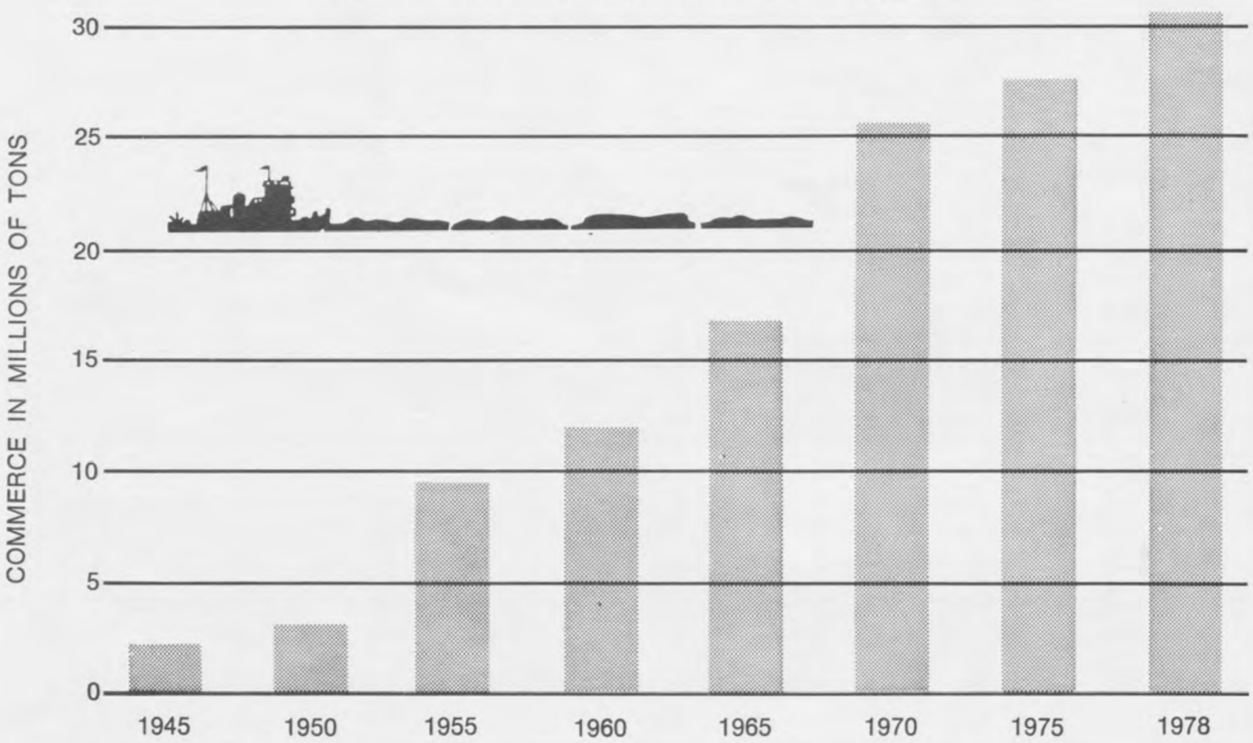
Water Resources Development in Alabama 1981

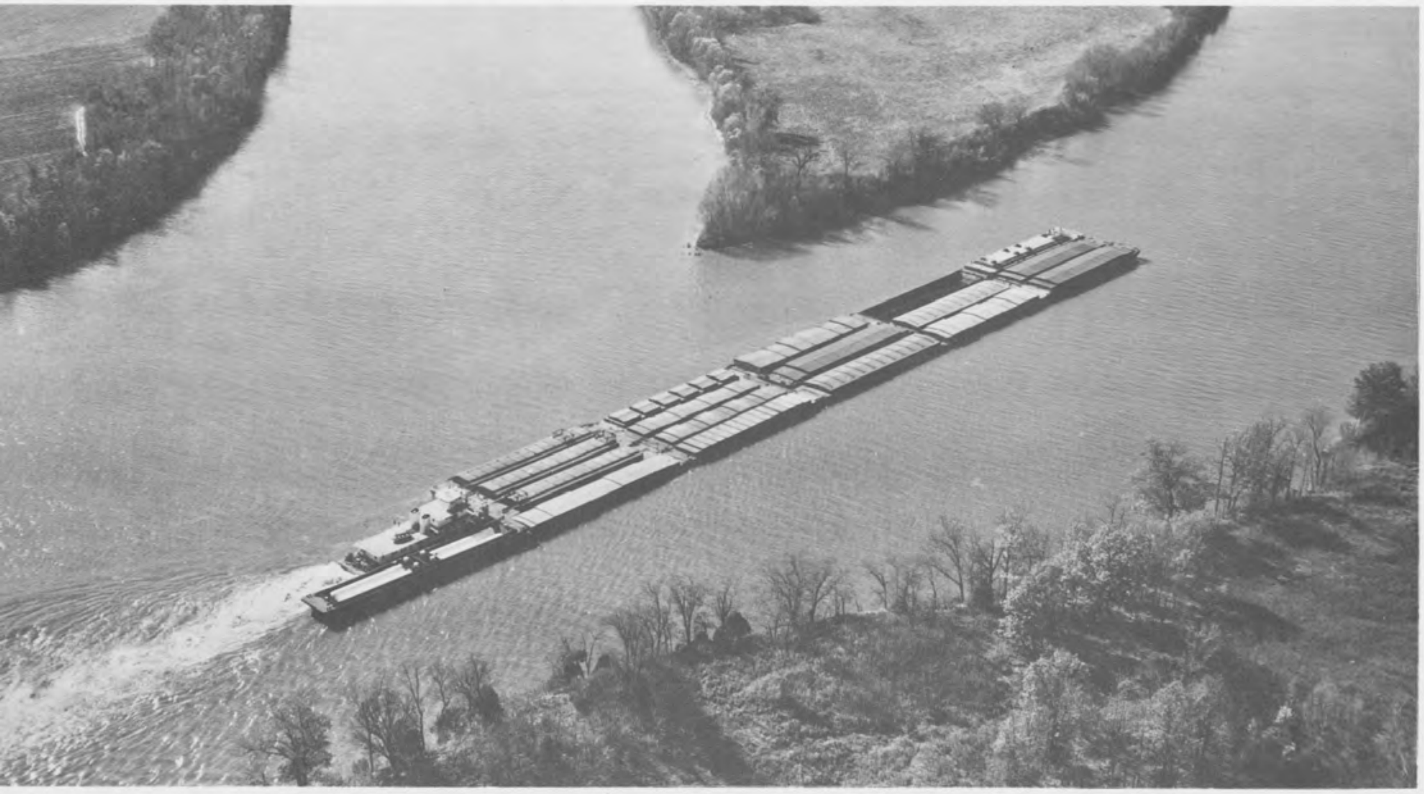
by the US Army Corps of Engineers

Army Corps
Engineers

South Atlantic Division

TRAFFIC ON THE TENNESSEE RIVER





Typical mixed tow on the Tennessee River

Water Resources Development In Alaska 1981

by the U.S. Army Corps of Engineers

Projects Under Way — Navigation

CORDOVA HARBOR

This project is an expansion of the existing harbor at Cordova, a city located in southcentral Alaska on the southeast shore of Orca Inlet near the eastern entrance to Prince William Sound.

Authorized for construction in 1980 under Section 107 of the 1960 River and Harbor Act, as amended, this project consists of a main west breakwater 1,150 feet in length, a northwestern breakwater 752 feet in length, and a south silt-barrier breakwater 650 feet in length. This will inclose an additional 20-acre area on the west side of the existing harbor and provide mooring space for 602 additional vessels.

Federal costs are $1,710,000 for construction and $3,400 for annual maintenance.

CRAIG HARBOR

Craig is a community of approximately 600 people in southeast Alaska on the west side of Prince of Wales Island 60 air miles west of Ketchikan.

Construction of two rubble mound breakwaters to protect the expanded float system in Craig's South Cove Harbor was authorized in 1979 under Section 107 of the 1960 Rivers and Harbor Act, as amended. The north breakwater is 160 feet in length and the south breakwater is 300 feet in length. Federal costs are $651,000 for construction and $2,000 for annual maintenance.

HOONAH HARBOR

Site of this project is the location of the Huna Indians, a Tlinglit tribe in southeastern Alaska. The communtiy is on the northeastern tip of Chichagof Island about 40 miles from Juneau.

The work here, authorized in 1972 under Section 201 of Public Law 89-298, is to produce a small boat mooring basin of 15.5 acres. It will have three armored breakwaters with an aggregate length of 2,400 feet, plus two diversion dike totaling 2,010 feet. A Federal entrance channel 150 feet wide and 16 feet deep leads into the harbor.

Federal costs are estimated at $4,000,000 for construction and $29,100 for annual maintenance. Construction started in fiscal year 1979 and will be completed in fiscal year 1981.

Project Costs

New Work	$3,953,743
Contributed Funds	1,125,004
TOTAL	5,078,747

KETCHIKAN, Bar Point Harbor

This project is an extension of the existing harbor at Ketchikan, a city on an island in the Alexander Archipelago of southeastern Alaska.

This work was authorized by the Chief of Engineers under terms of Section 107 of the River and Harbor Act of 1960 as amended.

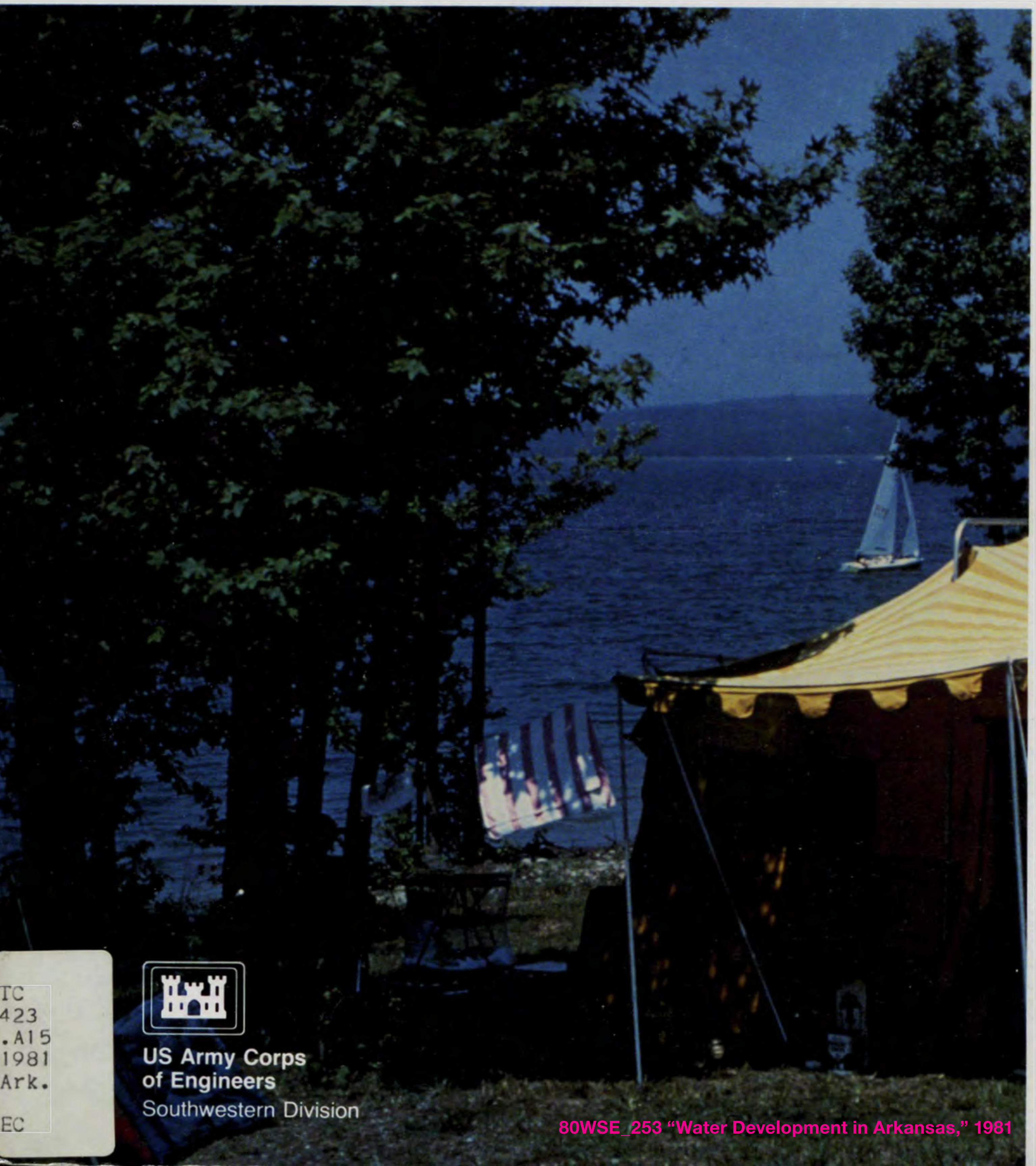
An additional 25-acre harbor area accommodating several hundred additional vessels was provided by construction of two concrete floating breakwaters, one 963 feet long, the other 120 feet.

Federal costs were $2,000,000 for construction which was completed in fiscal year 1980.

Jet probing at Craig Harbor in 1939

Water Resources Development in Arkansas 1981

by the U S Army Corps of Engineers



US Army Corps of Engineers
Southwestern Division

DeGray Lake Visitors Center

Blakely Mountain Dam and Lake Ouachita

Water Resources Development in Colorado 1981

by the US Army Corps of Engineers

US Army Corps
of Engineers
Missouri River Division

MULTIPLE-PURPOSE PROJECTS UNDERWAY

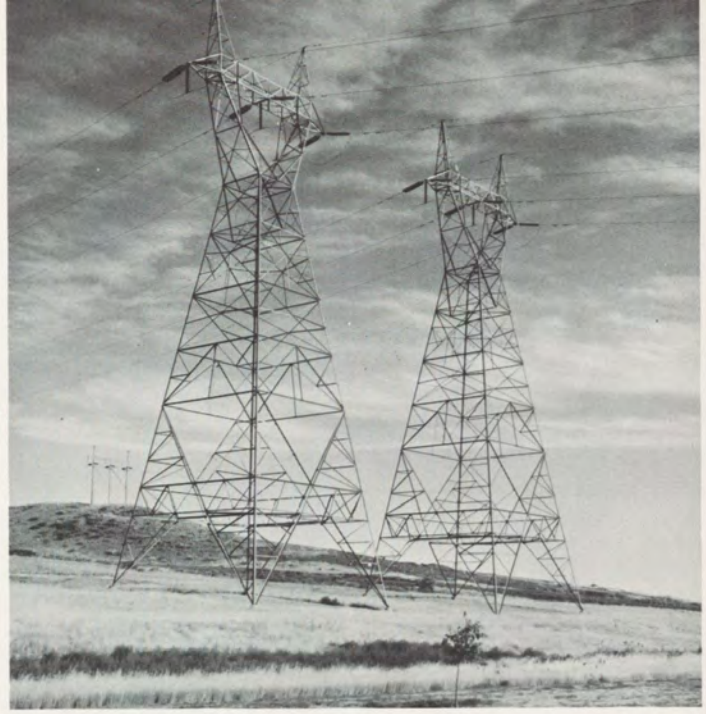
COMPREHENSIVE PLAN, MISSOURI RIVER BASIN

(Missouri River Division)

The 1944 Flood Control Act gave birth to the Nation's first attempt at solving its water resource problems through a comprehensive approach to river basin development. The legislation, which has come to be known as the Pick-Sloan Plan, was based upon separate proposals recommended by the Corps of Engineers and the Bureau of Reclamation (now the Water and Power Resources Service).

Designed primarily to provide four basic benefits — flood control, irrigation, generation of hydroelectric power, and improvements of navigation on the lower Missouri River — the measure has since derived benefits from improving municipal and industrial water supplies, providing land treatment and enhancement and water quality control, conservation of fish and wildlife, and the development of public recreation.

The plan originally provided for building some 103 dams and reservoirs to provide storage capacity of approximately 110 million acre-feet for multiple-purpose use; local levees and floodwalls to protect municipal, industrial, and agricultural areas; and a system of levees on both sides of the Missouri River between Sioux City, Iowa, and the mouth to protect hundreds of thousands of bottom land acres against flooding. Planned uses of stored water include the irrigation of some 4 million acres of land and



the generation, ultimately, of 13 billion kilowatt-hours of hydroelectric power annually for industrial and municipal expansion, for pumping irrigation water, and for other domestic uses. In addition, the regulation of riverflows provided by planned reservoirs is essential to navigation on the Missouri River.

New projects have been and are being authorized by Congress as their need becomes apparent and are added to the overall development. In the project formulation

Water Resources Development in Connecticut 1981

by the U S Army Corps of Engineers

US Army Corps
of Engineers
New England Division

REGIONAL STUDIES

NATIONAL STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION PROGRAM

Streambank erosion, which is a result of the regional topography, surface geology and social and cultural activities, is extensive in many sections of New England, especially upland areas. The flow direction and velocities of regional streams and tributaries are determined largely by the topography, although the vegetative cover substantially modifies the effects of topography and geology on streamflows.

Before . . . and After

Section 32 of the Water Resources Development Act of 1974 (P.L. 93-251) authorizes the Secretary of the Army, acting through the Chief of Engineers, to establish and conduct a national streambank erosion prevention and demonstration program. This section, referred to as the Section 32 Program, may be cited as the Streambank Erosion Control Evaluation and Demonstration Act of 1974. The act as amended authorizes appropriations to undertake the program not to exceed $50 million through 1981.

The program consists of:

- An evaluation of the extent of streambank erosion on navigable rivers and their tributaries.
- Development of new methods and techniques for bank protection, research on soil stability and identification of the causes of erosion.
- Report to the Congress on the results of such studies and the recommendations of the Secretary of the Army on means for the prevention and correction of streambank erosion.
- Demonstration projects, including bank protection works.

The assessment of existing New England streambank conditions, which updates an earlier assessment made in 1969, was accomplished in FY 1977. This was part of a national assessment by the Corps of Engineers and the Soil Conservation Service.

A demonstration project was completed in 1979 to protect a 2,600 foot reach of bank along the Connecticut River in Haverhill, New Hampshire. The project consists of several test sections that protect the toe of the bank with rock filled gabion mattresses, rubber tires, sand cement bags, and baled hay. The project also consists of over 20 different varieties of vegetative protection on the upper bank. A similar project is under construction at Northfield, Massachusetts. A third tentative project at Newbury, Vermont is now being planned for 1981 construction. The purpose of these projects is to experiment with a variety of structural and vegetative techniques for streambank protection and to demonstrate measures suitable for local implementation.

Water Resources Development in District of Columbia 1981

US Army Corps of Engineers
North Atlantic Division

The Potomac River Basin

The Potomac River Basin has a drainage area of 14,670 square miles and is located in parts of the States of Virginia, Maryland, West Virginia, Pennsylvania, and the entire District of Columbia. The main stream of the Basin, the Potomac River, originates in the rugged Allegheny mountains, moves eastward, while collecting flow from tributaries in its western watershed, through the Shenandoah Valley, the Piedmont Plateau, and the coastal plains east of Washington, to the Chesapeake Bay.

Floods in the Potomac River Basin can occur in all seasons of the year. Most of the major floods, however, have occurred between February and April, when snowmelt was accompanied by heavy spring rains, or in the summer and fall months, as a result of hurricane activity. Major floods have occurred in various portions of the basin over the years, the most damaging of which, the flood of March 1936, caused losses in the basin of $12.6 million. At today's prices and with the additional development in the basin, this figure would be much greater if an event comparable to the March 1936 flood recurred.

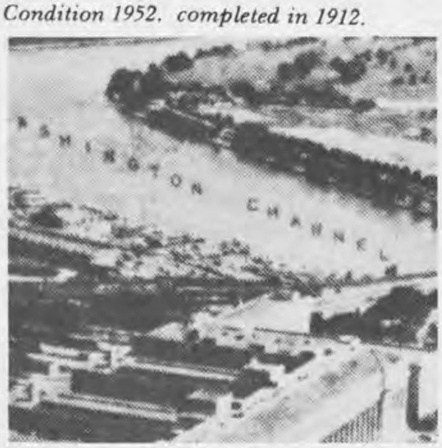
Two flood-protection projects have been authorized for the Washington, D.C., Metropolitan area, both of which have been completed; the Anacostia River project and the Washington, D.C., and vicinity project.

The Potomac River, at and below Washington, D.C., is used for navigation, and there are several navigation projects that have been authorized in that area. Three projects, the Potomac River below Washington, D.C., Washington Harbor, and Anacostia River, D.C., and Md., have been completed. A fourth project, the Potomac River, north side of Washington Channel, is in an inactive status.

Other miscellaneous projects in the Washington, D.C. Area include land reclamation in Anacostia Park, elimination of water chestnuts and removal of drift in the Potomac River, and the responsibility for water supply for the area.

Condition 1883. initiated in 1882.

Condition 1952. completed in 1912.



Washington Harbor Reclamation of Potomac River Flats (Potomac Park)

Descriptions of the projects in the basin in Washington, D.C., and their status, follow:

Project Completed Navigation—Channels and Harbors

Potomac River Below Washington

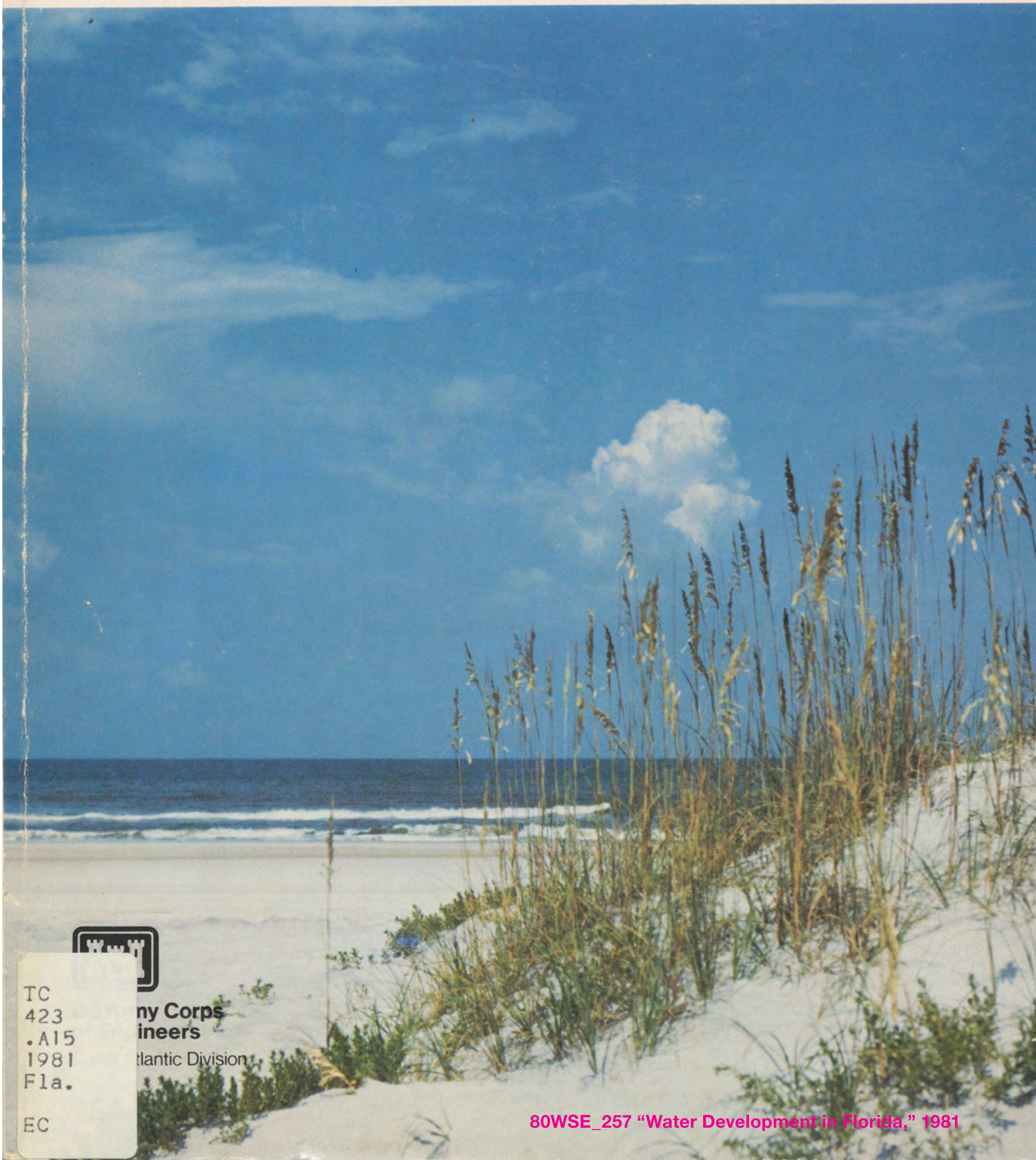
The project provides for a channel 24 feet deep and 200 feet wide from the Chesapeake Bay to Giesboro Point at Washington, D.C., a distance of 108 miles, of which the upper 4 miles are in the District of Columbia. The length of dredged channels along the route totals 17 miles. The project was completed in 1905 at a cost of $153,836. Waterborne commerce on the waterway consists of agricultural products, seafood, petroleum products, sand and gravel, sulphur, lumber, fertilizers, and manufactured products. Average annual traffic for the past ten years was 6.1 million tons, and 343,778 passengers. The traffic in 1976 was 6,037,072 tons and 343,173 passengers.

Washington Harbor

The project comprises the dredging of channels 400 feet wide and 24 feet deep at mean low water in: (1) the Virginia Channel of the Potomac River between Giesboro Point and Key Bridge, the width increased as necessary to provide a low-water cross-section area of 25,000 square feet; (2) the Washington Channel from Hains Point to the head of Washington channel at the foot of 14th Street, SW.; (3) the Anacostia River from Giesboro Point to Anacostia Bridge with a turning basin 800 feet wide and 2,400 feet long opposite the Naval Weapons Plant. Upstream from Anacostia Bridge the authorized channel dimensions are 200 feet wide and 24 feet deep at mean low water, with a turning basin 400 feet square at the head of the project at the foot of 15th Street, SE. While not fully completed in accordance with project plans, the project is listed as completed because existing chan-

Water Resources Development in Florida 1981

by the U.S. Army Corps of Engineers

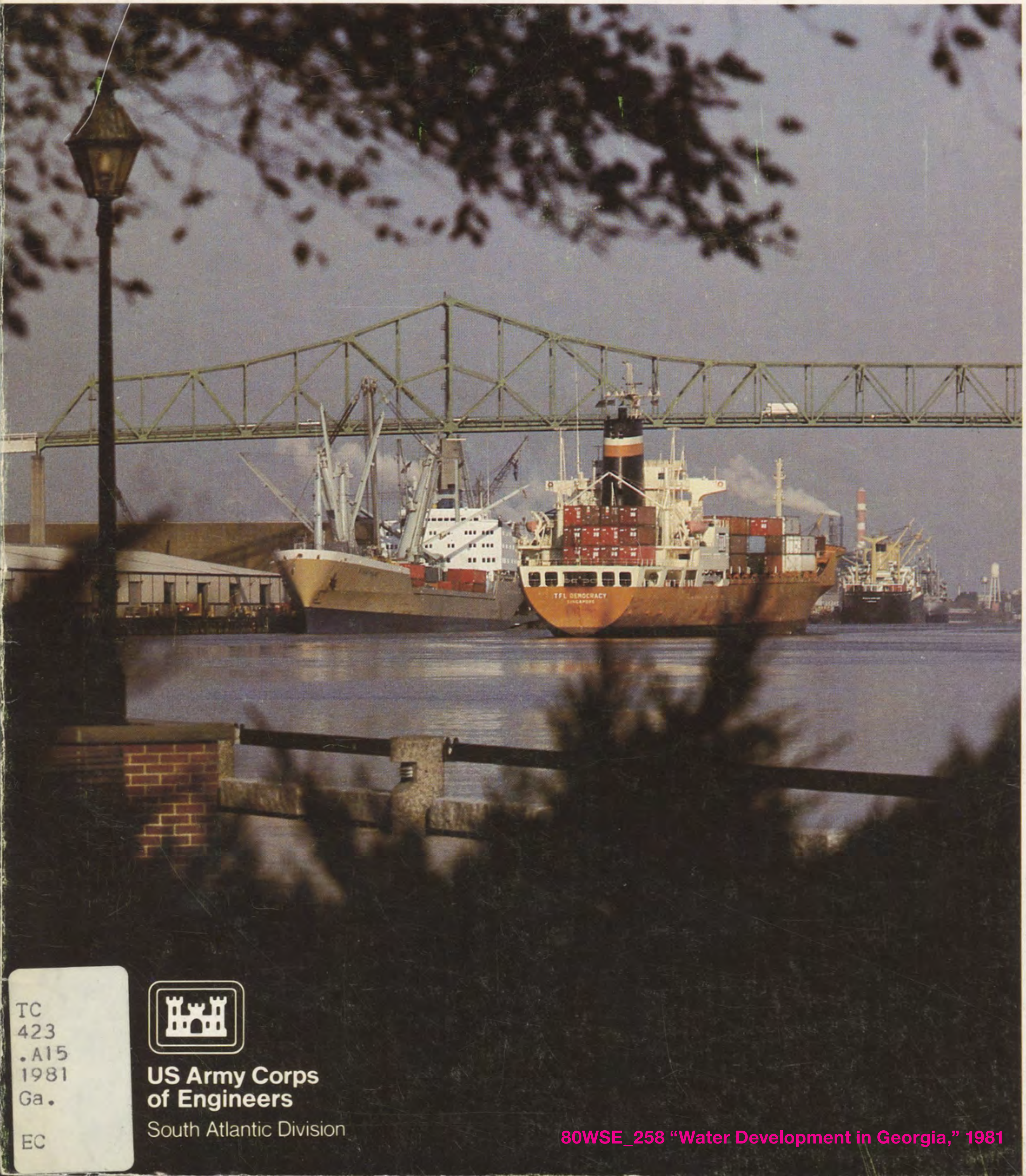


Eroded shoreline at Atlantic Beach (Duval County) before restoration.

Atlantic Beach after restoration.

Water Resources Development in Georgia 1981

by the US Army Corps of Engineers



US Army Corps of Engineers
South Atlantic Division

TYBEE ISLAND, GEORGIA
Underway Beach Erosion Control Project
(Savannah District)

Authorized for construction by Senate and House Resolutions in June 1971, actual construction began in November 1974. The project is designed to improve the shoreline of Tybee Island, located in Chatham County, directly south of Savannah Harbor entrance and about 17 miles east of Savannah, Georgia.

PROJECT DATA

Restoration and nourishment of approximately 13,300 linear feet (2.5 miles) of shore, beginning at the north end of Tybee Island and extending south to the vicinity of Eighteenth Street.

Estimated Project Costs (October 1980 price level):	
Federal Cost	$4,250,000
Non-Federal Cost	4,410,000
Total Project Costs	8,660,000

The improvement provides beach erosion control consisting of a groin system, beach restoration, and periodic beach nourishment. The first contract was awarded for construction of a 800-foot rock groin at the north end of Tybee Island near the Tybee Lighthouse and was completed in June 1975.

The second contract provided for placement of approximately 2.3 million cubic yards of sand fill along the entire length (13,300± feet) of ocean beach front. The sand was removed by hydraulic pipeline dredge from a massive sandbar borrow area off the south end of Tybee Island. Completed in April 1976, the contract provided approximately 124 feet of beach seaward of the seawall (accessible at average high tide), doubling the present public beach area.

The first of the scheduled renourishments, which was to take place during fiscal year 1979, was delayed due to additional studies of the causes of erosion at the beach. These studies resulted in a recommended scheme of constructing a new groin at the south end of the beach, raising the existing groin at the north end, and renourishment. The State of Georgia has agreed with this plan. Even though these studies are concluded, further Federal participation in the project is dependent on non-Federal interests constructing required parking and access facilities. Included in the project is a beach monitoring program after construction. The project also calls for 10 years of Federal participation in the renourishment program following initial nourishment. After that, additional sand nourishment will be borne by the local assurer unless reevaluation of benefits and techniques determine that Federal participation in periodic nourishment should be extended for the life of the project.

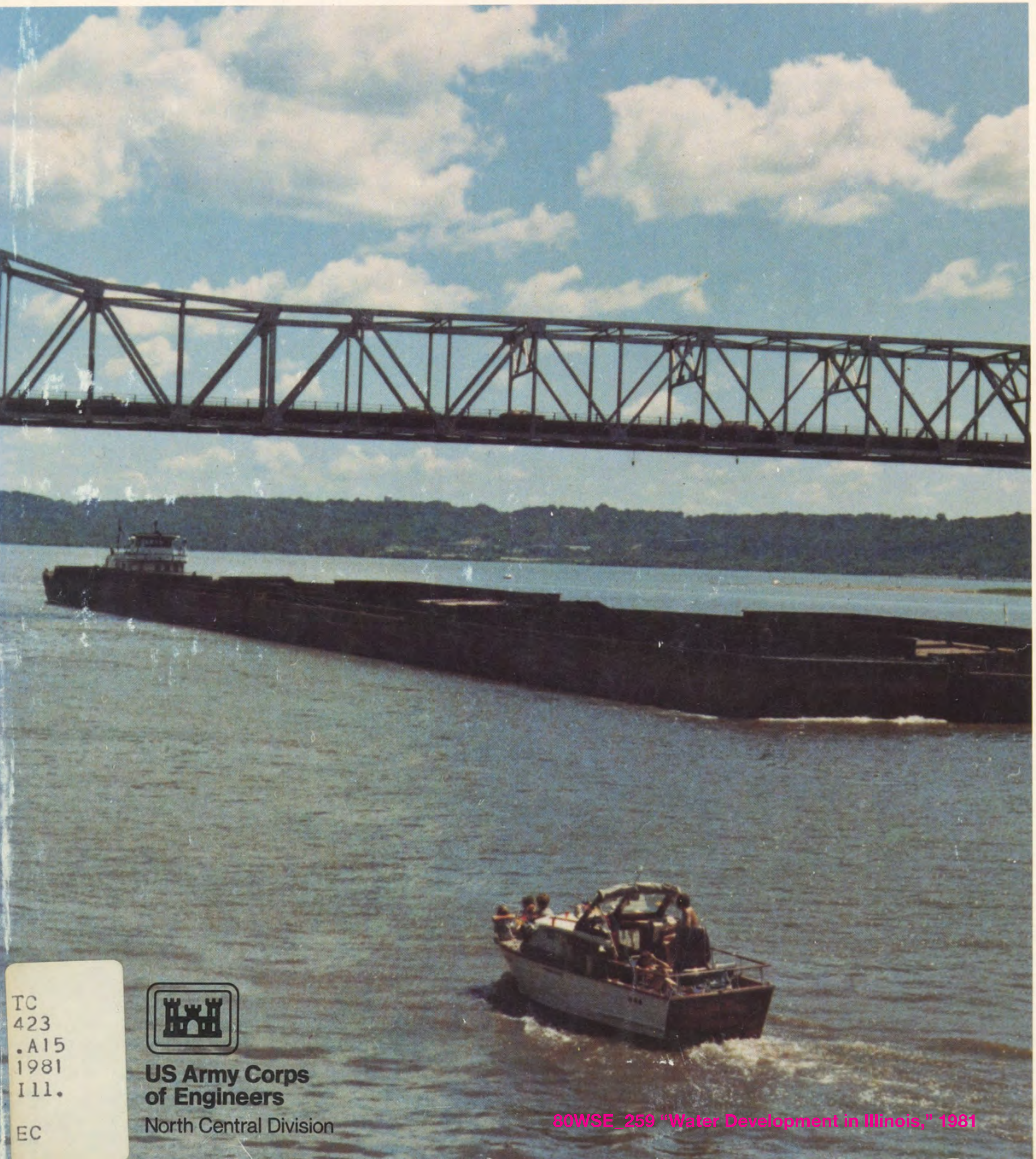
Current estimated cost of the 10-year project is $8,660,000. The local assurer, which includes the city of Tybee Island, formerly Savannah Beach, Chatham County (one time contribution), and the State of Georgia (Department of Natural Resources), pays 51 percent of the overall cost, with the Federal Government paying the remaining 49 percent. The local assurer also provides public parking and access to the beach; maintenance and repair of the groin and lands, easements, rights-of-way, and relocations required.

Cost of activities through fiscal year 1980 amounted to $4,450,000 (Federal and non-Federal funds).

Tybee Island

Water Resources Development in Illinois 1981

by the U S Army Corps of Engineers



US Army Corps of Engineers
North Central Division

Flood (1 percent): This is the same as a 100-year flood and is a flood that has a 1 percent chance of occurrence in any year.

Flood capacity: The flow carried by a stream or floodway at bank-full water level. Also, the storage capacity of the flood pool at a reservoir.

Flood crest: The highest or peak elevation of the water level during a flood in a stream.

Flood plain: Valley land along the course of a stream that is subject to inundation during periods of high water that exceeds normal bankful elevation.

Floodproofing: Techniques for preventing flood damage to the structure and contents of buildings in a flood-hazard area.

Floodwall: Wall, usually built of reinforced concrete, to confine streamflow to prevent flooding.

Freeboard: (1) Vertical distance between the normal maximum level of the surface of the liquid in a conduit, reservoir, tank, canal, etc., and the top of the sides of the conduit, reservoir, canal, etc.; (2) an allowance in protection above the design water surface level.

Gate bay walls: The gate bay walls include those portions of the lock in which the gate recesses, gate anchorages, gate machinery, and sometimes culvert valves and culvert bulkheads are located.

Gravity drainage outlet: (1) Outlets for gravity drains such as tiles, perforated conduits, etc., serving an agricultural area and discharging into a drainage ditch; (2) pipe, culvert, etc., used for dewatering ponded water by gravity.

Groin: A wall-like structure built perpendicular to the shore to trap sand and prevent beach erosion.

Guide pier: A structure that extends from the entrance to a lock, used to guide vessels safely into the lock.

Habitat: The total of the environmental conditions that affect the life of plants and animals.

Headwaters: (1) The upper reaches of a stream near its source; (2) the region where groundwaters emerge to form a surface stream; (3) the water upstream from a structure.

Ice booms: Structures installed across channels to retard the flow of ice but not that of water.

Ice floes: Free-floating sheets of ice, usually at least several inches thick, on a stream, lake, or sea.

Ice jam: Accumulation of ice packed together and piled up, choking the stream channel and causing a rise in water level above the jam.

Intercepting sewer: A conduit that receives flow from a number of transverse sewers or outlets and conducts such waters to a point for treatment or disposal.

Jetty: On open water, a structure extending into a body of water designed to prevent shoaling of a channel by littoral material and to direct stream or tidal flow. Usually built at the mouth of a river to help deepen and stabilize a channel.

Left or right bank of river: The left-hand or right-hand bank of a stream when the observer faces downstream.

Levee: A dike or embankment, generally constructed close to the banks of the stream, lake, or other body of water, intended to protect the landside from inundation or to confine the streamflow to its regular channel.

Lift: The difference in elevation between the upstream and downstream water surface levels in a lock and dam system.

Lift lock: A canal lock serving to lift a vessel from one reach of water to another such as from the downstream side to the upstream side of a navigation lock and dam system.

Lift span bridge: A bridge having a movable span that remains horizontal while being lifted vertically by cables arranged through towers at both ends.

Lift station: A small wastewater pumping station that lifts the wastewater to a higher elevation when the continuance of the sewer at reasonable slopes would involve excessive depths of trench.

Light-draft craft: A small boat, usually recreational, having a draft of about 10 feet or less.

Littoral drift: Material such as sand that is swept along the littoral zone by waves and current.

Littoral zone: The narrow area, including the land and water, bordering the shoreline.

Lock: An enclosed part of a canal, waterway, etc., equipped with gates so that the level of the water can be changed to raise or lower boats from one level to another.

Lock operation: Locks fill and empty by gravity, with no pumps required to raise or lower the water level. To raise the water level valves are opened above the upper gates and water flows into the lock through tunnels in both lock walls. This process is reversed to lower water in the lock. Valves are opened below the lower gates and water drains out of the lock through the tunnels. Gates at both ends of the lock open and close electrically after the proper water level has been reached.

Low water datum: A standard reference elevation, unique for each Great Lake, to which all depths on hydrographic charts are referred.

Maneuvering channel: A channel intended to facilitate maneuvering of vessels into and out of slips.

Meander: The name given to the winding course of a stream or river.

Miter gates: A type of gate commonly used to trap the water in a lock chamber.

Mouth of river: The exit or point of discharge of a stream into another stream, a lake, or the sea.

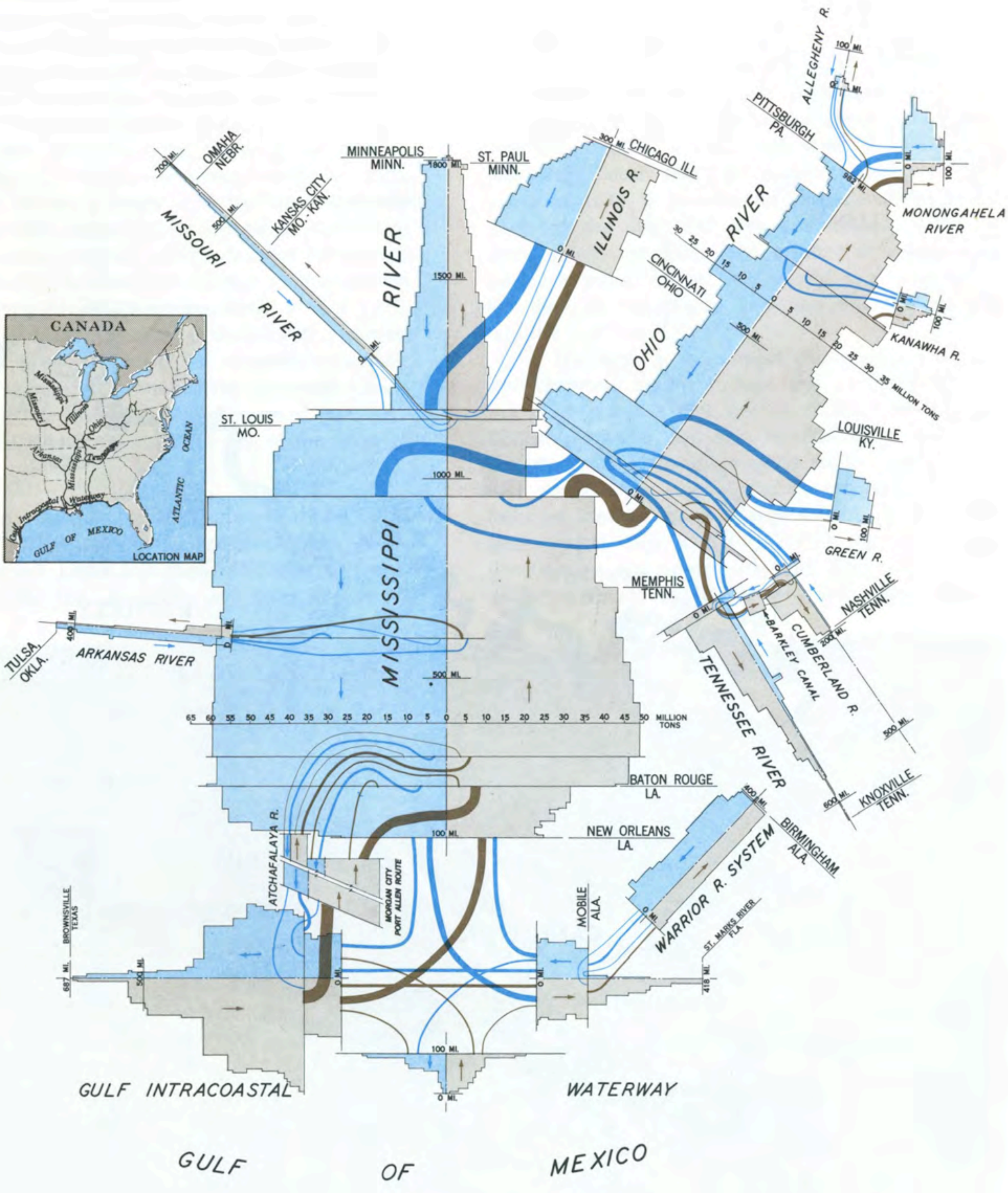
Oxbow lake: A lake formed in the meander of a stream, resulting from the abandonment of the meandering course because of the formation of a new channel course.

Pier: A structure which extends from the shore out into the lake and serves primarily for mooring and landing of boats. Also, the term is sometimes used synonymously with jetty.

Water Resources Development in Indiana 1981

by the U.S. Army Corps of Engineers

US Army Corps of Engineers
Louisville District



The widths of the light blue and light brown portions indicate tons of traffic moving on various portions of the system.

The darker lines show the various directions traffic takes as it moves from one portion of the system to the other and the varying widths of these lines indicate the relative amounts of traffic along each route.

Inland freight tonnage on the Mississippi River System and the Gulf Intracoastal Waterway, 1977

Water Resources Development in Iowa 1981

by the U S Army Corps of Engineers

US Army Corps of Engineers
North Central Division

Resource Management

The lands acquired by the Federal Government for construction of the 9-Foot Channel Project are managed to serve the general public, and many recreational opportunities are available as the result of the present navigation system.

The management plan for the Upper Mississippi River pools considers the unique wild character of the river bottom lands and the desirability of preserving their wildlife resources. Most of the lands acquired for the navigation project have been made available for concurrent administration by the Fish and Wildlife Service for waterfowl management. Generally, except for areas which are posted as waterfowl sanctuaries, these same lands may be used for wilderness camping and other recreational activities. All other Federal lands not leased or licensed for special purposes are also open to free public use.

Public Use Facilities

The Corps of Engineers operates many public use areas along the Upper Mississippi River 9-Foot Channel Project. These range in size from one to twelve acres. The degree of development varies from simple parking lots for fishermen to areas with boat-launching ramps and basic picnic and camping facilities. In addition, there are a number of public use areas on Corps land which have been developed and are operated by other agencies.

The locks and dams of the project themselves attract many sightseers. Visitors are always welcome at the locks and dams. Observation platforms have been built so that the public may have a better, and safer, view of lock operations.

The charts on the preceding pages list the locations and types of public use facilities provided by the Corps of Engineers along the channel project. More detailed information on specific public use areas may be obtained by contacting the appropriate District Engineer. The boundaries of the St. Paul, Rock Island, and St. Louis Districts are included in the chart. District office addresses are presented in the ''Foreword'' to this booklet.

Navigation charts, on sale in Corps District offices and at some boat docks and marinas, show federally owned lands under the jurisdiction of the Corps of Engineers and the Fish and Wildlife Service, the road network leading to the river, established river access points, facilities available at these points of access, and commercial recreational development on both privately owned and public lands.

Fish and Wildlife

Waterfowl food plants abound and great numbers of ducks and geese rest and feed in the project pools during fall and spring migrations. The waterfowl hunting season is long and highly productive. Sport fishermen find the pools attractive even in the winter, and their shanties dot the ice in many sloughs and backwaters. Commercial fishing is also a profitable undertaking which makes a contribution to the economy of the valley.

Water Resources Development in Kansas 1981

by the US Army Corps of Engineers

US Army Corps of Engineers
Southwestern Division

CHANNEL CUTOFF PROTECTS SALINA FROM SMOKY HILL RIVER

CHANNEL IMPROVEMENT FOR MUD CREEK AT ABILINE

Water Resources Development in Kentucky 1981

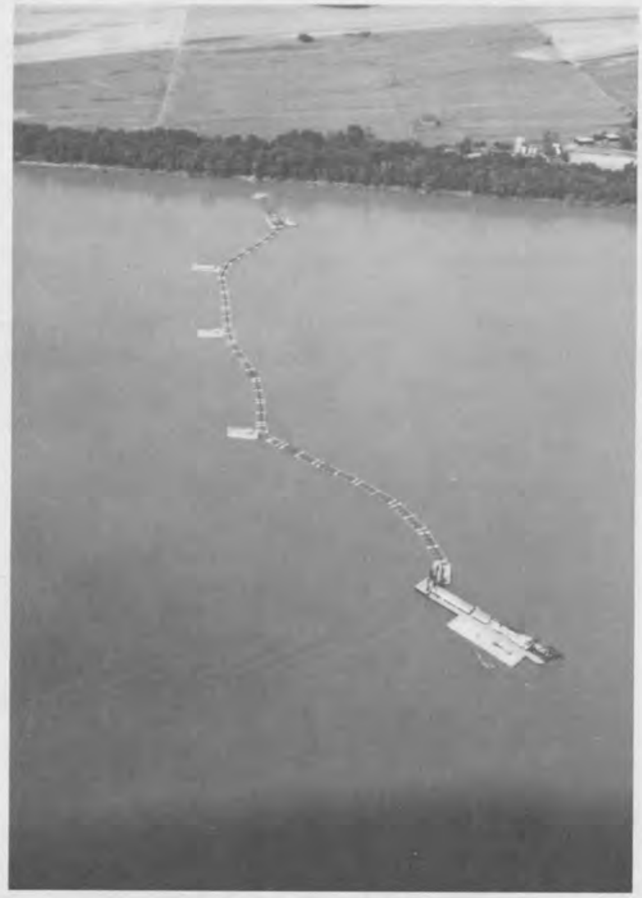
by the U.S. Army Corps of Engineers

US Army Corps
of Engineers
Louisville District

OHIO RIVER DREDGING MAINTENANCE IN KENTUCKY

Maintenance dredging on the Ohio River navigation system is being performed by contract dredge. An average of 3.2 million cubic yards of material is removed annually, providing a nine-foot channel for commercial traffic.

Dredge Boat of the 1800's



Modern Dredge Boat "Elco"

PROJECTS

Greenup Locks and Dam, Kentucky, Ohio and West Virginia (Completed) Huntington District 1–2. Greenup Locks and Dam, located on the Ohio River downstream from Greenup, was placed in operation in 1962 and completed in 1964. The project replaced four obsolete lock and dam structures on the Ohio River and one on the Big Sandy River. It provides a pool with a minimum navigable depth of 9 feet, extending 62 miles upriver to Gallipolis Locks and Dam. All of the important Huntington–Ashland–Ironton harbor area is located within this pool. The two lock chambers are side by side: the main lock is 110 feet by 1,200 feet in useable dimensions and the auxiliary lock is 110 feet by 600 feet. The dam is a nonnavigable gated structure with a lift of 30 feet between normal lower and upper pools.

The total federal cost of the completed project was about $55,700,000. Nonfederal funds were contributed for modifications of the substructure so that a highway bridge could be supported in the future. Design of the bridge is now in progress.

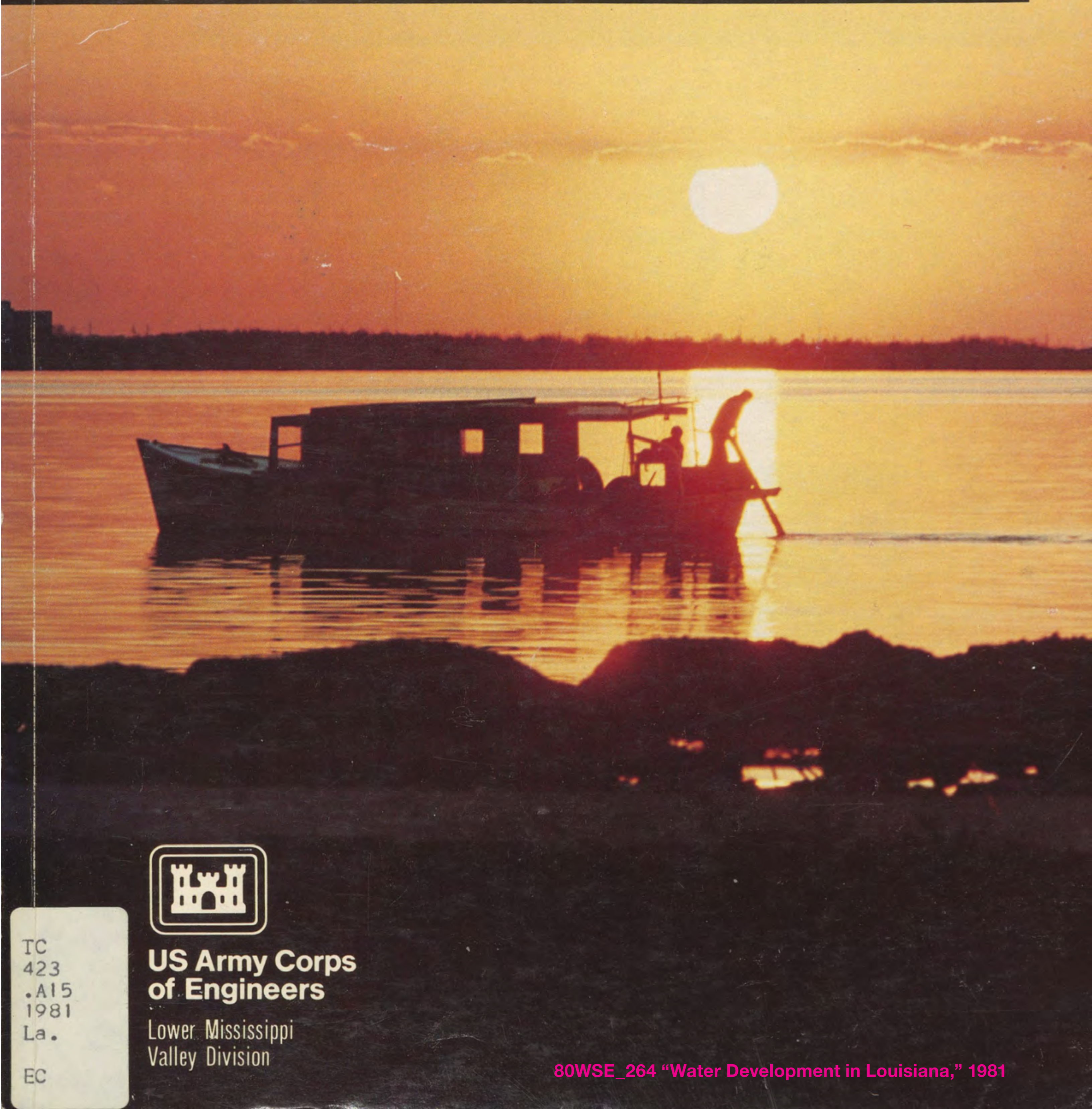
Recreational developments built in conjunction with the project include an observation platform and picnic facilities at the locks site, picnic facilities at the abutment site, and eight boat launching areas at various sites along the navigational pool. There are two boat launching ramps in Kentucky. Visitation in 1979 totaled 726,000. A visitor center complex is proposed for development at the Greenup locks site.

Vanceburg Electric Power and Heat System (Municipal), under license from the Federal Energy Regulatory Commission, is building a hydroelectric powerplant at the dam's right abutment on the Ohio side. Three turbines are to be installed, with a total generating capacity of 70,500 kilowatts. Completion is scheduled for 1982.

Captain Anthony Meldahl Locks and Dam, Kentucky and Ohio (Completed) Huntington District H–1. The Captain Anthony Meldahl Locks and Dam project is located on the Ohio River near Foster, Kentucky, and Chilo, Ohio. The project replaced four obsolete locks and dams on the Ohio River and provides a pool with a minimum navigable depth of 9 feet, extending 95 miles to Greenup Locks and Dam. The locks consist of two chambers, side by side: the main lock is 110 feet by 1,200 feet in useable dimensions and the auxiliary lock is 110 feet

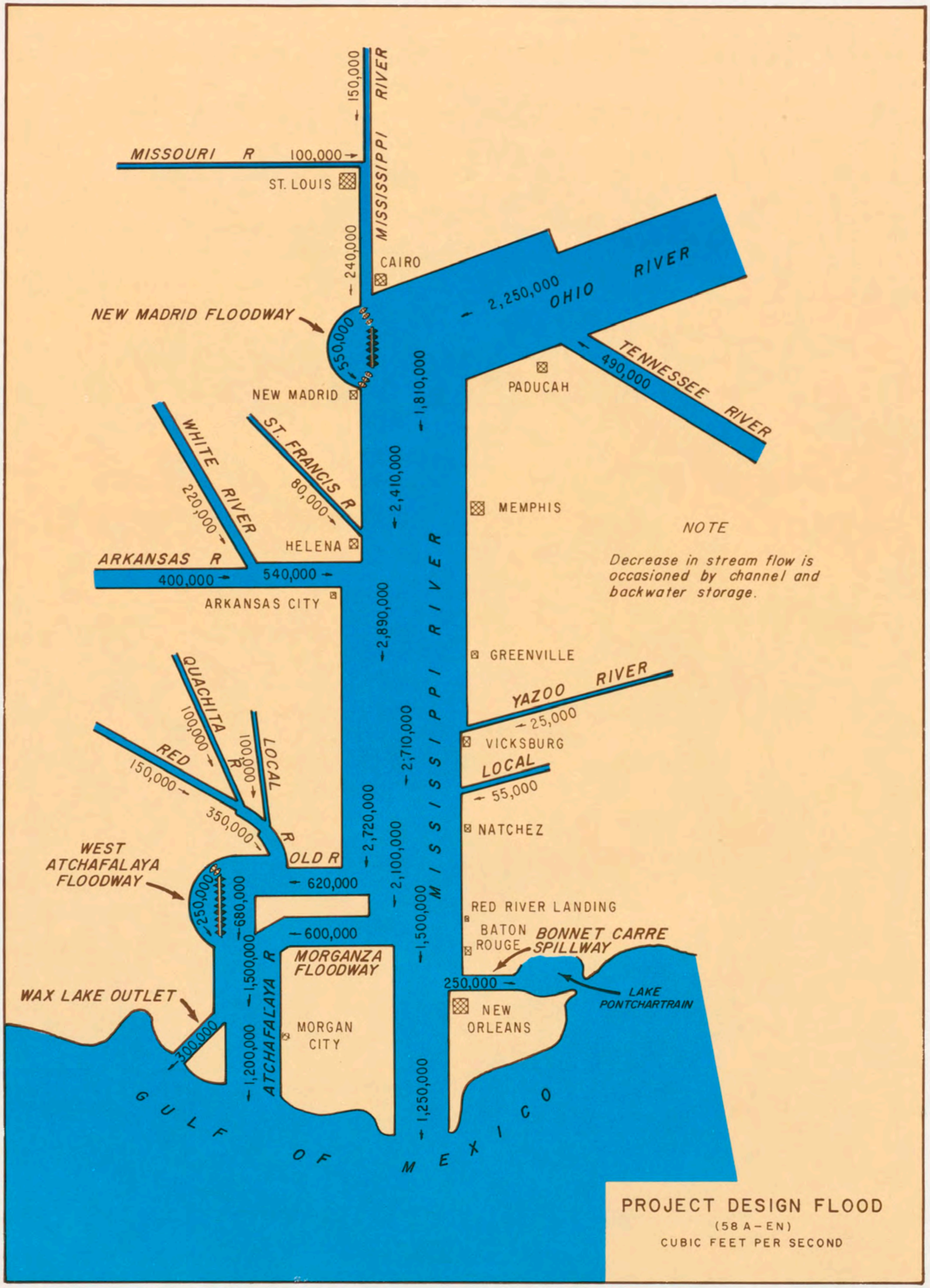
Water Resources Development in Louisiana 1981

by the US Army Corps of Engineers



US Army Corps of Engineers

Lower Mississippi Valley Division



80WSE_264 "Water Development in Louisiana," 1981

Water Resources Development in Maine 1981

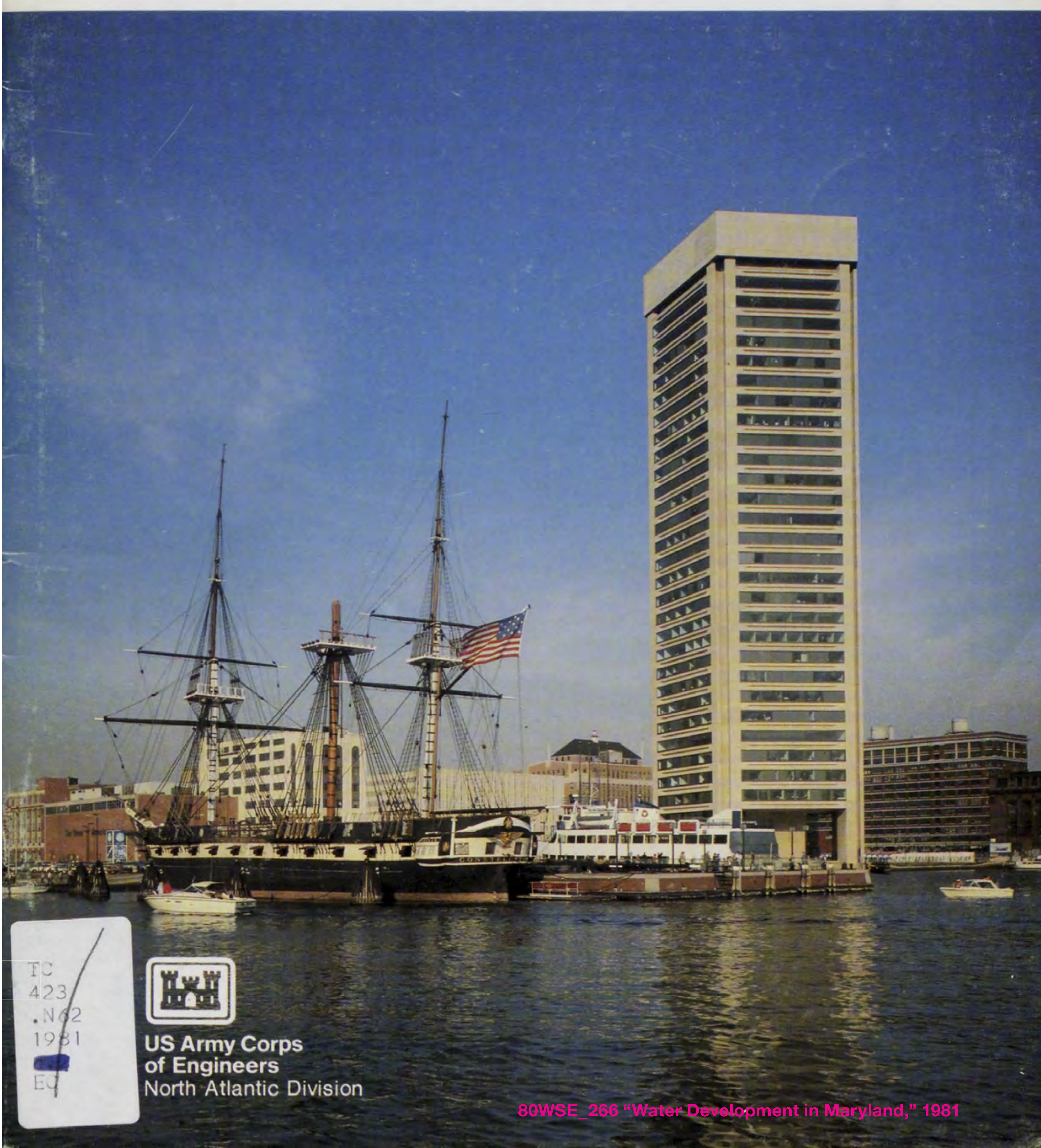
by the U S Army Corps of Engineers

US Army Corps
of Engineers
New England Division

TABLES

Sugarloaf Mountain

Water Resources Development in Maryland 1981



US Army Corps of Engineers
North Atlantic Division

BASIN DEVELOPMENT INVESTIGATIONS UNDERWAY

Chesapeake Bay Model, Matapeake, Maryland - Testing is being accomplished at the Baltimore Harbor portion of the model.

NAME	PURPOSE	STATUS
	BASIN DEVELOPMENT	
Chesapeake Bay Basin	To provide appraisal of the water resources needs and of the economic interrelations among the several portions of the basin, and their homogeneity, growth and sensitivity to development of water and related land resources. Results of completed studies in portions of the basin will be fully utilized. Water resources needs will be projected to the year 2020, and means for meeting those needs will be identified in general terms. The study area includes the entire Chesapeake Bay and its tidal tributaries from the Virginia Capes to the head of the tidal portion of the Susquehanna River. Water resources to be considered include navigation, fisheries, flood control, noxious weeds, water pollution, water-quality control, beach erosion, and recreation. A major	To be completed in 1983.

Water Resources Development in Massachusetts 1981

by the U S Army Corps of Engineers

MASSACHUSETTS COASTAL AREA

The Charles River in the wetlands of Natural Valley Storage

Water Resources Development in Michigan 1981

by the U S Army Corps of Engineers

US Army Corps
of Engineers
North Central Division

Harrisville Harbor provides well-needed refuge for light vessels.

Harrisville Harbor, Lake Huron

Recreational Navigation Project—Completed

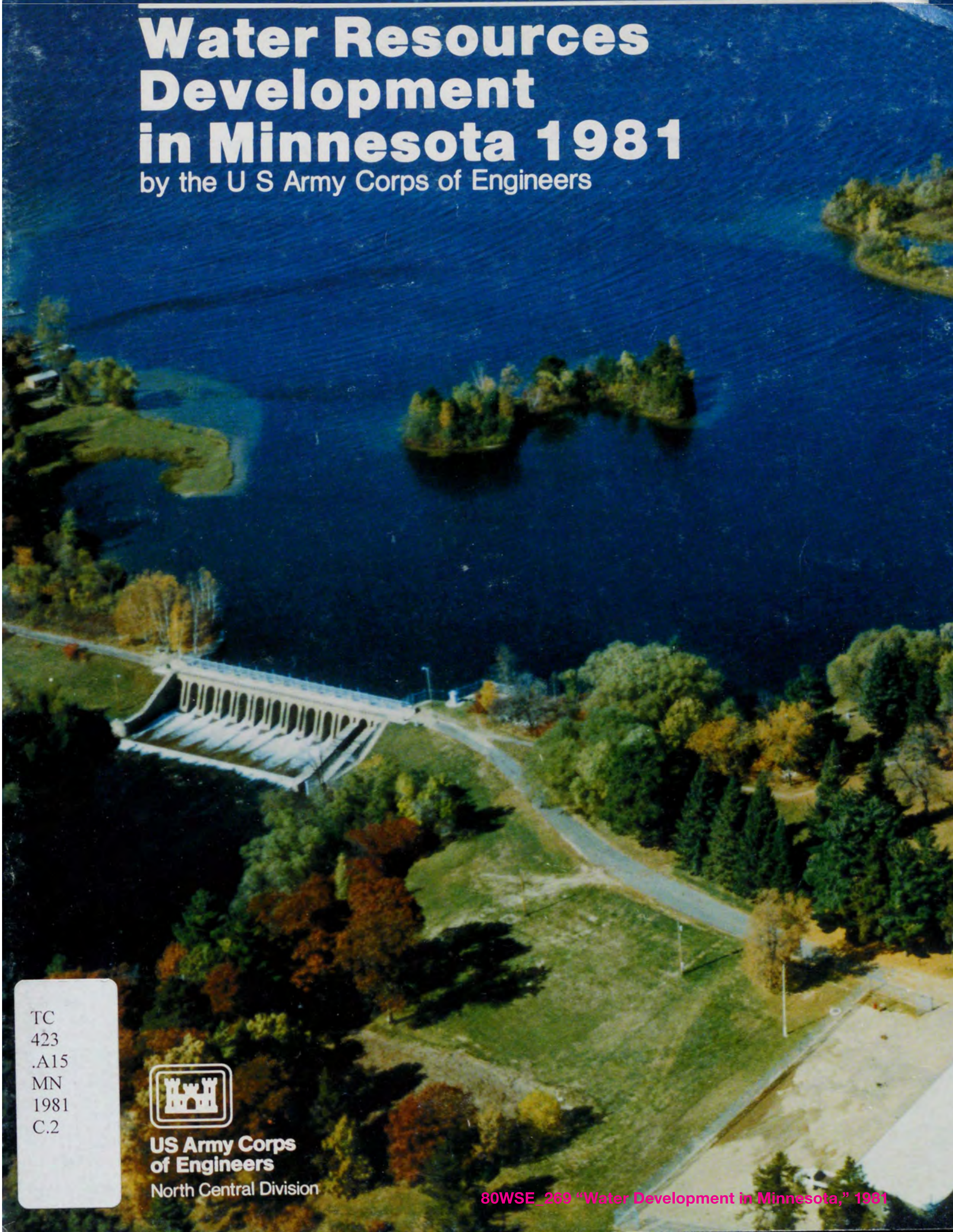
In 1945, Congress authorized the construction of a chain of harbors along the western shore of Lake Huron. One of these was at Harrisville, located 30 miles south of Alpena. It is the only safe refuge for light-draft craft between Alpena and Au Sable, Michigan.

There is a Weather Bureau Warning Display Station at Harrisville and a state park with camping facilities is located just south of the harbor. The Michigan State Waterways Commission and the city have provided a public dock with facilities for mariners.

Construction of two breakwaters and dredging to 10 feet in the harbor basin and 12 feet in the entrance channel, have cost the United States $1,562,637. Local interests contributed an additional $129,500 for construction of the breakwaters, channel, and basin. Maintenance costs amount to $592,074.

Water Resources Development in Minnesota 1981

by the U S Army Corps of Engineers



US Army Corps of Engineers

North Central Division

Lock and Dam No. 2 at Hastings, Minnesota, is one of a series of locks and dams constructed and operated on the Upper Mississippi River by the U.S. Army Corps of Engineers to provide a stairway of water for commercial barges and pleasure boats.

ing diagonally through the business district of Minneapolis for four miles, it forms the boundary between the Twin Cities of St. Paul-Minneapolis. The Minnesota River, first major tributary of the Mississippi, flows into the Mississippi at the Twin Cities. From the Twin Cities, the Mississippi River winds through an 856-mile stretch of high bluffs, rolling hills, and wild wetlands, passing near prairie farms and more than 500 forested islands. On its journey, it is joined near Prescott, Wisconsin, by the St. Croix River. For the next 137 miles the Mississippi River forms the Minnesota-Wisconsin state line. It continues southward, and near Genoa, Wisconsin, becomes the state line dividing Iowa and Wisconsin. In this stretch, the Wisconsin River joins the Mississippi River.

The Mississippi River forms the entire 312-mile eastern boundary of Iowa and the entire western boundary of Illinois. Along this reach, major Illinois tributaries and several Iowa tributaries flow into the Mississippi. The Rock River joins the Mississippi immediately below Rock Island, Illinois. Further downstream, the Illinois River — largest tributary of the Mississippi River above the mouth of the Missouri — flows into the Mississippi near Grafton, Illinois. Still further south, below East St. Louis, the Kaskaskia and the Big Muddy Rivers join. Iowa tributaries include the Turkey, Maquoketa, Wapsipinicon, Iowa, Cedar, Skunk, and the Des Moines Rivers. The Turkey flows into the Mississippi near the northern part of the state at Guttenberg, the Des Moines at the southern end of Keokuk. The others join the Mississippi River at random intervals and over the reach drain the eastern two-thirds of the State of Iowa. Tributaries draining the sections of the State of Missouri, which are included in the Upper Mississippi River Region, include the Fox, Wyaconda, and the Fabius Rivers.

The Upper Mississippi River Region ends at Cairo, Illinois, but the Mississippi continues southward passing through or past five more states on its journey to the Gulf of Mexico.

Upper Mississippi River Comprehensive Basin Study,
Completed Comprehensive Study (North Central Division)

An executive order providing for an Upper Mississippi River Basin Commission (UMRBC) was

Water Resources Development in Mississippi 1981

US Army Corps of Engineers

Lower Mississippi Valley Division

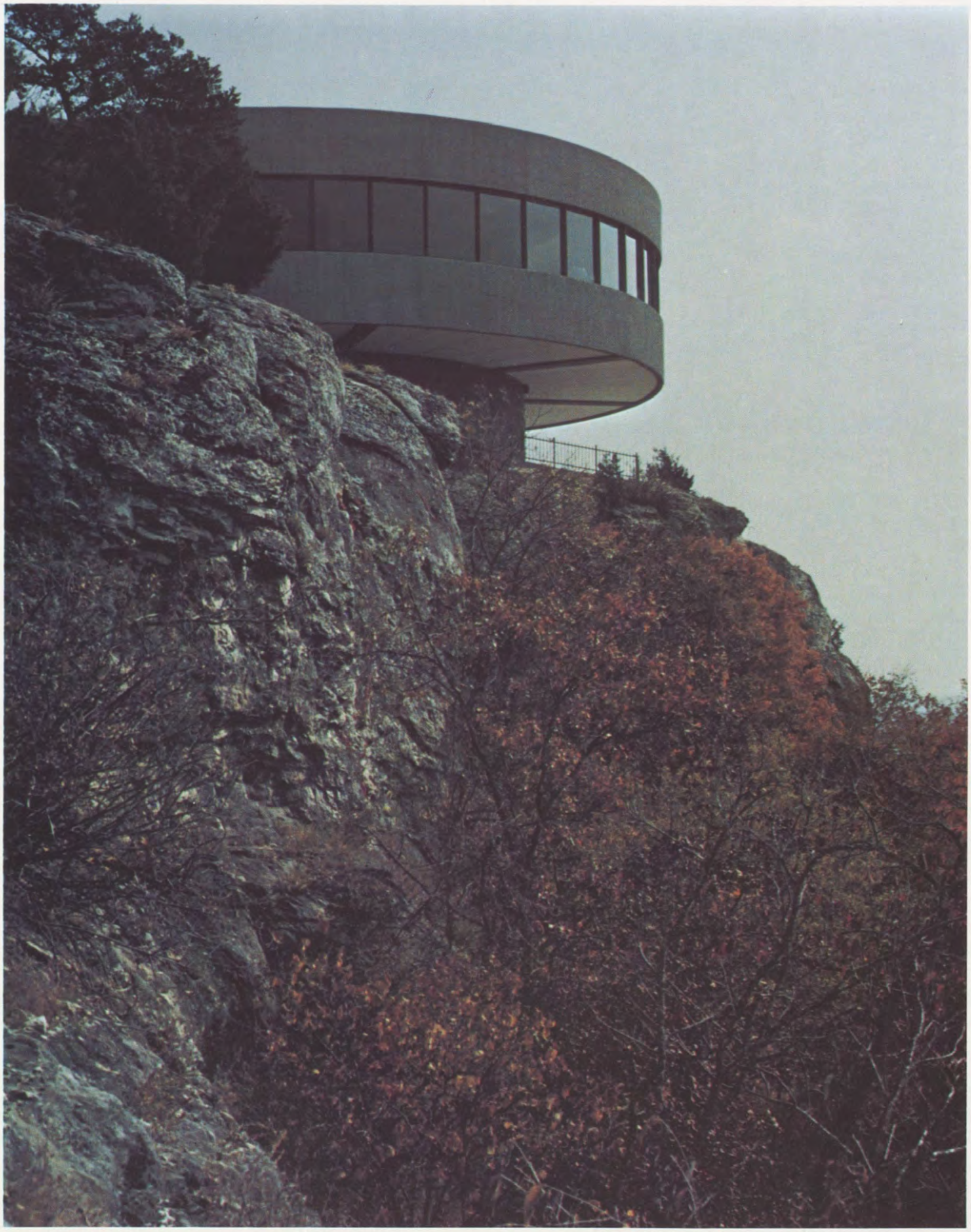
Bank grading for revetment work on the Mississippi River

Dredging operations on the Mississippi River

Water Resources Development in Missouri 1981

by the US Army Corps of Engineers

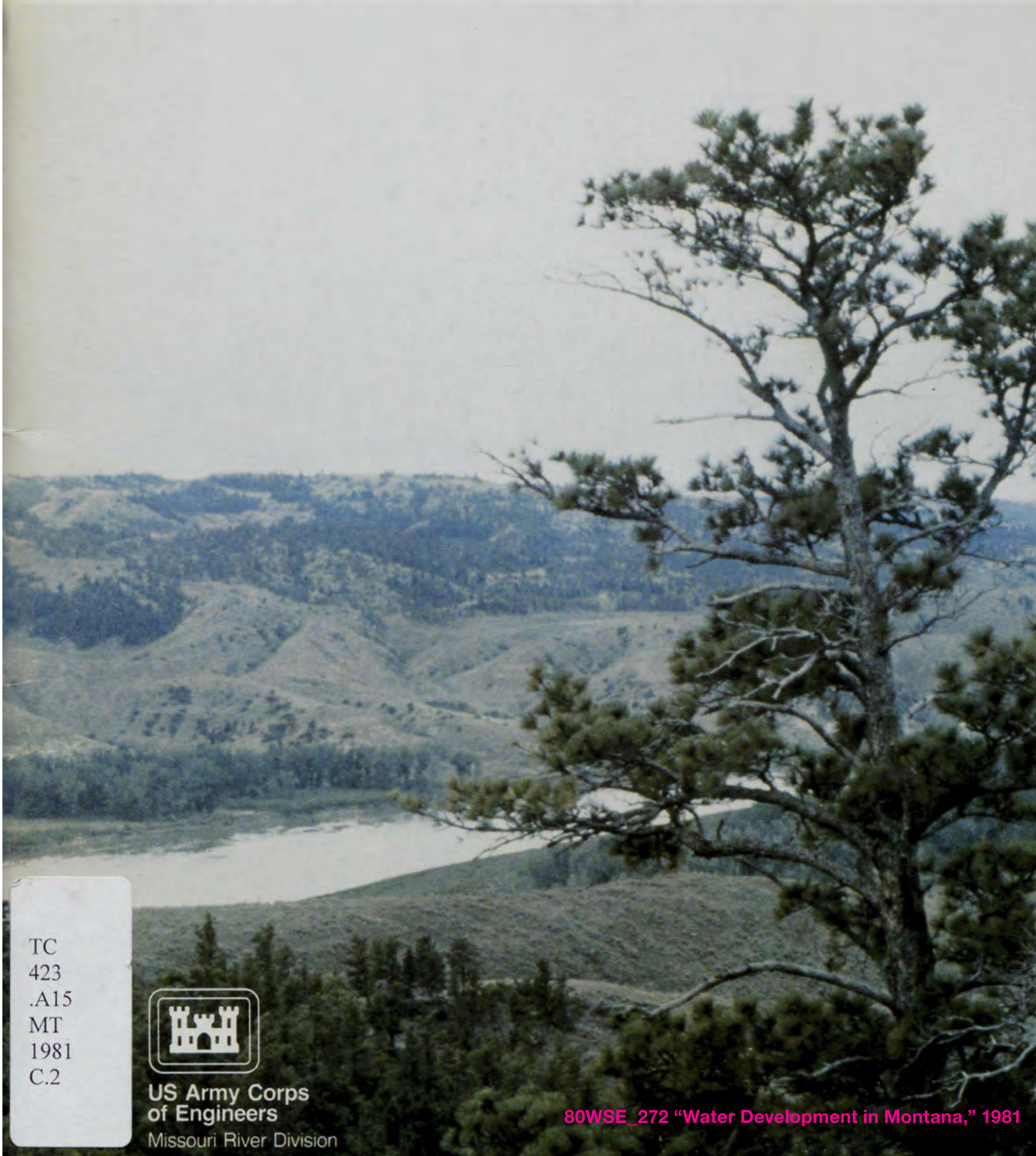
US Army Corps
of Engineers
Missouri River Division



OVERLOOK AND VISITOR CENTER AT HARRY S. TRUMAN DAM AND RESERVOIR

Water Resources Development in Montana 1981

by the US Army Corps of Engineers



US Army Corps of Engineers
Missouri River Division

A Past to Remember; A Future to Mold

As far back as 1804, Army Engineer Officers were traveling the Missouri River. Captain Merriweather Lewis and Captain William Clark journeyed along the Missouri River from Saint Louis to its source and then continued westward on their journey. The Lewis and Clark expedition was the first official contact by the U.S. Army in the Missouri River basin.

The mural shown above depicts the Lewis and Clark expedition as the explorers survey the horizon. The scenes travel through time and show the development of our Nation as the East merged with the West. The Corps of Engineers

Water Resources Development in Nebraska 1981

by the US Army Corps of Engineers

US Army Corps of Engineers
Missouri River Division

GLOSSARY

Acre-foot: A volume of water equivalent to 1 acre of land covered to a depth of 1 foot.

Advanced engineering and design work: Work done by Corps of Engineers in preparation of a project for construction.

Agricultural levee: A levee that protects agricultural areas where the degree of protection is usually less than that of a flood control levee.

Alluvial: Of, pertaining to, or composed of sediment deposited by flowing water, as in a riverbed, flood plain, or delta.

Appropriation: The setting aside of money by Congress, through legislation, for a specific use.

Arch-gravity structure: A structure which derives its resistance to the pressure of water from both an arching effect and its own weight.

Authorization: House and Senate Public Works Committee resolutions or specific legislation which provides the legal basis for conducting studies or constructing projects. The money necessary for accomplishing the work is not a part of the authorization but must come from an appropriation by Congress.

Bank and channel stabilization: The process of preventing bank erosion and channel degradation.

Basin: (1) Drainage area of a lake or stream, such as a river basin. (2) A naturally or artificially enclosed harbor for small craft, such as a yacht basin.

Concrete-gravity structure: A type of concrete structure in which resistance to overturning is provided only by its own weight.

Confluence: The place where streams meet.

Control dam: A dam or structure with gates to control the discharge from the upstream reservoir or lake.

Crest length: The length of a dam along its crest.

Dam: A barrier constructed across a valley for impounding water or creating a reservoir.

Damages prevented: The difference between damages without the project and the damages with the project in place.

Degree of protection: The amount of protection that a flood control measure is designed for, as determined by engineering feasibility; economic criteria; and social, environmental, and other considerations.

Dike: An embankment to confine or control water.

Diversion channel: (1) An artificial channel constructed around a town or other point of high potential flood damages to divert floodwater from the main channel to minimize flood damages. (2) A channel carrying water from a diversion dam.

Earthfill dam: A dam with its main section composed principally of earth, gravel, sand, silt, and clay.

Flood capacity: The flow carried by a stream or floodway at bankfull water level. Also, the storage capacity of the flood pool at a reservoir.

Flood plain: Valley land along the course of a stream which is subject to inundation during periods of high water that exceed normal bankfull elevation.

Flood proofing: Techniques for preventing flood damage to the structure and contents of buildings in a flood hazard area.

Floodwall: A wall, usually built of reinforced concrete, to confine a stream to prevent flooding.

Gilsonite: A natural black bitumen used in the manufacture of acid, alkali, and waterproof coatings.

Groin: A wall-like structure built perpendicular to the bank to prevent beach erosion.

Habitat: The total of the environmental conditions which affect the life of plants and animals.

Headwaters: (1) The upper reaches of a stream near its source. (2) The region where ground waters emerge to form a surface stream. (3) The water upstream from a structure.

Hydraulic earthfill dam: An embankment built up from waterborne clay, sand, and gravel carried through a pipe or flume.

Ice jam: Accumulation of ice packed together and piled up, choking the stream channel, and causing a rise in water level above the jam.

Jetty: A structure similar to a groin built on a seashore or riverbank to prevent erosion due to currents and/or tide.

Joint-use storage: Reservoir storage space which is used for more than one purpose. The operation may follow a fixed predetermined schedule or may be flexible and subject to adjustment, depending upon particular hydrologic conditions.

Left or right bank of river: The left-hand or right-hand bank of a stream when the observer faces downstream.

Levee: A dike or embankment, generally constructed close to the banks of the stream, lake, or other body of water, intended to protect the landside from inundation or to confine the streamflow to its regular channel.

Mouth of river: The exit or point of discharge of a stream into another stream, a lake, or the sea.

Oxbow lake: A lake formed in the meander of a stream, resulting from the abandonment of the meandering course due to the formation of a new channel course.

Paleontology: The study of fossils and ancient life forms.

Reach: A length, distance, or leg of a channel or other watercourse.

Reservoir: A pond, lake, tank, basin, or other space, either natural or created in whole or in part by the building of a structure such as a dam, which is used for storage, regulation, and control of water.

Revetment: (1) A facing of stone, concrete, or sandbags to protect a bank of earth from erosion. (2) A retaining wall.

Revetted levee: A stone or concrete faced embankment raised to prevent a river from overflowing.

Riprap: A layer, facing, or protective mound of randomly placed stones to prevent erosion, scour, or sloughing of a structure or embankment. Also, the stone so used.

Rock dike: An embankment built principally of rock.

Sill: (1) A horizontal beam forming the bottom of the entrance to a lock. (2) A low, submerged dam-like structure built to control riverbed scour and current speeds.

Spillway: A waterway on a dam or other hydraulic structure used to discharge excess water to avoid overtopping of a dam.

Stage: The elevation of the water surface above or below an arbitrary datum.

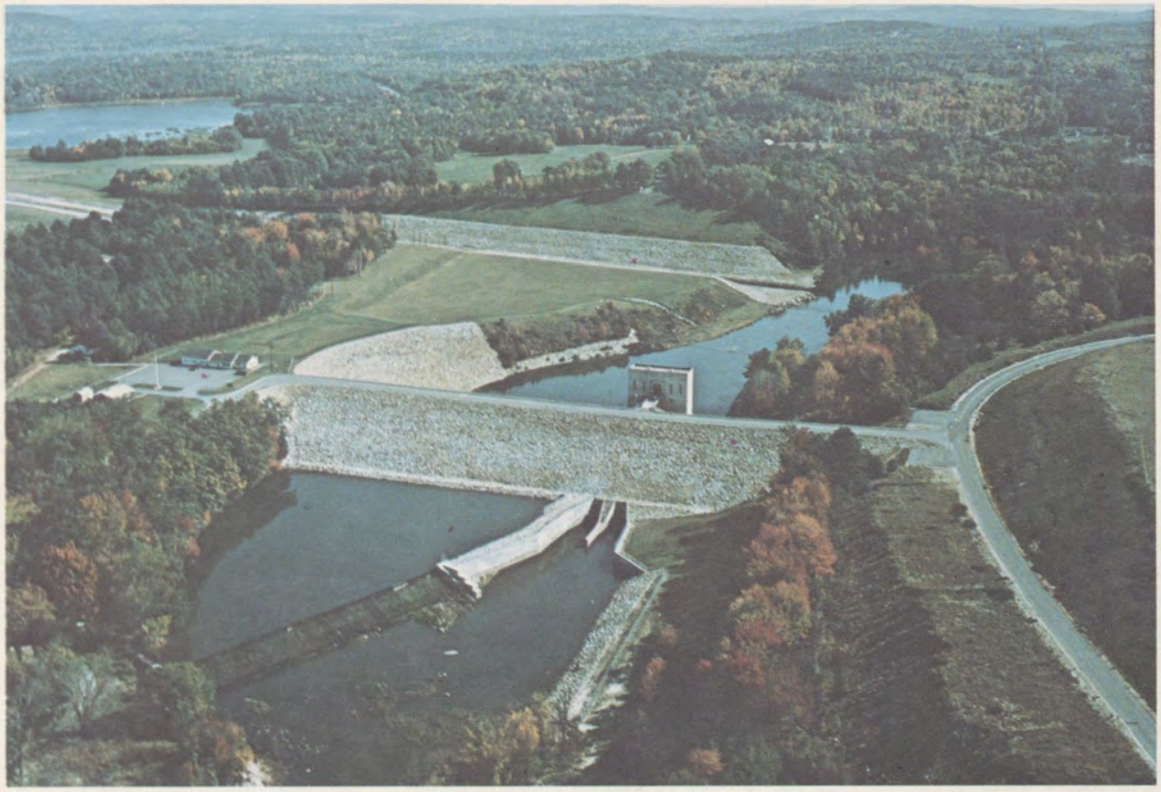
Standard project flood: A flood that may be expected from the most severe combination of meteorological and hydrological conditions that are reasonably characteristic of the geographical region involved, excluding extremely rare combinations.

Tributary: A stream or other body of water that contributes its water to another stream or body of water.

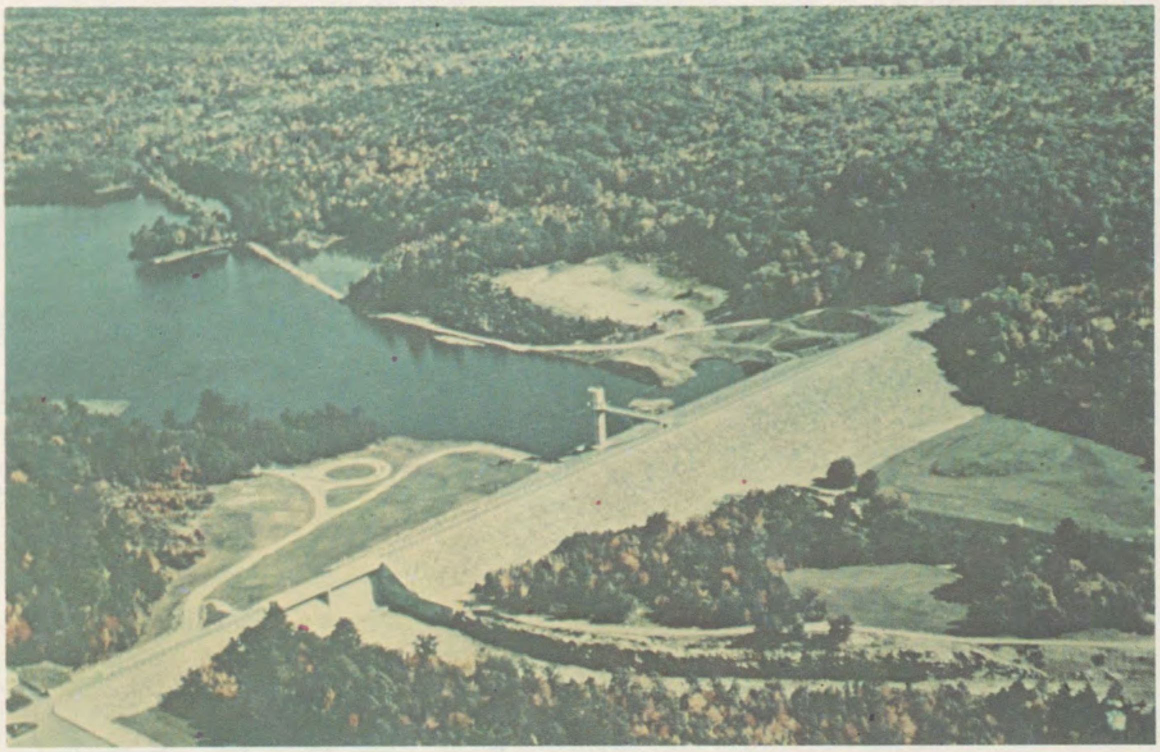
Water Resources Development in New Hampshire 1981

by the U S Army Corps of Engineers

US Army Corps
of Engineers
New England Division



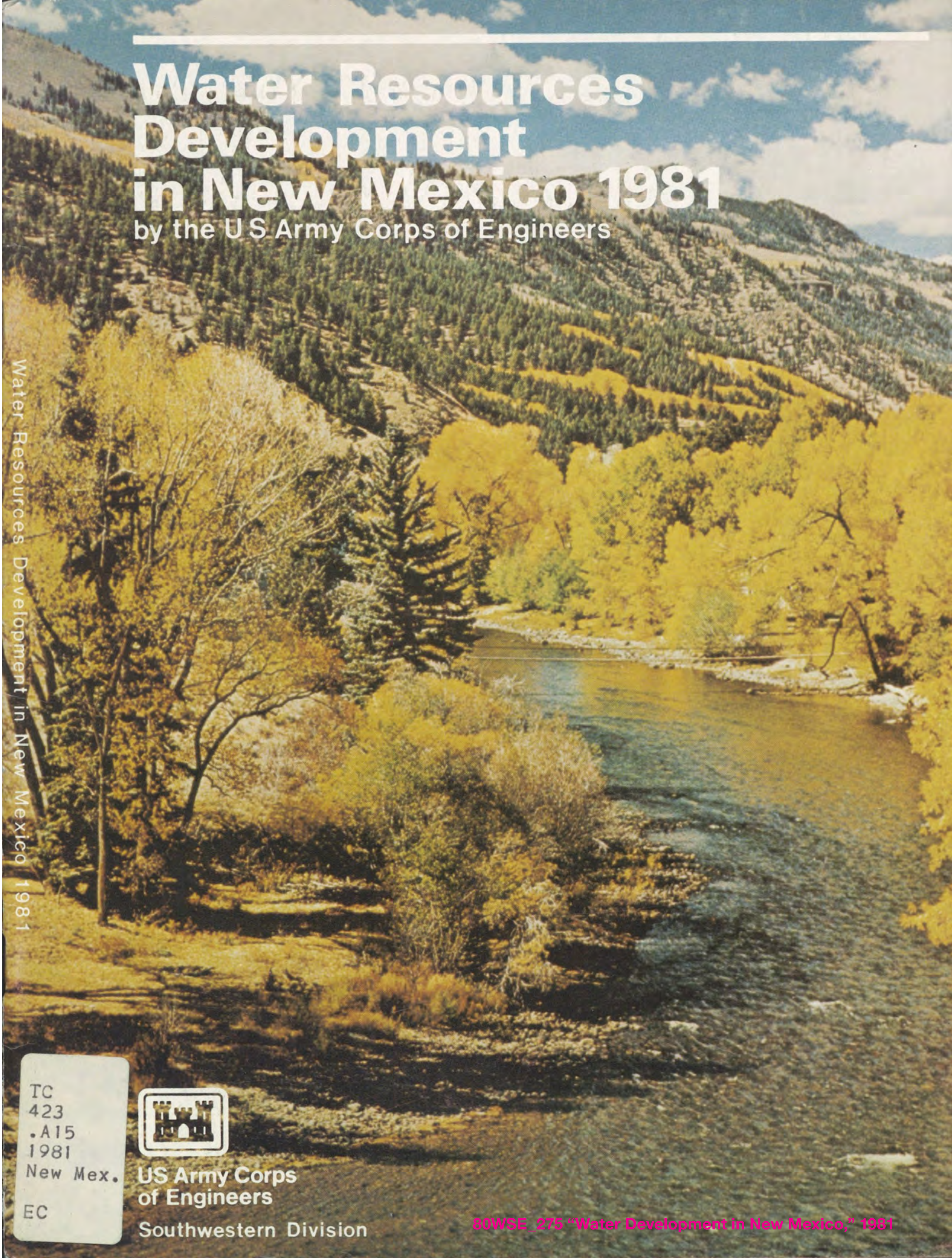
Hopkinton Lake



Everett Lake

Water Resources Development in New Mexico 1981

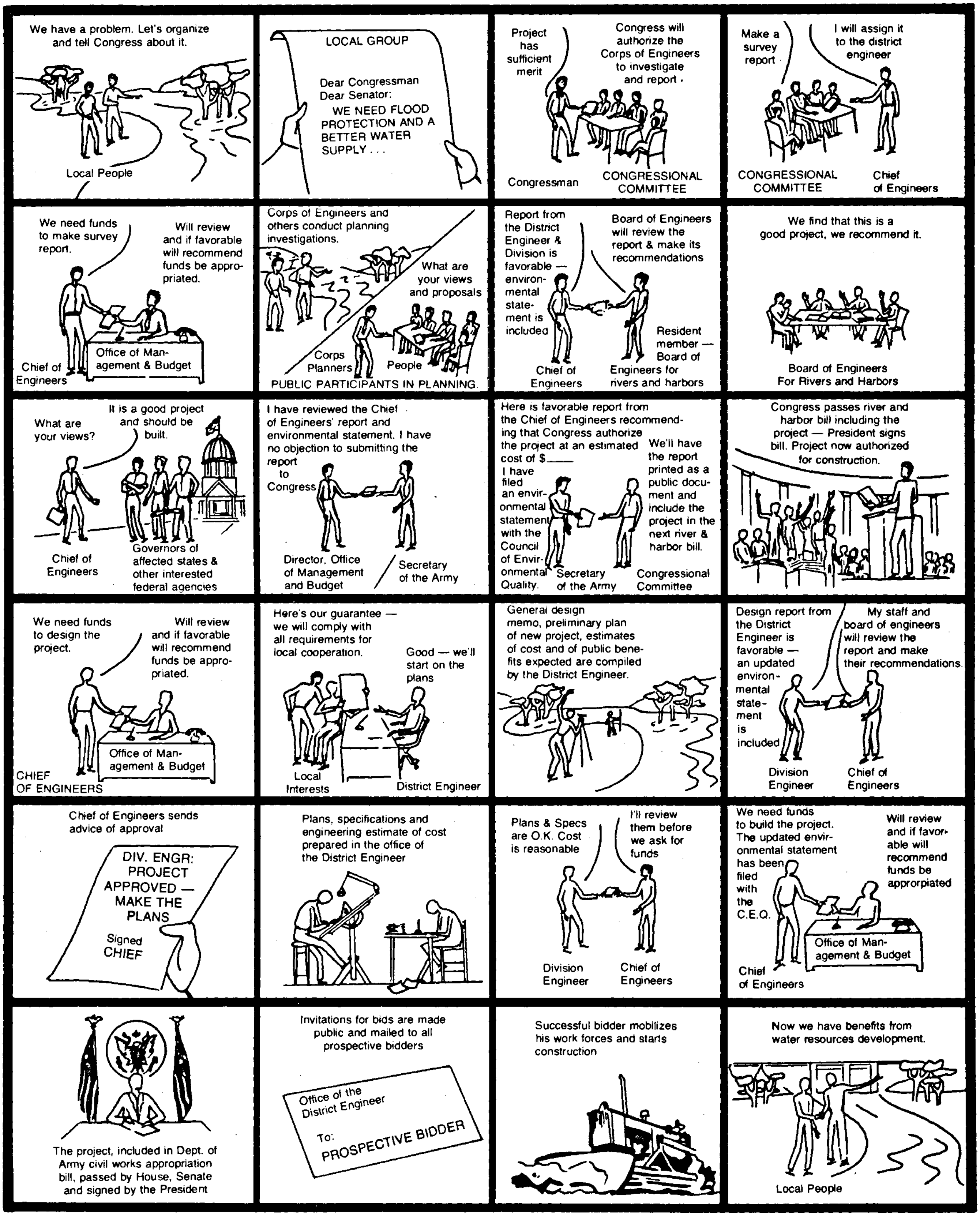
by the U S Army Corps of Engineers



Water Resources Development in New Mexico 1981

US Army Corps
of Engineers
Southwestern Division

How Corps of Engineers' Projects are Started, Authorized and Built



Water Resources Development in New York 1981

S Army Corps
f Engineers
orth Atlantic Division

inner channel 6 feet deep and 100 feet wide to the Long Island Intracoastal Waterway, a total distance of 1.7 miles. It also provides for rehabilitation of existing revetments and jetties, extension of existing jetties and provision of sand bypassing facilities. Estimated costs for the project are $7,050,000 Federal and $4,050,000 non-Federal (October 1978 price level).

Westhampton Beach, N.Y. — View showing erosion around building foundations.

Westhampton Beach, N.Y. — View showing breakthrough barrier caused by 1962 storm.

Emergency Operations

Westhampton Beach, N.Y. — View showing site of breakthrough after closure.

LONG ISLAND COAST — STORM OF 6-8 MARCH 1962

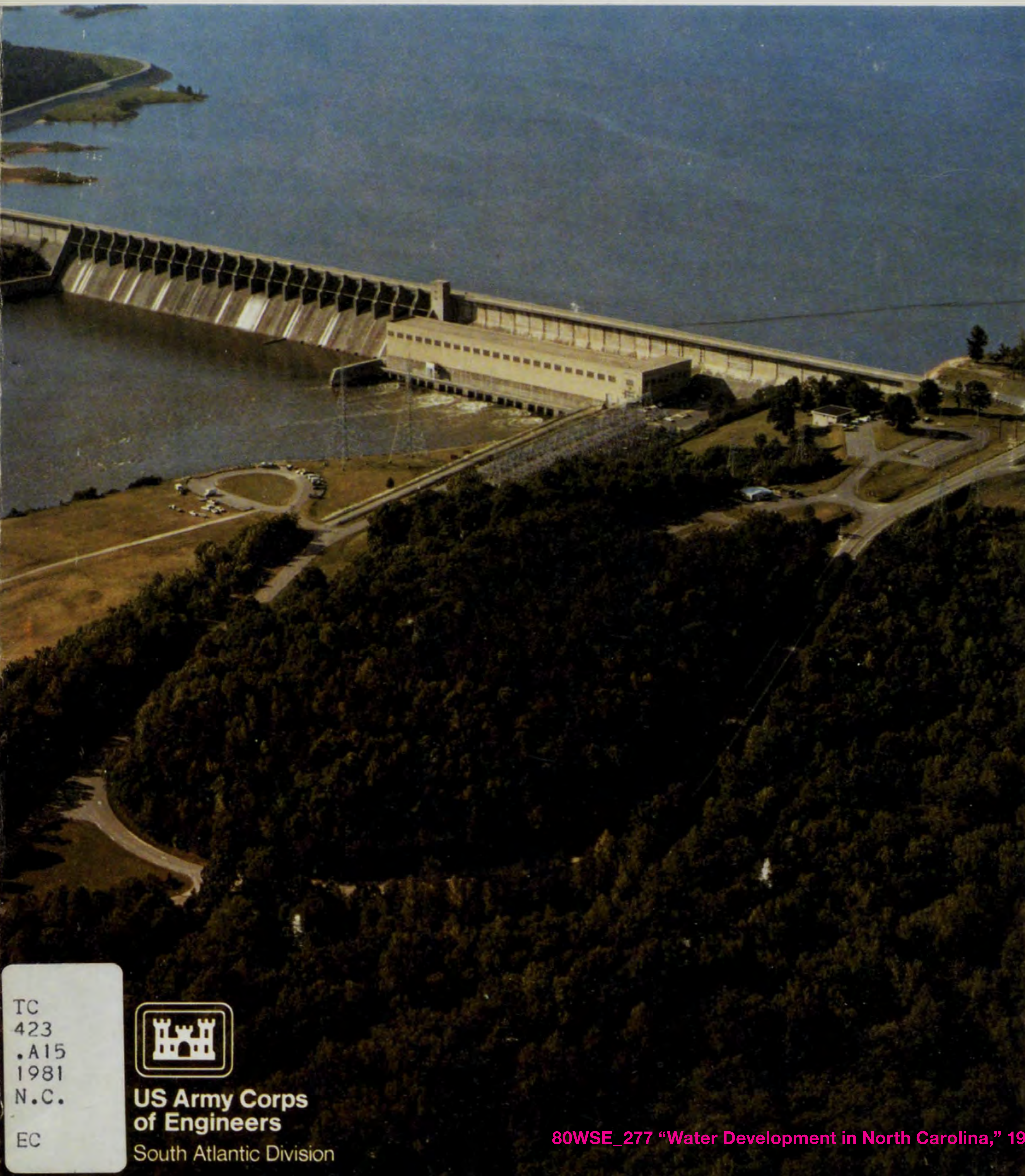
On 6-8 March 1962, a storm of sustained direction and intensity together with high spring tides resulted in vast unequaled damages to the Atlantic Coast from Florida to New England. As a result, the U.S. Army Corps of Engineers acting under Public law 87-875, and through the auspices of Office of Emergency Planning, Washington, D.C., performed engineering and construction services for emergency shore protection and rehabilitation.

Initial field engineering was started on 7 March 1962 by Federal personnel who made inspections, damages and soil surveys, aligned the initial protective dunes and prepared estimates of quantities and costs. The services of engineering and soil consultant firms were engaged to assist in surveys, damage estimates, locating suitable borrow areas and inspection and testing of materials used in construction. A description of the construction work performed by the Corps of Engineers for emergency shore protection and rehabilitation together with estimated Federal costs of the work at each location, at November 1962 price levels, follows:

A. Hampton Beach: Emergency rehabilitation of the beach and dune along the shore front in this community was commenced on 19 July and completed on 13 September 1962. About 100,000 cubic yards of

Water Resources Development in North Carolina 1981

by the US Army Corps of Engineers



US Army Corps of Engineers
South Atlantic Division

Philpott Lake (Wilmington District)

Project Data

Drainage area above dam 212 square miles

Dam:
- Type .... Concrete
- Crest length .... 892 feet
- Height .... 220 feet

Lake Capacity:

Flood control	154,500 acre-feet
Inactive	54,000 acre-feet
Power	110,000 acre-feet
Total Capacity	318,500 acre-feet

Power:
- Installed capacity .... 14,000 kilowatts

VISITATION—1979 1,193,500

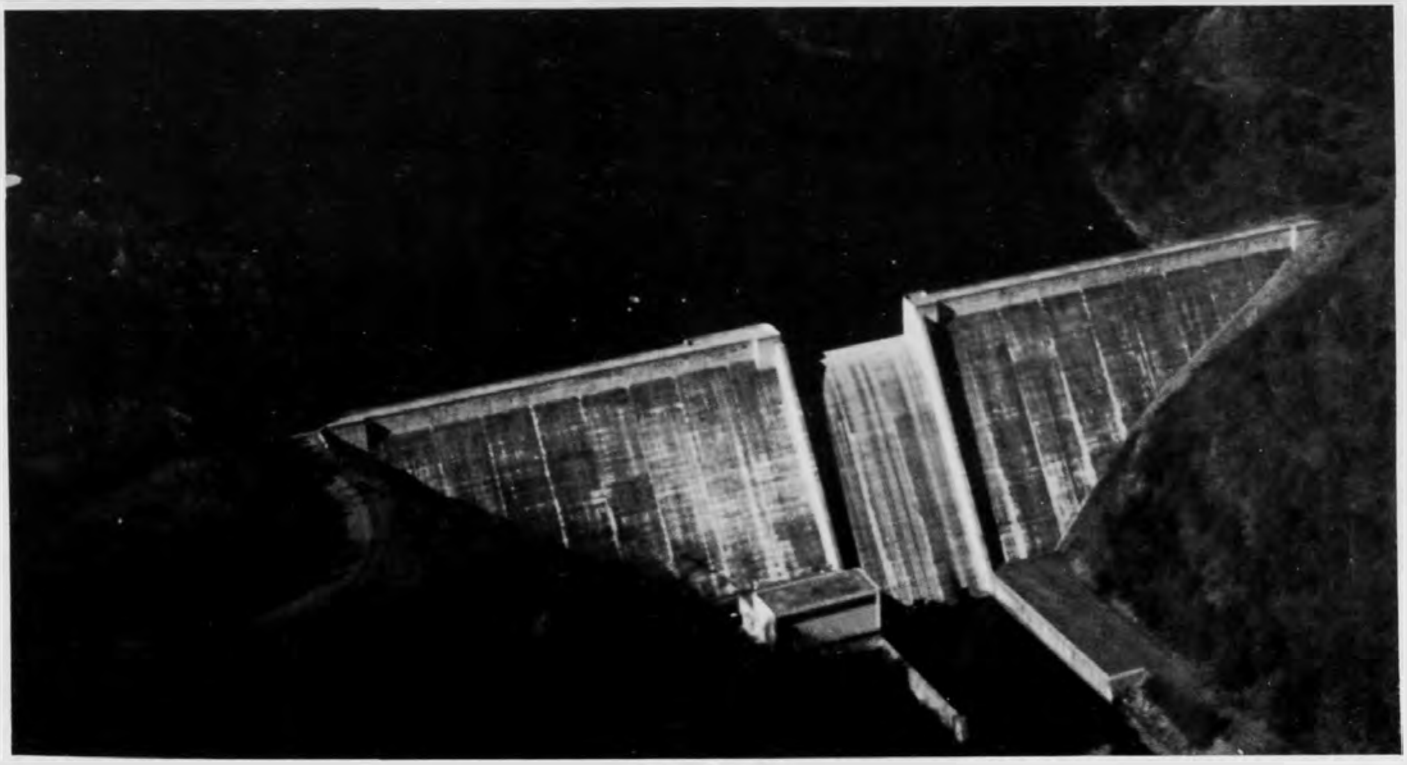
The Philpott Dam, authorized by the Flood Control Act of 1944, is an integral unit in the comprehensive plan for the development of the water resources of the Roanoke River Basin. The dam is on Smith River in Virginia, 7 miles upstream from Bassett and about 39 miles above the Virginia-North Carolina State line. The drainage area above the Philpott Dam is 212 square miles.

The dam consists of a concrete gravity structure, 892 feet long and 220 feet high at the highest point. The spillway section is ungated and is located in the old streambed. Three penstocks guide the flow of water through three generating units with a total capacity of 14,000 kw. The project affords 154,500 acre-feet of flood control storage. The reservoir at full power pool covers 2,880 acres.

The project is being operated primarily for the control of floods and generation of hydroelectric power. Other benefits will accrue to the project from fish and wildlife preservation and public use of the reservoir area. The estimated cost (1980) is $14,672,000 for new work.

Philpott Lake has seen considerable development of recreational resources since the project was completed. Additional facilities are under construction and being planned. Fishing, boating, swimming, picnicking, and related activities are enjoyed in season. In 1979, about 1,193,500 persons visited the reservoir area for sightseeing, fishing, hunting, boating, picnicking, swimming, and camping.

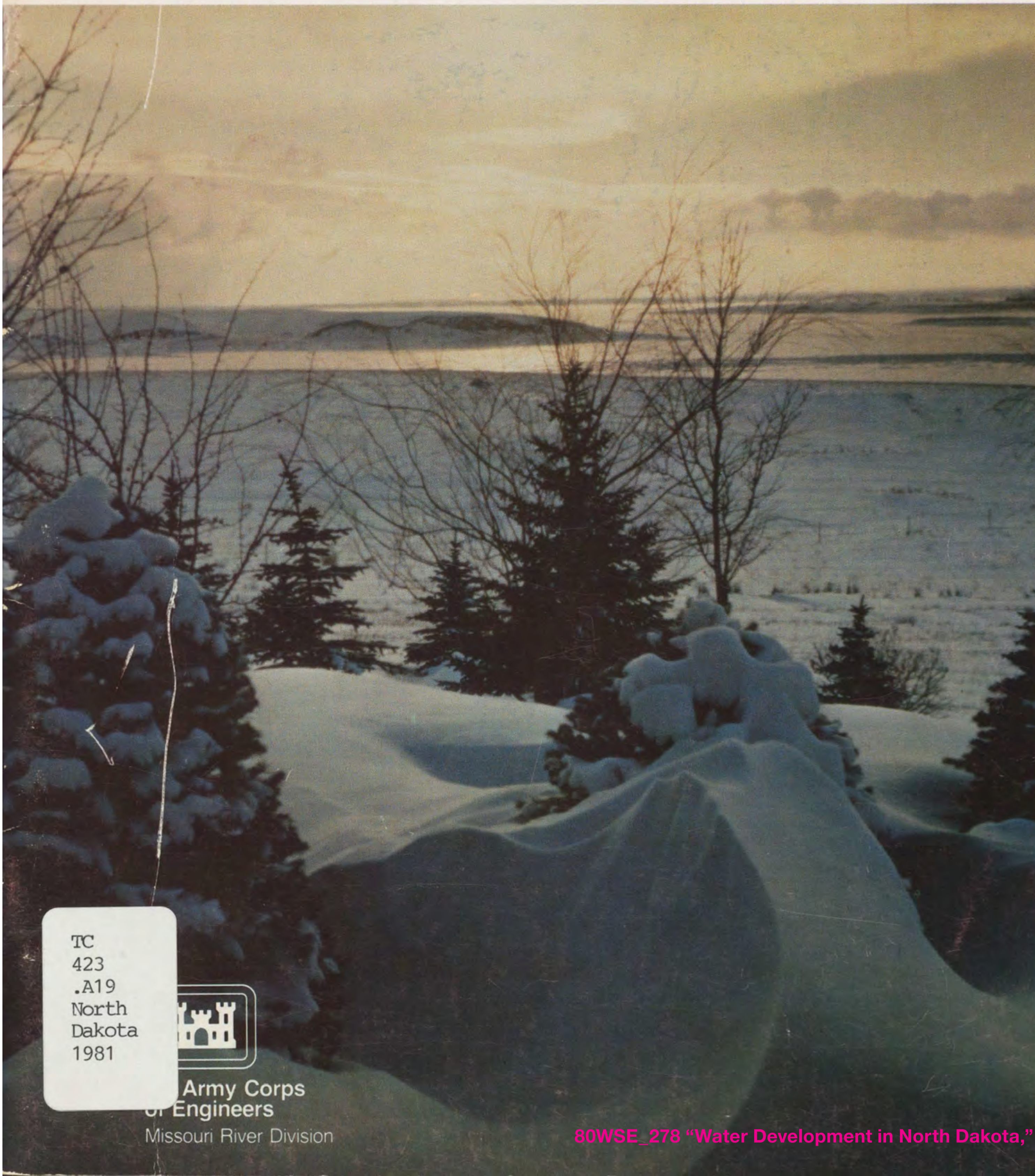
The project has provided flood protection to the cities and towns in the Smith River Valley below the dam, including Spray, N.C., since 1951. About $16,770,000 in flood damages have been prevented by the project. An estimated $5,000,000 of the total damages prevented was in connection with Hurricane Agnes in June 1972. "Agnes" was the first major flood since the flood control capacity became available in 1951. Generation of power began in September 1953. The Southeastern Power Administration markets power available at Federal projects in this area. Power revenues to September 30, 1980 amounted to $10,109,000 marketing expenses deducted.



Philpott Lake

Water Resources Development in North Dakota 1981

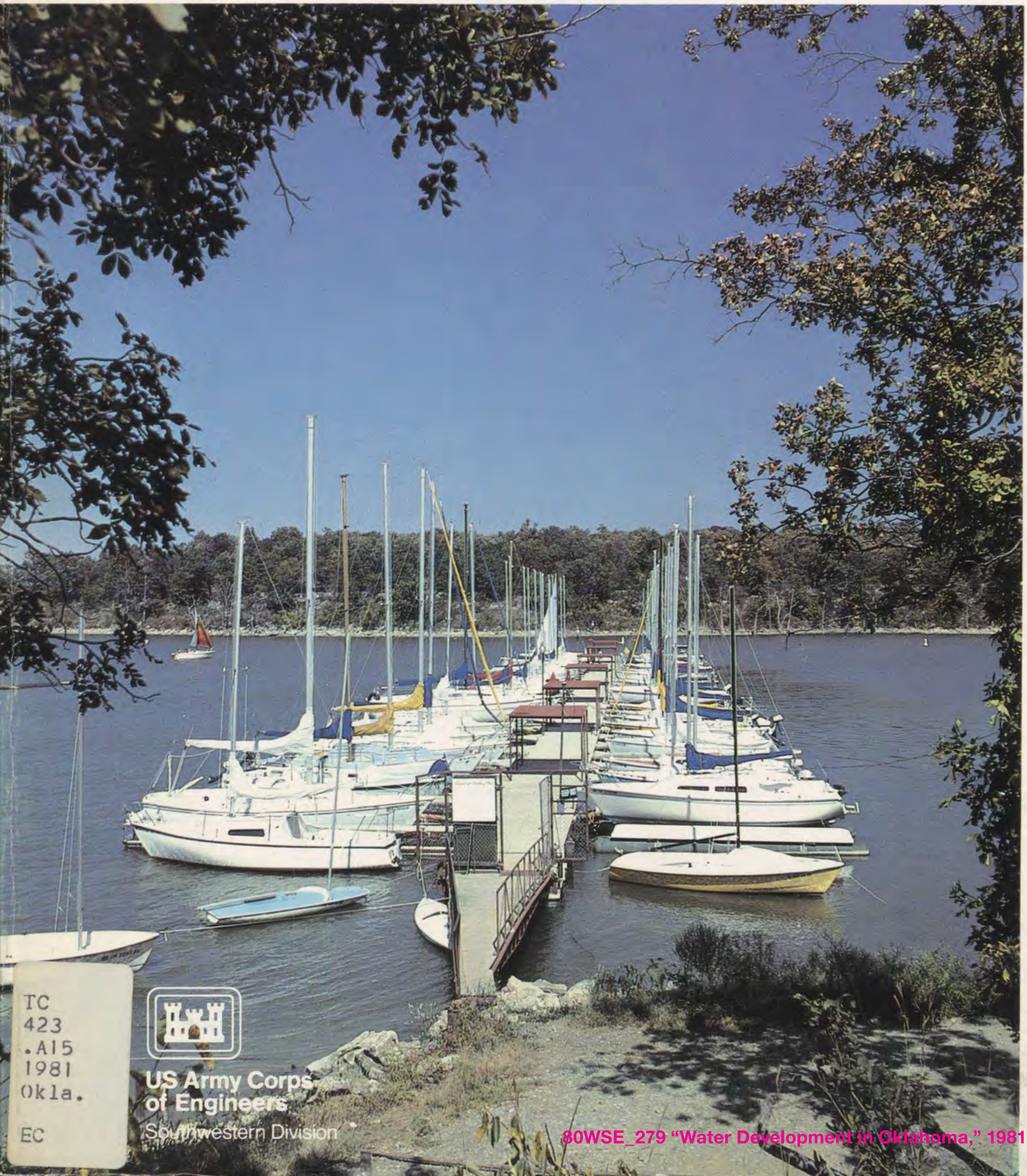
by the US Army Corps of Engineers



Army Corps
Engineers
Missouri River Division

Water Resources Development in Oklahoma 1981

by the US Army Corps of Engineers



US Army Corps of Engineers
Southwestern Division

A HUMMINGBIRD HOVERS OVER THE FLOWERS AT A EUFAULA PARK

Eufaula Lake is smack in the middle of one of the country's biggest flyways for snow and blue geese. The acorn-rich bottom lands yield excellent mallard duck shooting. The bushtails are thick, both the fox squirrels and the lightning fast grays. The bobwhite is at home around the lake, and the other game species sought by the sportsman include deer, cottontail rabbit, and mourning dove. Habitat improvement at Lake Eufaula will provide additional recreational opportunities for hunters, hikers, naturalists and birdwatchers. By planting food plots, controlling cattle grazing, and revegetating of the construction scars, all wildlife species will benefit.

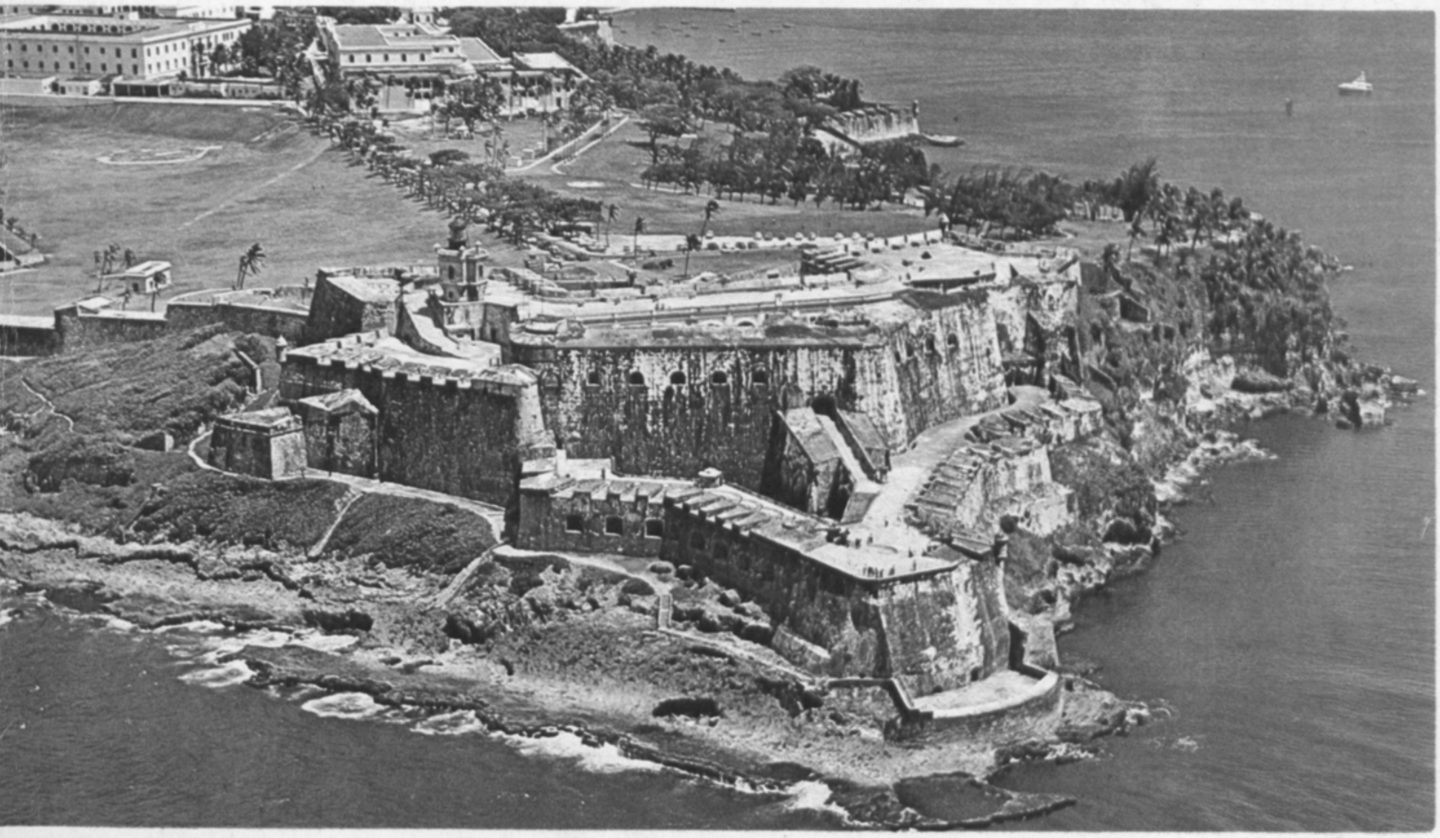
Eufaula Lake's rugged borders create numerous arms, coves and land points, some with steep banks, some ending in peaceful meadows. The scenic drives around the lake are especially beautiful in early spring, when flowering shrubs and leaf buds on the hickory and blackjacks lend a soft glow to the landscape. Again, when the fall colors start unfolding in the hills and valleys, dense with the variety of oaks and other hardwoods, the beauty of "Eufaulaland" is hard to match. Impressive views of the lake at almost every curve of the road makes a tour around the lake a memorable trip in any season.

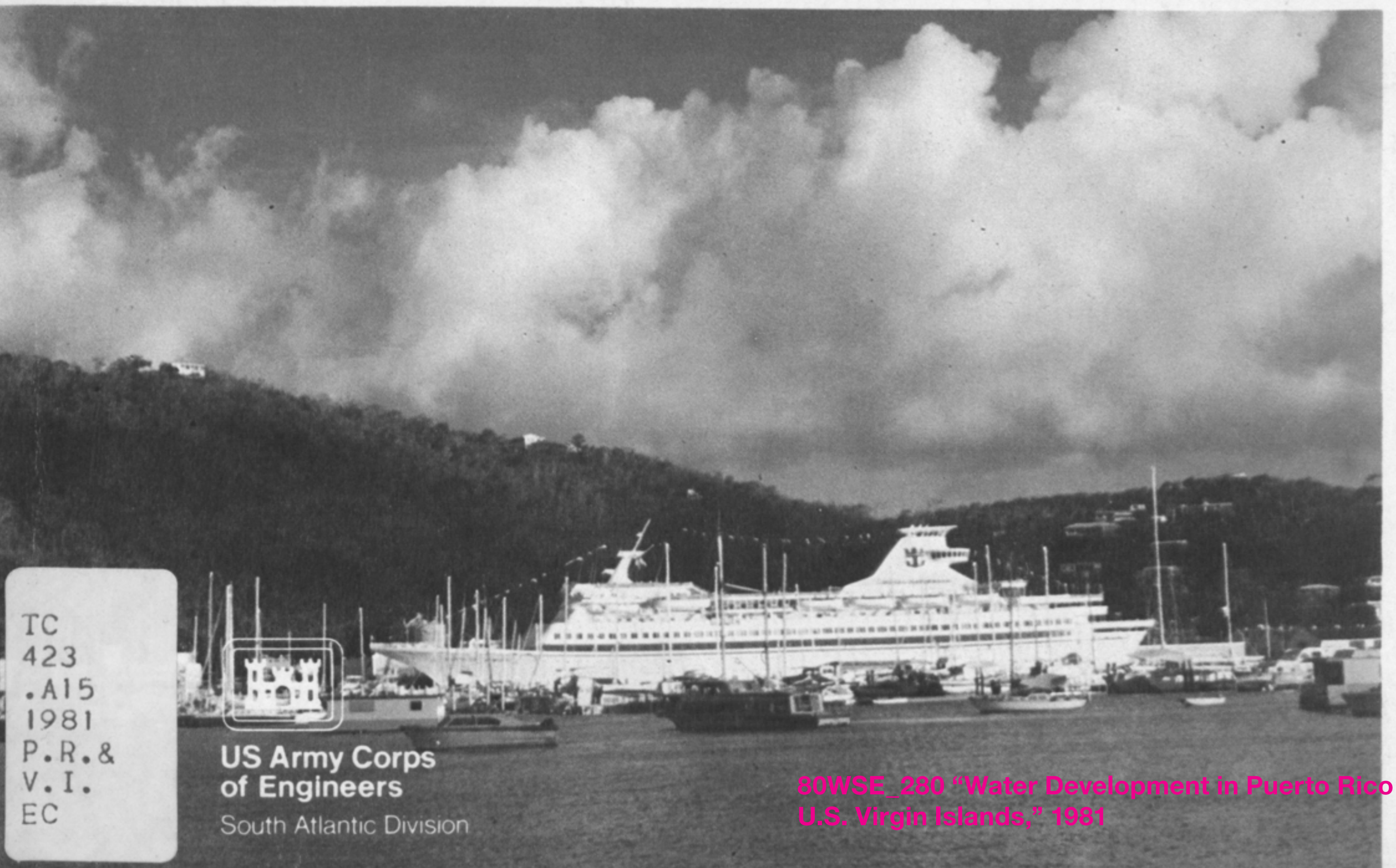
Additional Project Data

Dam: Rolled earthfill embankment with a total length of 3,200 feet including the powerhouse intake and concrete gated spillway. Maximum height of the structure above the streambed is 114 feet.
Power (Normal) Pool: Elevation 585.0 ft., contains 2,330,000 acre-feet for powerhead and water supply storage with a surface area of 102,200 acres.
Flood Control Pool: Elevation 597.0 ft., with a surface area of 143,700 acres, and a storage capacity of 1,468,000 acre-feet for controlling runoff from a contributing drainage area of 8,405 square miles.

Water Resources Development in Puerto Rico and the U.S. Virgin Islands 1981

by the U.S. Army Corps of Engineers





US Army Corps
of Engineers
South Atlantic Division

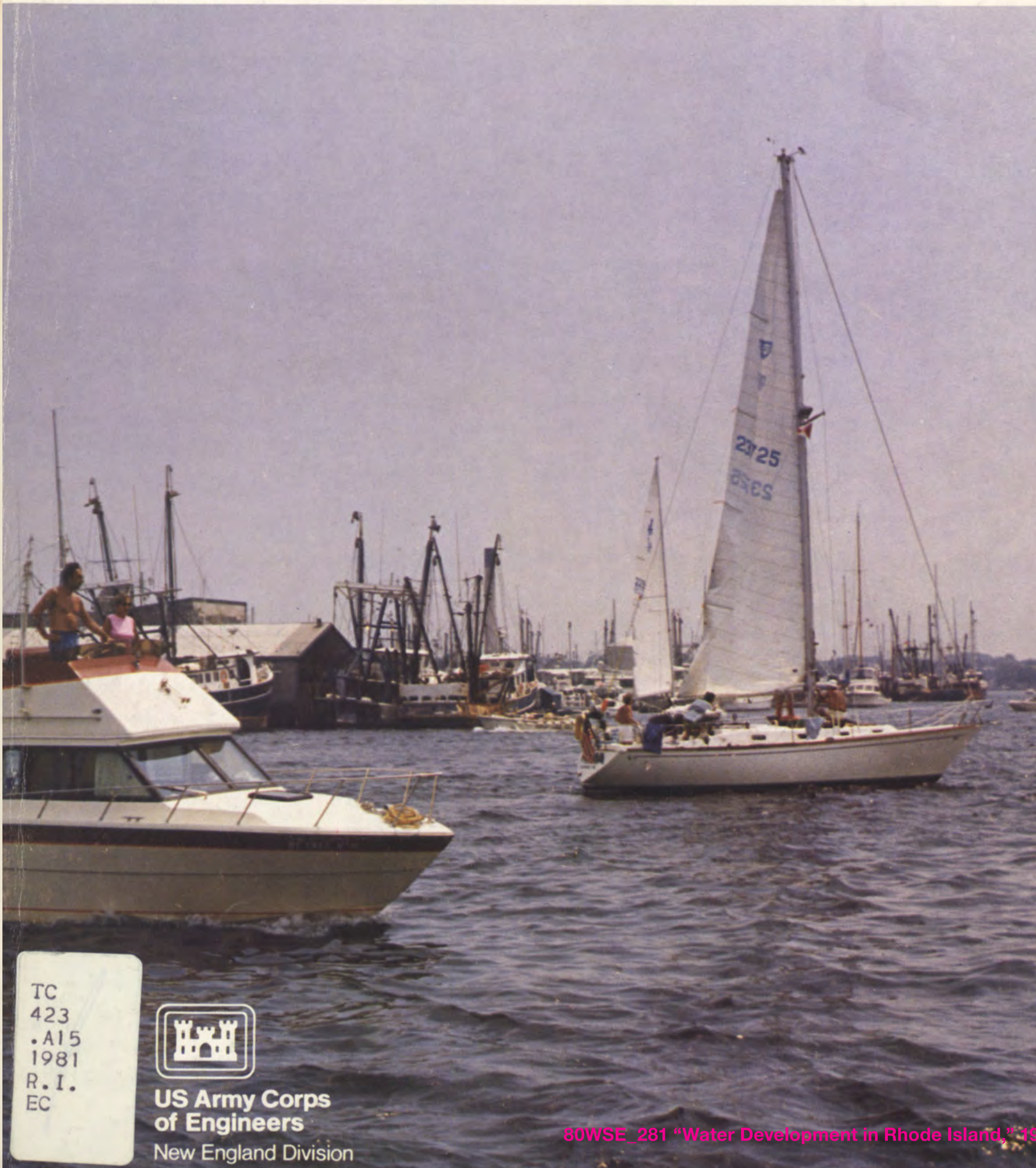
Small Flood Control Studies

Under Continuing Authority Section 205
Reconnaissance Reports

LOCALITY	PURPOSE	APPROXIMATE DATE TO BE COMPLETED
	Puerto Rico	
Naguabo	Flooding problems caused by Rio Santiago.	1980.
Sabana Grande	Extensive flooding by the Rio Guanajibo has affected new developments and industrial areas.	1981.
Penuelas Rio Tallaboa	Limited channel capacity and bridge structures have increased flooding problems on Rio Tallaboa.	1981.
	Virgin Islands	
Estate Mon Bijou St. Croix Christiansted	Large flood peaks cause major damages (55 to 60 homes) upstream watersheds with steep slopes and totally inadequate flood control facilities through Estate Mon Bijou.	Completed 1980.
La Grange	Large flood peaks cause extensive and frequent damages to the north section of Frederiksted due to inadequate flood control facilities.	1981.
Charlotte Amalie, St. Thomas	Savan Gut, a natural channel, floods the Jane Tutt School and a business district south of Back Street. Project to improve existing Savan Gut to reduce flooding of Jane E. Tutt School and the business district of Charlotte Amalie, St. Thomas in the Virgin Islands is under preparation.	1981.

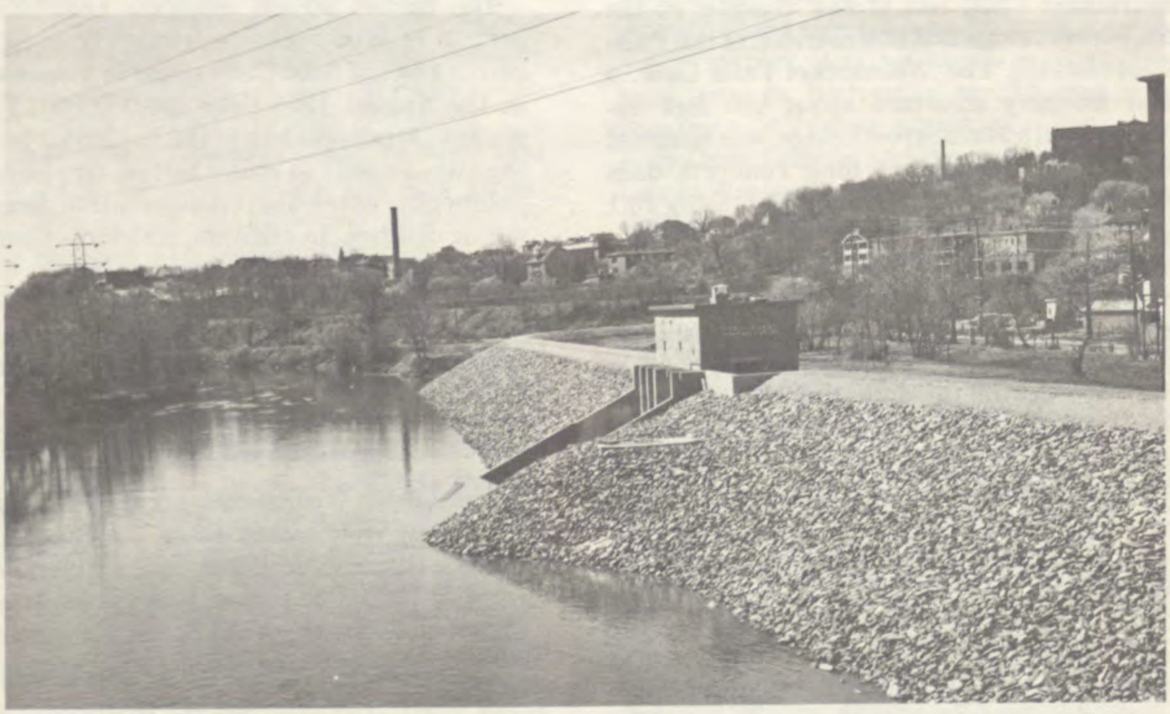
Water Resources Development in Rhode Island 1981

by the U S Army Corps of Engineers



US Army Corps
of Engineers
New England Division

Woonsocket Local Protection, showing effectiveness of tainter gates to reconstructed Woonsocket Falls Dam in reducing river stages during March 1968 flood.



Lower Woonsocket Local Protection, looking downstream at Blackstone River dike and Hamlet pumping station

Water Resources Development in South Carolina 1981

by the U.S. Army Corps of Engineers

US Army Corps
of Engineers
South Atlantic Division

Murrells Inlet jetties with Garden City in the background.

The cost of new work has been $1,786,100, and maintenance costs have totaled $5,860,500.

The harbor is used principally for shipping shellfish, clay products, nonmetalic mineral and residual fuel oil. Traffic total for 1977 was 173,488 tons.

Normally, only maintenance dredging by hopper dredge is required annually in the entrance channel. Contract maintenance dredging was performed in the inner harbor in 1969 for the first time since the project was completed.

Murrels Inlet, Georgetown County

Murrells Inlet is a small tidal inlet which cuts through a barrier beach on the coast of Georgetown County about 19 miles northeast of Georgetown and 13 miles southwest of Myrtle Beach. The inlet throat had become obstructed by extensive sand shoals, restricting passage of commercial and recreational vessels. The project, authorized by Congress in House Document No. 92-137 on November 13, 1971, provides a 10-foot entrance channel and an 8-foot inner channel for a total length of 3.5 miles. Rock jetties are provided on each side of the inlet. A weir section, built into the north jetty, permits littoral sands to pass over the jetty and into a deposition basin. Recreational facilities include a fishing walkway on the south jetty and a comfort station and parking area in Huntington Beach State Park. The walkway and parking area will be connected by a foot trail along the beach front. A contract for construction of the channels, jetty system, and fishing walkway was awarded in July 1977. All items of work in this contract, except for the deposition basin, have been completed as of June 1980. Excavation of the deposition basin is scheduled to begin in October 1980. Construction costs to date amount to $11,081,500.

A contract for the comfort station and parking area has been awarded and is scheduled for completion in April 1981.

Little River Inlet, Horry County

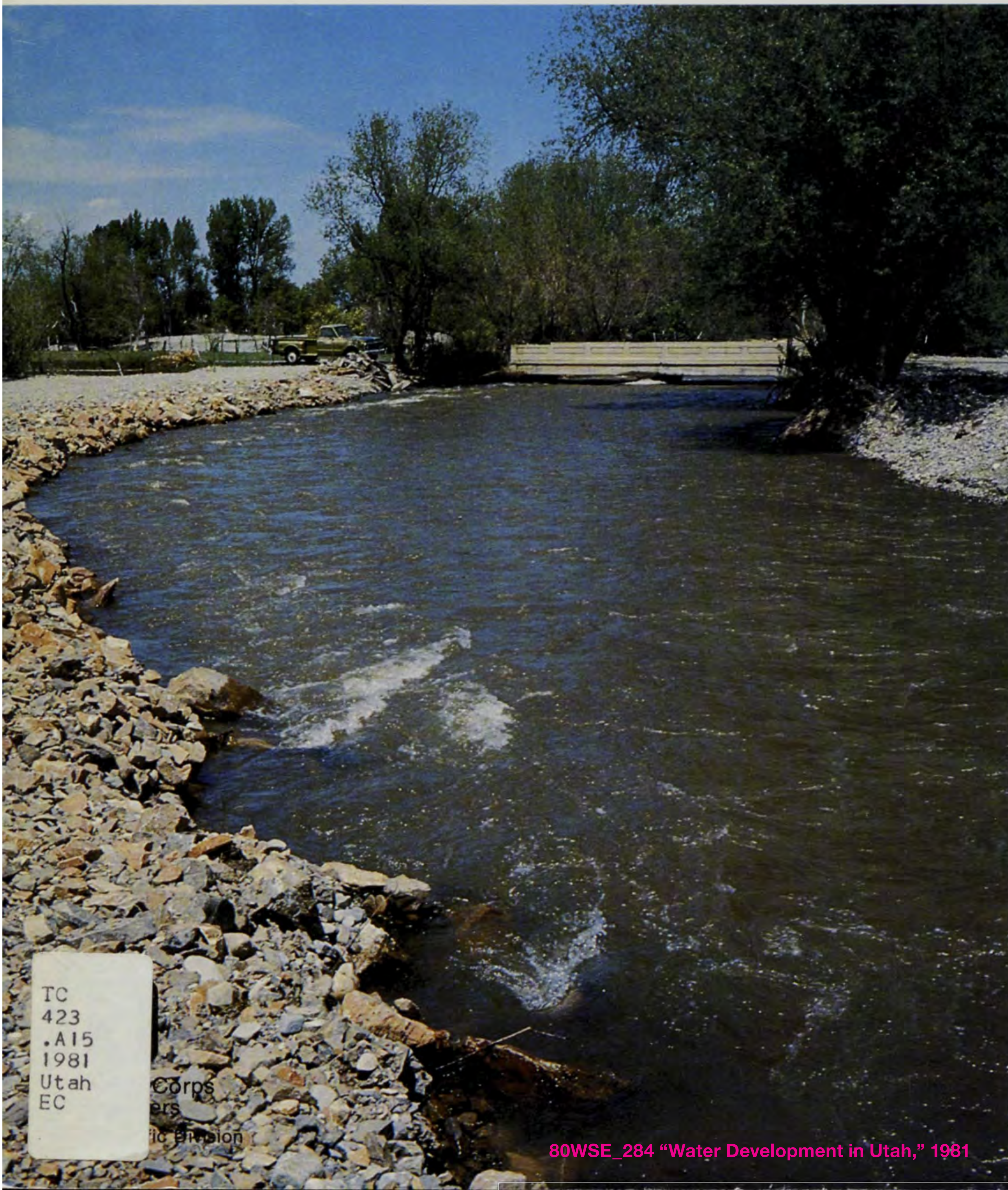
Little River Inlet is a small tidal inlet split by the State line. It lies in both Brunswick County, North Carolina, and Horry County, South Carolina. The inlet throat is generally obstructed by sand shoals, and the bar channel is unstable in depth and location, posing transit problems for charter boats and trawlers. The project, authorized on October 12, 1972, will provide an entrance channel 12 feet

Water Resources Development in South Dakota 1981

by the US Army Corps of Engineers

US Army Corps of Engineers
Missouri River Division

Water Resources Development in Utah 1981



Corps
ers
fic Division

Colorado-San Juan Basin

NAVAJO INDIAN RESERVATION, Arizona, New Mexico, and Utah (Los Angeles District)

A survey investigation at the Navajo Indian Reservation was authorized for flood damage prevention and allied purposes by section 176 of the 1976 Water Resources Development Act (Public Law 94-587).

The Navajo Indian Reservation, with a 1976 population of 149,000,[1] encompasses 26,000 square miles in Arizona, New Mexico, and Utah. The northern part of the reservation drains into the San Juan River, which is a major tributary to the Colorado River above Lees Ferry. The southern part drains into the Little Colorado River.

The investigation will assist the Navajo Indians in developing a comprehensive plan for water resources development in an extremely arid region. The Corps of Engineers will provide information on flood damage prevention and related water resources development problems, and will recommend, where justified, construction of specific projects.

During the flood of October 1972, several small earthfill dams in Arizona constructed by the Bureau of Indian Affairs failed. Also, severe damage occurred in several northeastern Arizona communities.

The investigation will begin when funds are made available.

VIRGIN RIVER NEAR ST. GEORGE, BANK PROTECTION (Los Angeles District)

Investigation of erosion and bank protection problems around bridge abutments, specifically in the vicinity of St. George, Washington County, was initiated in March 1980. The study was requested by county and state officials following flooding on the Virgin River in February 1980 that resulted in about $895,000 in damages ($401,000 to roads and bridges). One phase of the study, which is authorized under Section 14 of the 1946 Flood Control Act, as amended, includes investigating possible emergency flood control repairs to the Washington Fields, River Road, and Man-of-War bridges. The February 1980 flood caused about $73,000 in damages to the abutments of these three bridges, which span the Virgin River near St. George and provide access to nearby farmland and communities. Completion of the study is indefinite.

EMERGENCY WORK

The largest known flood in the Colorado-San Juan Basin occurred in September 1970 when heavy rains caused severe flooding on the lower reaches of McElmo Creek, on Montezuma Creek, and on the San Juan River. Industrial areas, utilities, and croplands suffered moderate flood damage. Two persons lost their lives, and damages exceeded $717,000. Emergency flood fighting and repair work under continuing authorities available to the Corps of Engineers totaled about $46,000.

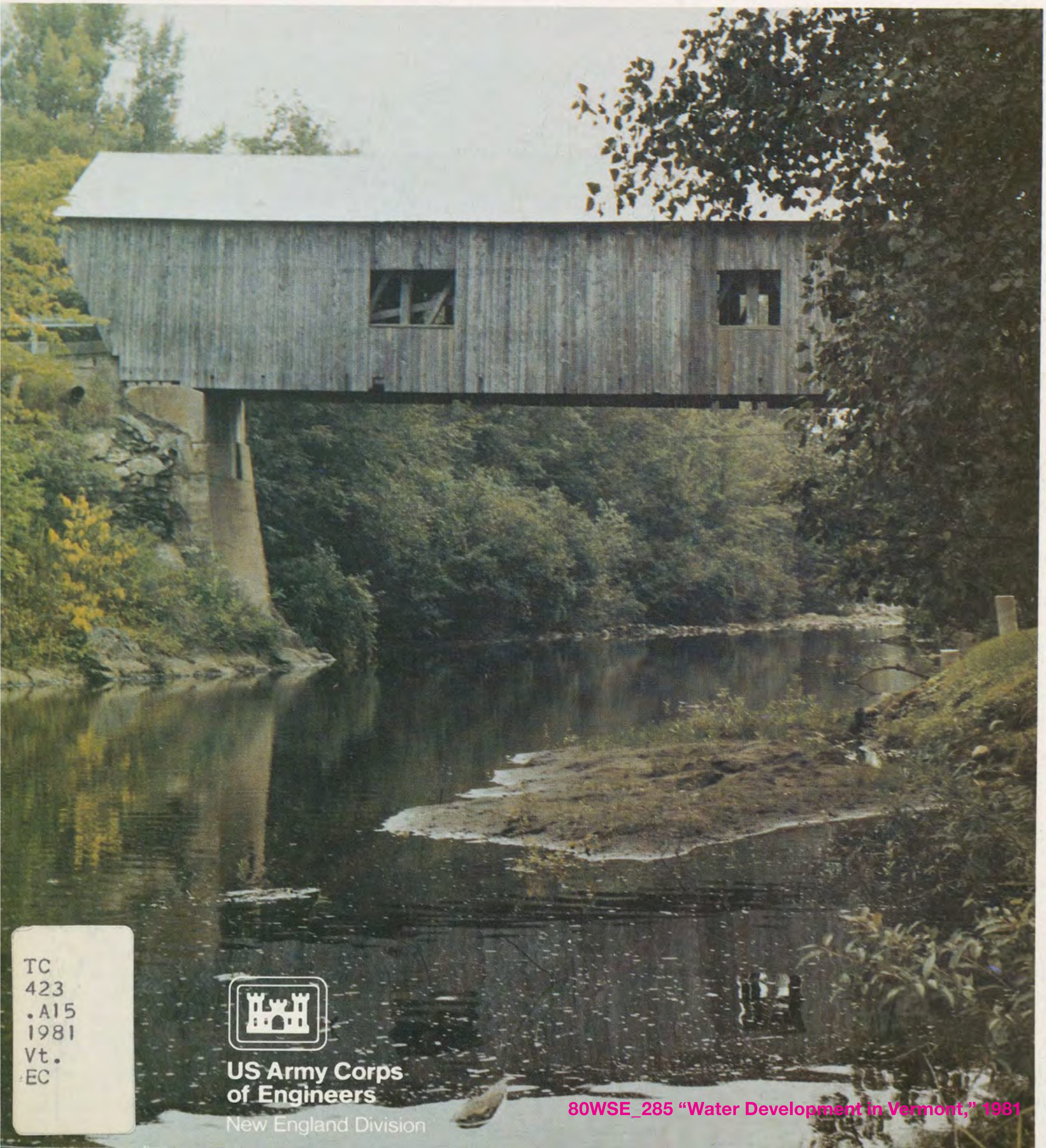
Emergency bank protection work, authorized by Section 14 of the 1946 Flood Control Act, was performed along a short reach of Mill Creek in Moab in 1978 at a cost of $55,000. Total costs for flood emergency work in the basin have totaled about $214,000.

Right bank abutment of State Highway 262 Bridge over McElmo Creek at Aneth was washed away during flooding in September 1970. (Desert News photo—Reed Madsen)

[1] *The most recent population figure—the 1980 census for the Navajo Indian Reservation not complete prior to going to print.*

Water Resources Development in Vermont 1981

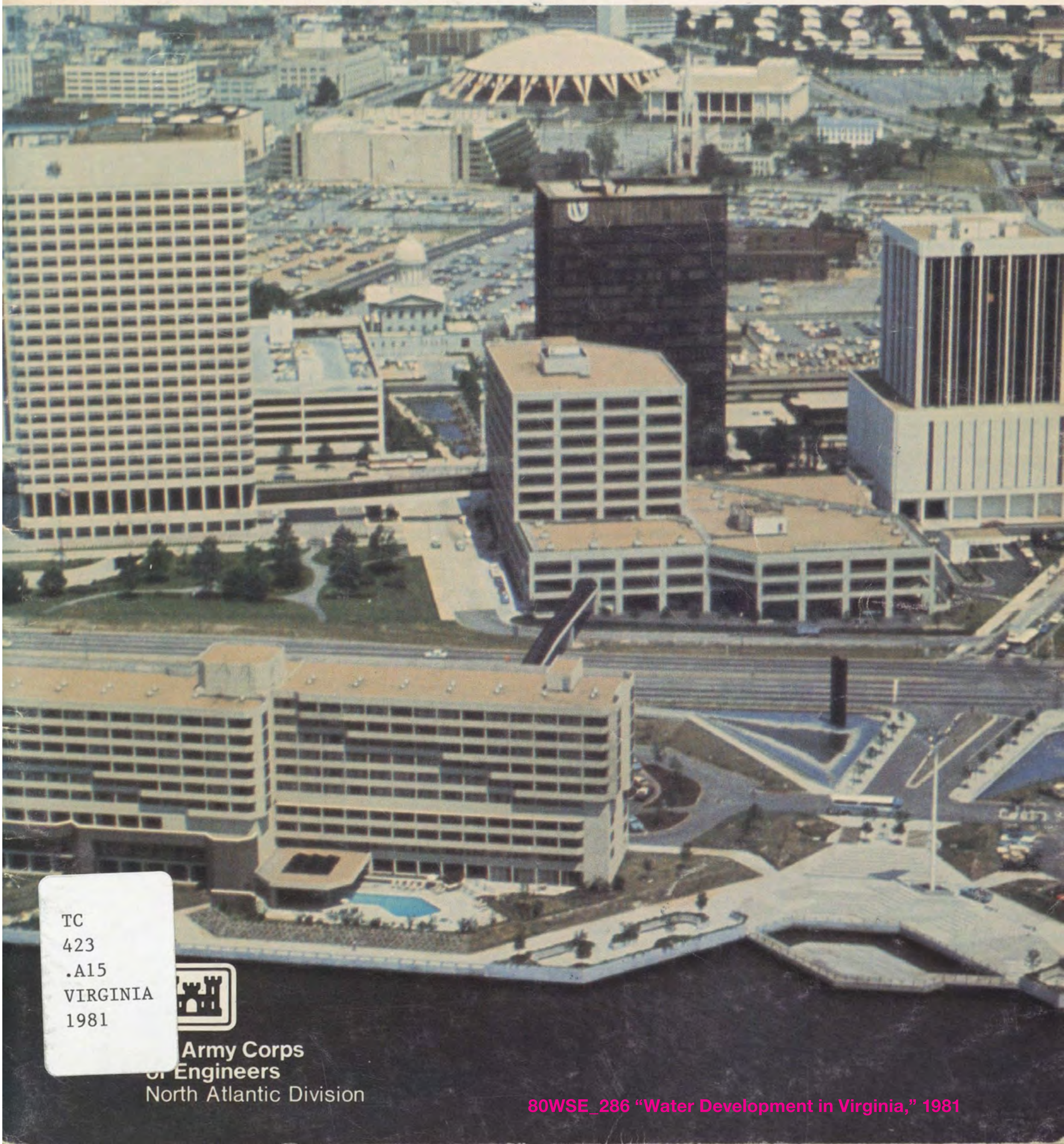
by the U S Army Corps of Engineers



US Army Corps
of Engineers
New England Division

Regulations of flows from Ball Mountain Dam provides "white water" for kayak and canoe races.

Water Resources Development in Virginia 1981

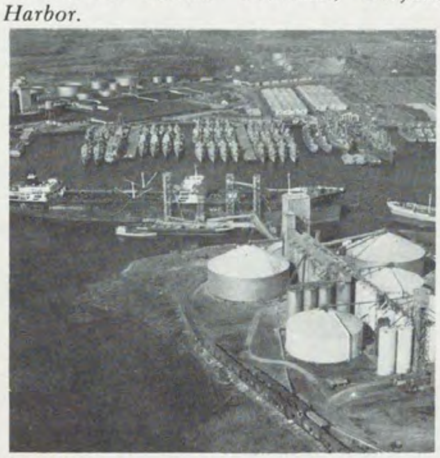


[US] Army Corps
[of] Engineers
North Atlantic Division

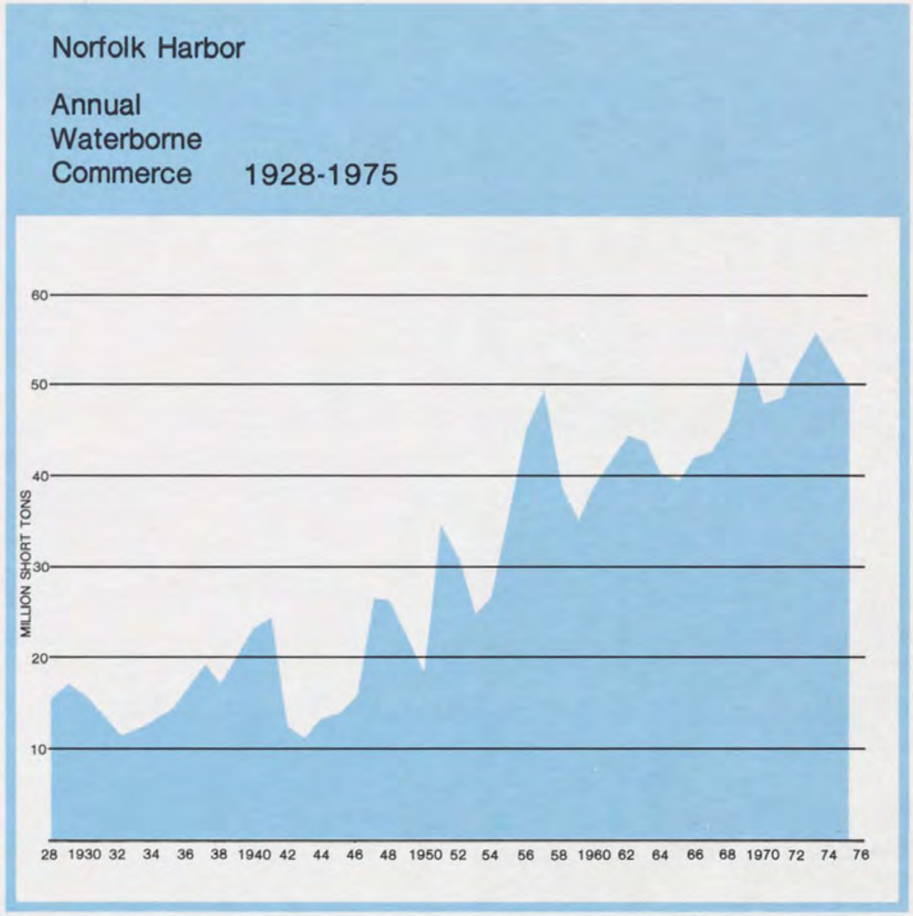
Norfolk Floodwall (first section) and Pumping Station.

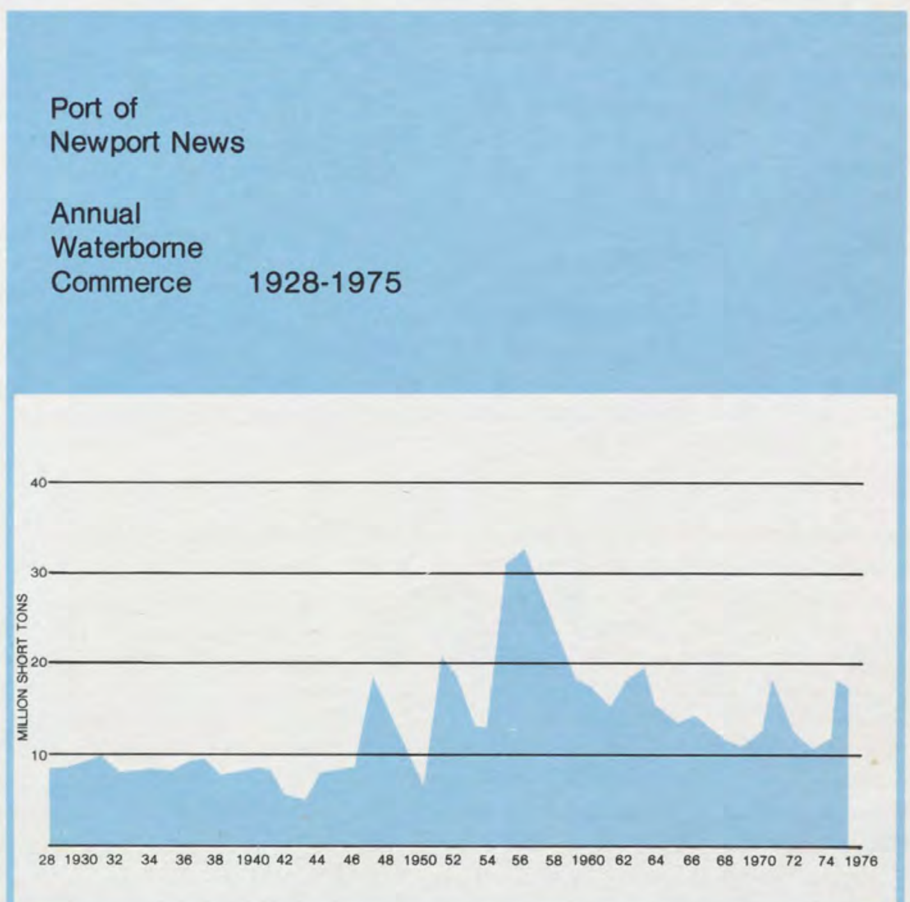
Norfolk International Terminal, Norfolk Harbor.

Portsmouth Marine Terminal, Norfolk Harbor.



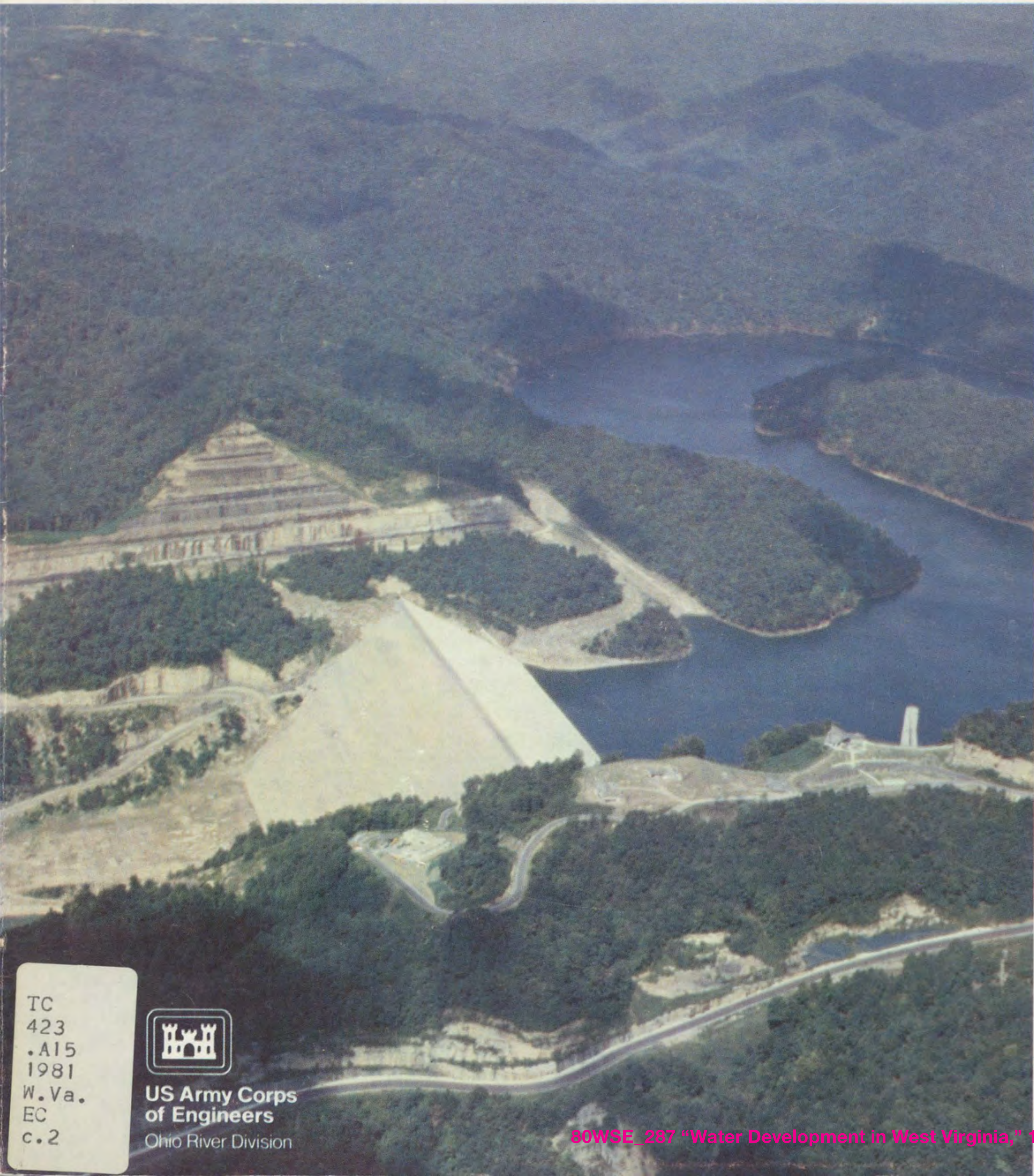
Vessel Loading Grain, Norfolk Harbor.





Water Resources Development in West Virginia 1981

by the U.S. Army Corps of Engineers



US Army Corps of Engineers
Ohio River Division

The committee gathered information and views from varied interests and held, in Cincinnati, on 14 June 1978, a Symposium on Winter Navigation at which reports of findings were presented. It was found that problems differed widely among the various locks and open-channel reaches, and that measures proposed were themselves generally subject to practical difficulties in their application.

Experience on the upper Mississippi indicated that navigation problems resulting from ice conditions are primarily industry problems and can be dealt with effectively mainly through industry collaboration under industry initiative.

A Corps operational procedure perceived to have prospectively useful effects on navigation conditions during periods of ice formation was the "bouncing" (varying the levels) of the navigation pools as a means of breaking up ice on the river and keeping it moving downstream.

Following from the committee's work, a reporting system has been organized to deal with ice conditions on the Ohio and to keep operators currently informed on these conditions.

A statistical summary of ice conditions on the Ohio River at Cincinnati for the ninety-year period from 1894 through 1964, prepared by the National Weather Service, shows that ice appeared on the river during sixty-two of the winters. The river was frozen over in thirteen of the ninety years.

Heavy running ice was experienced about every other year on the average, but the river froze over (at Cincinnati) on an average of one year out of every seven. In some years the duration of either condition was so short that overall impact on navigation was minimal.

HISTORY OF CORPS ACTIVITIES
Huntington, Pittsburgh, and Baltimore Districts

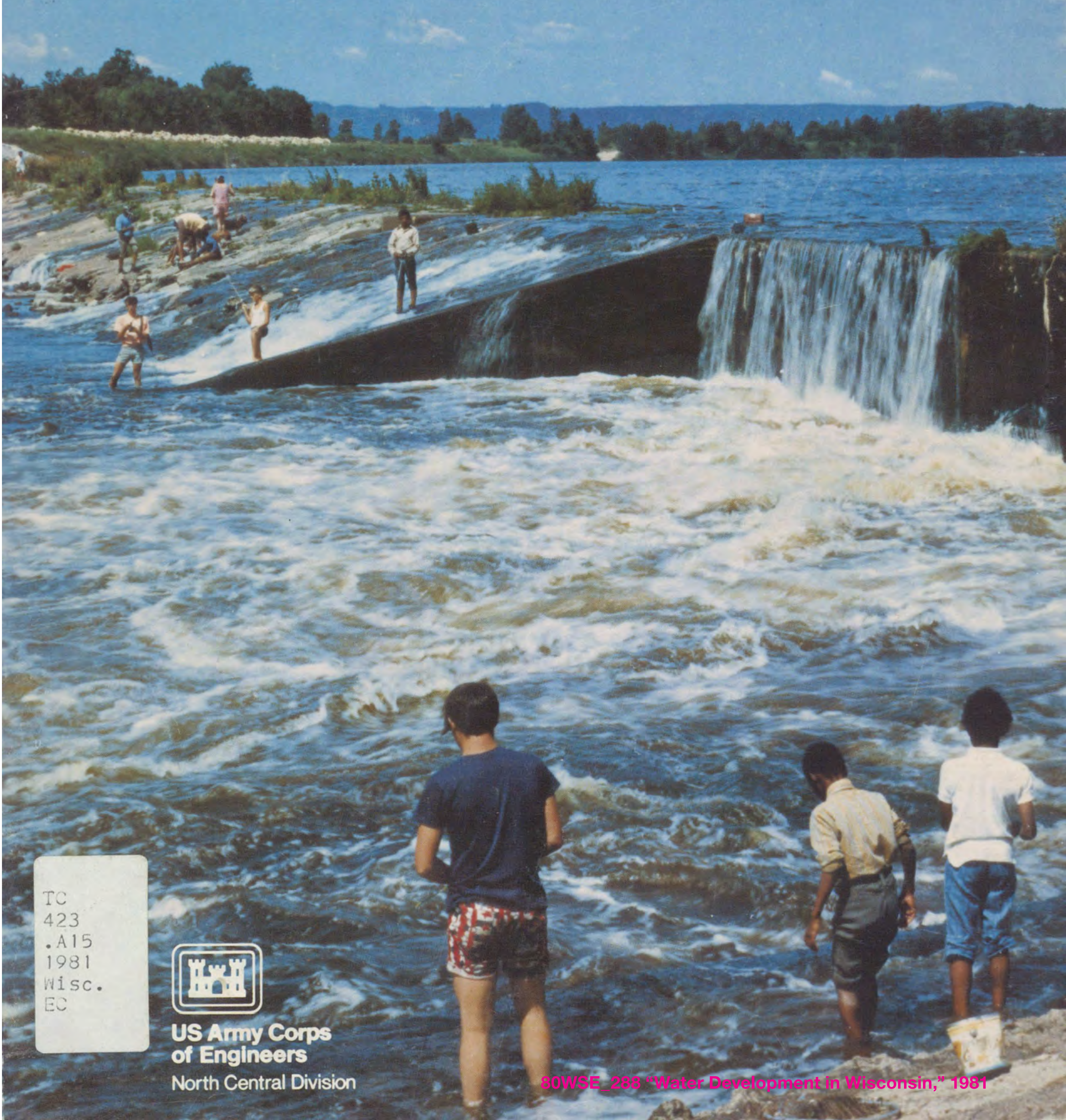
A program for the preparation and issuance of histories of Corps of Engineers field operating agencies was initiated by the Chief of Engineers in 1966. The first major history was that of the New Orleans District, published in 1971. Some forty histories have so far been issued, and the work on the other agency histories is in various stages of preparation, completion, or printing.

As relating to West Virginia, Corps histories have been published for the Huntington, Pittsburgh, and Baltimore Districts. The history of the Huntington District, entitled *Men, Mountains, and*

This placard, held by the Corps park aid who prepared the original, shows in color the common fish species found in Huntington District Corps lakes

Water Resources Development in Wisconsin 1981

by the U S Army Corps of Engineers



US Army Corps
of Engineers
North Central Division

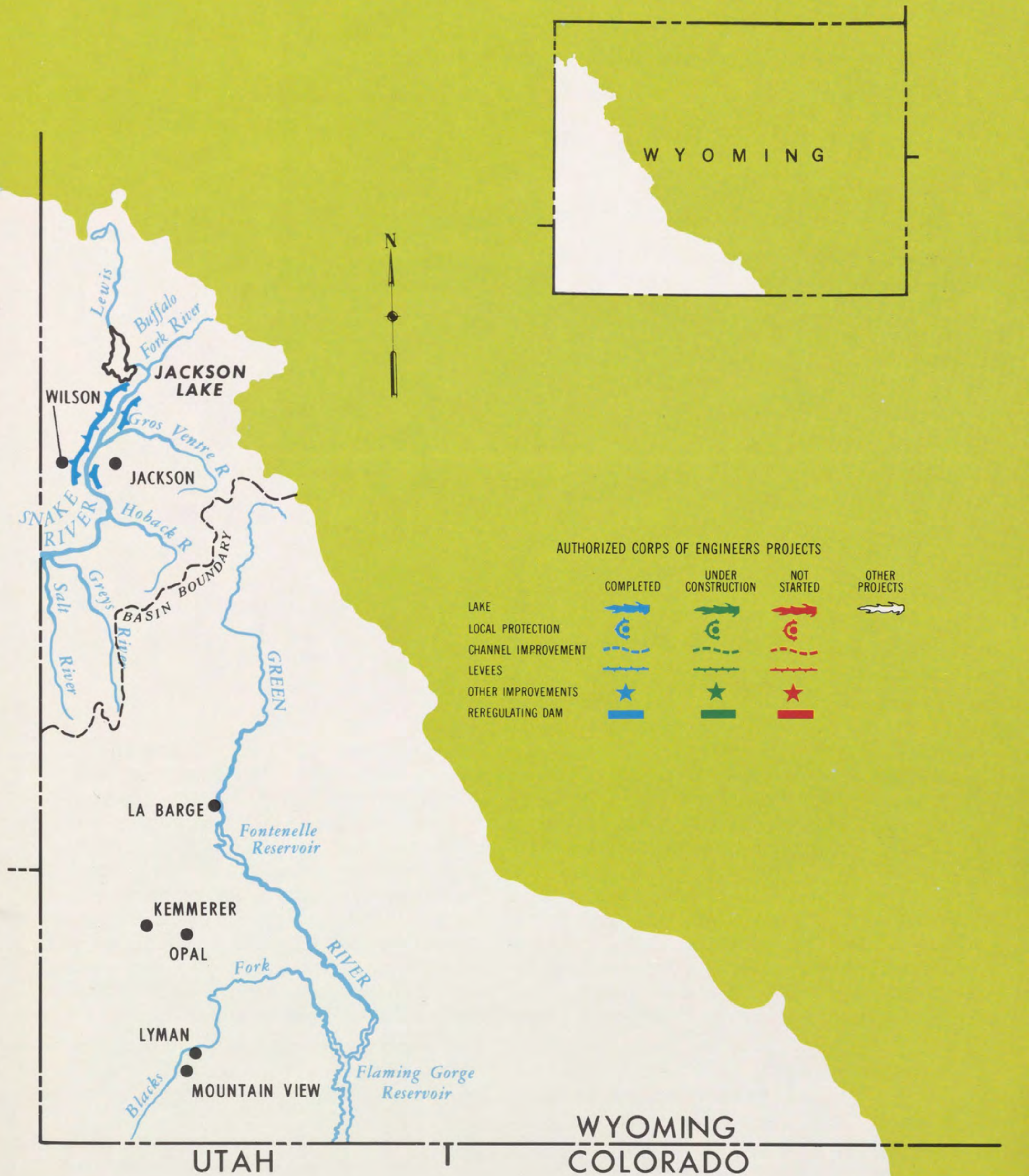
Meetings are held to inform the public of Corps studies and projects. Interested citizens are encouraged to participate in the planning process.

Several Corps of Engineers field sites sponsor ECO-EXPOZ, a program in which students receive tours and participate in educational games and exercises dealing with the environment surrounding the site.

Water Resources Development in Wyoming 1981

by the US Army Corps of Engineers

US Army Corps of Engineers
Missouri River Division



80WSE_289 “Water Development in Wyoming,” 1981

CLIMATE - MEAN TEMPERATURE

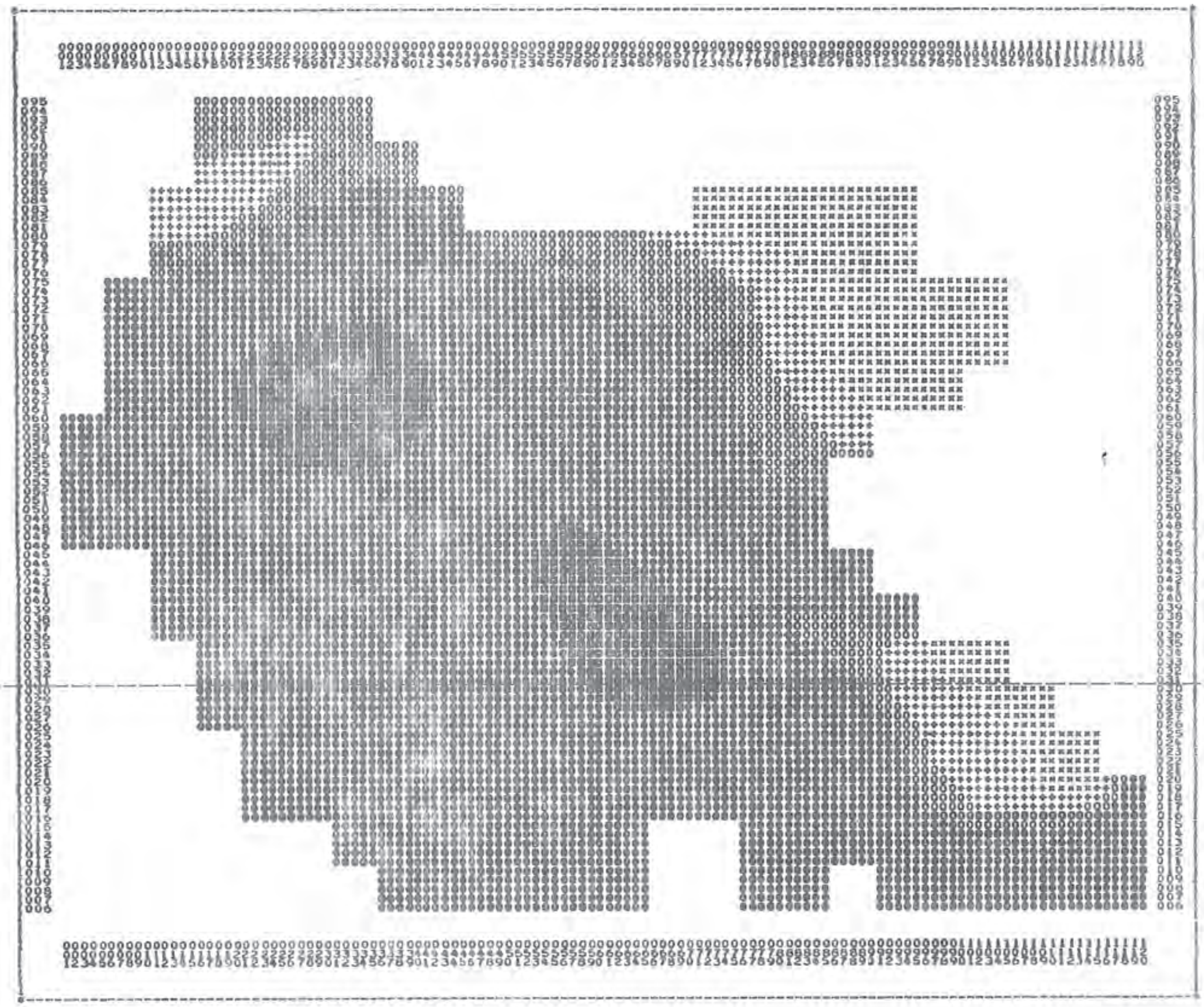
Legend:

2 = 40°-45°
3 = 45°-50°
4 = 50°-55°
5 = 55°-60°
6 = 60°-65°
7 = 65°-70°

Data Element Number: 47

Source: U.S. Weather Bureau Reports

Coding Method: Hand transfer of isolines to grid base map from USGS map
Accuracy: Best judgement
Problems: The U.S. Weather Bureau source had only 16 points of information in the area



SANTA ANA RIVER BASIN STUDY
CORPS OF ENGINEERS
LOS ANGELES DISTRICT

LEVELS	0	1	2	3	4	5	6	7	8	9
FREQUENCY	0	0	629	441	697	956	3897	580	0	0

OF REJECTED CELLS 0
OF CELLS LESS THAN ZERO 0
OF CELLS GREATER THAN NL 3600

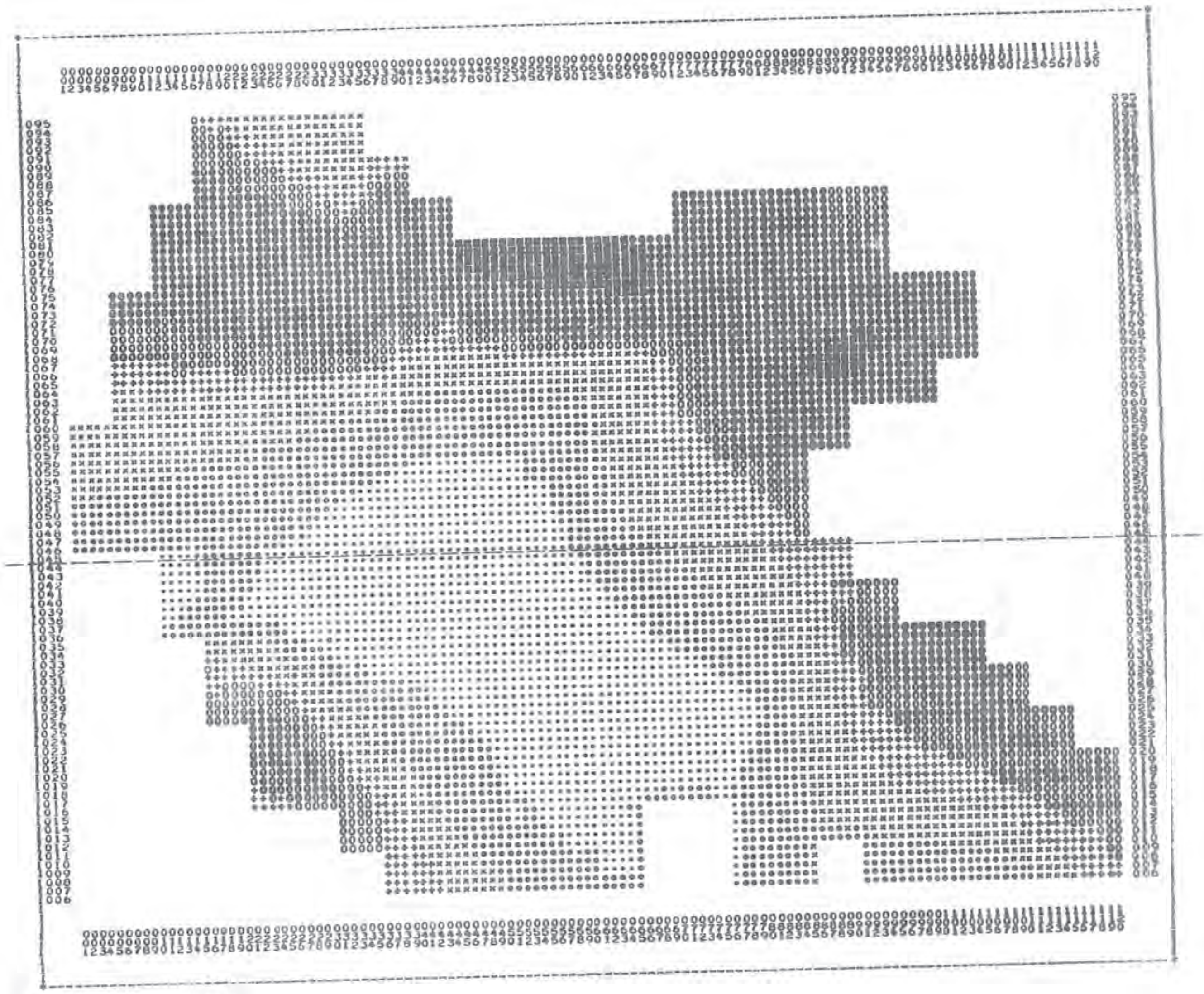
Figure 5.6.3 Climate - Mean Temperature

CLIMATE PRECIPITATION (MEAN)

Legend:	0 = 9.00-12.10 Inches/Year
	1 = 12.11-15.20 Inches/Year
	2 = 15.21-18.30 Inches/Year
	3 = 18.31-21.40 Inches/Year
	4 = 21.41-24.50 Inches/Year
	5 = 54.51-27.60 Inches/Year
	6 = 27.61-30.70 Inches/Year
	7 = 30.71-33.80 Inches/Year
	8 = 33.81-36.90 Inches/Year
	9 = 36.91-40.00 Inches/Year

Data Element Number: 9 (99)

Source:	Operations Branch, Corps of Engineers, Los Angeles District
Coding Method:	Hand transfer of isolines to grid base map from USGS map
Accuracy:	Within three grid cells
Problems:	Difficult to accurately interpolate lining up control points.



SANTA ANA RIVER BASIN STUDY
CORPS OF ENGINEERS
LOS ANGELES DISTRICT

LEVELS	0	1	2	3	4	5	6	7	8	9
FREQUENCY	1382	1457	1140	615	780	386	905	0	379	156

OF REJECTED CELLS 0
OF CELLS LESS THAN ZERO 0
OF CELLS GREATER THAN NL 3600

Figure 5.6.5 Climate Precipitation (Mean)

GROUNDWATER - TOP ELEVATION

Legend: Coded in hundreds of feet
Reduced for display as

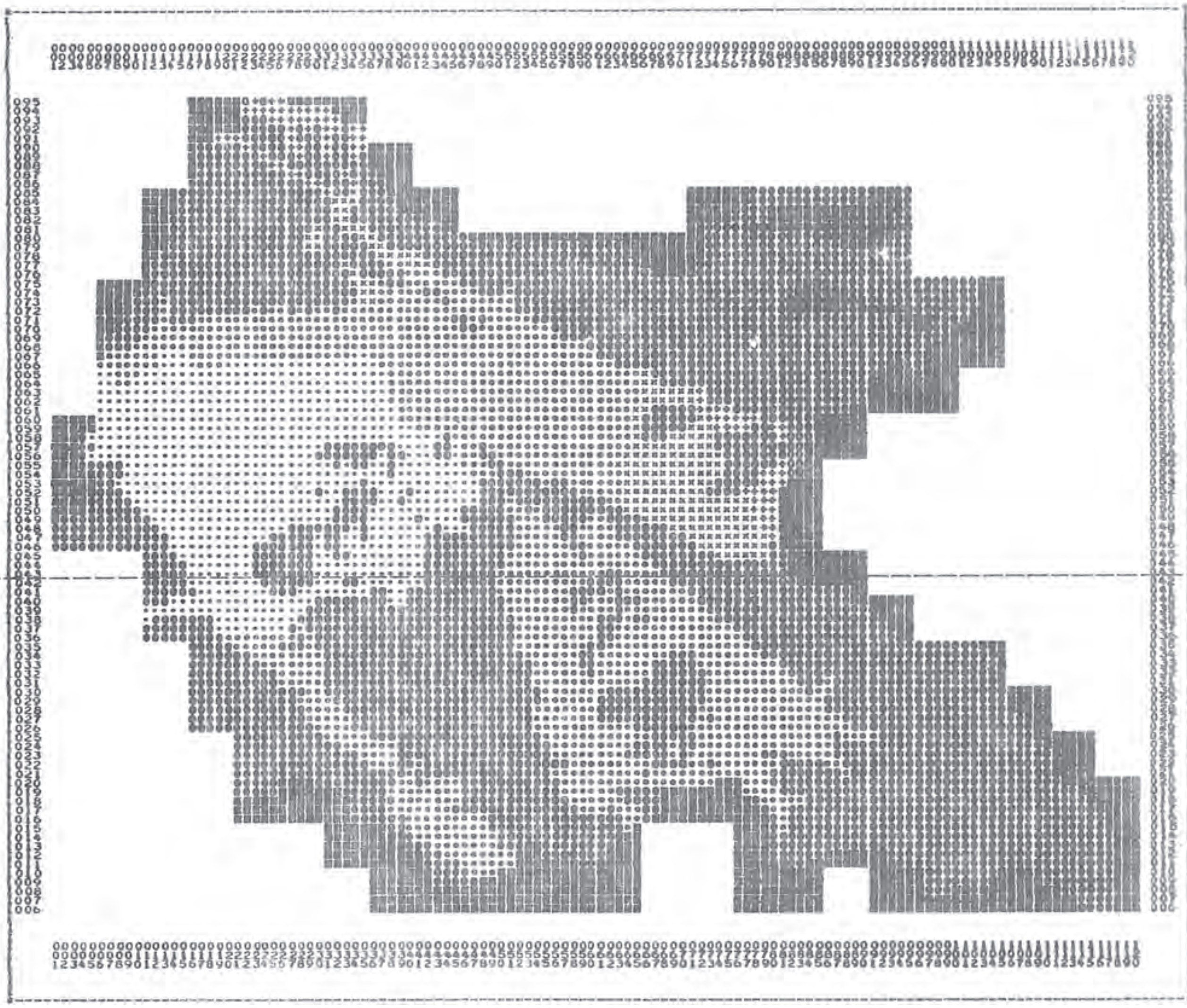
0 = 0- 880'
1 = 881-1760'
2 = 1761-2640'
3 = 2641-3520'
4 = 3521-4400'
7 = Water-bearing sediments
8 = Non water-bearing rocks
9 = No information

Data Element Number: 19

Source: Department of Water Resources - Geohydrology of Chino-Riverside, lines of equal elevation of groundwater in wells

Department of Water Resources - Santa Ana River investigation, lines of equal elevation of groundwater, Fall 1951 (1960); 1"= 2 Miles (1:126,720)

Coding Method: Grid overlay to source, centroid position
Accuracy: Within 2 grid cells
Problems: Maps too finely detailed; caused confusion



SANTA ANA RIVER BASIN STUDY
CORPS OF ENGINEERS
LOS ANGELES DISTRICT

LEVELS	0	1	2	3	4	5	6	7	8	9
FREQUENCY	795	1380	202	94	11	0	0	102	3495	1121

OF REJECTED CELLS 0
OF CELLS LESS THAN ZERO 0
OF CELLS GREATER THAN NL 3600

Figure 5.6.13 Groundwater - Top Elevation

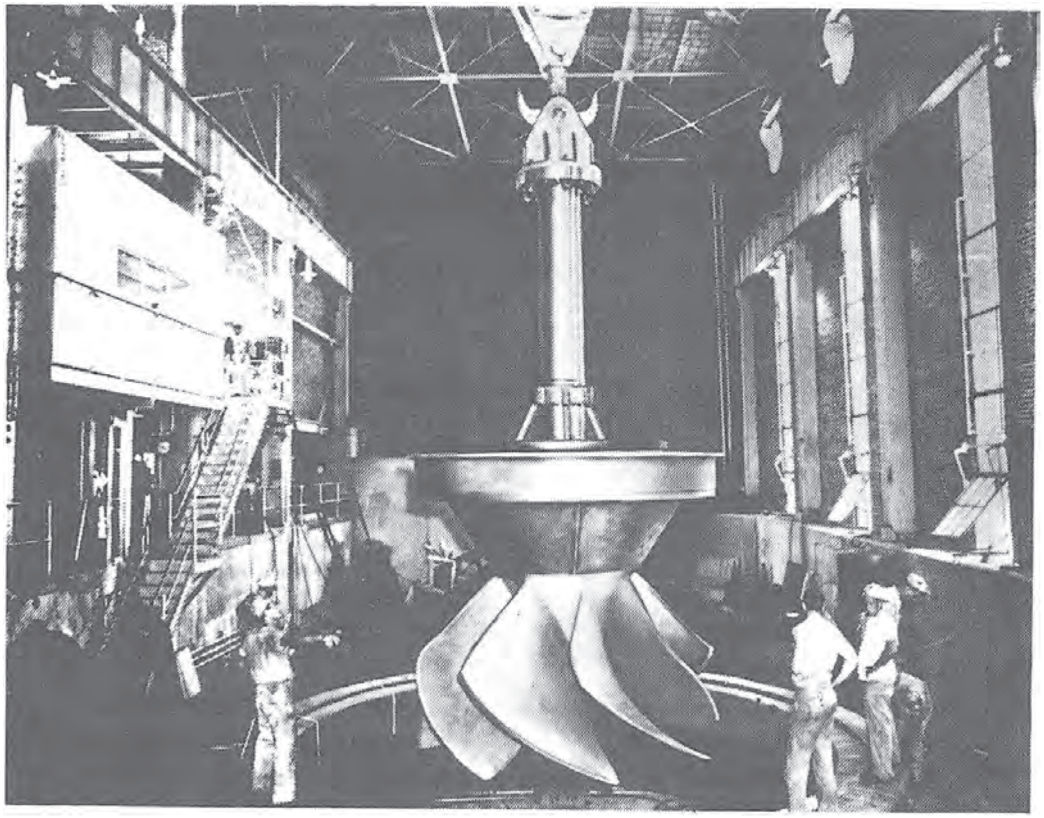


FIGURE A-4.3 —Installation of Head Cover and Runner Assembly for Yadkin Falls Plant

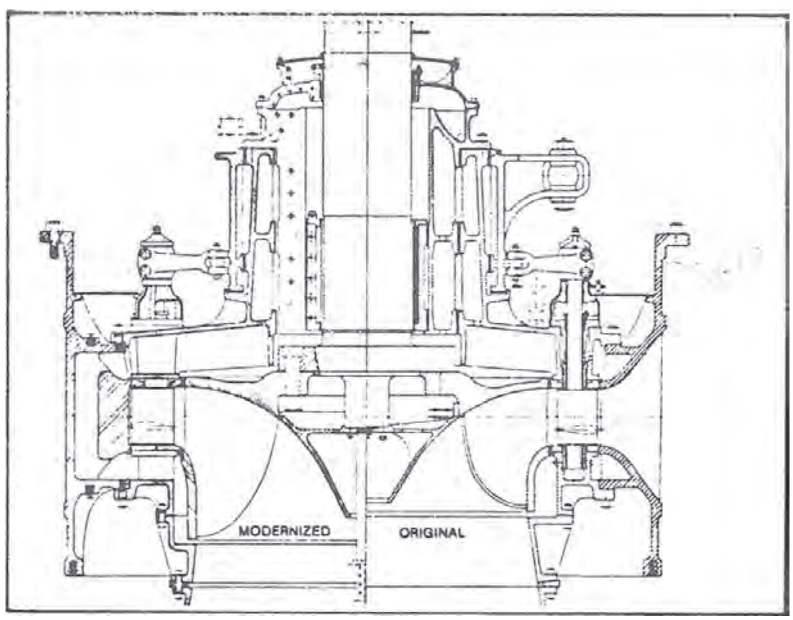
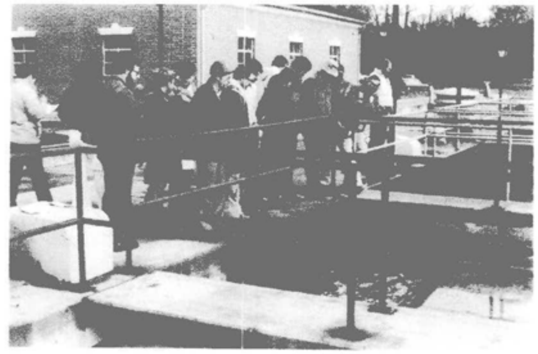
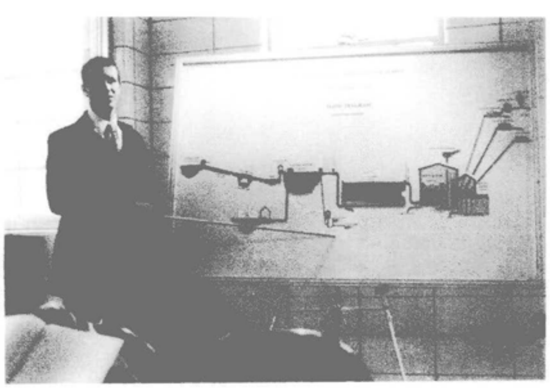


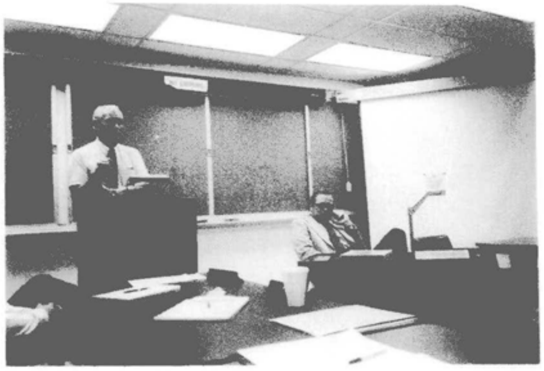
FIGURE A-4.4 Cross Section of Gorge Plant Hydraulic Turbine before and After Modernization

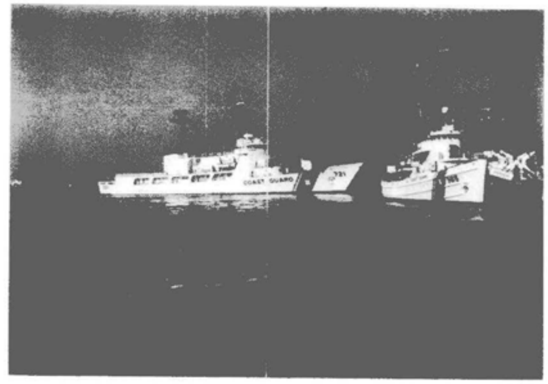
FIGURE A-4.6—One of the Replacement Stainless Steel Runners for Hoover Powerplant

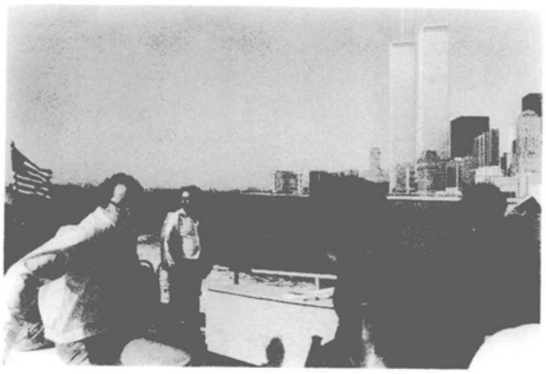


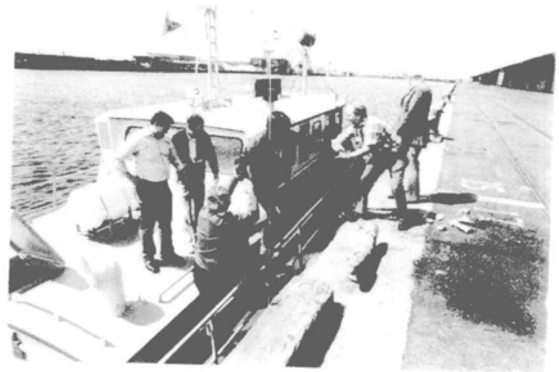


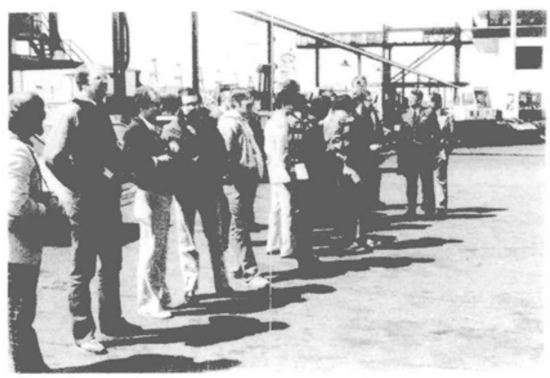
80WSE_293 “Annual Report: Planning Associates Program, Class of 1981,” 1981











80WSE_293 “Annual Report: Planning Associates Program, Class of 1981,” 1981

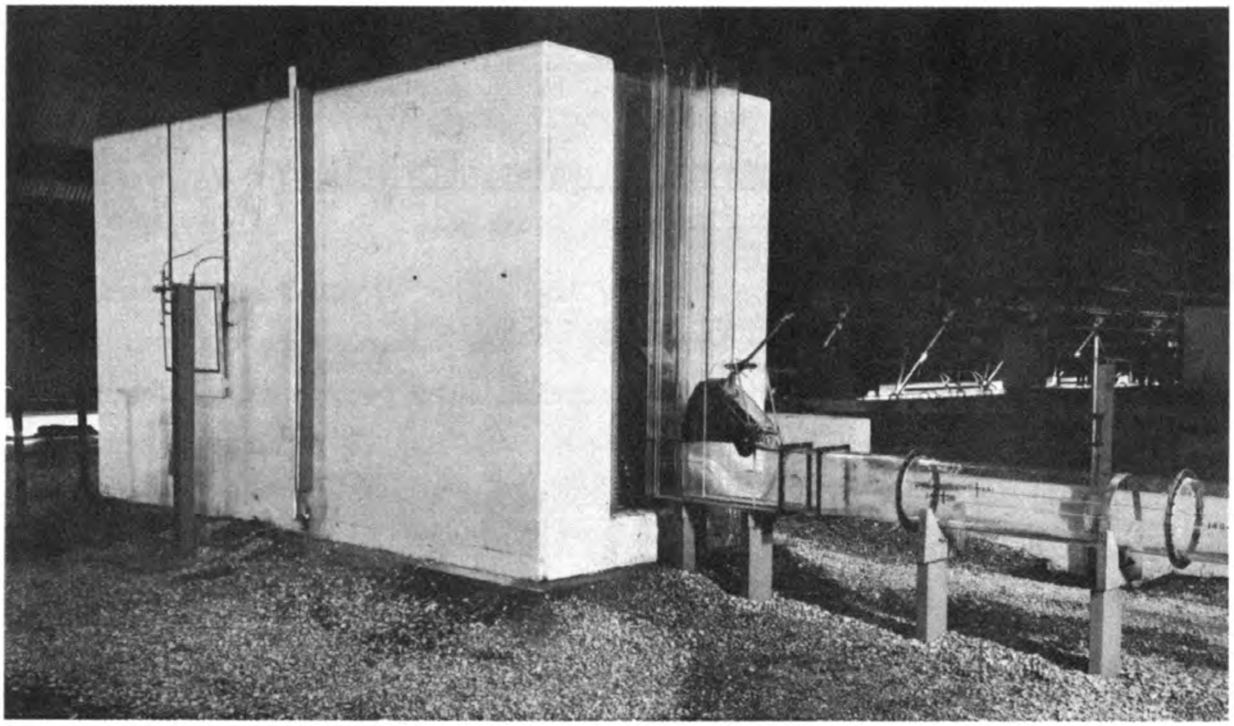


Fig. 3. Concrete headbay, plastic intake structure and section of tunnel used in flood-control tunnel tests

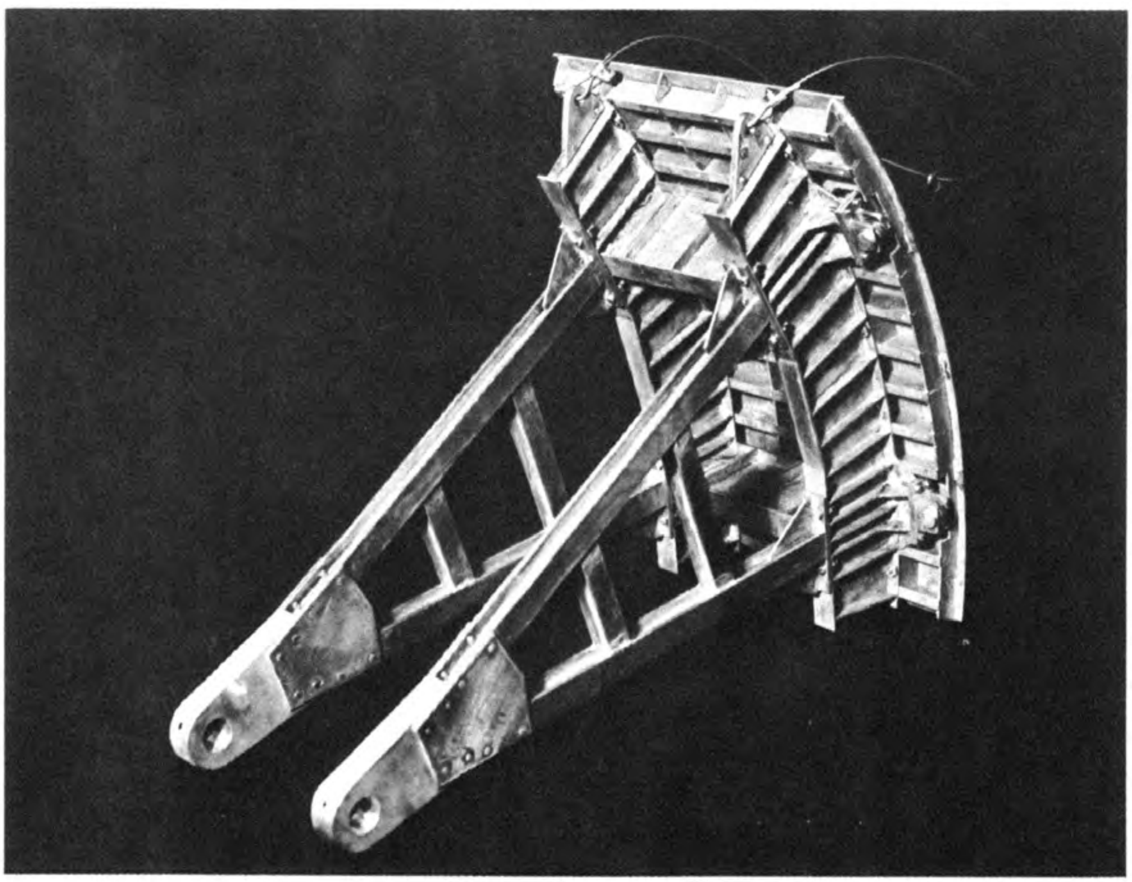


Fig. 4. Radial control gate for flood-control tunnel modeled in brass

actually rolling on ball bearings. The outlet transition was contained in a concrete flume and included about 225 ft of the upstream portion of the sloping apron of the stilling basin.

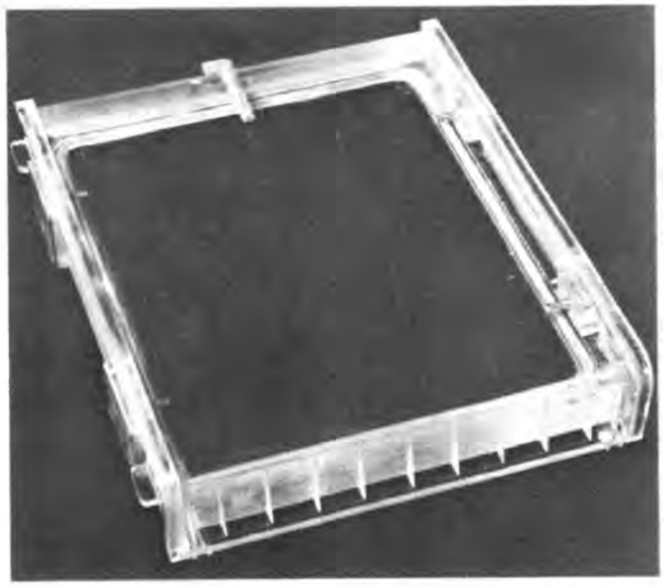


Fig. 5. Emergency gate for flood-control tunnel modeled in plastic

Apparatus

12. Water entering the model passed through a battery of venturi meters which permitted measurement of discharges. Piezometers were installed throughout the model for the measurement of pressures. Air entering the gate shafts was required to pass through a plastic pipe provided with orifice plates (fig. 6), which permitted measurement of the quantity of air being drawn into the tunnel. Forces on the model gates were measured both by mechanical and electromechanical methods (fig. 7). In the mechanical method no attempt was made to simulate the elastic properties of the prototype gate suspensions, and vibration amplitudes obtained using this method are only of qualitative value. However, since results could be obtained much more rapidly by the mechanical method, all preliminary tests were conducted in this manner. In the electromechanical method, which was used to obtain data on the final designs, the elastic properties of the prototype

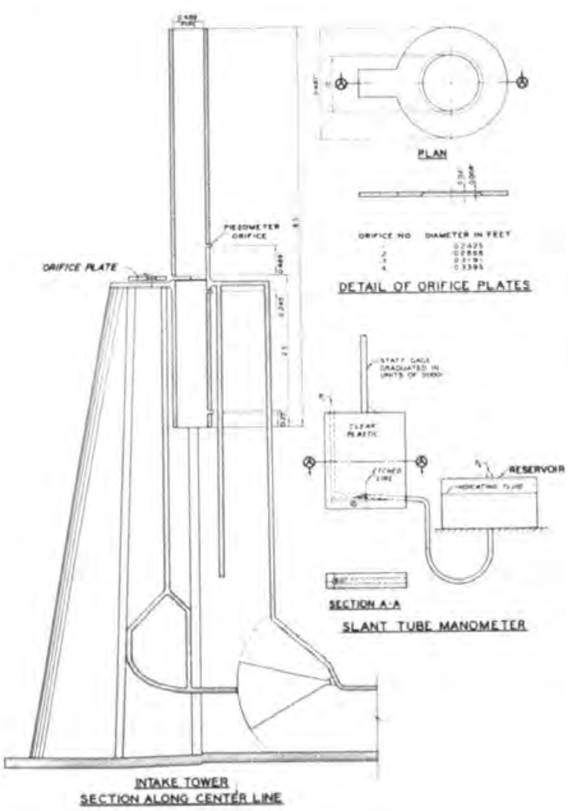


Fig. 6. Model air demand measuring apparatus

80WSE_294 “Outlet Works and Spillway for Garrison Dam, Missouri River, North Dakota: Hydraulic Model Investigation,” 1956 it was noted that the mechanical gage by which the downpull forces were measured showed a variation in load of about 20 tons for gate openings of 21 to 23 ft. Thus, at a 22.5-ft gate opening a maximum instantaneous downpull of about 35 tons was observed.

28. Type I-type IV cover plates. Since the downpull forces appeared to be caused by impingement of the eddy in the gate well on the exposed members of the gate, an attempt was made to reduce the downpull by streamlining the downstream side of the gate. This streamlining consisted of covering the downstream members of the original gate with skin plates to various degrees ranging from covering only the diaphragms (type I) to complete coverage (type IV); see fig. 12. Peak forces measured with the mechanical equipment were progressively reduced as the degree of coverage increased (plate 7). For the type IV gate a peak force of 14 tons was observed at a gate opening of 22.0 ft. Here again

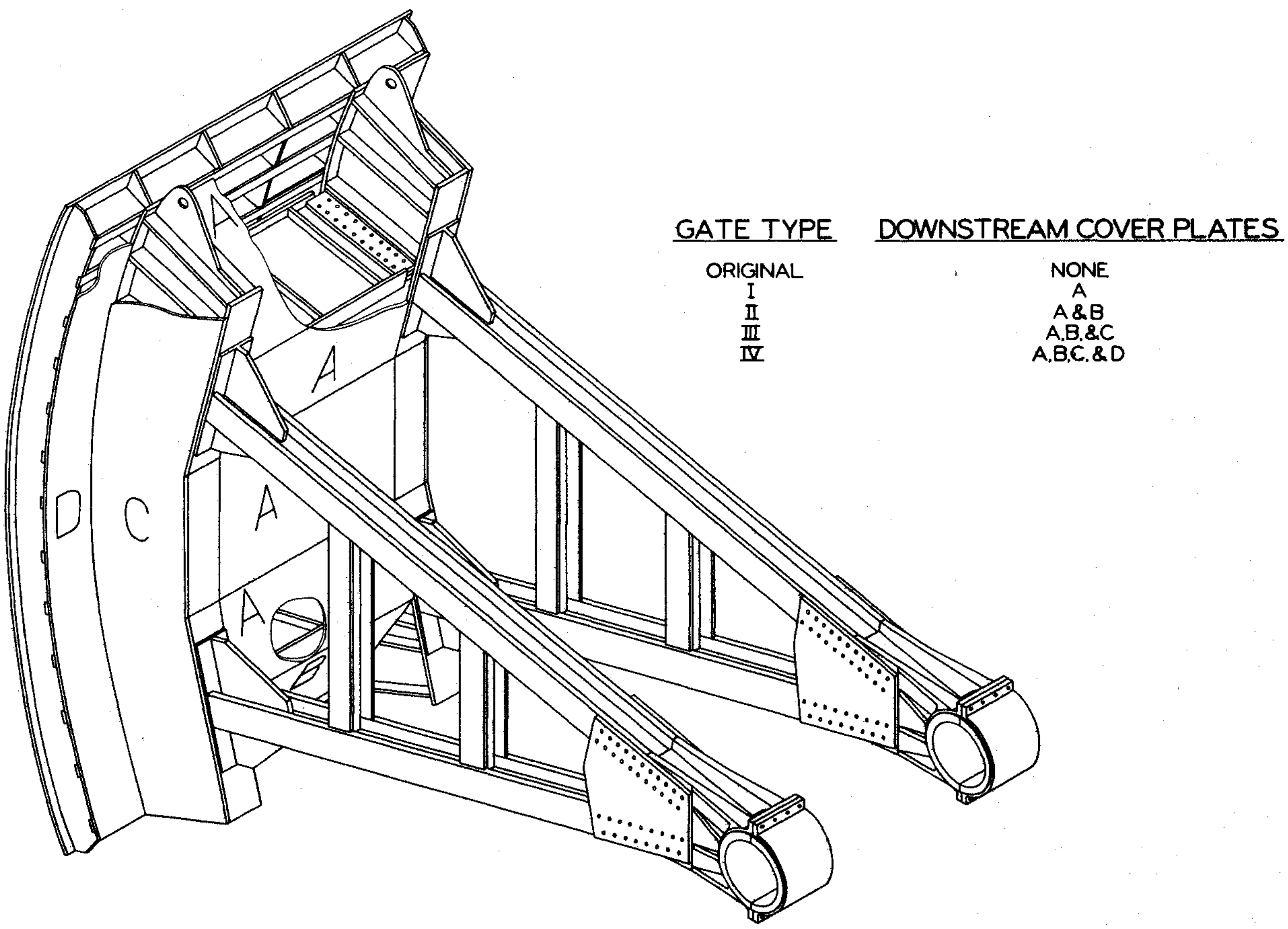


Fig. 12. Tainter type service gate with various downstream cover plates

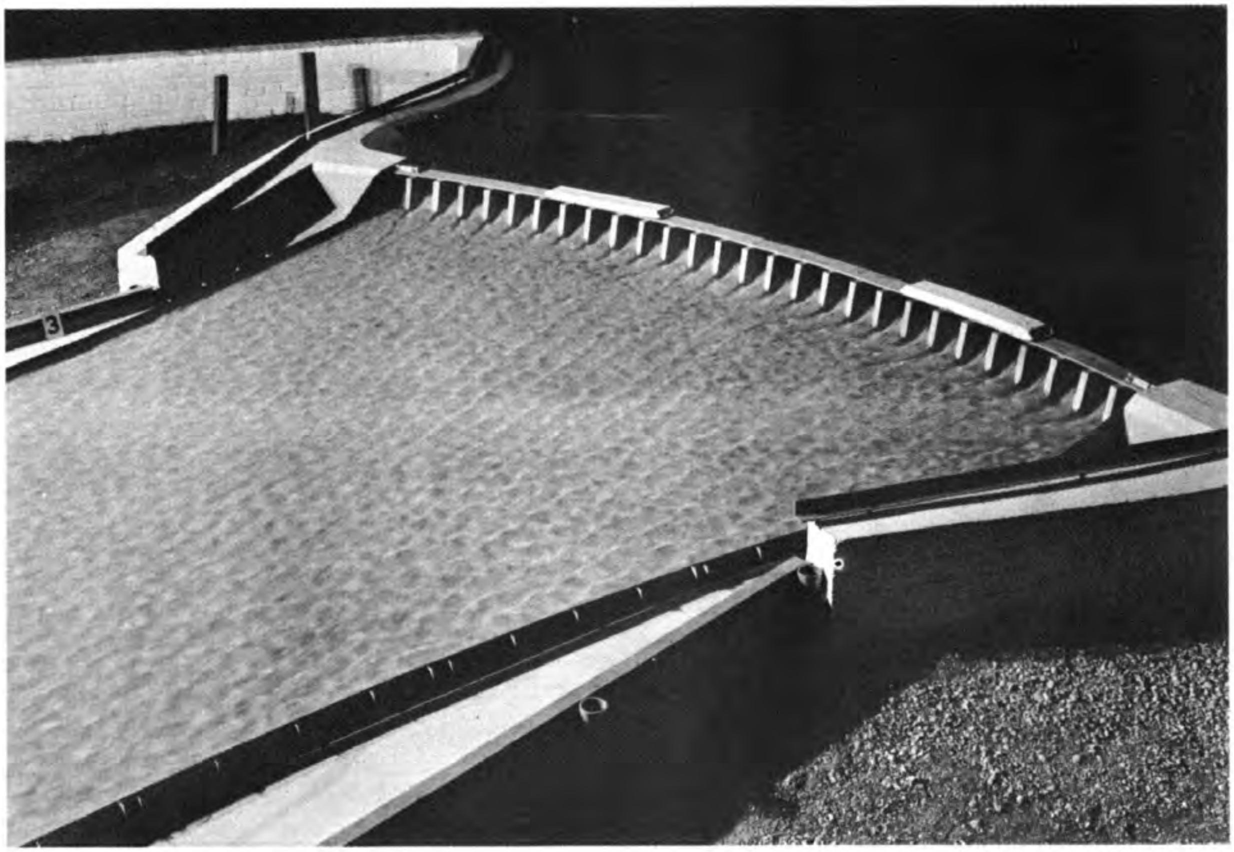


Fig. 22. Flow over spillway chute. Note diamond pattern of waves formed by piers

Stilling Basin

102. Engineers of the Garrison District stated that the stilling basin should perform well with the tailwater at ultimate retrogression (plot D, plate 64) and that its performance must be acceptable with the tailwater 5 ft less than ultimate retrogression. Acceptable performance was considered to obtain so long as the jump action remained in the stilling basin; spray action (jump swept into exit channel) was not considered acceptable.

103. Measurements in the 1:100-scale model of the energy of the flow in the chute immediately above the stilling basin revealed slightly greater losses in the model than had been computed. In order to assure that test results were on the safe side, model discharges were increased to accurately reproduce the computed pressure-momentum in the flow at

Table 1

STILLING-BASIN DESIGNS INVESTIGATED

Discharge, 152,000 cfs
Gate Opening, 8 ft

Upper Pool, 664.5
Tailwater, 658.55

Basin Design	Length of Basin	Height of End Sill	Distance of Baffles from Spillway Toe* 1st Row, ft	Distance of Baffles from Spillway Toe* 2nd Row, ft	Height of Baffles 1st Row, ft	Height of Baffles 2nd Row, ft	Maximum Velocity, fps 1.0 ft Above End Sill	Maximum Scour Depth ft**
1	52.6	3	13.6	30.6	4	4	10.2	0.0
1A	52.6	3	13.6	----	4	-	12.7	No scour test
1B	52.6	3	----	----	-	-	16.0	7.0
1C	52.6	3	13.6	30.6	6	6	9.6	No scour test
1D	52.6	3	13.6	----	6	-	11.8	No scour test
2	46.6	3	11.6	26.6	4	4	10.2	No scour test
2A	46.6	3	11.6	28.6	6	6	9.6	No scour test
3	42.6	3	7.6	22.6	4	4	9.6	No scour test
3A	42.6	3	15.6	----	4	-	12.7	No scour test
3B	42.6	3	5.6	20.6	6	6	7.6	No scour test
3C	42.6	3	13.6	----	6	-	11.3	No scour test
4	37.6	3	7.6	22.6	4	4	7.6	1.0
4A	37.6	3	15.6	----	4	-	14.0	3.0
4B	37.6	3	5.6	20.6	6	6	7.6***	0.0
4C	37.6	3	13.6	----	6	-	12.3	No scour test
4D	37.6	3	----	----	-	-	16.0	5.0
4E	37.6	-	----	----	-	-	19.3	No scour test
4F	37.6	6	----	----	-	-	16.0	8.0

* Distances are measured from intersection of spillway and stilling basin to upstream face of basin elements concerned.

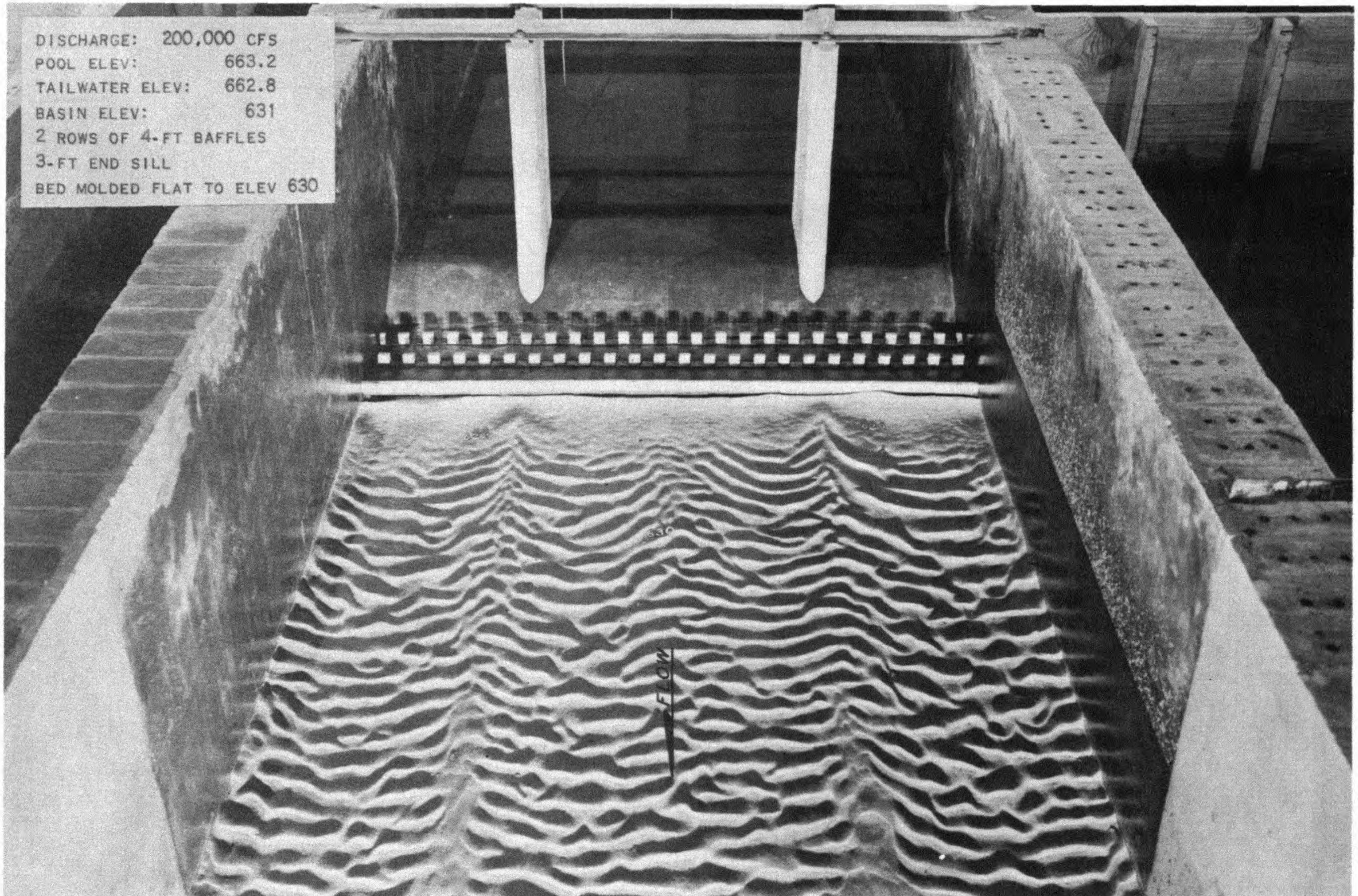
** For scour tests bed of exit area molded in sand to elevation 630

*** This velocity was obtained 4.0 ft above end sill; velocities were too low to be measured 1.0 ft above end sill

Notes: 1. Discharge based on flow through 2 bays multiplied by 6 to obtain discharge for entire spillway length

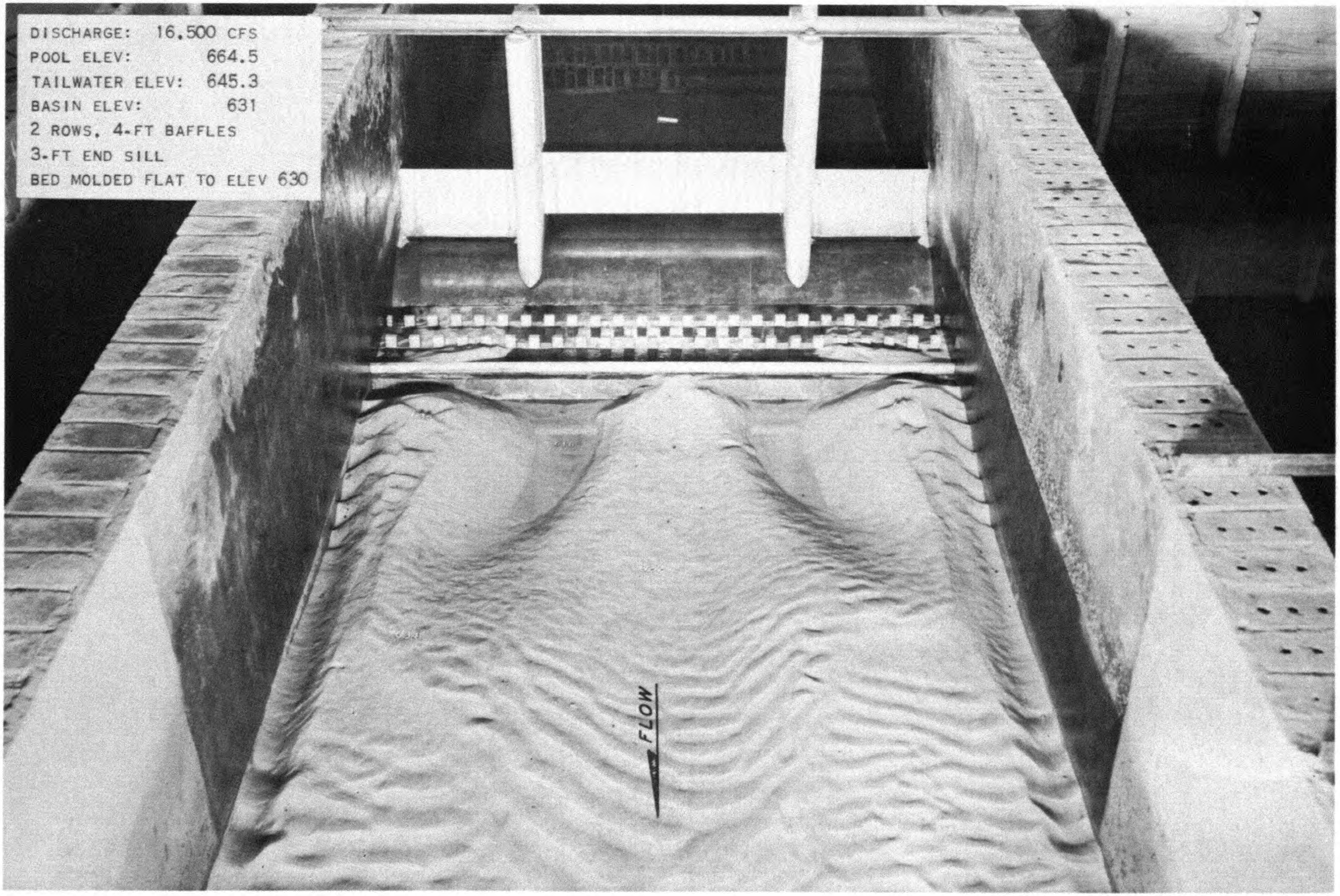
2. Tailwater elevation based on discharge for entire spillway length

3. Eight-foot gate opening selected for comparative tests as most critical velocities occurred at this opening

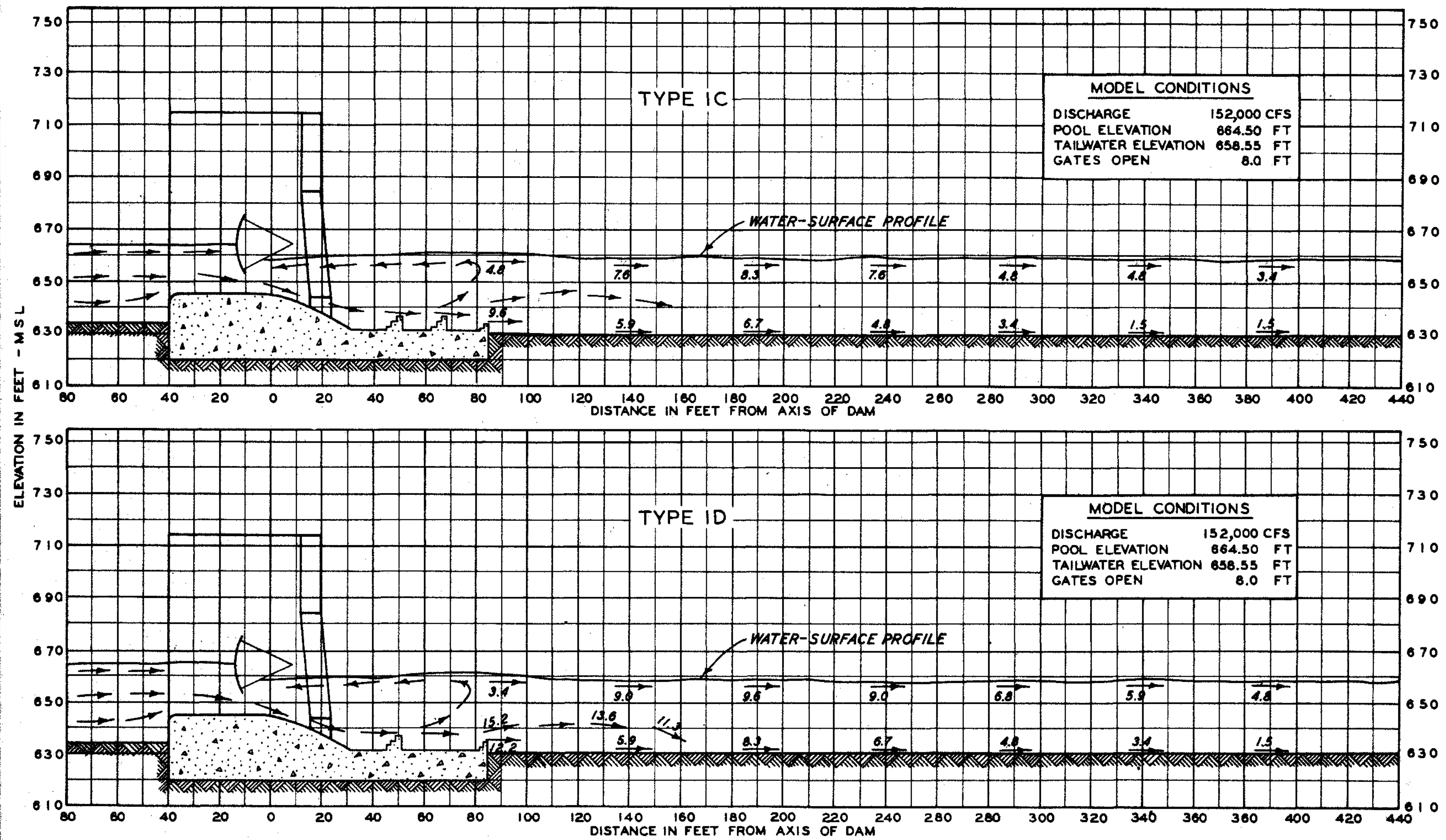


Photograph 6. Condition of exit area below type 1 basin after 1-hour scour test with gates open full

80WSE_295 “Spillway for New Cumberland Dam, Ohio River, West Virginia: Hydraulic Model Investigation,” 1954



Photograph 9. Condition of exit area below type 1 basin after 1-hour scour test with one gate open 8 ft



NOTE: WATER-SURFACE PROFILES MEASURED ALONG CENTER LINE OF MODEL.
VELOCITIES ARE IN PROTOTYPE FEET PER SECOND.

WATER-SURFACE PROFILES AND FLOW CHARACTERISTICS
TYPES IC AND ID BASINS
DISCHARGE: 152,000 CFS

80WSE_295 “Spillway for New Cumberland Dam, Ohio River, West Virginia: Hydraulic Model Investigation,” 1954

PART II: THE MODEL

Description

4. The model (fig. 2) was constructed to a scale of 1:25 and

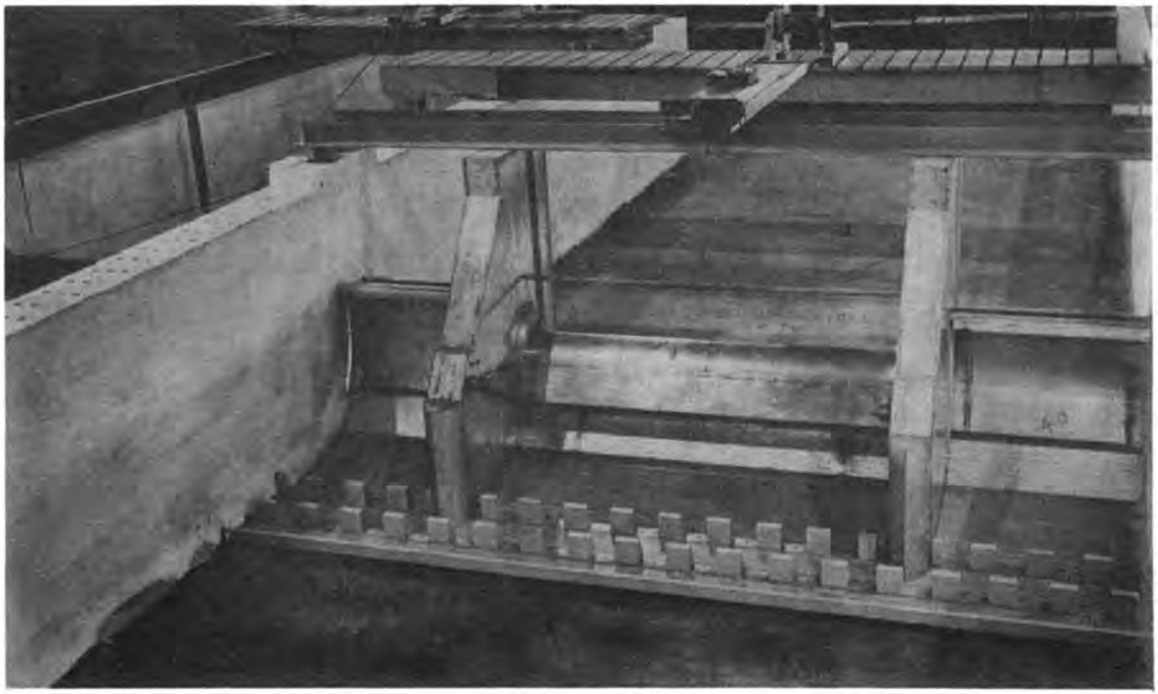
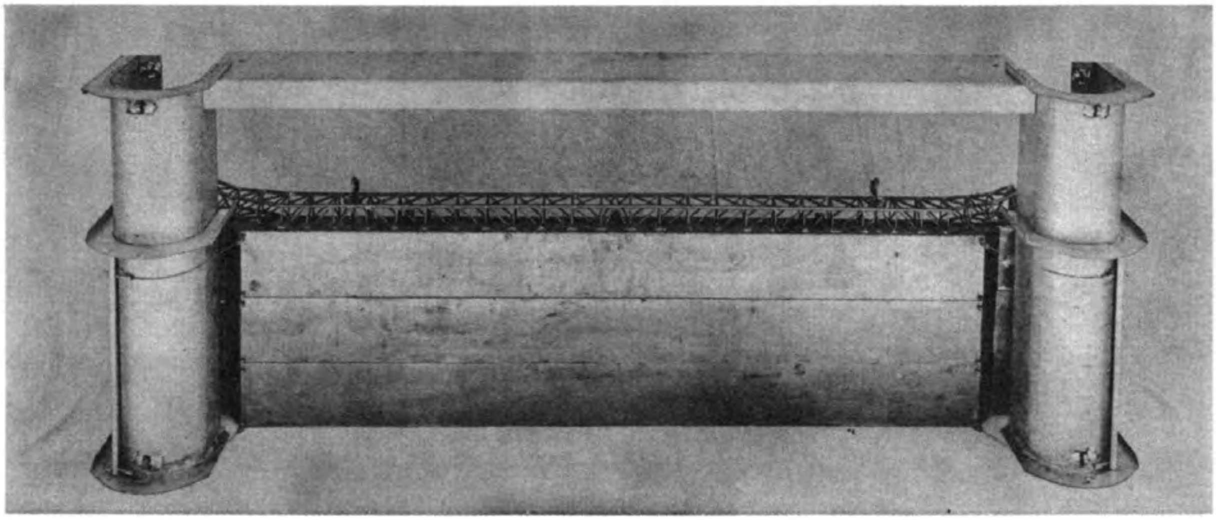


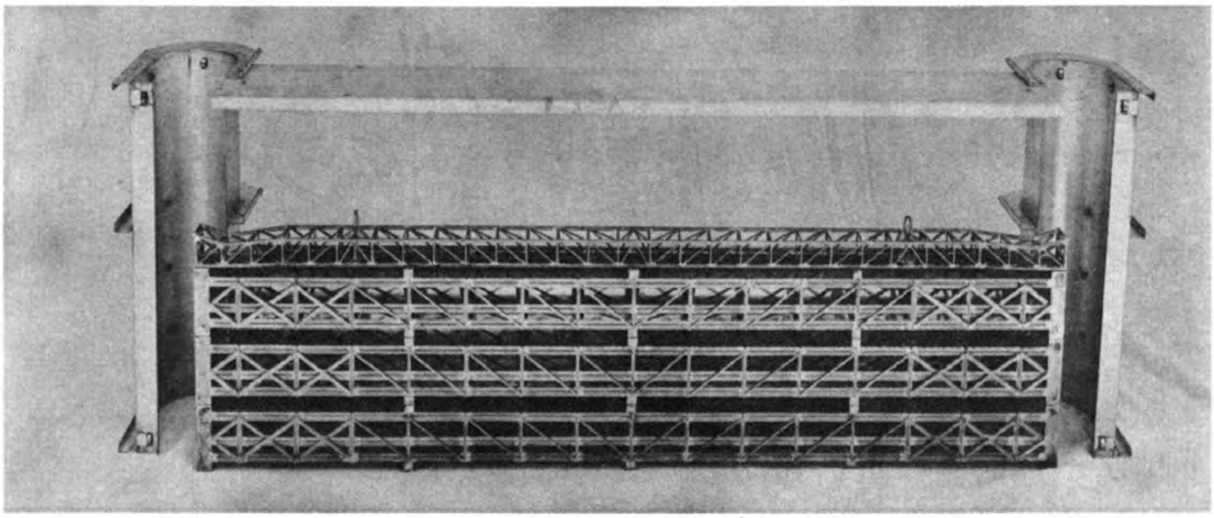
Fig. 2. 1:25-scale section model

reproduced one submergible-gate bay and adjacent half bays together with a corresponding width (240 ft) of a 400-ft-long section of the approach, the stilling basin, and 500 ft of the exit area. The portions of the model representing the spillway and gate piers were fabricated of sheet metal, and the stilling basin elements were modeled in wood. The test gate was reproduced of sheet brass accurately to scale in size, shape, and weight, and for the gate-vibration tests, the elongation characteristics of the prototype suspension were accurately reproduced. The model gate weighed 26.5 lb (an equivalent of 414 kips, the estimated weight of the prototype gate). The test bulkheads and lifting beam were constructed of sheet metal and were reproduced accurately to scale in size and shape (fig. 3).

5. In addition and supplementary to tests conducted in the model described above, observations of stilling basin action were made on a 1:40-scale section model installed in a 1-ft-wide glass-sided flume (fig. 4).



a. Looking downstream



b. Looking upstream

Fig. 3. 1:25-scale model of bulkheads and lifting beam

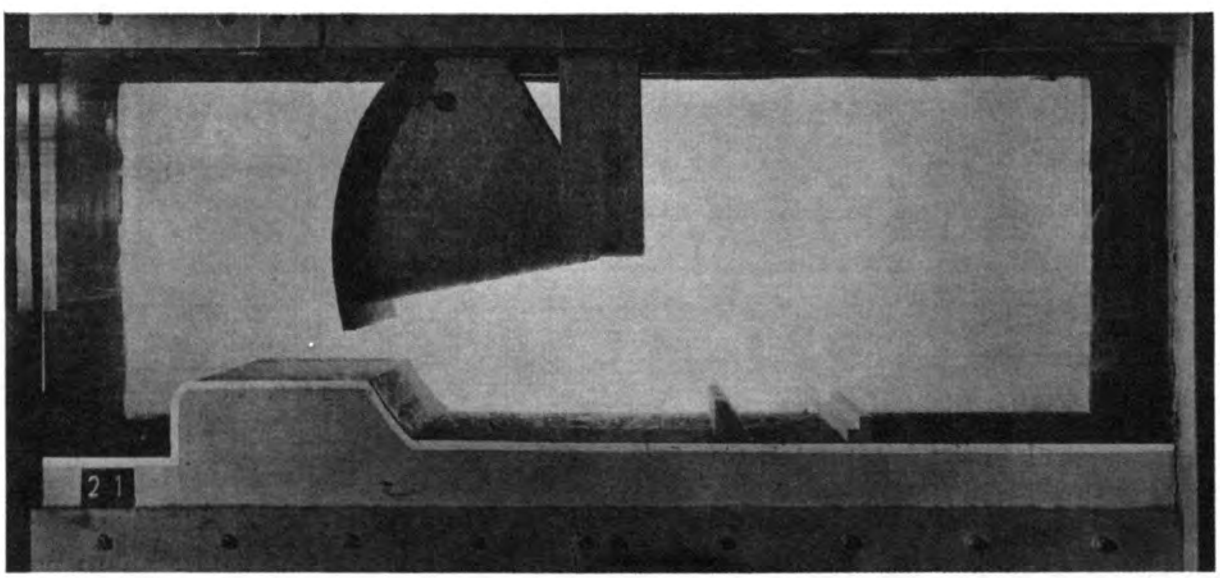
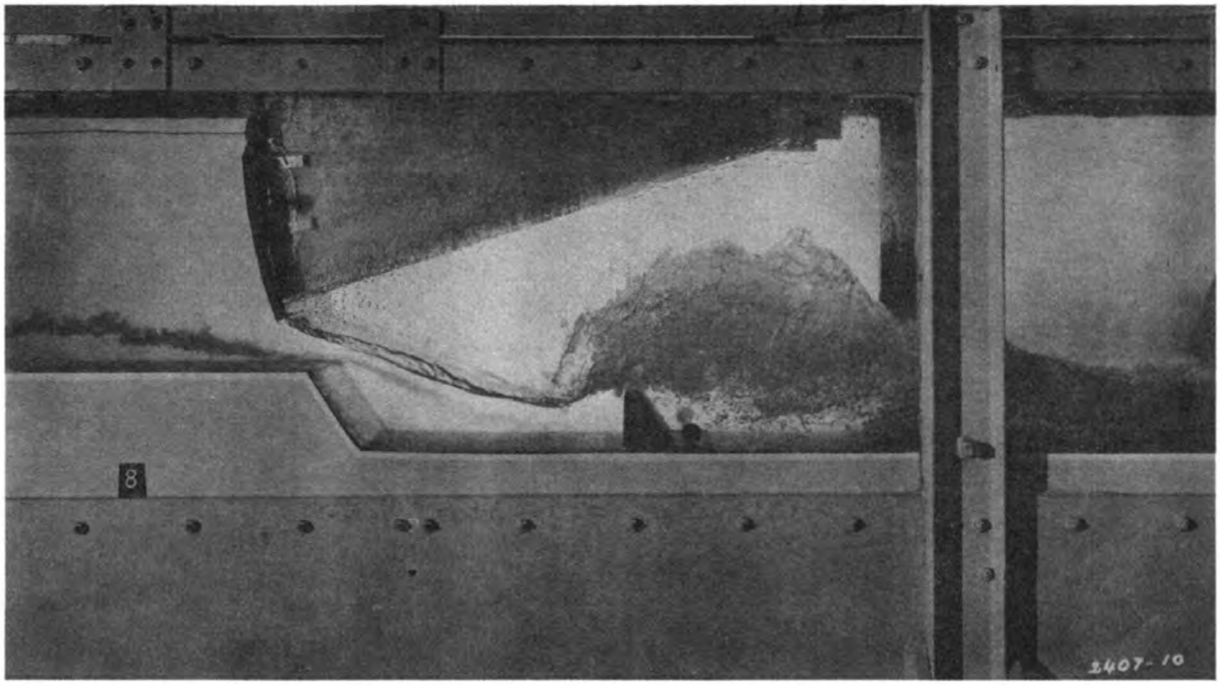
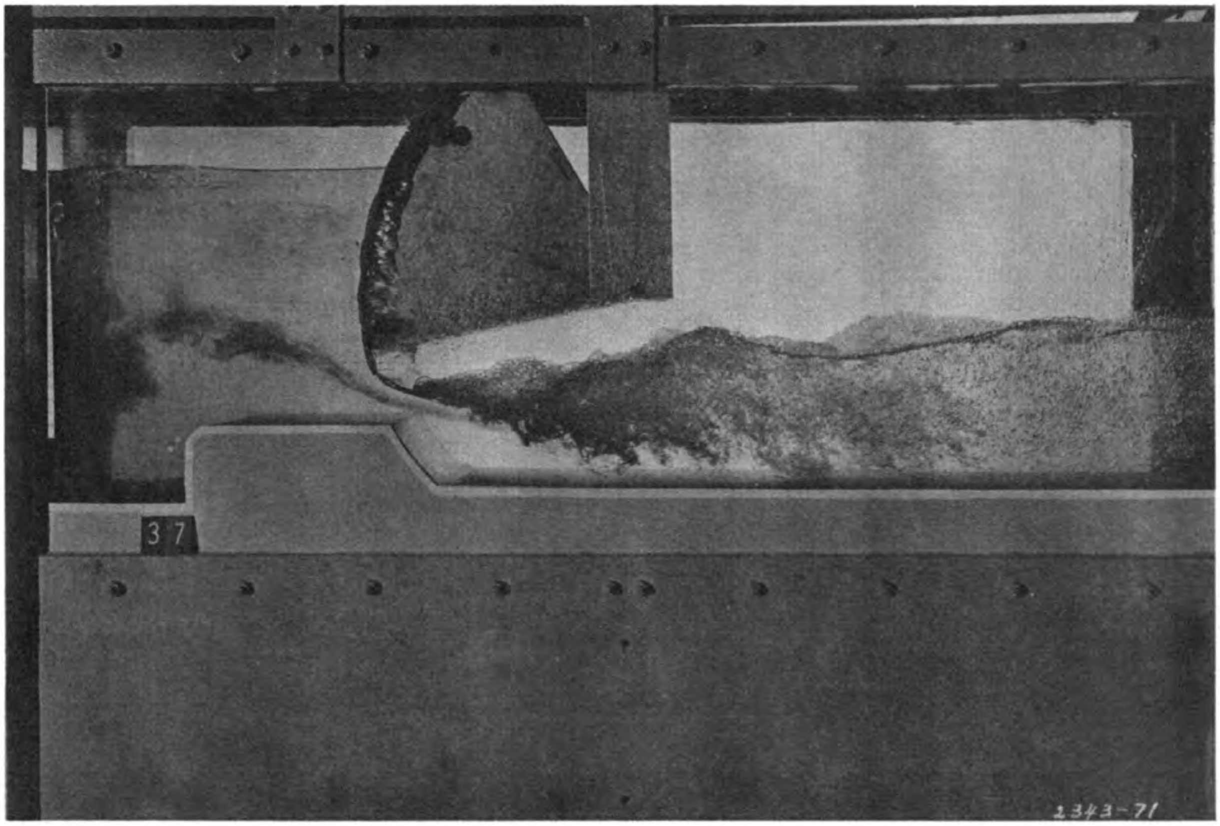


Fig. 4. 1:40-scale section model in glass-sided flume

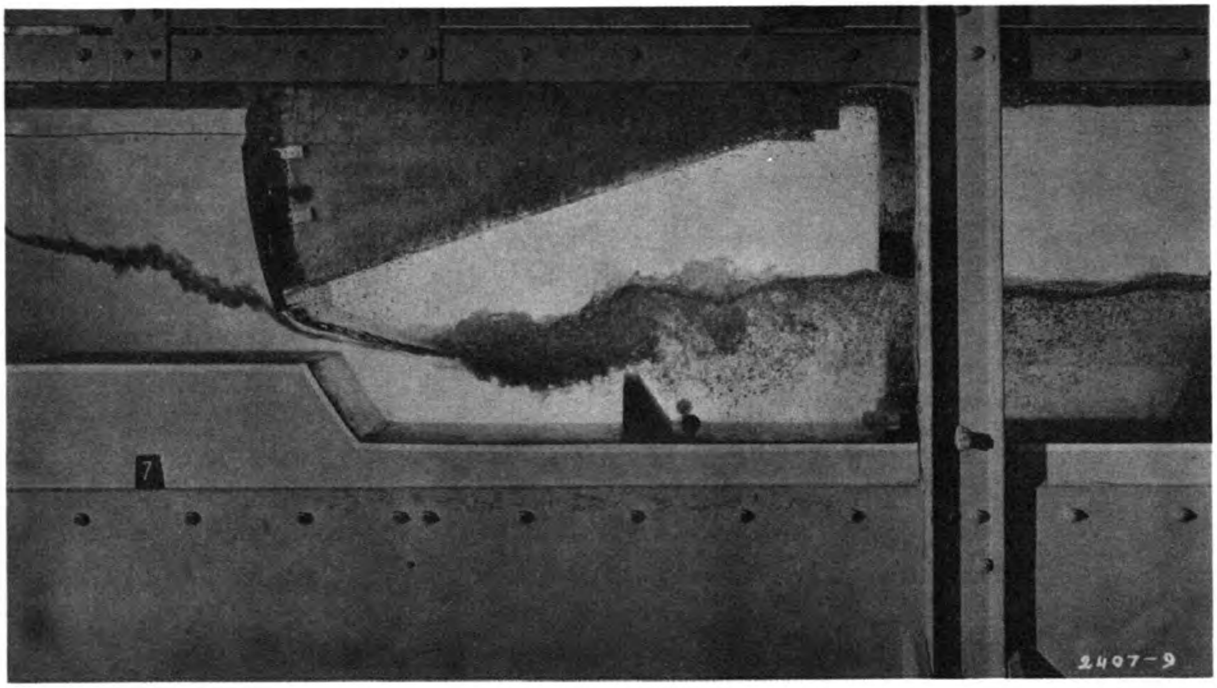


Photograph 1. Supercritical flow. The jet either swept through the stilling basin or impinged upon baffles or sills and sprayed into the exit area

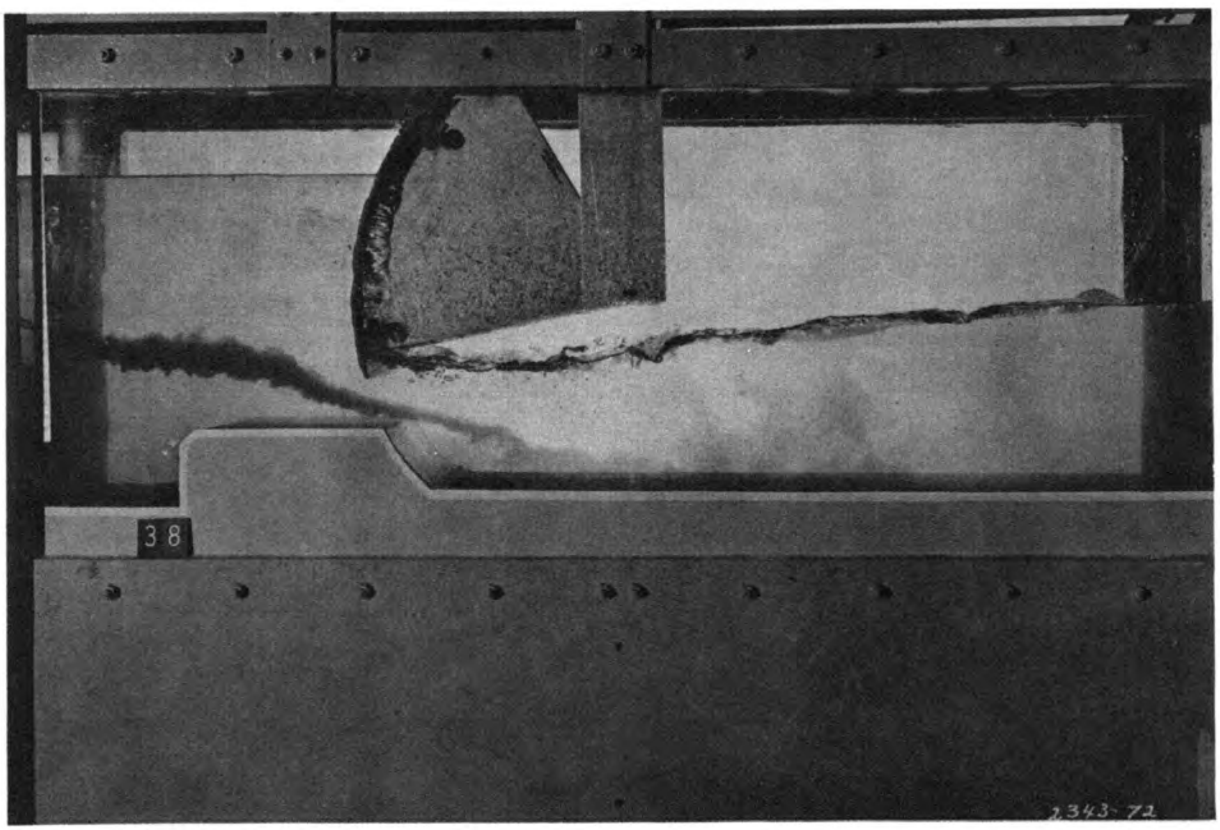


Photograph 2. Hydraulic jump. The jet was broken up by turbulence and spread throughout the entire depth of tailwater. Energy dissipation was excellent. Baffles and sills assisted in energy dissipation

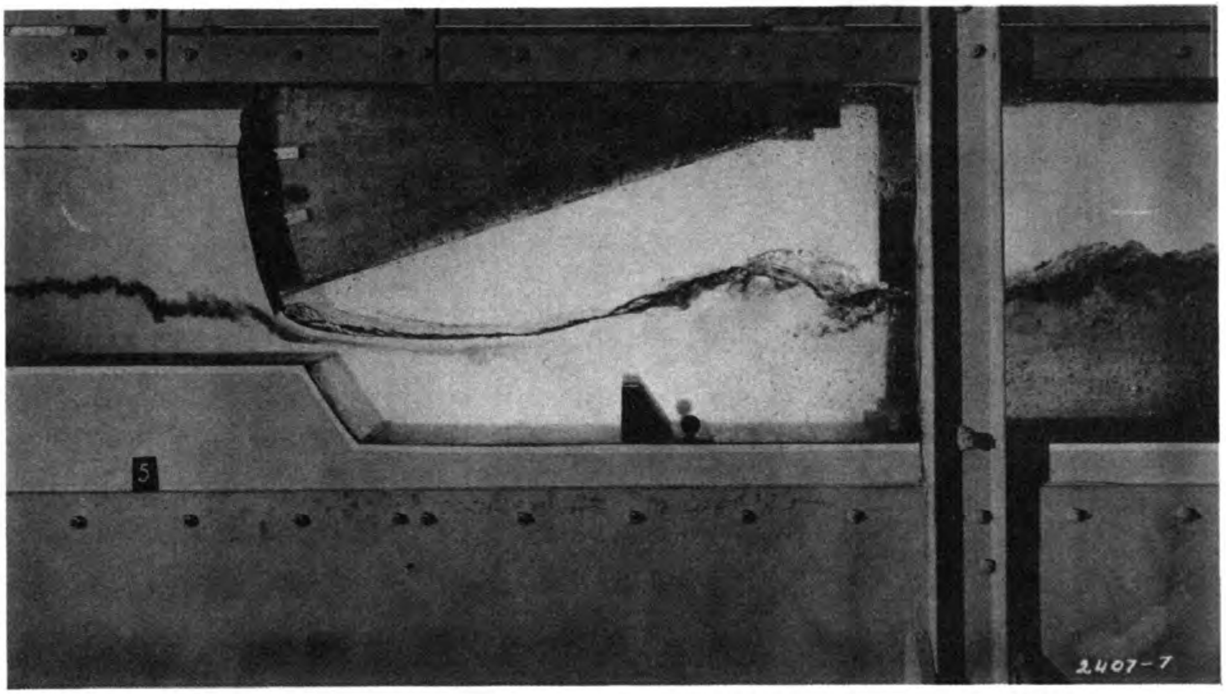
80WSE_296 "Spillway for New Cumberland Locks and Dam (Final Design), Ohio River, West Virginia: Hydraulic Model Investigation," 1961



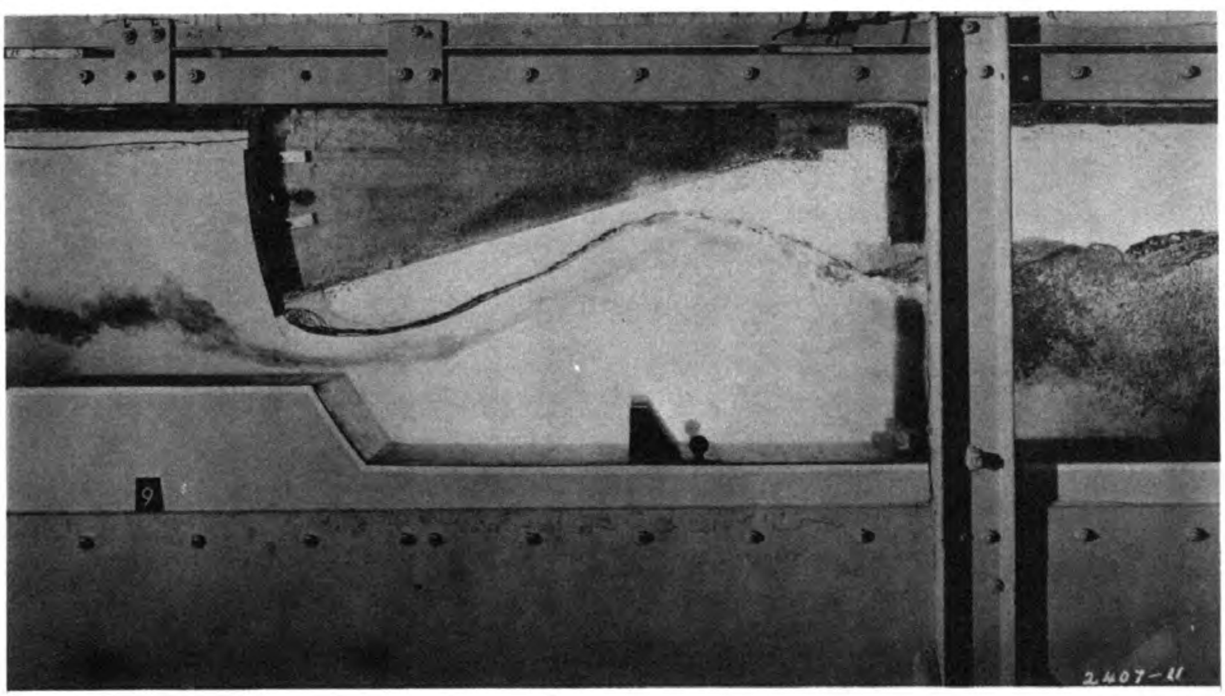
Photograph 3. Forced jump. Tailwater was less than that required for formation of hydraulic jump, and action was dependent primarily upon baffles or sills



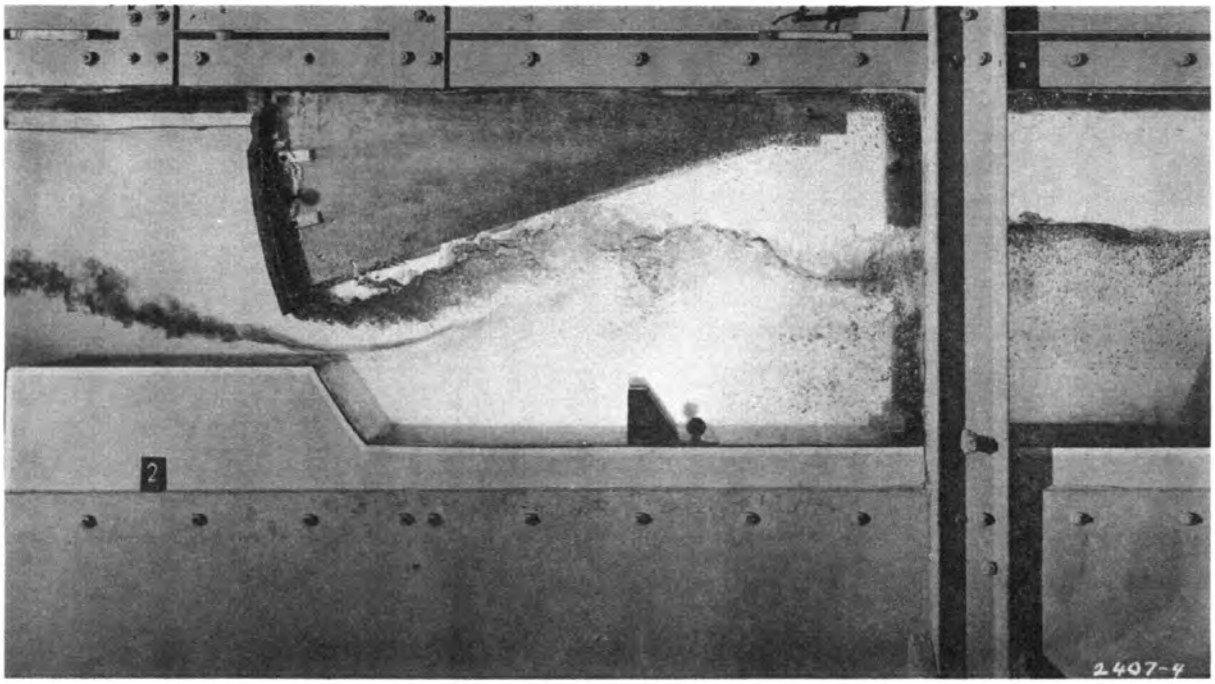
Photograph 4. Submerged jump. Tailwater was in excess of the optimum for a hydraulic jump, and the nappe at the gate lip was submerged. However, the jet spread from the apron throughout the entire depth of tailwater. Baffles and sills were effective



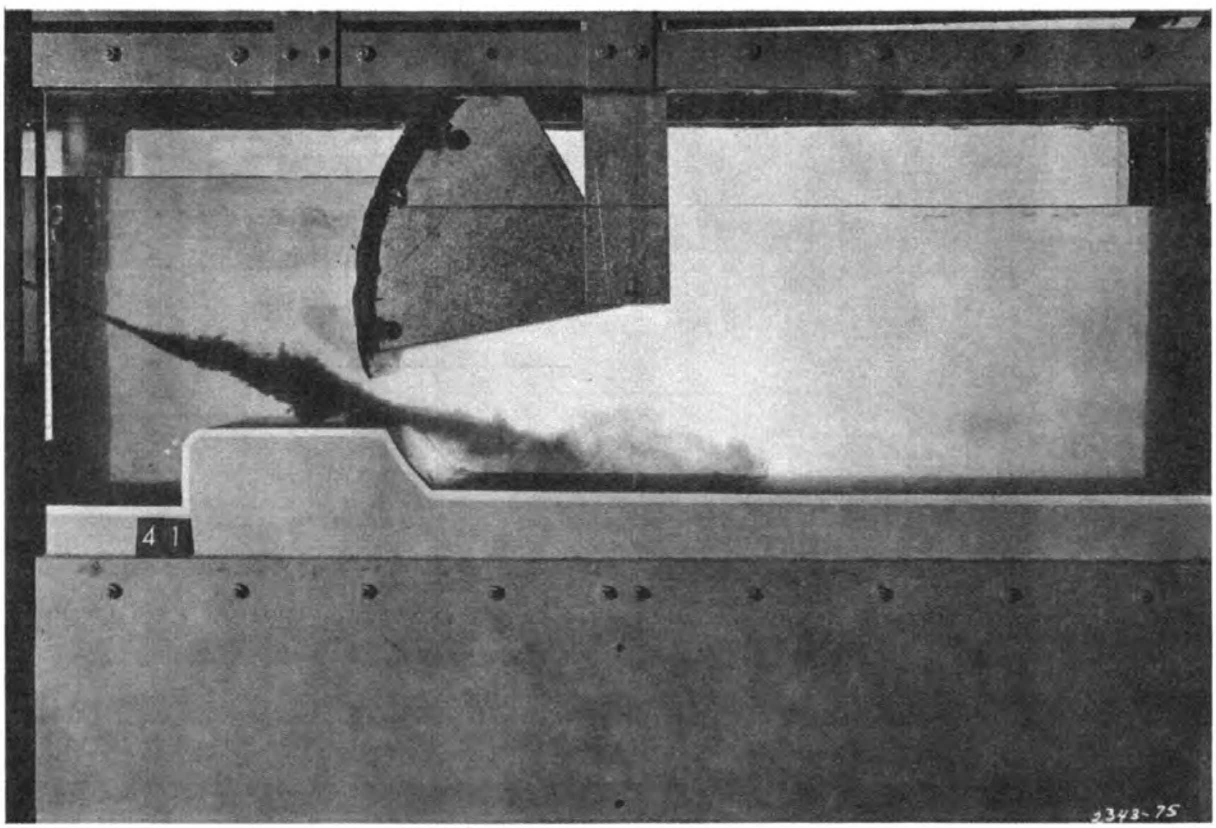
Photograph 5. Surface jet. Jet rode the surface of the tailwater. Bottom velocities usually were in upstream direction and of very low magnitude. However, high-velocity surface flow continued downstream and could result in serious bank erosion and/or undesirable navigation conditions



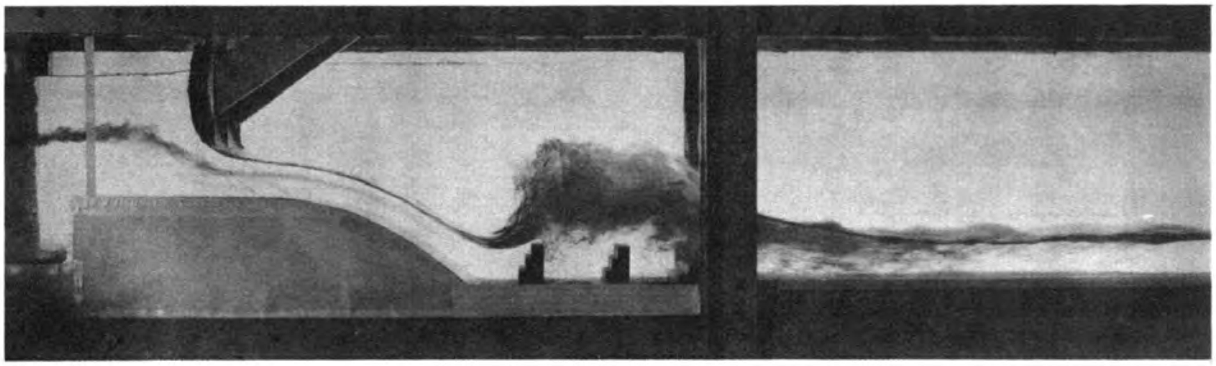
Photograph 6. Undulating jet (free nappe). Jet was forced to the surface immediately below the gate and then dove through the tailwater to the bottom. An excessively long stilling basin would be required to prevent serious attack on channel bottom



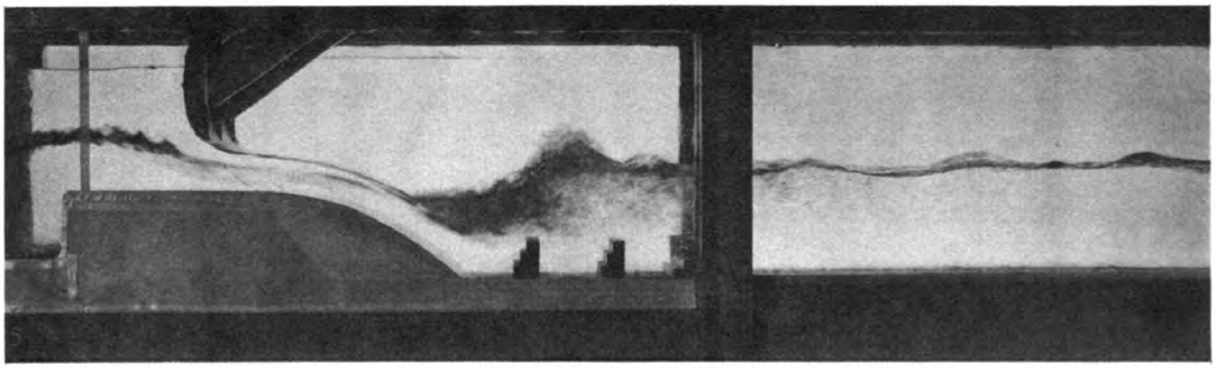
Photograph 7. Undulating jet (submerged nappe). Jet was generally the same as undulating jet (free nappe) except tailwater submerged nappe immediately below gate and was characterized by considerable surface wave action



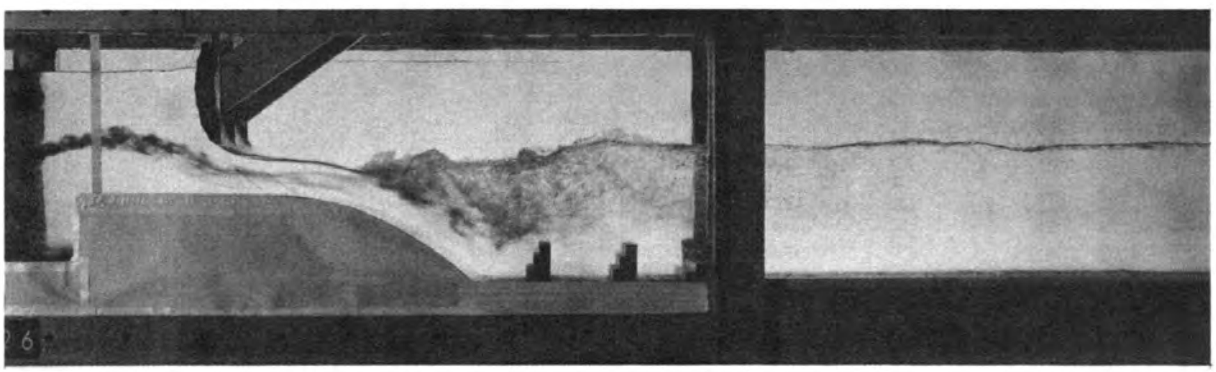
Photograph 8. Depressed jet. Jet was forced against the floor by excess tailwater and continued downstream along the floor



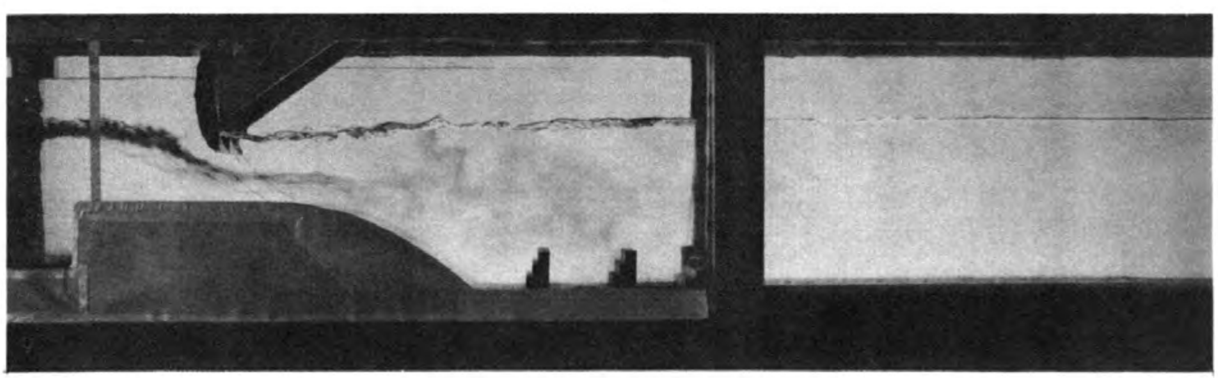
a. Spray action



b. Forced jump

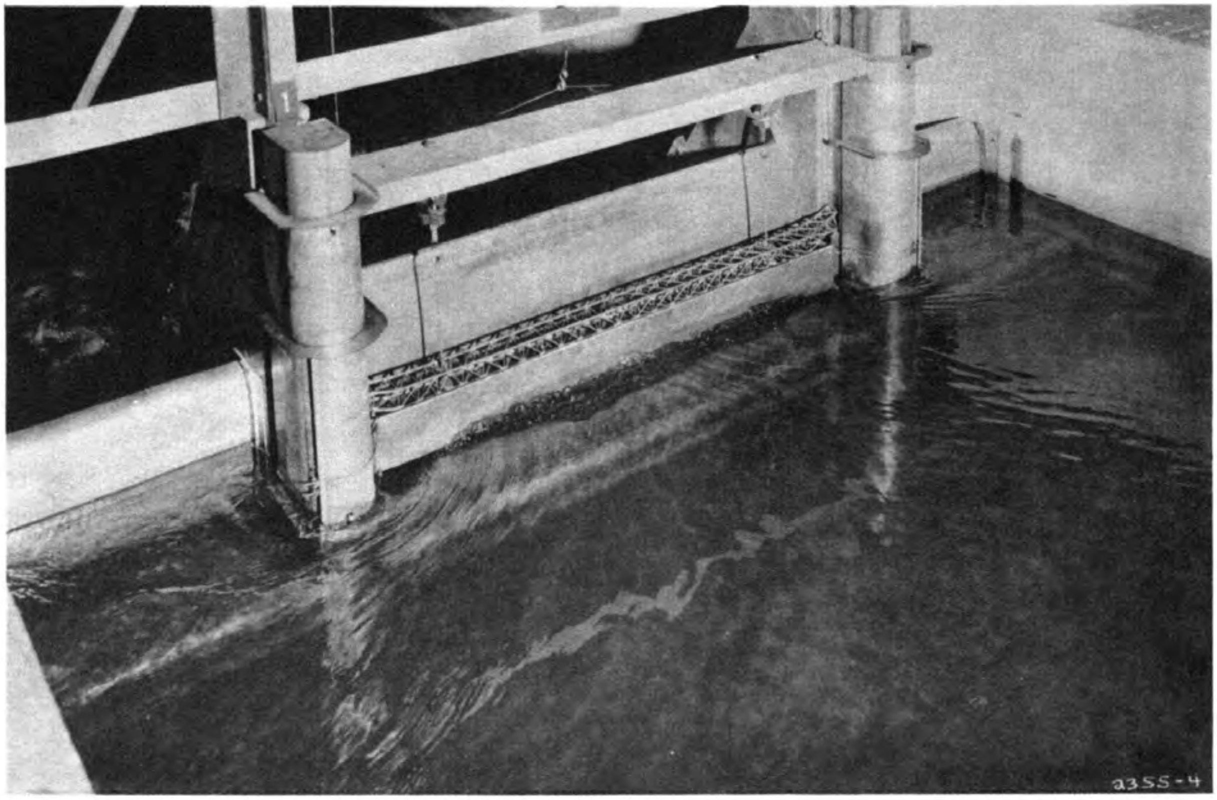


c. Hydraulic jump

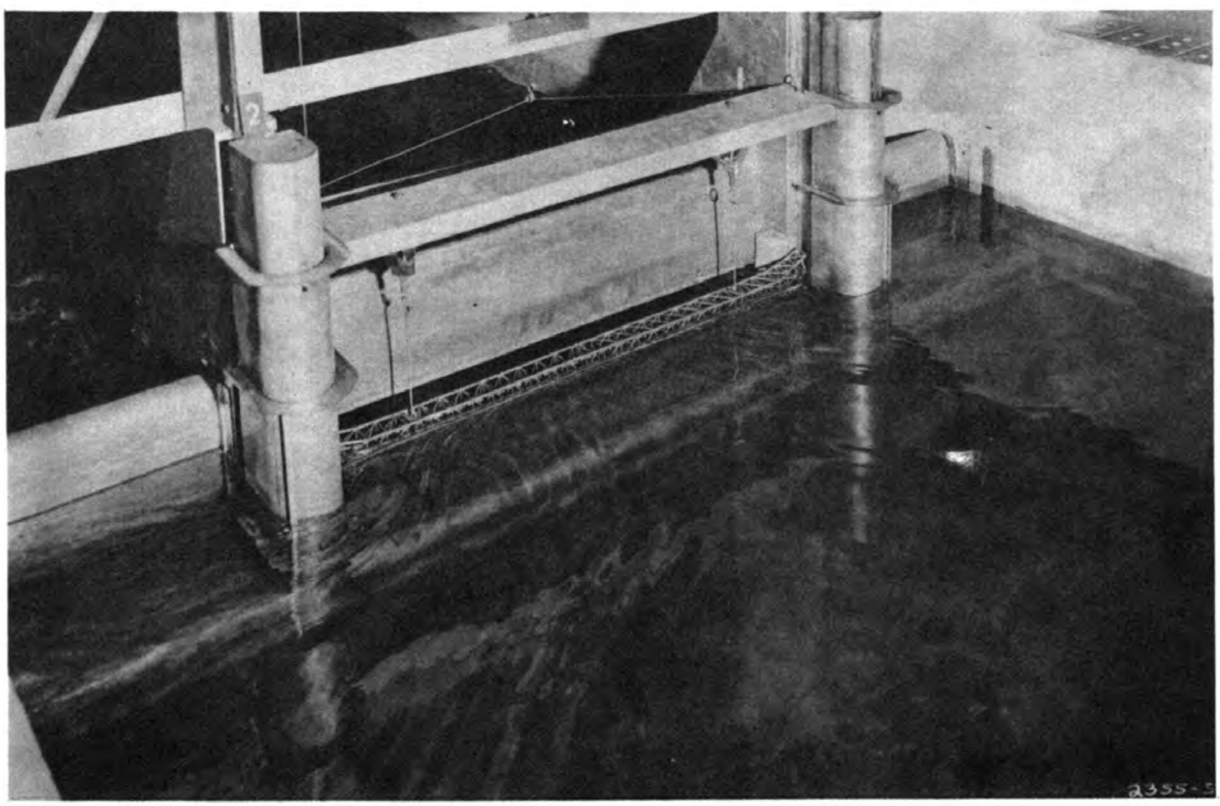


d. Submerged jump

Photograph 9. Types of basin action observed on 1:25-scale section model in 1-ft-wide glass-sided flume



Photograph 10. Flow conditions with first bulkhead section entering water. Upper pool el 664.5, tailwater el 642, spillway gate open full



Photograph 11. Flow conditions with bottom of first bulkhead section 9 ft above gate sill. Approximately 1.5-ft head over bulkhead. Upper pool el 664.5, tailwater el 642, spillway gate open full

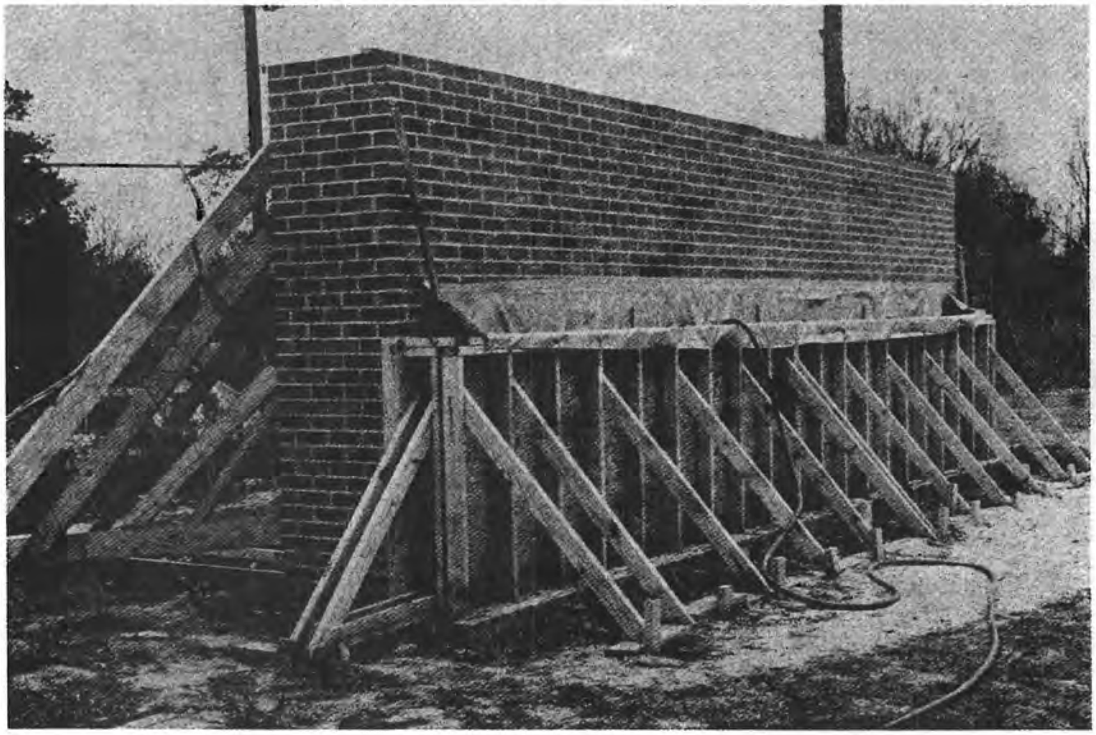


Figure 3. Overall view of Wall 1

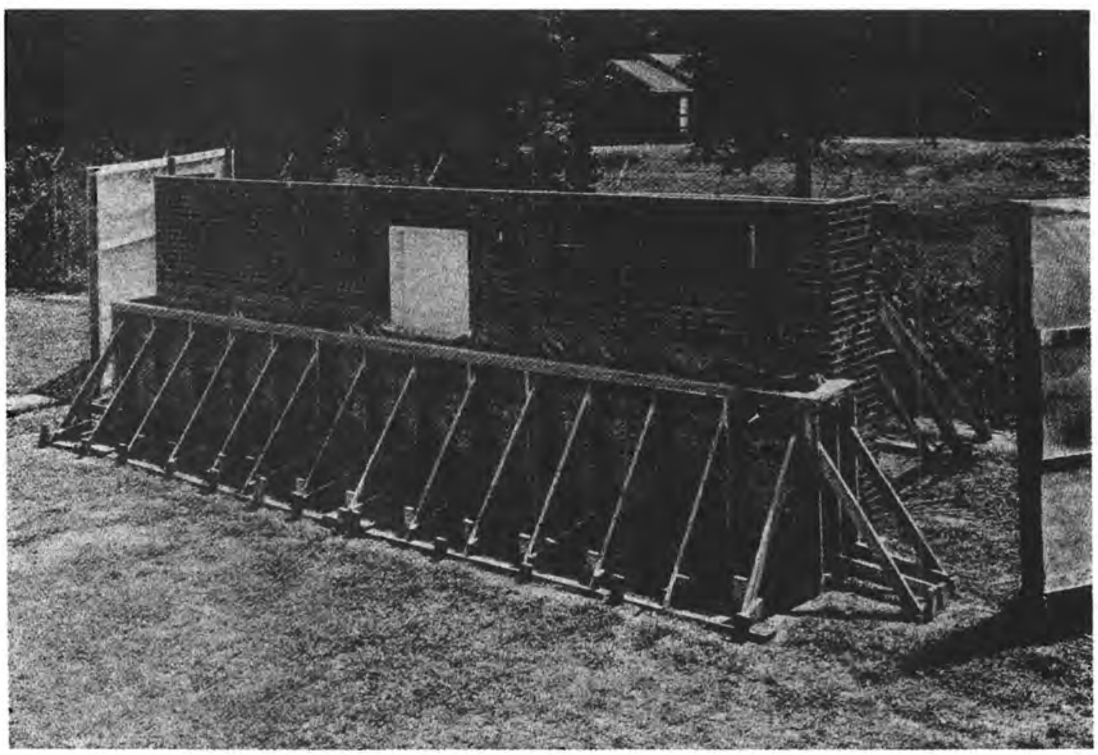


Figure 4. Overall view of Wall 2

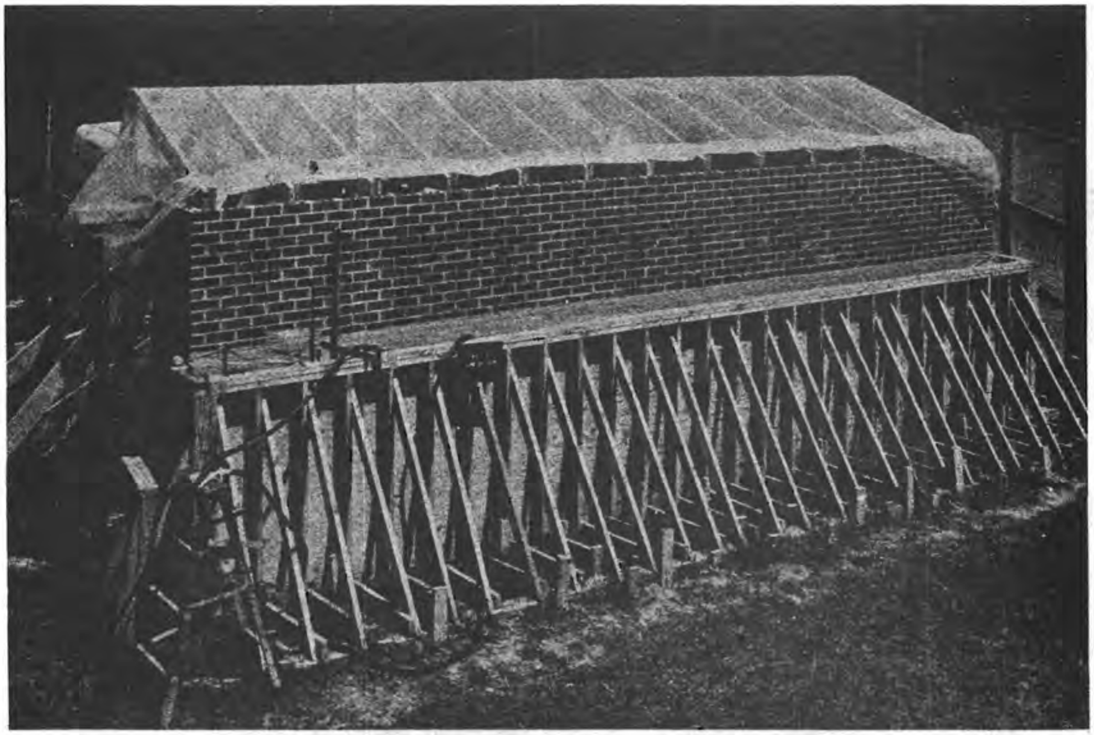


Figure 5. Overall view of Wall 3

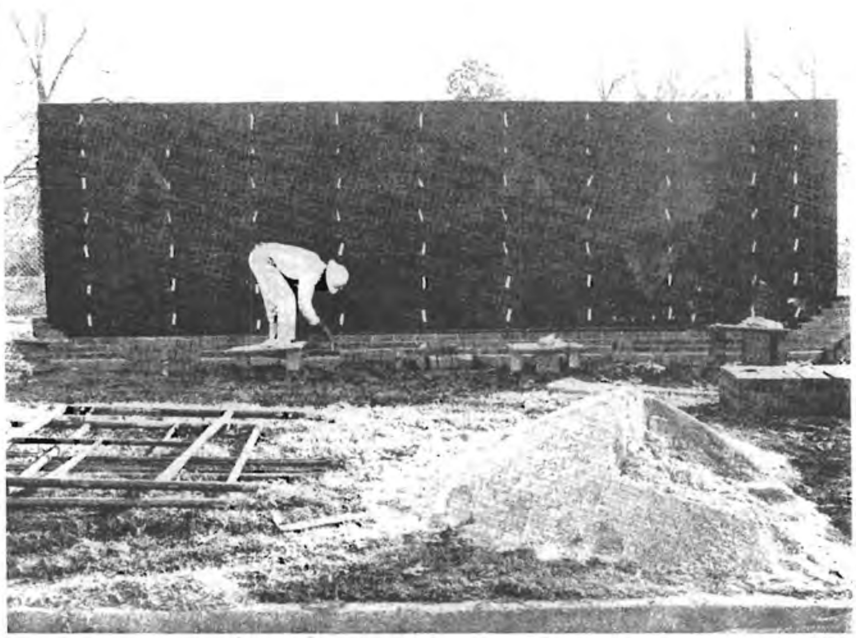


Figure 6. Stud wall, wallboard, and wall tie construction

Figure 9. Wall 1. Wall deflection

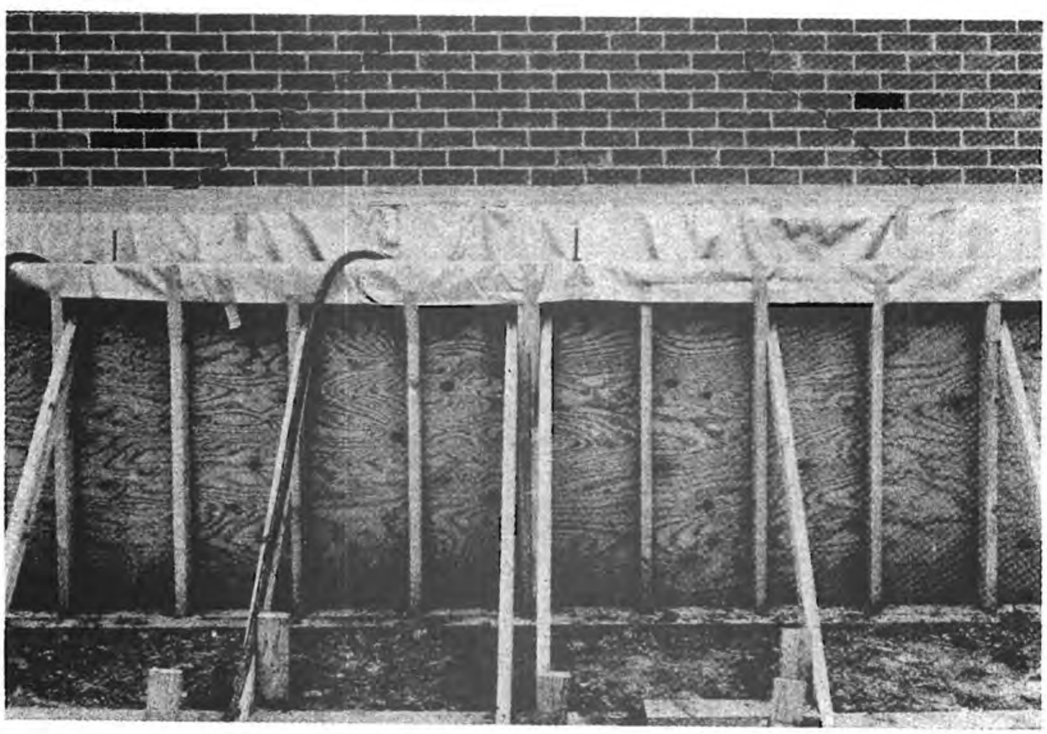


Figure 10. Wall 1. Failure pattern of wall

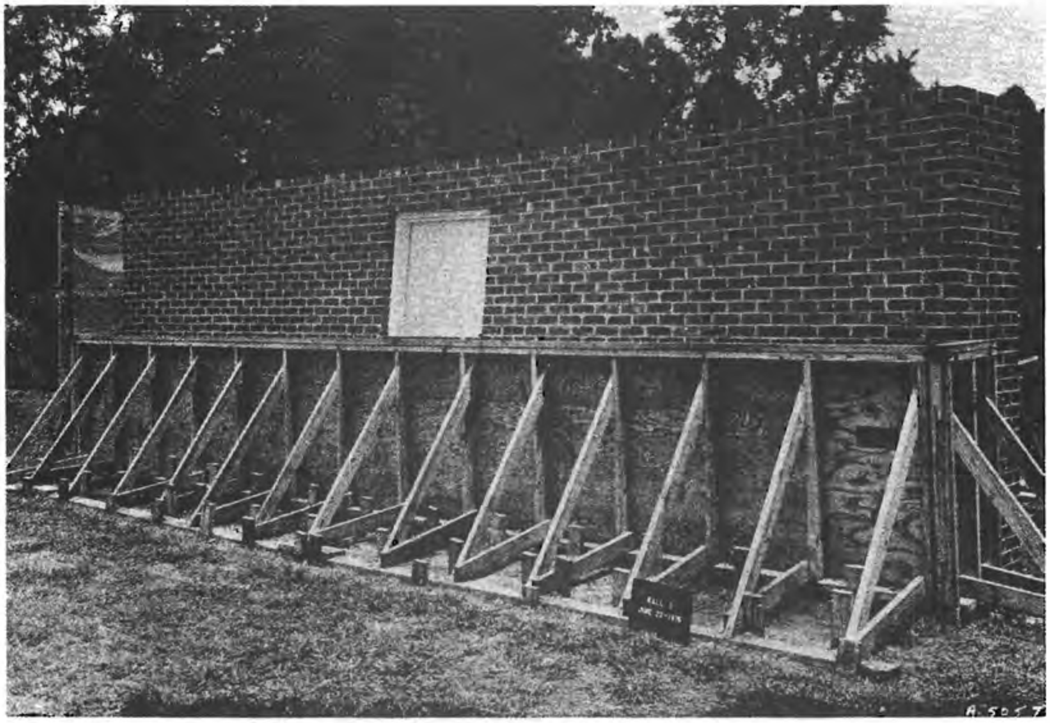


Figure 13. Wall 2. Wall failure, front view

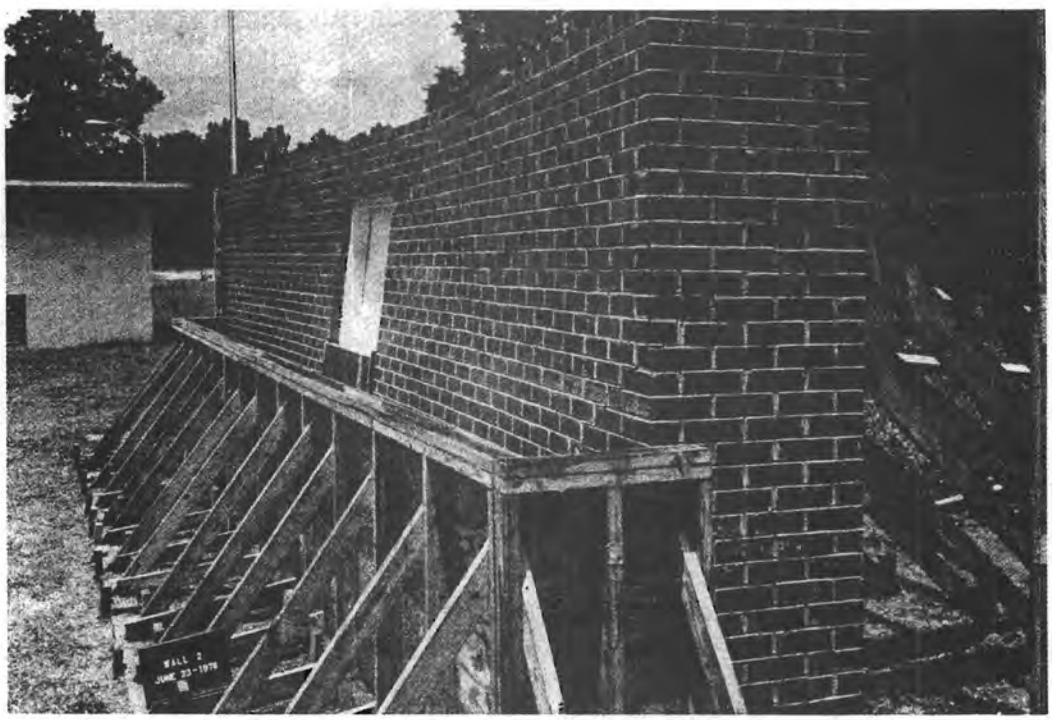


Figure 14. Wall 2. Wall failure, side view

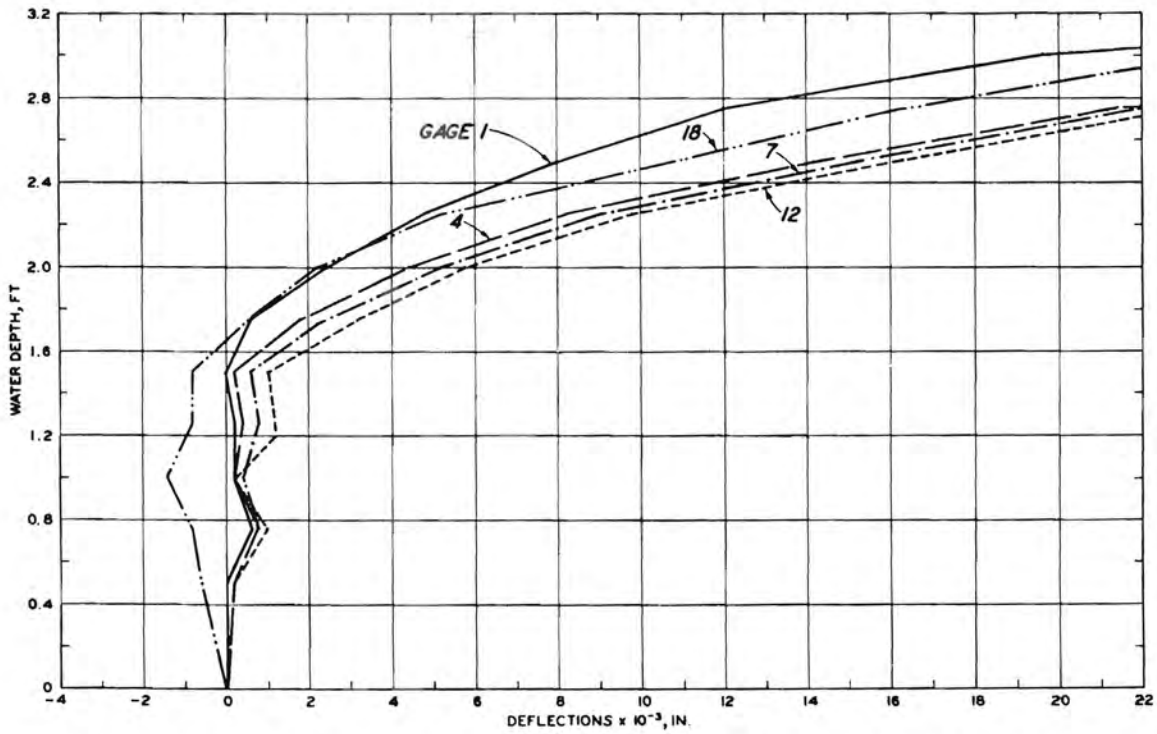


Figure 15. Wall 3. Brick-veneer wall deflection, vertical gage line

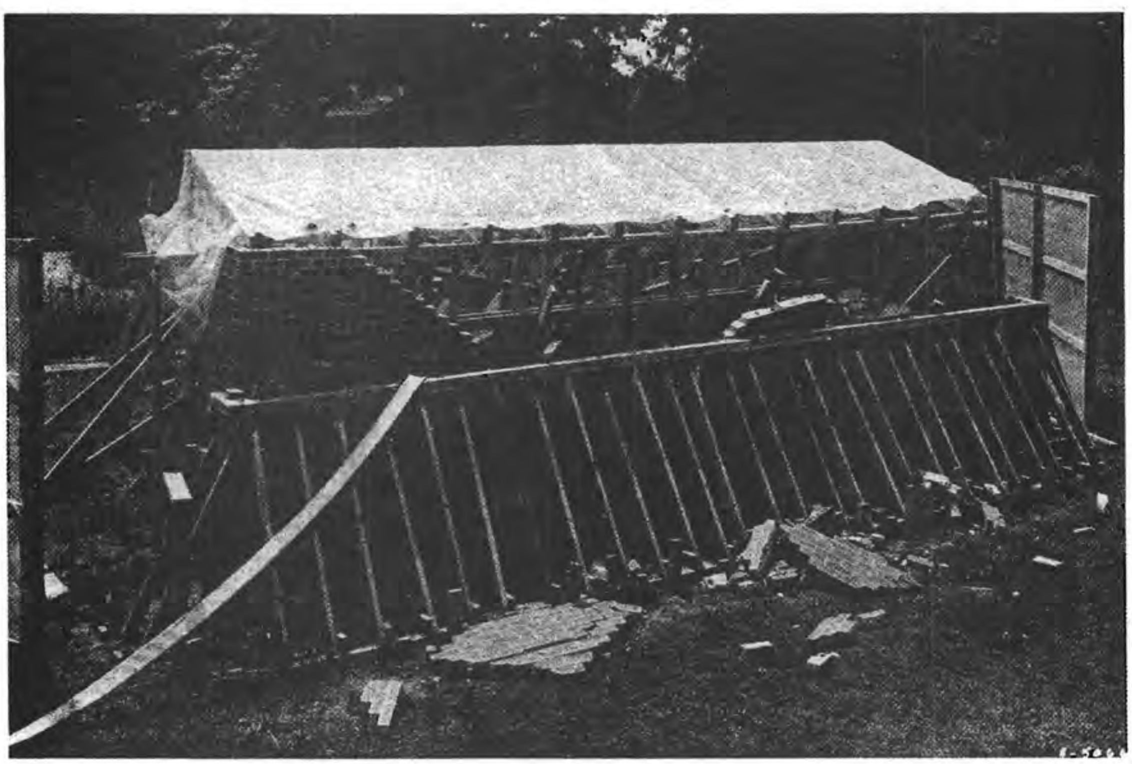
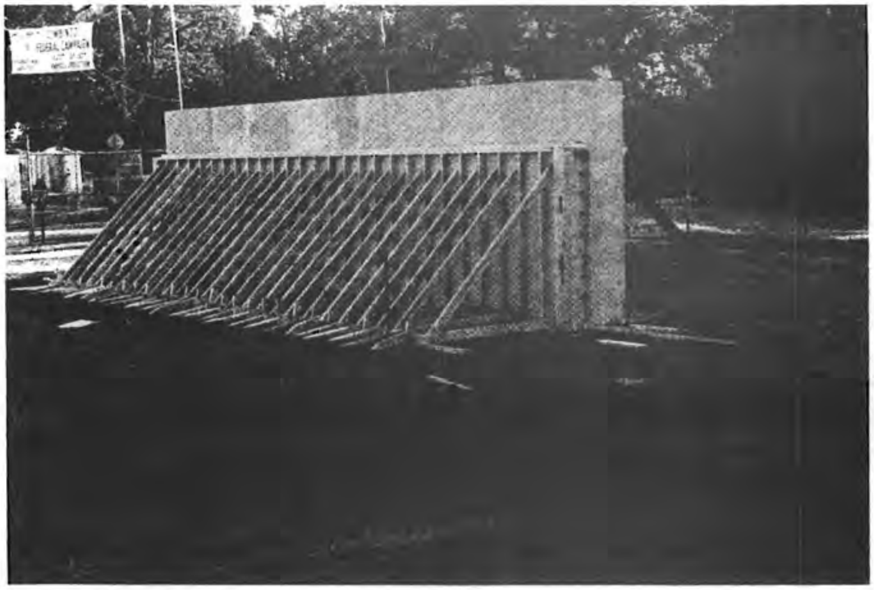
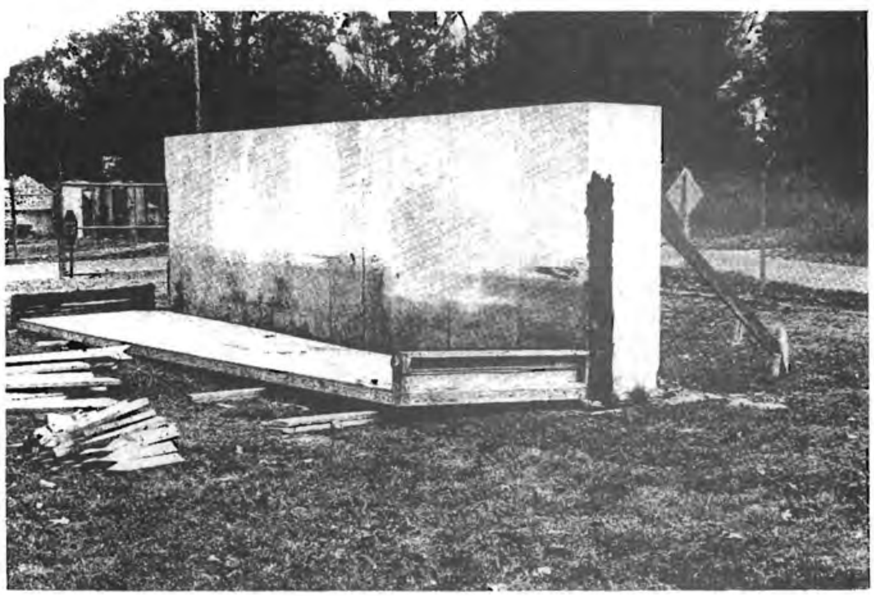


Figure 16. Wall 3. Wall failure, front view

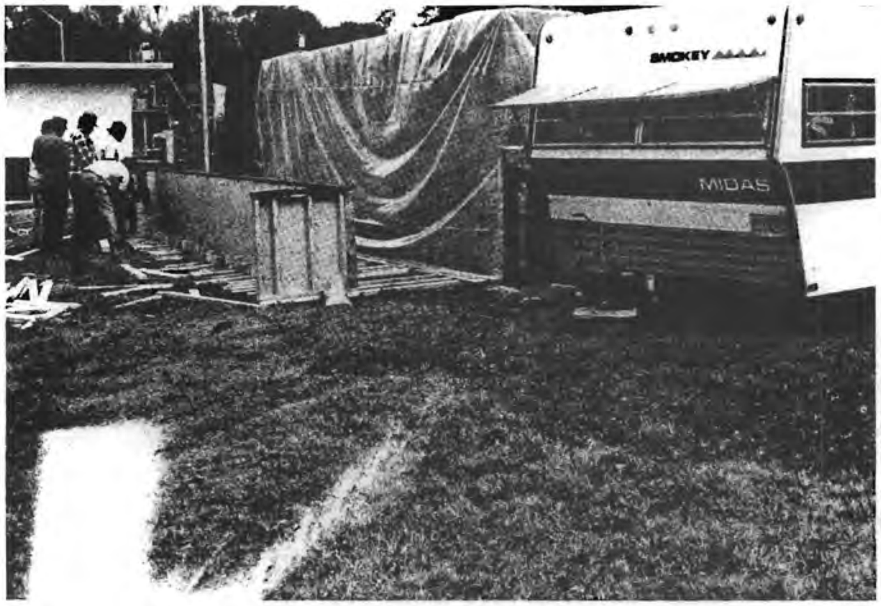


a. Before testing

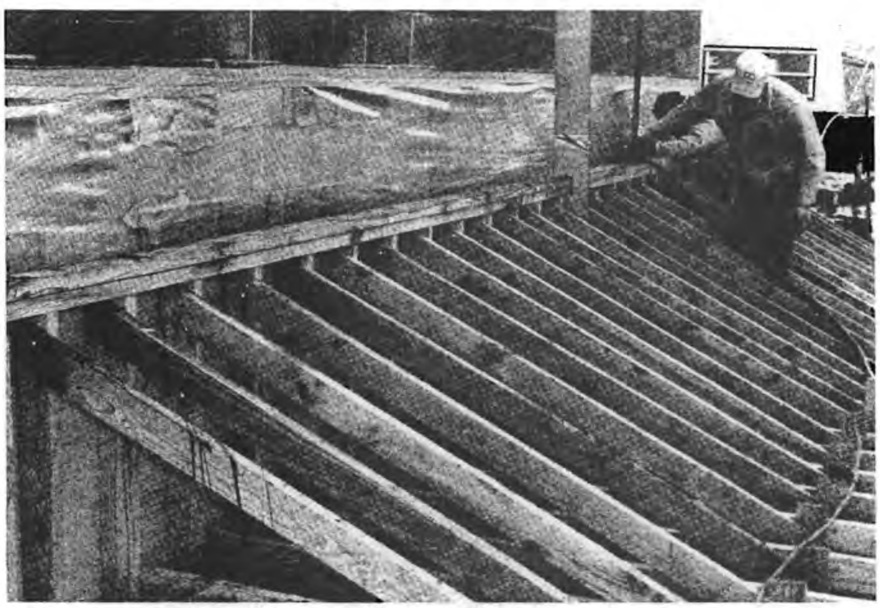


b. After testing

Figure 18. Block Wall 1



a. Preparing for testing



b. During testing

Figure 19. Block Wall 2

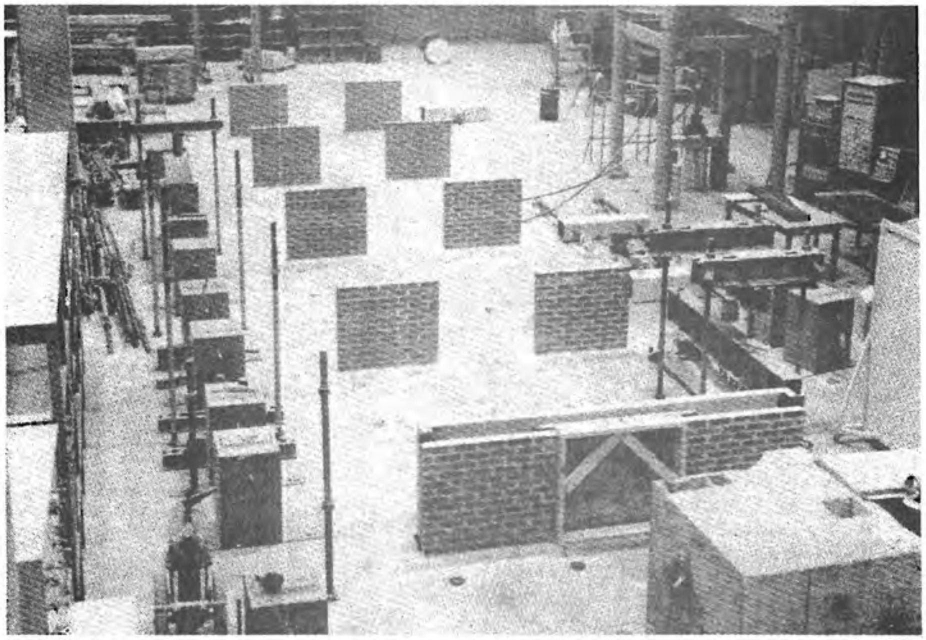


Figure 31. Five sets of short walls

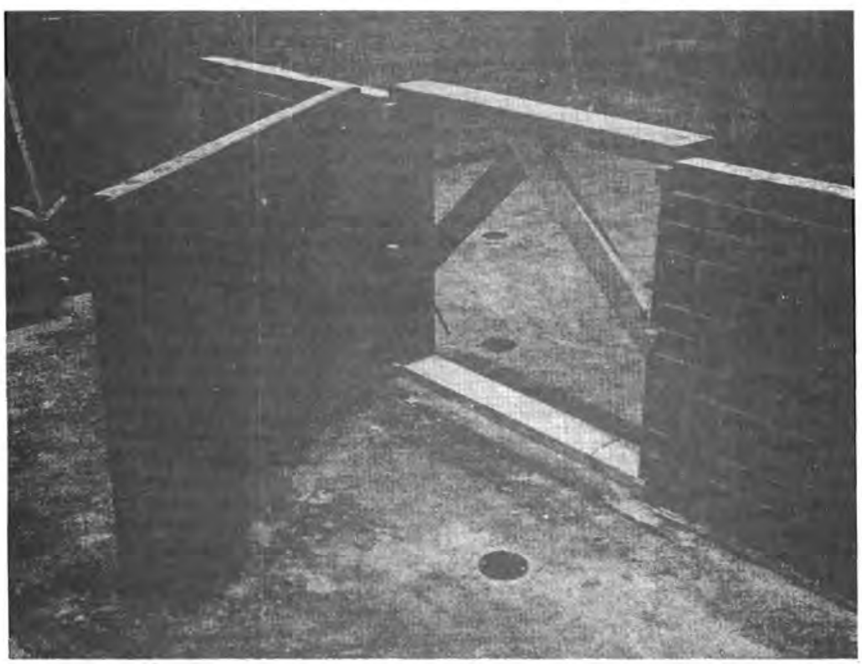


Figure 32. A restraining frame placed between the brick walls from which the closure can be pulled against the walls by springs

Figure 33. Springs used to pull the closure against the wall and down to the floor

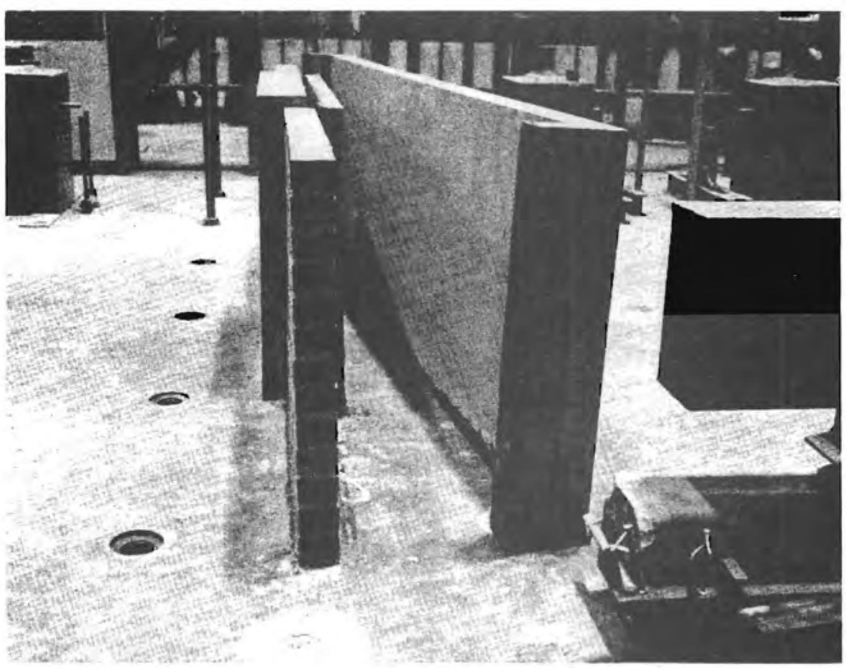


Figure 34. View 1. Bulkhead to contain water against the wall and closure

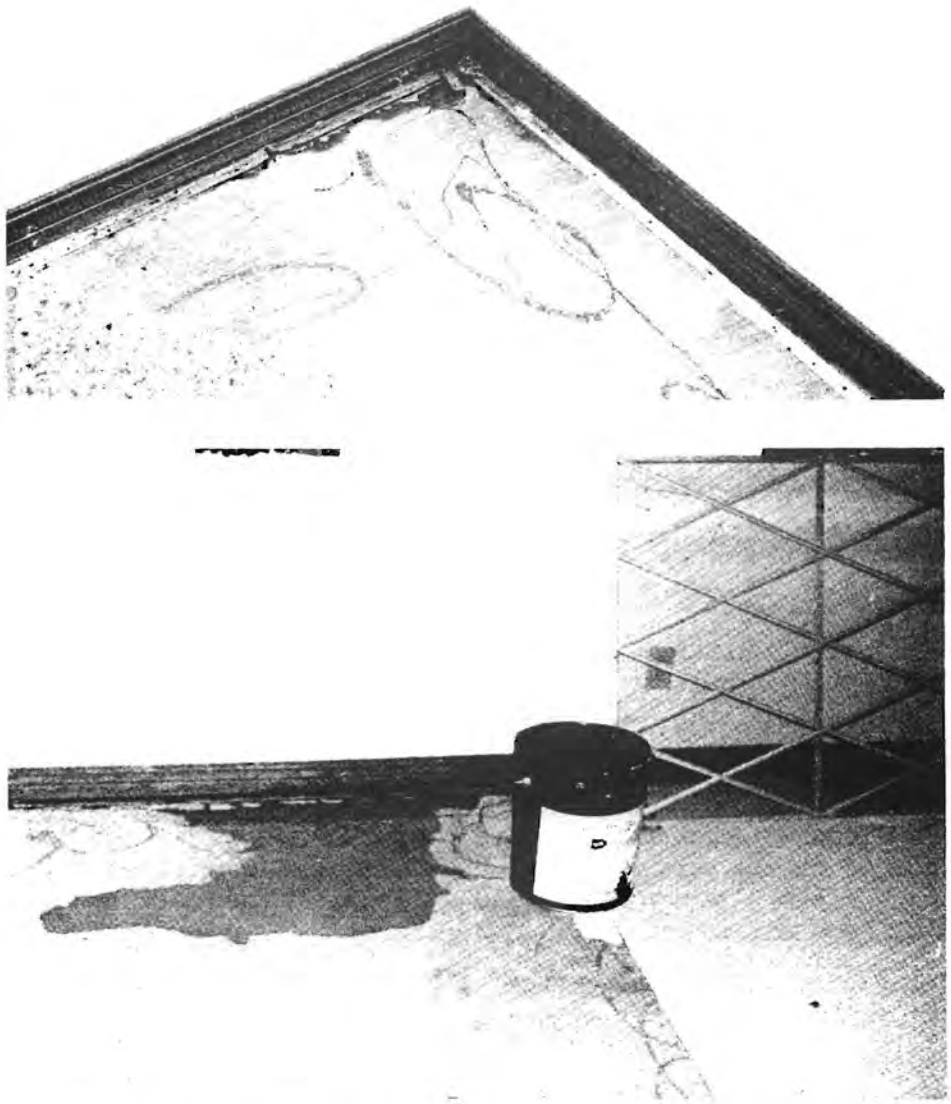


Figure 74. Seepage along baseboard due to leak in lap of fabric

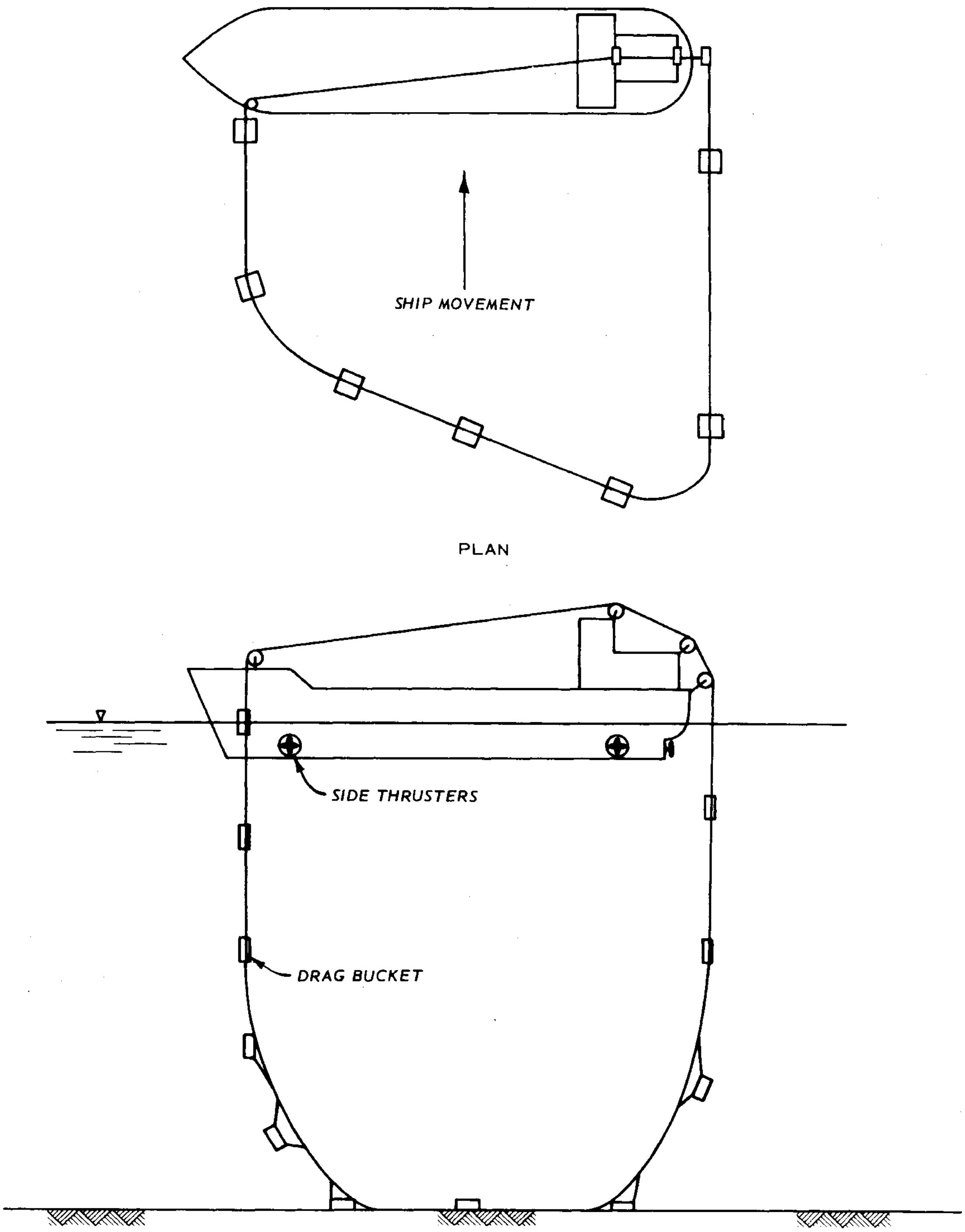
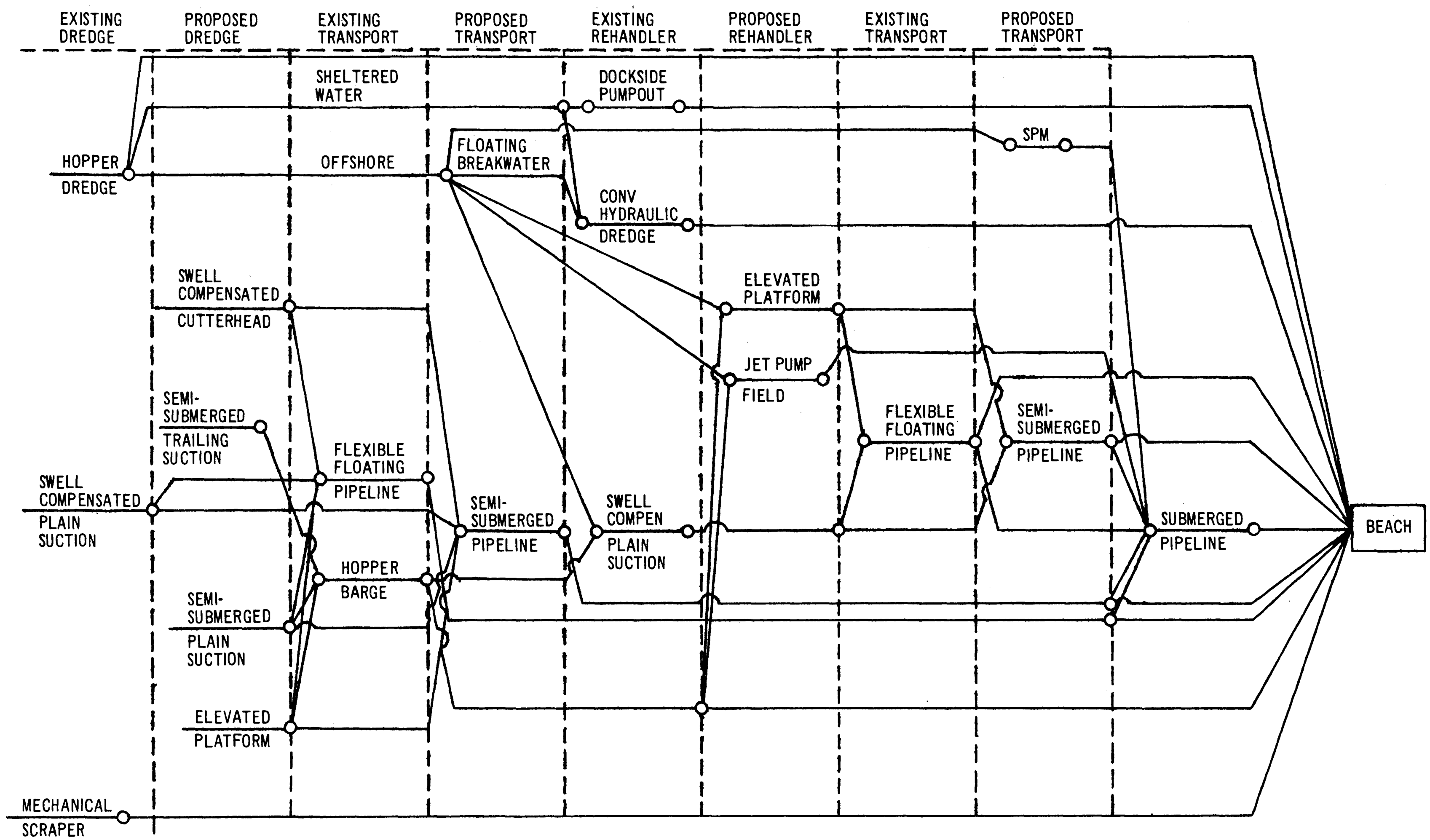


Figure 45. Deepwater bucket line dredge



SCHEMATIC OF
OFFSHORE BEACH NOURISHMENT SYSTEMS

PLATE 1

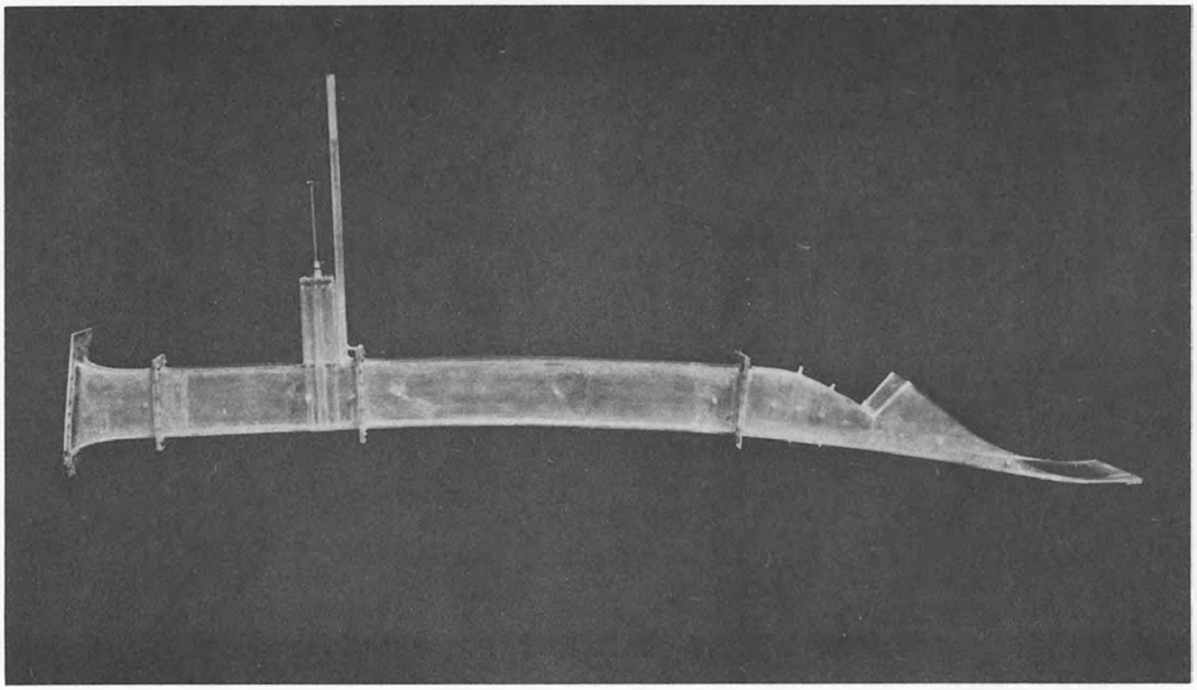


Fig. 1. Model sluice

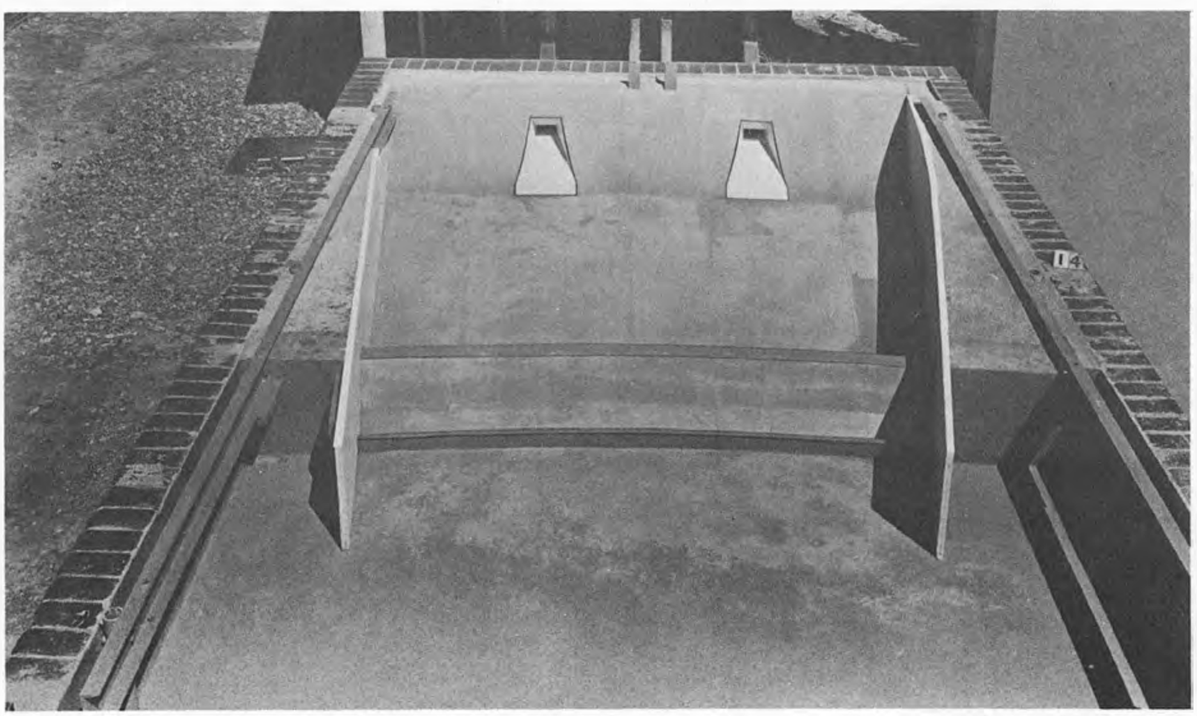


Fig. 2. Type A stilling basin

Fig. 16. Pool elev, 800
dischg, 3170 cfs; TW elev, 692.7

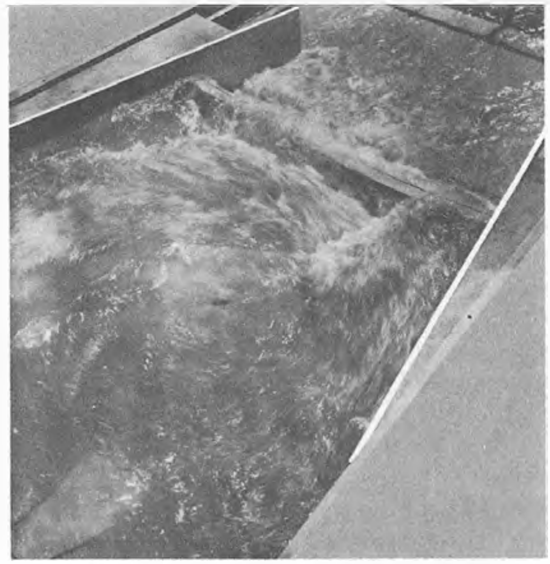


Fig. 17. Pool elev, 840
dischg, 3790 cfs; TW elev, 693.1

Type A-1 stilling basin, one sluice operating

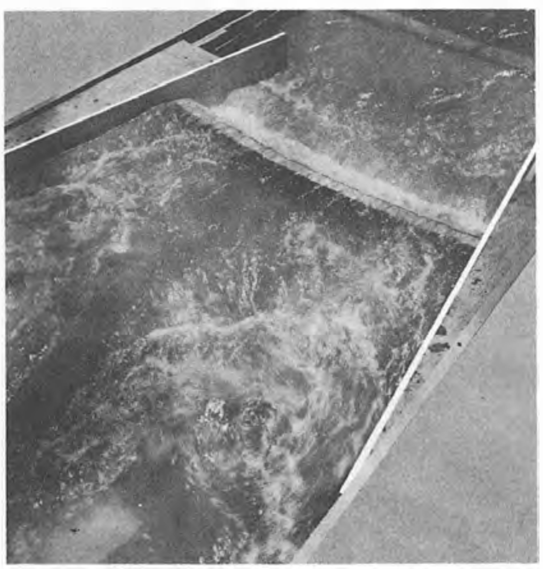


Fig. 18. Pool elev, 800
dischg, 6340 cfs; TW elev, 698.4

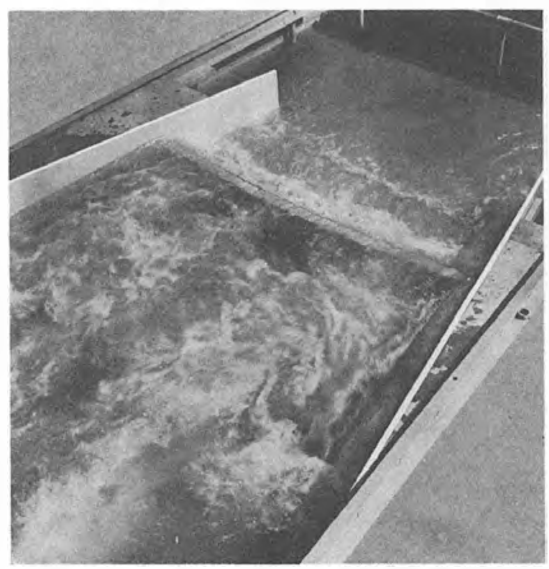


Fig. 19. Pool elev, 840
dischg, 7580 cfs; TW elev, 699.9

Type A-1 stilling basin, two sluices operating

End sill pressures

19. Since the jet from one sluice appeared to remain concentrated until it impinged on the end sill, it was feared that the end sill was

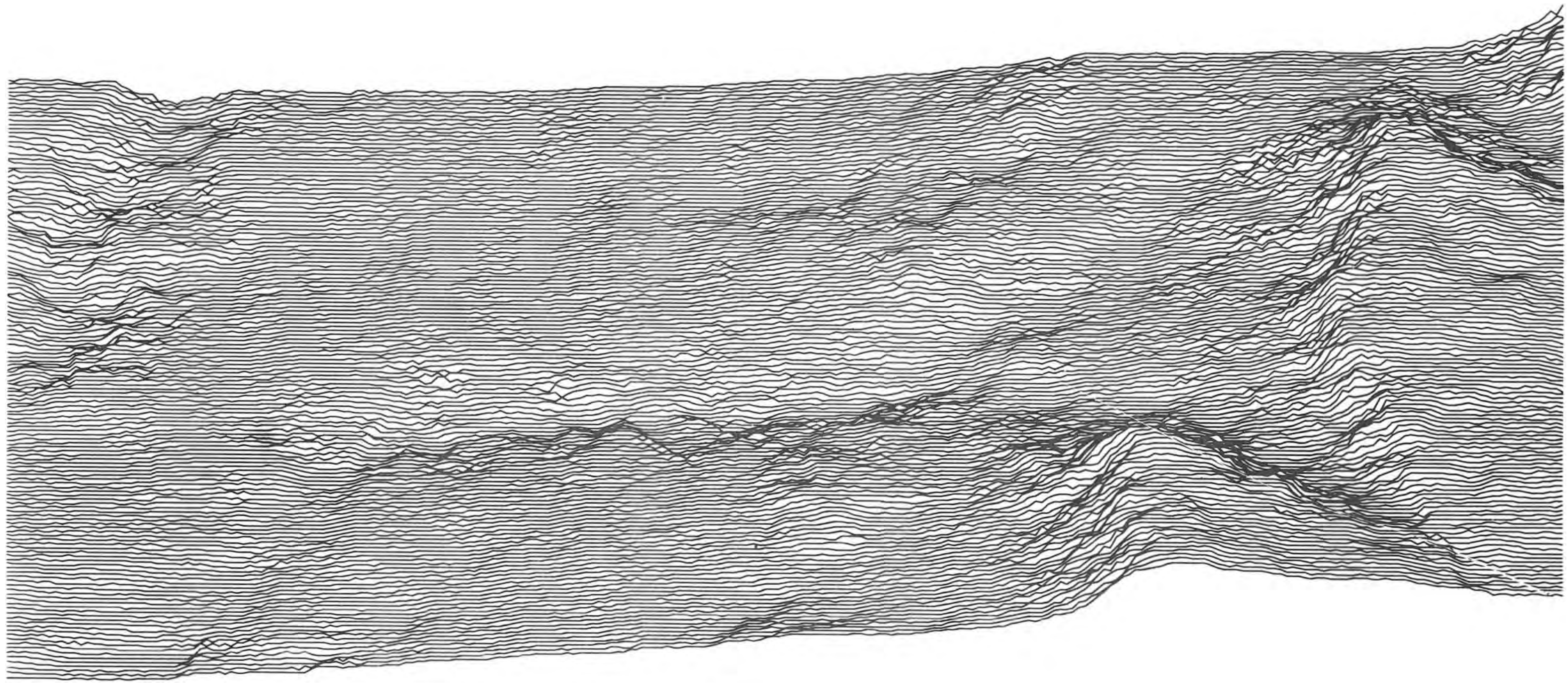


Figure 2. Computer line plot of the rectangular region shown in Figure 1

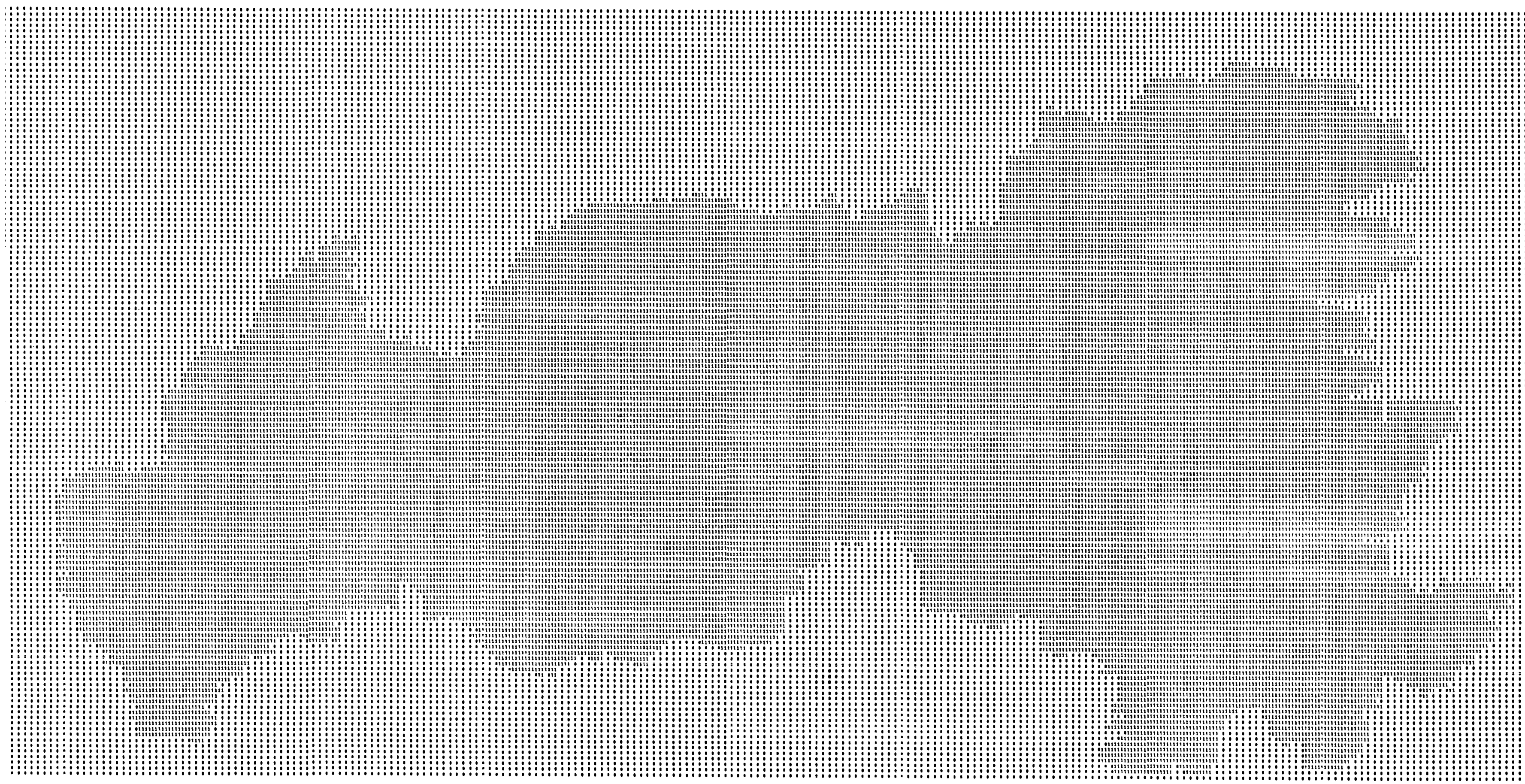


Figure 8. High-speed-printer map of watershed after deletion of islands

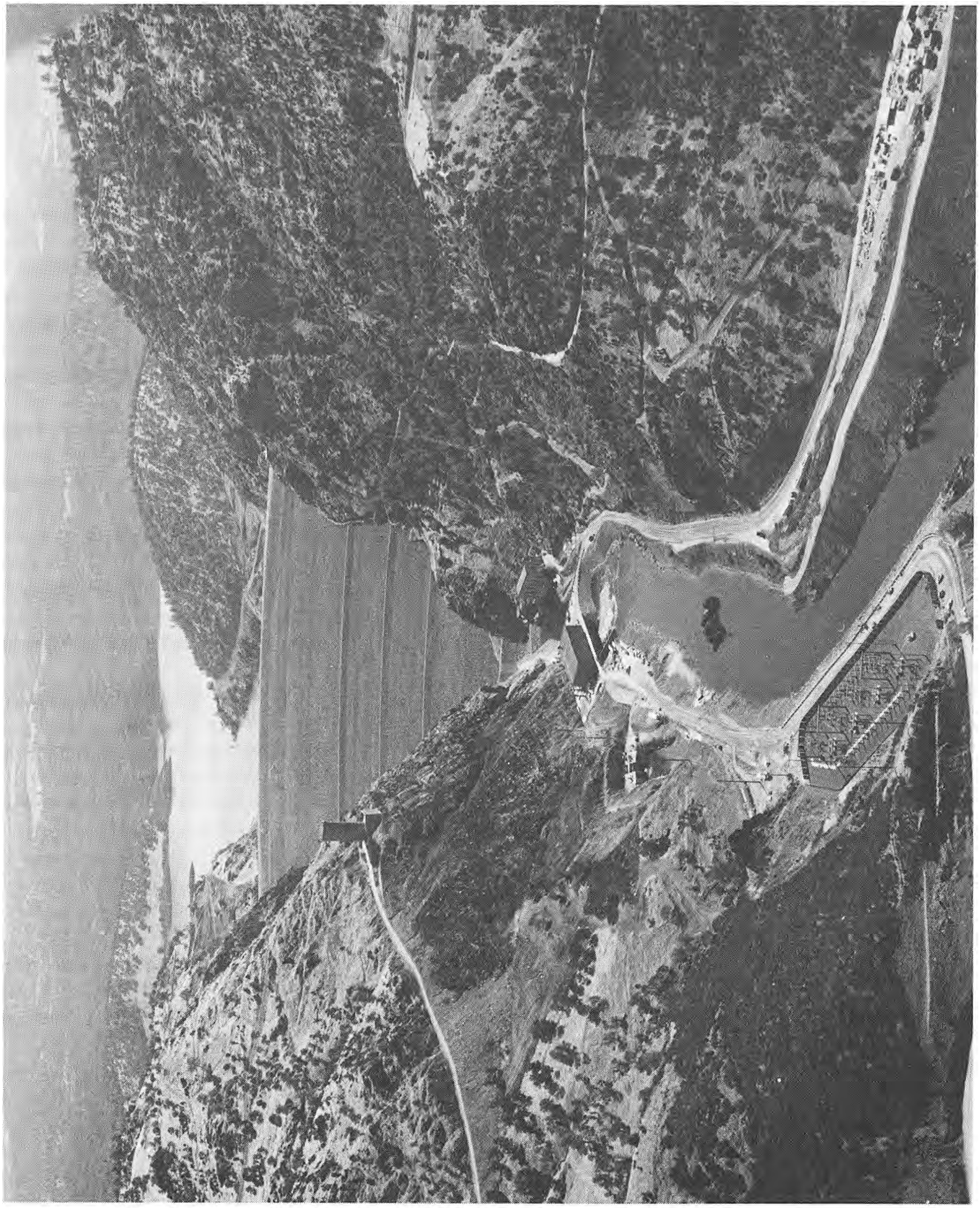


Figure 1. New Melones project

Figure 6. Recommended deflector plate design

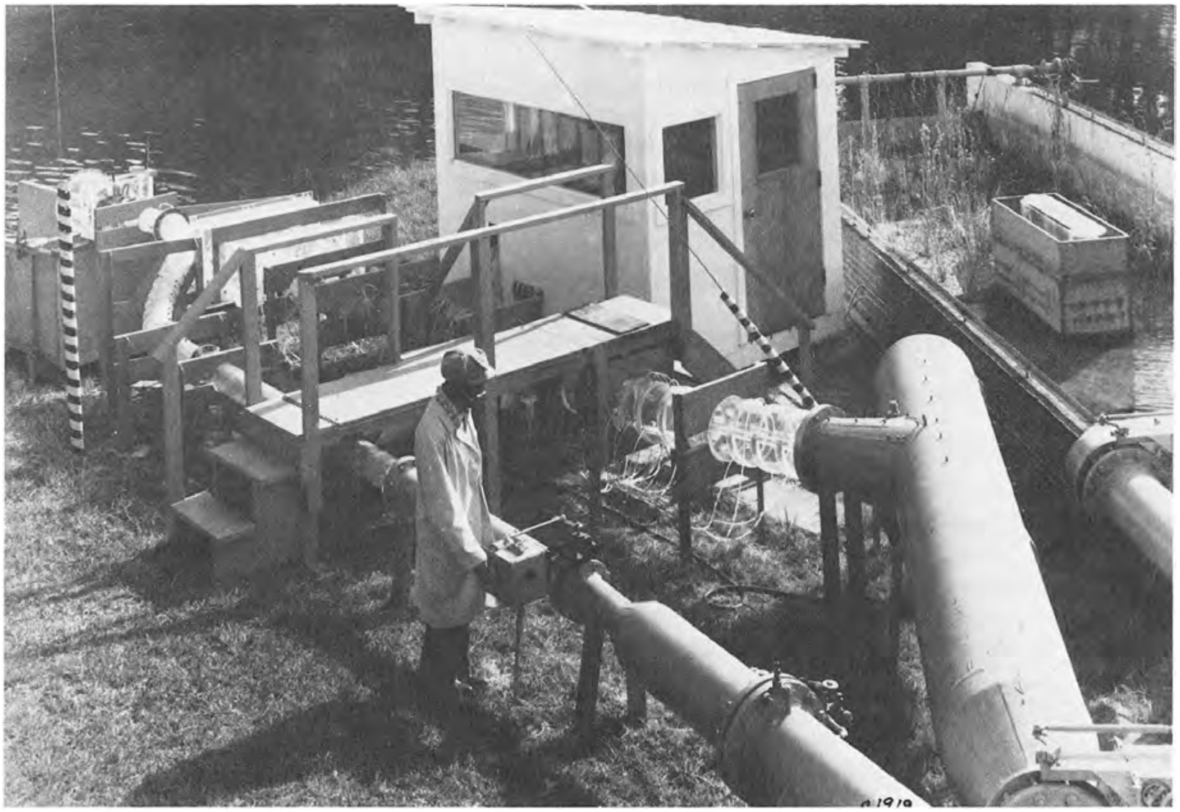


Figure 8. 1:12-scale model of New Melones fixed-cone valve

locations at which forces were measured is shown in Plate 16. The bottom vertical load cell (No. 5) was positioned near the center of gravity of the plastic hood and set to support the entire weight of the model hood (83 lb). This ensured that little vertical load was placed on the four horizontal load cells (Nos. 1-4). Rollers were provided with both vertical load cells (Nos. 5-6) to ensure freedom of movement in the horizontal direction.

35. Initial tests were conducted without the deflector plates to determine the force exerted upstream by the flow deflected by the backsplash cone. Load cell 6 was not used because no net upward force was anticipated. Time-histories of loading and the amplitude spectrum of loads measured with the maximum flow, pool el 1088, and a 28.1-in. sleeve travel are shown in Plates 17-21 for load cells 1-5, respectively. The left figure on each plate is the time-history of loading with the mean loading subtracted out. The right figure is the amplitude spectrum

Photo 2. Original design, failure of opposite bank, scour hole contour lines at el 495, 490, and 485

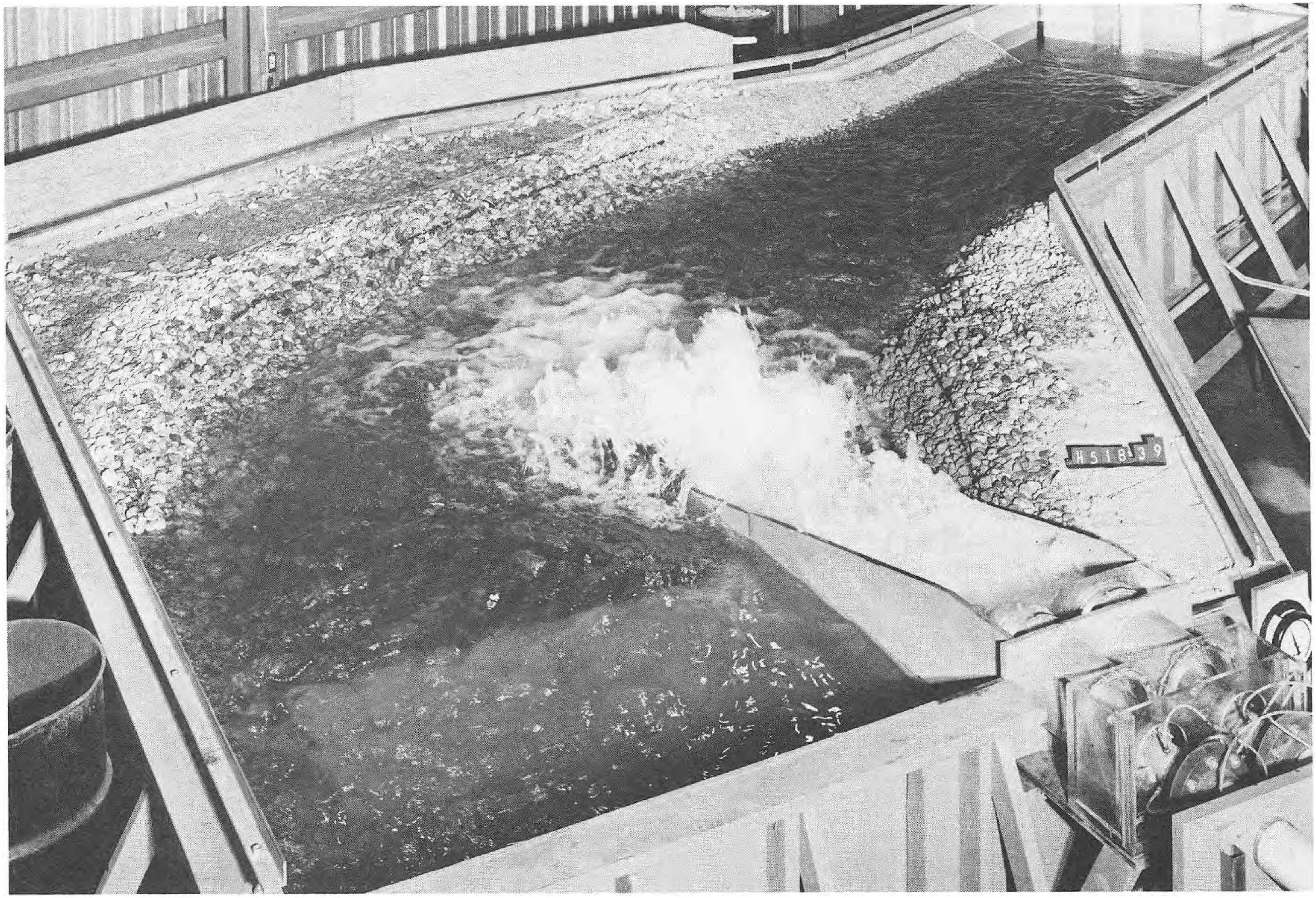


Photo 9. Recommended deflector plate design. Same flow as in Photo 9, showing attack on opposite bank

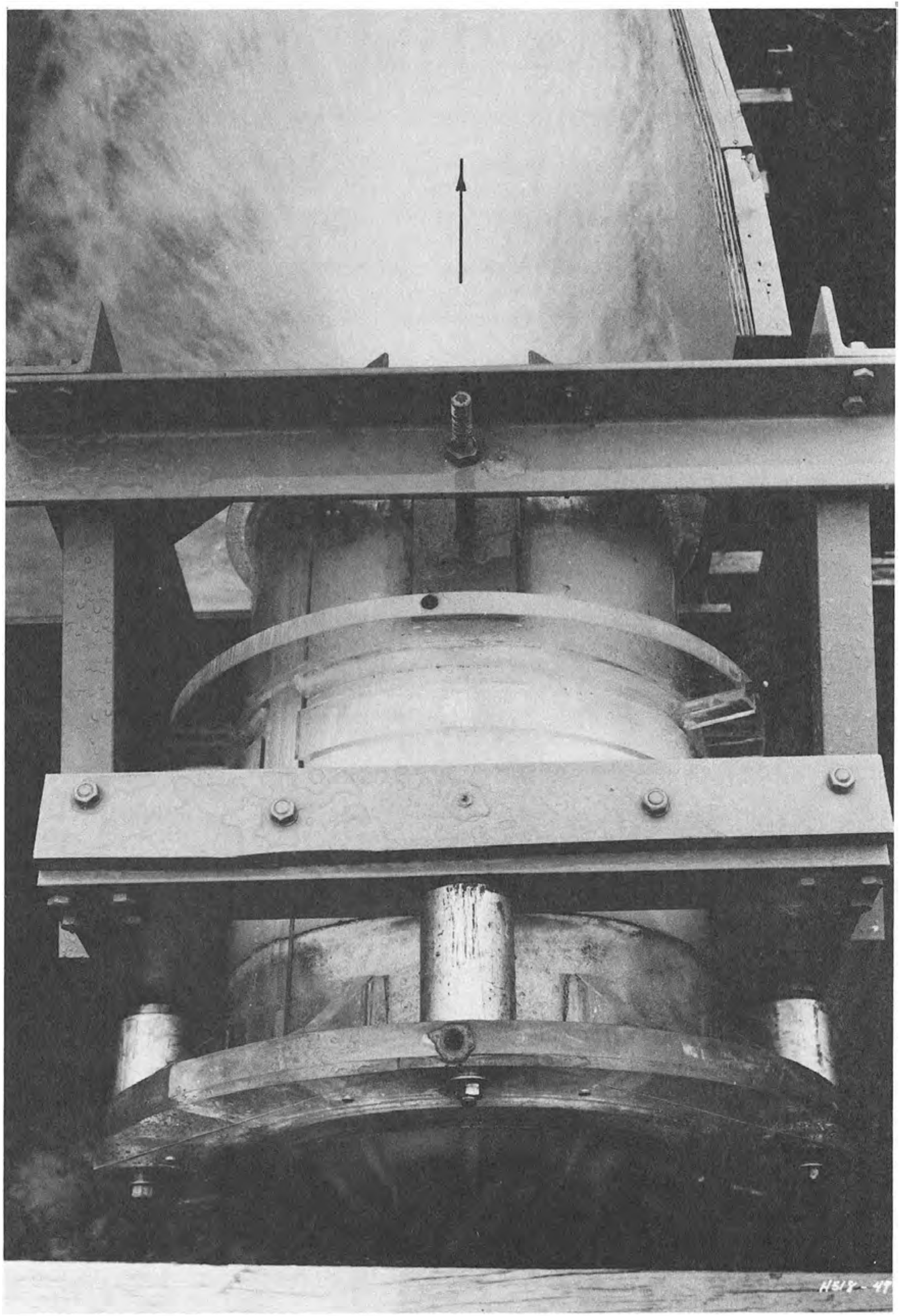


Photo 13. Recommended deflector plate design. Top view of flow conditions in 1:12-scale model

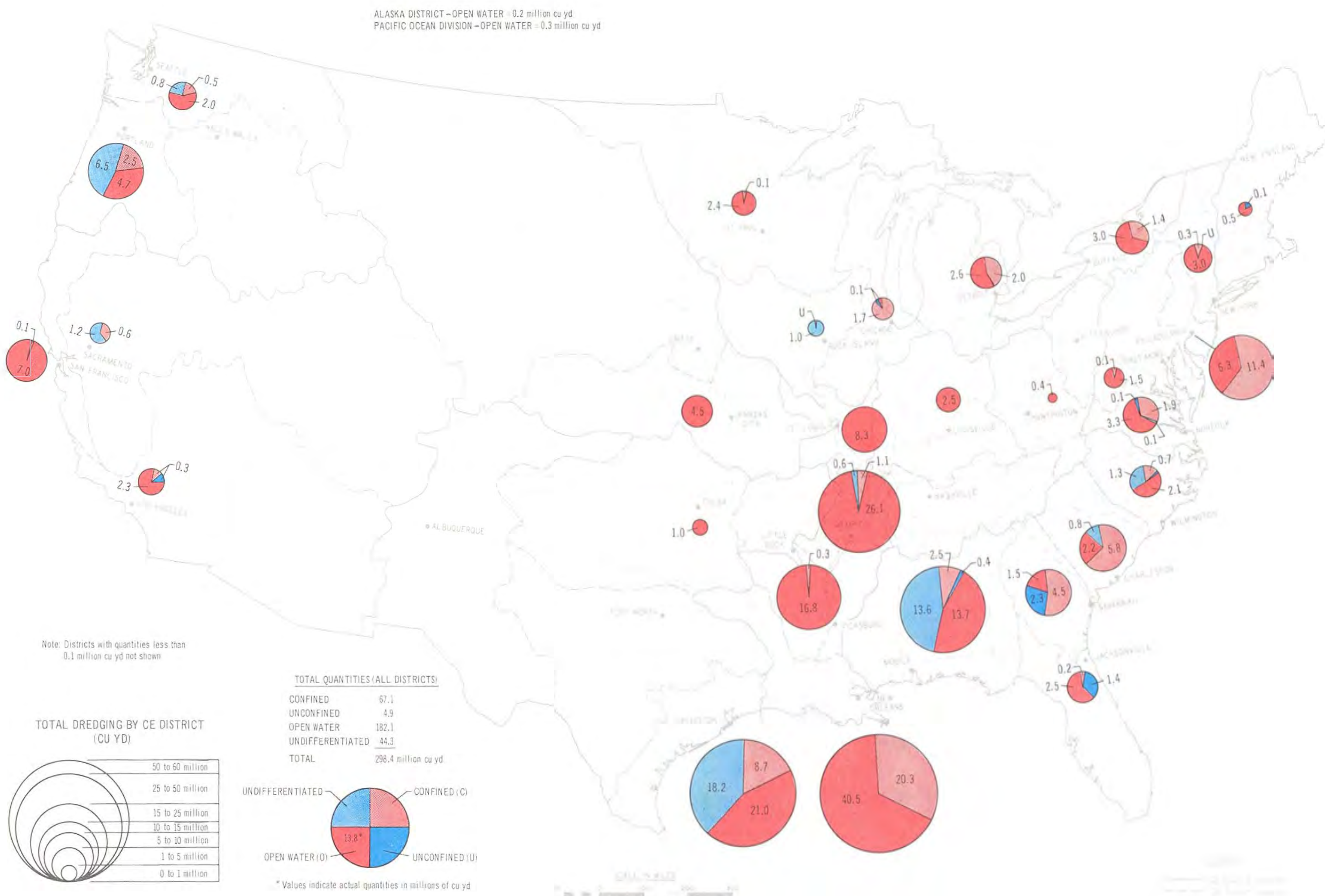


Fig. 2. Disposition of dredge spoil generated in maintenance dredging operations and average annual quantities of spoil disposed by area by District

80WSE_302 "Disposal of Dredge Spoil: Problem Identification and Assessment and Research Program Development," 1972

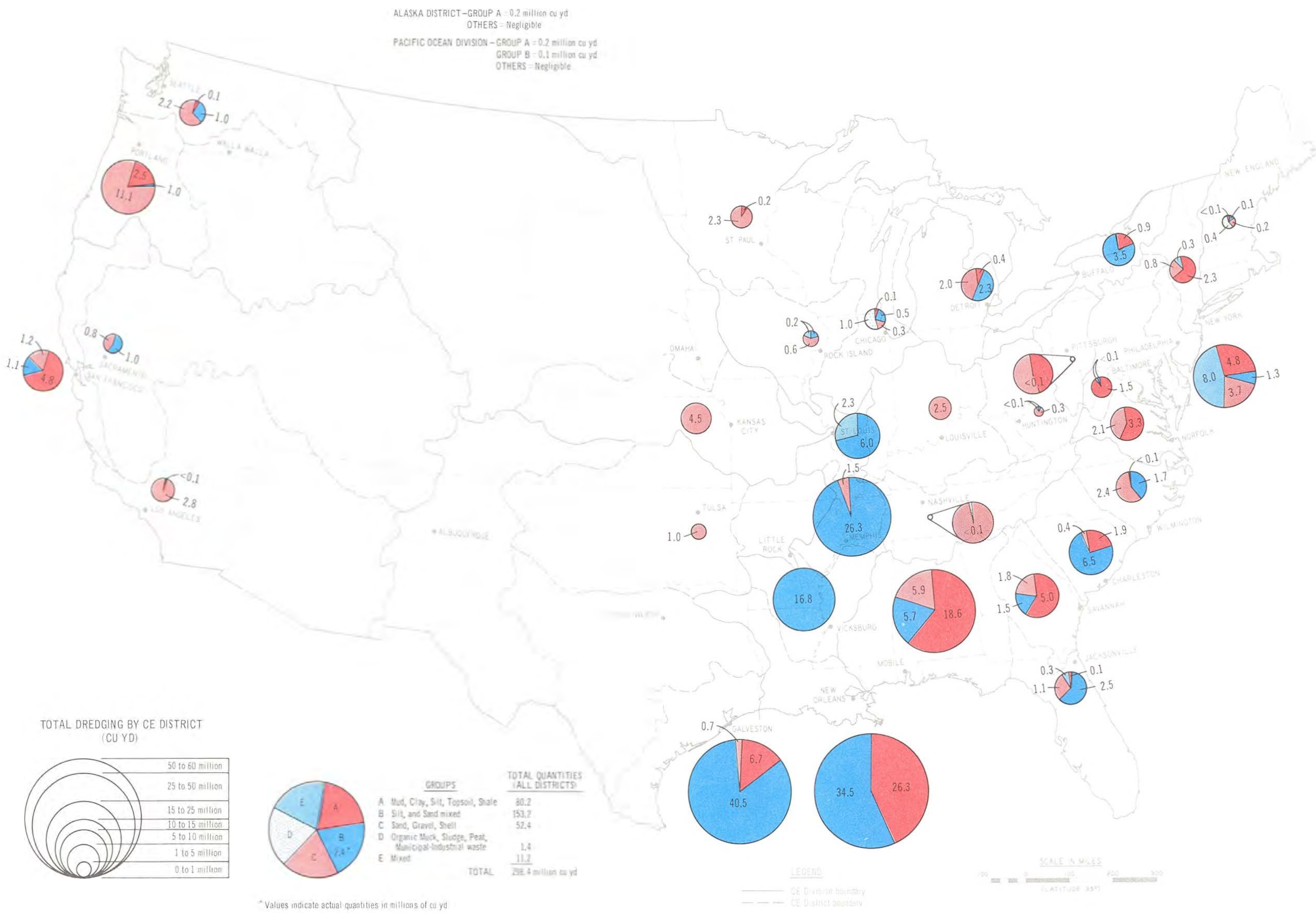


Fig. 3. Grain-size classification of spoil generated in maintenance dredging operations and average annual quantities of spoil by type by District

80WSE_302 "Disposal of Dredge Spoil: Problem Identification and Assessment and Research Program Development," 1972

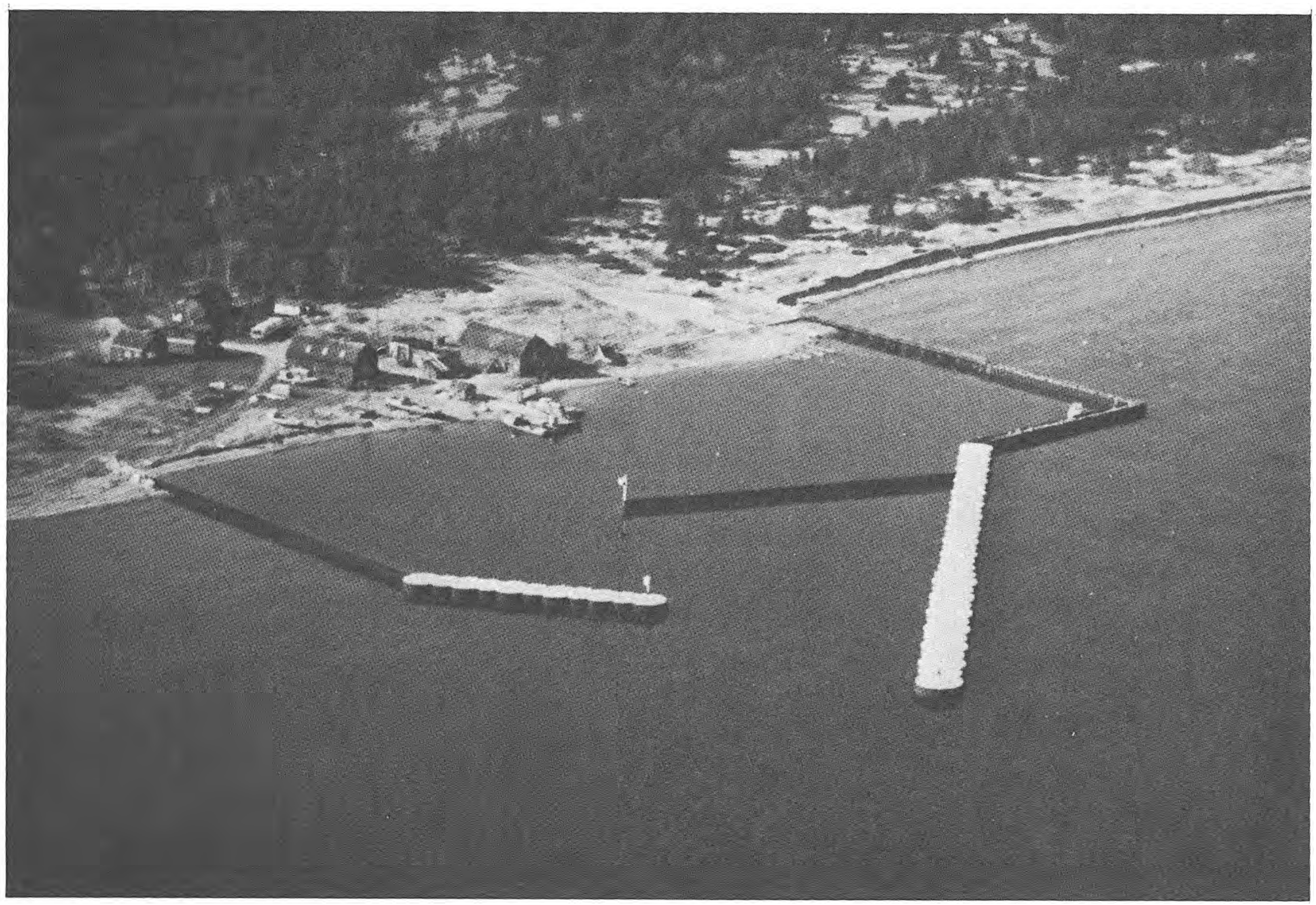


Figure 11. Combination sheet-steel pile and cellular breakwater construction, Whitefish Point Harbor of Refuge, Michigan, Lake Superior. (Additional construction has occurred in harbor since photograph was made.)

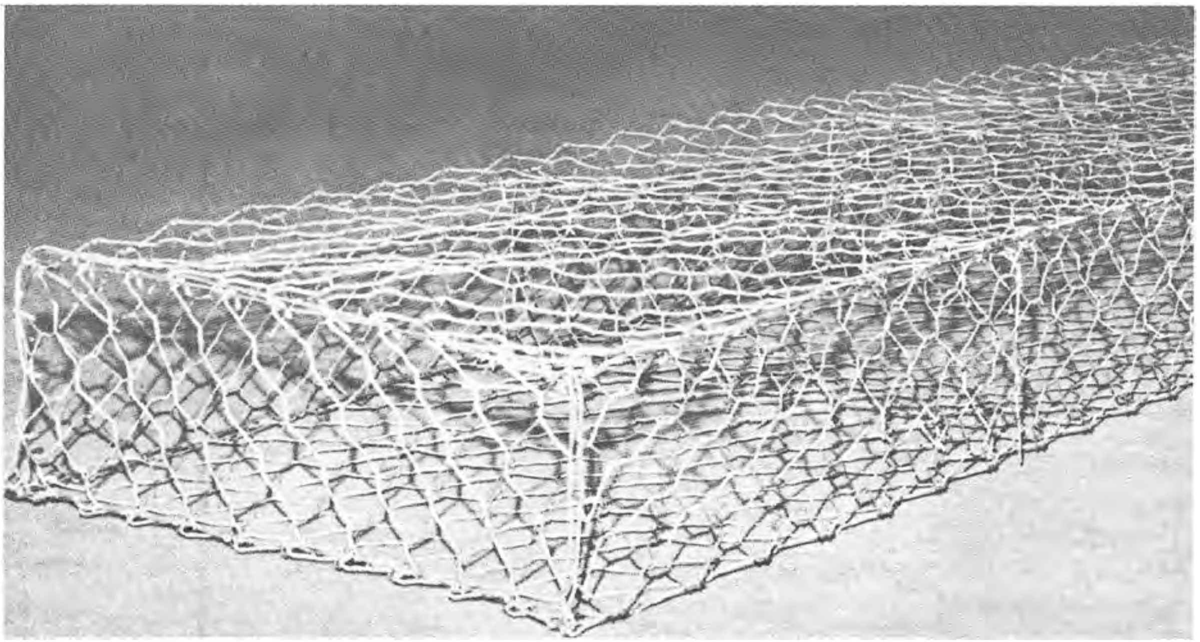
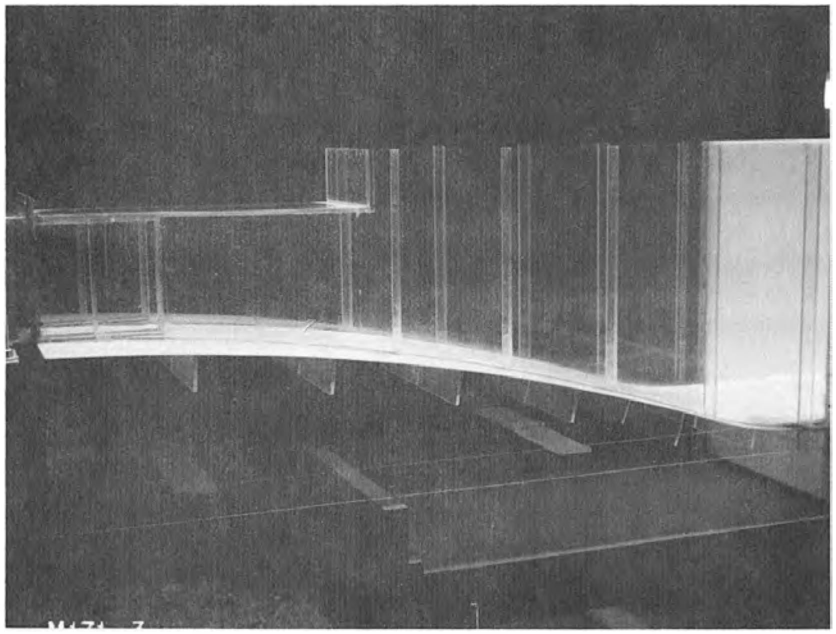


Figure 18. Gabion unit prior to filling with stone (after Terra Aqua Conservation, Bekaert Steel Wire Corp., 1977)

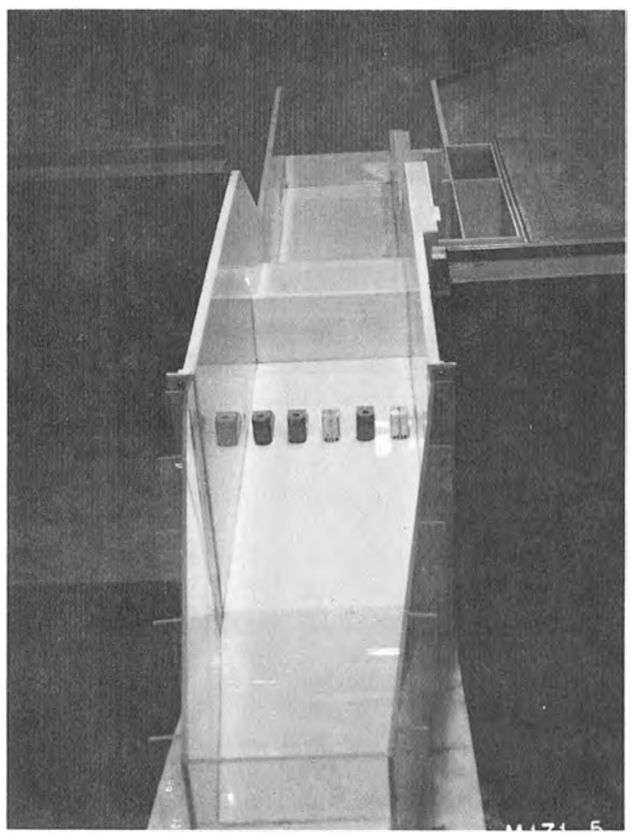
are compartmented rectangular containers made of galvanized steel hexagonal wire mesh and filled with stones. Compartments are formed of equal capacity by wire netting diaphragms or partitions. These partition walls add strength to the container and help retain its shape during the filling operation. They also provide assurance that the fill will remain evenly distributed, even after extensive settlement.

72. Gabion units are normally filled with hand-size stones, usually dumped into them mechanically. The filled gabion then becomes a large, flexible, and permeable building block from which a broad range of structures can be built. This is done by setting and wiring individual units together in courses and filling them in place, or by filling and then placing individual units. The wire mesh used in gabions is heavily galvanized. It may be safely used in fresh water and in areas where the pH (acidity indicator) is not greater than 11. For highly corrosive conditions, a PVC (polyvinyl chloride) coating should be used over the galvanizing. Such treatment is an economical solution to deterioration of the wire near the ocean, in some industrial areas, and in some polluted streams.

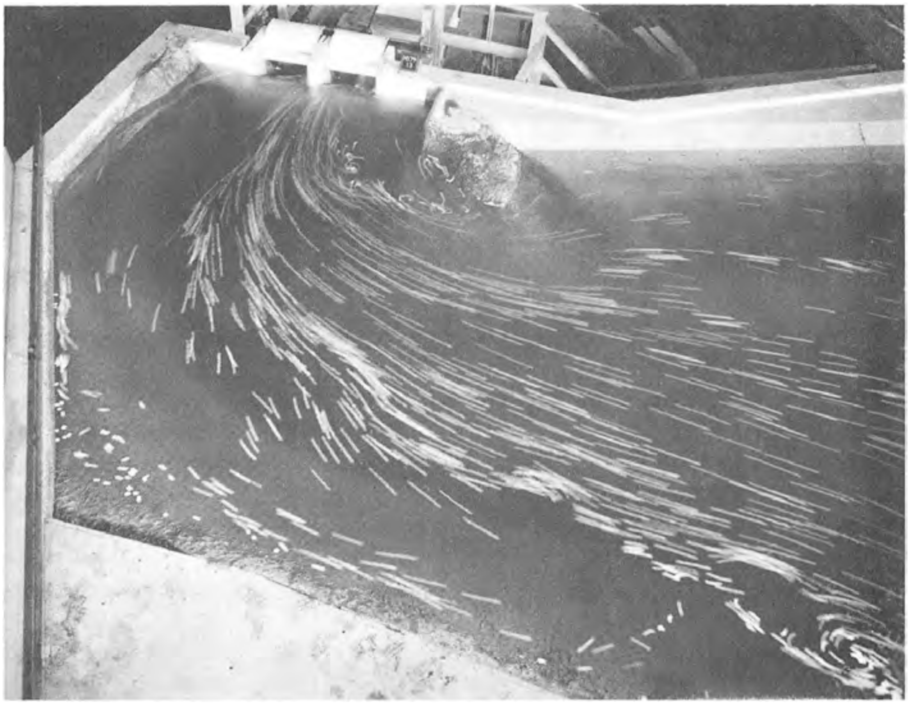
73. The foundation blanket specified for Masonboro Inlet will



Photograph 8. Tunnel exit and chute.



Photograph 9. Primary stilling basin.
View from upstream.



Photograph 13. Spillway approach channel at elevation 1,900, p/H_d = 1.01. Spillway discharge 93,600 cfs. Pool elevation 1,985.

80WSE_304 "Applegate Dam Spillway and Outlet Works, Applegate River, Oregon," 1984

Figure 19. Artificial islands add habitat and visual diversity to borrow pits and other aquatic areas. Although the island shown here is in an urban area and was landscaped for aesthetics, extensive use by waterfowl for nesting and loafing was observed

8 ft and were 4 in. thick. A nesting square 2 by 2 ft by 2 in. thick was placed in the center of each platform. Styrofoam was partly covered with chicken wire, soil, and brush. The entire platform was framed by 8-in.-wide logs and anchored with a rope and a 25- to 30-lb rock. An alternate design specifically for Canada geese is given by Will and Crawford (1970).

231. Islands created by excavation or fill can be created using dredges, bulldozers, draglines, or similar equipment. Coarse aggregates for base provide stability, while finer materials on the surface allow for the establishment of vegetation. Normally, vegetation will establish itself naturally, but planting may be advisable for some soils (see discussion on wildlife seeding and planting, paragraphs 317-347), and watering could be appropriate in arid climates (Giroux 1981a). If the primary objective is enhancement of nesting cover for the prairie duck complex, vegetation should not be allowed to grow densely on the island except on its windward side (Giroux 1981a). Periodic burning or mechanical scarification may be desirable (Duebbert 1982). To further minimize erosion at sensitive sites, islands may be situated along the lee shore of the basin and oriented parallel to prevailing winds and

Figure 21. Excess dredged material was deposited in thin layers on this foreshore site in Fulton, Ill., and will be planted to vegetation valuable as wildlife habitat

lush growth of vegetation that will provide shading for borrow pit ponds. Moreover, thin spreading will slightly raise the elevation of foreshore areas so that less flood-tolerant species can be used. Specific recommendations for developing upland habitats for wildlife from dredged material are provided by Hunt et al. (1978) and Walsh and Malkasian (1978).

280. If material is suitable for a low dike or earthen dam, ponds or wetlands can be created and serve as valuable wildlife habitat. Wetlands created by impoundment are more amenable to management through water control (paragraph 213) than those created by excavation.

281. Another option involves placing the excess material in mounds. Such placement is used to add diversity to an otherwise flat topography (Figure 22). Additionally, mounds that are placed on windward sites of borrow pits can shelter waterfowl and other wildlife during high winds, and mounds generally can serve as wildlife retreats during rapidly rising floodwaters (Grizzel and Vogan 1973).

282. The best height for material placement depends on resource

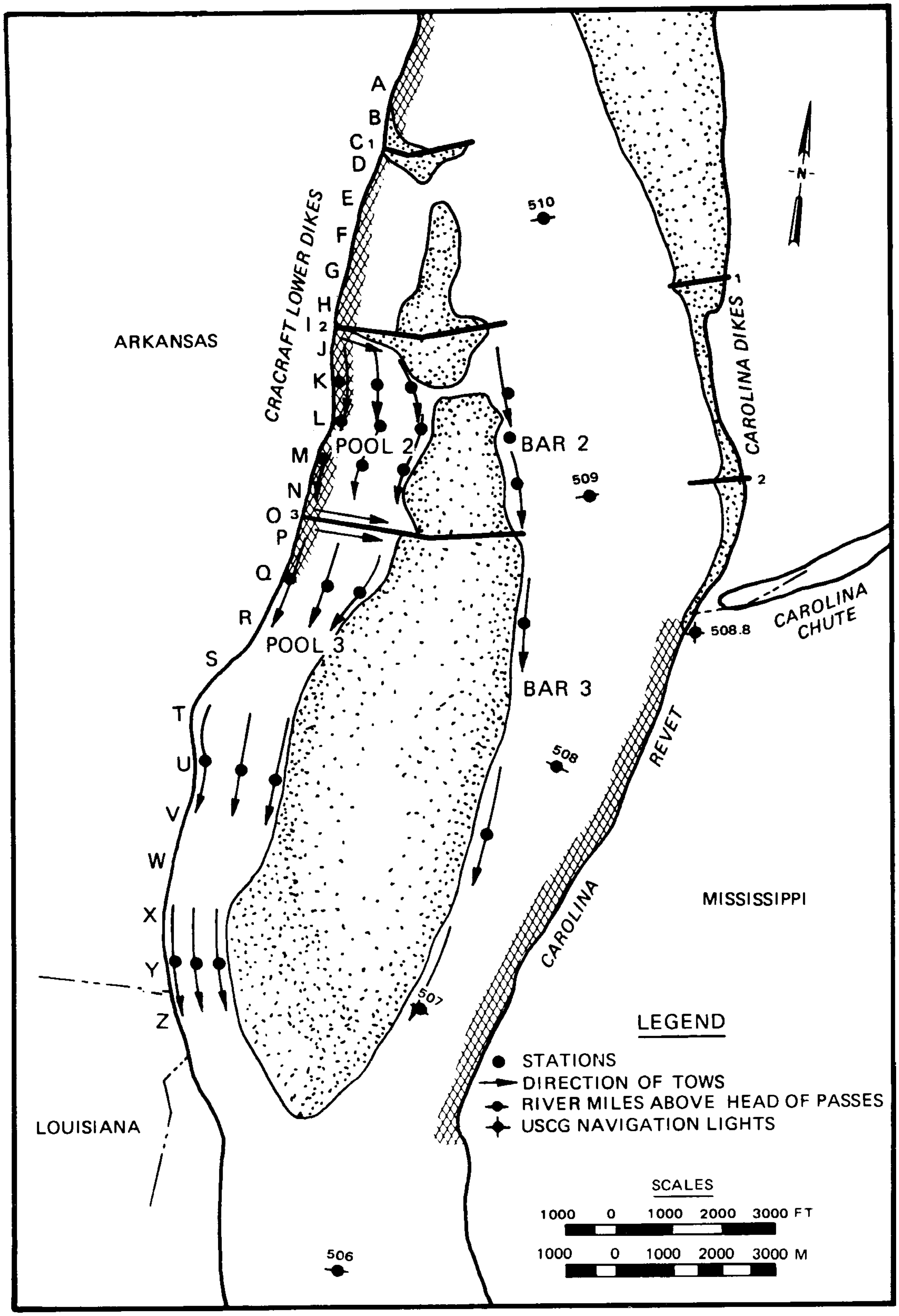


Figure 1. Map of Cracraft Dike Field with water quality and fish sampling stations indicated (USCG = U. S. Coast Guard)

80WSE_306 “Fish of Two Dike Pools in the Lower Mississippi River,” 1984

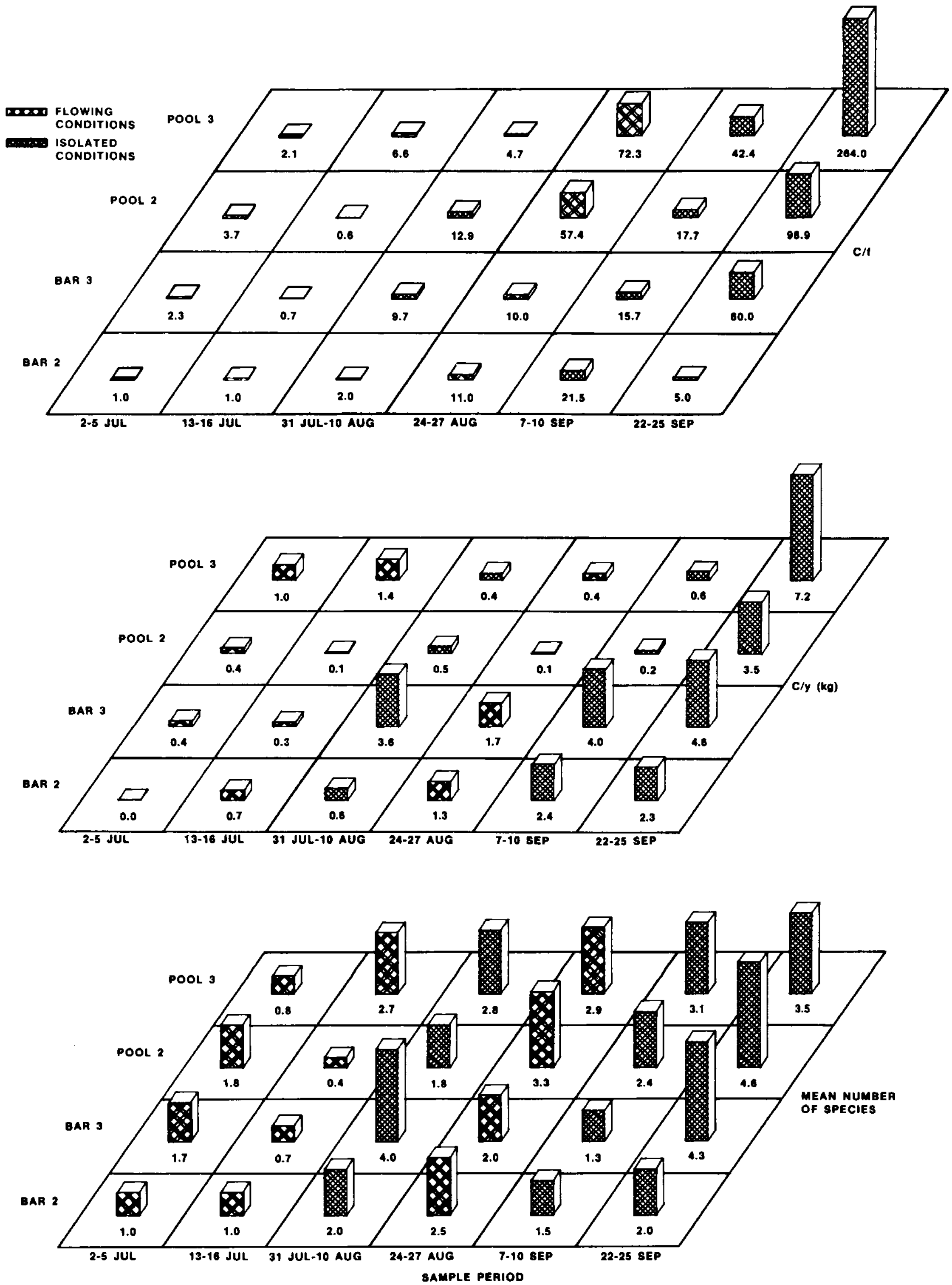


Figure 7. Mean catch per unit of effort (C/f), mean weight (C/y), and mean number of species for electroshocking data over time

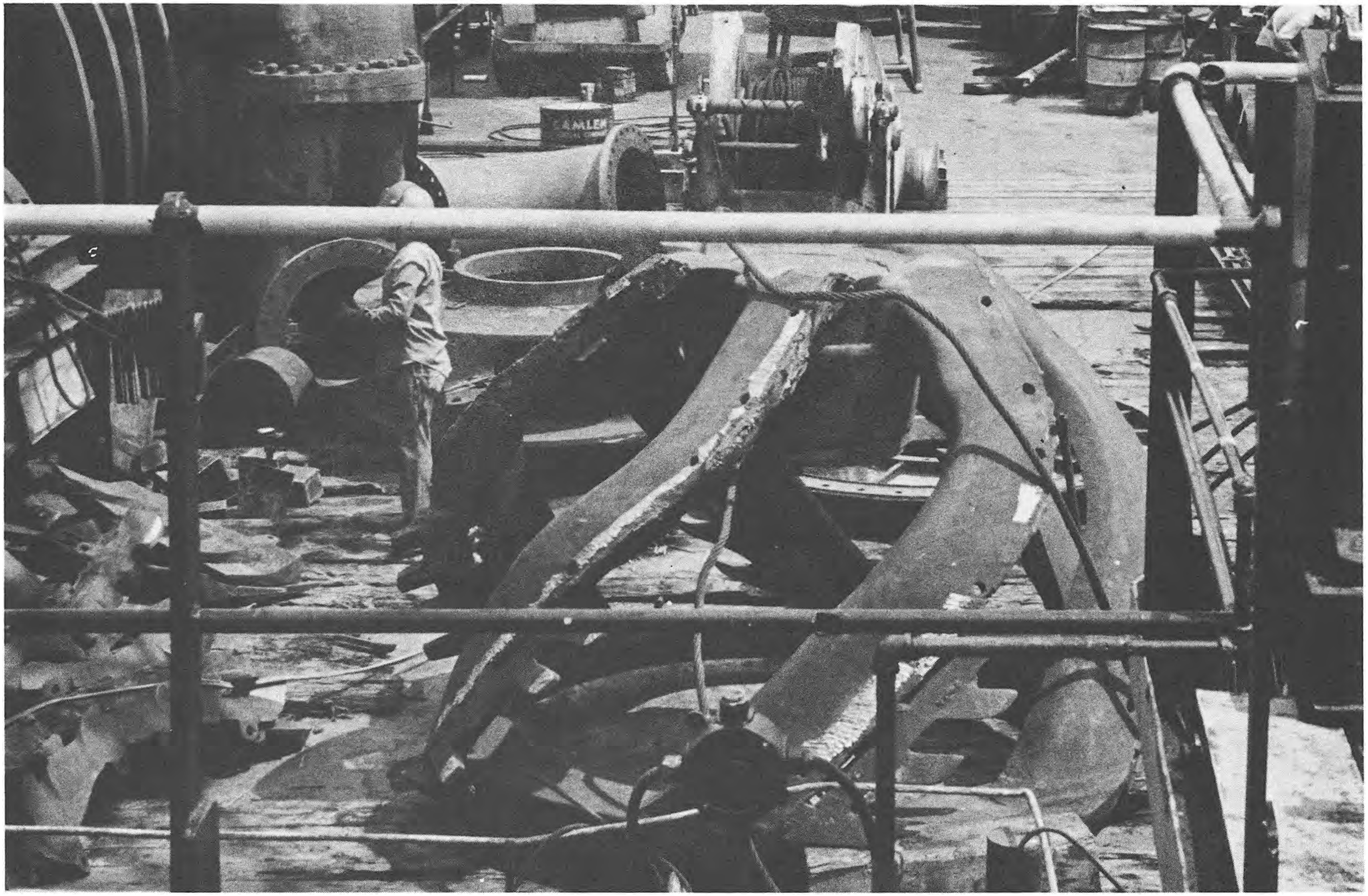


Figure 7. Sloppy housekeeping on a dredge. Cutter in foreground (similar to the one used during the tests) has about half of its teeth removed for replacement because of wear.

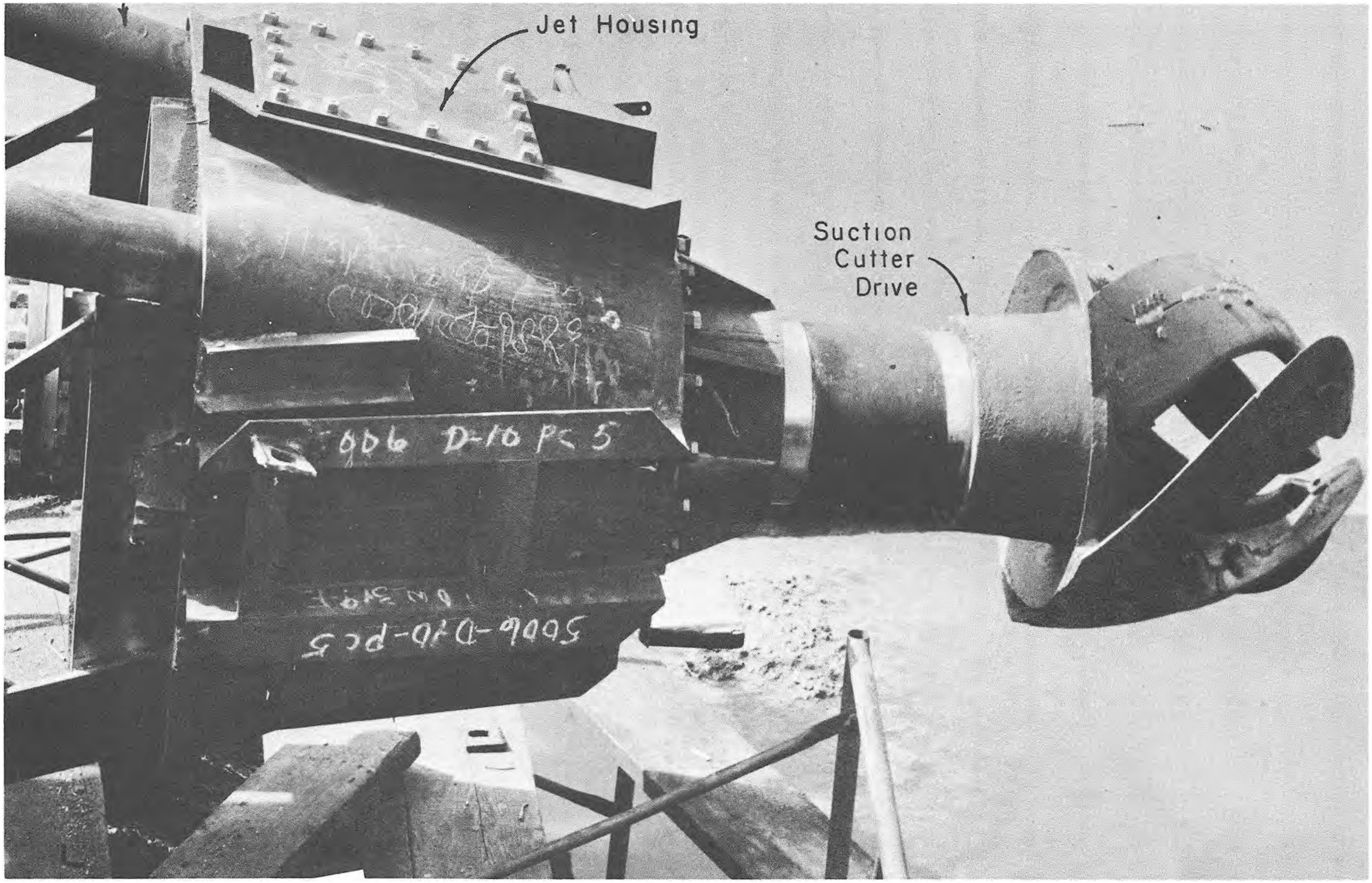


Figure 9. Cutter-suction assembly and jet on a cutterhead.

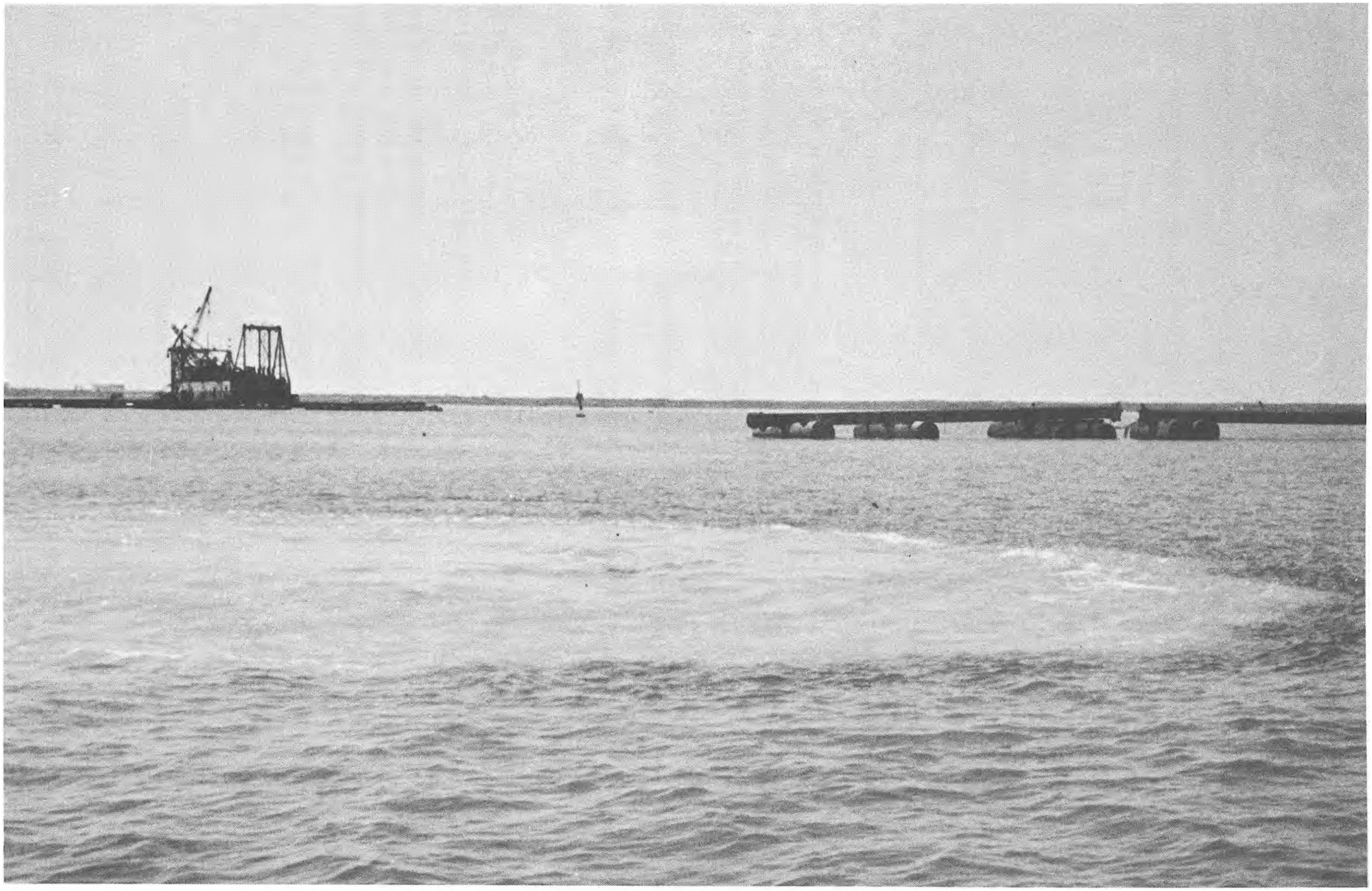


Figure 11. Tender-created turbidity during maneuvering.

Figure 12. Turbidity creation by dredge.plant equipment.

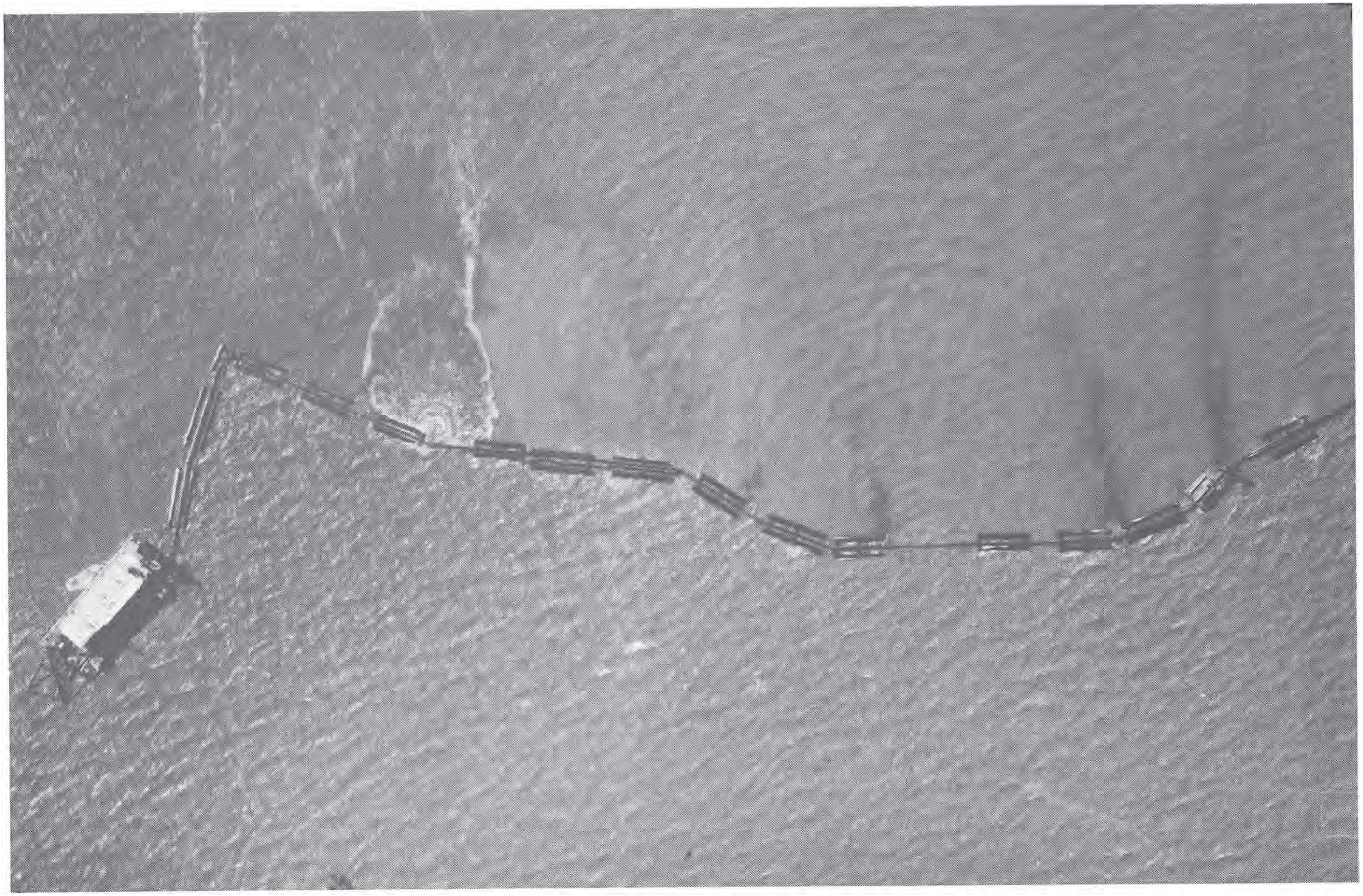


Figure 14. Turbidity creation by leaking floating line connections.

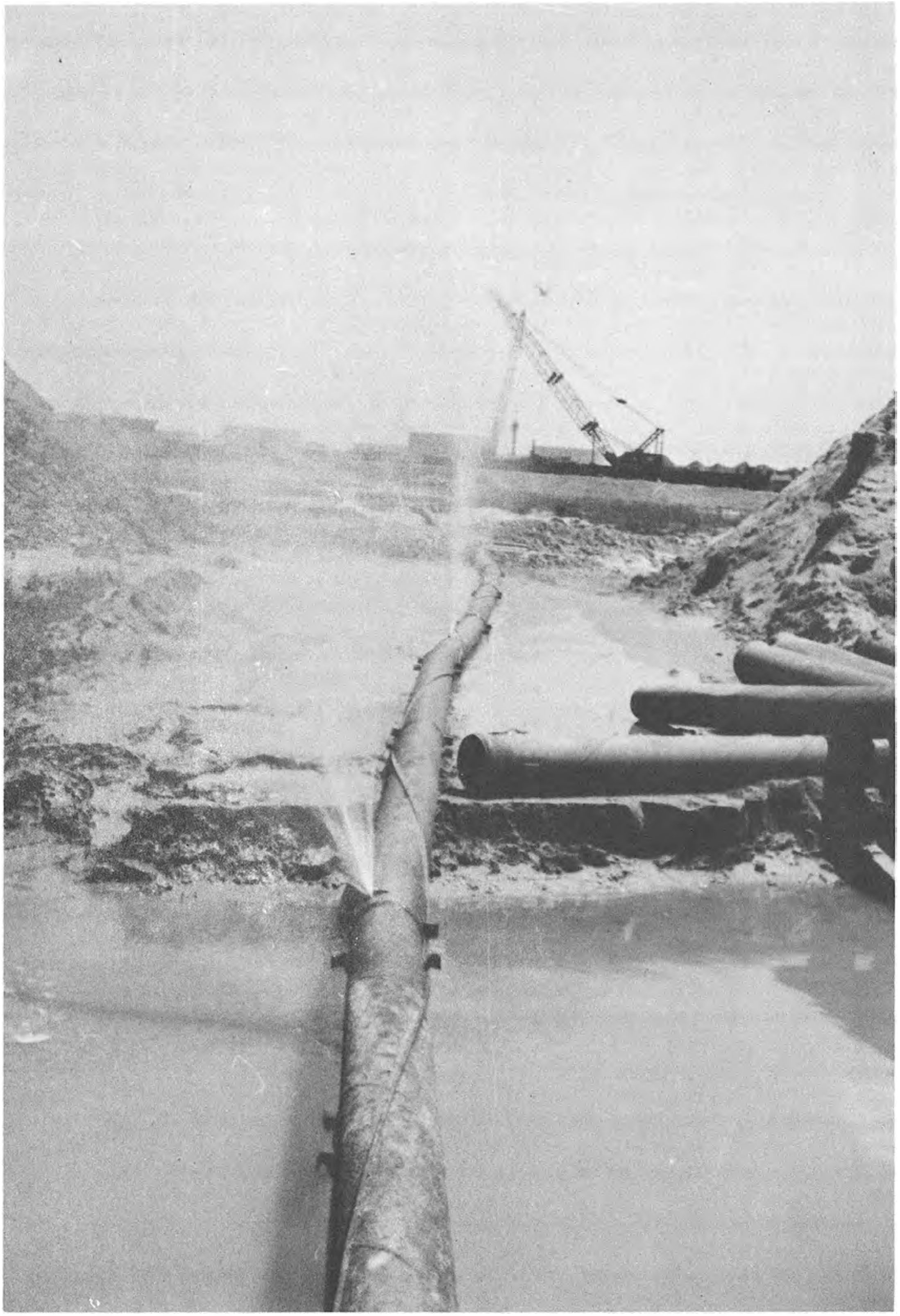


Figure 15. Shore line connection leakage under high pressure.

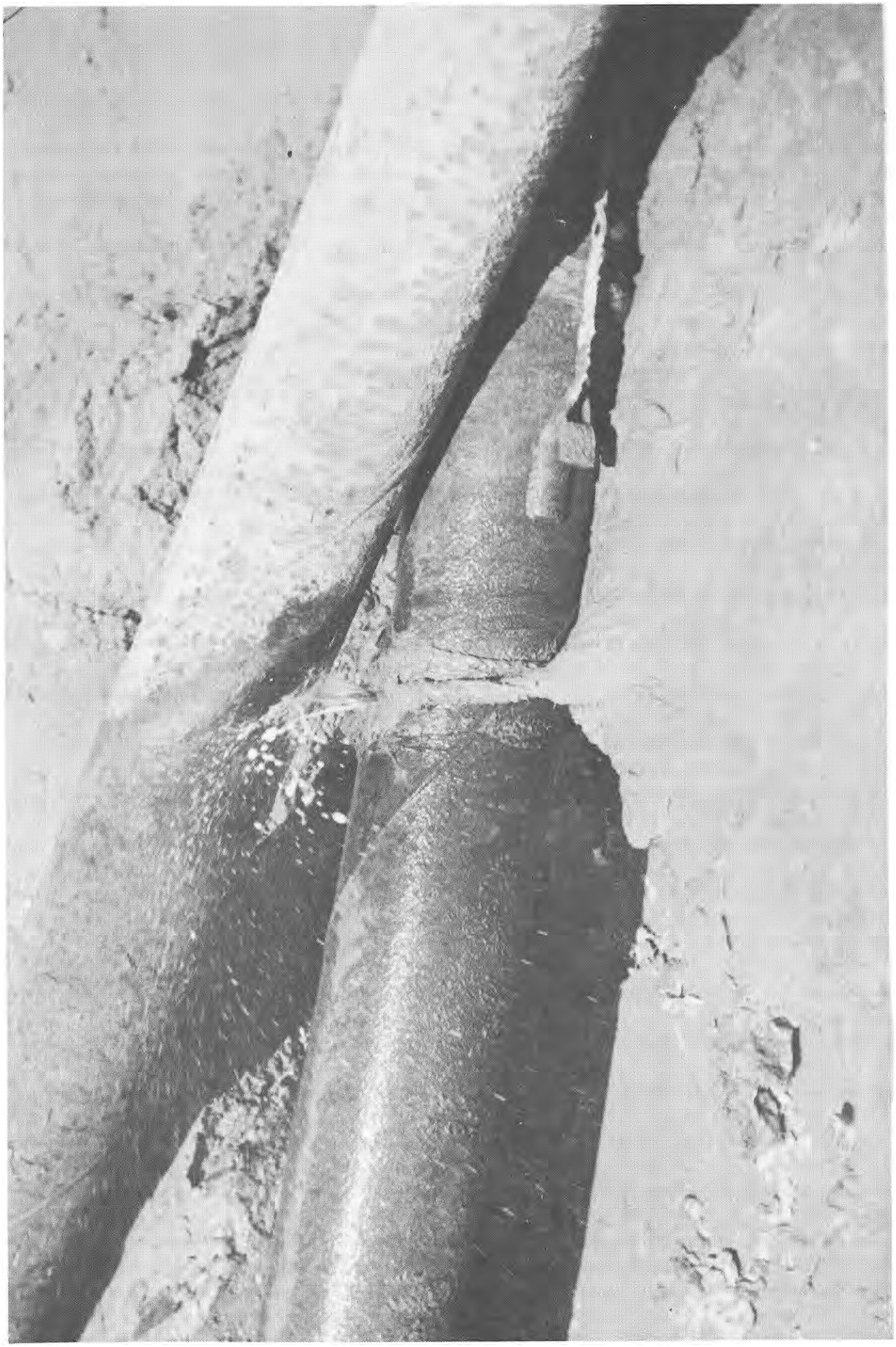


Figure 16. Shore line connection leakage.

Figure 21. Cutter that has been buried and all material not picked up.

Table 1

Plant Indicator Status Categories*

Indicator Category	Indicator Symbol	Definition
OBLIGATE WETLAND PLANTS	OBL	Plants that occur almost always (estimated probability >99%) in wetlands under natural conditions, but which may also occur rarely (estimated probability <1%) in nonwetlands. Examples: *Spartina alterniflora*, *Taxodium distichum*.
FACULTATIVE WETLAND PLANTS	FACW	Plants that occur usually (estimated probability >67% to 99%) in wetlands, but also occur (estimated probability 1% to 33% in nonwetlands). Examples: *Fraxinus pennsylvanica*, *Cornus stolonifera*.
FACULTATIVE PLANTS	FAC	Plants with a similar likelihood (estimated probability 33% to 67%) of occurring in both wetlands and nonwetlands. Examples: *Gleditsia triacanthos*, *Smilax rotundifolia*.
FACULTATIVE UPLAND PLANTS	FACU	Plants that occur sometimes (estimated probability 1% to <33%) in wetlands, but occur more often (estimated probability >67% to 99%) in nonwetlands. Examples: *Quercus rubra*, *Potentilla arguta*.
OBLIGATE UPLAND PLANTS	UPL	Plants that occur rarely (estimated probability <1%) in wetlands, but occur almost always (estimated probability >99%) in nonwetlands under natural conditions. Examples: *Pinus echinata*, *Bromus mollis*.

* Categories were originally developed and defined by the USFWS National Wetlands Inventory and subsequently modified by the National Plant List Panel. The three facultative categories are subdivided by (+) and (-) modifiers (see Appendix C, Section 1).

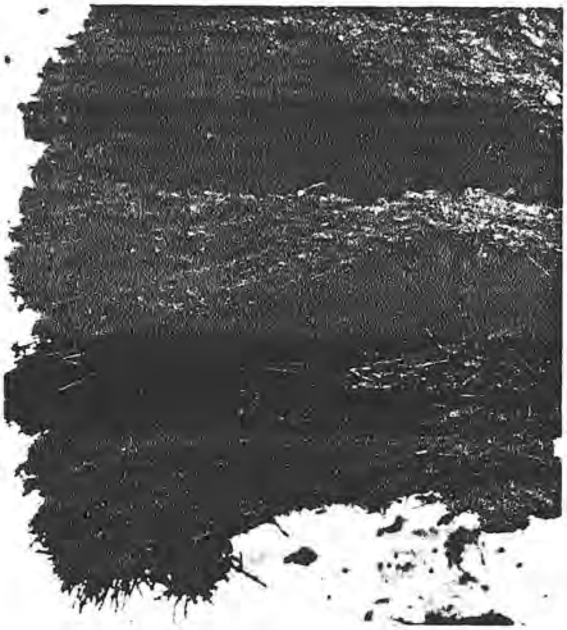


Figure 3. Organic soil

Figure 4. Gleyed soil

Figure 5. Soil showing matrix (brown) and mottles (reddish-brown)

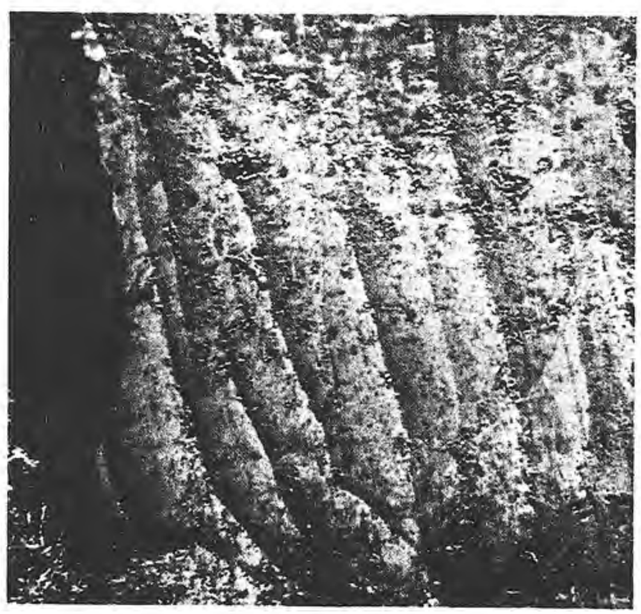


Figure 6. Iron and manganese concretions

Figure 7. Watermark on trees

Figure 8. Absence of leaf litter and drift line (extreme left)

Figure 9. Sediment deposit on plants

Figure 10. Encrusted detritus

DATA FORM 1

WETLAND DETERMINATION

Applicant Name:________________________ Application Number:_______________ Project Name:________________

State:__________County:__________Legal Description: Township:_____Range:_____

Date:________________ Plot No.:__________________ Section:_________________

Vegetation [list the three dominant species in each vegetation layer (5 if only 1 or 2 layers)]. Indicate species with observed morphological or known physiological adaptations with an asterisk.

Species	Indicator Status	Species	Indicator Status
Trees		Herbs	
1.		7.	
2.		8.	
3.		9.	
Saplings/shrubs		Woody vines	
4.		10.	
5.		11.	
6.		12.	

% of species that are OBL, FACW, and/or FAC:____. Other indicators:_________.

Hydrophytic vegetation: Yes ____ No ____. Basis:___________________________.

Soil

Series and phase:___________________ On hydric soils list? Yes_____; No____.

Mottled: Yes_____; No_____. Mottle color:__________; Matrix color:_________.

Gleyed: Yes_____ No_____ Other indicators:________________________________.

Hydric soils: Yes_____ No_____; Basis:_____________________________________.

Hydrology

Inundated: Yes_____; No_____. Depth of standing water:_____________________.

Saturated soils: Yes_____; No_____. Depth to saturated soil:_______________.

Other indicators:____.

Wetland hydrology: Yes_____; No_____. Basis:______________________________.

Atypical situation: Yes____; No_____.

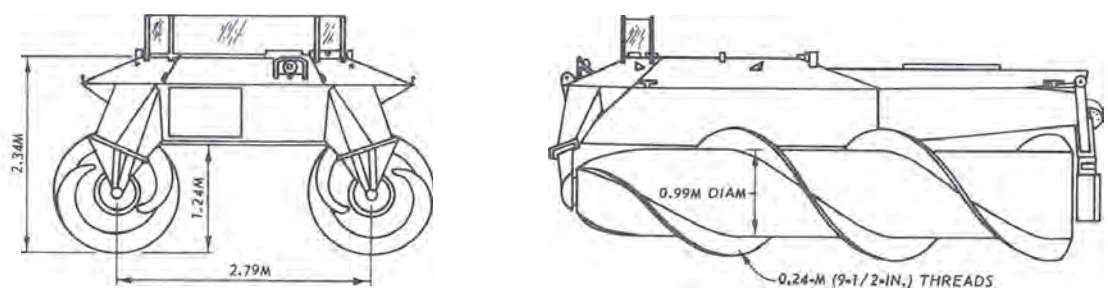
Normal Circumstances? Yes_____ No_____.

Wetland Determination: Wetland___________________; Nonwetland_______________.

Comments:

Determined by:_________________________

Figure 12. The Riverine Utility Craft or RUC



RIVERINE UTILITY CRAFT

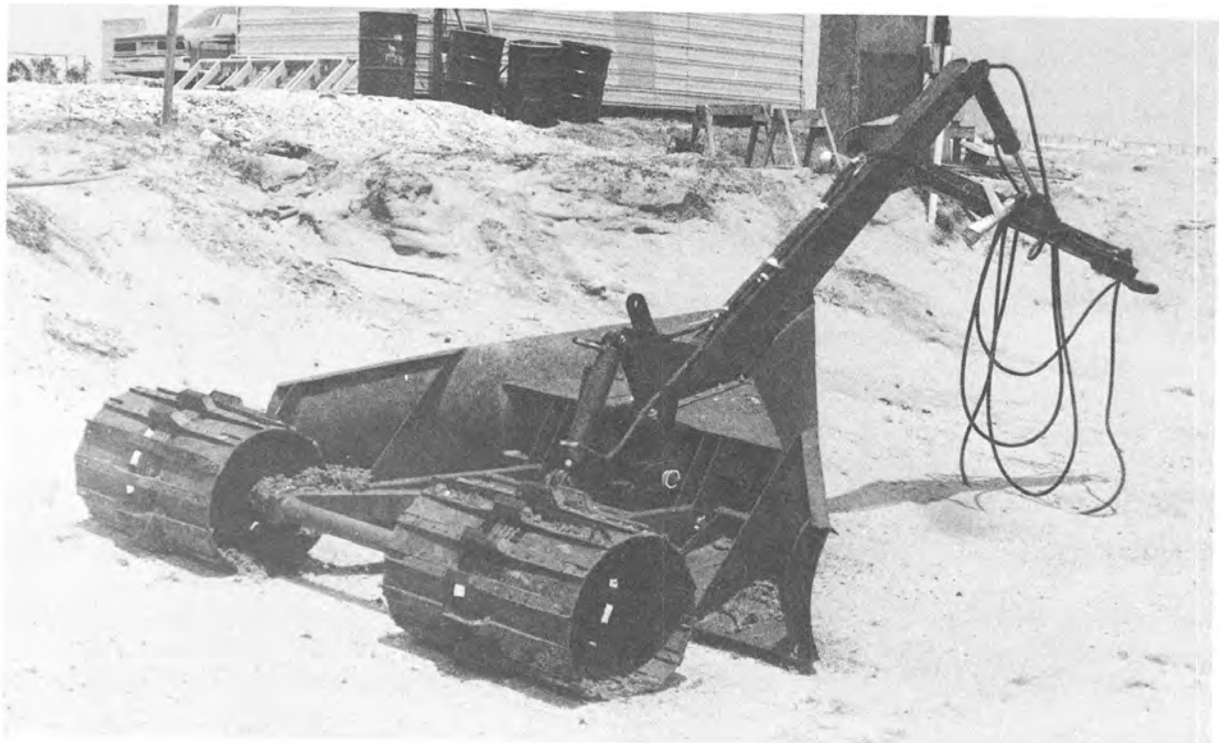
EMPTY WEIGHT (INCLUDING DRIVER AND FUEL)	4990 KG	(11,000 LB)
GROSS WEIGHT (DRIVER, FUEL AND PAYLOAD)	5900 KG	(13,000 LB)
LENGTH (OVERALL)	6 25 M	(21.5 FT)
WIDTH (OVERALL)	4.27 M	(14 FT)
HEIGHT (OVERALL, LESS WINDSHIELD)	2.34 M	(7.67 FT)
ROTOR SPACING (CENTER TO CENTER)	2.79 M	(110 IN.)
ROTOR DIAMETER (DRUM ONLY)	0.99 M	(39 IN.)
ROTOR DIAMETER (OVER HELIX)	1.47 M	(58 IN.)
ROTOR LENGTH (OVERALL)	5.64 M	(222 IN.)
ROTOR LENGTH (IN CONTACT WITH GROUND, NO RUT)	4.95 M	(195 IN.)
GROUND CLEARANCE	1.24 M	(49 IN.)
FLOATING DEPTH (EMPTY) (WATER)	0.55 M	(21.5 IN.)
FLOATING DEPTH (LOADED) (WATER)	0.61 M	(24 IN.)

Figure 13. General specifications for the RUC

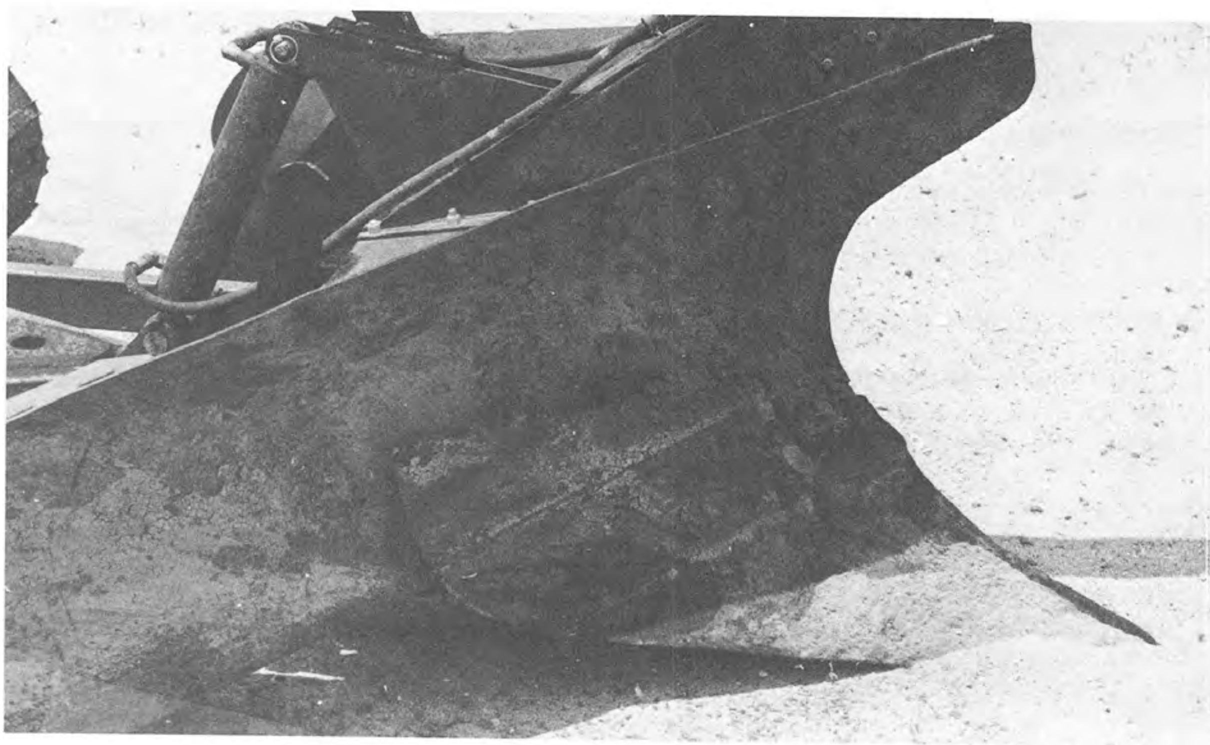
originally developed by the Dutch, who use the Amphirol, a small twin archimedean screw vehicle approximately one third the RUC's size, to make initial drainage trenches during their polder reclamation operations.[14] WES was not able to obtain an Amphirol for trenching evaluation, but studies were conducted with the Marsh Screw Amphibian shown in Figure 22, a prototype military vehicle approximately the same size and horsepower as the Amphirol. The Marsh Screw was found to be satisfactory for use in disposal area survey and reconnaissance activities but was not found suitable for extended disposal area dewatering. In the Dutch procedure, the Amphirol makes trenches on about 8 ft center-to-center spacings across the entire disposal area, though, if the crust is greater than about 2 in., shallow depressions are produced in the surface rather than a complete drainage trench cut through the crust, as constructed by the RUC.

86. Numerous problems will exist whenever RUC trenches across the disposal area intersect the perimeter RUC trench, as the vehicle will

Figure 22. The Marsh Screw, a smaller archimedean screw amphibious craft



a. Overall view of plow and stabilizing outriggers



b. View of plow point and moldboard

Figure 25. Drag plow used in Thiokol trenching operations

Figure 26. Trackmaster and drag plow trenching operation in progress

Figure 27. Thiokol drag plow produced trenches extending from the dragline perimeter trench into the disposal site

Photo 1. Gravel cover at head of Cottonwood Bar, mile 470, 26 September 1975

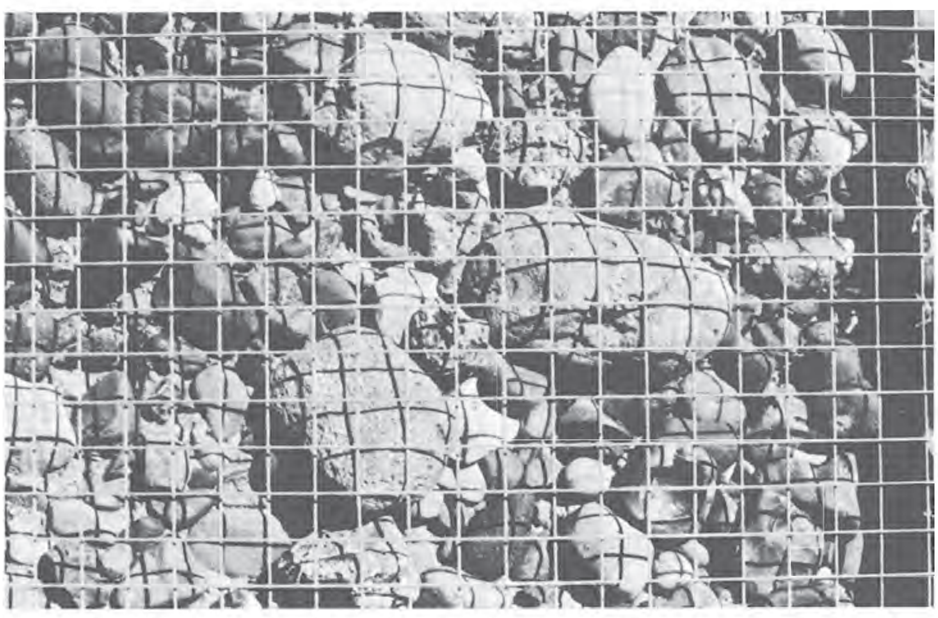


Photo 2. Cobbles on Cottonwood Bar, mile 470, 26 September 1975. Grid divisions are 2 cm

Photo 7. Gravel cover at head of Middle Ground Island, mile 409, 7 August 1974. Trench cut to expose underlying sand. Six-in. rule for scale

Figure 11. Surface flow patterns; spur dike angle 60 deg

Figure 12. Surface flow patterns; spur dike angle 75 deg

Figure 13. Surface flow patterns; spur dike angle 105 deg

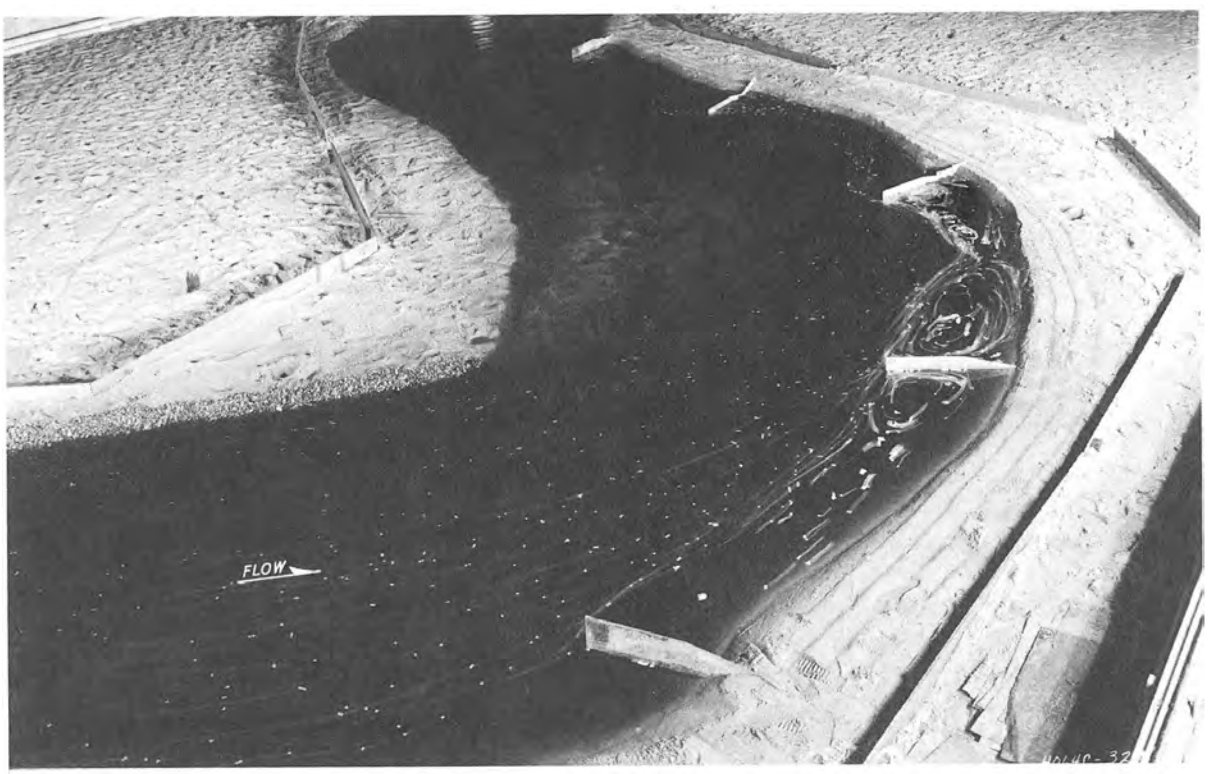


Figure 14. Surface flow patterns; spur dike angle 120 deg

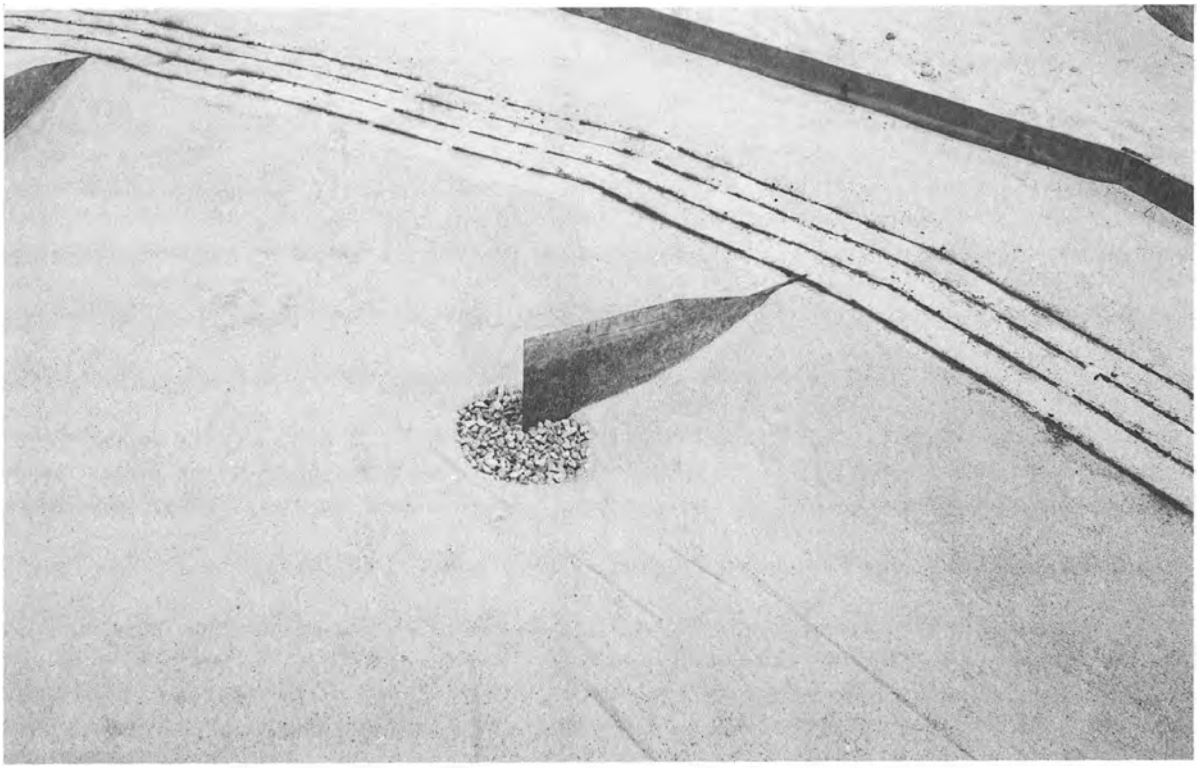


Figure 18. Initial placement of stone apron

Figure 19. Final conditions for stone apron after 24 hr

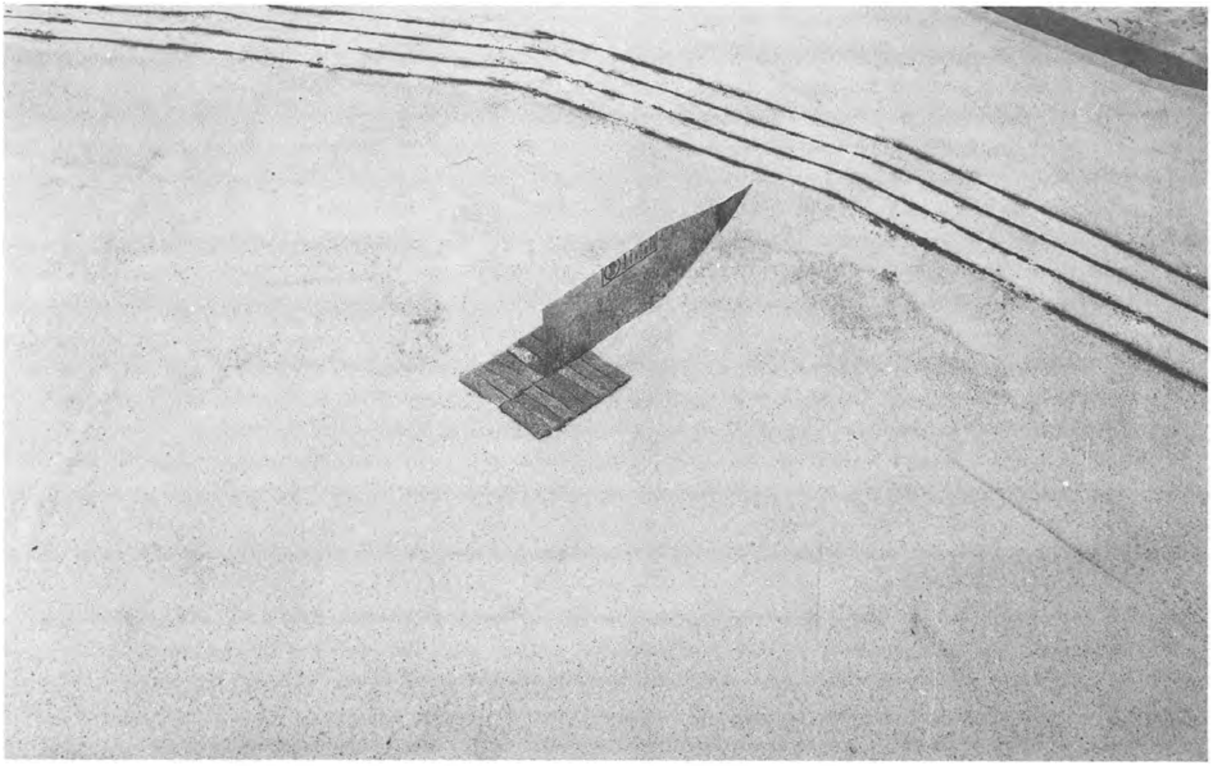


Figure 20. Initial placement of gabion apron

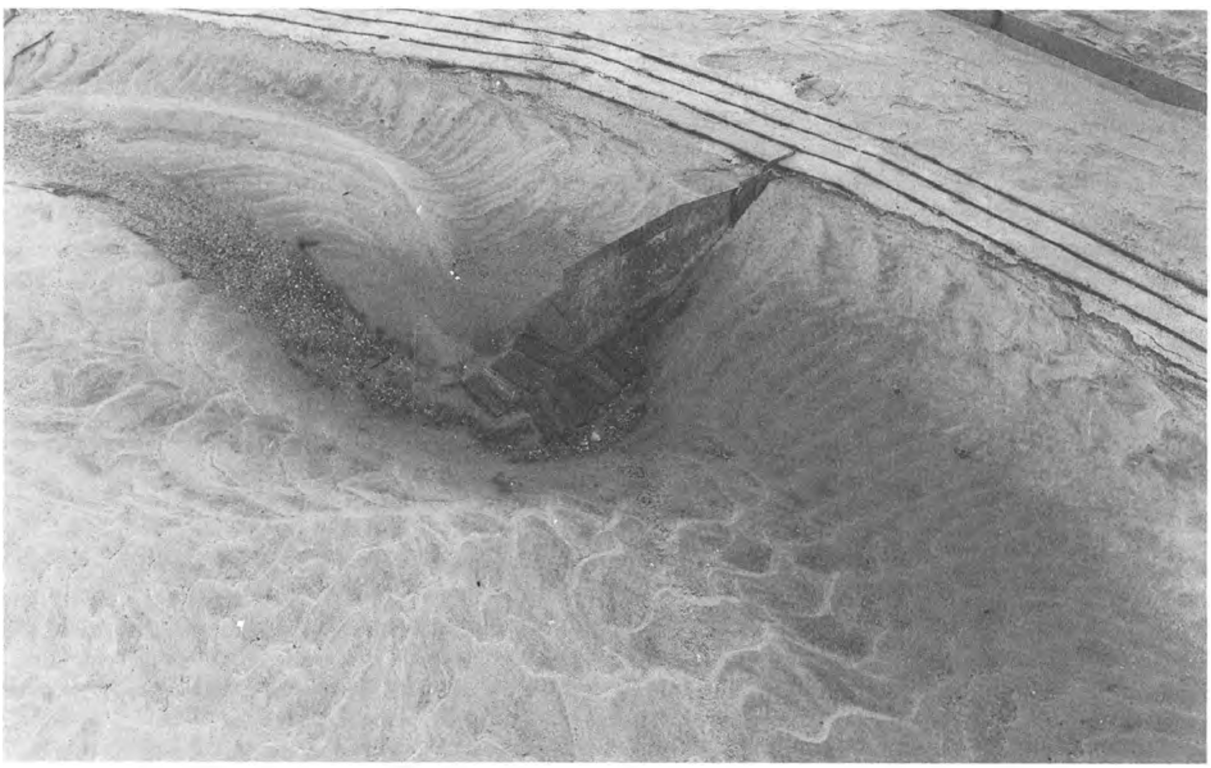


Figure 21. Final conditions for gabion apron after 24 hr

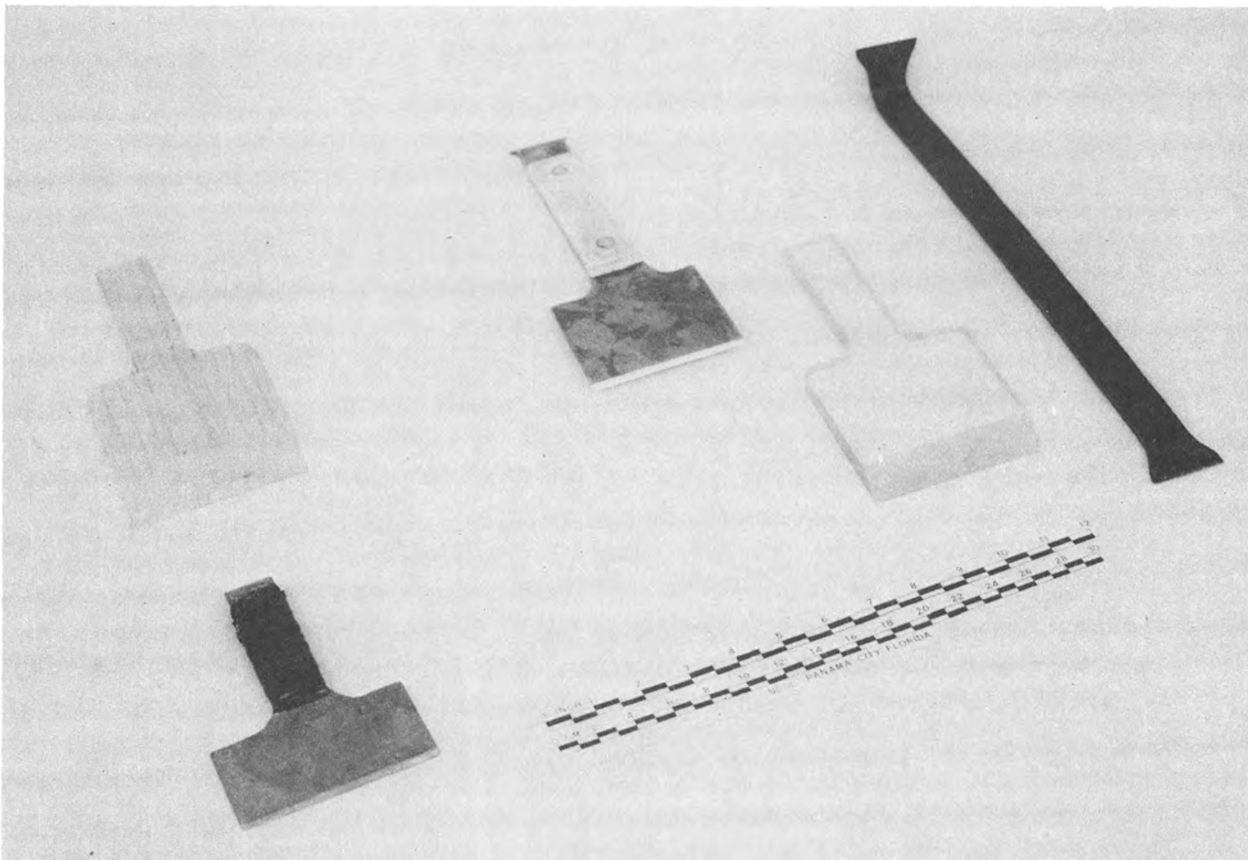


Figure 1. Hand held scrapers.

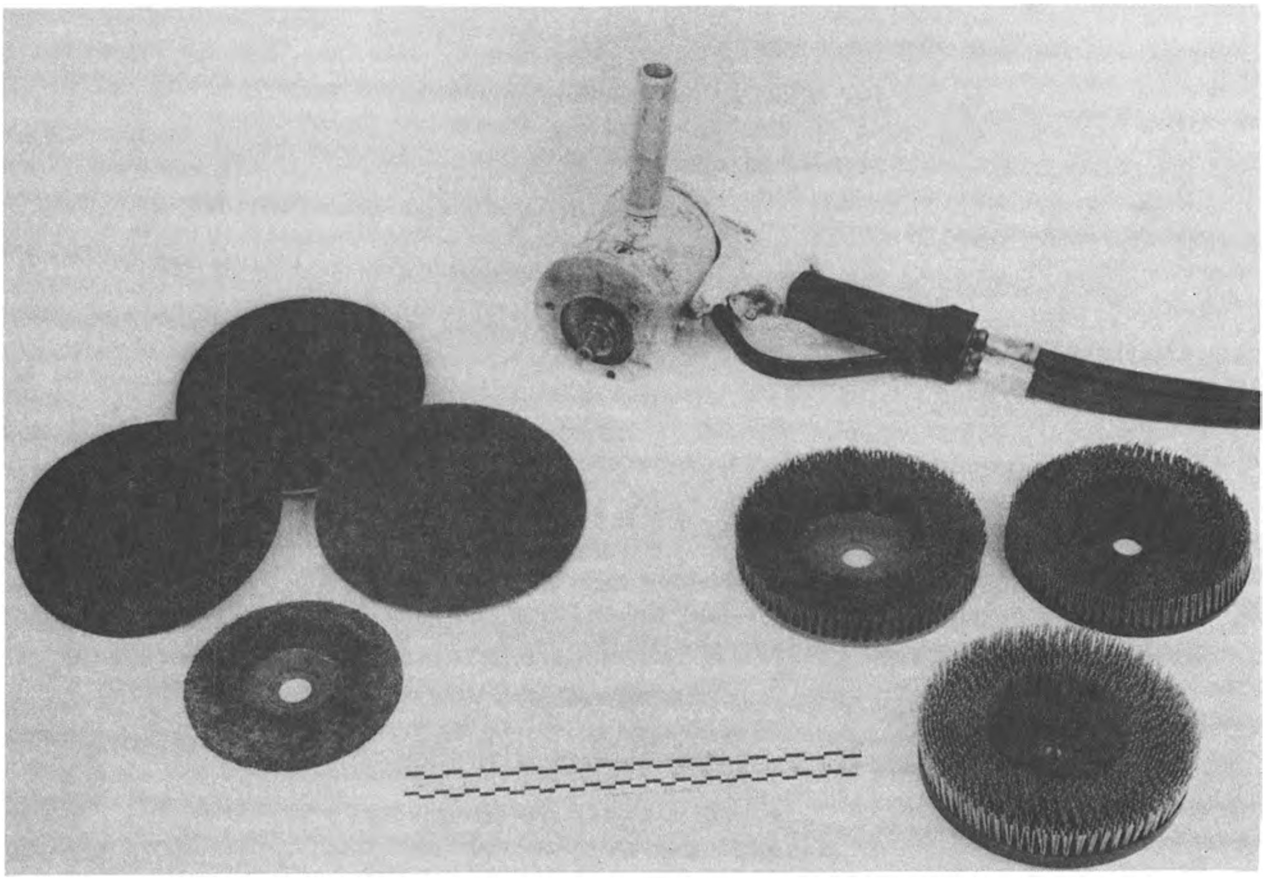


Figure 3. Brushes and abrasive discs.

Figure 4. The Whirl Away Rotary cleaning tool.

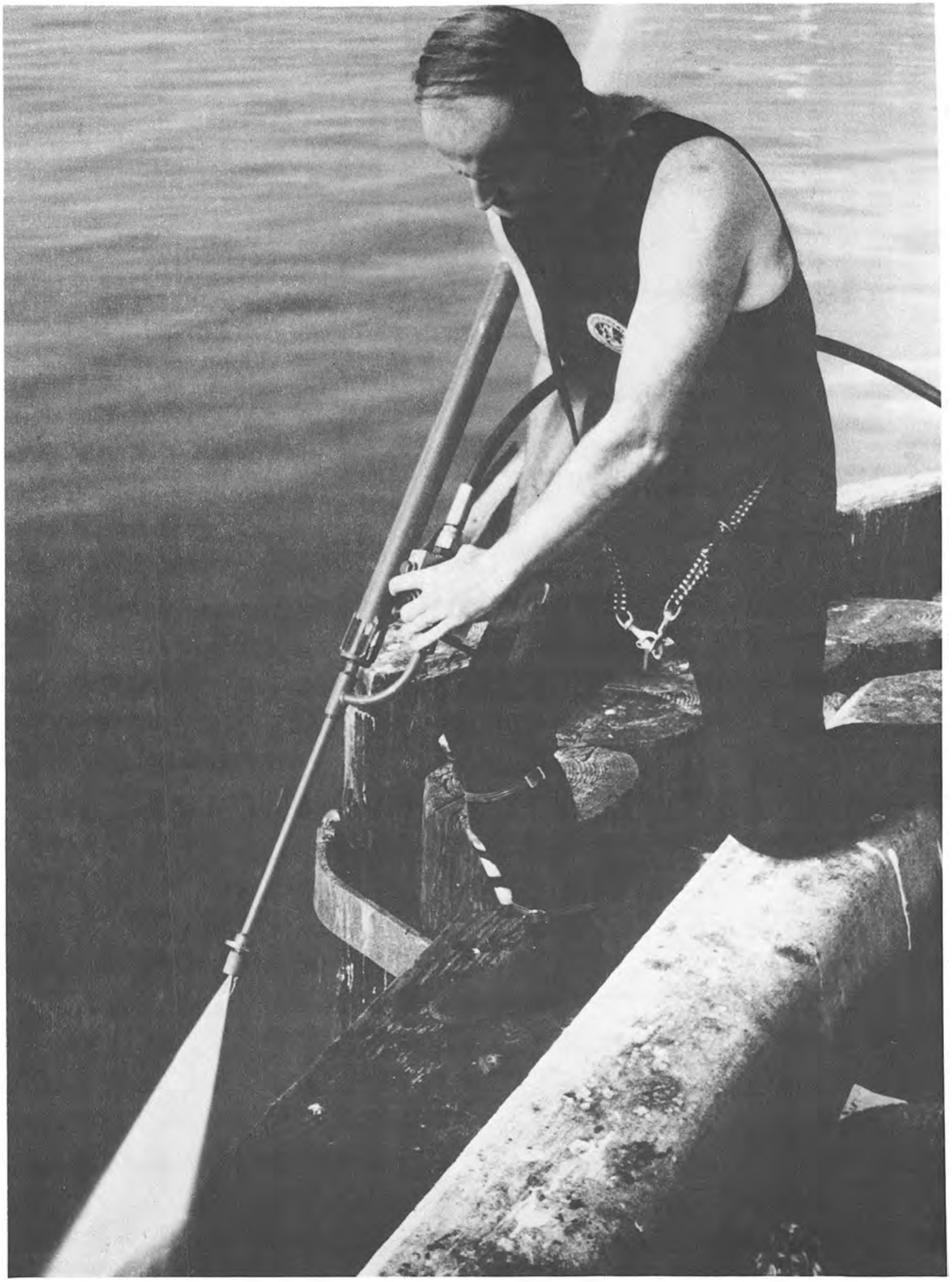


Figure 8. Jetin high pressure, high flow waterjet tool.

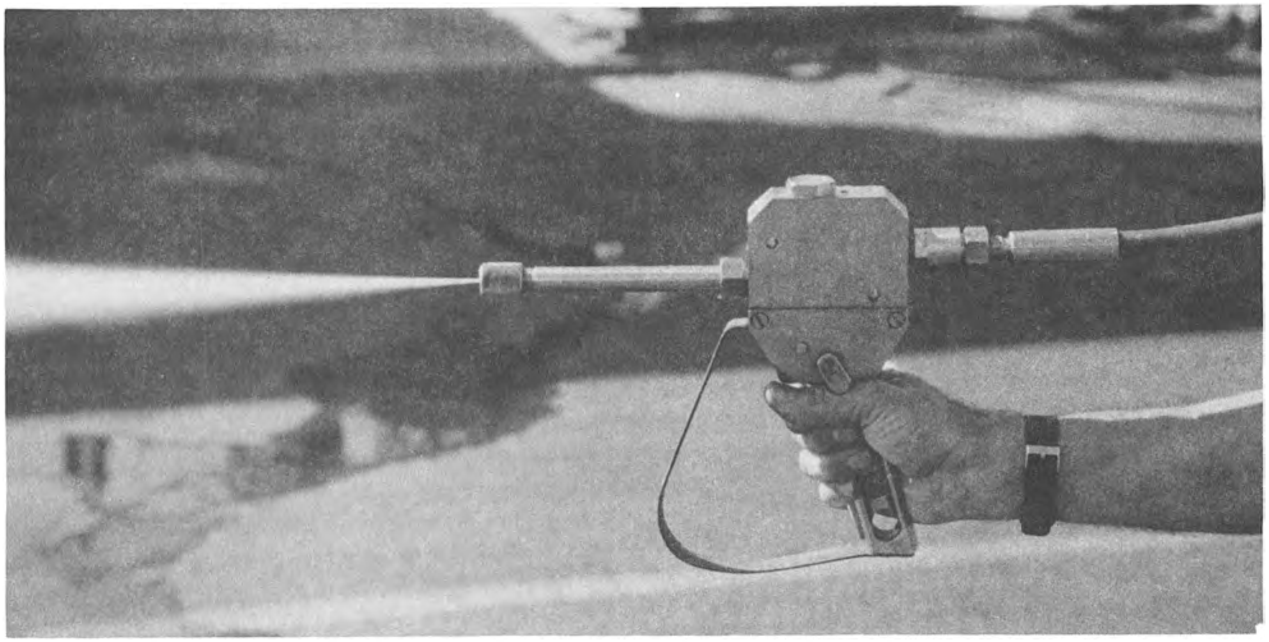


Figure 10a. Flow Industries high pressure, low flow cleaning tool.

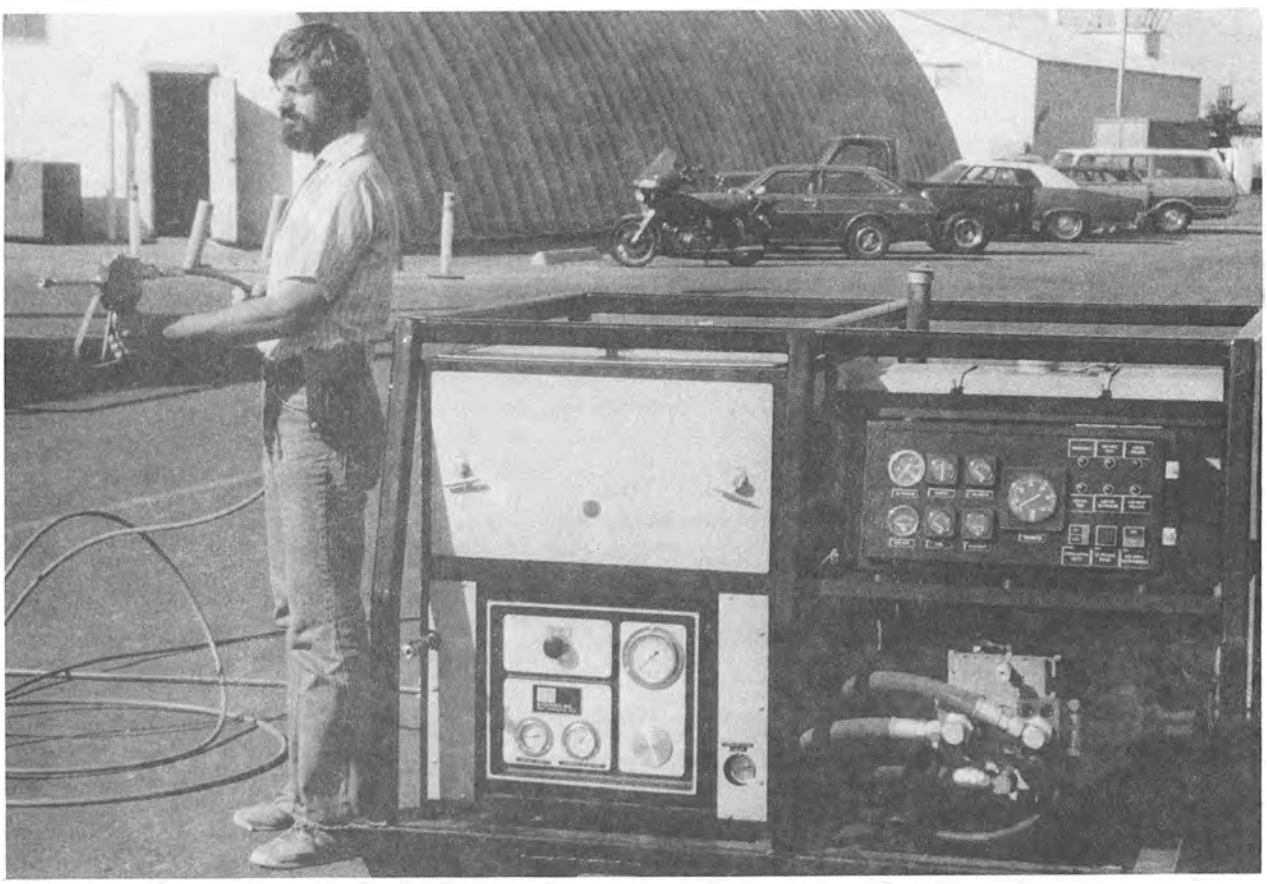


Figure 10b. Flow Industries waterjet cleaning system.

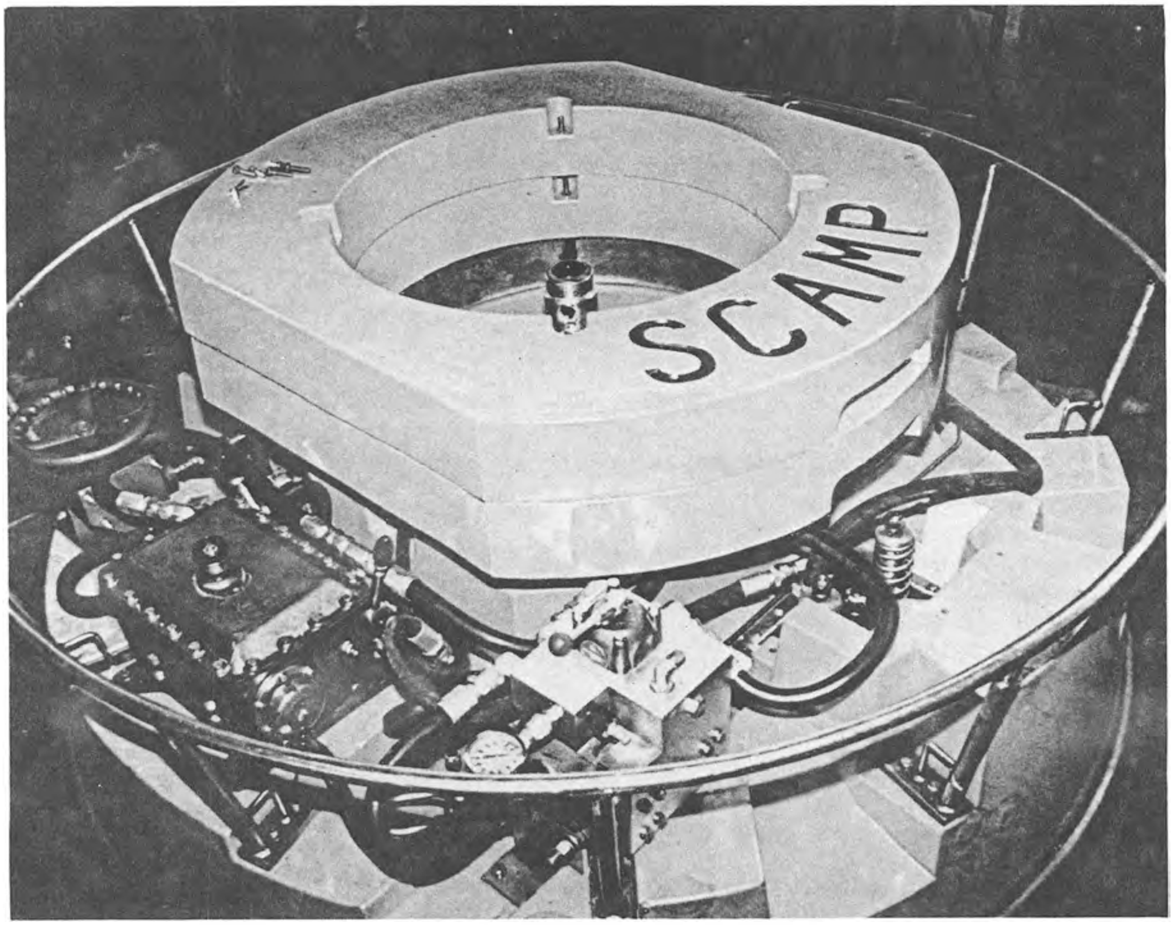


Figure 13. SCAMP self-propelled cleaning vehicle.

rate is about 450 square feet per minute. An impeller in a central duct secures the vehicle to the work surface with a thrust of 1,000 pounds. Power is supplied by a surface generator to a 15 horsepower submersible electric motor that drives a duplex hydraulic pump. One of the pump units powers the wheels and the cleaning brushes, while the second unit drives the impeller. The vehicle, connected to a surface control console with a coaxial cable, can be operated by remote control or directly steered by a scuba diver. The

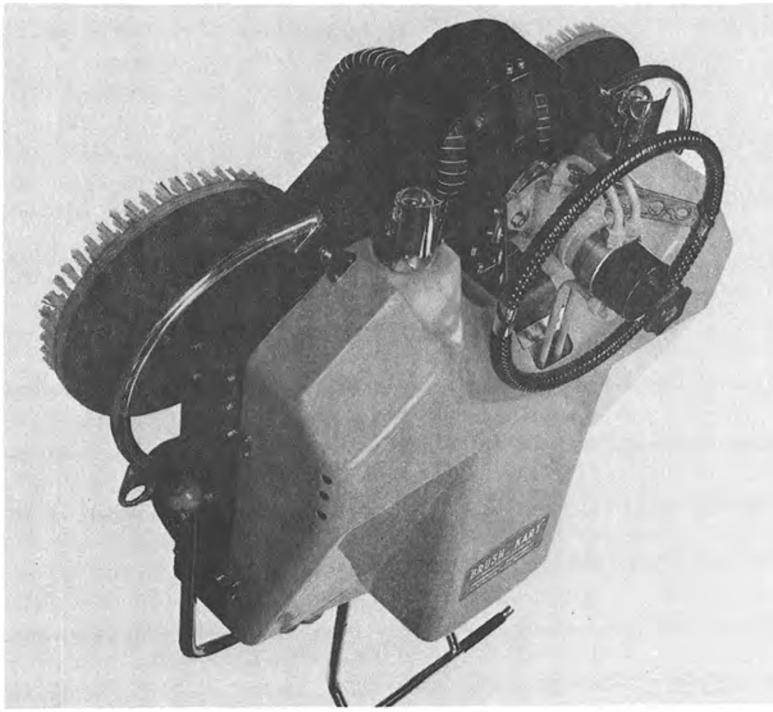


Figure 14a. Brush-Kart self-propelled vehicle.

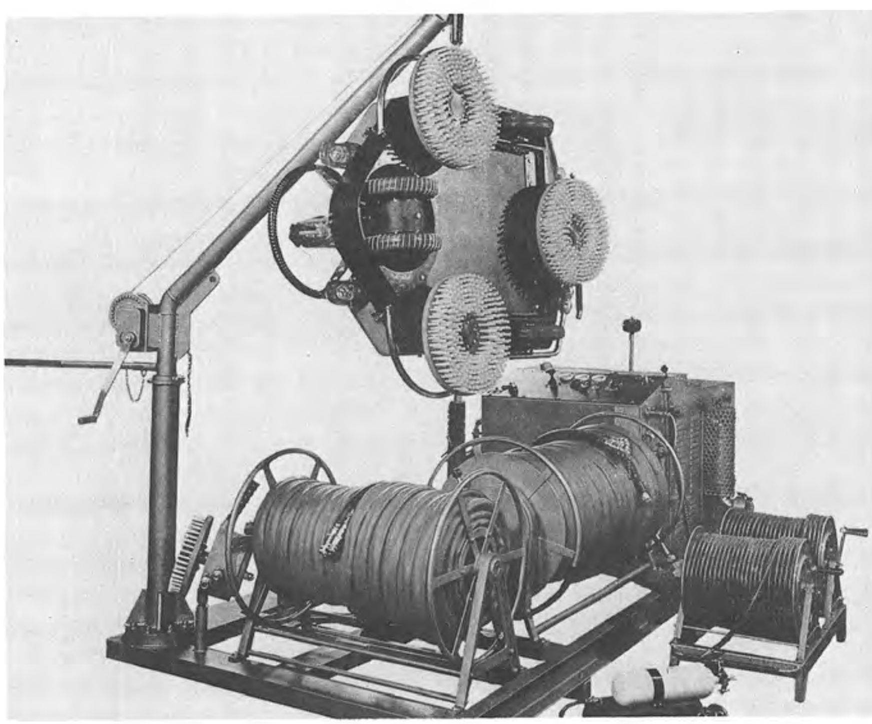


Figure 14b. Brush-Kart cleaning vehicle and deployment system.

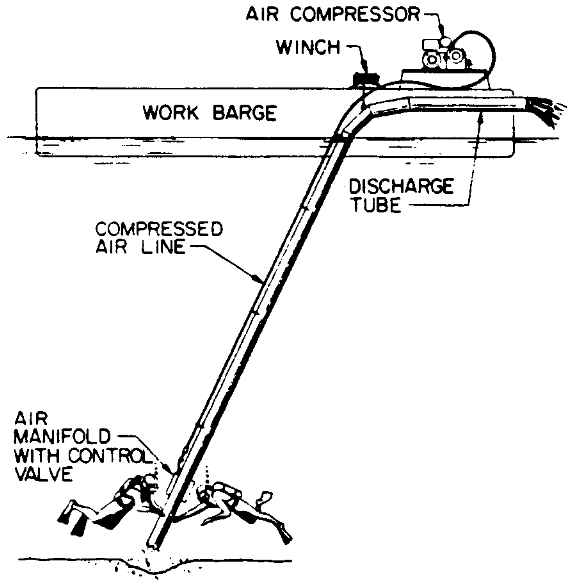


Figure 15. Air lift for removing submerged fouling materials.

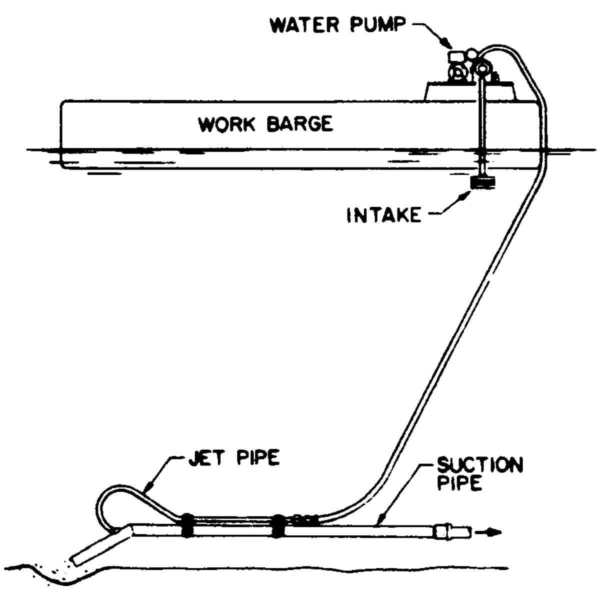


Figure 16. Underwater dredging system.

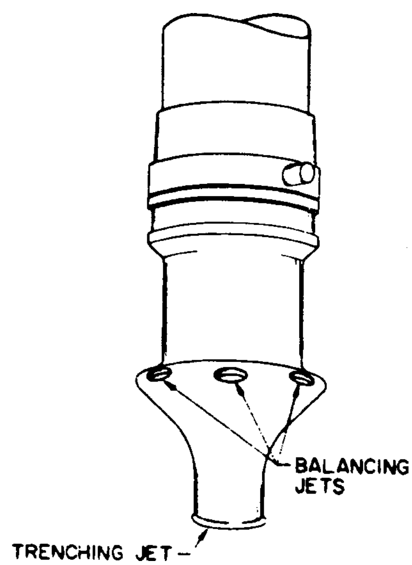


Figure 17. Jetting nozzle with balancing retrojets.

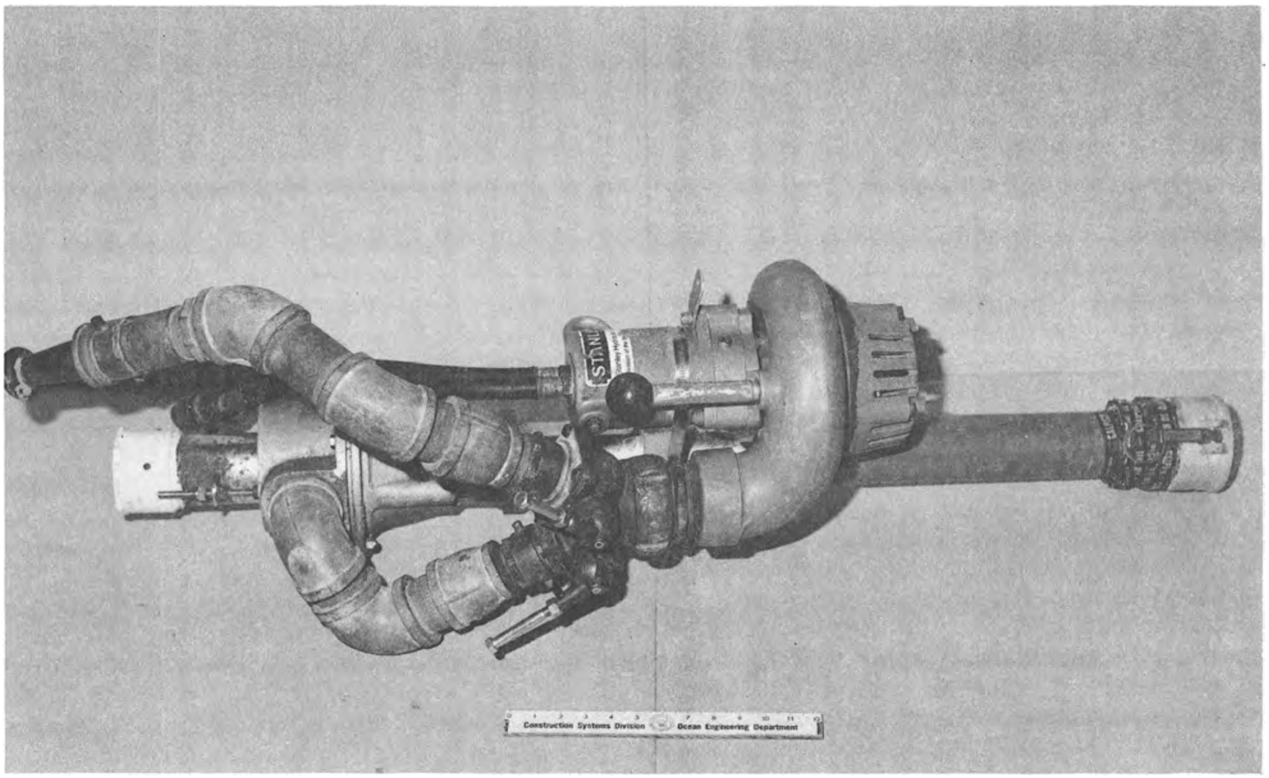


Figure 18. NCEL sediment excavation jet-dredge tool.

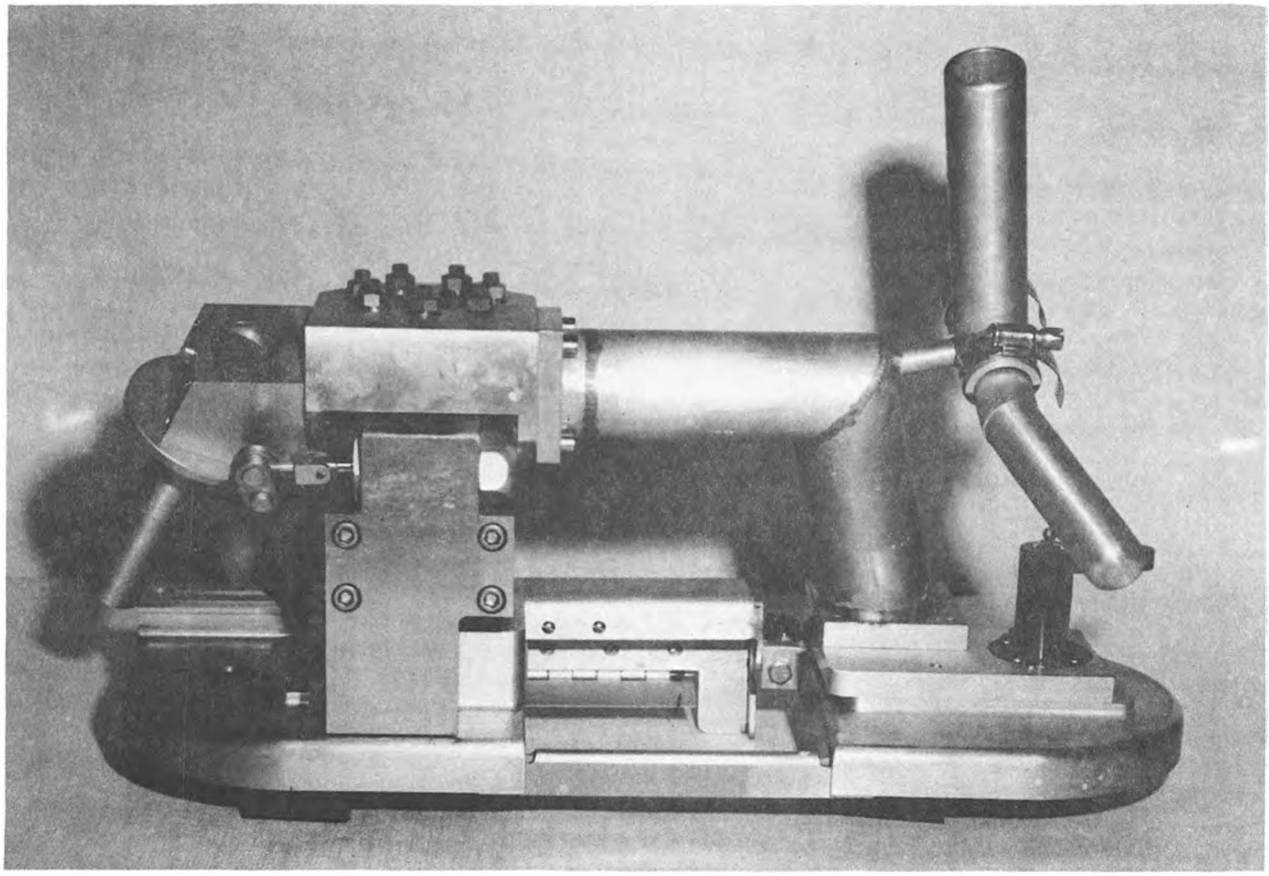


Figure 19. NCEL portable hydraulic bandsaw.

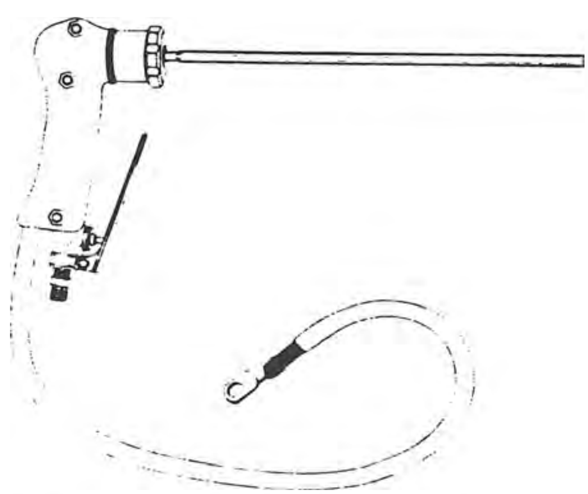


Figure 20. Ultrathermic cutting torch.

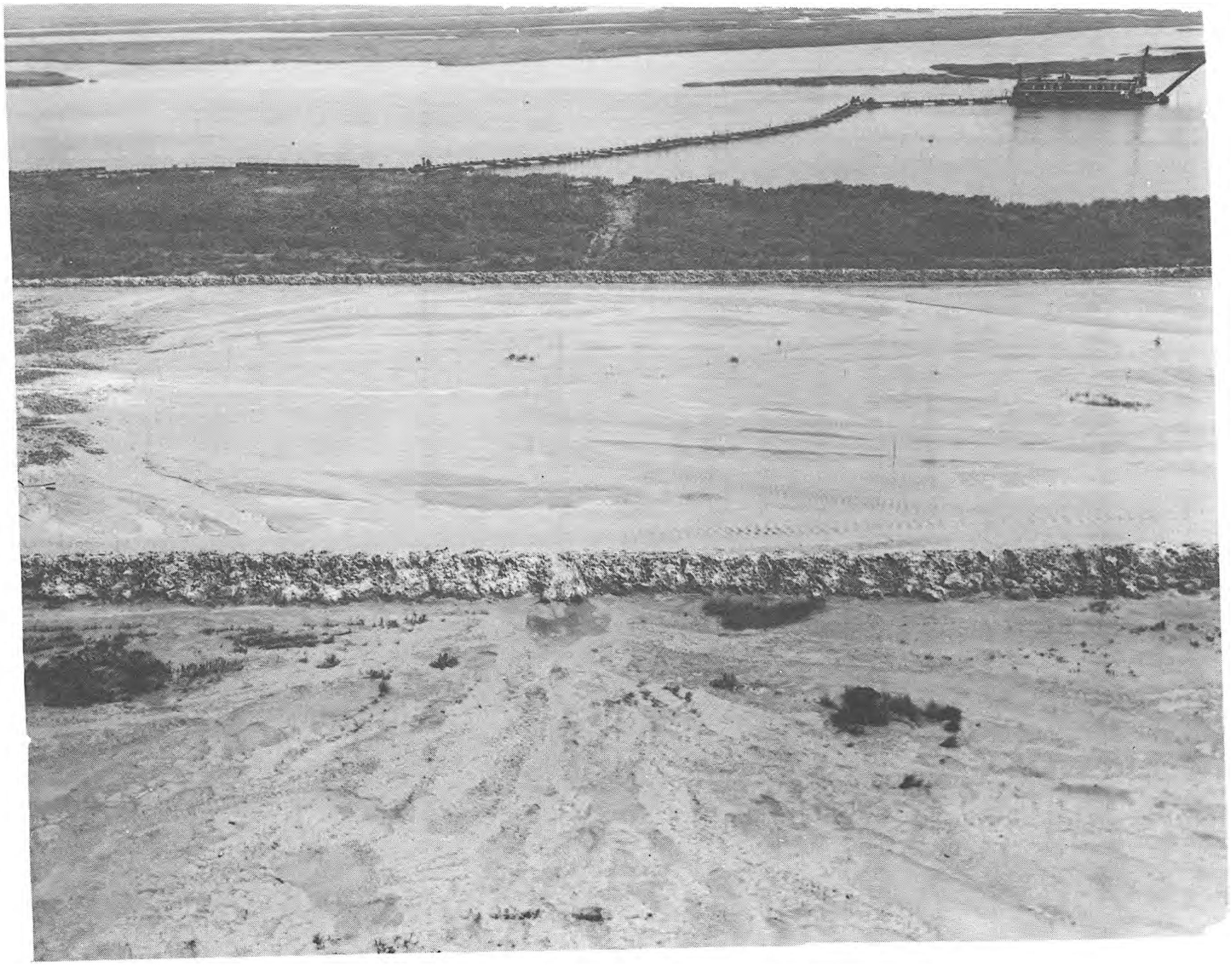


Figure 3

Typical Dredged Material Disposal Operation in Louisiana
(courtesy of the New Orleans CE District)

Figure 20

Detail of Surface Distortion Habitat Showing a
High Concentration of Mosquito Larvae

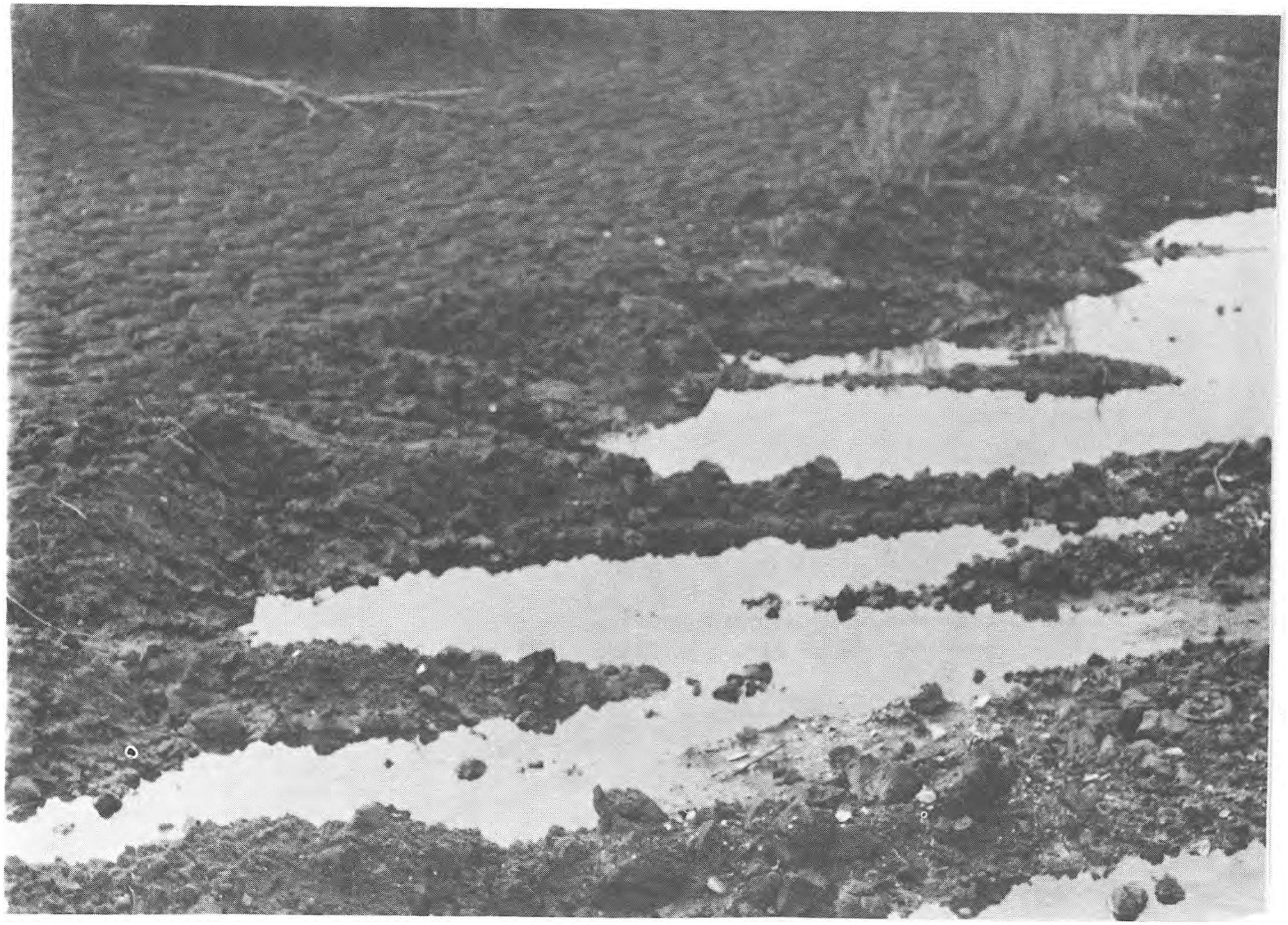


Figure 36

Detail of Poor Ditch Construction (note that water is separated in compartments and does not circulate)

Figure E6

Halophyte Successional Stage within a Dredged Material Disposal Site with Dense Stands of *Salicornia bigloveii* (foreground) and *Borrichia frutescens* (background)

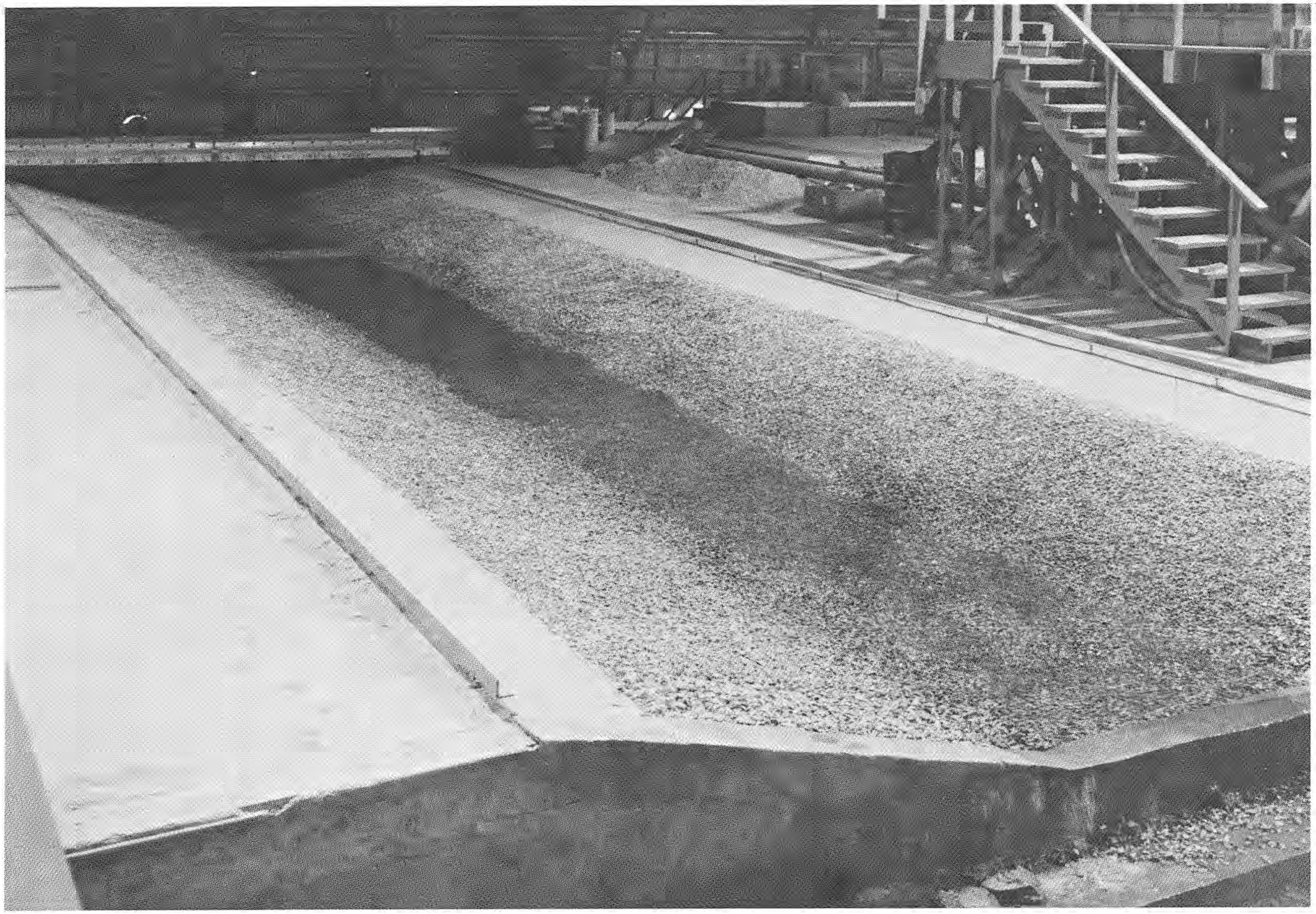


PLATE 2-1
Model Test Facility, Dry Bed

PLATE 2-2
Model Test Facility, Wet Bed

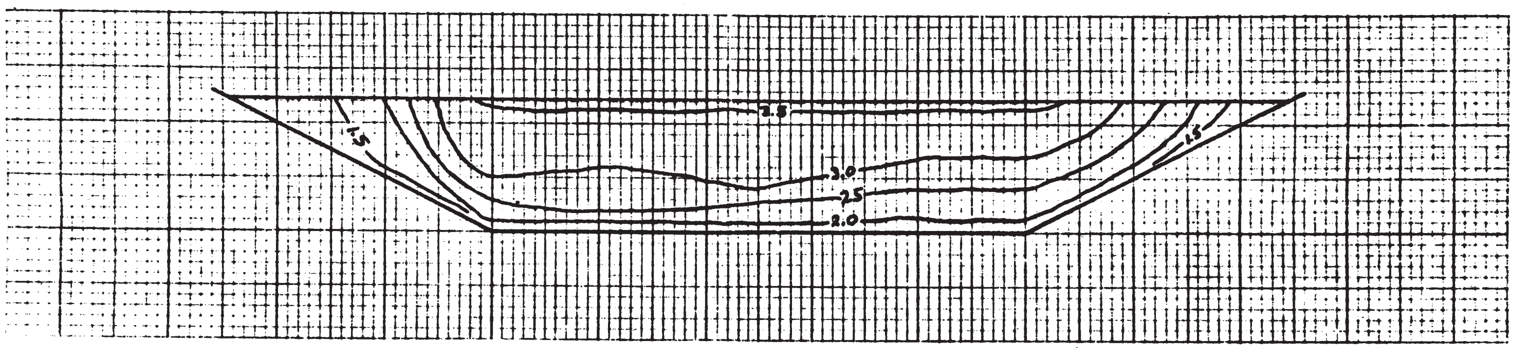


FIGURE 2-5
Velocity Profile - Q = 25 cfs, Depth = 1.22 ft, 1V:2HSS, D_{50} = 0.026 ft

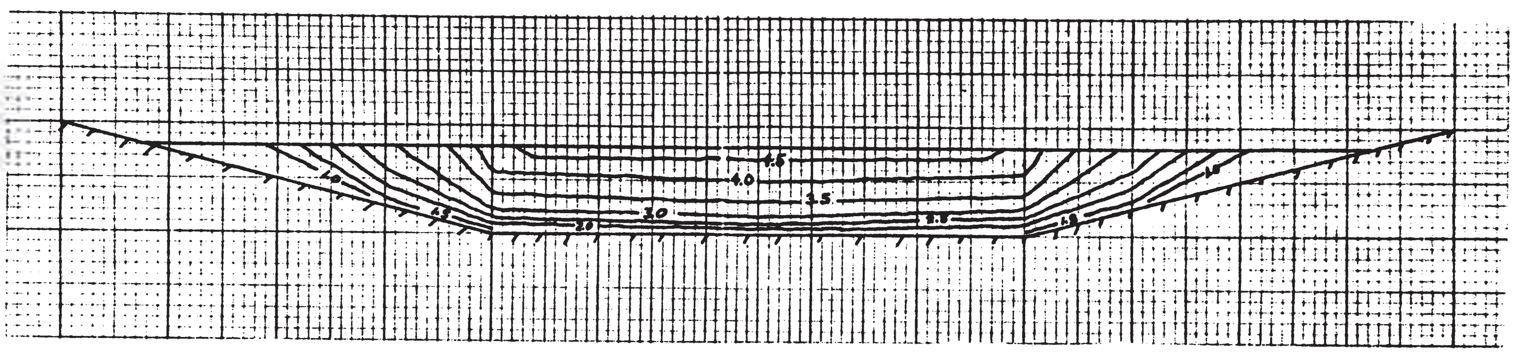
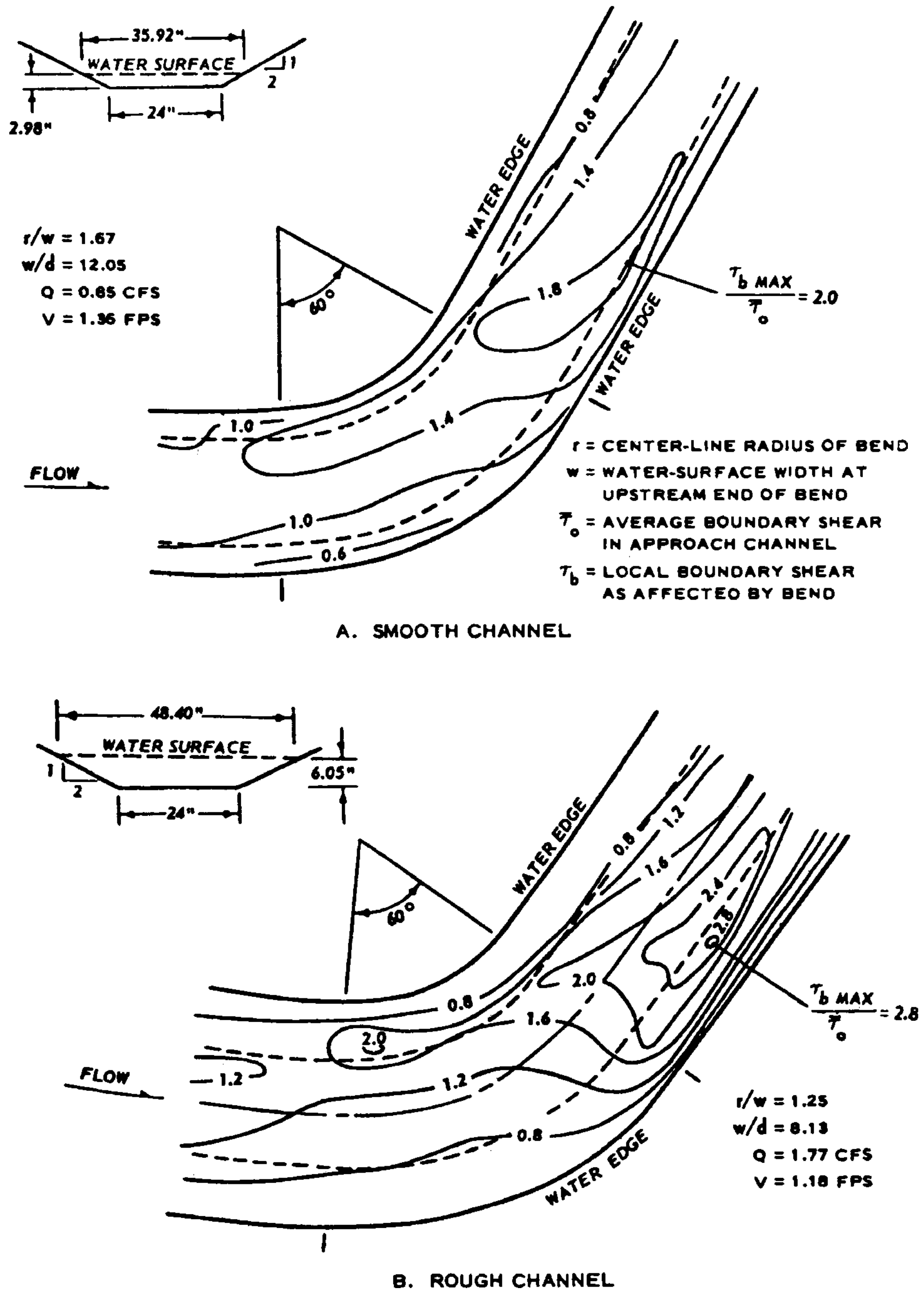


FIGURE 2-6
Velocity Profile - Q = 20 cfs, Depth = 0.81 ft, 1V:4HSS, D_{50} = 0.037 ft



NOTE: FIGURES REPRODUCED FROM REF 53.

FIGURE 5-1
Shear Distribution in Channel Bends (from EM 1110-2-1601 (4))

Table A2. Ice thickness measurements and observations of surface conditions on lakes, rivers and fast sea ice locations in Alaska.

Date	Ice thickness (cm)	Snow depth (cm)
1971 (cont'd)		
Apr 17	104	84
24	104	76
May 1	102	74
8	30 cm water overflow covering the ice.	
15	Water up 90 cm.	
16	Ice moves 90 m downstream.	
17	Ice running from bank to bank.	
21	River free of ice.	
1972		
Apr 15	91	46
22	91	43
29	91	38
May 6	91	43
6	30 cm water overflow on ice. Maximum ice thickness observed from 25 Mar to 6 May.	
7	60 cm water overflow on ice.	
8	No further information available.	
1973		
Apr 7	81	61
14	81	58
21	81	56
28	81	51
28	10 cm water overflow on ice.	
May 5	81	25
5	20 cm water overflow on ice.	
15	River free of ice.	
1974		
Apr 13	91	46
20	91	41
27	91	25
May 18	River free of ice.	

BARROW, ALASKA

Measurements made on Imikpuk Lake at approximately 100 to 150 m off the west shore toward the center of the lake.

Date	Ice thickness (cm)	Snow depth (cm)
1961		
Missing		
1962		
Apr 21	182	10
28	178	10
May 5	170	10
12	161	9
19	150	9
26	140	9
Jun 2	127	8
2	About 3 cm of water on the lake ice. No further measurements made.	
1963		
Missing		
1964		
Missing		
1965		
Incomplete		
1966		
Apr 18	154	36
19	No further information available.	
1967		
Apr 29	156	36
29	Numerous cracks.	
30	No further information available.	
1968		
Incomplete		
1969		
Incomplete		
1970		
Mar 21	Few cracks 1 to 2 cm wide visible in the occasional snow-free areas.	
28	191	15
28	Snow cover hard-packed, medium sized crystals in bottom layer.	
29	No further information available.	
1971		
May 22	185	28
23	185	28
23	Few visible cracks in bare areas.	
29	175	20
29	1-cm slush ice on surface of lake.	
30	No further information available.	
1972		
Apr 8	188	13
15	184	17
22	181	18
29	184	13
May 6	181	23
13	180	22
20	188	14
20	Cracks covered with 13 cm hard-packed snow during May.	
21	No further information available.	
1973		
Apr 28	193	17
May 12	193	13
19	193	13
26	198	13
26	Some above-freezing air temperatures, but no water observed on ice. Snow cover settling.	
26	Depth of water at measurement site 2.8 m, maximum depth of lake approx. 3 m.	
Jul 12	North Salt Lagoon free of ice.	
21	Middle Salt Lagoon free of ice.	
26	Imikpuk Lake free of ice.	
28	Sea ice along north shore in Chukchi Sea moved out, allowing ships to pass eastward from Pt. Barrow.	
1974		
Incomplete		

BARTER ISLAND, ALASKA

Measurements made on fresh water lakes located near the station at approximately 20 to 100 m offshore.

Date	Ice thickness (cm)	Snow depth (cm)
1961		
Missing		

Date	Ice thickness (cm)	Snow depth (cm)
1962		
	Note: Measurements during 1962 *only* were made on sea ice on Beaufort Sea approx. 1 km offshore north of the station.	
Apr 28	149	23
28	Few leads and some open sea water reported by aircraft 3 to 8 km from shore NW to NE of station. There is approx. 30 to 45 cm of water below the ice in the freshwater lake.	
May 5	145	18
12	141	18
19	135	5
26	126	Trace
26	Large area of open sea water reported approx. 8 km offshore to the NW through NE of the station. Beyond this point there are intermittent leads and open water for a considerable distance out. There is approx. 60 cm of water under the ice in the freshwater lake.	
27	No further information available.	
1963		
Incomplete		
1964		
Missing		
1965		
Incomplete		
1966		
Apr 30	213	3
May 7	213	5
7	Maximum ice thickness observed 16 Apr to 7 May.	
16	211	3
21	211	10
28	201	5
Jun 4	185	3
11	160	3
18	124	
25	99	
30	74	
1967		
Mar 25	154	22
Apr 3	156	8
8	152	8
15	140	6
22	132	5
29	127	4
May 5	124	5
12	122	1
19	114	1
26	107	
Jun 2	102	Trace
9	95	Trace
16	86	Trace
18	Open area 50×65 m in size in center of lake. Rapid thawing observed.	
23	61	
23	Ice surface soft, numerous puddles of water.	
30	37	
30	Breakup, ice considered unsafe for further measurements. Open leads and floating ice chunks.	
Aug 15	Camden Bay ice-free.	
18	Beaufort Sea ice-free.	
19	Simpson Cove ice-free.	
1968		
Mar 30	178	8

Date	Ice thickness (cm)	Snow depth (cm)
1968 (cont'd)		
Mar 30	Numerous cracks in ice, some quite deep. No leads or openings. Several large patches of snow-free ice on lake, but some drifts are 61 cm high. Ice thickness at water hole site is 124 cm, but that area is kept open artificially.	
Apr 6	178	8
13	178	15
20	175	8
20	Top layer of snow rather brittle.	
27	175	8
May 5	170	8
11	165	6
19	157	4
26	145	5
30	124	4
Jun 8	107	10
15	85	Trace
15	Lead extends across lake, 180 m long and 10 cm wide.	
22	55	
29	33	
29	Part of lake covered only with pancake ice. Ice cover in some areas soft. Ice measurements terminated due to rapid melting. Ice near shore still fast in numerous spots.	
Jul 30	Beaufort Sea and Camden Bay free of ice.	
1969		
Apr 5	198	13
12	196	14
19	194	19
26	191	13
26	Large crack extends NE to SW and numerous small cracks branch outward.	
May 3	185	9
10	182	8
17	174	5
24	170	4
31	165	3
31	Crack extends NE to SW, length undetermined due to snow cover. Ice appears to be decaying.	
Jun 7	132	
7	Pools of water starting to appear.	
14	98	
21	56	
21	Large leads developed near shore.	
28	13	
28	Entire ice pack merely floating on lake with about 18 m of open water between shore and ice. Ice observations terminated.	
1970		
Apr 11	198	
11	1/3 of ice surface clear of snow.	
18	196	
18	1/2 of ice surface clear of snow.	
25	198	1
May 2	192	3
9	196	
16	194	
16	40-knot winds removing snow from ice cover, except for an occasional drift.	
23	Ice soft and spongy at 46 cm and water encountered at 51 cm depth.	
30	Surface candled, numerous cracks. Ice spongy from top to bottom. Melt ponds along edge of ice.	

Date	Ice thickness (cm)	Snow depth (cm)
1970 (cont'd)		
Jun 6	185	
6	10 cm of candled ice on surface.	
13	175	
13	Bottom half of ice cover very soft. Area of open water observed around the edge of ice.	
20	165	
20	2 to 8 m of open water surrounds floating lake ice.	
27	147	
27	10 to 25 m of open water surrounds floating lake ice. 36 cm of candled ice on top and bottom ice slushy. Numerous melt ponds.	
1971		
Apr 3	204	10
3	Ice solid, no soft areas.	
10	206	10
10	Approx. 28 cm of water under ice at this part of the lake.	
17	208	10
24	203	10
24	Bottom of ice cover becoming soft.	
May 1	Ice thickness measurement, made in an area which has remained mostly clear of snow, 216 cm.	
8	198	8
8	Ice becoming soft 122 cm from the top and slushy 183 cm from the top.	
15	Ice thickness of 185 cm on this date does not appear representative because some ice on top was candled and the lower 122 cm was waterlogged.	
22	198	
29	193	
29	Surface candled.	
Jun 5	180	
5	Top 46 cm candled ice, remainder of ice rotten.	
12	173	Trace
12	3 to 6 m of open water around edge of lake.	
19	150	
19	15 to 30 m of open water around edge of lake. Melt ponds on ice with sections becoming quite rotten.	
26	Ice observations terminated, open water surrounding ice sheet. Surface ice candled with many cracks during June.	
1972		
Apr 22	203	
22	Bottom of lake ice becoming porous.	
28	211	
28	Ice cover solid on this date.	
May 5	210	
5	Bottom layer of ice becoming soft again.	
12	208	
12	Lowest 90 cm of ice sheet soft. Surface ice candling.	
19	208	8
19	Top 60 cm ice is hard, mid 60 cm soft and lowest portion of ice wet.	
26	210	10
Jun 16	160	
16	30 cm of candled ice over softer ice. Rapid deterioration of ice with numerous melt ponds on surface.	
24	137	
30	142	
1973		
Apr 24	188	10
May 5	189	15
12	189	10
19	191	10
26	188	8

Date	Ice thickness (cm)	Snow depth (cm)
1973 (cont'd)		
Jun 2	168	0
2	Large puddles of melt water on ice and 5 cm of candled ice on top.	
9	150	0
9	Numerous melt ponds, some 15 to 20 cm deep. Top 10 cm of ice is candled.	
16	142	0
16	Large areas of open water around shore of lake. Ice rotten down to 90 cm. Top 20 cm of ice is candled.	
23	119	3
Jul 16	Lake free of ice.	
1974		
Mar 30	190	0
Apr 6	189	0
13	185	0
20	184	1
27	185	0
Aug 1	Lake free of ice.	

BETHEL, ALASKA

Measurements made on the Kuskokwim River at distances ranging from 25 to 200 m off the north shore.

Date	Ice thickness (cm)	Snow depth (cm)
1962		
Mar 26	144	5
Apr 2	138	10
9	132	0
9	Top 38 cm of ice extensively honeycombed.	
16	138	0
16	Cold weather has solidified surface, though top 38 cm still porous.	
23	137	0
30	128	0
30	Ice extensively honeycombed.	
May 7	102	0
7	Top 30 to 46 cm extensively honeycombed and composed of alternate ice and water layers.	
15	93	0
15	Top 25 cm porous. Observer waded to hip boot height to get on ice. Last ice measurement.	
25	Surface slushy, shore ice.	
26	Surface jammed ice, numerous cracks.	
29	River channel free of ice. Only ice on bank and shore remain.	
31	River free of ice.	
1963		
Apr 21	107	11
27	102	10
27	Snow cover slushy except for drifted areas.	
May 5	100	
5	River ice soft and slushy. River has risen approx. 60 cm.	
6	South side of river ice under water for approx. 45 m from shore, due to a strong south wind.	
12	91	
12	Water overflow covering both banks and about 30% of river.	
13	River ice is honeycombing and crystallizing. River level up approx. 1.2 m.	
16	River ice has moved approx. 45 m from shore. Main ice is still solid.	

Date	Ice thickness (cm)	Snow depth (cm)
1963 (cont'd)		
May 18	Entire river ice is moving. Ice appears very honeycombed and crystallized.	
21	Main river ice has broken up.	
22	River is clear of ice, water level higher than normal.	
26	River free of ice.	
1964		
Apr 19	132	23
19	Snow over the ice consists of 5 cm slush-ice, 18 cm ice crystals. Between 0-5 cm of water and slush at bottom. Ice soft and watery.	
May 3	141	8
3	Ice consists of 118 cm solid ice, 10 cm slush-ice and 13 cm shell ice.	
10	135	3
10	Slush and shell ice frozen solid.	
17	140	3
17	Top 46 cm ice soft and contained water.	
24	132	
25	Water beginning to collect over the river ice on the south side.	
31	112	
31	Considerable overflow, 15 m area of open water on the south side of the river. River ice honeycombed and crystallized, no firm ice at test site.	
Jun 8	River free of ice.	
1965		
Mar 14	105	24
21	103	
21	Snow cover has turned into slush and water 15 cm deep.	
27	98	
27	All slush and water refrozen. 30 cm of soft and rotten ice beneath 4 cm of loose ice crystals.	
31	Water starts to flow on ice near the south bank of river.	
Apr 4	97	
4	Cover consists of 3 cm loose ice crystals at top, then 20 cm of firm ice followed by 30 cm soft ice and water, and the rest of ice firm.	
11	94	8
11	Top 30 cm of ice solid but crystallized, the rest soft.	
17	85	
17	Ice soft and starting to rot.	
24	71	
24	Top 20 cm ice crystallized and soft, the rest rotten and crystallized.	
28	South side of river covered with water, and on the north side overflow getting deeper.	
May 8	33	
8	River ice has only 5 cm of firm ice near the top, rest is rotten.	
17	Ice opened up at the bend on the west end of village.	
19	Ice has started moving, river level rising a few meters, shore ice still holding.	
20	River clear of ice except for a few stray cakes due to a jam up in river. North wind has broken the shore ice loose.	
23	Ice flowing heavy most of the day.	
24	A few cakes of ice flowing in the river in morning, by afternoon all the ice is gone.	
1966		
Apr 10	119	13
17	122	
17	Some overflow along shore. Snow cover across river varies from 3 to 30 cm.	
24	121	
1966 (cont'd)		
Apr 26	Slush on top of ice over approx. 1/3 of river.	
27	Approx. all of ice appears to have slush and snow crystals at surface.	
May 1	97	
1	Top 30 cm ice rotten, rest firm. Slush (up to 61 cm) covers most of river.	
4	Water over ice on south side of river.	
5	Approx. 1/5 of river on south side covered by water.	
8	74	
13	Overflow ice breaking up.	
17	River rising, overflow ice moving.	
18	Ice appears rotten and pitted at top.	
22	South side of river open.	
23	Ice moving down channel.	
24	Ice running in river entire day.	
25	Few floating ice cakes observed in river.	
26	River free of ice.	
1967		
Apr 16	117	
16	Cover consists of 8 cm crust ice over 15 cm water and slush, then 10 cm of very soft ice all over the main ice layer.	
23	114	
23	3 to 20 cm loose ice crystals on surface.	
30	91	
30	Few small open holes observed in ice. Overflow observed on south side of river during high tides.	
May 3	Water over ice on south side of river is increasing in depth.	
7	Shore ice is breaking up.	
8	Shore and river ice has started to move.	
9	Most of ice now small chunks and needle-like in appearance. River still solid with ice.	
11	River about 1/3 full of ice and running on south side.	
12	River passed its crest. Few stray ice chunks in river.	
14	All ice gone.	
1968		
Apr 7	108	Trace
14	107	3
14	Some water overflow on south side of river.	
21	103	
21	8 cm slush and 23 cm soft ice on top.	
28	Ice crystals 10 cm in depth observed on top of ice cover. Bottom 66 cm of ice cover soft.	
May 5	86	
6	Tide action has piled ice on south side of river near a sand bar.	
11	Water covering 1/3 of ice on south side; north side has considerable overflow.	
12	Shore ice breaking up and large holes observed.	
14	Ice moved a little and then jammed.	
16	Ice moved rapidly down channels on both sides of river.	
17	Loose ice running near center of river.	
19	Small amount of ice left near banks of river.	
20	Ice entirely gone.	
1969		
Mar 30	110	17
Apr 6	109	17
13	109	8
13	Top 46 cm soft ice. Slush and water 0 to 20 cm deep on river.	
16	Water overflow observed on ice.	
20	104	4
26	River water level rising.	
27	90	3

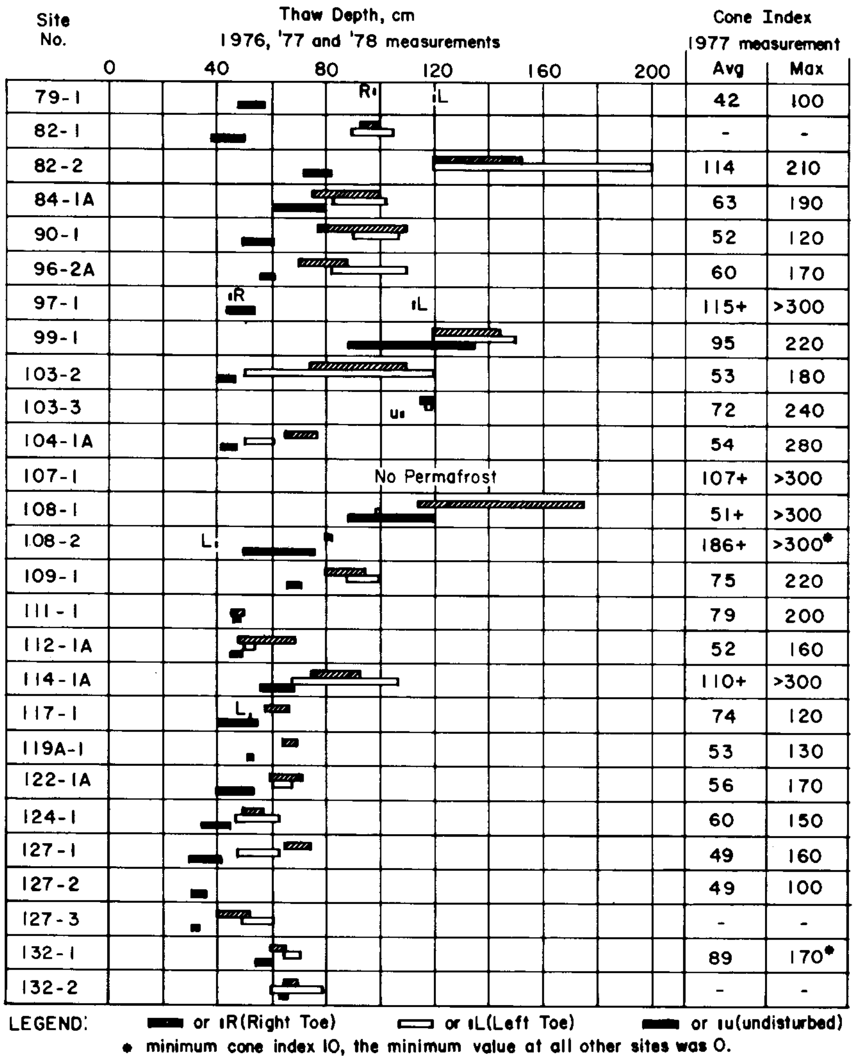


Figure 43. Thaw penetration and cone penetrometer index values of each test site.

areas adjacent to the road show no distinctly decreasing trend when progressing from the south to the north (Fig. 43), but upon close examination of the data, and recalling that Atigun Pass (on Alyeska Alignment Sheet 110) is the imaginary line between the Subarctic to the south and the Arctic to the north, some trends do appear. The first trend we note is a much wider variation in thaw depths at the individual sites in the subarctic region. Site 99-1 exhibited the largest variation in undisturbed thaw depths of the subarctic sites. The thaw depth was 88 cm in 1976 but 135 cm in 1978, a variation of 53%. The greatest variation north of Atigun Pass was at site 117-1 where the thaw depth ranged from 41 cm in 1976 to 55 cm in 1978, a differences of 34%.

The second notable trend is that undisturbed thaw depths from site 111-1 northward average less than those observed at sites 79-1 though 109-1. Another trend is also observed in Table 14, i.e. the average thaw penetration in the undisturbed areas has increased each year. Climatic observations (Table 1) indicate that 1977 was clearly the warmest thaw season south of Atigun Pass and only slightly warmer than the 1976 thaw season north of Atigun Pass. These data also confirm the main reason that thaw penetration depths are greater at the subarctic sites—the air thawing indexes are substantially larger.

These data are substantiated by additional data obtained during the 1977 vegetation transect studies (see *Vegetation* section). Thaw variations at approximate 5-km spacings along the

Figure 55. Upstream ponding caused by shoulder thaw consolidation and bowing of the culvert, site 91-C1, August 1977.

Figure 56. "Maintenance clogging" problem caused by surface material graded down sideslope of road, site 91-C2, August 1977.

Figure 105. Hydroseeder, with bales of wood cellulose fiber on trailer.

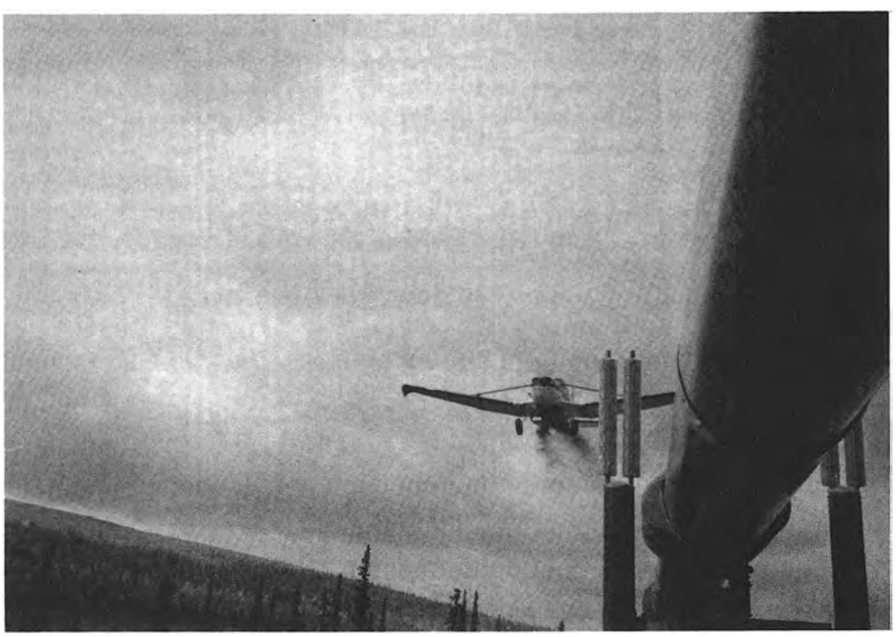


Figure 106. Aerial fertilizing of pipeline workpad in the vicinity of Old Man, September 1976.

Figure 109. Contour harrow used for seedbed preparation.

Figure 110. Harrowed ground in which seeds are buried, resulting in increased germination.

moved from the edge of the site, where organics and mineral soils were spread, to the middle of the site, which had no fine-grained soils on the surface.

Surface preparation. Following the initial year (1975), much more attention was placed upon surface preparation. In many areas special harrowing equipment, such as a contour harrow (consisting of a large wire mesh with heavy wires protruding downward) was towed over the surface both before and after seeding (Fig. 109). Contour harrows were first used north of Toolik during 1976. By 1978 the practice was employed along the entire Haul Road. Harrowing appeared to result in greatly increased germination rates and a subsequent increase in vegetative cover, application. For purposes of the numerical simulation, a rectangular region approximately 62,400 ft long in the alongshore direction and 29,400 ft wide in the offshore direction was considered. It included a portion of Pamlico Sound. The variable grid used for the simulation is shown in Figure 10. The grid was

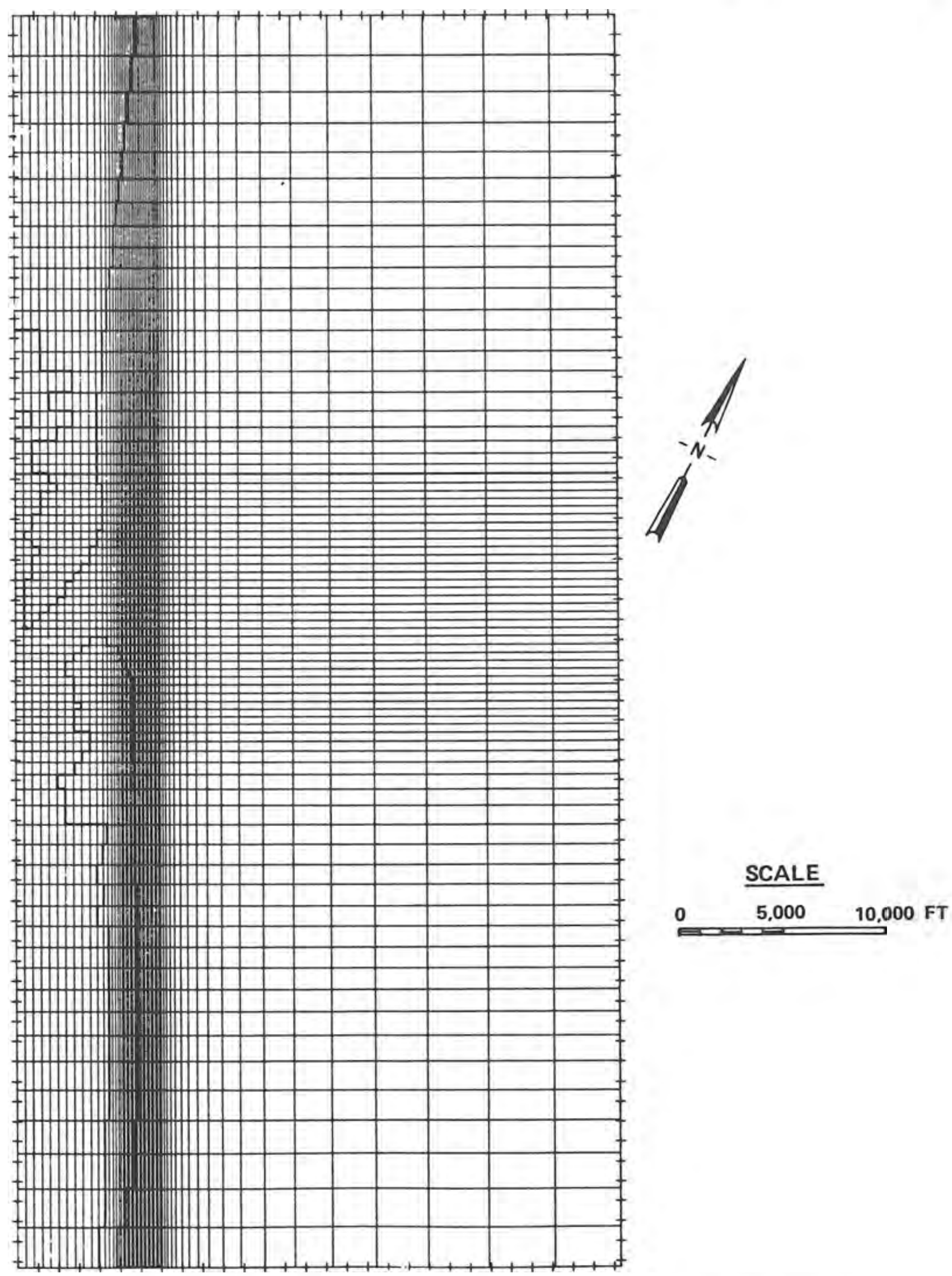


Figure 10. Numerical grid used for Oregon Inlet, North Carolina, simulation

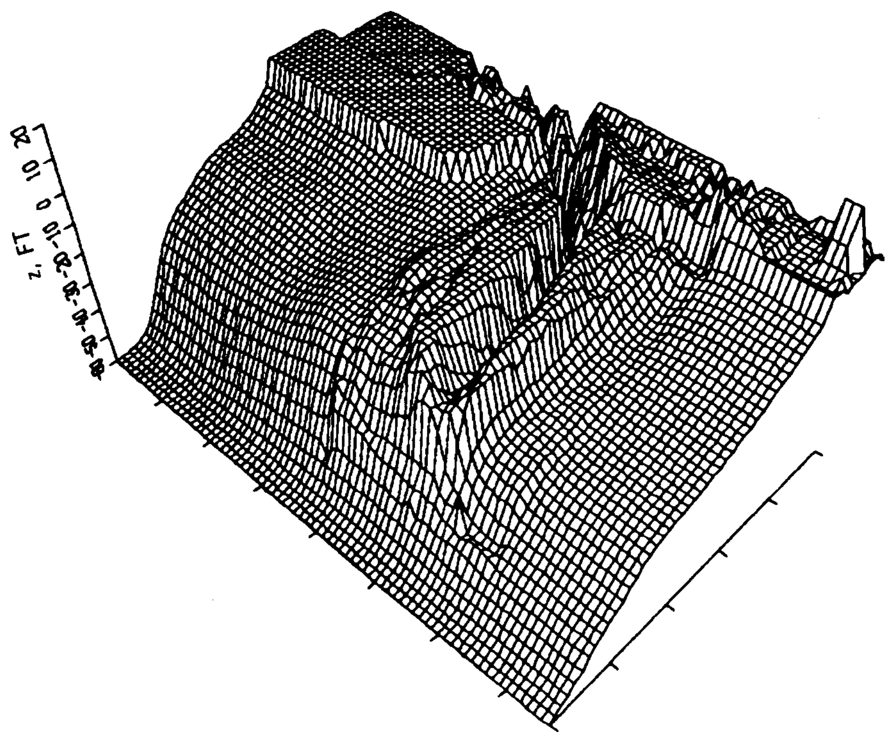


Figure 11. Topography used for Oregon Inlet, North Carolina, simulation

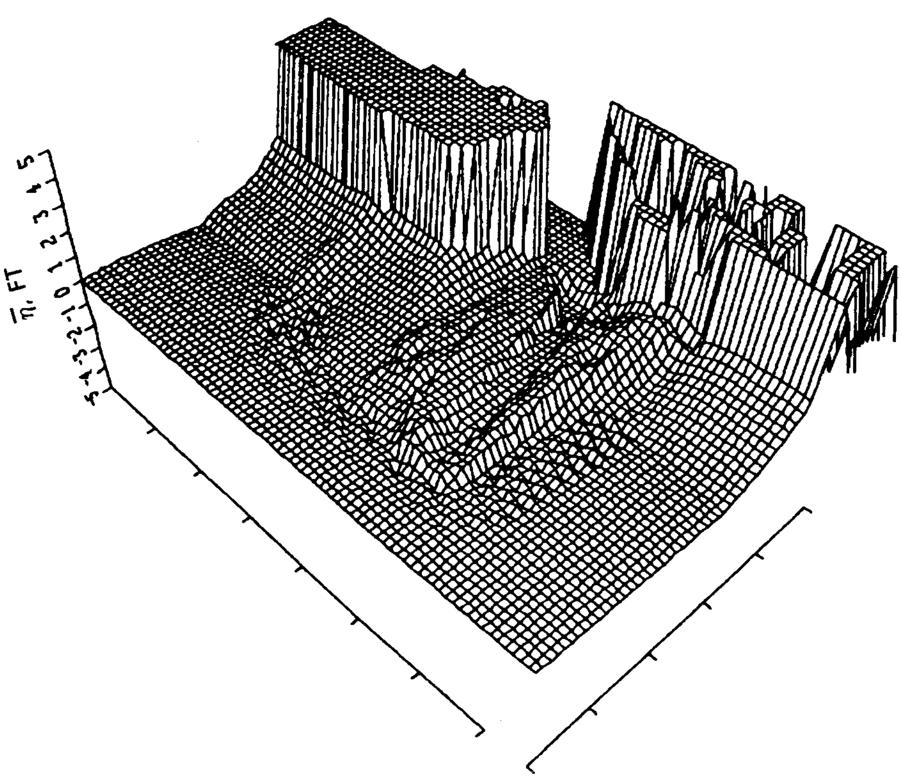


Figure 12. Water-surface elevation plot for Oregon Inlet, North Carolina, simulation

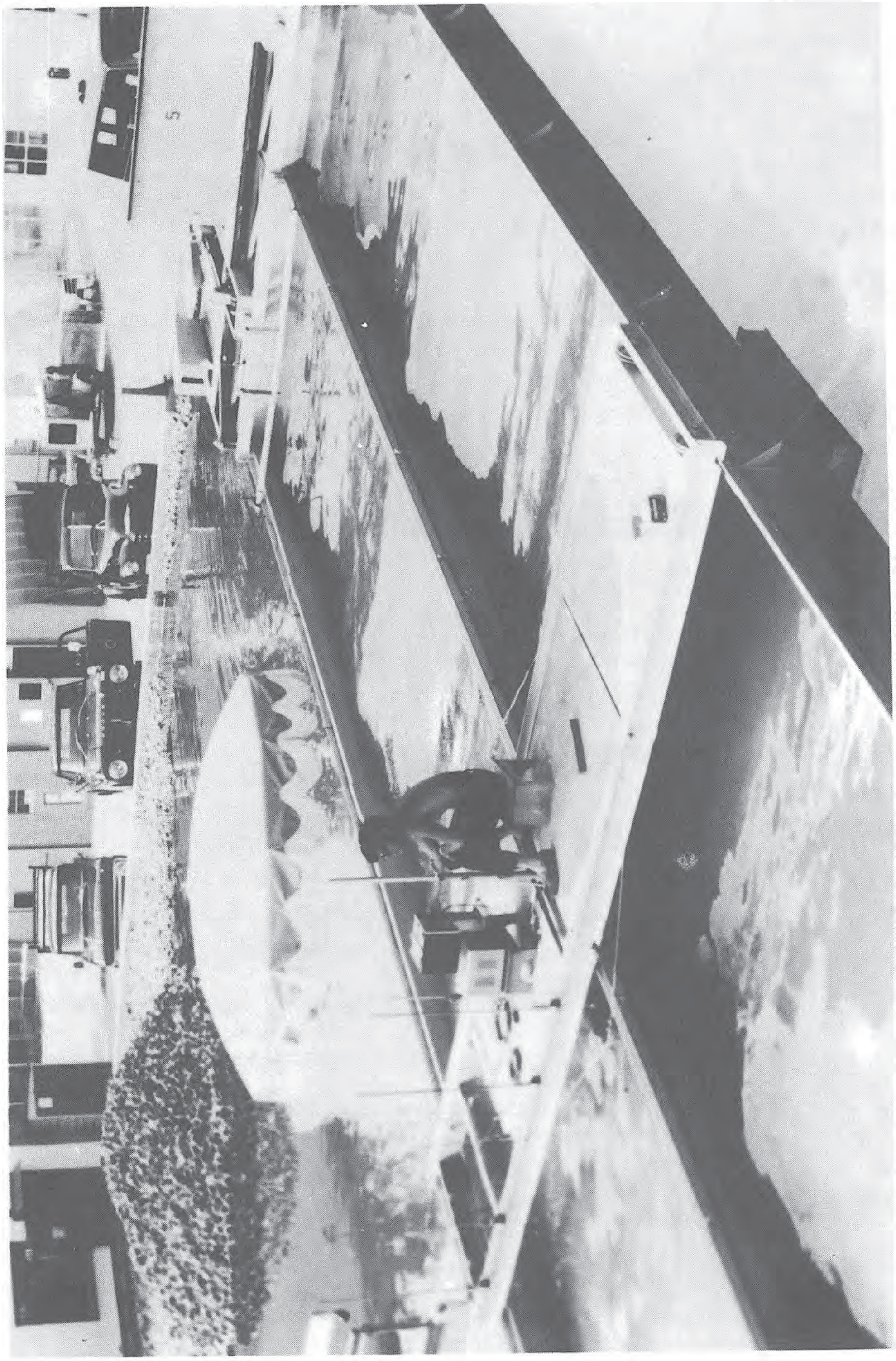


Figure 2. North section of Shore Processes Test Basin.

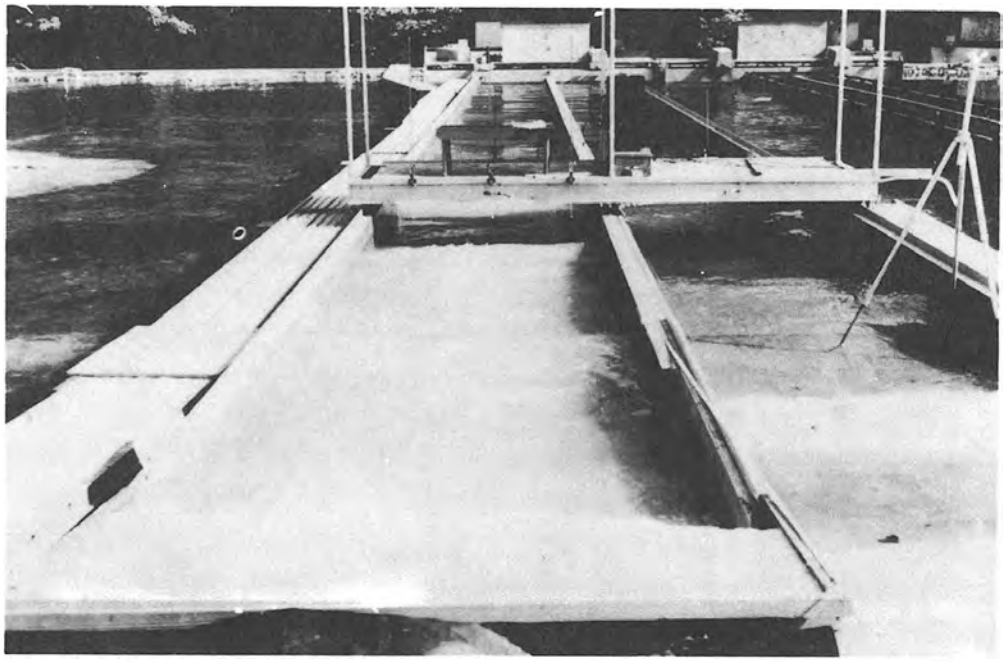


Figure 3. Seaward-looking view of 6-foot-wide wave tanks.

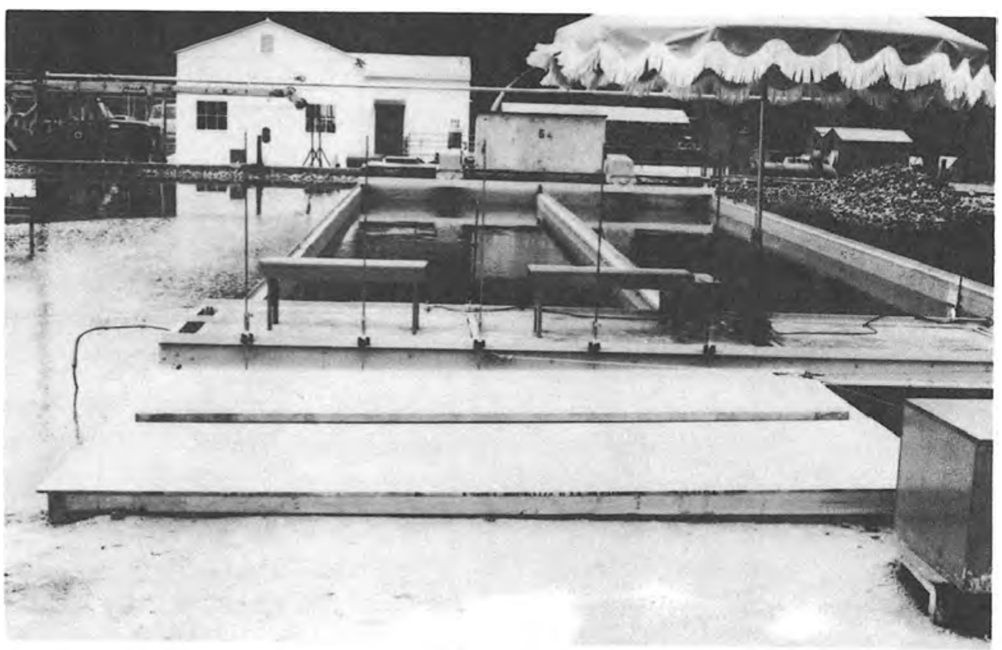


Figure 4. Seaward-looking view of 10-foot-wide wave tanks, with protective covers over subaerial beach.

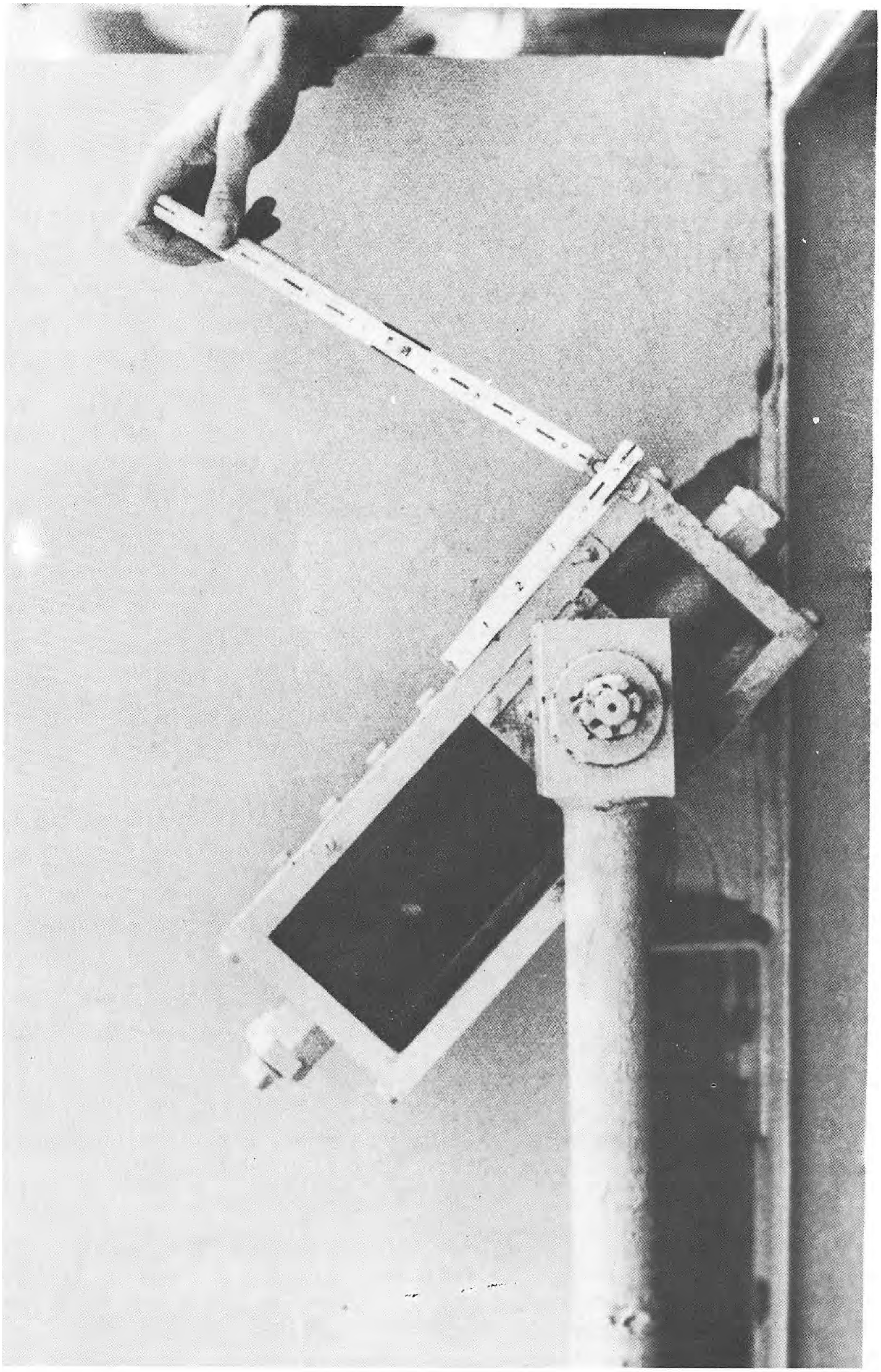


Figure 12. Eccentric setting on portable generator.

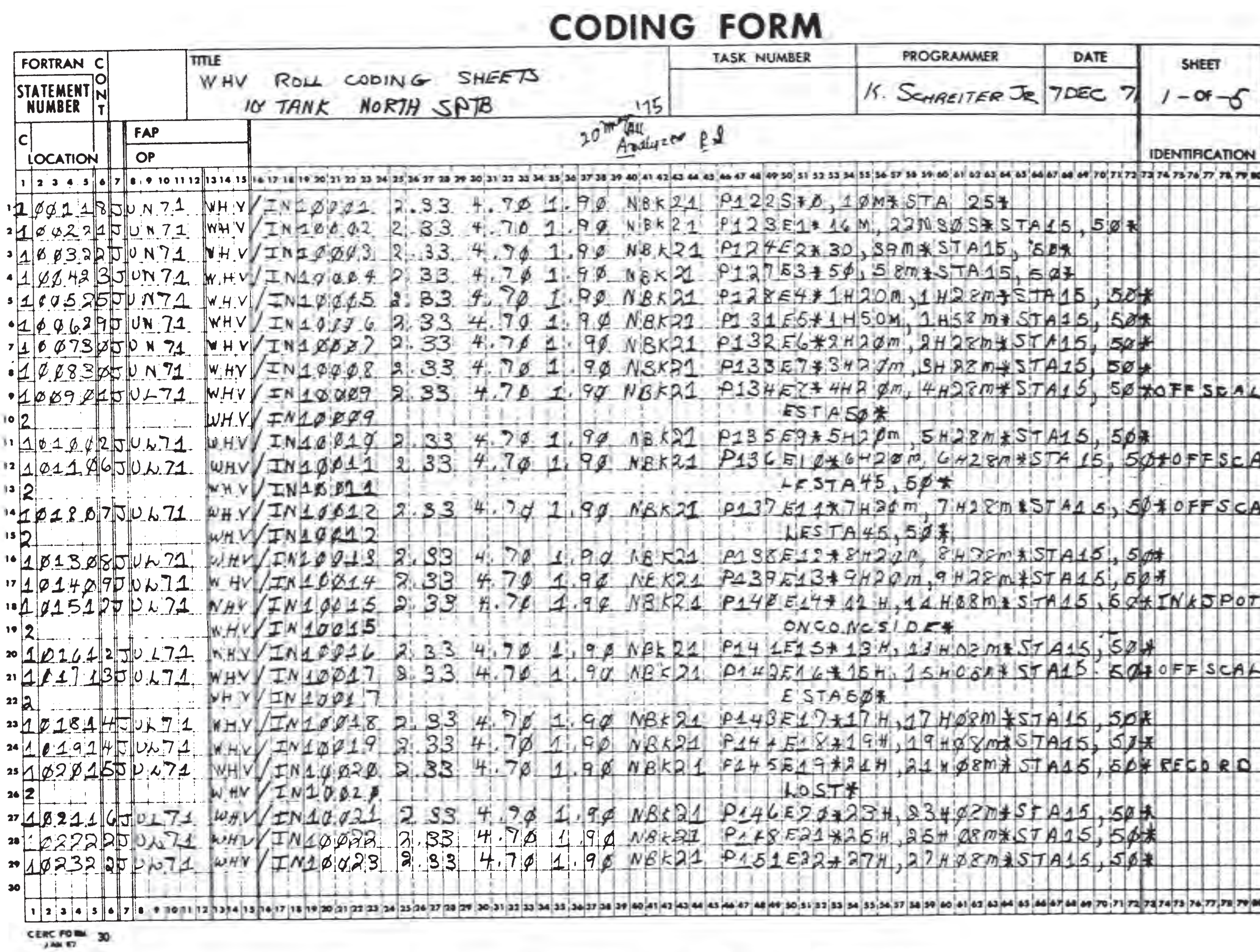


Figure A-4. Wave record lab coding form.

US Army Corps of Engineers
Waterways Experiment Station

Wetlands Research Program Technical Report WRP-DE-2

Wetland Evaluation Technique (WET)

Volume I: Literature Review and Evaluation Rationale

by Paul R. Adamus, Lauren T. Stockwell, Ellis J. Clairain, Jr.,
Michael E. Morrow, Lawrence P. Rozas, R. Daniel Smith

October 1991 – Final Report

Common Name	Scientific Name
Birds	
black duck	*Anas rubripes*
American coot	*Fulica americana*
bald eagle	*Haliaeetus leucocephalus*
bank swallow	*Riparia riparia*
black-bellied tree duck	*Dendrocygna autumnalis*
black skimmer	*Rynchops niger*
brant	*Branta bernicla*
bufflehead	*Bucephala albeola*
Canada goose	*Branta canadensis*
canvasback	*Aythya valisineria*
cliff swallow	*Petrochelidon pyrrhonota*
common goldeneye	*Bucephala clangula*
common merganser	*Mergus merganser*
fulvous tree duck	*Dendrocygna bicolor*
gadwall	*Anas strepera*
greater scaup	*Aythya marila*
white-fronted goose	*Anser albifrons*
harlequin duck	*Histrionicus histrionicus*
hooded merganser	*Lophodytes cucullatus*
least bittern	*Ixobrychus exilis*
lesser scaup	*Aythya affinis*
palm warbler	*Dendroica palmarum*
brown pelican	*Pelecanus occidentalis*
prothonotary warbler	*Protonotaria citrea*
red-breasted merganser	*Mergus serrator*
redhead	*Aythya americana*
ring-necked duck	*Aythya collaris*
ring-necked pheasant	*Phasianus colchicus*
roseate spoonbill	*Ajaia ajaja*
Ross' goose	*Chen rossii*
ruddy duck	*Oxyura jamaicensis*
seaside sparrow	*Ammospiza maritimus*
snow goose	*Chen caerulescens*
sora rail	*Porzana carolina*
whistling swan	*Olar columbianus*
wood duck	*Aix sponsa*
yellow-headed blackbird	*Xanthocephalus xanthocephalus*
Mammals	
beaver	*Castor canadensis*
manatee	*Trichechus manatus*
mink	*Mustela vison*
muskrat	*Ondatra zibethicus*
nutria	*Myocastor coypus*
raccoon	*Procyon lotor*
river otter	*Lutra canadensis*
sea otter	*Enhydra lutris*
swamp rabbit	*Sylvilagus aquaticus*
water shrew	*Sorex palustris*
white-tailed deer	*Odocoileus viginianus*
Virginia opossum	*Didelphis virginiana*

(Continued)

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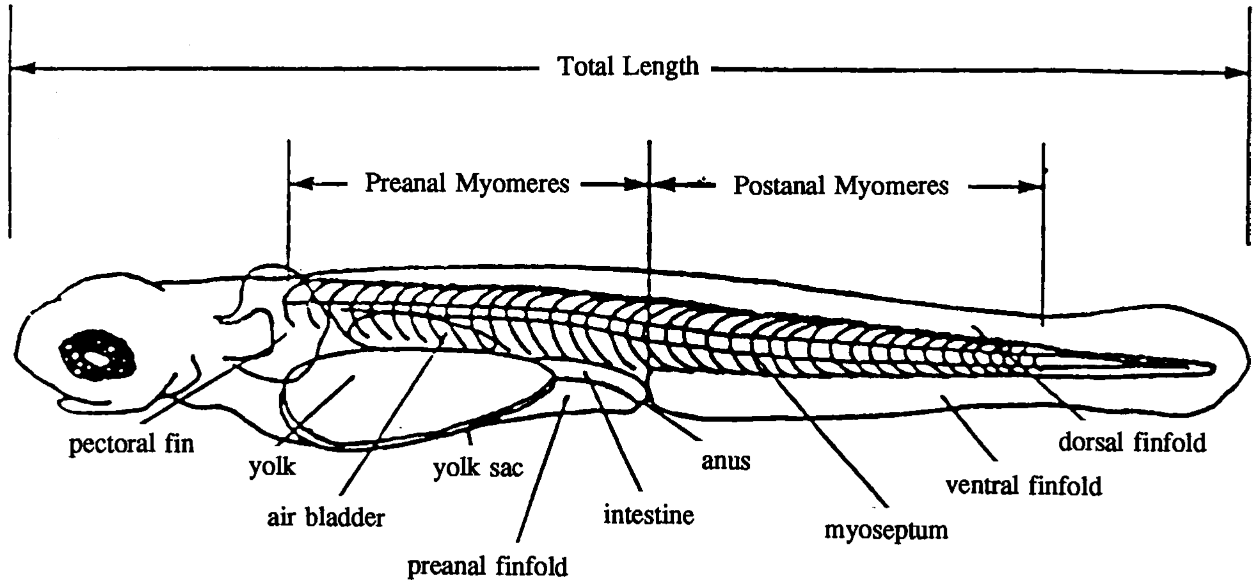
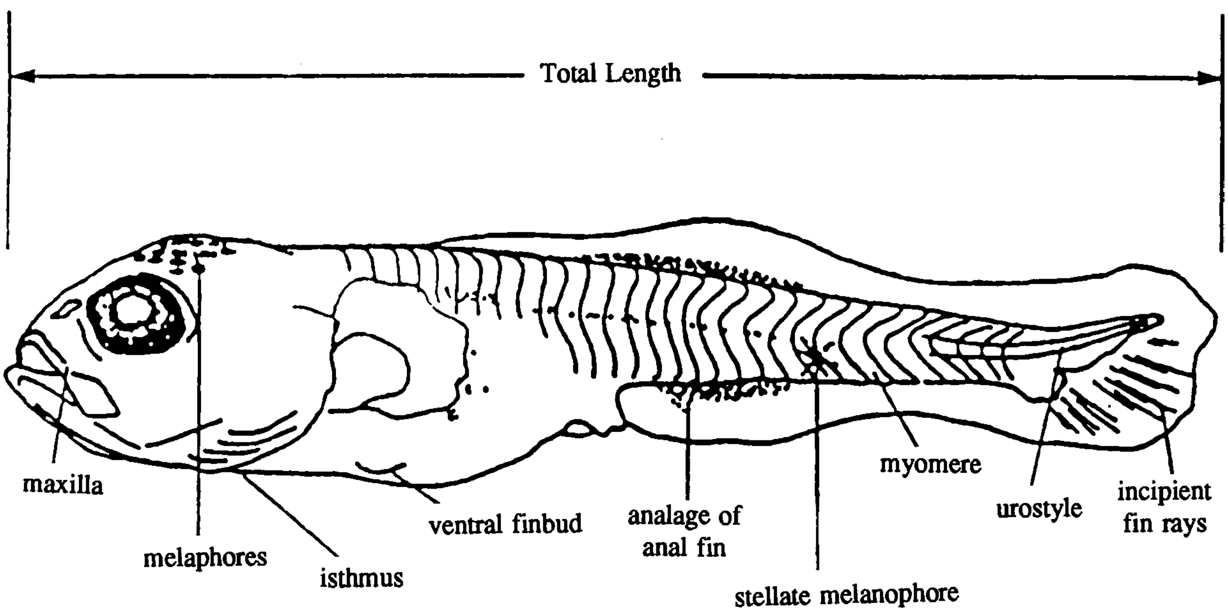


PROLARVA



POSTLARVA

Figure 2. Typical prolarval and postlarval fishes (after McGowen 1988)

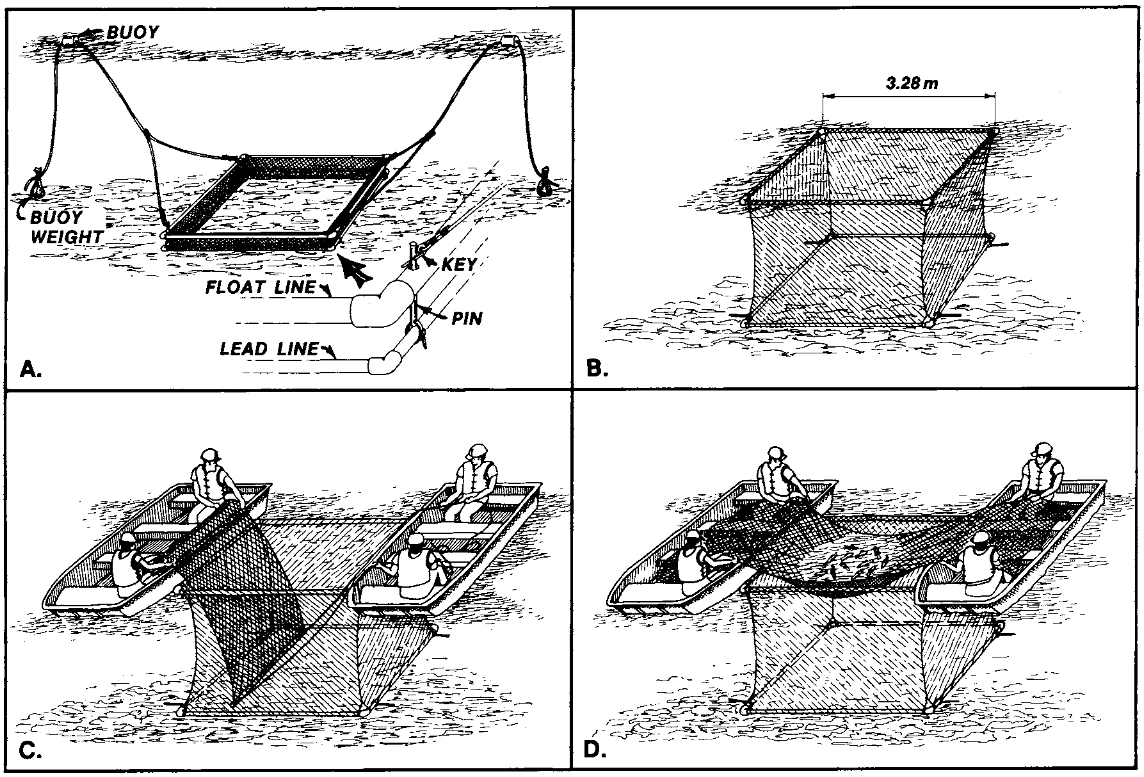
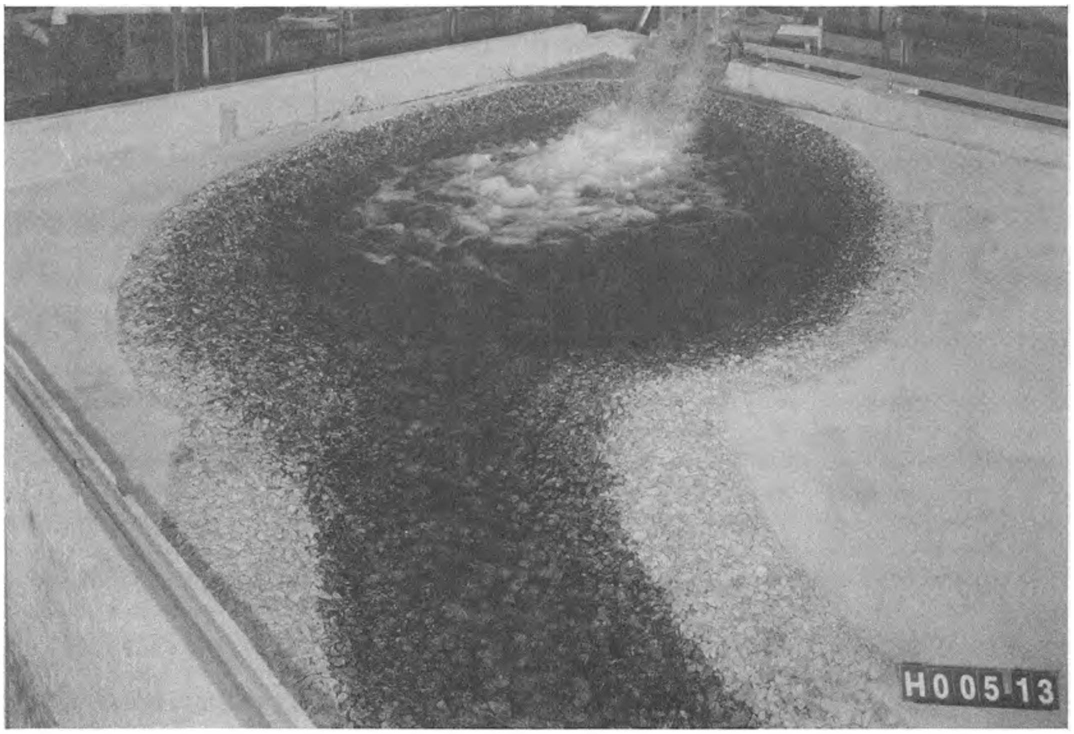
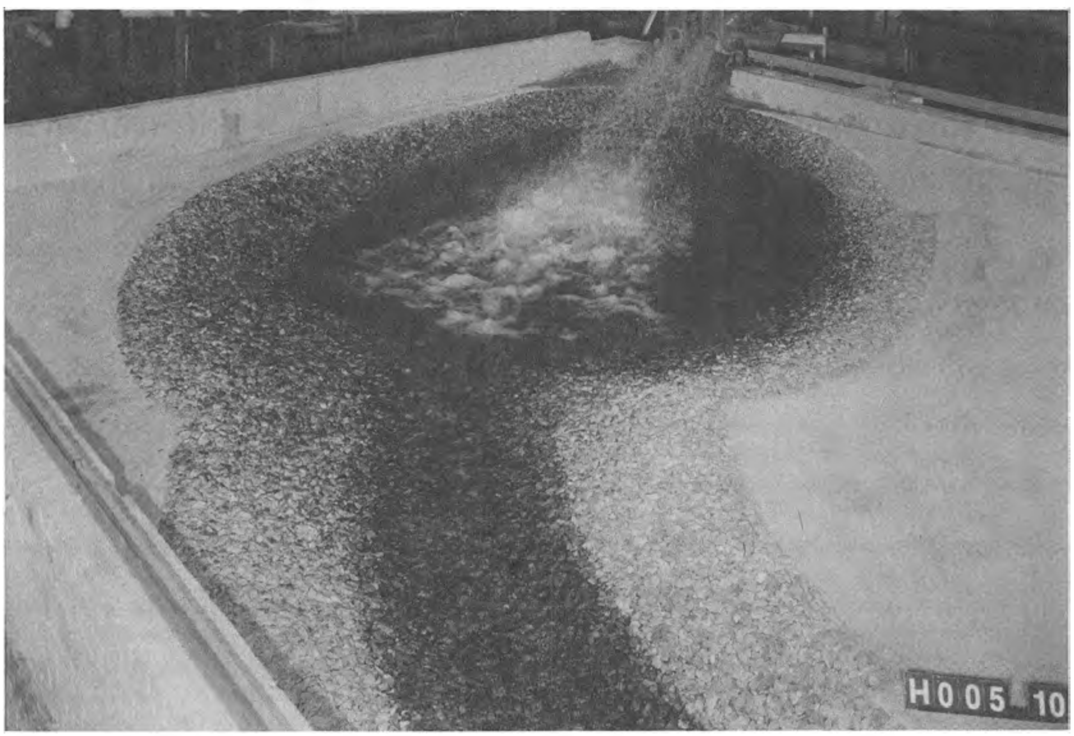


Figure 1. Schematic diagram of popnet system and the setup for fishing the net

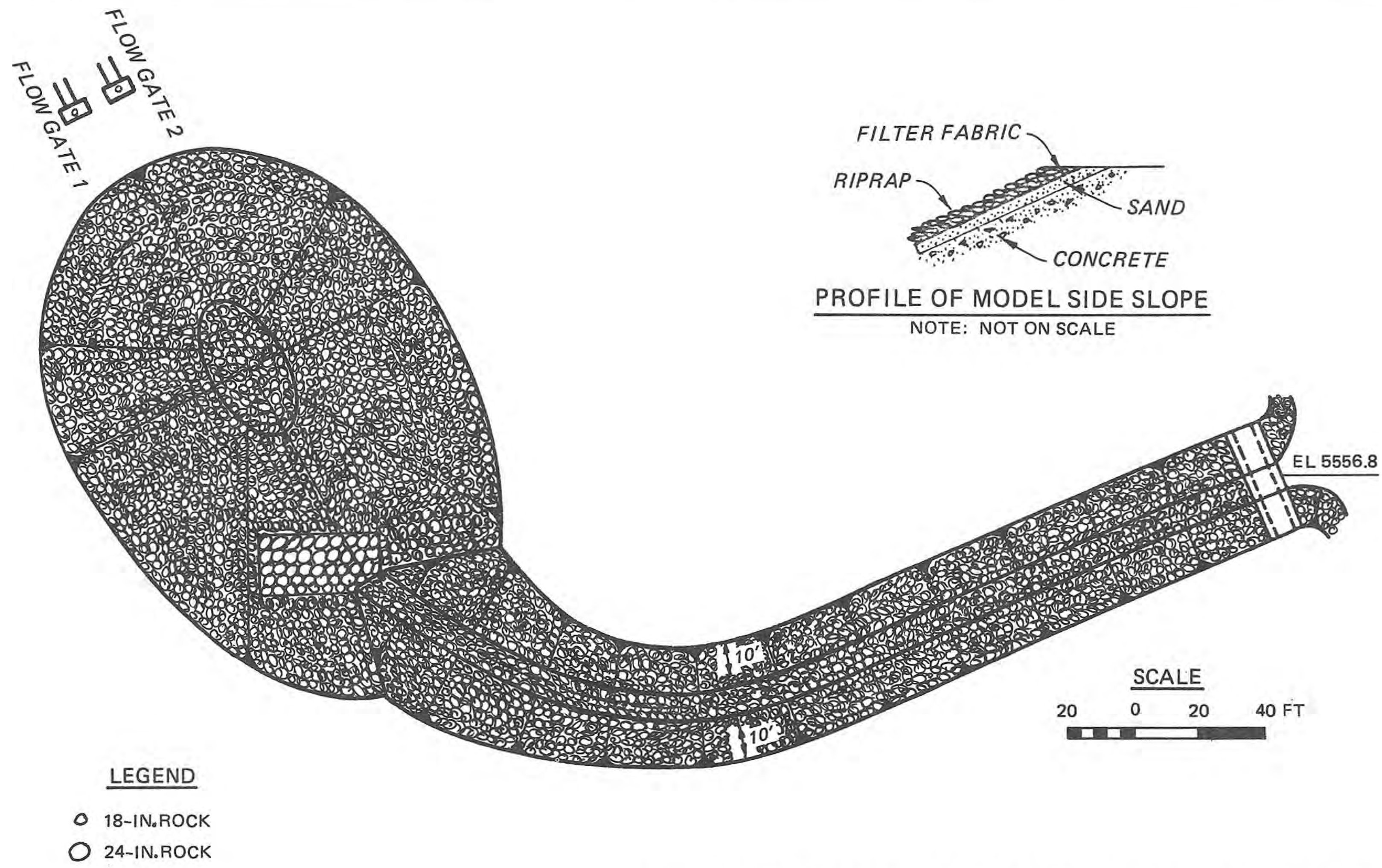


o. Pool el 5,740, gate 1 discharge 150 cfs and opening 66 percent, gate 2 discharge 150 cfs and gate opening 66 percent



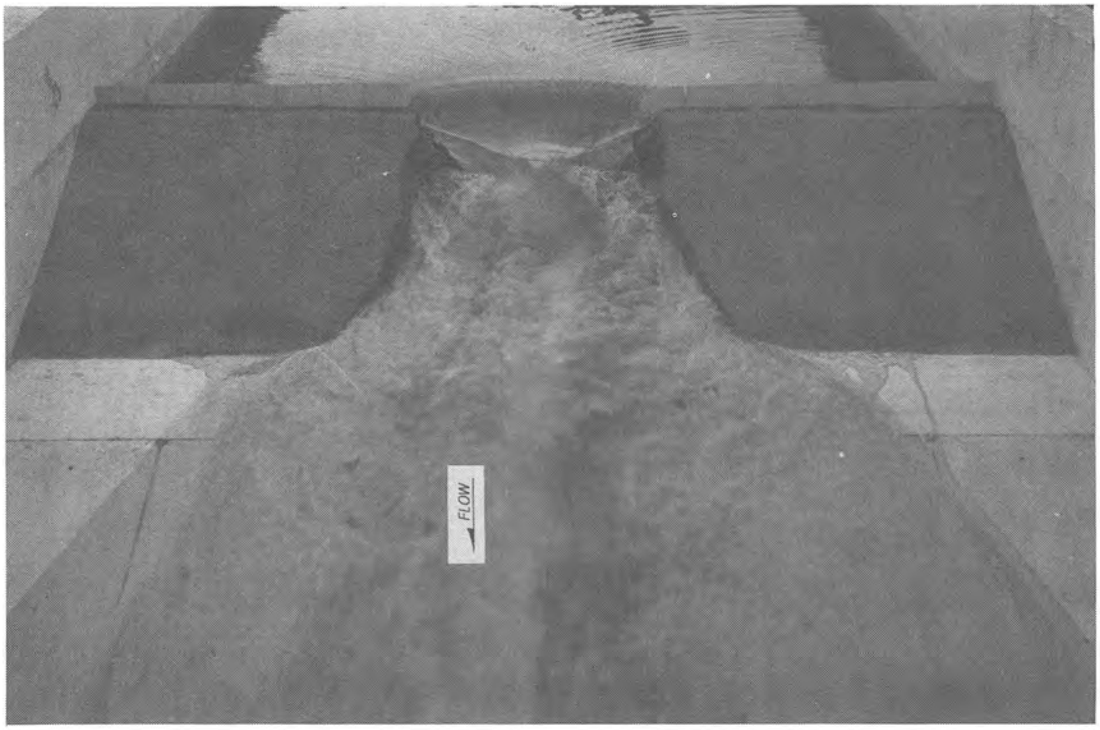
p. Pool el 5,798, gate 1 discharge 50 cfs and opening 15 percent, gate 2 discharge 50 cfs and opening 15 percent

Photo 3. (Sheet 8 of 9)

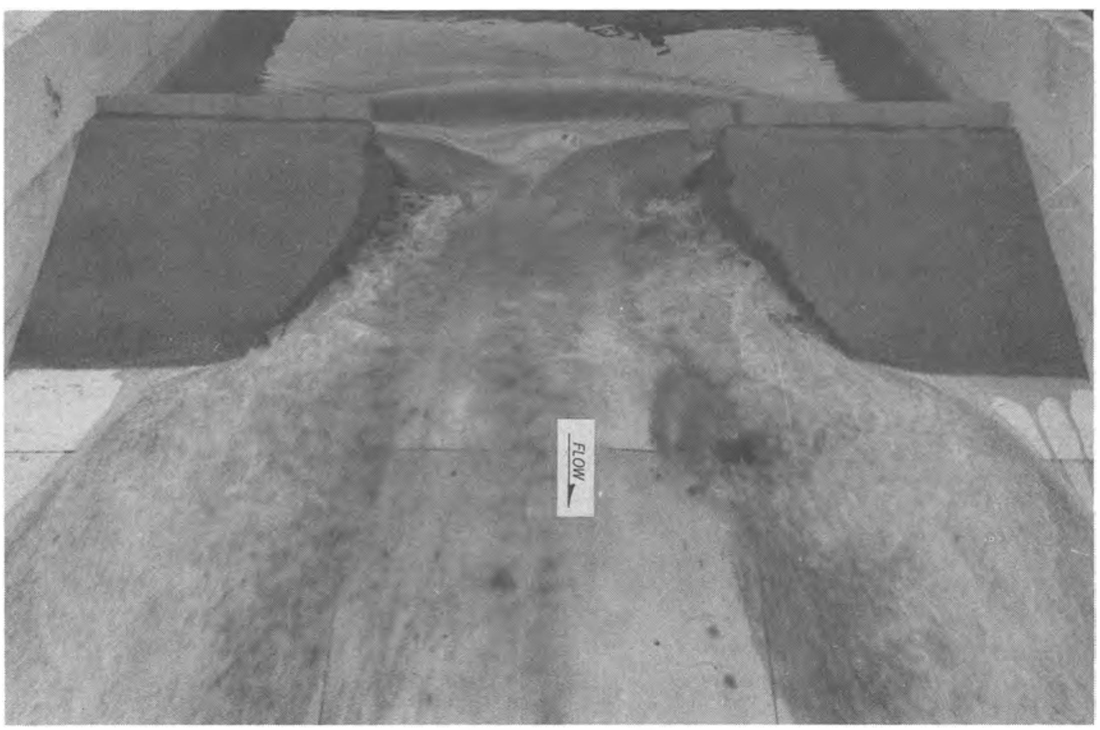


TYPE 2 PLUNGE POOL AND EXIT CHANNEL DESIGN
TYPE 3 RIPRAP DESIGN

PLATE 16

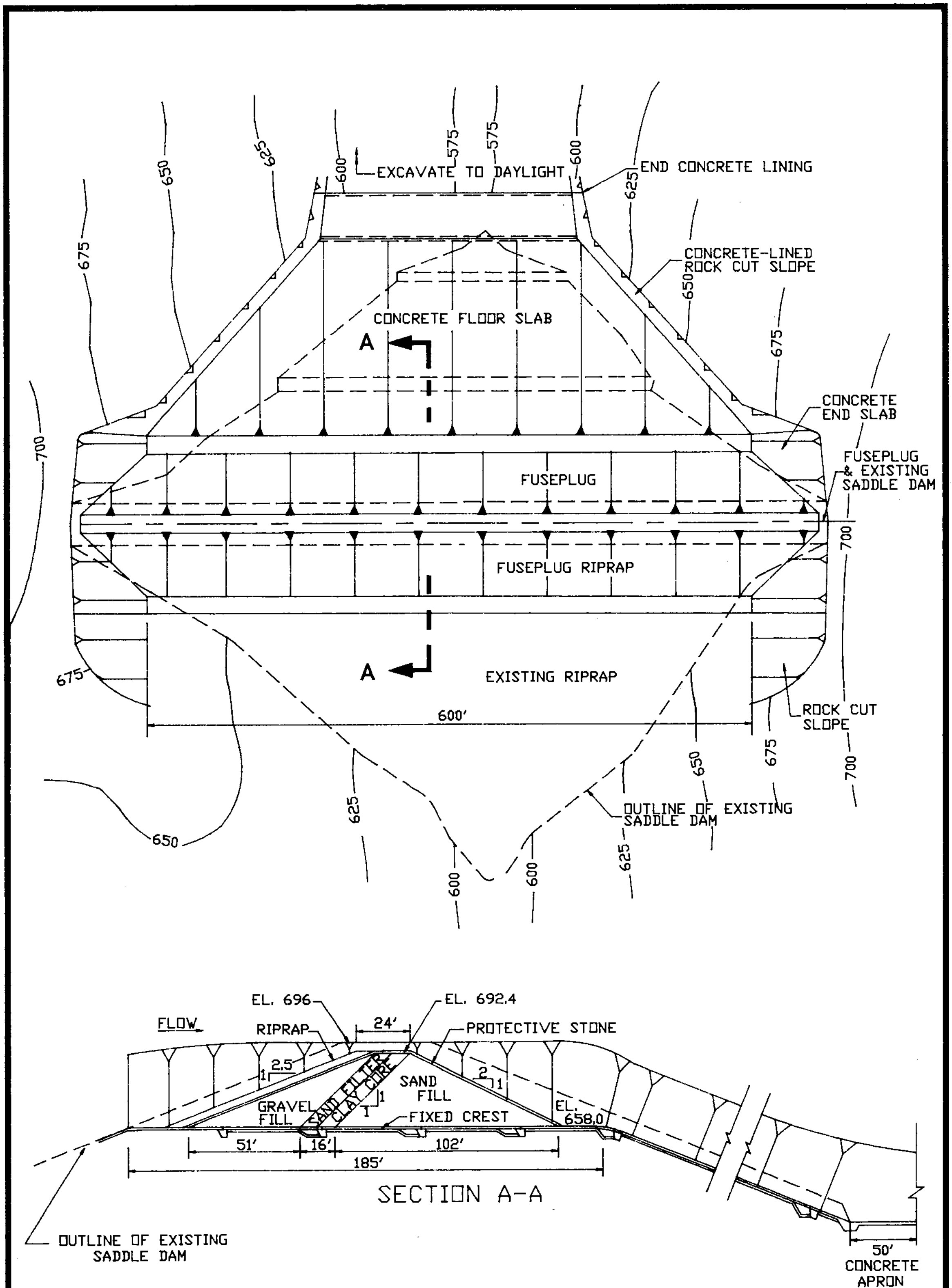


c. 75-ft-wide breach



d. Lateral erosion

Photo 4. (Concluded)



PLAN VIEW AND TYPICAL SECTION

PLATE 1

80WSE_322 "Center Hill Fuseplug Spillway, Caney Fork River, Tennessee: Hydraulic Model Investigation," 1992

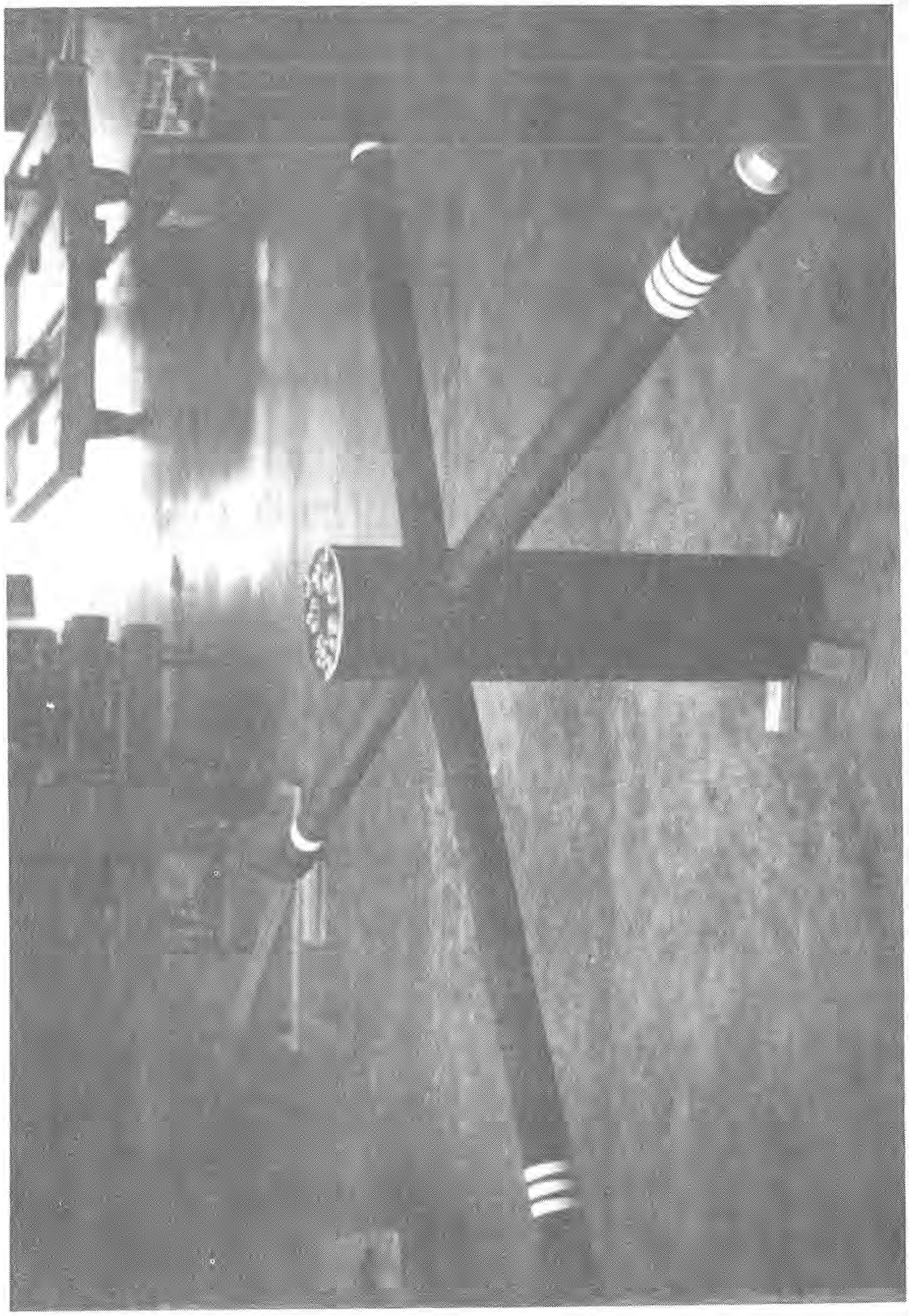


FIGURE III-1: The DPG Instrument.

FIGURE III-2: The DPG instrument secured to the steel cradle. The assembly is suspended beneath the Coastal Research Amphibious Buggy (CRAB) operated by the CERC Field Research Facility.

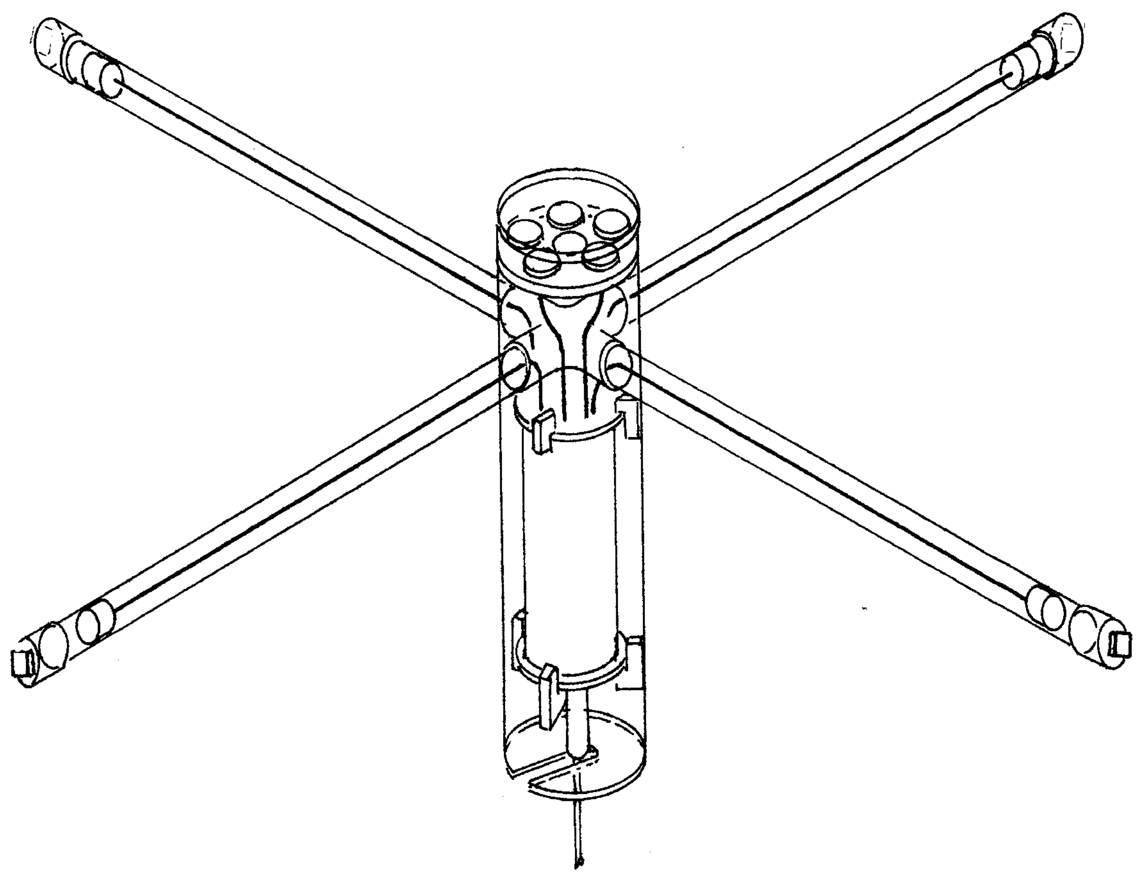


FIGURE III-3: Rendering of the DPG fuselage, arms, sensors, and water-tight instrumentation cylinder.

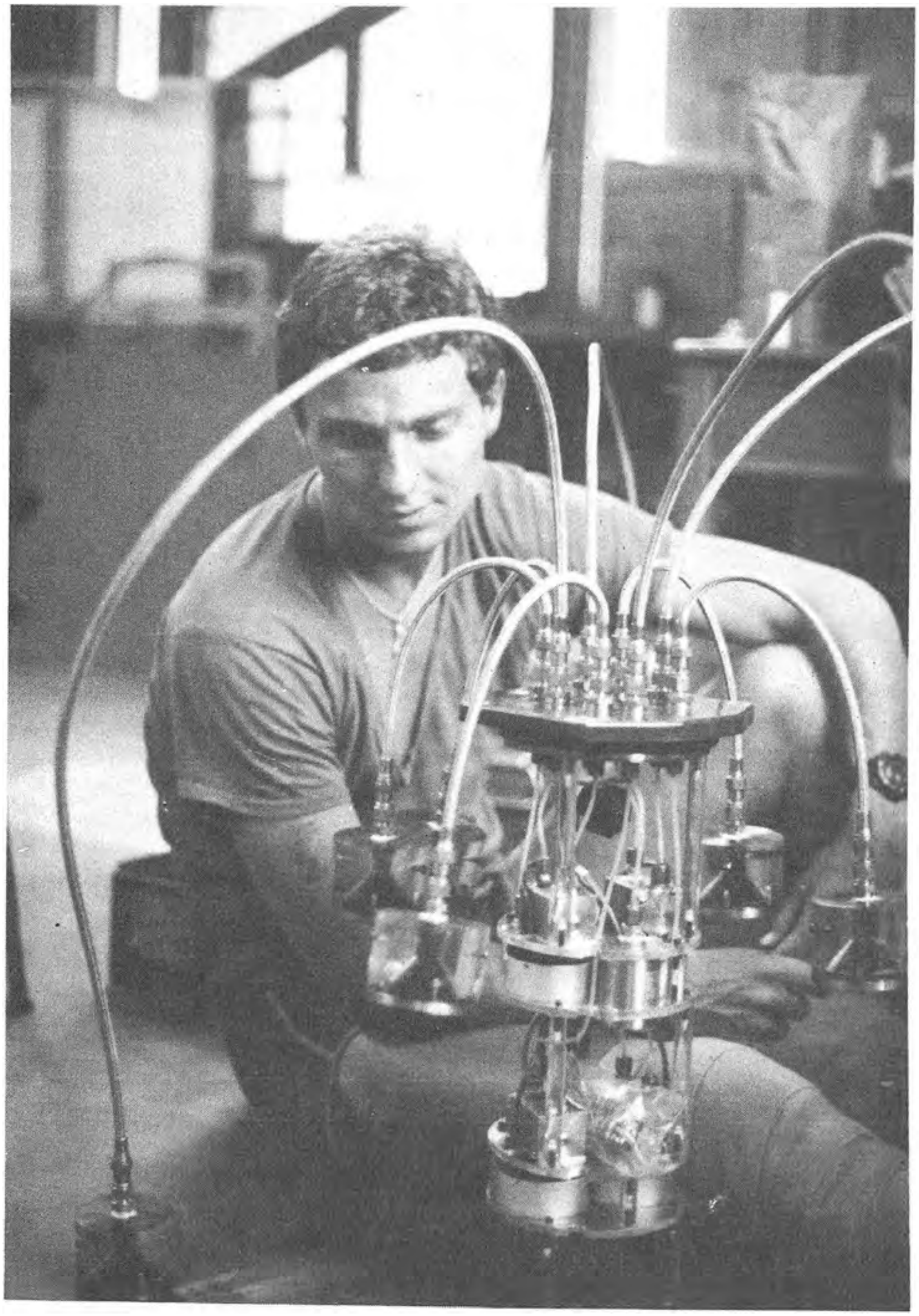


FIGURE III-5: Isolation sensor diaphragms shown attached by flexible and nylon tubing through the top of the water-tight instrumentation cylinder and into the transducers.

The instrument was calibrated twice; once with the 1/8 inch arm and 1/16 inch I.D. center inflexible tubing used at CERC, and two weeks later, with the 1/8 inch I.D. flexible tubing used on the

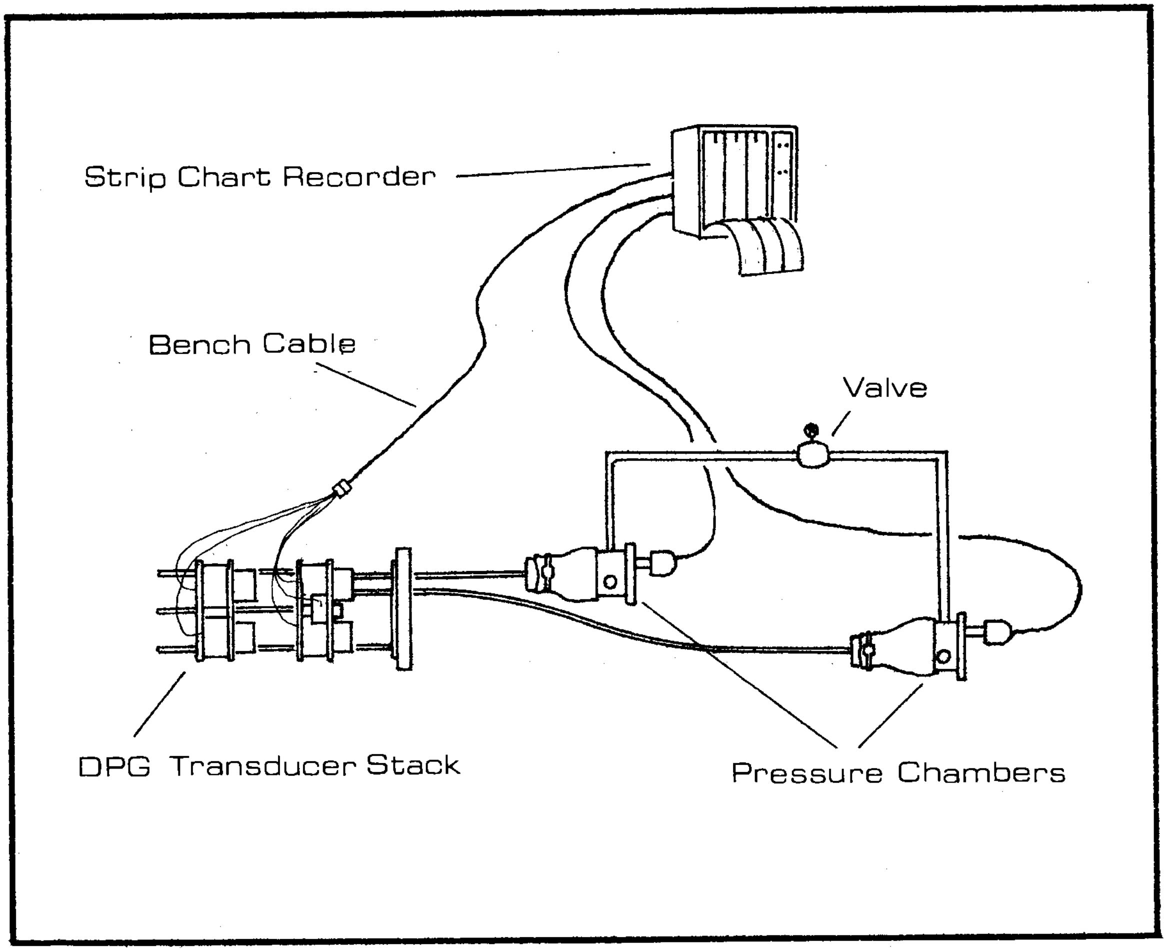


FIGURE V-3: The DPG bench calibration system. One pair of sensors shown for clarity.

final instrument installed at the FRF, Duck, NC. Early during the first calibration, one of the chamber transducers failed. This meant that only one side of a differential transducer could be loaded at a

FIGURE VI-3: Preparing for field installation: the DPG suspended underneath the operator's platform of the CRAB.

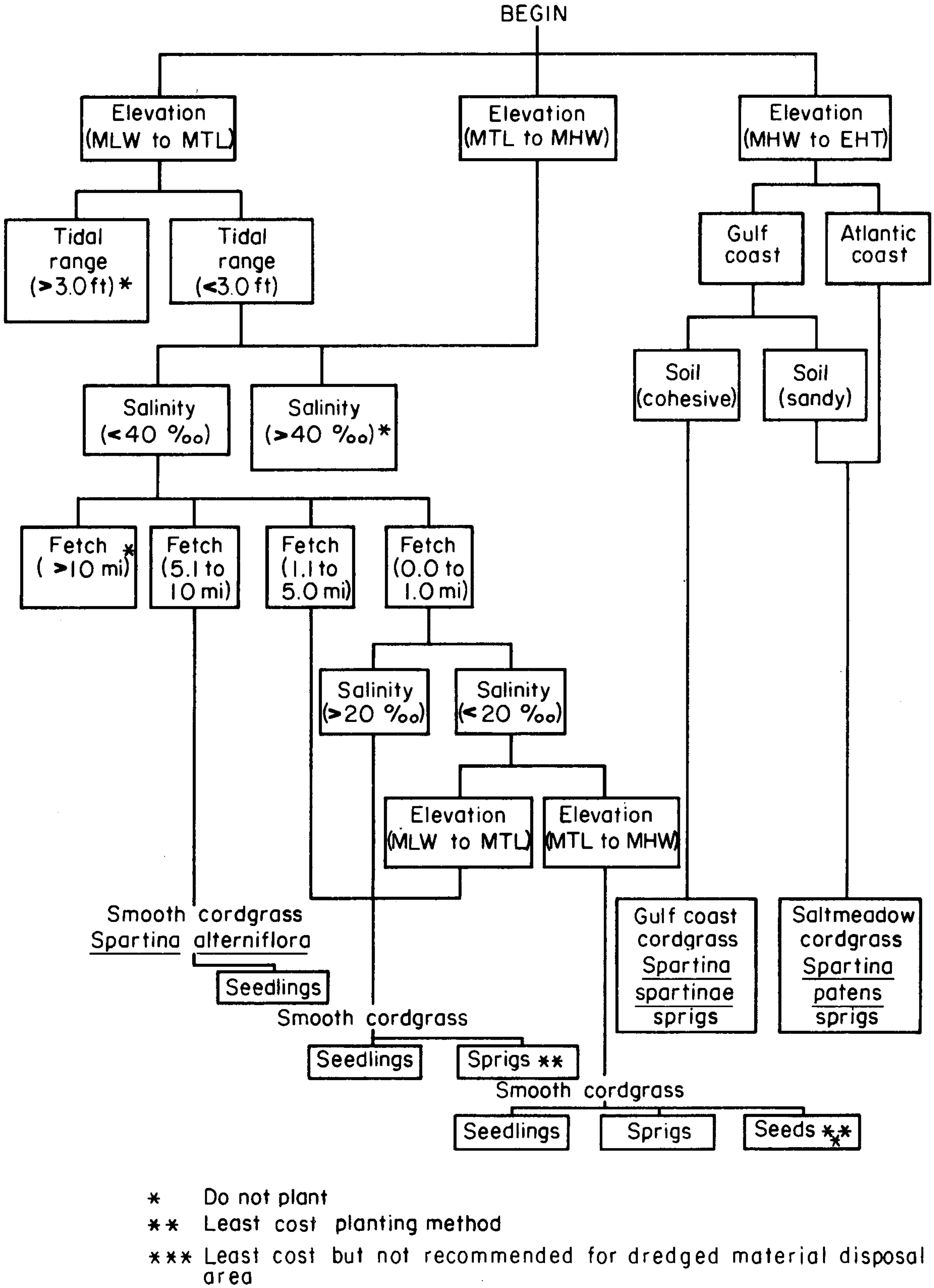
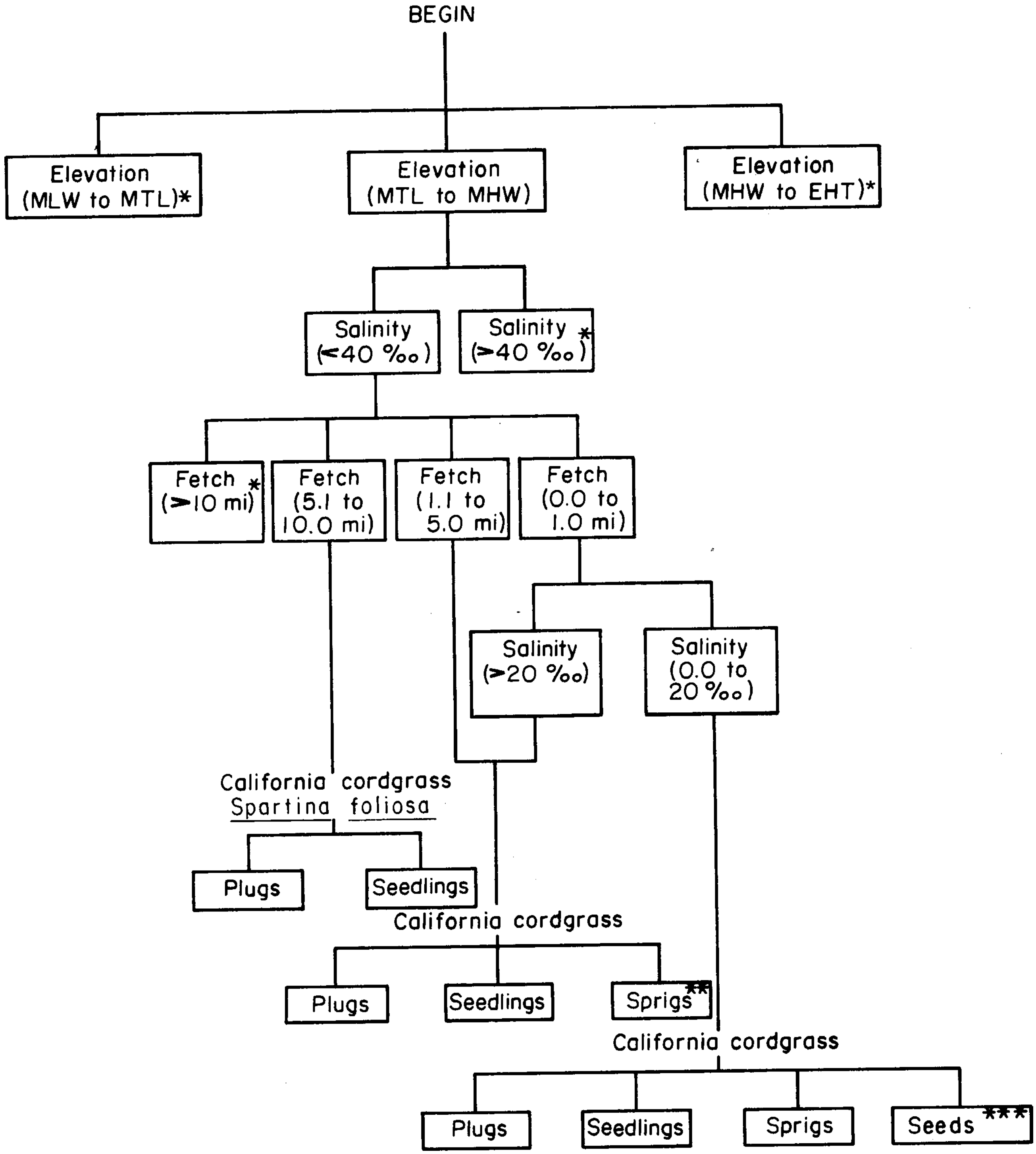


Figure 1. Planting decision key, Atlantic and gulf coasts.



* Do not plant

** Least cost planting method

*** Least cost but not recommended for marsh development on dredged material

Note: Tidal range not a determining factor on Pacific coast

Figure 2. Planting decision key, Pacific coast.

III. DETERMINING SEED APPLICATION RATE AND PLANT SPACING

1. Seeding.

The recommended application rate for seed is given by:

$$\text{Seed application rate} = Ra \times Qs$$

where Ra is a base application rate of 2 gallons per acre (19 liters per hectare) of seeds for California cordgrass (*Spartina foliosa*) or 1 gallon per acre (9.5 liters per hectare) of seeds for smooth cordgrass (*S. alterniflora*), and Qs is the seed quality index for the seed source. Qs is approximated by collecting and examining seed before harvest. The total number of spikelets examined divided by the number of full spikelets (Fig. 3) is the Qs.

Figure 3. Full spikelet.

Threshing will reduce the volume of harvested material by about 50 percent. Therefore, the volume which must be harvested in anticipation of planting is:

$$\text{Harvest volume} = 2(Ra \times Qs) \times A$$

where A is the area to be planted in acres.

2. Sprigs, Plugs, and Seedlings.

The Table summarizes recommended plant spacing for marsh development projects (plant cover in three growing seasons) and for bank stabilization projects (cover in one to two seasons).

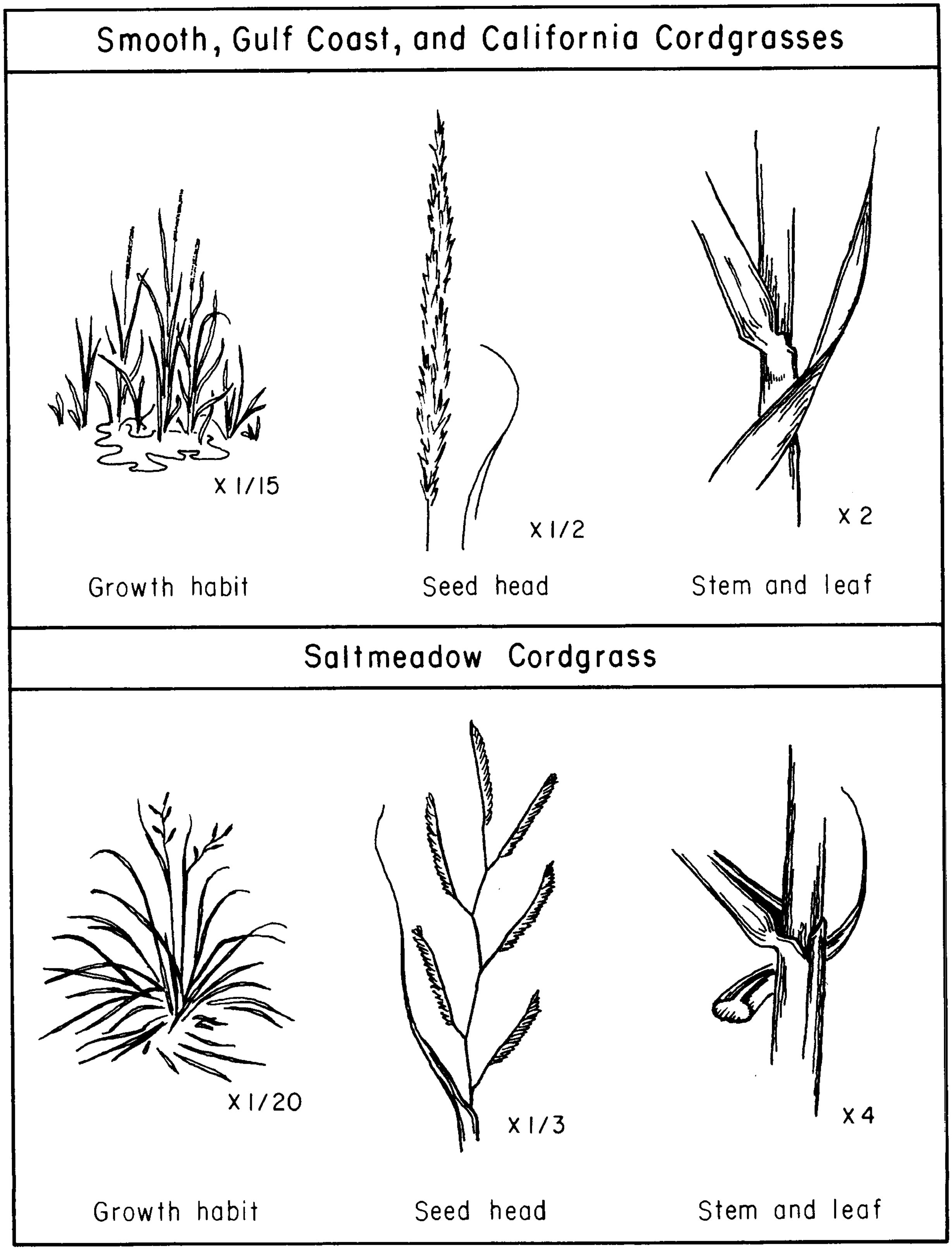


Figure A-1. Plant identification.

[redacted]

C-1

CRREL
REPORT 82-40

US Army Corps of Engineers

Cold Regions Research & Engineering Laboratory

Breaking ice with explosives

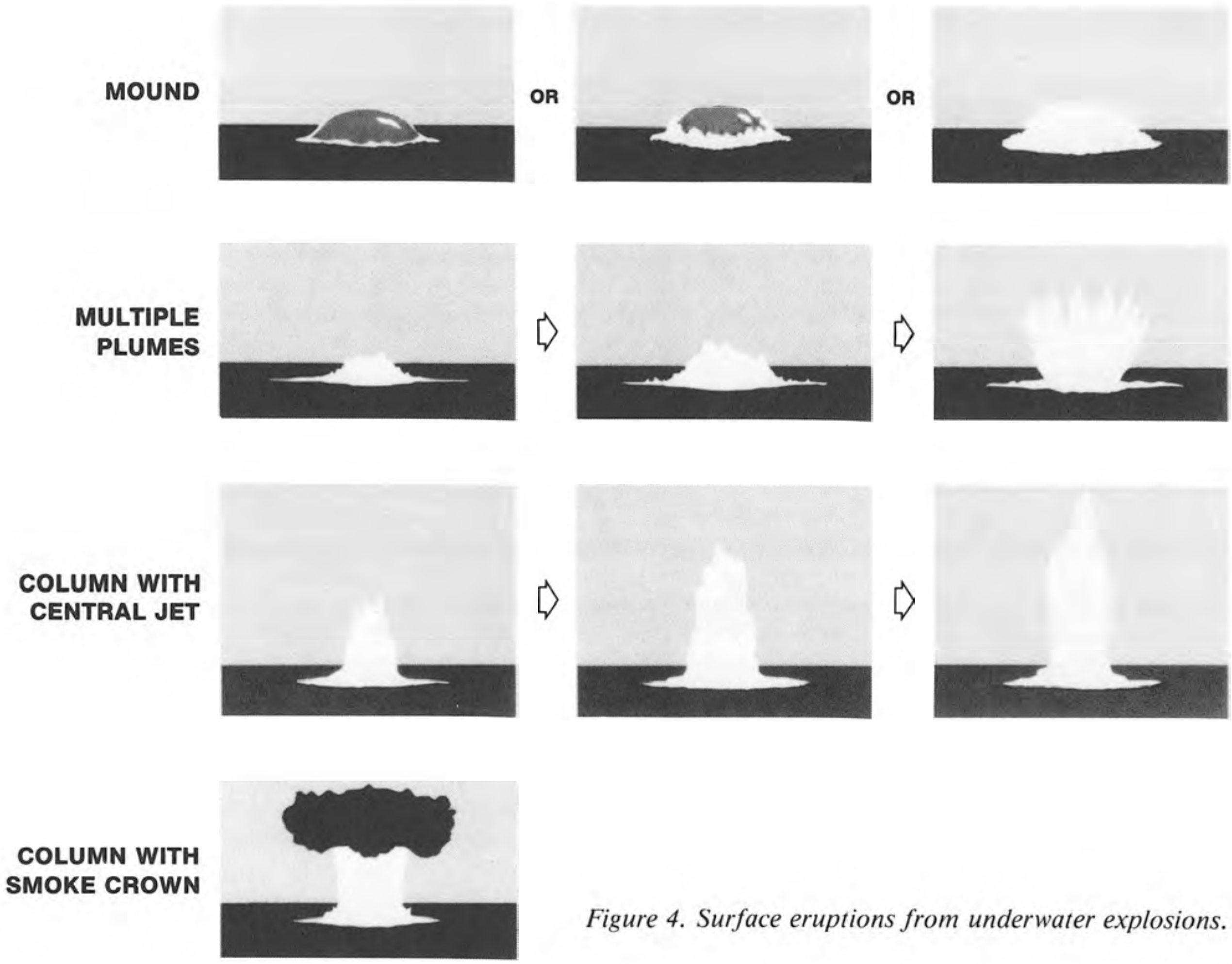


Figure 4. Surface eruptions from underwater explosions.

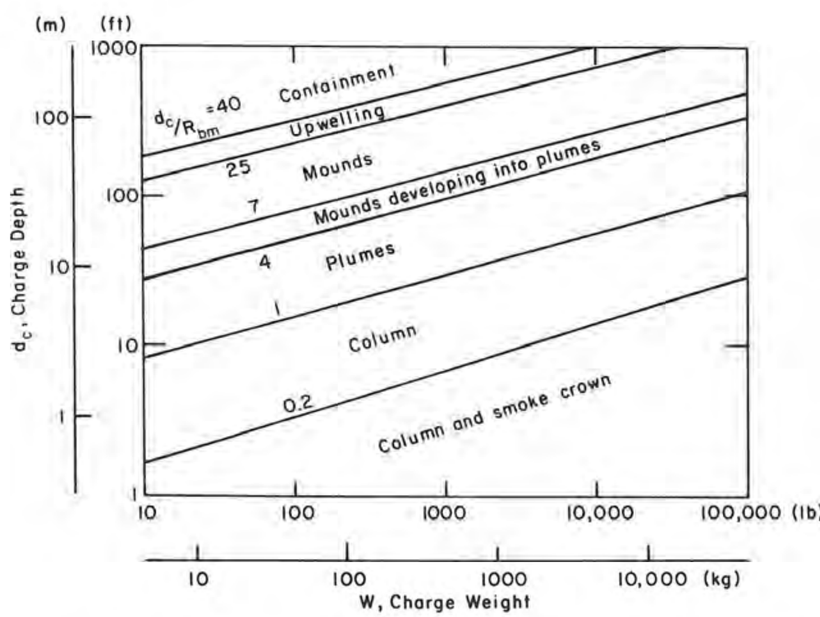


Figure 5. Surface effects from underwater explosions. (After Young 1971.)

Figure 7. *Spartina alterniflora* seedlings, 15 weeks old, in 10-centimeter-diameter peat pots (five seedlings per pot).

Figure 9. Seed germinating in sandfilled peat pots (center).

Figure 10. Seed germination in an environmentally controlled chamber.

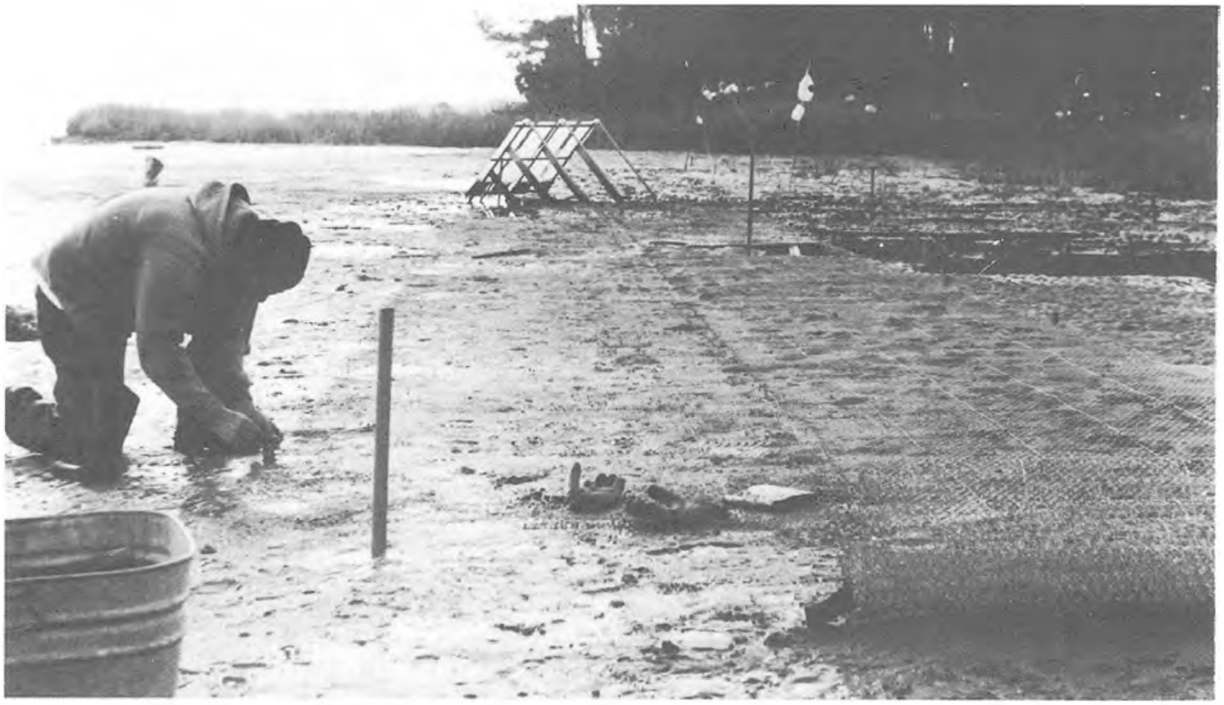


Figure 28. Placing of galvanized wire netting in the fall of 1973 to protect transplants from wildlife feeding.

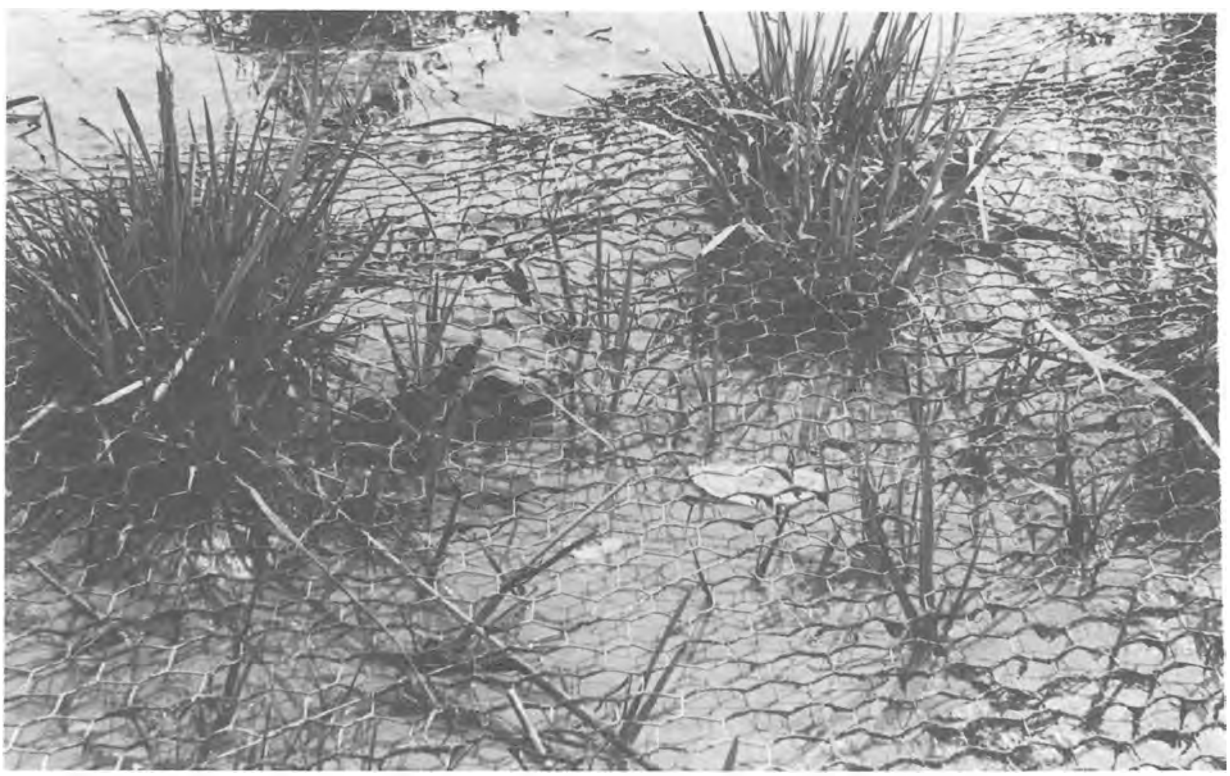


Figure 29. New plant growth emerging through protective netting, spring 1974.

Figure 33. Looking southward at transplants in section 4B at site 1 (Fig. 2) in July 1973. Note transplants of *S. alterniflora* (center) and of *S. patens* and *A. breviligulata* (left).

Figure 34. Looking southward at transplants in section 4B at site 1 (Fig. 2) in April 1974. Transplants of *S. alterniflora* (center) are barely visible. Note extensive drift material at left and in the region of *A. breviligulata* transplants.

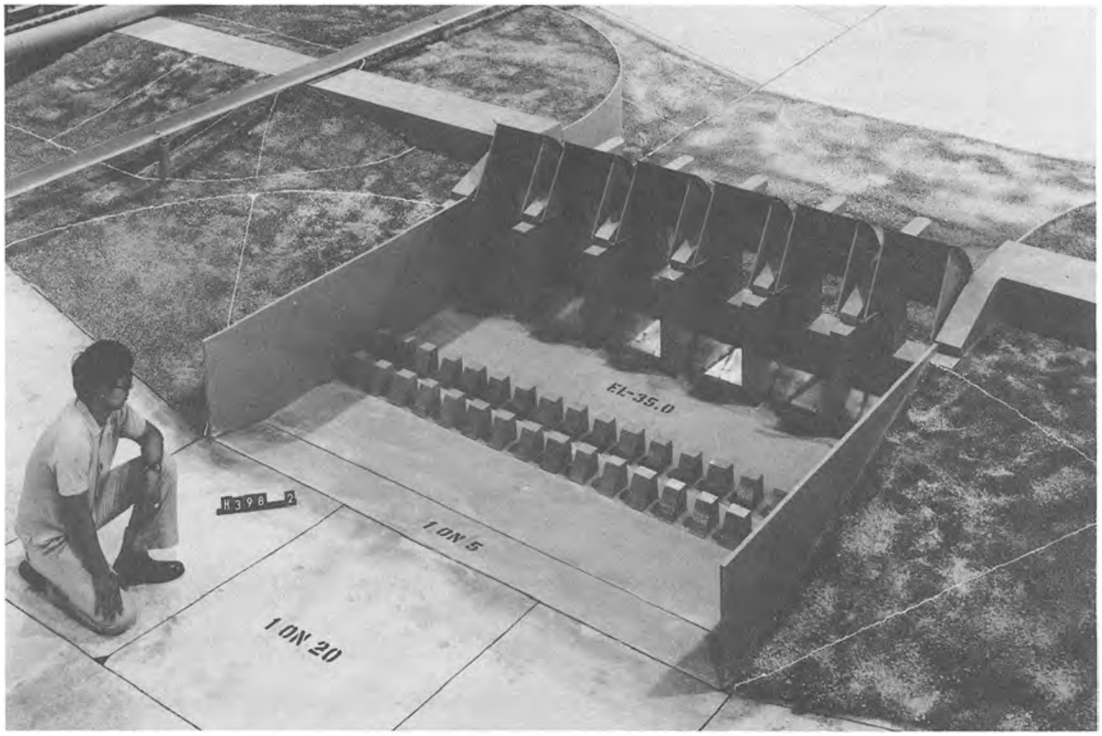


Figure 6. Original stilling basin design

evaluating hydraulic performance with various apron elevations. Tests and computations indicated that it would be hydraulically and economically feasible to elevate the apron. Flow conditions with the type 1 (original) design stilling basin apron are shown in Photos 5-7. Flow conditions with alternative designs, type 2 (apron el -25.0) and type 3 (apron el -15.0) design stilling basin aprons, are illustrated in Photos 8-9 and 10-11, respectively. The highest apron (el -15.0) did not provide satisfactory hydraulic performance with a discharge of 90,000 cfs passing through a fully opened single gate bay as excessive standing waves (Photo 10), turbulence, and unsatisfactory velocity distribution were observed. The type 4 (recommended) design stilling basin apron (el -20) was at the highest apron elevation that would maintain satisfactory hydraulic performance with fully open single gate operation. Velocities measured at the end of the apron with various apron elevations investigated are shown as isovels in Plates 28-39. A comparison of Plates 28-39 indicates that the best flow distribution is obtained with the apron located at el -20. Plate 40 indicates that the apron should not be higher than el -20 due to the excessive standing waves that occurred with a discharge of 90,000 cfs through a fully opened single gate. Additional tests

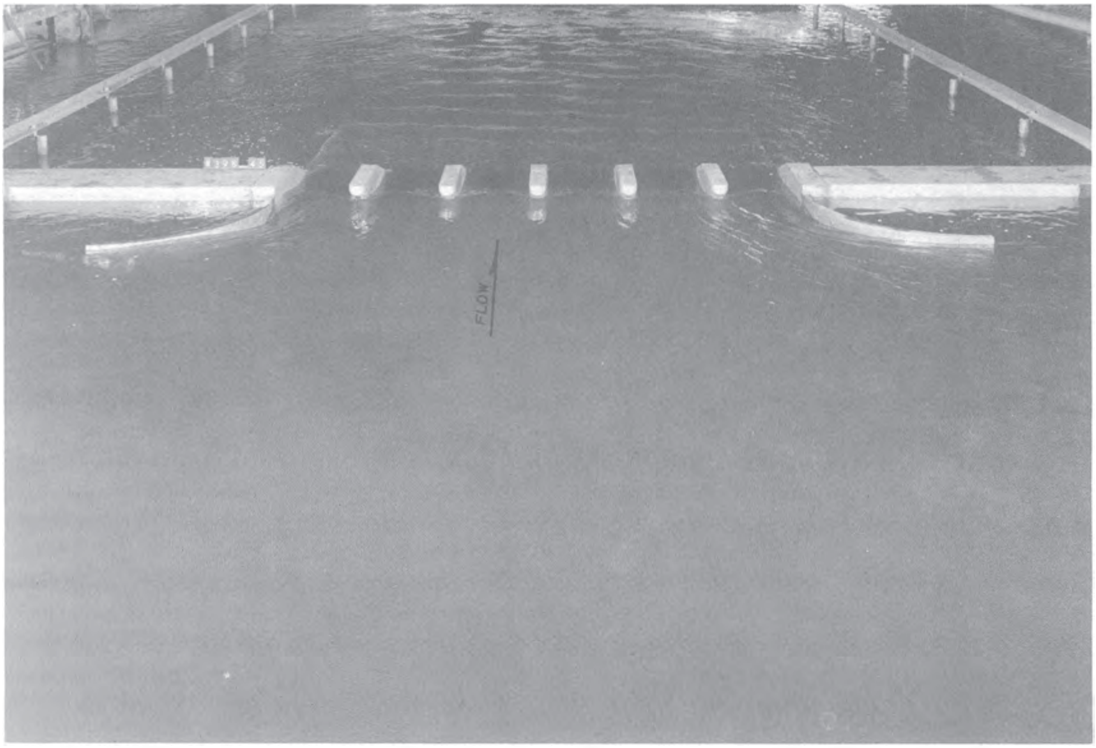


Photo 3. Approach flow, discharge 550,000 cfs, pool el 66.4, tailwater el 61.4, gates fully open

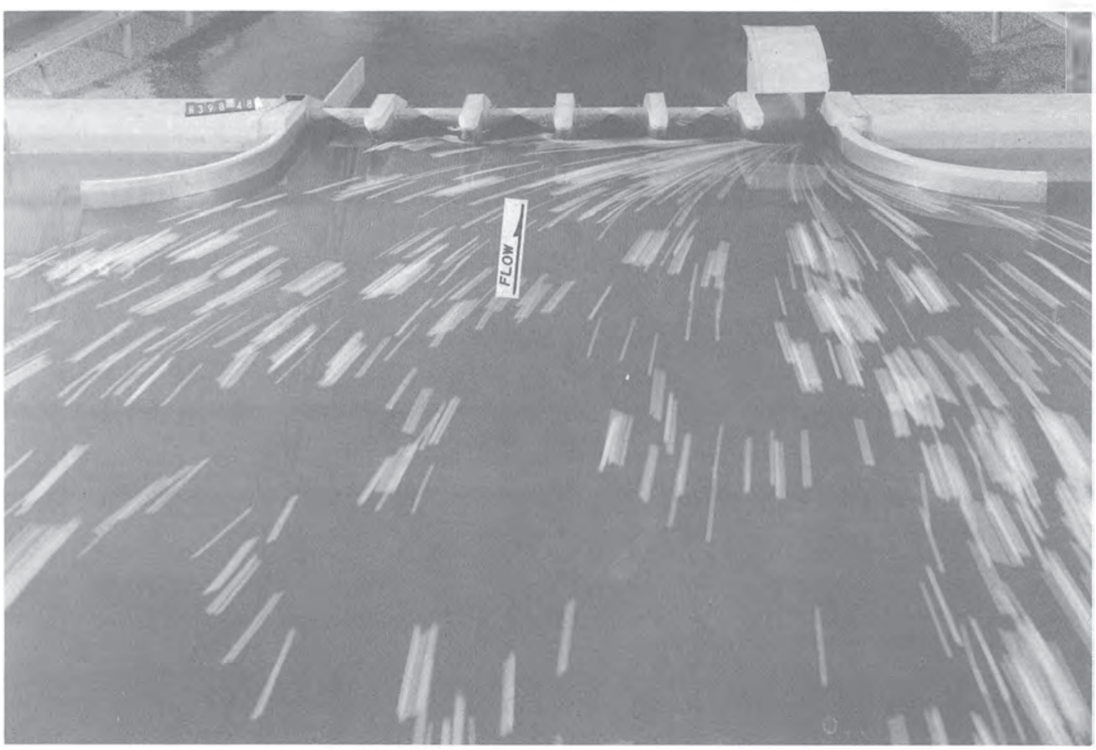


Photo 4. Approach flow, discharge 90,000 cfs, pool el 55.0, tailwater el 24.2, one gate fully open, 15-sec exposure time (prototype)

Figure 13. Masonboro Inlet, North Carolina, April 1968.

A suggested flow net grid system which includes important friction zones of the idealized inlet is shown in Figure B-5.

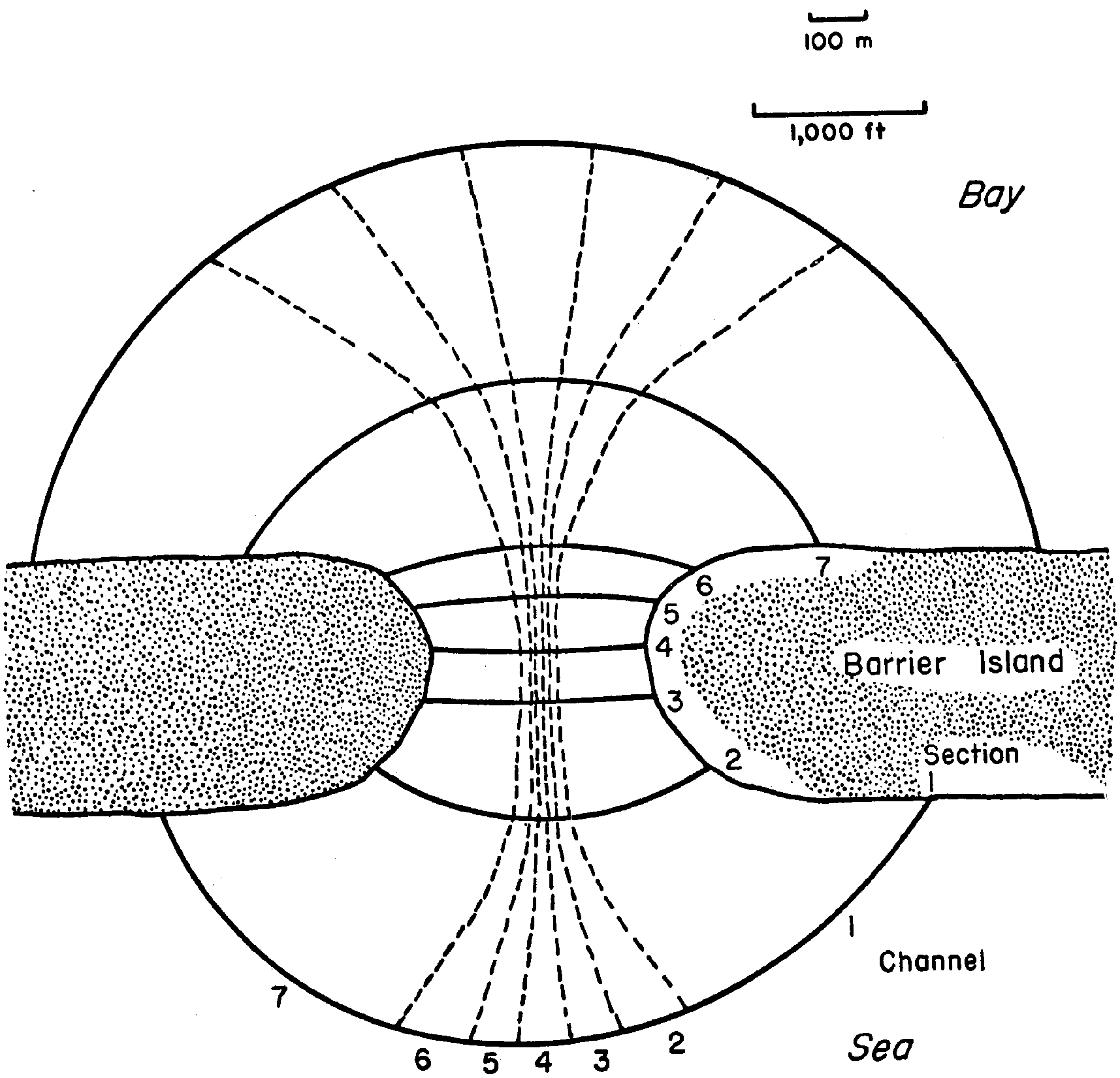
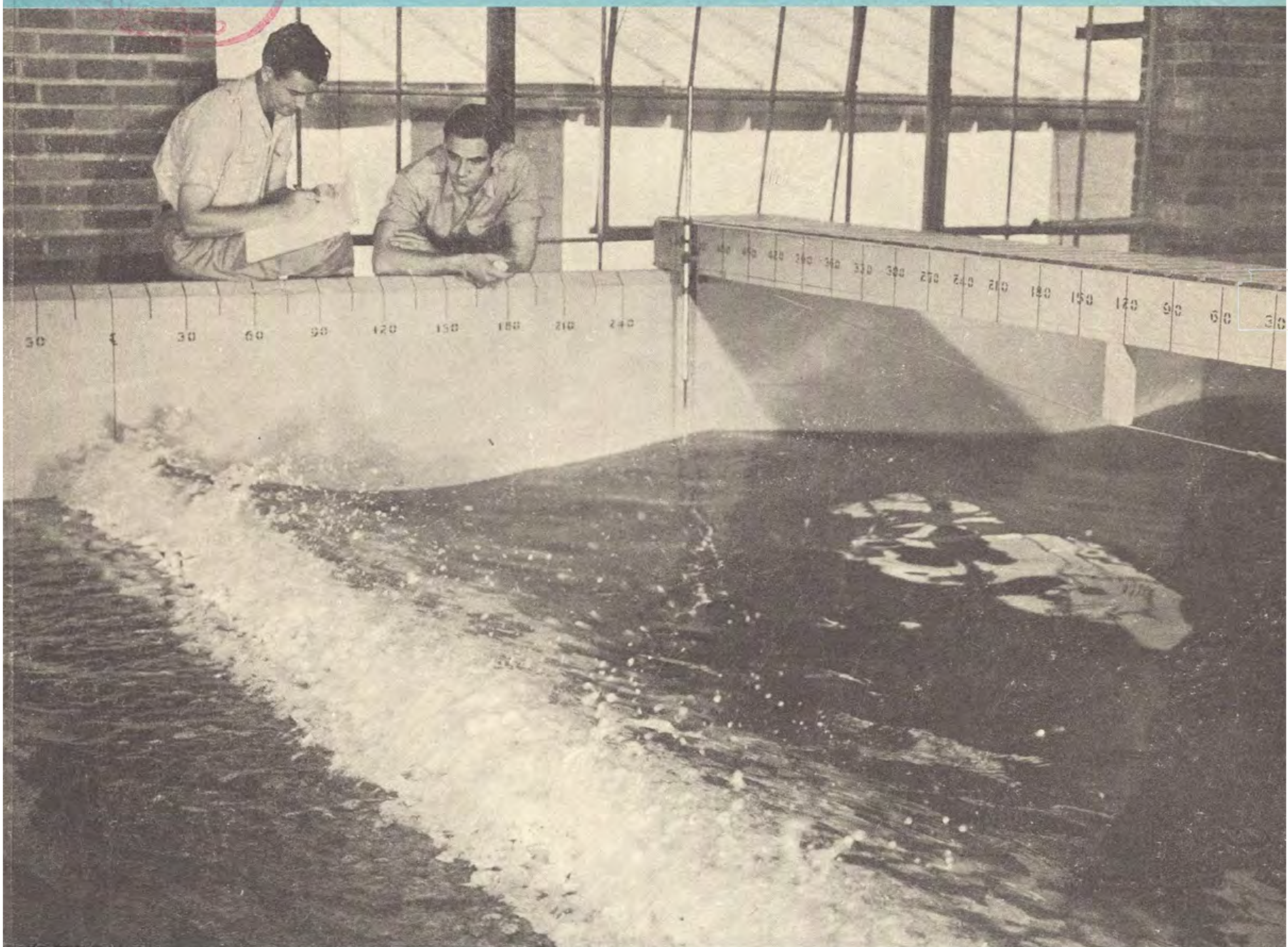


Figure B-5. A suggested grid cell system for the idealized inlet (seven channels and eight cross sections).

EMPIRICAL VERIFICATION OF TRANSFERENCE EQUATIONS IN LABORATORY STUDY OF BREAKWATER STABILITY



BULLETIN NO. 31

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

MISSISSIPPI RIVER COMMISSION

CORPS OF ENGINEERS

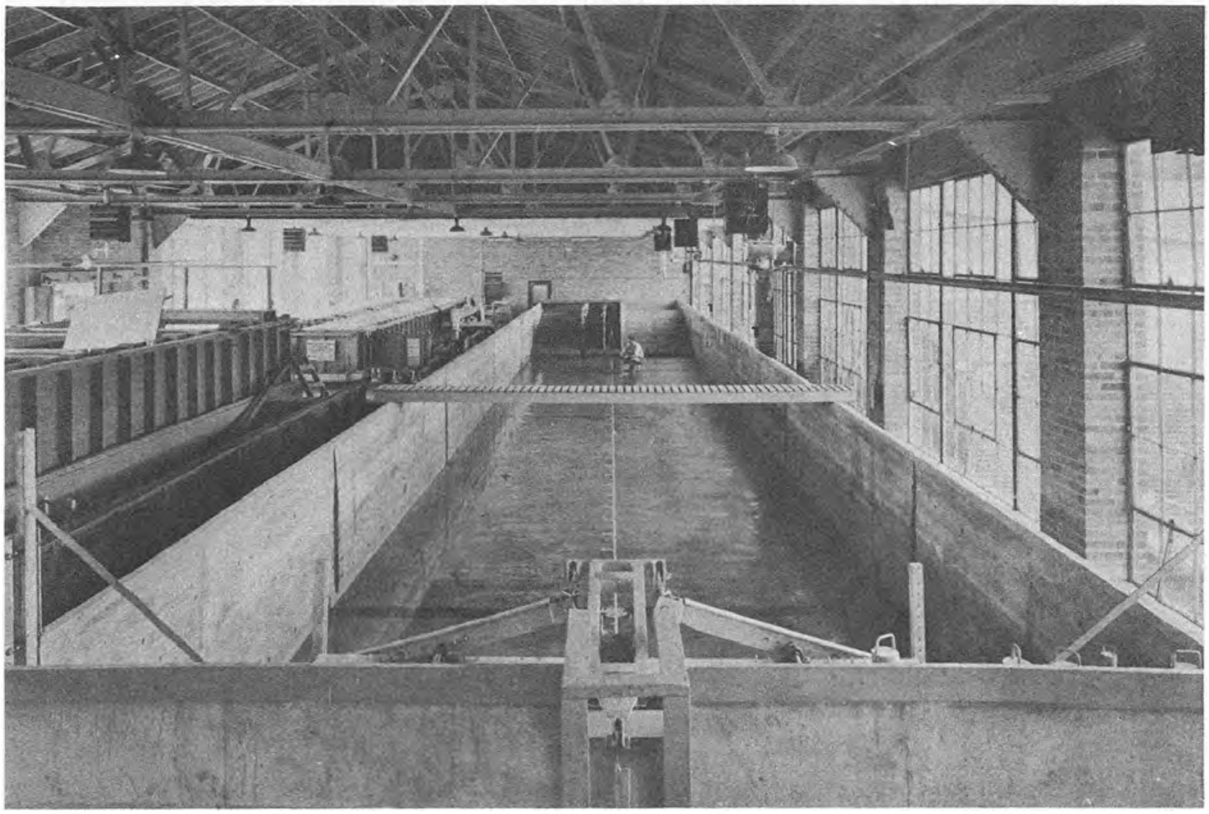


Fig. 1. The wave tank

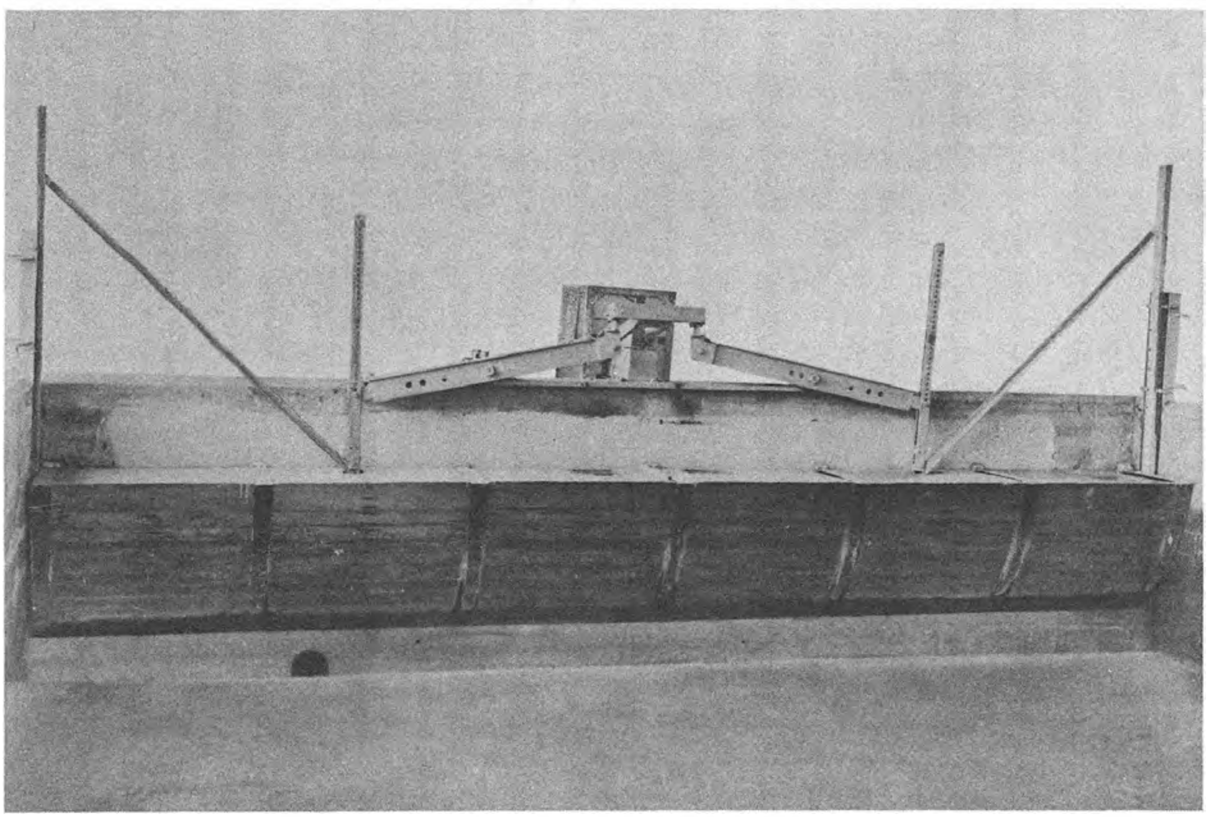


Fig. 2. The wave-generating device

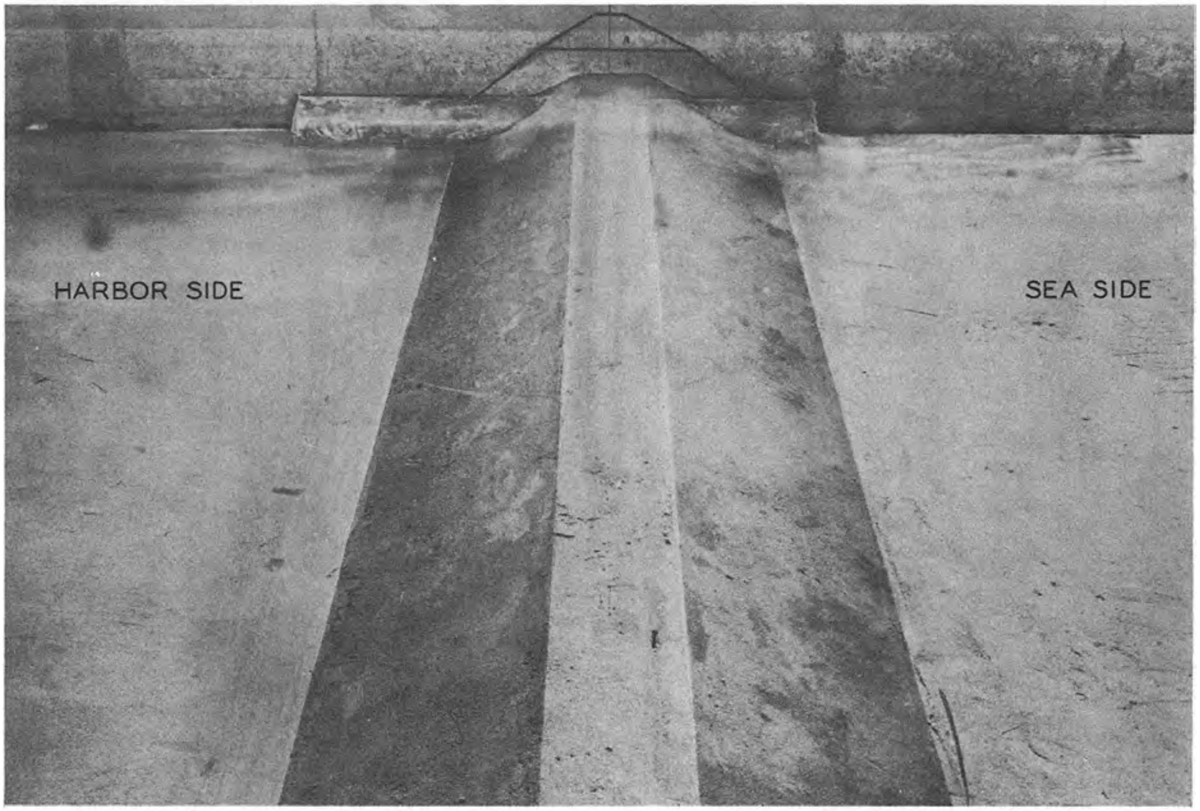


Fig. 4. Class C section before wave attack -- 1:45-scale model

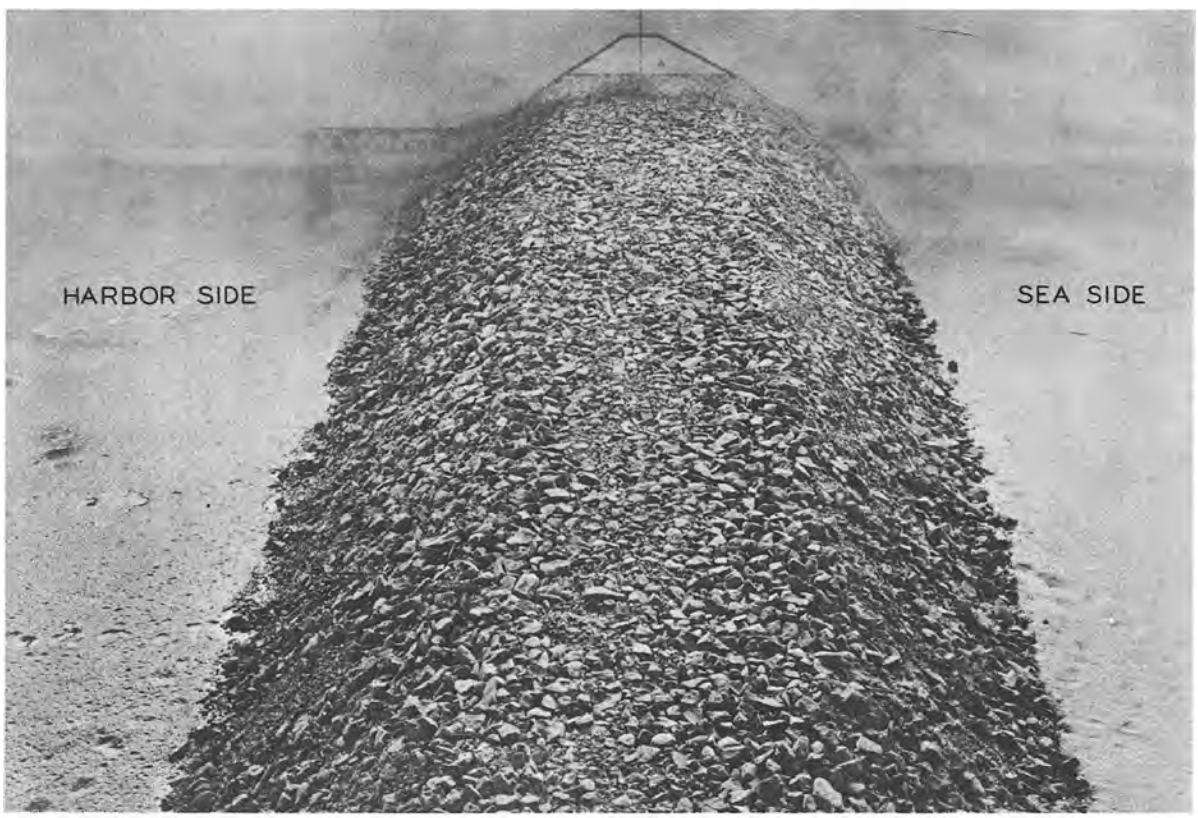


Fig. 5. Class B section before wave attack -- 1:45-scale model

Fig. 6. Complete breakwater section before wave attack -- 1:45-scale model

the total time and effort required for the tests. From previous tests it was known that the greater part of the damage experienced by the model breakwaters occurred early in each test and that thereafter erosion and displacement were comparatively slow. Therefore, ascertaining the precise point of stability for a given model, in the range where the movement of breakwater materials had become quite gradual, was a tedious and time-consuming procedure which necessitated repeated shutdowns for breakwater surveys after brief periods of progressive operation. Much of this delay was eliminated by following the simple procedure of determining the point of stability for one model selected as a pilot and running the other two models for equivalent periods of time as determined by the appropriate model scale. However, as a precaution to ensure the reliability of results, the model runs for these two models were extended beyond the computed equivalent times, and surveys were made both at the computed time and at the end of each test.

The 1:30-scale (or largest) model was selected as the pilot model for the study. On this model stability tests were performed for each of the breakwater sections that have been described using both 15- and 21-ft waves. The breakwater sections were considered stabilized by the forces of wave action when continued wave attack caused no further displacement of material other than such slight movement as must inevitably persist until all wave action ceased. Progress in the displacement of material was followed throughout the tests by frequently cross-sectioning the model breakwater. Cross sections were taken at 0.5-ft (model) stations along the middle 10-ft portion of the model, the extremities of the model adjacent to the walls being excluded because of the unnatural effects of the tank walls thereon. The

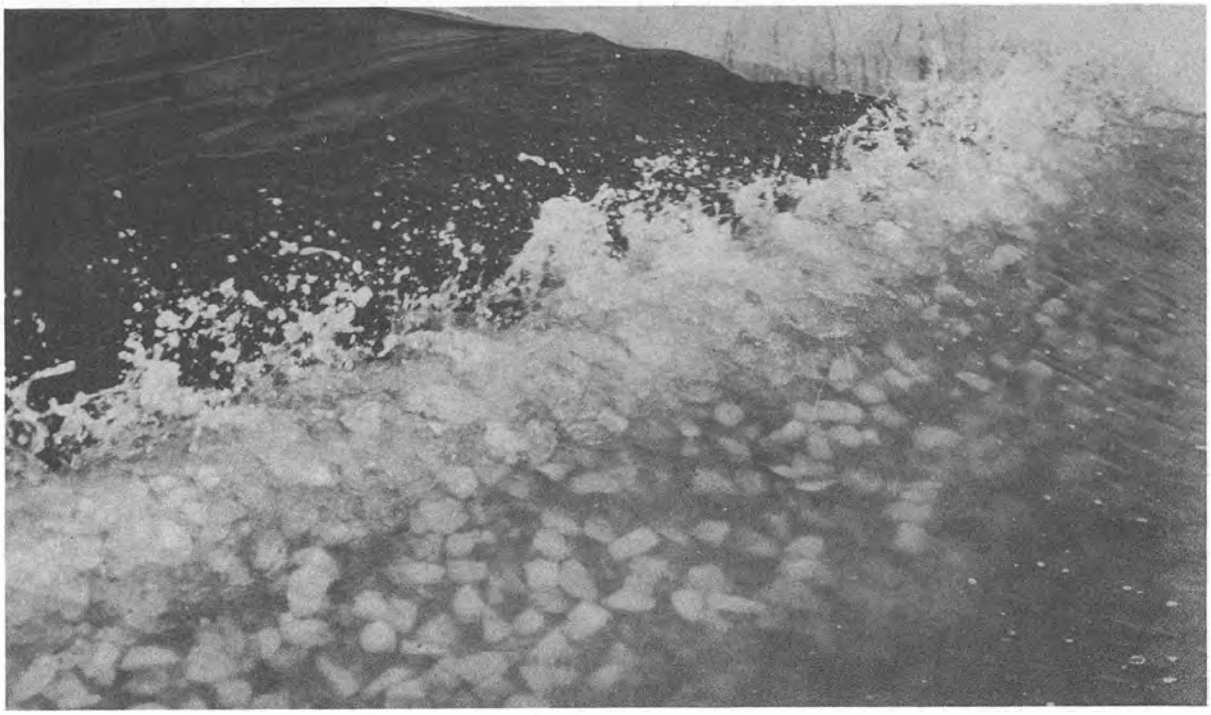
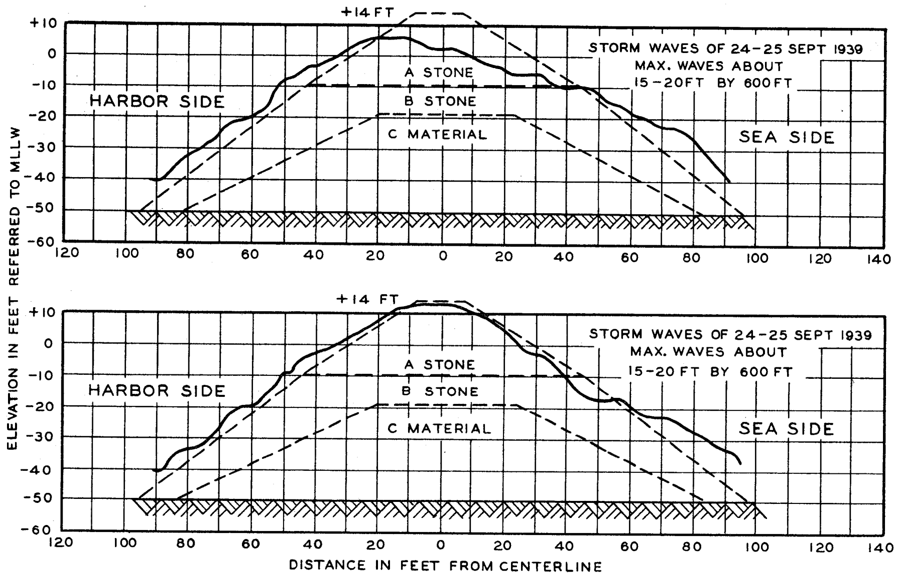
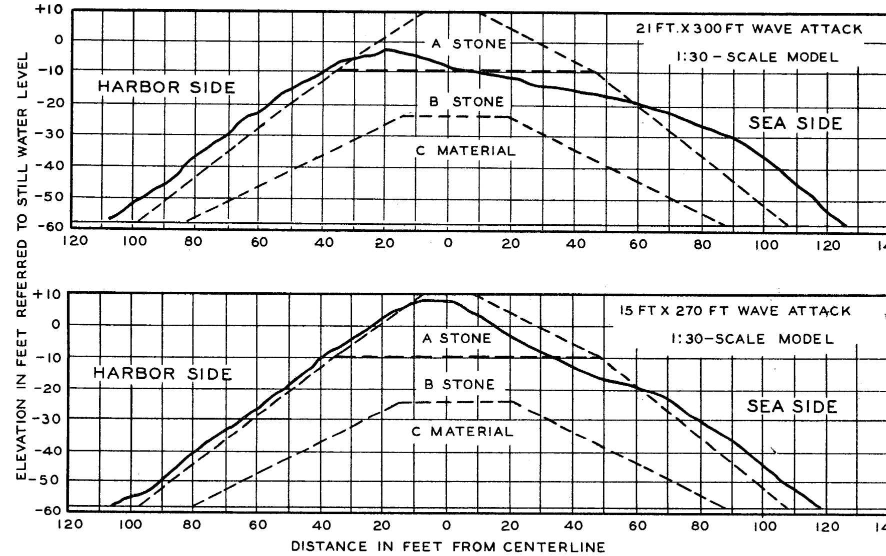


Fig. 7. A 21-ft wave breaking on the complete breakwater -- scale 1:30

During the course of the tests it was observed that the points where wave deformation began and where complete breaking occurred differed for the three sections tested and that within the limitations of visual observation these points were at about the same locations for both 15- and 21-ft waves. In prototype dimensions (the prototype sections are shown to scale in Fig. 3), observation showed that for tests of the Class C section (crown elevation at -24 ft swl) wave deformation began on the outer slope of the breakwater about 15 ft seaward of the center line, and complete breaking occurred on the inner slope about 45 ft harborward of the center line. For the Class B section (crown elevation at -10 ft swl) corresponding points were about 60 ft seaward and 45 ft harborward of the center line. These data show that these partial breakwater sections did not suffer the full force of breaking waves. The uniform shape and comparatively smooth surface of the partial sections after attack substantiate this observation. Fig. 8, 9, and 10 show the effects of 15-ft wave attack on the 1:60-scale models. Effects of the 21-ft waves were similar, but somewhat more pronounced. As can be seen in Fig. 8 and 9, the action of the waves caused a general flattening of the partial structures, with both landward and seaward displacement of material, but with somewhat greater deposition of material on the landward side in the direction of wave propagation. In tests of the complete, Class A section (crown elevation at +10 ft swl) wave deformation began on the outer slope of the breakwater about 75 ft seaward of the center line, and complete breaking occurred near the crown of the breakwater about 15 ft seaward of the center line. Appreciable damage resulted (see Fig. 10). Cap stones were toppled from the crown and dragged seaward, almost completely covering the Class B enrockment on the



MAXIMUM AND MINIMUM STORM DAMAGE TO DETACHED BREAKWATER, SAN PEDRO BAY, CALIFORNIA



DAMAGE TO MODEL BREAKWATER WITH WAVE ATTACK UNTIL MOVEMENT CEASED

SPECIFICATIONS OF BREAKWATER MATERIALS

SAN PEDRO BREAKWATER

CLASS A STONE. QUARRY RUN, WEIGHING NOT LESS THAN 160 LBS. PER CU. FT. (SOLID DRY), IN PIECES OF NOT LESS THAN ONE TON, OF WHICH NOT LESS THAN 50 PER CENT BY WEIGHT TO BE IN PIECES OF TEN TONS OR MORE EACH.

CLASS B STONE. QUARRY RUN, WEIGHING NOT LESS THAN 120 LBS. PER. CU. FT. (SOLID DRY), OF WHICH NOT MORE THAN 25 PER CENT BY WEIGHT TO BE IN PIECES OF LESS THAN 20 LBS. EACH, AND AT LEAST 40 PER CENT BY WEIGHT TO BE IN PIECES OF ONE TON OR MORE EACH.

CLASS C MATERIAL. A MOUND OF DREDGED SAND SURMOUNTED BY A MOUND OF DREDGED CLAY.

MODEL BREAKWATER

CLASS A STONE. SELECTED STONE FROM QUARRY (STONE FROM QUARRY UNDER CONSIDERATION WEIGHS 165-170 LBS. PER CU. FT.). NO PIECE TO WEIGH LESS THAN ONE TON, AND AT LEAST 75 PER CENT BY WEIGHT TO BE IN PIECES WEIGHING TEN TONS OR MORE EACH.

CLASS B STONE. QUARRY RUN, OF WHICH NOT MORE THAN 25 PER CENT BY WEIGHT TO BE IN PIECES OF LESS THAN 20 LBS., AND NOT LESS THAN 40 PER CENT BY WEIGHT TO BE IN PIECES OF ONE TON OR MORE EACH.

CLASS C MATERIAL. ALL RESIDUAL MATERIAL FROM QUARRY OPERATIONS, SIMILAR MATERIAL OBTAINED IN THE VICINITY OF THE QUARRY, OR DREDGED MATERIAL.

FIG. 14

COMPARISON OF KNOWN BREAKWATER DAMAGE WITH RESULT OF MODEL STABILITY TESTS

80WSE_329 "Empirical Verification of Transference Equations in Laboratory Study of Breakwater Stability," 1948

Fig. 7. A typical anchoring arrangement

pull out after a sampler drive has been started and partially completed. Whenever this occurs it is necessary to remove the sampler from the bore hole and clean out the boring in order to remove sand disturbed by the partial drive. A typical anchoring arrangement is shown by figure 7.

Drill rigs

The Waterways Experiment Station is presently using standard commercial-type truck-mounted rotary drill rigs in undisturbed sand sampling operations. The drill rigs use "N"-size drill rods and are equipped with oil-operated hydraulic drive systems capable of exerting a drive force in excess of 8000 lb. The length of drive is 2.5 ft. One of the drill rigs on a typical boring location is shown by figure 8.

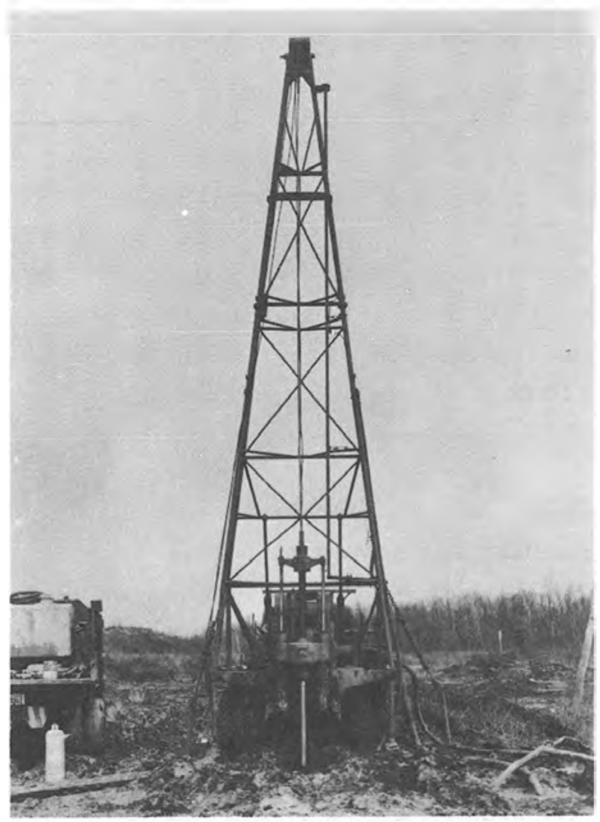


Fig. 8. Waterways Experiment Station drill rig on typical boring location

Drilling mud

The drilling mud is prepared on the job in sump pits by mixing approximately three parts of powdered bentonite and one part baroid by weight into the circulating water until the desired weight and consistency or viscosity are obtained. The preparation of the drilling mud can be facilitated by first filling the pit with water, then adding the baroid and bentonite through a mud gun and continuing circulation until a smooth texture is obtained. Figure 9 shows mud being results all samples should be drained before they are transported or any attempt is made to slice them or remove them from the sample tubes for density determination. The current procedures in use for draining, transporting, and determining density in the field and laboratory are discussed in the following paragraphs.

Draining samples. After the expanding packer has been placed in the bottom of the sample tube and the sampler head and piston removed, the tube is placed in a rack (figure 13) and allowed to drain. The sample tubes should be maintained in a vertical position from the time they are removed from the sampler until they are drained. Also, extreme care must be exercised to avoid disturbance of the sample by rough handling or jarring. The time required for proper drainage varies from 24 to 48 hours, depending on the amount and character of the fines in the sample. The sample never should be permitted to dry out under any circumstances, since it is impossible to slice satisfactorily a dry sand sample or to remove it from the tube in increments for density determinations.

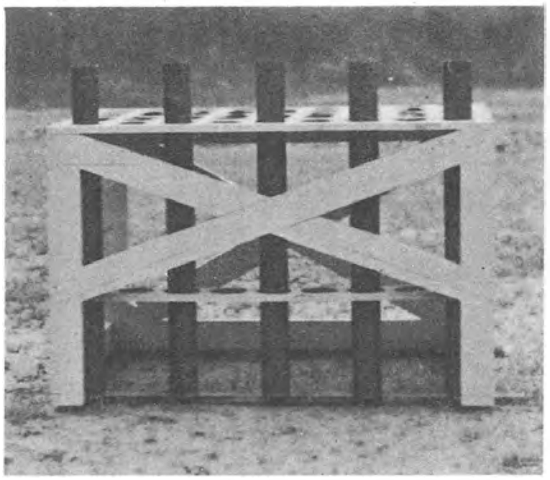


Fig. 13. Rack for draining samples

Transportation of samples. After the samples have been drained the drilling mud and all sand contaminated by drilling mud are removed from the top of each sample with the clean-out auger. Normally, the top 1 to 1-1/2 in. of each sample will contain drilling mud that has been mixed with sand in the bottom of the boring during previous clean-out operations. Mud penetration into the bottoms of the samples is usually less than 0.25 in. Expanding packers are then inserted in the tops of the sample tubes. The sample tubes containing the samples on which density determinations only are to be made are then placed in a horizontal position

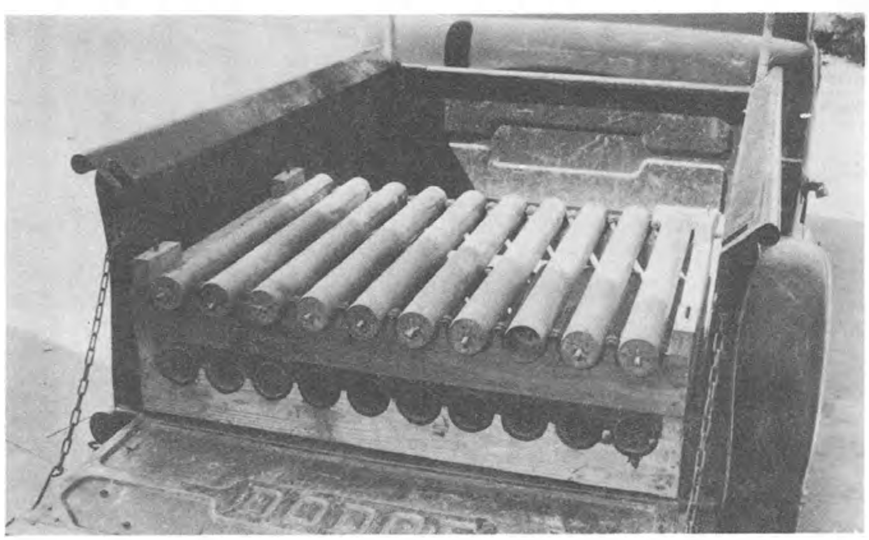
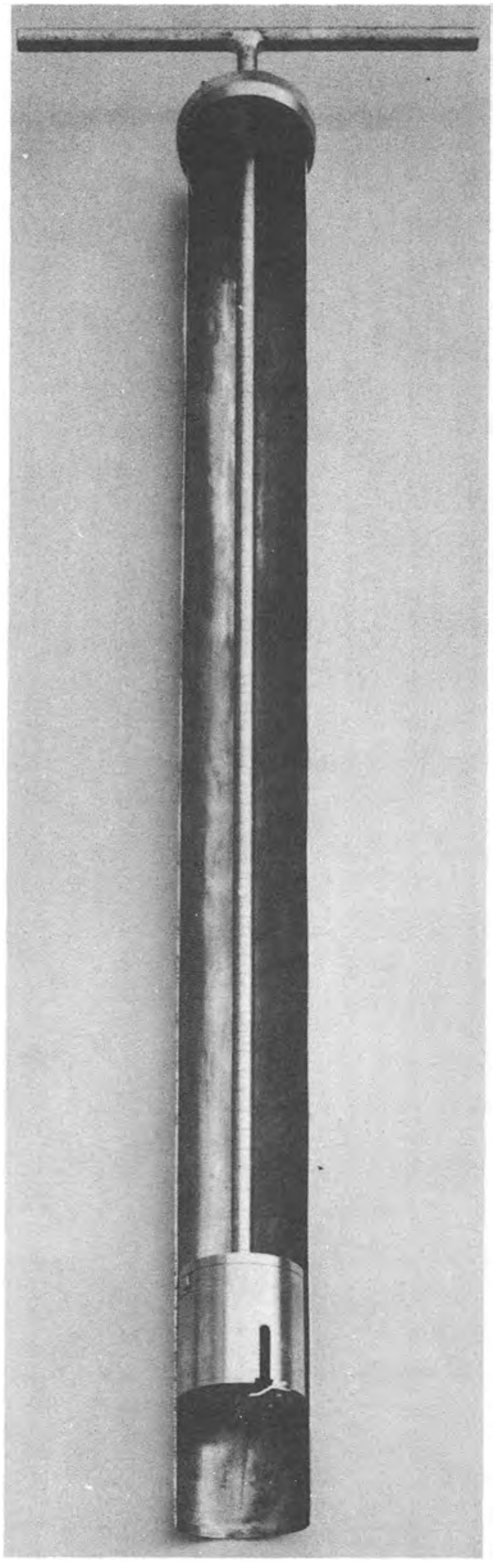
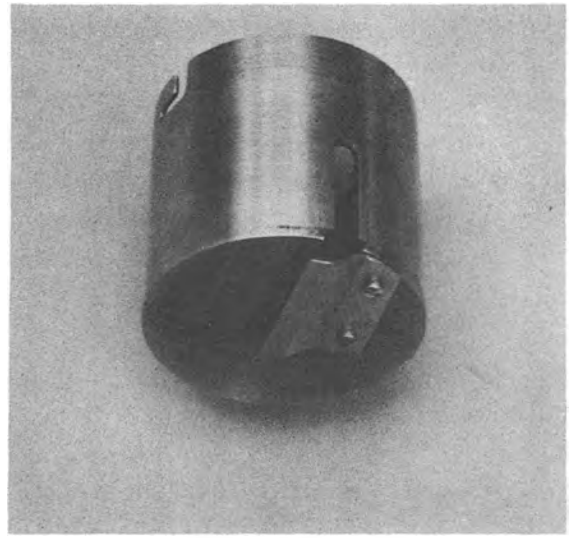


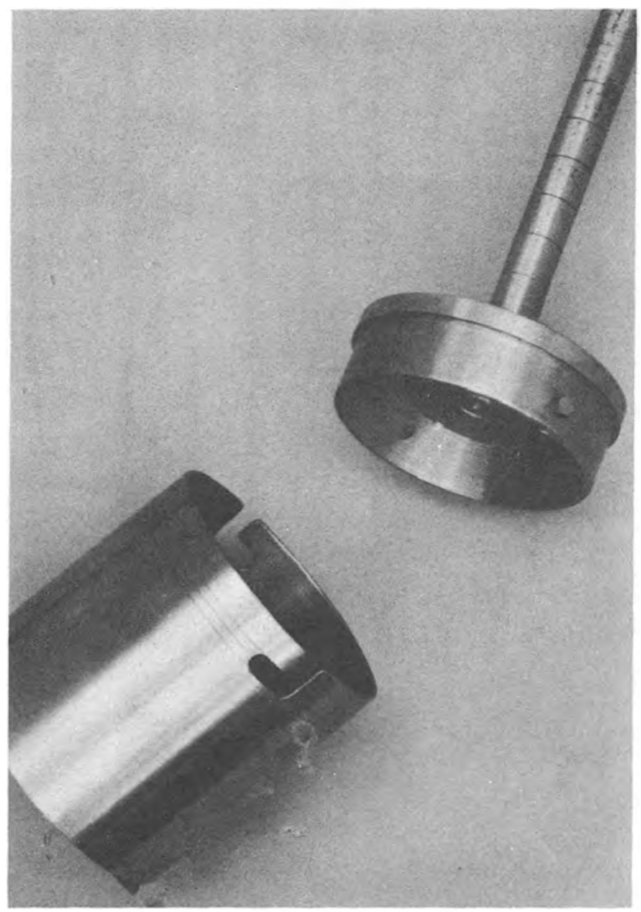
Fig. 14. Rack for transporting samples in horizontal position. Note packers in ends of tubes of top tier



CUP AUGER ASSEMBLY



AUGER CUP



AUGER CUP AND HEAD

Fig. 16. Cup auger

Fig. 18. 5-in.-diameter undisturbed sand sample from depth of 46.5 ft to 48.9 ft. Note discoloration at top of sample caused by intrusion of drilling mud

Fig. 19. 3-in.-diameter undisturbed sand sample from depth of 108.5 ft to 110.5 ft. Note that entire sample is highly stratified. Dark strata near bottom of sample contain lignite

Fig. 20. 3-in.-diameter undisturbed sand sample from depth of 96.5 ft to 99.0 ft. Note discoloration at top of sample caused by intrusion of drilling mud

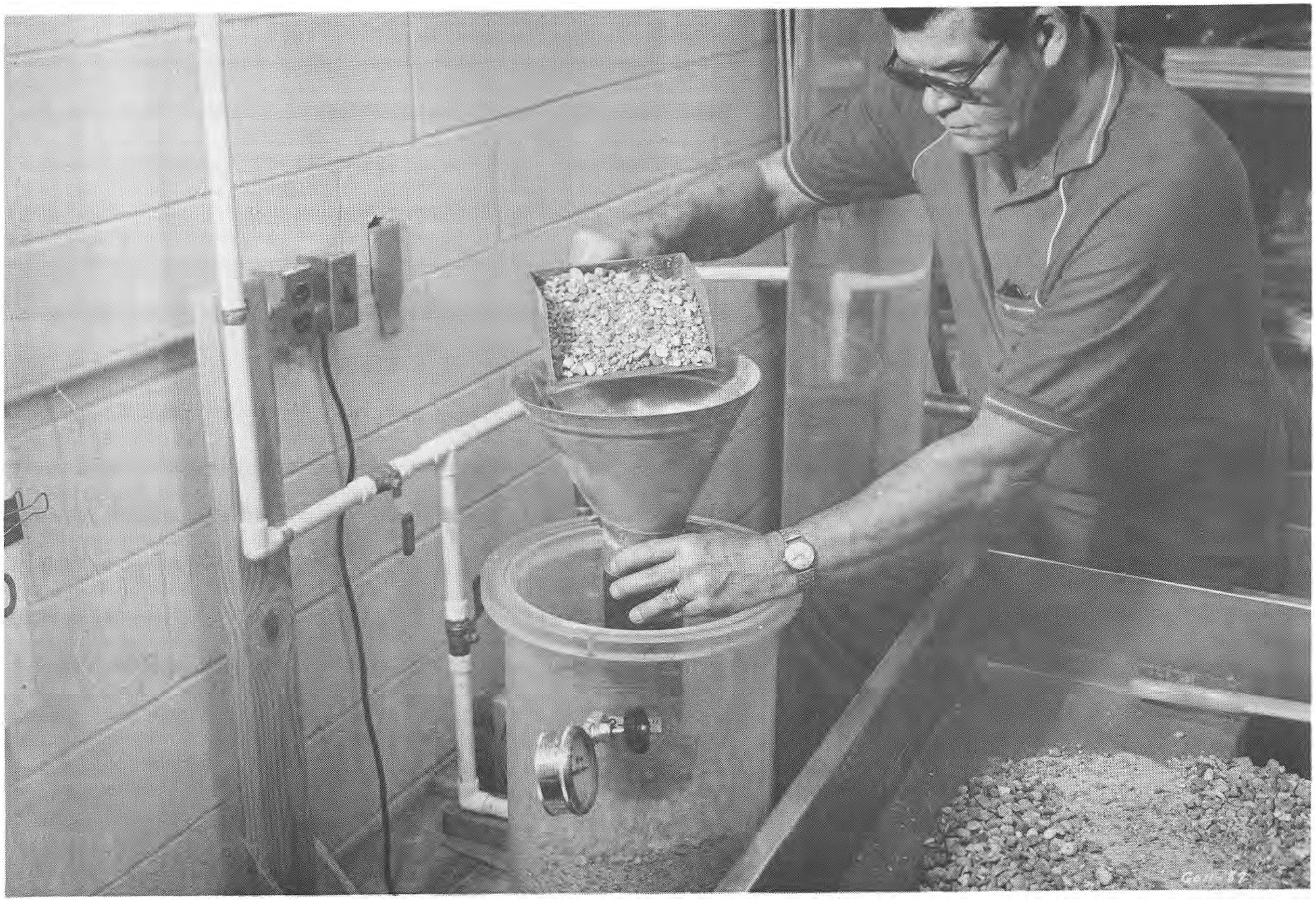


Figure 13. Placing filter material with funnel and open hose (Material shown is a poorly-graded sandy gravel used in Test No. 3A as discussed in Part IV.)

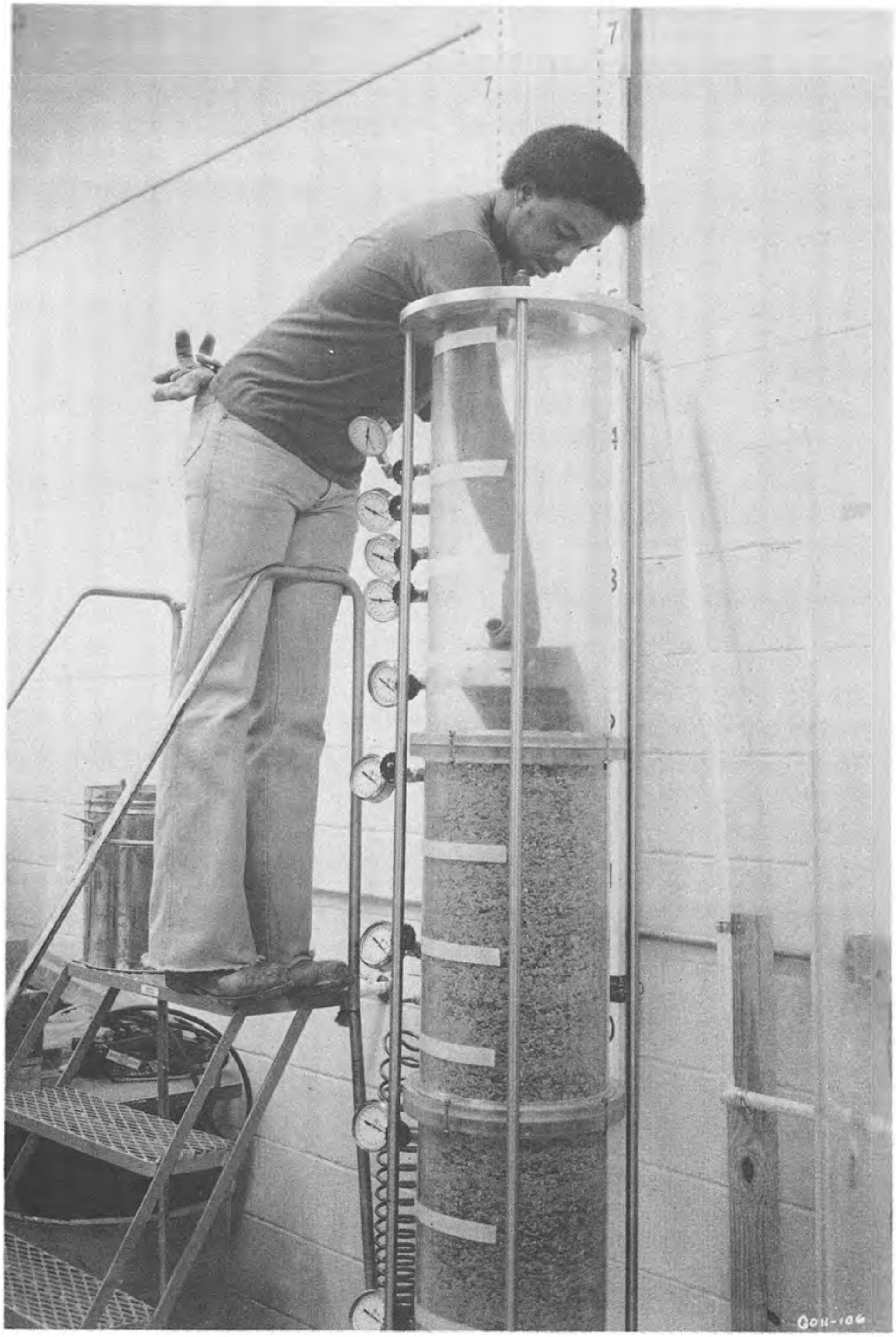


Figure 23. Removal of last increment of filter material during posttest test sampling (no base material for this test)

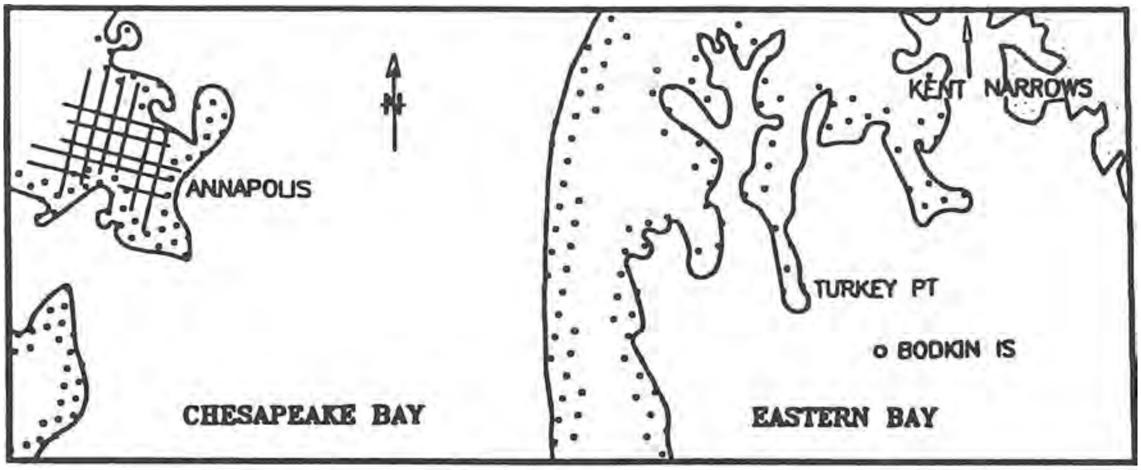


Figure 1. Location map

Figure 2. A view of Bodkin Island in January 1991, showing the bulkhead shoreline from the north, the standing vegetation, and the tall dead pine snag and osprey nest

80WSE_332 “Design of Habitat Restoration Using Dredged Material at Bodkin Island, Chesapeake Bay, Maryland,” 1992

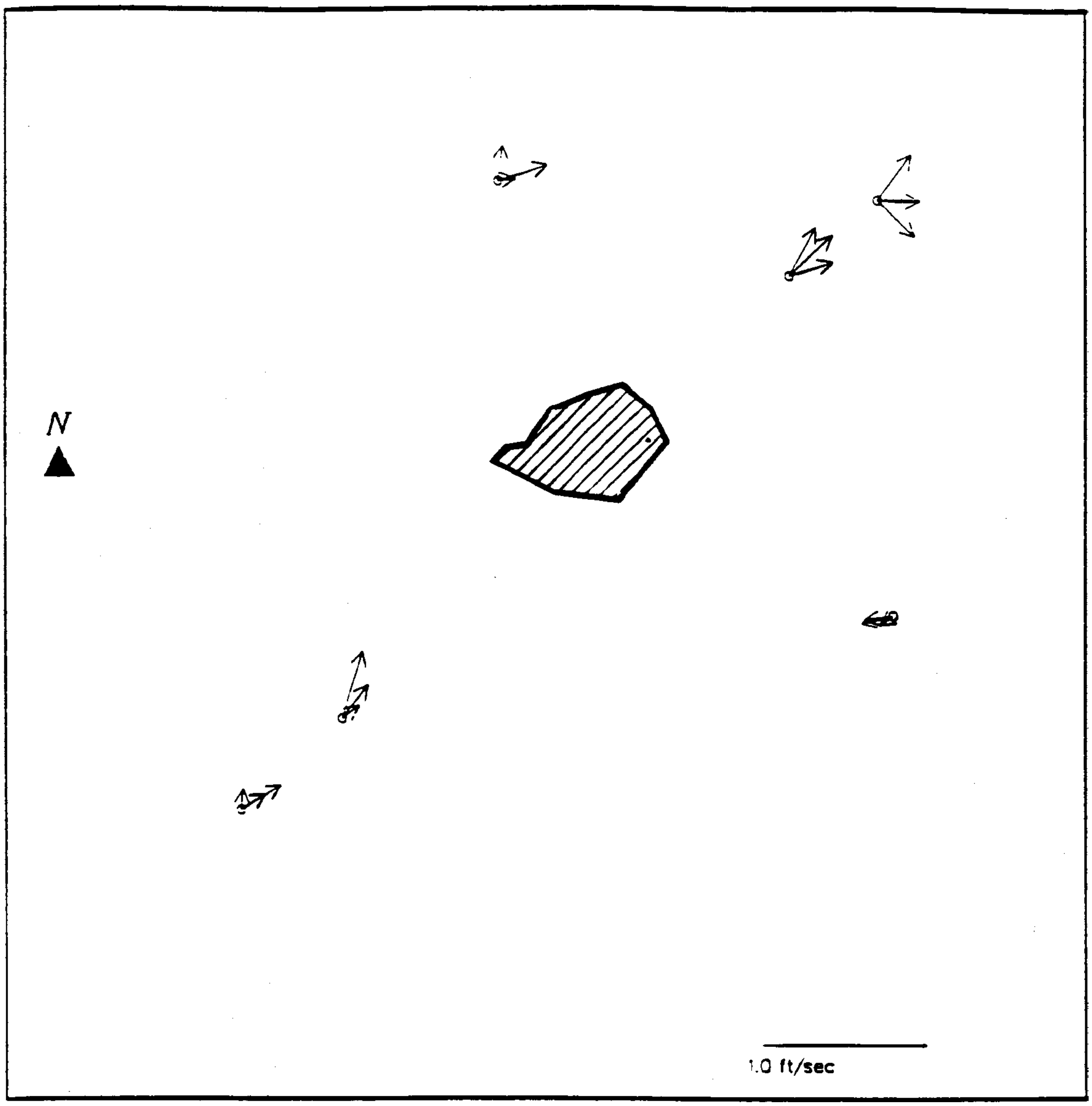


Figure 59. Velocity vectors, 17:00 to 17:20 EST, 8 June 1991

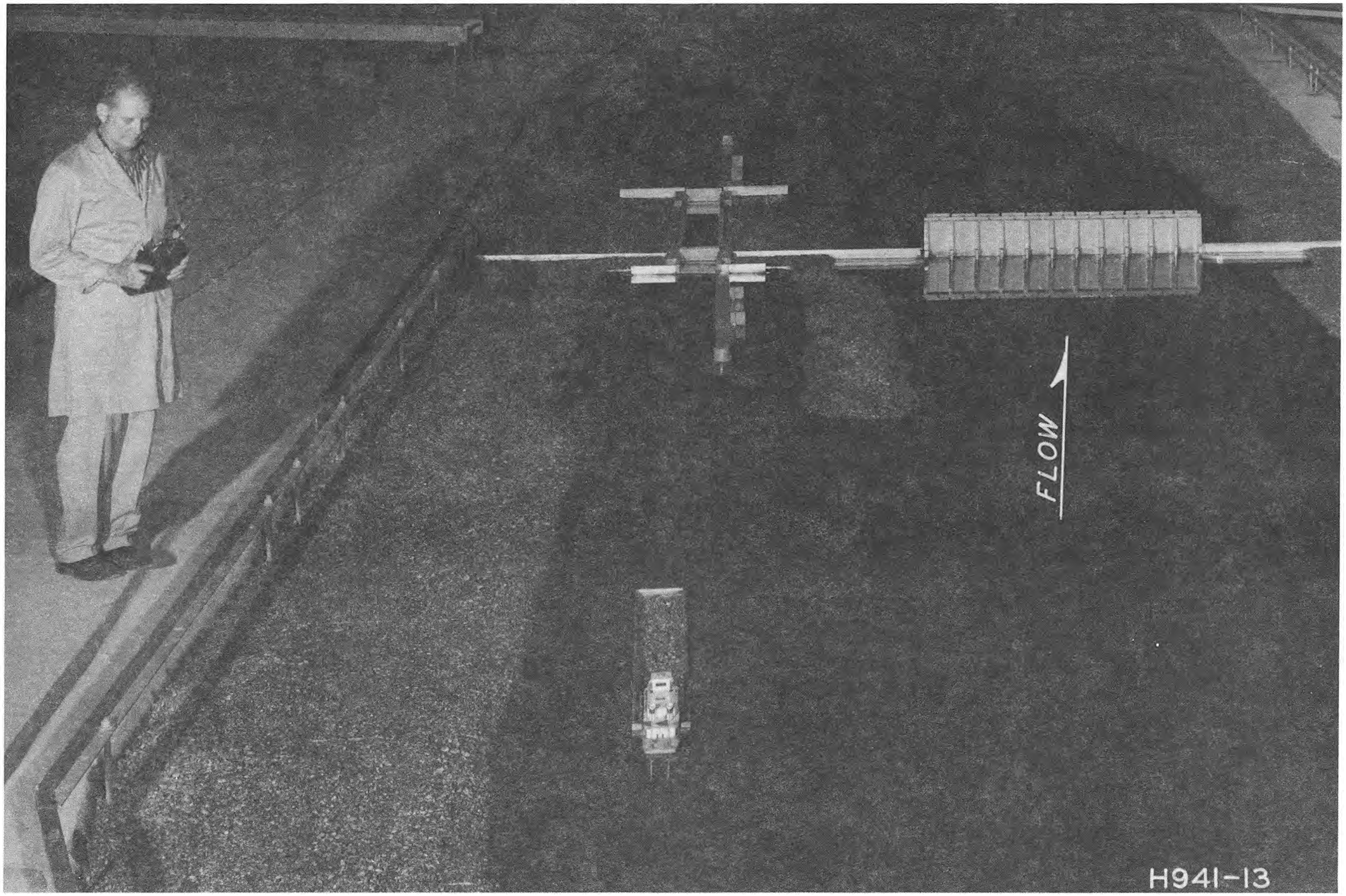


Figure 4. Remote-controlled towboat and tow in upper lock approach

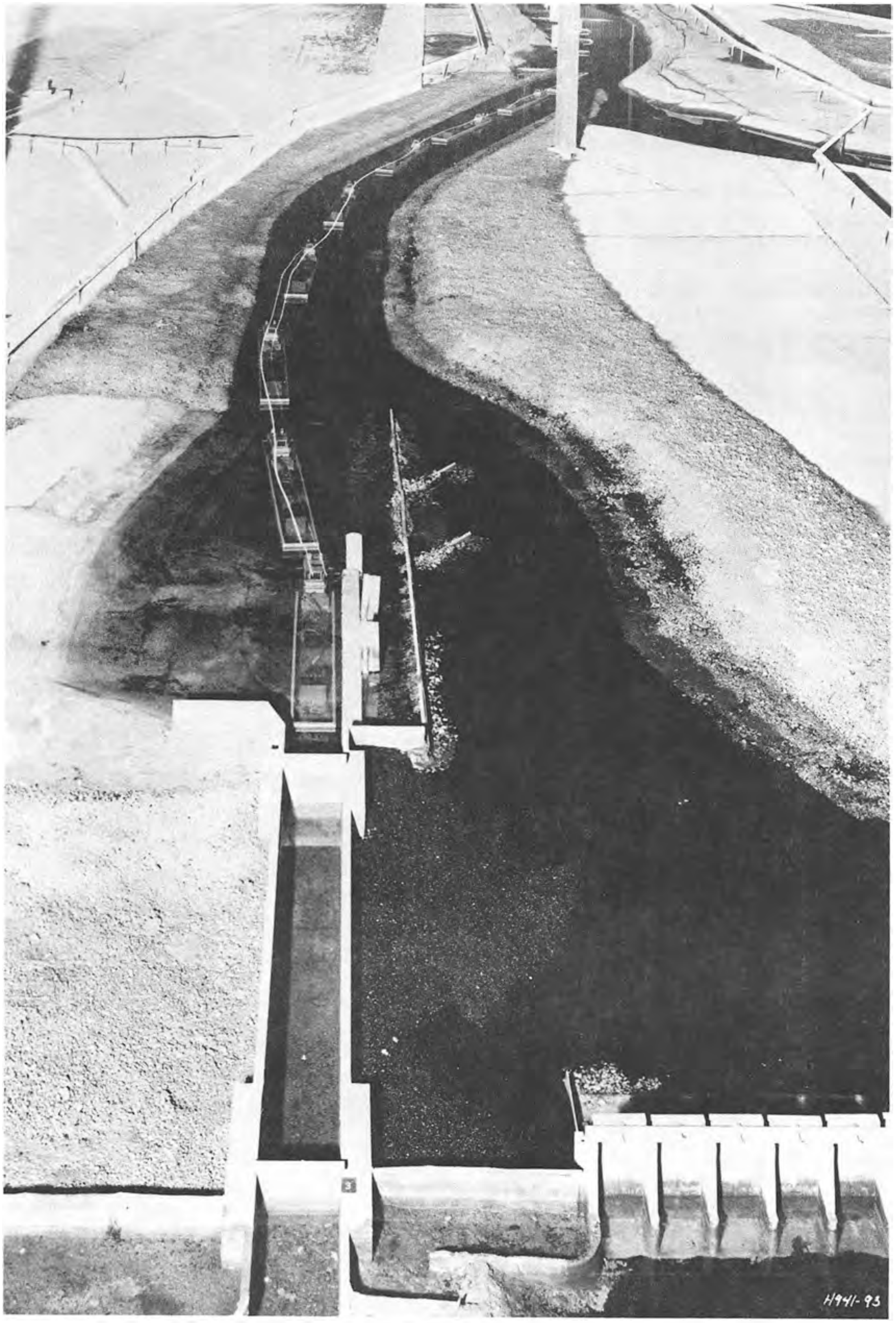
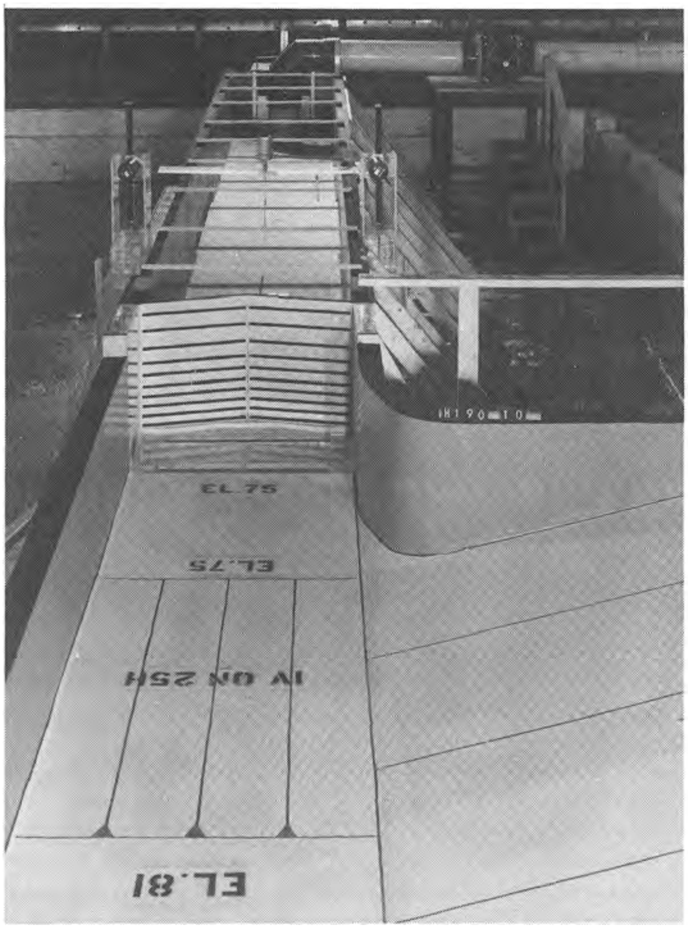
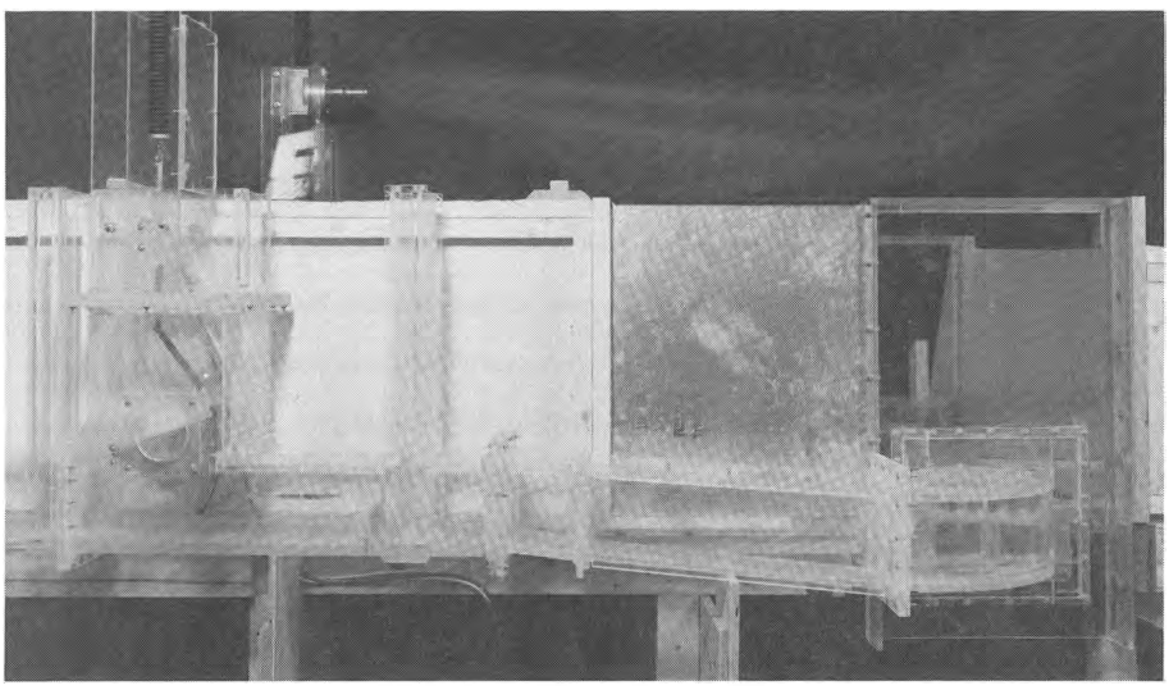


Photo 18. Plan B-35, upbound tow approaching the lock; discharge 31,000 cfs, tailwater el 21.0 (normal)



a. General view



b. Right emptying valve and culvert

Figure 3. The 1:25-scale model, type 1 (original) design

a. Normal valve

b. Single valve (left)

c. Single valve (right)

Photo 21. Maximum discharge and flow conditions for emptying operations with 4-min valve; type 13 design outlet area; initial head 3.5 ft; initial lock chamber el 121.7; lower pool el 118.2

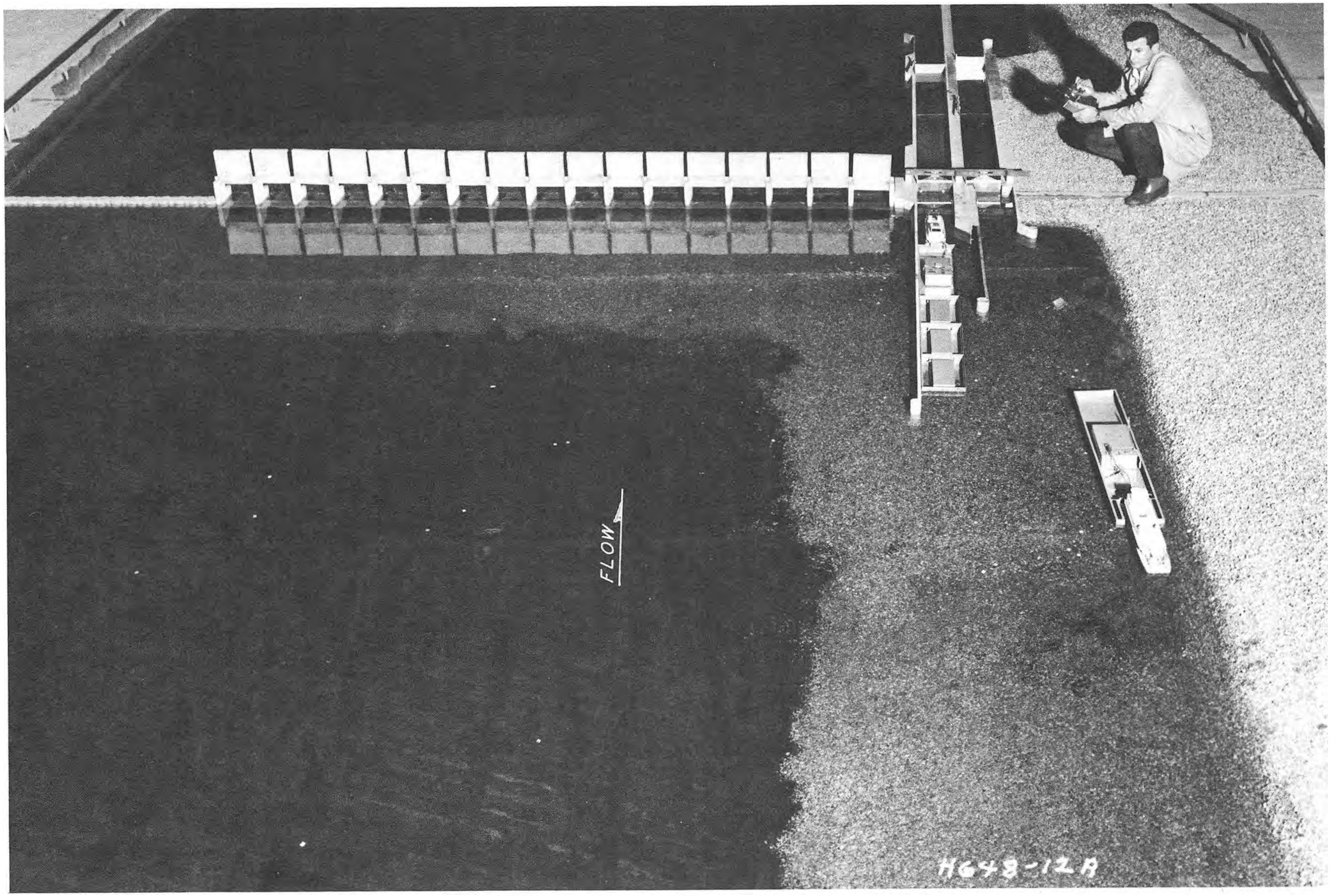
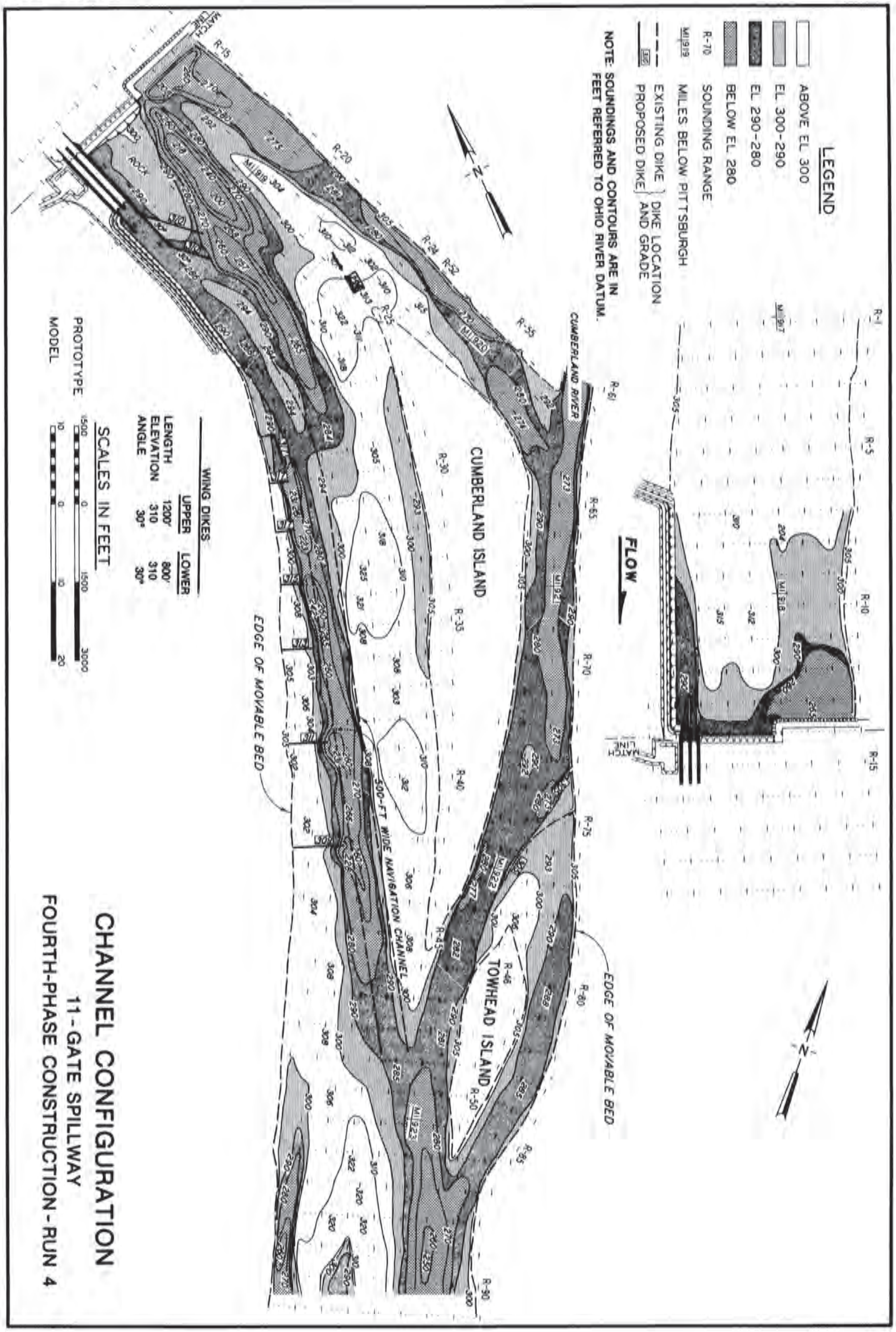


Figure 4. Model towboat and tow

PLATE 43



80WSE_335 “Smithland Locks and Dam, Ohio River: Hydraulic Model Investigation,” 1983

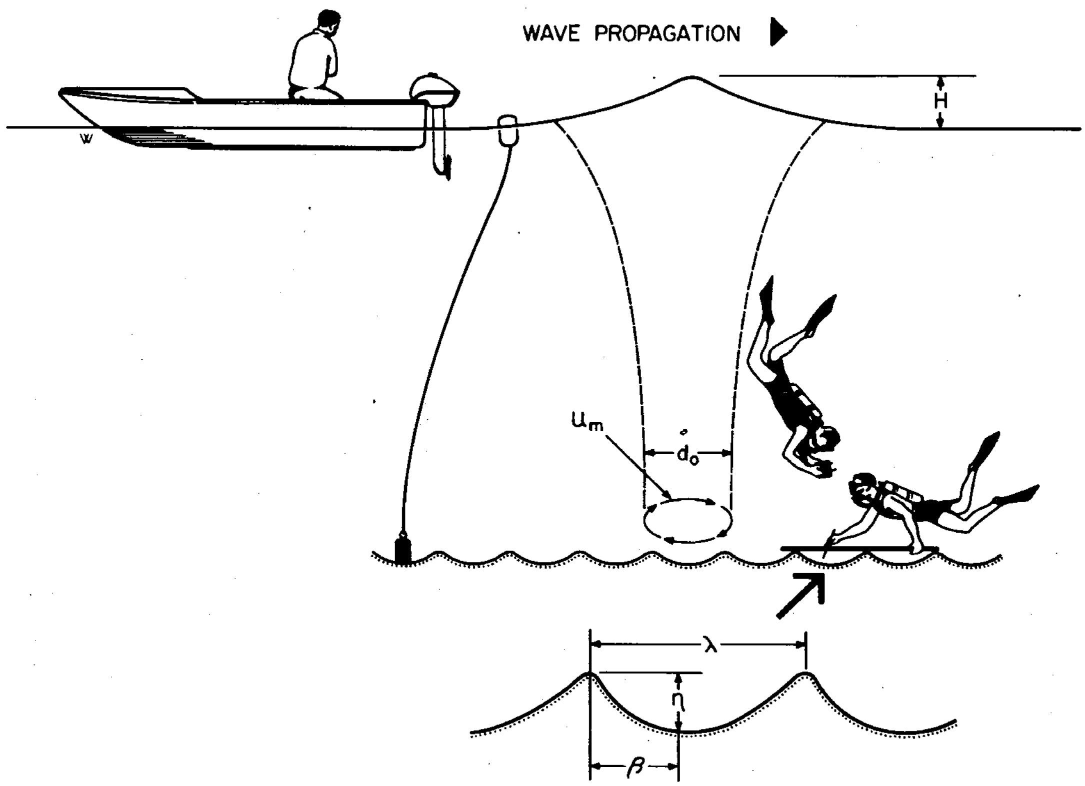


FIGURE I. SCHEMATIC DIAGRAM ILLUSTRATING THE TECHNIQUE USED IN MEASURING RIPPLES AND THE NOTATION FOR DESCRIBING WAVES AND RIPPLES. (u_m is the maximum horizontal orbital velocity of the waves, and d_o is the orbital diameter near the bottom. λ is the ripple wave length, η the ripple height, and λ/η and β/λ are the ripple index and measure of symmetry respectively.)

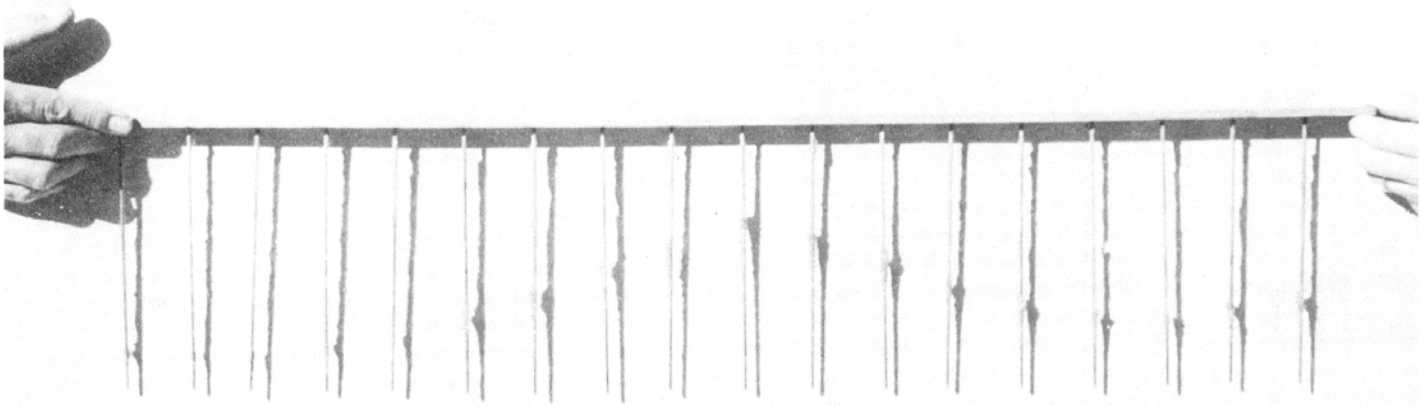


FIGURE 3. DEVICE USED FOR MEASURING RIPPLE PROFILE. (The prongs were coated with grease so that when forced into the sand bottom the imprint of the ripple profile is clearly marked in the grease. A spirit level (not shown) was attached to the bar to give a level datum. Typical profiles are shown in figure 4.)

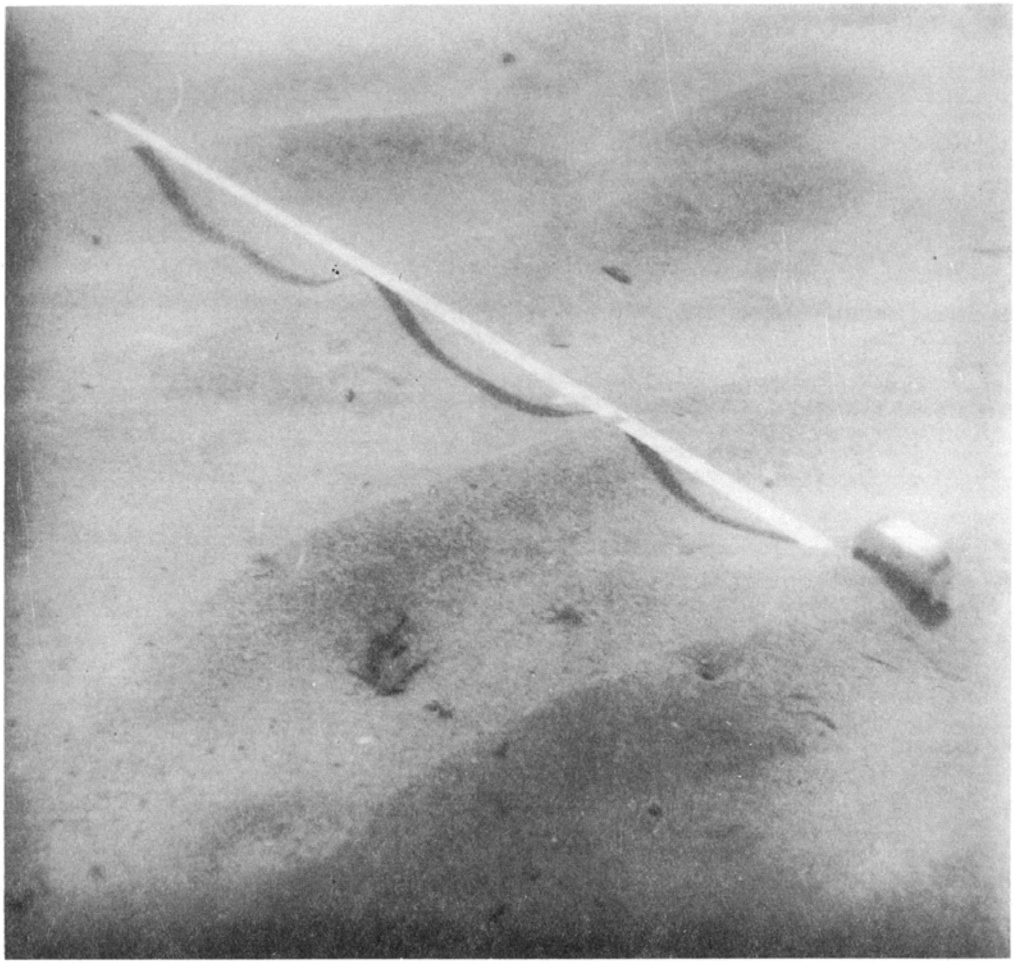


FIGURE 5. TROCHOIDAL RIPPLE PROFILE FROM THE PROTECTED SHELF OFF SAN CLEMENTE ISLAND, CALIFORNIA. Ripple wave length 0.67 ft., depth of water 21 ft. Sand on the ripple crest is slightly coarser than that in the trough. Complete description tabulated in Appendix III B.

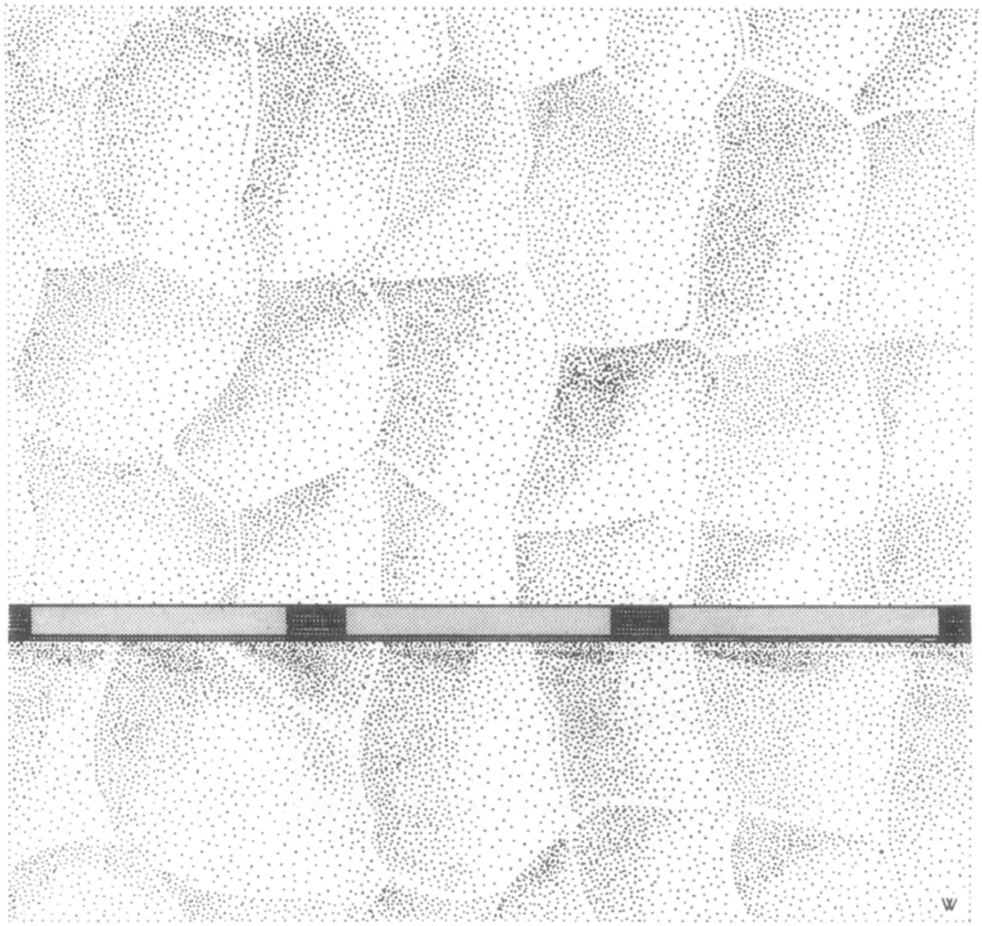


FIGURE 8. SHORT CRESTED RIPPLES (PATTERN 3). Ripples in fine sand at a depth of 52 ft. on D range. (Appendix I C, 23 November 1953). Ripple profile is trochoidal. Scale division 0.5 ft. (Drawing of a photograph).

Before and After Views of Shore Development, Prospect Beach, West Haven, Connecticut, 1956 and 1957

Before and After View of Beach Nourishment Project at Westhampton Beach, New York, 1969

The Castle Insigne was first worn by West Point Cadets in 1839.

This seal of the U.S. Engineer Department appears on drawings dated as early as 1815.

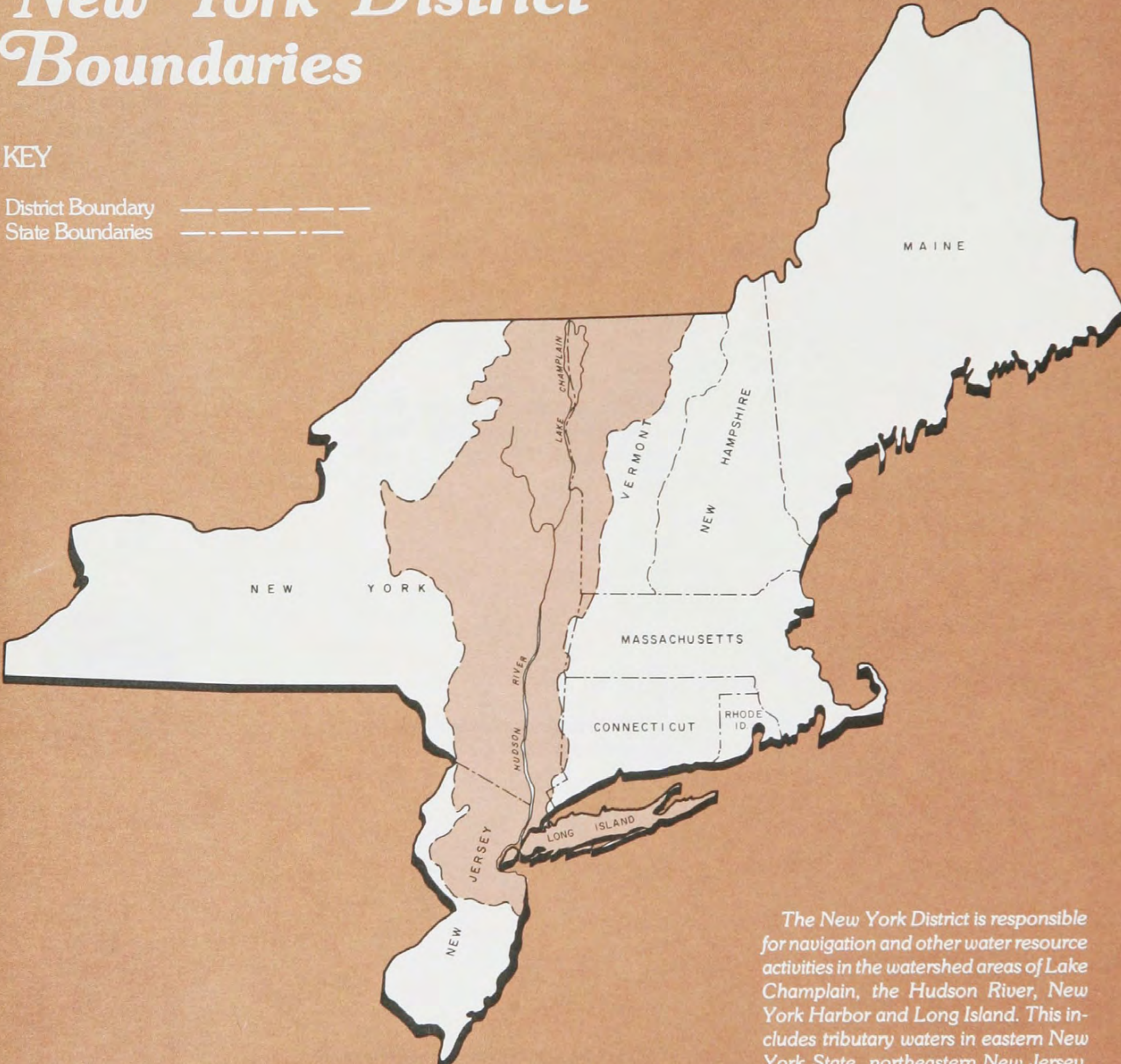
Chiefs of Engineers' letterheads bore this seal in the 1870's, 1880's.

The current seal, adopted in 1866 and made official by order of General Wilson in 1897.

New York District Boundaries

KEY

District Boundary — — — — —
State Boundaries —- —- —- -—



The New York District is responsible for navigation and other water resource activities in the watershed areas of Lake Champlain, the Hudson River, New York Harbor and Long Island. This includes tributary waters in eastern New York State, northeastern New Jersey, western Vermont and small portions of Massachusetts and Connecticut.

The District's military construction responsibilities include all of the New England states, New York, New Jersey and, outside of the U.S., Greenland, Labrador and the Azores.

Beach Erosion—Rockaway N.Y.—1973

Beginning near the westerly end of Long Island and proceeding easterly, the first project authorized in 1965 is from East Rockaway Inlet to Rockaway Inlet and Jamaica Bay, all in New York City, which provided combined protection through construction of an ocean floodwall and restoration of a protective beach along 6.2 miles of shore, construction of a gated hurricane surge barrier across the Rockaway Inlet entrance to Jamaica Bay along with connecting levees, dikes and other floodwalls. Construction of the beach erosion control portion was initiated in late 1974 after special authorization was made in the Water Resources Development Act of 1974 to construct this portion separately from the hurricane protection portion. The first increment of work for restoration of 3.0 miles of beach was completed in October 1975 with 3,800,000 cubic yards of sand fill being placed. A second increment of work to restore 1.2 miles of beach was scheduled for completion by November 1976.

Further eastward along the south shore from Fire Island Inlet to Jones Inlet, a beach erosion control project providing for the project shore by the establishment and maintenance of a feeder beach was authorized in 1958. By 1964, two increments of work were completed which resulted in a feeder beach being established at Cedar Beach, just west of the Inlet and a saw dike being constructed across the Oak Beach Channel with sand dredged from Fire Island Inlet. In 1965, plans for a project modification authorized in 1962 were approved by the President and Secretary of the Army. This modification combined the beach erosion control project with an existing navigation project at Fire Island Inlet and provided for bypassing sand across the Inlet to a feeder beach for shore nourishment. Construction of the modified project was initiated in 1973 to dredge a navigation channel and littoral reservoir and to establish an effective feeder beach.

Beach Restoration — Rockaway N.Y. — 1975

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

THE

BULLETIN

OF THE

BEACH EROSION BOARD

OFFICE, CHIEF OF ENGINEERS
WASHINGTON, D.C.

VOL. 11 JULY 1957 NO. 1

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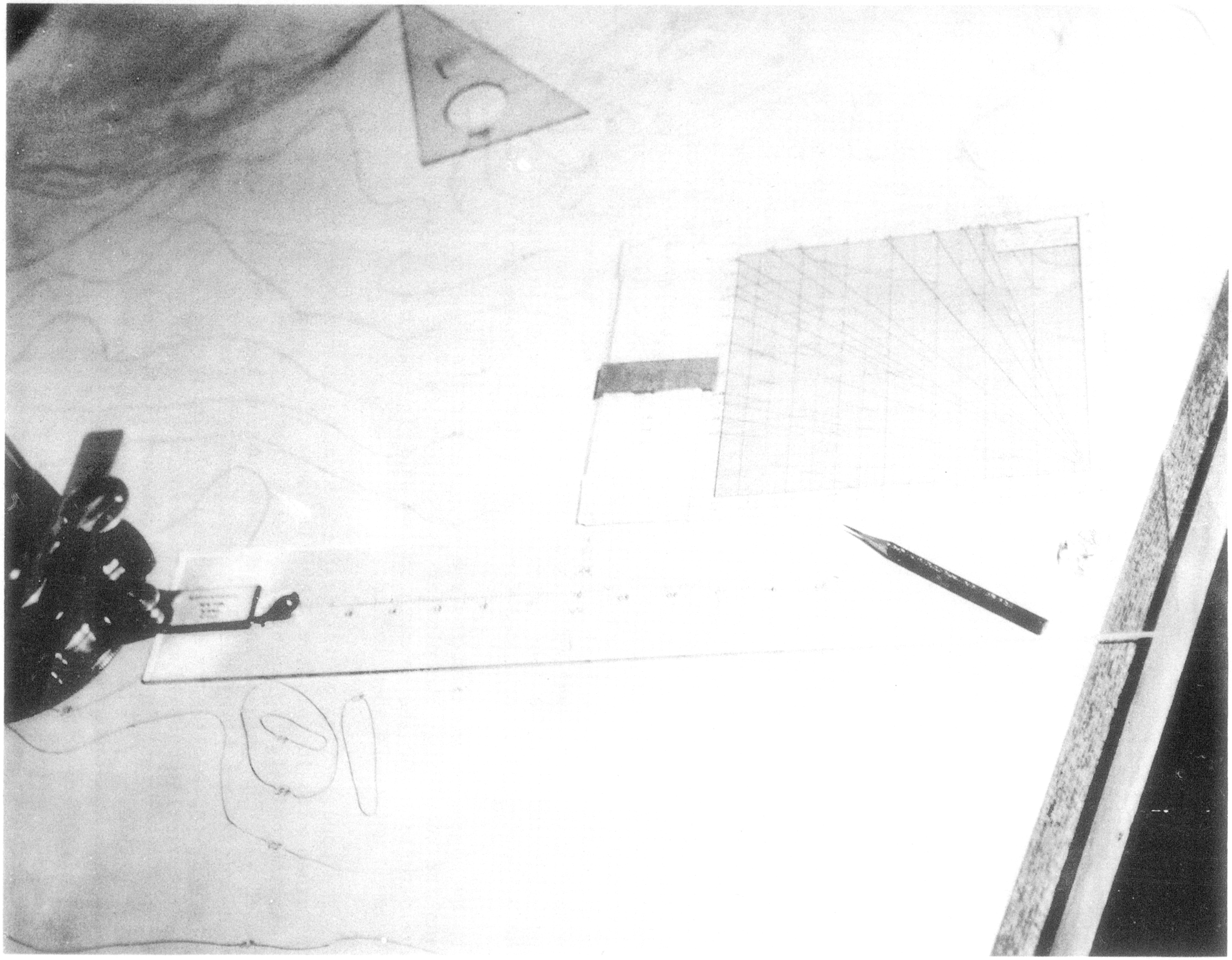


FIGURE I. WAVE REFRACTION PLOTTER ATTACHED TO DRAFTING MACHINE

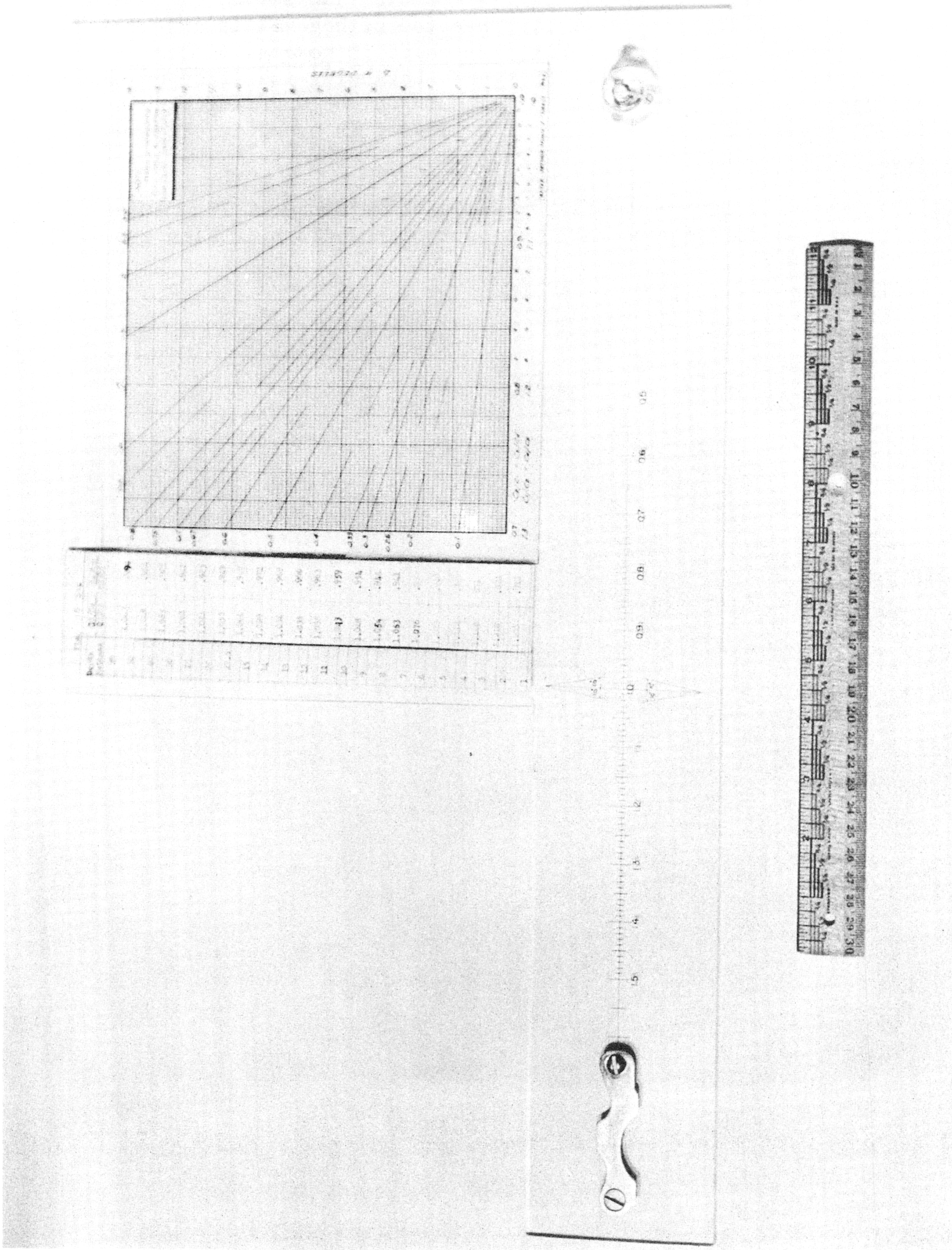


FIGURE 2. WAVE REFRACTION PLOTTER (CLOSE-UP)

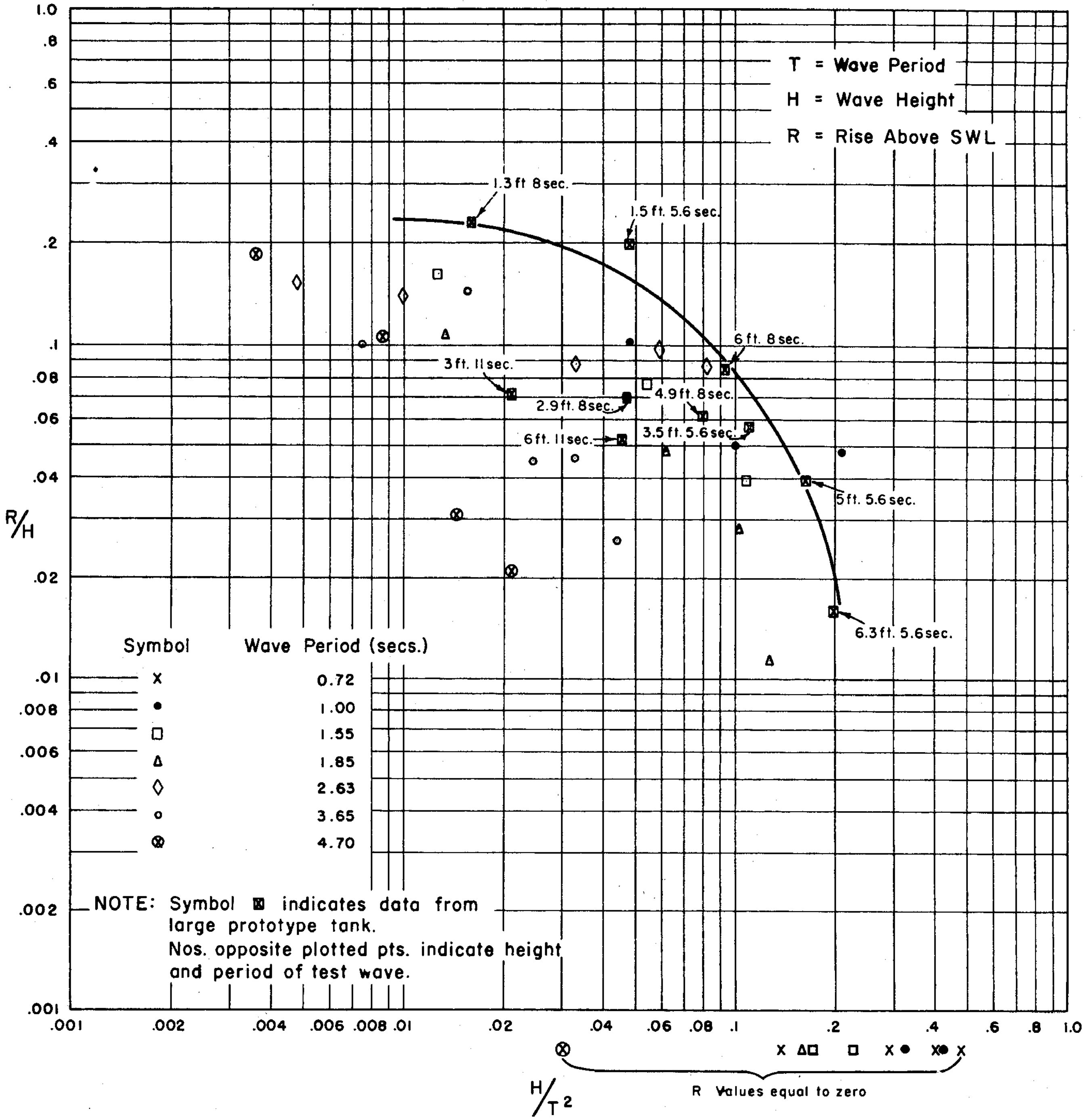
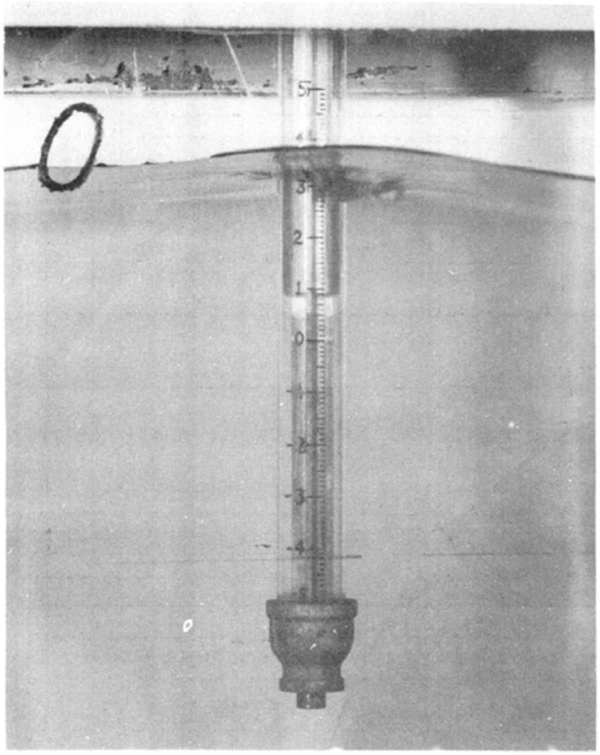
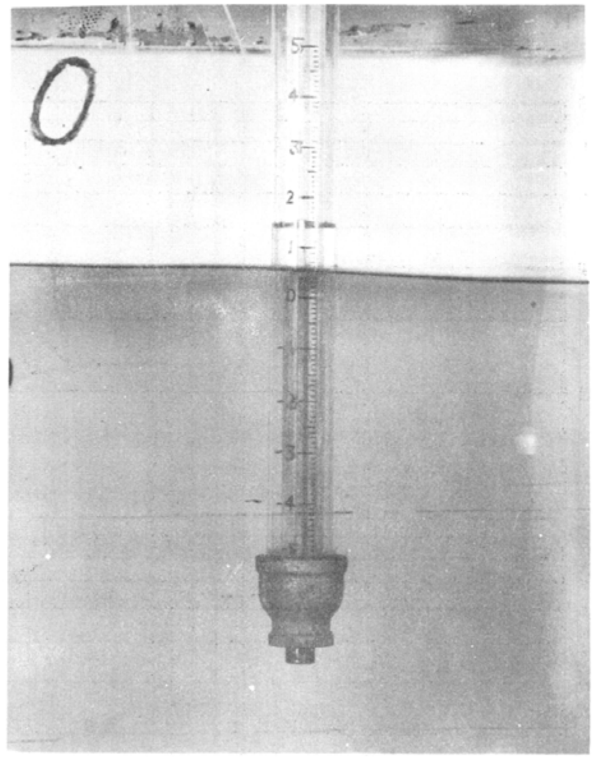


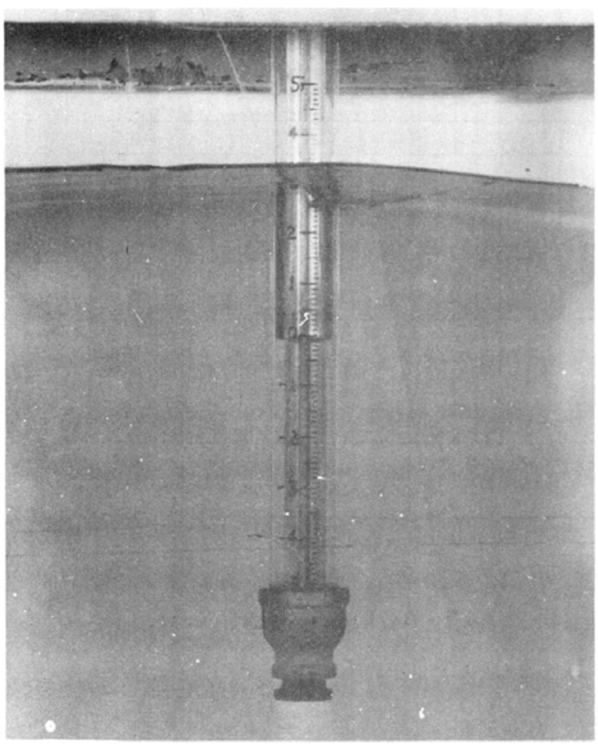
FIGURE 2. WATER LEVEL RISE DUE TO WAVE ACTION FOR CREST STAGE GAGE WITH REDUCED OPENING



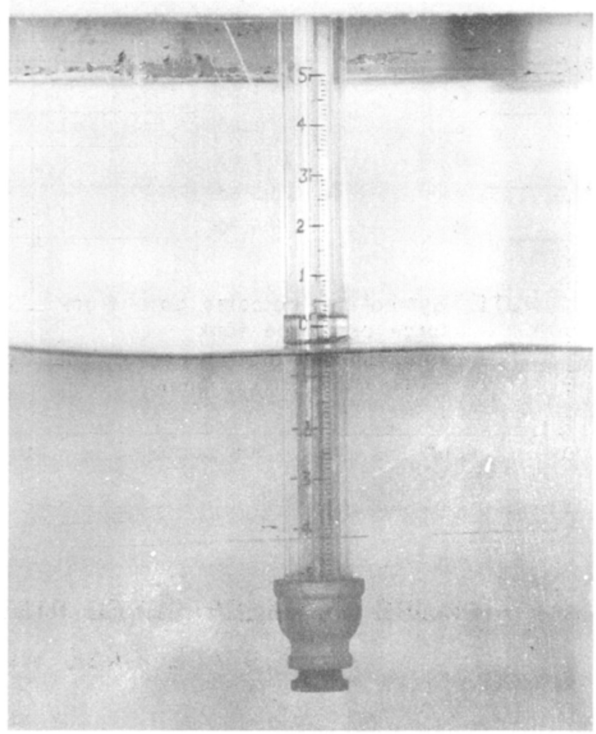
a. Wave Crest Passing (Regular Opening)



b. Maximum Water Level Rise (Regular Opening)



c. Wave Crest Passing (Reduced Opening)



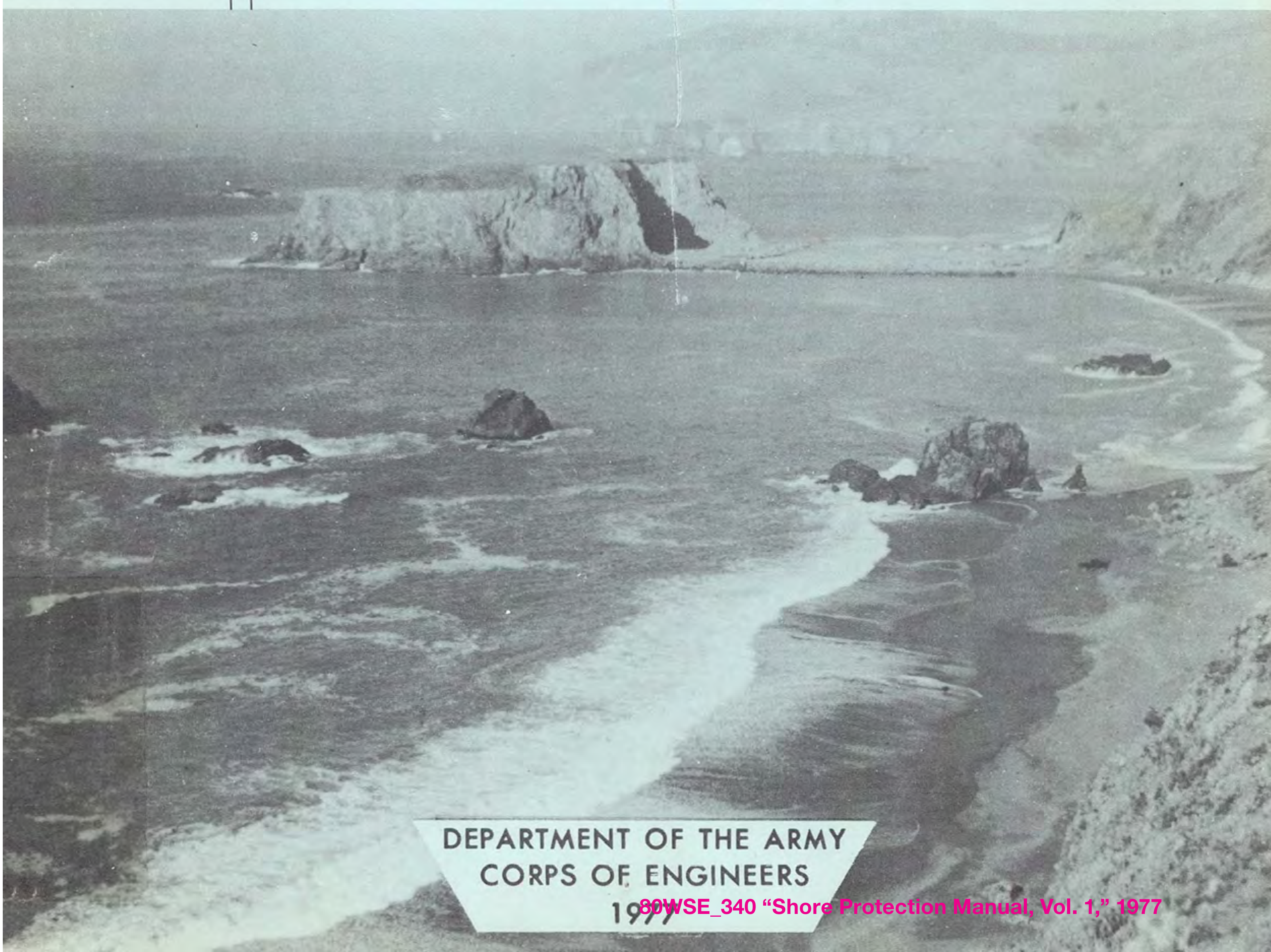
d. Maximum Water Level Rise (Reduced Opening)

FIGURE 3. PHOTOGRAPH OF CREST STAGE GAGE IN LABORATORY WAVE TANK

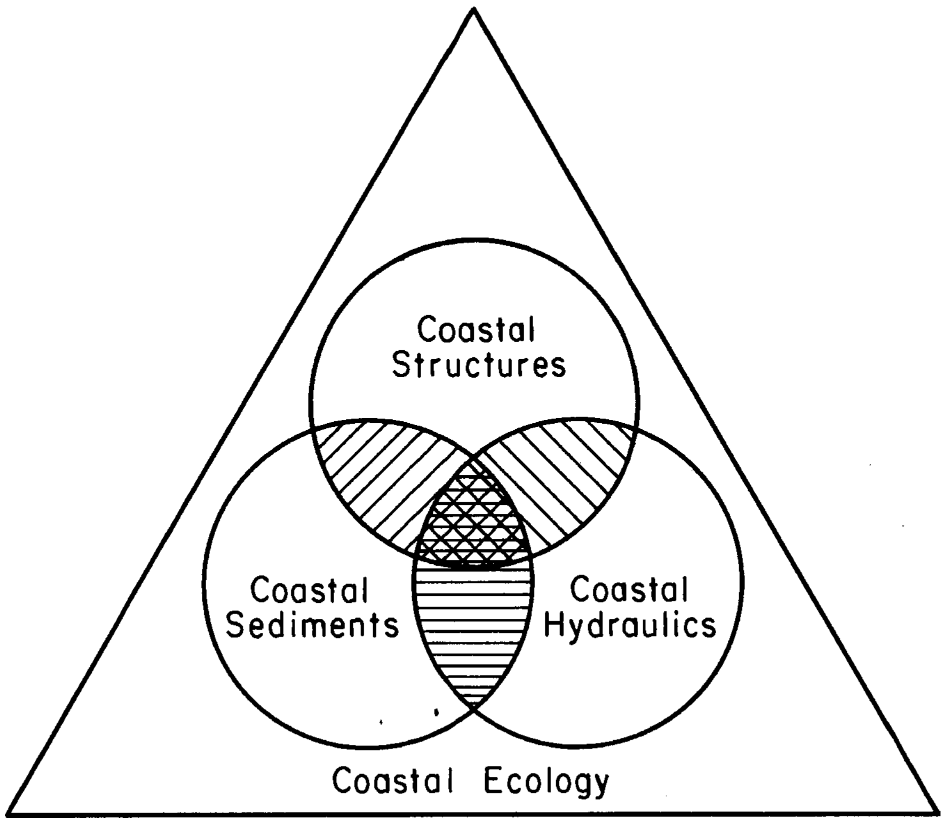
U.S. Army
Coastal Engineering
Research Center

SHORE PROTECTION MANUAL

Volume I



DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
1977



Coastal engineering: the art and science by which the interacting functions of structures, sediments, and hydraulics and the effects on the ecology in the coastal zone are made useful to man.

Original Printing 1973
Second Printing 1974
Second Edition 1975

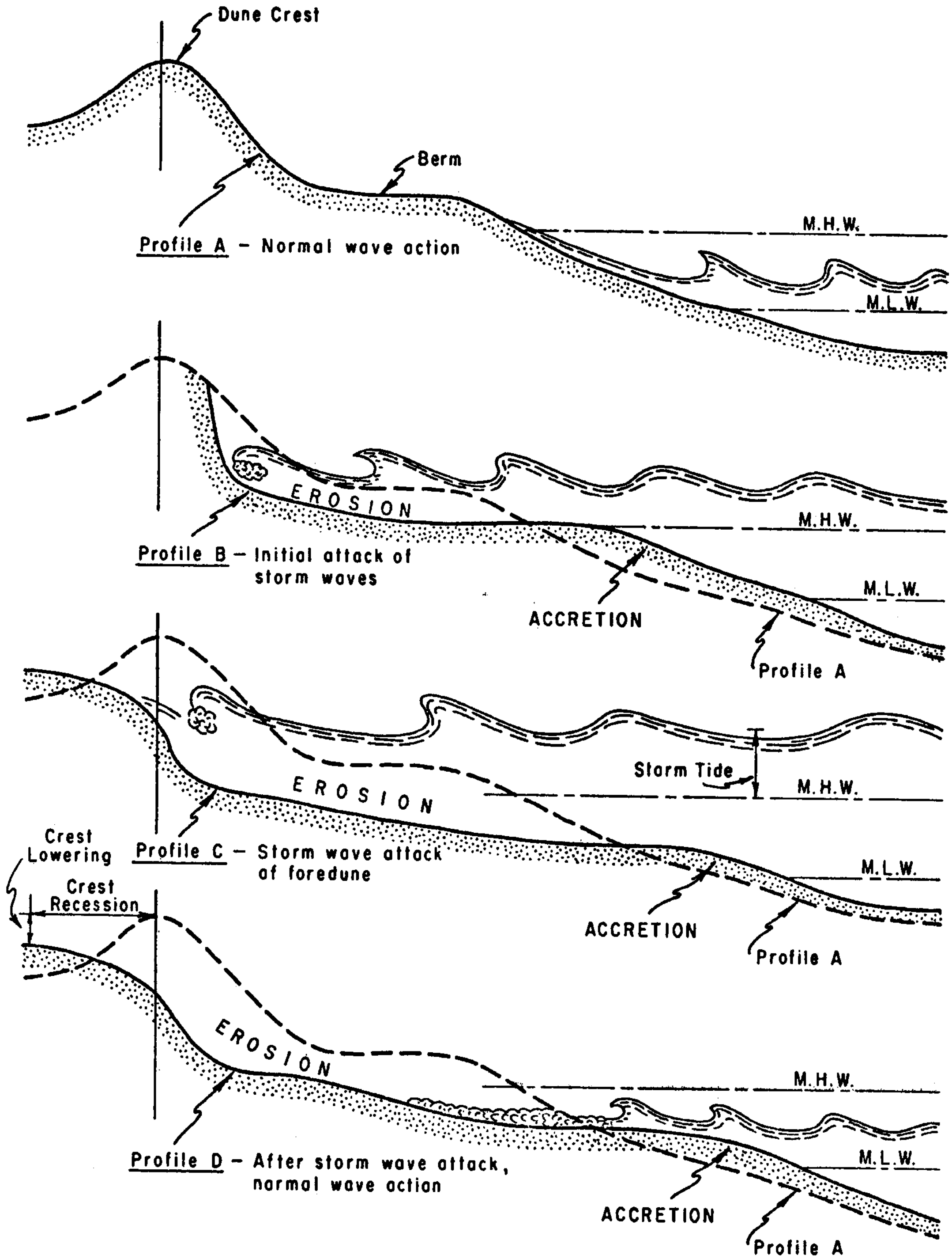
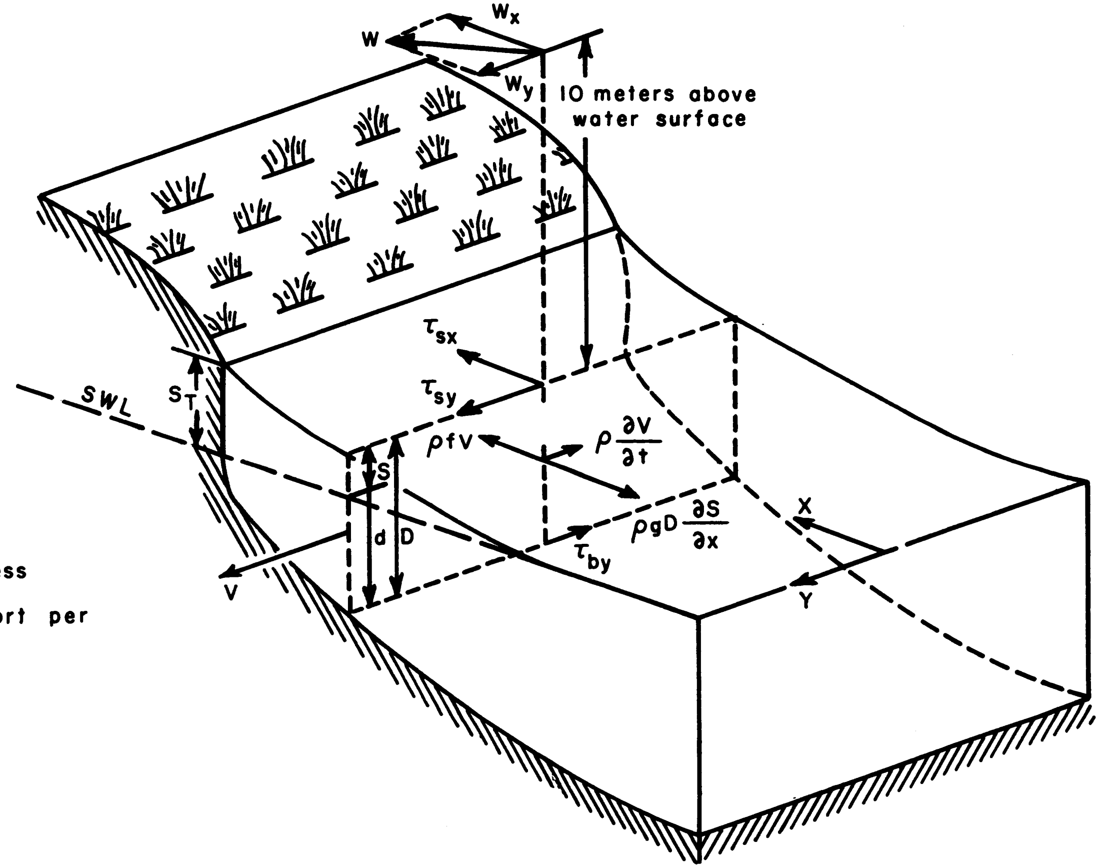


Figure 1-7. Schematic Diagram of Storm Wave Attack on Beach and Dune

NOTE: Various scales have been distorted to give a clearer pictorial representation.



LEGEND:

- SWL = Stillwater Level
- S_T = Total Setup at Shore
- S = Setup
- d = depth below SWL
- D = Total depth
- τ_{sx}, τ_{sy} = x,y components of wind stress
- V = y-component of water transport per unit width (of x)
- W = Wind Speed
- W_x, W_y = x,y components of wind speed
- τ_{by} = y-component of bottom stress
- f = coriolis parameter
- ρ = density of water
- g = gravity
- t = time

Figure 3-51. Schematic of Forces and Responses for Bathystrophic Approximation.

Figure 2-27. Wave Diffraction at Channel Islands Harbor Breakwater, California

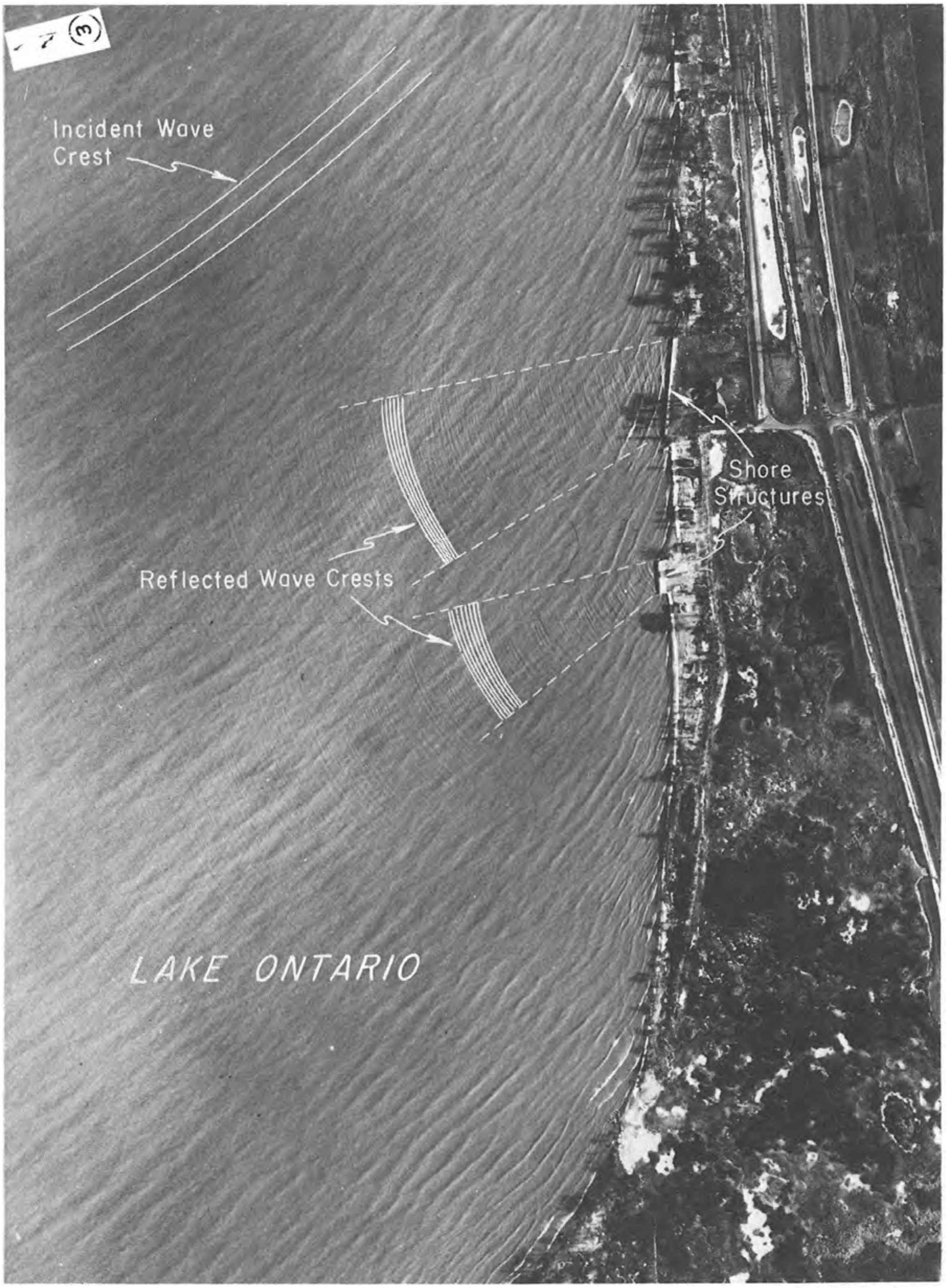


Figure 2-60. Wave Reflection at Hamlin Beach, New York

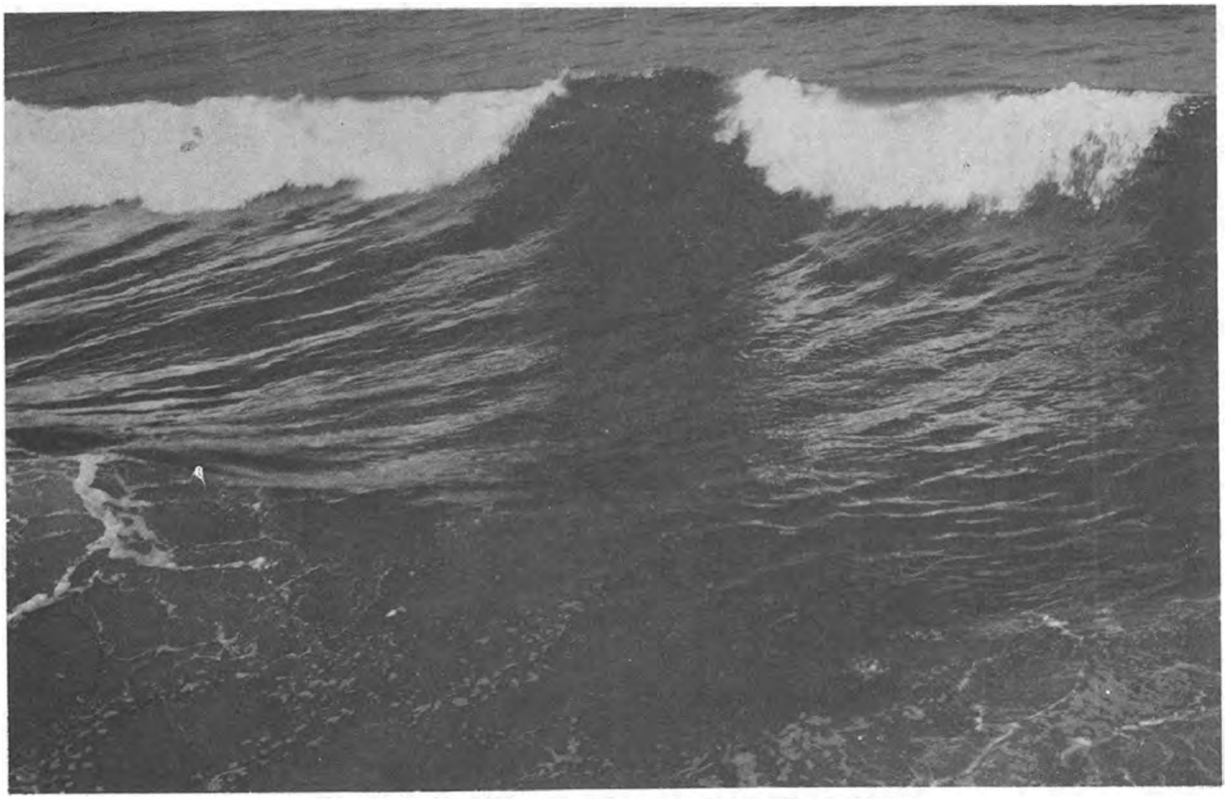


Figure 2-67. Spilling Breaking Wave

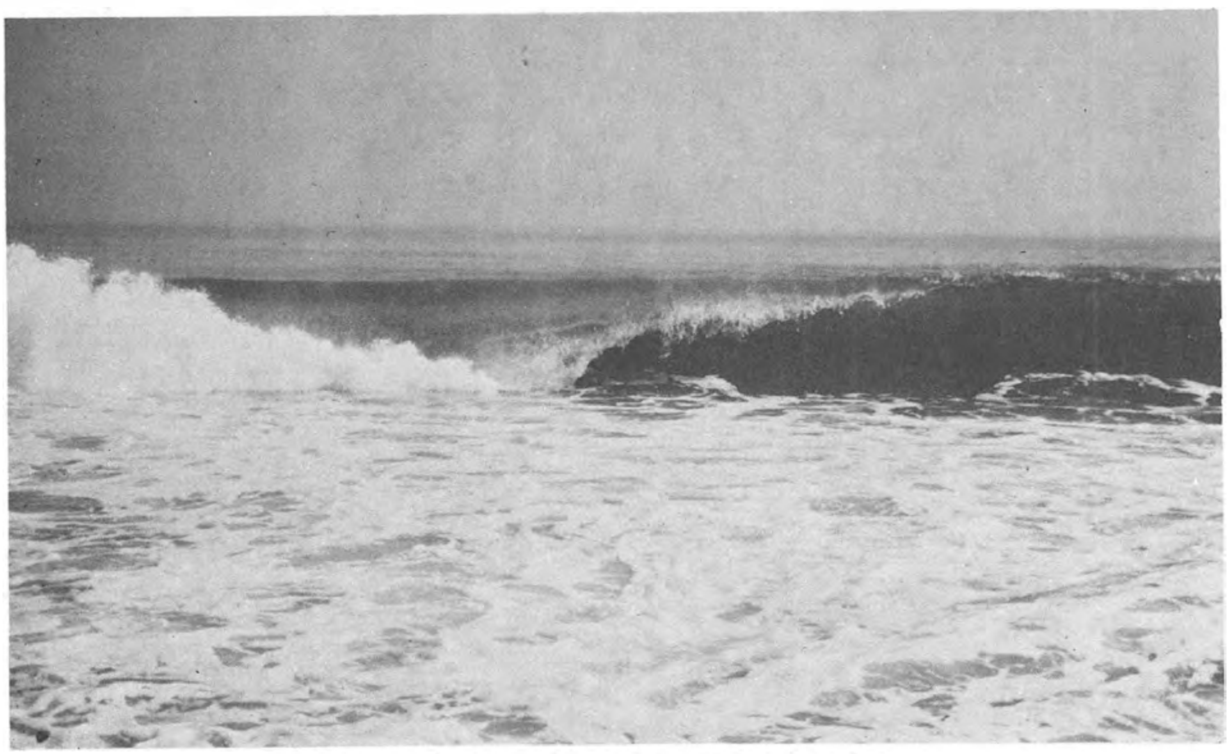


Figure 2-68. Plunging Breaking Wave

Figure 2-69. Surging Breaking Wave

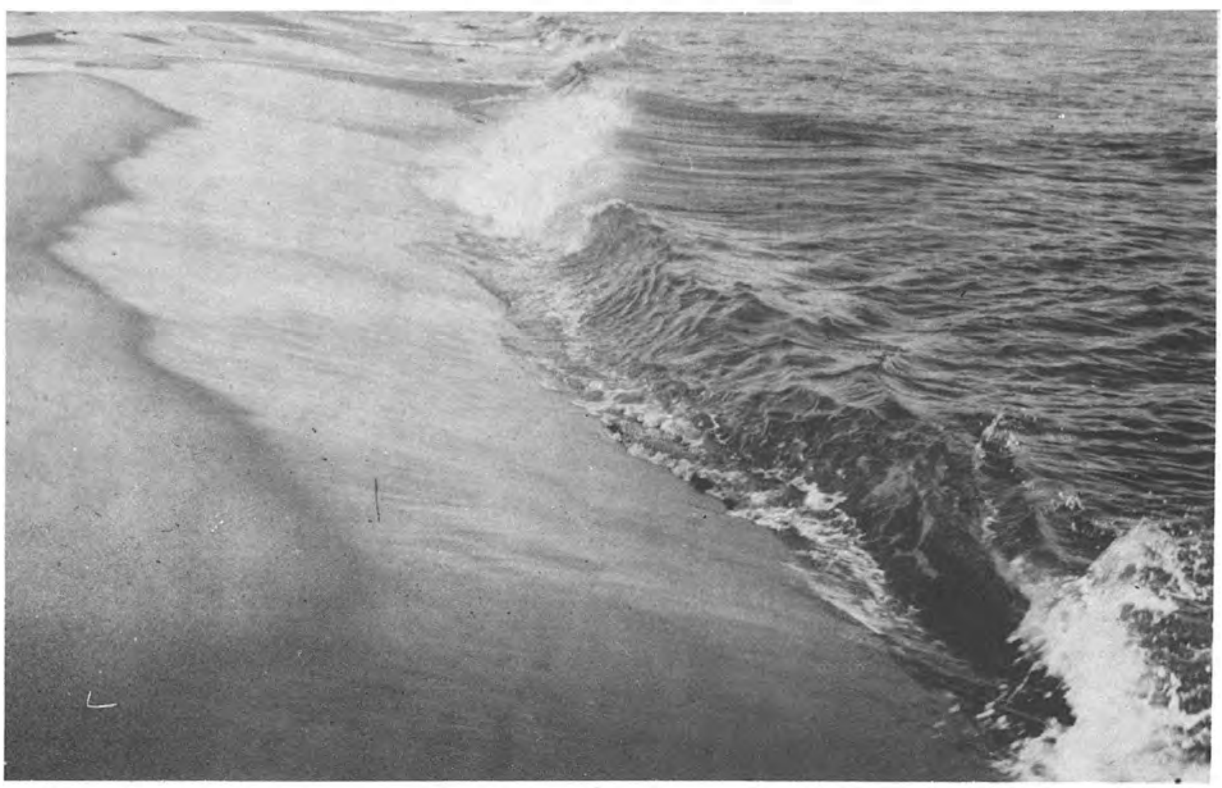


Figure 2-70. Collapsing Breaking Wave

CHAPTER 2

MECHANICS OF WAVE MOTION

CHAPTER 4

LITTORAL PROCESSES

U.S. Army
Coastal Engineering
Research Center

SHORE PROTECTION MANUAL

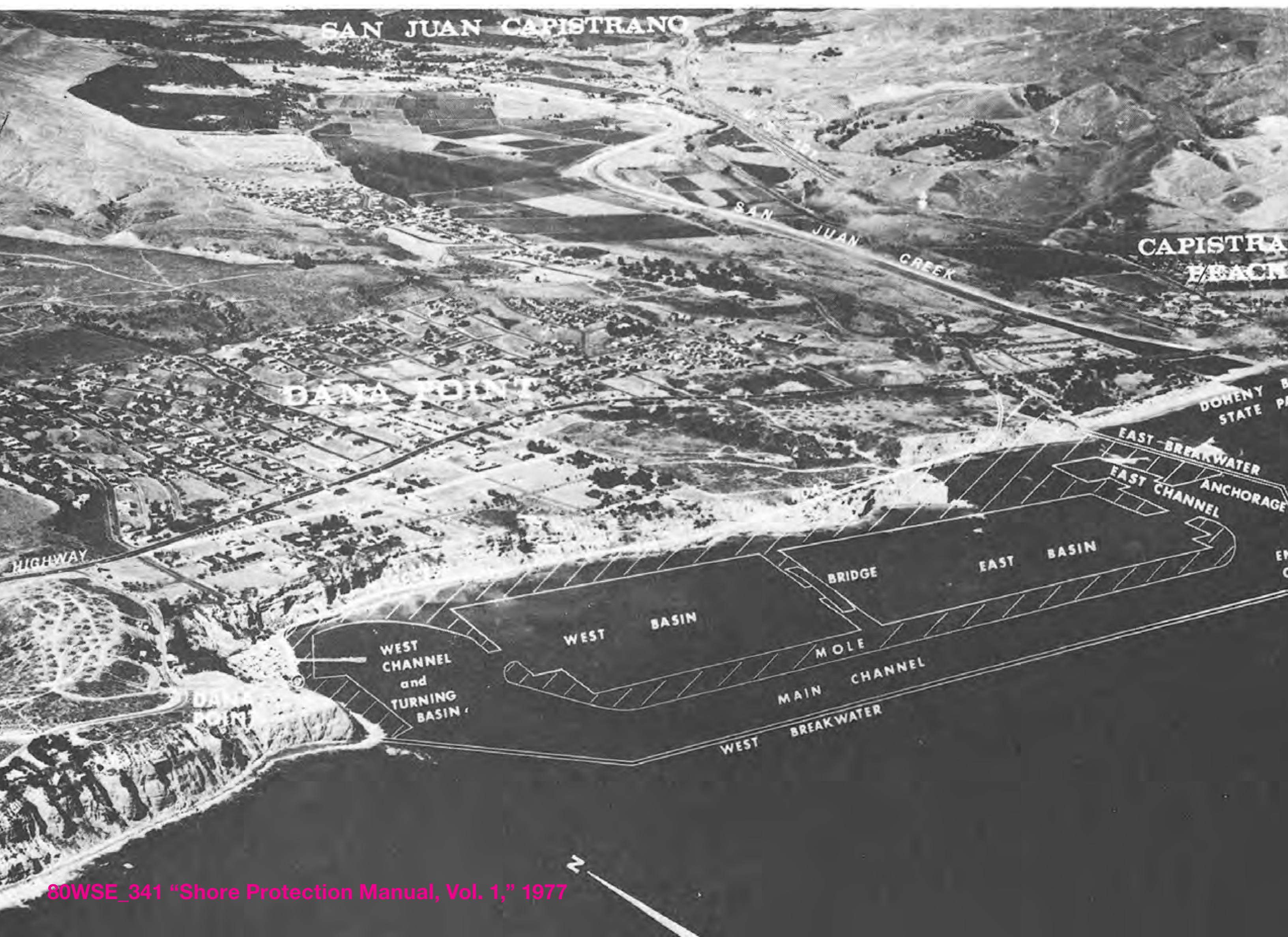
Volume II

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

1977

CHAPTER 5

PLANNING ANALYSIS



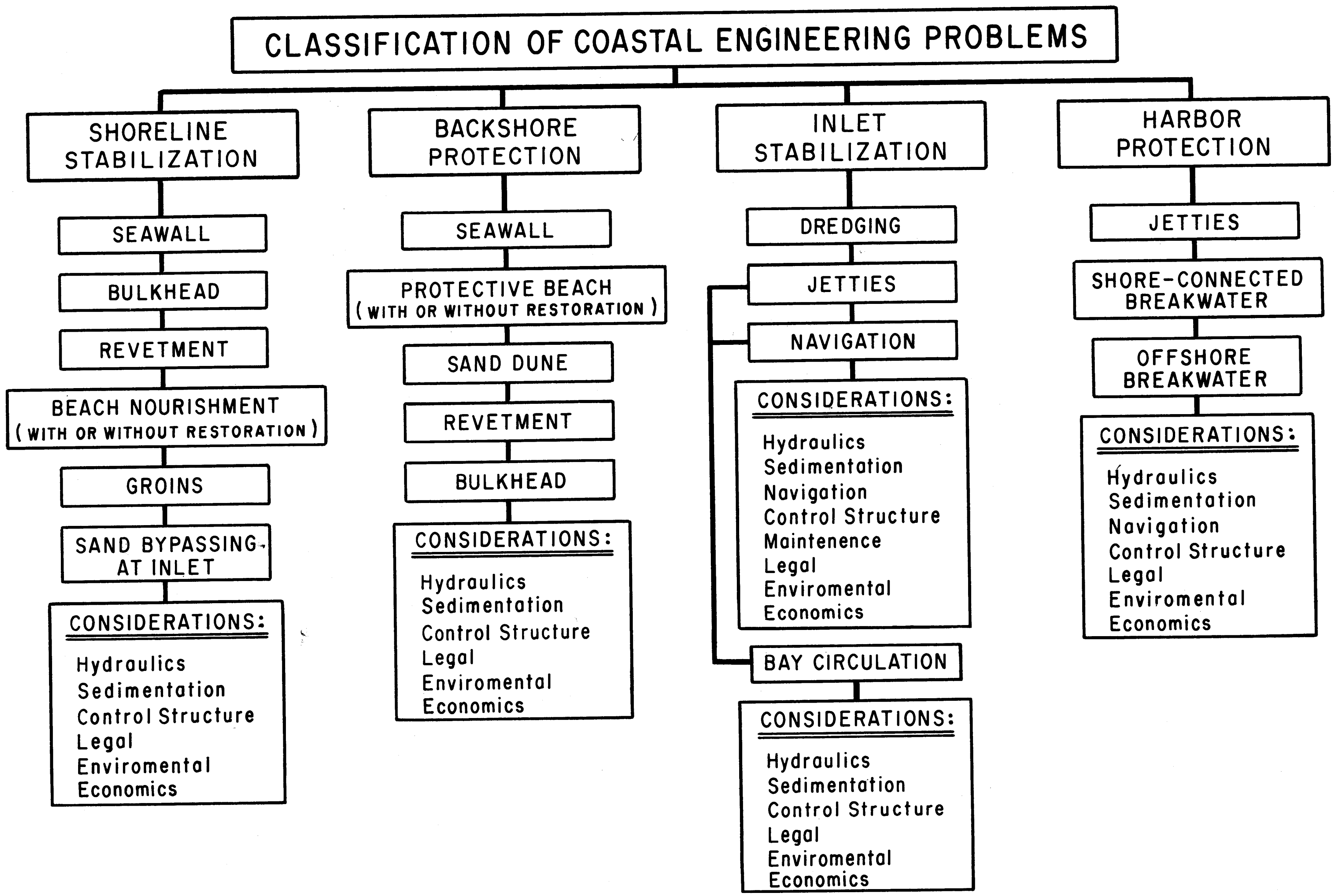


Figure 5-1. General Classification of Coastal Engineering Problems

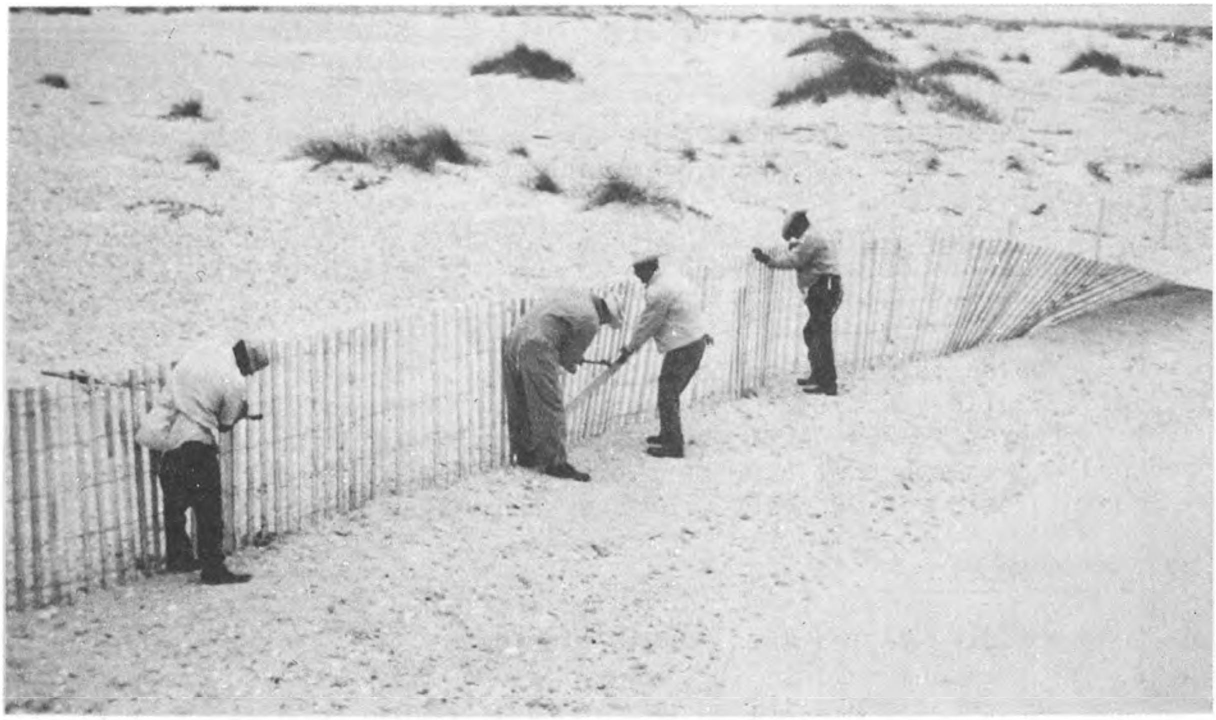
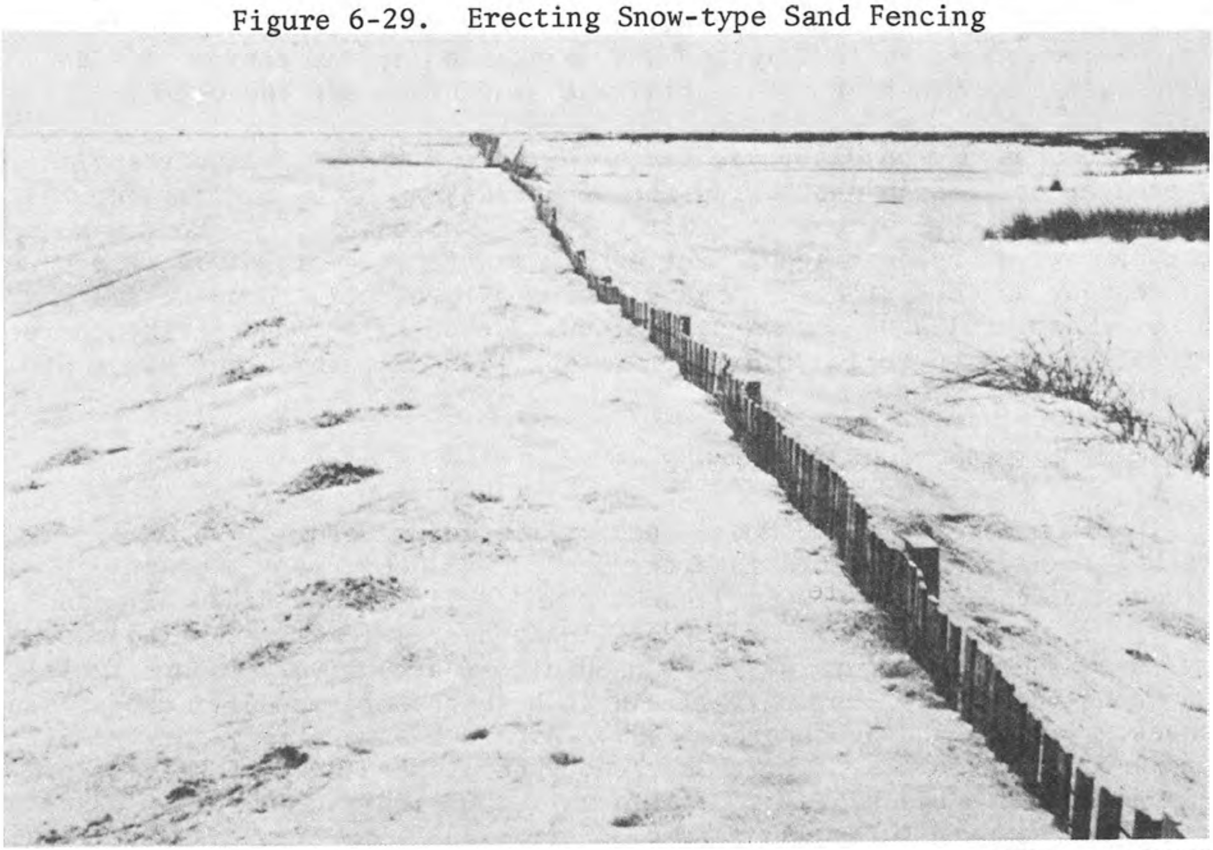
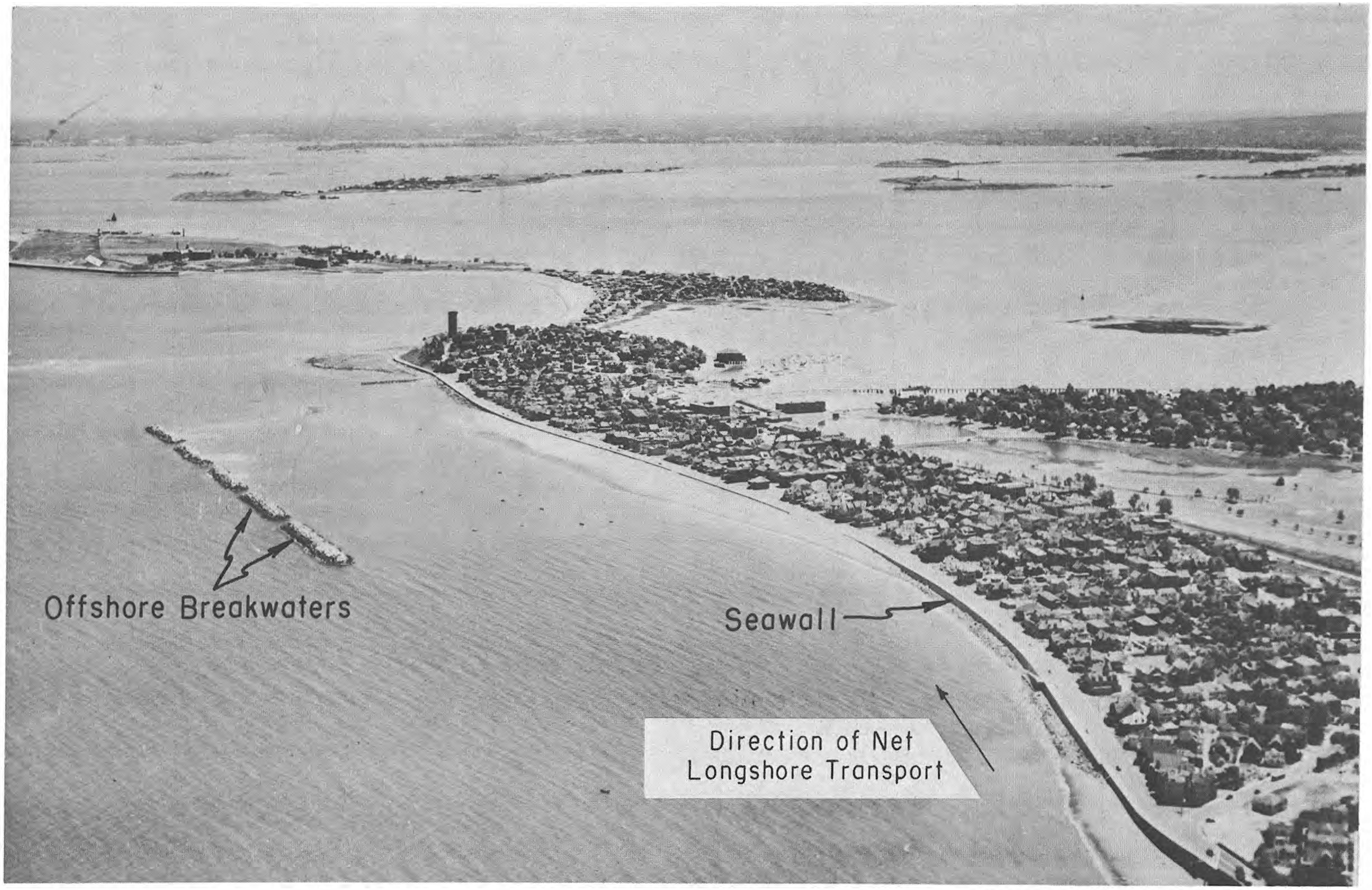


Figure 6-29. Erecting Snow-type Sand Fencing



(Savage, 1962)

Figure 6-30. Snow-type Sand Fencing Filled to Capacity



Winthrop Beach, Massachusetts - 1949

Figure 5-19. Siting Offshore Breakwaters Seaward of Seawalls for Protection

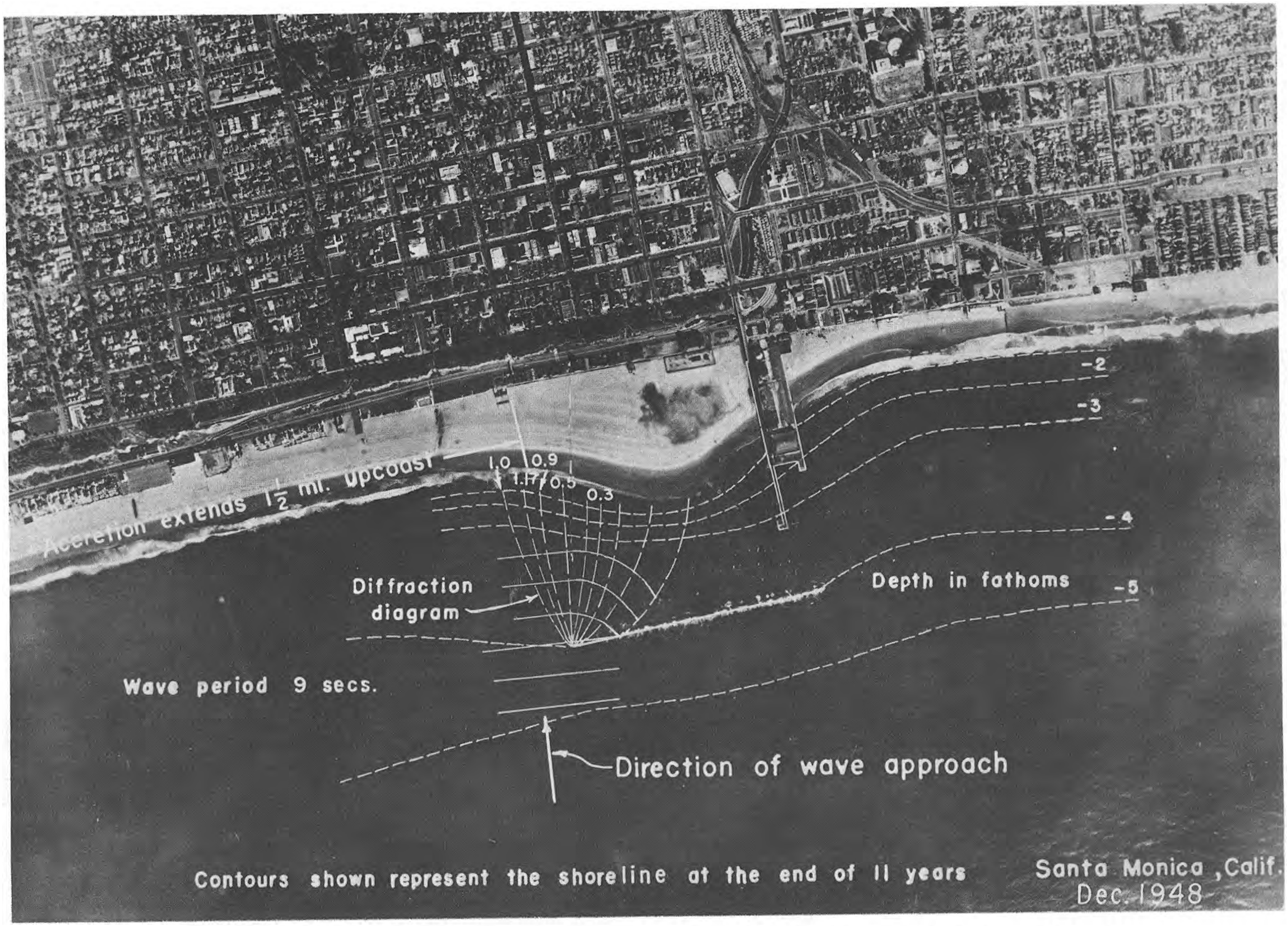


Figure 5-20. Operation of Breakwater in Diffraction of Wave Forces

Before Restoration (February 1965)

After Restoration (June 1965)

Figure 6-19. Protective Beach (Wrightsville Beach, North Carolina)

Before Restoration (1964)

After Restoration (1965)

Figure 6-21. Protective Beach (Carolina Beach, North Carolina)

Nantucket Island, Massachusetts (1972)
Photograph, Courtesy of U.S. Steel

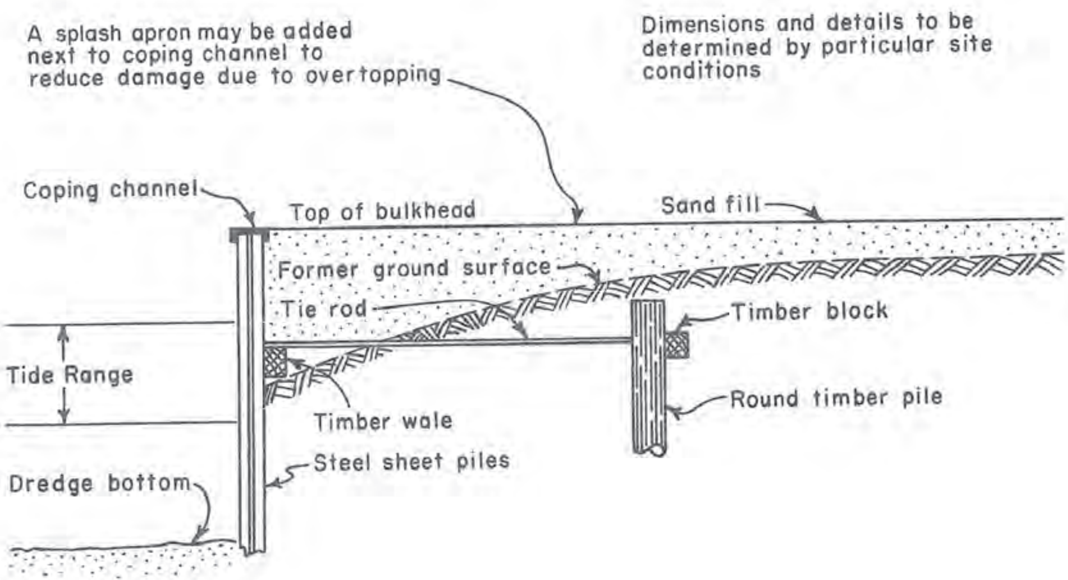
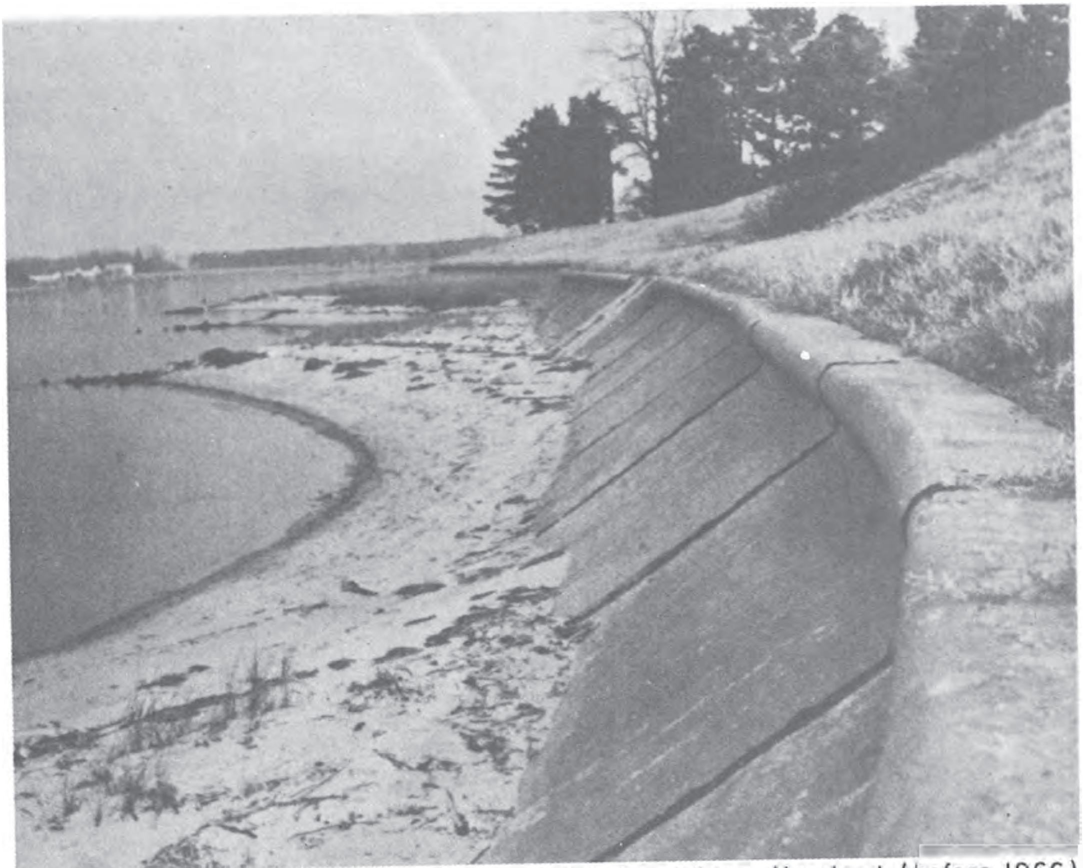


Figure 6-7. Steel Sheet-Pile Bulkhead



Pioneer Point, Cambridge, Maryland (before 1966)
Courtesy of Portland Cement Association

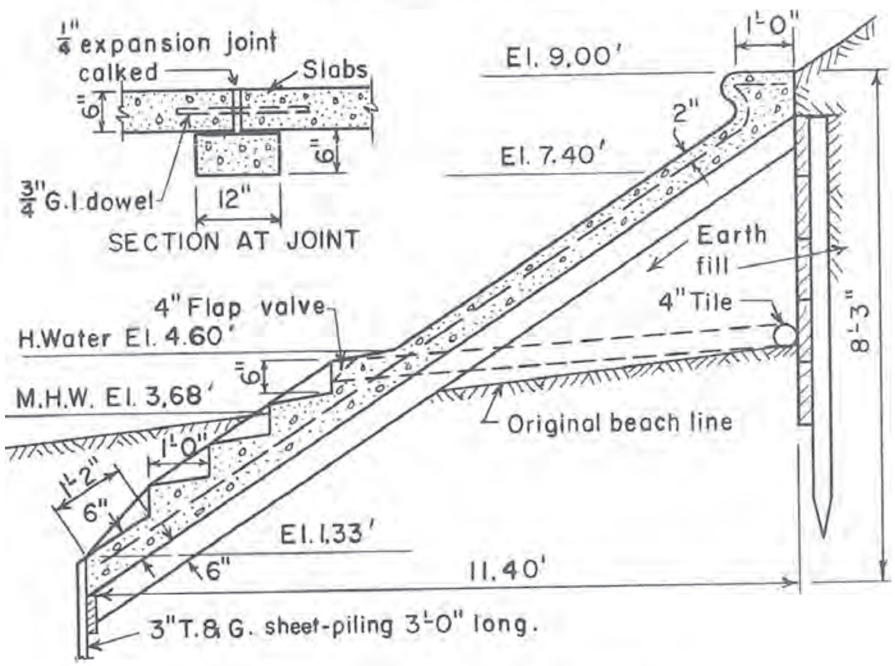
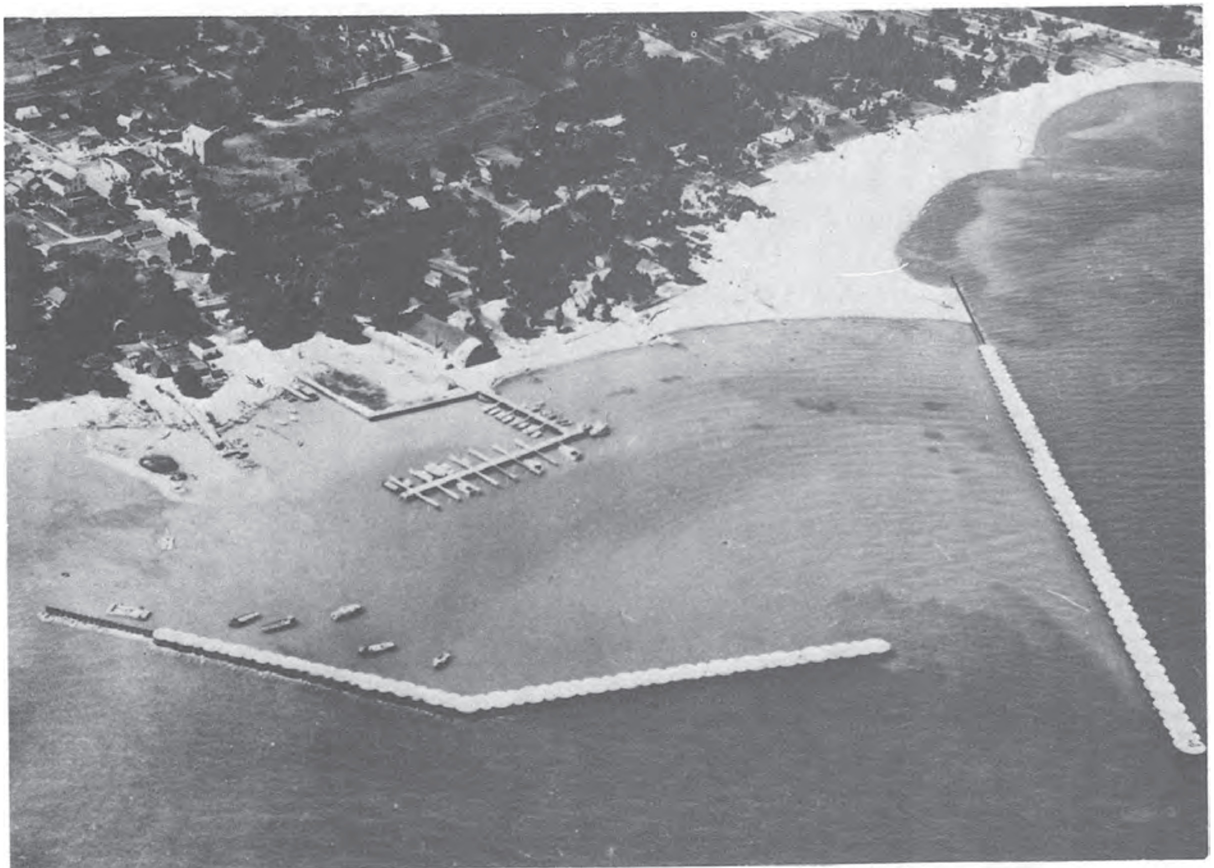


Figure 6-9. Concrete Revetment



Port Sanilac, Michigan (July 1963)

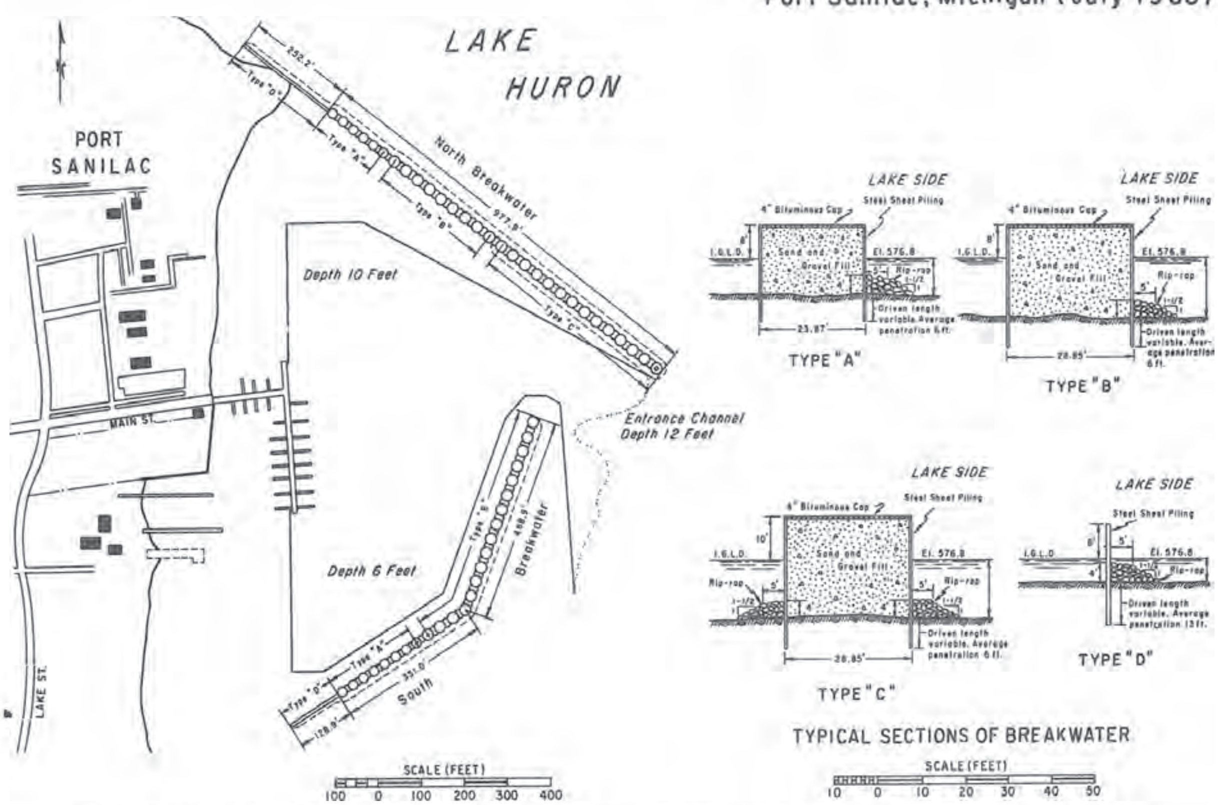
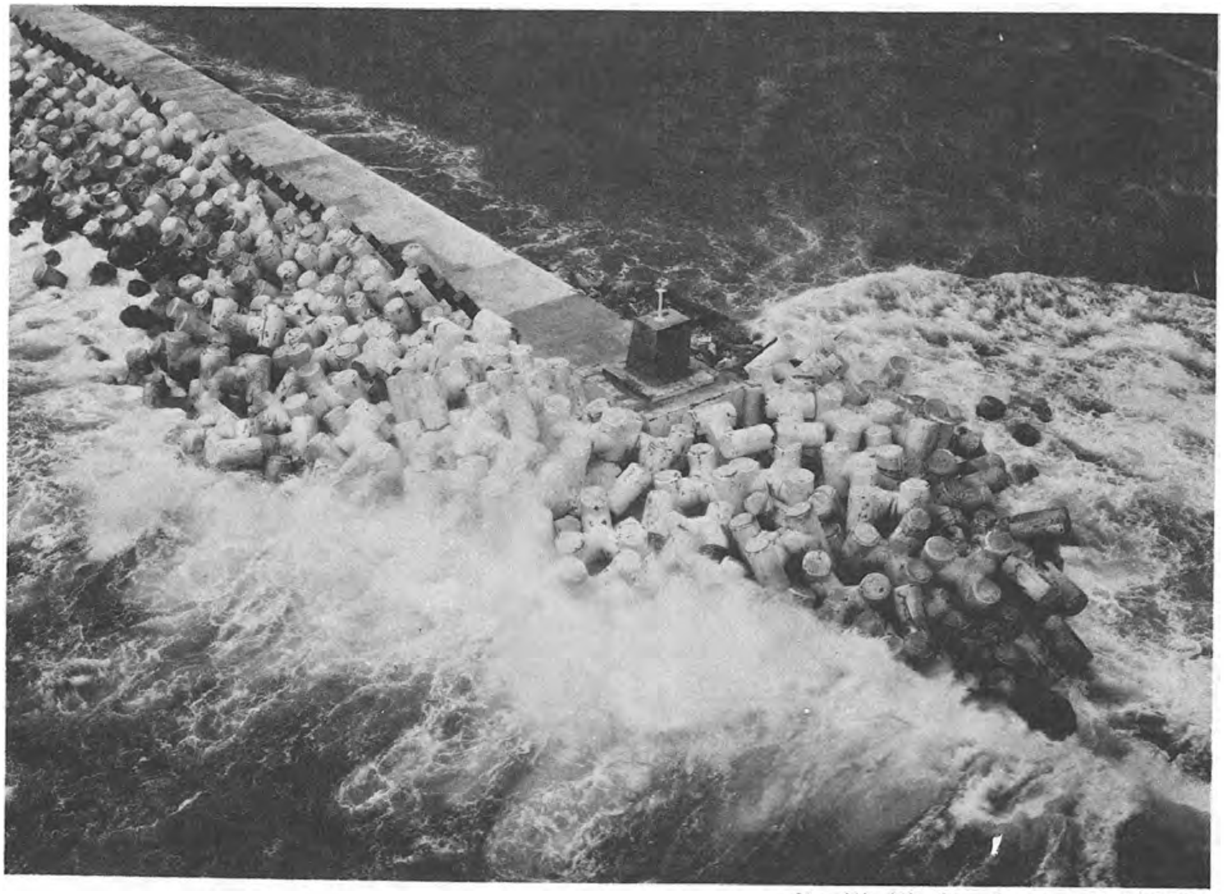


Figure 6-70. Cellular Steel Sheet-Pile and Sheet-Pile Breakwater



Nawiliwili, Kauai, Hawaii (1959)

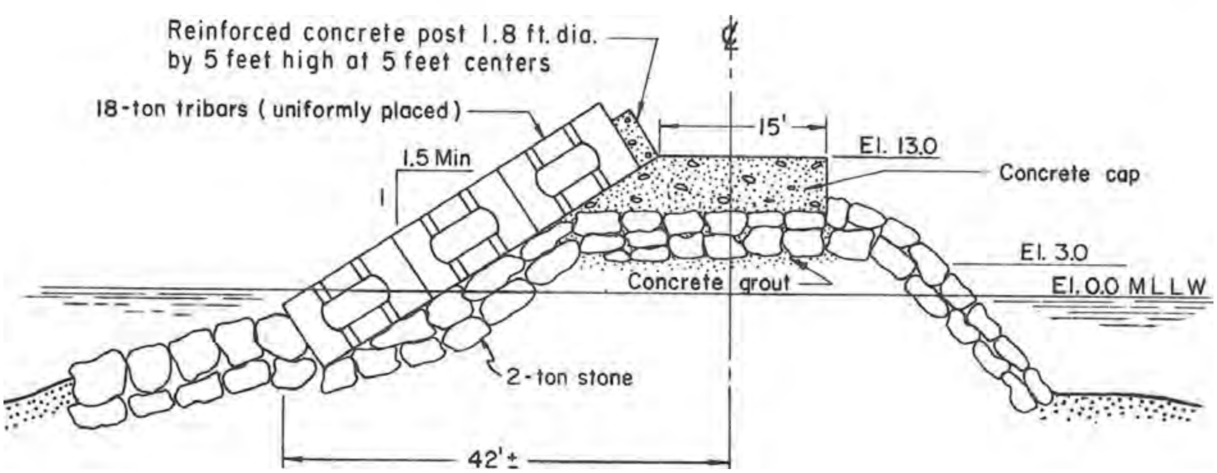


Figure 6-68. Tribar-Rubble-Mound Breakwater

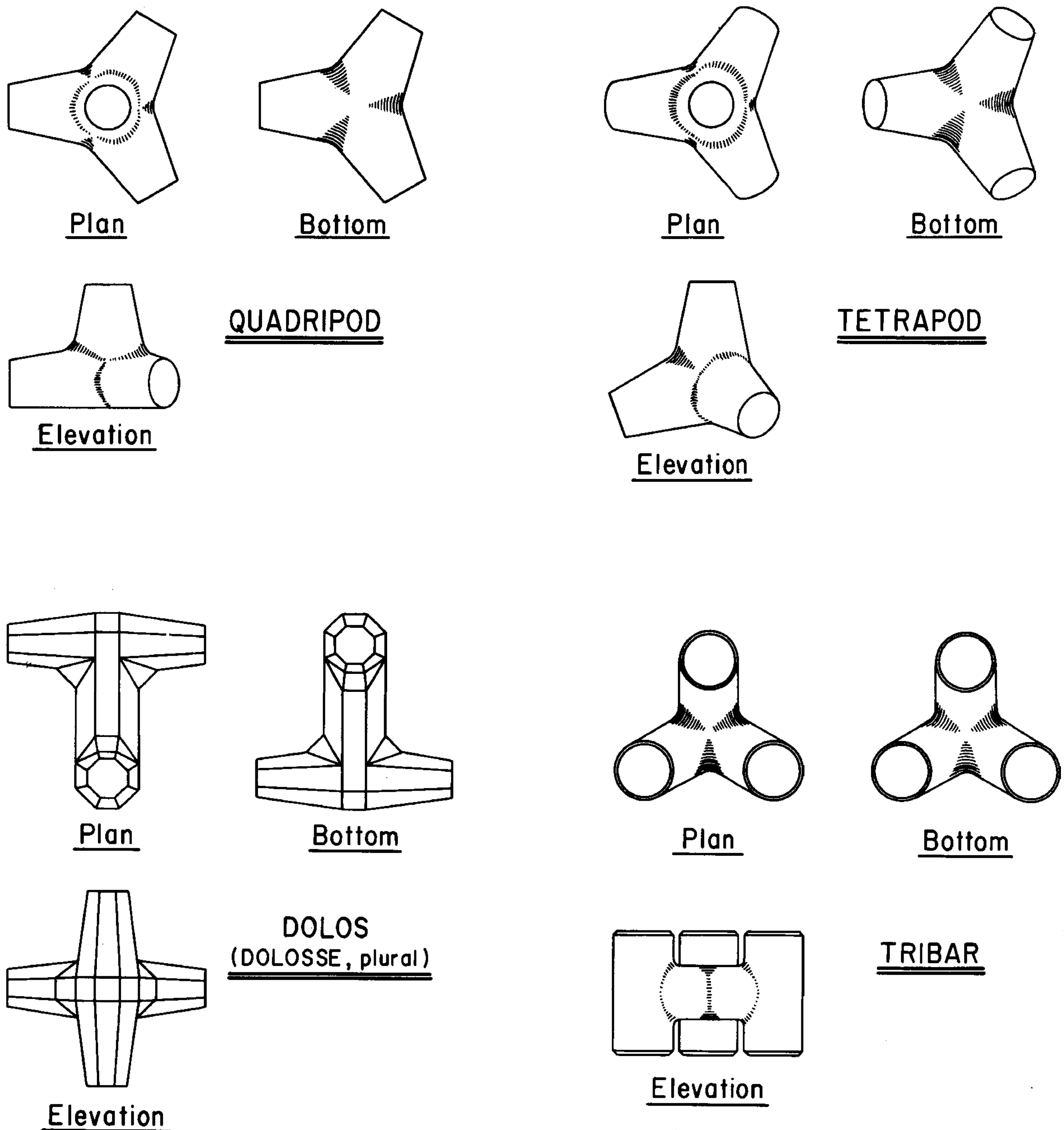


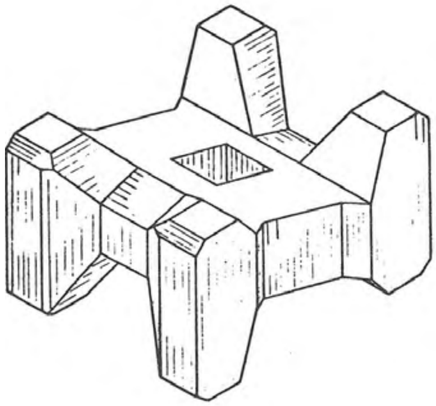
Figure 7-92. Concrete Armor Units

Table 7-14. Unit Weights and Internal Friction Angles of Soils

Classification	Unit Weight (lb/cu ft)					
	Dry		Wet		Submerged	
	Min. (loose)	Max. (dense)	Min. (loose)	Max. (dense)	Min. (loose)	Max. (dense)
GRANDULAR MATERIALS						
1. Uniform Materials						
Standard Ottawa SAND	92	110	93	131	57	69
Clean, uniform SAND (fine or Medium)	83	118	84	136	52	73
Uniform, inorganic SILT	80	118	81	136	51	73
2. Well-graded Materials						
Silty SAND	87	127	88	142	54	79
Clean, fine to coarse SAND	85	138	86	148	53	86
Micaceous SAND	76	120	77	138	48	76
Silty SAND and GRAVEL	89	146	90	155	56	92
MIXED SOILS						
1. Sandy or silty CLAY	60	135	100	147	38	85
2. Skip-graded silty CLAY with stones or rock fragments	84	140	115	151	53	89
3. Well-graded GRAVEL, SAND, SILT and CLAY mixture	100	148	125	156	62	94
CLAY SOILS						
1. CLAY (30 to 50 percent clay sizes)	50	112	94	133	31	71
2. Colloidal CLAY (−0.002 mm. ≥50 percent)	13	106	71	128	8	66
ORGANIC SOILS						
1. Organic SILT	40	110	87	131	25	69
2. Organic CLAY (30 to 50 percent clay sizes)	30	100	81	125	18	62

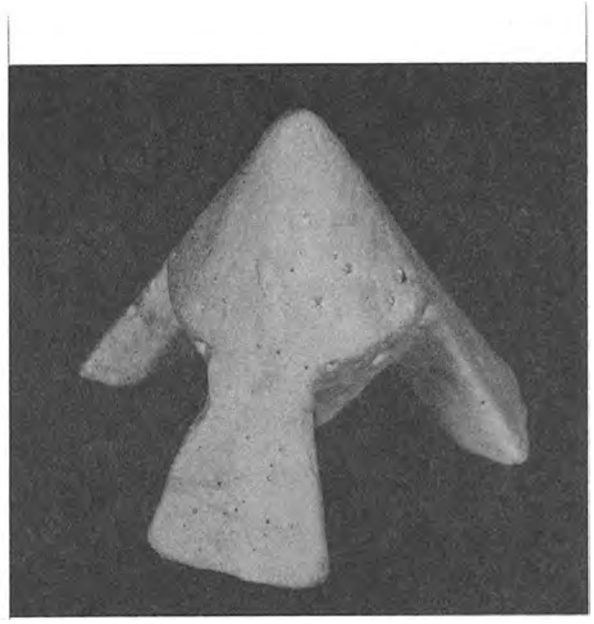
Classification	Friction Angle ϕ (degrees)	Density or Consistency	Unit Weight (lb/cu ft)		
			Soil	Equivalent Fluid	
				Active Case	Passive Case
Coarse SAND or SAND and GRAVEL	45	compact	140	24	820
	38	firm	120	29	510
	32	loose	90	28	290
Medium SAND	40	compact	130	28	600
	34	firm	110	31	390
	30	loose	90	30	270
Fine SAND	34	compact	130	37	460
	30	firm	100	33	300
	28	loose	85	31	280
Fine, silty SAND or sandy SILT	32	compact	130	40	420
	30	firm	100	33	300
	28	loose	85	31	280
Fine, uniform SILT	30	compact	135	45	400
	28	firm	110	38	300
	26	loose	85	33	220
CLAY-SILT	20	medium	120	59	245
		soft	90	44	183
Silty CLAY	15	medium	120	71	204
		soft	90	53	153
CLAY	10	medium	120	84	170
		soft	90	53	153
CLAY	0	medium	120	120	120
		soft	90	90	90

(after Hough, 1957)



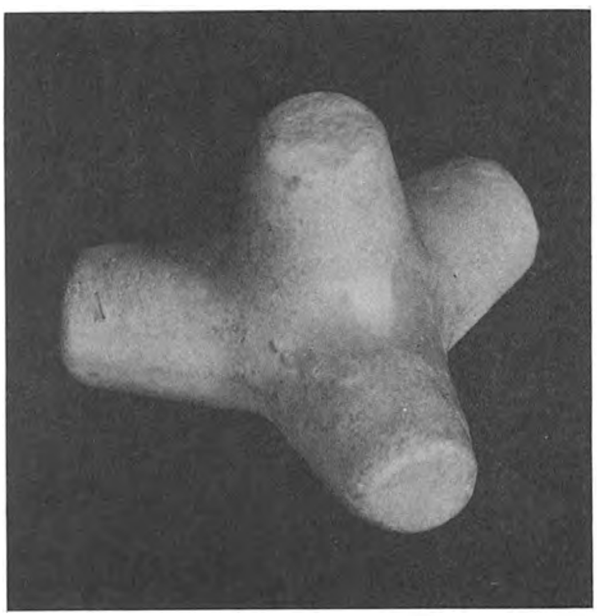
(After Nagai, 1962[28])

Fig. 13. N-shaped block



(After Jackson, 1961[15])

Fig. 14. Pelican stool

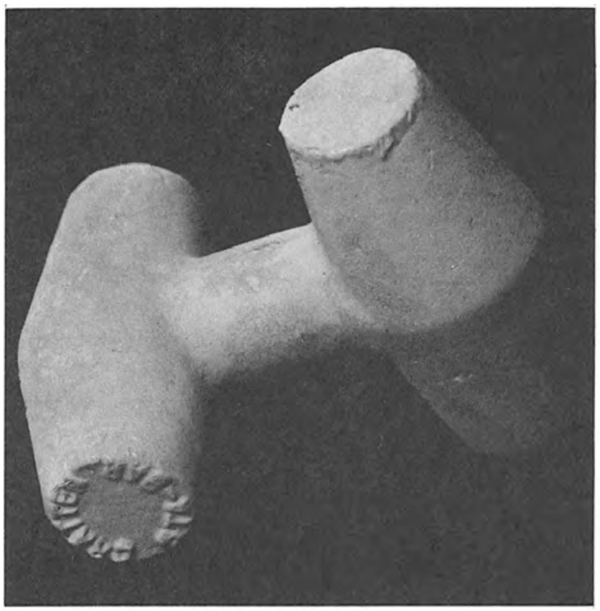


(After Jackson, 1968[12])

Fig. 15. Quadripod

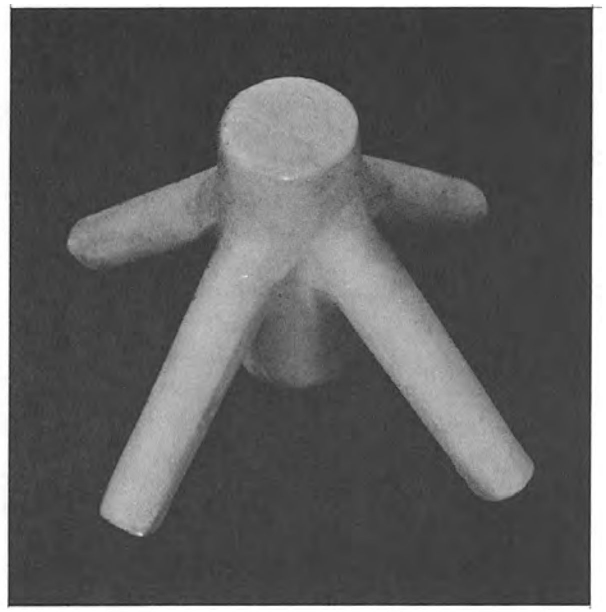
(Courtesy of Stabits Ltd., Sardinia House, 52 Lincoln's Inn Fields, London, W.C. 2)

Fig. 16. Stabit



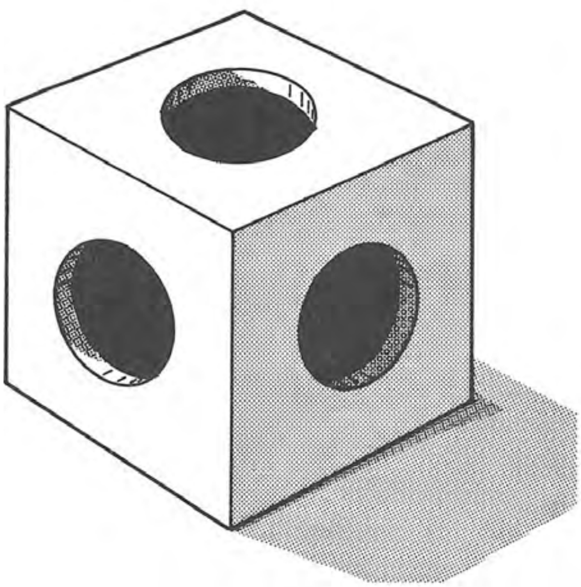
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Fig. 17. Sta-Bar



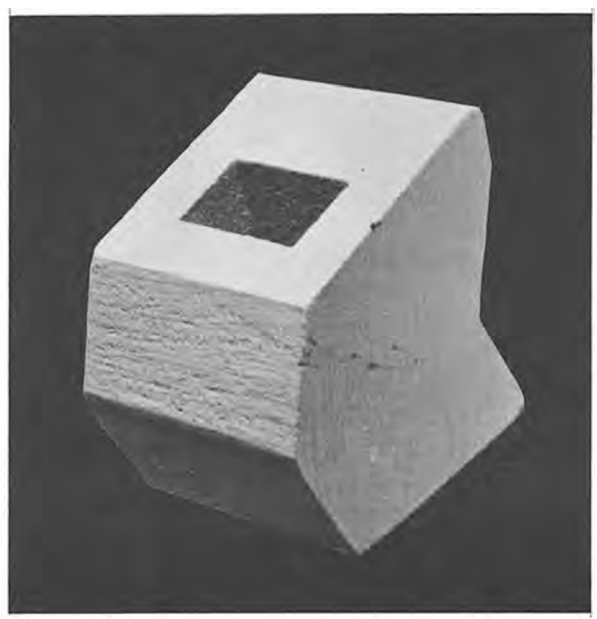
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N Y. 10710)

Fig. 18. Sta-Pod



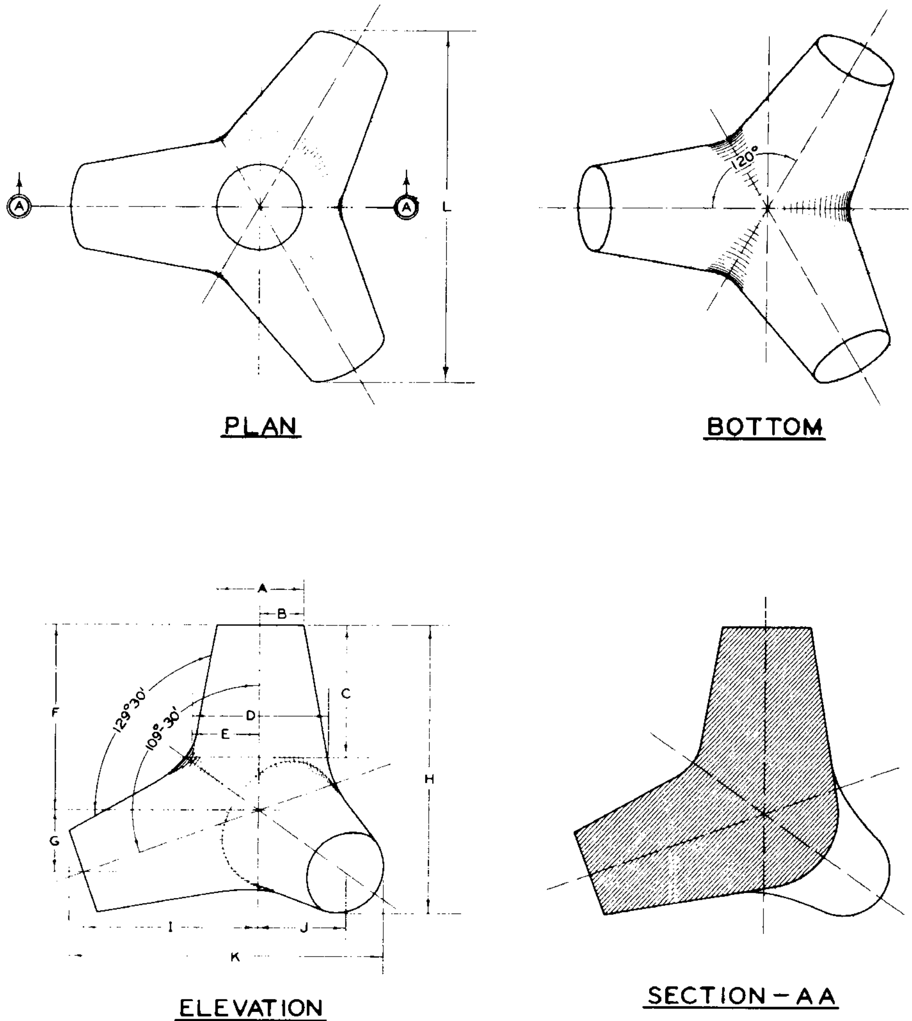
(Courtesy of B. Hakkeling, Ing, Merellaan 269, Maassluis, Netherlands)

Fig. 19. Stolk cube



(Courtesy of Noreno, Cort Adlers Gate 16, Oslo, Norway)

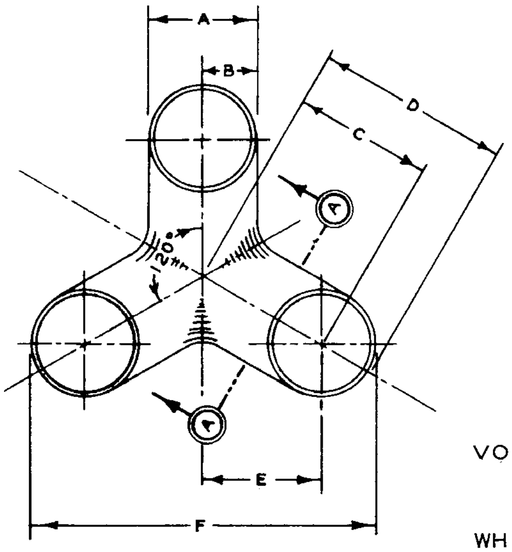
Fig. 20. Svee block



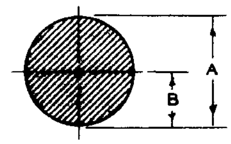
VOL = 0.280 H^3; A = 0.302 H; B = 0.151 H;
C = 0.477 H; D = 0.470 H; E = 0.235 H;
F = 0.644 H; G = 0.215 H; I = 0.606 H;
J = 0.303 H; K = 1.091 H; L = 1.201 H;

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12]

Fig. 36. Details of tetrapod armor unit



PLAN



SECTION A–A

VOLUME OF INDIVIDUAL ARMOR UNIT IS

$$V = A^3 (2.36\, k_v + 3.42)$$

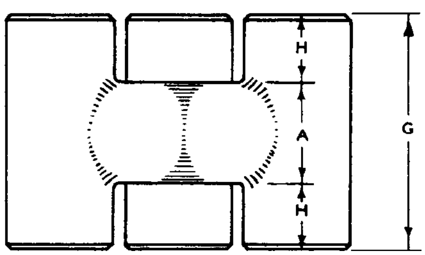
WHERE A = THE DIAMETER OF A LEG

$k_v = C/A = 1.2$

C = THE DISTANCE FROM THE CENTER OF THE UNIT TO THE CENTER OF A LEG

$G = 2A$

THUS $V = 6.252\, A^3$



ELEVATION

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12] AT PRESENT TIME PATENTEE RECOMMENDS $C = 1.25\, A$, AND FILLETS AT INTERSECTION OF HORIZONTAL AND VERTICAL MEMBERS WITH A RADIUS EQUAL TO $A/4$. THE EQUATION FOR VOLUME IN CU YD IS THEN, APPROXIMATELY, $V = 0.24^3$. DETAILS OF FORMS SHOULD BE OBTAINED FROM PATENTEE.

Fig. 37. Details of tribar armor unit

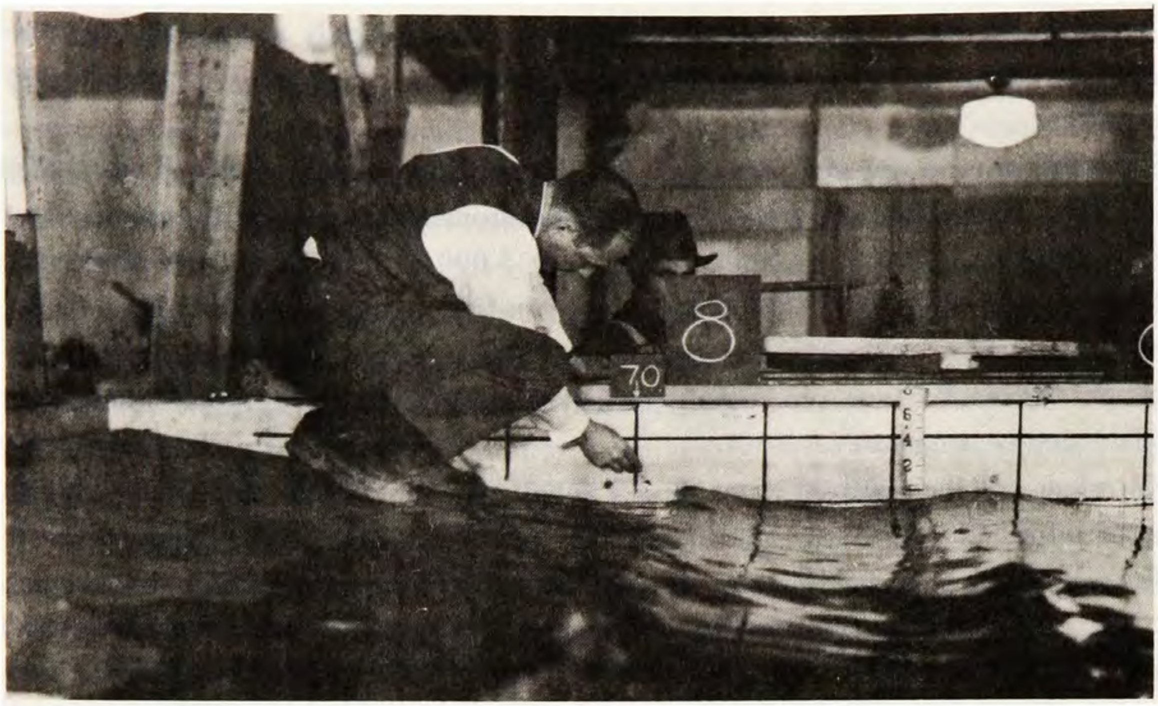


Figure 21. Jay V. Hall, Jr., involved in research work during World War II, using the BEB's 85-foot wave tank.

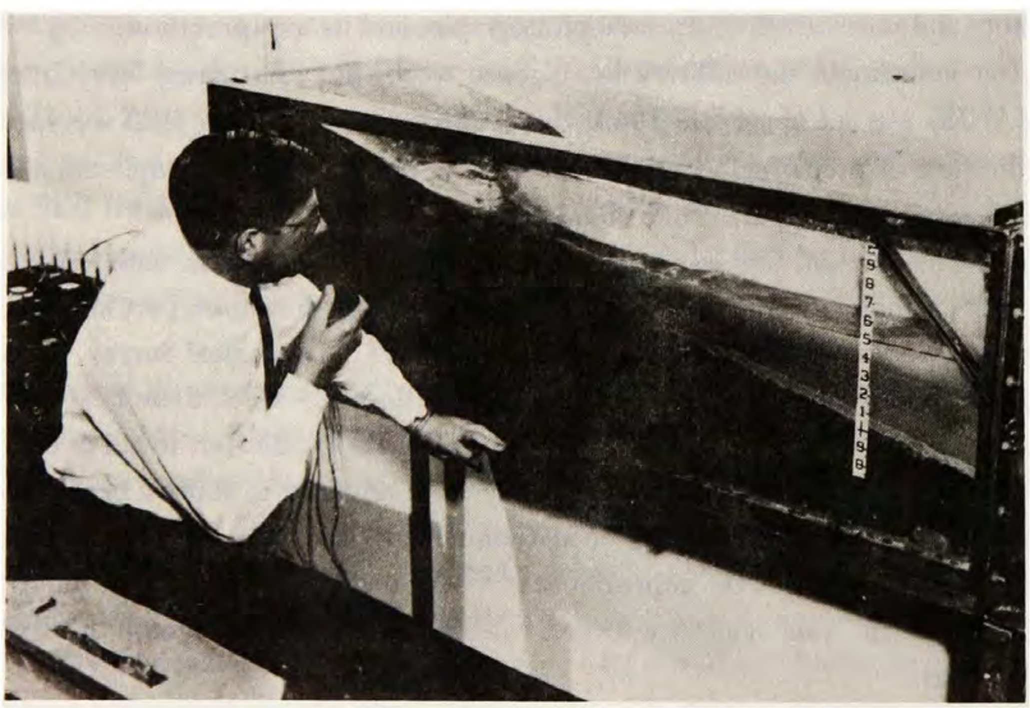


Figure 22. Martin A. Mason working on a wave experiment during World War II, using the BEB's 42-foot wave tank.

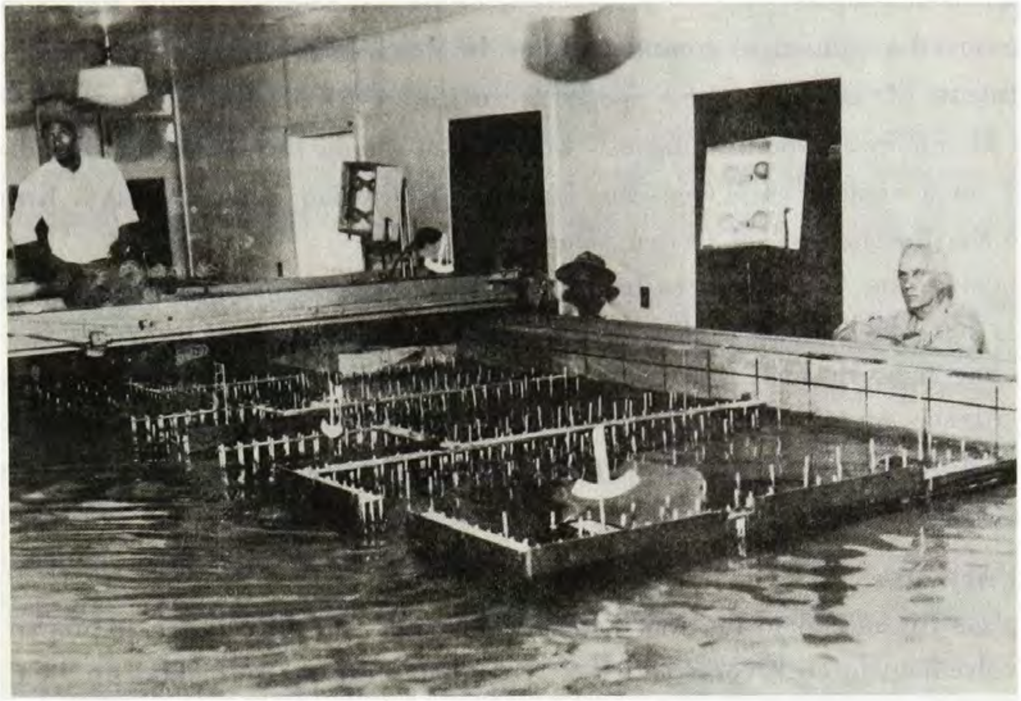


Figure 23. Left to right: Jim Mason, Leroy Harris, and Gen. John J. Kingman working on an experiment in the BEB's 85-foot wave tank, August 1943.

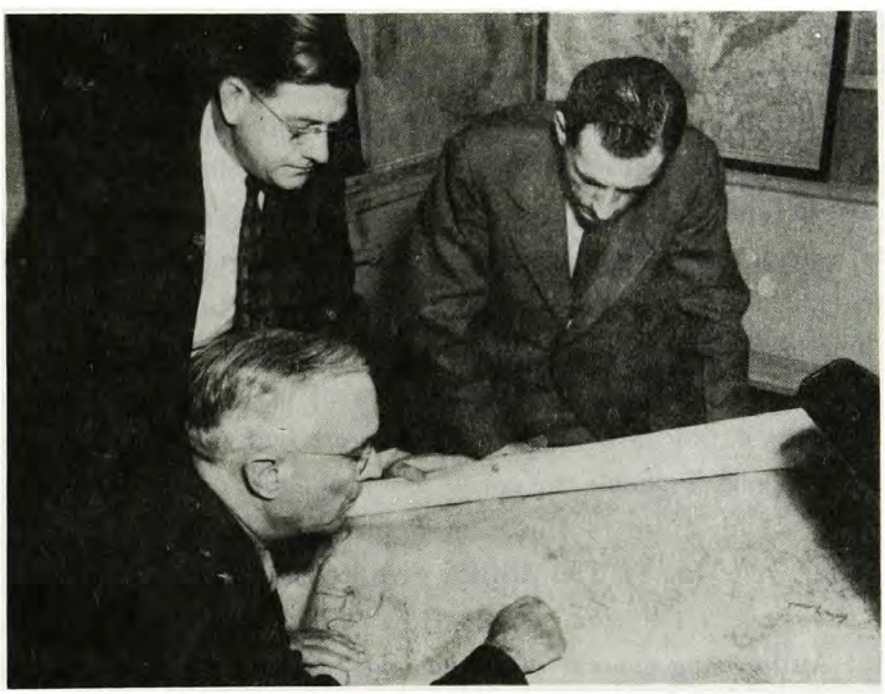


Figure 24. Gen. John Kingman (seated), Martin A. Mason (left), and William C. Krumbein studying a map in connection with the BEB's intelligence work during World War II.

Figure 27. Pouring concrete for the bed of the 635-foot wave tank, 30 September 1949.

Figure 28. Sides of the 635-foot tank beginning to take shape. To the right, grading for shore processes test basin is underway, 5 November 1949.

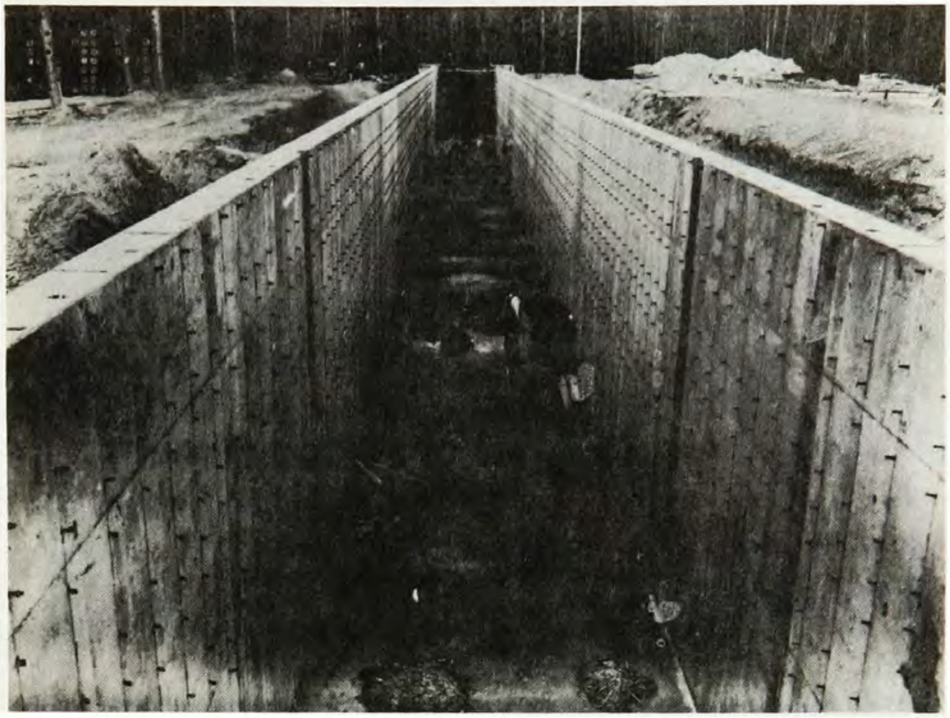


Figure 29. View looking down the 20-foot-deep, 635-foot tank toward area where generator will be located. On the right, shore processes test basin is under construction.

Figure 30. Another view of construction of BEB research facilities, 10 January 1950.

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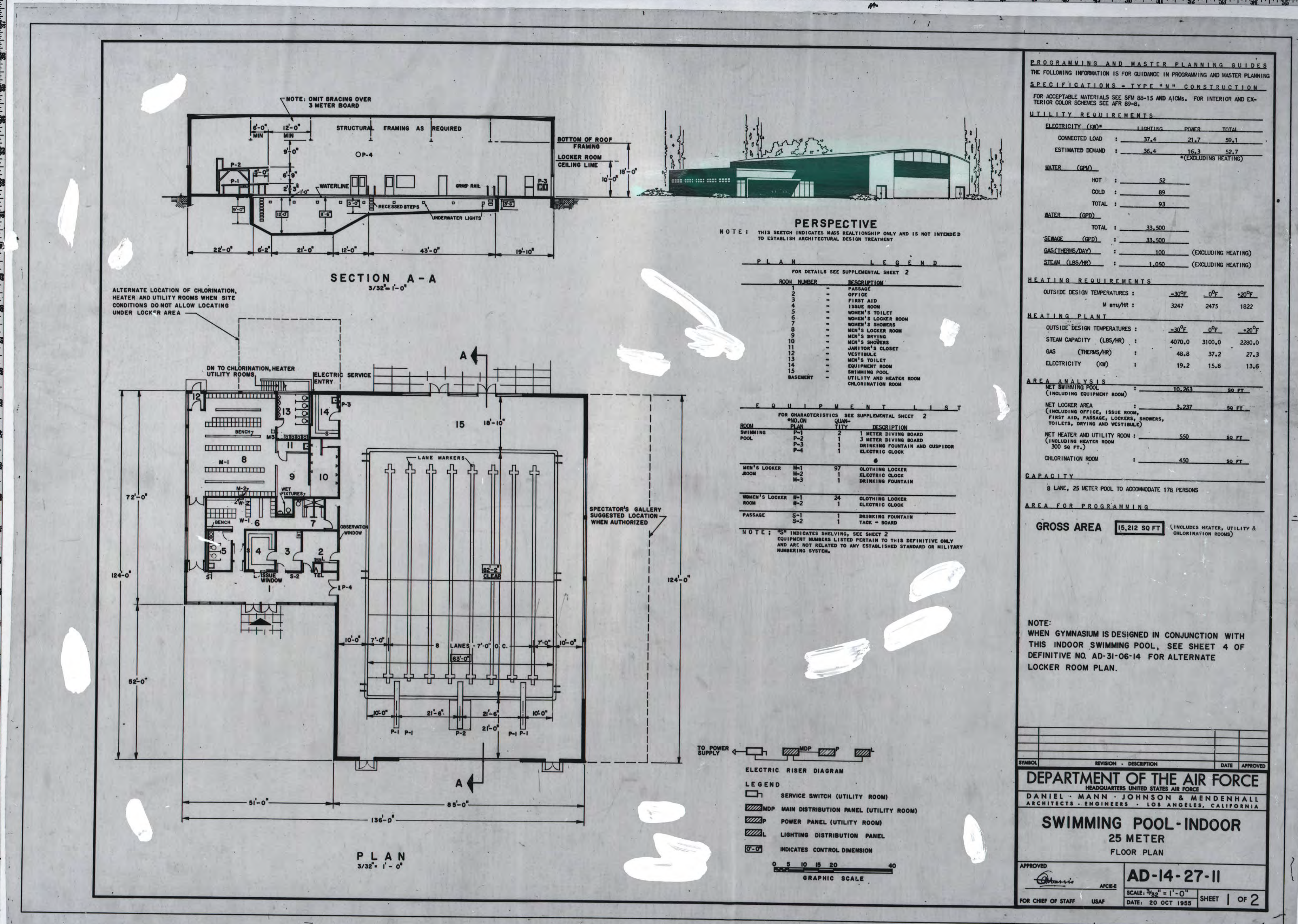
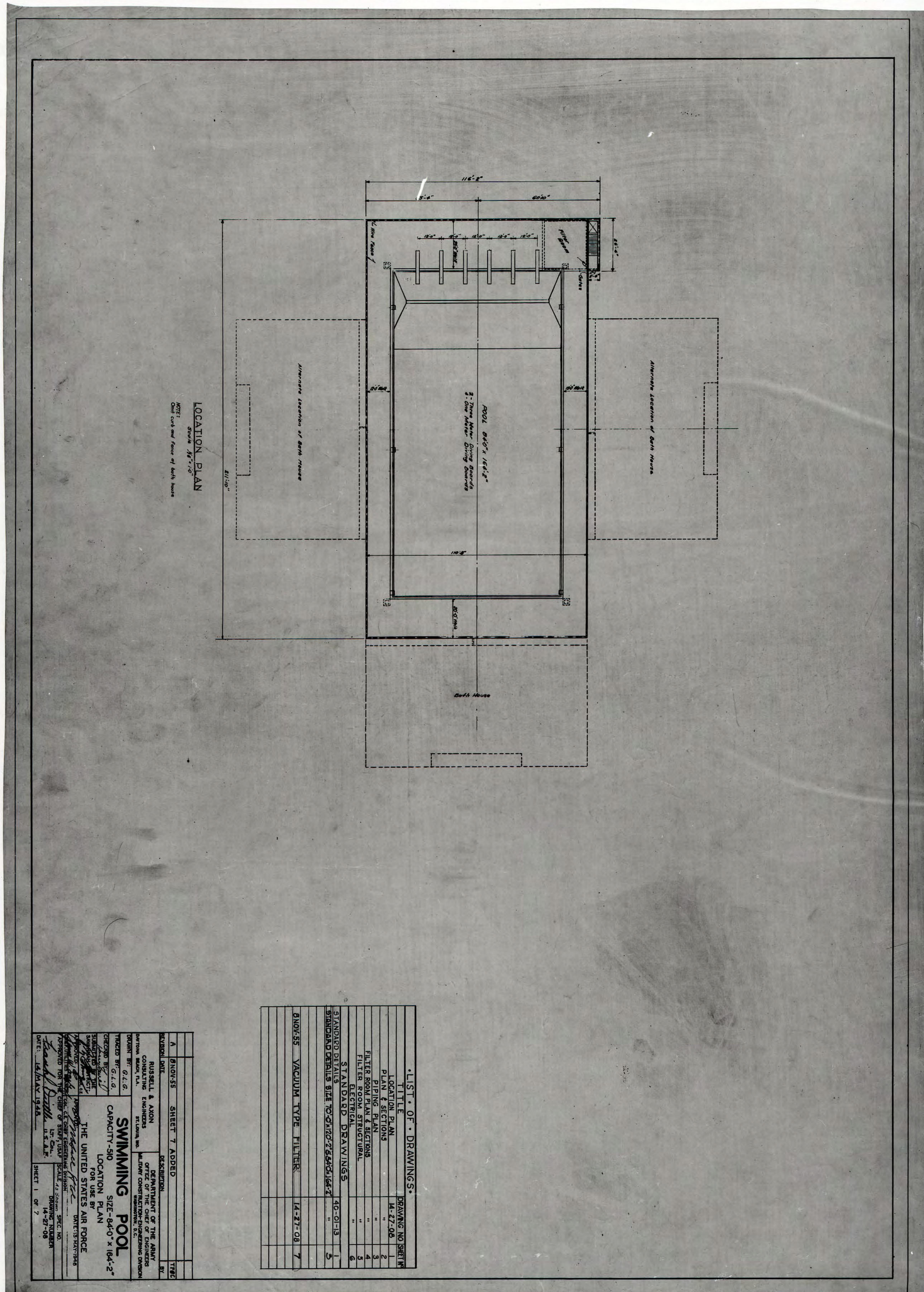
AUGUST 1956 8X

2701 WILSON BOULEVARD
ARLINGTON 1, VA. JAckson 5-1900



80WSE_345 “Swimming Pool,” 1955



80WSE_345 “Swimming Pool - Indoor,” 1955

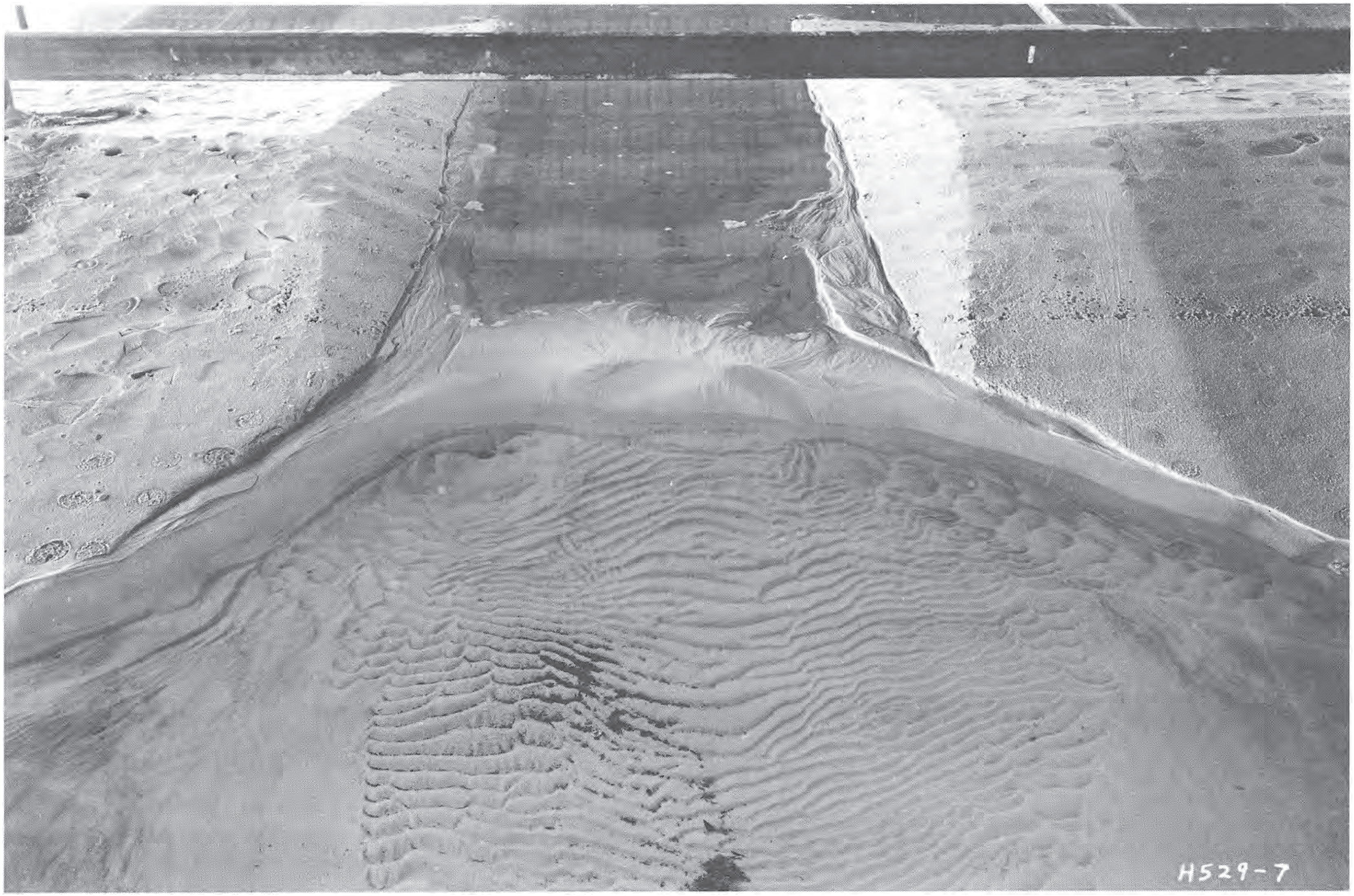
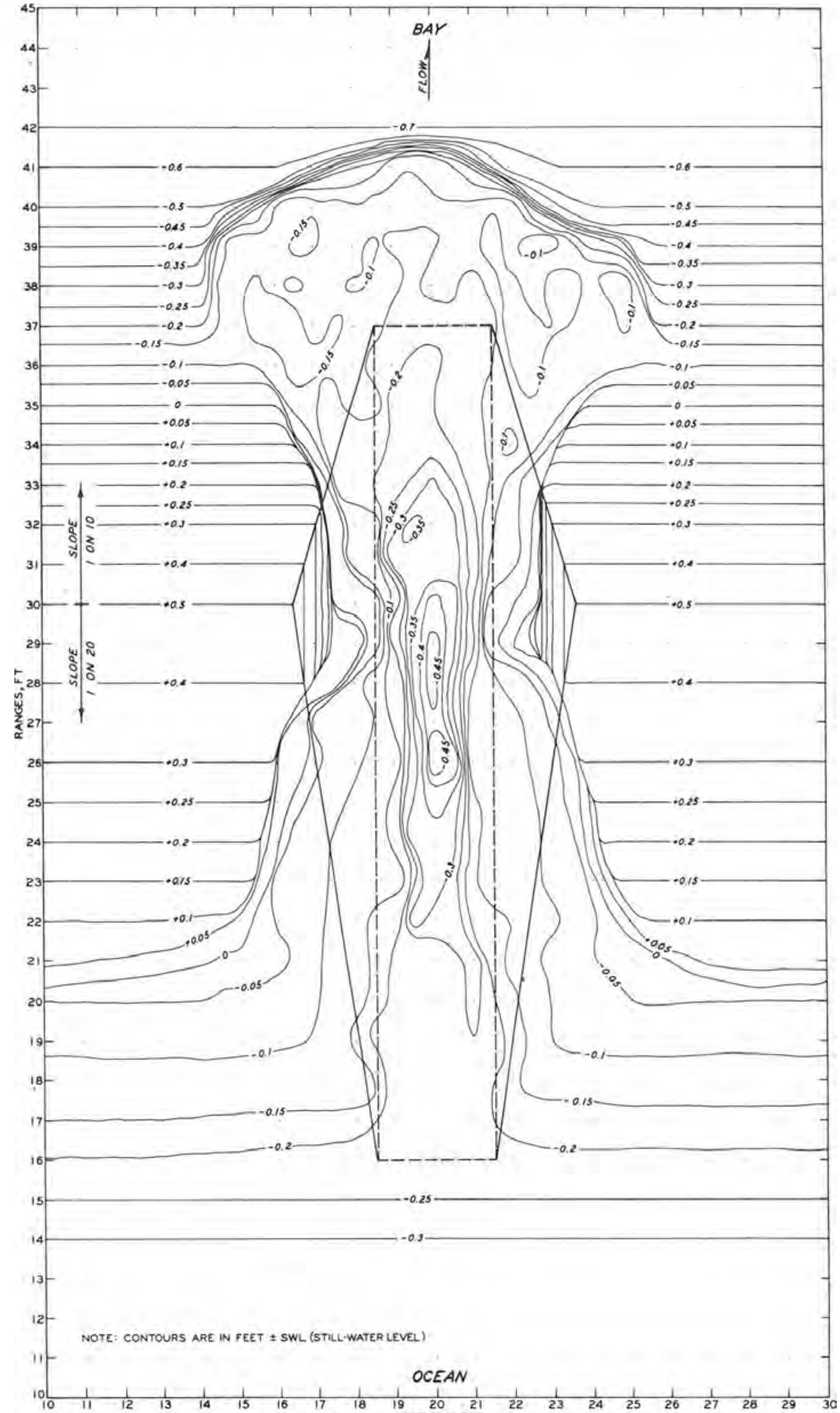


Photo 3. Test 1 after testing 11 hr 25 min with waves only, then 3 hr with waves and ebb flow of 0.1 cfs, then 1 hr with waves only. View from ocean to bay



TEST CONDITIONS

MATERIAL: SAND { SPECIFIC GRAVITY = 2.65, D_{50} = 0.25 mm }

FLOOD FLOW = 0.55 CFS
WAVE HEIGHT (H_0) = 0.065 FT
WAVE PERIOD = 1.0 SEC
WATER DEPTH = 1.0 FT SWL
WAVE DIRECTION = 0° (NORMAL)

HYDROGRAPHIC SURVEY

TEST 4A

CONTOURS AFTER TESTING 7 HOURS WITHOUT WAVES PLUS 24 HOURS WITH WAVES PLUS 7 HOURS WITHOUT WAVES

80WSE_346 "Model Materials Evaluation Sand Tests, Hydraulic Laboratory Investigation," 1976

Special Report 85-5

May 1985

US Army Corps of Engineers

Cold Regions Research & Engineering Laboratory

Workshop on permafrost geophysics

Golden, Colorado, 23-24 October 1984

J. Brown, M.C. Metz and P. Hoekstra, Editors

...eration with
...ON PERMAFROST
...ARCH BOARD
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...N, D.C.

pingo as constructed from elevation surveys and the radar sounding data. A core through the pingo revealed that the ice in it was composed of bands containing different bubble arrangements and geometries as well as sandy ice layers up to 0.7 m thick (Fig. 6)(pers. comm., Bruce Brockett, CRREL). The depth to the bottom of the ice and to the various internal layers was estimated by using an effective velocity of the radiated impulse wavelet in the bubbly ice and overburden of 0.172 m/ns. Using this value, and the wavelet time of flight, the calculated ice depth agreed within 2% with the drill-hole-measured depth of 13.1 m.

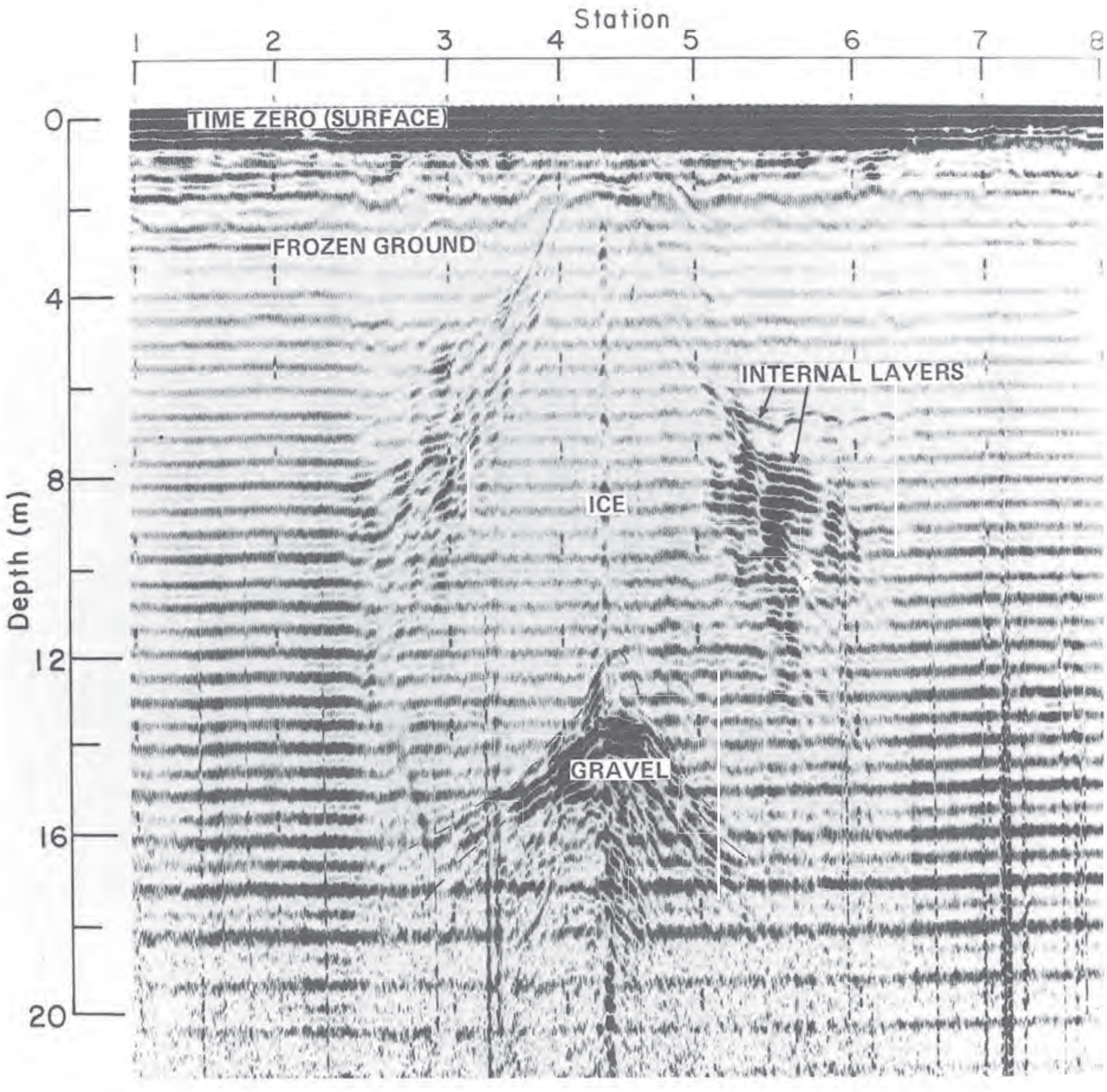


Figure 3. Second radar profile over Weather Pingo using a 300-MHz antenna. Horizontal banding is noise.

US Army Corps
of Engineers
Waterways Experiment
Station

Wetlands Research Program Technical Report WRP-DE-4

A Hydrogeomorphic Classification for Wetlands

by Mark M. Brinson



August 1993 – Final Report

Approved For Public Release; Distribution Is Unlimited

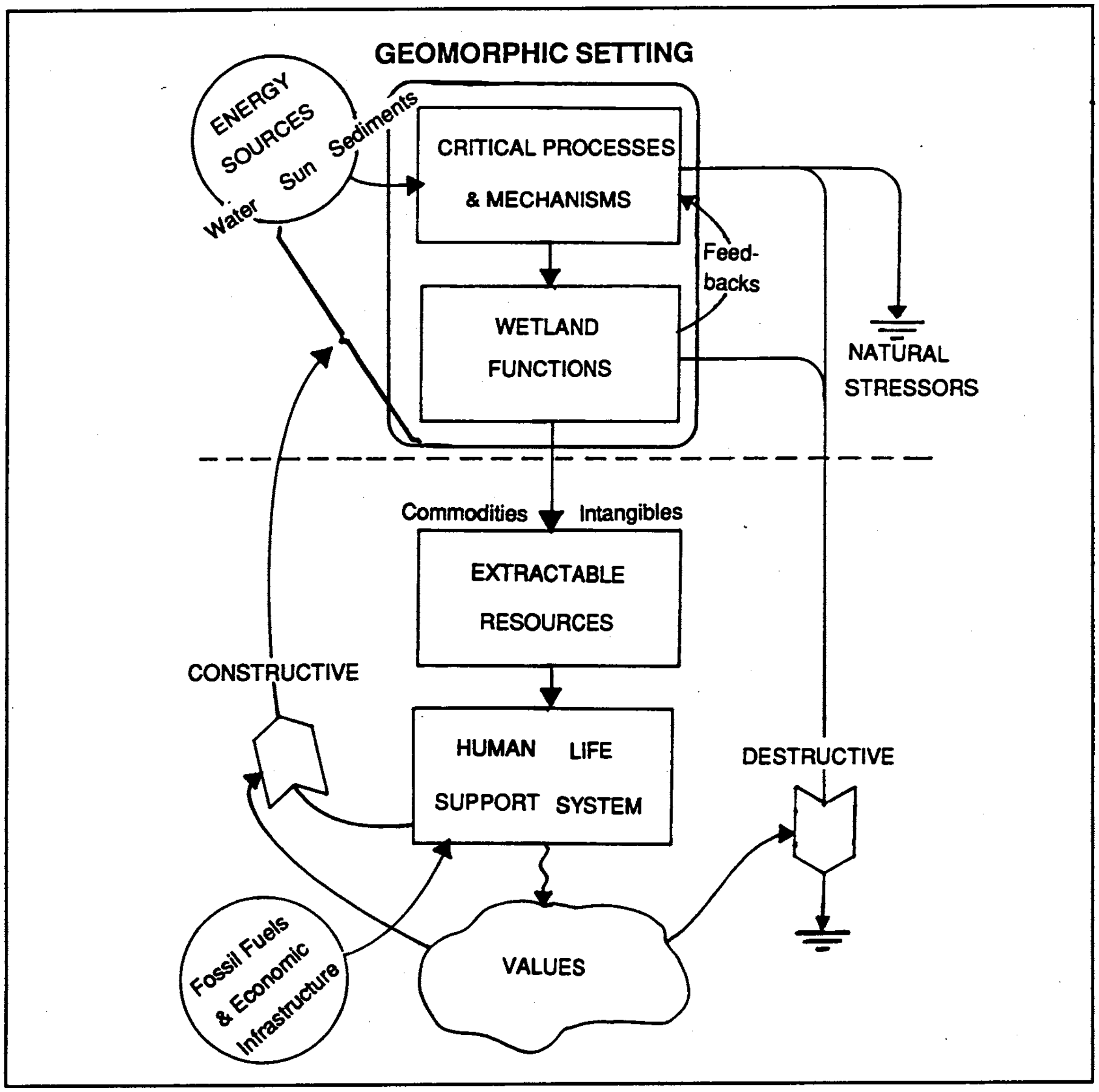
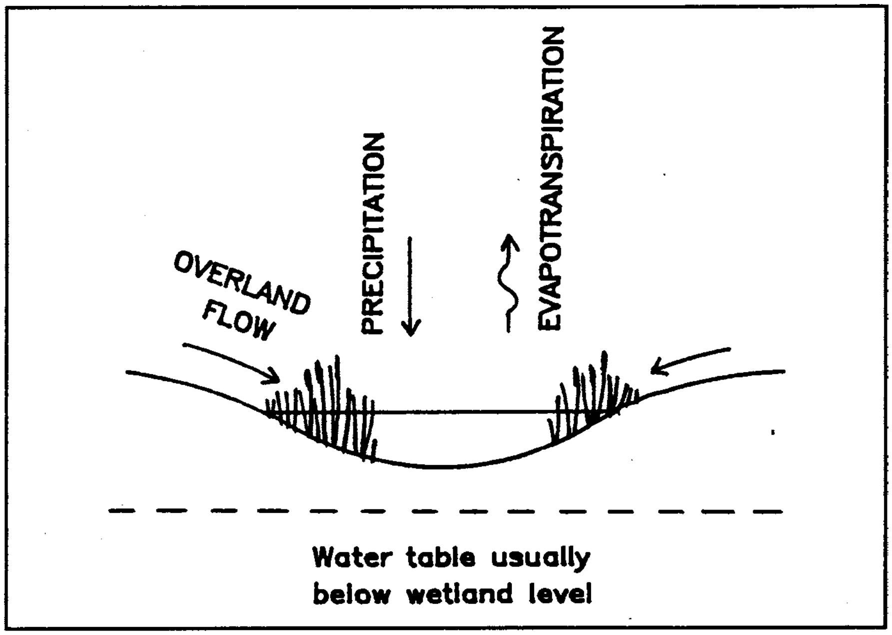
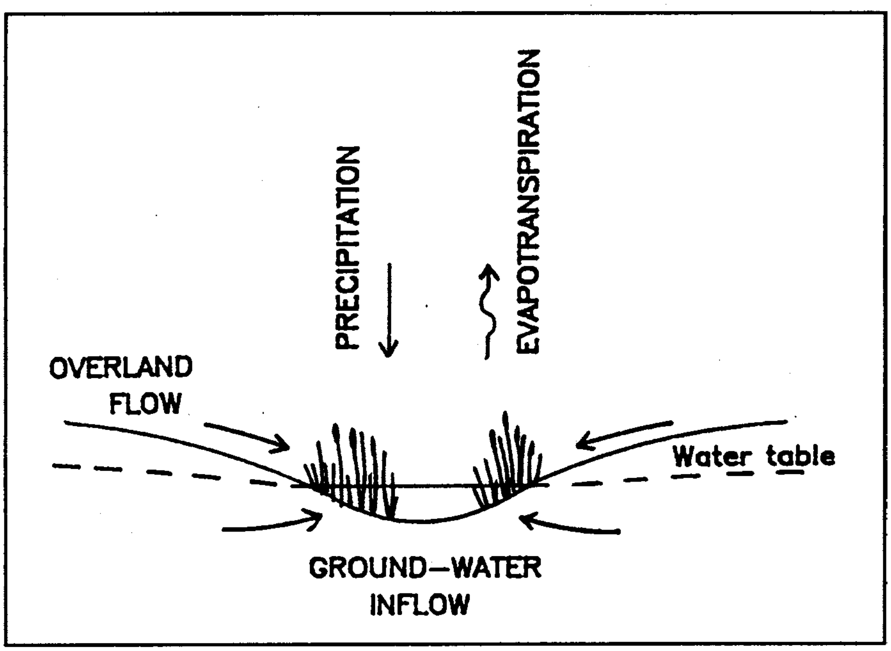


Figure 1. Diagram that conceptualizes the relationship between wetland functions and wetland values. The dashed line separates the geomorphic setting, which contains wetland functions, from societal interaction with wetlands. Items above the dashed line can continue in the absence of society; those below show the "uses" of wetlands by society. Critical processes and mechanisms (e.g., photosynthesis, microbial activity, and decomposition) and ecosystems functions (e.g., primary productivity, biomass accumulation, and nutrient cycling and retention) may become resources for human life support. The term extractable resources is meant to include intangibles, commodities, and all other goods and services that contribute to the human life support system. Note that human life support relies both on wetlands in their geomorphic settings and fossil fuels. Feedbacks initiated by societal values can be either constructive or destructive. While values are merely perceptions, they establish how the life support system interacts with the wetland resource. Adapted and modified from Twilley (personal communication, 1990, University of Southwestern Louisiana, Lafayette, LA), Taylor, Cardamore, and Mitsch (1990), E. Maltby (personal communication, 1990, University of Exeter, Exeter, U.K.), and Whigham and Brinson (1990)

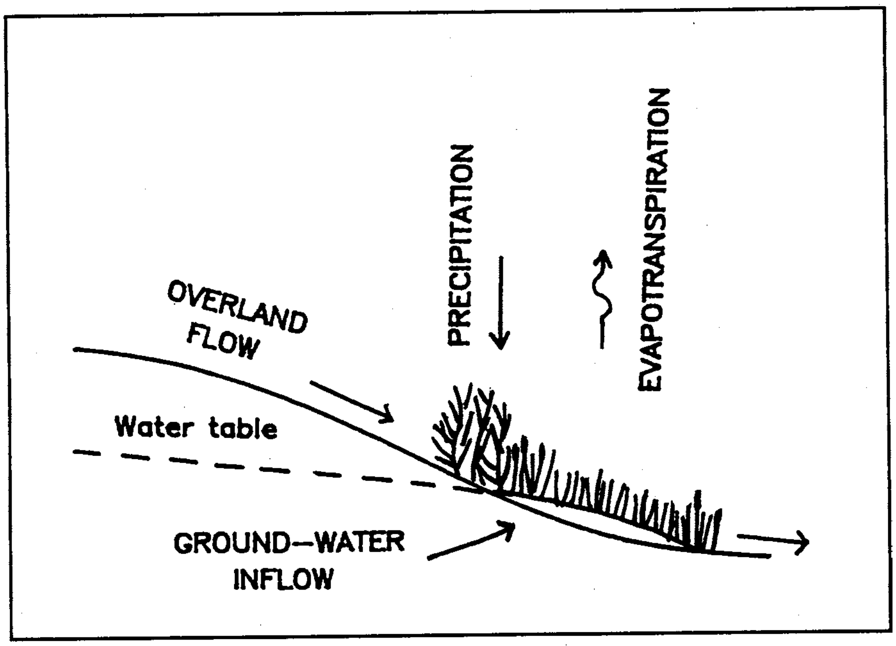


a. Surface water depression

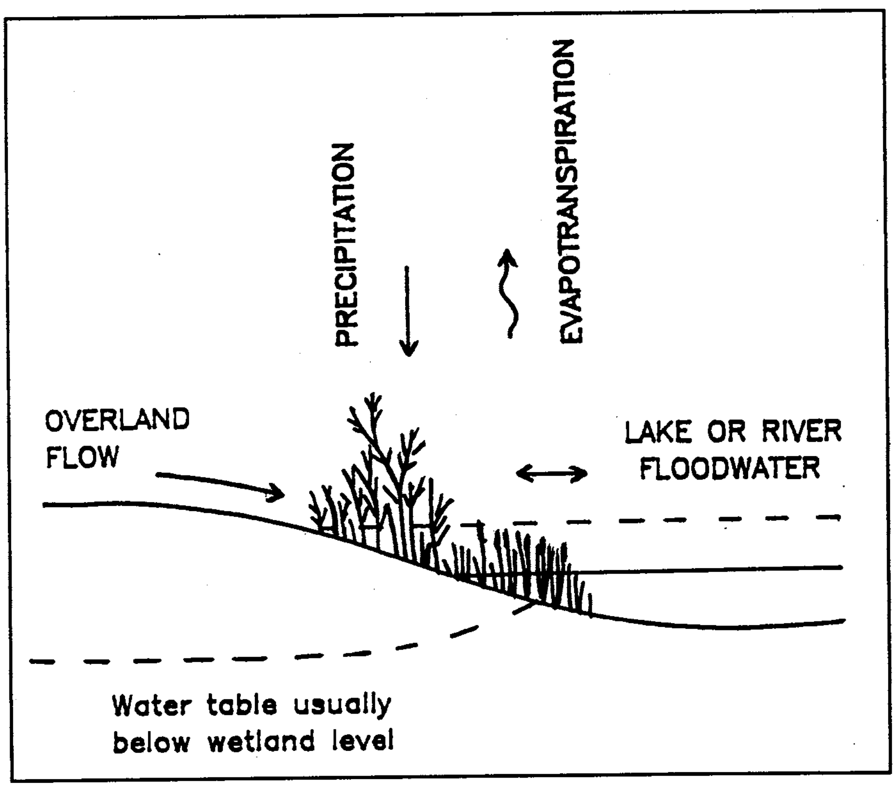


b. Groundwater depression

Figure 2. Four major hydrologic types of wetland types in Wisconsin (Continued)

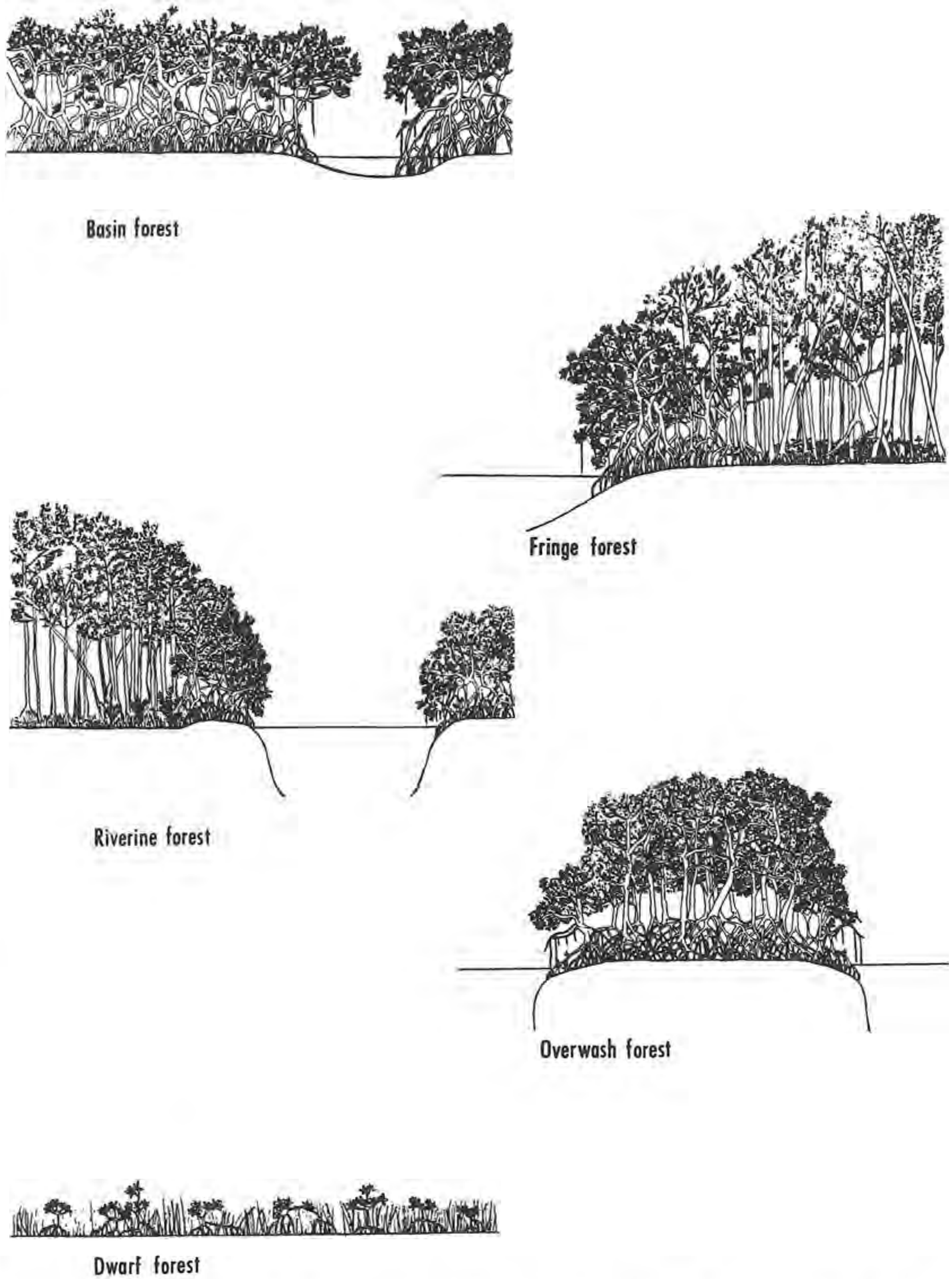


c. Groundwater slope



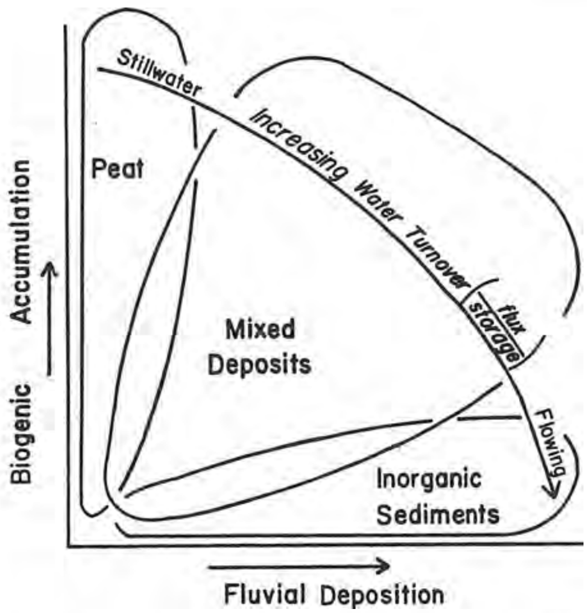
d. Surface water slope

Figure 2. (Concluded)

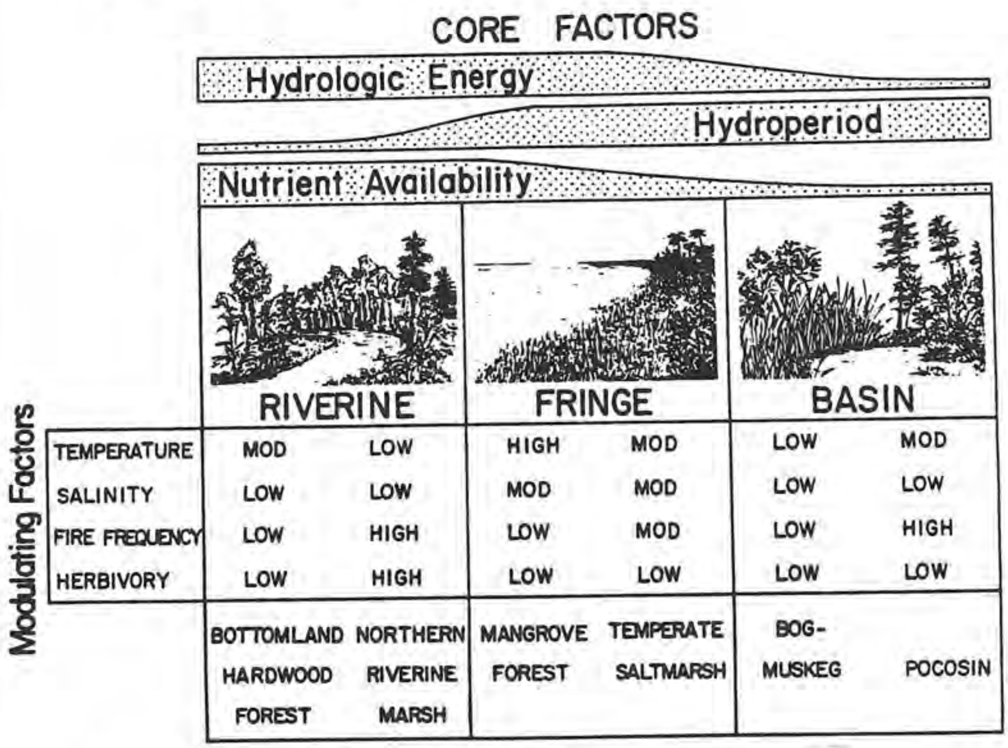


a. Five types proposed for mangroves in Florida by Lugo and Snedaker (1974) as illustrated in Wharton et al. (1977)

Figure 5. Classifications of wetlands based on their functional properties (Sheet 1 of 4)

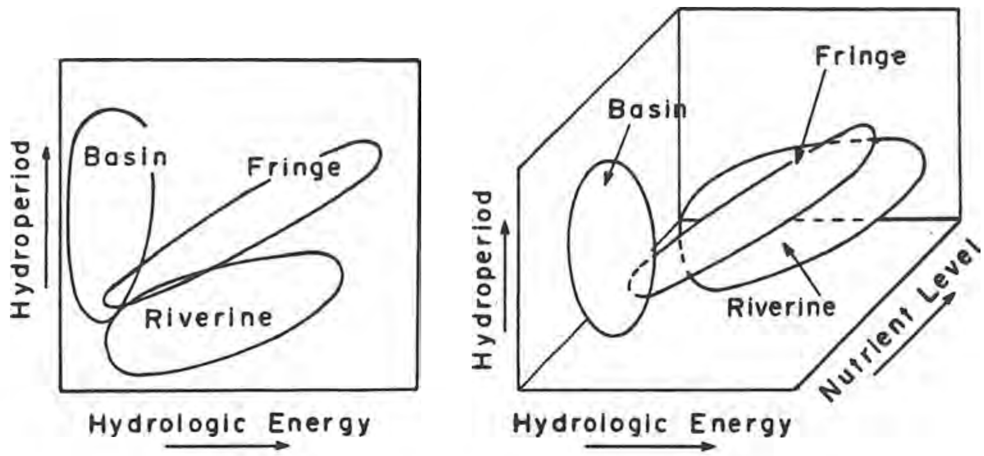


b. Flowing versus still-water proposed by Brown, Brinson, and Lugo (1979) for forested wetlands

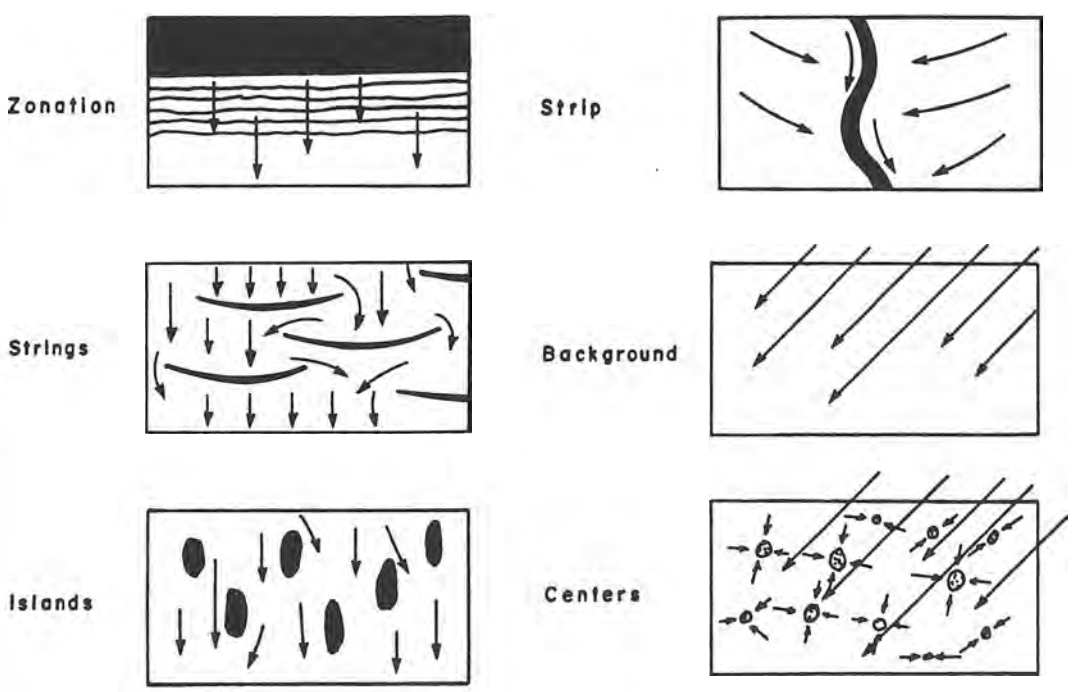


c. The use of modulating factors to characterize specific types of wetlands beyond the core factors (Brinson and Lee 1989)

Figure 5. (Sheet 2 of 4)

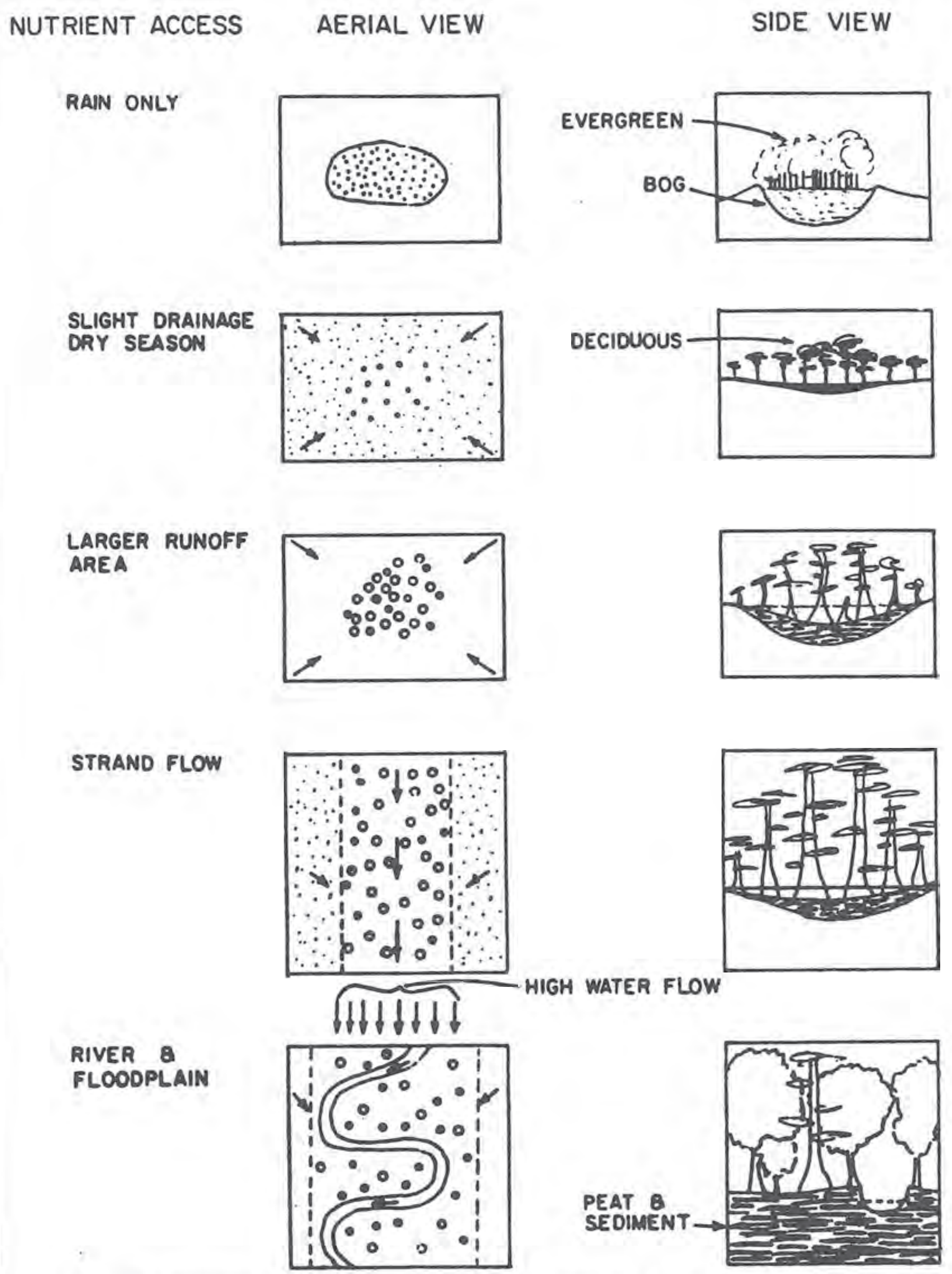


d. Riverine, basin, and fringe arranged around three core factors proposed by Lugo, Brinson, and Brown (1990b)



e. Six generic landform patterns that have equivalents for wetlands (Kangas 1990)

Figure 5. (Sheet 3 of 4)



f. Cypress wetlands of Florida arranged along increasing water and nutrient flows

Figure 5. (Sheet 4 of 4)