AD-A121 139 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

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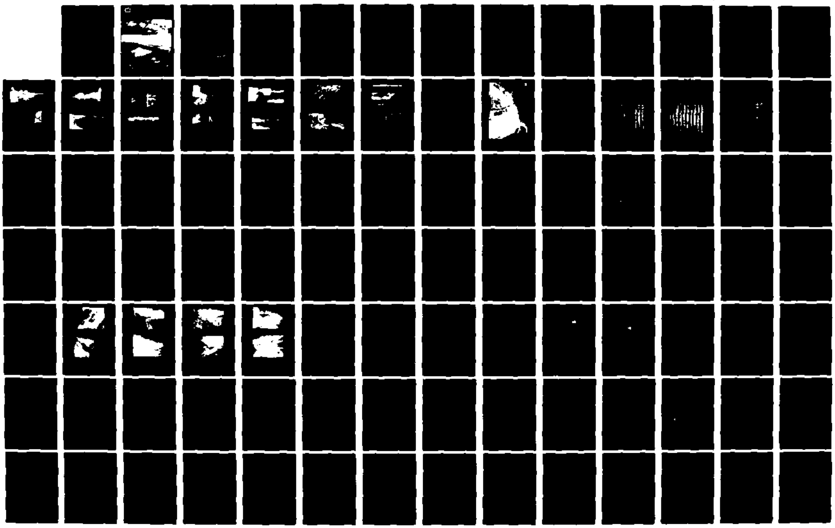
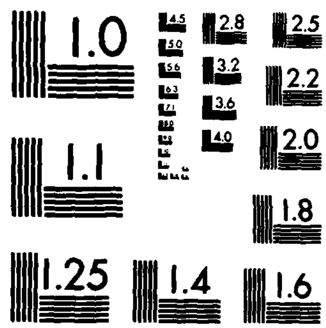




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

US Army Corps of Engineers

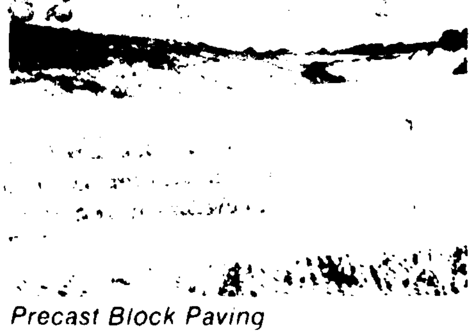
December 1981

THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 SECTION 32, PUBLIC LAW 93-251

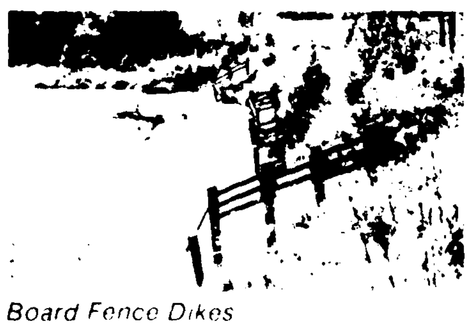
Appendix G - Demonstration Projects on Other Streams, Nationwide

Volume 2 of 2

Rock Toe With Tie-Backs



Precast Block Paving



Board Fence Dikes

FINAL REPORT TO CONGRESS

THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 SECTION 32, PUBLIC LAW 93-251

APPENDIX G

DEMONSTRATION PROJECTS ON OTHER STREAMS, NATIONWIDE

VOLUME 2 OF 2

Consisting of

A BRIEF SUMMARY REPORT AND INDIVIDUAL EVALUATION REPORTS ON TWENTY STREAMBANK EROSION CONTROL DEMONSTRATION PROJECTS ON SIXTEEN DIFFERENT STREAMS THROUGHOUT THE UNITED STATES

U.S. ARMY CORPS OF ENGINEERS
December 1981

APPENDIX G

Demonstration Projects on Other Streams, Nationwide

CONTENTS

VOLUME 1 of 2

VOLUME 2 of 2

IOWA RIVER AT
WAPELLO, IOWA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

IOWA RIVER AT WAPELLO, IOWA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Streambank Erosion Demonstration Project, Iowa River at Wapello, Iowa. See Plate 1 for location map.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem, the types of bank protection used, and a performance evaluation of a demonstration project on the Iowa River at Wapello, Iowa, constructed and monitored by the Rock Island District.

4. Problem Resume. The city of Wapello, Iowa, is the county seat of Louisa County and is located on the right bank of the Iowa River, approximately 16 miles above its confluence with the Mississippi River. At the upstream end of the community, the river makes a 90° bend, and over the years the right bank has been eroding. The main business district is within 180 feet of the riverbank; without man made modifications this commercial area will be subject to erosion damage in the near future. The general area is shown on Plate 2.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The combined basin of the Iowa and Cedar Rivers covers an area of 12,640 square miles of which 11,590 square miles are in Iowa and 1,050 square miles are in Minnesota. The basin is about 230

miles in length and averages 60 miles in width, reaching from north-central Iowa and south central Minnesota to southeastern Iowa.
The basins of both the Iowa River and the Cedar River are generally long and narrow. The Iowa River's average slope per mile is 1.9 feet per mile.

b. Geology. Generally, the Iowa-Cedar Basin is gently rolling prairie land, lying at elevations less than 150 feet above the streams. All of the basin has been covered by deposits of the two earliest ice sheets, the Nebraskan and the Kansan. In the lower reaches of the watershed, except in parts of Louisa and Muscatine Counties, the surface deposits are of the Kansan ice sheet, covering entirely those of the Nebraskan; and the topography is erosional and maturely drained. In parts of Louisa and Muscatine Counties, the surface glacial drift is of the Illinoian, the third ice sheet, and the topography is also mature. The streams have cut deeply into the drift of the Kansan and Illinoian and have usually developed wide flood plains.

North of the Johnson-Linn County line, extending into Mower County in Minnesota, the surface deposits over much of the basin are of the Iowan, the earliest substage of the Wisconsin, the fourth glaciation. This topography is generally erosional and well drained except in the upper reaches where isolated swamps and bogs, some of them now artificially drained, exist. The streams in the Iowa drift region are generally in steep valleys, though fairly wide flood plains are sometimes developed.

In the upper reaches of the Iowa River watershed and portions of the Cedar River watershed, the surface deposits are of the latest substage of the Wisconsin. The topography is characterized by irregularly spaced morainic hills and by marshes and peat bogs which were undrained or poorly drained before artificial drainage. The streams in this region have shallow channels in their upper reaches, but as their watersheds increase in area downstream, the channels are cut deeper into the glacial till and often into rock.

c. <u>Natural Resources</u>. The Iowa-Cedar River basin's most valuable natural resource is its rich farmland and generally abundant rainfall. In many parts of the basin, limestone can be found close to the surface. This limestone is used for the manufacture of Portland Cement, as building stone, and for macadam roads. However, in the Wapello area farming is the dominant livelihood.

d. <u>Land Development and Use</u>. About 95 percent of the basin is in farmland, and 77 percent of the farmland is used for crops and pasture. Farm woodland varies from about 8 to 10 percent in the southern part of the basin to 2 to 4 percent in the northern part.

e. <u>Transportation</u>. Wapello is located on US Highway 61, which is the major north-south route along the eastern edge of Iowa. Iowa Highway 99 also connects Wapello to points directly east and south. State Highway 91, which runs the width of Iowa, is located six miles north of Wapello at US 61. Wapello is also served by the CRI&P Railroad and numerous county roads.

f. <u>Hydrologic Characteristics</u>. The Iowa-Cedar watershed has a typical continental climate in the temperate zone. The climate is typified by conditions at Waterloo, where the average annual temperature is about 48 degrees. Extreme monthly averages are 19 degrees for January, and 74 degrees for July. Average annual precipitation is about 31.8 inches, with runoff of about 7 inches. Snowfall averages 29 inches. The Iowa River at the town of Wapello, which is near its mouth, has a bank-full capacity of 29,000 cubic feet per second, and at bank-full stage has a width of about 740 feet and a mean depth of 10.7 feet. At the city of Cedar Rapids, the Cedar River has a bank-full capacity of 10,000 cubic feet per second, and at bank-full stage has a width of 485 feet and a mean depth of 5.1 feet. No sediment, wave, or ice studies have ever been conducted on the Iowa River in the Wapello vicinity.

g. Environmental Characteristics. Five thousand dollars has been set aside for a study by Rock Island District environmental section staff to determine if any changes to the cultural, water quality, fish, etc., have occurred due to the streambank construction measures undertaken at Wapello.

6. Demonstration Site-Test Reach.

a. Hydrologic Characteristics. As previously stated, the annual precipitation at Waterloo, which is similar to Wapello, is 31.8 inches. The nearest stream gaging station is located at Wapello at the Highway 99 bridge on the downstream end of the project. Plate 3 is a river stage versus duration curve of the Iowa River at the Wapello gage. Plates 4 and 5 show the hydrographs developed from data obtained at this gaging station.

b. Hydraulic Characteristics. Flood flow velocities in the Iowa River range from 0.1 to 0.15 f.p.s. at discharges of about 900 c.f.s. to 6.0 to 7.0 f.p.s. as discharges approach 35,000 c.f.s. Plate 6 shows the velocity vs. discharge *relationship* taken at the Wapello gaging station. Channel cross-section locations are shown on Plate 7. Channel cross-sections were taken at 19 ranges along the test reach and are shown on Plates 8 thru 17. The velocity ranges are shown on Plate 18. Velocity distribution within the channel cross-section for these selected ranges are shown on Plate 19 thru 21. The flood of record for a period from 1916 to 1979 is the 1973 flood which had a peak stage of 28.63 feet and a peak discharge of 92,000 cubic feet per second. See Plate 25 for estimated yearly peak flood velocities.

c. Riverbank Description.

(1) Bank Materials. Materials composing the banks of the Iowa River are of two principal soil types. The top portion of the bank consists of a sandy clay till surface layer underlain with a brown sand which is fine to coarse and has a medium density. The bottom portion of

the bank is a glacial till that is a very tough gray clay. This layer is about seventeen feet thick. A typical section of the riverbank showing the principal soil types and their thicknesses is shown on Plate 22. Plate 23 shows the boring logs of three soil borings taken adjacent to the Iowa River in Wapello. These borings show that the depths of the principal soil types of the Iowa River bank adjacent to Wapello are quite uniform.

(2) Normal Bank Vegetation. Vegetation cover on the banks consists mainly of grasses, willows, cottonwoods, and various other shrubs and quick-growth trees. Vegetation is present in nearly all photographs.

(3) Bank Erosion Tendencies. The test site had been eroding at a rate of up to 2 feet per year. In some areas it has been determined that over 200 feet of the right riverbank have eroded away since 1848. Plate 2 displays the historical and projected erosion of the riverbank.

The erosion rate of the Iowa River bank at Wapello is controlled by the rate that the Iowa River flows wear away the tough clay comprising the lower portion of the riverbank. Of the two principal soil types that make up the riverbank (see plate 22), the lower clay portion is much more erosion resistant. However, due to the moderately dense vegetation on the upper sand portion of the riverbank, this portion has held up quite well during high flows that subject it to erosive forces. Over 90 percent of the time the river levels fluctuate within the lower clay portion of the riverbank as shown on Plate 3. Therefore, most of the erosive forces of the Iowa River act on this portion of the riverbank.

III. DESIGN AND CONSTRUCTION

7. General. Wapello is situated on a terrace that is 25 to 30 feet above the normal river levels. The Iowa River bank erosion has been a problem at Wapello since the community was established in the mid 1800's.

However, it was not until the mid 1900's that the erosion had progressed to the point that existing residential and commercial structures were being threatened.

In the past, the Wapello community and individual property owners have exerted considerable effort to stabilize the riverbank. A deflection da or jetty has been constructed between Mechanic Street and Van Buren Street. There have been large quantities of rubble dumped along the riverbank. These efforts may have reduced the erosion rate; however, they have not been effective in eliminating the bank erosion problem.

8. Basis for Design. The primary reason for selecting the combinatio plan was the fact that this project was a demonstration site. Using th combination of permeable timber jetties, erosion control mat, and Kelln Jacks, allows engineering scrutiny as to which erosion control measures are beneficial for future use. The jetties are designed to direct flov to the center of the channel; the jacks are placed to stabilize the toe of the sloping bank; and the erosion mat was designed to protect the ba against erosion during high flows.

9. Construction Details. The steel jacks were constructed of the materials as shown and identified on Plate 24. At the Wapello site, 7 of the steel jacks were arranged on one line on 15-foot centers extend upstream from the Highway 99 bridge and parallel to the bank. Plate 7 illustrates the layout of the steel jacks. Photo 1 shows the area of site before construction. The steel jacks during and after constructi are shown in photos 2 and 3, respectively.

The permeable timber jetties were constructed of the materials as show and identified on Plate 24. At the Wapello site, six ranges of the timber jetties were located as shown on Plate 7. Photos 4, 5, and 6 the site area before, during, and after construction, respectively.

The erosion control mat was constructed as shown on Plate 24. The mat was placed on the area indicated on Plate 7. Photos 7, 8, and 9 show the site area before, during, and after construction, respectively.

Construction was done from June through September 1978.

10. Cost. The total cost of fabrication and installation of streambank erosion control measures amounted to $223,015.97.

IV. PERFORMANCE OF PROTECTION

11. Rock Island District is now monitoring this project four times a year, including yearly surveys. Photo 10 is an example of the damage to the timber jetties. Some panels are bent or missing. Some panels were probably lifted off by ice action. Photo 11 illustrates some spalling on the surface of the erosion control mat, apparently the aftermath of a fisherman's campfire. The fabric covering of the erosion control mat has been cut by the jacks or cables in a few places. A break in the concrete erosion control mat was noted in April of 1981. The break is about 3 to 5 feet long. A knife blade could penetrate the break to about 1/2 inch, although the break is obviously the total depth of the mat. See photo 14. However, other than these minor problems, the fabric is in good condition. Photo 12 illustrates the damage to the steel jacks on the upstream end. The anchor cable securing these upstream jacks was broken during ice flow conditions.

12. Evaluation of Protection Performance. Flooding in the spring of 1979 resulted in some damage to timber jetties and to the steel jacks. This damage is shown in photos 10 and 12 as mentioned previously.

The erosion control mat and timber jetties protection has been effective in reducing additional bank erosion. The steel jacks, however, were not as effective as planned. This is probably due to two conditions. One, water and ensuing debris are moving too fast when they strike the steel jacks. Steel jacks operate better in catching debris and deflecting the current when the water is at a lower velocity. Second, ice lifting and heaving resulted in some of the steel jacks being flattened and moved from their original position in the river to a flattened position on the bank.

13. Rehabilitation. Rock Island District personnel from Operations Division did repair work on the timber jetties during the summer of 1980. This work consisted of repairing or replacing damaged and broken wood panels and welding pipe extensions to some of the pipe piling. This work was completed at a cost of $16,106.20. The broken cable previously mentioned was extended and anchored by Rock Island District personnel to a tree growing near the top of the riverbank. The cost of this repair work was $400.

14. Conclusion. The effectiveness of the erosion control mat, timber jetties, and steel jacks has been demonstrated by flooding at the Wapello site in the spring of 1979. An important consideration in determining the proper streambank erosion measure is to determine the water velocity at different reaches of the project. This will ensure that the proper type of streambank protection is placed where it will do the most good. Another important point to consider is ice causing damage to streambank improvements due to heaving and uplift of ice. The Corps of Engineers will continue to monitor this project.

PHOTO 1. LOOKING UPSTREAM AT WAPELLO RIVERBANK FROM HIGHWAY 99 BRIDGE. 8 JUNE 1978.

PHOTO 2. STEEL JACKS DURING CONSTRUCTION.

PHOTOS 1 AND 2

PHOTO 3. STEEL JACKS IN PLACE AFTER CONSTRUCTION.

PHOTO 4. LOOKING UPSTREAM FROM THE TOP OF IOWA RIVER BANK TO AREA WHERE THE TIMBER JETTIES WILL BE CONSTRUCTED. 8 JUNE 1978.

PHOTOS 3 AND 4

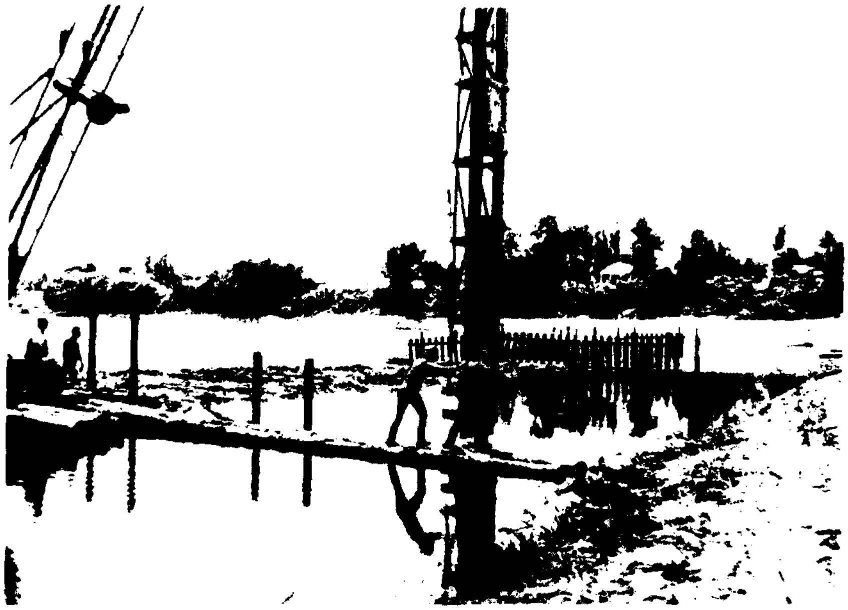


PHOTO 5. TIMBER JETTIES DURING CONSTRUCTION. 24 AUGUST 1978.

PHOTO 6. LOOKING UPSTREAM FROM STA 18+00 AT TIMBER JETTIES.
19 OCTOBER 1978.

PHOTOS 5 AND 6

PHOTO 7. AREA WHERE EROSION CONTROL MAT IS TO BE PLACED.
8 JUNE 1978.

PHOTO 8. PUMPING GROUT INTO THE EROSION CONTROL MAT.
24 AUGUST 1978.

PHOTOS 7 AND 8

PHOTO 9. STA 12+00 LOOKING UPSTREAM AT CONCRETE MAT.
14 NOVEMBER 1979.

PHOTO 10. DAMAGED TIMBER JETTY AT STA 26+00. 7 MAY 1980.

PHOTOS 9 AND 10

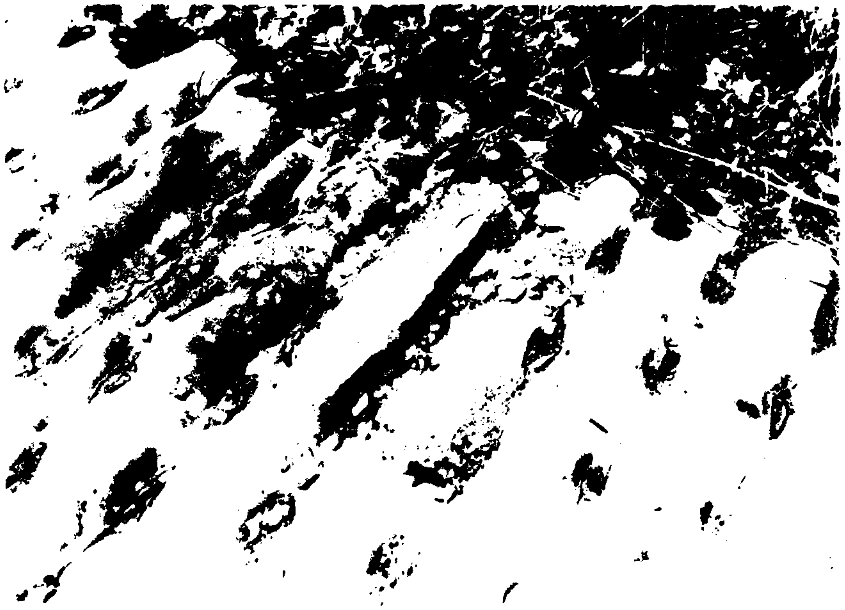


PHOTO 11. CONCRETE SPALLING OF THE EROSION CONTROL MAT. 1980.

PHOTO 12. STA 6+00 LOOKING UPSTREAM AT DAMAGED STEEL JACKS.
14 NOVEMBER 1979.

PHOTOS 11 AND 12

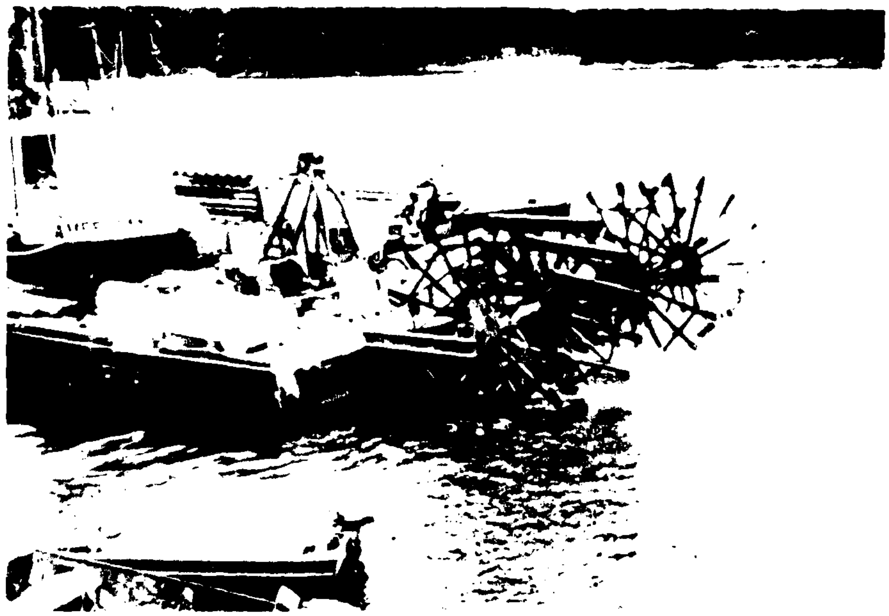


PHOTO 13. WORK BARGE USED DURING CONSTRUCTION.

PHOTO 14. BREAK IN CONCRETE EROSION CONTROL MAT.

PHOTOS 13 AND 14

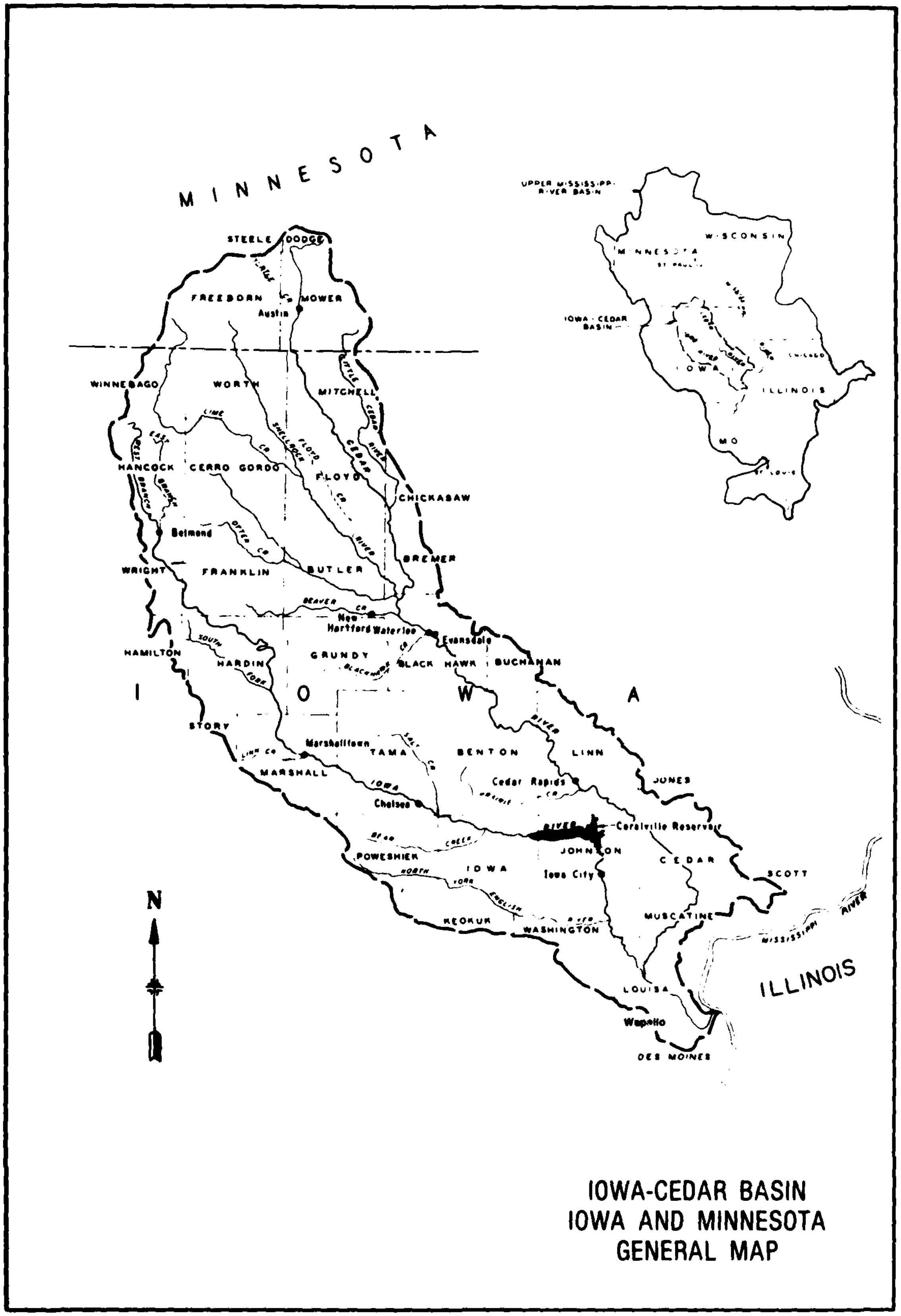
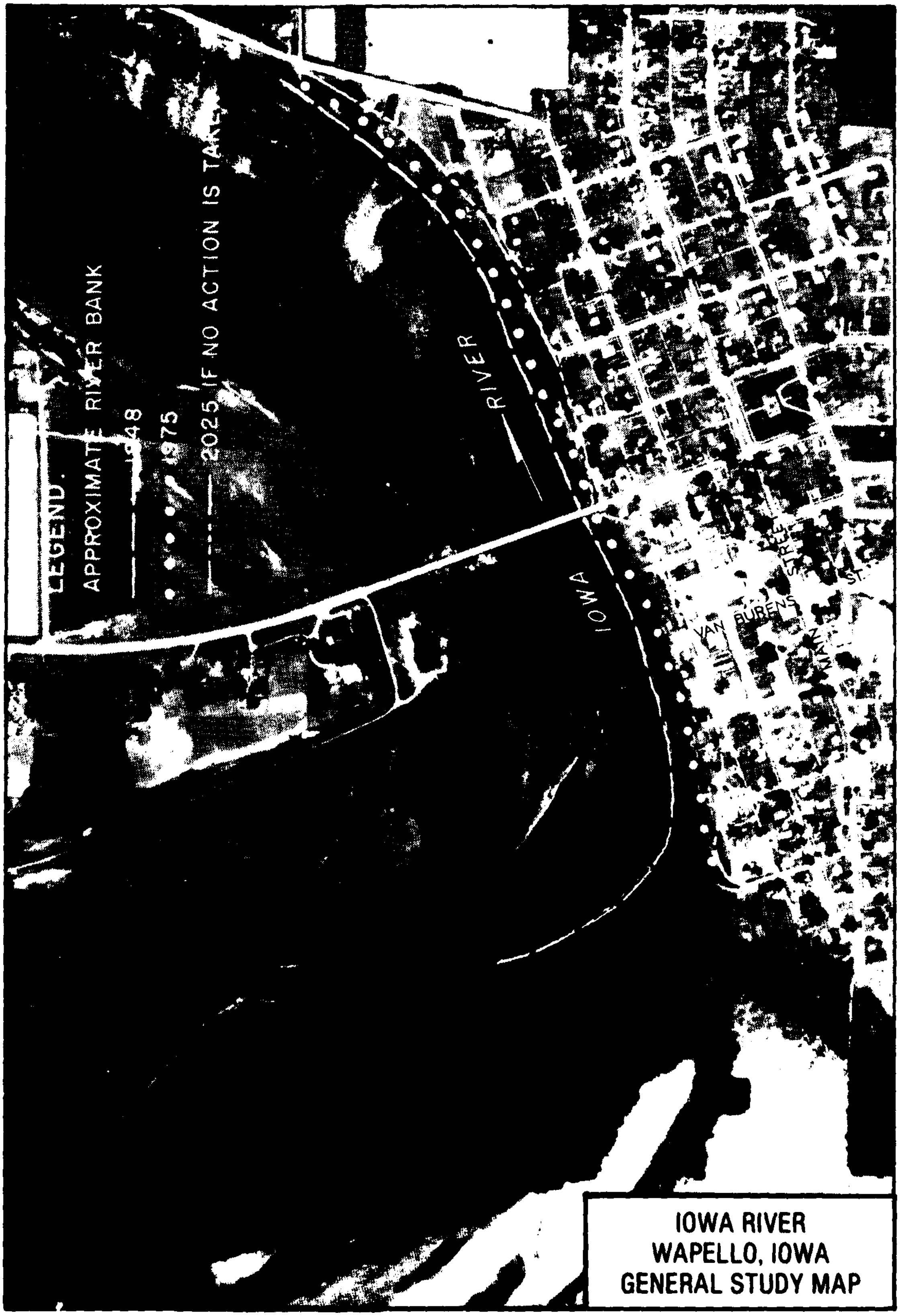
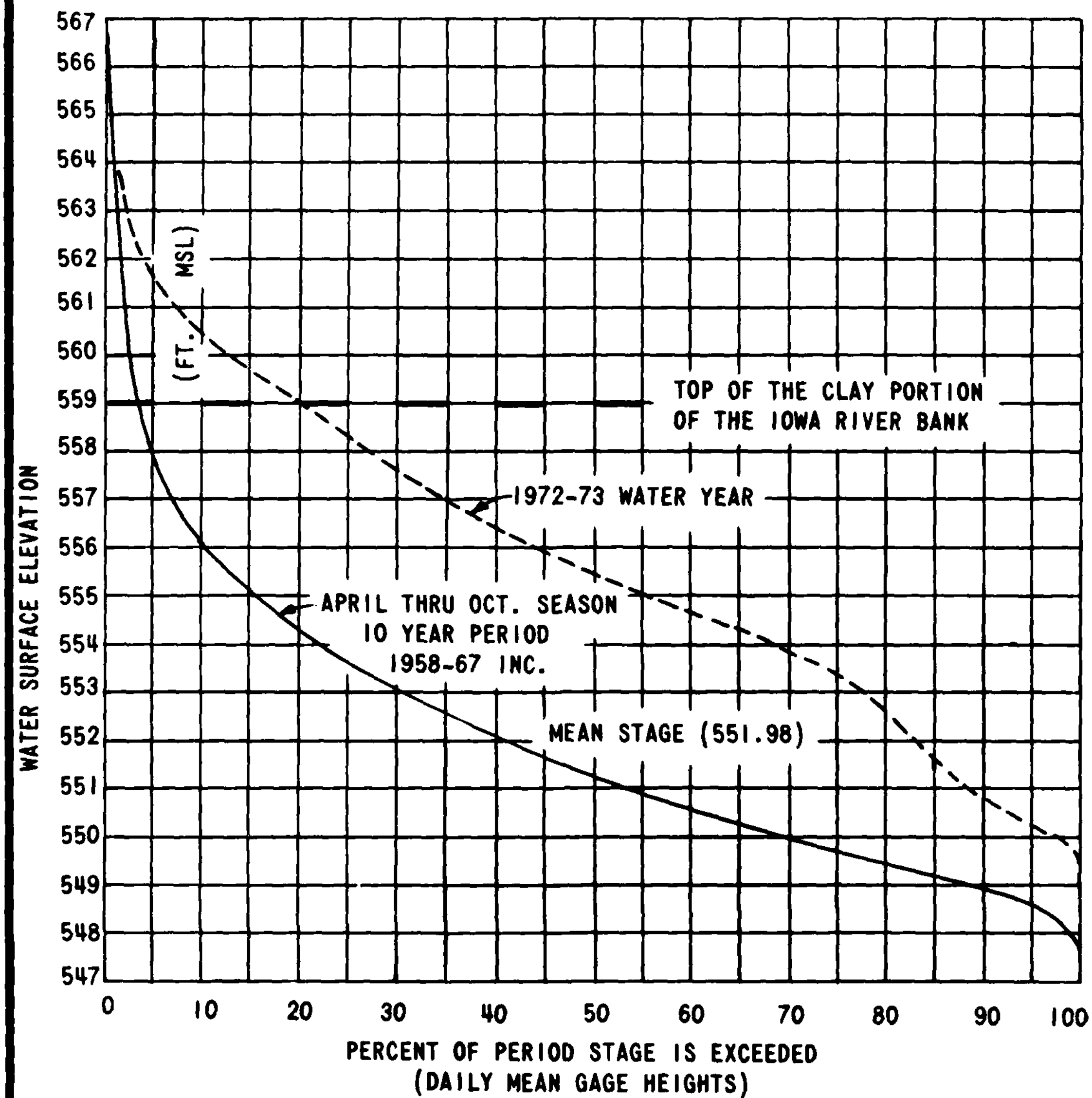

MINNESOTA
IOWA
ILLINOIS
STEELE
DODGE
FREEBORN
MOWER
Austin
WINNEBAGO
WORTH
MITCHELL
HANCOCK
CERRO GORDO
FLOYD
CHICKASAW
Belmond
WRIGHT
FRANKLIN
BUTLER
BREMER
New Hartford
Waterloo
Evansdale
HAMILTON
HARDIN
GRUNDY
BLACK HAWK
BUCHANAN
STORY
Marshalltown
TAMA
BENTON
LINN
MARSHALL
Cedar Rapids
JONES
Chelsea
Coralville Reservoir
JOHNSON
CEDAR
POWESHIEK
IOWA
Iowa City
SCOTT
KEOKUK
WASHINGTON
MUSCATINE
LOUISA
Wapello
DES MOINES
N
UPPER MISSISSIPPI RIVER BASIN
WISCONSIN
IOWA-CEDAR BASIN
ILLINOIS
MO
IOWA-CEDAR BASIN
IOWA AND MINNESOTA
GENERAL MAP

PLATE 1



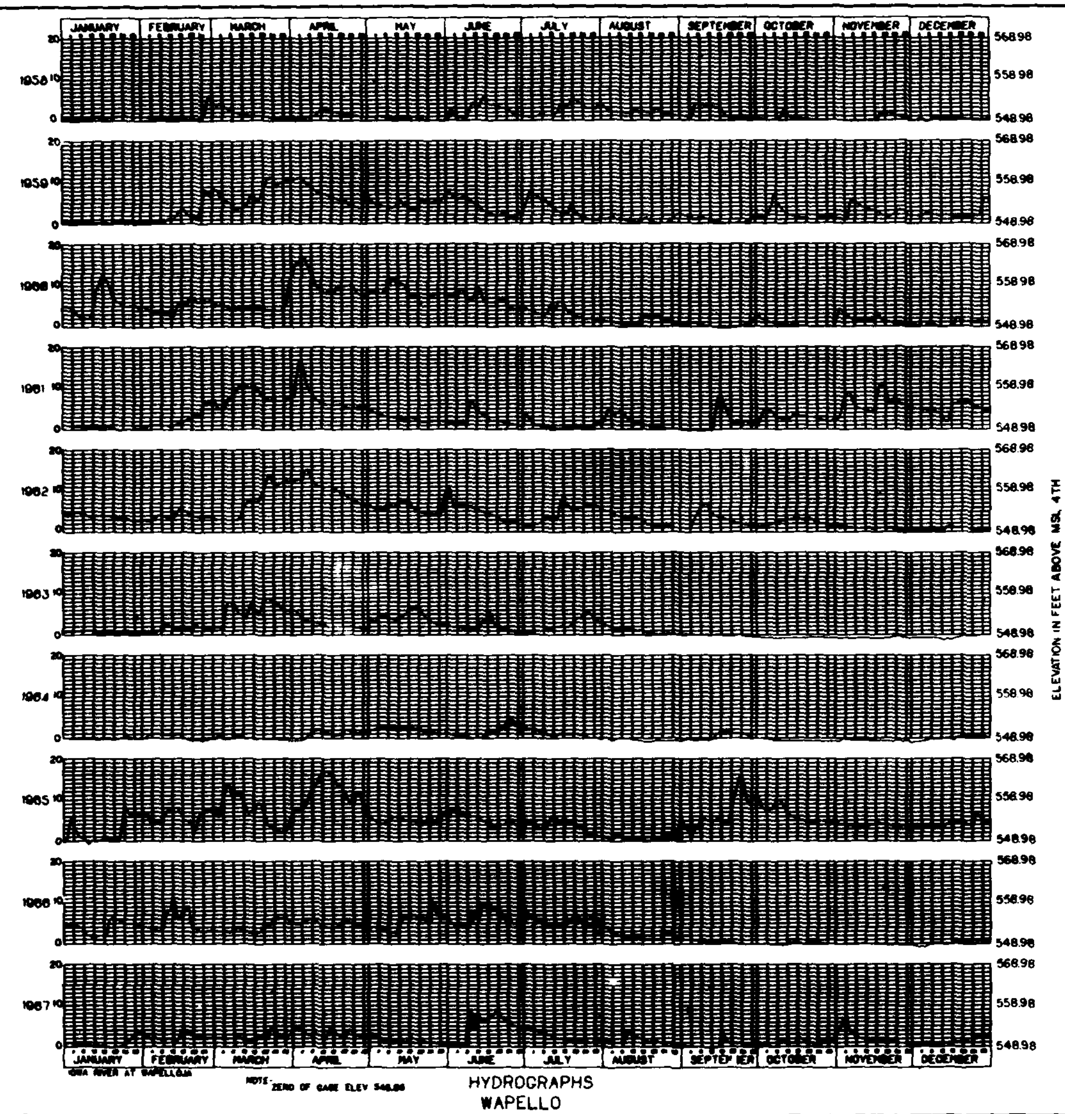
IOWA RIVER
WAPELLO, IOWA
GENERAL STUDY MAP

PLATE 2



IOWA RIVER
WAPELLO, IOWA
RIVER STAGE – DURATION CURVES

PLATE 3

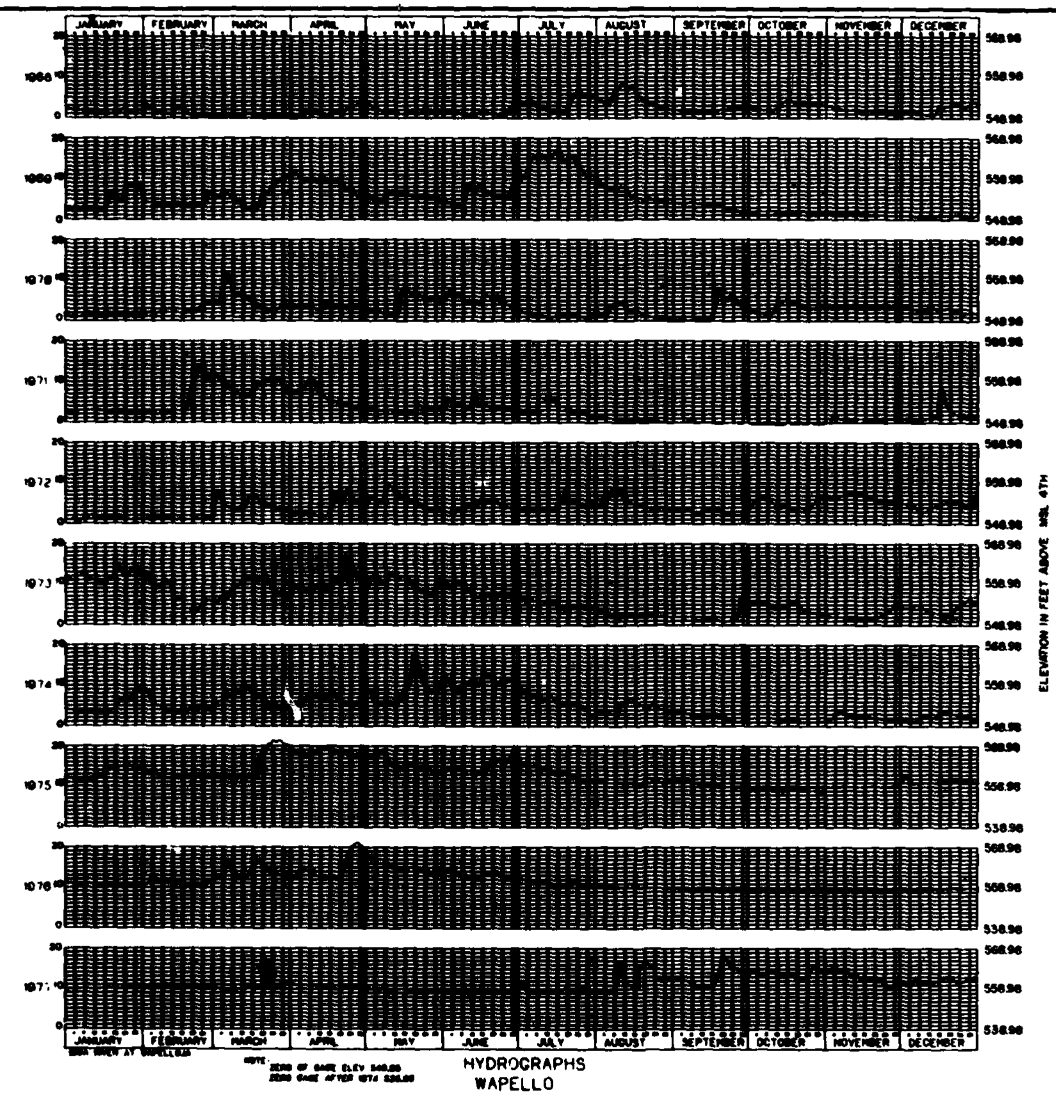


NOTE: GAGE HEIGHTS SHOWN IN LEFT MARGIN ARE 0, 10, AND 20 FEET. ELEVATIONS IN RIGHT MARGIN ARE 548.98, 558.98, AND 568.98 FEET

IOWA RIVER
WAPELLO, IOWA
HYDROGRAPHS
1958-1967

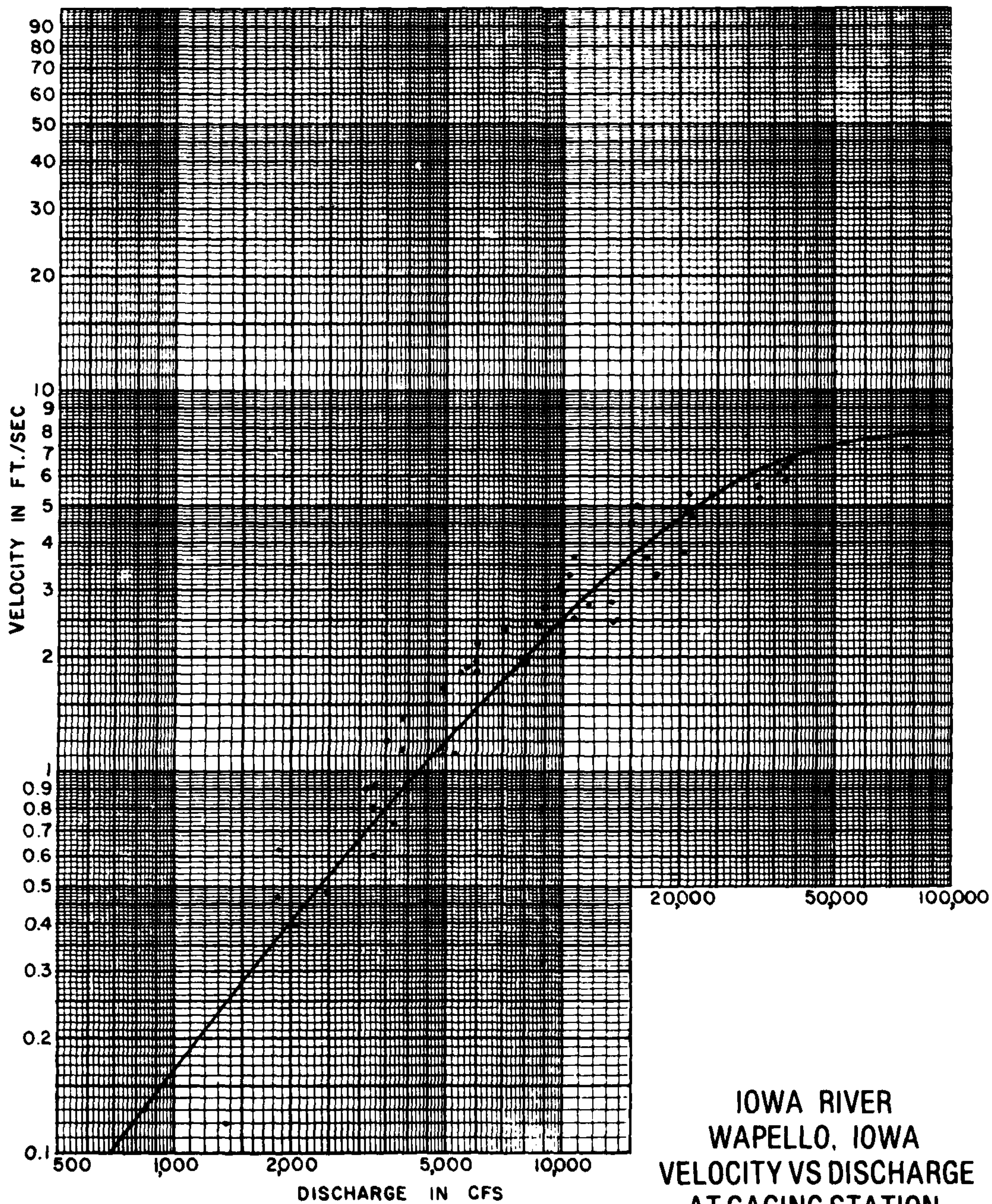
PLATE 4

PLATE 5



NOTE: GAGE HEIGHTS SHOWN IN LEFT MARGIN ARE 0, 10, AND 20 FEET. ELEVATIONS IN RIGHT MARGIN ARE 548.98, 558.98, AND 568.98 FEET THROUGH 1974. BEGINNING 1975 THEY ARE 538.98, 548.98, AND 558.98 FEET.

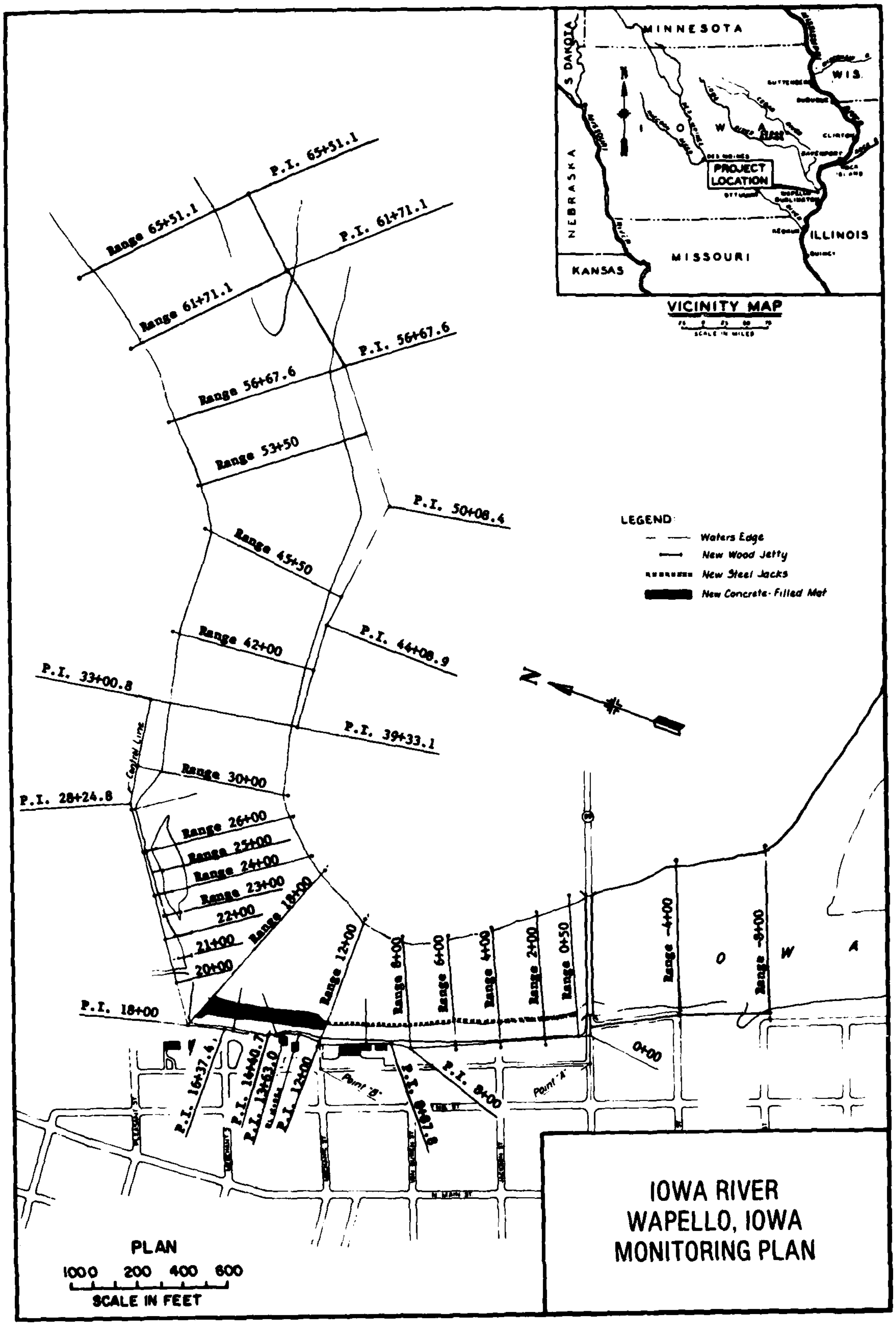
IOWA RIVER
WAPELLO, IOWA
HYDROGRAPHS
1968-1977



IOWA RIVER
WAPELLO, IOWA
VELOCITY VS DISCHARGE
AT GAGING STATION

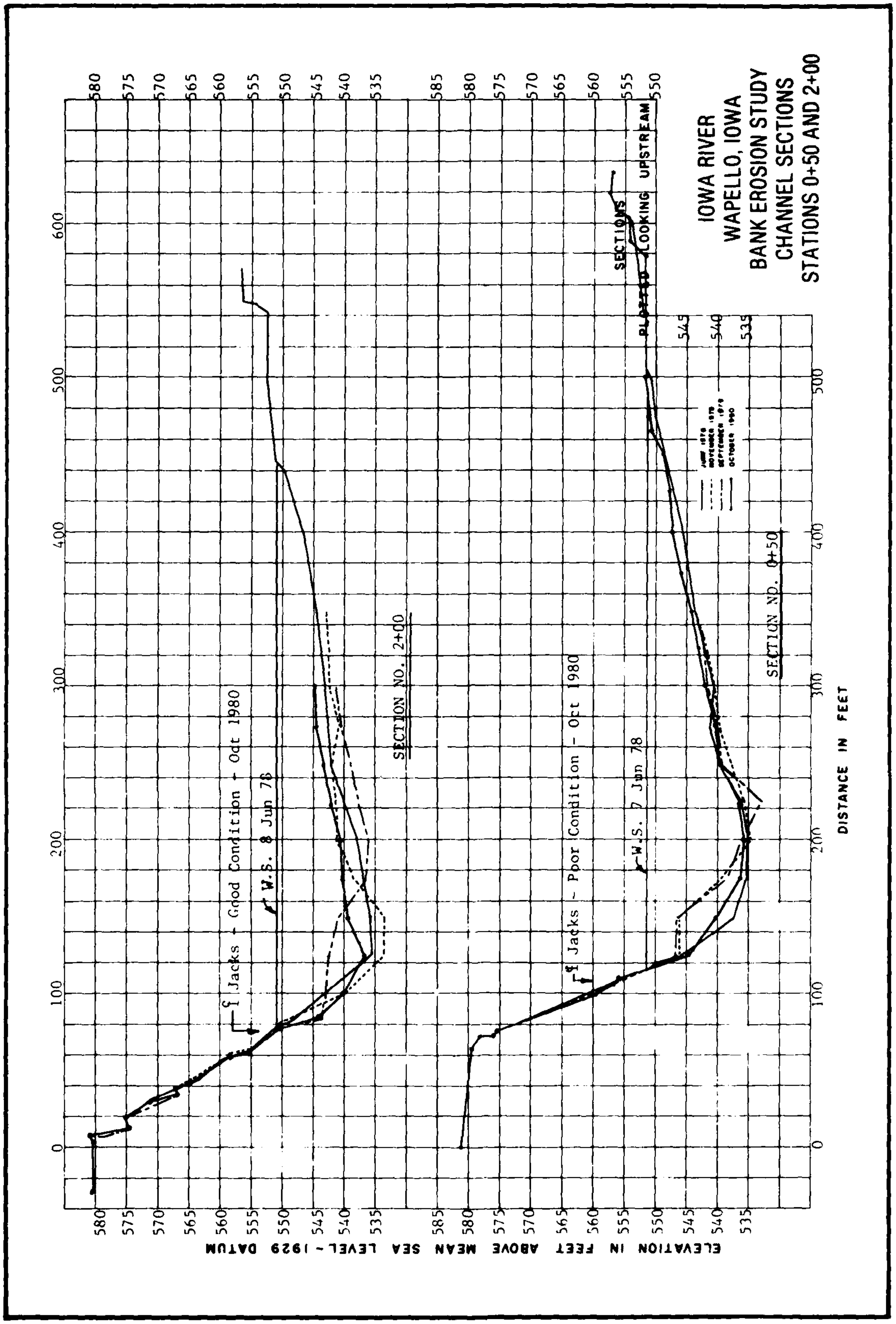
VELOCITIES ARE APPROXIMATELY 40 FT

PLATE 6



MINNESOTA
S DAKOTA
WIS.
IOWA
NEBRASKA
PROJECT LOCATION
ILLINOIS
MISSOURI
KANSAS
VICINITY MAP
SCALE IN MILES
P.I. 65+51.1
Range 65+51.1
P.I. 61+71.1
Range 61+71.1
P.I. 56+67.6
Range 56+67.6
Range 53+50
P.I. 50+08.4
Range 45+50
Range 42+00
P.I. 44+08.9
P.I. 33+00.8
P.I. 39+33.1
Control Line
Range 30+00
P.I. 28+24.8
Range 26+00
Range 25+00
Range 24+00
Range 23+00
22+00
21+00
20+00
Range 18+00
Range 12+00
Range 8+00
Range 6+00
Range 4+00
Range 2+00
Range 0+50
Range -4+00
Range -8+00
P.I. 18+00
P.I. 16+37.4
P.I. 14+40.7
P.I. 13+63.0
P.I. 12+00
P.I. 9+67.8
P.I. 8+00
0+00
Point "B"
Point "A"
LEGEND
Waters Edge
New Wood Jetty
New Steel Jacks
New Concrete-Filled Mat
N
PLAN
1000 200 400 600
SCALE IN FEET
IOWA RIVER
WAPELLO, IOWA
MONITORING PLAN

PLATE 7



IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 0+50 AND 2+00
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 2+00
SECTION NO. 0+50
℄ Jacks - Good Condition - Oct 1980
W.S. 8 Jun 78
℄ Jacks - Poor Condition - Oct 1980
W.S. 7 Jun 78
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 8

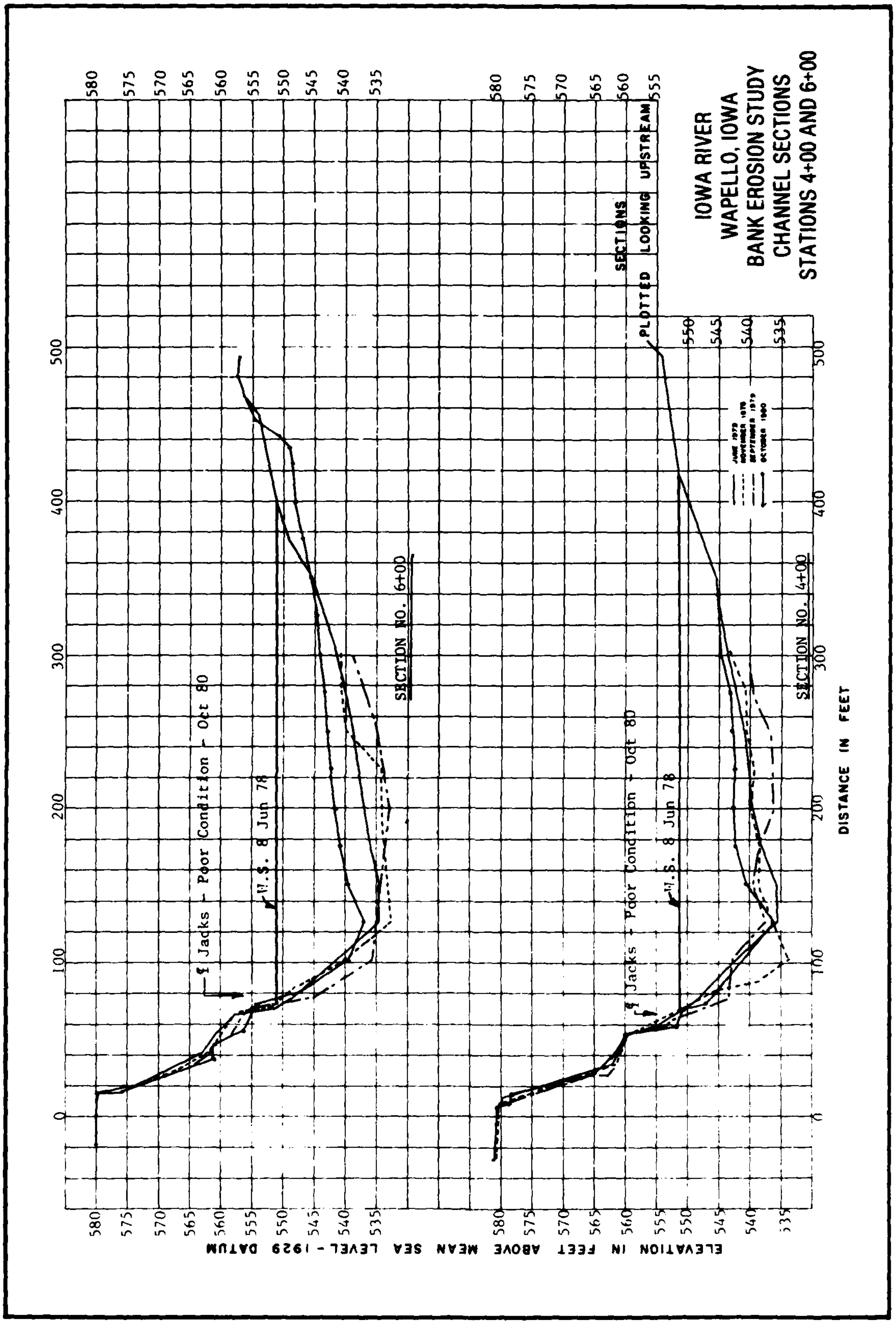

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 4+00 AND 6+00
PLOTTED LOOKING UPSTREAM
SECTIONS
SECTION NO. 6+00
SECTION NO. 4+00
℄ Jacks - Poor Condition - Oct 80
W.S. 8 Jun 78
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM
0
100
200
300
400
500
580
575
570
565
560
555
550
545
540
535

PLATE 9

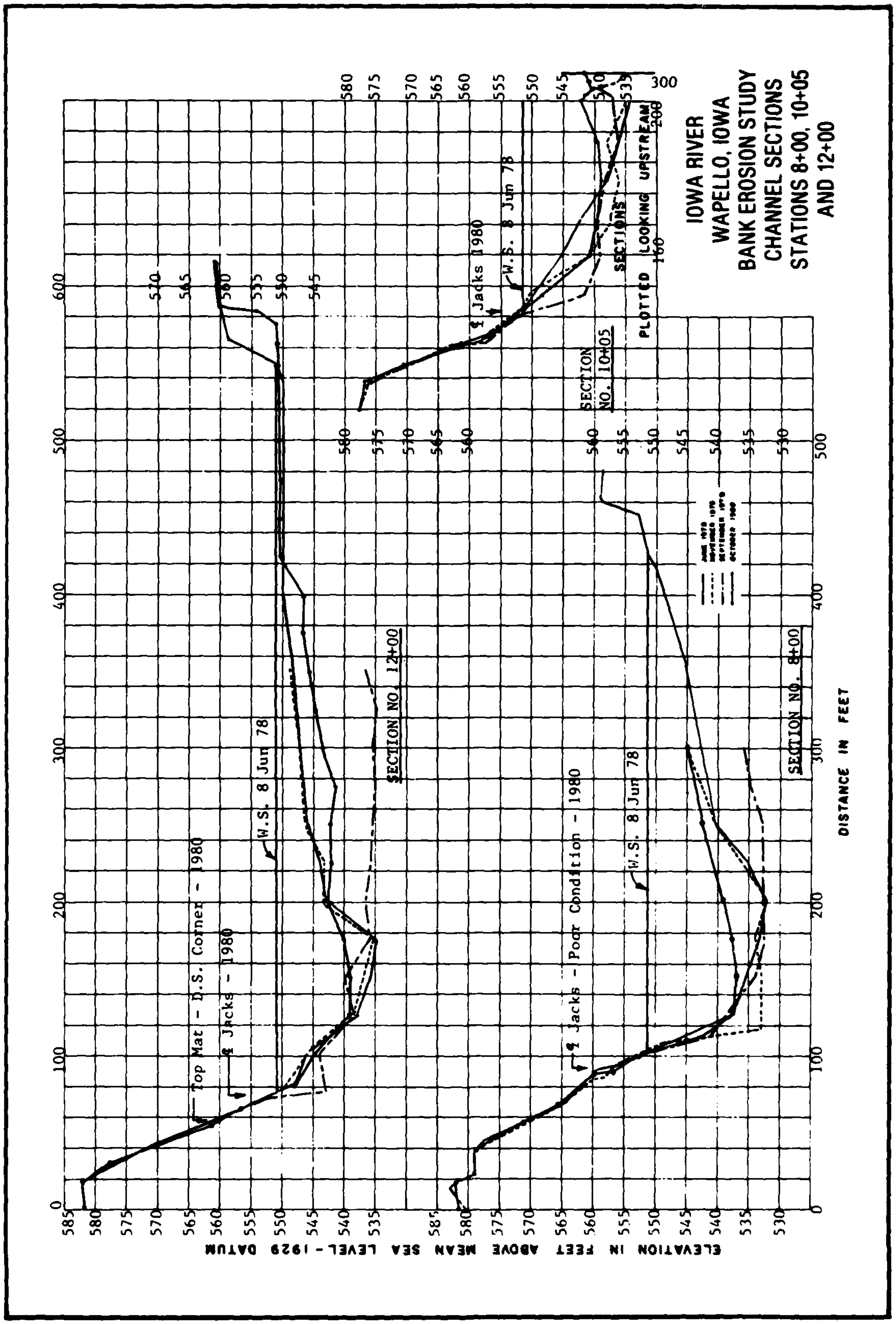

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 8+00, 10+05
AND 12+00
SECTION NO. 12+00
SECTION NO. 10+05
SECTION NO. 8+00
PLOTTED LOOKING UPSTREAM
SECTIONS
Top Mat - D.S. Corner - 1980
℄ Jacks - 1980
W.S. 8 Jun 78
℄ Jacks 1980
℄ Jacks - Poor Condition - 1980
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 10

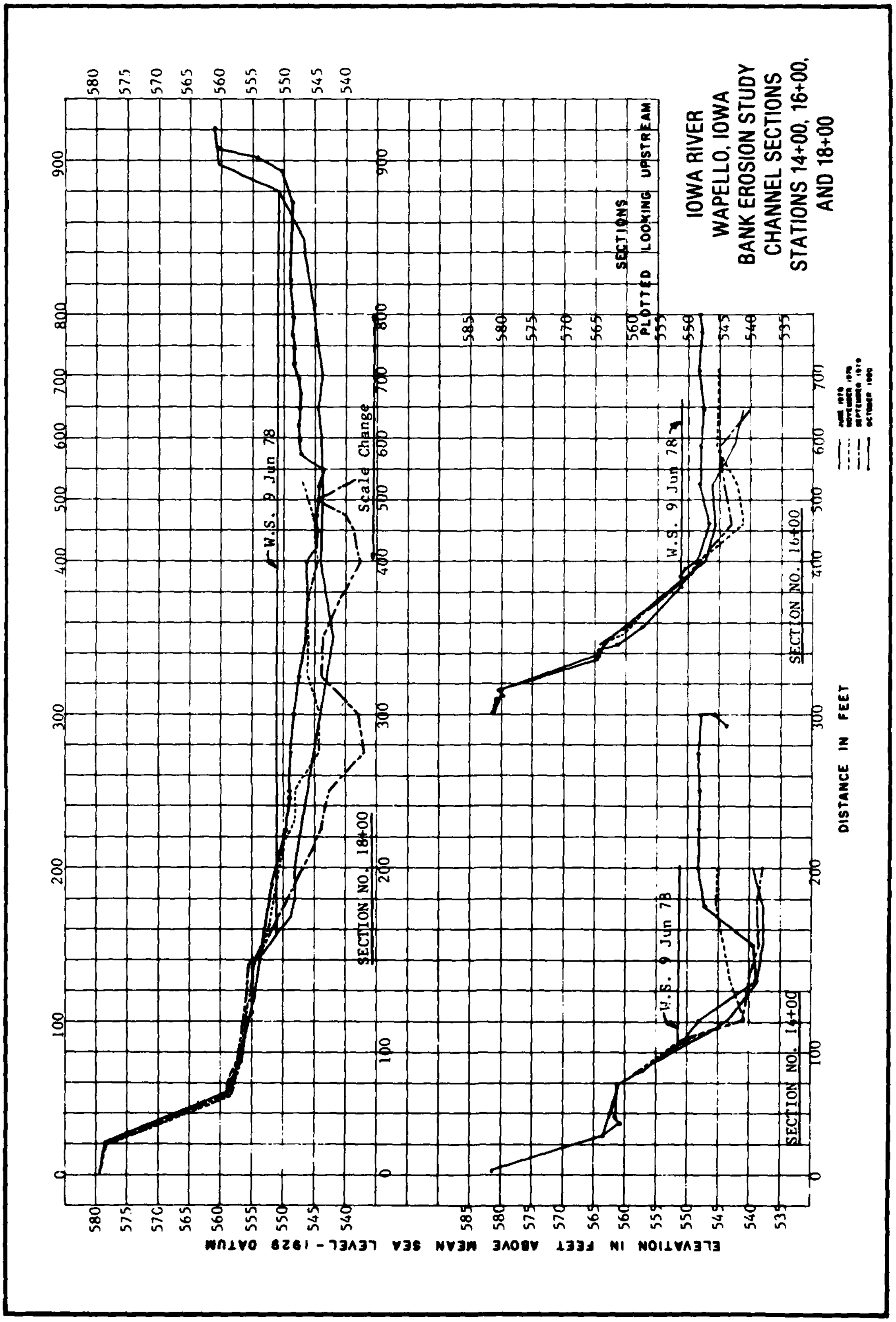

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 14+00, 16+00,
AND 18+00
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 14+00
SECTION NO. 16+00
SECTION NO. 18+00
W.S. 9 Jun 78
Scale Change
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL-1929 DATUM

PLATE 11

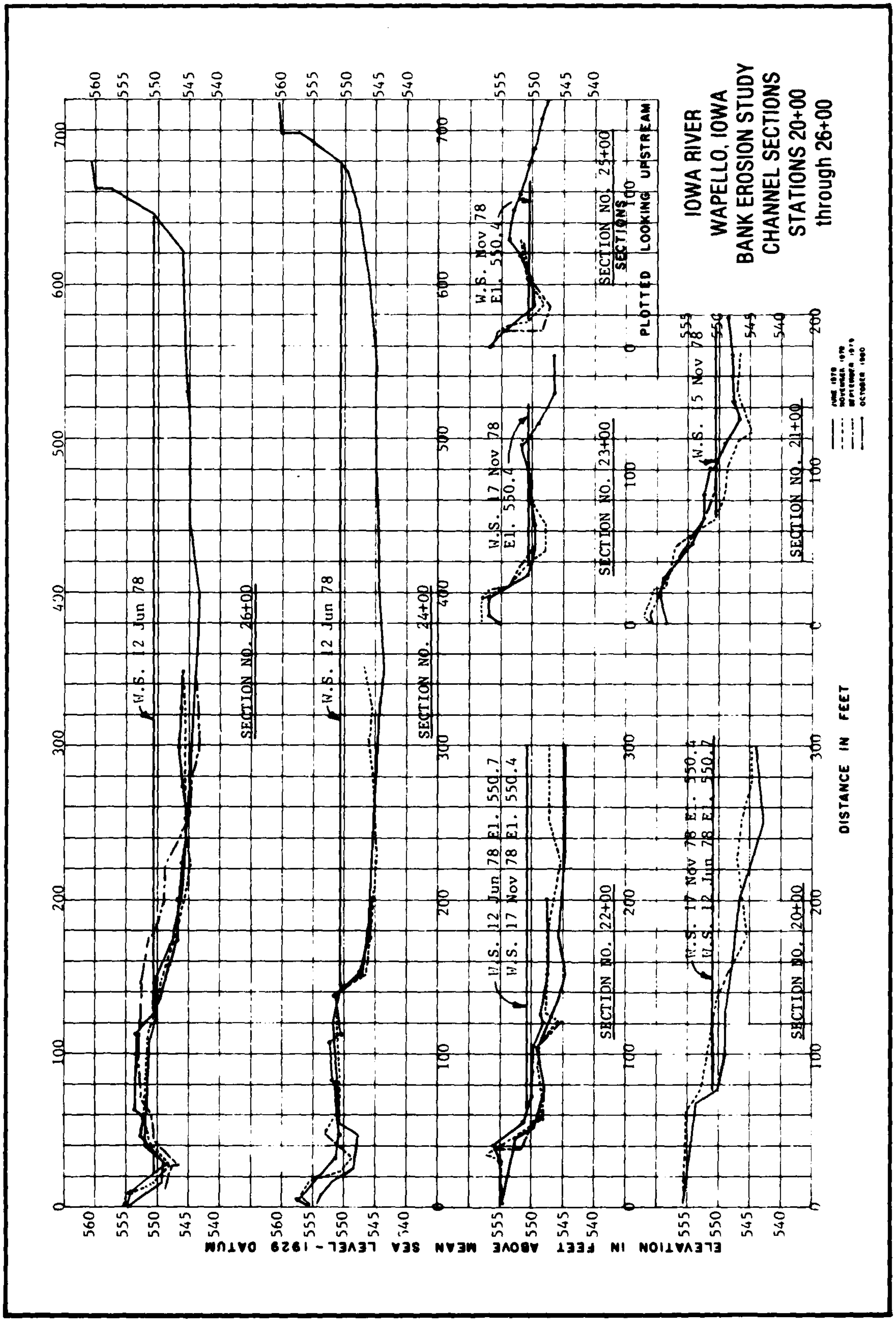
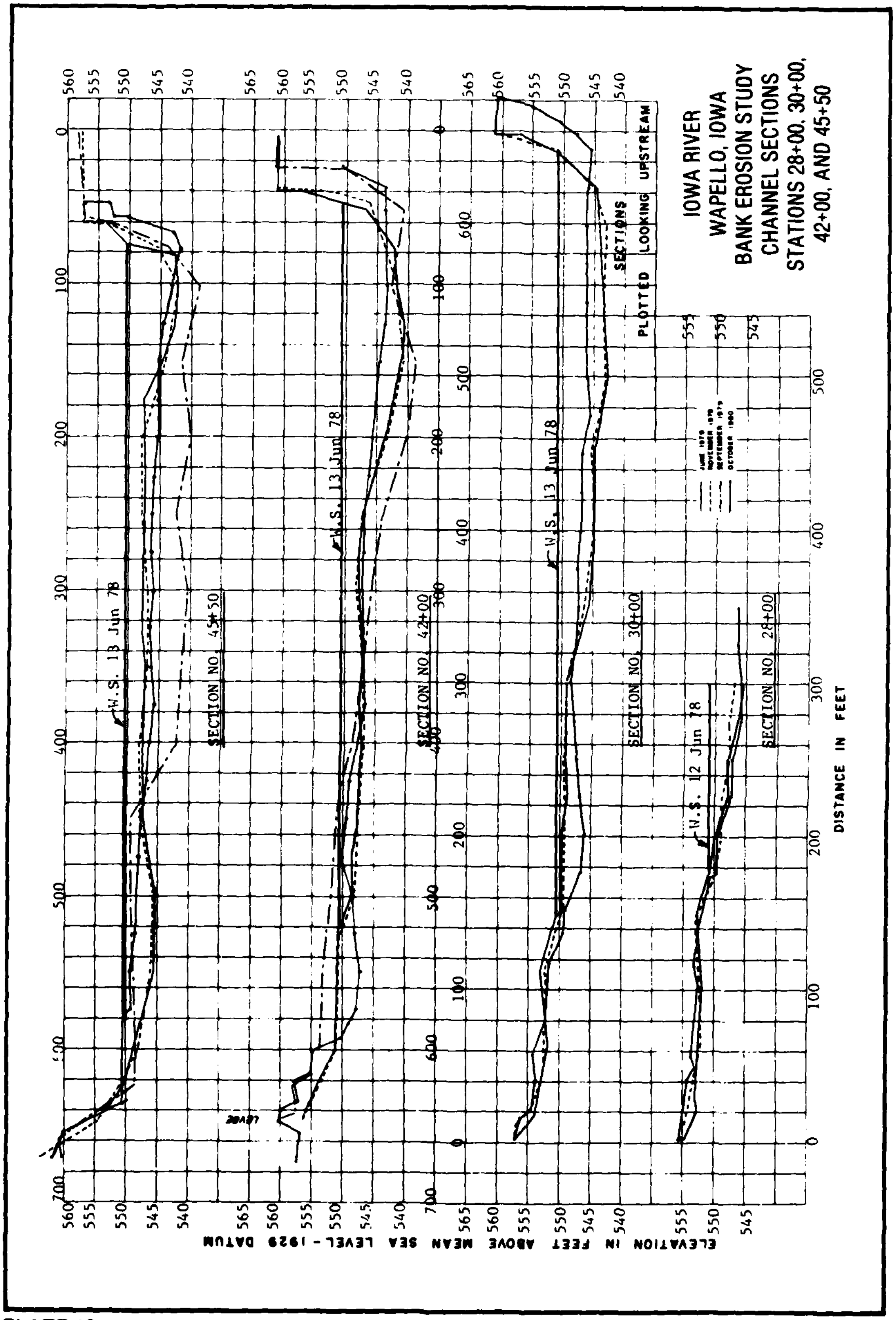

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 20+00
through 26+00
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 26+00
W.S. 12 Jun 78
SECTION NO. 24+00
W.S. 12 Jun 78
SECTION NO. 22+00
W.S. 12 Jun 78 El. 550.7
W.S. 17 Nov 78 El. 550.4
SECTION NO. 23+00
W.S. 17 Nov 78
El. 550.4
SECTION NO. 25+00
W.S. Nov 78
El. 550.4
SECTION NO. 20+00
W.S. 17 Nov 78 El. 550.4
W.S. 12 Jun 78 El. 550.7
SECTION NO. 21+00
W.S. 15 Nov 78
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM
DISTANCE IN FEET

PLATE 12



IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 28+00, 30+00, 42+00, AND 45+50
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 45+50
SECTION NO. 42+00
SECTION NO. 30+00
SECTION NO. 28+00
W.S. 13 Jun 78
W.S. 12 Jun 78
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 13

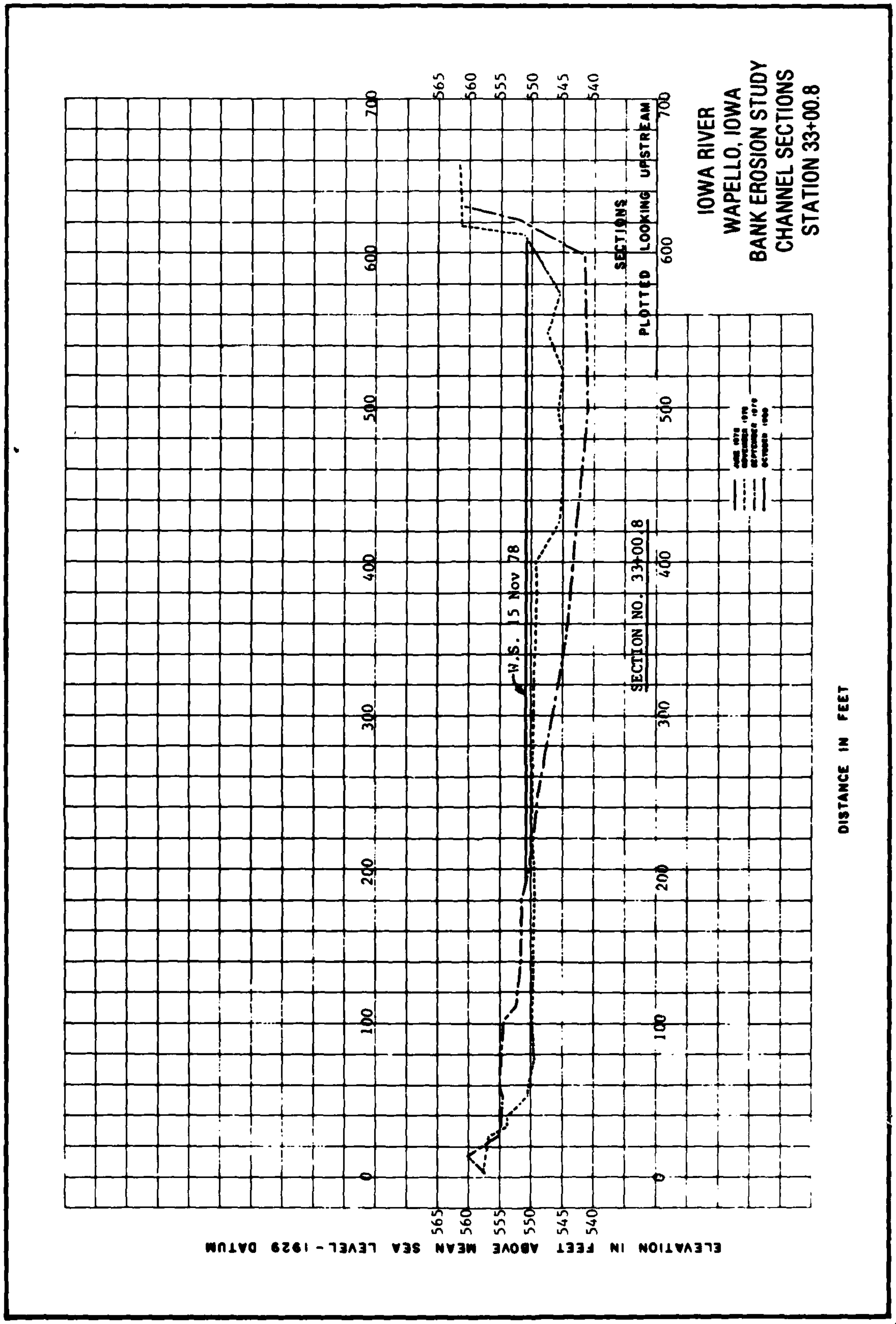

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATION 33+00.8
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 33+00.8
W.S. 15 Nov 78
0
100
200
300
400
500
600
700
565
560
555
550
545
540
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 14

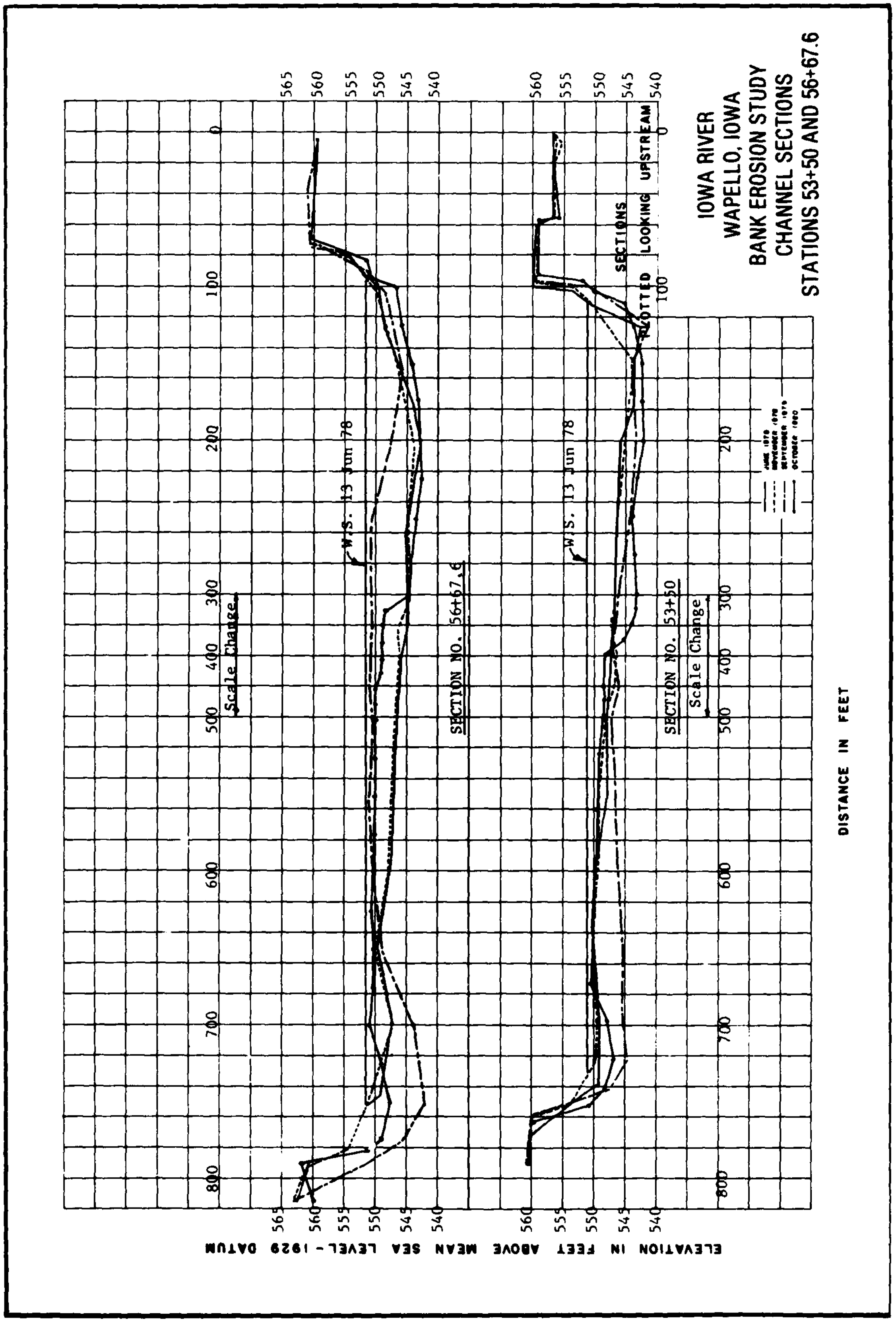
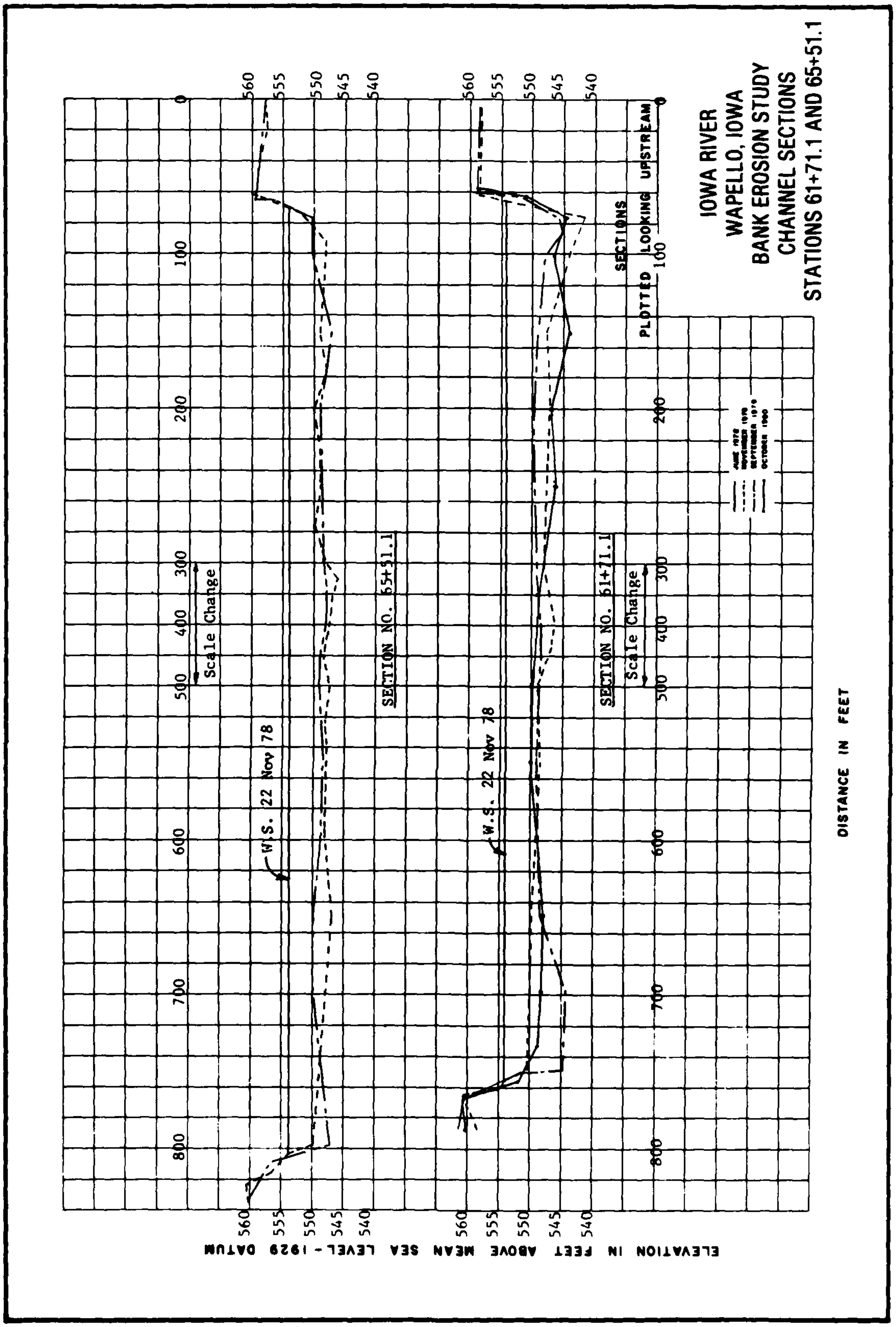

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 53+50 AND 56+67.6
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 56+67.6
SECTION NO. 53+50
W.S. 13 Jun 78
W.S. 13 Jun 78
Scale Change
Scale Change
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM
0
100
200
300
400
500
600
700
800
565
560
555
550
545
540

PLATE 15



IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 61+71.1 AND 65+51.1
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 65+51.1
SECTION NO. 61+71.1
W.S. 22 Nov 78
W.S. 22 Nov 78
Scale Change
Scale Change
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 16

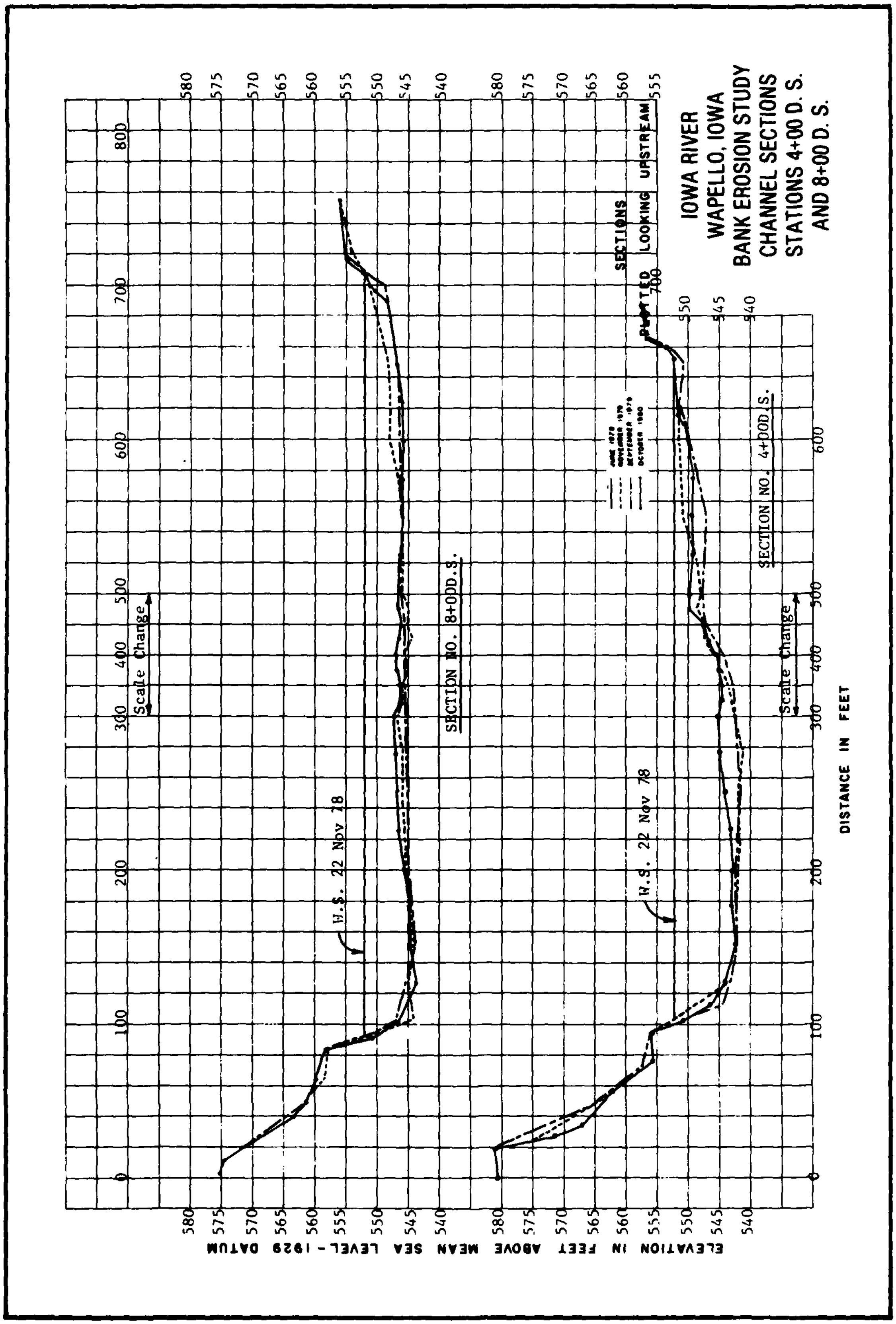
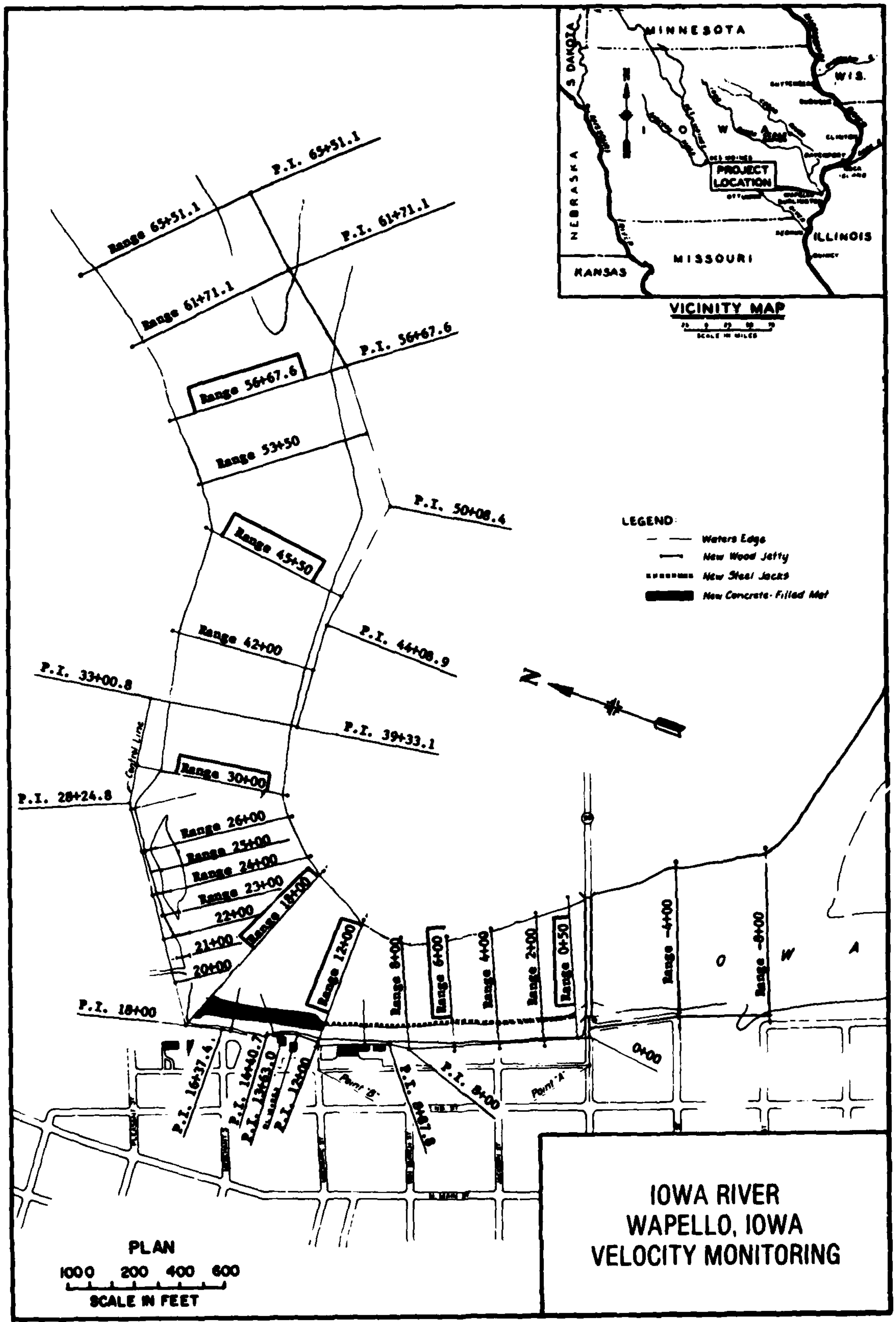

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
STATIONS 4+00 D. S.
AND 8+00 D. S.
SECTIONS PLOTTED LOOKING UPSTREAM
SECTION NO. 8+00D.S.
SECTION NO. 4+00D.S.
W.S. 22 Nov 78
Scale Change
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 17



VICINITY MAP
MINNESOTA
S DAKOTA
WIS.
NEBRASKA
IOWA
PROJECT LOCATION
ILLINOIS
MISSOURI
KANSAS
LEGEND
Waters Edge
New Wood Jetty
New Steel Jacks
New Concrete-Filled Mat
N
P.I. 65+51.1
Range 65+51.1
P.I. 61+71.1
Range 61+71.1
P.I. 56+67.6
Range 56+67.6
Range 53+50
P.I. 50+08.4
Range 45+50
Range 42+00
P.I. 44+08.9
P.I. 33+00.8
P.I. 39+33.1
Range 30+00
P.I. 28+24.8
Range 26+00
Range 25+00
Range 24+00
Range 23+00
22+00
21+00
20+00
Range 18+00
Range 12+00
Range 8+00
Range 6+00
Range 4+00
Range 2+00
Range 0+50
Range -4+00
Range -8+00
P.I. 18+00
P.I. 16+37.4
P.I. 14+40.7
P.I. 13+63.0
P.I. 12+00
P.I. 8+00
0+00
Point 'A'
Point 'B'
IOWA
PLAN
1000 200 400 600
SCALE IN FEET
IOWA RIVER
WAPELLO, IOWA
VELOCITY MONITORING

PLATE 18

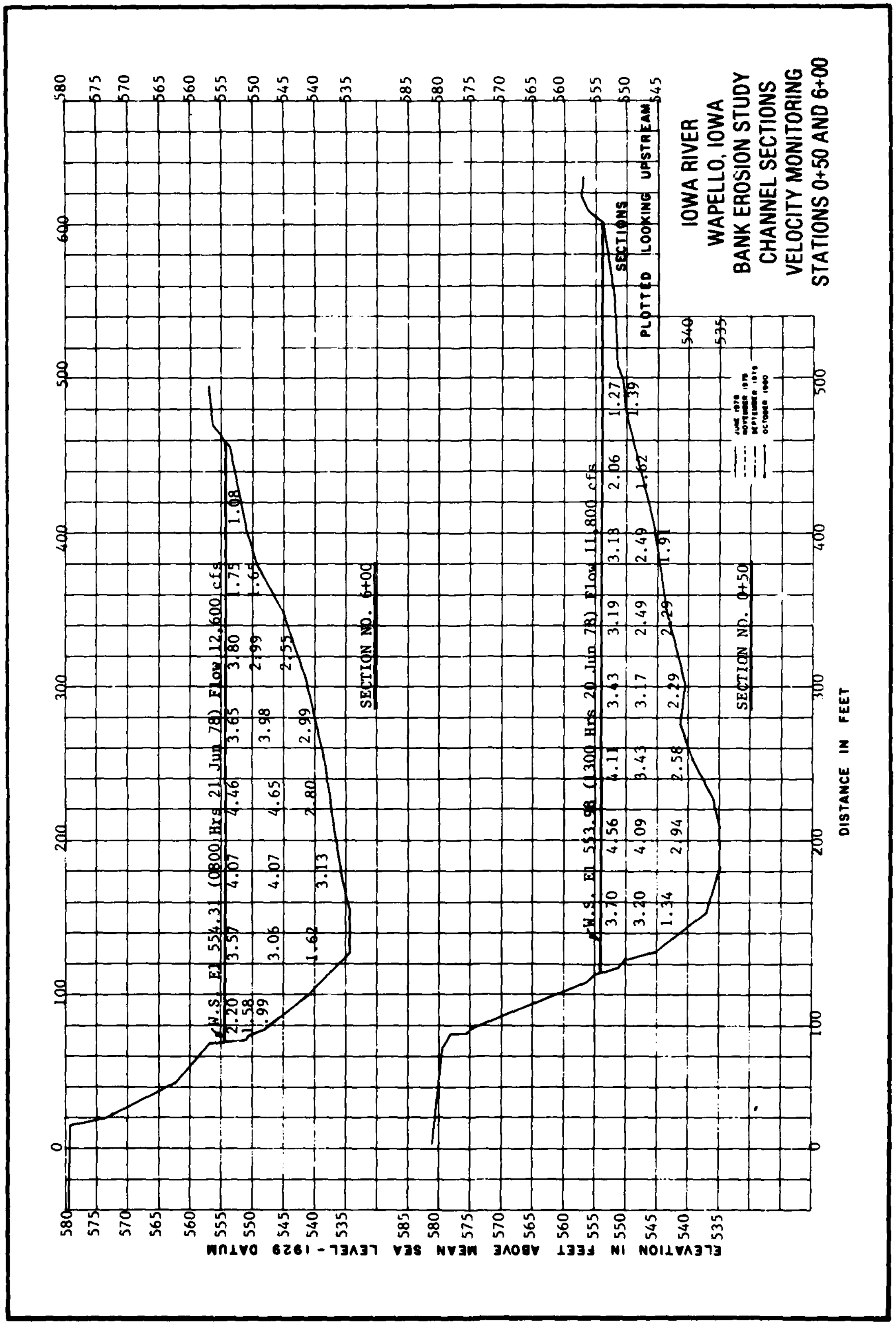

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
VELOCITY MONITORING
STATIONS 0+50 AND 6+00
PLOTTED SECTIONS LOOKING UPSTREAM
W.S. El 554.3 (0800 Hrs 21 Jun 78) Flow 12,600 cfs
SECTION NO. 6+00
W.S. El 553.98 (1300 Hrs 20 Jun 78) Flow 11,800 cfs
SECTION NO. 0+50
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 19

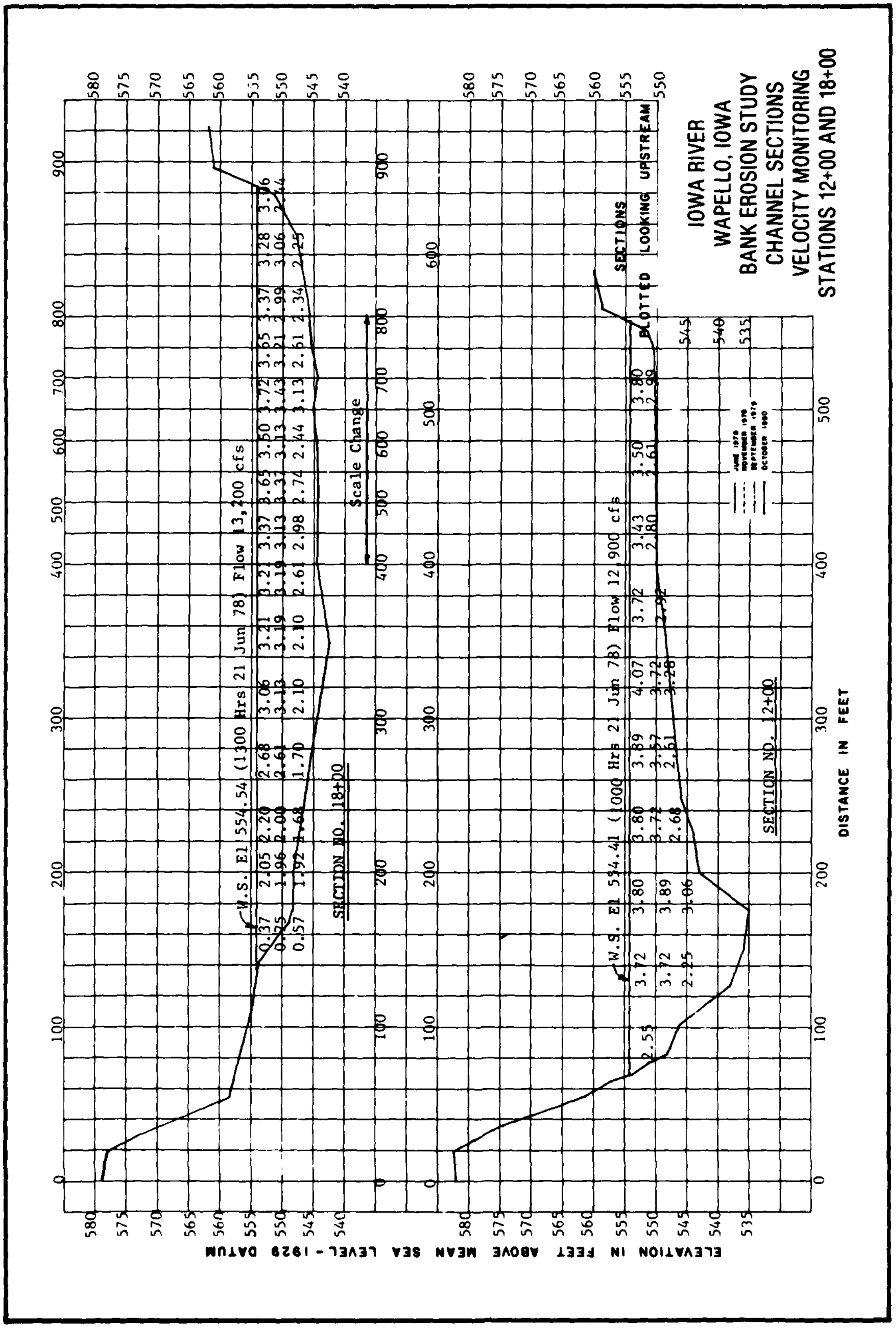

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
VELOCITY MONITORING
STATIONS 12+00 AND 18+00
SECTIONS PLOTTED LOOKING UPSTREAM
W.S. El 554.54 (1300 Hrs 21 Jun 78) Flow 13,200 cfs
SECTION NO. 18+00
Scale Change
W.S. El 554.41 (1000 Hrs 21 Jun 78) Flow 12,900 cfs
SECTION NO. 12+00
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM
JUNE 1978
NOVEMBER 1979
SEPTEMBER 1979
OCTOBER 1980

PLATE 20

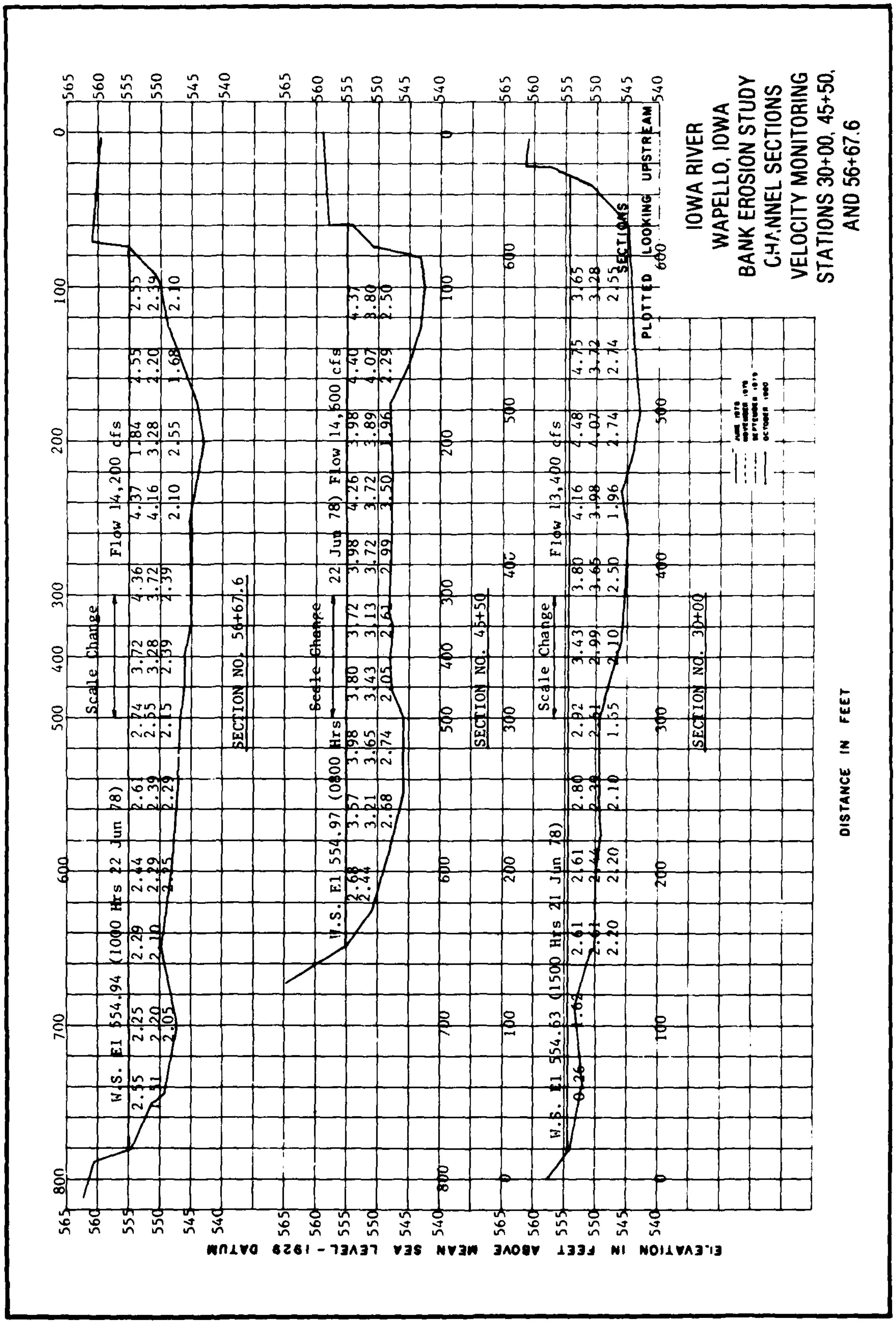
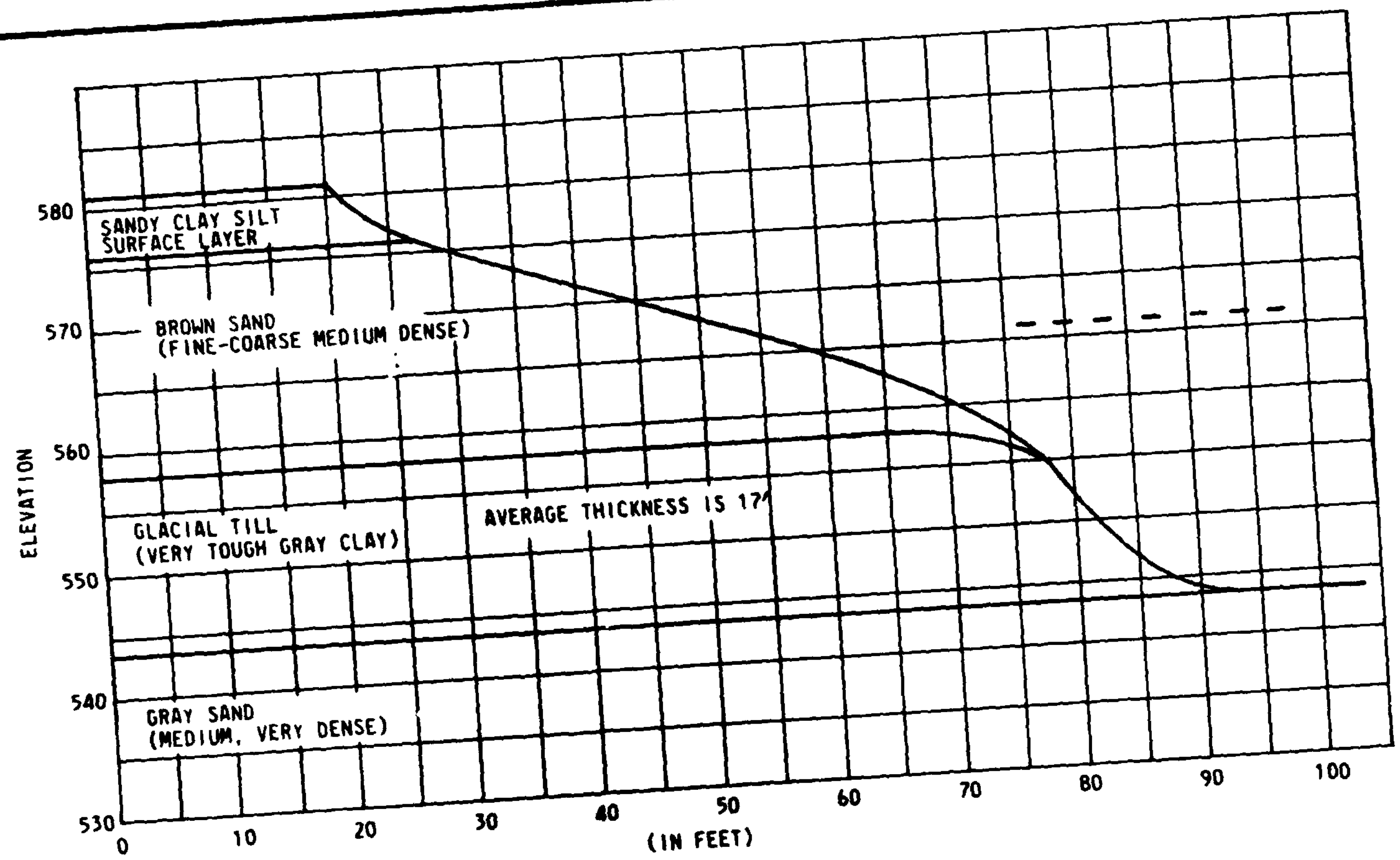
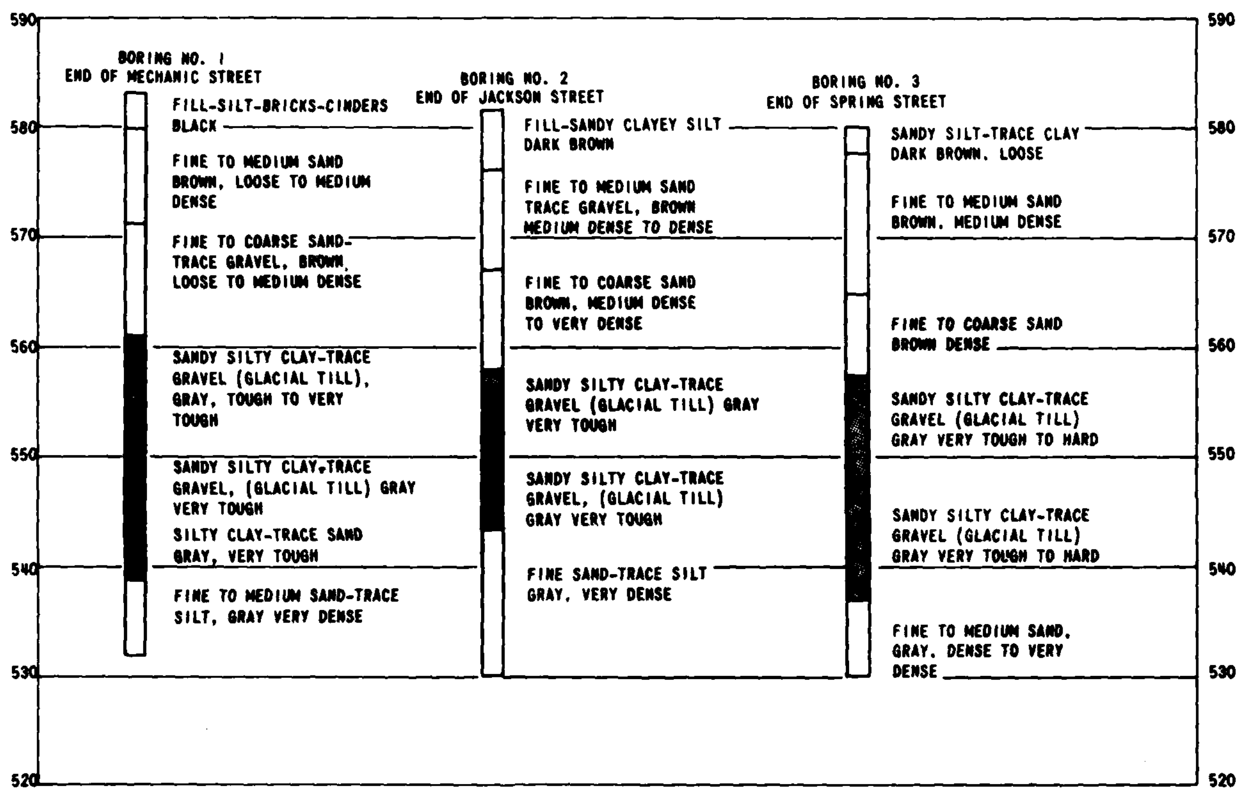

IOWA RIVER
WAPELLO, IOWA
BANK EROSION STUDY
CHANNEL SECTIONS
VELOCITY MONITORING
STATIONS 30+00, 45+50,
AND 56+67.6
PLOTTED LOOKING UPSTREAM
SECTIONS
SECTION NO. 56+67.6
W.S. El 554.94 (1000 Hrs 22 Jun 78)
Flow 14,200 cfs
SECTION NO. 45+50
W.S. El 554.97 (0800 Hrs 22 Jun 78) Flow 14,600 cfs
SECTION NO. 30+00
W.S. El 554.63 (1500 Hrs 21 Jun 78)
Flow 13,400 cfs
Scale Change
DISTANCE IN FEET
ELEVATION IN FEET ABOVE MEAN SEA LEVEL - 1929 DATUM

PLATE 21



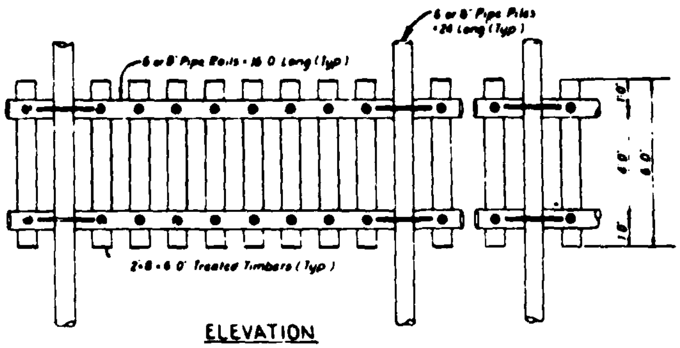
IOWA RIVER
WAPELLO, IOWA
TYPICAL SECTION OF RIVER BANK

PLATE 22



IOWA RIVER
WAPELLO, IOWA
LOG OF SOIL BORINGS

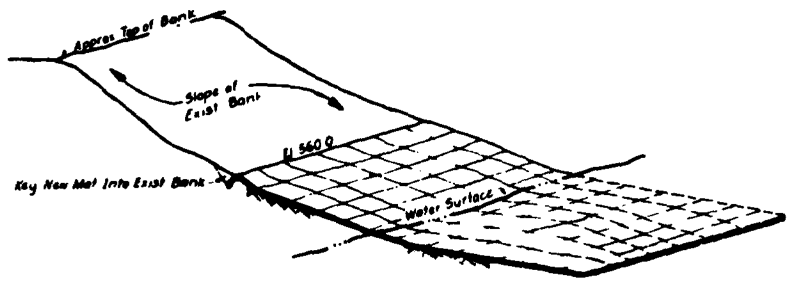
PLATE 23



ELEVATION

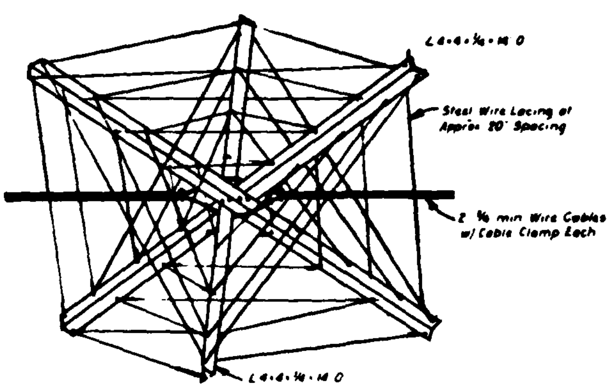
PERMEABLE TIMBER JETTY DETAILS

(NO SCALE)



CONCRETE - FILLED MAT DETAILS

(NO SCALE)



STEEL JACK UNIT

(NO SCALE)

IOWA RIVER
WAPELLO, IOWA
PROTECTION PLAN DETAILS

PLATE 24

The following table gives the estimated velocity of flood discharges taken at the Wapello stream gaging station. The gage is located at the Highway 99 bridge on the downstream end of the project. The velocity measurements were taken approximately 40 feet from the right edge of water (REW). The period of record is from water year 1916 to water year 1979.

Water Year	Date	Peak Stage (ft.)	Peak Discharge (c.f.s.)	Est Vel Approx. 40' from REW (ft./sec.)
1916	28 Mar	22.7	48,900	7.20
1917	29 Mar	23.2	52,000	7.35
1918	8 Jun	25.00	77,000	7.70
1919	23 Mar	20.83	38,100	6.70
1920	29 Mar	19.7	32,200	6.25
1921	24 Sep	19.6	32,400	6.30
1922	2 Mar	18.6	26,500	5.65
1923	7 Apr	19.6	31,700	6.20
1924	28 Jun	21.30	40,700	6.85
1925	21 Jun	15.20	12,900	3.25
1926	25 Sep	21.8	43,500	7.00
1927	26 May	20.1	34,200	6.40
1928	9 Oct	19.2	28,800	5.95
1929	21 Mar	24.6	72,200	7.70
1930	17 Jun	23.5	52,200	6.35
1931	27 Sep	13.1	6,740	1.63
1932	2 Dec	18.8	27,800	5.80
1933	7 Apr	25.38	62,000	7.60
1934	11 Apr	13.27	7,230	1.75
1935	12 Mar	21.04	35,500	6.55
1936	18 Mar	21.59	36,500	6.60
1937	7 Mar	24.64	53,800	7.40
1938	15 Jun	18.15	21,200	4.85
1939	14 Mar	21.31	37,000	6.60
1940	21 Mar	13.70	7,780	1.90
1941	25 Mar	17.10	16,600	4.00
1942	7 Aug	21.06	36,300	6.60
1943	4 Aug	18.68	24,400	5.40
1944	25 May	24.72	54,100	7.40
1945	22 Mar	24.82	56,400	7.50
1946	8 Jan	23.98	51,400	7.30
1947	18 Jun before levee failure	26.85	94,000	7.80
1948	21 Mar	24.68	60,000	7.50
1949	11 Mar	22.71	44,300	7.05
1950	14 Mar	22.79	44,800	7.10
1951	14 Apr	26.14	67,000	7.60
1952	14 Mar	22.10	41,300	6.90
1653	23 Feb	20.42	32,800	6.10
1954	29 Jun	21.98	40,800	6.90
1955	25 Apr	18.26	23,000	5.20
1956	1 Sep	14.18	9,340	2.32
1957	22 Jun	15.89	14,200	3.55
1958	27 Feb	16.48	13,000	3.25
1959	22 Mar	21.52	37,200	6.70
1960	5 Apr	27.02	69,000	7.65
1961	3 Apr	26.85	68,000	7.60
1962	6 Apr	25.38	53,700	7.40
1963	21 Mar	19.16	25,100	5.50
1964	25 Jun	15.25	11,800	2.95
1965	13 Apr	27.25	70,800	7.70
1966	25 May	20.36	30,300	6.10
1967	9 Jun	19.43	26,700	5.50
1968	10 Aug	18.70	21,500	4.90
1969	15 Jul	27.40	69,200	7.65
1970	6 Mar	21.81	34,600	6.50
1971	28 Feb	23.23	38,000	6.70
1972	8 Aug	19.37	24,500	5.40
1973	22 Apr	28.63	92,000	7.80
1974	19 May	28.12	82,200	7.75
1975	25 Mar	21.78	38,700	6.75
1976	27 Apr	21.41	36,600	6.60
1977	20 Sep	18.23	21,100	4.80
1978	22 Mar	20.14	30,300	6.10
1979	22 Mar	25.3	63,700	7.60

ESTIMATED FLOOD VELOCITIES

PLATE 25

LITTLE MIAMI RIVER AT
MILFORD, OHIO

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

LITTLE MIAMI RIVER AT MILFORD, OHIO
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Milford, Ohio Bank Protection, Little Miami River, Mile 13, Milford, Ohio. Plates 1 and 2 show a General Map and Location Plan for the site.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem, the types of bank protection used, and a performance evaluation of a demonstration project on the Little Miami River, Ohio, constructed and monitored by the Louisville District.

4. Problem Resume. Milford lies along both banks of the Little Miami River about 12 miles upstream from its mouth on the Ohio and about 1 mile upstream from its confluence with the East Fork of the Little Miami River as shown on the General Map, Plate 1.

The study area consists of a precipice about 75 feet high which has been cut by the river through unconsolidated glacial material. This material consists of sand and gravel deposits which are overlain by silt and clay. Erosion of this bank has progressed at the rate of about 5 feet a year since 1973; however, in recent years up to 10 feet has been lost per year.

The critical caving bank area, Plate 2, on the left (east) bank is now about 800 feet long and 75 feet high. Erosion has taken an alley, a garage, and sanitary sewerline. The sanitary sewerline has since been relocated about 50 feet away from the bank. There is evidence of ground loss prior to 1958.

Erosion progressed to the Mound Street Alley area during 1974. Most erosion has occurred since then. The purpose of the project was to stop erosion of the lower bank and to lessen and eventually stop erosion of the high bank and thereby reduce loss of private property and public utilities.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The topographic features along the left bank of Milford consist of a low flood plain and multilevel plateau areas. The topographic relief along the left overbank ranges from approximately 500 feet mean sea level along the left overbank, while less than 1/2-mile landward, an elevation of 690 feet mean sea level exists. The topographic information listed above was taken from 1970 photo revised U.S.G.S. quadrangle maps with 10 foot contour intervals. Most of the recent land development at Milford has been the upper plateaus away from the flood plain. The City of Milford has zoning ordinances which include the designation of flood plain lands.

The Little Miami River begins in Clark County, Ohio, not far from South Charleston, and flows south to the Ohio River. The main branch passes through Clark, Greene, Warren, and Hamilton Counties. It drops from an elevation of 1,137 feet mean sea level at its source to 448 feet mean sea level at its mouth, an average slope of 6.5 feet per mile. The Little Miami River through the Milford reach was designated a Scenic River by the state in 1969. The Little Miami River has three principal tributaries: Caesar Creek, Todd Fork and East Fork.

East Fork is a major tributary of the Little Miami River which has its confluence 1.5 miles downstream of Milford. East Fork has a basin area of 500.7 square miles in its 81.7 miles of length. The Corps of Engineers has recently completed East Fork Lake. The lake is a multipurpose project with water supply, water quality, recreation, and controls a 342 square mile drainage area for flood control purposes. The damsite is located about 21 miles above the confluence.

The drainage area of Little Miami River at Milford is 1,203 square miles. Recently, the Corps of Engineers has completed Caesar Creek Lake, a multipurpose project on Caesar Creek, a major tributary of the Little Miami River. This lake controls a drainage area of 237 square miles and is designed to provide for storage of water for water supply and water quality control, recreation, and flood control. The damsite is three miles above the mouth of Caesar Creek, a tributary of the Little Miami River, about 30 miles north of Milford.

b. Geology. The Village of Milford, Ohio, is approximately 10 miles east of Cincinnati. Preliminary study indicates that much of Milford is situated on unconsolidated, largely pervious, glacial outwash that is generally covered by a few feet of impervious till. The critical caving bank area is composed of thick deposits of permeable sand and gravel underlying relatively thin layers of fine sand and clay. Bedrock throughout most of the basin is overlain by unconsolidated clay, silt, sand, and gravel deposits of glacial origin. Two major types of deposits left by glaciers are till, composed of clay with sand and boulders, and outwash composed chiefly of sand and gravel. As a result of several periodic advances of glaciers, both till and outwash were deposited in the basin. Plate 8 is a natural section at the project site showing soil composition.

The soil deposits exposed in the bank slope consist of granular materials ranging from fine sands to coarse gravel. The deposits vary in gradation and are encountered at interbedded conditions with some zones exhibiting cemented formations. The stable bank slopes to either side of the problem area are covered with overburden and vegetation including trees. These slopes are standing at apparently stable condition at a slope of 1 horizontal to 1 vertical.

The slope conditions in the most severely affected problem areas reflect a vertical drop of 30 feet from the alley grade of elevation 570 down to elevation 540. From this level down, the materials which have accumulated from the gravitational "drop" from above have come to rest at an approximate slope of 1-1/2 horizontal to 1 vertical. Such is considered to be the angle of repose for these materials and therefore constitute a stable condition providing the toe of slope (at water line) is not undermined.

Buried valley aquifers have been identified in the Little Miami flood plain with yields estimates at 3 to 4 million gallons per day per valley mile. This water supply is rated as available but some exploratory boring would be necessary.

c. Locality, Development and Occupation. Milford, Ohio, is located on both banks of the Little Miami River, approximately 13 miles upstream of its confluence with the Ohio River (see Plate 1). It is in Miami Township, Clermont County, just west of the Clermont-Hamilton County line. Milford is within the Cincinnati metropolitan area, 15 miles east of the city's central business district. Terrace Park and Indian Hill Village are adjacent communities to the south and west of Milford, respectively.

Population in 1960 was 4,131; in 1975 was 6,000 and is expected to be about 10,000 by the year 2000. Because of the expansion of the metropolitan Cincinnati area, development has occurred on both banks of the Little Miami River in the Milford area. Milford is located on the left bank of the Little Miami River, while Terrace Park is situated downstream of Milford on the right bank. Indian Hills is located northwest of Milford.

Most of the Clermont County residents in the labor market are employed in Hamilton County. The employment structure of Clermont County in 1970 consisted of a high percent of manufacturing (42 percent), wholesale and retail (17 percent), and 11 percent in services. The remaining market consists of a relatively equal distribution of agriculture, construction, public services, government, and education.

Transportation facilities have recently been enhanced at Milford with the local completion of I-275 beltway located two miles east of the village. This beltway circumscribes the Cincinnati metropolitan area linking Interstate Highways I-71, I-74, and I-75. Another major transportation route serving the Milford area is U.S. 50, which links Milford to Mariemont and Cincinnati to the west, and Perintown to the east.

d. Hydrologic Characteristics. The drainage area of the Little Miami River at Milford is 1,203 square miles. The average discharge is 1,189 c.f.s. A record discharge of 84,100 c.f.s. occurred on 22 January 1959. The discharge of the historic high water of March 1913 exceeded this. Elevation at the project site was 519.0 for this flood. An elevation-frequency curve is Plate 16. Velocities at the site range from 2 to 5 feet per second for normal flows to around 10 feet per second for floods. The average slope of the stream is 6.5 feet per mile. Plate 9 is a profile of the stream showing previous flood levels, thalweg, and Ordinary High Water. Plate 7 is a full natural cross section at the project site. The main stem of the Little Miami River from Loveland to its headwaters in Clark County has been designated a scenic river by Ohio Department of Natural Resources. Elevation hydrographs at the site are shown on Plates 10 through 15. Air temperatures at the site are generally moderate, seldom above 100°F and only occasionally below 0°F.

The U.S. Geological Survey has a water-stage recorder gage located 500 feet downstream from U.S. Highway 50 at Milford. Gage zero is 499.20 feet mean sea level, adjustment of 1929. Gage data are available from 1915 to present with the exceptions of two periods: 1918-1925 and 1937-1938. The maximum flood of record is the January 1959 flood when a gage height of 22.3 feet was recorded. Current information indicates that the flood in March 1913 reached a stage of 25.5 feet, present datum.

The average annual precipitation at Milford is approximately 41 inches. The average runoff rate has been determined to be about 35 percent. Runoff rates are relatively low and vary considerably due to the pervious soil in the river basin. Antecedent rainfall in the basin is reflected in the ground water table which, in turn, effects the degree of runoff.

e. Channel Characteristics. The elevation-frequency curve, Plate 16, shows that flows which meet or exceed top of bank (near Milford) run more than once per year probably almost 3 to 5 times per year. During average years the stream always flows--never drying up. The channel is composed of random deeper pools, bars, and riffle areas. During high flow considerable movement of sand and gravel takes place--rearranging to some degree the location and/or size of these pools, bars and riffles. High water also results in erosion of banks and loss of some trees.

At the project site, and along much of the stream, there is a very high steep bank on one side of the stream (see Plan - Plate 2). This bank is susceptible to sloughing even though channel flows are always far from its top. Erosion of the lower bank due to rearrangement of channel features, as discussed above, and flow currents can cause sloughing to elevations 50 to 75 higher than the stream bottom.

f. Environment Considerations. The study area for this investigation lies entirely within the corporate limits of the Village of Milford, Ohio. As a result, the natural environment has been highly stressed by human activity. Residential and commercial properties extend to the river and the remaining natural vegetation consists primarily of a narrow bank of scattered mature trees along the riverbank. The ground cover is generally species of domesticated grasses. The predominant riparian tree species include sycamore, cottonwood, elm and hackberry. Boxelder and white mulberry are prevalent where an understory has been permitted to develop, and on the lower bank slopes and sand bars in the river, channel black willow and sand bar willow are prevalent. The highly stressed condition of the site would indicate potential for the occurrence of any threatened or endangered plant species as being slight.

The proposed project will exert short-term adverse impacts on water quality during project construction as a result of increased turbidity. The long-term impact on water quality should be positive as a result of decreased susceptibility of the bank to massive failure and erosion which will substantially lessen localized river turbidity.

There will also be a temporary increase in noise, erosion, and a decrease in air quality as a result of activities associated with construction.

There will be no long-term adverse impacts from these activities. Because of existing conditions on the site, project impact on natural vegetation and wildlife will be minimal. As the filled areas behind the dike will be stabilized by appropriate plantings, the overall effect of the project should be to enhance the value of the area for wildlife.

No structures of historical significance will be affected by the project. Although a potential archaeological site may exist within the project limits, it was not affected by construction activities as there was no excavation of the bank.

6. Demonstration Site--Test Reach.

a. Hydrologic Characteristics. The hydrologic characteristics are as previously stated in paragraph 5-d.

b. Hydraulic Characteristics. Flow velocities at the site range from 2 to 5 feet per second for normal flows (around 1,200 cubic feet per second) to around 10 feet per second for flood flows. A maximum recorded discharge, 84,000 cfs, occurred on 22 January 1959. Plate 9 shows this and other flood profiles at the site. Velocity distribution within the channel cross section was not determined.

c. Riverbank Description. The materials composing the bank are shown on the bank section on Plate 8. This plate was compiled from an onsite investigation by a Geotechnical Engineer. Test borings are not available at this site. The bank materials are generally loose and easily become unstable on slopes steeper than 2 horizontal to 1 vertical.

Vegetation cover at the site consists of grasses, weeds and a few scattered small trees and shrubs. Because the upper bank is still sloughing--seeking its angle of repose--little or no vegetation can take hold there. See inclosed photos. Erosion of the bank is discussed in paragraph 4. Plate 2 shows the present line of erosion of the upper bank.

III. DESIGN AND CONSTRUCTION

7. General. The very high bank, greater than 70 feet, created a special design problem in that the cost to protect this entire height would be

prohibitive. A method had to be devised to stop erosion of the lower bank or toe and thereby control erosion of the upper bank. A comparison of the lower protected area to the entire bank height can be seen on the photos.

8. Basis for Design. The method used to stabilize the lower bank had to be one which would be able to withstand both flow conditions of the stream and the expected future sloughing of the bank above it. Three methods were devised to do this. These are shown on Plates 4 through 6. These schemes (riprap, gabions and cribwall) all involve basically the same theoretical method. A rather large riprap berm was constructed along the reach in order to control erosion at the toe and thereby stabilize the entire reach eventually. Then riprap, gabion and cribwall revetments were placed on the berm (which was slightly above low water) in order to protect the natural bank up to the 5 year flood level.

9. Construction Details. See Plate 2 for a plan of the project. Protective works consist of a riprap berm along the left bank which slightly encroaches the channel. The berm extends approximately 800 feet and provides a footing [elevation 500) and toe (elevation 505) protection for three different types of revetment: riprap, gabions, and a cribwall. The protective works extends from 5 feet below the bottom of the river to elevation 510. Backfill between elevations 510 and record high water elevation 518 was to be protected by selected vegetation. A low masonry dam built across the river near the center of the problem area has been washed away over the years, except for the center third which diverts flows against the left bank even under low flow conditions. The remaining portion of the dam has been demolished as part of the plan of improvement. Plates 4 through 6 are sections through each type of protection. More details of design are shown on these plates. Plate 7 is a full natural cross section at the site. The project was begun in 1978 and completed in 1979. Photos 1 and 2 show the project under construction. Photo 3 shows the project soon after completion.

10. Cost. Total cost of the project was about $553,000 including Engineering, Design, Supervision and Administration. Actual construction cost for

each scheme is shown in the table below. Total cost and including supervision, administration, engineering, design, and construction is also included.

Scheme	Construction Cost/Square Foot	Construction Cost/Linear Foot	Total Cost/Linear Foot
Gabions	$ 7.44	$141.	$245.
Cribwall	12.70	241.	345.
Riprap	3.00	36.	169.
Foundation & Backfill	—	249.	278.

No reconstruction was required.

IV. PERFORMANCE OF PROTECTION

11. Monitoring Program. The 3-year monitoring program consists of quarterly inspections with up to 24 color photos of which 10-15 are selected for use in the inspection report. Velocity is an important factor in erosion at this site. Therefore, velocity is measured at two locations along the 800-foot revetment using floats and stopwatch. A U.S.G.S. water stage recorder 0.5 mile upstream provides stage and flow information. Plates 10 through 15 are hydrographs for the site from the start of project (1976) to present. Plate 3 shows the parameters monitored and the frequency.

12. Evaluation of Protection Performance. During September 1979 a 10-year flood occurred at the site. Water reached elevation 515 or about 5 feet above the top of the protection which is at the 5-year flood level. No significant damage occurred from this major flood. Some erosion has been continuing to occur upstream and downstream of the protection where a more gradual transition to natural banks should have been made. See photos for better insight into these areas. Another large flood occurred during July 1980. This was about a 6-year flood and reached 1 foot above top of revetment. Again, no significant damage occurred. Monitoring inspections have shown a steady sloughing of the upper bank. The locals have hydro-seeded this upper

slope in hopes of reducing or stopping the sloughing. Plate 2 shows the approximate existing line of erosion and expected final line when bank has stabilized.

13. Rehabilitation. None has been required. The District may add some type of protection at the transition areas at the upstream and downstream project limits to curtail erosion occurring there--see photographs 5 and 6.

14. Summary of Findings.

a. More gradual transition to natural bank should have been made upstream and downstream of the project.

b. Sloughing of the upper slope should be slowed by continued growth of weeds and grasses. However, the slope will continue to flatten toward its natural angle of repose (1.5H to 1V). Condition of the upper slope is the responsibility of the locals and they are aware of that.

c. The revetment dike itself has performed well. It was designed to create a permanent toe or lower bank from which the upper bank or bluff would base itself and eventually stabilize. It appears that this will occur, thoug more property at the top of the upper bank will be lost during the process.

d. The berm design (see Plates 2 and 4 through 6) has functioned well. It has withstood the sliding of large amounts of material from the bluff and the flow caused by a 10-year flood. The use of gabions and cribwall saved valuable space which will eventually cause a savings in landscaped private property and a sanitary sewer line. Therefore, the use of gabions and cribwall in combination with the riprap dike was probably justified, though the cost exceeds that of plain riprap. However, of the two types of retaini structures, gabions appear more economical.

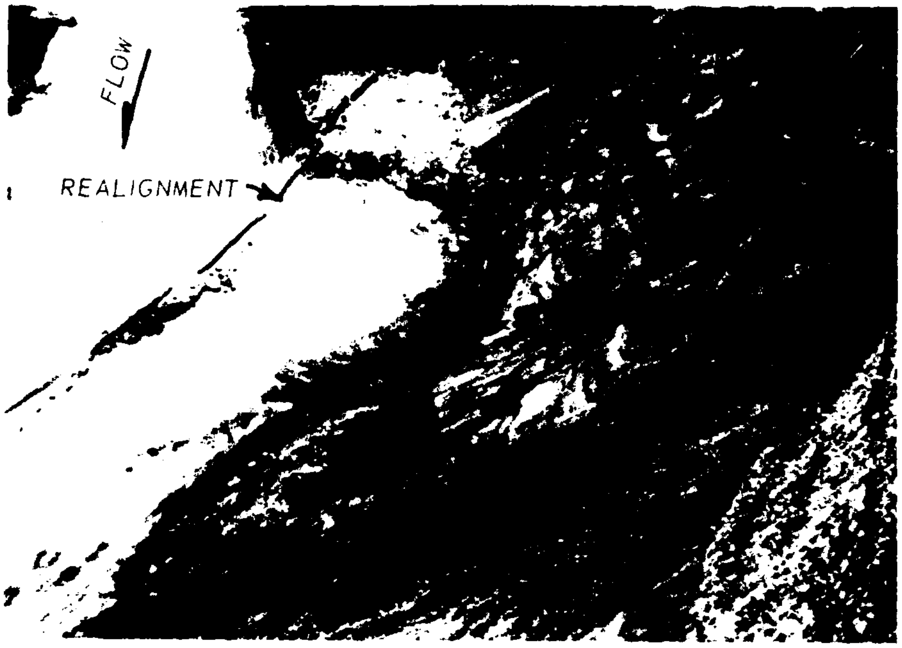


PHOTO NO. 1. 9 Nov. 78. Project under construction; application of base fill (rock) at upstream end.

PHOTO NO. 2. 9 Nov. 78. Under construction; downstream portion of project.

PHOTOS 1 AND 2

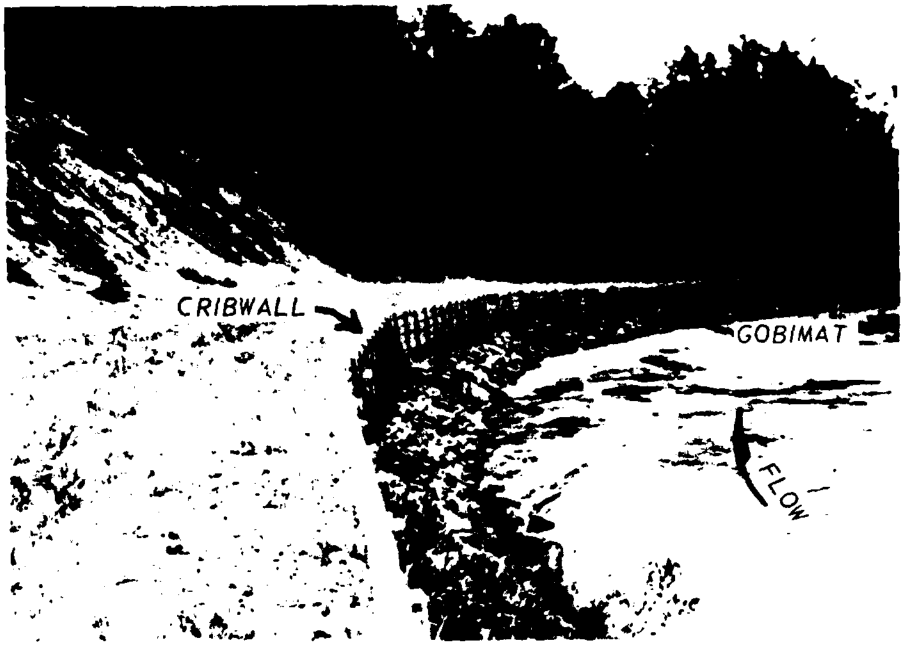


PHOTO NO. 3. 5 May 80. Completed project.

PHOTO NO. 4. Completed project showing unstable condition of upper bank. This bank is expected to gradually stabilize at its natural angle of repose.

PHOTOS 3 AND 4

PHOTO NO. 5. 5 Jan. 81. Erosion downstream end of project. Repair of this transition to natural section will be required.

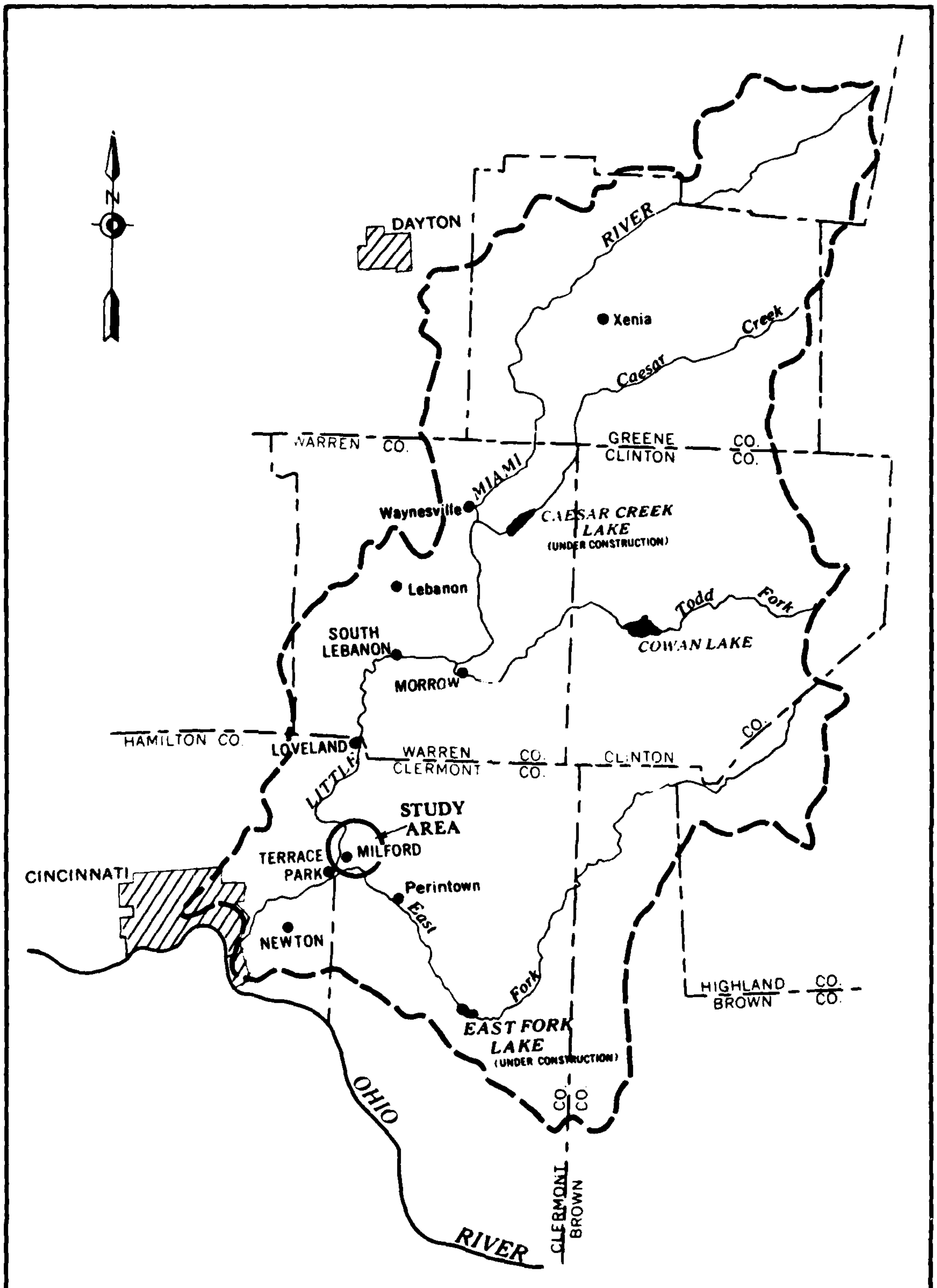
PHOTO NO. 6. Jan 81. Erosion upstream end of project. Transition repair will be required here also.

PHOTOS 5 AND 6

PHOTO NO. 7. Jan. 81. Very steep upper bank. Bank will continue to receed until stable slope is reached.

PHOTO NO. 8. Jan. 81. Note side of material from upper bank onto berm area. Some vegetation, mostly weeds, also occurring.

PHOTOS 7 AND 8



MILFORD, OHIO SITE
GENERAL MAP

PLATE 1

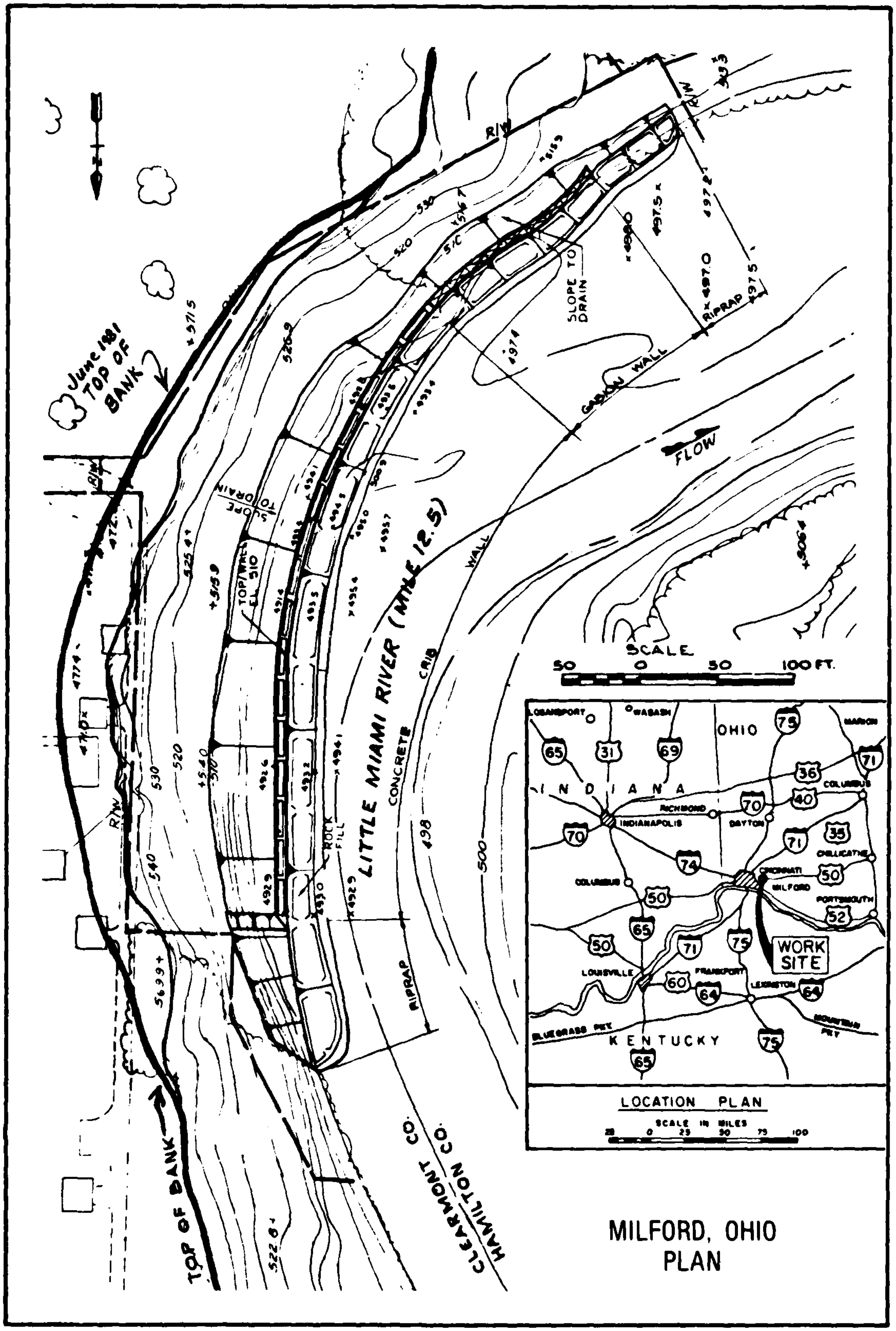

June 1981 TOP OF BANK
TOP OF BANK
R/W
SLOPE TO DRAIN
TOP/WALL EL 510
GABION WALL
FLOW
LITTLE MIAMI RIVER (MILE 12.5)
CONCRETE CRIB WALL
RIPRAP
ROCK FILL
CLEARMONT CO.
HAMILTON CO.
SCALE
50 0 50 100 FT.
OHIO
INDIANA
KENTUCKY
INDIANAPOLIS
RICHMOND
DAYTON
COLUMBUS
CINCINNATI
MILFORD
PORTSMOUTH
CHILLICOTHE
LOUISVILLE
FRANKFORT
LEXINGTON
WORK SITE
LOCATION PLAN
SCALE IN MILES
MILFORD, OHIO
PLAN

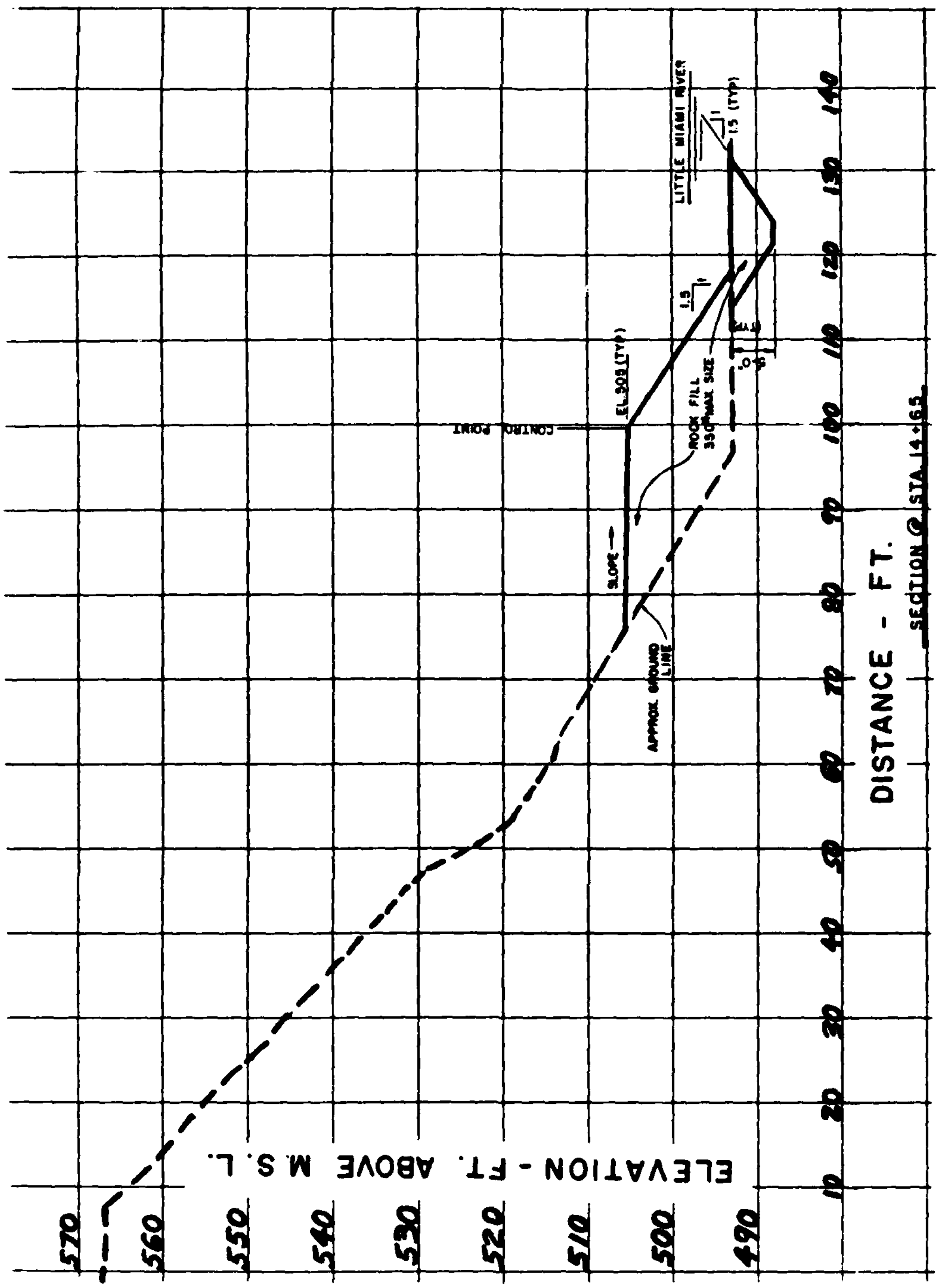
PLATE 2

DATA COLLECTION TABLE
MILFORD, OHIO SITE

Parameter	Item	Frequency
Geometry	1. Overbank cross sections thru various types of protection used. See Plates 4 thru 6.	Once-signif. changes would be resurveyed.
	2. Full channel cross sections.	Once
	3. Ground photos from fixed reference points.	Quarterly
Climate	1. Air temperature, precipitation, wind.	Continuous
	2. Ice conditions, snow cover noted from visual observations.	As available
Hydraulics	1. River stage record (U.S.G.S stage recorder .5 mile upstream).	Daily
	2. Stream velocity (measured using float and stop watch).	Quarterly
	3. Wave height (fixed staff gage).	Quarterly
	4. Other miscellaneous river conditions: current direction, turbidity, etc.	Quarterly
Streambank Protection	1. Monitor dimensional changes of marked structural and vegetal units through photos and manual measurement.	Quarterly
	2. Observe durability of marked units of structural material (qualitative).	Quarterly
	3. Observe condition of marked plants.	Quarterly
	4. Record initiation and measure progression of failures in bank protection.	Quarterly
Geology and Soils	1. Materials properties testing.	Once

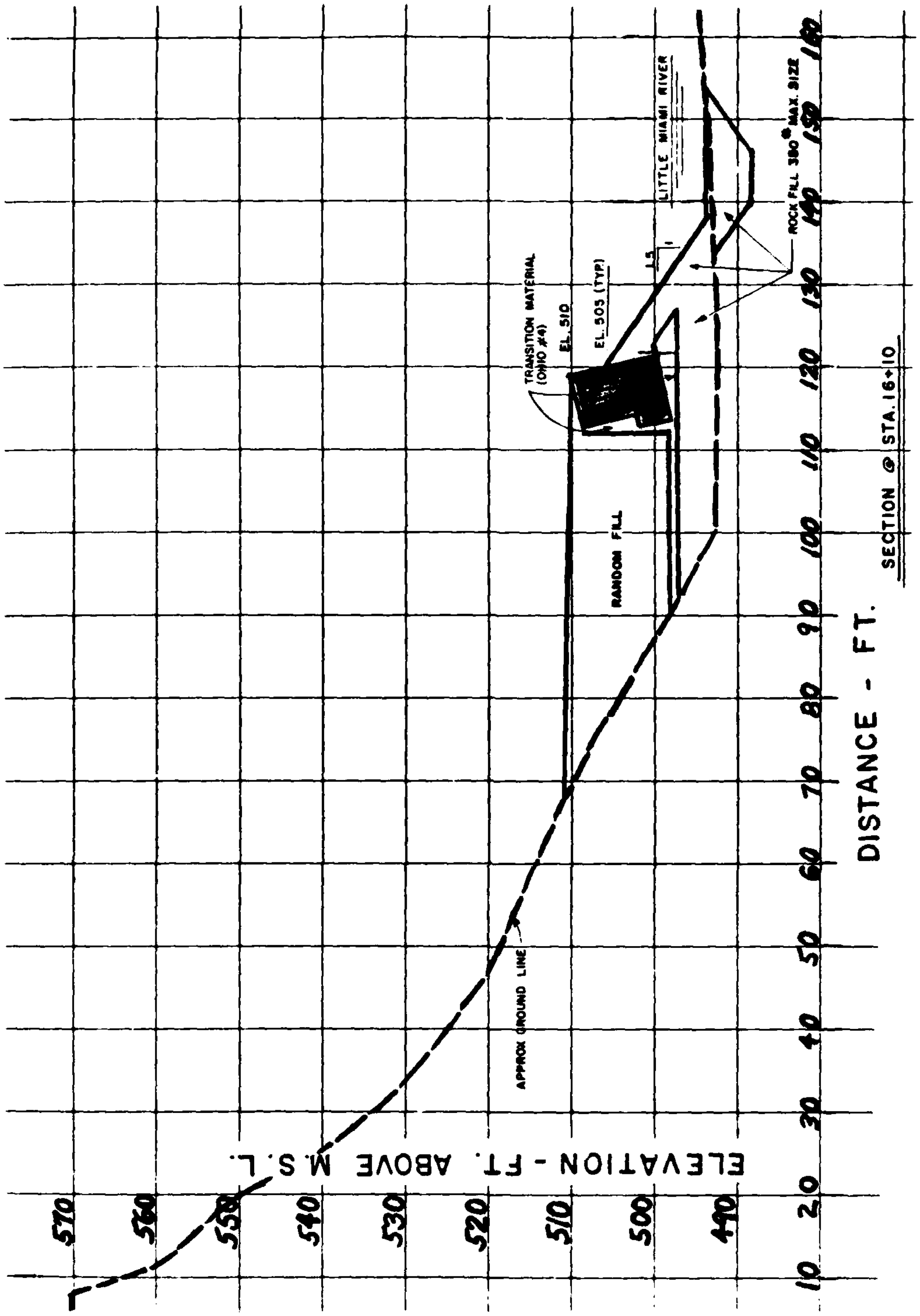
DATA COLLECTION TABLE

PLATE 3



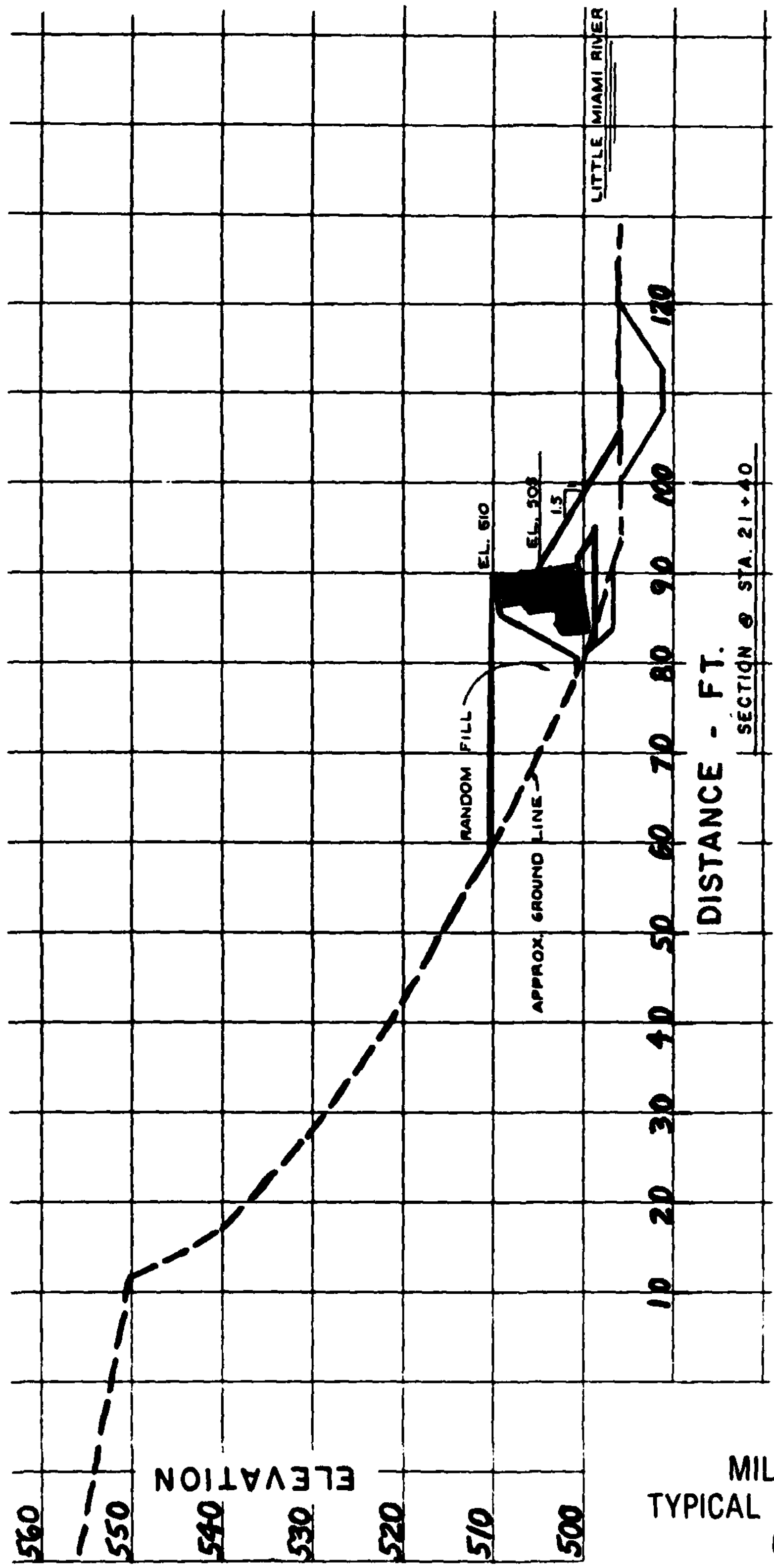
MILFORD, OHIO
TYPICAL CROSS SECTIONS
RIPRAP

PLATE 4



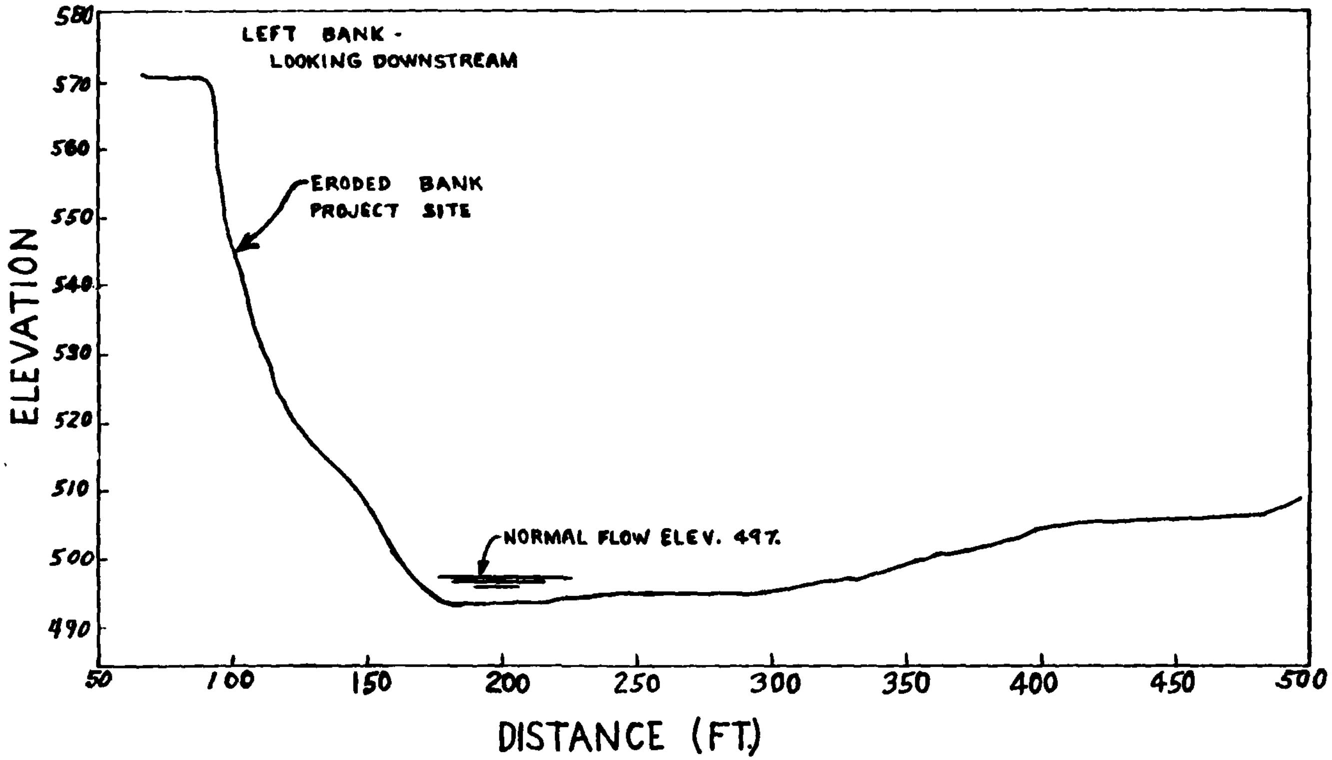
MILFORD, OHIO
TYPICAL CROSS SECTIONS
CRIB WALL

PLATE 5



MILFORD, OHIO
TYPICAL CROSS SECTIONS
GABIONS

PLATE 6



LITTLE MIAMI RIVER
NATURAL SECTION

PLATE 7

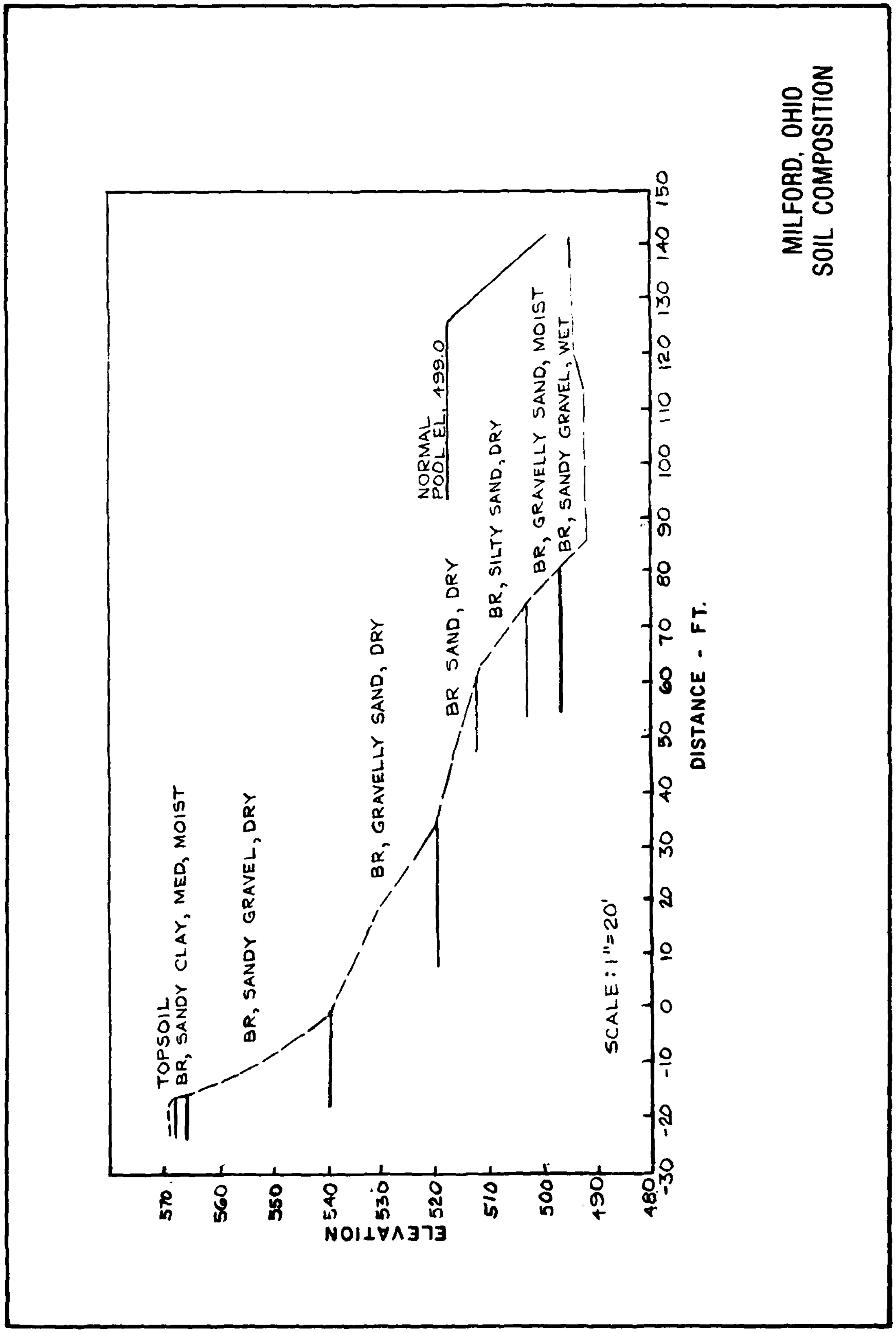
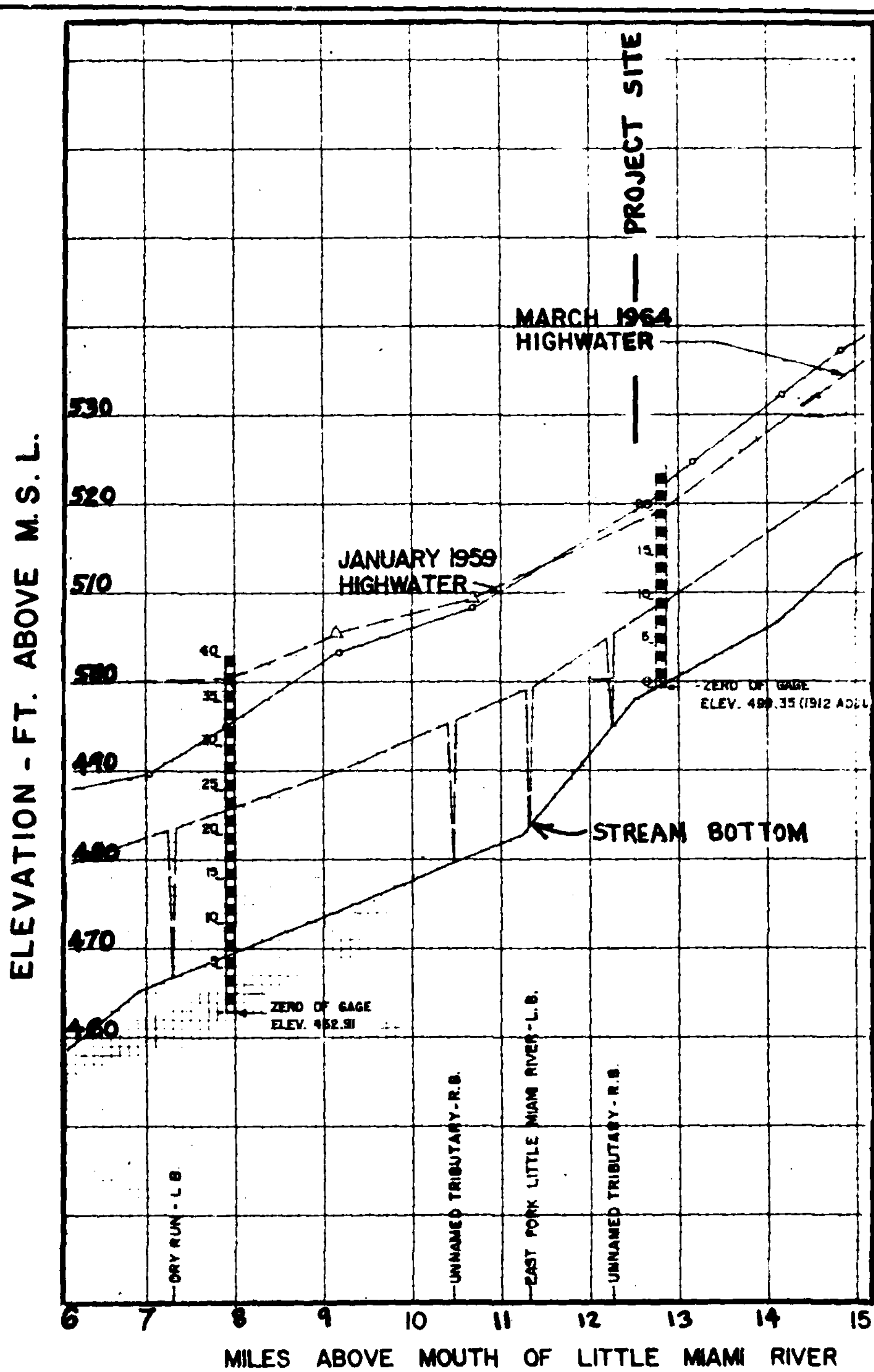
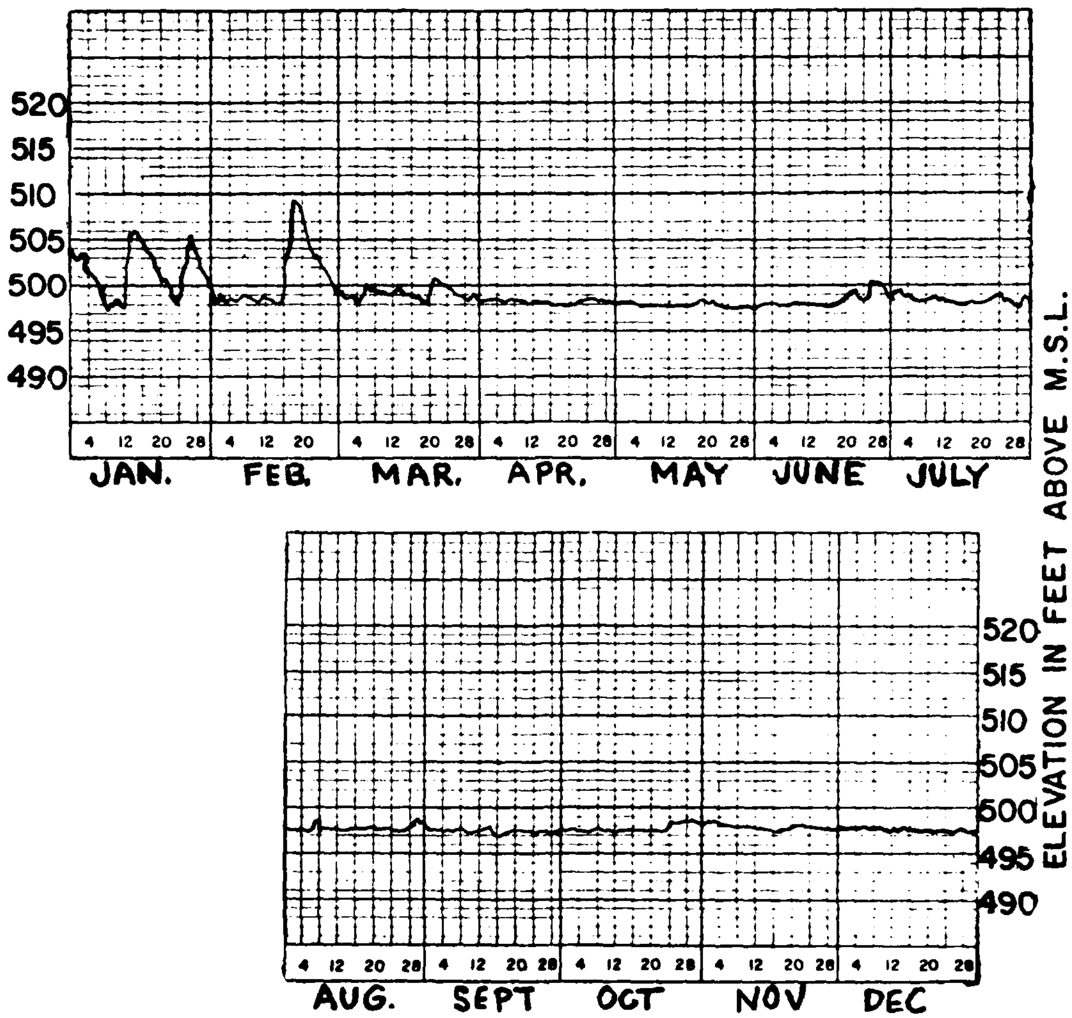

MILFORD, OHIO
SOIL COMPOSITION
TOPSOIL
BR, SANDY CLAY, MED, MOIST
BR, SANDY GRAVEL, DRY
BR, GRAVELLY SAND, DRY
BR SAND, DRY
BR, SILTY SAND, DRY
BR, GRAVELLY SAND, MOIST
BR, SANDY GRAVEL, WET
NORMAL
POOL EL. 499.0
SCALE: 1"=20'
ELEVATION
570
560
550
540
530
520
510
500
490
480
DISTANCE - FT.
-30
-20
-10
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150

PLATE 8



LITTLE MIAMI RIVER
PROFILES

PLATE 9

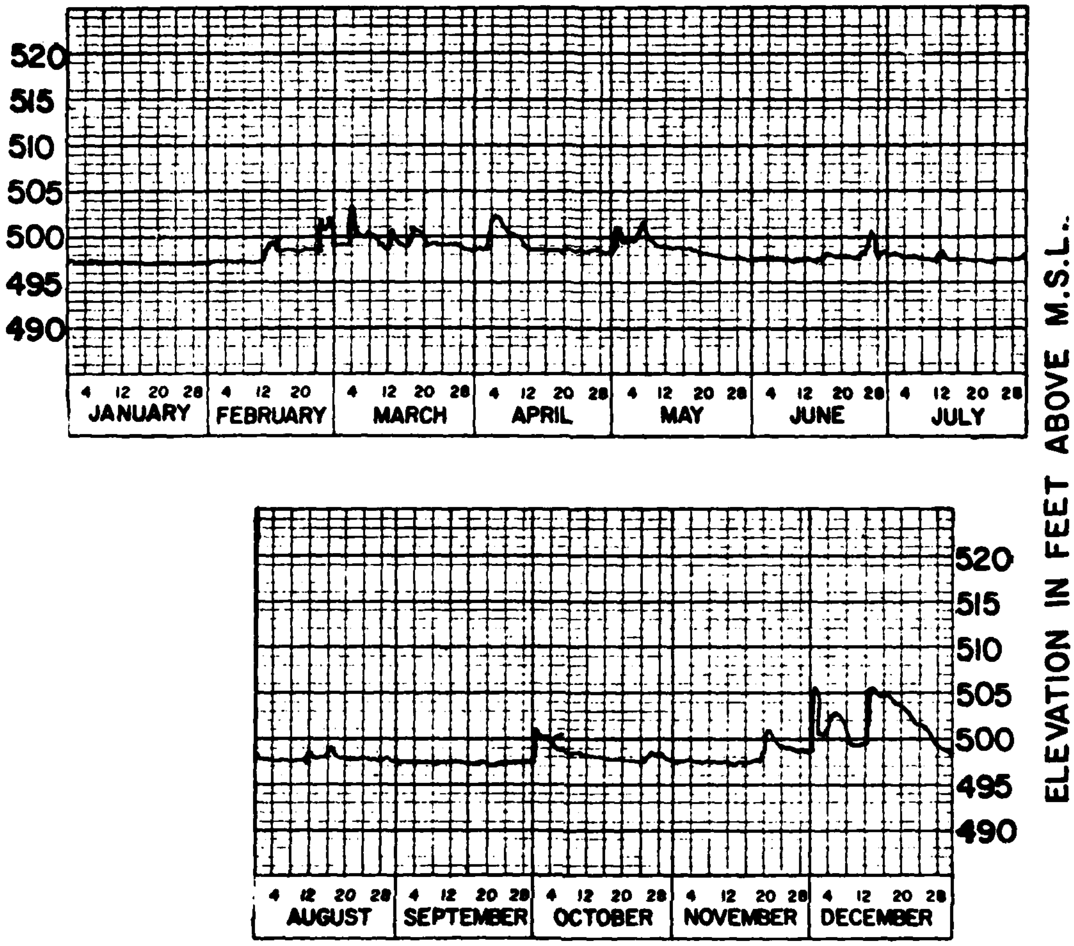


1976

HYDROGRAPHS WERE DERIVED FROM DAILY STAGE READINGS AT U.S.G.S. GAGE LOCATED ON U.S. 50 BRIDGE - MILE 12.8 LITTLE MIAMI RIVER

MILFORD, OHIO SITE
HYDROGRAPHS

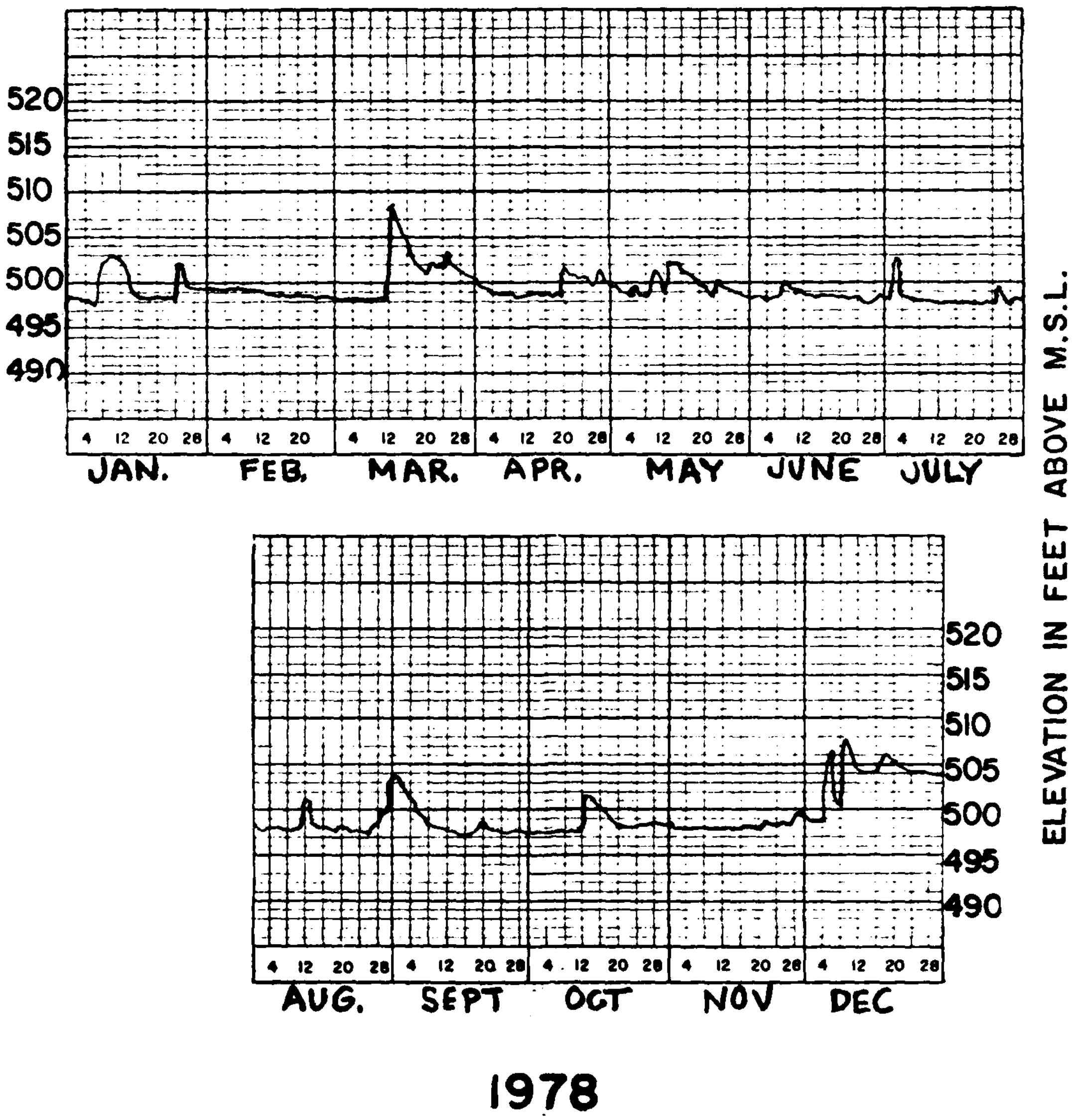
PLATE 10



1977

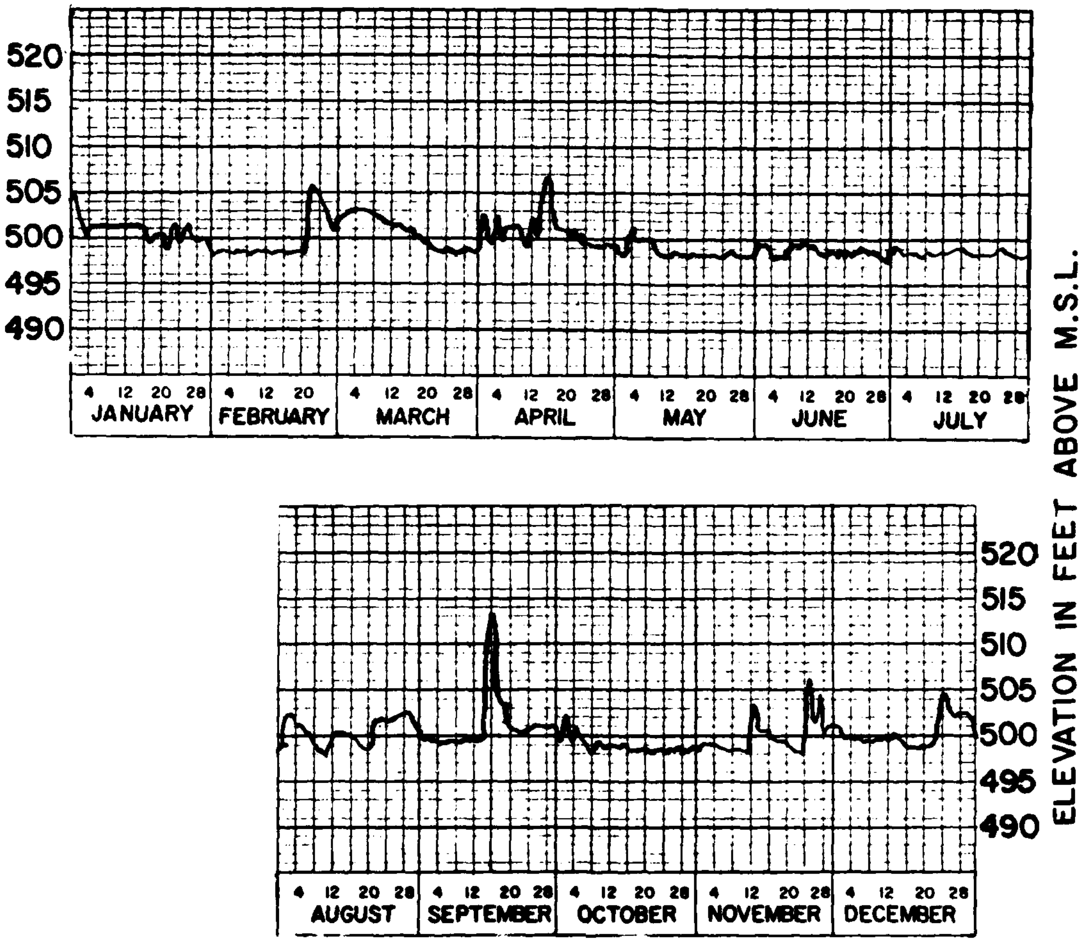
MILFORD, OHIO SITE
HYDROGRAPHS

PLATE 11



MILFORD, OHIO SITE
HYDROGRAPHS

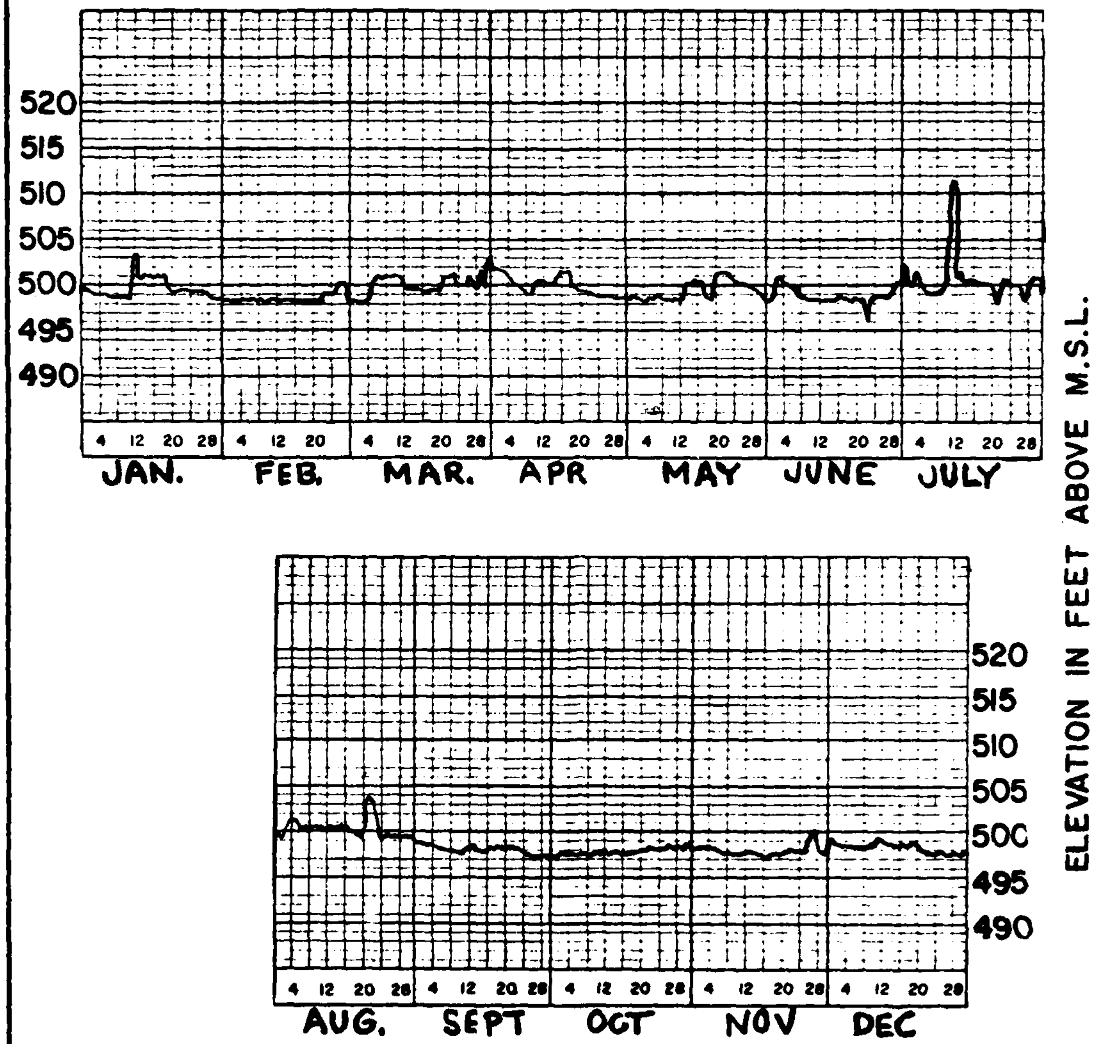
PLATE 12



1979

MILFORD, OHIO SITE
HYDROGRAPHS

PLATE 13

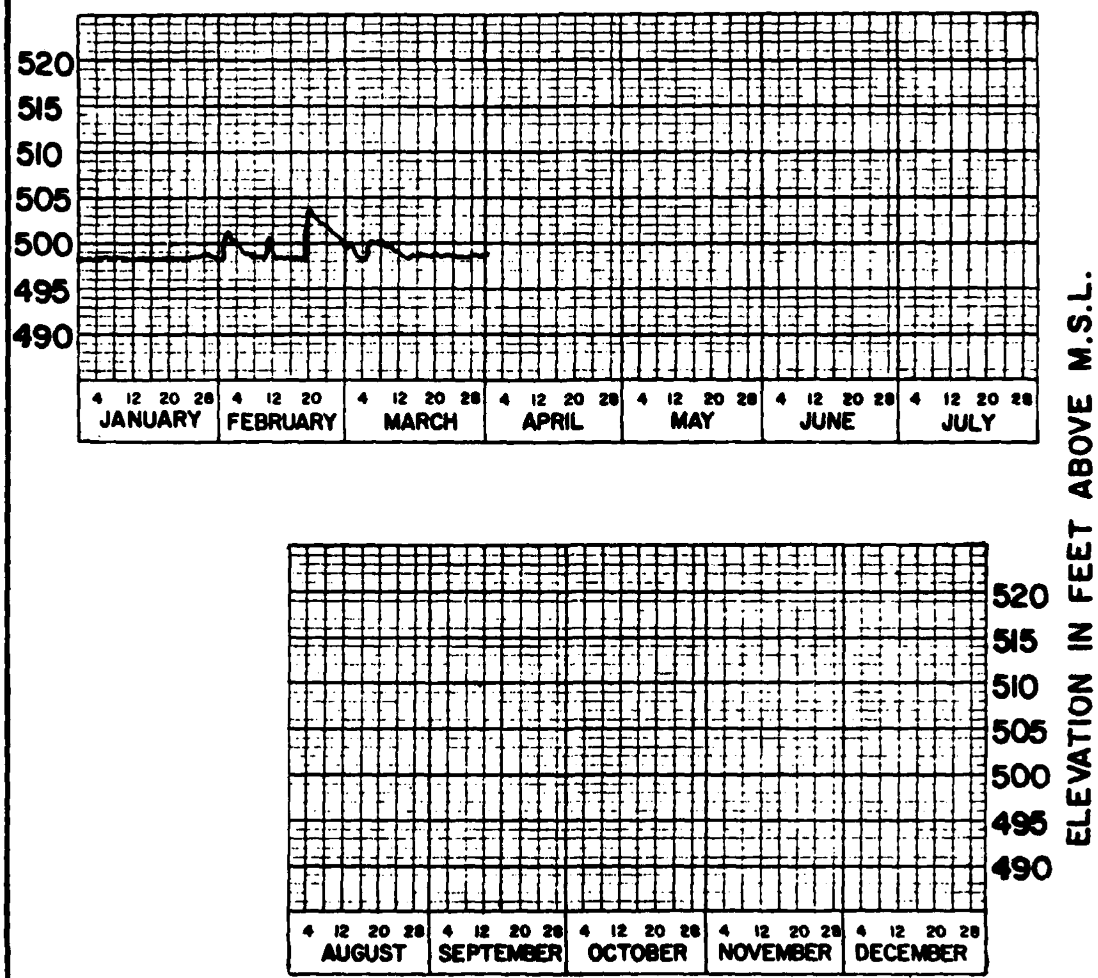


1980

HYDROGRAPHS WERE DERIVED FROM DAILY STAGE READINGS AT U.S.G.S. GAGE LOCATED ON U.S. 50 BRIDGE - MILE 12.8 LITTLE MIAMI RIVER

MILFORD, OHIO SITE
HYDROGRAPHS

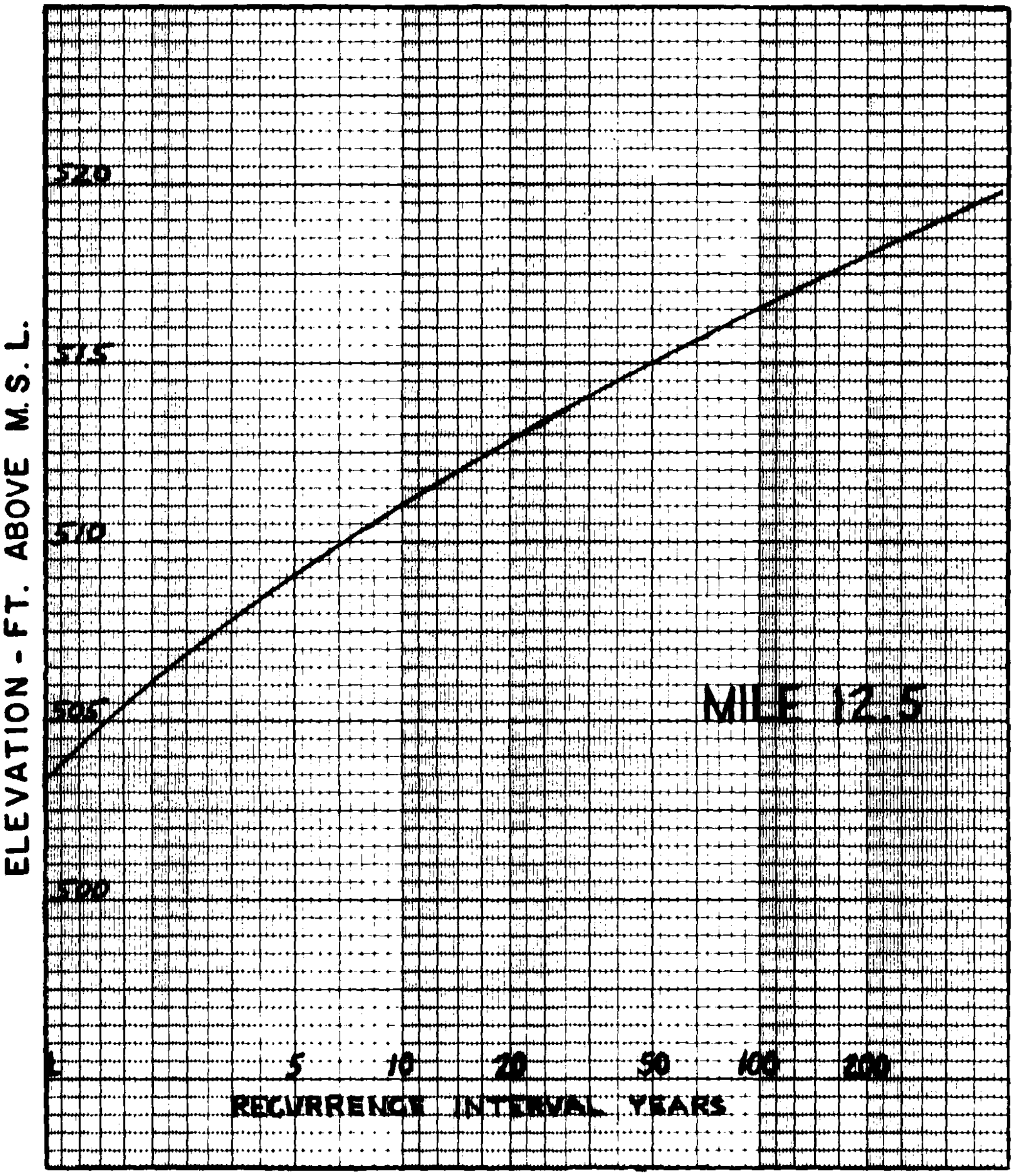
PLATE 14



1981

MILFORD, OHIO SITE
HYDROGRAPHS

PLATE 15



LITTLE MIAMI RIVER
ELEVATION FREQUENCY

PLATE 16

LOWER CHIPPEWA RIVER NEAR EAU CLAIRE, WISCONSIN

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

LOWER CHIPPEWA RIVER NEAR EAU CLAIRE, WISCONSIN
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. The project is on the lower Chippewa River near Eau Claire, Wisconsin. It extends about 9,900 bank feet along five different reaches from about river mile 15.4 to mile 35.5 above the Mississippi River. Plate 1 shows the project location.

2. Authority. The authority for the project is contained in the Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes the bank protection problem, the type of bank protection used, and an evaluation of the performance of the demonstration project on the Chippewa River. The project was constructed and is monitored by the St. Paul District, Corps of Engineers.

4. Problem Resume. Erosion along the Lower Chippewa River is of two types: (1) high bank erosion and (2) low bank erosion. The high bank chosen for protection is more than 100 feet high and is being eroded by undermining of the bank below the normal water surface which causes the material to slide. See photo 1 for typical high bank erosion. Low bank erosion occurs on banks generally 5 to 15 feet high. These banks are eroded in the same manner. Undermining beneath the water surface causes the material to slide while ground cover reinforces the soil at the top of the bank. This soil becomes cantilevered as erosion progresses and eventually breaks away. See photo 2 for typical low bank erosion.

Erosion of the banks below the water surface is primarily caused by the river current acting against the banks and by fluctuations in the water surface resulting from releases for hydropower. These fluctuations tend to saturate a narrow band of the silty sand banks during the higher stage. When the stage drops, the return seepage results in loss of strength in the first few inches of the surface soils causing minor sloughing or flow of material which allows the current to slowly carry or move it into the stream. These repeated actions steepen the bank causing failure.

Other factors which may contribute to erosion include ice movement, wave action, and wind.

The following criteria were used to select the sites for erosion control measures.

a. Site accessibility.

b. Stability of the reach with regard to flow patterns.

c. Workability of the bank.

d. Probability of local cooperation.

e. Environmental impacts of proposed actions.

f. Special suitability of the bank for certain types of erosion protection.

g. Value of the property protected or damaged by the project.

II. HISTORICAL DESCRIPTION

5. Stream.

a. General Topography. The Chippewa River basin, in the northwestern part of Wisconsin, extends 175 miles from Michigan to the Mississippi River. It has an area of 9,573 square miles. The land surface ranges from 670 feet above mean sea level at the mouth of the Chippewa River to 1952 feet at Tims Hill on the eastern basin divide. In the southern part of the basin, the land surface is irregular and ranges in elevation from 1200 feet along the divide to about 700 feet in the stream channels. Southwest of an eastward facing escarpment at Eau Claire, the country is hilly and composed of a maze of ridges and coulees. The lower course of the Chippewa River below Eau Claire has a uniform gradient of about 1.5 feet per mile and meanders broadly over its 1- to 2-mile wide floodplain. Erosional terraces more than 100 feet above the valley floor indicate the depth to which the valley was originally filled. The most striking feature of the lower Chippewa River is the delta, a great accumulation of sediment at its mouth, in the gorge of the Mississippi River. Because the gradient of the Chippewa River is much steeper than that of the Mississippi River, the smaller Chippewa River was able to bring in more and coarser debris than the master stream could handle. Thus, the delta was formed. Lake Pepin in the Mississippi River valley was created upstream of the dam-like delta.

b. Geology. The project area lies within the Western Upland geographic province of Wisconsin. The upland is a cuesta or plateau, capped with dolomite, in which the streams have cut deep, steep-sided valleys. Till and outwash were deposited over the area probably during the first substage of Wisconsin glaciation. The thin layer of sandy, silty, clayey till mantling the upland is now fairly well weathered, strongly leached, and therefore, acid. Most of the upland areas of till and rock are covered with a mantle of loess, a medium- to coarse-textured silt laid down by winds during Wisconsin glaciation. The silt is generally 3 to 5 feet thick. On the terraces and outwash plains adjoining the Chippewa River, the soils are mostly glacial sands and gravels. A mantle of silt and fine sand, laid down by wind, overlies the more granular soils on the terraces in many places. Figure 1 is a generalized cross section showing the geologic and topographic features in the vicinity of the project area.

Surface relief and soils of the area have been greatly influenced by the bedrock. The sequence of rock stratigraphy is, in ascending order, igneous and metamorphic Precambrian basement rocks which dip southwest at 15 feet per mile and lie 500 feet below the land surface at the mouth of the Chippewa River, easily erodible soft Cambrian sandstones which underlie the alluvial and outwash sediments in the valleys and abutting walls, and Ordovician dolomite which caps hills

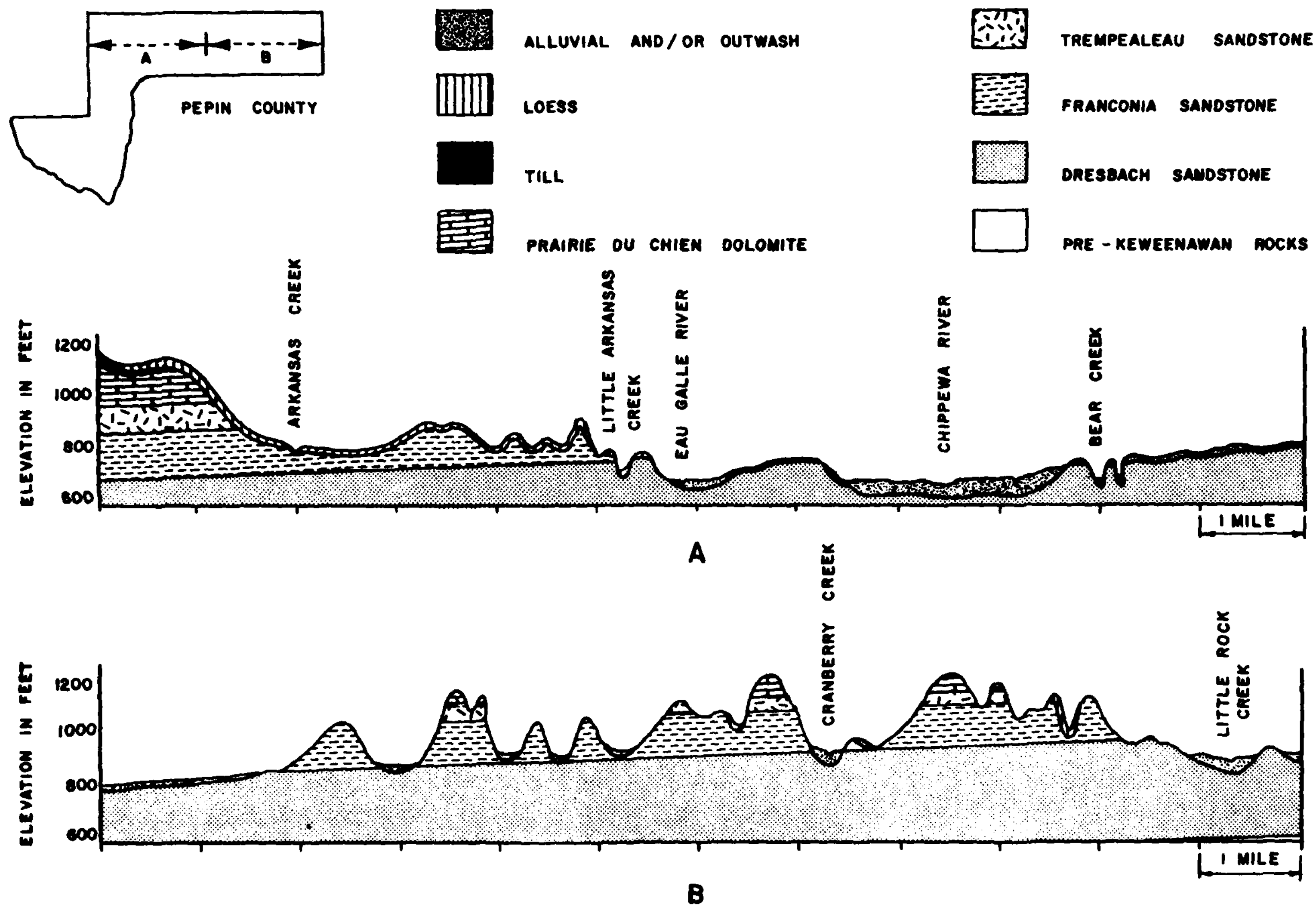


Figure 1. Geologic and topographic features

and ridges. The town of Durand is built on a sandstone terrace covered with a thin veneer of silts and sands from 4 to 15 feet thick.

Along the lower reaches of the Chippewa River, the valley bottom is deep and wide and bounded by uplands that rise abruptly 200 to 400 feet above the sandy floodplain. Along the main stream and its tributaries in the area, several levels of terraces and steep escarpments rise above the floodplains. The terraces were formed by the entrenchment of these streams, which cut deeply into the old floodplains. Erosional downcutting continues as the more steeply graded Chippewa River seeks the level of the Mississippi River. The natural lowering of the Chippewa River channel and streambank erosion are chiefly responsible for the heavy sediment load deposited in the Mississippi River at the mouth of the Chippewa River. Downstream 400 feet from site No. 1-Low, the floodplain is restricted to less than 1,200 feet in width as the river passes through a shallow rock gorge of sandstone at Round Hill. Sandstone is exposed on both sides of the river. Sandstone outcrops along the river in Spring Brook Township approximately 5½ miles upstream of Site No. 3-High. On the basis of well records and geologic interpretation, sandstone bedrock is expected to underlie the river at a depth of 50 feet or less in the project area.

c. Climate. The Chippewa River basin has a temperate climate characterized by marked seasonal changes. Average monthly temperatures range from 10° F for January in the north to 72° F for July in the south. The temperature extremes for the State of Wisconsin are 114° F on 13 July 1936 and -54° F for 22 January 1922. The average growing season is shorter than 100 days in the north and longer than 140 days in the south. In recent years, continuous temperatures below freezing have been recorded in the vicinity of the study area for about 60 to 70 days.

Precipitation is abundant in the basin, and periods of drought occur infrequently. Average annual precipitation for the basin was

31.2 inches from 1931 to 1960. February is normally the driest month (about 1 inch), and June is the wettest (about 5 inches). Snowfall averages about 50 inches annually and is approximately 16 percent of the average annual precipitation.

Water temperature records for the Chippewa River at Durand, Wisconsin, can be found in the Water Quality Records (1964-65, 1967, 1973 to current year) published by the U.S. Geological Survey in the Annual Water Resources Data for Wisconsin. The Chippewa River station is also a national stream quality accounting station, national pesticide monitoring network station, and national radio chemical surveillance station. Suspended sediment concentrations are published with the other water quality data. Mean suspended sediment concentrations for more than 20 percent of the year are estimated.

Surface wind rose data are available both annually and monthly. Published data show the percent of time wind blows from each of the 16 major directions. These data are available from climatic maps of the United States and regional summaries. Wind rose data from Eau Claire are shown on figure 2. Data on wind extremes are available from the National Oceanic and Atmospheric Administration for the first order stations and from the Federal Aviation Administration near airports.

d. <u>Existing Hydrologic Data.</u> Discharge records are available on the Chippewa River at Eau Claire for two periods: from November 1902 to March 1909 and from March 1944 to September 1954 (when the station was discontinued). The records were obtained from a former U.S. Geological Survey gaging station on the State Highway 37 and 85 bridge, about 2.8 miles downstream from the mouth of the Eau Claire River. Daily stage readings are available since June 1967 on the Chippewa River at Eau Claire, just below the junction of the Eau Claire River. Streamflow records have been kept on the Chippewa River upstream from Eau Claire at Chippewa Falls, Wisconsin, since June 1888 and at

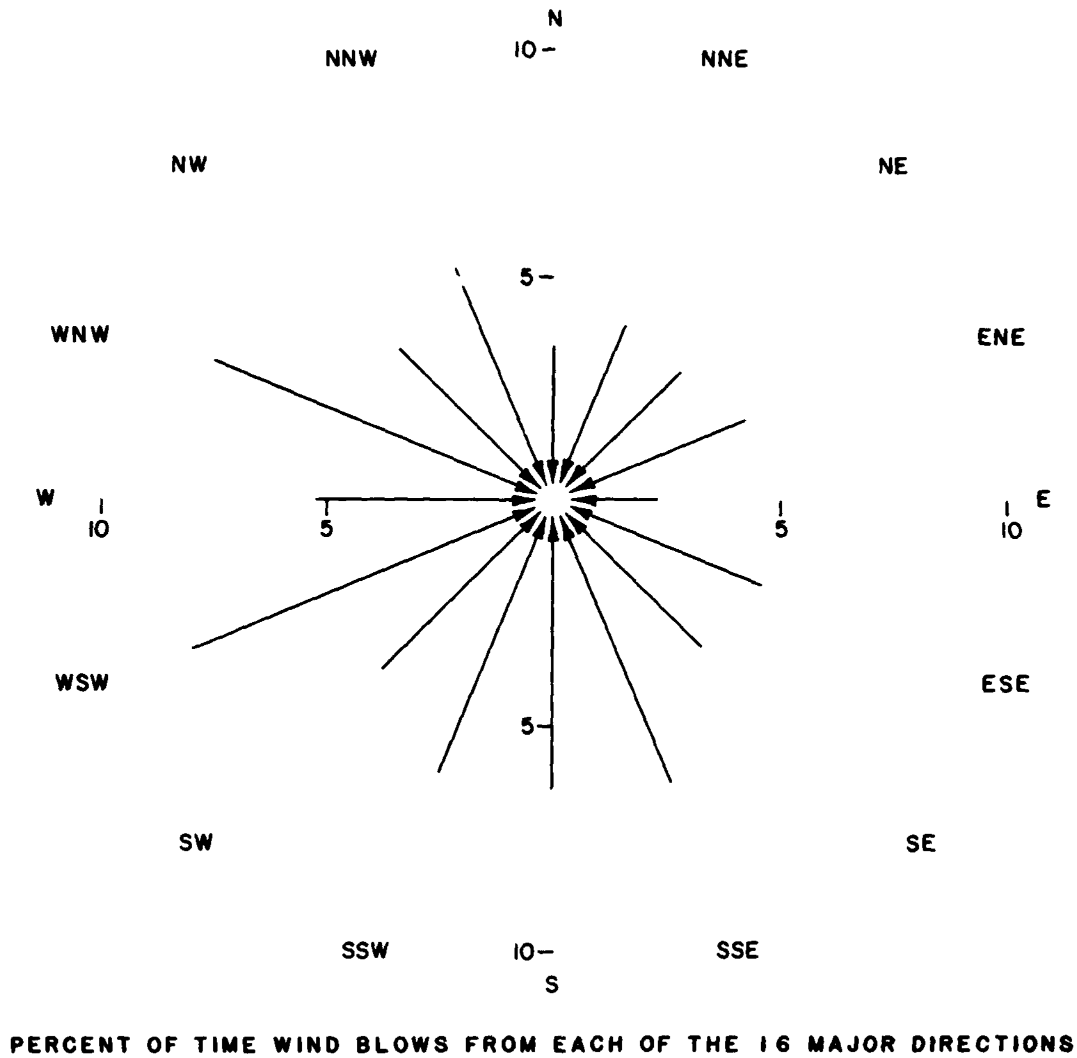


Figure 2. Wind Rose, Eau Claire, Wisconsin. All weather surface winds, 1960-1964, based on data by NOAA

Durand since July 1928. The average discharge for the 91 years of record at the Chippewa Falls station is 5,106 cfs (cubic feet per second). The recorded maximum discharge is 102,000 cfs (1 September 1941), and the recorded minimum discharge is 22 cfs (2 April 1934). The average discharge for the 51 years of record at Durand is 7,532 cfs. The recorded maximum discharge is 123,000 cfs (2 April 1967), and the recorded minimum discharge is 1,020 cfs (24 November 1950).

Two major tributaries, the Eau Claire River (drainage area = 881 square miles) and the Red Cedar River (drainage area = 1,870 square miles) enter the lower Chippewa River at Eau Claire and river mile 26.5, respectively. The gaging station on the Red Cedar River at Menomonie, Wisconsin (gage drainage area = 1,760 square miles), has a period of record from 1913 to the present. The average discharge is 1,248 cfs. The recorded maximum discharge is 40,000 cfs (4 April 1934), and the recorded minimum discharge is 21 cfs (9 December 1928). Streamflow records and characteristics are given in table 1.

On the Mississippi River, the long-term gaging stations closest to the mouth of the Chippewa River are the Prescott, Wisconsin, station (river mile 811.4) below lock and dam 2 (river mile 815.2) and the Winona, Minnesota, station (river mile 725.7) below lock and dam 5A (river mile 728.8). The Prescott station data include stages from 1891 and discharges from 1928 to the present.

Typical major and minor 30-day flood hydrographs are shown on plate 2 for the Chippewa River at Durand. The peak discharge for the major flood of 2 April 1967 is 123,000 cfs and represents a 2-percent exceedence frequency or 50-year flood. The peak discharge for the mino flood of 2 April 1976 is 62,300 cfs and represents approximately a 20-percent exceedence frequency or 5-year flood.

(1) Flow Duration Data. Flow duration data were computed fc the 51 years of record (1928-1979) for the Chippewa River at Durand. Monthly flow duration data were also computed for the 51-year record. Three monthly flow duration curves were plotted on each sheet and compared to the annual flow duration curve. The flow duration curve shows the percentage of time that a given flow is exceeded. These flow duration curves are shown on plates 3 and 4. The monthly flow duration curves show that the month of highest runoff is April, May is second highest, June is the third highest, and March is the fourth highest. August flow duration is the lowest of the year for the period of recor

(2) Runoff. The average discharge for the Chippewa River at Durand for the 51 years of record (1929-1979) is 7,532 cfs and represents 11.3 inches of average annual runoff. The maximum, minimum, and average discharges for the mean monthly and annual flows are summarized in figure 3 for the 51 years of record at Durand. The annual mean flows (water year and climatic year) and ranking for the period of record at Durand are summarized in table 2.

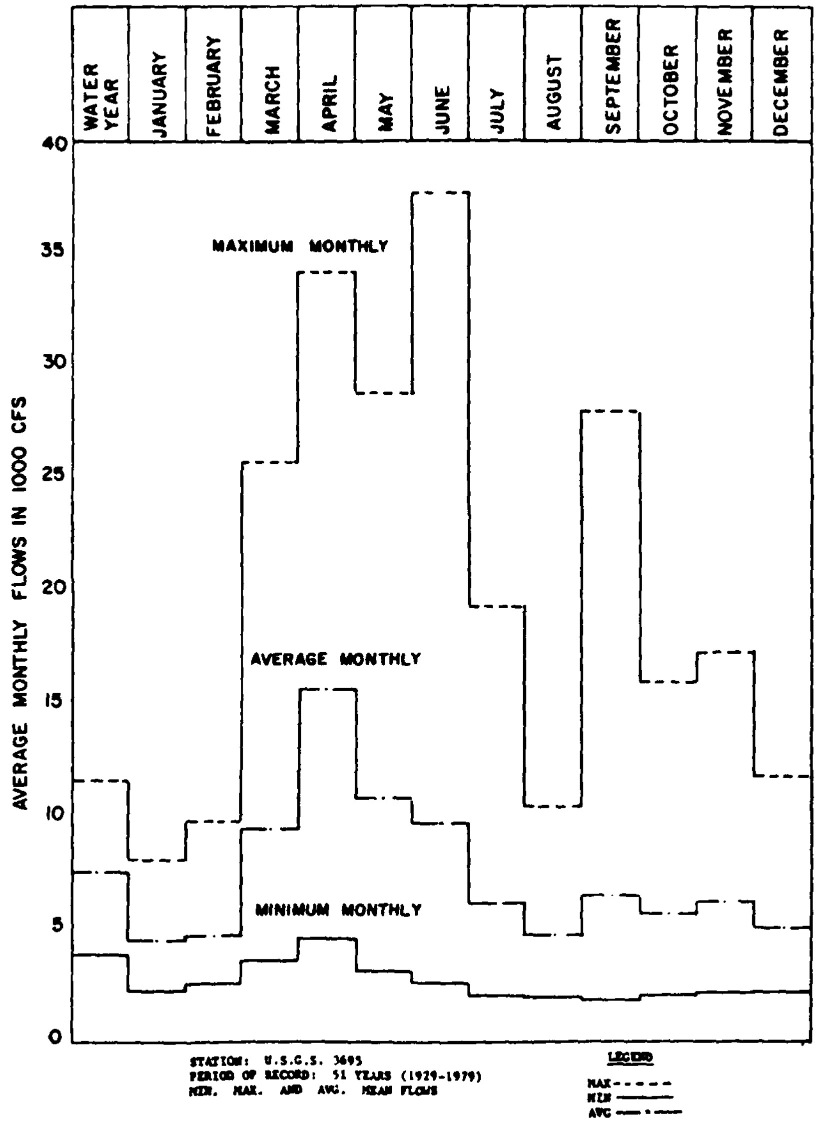


Figure 3. Table of average monthly flows, Chippewa River at Durand, Wisconsin

The average discharge for the Red Cedar River at Menomonie for the 67 years of record (1913-1979) is 1,248 cfs and represents 9.6 inches of average annual runoff.

(3) Flood Frequencies. Discharge records are available on the Chippewa River at Chippewa Falls since 1888, Eau Claire for a period of 17 years (1903-1908 and 1944-1954), and Durand since 1929. The Durand and Chippewa Falls records through 1967 were used to determine the frequency of discharges at Durand (the project area). In addition, peak flows for the six missing years at Chippewa Falls were estimated to give a total of 88 years (1880-1967). These estimates were made to incorporate the two large floods of 1880 and 1884.

An annual instantaneous peaks frequency curve was derived for Durand using the 39 years of record at that site (1929-1967) and statistically correlated to Chippewa Falls recorded and estimated flows. The log Pearson Type III method with zero skew and P_n adjustment was used to develop the curve shown on plate 5.

Another frequency curve for Durand was derived for all independent peaks. The upper part was based on statistical computations from Chippewa Falls data; the lower portion was plotted using Durand records adjusted by long records at Chippewa Falls. The independent peaks frequency curve is also shown on plate 5. Although the Durand frequency curves are based on records through 1967, they can be considered current. Updating the curves would not change them significantly.

e. Existing Channel Conditions. From its confluence with the Mississippi River to the town of Durand 16 miles up the valley, the Chippewa River is essentially a braided river with a sinuosity of 1.06. Sinuosity is the ratio of thalweg (greatest channel depth) length to the length of the river valley. The main channel is characteristically broad and shallow and contains shifting sandbars and sand islands.

The average channel width is 700 feet and the average depth is about 3 feet. The bank-full width is approximately 1,000 feet. Channel slope for this river reach is 1.76 feet per mile. Upstream from Durand to Eau Claire at river mile 61, the Chippewa River has a meandering configuration with a sinuousity of 1.49. This reach is characterized by eroding sand and gravel banks. The channel width is somewhat less than that below Durand, averaging about 600 feet. The channel slope for this reach is about 1.5 feet per mile.

A geomorphic study of the Chippewa River indicates that erosion of steep high banks is evident at several locations between Eau Claire and Durand. These locations include Yellow Bank at river mile 20.5, the right bank of the Chippewa River near Happy Island at river mile 35.5, and the left bank of the Lower Elk Creek entering the Chippewa River near river mile 45.5.

f. Locality, Development, and Occupation. Statistics on employment show that about two-thirds of the workers in the project area found employment in agriculture, services, and retail trade. Agriculture was the single largest sector of employment with about 25 percent.

Agriculture remains the predominant industry despite decreasing employment in this category from 1964 to 1974. Decreasing employment in agriculture appears to be primarily caused by a shift toward greater energy-intensive farming practices rather than a lessening of importance of agriculture in the region.

Population in the project area (Buffalo, Dunn, and Pepin Counties) was 53,313 in 1975, up 8.8 percent from 1950. About half this gain can be attributed to the national population increase in the area (i.e., birth rates exceeding mortality rates); the remainder resulted from immigration to the three-county area.

Median family income for the area (about $7,600) for 1969 shows families earn typically less than the median family income for Wisconsin ($10,068). These differences may be attributed to the predominance of agriculture in the area which usually has lower cash incomes associated with it.

g. Environmental Considerations. The floodplain vegetation of the Chippewa River is composed primarily of lowland hardwood and is characterized by species such as silver maple, green ash, elm, box elder, and willow. Vegetation along the upper bank includes oak, aspen, and birch with scattered stands of red and white pine along some of the higher banks. Upland areas adjacent to the Chippewa River are predominantly agricultural in nature.

Aquatic macrophytes in the study area are scarce because of the continual water level fluctuations, naturally occurring dark water color, shifting substrates, and harmful effluents. Sedges (carex sp.) are the most prevalent emergent aquatic vegetation in the study area.

A wide variety of wildlife may be found along the Chippewa River. Fauna of the region include 65 species of mammals, 44 species of reptiles and amphibians, and 237 species of birds. Some common species include white-tailed deer, fox squirrel, cottontail rabbit, muskrat, raccoon, woodchuck, leopard frog, garter snake, American toad, and turtle. Numerous passerine birds are common to the floodplain forest. Waterfowl such as mallards, wood ducks, and blue-winged teal inhabit the many ponds and marshy areas within the bottomland forest. Walleye, northern pike, black crappie, and channel catfish are the most abundant sport fish in the project area. Rough fish common to the area include sculpin, white sucker, northern redhorse, and carp.

The following species are listed as threatened or endangered by the U.S. Fish and Wildlife Service and could occur along the Chippewa River: prairie chicken, prairie falcon, peregrine falcon, bald eagle, Indiana bat, eastern timber wolf, and eastern cougar.

The Chippewa River, like many rivers in the Upper Mississippi River basin, is a nutrient-rich stream. Water quality degradation is caused more by agricultural drainage than municipal and industrial discharges. The water quality of the Chippewa River is considered fair for a warmwater stream.

Because of the localized nature of the anticipated impacts during construction, no significant adverse impacts on the fish and wildlife resources are anticipated.

The Chippewa River has potential for inclusion in the National Wild and Scenic Rivers System. The project was coordinated with appropriate State and Federal agencies to minimize the impacts and make the project more compatible in terms of its wild and scenic river potential. No negative comments were received from these agencies.

6. Demonstration Site - Test Reach.

a. Hydraulic Characteristics. Three across river sections were surveyed at each of the five sites. Velocity measurements were obtained in the fall of 1980 just before construction. These velocities ranged from less than 1 fps to 5.5 fps (feet per second) at discharges which ranged from 3,690 cfs to 18,100 cfs. Plate 6 shows the velocity distribution within the channel at the time the measurements were taken for two typical cross sections. This plate also shows that the banks have eroded over about a 1-year period. Velocity measurements obtained prior to construction are shown in Appendix A. Plate 7 shows the elevation-discharge relationship at the U.S. Geological Survey gage near Durand. This curve is based on historic records from the Geological Survey. Water surface profiles are shown on plate 8.

b. Riverbank Description.

(1) Bank Materials. The major soils of the low banks include undifferentiated sandy alluvial soils overlain by a shallow

silty horizon. The high bank is distinguished by a thick sandy subsurface horizon with up to 40 inches of silty topsoil. Mechanical analysis of sand samples taken from the banks in the test reaches indicate that the soils are basically poorly graded fine to medium sands to poorly graded silty sands. See plates 9-12 for typical gradation curves.

(2) Vegetation. The floodplain area vegetation consists primarily of lowland hardwoods (silver maple, green ash, elm, box elder, and willow). The vegetation near the upper banks is mostly scattered oak, aspen, and birch. Along some of the higher banks are scattered red and white pines.

III. DESIGN AND CONSTRUCTION

7. General. The five sites chosen for erosion control allowed for a design of 19 different test sections. The materials used for protection include concrete block, filter fabric, quarried rock fill, Enkamat, woven wire fencing, sandbags, snow fence, and local trees. Various placement techniques and configurations of materials make up the scheme of the demonstration project. Generally, construction will extend from the top of the bank into the water and along the bottom to a point less than 50 feet from the waterline at the river mean low-water level to limit erosion at the toe of the slope.

8. Basis for Design. The methods of protection were designed primarily on three considerations:

a. Use of inexpensive methods which may be applied by local interests.

b. Use of easily obtainable products or products designed for other applications that could be modified for erosion control.

c. Methods that have been effective or partially effective in other areas or other projects.

Some design configurations were altered so a determination could be made as to which configuration would be most effective.

Erosion of the banks caused by wind was not considered a major design factor. The geometry of the banks, and the materials and nature of the topsoil and ground cover indicate that erosion caused by wind is minor compared to erosion from river flow.

9. Construction Details. The design plan and sections for the five sites are shown on plates 13 through 34. Final surveys for these sites will be taken in spring 1981. Therefore, as-built drawings will not be included in this report. A brief description of the protection measure for each site follows.

a. Site No. 1 - Low (five test sections, plates 13-17).

(1) Two sections with a single layer of concrete blocks filled with sand above the water level. The blocks were placed with their major axis perpendicular to the shore for half of each section and parallel to the shore for the remaining half. One section was underlain with a filter fabric blanket; rock was placed at the toe of the other section.

(2) Three sections of various configurations of quarry-run rock fill.

b. Site No. 2 - Low (seven test sections, plates 18-24).

(1) One section of filter fabric weighted with rock fill at the top and bottom and rock laid randomly on the slope.

(2) Two sections with different styles of Enkamat soil reinforcement matting weighted with rock fill at the top and bottom and secured to the bank with metal pins.

(3) Three sections of upright woven wire fencing parallel to the bank with small trees, brush, and clippings placed landward of the fence.

(4) One section protected with nonbiodegradable sand-filled bags.

c. Site No. 3 - Low (five test sections, plates 25-28).

(1) Two sections of wire and wood snow fencing weighted with concrete blocks, partially submerged at the toe of the streambank, lying along the bottom parallel to the bank.

(2) One section with wire and wood snow fencing placed as above, with addition of upright fencing backfilled with brush along the streambank.

(3) Two sections with the trunks of large trees cabled to the bank at various spacings and lying in the river perpendicular to the bank or angled downstream.

d. Site No. 7 - Low (plates 29-31). Consists of two configurations of short rock-fill wing dams at various spacings.

e. Site No. 3 - High (plates 32-34). Erosion protection consists of varying quantities of rock fill dumped from the top of the 100-foot high bank.

Rock for the protection work was quarried stone with at least 50 percent greater than 25 pounds. The maximum size stone had to fit within the finished grade lines as shown for the typical sections. The finished structures presented a reasonably well-graded distribution of sizes.

10. Construction Problems. No significant problems were encountered in construction of the project. Dumping of rock fill over the top of the bank at Site No. 3 - High had unanticipated results. More than three-fourths of the rock lodged near the top of the bank during dumping. Most of the rock was expected to roll or slide to the bottom of the 100-foot high bank immediately. With spring runoff-induced erosion, more of the rock should make its way to the toe of the bank as originally intended. At Site No. 2 - Low, wooden stakes used to anchor the Enkamat pulled out of the bank and floated when placed beneath water. Metal stakes were fabricated and used to repin the matting. Distance between rock groins at Site No. 7 - Low was measured from the top of the bank at time of construction so their locations may vary with regard to the range lines as shown on the drawings.

No instrumentation was installed other than locations flagged for measuring velocity and range lines marked for surveying.

Construction activities had some temporary adverse impacts, which were generally restricted to the immediate project area. Clearing in the construction areas resulted in a minor loss of some vegetation. Erosion resulting from construction activities and consequent increases in the levels of suspended solids in the surface waters temporarily degraded water quality.

The placement of riprap and the construction of wing dams at selected demonstration sites resulted in temporary adverse impacts on shoreline and benthic communities because existing habitat was covered. However, these construction measures provided new habitat which was available for colonization by organisms outside of the construction area.

11. Costs. Costs for the project totaled $423,000. This included final site selection, preconstruction planning and coordination, project design, preparation of plans and specifications, project construction,

and administration and supervision of the construction contract. Construction costs including modifications for the project were $327,437. With a project length of 9,900 feet, the average cost for protection was about $33/foot. Costs ranged from $5.40/foot for trees anchored to the bank (Site No. 3 - Low, Section 5) to $87.70/foot for concrete blocks with rock fill toe protection (Site No. 1 - Low, Section 2). Table 3 summarizes costs for each section of the project.

IV. PERFORMANCE OF PROTECTION

12. Monitoring Program. The sites will be monitored after project construction. Monitoring will consist of gathering survey data, velocity measurements, and climatological and hydrologic data. Visual observations will be recorded and a complete photographic record will be made during site inspections.

Testing of materials in existing banks and of the rock fill is completed for each finished site and is not scheduled to be retested in the monitoring plan. The elements of the monitoring program are summarized in table 4.

13. Evaluation of Protection Performance. Because the completed sites were finished late in fall 1980, very little performance data have been collected. The prefinal inspection on 5 December 1980 revealed that an uprooted tree had floated into the bank and had broken and disrupted a small section of concrete blocks in section 1 of Site No. 1 - Low. At Site No. 2 - Low, ice movement had displaced some of the concrete blocks connected to the woven wire fencing in sections 5 and 6. This minor displacement causes no problems at this time; however, larger ice flows may distort the wire to which the block is connected. Deer tracks were seen on the sandbags in section 7 of Site No. 2 - Low, but the bags had not been damaged.

Because construction on most sites was so recently completed, results of revegetation (seeded and native) cannot be evaluated at this time.

Typical photos taken before, during, and after construction are shown on plates 35 through 47.

14. Rehabilitation. No major rehabilitation of the project or portions of the project is planned by the Corps. Because of the demonstration nature of the project and the strong potential for failure, the local sponsors (Pepin and Dunn Counties, Wisconsin) will not be required to reconstruct or maintain any portions of the project which fail during the monitoring period. They will, however, be required to maintain and operate the project later on provided that (1) the project has been deemed by the Government to be structurally and functionally sound for maintenance and the project is no longer needed for demonstration purposes and (2) the project has been formally turned over to the county (counties) for operation.

15. Conclusions. An evaluation of the effectiveness of the project is difficult at this time because most reaches were constructed in fall 1980. Preliminary indications are that:

a. Rock fill and concrete block configurations demonstrated at Site No. 1 - Low will perform satisfactorily under conditions similar to the Chippewa River with no or slight modifications from project designs.

b. Ice effects on filter fabric, Enkamat, and vertical longitudinal wire fencing as demonstrated at Site No. 2 - Low could preclude their use in areas where ice damage is a problem. However, Enkamat sections probably would perform much better if vegetation were established before ice effects occur. The St. Paul District in cooperation with The American Enka Company is contemplating reestablishment of Enkamat sections in early summer 1981. Sand-filled bags could also fail as a result of ice and weather effects although they withstood the winter of 1980-81 and spring high flows relatively intact.

c. Measures proposed at Site No. 3 - Low will be constructed in spring 1981. Therefore, no conclusions as to their effectiveness can be made at this time.

d. Wing dams constructed at Site No. 7 - Low performed well during the first 6 months after construction and were not affected by ice. High flows in spring 1981, however, have eroded the banks between some wing dams. Long-term effectiveness of wing dams and optimal spacing between them cannot be determined at this time.

e. Dumped rock fill at Site No. 3 - High cannot be evaluated at this time because of the before-mentioned construction problems. The site will continue to be monitored to determine whether the rock reaches the toe as intended and whether the rock is effective in preventing erosion if it reaches the toe.

AD-A121 139 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

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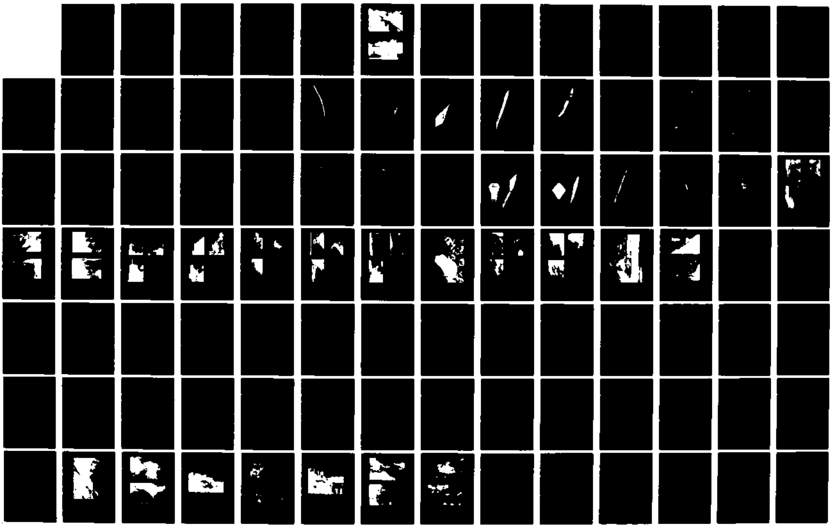
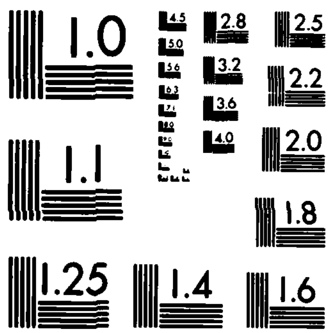




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Table 1. Streamflow Records and Characteristics, U. S. Geological Survey Gaging Stations

Station	River miles above mouth	Total drainage area (sq mi)	Gage zero elevation above msl (1929 adj)	Period of record		Maximum flood data			Minimum flow data		
				From	To	Date of maximum discharge	Discharge (cfs)	Gage height (feet)	Date of minimum discharge	Discharge (cfs)	Average discharge (cfs)
Chippewa River at Chippewa Falls, Wisconsin	75.4	5,680(1)	798.46(2)	Jun 1888	Date	1 Sep 1941	102,000	24.8(3)	2 Apr 1934	22	5,106
Chippewa River at Eau Claire, Wisconsin	56.8	6,630	750.00	Nov 1902 Mar 1944	Mar 1909 Sep 1954	2 May 1954	80,000	22.0	18 Dec 1907	190	5,755
						2 Apr 1967(4)	93,500	22.62			
Red Cedar River at Menomonie, Wisconsin		1,760	780.00	May 1913	Sep 1979	4 Apr 1934	40,000	16.0	9 Dec 1928	21	1,248
Chippewa River at Durand, Wisconsin	17.4	9,010	694.59	Jul 1928	Date	2 Apr 1967	123,000	16.93(5)	24 Nov 1950	1,020	7,532

(1) Revised USGS figure confirmed in 23 July 1963 letter from USGS.
(2) Prior to January 1914, nonrecording gage, and January 1914 to 19 June 1932, water stage recorded at site 1 mile upstream at different datum, 19 June 1932 to present, water stage recorded at present site and datum.
(3) Maximum stage known, 26.94 feet, 10 September 1884.
(4) Not operated as regular station, data observed at request of Corps of Engineers.
(5) Maximum stage known, 18.4 feet, 12 September 1884.

Table 2. Annual Mean Flows, Chippewa River at Durand
(Station Number 05369500)

Climatic year(1)	Average annual discharge (cfs)	Ranking	Water year(2)	Average annual discharge (cfs)	Ranking
1930	6620.00	18	1929	8550.0	19
1931	4860.00	6	1930	5500.0	41
1932	5410.00	10	1931	3990.0	51
1933	4810.00	5	1932	6180.0	39
1934	4240.00	2	1933	4790.0	46
1935	7040.00	26	1934	4030.0	50
1936	7940.00	32	1935	8660.0	17
1937	6460.00	17	1936	8100.0	21
1938	5660.00	12	1937	4970.0	43
1939	12800.00	50	1938	10400.0	5
1940	7580.00	28	1939	10800.0	4
1941	6930.00	23	1940	6270.0	36
1942	10000.00	43	1941	7690.0	24
1943	9960.00	42	1942	11600.0	1
1944	10400.00	44	1943	10900.0	3
1945	7980.00	33	1944	7400.0	30
1946	9360.00	41	1945	8590.0	18
1947	6900.00	22	1946	7450.0	29
1948	6680.00	20	1947	7630.0	28
1949	3720.00	1	1948	4510.0	47
1950	5250.00	9	1949	4470.0	48
1951	6660.00	19	1950	7360.0	31
1952	11000.00	47	1951	8910.0	14
1953	8290.00	35	1952	9460.0	6
1954	6700.00	21	1953	7200.0	33
1955	10900.00	46	1954	9280.0	7
1956	6270.00	15	1955	7710.0	27
1957	6210.00	14	1956	6440.0	35
1958	4750.00	4	1957	4820.0	45
1959	5480.00	11	1958	5460.0	42
1960	7660.00	30	1959	6220.0	38
1961	8420.00	36	1960	8790.0	16
1962	6030.00	13	1961	6680.0	34
1963	7580.00	29	1962	7210.0	32
1964	4700.00	3	1963	5790.0	40
1965	5050.00	7	1964	4440.0	49
1966	10400.00	45	1965	8260.0	20
1967	6330.00	16	1966	7790.0	26
1968	8190.00	34	1967	9110.0	10
1969	11800.00	49	1968	9140.0	9
1970	6980.00	25	1969	9030.0	11
1971	7730.00	31	1970	6230.0	37
1972	8500.00	37	1971	8820.0	15
1973	11200.00	48	1972	8990.0	12
1974	8970.00	39	1973	11500.0	2
1975	7380.00	27	1974	8010.0	22
1976	9350.00	40	1975	7850.0	25
1977	5180.00	8	1976	7970.0	23
1978	6960.00	24	1977	4860.0	44
1979	8670.00	38	1978	8960.0	13
			1979	9160.0	8

(1) In year ending 31 March.
(2) In year ending 30 September.

Table 3. Annual Mean Flows and Ranking

SITE 1 LOW

		COST/LINEAR FT.
SEC. 1	CONCRETE BLOCK OVER FILTER FABRIC	$ 55.60
SEC. 2	CONCRETE BLOCK WITH ROCK FILL AT TOE	$ 87.70
SEC. 3	ROCK FILL, EXISTING GRADE	$ 43.20
SEC. 4	ROCK FILL, CUT AND FILL TO GRADE	$ 45.00
SEC. 5	ROCK FILL TO THICKNESS ON EXISTING GRADE	$ 40.10

SITE 2 LOW

		COST/LINEAR FT.
SEC. 1	FILTER FABRIC WITH ROCK ANCHORING	$ 45.40
SEC. 2	ENKAMAT 7010	$ 68.80
SEC. 3	ENKAMAT 7020	$ 76.80
SEC. 4	VERTICAL FENCE AT SHORELINE WITH BRUSH BACKFILL	$ 34.40
SEC. 5	VERTICAL FENCE 3 FT RIVERWARD OF SHORELINE AND FENCE WIDTH PARALLEL TO SHORELINE AT FENCE BASE	$ 39.40
SEC. 6	VERTICAL FENCE AT SHORELINE AND FENCE WIDTH PARALLEL TO SHORELINE AT FENCE BASE	$ 33.40
SEC. 7	SANDBAGS	$ 48.40

SITE 3 LOW

		COST/LINEAR FT.
SEC. 1	FENCE WIDTHS PARALLEL TO SHORELINE GRADED TO FIT FLUSH WITH EXISTING GRADE	$ 15.10
SEC. 2	FENCE WIDTHS PARALLEL TO SHORELINE	$ 15.10
SEC. 3	VERTICAL FENCE BRUSH BACKFILL, PARALLEL FENCE WIDTH AT VERTICAL FENCE BASE	$ 11.80
SEC. 4	CABLE ANCHORED TREES AT ANGLE TO SHORELINE	$ 8.80
SEC. 5	CABLE ANCHORED TREES PERPENDICULAR TO SHORELINE	$ 5.40

SITE 7 LOW (SEC. 1 & SEC. 2)

$32,900 TOTAL COST
37 WING DAMS »»» $890/WING DAM

$14.40/LINEAR FT.

SITE 3 HIGH

DUMPED ROCK FILL OVER BANK EDGE $57.80/LINEAR FT.

Table 4. Monitoring Program - Chippewa River

Item	Frequency
Surveys - along all range lines	Prior to construction, after construction, at completion of the 5-year monitoring and when significant changes are noted during field inspections.
Velocity measurements - taken by USGS	Prior to construction and as determined necessary.
Visual observations	Semiannually or as necessary during and/or after large flow conditions.
Photography - complete photographic record taken from same location to allow comparison	Semiannually or as necessary during and/or after large flow conditions.
Materials testing - mechanical analysis run on existing bank conditions	Taken prior to placement of protection.
- mechanical analysis to ensure rock fill meets specifications	Five tests taken during construction.
Climatological and hydrologic data	Prior to construction and during the project monitoring.

PHOTO 1. SITE NO. 3 - HIGH, LOOKING UPSTREAM, BANK ABOUT 100 FEET HIGH, 30 OCTOBER 1980.

PHOTO 2. SITE NO. 2 - LOW, DOWNSTREAM SUBREACH, DOWNSTREAM TO RIGHT, BANK ABOUT 7 FEET HIGH, 25 SEPTEMBER 1980.

PHOTOS 1 AND 2

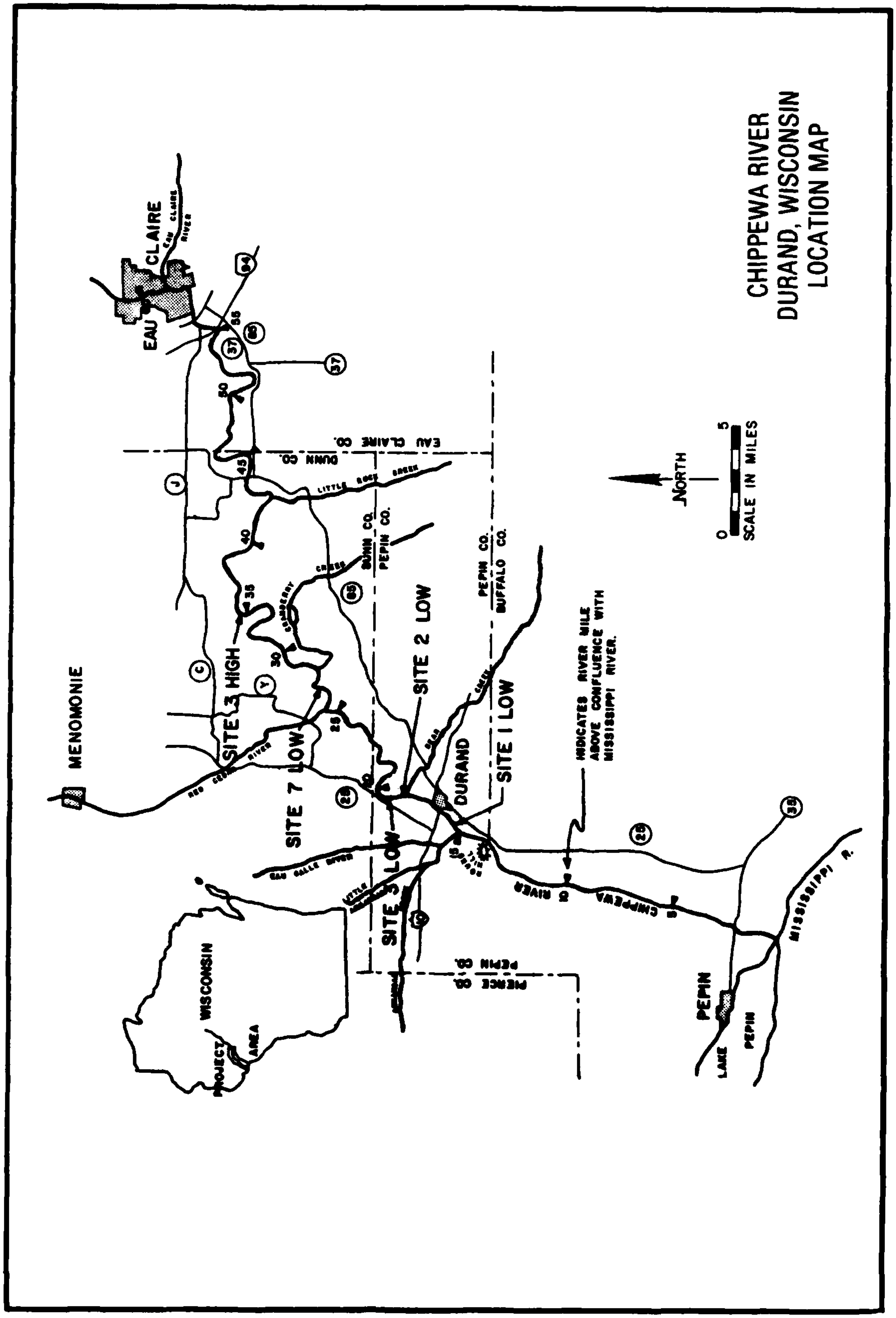
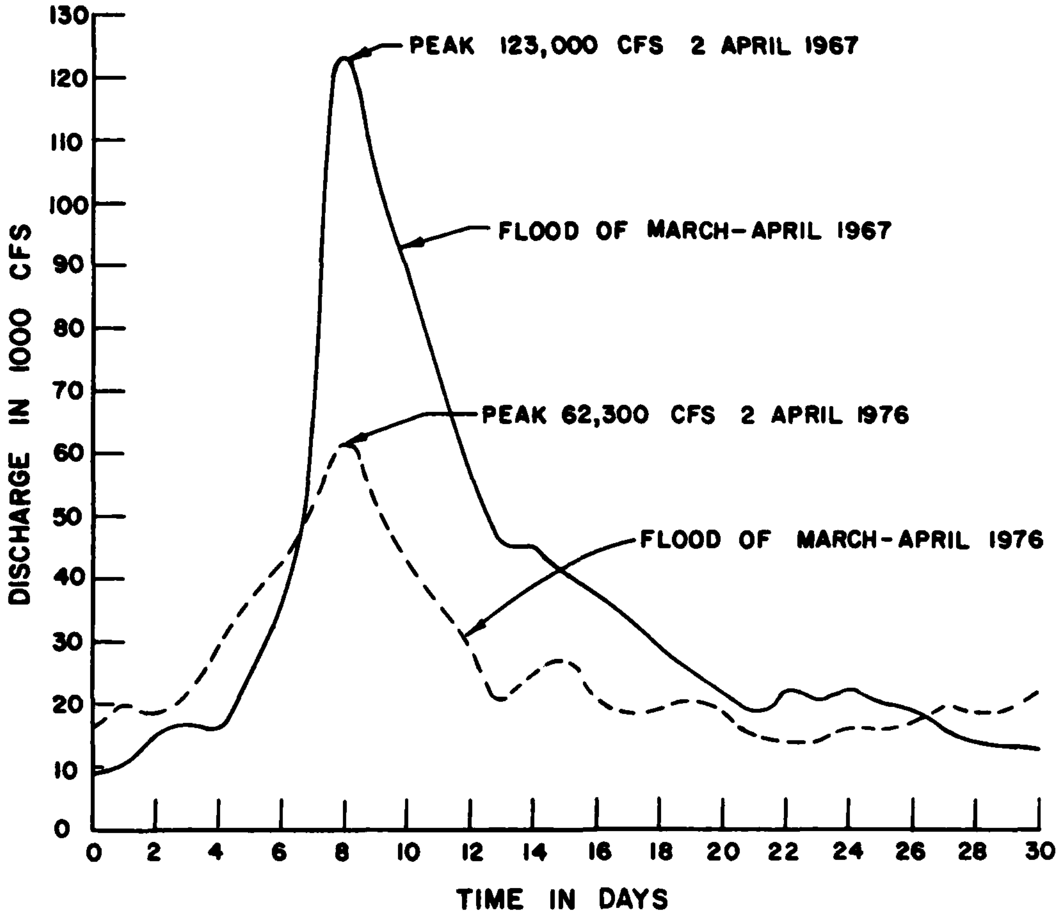
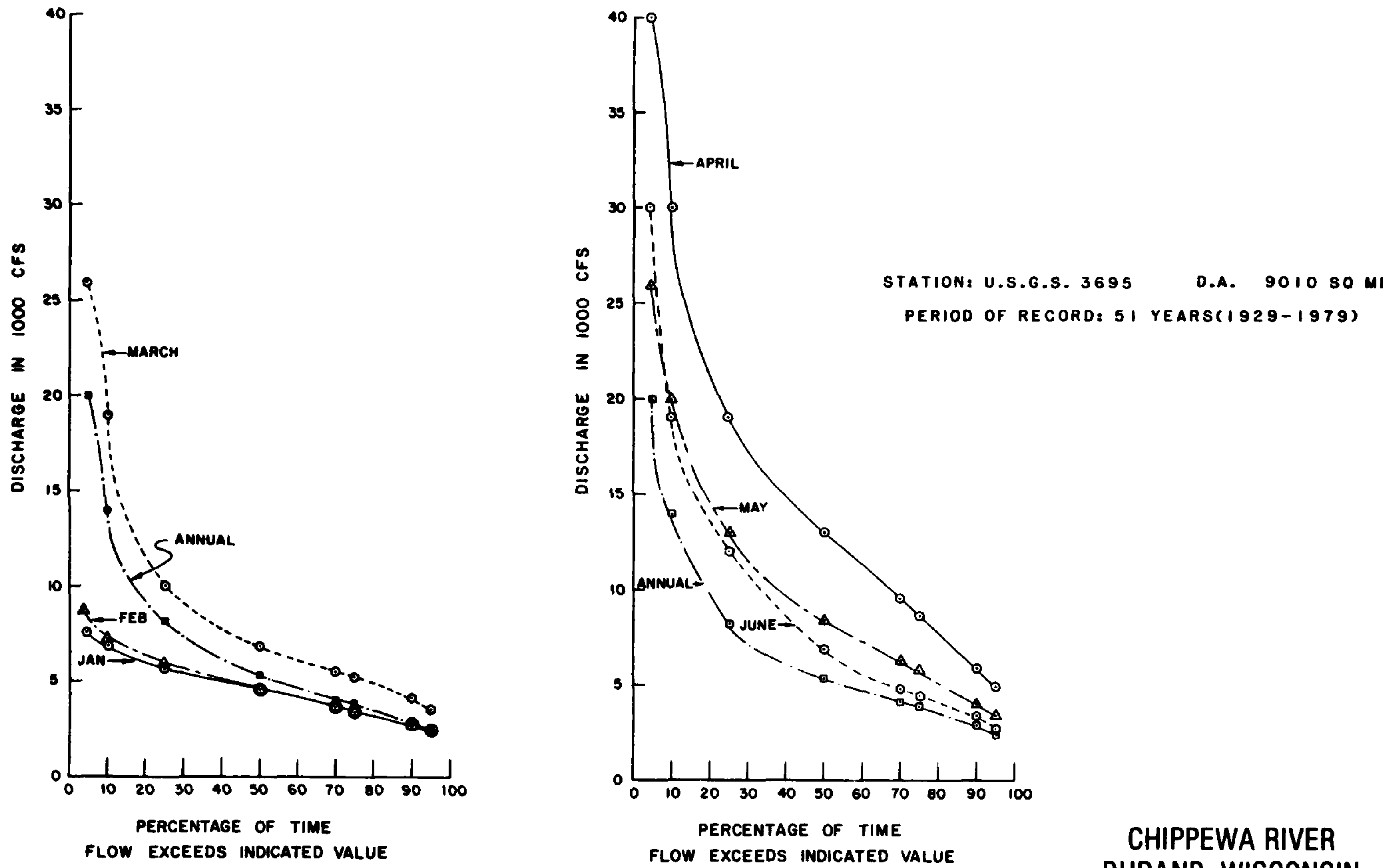

CHIPPEWA RIVER
DURAND, WISCONSIN
LOCATION MAP
EAU CLAIRE
EAU CLAIRE RIVER
MENOMONIE
DURAND
PEPIN
LAKE PEPIN
SITE 1 LOW
SITE 2 LOW
SITE 3 HIGH
SITE 5 LOW
SITE 7 LOW
EAU CLAIRE CO.
DUNN CO.
PEPIN CO.
BUFFALO CO.
PIERCE CO.
CHIPPEWA RIVER
MISSISSIPPI R.
RED CEDAR RIVER
EAU GALLE RIVER
LITTLE ROCK CREEK
INDICATES RIVER MILE ABOVE CONFLUENCE WITH MISSISSIPPI RIVER.
NORTH
0
5
SCALE IN MILES
WISCONSIN
PROJECT AREA

PLATE 1



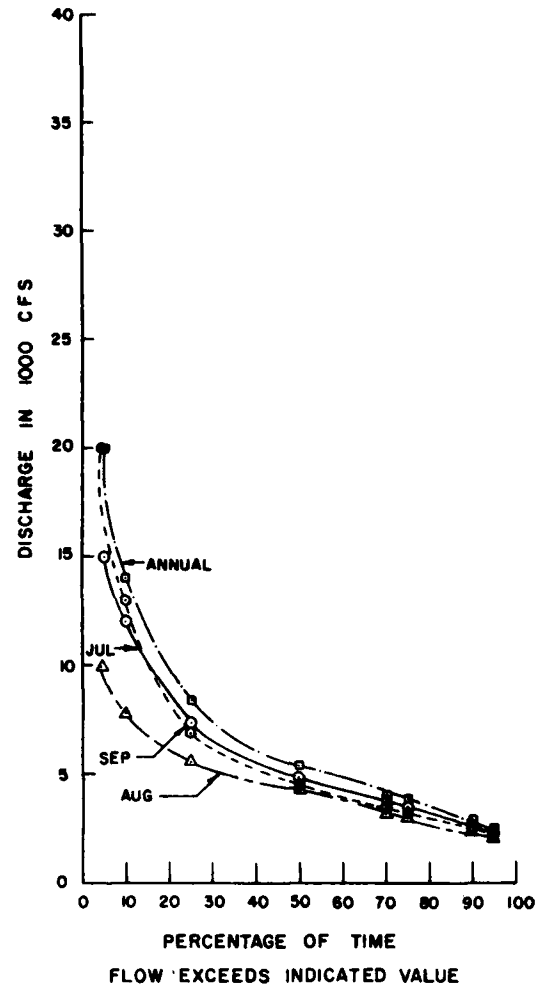
CHIPPEWA RIVER
DURAND, WISCONSIN
TYPICAL MAJOR AND MINOR FLOODS
HYDROLOGIC CHARACTERISTICS

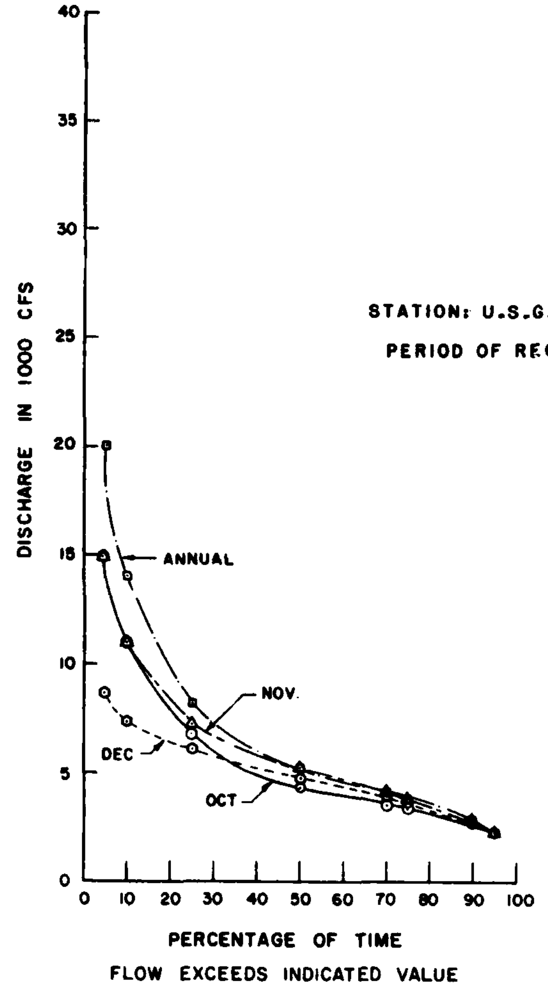
PLATE 2



CHIPPEWA RIVER
DURAND, WISCONSIN
FLOW DURATION CURVE

PLATE 3

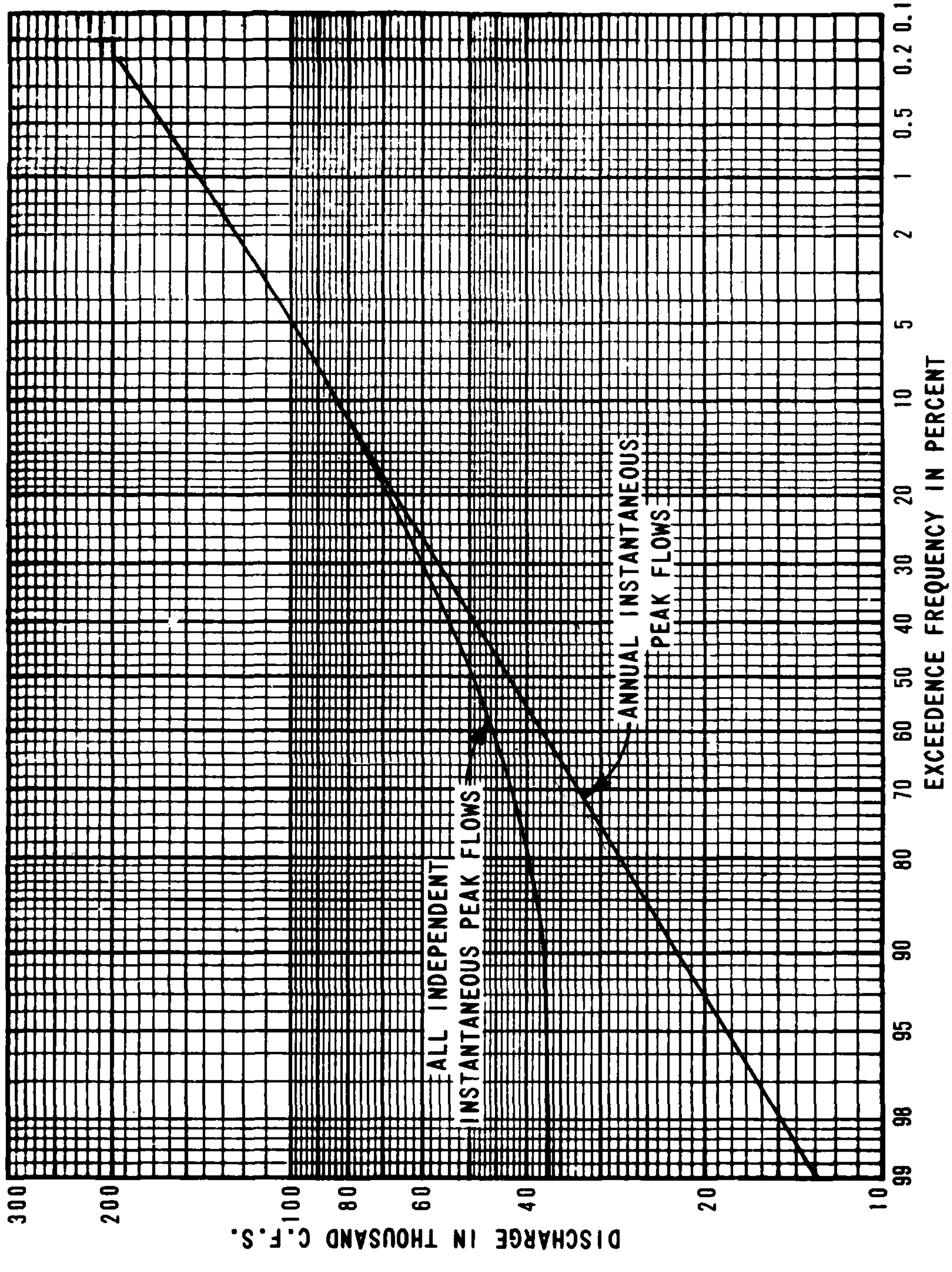




STATION: U.S.G.S. 3695 D.A. 9010 SQ MI
PERIOD OF RECORD: 51 YEARS(1929-1979)

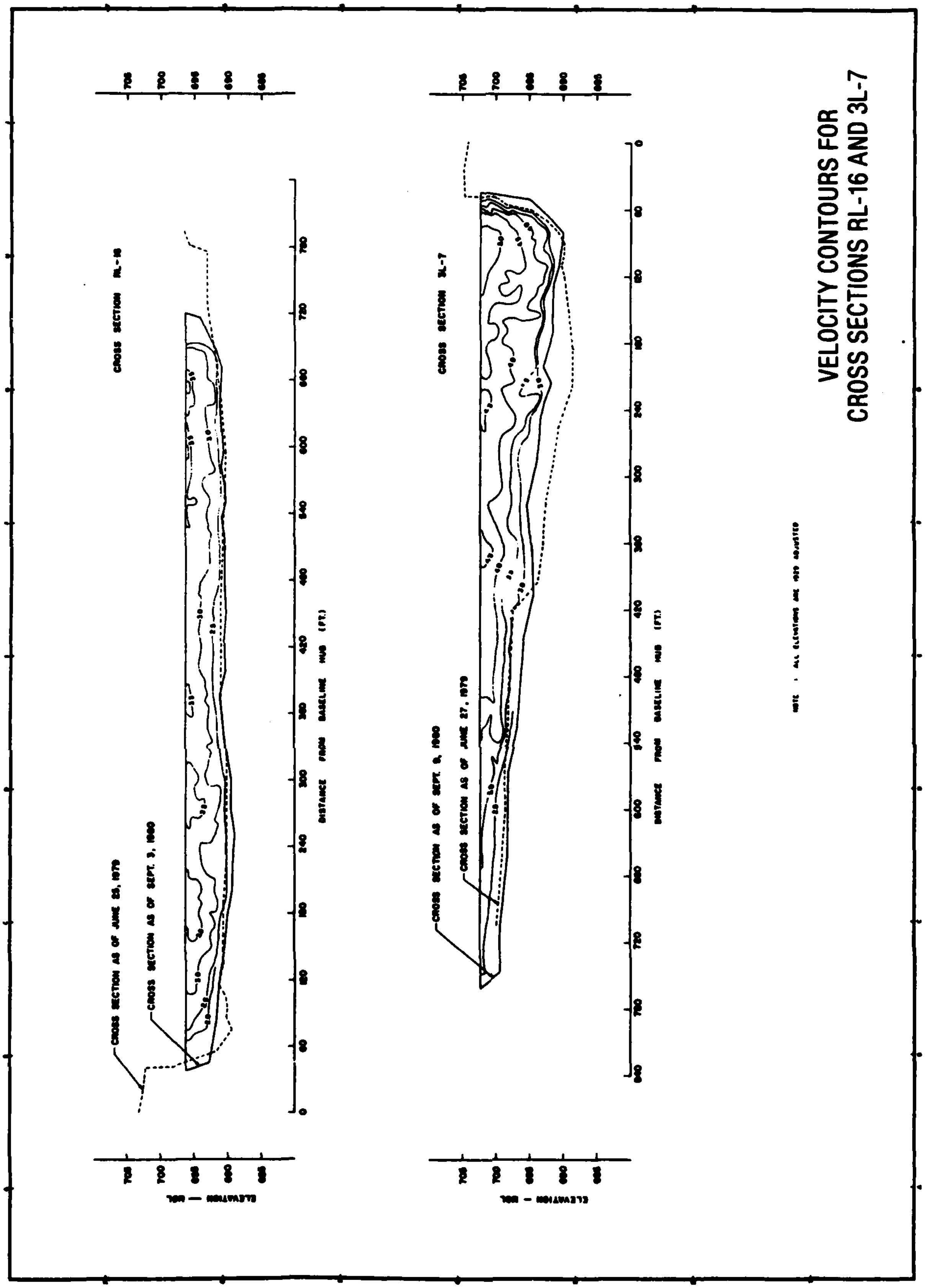
CHIPPEWA RIVER
DURAND, WISCONSIN
FLOW DURATION CURVE

PLATE 4



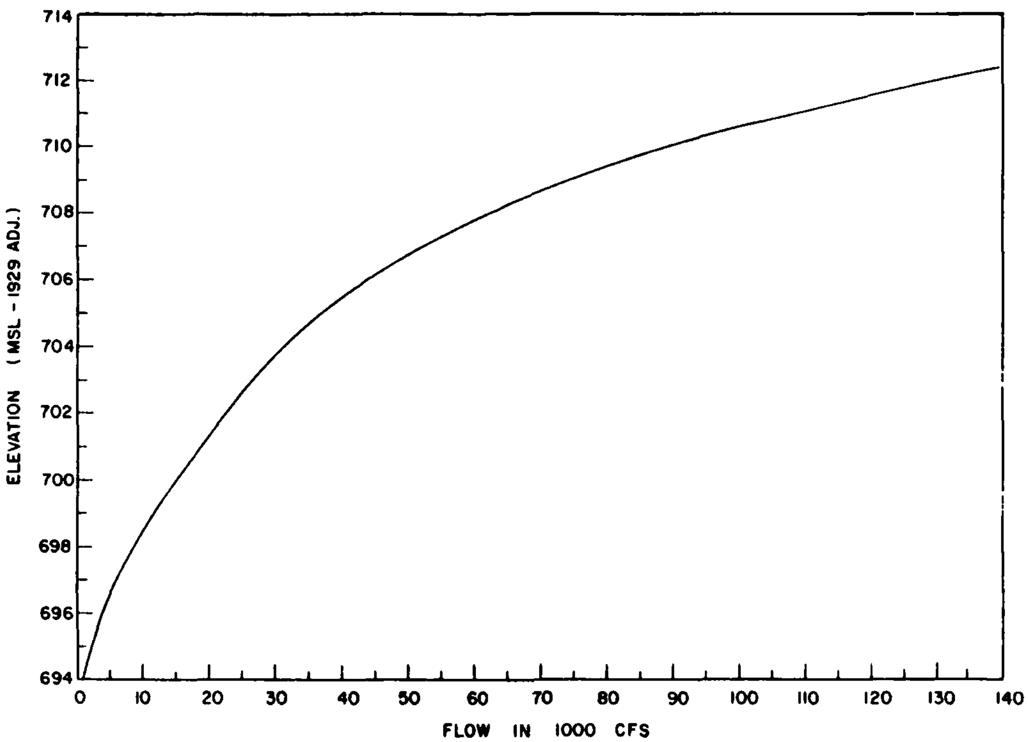
CHIPPEWA RIVER
DURAND, WISCONSIN
DISCHARGE - FREQUENCY CURVES

PLATE 5



VELOCITY CONTOURS FOR
CROSS SECTIONS RL-16 AND 3L-7
CROSS SECTION RL-16
CROSS SECTION 3L-7
CROSS SECTION AS OF JUNE 25, 1979
CROSS SECTION AS OF SEPT. 9, 1980
CROSS SECTION AS OF SEPT. 9, 1980
CROSS SECTION AS OF JUNE 27, 1979
DISTANCE FROM BASELINE HUB (FT.)
DISTANCE FROM BASELINE HUB (FT.)
ELEVATION — MSL
ELEVATION — MSL

PLATE 6



U.S.G.S. GAGE 05369500

GAGE ZERO: 694.59 1929 ADJ.

CHIPPEWA RIVER
DURAND, WISCONSIN
ELEVATION—DISCHARGE RATING CURVE

PLATE 7

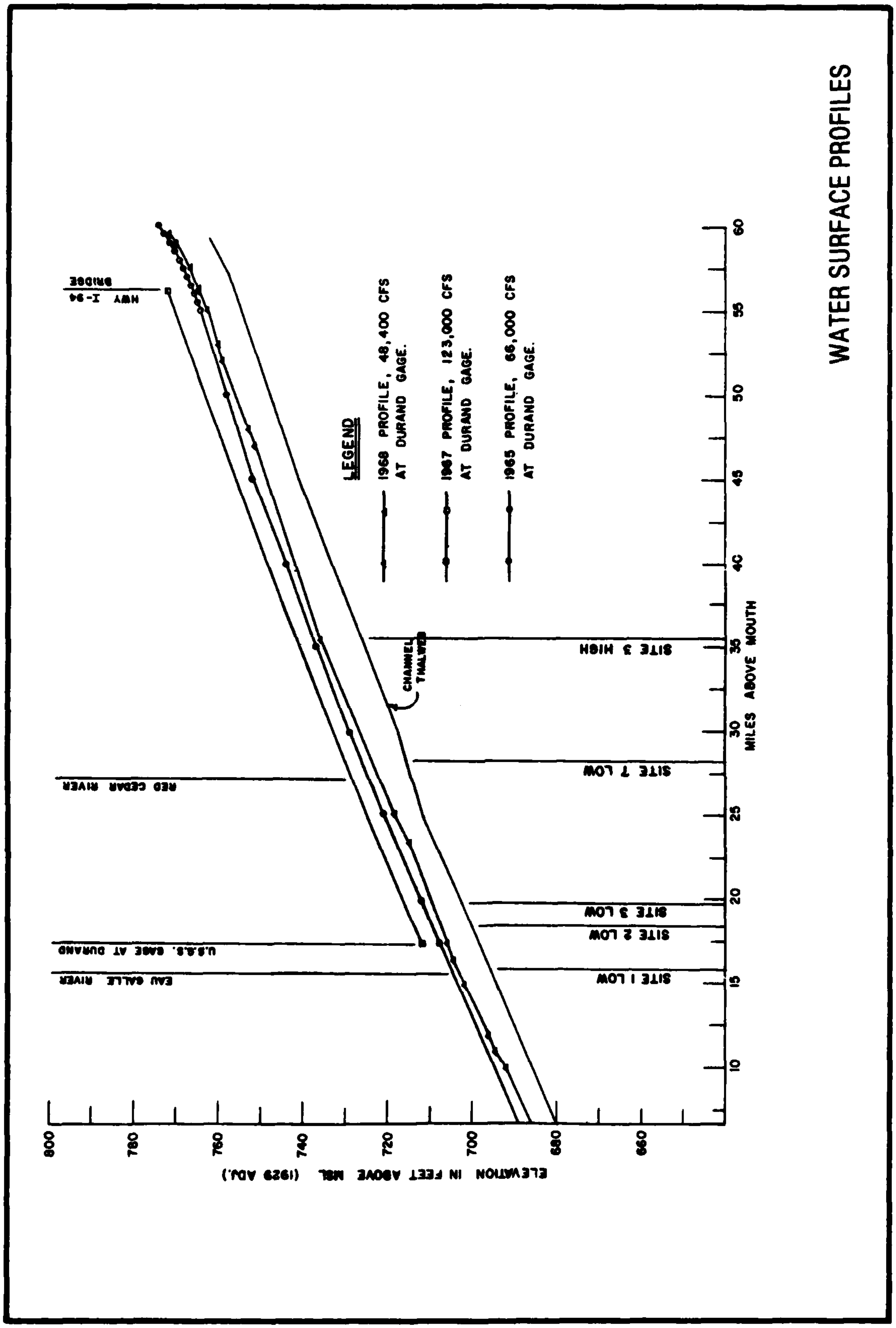
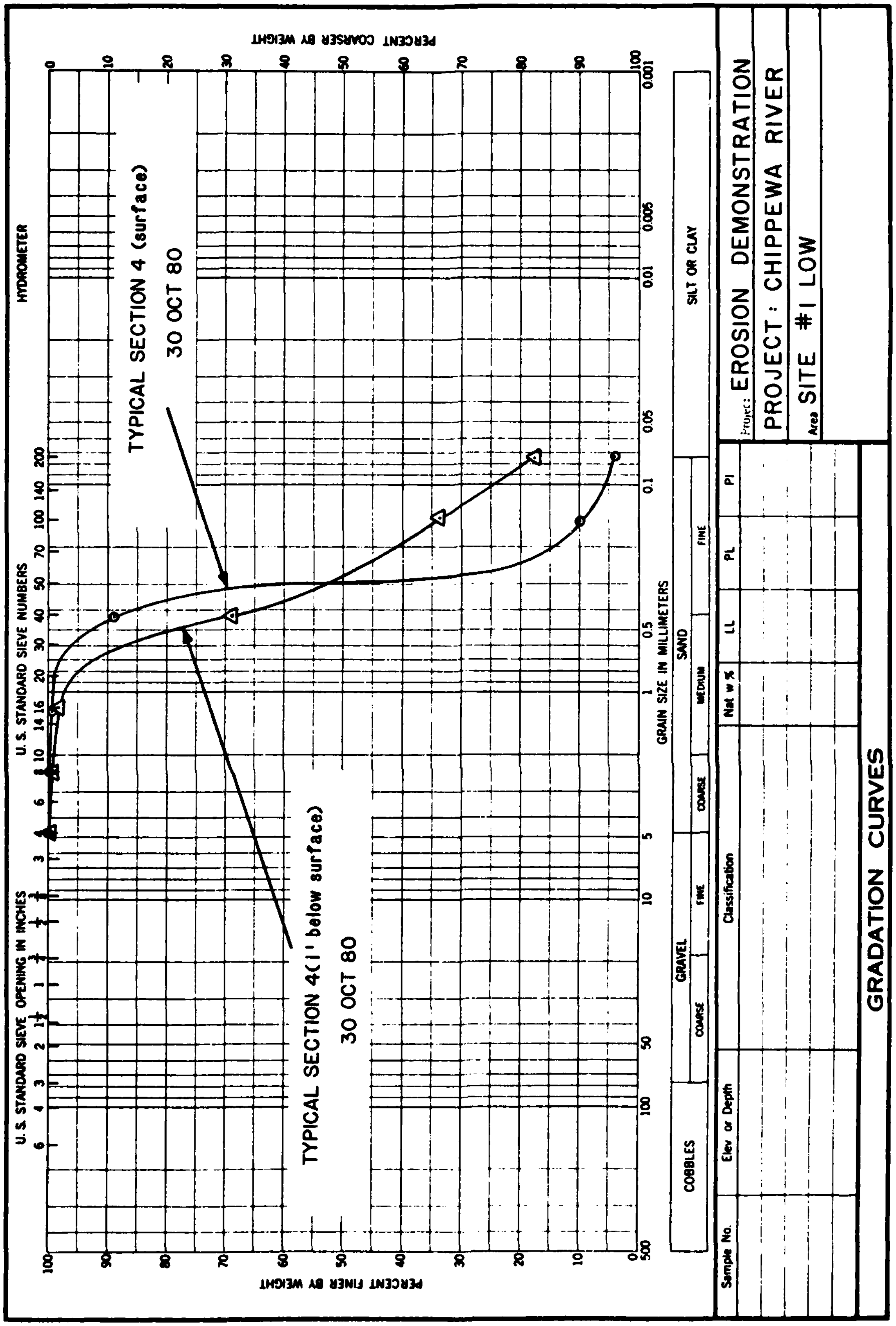
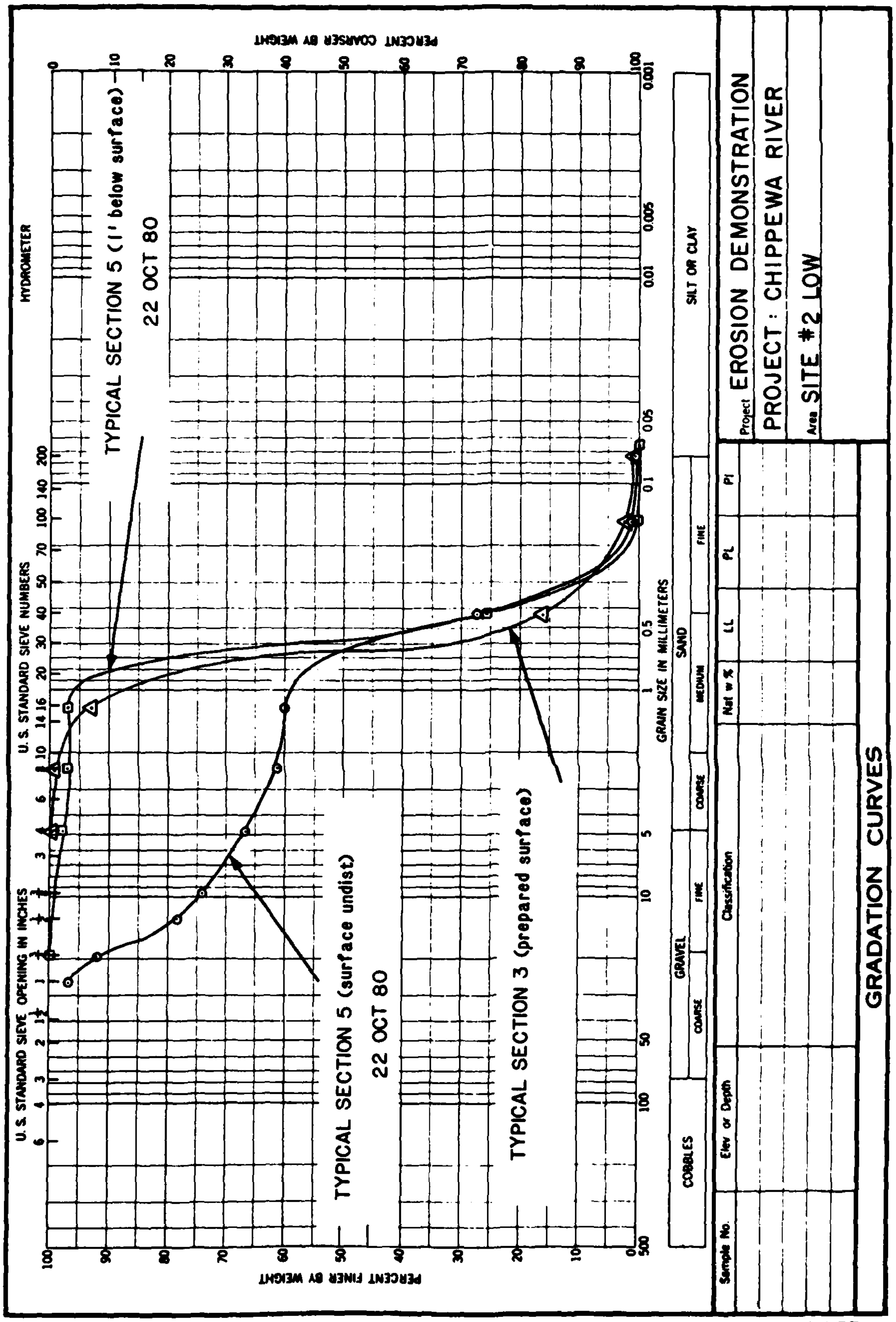

WATER SURFACE PROFILES
LEGEND
1968 PROFILE, 48,400 CFS AT DURAND GAGE.
1967 PROFILE, 123,000 CFS AT DURAND GAGE.
1965 PROFILE, 66,000 CFS AT DURAND GAGE.
CHANNEL THALWEG
HWY I-94 BRIDGE
RED CEDAR RIVER
U.S.G.S. GAGE AT DURAND
EAU GALLE RIVER
SITE 3 HIGH
SITE 7 LOW
SITE 3 LOW
SITE 2 LOW
SITE 1 LOW
MILES ABOVE MOUTH
10
15
20
25
30
35
40
45
50
55
60
ELEVATION IN FEET ABOVE MSL (1929 ADJ.)
660
680
700
720
740
760
780
800

PLATE 8



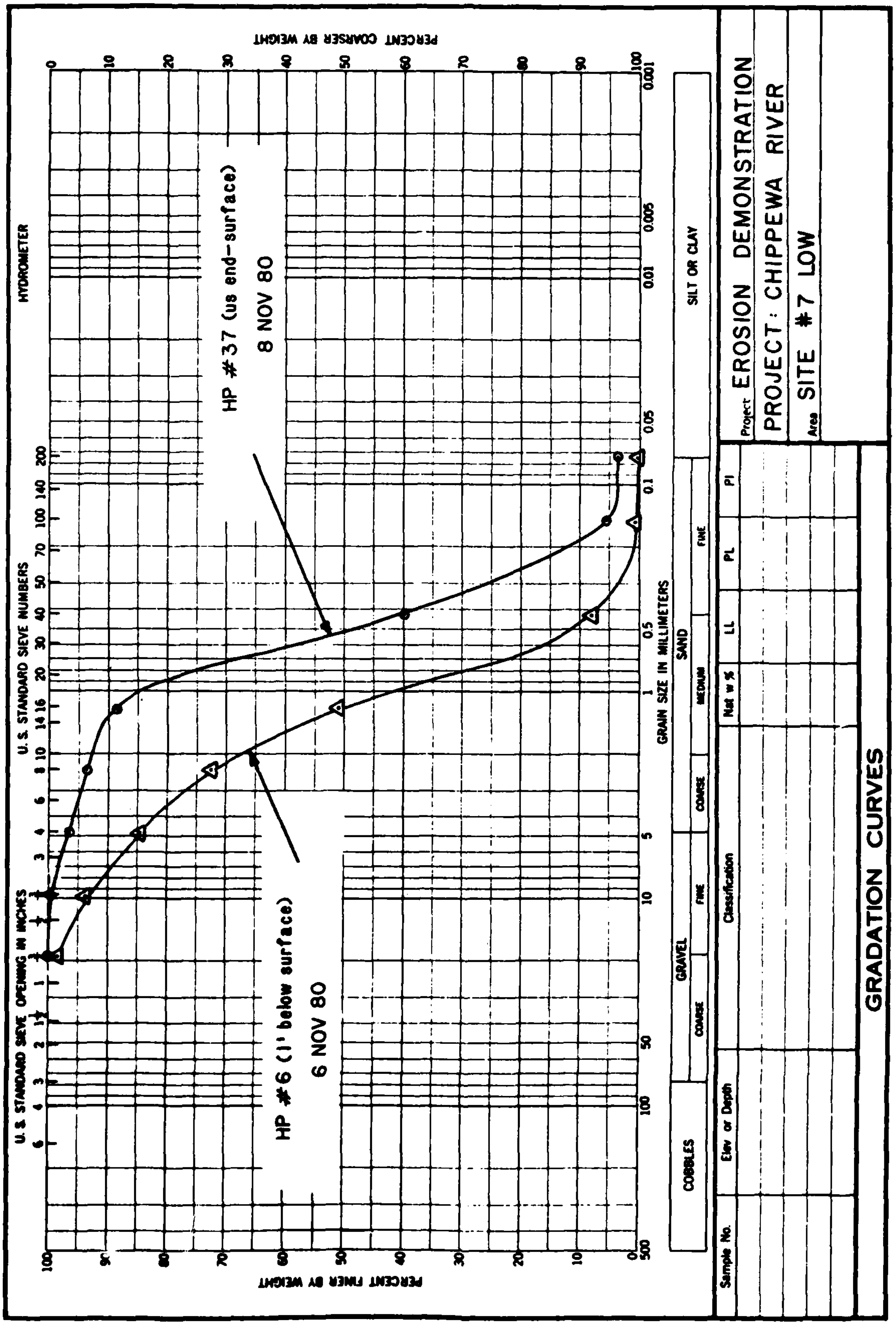
TYPICAL SECTION 4 (surface)
30 OCT 80
TYPICAL SECTION 4(1' below surface)
30 OCT 80
U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project: EROSION DEMONSTRATION
PROJECT : CHIPPEWA RIVER
Area SITE #1 LOW
GRADATION CURVES

PLATE 9



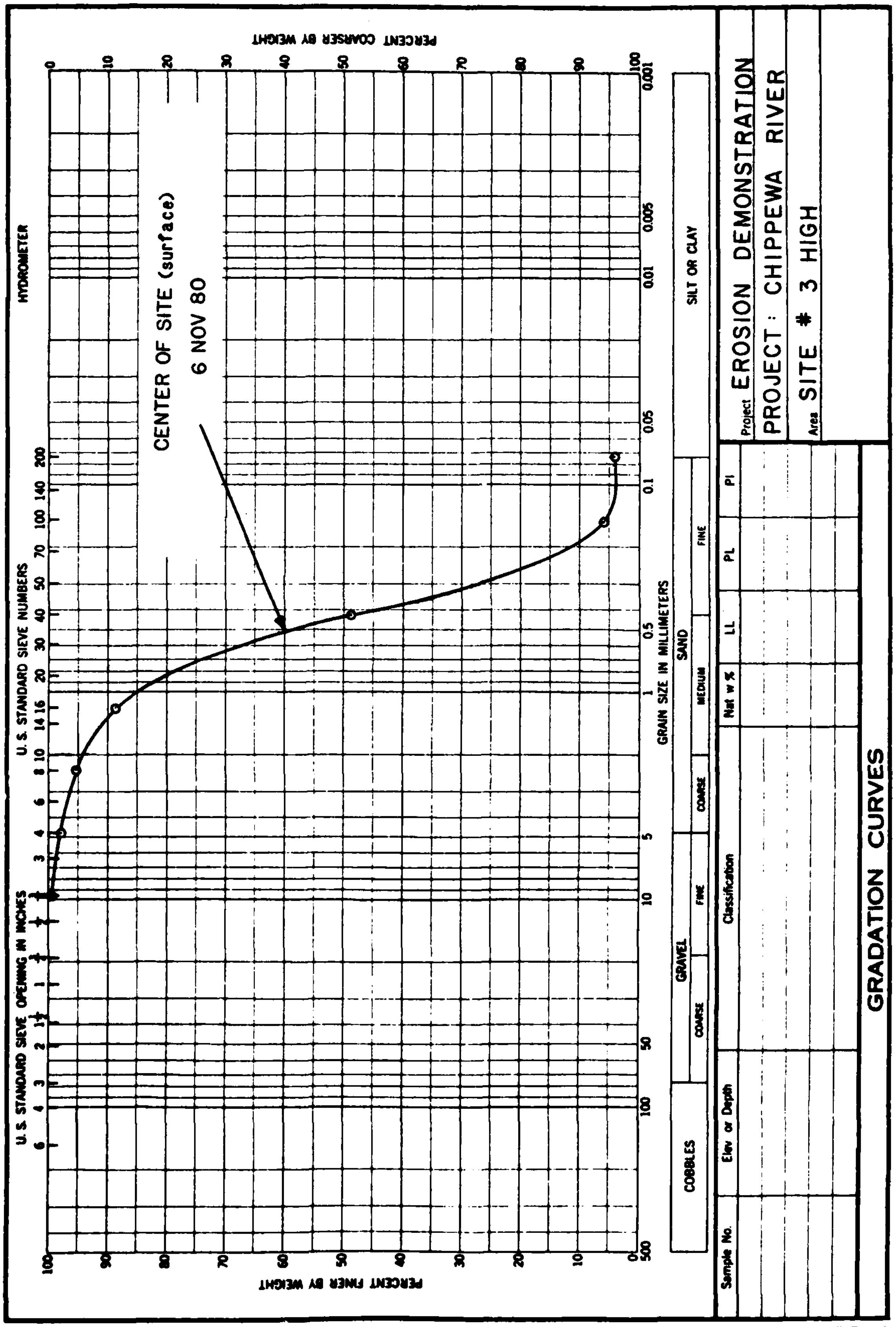
EROSION DEMONSTRATION
PROJECT: CHIPPEWA RIVER
Area SITE #2 LOW
GRADATION CURVES
TYPICAL SECTION 5 (1' below surface)
22 OCT 80
TYPICAL SECTION 5 (surface undist)
22 OCT 80
TYPICAL SECTION 3 (prepared surface)
U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT COARSER BY WEIGHT
PERCENT FINER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI

PLATE 10



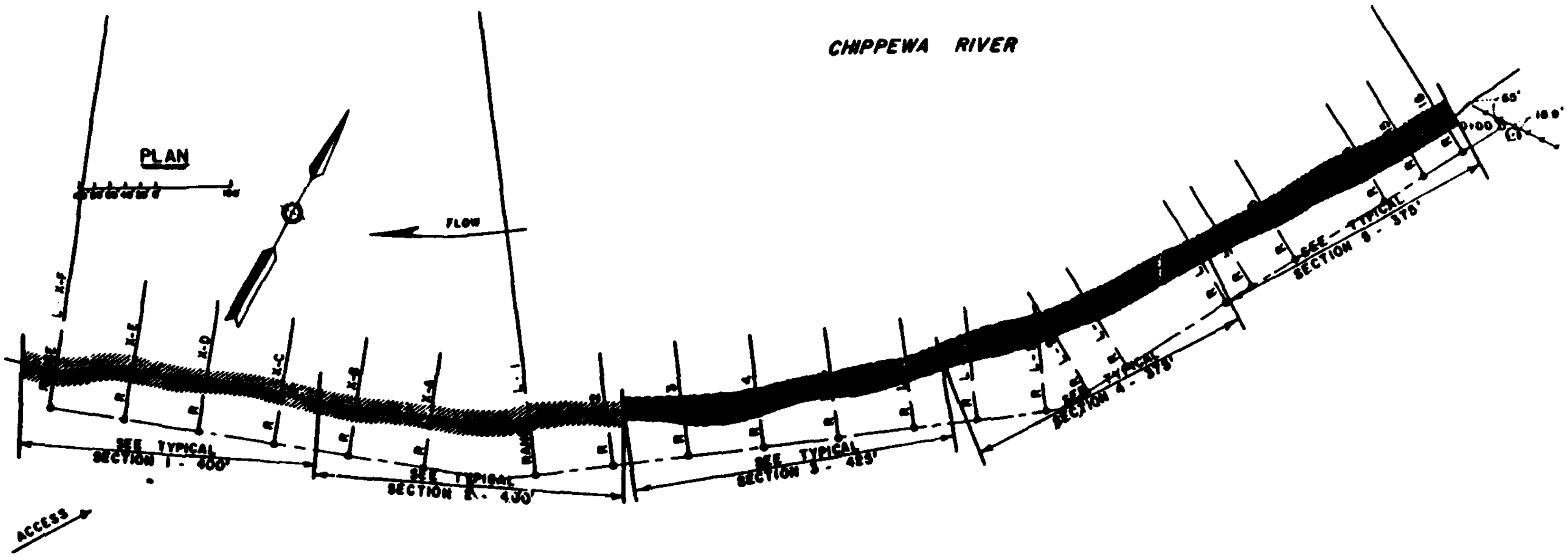
HP #37 (us end-surface)
8 NOV 80
HP #6 (1' below surface)
6 NOV 80
U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
MEDIUM
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project EROSION DEMONSTRATION
PROJECT: CHIPPEWA RIVER
Area SITE #7 LOW
GRADATION CURVES

PLATE 11



U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT COARSER BY WEIGHT
PERCENT FINER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
CENTER OF SITE (surface)
6 NOV 80
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project EROSION DEMONSTRATION
PROJECT : CHIPPEWA RIVER
Area SITE # 3 HIGH
GRADATION CURVES

PLATE 12

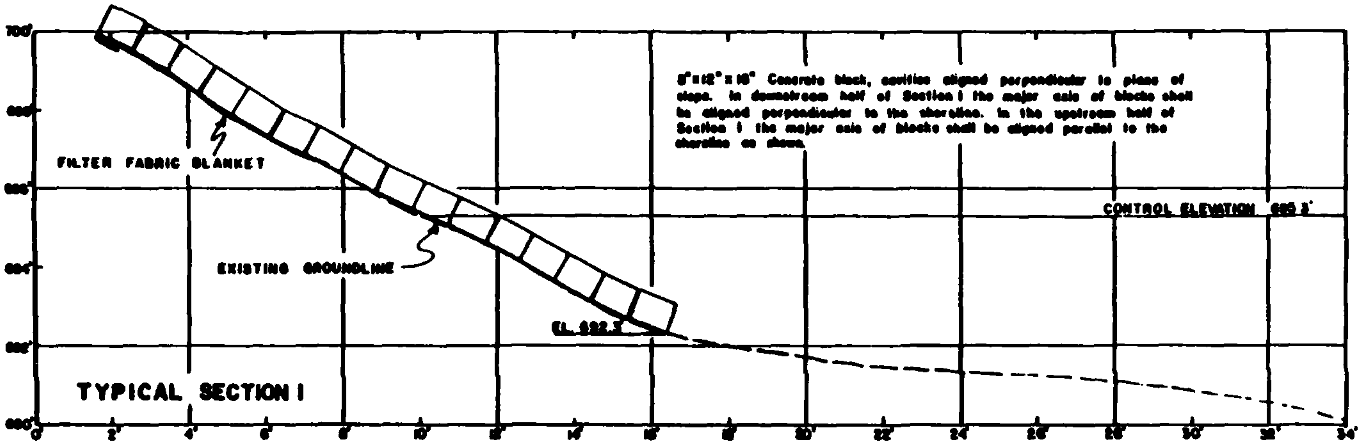


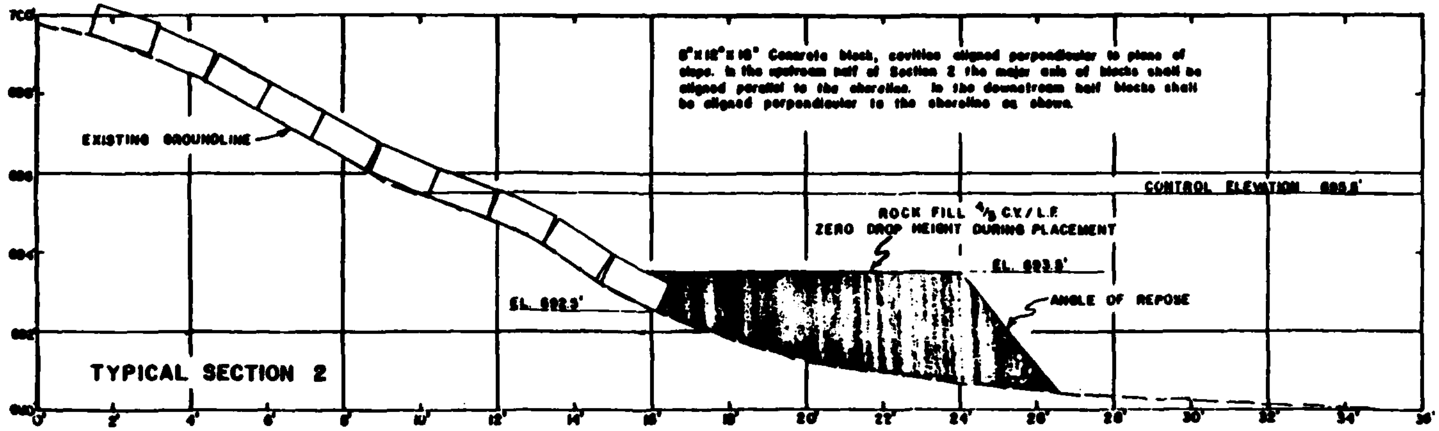
NOTES:

1. CONTROL ELEVATIONS GIVEN ON EACH SECTION ARE W.S. ELEVATIONS AT THE TIME OF SURVEYS. CONSTRUCTION SHALL BE CONTROLLED BY GIVEN ELEVATIONS AND SHALL BE INDEPENDENT OF WS AT THE TIME OF CONSTRUCTION
2. ABOVE ICE ELEVATION OF CONCRETE BLOCKS, GRADE WHERE NECESSARY TO INSURE A SLOPE NO GREATER THAN 1V ON 1H.
3. FOR 15' EITHER SIDE OF THE INTERSECTIONS OF SECTIONS 2 AND 3, 3 AND 4, 4 AND 5, 24" OF ROCK FILL SHALL BE PLACED WITH FILLING OR EXCAVATION AS NEEDED TO SMOOTHLY TRANSITION BETWEEN PROTECTION TYPES.
4. BLOCK CAVITIES TO BE FILLED ABOVE WATER SURFACE AFTER PLACEMENT

SITE 1 LOW
PLAN

PLATE 13





SITE 1 LOW
SECTION 1 AND 2

PLATE 14

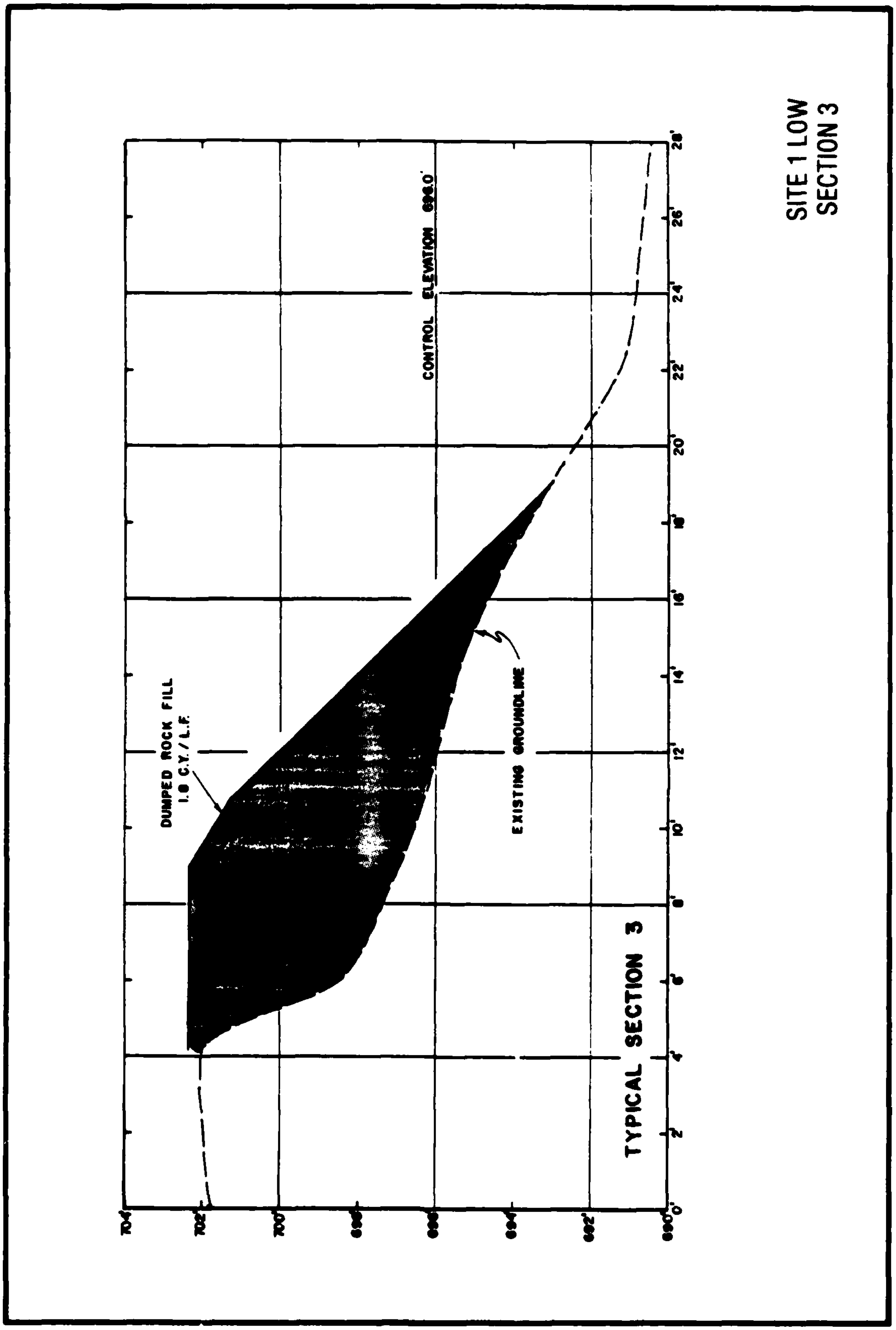
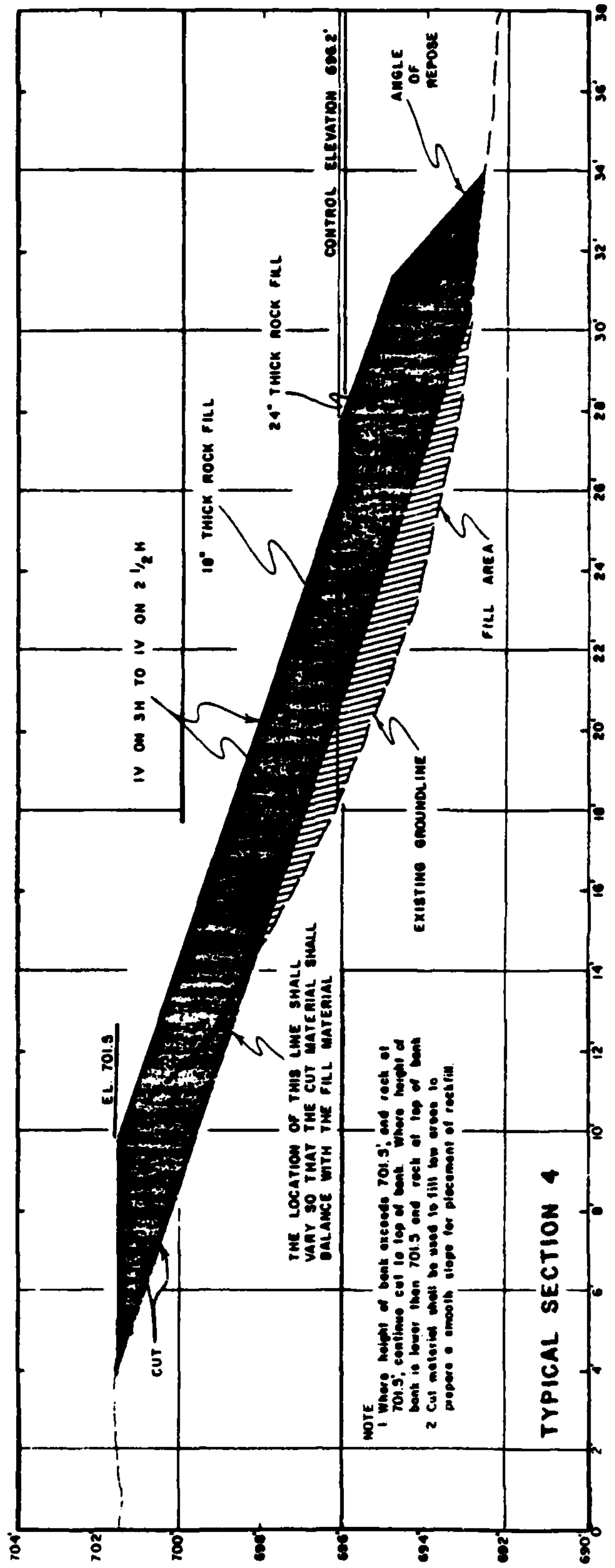

SITE 1 LOW
SECTION 3
TYPICAL SECTION 3
CONTROL ELEVATION 696.0'
DUMPED ROCK FILL
EXISTING GROUNDLINE
704
702
700
698
696
694
692
690
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28

PLATE 15



SITE 1 LOW
SECTION 4

PLATE 16

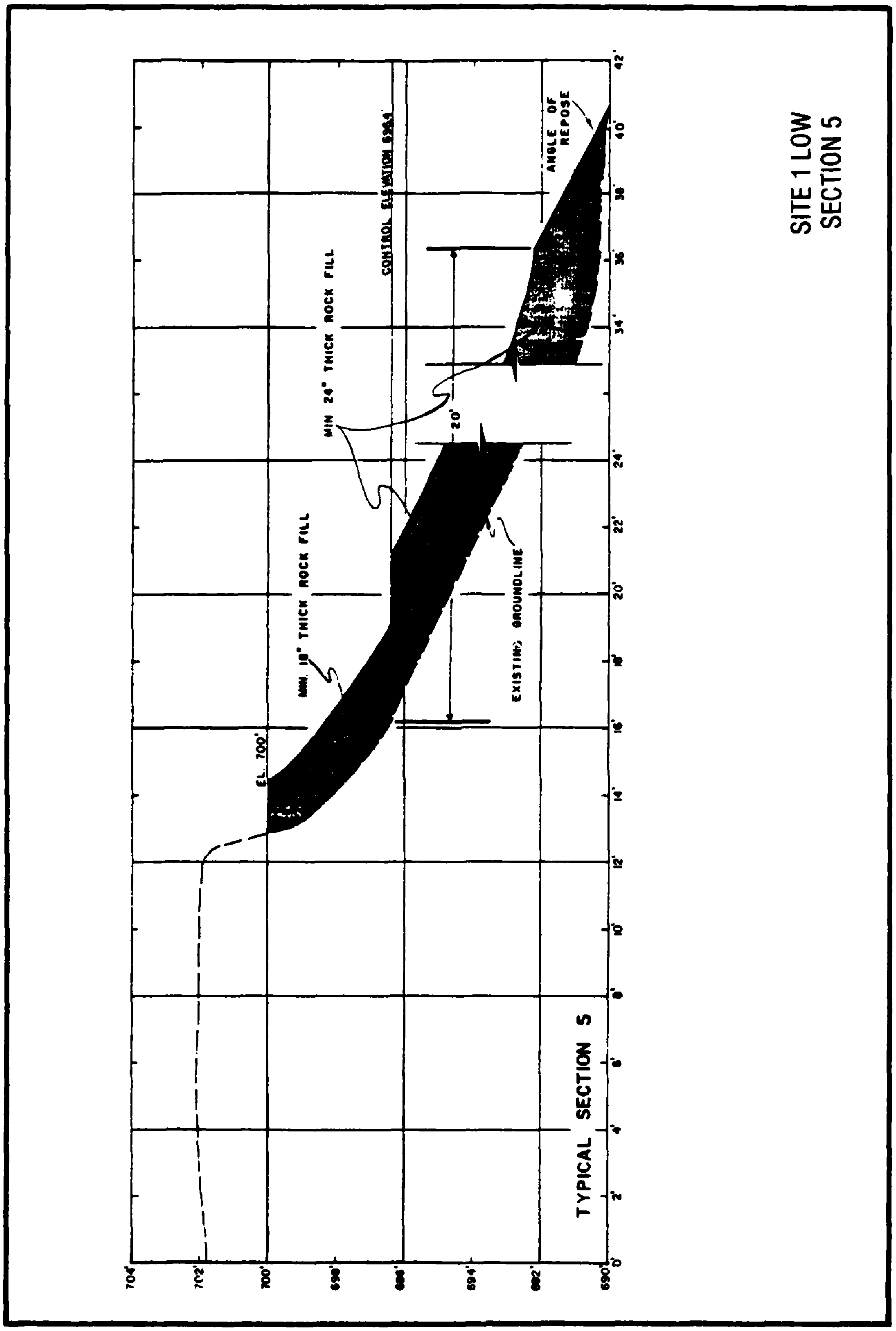
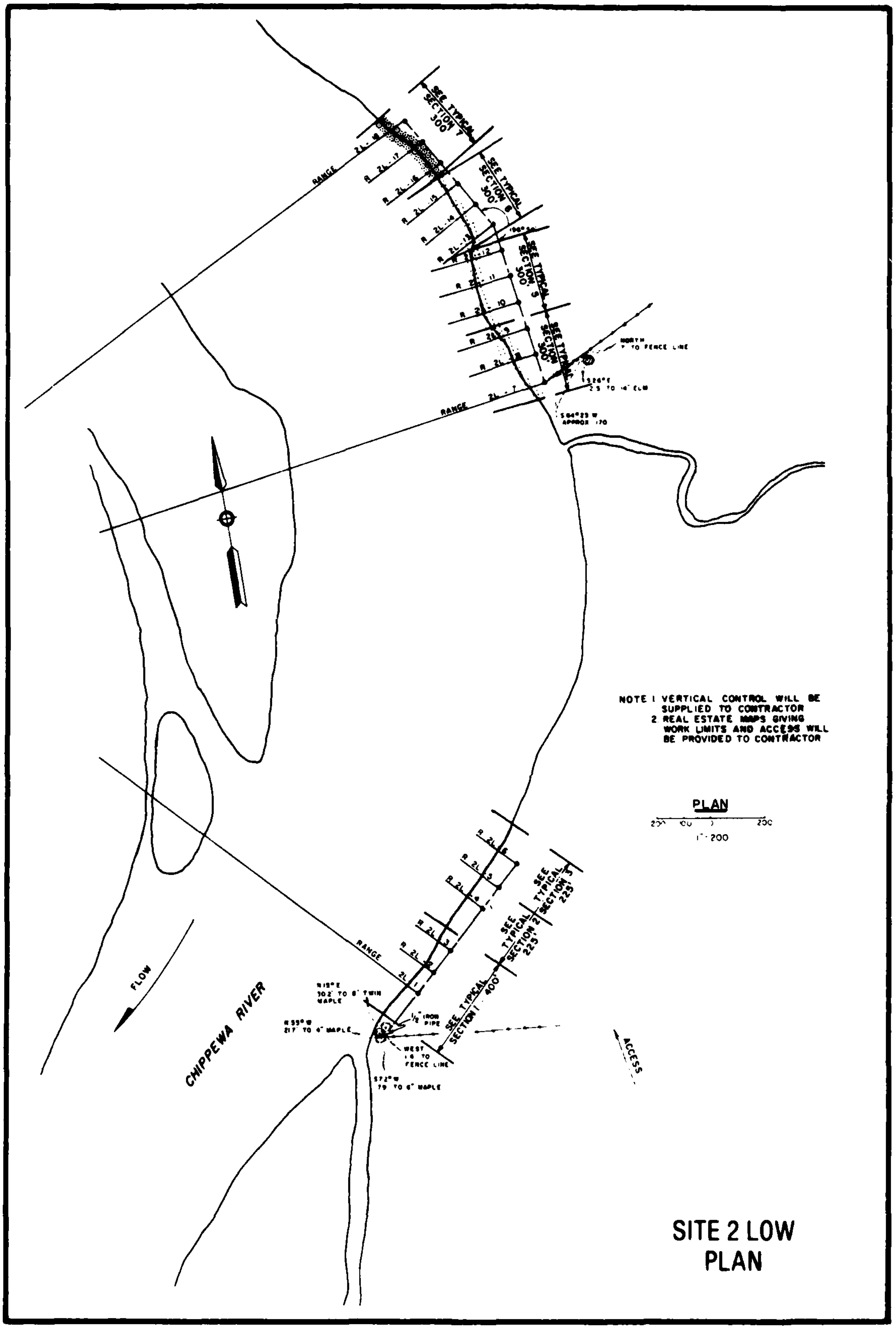

SITE 1 LOW
SECTION 5
TYPICAL SECTION 5
CONTROL ELEVATION
MIN 24" THICK ROCK FILL
MIN. 18" THICK ROCK FILL
EXISTING GROUNDLINE
ANGLE OF REPOSE
EL. 700'
20'
704'
702'
700'
698'
696'
694'
692'
690'
0'
2'
4'
6'
8'
10'
12'
14'
16'
18'
20'
22'
24'
34'
36'
38'
40'
42'

PLATE 17



SITE 2 LOW PLAN

PLATE 18

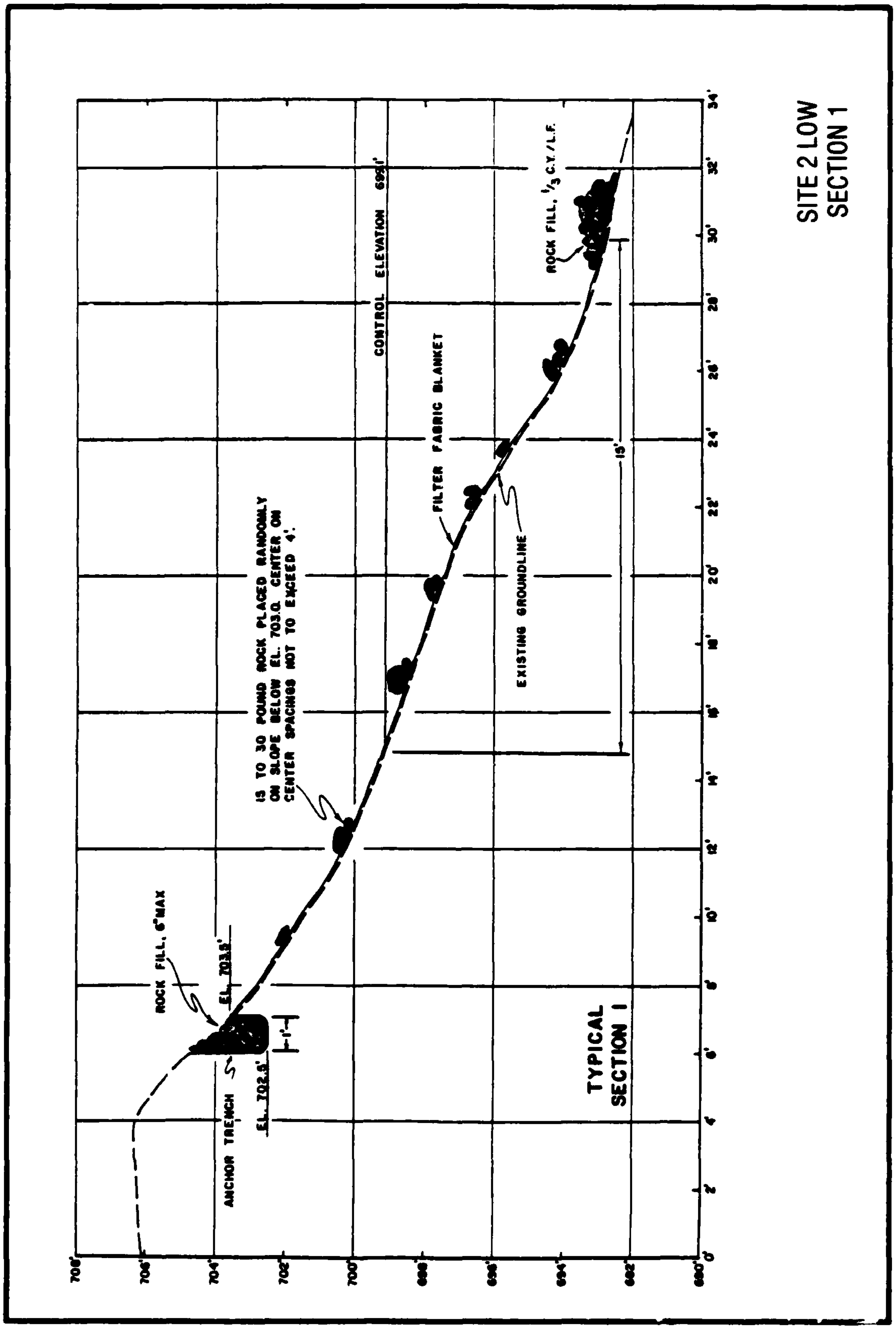
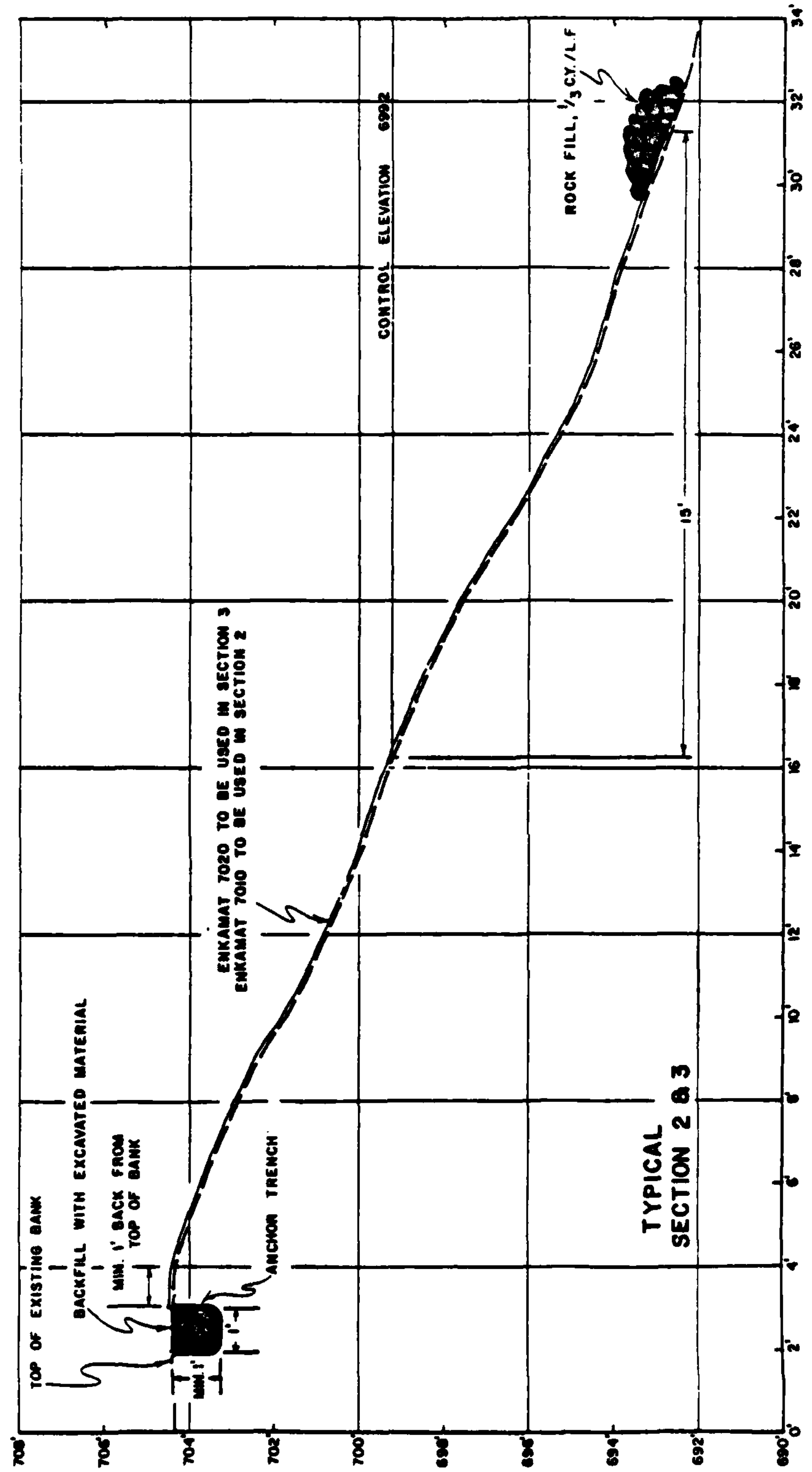

SITE 2 LOW
SECTION 1
TYPICAL
SECTION I
ROCK FILL, 1/3 C.Y./L.F.
CONTROL ELEVATION
FILTER FABRIC BLANKET
EXISTING GROUNDLINE
15 TO 30 POUND ROCK PLACED RANDOMLY
ON SLOPE BELOW EL. 703.0. CENTER ON
CENTER SPACINGS NOT TO EXCEED 4'.
ROCK FILL, 6" MAX
EL. 703.5'
EL. 702.5'
ANCHOR TRENCH
15'
1'
706'
704'
702'
700'
698'
696'
694'
692'
690'
0'
2'
4'
6'
8'
10'
12'
14'
16'
18'
20'
22'
24'
26'
28'
30'
32'
34'

PLATE 19



SITE 2 LOW
SECTIONS 2 AND 3

PLATE 20

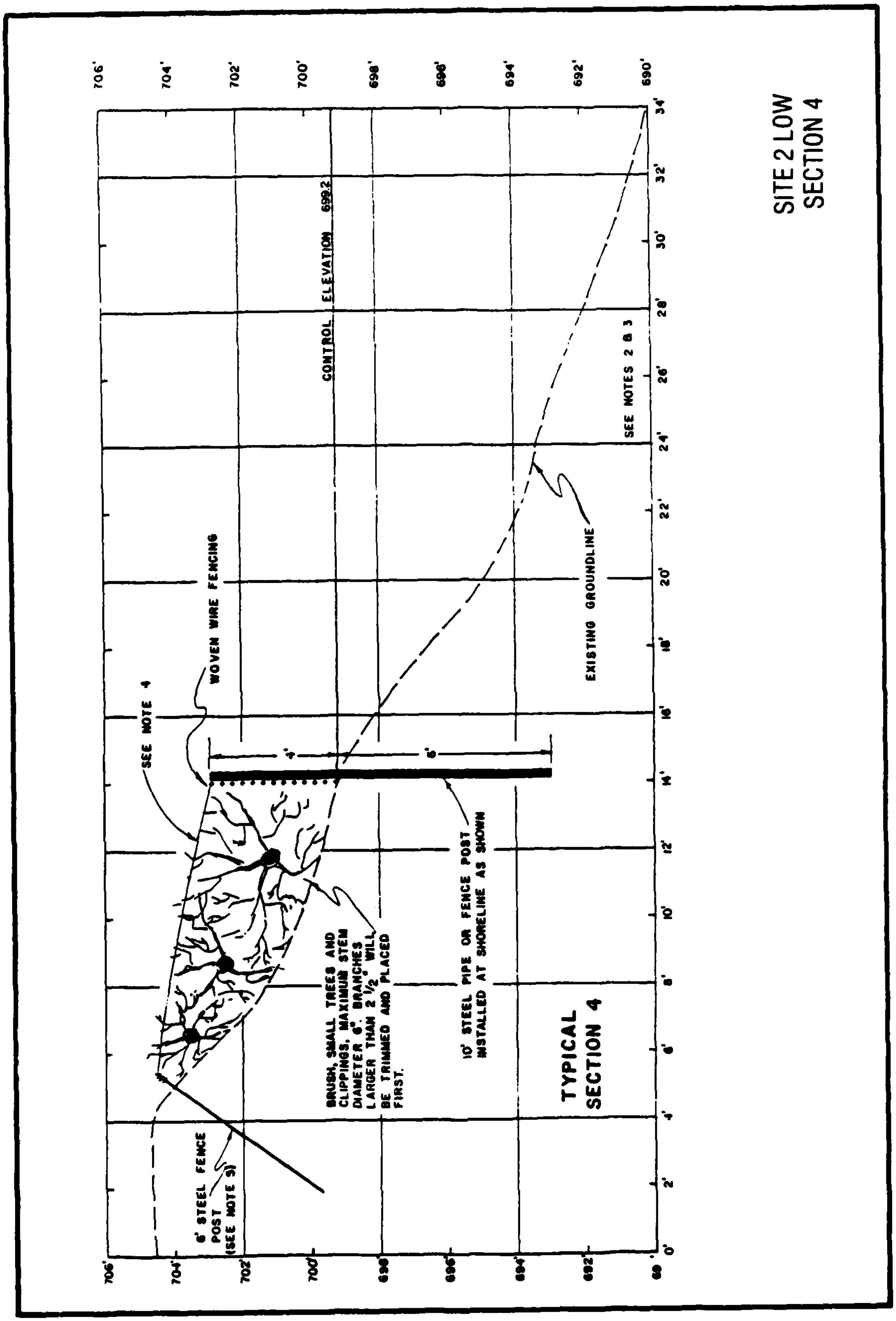

SITE 2 LOW
SECTION 4
TYPICAL
SECTION 4
CONTROL ELEVATION 699.2
SEE NOTES 2 & 3
EXISTING GROUNDLINE
WOVEN WIRE FENCING
SEE NOTE 4
4'
6'
10' STEEL PIPE OR FENCE POST INSTALLED AT SHORELINE AS SHOWN
BRUSH, SMALL TREES AND CLIPPINGS, MAXIMUM STEM DIAMETER 6". BRANCHES LARGER THAN 2 1/2" WILL BE TRIMMED AND PLACED FIRST.
6' STEEL FENCE POST (SEE NOTE 5)

PLATE 21

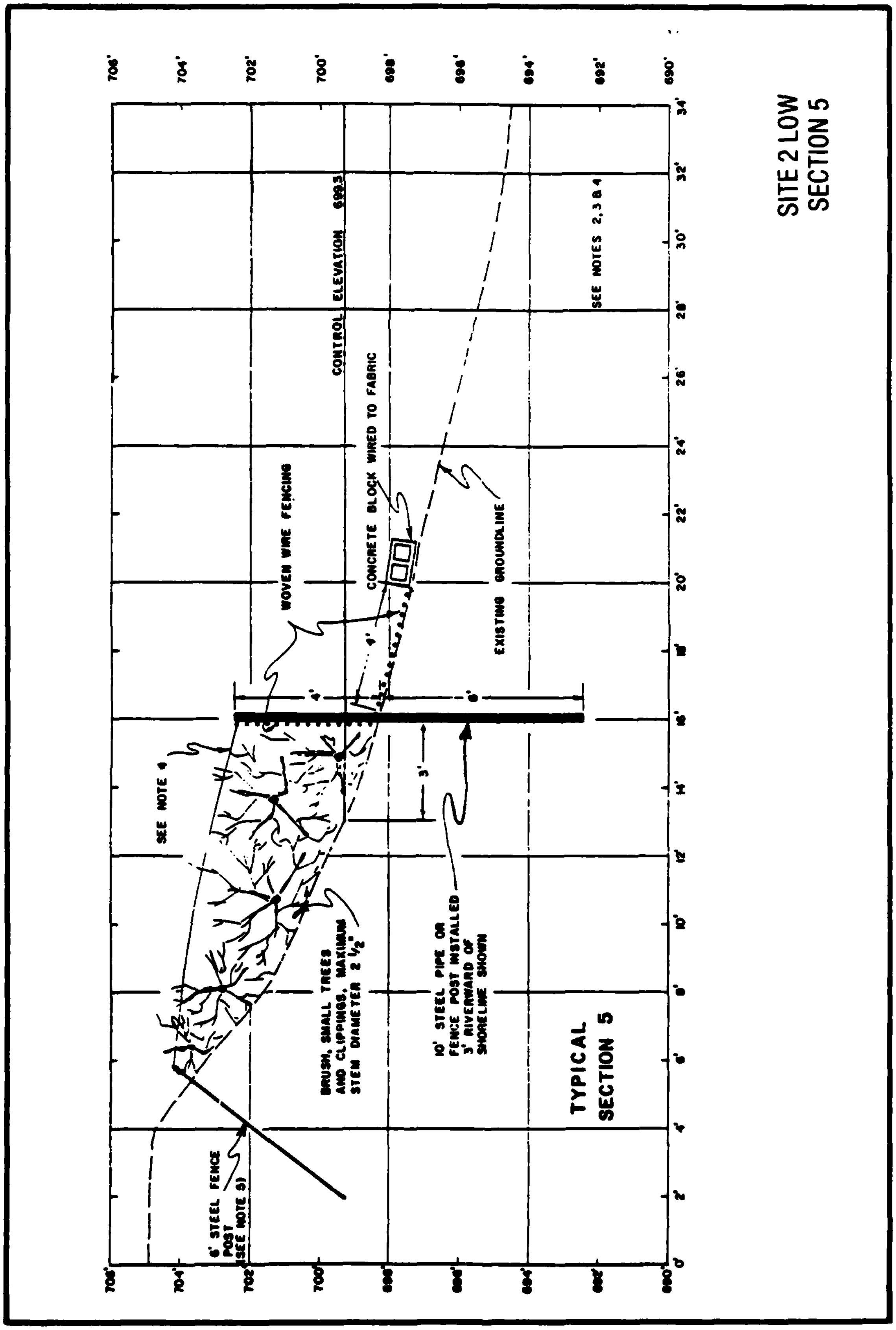

SITE 2 LOW
SECTION 5
TYPICAL
SECTION 5
CONTROL ELEVATION 699.3
SEE NOTES 2, 3 & 4
WOVEN WIRE FENCING
CONCRETE BLOCK WIRED TO FABRIC
EXISTING GROUNDLINE
SEE NOTE 4
BRUSH, SMALL TREES
AND CLIPPINGS, MAXIMUM
STEM DIAMETER 2 1/2"
10' STEEL PIPE OR
FENCE POST INSTALLED
3' RIVERWARD OF
SHORELINE SHOWN
6' STEEL FENCE
POST
(SEE NOTE 5)
3'

PLATE 22

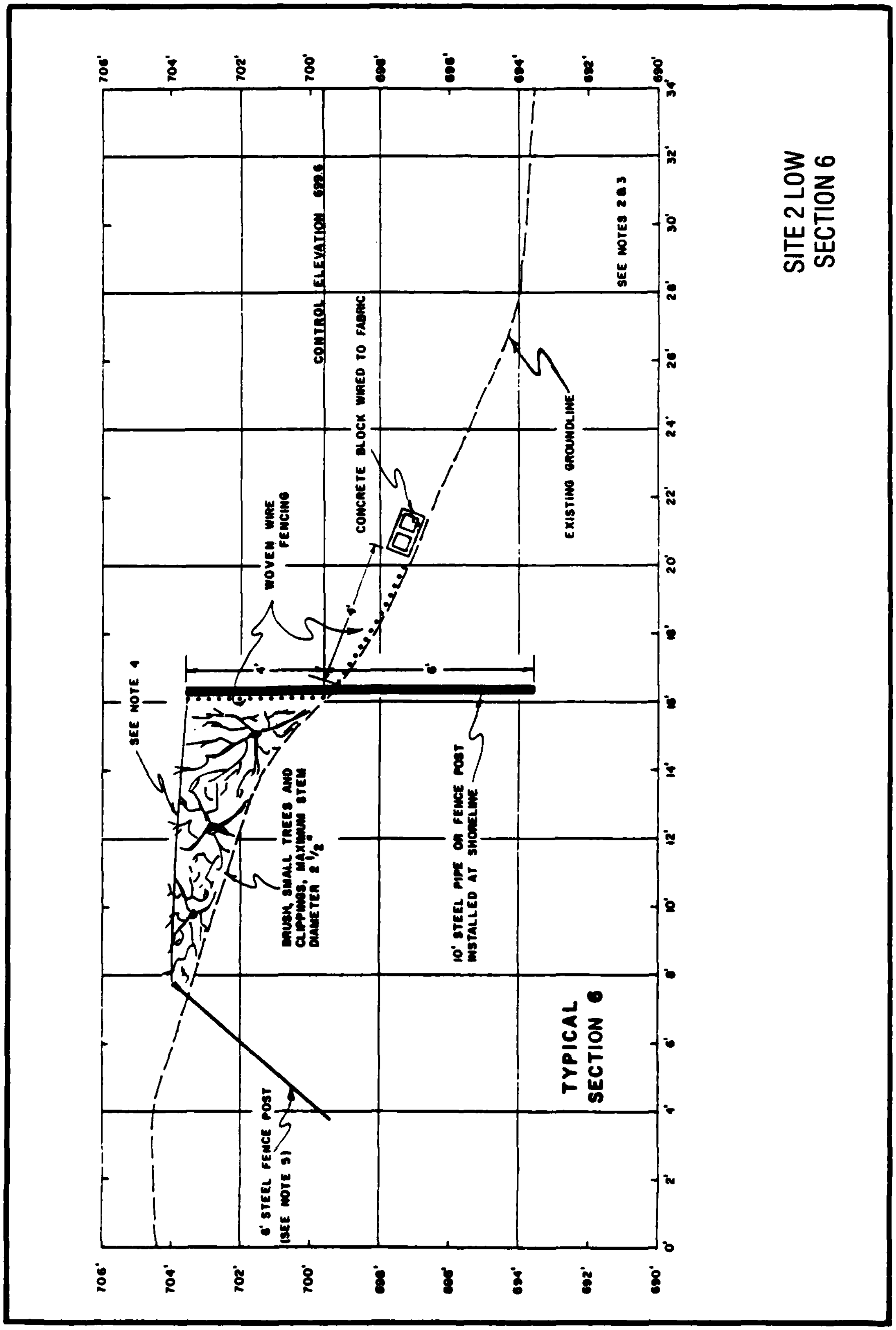
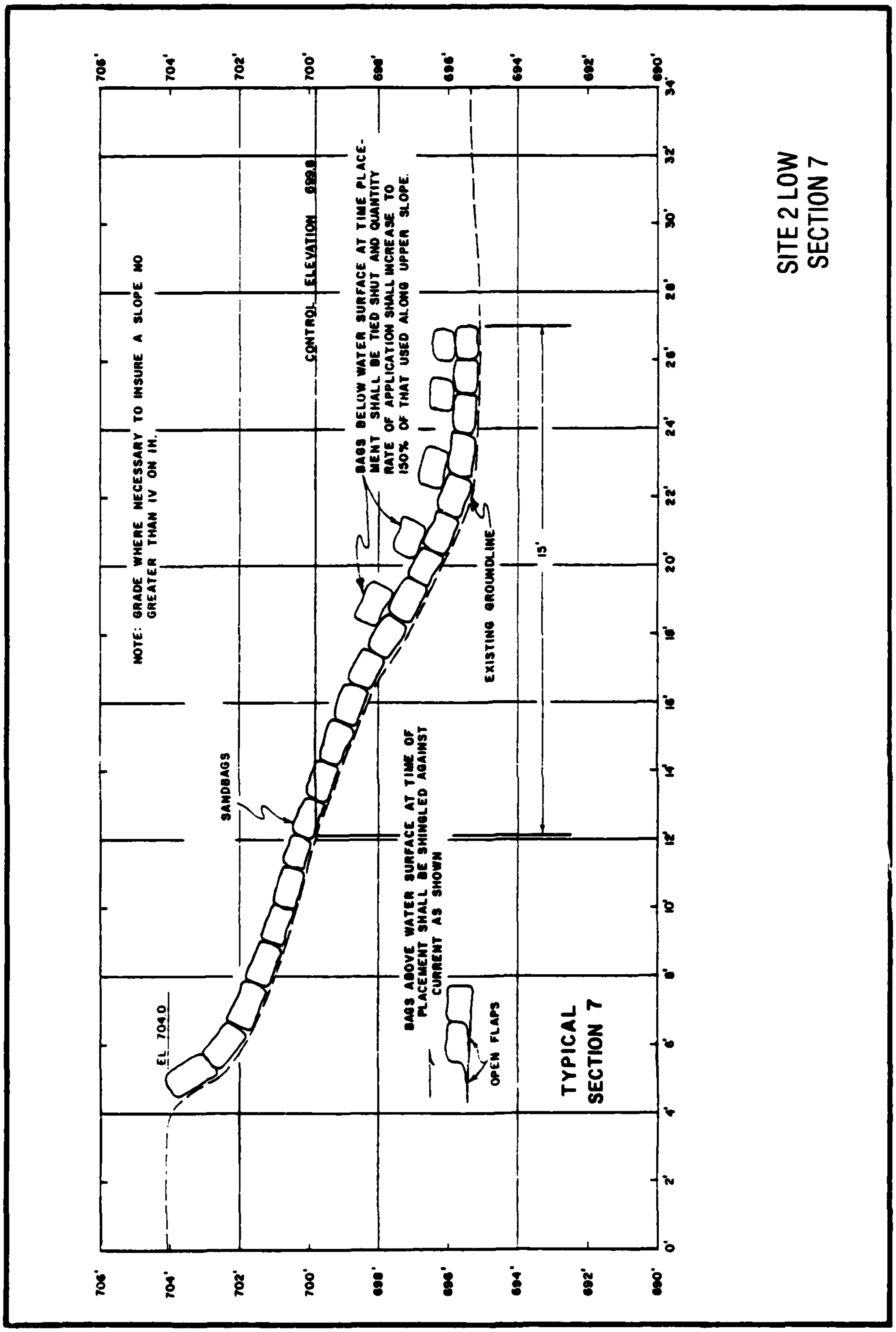
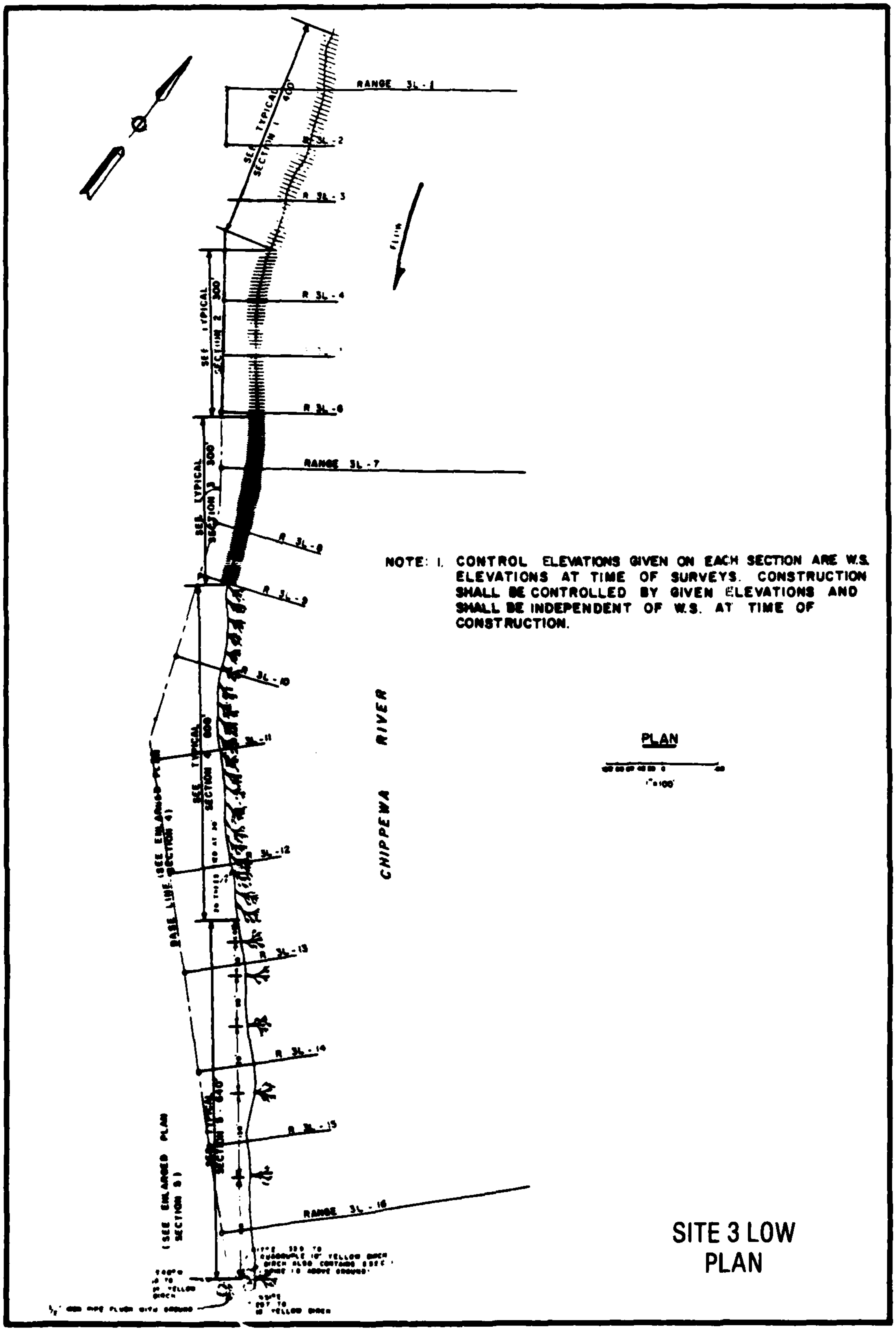

SITE 2 LOW
SECTION 6
TYPICAL
SECTION 6
CONTROL ELEVATION 699.5
SEE NOTES 2 & 3
SEE NOTE 4
CONCRETE BLOCK WIRED TO FABRIC
WOVEN WIRE
FENCING
EXISTING GROUNDLINE
BRUSH, SMALL TREES AND
CLIPPINGS, MAXIMUM STEM
DIAMETER 2 1/2"
10' STEEL PIPE OR FENCE POST
INSTALLED AT SHORELINE
6' STEEL FENCE POST
(SEE NOTE 5)
4'
6'
4'
706'
704'
702'
700'
698'
696'
694'
692'
690'
0'
2'
4'
6'
8'
10'
12'
14'
16'
18'
20'
22'
24'
26'
28'
30'
32'
34'

PLATE 23



706'
704'
702'
700'
698'
696'
694'
692'
690'
0'
2'
4'
6'
8'
10'
12'
14'
16'
18'
20'
22'
24'
26'
28'
30'
32'
34'
NOTE: GRADE WHERE NECESSARY TO INSURE A SLOPE NO GREATER THAN 1V ON 1H.
CONTROL ELEVATION 699.8
BAGS BELOW WATER SURFACE AT TIME PLACEMENT SHALL BE TIED SHUT AND QUANTITY RATE OF APPLICATION SHALL INCREASE TO 150% OF THAT USED ALONG UPPER SLOPE.
SANDBAGS
EXISTING GROUNDLINE
15'
EL 704.0
BAGS ABOVE WATER SURFACE AT TIME OF PLACEMENT SHALL BE SHINGLED AGAINST CURRENT AS SHOWN
OPEN FLAPS
TYPICAL SECTION 7
SITE 2 LOW
SECTION 7

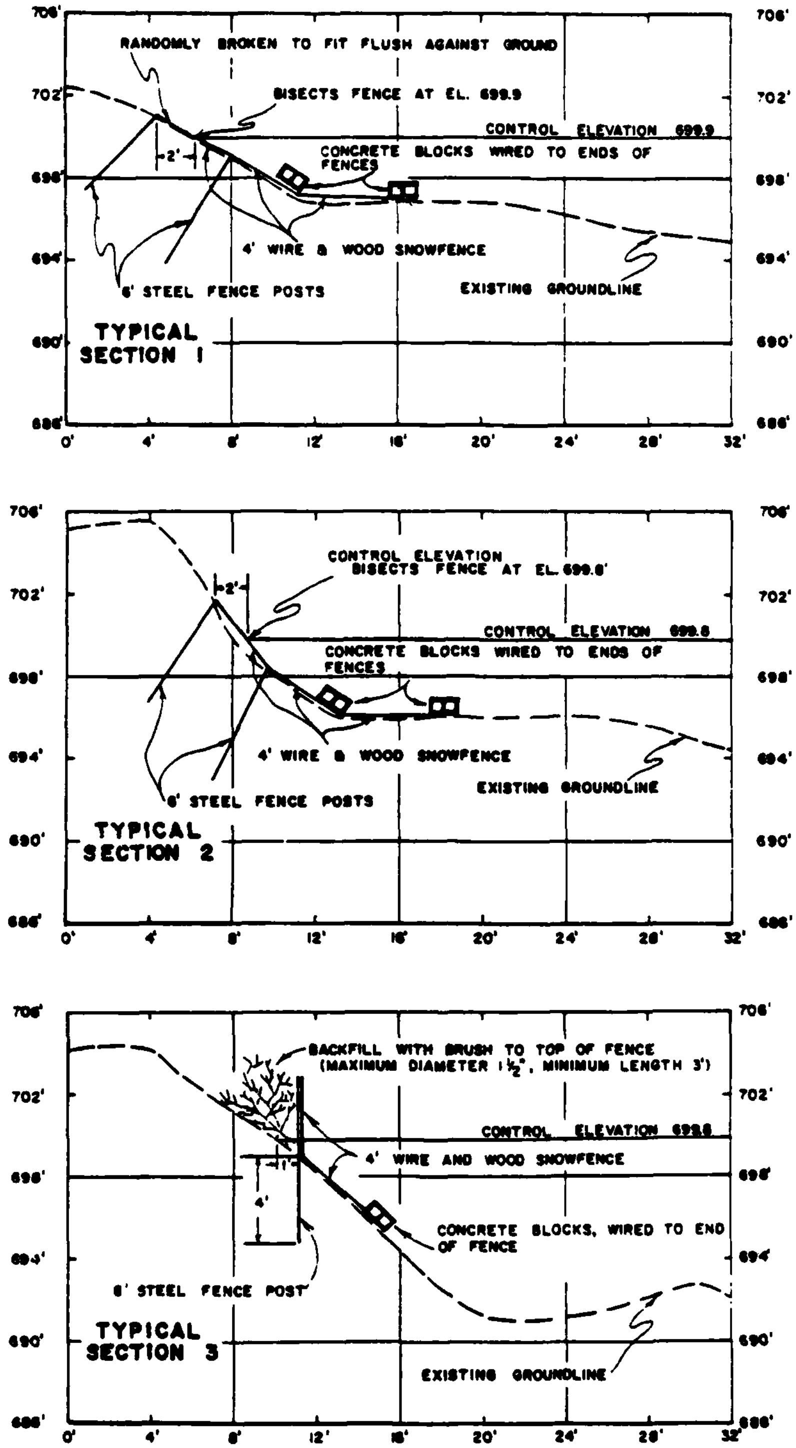
PLATE 24



NOTE: 1. CONTROL ELEVATIONS GIVEN ON EACH SECTION ARE W.S. ELEVATIONS AT TIME OF SURVEYS. CONSTRUCTION SHALL BE CONTROLLED BY GIVEN ELEVATIONS AND SHALL BE INDEPENDENT OF W.S. AT TIME OF CONSTRUCTION.

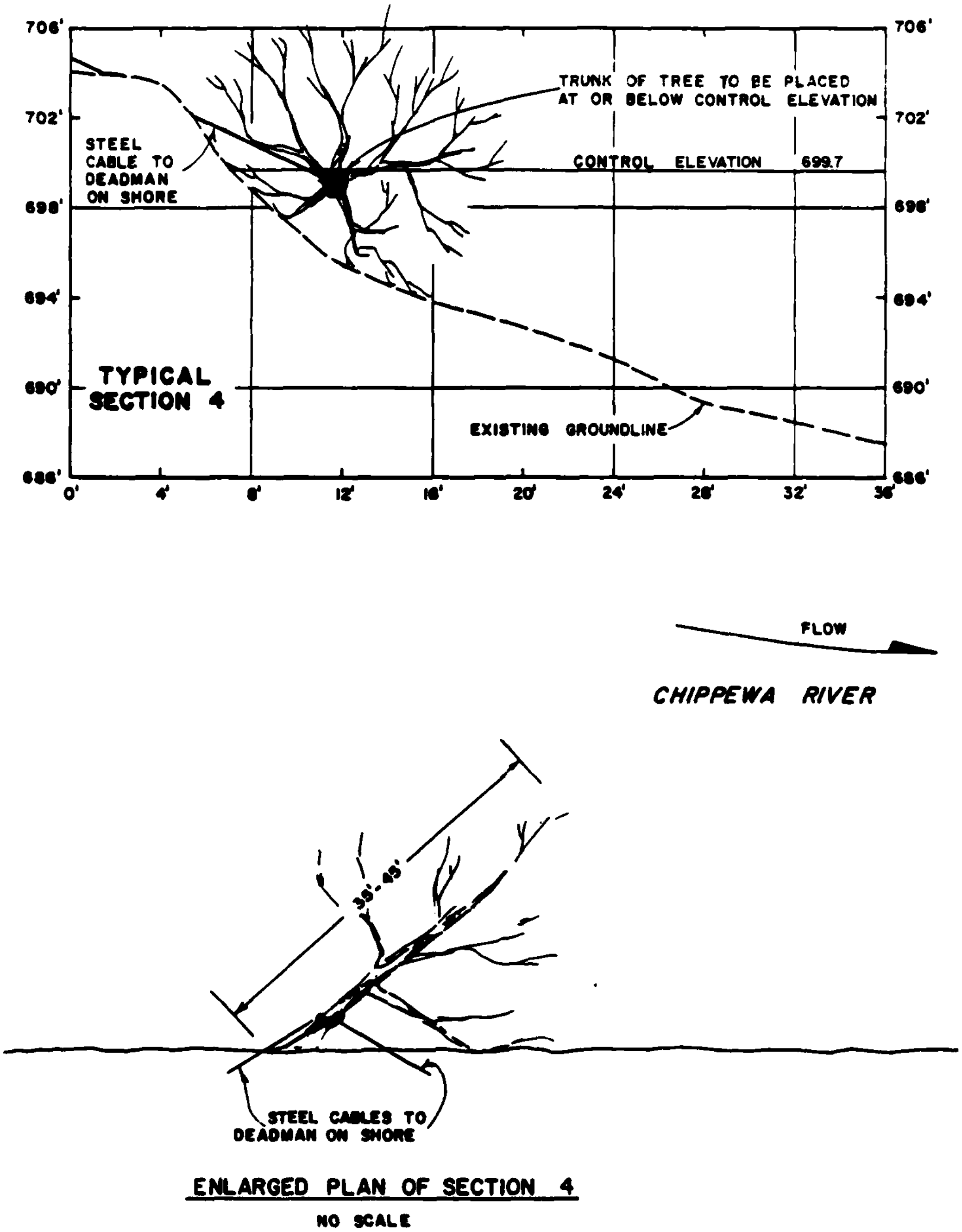
SITE 3 LOW
PLAN

PLATE 25



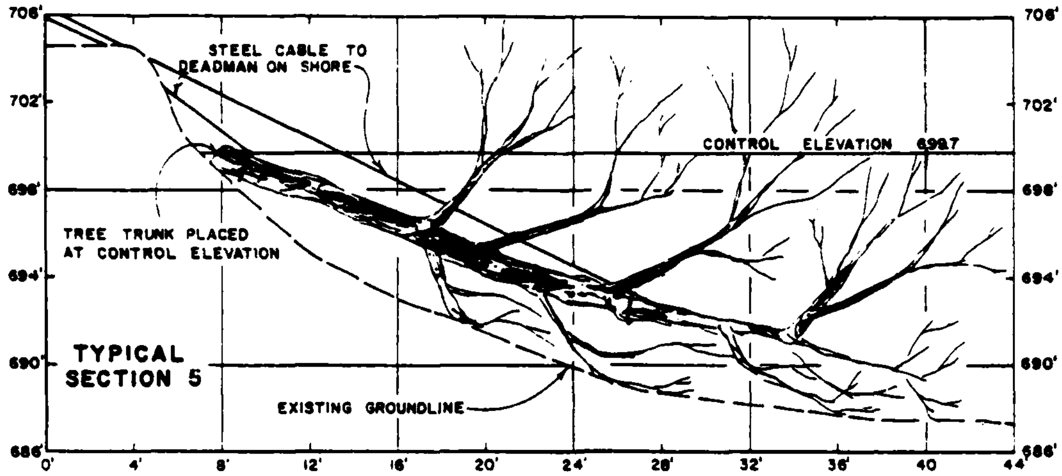
SITE 3 LOW
SECTIONS 1, 2, AND 3

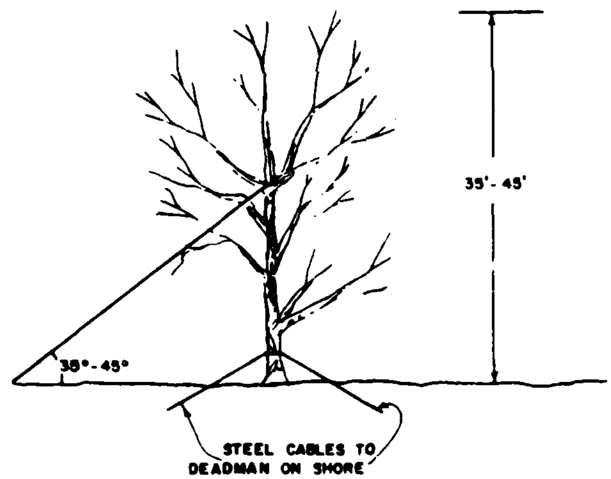
PLATE 26



SITE 3 LOW
SECTION 4

PLATE 27





ENLARGED PLAN OF SECTION 5

NO SCALE

SITE 3 LOW
SECTION 5

PLATE 28

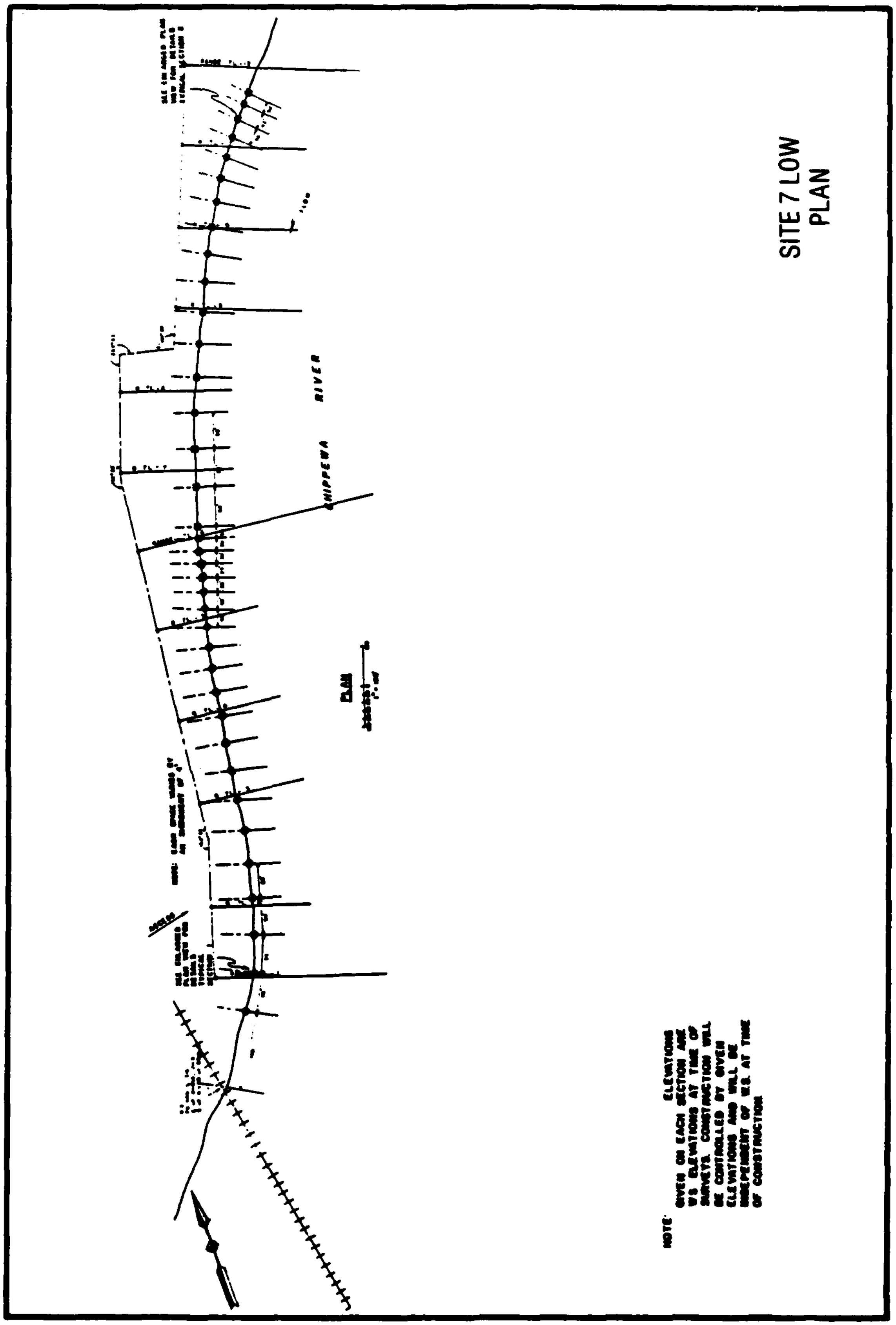
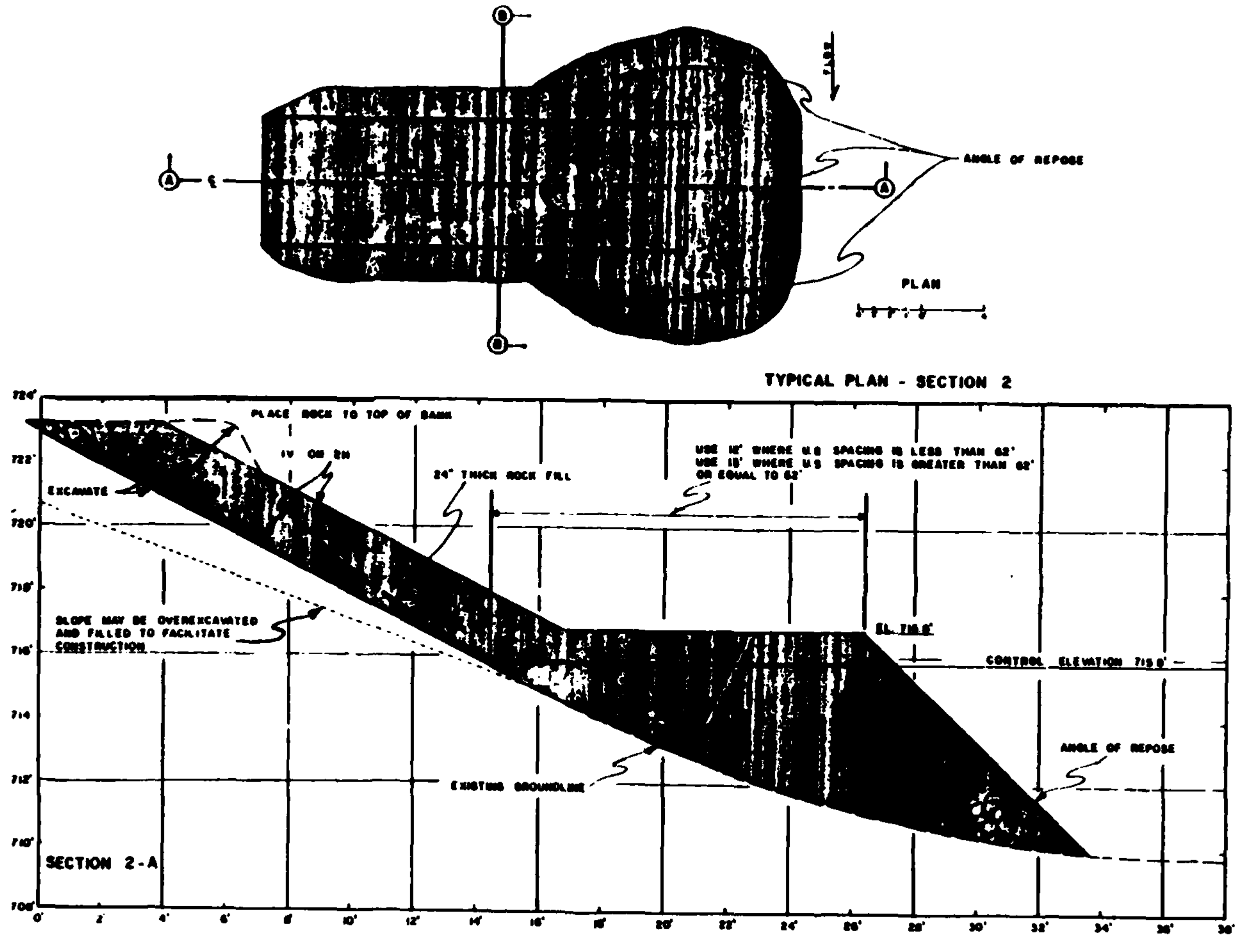
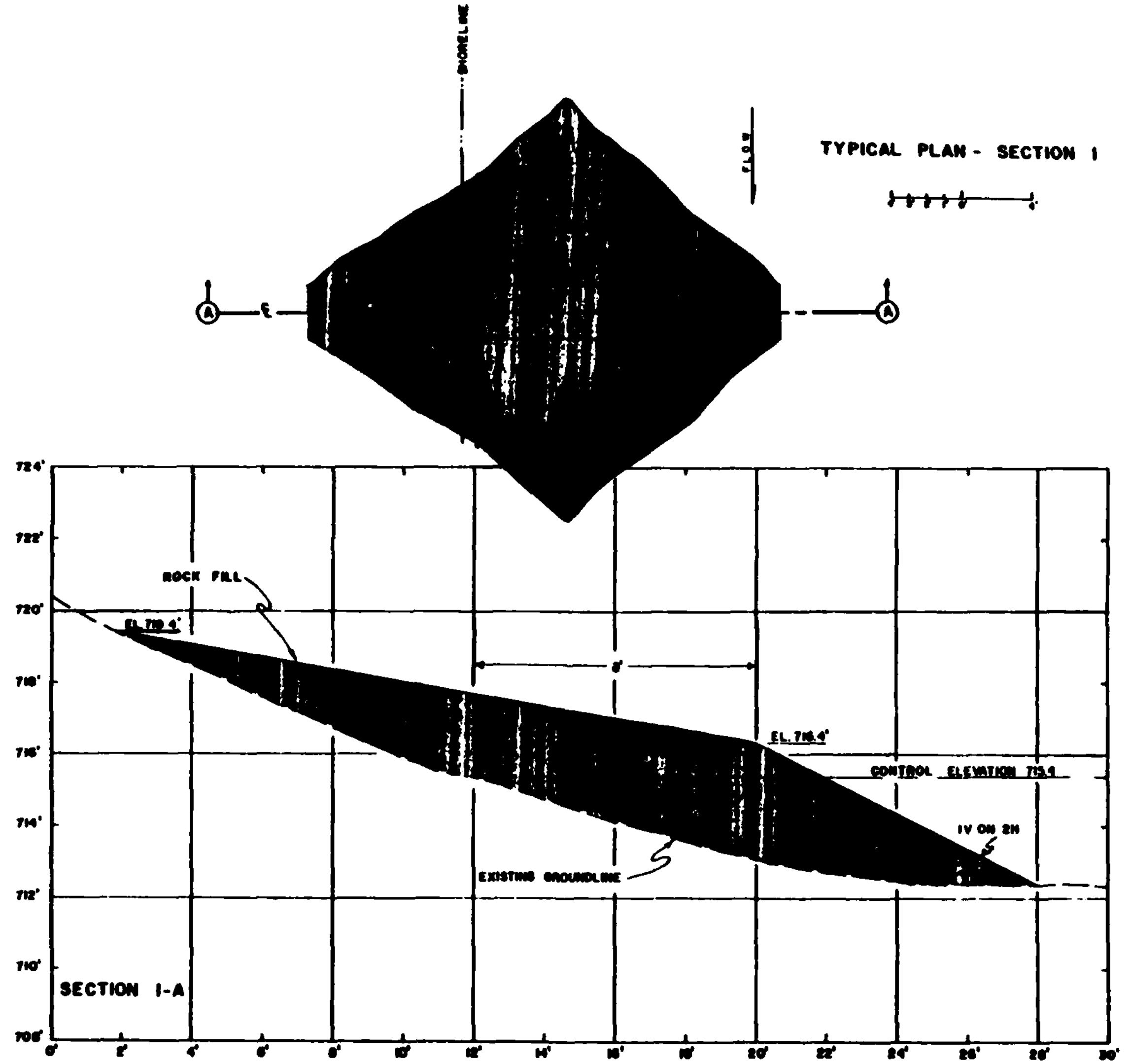

SITE 7 LOW
PLAN
CHIPPEWA RIVER
PLAN
NOTE: ELEVATIONS
GIVEN ON EACH SECTION ARE
W.S. ELEVATIONS AT TIME OF
SURVEYS. CONSTRUCTION WILL
BE CONTROLLED BY GIVEN
ELEVATIONS AND WILL BE
INDEPENDENT OF W.S. AT TIME
OF CONSTRUCTION

PLATE 29



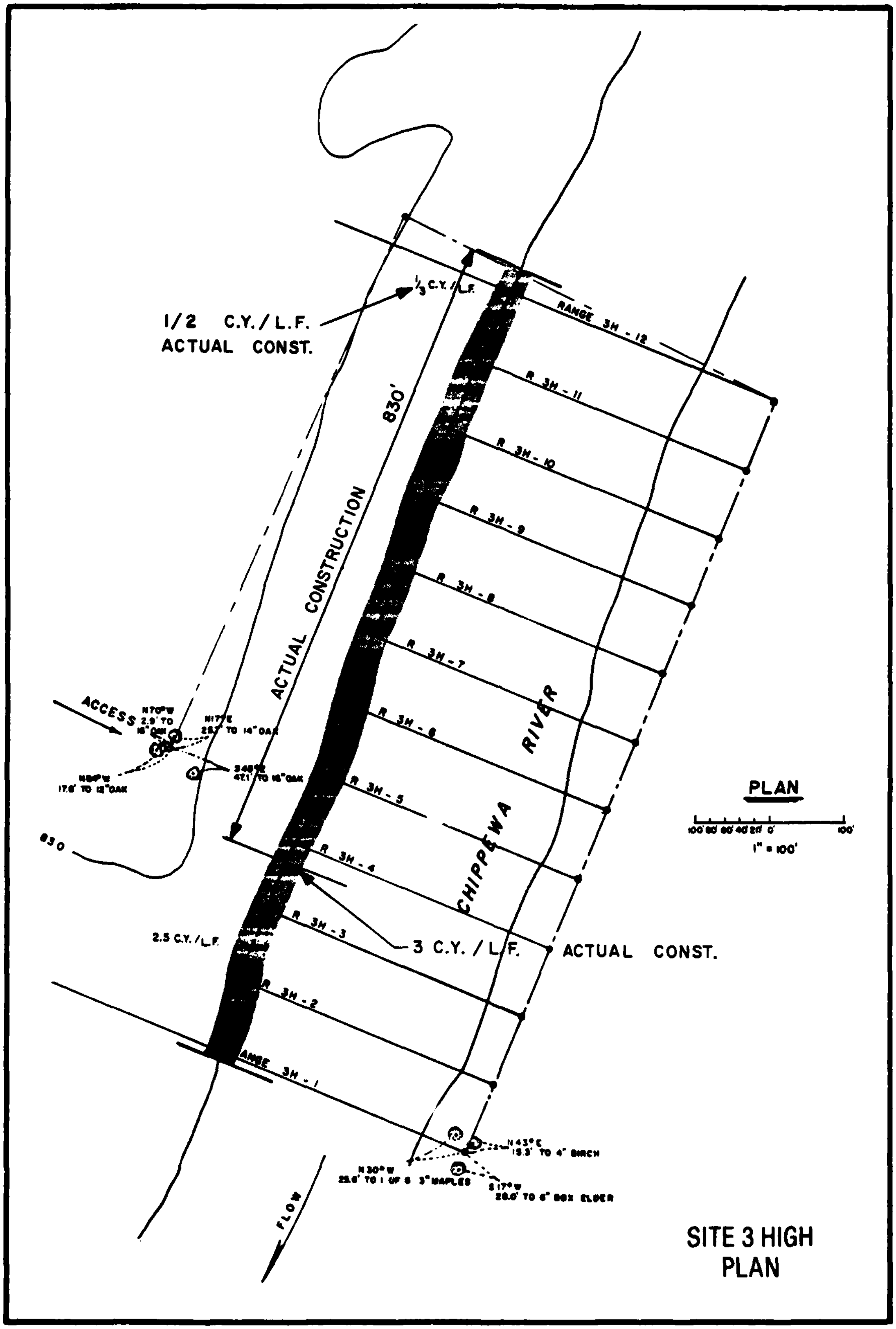
TYPICAL PLAN
SECTION 2

PLATE 30



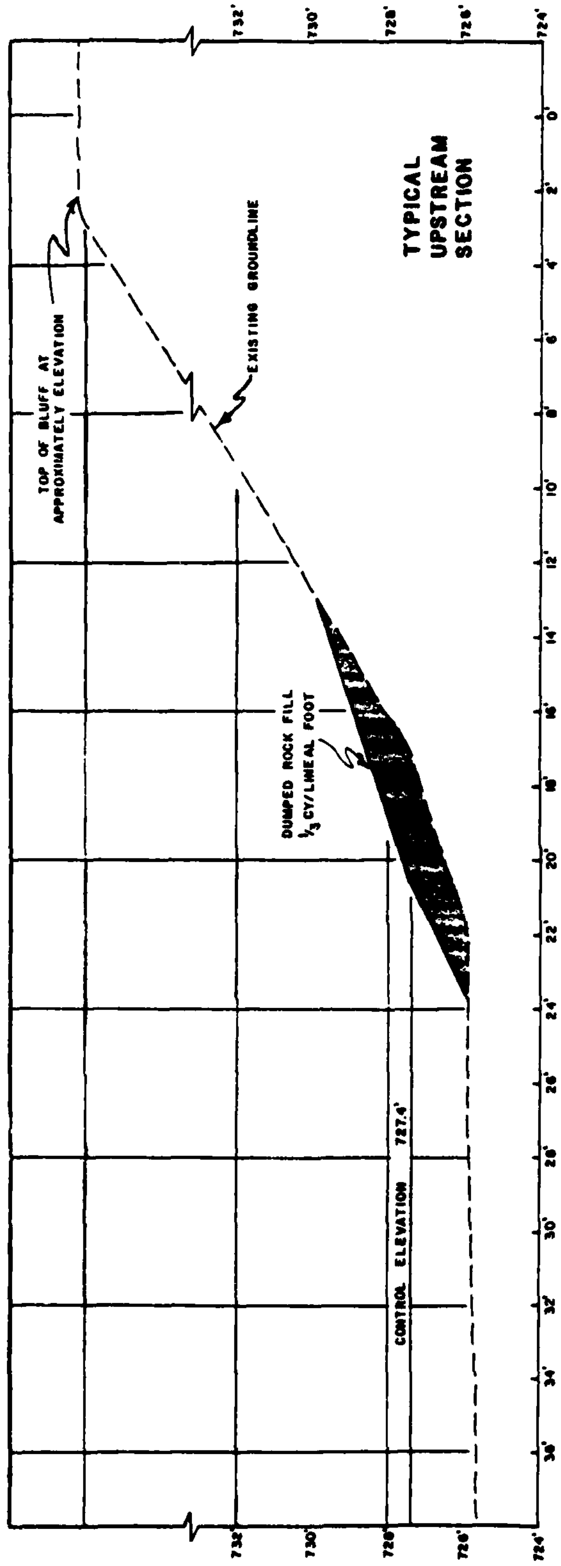
TYPICAL PLAN
SECTION 1

PLATE 31



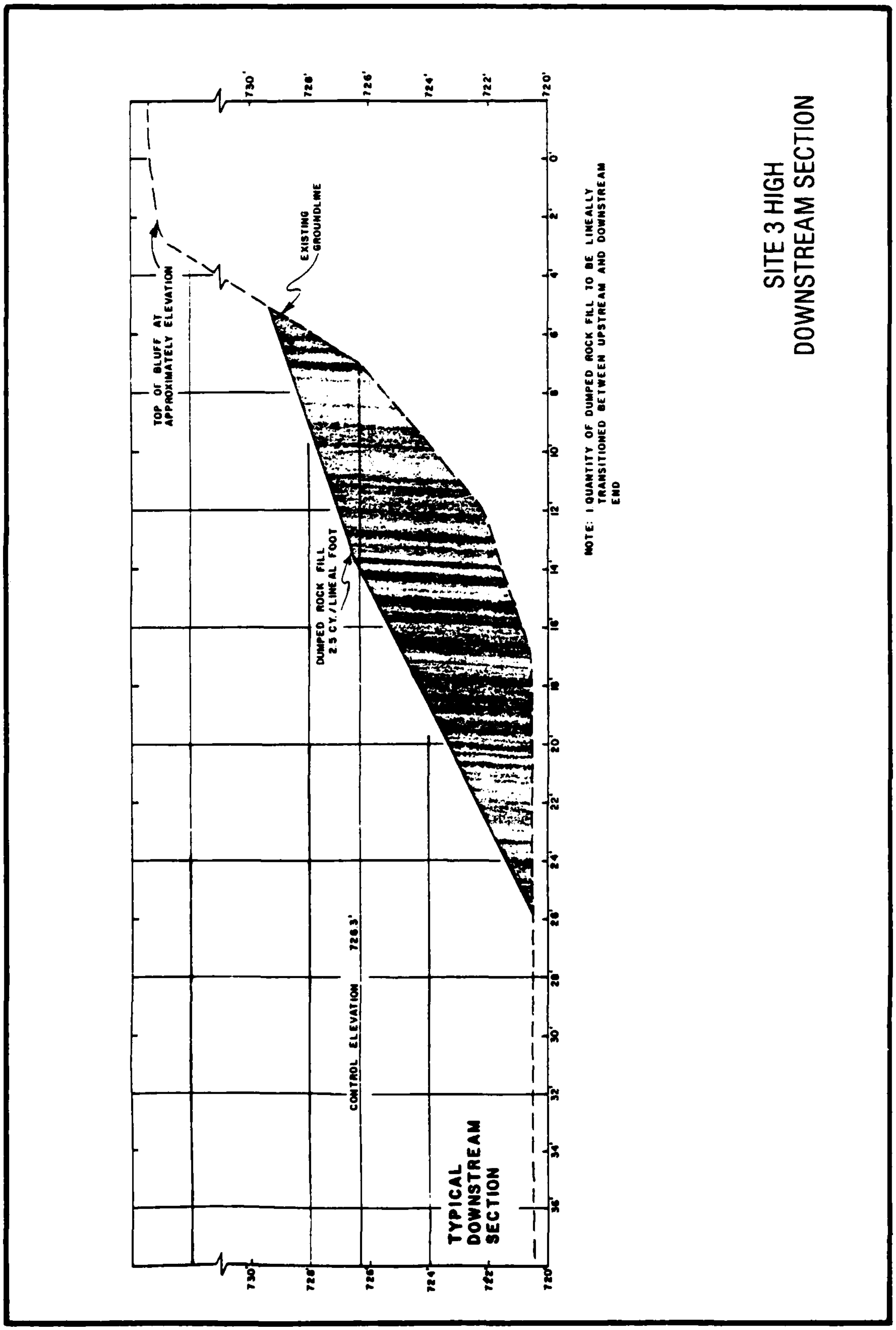
SITE 3 HIGH
PLAN

PLATE 32



SITE 3 HIGH
UPSTREAM SECTION

PLATE 33



SITE 3 HIGH DOWNSTREAM SECTION

PLATE 34

PHOTO 3

PHOTO 4

PHOTO 5

PHOTO 3. SECTION 1 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 4. SECTION 1 LOOKING UPSTREAM AFTER CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 5. SECTION 2 LOOKING UPSTREAM DURING CONSTRUCTION.

SITE 1 LOW

PLATE 35

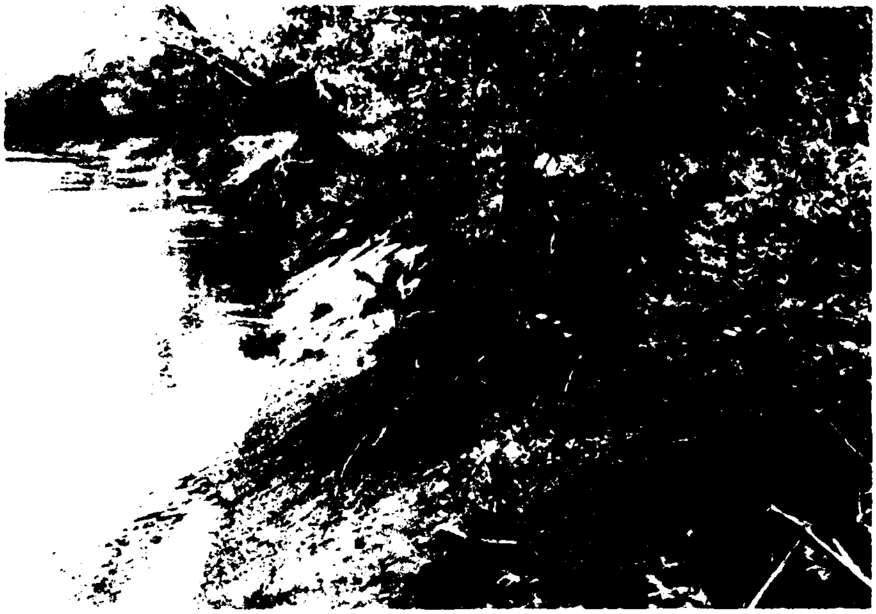


PHOTO 6. SECTION 4 LOOKING UPSTREAM BEFORE CONSTRUCTION.
27 AUGUST 1980.

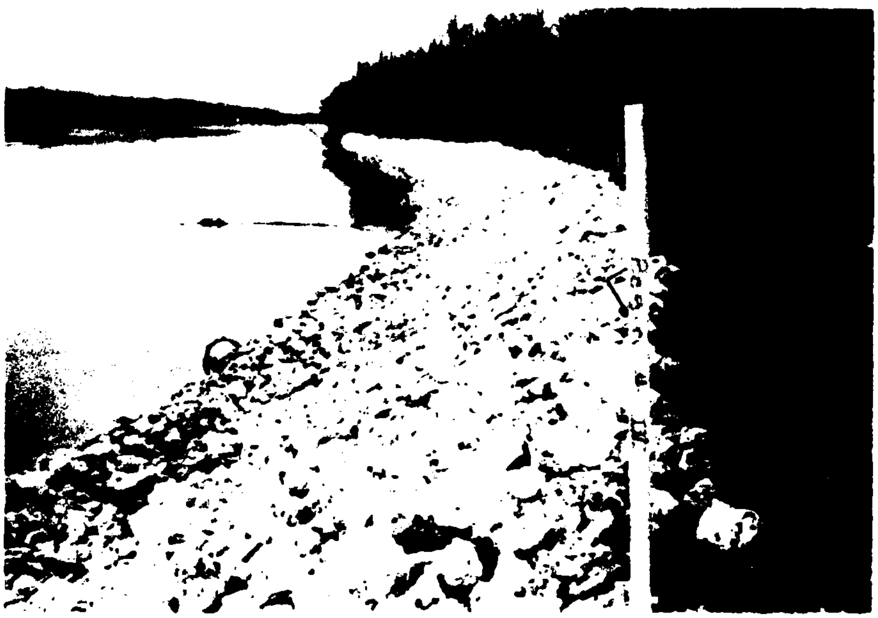


PHOTO 7. SECTION 4 LOOKING UPSTREAM AFTER CONSTRUCTION.
17 NOVEMBER 1980.

SITE 1 LOW

PLATE 36

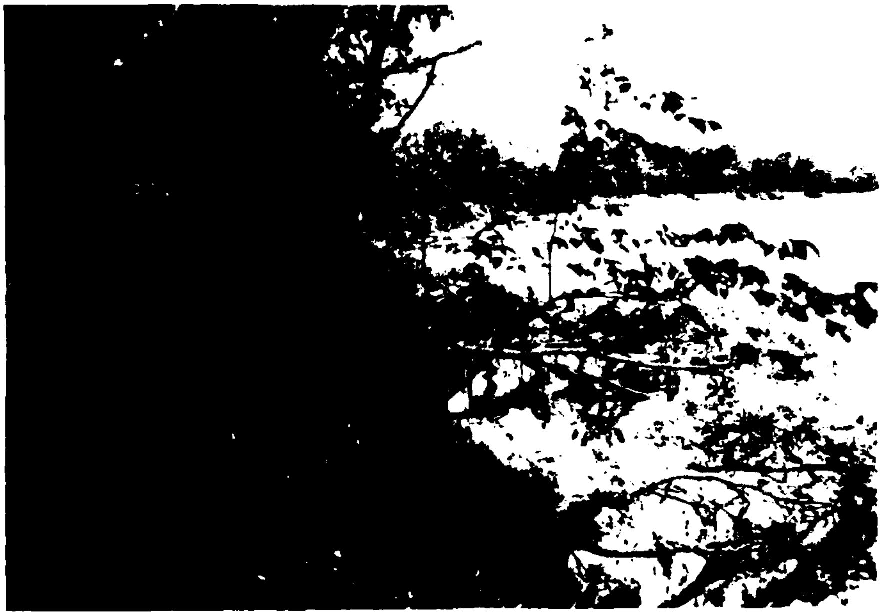


PHOTO 8. SECTION 5 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

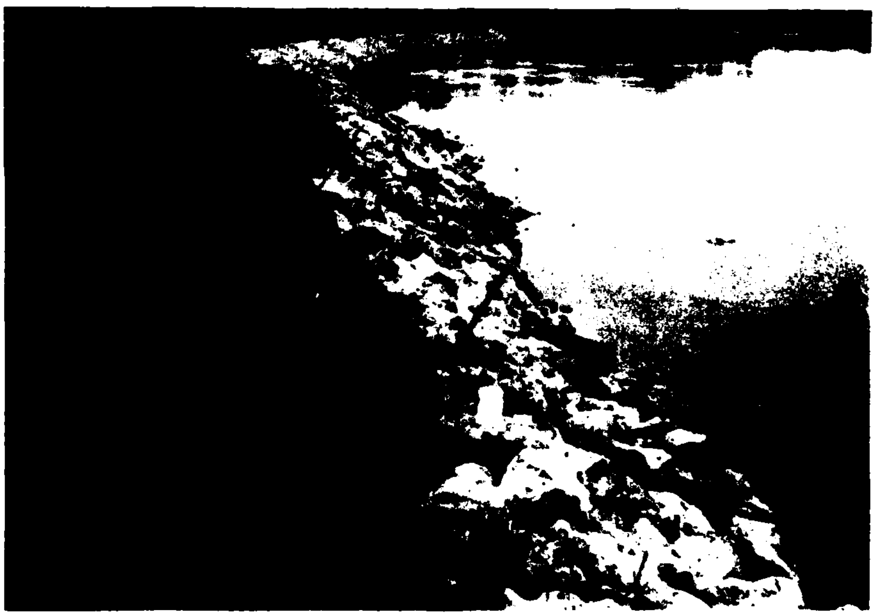


PHOTO 9. SECTION 5 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 17 NOVEMBER 1980.

SITE 1 LOW

PLATE 37

PHOTO 10

PHOTO 11

PHOTO 12

PHOTO 10. SECTION 1 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 11. SECTION 1 LOOKING DOWNSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 12. SECTION 1 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 2 LOW DOWNSTREAM SEGMENT

PLATE 38

PHOTO 13

PHOTO 14

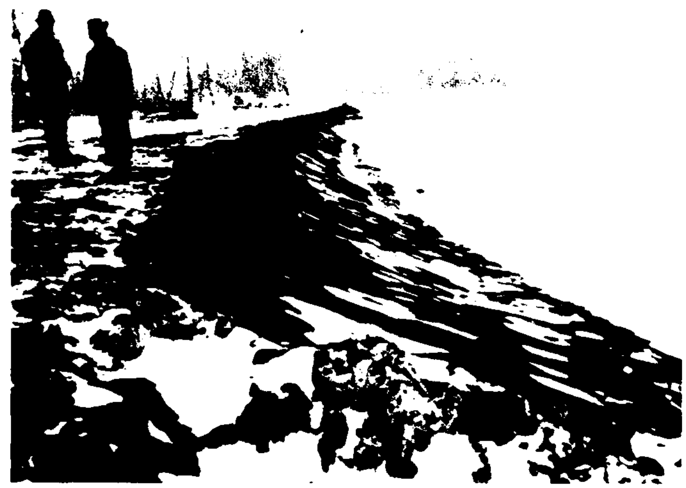


PHOTO 15

PHOTO 13. SECTION 3 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 14. SECTION 3 LOOKING DOWNSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 15. SECTION 3 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 2 LOW
DOWNSTREAM SEGMENT

PLATE 39

PHOTO 16

PHOTO 17

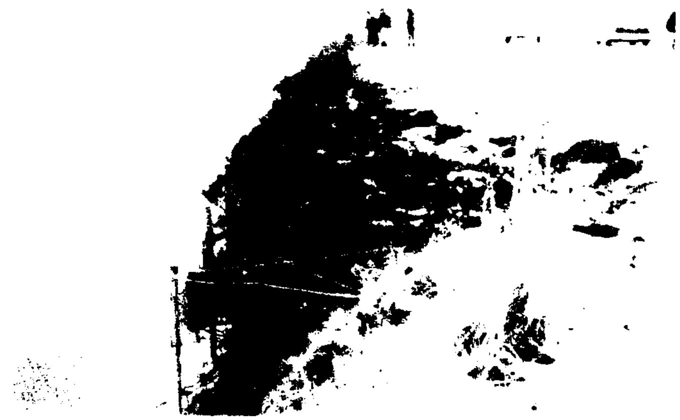


PHOTO 18

PHOTO 16. SECTION 4 LOOKING UPSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 17. SECTION 4 LOOKING UPSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 18. SECTION 4 LOOKING UPSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 2 LOW
UPSTREAM SEGMENT

PLATE 40

PHOTO 19

PHOTO 20

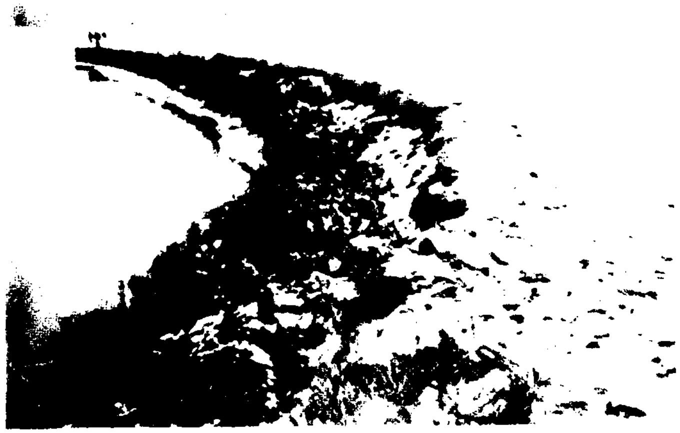


PHOTO 21

PHOTO 19. SECTION 6 LOOKING UPSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 20. SECTION 6 LOOKING UPSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 21. SECTION 6 LOOKING UPSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 2 LOW
UPSTREAM SEGMENT

PLATE 41

PHOTO 22

PHOTO 23

PHOTO 24

PHOTO 22. SECTION 7 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 23. SECTION 7 LOOKING DOWNSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 24. SECTION 7 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 2 LOW
UPSTREAM SEGMENT

PLATE 42

PHOTO 25. 24 NOVEMBER 1980.

SITE 7 LOW

PLATE 43

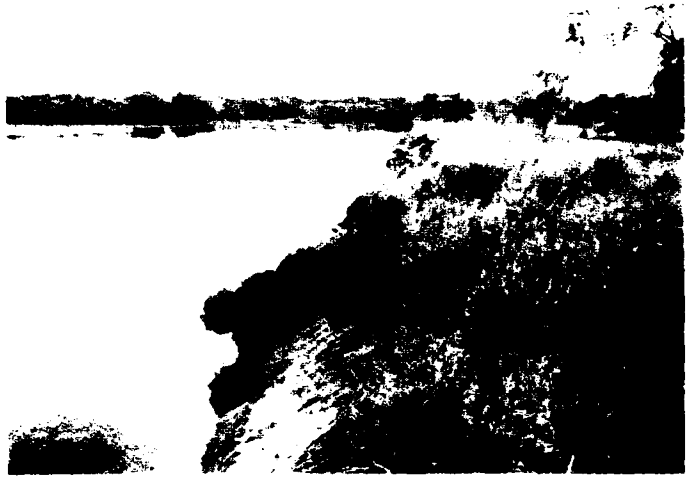


PHOTO 26

PHOTO 27

PHOTO 26. SECTION 1 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 27. SECTION 1 LOOKING DOWNSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 28. SECTION 1 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

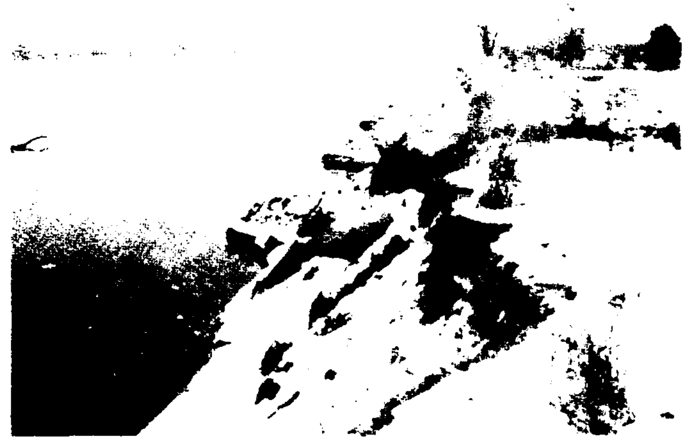


PHOTO 28

SITE 7 LOW

PLATE 44

PHOTO 29

PHOTO 30

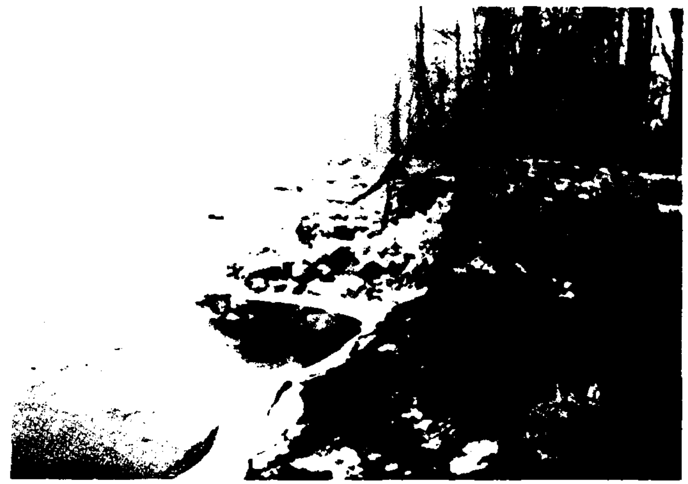


PHOTO 31

PHOTO 29. SECTION 2 LOOKING DOWNSTREAM BEFORE CONSTRUCTION. 27 AUGUST 1980.

PHOTO 30. SECTION 2 LOOKING DOWNSTREAM DURING CONSTRUCTION. 17 NOVEMBER 1980.

PHOTO 31. SECTION 2 LOOKING DOWNSTREAM AFTER CONSTRUCTION. 5 DECEMBER 1980.

SITE 7 LOW

PLATE 45

PHOTO 32. 24 NOVEMBER 1980.

SITE 3 HIGH

PLATE 46

PHOTO 33. LOOKING DOWNSTREAM BEFORE CONSTRUCTION.
27 AUGUST 1980.

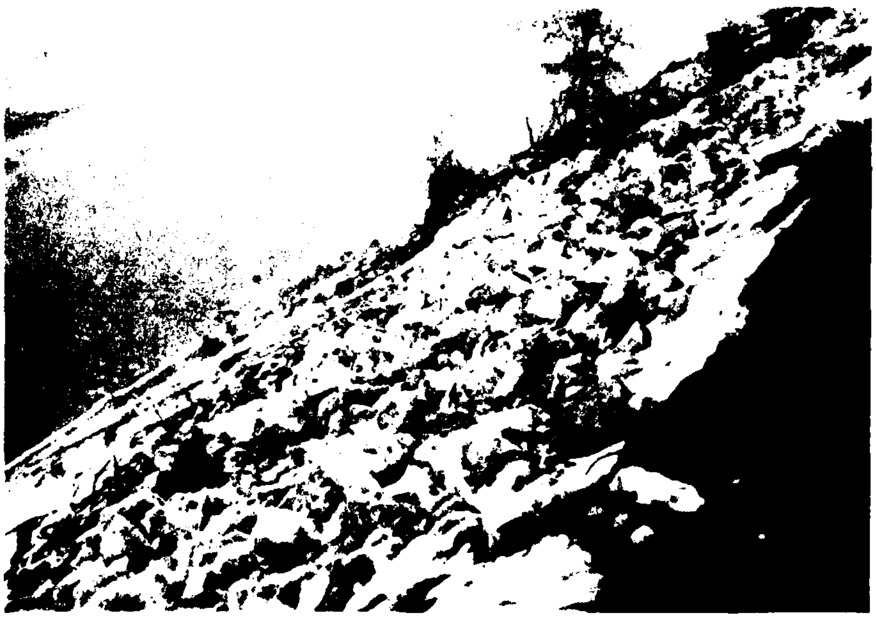


PHOTO 34. LOOKING DOWNSTREAM AFTER CONSTRUCTION.
5 DECEMBER 1980.

SITE 3 HIGH

PLATE 47

APPENDIX A

VELOCITY DATA

VELOCITY DATA, SITE 1 LOW, CROSS SECTION XF

WATER SURFACE ELEVATION 695.33 FEET, DISCHARGE 9,840 CFS, 2 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
963 - Bank Intersects Water Surface			
960	5.1	0.0	0.91
		1.02	0.81
		2.55	1.23
		4.08	0.74
945	11.4	0.0	2.35
		2.28	2.09
		5.70	2.49
		9.12	1.91
930	11.6	0.0	4.21
		2.32	3.72
		5.80	3.80
		9.28	3.44
915	9.8	0.0	4.25
		1.96	3.80
		4.90	3.37
		3.84	3.43
900	9.7	0.0	4.49
		1.94	4.37
		4.85	4.06
		7.76	3.27
885	9.4	0.0	4.39
		1.88	4.35
		4.70	3.88
		7.52	3.12
870	9.2	0.0	4.28
		1.84	4.35
		4.60	3.80
		7.36	2.92
855	9.4	0.0	4.59
		1.88	4.46
		4.70	4.25
		7.52	3.35
840	9.2	0.0	4.24
		1.84	4.15
		4.60	3.97
		7.36	3.05
825	8.2	0.0	4.46
		1.64	4.15
		4.10	3.97
		3.28	3.44
810	7.8	0.0	4.17
		1.56	3.97
		3.90	3.27
		6.24	3.12
795	6.7	0.0	4.45
		1.34	4.13
		3.35	4.25
		5.36	3.43
780	6.2	0.0	3.98
		1.24	3.97
		3.10	[illegible].64
		4.96	2.80
765	6.1	0.0	3.69
		1.22	3.35
		3.05	3.50
		4.88	2.92
740	5.7	0.0	3.88
		1.14	3.72
		2.85	3.72
		4.56	2.87
715	5.6	0.0	4.07
		1.12	3.80
		2.80	3.72
		4.48	3.12
690	5.8	0.0	3.46
		1.16	3.35
		2.90	3.50
		4.64	2.54
665	4.3	0.0	3.82
		2.15	3.19
		2.58	3.25
640	5.3	0.0	3.54
		1.06	3.27
		2.15	2.98
		4.24	2.74
615	4.9	0.0	3.59
		0.98	3.19
		2.45	2.92
		3.92	2.38
590	4.1	0.0	3.29
		2.05	2.87
		2.46	2.80
565	3.9	0.0	3.06
		1.95	2.73
		2.34	2.60
535	3.7	0.0	2.93
		1.85	2.80
		2.22	2.49
500	3.7	0.0	2.98
		1.85	2.66
		2.22	2.53
480	3.1	0.0	2.02
		1.55	1.64
		1.86	1.72
474 - Bank intersects Water Surface			

VELOCITY DATA, SITE 1 LOW, CROSS SECTION 1

WATER SURFACE ELEVATION 695.80 FEET, DISCHARGE 10,200 CFS, 3 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
950 - Bank Intersects Water Surface			
945	2.6	0.0	1.19
		1.56	1.01
930	5.4	0.0	2.26
		1.08	2.15
		2.70	2.04
		4.32	1.70
915	5.9	0.0	2.58
		1.18	2.38
		2.95	2.19
		4.72	2.00
900	6.8	0.0	2.36
		1.36	2.29
		3.40	1.99
		5.44	1.72
885	7.9	0.0	1.91
		1.58	1.68
		3.95	1.76
		6.32	1.57
870	8.0	0.0	2.35
		1.60	2.09
		4.00	1.87
		6.40	1.91
840	8.2	0.0	2.09
		1.64	1.91
		4.10	1.95
		6.56	1.64
825	7.8	0.0	2.41
		1.56	2.29
		3.90	2.04
		6.24	1.80
810	7.3	0.0	2.59
		1.46	2.60
		3.65	2.54
		5.84	1.80
795	6.5	0.0	3.21
		1.30	2.60
		3.25	2.54
		5.20	2.85
780	6.6	0.0	3.09
		1.32	3.12
		3.30	2.54
		5.28	2.13
765	7.0	0.0	3.12
		1.40	3.12
		3.50	2.73
		5.60	2.19
750	7.4	0.0	3.49
		1.48	3.27
		3.70	2.87
		5.92	2.66

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
725	7.3	0.0	3.58
		1.46	3.43
		3.65	3.43
		5.84	2.66
700	8.1	0.0	3.49
		1.62	3.27
		4.05	3.35
		6.48	2.66
680	7.2	0.0	4.11
		1.44	3.72
		3.60	2.98
		5.76	3.27
660	7.7	0.0	3.81
		1.54	3.88
		3.85	3.05
		6.16	2.60
640	8.2	0.0	3.76
		1.64	3.80
		4.10	3.43
		6.56	2.60
620	7.5	0.0	4.36
		1.50	4.06
		3.75	3.64
		6.00	3.35
600	7.9	0.0	3.88
		1.58	3.72
		3.95	3.57
		6.32	2.87
575	7.9	0.0	3.84
		1.58	3.80
		3.95	3.50
		6.32	2.73
550	7.7	0.0	3.79
		1.54	3.72
		3.85	3.50
		6.16	2.73
525	5.8	0.0	3.83
		1.16	3.64
		2.90	3.64
		4.64	2.87
500	5.6	0.0	3.99
		1.12	3.80
		2.80	3.50
		4.48	2.98
460	3.7	0.0	4.04
		1.85	3.64
		2.22	3.43
420	2.7	0.0	3.21
		1.35	2.80
		1.62	2.73
390	1.4 - No Flow		
360 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 1 LOW, CROSS SECTION 16

WATER SURFACE ELEVATION 696.22 FEET, DISCHARGE 10,900 CFS, 3 SEPTEMBER 1981

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
720 - Bank Intersects Water Surface			
716	2.1	0.0	0.47
		1.26	0.40
700	3.4	0.0	0.98
		2.04	0.83
685	4.6	0.0	3.53
		0.92	3.27
		2.30	3.19
		3.68	2.73
670	5.3	0.0	3.45
		1.06	3.19
		2.65	3.19
		2.12	2.68
655	5.2	0.0	4.05
		1.04	3.43
		2.60	3.12
		4.16	2.98
640	5.0	0.00	3.94
		1.00	3.57
		2.50	3.19
		4.00	3.12
625	5.1	0.0	3.46
		1.02	3.35
		2.05	3.27
		4.08	2.54
610	5.3	0.0	3.62
		1.06	3.35
		2.65	3.27
		4.24	2.80
595	5.5	0.0	4.21
		1.10	3.35
		2.75	3.25
		4.40	3.80
580	5.2	0.0	3.81
		1.04	3.43
		2.60	3.27
		4.16	3.05
565	5.8	0.0	3.62
		1.16	3.35
		2.90	2.85
		4.64	2.80
550	5.9	0.0	3.54
		1.18	3.35
		2.95	3.27
		4.72	2.66
535	5.3	0.0	3.57
		1.06	3.27
		2.65	3.08
		4.24	2.80
520	5.7	0.0	3.72
		1.14	3.35
		2.85	3.08
		4.56	2.98
500	5.7	0.0	3.46
		1.14	3.50
		2.85	3.19
		4.56	2.38
475	5.7	0.0	3.33
		1.14	3.12
		2.85	2.87
		4.56	2.54

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
450	5.9	0.0	3.28
		1.18	3.19
		2.95	2.80
		4.72	2.38
425	5.7	0.0	3.21
		1.14	3.12
		2.85	2.80
		4.56	2.33
400	5.8	0.0	3.16
		1.16	3.19
		2.90	2.92
		4.64	2.19
375	5.1	0.0	3.69
		1.02	3.43
		2.55	3.19
		4.08	2.85
350	5.6	0.0	3.36
		1.12	3.43
		2.80	3.27
		2.24	2.29
		0.0	3.61
325	5.9	1.18	3.27
		2.95	3.12
		4.72	2.87
300	6.8	0.0	3.83
		1.36	3.64
		3.40	2.87
		5.44	2.87
275	6.7	0.0	3.88
		1.34	3.72
		3.35	3.43
		5.36	2.87
250	7.3	0.0	3.44
		1.46	3.12
		3.65	3.12
		5.84	2.73
225	7.0	0.0	3.44
		1.40	3.19
		3.50	2.73
		5.60	2.66
200	7.0	0.0	3.68
		1.40	3.72
		3.50	3.27
		5.60	2.54
175	6.2	0.0	3.54
		1.24	3.10
		3.10	3.12
		4.96	2.92
150	5.8	0.0	3.41
		1.16	3.43
		2.90	3.19
		4.64	2.37
125	5.0	0.0	3.19
		1.00	3.19
		2.50	2.66
		4.00	2.24
80	4.4	0.0	2.69
		2.20	2.29
		2.64	2.29
45	3.5	0.0	1.24
		1.75	1.01
		2.10	1.05
30 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 2 LOW, CROSS SECTION 1

WATER SURFACE ELEVATION 701.63 FEET, DISCHARGE 17,900 CFS, 8 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
916 - Bank Intersects Water Surface			
910	5.3	0.0	2.23
		1.06	2.29
		2.65	2.04
		4.24	1.50
895	7.8	0.0	2.98
		1.56	2.66
		3.90	2.80
		6.24	2.40
880	10.5	0.0	4.26
		2.10	3.97
		5.25	3.72
		8.40	3.27
865	11.0	0.0	4.21
		2.20	3.88
		5.50	3.88
		8.80	3.27
850	11.0	0.0	4.36
		2.20	4.15
		5.50	3.88
		8.80	3.27
835	10.5	0.0	4.13
		2.10	4.15
		5.25	4.15
		8.40	2.87
820	10.3	0.0	4.48
		2.06	4.35
		5.15	3.64
		8.24	3.27
805	9.7	0.0	4.16
		1.94	3.80
		4.85	3.72
		7.76	3.27
790	10.0	0.0	4.18
		2.0	4.25
		5.0	4.15
		8.0	2.85
775	9.0	0.0	4.46
		1.80	4.15
		4.50	3.80
		7.20	3.43
760	8.8	0.0	4.21
		1.76	4.35
		4.40	3.35
		7.04	2.80
745	8.9	0.0	4.34
		1.78	4.25
		4.45	3.97
		7.12	3.12
730	8.9	0.0	3.88
		1.78	3.80
		4.45	3.97
		7.12	2.80
715	8.3	0.0	4.05
		1.66	3.97
		4.15	3.25
		6.64	2.92
690	7.2	0.0	4.46
		1.44	4.15
		3.60	4.15
		5.76	3.43
665	6.6	0.0	3.88
		1.32	3.80
		3.30	3.64
		5.28	2.80
640	6.0	0.0	4.16
		1.20	3.80
		3.00	3.27
		4.80	3.27
615	5.7	0.0	4.31
		1.14	4.06
		2.85	3.50
		4.56	3.27
590	6.1	0.0	4.02
		1.22	3.97
		3.05	3.50
		4.88	2.87
565	5.6	0.0	4.16
		1.12	3.64
		2.80	3.64
		4.48	3.44
540	5.7	0.0	4.02
		1.14	3.97
		2.85	3.57
		4.56	2.87
515	6.2	0.0	3.85
		1.24	3.88
		3.10	3.35
		4.96	2.66
490	6.4	0.0	3.83
		1.28	3.97
		3.20	3.57
		5.12	2.54
465	6.4	0.0	4.25
		1.28	3.80
		3.20	3.80
		5.12	3.43
440	6.5	0.0	4.13
		1.30	3.97
		3.25	3.88
		5.20	3.05
415	7.3	0.0	3.88
		1.46	3.72
		3.65	3.43
		5.84	2.87
390	7.8	0.0	4.01
		1.56	4.15
		3.90	3.57
		6.24	2.66
365	8.0	0.0	3.68
		1.60	4.06
		4.00	3.43
		6.40	2.19
340	7.9	0.0	4.02
		1.58	3.97
		3.95	3.80
		6.32	2.87
315	7.4	0.0	4.11
		1.48	3.80
		3.70	3.50
		5.92	3.19
290	6.9	0.0	3.98
		1.38	3.64
		3.45	3.35
		5.52	3.12
265	6.8	0.0	3.71
		1.36	3.43
		3.40	3.12
		5.44	2.87
240	5.2	0.0	3.25
		1.04	2.98
		2.60	2.80
		4.16	2.54
200	2.0	0.0	2.02
		1.0	1.72
		1.20	1.72
165	1.1	0.0	1.33
		0.55	1.13
		0.66	1.13
155 - Bank Intersects River Edge			

VELOCITY DATA, SITE 2 LOW, CROSS SECTION 7

WATER SURFACE ELEVATION 700.90 FEET, DISCHARGE 15,600 CFS, 8 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
944 - Bank Intersects Water Surface			
940	6.5	0.0	0.32
		1.30	0.30
		3.25	0.36
		5.20	0.24
925	10.7	0.0	1.59
		2.14	1.21
		5.35	1.68
		8.56	1.50
910	9.2	0.0	2.44
		1.84	2.11
		4.60	2.29
		7.36	2.04
895	9.1	0.0	4.00
		1.82	3.88
		4.55	3.35
		7.28	2.92
880	9.5	0.0	4.11
		1.90	4.06
		4.75	3.27
		7.60	2.92
865	9.5	0.0	4.08
		1.90	4.06
		4.75	3.97
		7.60	2.87
850	9.2	0.0	4.08
		1.84	4.06
		4.60	3.97
		7.36	2.87
835	9.0	0.0	4.12
		1.80	3.88
		4.50	3.35
		7.20	3.12
820	9.7	0.0	3.82
		1.94	4.06
		4.85	3.50
		7.76	2.43
805	9.6	0.0	3.74
		1.92	4.06
		4.80	3.25
		7.68	2.29
790	9.2	0.0	4.08
		1.84	4.06
		4.60	3.88
		7.36	2.87
775	8.7	0.0	4.26
		1.74	4.06
		4.35	3.72
		6.96	3.19
760	8.6	0.0	3.99
		1.72	4.06
		4.30	3.43
		6.88	2.73
745	8.3	0.0	4.16
		1.66	3.64
		4.15	3.35
		6.64	3.44
730	8.9	0.0	3.49
		1.78	3.64
		4.45	3.50
		7.12	2.29
690	7.7	0.0	3.98
		1.54	3.97
		3.85	3.19
		6.16	2.80
650	6.9	0.0	4.24
		1.38	3.64
		3.45	3.27
		5.52	3.57
610	6.3	0.0	3.67
		1.26	3.64
		3.15	3.43
		5.04	2.60
570	7.0	0.0	4.03
		1.40	3.80
		3.50	3.43
		5.60	3.05
530	5.8	0.0	4.21
		1.16	3.97
		2.90	3.57
		4.64	3.19
490	5.5	0.0	4.31
		1.10	3.97
		2.75	3.19
		4.40	3.35
450	4.8	0.0	3.66
		0.96	3.43
		2.40	3.27
		3.84	2.80
400	4.2	0.0	3.06
		2.10	2.66
		2.52	2.60
340	3.8	0.0	2.80
		1.90	2.49
		2.28	2.38
280	4.1	0.0	2.86
		2.05	2.80
		2.46	2.43
200	3.4	0.0	2.86
		1.70	2.54
		2.04	2.43
100	3.8	0.0	2.86
		1.90	2.60
		2.28	2.43
80	1.2	0.0	2.02
		0.72	1.72
70 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 2 LOW, CROSS SECTION 18

WATER SURFACE ELEVATION 700.81 FEET, DISCHARGE 16,800 CFS, 9 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
1040 - Bank Intersects Water Surface			
1030	4.4	0.0	1.80
		2.64	1.53
1015	5.7	0.0	3.07
		1.14	2.73
		2.85	2.80
		4.56	2.49
1000	5.8	0.0	2.68
		1.16	2.80
		2.90	2.24
		4.64	1.76
985	5.3	0.0	2.79
		1.06	2.66
		2.65	2.54
		4.24	2.09
970	4.4	0.0	2.99
		2.20	2.66
		2.64	2.54
955	4.3	0.00	2.80
		2.15	2.60
		2.58	2.38
940	4.2	0.0	2.64
		2.10	2.43
		2.52	2.24
925	4.3	0.0	2.69
		2.15	2.24
		2.58	2.29
910	4.3	0.0	2.64
		2.15	2.24
		2.58	2.24
895	4.3	0.0	2.86
		2.15	2.54
		2.58	2.43
880	4.8	0.0	2.79
		0.96	2.66
		2.40	2.54
		3.84	2.09
865	5.2	0.0	2.66
		1.04	2.54
		2.60	2.66
		4.16	1.99
850	5.3	0.0	2.98
		1.06	2.87
		2.65	2.80
		4.24	2.19
835	5.6	0.0	3.15
		1.12	2.98
		2.80	2.87
		4.48	2.38
800	6.0	0.0	3.50
		1.20	3.57
		3.00	3.27
		4.80	2.38

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
760	7.0	0.0	3.65
		1.40	3.72
		3.50	3.27
		5.60	2.49
720	7.4	0.0	3.71
		1.48	3.43
		3.70	3.57
		5.92	2.88
680	8.0	0.0	4.11
		1.60	3.72
		4.00	3.35
		6.40	3.27
640	7.9	0.0	4.58
		1.58	4.29
		3.95	4.06
		6.32	3.50
600	8.2	0.0	3.83
		1.64	3.64
		4.10	3.64
		6.56	2.87
560	9.1	0.0	4.02
		1.82	3.97
		4.55	3.50
		7.28	2.87
520	7.2	0.0	4.21
		1.44	4.35
		3.60	3.97
		5.76	2.80
480	6.7	0.0	4.39
		1.34	3.97
		3.35	4.15
		5.36	3.50
440	5.9	0.0	4.58
		1.18	4.15
		2.95	3.64
		4.72	3.64
400	7.0	0.0	4.55
		1.40	4.46
		3.50	3.88
		5.60	3.27
350	5.4	0.0	3.52
		1.08	2.80
		2.70	3.72
		4.32	3.19
300	3.9	0.0	4.38
		1.95	3.72
		2.34	3.72
200	2.5	0.0	5.00
		1.25	4.25
		1.50	4.25
150	2.4	0.0	4.47
		1.20	3.80
		1.76	3.80
135 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 LOW, CROSS SECTION 16

WATER SURFACE ELEVATION 701.71 FEET, DISCHARGE 17,100 CFS, 9 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
820 - Bank intersects water surface			
815	5.6	0.0	2.32
		1.12	2.33
		2.80	1.38
		4.48	1.61
800	9.2	0.0	4.64
		1.84	4.25
		4.60	4.06
		7.36	3.64
785	9.5	0.0	5.07
		1.90	4.74
		4.75	4.46
		7.60	3.88
770	9.1	0.0	5.05
		1.82	4.95
		4.55	4.74
		7.28	3.64
755	9.1	0.0	5.49
		1.82	5.18
		4.55	4.85
		7.28	4.15
740	9.1	0.0	5.81
		1.82	5.31
		4.55	5.07
		7.28	4.57
725	9.6	0.0	5.53
		1.92	5.43
		4.80	5.07
		7.68	3.97
710	9.6	0.0	4.48
		1.92	4.95
		4.80	4.55
		7.68	3.27
695	9.5	0.0	5.30
		1.90	4.95
		4.75	4.35
		7.60	4.06
680	9.2	0.0	5.61
		1.84	5.18
		4.60	4.85
		7.36	4.35
665	9.7	0.0	4.81
		1.94	4.74
		4.85	4.15
		7.76	3.43
650	9.5	0.0	4.88
		1.90	4.57
		4.75	4.46
		7.60	3.72

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
635	9.2	0.0	4.72
		1.84	4.15
		4.60	4.57
		7.36	3.88
620	8.8	0.0	4.81
		1.76	4.74
		4.40	4.64
		7.04	3.43
580	9.2	0.0	3.89
		1.84	4.74
		4.60	4.15
		7.36	1.87
540	8.6	0.0	3.86
		1.72	4.57
		4.30	3.97
		6.88	1.99
500	8.4	0.0	3.41
		1.68	3.80
		4.20	3.57
		6.72	1.99
460	7.1	0.0	4.21
		1.42	3.97
		3.55	3.72
		5.68	3.19
420	6.3	0.0	3.52
		1.26	3.19
		3.15	3.27
		5.04	2.80
380	5.7	0.0	3.27
		1.14	3.27
		2.85	2.98
		4.56	2.29
340	4.9	0.0	3.62
		0.98	3.72
		2.45	2.87
		3.92	2.43
300	4.4	0.0	3.67
		2.2	2.87
230	4.8	0.0	3.04
		0.96	3.12
		2.40	2.73
		3.84	2.04
160	4.9	0.0	3.04
		0.98	2.87
		2.45	2.73
		3.92	2.29
145 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 LOW, CROSS SECTION 7

WATER SURFACE ELEVATION 702.23 FEET, DISCHARGE 18,100 CFS, 9 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
842 - Bank Intersects Water Surface			
835	7.4	0.0	2.87
		1.48	3.27
		3.70	2.38
		5.92	1.61
820	10.0	0.0	4.75
		2.0	4.64
		5.0	4.15
		8.0	3.43
805	12.3	0.0	5.08
		2.46	5.07
		6.15	4.37
		9.84	3.57
790	12.0	0.0	5.00
		2.40	5.07
		6.00	4.57
		9.60	3.43
775	11.7	0.0	5.30
		2.34	4.95
		5.85	4.64
		9.36	4.06
760	11.0	0.0	5.18
		2.20	4.74
		5.50	4.35
		8.80	4.06
745	10.8	0.0	4.92
		2.16	4.57
		5.40	4.57
		8.64	3.80
730	10.8	0.0	4.89
		2.16	4.25
		5.40	4.15
		8.64	4.06
715	10.8	0.0	4.79
		2.16	4.64
		5.40	3.43
		8.64	3.50
700	10.0	0.0	4.60
		2.00	4.25
		5.00	3.97
		8.00	3.57
685	9.4	0.0	4.95
		1.88	4.35
		4.70	3.97
		7.52	4.06
670	10.6	0.0	4.05
		2.12	4.15
		5.30	3.97
		8.48	2.73
655	9.7	0.0	4.91
		1.94	4.46
		4.85	3.72
		7.76	3.88

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
640	9.0	0.0	4.35
		1.80	4.35
		4.50	4.15
		5.20	3.43
600	8.4	0.0	4.09
		1.68	4.35
		4.20	3.64
		6.72	2.60
560	7.0	0.0	4.21
		1.40	3.97
		3.50	3.88
		5.60	3.19
520	7.6	0.0	4.71
		1.52	4.57
		3.80	3.97
		6.08	3.43
480	7.9	0.0	4.13
		1.58	4.15
		3.95	3.27
		6.32	2.87
440	6.5	0.0	4.19
		1.30	4.46
		3.25	3.72
		5.20	2.66
400	6.0	0.0	4.19
		1.20	4.25
		3.00	3.64
		4.80	2.87
360	5.5	0.0	3.55
		1.10	3.44
		2.75	3.72
		4.40	2.60
320	3.8	0.0	3.35
		1.90	3.12
		2.28	2.85
250	4.1	0.0	3.06
		2.05	2.54
		2.46	2.60
180	3.0	0.0	2.80
		1.50	2.38
		1.80	2.38
140	2.8	0.0	2.86
		1.40	2.13
		1.68	2.43
125 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 LOW, CROSS SECTION 1

WATER SURFACE ELEVATION 701.43 FEET, DISCHARGE 14,900 CFS, 11 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
905 - Bank Intersects Water Surface			
880	5.0	0.0	2.42
		1.00	2.24
		2.50	2.13
		4.00	1.87
865	5.9	0.0	3.09
		1.18	2.87
		2.95	2.98
		4.72	2.38
850	5.5	0.0	3.89
		1.10	3.88
		2.75	3.72
		4.40	2.73
835	5.0	0.0	4.38
		1.00	4.25
		2.50	3.97
		4.00	3.19
820	5.1	0.0	4.31
		1.02	4.06
		2.55	3.88
		4.08	3.27
805	5.3	0.0	4.66
		1.06	4.57
		2.65	4.25
		4.24	3.35
790	5.6	0.0	4.50
		1.12	4.46
		2.80	4.15
		4.48	3.19
775	6.6	0.0	4.48
		1.32	4.57
		3.30	4.15
		5.28	3.05
760	7.9	0.0	4.48
		1.58	4.57
		3.95	4.15
		6.32	3.05
745	8.7	0.0	4.61
		1.74	4.64
		4.35	4.06
		6.96	3.19
730	9.0	0.0	4.41
		1.80	4.25
		4.50	4.15
		7.20	3.25
715	9.2	0.0	4.19
		1.84	4.25
		4.60	3.80
		7.36	2.87

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
680	8.2	0.0	4.26
		1.64	4.06
		4.10	4.06
		6.56	3.19
640	7.7	0.0	4.41
		1.54	4.06
		3.85	3.88
		6.16	3.43
600	7.1	0.0	4.32
		1.42	4.15
		3.55	3.88
		5.68	3.19
560	7.6	0.0	4.18
		1.52	4.06
		3.80	3.97
		6.08	3.05
520	7.8	0.0	4.34
		1.56	4.46
		3.90	3.88
		6.24	2.92
480	7.7	0.0	4.13
		1.54	4.15
		3.85	3.97
		6.16	2.87
440	7.0	0.0	4.31
		1.40	4.06
		3.50	3.97
		5.60	3.27
400	7.1	0.0	4.02
		1.42	3.97
		3.55	3.72
		5.68	2.87
360	5.8	0.0	3.44
		1.16	3.25
		2.90	3.50
		4.64	2.60
320	4.0	0.0	3.85
		2.00	3.35
		2.40	3.27
260	2.2	0.0	4.12
		1.10	3.43
190	2.6	0.0	2.58
		1.30	2.19
170	1.8	0.0	1.98
		0.90	1.68
160 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 7 LOW, CROSS SECTION 1

WATER SURFACE ELEVATION 716.88 FEET, DISCHARGE *, 10 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
0 - Bank Intersects Water Surface			
8	3.0	0.0	2.02
		1.50	1.83
		1.80	1.72
20	4.3	0.0	3.29
		2.15	3.05
		2.58	2.80
35	4.8	0.0	3.79
		0.96	3.64
		2.40	3.27
		3.84	2.80
50	5.3	0.0	4.02
		1.06	3.97
		2.65	3.72
		4.24	2.87
65	6.3	0.0	4.72
		1.26	4.46
		3.15	3.88
		5.04	3.57
80	7.2	0.0	4.59
		1.44	4.46
		3.60	4.46
		5.76	3.35
100	7.5	0.0	4.18
		1.50	4.06
		3.75	3.43
		6.00	3.05
120	7.4	0.0	4.04
		1.48	4.06
		3.70	3.72
		5.92	2.80
150	7.4	0.0	4.24
		1.48	4.15
		3.70	3.64
		5.92	3.05
200	7.3	0.0	4.47
		1.46	4.25
		3.65	3.88
		5.84	3.35

*Velocity Measurements were not taken across entire section. Stage was high and tag line could not be anchored on east side of river. Distance from initial point is estimated.

VELOCITY DATA, SITE 7 LOW, CROSS SECTION 6

WATER SURFACE ELEVATION 717.09 FEET, DISCHARGE 10,200 CFS, 10 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
819 - Bank Intersects Water Surface			
815	5.0	0.0	1.34
		1.00	1.25
		2.50	0.94
		4.00	1.03
800	10.0	0.0	2.15
		2.00	2.29
		5.00	1.80
		8.00	1.37
785	10.8	0.0	3.12
		2.16	3.50
		5.40	1.47
		8.64	1.80
770	10.9	0.0	3.25
		2.18	3.72
		5.45	2.24
		8.72	1.80
755	9.4	0.0	4.42
		1.88	4.46
		4.70	3.97
		7.52	3.05
740	9.5	0.0	4.81
		1.90	4.74
		4.75	4.25
		7.60	3.43
725	9.6	0.0	3.92
		1.92	4.06
		4.80	3.80
		7.68	2.60
710	10.0	0.0	4.31
		2.00	3.97
		5.00	2.98
		8.00	3.35
695	10.0	0.0	4.38
		2.00	4.57
		5.00	4.15
		8.00	2.87
680	10.1	0.0	4.11
		2.02	4.57
		5.05	4.26
		8.08	2.42
665	9.1	0.0	4.59
		1.82	4.46
		4.55	4.25
		7.28	3.35
650	9.0	0.0	4.62
		1.80	4.35
		4.50	4.06
		7.20	3.50
635	9.3	0.0	4.42
		1.86	4.25
		4.65	3.72
		7.44	3.27
620	9.3	0.0	4.17
		1.86	3.97
		4.65	3.57
		7.44	3.12
600	9.0	0.0	3.71
		1.80	3.64
		4.50	3.43
		7.20	2.66
580	8.5	0.0	3.93
		1.70	3.88
		4.25	3.57
		6.80	2.80
560	7.7	0.0	3.54
		1.54	3.35
		3.85	3.43
		6.16	2.66
540	6.7	0.0	3.79
		1.34	3.57
		3.35	3.43
		5.36	2.87
520	6.5	0.0	2.96
		1.30	3.05
		3.25	3.05
		5.20	1.99
500	6.0	0.0	3.10
		1.20	2.98
		3.00	2.66
		4.80	2.29
470	4.7	0.0	2.95
		0.94	2.73
		2.35	2.54
		3.76	2.29
430	2.4	0.0	2.20
		1.20	2.24
		1.44	1.87
390	0.6		
380 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 7 LOW, CROSS SECTION 12

WATER SURFACE ELEVATION 717.44 FEET, DISCHARGE 10,400 CFS, 10 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
762 - Bank Intersects Water Surface			
758	3.0	0.0	1.47
		1.50	1.16
		1.80	1.25
750	3.9	0.0	2.69
		1.95	2.38
		2.34	2.29
735	5.3	0.0	3.57
		1.06	3.27
		2.65	3.19
		4.24	2.80
720	7.7	0.0	3.83
		1.54	3.64
		3.85	3.35
		6.16	2.87
705	9.1	0.0	4.04
		1.83	3.88
		4.55	3.64
		7.28	2.98
690	9.6	0.0	4.19
		1.92	4.15
		4.80	3.88
		7.68	2.98
675	9.0	0.0	5.03
		1.80	4.35
		4.50	3.97
		7.20	3.35
660	8.4	0.0	4.66
		1.68	4.35
		4.20	3.97
		6.72	3.57
645	8.5	0.0	4.58
		1.70	4.35
		4.25	4.15
		6.80	3.44
630	8.3	0.0	4.38
		1.66	4.25
		4.15	3.97
		6.64	3.19
615	8.1	0.0	4.50
		1.62	4.15
		4.05	3.88
		6.48	3.50
600	7.8	0.0	4.31
		1.56	3.97
		3.90	3.50
		6.24	3.35

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
585	7.4	0.0	4.01
		1.48	4.15
		3.70	3.72
		5.92	2.66
570	7.7	0.0	4.21
		1.54	3.97
		3.85	3.57
		6.16	3.19
555	7.4	0.0	4.03
		1.48	4.25
		3.70	3.43
		5.92	2.60
530	7.5	0.0	4.26
		1.50	3.97
		3.75	3.64
		6.00	3.27
500	7.8	0.0	3.98
		1.56	3.64
		3.90	3.27
		6.24	3.12
470	6.4	0.0	3.98
		1.28	3.64
		3.20	3.27
		5.12	3.12
440	6.7	0.0	3.53
		1.34	3.27
		3.35	2.87
		5.36	2.73
410	5.7	0.0	3.36
		1.14	3.12
		2.85	3.60
		4.56	2.60
380	5.0	0.0	2.95
		1.00	2.73
		2.50	2.80
		4.00	2.29
350	4.6	0.0	2.44
		0.92	2.54
		2.30	2.29
		3.68	1.61
320	3.5	0.0	2.35
		1.75	2.29
		2.10	2.00
280	2.8	0.0	2.40
		1.40	2.09
240	2.8	0.0	2.29
		1.40	2.04
190	1.8	0.0	1.80
		0.90	1.57
160 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 HIGH, CROSS SECTION 1

WATER SURFACE ELEVATION 728.79 FEET, DISCHARGE 3,960 CFS, 4 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
2 - Bank Intersects Water Surface			
8	5.5	0.0	2.11
		1.10	2.09
		2.75	1.64
		4.40	1.50
20	7.0	0.0	4.12
		1.40	3.88
		3.50	3.35
		5.60	3.12
35	7.7	0.0	4.50
		1.54	4.46
		3.85	4.15
		6.16	3.19
50	7.7	0.0	4.86
		1.54	4.46
		3.85	4.25
		6.16	3.80
65	7.4	0.0	4.35
		1.48	4.35
		3.70	4.15
		5.92	3.05
80	6.0	0.0	4.12
		1.20	3.88
		3.00	3.64
		4.80	3.12
95	4.0	0.0	3.75
		2.00	3.43
		2.40	3.19
110	4.3	0.0	3.85
		2.15	3.43
		2.58	3.27
125	3.9	0.0	3.51
		1.95	3.57
		2.34	2.98
140	3.2	0.0	3.75
		1.60	3.27
		1.92	3.19

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
155	3.1	0.0	3.51
		1.55	3.12
		1.86	2.98
170	3.7	0.0	3.38
		1.85	2.66
		2.22	2.87
185	4.1	0.0	2.64
		2.05	2.60
		2.46	2.24
200	3.5	0.0	2.69
		1.75	2.38
		2.10	2.29
215	3.6	0.0	2.64
		1.80	2.24
		2.16	2.24
230	3.4	0.0	2.64
		1.70	2.04
		2.04	2.24
245	3.5	0.0	2.20
		1.75	2.04
		2.10	1.87
260	3.6	0.0	2.40
		1.80	2.19
		2.16	2.04
275	3.8	0.0	2.74
		1.90	2.24
		2.28	2.33
290	3.0	0.0	1.56
		1.50	1.28
		1.80	1.33
293 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 HIGH, CROSS SECTION 6

WATER SURFACE ELEVATION 729.50 FEET, DISCHARGE 4,630 CFS, 4 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
1 - Bank Intersects Water Surface			
5	3.4	0.0	3.38
		1.70	2.80
		2.04	2.87
15	5.0	0.0	4.09
		1.00	3.97
		2.50	3.35
		4.00	2.98
30	6.3	0.0	4.32
		1.26	4.15
		3.15	3.64
		5.04	3.19
45	7.5	0.0	4.58
		1.50	4.35
		3.75	4.35
		6.00	3.43
60	6.6	0.0	4.14
		1.32	4.06
		3.30	3.97
		5.28	2.98
75	5.6	0.0	4.05
		1.12	3.97
		2.80	3.64
		4.48	2.92
90	5.9	0.0	4.08
		1.18	3.88
		2.95	3.43
		4.72	3.05
105	5.3	0.0	4.42
		1.06	4.15
		2.65	3.97
		4.24	3.37
120	4.7	0.0	4.81
		0.94	4.46
		2.35	4.25
		3.76	3.72
135	4.6	0.0	4.42
		0.92	3.88
		2.30	3.88
		3.68	3.64
150	4.2	0.0	4.78
		2.10	4.46
		2.52	4.06

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
165	3.8	0.0	4.20
		1.90	3.97
		2.28	3.57
180	3.6	0.0	3.85
		1.80	3.37
		2.16	3.27
195	3.2	0.0	3.67
		1.60	3.05
		1.92	3.12
210	3.0	0.0	3.59
		1.50	2.98
		1.80	3.05
225	2.7	0.0	4.04
		1.35	3.35
		1.62	3.43
240	2.8	0.0	3.85
		1.40	3.19
		1.68	3.27
255	3.2	0.0	3.67
		1.60	3.72
		1.92	3.12
270	3.6	0.0	3.94
		1.80	3.35
		2.16	3.35
285	3.0	0.0	3.75
		1.50	3.19
		1.80	3.19
300	3.6	0.0	1.98
		1.80	1.87
		2.16	1.68
302 - Bank Intersects Water Surface			

VELOCITY DATA, SITE 3 HIGH, CROSS SECTION 12

WATER SURFACE ELEVATION 729.96 FEET, DISCHARGE 4,770 CFS, 4 SEPTEMBER 1980

Distance from Initial Pt. (ft.)	Depth to River Bottom (ft.)	Observation Depth (ft.)	Velocity at Pt. (ft/s)
5 - Bank Intersects Water Surface			
10	4.0	0.0	2.35
		2.00	1.97
		2.40	2.00
25	4.0	0.0	3.13
		2.00	3.19
		2.40	2.66
40	4.2	0.0	3.85
		2.10	3.50
		2.52	3.27
55	4.2	0.0	3.67
		2.10	3.50
		2.52	3.12
70	3.8	0.0	4.47
		1.90	3.80
		2.28	3.43
85	3.9	0.0	4.47
		1.95	3.80
		2.34	3.80
100	3.9	0.0	4.20
		1.95	4.06
		2.34	3.57
115	4.0	0.0	4.20
		2.00	3.72
		2.40	3.57
130	4.4	0.0	4.88
		2.20	4.57
		2.64	4.15
145	4.8	0.0	4.93
		0.96	4.74
		2.40	4.57
		3.84	3.64
160	5.3	0.0	5.18
		1.06	4.74
		2.65	4.46
		4.24	4.06
175	5.4	0.0	5.24
		1.08	4.85
		2.70	4.64
		4.32	4.06
190	5.4	0.0	5.02
		1.08	4.74
		2.70	4.74
		4.32	3.80
205	4.8	0.0	5.25
		0.96	4.95
		2.40	4.46
		3.84	3.97
220	5.1	0.0	5.25
		1.02	4.95
		2.55	4.95
		4.08	3.97
235	5.4	0.0	5.25
		1.08	4.95
		2.70	4.57
		4.32	3.97
250	5.6	0.0	4.89
		1.12	4.74
		2.80	4.35
		4.48	3.57
265	5.8	0.0	3.79
		1.16	3.64
		2.90	3.43
		4.64	2.80
275	4.4	0.0	1.36
		2.20	1.72
		2.64	1.16
282 - Bank Intersects Water Surface			

PEARL RIVER AT
MONTICELLO, MISSISSIPPI

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

PEARL RIVER AT MONTICELLO, MISSISSIPPI
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Section 32 Streambank Erosion control Demonstration Project, located on the Pearl River (Mile 191) at Monticello, Mississippi, as shown on Plates 1 and 2.

2. Purpose. This report describes three bank erosion problems, the types of protection used and a limited performance evaluation of the constructed project.

3. Problem. Bank caving along the Pearl River at Monticello has been a major concern. Several structures including a city maintenance shop, the county health clinic, the city library and a boat ramp were threatened by steady erosion. This erosion is directly attributable to direct current attack, hydrostatic forces induced in slopes after periods of rapid recession, and to local drainage.

4. Authority. The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, PL 93-251 authorizes the Secretary of the Army, acting through the Chief of Engineers, to establish and conduct a national streambank erosion prevention and demonstration program.

II. HISTORICAL DESCRIPTION

5. Stream Morphology. The Pearl River drains approximately 8760 square miles in South Central Mississippi. The river flows in a generally southern direction from its origin northeast of Jackson, to the Mississippi Sound at the Louisiana-Mississippi boundary. The approximate river slope at Monticello is between .6 and .8 feet per mile.

6. Hydrologic Characteristics. The weather conditions at Monticello are typical of the Central Mississippi region. The average rainfall is 56 inches per year and the average temperature, 65°F. The area averages 38 days per year when .5 inches of rainfall or greater is received. In

general, the major floods occur during the spring and are associated with frontal systems. The maximum recorded discharge of 122,000 cfs occurred on 20 April 1979 with the stage at elevation 192.7 feet NGVD. The next largest flood, with an approximate discharge of 100,000 cfs and a stage at elevation 191.7 feet NGVD, occurred in April 1902. This data is obtained from a USGS stream gage on the downstream side of the Highway 84 bridge, approximately one mile east of Monticello. Data for the station is provided in the annual "Water Resource Data for Mississippi" published by USGS. Precipitation and temperature stations exist at Monticello, Brookhaven,Columbia, Hattiesburg, Jackson and many other cites in the immediate area. This information is in the "Climatological Data Annual Summary, Mississippi," published by National Weather Service.

7. Geology. The site is in the Coastal Plain Geophysical Province, located in Lawrence County, Mississippi, along the Pearl River. The overburden is typically a yellowish brown loam containing much silt and clay which normally ranges in thickness from 3 to 5 feet. The Hattiesburg and Pascagoula formations outcrop in the river, along roads and in other cuts throughout the area. The formations are normally composed of sandy clay rock of light gray to brownish gray color with a cementing material of reddish brown to purplish iron oxide, but occasionally a hard thin layer of siltstone or limestone may be found. The formation is characterized by oyster fossils, indicating the marine or estuarine character of this deposit. The formations are of the oligocene and miocene geologic age. The logs of borings presented in Appendix A show the results of five borings. Holes No. 55-1-77 and 55-2-77 represent conditions at Sites 1 and 2 respectively, while Holes No. 55-1-78, 55-2-78 and 55-3-78 represent conditions at Site 3. Locations of these borings are shown on Plates 3 and 4.

8. Channel Conditions. Sites 1 and 2 shown on Plates 2 and 3 are on the outside of a 135^{0} bend situated on a bank 25-30 feet above low water. Site 3 (Plate 4) is on the outside and downstream of a 90^{0} bend on a bank which extends approximately 8 feet above low water. The upstream bend, near Sites 1 and 2, is characteristic of "abrupt angle" bends wherein the majority of the current attack is concentrated on

resistant material located at the base of the bluff. These type bends are also characterized by comparatively long straight reaches downstream from the bend.

9. Environmental Consideration. Implementation of the demonstration project has resulted in the stabilization, to date, of the existing bank erosion. Approximately 0.8 acres of eroded bank have been covered by the three stabilization methods. The biological productivity of the three areas involved, due to the steep slopes and exposed soil, is of low value. Therefore, the impact on it should be insignificant. The lower portions of the three protective measures extend into the water and provide a different substrate than that which existed prior to construction. Both the U.S. Fish and Wildlife Service and the Mississippi Game and Fish Commission were contacted and advised of the project plans. Both agencies had no objections to the project and felt that the fish habitats provided by each of the three different stabilization methods would be more advantageous than the natural sandy bottoms. They were also of the opinion that wildlife habitat would not be significantly affected by the project.

10. Demonstration Site Description.

a. Hydraulic Characteristics. Stage-frequency, stage-discharge, and average velocity-water surface elevation data are shown on Plates 6, 7 and 8, respectively. This data was provided by information taken from the previously mentioned USGS gaging station at the Highway 84 bridge. A typical cross-section (pre-construction) of the Pearl River in the project area is shown on Plate 5.

b. Overbank Description and Erosion Trends. The type of material discussed in the Geology Section (paragraph 7) is typical of the overbank material at Monticello. The excavation of these beds normally can be accomplished with conventional mechanical equipment. Although the base of the slope is highly resistant to erosion, the upper slope is, in several isolated reaches, unstable due to: (a) current attack on the erodible upper slope material, (b) loss of vegetative cover as a result of human activity, (c) hydrostatic forces from ground water, and (d) local drainage. Also, after a heavy rainfall in the upper portion of the basin, stages at Monticello increase as this runoff is passed downstream. After being at near peak stages for long periods of time, the river is subject to relatively sudden stage recession of 10 to 15 feet. As a consequence, the saturated stream bank sloughs into the river.

III. DESIGN AND CONSTRUCTION

11. Project Design. This demonstration project consists of three sites using differing bank stabilization measures. The three sites are located on the Pearl River near Monticello as shown on Plates 1 thru 4. Site 1 utilizes dumped rubble on a slope of 1 to 1 back to existing ground. The top width of the rubble is 10 feet, located at elevation 190.0. Site 2 utilizes a mat constructed of used tires. The tires are banded together and anchored at 10-foot centers to concrete anchors on the bank. The top elevation of the tire mat is also at elevation 190.0. Site 3 consists of a mat composed of concrete blocks, placed atop plastic filter cloth, and anchored with steel stakes. The mat is covered with topsoil and grassed. The top elevation for Site 3 is elevation 180.0 with riprap placed at the toe of the mat. Typical cross-sections for Sites 1, 2 and 3 are shown on Plate 9. Before-and-after photographs and after flooding photographs for each site are shown on Plates 10 through 15.

12. Construction Details.

a. Delays. Bids for construction were opened on four different occasions, dating back to the fall of 1978, before an awardable bid was received in the spring of 1980. This created an extremely long delay between final design and completion of construction and reduced the monitoring period to date to six months, starting in December 1980. (At present this program will be terminated as of 31 December 1981 in accordance with Public Law 93-251.)

b. Completion Date and Cost. All three sites were completed in December 1980 at a total cost of approximately $353,000. The unit costs per linear foot of Sites 1, 2 and 3 were $381, $266 and $471, respectively.

c. Other Methods Considered. Three other methods of protection were considered for this project, but were not chosen for various reasons. They were: (1) Fabriform, Hydromat, Gobi Blocks or other forms of articulated mat, (2) low bulkhead with vegetative protection along the upper slope, and (3) soil cement.

IV. MONITORING

13. Description of Monitoring Activities and Cost.

a. Cross Section. Cross sections will be taken annually at permanent survey points established during the pre-construction phase. The cost of this surveying is estimated to be $9,000 annually.

b. Photographic Coverage. Aerial photography of the three sites will be taken annually or after major flooding (Plate 16). Ground level photos will be taken during each site visit with total annual cost for photography estimated to be $1,500. (Plates 10 thru 15 show photos taken during various stages of the project.)

c. Visual Inspection. Site visits have been made prior to, and immediately after construction, and presently visits are planned to be made twice yearly at an annual cost of about $750.

14. Conclusions. The "final inspection" after construction was made in early December 1980. This trip concluded the construction phase and initiated the monitoring effort. During late April 1981 a minor flood occurred at Monticello. After this flood, a site visit was made to monitor and note any damage. Except for minor erosion at the toe of Site 3, all three sites appeared to be in excellent condition and have halted the erosion, to date, at each location.

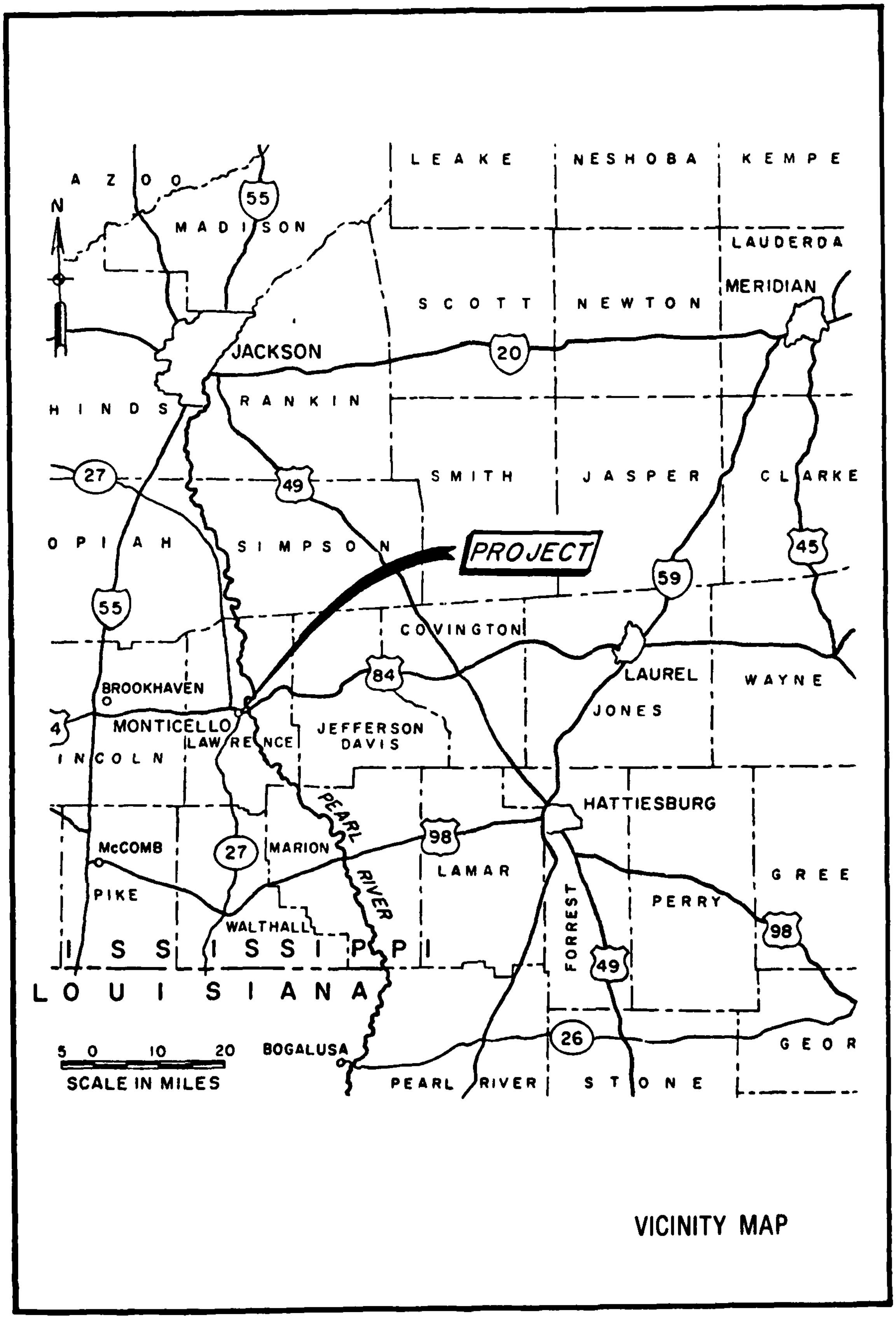
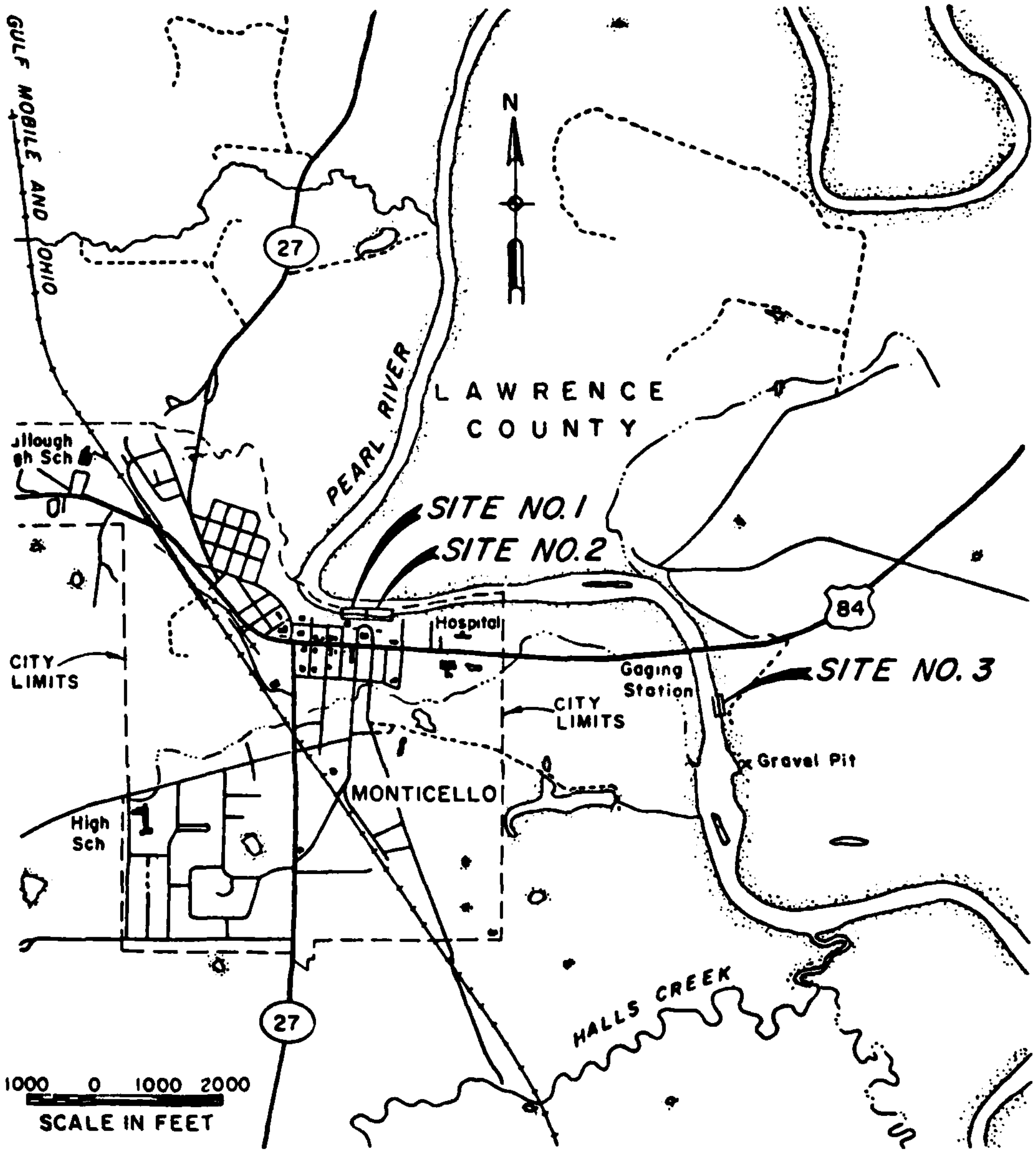
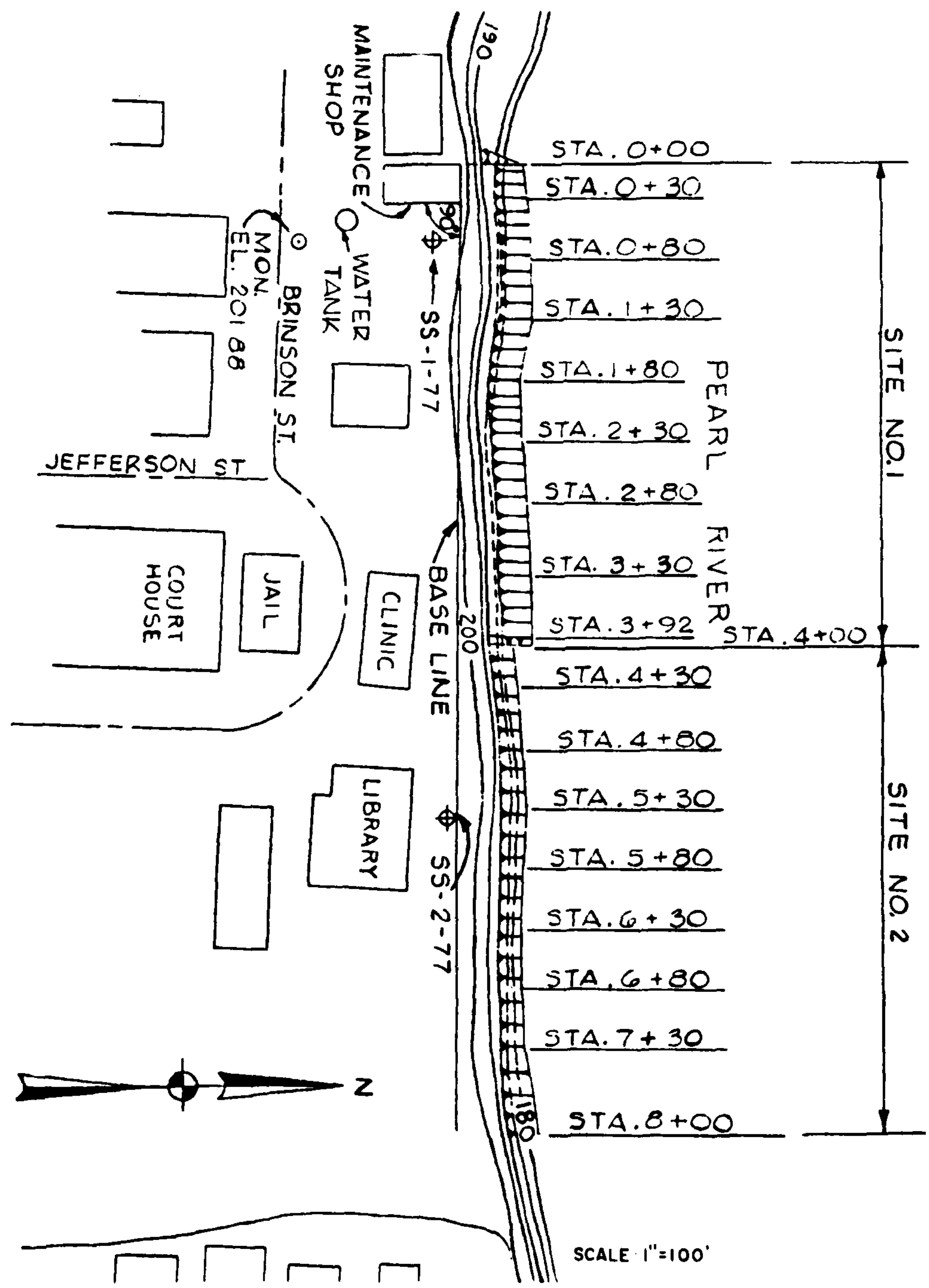

LEAKE
NESHOBA
KEMPE
AZOO
MADISON
55
LAUDERDA
MERIDIAN
SCOTT
NEWTON
JACKSON
20
HINDS
RANKIN
27
49
SMITH
JASPER
CLARKE
OPIAH
SIMPSON
PROJECT
45
59
55
COVINGTON
LAUREL
WAYNE
84
BROOKHAVEN
JONES
MONTICELLO
LAWRENCE
JEFFERSON DAVIS
INCOLN
PEARL
HATTIESBURG
98
McCOMB
27
MARION
LAMAR
RIVER
GREE
PIKE
PERRY
WALTHALL
98
FORREST
ISSISSIPPI
49
LOUISIANA
5 0 10 20
SCALE IN MILES
BOGALUSA
26
GEOR
PEARL RIVER
STONE
VICINITY MAP

PLATE 1



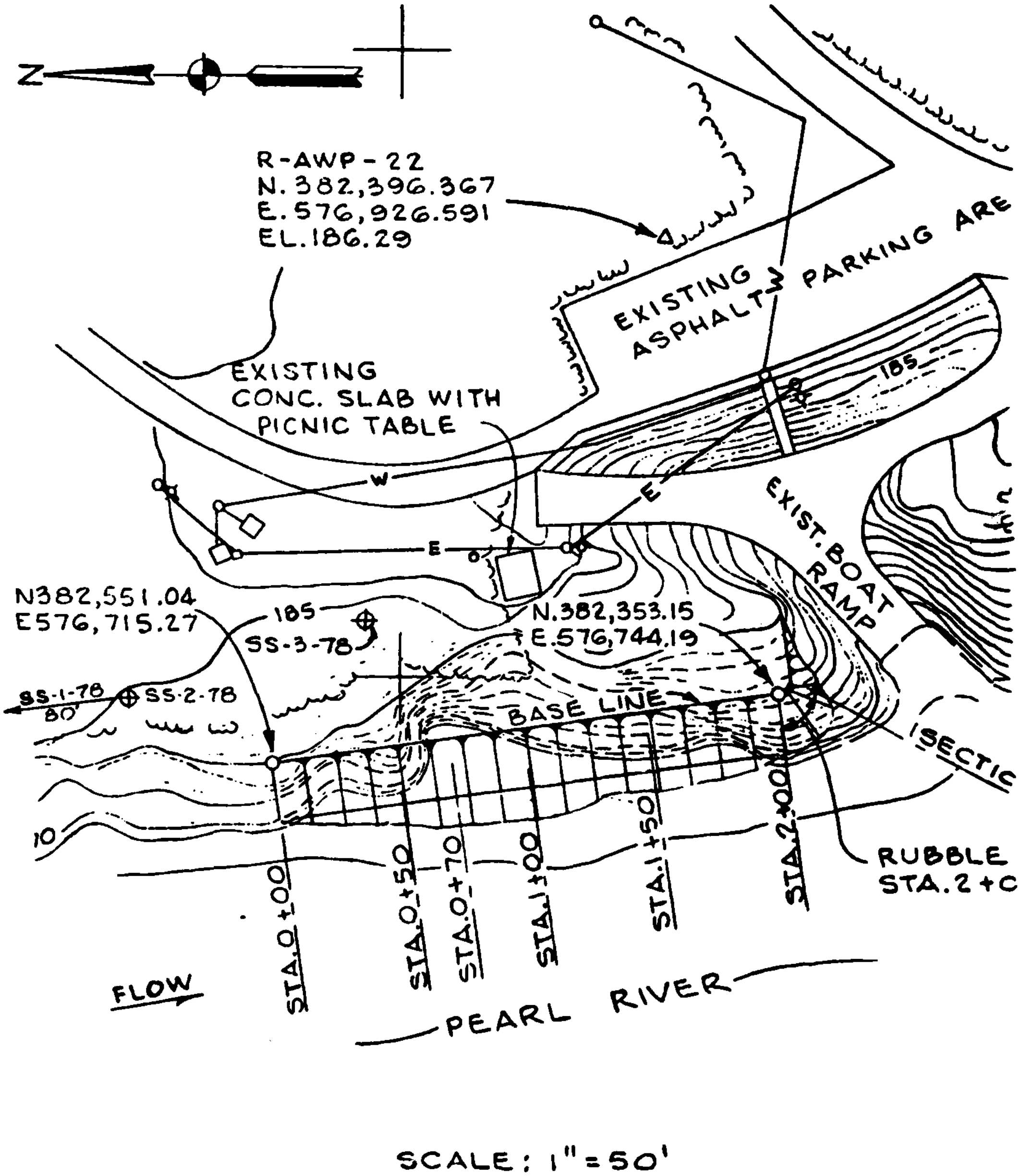
SITE PLAN

PLATE 2



SITES 1 AND 2
PLAN

PLATE 3



SITE 3
PLAN

PLATE 4

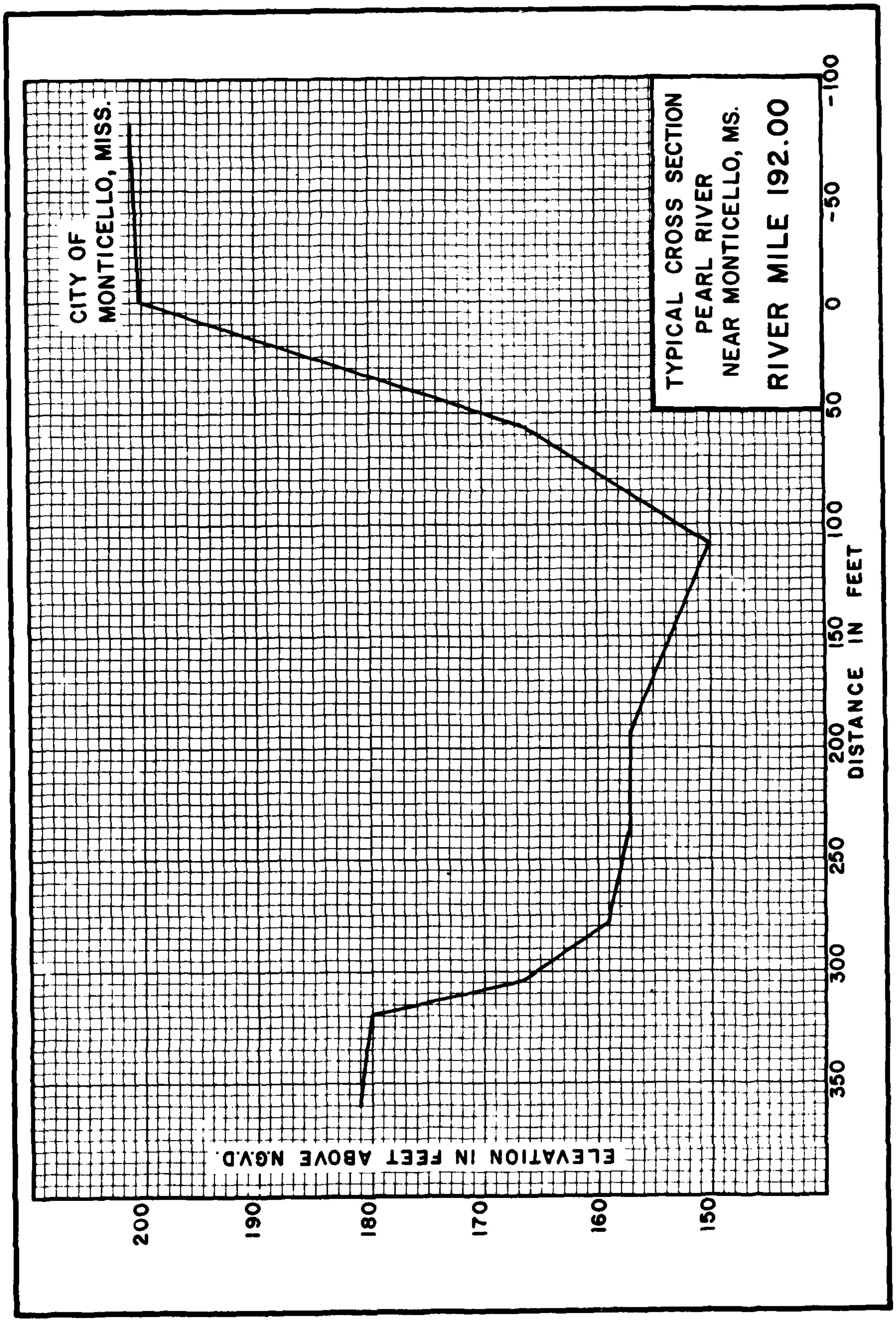

TYPICAL CROSS SECTION
PEARL RIVER
NEAR MONTICELLO, MS.
RIVER MILE 192.00
CITY OF
MONTICELLO, MISS.
DISTANCE IN FEET
-100
-50
0
50
100
150
200
250
300
350
ELEVATION IN FEET ABOVE N.G.V.D.
200
190
180
170
160
150

PLATE 5

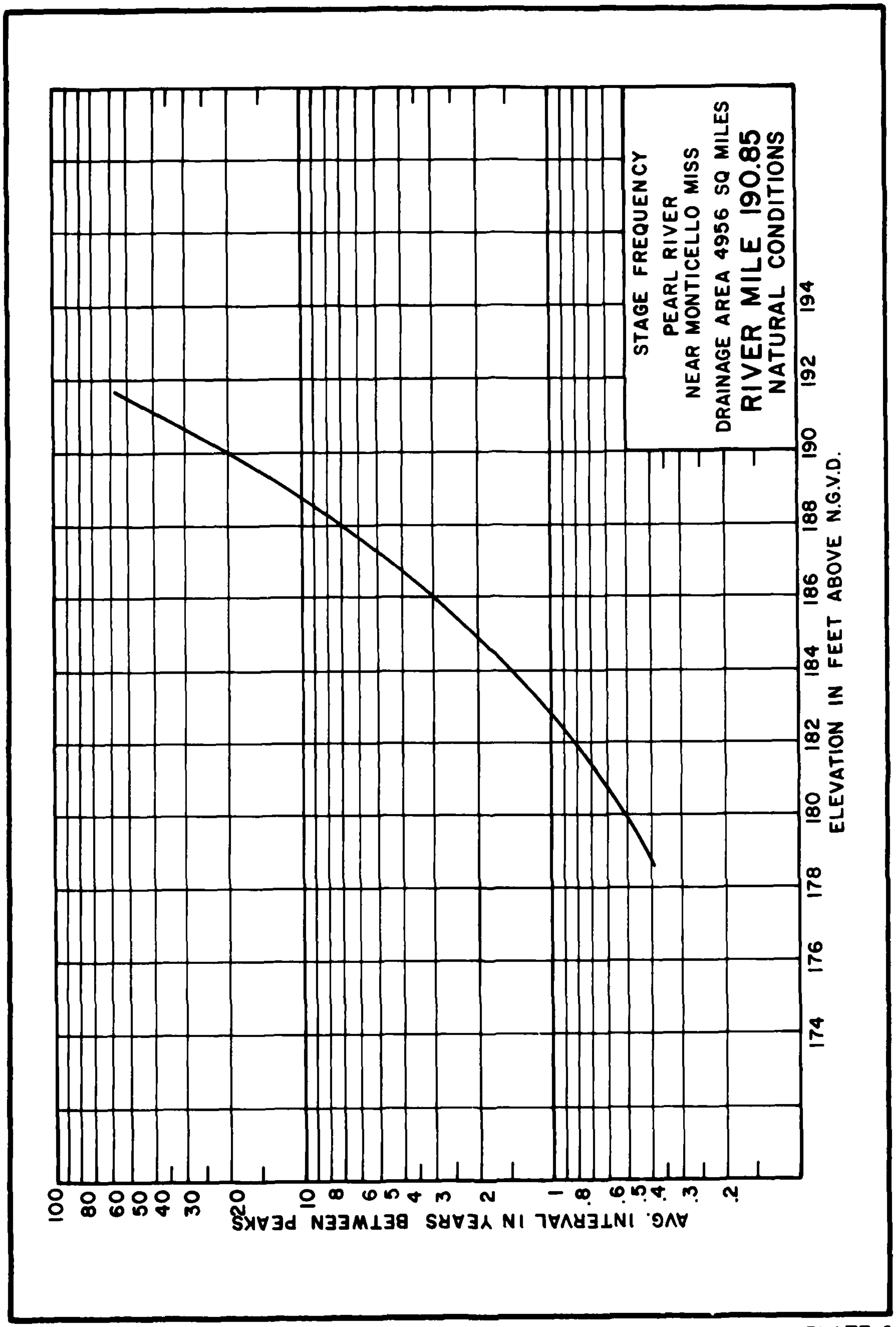

STAGE FREQUENCY
PEARL RIVER
NEAR MONTICELLO MISS
DRAINAGE AREA 4956 SQ MILES
RIVER MILE 190.85
NATURAL CONDITIONS
174 176 178 180 182 184 186 188 190 192 194
ELEVATION IN FEET ABOVE N.G.V.D.
100 80 60 50 40 30 20 10 8 6 5 4 3 2 1 .8 .6 .5 .4 .3 .2
AVG. INTERVAL IN YEARS BETWEEN PEAKS

PLATE 6

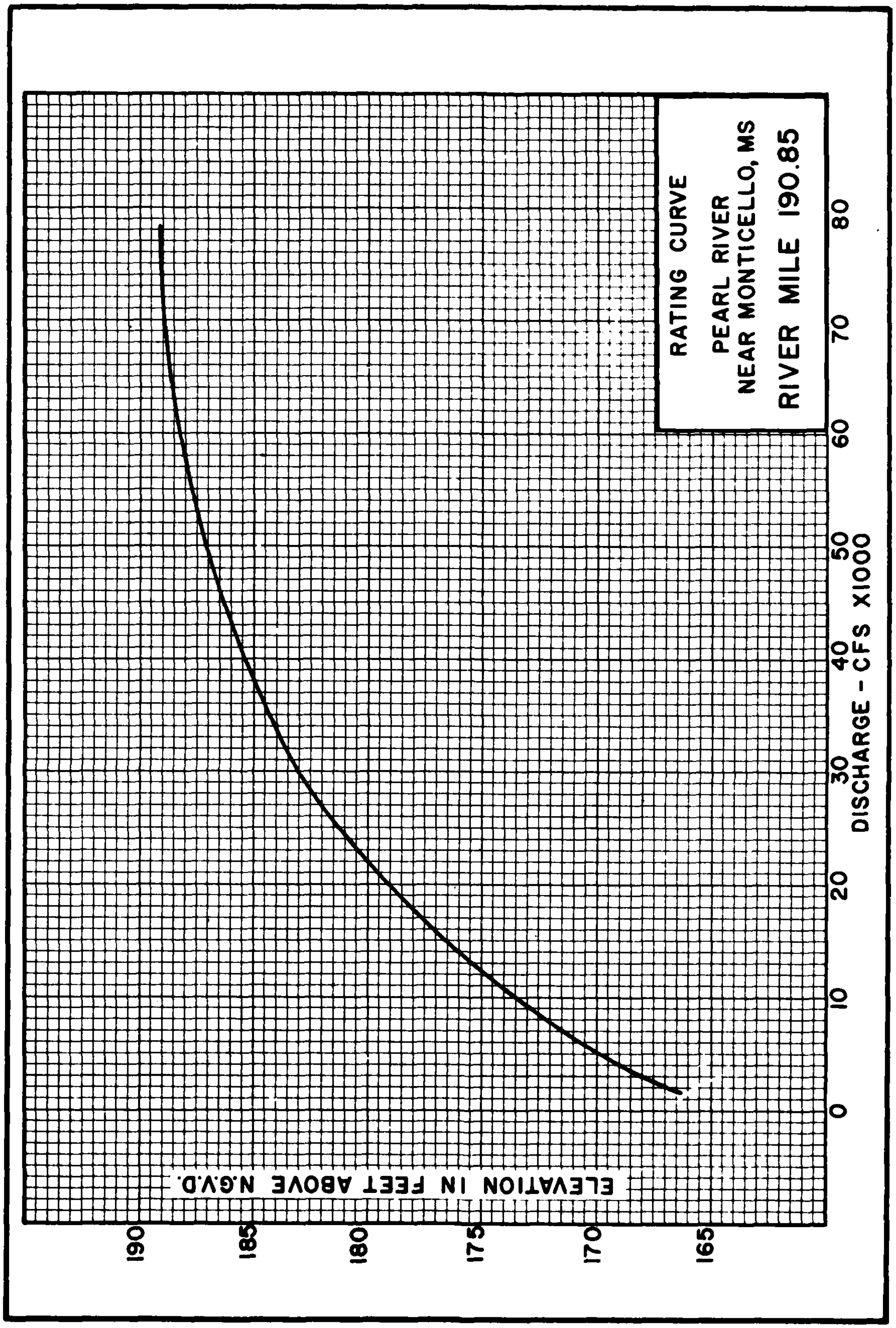

RATING CURVE
PEARL RIVER
NEAR MONTICELLO, MS
RIVER MILE 190.85
ELEVATION IN FEET ABOVE N.G.V.D.
190
185
180
175
170
165
DISCHARGE - CFS X1000
0
10
20
30
40
50
60
70
80

PLATE 7

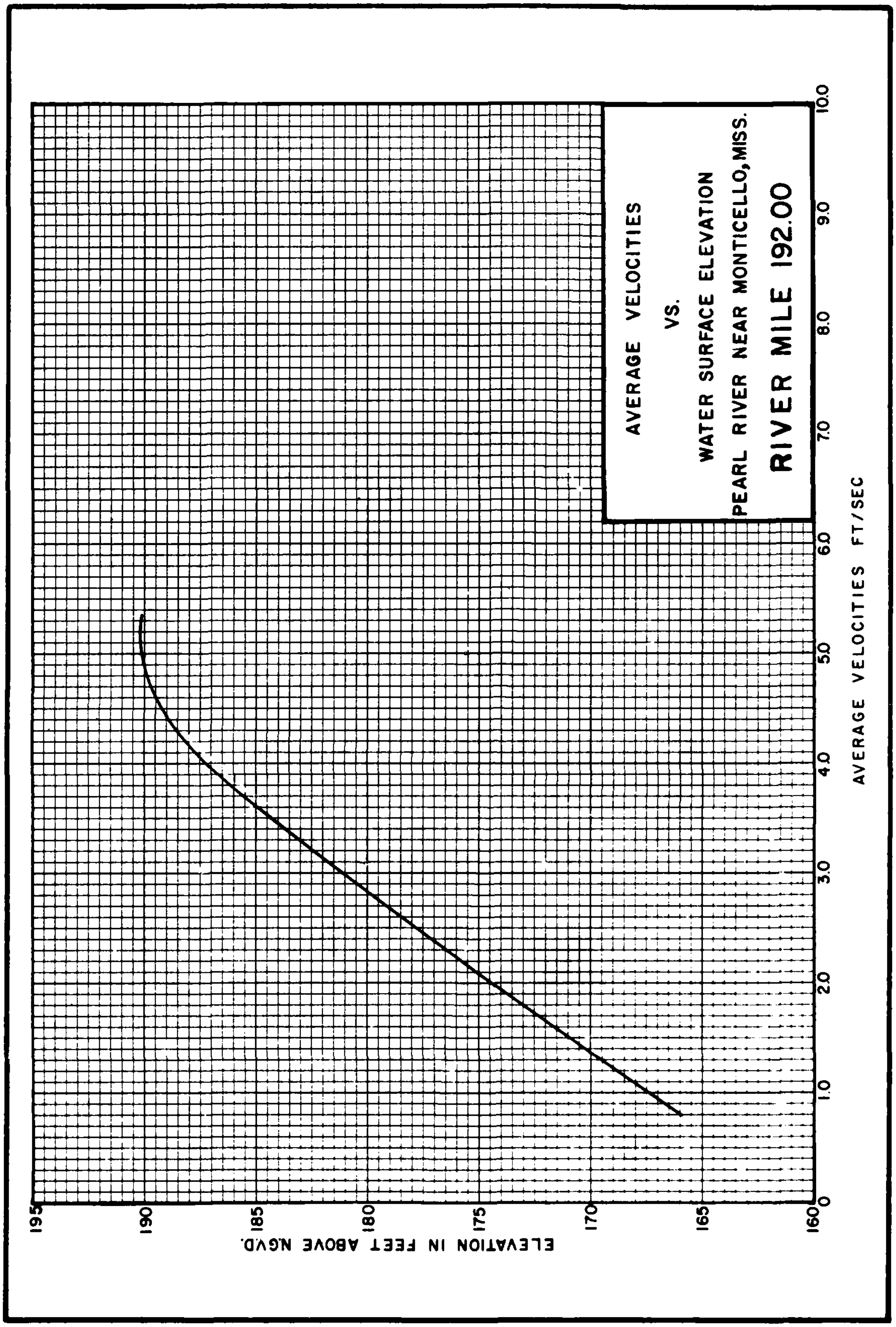

AVERAGE VELOCITIES
VS.
WATER SURFACE ELEVATION
PEARL RIVER NEAR MONTICELLO, MISS.
RIVER MILE 192.00
AVERAGE VELOCITIES FT/SEC
ELEVATION IN FEET ABOVE N.G.V.D.
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
160
165
170
175
180
185
190
195

PLATE 8

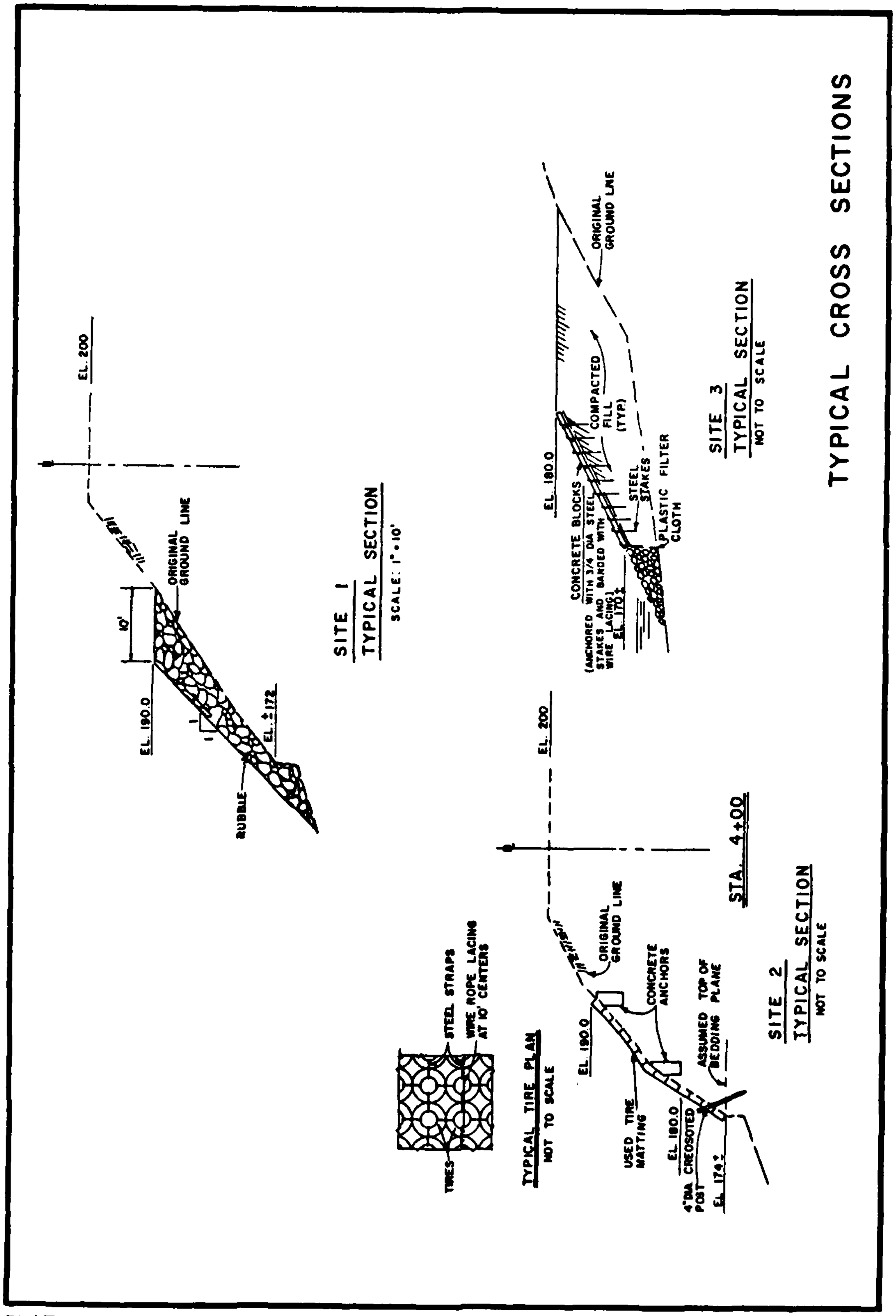

TYPICAL CROSS SECTIONS
EL. 200
ORIGINAL GROUND LINE
10'
EL. 190.0
EL. ±172
RUBBLE
SITE 1
TYPICAL SECTION
SCALE: 1"=10'
ORIGINAL GROUND LINE
COMPACTED FILL (TYP)
EL. 180.0
CONCRETE BLOCKS
(ANCHORED WITH 3/4 DIA STEEL STAKES AND BANDED WITH WIRE LACING)
EL. 170±
STEEL STAKES
PLASTIC FILTER CLOTH
SITE 3
TYPICAL SECTION
NOT TO SCALE
EL. 200
STA. 4+00
ORIGINAL GROUND LINE
CONCRETE ANCHORS
TOP OF PLANE
ASSUMED BEDDING PLANE
EL. 190.0
USED TIRE MATTING
EL. 180.0
4" DIA. CREOSOTED POST
EL. 174±
SITE 2
TYPICAL SECTION
NOT TO SCALE
TIRES
STEEL STRAPS
WIRE ROPE LACING AT 10' CENTERS
TYPICAL TIRE PLAN
NOT TO SCALE

PLATE 9

BEFORE CONSTRUCTION, MAY 1980. NOTE UPSTREAM VIEW, TOP OF BANK, APPROXIMATELY 200 FEET N.G.V.D., 19 FEET ABOVE WATER SURFACE.

SITE 1

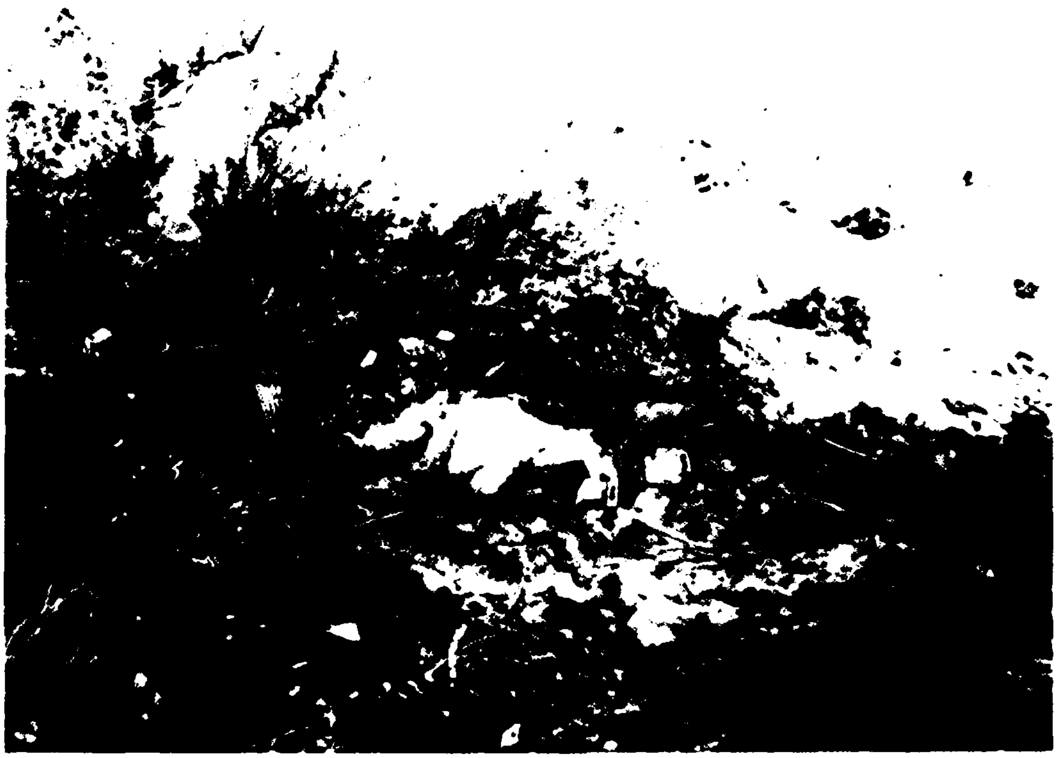
PLATE 10

AFTER CONSTRUCTION, DECEMBER 1980. NOTE UPSTREAM VIEW, TOP OF RUBBLE, 190 FEET N.G.V.D., 23 FEET ABOVE WATER SURFACE.

AFTER FLOOD OF MAY 1981. NOTE UPSTREAM VIEW, TOP OF RUBBLE 190 FEET N.G.V.D., 18 FEET ABOVE WATER SURFACE.

SITE 1

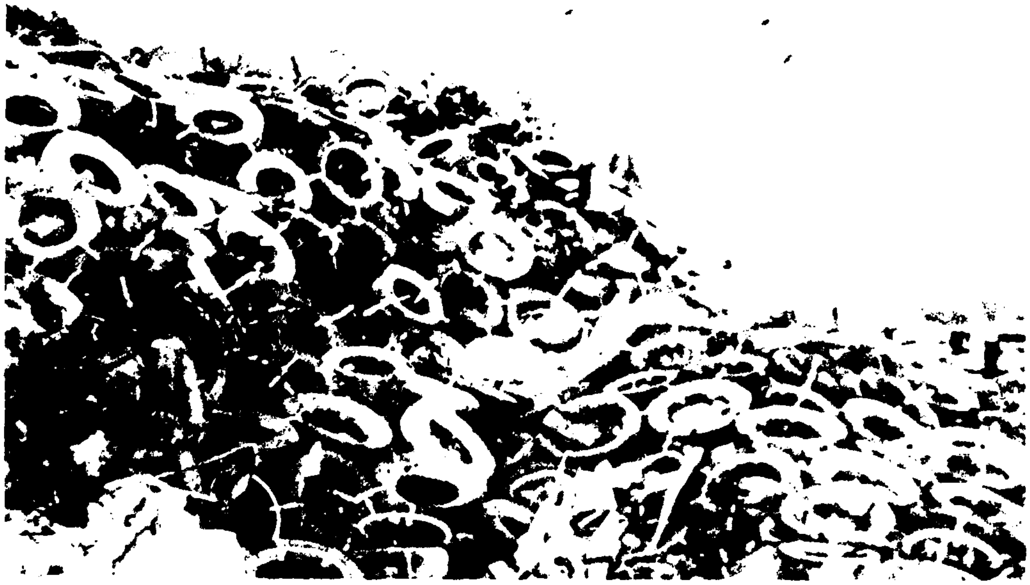
PLATE 11



BEFORE CONSTRUCTION, MAY 1980. NOTE FLOW TO THE RIGHT TOP OF BANK APPROXIMATELY 200 FEET N.G.V.D., 19 FEET ABOVE WATER SURFACE.

SITE 2

PLATE 12



AFTER CONSTRUCTION, DECEMBER 1980. NOTE FLOW TO THE RIGHT, TOP OF STRUCTURE 190 FEET N.G.V.D., 23 FEET ABOVE WATER SURFACE.

AFTER FLOOD OF MAY 1981. NOTE FLOW TO THE RIGHT, TOP OF STRUCTURE 190 FEET N.G.V.D., 18 FEET ABOVE WATER SURFACE.

SITE 2

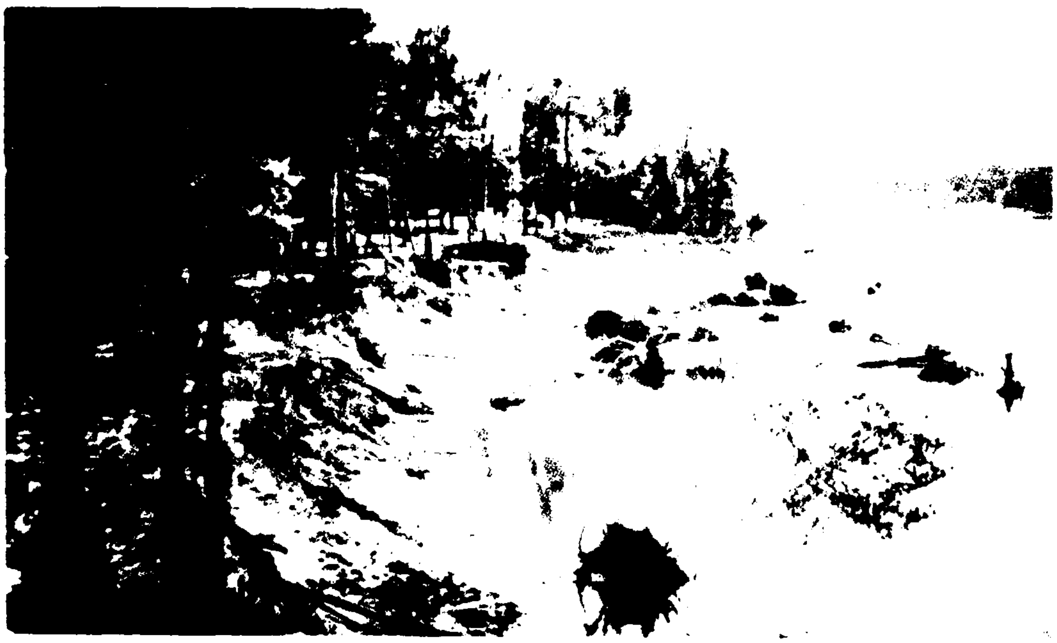
PLATE 13

BEFORE CONSTRUCTION, MAY 1980. NOTE DOWNSTREAM VIEW, TOP OF BANK APPROXIMATELY 185 FEET N.G.V.D., 10 FEET ABOVE WATER SURFACE.

SITE 3

PLATE 14

AFTER CONSTRUCTION, DECEMBER 1980. NOTE DOWNSTREAM VIEW, TOP OF STRUCTURE 180 FEET N.G.V.D., 8 FEET ABOVE WATER SURFACE.



AFTER FLOOD OF MAY 1981. NOTE DOWNSTREAM VIEW, TOP OF STRUCTURE 180 FEET N.G.V.D., 13 FEET ABOVE WATER SURFACE.

SITE 3

PLATE 15

SITE 2 SITE 1

SITES 1 AND 2. MAY 1981. NOTE FLOW TO THE LEFT, TOP OF BANK APPROXIMATELY 200 FEET N.G.V.D., 33 FEET ABOVE WATER SURFACE.

SITE 3. MAY 1981. NOTE FLOW TO THE RIGHT, TOP OF BANK APPROXIMATELY 185 FEET N.G.V.D., 18 FEET ABOVE WATER SURFACE.

SITES 1, 2, AND 3
AERIAL PHOTOGRAPHY

PLATE 16

LOGS OF BORINGS

APPENDIX "A" - LOGS OF BORINGS

GENERAL NOTES:

Boring logs shown on the following sheets shall not be copied or altered.

Ground water depths shown on the boring logs represent ground water surfaces encountered on the dates shown. Absence of water surface data on certain borings implies that no ground water data is available, but does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of these borings.

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local minor variations in characteristics of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially from the description shown with the logs or profiles.

Soils are classified in accordance with the Unified Soil Classification System, Technical Memorandum No. 3-357 dated April 1960 for civil projects and Military Standard 619B dated 12 June 1968 for military projects.

Driving resistances are shown graphically. Blows per foot are determined with a standard split spoon sampler (1-3/8" I.D., 2" O.D.) and a 140-lb. driving hammer with a 30" drop unless otherwise noted on the boring logs.

For location of borings see Plates 3 and 4.

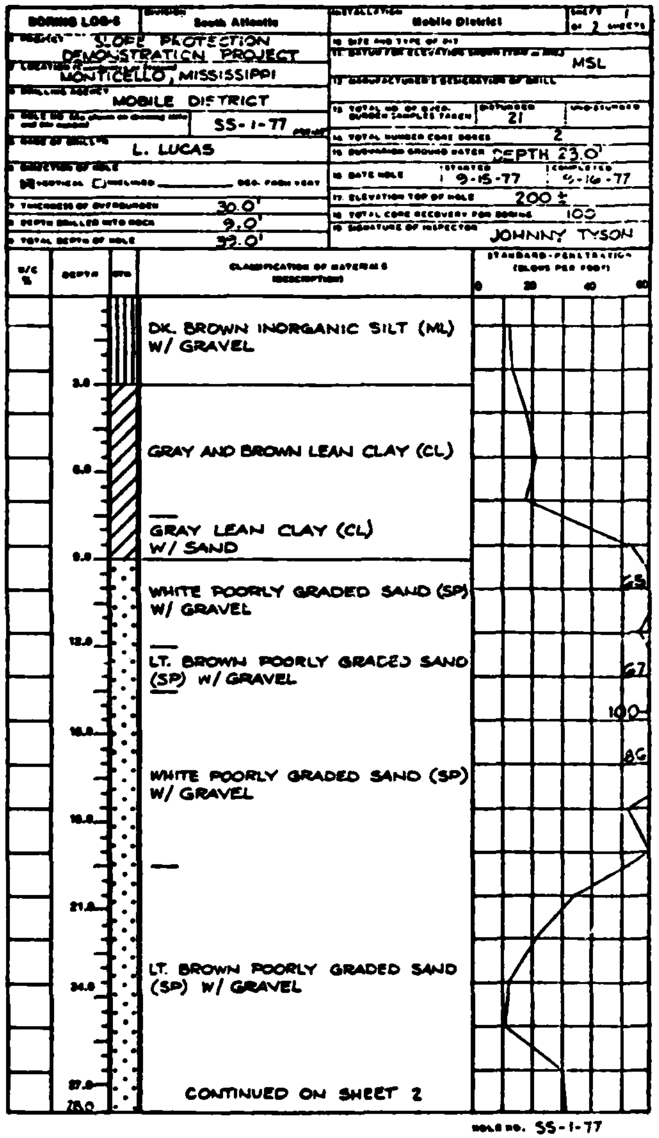
SPECIAL NOTE:

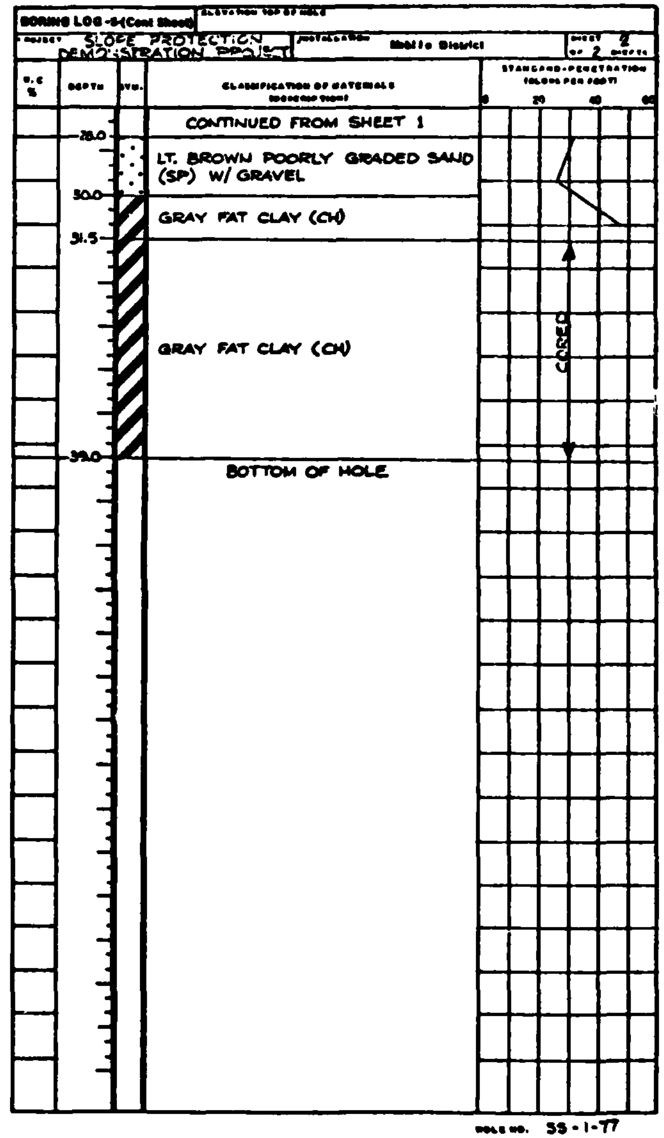
Water table shown is an approximation of the water elevation on the date shown. The water elevation may vary and may reach ground surface. Seepage above the water table can be expected at any time. Any conclusions drawn by the Contractor shall be the Contractor's sole responsibility.

LEGEND

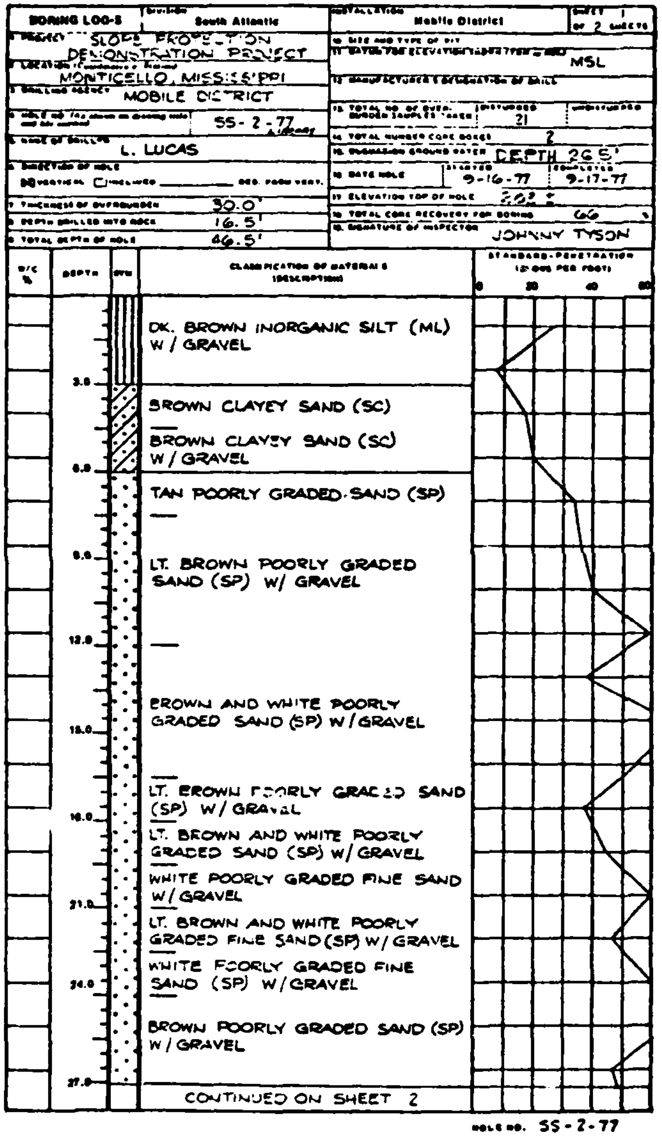
GW	Well graded gravels or gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
GM	Silty gravels, gravel-sand-silt mixtures.
GC	Clayey gravels, gravel-sand-clay mixtures.
SW	Well graded sands or gravelly sands, little or no fines.
SP	Poorly graded sands or gravelly sands, little or no fines.
SM	Silty sands, sand-silt mixtures.
SM-H	Same as above with high liquid limit.
SC	Clayey sands, sand-clay mixtures.
SC-H	Same as above with high liquid limit.
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
OH	Organic clays of medium to high plasticity, organic silts.
OL	Organic silts and organic silt-clays of low plasticity.
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
CH	Inorganic clays of high plasticity, fat clays.
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
PT	Peat and other highly organic soils.
	Sandstone
	No sample or recovery.

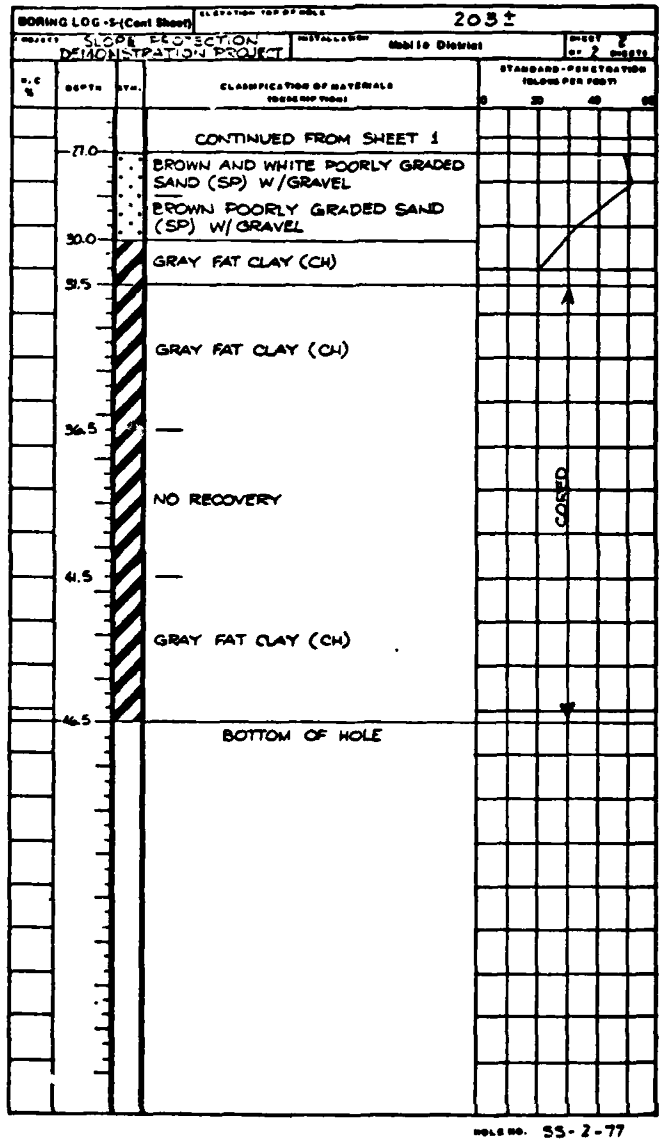
LOGS OF BORINGS





LOGS OF BORINGS





LOGS OF BORINGS

BORING LOG — Division: South Atlantic — Installation: Mobile District — Sheet 1 of 2 sheets

Project	SLOPE PROTECTION DEMONSTRATION PROJECT
Location	MONTICELLO, MS
Drilling agency	MOBILE DISTRICT
Hole no.	SS-1-78
Name of driller	BROWN
Direction of hole	Vertical
Total depth of hole	45.0'
Datum for elevation shown	MSL
Elevation ground water	DEPTH 15.0'
Date hole	Started 2-8-78; Completed 2-13-78
Elevation top of hole	185 ±
Signature of inspector	CHILDERS

W/C %	Depth	Classification of materials (Description)
		BROWN SILTY SAND (SM) W/SOME ROOTS
12	3.0	BROWN SILTY SAND (SM)
	4.5	BROWN CLAYEY SAND (SC)
	6.0	BROWN FINE SANDY CLAY (CL)
21	7.5	LT. BROWN LEAN CLAY (CL) W/LITTLE SAND
	9.0	GRAYISH TAN CLAYEY FINE SAND (SC)
	10.5	TAN SILTY FINE SAND (SM)
18	12.0	TAN-LTGRAY SANDY LEAN CLAY (CL)
	13.5	
	15.0	TAN POORLY GRADED SILTY SAND (SP-SM) W/MUCH GRAVEL
		TAN-WHITE POORLY GRADED GRAVEL (GP)
	18.0	LT. BROWN POORLY GRADED GRAVEL (GP) NON-PLASTIC
	21.0	WHITE POORLY GRADED GRAVEL (GP)
	24.0	BROWN-WHITE POORLY GRADED SANDY GRAVEL (GP)
	27.0	BROWN-WHITE POORLY GRADED GRAVEL (GP) NON-PLASTIC
		CONTINUED ON SHEET 2

Standard-penetration (blows per foot): 0 20 40 60

HOLE NO. SS-1-78

BORING LOG (Cont Sheet) — Elevation top of hole: 185 ± — Project: SLOPE PROTECTION — Installation: Mobile District — Sheet 2 of 2 sheets

W/C %	Depth	Classification of materials (Description)
	27.0	BLUISH GRAY FINE SANDY FAT CLAY (CH)
	28.5	GRAY-LT. BROWN LEAN CLAY (CL) W/LITTLE SAND
	30.0	BLUISH GRAY-TAN SILTY FAT CLAY (CH)
	36.0	TAN-GRAY LEAN CLAY (CL) LL=42, PL=19, PI=23
		TAN-GRAY LEAN CLAY (CL)
	39.0	BLUISH GRAY-TAN SILTY FAT CLAY (CH)
		GRAY-LT. BROWN FAT CLAY (CH) W/TR. SAND
		BLUISH GRAY-TAN-LT. PURPLE SILTY FAT CLAY
	45.0	BOTTOM OF HOLE N 382,683 E 576,720 NOTE: GROUTED HOLE WITH 4 BAGS CEMENT

Standard-penetration (blows per foot): 0 20 40 60

HOLE NO. SS-1-78

LOGS OF BORINGS

BORING LOG-S — Division: South Atlantic — Installation: Mobile District — Sheet 1 of 2 sheets

1. Project	SLOPE PROTECTION DEMONSTRATION PROJECT
2. Location	MONTICELLO, MS
3. Drilling agency	MOBILE DISTRICT
4. Hole No.	SS-2-78
5. Name of driller	BROWN
6. Direction of hole	☒ Vertical ☐ Inclined ___ deg. from vert.
7. Thickness of overburden	
8. Depth drilled into rock	
9. Total depth of hole	42.5'
10. Size and type of bit	
11. Datum for elevation shown	MSL
12. Manufacturer's designation of drill	
13. Total no. of over-burden samples taken	Disturbed: — Undisturbed: —
14. Total number core boxes	
15. Elevation ground water	DEPTH 15.0'
16. Date hole	Started 2-13-78; Completed 2-14-78
17. Elevation top of hole	85±
18. Total core recovery for boring	%
19. Signature of inspector	CHILDERS

W/C %	Depth	Sym.	Classification of materials (Description)	Standard-Penetration (Blows per foot)
			NO SAMPLE	AUGERED
21	3.0		BROWN SANDY INORGANIC SILT (ML)	WEIGHT OF HAMMER
	4.5		NO SAMPLE	AUGERED
	6.0		LT. BROWN FINE SANDY INORGANIC SILT (ML)	WEIGHT OF HAMMER
	7.5		NO SAMPLE	AUGERED
	11.0		TAN-BROWN LEAN CLAY (CL) W/ SOME SAND	12
	12.5		NO SAMPLE	AUGERED
	17.0		TAN POORLY GRADED GRAVEL (GP)	34
	18.5		NO SAMPLE	AUGERED
	23.0		TAN POORLY GRADED COARSE SAND (SP) W/GRAVEL	30
	24.5		NO SAMPLE	ROCK BIT
	27.0		CONTINUED ON SHEET 2	

Hole No. SS-2-78

BORING LOG-S (Cont Sheet) — Elevation top of hole: 185± — Sheet 2 of 2 sheets

Project: SLOPE PROTECTION — Installation: Mobile District

W/C %	Depth	Sym.	Classification of materials (Description)	Standard-Penetration (Blows per foot)
	27.0		NO SAMPLE	
	29.0		BLUISH GRAY SANDY FAT CLAY (CH)	29
	30.5		NO SAMPLE	ROCK BIT
	35.0		BLUISH GRAY SILTY FAT CLAY (CH)	49
	36.5		NO SAMPLE	ROCK BIT
	41.0		BLUISH GRAY SILTY FAT CLAY (CH)	25
	42.5		BOTTOM OF HOLE	

N 382,608
E 576,741

NOTE: HOLE GROUTED WITH 4 BAGS CEMENT.

Hole No. SS-2-78

LOGS OF BORINGS

BORING LOG	Division: South Atlantic	Installation: Mobile District	Sheet 1 of 2 sheets
1. Project	SLOPE PROTECTION DEMONSTRATION PROJECT	10. Size and type of bit	
2. Location	MONTICELLO, MS.	11. Datum for elevation shown	MSL
3. Drilling agency	MOBILE DISTRICT	12. Manufacturer's designation of drill	
4. Hole no.	SS-3-78	13. Total no. of overburden samples taken	
5. Name of driller	BROWN	14. Total number core boxes	
6. Direction of hole	Vertical	15. Elevation ground water	DEPTH 15.0' *
7. Thickness of overburden	27.7'	16. Date hole	Started 2-14-78, Completed 2-15-78
8. Depth drilled into rock	17.3'	17. Elevation top of hole	185 ±
9. Total depth of hole	45.0'	18. Total core recovery for boring	
		19. Signature of inspector	CHILDERS

W/C %	Depth	Classification of materials (description)	Standard penetration (blows per foot)
	0 – 1.5	BROWNISH GRAY POORLY GRADED SILTY SAND (SP-SM)	
	1.5 – 3.0	GRAY SILTY SAND (SM)	
18	3.0 – 4.5	BROWN LEAN CLAY (CL) W/SOME SAND	
	4.5 – 6.0	LT. BROWN CLAYEY SAND (SC)	
18	6.0 – 7.5	LT. BROWN SILTY SAND (SM)	
	7.5 – 10.5	TAN POORLY GRADED SILTY SAND (SP-SM)	
	10.5 – 12.0	TAN POORLY GRADED SAND (SP) W/ GRAVEL	
	12.0 – 13.5	TAN POORLY GRADED COARSE SAND (SP) W/GRAVEL	
	13.5 – 18.5	NO SAMPLE	ROCK BIT
	18.5 – 20.0	BROWN-TAN POORLY GRADED SANDY GRAVEL (GP)	31
	20.0 – 26.5	NO SAMPLE	ROCK BIT
	27.0	CONTINUED ON SHEET 2	

* STATIC AT 2.5'

Hole no. SS-3-78

BORING LOG (Cont Sheet)	Elevation top of hole: 185 ±	Installation: Mobile District	Sheet 2 of 2 sheets
Project: SLOPE PROTECTION			

W/C %	Depth	Classification of materials (description)	Standard penetration (blows per foot)
	27.7	GP PR GRD GRAVELLY SILTY SAND (SP-SM)	21
	28.0	BLUISH GRAY CLAYEY FN. SAND (SC)	
		TOP OF EUTAW	
	28.0 – 34.0	NO SAMPLE	ROCK BIT
	34.0 – 35.5	BLUISH GRAY CLAYEY FINE SAND (SC)	49
	35.5 – 40.0	NO SAMPLE	ROCK BIT
	40.0 – 41.5	GRAY SILTY FAT CLAY (CH)	39
	41.5 – 43.5	NO SAMPLE	ROCK BIT
	43.5 – 45.0	GRAY SILTY FAT CLAY (CH)	42

BOTTOM OF HOLE

N 382,515
E 576,775

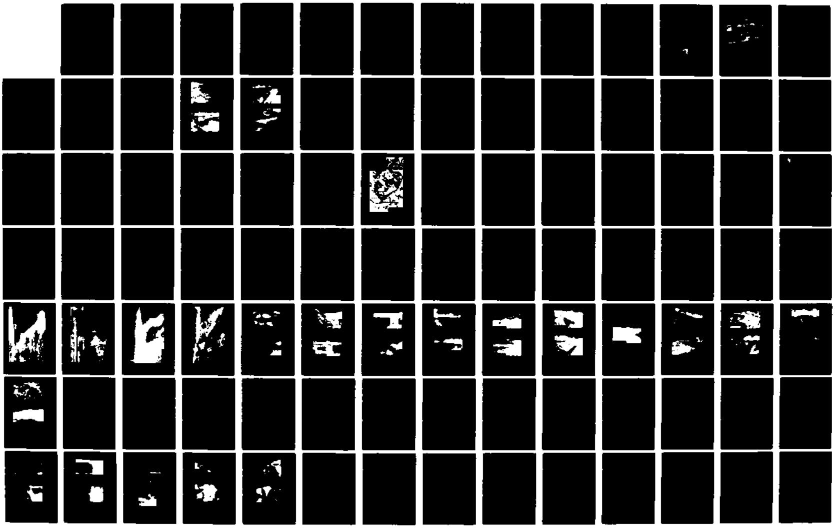
NOTE:
GROUTED HOLE WITH 4 BAGS CEMENT

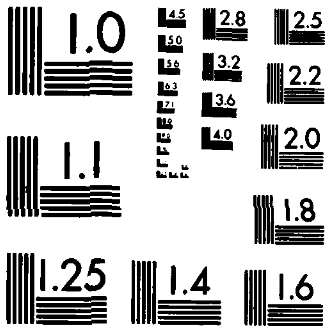
Hole no. SS-3-78

AD-A121 139 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

3/4

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

RIO CHAMA NEAR
ABIQUIU, NEW MEXICO

Section 32 Program Steambank Erosion Control
Evaluation and Demonstration Act of 1974

RIO CHAMA AT ABIQUIU, NEW MEXICO
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. The Rio Chama project is protection for an historic Hispanic chapel and village ruins located on Rio Chama, river mile 21 in Rio Arriba County, New Mexico. The project is located one and a half miles south of the community of Abiquiu, and ten miles downstream from Abiquiu Dam and reservoir which was constructed in 1963. See Plate 1 for Location and Vicinity Map.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem on the Rio Chama near an historic site listed on the National Register of Historic Places and the types of solutions used to prevent future threat by floodwater. The project was completed on 20 March 1981 and was judged to be in excellent condition at the time of the final inspection.

4. Problem Resumé. Bank erosion has been threatening the ruins of a Spanish Colonial Chapel and village site on the right bank of the Rio Chama at a critical bend (Plate 2, Artist's Conception). Some of the village site has already been claimed by the erosional process. Interested local residents alarmed at the irretrievable loss, incorporated to form La Asosiacion de Lima de Abiquiu, Inc., in 1976. This non-profit corporation worked closely with the Albuquerque District Corps of Engineers to accomplish the project.

II. HISTORICAL DESCRIPTION

5. Chapel and Village Site. The village site was settled in 1734 around a chapel dedicated to Saint Rose of Lima Peru, the first saint of the Americas. Abandoned in 1748 because of Indian attack, it was resettled in 1750. The community surrounding Capilla de Santa Rosa (Chapel of Saint Rose) became known as "La Capilla" and its population

numbered 254 in 1776. See Plate 3 for an archaeology map of the site.

The present day village of Abiquiu was established in 1754 when a land grant was made for the Abiquiu Genizaros, Hispanicized Indians. The two neighboring communities eventually merged as common problems of the frontier encouraged cooperation. For over 100 years the Abiquiu communities constituted the northernmost Hispanic settlement in New Mexico. The community of La Capilla is now abandoned with only one wall remaining of the old chapel. A shift in the course of the Rio Chama in the 1800's destroyed much of the adjacent farmland and the last inhabited house was abandoned in 1920. The remaining wall of the chapel is an important symbol to those whose ancestors are buried there. The descendents of the pioneers who lived there and other members of La Asosiacion de Santa Rosa de Lima de Abiquiu, Inc., have plans to construct a Spanish community museum on the site.

6. Rio Chama.

a. Topography. The headwaters of the Rio Chama are in the San Juan Mountains of southern Colorado in Conejos County. From there it flows south to El Vado, New Mexico then southeast through the Chama Basin to its confluence with the Rio Grande six miles north of Espanola, New Mexico, approximately 130 miles total. A map of the watershed is shown on Plate 1. Elevations range from about 12,000 feet in the mountains to 5,600 feet at the confluence. The drainage area is about 3,159 square miles, of which about 2,146 square miles are above Abiquiu Dam. The upper areas have steep slopes with a dense coniferous tree cover while slopes are much more gradual at lower elevations where the vegetation trends to pinon-juniper woodland, shrubs, and short grass prairie.

b. Geology. The project is situated in the Chama Basin, a shallow basinal structure merging on the norhtwest with the larger San Juan Basin. The basin is bounded on the west by the Gallina Arch, on the south by the Jemez Plateau, on the southeast by the Espanola Basin and on the northeast by the Brazos Uplift. The sedimentary rocks present within the Chama Basin Range in age from Pennsylvanian to Pliocene.

Igneous rocks of the basin are divided into the Precambrian intrusives of the San Pedro Mountains in the west and the Pliocene and Pleistocene extrusives of the Jemez Mountains to the south. The southeastern part of the basin is characterized by broad folds and gentle regional dips to the north and west. Steeply dipping normal faults with a general north to northeast trend are common and often exhibit throws in excess of 200 feet. Volcanic activity in the Valle Grande of the nearby Jemez Mountains began in the Miocene, and the thick tuffs of the Abiquiu formation were deposited by westward flowing streams. The Jemez Plateau continued as an active volcanic center during the Pleistocene, contributing andesitic and basaltic flows and rhyolitic tuffs to the Abiquiu formation.

c. Hydrology. The climate of the Rio Chama Basin ranges from humid and semi-humid in the mountainous upper reaches of the watershed to semi-arid in the lower elevations. The mean annual temperature over the entire basin is 42°F (Based on mean annual temperature for increments of elevation and weighted in proportion to the areas in the increments). Extreme maximum and minimum temperatures in Abiquiu since 1959 are 100°F and -25°F respectively. The average annual precipitation in the Rio Chama Watershed is about 19 inches.

The National Weather Service Rainfall Station at Chama, New Mexico (located about 30 miles north of the project), with 60 years of record, has a normal annual precipitation of 20.83 inches, a maximum of 32.14 inches in 1916, and minimum of 8.72 inches in 1956.

During the winter months, heavy snowfall occurs in the upper mountainous areas of the watershed, whereas over the lower portion snowfall is light. Snow usually remains in the mountains above elevation 8,000 feet from the beginning of heavy snows in December until early in April when snowmelt runoff begins. Snow below 8,000 feet seldom stays on the ground more than a few days. During the spring months, the depth and water content is measured at seven snow courses in the subbasin for purposes of estimating characteristics of spring runoff.

d. Channel Conditions. The soils in the project area are derived from the middle member of the Chinle shale formation. The parent materials are then predominantly Salitral shale and Poleo sandstone. Soil in the channel was characterized as SM, Silty sand. (Plate 12). Both streambank erosion and aggradation of the channel are a problem during high velocity flows. The variation in stream quantity and competence may be affected by rainfall and runoff, but for the most part it is subject only to the controlled release from Abiquiu Dam. In May 1980, a rainstorm occurred which was centered about 5 miles upstream from the project and this storm caused dramatic erosion due to interior drainage flows. The ruins sit on a high bank approximately 20 feet above the streambed so they are not subject to inundation. This storm caused severe sloughing of the bank at the village site and was an impetus for local residents to secure the area from future devastation.

7. Environmental Consideration. An environmental impact statement was not required for this project. No endangered, threatened, or otherwise unique species of vegetation or wildlife are known to be present on project lands or in the Rio Chama Valley in the vicinity. The peregrine falcon (*Falco peregrinus*) which is on the U.S. Department of Interior list of endangered wildlife and the golden eagle (*Aquila chrysaetos*), which is not listed by either the state or the U.S. Department of Interior, but which is capable of generating special enthusiasm and concern among birdwatchers, are worthy of mention. The ranges of these species include the project area but are not reported on project land.

a. Wildlife. The overall project area and adjacent lands are of generally marginal value as wildlife habitat. Upland areas have been heavily grazed and habitat at lower elevations has been adversely affected by flooding and sedimentation. Pinon-juniper areas near the project support mice, packrats, rock squirrels, least chipmunks, rock wrens, brown towees, house finches, mourning doves, rufus crowned sparrows and western flycatchers. Owls, kestrels, prairie falcons, meadowlarks, horned larks, and chipping sparrows were also noted. Modest numbers of mule deer, antelope, cottontail, muskrat, raccoon, skunk, coyote, and badger are seen in the vicinity by residents.

Seven species of fish are presently known to inhabit the Rio Chama: Channel catfish (Ictalurus punctatus), Carp (Cyprinus carpio), River carpsucker (Carpiodes carpio), White sucker (Catastomus commersoni), Rio Grande chub (Gila nigrescens), Brown trout (Salmo trutta), and Rainbow trout (Salmo dairdneri). The Abiquiu Reservoir is presently being managed as a "two-story" fishery in which both cold-water (Rainbow trout) and warm-water (Channel catfish) fish are maintained.

b. Terrestrial Vegetation. The vegetation in the vicinity of Abiquiu trends from short-grass prairie to pinon-juniper woodland depending primarily upon elevations, slope, exposure, and soils. The one-seed juniper is fairly prominent on the steeper slopes of the sides of dissected terraces of plateaus. The juniper and pinon pine are both common on the foothills. Ground cover is sparse due to low rainfall and poor soil. Woodlands were cleared to create increased gaging. Range conditions tend to be only fair. Cattle production is dominated by galleta, blue grama, and bottlebrush-squirreltail. Broom snakeweed and other shrubs are also present. Cultivated fields in the flood plain to the north of the project grow apples, corn, beans and other vegetables.

8. Demonstration Site-Test Reach.

a. Hydrologic Characteristics. The hydrologic characteristics are as previously stated. Streamflow data are available at five locations in the Rio Chama Basin below Abiquiu Dam. Two of these gage sites are on tributaries to the Rio Chama. They are the El Rito Creek gage near El Rito, New Mexico, and the Rio Ojo Caliente gage near La Madera, New Mexico. The other three gage sites are all on the Rio Chama. They are the Rio Chama below Abiquiu Dam, the Rio Chama near Abiquiu, and the Rio Chama near Chamita. Based on analysis from all of the gages except the one right below the dam (it reflects only the controlled release), a discharge-frequency curve was computed (Plate 11). The maximum flow for all frequencies from the annual to that approaching the Standard Project Flood should not exceed 4,000 c.f.s. However, for present channel conditions, a more realistic maximum flow would be in the range of 1,200 to 2,000 c.f.s.

b. Hydraulic Characteristics. Flood flow velocities in the Rio Chama range from 0 to 2.85 f.p.s. for a period of record of 1912 to 1978. Bankfull stage is 3.5 feet with a flow of 2,000 c.f.s., and an average recurrence interval of 2 years. Bankfull flow velocity is 2.85 f.p.s. with a flow of 2.0 f.p.s. near the bank. Maximum safe release from Abiquiu Dam would be 2,000 c.f.s. No groundwater bank seepage was observed in the project site. No rock of any kind was observed in the channel or in the vicinity of the project.

c. Riverbank Description. The riverbank is composed of the same silty sand as is found in the channel. Vegetation along the bank is sparse due to the low rainfall condition and is composed of the short, prairie grass mentioned earlier, merging to shrubs as elevation increases. The bank was eroding at an average rate of one-foot per year near the project site before construction. The bank is 20 feet high at the site and contains cultural debris washed down from the village above. A gradation of a typical riverbank soil sample is shown as Plate 12. No clay was present in any of the soil samples.

III. DESIGN AND CONSTRUCTION

9. General. Since both streambank erosion and aggradation of the channel are problems at the site, the objectives were to stabilize the bank and to increase the channel's transport capacity by increasing the channel velocity. Bank protection was designed to use conventional labor intensive methods. A major constraint was to provide protection which was architecturally compatible with the adjacent ruins.

10. Basis for Design. Three methods of erosion control were selected using methods proposed by the Waterways Experiment Station. They were:

a. Log cribs
b. Riprap
c. Gabion Groins

The layout of the protection works is given on Plate 4, and details of the log cribs, riprap and gabion groins are given on Plates 5 and 6.

11. Construction Details.

a. Riprap. The bedding materials for riprap were composed of hard, durable particles, free from adherent coating with a maximum size of 3 inches and not more than 2 percent by weight passing a standard number 200 sieve. The stone for riprap was river run cobbles, well graded, with a maximum of 12 inches and minimum size of 4 inches. Where compacted earth fill was required on the project it was placed in 8" layers and was compacted to a density equal to or greater than the density of the adjacent undisturbed areas. Plastic filter fabric was placed on prepared foundation for the easternmost 279 feet of the project.

b. Log Cribs. The log cribs were constructed of telephone poles as per option 2.2.1, Section 2c of the specifications.

c. Gabion Groins. The chain link fabric was 9 gage wire woven in a 2 inch mesh. Gabions were assembled from the wire mesh on the site using single unit construction to assure that any point of connection would be at least equal to the strength of the mesh.

d. Seeding. Seeding was required for this project. The contractor used a generous amount of the specified mix of pure live seed composed of blue grama, sideoats grama, indian rice grass, and western wheat grass. Permission was given for early planting due to favorable weather conditions and the results were found to be satisfactory at the time of the final inspection.

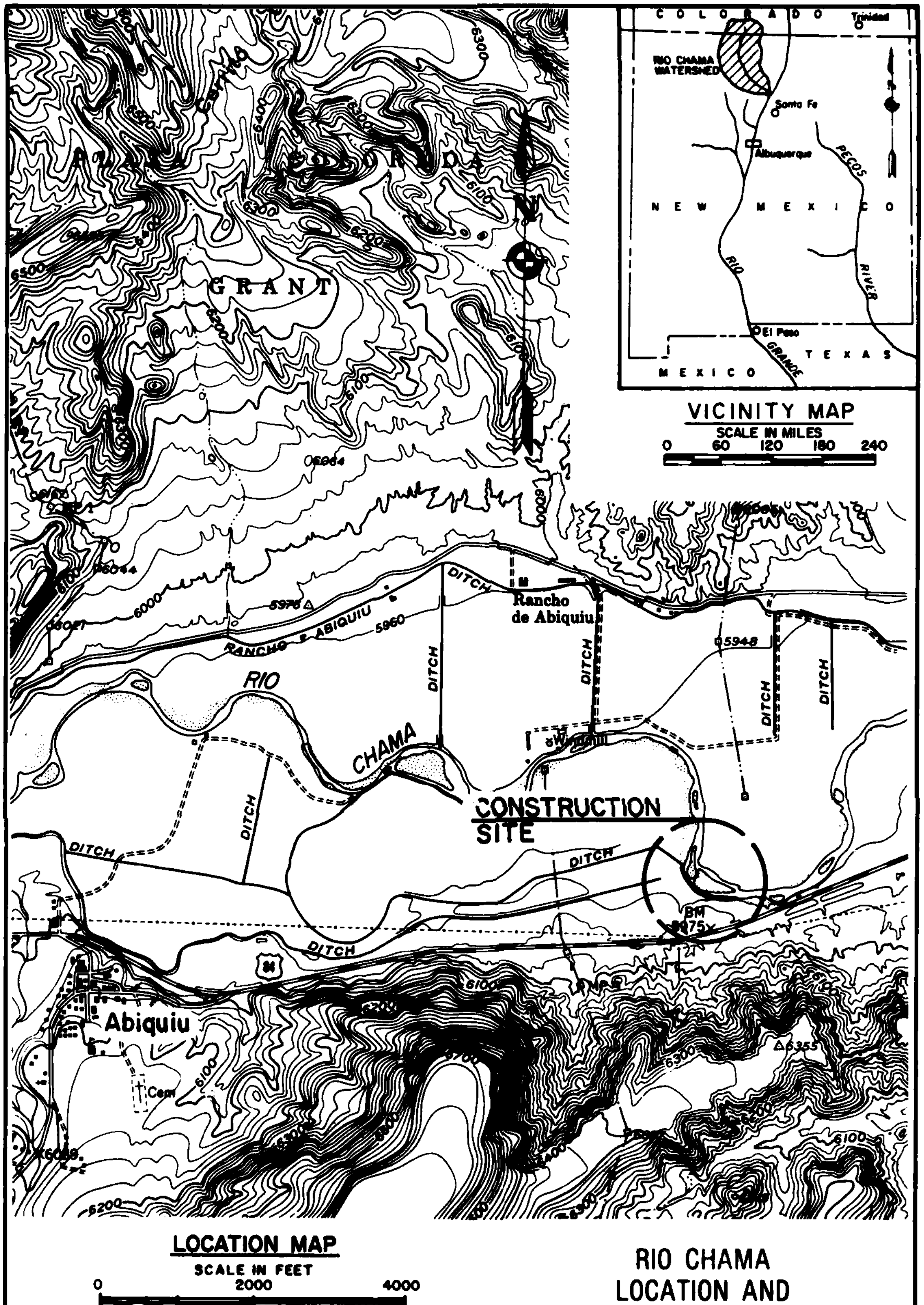
12. Cost. The total cost of construction including engineering and design of the three types of streambank protection amounted to $160,000 or approximately $223 per bank foot for the log cribs; $176 per bank foot for the riprap; $117 per bank foot for the gabion groins.

IV. PERFORMANCE OF PROTECTION

13. Monitoring Program. The elements of the monitoring program are summarized on Plate 9. Primary observations include baseline, annual and specified channel cross-sections surveys, velocity distribution,

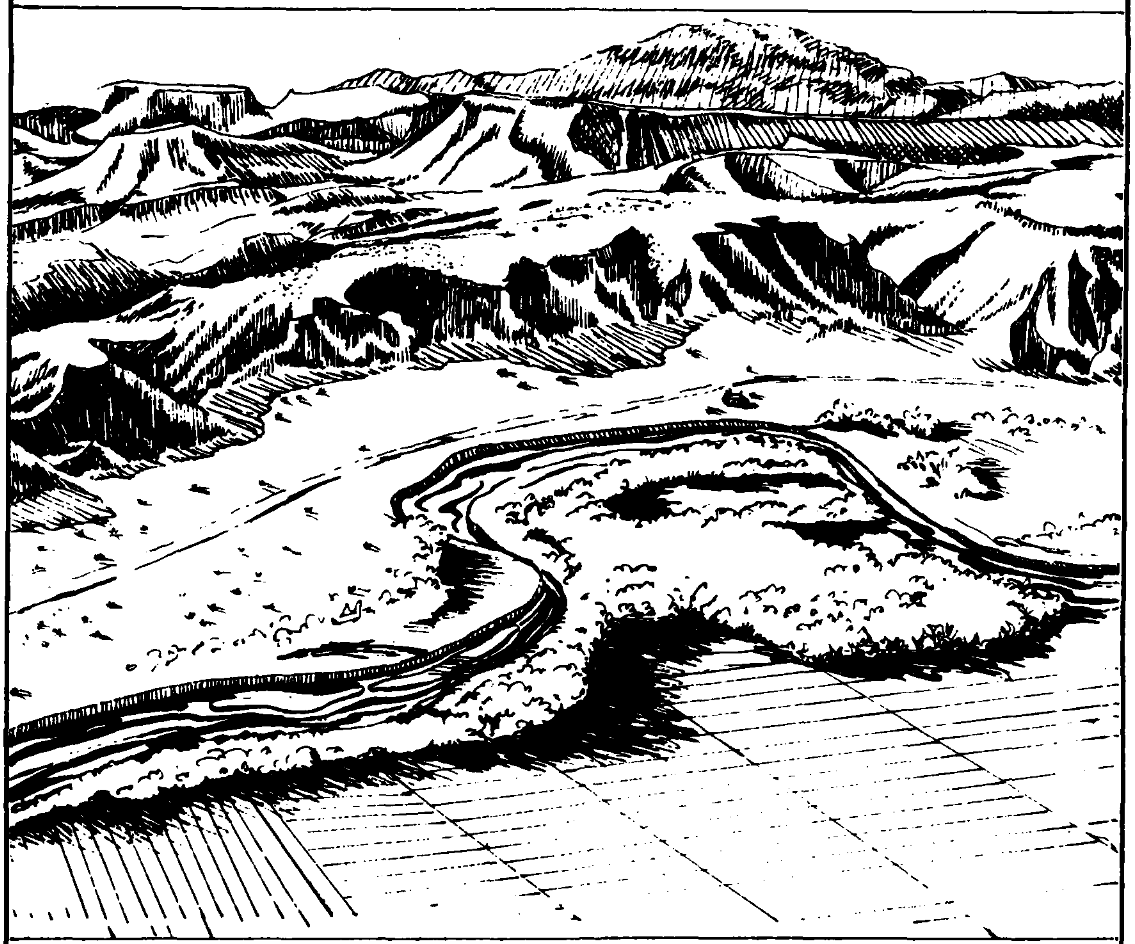
visual inspections, and periodic and special photographs (Plates 7 & 8). Any flood event in the project area will be observed by Corps of Engineers personnel from the Abiquiu Dam residence office. The project was completed on 20 March 1981, therefore, it is unchanged from the condition described in the final inspection. The site will be monitored after appreciable flows have been experienced and the resulting evaluation and conclusions reports furnished Southwest Division. See Plate 10, Evaluation of Existing Protection.

14. Conclusion. Based on the construction final inspection, the project was judged to be a very satisfactory performance on the part of the contractor. The ability of the design to function for the intended purposes will be decided later when an actual flood occurs. The residents of Abiquiu who participated with the Corps of Engineers in the real estate requirements were more than happy with the project. The esthetics of the design blend harmoniously with the environment and it can be seen from U.S. 84 by all who pass. The Corps of Engineers gained a positive political benefit with this project. This project brought letters of praise to the District Office from Abiquiu residents and will possibly have a long range effect to promote good public relations between the Corps and the people we are trying to serve.



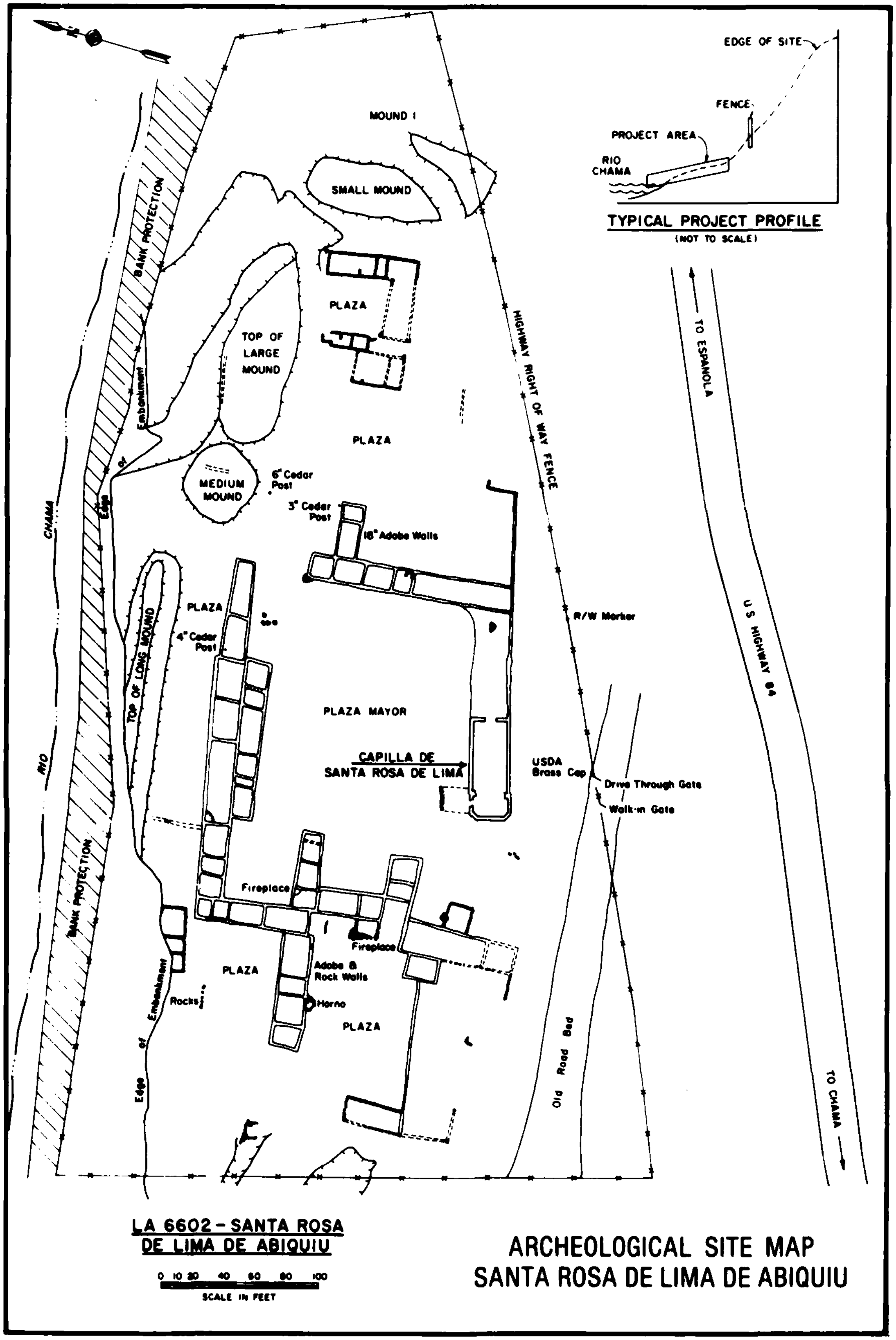
RIO CHAMA LOCATION AND VICINITY MAPS

PLATE 1



ARTIST'S CONCEPTION
OF VICINITY

PLATE 2



EDGE OF SITE
FENCE
PROJECT AREA
RIO CHAMA
TYPICAL PROJECT PROFILE
(NOT TO SCALE)
MOUND I
SMALL MOUND
BANK PROTECTION
PLAZA
TOP OF LARGE MOUND
Edge of Embankment
HIGHWAY RIGHT OF WAY FENCE
TO ESPANOLA
PLAZA
MEDIUM MOUND
6" Cedar Post
3" Cedar Post
18" Adobe Walls
RIO CHAMA
PLAZA
4" Cedar Post
TOP OF LONG MOUND
R/W Marker
US HIGHWAY 84
PLAZA MAYOR
CAPILLA DE SANTA ROSA DE LIMA
USDA Brass Cap
Drive Through Gate
Walk-in Gate
BANK PROTECTION
Fireplace
Fireplace
PLAZA
Adobe & Rock Walls
Rocks
Horno
PLAZA
Edge of Embankment
Old Road Bed
TO CHAMA
LA 6602 - SANTA ROSA DE LIMA DE ABIQUIU
0 10 20 40 60 80 100
SCALE IN FEET
ARCHEOLOGICAL SITE MAP
SANTA ROSA DE LIMA DE ABIQUIU

PLATE 3

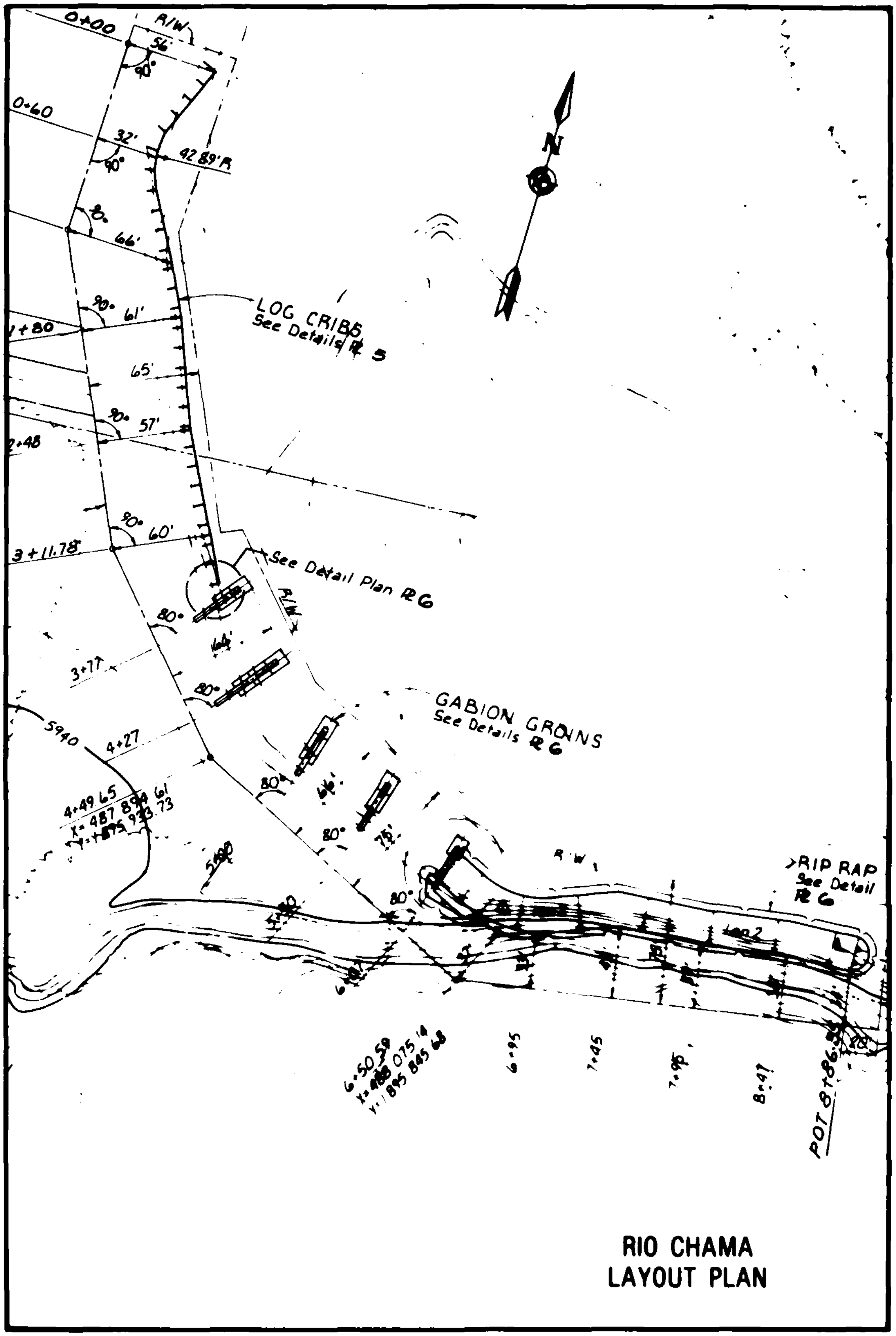
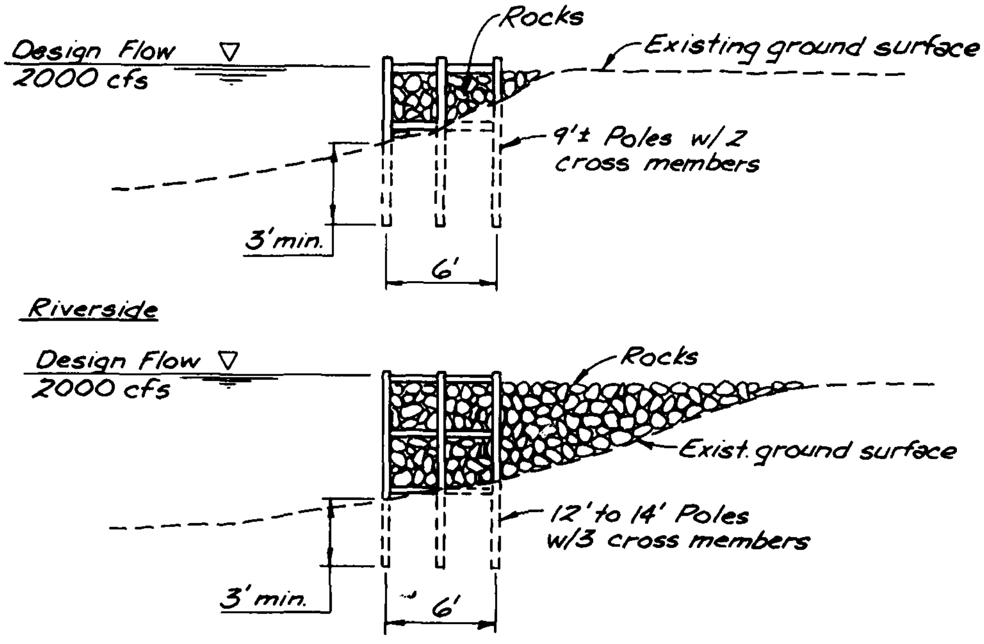
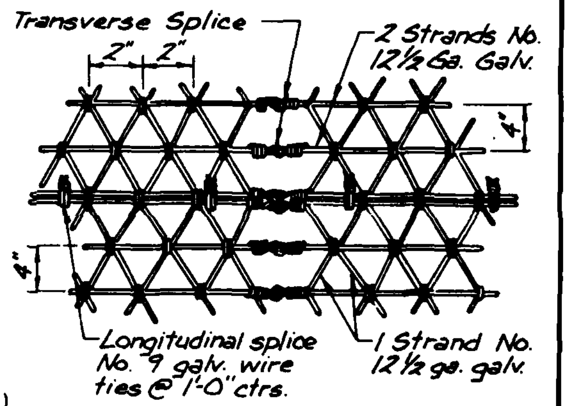

0+00
R/W
0+60
42.89' R
LOG CRIBS
See Details Pl 5
3+11.78
See Detail Plan Pl 6
GABION GROINS
See Details Pl 6
4+27
RIP RAP
See Detail
Pl 6
RIO CHAMA
LAYOUT PLAN

PLATE 4



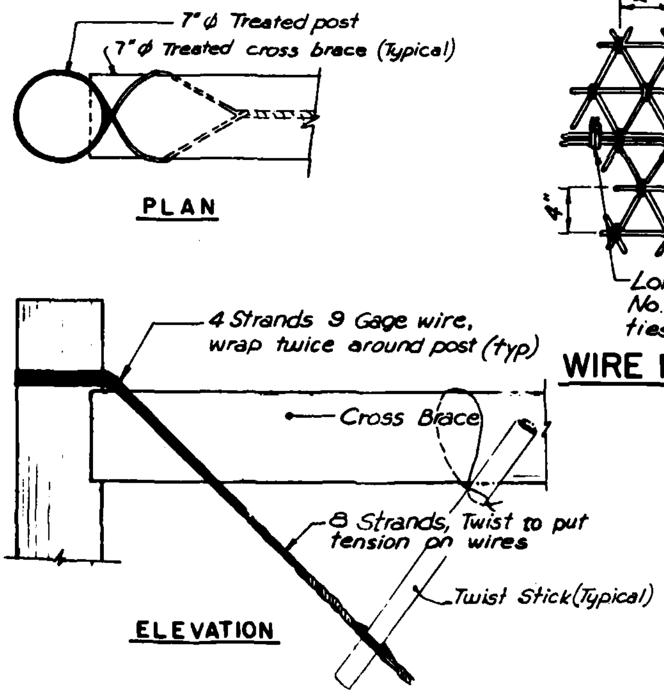
LOG CRIBS – TYPICAL SECTIONS

NOT TO SCALE



WIRE FABRIC SPLICING DETAILS

NOT TO SCALE

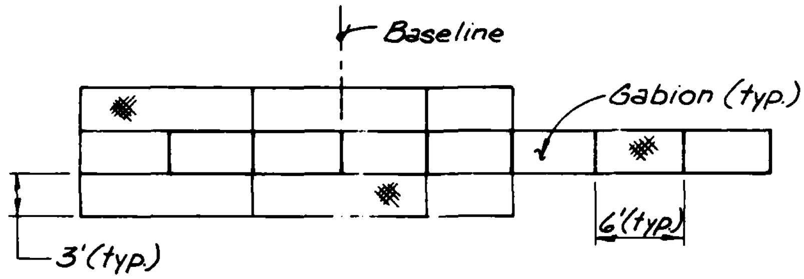


CROSS MEMBER DETAILS

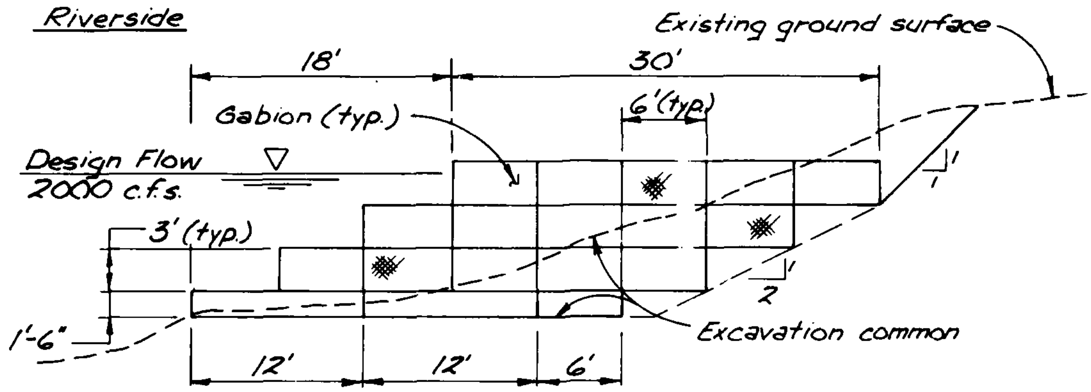
NOT TO SCALE

RIO CHAMA
BANK PROTECTION
SECTIONS AND DETAILS

PLATE 5



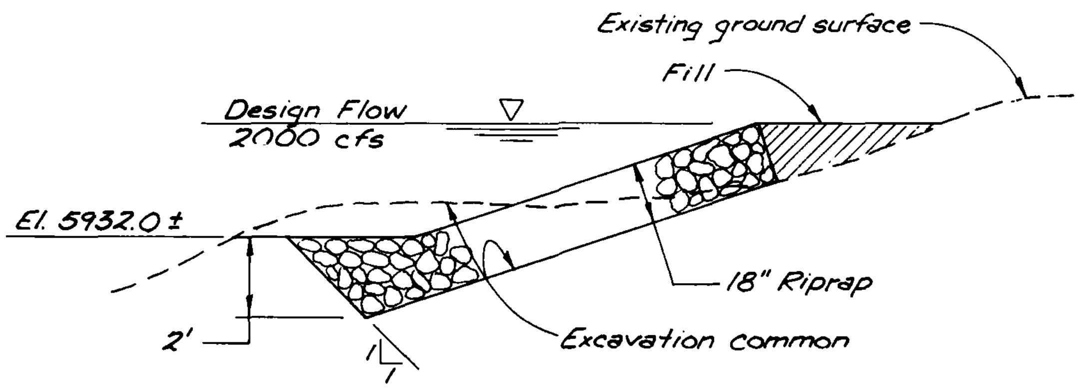
PLAN



SECTION

GABION GROIN

NOT TO SCALE



RIPRAP SECTION

NOT TO SCALE

RIO CHAMA
BANK PROTECTION
SECTIONS AND DETAILS

PLATE 6

PHOTO 1 LOOKING DOWNSTREAM BEFORE CONSTRUCTION
DATE: OCT 1980

PHOTO 2 LOOKING UPSTREAM AFTER CONSTRUCTION
DATE: APRIL 1981

VIEWS OF DEMONSTRATION SITE

PLATE 7

PHOTO 1 LOOKING DOWNSTREAM BEFORE CONSTRUCTION
DATE: OCT 1980

PHOTO 2 LOOKING DOWNSTREAM AFTER CONSTRUCTION
DATE: APRIL 1981

VIEWS OF DEMONSTRATION SITE

PLATE 8

Field Data of Physical Features	Frequency
1. Cross Sections for scour depth along structure, width, and quiet-water areas. 2. Velocity measurements for discharge distribution, velocity distribution near structure, and vertical velocity distribution at a standard distance. 3. Bank-line location survey. 4. Overbank cross section.	Preconstruction and annually postconstruction or when significant changes noted during field inspections.
5. Crown profiles and cross sections establishment of aids for visual observations. 6. Probings for underwater structure locations.	Immediately after construction and future dependent on visual observations.
Visual Observations	
1. Changes in aquatic habitat. 2. Aggradation-degradation processes. 3. Erosion and river conditions. 4. Changes in terrestial habitat and upper slope vegetation. 5. Changes in structure integrity and material durability (including effects of ice). 6. Surface current flow pattern.	Four times yearly for all visual observations.
Monitoring Program	
1. Mechanical analysis of river-bed material. 2. Classification of cutting bank material (mechanical analysis when appropriate). 3. Freeze-thaw durability for rock.	Once per source location unless material obtained for wide divergence in geologic structure.
4. Mechanical analysis for rock, sand, gravel, and clay. 5. Chemical analysis of construction materials when appropriate.	Once during construction for each site and as needed postconstruction.
Photography	
1. Ground level photography.	Preconstruction photography of site and a minimum annually after construction.

RIO CHAMA
MONITORING PROGRAM

PLATE 9

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 8

(1) Location

Stream Rio Chama River Mile 21 Side Right Bank

Local Vicinity Abiquiu Lat 36°12'21"N Long 106°17'29"W

At/Nr City Espanola County Rio Arriba State NM Cong Dist 1st

CE Office Symbol Albuquerque Responsible Agency

Site Map Sources U.S.G.S. 7.5 MINUTE QUADRANGLE SHEET

Land Use Information Sources

(2) Hydrology at or Near Site

Stage Range 0 to 35 ft; Period of Record 1912 to 1978

Discharge Range 0 to 2000 cfs; Velocity Range 0 to 2.85 fps.

Sediment Range 0 to 214,000 tpd; Period of Record 1962 to 1974

Bankfull Stage 3.5 ft; Flow 2000 cfs; Average Recurrence Interval 2 yrs.

Bankfull Flow Velocity: Average 2.85 fps; Near Bank 2.0 fps.

Comments Maximum safe release from Abiquiu Dam = 2,000 c.f.s.

(3) Geology and Soil Properties

Bank (USCS) SM (Silty Sand) Bed (USCS)

Data Sources Field Observation

Groundwater Bank Seepage None observed

Overbank Drainage

Comments No rock observed in vicinity of this project site.

(4) Construction of Protection

Need for Protection

Erosion Causative Agents Erosion and Sloughing of the Right Banks, aggredation, velocity scour, and stage fluctuation

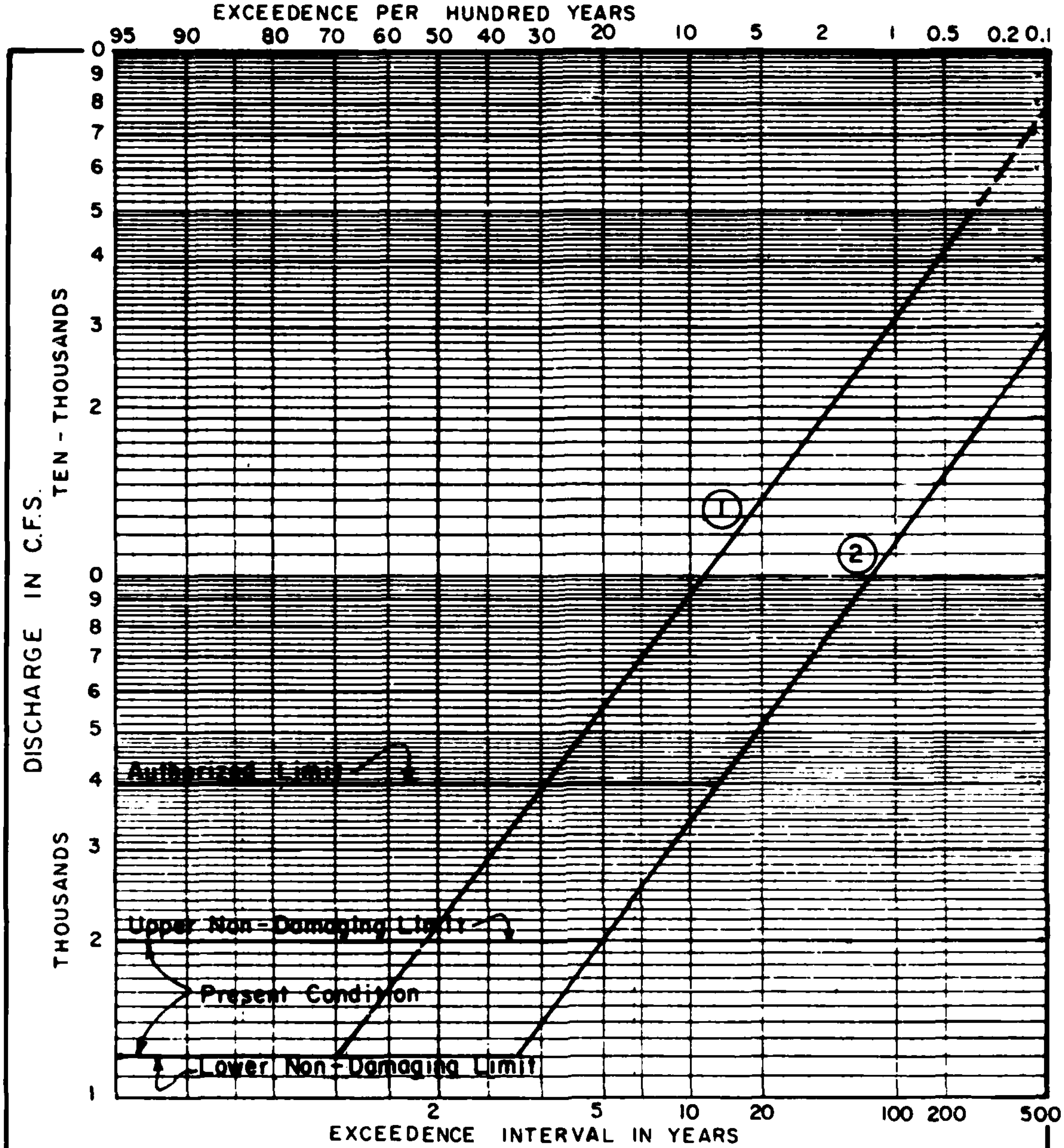
Protection Techniques Log cribs, riprap and gabion groins.

General Design

Project Length 886 ft; Construction Cost $ 131,000 ; Mo/Yr Completed 5-81

DEMONSTRATION PROJECT

PLATE 10



(1) Rio Chama Near Chamita, New Mexico

(2) Rio Chama Near Abiquiu, New Mexico

RIO CHAMA
BELOW ABIQUIU DAM
DISCHARGE-FREQUENCY

PLATE 11

cost symbol VW81293800-0 Cochiti Corps of Engineers Soils Lab

SWD FORM 275 Rev 1 Dec 65	AGGREGATE SIEVE ANALYSIS AND MISCELLANEOUS TESTS	DATE 28 Oct 1980
PROJECT Chama River		FIELD NO.
MATERIAL		~~SAMPLE NO.~~ Sample #1
~~REPORT NO.~~ Hole #1		TESTED BY sampled by: B. Elsner

SPECIFICATIONS date sampled 23 Oct 1980 Depth: surface

1. MECHANICAL ANALYSIS

SIEVE SIZE (U.S.B.S.) SQ. NO.	WEIGHT RETAINED SAMPLE #1	SAMPLE #2	PERCENT RETAINED #1	#2	AVG.	CUMULATIVE	PASSING	SPEC. REQ.
6 Inch								
5 "								
4 "								
3 "								
2 1/2 "								
2 "								
1 1/2 "								
1 "								
3/4 "	0					0	100	
1/2 "	3					0.5	99.5	
3/8 "	–					–	–	
3/16 " (No.4)	5					0.8	99.2	
No. #10	6					1.0	99.0	
No. #40	27					4.4	95.6	
No. 30	–					–	–	
No. 50	–					–	–	
No. 100	100					16.5	83.5	
No. 200	427					70.3	29.7	
PAN	487							
TOTAL	607							

2. MATERIAL FINER THAN NO. 200 — SAMPLE 1 | SAMPLE 2

ORIG DRY WT. ______
DRY WT. AFTER WASHING ______
DIFFERENCE ______
______ %
WT. DECANT (200) ______
______ %

3. FLOATATION

OVEN DRY WT. ______
WT. FLOATERS ______
% FLOATATION ______

Moist (#48)

4. ORGANIC IMPURITIES

PLATE NO. 1 2 3 4

STANDARD DARKER LIGHTER

Grad (#50)

REMARKS Test run in accordance w/Appendix V of EM 1110-2-1906 840g WW 666 DW 174 26.1% 765g WW 607g DW

TESTED BY:	COMPUTED BY:	CHECKED BY:
J. E. GUITERREZ	J. E. GUITERREZ	

AGGREGATE SIEVE ANALYSIS AND MISCELLANEOUS TESTS

PLATE 12

ROANOKE RIVER NEAR LEEVILLE, VIRGINIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

ROANOKE (STAUNTON) RIVER NEAR LEESVILLE, VIRGINIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Roanoke (Staunton) River near Leesville, Virginia, Section 32 Demonstration Project. Figure 1 shows location map.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem, the types of bank protection used, and (later) a performance evaluation of a demonstration project on two sites on the Roanoke (Staunton) River, Virginia, constructed (construction began 13 December 1980 and was completed 13 March 1981) and monitored by the Wilmington District.

4. Problem Resume. Operation of the hydropower facility at Leesville Dam results in normal river stage fluctuation from about 0.5 feet to about 5 feet twice daily. Portions of the 5 miles of streambank immediately downstream from the dam have eroded significantly, some areas receding as much as 200 feet. Two of the most severely eroded locations, which are pasturelands, have been selected as demonstration sites. Site A (1,300 feet long) is about 1 mile downstream from the Leesville Dam on the left bank facing downstream, Site C (700 feet long) is about 3-1/2 miles downstream from the Leesville Dam, at the town of Leesville, on the right bank facing downstream. Nearby river erosion of Virginia Highway 43 east of Leesville has been partially

solved by placement of sandstone riprap by the Virginia Department of Highways.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The Roanoke River watershed lies within four physiographic provinces known as the Valley and Ridge Province, the Blue Ridge Province, the Piedmont Plateau, and the Atlantic Coastal Plain. The Leesville project is in the western part of the Piedmont physiographic province which extends from northeast to southwest between the Blue Ridge Mountains and the Coastal Plain. The Piedmont Plateau is a low, eastward- sloping plateau, well dissected by long-continued erosion. The Plateau is characterized by rolling conformation with elevation ranging from 150 to 900 feet above mean sea level (ft., m.s.l.). The elevation near the Leesville project is between 500 and 600 ft., m.s.l. The Piedmont area is traversed by highlands which are cut by numerous tributary valleys. The flood plain along the Roanoke River in the Piedmont Plateau near the Leesville project ranges from 1,000 to 1,500 feet in width. The fall in the river at the project location is about 2.5 feet per mile of river.

b. Geology.

(1) Regional. The Piedmont area is formed largely in very old sedimentary and igneous rocks that have been so greatly altered through metamorphism that most of them bear but slight resemblance to the original rocks. The rock underlying the site is Wissahickon schist, which is a finegrained, chlorite schist whose principal mineral constituents are quartz, white mica, and chlorite. While it is a relatively soft rock, it is tough and resistant to erosion.

(2) Project Area. The project area is located in alluvial material which overlays the bedrock. The eroding material is generally silty sand. The erosion appears to be most active in the area from the Leesville Dam to approximately 5 miles downstream. The rate of erosion, although not determined exactly, is fairly high. Erosion has taken place on both sides of the river in this area, but primary loss area observed is on the left bank, in a moderate curve. Maximum width of erosion is about 150'-200'. The channel side slope is in alluvium that assumes an unstable vertical slope for about 6 feet below ground surface, then eroded material (silty sand) assumes about a 1-1/2 to 1 slope to the bottom of the stream. Vertical distance from top of bank to streambed varies from about 8 feet to 15 feet. Some old logs and leaf mold are exposed near the toe of slope. Many trees, mostly sycamore, have eroded from the bank and lie in the river. The toe of the slope in some areas is silt and in other areas appears to be "pipe clay," a sticky "weathered-in-place" residual from bedrock. Much small gravel is in the streambed and some bars of large boulders are in the river. In several areas the bank consists of fill material resulting from a reconstruction project several years ago. The banks are vertical and virtually void of vegetation in many areas. It was noted that the water level is raised from 0 to approximately 10 feet, two to three times daily, resulting in a continuous wetting and drying of these erodable soils.

c. Hydrologic Characteristics.

(1) Climate. Moderate temperatures generally prevail in the Roanoke River Basin at the project location, with occasional extremes. The highest and lowest temperatures recorded at nearby Roanoke, Virginia are 104°F and -4°F, respectively.

(2) Precipitation. The average annual precipitation over the Roanoke River Basin above the Leesville project is about 41 inches. The maximum annual rainfall over the Basin was 57 inches in 1937 and again near 57 inches in 1972, and the minimum was 30 inches in 1963.

Annual snowfall averages about 22 inches, but only a small amount accumulates for any length of time.

(3) Runoff. The annual runoff of the Roanoke River at Leesville averages about one cubic foot per second per square mile of drainage area and slightly greater than 30 percent of annual precipitation. Flood producing storms occur in all seasons of the year. Generally, floods are caused by brief periods of intense rainfall on a major portion of the watershed. In the winter and spring, the intense rainfall has occasionally been a part of a protracted storm period in which prior rainfall served to fill the stream channels and to soak the ground, thereby decreasing the absorptive capacity of the soil. In the summer and fall, intense rainfall is often associated with tropical hurricanes. Runoff is regulated by upstream hydropower reservoirs to some degree (Smith Mountain and Leesville Reservoirs).

d. Development. The setting of the project area is largely rural with row-crop farming and pasturing cattle being the major sources of income. The Leesville Dam and Reservoir, the lower unit of a pumped-storage hydroelectric system, is located about 1 mile upstream from site A of the project area.

e. Channel Conditions. Project area flows are controlled most of the time by releases from one or both of the turbines within the Leesville Dam. The flow from one turbine produces a flow depth of about 4 feet, which may persist for several hours each day, depending on demand. During heavy demand periods, both turbines operating produce flow depths of about 8 feet in the project area. Flow during nongenerating periods is on the order of 50-100 cubic feet per second, causing flow depths of about 1/2 feet. Floods, since the reservoir was built about 1965, have caused substantial overbank flooding and damages to crops and pastures. Cross sections surveyed in 1976 (see figure 1) showed that the channel at about station 2+00 on site A (section 1) had a bottom width of 210 feet, top width of 270 feet, and depth of 20 feet. The channel at about station 9+00 on site A (section 1A) had a bottom width of 275 feet, top width of 325 feet, and depth of 17 feet (this is the area which had experienced the greatest erosion). The channel about 1,500 feet downstream from site A (section 1B) had a bottom width of 90 feet, top width of 130 feet, and depth of 14 feet (this area has been relatively unaffected by erosion, but may have received some sediment from upstream erosion). The channel at about station 2+00 on site C (section 3) had a bottom width of 300 feet, top width of 325 feet, and depth of 15 feet (erosive effects of both river and Goose Creek flows are reflected here).

f. Environmental Considerations. The bank protection plan did not result in any losses of forest game habitat within the area, since trees that once lined the banks have been destroyed by erosion. The stabilization work included grassing on channel side slopes and top of bank areas which were raw earth from erosion. The grass provided is considered beneficial since it improves habitat for wildlife and improves the scenic character of the stream. In addition, the trend toward a stabilized geometry of the alluvial river could result in a deeper channel in some areas with more pools which will have a beneficial effect on the fish habitat. The project is expected to improve the water quality in the Roanoke (Staunton) River by reducing the load of suspended solids. This should enhance the scenic, recreational, and fish and wildlife value of the river. The project area is an important spawning reach for striped bass from Kerr Reservoir.

6. Demonstration Site - Test Reach.

a. Hydrologic Characteristics. The hydrologic characteristics are determined largely by operation of the Leesville hydropower facility the majority of the time. During floods, releases in excess of power releases are necessary, and bankfull stages may be exceeded (see figure 6). The United States Geological Survey (USGS) stream gage at Alta Vista, some 10 miles downstream from the Leesville Dam, provides water flow and water quality data (see figure 5). Monthly maximum and minimum tailwater elevations and companion discharges from the Leesville Dam are also available. (See figure 7.)

b. Hydraulic Characteristics. Stream velocities vary from about 2 feet per second at depths of 2 feet to about 5 feet per second at about 8 feet of depth in the site A project reach. Similar readings were obtained at site C. The mean velocity was 3.86 feet per second at a depth of 10 feet and discharge of 5,020 cubic feet per second at site A. The mean velocity at site C, at about 7 feet of depth, was 3.04 cubic feet per second (see figure 4).

c. Riverbank Conditions.

(1) Bank Materials. Silty sand and sandy inorganic silt comprise the majority of the bank material. The toe of the slope varies from inorganic silt to clay. Soil sample analyses are shown in figure 2. No soil borings were taken, so boring logs are not available. Samples analyzed were removed directly from the bank and placed in sample jars for analysis.

(2) Normal Bank Vegetation. Trees, mostly sycamores, willows, and various other shrubs and quick-growth trees provide some bank stabilization along the river in this general area. However, the original tree line has been eroded into the river on both sites A and C. Grass-covered areas, where the original tree line was sparse or absent, seem to experience the highest rates of erosion (see photos).

(3) Bank Erosion Tendencies. Both test sites have been eroding at a fairly consistent rate since monitoring began in 1977. Figure 8 shows the results of bank sloughing measurements.

III. DESIGN AND CONSTRUCTION

7. General. Three methods of erosion control were provided. Site A received 960 feet of stone rubble with vegetation and 532 feet of rubber tire mattress. Site C received 700 feet of rock windrow. The stone rubble with vegetation method consists of excavating a bench at elevation 537 feet, mean sea level, along the streambank, with a back slope of 1:1 and a 10-foot width. Locally produced stone, weighing between 20 and 290 pounds, was placed on the bench to a height of about 5 feet. All stone used was marble, tending to flat, elongate shape, from the Blue Ridge Stone Corporation, Lynchburg, Va. The landward upslope was graded to 3H and 1V until the natural ground intercept was reached, and the graded area was seeded to Kentucky 31 fescue grass (see photographs).

The rubber tire mattress method consists of placing a 3-foot-high rock toe in the streambed, top width 1-1/2 feet, base width 10-1/2 feet, and grading the entire slope from toe to top of bank to 3H to 1V. The lower 27 feet (measured along the slope) was covered with a tire mattress. The tires were fastened together into 4-tire units with steel bailing bands, and each unit anchored with screw anchors, 3/4-inch diameter, 66 inches long, with 6-inch blades. Steel cables, 3/8-inch diameter, hold the units to the anchors. The upslope area was seeded to Kentucky 31 fescue grass. A willow sprout was planted in each tire (see photographs).

The rock windrow method consists of a trench excavated near the top of the existing bank and filled with rock. The trench has a 10-foot-bottom width, 1:1 slide slopes, and 5-foot depth. The rock was covered with a 6-inch thickness of earth. No modifications were made to the existing bank.

8. Basis for Design. The primary reason for selection of these alternative methods is to demonstrate the effectiveness of low material cost, but high labor cost, approaches. Rock is abundant in the

vicinity, and reject tires can be obtained at low, or no, cost in nearby Danville, Virginia. If they prove successful, local farm interests could approach these problems during off seasons when labor demands are low.

9. Construction Details. Construction drawings are included as figure 3. Photos 13 through 17 show "during construction views of site A. Photos 18 through 25 show after construction views of the various types of protection.

10. Protection Costs. Total protection costs, including construction E&D, and S&I were $342,007. Total protection costs for each type of protection were: 960 lineal feet of stone rubble, $157,240; 532 lineal feet of rubber tire mattress, $103,935; and 700 lineal feet of stone windrow, $80,832. The total protection cost per lineal foot for each protective scheme was: stone rubble, $163.77; rubber tire mattress, $195.33; and stone windrow, $115.46. A minor modification ($525) consisted of providing, in May 1981, about 1,300 lineal feet of barbed wire fencing to protect project slopes from grazing cattle on site A.

11. Monitoring Costs. Total monitoring costs are $11,582 through 31 May 1981. Additional monitoring costs, through fiscal year 1982, are expected to be $13,500.

12. Environmental Impact Experiences During Construction. The willow sprouts placed in the tires of the higher locations of the slope, away from the water did not thrive as could be expected, since it was quite dry during and following construction. However, most of them survived.

All excavation was done during times of low flow, so sedimentation was minimized. There was no marked environmental impact during construction. When water rose over disturbed earth, very minor siltation occurred for a few days until the earth stabilized and grass growth started.

IV. PERFORMANCE OF PROTECTION

13. Monitoring Program. Observations, started in 1977, include stage-discharge data in tailwater of Leesville Dam, USGS stage-discharge data at Alta Vista, baseline survey (1978), velocity distribution curves by USGS (1978), visual inspections, periodic ground photography (every 3 months), and "sloughing" measurements from baseline points to top of bank (every 3 months). Monitoring continued during and after the construction period. Monitoring results are displayed in figures 7 and 8 and in photos 1 through 25. Monitoring is expected to continue through fiscal year 1982.

14. Evaluation of Protection Performance. The daily wetting and drying of bank material seems to be the primary cause of bank erosion, rather than flood flows. Since the project construction was completed in March 1981, only limited experience with the effectiveness of the completed project is available at this time.

15. Rehabilitation. The judgment on the need for rehabilitation cannot be made until 2 to 3 years following construction, or about Spring 1984.

16. Conclusion. None possible at this time. Initial impressions, based on experience over 3 months since construction was completed, are that the project is functioning well and as designed.

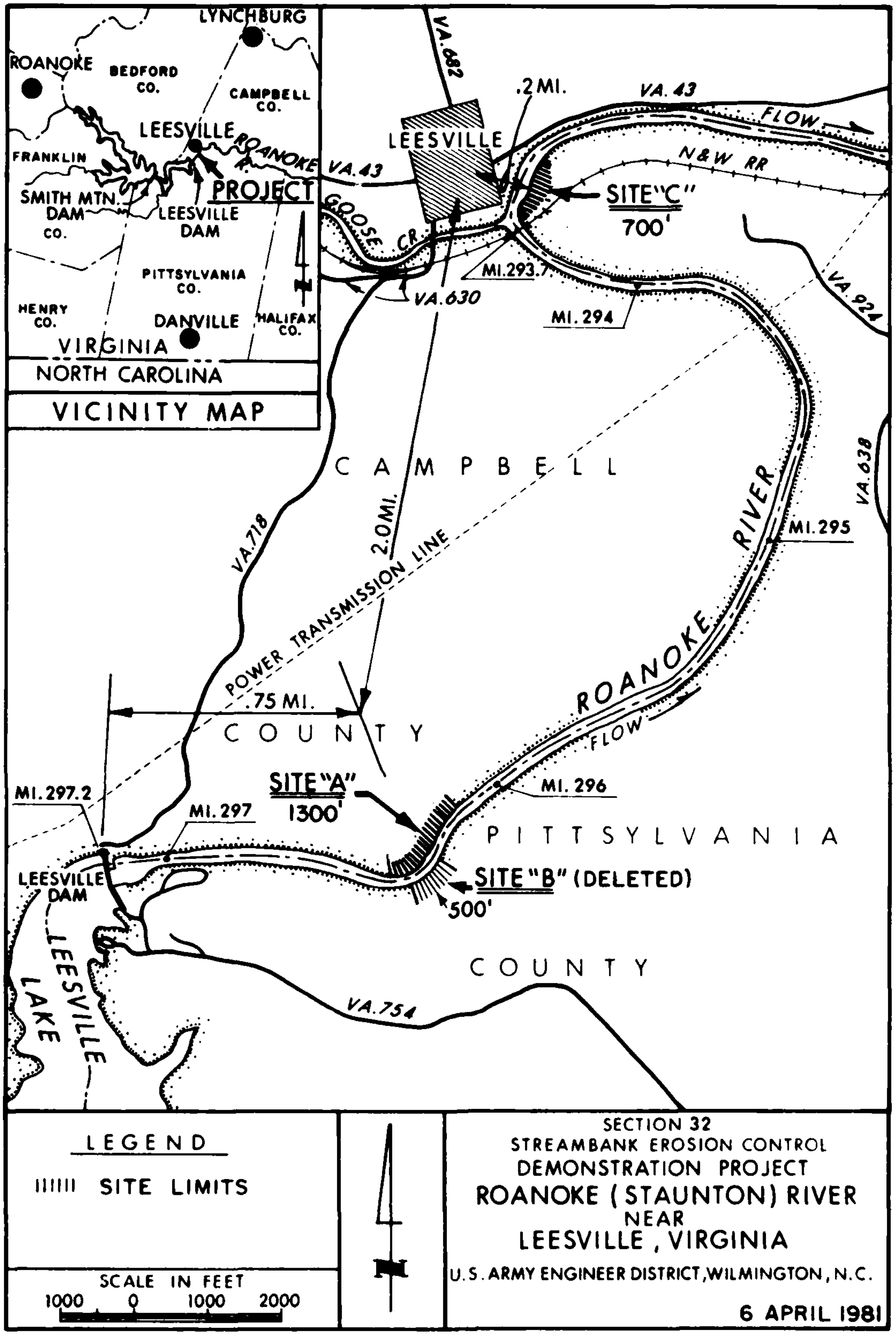


FIGURE 1 (SHEET 1 OF 2)

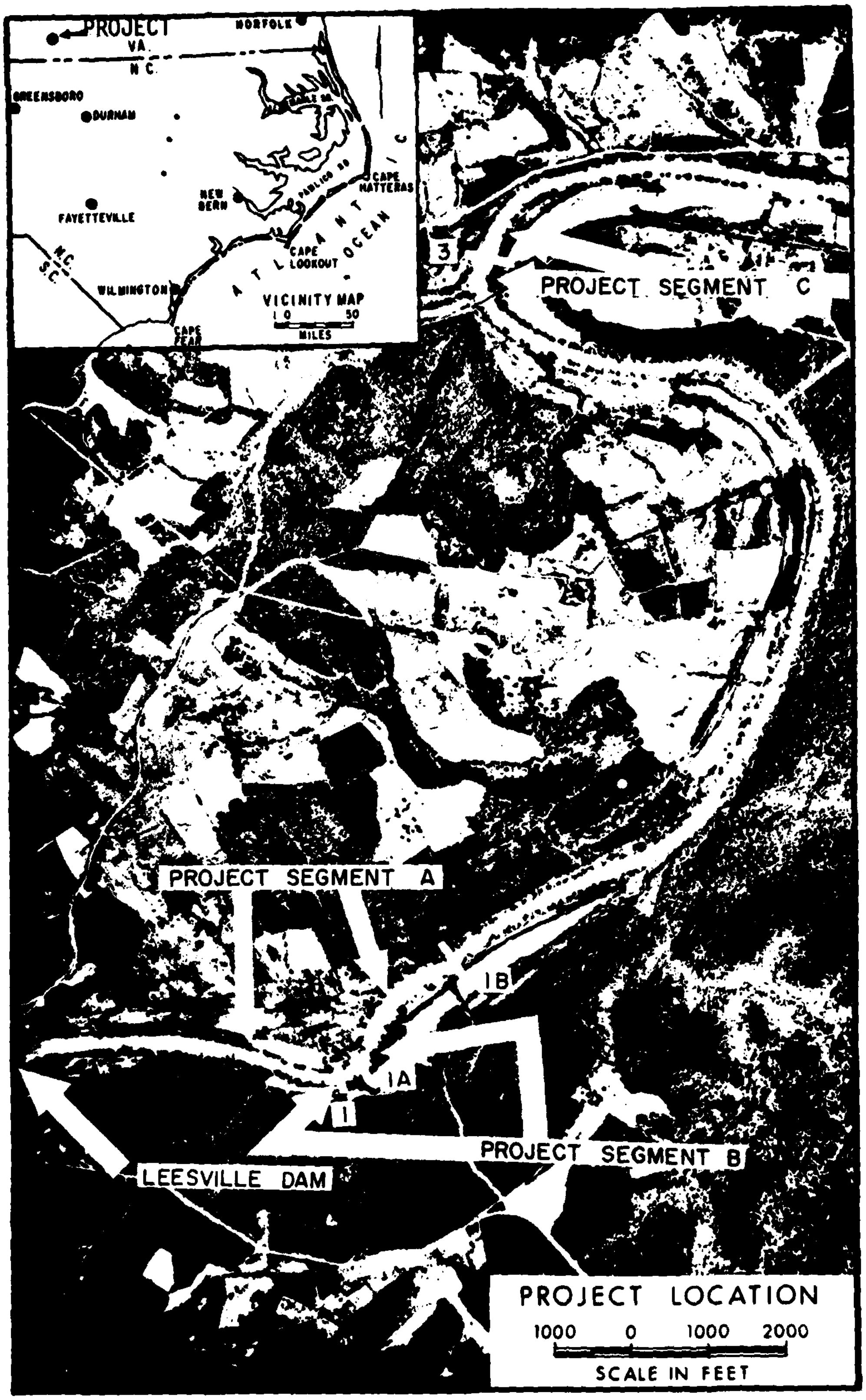


FIGURE 1 (SHEET 2 OF 2)

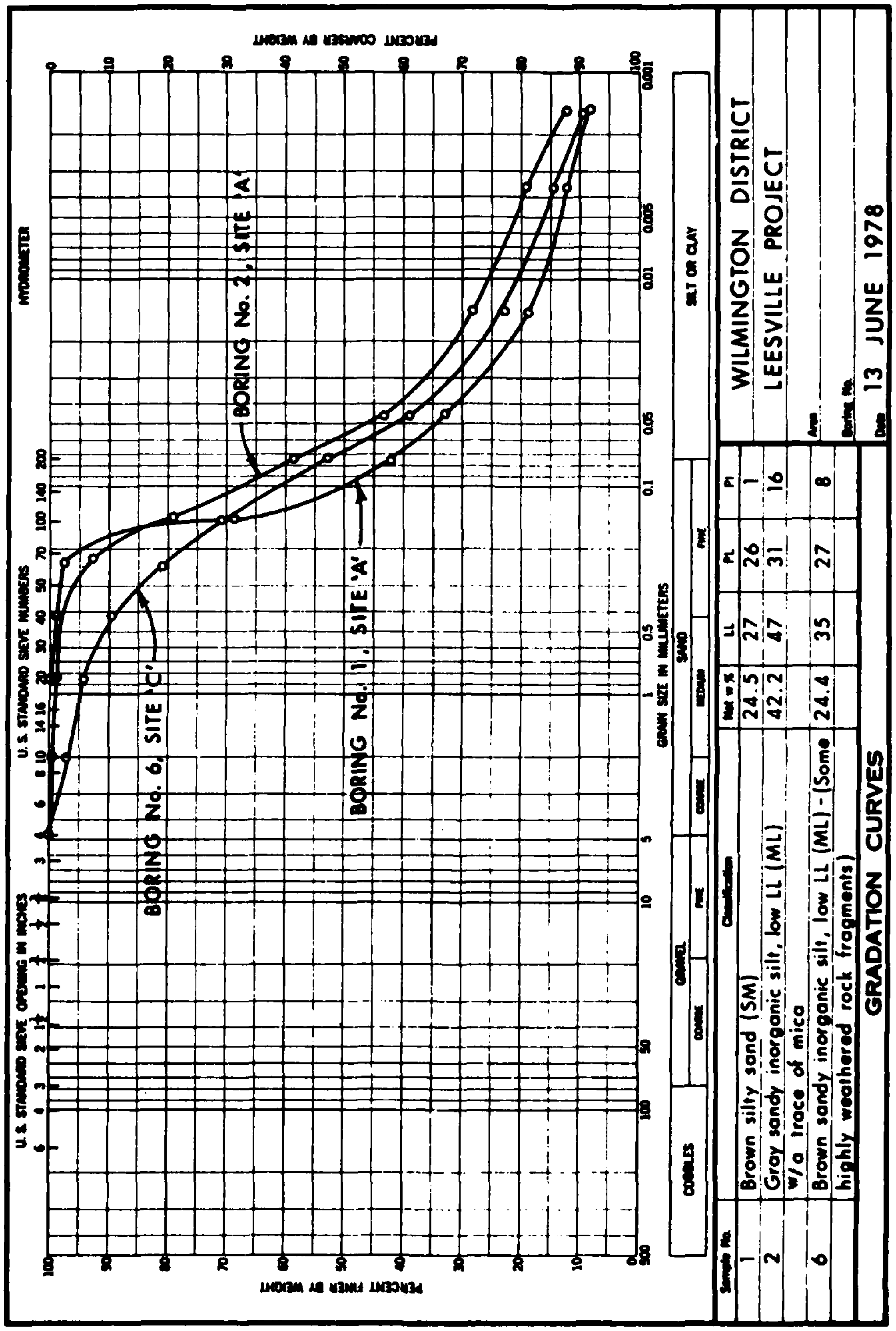
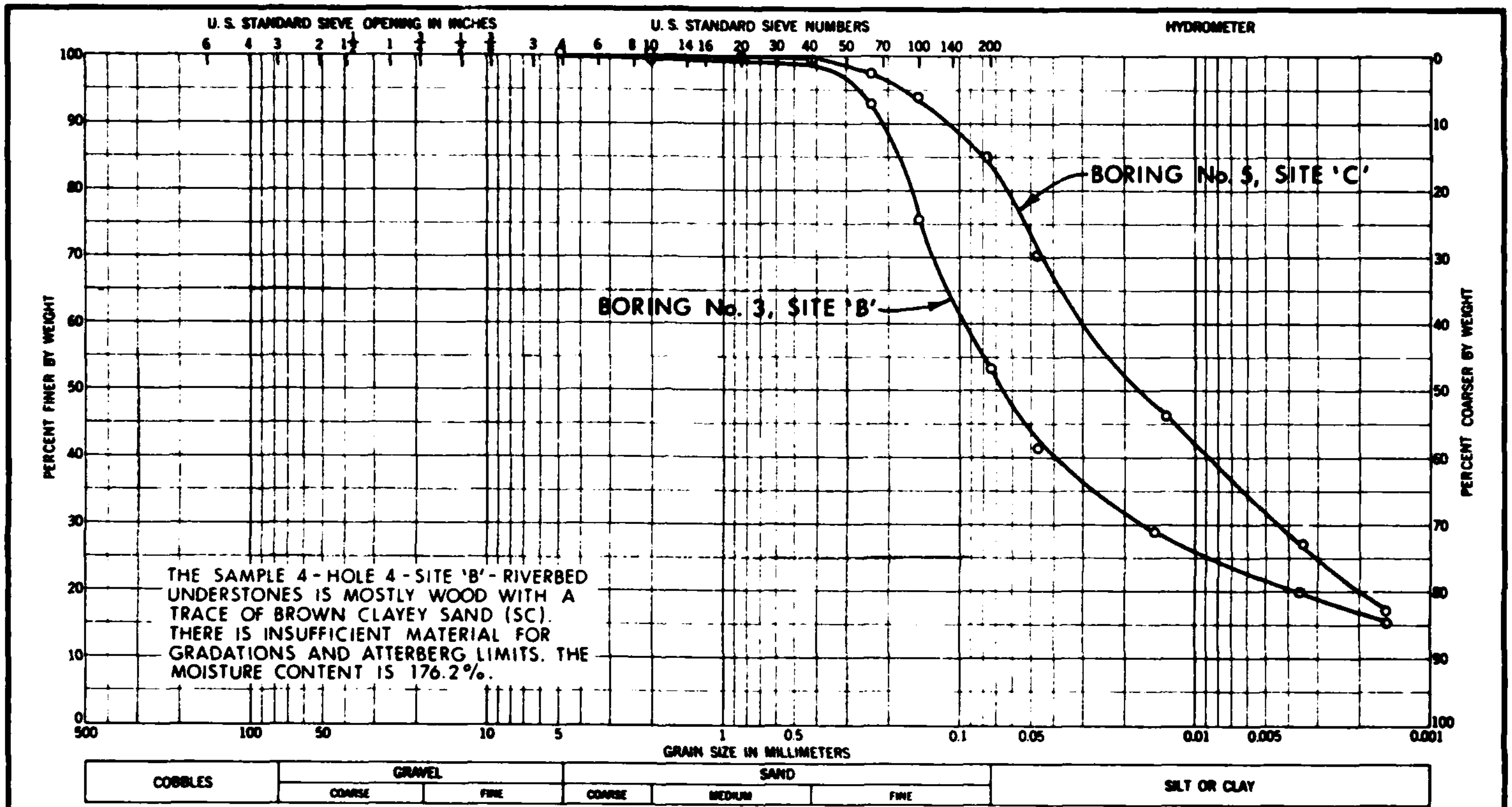


FIGURE 2 (SHEET 1 OF 2)



Sample No.	Classification	Nat w %	LL	PL	PI
3	Brown sandy lean clay (CL) w/a trace of mica	30.4	32	21	11
5	Brown inorganic silt, high LL (MH) w/some sand	32.5	51	32	19

WILMINGTON DISTRICT
LEESVILLE PROJECT
Area
Boring No.
Date 13 JUNE 1978

GRADATION CURVES

FIGURE 2 (SHEET 2 OF 2)

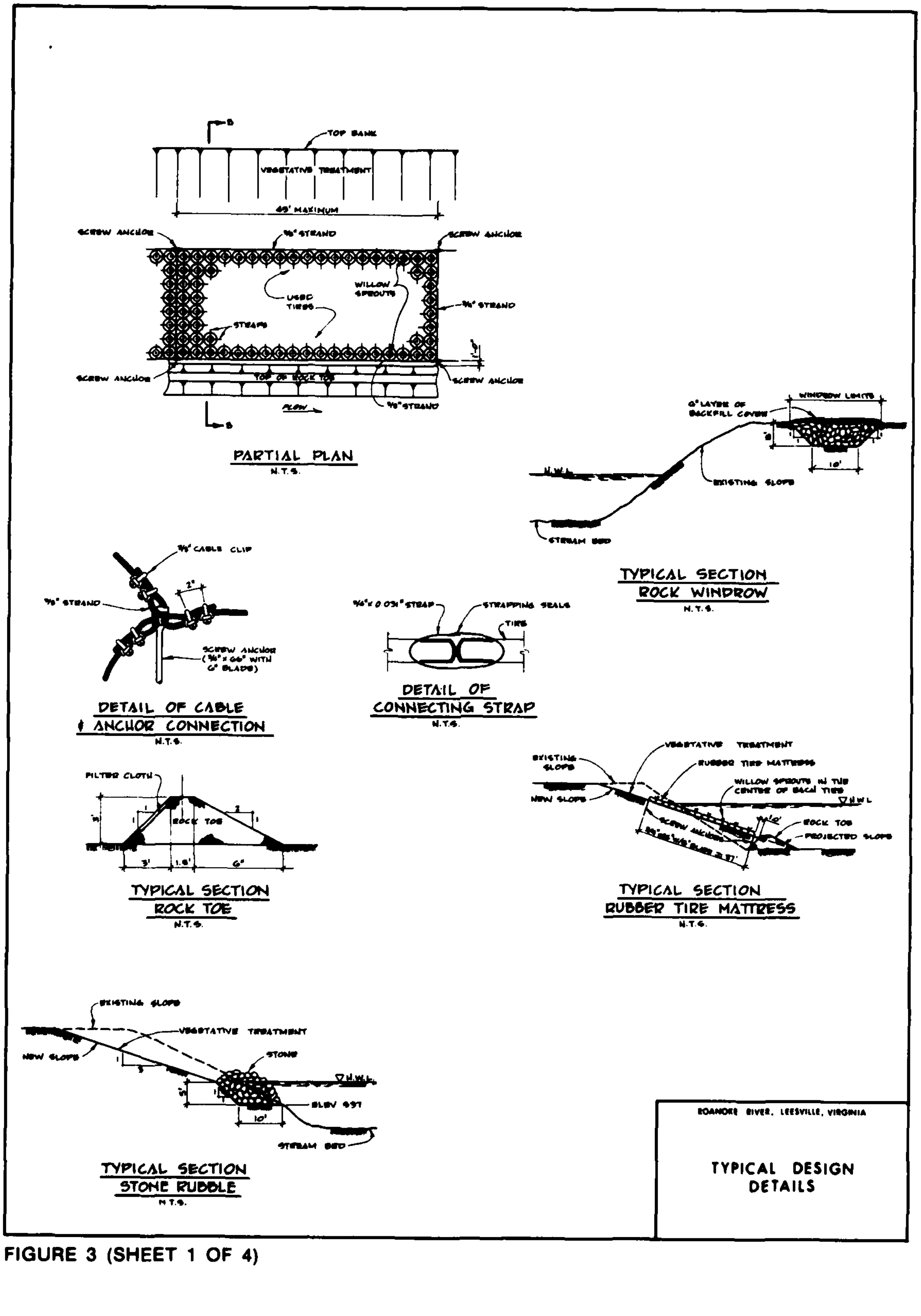


FIGURE 3 (SHEET 1 OF 4)

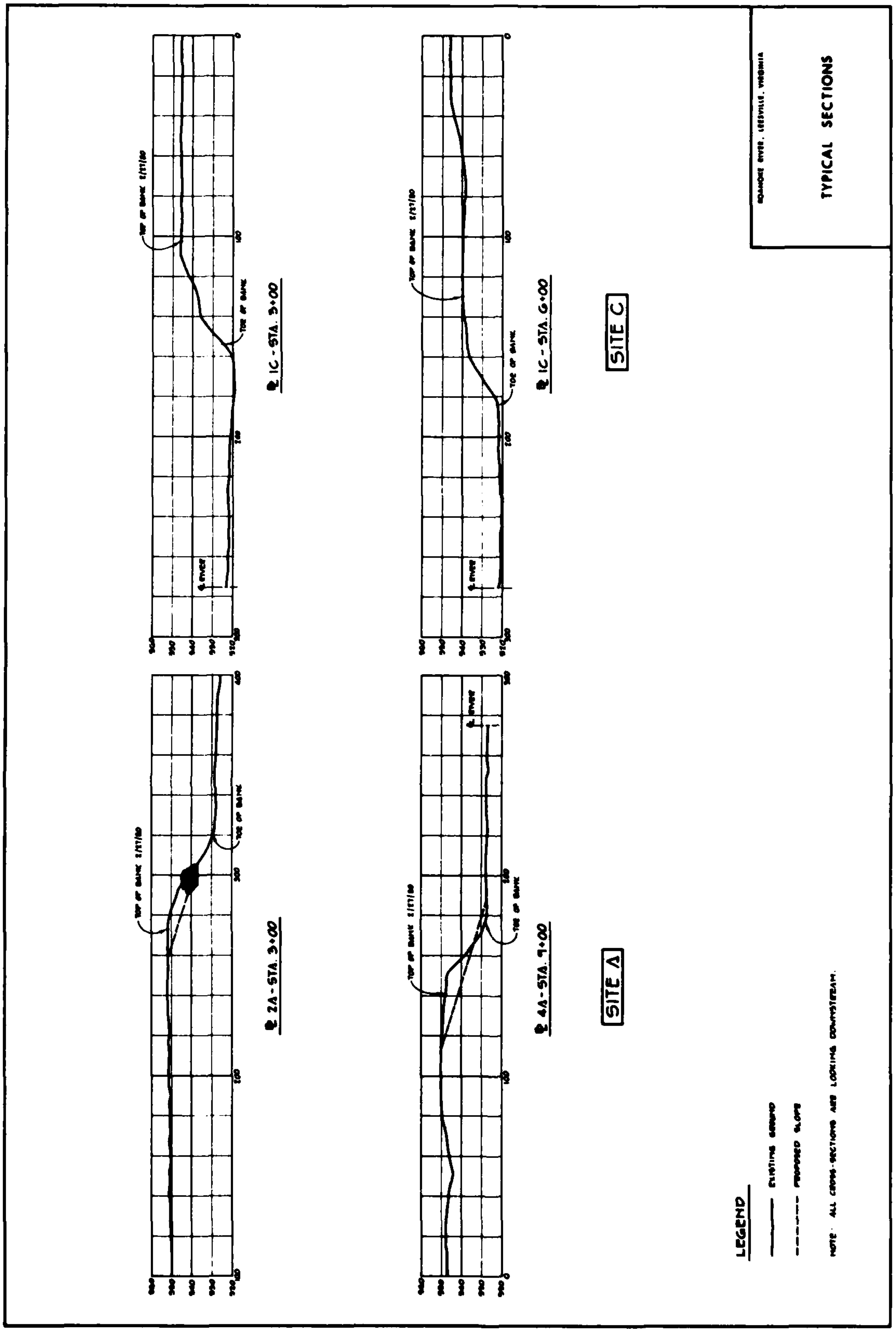


FIGURE 3 (SHEET 2 OF 4)

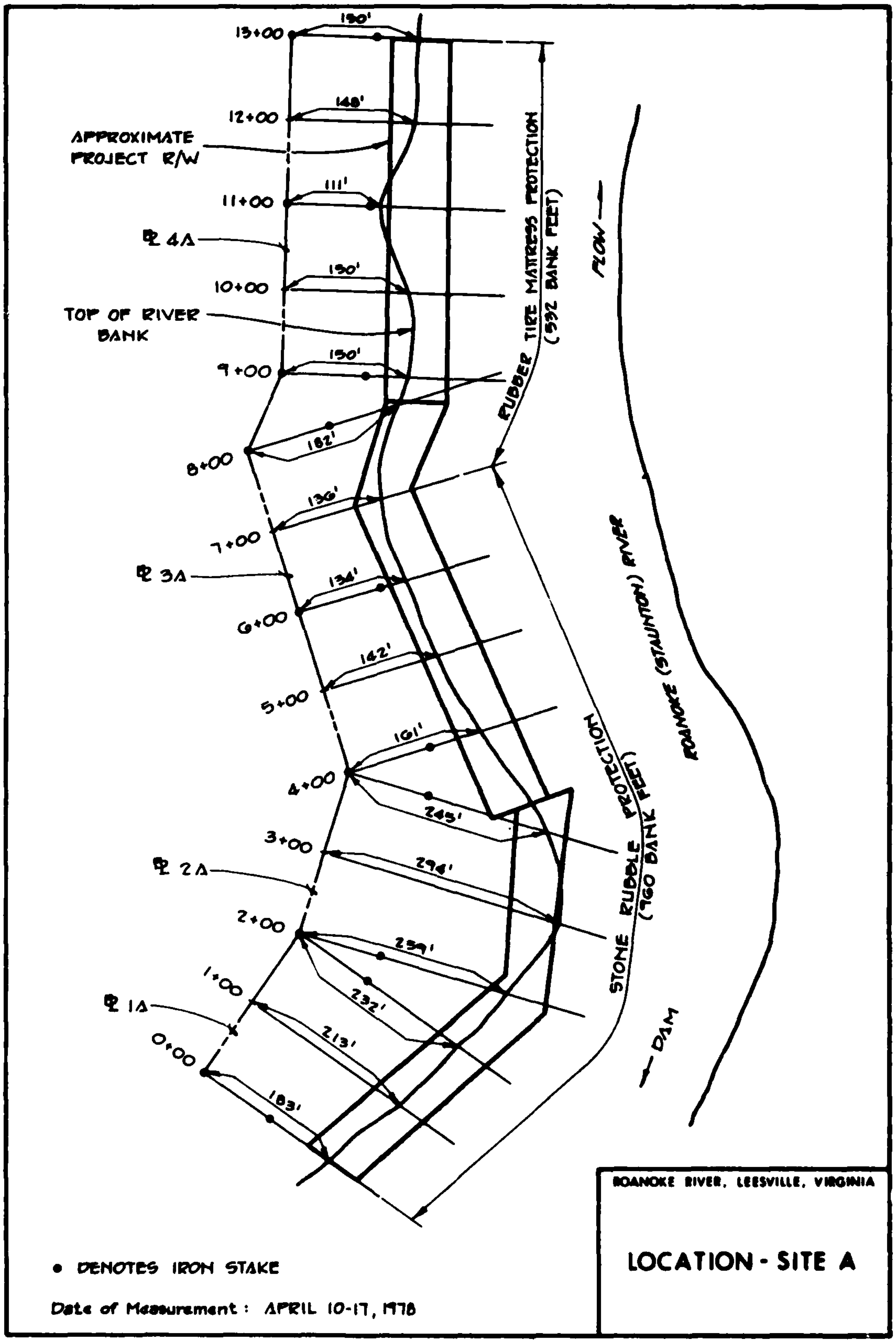


FIGURE 3 (SHEET 3 OF 4)

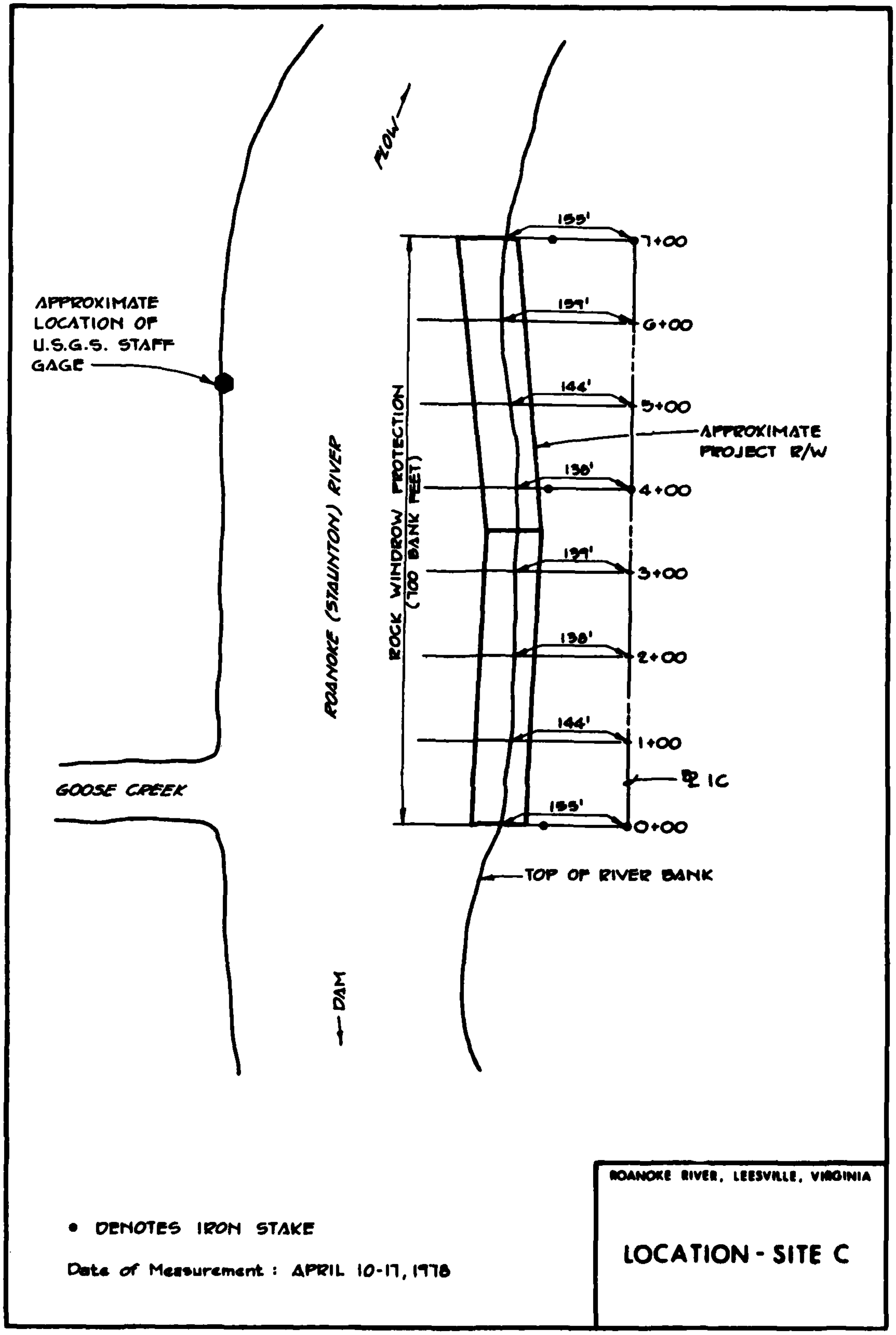


FIGURE 3 (SHEET 4 OF 4)

United States Department of the Interior

GEOLOGICAL SURVEY

WATER RESOURCES DIVISION
200 West Grace Street
Richmond, Virginia 23220

June 26, 1978

District Engineer
Wilmington District, Corps of Engineers
P.O. Box 1890
Wilmington, North Carolina 28402

Attention: Rex Phillips

Dear Sir:

In reference to our telephone conversation of today, I am enclosing the stage-velocity curves, the velocity distribution curves, and copies of the discharge measurements for Roanoke River below Leesville Dam made June 8, 1978. I am also returning your copy of instruction report H-77-1 and copies of our work curves showing the velocity data points. As we discussed, we were unable to complete the measurement at the higher discharge, however, we were able to determine the increase in stage at each site (4.0 ft at upper site and 4.3 ft at lower site) from which we were able to complete the stage-velocity curves.

As indicated by these curves, the mean velocity for the higher discharge would only increase about 1 foot per second. It appears that the velocities are not as much a problem as the alternately wetting and drying of the banks which causes them to cave in.

The results we have gotten look good and I regret we were unable to complete the higher measurement. I am inclined to believe these results will be sufficient for your needs. Please call me when you get these curves to let me know what you think.

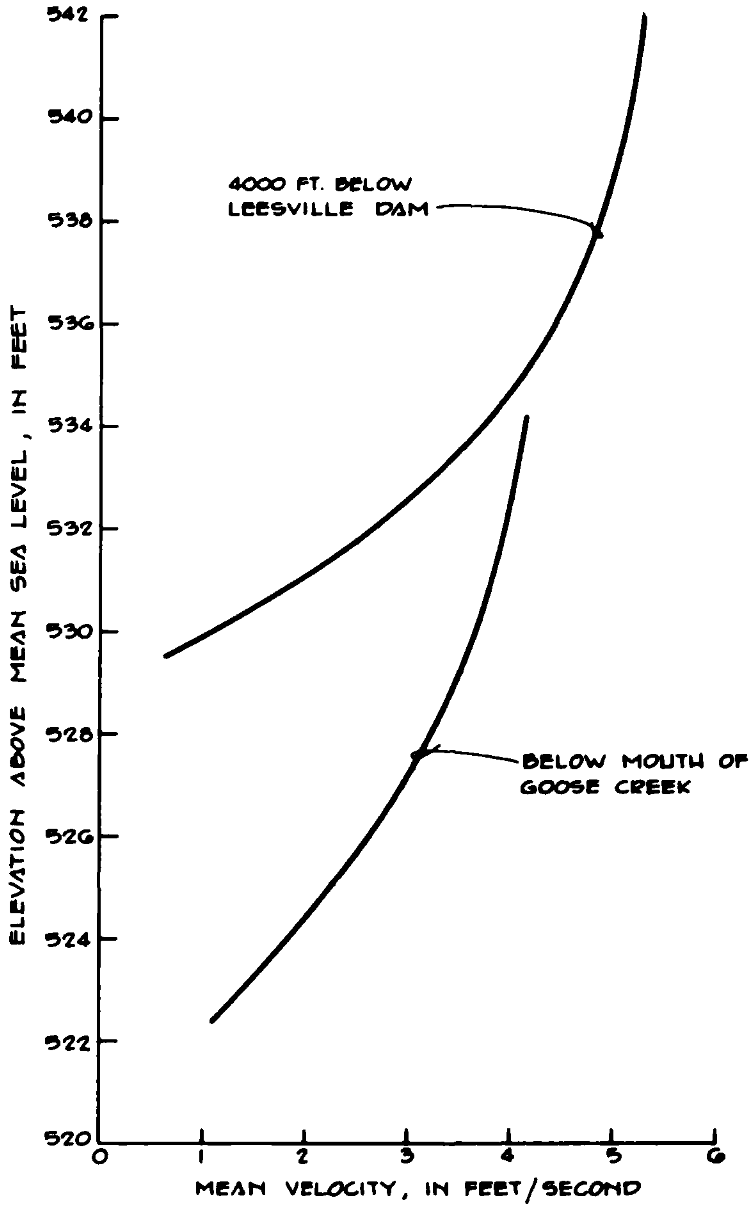
For the District Chief

Sincerely yours,

Prentis M. Frye
Supervisory Hydrologist

Enclosures

FIGURE 4 (SHEET 1 OF 3)

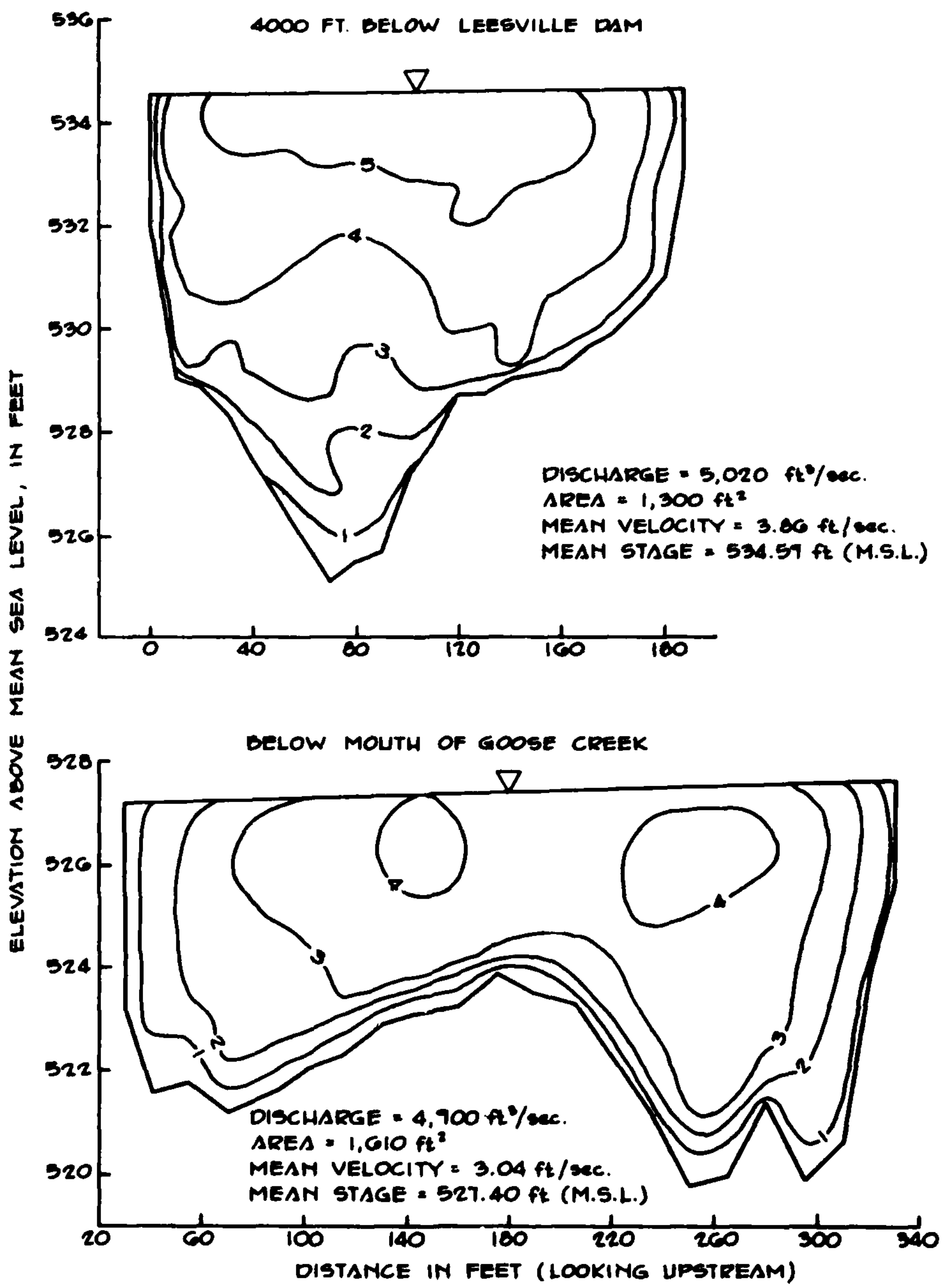


NOTE: CURVES DERIVED FROM ENCLOSURES TO LETTER (FIGURE 4, SHEET 1 OF 3).

ROANOKE RIVER, LEESVILLE, VIRGINIA

VERTICAL VELOCITY DISTRIBUTION

FIGURE 4 (SHEET 2 OF 3)



NOTE: CURVES DERIVED FROM ENCLOSURES TO LETTER (FIGURE 4, SHEET 1 OF 3).

LEGEND

▽ WATER SURFACE

—3— VELOCITY CURVE, IN FT/SEC

ROANOKE RIVER, LEESVILLE, VIRGINIA

VELOCITY CURVES

FIGURE 4 (SHEET 3 OF 3)

02060500 ROANOKE (STAUNTON) RIVER AT ALTAVISTA, VA

LOCATION.--Lat 37°06'16", long 79°17'44", Pittsylvania County, Hydrologic Unit 03010101, on right bank 12 ft (4 m) upstream from bridge on U.S. Highway 29, 0.3 mi (0.5 km) south of Altavista, 0.3 mi (0.5 km) downstream from Sycamore Creek, 3.5 mi (5.6 km) upstream from Big Otter River, and at mile 286.5 (461.0 km).

DRAINAGE AREA.--1,789 mi^2 (4,634 km^2).

WATER-DISCHARGE RECORDS

PERIOD OF RECORD.--August 1930 to current year.

REVISED RECORDS.--WSP 892: 1938(M). WSP 972: 1931-33. WSP 2104: Drainage area.

GAGE.--Water-stage recorder. Datum of gage is 503.10 ft (153.345 m) above mean sea level. Prior to Feb. 21, 1951, on left bank 50 ft (15 m) downstream at same datum.

REMARKS.--Records good. Flow regulated since 1962 by Leesville Lake (station 02059400) 9.5 mi (15.3 km) upstream and since 1963 by Smith Mountain Lake (station 02057400) 27.5 mi (44.2 km) upstream. Gage-height telemeters at station.

AVERAGE DISCHARGE.--47 years, 1,832 ft^3/s (51.88 m^3/s), 13.91 in/yr (353 mm/yr), adjusted for storage.

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 105,000 ft^3/s (2,970 m^3/s) Aug. 15, 1940, gage height, 40.08 ft (12.216 m), from floodmark, from rating curve extended above 52,000 ft^3/s (1,500 m^3/s) on basis of unit hydrograph and flood-routing studies by Corps of Engineers and records for other stations in Roanoke River basin; minimum, 13 ft^3/s (0.37 m^3/s) Jan. 30, 1966; minimum daily, 39 ft^3/s (1.10 m^3/s) July 10, 1966; minimum gage height, 1.66 ft (0.506 m) Jan. 31, 1934, result of freezeup.

EXTREMES FOR CURRENT YEAR.--Maximum discharge, 15,800 ft^3/s (447 m^3/s) Apr. 5, gage height, 16.97 ft (5.172 m); minimum, 74 ft^3/s (2.10 m^3/s) Jan. 2, gage height, 1.53 ft (0.466 m), result of freezeup; minimum daily, 125 ft^3/s (3.54 m^3/s) Sept. 5.

DISCHARGE, IN CUBIC FEET PER SECOND, WATER YEAR OCTOBER 1976 TO SEPTEMBER 1977
MEAN VALUES

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1580	1770	2390	195	974	492	997	951	962	826	734	782
2	271	1790	1320	196	987	1050	245	948	937	182	767	1010
3	281	1310	1100	1250	1100	974	252	939	1100	142	738	185
4	830	1380	226	1180	1030	932	6440	933	199	157	792	129
5	811	1380	227	1240	297	250	14300	904	178	2100	1040	125
6	794	471	1010	1330	283	279	11600	1590	850	802	160	741
7	980	248	5430	1440	991	1050	10300	986	729	514	132	1030
8	1360	1150	6690	368	939	992	6420	1030	752	734	714	1490
9	7180	1490	6520	202	918	963	5290	1010	772	176	916	1450
10	5460	943	5560	2230	982	955	2490	985	1420	144	802	353
11	5610	1230	819	3210	1020	944	4790	942	202	786	791	194
12	5850	1310	511	3240	275	226	2160	867	148	903	486	836
13	3680	377	6050	2870	264	634	2290	895	764	899	289	787
14	2440	227	3390	939	1060	3260	1810	759	957	852	176	782
15	1240	1140	2500	358	1450	2760	1530	763	959	1250	800	770
16	214	1160	5130	285	846	3290	456	928	956	214	800	942
17	270	1080	2770	4640	2020	1230	918	826	921	144	834	195
18	1120	1030	593	1980	609	1330	1050	824	221	877	895	165
19	1250	1160	309	1380	210	422	907	801	155	834	1030	784
20	5980	380	1440	1220	238	469	872	828	396	645	301	811
21	5430	199	1410	882	971	5200	987	853	1270	786	142	812
22	4370	453	1360	232	956	3850	1090	867	836	976	718	1080
23	768	495	1990	212	974	3860	888	882	816	166	789	728
24	319	1320	432	956	1080	1710	1050	870	1750	130	786	149
25	1660	378	246	1010	1060	1680	1100	971	355	776	407	139
26	7840	1220	316	979	267	581	1220	933	217	790	1000	771
27	7210	311	2190	974	248	314	1380	830	796	808	185	825
28	6020	356	2420	1120	1010	1290	3510	798	812	798	134	779
29	5590	3810	2770	223	---	2510	1750	813	993	933	731	777
30	2710	3560	1560	329	---	1430	972	867	1100	180	780	986
31	782	---	418	1020	---	1190	---	926	---	141	441	---
TOTAL	89880	34160	69097	37690	22979	45817	89464	28327	22523	19665	20638	20607
MEAN	2899	1139	2229	1216	821	1478	2982	914	751	634	666	687
MAX	7840	3810	6690	4640	2020	5200	14300	1590	1750	2100	1044	1490
MIN	214	199	226	195	210	226	245	759	148	130	132	125
(*)	+1081	+110	-230	-346	+111	+350	-25	-60	-156	-506	-485	-267
MEAN†	3980	1249	1999	870	932	1828	2957	854	595	128	181	420
CFSM†	2.22	.70	1.12	.49	.52	1.02	1.65	.48	.33	.07	.10	.23
IN†	2.57	.78	.29	.56	.54	1.18	1.84	.55	.37	.08	.12	.26

CAL YR 1976 TOTAL 554879 MEAN 1516 MAX 7840 MIN 172 MEAN† 1522 CFSM† .85 IN† 11.58
WTR YR 1977 TOTAL 500855 MEAN 1372 MAX 14300 MIN 125 MEAN† 1337 CFSM† .75 IN† 10.15

* Change in contents, equivalent in cubic feet per second, in Smith Mountain and Leesville Lakes; furnished by Appalachian Power Co.
† Adjusted for change in contents.

FIGURE 5 (SHEET 1 OF 4)

02060500 ROANOKE (STAUNTON) RIVER AT ALTAVISTA, VA--Continued

WATER-QUALITY RECORDS

PERIOD OF RECORD.--Water years 1951, 1953-56, 1968 to current year.

PERIOD OF DAILY RECORD.--
SPECIFIC CONDUCTANCE: October 1950 to September 1951, February 1953 to September 1956, April 1968 to current year.
WATER TEMPERATURES: October 1950 to September 1951, February 1953 to September 1956, April 1968 to current year.
SUSPENDED-SEDIMENT DISCHARGE: February 1953 to September 1956.

REMARKS.--Frequency of sample analysis changed from twice monthly to once monthly in March.

EXTREMES FOR PERIOD OF DAILY RECORD.--
SPECIFIC CONDUCTANCE: Maximum, 580 micromhos Jan. 17, 1969; minimum, 54 micromhos Aug. 18, 1955.
WATER TEMPERATURES: Maximum, 30.0°C Aug. 10, 1951; minimum, 0.0°C on many days during winter period.

EXTREMES FOR CURRENT YEAR.--
SPECIFIC CONDUCTANCE: Maximum, 540 micromhos Jan. 31; minimum, 83 micromhos Dec. 28, Jan. 4.
WATER TEMPERATURES: Maximum, 27.0°C July 29; minimum, 3.0°C Jan. 18.

WATER QUALITY DATA, WATER YEAR OCTOBER 1975 TO SEPTEMBER 1976

DATE	TIME	DIS-CHARGE (CFS)	SPE-CIFIC CON-DUCT-ANCE (MICRO-MHOS)	COLOR (PLAT-INUM-COBALT UNITS)	HARD-NESS (CA,MG) (MG/L)	NON-CAR-BONATE HARD-NESS (MG/L)	DIS-SOLVED CAL-CIUM (CA) (MG/L)	DIS-SOLVED MAG-NE-SIUM (MG) (MG/L)	DIS-SOLVED SODIUM (NA) (MG/L)	DIS-SOLVED PO-TAS-SIUM (K) (MG/L)
OCT										
01...	1900	4580	126	0	59	2	17	4.0	4.5	2.2
15...	1900	895	138	40	59	11	15	5.3	4.9	2.2
NOV										
01...	1500	204	164	5	67	18	18	5.3	5.0	2.2
15...	1500	383	166	0	68	17	17	6.3	4.8	2.2
DEC										
01...	1500	1990	105	10	57	8	16	4.2	5.1	2.2
15...	1500	214	100	10	53	7	14	4.3	6.0	2.0
JAN										
01...	1500	6970	103	50	52	11	13	4.7	4.1	2.6
15...	1500	3530	140	30	55	10	13	5.4	4.2	2.0
FEB										
01...	1500	391	305	5	52	13	13	4.8	33	1.9
15...	1500	259	139	0	53	11	13	5.0	4.5	2.0
MAR										
15...	1500	3990	140	0	54	10	13	5.2	5.8	1.9
APR										
15...	1900	1530	155	0	57	6	15	4.9	4.8	2.5
MAY										
15...	1900	875	150	0	60	2	16	4.9	5.0	2.6
JUN										
15...	1900	2120	150	0	61	3	15	5.8	5.4	2.3
JUL										
15...	1900	3140	130	0	66	4	16	5.8	5.9	2.4
AUG										
15...	0700	160	158	0	66	6	17	5.8	5.6	2.4
SEP										
15...	0700	611	160	5	68	6	18	5.6	5.7	2.6

FIGURE 5 (SHEET 2 OF 4)

02060500 ROANOKE (STAUNTON) RIVER AT ALTAVISTA, VA--Continued

WATER QUALITY DATA, WATER YEAR OCTOBER 1975 TO SEPTEMBER 1976

DATE	BICAR-BONATE (HCO3) (MG/L)	DIS-SOLVED SULFATE (SO4) (MG/L)	DIS-SOLVED CHLO-RIDE (CL) (MG/L)	DIS-SOLVED FLUO-RIDE (F) (MG/L)	DIS-SOLVED SILICA (SIO2) (MG/L)	DIS-SOLVED SOLIDS (RESI-DUE AT 180 C) (MG/L)	DIS-SOLVED NITRATE (N) (MG/L)	DIS-SOLVED ORTHO. PHOS-PHORUS (P) (MG/L)	DIS-SOLVED IRON (FE) (UG/L)
OCT									
01...	69	8.7	5.5	.1	8.6	99	.36	.00	0
15...	59	9.4	5.4	.1	9.0	75	.38	.01	10
NOV									
01...	60	7.4	4.8	.1	14	99	.24	.01	70
15...	63	9.7	6.1	.1	7.8	98	.30	.00	10
DEC									
01...	60	6.5	5.4	.2	13	88	.49	.01	10
15...	56	9.6	5.1	.1	13	86	.34	.10	90
JAN									
01...	50	9.6	5.6	.2	8.8	90	.46	.01	70
15...	54	8.5	5.4	.2	7.5	94	.37	.01	30
FEB									
01...	48	9.2	54	.2	12	160	.56	.00	40
15...	51	8.2	5.2	.3	7.6	68	.45	.00	10
MAR									
15...	54	8.6	6.2	.3	7.2	70	.42	.00	10
APR									
15...	62	7.1	5.2	.1	10	83	.57	.00	0
MAY									
15...	70	7.0	4.8	.1	9.8	80	.24	.00	0
JUN									
15...	71	10	5.9	.3	6.2	81	.41	.01	0
JUL									
15...	73	10	6.4	.2	6.5	73	.41	.02	0
AUG									
15...	73	8.7	6.0	.1	8.1	104	.37	.01	0
SEP									
15...	76	8.9	6.5	.2	8.6	102	.33	.00	0

SPECIFIC CONDUCTANCE (MICROMHOS/CM AT 25 DEG. C), WATER YEAR OCTOBER 1975 TO SEPTEMBER 1976
ONCE-DAILY

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	126	166	105	103	305	120	140	119	135	135	160	158
2	142	132	115	90	140	115	150	120	115	138	160	158
3	130	178	105	90	120	122	150	130	130	150	163	157
4	132	178	110	83	120	---	155	119	140	160	161	160
5	140	179	108	90	120	122	158	151	118	140	157	160
6	140	172	110	120	116	112	143	121	140	160	150	155
7	134	174	115	140	118	108	165	153	140	159	150	157
8	120	162	110	130	---	125	125	142	123	160	151	160
9	128	160	105	115	122	120	125	141	138	159	162	159
10	108	158	115	125	138	110	145	120	115	159	160	160
11	135	170	100	125	130	115	140	161	140	160	161	143
12	135	172	112	110	125	115	145	162	142	160	162	155
13	128	173	95	120	118	---	150	148	150	160	161	160
14	137	177	110	110	110	138	140	---	120	---	162	159
15	138	166	100	140	138	140	155	150	158	130	158	160
16	120	176	108	115	---	128	150	161	140	---	128	140
17	132	175	100	100	---	138	158	123	125	162	130	160
18	120	172	110	110	108	120	145	123	125	162	160	157
19	140	166	110	120	120	158	130	126	130	160	145	140
20	140	166	98	135	112	---	155	122	125	163	160	160
21	132	164	108	120	122	---	150	171	155	142	158	152
22	108	156	95	110	128	---	158	150	150	160	155	145
23	118	158	118	112	118	---	142	126	142	158	150	150
24	142	172	110	118	---	120	160	139	125	147	160	130
25	120	170	102	110	120	145	163	148	142	148	160	147
26	160	166	118	120	125	138	131	160	110	---	130	150
27	167	158	85	110	110	123	126	133	---	155	145	135
28	173	155	83	120	130	140	125	160	122	157	140	158
29	177	166	98	110	110	115	157	118	140	150	140	150
30	178	165	103	120	---	103	155	119	128	161	160	130
31	146	---	100	540	---	125	---	140	---	161	145	---
MONTH	137	167	105	128	129	124	146	138	133	154	153	152

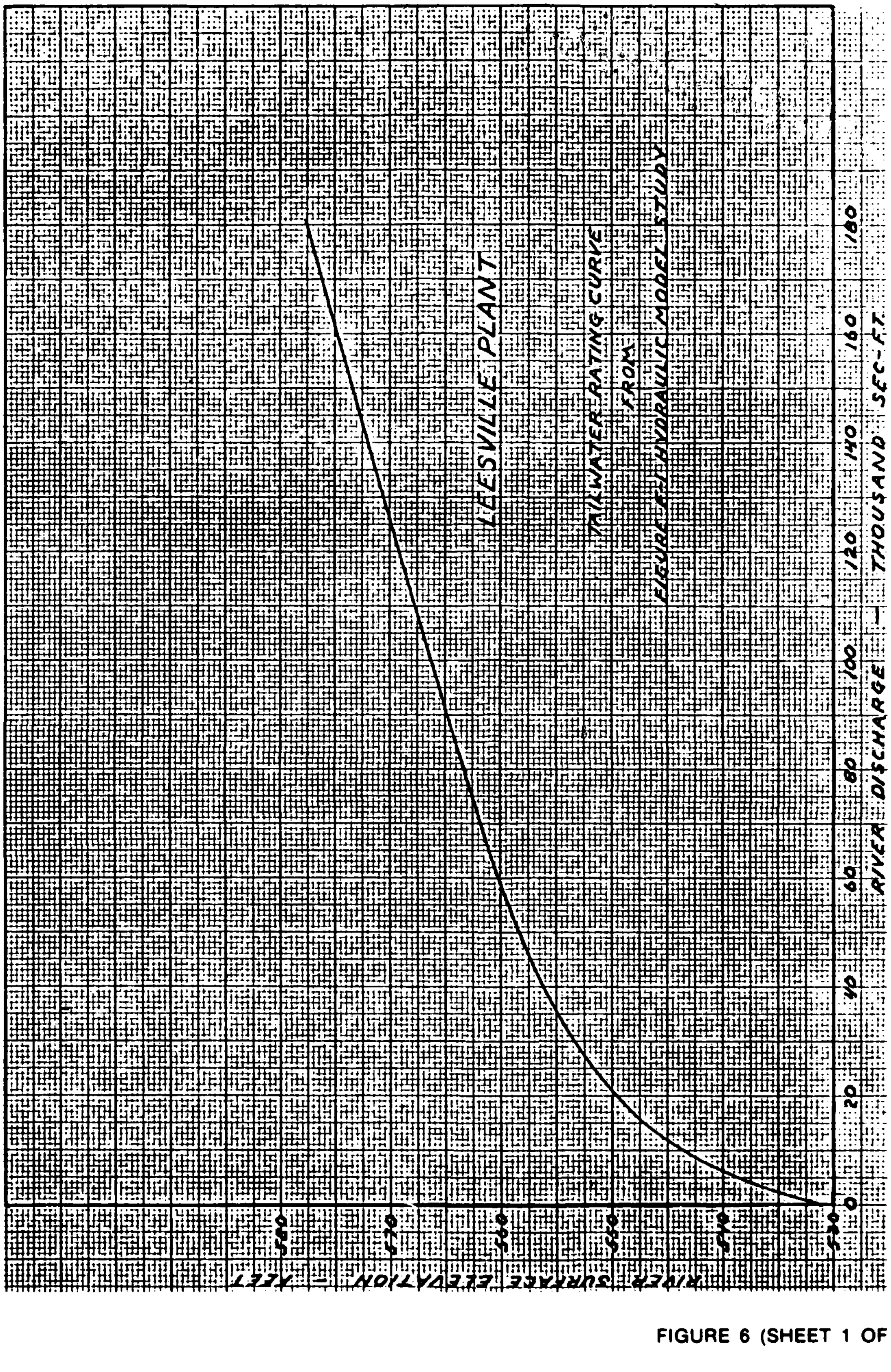
FIGURE 5 (SHEET 3 OF 4)

02060500 ROANOKE (STAUNTON) RIVER AT ALTAVISTA, VA--Continued

TEMPERATURE (DEG. C) OF WATER, WATER YEAR OCTOBER 1975 TO SEPTEMBER 1976
ONCE-DAILY

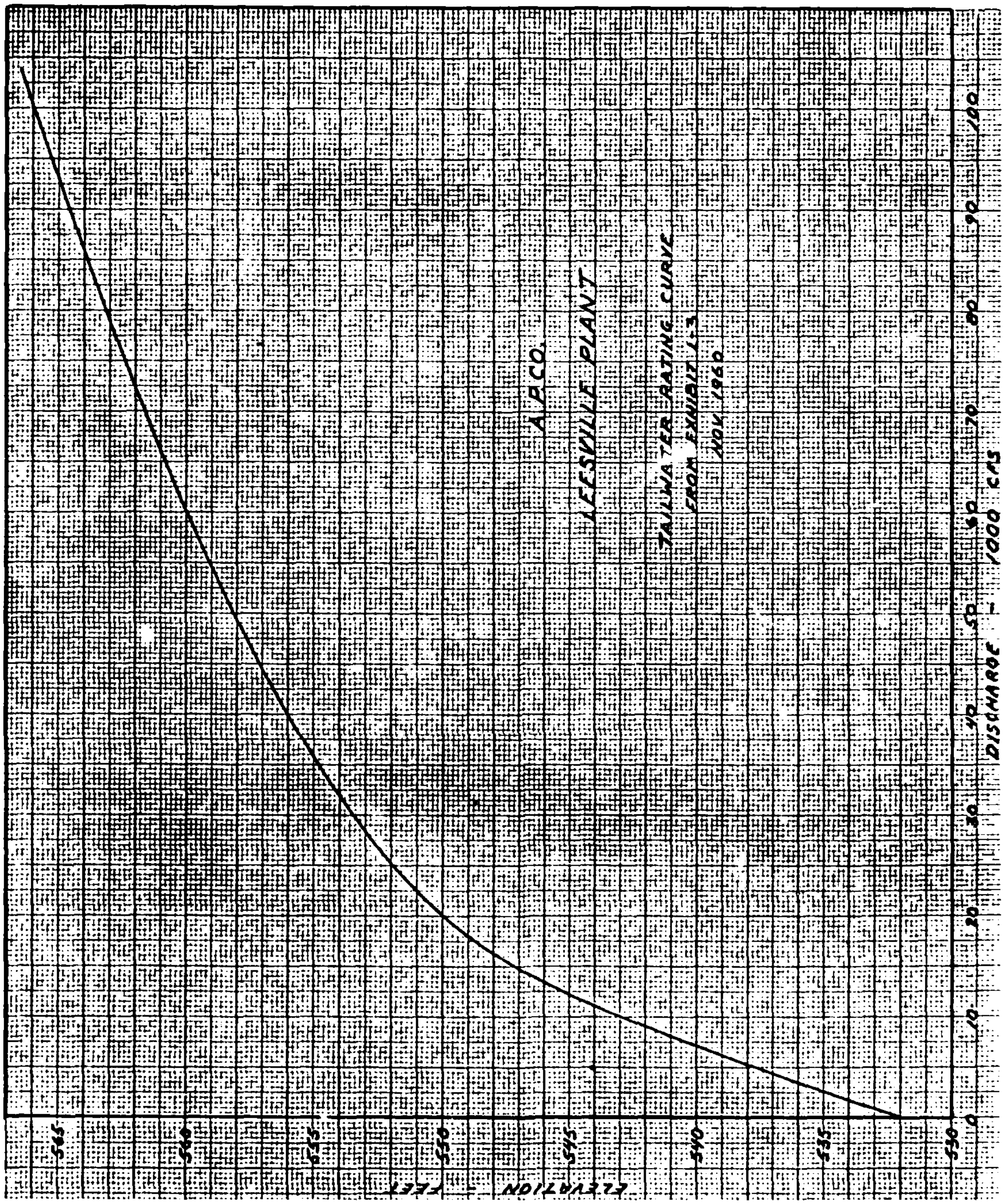
DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	20.0	18.0	11.0	9.0	3.5	12.5	12.0	18.0	18.0	21.0	26.0	22.0
2	20.0	17.0	11.0	9.0	5.0	13.0	12.0	17.0	18.5	21.5	23.0	22.5
3	20.0	18.0	13.0	9.0	6.0	13.5	15.0	18.0	18.0	22.0	23.5	23.0
4	19.0	18.0	13.0	6.5	7.0	---	15.0	18.0	18.5	22.0	22.0	25.0
5	20.0	19.0	12.5	6.5	7.0	14.0	13.5	17.0	18.0	23.0	22.5	25.5
6	21.0	18.0	12.5	8.0	8.0	13.0	15.0	18.0	20.0	21.0	22.0	23.0
7	21.0	19.0	12.0	8.0	6.0	13.5	15.0	18.0	18.0	21.5	24.0	23.0
8	20.0	20.0	13.0	7.0	---	13.0	16.0	18.0	18.0	21.0	24.5	22.5
9	21.0	20.5	13.0	7.0	7.0	13.5	16.0	17.0	18.5	21.0	23.5	22.0
10	22.0	18.0	12.0	5.0	7.5	14.0	16.0	17.0	19.0	23.0	23.0	22.0
11	21.0	17.0	11.0	5.5	8.0	14.0	14.0	17.0	19.5	24.0	23.0	24.0
12	21.0	18.0	11.0	8.0	8.0	13.5	14.0	18.0	18.0	22.0	24.0	25.5
13	20.0	18.0	13.0	8.0	8.5	---	14.5	18.0	18.0	22.0	24.5	22.0
14	20.0	17.0	13.0	5.0	9.0	14.0	15.0	---	18.5	---	26.0	22.5
15	20.0	16.0	13.0	5.0	10.0	12.0	15.0	19.0	17.0	22.5	26.0	22.0
16	20.0	16.0	11.0	5.0	---	11.5	14.0	19.0	18.0	---	23.0	21.0
17	19.0	17.0	10.0	5.0	---	11.0	16.5	19.0	18.0	24.0	23.5	21.5
18	18.0	17.0	10.0	3.0	9.5	15.0	17.0	19.0	18.5	25.0	23.0	24.0
19	18.0	16.0	7.0	5.0	10.0	15.0	16.0	19.0	18.0	22.5	23.0	24.0
20	20.0	17.0	4.0	5.5	10.0	---	16.0	18.0	19.0	22.0	23.5	20.0
21	20.0	17.0	4.0	5.0	12.0	---	17.5	17.0	19.0	23.5	25.0	20.5
22	19.0	11.0	4.0	5.5	11.0	---	17.0	17.0	20.5	23.5	24.0	20.0
23	19.0	10.0	8.0	5.5	13.0	---	17.0	18.0	20.0	23.0	22.0	20.0
24	18.0	15.0	8.0	5.5	---	11.5	17.0	18.0	21.0	26.5	22.5	20.0
25	18.0	13.0	7.0	5.0	12.5	14.0	17.0	18.0	21.5	26.0	22.0	22.5
26	19.0	13.0	7.0	6.0	12.0	14.0	17.0	18.0	22.0	---	22.5	23.0
27	18.0	13.0	7.0	6.0	12.0	12.0	18.0	18.0	---	23.0	22.0	19.5
28	18.0	13.0	6.0	6.0	11.5	12.0	18.0	18.0	20.0	26.0	25.5	19.0
29	19.0	9.5	8.0	6.0	11.0	14.0	19.0	18.0	20.5	27.0	25.0	19.0
30	18.0	9.0	8.0	6.0	---	14.0	19.0	18.0	21.0	24.5	22.0	19.0
31	18.0	---	9.0	4.5	---	14.0	---	19.0	---	26.0	22.5	---
MONTH	19.5	16.0	10.0	6.0	9.0	13.5	16.0	18.0	19.0	23.0	23.5	22.0

FIGURE 5 (SHEET 4 OF 4)



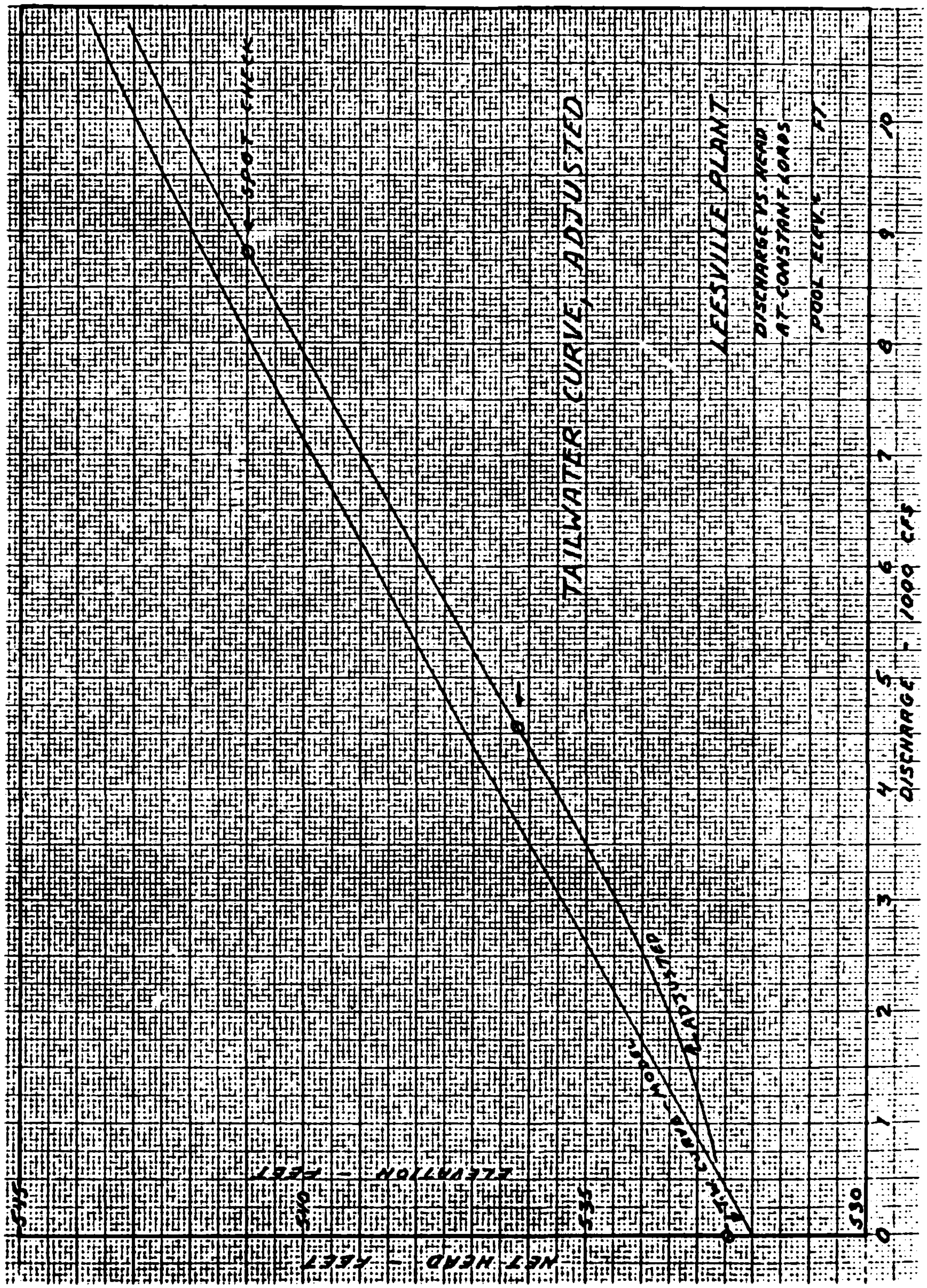
SOURCE: APPALACHIAN POWER CO., ROANOKE, VA

FIGURE 6 (SHEET 1 OF 6)



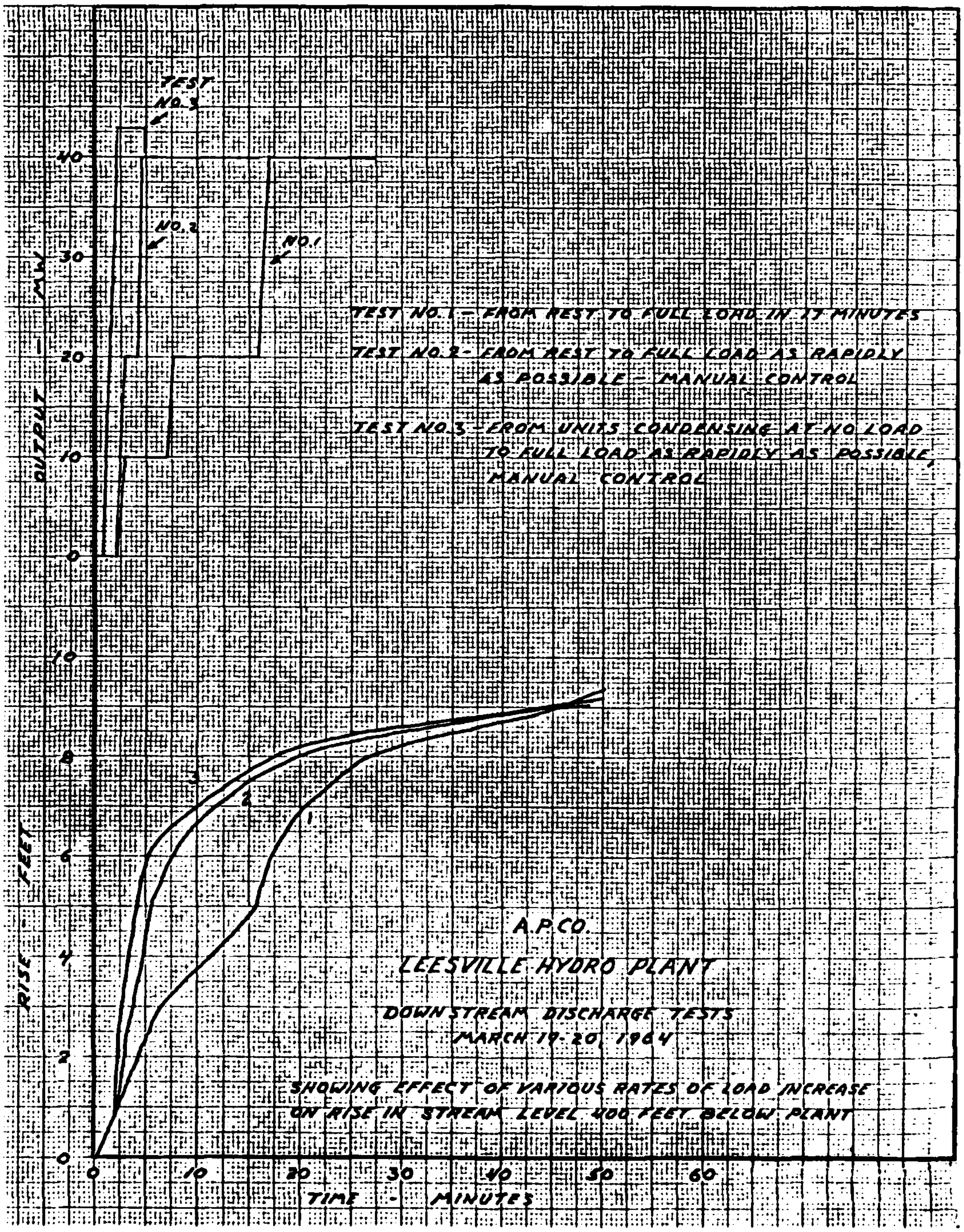
SOURCE: APPALACHIAN POWER CO., ROANOKE, VA

FIGURE 6 (SHEET 2 OF 6)



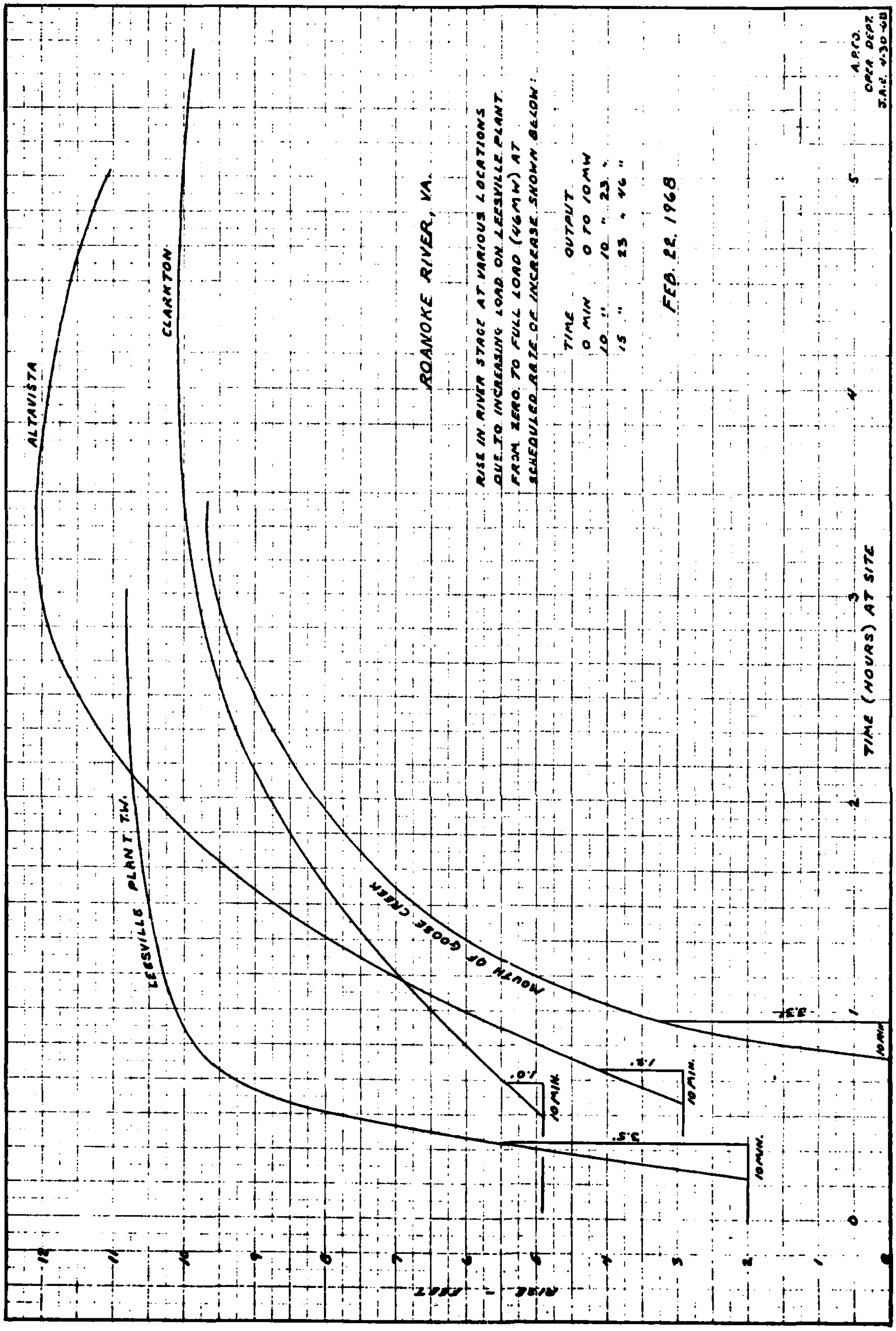
SOURCE: APPALACHIAN POWER CO., ROANOKE, VA

FIGURE 6 (SHEET 3 OF 6)



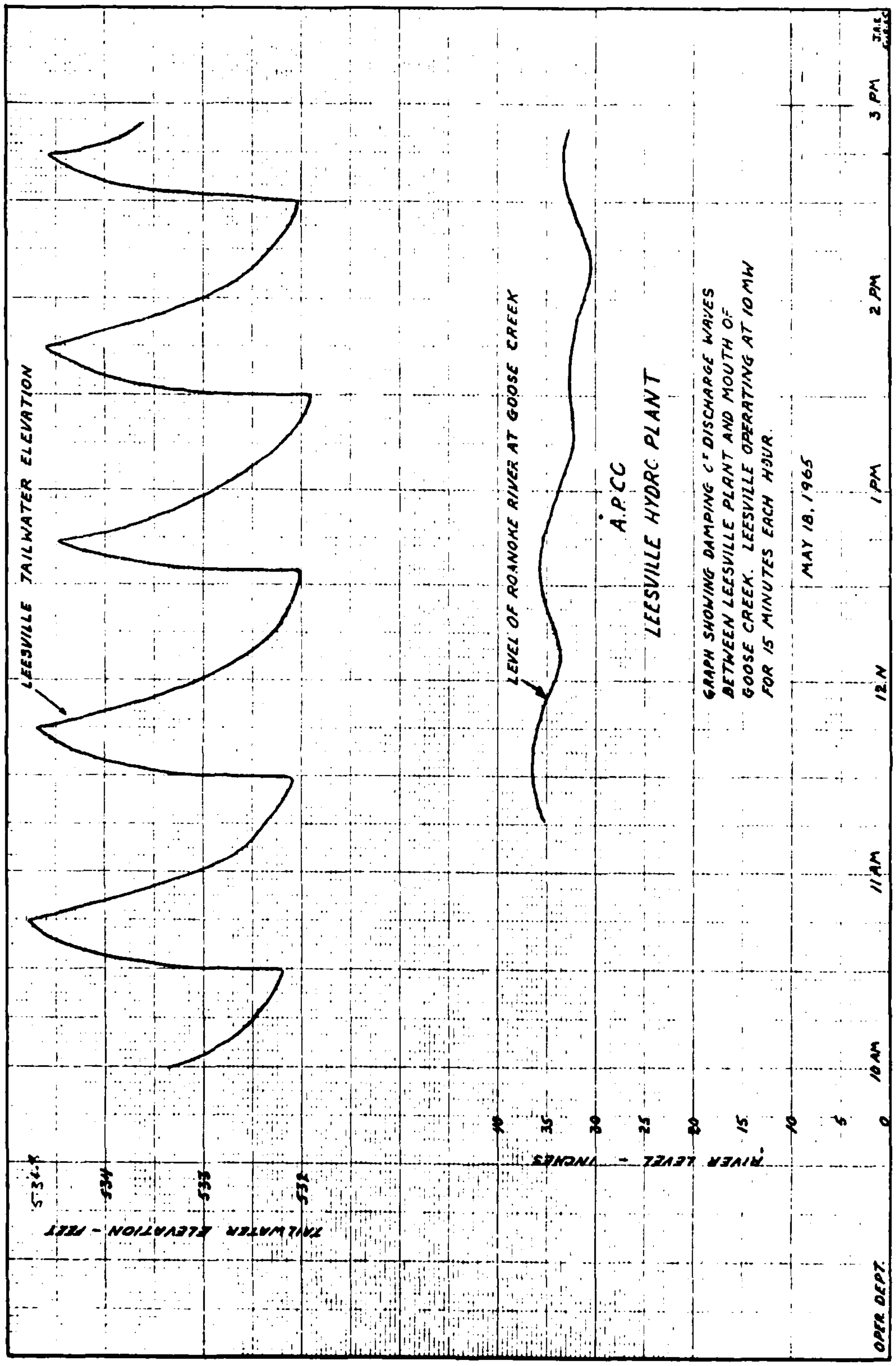
SOURCE: APPALACHIAN POWER CO., ROANOKE, VA

FIGURE 6 (SHEET 4 OF 6)



SOURCE: APPALACHIAN POWER CO. ROANOKE, VA

FIGURE 6 (SHEET 5 OF 6)



SOURCE: APPALACHIAN POWER CO., ROANOKE, VA

FIGURE 6 (SHEET 6 OF 6)

Leesville Dam Tailwater Stages
Minimum and Maximum
January 1978 through December 1980

1978 Month	Highest Average Hourly Discharge (CFS) Day	Discharge	T.W. (max.)	Lowest Average Hourly Discharge (CFS) Day	Discharge	T.W. (max.)
Jan	27	17,404	544.3	21	37	531.8
Feb	1	10,000	539.6	4	37	531.8
Mar	11	10,150	540.1	5	37	531.8
Apr	27	32,178	552.1	16	37	531.8
May	1	10,150	540.0	7	37	531.8
Jun	8	9,913	540.0	8	37	531.8
Jul	13	4,975	536.2	13	37	531.8
Aug	8	9,890	540.0	25	37	531.8
Sep	25	5,145	536.2	5	37	531.8
Oct	2	5,060	536.2	22	74	531.8
Nov	16	4,988	536.2	26	37	531.8
Dec	6	4,950	536.2	24	37	531.8
1979						
Jan	26	9,890	540.0	28	37	531.8
Feb	26	20,936	549.2	22	37	531.8
Mar	24	10,150	540.1	18	37	531.8
Apr	26	10,263	540.1	8	37	531.8
May	16	9,620	540.0	6	37	531.8
Jun	28	9,913	540.0	17	37	531.8
Jul	16	5,145	536.2	14	37	531.8
Aug	14	4,988	536.2	14	37	531.8
Sep	24	16,943	544.4	22	37	531.8
Oct	12	10,150	540.1	14	37	531.8
Nov	29	10,415	540.2	17	37	531.8
Dec	10	5,400	536.3	9	37	531.8
1980						
Jan	15	9,913	540.0	13	37	531.8
Feb	12	7,175	538.1	2	37	531.8
Mar	20	10,150	540.1	16	37	531.8
Apr	15	10,300	540.1	6	37	531.8
May	1	4,988	536.2	8	37	531.8
Jun	27	9,558	540.0	15	37	531.8
Jul	8	9,913	540.0	6	37	531.8
Aug	8	8,342	539.2	4	37	531.8
Sep	3	5,736	538.0	21	74	531.8
Oct	13	5,060	536.2	21	37	531.8
Nov	17	5,315	536.3	24	37	531.8
Dec	9	4,710	536.2	2	37	531.8

Source: Appalachian Power Co., Roanoke, VA

FIGURE 7

LEESVILLE, VIRGINIA
Streambank Monitoring Project (Section 32)

SURVEY MEASUREMENTS OF BANK SLOUGHING*

Site A

Station	4/10/78	8/4/78	11/29/78	5/1/79	7/3/79	10/24/79	2/27/80	9/3/80
0+00	183.0	172.0	172.0	170.0	170.0	170.0	170.0	170.0
1+00	213.0	213.0	213.0	213.0	213.0	213.0	212.0	212.0
2+00	232.0	225.0	224.0	223.0	222.0	223.0	223.0	223.0
	259.0	245.0	245.0	245.0	245.0	245.0	243.0	243.0
3+00	294.0	278.0	278.0	276.0	276.0	274.0	273.0	273.0
4+00	245.0	235.0	235.0	235.0	235.0	233.5	233.5	233.0
	161.0	156.0	156.0	156.0	156.0	156.0	156.0	156.0
5+00	142.0	137.0	137.0	137.0	137.0	136.0	136.0	135.0
6+00	134.0	134.0		135.0	135.0	135.0	135.0	135.0
7+00	136.0	131.0	131.0	131.0	131.0	131.0	131.0	131.0
8+00	182.0	176.5	176.0	175.5	175.5	175.5	175.5	175.0
9+00	150.0		146.0	146.0	146.0	146.0	146.0	146.0
10+00	150.0	148.5	144.0	144.0	144.0	144.0	144.0	144.0
11+00		133.0	114.0	111.0	111.0	111.0	111.0	111.0
12+00	148.0		145.0	143.0	143.0	142.0	142.0	143.0
13+00	150.0				145.0	145.0	145.0	

Site C

Station	4/10/78	9/7/78	7/5/79	4/10/80
0+00	155.0	155.0	153.0	119.0
1+00	144.0	144.0	140.0	106.0
2+00	138.0	138.0	133.0	128.0
3+00	139.0	139.0	123.5	103.0
4+00	138.0	138.0	135.0	108.0
5+00	144.0	144.0	128.0	116.0
6+00	159.0	155.0	136.0	130.0
7+00	155.0	145.0	144.0	144.0

*NOTE: All measurements are in feet and are perpendicular distances from a baseline generally parallel to the river. Stations are in feet, beginning at the upstream end of each site. See figure 3, sheets 3 and 4, for location of stations.

FIGURE 8

PHOTO 1. Site A, Station 7+00, downstream 16 Mar 78

PHOTO 1

PHOTO 2. Site C, upstream portion from across river 16 Mar 78

PHOTO 2

PHOTO 3. Site A, Station 6+00, downstream 29 Nov 78

PHOTO 3


PHOTO 4. Site A, Station 5+00, downstream 1 May 79
BANK HEIGHT ABOVE WATER SURFACE
IS ABOUT 6 FT

PHOTO 4

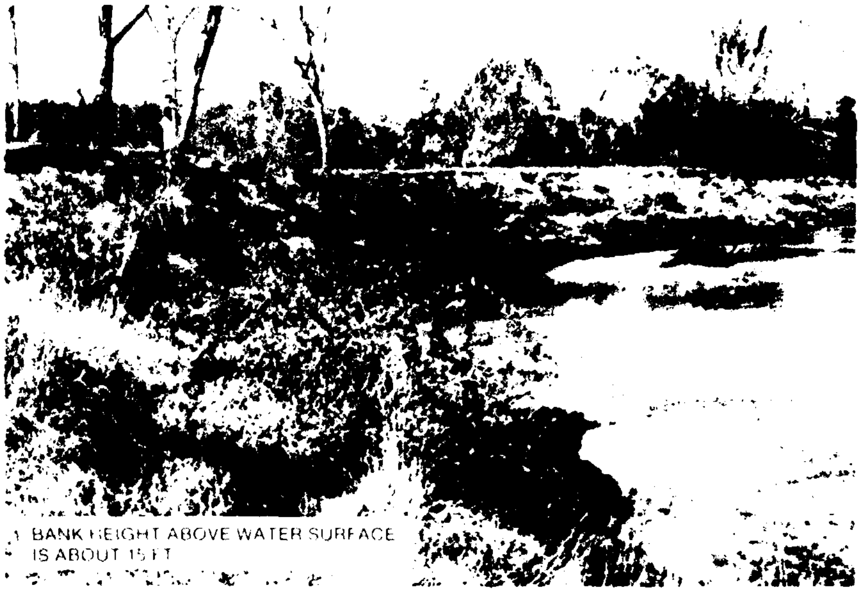


PHOTO 5. Site A, Station 5+00, downstream 24 Oct 79

PHOTO 6. Site C, Entire site viewed from across river 24 Oct 79

PHOTOS 5 AND 6

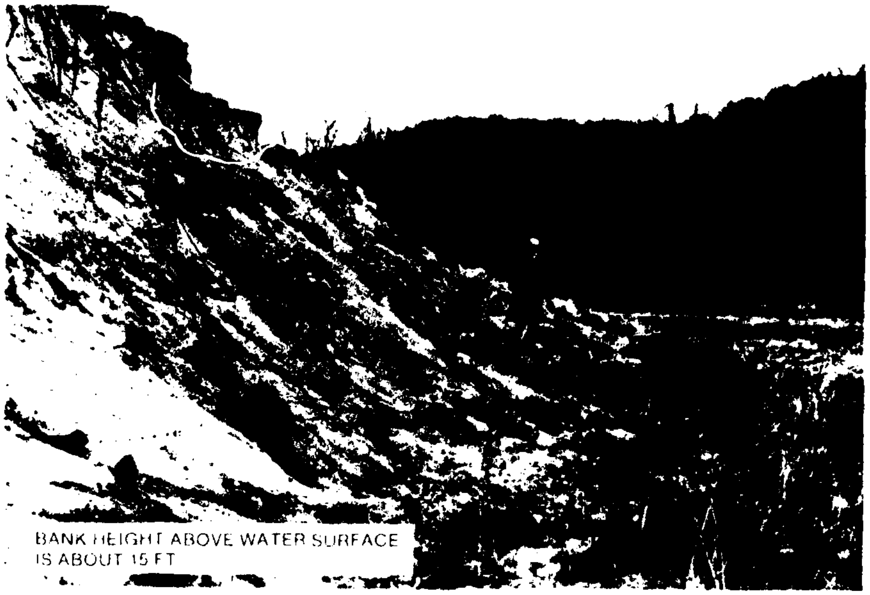


PHOTO 7. Site A, Station 2+50, downstream 27 Feb 80

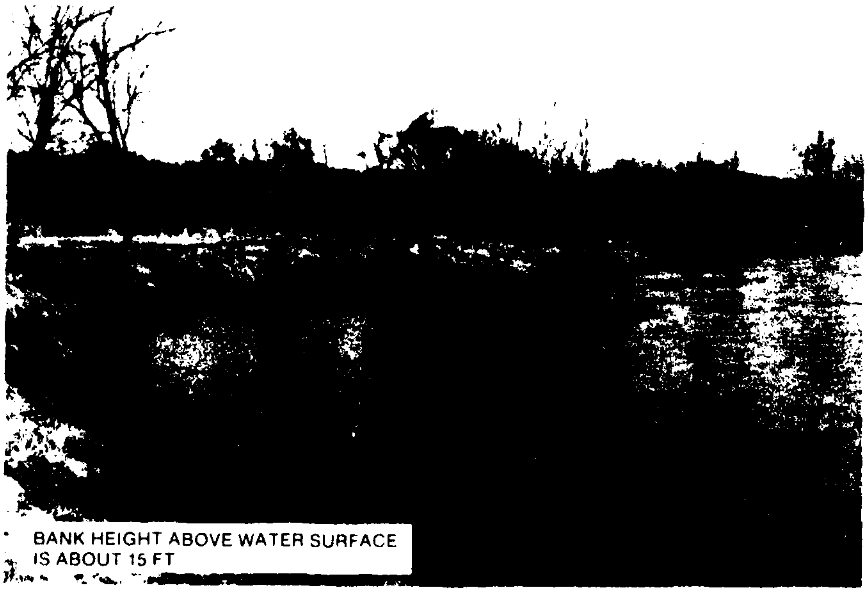


PHOTO 8. Site A, Station 4+50, downstream 27 Feb 80

PHOTOS 7 AND 8

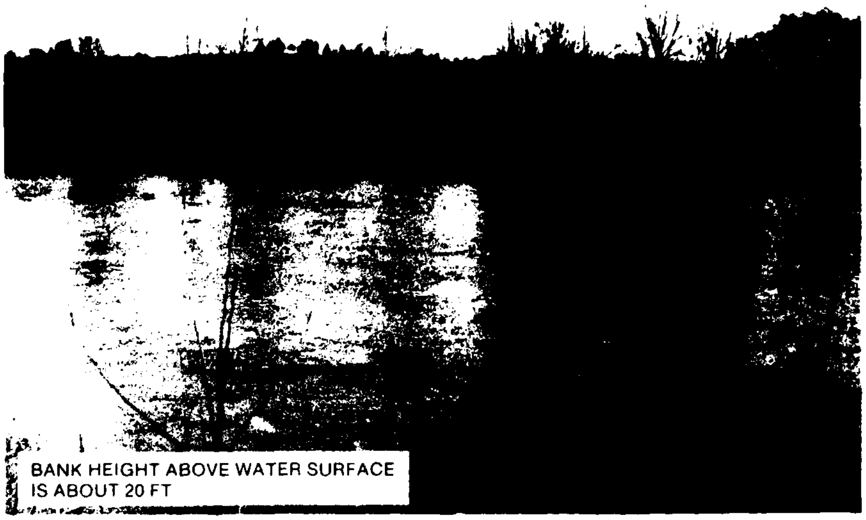


PHOTO 9. Site C, Center of Site C from across river
27 Feb 80

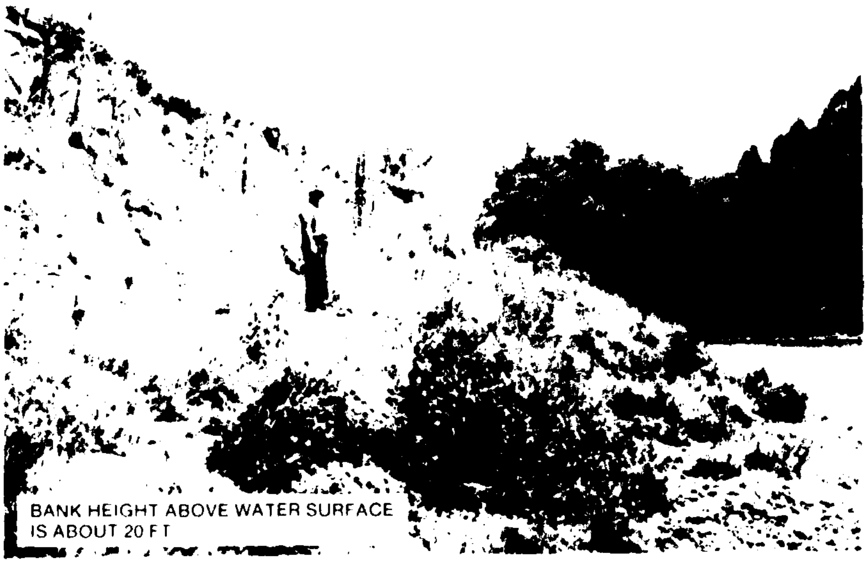


PHOTO 10. Site A, Station 2+50, downstream 3 Sep 80

PHOTOS 9 AND 10

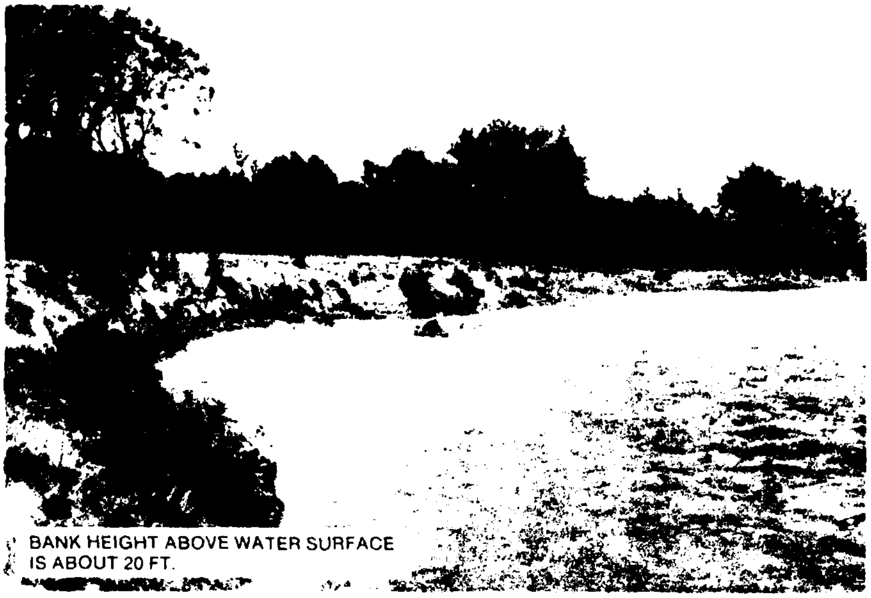


PHOTO 11. Site A, Station 4+50, downstream 3 Sep 80

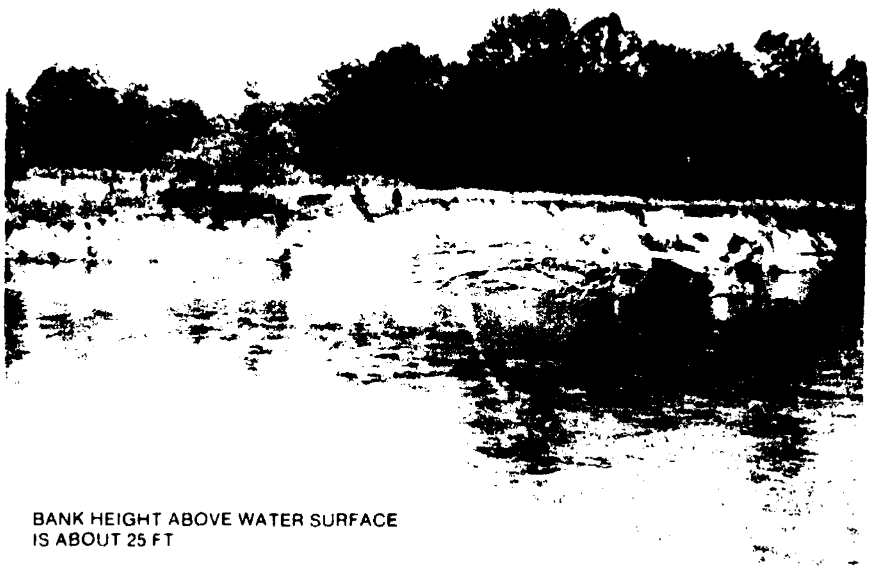


PHOTO 12. Site C, Station 4+00, near center of Site C from across river 3 Sep 80

PHOTOS 11 AND 12

PHOTO 13. Site A, Station 3+50, view of bank, stone rubble method 5 Jan 81

PHOTO 14. Station 3+50, downstream, stone rubble method 5 Jan 81

PHOTOS 13 AND 14

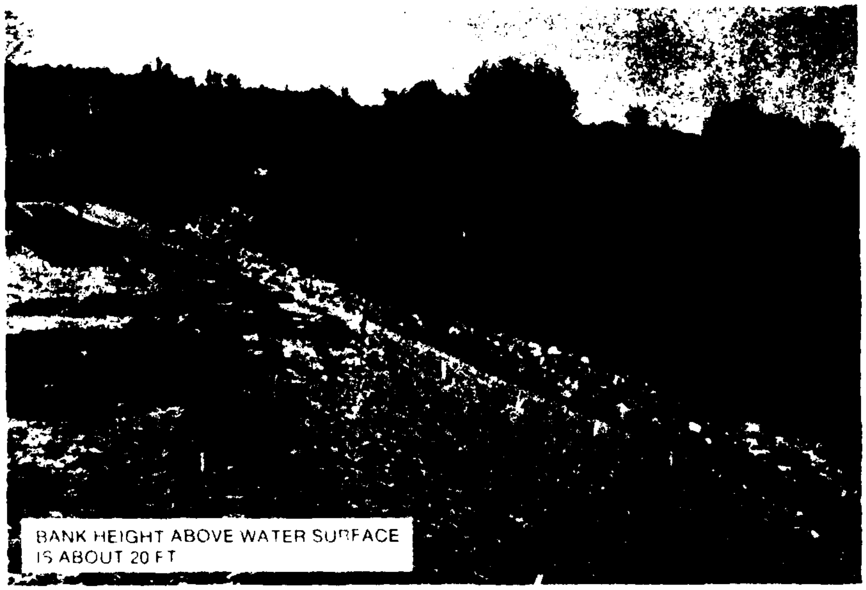


PHOTO 15. Site A, Station 4+00, downstream, completed stone rubble (foreground), beginning tire mattress-rock toe 5 Jan 81

PHOTO 16. Site A, Station 6+50, downstream, showing transition from rubble to tire mattress-rock toe method (background) 5 Jan 81

PHOTOS 15 AND 16

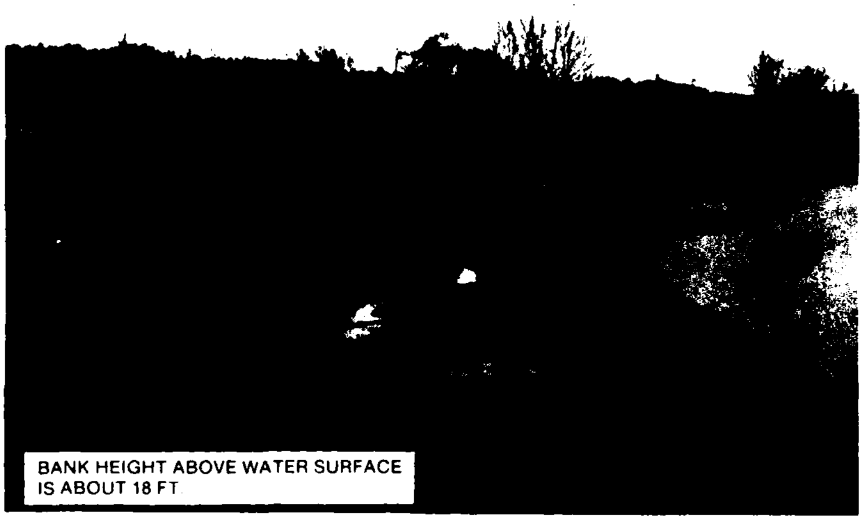


PHOTO 17. Site A, downstream portion, looking downstream just prior to construction 5 Jan 81

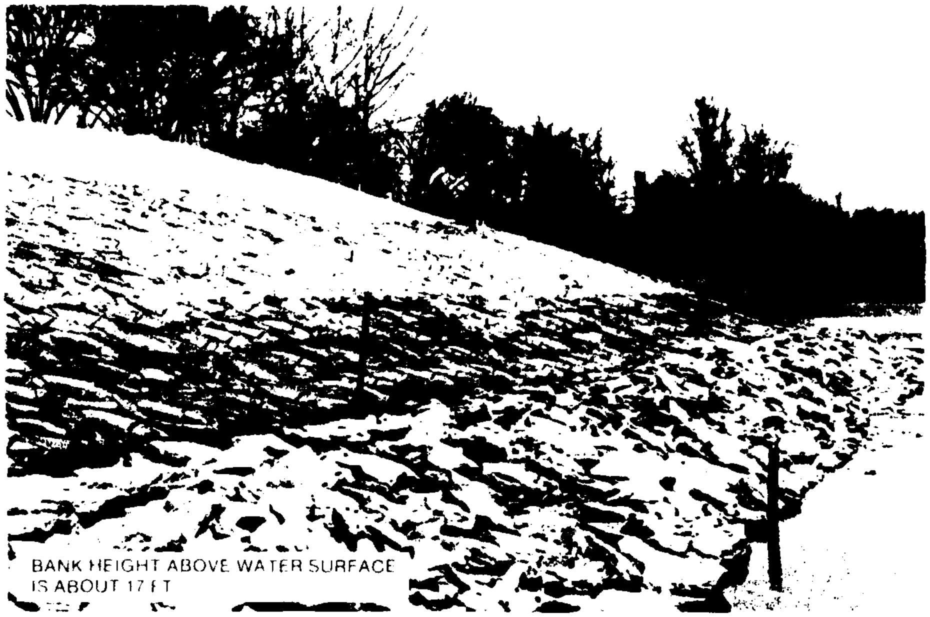


PHOTO 18. Site A, about Station 9+00. View of rubber tire mattress - rock toe immediately after construction. 21 Mar 81

PHOTO 19. Site A, Station 7+00, downstream. Transition from stone rubble to tire mattress - rock toe immediately after construction. 21 Mar 81

PHOTOS 18 AND 19

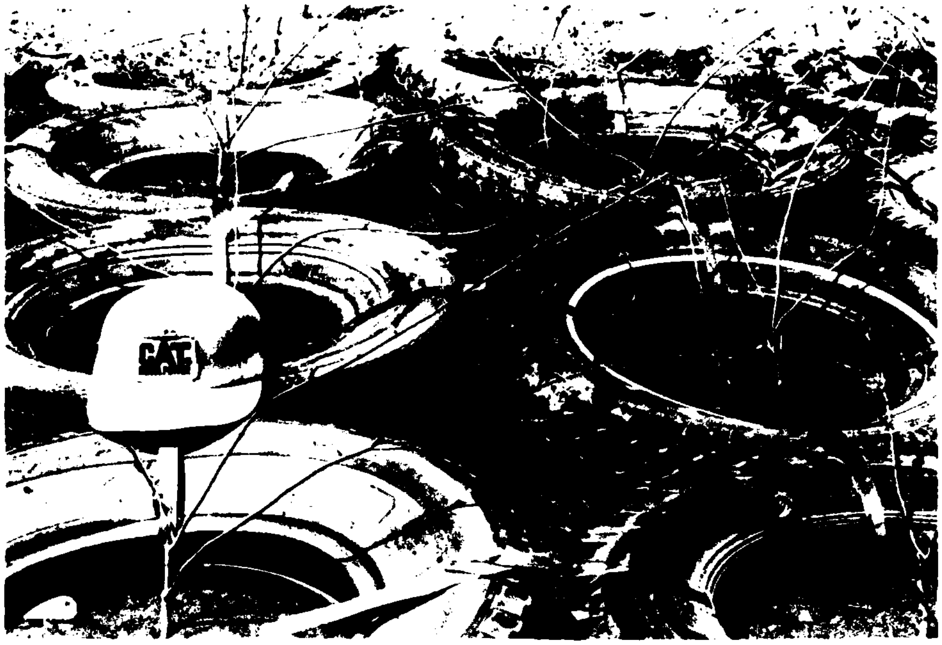


PHOTO 20. Site A, close-up of rubber tire mattress immediately after construction. 21 Mar 81

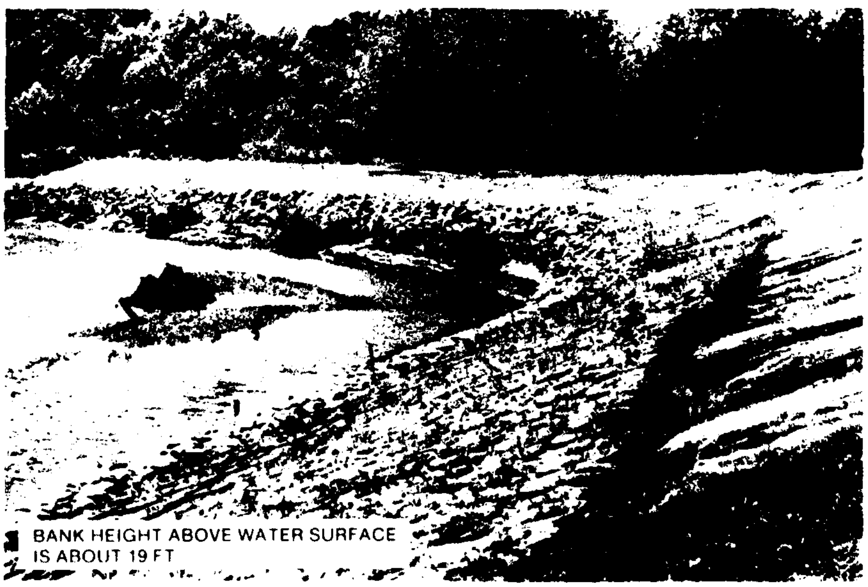


PHOTO 21. Site A, view upstream, standing at Station 8+00, curve and tree stump at Station 7+00 (transition from stone rubble to rubber tire mattress with photo coverage to Station 5+00, 2 months after construction. 21 May 81

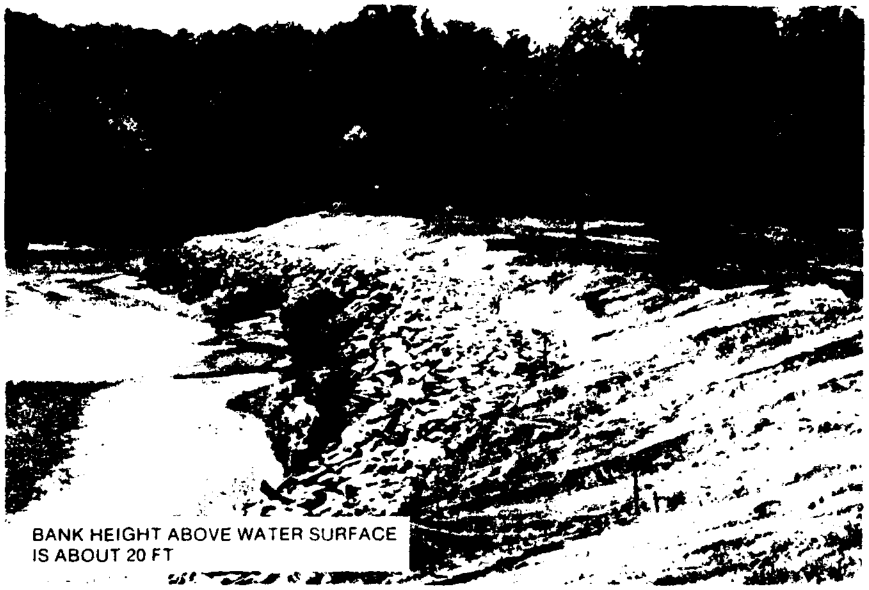


PHOTO 22. Site A, view upstream from Station 6+00 to 2+00, stone rubble method, 2 months after construction. 21 May 81

PHOTO 23. Site A, close-up of rubber tire mattress 2 months after construction. 21 May 81

PHOTOS 22 AND 23

PHOTO 24. Site A, downstream view, Station 12+50 to 13+00 showing lower 50 feet of project that received only rock toe and grass cover (no tires). 21 May 81

PHOTO 25. Site C, stone windrow method. Upstream view from Station 3+00 to 0+00. River on right. Immediately after construction. 21 May 81

PHOTOS 24 AND 25

SACRAMENTO RIVER NEAR ORDBEND, CALIFORNIA

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

SACRAMENTO RIVER NEAR ORDBEND, CALIFORNIA
DEMONSTRATION PROJECT PERFORMANCE REPORT

I - INTRODUCTION

1. Project Name and Location. - Sacramento River Streambank Erosion Control Evaluation and Demonstration Project, Glenn County, California. The demonstration site is located on the west bank of the Sacramento River just south of Ordbend and near River Mile 179.5. Plate 1 shows the location of the project.

2. Authority. - Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251. The California State Reclamation Board, as the local sponsor of this project, provided the assurances of local cooperation. The Board has agreed to operate and maintain the project upon termination of the Section 32 program.

3. Purpose and Scope. - Although construction was not yet completed at the time of report preparation, this report has been prepared for inclusion in the final report to the Congress on the Streambank Erosion Control Evaluation and Demonstration Program. The report describes the two types of bank protection measures constructed on the Sacramento River as part of the demonstration program. Included in this report is a description of the historical and existing stream conditions and characteristics, the demonstration project's design and construction, and the monitoring program for evaluating project performance. An evaluation of project effectiveness is not available at this time.

4. Problem Resume. - The concept described in this report was proposed to investigate alternatives to the use of stone riprap for streambank erosion protection along the Sacramento River. The natural banks, berms and manmade flood control levees along the Sacramento River are eroding. During recent years, stone riprap has been used almost exclusively in private, local, State and Federal programs for erosion control along the Sacramento River. The erosion at the demonstration project site is typical of the erosion problems along the upper reaches of the river. The upper bank is steep and primarily composed of sandy and silty clay of low plasticity. The lateral erosion rate at the site was about 15 feet per year based on recent bank erosion measurements taken by the California Department of Water Resources. (See Plate 2) Photographs of the site are shown on pages 13 through 17. The bank erosion measurements were analyzed by Corps personnel to determine if bank erosion (in this instance represented by bank surface area eroded away) is a function of discharge. Initially, cumulative erosion (surface area) was plotted against cumulative discharge as shown in Plate 3. A review of the data plotted on Plate 3 indicates that the surface area of bank erosion is related to discharge, but that cumulative discharge does not always adequately explain the increase or decrease in the rate of erosion. Although physical conditions are constantly changing at this site, subsequent analyses indicate that bank erosion varies exponentially with discharge.

II - HISTORICAL DESCRIPTION

5. Stream.

a. Topography. - The Sacramento River is the largest stream in California. It originates in northern California and has an extensive basin which drains an area of 26,000 square miles producing an average annual runoff of about 21,300,000 acre-feet. The river follows a 300 mile meandering course from Redding to Suisun Bay. The lower portion of the river is commercially navigable from Suisun Bay to the Port of Sacramento. The average gradient of the Sacramento River in the reach of the demonstration site is about 2 feet per mile.

b. Geology. - The project area is located within the Great Valley Geomorphic Province of California, a nearly flat, alluvial plain about 450 miles long with an average width of 50 miles. From the Tertiary to Recent Age, the Great Valley has been filled with a thick accumulation of sedimentary rocks and alluvial deposits. The topography of the area is generally flat, with soils derived from flood basin deposits consisting of clay and silt, which is the predominant riverbed composition. Through recent geologic ages, flood deposits created natural levees (rimlands) so that the river now flows through an incised channel flanked on either side by broad upraised rimlands and beyond by low-lying flood basins. The deposition phase of the main streams indicates that the Sacramento River and alluvial valley have reached a somewhat stable stage of development. Elevations range from 90 to 100 feet along the river adjacent to the project area.

c. Locality, development, and occupation. - Historically the project area was part of the Mexican land grant given to Jacinto Rodriquez. In 1867, Dr. Hugh J. Glenn purchased 55,000 acres surrounding the demonstration site to the north, south, and west. Dr. Glenn began large scale cultivation of wheat in the 1870's and at one time employed two to three hundred workers. In 1883, Dr. Glenn was shot to death and his vast agricultural holdings were eventually divided into small farms. Today the area remains in agricultural producation with wheat, corn, rice, walnuts and almonds being the predominant crops.

d. Hydrologic characteristics. - Heavy rainfall and accompanying runoff from the 12,260 square mile drainage basin upstream of the demonstration site is seasonal. Most of the heavy rainfall and runoff occur during the months of November through March. Normal annual precipitation ranges from 15 to 18 inches. Water temperatures average 47°F in the winter, increasing to 76°F in the summer. Flows in the river at Ord Ferry have ranged from a minimum of about 2,400 cubic feet per second (cfs) to about 370,000 cfs during the February 1940 flood. Maximum flow since completion of Shasta Dam was 265,000 cfs in January 1970. This flow included inbank and overbank flow. Flows in the overbank area are roughly estimated since the Ord Ferry gage measures flow only in the main channel. The Sacramento River Flood Control Project design flow at the site is 160,000 cfs. Discharges at this location may be correlated with discharges recorded at stream gaging station "Sacramento River at Ord Ferry" which has a drainage area of about 12,250 square miles and stream gaging station "Sacramento River near Butte City" which has a drainage area of 12,270 square miles. The peak discharge versus frequency relationship at the Ord Ferry station is presented in the following tabulation:

EVENT	SACRAMENTO RIVER AT ORD FERRY (flow in cfs)
500 year	666,000
100 year	370,000
50 year	300,000
20 year	240,000
10 year	210,000
5 year	150,000
2 year	98,000

A discharge versus stage relationship was developed for Sacramento River at Ord Ferry in 1977. The discharge versus stage relationship is presented in the following tabulation:

PEAK DISCHARGE (cfs)	GAGE HEIGHT* (feet)
20,000	50.78
50,000	57.55
100,000	64.50
150,000	67.00
200,000	68.55
250,000	69.60

*The zero on the gage is at Elevation 50.00 feet Corps of Engineers Datum

6. Demonstration Project Test Site.

a. Hydrologic characteristics. - The hydrologic characteristics are described in paragraph 5. The nearest gaging station is located at Ord Ferry approximately 4.5 miles upstream of the project site.

b. Hydraulic characteristics. - Average flow velocities in Sacramento River in the vicinity of the demonstration site range from 4 to 6 fps at a discharge of about 30,000 cfs, and from 8 to 11 fps at discharges approaching 160,000 cfs. The velocity ranges are based on the mean cross-sectional velocity as determined from discharge measurements at the Ord Ferry gaging station and channel cross sections along the test reach. Velocity distribution within the channel cross section was not determined.

c. Site conditions prior to construction.

(1) Riverbank description. - The natural ground elevation at the site is about 20 feet above normal summer water elevations and the bank slopes varied from vertical to about 1V on 2H. The curve radius at the site is about 3,000 feet and the channel has an overall streambed width of about 400 feet. The Sacramento River Flood Control Project levee is adjacent to the site. The river frequently flows over the top of the natural bank and inundates the project site.

(2) Bank materials. - Materials composing the banks and valley floor of the Sacramento River are sandy and silty clay with deposits of sand, gravel, and clay. Logs of borings at the demonstration site are shown on Plate 4. Soil classification for bank material is given in Plate 5.

(3) Normal bank vegetation. - Vegetation on the bank consists of grasses, willows, cottonwood, buttonbush, and various other shrubs and trees. Prior to construction, most of the vegetation located at the project site was on a low level clay bench. The bank

slopes above the bench were sandy and did not sustain vegetation because of the vertical slopes.

(4) Bank erosion tendencies. - Summer low flows erode the toe of the bank and sloughing of the upper bank occurs when the bank is saturated by winter floods. The downstream end of the test site had been eroding laterally at a rate of about 8 to 10 feet per year for the past several years. Recent analysis indicates that the lateral erosion rate was about 15 feet per year immediately prior to the start of construction of the demonstation site. Plate 3 shows the cumulative bank erosion from January 1977 to April 1979, and Plate 2 shows the lateral bank surface area lost to erosion.

d. Environmental considerations. - Environmental aspects of the project, including water quality, vegetation, fish, wildlife, benthos, rare and endangered species, and cultural resources, were coordinated with other interested agencies. An assessment was prepared in accordance with Section 404(b) of Public Law 92-500. A public notice describing the proposed project was prepared and distributed to about 125 Federal, State and local officials and agencies, including property owners in the vicinity of the project. The assessment of the impacts of construction at the site was included. The selected bank protection concepts reflect concern for existing on-site environmental values.

III - DESIGN AND CONSTRUCTION

7. General. - Two types of bank protection are being constructed at the site: (1) vegetative cover and (2) embedded concrete pile groins. Schematic drawings of the project are shown on Plates 6, 7 and 8. These designs were based on engineering judgement, physical characteristics of the site, soils and foundation investigations and, in the case of the vegetative cover design, information on various planting schemes developed by others. Use of vegetation for erosion control along the Sacramento River is being investigated because

vegetation is considered by some to be esthetically pleasing and it may provide wildlife habitat. The purpose of the vegetation cover demonstration site is to evaluate the erosion resistance potential of selected vegetation and planting schemes. Many areas along the Sacramento River have vegetation buffer strips along the top of bank, and the concept of the concrete pile groin design was to allow construction of erosion control facilities behind and/or at intermittent intervals throughout vegetation buffer strips. This method would eliminate the disturbances at the existing land-water interface that are associated with construction of stone riprap. The erosion would continue until retarded by the groins, thereby forming an irregular bank line. This method of intermittent erosion protection would preserve some of the natural land-water interface along the bank and the area between the groins would be available for root systems of vegetation.

8. Basis of Design.

a. Vegetative cover. - The purpose of the vegetative cover site is to test the erosion control potential of selected plants and planting schemes without the use of artificial supports or soil stabilizers. The slope for the 1,000-foot long section of the vegetation demonstration site is 1V on 2H below normal water elevations and 1V on 3H above normal water elevations. A continuous 2-foot thick layer of stone riprap was selected for protecting the toe of the slope below normal water. A 1V on 3H slope was selected for the vegetation to be planted above normal water. This slope was considered to be the steepest slope that could be stabilized by the vegetation. In addition, a flatter slope would have required greater disturbance of the existing bank slopes and berm area. The slopes at each end of the site were warped to conform to the existing bank slopes. Literature on various vegetation studies conducted along the Sacramento River was reviewed, and a list of plants to use in the vegetation demonstration site developed. The plant list includes trees, shrubs, ground cover, vines, and various grasses. The use of various materials for adding support and strength to the soil and plant root systems was not considered appropriate for this demonstration site since the purpose was to test the erosion control potential of vegetation, not artificial soil stabilizers. The common names of plant species selected are listed below:

TREES	SHRUBS	GROUND COVER, VINES & ANNUALS
Valley Oak	California Wild Rose	Oregon Grape
Coulter Pine	Elderberry	California Grape
California Walnut	Holly-leaf Cherry	California Poppy
California Redbud	Coffeeberry	Blue Lupine
Box Elder	Toyon	
Cottonwood	Ceanothus	
Incense Cedar		
Black Locust		
Western Sycamore		
Willow		

Various parts of the vegetation site were hydromulched after planting to permit observation of the results. The construction contract required the vegetation to be planted in the spring, and the contractor is responsible for watering the vegetation through the first summer.

b. Concrete pile groins. - The downstream 500-foot long groin demonstration site consists of poured-in place reinforced concrete pile groins, as shown on Plates 6 and 7. The piles form a series of embedded groins. The groins are each about 20 feet long and placed perpendicular to the river flow at about 30-foot intervals. The spacing and length of the groins was determined based on information in a report published by the United Nations Economic Commission for Asia and the Far East entitled "River Training and Bank Protection." The spacing of 1-1/2 times the groin lengths was adopted for this test, recognizing the narrowness of the Sacramento River, the relatively high velocities, and soil conditions at the site. The groin lengths are considered to be sufficient to move the eroding current away from the bank. The length of each groin was also limited by site conditions. The piles are reinforced and tied together with a concrete cap on top. Minimum compressive strength for the concrete was specified as 2,000 psi. The length of the piles was determined based on various loading conditions including the impact of high flows and anticipated soil loads once material between the groins is eroded.

9. Construction Details. - The invitation for bids for construction was issued on 17 March 1980. A contract was awarded on 10 September 1980, construction is in progress, and the estimated completion date is October 1981. The contractor completed construction of the concrete pile groins in the latter part of December 1980. Construction operations ceased for the winter months and resumed in May 1981. Subsurface conditions encountered during construction varied from the subsurface conditions shown in the contract documents, resulting in constructive change orders. The contractor altered his drilling and placement procedure. Construction of the concrete pile groins involved working two adjacent groins simultaneously. A crane-mounted hydraulic vibrator would drive a 24-inch diameter casing 4 feet below pile tip elevation in one groin while a crane mounted auger drilled out the previously driven casing in an adjacent groin. The drill crane would then hang the rebar cage in the drilled out casing while a concrete pumper located between the two cranes placed concrete fill using a full depth steel trunk. The vibrator crane then pulled out the completed pile casing at the second groin and immediately drove the same casing into the first groin. The crane auger then started drilling out the next adjacent casing and the cycle continued until completion of the two groins. It is not expected that differing site conditions will be encountered at the vegetative cover portion of the demonstration site. Construction of the vegetation site consists of excavating the required slopes, placing the stone riprap toe protection, planting the vegetation as described in paragraph 8, and watering the vegetation through the first summer.

10. Cost. - The estimated construction cost for the 1,000-foot long vegetative cover site is $220,000 or $220 per lineal foot of bank. Due to the constructive change orders, the estimated cost for construction of the 500-foot long concrete pile groin site has increase from $220,000 to $380,000, or from $440 to $760 per lineal foot of bank. Barring any additional constructive change orders, the estimated total construction cost for the demonstration site will be about $600,000. The estimated final allocation of total project costs is shown below:

Contract Construction Cost	$600,000
Engineering and Design	67,000
Supervision and Administration	34,000
Operation and Maintenance Manual	4,000
Final Report	3,000
Total Allocated Funds	$708,000

The above costs do not include costs incurred by the State Reclamation Board in procuring rights-of-way for the project.

IV - PERFORMANCE OF PROTECTION

11. Monitoring Program. - The monitoring program that will be used at this site is outlined on Plate 9. Field observations, surveys and photographs will form the basis for evaluating the erosion control performance of the bank protection demonstration methods. Bank erosion will be monitored and documented by photographs and surveys. Displacement of project structures will be documented, and should displacement occur, pertinent hydraulic and soils data will be analyzed so that corrective actions can be considered. District personnel will note predominant current directions and eddy currents and will obtain velocity data adjacent to the site by float measurements. Point bar growth just opposite and downstream of the site will be monitored. During the monitoring phase of the project, spot surveys of river cross sections will be made annually. Should these spot surveys indicate that significant changes in bank or river

cross sections are occurring, more extensive surveys will be considered on an as-needed basis. Natural environmental conditions such as vegetative growth, flora, and wildlife in the project vicinity will be noted and photographs taken. The project area will also be video taped prior to, during and immediately after construction.

12. Evaluation of Protection Performance. - Preliminary observation of the performance of the concrete pile groins has been possible since the groins have sustained one winter's flow. Seventeen concrete pile groins were constructed as shown on Plate 6. The groins were inspected in April 1981. Most of the groins were found to be completely embedded in the natural river bank with no portion of the structure being visible. However piles 5, 11, 14, 15 and 17 had been partially exposed due to the erosive action of the river and bank failure caused by flow over the berm. Piles 5, 11 and 17 appeared to have been exposed by river currents, whereas piles 14 and 15 appeared to have been exposed by flows over the berm returning to the river. During construction of the vegetation cover section, the area immediately behind the groins was regraded so return flows on the berm would be diverted away from the groins.

13. Rehabilitation. - None.

14. Conclusion. - The only conclusion that can be made at this time involves the engineering and construction of the concrete pile groins. The concrete pile groin method involves greater engineering costs than conventional bank protection practices because of the need for detailed explorations to identify subsurface conditions. Even after exploring, there are still unknowns that can increase construction costs, as was experienced at this demonstration site. Soil conditions varied from one drilled hole to the next, with sand and gravel layers at different elevations between holes. Also encountered was the unexpected presence of trees, stumps, and other subsurface debris, all of which, acting together, increased

construction costs. The concept of the concrete pile groin design was to allow construction of erosion control facilities set back from and/or at intermittent intervals along the river bank, thereby leaving the river bank or strips along the bank undisturbed by construction. However, the construction area actually required for maneuvering the cranes and the additional area needed for access of the pump and concrete trucks is greater than anticipated, and the berm was disturbed for the entire length of the groin site.

BEFORE CONSTRUCTION

Station 22+60 in center of photo - Looking upstream - Bank height about 20 feet (January 1980)

BEFORE CONSTRUCTION

Station 22+60 - Looking upstream at limits of vegetative planting (January 1980)

BEFORE CONSTRUCTION

Station 22+60 - Looking downstream at area for concrete pile groins (January 1980)

BEFORE CONSTRUCTION

Station 17+20 - Downstream limits for concrete pile groins - bank height about 20 feet (January 1980)

DURING CONSTRUCTION

Construction of 17 concrete pile groin structures - Looking east at project site (December 1980)

DURING CONSTRUCTION

Drilling of 2' Ø holes for piles - Note the use of two cranes (December 1980)

AFTER CONSTRUCTION

Station 18+20 - Looking at downstream side of groin Number 14 - Erosion to a depth of 10 feet (April 1981)

AFTER CONSTRUCTION

Station 17+90 - Looking at upstream side of groin Number 15 - Erosion at 2' Ø pile to a depth of about 8 feet (April 1981)

AFTER CONSTRUCTION

Station 17+40 - Looking upstream at groin Number 17 (April 1981)

AFTER CONSTRUCTION

Station 20+90 - Looking at downstream side of groin Number 5 (April 1981)

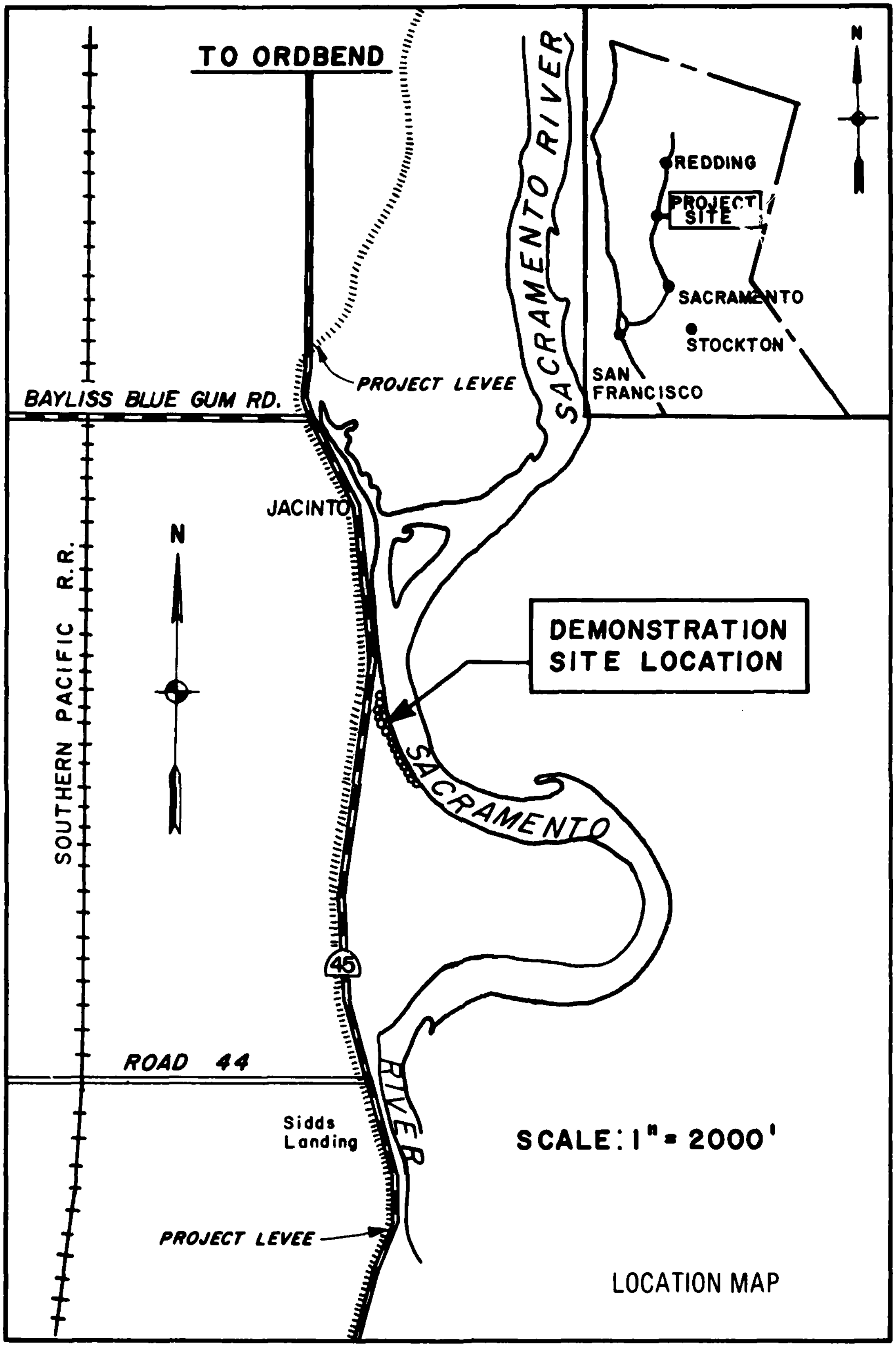

TO ORDBEND
SACRAMENTO RIVER
N
REDDING
PROJECT SITE
SACRAMENTO
STOCKTON
SAN FRANCISCO
PROJECT LEVEE
BAYLISS BLUE GUM RD.
JACINTO
N
SOUTHERN PACIFIC R.R.
DEMONSTRATION SITE LOCATION
SACRAMENTO
45
ROAD 44
RIVER
Sidds Landing
SCALE: 1" = 2000'
PROJECT LEVEE
LOCATION MAP

PLATE 1

BANK EROSION MEASUREMENTS[1/]

(LATERAL BANK SURFACE AREA LOST DUE TO EROSION)

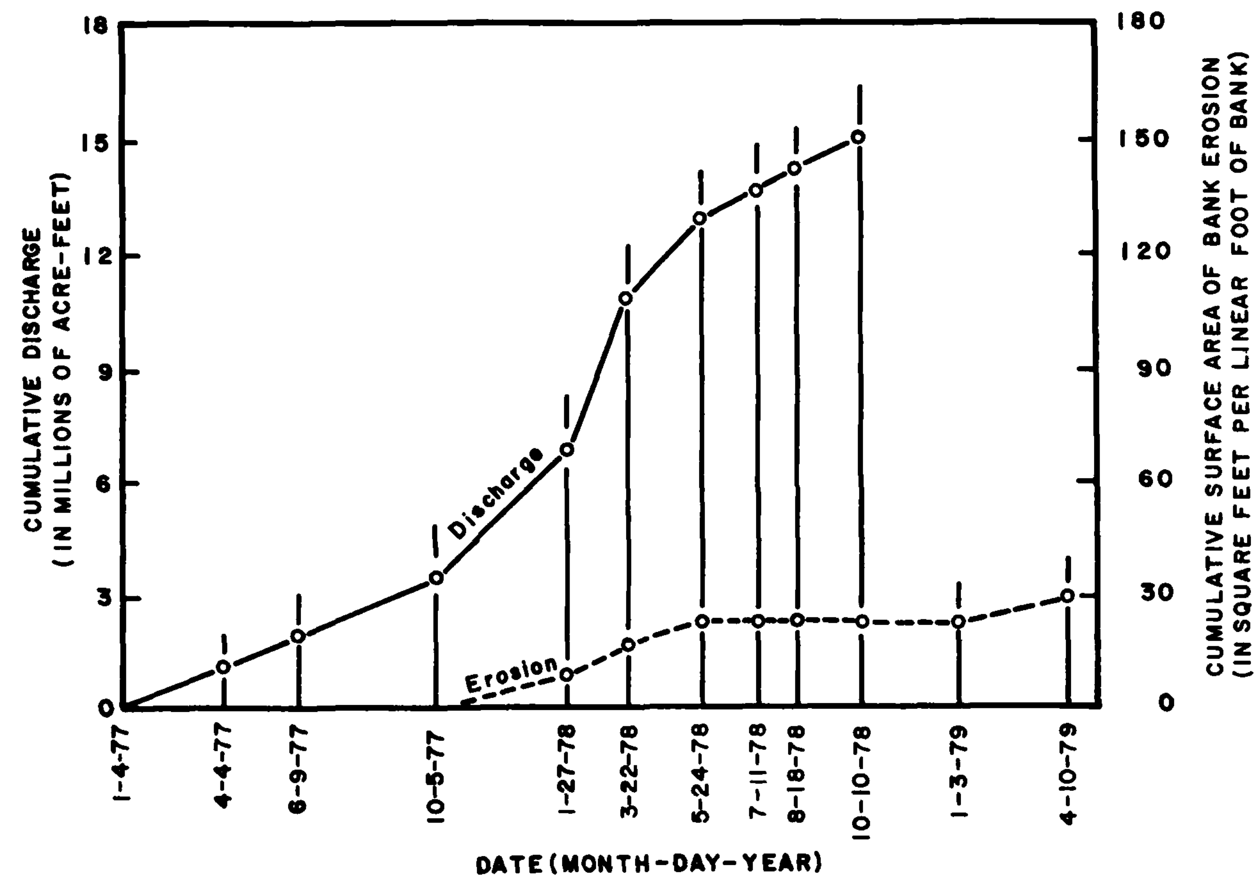
Site[2/]	Monitored Length (feet)	Date Measured	Bank Surface Area Loss (square feet)
Jacinto River Mile 179.5R	2,750	1-4-77	
			0
		4-4-77	
			0
		6-9-77	
			0
		10-5-77	
			27,000
		1-27-78	
			22,200
		3-22-78	
			17,100
		5-24-78	
			0
		7-11-78	
			0
		8-16-78	
			0
		10-10-78	
			0
		1-3-79	
			19,700
		4-10-79	

1/ Measurements taken by personnel of the California Department of Water Resources.

2/ L and R designate the left or right bank looking downstream.

BANK EROSION

PLATE 2



NOTE: DISCHARGE AS MEASURED AT STATE OF CALIFORNIA GAGE A02570 (SACRAMENTO RIVER AT ORD FERRY).

CUMULATIVE SURFACE AREA OF BANK EROSION IN RELATION TO CUMULATIVE DISCHARGE

SACRAMENTO RIVER MILE 179.5

PLATE 3

2F-79-1

Depth		N	GR	SA	FI	LL	PI	MC	Description
0.0'	CL	—	—	—	—	—	—	14	* CLAY, dark yellow brown, 95% medium plastic fines, fine sand, scattered gravels to 2" max. at surface
		18							
5.0'		21	—	—	—	—	—	23	* CLAY, dark gray brown, 95% mediam plastic graded sand
8.0'			0	2	96	32	12	20	CLAY
10.5'		6	—	—	—	—	—	—	* CLAY, same as at 5.0' to 8.0'
13.0' ▽		8							
16.0'			8	11	81	35	14	22	SANDY CLAY
17.0'	SC		—	—	—	—	—	18	* GRAVELLY CLAYEY SAND, dark gray brown, 20% medium plastic fines, 60% graded sand, 15% gravels to .75" max.
19.0'	CL	10	—	—	—	—	—		SANDY CLAY, olive, 80% medium plastic fines, fine sand, loose, micaceous
22.0'		19	—	—	—	—	—	28	* CLAY, light olive brown, 96% medium plastic fines, graded sand, trace of gravels, weakly cemented, slight iron oxide staining, firm to stiff
24.0'			0	17	83	41	16	25	SANDY SILTY CLAY
26.5'		25	—	—	—	—	—	26	* CLAY, light olive brown, 90% medium plastic fines, graded sand, trace of gravels, weakly cemented
30.0'		14	—	—	—	—	—	28	* CLAY, same as above
32.0'			0	7	93	41	17	23	SILTY CLAY, light grayish tan, firm
33.8'			—	—	—	—	—	31	* CLAY, light olive brown, 90% medium plastic fines, graded sand, trace of gravel
36.0'		47	—	—	—	—	—	16	* CLAY, olive brown, 95% medium plastic fines, medium sand, trace of gravels, very stiff, iron oxide staining.
37.5'			0	23	77	30	13	21	SANDY CLAY, olive brown
40.0'									

NOTE: See Plate 5 for symbol explanation.
See Plate 6 for location of boring.

BORING

PLATE 4 (SHEET 1 OF 2)

2F-79-2

Depth		N	GR	SA	FI	LL	PI	MC	Description
0.0'	CL	—	—	—	—	—	—	12	* CLAY, dark yellow brown, 90% medium plastic fines, fine sand, scattered subrounded gravels to .5" max. scattered roots
		3							
5.0'		12	—	—	—	—	—	17	* CLAY, dark yellow brown, 95% medium plastic fines, fine sand, trace of gravels
8.0'			0	5	95	29	11	18	CLAY, firm
10.0'		7	0	5	95	34	15	22	CLAY, dark tannish brown, soft
13.5' ▽									* GRAVELLY CLAY, dark gray brown, 85% medium plastic fines, 10% gravel to 1" max, soft to firm
14.0'		10	—	—	—	—	—	15	
15.5'	GP-GM		—	—	—	—	—	8	* SILTY SANDY GRAVEL, olive brown, non-plastic fines, 35% graded sand, 60% gravels to 1" max.
16.4'	GP		58	38	4	—	—	8	SANDY GRAVEL
18.7'	GP-GM	22	—	—	—	—	—	—	* SILTY SANDY GRAVEL, same as at 15.5 to 16.4'
22.5'	CL	20	—	—	—	—	—	29	* CLAY, olive yellow, 90% medium plastic fines, graded sand, weakly cemented, slight iron oxide staining
24.0'	ML		1	2	97	42	14	28	CLAYEY SILT
26.5'	CL	22	—	—	—	—	—	28	* CLAY, light olive brown, 90% medium plastic fines, fine sand
30.0'		20	—	—	—	—	—	26	* CLAY, light olive brown, 90% medium plastic fines, fine sand, stiff, iron oxide staining
32.0'			0	13	87	34	16	23	SANDY CLAY
34.5'		26	—	—	—	—	—	25	* CLAY, light olive brown, 95% medium plastic fines, medium sand, organic material
37.5'			0	24	76	38	19	23	SANDY CLAY
40.0'									

NOTE: See Plate 5 for symbol explanation.
See Plate 6 for location of boring.

BORING

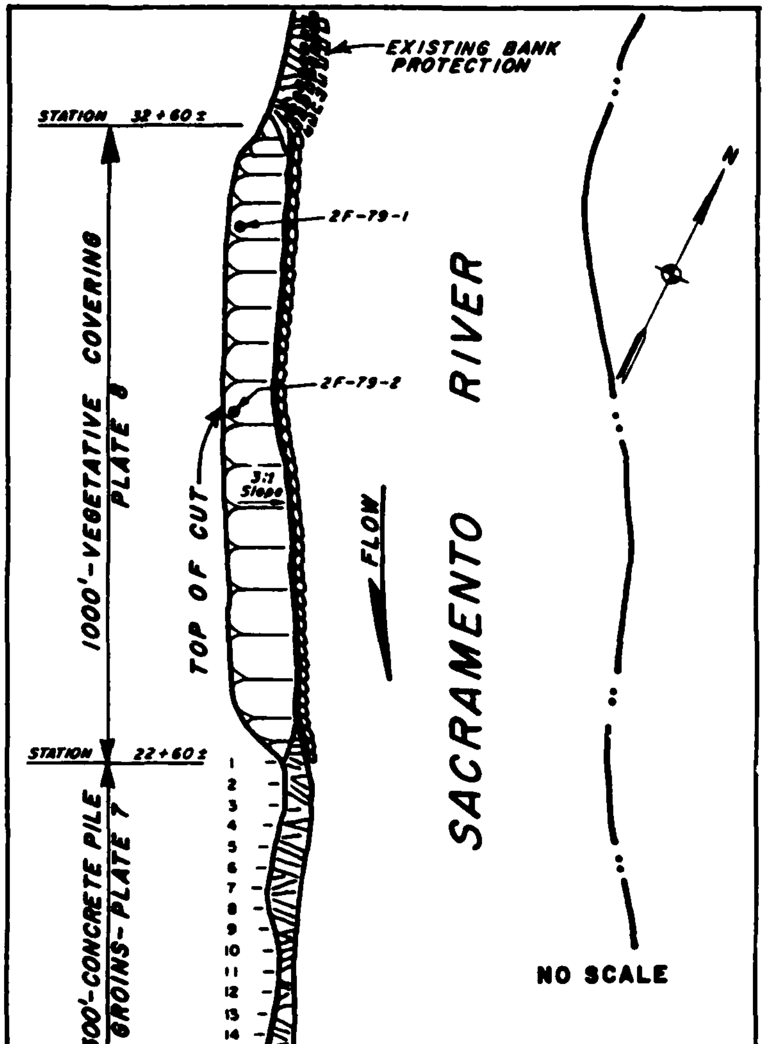
PLATE 4 (SHEET 2 OF 2)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS - 50% or more retained on the No. 200 sieve.	GRAVELS - More than half of coarse fraction retained on the No. 4 sieve.	Clean sands	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty gravels, gravel-sand silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS - More than half of coarse fraction passing the No. 4 sieve.	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS - More than 50% passing the No. 200 sieve	SILTS AND CLAYS	Liquid Limit below 50%	ML	Inorganic silts and very fine sands, rock flour, silty fine sands, or silts. Plasticity below "A" line.
			CL	Inorganic clays, gravelly clays, sandy clays, lean clays. Plasticity above "A" line.
			OL	Organic silts and organic clays. Plasticity below "A" line.
		Liquid Limit 50% and above	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. Plasticity below "A" line.
			CH	Inorganic fat clays. Plasticity above "A" line.
			OH	Organic clays or organic silts. Plasticity below "A" line.
Highly organic soils			PT	Peat, organic content greater than 60%.

*** - Laboratory Visual Classification**
N - Number of blows of standard penetrometer
GR - Gravel, percent by weight passing 3-inch sieve and retained on the No. 4 sieve
SA - Sands, percent by weight passing the No. 4 sieve and retained on the No. 200 sieve
FI - Fines, percent by weight passing the No. 200 sieve
LL - Liquid Limit
PI - Plasticity Index (Liquid Limit minus Plastic Limit)
MC - Field Moisture Content in percent of dry weight

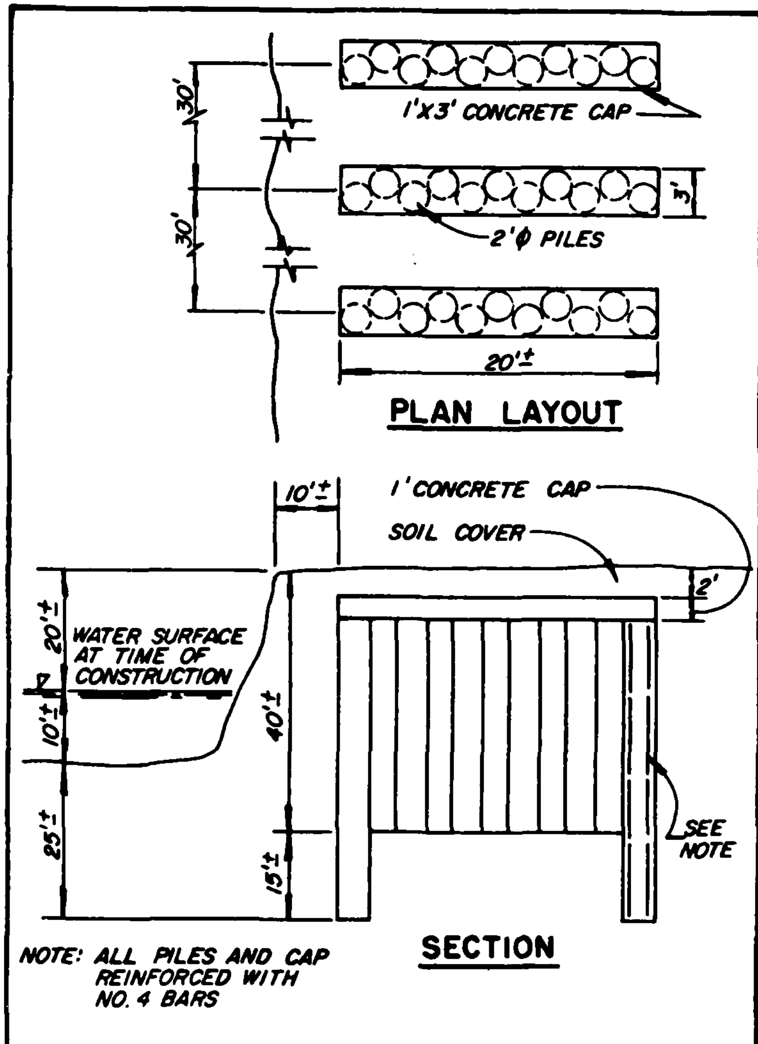
UNIFIED SOIL CLASSIFICATION SYSTEM

PLATE 5



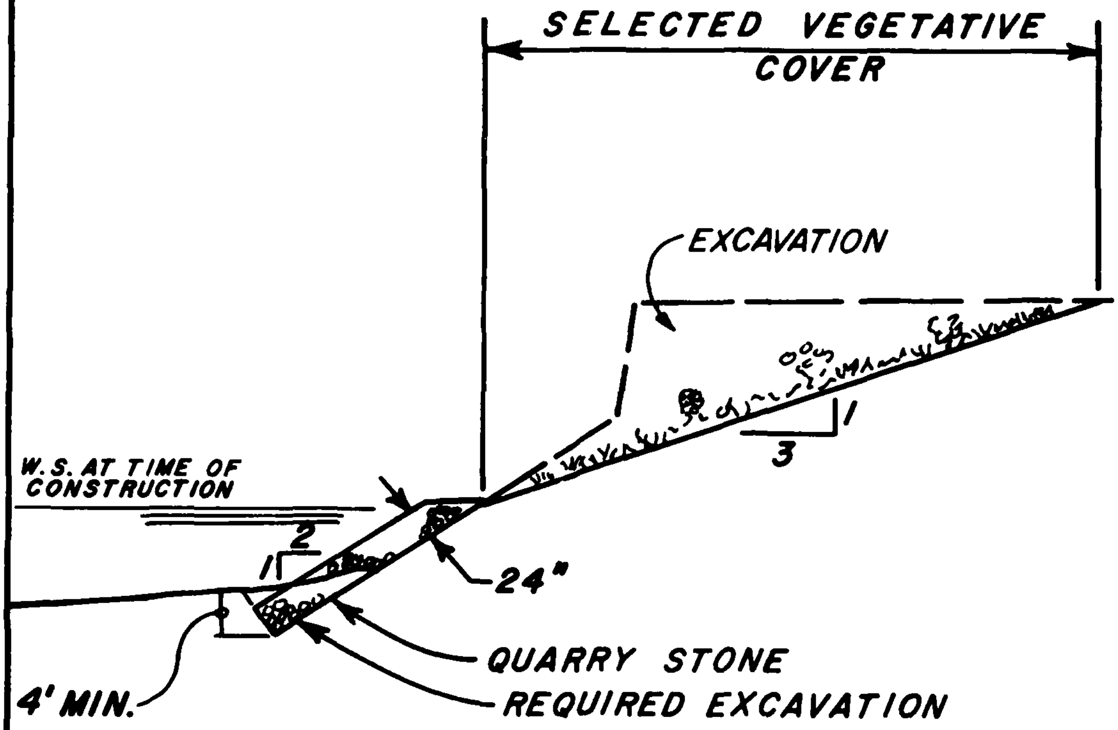
SKETCH OF PLAN

PLATE 6



CONCRETE PILE GROINS

PLATE 7



SECTION

NOT TO SCALE

VEGETATIVE COVER

PLATE 8

SACRAMENTO RIVER
GLENN COUNTY, CALIFORNIA
STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION PROJECT

June 1981

MONITORING PROGRAM

Task	Responsibility: District Office	Responsibility: Construction Project Office	Responsibility: Other	Performance Period: Prior to Construction	Performance Period: During Construction	Performance Period: After Construction	Frequency of Post-Construction Data Collection
1. Climatology							
General			X 1/	X	X	X	Hourly
2. Hydrology							
a. Discharge			X 2/	X	X	X	Continuous
b. Current Velocities	(	X		X		X	6/Year
3. Surveys							
Cross Sections	X	X		X		X	Annually
4. Soils and Geology							
a. Subsurface	X			X			
b. Pierometers (2)	X	X		X	X	X	8/Year
5. Environment							
a. Assessment	X			X			
b. Observations	X	X		X	X	X	As Necessary
6. ?roject Performance							
a. Erosion	X	X		X		X	Annually
b. Structures	X	X				X	Annually
c. Photographs							
(1) Ground	X	X		X	X	X	8/Year
(2) Aerial	X			X		X	Variable

1/ National Weather Service
2/ U. S. Geological Survey

MONITORING PROGRAM

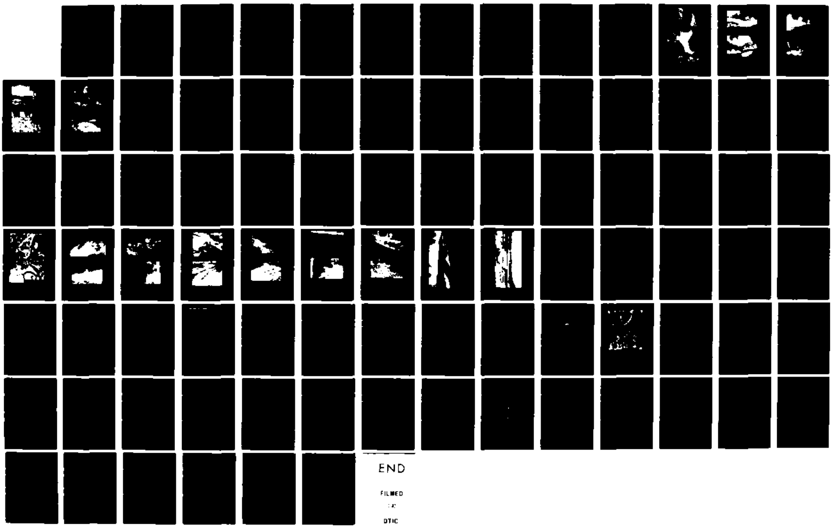
PLATE 9

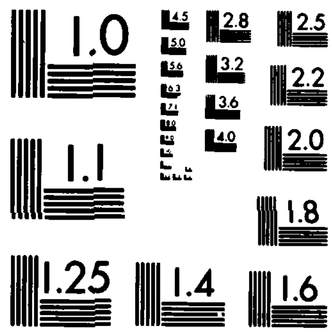
WHITE RIVER NEAR JACKSONPORT, ARKANSAS

AD-A121 139 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

4/4

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

JACKSONPORT STATE PARK BANK PROTECTION WORKS DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. This project is called the Jacksonport State Park Bank Protection Works. It is located on the White River, Mile 260, near the town of Jacksonport, in Jackson County, Arkansas, Photo 1 and Plates 1 and 2 show the project location.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a bank erosion problem, the types of bank protection used, and a performance evaluation of a demonstration project on the White River at Jacksonport State Park. The protection works were constructed and monitored by the Little Rock District.

4. Problem Resume'. The average recession of the park bankline between 1949 and 1975 was 6 feet annually. Within the project area, erosion of the park shoreline caused as much as 35 feet of parkland to cave into the river between 1973 and 1975. Continuing erosion and loss of parkland produces significant damage and adversely affects the navigation channel, Jacksonport State Park, and surrounding area in the following ways:

a. It discourages development of the park along the riverbank by threatening facilities placed near the bank.

b. It threatens the existing locally constructed Jacksonport levee.

c. It threatens, and will eventually destroy, the overall historic and environmental setting of this park which is associated with early river navigation during the steamboat era, and is listed in the National Register of Historic Places.

d. It presents the possibility that the river may eventually cut across the narrow neck north of Mason Bend, creating an island and cutting off the lower end of the park from land access.

There were no existing improvements related to the correction of bank erosion problems prior to project inception.

II. HISTORICAL DESCRIPTION

5. Stream Description, General. The White River rises in northwest Arkansas, flows northerly into Missouri, thence easterly and southeasterly through eastern Arkansas to join the Mississippi River about 61 miles upstream from Greenville, Mississippi. It is about 720 miles long and has a total drainage area of 27,765 square miles, 10,622 in southern Missouri and 17,143 in northern and eastern Arkansas.

6. Topography. The White River rises in the Boston "Mountains" in western Arkansas. The elevation of its source is about 2,050 feet above mean sea level and low water elevation at the mouth is 107 feet above mean sea level. The fall ranges from 25 feet per mile near the source to an average of 0.3 feet per mile downstream from the Black River.

The demonstration project is about 0.5 mile downstream from the mouth of Black River. The topography in the project area is flat with a regional slope to the south in this vicinity, the river is characterized by a meandering channel and flat slopes. Riverbanks range from 20 to 25 feet in height and channel widths from 200 to 400 feet. Along the park shoreline, the low bank area is about 215 feet above mean sea level and high park land is 220 feet above mean sea level.

7. Geology. The demonstration project lies in the Mississippi Alluvial Plain of the Coastal Plain Province. The site is characterized by two flat surface configurations. The flood plain comprising the lower surface is underlain by sediment deposited in recent times. The higher surface is a dissected plain or terrace about 10 to 35 feet above the flood plain and underlain by deposits of glacial age. The well defined traces of river migration on the surface of the flood plain show that all the flood plain area is a part of the meander belt.

Alluvial deposits under the flood plain have a total thickness of about 75 feet. These consist of a coarse grained substratum of coarse sand and gravel and a fine grain top stratum of clays, silts, and fine sands. The top stratum sediments have been deposited by the present day White and Black Rivers and generally form a continuous blanketing layer averaging 10 to 20 feet in thickness, although locally it may be just a few feet thick or absent. Underlying the bottom stratum deposits are interbedded, fine-to-coarse-grained sands and carbonaceous clays of the Wilcox Group of Teritiary Age. Point bar deposits are generally less than 20 feet thick. Channel filling in abandoned meanders are generally less than 30 feet thick. Soils borings were made along the project length. Specific soils logs and borings locations are shown on Plates 4 and 7.

8. Hydrologic Characteristics. The climate of the demonstration area is humid. Summers are usually long and hot and winters relatively short and mild. The maximum and minimum temperatures as recorded at Newport, Arkansas, (located about 3 miles from the demonstration site) were 114 degrees and minus 14 degrees, respectively, and the average annual temperature at that location is about 61 degrees. The prevailing wind is from the south and has an average velocity of about 7.5 miles per hour. The average annual precipitation over the area, based on the standard United States Weather Bureau mean at the Newport station, is 50.0 inches. The wettest and driest years of record had 80.88 and 22.35 inches of precipitation, respectively. The average annual snowfall, based on records from the Newport station, is about

6.0 inches. Runoff from the melted snow seldom contributes to flooding in the area.

9. Flooding conditions.

General. The community of Jacksonport, Arkansas, is subject to flooding by the White and Black Rivers. Partial protection from frequent flooding is provided by a locally constructed levee which separates the community and portions of the park from the riverfront parkland and facilities. Riverfront parklands are about 220 feet above mean sea level and are flooded normally about once a year for a period of about three days.

Normally, the principal flood season occurs during the late winter to early summer season. However, intense local storms can occur at any time of the year. Since 1964, discharges and flow durations on the White River at Jacksonport have been modified by five major flood control reservoirs constructed in the upper basin. A tabulation of mean monthly flows for 13 years is shown on Plate 6. Hydropower generation has also affected daily flows past the site. Typical flood hydrographs are shown on Plate 5. Discharge rating and discharge duration curves are shown on Plate 5. Discharges at Jacksonport for 5, 10, 50 and 100 year frequencies of recurrance are about 112,000, 164,000, 340,000, and 388,000 c.f.s. River discharge profiles through the reach are shown on Plate 3.

10. Bank erosion. The riverbank at the project site is subject to continuous attack by river currents. The high bank along the west side of the park was practically vertical, unstable, and dangerous to park visitors. Typical pre- project riverbank sections are shown on Photo 2 through 9 and on Plates 9 through 12. Loss of valuable parkland, trees, and vegetation has occurred as a result of bank caving and sloughing.

A comparison of 1964 and 1975 aerial photographs indicates serious bank caving has occurred along the entire project site. As much as 35 feet of parkland caved into the river during that period. The parkland river bank receded an average of about 6 feet annually between 1949 and 1975.

11. Environmental considerations. A large portion of the area surrounding the project site is agricultural land and clearing has occurred almost to the river banks leaving a small corridor of timber and grassland. Within the lower White River region, this segment has a low wildlife potential. Most understory species of plants have been eliminated. Within the immediate project site, willows and grasses predominate in the low bank area and native trees occupy the high bank area of the park. Small mammals and birds are well represented in the park. Numerous species of fresh water fishes are found in the White River. Amphibian and reptilian fauna is abundant and quantitatively consistent in the area. No rare or endangered species are known to be in the vicinity of the work. The Little Rock District Engineer found that no long- range adverse environmental impacts will occur as a result of the implementation of the improvements and there are no unresolved conflicts. In accordance with the National Environmental Act of 1969 (PL 91-190), he determined that an Environmental Statement is not required.

III. DESIGN AND CONSTRUCTION

12. Scope. A boat mooring slip is located at the Jacksonport waterfront. The slip plus about 150 feet of the left bankline upstream and downstream from the slip are protected against river velocity erosion with a quarry run stone paved revetment section. The demonstration site extends along the left bank of the river for a distance of over 4,000 feet downstream from the boat slip protection. A plan view with project stationing is shown on Plate 7.

13. Basis of Design. The demonstration project protection consists basically of a standard quarry run stone toe revetment section along the full length of the site plus various combinations of stone, compacted clay, and uncompacted clay in situ sections on the upper bank slopes.

The elevations of the protection sections were related to the construction reference plane (CRP) of the project reach. The CRP corresponds to the water surface profile for a very low river discharge - about 3,000 c.f.s. The toe trench section was designed with an adequate volume of stone to launch and protect the bank from all anticipated riverbed scouring. Greater scouring tendencies were anticipated at the upstream end of the site, and larger stone sections were provided. The bottom of the toe trench was established so that its excavation would be not more than 5 to 6 feet below the anticipated water surface during the expected construction period. The top of the toe trench section was set 10 feet above the CRP.

a. Test Sections. Eight different revetment cross-section designs incorporating various toe sizes, types of material (clay or stone) and heights of upper bank protection were developed. Label Sections A through H, details of each are shown on Plates 9 through 12.

b. Test Reaches. As layed out in the field, the demonstration project consists of 13 test reaches each incorporating various lengths of one or two of the test sections. Each test reach is described in the tabulation on the following page.

c. Stone. Stone fill material for protection design is based on dumped quarry-run stone available from a local quarry about 18 miles from the site. Specified gradation was maximum size of 350 pounds with 50 percent weighing 25 pounds or more.

d. Clay material. Samples of clay from borings JP-7 through JP-12 were tested for dispersion by pinhole erosion, total cation, and SCS dispersion tests. The samples all tested non-dispersive.

Twelve samples of clay from borings JP-7 through JP-12 were classified in the laboratory. One sample from 7.5 to 8.0 feet depth in boring JP-9 had 1 percent sand and 99 percent fines. It had a liquid limit of 51, a plastic limit of 18, a plasticity index of 33, and was classified a fat

Test Reach	Between Stations	Length feet	Reach Description	Cost/foot*
1	6+60 16+00	940	Section G	144
2	16+00 16+80	80	Alternating length of Sections A (20 ft long) and B (20 ft long)	163
3	16+80 18+00	120	Alternating lengths of Sections A (40 ft long) and B (20 ft long)	158
4	18+00 19+60	160	Alternating lengths of Sections A (60 ft long) and B (20 ft long)	156
5	19+60 21+60	200	Alternating lengths of Sections A (80 ft long) and B (20 ft long)	154
6	21+60 26+30	470	Section C	115
7	26+30 31+00	470	Section H	111
8	31+00 31+80	80	Alternating lengths of Sections D (20 ft long) and E (20 ft long)	81
9	31+80 33+00	120	Alternating lengths of Sections D (40 ft long) and E (20 ft long)	81
10	33+00 34+60	160	Alternating lengths of Sections D (60 ft long) and E (20 ft long)	81
11	34+60 36+60	200	Alternating lengths of Sections D (80 ft long) and E (20 ft long)	81
12	36+60 42+00	540	Section F	104
13	42+00 45+00	300	Section G	144
		3840		123 Avg

* Construction, engineering and design cost.

clay CH. The natural water content was 34.7 percent. The other 11 samples were classified as lean clay CL with liquid limit from 35 to 49, plastic limit from 14 to 20, and plasticity index from 19 to 31. Natural water contents of these samples ranged from 20.1 to 25.9 percent. Results of the tests are in SWDED-FL Reports 12848 and 12848-1 dated 9 July 1979 and 23 August 1979, respectively.

The graded clay bank paving and clay fill bank paving is compacted by making not less than 6 passes over the entire surface by a track-type tractor weighing at least 10 tons and exerting a unit pressure of not less than 6 pounds per square inch or by 6 passes of a sheep's-foot roller.

e. Disposal site. Disposal of excess earth from revetment excavation is placed in a berm along the east border of the park. It is a semi-compacted earth embankment having a crown width of 24 feet, grade elevation 226 feet, m.s.l. and side slopes of 1V on 3H.

14. Construction Details. The contractor started clearing, grubbing, and grading the top bank in the fall of 1978. He was delayed by high river stages until June 1979. To expedite the work, in 1979 he was allowed to build the toe section 3 feet higher than specified until lower river stages occurred in late September. Transition to the specified lower section was accomplished in the vicinity of Station 18+75.

The contractor used the top of the toe section berm for truck delivery of stone fill for revetment construction. Temporary down ramps at convenient points along the bank were allowed for access to the toe berm and deliveryof stone fill material and equipment. Where a bermed toe section was not specified (Station 30+20 to 41+15), the contractor added, temporarily, an excess of stone fill to the toe section as a working berm for stone placement. At a convenient time, excess stone fill was removed for bank paving fill and the toe section restored to its specified dimensions.

During the winter of 1978-1979, high river flows of 122,000 c.f.s. resulted in stages of about elevation 224 and caused some bank caving in the vicinity of Station 29+00. About 200 to 300 feet of bank was affected. As a result, the revetment was realigned about six to eight feet landward of the proposed alignment.

Project construction was completed in February 1980.

The total contract cost of the Jacksonport Demonstration Project amounted to $373,000. Adding construction engineering and design, the total cost was $473,600 or an average of $123 per bank foot.

IV. PERFORMANCE OF PROTECTION

15. Monitoring Program. The approved monitoring program for the test site includes periodic observations, photographs, and measurements of the following data: River stages, stream velocity, and structure damage.

16. Evaluation of Protection Performance. No floods or unusually high flows have occurred on the White River between the time of project completion and preparation of this report. It should be noted however, that heavy local rainfall during early spring of 1980 caused extensive rill channels to be cut in most clay bank paving where turfing was not specified. Accordingly, the construction contract was extended to include the seeding and mulching of the exposed clay bank paving. An inspection in October 1980 confirmed a limited stand of vegetative cover. Little Rock District proposes to inspect the site following the occurance of a major river flow and prepare a report of findings and conclusions on project performance for transmittal to the Southwestern Division Office.

17. Conclusion. The effectiveness of the Jacksonport State Park Demonstration Project cannot be determined at this time owing to lack of flow conditions under which performance can be evaluated.

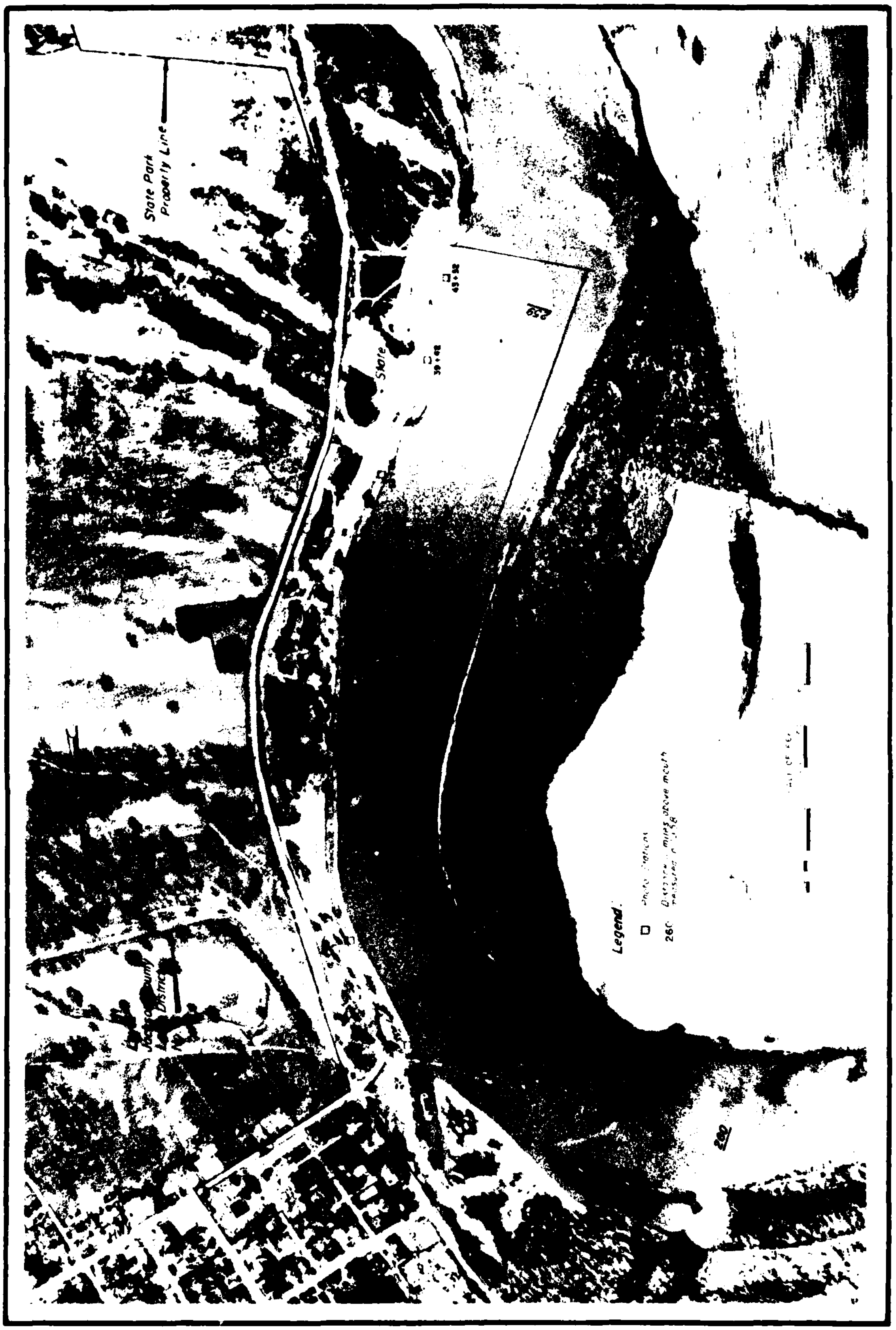


PHOTO 1

PHOTO 2 STA 11+00

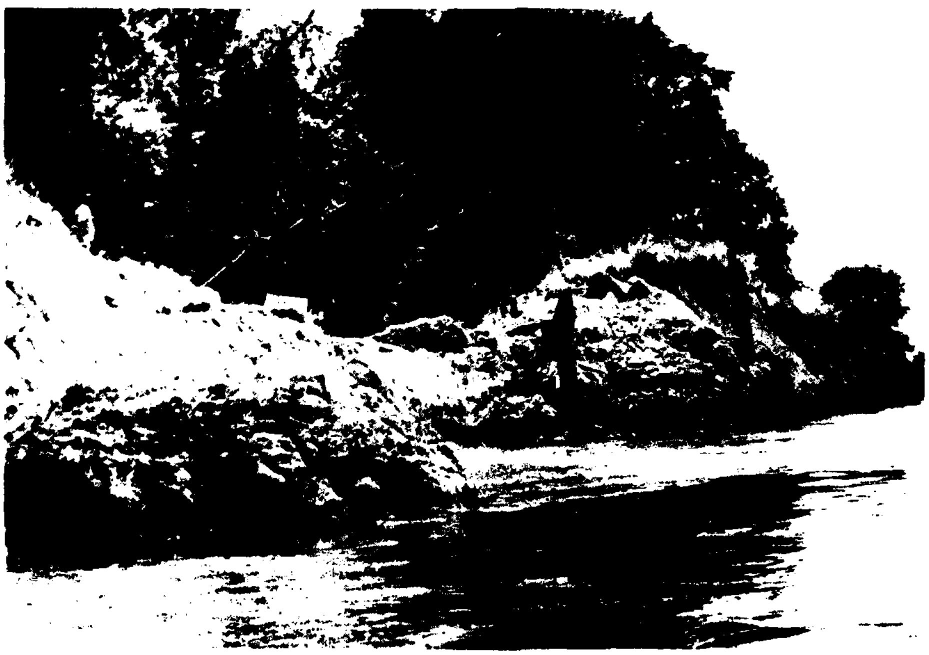


PHOTO 3 STA 19+00

UPPER END
PREPROJECT BANK LINE

PHOTOS 2 AND 3

PHOTO 4 STA 24+00

PHOTO 5 STA 40+00

MIDDLE AND LOWER END
PREPROJECT BANK LINE

PHOTOS 4 AND 5

PHOTO 6 STA 19+00

PHOTO 7 STA 24+00

UPPER END AND MIDDLE
PROTECTED BANK LINE

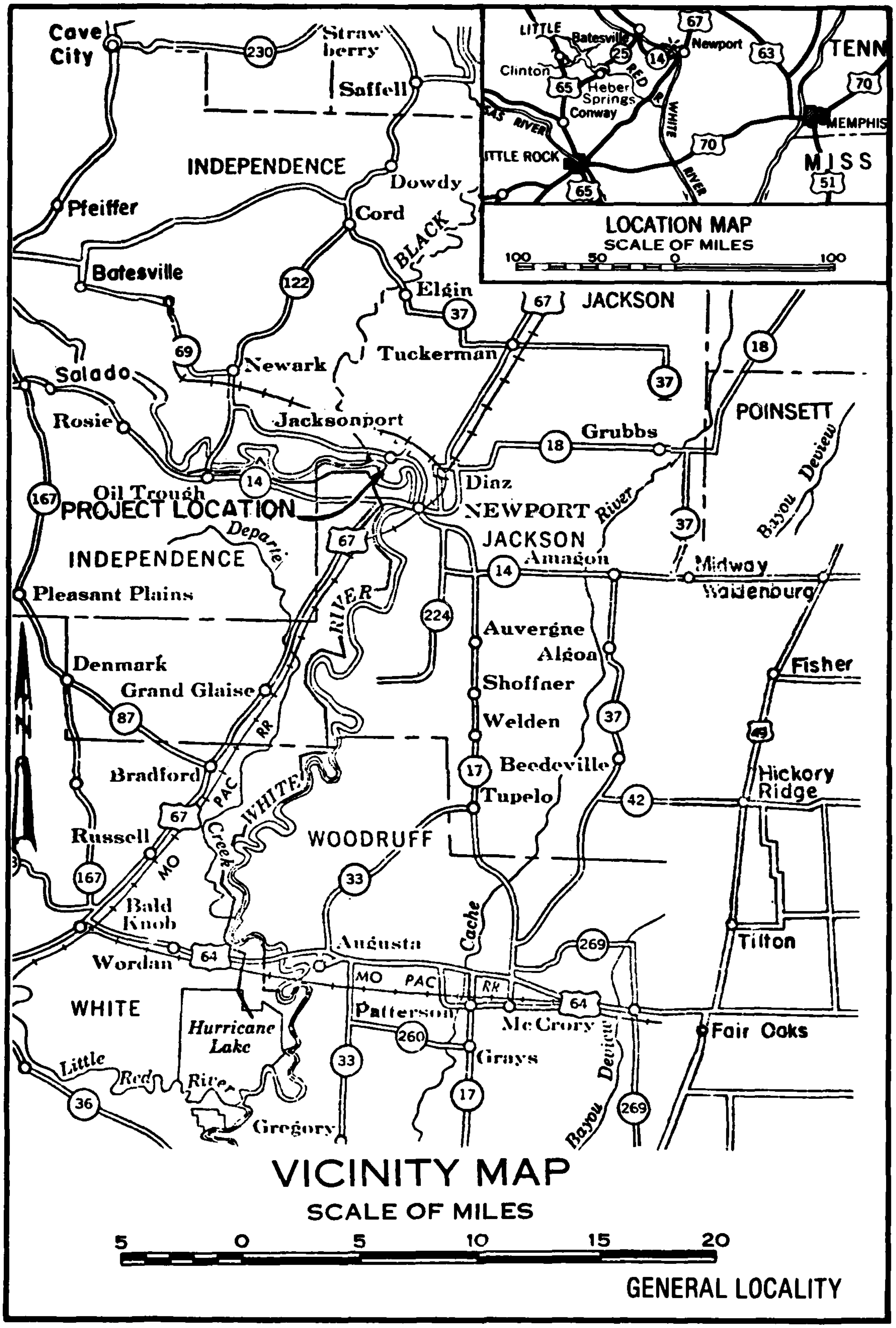
PHOTOS 6 AND 7

PHOTO 8 STA 29+00

PHOTO 9 STA 40+00

MIDDLE AND LOWER END
PROTECTED BANK LINE

PHOTOS 8 AND 9



Cave City
Strawberry
Saffell
INDEPENDENCE
Dowdy
Pfeiffer
Cord
BLACK
Batesville
Elgin
JACKSON
Tuckerman
Newark
Solado
Rosie
Jacksonport
Grubbs
POINSETT
Oil Trough
PROJECT LOCATION
Diaz
NEWPORT
River
Bayou Deview
JACKSON
Departie
INDEPENDENCE
Amagon
Midway
Waldenburg
Pleasant Plains
RIVER
Auvergne
Algoa
Denmark
Fisher
Grand Glaise
Shoffner
Welden
RR
Bradford
Beedeville
Hickory Ridge
PAC
WHITE
Tupelo
Russell
Creek
WOODRUFF
MO
Bald Knob
Augusta
Cache
Tilton
Wordan
MO PAC RR
WHITE
Patterson
McCrory
Fair Oaks
Hurricane Lake
Grays
Little Red River
Gregory
Bayou Deview
LOCATION MAP
SCALE OF MILES
100 50 0 100
LITTLE
Batesville
Newport
Clinton
Heber Springs
Conway
RED
WHITE
RIVER
TENN
MEMPHIS
LITTLE ROCK
MISS
VICINITY MAP
SCALE OF MILES
5 0 5 10 15 20
GENERAL LOCALITY

PLATE 1

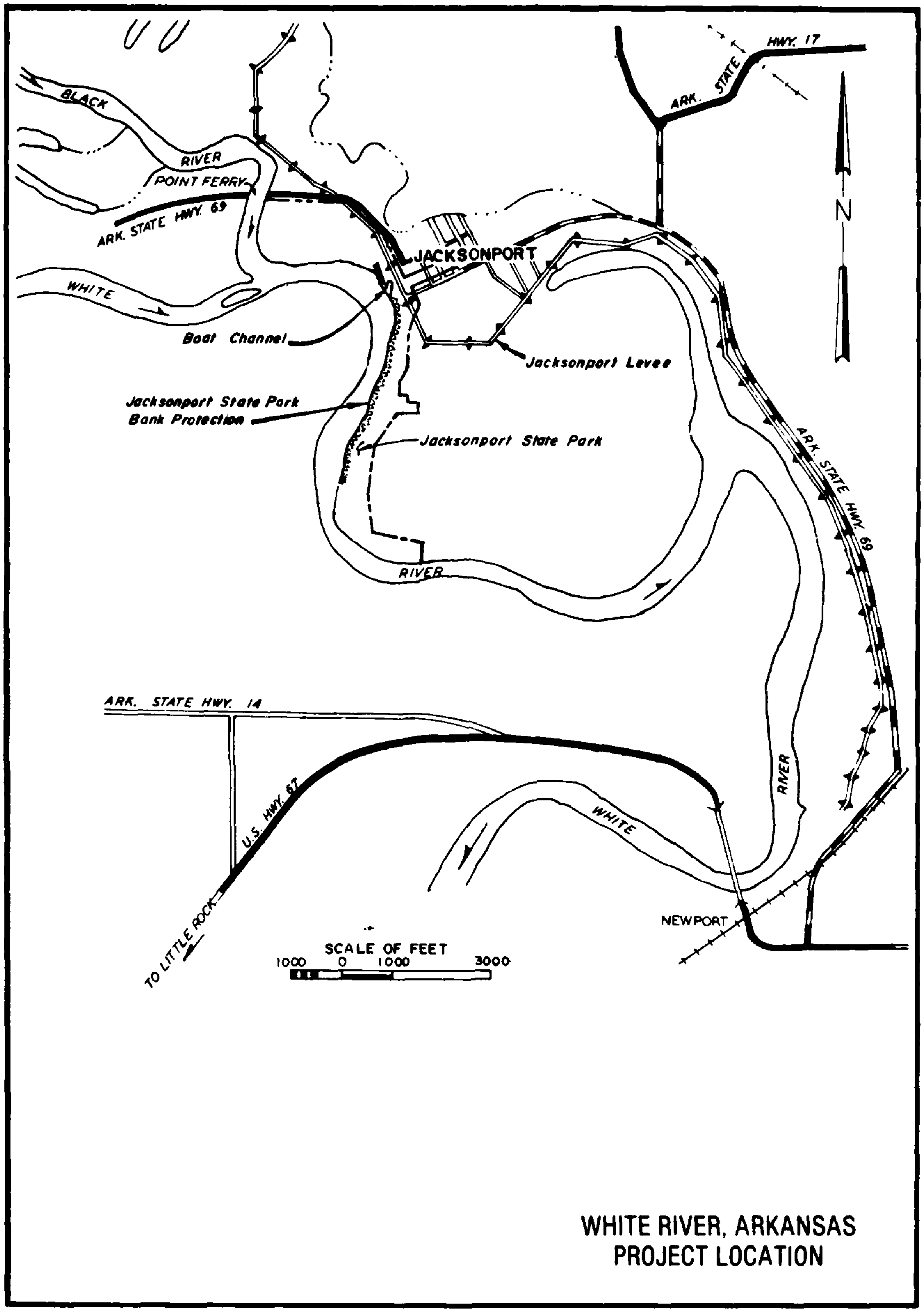
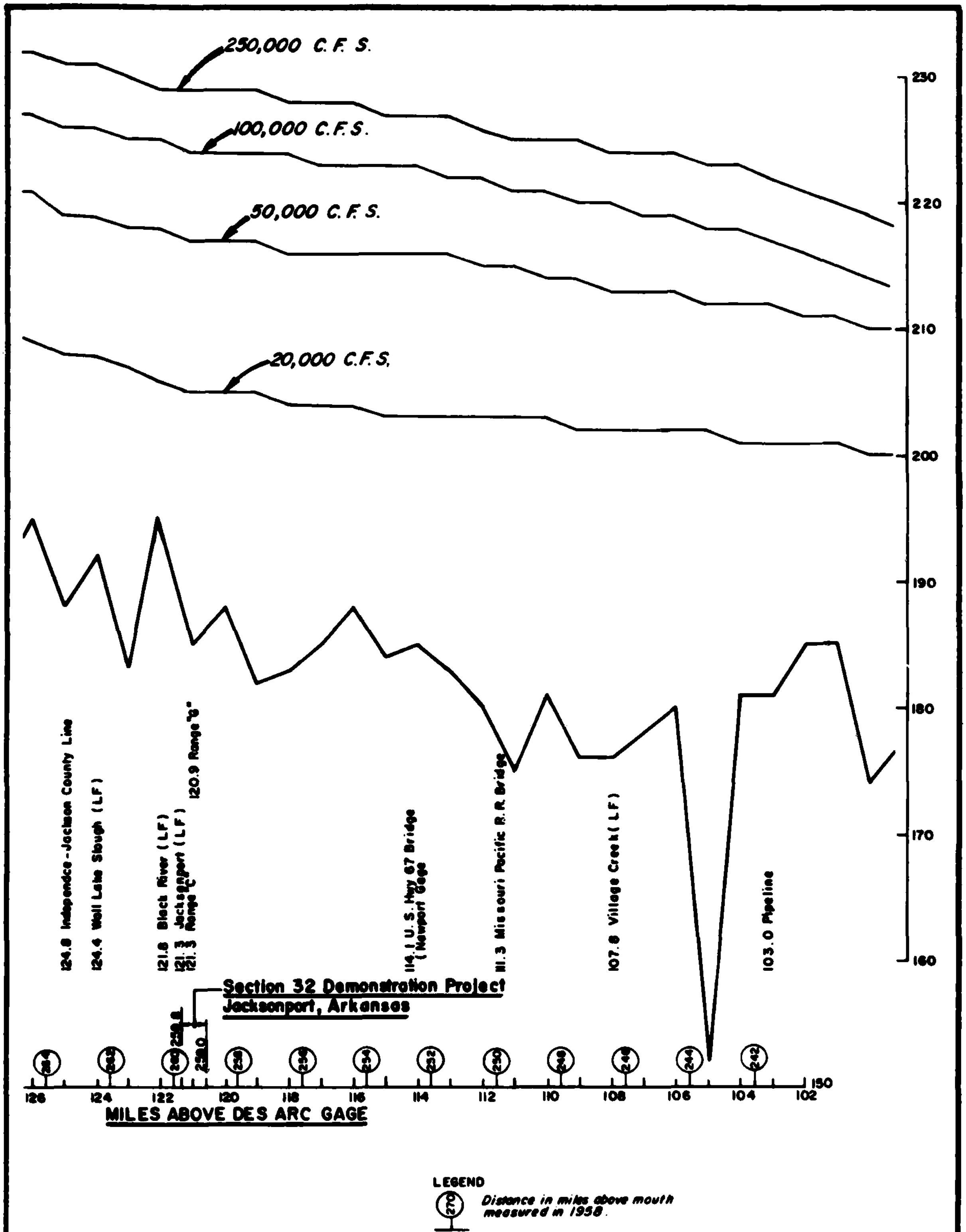
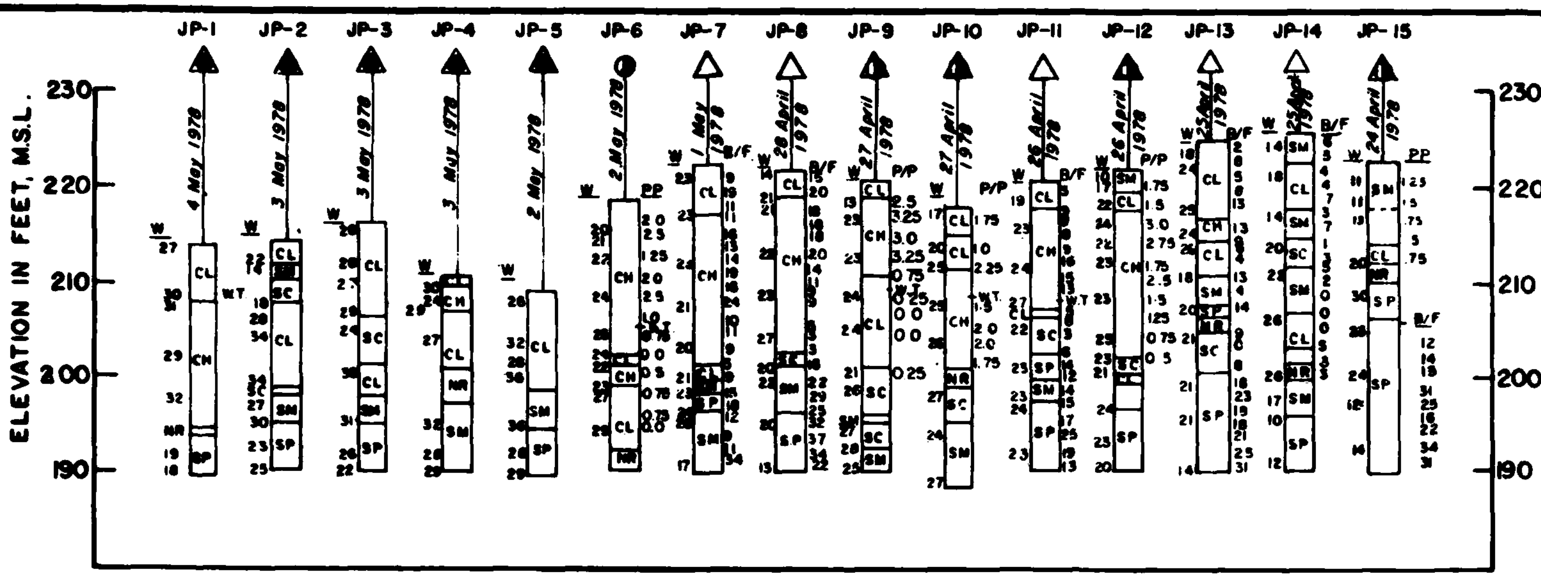

BLACK
RIVER
POINT FERRY
ARK. STATE HWY. 69
WHITE
JACKSONPORT
Boat Channel
Jacksonport Levee
Jacksonport State Park
Bank Protection
Jacksonport State Park
RIVER
HWY. 17
ARK. STATE
N
ARK. STATE HWY. 69
ARK. STATE HWY. 14
U.S. HWY. 67
WHITE
RIVER
NEWPORT
TO LITTLE ROCK
SCALE OF FEET
1000 0 1000 3000
WHITE RIVER, ARKANSAS
PROJECT LOCATION

PLATE 2



WHITE RIVER, ARKANSAS
RIVER CHANNEL PROFILE

PLATE 3



SOILS LOGS

SOILS LEGEND

- ◐ *3" Shelby tube*
- △ *(1 3/8" I.D. x 2" O.D.) std. penetration split spoon sampler*
- *Std penetration split spoon sampler & 3" Shelby tube*
- ▲ *4" hand auger and 3/4" Harris drive*

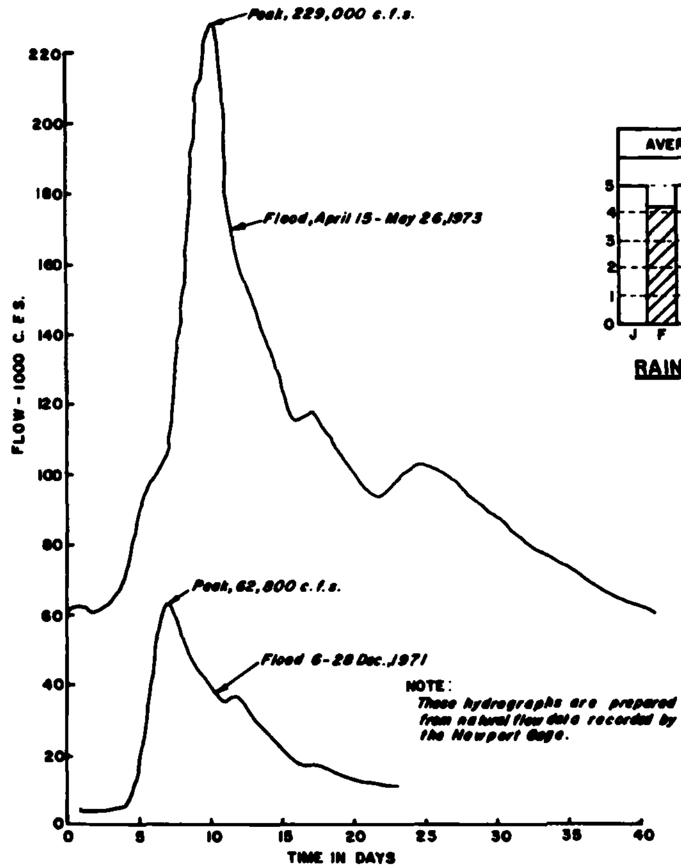
NR	*No recovery*
W	*Natural water content of sample in % dry weight of sample*
P/P	*Pocket penetrometer reading in tons per sq. foot*
B/F	*Blows per foot determined with a standard split spoon sampler. (1 3/8" I.D. x 2" O.D.) and a 140 lb driving hammer with a 30" drop.*
PL	*Plastic limit*
SP	*Poorly graded sands or gravelly sands little or no fines.*
SM	*Silty sands, sand silt mixtures.*
SC	*Clayey sands, sand clay mixtures.*
CL	*Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.*
CH	*Inorganic clays of high plasticity, fat clays.*

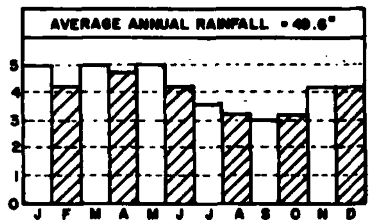
SOILS NOTES:

1. *Figures shown above boring logs are boring identification numbers and dates of exploration.*
2. *Water table (W.T.) elevations are for the dates borings were made.*
3. *Soils classifications shown are in accordance with the Corps of Engineers unified soils classification system and were determined by visual inspection and mechanical analysis.*
4. *All borings terminated at preselected depth.*

SOILS DATA
LEFT RIVER BANK

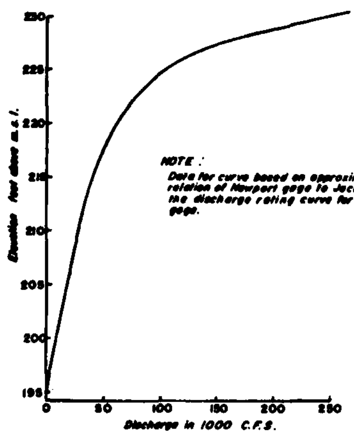
PLATE 4



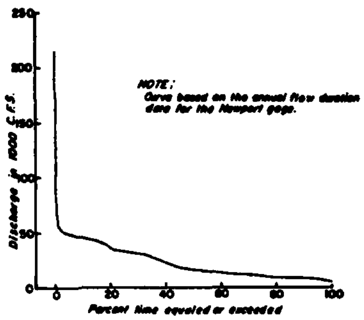


RAINFALL AT PROJECT SITE

TYPICAL MAJOR AND MINOR FLOODS
WHITE RIVER, ARKANSAS



DISCHARGE RATING CURVE



ANNUAL FLOW DURATION CURVE

WHITE RIVER, ARKANSAS
HYDROLOGIC CHARACTERISTICS

PLATE 5

Year	January Number Days 50,000 cfs or more	January Mean Monthly cfs	February Number Days 50,000 cfs or more	February Mean Monthly cfs	March Number Days 50,000 cfs or more	March Mean Monthly cfs	April Number Days 50,000 cfs or more	April Mean Monthly cfs	May Number Days 50,000 cfs or more	May Mean Monthly cfs	June Number Days 50,000 cfs or more	June Mean Monthly cfs
1964	--	5,346	--	7,052	12	42,870	4	31,900	--	15,900	--	8,561
1965	--	13,380	--	18,910	--	17,380	--	29,730	--	17,930	--	14,110
1966	12	44,480	9	38,340	--	26,700	7	35,260	9	46,380	--	19,000
1967	--	11,000	--	12,760	--	13,080	--	11,670	--	22,230	--	9,849
1968	--	24,060	3	41,830	10	36,400	5	46,720	10	46,650	--	24,460
1969	23	54,460	25	70,460	3	46,020	16	52,010	--	32,550	--	12,290
1970	--	14,950	--	10,840	--	18,380	9	31,570	9	38,760	--	15,140
1971	--	34,140	--	29,010	--	25,040	--	11,410	--	14,490	--	12,490
1972	--	13,030	--	9,561	--	10,670	--	20,570	2	28,750	--	10,170
1973	1	34,490	7	41,150	21	57,440	30	94,600	29	82,540	--	35,910
1974	6	42,780	14	49,560	2	46,350	--	40,620	--	38,050	--	36,590
1975	2	40,430	5	39,530	28	69,130	19	69,250	9	34,230	--	15,250
1976	--	20,250	--	16,550	--	18,130	--	21,610	--	16,830	--	19,860

Year	July Number Days 50,000 cfs or more	July Mean Monthly cfs	August Number Days 50,000 cfs or more	August Mean Monthly cfs	September Number Days 50,000 cfs or more	September Mean Monthly cfs	October Number Days 50,000 cfs or more	October Mean Monthly cfs	November Number Days 50,000 cfs or more	November Mean Monthly cfs	December Number Days 50,000 cfs or more	December Mean Monthly cfs	Mean Actual cfs
1964	--	8,533	--	8,828	--	7,754	--	6,605	--	6,909	--	10,110	13,364
1965	--	11,700	--	10,520	--	14,440	--	9,588	--	8,446	--	9,691	14,652
1966	--	13,250	--	11,070	--	8,940	--	8,663	--	7,782	--	9,349	22,435
1967	--	14,140	--	9,046	--	7,158	--	10,030	--	12,820	1	32,610	13,866
1968	--	19,380	--	12,830	--	10,780	--	12,700	--	22,760	10	48,760	28,927
1969	--	12,100	--	10,110	--	7,857	--	9,077	--	8,271	--	9,980	27,095
1970	--	11,710	--	14,120	--	15,980	--	20,900	--	23,010	--	28,580	20,328
1971	--	7,917	--	8,728	--	8,375	--	7,288	--	6,575	3	19,510	15,414
1972	--	10,690	--	10,680	--	9,240	--	11,690	--	41,4[illegible]	--	28,530	16,834
1973	--	28,610	--	28,440	--	26,720	--	16,860	6	32,880	14	60,050	45,024
1974	--	22,270	--	20,780	--	19,550	--	15,750	--	24,640	--	25,490	31,869
1975	--	11,380	--	15,020	--	14,260	--	10,700	--	10,910	--	22,690	29,398
1976	--	25,830	--	16,290	--	8,768	--	11,070	--	8,238	--	7,808	15,936

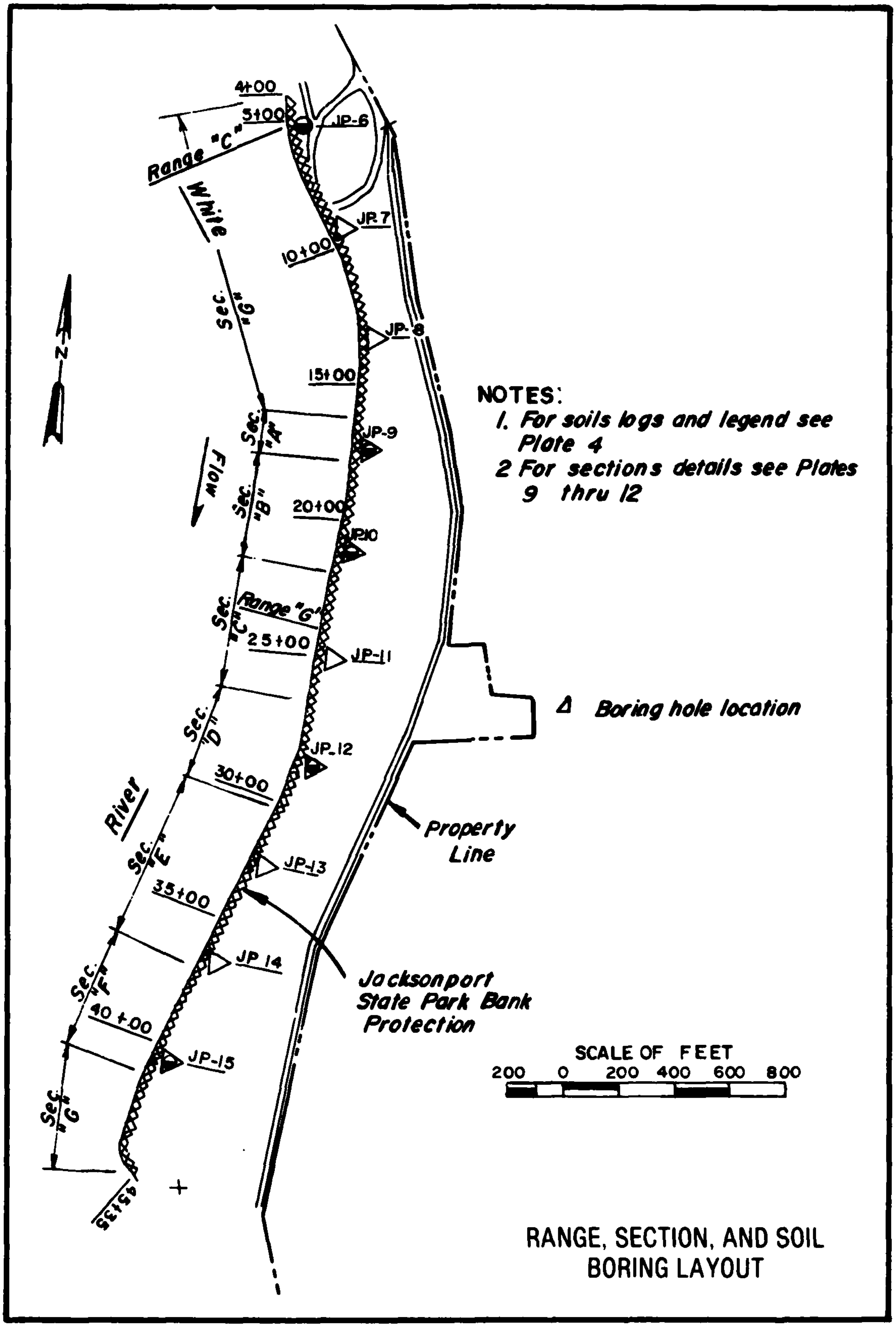
Notes:

a. The above data were obtained from Newport recording gage at bridge on U.S. Highway 67 at Newport, Arkansas, 7.2 miles downstream from Black River.

b. Natural flows are regulated by five upstream flood control reservoirs, namely Norfork (1943), Clearwater (1948), Bull Shoals (1950), Table Rock (1956), and Beaver (1964) Reservoirs.

WHITE RIVER, ARKANSAS
REGULATED FLOW CHARACTERISTICS

PLATE 6



RANGE, SECTION, AND SOIL BORING LAYOUT

PLATE 7

RIVER SECTION PROFILES
RANGES "C" AND "G"

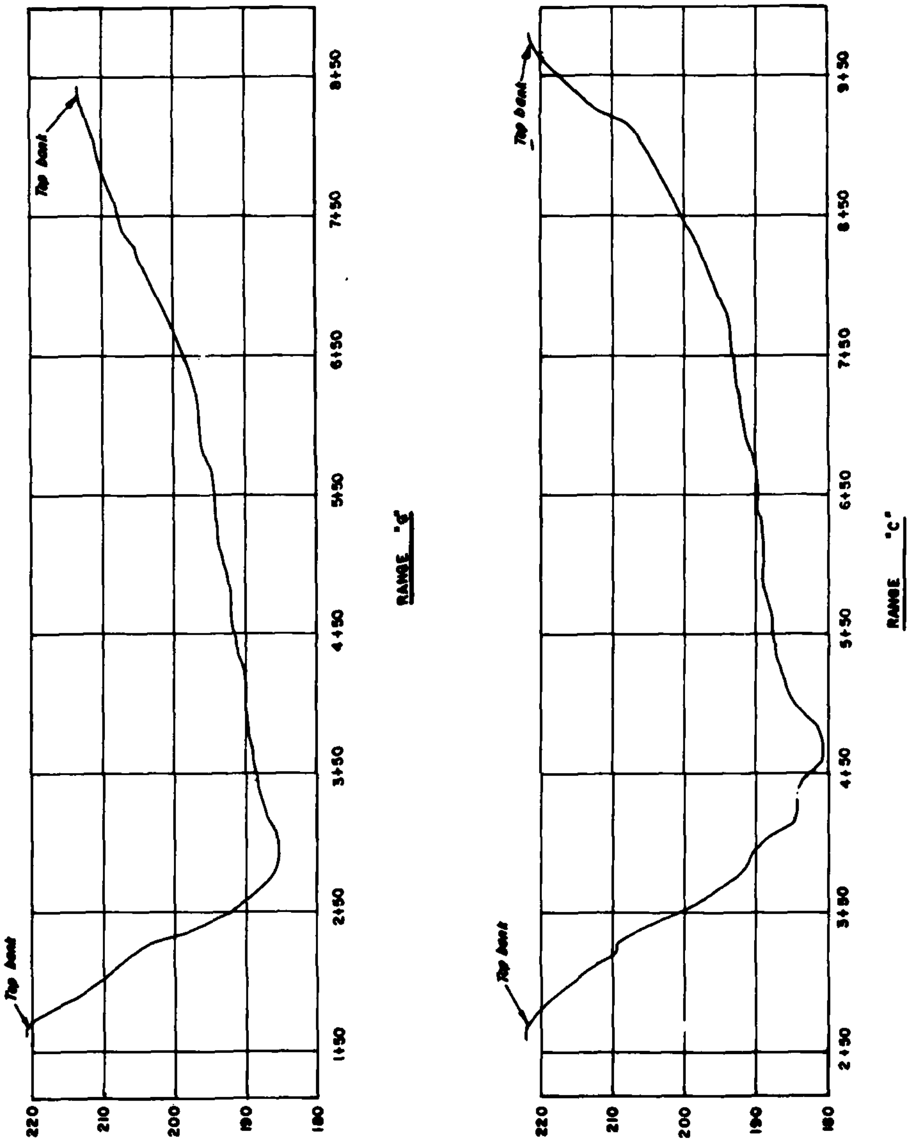
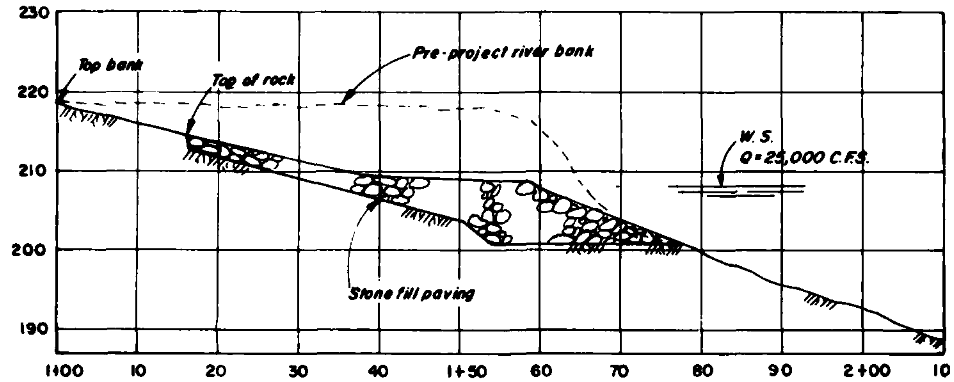
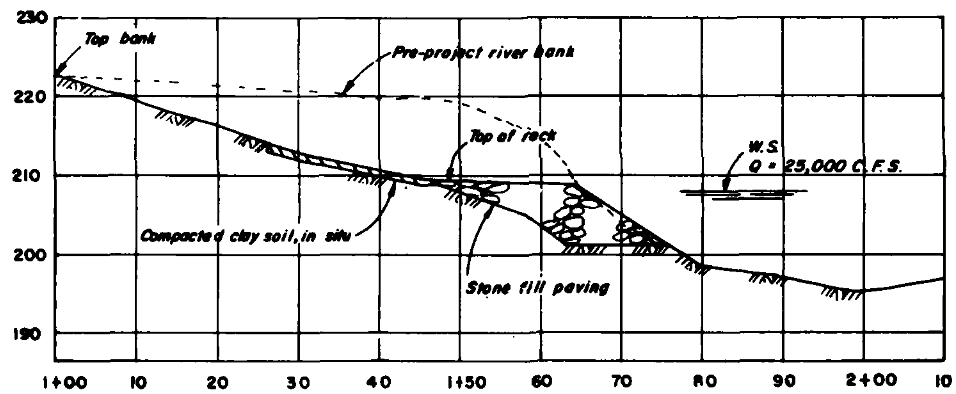


PLATE 8



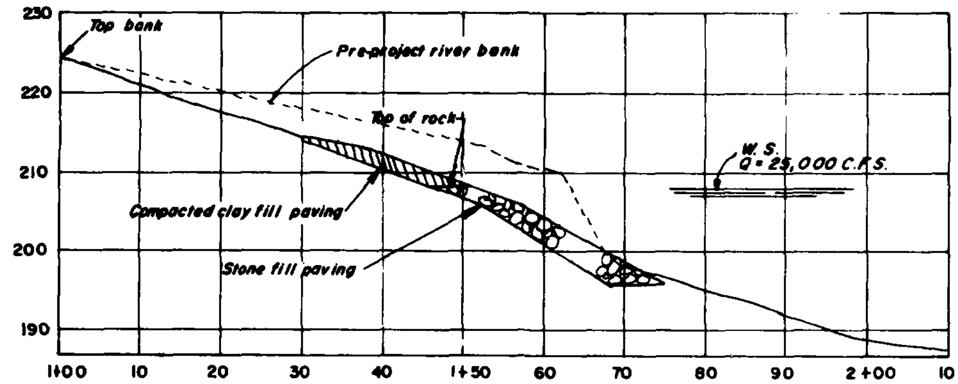
SECTION B



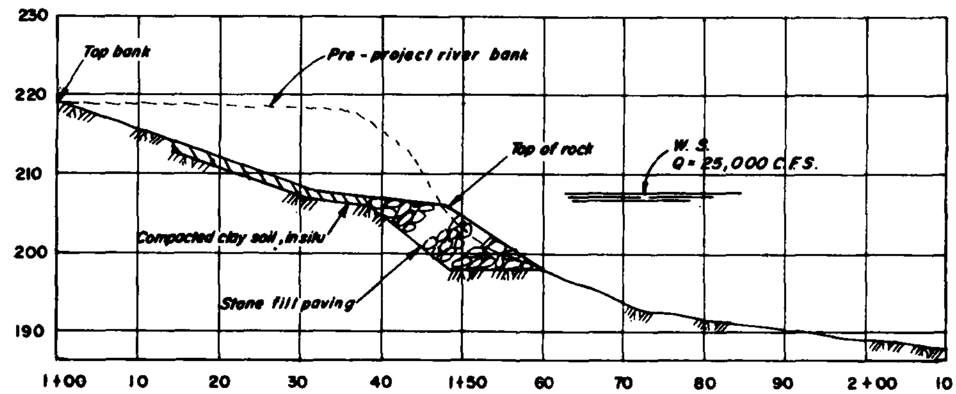
SECTION A

BANK-STRUCTURE SECTIONS
SECTIONS A AND B

PLATE 9



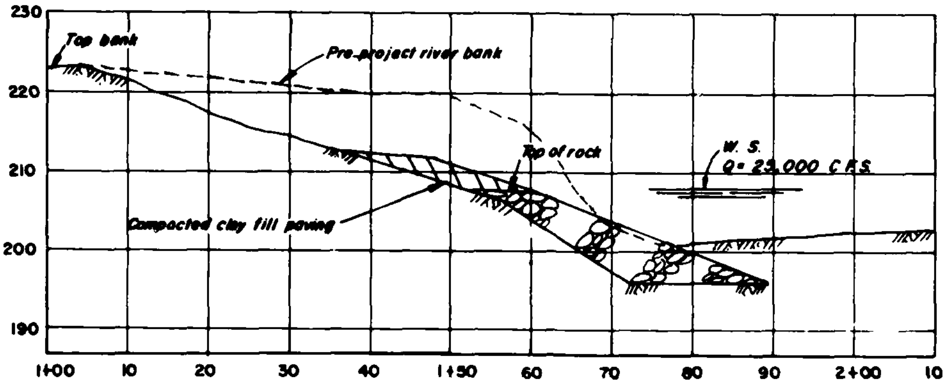
SECTION D



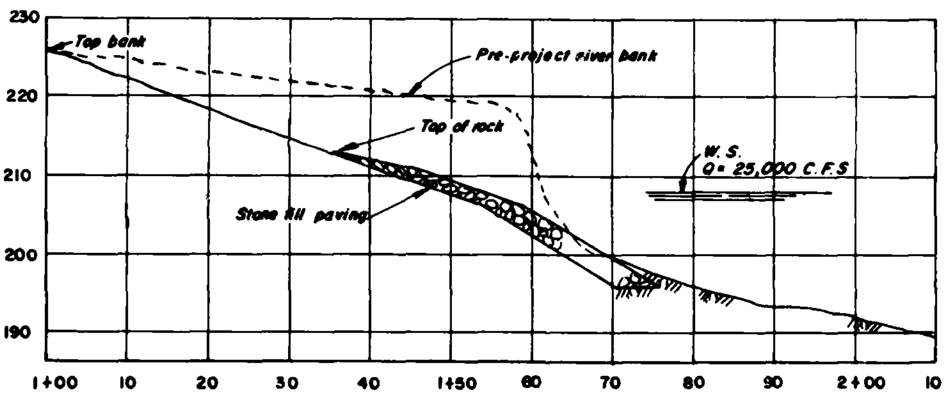
SECTION C

BANK-STRUCTURE SECTIONS
SECTIONS C AND D

PLATE 10



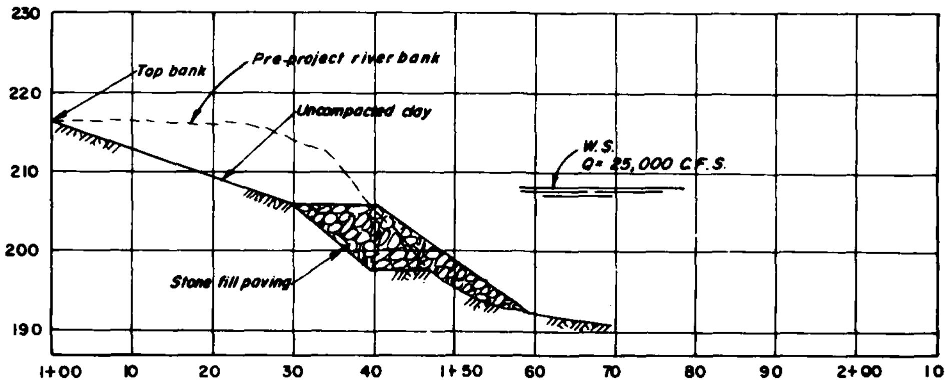
SECTION F



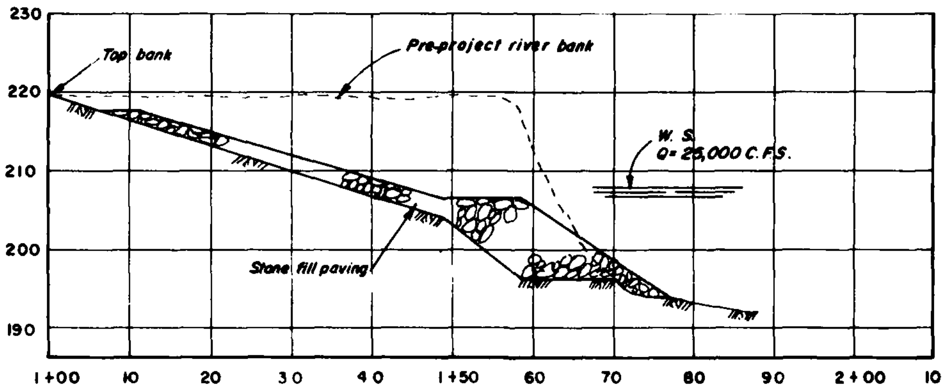
SECTION E

BANK-STRUCTURE SECTIONS
SECTIONS E AND F

PLATE 11



SECTION H



SECTION G

BANK-STRUCTURE SECTIONS
SECTIONS G AND H

PLATE 12

WHITE RIVER AT
DES ARC, ARKANSAS

Section 32 Program Streambank Erosion Control
Evaluation and Demonstration Act of 1974

WHITE RIVER AT DES ARC, ARKANSAS
DEMONSTRATION PROJECT PERFORMANCE REPORT

I. INTRODUCTION

1. Project Name and Location. Streambank Erosion Control at Des Arc, Arkansas, White River Mile 143.

2. Authority. Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251.

3. Purpose and Scope. This report describes a streambank erosion problem, the types of bank protection used, and a preliminary performance evaluation of a demonstration project on the White River at Des Arc, Arkansas, constructed and monitored by the Memphis District.

4. Problem Résumé. Bank caving along the city front of Des Arc had caused the loss or relocation of some houses, the loss of portions of two city streets, and was threatening a project levee which protects portions of the city from flooding. The type of bank caving was unique within the experience of the Memphis District, and the City was powerless to stop it.

II. HISTORICAL DESCRIPTION

5. Stream.

a. Topography. The White River begins in the Ozark Mountains of Arkansas and Missouri and meanders through alluvial deposits of Eastern Arkansas on its course to join the Mississippi River. Des Arc is located on the bluffs along the western edge of the Mississippi River Embayment some 143 miles above the mouth of the White River. The town is situated on a more or less peninsula that projects out into the river's flood plain. This peninsula is probably an erosional remnant left by the meandering of the river. This peninsula is located on an outside bend of the White River and is subjected to potentially scouring velocities. (See Photo No. 1.)

b. Geology. The demonstration site and city of Des Arc are located on the White River along the western edge of the Mississippi River Embayment in east central Arkansas. The bluffs in this area represent the limit of the meander pattern of the river. The river in this area is located on point bar deposits which consist of fine grained surface materials, predominantly silts and clays, overlying sand and gravel deposits which extend to the top of tertiary.

c. Hydrologic Characteristics. The White River Basin is 690 miles in length and drains an area of 27,765 square miles. It is a major tributary of the Mississippi River, which it joins at Mile 599 AHP. The winter and spring months are the high water season and late summer and fall months the low water period. Stream flow is partially controlled by five dams and major reservoirs. Very large, attractive lakes are formed and hydroelectric power is produced. The annual average precipitation is about 46 inches, but has varied from about 92 inches to as low as 20 inches. Average annual runoff is about 17 inches and average annual flow of the White River in the vicinity of the demonstration project at Des Arc is around 27,000 cfs. Minimum flow has ranged to as low as about 4,700 cfs. The lower reaches of the river which include the demonstration site are alluvial plain and characterized by wide flat valleys subject to frequent overflow except where protected by levees. The climate of the region is classified as humid and continental. Temperatures have ranged from 110 to -13 degrees F in this region. The average temperature is about 65°F.

d. Channel Conditions. The White River is navigable to commercial traffic from its mouth to Newport, Arkansas, a distance of 255 river miles. The navigation channel below Augusta, Arkansas, which reach includes the project site at Des Arc, is maintained to a width of 125 feet and minimum depth of 5 feet. A minimum depth of 8 feet is maintained at stages above 12 feet on the Clarendon, Arkansas gage, which is about 76 percent of the time. In this reach of the river, bank to bank widths range from 400 to 800 feet. Oxbow lakes formed by natural cutoffs of old meanders occur along both sides of the river, all within a few miles of the present channel. Lowlands and swamps are common along the river. From its mouth to mile 130 the river meanders through a bottomland hardwood forest. From mile 130 to Augusta (Mile 198) about 70 percent of the banks are forested, and the remainder is mostly in crops or pasture. The slope of the navigable reach of the stream is about one-third foot per mile.

e. Environmental Considerations. The demonstration site prior to construction of the project consisted of a raw caving bank with trees and remnants of houses and streets falling into the river. The project has apparently stopped the caving action and the flattened slopes are covered with relatively attractive paving. The upper slope and adjacent overbank areas were established with grasses. Willow shoots were set in the center of all the tires which were used in the middle section of the project. The record heat and drought throughout the first growing season reduced the survival rate of this attempted vegetative treatment. This area was also fertilized and seeded to improve resistance to erosion and improve appearance. It is expected that the tires will cause silt deposition and the area will develop full vegetative cover naturally. The town of Des Arc owns some land adjacent to the project and has plans to develop it into a park, picnic, and overlook area. No detrimental environmental effects are anticipated.

6. Demonstration Site-Test Reach.

a. Hydrologic Characteristics. The hydrologic characteristics are as stated in paragraph 5c.

b. Hydraulic Characteristics. The average water surface slope is about one-third foot per mile at both low and high stages. Velocity of flow averages about one foot per second at low stages but may range up to 5 feet per second at flood stages. Discharge varies from about 4,700 cfs to 390,000 cfs and averages about 27,000 cfs. The project site is located on the outside of a bend. The higher velocities encountered in this portion of the channel probably speeded the erosional forces by carrying away some of the soft remolded talus from the foot of the bluff, allowing it to flow toward the river, which reduced lateral forces on the foot of the bluff, triggering another cycle of caving and erosion.

c. Riverbank Description.

(1) Bank Materials. The face of the caving bluff was composed of about 8 feet of gray and brown very stiff silty clay, possibly of loessial origin, underlain by a very stiff red clay of high plasticity, and farther underlain by a stiff red silt interbedded with layers of stiff red clay. The material exposed in the bluffs appeared to be heavily overconsolidated, the stratum of red clay in particular. It exhibited a very complex network of joints, fractures, and fissures in both the vertical and horizontal directions, and was laminated in the horizontal direction. Many of the joints were twisted and distorted, indicating a history of high stress. A boring log indicates that the exposed bluff face is underlain by a stiff brown to gray clay to a depth of 47 feet and this is underlain by a gray fine sand. Inclosure 5 to Memo for Record attached to Appendix A shows the log of a boring taken just behind the top of bank in 1975. The boring log agrees generally with the soil types exhibited on the bluff face but the extent of jointing and slickensides was less in the boring samples than was shown on the exposed face. Additional borings were taken in 1978 and the log of these borings is shown on Plate A-1. Soil types are also shown for the inclinometer tube borings on Plates B-2 through B-5.

(2) Normal Bank Vegetation. The normal vegetation on and behind the top of the caving bank was mostly lawn sod and large pecan trees. The bank itself was bare earth standing on an almost vertical bluff about 15 feet high. At the foot of the bluff was a low shelf about 100 to 150 feet wide composed of soft, remolded material with large blocks of intact bluff material floating in it. These blocks were oriented much as they were when part of the bluff and some large trees continued to live. Grass, weeds, and vines grew on this shelf in summer months and it was ususlly inundated during winter and spring. Small willows sometimes became established on the shelf before slowly creeping into the river.

(3) Bank Erosion Tendencies. The riverbank at the project site and an adjacent 1,000 feet of bank upstream of the work have a long history of bank recession. All available records of inspections at this location describe the same characteristics of caving; that is, a bluff bank 15 to 20 feet high with a relatively flat shelf 100 to 200 feet wide at low river stages. The shelf is composed of large chunks of former bank material "floating" in a soft bed of remolded bank material. Photo No. 2 shows this area from the caving bluff to the river and Inclosure 1 to the Memo for Record attached to Appendix A shows it schematically.Typically, a noticeable crack or crevice would develop maybe 8 to 12 feet behind top bank for 20 or 30 feet. Then after several weeks or a few months this big chunk suddenly settles more or less straight down, with any trees and vegetation usually remaining intact. The lower portion of the chunks appear to be remolded, and the upper several feet remain intact. Caving appears to have been more severe during and immediately following periods of high water. By 1937 caving was threatening the Rock Island Railroad line and railroad station in the reach immediately upstream of the project. The rail line and railroad station have been abandoned now and part of the track has caved in. This area had become relatively stable in recent years, apparently because the flat shelf had increased to 400 or 500 feet wide. The extent of bank caving at the project site has been documented since early 1975. Between January 1975 and October 1976 over 100 feet of bank was lost at range 118. Maximum recession from January 1975 to the final stabilized top bank location amounted to 160 feet and occurred at Range 118. Photos 3 and 4 show additional details.

III. DESIGN AND CONSTRUCTION

7. General. A plausible description of the likely bank failure mechanism is as follows: The bluff face, which once experienced high lateral stresses, would experience reduction or loss of lateral stress. Above the talus berm the stress would be zero at the face and increase horizontally into the bluff. Below the bluff base, the bank is confined to some degree by the talus, but the lateral stress is still less than previously experienced. The absence or reduction in lateral stresses and the drying action on the bluff face allow the many joints and fractures to open up. Water from precipitation, percolation, high river stages,

etc., can enter the opened joints, softening the clays at the faces of the joints and causing extensive reduction in shear strength. This shear strength reduction is furthered by the presence of the highly plastic clays on the joint faces. The talus, which consists primarily of the softened and remolded material from the bluffs, is barely stable under its own weight but does offer lateral support at the foot of the bluff. However, with high stages on the river, the talus becomes fully saturated and experiences loss to erosion. Water also enters the joints in the bluff clays, lessening their shear strength. On recession of stages, hydrostatic pressures are generated in the soil mass. This, along with the softened and reduced mass of the talus, could lead to bluff failure and movement of the talus. Movement of the talus could generate shearing strains in the underlying sands, causing excess pore water pressures to develop and a liquefaction failure at the toe of the talus, increasing the rate of movement. With movement of the talus, lateral pressures are also reduced at the foot of the bluff, allowing collapse of the bluff. This occurrence adds mass to the talus, causes more movement, causing more liquefaction failures at the toe of the talus and further collapse of the bluff. The mechanism, once initiated, is essentially self-perpetuating and should continue until the slope of the entire bank is flat enough to attain stability. Except for erosion at the toe of the talus and possibly some minor erosion of its top surface, a state of equilibrium would eventually develop. Inclosure 6 to Memo for Record attached to Appendix A schematically illustrates this theory. A precarious state of such stability has apparently occurred in the approximately 1,000-foot section immediately upstream of the project. Obviously, no erosion control measure can be effective unless the causative failure mechanism can first be halted. This could possibly be done by halting the erosion and riverward migration of the talus shelf. This could possibly have been accomplished by covering the leading edge and most of the talus mass with a filter cloth and riprap paving. Then subsequent erosion and flattening of the bank would eventually lead to an overall stable slope. This would, however, have allowed loss of the project levee (which, incidently, is quite small in this area) as well as additional city lots, streets, and homes. The work under such an approach would also likely have taken frequent repair and reinforcement work before the advancing mass could be halted and stabilized. Again, the above described failure mechanism is somewhat conjectural but appears to be supported by the tentative stability of the adjacent upstream reach. Another, and more immediately obtainable, plan which does not sacrifice significantly more bank, consists substantially of grading and paving the bluff and placing a massive stone fill between the solid bank material and the talus berm throughout the length of the project.

8. Basis of Design. The project design consists of halting progress of the bluff caving by grading the bank to a stable slope; excavating between the original bank material and the talus berm; backfilling with a massive stone fill to provide lateral stability at the foot of the slope; and then provide different types of experimental bank paving on the face of the slope. Several soil borings were taken at the site and a slope stability analysis was performed on a typical section using shear strength parameters from the borings. The soil stability analysis indicated a 1V on 2.5H bank slope and a requirement for stone fill 17 feet thick extending to the underlying sands, and a required quantity of approximately 43 tons of stone per linear foot of bank to provide the necessary mass to assure resistance of the lateral forces at the base of the slope. The projected cost of this design was about double the funding that could be made available for the work. Reducing the length to about one-half the length of actively caving bank was rejected because the shorter length would be of little value to the local sponsors, the whole project could be lost to caving from one or both ends, and such a reduced length would provide too short a reach to adequately demonstrate the effectiveness of the different experimental erosion protection schemes. Since the work envisioned under the Section 32 program is experimental in nature, it was concluded that the factor of safety which might otherwise have been prudent could be reduced substantially and still maintain a reasonable likelihood of successful project accomplishment. A design stone fill quantity of 20 tons per linear foot was arrived at as a figure that was affordable and still stood a reasonable chance, in the designers' view, of achieving the project purpose. The trench for the stone fill section was designed to be excavated as deep as could likely be accomplished with equipment which could work on the soft talus and without going too far below the water table at low river stages, where the material might flow. Side slopes of the trench were as steep on both the original material and talus sides as would stand during construction in order

to provide the most compact stone section. Experimental erosion protection measures which were considered for use on the graded slope included use of a section of riprap paving 12 inches thick to serve as a control against which the other measures could be compared. This is generally considered the most practicable and inexpensive proven method of erosion control in this area. This was rejected because the expected level of performance can be anticipated sufficiently close to permit an objective judgment of the chosen methods. Three bank protection measures were selected whose economy and effectiveness were considered experimental. The three protective measures chosen for use on the slope consisted of: (1) a grout-filled fabric mattress, (2) a revetment of used tires banded together, and (3) a mixed-in-place soil cement paving. These measures were chosen because they are adaptable to small bank protection projects, could conceivably be used by individuals but more likely companies, municipalities, or local governments with their own resources. A plan view and typical cross sections for the work are shown on Plates 1 through 3.

9. Construction Details. The bank was graded and excavation of the toe trench was begun. At this time the bank began to collapse in the upstream and middle portions of the project. Excavation of the toe trench proceeded, followed closely by placement of the stone fill at the rate of 20 tons per linear foot. Excavation of the toe trench was not a factor in collapse of the bank, as the bank failure occurred just as excavation was commenced at the upstream end of the project. As the trench was excavated along the failed slope, the landward wall of the trench would move riverward perceptibly before the trench could be filled with stone. The material continued to settle and flow out over the stone filled toe trench. Before grading and excavation operations were completed, the lower half of the bank had failed throughout virtually the entire 1,000-foot length of the project. Geotechnical engineers determined that the disturbed material in the failure zone would have to be removed because it had almost zero shear strength. This remolded or failed bank material was removed and replaced with a semi-compacted clay gravel. A typical section of this is shown on Plate 4 and Photos 5 through 8 show the failure condition and repairs in progress. The upstream 333-foot section consists of a concrete grout with a minimum thickness of 4 inches placed between two layers of fabric held not

closer together than 4 inches at close intervals. The specifications were written to encourage the contractor to improvise in this construction, if feasible, but he chose to have this section performed by a nationally known firm that specializes in this type of bank protection. In this method two very strong layers of woven nylon fabric are used. At about 3-inch spacing each way, the fabric has nylon strands to limit separation of the two layers of fabric to not less than the specified minimum thickness when a slurry of grout is pumped between the layers. The grout-filled fabric mattress was keyed into the bank at the upstream edge and ends and lapped onto the stone-filled trench at the foot of the slope. Photos 9 and 10 show details of this construction. The middle one-third of the 1,000-foot section is a used tire revetment. Used tires were laid on the graded bank in a close-packed arrangement and banded together at the contact points with 3/4" X 0.020" galvanized steel strapping and heavy duty galvanized steel seals. Tire sizes permitted were automobile or truck type tires 13" to 16" wheel size. In retrospect, it would have been better if a mixture of sizes had not been permitted because the mixed tire sizes did not lend themselves to a close packed arrangement with edges touching and some bands had to span several inches to reach a small tire between larger ones. Screw anchors 3/4" X 66" with 6" blades were set to full depth in the bank on 30-foot spacing each way and 3/8 inch galvanized steel strand attached between them. Tires along the strands were banded to them to anchor the whole system to the riverbank. Photos 11 and 12 show construction of the used tire revetment. A native willow shoot or sprout about 18 inches long and 1/4" to 1" diameter was planted approximately 12" deep in the center of each tire. The area was also fertilized and seeded to help hasten the establishment of vegetative cover. The downstream 333 feet of sloped bank was protected with mixed-in-place soil cement paving. Due to the high clay content, the bank was first stabilized by cutting in 133 pounds per square of hydrated lime, which was allowed to set for at least two days. Then the soil moisture content was to be brought within the range of 31 to 33 percent and portland cement applied at the rate of 575 pounds per square and tilled or disked thoroughly into the ground to a depth of six inches. Some confusion developed during this phase of construction and the actual construction procedure consisted of all the cement for the entire area being distributed and lightly disked in on one day. Water was applied to the slope to raise the soil cement mixture to the specified range and the mixture was thoroughly disked in on the following day. The ground was then thoroughly compacted with a sheepsfoot roller. About 24 hours elapsed from the first light disking in of the cement to completion of the compaction. The extent of cement hydration by natural soil moisture overnight and the loss of strength from working the mixture a full day is unknown, but so far this section of paving looks good.

10. Cost. The total cost of the construction contract amounted to $508,008.13, or $508 per linear foot of bank protected. However, the erosion protection measures placed on the slope, exclusive of the stone toe trench and earthwork, amounted to only $66.66 per linear foot for the grout filled mattress, $103.15 for the used tire revetment, and $63.07 for the soil cement bank protection.

IV. PERFORMANCE OF PROTECTION

11. Monitoring Program. The project was completed in May 1980. The site has not been subjected to a prolonged high-water period at the time of this writing. There was a brief period of high water in June 1981 when bankfull stages prevailed for two days, but that was too short duration to saturate the bank or to test the protection. Monitoring has been accomplished by use of topographic and hydrographic surveys, photographs, personal inspections, and the installation and reading of inclinometers. Inclinometers were placed on top bank, in about the midpoint of the slope, through the stone toe trench, and on the shelf riverward of the stone toe fill. All of them extend down into the underlying sand and their bottoms are assumed to be stationary. Monitoring results to date are as follows: Before completion of the grout-filled fabric section, a crack developed in it approximately along the line of the previous bank failure or junction of original bank slope and replaced material. By the time the entire project was completed about two months later, the crack was a sideways "y" shape extending the full length of the grouted mattress section and had separated 12 to 15 inches near the upstream end and only an inch or two at the downstream end. Photo No. 13 shows this condition. In June 1981 the crack was observed to be two to two and one-half feet separation at the upstream end and about one foot at the downstream end. When the project was completed, the crack could be detected

at intervals in the used tire revetment and soil cement paving. These areas have not worsened as of August 1981. The presence of these cracks in the bank paving is not considered a threat to the integrity of the project, and plans are to just monitor their condition at this time. If the bank stabilizes, the grout-filled mattress probably should be broken up in the vicinity of the widest cracks and riprap stone added to the broken grout to provide full coverage of the slope. Measurement of the movement in the inclinometers shows that the massive stone toe fill has moved as much as 2.3 inches. Settlement of the clay gravel fill and breakup of the grout-filled mattress began before any movement of the stone toe fill was recorded. This was possibly a result of viscous flow of the trench wall into the intersticies or voids of the stone fill. If movement continues in the clay layer underneath the stone, loss of the slope protection may result and further bank recession may occur. The massive stone fill may settle into the shearing soil layer and retard or halt the movement, thus slowing or possibly stopping the recession.

Installation of the seven inclinometers was completed in August 1980. They were read on a biweekly basis for 6 months, then once per month unless there was a substantial change in river stages during the month, in which case biweekly readings were resumed. Results through August 1981 indicate maximum horizontal displacements of 0.4 inch behind top bank, 2.4 inches in the grout filled fabric, 2.3 inches in the stone filled toe trench, and 1.8 inches on the shelf riverward of the work. As can be seen from the displacement plots on Plates B-4 and B-5, the movement of material in the slope and below the stone filled toe trench is confined to shear in a narrow layer two to three feet thick. Plans for further monitoring include occasional site visits; repeating topographic and hydrographic survey coverage to detect changes in the bank, the talus shelf, and the adjacent section of river; and continued reading of the inclinometers.

12. Evaluation of Protection Performance. The work has not been in place long enough to permit an evaluation of its performance except in a very general sense. Performance of the stone filled toe trench will be the determining factor in the ultimate success or failure of the project. Movement recorded to date in the inclinometers is considered a serious indication of bank instability and is

viewed with some alarm by geotechnical engineers. The rigid concrete grout filled fabric is unable to articulate and conform to changes in the subgrade and tends to break into large pieces and all the separation is at the break point. This method of bank protection produces a very esthetic appearance and would probably be a good choice for more ordinary applications. The used tire revetment, which was not anticipated to perform too well, now seems quite promising. A slight river rise during construction put flow two or three feet up on the paved toe of the slope and left a noticeable amount of fill, consisting mostly of what appeared to be silt, in the used tire revetment section. The June 1981 rise in the river did not appear to have any effect on the bank. The record setting heat and drought in the summer and fall of 1980 were very hard on the willow shoots. Only an estimated one percent lived. However, the area is generally well covered with grass and weeds. No changes have occurred in the soil cement slope protection after 15 months of exposure, but it has not gone through a high-water period. However, the bank is lower in this area and was much more stable before and during construction, and the adjacent riverbank below this section is a "normal" riverbank without the unique characteristics described earlier for the project site and the area upstream of it.

13. Conclusion. Inclusion of the bank protection project at Des Arc in the Section 32 program permits demonstration of an extremely complex and possibly unique bank erosion problem coupled with the demonstration of other erosion protection measures which would be generally applicable to many areas and streams in the Mississippi Valley. The demonstration stabilization project is designed to hold the bank, but the shelf extending out into the river is expected to erode away since eroded material is no longer being replaced by caving bank material. Erosion along the riverward side of the wide semi-stable shelf upstream of the project may lead to another round of active bank recession upstream of the project. The presence of the project, while not a causative agent, may possibly hasten this likely occurrence. The project will, however, serve as a hard point and should tend to limit the severity of future bank recession upstream from it.

14. Recommendation. The project was designed within the constraint of available funds being about half the estimated cost of the design suggested in the Bank Stability Study (Appendix A). The conclusions of Mr. Joseph F. Keithley (Chief, Geotechnical Engineering and Survey Branch) stated in Appendix B are that the movements recorded in the inclinometer tubes suggest that a general overall failure is impending. The visible condition of the bank and the experimental erosion control measures is good, and only minor remedial work in the grout-filled fabric mattress section appears to be needed. The cracks in this section do not materially affect the effectiveness of the mattress, and this minor repair should be held in abeyance while stability of the overall soil mass is being monitored. In view of the questionable stability of the bank and soil mass and the deficiency of design as compared to that recommeded in Appendix A, it is recommended that this project not be turned over to the local sponsors until it has been monitored/maintained/rehabilitated for about 3 more years. If possible, a contingency fund should be established for projects such as Des Arc where a sudden and total loss of protection is possible.

DES ARC,
ARKANSAS

PHOTO 1

MATCH LINE

LEFT HALF PHOTO 2

PHOTO 3

PRE-PROJECT CONDITIONS

PHOTOS 2 AND 3

MATCH LINE

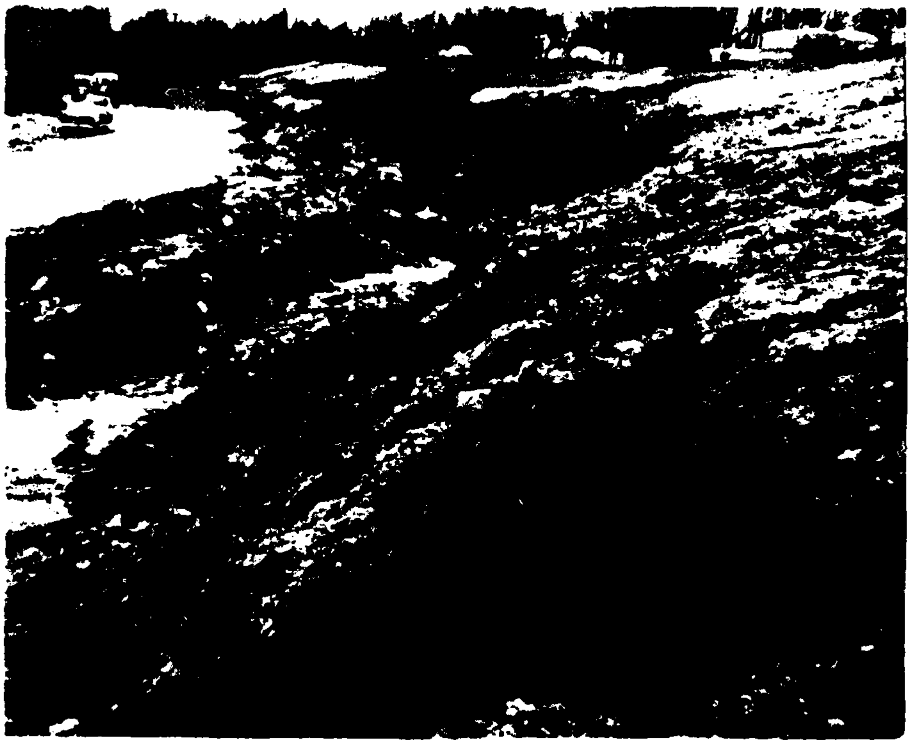
RIGHT HALF PHOTO 2

PHOTO 4

PRE-PROJECT CONDITIONS

PHOTOS 2 AND 4

VIEW UPSTREAM PHOTO 5



VIEW DOWNSTREAM PHOTO 6

BANK FAILURE AFTER GRADING

PHOTOS 5 AND 6

PHOTO 7

VIEW UPSTREAM

PHOTO 8

VIEW DOWNSTREAM

REPLACING FAILED BANK MATERIAL WITH
SEMI-COMPACTED CLAY GRAVEL

PHOTOS 7 AND 8

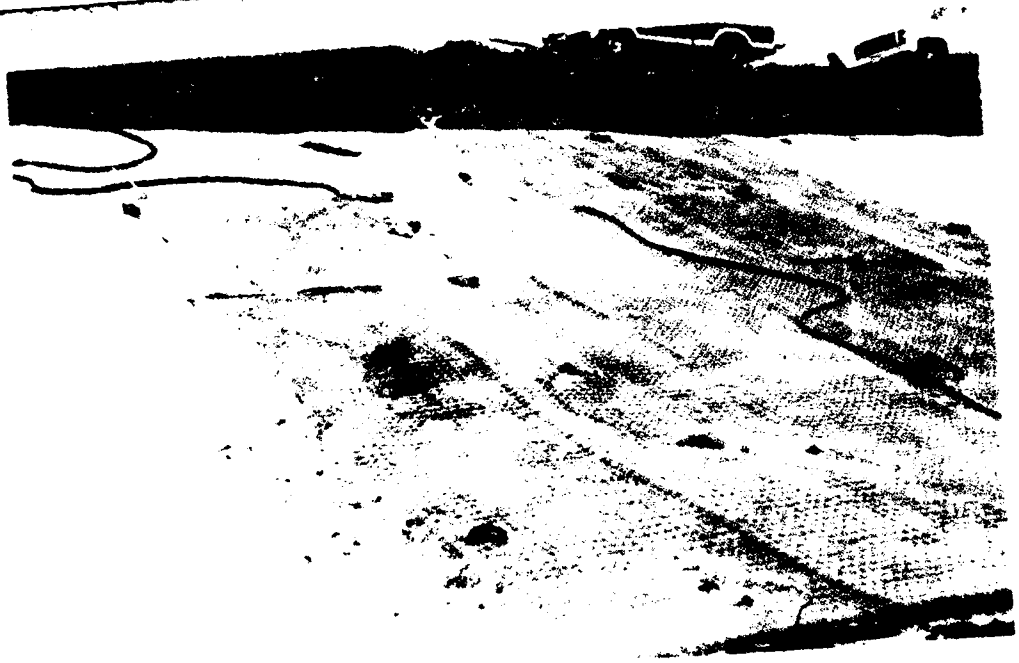


PHOTO 9

CONSTRUCTION OF GROUT-FILLED FABRIC

PHOTO 10

DETAIL OF GROUT FILLING OF NYLON FABRIC

PHOTOS 9 AND 10

PHOTO 11

PLACEMENT AND ANCHORAGE OF USED TIRES

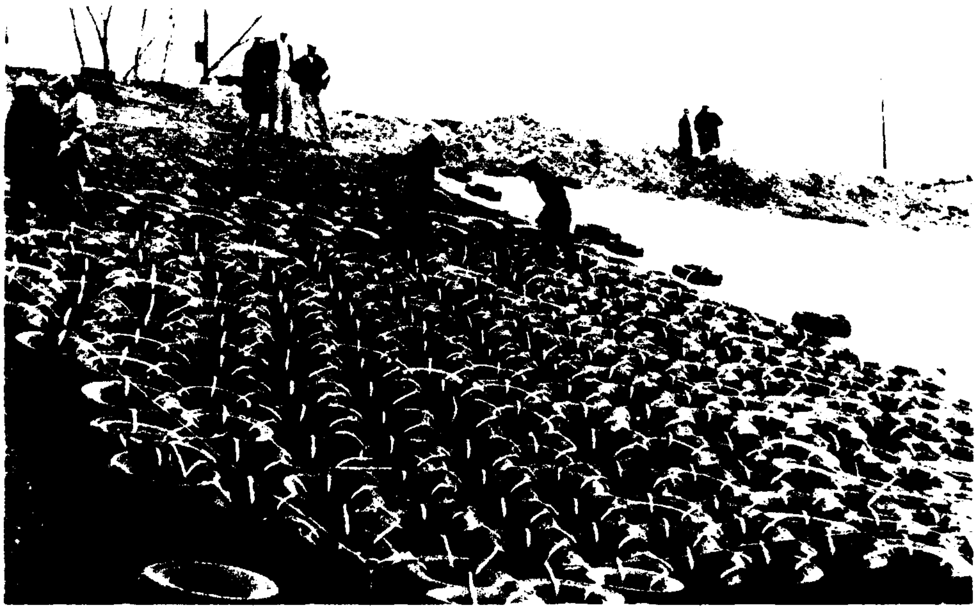


PHOTO 12

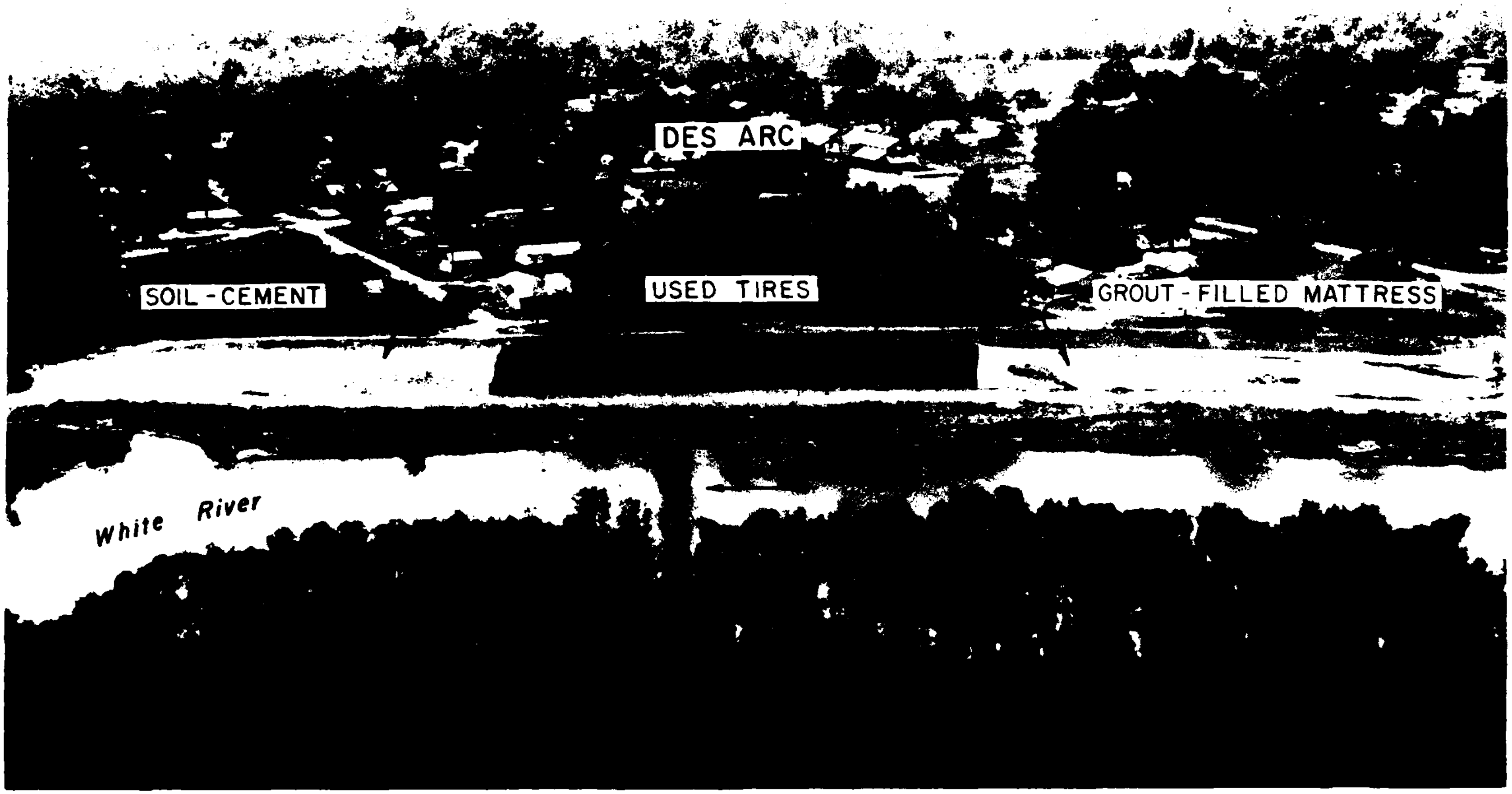
USED TIRE REVETMENT

PHOTOS 11 AND 12

PHOTO 13

GROUT-FILLED MATTRESS 2 MONTHS AFTER CONSTRUCTION
BREAKAGE AND SEPARATION DUE TO SUBGRADE MOVEMENT

PHOTO 14



VIEW OF COMPLETED PROJECT SHOWING
3 TYPES OF BANK PROTECTION

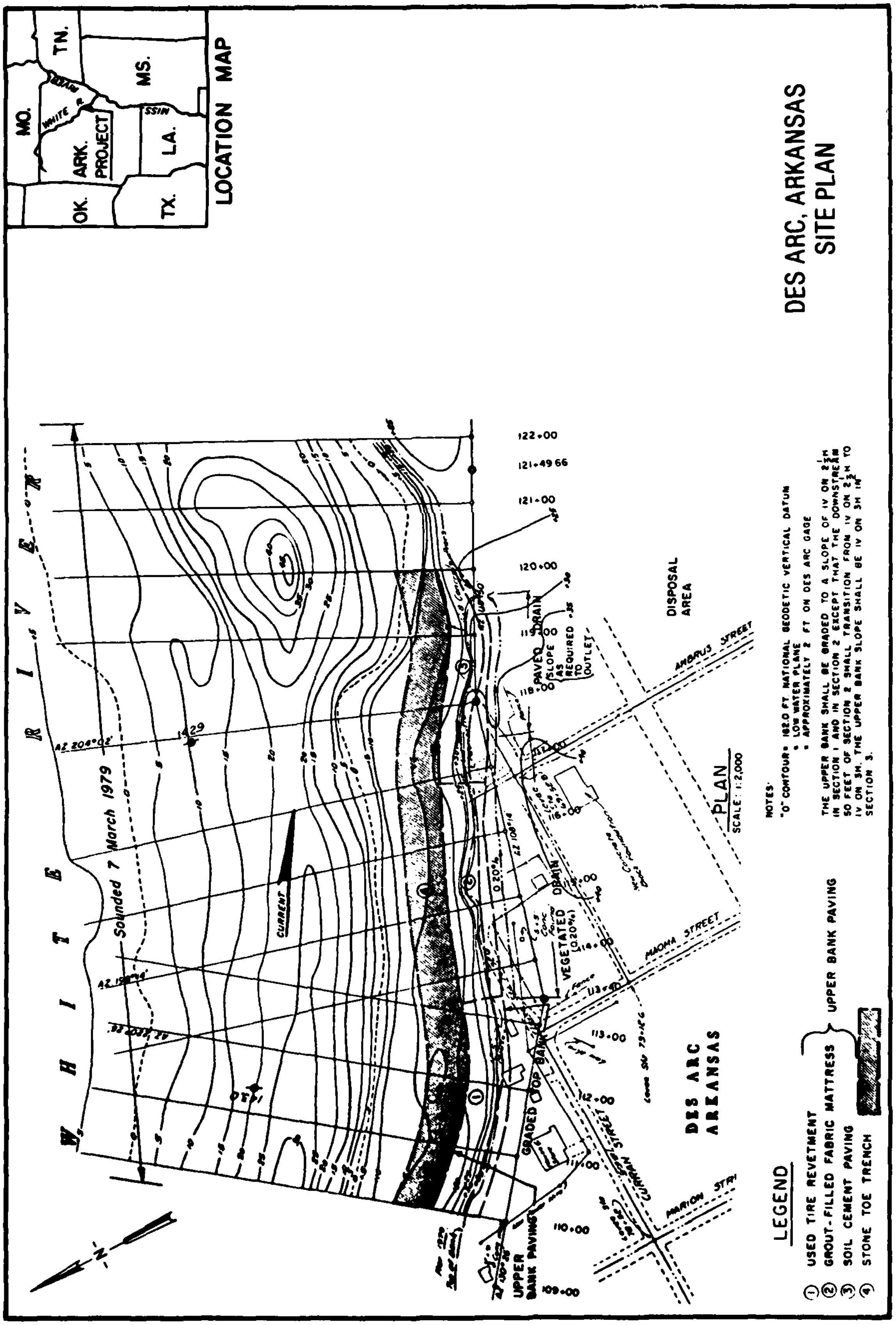
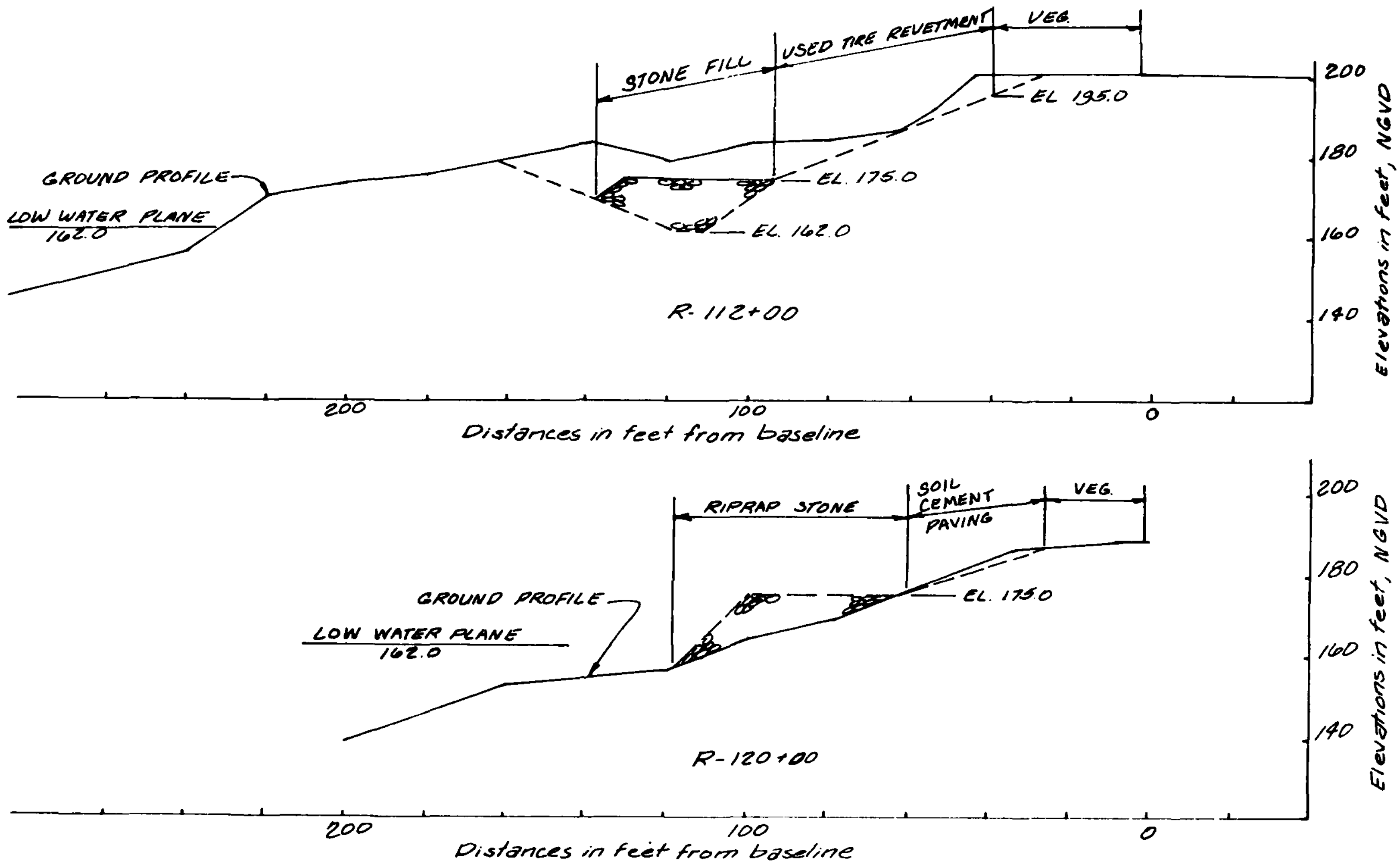

LOCATION MAP
MO.
TN.
MS.
ARK.
PROJECT
WHITE R.
OK.
TX.
LA.
DES ARC, ARKANSAS
SITE PLAN
WHITE RIVER
Sounded 7 March 1979
CURRENT
DISPOSAL AREA
DES ARC ARKANSAS
PLAN
SCALE: 1:2,000
NOTES:
"0" CONTOUR = 162.0 FT. NATIONAL GEODETIC VERTICAL DATUM
= LOW WATER PLANE
= APPROXIMATELY 2 FT ON DES ARC GAGE
THE UPPER BANK SHALL BE GRADED TO A SLOPE OF 1V ON 2½H IN SECTION 1 AND IN SECTION 2 EXCEPT THAT THE DOWNSTREAM 50 FEET OF SECTION 2 SHALL TRANSITION FROM 1V ON 2½H TO 1V ON 3H. THE UPPER BANK SLOPE SHALL BE 1V ON 3H IN SECTION 3.
LEGEND
① USED TIRE REVETMENT
② GROUT-FILLED FABRIC MATTRESS
③ SOIL CEMENT PAVING
④ STONE TOE TRENCH
UPPER BANK PAVING
PAVED DRAIN SLOPE AS REQUIRED TO OUTLET
VEGETATED (0.20%)
GRADED TOP BANK
UPPER BANK PAVING
ANDRUS STREET
MAONA STREET
MARION STREET
CURRAN STREET
122+00
121+49.66
121+00
120+00
119+00
118+00
116+00
114+00
113+00
112+00
111+00
110+00
109+00

PLATE 1



TYPICAL BANK SECTIONS

PLATE 2

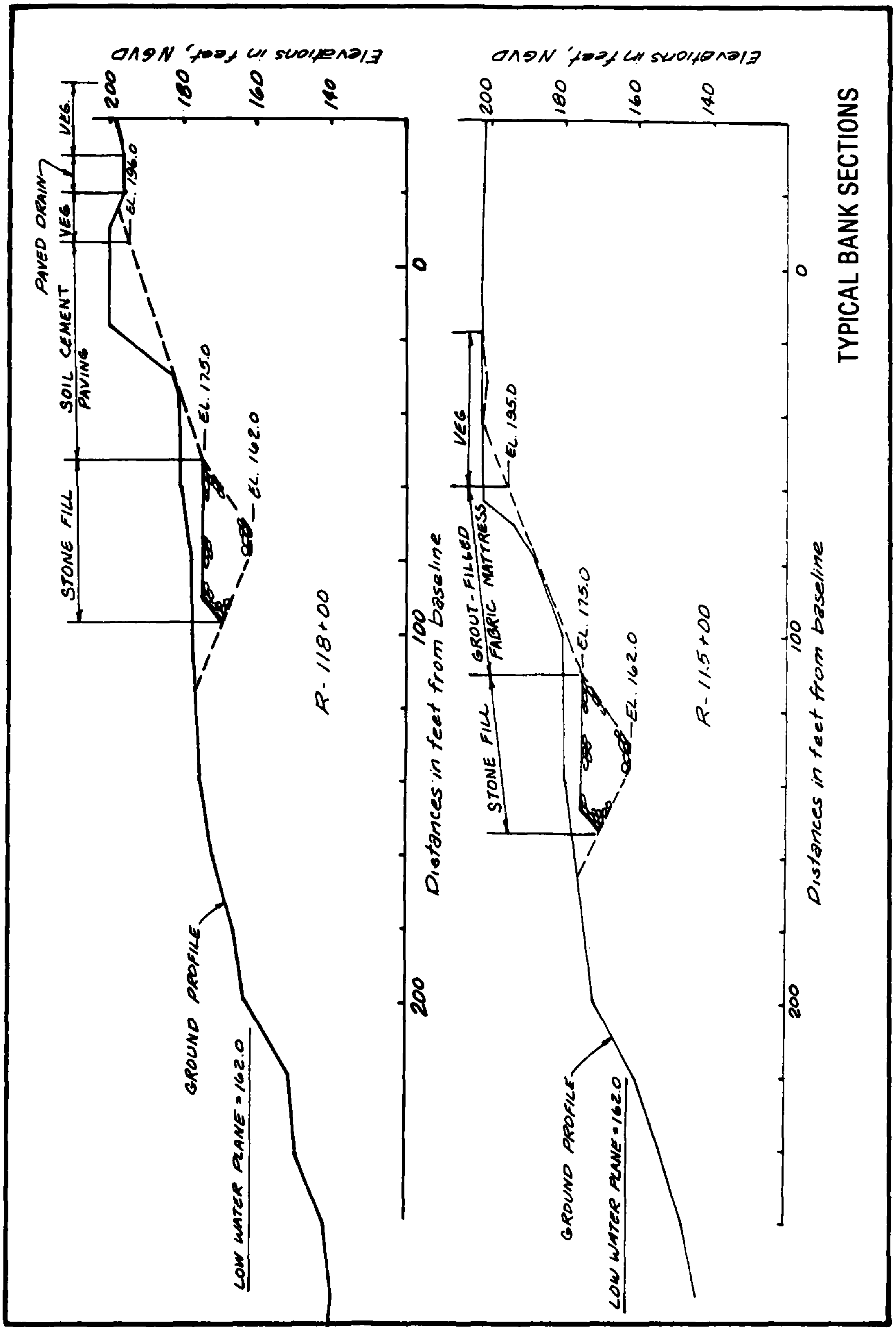

TYPICAL BANK SECTIONS
Elevations in feet, NGVD
200
180
160
140
VEG.
PAVED DRAIN
VEG
EL. 196.0
SOIL CEMENT PAVING
EL. 175.0
EL. 162.0
STONE FILL
R-118+00
0
100
200
Distances in feet from baseline
GROUND PROFILE
LOW WATER PLANE = 162.0
Elevations in feet, NGVD
200
180
160
140
VEG
EL. 195.0
GROUT-FILLED FABRIC MATTRESS
EL. 175.0
EL. 162.0
STONE FILL
R-11.5+00
0
100
200
Distances in feet from baseline
GROUND PROFILE
LOW WATER PLANE = 162.0

PLATE 3

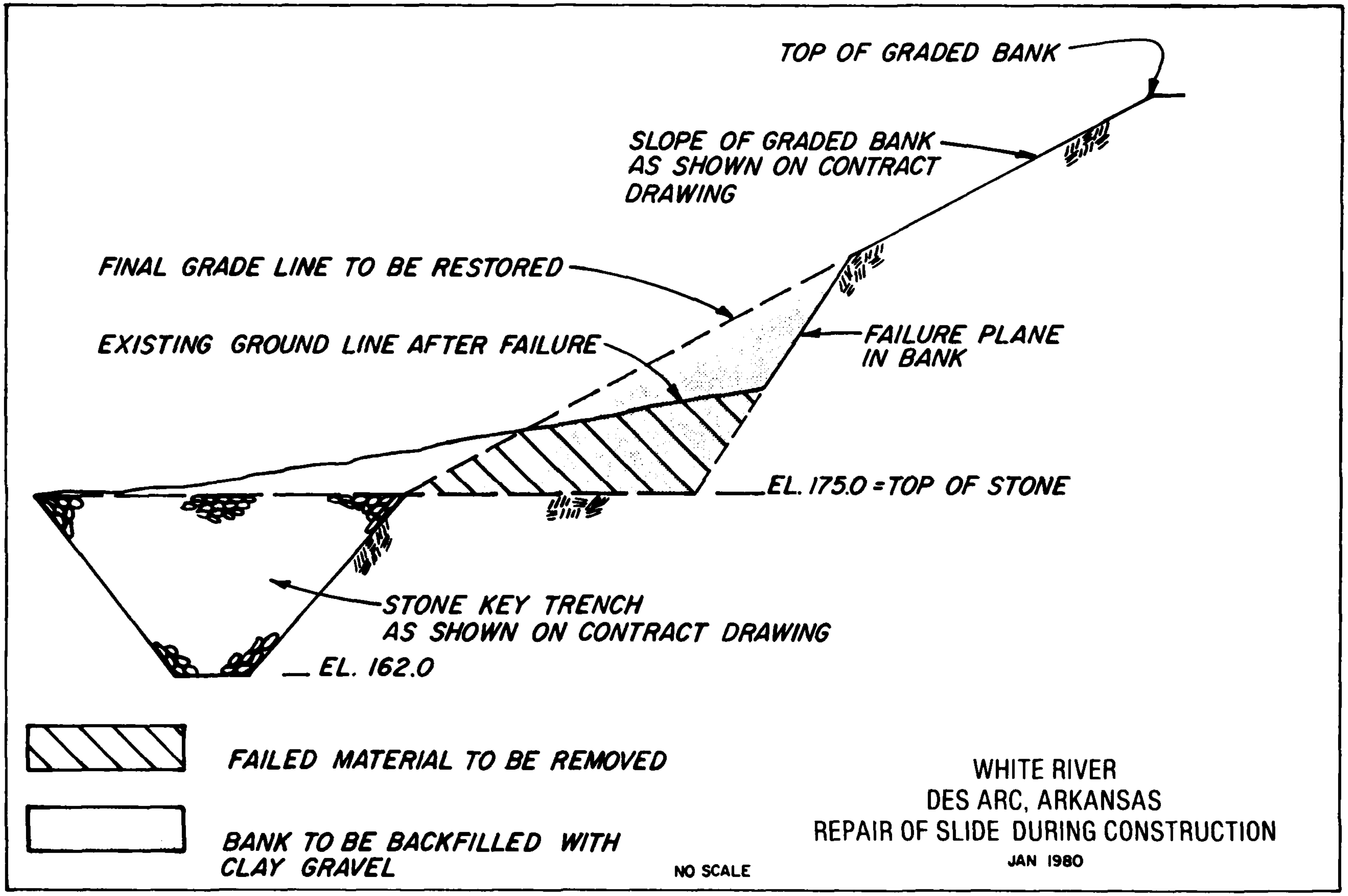

TOP OF GRADED BANK
SLOPE OF GRADED BANK AS SHOWN ON CONTRACT DRAWING
FINAL GRADE LINE TO BE RESTORED
EXISTING GROUND LINE AFTER FAILURE
FAILURE PLANE IN BANK
EL. 175.0 = TOP OF STONE
STONE KEY TRENCH AS SHOWN ON CONTRACT DRAWING
EL. 162.0
FAILED MATERIAL TO BE REMOVED
BANK TO BE BACKFILLED WITH CLAY GRAVEL
NO SCALE
WHITE RIVER
DES ARC, ARKANSAS
REPAIR OF SLIDE DURING CONSTRUCTION
JAN 1980

PLATE 4

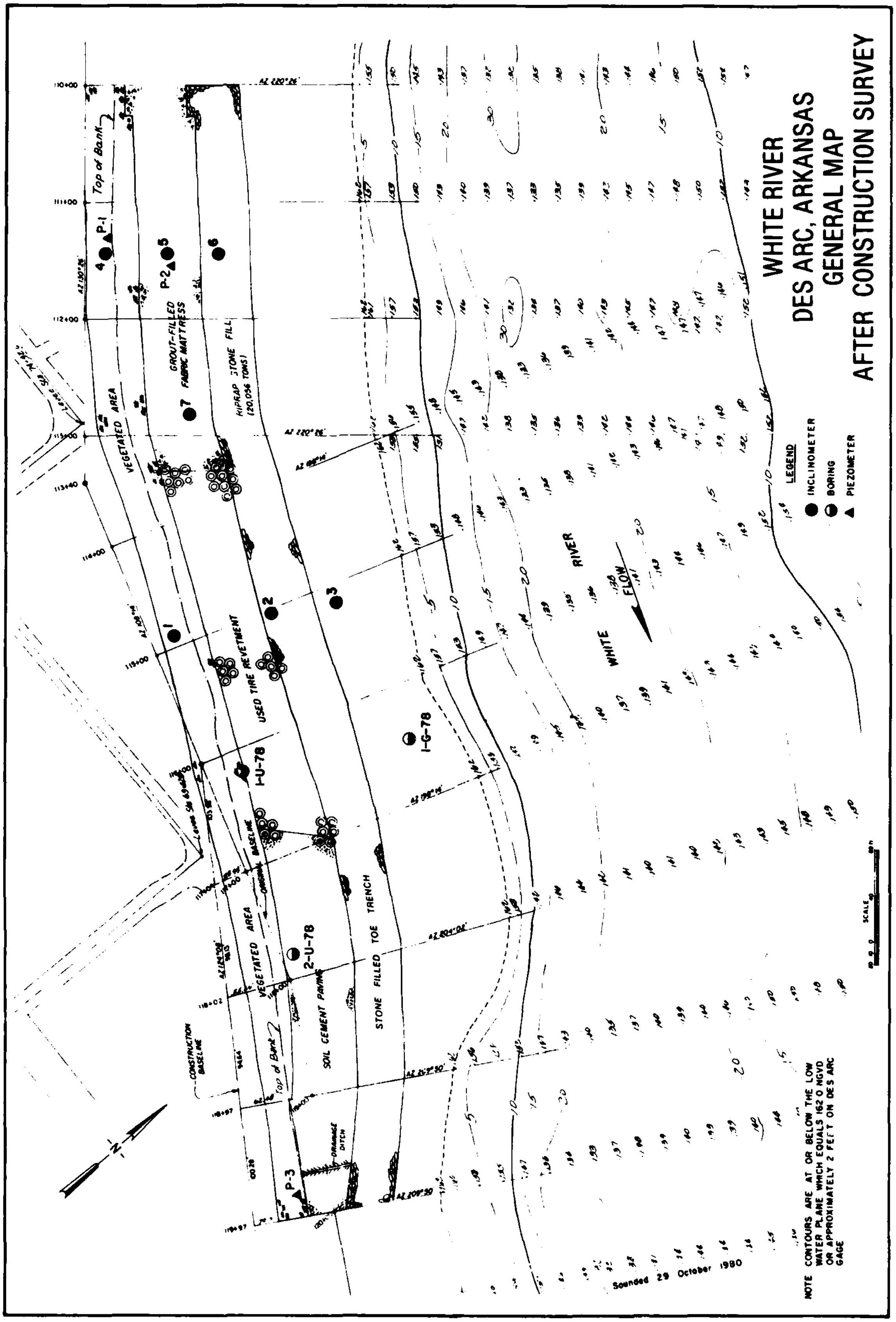

WHITE RIVER
DES ARC, ARKANSAS
GENERAL MAP
AFTER CONSTRUCTION SURVEY
LEGEND
INCLINOMETER
BORING
PIEZOMETER
VEGETATED AREA
GROUT-FILLED FABRIC MATTRESS
RIPRAP STONE FILL (20,056 TONS)
USED TIRE REVETMENT
STONE FILLED TOE TRENCH
SOIL CEMENT PAVING
VEGETATED AREA
Top of Bank
CONSTRUCTION BASELINE
ORIGINAL BASELINE
DRAINAGE DITCH
WHITE RIVER
FLOW
1-U-78
2-U-78
1-G-78
P-1
P-2
P-3
Sounded 29 October 1980
NOTE CONTOURS ARE AT OR BELOW THE LOW WATER PLANE WHICH EQUALS 162.0 NGVD OR APPROXIMATELY 2 FEET ON DES ARC GAGE
SCALE

PLATE 5

APPENDIX A
DES ARC, ARKANSAS
BANK STABILITY STUDY

a. General. The town of Des Arc is situated atop the bluffs located along the west bank of the White River in east central Arkansas. The western bank of the White River immediately alongside the town has been the scene of serious bank caving in recent years. Several inspections of the area have been made in the past in an effort to determine what remedial measures would be appropriate. Memo for Record, LMMED-F, dated 21 December 1976, (copy attached) gives a detailed account of one such inspection. The Memo for Record also addresses the geological and subsurface conditions at the site, past occurrences of bank caving, probable cause and mechanism of bank failure, and possible remedial measures.

b. Soils Exploration. The soil conditions in the vicinity of the bank caving were investigated by taking 2 undisturbed borings and one general sample boring. Samples were taken with auger, 5-inch tube, and split spoon samplers. The boring locations are shown on Plate 5 and the logs of these borings are shown on Plate A-1.

c. Selected Plan of Repair. The failure mechanism associated with the bank caving was assumed to be similar to that as described in para 5 of attached Memo for Record. Therefore, it was felt that the placement of erosion protection on the bank surface alone would not be adequate to stabilize the banks. Several remedial measures were considered for stabilizing the bank section, but the one which appeared to have the most merit consisted of grading the banks to a stable slope, providing slope protection, and constructing a massive rock toe throughout the length of the affected area. A section illustrating the proposed method of repair is shown on Plate A-2.

d. Shear Strength Characteristics. The selection of design values for cohesive and semi-cohesive soils for the Q and R condition based was on consistencies indicated by the boring logs, natural moisture contents, the Atterberg limits, and the unconfined compression test results when available. Design values for the S condition for these soils were based on a plot of the plasticity index versus ϕ'

correlation shown on page 5 of TR-3-604. The cohesion for the S case was assumed equal to zero. For all cohesionless soils, shear strength design values of $\phi'=33^{o}$ and c=0 were selected for the Q, R, and S conditions. Values of $\phi'=38^{o}$ and c=0 were selected for the stone fill.

e. Stability Analysis. Logs and test results from Borings 1-U-78, 2-U-78 and 3-G-78 were examined to determine appropriate soil stratification and shear strengths to use in slope stability analyses. Bank slopes were analyzed using criteria specified in DIVR 1110-1-400, Section 5, Part 4, Item 1, dated March 1973. Because the bank caving was located in an urban area where availability of right-of-way was somewhat restricted, it was intended to provide the steepest stable bank slope possible to minimize right-of-way requirements. A series of slope stability analyses were performed on a typical section using shear strength parameters and soil stratification from the borings. It was assumed in the analyses that a trench would be excavated at the base of the slope between the bluff material and the talus material for the placement of the rock toe (See Plate A-2). The cut on the bluff side would be made through undisturbed bluff material. It was also assumed that the trench would be excavated down to sand (El. 158±) and would have a bottom width of six feet. The trench would then be backfilled to elevation 175.0 with a sufficient quantity of stone to provide a stable bank section. The section was initially analyzed to determine the required slopes to provide adequate stability during excavation of the trench. Before excavation slopes in the talus could be determined, it was necessary to assign shear strength to the talus material. Therefore a stability analysis as shown on Plate A-3 was performed in an effort to determine the cohesive strength of the talus material. Since the talus was actively failing at the time of inspection, it was assumed that the factor of safety for the section was probably somewhere in the range between 0.95 and 1.00. From the results of several trials it was found that a cohesive value of 240 psf for the talus material yielded a factor of safety in this range. A cohesive value of 240 psf was then used in the analysis as shown on Plate A-4 to determine the appropriate excavation slope for the talus material. This analysis indicated that a 1V on 2.75 H excavation slope was required. An additional stability analysis was performed for the bluff side of the proposed trench to determine the required excavation slopes there. Soil stratification and shear strength values for the bluff material were based on a composite of Borings 1-U-78 and 2-U-78. The analysis as shown on Plate A-5 indicates that a 1V on 1.5 H slope from elevation 158.0 to 175.0 and a 1V on 2.5 H slope from elevation 175.0 to top bank is adequate. An overall analysis indicating the stability of the section after the placement of stone is presented on Plate A-6. It was assumed that approximately 43 tons of stone per linear foot of bank would initially be placed as shown on Plate A-7. For purposes of analysis, however, it was assumed that bank scour and failure in the talus would progress even after placement of the stone fill until enough stone was displaced from the riverside portion of the trench to line and protect the slope immediately below it from further scour. Plate A-6 represents the assumed configuration of this bank section.

All stability analyses were performed for the after construction loading condition using both wedge and circular arc methods of analysis with applicable WES computer programs SSWT28 and SSA003. Since the banks are composed of predominantly clay materials, the sudden drawdown and partial pool loading conditions were not considered applicable.

f. Recommendations. Based on available data and the results of the stability analyses, the following is the recommended plan for bank stabilization:

The existing bank should be graded to provide a slope no steeper than 1V on 2.5 H from top bank down to elevation 175.0. Slope protection should be provided for this area. A trench should be excavated at the base of the slope between the bluff material and the talus material for placement of a stone fill along the toe of the bank section. The trench should be excavated from elevation 175.0 down at least to the top of sand (El. 158±). Excavation side slopes should not be steeper than 1V on 1.5 H on the bluff side of the trench and should not be steeper than 1V on 2.75 H on the talus side. Provisions should be made so that the excavation of the trench on the bluff side is made through the undisturbed bluff materials. The trench should have a minimum bottom width of six feet. A stone fill should be placed in the trench from elevation 158 to elevation 175. This will require a stone quantity of approximately 43 tons per linear foot of bank section. The recommended plan of stabilization is illustrated on Plate A-7.

DISPOSITION FORM

For use of this form, see AR 340-15; the proponent agency is The Adjutant General's Office.

REFERENCE OR OFFICE SYMBOL	SUBJECT
LMMED-F	Memo for Record Bank Failure, Des Arc, Ark.

TO	FROM	DATE	CMT 1
THRU: LMMED LMMED-D TO: LMMED-DR	LMMED-F	21 Dec 76 Keithley/mlo/3381	

1. On 2 Dec 1976 J. E. Keithley of this office along with Mssrs. Steve Smith and James Lowery, River Stabilization Sec., inspected bank caving on the White River at Des Arc, Arkansas. The bank section inspected had been caving for some time and remedial action had been proposed under the Streambank Erosion Control Evaluation and Demonstration Act of 1974 (Section 32). The purpose of this inspection was then to gather evaluations and opinions from this office as to the applicability of remedial measures allowable under this act to the caving banks.

2. The following observations were made at the site: From a point just downstream of the old highway bridge location and extending for a distance of approximately 1200 ft. upstream, active bank failure was in progress. The most active portion of the failure was confined to a reach just 500 ft. upstream of the old bridge site. The failure was typified by a vertical bluff approximately 10 to 15 ft. in height with a talus or berm of failed material at the toe of the bluff. The talus extended from the face of the bluff to the waters edge, a distance of approximately 100 to 150 ft., on a very flat slope. The face of the bluffs were observed to be composed of approximately eight ft. of gray to gray and brown very stiff silty clay, possibly of loessial origin, underlain by a very stiff red clay of apparent high plasticity further underlain by a stiff red silt interbedded with thin lenses of stiff red clay. The material exposed in the bluffs appeared to be heavily overconsolidated. The stratum of red clay in particular appeared to be heavily overconsolidated. This clay exhibited a very complex network of joints, fractures and fissures, in both the horizontal and vertical directions and was laminated in the horizontal direction. Many of the joints were twisted and distorted, indicating a history of high stress. The faces of the joints and fractures were coated with a very thin layer of tan, black, red, grey or yellow silt or clay. The clay coatings appeared to be very highly plastic and some appeared to be either organic or contain high percentages of montmorillonite. The material on the face of the bluff was very dry and would crumble to cubes of approximately 1/4 inch by hand pressure. The talus was composed of intact blocks of the bluff material that appeared to be "floating" in a mass of very soft material. In addition to the previously described bluff materials, the blocks also included a stiff grey clay which also exhibited laminations, joints and fissures but not to the magnitude exhibited by the red clay. The soft material appeared to be composed of the remolded and softened material from the bluff. The talus is at present active, flowing toward the river. At the time of inspection the talus surface was covered with a heavy frost. Slip surfaces were observed within the talus, near the waters edge. The upper part of the slip surfaces were observed to be covered with ice

crystals while the lower six to eight inches of the surfaces were covered with a thin sheen of water which was free of ice crystals, indicating that movement had occurred just prior to the inspection. Several dead trees, leaning at various angles, were observed out in the river, indicating that the talus extended for some distance past the waters edge. Sketches of the failure section are included. (Inclosures 1 and 2). The location of the failure is shown on Inclosure 3 and 4.

3. A review of correspondence files indicates that bank caving has long been a problem at this location. The present series of failures appears to have begun following the 1973 highwater and was hastened by the 1975 highwater. Early in 1975 a boring was taken near the site of the present bank failures and remedial action was recommended by this office. However, funding was not available and no work was performed.In 1953, following another highwater season, representatives of the District visited the site to inspect bank caving. The caving at this time was indicated to be centered approximately 2000 ft. above the site of the old highway bridge and endangered the railroad located near top bank. Remedial action was recommended but there is no record of any action being taken. Although not specifically stated, the correspondence relating to the 1953 incident implies that bank caving occurred following the 1937 high water. A review of available topographical data and aerial photos indicates that in the very recent geological past there may have occurred a slide just downstream of the new highway bridge of a magnitude sufficient to alter the course of the river. All of the above indicate a general instability of the bluffs and all of the slides of record appeared to follow a period of significant high water.

4. The town of Des Arc is located on the bluffs along the western edge of the Mississippi River Embayment. The bluffs represent the limit of the meander pattern of the river. The bluffs in this area are composed of sands and silts and are backswamp deposits. Material of this type is poorly consolidated and is subject to rapid erosion. The river is located on point bar deposits which consists of fine grained surface material predominantly silts and clays overlying sand and gravel which extends to the top of the Tertiary. The point bar deposits are crossed at various places by abandoned channels of the White River or other streams. As mentioned, a boring was taken at the site in early 1975. The log of the boring is shown on Inclosure 5. In general, the boring log exhibits the soil types as those exposed on

the bluff face. The red clays and silts that are exposed on the bluff face are underlain by a stiff brown to gray clay to a depth of 47 ft. which is underlain by a gray fine sand. The samples from the boring exhibited some slickensides and jointing but not to the degree as the material exposed on the face of the bluff.

5. Based on the available data and the history of bank failures at the site an explanation of the possible mechanism of failure can be made; the bluffs on which the town of Des Arc is located represent an erosional remnant left by the meandering of the White River. The town is situated on a more or less peninsula that projects out into the flood plain of the White River. This peninsula is located on an outside bend of the White River and is subject to potential scouring velocities. Since history indicates that bank failures occur after a significant high water it must be concluded that this event is a prime contributor to bank failure. It is felt however that the high river stages and accompanying high velocities serve to trigger the slides rather than act as the primary cause. Consider the bluffs during periods of relatively low water, the bluff face which once experienced high lateral stresses now experiences a reduction in lateral stress. For that portion of the bluff above the talus berm the stress reduction is to zero at the face and increases horizontally into the bluff. That portion of the bluff below the talus is confined to some degree by the talus but the lateral stresses are still less than those previously experienced. As with the portion of the bluff above the talus the lateral stresses should increase at points in a horizontal direction into the bluff. The absence or reduction of lateral stresses and the drying action on the bluff face allows the clays to expand and the many joints and fractures will open up. The clay, being overconsolidated, has a natural affinity for water. Water, either from precipitation, percolation through the loess overburden, or artesian water in the lower sands, can enter the opened joints, soften the clays at the faces of the joints and cause an extensive reduction in shear strength. This shear strength reduction is furthered by the presence of the highly plastic clays on the joint faces. The lateral dimension of the loss in shear strength is variable but should extend several feet into the face of the bluff. (5 to 10 ft. or more). The talus, which consists primarily of the softened and remolded material from the bluffs, is barely stable under its own weight but offers enough lateral support to the bluffs to prevent their collapse. The talus may exhibit movement to a small degree during this period but is generally stable. The

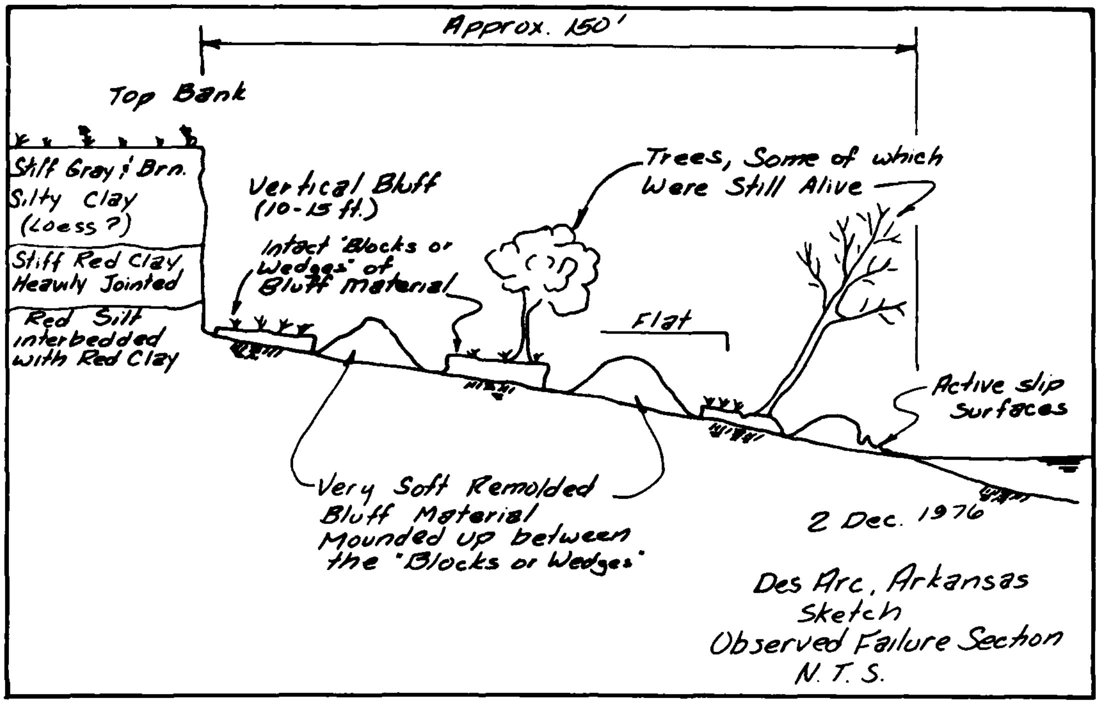
whole systems, bluff and talus, may then be considered to be in a state of somewhat precarious equilibrium. The occurrence of high stages on the White River can act to disrupt this state of equilibrium in two ways. The water can fully saturate the talus and further weaken its resistance to movement. The water can also enter the expanded joints in the bluff clays, further lessen their shear strength and on recession, generate hydraulic pressures within the soil mass. With sufficiently high velocities in the river, the toe of the talus could be eroded away. This, along with the softened condition, could trigger movements in the talus. Movement of the talus could generate shearing strains in the underlying sands, causing excess pore water pressures to develop and a liquefaction failure at the toe of the talus, increasing the rate of movement. With movement of the talus, lateral pressures are also reduced at the face of the bluff, allowing collapse of the bluff. This occurrence adds mass to the talus, causes more movement, causing more liquefaction failures at the toe and further collapse of the bluff. The mechanism, once initiated is essentially self perpetuating and should continue until the slope of the entire bank is flat enough to attain stability. A sketch illustrating the mechanism of failure is shown as Inclosure 6.

6. As stated, the bank caving should continue until an equilibrium condition is reached. The limits to which caving may be expected is shown on Inclosure 4. It should be noted that the location of the limits are nothing more than a guess on our part. On reaching a slope of equilibrium it cannot be assumed that the banks are permanently stabilized. With the recurrence of erosion at the toe of the talus, the failure mechanism may be again generated. This is not an unlikely event but rather an expected natural occurrence.

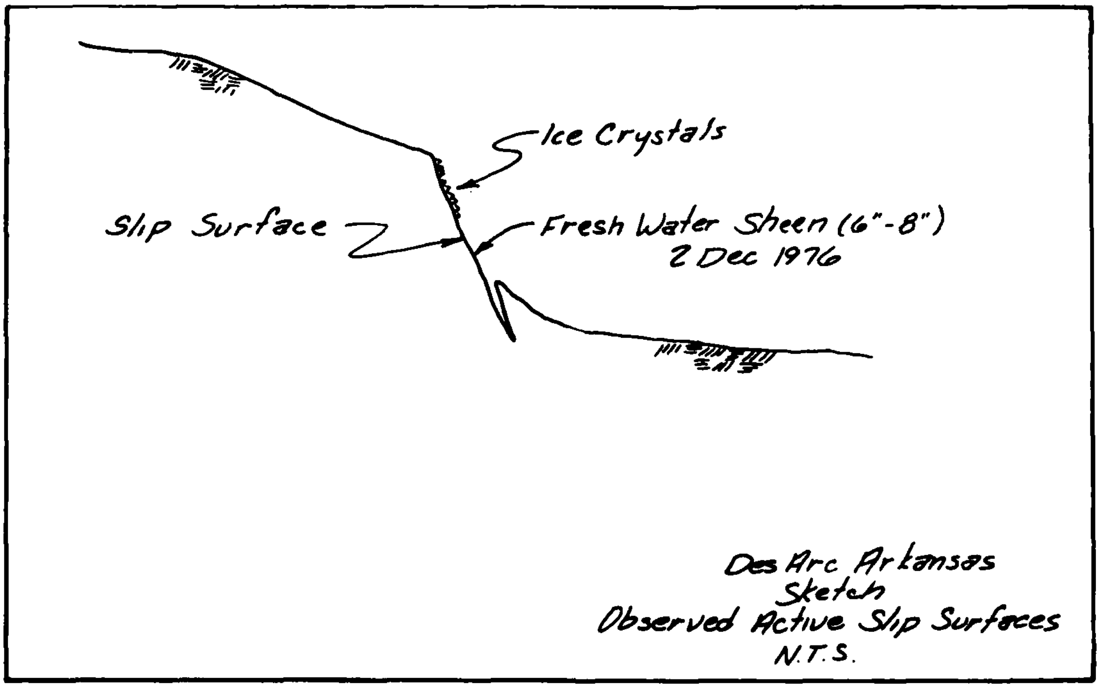
7. There are several remedial measures which may be undertaken to stabilize the bank section. Those which are thought to meet with the most success will necessitate the placement of large quantities of stone. (A massive rock toe, bank grading and paving or a system of stone dikes and bank grading and some paving). It is not felt that the work allowed under the Streambank Erosion Control Evaluation and Demonstration Act of 1974 will be adequate to stabilize the banks. If the failure mechanism is similiar to that described the placement of erosion protection on the bank surface will not be adequate to stabilize the banks. It should be carefully noted that the described failure mechanism is merely a conjecture on our part. Before any course of remedial action is taken, it is recommended that

the bank failures be carefully studied to clearly define the cause and method of failure. The apparent uniqueness of the mechanism of the bank failures may be such that the problem merits special study to clearly define the events and parameters governing failure. A somewhat abbreviated review of available literature indicates that many publications speak of failures in overconsolidated clays and progressive liquefaction failures in sands. However, none were found that describe a problem where both occur at the same site and where one is complimented by the other. It is recommended that consideration be given to performance of a special study to define the mechanism of failure at this site and the factors that influence failure. The study should include recommendations and design of remedial measures to be undertaken. Should funding be made available for construction of the remedial measures, they should be monitored after construction to determine their adequacy.

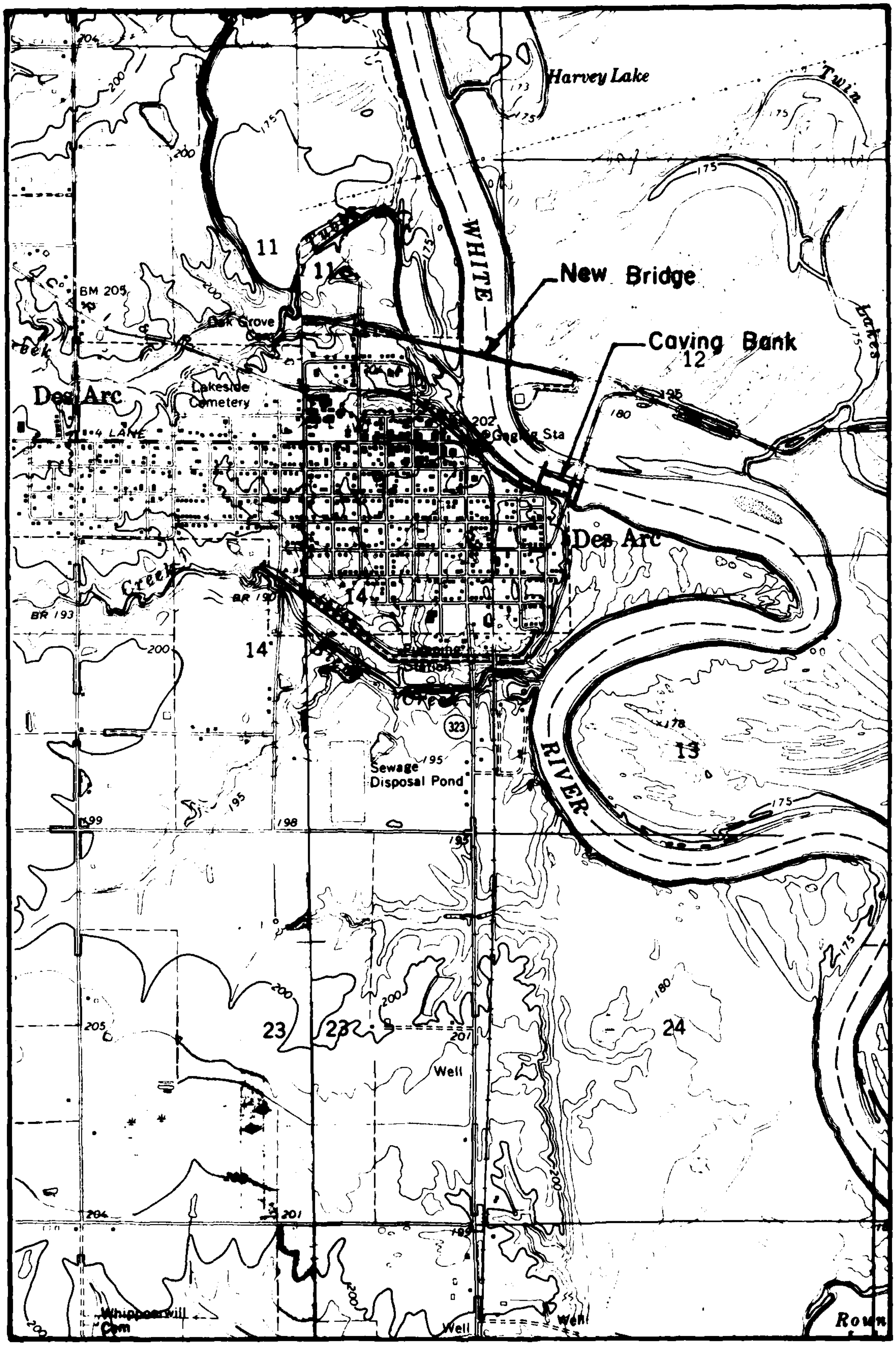
K. R. Akers
K. R. AKERS
C/F&M Br.



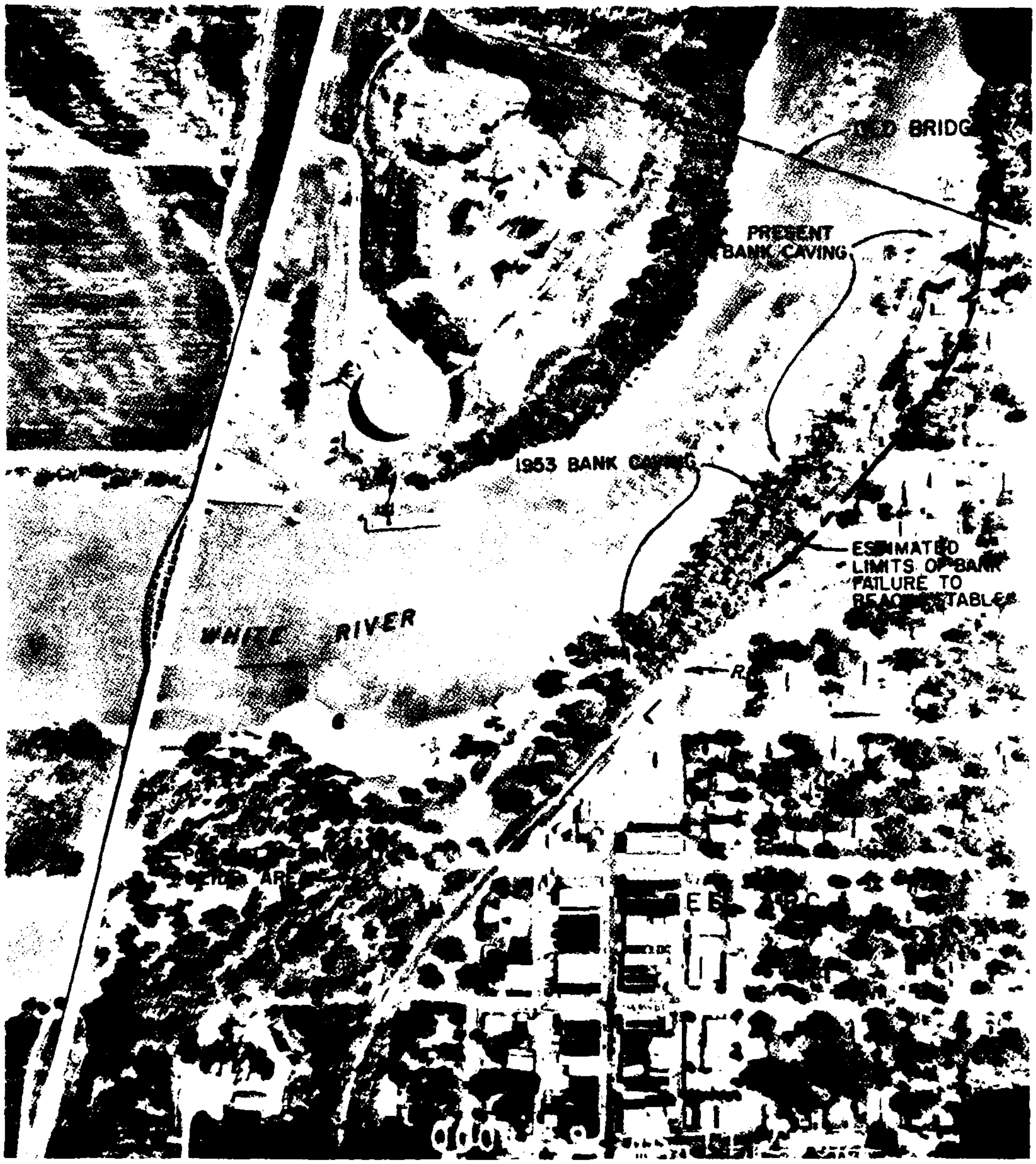
INCLOSURE 1



INCLOSURE 2

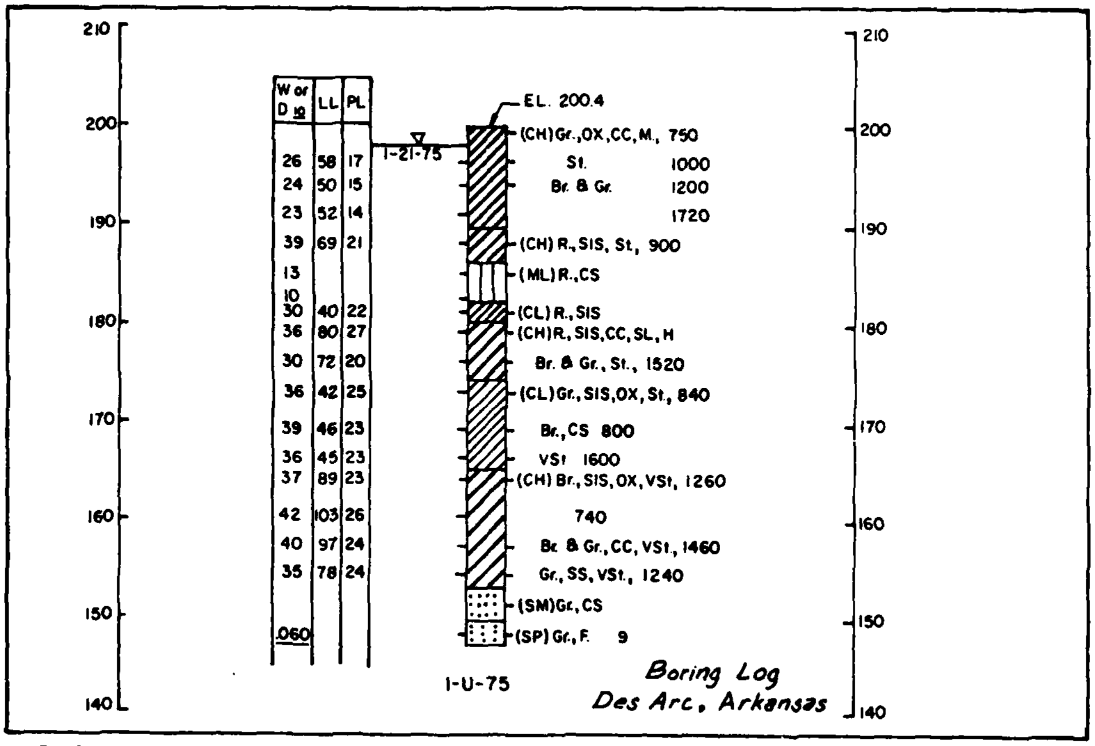

Harvey Lake
Twin
New Bridge
Caving Bank
WHITE
RIVER
Des Arc
Des Arc
Lakeside Cemetery
Oak Grove Cem
BM 205
Creek
Sewage Disposal Pond
Well
Well
Well
Whippoorwill Cem
Lakes
11
11
12
13
14
14
23
23
24
323
175
180
195
200
204
205
199
198
201
202

INCLOSURE 3

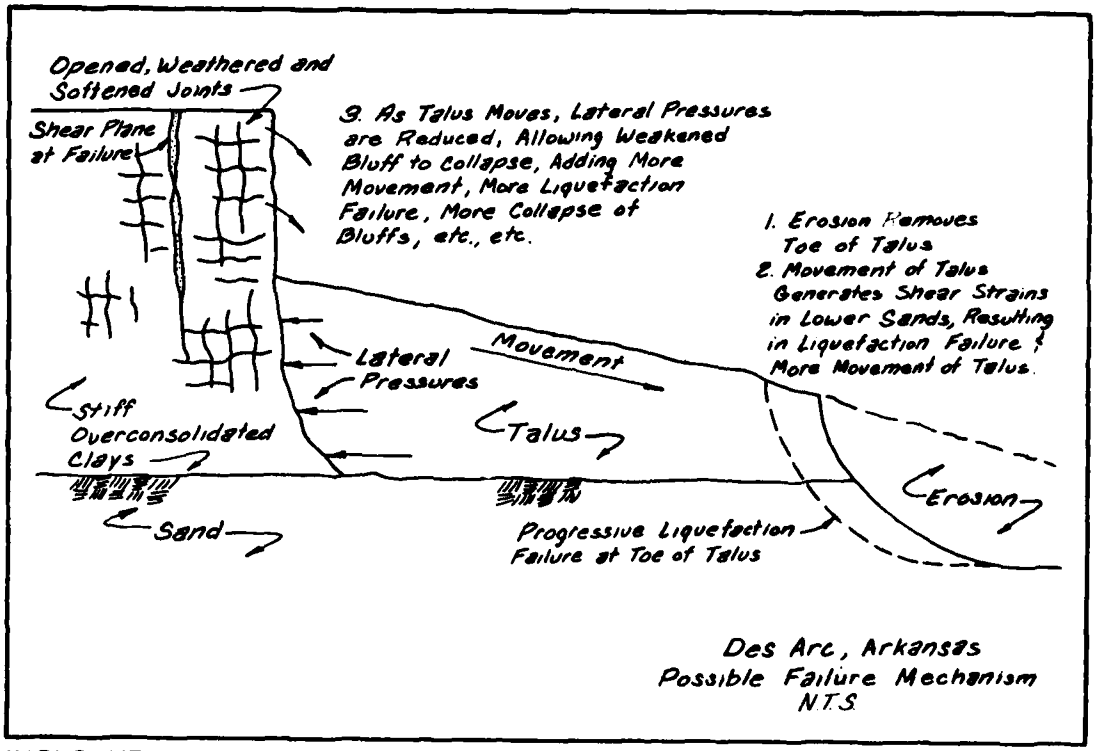


DES ARC, ARKANSAS

INCLOSURE 4



INCLOSURE 5



INCLOSURE 6

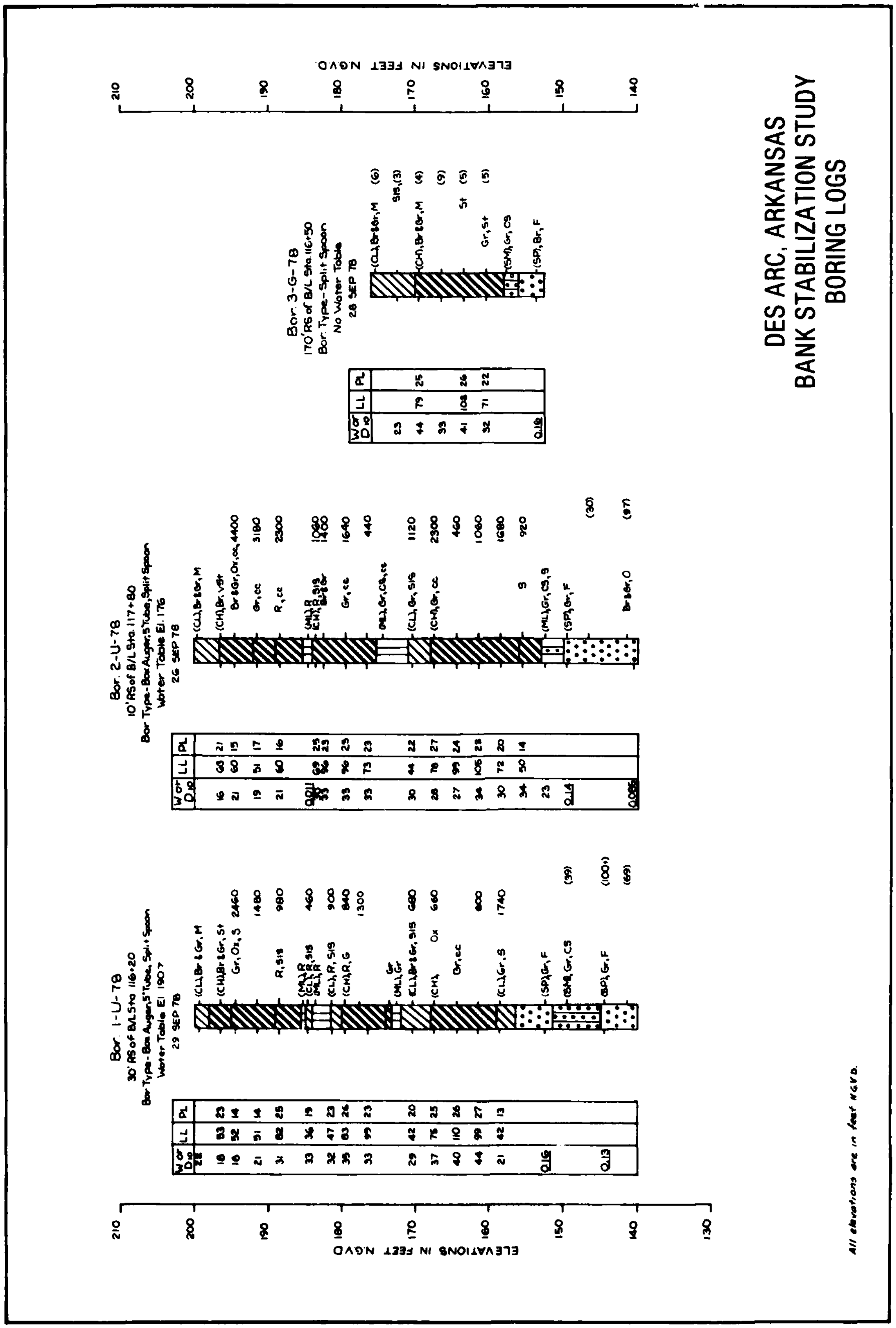
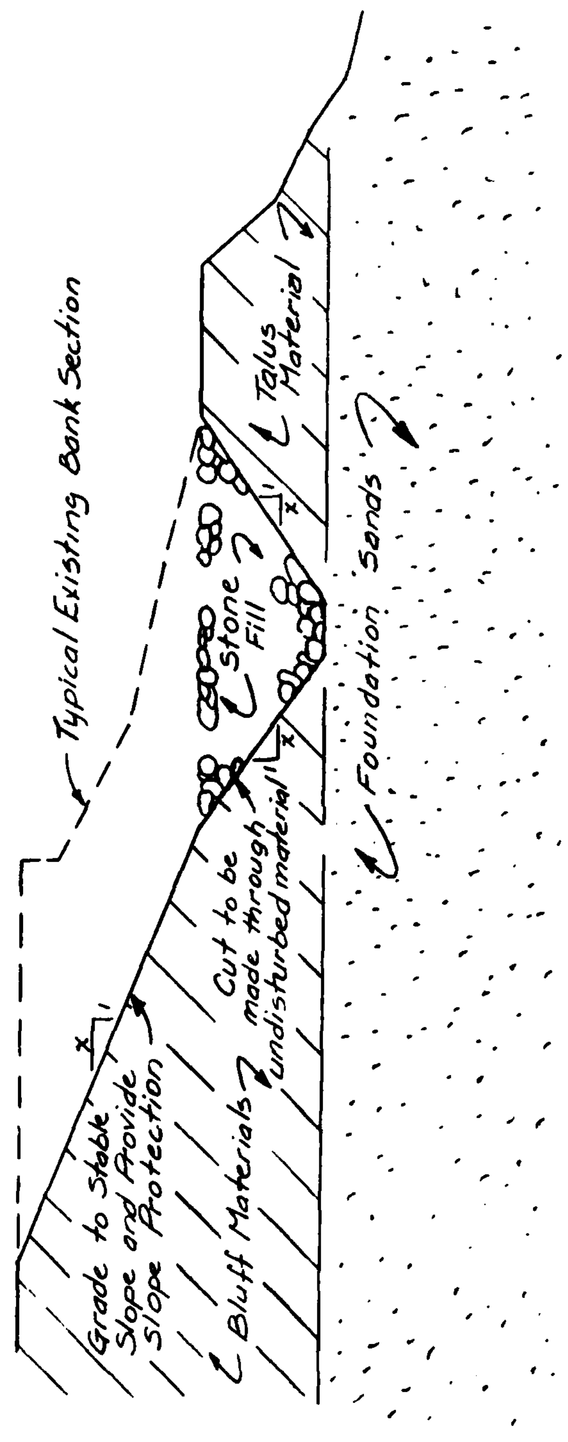
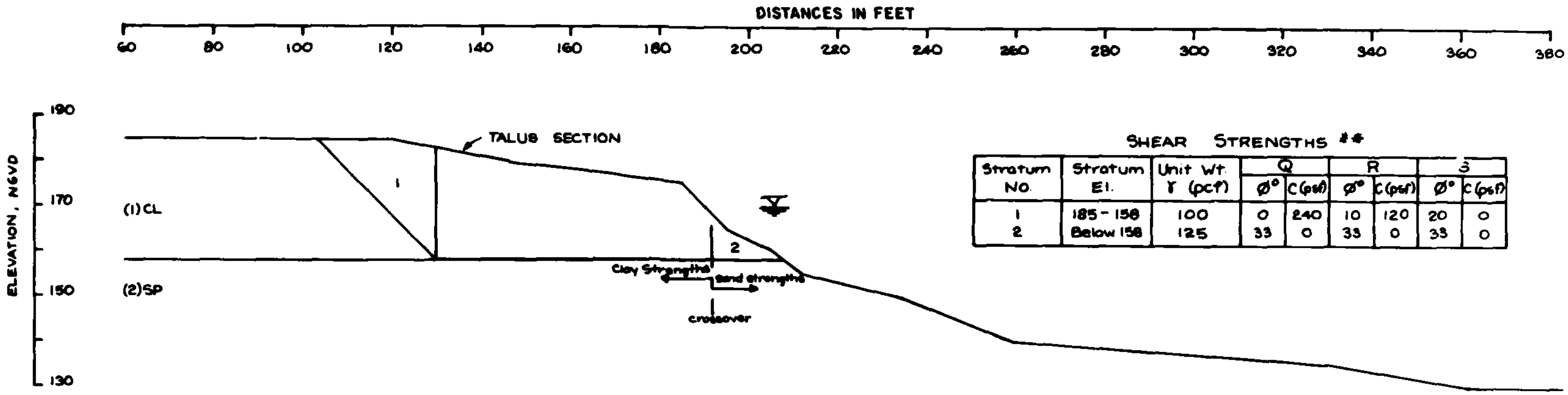

DES ARC, ARKANSAS
BANK STABILIZATION STUDY
BORING LOGS
Bor. 1-U-78
30' RS of B/L Sta 116+20
Bor Type - Box Auger, 5" Tube, Split Spoon
Water Table El 190.7
29 SEP 78
Bor. 2-U-78
10' RS of B/L Sta. 117+80
Bor Type - Box Auger, 5" Tube, Split Spoon
Water Table El. 176
26 SEP 78
Bor. 3-G-78
170' RS of B/L Sta. 116+50
Bor. Type - Split Spoon
No Water Table
28 SEP 78
W or D10
LL
PL
ELEVATIONS IN FEET NGVD
All elevations are in feet NGVD.

PLATE A1



DES ARC, ARKANSAS
BANK STABILIZATION STUDY
PROPOSED METHOD OF REPAIR

PLATE A2



SHEAR STRENGTHS ##

Stratum No.	Stratum El.	Unit Wt. γ (pcf)	Q $\phi°$	Q C (psf)	R $\phi°$	R C (psf)	S $\phi°$	S C (psf)
1	185 - 158	100	0	240	10	120	20	0
2	Below 158	125	33	0	33	0	33	0

COMPUTER RESULTS - ARC METHOD

Arc Coordinates X (ft)	Y (elev)	Radius (ft)	Factors of Safety A.C.
200	241	85	1.27
180	282	124	1.10
170	228	70	1.15
240	230	92	1.66
175	282	124	1.11
180	285	127	1.10

COMPUTER RESULTS - WEDGE METHOD

Neutral Block Coordinates X_L (ft)	X_R (ft)	Failure Plane Elev.	Factors of Safety A.C.
185	207	158	1.08
140	207	158	0.99
185	233	150	1.43
185	259	140	1.75
130	208	158	0.96 *
145	208	158	1.02

* case presented

MANUAL CALCULATIONS

Element	Weight (kips)
1	30.96
2	2.76

$R_a = 2 W \tan \phi + 2 C H \tan(45° - \phi/2) = 2(240)(27) = \underline{12.96^K}$

$R_b = W \tan \phi + CL = 2.76 \tan 33° + .24(62) = \underline{16.67^K}$

$R_p = 0$

$D_a = W \tan(45° + \phi/2) = \underline{30.96^K}$

$D_p = 0$

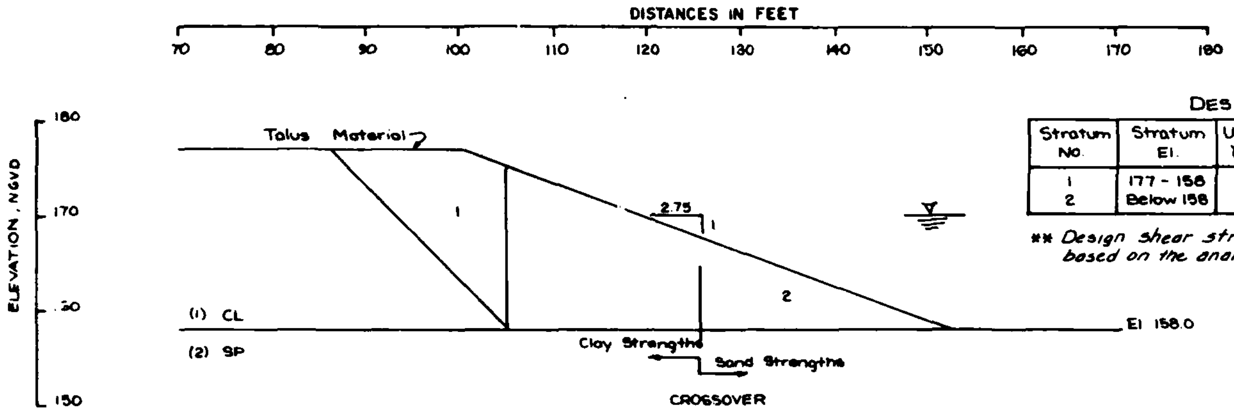
$F.S. = \dfrac{R_a + R_b + R_p}{D_a - D_p} = \dfrac{29.63}{30.96} = \underline{\underline{0.96}}$

All elevations are in feet NGVD

Note: This stability analysis was performed in an effort to determine the cohesive strength of the talus material. Since the talus was actively failing at the time of inspection, it was assumed that the factor of safety for the section was probably somewhere in the range between 0.95 and 1.00. From the results of several trials it was found that a cohesive value of 240 psf for the talus material yielded a factor of safety in this range as indicated by the above analysis. A cohesive value of 240 psf was then used in later analyses to determine the proper excavation slopes for the talus section (See Plate 5)

DES ARC, ARKANSAS
STABILIZATION STUDY
STABILITY ANALYSIS

PLATE A3



DESIGN SHEAR STRENGTHS **

Stratum No.	Stratum El.	Unit Wt. γ (pcf)	Q $\phi°$	Q c(psf)	R $\phi°$	R c(psf)	S $\phi°$	S c(psf)
1	177 - 158	100	0	240	10	120	20	0
2	Below 158	125	33	0	33	0	33	0

** *Design shear strengths for the talus material are based on the analysis presented on Plate 4.*

COMPUTER RESULTS - ARC METHOD

Arc Coordinates X (ft)	Y (elev)	Radius (ft)	Factors of Safety A.C.
118	188	30	1.63
119	208	50	1.60
114	218	60	1.67
137	268	113	1.76
137	238	83	1.65

COMPUTER RESULTS - WEDGE METHOD

Neutral Block Coordinates X_L (ft)	X_R (ft)	Failure Plane Elev.	Factors of Safety A.C.
105	152.5	158	1.32 *
110	152.5	158	1.37
103	152.5	158	1.33
107	152.5	158	1.33
107	150.0	150	1.76

* Case presented

MANUAL CALCULATIONS

Element	Weight (kips)
1	13.11
2	4.99

$R_a = 2W \tan\phi + 2CH \tan(45° - \phi/2) = 2(.240)(19) = \underline{9.12^K}$

$R_b = W \tan\phi + CL = 4.99 \tan 33° + .240\,(20.5) = \underline{8.16^K}$

$R_p = 0$

$D_a = W \tan(45 + \phi/2) = \underline{13.11^K}$

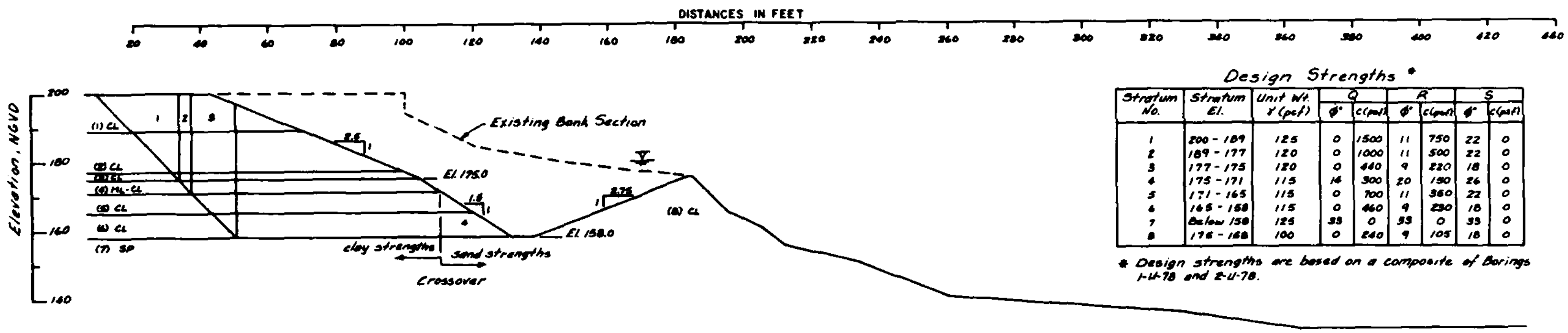
$D_p = 0$

$F.S. = \dfrac{R_a + R_b + R_p}{D_a - D_p} = \dfrac{17.28}{13.11} = \underline{1.32}$

All elevations are in feet NGVD

DES ARC, ARKANSAS
STABILIZATION STUDY
TALUS EXCAVATION SLOPE
STABILITY ANALYSIS

PLATE A4



Design Strengths *

Stratum No.	Stratum El.	Unit Wt. γ (pcf)	Q		R		S	
			φ°	c(psf)	φ°	c(psf)	φ°	c(psf)
1	200 - 189	125	0	1500	11	750	22	0
2	189 - 177	120	0	1000	11	500	22	0
3	177 - 175	120	0	440	9	220	18	0
4	175 - 171	115	14	300	20	150	26	0
5	171 - 165	115	0	700	11	360	22	0
6	165 - 158	115	0	460	9	230	18	0
7	Below 158	125	33	0	33	0	33	0
8	176 - 168	100	0	240	9	105	18	0

* Design strengths are based on a composite of Borings 1-U-78 and 2-U-78.

Computer Results - Arc Method

Arc Coordinates			Factors of Safety
X (ft.)	Y (elev.)	Radius (ft.)	A.C.
71.67	221	44	3.59
79.64	251	56	2.65
78.98	221	50	2.19
77.67	211	46	2.09
92.40	221	68	1.46
86.99	261	105	1.92
81.27	251	66	1.91
109.51	281	126	1.41

Computer Results - Wedge Method

Neutral Block Coordinates			Factors of Safety
X_L (ft.)	X_R (ft.)	Failure Plane Elev.	A.C.
55	100	177	2.68
55	105	175	2.32
60	111	171	2.19
65	120	165	1.78
50	130	158	1.25 **
45	130	158	1.26
53	130	158	1.26
50	130	165	1.76

** case presented

Manual Calculations

Element	Weight (kips)
1	37.79
2	8.91
3	41.61
4	7.16

$R_a = 2W \tan\phi + 2CH \tan(45° - \phi/2) = 2[1.5(11) + 1(12) + .44(2) + 8.91 \tan 16° + .3(4) \tan 38° + .7(6) + .46(7)] = \underline{79.92}^{k}$

$R_b = W \tan\phi + CL = 7.16 \tan 33° + .46(60.79) = \underline{32.61}^{k}$

$R_p = 0$

$D_a = W \tan(45° + \phi/2) = 37.79 + 8.91 \tan 52° + 41.61 = \underline{90.80}^{k}$

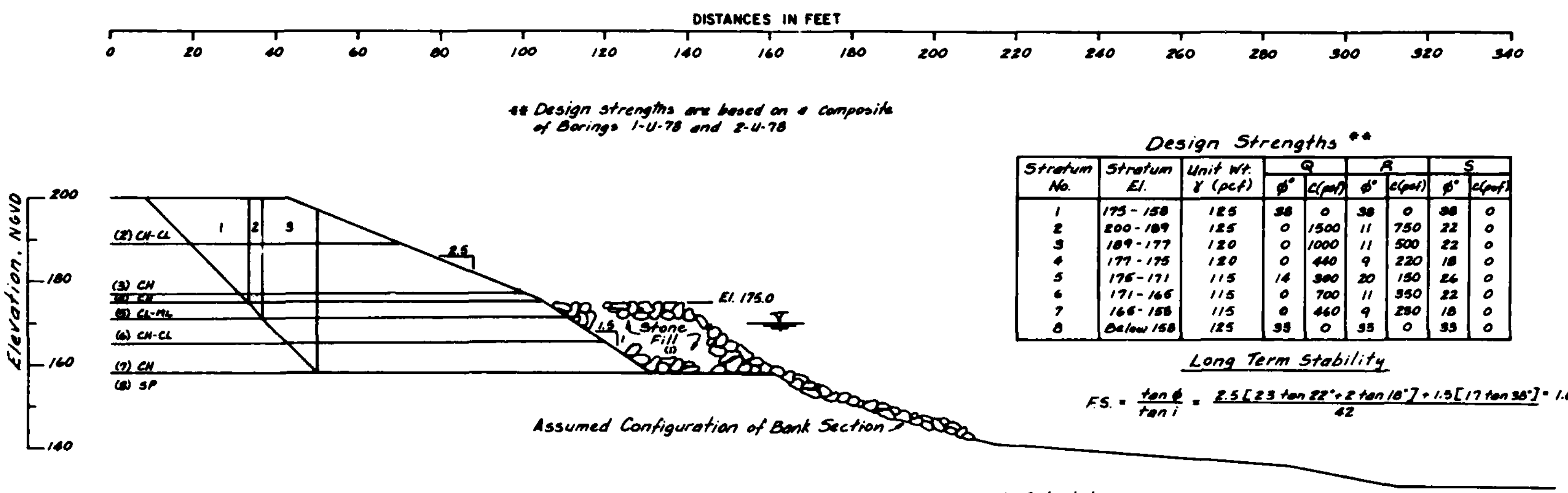
$D_p = 0$

$F.S. = \dfrac{R_a + R_b + R_p}{D_a - D_p} = \dfrac{112.53}{90.80} = \underline{1.24}$

All elevations are in feet N.G.V.D.

DES ARC, ARKANSAS
BANK STABILITY STUDY
EXCAVATION SLOPE
STABILITY ANALYSIS

PLATE A5



** Design strengths are based on a composite of Borings 1-U-78 and 2-U-78

Design Strengths **

Stratum No.	Stratum El.	Unit Wt. γ (pcf)	Q		R		S	
			$\phi°$	c(psf)	$\phi°$	c(psf)	$\phi°$	c(psf)
1	175 - 158	125	38	0	38	0	38	0
2	200 - 189	125	0	1500	11	750	22	0
3	189 - 177	120	0	1000	11	500	22	0
4	177 - 175	120	0	440	9	220	18	0
5	176 - 171	115	14	380	20	150	26	0
6	171 - 166	115	0	700	11	350	22	0
7	166 - 158	115	0	440	9	250	18	0
8	Below 158	125	33	0	33	0	33	0

Long Term Stability

$$F.S. = \frac{\tan\phi}{\tan i} = \frac{2.5[23\tan 22° + 2\tan 18°] + 1.5[17\tan 38°]}{42} = 1.07$$

Computer Results - Arc Method

Arc Coordinates			Factors of Safety
X (ft)	y (elev)	Radius (ft)	A.C.
69	241	64	3.55
72	241	66	2.56
77	241	70	2.24
72	221	56	1.86
79	221	63	1.40
76	231	73	1.38
71	231	73	1.59
91	251	96	1.93
144	301	160	1.80
198	236	95	1.58

Computer Results - Wedge Method

Neutral Block Coordinates			Factors of Safety
X_L (ft)	X_R (ft)	Failure Plane Elev.	A.C.
50	100	177	2.52
52	105	175	2.23
52	111	171	2.20
52	120	158	1.54
52	162	158	1.35
55	160	156	2.02
50	162	158	1.34 *
45	162	158	1.35
50	155	168	1.35
55	213	141	2.02

* case presented

Manual Calculations

Element	Weight (kips)
1	38.57
2	10.28
3	49.61

$R_a = 2W\tan\phi + 2CH\tan(45° - \phi/2) = 2[1.5(11) + 1(12) + .44(2) + 10.28\tan 14° + .3(4)\tan 38° + .7(6) + .46(7)] = \underline{80.60^k}$

$R_b = W\tan\phi + CL = 23.58\tan 33° + .46(81) = \underline{52.54^k}$

$R_p = 0$

$D_a = W\tan(45° + \phi/2) = 38.57 + 10.28\tan 52° + 49.61 = \underline{101.34^k}$

$D_p = 0$

$$F.S. = \frac{R_a + R_b + R_p}{D_a - D_p} = \frac{133.14}{101.34} = \underline{1.31}$$

All elevations are in feet NGVD

DES ARC, ARKANSAS
BANK STABILIZATION STUDY
STABILITY ANALYSIS

PLATE A6

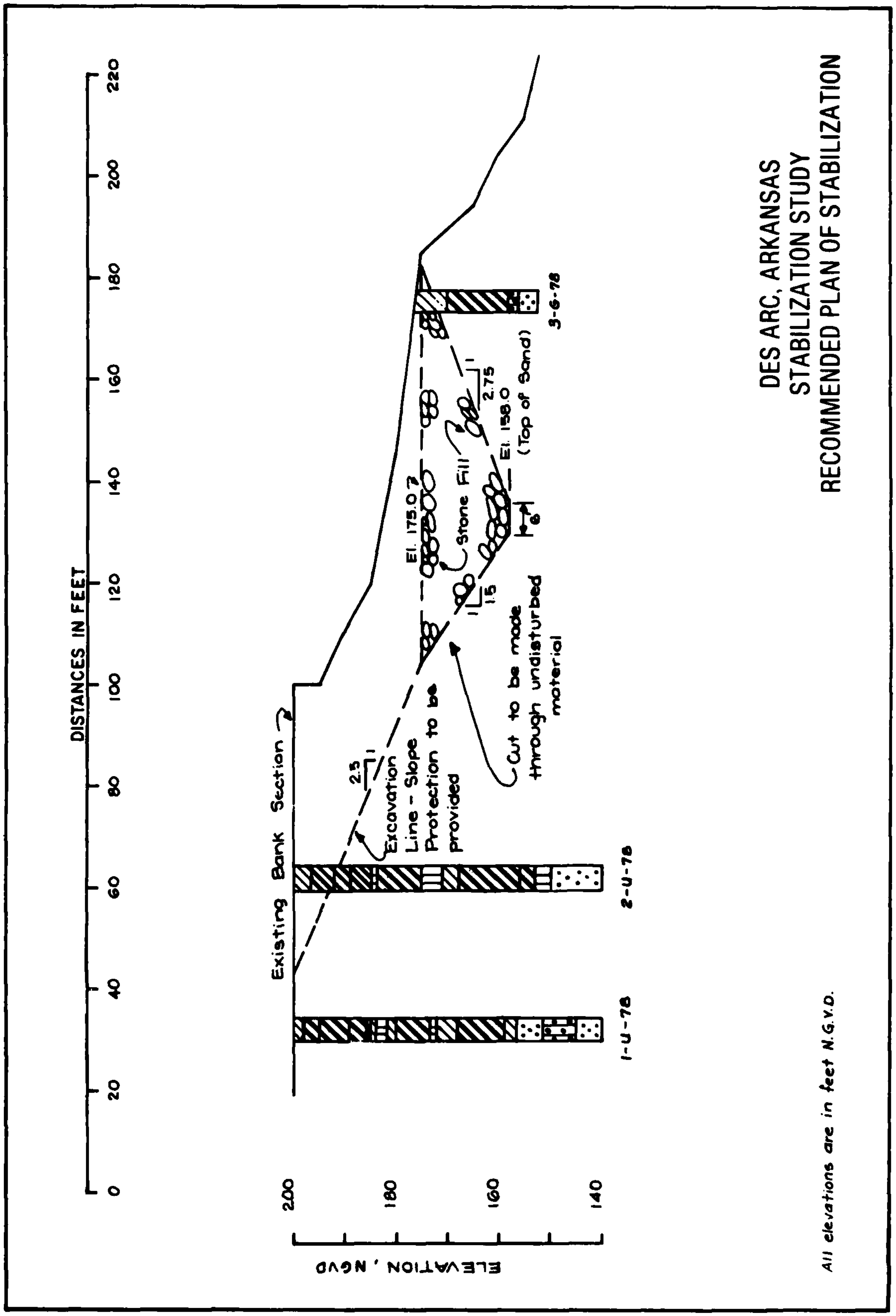

DES ARC, ARKANSAS
STABILIZATION STUDY
RECOMMENDED PLAN OF STABILIZATION
DISTANCES IN FEET
ELEVATION, NGVD
Existing Bank Section
Excavation Line - Slope Protection to be provided
Cut to be made through undisturbed material
El. 175.0
Stone Fill
El. 158.0 (Top of Sand)
2.5
1.5
2.75
6'
1-U-78
2-U-78
3-6-78
All elevations are in feet N.G.V.D.

PLATE A7

APPENDIX B
REPORT OF MONITORING PROGRAM
WHITE RIVER AT DES ARC, ARKANSAS

a. General. Following completion of construction of the Streambank Erosion Control Project on the White River at Des Arc, Arkansas, a monitoring program was initiated to evaluate the effectiveness of the project. The purpose of this report is to report the significant findings of the program to date and preliminary conclusions that may be drawn.

b. Instrumentation. During the month of August 1980, seven slope indicator tubes and three piezometers were installed at the project site. Locations of these items are shown on Plate 5. Following installation, the slope indicator tubes were sounded using a Digitilt Model 50301 borehole inclinometer in order to establish an initial base reading. Subsequent readings were all made using the same device.

c. Monitoring Interval. The slope indicators and peizometers were monitored weekly following installation until approximately February 1981 when the monitoring interval was changed to a biweekly schedule. Along with the monitoring of instrumentation, visual monitoring of the project was also performed with changes in surface characteristics noted.

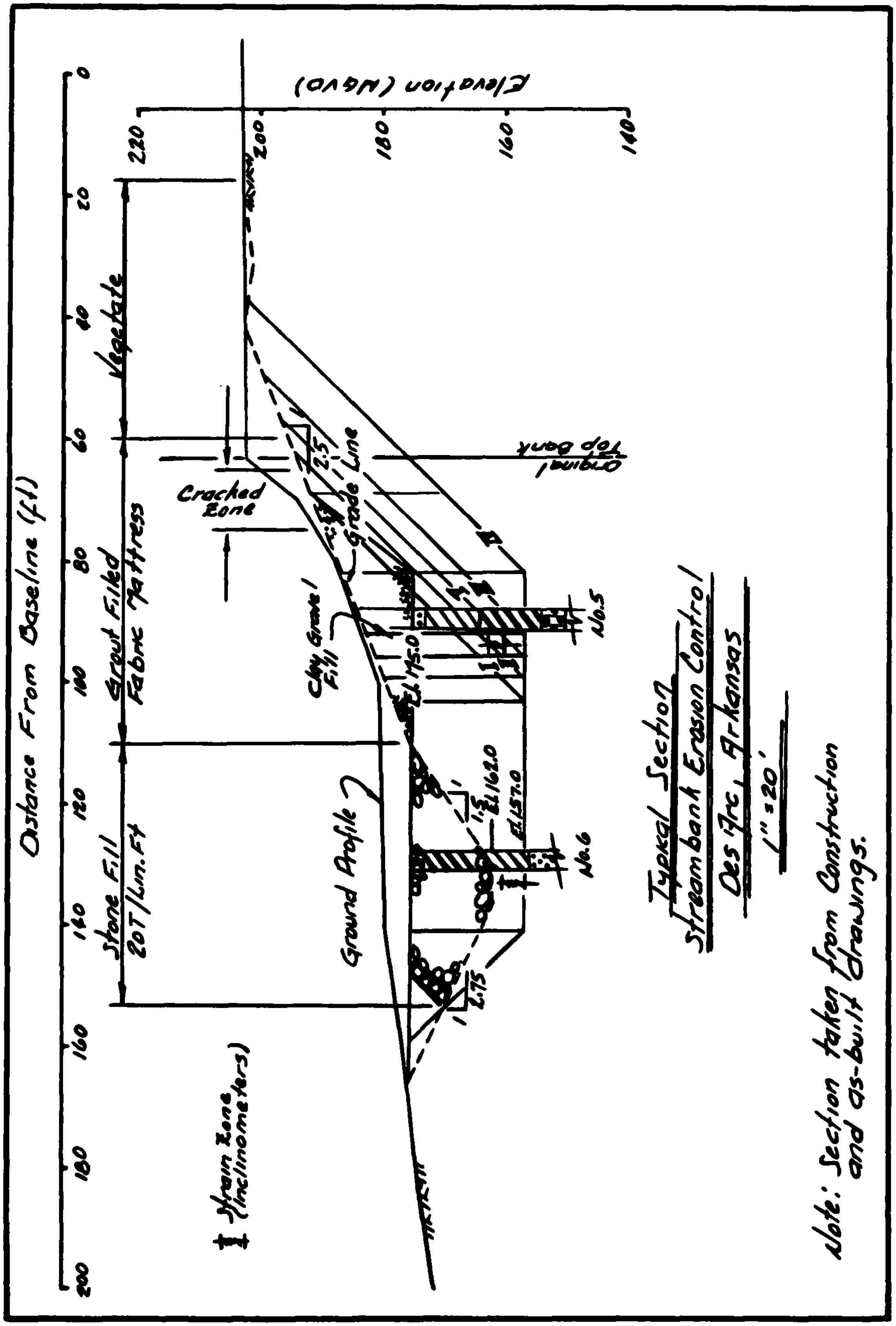
d. Discussion of Data Collected. From the time of installation to June 1981, slope indicator readings indicated little movement of the riverbank. Although there was a general indication of movement toward the river and downstream, the movements measured were slight. However, slope indicator data recorded in June 1981 indicated significant movement had occurred within the riverbank, particularly in the area protected by the grouted mattress (inclinometer tubes 5, 6, and 7). The plane of movement was indicated to occur between approximate elevations 157 and 165 NGVD and exhibited the same pattern of movement toward the river and downstream. Plots of movement versus elevation are shown on Plates B-2 through B-5. For the sake of clarity, only selected data is shown. During the same interval, June 1981, the piezometric surface, which had been fairly consistent previously, showed a significant rise, especially Piezometer No. 2 which indicated a rise of approximately 10 feet to within approximately seven feet of ground surface.

e. Discussion of Reported Visual Observations. During construction of the project, a slope failure occurred along the landside of the trench excavation. On completion of the project, a crack opened along the grouted mattress portion of the project along the juncture of the original slope and the clay gravel fill which was used to replace the failed material. The crack appeared to extend through the tire mattress and the soil cement area. A review of survey information indicates that the slope failure and the crack follow the orginal top bank that existed prior to construction. As with the slope indicators, the crack showed little movement until June 1981 when significant movement was observed. The river also experienced a slight rise at this time, as a drift line was observed approximately one to two feet above the toe of the reveted slopes.

f. Evaluation of Data. While no evaluation as to the effectiveness of the erosion prevention measures may be advanced at this time since no significant water has been against the protected slopes, a preliminary evaluation of the stability of the riverbank has been performed. The evaluation was performed using the wedge method of analysis and by placing the failure surface through the planes of movement indicated by the slope indicator data. The assumptions made, calculations and results of the evaluation are shown in attached sheets 1 through 9.

g. Conclusions. Based on the instrumentation data, visual observations and stability evaluation, it is concluded that localized failure has occurred within the river bank in the area represented by inclinometer tubes 4, 5, and 6 and is in the early stages of development in the area represented by tubes 1, 2, and 3 (see Plates Nos. B-2 - B-4). The failure surface appears to be confined to the failed bluff material that had formed the bench at the base of the bluffs and passes beneath the rock filled toe trench. Based on the behavioral history of this material, it must also be concluded that these localized failures will be progressive in nature and will continue until general overall failure of the bank has occurred. The progression of failure may occur at a relatively rapid rate but, if not occurring beforehand, may very well be expected following the recession of the next highwater against the banks. The rock filled toe trench lacks adequate depth and mass to be effective in halting failure and may be expected to be carried out with the failing embankment. Following the occurrence of the expected failure it may be anticipated that a mechanism of failure similar to that described in the Memo for Record attached to the Bank Stability Study (Appendix A) will again develop within the bluffs. It is further concluded that the developing failure may be attributed to the failure to construct the rock filled toe trench to adequate depth and section and to the failure to set the grading limits back sufficiently to assure all of the graded bank landward of the toe trench would be in undisturbed material.

h. Recommendations. Since the program under which the project was constructed is experimental in nature, it is recommended that the monitoring of the instrumentation be contimed with significant occurrences being evaluated and documented. Due to the experimental nature of the program, funds should be made available to rectify problem areas which develop or rehabilitate the project in the event of a general failure of the soil mass.


Distance From Baseline (ft)
Elevation (NGVD)
Vegetate
Grout Filled Fabric Mattress
Stone Fill 20T/Lin.Ft
Cracked Zone
Original Top Bank
Grade Line
Clay Gravel Fill
Ground Profile
El.195.0
El.162.0
El.157.0
No.5
No.6
Strain Zone (Inclinometers)
Typical Section
Streambank Erosion Control
Des Arc, Arkansas
1" = 20'
Note: Section taken from Construction and as-built drawings.

SHEET 1 OF 9

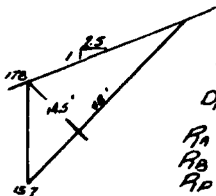
PROJECT	PAGE	COMPUTED BY	DATE
Streambank Erosion Control, Des Arc, Ark.	2 of 9	JEK	18 Aug 81
SUBJECT		**CHECKED BY**	**DATE**
Evaluation of Bank Stability - Inclinometer Data		WJS	19 Aug 81

See sketch, Page 1.

Assumptions,

Unit Weight of soil ranges from 100-110 pcf
Shear Strength of soil based on cohesion only and ranges from 200 psf to 240 psf (See original Soils report)
Failure is wedge type.
Hydrostatic Forces may act along existing cracks in slope.
Portions of upperbank possess sufficient strength to stand alone without contributing to active drive or resisting forces.

Analyze Failure Surface I



$Area = \frac{16.5 \times 48}{2} = 348 \square'$

Using $\gamma = 110$ pcf, $D_A = 38.28$ K

$D_P = \left(\frac{18 \times 18}{2}\right) \times 0.11 = 17.82$ K

$R_A = 2(21)(0.24) = 10.08$ K
$R_B = 38(0.24) = 9.12$ K
$R_P = 2(18)(0.24) = 8.68$ K
} c = 240 psf

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{10.08 + 9.12 + 8.68}{38.28 - 17.82} = \frac{27.88}{20.46} = 1.36$

Using c = 200 psf, F.S. = 1.13

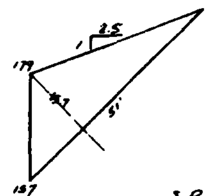
Using γ = 100 pcf, F.S. = 1.50

Using γ = 100 pcf & c = 200 psf, F.S. = 1.25

SHEET 2 OF 9

PROJECT	PAGE	COMPUTED BY	DATE
Streambank Erosion Control, Des Arc, Ark.	3 of 9	JEK	18 Aug 81
SUBJECT		**CHECKED BY**	**DATE**
Evaluation of Bank Stability - Inclinometer Data		WJS	19 Aug 81

Analyze Failure Surface II



$Area = \frac{15.7 \times 51}{2} = 400.35 \square'$

Using $\gamma = 110$ pcf, $D_A = 44.04$ K

$D_P = 17.82$ K
$R_A = 2(22)(0.24) = 10.56$ K
$R_B = 41(0.24) = 9.84$ K
$R_P = 8.68$ K
} c = 240 psf

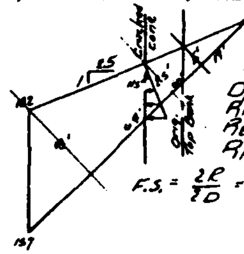
$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{10.56 + 9.84 + 8.68}{44.04 - 17.82} = \frac{29.08}{26.22} = 1.11$

Using c = 200 psf, F.S. = 0.925

Using γ = 100 pcf, F.S. = 1.22

Using γ = 100 pcf & c = 200 psf, F.S. = 1.02

Analyze Failure Surface III



$Area = \frac{18 \times 64}{2} = 576 \square'$

Using $\gamma = 110$ pcf, $D_A = 63.36$ K

$D_P = 17.82$ K
$R_A = 2(25)(0.24) = 12.0$
$R_B = 49(0.24) = 11.76$ K
$R_P = 8.68$ K
} c = 240 psf

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{12.0 + 11.76 + 8.68}{63.36 - 17.82} = \frac{32.44}{45.54} = 0.71$

Using c = 200 psf, F.S. = 0.59

Using γ = 100 pcf, F.S. = 0.78

Using γ = 100 pcf & c = 200 psf, F.S. = 0.65

SHEET 3 OF 9

PROJECT Streambank Erosion Control Des Arc, Ark.	Page 4 of 9	COMPUTED BY JEK	DATE 18 Aug 81
SUBJECT Evaluation of Bank Stability-Inclinometer Data		CHECKED BY WJS	DATE 20 Aug 81

<u>Assume that portion of upper bank above original top bank is free standing and does not contribute to driving forces;</u>

$Area = 576 - \frac{5(19)}{2} = 528.5\ \square'$

Using $\gamma = 110$ pcf, $D_A = 58.14$ K

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{32.44}{58.14-17.82} = \frac{32.44}{40.32} = 0.80$ ($c = 240$ psf)

Using $c = 200$ psf, $F.S. = 0.67$

Using $\gamma = 100$ pcf, $F.S. = 0.88$

Using $\gamma = 100$ pcf & $c = 200$ psf, $F.S. = 0.73$

<u>Assume that portion of upper bank above cracked zone is free standing and does not contribute to driving forces;</u>

$Area = 576 - \frac{8.5 \times 30}{2} = 448.5\ \square'$

Using $\gamma = 110$ pcf, $D_A = 49.34$ K

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{32.44}{49.34-17.82} = \frac{32.44}{31.52} = 1.03$ ($c = 240$ psf)

Using $c = 200$ psf, $F.S. = 0.86$

Using $\gamma = 100$ psf, $F.S. = 1.13$

Using $\gamma = 100$ psf & $c = 200$ psf, $F.S. = 0.94$

<u>Assume Hydrostatic forces act along cracked zone;</u>

$H = \frac{\gamma_w h^2}{2} = \frac{0.0625(11.5)^2}{2} = 4.13$ K

Using $\gamma = 110$ pcf, $F.S. = \frac{32.44}{31.52+4.13} = 0.91$ ($c = 240$ psf)

Using $c = 200$ psf, $F.S. = 0.76$

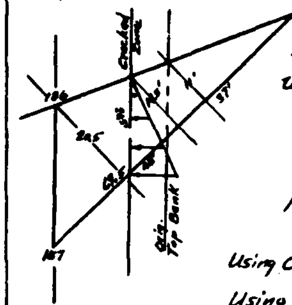
Using $\gamma = 100$ pcf, $F.S. = 0.99$

Using $\gamma = 100$ pcf & $c = 200$ psf, $F.S. = 0.83$

SHEET 4 OF 9

PROJECT Streambank Erosion Control Des Arc, Ark.	Page 5 of 9	COMPUTED BY JEK	DATE 18 Aug 81
SUBJECT Evaluation of Bank Stability-Inclinometer Data		CHECKED BY WJS	DATE 20 Aug 81

<u>Analyze Failure Surface IV</u>



$Area = \frac{20.5 \times 69.5}{2} = 712.38\ \square'$

Using $\gamma = 110$ pcf, $D_A = 78.36$ K

$D_P = 17.82$ K

$R_A = 2(29)(0.24) = 13.92$ K

$R_B = 59(0.24) = 14.16$ K

$R_P = 8.68$ K

} $c = 240$ psf

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{13.92+14.16+8.68}{78.36-17.82} = \frac{36.76}{60.54} = 0.61$

Using $c = 200$ psf, $F.S. = 0.51$

Using $\gamma = 100$ pcf, $F.S. = 0.67$

Using $\gamma = 100$ pcf & $c = 200$ psf, $F.S. = 0.56$

<u>Assume that portion of upper bank above original top bank is free standing and does not contribute to driving forces;</u>

$Area = 712.38 - \frac{11(37)}{2} = 508.88\ \square'$

Using $\gamma = 110$ pcf, $D_A = 55.98$ K

$F.S. = \frac{36.76}{55.98-17.82} = 0.96$ ($c = 240$ psf)

Using $c = 200$ psf, $F.S. = 0.80$

Using $\gamma = 100$ pcf, $F.S. = 1.06$

Using $\gamma = 100$ pcf & $c = 200$ psf, $F.S. = 0.88$

SHEET 5 OF 9

Project	Page	Computed by	Date
Streambank Erosion Control, Des Arc, Ark.	6 of 9	JEK	18 Aug 81
Subject		**Checked by**	**Date**
Evaluation of Bank Stability-Inclinometer Data		WJS	20 Aug 81

Assume that portion of upperbank above cracked zone is free standing and does not contribute to driving forces;

$Area = 712.38 - \frac{14.5 \times 48}{2} = 364.38 \square'$

Using $\gamma = 110$ pcf, $D_A = 40.08$ K (c = 240 psf)

$F.S. = \frac{36.76}{40.08 - 17.82} = 1.65$

Using C = 200 psf, F.S. = 1.38

Using $\gamma = 100$ pcf, F.S. = 1.82

Using $\gamma = 100$ pcf & C = 200 psf, F.S. = 1.51

Assume Hydrostatic forces act along cracked zone:

$H = \frac{\gamma_w h^2}{2} = \frac{0.0625(20.5)^2}{2} = 13.13$ K

Using $\gamma = 110$ pcf, $F.S. = \frac{36.76}{40.08 + 13.13 - 17.82} = 1.04$ (c = 240 psf)

Using C = 200 psf, F.S. = 0.87

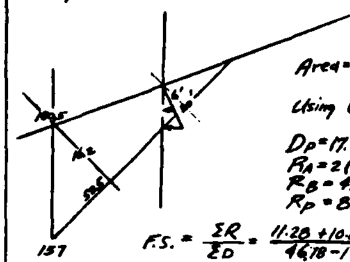
Using $\gamma = 100$ pcf, F.S. = 1.14

Using $\gamma = 100$ pcf & C = 200 psf, F.S. = 0.95

SHEET 6 OF 9

Project	Page	Computed by	Date
Streambank Erosion Control, Des Arc, Ark.	7 of 9	JEK	19 Aug 81
Subject		**Checked by**	**Date**
Evaluation of Bank Stability-Inclinometer Data		WJS	20 Aug 81

Analyze Failure Surface V



$Area = \frac{16.2 \times 52.5}{2} = 425.25 \square'$

Using $\gamma = 110$ pcf, $D_A = 46.78$ K

$D_P = 17.82$ K

$R_A = 2(23.5)(0.24) = 11.28$ K
$R_B = 45(0.24) = 10.8$ K
$R_P = 8.68$ K
$\}$ c = 240 psf

$F.S. = \frac{\Sigma R}{\Sigma D} = \frac{11.28 + 10.8 + 8.68}{46.78 - 17.82} = \frac{30.76}{28.96} = 1.06$

Using C = 200 psf, F.S. = 0.88
Using $\gamma = 100$ pcf, F.S. = 1.17
Using $\gamma = 100$ pcf & C = 200 psf, F.S. = 0.97

Assume that portion of upperbank above cracked zone is free standing and does not contribute to driving forces:

$Area = 425.25 - \frac{6(20)}{2} = 365.25 \square'$

Using $\gamma = 110$ pcf, $D_A = 40.18$ K

$F.S. = \frac{30.76}{40.18 - 17.82} = 1.38$ (c = 240 psf)

Using C = 200 psf, F.S. = 1.15

Using $\gamma = 100$ pcf, F.S. = 1.52

Using $\gamma = 100$ pcf & c = 200 psf, F.S. = 1.26

Assume Hydrostatic forces act along cracked zone

$H = \frac{\gamma_w h^2}{2} = \frac{0.0625(9)^2}{2} = 2.53$ K

$F.S. = \frac{30.76}{40.18 + 2.53 - 17.82} = 1.23$ (c = 240 psf)

Using c = 200 psf, F.S. = 1.03

Using $\gamma = 100$ pcf, F.S. = 1.36

Using $\gamma = 100$ pcf & c = 200 psf, F.S. = 1.13

SHEET 7 OF 9

PROJECT: Embankment Erosion Control, Des Arc, Ark.	Page 8 of 9	COMPUTED BY: JEK	DATE: 19 Aug 81
SUBJECT: Evaluation of Bank Stability-Inclinometer Data		CHECKED BY: WJS	DATE: 20 Aug 81

<u>Summation of Computed Factors of Safety</u>

Loading Case	Factors of Safety			
	γ = 110 pcf		γ = 100 pcf	
	C = 240 psf	C = 200 psf	C = 240 psf	C = 200 psf
I	1.36	1.13	1.50	1.25
II	1.11	0.925	1.22	1.02
III (1)	0.71	0.59	0.70	0.65
III_A	0.80	0.67	0.88	0.73
III_B	1.03	0.86	1.13	0.94
III_C	0.91	0.76	0.99	0.83
IV (1)	0.61	0.51	0.67	0.56
IV_A	0.96	0.80	1.06	0.88
IV_B	1.65	1.38	1.82	1.51
IV_C	1.04	0.87	1.14	0.95
V	1.06	0.88	1.17	0.97
V_B	1.38	1.15	1.52	1.26
V_C	1.23	1.03	1.36	1.13

Subscript A denotes assumption that portion of upper bank above original top bank is free standing and does not contribute to driving forces.

Subscript B denotes assumption that portion of upper bank above cracked zone is free standing and does not contribute to driving forces.

Subscript C denotes same assumption as B but with hydrostatic forces applied along cracked zone.

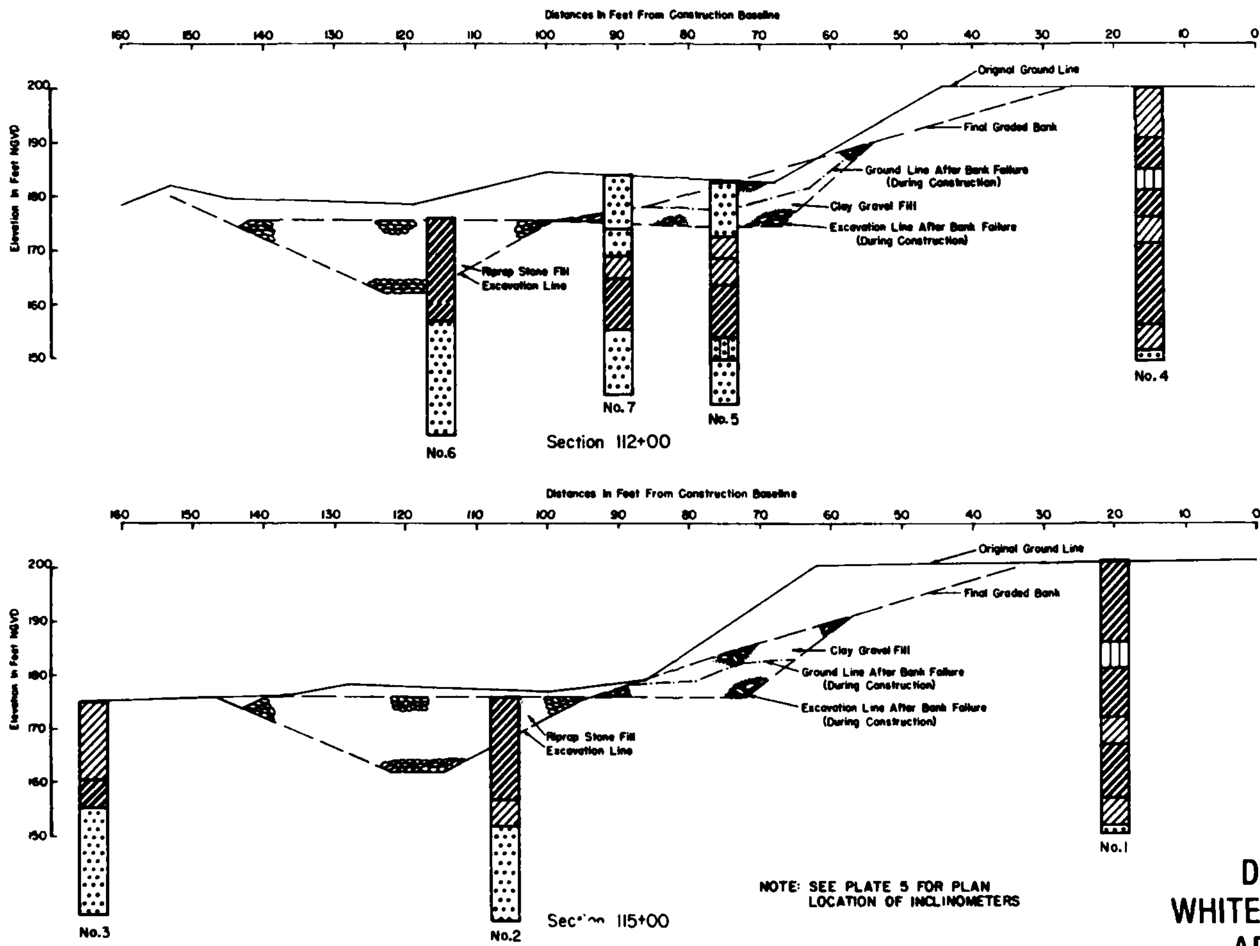
(Continued on next page)

PROJECT: Embankment Erosion Control, Des Arc, Ark.	Page 9 of 9	COMPUTED BY: JEK	DATE: 19 Aug 81
SUBJECT: Evaluation of Bank Stability-Inclinometer Data		CHECKED BY: WJS	DATE: 20 Aug 81

(1) Considered overly conservative as failure surface passes through "undisturbed" upper bank soils which possess a high shear strength.

<u>Comments on Analyses:</u>

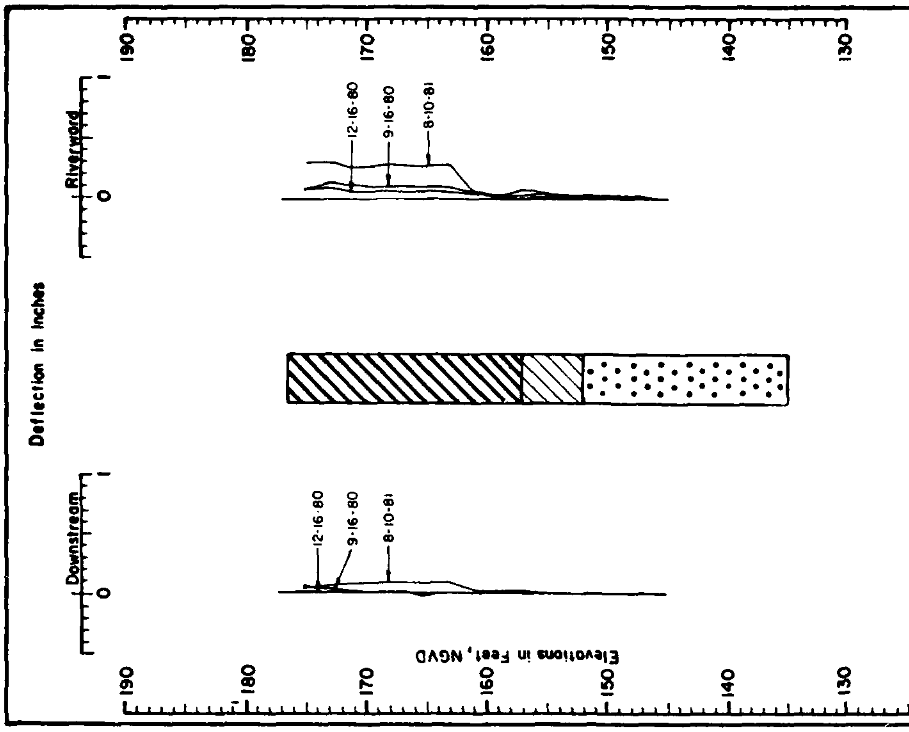
Methodology used for analyses is simplistic in nature but is consistent with ability to determine shear strength of the soils, inclinometer data and observed surface conditions. Inclinometer data collected to date indicates a progressive pattern of movement of the banks toward the river. The stability analyses indicate the distinct probability of failure of the slopes along the planes of movement indicated by the inclinometer data. The failure surface appears to be developing within the disturbed soil within the bank and through the clay stratum beneath the rock toe trench. As movement continues it may be anticipated that the shear strength of the soil will be further reduced through remolding until it approaches that used in the analyses. Conclude that the bank is not stable and that failure of the bank, through the disturbed soil within the bank and beneath the rock toe trench, will, in all probability, occur in the relatively near future.



NOTE: SEE PLATE 5 FOR PLAN LOCATION OF INCLINOMETERS

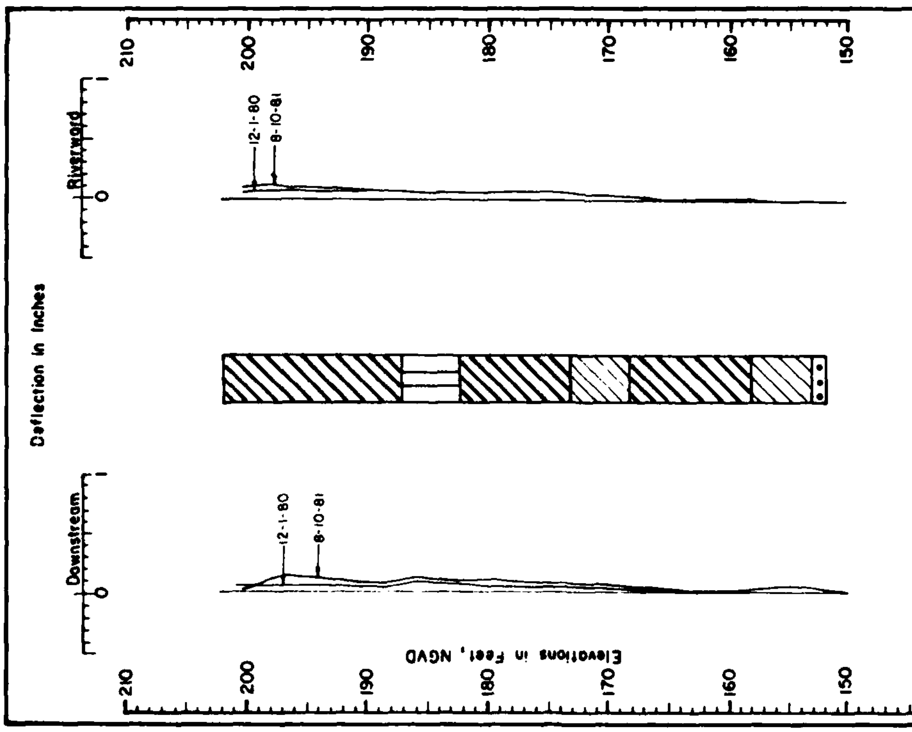
DES ARC, ARKANSAS
WHITE RIVER - BANK SECTIONS
AFTER CONSTRUCTION
OCTOBER 29, 1980

PLATE B1



DES ARC, ARK.

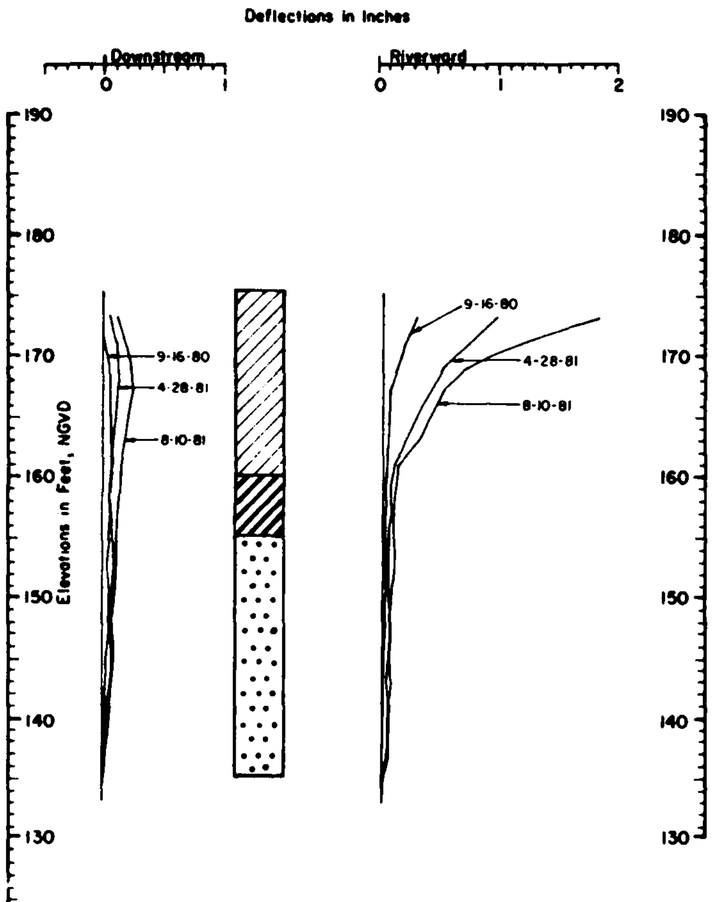
INCLINOMETER NO. 2 IN LANDWARD EDGE OF STONE TOE FILL BELOW USED TIRE REVETMENT, NEAR CENTER OF PROJECT



DES ARC, ARK.

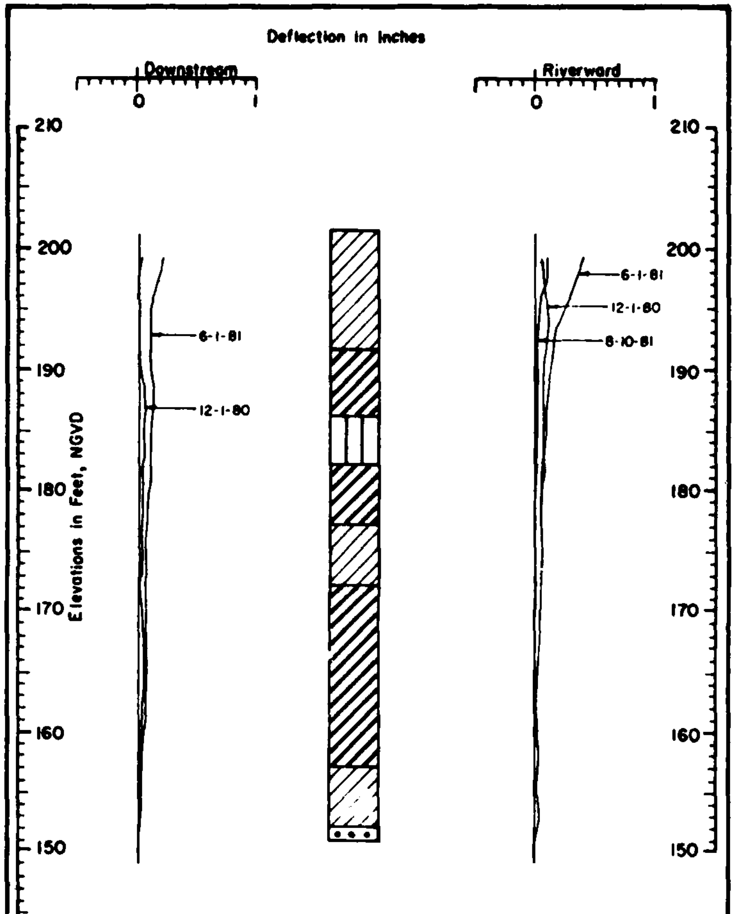
INCLINOMETER NO. 1 LOCATED LANDWARD TOP OF BANK BEHIND USED TIRE REVETMENT, NEAR CENTER OF PROJECT

PLATE B2



DES ARC, ARK.

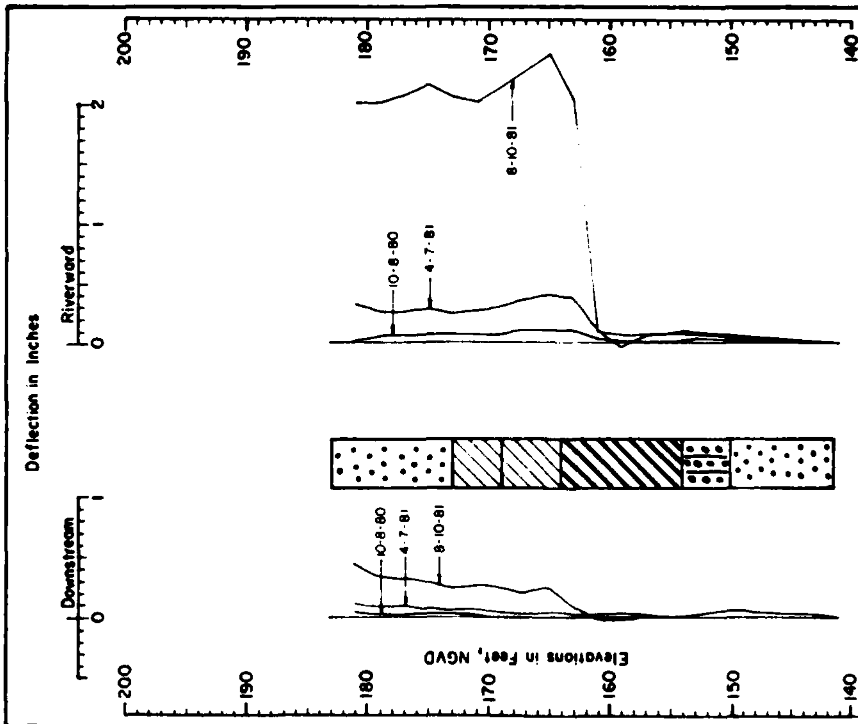
INCLINOMETER NO. 3 LOCATED IN TALUS BERM RIVERWARD OF THE STONE FILLED TOE TRENCH, NEAR MIDDLE OF PROJECT



DES ARC, ARK.

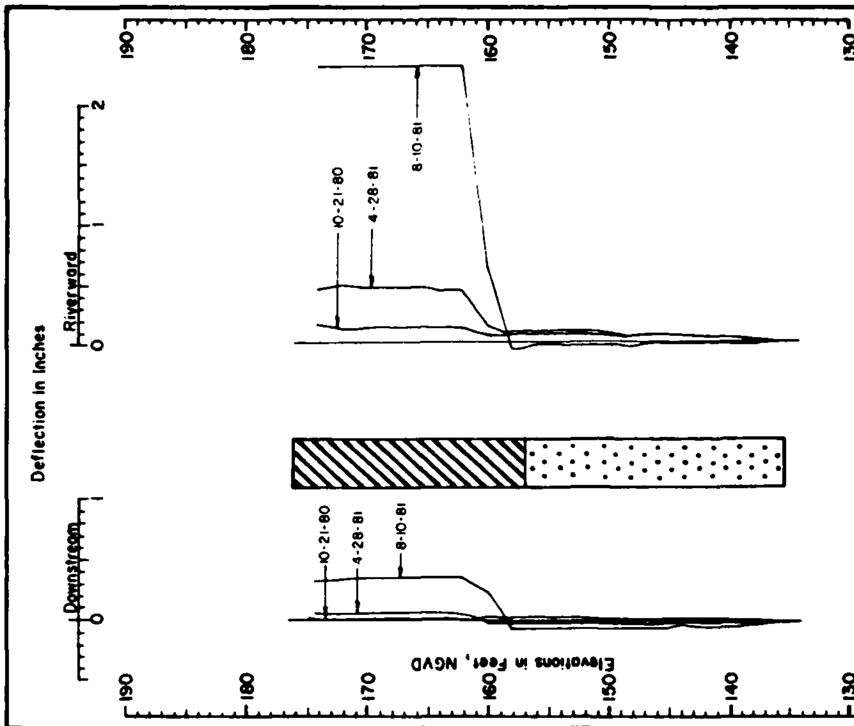
INCLINOMETER NO. 4 LOCATED BEHIND TOP OF BANK NEAR UPSTREAM END OF PROJECT

PLATE B3



DES ARC, ARK.

INCLINOMETER NO. 5 LOCATED ON SLOPE NEAR CENTER OF GROUT FILLED FABRIC SECTION, NEAR UPSTREAM END OF PROJECT



DES ARC, ARK.

INCLINOMETER NO 6 LOCATED IN STONE FILLED TOE TRENCH NEAR UPSTREAM END OF PROJECT

PLATE B4

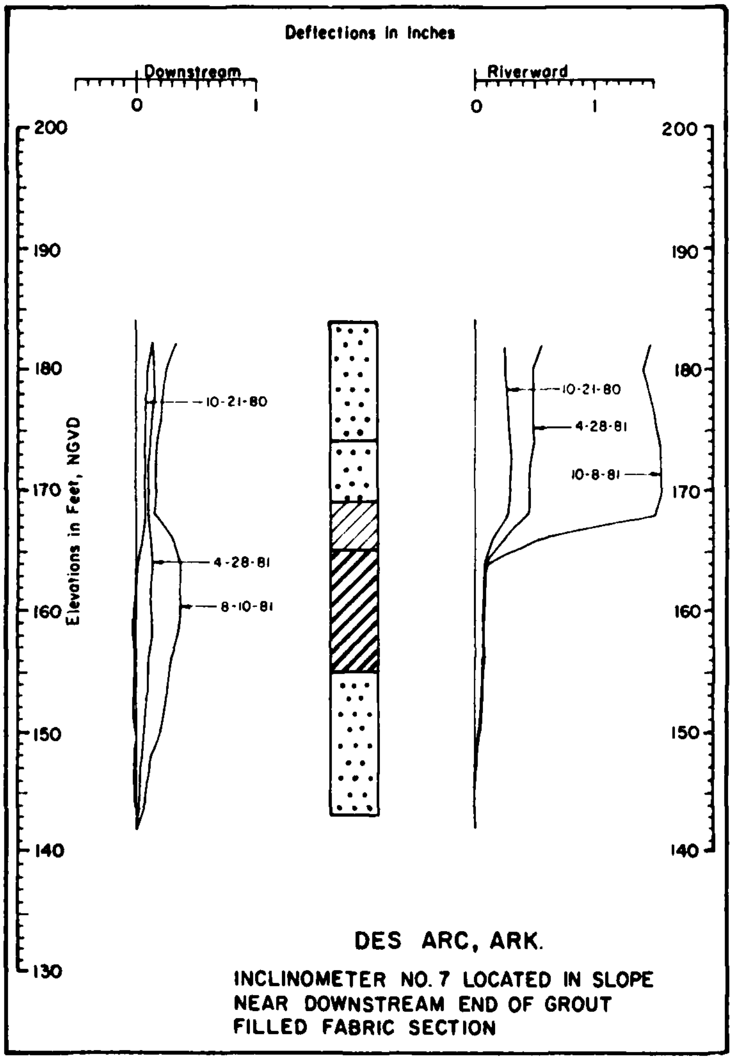

Deflections in Inches
Downstream
0
1
Riverward
0
1
200
190
180
170
160
150
140
130
Elevations in Feet, NGVD
10-21-80
4-28-81
8-10-81
10-21-80
4-28-81
10-8-81
DES ARC, ARK.
INCLINOMETER NO. 7 LOCATED IN SLOPE NEAR DOWNSTREAM END OF GROUT FILLED FABRIC SECTION

PLATE B5

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER Technical Report H-77-9	2. GOVT ACCESSION NO. AD-A141 139	3. RECIPIENT'S CATALOG NUMBER
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		6. PERFORMING ORG. REPORT NUMBER
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18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Bank erosion
Bank protection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A preliminary study of streambank erosion control was conducted with the major emphasis on an extensive literature survey of known streambank protection methods. In conjunction with the survey, preliminary investigations were conducted to identify the mechanisms that contribute to streambank erosion and to evaluate the effectiveness of the most widely used streambank protection methods. The results of the literature survey and the two preliminary investigations are presented herein.

(Continued)

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued).

The text of the "Streambank Erosion Control Evaluation and Demonstration Act of 1974" is presented in Appendix A. A list of commercial concerns that market streambank protection products is provided in Appendix B. Appendix C contains a glossary of streambank protection terminology. A detailed bibliography resulting from the literature survey is provided in Appendix D, and a listing of selected bibliographies related to streambank protection are provided in Appendix E.