Man-Induced Channel Adjustment in Tennessee Streams

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PREFACE

This report is an attempt to relate the effects of channel modifications to the resulting instabilities in the fluvial system. The primary objective is to provide the Tennessee Department of Transportation with information concerning channel stability in relation to river crossing structures. Several new analytical techniques are presented that can aid in the understanding of channel adjustment to natural and man-induced stress. The methods of analyses presented herein should be applicable to other areas with alluvial, sand-bed channels, especially in the Gulf Coastal Plain States.

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CONVERSION FACTORS

For readers who may prefer to use the International System of Units (SI,) rather than the inch-pound units used herein, the conversion factors are listed below:

Inch-pound	Multiply by	To obtain
foot per second (ft/s)	3.048×10^{-1}	meter per second (m/s)
cubic foot per second (ft^3/s)	2.832×10^{-2}	cubic meter per second (m^3/s)
foot (ft)	3.048×10^{-1}	meter (m)
square foot (ft^2)	9.29×10^{-2}	square meter (m^2)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)

National Geodetic Vertical Datum of 1929 (NGVD of 1929): A geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called "mean sea level".

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ABSTRACT

Channel modifications in Tennessee, particularly in the western part, have led to large-scale instabilities in the channelized rivers and may have contributed to several bridge failures. These modifications, together with land-use practices, led to downcutting, headward erosion, downstream aggradation, accelerated scour, and bank instabilities. Changes in gradient by channel straightening caused more severe channel response than did dredging or clearing.

Large-scale changes continue to occur in all the channelized rivers: the Obion River, its forks, and the South Fork Forked Deer River. However, the non-channelized Hatchie River in west Tennessee not only withstood the natural stresses imposed by the wet years of 1973 to 1975 but continues to exhibit characteristics of stability.

Water-surface slope, the primary dependent variable, proved to be a sensitive and descriptive parameter useful in determining channel adjustment. Adjustments to man-induced increases in channel-slope are described by inverse exponential functions of the basic form $S=ae^{-b(t)}$; where "S" is some function describing channel-slope, "t" is the number of years since completion of channel work, and "a" and "b" are coefficients.

Response times for the attainment of "equilibrium" channel slopes are a function of the magnitude and extent of the imposed modification. The adjusted profile gradients attained by the streams following channelization are similar to the predisturbed profile gradients, where no alteration to channel length was made. Where the channels were straightened by constructing cut-offs, slope adjustments (reduction) proceeded downstream and upstream imposing new profiles with lower gradients.

INTRODUCTION

Man-induced hydraulic and morphologic changes to stream channels in Tennessee may have contributed to four bridge failures between June 1, 1973, and May 31, 1974. As a result of the bridge failures, an investigation of stream channels at bridge crossings was initiated on July 1, 1974, by the U.S. Geological Survey in cooperation with the Tennessee Department of Transportation, Bureau of Planning and Development, Division of Structures.

The purposes of the investigation, as originally proposed, were to (1) evaluate problems of local scour at bridges as well as general scour in the vicinity of bridge crossings, (2) improve estimations of potential scour, and (3) develop a system of site inspection to help forewarn of possible bridge failures due to scour. Initially, local scour was thought to be the major cause of bridge failures. However, as site specific data were gathered, it was obvious that these data were inadequate to determine the causes of bridge failures. The investigation was therefore shifted to include the hydrology and morphology of entire stream systems.

This report focuses on the effects of man-made channel modifications to the river mechanics, fluvial processes, and morphology of streams in western Tennessee. These effects are described for several stream reaches where bridge failures have occurred or where structural problems may occur. In addition, this report provides examples which demonstrate the application of existing streamflow records to the analysis of stream-channel changes with time.

The project area is statewide (plate 1). However, the majority of stream channel problems exist in the western one-third of the State which is characterized by erodible, unconsolidated, alluvial channel material. The most extensive data are available for the Forked Deer and Obion River basins in western Tennessee where two of the bridge failures have occurred.

Five rivers (three river basins) in west Tennessee with varying amounts and types of channel work were included in a study of channel adjustment to channel modification. Table 1 lists these rivers by reach and the types and dates of channel modification. Stage trends at selected sites were used to identify channel aggradation and degradation, and stream channel cross sections were used to identify change in cross section geometry.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the cooperation of the Corps of Engineers, Memphis District, in furnishing dredging plan sheets, stream channel cross sections, and streamflow data for 13 gaging stations in western Tennessee. The Tennessee Department of Transportation, Bureau of Planning and Development, Division of Structures generously furnished stream channel cross sections, bridge design data, and aerial photographs. The cooperation of the U.S. Department of Agriculture, Soil Conservation Service, in furnishing soil erosion data and tributary channel profiles is gratefully acknowledged. The authors express their appreciation to Lloyd E. Peterson for his assistance in data acquisition. Mr. Peterson was formerly employed with the Geological Survey.

Table 1.--Channel modification in west Tennessee studied in this report

River	Limits of channel modification Mile to mile	Type of modification	Date Begin	Date End
Obion River basin				
Main Stem	7.7 - 11.8*	Channel enlargement**	12-28-74	2- 1-77
Do.....	11.8 - 27.1*	.........do.........	7-12-77	11- 3-79
Do.....	27.1 - 35.3*	.........do.........	1959	1960
Do.....	35.3 - 41.6*	.........do.........	2-08-61	6-27-62
Do.....	41.6 - 46.1*	.........do.........	7-29-61	12-03-62
Do.....	46.1 - 50.8*	.........do.........	5-16-62	6-27-63
Do.....	50.8 - 55.7*	.........do.........	8-22-62	2-04-64
Do.....	55.7 - 60.5*	.........do.........	2-19-63	9-18-64
Do.....	60.5 - 65.5*	.........do.........	7-01-64	11-18-65
Do.....	65.5 - 69.9*	.........do.........	8-18-64	10-10-66
North Fork	0.0 - 10.9	Dredging and clearing	7-12-65	1-16-67
South Fork	0.0 - 5.2	.........do.........	9-09-65	10-10-67
Do......	5.2 - 6.2	.........do.........	Oct. 68	2-07-69
Do......	19.0 - 25.0	Snagging and clearing	1978	1979
Forked Deer River basin				
South Fork	0.0 - 4.4*	Channel enlargement	7-26-68	7-02-69
Do......	above 4.4*	Snagging and clearing	1973	-
Hatchie River basin				
Main Stem	130 - 190	Channel snagging	1945	1952

* Redefined river mile designation for modified channel
** Channel enlargement refers to straightening, dredging, and clearing

FACTORS AFFECTING CHANNEL STABILITY

Stream channels tend to be among the most actively changing of all geomorphic forms. An alluvial river channel, for example, is continually changing its position and shape as a response to varying hydrologic conditions and to hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental conditions or from man's activities.

All rivers are governed by the same forces, yet it is the relative significance of each of these variables that makes each channel unique. Awareness of the following interrelated factors and characteristics is essential to understanding these forces: (1) geologic factors, including structure, resistance to erosion, and soil characteristics; (2) hydrologic factors, including flow variance, runoff characteristics, sediment load, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream channel, and (4) hydraulic characteristics such as depths, slopes, channel roughness, and velocity distributions of streams.

According to the concept of channel equilibrium (Mackin, 1948), modifications made to a river channel will cause channel adjustments for some distance upstream and downstream depending on the magnitude of the change. Equilibrium implies an ability to adjust (respond) to changes in controlling variables (listed above) as well as a stability in form and profile. Stability is a trait of equilibrium and is distinguished from aggradation and degradation (the progressive raising or lowering of the channel bed). Scour and fill may occur within a channel as short-lived changes around a mean condition in response to varying hydrologic conditions, but the instability of gradation (aggradation or degradation) refers to changes over periods longer than a few years.

Two important questions related to equilibrium are:

1. What are the conditions in which an alluvial channel will be in equilibrium?

2. If an alluvial channel exhibits instability, how will it regain its equilibrium condition?

A channel is in equilibrium if it exhibits minimum rate of energy dissipation (loss) under the existing climatic, hydrologic, hydraulic, geologic, and man-made constraints (Yang, 1976). If for some reason the alluvial channel deviates from its minimum rate of energy dissipation, it will accordingly adjust its velocity, slope, roughness, geometry, and pattern of channel shifting so that energy dissipation can again be minimized (Yang and Song, 1979).

Lane (1955) describes the general relationship of the stream power concept, which can be related to minimum rate of energy dissipation, in the following expression:

$$QS \propto Q_s d_{50}, \qquad (1)$$

where:

Q = water discharge,
S = channel slope or energy gradient,
Q_s = sediment discharge, and
d_{50} = median particle size of bed material.

The discharge component (Q) can be expressed in a form of the continuity equation for streamflow which is:

$$Q = VA, \qquad (2)$$

where

Q = water discharge, in cubic feet per second,
V = mean stream velocity, in feet per second, and
A = cross-sectional area, in square feet.

By assuming that streamflow is uniformly distributed throughout the cross section and that cross-sectional area (A) is unchanging, QS can be divided by the product of the water-surface width and the depth of water covering a unit bed area yielding VS (stream power). This implies that an increase in V and/or S will result in a proportionate increase in Q_s or d_{50}. Furthermore, if the water in a stream at a given elevation represents the potential energy input to the stream at that point, and if potential energy decreases downstream due to the loss of elevation, then the expression VS represents the total rate of energy dissipation at a given cross-section.

Energy dissipation is caused by friction from roughness along the wetted perimeter (2 x depth + width) of the channel, by friction between flow lines within the current, and by transportation of sediment and debris. Rubey (1933) estimates that roughly 97 percent of the total energy losses within a stream can be accounted for by friction. Consequently, energy utilized for transportation is very small in comparison to that dissipated by friction. If total energy (VS) is constant, then relatively slight changes in channel characteristics which affect frictional losses cause very significant changes in transporting power and consequently, channel morphology (Mackin, 1948). Change inflicted on an alluvial channel by man may very well involve tampering with both the wetted perimeter (place of energy dissipation) and the stream gradient (stream power function) such that a long period of instability can be anticipated.

CHANNELIZATION

Stream channels are altered by snagging, clearing, dredging, widening, or straightening in connection with flood plain drainage problems or bridge construction. Generally a straightened channel shortens the flow line, thereby increasing gradient and flow capability within the channel. An increase in gradient leads to an increase in velocity and consequently more water can flow through the same cross-sectional area. Because gradient is directly proportional to the square of velocity, and discharge is directly proportional to and a function of velocity (eq. 2), then by equation (1) an increase in gradient (S) by channelization will lead to large changes in sediment discharge. The processes involved will be erosion locally and upstream of the change with subsequent deposition downstream. If significant lengths of the river are channelized (shortened, deepened, widened), there can be a noticeable decrease in the elevation of the water surface profile for a given discharge in the main channel. Tributaries emptying into the main channel in such reaches are significantly affected. A lower local bed level in the main channel for a given discharge means that the tributaries entering in that vicinity are subjected to a steepened gradient and thus an increase of stream power at that point. This in turn leads to headward erosion (degradation) in the tributaries.

An example of headward erosion in a tributary is Mud Creek, a tributary to the South Fork Obion River at river mile 2.0 in eastern Obion County (plate 1). Longitudinal bed profiles of Mud Creek from 1965-81 (obtained from Soil Conservation Service, Dresden, Tenn.) are shown in figure 1 and indicate progressive upstream degradation resulting from dredging in the South Fork Obion River. In some cases, sufficient degradation and lateral erosion can be induced to cause bridge failures on the tributary systems. As degradation occurs in the tributaries, bank instabilities are induced and the sediment loads are greatly increased (Culbertson and others, 1967).

With few exceptions, sites with gradation problems occur along most streams in the western one-third of Tennessee, an area of easily erodible soil and alluvium, and of generally high sediment yields (U.S. Department of Agriculture, 1980). Such streams are more likely to change course or shape than streams flowing in bedrock channels or channels with very large bed material which are characteristic of the eastern two-thirds of the State. Gradation, as used in this text, refers to the consistent vertical movement of the channel bed which takes place over long distances, establishing a trend, up or down, for more than several years at a time. Vertical movement of the channel bed within a reach after a single flow event will be either scour or fill, depending upon the direction of movement, and represents adjustments around some mean equilibrium condition (steady-state).

Gradation problems at river crossings require an intensive study of the river system as a whole. This is especially true in alluvial stream channels where a change at one point is propagated in different ways upstream and downstream. Alluvial channels also move laterally as well as vertically in response to changing environmental and man-made stresses. Rivers must be treated as dynamic systems because streambeds are in various states of equilibrium with the flowing water.

ANALYSES OF CHANNEL ADJUSTMENTS

METHOD OF STUDY

Site Inspection

Data were collected at 95 bridge crossings where channel problems were believed to exist (plate 1 and table 2). At each site, cross-sectional changes and stream-channel gradation were documented and photographs were taken. At sites where bridge failures occurred, or where visible hydraulic and morphologic changes were occurring, additional data were collected, such as longitudinal stream-bed profiles, discharge and velocity distributions, channel geometry, and observations of flow patterns.

Streamflow Records

For selected streams, records from U.S. Army Corps of Engineers (USCE) and U.S. Geological Survey (USGS) stream gages were used to plot low-pool elevations and specific-gage elevations against time for each station (table 3). These plots provide records of gradation trends and gradation rates of the selected stream beds.

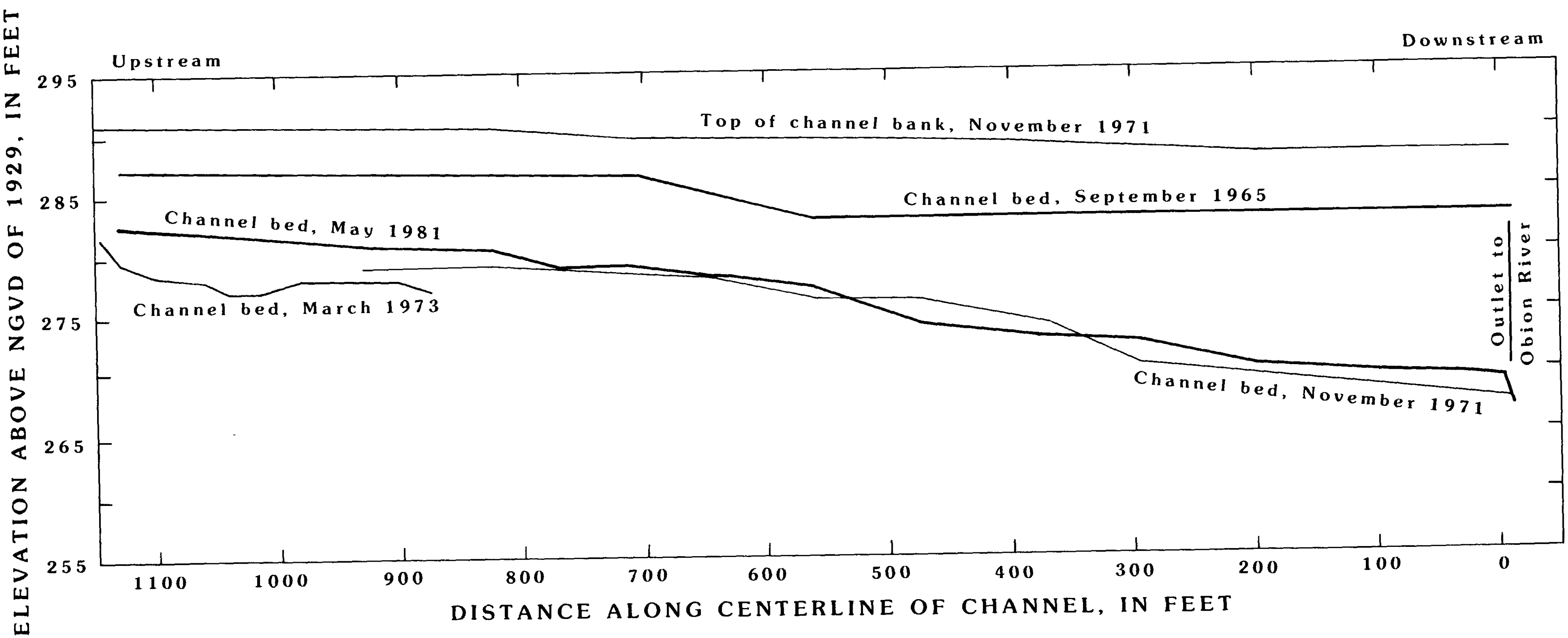


Figure 1.--Channel-bed profiles of Mud Creek, tributary to the Obion River, from 1965-81. (Modified from Weakley County Soil Conservation Service, Dresden, Tennessee.)

Table 2.--Summary of project investigation sites in Tennessee

[Highway Crossings: SR, State route; S, secondary; R, rural; M, Metro; I, Interstate]

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
1	Beaverdam Creek	SR-48	Hickman	22	9-31-74	Probable major flood, 12-72.	--------------------	Bed at pile cap level.	Poor entrance conditions.
2	Cane Creek	SR-87	Lauderdale	79.6	6-24-75	--------------	Enlargement and straightening, 1970.	Degradation and lateral channel movement.	Channel degradation.
3	Do ---------	SR-87A	Lauderdale	83.9	6-24-75	--------------	Enlargement and straightening, 1970.	Degradation and lateral channel movement.	Do.
4	Do ---------	Old SR-3	Lauderdale	26	10-10-74 2-27-75 6-15-76	--------------	Downstream enlargement and straightening, 1970.	Degradation and lateral channel movement.	Do.
5	Do ---------	New SR-3	Lauderdale	32	10-10-74 2-27-75 6-15-76	--------------	Enlargement and straightening, 1970.	Scour at downstream end of box culvert.	Downstream degradation.
6	Do ---------	SR-19	Lauderdale	33.9	10-10-74 2-27-75 6-15-76	--------------	Englargement and straightening, 1970.	Degradation and lateral channel movement.	Channel degradation.
7	Cub Creek	SR-125	Hardeman	14	10-31-74 7-17-75 6-15-76	Possible 50-year flood, 3-73.	Channel relocated at site 1940's, enlargement and straightening, 1970.	Bed approximately 1 foot above pile cap.	Channel modification.
8	Cypress Creek	SR-15	Fayette	36	10-31-74	Second highest stage in 18 years of record, 4-73.	Probable channel modification, 1920's.	Exposed pile caps	Channel degradation.
9	Edmundson Creek	SR-5	Gibson	9.5	6-25-75 2- 8-77 6-14-77 11- 1-78 1-16-80	--------------	Enlargement and straightening downstream, 1976.	Lateral movement near bridge.	---------------
10	Ferguson Branch	S-6330	Smith	2.2	12-18-75	--------------	Downstream channel modification.	Scour at left abutment.	Poor approach conditions, channel degradation.
11	Fletcher Creek Tributary.	SR-15	Shelby	-------	6-23-75	--------------	Channel modification	Channel degradation	---------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
12	Fulwood Creek	S-8085	McNairy	3.6	4- 6-76	---------------	--------------------	Maintenance problem	-----------------
13	Hatchie River	I-40	Haywood	1,870	2-27-75 3- 5-75	Largest since 1929, 3-73 10-year flood, 1-74.	--------------------	Scour at piers	Pier alinement.
14	Do ---------	SR-100	Hardeman	1,547	12-21-76	---------------	--------------------	Lateral movement to left.	Channel bend at bridge.
15	Hicks Creek	S-8206	Madison	2.9	10- 4-76	---------------	--------------------	Scour about 6 feet deep.	Poor approach conditions.
16	Horse Creek	S-8243	Hardin	104	3-28-74 7-17-75	Record flood, 3-73.	--------------------	No problem	-----------------
17	Do ---------	S-6194	Hardin	114	3-28-74 7-17-75	Record flood, 3-73.	--------------------	Scour at piers 3 and 4 in 1973.	High velocities, poor approach conditions.
18	Do ---------	SR-15	Hardin	-------	3-28-74 7-17-75	Record flood, 3-73.	--------------------	Scour at main bridge in 1973, and overflow bridge in 1975.	High velocities, eccentric openings.
19	Do ---------	SR-128	Hardin	-------	3-28-74 7-17-75	Record flood, 3-73.	--------------------	Scour at main bridge in 1973.	High velocities, eccentric openings.
20	Hyde Creek	Old SR-3	Lauderdale	7	10- 9-74	---------------	Enlargement and straightening, 1970.	Scour at pier nose, degradation, bank failure.	Channel modification.
21	Do ---------	New SR-3	Lauderdale	9.5	10- 9-74 2-27-75 6-15-76	---------------	Enlargement and straightening, 1970.	Possible head-cutting downstream from box culvert.	Do.
22	Johns Creek	S-8100	Shelby	21	12- 5-74	Possible 25-year flood.	Downstream lined channel.	Major degradation, vertical banks.	Downstream lined channel.
23	Lick Creek	M-2202	Gibson	2.1	2-25-76	Large flood, 2-76.	Small concrete lined channel upstream.	Scour at bridge and 43 feet upstream.	High velocities, poor entrance conditions.

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
24	Middle Fork Obion River.	SR-43	Weakley	275	10-30-74 7-16-75 12-18-75 4- 1-76 6-14-76 6-14-77 11- 1-78 1-15-80	5-year flood, 4-73.	Enlargement and straightening 1920's or 1930's, downstream modification 1969.	No scour at time of inspection. Previous bridge failure due to scour 1967, bridge replaced 1969.	-----------------
25	Mill Creek	SR-21	Obion	5.8	6-24-75	-------------	-------------------	Minor scour at left abutment.	-----------------
26	Mud Creek Tributary.	S-8014	Weakley	-------	1- 7-76	-------------	-------------------	Large scour hole downstream and under downstream cut-off wall.	Channel degradation downstream.
27	Nonconnah Creek	SR-3	Shelby	172	5-14-74 6-23-75 4- 7-76	-------------	Channel modified by urban development.	Scour at 2 main piers.	High velocities due to bridges blocked by landfill and drift.
28	Do ---------	Perkins Road (S-8180)	Shelby	114	4- 7-76	-------------	Channel modified by urban development.	Center piles failed due to scour.	Fill material mining downstream.
29	North Fork Mud Creek.	SR-22	McNairy	1.2	12-21-76	-------------	-------------------	No problem	-----------------
30	North Fork Mud Creek Tributary.	SR-22	McNairy	0.7	12-21-76	-------------	-------------------	----do---------	-----------------
31	North Indian Creek.	SR-81	Unicoi	63.5	7- 9-76 11-29-77 4-18-78	Flood, 11-6-77	-------------------	Head cutting	Gravel mining downstream.
32	Obion River	SR-78	Dyer	2,216	8-14-74	5-year flood 1972, 1973, 1974.	Enlargement and straightening, 1962.	Bank failure. No cross section recorded.	-----------------
33	Do----------	SR-20	Dyer	2,398	2-28-75	-------------	Enlargement and straightening 1930's, channel cleanout 1959.	No problem	-----------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
34	Obion River	S-8005	Dyer	2,000	8-14-75 2-28-75 4- 4-75 5-21-75 12-17-75 3-31-76 6-14-76 11-10-76 1-17-80	22 floods near or above bank-ful stage from 1-29-72 to 1-14-74.	Enlargement and straightening, 1963.	Bridge failure due to scour 1-27-74.	Chanel and flood plain modification lowered base level.
35	Do ---------	SR-5	Obion	1,736	6-24-75	-------------	-------------------	No problem	-----------------
36	Paul Short Creek	S-8165	Obion	-------	10- 5-76	-------------	Channel straightened downstream.	Scour at abutments.	Poor approach conditions.
37	Porters Creek	SR-125	Hardeman	40	10-31-74 7-17-75 6-15-76	Possible 50-year flood, 3-73.	Enlargement and straightening, 1972.	Bank failure, lateral movement.	Channel modification.
38	Rock Branch	S-8165	Obion	0.9	10- 5-76	-------------	-------------------	Scour at abutment	Poor approach condition, water falls upstream.
39	Rutherford Fork Obion River.	SR-1	Carroll	35	10-30-74 6-25-75 6-14-76	5-year flood, 4-73.	Channel modification 1920's or 1930's, downstream enlargement and straightening, 1967.	Scour 4 feet below pile cap.	Drift accumulation.
40	Sequatchie River	I-24	Marion	583	1-14-76	-------------	-------------------	No problem	-----------------
41	South Fork Forked Deer River.	S-8164	Chester	158	9-30-74 10-10-74 4- 3-75 5-20-75 6-16-75	Possible 50-year flood, 4-73, 3 foot higher 3-75.	Channel modification, 1920's.	Scour at pier, removal of riprap at left abutment.	Size of pier, abutment set in main channel.

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
42	South Fork Forked Deer River.	SR-3	Dyer	1,140	12-17-73 10-10-74 2-27-75 4- 4-75 4-29-75 5-21-75 12-17-75 3-30-76 6-15-76 10-30-78 1-17-80	-------------	Downstream enlargement and straightening 1969, channel cleanout 1974.	Lateral movement. Bridge failure due to scour 12-4-73.	Channel modification, drift accumulation.
43	Do ---------	I-40	Madison	645	6-23-75	Record flood, 3-75.	Channel modification, 1920's.	Scour at several bents.	-----------------
44	Do ---------	S-8058	Madison	278	10- 4-76	Record flood, 3-75	Channel modification, 1920's.	Scour at right pier and abutment.	Poor approach conditions.
45	Do ---------	S-8206	Madison	1,144	10- 4-76	Record flood, 3-75.	Channel modification, 1920's.	Left approach fill washed out 3-75.	Embankment overtopped.
46	South Fork Forked Deer River Tributary.	S-8131	Haywood	-------	10- 4-76	Record flood on South Fork Forked Deer River, 3-75.	-------------------	Scour 10 feet deep	High head due to river flow.
47	Do ---------	S-8171	Crockett	1.9	10- 4-76	-------------	Channel enlarged and straightened downstream.	Scour at abutments	Poor approach conditions.
48	South Fork Mud Creek.	SR-22	McNairy	3.8	12-21-76	-------------	-------------------	No problem	-----------------
49	Tom's Creek	S-6243	Perry	17.7	3- 5-75 4- 3-75	Residents say largest flood known, 3-75.	-------------------	Piers with spread footings sank 7 feet 3-13-75.	Mobile bed material, drift.
50	Tommy Creek	S-8011	Weakley	3.8	10- 5-76	-------------	Channel modification during bridge construction.	--------------	-----------------
51	Town Creek	SR-15	Fayette	2.5	12- 6-74 6-15-76	-------------	-------------------	Exposed pile caps	Lowered base level.
52	Wolf River	S-8208	Shelby	709	10- 6-76	-------------	-------------------	No problem	-----------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
53	Wolf River	SR-57	Fayette	306	6-15-76	--------------	--------------------	No problem	-----------------
54	Bethel Creek	SR-104	Dyer	17.4	6-15-77 11- 2-78 1-16-80	--------------	--------------------	-----do--------	-----------------
55	Black Creek Tributary.	SR-88	Crockett	-------	6-24-75	--------------	--------------------	Bank failure and scour at left bent.	-----------------
56	Buck Creek	S-8031	Gibson	48.4	6-15-77 11- 2-78 1-16-80	--------------	Channel modification 1930's or 1940's.	No problem	-----------------
57	Doakville Creek	Old SR-104	Dyer	27.6	6-15-77	--------------	--------------------	-----do--------	-----------------
58	Do --------	SR-104	Dyer	27.6	6-15-77 11- 2-78 1-16-80	--------------	--------------------	Channel degrading	-----------------
59	Grove Creek	SR-22	Obion	-------	6-13-77 11- 1-78 1-16-80	--------------	--------------------	No problem	-----------------
60	Kail Creek	R-9341	Haywood	-------	2-10-77 6-17-77 10-20-78 11- 3-78 1-18-80	--------------	Enlargement and straightening 1960's or 1970's.	-----do--------	
61	Lewis Creek	SR-104	Dyer	29.2	6-15-77 10-31-78 1-16-80	--------------	Clearing and snagging, 1978.	Left upstream bank sloughing.	-----------------
62	Locust Grove Creek.	SR-105	Gibson	4.9	2- 9-77 6-14-77 11- 1-78 1-15-80	--------------	Clearing and banks smoothed, 1976.	Scour at right abutment.	Channel modification and degradation.
63	Lost Creek Tributary.	S-8127	Haywood	7.5	2-10-77 6-16-77 11- 3-78 1-17-80	--------------	--------------------	No problem	-----------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
64	Middle Fork Obion River	S-8250	Weakley	310	2- 9-77 6-14-77 11- 1-78 1-15-80	-------------	Channel dredged 1920's and banks cleared 1978.	No problem	-----------------
65	Miller Creek	County Road	Dyer	-------	6-15-77 11- 2-78 1-16-80	-------------	Channel dredged and straightened in 1930's or 1940's.	-----do--------	-----------------
66	Mud Creek	SR-77	Gibson	10.7	2- 9-77 6-14-77 11- 1-78 1-16-80	-------------	Upstream left bank cleared and smoothed 1977. Channel dredged in 1930's or 1940's.	Scour at right abutment.	Previous channel modification.
67	Mud Creek	SR-76	Haywood	25.0	2-11-77 6-17-77 11- 3-78 1-18-80	-------------	Dredged and straightened in 1930's.	No problem	-----------------
68	Nixon Creek	S-8127	Haywood	105.5	2-10-77 6-16-77 11- 3-78 1-17-80	-------------	Dredged and straightened in 1920's or 1930's. Banks cleared 1978.	-----do--------	-----------------
69	North Fork Forked Deer River.	SR-104	Gibson	202	6-15-77 11- 2-78 1-16-80	-------------	Dredged and straightened 1955. Cleared and snagged 1977.	Scour near left abutment, and bank sloughing.	Constriction of valley.
70	Do -------	S-8029	Gibson	176	6-15-77 11- 2-78 1-16-80	-------------	Enlargement and straightening 1955. Banks cleared 1978.	Bank sloughing	-----------------
71	Do -------	Old SR-104	Gibson	213	6-15-77 11- 2-78 1-16-80	-------------	Cleared and snagged 1977.	No problem	-----------------
72	North Fork Obion River.	SR-22	Obion	480	6-13-77 11- 1-78 1-15-80	-------------	Enlargement and straightening, 1967.	Large scour hole below bridge, moving upstream.	Channel modification and degradation. Bridge is now a major constriction to flow.
73	Do -------	SR-43	Weakley	372	6-14-77 11- 1-78 1-15-80	-------------	Enlarged and straightened 1920's or 1930's.	Scour holes near each abutment.	Constriction of flood plain.

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
74	Owl Creek	SR-142	Hardin	33.2	3- 7-77 3-15-77	-------------	Enlarged and straightened downstream 1950's or 1960's. Large flood 3-4-77.	---------------	Bridge failure due to drift.
75	Pond Creek	R-9442	Dyer	69.6	6-16-77 11- 3-78 1-16-80	-------------	Dredged and straightened in 1930's or 1940's. Cleared and snagged 1977.	No problem	-----------------
76	Do--------	SR-20	Dyer	65.6	6-16-77 10-31-78 1-16-80	-------------	Dredged and straightened in 1930's or 1940's. Cleared and snagged 1977.	-----do--------	-----------------
77	Do--------	R-9778	Dyer	-------	6-16-77 11- 2-78 1-17-80	-------------	Enlarged and straightened 1930's or 1940's. Cleared and snagged 1978.	Degradation and bank widening.	Channel clearing.
78	Do--------	County Road	Dyer	-------	6-16-77 11- 2-78 1-17-80	-------------	Enlarged and straightened 1930's or 1940's. Cleared and snagged 1978.	Bank erosion at upstream right bank.	Bridge failure due to bank widening behind right abutment.
79	Do--------	R-9201	Crockett	-------	6-16-77 11- 2-78 1-17-80	-------------	Enlarged and straightened 1930's or 1940's. Cleared and snagged 1978.	Bank sloughing upstream and downstream of bridge.	-----------------
80	Rutherford Fork Obion River.	S-8179	Gibson	217	2- 7-77 1-15-80	-------------	Channel modification 1920's or 1930's. Cleared and snagged downstream 1976.	No problem	-----------------
81	Do--------	SR-54	Gibson	201	2- 7-77 11- 1-78 1-15-80	-------------	Modified 1920's or 1930's, right bank cleared 1978.	-----do--------	-----------------
82	Do--------	SR-105	Gibson	238	2- 8-77 11- 1-78 1-16-80	-------------	Cleared and snagged, 1974.	Right upstream bank sloughing, left bank has widened about 10 feet.	-----------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
83	Rutherford Fork Obion River Tributary.	SR-5	Gibson	2.2	2- 8-77 6-14-77 11- 1-78 1-16-80	-------------	Upstream channel modified, 1940's.	Upstream right bank erosion.	Bridge alinement.
84	South Fork Forked Deer River.	SR-88	Crockett Lauderdale	932	2-10-77 6-16-77 10-20-78 1-17-80	-------------	Enlarged and straightened 1920's. Cleared and snagged upstream 1976.	Scour at left abutment and on top of left bank.	Channel modification.
85	Do--------	SR-54	Haywood	-------	2-10-77 6-17-77 10-20-78 11- 3-78 1-18-80	-------------	Enlarged and straightened 1920's. Banks and channel cleared 1978.	Scour at right abutment.	Constriction of channel from old abutments and drift.
86	Do--------	SR-76	Haywood	687	2-11-77 6-17-77 11- 3-78 1-18-80	-------------	Enlarged and straightened, 1920's.	Sloughing of material from both banks.	-----------------
87	South Fork Obion River.	S-8014	Obion	760	2- 8-77 6-14-77 11-11-78 1-16-80	-------------	Dredged, 1967.	No problem	-----------------
88	Do--------	S-8030	Gibson	-------	6-14-77 11- 1-78 1-15-80	-------------	Enlarged and straightened, 1920's or 1930's. Cleared and snagged, 1978.	Channel degrading	-----------------
89	Do--------	S-8008	Gibson	-------	2- 9-77 6-14-77 11- 1-78 1-16-80	-------------	Enlarged and straightened, 1920's and 1948.	-----do--------	-----------------
90	Stokes Creek	County Road	Dyer	-------	6-15-77 11- 2-78 2-16-80	-------------	Enlarged and straightened, 1940's.	No problem	-----------------
91	Do--------	County Road	Crockett	-------	6-15-77 11- 2-78 1-16-80	-------------	Enlarged and straightened, 1940's.	Slight degradation and widening.	-----------------
92	White Oak Creek	SR-22	McNairy	49.2	3-15-77	Large flood, 3-4-77.	Cleared, 1976	No problem	-----------------

Table 2.--Summary of project investigation sites in Tennessee--Continued

Site No.	Stream	Highway crossing	County	Drainage area (mi^2)	Date inspected	Floods	Channel modification	Condition	Possible factors influencing scour
93	White Oak Creek	County Road	McNairy	-------	3-15-77	Large flood, 3-4-77.	Cleared, 1976	Bridge failure due to scour.	High velocities.
94	South Fork Forked Deer River.	County Road	Lauderdale	-------	10-20-78 11- 2-78 1-17-80	--------------	Dredged and straightened in 1920's.	Degradation and bank widening.	Channel modification downstream.
95	Do--------	County Road	Lauderdale	-------	10-20-78 1-17-80	--------------	Dredged and straightened in 1920's.	Degradation and bank widening.	Channel modification downstream.

Table 3.--Gaging stations used in analyses

River and station	River mile of station before and after channel modification: Natural channel	Finished channel	Type of data	Agency[1]	Period of record
Obion:					
Menglewood	20.8	[2]29.0	Daily stage	USCE	1960 to present.
Bogota	36.7	[2]42.0	Continuous stage and discharge.	USCE	1939 to present.
Obion	62.4	60.7	-------do-------	USGS	1929 to present.
Rives	71.4	66.5	-------do-------	USCE	1939 to present.
North Fork Obion:					
Rives	--	5.9	Daily stage	USCE	1939-70
Union City	--	10.0	Continuous stage and discharge.	USGS	1929-70
Martin	--	18.0	-------do-------	USCE	1939 to present.
Palmersville	--	34.9	-------do-------	USCE	1969 to present.
South Fork Obion:					
Kenton	--	5.8	Daily stage and measured discharge.	USCE	1939 to present.
Greenfield	--	19.2	Continuous stage and discharge.	USGS	1929 to present.
McKenzie	--	34.4	-------do-------	USCE	1969 to present.
South Fork Forked Deer:					
Yellow Bluff	5.7	3.3	Daily stage	USCE	1939 to present.
Halls	10.6	7.9	Continuous stage and discharge.	USCE	1939 to present.
Gates	19.0	16.3	-------do-------	USCE	1969 to present.
Hatchie:					
Bolivar	135.1	--	Continuous stage and discharge.	USGS	1929 to present.
Pocahontas	178.0	--	-------do-------	USCE	1940 to present.
Walnut	185.0	--	-------do-------	USCE	1946 to present.

1 USCE, U.S. Army Corps of Engineers; USGS, U.S. Geological Survey.
2 Mouth of Obion River and river mile designation redefined by U.S. Army Corps of Engineers.

Low pool

"Low pool" in this report refers to the lowest water-surface elevation (gage-height) that was recorded at the gaging station during a selected water year (October 1 to September 30). The low-pool elevations do not represent a constant flow duration, frequency, or discharge. The low-pool elevation is climatically dependent and may vary annually. Low-pool elevations are used in this report to identify gradation trends and should not be used to determine gradation rates, or quantitative changes in channel-bed elevations.

Specific gage

"Specific gage" in this report refers to the water-surface elevation (gage-height) at which a selected discharge [for example, 1,000 ft^3/s] occurs. Specific gage was derived from stage-discharge relationships (rating curves) for all continuous stage and discharge stations on the Obion, Forked Deer, and Hatchie Rivers. Blench (1969) and Gessler (1971) concluded that specific gage at a single gaging station is not an accurate analytical tool because (1) changes in channel geometry may affect the water-surface elevation and (2) stage gives no indication about development of channel slope. However, for this study specific gage is considered acceptably accurate provided there are at least two gaging stations per channel reach and the lengths of the records are at least 10 years. This study uses specific gage to identify quantitative changes in channel geometry and to compute water-surface slope in a channel reach between two gaging stations.

To obtain an acceptably accurate specific-gage record, the rating curve must be defined through a large range of discharges. To use specific gage to identify quantitative changes in channel geometry, a gaging station network of sufficient density and length of record is necessary. In the case of the Obion, Forked Deer and Hatchie Rivers, 17 gaging stations were used; record lengths range from 12 years (1969-81) to 52 years (1929-81). Analyses of records for those stations identified short- and long-term trends for changes in channel-bed elevations and water-surface slopes.

The following criteria was used for selecting specific-gage elevations; (1) a high intermediate discharge was selected that occurs frequently during periods of steady streamflow (normally occurring between August and October), (2) the selected discharge water-surface elevation (specific gage) was lower than bankfull stage (the water-surface elevation attained by the stream when flowing at capacity and the stage above which banks are overflowed) and higher than baseflow stage (the water-surface elevation attained by the stream when the discharge entering the channel is from ground water or other delayed sources), (3) once a specific-gage elevation is selected for a particular channel cross section, it is used for all the gaging stations on that channel.

Specific-gage data were transferred from the continuous-record stations to the daily-stage-only stations using the following procedure. To estimate the specific gage at 1,000 ft^3/s at a daily-stage-only station downstream from a continuous-record station, periods of steady flow at 1,000 ft^3/s are identified at the continuous-record station for each year of interest. The identical periods of steady flow and corresponding stage at the downstream station are recorded. The stage for each of these periods is assumed to be the specific gage for 1,000 ft^3/s at the downstream station provided there is less than a 20 percent increase in drainage area and no major tributaries between the stations.

Water-surface Slope

Elevations of specific gage at various points along the stream were used to obtain values of water-surface slope. A water-surface slope was calculated for each channel reach by dividing the difference in specific-gage elevations by the distance between the gaging stations. The slope of the water surface and the gradient of the channel bed are assumed to be parallel. Therefore, water-surface slope and channel-bed gradient are used interchangeably in this report. Local discontinuities in the water surface probably occur through a large range of discharge; however, by using a high intermediate discharge when obtaining specific-gage elevations, it is assumed that these local discontinuities are minimized. Specific gage at a discharge rate of 500 ft^3/s was used for the Obion River and its forks, whereas 1,000 ft^3/s and 2,000 ft^3/s were used respectively for the South Fork Forked Deer and Hatchie Rivers. Utilizing specific-gage elevations to define channel-bed gradient and water-surface slope reduces the effects of site-specific fluvial processes and yearly climatic variations.

Although slope is one of the most difficult hydraulic variables to define (Andrews, 1979), it provides information on a channel reach as opposed to a specific point or cross section. Gradient or slope is a controlling variable to river adjustment and the primary dependent variable (Mackin, 1948). This implies that variations in other variables, such as sediment and water discharge, bedload sediment size, as well as other geologic, hydrologic and geometric parameters, are accounted for by changes in channel slope. In natural alluvial channels that are not being stressed, slope is independent of time, and a statistical relation does not exist. However, in highly stressed, significantly altered alluvial channels, slope is statistically related to time. The relation between slope and time is not linear, but it can be defined by exponential or log-normal regression.

This approach is beneficial when dealing with the propagation of an adjustment along the course of a channel. Furthermore, stream power (VS), as described by Yang (1976), utilizes slope as a dominant variable in describing an equilibrium condition. Analysis of the variation in water-surface slope with time offers insight for the analysis of fluvial adjustments. For this reason water-surface slope, and when possible, stream power (VS) were used as measures of channel adjustments.

METHOD OF ANALYSES

Depending on available data, four types of slope analyses were undertaken to depict channel adjustment. All analyses refer to water-surface slope, and are a function of time relative to the completion of channel work. Four equations were derived for the various types of slope analyses.

Slope Regression

The first, and probably the most basic, is an exponential regression between slope and time since completion of channel work which describes slope reduction following channelization. The equation is given below.

$$S = e^{-b(t)-a} \quad (3)$$

where:

S = water surface slope in channel reach in feet per foot x 10^{-4},
t = time in years since completion of channel work, and
a,b = regression coefficients.

Sum of Slopes

The second analysis is a measure of the gradual reduction in water-surface slope and is termed "sum of slopes" (ΣS). The equation is given below.

$$\sum_{1}^{n} S = S_1 + S_2 + \ldots S_n \quad (4)$$

where:

S = water surface slope in channel reach in feet per foot x 10^{-4}.

The water-surface slopes for several reaches within the same channel are summed by year, and the resultant sum of slopes is then plotted against time. Channel-response time (the time required for a channel to reach the designed or a stable gradient) is evaluated by comparing the resultant sum of slopes values with either the channel design gradient or a historical (adjusted) sum-of-slopes value. A plot of sum of slopes against time using an exponential regression shows the overall slope adjustment trend for the entire channel rather than an individual channel reach. The equation is given below.

$$\Sigma S = e^{-b(t)-a} \quad (4a)$$

where:

S = water surface slope in channel reach in feet per foot x 10^{-4} and
t = time in years since completion of channel work.

Unit Stream Power

The third analysis involves unit stream power (θ) as a measure of channel response over time. The equation for this analysis follows:

$$\theta = \frac{VS}{L} \quad (5)$$

where:

V = mean velocity at predetermined specific-gage elevation in feet per second,
S = water-surface slope between gaging stations at predetermined specific-gage elevation in feet per foot x 10^{-4}, and
L = length of channel reach in miles.

For any gaged site, instantaneous discharge is plotted against mean velocity for all discharge measurements made at the site in a given year. By regression analysis, a line of relation is determined for each year, yielding a velocity for any selected discharge. The selected discharge is the same discharge used for specific gage at the site. Unit stream power values were calculated by dividing the product of velocity and slope by reach length. These values (θ) were plotted against time in years. Plots of θ for each reach give insight into the magnitude of the fluvial processes (erosion or deposition) within the reach and the time required for the reach to adjust to the new hydraulic conditions.

Exponential Decay Function

A fourth analysis was obtained by plotting absolute values of water-surface slope change against time since the completion of channel alterations. See the following equation:

$$|\Delta S| = ae^{-b(t)} \qquad (6)$$

where:

$|\Delta S|$ = absolute value of change in water-surface slope in channel reach between two consecutive years,

t = time in years since completion of channel work, and

a,b = regression coefficients.

This analysis was used to estimate the magnitude of slope adjustment by year. The absolute value of slope change was used to avoid computations with negative numbers in attempting to define statistically a wave function. An idealized plot of this function with its associated equation is shown in figure 2.

Each peak in the wave is a maximum absolute value of slope change between successive years and represents the adjustment passing one of the measuring points in the reach. In the idealized plot, the amplitude of each successive peak decreases by an exponential decay function. The reasons are:

(1) As the transition slope moves away from the upstream end of the altered reach, its effect upstream or downstream is proportionately reduced;

(2) As the transition slope propagates upstream, its steepness and amount of change decreases due to the non-cohesive nature of the sediment; and

(3) The data indicate that following a disturbance, slope adjustment is rapid and that this rate of change slowly tends towards a minimum.

The peaks are a measure of time of adjustment movements and reflect the magnitude of disturbance. Therefore, the spacing of the peaks describe a velocity function (distance over time) for phases of adjustments (aggradation or degradation) along the channel between successive points. As shown in figure 2, their amplitude defines the maximum rate of slope change, whereas the wavelength measured at their bases is the number of years in which the reach is affected by a particular adjustment. The exponential decay function describes channel adjustment that may take hundreds of years under natural (no stress) conditions, but due to man's involvment, can be seen in a period of a few years. This decay function enables one to predict the time required for the adjustment of slope to reach some new equilibrium profile following channel disruption.

Alexander (1981) offers a discussion that further explains, in a physical sense, the reason for the waves and their declining amplitudes. Alexander states that a shock to the channel system will not only create a readjustment among the hydraulic variables (gradient, hydraulic radius, bed shear, size and variability of streamflow and bed load), but will be overcompensated for, such that degradation and aggradation alternate in the form of an oscillatory response. Furthermore, a shock of enough magnitude is capable of producing several periods of entrenchment in a single reach, giving rise to sequentially smaller oscillations as the response dwindles.

Depending on available data, examples of the four analyses are presented in the following case-study discussions.

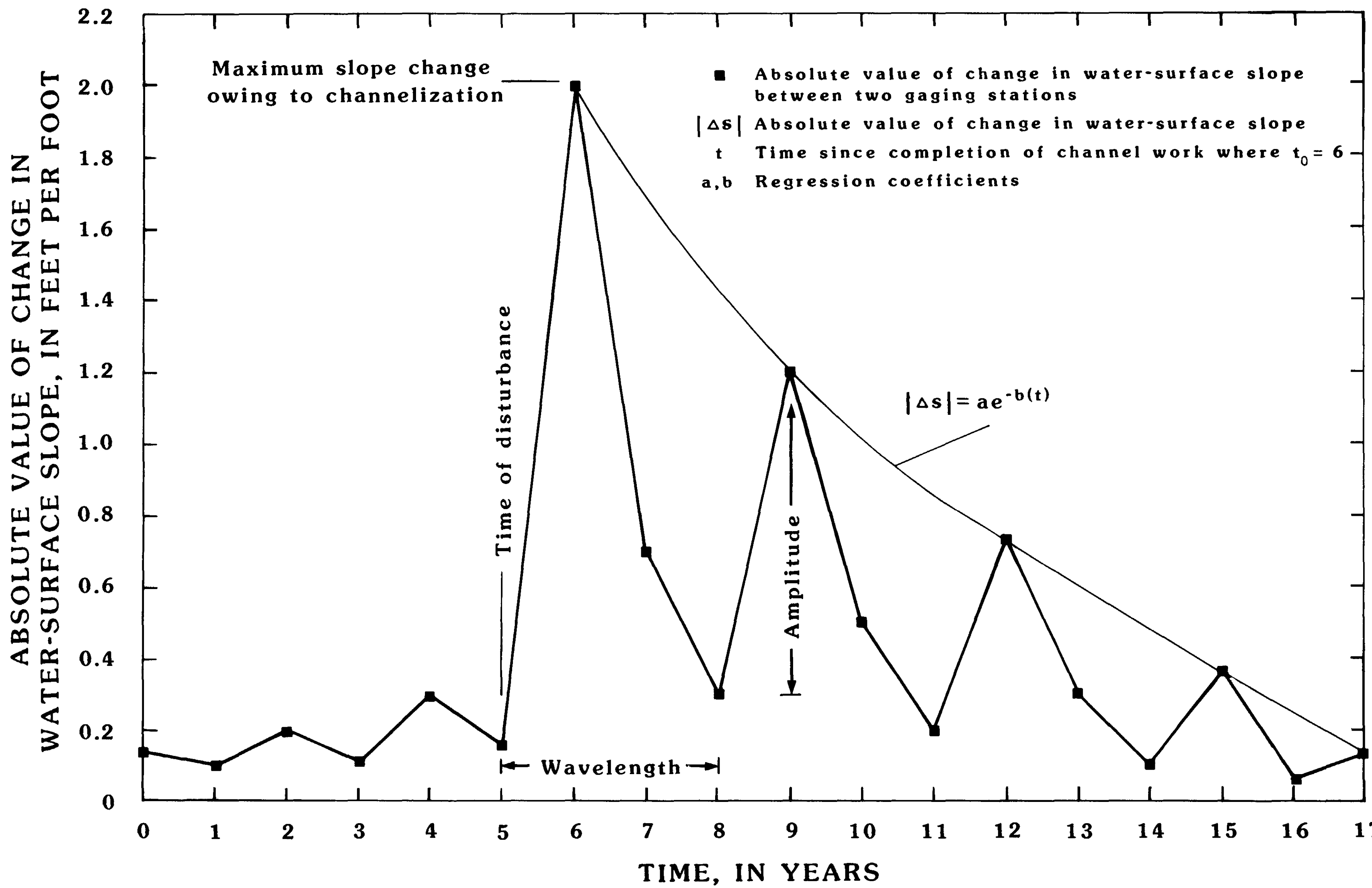


Figure 2.--Idealized plot of the absolute value of change in water-surface slope versus time between two gaging stations.

CASE STUDIES

In the following six case study discussions, a history of channel modification for specific stream reaches is given. Except for the North Indian Creek at Erwin, Tenn., discussion, each analysis is based on long-term streamflow records, stream-channel cross-section comparisons, and when available, channel-bed profiles. The North Indian Creek at Erwin, Tenn., case-study discussion (Supplement A) is presented solely to document the unusual photo history of a man-induced knickpoint migration in a consolidated bedrock formation.

Bridge failures occurred at (1) Nonconnah Creek (site 28, Shelby County) in 1980, (2) Toms Creek (site 49, Perry County) in 1975, (3) Owl Creek (site 74, Hardin County) in 1977, and (4) White Oak Creek (site 92, McNairy County) in 1977. Channel modifications at these sites are not discussed in this report because adequate data are not available for analysis. However, these sites are shown in plate 1 and listed in table 2.

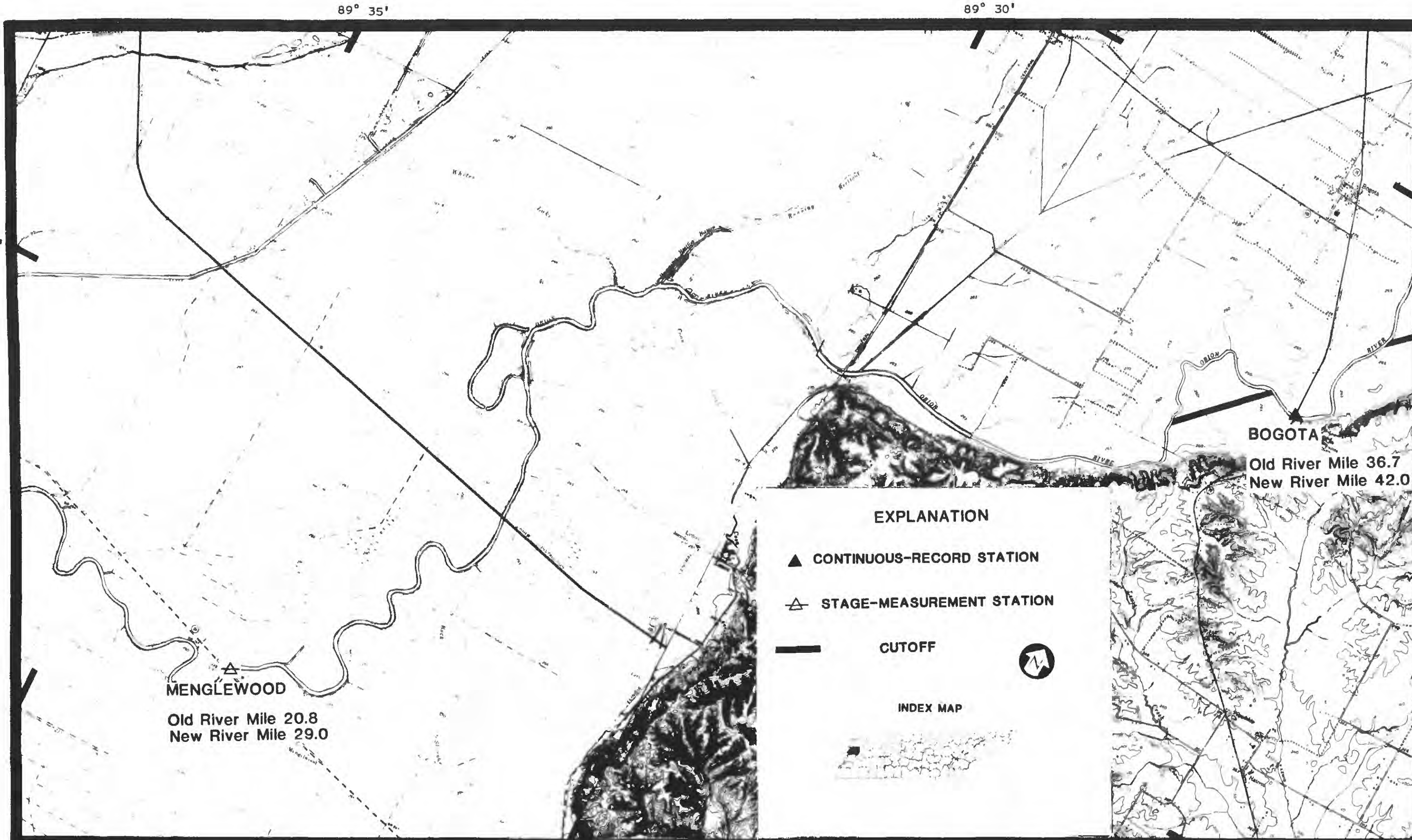
OBION RIVER - MENGLEWOOD TO RIVES

Prior to channelization, the Obion River in the study reach (old river miles 20.8 to 71.4, new river miles 29.0 to 66.5) followed a sinuous course in a flood plain ranging in width from approximately 1.0 miles at the upstream end of the study reach to approximately 3.5 miles at the downstream end (figs. 3, 4, and 5). A 1912 channel-bed profile (Hidinger and Morgan, 1912) shows a natural channel gradient of approximately 1.46×10^{-4} ft/ft.

To improve flood-plain drainage, local drainage districts conducted snagging and clearing operations on approximately 170 miles of main channel and tributaries of the Obion River system from 1938-43. Aerial photographs, taken in August 1941, indicate that channel-top width ranged from 90 to 130 feet. Cross sections made prior to 1959 show that channel depths ranged from 12 to 19 feet.

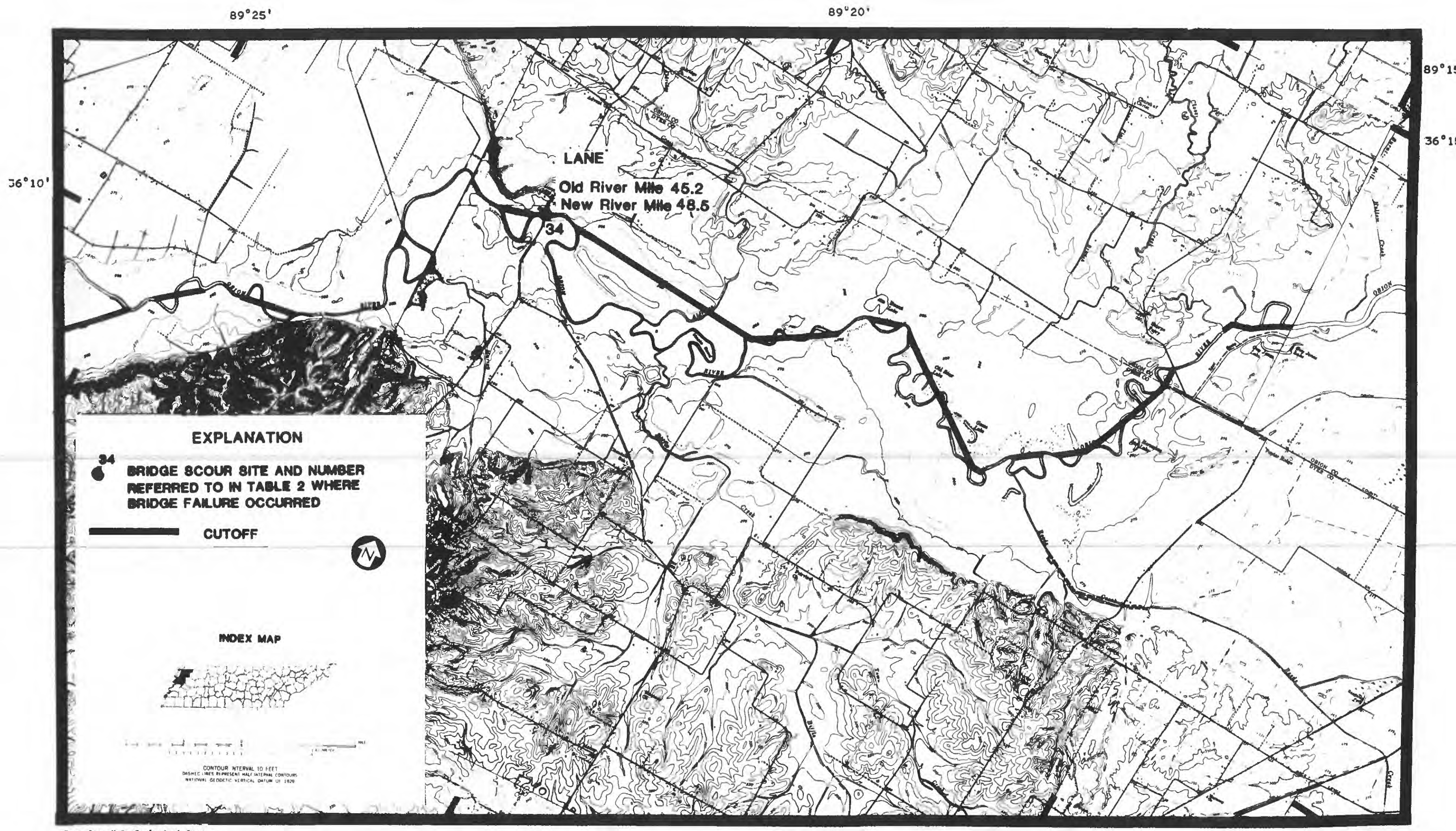
During 1959-60 channel enlargement including straightening and dredging was completed between old river miles 18.8 and 28.7 (new river miles 27.1 and 35.3), or to about 6 miles upstream of the Menglewood gage. From 1961-66 channel enlargement from old river mile 28.7 to old river mile 79.3 (new river miles 35.3 to 69.8) deepened the channel from approximately 15 feet to about 25 feet, and widened the channel top width from approximately 150 feet to approximately 220 feet. Upon completion, the total channel length had been shortened by approximately 17.8 miles (29.4 percent). The relation between old and new river miles can be seen in figure 6. According to the USCE dredging plans, the overall design gradient between the Menglewood and Rives gage was 1.29×10^{-4} ft/ft.

Plots of specific-gage and low-pool elevations against time at the Obion and Rives gaging stations show that the upstream reaches of the Obion River were steadily aggrading before the major channel alterations took place from 1959-66 (fig. 7). The water-surface slope, at a discharge rate of 500 ft^3/s, between Bogota and Obion (1939-59) ranged from 0.89×10^{-4} ft/ft to 1.13×10^{-4} ft/ft, and between Obion and Rives (1939-59) from 2.30×10^{-4} ft/ft to 4.10×10^{-4} ft/ft.



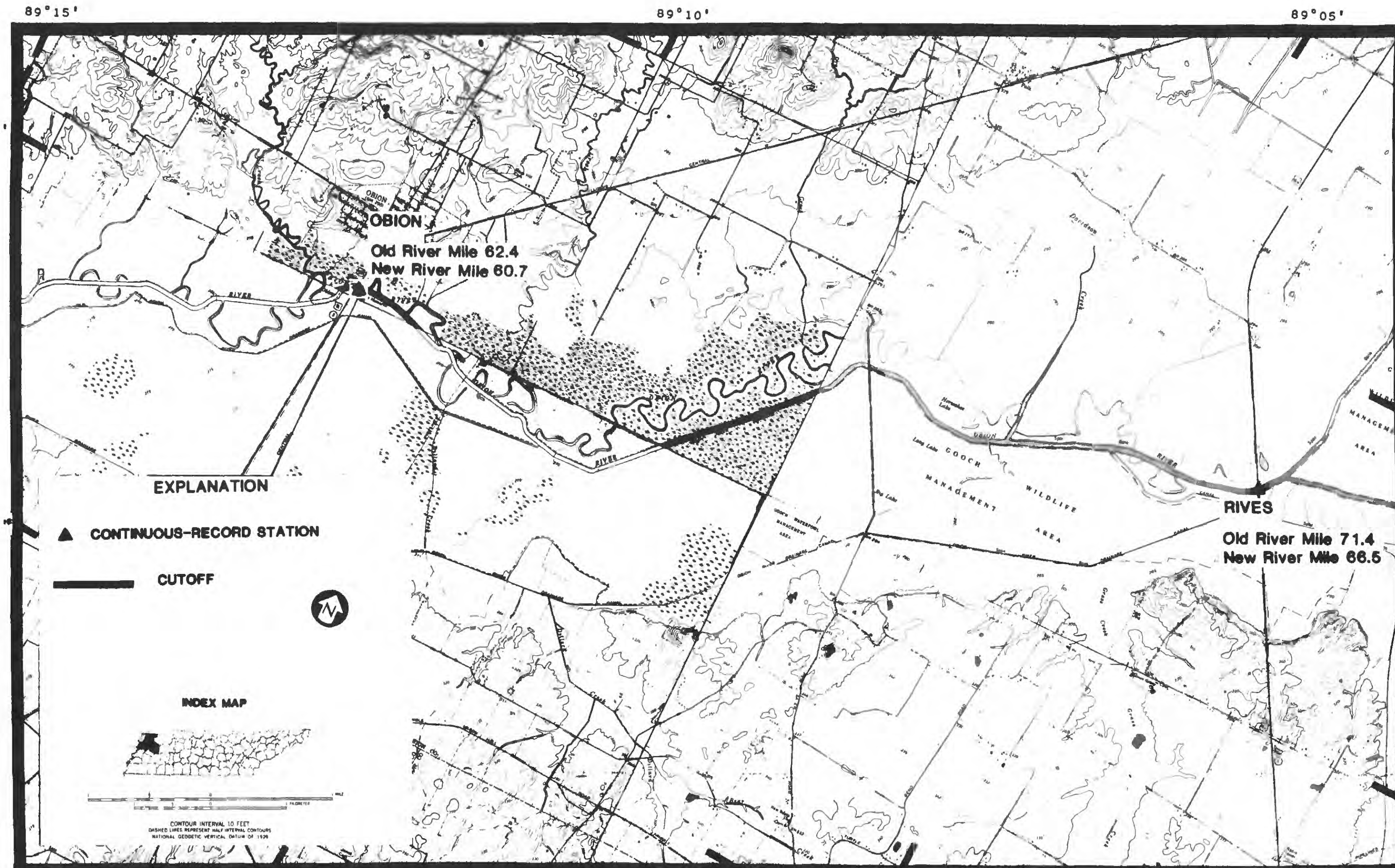
Base from U.S. Geological Survey
Caruthersville SE, 1:24,000, Cottonwood Point, 1:24,000, Tennemo, 1:24,000, 1971; Dyersburg, 1:24,000, Miston, 1:24,000, 1952

Figure 3.--Location of Menglewood and Bogota gaging stations and channel work in western part of Obion River, Tennessee.



Base from U.S. Geological Survey
Obion, 1:24,000, 1951; Dyersburg, 1:24,000,
Lane, 1:24,000, Miston, 1:24,000, Newbern,
1:24,000, 1952; Hornbeak, 1:24,000, 1964;
Trimble, 1:24,000, 1965

Figure 4.--Location of bridge-failure site at Lane and channel work in the middle part of the Obion River, Tennessee.



Base from U.S. Geological Survey
Obion, 1:24,000, 1951; Rives, 1:24,000, 1950,
revised 1980; Trimble, 1:24,000, 1965; Kenton,
1:24,000, 1965, revised 1981

Figure 5.--Location of Obion and Rives gaging stations and channel work in eastern part of Obion River, Tennessee.

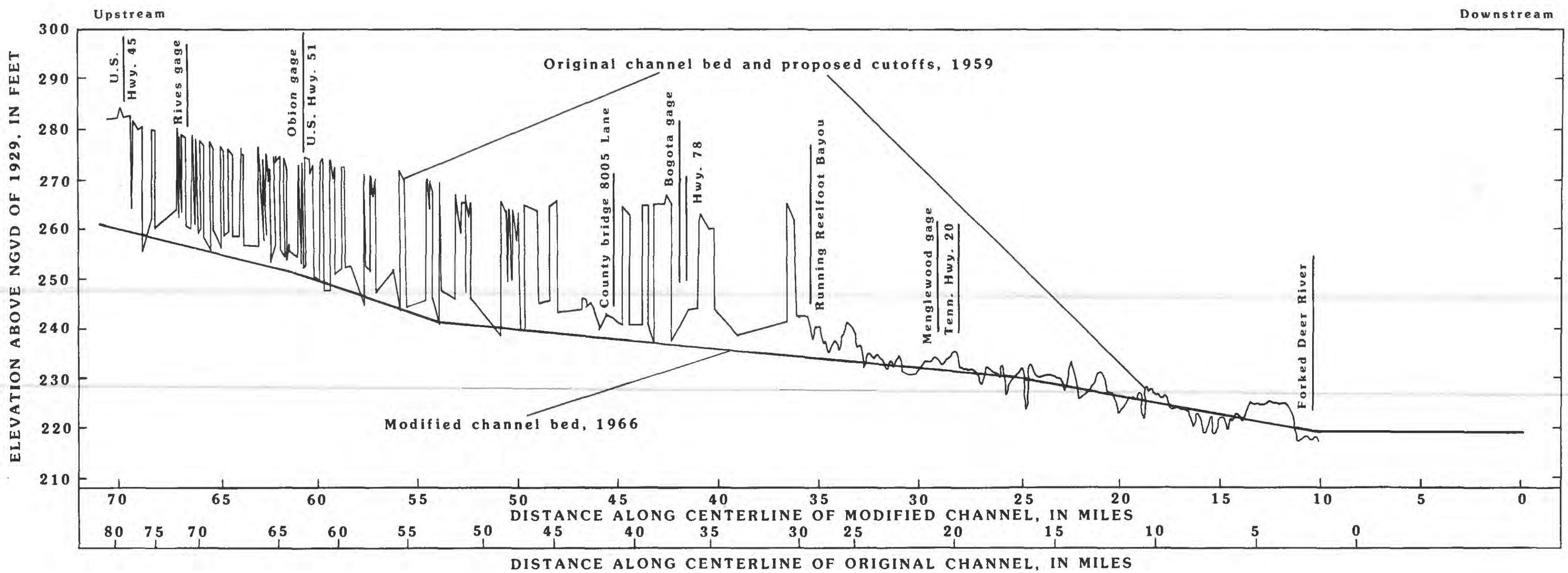


Figure 6.--Channel-bed profile from U.S. Army Corps of Engineers dredging plans showing channel work on the Obion River, Tennessee.

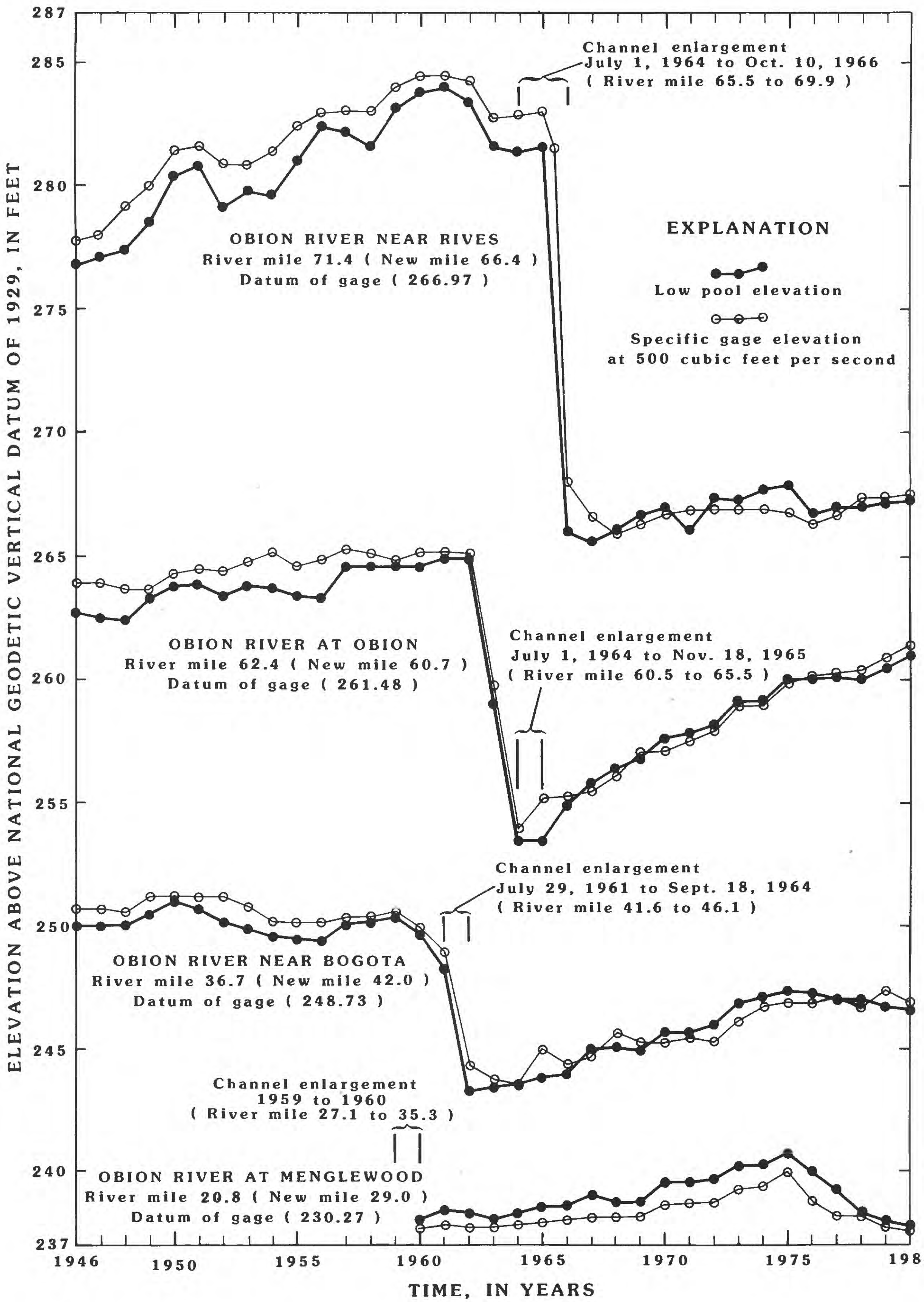


Figure 7.--Specific-gage and low-pool elevations versus time for gaging stations on the Obion River, Tennessee.

Plots of water-surface slope, at a discharge rate of 500 ft^3/s, against time (fig. 8) show that three years after channelization began in 1959, slope between Bogota and Obion had increased 70 percent from an average of 0.98×10^{-4} ft/ft to a maximum of 1.67×10^{-4} ft/ft. Within 5 years, slope between Obion and Rives had increased 170 percent from an average of 3.37×10^{-4} ft/ft to a maximum of 9.11×10^{-4} ft/ft. The immediate response of the river to increased channel gradient was headward erosion upstream as far as new river mile 66.5 (Rives) by the end of 1962.

Degradation was induced in the channel many miles above the channel work in progress due to the forced increase in channel gradient. Much of the material removed from the channel between Bogota and Obion between 1962-64 occurred by degradation as the river attempted to reduce its gradient in response to the imposed disturbances. For example, water-surface elevation, at a discharge rate of 500 ft^3/s, at Obion for the three years prior to channel work was approximately 265 feet above NGVD of 1929 (fig. 7). The combined effects of channel work and degradation resulted in an elevation of 254 feet above NGVD of 1929 by 1964, a decrease of approximately 11 feet (fig. 8).

As the upstream reaches of the channel eroded, large amounts of sediment were transported to the more gentle sloping downstream reaches and aggradation was initiated. By 1965, channel adjustment had reduced water-surface slope to 1.02×10^{-4} ft/ft between Bogota and Obion (fig. 8). Due to a higher aggradation rate at Obion (0.53 ft/yr) than at Bogota (0.30 ft/yr) (fig. 7), the water-surface slope between Bogota and Obion started increasing again in 1966 (fig. 8).

Slope adjustment of the main-stem Obion River along three successive reaches covering 37.5 miles (originally 50.6 miles) of channel from Menglewood to Rives is shown schematically in figure 9. The reduction in water-surface slope between Obion and Rives was a result of aggradation which took place in the vicinity of the Obion gage (0.53 ft/yr) between 1964-75 (fig. 7). Bogota, almost 19 river miles (originally 25.7 river miles) further downstream aggraded proportionately slower (0.30 ft/yr) whereas bed level at the Menglewood gage increased at an even slower rate (0.18 ft/yr). By considering that the "maximum" disturbance (based on percent slope change) took place at Rives (new river mile 66.5), plotting the aggradation rates at each gaging station against their distance from the Rives gage yields a very good relation [correlation coefficient $(r)=-0.997$]. This relation, with its associated equation for the main-stem of the Obion River is shown in figure 10. The equation, based on a minimum of 11 years of record (1964-75), allows calculation of the rate of aggradation following the completion of channel work at any point along the main stem of the Obion River. At Lane, which is 18.0 river miles (originally 26.2 river miles) downstream from the Rives gage, the channel aggraded at a calculated average rate of 0.36 feet per year.

Rainfall in West Tennessee for the period 1960-71 averaged about 7 percent below normal (based on 24 rain gaging stations in the study area) and average streamflow in the Obion River for the same period was at or slightly below the long-term average (1929-71). However, during 1972, 1973, and the first 10 months of 1974, rainfall was 22, 26 and 31 percent above normal, respectively (based on the same 24 rain gaging stations in the study area). Average

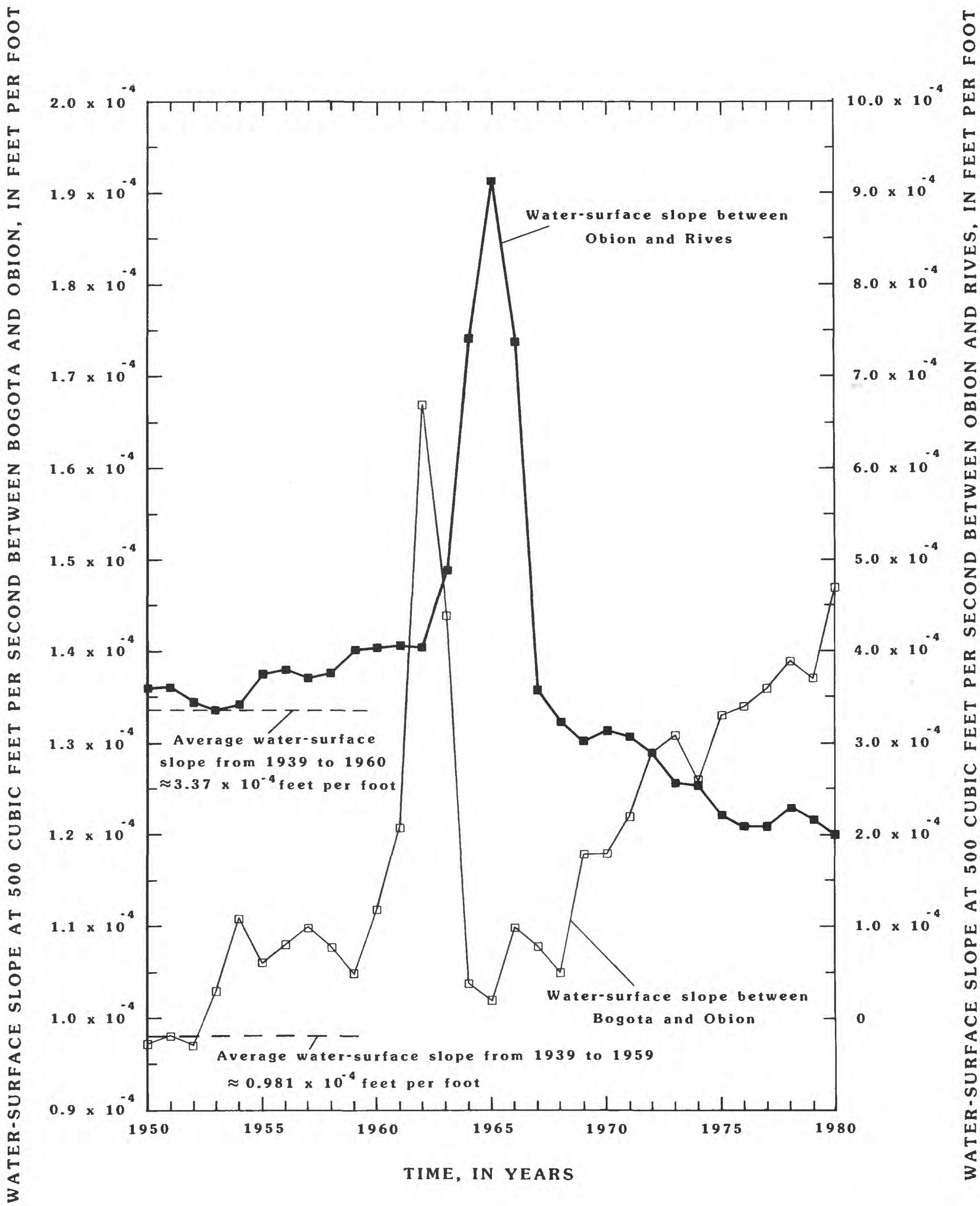
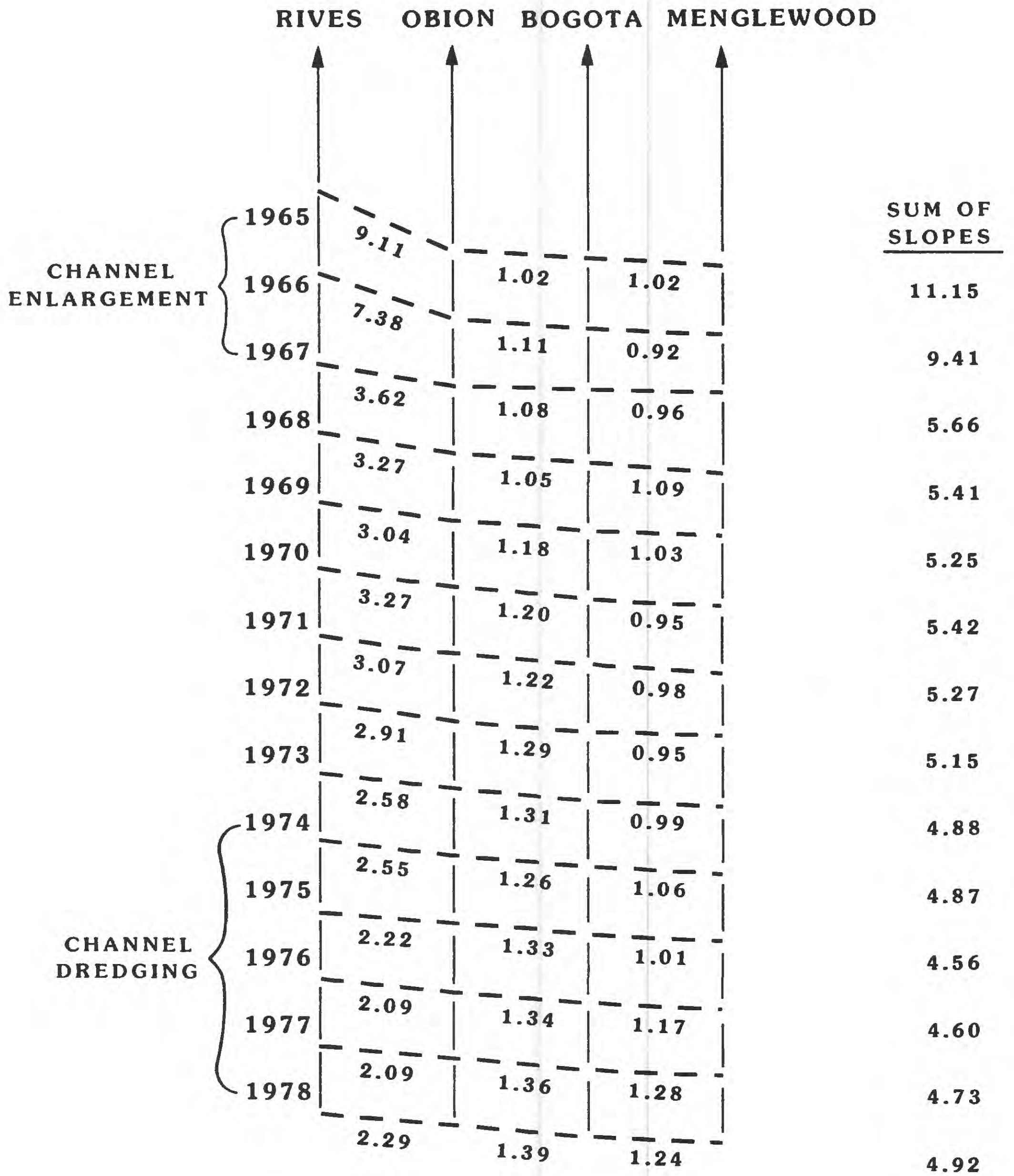


Figure 8.--Change in water-surface slope at a discharge rate of 500 cubic feet per second versus time between gaging stations on the Obion River, Tennessee.



Note : Horizontal distance and vertical slope not to scale

Figure 9.--Schematic representation of channel slope profile adjustments on the Obion River after channelization.

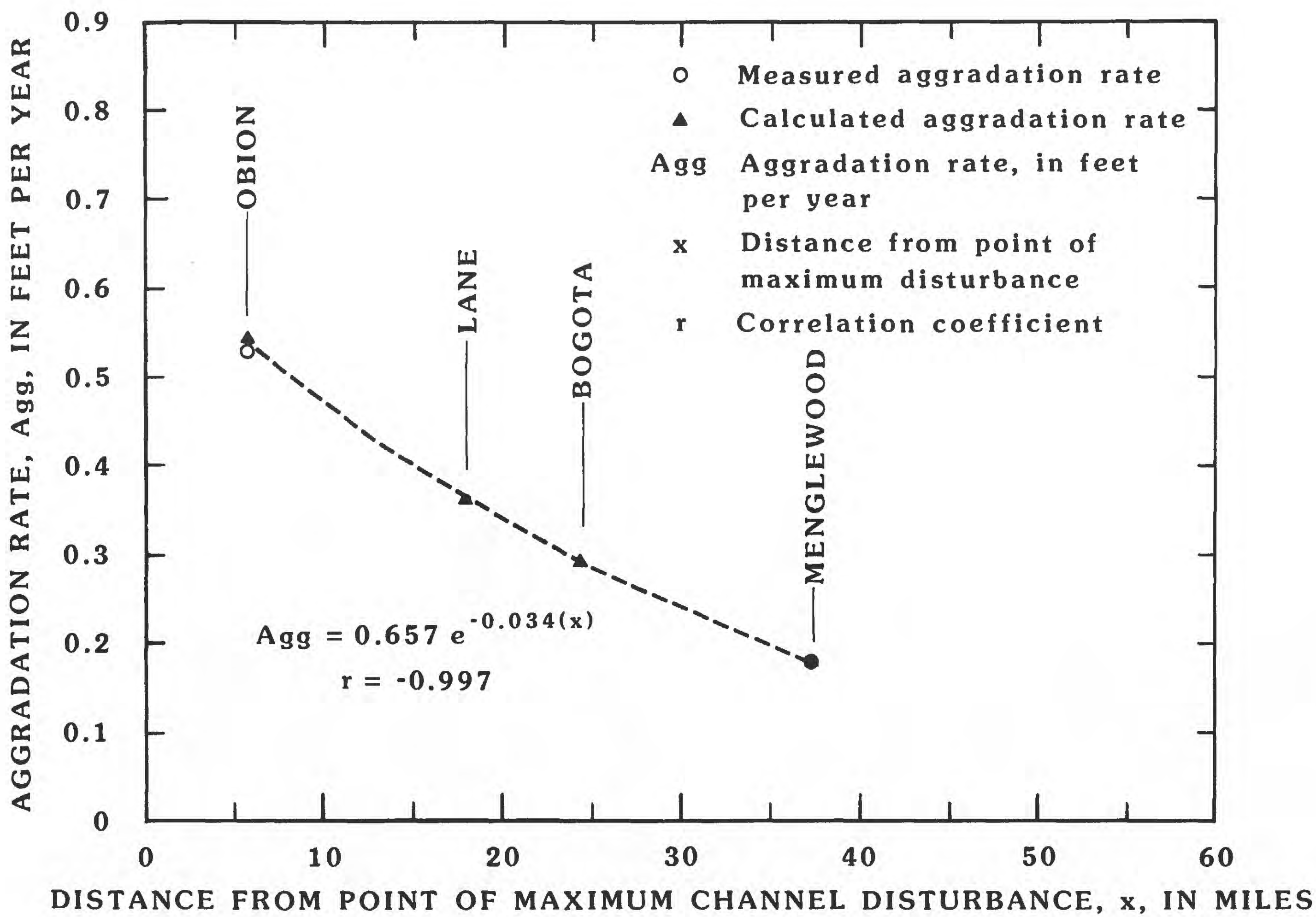


Figure 10.--Aggradation rates on Obion River (from 1964-75) as a function of distance from point of maximum channel disturbance.

streamflow for the same period was 5 percent less than, and 115 and 62 percent more than the long-term average, respectively. Based on gaging-station records at Obion (drainage area 1,852 square miles) 17.2 miles upstream from Lane, 22 flows near or above bankfull stage occurred between January 29, 1972, and January 14, 1974. From a frequency curve based on 52 years of record (1929-81), most of these flows ranged from 1.5- to 2.0-year frequency; however, flows on December 13, 1972, April 24, 1973, and January 14, 1974, were approximately 5-year frequency.

At about 6 a.m., on January 27, 1974, the S-8005 bridge at Lane (site 34) failed (fig. 4). Field inspection indicated that bent 4 settled enough to cause the span between bents 3 and 4 (bridge deck supports outside main channel) to slip and drop into the channel. This caused a loss of stability which in turn caused bent 4 and the two main channel piers to fall toward the left end of the bridge. The peak discharge during the flood of January 14, 1974, was 38,700 ft^3/s. The discharge at site 34 at the time of failure (January 27, 1974) was about 7,700 ft^3/s and rising.

Channel soundings around the bridge and about 500 feet upstream and downstream were made in March 1974 by the Tennessee Department of Transportation (TDOT). Although local scour around the piers could not be determined, the soundings indicated that about 8 feet of general scour had occurred in the channel since the completion of channel work in 1963. A scour hole about 32 feet downstream from the bridge may have resulted from sill action over the collapsed decking after the bridge had fallen.

The dredged channel at Lane (completed in June 1963) is in a flood plain that averages approximately 1.5 miles wide. The channel is straight for approximately 0.75 miles above the bridge and several hundred feet below the bridge. The bed level at Lane was lowered approximately 6 feet by the dredging (fig. 6). The present highway was constructed in 1959 and is immediately upstream from an old highway embankment that crosses the valley about normal to the direction of flood flow. Land owners filled in the old overflow bridge openings in the embankment and increased the height of the embankment to form a solid levee across most of the left flood plain.

Channel-top width in the vicinity of the S-8005 bridge at Lane (site 34) remained fairly constant at approximately 220 feet from the time of channel completion at Lane in June 1963 until the time of the bridge failure in January 1974 (fig. 11). An accurate determination of the channel-widening process within the reach cannot be made from the cross sections taken at the bridge. The bridge exerts such a profound hydraulic influence on channel characteristics that channel changes at the bridge are not necessarily indicative of changes taking place within the channel reach. Although channel-top width decreased approximately 16 percent at site 34 from 1963-74, extended periods of high water from 1972-74 and lateral channel shifting may have induced instabilities in the artificially sloped channel banks, causing bank sloughing and caving above and below the bridge. The cross sections and channel-bed profiles in figure 11 show the progressive change in channel geometry at the S-8005 bridge at Lane.

During 1963-74, channel flow capacity at Lane (site 34) decreased from 17,800 ft^3/s to approximately 10,800 ft^3/s, 39 percent, as a result of aggradation. The channel flow capacities were determined by indirect methods

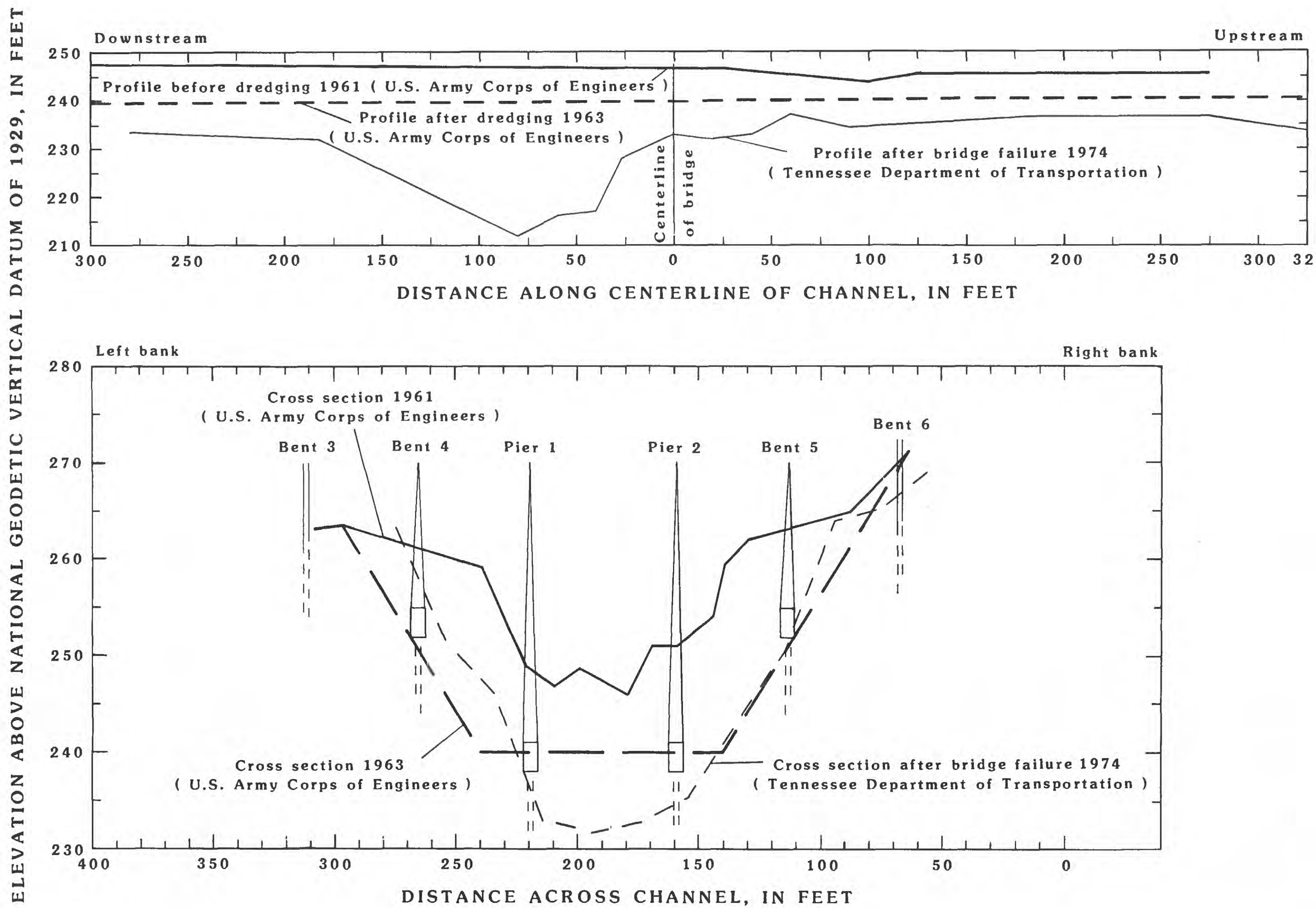


Figure 11.--Stream-channel cross sections and channel-bed profiles at old S-8005 bridge on Obion River at Lane, Tennessee.

using channel areas, slopes, and roughness coefficients. By comparing cross sections taken upstream from the S-8005 bridge at Lane (site 34) in 1963, 1973, and 1974, the decrease in cross sectional area, which would affect channel flow capacity can be seen (fig. 12).

At the time of the bridge failure (January 1974), channel soundings could not determine the local scour around the piers. However, two techniques using equations developed for sand channels were used to estimate the probable depth of local scour.

Method 1. Federal Highway Administration, equation 6.5.10 for depth of local scour, Design and Training Manual, page 6-38 (U.S. Department of Transportation, 1975):

$$Y_s = 2.2a^{0.65}\ Y_1^{\ 0.35}\ Fr_1^{\ 0.43} \qquad (7)$$

where Y_s = scour depth below original bed surface, or equilibrium scour depth, in feet,
Y_1 (depth of flow upstream of pier, in feet) = 42 ft,
a (width of pier or footing, in feet) = 6 ft, and
Fr_1 = upstream Froude Number (dimensionless)

$$= \frac{V_1}{\sqrt{gY_1}} \qquad (8)$$

where V_1 (average upstream velocity, in feet per second) = 5.29 ft/s, and g (gravitational acceleration, in feet per second squared) = 32.2 ft/s^2

Therefore: $Fr_1 = \frac{V_1}{\sqrt{gY_1}} = \frac{5.29}{\sqrt{32.2\ (42)}} = 0.144$ ← Froude Number

and $Y_s = 2.2\ (6)^{0.65}\ (42)^{0.35}\ (.144)^{0.43}$
$= 11.3$ ft ← Equilibrium scour depth

$$Ymax = 1.3\ (Y_s) = \qquad (9)$$

where Ymax = maximum scour depth, in feet

Therefore: Ymax = 1.3 (11.3)

= 14.7 ft ← Maximum scour depth

Maximum scour depth assumes movement of bed forms (dunes) through the local reach, no armoring, and a = 6 ft is constant for entire depth of scour. The average upstream velocity was obtained from U.S. Geological Survey files (C. T. Jenkins and I. J. Hickenlooper, written commun., 1957).

Method 2. Shen equation for clear-water scour (Shen, 1971):

$$d_{se} = 0.00073\ R^{0.619} \qquad (10)$$

where d_{se} = equilibrium depth of scour measured from mean bed elevation, in feet, and
R = pier Reynolds Number (dimensionless)

$$= \frac{Vb}{\nu}, \qquad (11)$$

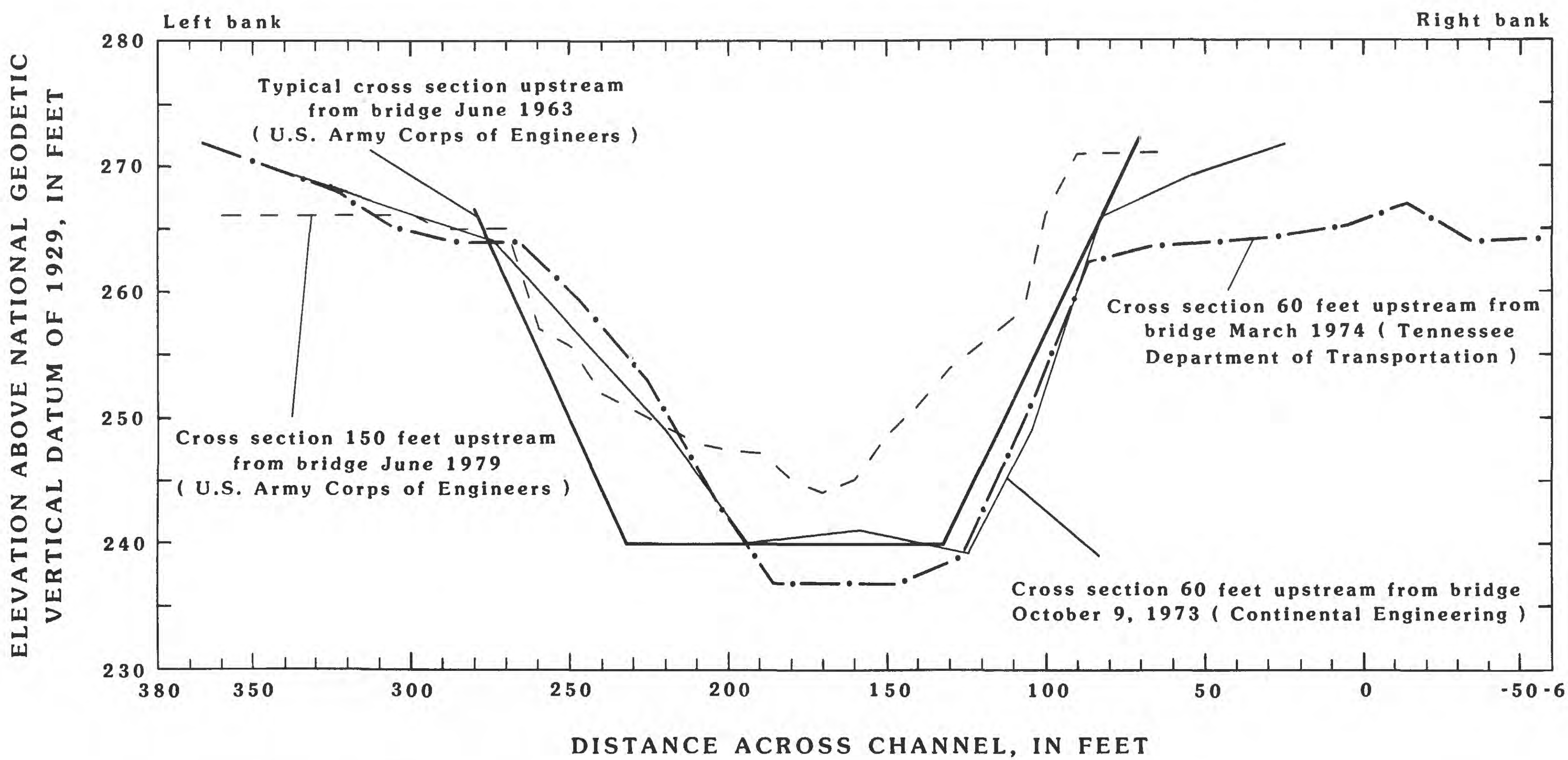


Figure 12.--Stream-channel cross sections of Obion River upstream of S-8005 bridge at Lane, Tennessee.

where V (mean velocity of undisturbed flow, or mean upstream velocity, in feet per second) = 5.29 ft/s

b (width of pier, or footing, in feet) = 6 ft, and

(kinematic viscosity of water at 40°F, in feet squared per second) = 1.66×10^{-5} ft^2/s

Therefore: $d_{se} = 0.00073 \left(\dfrac{5.29(6)}{1.66 \times 10^{-5}} \right)^{0.619}$

$= 5.6$ ft ← Equilibrium scour depth

$$d_{max} = 1.3(d_{se}) \qquad (12)$$

where d_{max} = maximum scour depth, in feet

Therefore: $d_{max} = 1.3(5.6)$

$= 7.3$ ft ← Maximum scour depth

Major factors or a combination thereof which contributed to the scour problem and bridge failure are inferred as follows:

1. Flows in the overflow bridges were blocked by the old highway embankment downstream, and the reduced channel capacity forced additional flow through the main channel bridge. Although the January 14, 1974, flood was only about a 5-year frequency flood, the discharge at the main channel bridge was estimated to be equivalent to the structure's design flood (50 year) but at an elevation 4.5 feet lower.
2. Channel straightening upstream and downstream from the bridge increased the original channel gradient 70 percent from about 0.981×10^{-4} ft/ft to 1.67×10^{-4} ft/ft in 1962. Induced erosion upstream of Obion resulted in increased sediment movement, and aggradation downstream. These processes produced an unstable channel. Channel straightening also changed the flow distribution for which the bridge was designed and concentrated the majority of flow through the main channel bridge (fig. 13).
3. Field inspection suggested that bank failure on the right side of the channel upstream from the bridge deflected the flow toward pier 1 (fig. 14).
4. Field inspection also suggested that large amounts of debris lodged on the two main piers reduced the bridge waterway causing an increase in velocity through the bridge. The debris probably caused large scale eddies which increased the amount of scour. The longitudinal profile in figure 11 shows the enlarged scour hole as it appeared shortly after the failure in 1974.
5. Valley conditions were significantly altered by channel enlargement and straightening. The conditions for which the bridge was designed no longer existed. Of the items listed, the major ones were the channelization (causing aggradation), reduced capacity, scour at the downstream side of the bridge due to reduced bridge channel capacity, and altered flow distribution.

The bridge at Lane was rebuilt at the same location in 1974 using longer deck spans, which provided for a larger distance between piers and fewer bents. Between 1974-79, the Obion River was dredged from approximately three miles below the confluence of the Forked Deer and Obion Rivers up to about 2 miles downstream of the Menglewood gage, new river mile 27.1 (site 33, plate 1).

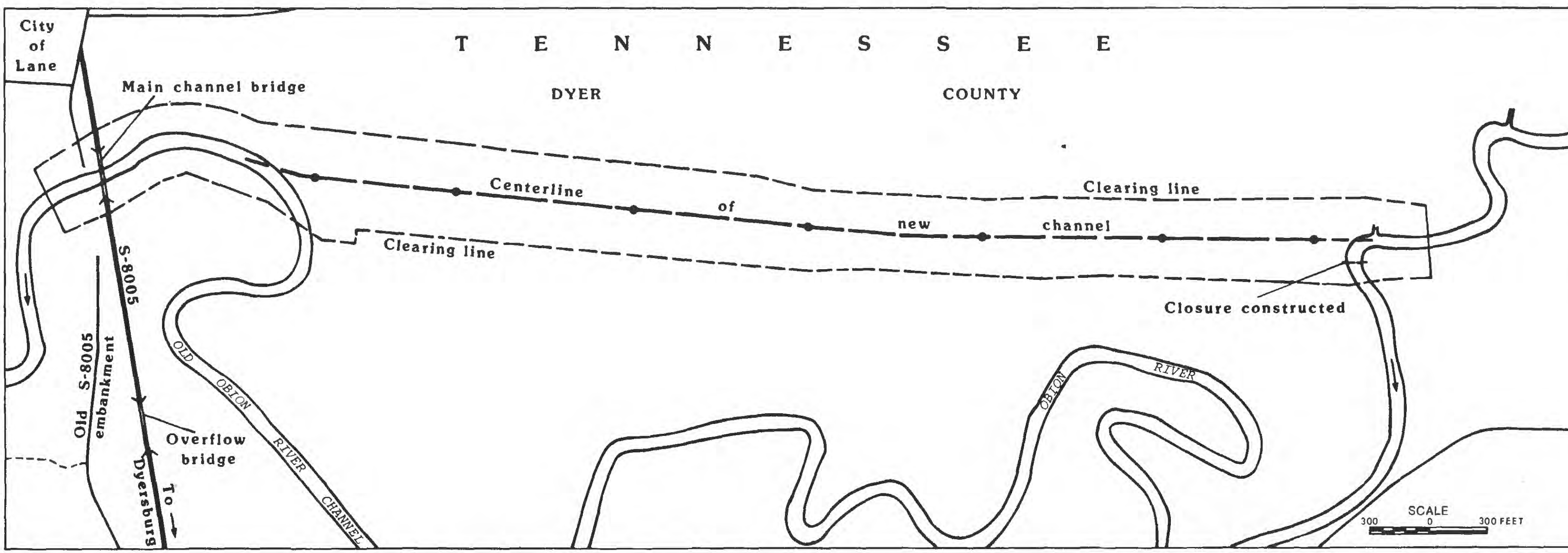


Figure 13.--Sketch showing original channel and present-day channel (1982) in relation to bridge openings on Obion River at Lane, Tennessee.

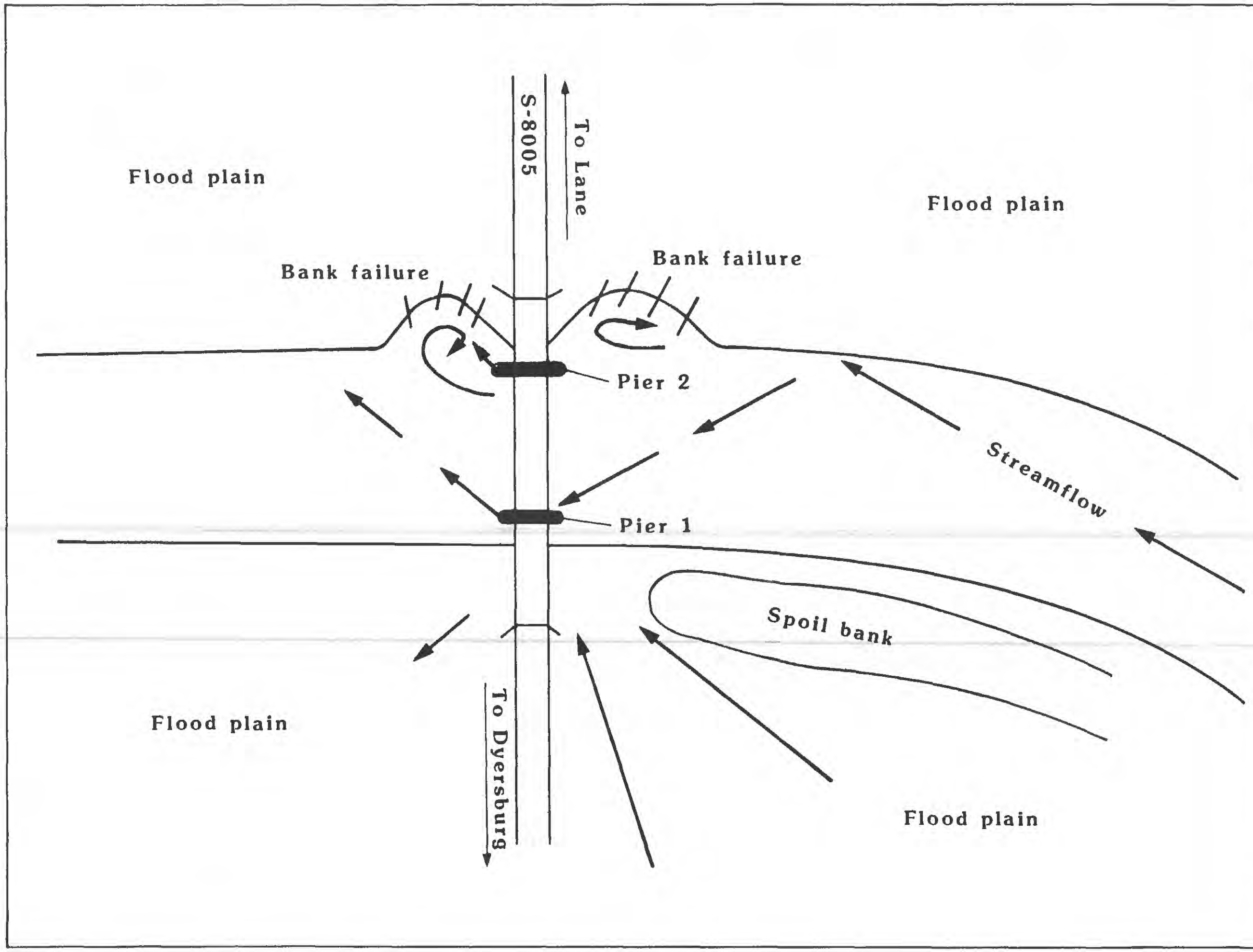


Figure 14.--Diagram showing concept of streamflow pattern on Obion River at Lane, Tennessee.

Longitudinal channel profiles were run in May and December 1975 from approximately 7 miles downstream to several hundred feet upstream of the new bridge at Lane (fig. 15). The profiles indicate that the channel was aggrading. Profiles of the channel at the new bridge are plotted on figure 16, along with three cross sections from discharge measurements in March and April 1976 and January 1980. The profiles indicate that the scour hole has filled below the bridge, and the cross sections indicate that the channel section at the bridge is shifting laterally. The cross sections may not be entirely representative of current channel processes because the old bridge deck was never removed from the channel bottom and may affect the cross section geometry and velocity distribution at the bridge. A cross section taken approximately 150 feet upstream from the bridge in June 1979 is shown in figure 12. Compared with the cross sections prior to the bridge failure at approximately 60 feet upstream of the bridge, it is apparent that the channel has aggraded since March 1974. The 1979 channel capacity was approximately 8,120 ft^3/s, or a loss in channel flow capacity of 54 percent since 1963.

Sum of slopes data (fig. 17) show the "natural" tendency of water-surface slope reduction from 1967-75. The sum of slopes increased in 1976 as a response to increased sediment loads caused by headward erosion resulting from further dredging below Menglewood in the mid-1970's. This disturbance increased water-surface slope between Menglewood and Bogota from 0.99×10^{-4} in 1976 to 1.36×10^{-4} ft/ft by 1979 (37 percent increase). Headward erosion is expected to occur in the main stem as this disturbance propagates upstream. As of 1979, the adjustment had not yet reached Bogota (new river mile 42.0), as indicated by specific-gage records in figure 7.

Due to the higher rate of aggradation at Obion than at Bogota, the water-surface slope in this reach started increasing again in 1966 and reached a secondary maximum in 1977 of 1.36×10^{-4} ft/ft (fig. 8). Although water-surface slope increased between Bogota and Obion from 1966-77 (fig. 8), figure 17 shows that the trend for the entire main stem of the Obion River is generally the reverse. The slight increase from 1975-78 (fig. 17) is a result of channel dredging below Menglewood.

Regressing the sum-of-slopes data against time yields a very good relation [correlation coefficient (r) = -0.94]. This relation, with its associated equation is shown in figure 17. The equation can be used to define the channel adjustment from 1967-75 as follows:

$$\Sigma S = e^{-0.0233(t)-7.4793} \qquad (13)$$

where

ΣS = the sum of slopes of three reaches, and

t = time in years since the completion of channel work where t_o = 1967.

By summing the means of the individual channel-reach water-surface slopes prior to the 1960's channel work, a sum of slopes, ΣS, of 4.51×10^{-4} ft/ft is obtained for original channel conditions. Substituting this value for ΣS and solving equation (13) for time (t) indicates that this water-surface slope should have been attained by 1977. Because the overall channel length was reduced by 29.4 percent, slope reduction by downstream aggradation is expected to continue beyond this value to a lower profile gradient. According to the USCE dredging plans, the overall design gradient between the Menglewood and Rives gages was to be 1.29×10^{-4} ft/ft, as compared to an average water-surface slope, at a discharge rate of at 500 ft^3/s, of 3.14×10^{-4} ft/ft

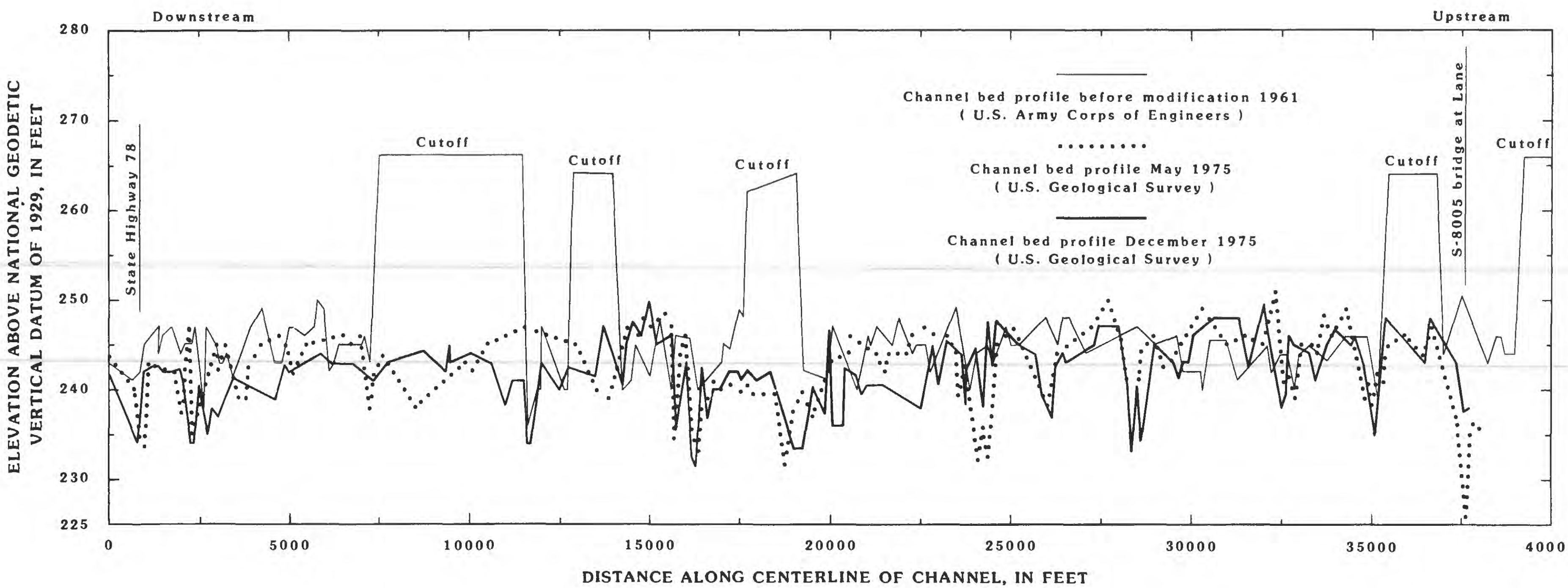


Figure 15.--Channel-bed profiles of the Obion River from below State Highway 78 to above S-8005 bridge at Lane, Tennessee.

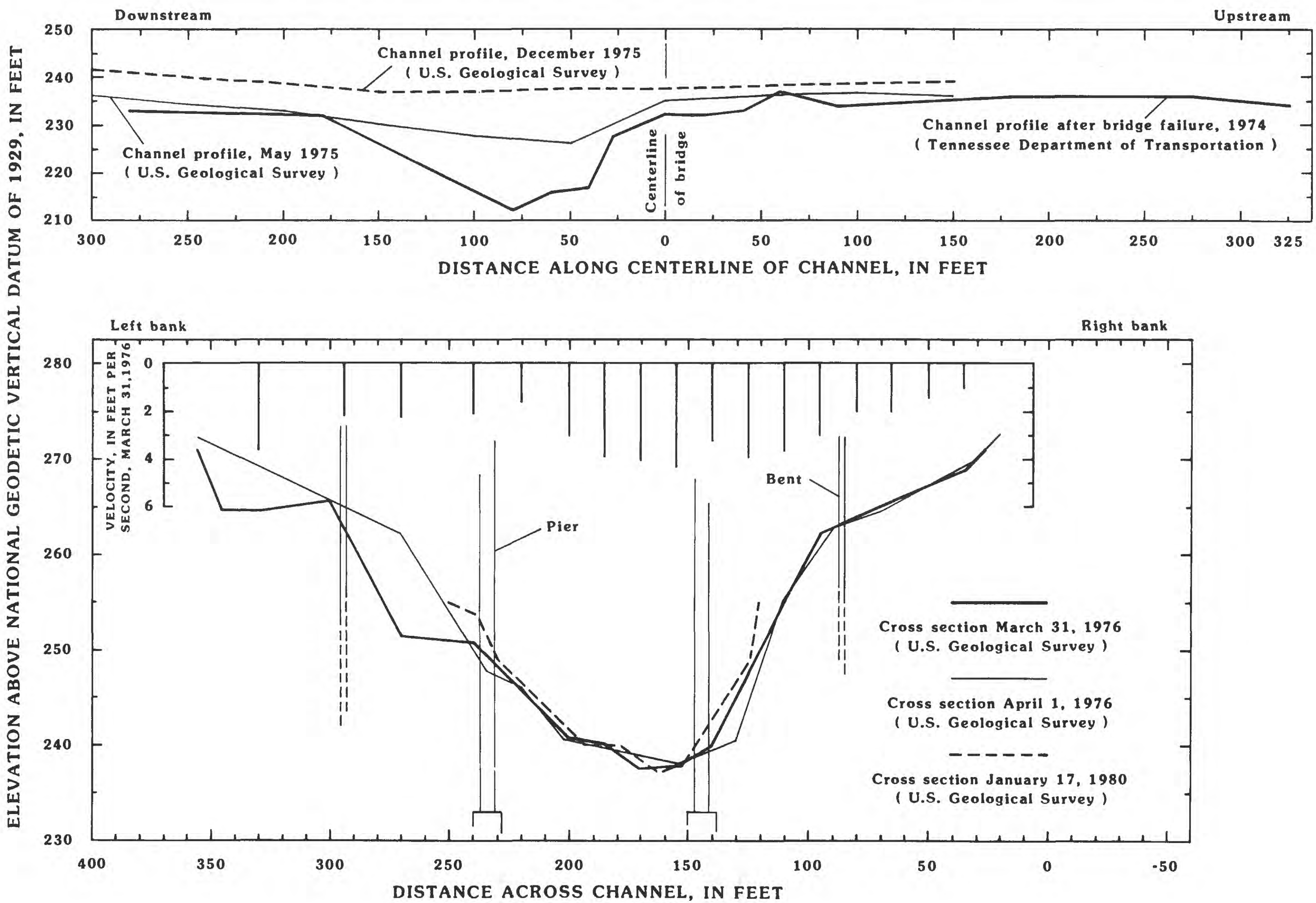


Figure 16.--Stream-channel cross sections showing velocity distribution and channel-bed profiles at new S-8005 bridge on the Obion River at Lane, Tennessee.

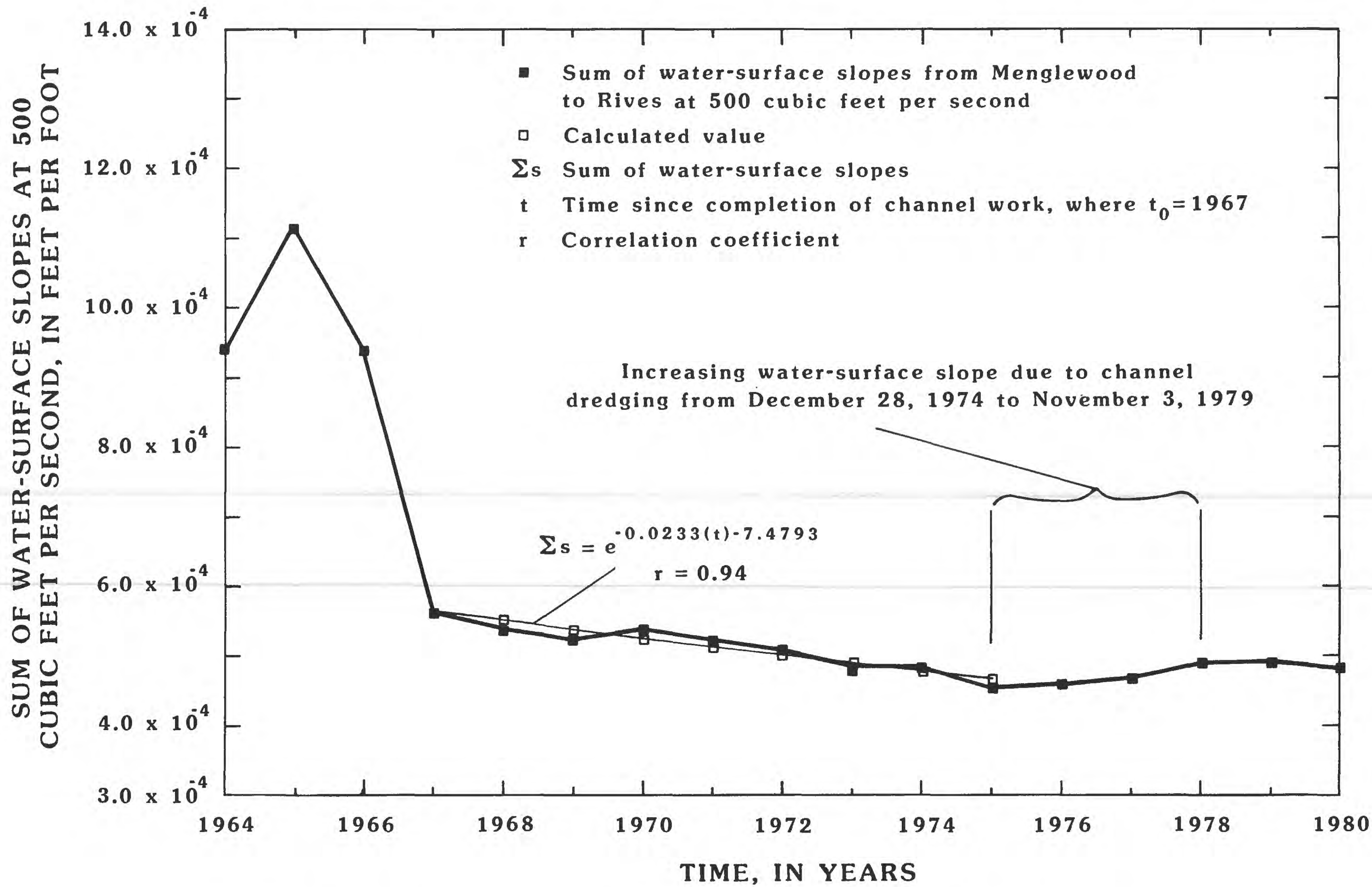


Figure 17.--Sum of water-surface slopes at a discharge rate of 500 cubic feet per second versus time on Obion River, from Menglewood to Rives, Tennessee.

at the time of completion in 1966. By equation (13), this proposed design grade would be achieved approximately 25 years following the completion of channel work, or by 1992.

Because channelization in the 1960's progressed steadily upstream on the main-stem Obion River, rates of adjustment propagation within individual reaches cannot be determined from the specific-gage plots. However, the exponential decay function can be used to determine long-term trends and the relative effects of channel work along the river between 1967-75. Calculated response time does not include the effect of further channel work in the mid-1970's. Plots of the exponential decay function for yearly slope response in the three measured reaches are shown in figures 18, 19, and 20. The actual value of change in slope for a given year varies proportionately with the distance from Rives (point of "maximum" disturbance; therefore, there is a greater fluctuation in the upper reaches). All three curves approach a minimum value (are asymptotic) at roughly 20 years from their maximum imposed slope change. By assuming that the response in the upstream reach (Obion to Rives) would be completed last, or by 1987 (1967 plus 20 years), the resulting sum of slopes, ΣS, by equation (13) will be 3.56×10^{-4} ft/ft. This value represents an additional 22 percent decrease in the 1975 sum of slopes profile that resulted from the 26 percent reduction in channel length from the 1960's channel work.

Absolute values of yearly change in slope data for the Menglewood to Bogota reach are shown in figure 18. Note the reduction in the amplitude of the peaks through the early 1970's as the effect of the disturbance is lessened in the reach. The slope-change peak in 1976 appears to be directly related to the dredging initiated in 1974. Two successive years of higher than average flow in 1973 and 1974 caused less change than the continued channelization. A second exponential decay function could be described to define the response of the reach after 1976 if more data were available.

SOUTH FORK FORKED DEER RIVER - YELLOW BLUFF TO GATES

The South Fork Forked Deer River in the study reach (old river miles 5.7 to 19.0, new river miles 3.3 to 16.3) followed a sinuous course prior to channelization in a flood plain ranging in width from approximately 1.5 miles at the upstream end of the study reach to approximately 3.0 miles at the downstream end (figs. 21, 22, and 23). The natural channel ranged in width from 70 to 110 feet and in depth from 12 to 15 feet (Hidinger and Morgan, 1912). Channel gradient ranged from 1.0×10^{-4} ft/ft to 2.7×10^{-4} ft/ft.

Initial work on the straightening of the South Fork Forked Deer River began in approximately 1914 and ended in the early 1920's. The channel work extended from the vicinity of the present U.S. 51 bridge (site 42), old river mile 9.0, to Jackson, Tennessee, old river mile 90. The channel work was done by local drainage districts to provide better drainage for the flood plain. Approximately 80 miles of channel with a sinuosity of 2.1 were transformed to a series of long straight reaches with a sinuosity of 1.3 and an average gradient of 3.2×10^{-4} ft/ft (L. E. Peterson, written commun., 1981). Cross sections which were taken at the time of channelization and 5 years later, showed a maximum channel widening of 25 feet (Ramser, 1930).

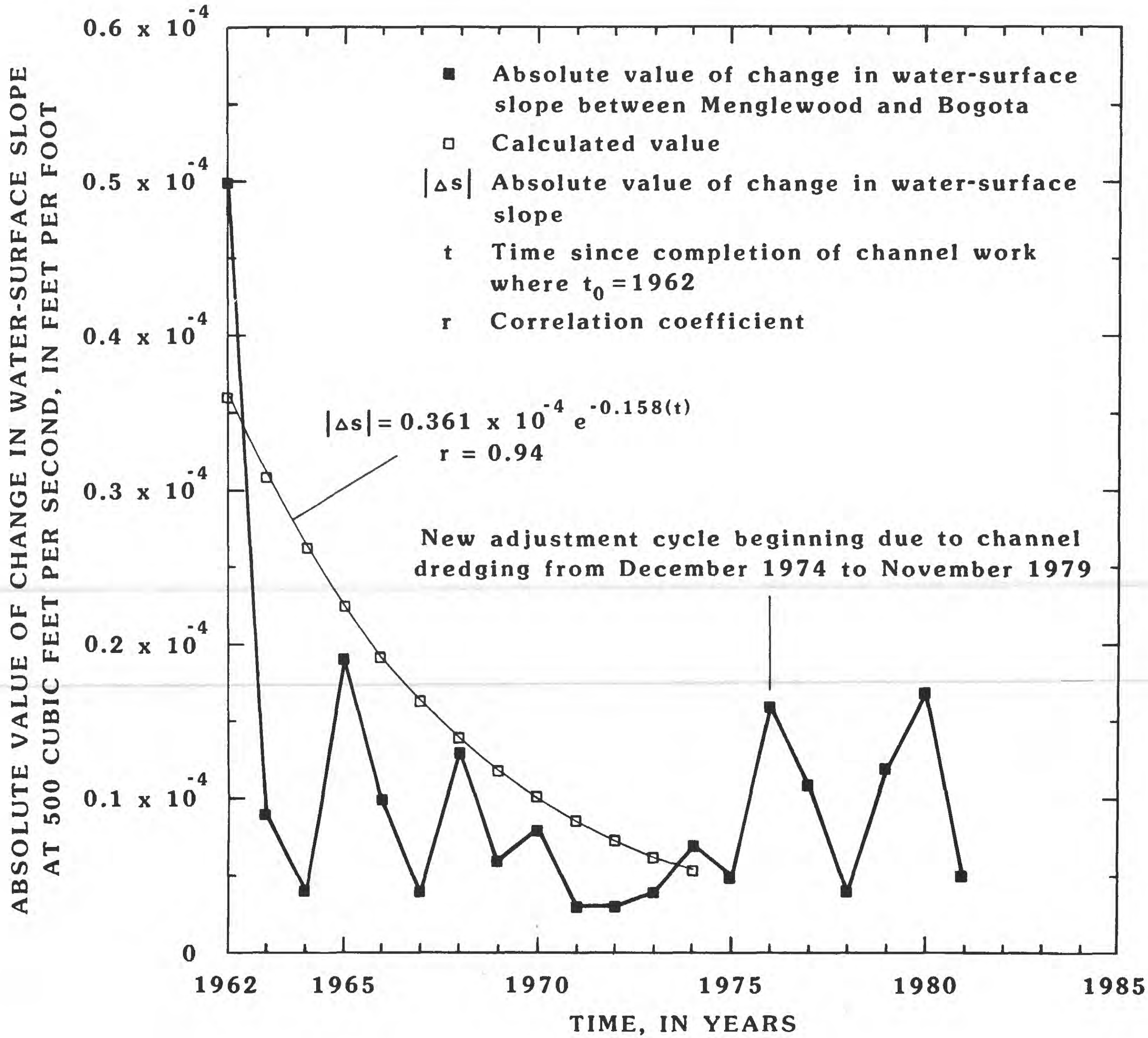


Figure 18.--Exponential decay function describing the yearly variation in water-surface slope fluctuation on the Obion River between Menglewood and Bogota, Tennessee.

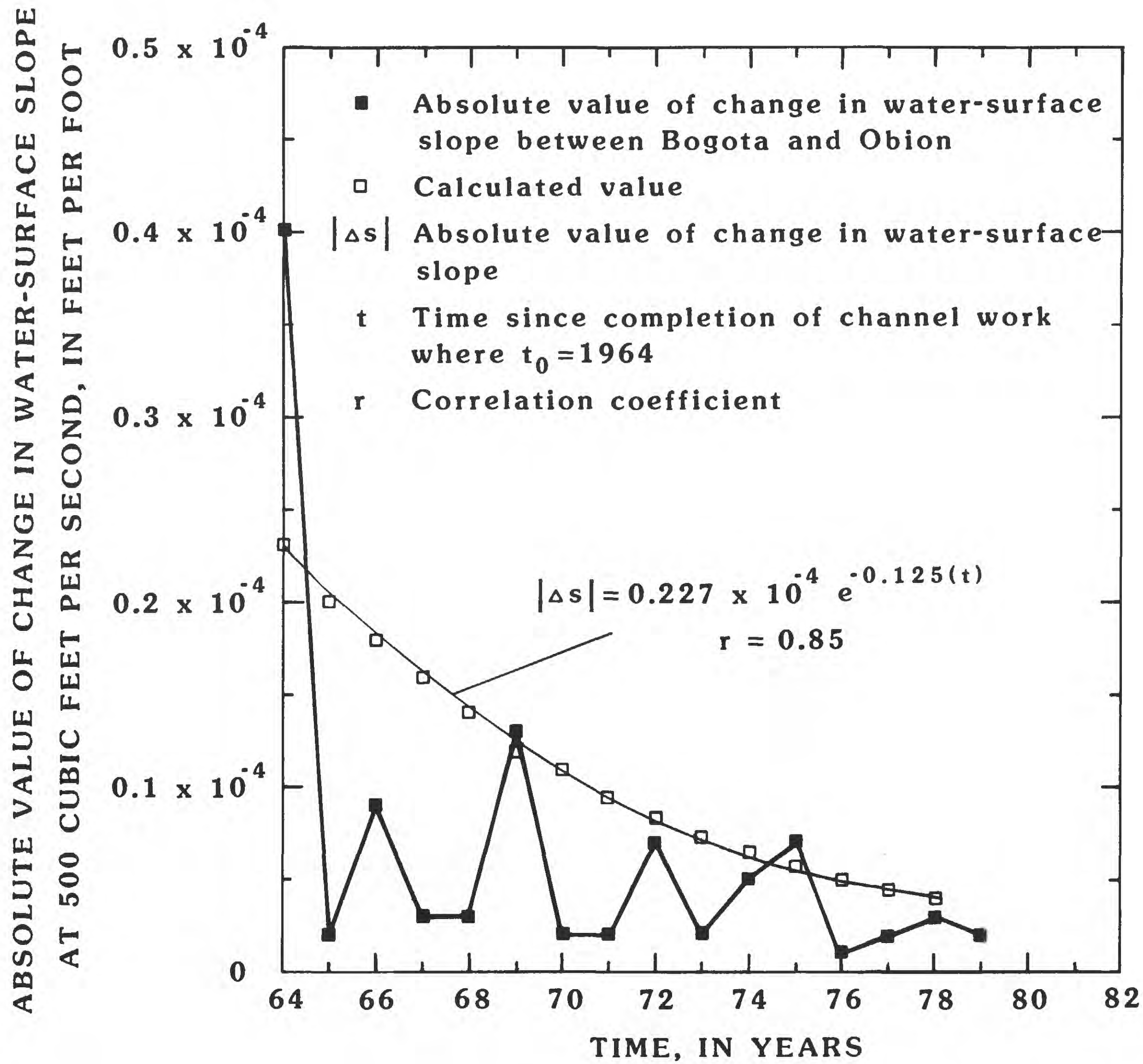


Figure 19.--Exponential decay function describing the yearly variation in water-surface slope fluctuation on the Obion River between Bogota and Obion, Tennessee.

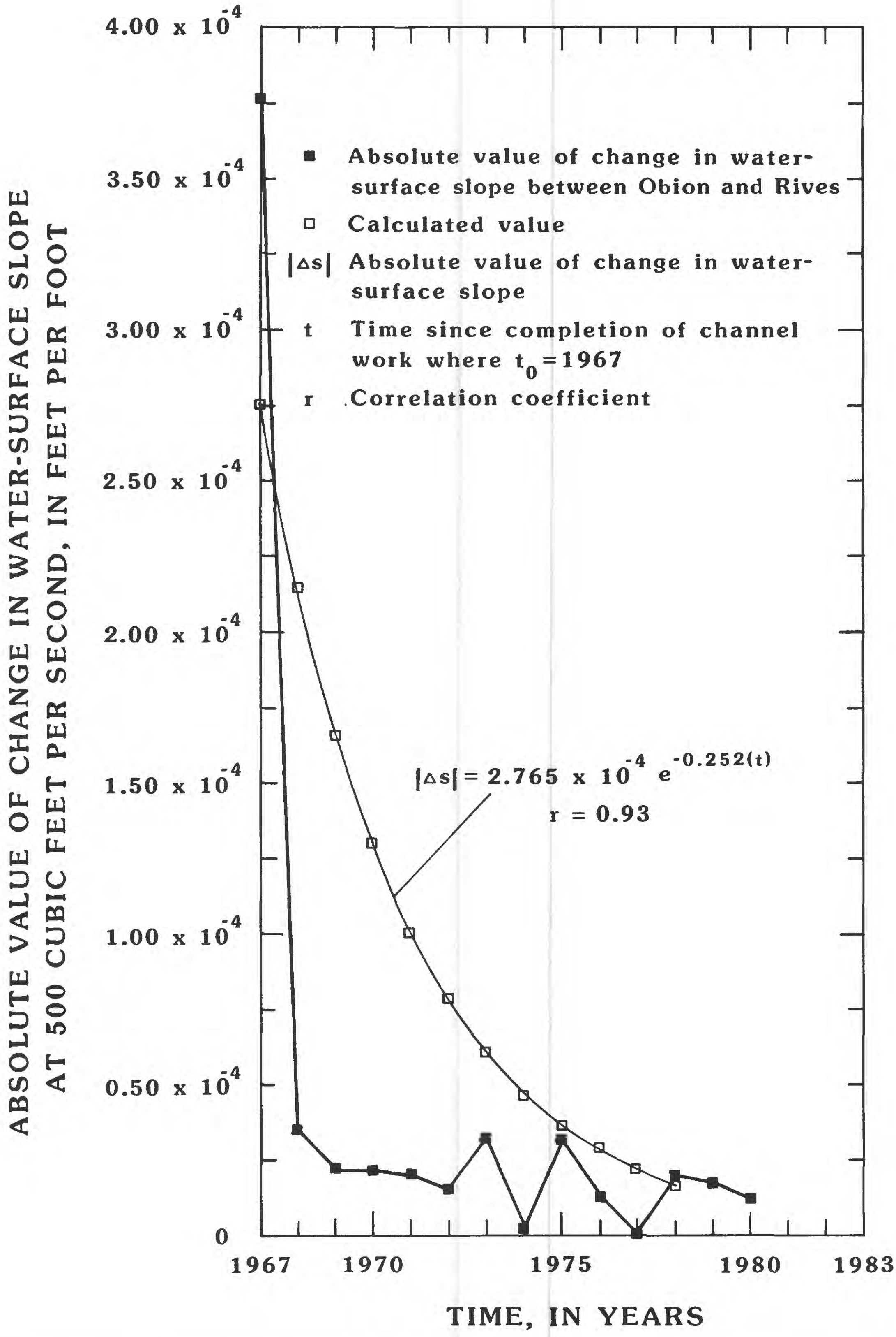
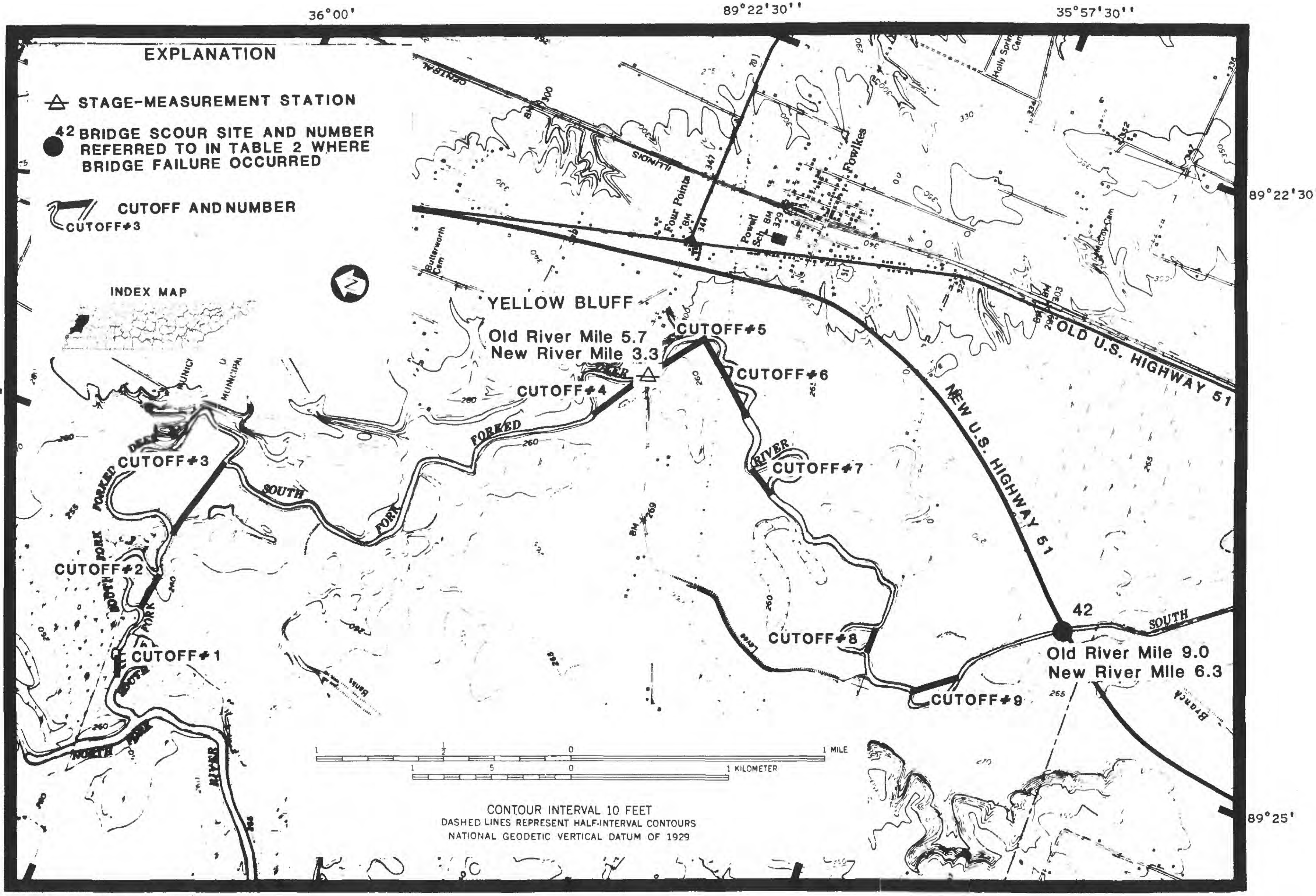
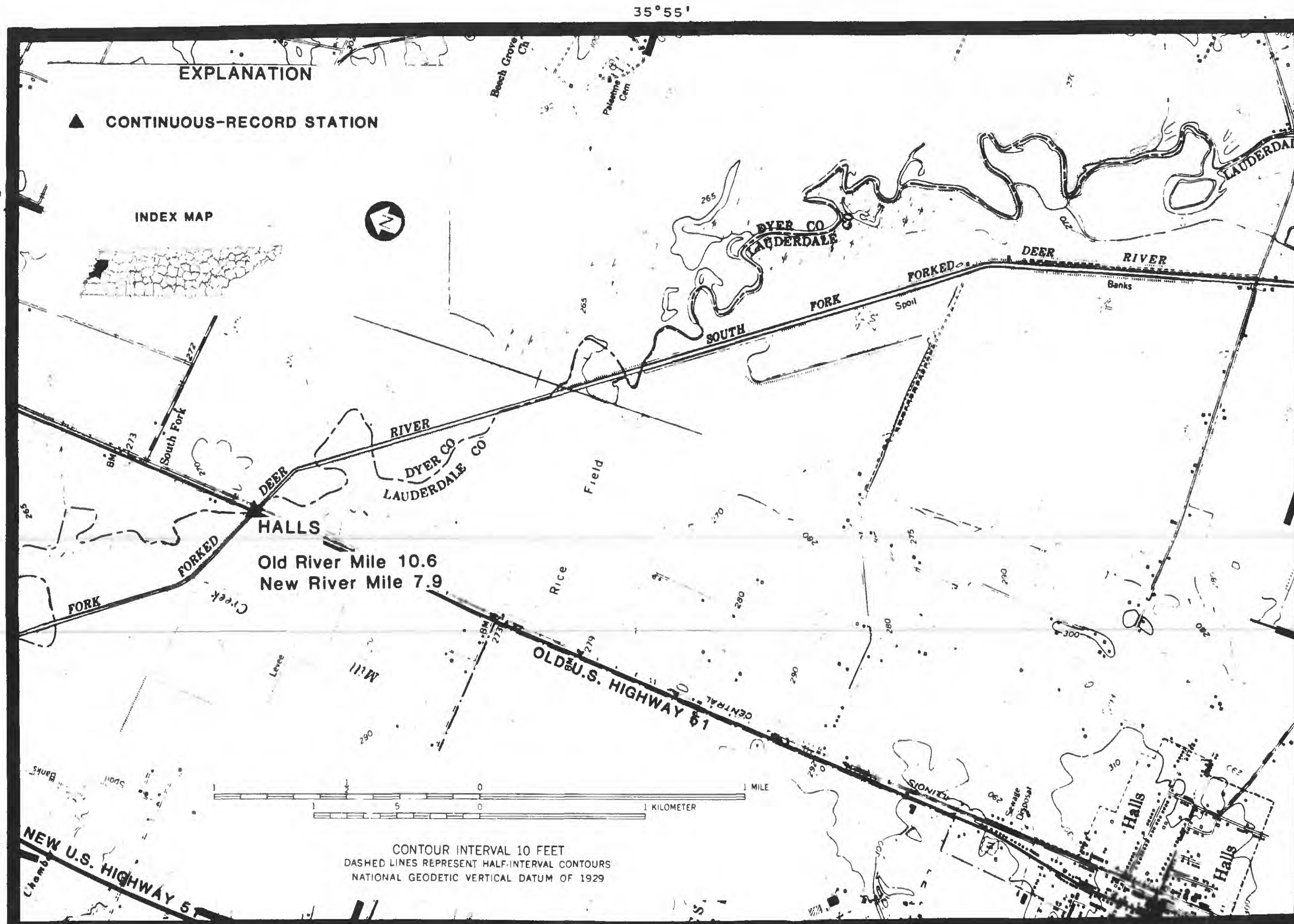


Figure 20.--Exponential decay function describing the yearly variation in water-surface slope fluctuation on the Obion River between Obion and Rives, Tennessee.



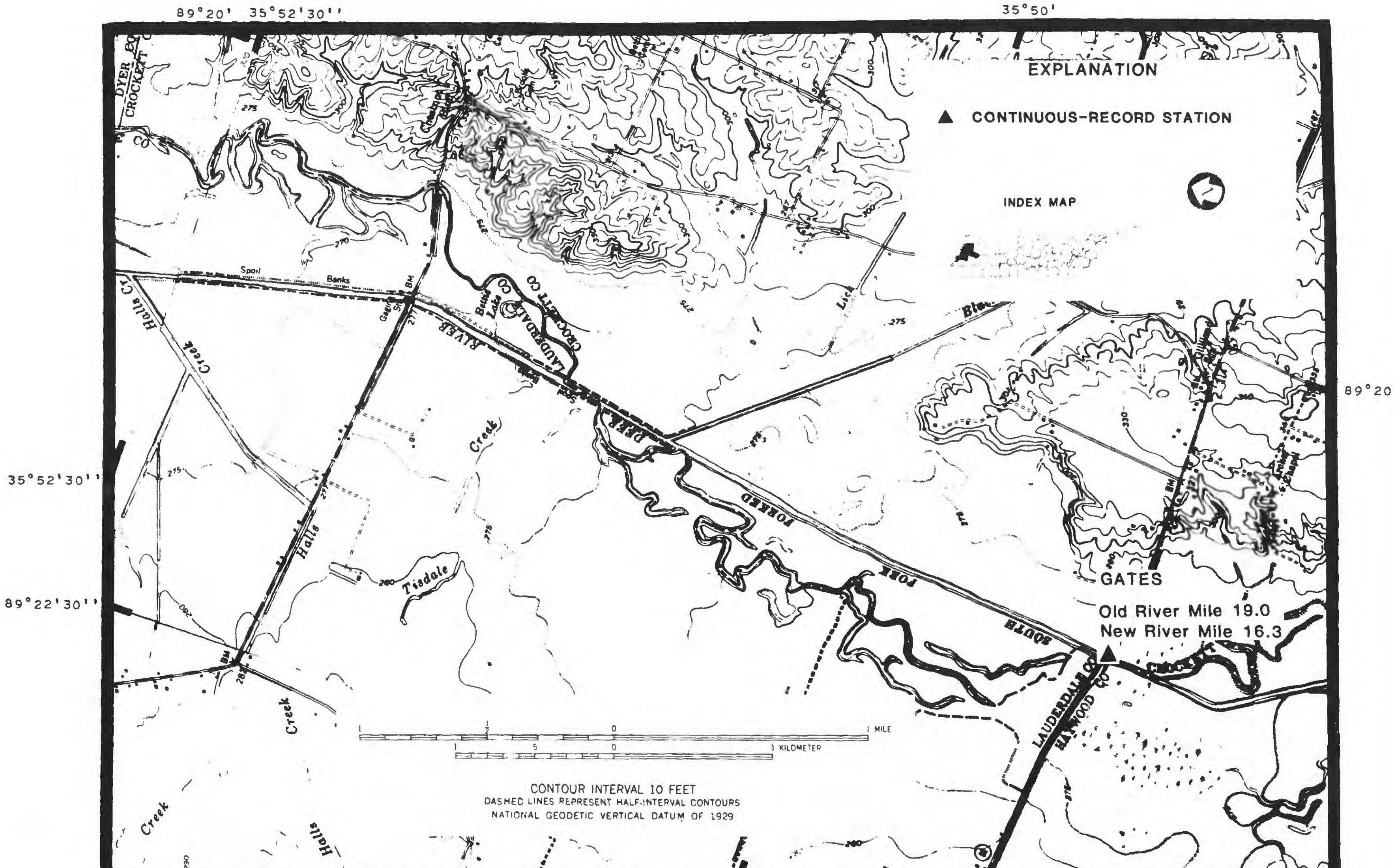
Base from U.S. Geological Survey
Bonicord, 1:24,000, Dyersburg, 1:24,000, Fowlkes, 1:24,000, 1952

Figure 21.--Location of Yellow Bluff gaging station, bridge-failure site near Halls, and channel work in western part of South Fork Forked Deer River, Tennessee.



Base from U.S. Geological Survey
Bonicord, 1:24,000, and Fowlkes, 1:24,000, 1952

Figure 22.--Location of Halls gaging station in middle part of South Fork Forked Deer River, Tennessee.



Base from U.S. Geological Survey
Bonicord, 1:24,000, Chesnut Bluff, 1:24,000,
Gates, 1:24,000, 1952

Figure 23.--Location of Gates gaging station in eastern part of South Fork Forked Deer River, Tennessee.

Between 1964-66, the U.S. Army Corps of Engineers (USCE), enlarged and straightened the main-stem Forked Deer River from the confluence of the North and South forks of the Forked Deer River (old river mile 13.6) to its confluence with the Obion River (river mile 0.0). Between 1968-69, the U.S. Army Corps of Engineers (USCE) modified the lowest 7 miles of the South Fork Forked Deer River channel that had previously been left untouched. The channel was deepened by approximately 10 feet, widened by approximately 50 feet, and shortened by 2.3 miles when straightened by seven large cutoffs (fig. 21). At the upstream end of the work, the modified channel bed was 12.1 feet below the existing channel bed (fig. 24). A 1,000 foot long transition channel, at a gradient of 12.0×10^{-4} ft/ft, was used to equalize the difference between the modified and existing channel. In 1969, two more cutoffs (8 and 9) were constructed by the Dyer County Levee and Drainage District that brought the modified channel to about 2,500 feet downstream of the U.S. 51 bridge (fig. 21). This channel work plus the channel work of the U.S. Army Corps of Engineers shortened the length of the South Fork Forked Deer River by 2.7 miles. The relation between old and new river miles can be seen in figure 24. In 1973, the Obion-Forked Deer Basin Authority (OFDBA) began clearing and reshaping the channel banks starting in the vicinity of the U.S. 51 bridge and working upstream.

Specific-gage and low-pool elevations plotted against time and channel work are shown in figure 25. This figure shows that (1) the South Fork Forked Deer River was steadily aggrading prior to the channel work between 1968-69; (2) the channel bed at Yellow Bluff (old river mile 5.7, new river mile 3.3) was lowered approximately 12 feet in 1969 when the channel work passed through this reach; and (3) that within two years of the channel work at Yellow Bluff, the channel 4.6 river miles upstream at Halls (old river mile 10.6, new river mile 7.9) began to degrade.

It is assumed that as the transition slope between the modified and existing channel bed progressed upstream, its gradient flattened due to the non-cohesive nature of the bed material. Degradation rates at Halls and presumably at the U.S. 51 bridge (site 42), 1.6 river miles downstream, averaged 0.87 ft/yr from 1969-81. However, from 1969-70 and from 1973-74 the channel-bed at Halls degraded 2.9 and 3.6 feet, respectively (fig. 25).

Water-surface slope in the channel reach between Yellow Bluff and Halls averaged about 2.53×10^{-4} ft/ft over the 10 years prior to channel work. By the time channelization had been completed in this reach in 1969, water-surface slope had increased 210 percent to 7.84×10^{-4} ft/ft (fig. 26). Between Halls and Gates, water-surface slope increased 114 percent from 1.60×10^{-4} ft/ft in 1969 to 3.42×10^{-4} ft/ft in 1975 due to the steady progression of degradation upstream. Although the 1969 water-surface slope between Halls and Gates is not a long-term average, it does represent a "pre-effected" condition (fig. 25) and thereby serves as a basis for comparison.

The bank clearing and reshaping in 1973 reduced the hydraulic roughness in the channel. The combined effects of reduced hydraulic roughness and increased water-surface slope increased velocity, which in turn increased shear stress in the channel. At a discharge of 1,000 ft^3/s, calculated mean velocity at Halls increased 22 percent from 1.74 to 2.13 ft/s from 1973-74; at Gates (old river mile 19.0, new river mile 16.3), 8 river miles upstream,

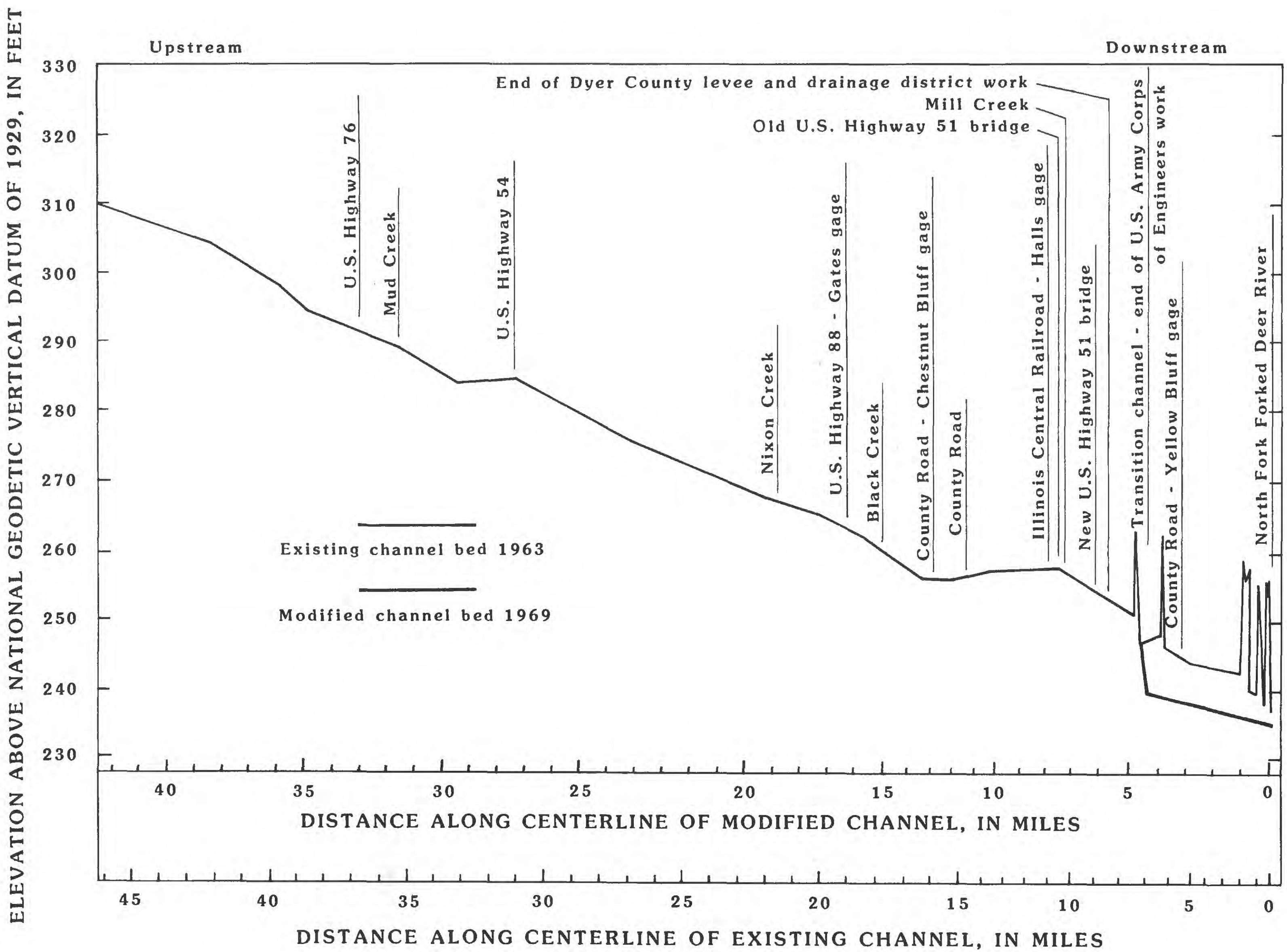


Figure 24.--Channel-bed profile from U.S. Army Corps of Engineers dredging plans showing channel work on South Fork Forked Deer River, Tennessee.

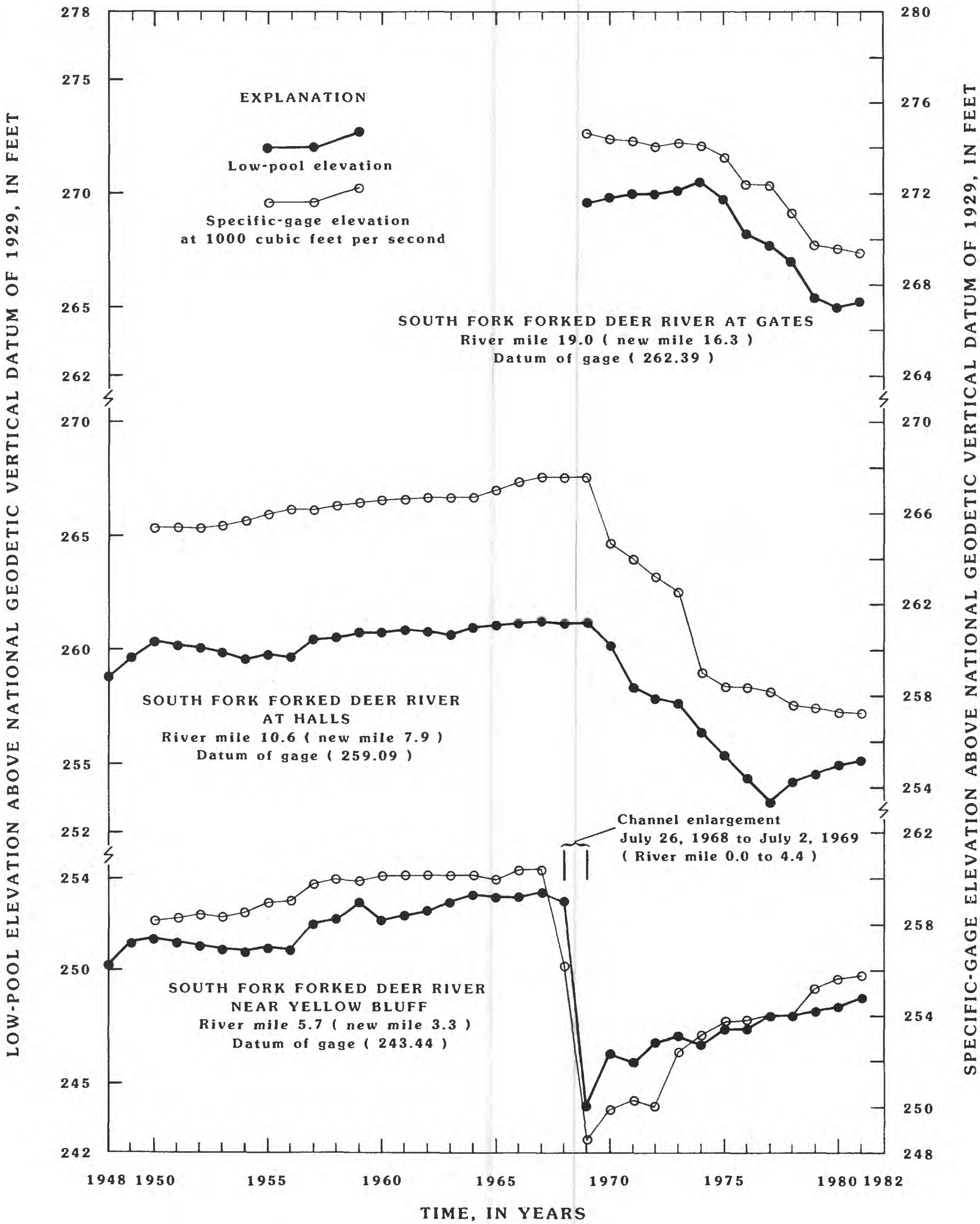


Figure 25.--Specific-gage and low-pool elevations versus time for gaging stations on the South Fork Forked Deer River, Tennessee.

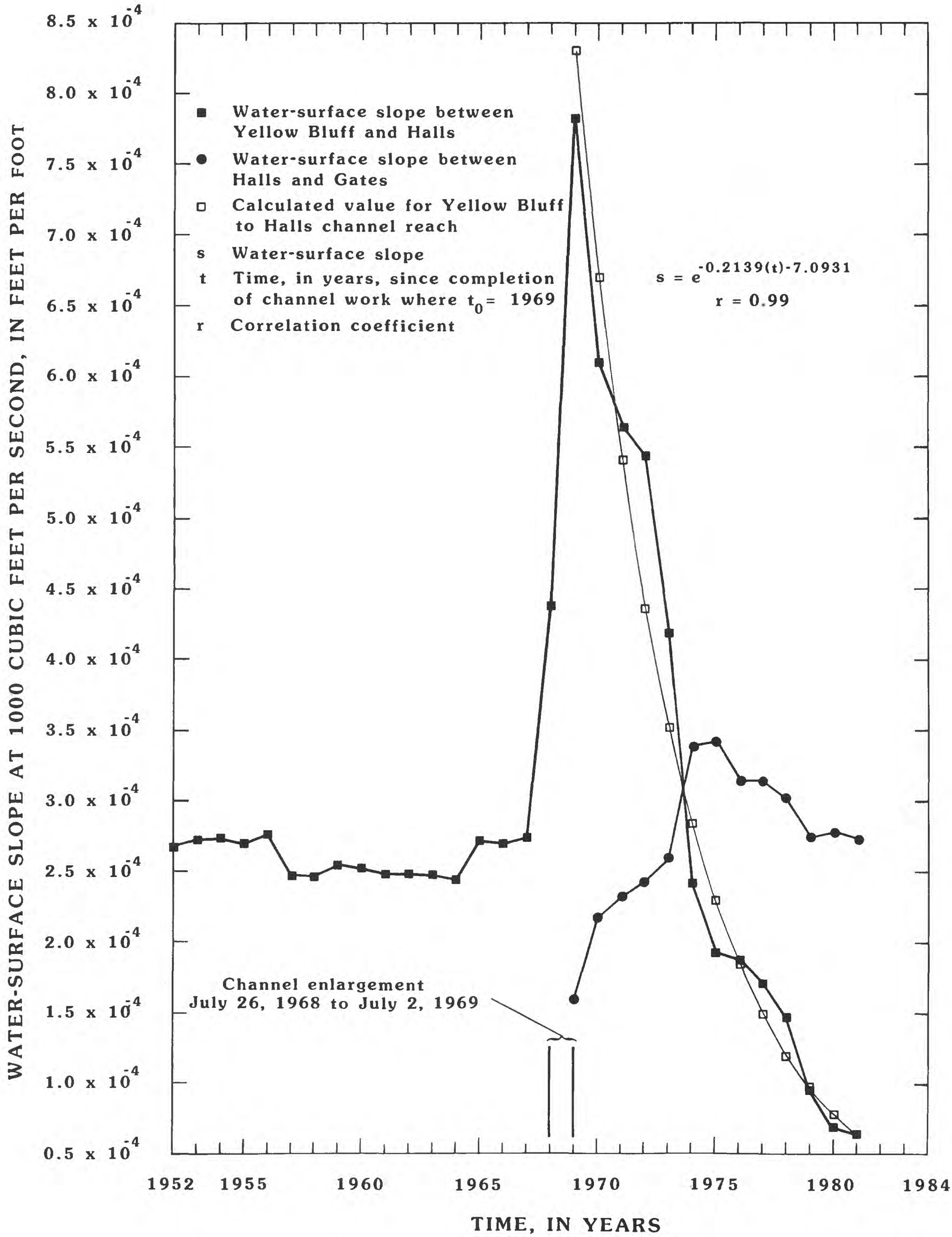


Figure 26.--Change in water-surface slope at a discharge rate of 1,000 cubic feet per second versus time between gaging stations on South Fork Forked Deer River Tennessee.

calculated velocity increased 20 percent from 1.87 to 2.25 ft/s during the same period. The increased erosive power accelerated the downcutting and channel widening, which in turn increased the channel flow capacity.

Two years of above average flow in the basin began in 1973. In December 1973, the northbound lane of the U.S. 51 bridge (site 42) over the South Fork Forked Deer River failed at bent 8. Field inspection after the failure indicated a large scour hole at the bridge. In 1975, both lanes of the bridge were rebuilt with greater distance between piers and deeper footings. Five cross sections taken between 1968-78 at the U.S. 51 bridge (site 42) are shown in figure 27. Channel flow capacity increased 528 percent from approximately 1,640 ft^3/s in 1968 to 10,300 ft^3/s in December 1973. The channel flow capacities were determined by indirect methods using channel areas, slopes, and roughness coefficients. Channel-top width doubled during this same period (fig. 27). Two pictures, taken from the same vantage point, in 1974 and 1981, illustrate the change in channel-top width from October 1974 to March 1981 (figs. 28 and 29). Further evidence of the degradation within the channel is shown in figure 30. Comparing the August 1973 and the 1966 channel-bed profiles, the channel bed at the U.S. 51 bridge (site 42) degraded approximately 12 feet. An additional 6.0 feet of degradation is indicated by the December 1973 channel-bed profile, giving a total degradation of approximately 18 feet.

Plots of stream power (θ) with time at Halls and Gates are shown in figure 31. Stream power reached a maximum at Halls in 1970 followed by a secondary peak in 1972. At Gates, stream power reached a maximum in 1974, which suggests that the channel adjustment was progressing upstream.

The exponential decay function plot (fig. 32) indicates several characteristics of the South Fork Forked Deer River's response to channelization. The first large peak in 1969 (Yellow Bluff to Halls) encompasses channel response from 1967-72 and coincides with the completion of channel work. The 1969 peak represents the increase in channel slope as a direct result of channel straightening. The first wave of degradation at Halls and the consequent aggradation downstream at Yellow Bluff represents the initial channel adjustment (1970-72) in the reach. This peak is substantiated by the stream power plot in figure 31, which shows that the largest values at Halls occurred from 1970-72. The next peak (fig. 32) occurred in 1974 for the Yellow Bluff to Halls and Halls to Gates reaches. Because slope values for both of these reaches are a function of the bed level at Halls, it appears that the peaks for both reaches in 1974 are due to the 3.6 feet of degradation at Halls from 1973-74 (fig. 25) following bank clearing and reshaping. Therefore, the 1974 peaks may represent not only the propagation of the original adjustment upstream, but probably include and are influenced by the additional channel work.

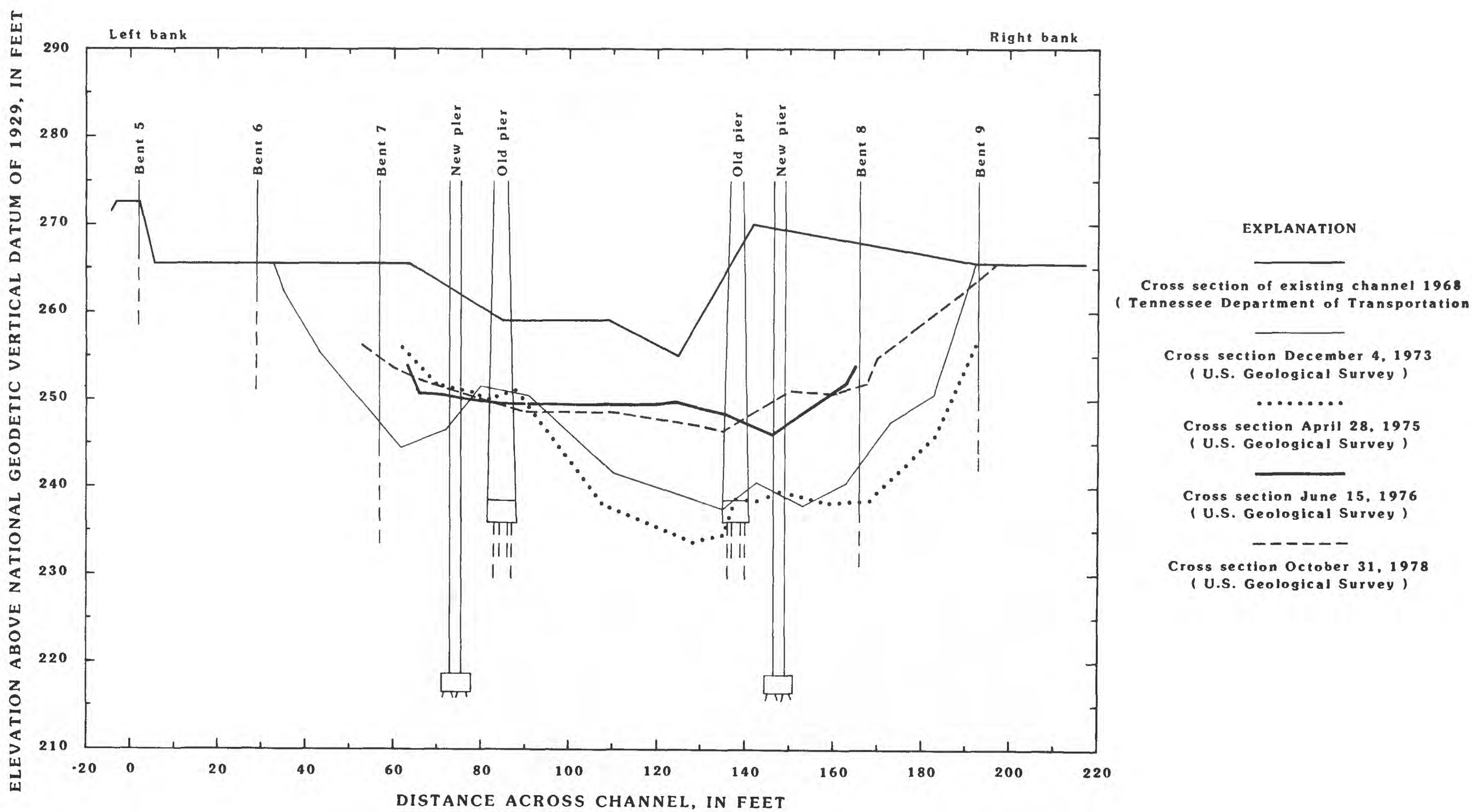


Figure 27.--Stream-channel cross sections at U.S. 51 bridge on South Fork Forked Deer River near Dyersburg, Tennessee.

Photograph by Charles R. Gamble, U.S. Geological Survey

Figure 28. -- South Fork Forked Deer River at new U.S. 51 bridge. View is upstream from bridge, October 10, 1974.

Photograph by Charles R. Gamble, U.S. Geological Survey

Figure 29. -- South Fork Forked Deer River at new U.S. 51 bridge. View is upstream from bridge, March 9, 1981.

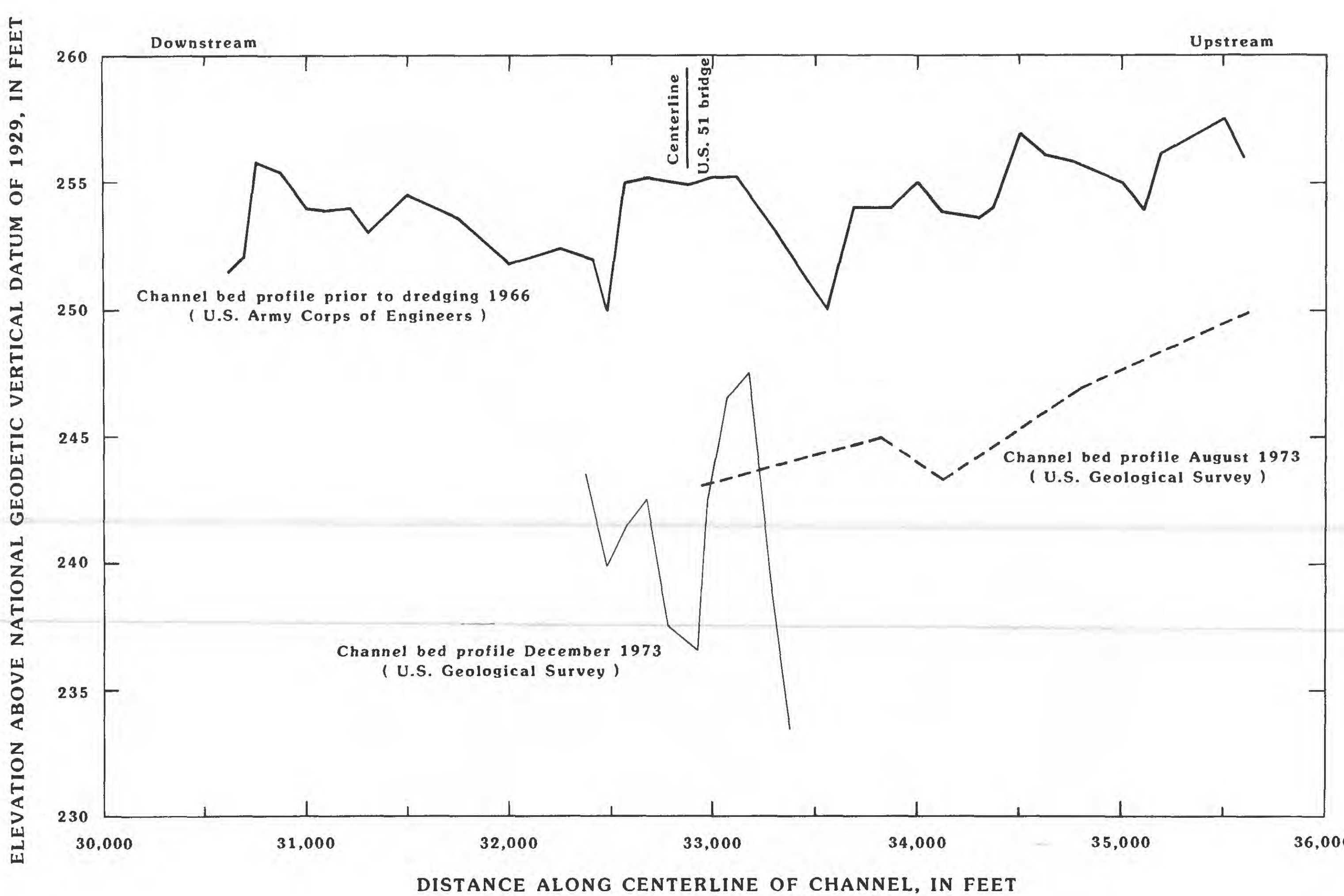


Figure 30.--Channel-bed profiles of South Fork Forked Deer River, 1966-73.

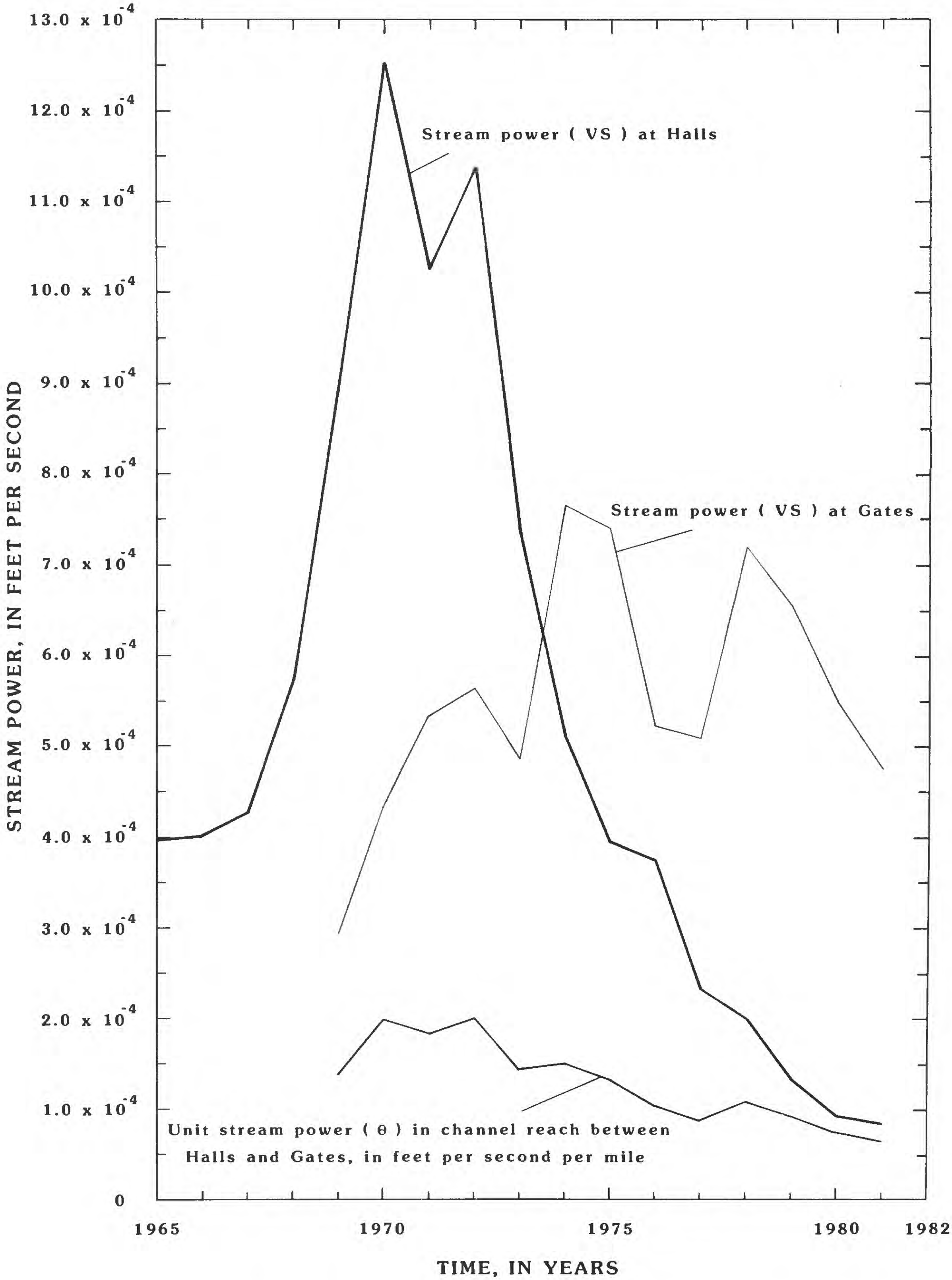


Figure 31.--Stream power versus time for gaging stations on South Fork Forked Deer River, Tennessee.

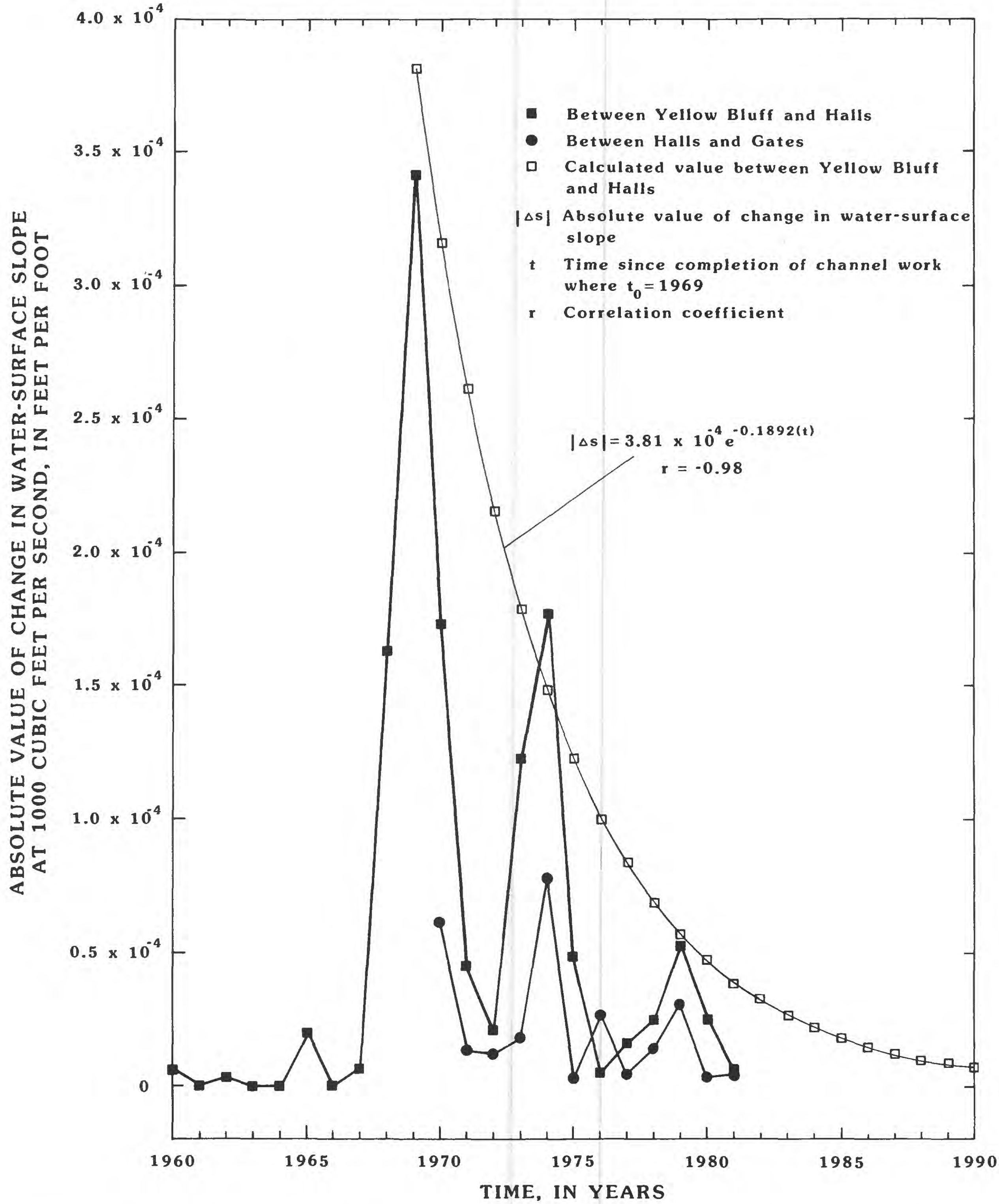


Figure 32.--Exponential decay function describing the yearly variation in water-surface slope fluctuation for two reaches on the South Fork Forked Deer River, Tennessee.

The small peak in 1976 on figure 32 for the Halls to Gates reach is indicative of greater bed-level lowering at Gates than at Halls, thereby initiating a trend of slope reduction in the reach (fig. 25 and 26). The amplitude of the 1976 peak (fig. 32) is smaller in relation to the peaks observed in the downstream reach and could be caused by two factors: (1) the flattening of the transition slope as it progressed upstream; and (2) Gates is 8 river miles further upstream from the point of "maximum" disturbance (the transition slope at new river mile 4.8, old river mile 7.4) than Halls, and therefore is effected less by it. It appears from figure 25 that degradation began at Gates between 1974-75 but was masked in figure 32 by the 3.6 feet of bed lowering at Halls, which was due to an additional disturbance. Therefore, if the primary adjustment was at Halls in 1970 and at Gates in 1975, degradation was progressing upstream at a rate of 1.62 miles per year.

Almost all of the factors contributing to the failure of the new U.S. 51 bridge in 1973 resulted from man-induced channel instabilities. An exception is the extended periods of high flow in 1973 that kept the channel banks saturated.

Channel-bed profiles for 1966 and May and December 1975 are shown in figure 33. The 1975 channel bed was lower than that of 1966, but the channel reaches below the new U.S. 51 bridge had begun to aggrade and channel irregularities were beginning to smooth out. The cross section from April 28, 1975, (fig. 27) indicates that further downcutting and channel widening had occurred since December 1973. However, as the disturbance proceeded upstream in the form of degradation, aggradation became the dominant trend downstream between Yellow Bluff and the new U.S. 51 bridge. Cross sections taken on June 15, 1976, and October 31, 1978 (fig. 27) indicate that the channel in the vicinity of the new U.S. 51 bridge aggraded approximately 14 feet between April 1975 and October 1978. This aggradation effectively reduced channel flow capacity 43 percent to approximately 5,900 ft^3/s. The specific-gage data for Yellow Bluff (fig. 25) shows that this site aggraded 7.1 feet from 1969-81 at a rate of 0.60 ft/yr. This accumulation represents a 60 percent recovery in bed level since 1969, indicating the river's natural tendency to adjust its gradient to the altered environment.

Channel adjustment by concurrent upstream erosion at Halls and Gates and downstream deposition at Yellow Bluff effectively reduced water-surface slope. By 1981, water-surface slope had been reduced approximately 1,100 percent from the maximum of 7.84×10^{-4}ft/ft that was reached in 1969, to 0.64×10^{-4} ft/ft, for the reach between Yellow Bluff and Halls. The water-surface slope in the Yellow Bluff to Halls channel reach as of 1981 was 280 percent less than its pre-channelized water-surface slope. This indicates that a shortened channel requires less gradient to accomplish the same amount of work.

The time distribution of slope variation as the degradation progressed upstream is shown in figure 26. Sum of slopes plotted against time (fig. 34) quantify the overall response of the two reaches and depicts the steady reduction in water-surface slope between Yellow Bluff and Gates. This overall trend in water-surface slope reduction continued even though channel gradient increased between 1969-75 in the upstream channel reach as accelerated erosion was taking place. The river's tendency to return to an equilibrium condition is indicated in figures 26 and 27. Regressing the water-surface slope data

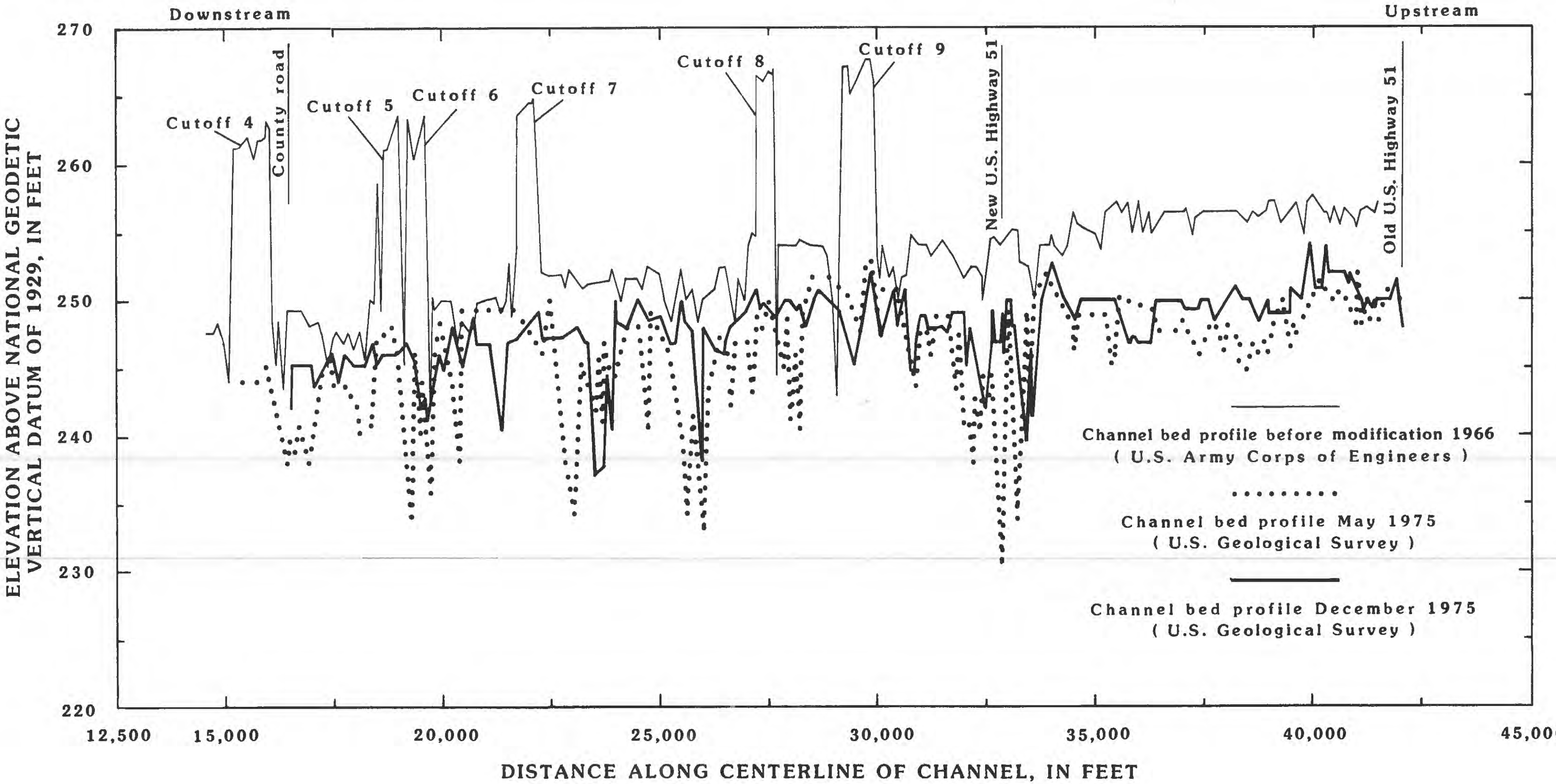


Figure 33.--Channel-bed profiles on South Fork Forked Deer River from confluence with North Fork Forked Deer River to old U.S. 51 bridge, 1966-75.

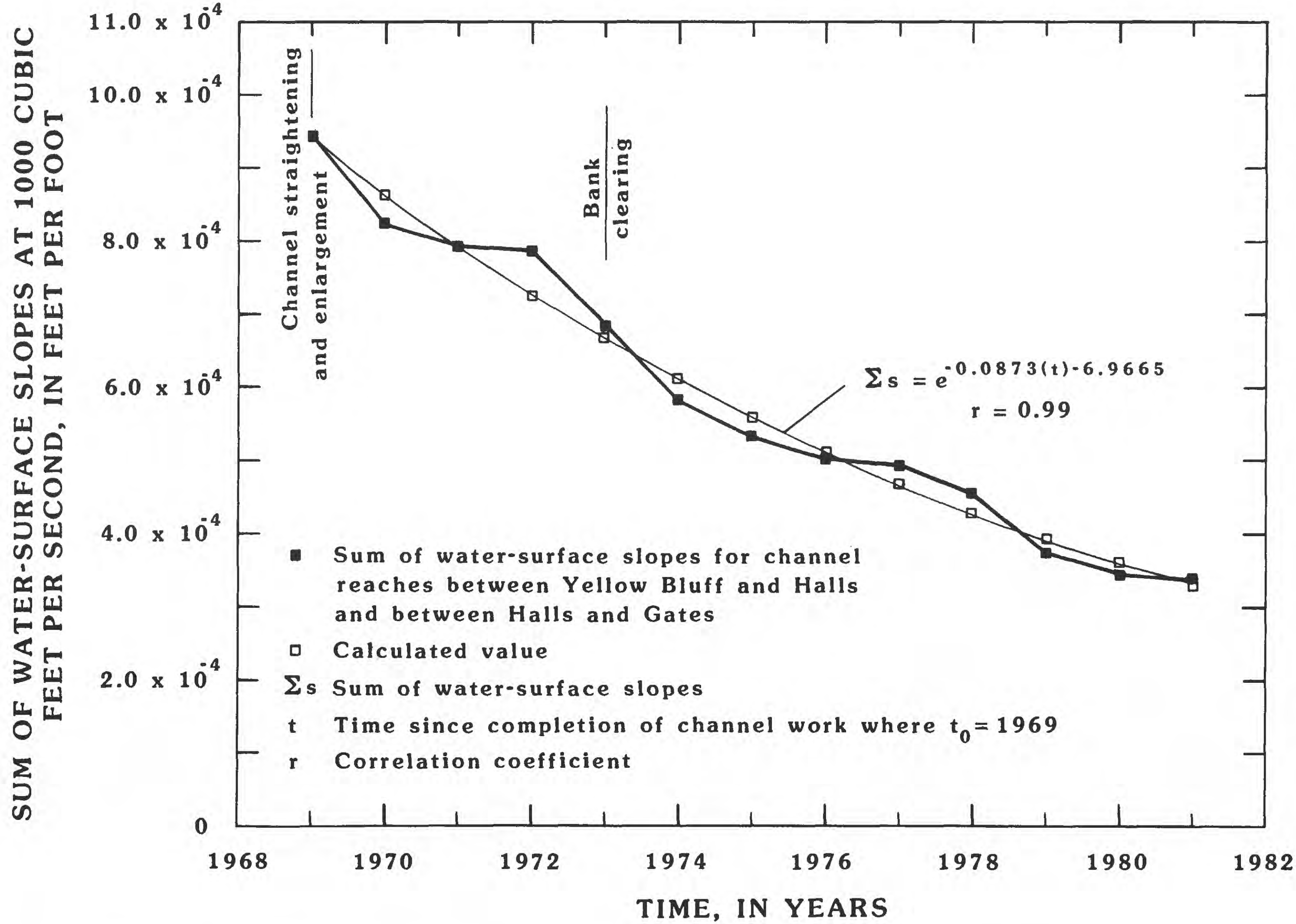


Figure 34.--Sum of water-surface slopes at a discharge rate of 1,000 cubic feet per second versus time between Yellow Bluff and Gates, Tennessee.

for the Yellow Bluff to Halls reach against time yields a very good relation [correlation coefficient (r) = -0.99]. This relation with its associated equation is shown in figure 26, and the equation is also given below.

$$S = e^{-0.2139(t)-7.0931} \tag{14}$$

where

S = water-surface slope, and
t = time in years since completion of channel work where t_o = 1969.

Solving equation 14 for time, t, and substituting 2.53×10^{-4} ft/ft (average water-surface slope at 1,000 ft^3/s in this reach prior to channelization) for slope, S, the calculated length of time for the reach to adjust to this gradient is slightly over 5 years, or until 1974. Actually, a water-surface slope of 2.40×10^{-4}ft/ft was attained in 1974 in this reach. Yet, as can be seen in figure 26, slope adjustment continued beyond this value and time. The shortened channel requires a more gently sloping profile than the pre-channelized condition and therefore will continue to adjust its gradient accordingly. As degradation progresses headward, aggradation is expected to become the dominant trend at Halls, and sometime later at Gates. Regressing the sum-of-slopes data against time also yields a good relation [correlation coefficient (r) = -0.99]. This relation with its associated equation is shown in figure 34. The equation can be used to define channel adjustment, or the response time for channel adjustment as follows:

$$\Sigma S = e^{-0.0873(t)-6.9665} \tag{15}$$

where

ΣS = sum of water-surface slopes, and
t = time in years since completion of channel work where t_o = 1969.

Solving equation 15 for t and substituting 4.13×10^{-4} ft/ft (sum of slopes for the two channel reaches between Yellow Bluff and Gates prior to channelization) for ΣS, the response time for the two reaches to adjust to their pre-disturbed gradient profile is calculated to be approximately 10 years, or until 1979. Again, because channel length on the South Fork Forked Deer River was shortened approximately 15 percent, gradient adjustment tends to proceed past the prechannelized profile to a new, lower gradient profile. Therefore, the response time will be somewhat more than the 10 years calculated from equation (15).

Stream power for the two reaches under consideration are trending toward minimum values (fig. 31). Stream power at Halls is presently lower than its prechannelized values. This reflects the straightening and shortening of the channel such that less energy is dissipated by friction along the wetted perimeter and thus stream power is effectively a minimum. Yang and Song (1979) state that stream power will decrease as an alluvial river approaches equilibrium.

The exponential decay function describing the yearly variation in water-surface slope, for the Yellow Bluff to Halls reach, is defined by three peaks encompassing 12 years of data (fig. 32). Regressing these peaks against time yields a very good relation [correlation coefficient (r) = -0.98]. The equation can be used to estimate the adjustment period for the Yellow Bluff to Halls reach. The relation is defined as follows:

$$|\Delta S| = 3.81 \times 10^{-4} (e)^{-0.1892(t)} \quad (16)$$

where $|\Delta S|$ = absolute value of the yearly change in water-surface slope, and

t = time in years since completion of channel work where $t_0 = 1969$

Data obtained from the Hatchie River (non-channelized control basin for this study) suggests that yearly channel-slope variation may reach 4 percent of the average water-surface slope. Applying this 4 percent variation value to the average pre-channelized water-surface slope (2.53×10^{-4} ft/ft) in the Yellow Bluff to Halls reach, a hypothetical "natural" variation of 1.01×10^{-5} (ft/ft)/yr in water-surface slope is obtained.

Solving equation 16 for time, t, and substituting 1.01×10^{-5} (ft/ft)/yr for $|\Delta S|$, an adjustment time of 19 years (1988) is calculated for the Yellow Bluff to Halls channel reach (fig. 32). It is expected that the aggradation phases of adjustment should be completed by 1988 at Yellow Bluff and Halls, thereby bringing stability to the channel reach. It is anticipated that channel adjustment in the Halls to Gates reach will continue into the 1990's and bridges within this reach will be subjected to the same hydraulic and geomorphic forces that contributed to the failure of the U.S. 51 bridge in December 1973.

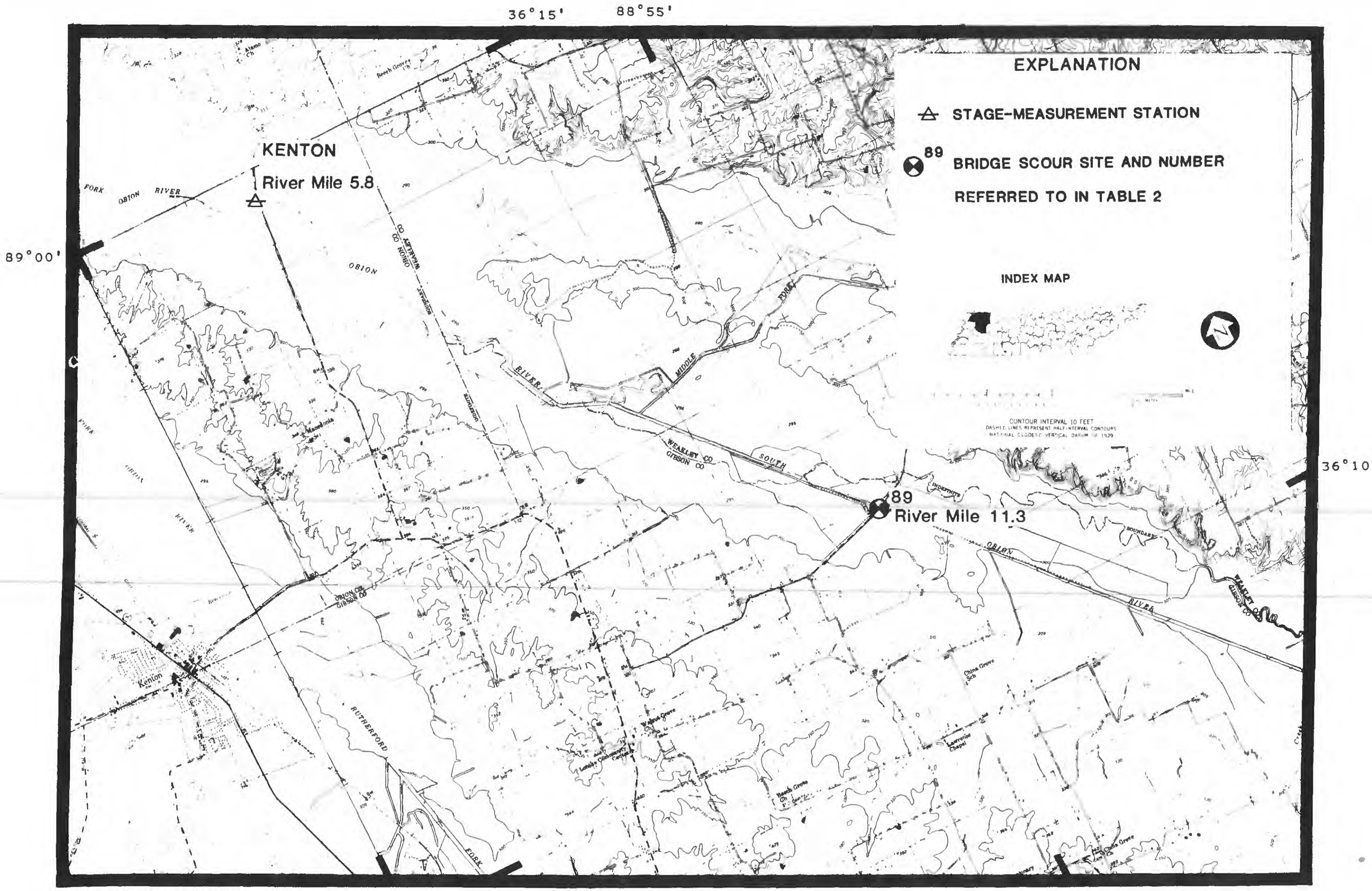
SOUTH FORK OBION RIVER - KENTON TO MC KENZIE

The South Fork Obion River in the study reach (river miles 5.8 to 34.4) prior to channelization followed a sinuous course in a flood plain ranging in width from approximately 1.0 miles at the upstream end of the study reach to approximately 2.0 miles at the downstream end (figs. 35, 36, and 37). A 1912 channel-bed profile (Hidinger and Morgan, 1912) shows a natural channel gradient of approximately 3.60×10^{-4} ft/ft from the confluence of the South Fork and Middle Fork Obion Rivers to the confluence of the South Fork and Obion River main stem.

The South Fork Obion River channel was dredged and straightened in the late 1920's to improve flood-plain drainage. Sections of the channel were dredged again in the late 1940's to recover channel flow capacity lost by filling of the channel by sediment.

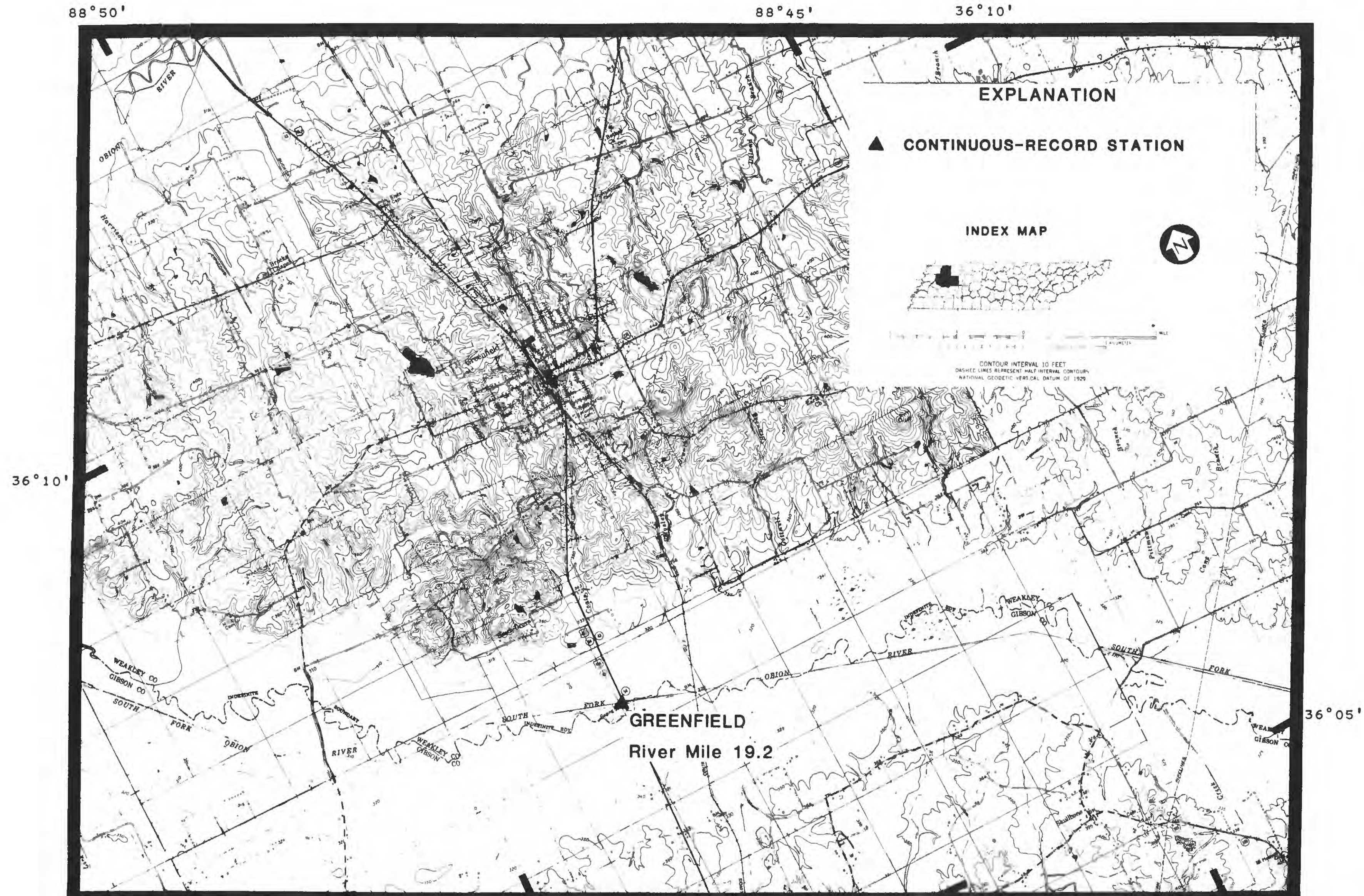
Low-pool and specific-gage elevations at the Kenton (river mile 5.8) and Greenfield (river mile 19.2) gaging stations indicate that the South Fork Obion River aggraded steadily after the major channel work of the 1920's. From 1939-65, the river channel at Kenton aggraded at an average rate of 0.35 feet per year, and from 1929-47 the river channel at Greenfield aggraded at an average rate of 0.56 feet per year. In 1948 the river channel at Greenfield began to degrade at an average rate of 0.09 feet per year in response to the dredging between Kenton and Greenfield in the late 1940's.

During the period 1939-65, water-surface slope at a discharge rate of 500 ft^3/s between gaging stations at Kenton and Greenfield varied from 2.42×10^{-4} ft/ft to 3.21×10^{-4} ft/ft; the average slope for the period was 2.79×10^{-4} ft/ft. This 33 percent fluctuation in water-surface slope indicates that an adjustment of channel gradient due to the channelization in the 1920's and the dredging in the 1940's was still occurring in 1965.



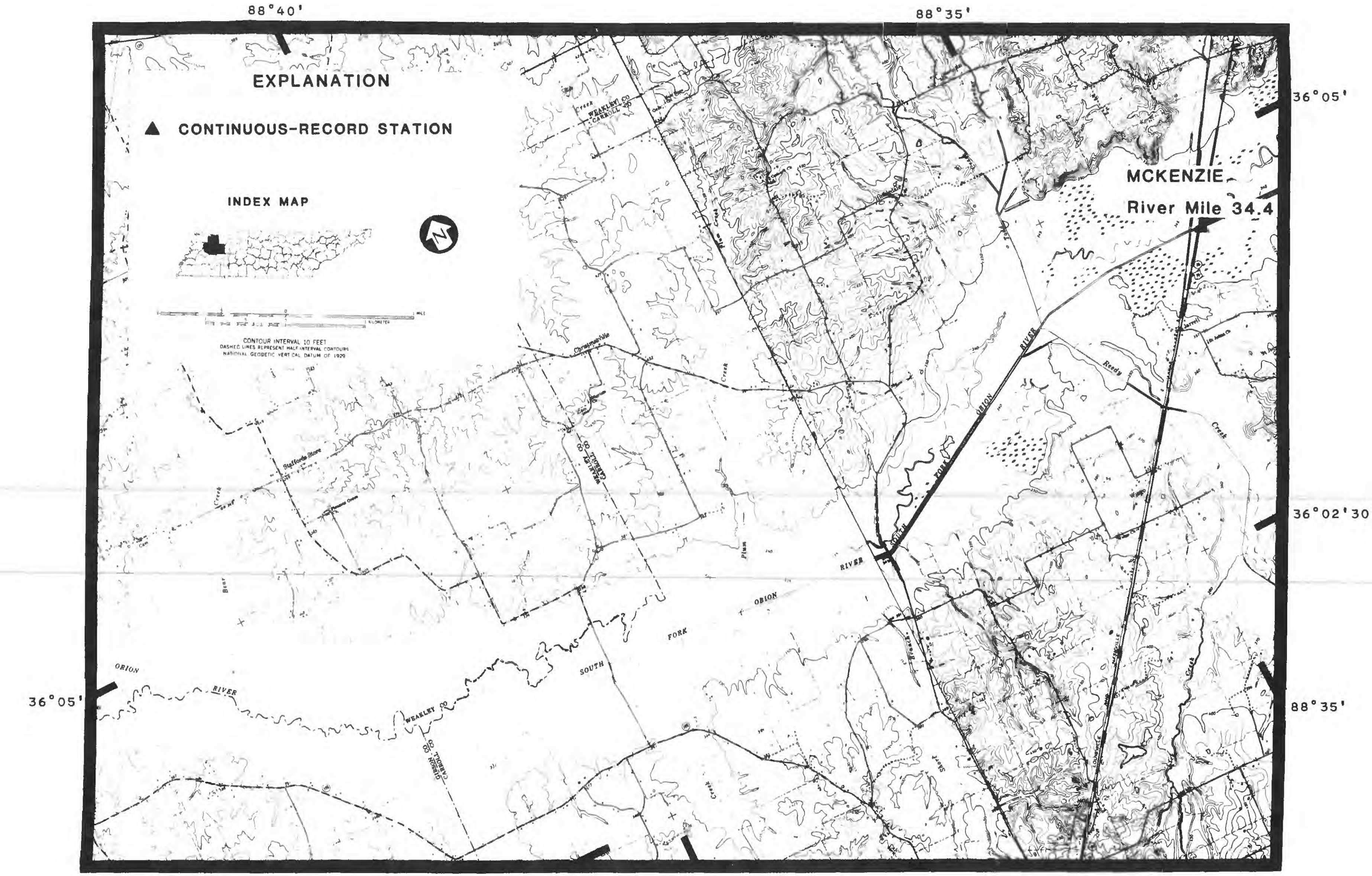
Base from U.S. Geological Survey
Gardner, 1:24,000, 1956, Greenfield, 1:24,000, 1954, revised 1981, Kenton, 1:24,000, 1965, revised 1981, Rutherford, 1:24,000, 1954, revised 1981

Figure 35.--Location of Kenton gaging station and bridge scour site 89 in western part of South Fork Obion River, Tennessee.



Base from U.S. Geological Survey
Dyer, 1:24,000, 1953; Trezevant West, 1:24,000, 1966; Pillowville, 1:24,000, 1967, revised 1980; Bradford, 1:24,000, Greenfield, 1:24,000, Rutherford, 1:24,000, 1954, revised 1981

Figure 36.--Location of Greenfield gaging station in middle part of South Fork Obion River, Tennessee.



Base from U.S. Geological Survey
Trezevant West, 1:24,000, 1966; Trezevant East,
1:24,000, 1967; Pillowville, 1:24,000, 1967,
revised 1980

Figure 37.--Location of McKenzie gaging station in eastern part of South Fork Obion River, Tennessee.

Between 1965-69, the South Fork Obion River channel was dredged from mile 0.0 to mile 6.2 (fig. 38). This work deepened the channel by an average of 6 feet and widened the channel from 60 feet to 100 feet. However, specific-gage elevations (fig. 39) show a local bed-level lowering of approximately 15.5 feet at Kenton (river mile 5.8) from 1965-69. At the upper end of the channel work, a transition channel with a gradient of 12.0 x 10^{-4} ft/ft was constructed to equalize the difference between the modified and existing channel beds.

Water-surface slope, at a discharge rate of 500 ft^3/s, between Kenton and Greenfield increased 76 percent, from 2.61 x 10^{-4} ft/ft to 4.61 x 10^{-4} ft/ft, between 1965-69 (fig. 40). The increase in water-surface slope from dredging resulted in increased stream power and accelerated erosion upstream and deposition downstream.

Four channel cross sections with two sets of velocity measurements at the Greenfield gaging station are shown in figure 41. The channel-top width remained constant at approximately 106 feet from 1938 through 1978. Channel flow capacity also remained constant at approximately 2,000 ft^3/s during the same period. The channel flow capacity was determined by using discharge measurements at bankfull stages.

During the late summer of 1978, snagging in the channel and clearing of the banks was begun above and below the bridge at Greenfield. It is extremely difficult to separate the effects of the downstream dredging from the localized snagging and clearing at Greenfield. Average velocities are assumed to have increased in the vicinity of the clearing due to decreased hydraulic roughness as obstructions to flow were removed. In addition, channel bank stability was reduced by the removal of vegetation. Specific gage at a discharge rate of 500 ft^3/s at Greenfield dropped 2.0 feet from 1978-79 and a total of 3.1 feet by July 1981, indicating that degradation was taking place (fig. 39). The cross sections taken in January 1980 and July 1981 (fig. 41) show that channel widening occurred between December 1978 and January 1980. Discharge measurements indicate that channel flow capacity had increased between the 1978 and 1981 measurements by 30 percent to approximately 2,600 ft^3/s. Three cross sections taken 7.9 river miles downstream from Greenfield at county bridge S-8008 (river mile 11.3) are shown in figure 42. The cross sections show that between June 1977 and January 1980 mean channel depth increased, suggesting that the degradation at Greenfield from 1978-81 was more than a response to local channel disturbance (snagging and clearing) at the Highway 45-E bridge.

The beginning of accelerated erosion at Greenfield in 1978 coincides with, but is only partly attributable to, bank clearing and channel snagging. Water-surface slope between Kenton and Greenfield at a discharge rate of 500 ft^3/s (fig. 40) shows a general decrease after 1975 as sediment from the upstream channel reach was deposited downstream in the vicinity of the Kenton gage. Relative to recent water-surface slope fluctuations in the Kenton to Greenfield channel reach, a decrease in gradient of 0.73 x 10^{-4} ft/ft (16 percent) occurred from 1975-81 as both bed-level lowering and channel-bank widening were occurring at Greenfield. Because degradation, causing over-steepened banks, generally preceeds channel widening (Kirkby and Kirkby, 1968), it is apparent that two sets of effects are at work at Greenfield. First, the river's adjustment from the downstream dredging reached Greenfield during 1979, causing the observed degradation, and second, the concurrent channel widening

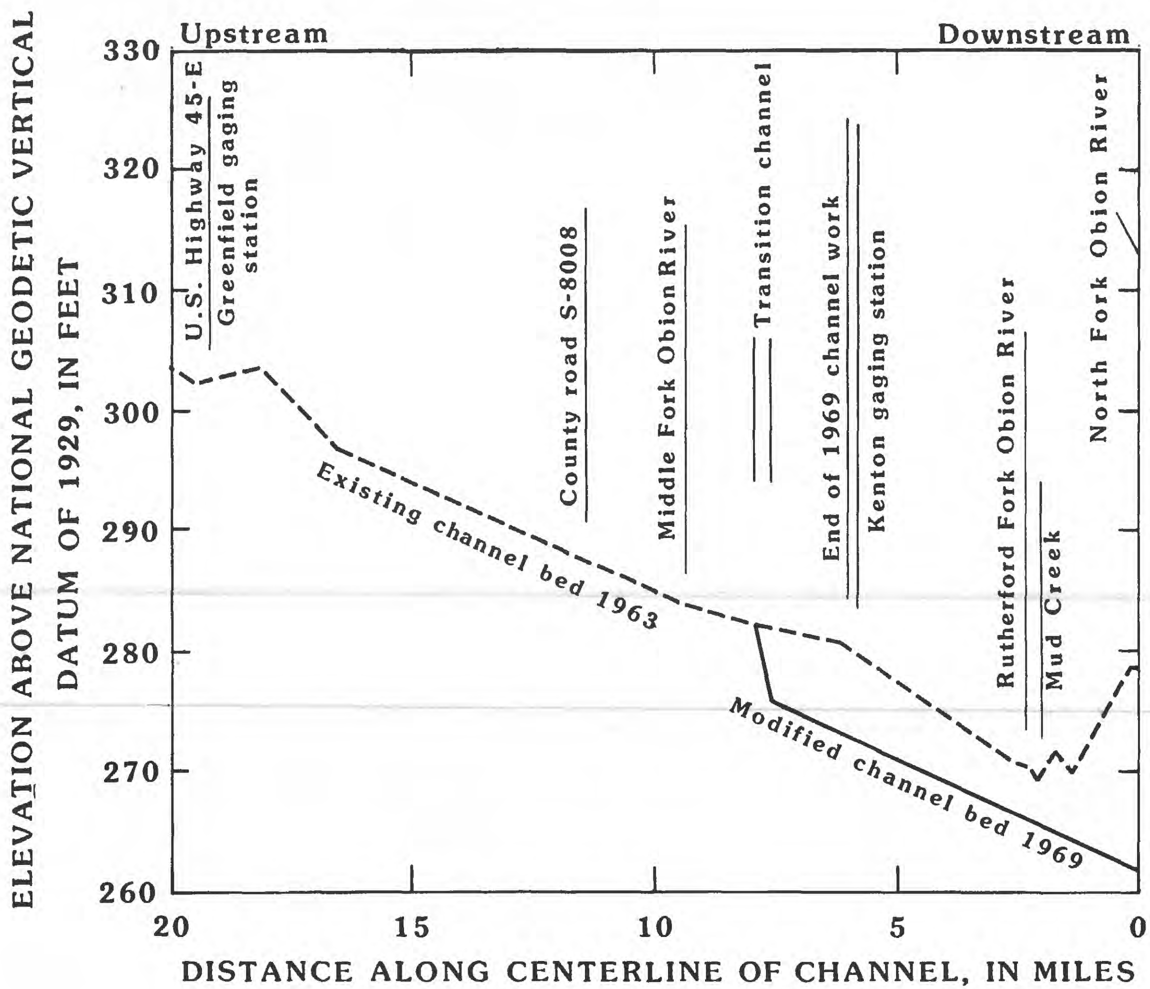


Figure 38.--Channel-bed profile from U.S. Army Corps of Engineers dredging plans showing channel work on the South Fork Obion River, Tennessee.

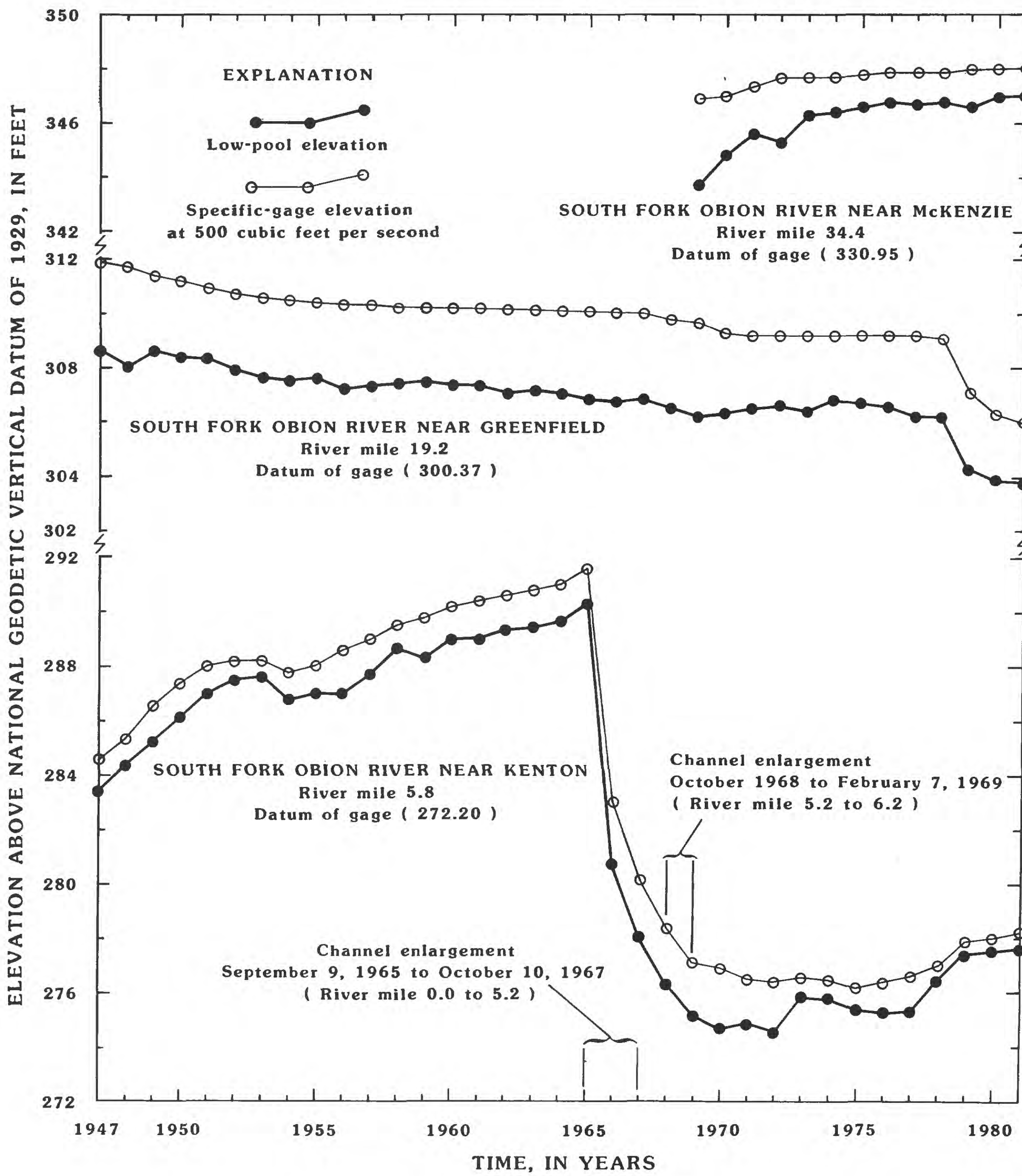


Figure 39.--Specific-gage and low-pool elevations versus time for gaging stations on the South Fork Obion River, Tennessee.

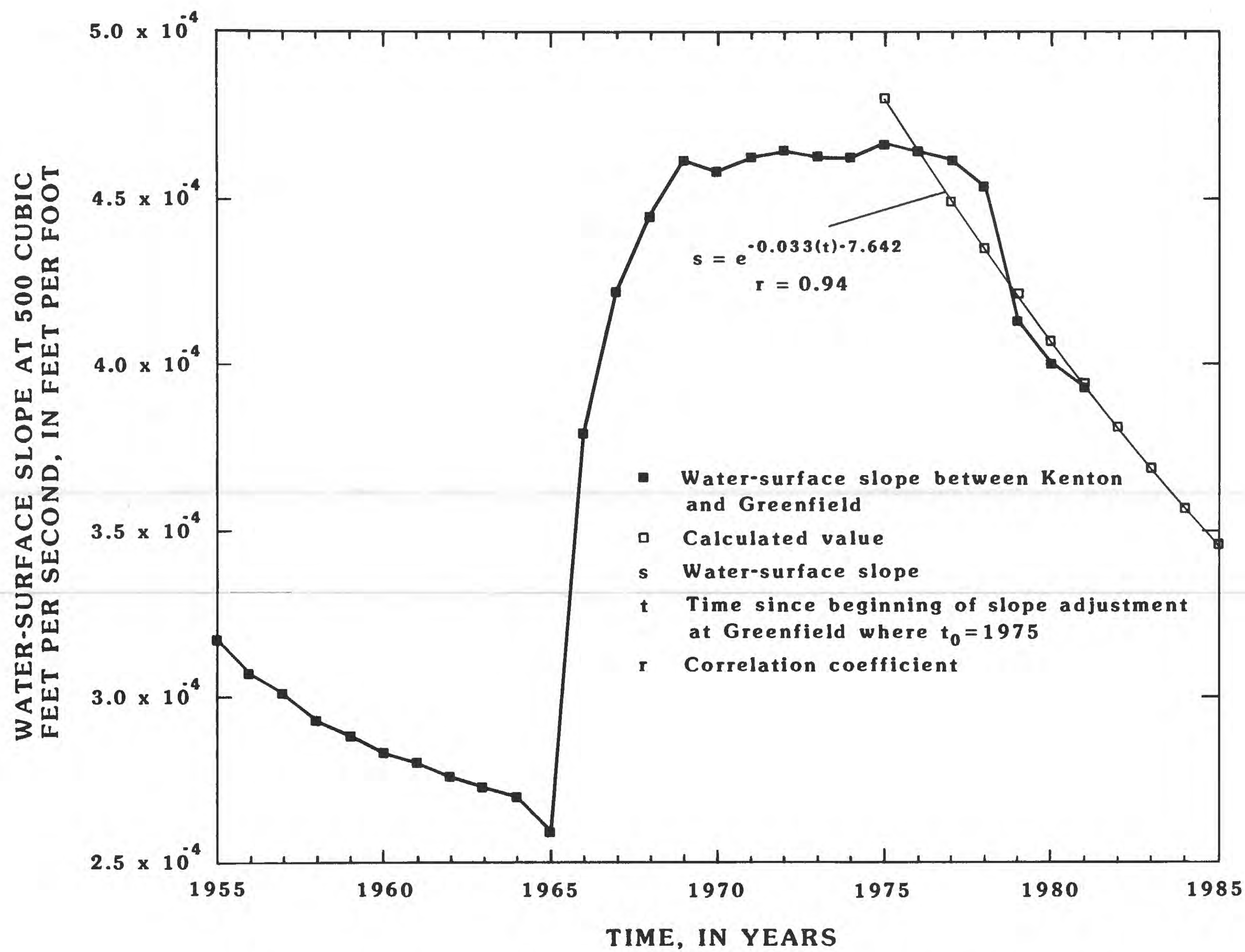


Figure 40.--Change in water-surface slope at a discharge rate of 500 cubic feet per second versus time on the South Fork Obion River between Kenton and Greenfield, Tennessee.

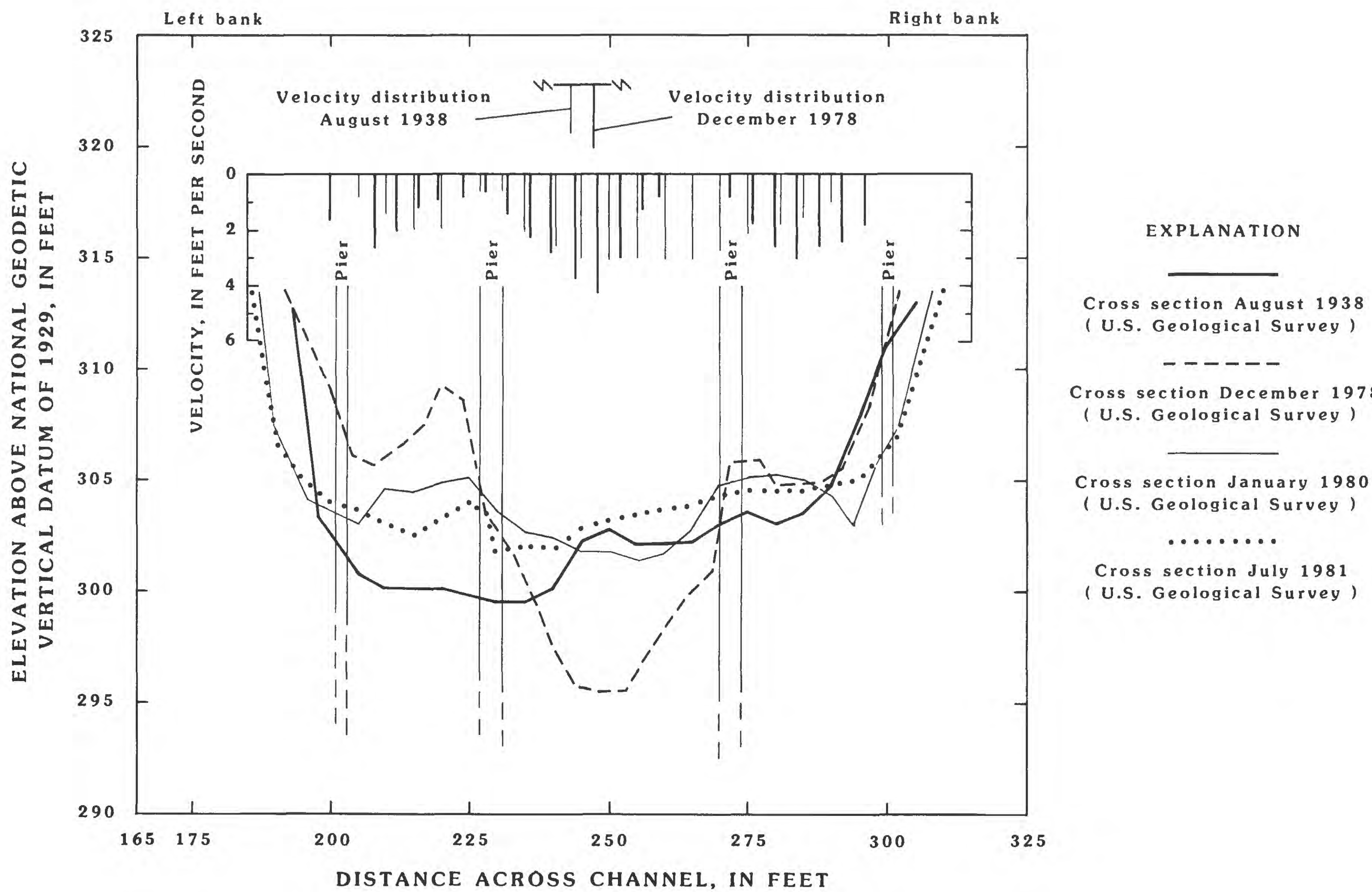


Figure 41.--Stream-channel cross sections showing velocity distributions for South Fork Obion River at State Highway 45-E bridge near Greenfield, Tennessee.

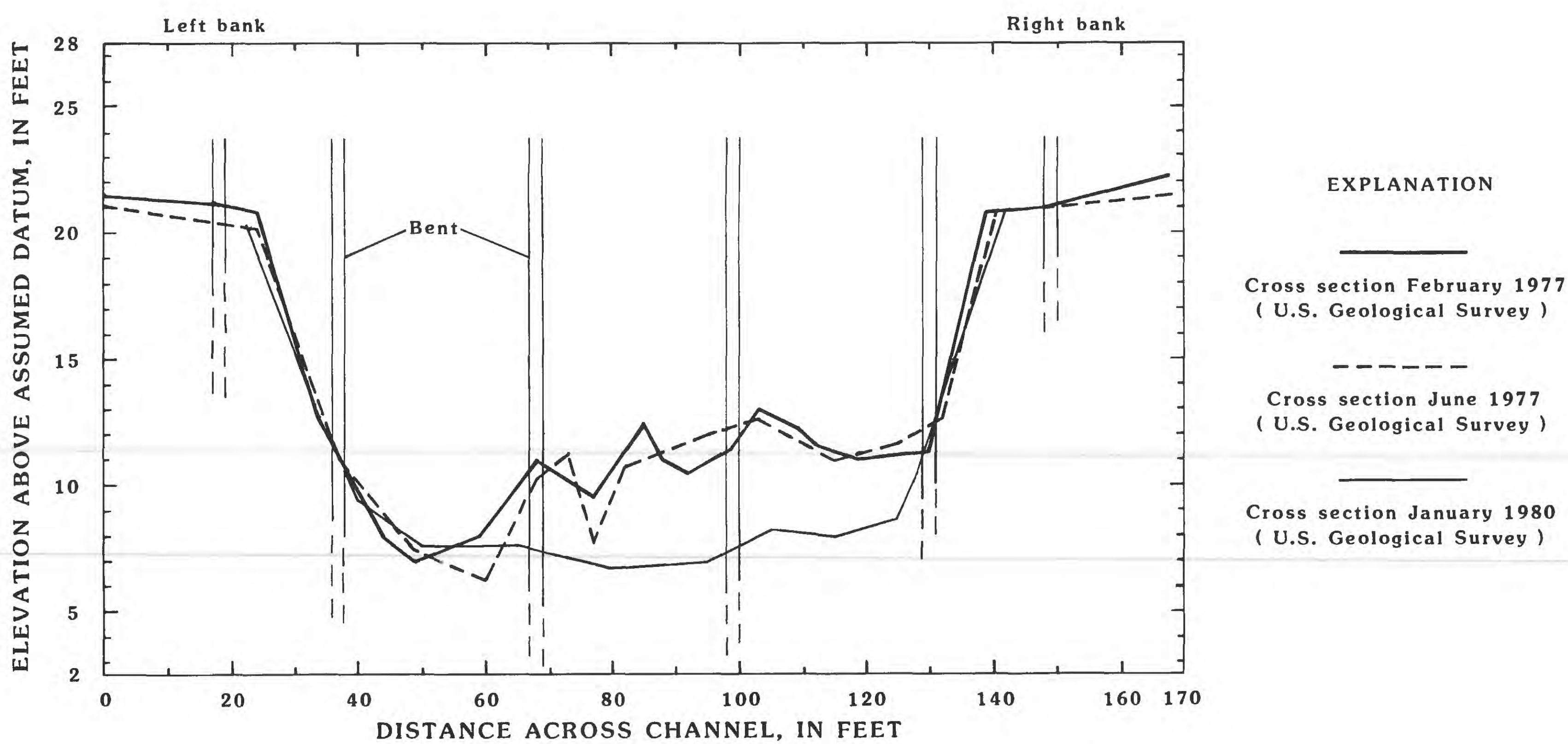


Figure 42.--Stream-channel cross sections for South Fork Obion River at S-8008 bridge near Greenfield, Tennessee.

was partly due to the bank clearing. An aerial photograph of the South Fork Obion River at Highway 45-E, as of July 1979, is shown in figure 43. The scalloped channel banks, most prominent downstream of the bridge, are an indication of large-scale channel instability.

The depth of degradation that will occur at Greenfield cannot be predicted accurately. However, as the channel gradient adjusts, aggradation downstream and degradation upstream is expected to continue until a hydraulically compatible channel gradient is attained. The channel bed at Greenfield will most likely respond according to a function of (1) its distance from the point of the maximum disturbance and (2) the magnitude of the initial disturbance 13 river miles downstream. Within the 13.4 mile channel reach between Kenton and Greenfield, aggradation and degradation are occurring simultaneously (fig. 39). The adjustment to the bed-level lowering from river mile 0.0 to river mile 6.2 took approximately 10 years to advance upstream to Greenfield (river mile 19.2), a rate of 1.3 miles per year (fig. 39). If headward erosion continues at its present rate of 1.3 miles per year, it is expected that degradation will not last longer than 10 years, or until approximately 1990 at Greenfield. Because channel adjustments are rapid at first and then decrease, and knowing that the channel at Greenfield has been degrading at an average rate of 1.0 foot per year (1978-81), the channel bed could be lowered an additional 7 feet, or a total of about 10 feet under present hydrologic conditions.

Assuming that degradation could lower the present streambed elevation at Greenfield approximately 10 feet, general scour due to contraction may cause significant structural problems at the bridge. Using equation 7 and assuming present conditions, it is calculated that channel-bed lowering near the Highway 45-E bridge, owing to local scour around the piers plus degradation, may exceed 20 feet.

$$Y_s = 2.2\, a^{0.65}\, Y_1^{0.35}\, Fr_1^{0.43} \qquad (7)$$

where $Y_1 = 13$ ft
$a = 6$ ft
$V_1 = 3.0$ ft/s
$Fr_1 = 0.147$

Therefore: $Y_s = 2.2\,(6)^{0.65}\,(13)^{0.35}\,(0.147)^{0.43}$

$= 7.60$ ft ← Equilibrium scour depth

$$Y_{max} = 1.3\,(Y_s) \qquad (9)$$

Therefore: $Y_{max} = 1.3(7.60) = 9.9$ ft ← Maximum scour depth

Velocity, depth of flow, and pier width were obtained from discharge measurements at this site.

Estimated depth of degradation (10 feet) plus maximum scour depth (9.9 feet) gives a potential depth of 19.9 feet of bed lowering at the bridge. The additional channel depth due to degradation (10 feet) would give a total bank full depth of water at the bridge (not including scour) of approximately 23 feet. This additional depth and width due to degradation would cause a larger portion of flood water to be contained within the channel.

Photograph from Tennessee Department of Transportation

Figure 43. -- Aerial photograph of South Fork Obion River at State Highway 45-E near Greenfield, Tennessee, showing channel instability on July 31, 1979.

As the hydraulic channel conditions change in response to adjusting channel gradient, the Highway 45-E bridge is expected to be affected by the previously mentioned channel adjustments; (1) channel degradation which may expose the pilings at piers and bents, and (2) channel widening (lateral stream migration), which may undercut parts of the bridge structure. After the degradation cycle passes the Greenfield site, by approximately 1990 under prevailing conditions, aggradation is expected to become the dominant trend.

The slower adjustment rate of 1.3 miles per year in the South Fork Obion River, as compared to 1.62 miles per year in the South Fork Forked Deer River, is probably due to the fact that the South Fork Obion River channel was dredged without alteration to channel length. The difference is that the straightening on the South Fork Forked Deer River resulted in a 210 percent increase in water-surface slope, whereas the dredging on the South Fork Obion River caused a 76 percent increase in water-surface slope and consequently a slower adjustment rate.

The 76 percent increase in water-surface slope from 1965-69 had a larger effect near the Kenton gage than at the Greenfield gage because the transition slope was 0.2 river miles upstream of this gage. The steeper gradient of the transition slope allowed the channel material eroded by adjustment upstream to be transported through the Kenton gage channel reach and deposited further downstream.

Channel adjustment is not observed until 1976 (figs. 39 and 40) because of the location of the two gaging stations (Kenton and Greenfield) relative to the channel modification. In 1976, aggradation in the stream channel started at the Kenton gage and water-surface slope within the channel reach began to adjust. In addition, accelerated erosion at the Greenfield gage in 1978 enhanced water-surface slope change.

Regressing the water-surface slope data for the Kenton to Greenfield reach against time yields a good relation [correlation coefficient (r) = -0.94]. This relation with its associated equation is shown in figure 40. The equation can be used to define the reduction in water-surface slope between Kenton and Greenfield, or to determine the response time for channel adjustment as follows:

$$S = e^{-0.033(t)-7.642} \qquad (17)$$

where

S = water-surface slope at 500 ft^3/s, and

t = time in years since beginning of slope adjustment at Greenfield where t_o = 1975.

Solving equation 17 for time, t, the number of years can be estimated that it will take for the channel reach to attain a particular gradient. Accordingly, it will have required this channel reach approximately 9 years since slope adjustment began at Greenfield in 1975 (15 years from the completion of dredging in 1969), or until about 1984, to reach the USCE design gradient of 3.58×10^{-4} ft/ft. Although this gradient is almost identical to the 1912 profile gradient (3.60×10^{-4} ft/ft), it does not represent a hydraulically compatible gradient under the prevailing conditions. It is more likely that the channel reach will steadily adjust its gradient until an average slope of approximately 2.79×10^{-4} ft/ft (1920's post-straightening average water-surface slope) is achieved. Calculations from equation 17 indicate a 17-year response time at Greenfield or a total of 23 years since the completion of dredging in 1969. Therefore, this gradient is not expected to be attained until approximately 1992. This response time supports the assumption discussed earlier suggesting that degradation is expected to continue at Greenfield until approximately 1990.

NORTH FORK OBION RIVER - RIVES TO PALMERSVILLE

Prior to channelization, the North Fork Obion River in the study reach (river miles 5.9 to 34.9) followed a sinuous course in a flood plain ranging in width from approximately 1.0 miles at the upstream end of the study reach to approximately 2.0 miles at the downstream end (figs. 44, 45, and 46).

The North Fork Obion stream channel was originally straightened and widened in the late 1920's to improve flood plain drainage. The old channel was abandoned and a new channel was dug approximately parallel to the old stream course. Streamflow and channel geometry data are not available for the original natural channel; however, a USGS gaging station was installed at the Highway 22 bridge at Union City in 1929 shortly after completion of the new channel.

From 1938-43, snagging and clearing was performed on the North Fork Obion River. Specific-gage and low-pool elevation records reflect this channel work with a degradation cycle beginning in early 1940 and ending in mid-1947. Channel-bed profiles and cross sections from USCE dredging plans (1963) show a channel gradient of approximately 1.67×10^{-4} ft/ft, an average channel width of 80 feet, and a depth of 8 feet.

According to streamflow records at the Union City gage, bankfull channel capacity was approximately 9,200 ft^3/s in January 1932. Specific-gage and low-pool elevations indicate that the channel dug in the late 1920's began to aggrade shortly after completion. Discharge measurements and cross sections at this site indicate that by 1947 aggradation had effectively reduced the bankfull channel capacity to approximately 5,400 ft^3/s, a decrease of 41 percent. By 1953 the bankfull channel capacity was approximately 4,350 ft^3/s, a total decrease of 53 percent.

The water-surface slope, at a discharge rate of 500 ft^3/s, between gaging stations at Rives and Union City varied from 2.33×10^{-4} ft/ft in 1939 to 4.75×10^{-4} ft/ft in 1965 and averaged approximately 4.09×10^{-4} ft/ft. Between the Union City and Martin gaging stations, water-surface slope, at a discharge rate of 500 ft^3/s, varied from 2.83×10^{-4} ft/ft in 1939 to 4.36×10^{-4} ft/ft in 1965 and averaged approximately 3.81×10^{-4} ft/ft.

In July 1965, channel dredging and clearing was begun. It extended from the mouth of the North Fork Obion River where main-stem channelization was completed in 1966 to river mile 10.9, approximately 0.9 river miles upstream of the Union City gage (fig. 47). According to USCE dredging plans, the channel bed was to be lowered 5.5 feet at Rives and 1.0 foot at Union City (fig. 47). However, specific-gage records (fig. 48) indicate that the channel

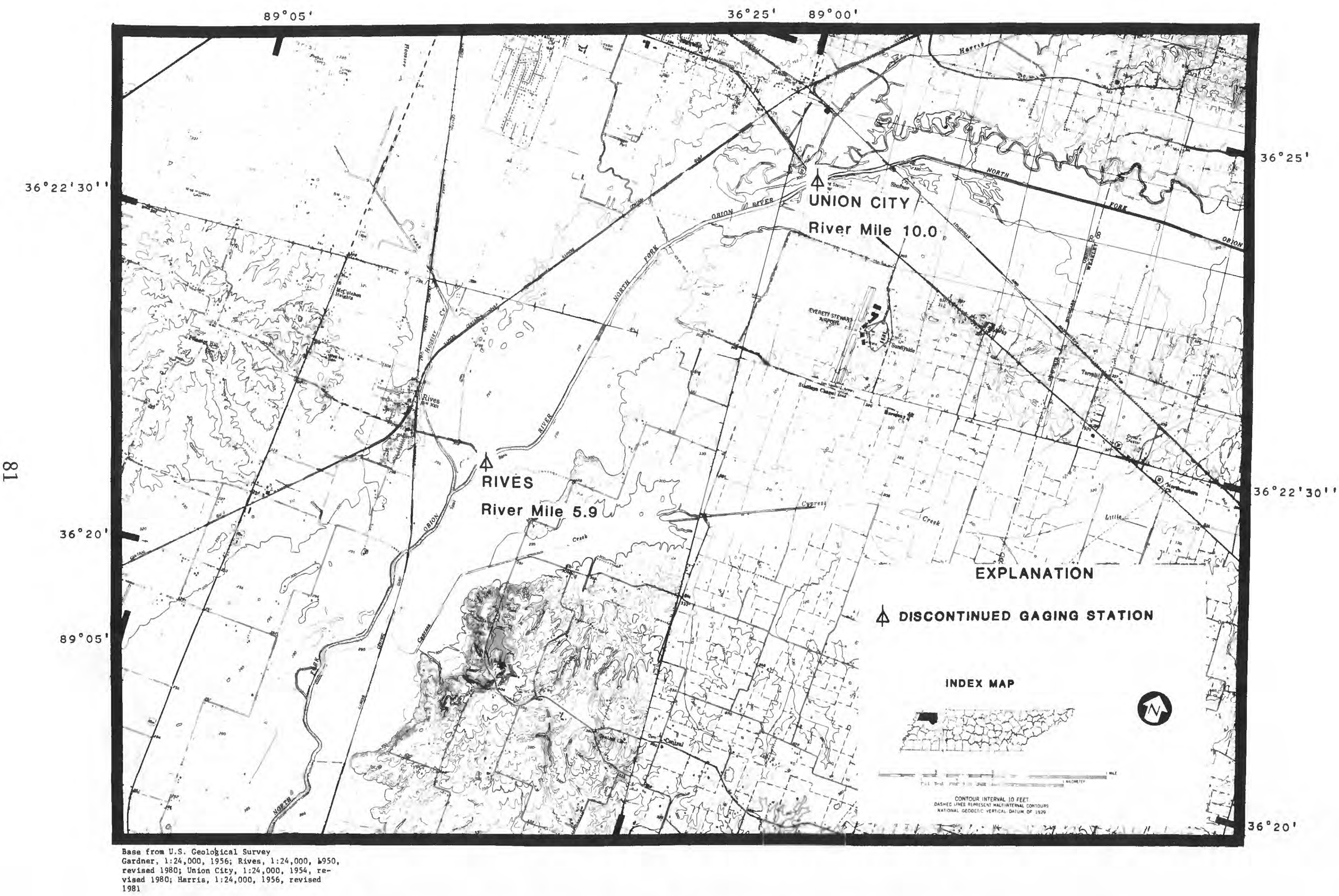
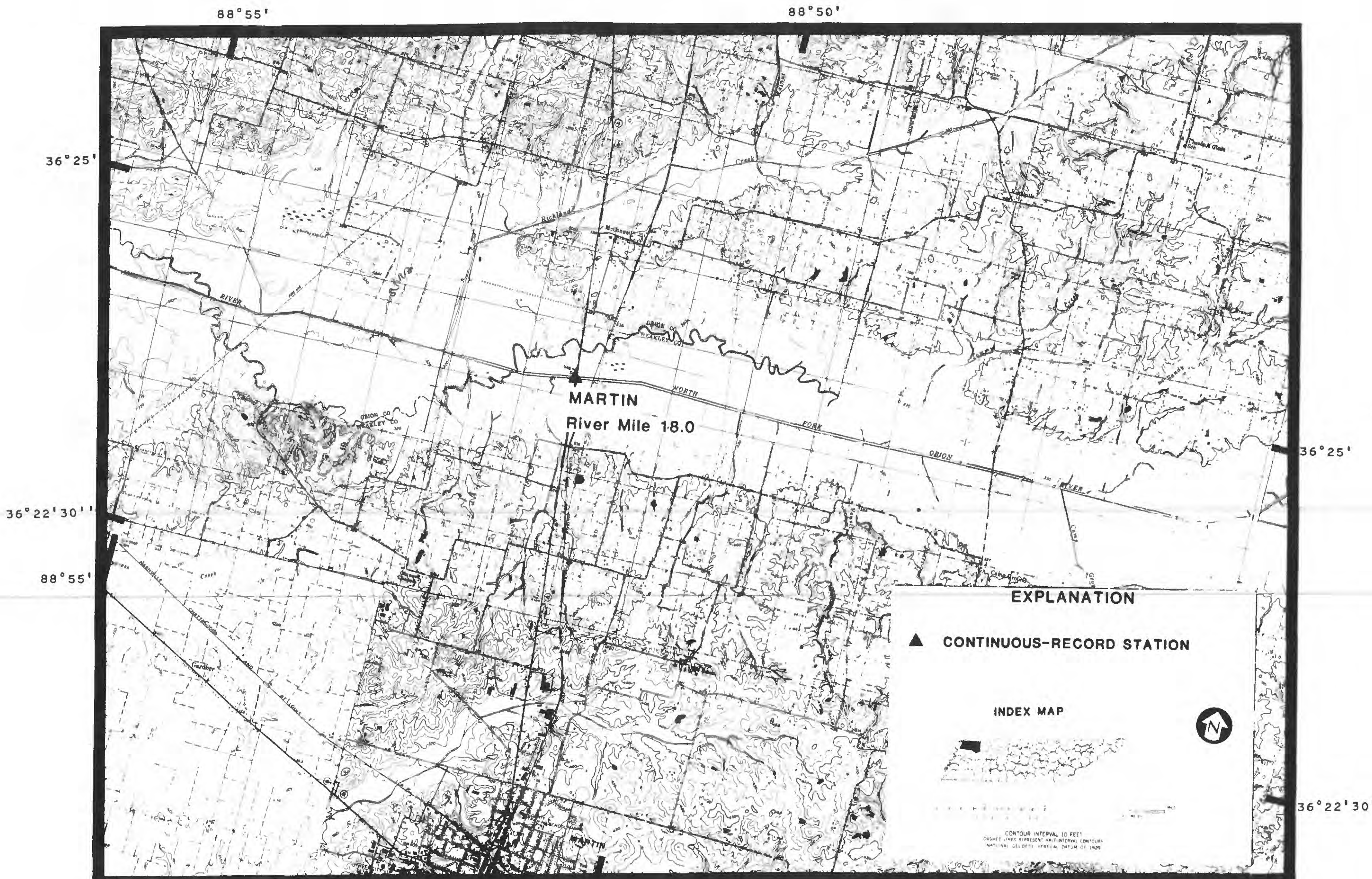
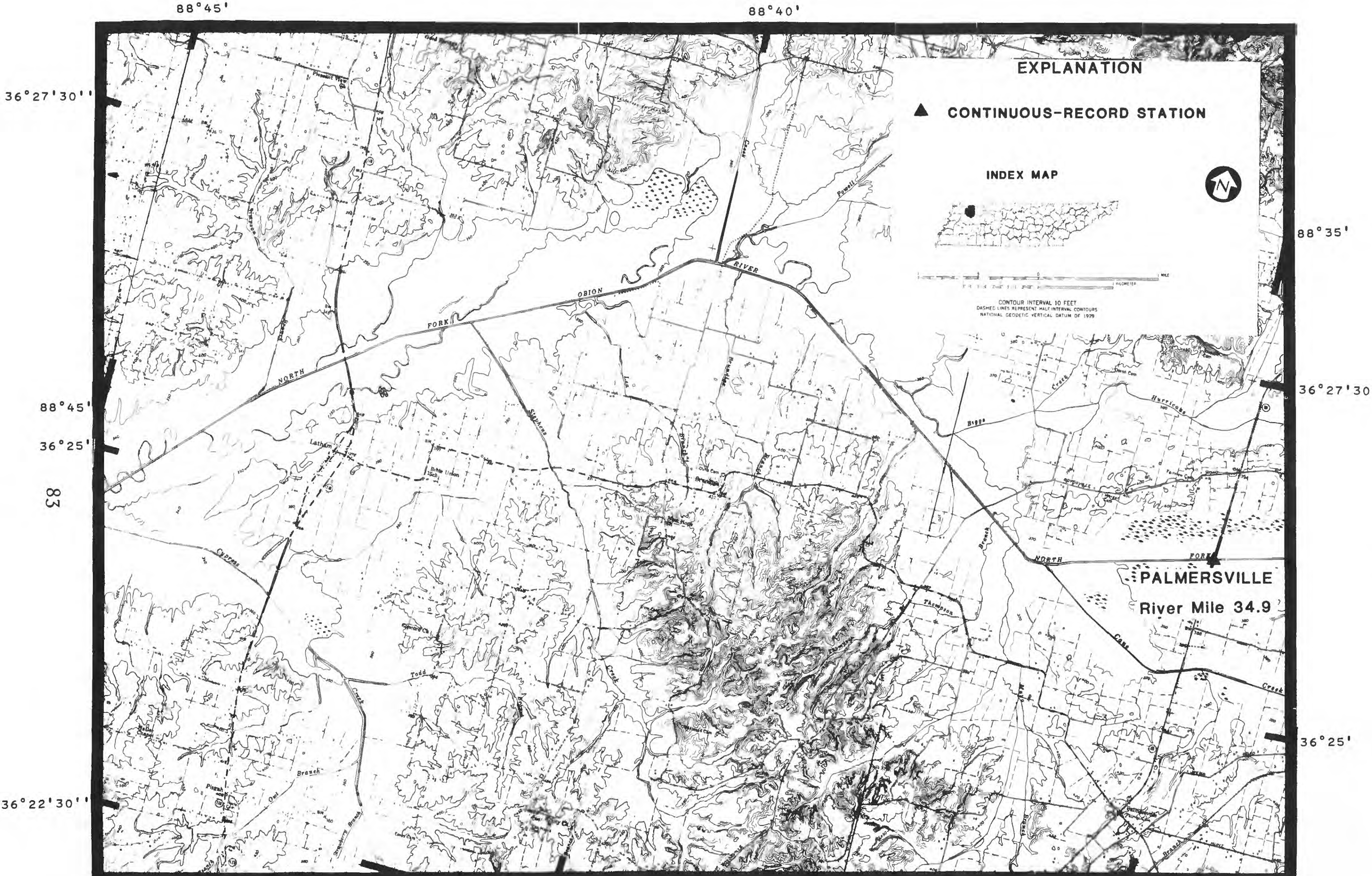


Figure 44.--Location of Rives and Union City gaging stations in western part of North Fork Obion River, Tennessee.



Base from U.S. Geological Survey
Gardner, 1:24,000, Latham, 1:24,000, 1956, Martin,
1:24,000, 1950, revised 1981; Dresden, 1:24,000,
1952, revised 1981; Herris, 1:24,000, McConnell,
1:24,000, 1956, revised 1981

Figure 45.--Location of Martin gaging station in middle part of North Fork Obion River, Tennessee.



Base from U.S. Geological Survey
Latham, 1:24,000, Palmersville, 1:24,000, 1956;
Dreaden, 1:24,000, 1952, revised 1981

Figure 46.--Location of Palmersville gaging station in eastern part of North Fork Obion River, Tennessee.

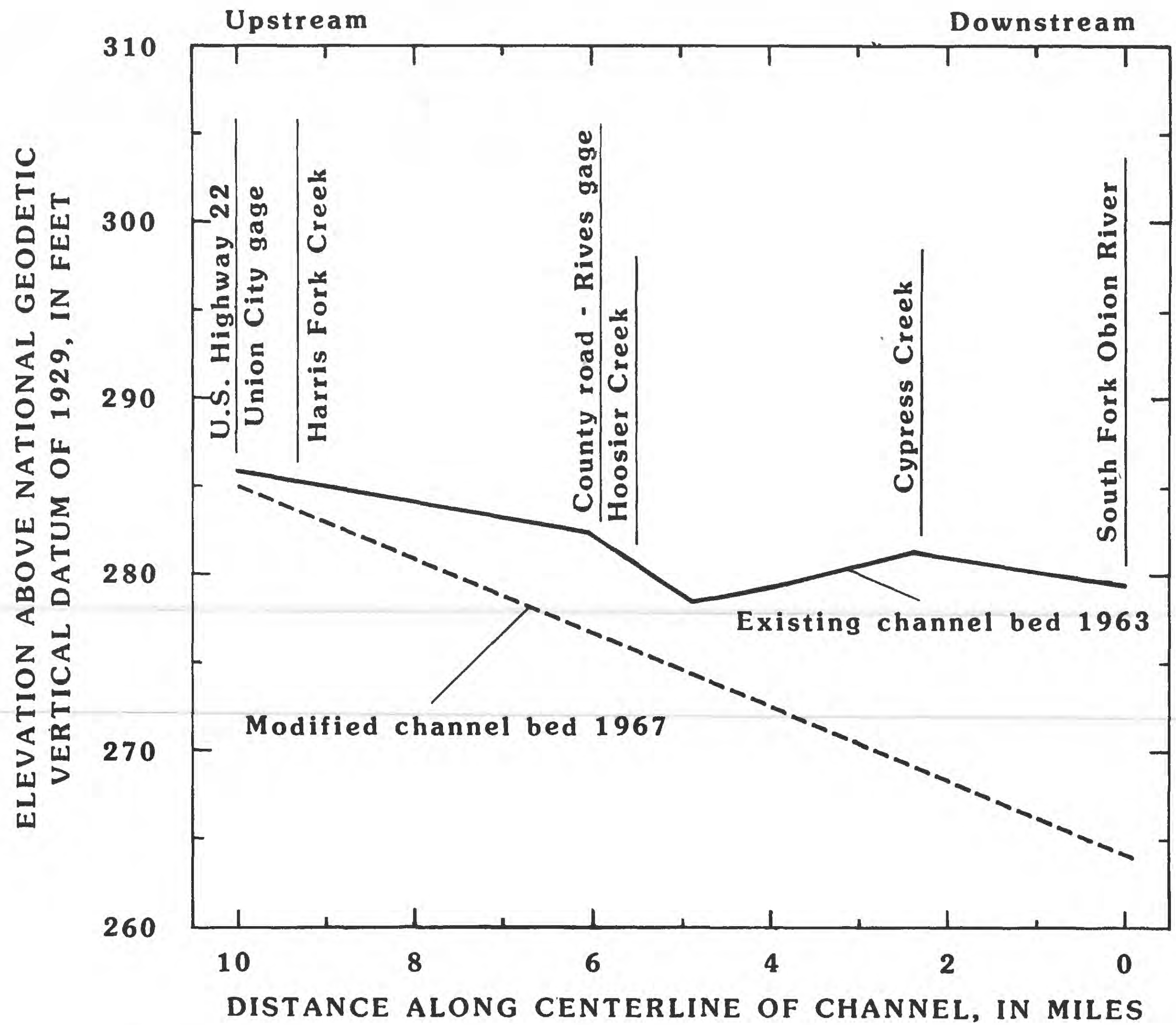


Figure 47.--Channel-bed profile from U.S. Army Corps of Engineers dredging plans showing channel work on the North Fork Obion River, Tennessee.

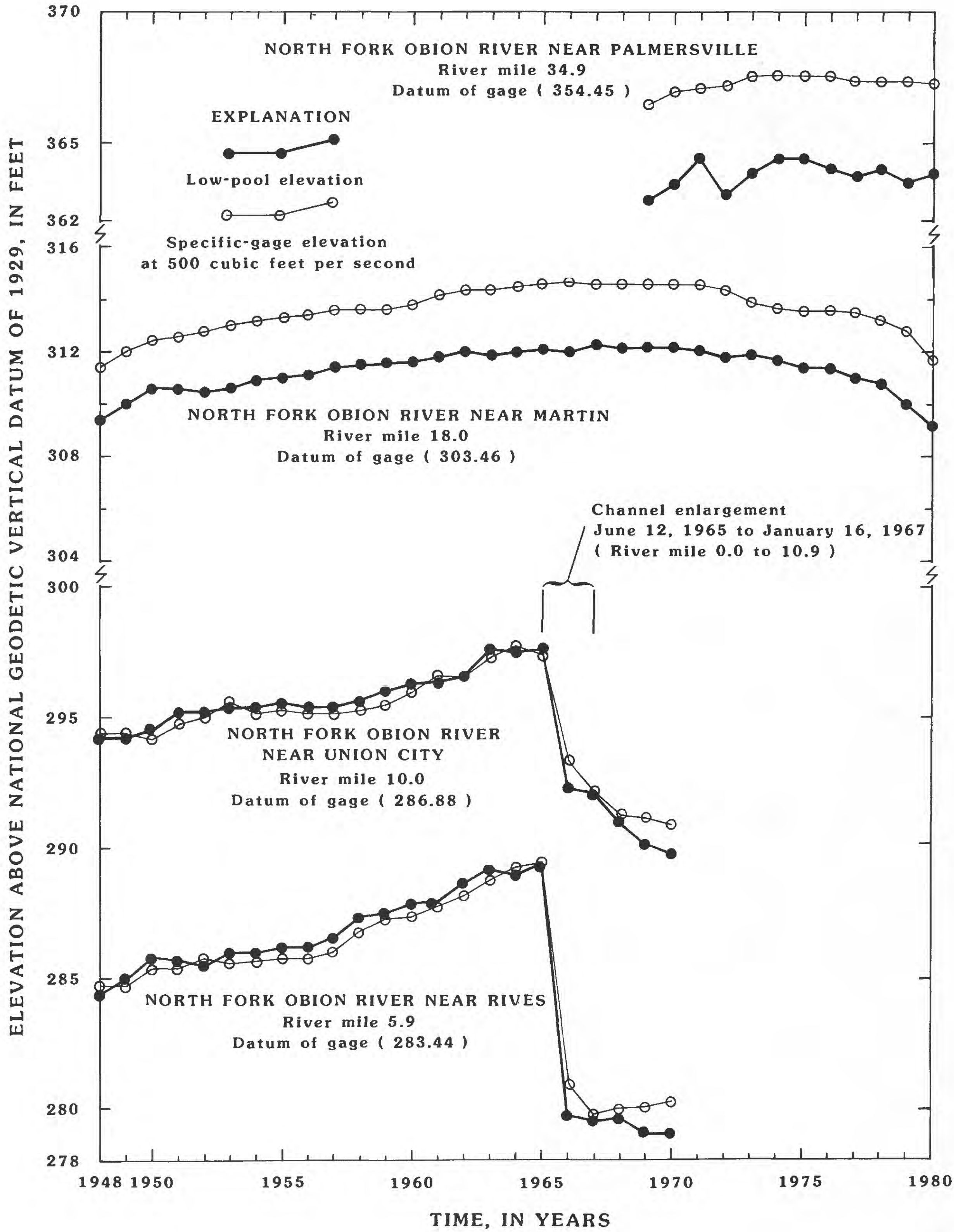


Figure 48.--Specific-gage and low-pool elevations versus time for gaging stations on the North Fork Obion River, Tennessee.

bed was lowered 9.7 feet at Rives and 5.2 feet at Union City by the time channel work was completed in early 1967. As a result, water-surface slope, at a discharge rate of 500 ft^3/s, had increased 52 percent from 3.79 x 10^{-4} ft/ft in 1965 to 5.77 x 10^{-4} ft/ft in 1967 in the channel reach between Rives and Union City (fig. 49).

As reaches of the North Fork Obion River were dredged and cleared, the channel was reshaped with 35-foot bottom widths, 90-foot top widths and an average channel depth of 19 feet. Without the aid of perennial vegetation to stabilize the banks, oversteepening due to downcutting, sloughing, and caving of bank material along the effected reach occurred. By 1981, channel width for 0.25 miles above the Highway 22 bridge near Union City had increased to 150 feet. The net result of the channelization was an increase in channel flow capacity immediately above and below the bridge. From 1969-81, channel flow capacity at the Highway 22 bridge increased 108 percent, from 8,700 ft^3/s to 18,100 ft^3/s. The channel flow capacities were determined by using discharge measurements and indirect methods using channel areas, slopes, and roughness coefficients.

Although a channel cross section taken at a bridge may be unrepresentative of natural conditions owing to the influence of the bridge, sections measured as late as 1974 indicate that channel-bed lowering was occurring at Union City (fig. 50). Field inspections and cross sections taken above and below the Highway 22 bridge indicated the development of a large scour hole immediately downstream of the bridge. Its 1974 dimensions were approximately 300 feet long, 200 feet wide, and 12.5 feet deep. By 1981, it had deepened to 17.5 feet and was extending upstream around the bridge piers (fig. 51). What appears to be additional bed-level lowering in the 1974 and 1981 cross sections (fig. 50) is at least partly attributable to the leading edge of the scour hole as it extended upstream under the bridge.

In 1981, channel flow capacity 120 feet upstream of the Highway 22 bridge was 37,700 ft^3/s. Bankfull streamflow is forced through the two main bridge piers which have a 60 foot opening that had a design flow of 18,100 ft^3/s. The constriction at the piers causes water to "pile-up" on the upstream side of the bridge resulting in a hydraulic jump through the opening and increased velocities for some distance downstream of the bridge. This process is responsible for the development of the scour hole immediately downstream of the bridge. The scour hole is also shown in an aerial photograph taken in 1981 (fig. 52).

By March 1981, the scour hole had extended upstream past part of the right main pier, exposing the pile cap and about four feet of the piles. The scour hole will probably continue to enlarge vertically and horizontally until it encompasses the entire downstream side of the constriction. Furthermore, the constriction causes large-scale eddying that further enhances scour at both piers. Figures 51 and 53 show both planametric and cross-sectional views of the scour hole.

Equation 7 is used to estimate local scour around the piers due to the contraction at the bridge, and to compare differences in erosive stream power caused by increased velocities and depth of flow at the bridge for 1954 and 1981.

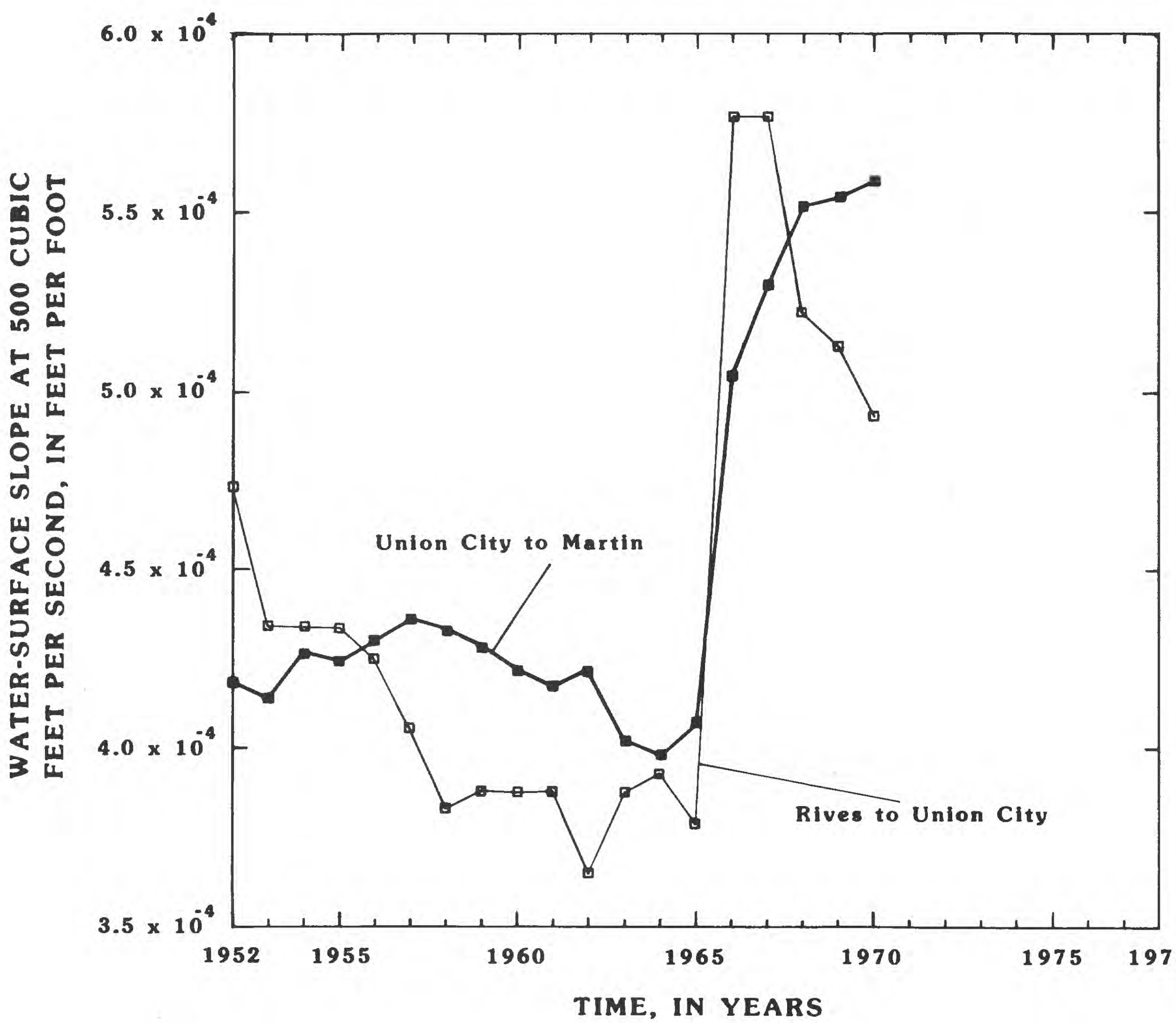


Figure 49.--Change in water-surface slope at a discharge rate of 500 cubic feet per second versus time between gaging stations on the North Fork Obion River, Tennessee.

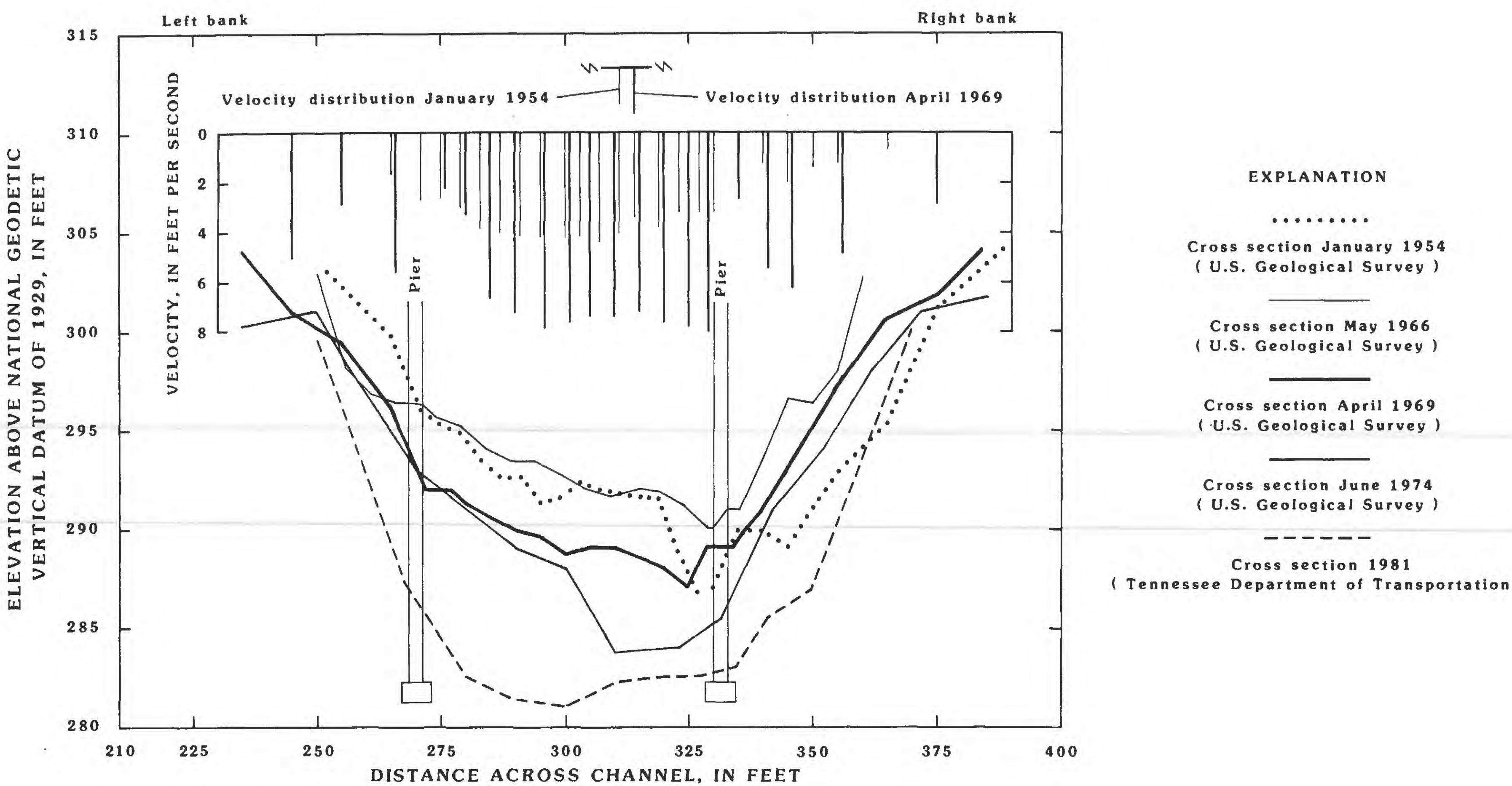


Figure 50.--Stream-channel cross sections showing velocity distributions for North Fork Obion River at State Highway 22 bridge near Union City, Tennessee.

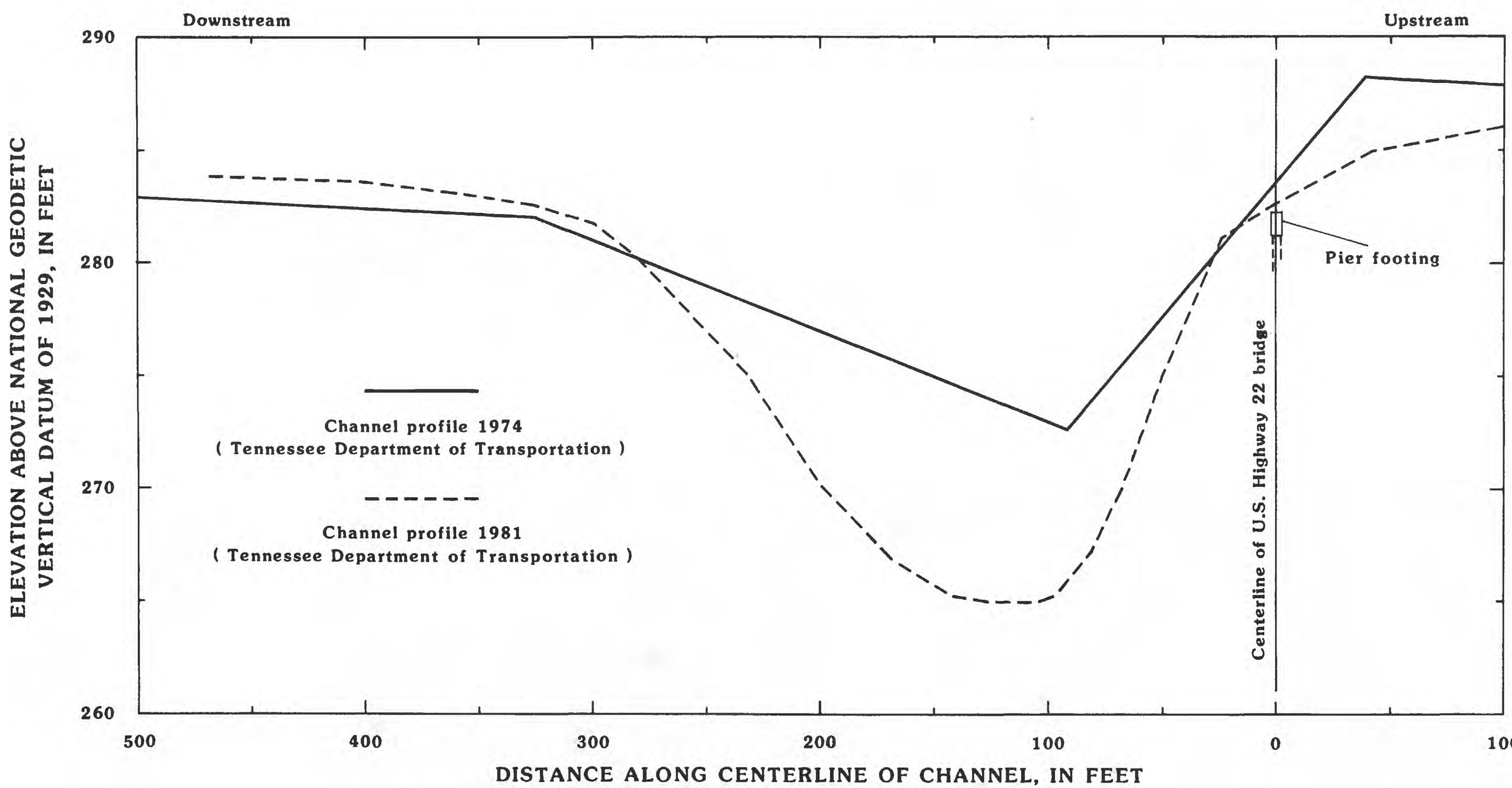
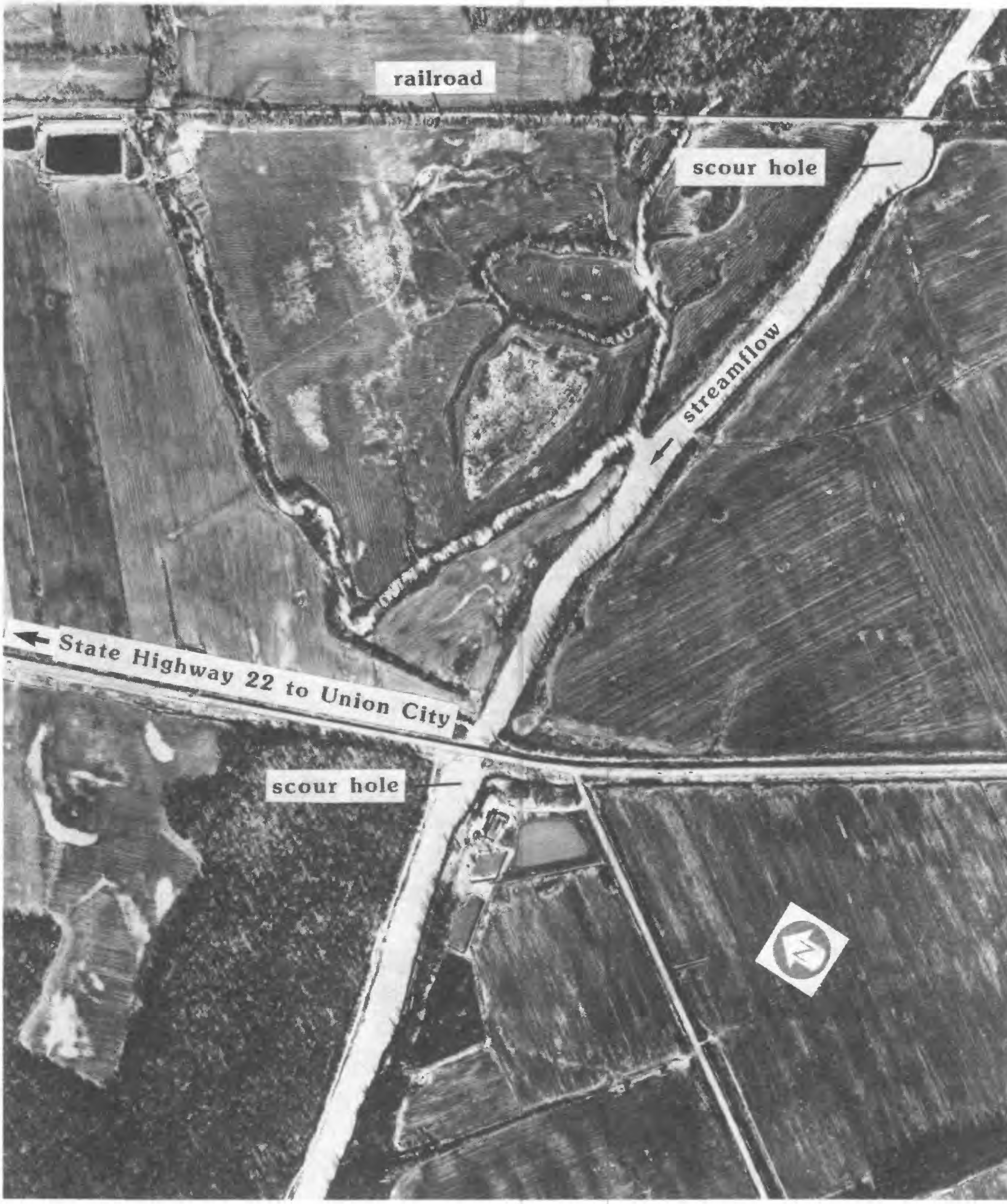


Figure 51.--Channel-bed profile of North Fork Obion River at State Highway 22 bridge near Union City, Tennessee.



Photograph from Tennessee Department of Transportation

Figure 52. -- Aerial photograph of North Fork Obion River at Highway 22 near Union City, Tennessee, showing scour holes below the highway bridge and railroad bridge, May 1980.

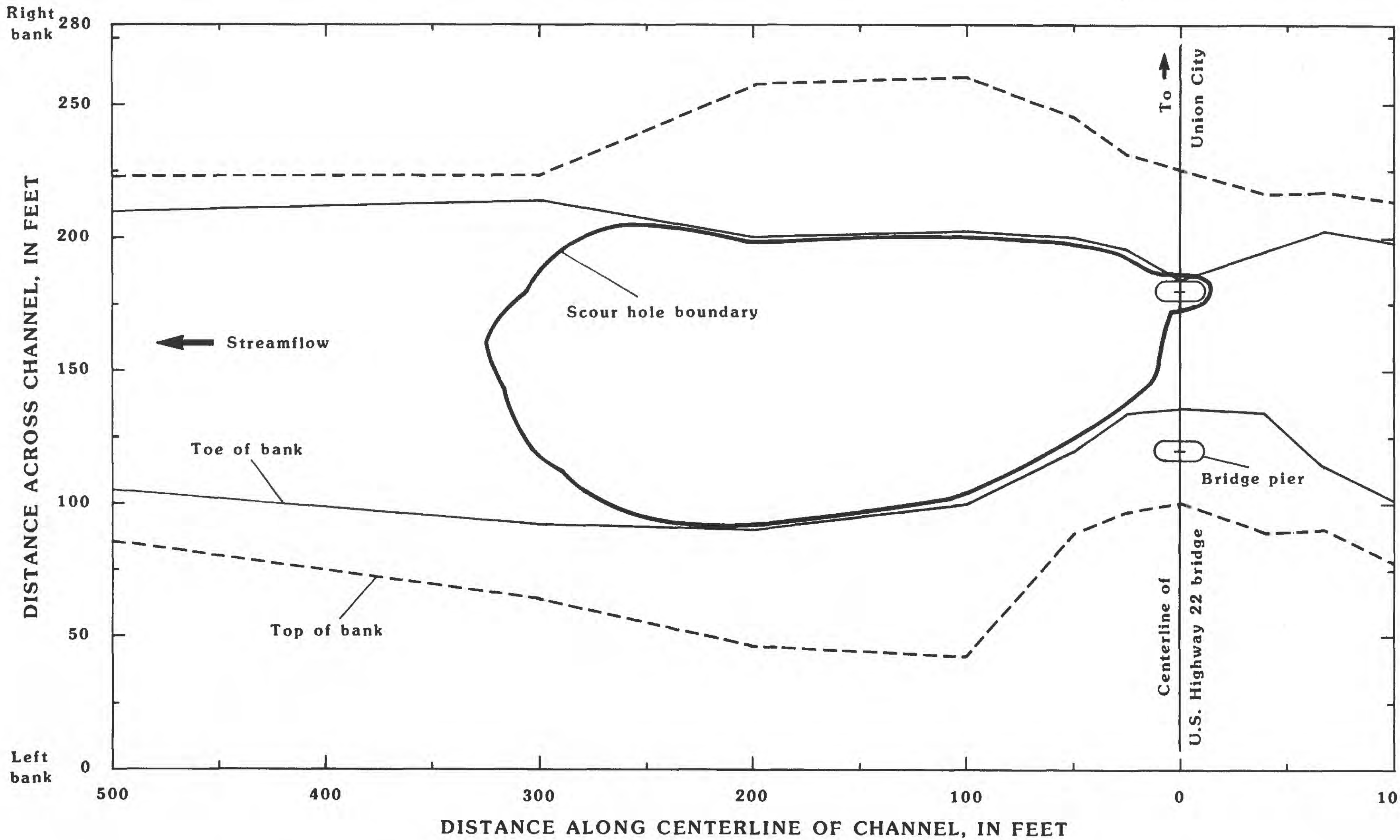


Figure 53.--Plan view of scour hole below State Highway 22 bridge on North Fork Obion River near Union City, Tennessee, from Tennessee Department of Transportation channel survey, March 1981.

Estimation of local scour for 1954 channel and bridge conditions

$$Y_s = 2.2a^{0.65} Y_1^{0.35} Fr_1^{0.43} \quad (7)$$

where $Y_1 = 12$ ft
$a = 6$ ft,
$V_1 = 2.3$ ft/s, and
$Fr_1 = 0.117$

Therefore: $Y_s = 2.2\,(6)^{0.65}\,(12)^{0.35}\,(.117)^{0.43}$
$= 6.73$ ft ← equilibrium scour depth

$$Y_{max} = 1.3\,(Y_s) \quad (9)$$

Therefore: $Y_{max} = 1.3\,(6.73) = 8.75$ ft ← Maximum scour depth

Estimation of local scour for 1981 channel and bridge conditions

$$Y_s = 2.2a^{0.65} Y_1^{0.35} Fr_1^{0.43} \quad (7)$$

where $Y_1 = 21$ ft
$a = 6$ ft,
$V_1 = 5.8$ ft/s, and
$Fr_1 = 0.223$

Therefore: $Y_s = 2.2\,(6)^{0.65}\,(21)^{0.35}\,(0.223)^{0.43}$
$= 10.73$ ft ← equilibrium scour depth

$$Y_{max} = 1.3\,(Y_s) \quad (9)$$

Therefore: $Y_{max} = 1.3\,(10.73) = 14.0$ ft ← Maximum scour depth

Velocity, depth of flow, and pier width for the 1954 conditions were obtained from discharge measurements made at this site. Velocity, depth of flow, and pier width for the 1981 condition were obtained from U.S. Geological Survey files (C. R. Gamble, written commun., 1981). Compared to the 1954 value, the 1981 value represents a 60 percent increase in maximum potential scour depth.

At the Martin gaging station (river mile 18.0) a trend of degradation began in 1972 (fig. 48). Channel work was completed at river mile 10.9 in January 1967. Therefore, it took six years (January 1967-72) for the channel adjustment to reach the gage at Martin, a rate of 1.18 miles/year. According to specific-gage records the channel bed at Martin had degraded a total of 2.9 feet from the end of 1971 to 1980 (fig. 48).

By regressing the degradation rate (Deg), in feet per year, for each gaging station against the distance (x), in miles, that each gaging station is located from the "maximum disturbance" (river mile 10.9) a good relation (correlation coefficient (r) = -0.99) results:

$$Deg = -0.045(x) + 0.642 \quad (18)$$

where Deg = degradation rate in feet per year, and
x = distance from point of maximum channel disturbance in miles.

The relation and its associated equation (fig. 54) are based on only 4 years of streamflow record (1968-70) at the Rives and Union City gaging stations. Therefore, the applicability of equation 18 is limited and should be used only to calculate the degradation rate along the channel during the present phase of channel adjustment.

As the channel adjustment progressed upstream in the Obion River, aggradation probably began in the mid-1970's in the lower reaches of the North Fork Obion River as the transported sediment load was deposited along the more

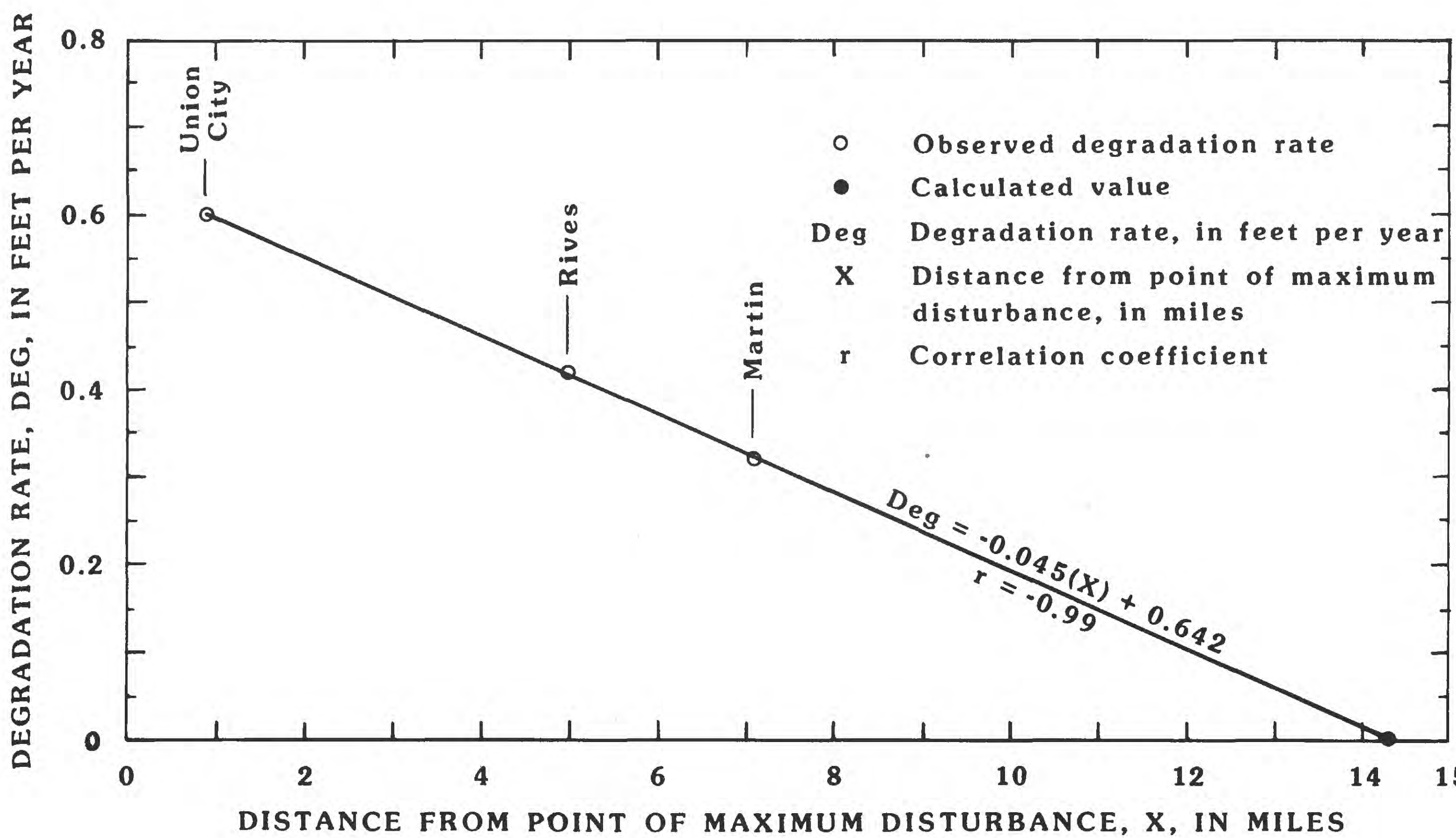


Figure 54.--Degradation rates on North Fork Obion River as a function of the distance from the point of maximum channel disturbance (river mile 10.9).

gently sloping reaches. The sum-of-slopes plots (fig. 55) peak in 1967, showing the effect of the channel work and then trend downward suggesting the initiation of channel adjustment to the imposed changes.

The sum of the average historical water-surface slopes for the North Fork Obion River, at a discharge rate of 500 ft^3/s, (1939-65) between Rives and Union City (4.09×10^{-4} ft/ft), and between Union City and Martin (3.81×10^{-4} ft/ft) is 7.9×10^{-4} ft/ft. By assuming that the average historical water-surface slope represents an adjusted gradient, and that the river will attempt to return to this sum-of-slopes by upstream degradation and downstream aggradation, the adjustment response time can be estimated. Regressing the sum-of-slopes data for the two channel reaches against time (B in fig. 55) yields a good relation [correlation coefficient (r) = -0.96]. This relation with its associated equation is shown in figure 55. The equation can be used to estimate the adjustment response time as follows:

$$\Sigma S = 0.0011(e)^{-0.0154(t)} \quad (19)$$

where ΣS = sum of water-surface slopes, and
t = time in years since the completion of channel work where $t_o = 1967$

Solving equation 19 for time, t, and substituting 7.9×10^{-4} ft/ft for the sum of water-surface slopes, ΣS, the response time for the two reaches to attain an adjusted gradient is approximately 21 years, or until 1988. Because the sum-of-slopes value used in this calculation (7.9×10^{-4} ft/ft) represents a straightened channel condition (straightened in the late 1920's) and the North Fork Obion River was only dredged (no straightening) in 1965, it is anticipated that the adjusted sum-of-slopes profile gradient for this river will return to this gradient (7.9×10^{-4} ft/ft) especially as the sum-of-slopes regression for the main stem of the Obion (fig. 17) is nearly identical to that of the North Fork Obion (fig. 55). The Union City and Rives gages were discontinued at the end of 1970 and only four years of slope data are incorporated into the regression equation. However, the North Fork Obion River exhibits adjustments similar to other rivers discussed previously. Thus, the estimated trends appear significant.

The magnitude of the imposed stress on the North Fork Obion River appears to be the least of any of the rivers analyzed in this report where channels have been modified. The average percent increase in water-surface slope after channelization (77.5 percent) is lower, and the disturbance propogation rate (1.18 mi/yr) is proportionately slower than the other rivers.

Following the completion of channel work 1967, further degradation occurred at Rives and 6.1 river miles upstream at Union City. Assuming that the rate of propagation of the channel adjustment continues at a steady pace (1.18 mi/yr), the adjustment would reach the Palmersville gage (river mile 34.9) by 1986. Significant channel adjustments are expected to end before this time, and thus never reach the Palmersville gage for the following reasons:

(1) Calculated degradation rates for the North Fork Obion River (fig. 54) trend toward zero at a point in the channel approximately 14 river miles upstream of the "maximum" disturbance (river mile 10.9) or at river mile 25. This indicates that channel gradient downstream has been sufficiently adjusted to transport the sediment load delivered from upstream, and that further gradation is unlikely (Deg = -0.045 (x) + 0.642 (Eq. 18)).

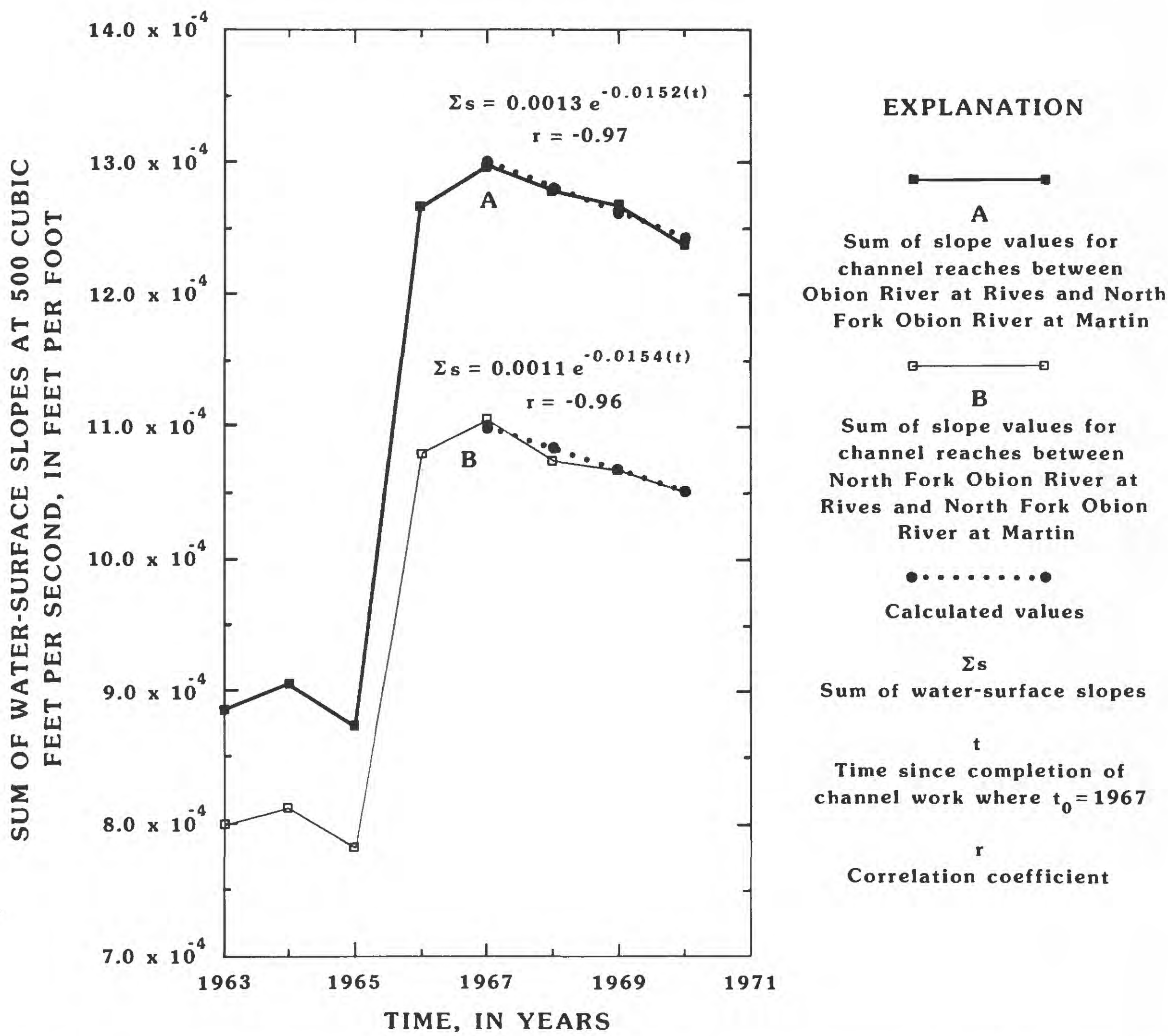


Figure 55.--Sum of water-surface slopes at a discharge rate of 500 cubic feet per second versus time on North Fork Obion River, Tennessee.

(2) The sum-of-slopes regression (fig. 55) calculated for the average historical sum-of-slopes (1939-65) results in a response time of 21 years, or until approximately 1988. This indicates that sufficient channel gradient adjustment is expected to have occurred between river mile 0.0 and river mile 25 to yield an adjusted sum-of-slopes profile by 1988.

The Highway 22 bridge near Union City is threatened by a scour hole that developed from channel modifications and their related effects. Considering that the channel bed was lowered more at Rives (river mile 5.9) than at Union City (river mile 10.0), related processes such as channel widening are probably even more effective at Rives than at the Union City site where substantial lateral cutting has also occurred. Potential bridge problems along this river are presumably restricted to areas effected directly by removal of channel material. This inference differs from those of the previous two case studies, in which river responses are apparently contributing to potential bridge problems far upstream from the channel modifications.

HATCHIE RIVER - BOLIVAR TO WALNUT

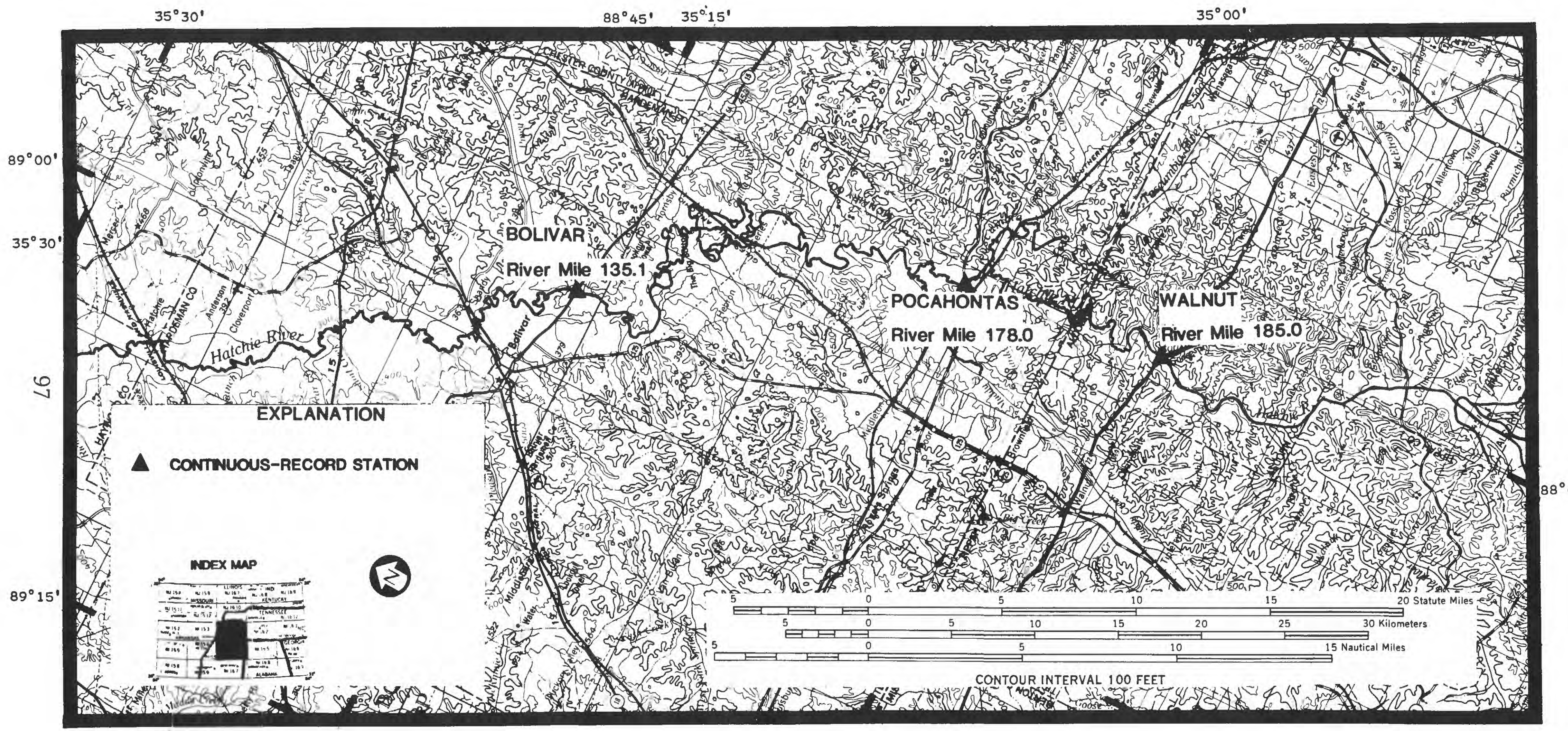
The Hatchie River represents the control basin in this report. It was analyzed by some of the same tests as were those basins where channel modifications have taken place. This river drains approximately 2,609 square miles in west Tennessee and has an almost "natural" fluvial development. The drainage area at Bolivar, the most downstream gaging station used in this analysis, is 1,480 square miles (fig. 56).

The Hatchie River valley has a broad flood plain (1.5 to 2.5 miles wide) and a sinuous channel with numerous swamps (fig. 56). Agricultural development has been limited to the upland areas of the watershed and the flood plain is heavily vegetated with trees and shrubs. The only alteration of this river by man was some channel snagging from 1938-43 (affecting 145 river miles) and from 1945-52 between river miles 130 and 190.

Mean water-surface slope, at a discharge rate of 2,000 ft^3/s, from 1960-77 between Bolivar (river mile 135.1) and Pocahontas (river mile 178) was 1.33×10^{-4} ft/ft, with a standard deviation of 2.0×10^{-6} ft/ft. The data were divided into two groups, 1960-68 and 1969-77, to test for significant differences in water-surface slope. Mean slope values of 1.32×10^{-4} ft/ft for 1960 through 1968 and 1.33×10^{-4} ft/ft for 1969 through 1977 were defined. Water-surface slopes have fluctuated in a narrow range during this 17-year period.

Water-surface slopes from 1960-77 between Pocahontas and Walnut (river mile 178-185) averaged 7.97×10^{-4} ft/ft. Splitting the data set in half, water-surface slopes were 8.04×10^{-4} ft/ft (1960-68) and 7.90×10^{-4} ft/ft (1969-77), a 1.7 percent difference. The data were divided to approximate the concurrent time periods when other rivers were undergoing large-scale slope changes.

The variation in water-surface slopes for the two reaches is shown in figure 57. Although the plot of the upstream reach appears to portray substantial variation, the difference between the maximum water-surface slope



Base from U.S. Geological Survey
Bltheville, 1:250,000 and Tupelo, 1:250,000, 1970

Figure 56.--Location of Hatchie River gaging stations used in analysis.

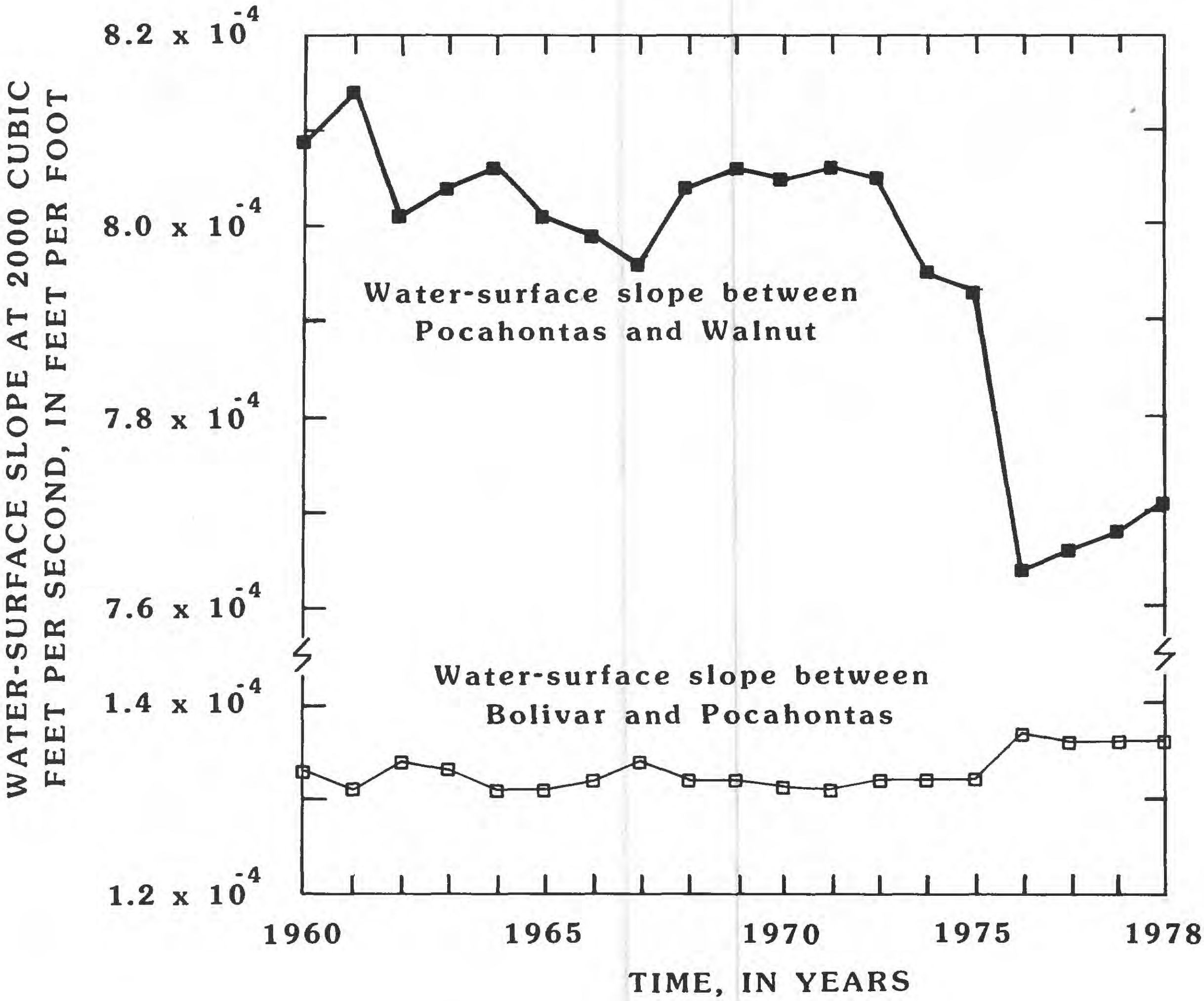


Figure 57.--Change in water-surface slope at a discharge rate of 2,000 cubic feet per second versus time between gaging stations on the Hatchie River, Tennessee.

in 1961 (8.14×10^{-4} ft/ft) and the minimum water-surface slope in 1975 (7.63×10^{-4} ft/ft) is only 6.3 percent.

The stability in profile is indicated in the specific-gage record, at a discharge rate of 2,000 ft^3/s, for the same three sites (fig. 58). Bed level at the Walnut gage remained nearly constant from 1960-78. Deposition rates at Pocahontas and Bolivar were 0.09 and 0.06 ft/yr, respectively. These reaches of the Hatchie River are exhibiting gradients which are characteristic of channels in a relatively stable condition.

Sum-of-slopes data (fig. 59) illustrate a maximum fluctuation of 1 percent around a mean value of 9.36×10^{-4} ft/ft from 1960-72. Larger variations began in 1973 and continued through 1975. This period had three years of above normal percipitation and above average streamflow in the basin. Mean annual discharge at the Bolivar gage for 1973, 1974, and 1975, respectively, were 36, 54, and 39 percent more than the mean annual discharge for the period of record (1929-81). By 1975, water-surface slope in the upper reach (Pocahontas to Walnut) had decreased approximately 5 percent whereas, the lower section increased 4 percent.

Bed levels, however, remained fairly constant, reflecting channel stability. The only noticeable change occurred from 1974-75 at Pocahontas, where bed level increased 1.2 feet. The Pocahontas gage is approximately 1.6 river miles downstream from the confluence of the Hatchie River and the Tuscumbia River (fig. 56). The Tuscumbia River was dredged and straightened between 1938 and 1943, and like the other channelized rivers studied in this report, it is assumed that the Tuscumbia River is experiencing aggradation in its lower reaches. This inherent instability was probably accentuated during the period of higher than normal streamflows (1973-75), causing a greater amount of sediment to be delivered to the Hatchie River. This process is assumed to be the cause of the bed level increase at Pocahontas.

The yearly fluctuation in slope (fig. 60) illustrates the river response to increased streamflows. The maximum peaks in slope occurred in 1975, indicating that increased flows caused a simultaneous response in the two reaches. The channel changes in the Hatchie River are extremely small in comparison to the other rivers.

SUMMARY OF CHANNEL ADJUSTMENT IN WEST TENNESSEE

The predictive equations designed and described in this report represent an empirical analysis of man-induced channel admustment. These equations, of the generalized form

$$S = ae^{-b(t)}$$

where S = a parameter of slope,
t = time in years since completion of channel work, and
a and $-b$ = coefficients

define the end product of the interaction of interrelated variables in the fluvial environment. It is understood that the actual values of these coefficients may vary by channel reach and basin as a function of the relative significance of the controlling variables. Factors such as drainage area, magnitude of imposed disturbance, type of bed and bank material, sediment

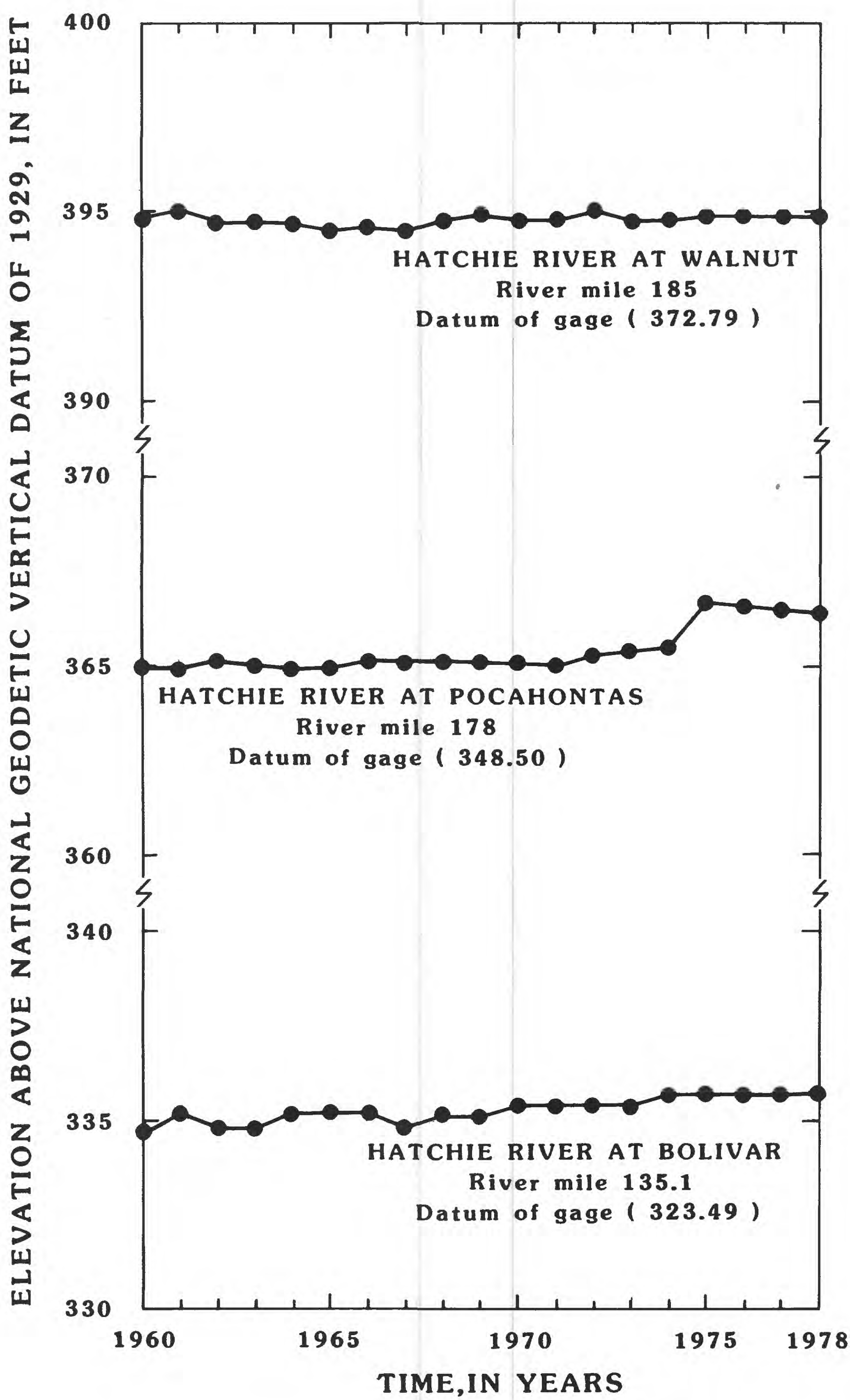


Figure 58.--Specific-gage elevations at a discharge rate of 2,000 cubic feet per second versus time for Hatchie River gaging stations.

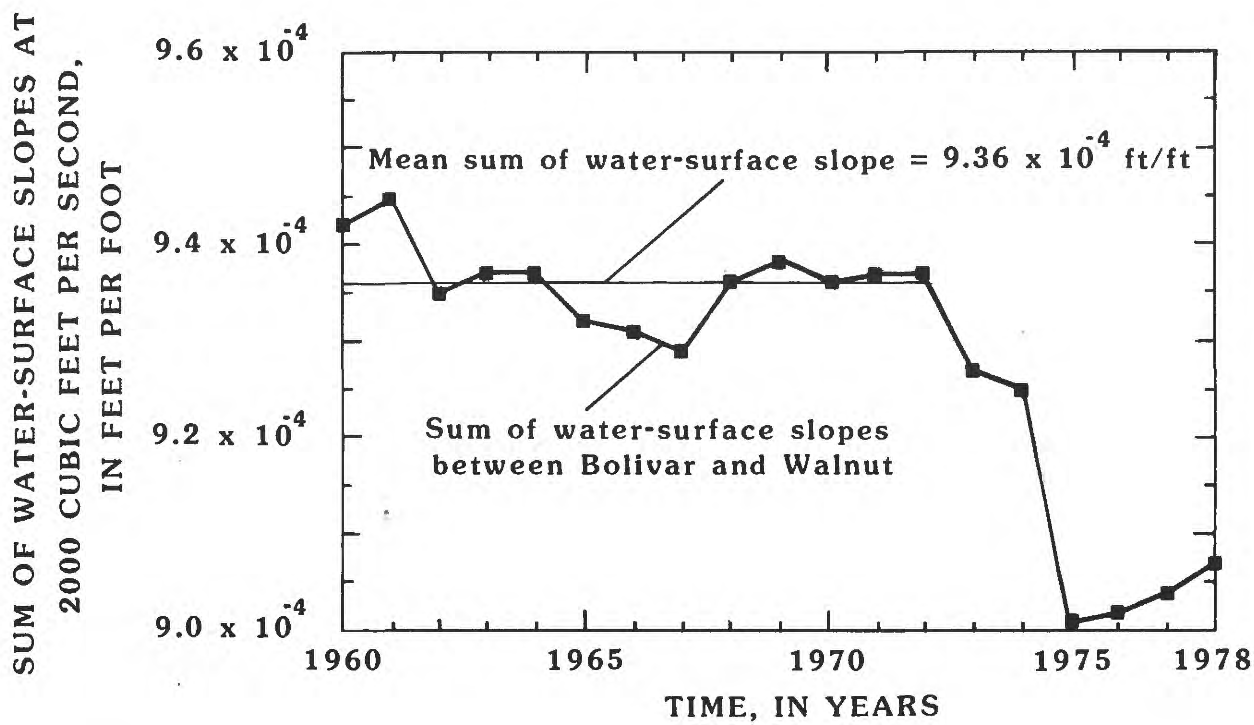


Figure 59.--Sum of water-surface slopes at a discharge rate of 2,000 cubic feet per second versus time on the Hatchie River between Bolivar, Tennessee, and Walnut, Mississippi.

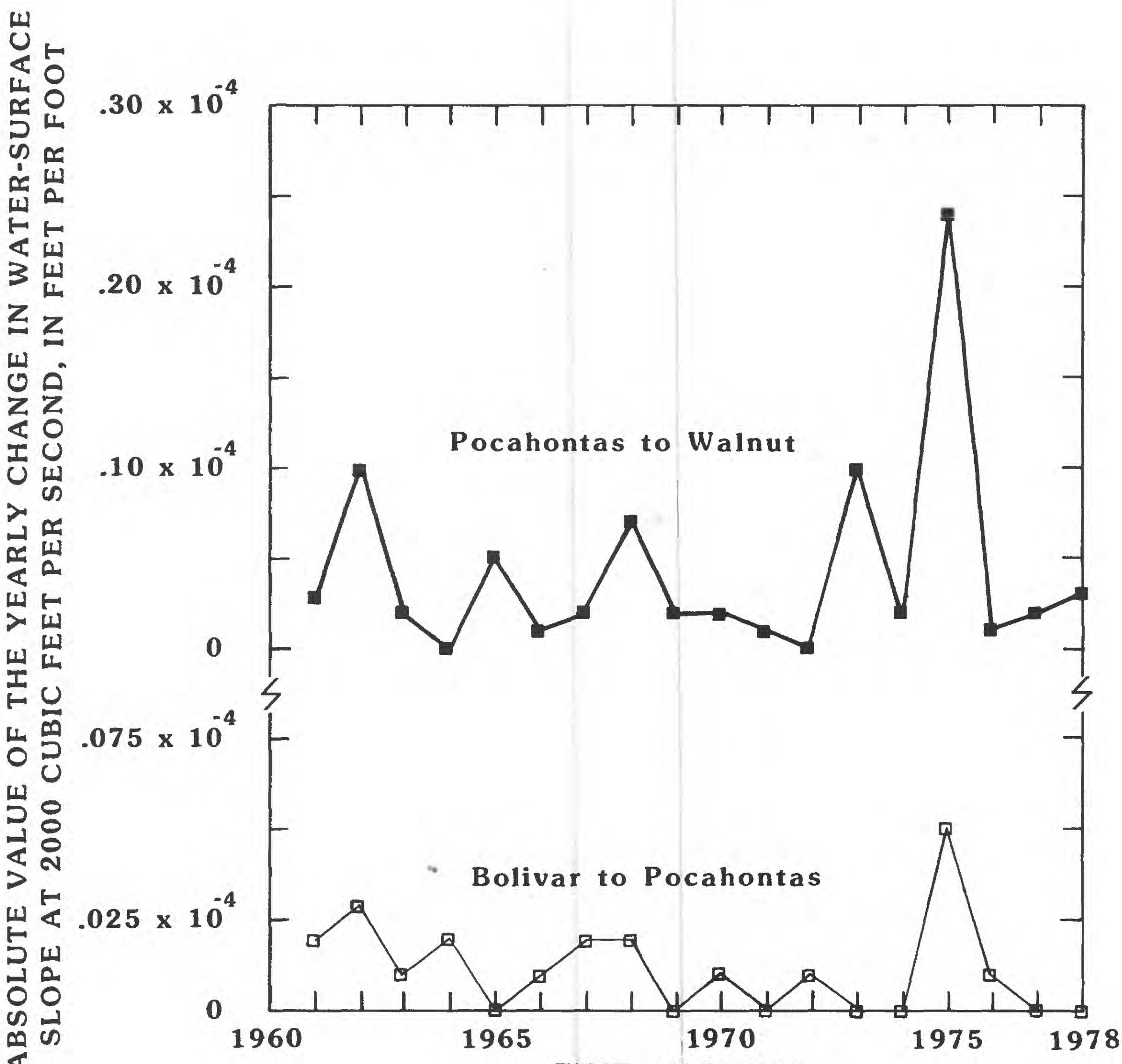


Figure 60.--Absolute value of yearly change in water-surface slope between gaging stations on the Hatchie River, Tennessee.

loads, flow characteristics, and antecedent conditions all exert an influence of unspecified magnitude on the response of the stream channel to stress. Still, the equations adequately evaluate and predict stream channel adjustment over time for the study reaches discussed in this report. The data used for each analysis is given in Supplement B.

The relation between the magnitude of the channel disturbance, as described by the percent increase in slope and the speed at which the channel adjustment progresses upstream, is shown in figure 61. Regressing the rate of adjustment propagation against the percent change in water-surface slope after channel modification yields a good relation [correlation coefficient (r) = 0.95]. The general relation is defined by using channel adjustment propagation rates and percent change in water-surface slope of the North and South Fork Obion Rivers and the South Fork Forked Deer River and is expressed by the following equation:

$$R = 2.81 \times 10^{-3}(z) + 1.03 \quad (20)$$

where R = the rate of adjustment propagation, in miles per year, and

z = the percent change in water-surface slope.

The relation (fig. 61) indicates that the greater the percent change in water-surface slope, the faster the adjustment propagation rate. The South Fork Forked Deer River, where the channel was shortened and water-surface slope was increased 210 percent, showed the most rapid and probably the most large-scale channel adjustments of the sites discussed here. In comparison, the South Fork Obion River (76 percent slope increase) underwent nearly 18 feet of channel-bed lowering at Kenton, and an estimated 10 feet of channel-bed lowering is expected to occur by degradation at Greenfield. Within the next 7 to 10 years bridges at both locations may be damaged due to sloughing and caving of over-steepened banks and local scour caused by altered channel conditions.

The large-scale changes occurring in the Obion River, its forks, and the South Fork Forked Deer River are not taking place in the Hatchie River. Because this channel has not been modified by man, drains the same types of materials as do other basins of west Tennessee, and is exposed to similar climatic conditions as the other basins, it appears that it can withstand natural stresses without exhibiting characteristics of instability. The wet years of 1972 through 1974 in west Tennessee resulted in accelerated down-cutting, oversteepening of banks, and subsequent bank sloughing and caving in the channelized rivers. The bridge failures occurred in the channelized rivers or their tributaries while stability was maintained in the Hatchie River.

Table 4 summarizes areas where bridge problems are most likely occurring, or where they may occur in the near future. These estimates are based on the present state of instability and the phase of channel adjustment exhibited by a particular river. Tributaries to these channelized rivers will be affected by the channel adjustment ongoing in the main river. The magnitude and rate of adjustment will presumably be enhanced in the tributaries because the smaller drainage areas will be less capable of coping with the bed level changes occurring in the trunk streams.

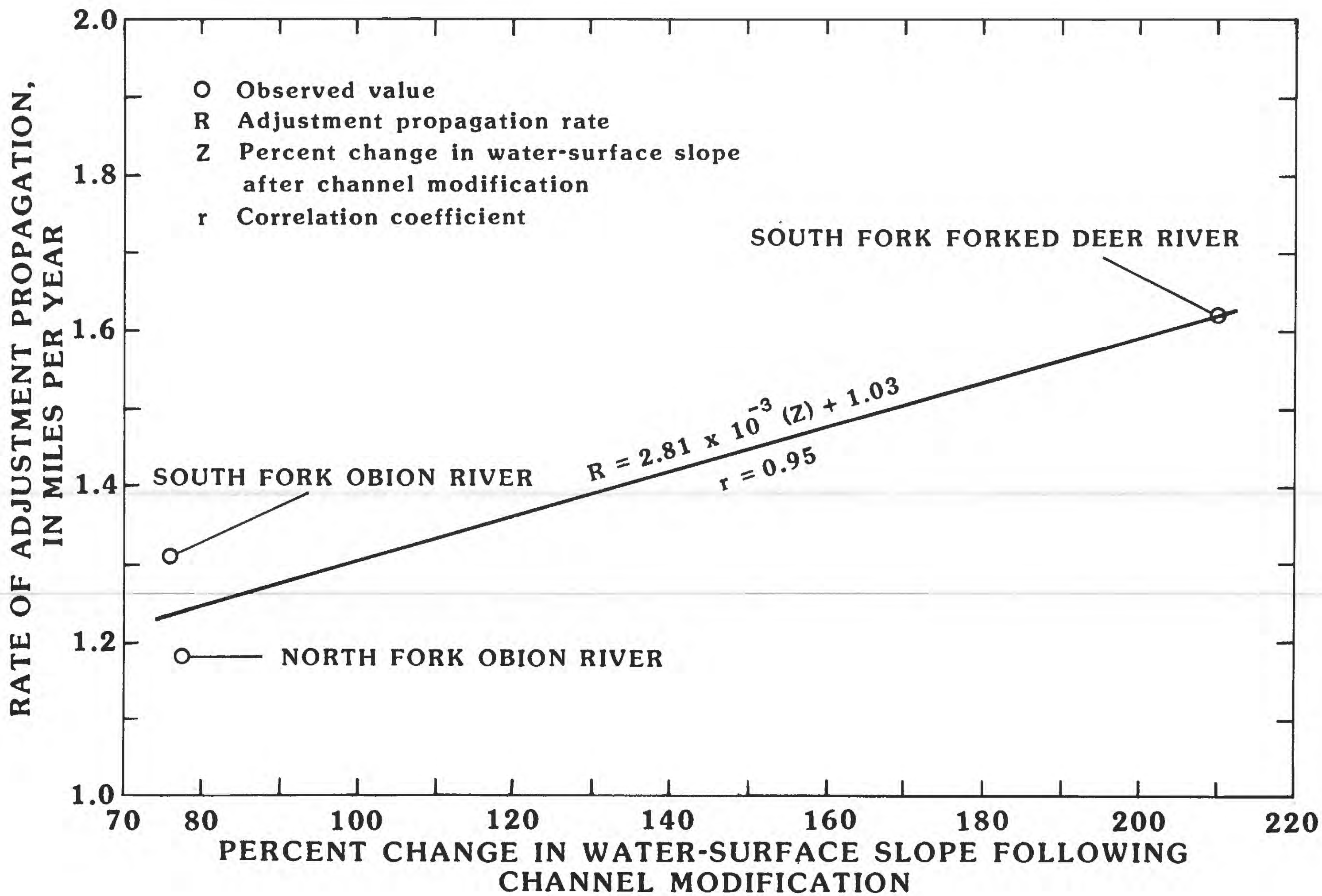


Figure 61.--Relative rates of channel adjustment propagation as a function of the magnitude of the imposed disturbance.

Table 4.--Channel conditions and areas of potential bridge problems

[RM, River Mile]

River	Adjustment phase	Cause and date	Associated problems
Obion	Degradation headward of Menglewood gage (RM 29.0).	Channelization, 1970's.	Downcutting, bank instabilities, headward erosion in stem and tributaries.
	Aggradation, downstream of Rives gage (RM 66.5).	Channelization, 1960's.	Reduced channel capacity, concentrated flows.
North Fork Obion	Degradation, at and down stream of RM 24.0.	Channelization, 1960's.	Downcutting, widening, scour from contracted openings headward erosion in stem and tributaries.
South Fork Obion	Degradation, upstream of Kenton (RM 5.8)	Channelization, 1960's.	Downcutting, widening, headward erosion in stem and tributaries.
	Aggradation, at and down stream of Kenton (RM 5.8)	Channelization, 1960's.	Oversteepened owing to degradation phase of adjustment.
South Fork Forked Deer	Degradation, upstream of Yellow Bluff (RM 3.2).	Channelization, 1960's.	Downcutting, widening, headward erosion in stem and tributaries.
North Indian Creek	Degradation, upstream Erwin (RM 2.2).	Dredging, 1960's.	Headward erosion.
Hatchie	Stable	---	---

APPLICATION OF ANALYTICAL TECHNIQUES TO CHANNEL STABILITY STUDIES

DATA REQUIREMENTS

Application of the analytical techniques presented in this report to channel-stability studies requires at least two continuous streamflow gaging stations with 10 or more years of record on the channel reach of interest. Time-series channel cross sections, bed profiles, and low-level aerial photographs at several locations within the channel reach, for the identical time period, are also needed. A complete history of channel modification above and below the channel reach of interest is needed to determine if the channel is reacting to natural or man-induced stress.

PROCEDURE

After the appropriate data have been collected, select an intermediate discharge value and plot its corresponding water-surface elevation (specific gage) against time for each gaging station. Use the same discharge value for all gaging stations on the same channel. The specific-gage plots will indicate rates of gradation trends within the channel reach. Compare the channel cross sections and channel-bed profiles with the specific-gage plots to verify any gradation trends indicated by the specific-gage plots.

Comparing channel cross sections at a specific location will indicate changes in channel flow capacity or lateral channel shifting. Channel bed profile comparisons will detect channel-bed irregularities. The bed profiles can also be used to estimate channel slope.

Aerial photographs are used to detect channel instabilities. Channel bank scalloping indicates that the channel bed may be degrading due to removal of material from the toe of the bank. The aerial photographs can also be used to measure channel widening if the photograph scale is known.

Once these preliminary comparisons have been completed, any or all of the four analytical techniques (slope regression, sum of slopes, unit stream power, and exponential decay function) can be applied. If results of the analytical techniques indicate (1) that the water-surface slope is relatively constant around some mean, (2) the channel cross sections indicate small changes in channel flow capacity, and (3) the channel-bed profiles indicate a stationary channel bed, then the channel reach is likely in equilibrium with the surrounding hydraulic and hydrologic environment.

However, if (1) the water-surface slope is rapidly increasing or decreasing, (2) the channel cross sections indicate large changes in channel flow capacity, and (3) the channel-bed profiles indicate a progressive raising or lowering of the channel bed, then the channel reach is probably in an adjustment cycle due to imposed stress upstream or downstream. In this case, the slope regression or the exponential decay function equations can be solved to estimate the time required for the channel reach to complete its adjustment under prevailing hydraulic and hydrologic conditions.

SUMMARY AND CONCLUSIONS

The fluvial environment is a combination of inter-related variables continuously acting upon each other to produce channel and stream network conditions that are compatible or in "equilibrium" with prevailing hydrologic conditions. These conditions can be "natural" or man-induced. Streams are continually adjusting to a fluctuating environment; therefore distinctions must be made between changes that produce short-term, small-scale adjustments, and changes that lead to channel instability and associated systematic long-term adjustments (gradation).

This report provides (1) results of an investigation to determine the effects of channelization upon channel characteristics and processes, and (2) insight into the reliability of using these characteristics and processes as diagnostic measures of fluvial adjustment. Four rivers, in two basins of west Tennessee, were evaluated for their adjustments to channel modifications. They are the main-stem Obion River, the North and South Forks of the Obion River, and the South Fork Forked Deer River. To separate man-induced responses from those initiated naturally, the Hatchie River, also in west Tennessee, was used for comparison. The only alteration of this river by man was some channel snagging from 1938-43 and 1945-52. The drainage basin of the Hatchie is similar in topography, geology, and size to the other rivers used in this study. Therefore, it served as an example of "natural" fluvial development in west Tennessee.

This study developed and utilized a series of analytical methods to describe channel adjustment. The prediction of channel adjustment and response time is based on a slope function that describes the effect of the imposed change on the hydraulically compatible, or "equilibrium," gradient eventually attained by the channel. Water-surface slope, as the primary dependent variable, appeared to be a sensitive and descriptive parameter that reduced bias due to local conditions. The basic unit of measurement is the channel reach, which seems necessary when considering the propagation of an adjustment along a channel.

Stream power (Θ) and the theory of minimum energy dissipation served as useful methods to describe altered river conditions. Stream power and slope proceed toward a minimum value or equilibrium condition. This agrees with the findings of Yang and Song (1979) and indirectly with those of Osterkamp and Harrold (1981) pertaining to a "natural" tendency of decreasing gradient in alluvial channels. Although some channel reaches showed increases in slope as accelerated erosion progressed upstream, the overall adjustments of each stream was a net decrease in slope.

The sum of slopes and the exponential decay function representing the yearly change in slope, provide methods to predict adjustment times to channelization for reaches or whole lengths of rivers. Furthermore, gradation rates within a particular phase of adjustment are a function of the distance along the channel from the maximum disturbance.

Changes in gradient by channel straightening caused larger-scale channel adjustments than did channel dredging and clearing. The magnitude of a disturbance, estimated by the percent change in gradient, can be related by linear regression to the rate of channel adjustment propagation upstream. Channel change induced by wet and dry periods alone (as indicated by the Hatchie River) appeared negligible and was simultaneous along the river course in comparison to the propagation upstream of a gradient adjustment in a channel following man-induced changes.

Data indicate that the profile eventually attained by a stream following channelization is similar to the original gradient if no alteration to channel length occurred. If the channel was straightened and shortened by cut-offs, slope adjustment (reduction) proceeds past the original profile gradient to a new profile with a lower gradient. This finding suggests that possibly the channels of west Tennessee can be maintained without inducing larger-scale instabilities. The exponential decay function defining yearly variation in slope can be used to estimate the length of time required for a stream to attain its adjusted gradient. Furthermore, by substituting the resultant length of time in either the sum of slopes or slope regression equation(s), the hydraulically compatible gradient profile can be estimated.

Stresses imposed on stream channels by channel modifications led to downcutting, headward erosion, downstream aggradation, accelerated scour, bank instabilities, and in some cases contributed to bridge failure. Combinations of these effects are still affecting bridge structures spanning the main-stem Obion River, the North and South Forks of the Obion River, the South Fork Forked Deer River, their tributaries, and probably other channelized streams in west Tennessee.

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SUPPLEMENT A

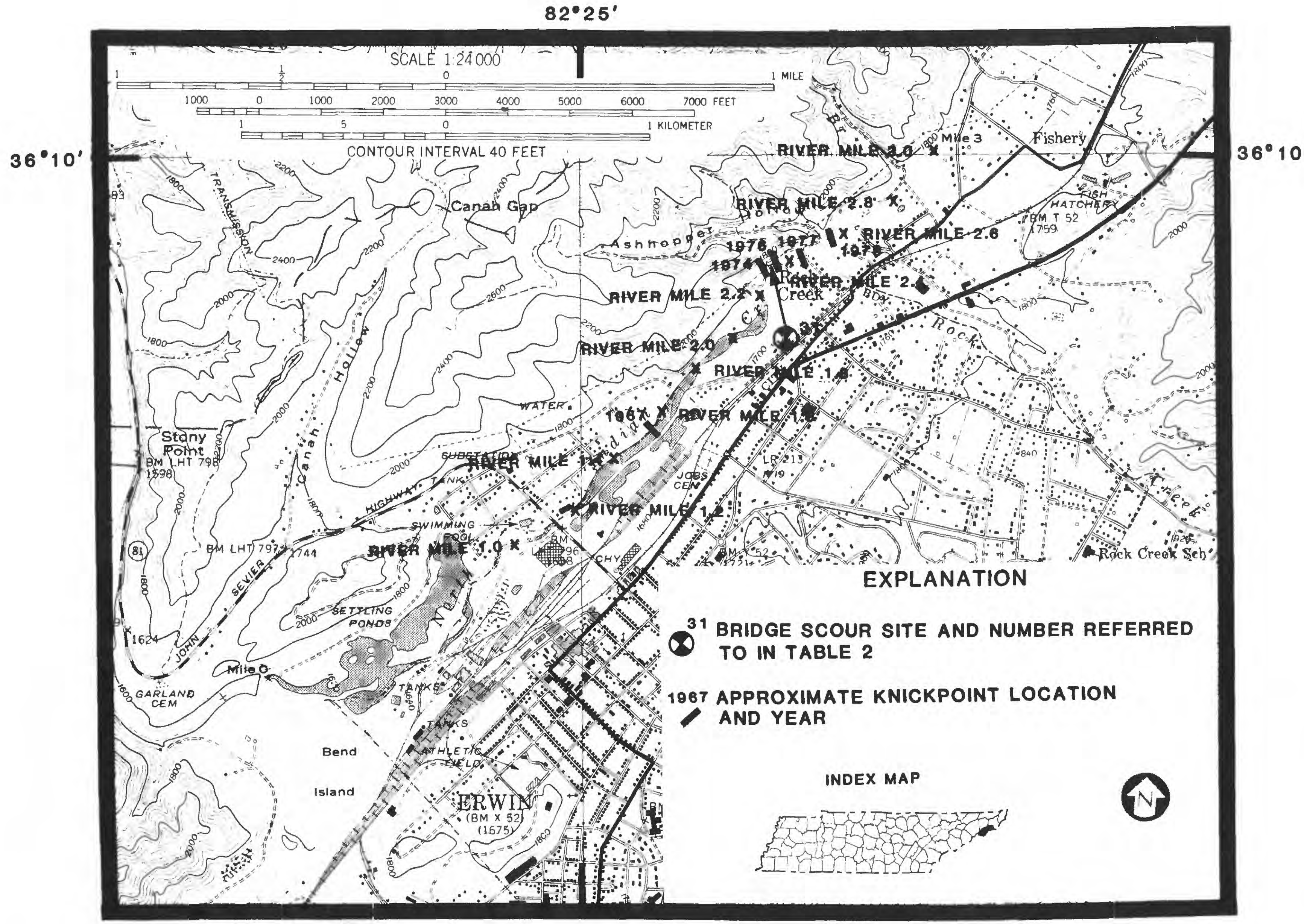
North Indian Creek at Erwin, Tennessee, Unicoi County, Site 31

Site 31 (plate 1, table 2) is in northeast Tennessee, approximately 1.5 miles north of Erwin, and approximately 2.2 river miles above the confluence with the Nolichucky River (fig. 62). The drainage area above site 31 is 57.3 square miles. The river valley at the time of initial investigation contained a flood plain approximately 800 feet wide, a stream channel approximately 5 feet deep and 40 feet wide, and a fairly steep channel gradient. The channel bed consists of a thin (2 to 3 feet) armoring of gravel and cobbles overlying weathered bedrock. Six longitudinal profiles taken at North Indian Creek at Erwin from 1939 to 1978 are shown in figure 63. According to the 1939 profile, channel gradient between river mile 1.36 and 2.70 was 8.42×10^{-3} ft/ft.

From 1966-67 a construction-products company removed sand and gravel from the streambed in the vicinity of river mile 1.43 (fig. 63), which effectively lowered the bed 25 feet. Consequently, a knickpoint was formed in the channel at approximately river mile 1.57 and the channel gradient was increased to 1.03×10^{-2} (22 percent) by 1967 (fig. 63).

By May 1974, the knickpoint had moved upstream to river mile 2.30 at a rate of approximately 543 feet per year. The erosion increased the channel gradient to 2.76×10^{-2} ft/ft (168 percent increase) between river miles 1.98 and 2.30, and reduced the channel gradient to 6.15×10^{-4} ft/ft (94 percent decrease) between river miles 1.36 and 1.98. By July 1, 1976, the knickpoint had moved upstream an additional 75 feet to river mile 2.32. Figure 64 is a picture of the knickpoint as it appeared on July 1, 1976. On November 6, 1977, a flood of approximately 3,500 ft^3/s occurred, and on November 29, 1977, another profile was surveyed (fig. 63) and another picture was taken (fig. 65). The knickpoint had moved an additional 138 feet upstream and flattened somewhat since July 1, 1976 (fig. 65). A fifth profile was surveyed and pictures were taken on April 18, 1978, and as illustrated in figures 63, 66, and 67, the knickpoint had proceeded another 650 feet upstream. The basic shape of the knickpoint was maintained througout its propagation upstream because of the nature of material through which the knickpoint was moving. This knickpoint differs in shape and rate of propagation from the adjustments documented in west Tennessee channels.

Figures 68-70 show the North Indian Creek channel as it appeared in May 1981. The entire valley has been altered by two processes: (1) natural channel adjustment due to bed lowering, and (2) highway construction along the right bank. Streamflow data and time-series cross-section data are not available for this site, and for this reason no further analysis is offered for this case study.



Base from U.S. Geological Survey
Erwin 1:24,000, 1939, revised 1971

Figure 62.--Location of North Indian Creek at Erwin, Tennessee.

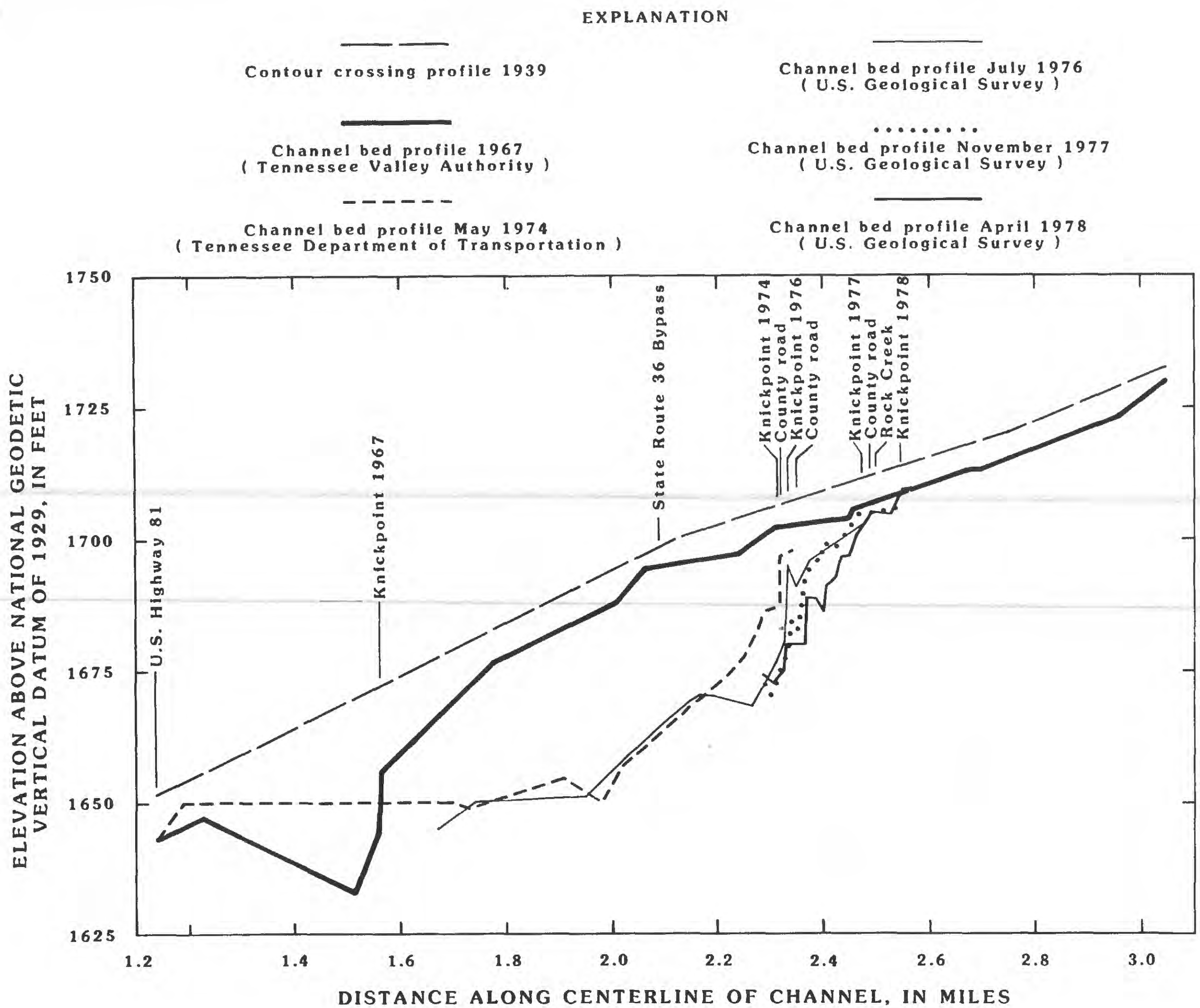


Figure 63.--Channel-bed profiles of North Indian Creek at Erwin, Tennessee.

Photograph by Randy S. Parker, U.S. Geological Survey

Figure 64. -- North Indian Creek at Erwin, Tennessee. View is upstream at falls and bridge from right bank, July 1, 1976.

Photograph by Randy S. Parker, U.S. Geological Survey

Figure 65. -- North Indian Creek at Erwin, Tennessee. View is upstream at falls and bridge from right bank, November 9, 1977.

Photograph by Charles R. Gamble, U.S. Geological Survey

Figure 66. -- North Indian Creek at Erwin, Tennessee View is downstream from left bank, April 18, 1978.

Photograph by Charles R. Gamble, U.S. Geological Survey

Figure 67. -- North Indian Creek at Erwin, Tennessee. View is upstream at knickpoint migration from left bank, April 18,1978.

Photograph by James G. Lewis, U.S. Geological Survey

Figure 68. -- North Indian Creek at Erwin, Tennessee. View upstream at old bridge location from right bank. May 1981.

Photograph by James G. Lewis, U.S. Geological Survey

Figure 69. -- North Indian Creek at Erwin, Tennessee. View is downstream at old bridge location from right bank, May 1981.

Photograph by James G. Lewis, U.S. Geological Survey

Figure 70. -- North Indian Creek at Erwin, Tennessee. View is upstream at old bridge location and new valley conditions, May 1981.

SUPPLEMENT B

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SUPPLEMENT B

Data used for analysis of Obion River

[S = water-surface slope in channel reach in feet per foot, $|\Delta S|$ = absolute value of change in water-surface slope in channel reach between two consecutive years, and ΣS = sum of water-surface slopes in feet per foot]

[In order to obtain actual values of S, $|\Delta S|$, and ΣS, multiply values shown in table by 10^{-4}]

Year	Menglewood specific gage at 500 ft^3/s	Bogota specific gage at 500 ft^3/s	Obion specific gage at 500 ft^3/s	Rives specific gage at 500 ft^3/s	S between Menglewood and Bogota	S between Bogota and Obion	S between Obion and Rives	$\lvert\Delta S\rvert$ between Menglewood and Bogota	$\lvert\Delta S\rvert$ between Bogota and Obion	$\lvert\Delta S\rvert$ between Obion and Rives	ΣS between Menglewood and Rives
1950		251.20	264.30	281.40		0.97	3.60				
1951		251.19	264.45	281.60		0.98	3.61		.01	.01	
1952		251.19	264.40	280.80		0.97	3.45		.01	.16	
1953		250.80	264.80	280.80		1.03	3.37		.06	.08	
1954		250.20	265.20	281.40		1.11	3.41		.08	.04	
1955		250.20	264.60	282.40		1.06	3.75		.05	.34	
1956		250.20	264.80	282.90		1.08	3.81		.02	.06	
1957		250.40	265.30	283.00		1.10	3.72		.02	.09	
1958		250.40	265.10	283.00		1.08	3.77		.02	.05	
1959		250.60	264.90	284.00		1.05	4.02		.03	.25	
1960	237.72	250.00	265.20	284.40	1.46	1.12	4.04		.07	.02	6.62
1961	237.88	249.00	265.20	284.40	1.46	1.21	4.04	0	.09	0	6.71
1962	237.80	244.40	265.10	284.25	0.96	1.67	4.03	.50	.46	.01	6.66
1963	237.82	243.80	259.80	282.80	0.87	1.44	4.90	.09	.23	.87	7.21
1964	237.90	243.60	254.00	282.85	0.83	1.04	7.40	.04	.40	2.50	9.27
1965	237.99	245.00	255.20	283.00	1.02	1.02	9.11	.19	.02	1.71	11.15
1966	238.09	244.40	255.30	268.00	0.92	1.11	7.38	.10	.09	1.73	9.41
1967	238.21	244.80	255.50	266.60	0.96	1.08	3.62	.04	.03	3.76	5.66
1968	238.22	245.70	256.10	265.90	1.09	1.05	3.27	.13	.03	.35	5.41
1969	238.23	245.30	257.00	266.30	1.03	1.18	3.04	.06	.13	.23	5.25
1970	238.73	245.25	257.10	266.70	0.95	1.20	3.27	.08	.02	.23	5.42
1971	238.77	245.50	257.50	266.90	0.98	1.22	3.07	.03	.02	.20	5.27
1972	238.78	245.30	257.90	266.90	0.95	1.29	2.91	.03	.07	.16	5.15
1973	239.30	246.10	258.90	266.90	0.99	1.31	2.58	.04	.02	.33	4.88
1974	239.42	246.70	258.95	266.90	1.06	1.26	2.55	.07	.05	.03	4.87
1975	239.97	246.90	259.85	266.80	1.01	1.33	2.22	.05	.07	.33	4.56
1976	238.81	246.85	260.20	266.40	1.17	1.34	2.09	.16	.01	.13	4.60
1977	238.21	247.00	260.30	266.70	1.28	1.36	2.09	.11	.02	0	4.73
1978	238.19	246.70	260.40	267.40	1.24	1.39	2.29	.04	.03	.20	4.92
1979	237.70	247.40	260.90	267.40	1.41	1.37	2.12	.17	.02	.17	4.90
1980	237.55	246.90	261.40	267.50	1.36	1.47	1.99	.05	.10	.13	4.82

SUPPLEMENT B--Continued

Data used for analysis of South Fork Forked Deer River

[S = water-surface slope in channel reach in feet per foot, |ΔS| = absolute value of change in water-surface slope in channel reach between two consecutive years, ΣS = sum of water-surface slopes in feet per foot, and VS/L = unit stream power for channel reach in feet per second per mile]

[In order to obtain actual values of S, |ΔS|, ΣS, and VS/L, multiply values shown in table by 10^{-4}]

Year	Yellow Bluff specific gage at 1000 ft^3/s	Halls specific gage at 1000 ft^3/s	Gates specific gage at 1000 ft^3/s	S between Yellow Bluff and Halls	S between Halls and Gates	|ΔS| between Yellow Bluff and Halls	|ΔS| between Halls and Gates	ΣS between Halls and Gates	Velocity at Halls	Velocity at Gates	VS/L between Yellow Bluff and Halls	VS/L between Halls and Gates
1950	258.20	265.40		2.78								
1951	258.30	265.40		2.74		.04						
1952	258.50	265.38		2.66		.08						
1953	258.38	265.43		2.72		.06						
1954	258.57	265.69		2.75		.03						
1955	259.00	266.00		2.71		.04						
1956	259.05	266.20		2.76		.05						
1957	259.80	266.20		2.47		.29						
1958	260.00	266.40		2.47		0						
1959	259.95	266.50		2.53		.06						
1960	260.20	266.60		2.47		.06			1.72		0.860	
1961	260.20	266.60		2.47		0			1.70		0.864	
1962	260.20	266.70		2.51		.04			1.65		0.838	
1963	260.20	266.70		2.51		0			1.48		0.752	
1964	260.20	266.70		2.51		0			1.70		0.850	
1965	260.00	267.00		2.71		.20			1.46		0.810	
1966	260.40	267.40		2.71		0			1.50		0.823	
1967	260.40	267.60		2.78		.07			1.55		0.873	
1968	256.20	267.60		4.41		1.63			1.31		1.174	
1969	248.60	267.60	274.60	7.82	1.58	3.41		9.40	1.14	1.84	1.942	0.350
1970	249.90	264.70	274.40	6.09	2.19	1.73	.61	8.28	2.05	2.00	2.723	0.519
1971	250.31	264.00	274.30	5.64	2.32	.45	.13	7.96	1.83	2.30	2.238	0.635
1972	250.00	263.20	274.00	5.43	2.44	.21	.12	7.87	2.09	2.34	2.476	0.674
1973	252.40	262.60	274.20	4.20	2.62	1.23	.18	6.82	1.74	1.87	1.596	0.579
1974	253.10	259.00	274.10	2.43	3.40	1.77	.78	5.83	2.13	2.25	1.116	0.911
1975	253.70	258.40	273.60	1.94	3.43	.49	.03	5.37	2.02	2.17	0.861	0.883
1976	253.80	258.40	272.40	1.89	3.16	.05	.27	5.05	2.05	1.67	0.820	0.624
1977	254.00	258.20	272.39	1.73	3.20	.16	.04	4.93	1.32	1.60	0.511	0.608
1978	254.00	257.60	271.15	1.48	3.06	.25	.14	4.54	1.37	2.35	0.411	0.856
1979	255.20	257.52	269.73	0.96	2.75	.52	.31	3.71	1.42	2.39	0.296	0.782
1980	255.61	257.29	269.62	0.69	2.78	.27	.03	3.47	1.35	1.98	0.202	0.655
1981	255.73	257.29	269.39	0.64	2.73	.05	.05	3.37	1.33	1.74	0.185	0.565

SUPPLEMENT B--Continued

Data used for analysis of South Fork Obion River

[S = water-surface slope in channel reach in feet per foot and $|\Delta S|$ = absolute value of charge in water-surface slope in channel reach between two consecutive years]

[In order to obtain actual values of S and $|\Delta S|$, multiply values shown in table by 10^{-4}]

Year	Kenton specific gage at 500 ft^3/s	Greenfield specific gage at 500 ft^3/s	McKenzie specific gage at 500 ft^3/s	S between Kenton and Greenfield	$\|\Delta S\|$ between Kenton and Greenfield
1950	287.40	311.20		3.36	
1951	288.00	310.90		3.24	.12
1952	288.20	310.70		3.18	.06
1953	288.20	310.60		3.17	.01
1954	287.80	310.50		3.21	.04
1955	288.00	310.40		3.17	.04
1956	288.60	310.30		3.07	.10
1957	289.00	310.30		3.01	.06
1958	289.50	310.20		2.93	.08
1959	289.80	310.20		2.88	.05
1960	290.20	310.20		2.83	.05
1961	290.40	310.20		2.80	.03
1962	290.60	310.10		2.76	.04
1963	290.80	310.10		2.73	.03
1964	291.00	310.10		2.70	.03
1965	291.60	310.00		2.59	.11
1966	283.00	310.00		3.79	1.20
1967	280.20	310.00		4.21	.42
1968	278.40	309.80		4.44	.23
1969	277.10	309.70	346.90	4.61	.17
1970	276.90	309.30	347.00	4.58	.03
1971	276.50	309.20	347.40	4.62	.04
1972	276.40	309.20	347.70	4.64	.02
1973	276.50	309.20	347.70	4.62	.02
1974	276.45	309.20	347.70	4.62	0
1975	276.20	309.20	347.80	4.66	.04
1976	276.40	309.20	347.80	4.64	.02
1977	276.60	309.20	347.90	4.61	.03
1978	277.00	309.10	347.90	4.54	.07
1979	277.90	307.10	348.00	4.13	.41
1980	278.00	306.30	348.00	4.00	.13
1981	278.20	306.00	348.00	3.93	.07

SUPPLEMENT B--Continued

Data used for analysis of North Fork Obion River

[S = water-surface slope in channel reach in feet per foot and ΣS = sum of water-surface slopes in feet per foot

[In order to obtain actual values of S and ΣS, multiply values shown in table by 10^{-4}]

Year	Rives' specific gage at 500 ft^3/s	Union City specific gage at 500 ft^3/s	Martin specific gage at 500 ft^3/s	Palmersville specific gage at 500 ft^3/s	S between Rives' and Union City	S between Union City and Martin	S between Martin and Palmersville	S between Rives and Rives'	ΣS between Rives' and Martin	ΣS between Rives and Rives'
1950	285.40	294.20	312.40		4.07	4.31			8.38	
1951	285.40	294.80	312.60		4.34	4.21			8.55	
1952	285.79	295.00	312.79		4.25	4.21			8.46	
1953	285.60	295.60	313.00		4.62	4.12			8.74	
1954	285.70	295.20	313.20		4.39	4.26			8.65	
1955	285.80	295.30	313.30		4.39	4.26			8.65	
1956	285.80	295.20	313.40		4.34	4.31			8.65	
1957	286.00	295.20	313.60		4.25	4.36			8.61	
1958	286.80	295.30	313.60		3.93	4.33			8.26	
1959	287.30	295.45	313.60		3.76	4.30			8.06	
1960	287.40	295.95	313.60		3.95	4.23			8.18	
1961	287.80	296.60	314.19		4.07	4.16			8.23	
1962	288.20	296.60	314.40		3.88	4.21			8.09	
1963	288.80	297.35	314.40		3.95	4.04		.867	7.99	8.86
1964	289.30	297.80	314.50		3.93	4.19		.932	8.12	9.05
1965	289.40	297.60	314.60		3.79	4.02		.932	7.81	8.74
1966	281.00	293.40	314.70		5.73	5.04		1.89	10.77	12.66
1967	279.80	292.30	314.60		5.77	5.28		1.92	11.05	12.97
1968	280.00	291.30	314.60		5.22	5.52		2.04	10.74	12.78
1969	280.10	291.20	314.60	366.50	5.13	5.54	5.82	2.01	10.67	12.68
1970	280.31	290.98	314.60	367.00	4.93	5.59	5.87	1.84	10.52	12.36
1971			314.59	367.10			5.88			
1972			314.40	367.20			5.92			
1973			313.90	367.60			6.02			
1974			313.65	367.60			6.05			
1975			313.60	367.60			6.05			
1976			313.60	367.60			6.05			
1977			313.54	367.40			6.04			
1978			313.27	367.40			6.07			
1979			312.85	367.40			6.11			
1980			311.73	367.30			6.23			

Rives - Obion River at Rives
Rives' - North Fork Obion River at Rives

SUPPLEMENT B--Continued

Data used for analysis of Hatchie River

[S = water-surface slope in channel reach in feet per foot, $|\Delta S|$ = absolute value of change in water-surface slope in channel reach between two consecutive years, and ΣS = sum of water-surface slopes in feet per foot]

[In order to obtain actual values of S, ΣS, and $|\Delta S|$, multiply values in table by 10^{-4}]

Year	Bolivar specific gage at 2000 ft^3/s	Pocahontas specific gage at 2000 ft^3/s	Walnut specific gage at 2000 ft^3/s	S between Bolivar and Pocahontas	S between Pocahontas and Walnut	ΣS between Bolivar and Walnut	$\lvert\Delta S\rvert$ between Bolivar and Pocahontas	$\lvert\Delta S\rvert$ between Pocahontas and Walnut
1960	334.70	364.90	394.80	1.33	8.09	9.42		
1961	335.20	364.90	395.00	1.31	8.14	9.45	.02	.03
1962	334.80	365.10	394.70	1.34	8.01	9.35	.03	.10
1963	334.80	365.00	394.70	1.33	8.04	9.37	.01	.02
1964	335.20	364.90	394.70	1.31	8.06	9.37	.02	0
1965	335.20	364.90	394.50	1.31	8.01	9.32	0	.05
1966	335.20	365.10	394.60	1.32	7.99	9.31	.01	.01
1967	334.80	365.10	394.50	1.34	7.95	9.29	.02	.02
1968	335.10	365.10	394.80	1.32	8.04	9.36	.02	.07
1969	335.10	365.10	394.90	1.32	8.06	9.38	0	.02
1970	335.40	365.05	394.80	1.31	8.05	9.36	.01	.02
1971	335.40	365.00	394.80	1.31	8.06	9.37	0	.01
1972	335.40	365.25	395.00	1.32	8.05	9.37	.01	0
1973	335.40	365.40	394.80	1.32	7.95	9.27	0	.10
1974	335.70	365.50	394.80	1.32	7.93	9.25	0	.02
1975	335.70	366.65	394.90	1.37	7.64	9.01	.05	.24
1976	335.70	366.60	394.90	1.36	7.66	9.02	.01	.01
1977	335.70	366.50	394.90	1.36	7.68	9.04	0	.02
1978	335.70	366.40	394.90	1.36	7.71	9.07	0	.03