 | 17,900 | Can't Tell | | | | | |
| | | 4123 | 1:12:45 | End Greasy Cr. | | | | | | | |
| 3-11 | Moore Creek | 4208 | 1:14:52 | Mouth Moore Cr. | 0 | Can't Tell | | | | | |
| | | | | on Otoucalofa Cr. | (57,500) | | | | | | |
| | | 4222 | 1:15:14 | Bridge-Hwy 315-Sec.6 | 4,000 | | | | | | |

## Table 31 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Moore Creek | 4232 | 1:15:29 | Sand Cr.-Right | 7,000 | | | | | | |
| | (Continued) | 4268 | 1:16:24 | End Moore Cr. | | | | | | | |
| 3-11 | Sand Creek | 4280 | 1:16:42 | Mouth of Sand Creek | | Can't Tell | | | | | |
| | | 4342 | 1:18:19 | End Sand Cr. | | | | | | | |
| 3-11 | Gordon Branch | 4148 | 1:13:21 | Mouth Gordon Branch | 0 | Can't Tell | | | | | |
| | | | | on Otoucalofa Cr. | (59,500) | | | | | | |
| | | 4158 | 1:13:36 | Bridge-Grvl.Rd.Sec.7 | 3,000 | | | | | | |
| | | 4166 | 1:13:49 | End Gordon Branch | | | | | | | |
| 3-11 | Otoucalofa S.#1 | 4378 | 1:19:14 | Mouth Otoucalofa South #1 | 0 | Can't Tell | | | | | |
| | | 4392 | 1:19:37 | on Otoucalofa Cr. | (63,600) | | | | | | |
| | | 4396 | 1:19:42 | Bridge-Pvd.Rd.Sec.5 | 4,500 | | | | | | |
| | | 4397 | 1:19:43 | Driveway-Sec.4&5 | 5,400 | | | | | | |
| | | 4398 | 1:19:45 | Spring Cr.-Left | 5,500 | | | | | | |
| | | 4409 | 1:20:03 | End Otoucalofa South #1 | | | | | | | |
| 3-11 | Spring Creek | 4418 | 1:20:17 | Mouth Spring Creek | 0 | Can't Tell | | | | | |
| | | | | on Otoucalofa South #1 | (5,500) | | | | | | |
| | | 4442 | 1:20:54 | End Spring Cr. | | | | | | | |
| 3-11 | Mills Creek | 4447 | 1:21:02 | Mouth Mills Cr. | 0 | Sand-bed | Steep banks | Grass, some | Cultivated & | Seems OK | Tending to unstable |
| | | | | on Otoucalofa Cr. | (75,700) | Low sinuosity | unstable | shrubs | woodland | | |
| | | 4455 | 1:21:14 | Bridge-Hwy 315-Sec.33 | 2,500 | appears | | | | | |
| | | 4480 | 1:21:54 | End Mills Cr. | | Incised | | | | | |
| 3-11 | Smith South | 4481 | 1:21:55 | Mouth Smith South | 0 | Sand-bed | Steep banks but | Woody veg | " | " | |
| | | | | on Otoucalofa Cr. | (79,800) | Low sinuosity | mostly stable | | | | |
| | | 4491 | 1:22:11 | Bridge-Pvd.Rd.Sec.3 | 4,200 | | | | | | |
| | | 4561 | 1:24:03 | Pipeline-Sec.11 | 12,800 | Can't Tell | | | | | |
| | | 4600 | 1:25:05 | Grvl.Rd.Sec.11&12 | 18,000 | | | | | | |
| | | 4600 | 1:25:05 | End Smith South Cr. | | | | | | | |
| 3-11 | Sarter Creek | 4607 | 1:25:17 | Mouth Sarter Cr. | 0 | Low sinuosity | Steep banks | Grass & shrubs | Mainly cultivated | OK | Narrow, incised channel. |
| | | | | on Otoucalofa Cr. | (86,400) | Can't tell bed | | | some wood | | Possible future instability? |
| | | 4631 | 1:25:55 | Bridge-Hwy 315-Sec.33 | 5,200 | | | | | | |
| | | 4653 | 1:26:31 | Bridge-Grvl.Rd. | 6,800 | Sinuous | Steep banks | " | Cultivated & wood | Can't Tell | |
| | | 4661 | 1:26:43 | Left Sarter-got on trib. | | | | | | | |
| | | 4680 | 1:27:14 | Bridge-Grvl.Rd.Sec.26 | 9,400 | | | | | | |
| | | 4701 | 1:27:48 | Bridge-Hwy9W-Sec.23 | 15,400 | Can't Tell | | | | | |
| | | 4704 | 1:27:53 | Bridge-Driveway | 16,200 | | | | | | |
| | | 4737 | 1:28:47 | Grvl.Rd.Sec.13 | 20,000 | | | | | | |
| | | 4753 | 1:29:13 | End Sarter Cr. | | | | | | | |

## Table 31 (Concluded)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Smith & Dickey Creek | 4765 | 1:29:32 | Mouth of Otoucalofa | (109,500) | Straight ditch | Steep banks | Grass | Cultivated | Can't Tell | Possible future width instability |
| | | 4771 | 1:29:42 | Bridge-Hwy 9W-Sec.31 | 2,000 | Bed material | | | | | |
| | | 4775 | 1:29:49 | Dickey Cr. | 2,900 | obscured | | | | | |
| | | 4776 | 1:29:51 | Bridge-Grvl.Rd.Sec.31 | 3,500 | Incised? | | | | | |
| | | 4796 | 1:30:25 | End Smith & Dickey Cr. | | | | | | | |
| 3-11 | Shippy Creek | 4801 | 1:30:32 | Mouth Shippy Cr. on Otoucalofa | 0 (114,000) | Gravel? Bed Low sinuosity | Steep, eroding banks | Trees, grass, shrubs | Mainly cultivated | Can't Tell | Unstable |
| | | 4808 | 1:30:43 | Bridge-Grvl.Rd.Sec. | 4,000 | | | | | | |
| | | 4818 | 1:31:00 | Bridge-Hwy 9W-Sec.5 | 7,100 | | | | | | |
| | | 4830 | 1:31:20 | End Shippy Cr. | | | | | | | |

## Table 32
## Log of Aerial Videotape 3

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Burney Branch | 0029 | 0:00:23 | Mouth of Burney Branch | | Sinuous Sand-bed | Mostly stable | Woody veg | Woodland & cultivated | OK | No apparent problem |
| | | 0043 | 0:00:32 | Structure | | | | | | OK | |
| | | 0060 | 0:00:42 | Bridge of Fudgetown Rd. | | | | | | Can't Tell | |
| | | 0088 | 0:01:00 | Structure | | | | | | | |
| | | 0172 | 0:01:54 | Structure | | Low sinuosity Sand-bed w/pt bars | Mostly stable | Medium woody buffer | Woodland & suburban | OK | No apparent problem |
| | | 0198 | 0:02:11 | Driveway to Retardation Center | | | | | | | |
| | | 0208 | 0:02:18 | Driveway to Retardation Center | | | | | | | |
| | | 0220 | 0:02:26 | Driveway to Industrial Park | | | | | | | |
| | | 0220 | 0:02:26 | Structure | | | | | | | |
| | | 0226 | 0:02:30 | Hwy. 7 | | | | | | | |
| | | 0255 | 0:02:49 | Bridge on Belk Drive | | Straight sand bed ditch | Mostly stable | Grass & shrubs | Suburban | OK | No apparent problem |
| | | 0281 | 0:03:07 | Hwy. 6 & street | | | | | | | |
| | | 0304 | 0:03:23 | University Ave. | | | | | | | |
| | | 0311 | 0:03:27 | Jackson Ave. | | | | | | | |
| | | 0319 | 0:03:19 | School Driveways | | | | | | | |
| | | 0334 | 0:03:43 | End of Sisk Ave. | | | | | | | |
| | | 0334 | 0:03:43 | Start detailed shots of Burney Br. | | Nothing to add to above | | | | | |
| | | 0453 | 0:05:07 | End detailed shots of Burney Br. | | | | | | | |
| | | 0474 | 0:05:22 | Mouth of Tributary #1 | | As Above | | | | | |
| | | 0474 | 0:05:22 | Hwy. 7 | | | | | | | |
| | | 0480 | 0:05:34 | Old Hwy. 7 | | | | | | | |
| | | 0578 | 0:06:37 | End of Tributary #1 | | | | | | | |
| | | 0636 | 0:07:21 | Mouth of Tributary #2 | | | | | | | |
| | | 0636 | 0:07:21 | Hwy. 7 | | | | | | | |
| | | 0657 | 0:07:37 | Old Hwy. 7 | | Can't Tell | | | | | |
| | | 0665 | 0:07:43 | Pvd. street | | | | | | | |
| | | 0701 | 0:08:11 | Hwy. 6 | | | | | | | |
| | | 0749 | 0:08:48 | End of Tributary #2 | | | | | | | |
| 3-11 | Toby Tubby Creek | 0852 | 0:10:09 | Mouth Toby Tubby Cr. on Sardis Lake | | Highly sinuous Sand-bed w/dunes & well developed bars | Mostly stable | Woody | Woodland | OK | No apparent problem |
| | | 0982 | 0:11:54 | Bridge-Old Sardis Rd.Sec.33&34 | | | | | | | |
| | | 1019 | 0:12:25 | Bridge-Grvl.Rd.Sec.3 | | | | | | | |
| | | 1447 | 0:14:11 | Pvd.Rd.Bridge | | | | | | | |
| | | 1501 | 0:15:01 | Old Sardis Rd. | | Can't Tell | | | | | |
| | | 1511 | 0:15:11 | End at Lab | | | | | | | |
| | | 1511 | 0:15:11 | End Toby Tubby Cr. | | | | | | | |

## Table 32 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3-11 | Berry Creek | 1523 | 0:15:22 | Mouth of Berry Cr. | | Can't Tell | | | | | |
| | | 1548 | 0:15:48 | Old Sardis Rd. | | | | | | | |
| | | 1610 | 0:16:45 | Pvd. Rd. | | | | | | | |
| | | 1708 | 0:18:19 | End Berry Cr. | | | | | | | |
| 3-11 | Goose Creek | 1726 | 0:18:37 | Mouth of Goose Cr. | | Can't Tell | | | | | |
| | | 1885 | 0:21:16 | Hwy. 6 | | | | | | | |
| | | 1920 | 0:21:51 | End Goose Cr. | | | | | | | |
| 3-11 | East Goose Creek | 1929 | 0:22:00 | Mouth of East Goose Cr. | | | | | | | |
| | | 1994 | 0:23:08 | Pvd. Street | | | | | | | |
| | | 2120 | 0:25:21 | West Jackson Ave. | | Can't Tell | | | | | |
| | | 2128 | 0:25:29 | Hwy. 6 | | | | | | | |
| | | 2154 | 0:25:57 | End East Goose Cr. | | | | | | | |
| 4-23 | Hickahala Creek | 2301 | 0:28:37 | Mouth Hickahala on Arkabutla Res. | 0 | Sand-bed w/dunes | Stable | Woody bank veg | Woodland & swamp | OK | No apparent problem |
| | | 2362 | 0:29:44 | Bridge-Pvd.Rd.Sec.36 | 10,200 | | | | | | |
| | | 2461 | 0:31:36 | Bridge-Hwy 51 & R.R. | | Very straight | | | | | |
| | | 2514 | 0:32:37 | Bridge-I 55 | 34,000 | planform | | | | | |
| | | 2520 | 0:32:53 | Senatobia Cr.-Left | 36,000 | Bars masked by | | | | | |
| | | 2569 | 0:33:41 | Bridge-Pvd.Rd.Sec.22 | 43,900 | high flow | | | | | |
| | | 2587 | 0:34:01 | Pipeline (2) | 47,400 | | | | | | |
| | | 2587 | 0:34:01 | Thornton Cr.-Right | 48,000 | | | | | | |
| | | 2591 | 0:34:06 | Basket Cr.-Left | 48,600 | Sand-bed w/dunes | Mostly stable | Shrubs & trees | Cultivated & | OK | No apparent problem |
| | | | | (Breakoff) | 54,400 | & poorly | some scour & | | woodland | | |
| | | 2659 | 0:35:26 | Lick Cr.-Right | 64,000 | developed pt. | instability | | | | |
| | | 2669 | 0:35:38 | Lick Br.-Right | 66,000 | bars | | | | | |
| | | 2686 | 0:35:58 | New Bridge | | Low sinuosity | | | | | |
| | | 2694 | 0:36:07 | Structure-Sec.17 | 71,000 | | | | | | |
| | | 2699 | 0:36:13 | James Wolf-Left | 72,000 | | | | | | |
| | | 2784 | 0:37:55 | Hwy 305 | 80,200 | Sand-bed w/dunes | Steep banks | Woody buffer | Cultivated | OK | |
| | | 2844 | 0:39:08 | Bridge-Hwy 306 | 96,000 | Poorly developed | potentially | | | | |
| | | 2851 | 0:39:16 | Beards Cr.-Right | 97,500 | bars v. low | unstable | | | | |
| | | 2868 | 0:39:37 | Low Drop Structure | 101,000 | sinuosity | | | | | |
| | | 2868 | 0:39:37 | Bridge-Pvd.Rd. | 101,200 | | | | | | |
| | | 2873 | 0:39:43 | Cathey Cr.-Left | | | | | | | |
| | | 2887 | 0:40:01 | Ditch-Right | 104,600 | | | | | | |
| | | 2904 | 0:40:21 | Low Drop Structure | 108,800 | | | | | | |
| | | 2912 | 0:40:31 | Bridge-Grvl.Rd. | 110,000 | As Above | | | | | |
| | | 2946 | 0:41:13 | Bridge-Grvl.Rd. | 119,300 | | | | | | |
| | | 2959 | 0:41:30 | Low Drop Structure | 121,700 | | | | | | |
| | | 2967 | 0:41:39 | Tributary left-mouth of North Fork Creek | 123,000 | | | | | | |

# Table 32 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-23 | Hickahala S.Fork | 2967 | 0:15:34 | Trib.-Left | 123,000 | Incised Sand-bed Not much bed deposition Sinuous | Steep banks, potentially unstable | Narrow shrub buffer | Mainly cultivated | OK | Tending to unstable |
| | | 2967 | 0:15:37 | Low Drop Structure +400' from fork | | | | | | | |
| | | 3002 | 0:16:08 | Bridge-Grvl.Rd. | | | | | | | |
| | | 3007 | 0:16:29 | Low Drop Structure | +1,200 | | | | | | |
| | | 3036 | 0:17:12 | Low Drop Structure-Sec.13 | | | | | | | |
| | | 3055 | 0:43:31 | Culverts-Hwy 4 1000'W. Marshall Co. | | | | | | | |
| | | 3055 | 0:43:31 | End Hickahala S. Fork | | | | | | | |
| | | 3056 | 0:43:32 | Re-flew S.Fork from upstream to mouth with close-ups of structures | | | | | | | |
| | | 3332 | 0:49:29 | | | | Nothing to add | | | | |
| 4-23 | James Wolf | 3354 | 0:49:58 | James Wolf Trib. | | Sand-bed Sinuous | Steep, unstable | Shrubs | Cultivated | Threat from bank erosion | Unstable width |
| | | 3411 | 0:51:13 | Bridge-Hwy 4 | 11,100 | | | | | | |
| | | 3462 | 0:52:22 | Bridge-Sec.35 - New Drop Structure under construction | 22,500 | Sand-bed Sinuous Not much bed deposition | Steep, unstable | Grass, shrubs | Cultivated | OK | Tending to unstable |
| | | 3479 | 0:52:45 | James Wolf-Martin Dale Fk (J.W. turns to left) | 26,500 | | | | | | |
| | | 3484 | 0:52:52 | James Wolf-Martin Dale Fk (J.W. turns to left) | 26,500 | | | | | | |
| | | 3554 | 0:54:34 | Rock Stabilizer | 36,500 | Sand-bed Sinuous Point bars | Incised, steep, unstable banks some deep-seated failures | Shrubs | Cultivated | Seem OK | Tending to unstable |
| | | 3565 | 0:54:42 | New Drop Structure under construction | 38,500 | | | | | | |
| | | 3573 | 0:54:64 | Bridges(2)-Hwy 306 | 40,700 | | | | | | |
| | | 3414 | 0:55:50 | Bridge-Pvd.Rd.Sec.28&29 | 46,300 | | | | | | |
| | | 3650 | 0:56:40 | Drop Structure | 49,700 | | | | | | |
| | | 3660 | 0:56:54 | Bridge(concrete)-Drt.Rd. | 52,700 | | | | | | |
| | | 3669 | 0:57:06 | took N. Fk. J.W. | 55,200 | | | | | | |
| | | 3694 | 0:57:41 | Iron Bridge-Drt.Rd. (N.Fk.J.W.Btwn Sec.34&35) | | As Above | | | | | |
| | | 3731 | 0:58:34 | Bridge-Drt.Rd.Sec.25 | <1.5mi. | | | | | | |
| | | 3741 | 0:58:48 | 2nd Bridge-Drt.Rd.Sec25 | | Can't Tell | | | | | |
| | | 3741 | 0:58:48 | End James Wolf Canal | | | | | | | |
| 4-23 | Senatobia Creek | 3924 | 1:03:09 | Hickahala passing under Hwy.51 | | | | | | | |
| | | 3950 | 1:03:47 | Hickahala passing under I55 | 34,000 | As for Hickahala | | | | | |
| | | 3960 | 1:04:00 | Mouth Senatobia Cr. on Hickala Cr. | 0 (36,000) | Straight | Stable | Woody | Mostly woodland | OK | No apparent problems |
| | | 4000 | 1:04:59 | Bridge-Hwy 4 | 7,500 | Bed obscured | | | | | |
| | | 4093 | 1:07:17 | Structure | 28,400 | Low sinuosity Sand-bed w/dunes | Incised steep, unstable banks | Narrow woody buffer | Cultivated | OK at present | Tending to unstable |
| | | 4098 | 1:07:23 | Senatobia-Nelson Fork (new concrete) | 29,300 | | | | | | |
| | | 4135 | 1:08:18 | Bridge-Pvd.Rd.Sec.19&24 | 39,400 | Sinuous Sand-bed | Incised steep | Shrubs | Cultivated | OK - some | Unstable |

## Table 32 (Concluded)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-23 | Senatobia Creek (Continued) | 4180 | 1:09:26 | Bridge-Grvl.Rd. | 51,200 | w/bars | unstable banks | | | potential threat from erosion | |
| | | 4196 | 1:09:51 | Ford | 56,000 | | | | | | |
| | | 4213 | 1:10:18 | Ford | 60,000 | | | | | | |
| | | 4228 | 1:10:38 | Bridge-Grvl.Rd. | 62,900 | | | | | | |
| | | 4274 | 1:11:49 | Bridge-Grvl.Rd. | 74,900 | Sinuous | Incised steep eroding banks | Grass | Cultivated some wood | OK | Unstable |
| | | 4314 | 1:12:49 | Ford | 79,900 | Sand-bed w/dunes | | | | | |
| | | 4329 | 1:13:12 | Reservoir at Headwater | 87,000 | Point bars | | | | | |
| | | 4329 | 1:13:12 | End Senatobia Cr. (Sec.21 Range 5WT.6S) | | | | | | | |

(Sheet 4 of 4)

# Table 33
# Log of Aerial Videotape 4

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-29 | Batupan Bogue | 0084 | 0:00:57 | Mouth Batupan Bogue | 0 | Sand-bed w/dunes | Mostly stable | Trees & shrubs | Woodland & | OK | |
| | | 0151 | 0:01:39 | Bridge-Hwy 8-Sec.17 | 8,000 | Extensive pt. bars | | | cultivated | | |
| | | 0262 | 0:02:53 | Mouth of Perry Cr.-Left | 13,000 | Sand-bed w/pt. | Steep but mostly | Medium woody | Cultivated some | OK | No apparent problem |
| | | 0402 | 0:04:30 | Bridge-Pvd.Rd.Sec.28 | 26,700 | bars. | stable | buffer | woodland | | |
| | | 0515 | 0:05:51 | Jack Cr.-Left-Sec.33 | 30,500 | Meandering | | As Above | | | |
| | | 0749 | 0:08:47 | Bridge-Pvd.Rd.Sec.14 | 52,500 | Sand-bed w/bars | Steep banks | Shrubs | Mainly cultivated | Bank protec- | Tending to unstable |
| | | 0781 | 0:09:12 | Little Bogue-Right | 56,400 | Meandering | unstable | | | tion failed | |
| | | 0781 | 0:09:12 | End Batupan Bogue | | | | | | u/s of bridge | |
| 4-29 | Big Bogue Creek | 0781 | 0:09:12 | Mouth Big Bogue Creek | 0 | | | | | | |
| | | 0906 | 0:10:52 | Sykes Cr.-Left | 8,100 | | | | | | |
| | | 0922 | 0:11:05 | Bridge-Grvl.Rd.Sec.24 | 9,500 | | | Can't Tell Obscured by Veg | | | |
| | | 0924 | 0:11:07 | Wilkens Cr.-Left | 10,000 | | | | | | |
| | | 1030 | 0:12:34 | Old Military Bridge site Sec.25 | 15,100 | | | | | | |
| | | 1086 | 0:13:22 | Jackson Cr.-Left-Sec.36 | 20,600 | | | | | | |
| | | 1148 | 0:14:15 | Bridge-Hwy 404-Sec.31 | 28,500 | | | | | | |
| | | 1226 | 0:15:23 | Mouth Worsham Cr. | 0 | | | | | | |
| | | 1226 | 0:15:23 | End Big Bogue Cr. | (35,000) | | | | | | |
| 4-29 | Worsham Creek | 1370 | 0:17:33 | Bridge-Grvl.Rd.Sec.9 | 15,400 | Incised | Unstable deep- | Grass | Cultivated | Bridges | Unstable width - channel |
| | | 1431 | 0:18:28 | Structure | 21,000 | Fairly straight | seated | | | under threat | widening |
| | | 1431 | 0:18:28 | Bridge-Pvd.Rd.Sec.16&21 on structure | 21,500 | Bed obscured | failures | | | from erosion (scour & bank) | |
| | | 1458 | 0:18:53 | Structure-Sec.21 | 26,500 | | | | | | |
| | | 1479 | 0:19:13 | Structure | 26,500 | | | Can't Tell | | | |
| | | 1514 | 0:19:46 | Gravel Rd. | 30,800 | | | | | | |
| | | 1514 | 0:19:46 | End Worsham Cr. | 30,800 | | | | | | |
| 4-29 | Middle Fork Worsham Creek | 1527 | 0:19:58 | Middle Fk. Worsham-Sec.15 | | Incised | Steep, unstable | Grasses | Cultivated | Seem OK | Tending to unstable |
| | | 1544 | 0:20:15 | Structure #1-Sec.22 | | Sand-bed | banks | Isolated trees | | | |
| | | 1544 | 0:20:15 | Bridge-Pvd.Rd.Sec.22 | | | | | | | |
| | | 1570 | 0:20:39 | Structure #2.Sec.22 | | | | | | | |
| | | 1580 | 0:20:49 | Structure #3-Sec.22 | | | | | | | |
| | | 1601 | 0:21:09 | End Middle Fk. Worsham | | | | | | | |
| 4-29 | East Fork Worsham Creek | 1657 | 0:22:03 | East Fk. Worsham Cr. | | | | | | | |
| | | 1685 | 0:22:31 | Structure #1-on East Fk. | | | | | | | |
| | | 1685 | 0:22:31 | Bridge-Pvd.Rd.Sec.22 | | | | | | | |

## Table 33 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-29 | East Fork | 1689 | 0:22:35 | Structure #2-on East Fk. | | | | | | | |
| | Worsham Creek | 1701 | 0:22:46 | Structure #3 | | | Can't Tell - Obscured by Vegetation | | | | |
| | (Continued) | 1769 | 0:23:54 | Bridge-Driveway-Sec.24 | | | | | | | |
| | | 1807 | 0:24:32 | Gravel Rd. Sec.24 | | | | | | | |
| | | 1807 | 0:24:32 | End Worsham Cr. | | | | | | | |
| | | 1833 | 0:24:58 | Tributary of East Fork | | | | | | | |
| | | 1905 | 0:26:12 | End of East Fork Tributary | | | | | | | |
| 4-29 | Eskridge Creek | 1959 | 0:27:07 | Mouth Eskridge Cr. | 0 | Sand-bed | Steep, some outer | Narrow woody | Cultivated | Seems OK | Tending to unstable |
| | | | | on Big Bogue | (35,000) | Point bars | bank erosion | buffer | | | |
| | | 2030 | 0:28:21 | Bridge-Pvd.Rd.Sec.8 | 5,400 | Sinuous | tending to | | | | |
| | | 2136 | 0:30:14 | Structure-Sec.20 | 20,000 | | unstable | | | | |
| | | 2141 | 0:30:20 | Bridge-Pvd.Rd.Sec.20&29 | 20,900 | Sand-bed | Steep, unstable | Shrubs | Mainly cultivated | OK | Tending to unstable |
| | | 2162 | 0:30:43 | Structure-Sec.20&29 | 24,000 | Pt bars | banks | isolated trees | | | |
| | | 2162 | 0:30:43 | Bridge-Driveway | 24,100 | Sinuous | | | | | |
| | | 2213 | 0:31:39 | End Eskridge Cr. | | | | | | | |
| 4-29 | Jackson Creek | 2225 | 0:31:51 | Mouth Jackson Cr. | 0 | | | | | | |
| | | | | on Big Bogue | (20,600) | | | | | | |
| | | 2231 | 0:31:58 | Bridge-Hwy 51-Sec.36 | 1,600 | Can't Tell | | | | | |
| | | 2240 | 0:32:08 | Bridge-Driveway | 3,000 | | | | | | |
| | | 2251 | 0:32:21 | Bridge-Hwy 404-Sec.36 | 5,200 | | | | | | |
| | | 2323 | 0:33:40 | End Jackson Cr. | | | | | | | |
| 4-29 | Wilkins Creek | 2327 | 0:33:45 | Mouth Wilkens Cr. | 0 | Sand-bed | Steep banks | Shrubs | Mainly cultivated | OK | Active lateral migration |
| | | | | on Big Bogue | (9,900) | Point bar | Outer bank | | | | |
| | | 2358 | 0:34:19 | Bridges-Hwy51&R.R.Sec.25 | 7,200 | deposits | erosion | | | | |
| | | 2420 | 0:35:30 | Bridge-Hwy 4-Sec.35 | 18,300 | Meandering | | | | | |
| | | 2518 | 0:37:23 | Bridge-Grvl.Rd. | 36,700 | | | | | | |
| | | 2518 | 0:37:23 | End Wilkens Cr. | | | | | | | |
| 4-29 | Sykes Creek | 2540 | 0:34:49 | Mouth Sykes Cr. | 0 | Sand-bed | Steep, unstable | Woody buffer | Cultivated some | OK | Tending to unstable |
| | | | | on Big Bogue | (8,100) | Extensive point | banks | | woodland | | |
| | | 2564 | 0:38:16 | Bridge R.R.-Sec.23 | 7,600 | bars | | | | | |
| | | 2564 | 0:38:16 | Bridge-Hwy 51-Sec.23 | 7,800 | Meandering | | | | | |
| | | 2629 | 0:39:33 | Bridge-Grvl.Rd.Sec.27 | 20,800 | | | | | | |
| | | 2676 | 0:40:29 | Bridge-Hwy 404-Sec.33 | 29,800 | | | | | | |
| | | 2694 | 0:40:51 | Bridge-Hwy 55 | 32,700 | Can't Tell | | | | | |
| | | 2694 | 0:40:51 | End Sykes Cr. | | | | | | | |
| 4-29 | Jack Creek | 2701 | 0:40:59 | Mouth Jack Cr. | 0 | Sand-bed w/dunes | Steep, unstable | Narrow woody | Mainly cultivated | OK | |
| | | | | on Batupan Bogue | (30,500) | Extensive point | outer banks | buffer | | | |
| | | 2739 | 0:41:45 | Bridge-R.R.-Sec.33 | 7,400 | bars | | | | | |
| | | 2747 | 0:41:54 | Bridge-Hwy 51-Sec.33 | 9,000 | | | | | | |
| | | 2856 | 0:44:08 | End Jack Cr. | | | | | | | |

## Table 33 (Continued)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-29 | Perry Creek | 3116 | 0:49:36 | Mouth Perry Cr. | 0 | Sand-bed with | Steep banks | Shrubs some | Mainly cultivated | OK | Tending to unstable |
| | | | | on Batupan Bogue | (13,700) | extensive point | potentially | woody | | | |
| | | 3102 | 0:49:17 | Structure | 4,000 | bars | unstable | | | | |
| | | 3099 | 0:49:14 | Bridge-Pvd.St. | 5,000 | Meandering | | | | | |
| | | 3089 | 0:49:00 | Bridge-Pvd.St. | 8,000 | | | | | | |
| | | 3068 | 0:48:34 | Structure | 12,000 | | | | | | |
| | | 3025 | 0:47:40 | Bridge-Hwy 51-Sec.29 | 15,000 | | | | | | |
| | | 3019 | 0:47:31 | Bridge-Pvd.St. | 16,800 | | | | | | |
| | | 2988 | 0:46:53 | Bridge-Pvd.Rd. | 21,400 | | | | | | |
| | | 2954 | 0:46:10 | Bridge-I 55-Sec.36 | 29,500 | Sand-bed | Blowout-rapidly eroding | Grass | Cultivated | Under threat from blowout | Unstable |
| | | 2924 | 0:45:53 | Site #1 | | Sand-bed | Steep unstable banks | Grass | Mainly cultivated | OK | Tending to unstable |
| | | 2924 | 0:45:31 | Site #2 | | | | | | | |
| | | 2910 | 0:45:15 | Site #3 | | | | | | | |
| | | 2902 | 0:45:05 | Site #4 | | | | | | | |
| | | 2868 | 0:44:22 | End Perry Cr. | | | | | | | |
| 4-29 | Little Bogue Creek | 3130 | 0:49:53 | Mouth Little Bogue Cr. | | Sand bed w/dunes | Unstable banks | Woody & shrubs | Mainly cultivated | Some bank | |
| | | | | on Batupan Bogue Cr. | 56,400 | & point bars | | veg. | | erosion | |
| | | 3154 | 0:50:25 | Bridge-Pvd.Rd.Sec.18 | 63,100 | Sinuous | | | | threat | |
| | | 3208 | 0:51:36 | Bridge-Pvd.Rd.Sec.20&21 | 75,900 | | | | | | |
| | | 3243 | 0:52:21 | Campbell Cr.-Right | 83,000 | Sand/bed Point bars Tortuous meanders | Unstable | Shrubs | Mainly cultivated | OK | Unstable |
| | | 3326 | 0:54:12 | Powell Cr.-Right | 98,900 | Sand bed point | Steep unstable | Woody buffer | Cultivated some | OK | Tending to unstable |
| | | 3331 | 0:54:18 | Bridge-Grvl.Rd.Sec.29 | 100,500 | bars | | | woodland | | |
| | | 3357 | 0:54:52 | Mouse Cr.-Left | 106,000 | Sinuous | | | | | |
| | | 3378 | 0:55:21 | Bridge-Grvl.Rd.Sec.29 | 110,400 | | | | | | |
| | | 3407 | 0:56:00 | Bridge-Grvl.Rd.Sec.28 | 117,800 | Sand-bed | Unstable banks | Shrubs | Mostly cultivated | OK | Tending to unstable |
| | | 3427 | 0:56:28 | Caffe Cr.-Left | 122,000 | Point bars | | | | | |
| | | 3430 | 0:56:32 | Structure | 123,500 | | | | | | |
| | | 3482 | 0:57:42 | End Little Bogue Cr. | | | | | | | |
| 4-29 | Caffe Branch | 3493 | 0:57:57 | Mouth Caffe Branch | 0 | Sand-bed | Steep banks | Narrow woody | Cultivated | OK | Tending to unstable |
| | | | | on Little Bogue Cr. | (122,000) | Incised | | buffer | | | |
| | | 3502 | 0:58:10 | Structure | 2,000 | | | | | | |
| | | 3502 | 0:58:10 | Bridge-Grvl.Rd.Sec.21 | 2,900 | Can't Tell | | | | | |
| | | 3531 | 0:58:49 | End Caffe Branch | 21,000 | | | | | | |
| 4-29 | Crowder Creek | 3577 | 0:59:53 | Mouth Crowder Cr. | 0 | Sand-bed with | Steep banks | Woody buffer | Mainly cultivated | OK | Tending to unstable |
| | | | | on Little Bogue Cr. | | point bars | | | | | |
| | | 3587 | 1:00:07 | Bridge-Military Rd.Sec.18 | 2,200 | Sinuous | | | | | |
| | | 3598 | 1:00:22 | Bridge-Military Rd.Sec.7 | 5,200 | | | | | | |
| | | 3603 | 1:00:29 | Bridge-Military Rd.Sec.7 | 8,000 | | | | | | |

## Table 33 (Concluded)

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 4-29 | Crowder Creek (Continued) | 3628 | 1:00:53 | New Structure Sec.6 | 12,100 | | | | | | |
| | | 3630 | 1:01:07 | Structure-Sec.5 | 15,000 | | | | | | |
| | | 3717 | 1:03:10 | End Crowder Cr. | | | | | | | |
| 4-29 | Epison Creek | 3754 | 1:04:03 | Mouth Epison Cr. on Little Bogue Cr. | 0 (66,100) | | | Can't Tell | | | |
| | | 3769 | 1:04:25 | Bridge-Pvd.Rd.Sec.17 | 5,100 | | | | | | |
| | | 3796 | 1:05:03 | End Epison Cr. | | | | | | | |
| 4-29 | Campbell Creek | 3830 | 1:05:52 | Mouth Campbell Cr. on Little Bogue Cr. | 0 (83,000) | Bed obscured Appears incised | Steep, some blow outs | Shrubs | Mainly cultivated | OK | Tending to unstable |
| | | 3856 | 1:06:29 | New structure | | Relatively | | | | | |
| | | 3856 | 1:06:29 | Bridge-Pvd.Rd.Sec.23 | 8,700 | straight | | | | | |
| | | 3886 | 1:07:12 | Bridge-Military Rd. | | As Above | | | | | |
| | | 3907 | 1:07:43 | Pvd.Rd.Sec.2 | 23,500 | Can't Tell | | | | | |
| | | 3907 | 1:07:43 | End Campbell Cr. | | | | | | | |
| 4-29 | Powell Creek | 3919 | 1:08:00 | Mouth Powell Cr. on Little Bogue Cr. | 0 (98,800) | Sand-bed Low sinuosity | Steep banks potentially unstable | Grass | Mainly cultivated | OK | Stable at present |
| | | 3925 | 1:08:08 | Bridge-Grvl.Rd.Sec.24 | 2,000 | | | | | | |
| | | 3947 | 1:08:42 | Bridge-Grvl.Rd.Sec.18&19 | 8,400 | | | | | | |
| | | 3999 | 1:09:59 | End Powell Cr. | | | | | | | |
| 4-29 | Mouse Creek | 4020 | 1:10:29 | Mouth Mouse Cr. on Little Bogue Cr. | 0 (106,000) | Bed obscured Low sinuosity | Can't Tell | Shrubs | Cultivated | OK | |
| | | 4026 | 1:10:39 | Bridge-Hwy 404-Sec.30 | 1,900 | bank line Sinuous thalweg | | | | | |
| | | 4044 | 1:11:06 | Structure-Sec.32 (under construction) | 6,000 | Sand-bed Sinuous | Steep, unstable | Grass | Cultivated | OK | Width instability |
| | | 4072 | 1:11:47 | Bridge-Grvl.Rd.Sec.5 | 11,200 | Point bars | | | | | |
| | | 4200 | 1:15:01 | Beat 3 Lake | 24,000 | Can't Tell | | | | | |
| | | 4200 | 1:15:01 | End Mouse Cr. | | | | | | | |

# Table 34
# Log of Aerial Videotape 5

| Date | Stream Name | Footage Tape Start | VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6-16 | Hurricane Creek | 0379 | 0:04:13 | Mouth Hurricane Cr. | | Straight | Natural levees | Woody | | N/A | No apparent problem |
| | | 0379 | 0:04:13 | Delta extending into | | Bed obscured | stable | | | | |
| | | | | Arkabutla Res. | | | | | | | |
| | | 0474 | 0:05:20 | Bridge-Hwy 304 | | Bed obscured | Stable banks | Thin woody | Cultivated | OK | No problem |
| | | 0520 | 0:05:23 | Bridge-Pvd.Rd.Sec.7&8 | | Low sinuosity | | buffer | | | |
| | | 0712 | 0:08:17 | Bridge-Pvd.Rd.Sec.26&35 | | | | | | | |
| | | 0735 | 0:08:34 | Bridge-R.R.-Sec.36 | | Sand-bed with | Mostly stable | Thin woody | Cultivated | OK | |
| | | 0764 | 0:08:57 | Bridge-Hwy 51 | | point bar | | buffer | | | |
| | | 0774 | 0:09:35 | Bridge-I 55 | | Moderate sinuosity | | | | | |
| | | 0847 | 0:10:02 | Bridge-Pvd.Rd.Sec.27&28 | | Can't Tell | | | | | |
| | | 0847 | 0:10:02 | End Hurricane Cr. | | | | | | | |
| 6-16 | Mussacuna Creek | 1142 | 0:14:06 | Mouth Mussacuna Cr. on | | | | | | | |
| | | | | Coldwater River | | | | | | | |
| | | 1321 | 0:16:42 | Bridge-Pvd.Rd.Sec.26 & 27 | | Can't Tell | | | | | |
| | | 1400 | 0:17:53 | Bridge-Grvl.Rd.Sec.25&26 | | | | | | | |
| | | 1424 | 0:18:15 | End Mussacuna Cr. | | | | | | | |
| 6-16 | Cane Creek | 1482 | 0:19:09 | Mouth Cane Creek-on | | Sand-bed | Appears mostly | Woody & shrubs | Mainly cultivated | Can't Tell | |
| | | | | Coldwater River | | Moderately | stable | | | | |
| | | 1582 | 0:20:43 | Crossing-Sec.3&4 | | sinuous | | | | | |
| | | 1615 | 0:21:14 | Bridge-Pvd.Rd.Sec.34&35 | | | | | | | |
| | | 1698 | 0:22:34 | Hwy. 51 | | | | | | | |
| | | 1698 | 0:22:34 | End Cane Cr. | | | | | | | |
| 6-16 | Secret Creek | 1740 | 0:23:15 | Mouth Secret Cr.-on Cane Cr. | | Bed obscured | Mostly stable | Shrubs & grass | Cultivated | OK | |
| | | 1803 | 0:18:03 | Bridge-Pvd.Rd.Sec.10&11 | | Straight | some local | | | | |
| | | 1803 | 0:24:17 | End Secret | | | failures | | | | |
| 6-16 | Wolf Creek | 0097 | 0:01:05 | Mouth Wolf Cr.-on | | Can't Tell | | | | | |
| | | | | Hurricane Cr. | | | | | | | |
| | | 0161 | 0:01:46 | Bridge-Hwy 301 | | | | | | | |
| | | 0333 | 0:03:41 | End Wolf Cr. | | | | | | | |
| 6-16 | Panther Creek | 0890 | 0:10:36 | Mouth Panther Cr.-on | | Can't Tell | | | | | |
| | | | | Hurricane Cr. | | | | | | | |
| | | 0944 | 0:11:20 | Bridge-Hwy 304 | | | | | | | |
| | | 1102 | 0:13:32 | Bridge-Pvd.Rd.Sec.2&3 | | | | | | | |
| | | 1102 | 0:13:32 | End Panther Cr. | | | | | | | |

## Table 34 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6-16 | Coldwater River | 1818 | 0:24:33 | Mouth Coldwater on Tallahatchie River | | Bed obscured Meandering | Mostly stable | Woody & shrubs | Swamp | OK | No app. problem |
| | | 1876 | 0:25:31 | Bridge-Hwy 51 | | | | | | | |
| | | 1949 | 0:26:46 | Bridge-R.R. | | As Above | | | | | |
| | | 1949 | 0:26:46 | Bridge-I 55 | | | | | | | |
| | | 2128 | 0:29:53 | Channel jumped out | | Bed obscured | Steep but stable | Woody | Woodland | Can't Tell | |
| | | 2183 | 0:30:52 | Short Cr.-Right | | Dunes visible | | | | | |
| | | 2220 | 0:31:32 | Bridge-Pvd.Rd.Sec.30 | | sinuous | | | | | |
| | | 2347 | 0:33:32 | Bridge-Hwy 305 | | Sand bed w/dunes | Mostly stable | Woody | Woody | Can't Tell | |
| | | 2433 | 0:35:28 | Bridge-Hwy 304 | | Meandering | | | | | |
| | | 2562 | 0:37:55 | Bridge-Hwy 78 | | | | | | | |
| | | 2562 | 0:37:55 | Bridge-Old Hwy 78 & R.R. | | As Above | | | | | |
| | | 2668 | 0:39:47 | Grvl.Rd.Sec.10 | | | | | | | |
| | | 2676 | 0:40:09 | Pipeline | | | | | | | |
| | | 2719 | 0:40:59 | Byhalia Rd. | | | | | | | |
| | | 2745 | 0:41:31 | Pipeline | | | | | | | |
| | | 2869 | 0:44:01 | Bridge-Victoria Rd.Sec.15&16 | | | | | | | |
| | | | | Tributary North of Coldwater | | | | Can't Tell - Obscured by Veg & High Flow | | | |
| | | 2995 | 0:46:36 | Bridge-Pvd.Rd.Sec.19&24 | | | | | | | |
| | | 3019 | 0:47:07 | Power Line | | | | | | | |
| | | 3165 | 0:50:11 | Bridge-Hwy 311 | | | | | | | |
| | | 3187 | 0:50:41 | Bridge-Hwy 311 | | | | | | | |
| | | 3307 | 0:53:17 | Bridge-Hwy 7 | | | | | | | |
| | | 3307 | 0:53:17 | End Coldwater River | | | | | | | |
| 616 | Holly Springs Experiment Station | 3308 | 0:53:18 | Start flight over station | | N/A | | | | | |
| | | 3377 | 0:54:49 | End flight over station | | | | | | | |
| 6-16 | Beartail Creek | 3417 | 0:55:42 | Mouth Beartail Cr. on Coldwater River | | | | | | | |
| | | 3467 | 0:56:50 | Bridge-Pvd.Rd.Sec.15 | | | | | | | |
| | | 3531 | 0:58:16 | Bridge-Pvd.Rd.Sec.19&24 | | | | | | | |
| | | 3555 | 0:58:49 | Buttermilk Cr.-Left | | | | | | | |
| | | 3567 | 0:59:05 | Little Beartail-Left | | | Can't Tell Much Detail | | | | |
| | | 3591 | 0:59:39 | Bridge-Pvd.Rd.Sec.16 | | | | | | | |
| | | 3619 | 1:00:14 | Bridge-Grvl.Rd.Sec.10&15 | | | | | | | |
| | | 3632 | 1:00:35 | Bridge-Hwy 305 | | | | | | | |
| | | 3643 | 1:00:50 | Pipeline | | | | | | | |
| | | 3662 | 1:01:16 | Bridge-Grvl.Rd.Sec.11 &12 and construction | | | | | | | |
| | | 3670 | 1:01:28 | End Beartail Cr. | | | | | | | |

# Table 34 (Continued)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet)/ | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6-16 | Grays Creek | 3741 | 1:03:08 | Mouth Grays Cr. on Coldwater River | | Sand-bed Point bars sinuous | Mostly stable | Shrubs & woody | Mainly cultivated | OK | |
| | | 3764 | 1:03:39 | Bridge-Pvd.Rd.Sec.34&35 | | | | | | | |
| | | 3801 | 1:04:32 | Hwy. 55 on tributary | | | | | | | |
| | | 3801 | 1:04:32 | End Grays Cr. | | | | | | | |
| 6-16 | Short Fork Creek | 3809 | 1:04:42 | Mouth of Short Fork Cr. | | Sand-bed Low sinuosity | Steep banks some failures | Shrubs & grasses | Mainly cultivated | OK | Tending to unstable |
| | | 3829 | 1:05:11 | Bridge on Pvd.Rd.Sec.25 | | | | | | | |
| | | 3852 | 1:05:45 | Bridge on Pvd.Rd.Sec.23 | | | | | | | |
| | | 3901 | 1:06:55 | Bridge on Pvd.Rd.Sec. 9 & 10 | | | | | | | |
| | | 3901 | 1:06:55 | End of Short Fork Cr. | | | | | | | |
| 6-16 | Camp Creek | 3923 | 1:07:26 | Mouth Camp Cr. on Coldwater River | | Sand bed w/dunes & pt. bars | Steep but mostly stable some outer bank erosion | Woody | Woodland | OK | |
| | | 3973 | 1:08:39 | Bridge-Pvd.Rd.Sec.25 | | | | | | | |
| | | 4055 | 1:10:39 | Bridge-Hwy 304 | | | | | | | |
| | | 4133 | 1:12:35 | Bridge-Pvd.Rd.Sec.7&8 | | Sand-bed w/point bars | Steep some unstable | Narrow woody buffer | Mainly cultivated | OK | Tending to unstable |
| | | 4164 | 1:13:20 | Hole Hoe Cr.-Right | | | | | | | |
| | | 4169 | 1:13:29 | Bridge-Grvl.Rd.Sec.6 | | | | | | | |
| | | 4221 | 0:14:47 | Ford-Sec.32 | | Sand-bed w/point bars Meandering | Steep, unstable banks | Shrubs & trees | Cultivated some woods | OK | Tending to unstable |
| | | 4237 | 0:15:11 | Culvert-Hwy 78 | | | | | | | |
| | | 4241 | 0:15:16 | Bridge-Hwy 305 | | | | | | | |
| | | 4261 | 0:15:47 | Bridge-Hwy 78 & R.R. | | | | | | | |
| | | 4270 | 1:16:01 | End Camp Cr. | | | | | | | |
| 61-16 | Pigeon Roost Cr. | 4320 | 1:17:18 | Mouth Pigeon Roost Cr. on Coldwater River | | Sand-bed w/point bars & dunes Straight | Steep, mostly stable some failures | Shrubs | Cultivated | OK | Mostly stable - tending to meander |
| | | 4347 | 1:17:58 | Bridge-Hwy 305 | | | | | | | |
| | | 4371 | 1:18:35 | Byhalia Cr.-Right | | | | | | | |
| | | 4392 | 1:19:08 | Bridge-Pvd.Rd.Sec.13 | | | | | | | |
| | | 4395 | 1:19:12 | Red Banks Cr.-Right | | | | | | | |
| | | 4478 | 1:21:22 | Bridge-Grvl.Rd.Sec.5&32 | | As Above | | | | | |
| | | 4551 | 1:23:16 | Bridge-Grvl.Rd.Sec.2 | | | | | | | |
| | | 4581 | 1:24:03 | Bridge-Hwy 309 | | As Above | | | | | |
| | | 4633 | 1:25:27 | Cuffawa Cr.-Left | | | | | | | |
| | | 4702 | 1:27:17 | Bridge-Pvd.Rd.Sec.11&12 (P.R.Sta.#17 Red Banks/ Marianna Rd.) | | Sand-bed w/point bars Sinuous | Steep unstable | Narrow woody buffer | Cultivated | OK | Tending to unstable |
| | | 4723 | 1:27:51 | Bridge-Grvl.Rd.Sec.7&12 (Mt. Moriah Church Rd.) | | | | | | | |
| | | 4766 | 1:29:00 | Bridge-Pvd.Rd.Sec.17 | | | | | | | |
| | | 4869 | 1:31:50 | Hwy 78 | | Can't Tell | | | | | |
| | | 4869 | 1:31:50 | End Pigeon Roost Cr. (on Jones Creek) | | | | | | | |

## Table 34 (Concluded)

| Date | Stream Name | Footage Start | Tape VCR Time | Description/Location | ARS Range (Feet) | Bed | Bank | Vegetation | Flood Plain Land Use | Condition of Structural Elements | Notes/Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 6-16 | Cuffawa Creek | 4900 | 1:32:41 | Mouth Cuffawa Cr. on Pigeon Roost Cr. | | Sand-bed Extensive point bars | Steep eroding | Narrow woody buffer | Cultivated | OK at present | Tending to unstable |
| | | 4933 | 1:33:34 | Bridge-Grvl.Rd.Sec.9&16 | | | | | | | |
| | | 4969 | 1:34:35 | Bridge-Grvl.Rd.Sec.21&28 (P.R.Sta.#32) | | Sinuous<br>As Above | | | | | |
| | | 4986 | 1:35:03 | Ford-Sec.28 | | | | | | | |
| | | 5054 | 1:36:57 | Bridge-Hwy 4 @ Chulahoma (P.R.Sta.#35) | | Sand-bed Extensive bars | Steep eroding | Narrow woody buffer | Cultivated | OK | Tending to unstable |
| | | 5078 | 1:37:38 | Bridge-Grvl.Rd. to Laws Hill | | Meandering | | | | | |
| | | 5161 | 1:40:00 | Bridge-Grvl.Rd.Sec.13&14 | | Can't Tell | | | | | |
| | | 5182 | 1:40:36 | End Cuffawa Cr. | | | | | | | |
| 6-16 | Byhalia Creek | 5189 | 1:40:47 | Mouth Byhalia Cr. on Pigeon Roost Cr. | | Sand-bed w/small point bars | Mostly stable | Shrubs & trees | Cultivated some wood | OK | No apparent problem |
| | | 5215 | 1:41:32 | Structure | | Low sinuosity | | | | | |
| | | 5227 | 1:41:53 | Bridge-Pvd.Rd.Sec.7 | | Sand-bed | Steep unstable | Shrubs | Cultivated | OK | Tending to unstable |
| | | 5265 | 1:42:58 | Bridge-Grvl.Rd.Sec.9 | | Sinuous | | | | | |
| | | 5316 | 1:44:27 | Bridge-Hwy 309 | | | | | | | |
| | | 5331 | 1:44:52 | Hwy 78 | | As Above | | | | | |
| | | 5331 | 1:44:52 | End Byhalia Cr. | | | | | | | |
| 6-16 | Red Banks Creek | 5345 | 1:45:18 | Mouth Red Banks Cr. on Pigeon Roost Cr. | | Can't Tell | | | | | |
| | | 5382 | 1:46:22 | Bridge-Pvd.Rd.Sec.29&30 | | | | | | | |
| | | 5466 | 1:48:50 | Bridge-Hwy 309 | | Sand-bed w/point bars Incised | Steep unstable | Almost absent | Cultivated | OK at present | Tending to unstable |
| | | 5546 | 1:51:13 | Bridge-Grvl.Rd.Sec.21&22 | | As Above | | | | | |
| | | 5612 | 1:53:12 | Bridge-Pvd.Rd.Sec.13&18 (intersects Hwy 78) | | Sand-bed Extensive bars | Mostly unstable | Woody | Cultivated | OK | |
| | | 5630 | 1:53:44 | End Red Banks Cr. | | Sinuous | | | | | |

# REPORT DOCUMENTATION PAGE

*Form Approved*
*OMB No. 0704-0188*

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

**1. AGENCY USE ONLY** *(Leave blank)*

**2. REPORT DATE**
May 1995

**3. REPORT TYPE AND DATES COVERED**
Final report

**4. TITLE AND SUBTITLE**
Demonstration Erosion Control Project Monitoring Program, Fiscal Year 1993 Report; Volume I: Main Text

**5. FUNDING NUMBERS**

**6. AUTHOR(S)**
Nolan K. Raphelt, William A. Thomas, Bobby J. Brown, David D. Abraham, David L. Derrick, Billy E. Johnson, Brenda L. Martin, Lisa C. Hubbard, Michael J. Trawle, Chester C. Watson, Steve R. Abt, Colin R. Thorne

**7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)**
See reverse

**8. PERFORMING ORGANIZATION REPORT NUMBER**
Technical Report HL-94-1

**9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)**
U. S. Army Engineer District, Vicksburg
3550 I-20 Frontage Road
Vicksburg, Mississippi 39180-5191

**10. SPONSORING/MONITORING AGENCY REPORT NUMBER**

**11. SUPPLEMENTARY NOTES**
Appendixes A-E were published under separate cover. Copies of this report and the Appendixes are available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

**12a. DISTRIBUTION/AVAILABILITY STATEMENT**
Approved for public release; distribution is unlimited.

**12b. DISTRIBUTION CODE**

**13. ABSTRACT** *(Maximum 200 words)*

The purpose of monitoring the Demonstration Erosion Control (DEC) Project is to evaluate and document watershed response to the implemented DEC Project. Documentation of watershed responses to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds. The monitoring program includes 11 technical areas: stream gauging, data collection and data management, hydraulic performance of structures, channel response, hydrology, upland watersheds, reservoir sedimentation, environmental aspects, bank stability, design tools, and technology transfer.

This report includes detailed discussion of the eight technical areas that were investigated by the U. S. Army Engineer Waterways Experiment Station during Fiscal Year 1993, i.e., all of these areas except upland watersheds, reservoir sedimentation, and environmental aspects.

In the area of data collection and data management, in[illegible]llation of continuous stage gauge instrumentation at 33 sites and crest gauges at an additional 42 sites was completed and data collection initiated. The initial development of the engineering database on Intergraph workstations was completed and made available to the U.S. Army Engineer District, Vicksburg, for testing.

(Continued)

**14. SUBJECT TERMS**

| | |
|---|---|
| Channel degradation | Hydraulic data collection |
| Engineering database | Hydrologic modeling |
| Erosion control | Sedimentation |

**15. NUMBER OF PAGES**
258

**16. PRICE CODE**

| 17. SECURITY CLASSIFICATION OF REPORT | 18. SECURITY CLASSIFICATION OF THIS PAGE | 19. SECURITY CLASSIFICATION OF ABSTRACT | 20. LIMITATION OF ABSTRACT |
|---|---|---|---|
| UNCLASSIFIED | UNCLASSIFIED | | |

**7. (Concluded).**

U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199

Civil Engineering Department
Engineering Research Center
Colorado State University
Fort Collins, Colorado 80523

University of Nottingham
Department of Geography
University Park
Nottingham, England NG7 2RD

**13. (Concluded).**

In the area of hydraulic performance of structures, a model study to determine the feasiblity of a low-drop structure using a 10-ft drop was conducted. Selected high- and low-drop structures were instrumented with stage gauges. The stage data will be used in calculating discharge coefficients for rating curves.

In the area of channel response, the first detailed topographic survey of the 20 long-term sites was completed. The initial broad-based geomorphic studies of 10 watersheds and detailed geomorphic studies of 3 watersheds were completed.

In the area of hydrology, development of HEC-1 hydrology models for 10 watersheds was initiated. The evaluation of the CASC2D hydrology model using the Goodwin Creek watershed was initiated.

In the area of bank stability, a model study to determine the applicability of the bendway weir concept for bank stabilization was conducted.

In the area of design tools, a riser pipe design system housed on the engineering database (Intergraph) was developed, tested, and made available for District use on the Coldwater River watershed.

In the area of technology transfer, a video report on the DEC Project was completed, and a second video report on channel degradation processes was initiated.

Destroy this report when no longer needed. Do not return it to the originator.