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THE STREAMBANK EROSION CONTROL EVALUATION AND
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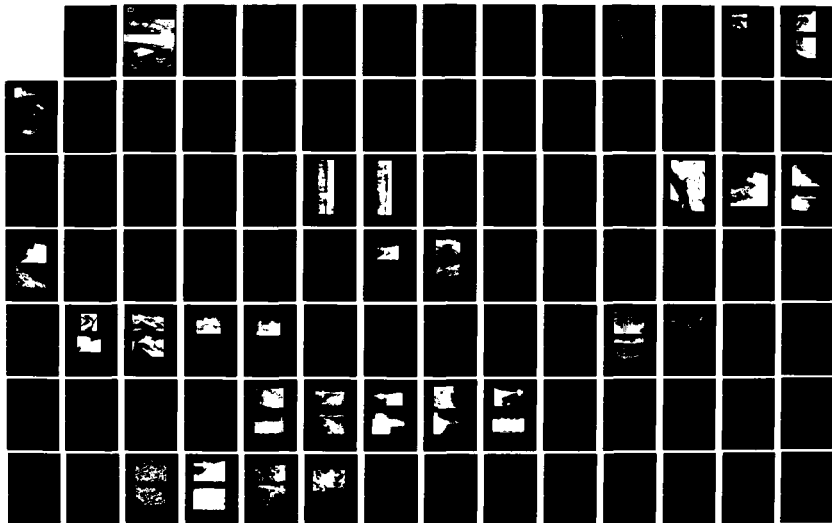
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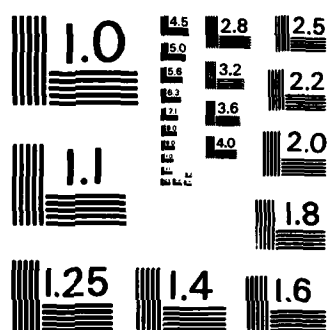
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December 1981

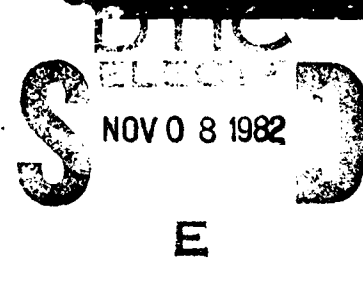
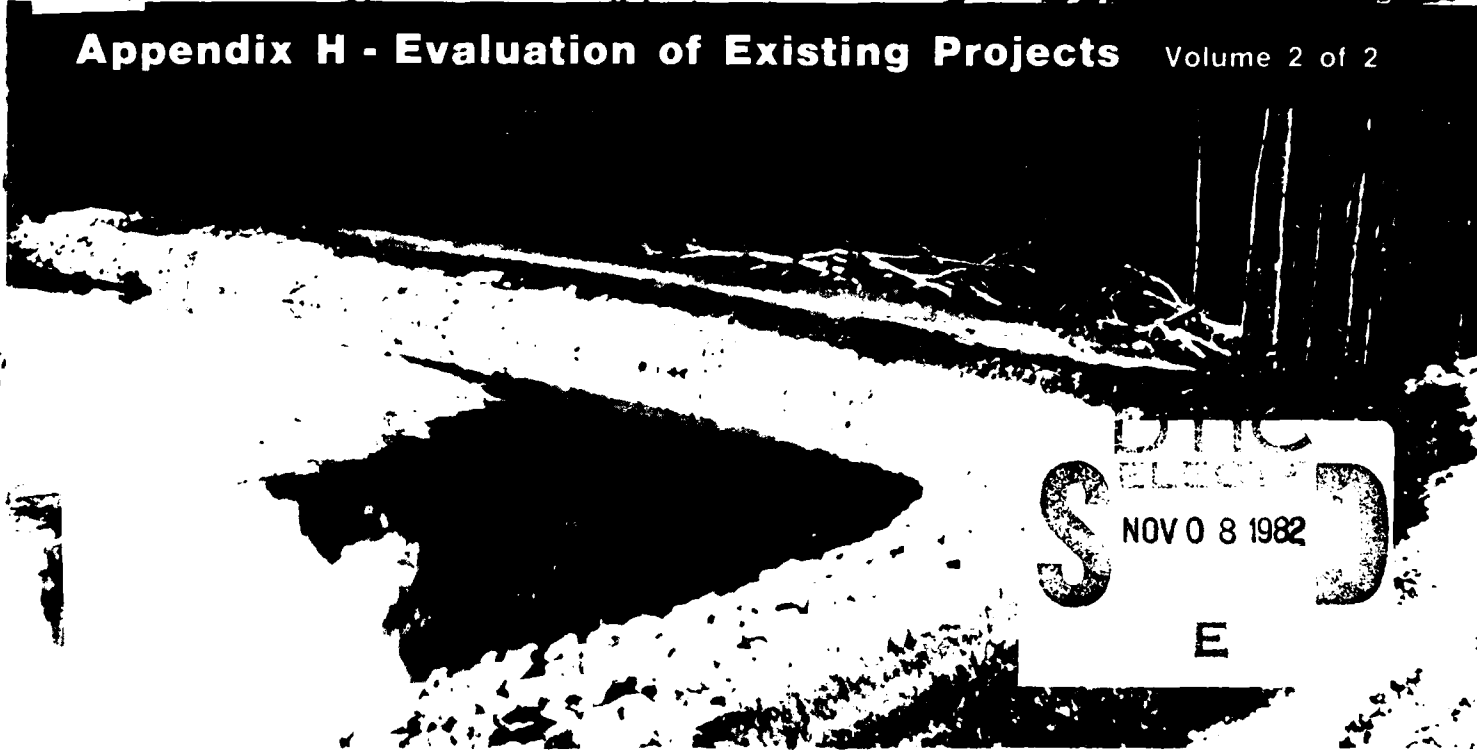
**THE STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION ACT OF 1974
SECTION 32, PUBLIC LAW 93-251**

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Appendix H - Evaluation of Existing Projects 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 H EVALUATION OF EXISTING PROJECTS VOLUME 2 OF 2

Consisting of
A BRIEF SUMMARY REPORT AND INDIVIDUAL EVALUATION
REPORTS ON FIFTY EXISTING STREAMBANK EROSION CONTROL
PROJECTS CONSTRUCTED PRIOR TO OR SEPARATE FROM THE
SECTION 32 PROGRAM



U.S. ARMY CORPS OF ENGINEERS
December 1981

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APPENDIX H

EVALUATION OF EXISTING PROJECTS

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**HAYWARD CREEK
QUINCY, MASSACHUSETTS**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Hayward Creek River Mile N/A Side Both
Local Vicinity Quincy/Braintree line Lat N42°15' Long W71°
At/Nr City Quincy County Norfolk State MA Cong Dist 11
CE Office Symbol NED Responsible Agency Corps of Engineers
Site Map Sources NED
Land Use Urban

(2) Hydrology at or Near Site

Stage Range ukn to - ^{*ungaged watershed} ft; Period of Record 19 - to 19 -
Discharge Range ukn to - cfs; Velocity Range - to - fps
Sediment Range ukn to - tpd; Period of Record 19 - to 19 -
Design Stage 2.5 ft; Flow 33 cfs; Average Recurrence Interval 24 yr
Design Flow Velocity: Average 3-4 fps; Near Bank ukn fps

Comments Channel discharges effectively controlled by upstream Corps flood control storage reservoirs.

(3) Geology and Soil Properties

Right bank-landfill Brown silty sandy gravel
Bank (USCS) Left bank-brown, silty sand Bed (USCS) with pockets of mud fill
Data Sources gravel with pockets of mud-fill
Project plans and specs.
Groundwater Bank Seepage None observed
Overbank Drainage None observed
Comments None

(4) Construction of Protection

Need for Protection Protect against potential streambank erosion in enlarged creek channel. (Creek connects flood control reservoirs and pressure conduit)
Erosion Causative Agents Excessive velocities on bare bank and channel invert

Protection Techniques Installation of "Monoslab" concrete blocks.
General Design Monoslab grid system on channel invert and both side slopes to top of bank.
Project Length 500 ft; Construction Cost \$ 28,500 Mo/Yr Completed 9/77

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Ungaged watershed

Repairs and Costs (Item, Cost, Date) None to date

Comments: Monoslab has worked well and due to nature of material the problems experienced with riprap(i.e. vandalism) have not occurred.

(6) Performance Observations and Summary

Monitoring Program Semiannual inspections (spring and fall)

Documentation Sources Photographs, project plans and specs, and trip reports

Project Effect on Stream Regime Minimal

Project Effect on Environment Beneficial. Vegetation has grown over monoslab very well and aesthetic value greatly enhanced along the streambank.

Successful Aspects Monoslab has successfully stabilized the streambank and has not required maintenance or repair since installation.

Unsuccessful Aspects No major unsuccessful aspects. Some minor settlement of streambank where low-lying drainage areas exist behind channel.

General Evaluation Monoslab has stabilized the bank. No erosion problems since its installation. Vegetation has become well established.

Recommendations Due to lack of high flows, long-term stability unknown, so far, very effective in urban setting because it cannot be removed and thrown into channel as can riprap.

(7) Additional Information, Comments, and Summary

Map No. 26. Monoslab has good durability. Placement of slabs very laborious and would probably prove too expensive along large channel reaches as only one slab can be laid at a time. Erosion had not been a severe problem at this site in the past. Channel work was done to provide uniform entry to conduit and for aesthetic reasons.

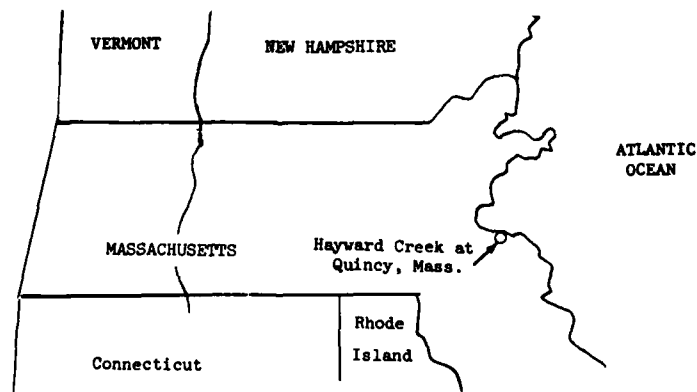
Attached Items:

26-1	Project summary and location	26-4	Project cross sections and photo
26-2	Project site	26-5	Photos 1 yr after const.
26-3	Monoslab dimensions	26-6	" 2 yr " "

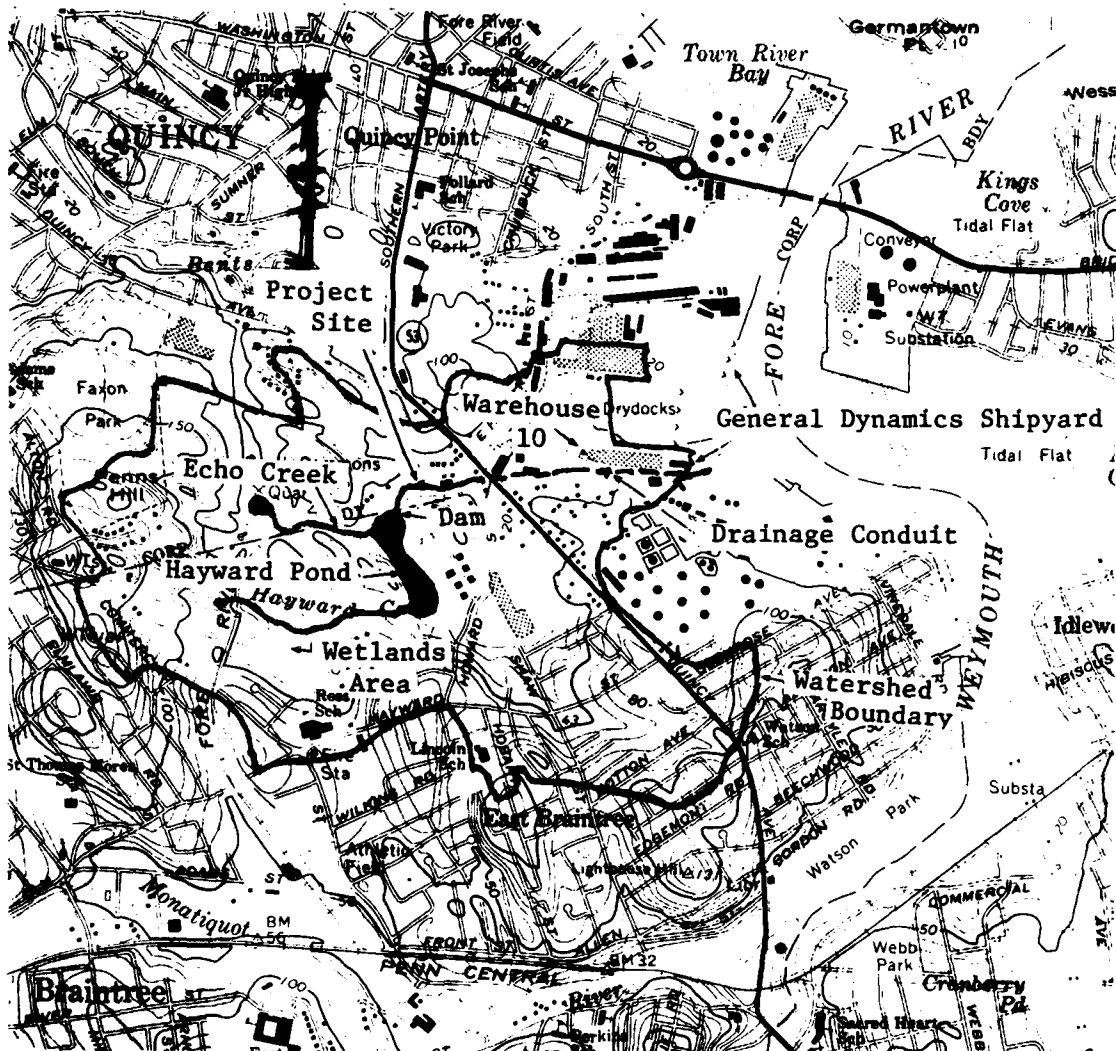
HAYWARD CREEK
AT QUINCY, MASS.

Channel improvement for Hayward Creek consisted of a precast cellular block installation (completed in September 1977) to protect the banks and bottom from erosion. The blocks were placed over a sand layer and gravel bedding. Upper banks were seeded and mulched. In July 1979, minor settlement (about 1 in.) was detected on two areas of the bank; however, the settlement has not affected the integrity of the revetment. Grass growth is still sparse but it has been thus far proved adequate for erosion protection. New England Division, CE, has made the following recommendations regarding the use of precast cellular blocks:

- a. The blocks should be laid on a uniform slope.
- b. A base course, usually consisting of gravel, should be laid prior to block placement. A sand bedding should be used between the gravel and the blocks.
- c. The revetment should be extended past the top of the base.
- d. Provisions for the drainage of overland flow at the crest of the bank should be included in the project design, preferably an exposed open channel conduit running parallel to the channel alignment with provisions for discharge into the channel at periodic intervals.



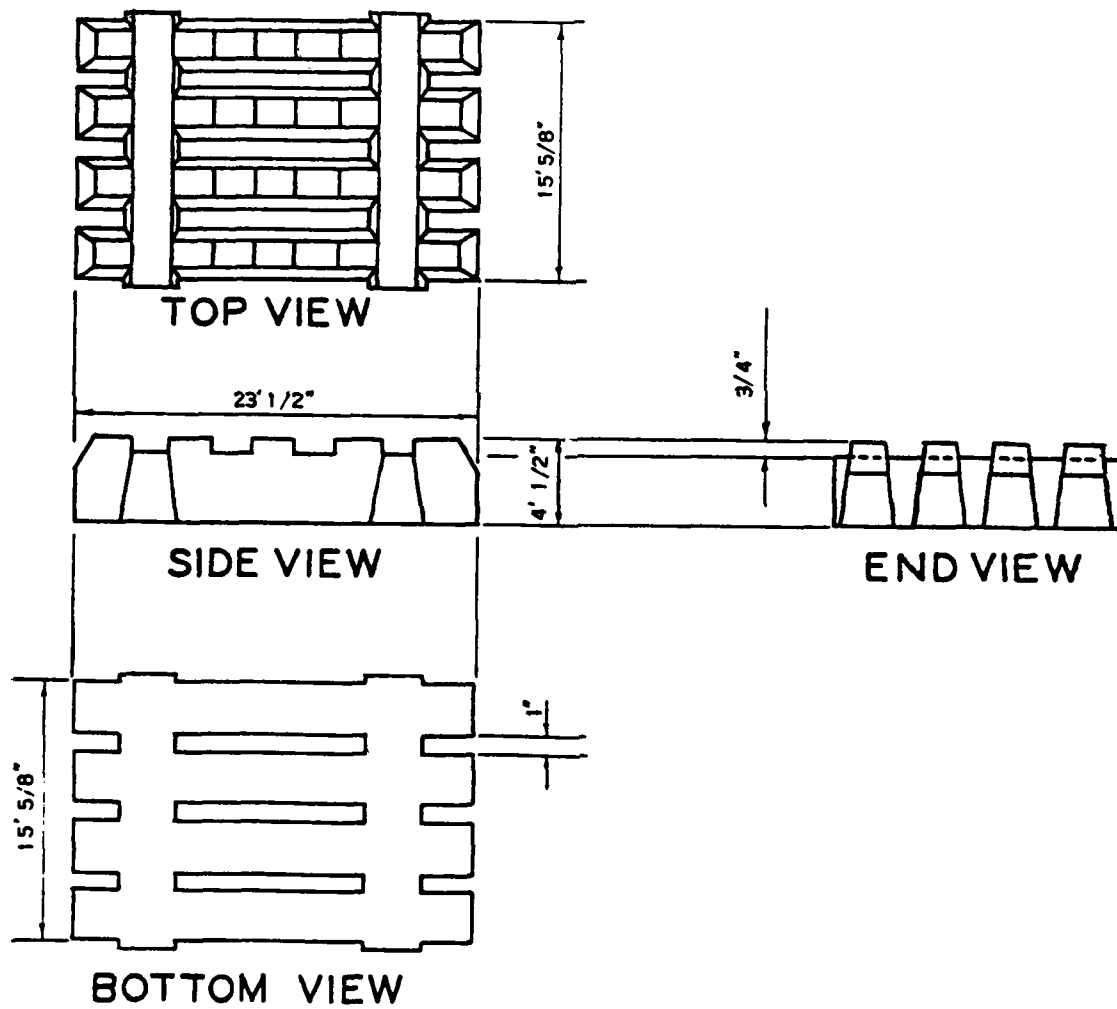
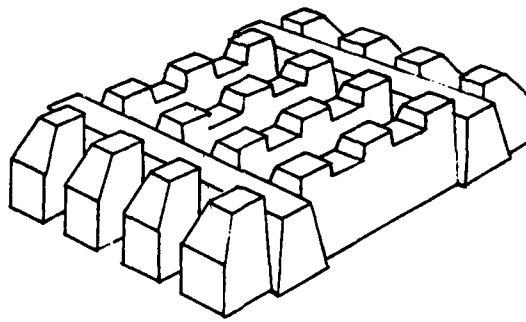
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Hayward Creek Basin (Source: USGS 1:24,000 topographic quadrangle map for Weymouth, Mass., 1971)

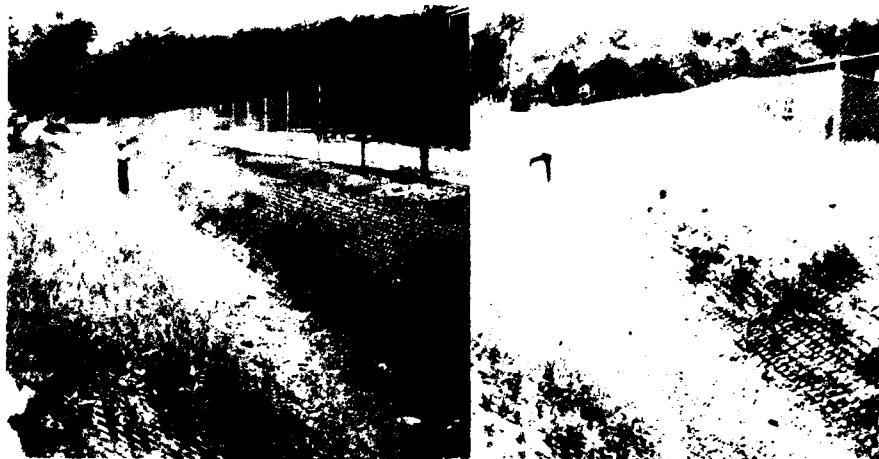
ITEM 26-2

H-26-4

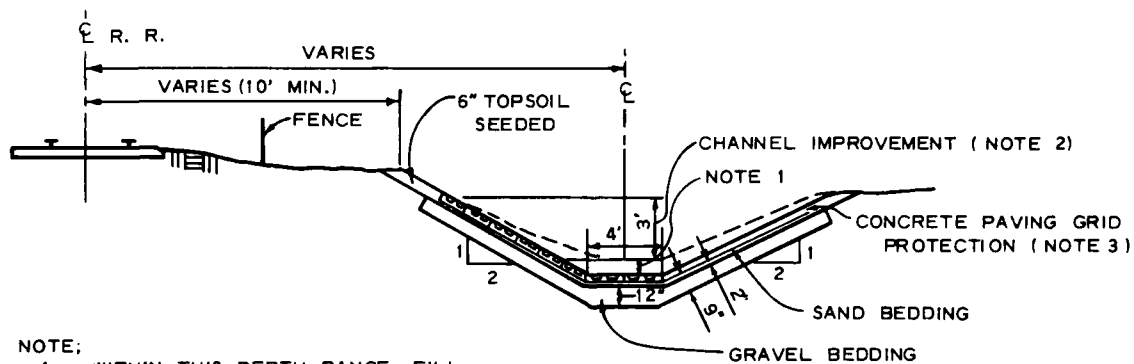


Monoslab dimensional measurements

ITEM 26-3



Upstream stereoscopic view of Monoslab revetment,
showing trapezoidal channel cross section, Hayward
Creek at Quincy, Massachusetts (25 July 1978)



NOTE:

1. WITHIN THIS DEPTH RANGE, FILL IN GRIDS WITH CRUSHED STONE FILL.
2. WITHIN THIS DEPTH RANGE, FILL VOIDS IN GRIDS WITH TOP SOIL MIX. SEED AND MULCH.
3. 6 GRIDS AT 1.5' ON EACH SLOPE.

Cross-sectional view of Monoslab revetment,
Hayward Creek at Quincy, Massachusetts

ITEM 26-4

H-26-6



Looking downstream at Monoslab revetment,
Hayward Creek at Quincy, Massachusetts
(photograph furnished by New England
Division, July 1978)



Upstream view of Monoslab revetment, showing
trapezoidal channel cross section, Hayward
Creek at Quincy, Massachusetts (25 July 1978)

PHOTOGRAPHS OF HAYWARD CREEK ONE YEAR AFTER CONSTRUCTION

ITEM 26-5

H-26-7



Looking upstream from Sta 1+25



Settlement of upper right slope at Sta 3+50

MONOSLAB REVETMENT TWO YEARS AFTER CONSTRUCTION
HAYWARD CREEK AT QUINCY, MASSACHUSETTS
(SEPTEMBER 1980)

ITEM 26-6

H-26-8

**WINOOSKI RIVER
NORTH WILLISTON, VERMONT**

EVALUATION OF EXISTING BANK PROTECTION WORKS

Stream Winooski River (2 sites) River Mile 24.5 Side Right
Local Vicinity Williston-Jericho Lat N44°27'12"&25" Long W73°01'15"&28"
At/Nr City N. Williston County Chitt. State VT Cong Dist 1
CE Office Symbol NAD Responsible Agency _____
Site Map Sources USGS, HUD, F.I.S.
Land Use Information Sources _____

Stage Range _____ to 23.5 ft; Period of Record 1935 to 1979.
 Discharge Range _____ to 45,000 cfs; Velocity Range _____ to _____ fps
 Sediment Range _____ to _____ tpd; Period of Record 19__ to 19__.
 Bank-full Stage 15± ft; Flow _____ cfs; Average Recurrence Interval _____ yr
 Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
 Comments _____

Bank (USCS) SP, SP-SM Bed (USCS) Gravel and cobbles
 Data Sources _____
 Groundwater Bank Seepage None observed
 Overbank Drainage No concentrated drainage
 Comments _____

Need for Protection Bank erosion

Erosion Causative Agents Water, ice, debris

Protection Techniques Log cribbing and riprap

General Design _____

H-27-1

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Data) _____

Comments: _____

(6) Performance Observations and Summary

Monitoring Program _____

Documentation Sources _____

Project Effect on Stream Regime Has probably changed meander pattern in this area.

Project Effect on Environment Negligible

Successful Aspects Stone has collapsed into a compact riprap that has held the bank in most places.

Unsuccessful Aspects Log cribbing rotted away and/or destroyed by ice.

General Evaluation The poplar log cribbing was a complete failure. The stone has held up well in most places.

Recommendations Riprap is one of the most common and effective methods of streambank protection, but does require maintenance at least after every high water. Vegetation covers along the streambank requires inspection at least annually, preferably in early spring. Bare places should be replanted. Poplar logs served only as a temporary solution.

(7) Additional Information, Comments, and Summary

Map No. 27.

Attached Items.

27 - 1 Project Description & Details

27 - 2 - Project Location

27 - 3 - Basin Map and Profile

27 - 4 & 5 - Discharge records

27 - 6 - Details of Stone-filled Log Crib

27 - 7 - Typical sections

(Downstream)

27 - 8 - Typical Sections

(Upstream)

27 - 9 - Photo of upstream site

27 - 10 - Photo of downstream site

Winooski River at North Williston, Vermont
Demonstration Project Performance Report

I. INTRODUCTION

1. Project Name and Location. Winooski River Demonstration Projects, Winooski River, North Williston, Vermont (plate 1, page H-27-10).
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 Winooski River, Vermont, constructed by Soil Conservation Service (SCS) in late 1930's and monitored recently by the New York District, Corps of Engineers.
4. Problem Resume. Until 1927, streambank erosion caused little trouble. Riverbank farmers kept erosion in check by simple erosion control measures such as patching eroded areas with stones and granite blocks, and removal of lodged trees to prevent stream turbulence. A devastating flood in the autumn of 1927 caused such overwhelming damages that homemade structures were no longer adequate. Also, grazing of vegetation by farm animals along the bank has diminished the protection needed. At the request of the local people, a study of soil erosion control demonstration project was conducted by the Soil Conservation Service beginning in 1936. The final construction of the demonstration project was completed in late 1930's. Upon the field review conducted in October 1980, there was no sign of erosion on the streambanks. The vegetations provided good coverage and the riprapping is quite stable.

II. HISTORICAL DESCRIPTION

5. Stream.
 - a. Topography. The Winooski River drains an area approximately

ITEM 27-1
(Sheet 1 of 7)

1,065 square miles and length of about 90 miles. The basin occupies all of Washington County, a little less than one-half of Chittenden County, and small parts of Lamoille, Orange, Addison, and Caledonia Counties in the upper central part of Vermont. The river flows in an east-west direction from its source and empties into Lake Champlain about 4 miles north of the City of Burlington, Vermont. From its mouth to Montpelier, the river valley is rich in arable land and well suited to cultivation; it is generally flat and cleared except for narrow rock gorges at Winooski, Bolton, and Middlesex. Above Montpelier the valleys of the main stream and tributaries are generally narrow, steep and rugged, with heavily timbered slopes. The river floor ranges in elevation from 98 ft MSL - 900 ft MSL (plate 2).

b. Geology. The prevailing rocks are metamorphic gneisses, schists, and quartzites of Cambrian and perhaps pre-Cambrian age. Winooski River is an antecedent stream which existed prior to the uplift of the Green Mountains, and which succeeded in cutting its valley across the range during the uplift period. The valley floor below Montpelier is deeply covered with delta deposits of gravel, sand and clay. Glacial deposits such as boulders and unsorted debris also occur in this area. Although much of the area is wooded, the soils are very thin, forming a shell over the parent rock.

c. Land Use and Development. The average length of growing season in the valley is 160 days. Because of the short growing season the valley of the Winooski Basin is largely devoted to the raising of fodder crops in support of dairying. Approximately 59 percent of the watershed area is in farmland. The remainder of the area is largely in woodland with only minor urbanization. There are very few industrial and residential developments within the study area with the exception of homes located along Route 117 east of Essex Junction.

d. Hydrologic Characteristics. The climate of the Winooski watershed is characterized by long cold winters with considerable snowfall

and comparatively short summers. The average annual temperature is 44.5 degrees Fahrenheit at Burlington and 41.4 degrees at Northfield, with extremes varying from 41 degrees below zero to 101 degrees above zero. The average annual precipitation over the watershed, based on data in and adjacent to the basin, is about 34.5 inches. The minimum recorded annual precipitation at Burlington was 20.99 inches to a maximum of 49.44 inches in 1883. The Winooski River has a long history of flooding resulting either from excessive rainfall, runoff from snowmelt, or a combination of the two. The flood of November 1927 was by far the greatest and most destructive in the state's history, taking 55 lives in the Winooski River basin alone. The record storm resulted in discharges of 113,000 cubic feet per second (plate 3). The majority of the storms occurring over this region are transcontinental and coastal types. The former type, and most frequent rain producer, is most likely to occur in the winter and early spring. The coastal type, and most intense rain producer, is most likely to occur in the late summer and early fall. Of several hundred rainstorms which have been recorded in this area since 1869, the greatest number have occurred in August; but those producing the worst floods have occurred from September through November.

e. Channel Conditions. The Winooski valley is characterized by steep headwater tributaries which promote rapid runoff up to 20 feet per second into the main channel and a succession of long flat meandering reaches in the main stream which terminate in narrow precipitous rock gorges. From the mouth of the river to the village of Winooski is a tortuous stream continually shifting its course across the alluvial plain. In this reach, the stream has an average width of 350 feet, with generally unstable banks about 8 feet high, composed of silt and some gravel. Channel capacities on the main river are limited by the long flat reaches of small slope and the low adjoining banks. Under extreme flood conditions the Winooski, Bolton, and Middlesex Gorges become increasingly important as restrictions to flow, a condition which becomes considerably aggravated by accumulations of drift or ice.

ITEM 27-1
(Sheet 3 of 7)

6. Demonstration Site - Test Reach.

a. Hydrologic Characteristics. As previously stated, the average annual rainfall over the watershed was about 34.5 inches. The amount of runoff at any given time depends largely upon the soil conditions. The spring runoff may be as high as 400 percent due to the melting of accumulated snow. Furthermore, deep percolation is reduced by the relatively thin layer of soil cover and impermeable bedrock. The nearest gaging station is located at Essex Junction approximately 3 miles downstream of the demonstration site.

b. Hydraulic Characteristics. The average overbank flow velocity in the Winooski River varies from 2-3 feet per second. The channel capacity of the Winooski River between Richmond and North Williston, Vermont, is about 18,000 to 19,000 cubic feet per second. The United States Geological Survey has maintained an automatic recording stream-gage station near Essex Junction, Vermont (plate 4). Discharges measured at Essex Junction gage are slightly higher than those at North Williston.

c. Riverbank Description.

(1) Bank Materials. The predominant soil types for the project area are Pittsfield and Woodbridge loam, Hollis and Sheldon fine sandy loam, Suffield and Saco silt loam, Adams loamy fine sand, Hadley very fine sandy loam, and rough stony land. The riverbed is classified as gravels and cobbles.

(2) Normal Bank Vegetation. Vegetation cover on the banks consists mainly of grasses, different types of weeds, cockleburrs, vines, shrubs, and hardwood trees.

(3) Bank Erosion Tendencies. Wood products are important in the economy of many farms, especially in hilly sections. Almost three-fourths of the woodland was logged except for the steep slopes, which were inaccessible. This resulted in greater runoff and soil loss. Less water is absorbed and stored in the soils. Snow melts more quickly in

the spring. Consequently, runoff has increased and the damage caused by floods in the valleys grows proportionately. There was evidence to indicate that cattle and other farm animals have damaged the streambanks by trampling and heavy grazing. This grazing resulted in a depletion of vegetation to the point where it was of little aid in holding the soil on the banks. Other causes of erosion were probably ice and debris.

III. DESIGN AND CONSTRUCTION

7. General. No construction plans or photos were available for either the upstream or downstream site. A circular published by the Soil Conservation Service titled "Streambank Erosion Control on the Winooski River, Vermont" was the only document on the demonstration projects. Based on interviews with two nearby landowners who remember the original construction, the upstream and downstream sites consisted of stone-filled log cribbing and hand-placed riprap. The construction of the two projects was accomplished around the late 1930's.

8. Basis for Design. Prior to 1927, streambank erosion caused little damage. The valley farmers were able to control erosion with homemade structures such as patching small eroded areas with stones and granite blocks, and removal of lodged trees to prevent stream turbulence. The 1927 flood of record caused such damages to the streambanks that homemade structures were no longer adequate. Another severe flood in 1936 caused additional damages in the valley. At the request of local peoples and the University of Vermont, a survey was conducted by the Soil Conservation Service. A plan was approved giving full consideration to the soil conservation needs of the entire watershed and for treating all lands, as needed, in accordance with complete farm conservation plans.

9. Construction Details. Funds available for streambank erosion control were limited. There was no prospect of obtaining local, State, or Federal funds for maintenance of structures after completion. Therefore, all measures had to be simple in design and easy to construct in order

ITEM 27-1
(Sheet 5 of 7)

to be maintained by the local farmers with materials available on the farm. With these limitations, standard types of permanent structures could not be built. The temporary structure was designed primarily to protect the banks until vegetative control could be established. Stone riprap however, was used to permanently protect the base of slopes. The temporary structure used for both the upstream and downstream sites under consideration is a stone-filled log crib (plate 5). Based on available information, the log used on the structure must be 12 inch minimum in diameter. For the upstream and downstream sites, the length of the cribbing are approximately 880 feet to 150 feet, respectively. Prior to 1927, grazing and trampling by farm livestock on the bank resulted in a depletion of vegetative cover and loosening of the soil. In order to establish a plant cover for erosion control, it was necessary to exclude farm livestock. This was done by fencing off the bank portion of the pasture from the remainder of the field.

10. Cost. No data is available as to the cost of the structures.

IV. PERFORMANCE OF PROTECTION

11. Evaluation of Protection Performance. The two sites on the Winooski River were monitored in October 1980. From the field review, it appears that the riprapping was the most effective where the banks were subject to undercutting. The materials used varied from "one-man" to larger size stones and boulders. The only thing left of the temporary structure are the stones remnant of the log crib. As shown on plates 6 and 7, the stones from the log cribbing have settled on the bank and along with the riprapping, provided an excellent blanket against scouring. Based on interviews with landowners who remember the original construction, ice took out the stoplogs and the logs rotted after 4 years. What remains is quite stable. Fine material has filled the voids in the stones and native vegetation provided good coverage thus creating an extremely stable bank. Photos of the present condition are shown on plates 8 and 9.

12. Reconstruction. At this time, there seems to be no need for reconstruction of the two sites. Vegetation has grown to a point where adequate protection to the upper portion of the banks is provided.

13. Conclusion. What remains of these structures constructed in the late 1930's has proved to be very effective for erosion control. The intended purpose of these structures was to provide permanent protection on the streambank and temporary protection for banks above the normal water surface to allow vegetation to grow.

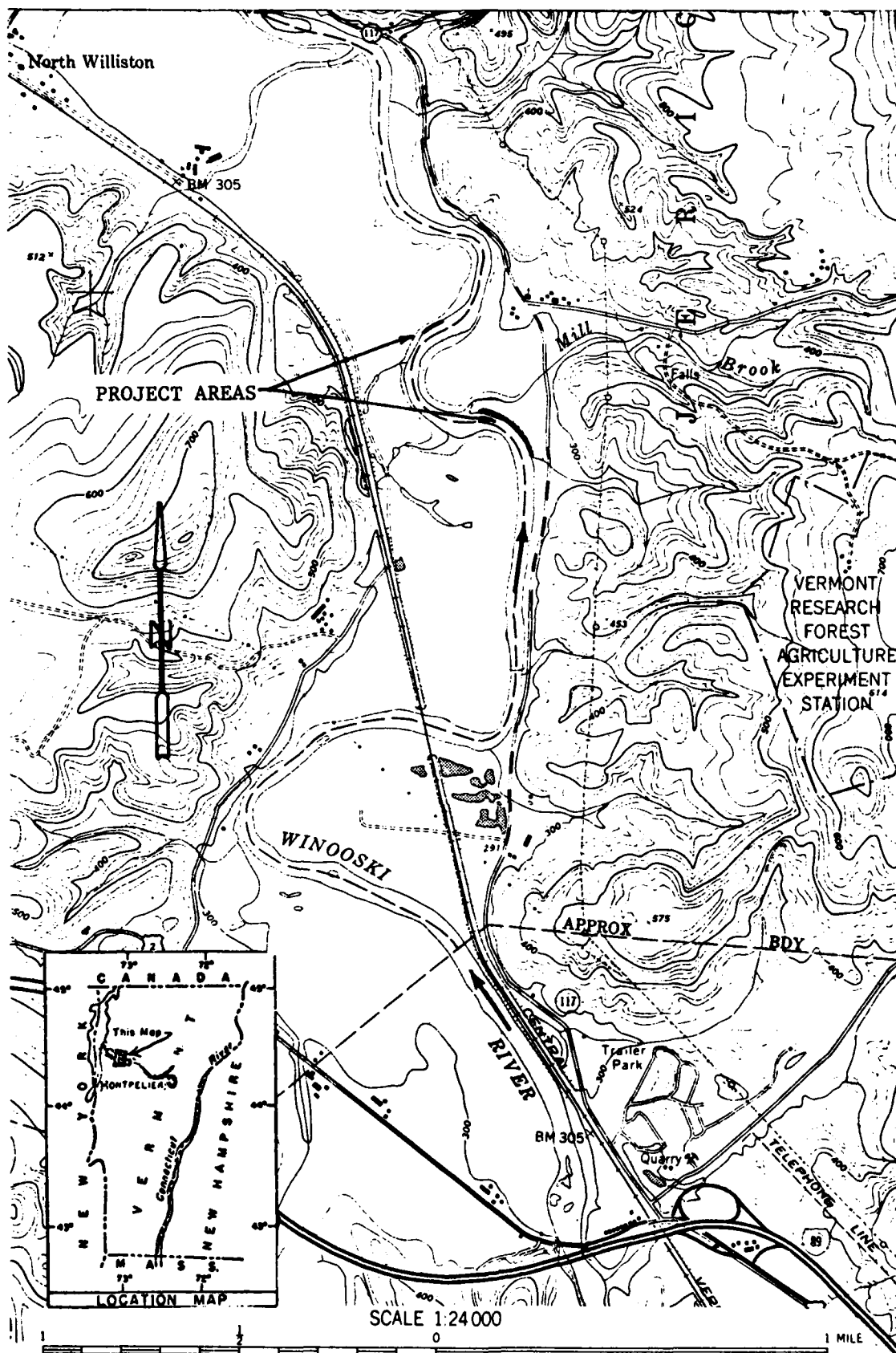


PLATE 1
ITEM 27-2

Project location

H-27-10

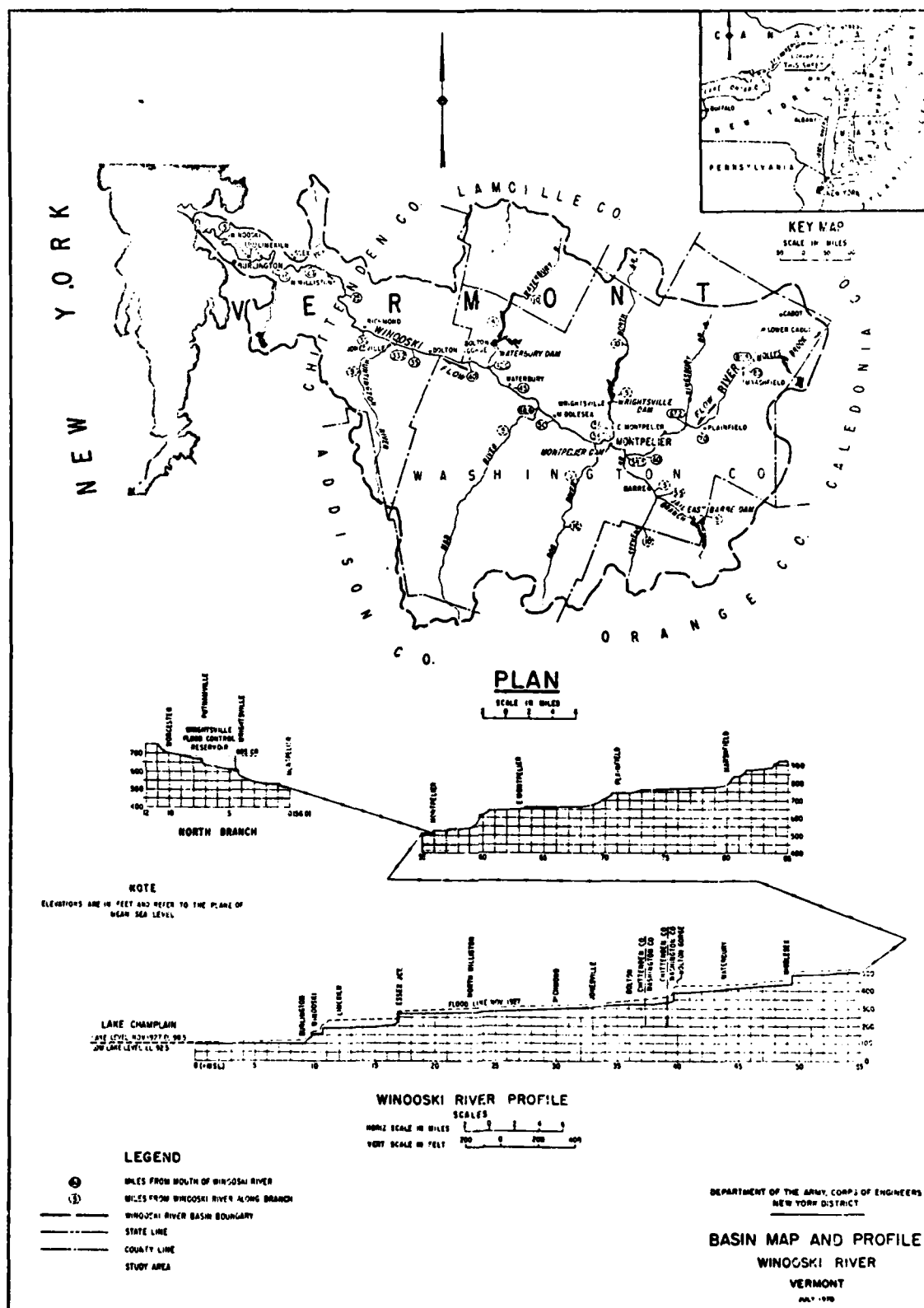


PLATE 2
ITEM 27-3

H-27-11

MAXIMUM AND MINIMUM STAGE AND DISCHARGE RECORDS AT U.S.G.S. STATIONS

Name of Station	Stream	Drainage Area (sq.mi.)	Period of Record	Datum of Gage	Date	Maximum		Minimum	
						Discharge (c.f.s.)	Gage Height (feet)	Discharge (c.f.s.)	Gage Height (feet)
Essex Junction	Winooski River	1044	1928-1954	---	4 Nov. 27	113,000	50.4	25 Sept. 37	70 ^(a) 0.1
Montpelier	Winooski River	397	1909-1923 1928-1954	499.59	3 Nov. 27	57,000	27.1	30 Sept. 21	6 ^(a) 2.58
Northfield Falls	Dog River	76.1	1934-1954	603.00	21 Sept. 38	9,750	11.53	7 Sept. 42	4.3 0.5
East Barre	Jail Branch	40.4 ^(b)	1920-1923 1933-1954	1,071.59	3 Nov. 27	11,500 ^(c)	---	18 Aug. 50	0.1 0.6
Wrightsville	North Branch	69.2	1934-1954	550.53	3 Nov. 27	17,200	---	13 Aug. 41	0.2 ^(a) 0.0
Waterbury	Waterbury River	111	1935-1954	428.00	3 Nov. 27	23,400 ^(c)	---	6 July 41	0.6 ^(a) 4.7
Moretown	Mad River	139	1928-1954	---	3 Nov. 27	23,000	19.4	1 Oct. 30	1.4 1.73

- (a) Minimum discharges are subject to regulation by reservoirs.
 (b) Includes 11.3 square miles diverted for East Barre Water Supply.
 (c) Estimated.

Winooski River near Essex Junction, Vt.

Location.—Lat 44°28'40", long 73°08'20", on right bank half a mile downstream from Muddy Brook and 2 miles southwest of Essex Junction, Chittenden County.

Drainage area.—1,044 sq mi.

Gage.—Recording. Altitude of gage is 185 ft (from topographic map).

Stage-discharge relation.—Defined by current-water measurements below 27,000 cfs and extended above on basis of indirect measurements at 32,600, 45,300, and 113,000 cfs.

Remarks.—Flow regulated by power plants above station, by Peacham Pond and Mollys Falls Reservoir, by Waterbury Reservoir since 1937, and by East Barre and Wrightsville Detention Reservoirs since 1935. Base for partial-duration series, 12,500 cfs.

Peak stages and discharges

Water year	Date	Gage height (feet)	Discharge (cfs)	Water year	Date	Gage height (feet)	Discharge cfs
1928	Nov. 4, 1927	50.4	113,000	1937	Apr. 7, 1937	11.10	18,200
					May 16, 1937	15.07	26,400
1929	Mar. 17, 1929	11.64	19,300				
	Mar. 23, 1929	9.90	15,500	1938	Mar. 24, 1938	12.53	20,000
	Apr. 6, 1929	9.80	15,300		Sept. 22, 1938	18.72	34,300
	Apr. 27, 1929	8.84	12,700				
1930	Jan. 9, 1930	12.60	21,300	1939	Dec. 7, 1938	10.06	16,200
	Mar. 27, 1930	8.78	12,700		Apr. 20, 1939	11.07	18,200
	Apr. 8, 1930	10.52	17,200		Apr. 23, 1939	11.00	18,000
1931	Apr. 11, 1931	13.22	22,600		Apr. 27, 1939	8.79	10,600
				1940	Apr. 13, 1940	8.30	12,700
1932	Apr. 9, 1932	10.49	16,900		May 3, 1940	13.38	22,800
	Apr. 13, 1932	13.68	23,600		May 29, 1940	8.31	12,700
1933	Oct. 7, 1932	8.83	13,600	1941	Dec. 30, 1940	11.82	
	Nov. 20, 1932	13.42	22,900		Apr. 14, 1941	7.90	11,900
	Apr. 8, 1933	11.97	20,000				
	May 4, 1933	8.40	12,900	1942	Apr. 16, 1942	13.15	22,400
					June 15, 1942	8.50	13,100
1934	Mar. 28, 1934	11.65	14,000	1943	May 12, 1943	8.35	12,900
	Apr. 3, 1934	11.70	19,400				
	Apr. 13, 1934	17.32	31,600	1944	Nov. 9, 1943	11.00	18,000
	Apr. 17, 1934	11.00	18,000		Apr. 11, 1944	10.78	17,600
1935	Jan. 10, 1935	16.96	30,900	1945	Mar. 18, 1945	9.50	13,500
					May 14, 1945	8.22	12,500
1936	Mar. 13, 1936	16.10	28,500	1946	Oct. 2, 1945	11.11	18,200
	Mar. 19, 1936	23.54	45,300		Mar. 10, 1946	12.80	21,600
	Mar. 22, 1936	12.57	21,200		Mar. 15, 1946	12.30	18,600
	Apr. 7, 1936	9.32	14,600				

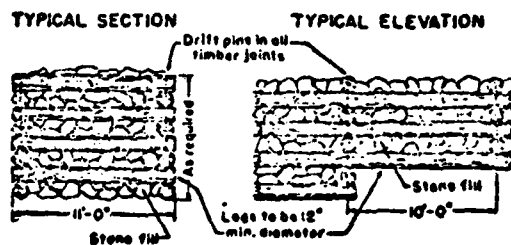
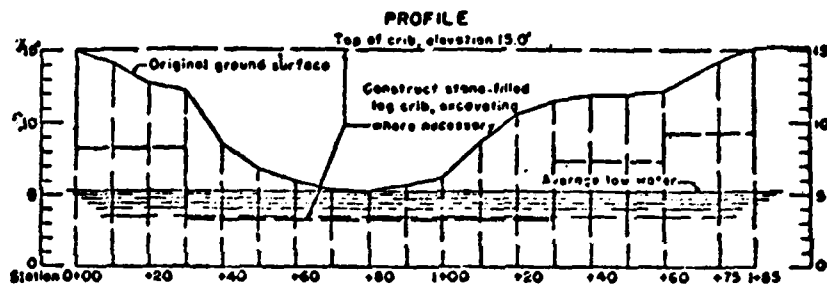
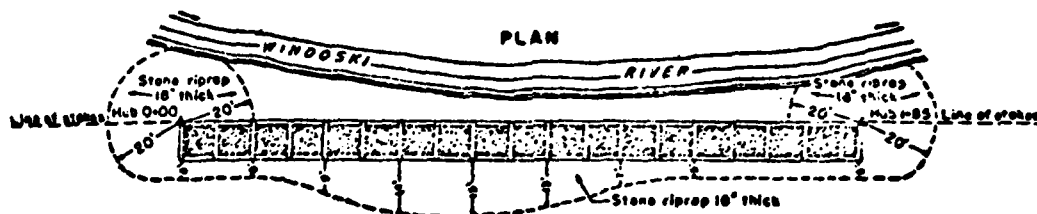
PLATE 4 (Sheet 1 of 2)
ITEM 27-5

Peak stages and discharges							
Water year	Date	Gage height (feet)	Discharge (cfs)	Water year	Date	Gage height (feet)	Discharge cfs
1947	Mar. 26, 1947	8.59	13,200	1959	Apr. 3, 1959	12.98	19,000
	Apr. 7, 1947	8.75	13,500		Apr. 7, 1959	8.06	13,000
	Apr. 12, 1947	12.44	20,900				
	May 30, 1947	10.50	17,000	1960	Nov. 28, 1959	14.00	24,100
	June 4, 1947	16.27	28,900		Apr. 1, 1960	11.71	19,400
					Apr. 5, 1960	10.98	18,000
1948	Mar. 22, 1948	10.08	16,200				
	Mar. 28, 1948	8.30	12,700	1961			
					Apr. 23, 1961	8.52	12,800
1949	Dec. 31, 1948	16.09	28,500		May 10, 1961	8.39	12,600
1950	Apr. 5, 1950	10.28	16,600	1962	Mar. 31, 1962	10.66	16,800
	Apr. 21, 1950	8.40	12,900		Apr. 8, 1962	11.83	19,200
1951	Dec. 5, 1950	11.17	18,300	1963	Apr. 4, 1963	10.28	16,100
1951	Mar. 31, 1951	8.86	13,800				
	Apr. 26, 1951	9.36	14,700	1964	Mar. 6, 1964	14.68	17,500
1952	Apr. 6, 1952	11.43	19,500	1965	Mar. 9, 1965	5.49	5,580
	June 2, 1952	14.16	24,900				
				1966	Mar. 25, 1966	8.03	10,300
1953	Dec. 12, 1952	8.71	14,000	1967	Apr. 3, 1967	12.60	18,700
	Mar. 25, 1953	11.80	19,600				
	Mar. 27, 1953	11.58	19,200	1968	Mar. 22, 1968	14.23	19,600
	Apr. 27, 1953	8.65	13,900				
				1969	Apr. 19, 1968	13.68	20,900
1954	Apr. 9, 1954	9.09	14,600				
	Apr. 12, 1954	8.02	13,000	1970	Apr. 25, 1970	13.52	20,500
1955	Apr. 7, 1955	8.75	14,100	1971	May 4, 1971	13.62	20,700
	Apr. 12, 1955	7.89	12,800				
	Apr. 15, 1955	9.36	15,100	1972	May 5, 1972	16.87	27,400
	Apr. 10, 1955	9.01	14,500				
				1973	July 1, 1973	17.47	28,700
1956	Apr. 17, 1956	8.03	13,000		Dec. 22, 1973	14.77	23,000
	Apr. 30, 1956	8.49	13,400				
	May 28, 1956	9.85	15,900	1975	Apr. 20, 1975	14.35	22,200
1957	Jan. 23, 1957	10.60	11,400	1976	Apr. 2, 1976	17.09	27,900
1958	Dec. 21, 1957	12.08	20,200	1977	Mar. 14, 1977	19.06	32,000
	Apr. 19, 1958	11.33	18,700		Oct. 17, 1977	14.06	21,600
	Apr. 22, 1958	13.03	22,100				
				1978	Jan. 10, 1978	13.17	19,800
				1979	Mar. 6, 1979	15.71	24,800

PLATE 4 (Sheet 2 of 2)
ITEM 27-5

H-27-14

TYPICAL DETAIL OF STONE-FILLED LOG CRIB



GENERAL NOTES

River side of crib and chute behind crib and around
ends to be planted.

Stone riprap 18" thick to be placed within broken lines as shown on plan.

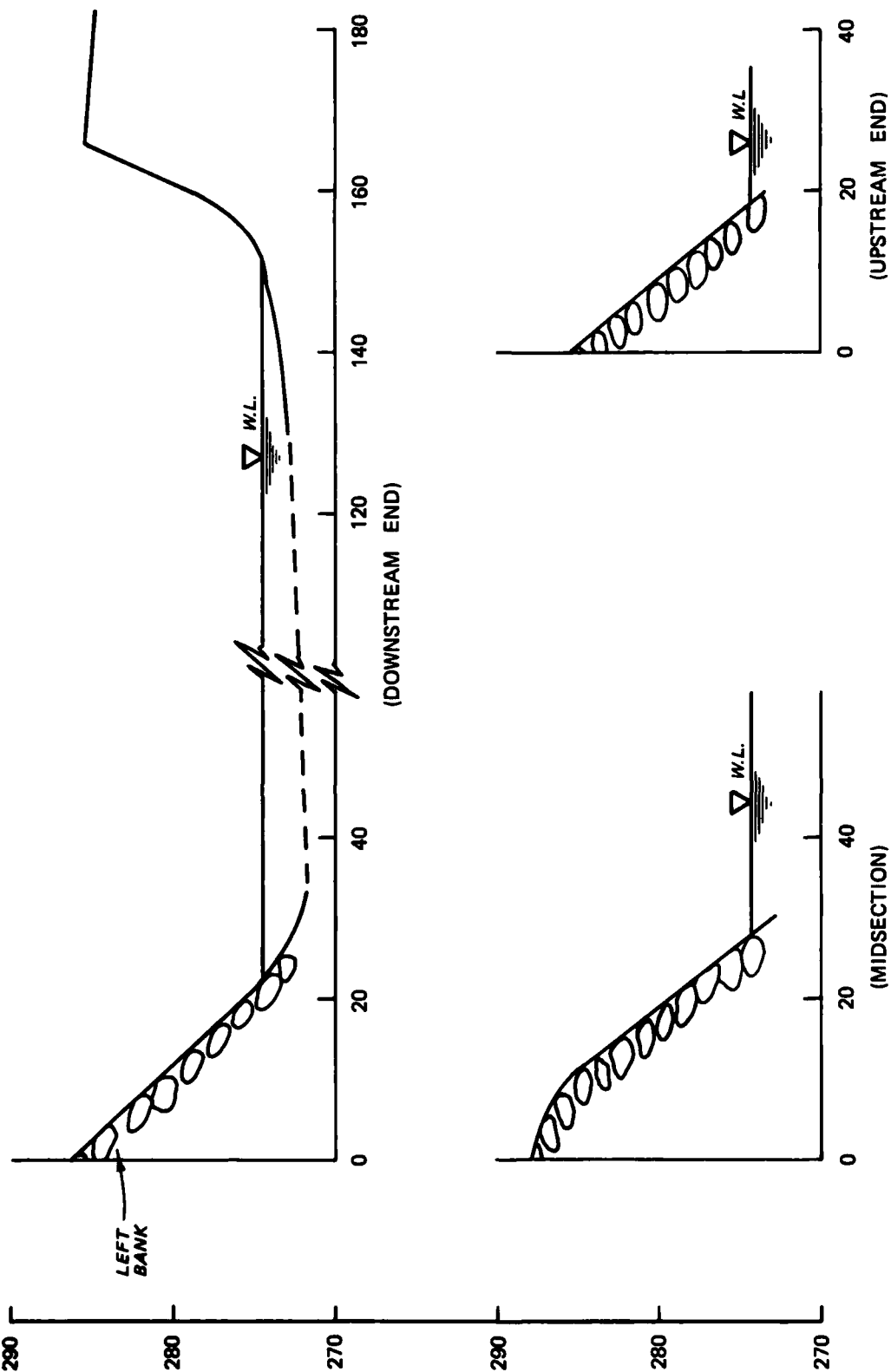
Earth backfill around cribbing to be tamped
in 6" layers.

Excess excavation to be placed on river side of crib for planting.

PLATE 5
ITEM 27-6

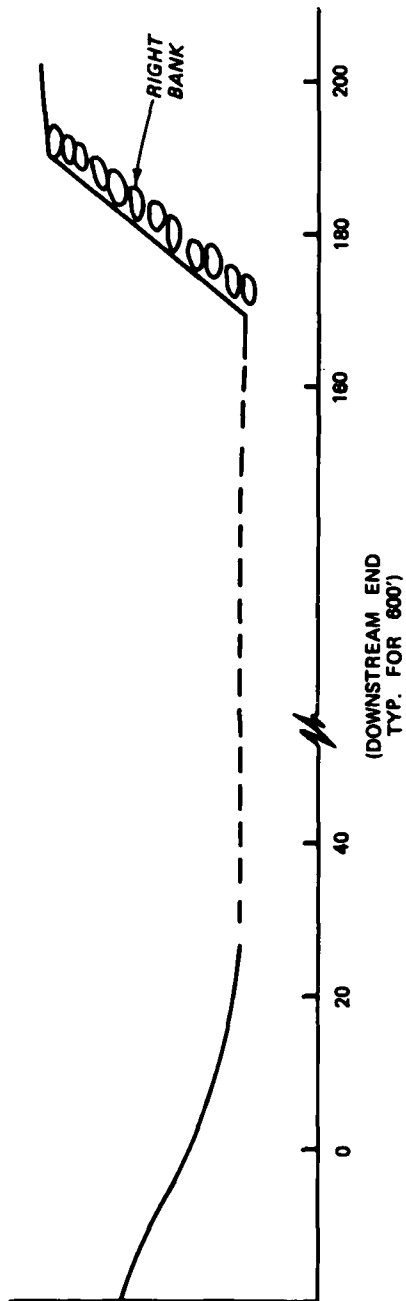
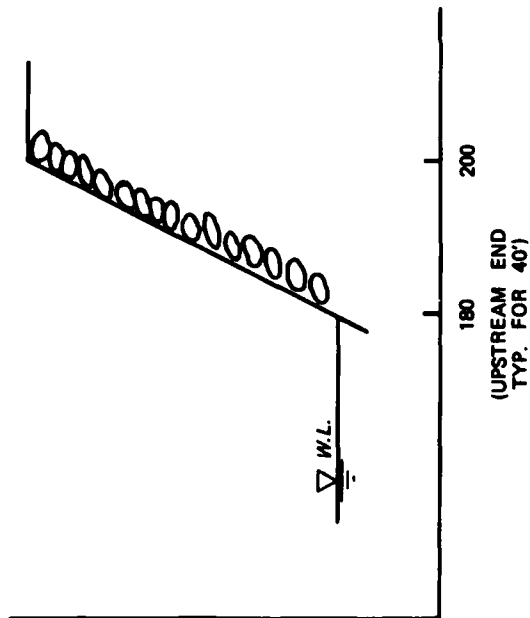
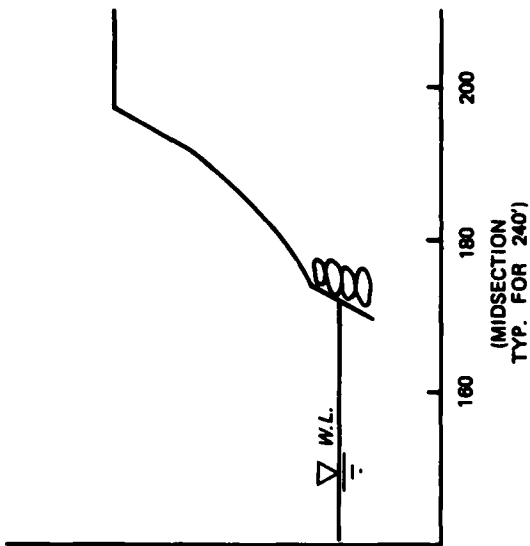
PLATE 6
ITEM 27-7

H-27-16



TYPICAL SECTIONS DOWNSTREAM SITE
SURVEYED 10/8/80

290 280 270 260 250



TYPICAL SECTIONS UPSTREAM SITE
SURVEYED 10/8/80

PLATE 7
ITEM 27-8

PLATE 8
ITEM 27-9



Right bank of the upstream site. Toward the left of the photo is the confluence of Mill Brook. A few of the stones, remnant of the log cribbing, are visible on the bank.

H-27-18



Left bank of the downstream site. Toward the left of the photo, exposed on the surface of the water, are two concrete blocks which are probably not part of the erosion control measures. There was a streambank failure behind the concrete blocks before the streambank control structure was built.

H-27-19

PLATE 9
ITEM 27-10

**ST MARY'S RIVER
MISSION POINT, MICHIGAN**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream St. Marys River River Mile 3 Side Right
Local Vicinity Rotary Island Lat N46°29' Long W84°20'
At/Nr City Sault Ste. Marie County Chippewa State MI Cong Dist 11
CE Office Symbol NCE Responsible Agency Detroit District, C.O.E.
Site Map Sources USGS Sault Ste. Marie East 7.5' quadrangle
Land Use Recreational park facility

(2) Hydrology at or Near Site

Stage Range 578.77 to 582.4 ft; Period of Record 1968 to 1980
Discharge Range 41,000 to 118,000 cfs; Velocity Range 1.5 to fps
Sediment Range - to - tpd; Period of Record 19-- to 19--
Bank-full Stage 582.4 ft; Flow 118,000 cfs; Average Recurrence Interval - yr
Bank-full Flow Velocity: Average 2.0 fps; Near Bank fps
Comments Discharge controlled by Compensating Works. Low Water Datum at Mission Point is 577.6 ft IGLD (1955).

(3) Geology and Soil Properties

Bank (USCS) Silt loam Bed (USCS) Silt loam
Data Sources Visual observation 26 Sept 1980
Groundwater Bank Seepage None
Overbank Drainage Not significant as regards erosion
Comments The site is relatively flat, low-lying and vegetated to the bank. There is a 1 to 2 foot high scarp at the banks, between the bank and the riprap which probably formed due to pre-protection erosion, but is now stabilized with grass.

(4) Construction of Protection

Need for Protection Recreational island subjected to high waves due to wind and commercial vessels.
Erosion Causative Agents Wave action, as stated above
Protection Techniques Shaping banks, placement of limestone riprap.
General Design Riprap revetment, 500 cubic yards of blasted limestone. No other design data available.
Project Length 495 ft; Construction Cost \$ 4,000 Mo/Yr Completed 1974

(5) Maintenance

Experienced Flows (Stage, cfs, Date) No data available

Repairs and Costs (Item, Cost, Date) None

Comments: No repairs have been necessary nor are repairs necessary now.

(6) Performance Observations and Summary

Monitoring Program Visual observations and photographs

Documentation Sources Photographs, 1975, 1980.

Project Effect on Stream Regime Insignificant

Project Effect on Environment Insignificant

Successful Aspects Prevents or retards streambank erosion due to ship and wind waves.

Unsuccessful Aspects None

General Evaluation Appears to be little change since installation; project performs to meet needs and expectations.

Recommendations No changes necessary. Other eroding areas in the vicinity may benefit from similar projects.

(7) Additional Information, Comments, and Summary

Map No. 28. See attached item 28-1*

Attached items:

28-1 Project summary & location

28-2 Project photograph

28-3 Project photograph

28-4 Project photographs

28-5 Project photographs

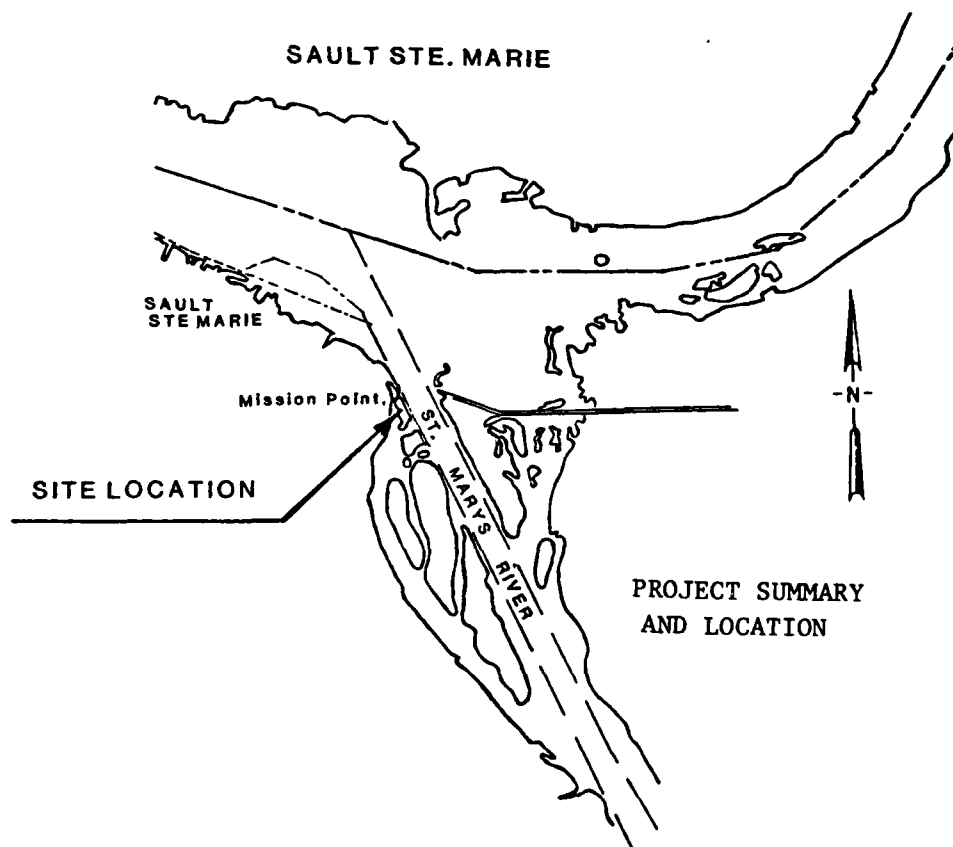
MISSION POINT SITE

The site is located on the St. Marys River approximately 3 miles downstream from the Sault Ste. Marie Compensating Works, which controls flow past the site. In 1974, limestone riprap was placed along the bank at Rotary Island Park, which is subjected to high waves generated by wind and by commercial vessels.

The protection has performed very satisfactorily. The riprap is stable and bank erosion is negligible. Nearby unprotected areas continue to erode. No repair or maintenance is necessary, but periodic monitoring, including photographs and visual observations, should be continued.

*The site was observed on relatively windy days and during ship passage. Ship waves appeared to be 1/2 foot or less in height, which can be considered a typical value since the speed of commercial vessels in the channel is regulated by the U.S. Coast Guard. Vessels and other craft violating regulations could cause larger waves which erode the banks. Observed waves did not come within 2 to 3 feet of the bank. The toe of the bank is vegetated. River stage on 26 September 1980 was about 580.4 feet IGLD (1955). The bank immediately downstream of the protected area is eroding.

Any settling of the revetment should be complete now. Slope stability is not a problem, since the riprap slope is gentle. Some of the finer gravel included with the riprap may occasionally wash away, but, overall, erosion of the revetment is insignificant.



ITEM 28-1



Aerial photo of St. Marys Project (27 June 1978)

ITEM 28-2

H-28-4



St. Marys River Project, 4 years after construction, 27 June 1978

ITEM 28-3

H-28-5



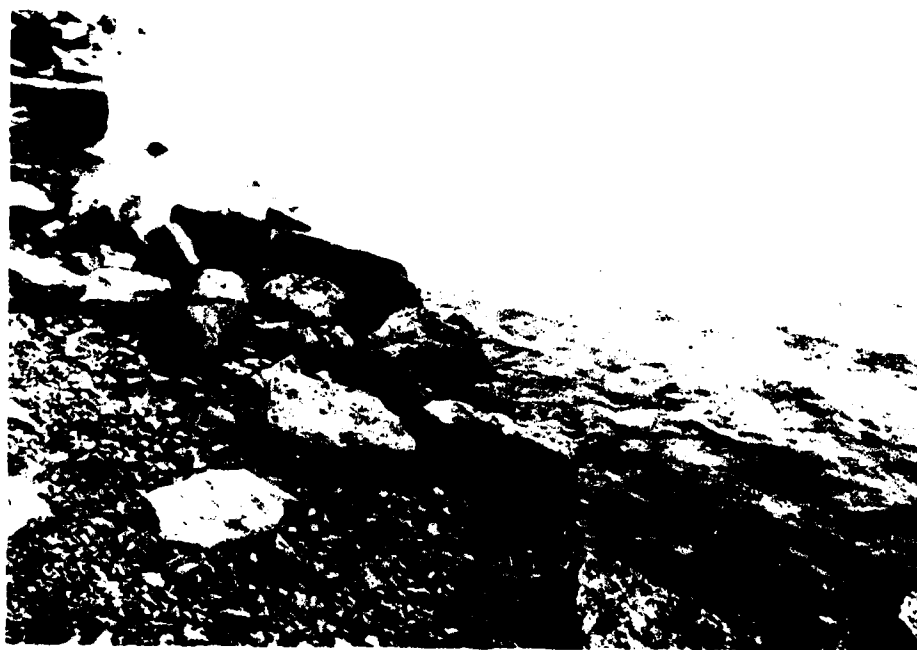
Examples of typical wave-surge conditions on
the St. Marys River - 1980

ITEM 28-4

H-28-6



Looking downstream (south) - 1978



Closeup view of riprap material - 1980

ITEM 28-5

H-28-7

**ILLINOIS WATERWAY
BANNER LEVEE, ILLINOIS**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Illinois River River Mile 144.2 to 145.4 Side Right
Local Vicinity Banner Levee near Peoria, Ill. Lat N 40° 31' Long W 89° 46'
At/Nr City Kingston Mines County Peoria State IL Cong Dist 18
CE Office Symbol NCR Responsible Agency Rock Island District, Corps
Site Map Sources of Engineers
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range 1.7 to 26.02 ft; Period of Record 19 39 to 19 81
Discharge Range 1,810 to 83,100 cfs; Velocity Range 0.6 to 3.54 fps
Sediment Range _____ to _____ tpd; Period of Record 19 _____ to 19 _____
Bank-full Stage 20.0 ft; Flow 48,390 cfs; Average Recurrence Interval 2.8 yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) Clay Bed (USCS) Silt and sand
Data Sources Visual and levee design
Groundwater Bank Seepage None observed
Overbank Drainage Insignificant - from top of levee only
Comments _____

(4) Construction of Protection

Need for Protection The revetment protects approx. 400 acres of agricultural land behind the levee and a Farm Supply Company tank farm.
Erosion Causative Agents Wave damages were caused by high winds and increase in size and number of barges and tows.
Protection Techniques Bank replacement, bedding and riprap.
General Design The damaged levee was repaired by placing new compacted fill material, bedding, and riprap on the prepared, sloped earth foundation. The levee top was seeded.
Project Length 11.790 ft; Construction Cost \$ 188 k Mo/Yr Completed 1976

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Top of riprap, 25.22 feet; 72,300 cfs;
24 March 1980.

Repairs and Costs (Item, Cost, Data) None

Comments: The levee slope and riprap are in good condition. There are no
indications that maintenance work will be required in the foreseeable
future.

(6) Performance Observations and Summary

Monitoring Program Visual site observation and photographs

Documentation Sources Photos - 1976, 1980

Project Effect on Stream Regime Negligible

Project Effect on Environment Negligible

Successful Aspects Protection has been effective

Unsuccessful Aspects None observed

General Evaluation Project looks good

Recommendations Similar protection recommended for other sites with wind
and navigation wave damage.

(7) Additional Information, Comments, and Summary

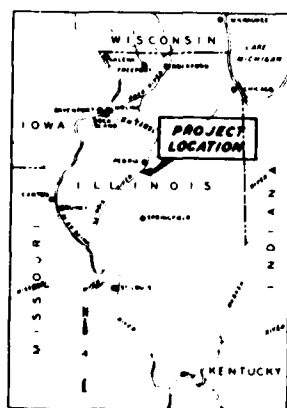
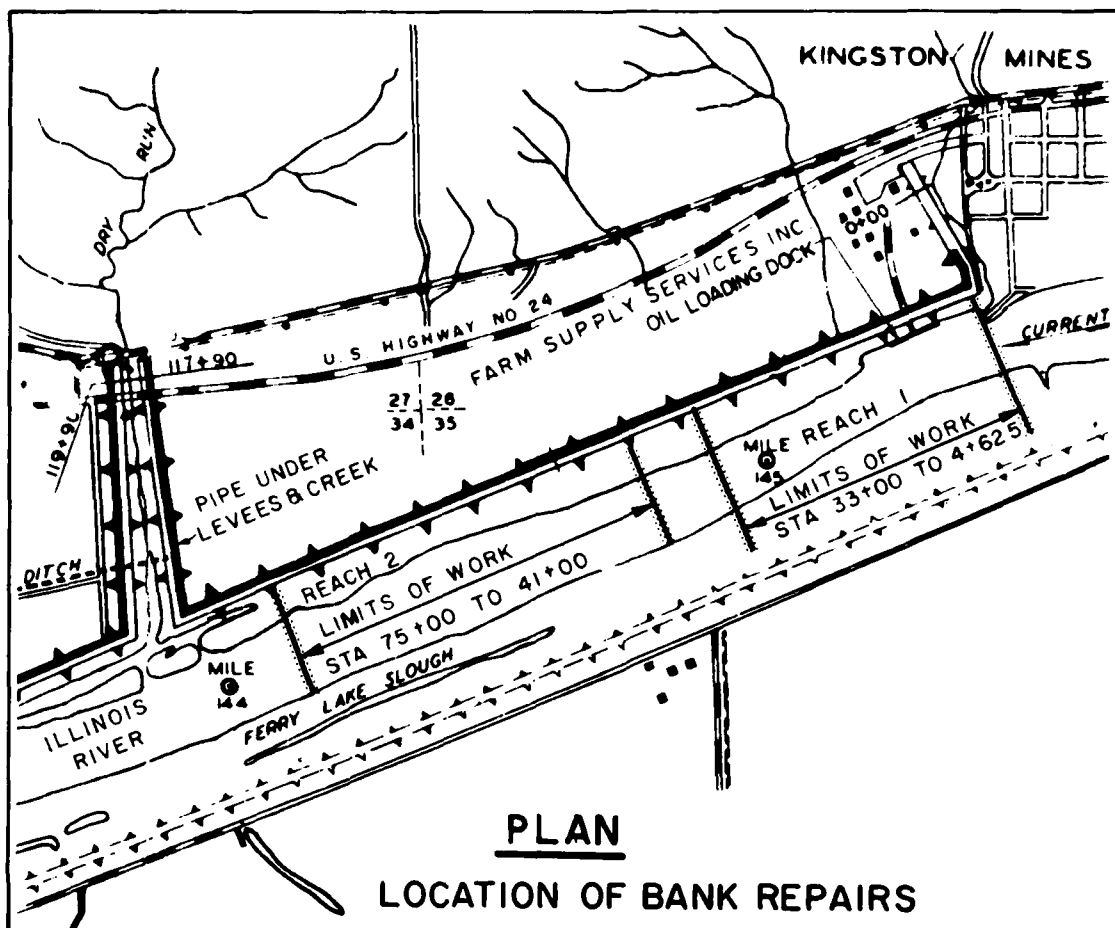
Map No. 29. The bank at river's edge is on a flatter slope than the rip-
rap and is armored with coarse gravel to small cobbles, and covered with
silt. There has been no regrowth of woody vegetation on the bank. There
is a barge docking facility for petroleum products or liquid farm
chemicals at the site.

Attached Items.

29 - 1 - Project plan and general location

29 - 2 - Typical cross section

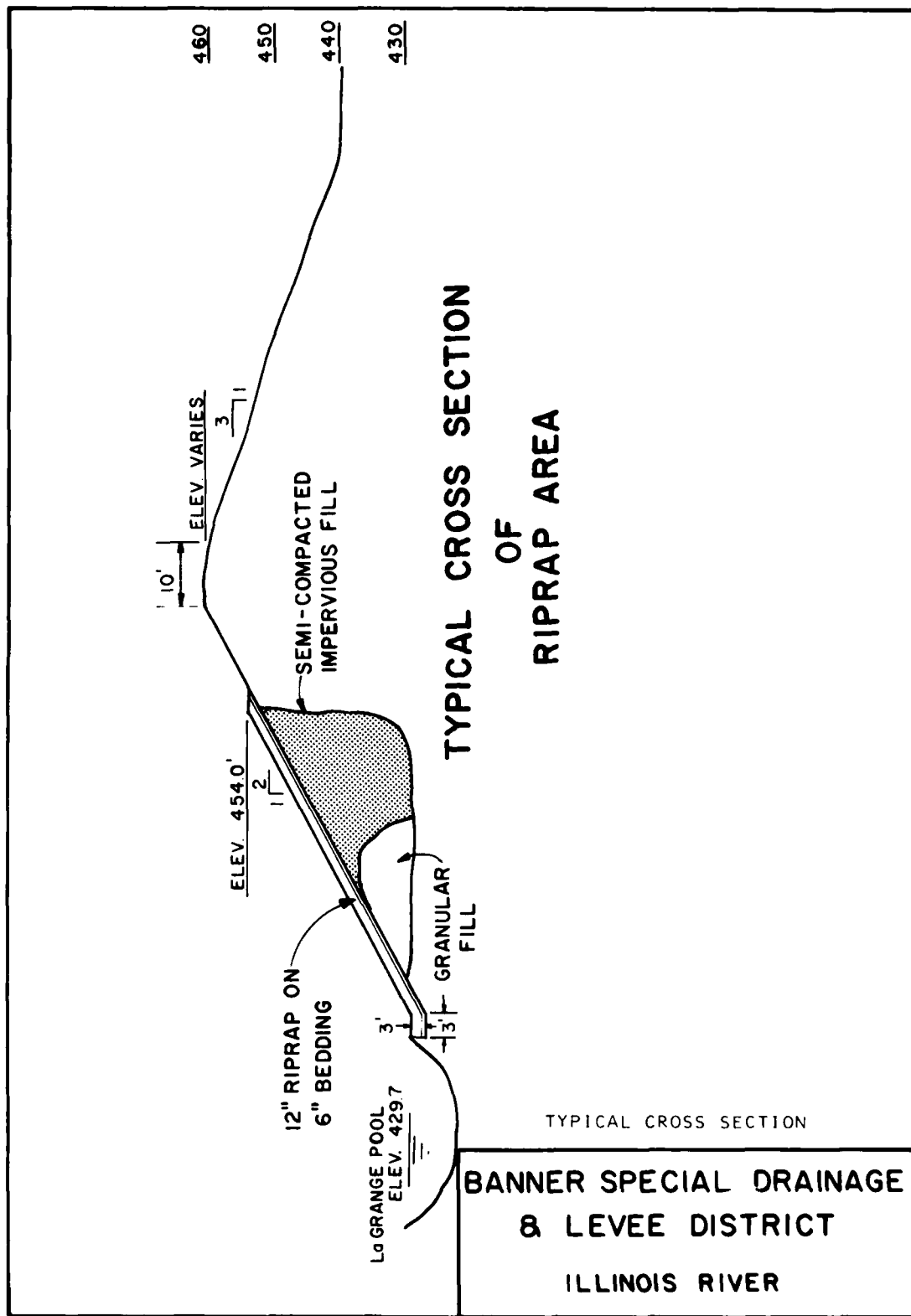
29 - 3 & 4 - Photos: before construction; after construction; after
substantial flow.



PROJECT PLAN AND LOCATION

**BANNER SPECIAL DRAINAGE
& LEVEE DISTRICT
ILLINOIS RIVER**

ITEM 29-1



ITEM 29-2

H-29-4



**JUNE 1976 - LOOKING DOWNSTREAM. PHOTO SHOWS
SEVERE WAVE WASH AND SCOUR
DAMAGES TO EXISTING LEVEE.**

PHOTOGRAPH BEFORE
CONSTRUCTION

**BANNER SPECIAL
DRAINAGE AND
LEVEE DISTRICT
ILLINOIS RIVER**

ITEM 29-3



**SEP. 1976 - LOOKING UPSTREAM IMMEDIATELY AFTER
LEVEE REPAIR.**



**OCT. 1980 - LOOKING DOWNSTREAM POST
CONSTRUCTION. PHOTO SHOWS
LEVEE SLOPE AND RIPRAP
ARE IN GOOD CONDITION.**

POST CONSTRUCTION
PHOTOGRAPHS

**BANNER SPECIAL
DRAINAGE AND
LEVEE DISTRICT
ILLINOIS RIVER**

ITEM 29-4

H-29-6

**BUREAU CREEK
BUREAU COUNTY, ILLINOIS**

**Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2**

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream West Bureau Creek River Mile _____ Side Left
Local Vicinity Near I&M Canal, Locks 10&13, IL Lat N41°20' Long W80°32'
At/Nr City Tiskilwa County Bureau State IL Cong Dist 18
CE Office Symbol NCR Responsible Agency Rock Island District, COE
Site Map Sources _____
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range 3.9 to 12.3 ft. Period of Record 19 36 to 19 81
Discharge Range 200 to 10,000 cfs; Velocity Range 0 to 10 fps
Sediment Range _____ to _____ tpd; Period of Record 19 _____ to 19 _____
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments Bed gradient 7'0"/mile

(3) Geology and Soil Properties

Bank (USCS) Clay borrow Bed (USCS) _____
Data Sources _____
Groundwater Bank Seepage Minor or none; assumed not to be a major contributor
to bank failure
Overbank Drainage Minor, unless the canal bank fails.
Comments _____

(4) Construction of Protection

Need for Protection See Attached Item 30-1*
Erosion Causative Agents Spring flooding. The failed bank is on the outside
of a curve in Bureau Creek.
Protection Techniques Rebuilt bank and installed steel jacks.
General Design 30 steel jacks at 10' spacing in 3 rows 15' apart.
Project Length _____ ft; Construction Cost \$ 7,000 Mo/Yr Completed Jun 74

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 9.80 ft; 3,450 cfs; 20 March 1979

Repairs and Costs (Item, Cost, Date) No repairs to the system were required or made while the project was still a Corps of Engineers responsibility.

Comments: The I&M Canal project has been turned over to the State of Illinois

(6) Performance Observations and Summary

Monitoring Program One yearly inspection

Documentation Sources Photos

Project Effect on Stream Regime Minor

Project Effect on Environment Unknown

Successful Aspects The very upstream end of the jack system has an established woody growth and the bank is stable. This may be partly due to two old car bodies.

Unsuccessful Aspects The bank has failed in the middle and downstream portion of the project. Erosion of the bank extends downstream past the end of the jacks.

General Evaluation See Attached Item 30-1**

Recommendations This site is not under the control of the COE and no repair work or maintenance work is anticipated by the Corps.

(7) Additional Information, Comments, and Summary

Map No. 30. See Attached Item 30-1***

Attached Items:

30-1	Project summary	30-6	Project photographs Lock 13
30-2	Project location	30-7	Project photographs Lock 10
30-3	Project plan		
30-4	Project photographs Lock 13		
30-5	"		

Bureau Creek, Bureau County, Illinois

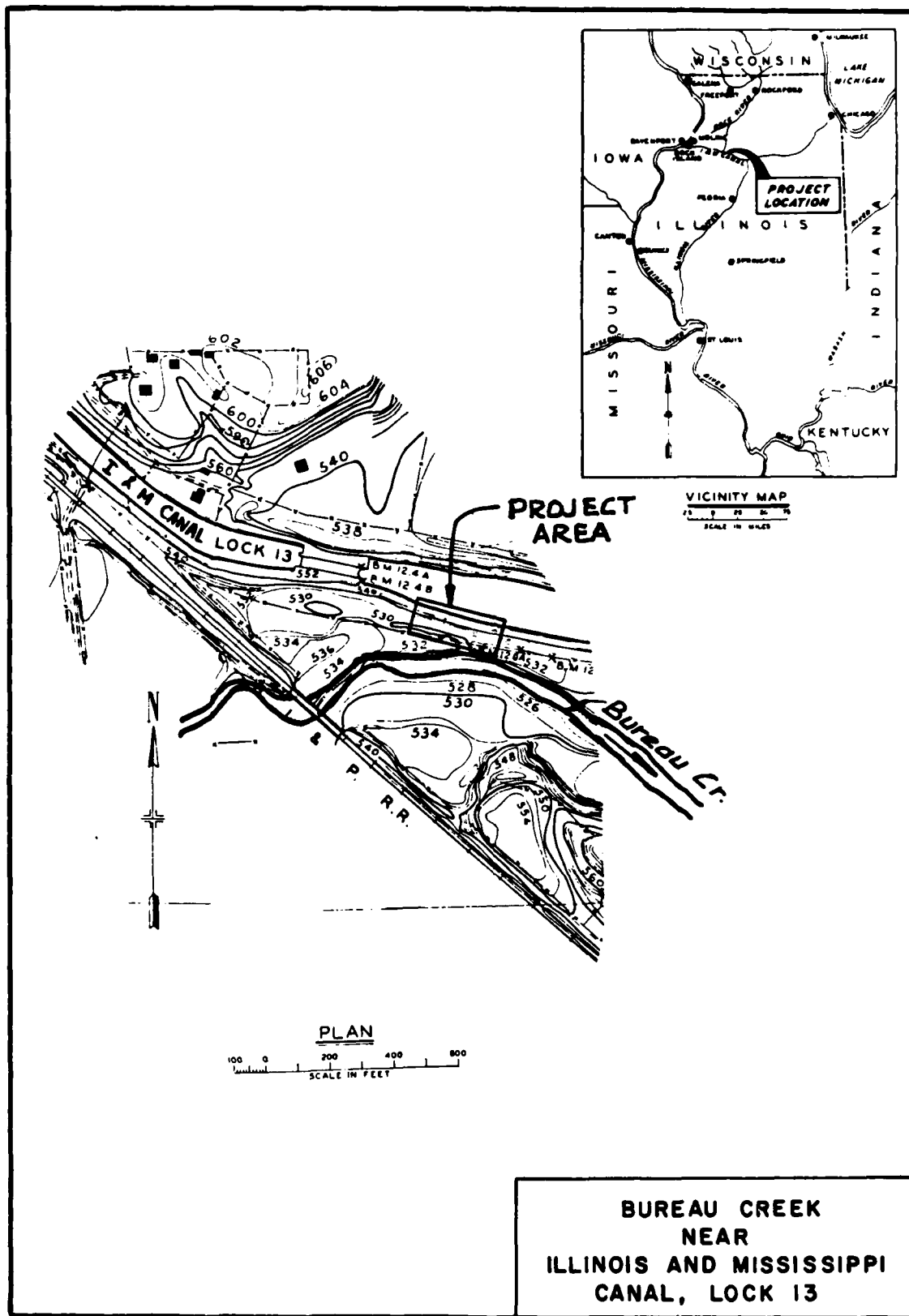
Steel jacks were placed at two locations on Bureau Creek in late 1973 and early 1974. At one location (vicinity Lock No. 10, I&M Canal), where two rows containing 133 jacks were installed, a subsequent location modification was necessary. At the other location (vicinity Lock 13 I&M Canal), where 3 rows of 30 jacks were installed, no modification was necessary. The jacks were effectively turning the flow and protecting the bank at the upstream end of the jack system with woody growth being established on the stable bank. The downstream end of the jack field has drifted out into the stream. Bank failure requiring some maintenance has occurred downstream past the end of the jack system.

*The I&M Canal and Bureau Creek are adjacent to each other at this location. The I&M Canal bottom is higher than the bottom elevation of the creek. During the spring flood of 1974, the bank between the canal and creek failed. This caused the canal water to flow into Bureau Creek.

**The jacks are in place on the upstream end of this site. They are collecting some trash and sediment, and there is a woody growth coming up in the upstream end of the field. In the middle part of the jack field, Bureau Creek is now flowing on both sides of the jacks. These jacks are collecting brush and dead trees, and low flows would not significantly overtop them. The downstream end of the jack field has drifted out into the stream. It is submerged by low flows. The sloped bank behind the jack system at the downstream end has failed and parts of the bank are now vertical. The bank failure has extended downstream past the end of the jack system. There is no evidence that trees were growing on the failed portion of the repaired bank.

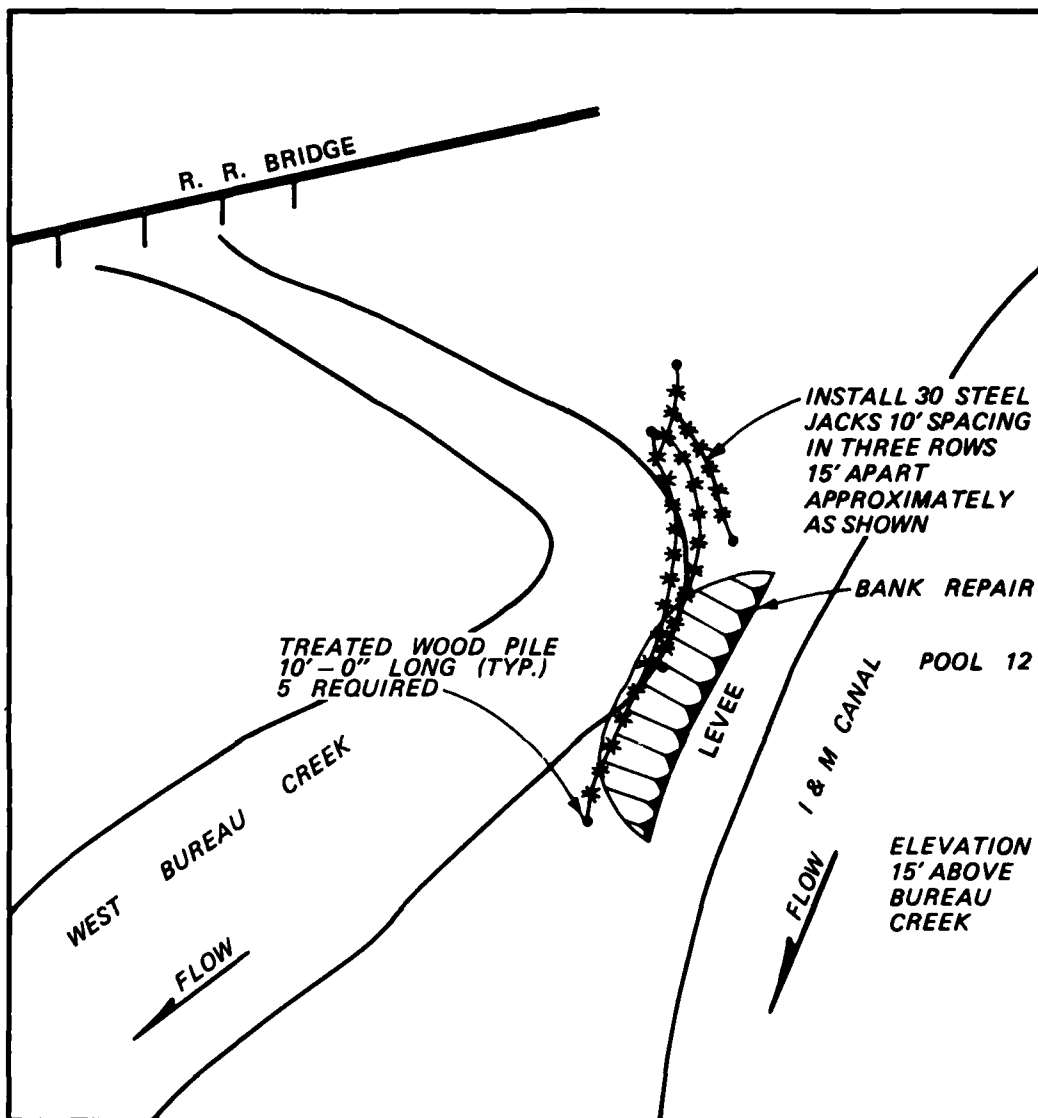
***The design philosophy was to redirect Bureau Creek with the jack system. The upstream end appears to be effective in turning the stream; however, it appears that the jack system should have been extended farther downstream. The bank failure appears to be occurring in the middle and downstream portion of the jack system. Some consideration should be given to planting an appropriate tree seedling in the jack field as a part of construction.

ITEM 30-1



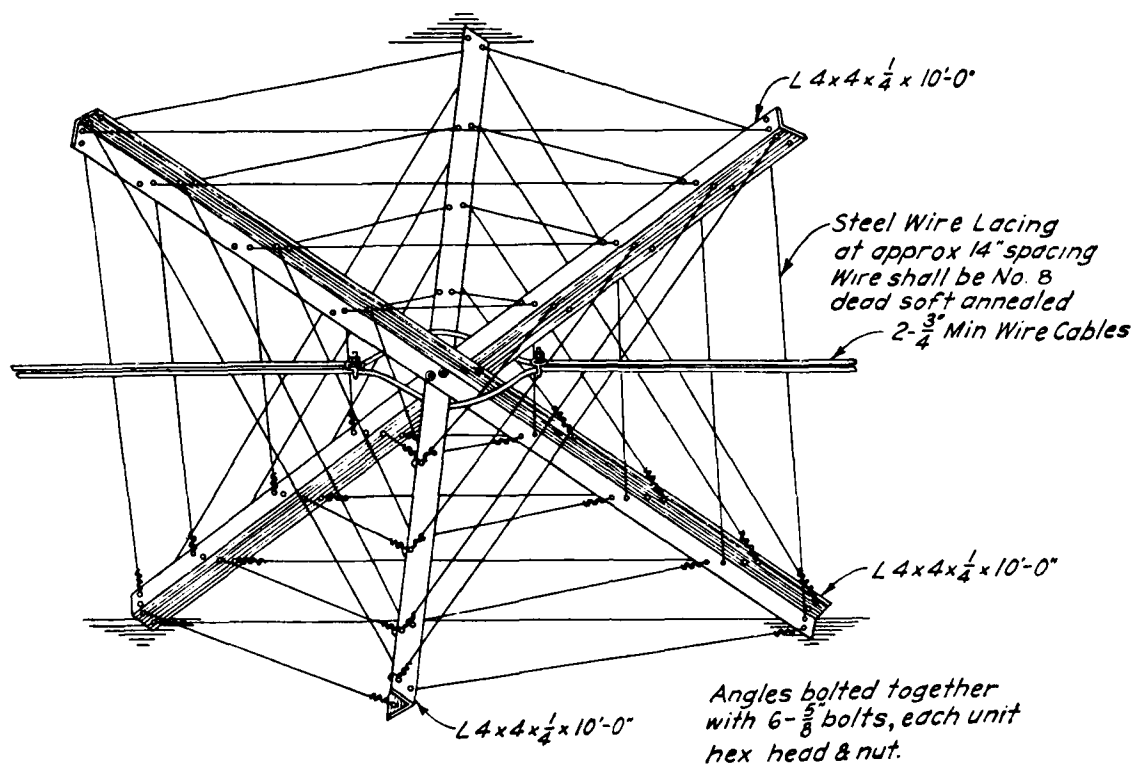
ITEM 30-2

H-30-4



BUREAU CREEK
NEAR
ILLINOIS AND MISSISSIPPI
CANAL, LOCK 13

ITEM 30-3a



STEEL JACK UNIT

NO SCALE

Construction diagram of jack used in Kellner Jack Fields on Bureau Creek near Locks 10 and 13 of the Illinois-Mississippi Waterway. Source: Illinois and Mississippi Canal - Repair of structures, sheet 5 of 7, NCR

ITEM 30-3b

H-30-6



MARCH 1974 - AERIAL VIEW OF ERODED
CREEK BANK DURING
CONSTRUCTION.



JULY 1976 - LOOKING UPSTREAM AT REPAIRED
CREEK BANK.

BUREAU CREEK
NEAR
ILLINOIS AND MISSISSIPPI
CANAL, LOCK 13

ITEM 30-4



Kellner Jack Field near Lock 13 of I&M Canal



Kellner Jack Field near Lock 13 of I&M Canal.
Road in center is along the top of the levee;
the canal is in the left portion of the figure

LOOKING DOWNSTREAM AT BUREAU CREEK - 1977

ITEM 30-5

H-30-8



OCT. 1980 - LOOKING UPSTREAM, PHOTO SHOWS
MIDDLE¹ AND DOWNSTREAM² SECTIONS OF THE
JACK FIELD HAS DRIFTED OUT INTO STREAM.

BUREAU CREEK
NEAR
ILLINOIS AND MISSISSIPPI
CANAL, LOCK 13

ITEM 30-6

H-30-9



OCT. 1980 - LOOKING UPSTREAM, PHOTO SHOWS
MIDDLE¹ AND DOWNSTREAM² SECTIONS
OF THE JACK FIELD HAS DRIFTED OUT
INTO STREAM.

BUREAU CREEK
NEAR
ILLINOIS AND MISSISSIPPI
CANAL, LOCK 10

ITEM 30-7

H-30-10

**IOWA RIVER
LOUISA COUNTY, IOWA**

**Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2**

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Iowa River River Mile 33.5 Side Left
Local Vicinity Southeast Iowa Lat N41°10' Long W91°22'
At/Nr City Columbus Junction County Louisa State IA Cong Dist 1
CE Office Symbol NCRED-PB-SS Responsible Agency Rock Island District, COE
Site Map Sources _____
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range 2.8 to 18.97 ft; Period of Record 19 56 to 19 81.
Discharge Range 69 to 35,700 cfs; Velocity Range ^{No info} avail- to _____ fps
Sediment Range *0.9 to 177,000 pd; Period of Record 19 _____ to 19 _____.
Bank-full Stage 15.0 ft; Flow 14,000 cfs; Average Recurrence Interval 5.1 yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments *Sediment concentrations of 2 mg/l to 7,800 mg/l were recorded at Iowa City

(3) Geology and Soil Properties

Bank (USCS) Mixed alluvium Bed (USCS) Same as banks
Data Sources _____
Groundwater Bank Seepage None observed
Overbank Drainage Minor
Comments _____

(4) Construction of Protection

Need for Protection To protect a pipeline on the Iowa riverbank from exposure due to bank erosion
Erosion Causative Agents Pipeline construction. The jetties were placed concurrent with the pipeline to prevent bank failure in the disturbed area.
Protection Techniques Seven permeable timber jetties on 50' centers
General Design The jetties were designed and constructed by private interests (pipeline company).
Project Length 300 ft; Construction Cost \$ Not available Mo/Yr Completed 1976

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 17.01; 22,200 cfs; 20 March 1979

Repairs and Costs (Item, Cost, Date) In May 76 about 20 to 30 ft. of the bank
between the land side of the jetty and the riverbank was eroded. The
repairs amounted to adding another panel to the jetty. Maintenance is
the responsibility of private interests (pipelines company).

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Yearly visual inspection

Documentation Sources District Section 32 photo files

Project Effect on Stream Regime Minor

Project Effect on Environment Negligible.

Successful Aspects To date the bank has not been protected from erosion.

Some bank buildup was observed.

Unsuccessful Aspects The river is changing its angle of attack on the jetties
and appears to be eroding away the built up material.

General Evaluation Protection has not been effective. As the thalweg moves the
effectiveness of the protection changes.

Recommendations Should not attempt to protect highly erodible non-
cohesive banks with permeable timber jetties.

(7) Additional Information, Comments, and Summary

Map No. 31. See Plate 4 for Project Summary

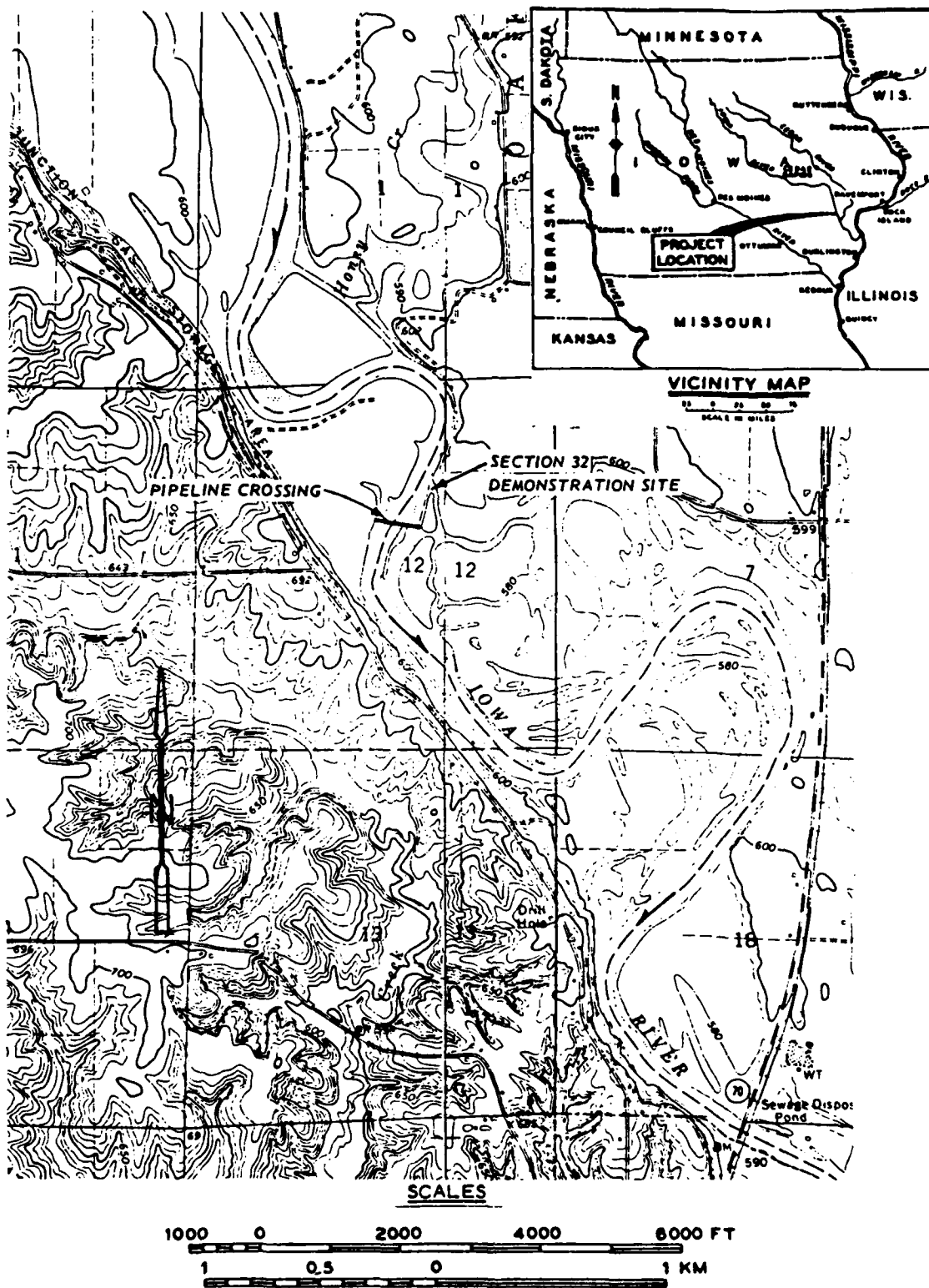
Attached Items.

31 - 1 - Project Location

31 - 2 - Project Plan

31 - 3 - 1976 Photographs

31 - 4 - Project Summary and 1979 Photograph



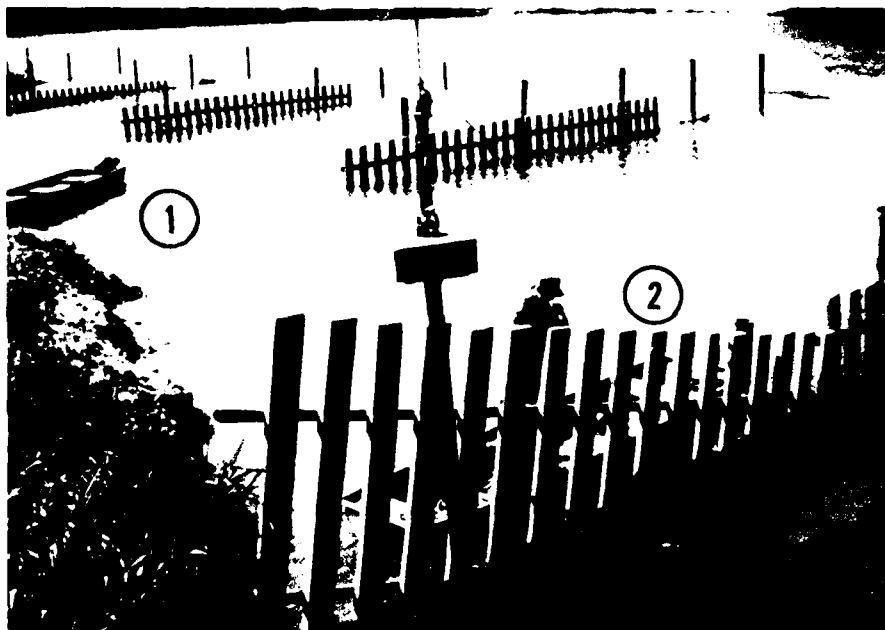
PROJECT LOCATION

IOWA RIVER

NORTH OF COLUMBUS JUNCTION, IA.

ITEM 31-1

H-31-3



May 1976 - Looking downstream at timbers during repairs.
Photo shows erosion¹ and placement of additional panel²



Sept 1976 - Looking downstream at timber jetties after repairs

1976 PHOTOGRAPHS
IOWA RIVER NORTH OF COLUMBUS JUNCTION, IA.

ITEM 31-3

H-31-5



In 1973, the National Gas Pipeline Co. of America (NGPCA) constructed a pipeline across the Iowa River. The disturbed bank was filled with sand taken from a sandy island on the Iowa River and seeded with rye grass in 1974; however, the attempt at bank stabilization was unsuccessful. Permeable timber jetties were installed in 1975. In May 1976, about 20 to 30 ft of bank between the landward end of the jetty system and the riverbank was eroded, so additional panels were placed landward of the system. At the time of the May 1977 inspection, the system had not failed; however, by late 1979, when more caving had occurred, the decision was made to remove the jetties and revet the bank with stone riprap.

PROJECT SUMMARY

1979 Photograph - North of Columbus Junction, Ia.

ITEM 31-4

H-31-6

**MINNESOTA RIVER
SAVAGE, MINNESOTA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Minnesota River River Mile 13.4 Side Right
Local Vicinity South-central Minnesota Lat N44°45' Long W 93°20'
At/Nr City Savage County Scott State MN Cong Dist _____
CE Office Symbol NCS Responsible Agency St. Paul District, Corps of
Site Map Sources _____
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range 687.2 to 719.5 ft; Period of Record 1934 to 1978
Discharge Range 79 to 117,000 cfs; Velocity Range 0.03 to 5.2 fps
Sediment Range 137 to 11,200 tpd; Period of Record 1977 to 1978
Bank-full Stage 700.0 ft; Flow 7,000 cfs; Average Recurrence Interval 2 yr
Bank-full Flow Velocity: Average 1.1 fps; Near Bank 1.8 fps
Comments On bend

(3) Geology and Soil Properties

Bank (USCS) Clay with silt & fine sand Bed (USCS) Same as banks
Data Sources Visual
Groundwater Bank Seepage Fairly heavy between water surface and 3 ft
above water surface
Overbank Drainage Minor
Comments Banks consist of weak deposits with occasional layers and seams
of soils with differing strengths and highly varying permeabilities.

(4) Construction of Protection

Need for Protection To protect bank from erosion by high-velocity flows
during flood stage and conditions created by passing tows.
Erosion Causative Agents The prime agent appears to be velocity and/or turbu-
lence caused by towboats in a rather narrow channel. Prop wash appears to
have only a minor effect.
Protection Techniques Shaping of banks and placement of stone.
General Design See Attached Item 32-1*

Project Length 1,505 ft. Construction Cost \$ 40/100ft² Mo/Yr Completed 7/66

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 717.1 (1912 adj.); 84,600, 14 Apr 1969

Repairs and Costs (Item, Cost, Date) None

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Visual site observation and photographs

Documentation Sources Photos, 1966, 1967, 1968, 1969, 1971, 1979

Project Effect on Stream Regime Negligible

Project Effect on Environment Negligible

Successful Aspects Has held toe of bank from eroding and prevented major sloughing. Permitted the upper bank to become vegetated. Prevented stream from migrating.

Unsuccessful Aspects Occasional local sloughing near and slightly above waterline. Loss of some protection starting above 1 foot below and extending about 3 ft above waterline.

General Evaluation See attached Item 32-1**

Recommendations No work is required at this time, However, the site should be observed carefully. Future work may consist of placing more stone to about 2 feet above and below normal pool elevation.

(7) Additional Information, Comments, and Summary

Map No. 32. See Attached Item 32-1***

Attached Items:

32-1 Project summary and location

32-2 Project plan and typical section

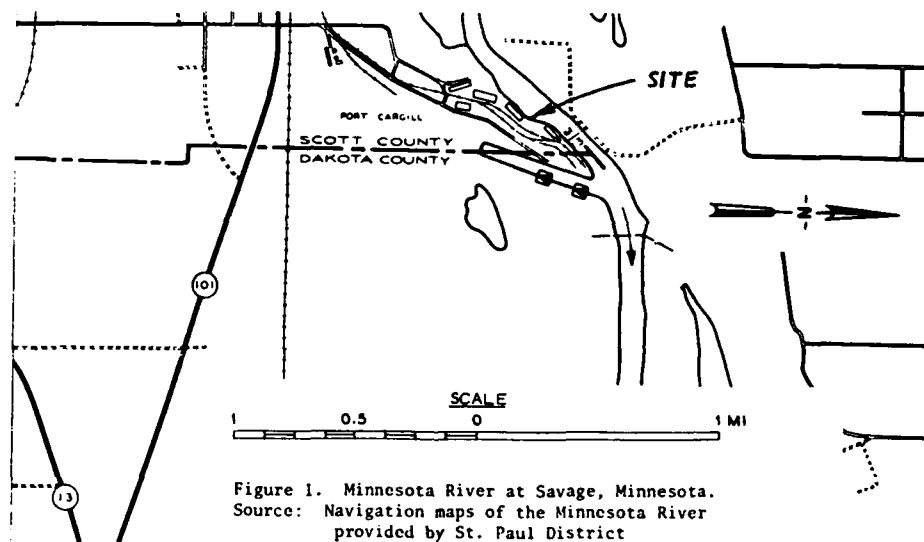
32-3 Project photographs

32-(4-7) "

SAVAGE

This site is part of an existing navigation project on the Minnesota River, as shown on the inclosed map. It is an example of quarry-run riprap used for slope protection on shaped banks consisting of cohesive subgrade material. The site has been monitored since 1966. Continued monitoring of the site helps to evaluate the effectiveness of this type of riprap and determine the advisability of using this type of material on similar projects.

The protection is generally performing satisfactorily. Most of the exposed riprap has been covered through siltation and vegetation. In an area where the armor is partially breached, failure probably results from a washing action. However, excessive erosion is not occurring and immediate repair work does not appear warranted.



ITEM 32-1
(Sheet 1 of 2)

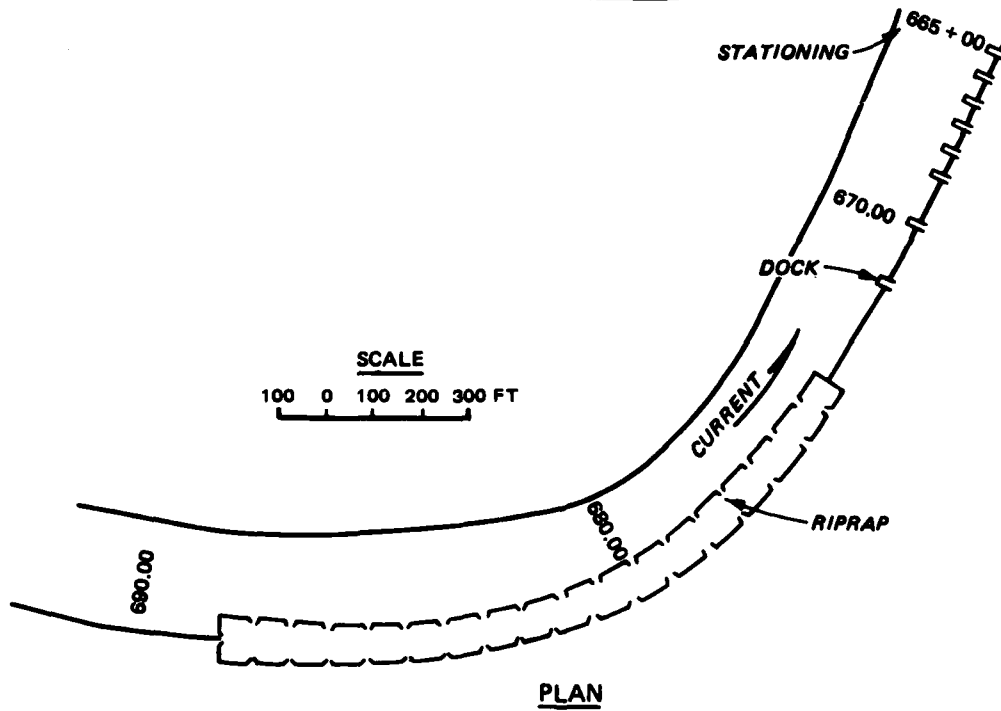
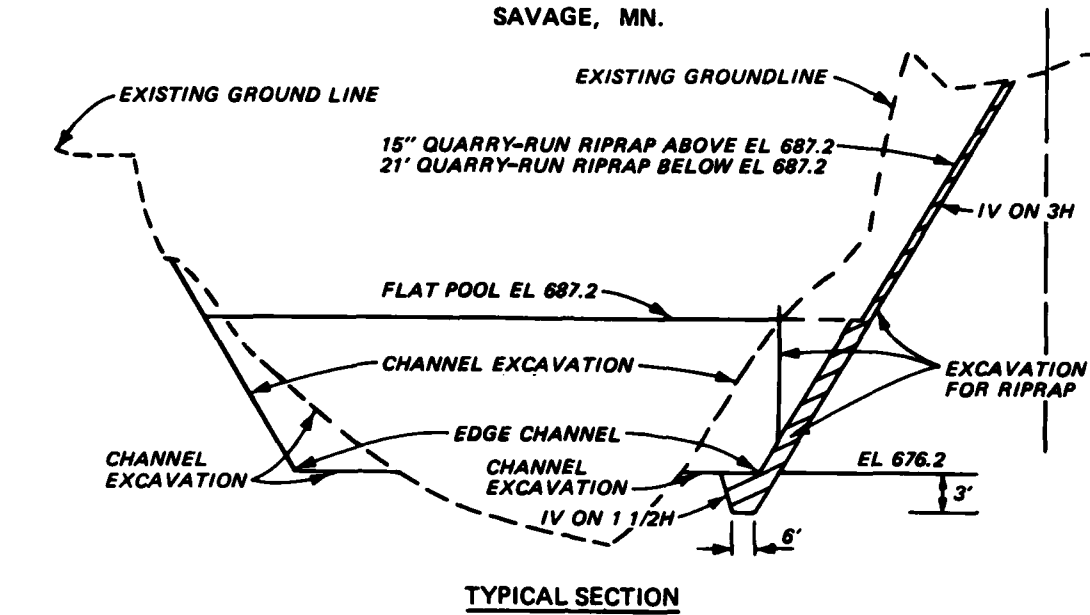
*15-in quarry-run riprap above water and 21-in quarry-run riprap below water. No bedding layer was used and the quarry-run rock was not processed. The stone was to be well graded between a maximum size of 250 pounds and not more than 3 percent smaller than the #100 sieve.

**Protection has been effective. Failure probably results from combination of several forces: seepage, frost action, and weak soil, resulting in minor bank instability. This results in a breach of the armor and is complicated by frequent velocity and water-level changes caused by passing tows. Approach of the tows forces a rise in the water surface. As the tows pass, the water level drops, causing a minor sudden drawdown. This continual action prevents buildup of silt or vegetation, and slowly removes the finer material from the rock mass. The failure has not affected the ability of the project to function as designed. The protection is preventing erosion at the toe. If erosion occurred at the toe, the bank would fail completely.

***In several areas, the stone is completely covered with silt and vegetation. Considerably less protection was used on this project than is required by strict compliance with guidelines. Heavy or severe snagging, clearing, and grubbing of the unprotected banks was not done (this appears to have been favorable). The root systems of the large trees are limiting the erosion on many of the exposed banks. Areas where tree growth has been removed exhibit considerably more erosion than adjacent wooded areas. Efforts applied by the adjacent landowners have also been favorable. Through placement of rubble on areas where erosion has started, their efforts have prevented further damage and have provided a basis for natural silting. The localized failure started within the first 3 years after construction, but the protection functioned successfully on the lower and upper bank and has allowed vegetation to become reestablished. As long as the protection stays intact at the toe, the lower bank should remain stable. There will be some erosion and sliding of the upper bank, but until it breaches the landward edge of the protection, there should be no need to repair. This type of bank protection is used on selected banks throughout the 14.7 mile project, and the overall protection is performing satisfactorily.

ITEM 32-1
(Sheet 2 of 2)

SAVAGE, MN.



PROJECT PLAN
AND TYPICAL SECTION

ITEM 32-2

PHOTOGRAPHS

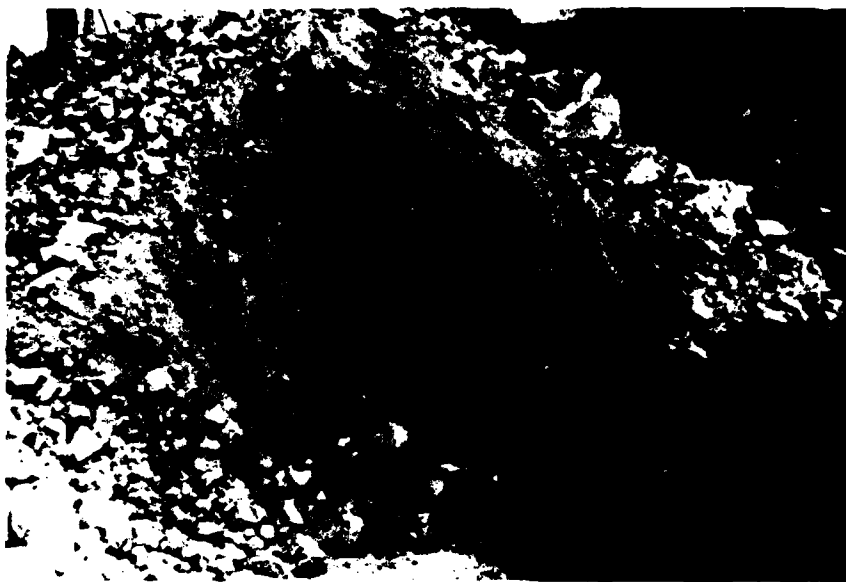


Photo 1

Unloading of first bargeload of riprap. Coarse material is on the outside of the load as a result of the manner in which the barge was loaded.

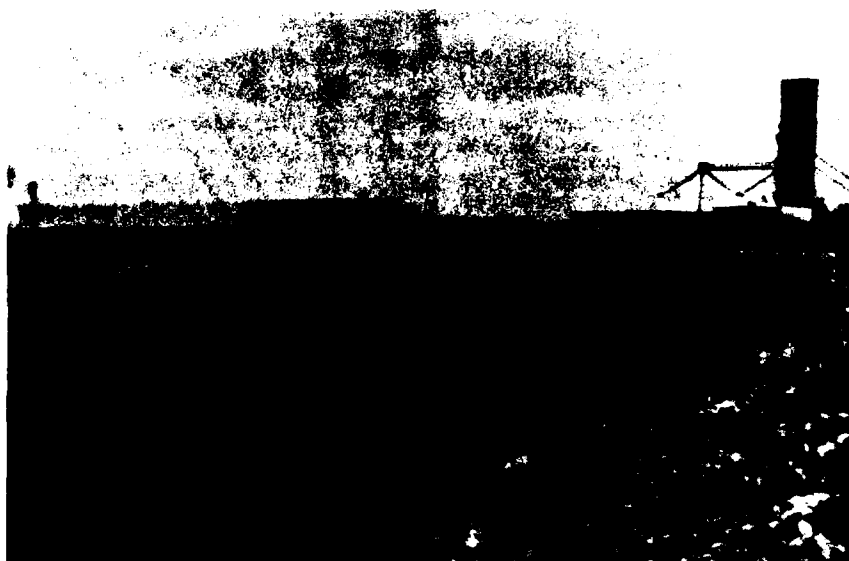


Photo 2

Taken 15 May 1967 from upstream end of riprap. Shows a fair representation of the test section after completion.

ITEM 32-3

H-32-6



Photo 3

Taken 10 August 1966 looking downstream from creek inlet. Shows bank protection after completion. The 1969 high water overtopped the top of the riprapped bank by almost 10 feet.

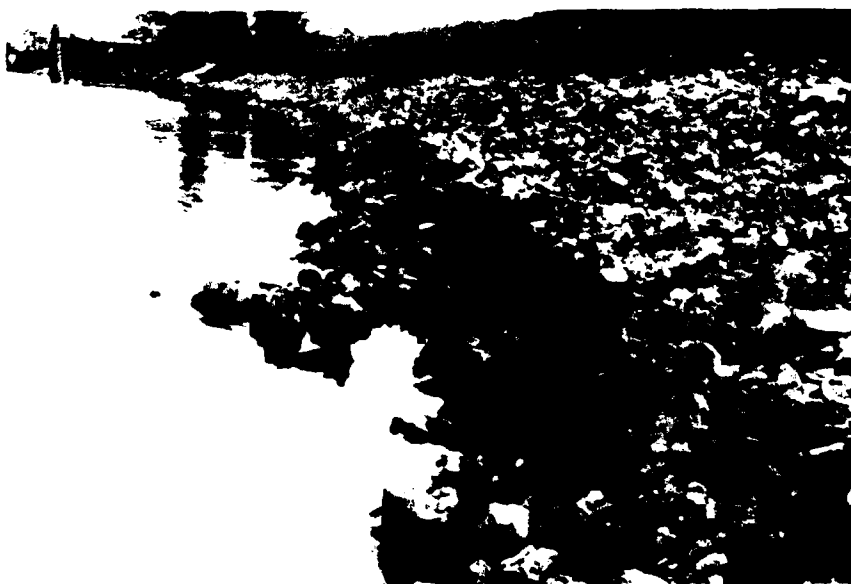


Photo 4

Taken 26 September 1969 looking downstream from creek inlet. Bank is beginning to cave at the water surface. The caving is probably caused by erosion and heavy bank seepage.

ITEM 32-4

H-32-7



Photo 5

Taken 28 June 1971 looking downstream from creek inlet. Water surface is above the caving area on the bank. Indicates no significant change since 1969.



Photo 6

Taken 6 November 1979 downstream of creek inlet looking upstream. Shows caving banks.

ITEM 32-5

H-32-8



Photo 7

Taken 27 July 1979. Same area as previous photo.



Photo 8

Taken 27 July 1979 looking upstream from creek inlet. Shows heavy growth of vegetation which has become established because placement of rock has prevented bank failure.

ITEM 32-6

H-32-9



Photo 9
Taken 1971. Same area as preceding photo.



Photo 10
Taken 6 November 1979. Upstream end of protection.
Shows difference between protected and unprotected
bank. In this area, displacement of water by tows
and river currents are not forced directly into the
bank.

ITEM 32-7

H-32-10

**MINNESOTA RIVER
MANKATO, MINNESOTA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Minnesota River River Mile 106.5 Side Both
Local Vicinity South-central Minnesota Lat N44°10'04" Long W94°00'26"
At/Nr City Mankato-North Mankato County Blue Earth-State MN Cong Dist _____
CE Office Symbol NCS ED-HF Responsible Agency Nicolett St. Paul District, Corps of Engineers
Site Map Sources _____
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range 748 to 778.8 ft; Period of Record 1903 to 1978
Discharge Range 26 to 94,100 cfs; Velocity Range 0 to 6.5 fps
Sediment Range 5.2 to 247,000 tpd; Period of Record 1963 to 1978
Bank-full Stage 774 ft; Flow 65,000 cfs; Average Recurrence Interval 25 yr
Bank-full Flow Velocity: Average 5.3 fps; Near Bank 5.3 fps
Comments Design Q = 150,000 cfs: Stage - 784.2; V = 8.6 fps

(3) Geology and Soil Properties

Bank (USCS) See Attached Item 33-1* Bed (USCS) Sand and gravel
Data Sources Machine borings
Groundwater Bank Seepage Minor
Overbank Drainage None, except that which falls directly on the slope.
Comments _____

(4) Construction of Protection

Need for Protection High channel velocities during high flows.
Erosion Causative Agents Stream velocity
Protection Techniques Shaping of banks and placement of stone.
General Design See Attached Item 33-1**
Project Length 2,700 ft; Construction Cost \$ Attached Mo/Yr Completed Left-1971
Item 33-1*** Right-1979

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 767.8; 34,000 cfs; 2.6 fps - 3 Apr 1979

Repairs and Costs (Item, Cost, Date) None to date

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Visual observations and photographs

Documentation Sources Photos - 1979

Project Effect on Stream Regime Confined the stream channel between the two
riprapped banks

Project Effect on Environment Minor, eliminates vegetal growth on banks and
associated small animals and birds.

Successful Aspects Stopped any further erosion of riverbanks, aesthetically
pleasing to some. Prevents bank failure caused by the erosion.

Unsuccessful Aspects See attached item 33-1****

General Evaluation The minor collapse of the rock on the left bank generally
leaves a depression 4-8 in. in the surface but does not break the con-
tinued cover. Protection on the right bank appears sound and adequate.

Recommendations Quarry-run material should be used primarily on cohesive
soils. When used on cohesionless soils, the thickness should be in-
creased; the amount of smaller stones in the material should be increased.

(7) Additional Information, Comments, and Summary

Map No. 33. See attached item 33-1*****

Attached Items:

33-1 Project summary and location

33-2 Typical sections left and right bank

33-3 Stone gradations

33-4 Typical gradation photographs

33-5 Project photographs

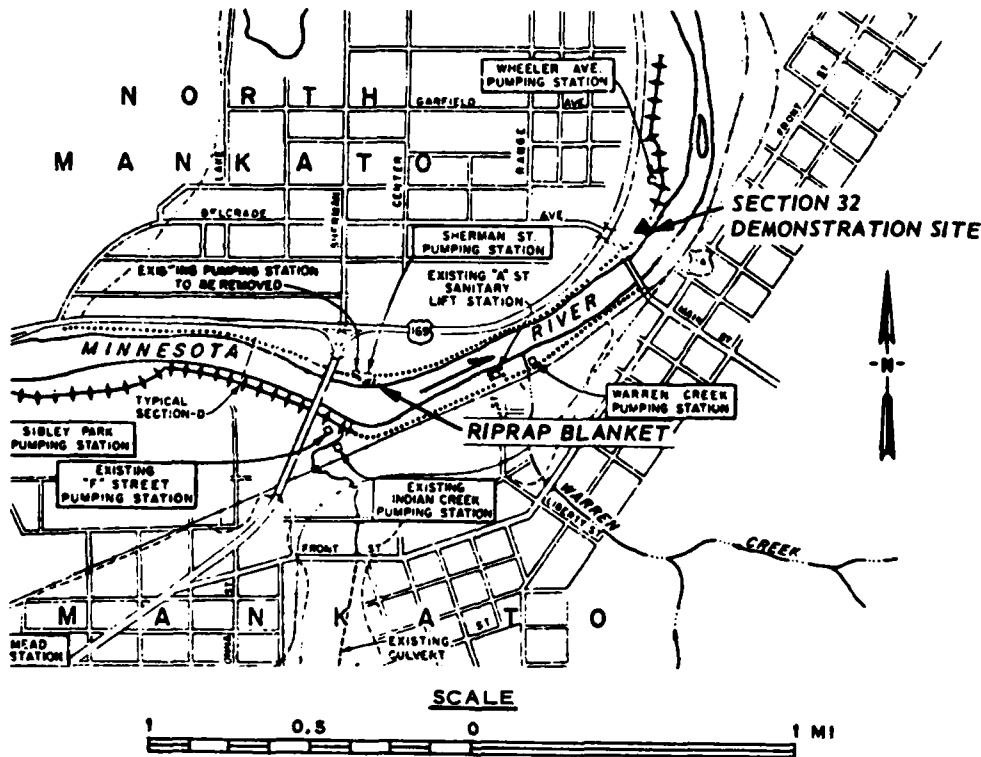
33-6 "

33-7 "

MANKATO

This site is part of the existing flood control project on the Minnesota River at Mankato, Minnesota, as shown below. The major purpose of observing this site was to compare the use of quarry-run riprap to a graded riprap with bedding. The study began 27 July 1979. The inspection team from Waterways Experiment Station (WES) visited a Mankato site downstream of the Main Street Bridge. Although the two sites are similar, they are not comparable because of other factors. The site chosen for this report is typical of sites in a straight section of the river. This site permits comparison of the left bank, which is protected with quarry-run stone, with the right bank, where graded riprap was placed, under as near as possible the same flood flow pattern. If this site were used, a good comparison could not be made because the quarry-run rock would have been on the inside of a curve and the graded material on the outside of the curve.

At this time, both types of protection are performing satisfactorily. The quality of the materials and placement techniques were good.



Minnesota River at Mankato, Minnesota.

ITEM 33-1
(Sheet 1 of 2)

*Predominantly sand and silty sand with clay and silt layers. In some areas, pervious and impervious material has been placed on the banks (see typical sections).

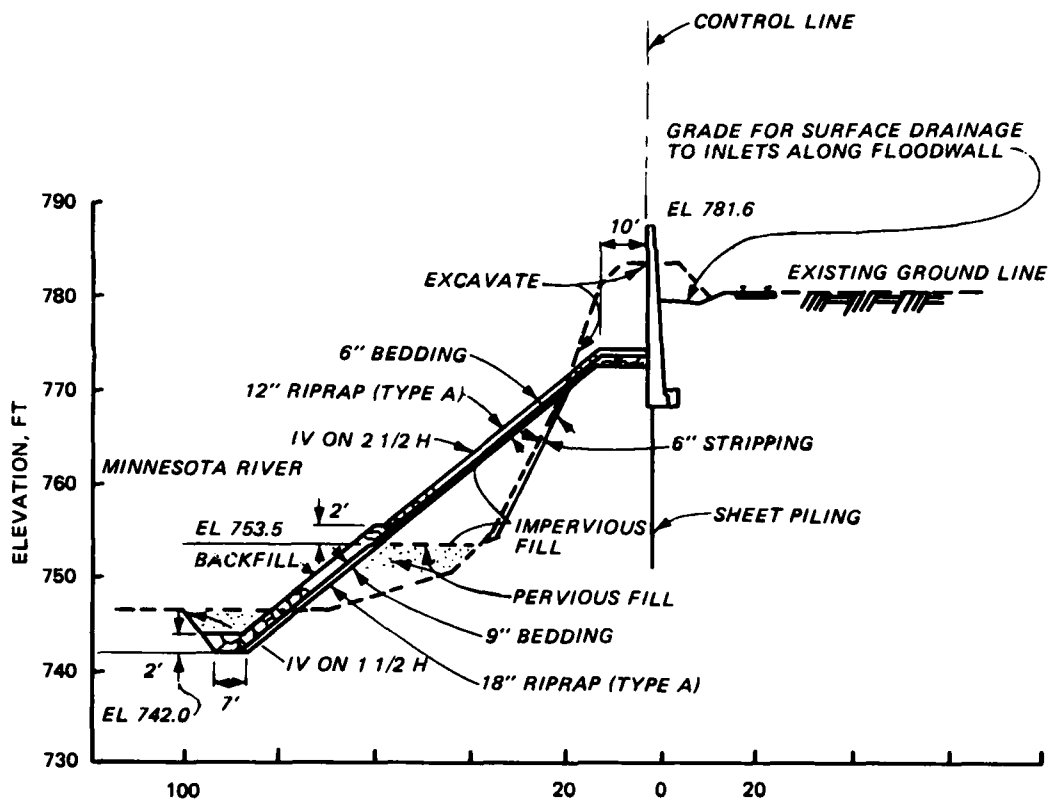
** Right bank - 12" graded riprap on 6" bedding above water and 18" graded riprap on 9" bedding below water
Left bank - 15" quarry-run riprap above water and 18" graded riprap on 9" bedding below water. The graded riprap and bedding was used below water because the foundation soil was a fine sand.

*** Right bank - Graded riprap \$74/100 ft² of placement
- Bedding \$37/100 ft² of placement Fall 1979
Left bank - Quarry-run rock \$47/100 ft² of placement Aug 1971

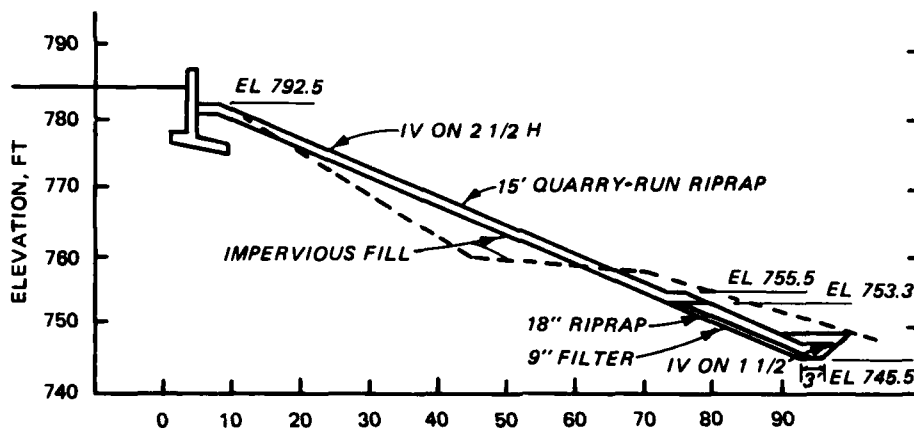
****In some areas on the left bank, noticeable concentrations of fine material have been washed out from the quarry-run riprap by surface runoff. This leaves medium-sized voids in the rock until the rock collapses to fill the voids. The voids are more pronounced in areas where the quarry-run rock lacked fines. This is especially true downstream of the Main Street Bridge where fines were removed before placement because of a misunderstanding of the contractor.

*****Rock and construction control on the right bank was probably as good as possible. This section and work look excellent. The major problem encountered with the quarry-run riprap on the left bank was insufficient small material. The lack of fines did not permit filling of the voids in the rock matrix. The voids between the larger stones which were open or only partly filled with finer material created a flow path for surface runoff. These paths also permitted the smaller fines to be transmitted downward through the mass to the subgrade and created a tendency for the surface runoff to become concentrated without having sufficient fines to protect the clay subgrade from eroding. This allowed channels to begin forming and offered an easier path for the flow of surface runoff which enhanced the erosion resistance. The material produced for this job was taken from massive stone. Because of the hardness of the rock, the quarry-run material also encourages a faster growth of vegetation which may cause more maintenance problems under certain climatic and flow conditions. However, the cost of quarry-run rock in place as compared to graded riprap and bedding is considerably less, and its degree of protection appears adequate.

ITEM 33-1
(Sheet 2 of 2)



RIGHT BANK-GRADED RIPRAP



LEFT BANK-QUARRY-RUN RIPRAP
RIVER MILE 106.5

MANKATO SITE
TYPICAL SECTIONS WITH PROTECTION
TECHNIQUE DETAILS

ITEM 33-2

H-33-5

Gradations of slope protection materials			
Location	Type	Material	Weight of stones (pounds)
		Percent by weight	
Left bank	Quarry-run riprap	100	170 max
		50	50 min
		15	10 max
	(Not more than 3 percent passing #100 sieve)		
	Graded riprap	100	80-170
		50	35-70
		15	13-35
	Bedding (filter material)	<u>Sieve size</u>	<u>Percent passing</u>
		6-inch	100
		3-inch	70-100
		1 1/2-inch	53-80
		3/4-inch	38-60
		#4	15-34
		#10	0-15
		#20	0-5
Right bank	Graded riprap (type A)	100	86-35
		50	36-17
		15	18-5
	Bedding	<u>Sieve size</u>	<u>Percent passing</u>
		6-inch	100
		3-inch	70-100
		1 1/2-inch	53-77
		3/4-inch	38-60
		#4	15-34
		#10	0-15
		#40	0-5

ITEM 33-3

H-33-6



Photos 1 and 2

Right bank - typical gradation for graded riprap.
Shows uniformity and good size distribution.

PHOTOGRAPHS
(taken 27 July 1979)

ITEM 33-4

H-33-7



Photo 3

Right bank. The gray stone shows the depth of flow of normal water surface. The light brown stone probably represents the highest water level recorded (elevation 767.8 or about 6.2 feet below the top of the riverward berm on 3 April 1979) since this stone was placed.



Photo 4

Left bank. Typical gradation for quarry-run riprap. Note the non-uniformity and clustering of different sizes.

ITEM 33-5

H-33-8



Photo 5
Left bank. Indicates amount of vegetation on bank.



Photo 6
Left bank. Note small stone washed out of riprap by surface runoff. The cause is believed to be insufficient fines in the mass. The small material appears to generally wash from the mass down to the subgrade and be carried in small channels under the rock until it becomes blocked. It then washed out through the open texture of the mass. The clusters of small material generally show up below these channels which show a lack of fines.

H-33-9

ITEM 33-6



Photo 7

Left bank. Extreme case where the small stone washed out creating voids and exposing the subgrade. The rock is collapsing because of loss of fines. Because it is collapsing, the protection will probably self-heal over time.

ITEM 33-7

H-33-10

**TANANA RIVER
FAIRBANKS, ALASKA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Tanana River (Tree Revetment) River Mile _____ Side Right
Local Vicinity Fairbanks, Alaska Lat N64°46' Long W147°30'
At/Nr City Fairbanks County _____ State AK Cong Dist 1
CE Office Symbol NPD Responsible Agency Corps of Engineers
Site Map Sources Corps of Engineers
Land Use Information Sources Corps of Engineers

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft; Period of Record 19__ to 19__
Discharge Range 3,100 to 125,000 cfs; Velocity Range 2.2 to 6.5 fps
Sediment Range _____ to _____ tpd; Period of Record 19__ to 19__
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) Silty sand, silt, gravelly Bed (USCS) No information available
sand, sandy gravel-a nearby site
Data Sources _____
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection Need for an expedient erosion protection scheme led to
the construction of this experimental revetment utilizing a material
which is readily available at a reasonable cost.
Erosion Causative Agents High-stage stream flow through a relatively straight
reach

Protection Techniques Tree revetment; groups of single or multiple tree clumps,
with roots & without roots, spaced at 14-ft intervals, cabled to riverbank.
General Design _____

Project Length * ft; Construction Cost \$ * Mo/Yr Completed Sept '77
* \$24.70 per lineal foot.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Date) The most significant damage was due to ice during breakup. Trees were busted and torn loose. Some damage has occurred due to vandalism (firewood gatherers). No repairs have been made.

Comments: The cost estimate given does not include the cost of the trees, which were available in the area at no cost.

(6) Performance Observations and Summary

Monitoring Program No more monitoring required.

Documentation Sources _____

Project Effect on Stream Regime None detected.

Project Effect on Environment The angle and location of the main current changed following construction of the tree revetment, shifting downstream reducing flow velocities.

Successful Aspects _____

Unsuccessful Aspects None.

General Evaluation Tree revetments with roots attached and inclumps of 3 or more trees performed better than single trees or small groups with and without roots. A closer spacing would provide better protection. Trees without roots were relatively ineffective since they tend to float at the waters edge providing little slope protection. Some trees without roots were deposited on top of the riverbank during overbank flows. All trees suffered from ice damage during breakup and from wood gatherers.

(7) Additional Information, Comments, and Summary

Map No. 34a

Attached Items:

34a-1 Unique Project Features	34a-5 Velocities (tree mattress)
34a-2 Location	34a-6 Tree Revetment Details
34a-3 Bankline Surveys	34a-7 Tree mattress Details
34a-4 Velocities (tree revetment)	34a-(8-10) Project Photographs

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Tanana River (Timber Mattress) River Mile _____ Side Right
Local Vicinity Fairbanks, Alaska Lat N64°46' Long W147°30'
At/Nr City Fairbanks County _____ State AK Cong Dist _____
CE Office Symbol NPD Responsible Agency Corps of Engineers
Site Map Sources Corps of Engineers
Land Use Information Sources Corps of Engineers

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft; Period of Record 19____ to 19____
Discharge Range 3,100 to 125,000 cfs; Velocity Range 2.6 to 7.8 fps
Sediment Range _____ to _____ tpd; Period of Record 19____ to 19____
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) Silty sand, silt, gravelly Bed (USCS) No information available
sand, sand gravel at nearby sites
Data Sources _____
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection Need for an expedient erosion protection scheme led to
the construction of the experimental timber mattress.
Erosion Causative Agents See page 34-1.
Protection Techniques Timber mattresses, 8'x20' constructed of 6"x6"x20" timbers
with concrete weights, tied together and cabled to riverbank.
General Design _____
Project Length * _____ ft; Construction Cost \$ _____ * Mo/Yr Completed Oct '78
* \$110 per lineal foot of riverbank.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Date) Timber mattresses have failed to protect the upper bank, which has eroded from under the cable ties allowing the mattress to slip down the bank. No repairs have been made or are planned.

Comments: Construction costs could be significantly reduced in a larger scale operation. Mattresses can be constructed offsite and transported to site.

(6) Performance Observations and Summary

Monitoring Program No more needed.

Documentation Sources _____

Project Effect on Stream Regime _____

Project Effect on Environment _____

Successful Aspects _____

Unsuccessful Aspects Timber mattresses acted as a hard point contributing to severe erosion downstream. Downstream end effects increased upper bank erosion.

General Evaluation Timber mattresses proved to be a expedient erosion protection scheme, since it can be constructed offsite and transported to the site when needed. A larger scale operation could significantly

~~Recommendations~~ reduce the cost/100 L.F. of the timber mattresses. The design failed to protect the upper riverbank and suffered some ice

damage. A redesign would have to consider relocating the upper concrete weight, establishing some high-water freeboard and further protection for the upper river-

(7) Additional Information, Comments, and Summary

Map No. 34a

Attached Items:

34a-1 Unique Project Features	34a-5 Velocities (tree mattress)
34a-2 Location	34a-6 Tree Revetment Details
34a-3 Bankline Surveys	34a-7 Timber Mattress Details
34a-4 Velocities (tree revetment)	34a-(8-10) Project Photographs

The Tanana River is a major tributary of the Yukon River. It flows generally to the northwest for its entire length of 531 miles and drains about 20,640 square miles above the confluence of the Chena River. The Chena River, a major tributary, enters the Tanana River from the north about five miles southwest of Fairbanks. The city of Fairbanks is on the floodplain between the Tanana and the Chena Rivers and is subject to flooding and damage from either or both rivers. As a result, the Chena Flood Control project was authorized by Congress to protect the city of Fairbanks from flooding. One principal feature of the Flood Control Project is the Tanana River Levee. The Levee extends 20.4 miles downstream from the Moose Creek Dam (another principal feature) to the confluence of the Chena and Tanana Rivers. Riverbank erosion on the north bank of the Tanana River poses a significant threat to certain sections of the Levee system. As a result, several diversion groins have been constructed along the Tanana River to protect the Levee. The high cost of the riprap, the scarcity of hard durable rock in the vicinity of Fairbanks, Alaska, and the need for an expedient erosion protection scheme, led to the construction of the experimental streambank protection works described in this report. The demonstration site is located adjacent to a diversion groin located nine miles upstream of Fairbanks, Alaska, on the right bank. Photo 1 shows the study site just south of the diversion groin. This site was selected because of its erosion characteristics and site accessibility. Two types of streambank protection works have been constructed and evaluated at this site: the tree revetments in the fall of 1977 and the timber mattresses in the fall of 1978.

TREE REVETMENTS

UNIQUE PROJECT FEATURES

1. Surrounding Area Housing and Industry There is no housing or industry in the area of the tree revetments.

ITEM 34a-1
(Sheet 1 of 8)

2. Details on Design Considerations

a. Flow Velocity Design velocities were 6 to 8 feet per second (fps). Figures 2 and 3 show some partial cross sections and velocity distributions measured during 1979. Measured velocities ranged from 2.2 to 6.5 fps.

b. Discharge Measured discharges of the Tanana River at Fairbanks have ranged from 3,100 to 125,000 cubic feet per second (cfs). The usual high summer flow at Fairbanks is 60,000 cfs. The estimated discharge for the 5 year flood is 92,000 cfs and 129,000 cfs for the 25 year flood.

c. Wave Heights Wave heights were not considered to be a design problem.

3. Details on Construction Type and Technique The tree revetments were constructed on the Tanana River during the week of 28 August to 3 September 1977. Two basic types of tree revetment were investigated, trees with roots attached (photo 3) and trees without roots (photo 4). The revetments consist of 51 groups of single and multiple tree clumps, spaced at approximately 14 foot intervals. Live and dead spruce trees of various sizes and lengths were used to determine their effectiveness for revetment. The layout of the tree revetment construction is shown in figure 1. The first 400 feet of the revetment consists of trees with roots. The next 325 feet consists of trees without roots (45 feet of the downstream end was removed in October of 1978 to provide room for the timber mattress test section). Typical details for the tree revetment construction are shown in figure 6. Smaller trees both dead and alive were pulled out by the roots and cabled together in clumps of two or more depending on the size of the trees. The larger live spruce trees were sawed down and transported by highbed semi-trailer to the assembly area. The trees were placed on the riverbank with the tops toward the water. All trees were anchored to buried deadmen at least 35 feet from the bank line using 1/2" diameter wire

rope. Following placement and anchorage of all trees, they were pushed into the river starting at the downstream end until only the butt (of sawed trees) or root clump (of whole trees) remained on the bank. The current would carry each tree downstream and swing it into the riverbank. Anchor cables to the deadmen were then tightened, so that the trees could go no farther. As the work progressed upstream, each tree would be carried down and into the previously placed tree.

4. Details on Observed Conditions Since Construction Since the construction of the tree revetment, the main current shifted downstream, reducing flow velocities in the area of the tree revetment. Erosion characteristics are best described as mild on the upstream end and moderate on the downstream end of the test section. During the breakup of 1979 the site experienced an overbank flow of 2 to 3 feet with heavy ice flow.

5. Maintenance History There has been no maintenance on the tree revetment since its construction.

6. Discussion of Any Possible Adverse Effects Caused by Construction No adverse effects have been detected as a result of the construction of the tree revetment.

EVALUATION

1. Social and Political Acceptance and How Well Structure Fits Natural Surroundings The tree revetment structures fit the natural surroundings quite well. The vegetation that was disturbed during construction has reestablished itself and blends in with the tree revetment construction. Social and political acceptance of the tree revetment structure is considered to be favorable since the tree revetment is a very Alaskan way of doing things.

2. How Well Structure Functions for Design Conditions The tree revetments with roots attached performed better than the trees without

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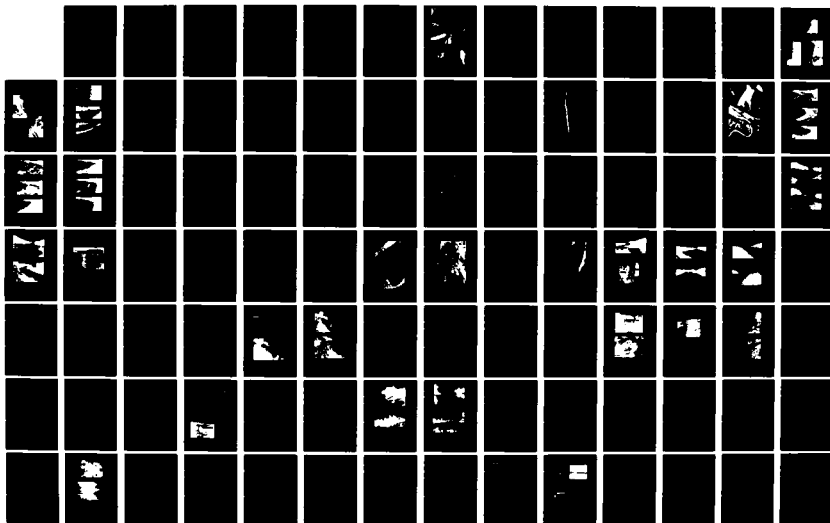
THE STREAMBANK EROSION CONTROL EVALUATION AND
DEMONSTRATION ACT OF 1974 S. (U) ARMY ENGINEER
WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.
M P KEOWN ET AL. DEC 81

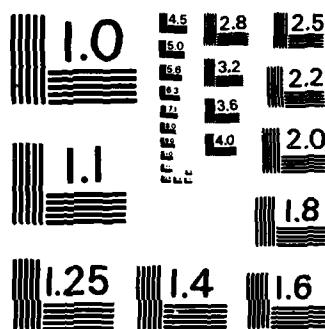
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MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

roots. Tree revetment clumps of 3 or more trees performed better than single or double trees. The tree revetment with roots kept their original positions better than the tree revetments without roots (photo 5). The tree revetments without roots were relatively ineffective once they fell into the river since they float at the water's edge, providing little slope protection (photo 6). Some trees without roots were also deposited on top of the riverbank during periods of overbank flow (photo 6). All tree revetments suffered from ice damage during breakup (photo 5). The tree revetments also suffered some vandalism or damage from firewood collectors.

3. Comments on Initial Cost and Maintenance Cost The cost of the tree revetment installation was \$2,470 per 100 lineal feet of riverbank (1977 costs). The cost estimate does not include the cost of the trees which were obtained in the area at no cost. Maintenance costs were not incurred with this structure.

4. Aesthetics of Structure The tree revetment installation has a natural look to it, particularly after the regrowth of the natural riverbank vegetation that was disturbed during installation.

5. Need for Further Monitoring or Possible Visit by Evaluation Committee No further monitoring will be done on the tree revetments. A visit from the evaluation committee is not considered necessary.

6. Discussion of Good and Bad Points of Structure and Any Possible Recommendations for Improvements or Whether Structure Should Not Be Considered for Further Use The tree revetments utilized a material which is readily available in the Fairbanks area at a reasonable cost with little environmental impact. Trees with roots attached, in clumps of three or more trees, provide fair to adequate protection in cases of moderate erosion. Tree revetments are subject to ice damage during breakup and damage by firewood gatherers. The tree revetment design could be improved by utilizing trees with roots in clumps of three or more trees, and increasing the density of the tree revetment

clumps by a factor of two or three. This would also increase the cost/100 lineal feet by about the same factor. Tree revetments are useful in cases of moderate erosion.

TIMBER MATTRESSES

UNIQUE PROJECT FEATURES

1. Surrounding Area Housing and Industry There is no housing or industry in the area of the timber mattresses.

2. Details on Design Considerations

a. Flow Velocity Design velocities were 6 to 8 feet per second (fps). Figures 4 and 5 show some partial cross sections and velocity distributions measured during 1979. Measured velocities ranged from 2.6 to 7.8 fps.

b. Discharge Measured discharges of the Tanana River at Fairbanks have ranged from 3,100 to 125,000 cubic feet per second (cfs). The usual high summer flow at Fairbanks is 60,000 cfs. The estimated discharge for the 5 year flood is 92,000 cfs and 129,000 cfs for the 25 year flood.

c. Wave Heights Wave heights were not considered to be a design problem.

d. Ice Heavy ice flows with ice thicknesses of 2 to 3 feet during breakup.

3. Details on Construction Type and Technique Typical details of the timber mattress construction are shown in figure 7. Timber mattresses were constructed in units that are "transportable" from an assembly site to the erosion site. The timber mattress unit measures 8' X 20' and was constructed of 6" X 6" X 20' timbers connected to a concrete

weight at both ends. Erosion matting was attached to the timber mattress to assist in protecting the riverbank. The timber mattress units were constructed upside down as shown in figure 7. Forms were prepared for the concrete weights with steel brackets and loops located as shown. Side boards were all that was necessary for form work since the ground was used as the bottom of the form and the concrete poured to a depth of 9". Once the concrete set up, the forms were removed and the timbers added to the concrete weights. The timbers were then tied to the concrete weights with 1/2" diameter cable through the timbers and steel brackets anchored in the concrete. Eighteen timber mattress units were constructed in this manner. The units were then flipped over so that the concrete weights are on top. Erosion matting was attached to the top surface of half of the units and on the bottom of the other half. The riverbank was prepared as shown in figure 7. Steel cables 1/2" in diameter were cut to a length sufficient to go from the deadmen to the timber mattresses; the deadmen were then placed in the riverbank as shown in figure 7. The timber mattress units were trucked, three at a time, from the assembly point to the erosion site, a distance of 14 miles, and installed on the riverbank. Thirty minutes per unit was the time required to offload and install on the riverbank.

4. Details on Observed Conditions Since Construction During the 1979 breakup the timber mattress site experienced 2 to 3 feet of overbank flow and heavy ice flow. The design flow velocities were encountered at the site.

5. Maintenance History There has been no maintenance on the timber mattresses since their construction.

6. Discussion of Any Possible Adverse Effects Caused by Structure The timber mattress structure has acted as a hard point since its construction with considerable erosion downstream of the structure. End effect erosion has contributed to the upper bank erosion behind the structure. A partial breakup of any one of the timber mattress

units could be dangerous to riverboat traffic and would reduce the effectiveness of the structure. A partial breakup would require maintenance or replacement of the unit.

EVALUATION

1. Social and Political Acceptance and How Well Structure Fits Natural Surroundings The timber mattress has a low profile but is highly noticeable on the riverbank due to the visibility of the upper concrete block. Social and political acceptance of the timber mattresses is considered to be favorable since riverbank protection schemes are a common sight in the Fairbanks area.

2. How Well Structure Functions for Design Conditions During the breakup of 1979 one timber mattress unit experienced damage due to ice. The upper concrete block was ripped loose and deposited on top of the riverbank as shown in photo 8. Maintenance was not done in order to study the effect of the damaged unit. The overbank flow during breakup of 1979 got behind the timber mattress structure and began to erode the upper riverbank. This action in combination with the end effect erosion describes the failure mode for the timber mattress structure. The erosion of the upper riverbank allowed the timber mattress units to slide further down the riverbank slope. Loss of freeboard contributes to the upper riverbank erosion by allowing high flows access to the upper riverbank as shown in photo number 9. The timber mattress units are providing some riverbank slope protection but fail to provide upper riverbank protection. The timber mattress units have acted as a hard point as can be seen in the bankline surveys in figure 1, showing complete erosion of the control area downstream.

3. Comments on Initial Cost and Maintenance Cost The timber mattress structure cost \$11,000 per 100 lineal feet of riverbank (1978 costs). These construction costs could be significantly reduced in a larger scale operation, where timber mattresses are constructed offsite and transported to erosion sites as needed.

4. Aesthetics of Structure The most noticeable aspect of the timber mattresses is the upper concrete block which is highly visible from the river. A modification of this block to retain the weighted end but reduce its visibility would make the structure more aesthetically acceptable.

5. Discussion of Any Need for Further Monitoring or Possible Visit by Evaluation Committee No further monitoring will be done on the timber mattresses. A visit from the evaluation committee is not considered necessary.

6. Discussion of Good and Bad Points of Structure and Any Possible Recommendations for Improvement or Whether Structure Should Not Be Considered for Further Use The timber mattresses have the potential to be an expedient erosion protection scheme. They can be constructed offsite and transported to an erosion site when needed. They provided reasonably good protection for the riverbank slope but failed to provide adequate protection for the upper riverbank. A redesign would have to provide for improved upper riverbank protection and relocate the upper concrete weight. The upper concrete weight could be designed with a lower profile or relocated beneath the timbers. The timber mattresses should be considered for future use in cases of moderate to severe erosion.



Photo 1 Location of tree revetment and timber mattress site on the Tanana River near Fairbanks, Alaska, 3 July 1979, river discharge is 39,200 cfs.

ITEM 34a-3

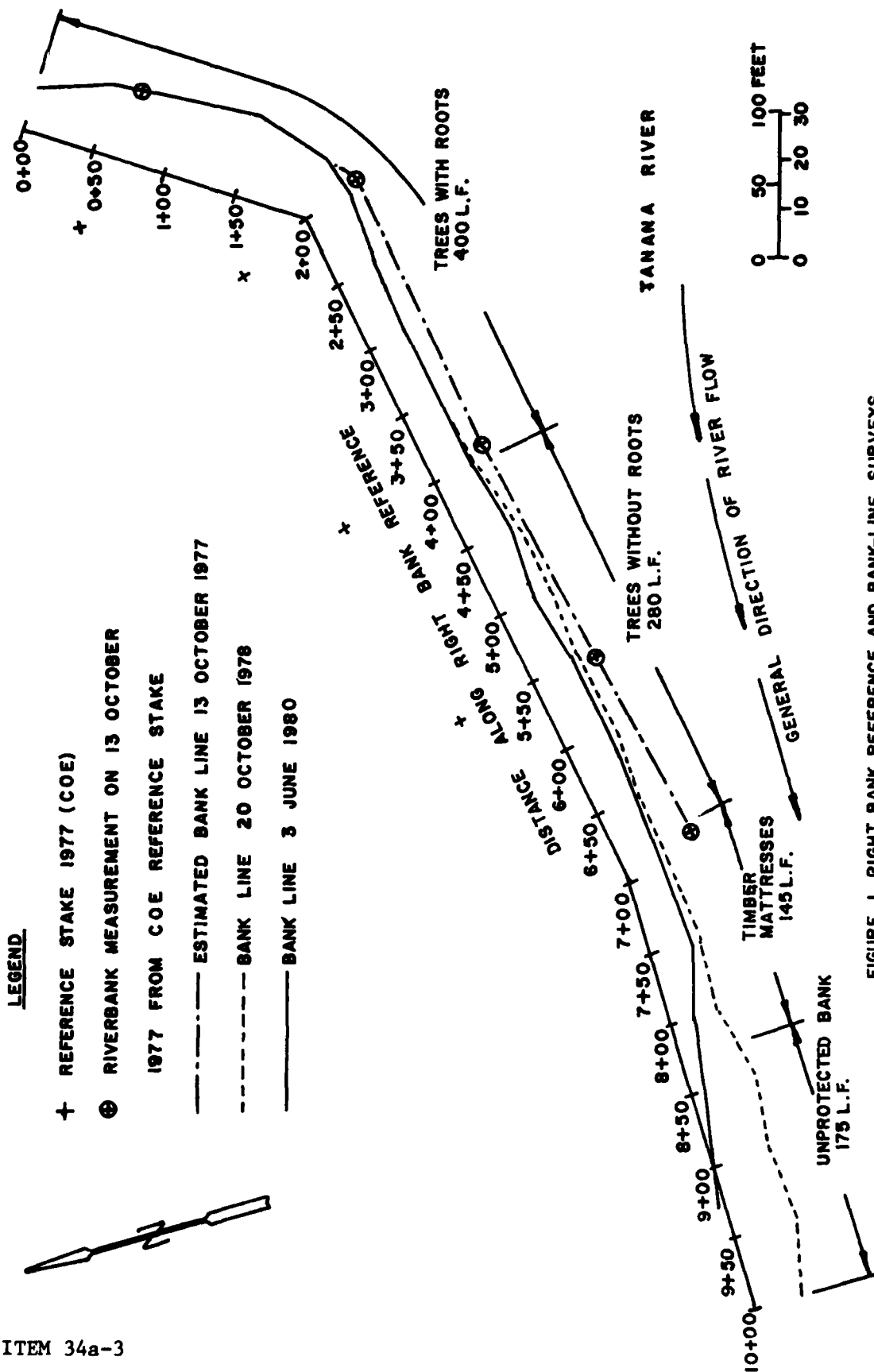


FIGURE 1 RIGHT BANK REFERENCE AND BANK-LINE SURVEYS
TREE REVETMENT AND TIMBER MATTRESS SITE

H-34a-14

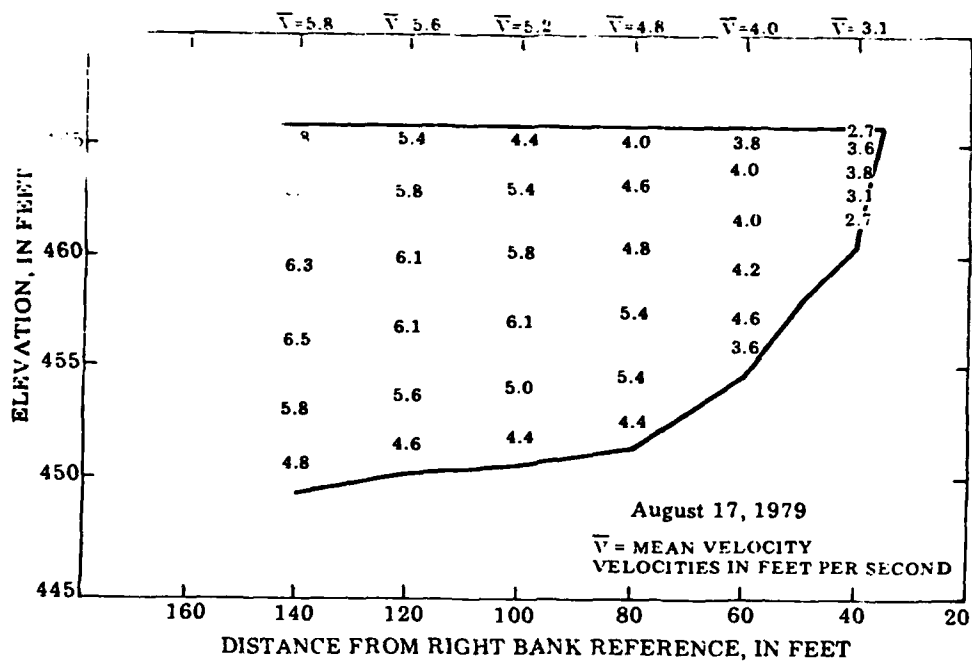
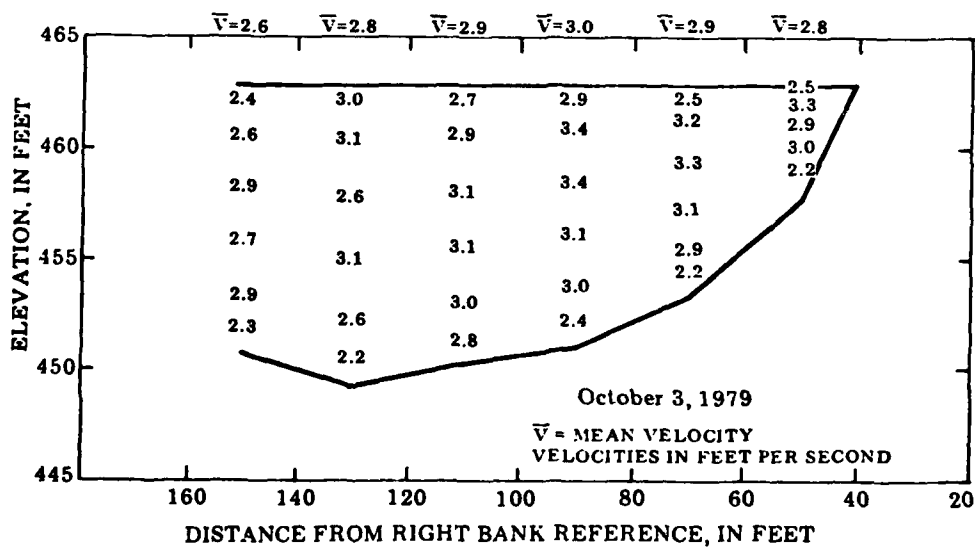


Figure 2

Partial cross sections and velocity distributions near right bank, station 4+50, tree revetment site.

Figure 3



ITEM 34a-4

H-34a-15

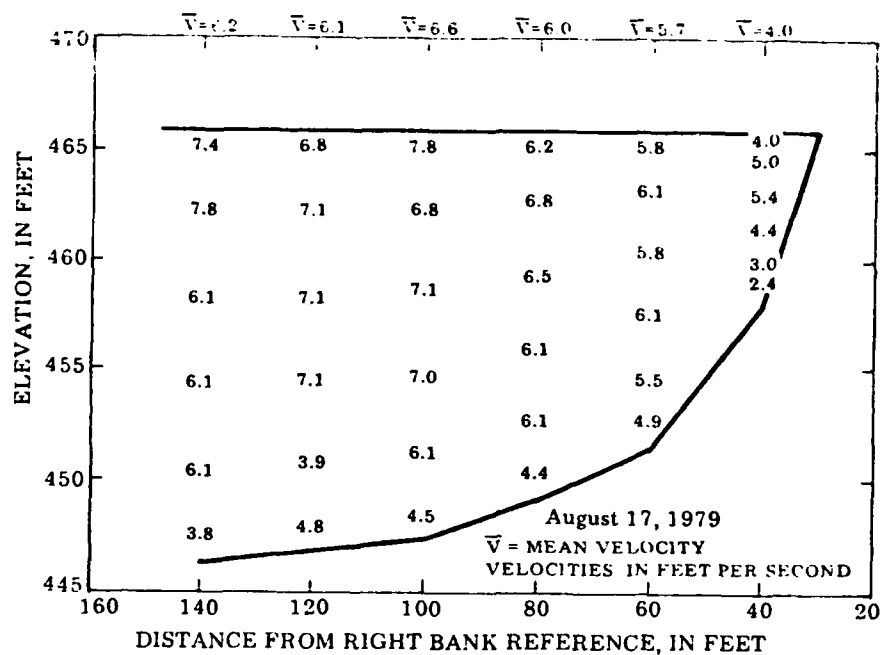
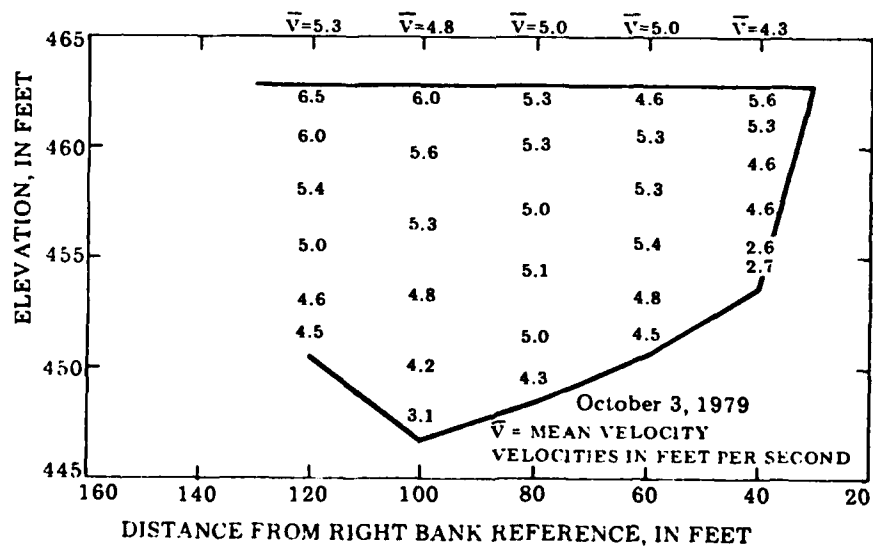


Figure 4

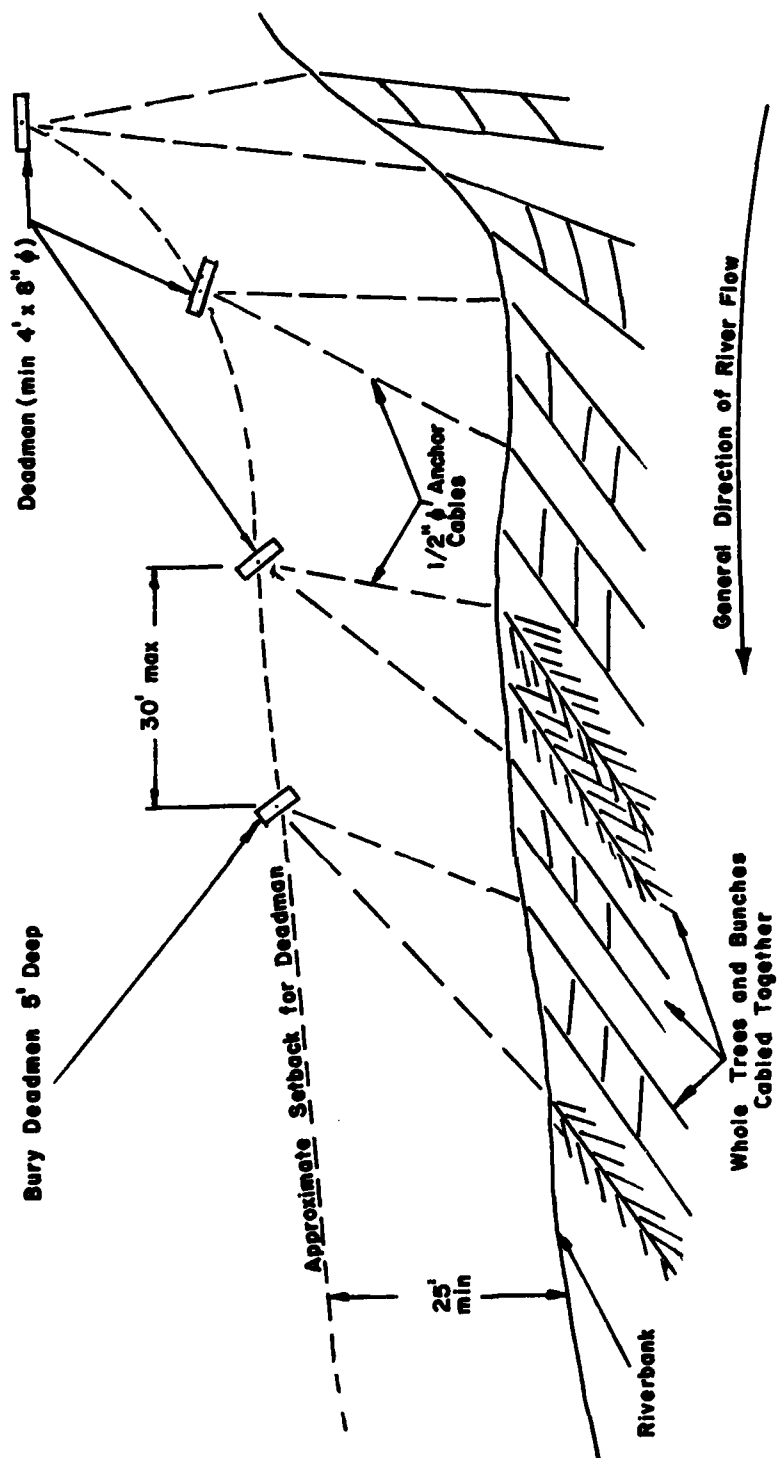
Partial cross sections and velocity distributions near right bank, station 7+00, timber mattress site.

Figure 5



ITEM 34a-5

H-34a-16

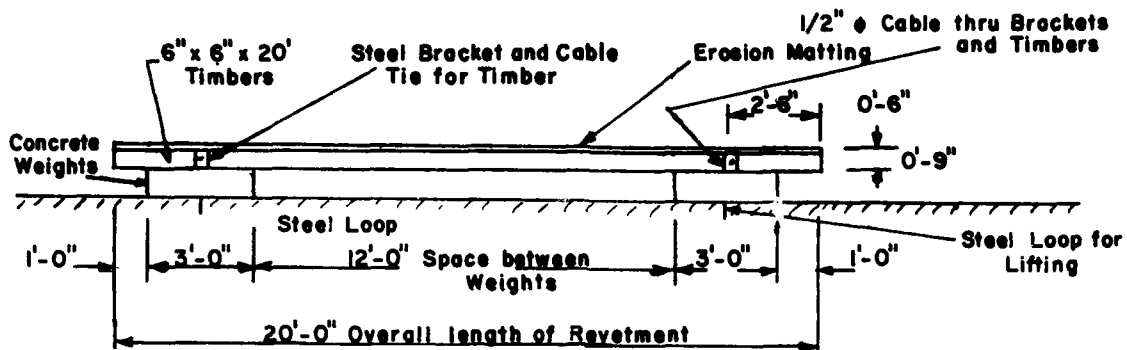


NOT TO SCALE

FIGURE 6 TREE REVETMENT - TYPICAL DETAIL

ITEM 34a-6

H-34a-17



Timber Mattress Constructed in Position Shown Above, Then Flipped over for Installation on Riverbank.

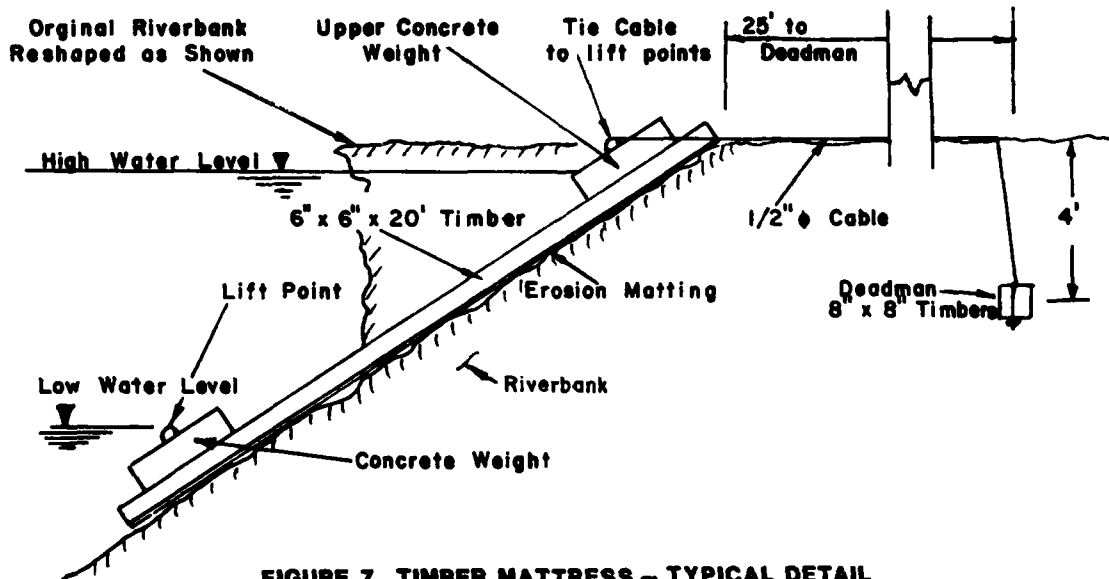
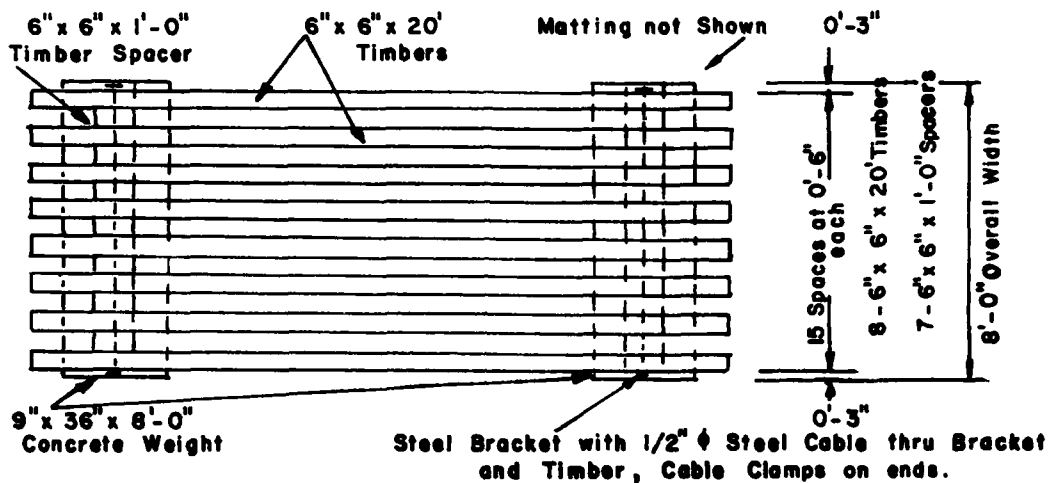


FIGURE 7 TIMBER MATTRESS - TYPICAL DETAIL

ITEM 34a-7

H-34a-18

Photo 2 June 1977, mean river discharge was 37,400 cfs,
view of riverbank prior to construction of tree revetments.



Photo 3 6 May 1978, river
discharge is 18,000 cfs, tree
revetment with roots attached.



Photo 4 5 May 1978, tree
revetment without roots,
tree limbs intact.



ITEM 34a-8

H-34a-19



Photo 5 9 May 1979, river
discharge is 36,600 cfs, tree
revetment with roots attached
showing ice damage and
position on riverbank.



Photo 6 9 May 1979, tree
revetment without roots,
Note tree left on top of
riverbank due to overbank flow

ITEM 34a-9

H-34a-20



Photo 7 June 1977, mean river discharge was 37,400 cfs, view of riverbank prior to construction of timber mattresses.

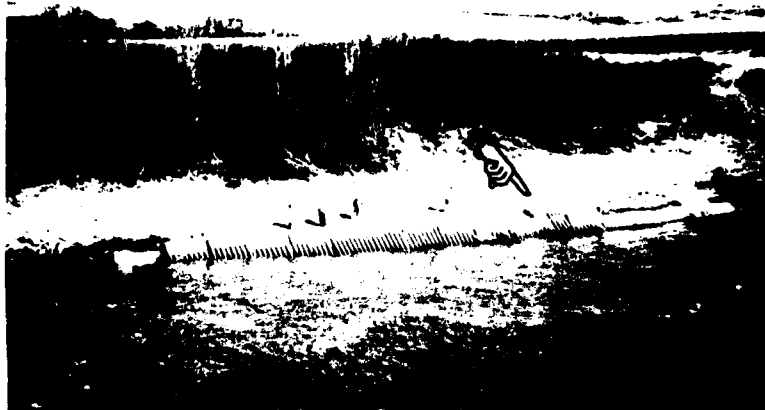


Photo 8 9 May 1979, discharge is 36,600 cfs, timber mattresses after break-up, failure mode starting on downstream end, one unit shows some ice damage



Photo 9 9 May 1979, timber mattresses showing failure mode, upper riverbank eroding due to end effect and overbank flow.

ITEM 34a-10

H-34a-21

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Tanana River (ice revetment) River Mile _____ Side Right
Local Vicinity Fairbanks, Alaska Lat N64°48' Long W147°52'
At/Nr City Fairbanks County _____ State AK Cong Dist _____
CE Office Symbol NPD Responsible Agency Corps of Engineers
Site Map Sources Corps of Engineers
Land Use Information Sources Corps of Engineers

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft; Period of Record 19____ to 19____
Discharge Range 3,100 to 125,000 cfs; Velocity Range 1.8 to 9.9 fps
Sediment Range _____ to _____ tpd; Period of Record 19____ to 19____
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) Silty sand, sandy silt, Bed (USCS) Same as bank.
sandy gravels, silty sandy gravels
Data Sources _____
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection Railroad spur supplying fuel to Fairbanks International
Airport was threatened due to erosion of riverbank.
Erosion Causative Agents High stage streamflow against concave bank of
bendway.
Protection Techniques Ice revetment
General Design _____
Project Length * ft; Construction Cost \$ * Mo/Yr Completed Apr '79
* \$30 to \$40 per lineal foot.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Ice revetment failed to reduce the erosion rate in the area of complete failure. No repairs were made to the revetment.

Repairs and Costs (Item, Cost, Date) _____

Comments: This type of construction is extremely weather dependent. Contractors must be ready when the weather is favorable for ice building.

(6) Performance Observations and Summary

Monitoring Program _____

Documentation Sources _____

Project Effect on Stream Regime No measurable effects

Project Effect on Environment _____

Successful Aspects _____

Unsuccessful Aspects _____

General Evaluation The construction of the experimental ice revetment was done using a variety of ice building methods to determine the best means of constructing a massive ice structure of this type. The structure was not completed to design standards due to the short construction period and the time it took to optimize the ice building method. The ice revetment provided good protection for the riverbank from ice damage but failed to inhibit the erosion rate in the area of complete failure. Ice revetment protection of this type may be considered for cases where ice damage occurs during breakup on rivers or lakes. An ice revetment used for protection from ice damage leaves little impact since it would melt in place. An ice revetment in conjunction with a riverbank freezing operation would probably be more effective. The protection is only temporary but there are times when temporary solutions are desirable.

Attached Items:

<u>34b-1 Unique Project Features</u>	<u>34b-5 Ice Revetment - Details</u>
<u>34b-2 Bankline Surveys</u>	<u>34b-6 Location</u>
<u>34b-3 Ice Revetment Failure</u>	<u>34-(7-9) Project Photographs</u>
<u>34b-4 Velocities</u>	

ICE REVETMENT

UNIQUE PROJECT FEATURES

1. Surrounding Area Housing and Industry The ice revetment site is located 1 mile south of the Fairbanks International Airport on the right bank of the Tanana River. This site has been a highly active erosion area for the last 10 years with an average erosion rate of 110 feet per year. The maximum rate of erosion in one year was 300 feet. River banklines from 1976 to 1979 are shown in figure 1. The ice revetment site is adjacent to a railroad spur which supplies fuel to the Fairbanks International Airport. A holding action was necessary to protect the railroad spur from the Tanana River. Local governments could not construct riverbank protection until the summer of 1979. Consequently, an experimental ice revetment was suggested as an attempt to reduce the erosion rate until a more permanent or substantial revetment could be built in the summer.

2. Details on Design Considerations

a. Flow Velocity Design flow velocities were 6 to 10 feet per second (fps). Figures 3 and 4 show some partial cross sections and velocity distributions taken during the summer of 1979. Measured flow velocities range from 1.8 to 9.9 fps.

b. Discharge Measured discharges of the Tanana River at Fairbanks have ranged from 3,100 to 125,000 cubic feet per second (cfs). The usual high summer flow at Fairbanks is 60,000 cfs. The estimated discharge for the 5 year flood is 92,000 cfs and 129,000 cfs for the 25 year flood.

c. Wave Heights Wave heights were not considered to be a design problem.

d. Ice Heavy ice flows with ice thicknesses of 2 to 3 feet are

expected during breakup.

3. Details on Construction Type and Technique The concept behind the ice revetment was to remove the snow cover on the riverbank allowing the riverbank to freeze to a depth deeper than that which usually occurs during the winter and to protect the riverbank with a large ice structure. The ice revetment was constructed by building a temporary dike in front of the riverbank and pumping water from the river into the area between the dike and the riverbank. The flooding was done with a submersible pump in 3 to 4 inch lifts and allowed to freeze overnight. The method works quite well as long as there is enough cold weather to freeze the layer of water completely solid before the next layer is applied. Developing the ice building technique and the right type of dike, for containing the flooding, were two of the major problems involved in the project. The most effective dike was a simple wooden wall built out of 2 X 4's and 3/8" plywood set in the ice sheet. The most effective way to build ice was to use a submersible pump, pumping the water from the river over the dike and into the area between the dike and the riverbank. The site layout, showing the location of the ice revetment, is shown in figures 1 and 2. Typical details of the ice revetment construction are shown in figure 5. Ice building operations began on the 9th of January 1979 and ended on the 23rd of March 1979. The ice revetment was not built to the design goal of 2 to 3 feet over the top of the riverbank, due to the short construction period, size of the structure, and the time it took to optimize the ice building method. The ice revetment as constructed was 800 feet long, 20 to 30 feet wide, had a surface area of 20,100 square feet, and a maximum thickness of 12 to 13 feet. The ice revetment was grounded out on the riverbed slope in several locations and the riverbed slope frozen to a depth of 1 to 2 feet. The riverbank itself had been cleared of snow and had frozen to a depth of 5 to 5.5 feet. Once the ice building operations ceased, the wooden forms were removed and the top surface of the ice revetment covered with sawdust to slow down the melting (photos 2 and 3). In addition, the riverbank itself was also covered with sawdust (photo 4).

4. Details on Observed Conditions Since Construction The breakup flow in the area was 6 inches overbank and the design conditions were experienced. The ice revetment survived breakup and was intact until the morning of the 3rd of May. On the morning of the 3rd a 385 foot section of the ice revetment failed exposing the riverbank to the full force of the river. Time-lapse photography showed a large crescent shaped slump in the area of the failed ice revetment. Two additional slumps established themselves in the area of the ice revetment by the end of May. During the latter part of June, a larger slump area was in evidence at the downstream end of the ice revetment area. The slump areas were subaqueous failures of the riverbank resulting in a upper bank failure.

5. Maintenance History No repairs were made to the ice revetment. The ice revetment failed to reduce the erosion rate in the area of complete failure. The possibility of further slumping of the riverbank, involving the railroad spur, led to the construction of a 1310 foot revetment consisting of 36,070 tons of unclassified rock and 2147 cubic yards of borrow which was completed in August of 1979 of a cost of \$16,500 per 100 lineal feet.

6. Discussion of Any Possible Adverse Effects Caused by Construction The ice revetment acted as a hard point while it was being constructed and during breakup. The deep scour at the toe of the ice revetment and perhaps the ice revetment itself contributed to the large slump which caused its failure.

EVALUATION

1. Social and Political Acceptance and How Well Structure Fits Natural Surroundings Social and political acceptance was considered favorable since once the ice revetment melts there is virtually no environmental impact. While the ice revetment was in place, it fit its natural surroundings quite well.

ITEM 34b-1
(Sheet 3 of 4)

2. How Well Structure Functions for Design Conditions The ice revetment functioned extremely well in protecting the riverbank from ice during breakup. Once the 385 foot section of ice revetment failed the riverbank was unprotected in that area and erosion was rapid. The 300 foot section of the ice revetment that melted in place continued to protect the riverbank until it melted. Photos 2 through 12 show the ice revetment as constructed, breakup, the ice revetment failure, and the subsequent erosion. Figure 2 shows bankline surveys of the site.

3. Comments on Initial Cost and Maintenance Cost The ice revetment cost was estimated to be \$3,000 to \$4,000 per 100 lineal feet (1979 costs). Maintenance costs are not involved since the ice revetment will ultimately melt in place and not require any maintenance.

4. Aesthetics of Structure The aesthetics of an ice revetment are considered to be favorable.

5. Need for Further Monitoring or Possible Visit by Evaluation Committee No further monitoring of the ice revetment site will be done. A visit from the evaluation committee is not necessary.

6. Discussion of Good and Bad Points of Structure and Any Possible Recommendations for Improvements or Whether Structure Should Not Be Considered for Further Use An ice revetment provides good riverbank protection from ice damage during breakup. An ice revetment would not be recommended for cases of severe erosion but may provide some reduction in the erosion rate in cases of moderate erosion. An ice revetment leaves little environmental impact since it melts in place. An ice revetment in conjunction with a riverbank freezing operation would probably be more effective.

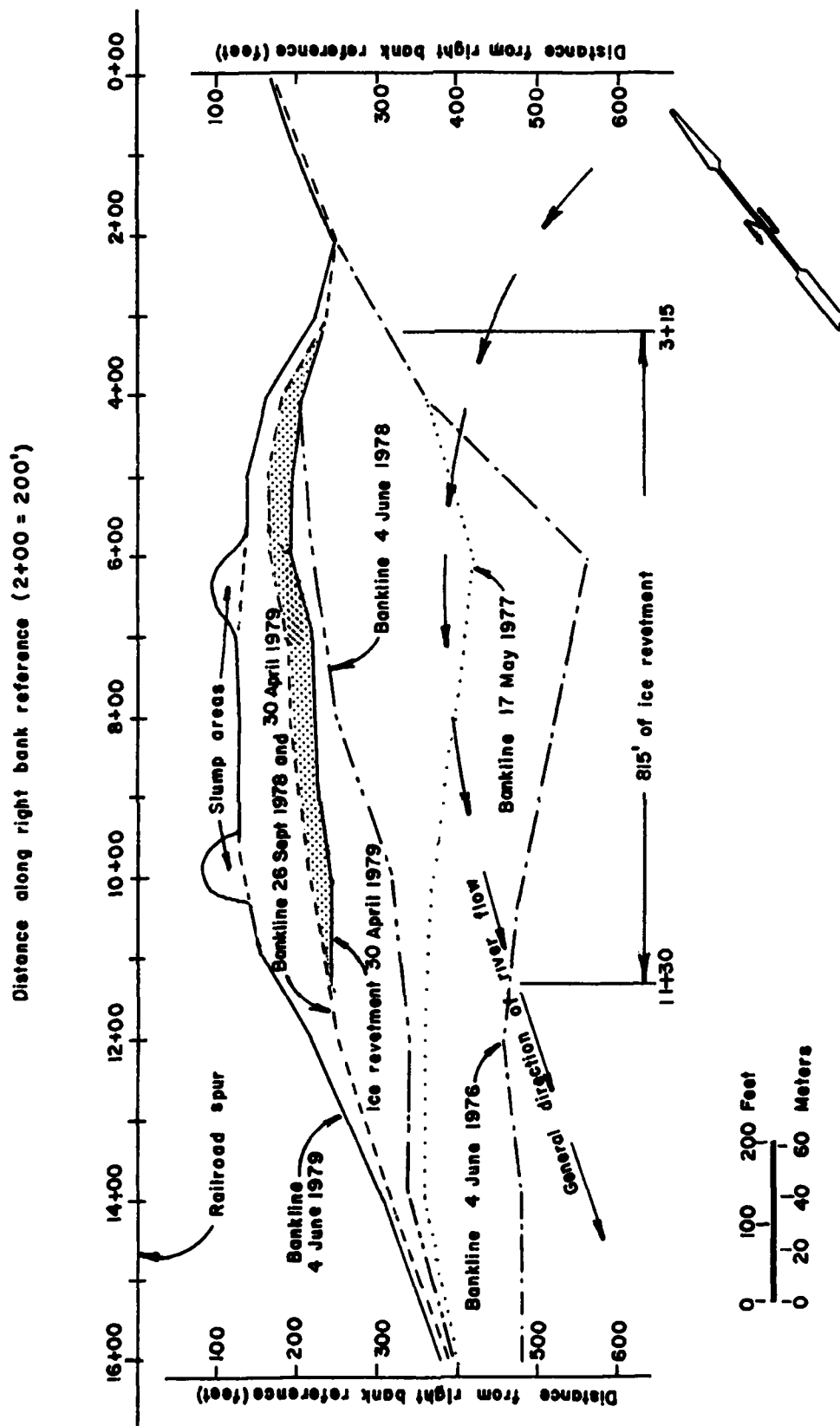
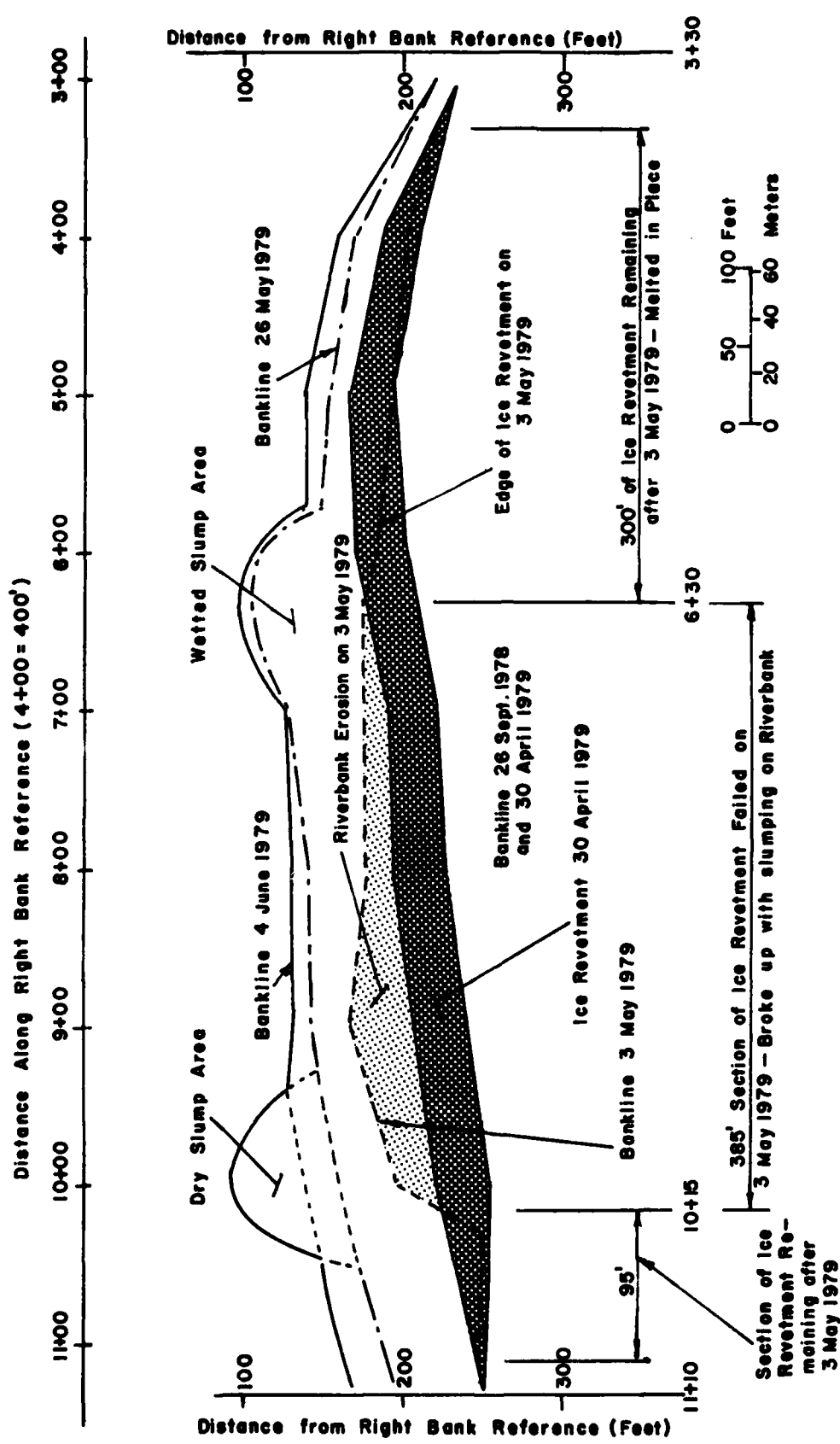


FIGURE 1 RIGHT BANK REFERENCE AND BANK-LINE SURVEYS - ICE REVETMENT SITE

ITEM 34b-2

H-34b-7



ITEM 34b-3

H-34b-8

FIGURE 2 BANK-LINE SURVEYS SHOWING ICE REVETMENT FAILURE.

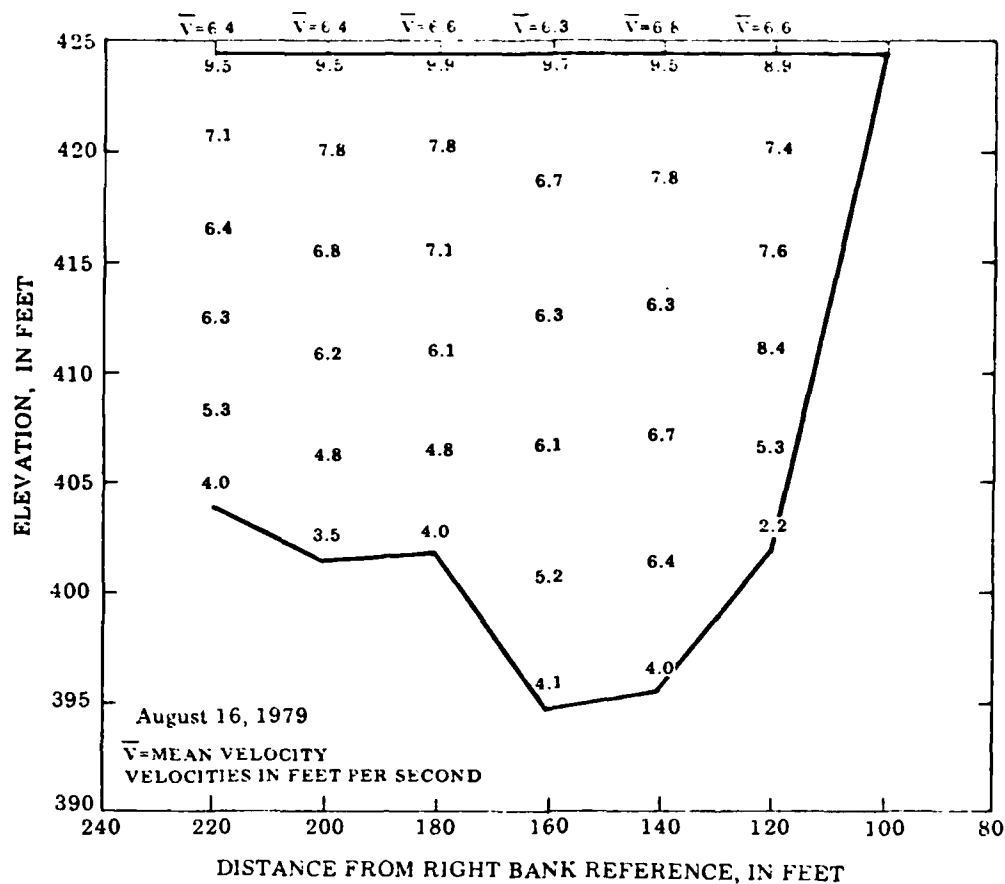
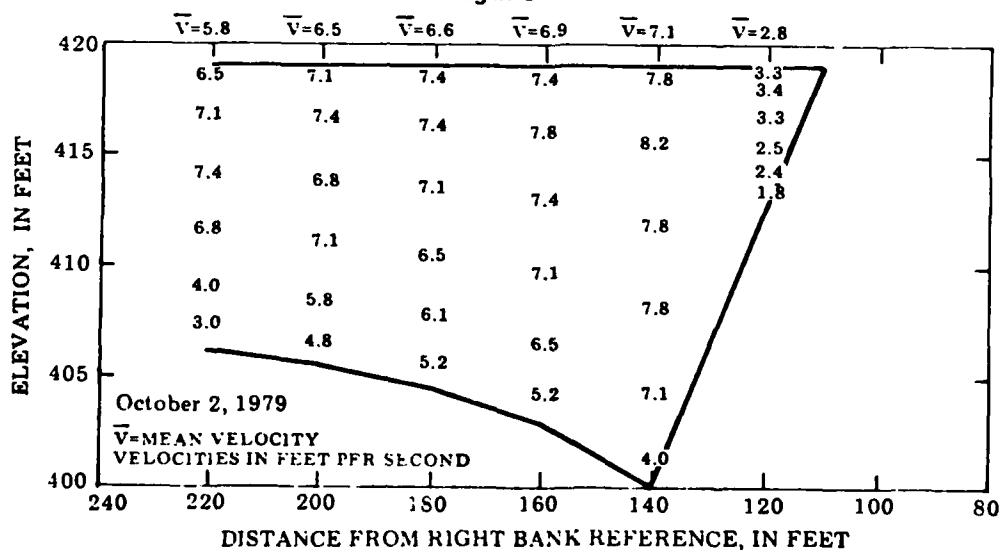


Figure 3

Partial cross sections and velocity distributions near right bank, station 8+100, ice revetment site.

Figure 4



ITEM 34b-4

H-34b-9

DIKE

2" x 4" Supports
3/8" Plywood Sheets
Polyethylene on Inner
Face Poly Tube Seal.

DIKE

2" x 4" Supports

3/8" Plywood Sheets

Polyethylene on Inner Face Poly Tube Seal.

20' to 30'

10'

2"

3/8 Plywood, 2'-3' high, Nailed to Timber, Inner Face Sealed with Polyethylene.

Access Road

Snow Cover

Polyethylene Tube Filled with Water to Seal Dike - optional.

Riverbank

Riverbank Should Freeze to a deeper depth if snow cover is removed.

Idealized Shape for Completed Ice Revetment.

Submersible Pump

Ice Sheet

Hose

20' to 30'

2'-3' to 4'

NOT TO SCALE

NOT TO SCALE

FIGURE 5 ICE REVETMENT - TYPICAL DETAIL



Photo 1 Location of ice revetment site on the Tanana River near Fairbanks, Alaska,
4 June 1978, river discharge is 21,600 cfs.

ITEM 34b-6

H-34b-11



Photo 2 31 March 1979, river discharge is 4,600 cfs,
ice building for revetment completed, forms removed.



Photo 3 13 April 1979, river discharge is 5,200 cfs,
sawdust spread on top of ice revetment.



Photo 4 28 April 1979, river discharge is 18,000 cfs, more sawdust spread
onto riverbank, depressed ice sheet shows overflow in front of revetment.

ITEM 34b-7

H-34b-12



Photo 5 1 May 1979, river discharge is 58,000 cfs, during breakup ice sheet broke up on top of ice revetment.



Photo 6 2 May 1979, river discharge is 52,000 cfs, post breakup, ice revetment intact, river stage is dropping.

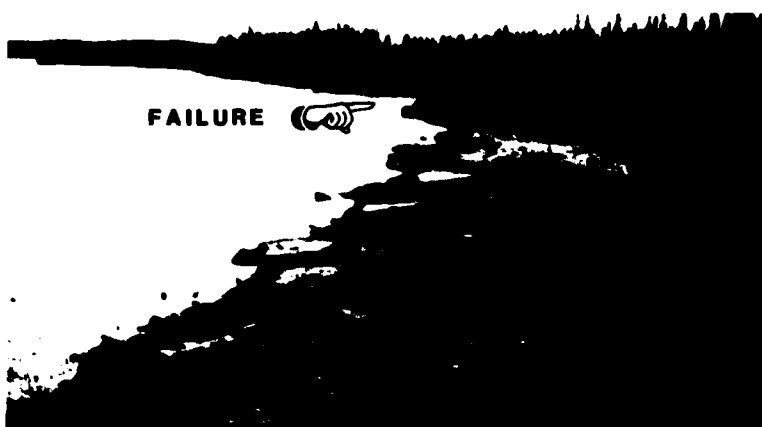


Photo 7 3 May 1979, river discharge is 47,000 cfs, ice revetment failure shown on downstream end, revetment still intact on upstream end

ITEM 34b-8



Photo 8 5 May 1979, discharge is 55,000 cfs, slumping and erosion of riverbank on downstream end, ice revetment still in place on upstream end.



Photo 9 12 May 1979, river discharge is 27,400 cfs, ice revetment completely melted, riverbank erosion under way on upstream end.



Photo 10 4 June 1979, river discharge is 30,200 cfs, riverbank still frozen under sawdust, erosion progressing rapidly.

Ice Revetment (Failure)

ITEM 34b-9

H-34b-14

**FISHER RIVER
LIBBY, MONTANA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Fisher River River Mile 0.0-10.9 Side L/R
Local Vicinity Near Libby, Montana Lat N48°17' Long W115°17'
At/Nr City Libby County Lincoln State MT Cong Dist 1
CE Office Symbol NPS Responsible Agency US Army Engineer District, Seattle
Site Map Sources General Plan Map, Libby Dam Project, Great Northern Line Change,
US Army Engineer District, Seattle (1965)
Land Use Primary land-use activities are timber production
and cattle ranching

(2) Hydrology at or Near Site

Stage Range 2136.37 to 2143.29 ft; Period of Record 1967 to 1978
Discharge Range 35 to 7280 cfs; Velocity Range _____ to _____ fps
Sediment Range 0.2 to 91,200 tpd; Period of Record 1967 to 1976
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average 14 fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) GM Bed (USCS) GW
Data Sources Libby Dam Project Office - Libby, Montana
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection Major channel changes along the Fisher River were required to
relocate the Great Northern Railroad east/west main line.

Erosion Causative Agents The Fisher River channel between Jennings, MT, upstream to
the Fisher's confluence with Wolf Creek was shortened by 4,815 ft, increasing
the bed gradient from 34 to 37 ft/mile.

Protection Techniques Stone riprap was placed on side slopes and 67 rock groins were
placed through the 16 channel changes.
General Design Groins were constructed of rock placed over a 12-in.-thick layer of
gravel bedding. Rock extended to a depth of 2.5 ft below and 2 ft above the
channel bed.
Project Length 22,625 ft; Construction Cost \$ 5,103,505.11 Mo/Yr Completed Fall 1967

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Date) Twenty-eight groins were completed prior to the spring 1967 runoff. Four groins were badly damaged during the runoff and 24 others were modified to varying degrees. The remainder of the groins were constructed of stone weighing from 200 to 2000 lbs, with 50 percent weighing over 1000 lbs.

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Seattle District, CE (NPS) conducted annual inspections from 1967-1971.

Documentation Sources NPS, Libby Dam Project Office

Project Effect on Stream Regime Groins have stabilized the channel bed and promoted shallow pool development.

Project Effect on Environment A study conducted by the State of Montana Department of Fish and Game indicated that the channel changes had adversely affected the aquatic environment.

Successful Aspects The project reach is stable.

Unsuccessful Aspects Stone weighing less than 200 lb moved out of groins and was swept downstream.

General Evaluation Groin rock is scattered downstream at some locations, but many groins are still intact; no riprap blankets placed on the side slope have failed. No serious channel degradation or aggregation has occurred.

Recommendations _____

(7) Additional Information, Comments, and Summary

Map No. 35.

For additional information see "Field Inspection of the Fisher River Channel Realignment Project near Libby, Montana," prepared by Malcolm P. Keown.

US Army Engineer Waterways Experiment Station, Vicksburg, MS, April 1981.

Attached Items: 35-1 Unique Project Features & Equations
35-2 Project Location 35-5 Typical Cross-Channel
35-3a,b,c Project Realignment 35-(6-8) Project
35-4 Cross Section Photographs

Unique Project Features

1. Several major relocations were required in conjunction with the construction of Libby Dam on the Kootenai River in Montana. One of these relocations was the east/west main line of the Great Northern Railroad (Figure 1). Extensive channel realignment was necessary to provide sufficient width to accommodate both the streambed and railroad bed. The modified channel between Jennings upstream to the Fisher's confluence with Wolf Creek constitutes the Section 32 existing site.

2. A total of 16 channel changes were required (Figures 2a-2c), resulting in a channel loss of 4,815 ft which steepened the gradient from 34 to 37 ft/mile. Sixty-seven groins were placed through the channel changes to prevent degradation and to mitigate impacts on the fish communities. Groins were constructed of rock placed over a 12-inch-thick layer of gravel bedding. Rock extended to a depth of 2.5 ft below channel grade to prevent undercutting and 2 ft above channel grade to provide fish resting pools and reduce stream velocities (Figures 3 and 4). Views of channel change 17 during construction and after completion are shown in Figure 5. The project was completed in the fall of 1967.

Evaluation

3. In September 1967, US Army Engineer District Seattle (NPS), conducted an inspection of the completed groins in channel changes 2, 11, 12, and 13 and prepared a report on the findings. Construction records indicated that 28 groins were in place prior to the spring 1967 runoff. Four of these groins were badly damaged, three to the extent that they were hardly recognizable, and the remaining 24 all appeared to have been modified to varying degrees, mostly in the weir sections. Stone weighing less than 200 lbs generally appeared to have been moved out of the groin and swept downstream; therefore, NPS recommended that the remainder of the groins be constructed of stone ranging from 200 to 2,000 lbs, with 50 percent of the stone weighing over 1,000 lbs. These recommendations were accepted and the remainder of the groins were constructed according to these guidelines. Additional inspections of the groins were conducted through August 1971.

ITEM 35-1
(Sheet 1 of 2)

4. After completion of the channel realignment project, the State of Montana Department of Fish and Game conducted a Federally funded study (1 July 1967 30 June 1972) to evaluate the impact of the channel realignment on the fish community. This study indicated that the groins installed in the channel changes had been successful in promoting shallow pool development and stabilizing the channel bed, but they had produced considerably more sucker habitat than trout habitat. Further, the report indicated that the channel realignment of Fisher River had adversely affected the aquatic environment by: (a) increasing sediment loads, (b) destroying riparian vegetation, (c) raising water temperatures in denuded channels, (d) shortening the stream length, and (e) producing unsuitable physical habitat for game fish. The report concluded that bridges should be utilized whenever possible as an alternative to channel realignment.

5. The US Army Engineer Waterways Experiment Station Section 32 evaluation team inspected the Fisher River Channel Realignment Project on 8 August 1979. At that time the project reach was stable with no serious channel degradation or aggradation evident. Although groin rock was scattered downstream at some locations (Figure 6), many of the groins were intact (Figure 7). No riprap blankets placed on side slopes had failed.

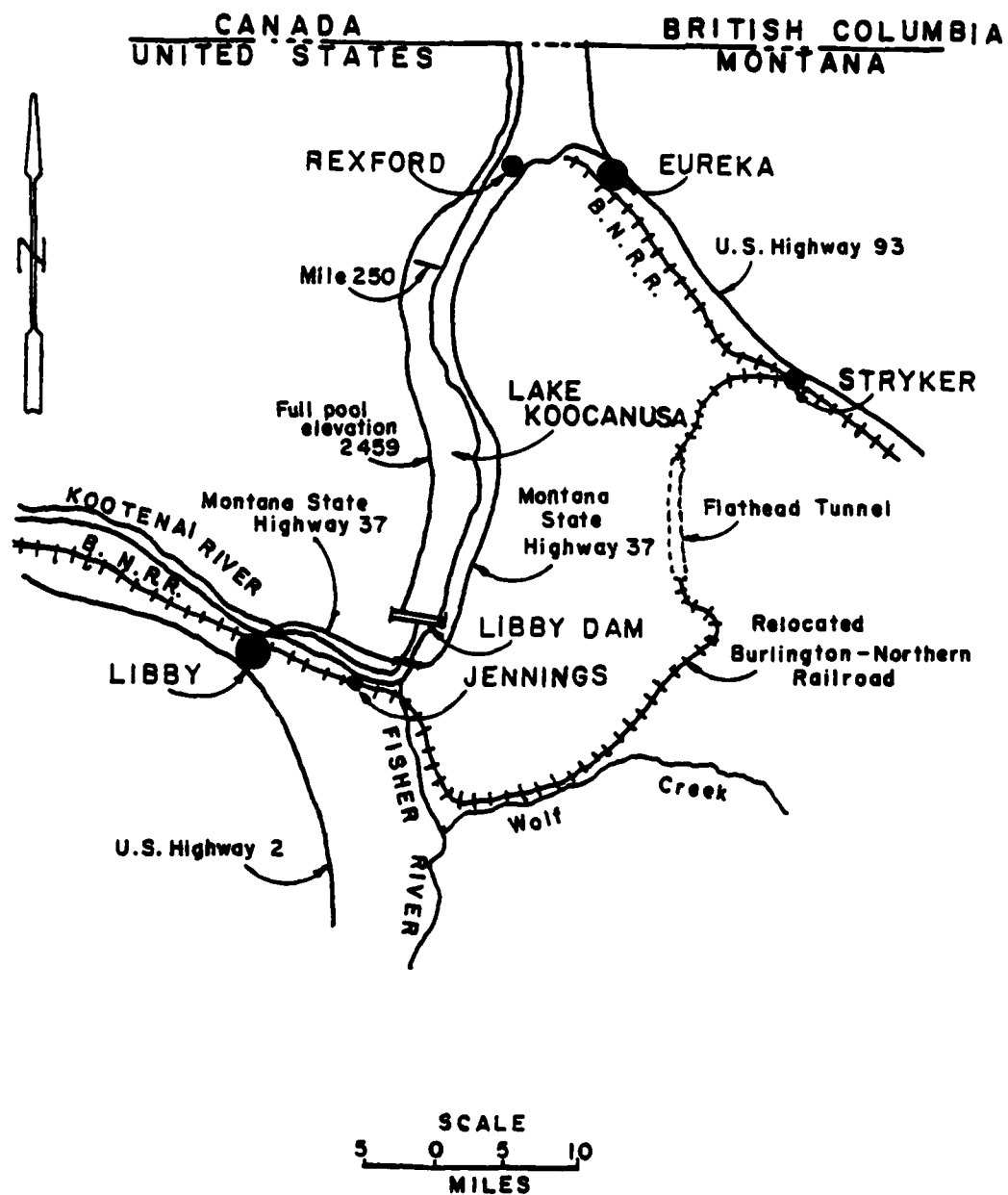
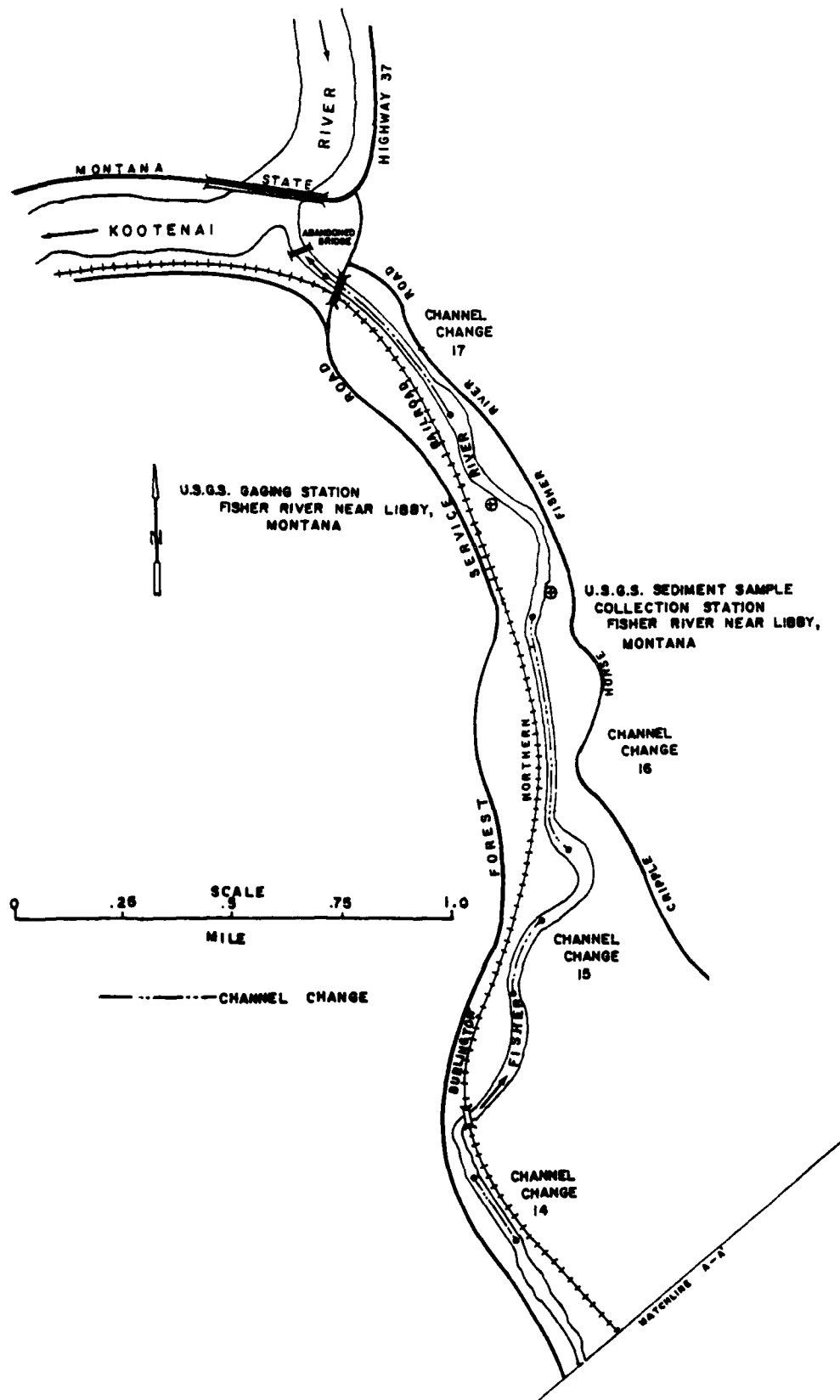


Figure 1. Libby Dam Project and Reservoir location

ITEM 35-2

H-35-5



ITEM 35-3
(Sheet 1 of 3)

Figure 2a. Fisher River Realignment Project

H-35-6

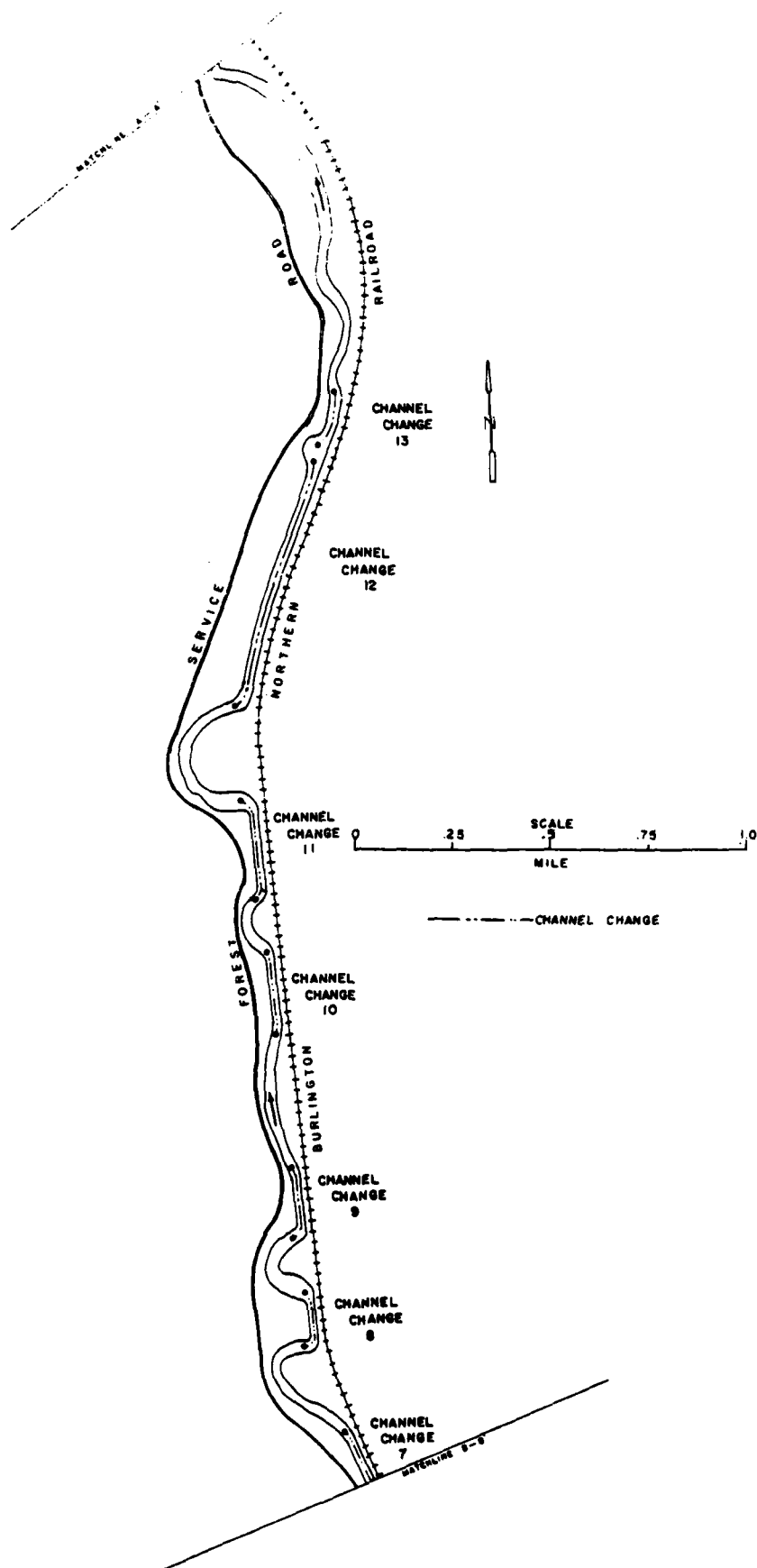


Figure 2b. Fisher River Redalignment Project

ITEM 35-3
(Sheet 2 of 3)

H-35-7

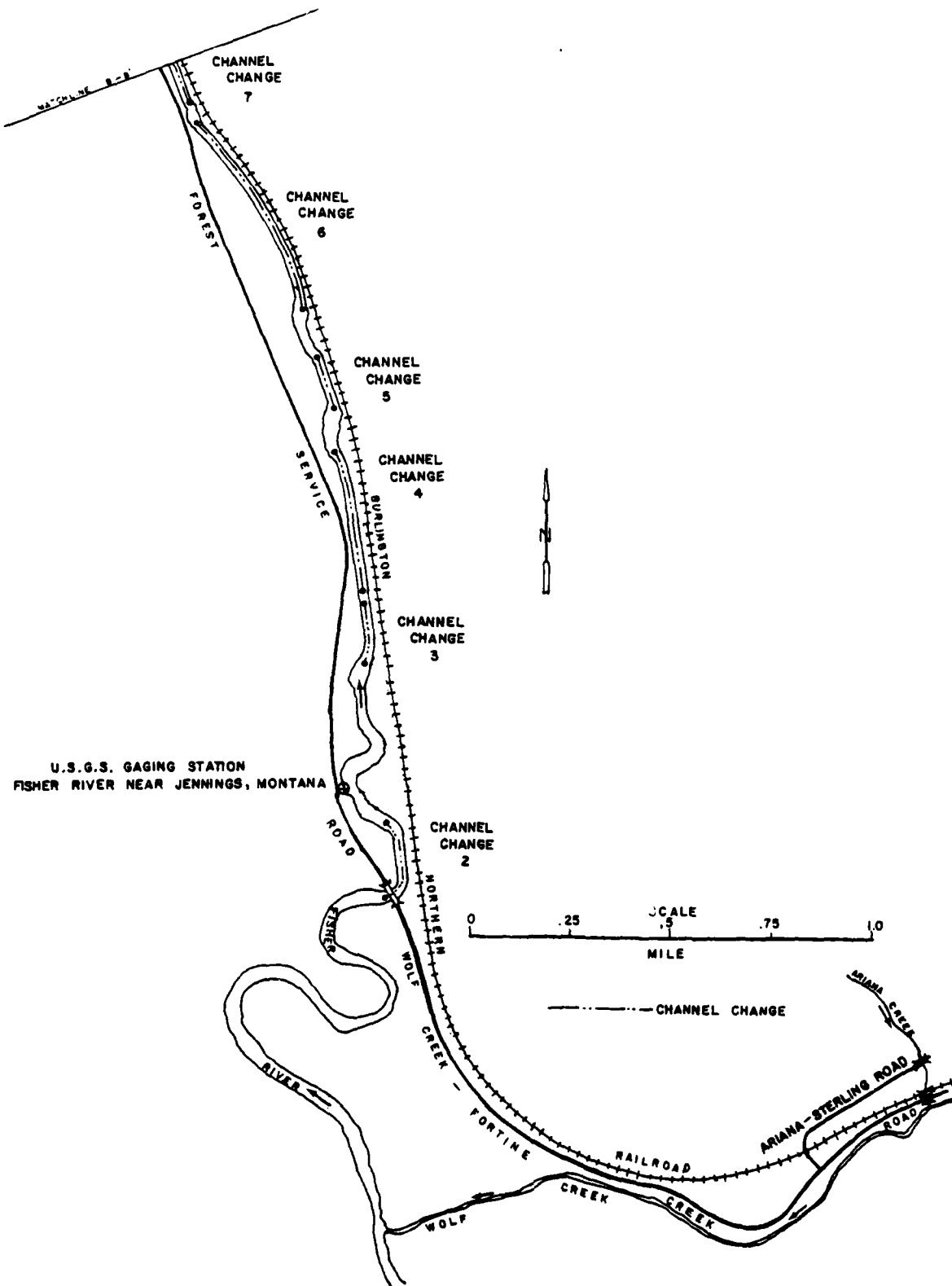


Figure 2c. Fisher River Realignment Project

ITEM 35-3
(Sheet 3 of 3)

H-35-8

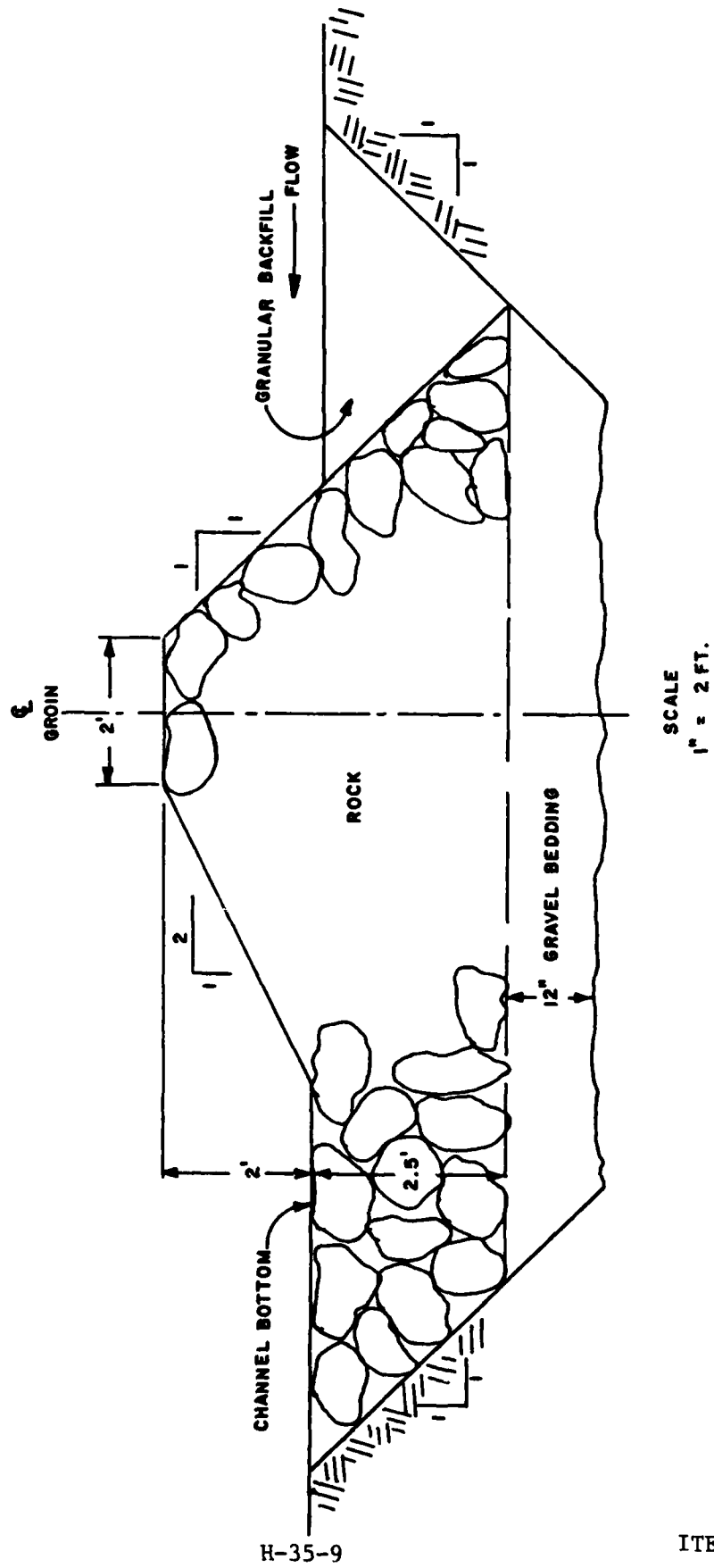


Figure 3. Cross-sectional view of groins (adapted from Great Northern Line Change, Ariana Creek to Jennings, Groins, Drawing E-53-33-81, December 1965, NPS)

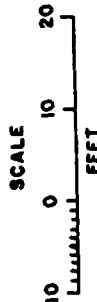


Figure 4. Typical cross-channel view of groin and railroad embankment (adapted from Great Northern Line Change, Ariana Creek to Jennings, Groins, Drawing E-53-33-81, December 1965, NPS)

H-35-10



a. Under construction (24 Aug 67)



b. Completed channel change (1 Sep 67). Measured discharge when photograph was taken was 11 cfs. Note railroad bed under construction at right

Figure 5. Channel change 17.

ITEM 35-6

H-35-11



a. Condition of groins on 13 Aug 70, three years after placement.



b. The groins were still recognizable on the date of the WES inspection visit (8 Aug 79); however, the rock was widely scattered

Figure 6. Channel change 17 viewed upstream from bridge
ITEM 35-7



Figure 7. Channel change 9 on the date of the WES inspection visit (8 Aug 79). Many of the groins were still intact

ITEM 35-8

H-35-13

**HOCKING RIVER
ATHENS, OHIO**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Hocking River River Mile 29.2-34.2 Side Right and Left
Local Vicinity Athens, Ohio Lat N39°20' Long W82°05'
At/Nr City Athens County Athens State OH Cong Dist 10
CE Office Symbol ORHED-HH Responsible Agency Hocking Conservancy District
Site Map Sources Contract Drawings and USGS 15' Quads
Land Use Information Sources Athens County Planner

(2) Hydrology at or Near Site

Stage Range 2+ to 20.41 ft; Period of Record 19 71 to 19 76
Discharge Range 50 to 17,600 cfs; Velocity Range 0.25 to 3.25 fps
Sediment Range 0.5 to 44,000 tpd; Period of Record 19 54 to 19 64
Bank-full Stage 27.24 ft; Flow 39,000 cfs; Average Recurrence Interval 60 yr
Bank-full Flow Velocity: Average 5.0 fps; Near Bank 5.6 at foe (computed)
Comments Maximum sediment load is for the March 1963 flood

(3) Geology and Soil Properties

Bank (USCS) Graphic logs Bed (USCS) Graphic logs
Data Sources Contract Dwgs 024-PA-10/1-7
Groundwater Bank Seepage None evident
Overbank Drainage A drainage interceptor system, consisting of shallow ditches parallel to top of banks and spills over channel slopes at 2000 ft intervals via grouted flumes

(4) Construction of Protection

Need for Protection To protect the new channel slopes composed mainly of fine sands against erosive forces.
Erosion Causative Agents Runoff down the channel slopes and boundary shear forces generated by the channel flow.
Protection Techniques Stone slope protection, coarse gravel and crown vetch.
General Design Contract Dwg 024-PA-16/6
Project Length 26,000 ft; Construction Cost \$ 5.2 million Mo/Yr Completed 9/71

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 6,000⁺ cfs on 14 Jan 1980

Repairs and Costs (Item, Cost, Date) None associated with channel bank slope protection techniques.

Comments No erosion of channel banks observed. Dredging needs to be completed especially at interior bend locations.

(6) Performance Observations and Summary

Monitoring Program Semiannual inspections

Documentation Sources _____

Project Effect on Stream Regime Channel straightening and widening of existing sinuous river.

Project Effect on Environment Land was reclaimed without degradation of the environment and the natural landscape was preserved.

Successful Aspects Excellent performance of the channel bank protection.

Unsuccessful Aspects The crown vetch is a perennial vegetative cover and is susceptible to permanent damage during long periods of inundation.

General Evaluation The channel slope protection seems to be performing its design function. There were no bank erosion problems observed during the inspection.

Recommendations Continue channel dredging and reestablish crown vetch on channel bank slopes as needed.

(7) Additional Information, Comments, and Summary

Map No. 36. Photos and Contract Dwg 024-PA-16/1 showing station locations where photos were taken during the insp. are included as attachment #1.
The photos were taken in approximately the same locations as in the previous inspection for comparison purposes.

Attached Items:

36-1 Project summary and location

36-2 Aerial view 1970

36-3 "

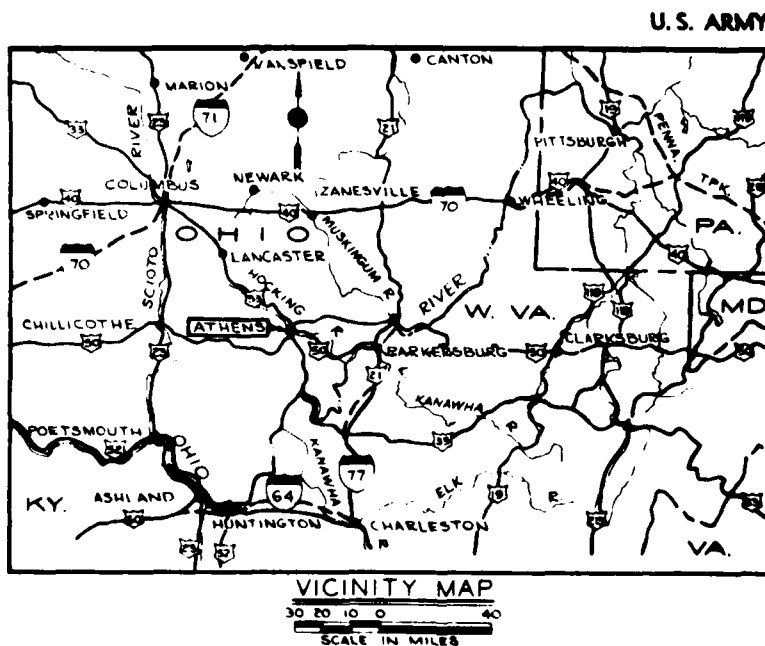
36-4 Cross sections

36-5 Aerial view 1975

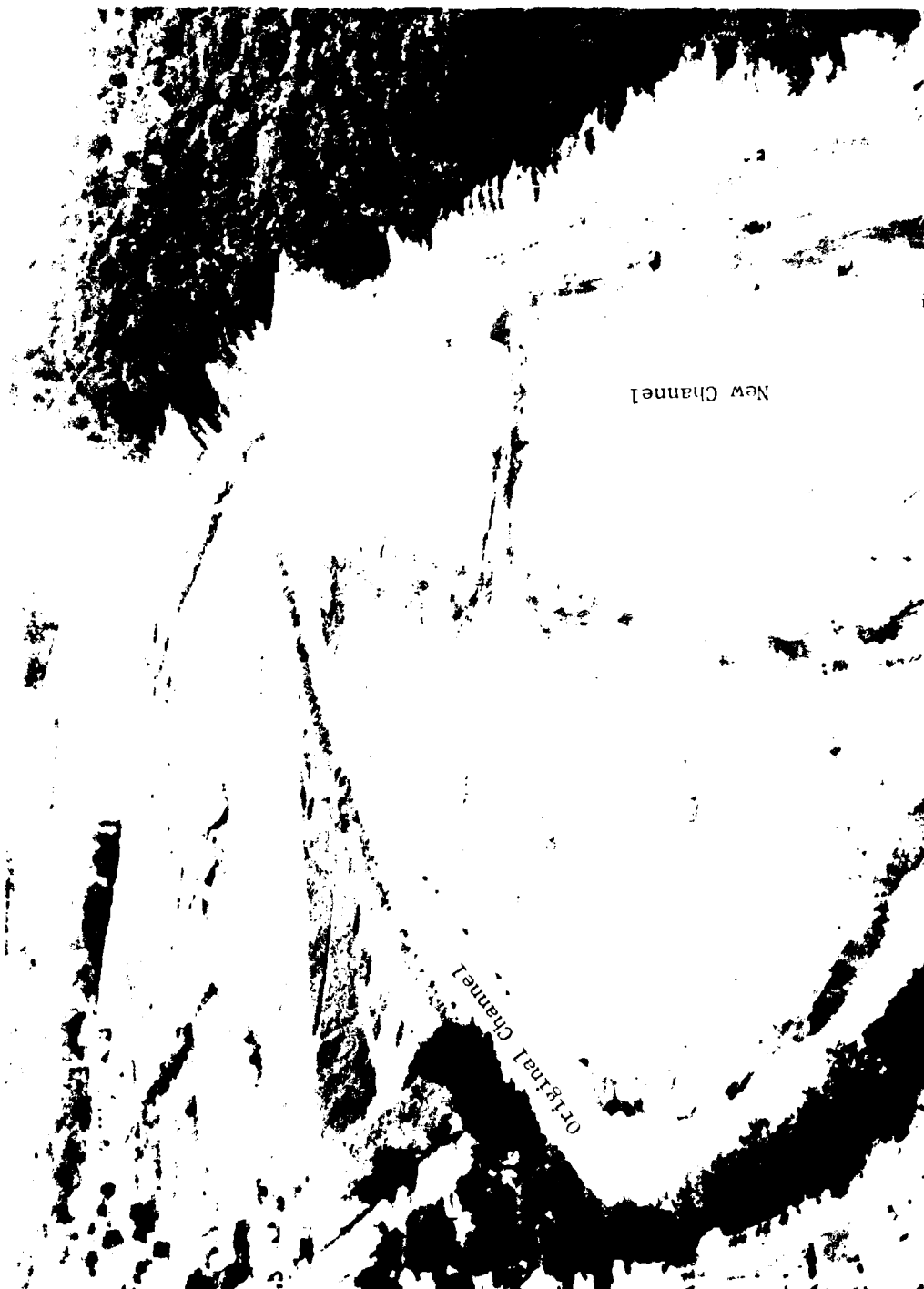
36-6-8 Project photographs

HOCKING RIVER LPP AT ATHENS, OHIO

The Local Protection Project (LPP), completed 1971, consists of a straight to moderately curved and somewhat shorter channel replacing sinuous reach of the Hocking River that flowed through the city of Athens, Ohio. Slopes on the outside of bendways were protected by stone riprap placed over graded bedding. Other reaches were protected with a gravel blanket. As an integral part of the project, Huntington District, CE (ORH), designed a drainage interceptor system to protect against erosion from overbank drainage. The upper banks were seeded with crown vetch, and disturbed areas were seeded with fescue. ORH inspected the protection in 1974 and found that the lower bank near the gravel-crowned vetch interface lacked substantial growth due to prolonged submergence. Annual inspections by the Hocking Conservancy District (local sponsor) indicate generally satisfactory performance of the protection scheme; however, these inspections did note riprap failures at the confluences of Coates Creek and Pork Riffle Run, two small tributaries within the project reach. They attribute these failures to overexcavation of the channel at the confluences of these two tributaries with the main stream. At the date of the WES inspection (June 1978), the crown vetch and other volunteer vegetation had become well established and covered much of the riprap and gravel on the slopes. In a few places the riprap was still visible but it appeared to be undisturbed. One of the serious problems associated with this project is channel deposition that has led to the formation of bars. Most of this problem can be attributed to the wider channel that promoted deposition by reduced flows. In 1979 and 1980, ORH personnel made an inspection of the LPP and found no evidence of serious revetment failure nor did they note problems with channel deposition. They do recommend, however, that the channel be dredged to project depth since the new channel is attempting to braid itself.



ITEM 36-1



Hocking River at Athens, Ohio. Downstream aerial view of construction in progress as it appeared in May 1970

ITEM 36-2

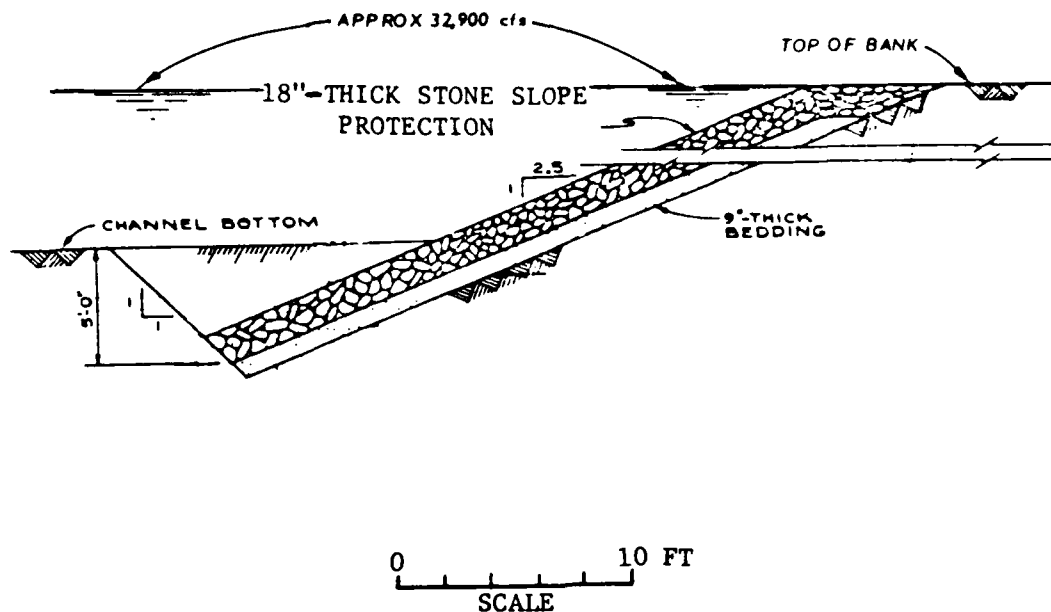
H-36-4



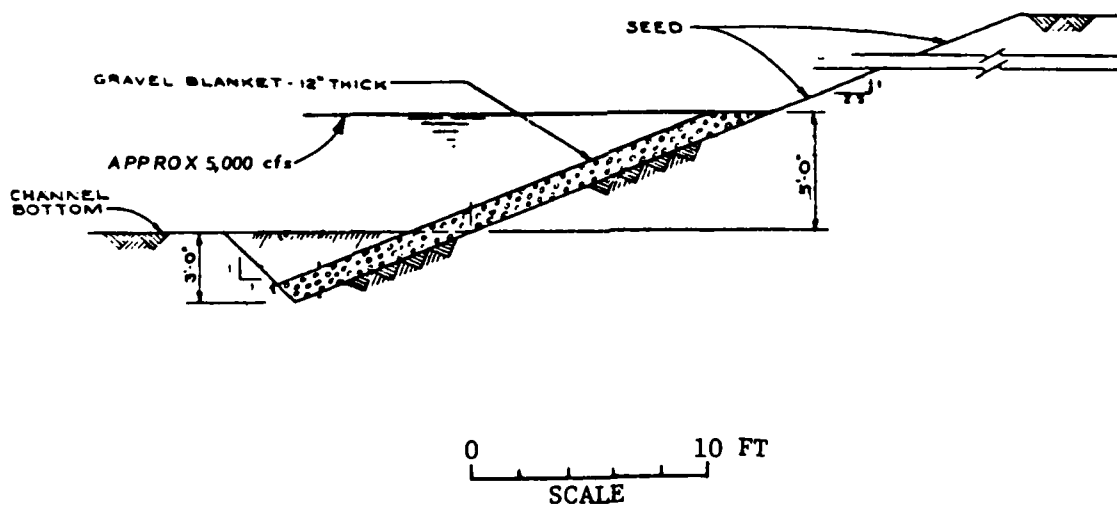
Hocking River at Athens, Ohio. Downstream aerial view of construction
as it appeared in April 1971. Compare with Item 36-2

ITEM 36-3

H-36-5



Hocking River at Athens, Ohio. Typical stone slope protection.
 (Source: Hocking River Local Protection Project, Plan and Profile, sta 86+70 to 108+81, Drawing No. 024-PA-16/6, Sheet 6 of 40, ORH, November 1968)



Hocking River at Athens, Ohio. Typical gravel blanket protection.
 (Source: Hocking River Local Protection Project, Plan and Profile, sta 86+70 to 108+71, Drawing No. 024-PA-16/6, Sheet 6 of 40, ORH, November 1968)

ITEM 36-4

H-36-6



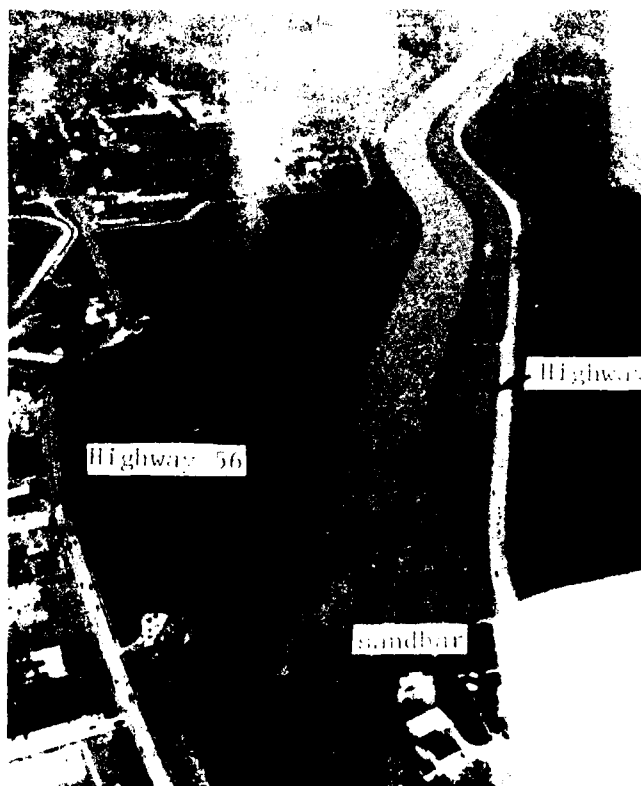
Hocking River at Athens, Ohio. Upstream view of completed project (March 1975)

ITEM 36-5

H-36-7



a. Downstream view from upper limit of project



b. Downstream view of project reach

Hocking River at Athens, Ohio. Aerial views taken from above upstream limit of project. Compare position of sandbar with a (21 June 1978).

ITEM 36-6

H-36-8



Looking downstream from Whites Mill Dam. (Dredged material on left.) April 1980.



Looking downstream from U.S. Rt. 50 Bridge (Richland Ave).

Hocking River at Athens, Ohio.
Two Views After Dredging. (1980)

H-36-9

ITEM 36-7



Looking downstream along left descending bank.



Looking upstream along left descending bank.

ITEM 36-8

Hocking River at Athens, Ohio. (1980)

H-36-10

**OHIO RIVER
CLOVERPORT, KENTUCKY**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Ohio River River Mile 711 Side Left
Local Vicinity Cloverport, KY Lat N37°45' Long W86°45'
At/Nr City Cloverport County Breckinridg State KY Cong Dist 2
CE Office Symbol ORLED-H Responsible Agency City of Cloverport
Site Map Sources Louisville District, Corps of Engineers
Land Use Information Sources City of Cloverport

(2) Hydrology at or Near Site

Stage Range 382 to 408 ft; Period of Record 1931 to 1969
Discharge Range 14,000 to 865,000 cfs; Velocity Range 1 to 6 fps
Sediment Range - to - tpd; Period of Record 19 - to 19 -
Bank-full Stage 410 ft; Flow 1,500,000 cfs; Average Recurrence Interval 1,000 yr
Bank-full Flow Velocity: Average 7 fps; Near Bank 7 fps
Comments Groundwater seepage was major cause of instability

(3) Geology and Soil Properties

Bank (USCS) Silty clay Bed (USCS) Silty sand
Data Sources Visual inspection
Groundwater Bank Seepage None visible; some noted prior to construction
Overbank Drainage None
Comments None

(4) Construction of Protection

Need for Protection To protect main business street which is also a US high-
way
Erosion Causative Agents Subsurface drainage, rapid drawdown, wind and boat
wave action.
Protection Techniques Minimum weight, graded riprap over stone bedding.
General Design Stone size, normally sufficient for Ohio River sites, is
marginal because of steep slope (1.5H:1V)
Project Length 350 ft. Construction Cost \$ 35,000 Mo/Yr Completed Oct 73

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Elev. 400,610,000 cfs

Repairs and Costs (Item, Cost, Date) None

Comments: None

(6) Performance Observations and Summary

Monitoring Program Inspections conducted on 20 June 1978, 9 May 1979, 10 April 1980, and 13 November 1981

Documentation Sources

Project Effect on Stream Regime None

Project Effect on Environment None

Successful Aspects Stable, economical, little or no maintenance.

Unsuccessful Aspects Minor slippage due possibly to collisions with ice or vessels. Some displacement of larger stones noted in Apr 80 inspection.

General Evaluation Effectively demonstrates that 26 lb max. graded riprap is generally adequate for current and wave action on Ohio River.

Recommendations Future Ohio River projects not subject to unusual turbulence, should be protected by a 26 lb max. grade stone (layer thickness 12 in.) or slopes flatter than 2H:1V.

(7) Additional Information, Comments, and Summary

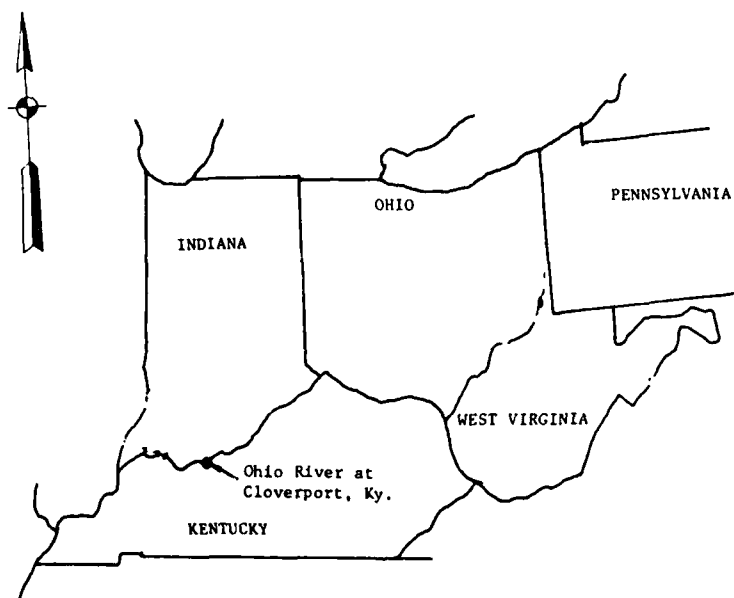
Map No. 37. 26 lb graded riprap should have 50% by weight between 5 lbs max wt. and 11 lbs max wt. 15% by weight should be between 5 lbs max. wt and 2 lbs max wt. For slopes steeper than 2H:1V, layer thickness should be increased to 15" to 18".

Attached Items:

- 37-1 Project summary and location
- 37-2 Project plan
- 37-3 Cross section and photographs
- 37-4 Photos after substantial flow

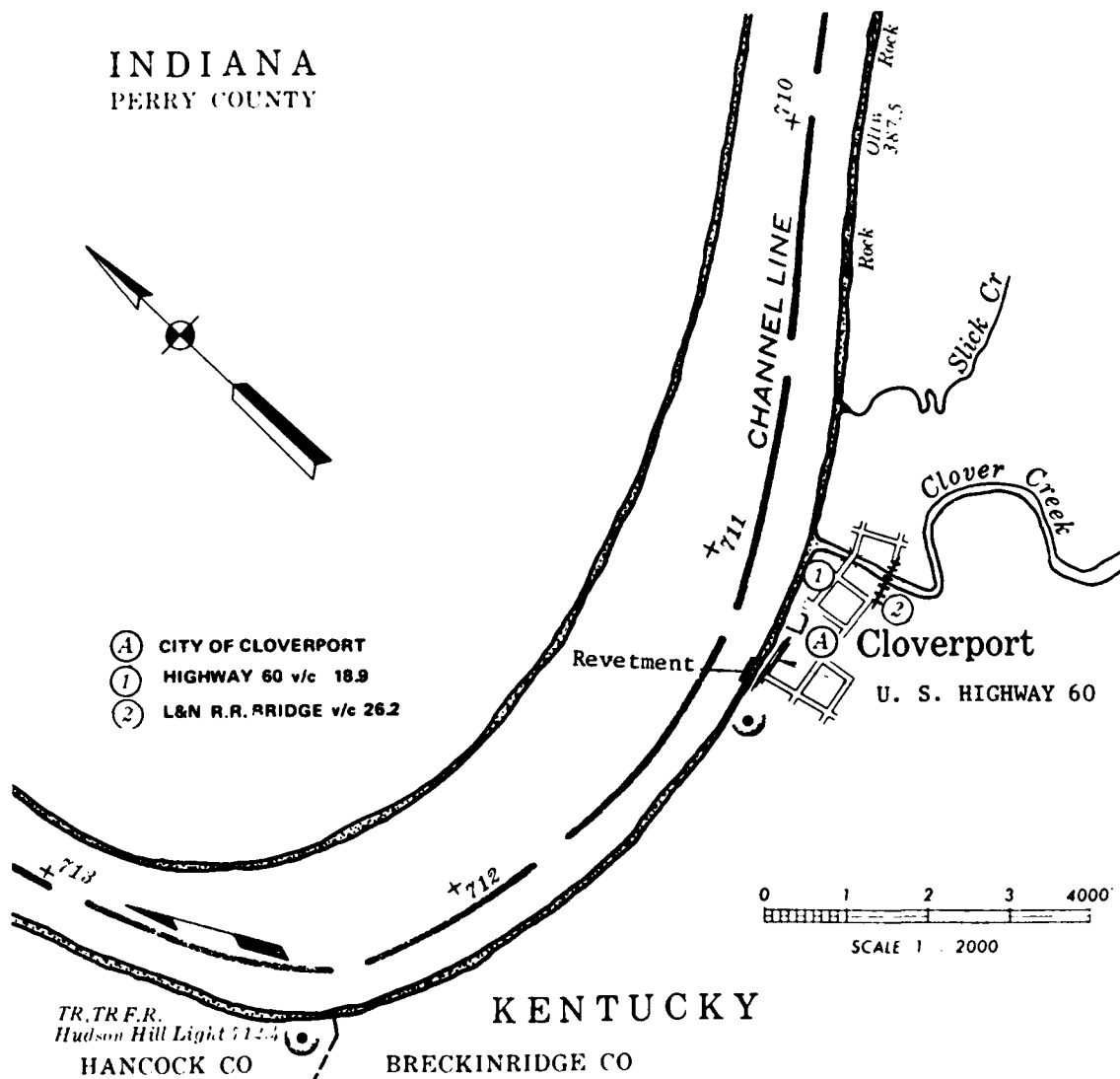
OHIO RIVER AT CLOVERPORT, KENTUCKY

In late 1973, Louisville District, CE (ORL), constructed a 350-ft-long stone riprap revetment, which was placed on gravel and crushed stone bedding, to protect a portion of the left bank of the Ohio River at Cloverport, Kentucky. The maximum weight of the stone used was 26 pounds, which was considerably lighter than that normally used on the Ohio River. This revetment replaced earlier unsuccessful attempts (e.g., dumping stone, tile, etc., from top bank) by local interests to protect the eroding streambank. All areas disturbed by the contractor but not protected with stone were seeded. Slope of the revetment is 1.0V on 1.5H. At the time of the June 1978 inspection, the revetment was generally intact, and a good stand of vegetation had become established in the seeded area. The riprap and bedding layer appeared to be effective in controlling pre-project erosion resulting from groundwater seepage and saturated bank failure while adequately resisting wave action from wind and river traffic. However, during early 1978, the structure experienced two five-year flood flows. As a result, the riprap has been shifted from one small section about 15 ft long by 2 ft wide near the toe of the bank. This damage may have been caused or aggravated by the grounding of a vessel. Additional inspections conducted by ORL in May 1979 and April 1980 indicated that no further changes in the revetment condition had occurred since June 1978 inspection. ORL reports that some displacement of larger stones was noted in April 1980 and that this revetment effectively demonstrates that 26-pound maximum weight graded riprap is generally adequate protection for currents and wave action along the Ohio River. Future projects on the Ohio River not subject to unusual turbulence should be protected by 50-pound maximum weight graded stone (the next largest size) where slopes are steeper than 1V on 2.5H to provide some factor of safety.



ITEM 37-1

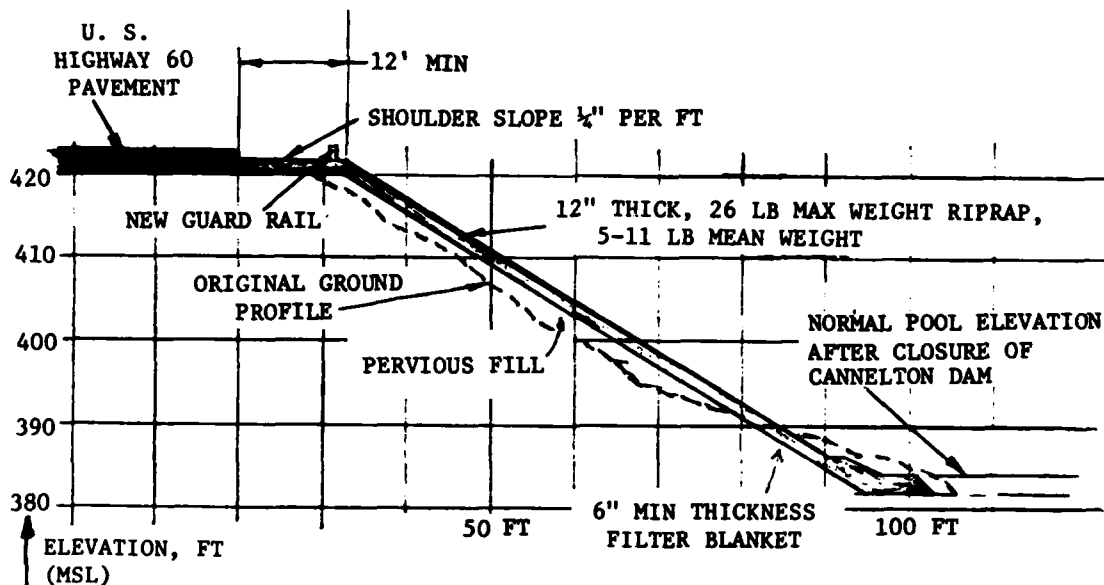
H-37-3



Ohio River at Cloverport, Ky. (Source: Chart 54, Ohio River Navigation Charts, Cairo, Ill., to Foster, Ky., U. S. Army Engineer District, Louisville, Jan. 1979).

ITEM 37-2

H-37-4



Ohio River (left bank) at Cloverport, Ky., cross section of revetment.
 (Source: File information provided by ORL)



Ohio River (left bank) at Cloverport, Ky. Vegetation had become well established along the top of the revetment at the time of the WES inspection visit (19 June 1978). The pool elevation is 348 ft. U. S. Highway 60 is to the left; the view is downstream

ITEM 37-3

H-37-5



Upper Bank



Typical Riprap Revetment

Ohio River (left bank) at Cloverport, Ky. Downstream view shows toe of revetment under wave attack due to passing barge traffic (19 Jun 1978)

ITEM 37-4

H-37-6

**OHIO RIVER
NEWBURGH, INDIANA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Ohio River River Mile 778 Side Right
Local Vicinity Newburgh Lat N37°58' Long W87°26'
At/Nr City Newburgh County _____ State IN Cong Dist _____
CE Office Symbol ORLED-H Responsible Agency City of Newburgh
Site Map Sources Louisville District Corps of Engineers
Land Use Information Sources City of Newburgh

(2) Hydrology at or Near Site

Stage Range 342 to 380 ft; Period of Record 1931 to 1969
Discharge Range 14,000 to 865,000 cfs; Velocity Range 1 to 7 fps
Sediment Range _____ to _____ tpd; Period of Record 19____ to 19____
Bank-full Stage 372 ft; Flow 500000 cfs; Average Recurrence Interval 2 yr
Bank-full Flow Velocity: Average 6 fps; Near Bank 6 fps
Comments Banks drop to deepest portion of channel.

(3) Geology and Soil Properties

Bank (USCS) Silty clay Bed (USCS) Silty sand
Data Sources Visual inspection
Groundwater Bank Seepage Some through sand levees
Overbank Drainage Extensive - controlled by storm sewers.
Comments _____

(4) Construction of Protection

Need for Protection To protect commercial district, overlook areas, private residences.
Erosion Causative Agents High stage streamflow against concave bank of bendway, wave action, seepage.
Protection Techniques Modification of quarrystone.
General Design Careful disposal of surface drainage through storm sewer outfalls.
Project Length 4000 ft; Construction Cost \$1.2million Mo/Yr Completed 1975

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Elev 377, 704,000 cfs September 1978.

Repairs and Costs (Item, Cost, Data) None.

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Inspections conducted on 8 May 1979 and 6 December 1979.

Documentation Sources As-built plans

Project Effect on Stream Regime None apparent

Project Effect on Environment None apparent

Successful Aspects Banks stabilized, surface runoff controlled.

Unsuccessful Aspects Surface runoff causing erosion at upstream project limit. Storm sewers outfalls difficult to inspect because of submergence for over 50% of the time.

General Evaluation _____

Recommendations For future projects, outfalls should emerge at a higher level with grouted riprap between outfall and normal pool. Surface runoff from upstream area should be controlled by curbs and sewers.

(7) Additional Information, Comments, and Summary

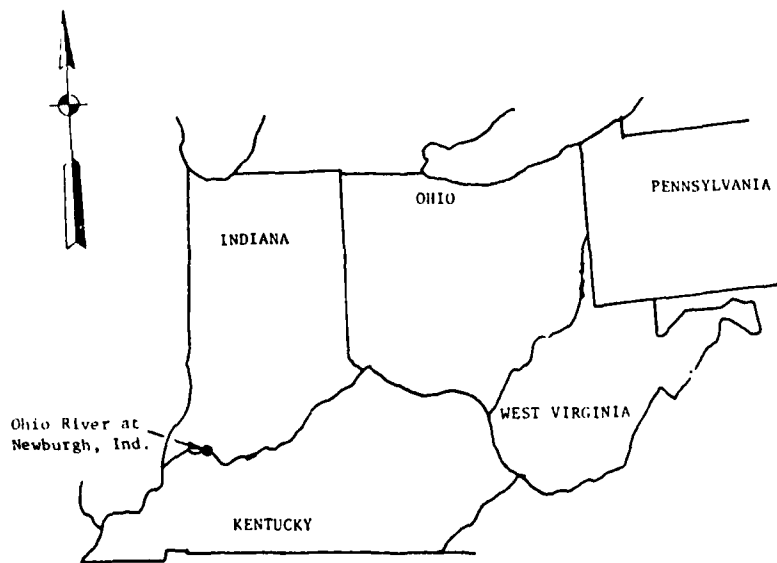
Map No. 38.

Attached Items:

- 38-1 Project summary and location
- 38-2 View of bank caving
- 38-3 Project cross section and protection
- 38-4 Views of downstream riprap

Ohio River at Newburgh, Indiana

The thalweg of the Ohio River is adjacent to the right of the Ohio River at Newburgh, Indiana. As a result this bank has been threatened by erosion. The Works Progress Administration riprapped portions of this bank in the late 1930's. In 1957, Louisville District (ORL) placed 300 additional linear feet of riprap near the intersection of State and Water Streets; additional caving occurred in 1969 and early 1970. This section was shaped and riprapped in 1970. The 1957 and 1970 bank protection projects, performed under Section 14 of the Flood Control Act of 1946, allowed ORL to take remedial action when public utilities were endangered by floodwaters. Emergency bank protection work was also undertaken in 1973 when Water Street was threatened in January 1975 and again in March of the same year as emergency bank protection projects using 50 lb. maximum weight stone were again necessary. In 1976, as part of its regular channel maintenance dredging contract with ORL, the St. Louis District CE placed hydraulic fill against the eroding bank to provide material for bank preparation. The bank was then shaped to a 1V-on-3H slope. ORL placed 6,200 ft of revetment at this site during 1976. This project was largely performed from the river using floating equipment. Where new stone protection joined existing revetment, the new riprap was adjusted to conform to the slope of the in-place material. Below the 337-ft elevation, 150-lb maximum weight quarry-run stone over a 6-in. lift of bedding material was placed in a 24-in. layer; above el 337, a 12-in. layer of 86-lb maximum weight graded riprap was used. The average top of the protection is about 30 ft above normal pool elevation and the average toe protection about 20 ft below normal pool elevation. All hydraulic fill above el 370 was covered with 4-in. topsoil and seeds. At the time of the WES visit (June 1978) and subsequent ORL visits (May and December 1979), the revetment was intact, and grass was established on the upper bank. ORL has noted some problems with surface runoff at the upstream end of the project. The District has also noted that the weight of the stone specified was possibly an overdesign and that a 50-lb maximum weight stone would have been adequate.



ITEM 38-1

H-38-3



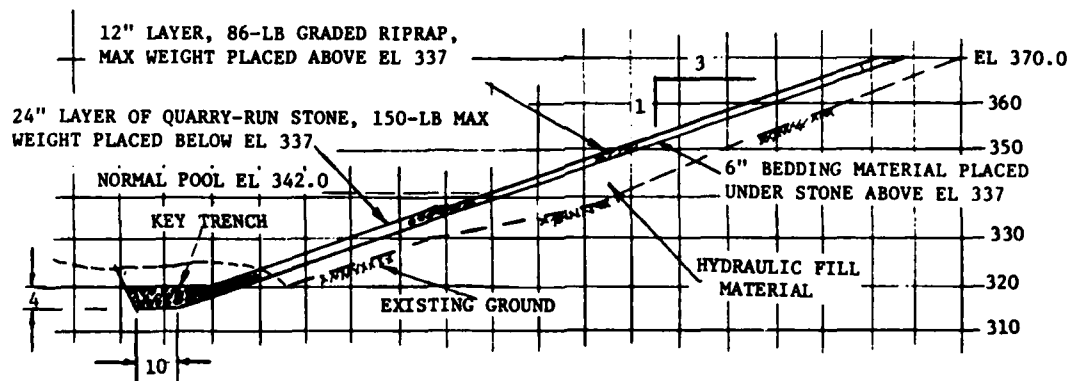
Ohio River (right bank) at Newburgh, Ind. Views of bank
caving prior to placement of riprap in 1957

ITEM 38-2

H-38-4



Ohio River (right bank) at Newburgh, Ind.
Downstream view showing emergency work in
1975. Water Street is to the right



Ohio River (right bank) at Newburgh, Ind.
Cross section of revetment (adapted from
Ohio River, Newburgh, Ind., Bank Protection
Sections, Drawing OC 4606/2, April 1973)

ITEM 38-3

H-38-5



a. View from top of revetment.



b. View from toe of revetment.

Ohio River (right bank) at Newburgh, Inc. Two downstream views of the riprap revetment (19 Jun 1978). The pool elevation is 343 ft.

ITEM 38-4

H-38-6

**WHITE RIVER LEVEE UNIT 8
EDWARDSPORT, INDIANA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream White River River Mile 89 Side Right
Local Vicinity Levee Unit 8 Lat N38°30' Long W87°30'
At/Nr City Plainville County Daviess State IN Cong Dist 8
CE Office Symbol ORLED-H Responsible Agency Levee Unit 8, Levee District
Site Map Sources Louisville District, Corps of Engineers
Land Use Information Sources Levee Unit 8, Levee District

(2) Hydrology at or Near Site

Stage Range 432 to 461 ft; Period of Record 1939 to 1978
Discharge Range 5000 to 140,000 cfs; Velocity Range 2 to 7 fps
Sediment Range - to - tpd; Period of Record 19 to 19
Bank-full Stage 458 ft; Flow 12000 cfs; Average Recurrence Interval 6 yr
Bank-full Flow Velocity: Average 6 fps; Near Bank 6 fps
Comments Pilot channel cutoff has enlarged to full channel

(3) Geology and Soil Properties

Bank (USCS) Silty clay Bed (USCS) Silty sand
Data Sources Visual inspection
Groundwater Bank Seepage None
Overbank Drainage None
Comments Extremely erodible material

(4) Construction of Protection

Need for Protection To divert flows from eroding area near toe of agricultural (Corps built) levee
Erosion Causative Agents Relatively high velocities attacking outside of bend in alluvial channel with silty-sand banks.
Protection Techniques Pilot channel across throat of oxbow.
General Design Pilot channel had average depth of 17 ft. and 10 ft bottom width 2H:1V side slopes.
Project Length 1350 ft; Construction Cost \$ 80,000 Mo/Yr Completed 1975

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Elev. 457, 35,000 cfs March 1978

Repairs and Costs (Item, Cost, Date) None

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Topographic survey Apr 79; aerial photos Apr 80; Insp Jun 78

Documentation Sources As-built plans, aerial photos '59, '63, '73, '76

Project Effect on Stream Regime Pilot channel widened to become main channel in 2 years. Active downstream erosion continued.

Project Effect on Environment None apparent. Rate of bank erosion downstream did not increase after 4 years.

Successful Aspects Erosion at toe of levee reduced to negligible amount

Unsuccessful Aspects Overland access to wooded oxbow area eliminated

General Evaluation Project considered successful. However, final determination should be used on longer period.

Recommendations Costs of this method should be compared with recent types of bank protection at future sites.

(7) Additional Information, Comments, and Summary

Map No. 39. See Attached Item 39-1*

Attached Items:

39-1 Project summary

39-2 Project plan

39-3 Pilot channel plan

39-4 Cross section of levee

39-5 Upper end of pilot channel and bank caving

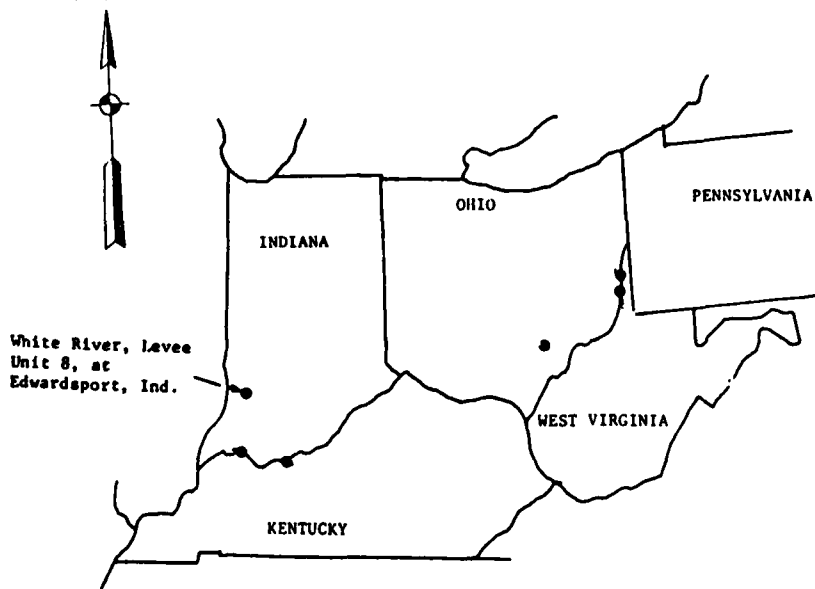
39-6 Project photographs

WHITE RIVER AT LEVEE UNIT 8, EDWARDSPORT, INDIANA

Levee Unit 8 was constructed on the left bank of the White River at Edwadsport, Indiana (mile 67.0-mile 92.0), by the U. S. Army Corps of Engineers (CE) in 1940 to provide 17.6 miles of agricultural levee. The structure is now administered by a local levee board. Four pilot channels for river relocation have been excavated since the initial work to relieve flow against the levee toe. Pilot Channel 1 at mile 87.0 has been selected as a Section 32 Program existing site. Pilot channel placement at mile 87.0 was first suggested by Louisville District, CE (ORL), in 1955 as a means of protecting the levee, which was potentially threatened by stream attack; however, work was not completed on Pilot Channel 1 until December 1975. The completed pilot channel at mile 87.0 is 1,341 ft long with a bed gradient of 3.2 ft/mile and bed width of 10 ft. The contractor placed the excavated materials parallel to the channel and both the excavated materials and channel banks were seeded.

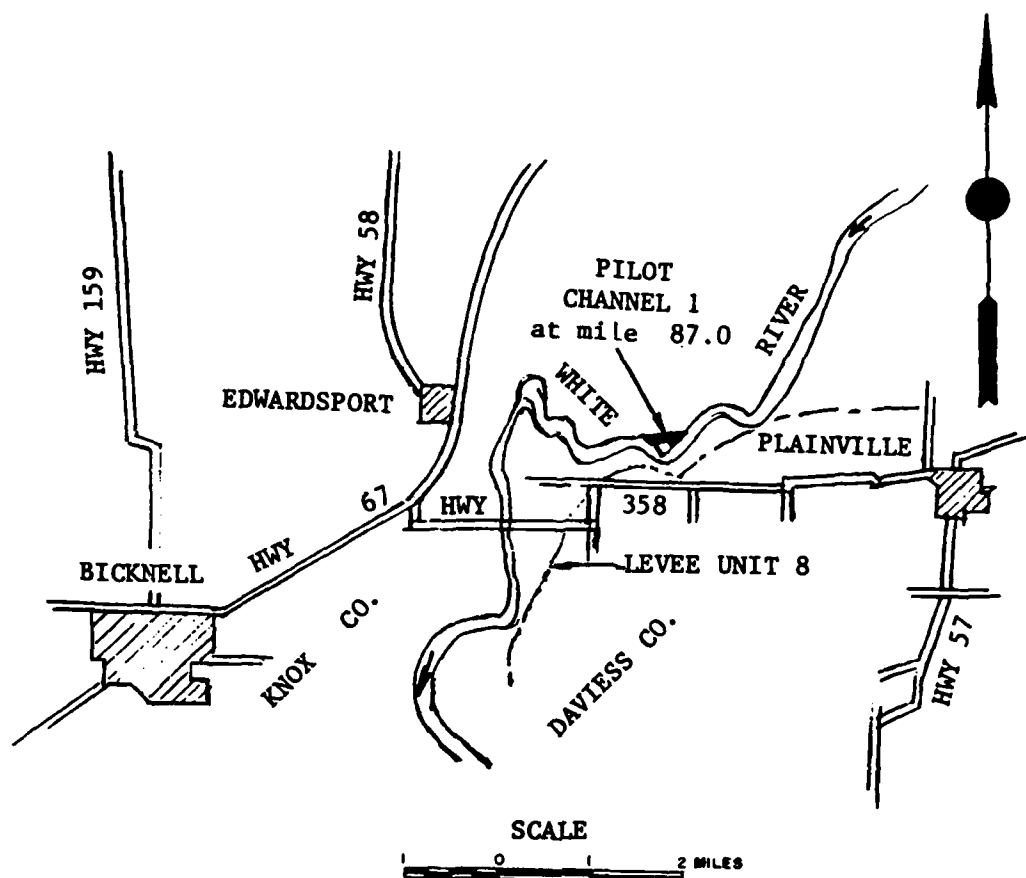
This cutoff reduced the stream length by 0.5 miles and the sinuosity from 1.8 to 1.6. Aerial photographs taken in 1963, 1976, and 1980 indicated that streambank erosion has occurred throughout the study, and that since the pilot channel was completed in 1975, bank erosion has occurred for about 1 mile downstream. However, due to the continuing erosion throughout the study, it is difficult to attribute specific erosion problems to a particular cutoff. The use of cutoffs as a bank protection measure will generally transfer erosion problems to other locations and increase the intensity of the problems. However, if the threatened area is critical (e.g. levee), then this approach could be justified.

*A 1980 memorandum by Ronald Copeland, WESHS, pointed out that in general, cutoffs are not a satisfactory method of erosion control since they increase velocities and shift location of erosion to new areas. This should be considered in the future when meanders threaten levees.



ITEM 39-1

H-39-3



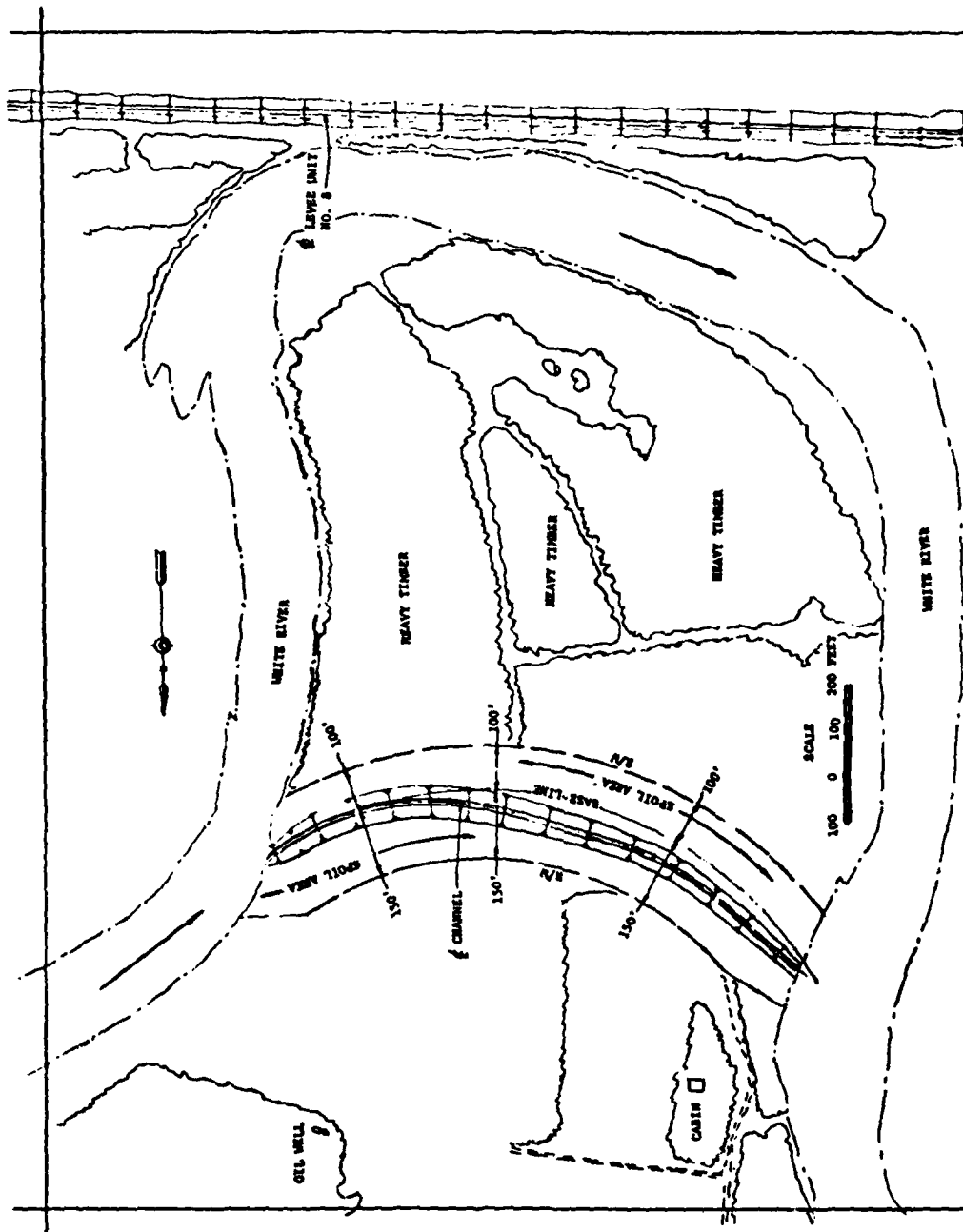
Pilot Channel 1, White River, Levee Unit 8 at Edwardsport, Ind.
(adapted from Levee Unit 8, White River, Levee Repairs, General
Plan, ORL, Sep 1943).



White River, Levee Unit
8 at Edwardsport, Ind.
An inspection conducted
by ORL on 9 May 1973 in-
dicated that the berm
riverward of the left
bank levee had been
eroded to within 2 ft
of the toe.

ITEM 39-2

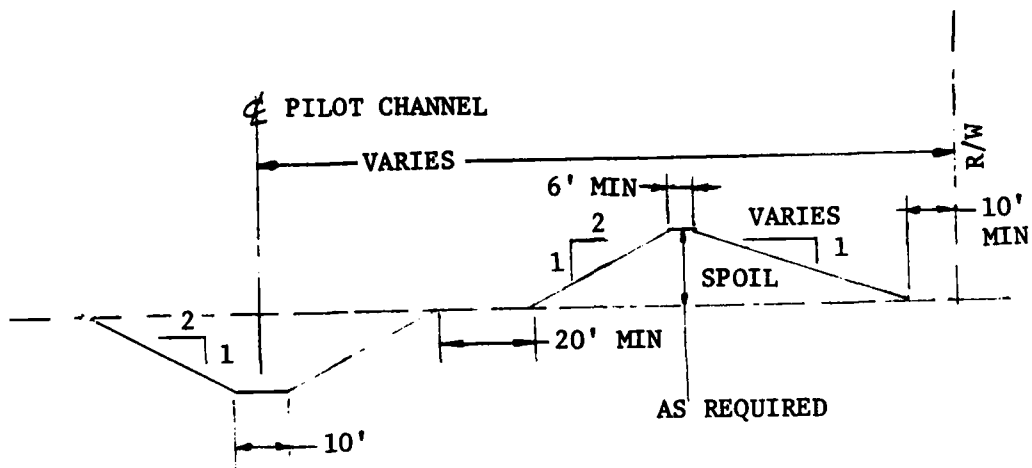
H-39-4



Project plan for Pilot Channel 1, White River, Levee Unit 8 at Edwardsport, Ind.
(adapted from Levee Unit 8, White River Pilot Channel, ORL, Nov 1954)

ITEM 39-3

H-39-5



Pilot Channel 1, White River, Levee Unit 8 at
Edwardsport, Ind. Cross-sectional view
(adapted from Levee Unit 8, White Pilot Channel,
ORL, Nov 1954)

ITEM 39-4

H-39-6



Pilot Channel 1, White River, Levee Unit 8 at Edwardsport, Inc. The upstream end of the pilot channel is to the right; the natural streambed is to the left. Note the levee in the background and deposition in the natural channel. The estimated total discharge is approximately 5,600 cfs, with 95 percent flowing through the pilot channel (20 June 1978).
View toward downstream direction



Pilot Channel 1, White River, Levee Unit 8 at Edwardsport, Ind. Severe bank caving is being experienced along the left bank, being most severe at the upstream end of the project. The view is upstream (20 June 1978)



Pilot Channel 1, White River, Levee Unit 8 at Edwardsport, Ind. Downstream view of lower end of pilot channel at confluence with natural channel of White River (in background) (June 1978)



Pilot Channel 1, White River at Levee Unit 8, Edwardsport, Ind. Planted grass has been well established on the excavated pilot channel materials placed on both banks (20 June 1978)

ITEM 39-6

H-39-8

**MONONGAHELA RIVER
CALIFORNIA, PENNSYLVANIA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Monongahela River Mile 48.2 Side Left
Local Vicinity Roscoe, Pa. Lat N40°04'40" Long W79°51'24"
At/Nr City California County Washington State PA Cong Dist 22
CE Office Symbol ORP Responsible Agency Private owner
Site Map Sources Mon. River L/D 4 Project Topo, May 1962.
Land Use Information Sources U.S.D.A. - SCS Washington City, Soil Survey 1974

(2) Hydrology at or Near Site

Stage Range 743.5 to 756.0 ft; Base Period of Record 1977 to 1978 June
Discharge Range 350 to 120,700 cfs; Jan 78 Velocity Range 0.1 to 5.0 fps
Sediment Range ukn to -- tpd; Period of Record 19-- to 19--
Bank-full Stage 755 ft; Flow 115,000 cfs; Average Recurrence Interval 5 yr
Bank-full Flow Velocity: Average 5.0 fps; Near Bank 3.3 fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) ML Bed (USCS) GM
Data Sources Visual inspection/borings from reconstruction of Dam 4
Groundwater Bank Seepage None observed
Overbank Drainage None - diverted by grading and construction railroad tie
curb
Comments _____

(4) Construction of Protection

Need for Protection Primarily to protect private property & secondly to act
as pleasure boat docking area.
Erosion Causative Agents Water velocities from high water, drawdown effects of
high water.
Protection Techniques Hand-placed, concrete, brick, & stone filled rubber
tires
General Design Two low walls of tires, each consisting of three tiers placed
in a staggered stacking arrangement & filled. The walls are separated by
a 7-ft-wide terrace
Project Length 140 ft; Construction Cost \$ 60.00 Mo/Yr Completed 5/77

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Elev. 751.5, Q = 90,400 cfs V = 4.5 fps on
5 March 1979 - Peak value

Repairs and Costs (Item, Cost, Date) To date minor loss by vandalism of rubble in
tires, no cost.

Comments: Each spring, debris and sediment is cleaned off terrace.

(6) Performance Observations and Summary

Monitoring Program Visual, twice annually

Documentation Sources Owner

Project Effect on Stream Regime Insignificant

Project Effect on Environment Lost rubber tires beaching in downstream areas.

Successful Aspects The walls have halted erosion at the toe of slope thus
prevented slumping of bank as experienced U/S and D/S of site.

Unsuccessful Aspects Tire walls do not protect upper bank from erosion during
highwater as evidenced by loss of soil above upper wall. However, the
river stage exceeds the elev. of the upper wall only about once a yr.
General Evaluation Project has performed well since construction.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 40. On 19 Aug 80 this reach of river bank experienced a major
flood that peaked at approx. elev. 756, 1.5 ft below top of bank.

Although bank scour occurred and trees were damaged within the wall
itself, bulkhead remained intact and functioned as intended.

Attached Items:

40-1 Project summary	40-4 Channel cross section
40-2 Project location	40-5 Photos after major flood
40-3 Rubber tire protection plan	

Monongahela River near California, Pennsylvania

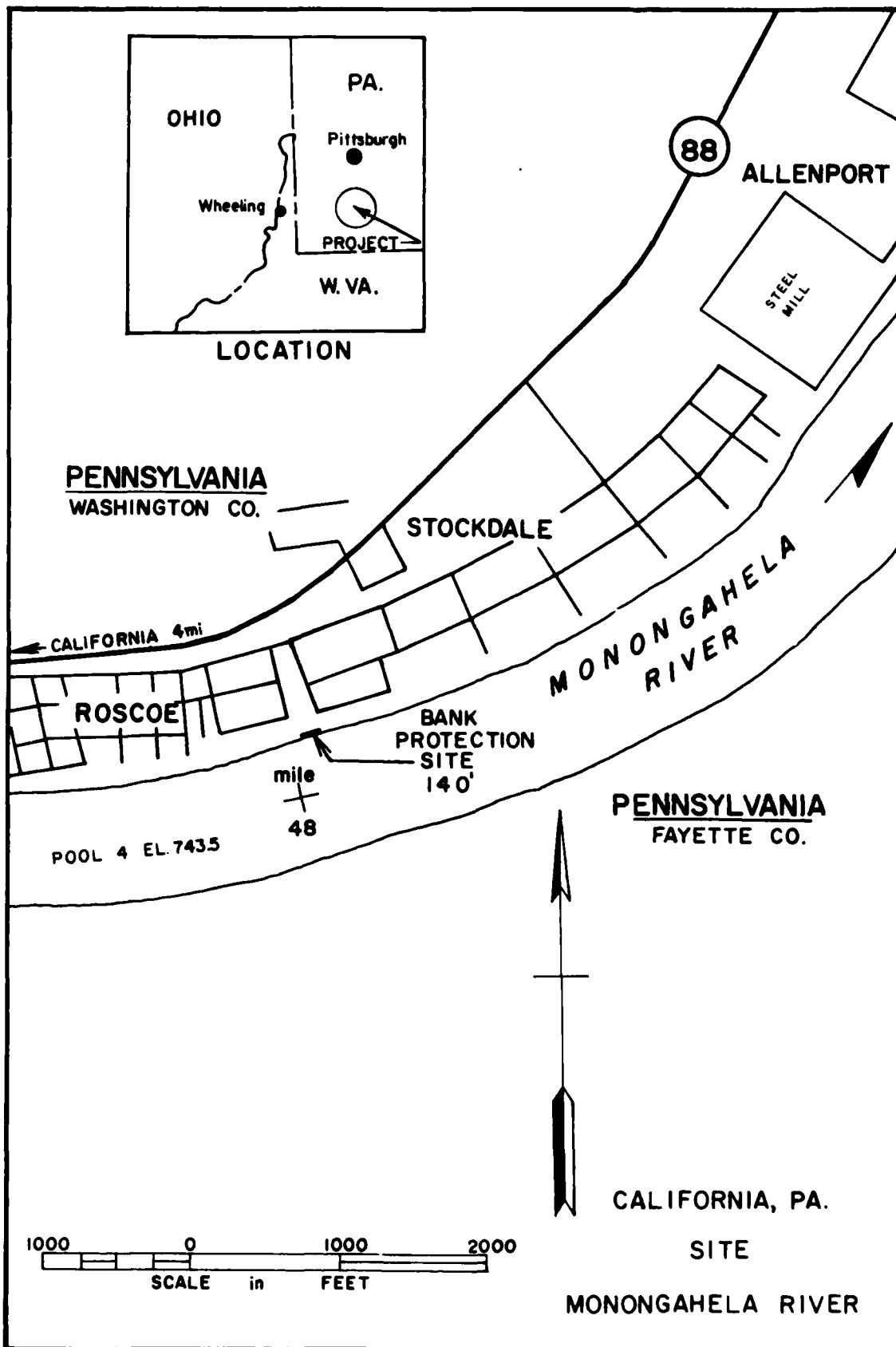
Project Summary

The used-tire revetment in the pool of Lock and Dam 4 on the left bank of the Monongahela River was constructed by the property owner in early summer of 1977. There are two walls consisting of three tiers of tires, each separated by a terrace. These tires were staggered and placed along the 140 ft of his property fronting the river. Each tire was filled with concrete, stone, and brick rubble, and no anchors were used to attach the tires to the streambank. The owner planted ivy in the top tier with the intent of eventually camouflaging the tires; he also made provisions to handle the storm drainage from a 24-in. culvert by incorporating an indentation in the design of the revetment at the point where the culvert discharges into the river. The structural concepts used in the design were developed by the owner, based on his experiences in protecting the riparian properties of his parents and friends. The design included a protrusion in the middle portion of the lower wall that was constructed to cover the tree stumps left a decade earlier by clearing performed in conjunction with the raising of the Lock and Dam 4 navigation pool. In addition to providing the obvious benefit of bank protection, the tire revetment with its lower wall protrusion serves as a pleasure boat dock.

In 1978 the site was inspected and found that in some places vegetation had begun to take root in the sediment that had been deposited in and behind the tires and that the tires had intercepted debris. The bank appeared to be stable, and the revetment was undamaged. There was no growth of ivy detected above the top tier of tires. Pittsburgh District, CE (ORP), personnel made a follow-up inspection (March 1979) as part of their monitoring program for this site. They found that although the annual vegetation had died during the winter, the revetment had remained stable, had induced sediment deposition, and had intercepted more debris. Both the land owner and ORP are satisfied with the performance of the used-tire revetment thus far. Erosion seems to have been retarded, and vegetation has become established in the sediment that has been deposited. ORP does note, however, that there are lost rubber tires beaching in the downstream area and that the elevation of the revetment is not sufficient to withstand erosion during the high water that occurs about once a year. There has also been minor loss due to vandalism of the rubble in the tires. In general, the revetment has performed well, and this site is an excellent example of how a landowner can use discarded materials to protect his riparian property from current and wave action for very low cost.

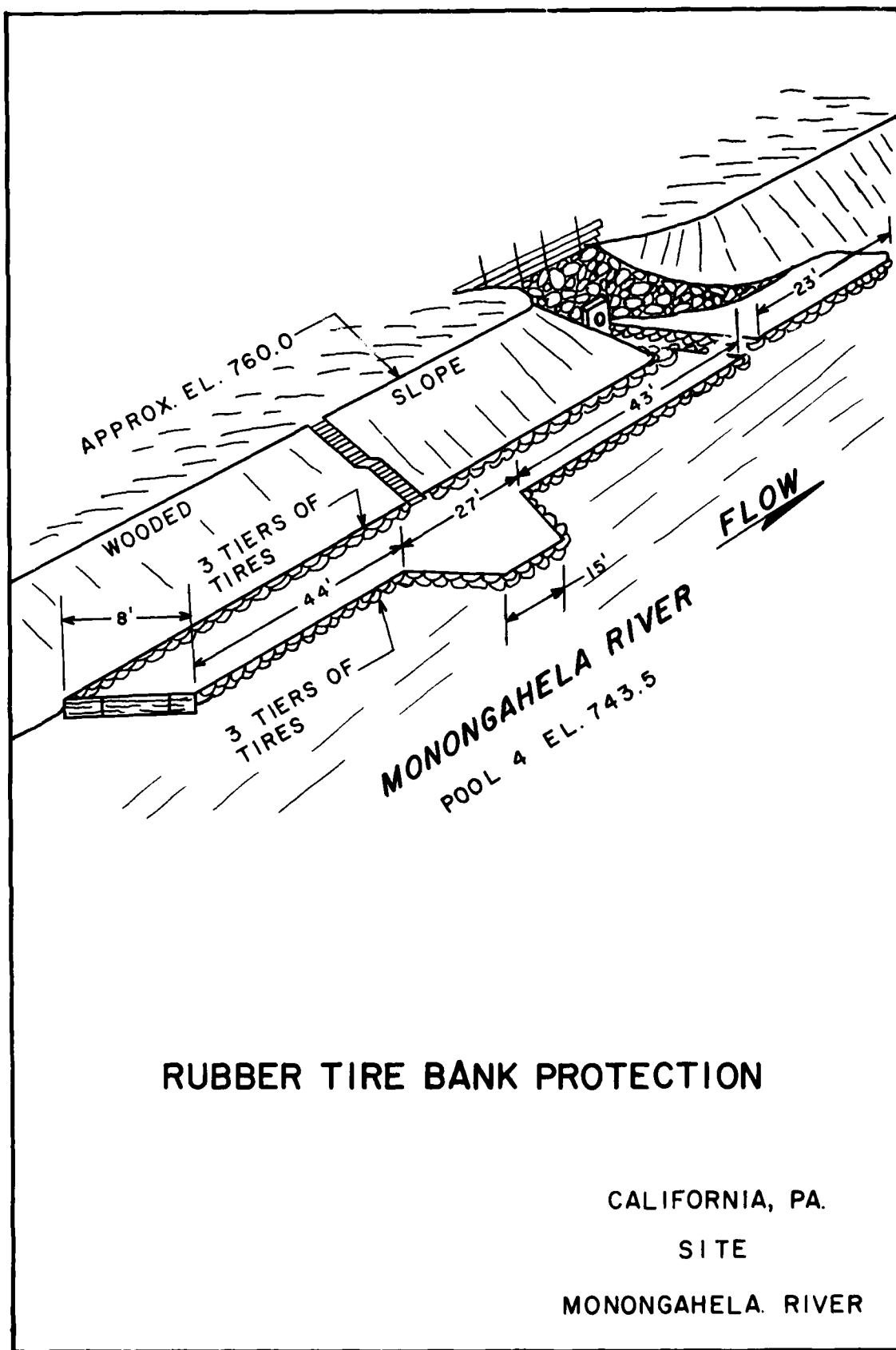
ITEM 40-1

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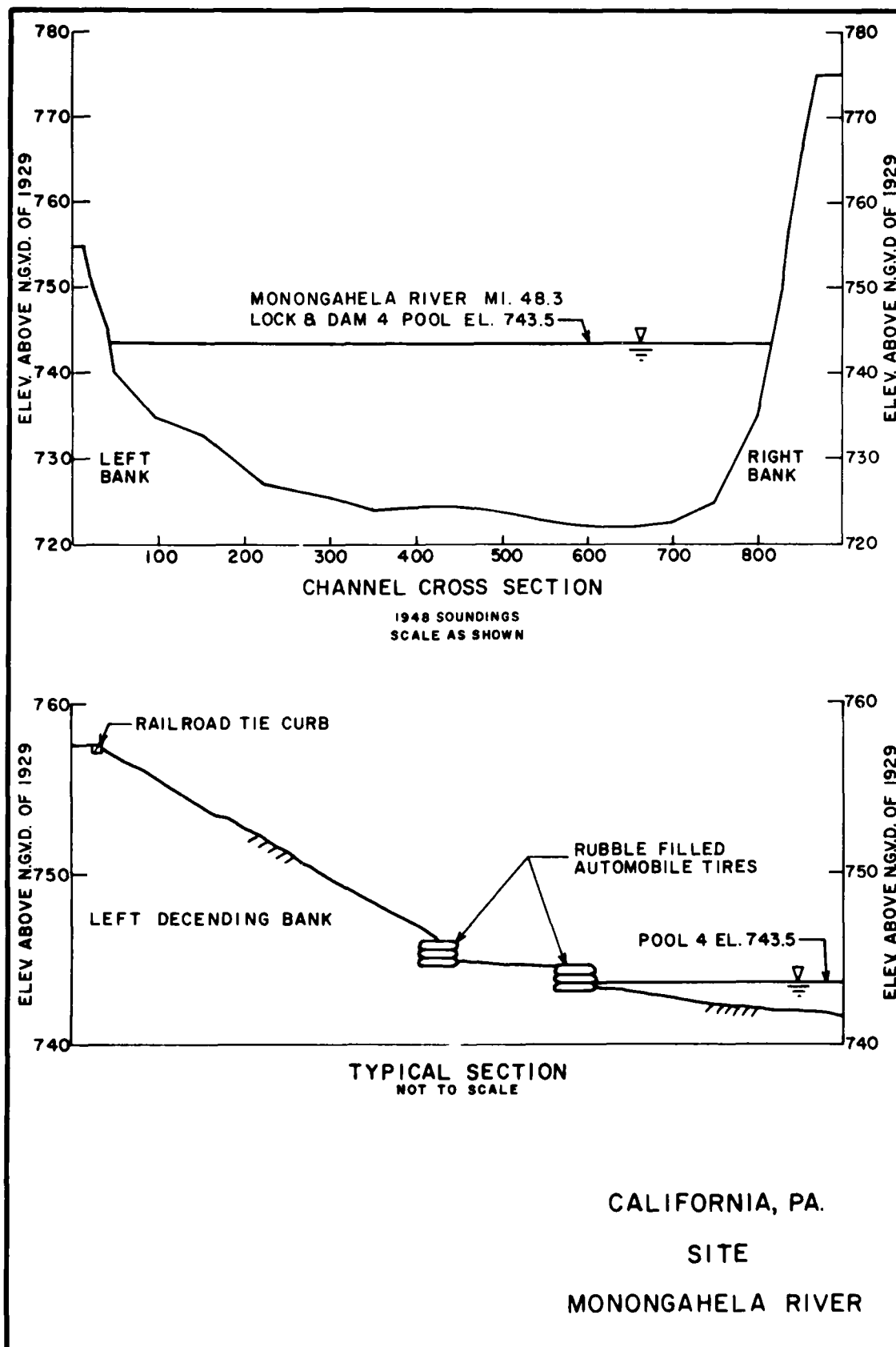
ITEM 40-2

H-40-4



ITEM 40-3

H-40-5



ITEM 40-4

H-40-6



Downstream view
8 October 1980



View looking downstream along the riverbank
8 October 1980

PROJECT PHOTOGRAPHS
AFTER MAJOR FLOOD
CALIFORNIA, PA.
SITE
MONONGAHELA RIVER

**OHIO RIVER
WHEELING, WEST VIRGINIA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Ohio River Mile 90.6 Side Left
Local Vicinity Wheeling Parking Wharf Lat N40°04'00" Long W80°43'24"
At/Nr City Wheeling County Ohio State WV Cong Dist 1
CE Office Symbol ORP Responsible Agency ORP-City of Wheeling
Site Map Sources Hannibal L/D Project Topo & Soundings, June 1967
Land Use Information Sources Hannibal L/D Project Topo & Soundings June 1967

(2) Hydrology at or Near Site

Stage Range 623.0 to 654.8 ft; Period of Record 19 71 to 19 78 .
Discharge Range 6300 to 348,000 cfs; Velocity Range 0.1 to 5.9 fps
Sediment Range ukn to - tpd; Period of Record 19 -- to 19 -- .
Bank-full Stage 645 ft; Flow 235,000 cfs; Average Recurrence Interval 6 yr
Bank-full Flow Velocity: Average 5.5 fps; Near Bank 3.7 fps
Comments Note that bank-full stage occurred 2 Feb. 79

(3) Geology and Soil Properties

Bank (USCS) Unknown Bed (USCS) GM-GP
Data Sources _____
Groundwater Bank Seepage None observed.

Overbank Drainage Top of bank is paved parking lot sloped to the river & all overland drainage cascades over stone protection.
Comments The fill, used to raise the lower level & on which the extended stone protection rests, is a granular material all of which is less than 8" with only 35% allowed to pass the No. 200 sieve.

(4) Construction of Protection

Need for Protection Original slurry protection to protect parking wharf structure; additional stone protection needed when lower level of wharf was raised Oct. 71.
Erosion Causative Agents Water velocities & turbulence from high water, overbank drainage, drawdown effects.

Protection Techniques Slurry concrete & stone protection over filter fabric
General Design One foot thick stone protection over filter fabric slushed with concrete at very top and toed in at top of old slurry concrete protection.

Project Length 624 ft; Construction Cost \$ 16,000 Mo/Yr Completed 10/71
Repair 34,000 Repair 3/72

(5) Maintenance

Experienced Flows (Stage, cfs, Date) El. 647.0, Q=256,000 cfs, V= 5.5 fps
on Feb. 26, 1979; El. 644.0, Q=225,000 cfs, V=5.5 fps on 6 Mar 79

Repairs and Costs (Item, Cost, Date) Repair in March 72 for \$34,000; parking authority
has added concrete rubble since then - on what occasions & at what cost is
unknown.

Comments: Experienced flows are peak values, attached stage hydrographs are
averaging values.

(6) Performance Observations and Summary

Monitoring Program Visual, twice annually.

Documentation Sources District files.

Project Effect on Stream Regime None

Project Effect on Environment None

Successful Aspects Has maintained the integrity of the parking structure.

Unsuccessful Aspects In five small areas within the project reach, the filter
fabric is exposed (area 5' x 10'); in other areas the parking authority
has had to replace lost stone with rubble concrete to maintain the integrity
of the protection.

General Evaluation Apparently the repair work done in 1973 has not been
100% effective since the parking authority still has to replace lost stone
protection. There appears to have been no loss of stone since last inspection.

Recommendations Continue to monitor the loss and replacement of stone.

(7) Additional Information, Comments, and Summary

Map No. 41.

Attached Items.

41 - 1 - Pertinent Project Details

41 - 2 - Location Map

41 - 3 - Typical Cross section

41 - 4 - Photos: before construction; after construction; after
substantial flow

Wheeling, W. Va. Site, Ohio River

1. Location.

In 1974 the Pittsburgh District completed construction of Hannibal Locks and Dam on the Ohio River at mile 126.4. As a result of this action, the pool which includes the Wheeling, W. Va., reach, was raised from an elevation of 617.8 ft (elevation of the old Dam 13 pool) to its present normal pool elevation of 623 ft which extends upstream to Pike Island Locks and Dam (mile 84.2). The 623-ft elevation of the new Hannibal pool required raising the elevation of several riverfront structures and their associated protection works to a level that would accommodate anticipated flows. Among the structures affected was the Wharf Parking Garage (mile 90.6), which is operated by the City of Wheeling and located on the left bank of the main channel of the Ohio River directly across from the center of Wheeling Island (Item 41-2), and approximately 0.2 mile downstream from the historic Wheeling Suspension Bridge.

2. Hydrologic and Geologic Description.

Unpublished discharge data are available for the National Weather Service gaging station at the Wharf Parking Garage. Maximum, mean, and minimum daily discharges for the life of the project (1971 to present) are 348,000 cfs, 44,800 cfs, and 6,300 cfs, respectively. Discharges of 235,000 cfs or more are considered flood flows. The maximum flow measured at this gage occurred on 22 June 1972. Since October 1971 other flood flows have been measured on 18 and 25 February 1975 and 2 February 1979. Throughout the period of record there have been many other days on which the flows have approached flood level. Flow velocities range from 0.1 to 5.9 fps. No suspended-sediment load or bed-material gradation samples have been taken in this reach. The bed gradient is 0.80 ft/mile with the bed material consisting of sand and gravel. The bank slopes are 1V on 2H in the vicinity of the parking facility, with the soils being clay, sandy clay, and silt. The depth to bedrock ranges from 15 to 25 ft.

3. Design and Construction.

The original bank protection at the Wheeling site consisted of 624 lin ft of a concrete slurry blanket, approximately 12 in. thick, constructed to an elevation varying between 625.5 and 626.5 ft. The date of placement of this slurry is not known. With the anticipated rise of the Hannibal Pool, the Pittsburgh District decided to raise the elevation of the lower level of the parking garage to 631 ft and to extend the slope protection to the 613-ft elevation.

In mid-1971, ORP advertised for bids to raise and resurface the lower parking level with bituminous pavement, to remove and replace the existing steel guardrail, and to place 624 lin ft of riprap bank protection in a 12-in. layer over filter fabric and granular fill, extending from the elevation of the existing concrete slurry revetment (625.5-626.5) to an

ITEM 41-1
(Sheet 1 of 2)

elevation of 631 ft. Item 41-3 shows a typical section of this revetment. On 27 July 1971 the contract for the work was awarded to the James White Construction Company of Weirton, W. Va. Work on the project began on 9 August 1971 and was completed by 20 October 1971. The total cost was \$107,348.51 which included \$12,342.00 for 374 cu yd of riprap and \$3,910.00 for 1,150 sq yd of Filter-X fabric, manufactured by Carthage Mills, of Cincinnati, Ohio.

Partial failure of the revetment occurred in March 1972 (see Item 41-4). As a result of this failure, an inspection was conducted by ORP personnel on 29 March 1972 after the water had receded to 4 ft below the top of the original concrete slurry revetment. The investigation revealed that in many locations the 1971 protection had been placed on top of the original concrete slurry revetment with no toe support to prevent the stone from sliding downslope. The original contract drawings clearly indicate that the new riprap should have been keyed through the surface of the existing protection as toe support. In a number of sections the riprap had slid into the river thus exposing and often tearing the filter fabric.

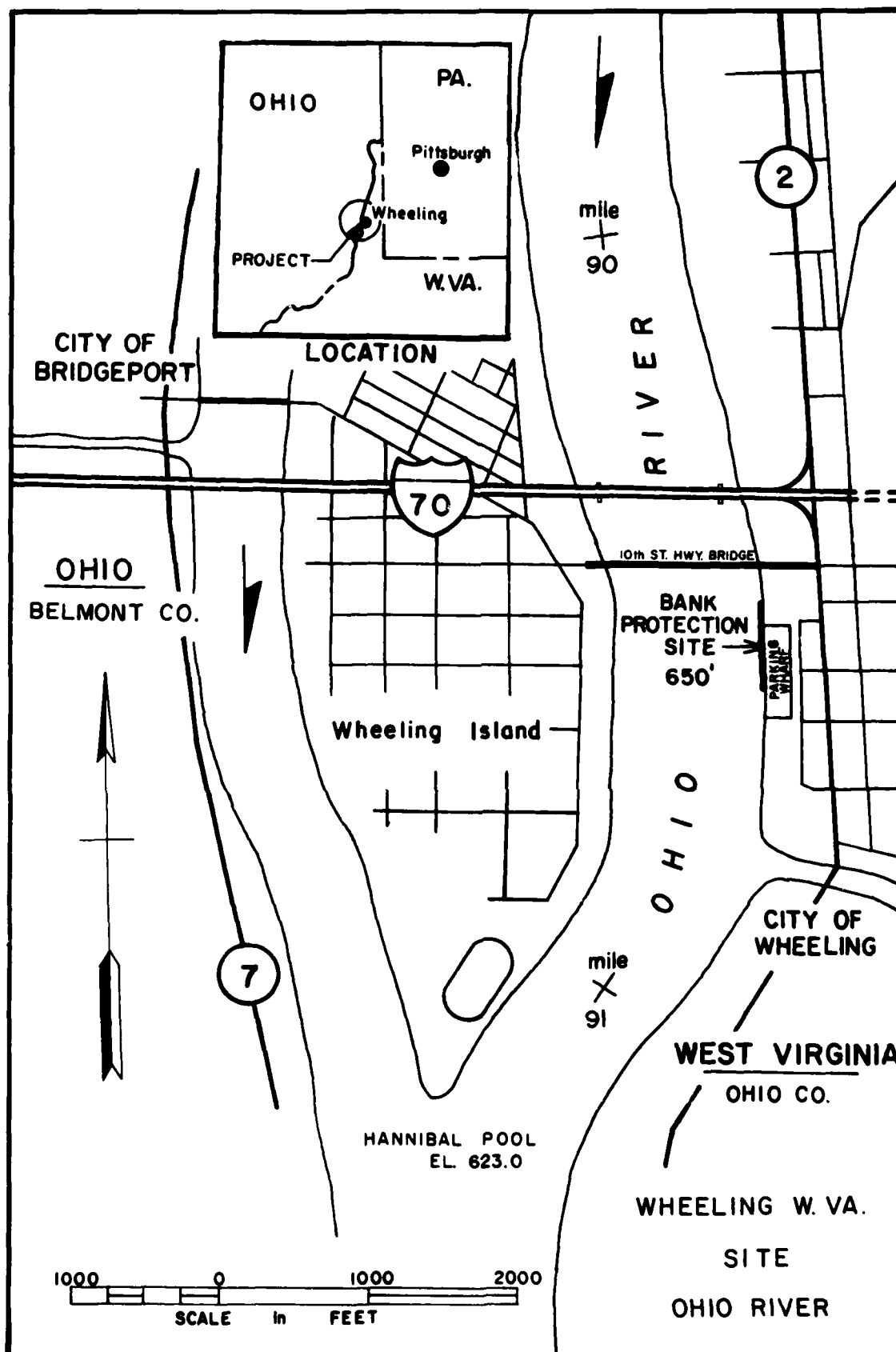
Repair of the damaged revetment was requested on 19 June 1973 by the Engineering Division of the Pittsburgh District and was completed by personnel of the Operations and Maintenance Branch on 13 September 1973. The total cost of the 1973 repairs was \$33,700 which included removal of the 1971 revetment and part of the lower bituminous pavement and the purchase of new filter fabric (Poly-Filter X).

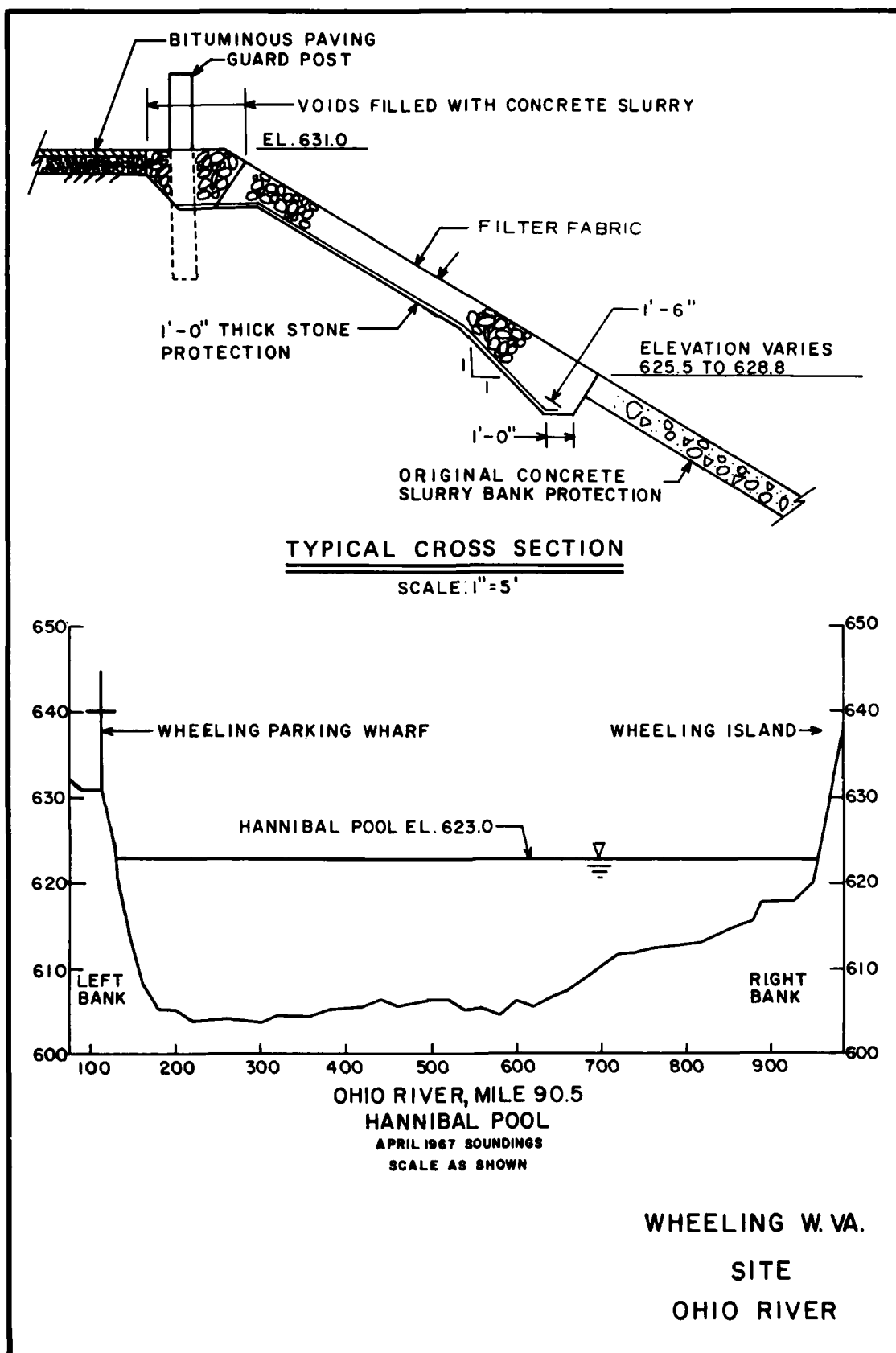
4. Conclusions and Recommendations.

Although generally effective, the stone and filter fabric revetment has a history of chronic localized failures related to sliding stability of the stone on the filter fabric. Sliding stability is reduced through external hydraulic forces experienced during high-water events as well as oversteepening of the slope due to shifting of the bank soil. Such soil displacements may result from settlement or internal seepage. The performance of the protection at the Wharf Parking Garage shows that plastic filter fabric may be inappropriate for use where there is marginal sliding stability. At this site a graded sand and gravel filter might have provided sufficient additional sliding resistance to prevent the localized failures.

ITEM 41-1
(Sheet 2 of 2)

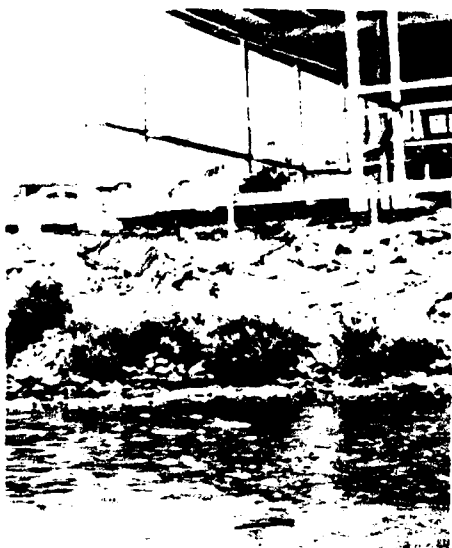
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ITEM 41-3

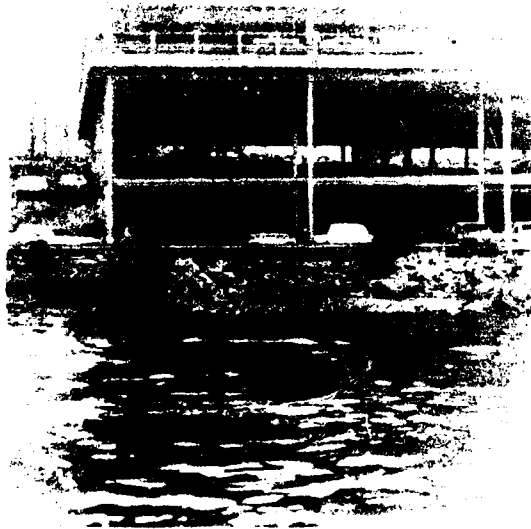
H-41-6



Failure after construction
August 1972



Restored bank
10 May 1977



Partial failure of restored
bank visible, 1 August 1978



View looking downstream
29 March 1979

WHEELING, W. VA.
SITE
OHIO RIVER

ITEM 41-4

H-41-7

**OHIO RIVER
TILTONSVILLE, OHIO**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Ohio River Mile 83 Side Right
Local Vicinity Tiltonsville, Ohio Lat N40°10'00" Long W80°41'48"
At/Nr City Tiltonsville County Jefferson State OH Cong Dist 18
CE Office Symbol ORP Responsible Agency ORPED
Site Map Sources Pike Island L/D Pool Topo & Soundings, March 1959
Land Use Information Sources Same as above

(2) Hydrology at or Near Site

Stage Range 644.5 to 659.3 ft; Period of Record June 68 to June 72
Discharge Range 6,300 to 348,000 cfs; Velocity Range 0.1 to 6.2 fps
Sediment Range ukn to --- tpd; Period of Record 1972 to 1980
Bank-full Stage 660 ft; Flow --- cfs; Average Recurrence Interval 50 yr
Bank-full Flow Velocity: Average 6.0 fps; Near Bank 4.0 fps
Comments Pike Island L/D is just downstream at R.M. 84.2

(3) Geology and Soil Properties

Bank (USCS) SM-SP Bed (USCS) GM-GP
Data Sources ---
Groundwater Bank Seepage Minor
Overbank Drainage From residences back yards and abandoned cisterns
Comments ---

(4) Construction of Protection

Need for Protection Bank erosion along Main Street riverfront
Erosion Causative Agents Water velocities from high water, overbank drainage, and possibly seepage.
Protection Techniques Graded gravel to act as riprap protection
General Design 3800 tons of graded gravel between 3/8" and 4-1/2" placed in the eroded pool level notch on a 1-vertical-on-2-horizontal slope.
Project Length 2500 ft. Construction Cost \$ 27,000 Mo/Yr Completed 7/68

(5) Maintenance

Experienced Flows (Stage, cfs, Date) E1. 650, Q=256,000 cfs, V=5.2 fps, on 26 Feb. 79; E1. 647.4, Q=225,000 cfs, V=4.9 fps on 6 March 79

Repairs and Costs (Item, Cost, Data) None to date.

Comments: Experienced flows are peak values

(6) Performance Observations and Summary

Monitoring Program Visual, twice annually

Documentation Sources District files

Project Effect on Stream Regime Insignificant

Project Effect on Environment Stabilized environment within project reach.

Successful Aspects Halted erosion within the zone it was placed, elev. 642.0 to approximate elev. 647.0.

Unsuccessful Aspects Minor erosion still takes place above the protected zone.

General Evaluation Although some shifting of the gravel has occurred in the past 10 yrs., the protection appears to be functioning satisfactorily.

Recommendations Future designs should be placed on a flatter slope than 1V on 2H initially or increase volume of gravel to prevent decreasing effectiveness as the slope flattens.

(7) Additional Information, Comments, and Summary

Map No. 42.

Attached Items.

42 - 1 - Additional project information.

42 - 2 - Project Location

42 - 3 - Typical Cross Section

42 - 4 - After substantial flow

42 - 5 - After substantial flow

Tiltonsville, Ohio Site, Ohio River

1. General.

In 1968 gravel protection was placed by the Pittsburgh District Operations and Maintenance Branch along approximately 2500 ft of the right bank of the Ohio River at Tiltonsville, Ohio (Item 42-2). Tiltonsville is located upstream of Pike Island Dam (mile 84.2) which maintains a normal pool elevation of 644 ft through the Tiltonsville reach (variation less than 1 ft). The top of the bank at this site is approximately 20 ft above the Pike Island pool. River traffic in the vicinity of this bank is infrequent because the Pike Island Locks are downstream on the left bank of the river.

2. Hydrologic and Soils Description.

No discharge or sediment data are available for the reach that includes this revetment. Unpublished discharge data are available for the CE gaging station located at Pike Island Locks and Dam (mile 84.2). Maximum, mean, and minimum daily discharges for the period of record of this station (1963 to present) are 368,000 cfs, 39,800 cfs, and 2,900 cfs, respectively. Discharges of 250,000 cfs or more are considered flood flows. The maximum flow measured at the Pike Island Dam gage occurred on 11 March 1964. Other flood flows have been measured during March 1963, February 1966, June 1972 and February 1979; throughout the period of record there have been many other days on which the flows have approached flood level. Flow velocities at the Tiltonsville site range from 0.25 fps to 7.0 fps. The bed gradient is 0.8 ft/mile. The bank soils are sandy silt and silty sand.

3. Design and Construction.

The bank protection consists of a wedge of gravel fill placed in the eroded notch to a 1-on-2 outer slope. Work was done from barges on the river. The gravel was graded between $3/8$ and $4\frac{1}{2}$ in., with 50 percent being $1\frac{1}{4}$ in. in diameter. The gravel protection was placed on the bank from 3 ft above normal pool level to 2 ft below pool level (Item 40-3), with the cost of the protection being approximately \$27,000. Although the size of the gravel appears to be small for this bank protection application, the Pittsburgh District considered this material adequate because most of the river traffic is near the opposite bank and because Pike Island Dam maintains normal pool level fluctuations within a relatively narrow range.

4. Conclusions and Recommendations.

Field observations indicate that the gravel protection has assumed a stable beach configuration under the action of river stage fluctuations and waves generated by wind and river traffic. Much of the gravel

ITEM 42-1
(Sheet 1 of 2)

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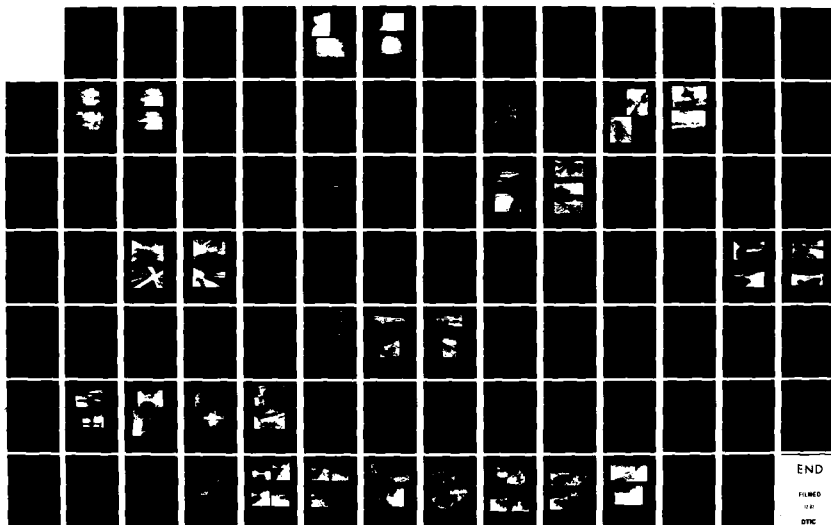
THE STREAMBANK EROSION CONTROL EVALUATION AND
 DEMONSTRATION ACT OF 1974 5. (U) ARMY ENGINEER
 WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.

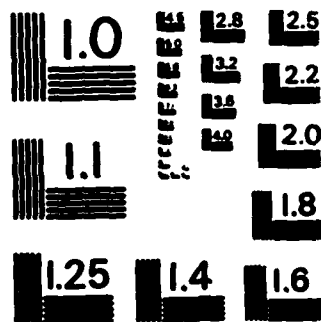
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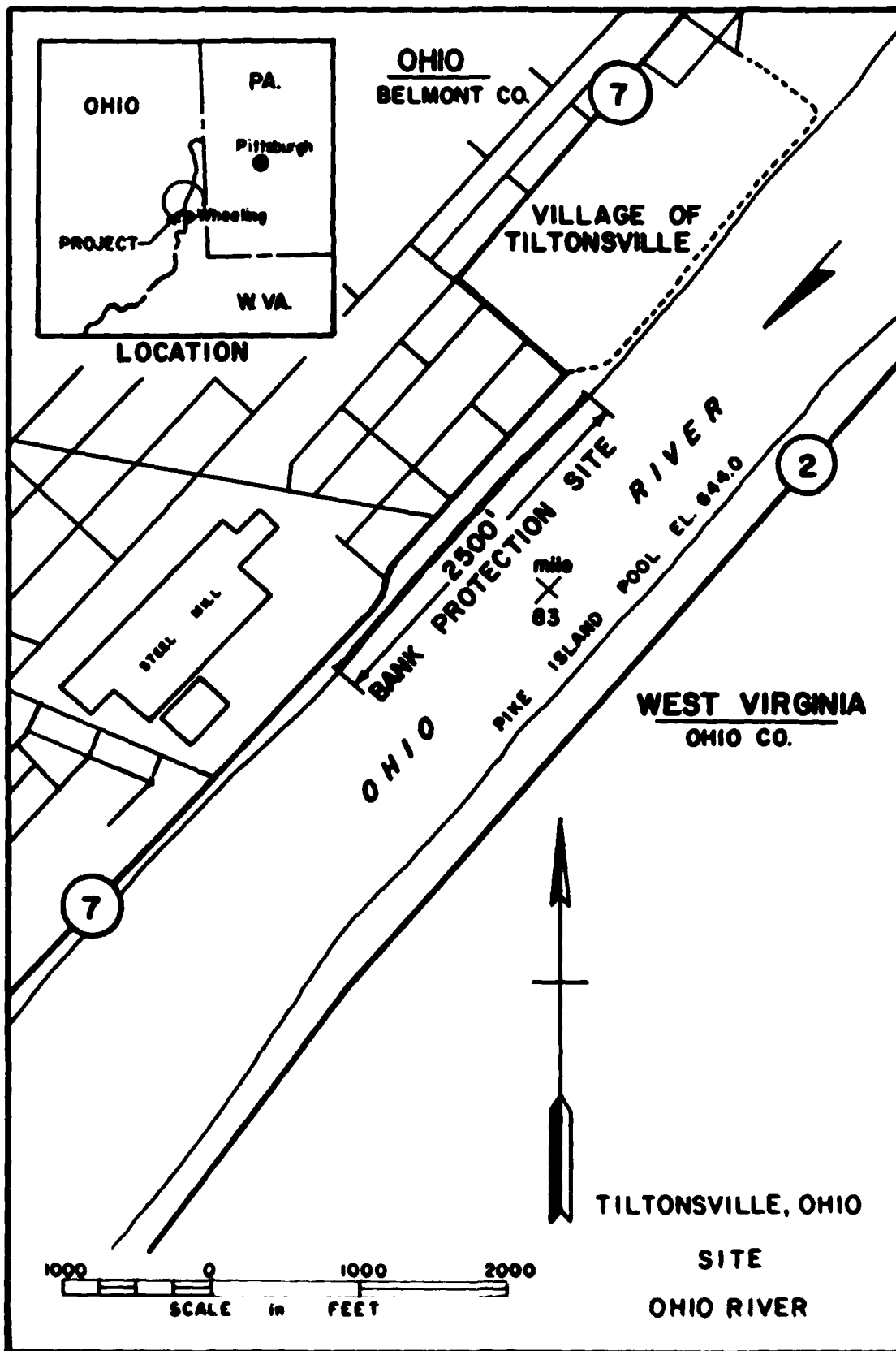


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

protection above approximately elevation 646 has, however, washed down onto the beach or into the river (Items 40-4 and 40-5). The gravel beach now has inclinations ranging from about 1 on 4 to 1 on 10 and averaging about 1 on 5 to 1 on 6; the gravel beach continues to perform its intended function of protecting the toe of bank from wave action at normal pool level. However, the bank continues to fail and erode, primarily during river stages above normal pool, by complex geotechnical processes that include retrogressive sliding induced by flood scouring, drawdown, and related piping. In future designs incorporating gravel toe protection consideration must be given to stabilization and protection of the upper bank.

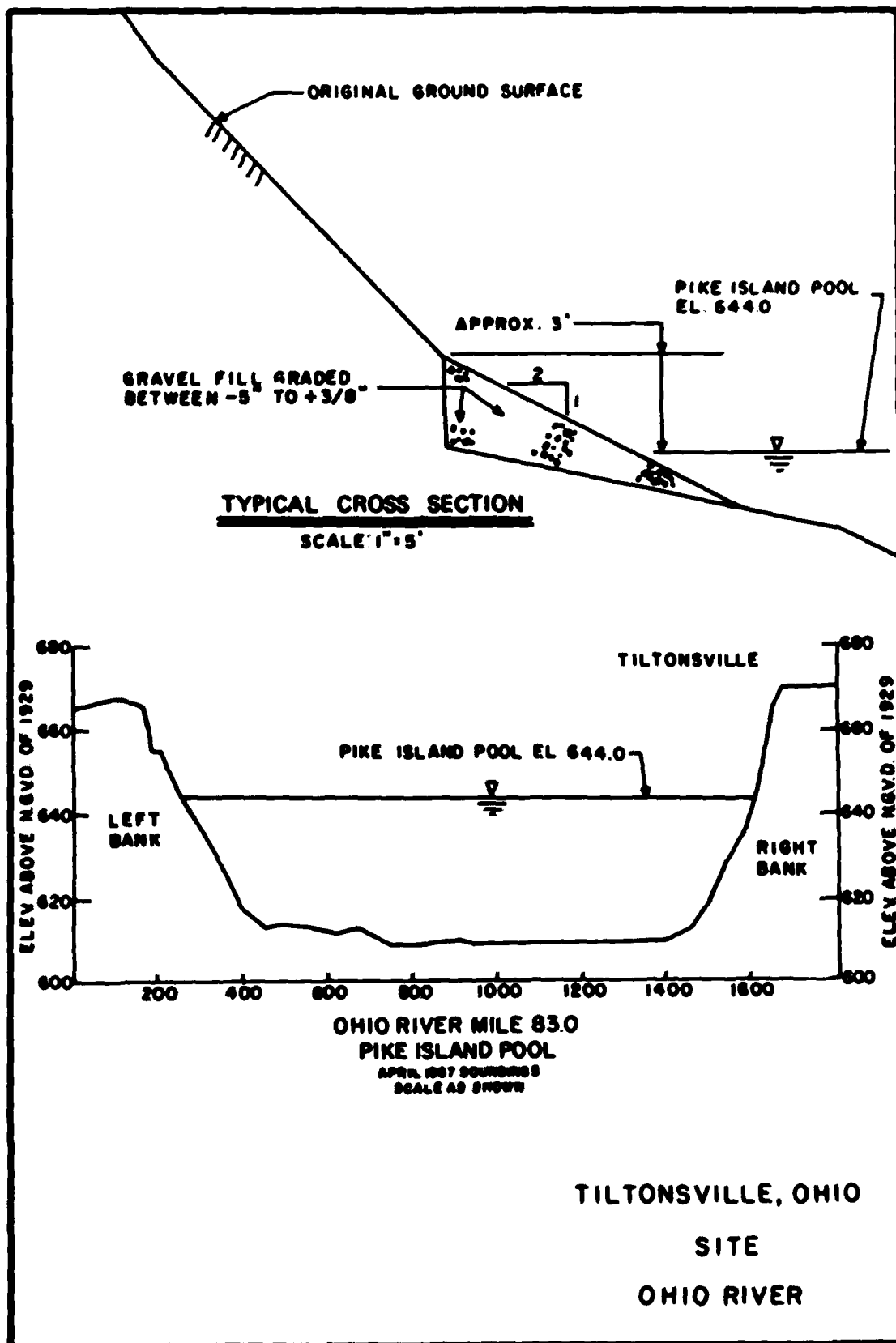
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ITEM 42-2

H-42-5



ITEM 42-3

H-42-6



Downstream view of beach and
upper bank, 10 January 1980



Upstream view of beach and upper bank
10 January 1980

TILTONSVILLE, OHIO
SITE
OHIO RIVER

H-42-7

ITEM 42-4



Downstream view of beach
10 January 1980



View of beach material
10 January 1980

TILTONSVILLE, OHIO
SITE
OHIO RIVER

ITEM 42-5

R-42-8

**WOODCOCK CREEK
SAEGERTOWN, PENNSYLVANIA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Woodcock Creek River Mile 3.9 Side Left and Right
Local Vicinity Saegertown, PA Lat N41°42' Long W80°05'
At/Nr City Headville County Crawford State PA Cong Dist 24
CE Office Symbol ORP Responsible Agency US Army Engineer Dist., Pittsburgh
Site Map Sources US Army Engineer Dist., Pittsburgh
Land Use Information Recreation, Farming

(2) Hydrology at or Near Site

Stage Range 2 to 6.5 ft. Period of Record 1974 to 1980
Discharge Range 7 to 1400 cfs. Velocity Range 0 to 10 (est) fps
Sediment Range low to _____ tpd. Period of Record 1974 to 1980
Bank-full Stage 5.8 ft. Flow 1000 cfs. Average Recurrence Interval 4 yr
Bank-full Flow Velocity Average 3 fps. Near Bank 2-4 fps
Comments Data applicable after construction of dam just above study reach.

(3) Geology and Soil Properties

Bank (USCS) GM Bed (USCS) GM
Data Sources Borings from construction of Woodcock Dam
Groundwater Bank Seepage Minor
Overbank Drainage Minor
Comments _____

(4) Construction of Protection

Need for Protection Arrest bank erosion, protect high-pressure gas line, and maintain natural channel alignment.
Erosion Causative Agents High-velocity attack on outside of bends, eddy action, and surges.
Protection Techniques Gabion deflectors (spurs) and riprap.
General Design Spurs on outside of bends to keep high velocities away from bank; one segment also riprapped.
Project Length 1200 ft. Construction Cost \$ 10,000 Mo/Yr Completed 10/73

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Date) _____

Comments No maintenance to date

(6) Performance Observations and Summary

Monitoring Program Included in periodic inspections of Woodcock Dam

Documentation Sources Periodic Inspection Reports and "Woodcock Creek Gabion Study".

Project Effect on Stream Regime Project has helped maintain existing regime of stream.

Project Effect on Environment Minimal

Successful Aspects Erosion is not evident along protected banks for the most part; channel alignment is not changed.

Unsuccessful Aspects Serious erosion at one location; some riprap washed away; some gabions undercut and/or broken.

General Evaluation Project failed to adequately protect outside of one sharp bend, otherwise it has had the desired effect.

Recommendations Place riprap on bank at bend where erosion is occurring or extend gabion spurs streamward in this area, repair broken gabions.

(7) Additional Information, Comments, and Summary

Map No. 43. See "Woodcock Creek Gabion Study", a report prepared by the Pittsburgh District, Corps of Engineers in August 1980, for additional information.

Attached Items:

43-1 Project summary
43-2 Vicinity map
43-3 Project plan

43-4 Typical sections
43-5 Project photos
43-6 " "

Woodcock Creek Dam Site, Woodcock Creek

1. General.

In July 1973 the Pittsburgh District completed construction of the Woodcock Dam which is located on Woodcock Creek. Gabion spurs were constructed along Woodcock Creek to arrest its natural meandering tendencies, in order to maintain for future recreation use the Federal lands adjacent to the creek downstream of the dam.

2. Hydrology and Soils Description.

Discharge data from the dam are available at the project. Maximum, mean and minimum daily discharges for the life of Woodcock Creek Dam project (1973 to present) are 1400 cfs, 710 cfs, and 15 cfs respectively. Discharges of 1000 cfs are bank-full flows. The maximum flow within the period of record (1400 cfs) occurred in March 1974. With a discharge of 710 cfs, flow velocities range from 0.2 fps near the banks to 12.5 fps along the riprapped reach at the downstream end of the protection. No suspended-sediment load or bed-material gradation samples have been taken in this reach. The bed gradient within the protected reach is 17 feet/mile. Woodcock Creek flows northwest in a northwesterly trending preglacial valley eroded in bedrock which was backfilled with till, lake sediments, and lane deposits. The creek bed and bank soils consist of silty sandy gravel and silty gravelly sand.

3. Design and Construction.

On 20 November 1972 the Foster Grading Company received notice to proceed for the construction of a recreation area below Woodcock Creek Dam. Included in this work was the placement of the gabion spurs and riprap along the creek channel. The cost for the 320 cubic yards of stone-filled wire mesh gabions and 60 cubic yards of riprap was \$6,400 and \$2,000, respectively. The riprap was needed to protect an exposed gas line. A modification to the contract added gabion spurs numbered 6 and 7 and lowered gabion spurs numbered 5, 8 and 9 to one foot below the existing streambed. The quantity of gabions was increased by 55 cubic yards and the total cost of the modification was \$1,635. The contract was completed 16 October 1973.

As a result of the 4th Periodic Inspection of Woodcock Dam on 12 July 1979, the Pittsburgh District undertook a hydraulic study to evaluate the effectiveness of the gabion spurs since some bank erosion was occurring and some gabions had been displaced. The conclusion to this study was that additional riprap will be placed along the bank between gabion spurs numbered 10 and 11.

4. Conclusions and Recommendations.

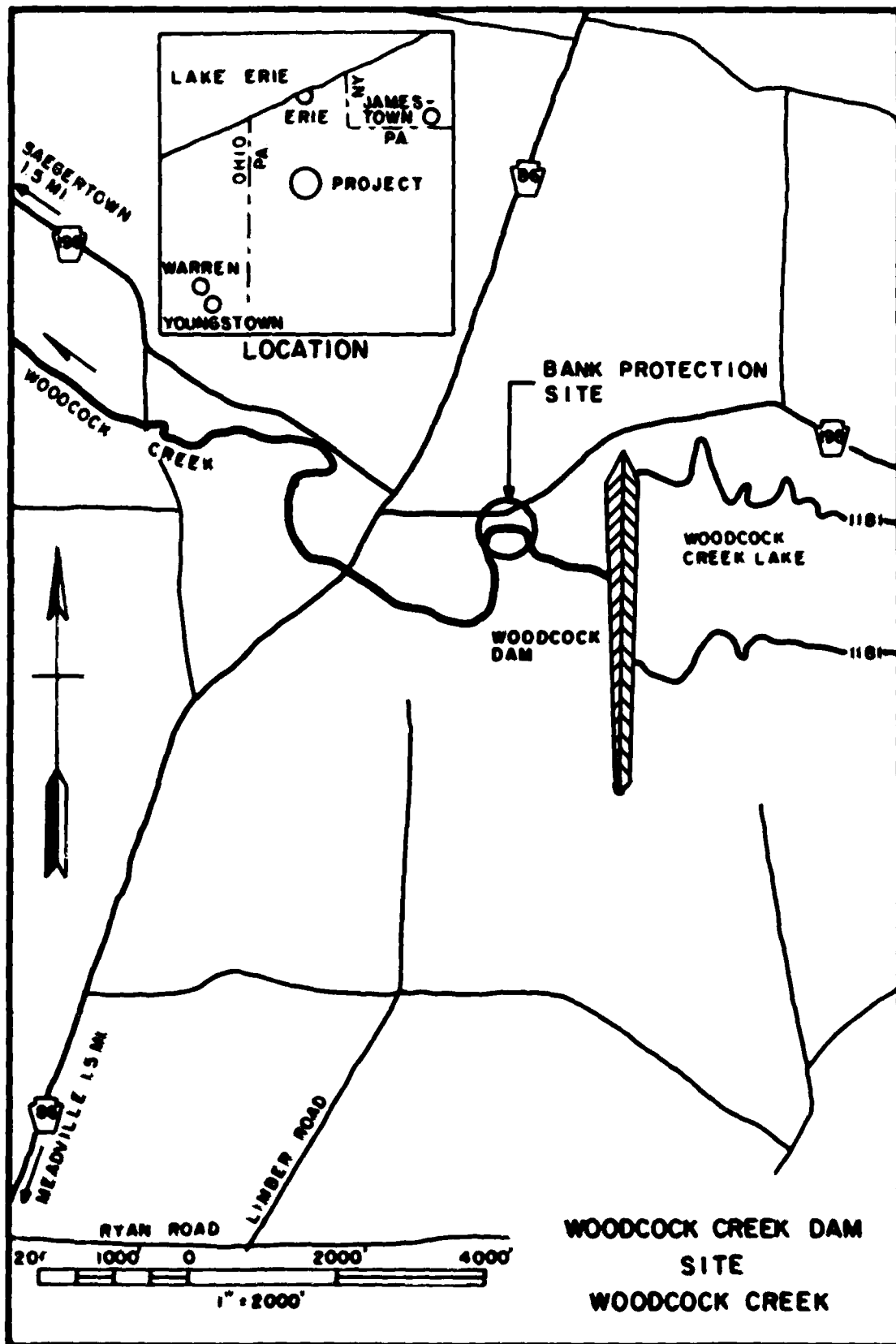
The choice of the spur design over a revetment design was based on

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(Sheet 1 of 2)

economics. The design and layout depended heavily on the practical knowledge and experience of the design engineer. Apparently, the gabion spurs have performed as intended, abating erosion and maintaining channel alignment. Some serious erosion, however, has taken place between spurs numbered 10 and 11 where remedial work is now planned. Erosion occurring between spurs numbered 5 and 6 and spurs numbered 9 and 10 is considered minor. Although spurs numbered 11 and 13 have been slightly displaced, they continue to function.

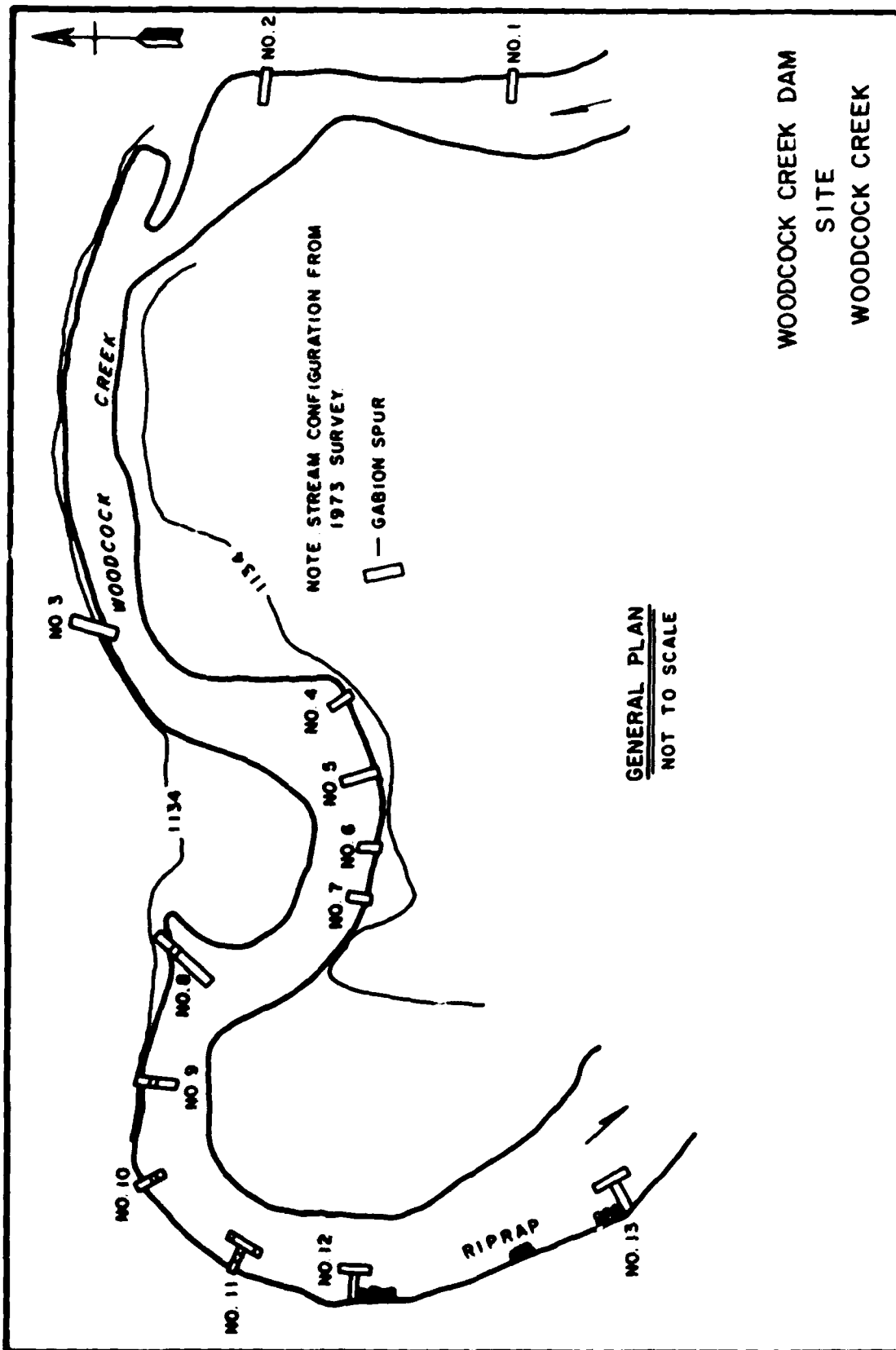
ITEM 43-1
(Sheet 2 of 2)

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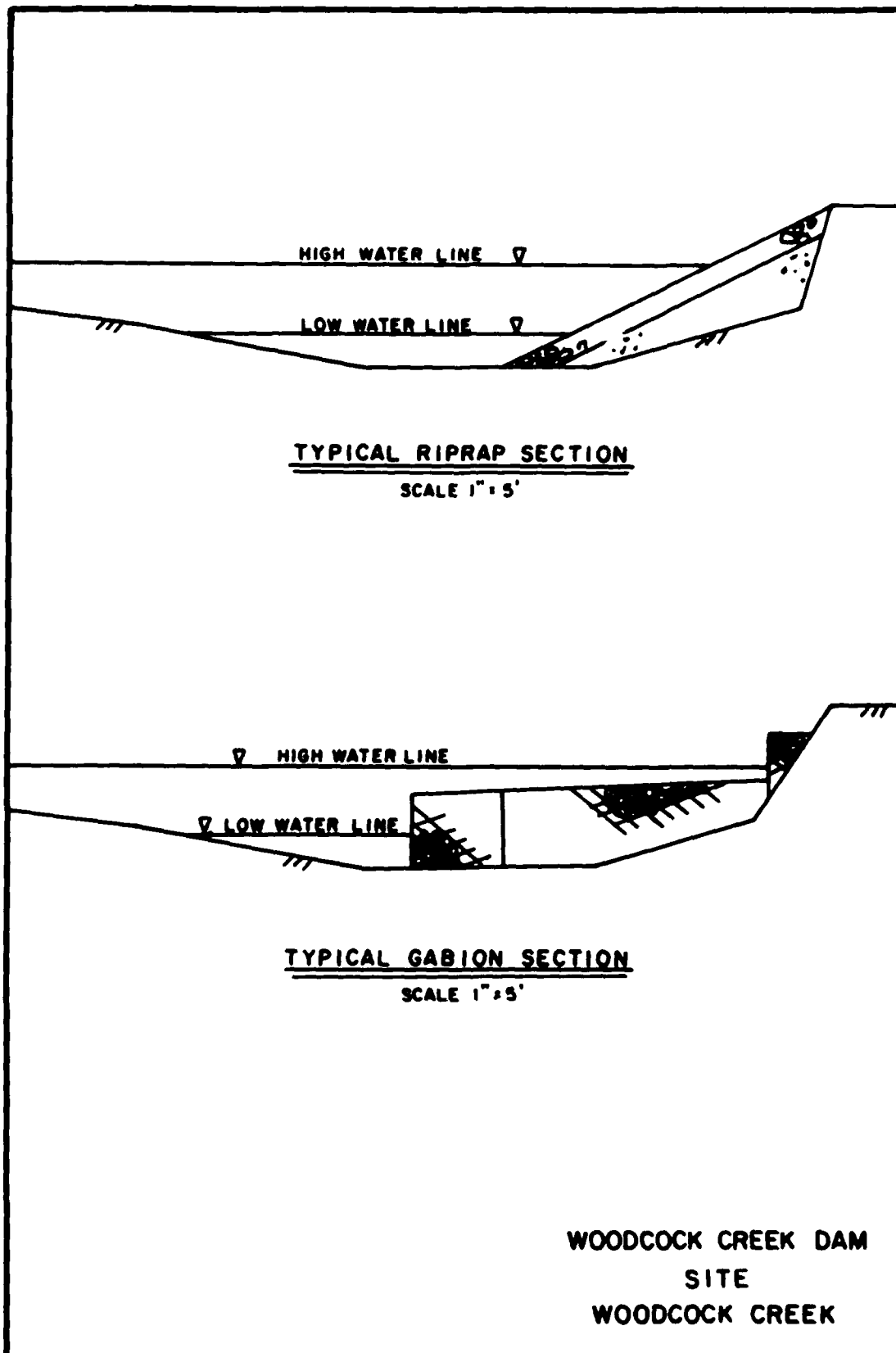
ITEM 43-2

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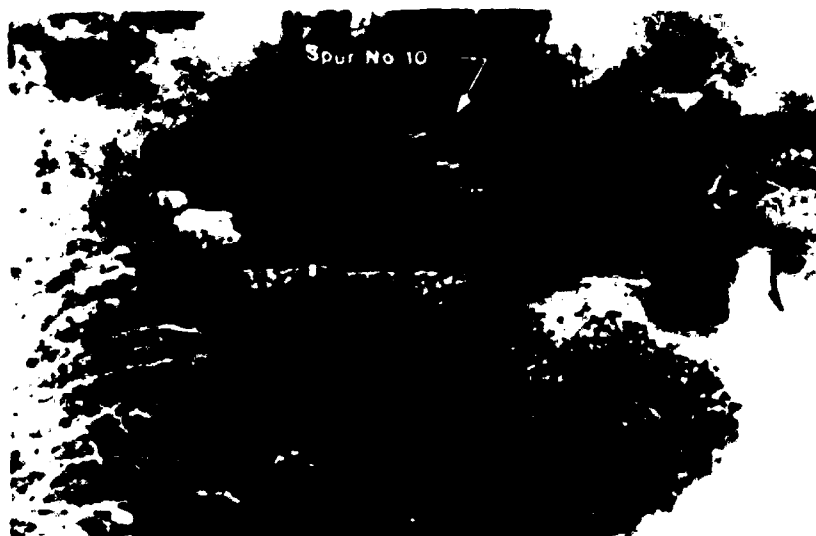
ITEM 43-3

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ITEM 43-4

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Upstream view of spurs No. 10 and No. 11 (8 July 1980)



Upstream view of spurs No. 10 and No. 11 after
a test discharge of 710 cfs from the Woodcock
Creek Dam Outlet Works (15 July 1980)

UPSTREAM VIEW
WOODCOCK CREEK DAM
SITE
WOODCOCK CREEK

ITEM 43-5

H-43-8



Downstream view from spur No. 11 (8 July 1980)



Downstream view from spur No. 11 after
a test discharge of 710 cfs from the
dam outlet works (15 July 1980)

DOWNSTREAM VIEW
WOODCOCK CREEK DAM
SITE
WOODCOCK CREEK

ITEM 43-6

H-43-9

**LITTLE ROCKFISH CREEK
HOPE MILLS, NORTH CAROLINA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Rockfish Creek River Mile 1.0 Side downstream Right looking
Local Vicinity Near Fayetteville Lat N35°00' Long W79°00'
At/Nr City Hope Mills County Cumberland State NC Cong Dist 7th
CE Office Symbol SAWEN-PP Responsible Agency Corps of Engineers
Site Map Sources USGS Quadrangle sheet & plate 3 from March 74 Section 14
Report
Land Use Information Sources Site investigations

(2) Hydrology at or Near Site

Stage Range 1.7 to 10.0 ft. Period of Record 1978 to 1981
Discharge Range 20 to 1100 cfs. Velocity Range 1 to 2.5 fps
Sediment Range ukn to ukn ipd. Period of Record 1976 to 1980
Right
Bank-full Stage 35 ft. Flow ukn cfs. Average Recurrence Interval ukn yr
Bank-full Flow Velocity: Average about 3.4 fps. Near Bank about 1-1.5 fps
Comments Right (eroded) bank is on bluff, 35 ft. high, in 90° curve

(3) Geology and Soil Properties

Right top 12' is medium to fine silty sand
Bank (USCS) lower is lat clay Bed (USCS) Sandy alluvium overlaying clay
Data Sources Field observation
Groundwater Bank Seepage Horizontal flow on top of clay layer
Overbank Drainage Flow from street down bank
Comments

(4) Construction of Protection

Need for Protection Floods of 1972-73 caused high stages and velocities which
contributed to loss of protective vegetation bank cover causing erosion
to begin and continue
Erosion Causative Agents No vegetative cover.

Protection Techniques Gabions on 1.5H to 1V slope, 24 ft. high
General Design Streambed apron section of 3X3X12, w/2 1X3X12 sections in front;
bank paving of 2 1X3X12 sections up 1.5H to 1V slope. Area from gabions
to top bank grassed
Project Length 280 ft. Construction Cost \$ \$76,040.47 Mo/Yr Completed My 76

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 10.0 ft., about 1,000 cfs, Sept. 1978
and 10.8 ft., about 1100 cfs Feb. 79 (dam governs small releases)

Repairs and Costs (Item, Cost, Date) 30 ft. long bulkhead section at base and
crusher run gravel; \$2,000; Sept. 77

Comments Filter system to intercept water table in advance of placing
gabions would be desirable on future projects.

(6) Performance Observations and Summary

Monitoring Program Ground photography, velocity meas at low, med, high flow;
daily stage records

Documentation Sources US Geological Survey measurements & records

Project Effect on Stream Regime Elimination of bank erosion at project site

Project Effect on Environment Reduction of suspended sediment load of stream by
elimination of erosion at project site.

Successful Aspects Erosion stopped.

Unsuccessful Aspects Short portion (30 lin ft) of gabion bank cover slipped
into stream

General Evaluation Seepage on top of clay layer entered bank back of gabions,
slippage of gabions resulted.

Recommendations Repaired damaged area with blanket of crushed stone, pro-
viding timber toe at streambed level.

(7) Additional Information, Comments, and Summary

Map No. 44. The site is near mouth of 18.2 mile long stream with 95
sq. mile drainage area, and is about 500 ft. downstream from a small dam
(110 acre reservoir). The reservoir is generally kept full for recreational
purposes.

Attached Items.

44 - 1 - Project Summary
44 - 2 - Project location
44 - 3 - Cross sections

44 - 4 - Preproject photos
44 - 5 - Photos during repairs
44 - 6 - Photos after repairs and
substantial flow

Little Rockfish Creek

Project Summary

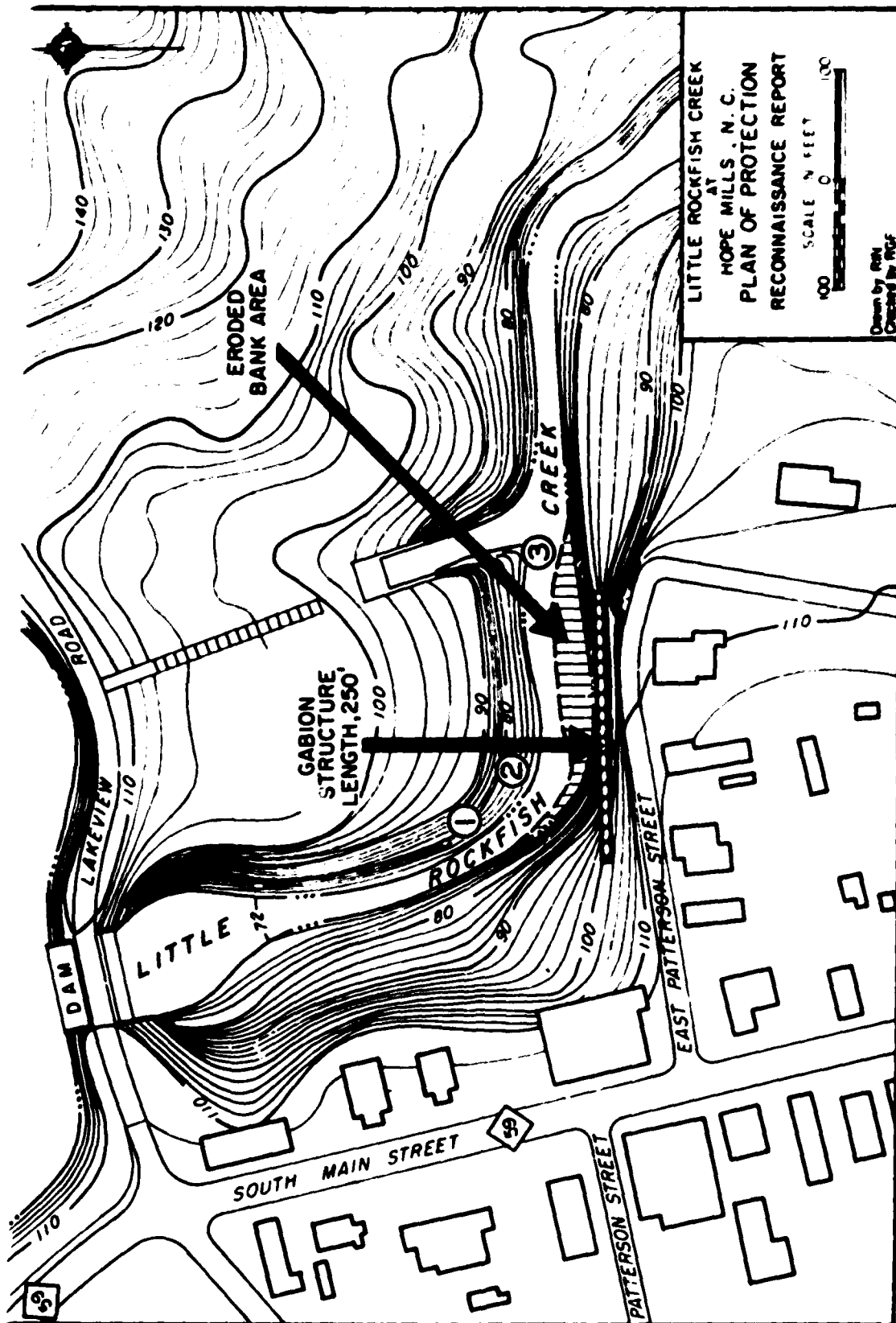
Prior to 1972, serious streambank erosion had been experienced on the cut bank of Little Rockfish Creek in the bend below the dam owned by Dixie Yarns Mill Textile Company. Local officials felt that continued erosion would eventually cause not only a washout of East Patterson Street, but also the loss of town-owned utilities, a 6-in. sewerline, and a 2-in. waterline located beneath the street. Heavy rains during 1972-1973 caused abnormally high stages and correspondingly high velocities in Little Rockfish Creek which resulted in the loss of protective vegetative cover. The town of Hope Mills attempted to retard the failure of the streambank by dumping broken concrete rubble on the slope, but this did not prove to be a successful method to minimize the erosion.

In response to a request from the Mayor of Hope Mills, Wilmington District, CE (SAW), prepared a report that contained recommendations for protection of the eroding streambank and showed that an installation of gabions on filter fabric would be more economical than either a 30-in. layer of riprap placed on filter fabric or the excavation of a new channel. In addition, SAW indicated that without protection a 225-ft section of East Patterson Street would be washed out a minimum of three times during a 50-yr evaluation period; however, with protection a benefit-cost ratio of 1.1 to 1 would be realized. SAW constructed a revetment on Little Rockfish Creek consisting of gabions placed on filter fabric with fescue and rye grass planted on the bank slopes landward of the gabions.

ITEM 44-1
(Sheet 1 of 2)

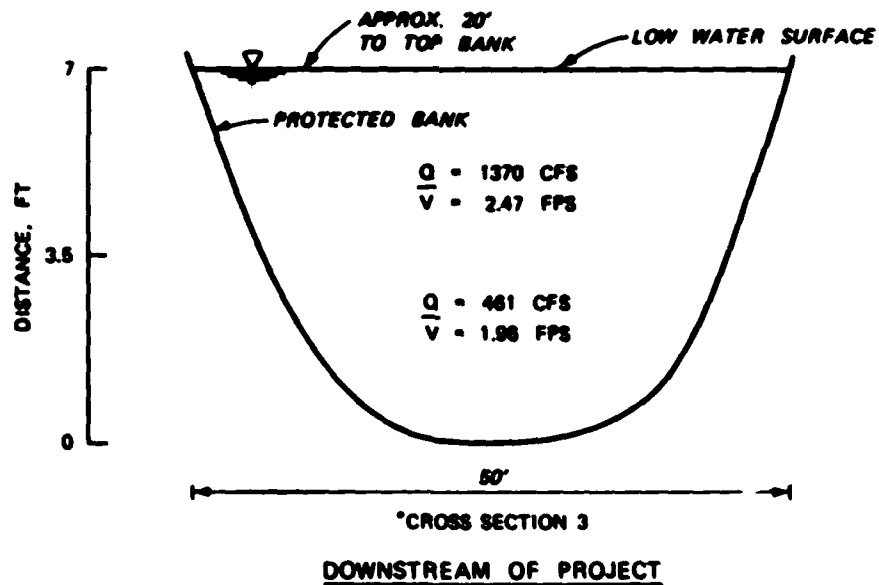
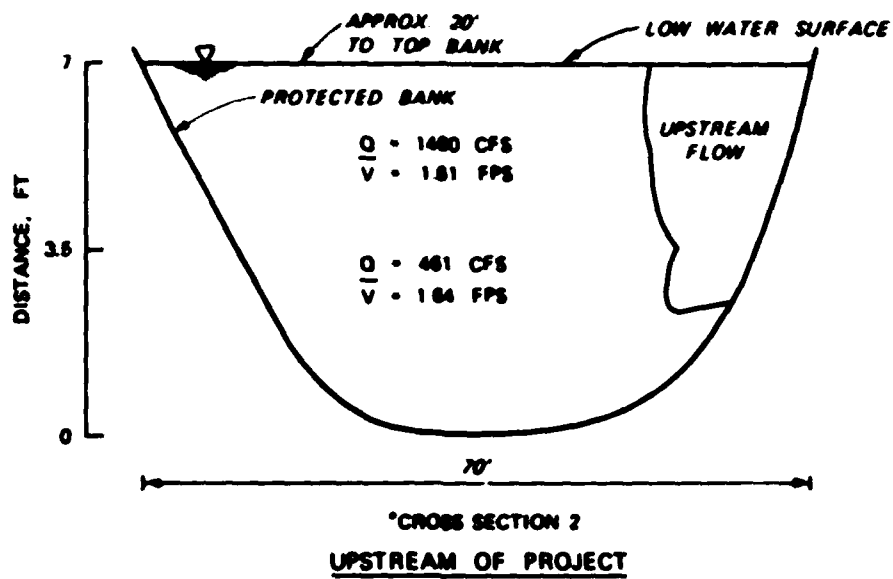
The total length of the completed gabion revetment is approximately 250 ft with the gabions and filter fabric extending 15 ft above the streambed elevation. The installation was completed in May 1976. During construction, the contractor encountered a perched water table resulting in saturation of the construction fill material. In January 1977, approximately 20 lin ft of the revetment slipped 6 to 8 ft vertically. The suspected cause of this failure was groundwater seepage and possibly improper compaction of the fill material beneath the filter fabric, rather than high streamflows or clogging of the filter fabric. Had subsurface investigations been conducted prior to construction of the revetment, the perched water table condition would have been discovered, and measures to divert the groundwater away from the streambank could have been incorporated into the project design. Repairs to the damaged portion of the revetment were made in November 1977. In May 1978, SAW personnel inspected the repaired section and found no evidence of additional failure. A November 1980 inspection by WES and SAW noted very little change with the exception of a scour pocket that had developed at the upstream end of the gabion mattress. SAW concluded that the revetment has performed as designed.

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(Sheet 2 of 2)



ITEM 44-2

H-44-5



*SEE ITEM 44-2

LITTLE ROCKFISH CREEK
AT HOPE MILLS N. C.
PROJECT CROSS SECTIONS

ITEM 44-3

H-44-6

Little Rockfish Creek, view
downstream, preproject con-
ditions. April 1973



Little Rockfish Creek, view
upstream showing middle and
upper end of project, pre-
project conditions. April
1973

ITEM 44-4

H-44-7



Four and one-half years after construction. October 1980



Four and one-half years after construction. Note stability
of repaired section two and one-half years after repair.
October 1980

Little Rockfish Creek, after repairs and substantial flow

ITEM 44-6

H-44-9

**MILL CREEK
MILL CREEK LEVEE, CALIFORNIA**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Mill Creek River Mile _____ Side _____
Local Vicinity Redlands Lat N 34°05' Long W 117° 06'
At/Nr City Mentone County San Bernadino State CA Cong Dist _____
CE Office Symbol SPL Responsible Agency Corps of Engineers
Site Map Sources _____
Land Use Information Sources _____

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft. Period of Record 19____ to 19____
Discharge Range _____ to 18,000 cfs. Velocity Range 8 to 18 fps
Sediment Range _____ to _____ fpd. Period of Record 19____ to 19____
Bank-full Stage _____ ft. Flow _____ cfs. Average Recurrence Interval _____ yr
Bank-full Flow Velocity Average _____ fps. Near Bank _____ fps
Comments Standard Project Flood 33,000 cfs

(3) Geology and Soil Properties

Bank (USCS) _____ Bed (USCS) Sands, gravel, cobbles &
boulders up to 3' in diam
Data Sources _____
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection Protects the integrity of a portion of Mill Creek Levees
and the cities of Mentone and Redlands
Erosion Causative Agents Channel flow
Protection Techniques Mid-floodway gabion structure
General Design _____
Project Length 1,500 ft. Construction Cost \$ 34,000 Mo/Yr Completed 1970

¹⁹⁶⁸
79,000

(5) Maintenance

Experienced Flows (Stage, cfs, Date) _____

Repairs and Costs (Item, Cost, Date) Initial cost in 1968, \$34,000. Repairs in 1979 cost \$79,000.

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Monitoring should continue because of unique hydraulic problems.

Documentation Sources _____

Project Effect on Stream Regime _____

Project Effect on Environment Blends well with natural surroundings.

Successful Aspects _____

Unsuccessful Aspects _____

General Evaluation The mid-floodway gabion barriers have successfully protected the Mill Creek levees during major storms of 1969 & 1978. The barrier

has caused no adverse effects and blends well with natural surroundings.

See photos 1-5.

Recommendations _____

(7) Additional Information, Comments, and Summary

Map No. 45.

Attached Items. _____

45 - 1 - Project features

45 - 2 - Project location

45 - 3 - Project layout and Levee section

45 - 4 & 5 - Sections of gabion barrier

45 - 6 - Photos of project as constructed, and after 1969 flood

45 - 7 - Photos of gabion barrier in 1978 with and without flow.

MID-FLOODWAY GABION BARRIERS

MILL CREEK, SAN BERNARDINO COUNTY, CALIF.

PROJECT FEATURES

1. Description of Original Project. Mill Creek is tributary to Santa Ana River near Mentone, some 5 miles northeast of Redlands, Calif., which drains about 52 square miles of rugged mountain terrain, ranging in elevation from nearly 11,500 feet at San Gorgonio peak to about 1,700 feet at the confluence with the river. The downstream 3 miles traverse an alluvial cone at grades ranging from 3 to 4 percent. In 1960, the Los Angeles District constructed 2.36 miles of levee (see Item 45-3) in an attempt to confine flows to a relatively wide floodway along the right side of the alluvial cone. Incorporated into the project were 2,000 feet of 24-foot-high vertical masonry wall, previously constructed by local interests.

2. Design Considerations. The Mill Creek improvement was designed for a standard project flood of 33,000 cfs (with design velocities ranging from 8 to 18 fps); and the cross-sectional area of the floodway is many times larger than would be required to confine this discharge. This additional area was provided because a number of low-flow channels existed on the steeply sloped alluvial fan. These channels meander throughout the fan, and therefore this extra capacity was incorporated in the design as a contingency against these adverse effects. The stream gradient approximates 4 percent and the debris load is tremendous, including boulders weighing perhaps 10 tons. The effect of this debris load on the stream pattern was underestimated when the improvement was constructed. The cost of the Federal project, not including the masonry wall, was \$654,000 of which \$618,000 were Federal funds.

The floods of 22 November 1965 and 6 December 1966, with a peak discharge of 10,000 cfs estimated for each flood, clearly demonstrated that the Mill Creek Levee project was inadequate to perform its intended purpose, and that satisfactory protection of Redlands and the intermediate area can be achieved only by a different concept of protection. Such protection will be expensive and the best plan can be determined only through a detailed study, with consideration of all alternates.

3. Construction Type and Technique. As an interim measure in 1967 work including restoration of the failed levees, restoration of the graded pilot channel and berm near the upstream end of the project (substantially repetition of work that had been done in 1966), and restoration of the berm and incidental construction of a low-flow channel for nearly a mile downstream of Garnet St. was accomplished. In addition a mid-floodway gabion barrier (Figure 2 on Item 45-4) about 1,500 feet long (See photo 1 on Item 45-6), beginning just downstream of the Garnet Street bridge and designed to prevent development of the cross-channel flow, was constructed. The barrier was supplemented by extensive removal of streambed material on the side of the barrier and continuing downstream for some distance. The spoil was used as an embankment to continue the barrier alignment.

4. Observed Conditions Since Construction. Late in January 1969, there occurred a flood with a peak discharge of 18,100 cfs. The mid-floodway barrier of gabions performed admirably. It was not topped, but at several locations the streambed immediately adjacent to it eroded to a depth of perhaps 5 feet lower than the toe of the structure. (See photo 2 on Item 45-6.) Destruction was prevented by the 1-foot-thick gabion mattress (at the base of the structure), which deflected flow as intended into the holes and prevented undercutting, which would have toppled the structure into the floodway. However, several hundred feet upstream from Garnet Street, where there was no flooding in 1965 or 1966, a small flow broke away from the main stream, crossed and destroyed Garnet Street several hundred feet from the bridge, and followed down between the project levee and the mid-floodway barrier. After-the-flood observations indicated that this flow probably did not exceed 1,000 cfs, but the gabion structure successfully handled at least 17,000 cfs.

San Bernardino County Flood Control District carried on an aggressive flood fight throughout this flood, and subsequently removed many thousand cubic yards of streambed material, pushing it up in a substantial barrier a thousand or more feet long to cut off the newly developed channel, which had resulted in the bridge being bypassed. The County's work also included channel rectification downstream of Garnet Street, some of the material being pushed in against the toe of the mid-floodway gabion barrier, particularly where erosion had undercut the gabion mattress and caused it to deflect.

An equally serious flood occurred on 25 February 1969. The recorded peak discharge is 18,000 cfs. The barrier just upstream of Garnet Street was badly eroded on the stream side, but by working heavy equipment behind it, San Bernardino County forces were able to continuously reinforce it with boulders. This prevented any flow from bypassing the bridge. Downstream of Garnet Street, the gabion barrier performed as before. Erosion along its toe apparently was not deeper than 5 feet, and while the mattress may have been deflected at additional locations, it again prevented undercutting of the gabion superstructure. In contrast, at one downstream location, streambed boulders and cobbles were piled against the face of the structure to the elevation of its top, and some discharge topped it. The vertical faces of the gabions appear to make this type of a structure much less vulnerable to the climbing characteristics of Mill Creek. The overflow was small and did no damage.

5. Maintenance History. Following the 1969 floods, the gabion barrier downstream of Garnet Street was modified (as shown in Figure 3, Item 45-4) to protect the integrity of the structure. Additionally, two new gabion barriers were placed immediately upstream of Garnet Street Bridge to divert flows away from the levees and under the bridge. Item 45-2 shows the location of these gabions, and the gabion section is shown as Figure 4, Item 45-5. The farthest upstream barrier is 675 feet long whereas the barrier located 300 feet upstream of Garnet Street is 500 ft long.

Runoff from the storms of 8-14 February 1978 and 3-6 March 1978 caused some damage to a 200 ft portion of the most upstream gabion barrier shown

ITEM 45-1

(Sheet 2 of 3)

H-45-4

in photo 3 on Item 45-7. As shown in photos 4 and 5 on Item 45-7 water impinged upon the gabion barrier. The only damage to the barrier was the erosion of the top wires of the mattress. To maintain the integrity of this portion of the gabion barrier the modification shown in figure 5 on Item 45-5 was constructed in September 1978. There was no damage to the gabion barriers downstream of Garnet Street.

6. Adverse Effects Caused by Structure. There have been no adverse effects caused by the mid-floodway gabion barriers.

7. Acceptance of Structure and Blending with Natural Surroundings. The mid-floodway gabion barriers blend well with the natural surroundings. This may be observed in photos 1 through 5 on Items 45-6 and 7. The barriers have local acceptance.

8. Performance of Structure. The performance of the barriers is documented in earlier sections of this report.

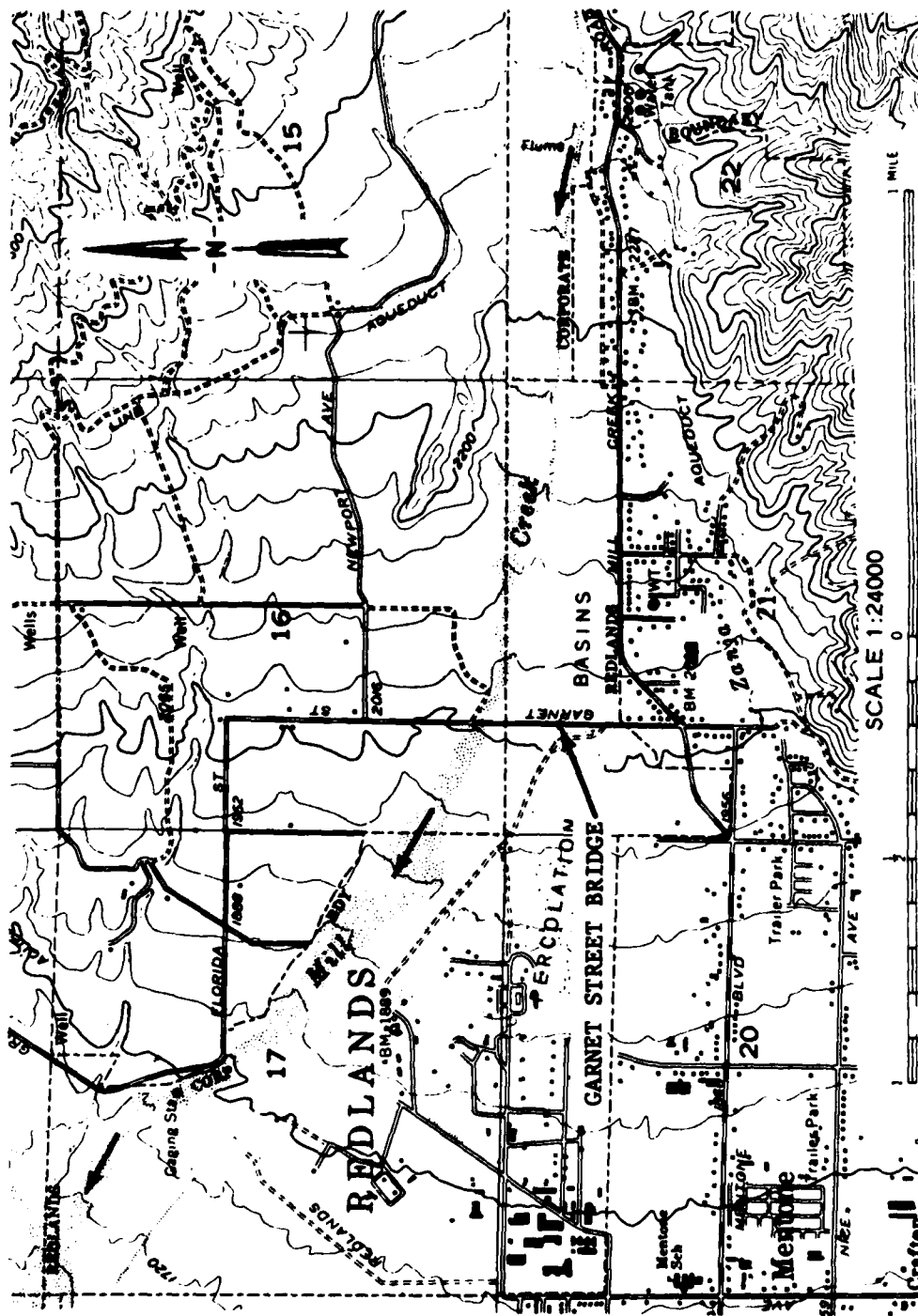
9. Initial Cost and Maintenance Costs. The initial cost of the gabion barriers downstream of Garnet Street was \$34,000 in 1968. In 1970 \$79,000 was spent for repair of these gabions and for construction of the barriers upstream from Garnet Street.

10. Aesthetics of Structure. The gabions blend well with the natural surroundings and are aesthetically acceptable.

11. Monitoring and Site Visit. Monitoring and documentation of this site should be continued because of the unique hydraulic problems at Mill Creek. If the evaluation committee were in the vicinity on other business in the near future a visit to Mill Creek would be of interest.

12. Summary. The mid-floodway gabion barriers have successfully protected the Mill Creek levee during major storms in 1969 and 1978. This location is recommended as a monitoring site to evaluate the performance of mid-floodway gabion barriers under extremely adverse flow conditions. Photographic documentation is available.

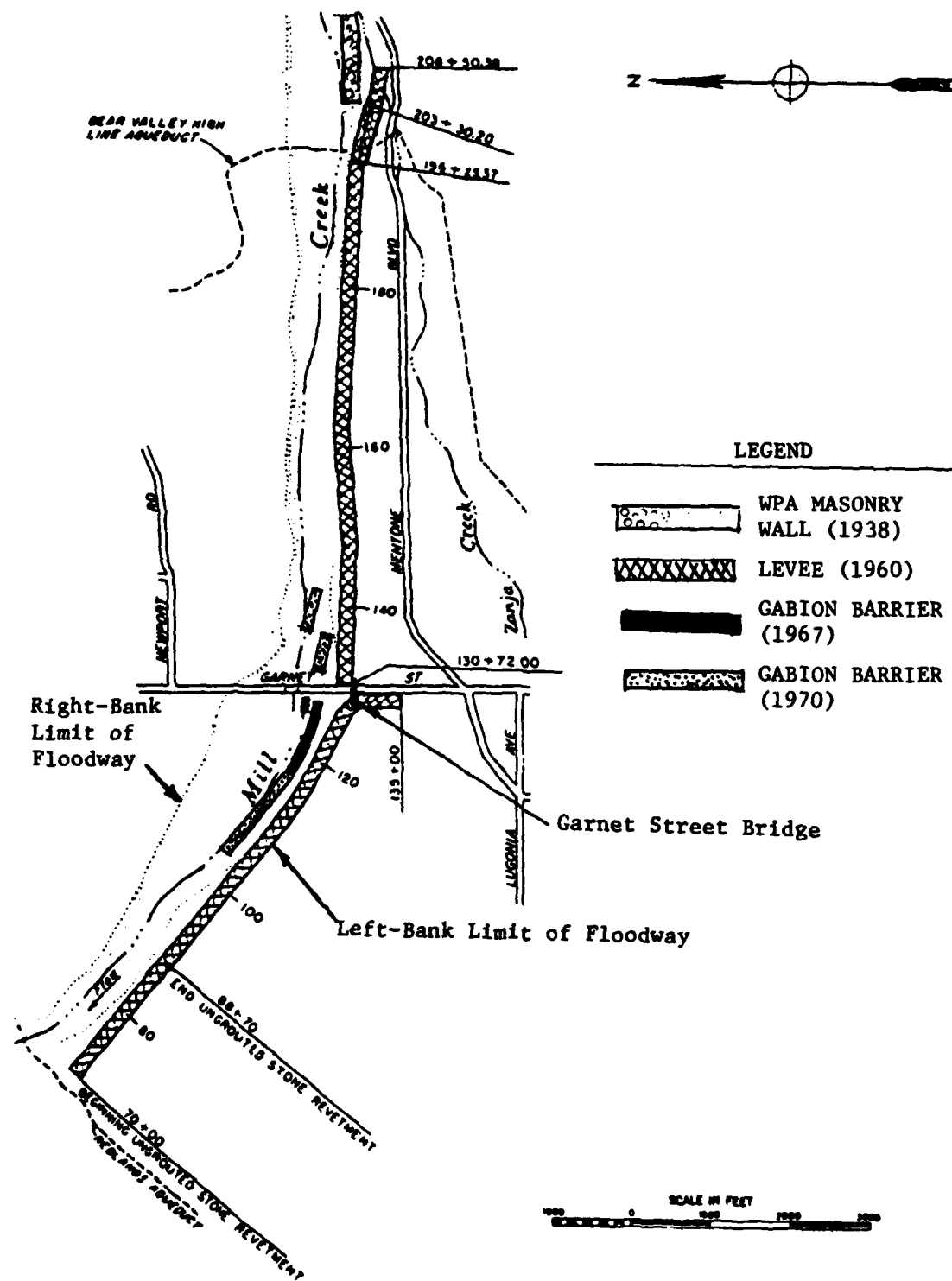
ITEM 45-1
(Sheet 3 of 3)



Mill Creek near Mentone, Calif. (Source: USGS 1:24,000 topographic quadrangle map for Yucaipa, Calif., 1967) (photo revised 1973)

ITEM 45-2

H-45-6



Limits of Mill Creek floodway and locations of WPA masonry wall, levee, and gabion barrier. Station numbers are referenced to 1960 project control line

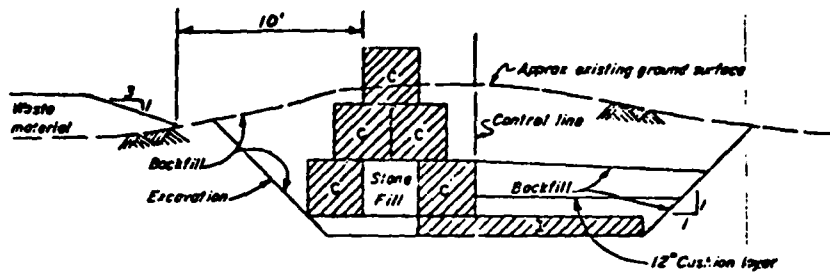


FIGURE 2 SECTION OF MID-FLOODWAY GABION BARRIER DOWNSTREAM OF GARNET ST. BRIDGE, CONSTRUCTED AFTER THE 1966 FLOOD

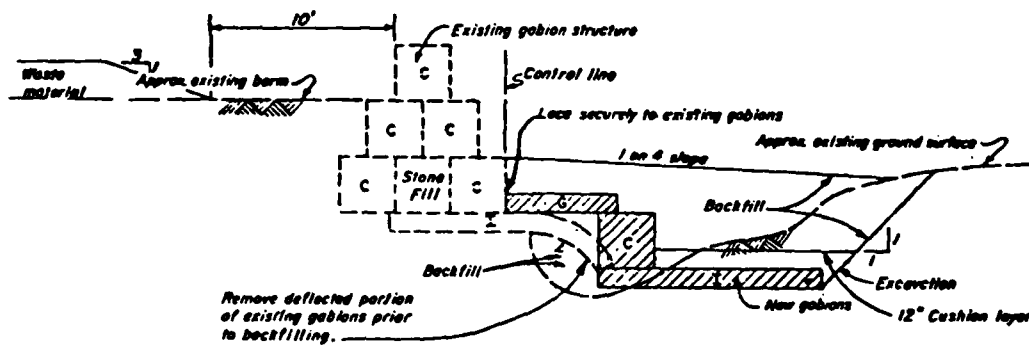


FIGURE 3 MODIFICATION TO GABION BARRIER IMMEDIATELY DOWNSTREAM OF GARNET ST BRIDGE, CONSTRUCTION AFTER 1969 FLOODS

ITEM 45-4

H-45-8

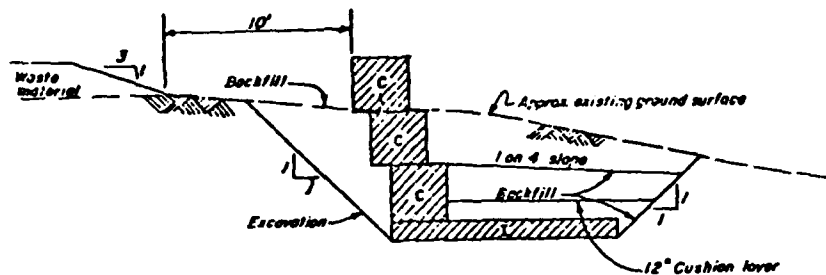


FIGURE 4 SECTION OF MID-FLOODWAY BARRIERS UPSTREAM OF GARNET ST. BRIDGE, CONSTRUCTED AFTER 1969 FLOODS

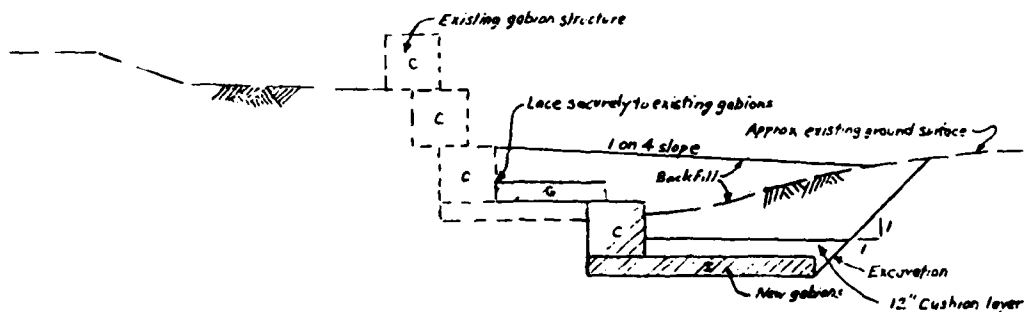


FIGURE 5 PROPOSED MODIFICATION TO A PORTION OF GABION BARRIER UPSTREAM OF GARNET ST. BRIDGE, SCHEDULED FOR CONSTRUCTION IN SEPT. 1978

ITEM 45-5



Photo 1 Mid-Floodway Gabion Barrier Looking Downstream
from Garnet St. Bridge as Constructed

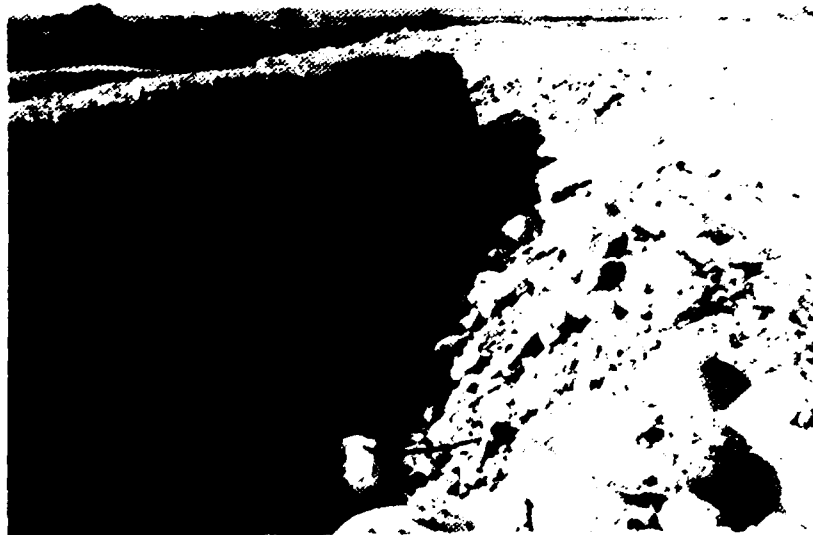


Photo 2 Mid-Floodway Gabion Barrier Looking Downstream
from Garnet St. Bridge After 1969 Floods

ITEM 45-6

H-45-10



Photo 3 Gabion Barrier Upstream of Garnet St. Looking Upstream from Streambed (July 1978)



Photo 4 Gabion Barrier Upstream of Garnet St. Looking Upstream at Flow Adjacent to Barrier (July 1978)



Photo 5 Gabion Barrier Upstream of Garnet St. Looking Downstream at Flow Adjacent to Barrier (July 1978)

ITEM 45-7

**RIO GRANDE RIVER
ESPANOLA, NEW MEXICO**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Rio Grande River Mile 1622.6 Side Rt
Local Vicinity North Central New Mexico Lat N35°57' Long W106°06'
At/Nr City Espanola County Rio Arriba State NM Cong Dist 1st
CE Office Symbol SWA Responsible Agency Santa Clara Pueblo
Site Map Sources Corps of Engineers, Albuquerque Dist.
Land Use Farming

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft: Period of Record ¹⁸~~19~~ 95 to 19 80
Discharge Range 60 to 24,400 cfs: Velocity Range NA to NA fps
Sediment Range _____ to _____ tpd: Period of Record 19 ____ to 19 ____
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average NA fps; Near Bank NA fps
Comments Bed gradient is 9 ft/mi. Average flow is 1506 cfs

(3) Geology and Soil Properties

Bank (USCS) Sand, silt, gravel Bed (USCS) Sands, gravel
Data Sources Visual classification
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection To prevent loss of irrigation canal paralleling river bank
Erosion Causative Agents Velocity and channel alignment
Protection Techniques Steel jetties to promote sediment deposition.
General Design Two front line jetties and six retard lines cabled together and anchored
Project Length 4,313 ft; Construction Cost \$ 35,059 Mo/Yr Completed 7/51

(5) Maintenance

Experienced Flows (Stage, cfs, Date) See (2)

Repairs and Costs (Item, Cost, Date) Replace 6 jetties in 1969, cost not known

Comments: _____

(6) Performance Observations and Summary

Monitoring Program Visual inspections

Documentation Sources Corps of Engineers, Albuquerque District

Project Effect on Stream Regime Riverflow shifted away from bank

Project Effect on Environment Sediment deposited in jetty field has built up
bank and revegetated

Successful Aspects Bank stabilized

Unsuccessful Aspects None

General Evaluation Project has functioned as planned

Recommendations None

(7) Additional Information, Comments, and Summary

Map No. 46

Attached Items: _____

46-1 Project Summary

46-2 Project Location Map

46-3 Project General Plan

46-4 Photographs

46-5 Photographs

KELLNER JACK FIELD ON RIO GRANDE
ESPANOLA, NM

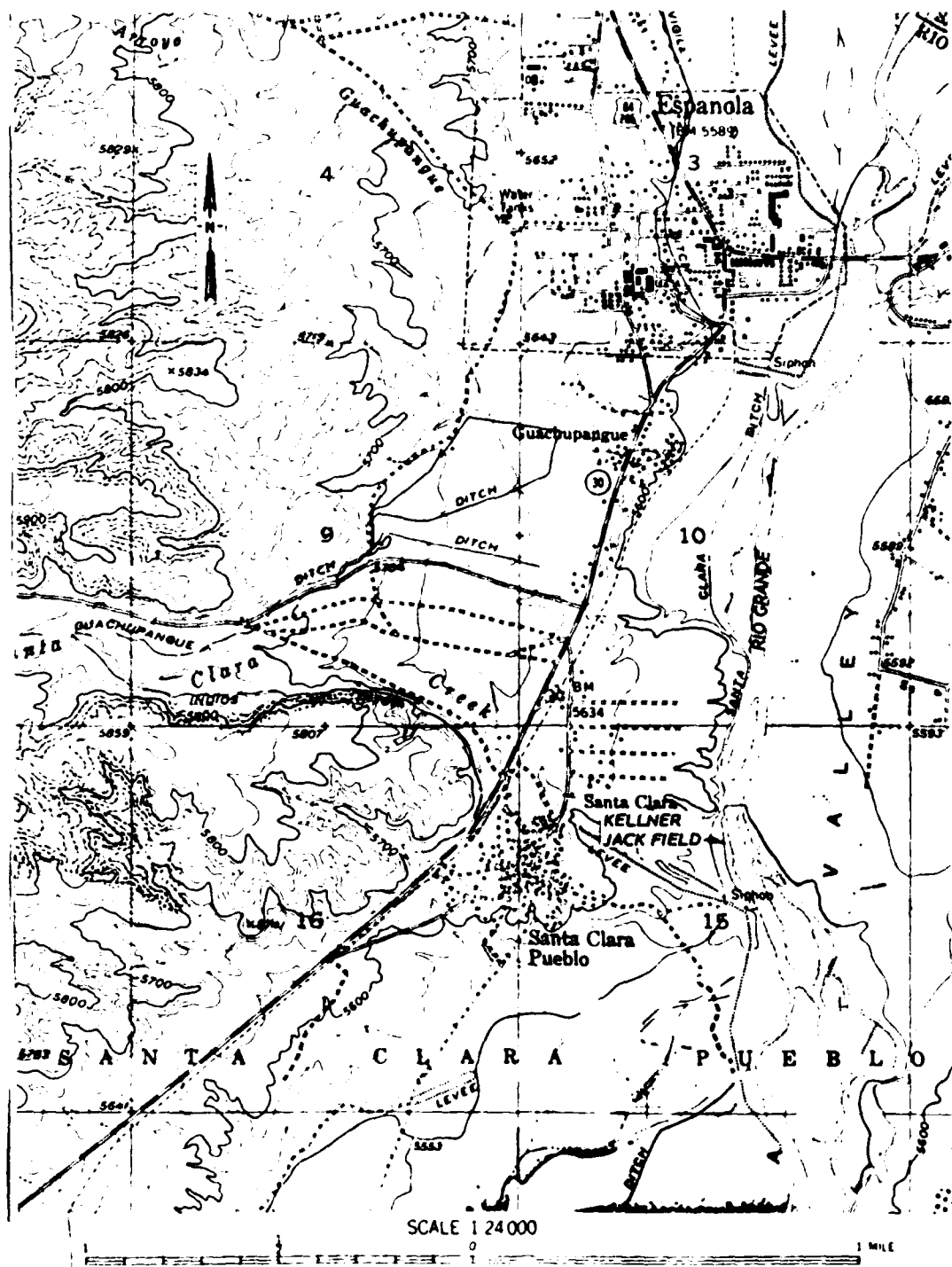
1. The purpose of this project was to provide protection for the Santa Clara irrigation canal which parallels the west bank of the Rio Grande on the Santa Clara Pueblo near Espanola, NM. See the inclosed map (Item 46-2). The bed gradient through this reach is 9 ft/mile. Prior to construction of the project, the river had eroded the west bank until the canal was in danger of being undermined and destroyed. F. D. Shufflebarger, Inc. of Albuquerque installed a Kellner Jack Field measuring 4,313 linear feet in 1951, as shown on Item 46-3. This installation consists of two main-line jetties (2,900 and 700 feet in length) and six backup retards varying from 133 to 265 feet in length. The jack units consist of three linear members (each 15 feet in length) bolted together at their midpoints such that each member is perpendicular to the other two. The members are laced with #10 steel wire; the jack units are connected with cable. The Bureau of Indian Affairs sponsored this project at a funding level of \$24,108 (1951 dollars).

2. Inspections performed by Albuquerque District (AD) personnel in 1955, 1956, 1958, 1960, 1963, and 1968 indicated that the project was performing its function; some of the jacks were in poor condition at the time of the 1968 inspection. Considerable sediment deposition had occurred, providing a substrate for dense growths of willow, Russian olive, and underbrush that have measurably improved the stability of the bank.

3. Six new jacks were required in 1969 as a repair measure. Beginning in 1972 a gravel bar began to build up in front of the project which has served to direct the flow away from the Kellner Jack Field (Item 46-5). The last inspection, conducted in November 1976, indicated that the project has been effective in preventing further bank erosion and has encouraged sediment deposition and general restoration of the eroded bank.

ITEM 46-1

H-46-3



Rio Grande River near Espanola, New Mexico. Source:
USGS quadrangle map for Espanola, New Mexico (1953)

ITEM 46-2

H-46-4



ITEM 46-3



Section of main-line jetty



Jacks units are connected with steel cable; this figure shows the cable configuration at the junction of a main-line jetty and retard line

ITEM 46-4

H-46-6



Dense vegetation growths have improved the stability
of the bank



A gravel bar that began forming in 1972 has directed
the streamflow away from the project

ITEM 46-5

H-46-7

**CUCHILLO NEGRO CREEK
TRUTH OR CONSEQUENCES, NEW MEXICO**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Cuchillo Negro Creek River Mile 0 Side Rt & Lt
Local Vicinity South Central New Mexico Lat N33°9' Long W107°17'
At/Nr City Truth or Consequences County Sierra State NM Cong Dist 2nd
CE Office Symbol SWA Responsible Agency Sierra County
Site Map Sources Corps of Engineers, Albuquerque District
Land Use Residential and Commercial

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft; Period of Record 19__ to 19__
Discharge Range _____ to _____ cfs; Velocity Range _____ to _____ fps
Sediment Range _____ to _____ tpd; Period of Record 19__ to 19__
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments No data available. Estimated peak discharge 21,400 cfs.
Bed gradient is 40 ft/mi

(3) Geology and Soil Properties

Bank (USCS) Sandy gravel Bed (USCS) Sandy gravel
Data Sources Visual classification
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection To prevent loss of levees and bridge abutments
Erosion Causative Agents Steep bed gradients. Channel meandering
Protection Techniques Gabion revetment and groins
General Design Fill placed in eroded levees, installed sections of gabions
as levee toe protection and gabion groins
Project Length 3400 ft; Construction Cost \$ 60,000 Mo/Yr Completed ?/76

(5) Maintenance

Experienced Flows (Stage, cfs, Date) See (2)

Repairs and Costs (Item, Cost, Date) _____

Comments: Project sponsor has not accomplished recommended repairs

(6) Performance Observations and Summary

Monitoring Program Visual inspections

Documentation Sources Corps of Engineers, Albuquerque, NM

Project Effect on Stream Regime Channel alignment has been straightened

Project Effect on Environment Flood damage on adjacent development has been prevented

Successful Aspects Flows have been contained between levees and bridge abutments

Unsuccessful Aspects Flows have undercut 150 foot section of gabion revetment causing wall to drop 4 to 5 feet.

General Evaluation Project has functioned as planned under emergency conditions (PL 99).

Recommendations Deeper placement of gabion revetment would have prevented undercutting

(7) Additional Information, Comments, and Summary

Map No. 47. Corps of Engineers recommended to sponsor that repairs be made to gabion revetment. No repairs made to date.

Attached Items:

47-1 Project Summary

47-2 Project location map.

47-3 Project General plan

47-4 Sections and Detail

47-5&6 Photographs

CUCHILLO NEGRO CREEK
AT TRUTH OR CONSEQUENCES, NM

1. Cuchillo Negro Creek, a west bank tributary, enters the Rio Grande just upstream from Truth or Consequences. See the inclosed map (Item 47-2). The creek, which originates along the Continental Divide in the western part of Sierra County, is 60 miles long and drains an area of 325 square miles. A low 25-acre area along the south bank of the creek is protected by an existing levee extending from high ground to the New Mexico Highway 51 Bridge. A lower levee is located along the north bank. Both of these levees are constructed of native materials (sandy gravel) and are highly erosive, even during low flows. Failure of the south bank levee would result in flash flooding to the low area which contains three permanent residences, 50 to 60 mobile homes, a ready-mix concrete plant, a boat storage facility, and a marine service.
2. This reach is characterized by steep bed gradients (40 ft/mile) and high peak discharges that occur during periods of intense rainfall. The upper reach of the stream above the community of Cuchillo is perennial; however, downstream from there the stream becomes ephemeral due to agricultural irrigation. Major flows occur during the rainy season from July to September each year. The riverbed in the vicinity of the bank protection works is dry during most of the remaining months. No regular stage recording stations are located on the creek; however, during the August 1975 flood, the USGS recorded a peak discharge of 21,400 cfs. Sparse vegetation has become established on the levee faces.
3. The construction date is not known for either levee nor is the builder. Local residents said the levees were pre-1955; however, no other information is available. The Corps became involved in the project after a flood disaster in 1972. Funds were made available under the continuing authority of PL 84-99 (28 June 1955) to preserve and repair existing flood protection works. The project was awarded to Vesper Materials Co., Espanola, NM, on 14 November 1972. Work commenced on 4 December 1972 and was completed on 26 January 1973. This effort consisted of extensive levee repairs on both banks along approximately 3400 feet of Cuchillo Negro Creek and the installation of seven pairs of rock-filled gabion-type training dikes to control the channel approach to the Highway 51 bridge.
4. An inspection conducted by Albuquerque District (AD) personnel on 11 April 1974 indicated that the seven pairs of gabion dikes and the north levee were in an as-built condition. The south levee was intact except for a minor toe cut about 2 feet high by 50 feet long due to low flows in 1973.
5. In 1974, the Corps of Engineers agreed to restore the damaged levee embankment and furnish additional gabions to Sierra County. The county agreed to furnish the labor required, equipment, and rock fill for the gabions. The plan was to place several courses of gabions parallel to

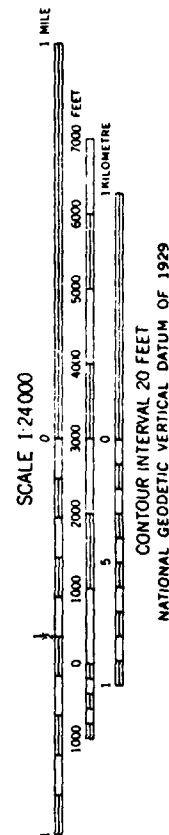
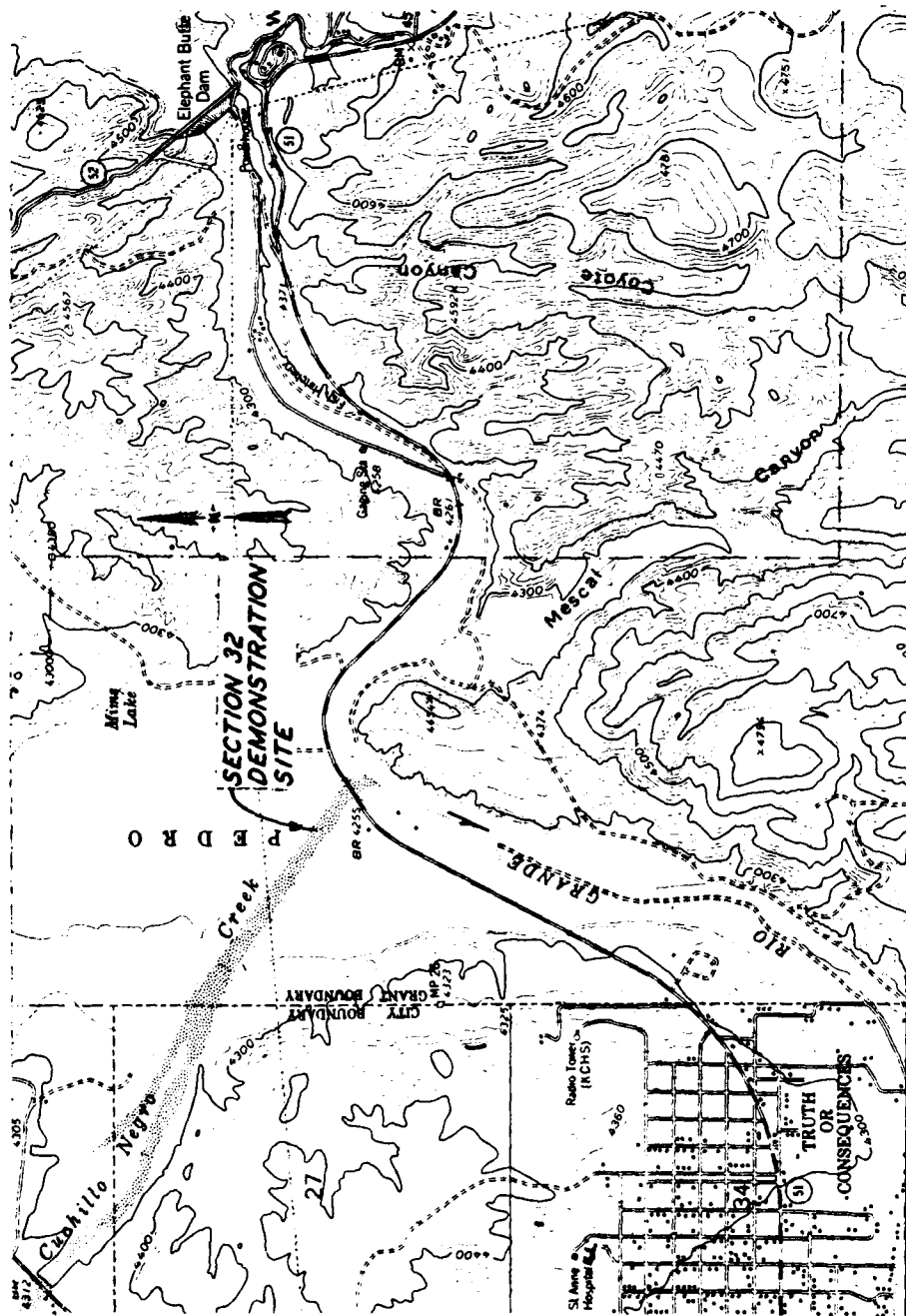
ITEM 47-1
(Sheet 1 of 2)

the bank locations subject to severe erosion. The county proceeded with the gabion installation in August 1975. On 5 September 1975 when the first course of gabions had been partially installed, high creek flows caused a suspension of the work. These flows resulted in damage to the incomplete gabion revetment and to the levee embankment. An additional inspection conducted by AD personnel on 10 October 1975 indicated that the upstream half of this project had been severely damaged. About 600 feet of the north levee was eroded by streamflows in 1974 and September 1975; 200 feet were nearly gone. The right-of-way fence was undercut and still hangs in the gap. The present bank at this location is about three feet high. Floodwater here was within two inches of overtopping this cut bank. The south levee was deeply cut but not breached. The flows that passed over the incompleted revetment cut the levee halfway through. The lower half of this project was in good condition. The seven pairs of gabion training dikes completely controlled the floods and protected the levees on both sides.

6. Cuchillo Negro Creek flowed only two to three feet deep at flood crest, yet it cut and removed hundreds of feet of levee 12 to 15 feet high by 25 feet wide at the crest. Toe and slope protection are vital in sandy projects, as the lower half of this project attests.

7. Before repairs of the 1975 flood damage, another 15,000-25,000 cu yd of levee erosion occurred in the September 1976 flood. In October 1976 and April 1977 the south levee was repaired. The now completed work includes: (a) placement of compacted fill secured from the channel bed in the eroded areas of the levee; (b) placement of rock-fill protection in selected portions of the restored levee; (c) completion of the gabion revetment installation on the west end of the levee; (d) installation of eight training dikes between the existing training dikes and the gabion revetment installation. A project map for this work is shown in Item 47-2. Federal and local participation costs were \$38,000 and \$22,000, respectively. Land use reevaluation of the area behind the north levee indicated that there was nothing of value to protect; therefore, no efforts were made to protect this levee, thus allowing the creek width to expand and reduce flow concentrations along the south levee.

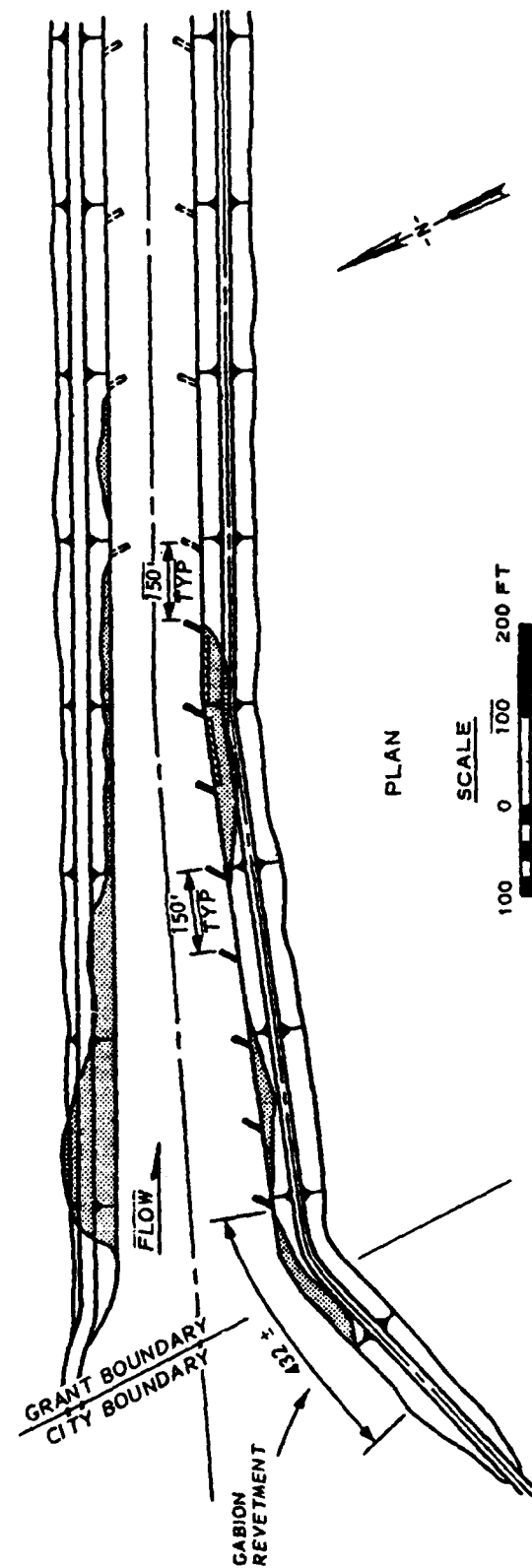
ITEM 47-1
(Sheet 2 of 2)



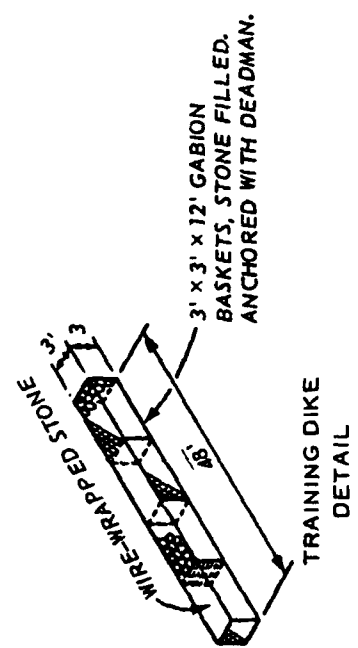
Cuchillo Negro Creek near Truth or Consequences, New Mexico.
Source: USGS quadrangle map for Elephant Butte, New Mexico (1959)

ITEM 47-2

H-47-5



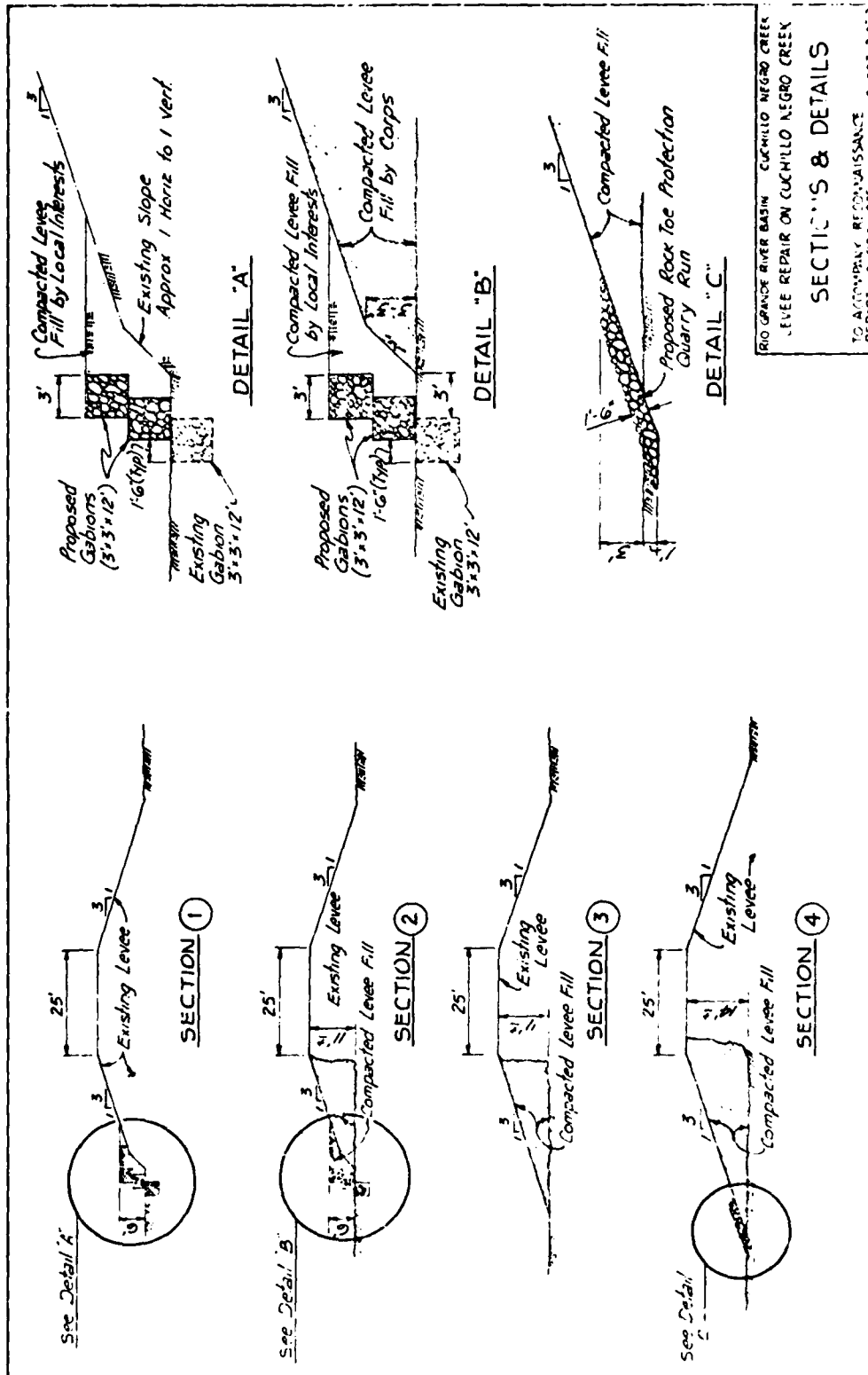
- LEGEND**
- ERODED AREAS
 - EXISTING STONE TRAINING DIKES
 - PROPOSED ROCK TOE PROTECTION
 - PROPOSED STONE TRAINING DIKES (SEE DETAIL)



Project map for work completed in 1977

ITEM 47-3

H-47-6



ITEM 47-4

H-47-7



Gabion training dike placed by Vesper Materials Co.
in December 1972



Right-of-way fence undercut on north levee

ITEM 47-5

H-47-8



Fill material for eroded levee areas was secured
from the channel bed



Completed gabion revetment on west end of south levee

ITEM 47-6

H-47-9

**TRINITY RIVER
MOSS HILL, TEXAS**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Trinity River River Mile 71 Side Left (East)
Local Vicinity N Lat 30°16'38" Long W94°47'52"
At/Nr City Moss Hill County Liberty State TX Cong Dist 2
CE Office Symbol SWGED-PS Responsible Agency St. Dept. of Highways & Pub. Trans.
Site Map Sources USGS Rayburn, TX (1:62500); Hwy. Dept. County Map
Land Use Forest

(2) Hydrology at or Near Site (Rumayor, TX)

Stage Range to ft; Period of Record 1924 to 1979
Discharge Range 102 to 111,000 cfs; Velocity Range 0.9 to 7.2 fps
Sediment Range 13 to 273,000 tpd; Period of Record 1968 to 1971
Bank-full Stage ft; Flow cfs; Average Recurrence Interval yr
Bank-full Flow Velocity: Average fps; Near Bank fps
Comments Scattered measurements, 1973-1975.

(3) Geology and Soil Properties

Bank (USCS) Clay, sand (Test Hole #6) Bed (USCS) Clay, sand (Test Holes #4-5)
Data Sources Soil borings, St. Dept. of Highways & Pub. Trans., circa 1964.
Groundwater Bank Seepage No data available.
Overbank Drainage
Comments

(4) Construction of Protection

Need for Protection To prevent bank erosion upstream from Texas FM Road
162 Bridge.
Erosion Causative Agents High stage streamflows against concave bank of
bendway.
Protection Techniques Five timber diverters and riprap
General Design Divert riverflows using timber diverters.
Project Length 600 ft; Construction Cost \$ 171,000 Mo/Yr Completed 6/78

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Maximum discharge since bridge completed
99,000 cfs in June 1973.

Repairs and Costs (Item, Cost, Date) Replacement or repair not accomplished by
1 Oct 1981.

Comments: Repairs or replacement of one timber diverter is needed.

(6) Performance Observations and Summary

Monitoring Program Cross sections were taken 23-24 Oct 1979.

Documentation Sources Inspection

Project Effect on Stream Regime _____

Project Effect on Environment _____

Successful Aspects Four diverters are in good condition and have slowed the
erosion.

Unsuccessful Aspects The second diverter upstream from the bridge is in
need of extensive repairs or replacement.

General Evaluation Riprap revetment required on bank and at two bridge piers
in addition to timber diverters for erosion control.

Recommendations Timber diverters should be used as temporary measures
and adequate stone protection for long-term protection.

(7) Additional Information, Comments, and Summary

Map No. 48

Attached Items: _____

48-1 Project Summary 48-4 Views of Bank Protection 1974-78

48-2 Project Plan & Location 48-5 Views of Bank Protection 1979

48-3 Typical Sections of Bank Protection

TRINITY RIVER AND TRIBUTARIES, TEXAS, FM 162 BRIDGE AT MOSS HILL
EVALUATION OF EXISTING STREAMBANK PROTECTION WORKS

Problem. Channel scour and bank erosion are being experienced along the concave left descending (east) bank of the Trinity River at the FM 162 Bridge as the river channel attempts to migrate eastward. Following construction of the bridge in 1966, the State Department of Highways and Public Transportation constructed five timber diverters upstream from the bridge along the east bank to divert riverflows and slow the process of erosion. The diverters achieved a measure of success; however, erosion occurred downstream from the diverter system and between 1966 and 1976 the bank receded about 60 ft, the toe of the channel shifted about 110 ft, and about 20 ft of vertical scouring occurred at the bridge. Erosion is endangering the bridge bent on the east bank, and scouring action has almost exposed the top of the piles supporting two bridge bents in the river.

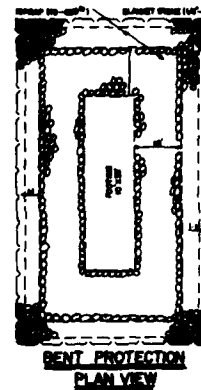
Protection. Under Section 14 of the 1946 Flood Control Act, as amended, the Galveston District will protect the FM 162 Bridge by shaping the east bank and placing blanket stone and riprap bank protection along about 240 lin ft of the bank in the immediate vicinity of the bridge, and by placing blanket stone and riprap around the base of the two bridge bents in the river. A contract for this work was awarded in February 1978. The layout of the existing diverters and the proposed bank and bent protection is shown in Item 48-2 and the details are shown in Item 48-3.

Cost. The five timber diverters cost \$62,500 when they were constructed in 1967. The Highway Department estimates their replacement cost at \$175,000 (Nov 1975 price levels). The Section 14 project had a total cost of \$171,000.

Monitoring. Existing conditions are established from hydrographic and topographic surveys and both aerial and water-level photographs. Additionally, photographs will be made after completion of the Section 14 project.

Status. Some erosion has occurred behind the diverters since they were constructed in 1967, but generally, four of the structures are in good condition. The second diverter upstream from the bridge is in need of extensive repairs or complete replacement. Construction of the Section 14 project was commenced in April 1978 and completed in June 1978. Photographs taken during 1974, 1978, and 1979 are provided in Items 48-4 and 48-5.

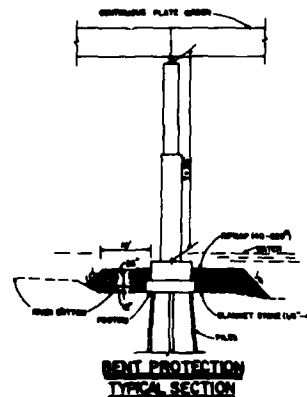
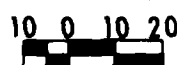
ITEM 48-1



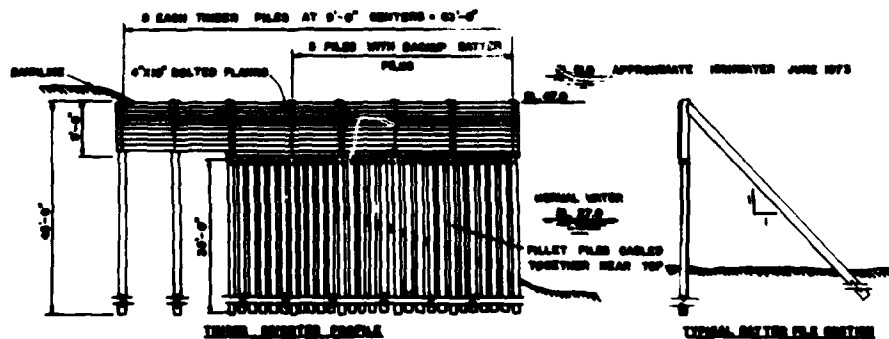
TOP OF BANK PROTECTION TO
EXTEND TO ELEVATION 40 APPROXIMATE
AND DOWNSTREAM OF BRIDGE
EXTEND DOWN TO TOP OF BANK
ONLY. SEE PLAN SHEET

TYPICAL BANK PROTECTION

SCALE IN FEET



NOTE: ALL PILES HAVE 6" TIP AND 15" BUTT.



TRINITY RIVER, TEXAS LEFT BANKLINE PROTECTION UPSTREAM FROM FM 162 BRIDGE
INVERTED FENCE DETAIL

TYPICAL SECTIONS OF BANK PROTECTION
FM 162, TRINITY RIVER, TEXAS

ITEM 48-3

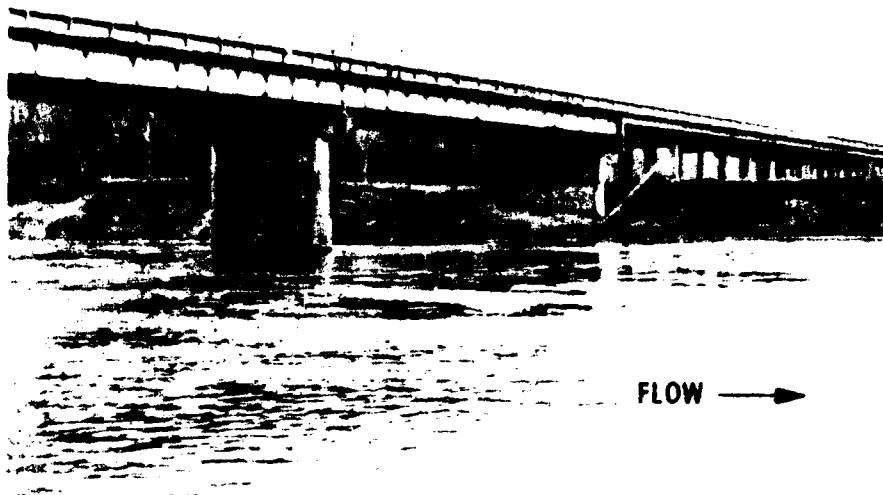


PHOTO 1: LOOKING EASTWARD AT LEFT DESCENDING
BANK BEFORE CONSTRUCTION OF SECTION 14 PROJECT
DATE: FEBRUARY 1978 BANK HEIGHT: 15 FEET

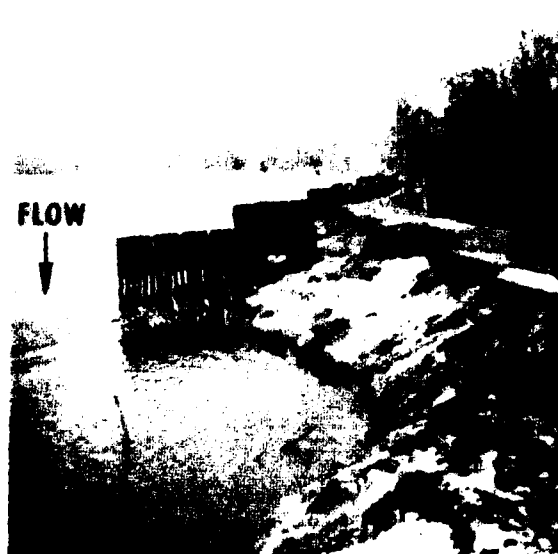


PHOTO 2: LOOKING UPSTREAM AT DIVERTERS
DATE: MAY 1974 BANK HEIGHT: 20 FEET

VIEWS OF BANK PROTECTION
FM 162, TRINITY RIVER, TEXAS

ITEM 48-4

H-48-6

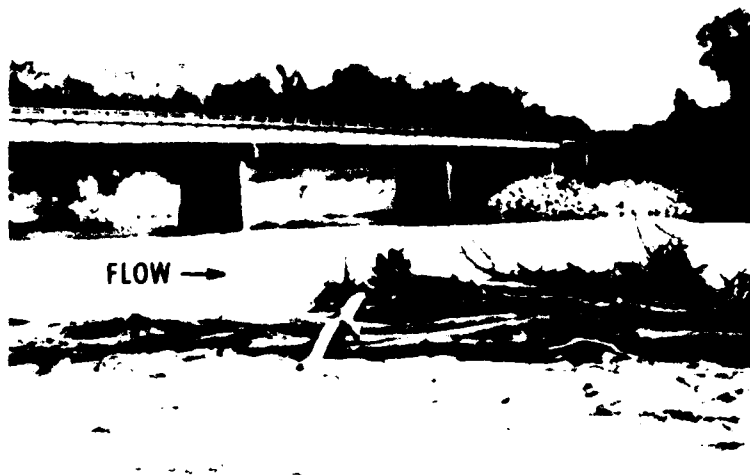


PHOTO 3: VIEW OF RIPRAP BANK PROTECTION 1.5 YEARS
AFTER COMPLETION
DATE: NOVEMBER 1979

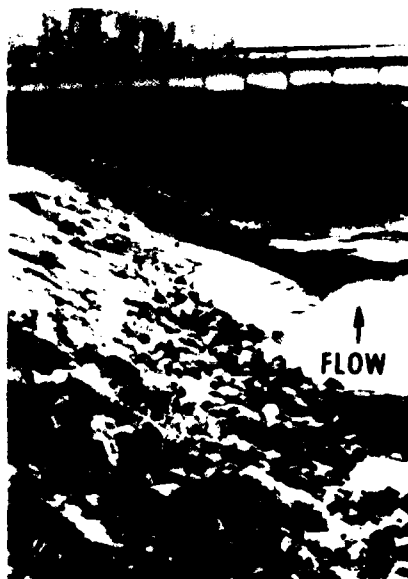


PHOTO 4: LOOKING DOWNSTREAM
AT RIPRAP BANK PROTECTION
DATE: NOVEMBER 1979

VIEWS OF BANK PROTECTION
FM 162, TRINITY RIVER, TEXAS

ITEM 48-5

**ARKANSAS RIVER
MERRISACH LAKE, ARKANSAS**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Merrisach Lake River Mile 14.5 Side Left
Locality Vicinity South East, Arkansas Lat N34°00' Long W91°15'
Attn City Gillett County Arkansas State AR Cong Dist 2
CE Office Symbol LRD Responsible Agency Corps of Engineers
Site Map Sources Corps of Engineers, Little Rock, AR
Land Use Information Sources Recreation

(2) Hydrology at or Near Site

Stage Range 160 to 168 ft; Period of Record 19 72 to 19 81
Discharge Range N/A to _____ cfs; Velocity Range N/A to _____ fps
Sediment Range N/A to _____ tpd; Period of Record 19 _____ to 19 _____
Bank-full Stage N/A ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average N/A fps; Near Bank _____ fps
Comments Site is located on a lake, erosion caused by wind and boat
generated waves not current.

(3) Geology and Soil Properties

Bank (USCS) CL-clay Bed (USCS) CL-clay
Data Sources Borings for park roads (dwg 8635-53/1215)
Groundwater Bank Seepage Not visible
Overbank Drainage Limited
Comments Overbank drainage and wave splash pools behind wall contributing
to deterioration.

(4) Construction of Protection

Need for Protection To prevent damage to developed recreation area
Erosion Causative Agents Wind and boat generated waves
Protection Techniques Wood sheet-pile bulkhead
General Design Job built tounge and groove pieces (3"x12") driven into
ground and capped with stringer at bank level.
Project Length 3,325 ft; Construction Cost \$ 91,000 Mo/Yr Completed Sept 1972

(5) Maintenance

Experienced Flows (Stage, cfs, Date) N/A

Repairs and Costs (Item, Cost, Date) None to date

Comments: Addition of riprap at end of walls is planned; damage and cost are minor

(6) Performance Observations and Summary

Monitoring Program Visual inspections

Documentation Sources Corps of Engineers, Little Rock District

Project Effect on Stream Regime N/A

Project Effect on Environment Minimum, bulkhead presents an artificial appearance but is no more unsightly than eroding shore.

Successful Aspects Shoreline stabilized, damage and maintenance minimal.

Unsuccessful Aspects

General Evaluation Wave-caused erosion prevented, wall allows maximum positive drainage through wall.

Recommendations Replace cap boards and fill scallops with stone.

(7) Additional Information, Comments, and Summary

Map No. 49. Repairs could be done at relatively small expense. Continuous minor repair work will maintain the usefulness of this shore protection for the public good. On new projects, scallops at ends of the wall should be controlled by use of an adequate riprap blanket at turnbacks. Use cypress timber.

Attached Items:

49-1 Project summary

49-4 Details of wood wall

49-2 Project location

49-(5-8) Photographs

49-3 Typical section

MERRISACH LAKE PROJECT

The Southwestern Division proposed that the Little Rock District's bank protection project at the Merrisach Lake public use area be included in the Section 32 Program for monitoring of existing sites. The public use area, as shown on Item 49-2, is located adjacent to the navigation channel of the Kerr-McClellan Arkansas River Navigation System.

The bank protection consists of five sections of timber sheet-pile walls as shown on Item 49-4, typical cross sections and pile details are shown on Item 49-3. Photographs of the completed structure are shown on Items 49-5, 6, 7, and 8.

The Merrisach Lake protection includes 3,325 feet of wall. Completed in September 1972, the in-place costs are summarized below.

<u>Item</u>	<u>Units</u>	<u>Unit Cost</u>	<u>Total Cost</u>
Timber Pile Walls	3,325 ft	\$21.49	\$71,500
Filter Fabric	20,000 ft ²	.25	5,100
Select Fill	1,800 yds ³	3.23	5,800
Sand	860 yds ³	10.00	8,600
TOTAL			\$91,000

The resulting cost of the protection per linear foot of bank line was \$27.37

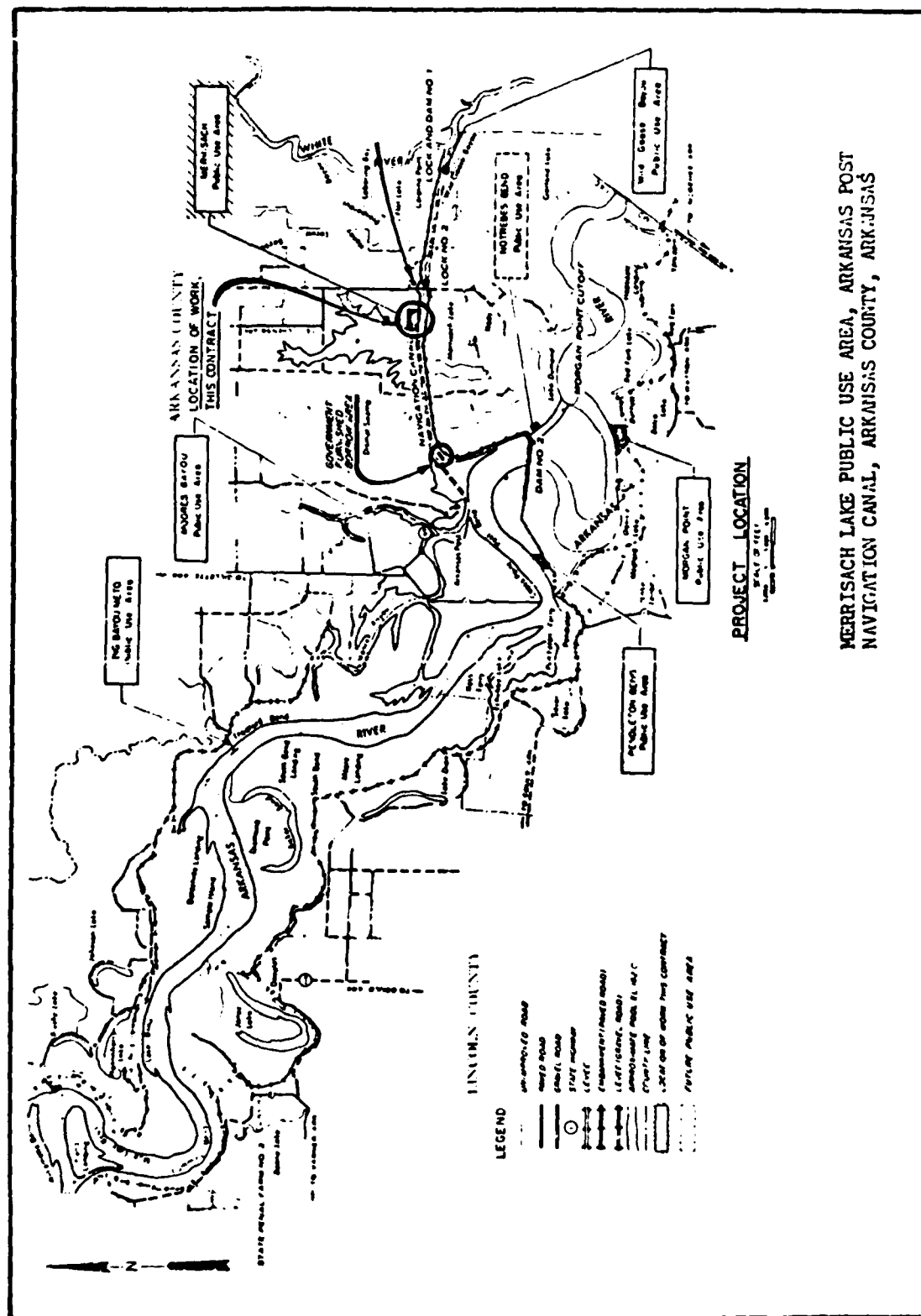
The protection measures were provided to prevent the loss of Merrisach Lake public use area facilities to bank erosion. The major erosive action results from the natural wave action on Merrisach Lake, from pleasure boat wakes in the lake, and from commercial navigation traversing the Arkansas Post Canal. Stream velocities along the bank line are negligible. The water level is controlled by the spillway at Dam No. 2 and is generally maintained between elevations 162.0 and 163.0 except during major flooding periods of either the Arkansas or Mississippi Rivers. During the peak of the 1973 floods, the entire park was submerged to about elevation 168.8. No damage has been sustained by the protection measures until 1978 when some cap boards became loose. In 1980 about 10 percent of wall required some repair to cap boards.

ITEM 49-1

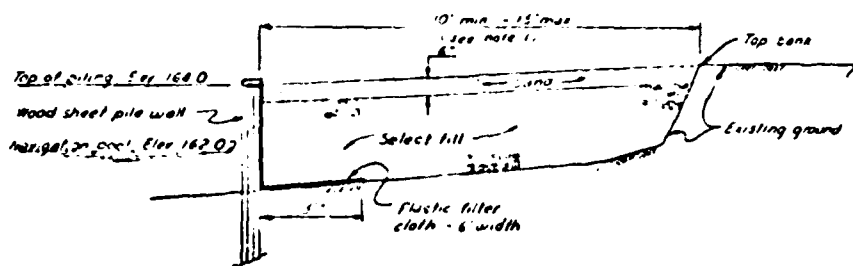
H-49-3

ITEM 49-2

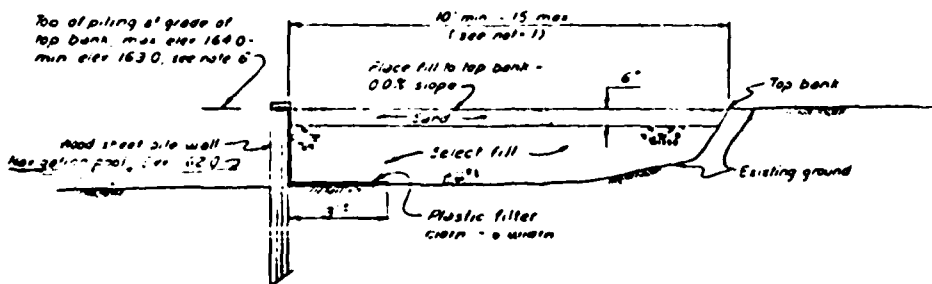
H-49-4



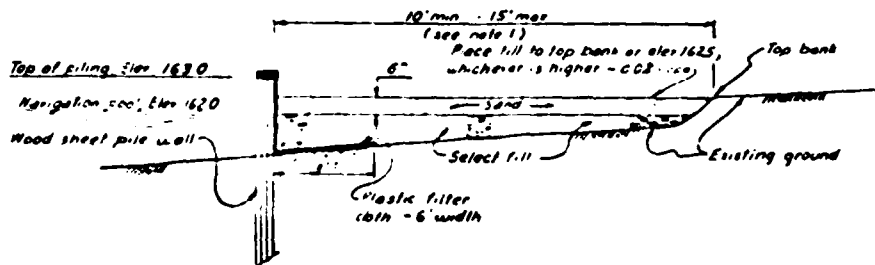
MERRISACH LAKE PUBLIC USE AREA, ARKANSAS POST
NAVIGATION CANAL, ARKANSAS COUNTY, ARKANSAS



CONDITION WHERE TOP BANK IS ABOVE ELEVATION 164.0



CONDITION WHERE TOP BANK IS AT OR BETWEEN ELEVATIONS 164.0 & 163.0



CONDITION WHERE TOP BANK IS BELOW ELEVATION 163.0

TYPICAL SECTIONS

SCALE 1"=8'-0"



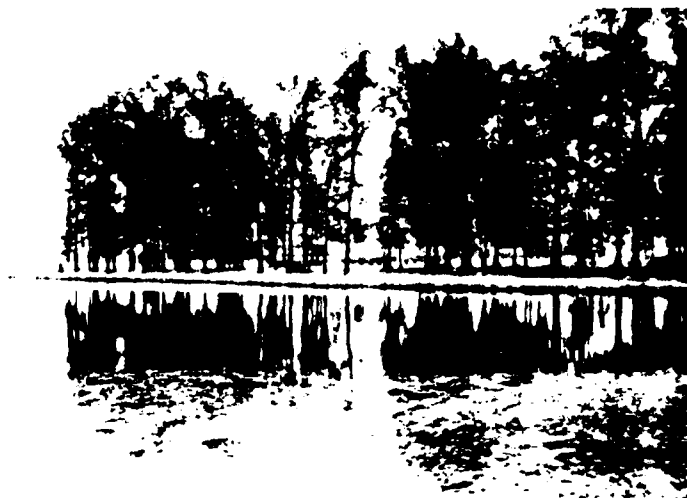
MERRISACH LAKE PUBLIC USE AREA, ARKANSAS POST
NAVIGATION CANAL, ARKANSAS COUNTY, ARKANSAS



Arkansas
Post Canal



Aerial view of Lake Merrisach on Arkansas Post Canal



Typical timber sheet-pile walls eight years after construction

LAKE MERRISACH, ARKANSAS
1976 PHOTOGRAPHS

ITEM 49-5

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Timber sheet-pile wall boat dock eight years after construction



Example of ten percent
of wall needed repair
after eight years

LAKE MERRISACH, ARKANSAS
1980 PHOTOGRAPHS

ITEM 49-6

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Kerr-McClellan Arkansas River Navigation Project
Arkansas Post Canal

Merrisach Lake Public Use Area
BANK PROTECTION MEASURES

ITEM 49-7

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Wall V
Loss of some embankment fill behind wooden wall

Kerr-McClellan Arkansas River Navigation Project
Arkansas Post Canal

Merrisach Lake Public Use Area

BANK PROTECTION MEASURES
17 April 1979

ITEM 49-8

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**ARKANSAS RIVER
ELLINWOOD, KANSAS**

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Arkansas River River Mile 853-860 Side Right
Local Vicinity Central Kansas Lat N38°20' Long W98°32' to 34'
At/Nr City Ellinwood (4 sites) County Barton State KS Cong Dist _____
CE Office Symbol SWT Responsible Agency Corps of Engineers
Site Map Sources Corps of Engineers, Tulsa District and USGS
Land Use Information Homes and farming

(2) Hydrology at or Near Site

Stage Range _____ to _____ ft; Period of Record 1940 to 1981.
Discharge Range 0 to 27,800 cfs; Velocity Range _____ to _____ fps
Sediment Range _____ to _____ tpd; Period of Record 19____ to 19____.
Bank-full Stage _____ ft; Flow _____ cfs; Average Recurrence Interval _____ yr
Bank-full Flow Velocity: Average _____ fps; Near Bank _____ fps
Comments _____

(3) Geology and Soil Properties

Bank (USCS) Fine sand, silt Bed (USCS) Sand, silt
Data Sources Visual observation
Groundwater Bank Seepage _____
Overbank Drainage _____
Comments _____

(4) Construction of Protection

Need for Protection To prevent damage to farmland
Erosion Causative Agents High flows and channel alignment
Protection Techniques Steel jacks
General Design 36 to 72 main-line jacks connected by two 3/4" cables and anchored by retard cables and retard jacks at sites 1-4.
Project Length See 50-1ft; Construction Cost \$ See 50-1 Mo/Yr Completed 9/74

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 9.2 ft., 2740 cfs, 9/24/76; 12.8 ft.,
4910 cfs, 7/29/79; 17.9 ft., 13,600 cfs, 6/15/81

Repairs and Costs (Item, Cost, Date) None

Comments: Flows measured at Great Bend, KS, gage

(6) Performance Observations and Summary

Monitoring Program Visual inspections with photographs June 1977, June 1978,
April 1980, August 1981

Documentation Sources Corps of Engineers

Project Effect on Stream Regime Smoothed the alignment

Project Effect on Environment Reduced siltation of stream; allowed revegetation
of streambank

Successful Aspects Controlled erosion and reestablished vegetation

Unsuccessful Aspects _____

General Evaluation The project is functioning as planned

Recommendations _____

(7) Additional Information, Comments, and Summary

Map No. 50

Attached Items:

50-1 Project Summary

50-6 Site 2 Plan

50-2 Project Location

50-7 Site 3 Plan

50-3 Typical Section

50-8 Site 4 Plan

50-4 Construction Details

50-9-12 Site 1 Photographs

50-5 Site 1 Plan

50-13-16 Site 3 Photographs

ARKANSAS RIVER, ELLINWOOD, KANSAS

The project consists of Kellner Jack (jetty) Fields at four sites located along the right bank of the Arkansas River as shown in Item 50-2. A plan view of the jack layout at each site is shown in Item 50-3. Individual jack construction and cable anchoring details are shown in Item 50-4. Plan view of the four sites are shown in Items 50-(5-8). Special efforts were made in the design to minimize the number of jacks used and to keep costs as low as possible. The project construction was completed in September 1974. The construction costs based on \$225 per jack in place are summarized in the following chart:

<u>Site No.</u>	<u>Structure Length</u>	<u>Jacks Used</u>	<u>Cost Per Foot</u>	<u>Total Cost</u>
1	450 ft	37	\$ 18.50	\$ 8,330
2	900 ft	111	27.70	24,970
3	650 ft	70	24.20	15,750
4	550 ft	53	21.70	11,930
TOTAL	2,550 ft	(Average	\$ 23.90)	\$60,980

From 1940 to 1975, maximum, mean, and minimum Arkansas River discharges at Great Bend, Kansas (about 12 miles upstream from the site), have been 27,800, 373, and 0 cfs, respectively; since project completion, 1,370, 54, and 3 cfs, respectively. Consequently, the bank protection measures have not been subject to significant flows whereby their performance could be evaluated.

Photographic coverage of physical changes to the bank line and protection system is provided in Items 50-(9-11) Site 1 and 50-(12-14) Site 3. Water level was above normal due to rain on previous day. (All four sites.)

General Comments: Site No. 1

Jack revetment is intact (originally installed on an earthen berm); most of the berm has eroded away. Water at toe of jack units up to 3-foot depth varying to 0 depth on left bank. All jacks, anchor cables, wires intact. Small cottonwood and willows are becoming established between the jack revetment and the levee.

Levee is covered with clover and cheat. Levee material is sandy loam, fine-grained, buff color. Car bodies existing along preproject bank are located between the jack revetment and bank.

General Comments: Site No. 2

Jack system is intact. Upstream portion originally installed on an earthen berm. Lower portion (downstream of scour area where retard lines are located) was assembled on bank and pushed off into river using a front end loader. This was bad practice and resulted in some wires being broken and poor spacing of the jack units. A low sand bar is forming 30 feet out of jack line at lower end. Lower half of jack revetment is in water. This applies from retard lines and extends up to 80-feet riverward of the jack revetment. Foreshore area is covered with young

ITEM 50-1
(Sheet 1 of 2)

trees ranging from very small up to 10 feet tall.

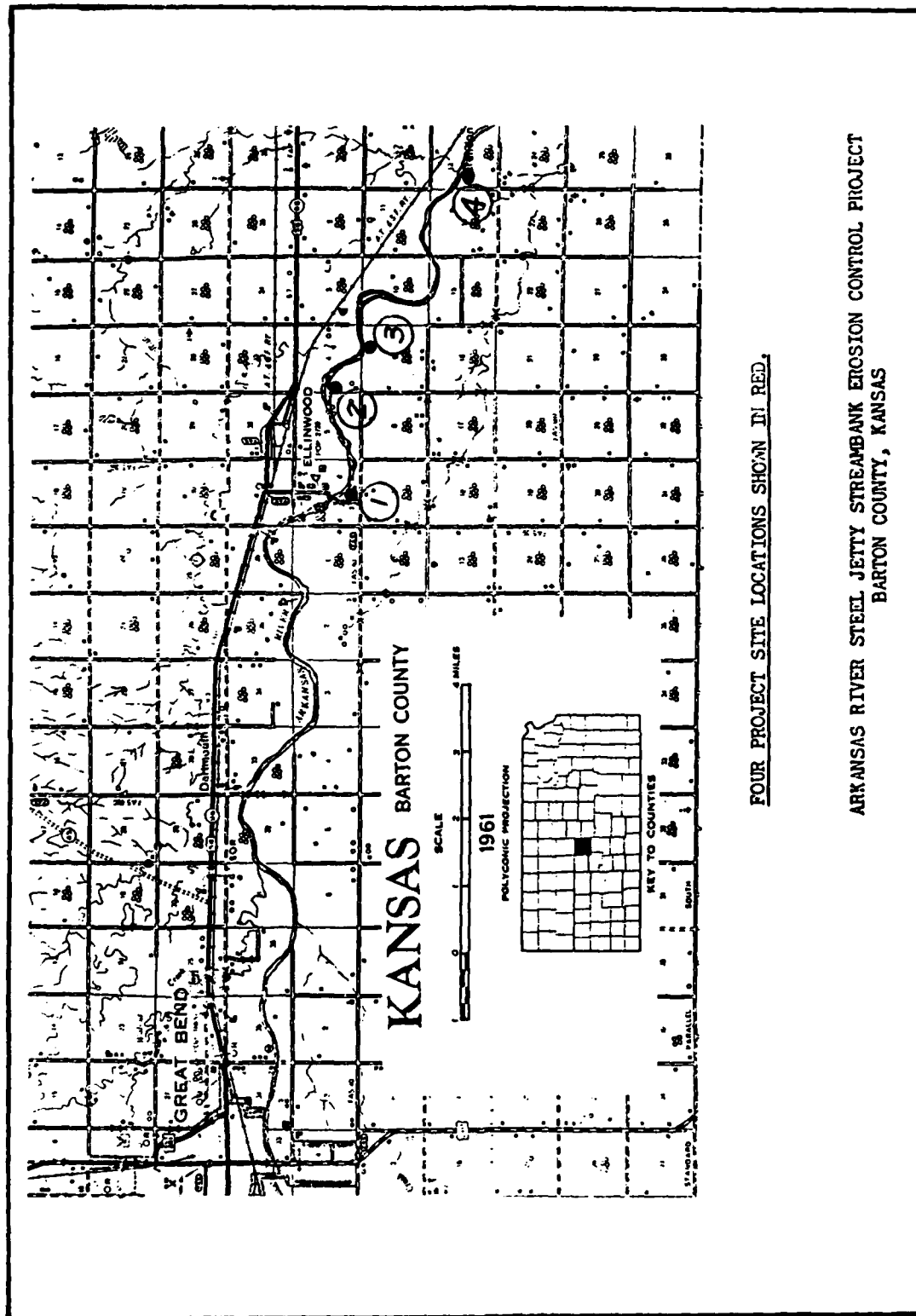
All jack units, cables, and wires in same condition as originally installed. Levee has good grass cover, mostly cheat and ragweed. High water has occurred since construction and an average of about 6 inches of silt has deposited in the dike field.

General Comments: Site No. 3

All anchor cables, jack units and wire are in good condition. Retard field was filled in above water level before installation of retards at owner's request; there is no evidence that high waters have exceeded this level. Jacks were originally installed on berm but jacks were in water at time of the 1978 inspection. Levee covered with grass; mostly cheat and Johnson grass. Bar has formed an island at upstream end of jack revetment and is covered with cottonwood and willow. Bar is 2 feet above water surface at highest point. Mainflow is against jacks at upper end and deepest at jack toe. There is also a small bar forming at lower end of jack revetment.

General Comments: Site No. 4

Jack revetment is in good condition. Alignment in reach is excellent and a good foreshore has developed riverward of the jack revetment covered with willow and cottonwood to 12 feet high. No attack on jacks; no deep water, cross section is uniform with no deep areas from bank to bank. Water velocities are also uniform across the section. Levee has good grass cover of brome, cheat and clover. No sandbars or evidence of channel attempting to form braids.



FOUR PROJECT SITE LOCATIONS SHOWN IN RED.

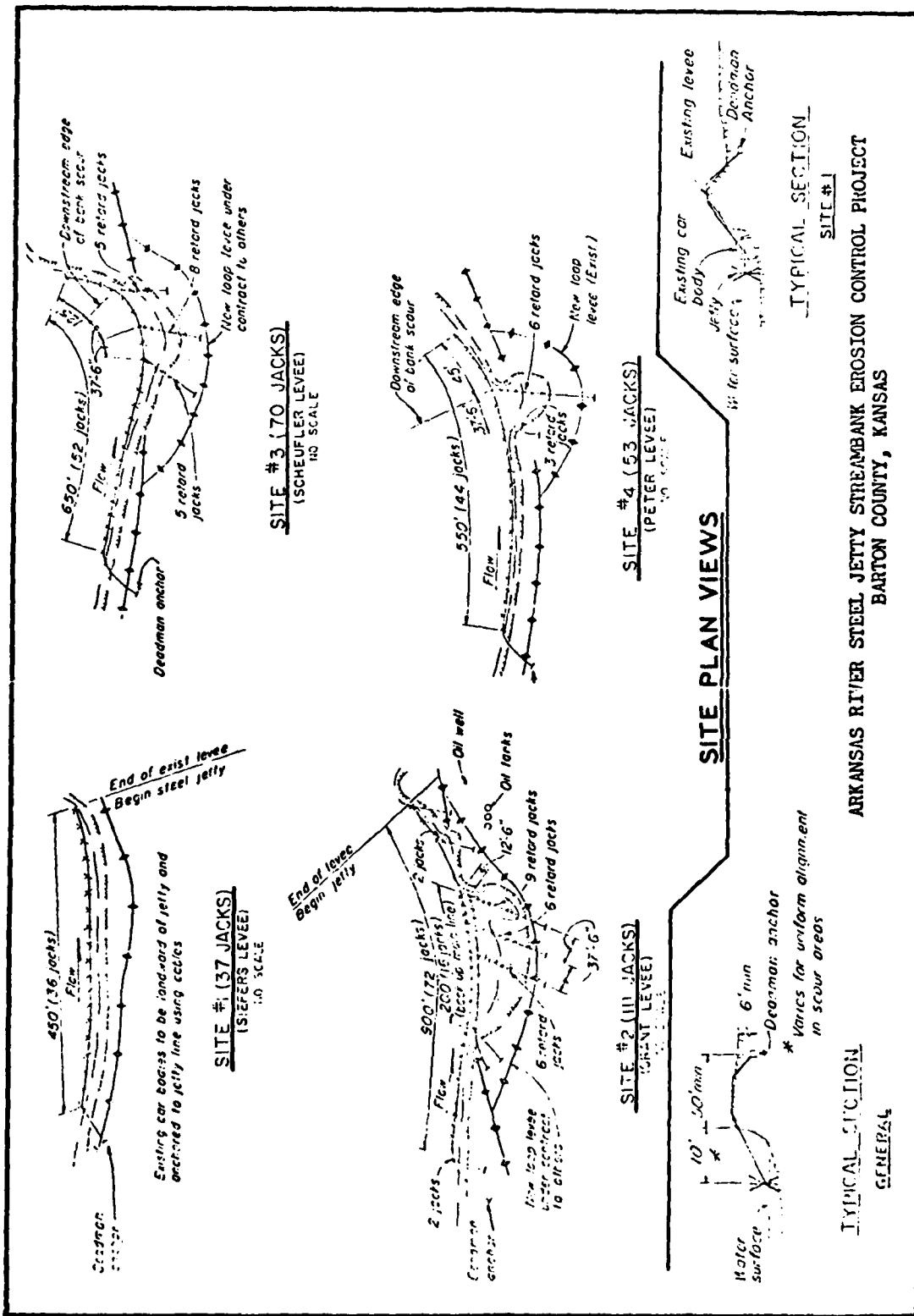
ARKANSAS RIVER STEEL JETTY STREAMBANK EROSION CONTROL PROJECT
BARTON COUNTY, KANSAS

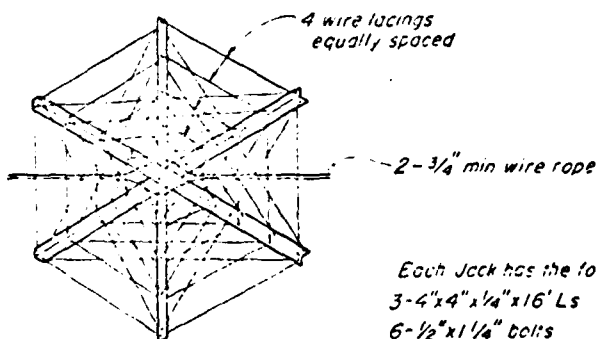
ITEM 50-2

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ITEM 50-3

H-50-6





Note: Minor deviation at location
of lacing holes will be permitted
if approved by the contracting officer

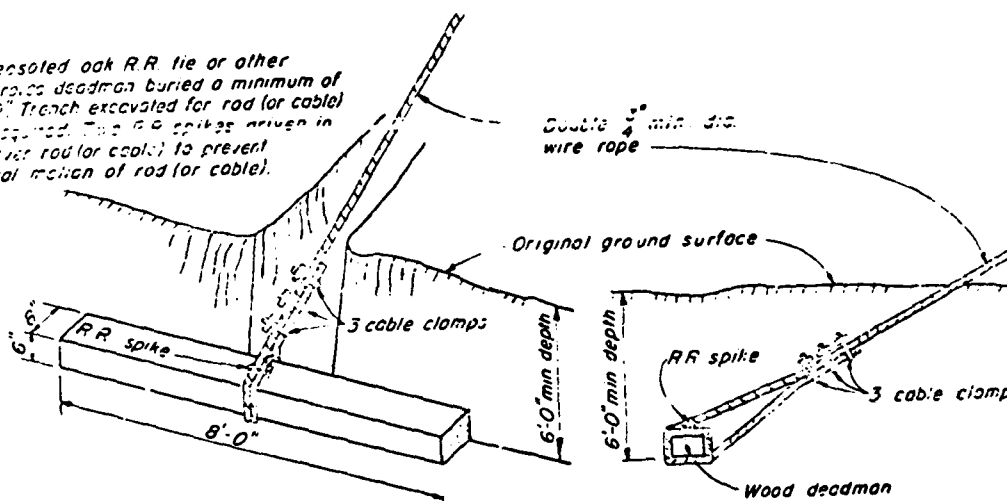
JACK DETAIL

NO SCALE

Each Jack has the following:
3-4"x4"x1/4"x16' Ls
6-1/2"x1 1/4" bolts
2-3/4" minimum wire ropes
2- wire rope clips
All lacing wires are No 6
Min of 4 wraps at lacing wire ties

NOTE

Seasoned oak R.R. tie or other
approved deadman buried a minimum of
6'-0". Trench excavated for rod (or cable)
as required. Two R.R. spikes driven in
tie over rod (or cable) to prevent
lateral motion of rod (or cable).



TYPICAL DEADMAN

1:0 SCALE

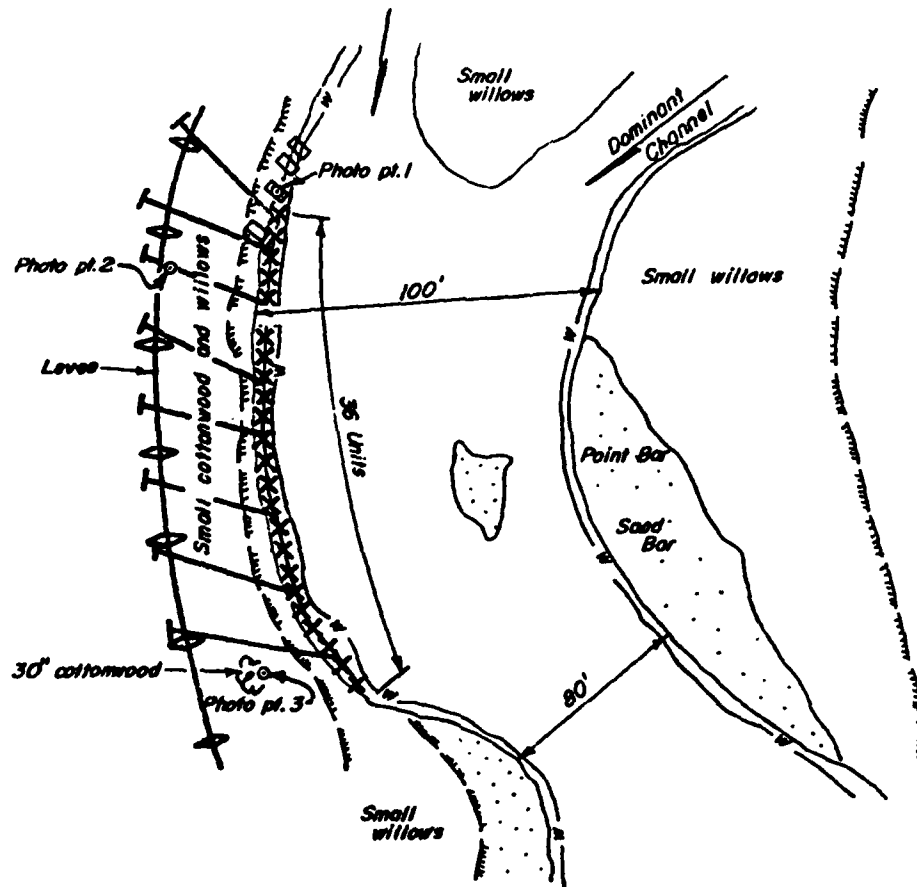
CONSTRUCTION DETAILS

ARKANSAS RIVER STEEL JETTY STREAMBANK EROSION CONTROL PROJECT
BARTON COUNTY, KANSAS

ITEM 50-4

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SITE NO. 1

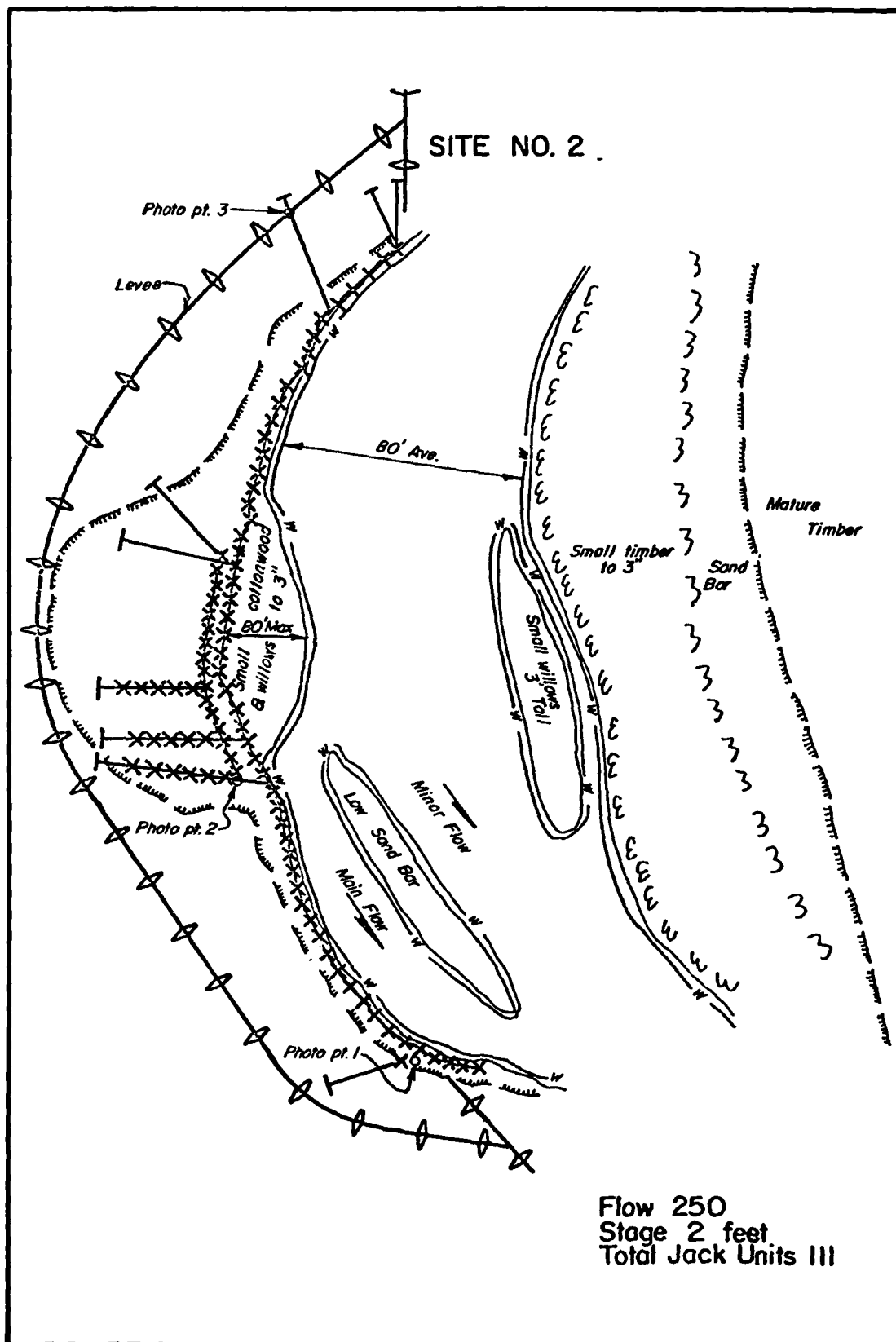


□ Existing car bodies

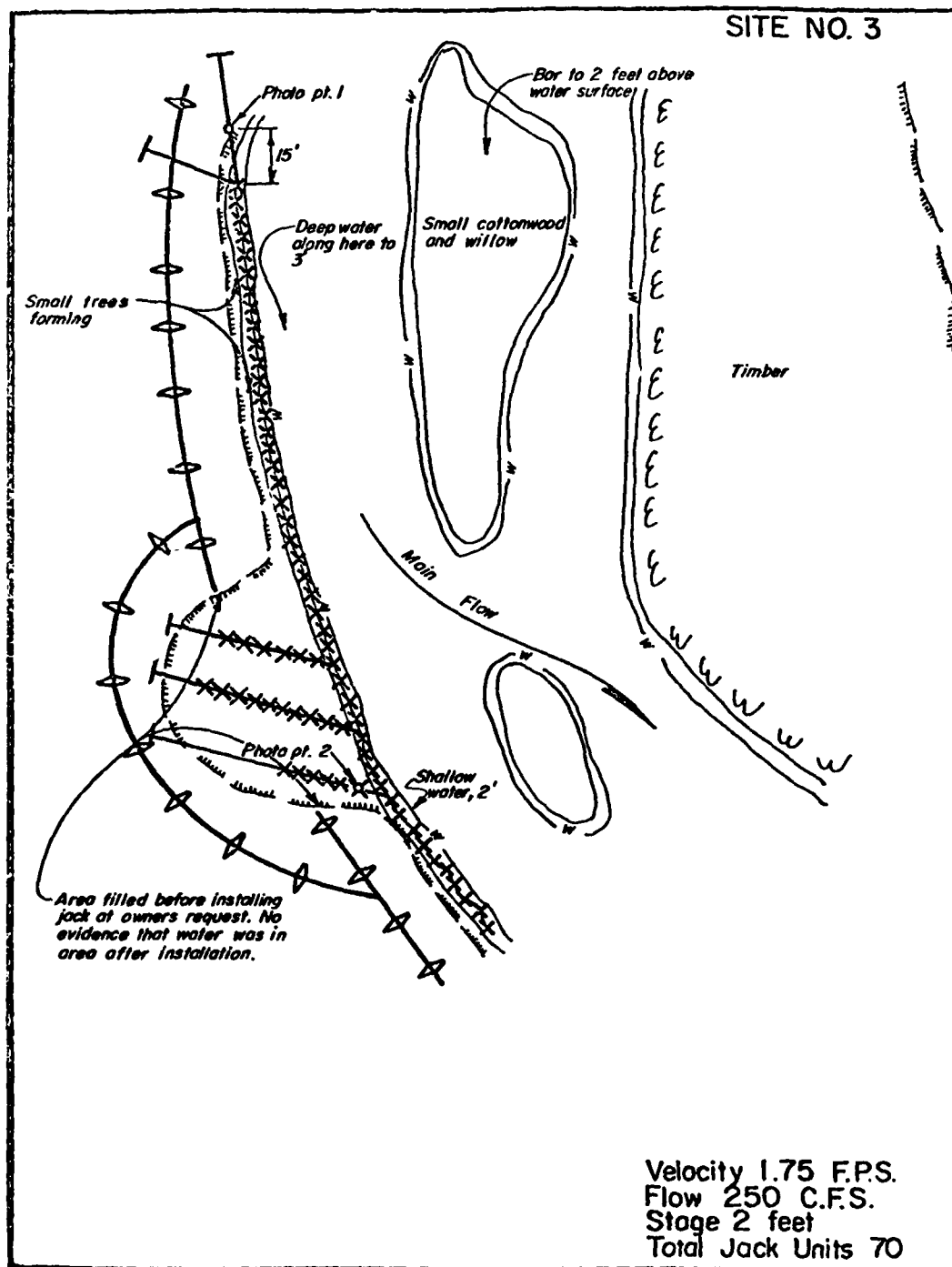
Stage 2 feet
Flow 250 C.F.S.
Velocity = 2 ft./sec.
Total Jack Units 36

ITEM 50-5

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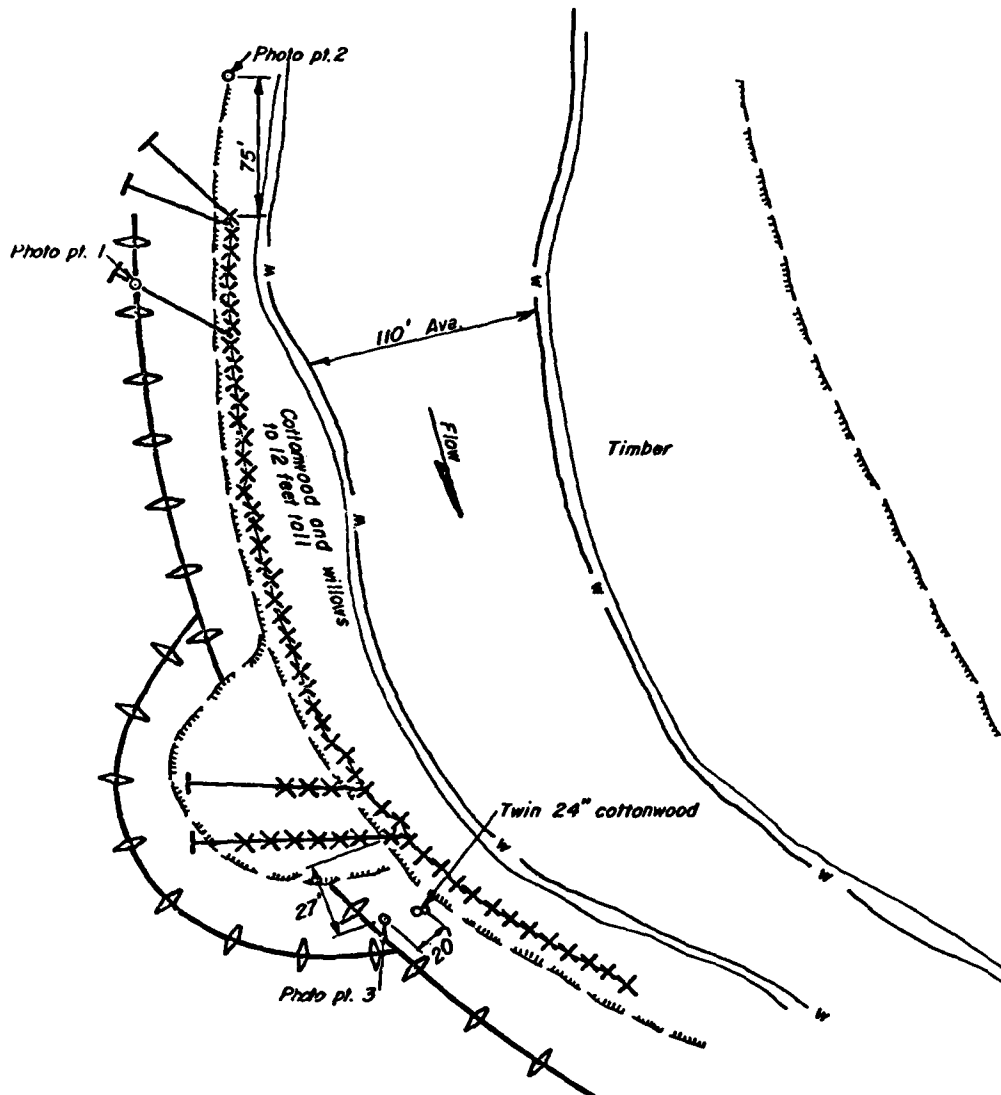
ITEM 50-6



ITEM 50-7

H-50-10

SITE NO. 4



Velocity 1.7 F.P.S.
Flow 250 C.F.S.
Stage 2 feet
Total Jack Units 53

ITEM 50-8

H-50-11



Looking downstream, photo point 1



Looking downstream, photo point 2



Looking upstream, photo point 3

SITE 1, 1977

ITEM 50-9

H-50-12



View downstream, photo point 1



View downstream, photo point 1



View downstream, photo point 2



View downstream, photo point 3

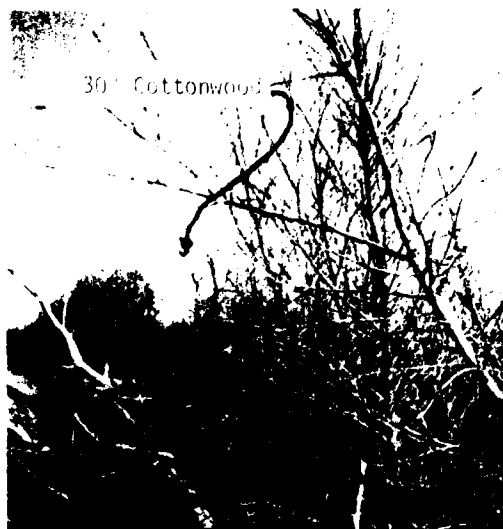
SITE 1, 1978

ITEM 50-10

H-50-13



View downstream, photo point 1



View downstream, photo point 1



View downstream, photo point 2



View upstream, photo point 3

SITE 1, 1980

ITEM 50-11

H-50-14



Looking downstream, photo point 1

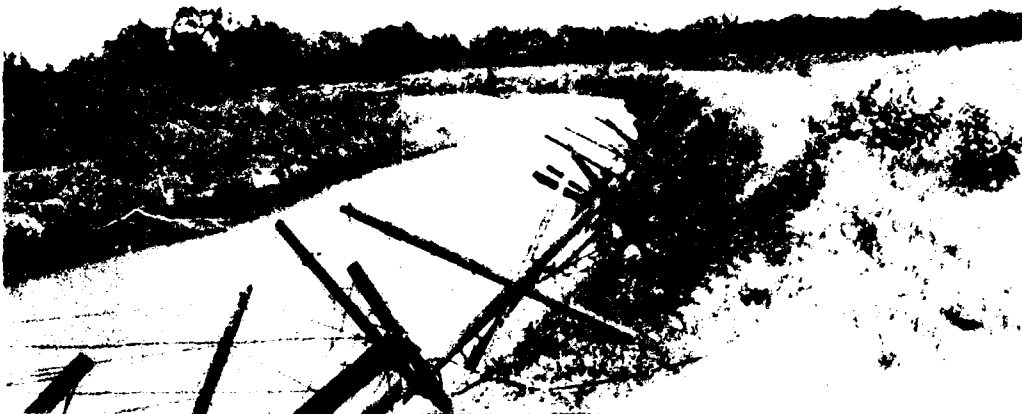


Looking downstream, 35 ft left of photo point 1

SITE 1, 1981

ITEM 50-12

H-50-15



Looking downstream, photo point 1



Looking upstream, photo point 2

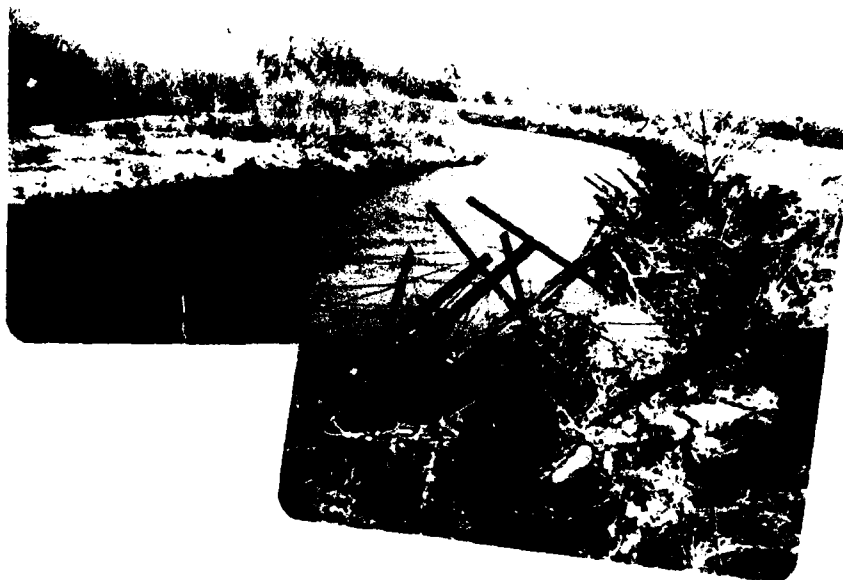


Looking downstream, photo point 2

SITE 3, 1977

ITEM 50-13

H-50-16



View downstream, photo point 1



View upstream, photo point 2



View downstream, photo point 2

SITE 3, 1978

ITEM 50-14

H-50-17



View downstream, photo point 1



View upstream, photo point 2



View downstream, photo point 2

SITE 3, 1980

ITEM 50-15

H-50-18



Looking downstream, photo point 1



Looking upstream, photo point 2

SITE 3, 1981

ITEM 50-16

H-50-19

Unclassified

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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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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.

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