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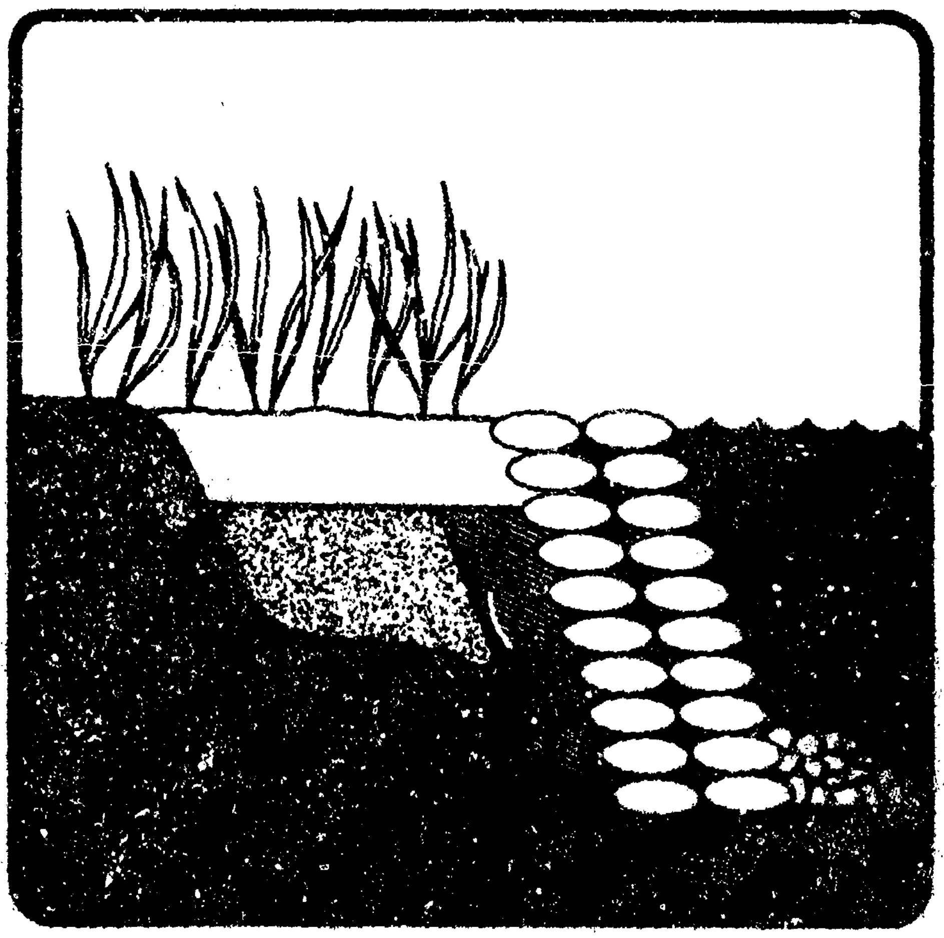
Office of the Chief of Engineers

Low-Cost Shore Protection

Final Report on the Shoreline Erosion Control Demonstration Program (Section 54)

1981



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This report documents the results of a program conducted by the U.S. Army Corps of Engineers to develop and demonstrate low-cost methods of shore protection in accordance with the provisions of Section 54, Public Law 93-251, 93d Congress, approved March 7, 1974. The program objectives were to provide a data base for use in the logical selection of devices or combination of devices to protect inland or sheltered shorelines in any region of the United States, and to develop techniques for making such a selection. Topics

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LOW COST SHORE PROTECTION

A Report on the
Shoreline Erosion Control Demonstration Program
(Section 54 Program)

by the

Office, Chief of Engineers
U.S. Army Corps of Engineers
Washington, D.C. 20314

This investigation authorized by
Section 54 of the
Water Resources Development Act of 1974
(PL 93-251)

1981

PREFACE

This report is the history of the Shoreline Erosion Control Demonstration Program. The program was authorized by Section 54 of Public Law 93-251, the Water Resources Development Act of 1974.

The Section 54 Program was accomplished by 16 Corps of Engineers districts under the supervision of 7 Corps divisions, and with program management by the Office of the Chief of Engineers. The Coastal Engineering Research Center (CERC) provided technical and administrative support to the Office of the Chief of Engineers. CERC provided the contract administration for analysis of the program monitoring data and for the preparation of a draft report by James W. Dunham, Project Manager for Moffatt and Nichol, Engineers, which forms the basis for this report. Mr. John G. Housley, of the Planning Division, Directorate of Civil Works, provided program management for the Office of the Chief of Engineers throughout.

The participating Corps district offices accomplished the planning, design, construction, and monitoring of the demonstration projects within their geographic areas. Where the project was constructed on non-Federal land, a local sponsor provided 25 percent of the construction cost, and took complete authority for the project after the Section 54 Program monitoring was completed. All other costs were borne by the Federal government.

The vegetative components of the demonstration program were accomplished with the advice and assistance of the Soil Conservation Service. Mr. Robert S. MacLauchlan, Chief Plant Materials Specialist, provided invaluable leadership and coordinated the unstinting work of the SCS field personnel.

The work of the dedicated Corps of Engineers professionals, too numerous to credit by name here, made the success of the Section 54 Program possible. The objective of the program is to provide information on low-cost methods of shoreline erosion control to private property owners who must make decisions about how they will treat their erosion problems. Although monitoring of devices in the Section 54 Program for additional time would doubtless have yielded results of increasing value, Congress did not extend the 5-year program. The results that were obtained, however, are significant, and it is important that what has been learned be disseminated to the public in a timely fashion. The knowledge gained will help embattled shore property owners to help themselves.

The Shoreline Erosion Advisory Panel (SEAP), authorized as part of this program to advise the Corps, was appointed by the Chief of Engineers, first met in December 1974, and met for the sixteenth and last time in December 1980. The chairman of the SEAP, Mr. Joseph M. Caldwell, was a primary influence on the activity of the SEAP until his death in December 1980. This report is dedicated to the memory of his outstanding guidance and inspiration.

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EXECUTIVE SUMMARY

This report documents the results of a program conducted by the U.S. Army Corps of Engineers to develop and demonstrate low-cost methods of shore protection in accordance with the provisions of Section 54, Public Law 93-251, 93d Congress, approved March 7, 1974. The objectives of the program were (a) to provide a data base that could be used for the logical selection of devices or combinations of devices to protect inland or sheltered shorelines in any given region of the United States, (b) to develop techniques for making such selection, and (c) to disseminate this information.

As authorized by the legislation, the Chief of Engineers appointed a Shoreline Erosion Advisory Panel (SEAP) to advise him on the conduct of the program. The Panel included no Federal employees, and represented various geographical areas, institutions, backgrounds, and organizations. All the members were knowledgeable in some aspect of shoreline erosion. The Chief of Engineers selected 10 sites from those recommended by the SEAP, in addition to the 6 sites in Delaware Bay that were mandated by the law, where the various shore protection devices were to be demonstrated. These devices were selected from a large listing of low-cost shore protection devices, including vegetation, which had been tried or proposed in the past. The selection of devices to be demonstrated at specific sites was made by the Chief of Engineers with the concurrence of non-Federal sponsors, who were required by law to pay at least 25 percent of construction costs where a project site was located on non-Federal land. These devices were then installed and monitored for effectiveness by the Corps. Each Corps of Engineers District in which a demonstration project was undertaken implemented a public information program to make the local public aware of the purpose and objectives of that project.

A number of low-cost shore protection devices, already installed at other sites under other programs, were also monitored by the Corps under the Shoreline Erosion Control Demonstration Program. These additional sites allowed for the observation and evaluation of a larger number of devices and environmental conditions than had been possible with only the 16 mandated sites.

An important objective of the program was to demonstrate the effectiveness of vegetation as a low-cost, environmentally responsible shore protection device, either by itself in low wave-energy areas or in conjunction with structural devices in areas subjected to the impact of higher waves. Much of the vegetative work was done by the Soil Conservation Service of the Department of Agriculture. The indigenous vegetation species at each site were examined and those considered suitable for shore protection were incorporated into many of the projects. Selection of the species was based on knowledge of plants likely to adapt to local conditions.

Monitoring of the sites included collection of several types of data. Local personnel trained under the Corps Littoral Environmental Observations (LEO) program made observations at each site either once or twice daily, depending on the site. These wind, wave, and current data were used to evaluate the performance of the structures. Bathymetric surveys, made before installation of devices and periodically thereafter, were used to evaluate beach and offshore profile changes and to trace the movement of littoral drift. Aerial photography, flown at about 3-month intervals, was used to trace changes in shoreline position and to study beach and backshore terrain changes. Sediment samples collected from beach and offshore areas were analyzed to determine grain size and other characteristics of littoral sediments. Each site was visited monthly by the Corps District project engineer, who took photographs of the installations and reported changed conditions, such as damages from storms. Special reports were prepared on vegetation plantings, which included survival counts, performance of plantings, and effectiveness of fertilization efforts.

The monitoring data were analyzed by Moffatt and Nichol, Engineers, and a draft report was prepared. The shore protection systems evaluated, code-marked to indicate degree of success and locations of sites, are included in a listing at the end of this summary.

Of the various materials investigated, it was found that quarrystone performed well and survived longest, but it could be rather expensive for use in low-cost shore protection structures. Asphalt mastic, though costly, proved effective in sealing voids in quarrystone groins to make them sand-tight. Portland cement concrete was also long lasting when adequately quality controlled, and it performed well in properly designed structures and in devices using concrete modules. Concrete rubble provided satisfactory shore revetments only when used in large quantity and when provided with adequate filtering material. Shape-sizing, to reduce elongated pieces or large flat slabs and to eliminate small pieces and debris, improved performance. The use of concrete rubble for shore protection was considered a good way to dispose of large quantitites of this material, but care must be exercised to assure adequacy of design and proper construction.

Timber proved to be a very useful material because of the ease with which it can be shaped and fastened together for fabrication of a number of types of shore protection structures. It should have a good preservative treatment to assure long life, especially where ground contact exists. Proper design and installation of bolts and other fasteners were required to prevent structural failure. Structures designed to retain fills required sand-tightness, which was best provided by tongue-and-groove sheet piles or backing of sheathing planks with filter material.

The gabion basket, when filled with stone larger than 4 inches in diameter, proved useful only for revetments in sites exposed to relatively low wave energy. Gabion structures built in or extended into deeper water were badly damaged by wave action and wave-borne debris; some baskets which were ripped open and from which the stone fill was lost still performed well. At sites where large stone for fill was not available, liners of wire screening and filter cloth were used to retain small stones and gravel fill. None of these liners survived battering by waves, and most of the fill was washed out of the baskets. Periodic refilling of the baskets appears to be essential to gabion survival.

Revetments of used steel fuel barrels bolted together and filled with gravel were very effective in arctic regions of Alaska where the barrels have little salvage value and accumulate in large quantities. When used as groins, however, their outer ends were badly damaged by ice and debris. Alaskan experience south of the Arctic Circle indicated that the barrels tend to rust out in a few years.

Longard tubes were effective as low breakwaters, bulkheads, and groins, but at every site they were badly damaged either by vandalism or by floating debris. When placed as bulkheads at the base of steep bluffs, overtopping waves and the unstable bluff slopes caused the bluffs to slide, pushing or rolling the tubes and distorting and tearing the fabric. A sand-epoxy coating of exposed surfaces helped to minimize damage caused by vandalism. The coating was not effective in reducing damage caused by debris. The tubes have proved successful in emergency situations because of the speed with which they can be placed and filled, but their vulnerability to damage makes their effectiveness as permanent shore protection structures questionable.

Filter cloth made of synthetic fibers proved very useful in reducing settlement of structures in soft material and in preventing loss of embankment through revetments and loss of backfill through bulkheads, seawalls, and sills. In revetments, its porosity prevented buildup of pore pressure and resultant slope failure, but the type of cloth used appeared to be an important factor. The woven types with distinct and uniform openings tended to perform better than the nonwoven types, although insufficient comparisons were made to conclusively determine this. Filter cloth exposed to direct wave impact after removal of overlying protective covers usually did not survive long. At one site, a covering of woven filter cloth over a sandbag breakwater survived well through several high wave episodes in a debris-free environment, but the observations ended before a true indication was given of what the life of the cloth might be under these conditions.

Sandbags proved most effective in revetments, breakwaters, sills, and groins when filled with a sand-cement mixture. Most fabrics have a relatively short life when exposed alternately to sunlight and wave impact, but the sand-cement hardens into concrete modules that hold their shape and bond together after the fabric deteriorates. Here again, adequacy of design proved to be an important factor. One revetment of one-bag thickness on a 60° slope failed, while another revetment of two-bag thickness on a 45° slope survived. Some breakwaters, sills, and groins constructed of large nylon bags filled in place with sand performed reasonably well. Two groins of such construction have survived several years, although the fabric is now deteriorating. None of the small sandfilled bag structures survived the first season of exposure. Such modules are too small for stability under moderate wave action, and without the bonding that results from the sand-cement process, the bags are readily displaced by wave forces.

Discarded rubber tires were used successfully in some structures and failed in others. This method of disposing of the large quantity of used tires now accumulating in the United States can be effective in shoreline protection and environmentally acceptable if done properly. The tires are too light and bulky for stacking by themselves in fixed shore protection structures and must be filled with heavier material and securely fastened together for satisfactory performance. Floating tire breakwaters adequately attenuate the short-period waves that predominate along most inland or sheltered shorelines, but more development is needed to devise an effective, low-cost method of interconnecting the modules. The results of this program will provide a base for further experimentation that may ultimately lead to the development of reliable tire shore protection systems.

Artificial beach fills and the use of sills to retain perched beaches proved to be very effective as well as environmentally responsible. The possibility of using each of these systems for shore protection, either alone or in combination, at any given site merits first priority in the system selection process. The circumstances that make for effective use of these systems are discussed in this report.

The establishment of vegetation as a primary means of shore protection was successful only in very low wave-energy areas where underlying soil types provided required stability for the plants and where weather conditions were amenable to sustained plant growth. The obvious advantage of vegetation is its low initial cost. The effectiveness of the vegetation program varied mainly with the amount of wave reduction provided by the physiography and offshore bathymetry of the site or with the degree of wave attenuation provided by structural devices. The best results were obtained where the veneer of sand was thin and overlayed loam or peat. The most successful plantings were made with large plants, i.e., plugs or pots for cordgrass and trees with prop roots for mangroves. This is related to their stability. Well-designed fertilization procedures accelerated plant growth and penetration of roots into substrate that could sustain and proliferate the vegetative cover. Types of vegetation that proved most effective were: smooth cordgrass (Spartina alterniflora) on the Atlantic and gulf coasts and tidewater areas; Pacific cordgrass (Spartina foliosa) on the Pacific coast; saltmeadow cordgrass (Spartina patens) in the wave uprush zone; and American beachgrass (Ammophilia breviligulata) above the tidal zone. Other types of vegetation showed promise of becoming effective under special conditions in more restricted geographical locations. Full cover had not developed at any but the monitoring-only sites at the conclusion of the program.

Although the monitoring period was too short for final judgment of the effectiveness and longevity of many of the systems monitored in this program, the findings in many instances are conclusive and early trends noted in other instances are indicative of future performance. The results of the program, and the suggestions and recommendations contained in this report, should be useful wherever shore protection along inland shorelines or sheltered shores is needed.

Locations and Performance of Systems Investigated

Bulkheads and Seawalls	
Treated timber[1]	Oak Harbor, Wash.; Buckroe Beach, Va.; Folly Beach, S.C.
Steel and timber[1]	Port Wing, Wis.
Concrete sheet pile[1]	Folly Beach, S.C.
Rubber tire and post[2]	Oak Harbor, Wash.
Longard tube[2]	Ashland, Wis.; Sanilac Section 11, Mich.
Earthfilled concrete pipe[2]	Beach City, Tex.
Rubber tire stack[2]	Port Wing, Wis.
Untreated timber[2]	Oak Harbor, Wash.; Ashland, Wis.
Hogwire fence and sandbags	Basin Bayou State Recreation Area, Fla.
Concrete and timber	Folly Beach, S.C.
Revetments	
Stone riprap[1]	Folly Beach, S.C.; Muskegon State Park, Mich.; Tawas Point, Mich.
Sand-cement-filled bags[1]	Alameda, Calif.; Oak Harbor, Wash.
Concrete blocks[2]	Fontainebleau State Park, La.; Port Wing, Wis.; Stuart-Jensen Beach Causeways, Fla.; Holly Beach, La.; Little Girls Point, Mich.
Gabions[2]	Kotzebue, Alaska; Ninilchik, Alaska; Oak Harbor, Wash.
Concrete rubble[2]	Alameda, Calif.; Shoreacres, Tex.
Steel fuel barrels[2]	Kotzebue, Alaska
Concrete slabs	Alameda, Calif.
Sandfilled bags	Alameda, Calif.
Fabric	Fontainebleau State Park, La.; Alameda, Calif.
Tire and fabric	Fontainebleau, La.
Breakwaters and Sills	
Stone rubble[1]	Kitts Hummock, Del.; Siuslaw River, Oreg.
Timber sheet piles[1]	Slaughter Beach, Del.
Tires on piles[1]	Fontainebleau State Park, La.
Sand-cement-filled bags[1]	Fontainebleau State Park, La.; Alameda, Calif.
Floating tires[2]	Pickering Beach, Del.; Stuart-Jensen Beach Causeways, Fla.
Longard tubes[2]	Alameda, Calif.; Basin Bayou State Recreation Area, Fla.
Gabions[2]	Geneva State Park, Ohio
Concrete boxes[2]	Kitts Hummock, Del.; Slaughter Beach, Del.
Z-wall[2]	Geneva State Park, Ohio
Surgebreaker[2]	Basin Bayou State Recreation Area, Fla.
Sandgrabber[2]	Basin Bayou State Recreation Area, Fla; Folly Beach, S.C.; Kualoa, Hawaii; Bellows Air Force Station, Hawaii
Sta-pods	Geneva State Park, Ohio

Locations and Performance of Systems Investigated (Continued)

Breakwaters and Sills (Continued)

Sandfilled bags	Basin Bayou State Recreation Area, Fla.; Kitts Hummock, Del.; Slaughter Beach, Del.; Buckroe Beach, Va.
Brush dike	Fontainebleau State Park, La.

Groins

Timber[1]	Broadkill Beach, Del.; Ninilchik, Alaska; Buckroe Beach, Va.; Lincoln Township, Mich.
Timber and rock[1]	Folly Beach, S.C.; Sanilac Section 26, Mich.
Stone rubble[1]	Siuslaw River, Oreg.
Concrete rubble[1]	Broadkill Beach, Del.
Sand-cement-filled bags[1]	Alameda, Calif.
Corrugated metal pipe[1]	Ninilchik, Alaska
Rock asphalt mastic[1]	Sanilac Section 26, Mich.
Longard tubes[2]	Ashland, Wis.; Lincoln Township, Mich.; Sanilac Section 26, Mich.
Gabions[2]	Kotzebue, Alaska; Ninilchik, Alaska; Sanilac Section 26, Mich.
Steel fuel barrels[2]	Kotzebue, Alaska
Sandfilled bags	Alameda, Calif.; Bowers, Del.; Kotzebue, Alaska; Sanilac Section 26, Mich.

Nonstructural

Perched beach[1]	Alameda, Calif.; Slaughter Beach, Del.
Beach fill[1]	Alameda, Calif.; Bowers, Del.; Broadkill Beach, Del.; Lewes, Del.; Muskegon State Park, Mich.; Sunnyside Beach, Wash.
Artificial seaweed (not monitored)	Roanoke Island, N.C.
Vegetation alone[2]	Pickering Beach, Del.; Slaughter Beach, Del.; Kitts Hummock, Del.; Fontainebleau State Park, La.; Basin Bayou State Recreation Area, Fla.; Roanoke Island, N.C.; Duck, N.C.; Hampton Natural Wildlife Refuge, Va.; Key West, Fla.; Uncle Henry's Fish Camp, N.C.; Alameda, Calif.; Stuart-Jensen Beach Causeways, Fla.
Vegetation with structure[2]	Alameda, Calif.; Basin Bayou State Recreation Area, Fla.; Geneva State Park, Ohio; Oak Harbor, Wash.; Key West, Fla.; Roanoke Island, N.C.; Bogue Sound, N.C.; Stuart-Jensen Beach Causeways, Fla.

[1] Systems that proved successful.

[2] Systems that could be made successful with minor changes or that should be used only in special environments or circumstances.

Note — Unmarked systems are those that failed structurally or functionally.

LOW-COST SHORE PROTECTION

I. INTRODUCTION

1. History and Objectives of the Shoreline Erosion Control Demonstration Act.

For many years, the coastal engineering profession has been designing shore protection systems to resist wave erosion in critical stretches of coastline exposed to high waves. Through model experiments and observation of actual structures, the state-of-the-art for design and construction of such open-coast works has advanced considerably, but the price for these structures is high. Where private property owners have shore frontage with exposure to less violent waves, there is seldom the need for or the resources available to build massive structures. However, little guidance has been provided for the design of smaller or less costly protective structures to meet their needs and fit their pocketbooks. Many mistakes have been made and failures of homemade and improperly designed and constructed protective works have been numerous.

The need for more definitive information on shoreline erosion control techniques was highlighted in the National Shoreline Study, a report by the U.S. Army Corps of Engineers (1971). That study showed that of the 84,000 miles of U.S. shoreline investigated, more than 20,000 miles was undergoing significant erosion. (Based on 1970 prices, to stop the erosion on the most critically eroding 2,700 miles would require an initial expenditure of $1.8 billion.) The shoreline of the United States (excluding Alaska) is about 70 percent in private ownership. Thus, the bulk of the shoreline erosion problem is a nonpublic one, and as such is an area of concern to persons who are not eligible for relief of the problem under existing Federal laws. The use to which an owner puts his piece of property with an eroding shoreline is one of the key factors when trying to determine an appropriate approach to a shoreline erosion control problem. If the land is in agricultural use, or is a homestead, or is highly developed for a commercial use, three different sets of values will obtain. The landowner must choose from among a variety of options (including relocation, abandonment, massive structural defenses, etc.) depending on his resources and on several intangible considerations. The Shoreline Erosion Control Demonstration Program is an outgrowth of the Congress's concern that the private property owner has available to him specific information on low-cost methods to help him help himself.

The Shoreline Erosion Control Demonstration Act of 1974, Section 54, Public Law 93-251, authorized a 5-year program to develop, demonstrate, and disseminate knowledge about low-cost methods of shore protection. As a result, the Secretary of the Army, acting through the Chief of Engineers, conducted a national Shoreline Erosion Control Demonstration Program consisting of planning, designing, constructing, monitoring, and evaluating shoreline erosion control devices, both structural and vegetative, for erosion problems on the shores of sheltered or inland waters. Technical assistance with the vegetative aspects of the program was provided by the Department of Agriculture. The objectives of the program were (a) to provide a resource that could be used for the logical selection of devices or combinations of devices to protect inland or sheltered shorelines in various coastal areas or Great Lakes of the United States, (b) to develop techniques for making such selection, and (c) to disseminate this information. The full text of the Congressional Act is presented as follows:

Public Law 93-251, 93d Congress (88 STAT. 26-28)
Approved March 7, 1974

Section 54. (a) This section may be cited as the "Shoreline Erosion Control Demonstration Act of 1974".

(b) The Congress finds that because of the importance and increasing interest in the coastal and estuarine zone of the United States, the deterioration of the shoreline within this zone due to erosion, the harm to water quality and marine life from shoreline erosion, the loss of recreational potential due to such erosion, the financial loss to private and public landowners resulting from shoreline erosion, and the inability of such landowners to obtain satisfactory financial and technical assistance to combat such erosion, it is essential to develop, demonstrate, and disseminate information about low-cost means to prevent and control shoreline erosion. It is therefore the purpose of this section to authorize a program to develop and demonstrate such means to combat shoreline erosion.

(c) (1) The Secretary of the Army, acting through the Chief of Engineers, shall establish and conduct for a period of five fiscal years a national shoreline erosion control development and demonstration program. The program shall consist of planning, constructing, operating, evaluating, and demonstrating prototype shoreline erosion control devices, both engineered and vegetative.

(2) The program shall be carried out in cooperation with the Secretary of Agriculture, particularly with respect to vegetative means of preventing and controlling shoreline erosion, and in cooperation with Federal, State, and local agencies, private organizations, and the Shoreline Erosion Advisory Panel established pursuant to subsection (d).

(3) Demonstration projects established pursuant to this section shall emphasize the development of low-cost shoreline erosion control devices located on sheltered or inland waters. Such projects shall be undertaken at no less than two sites each on the shorelines of the Atlantic, Gulf and Pacific coasts, the Great Lakes, and the State of Alaska, and at locations of serious erosion along the shores of Delaware Bay, particularly at those reaches known as Pickering Beach, Kitts Hummock, Bowers, Slaughter Beach, Broadkill Beach, and Lewes in the State of Delaware. Sites selected should to the extent possible, reflect a variety of geographical and climatic conditions.

(4) Such demonstration projects may be carried out on private or public lands except that no funds appropriated for the purpose of this section may be expended for the acquisition of privately owned lands. In the case of sites located on private or non-Federal public lands, the demonstration projects shall be undertaken in cooperation with a non-Federal sponsor or sponsors who shall pay at least 25 percent of construction costs at each site and assume operation and maintenance costs upon completion of the project.

(d) (1) No later than one hundred and twenty days after the date of enactment of this section the Chief of Engineers shall establish a Shoreline Erosion Advisory Panel. The Chief of Engineers shall appoint fifteen members to such Panel from among individuals who are knowledgeable with respect to various aspects of shoreline erosion, with representatives from various geographical areas, institutions of higher education, professional organizations, State and local agencies, and private organizations, except that such individuals shall not be regular full-time employees of the United States. The Panel shall meet and organize within ninety days from the date of its establishment, and shall select a Chairman from among its members. The Panel shall then meet at least once each six months thereafter and shall expire ninety days after termination of the five-year program established pursuant to subsection (c).

(2) The Panel shall-

(A) advise the Chief of Engineers generally in carrying out provisions of this section;

(B) recommend criteria for the selection of development and demonstration sites;

(C) recommend alternative institutional, legal, and financial arrangements necessary to effect agreements with non-Federal sponsors of project sites;

(D) make periodic reviews of the progress of the program pursuant to this section;

(E) recommend means by which the knowledge obtained from the project may be made readily available to the public; and

(F) perform such functions as the Chief of Engineers may designate

(3) Members of the Panel shall, while serving on business of the Panel be entitled to receive compensation at rates fixed by the Chief of Engineers, but not in excess of the maximum rate of pay for grade GS-18, as provided in the General Schedule under section 5332 of title 5 of the United States Code, including travel time, and while away from their homes or regular places of business, they may be allowed travel expenses, including per diem in lieu of subsistence, as authorized by law (5 U.S.C. 73b-2) for persons in Government service employed intermittently.

(4) The Panel is authorized, without regard to the civil service laws, to engage such technical and other assistance as may be required to carry out its functions.

(e) The Secretary of the Army, acting through the Chief of Engineers, shall prepare and submit annually a program progress report, including therein contributions of the Shoreline Erosion Advisory Panel, to the Committees on Public Works of the Senate and House of Representatives. The fifth and final report shall be submitted sixty days after the fifth fiscal year of funding and shall include a comprehensive evaluation of the national shoreline erosion control development and demonstration program.

(f) There is authorized to be appropriated for the first fiscal year following enactment of this section, and the succeeding four fiscal years, a total of not to exceed $8,000,000 to carry out the provisions of this section.

*****

2. Administration of the Program.

a. Corps of Engineers Responsibility. Overall responsibility for the program, which involved site selection, determination of which devices to be tested at each site, and the installation of those devices, was that of the U.S. Army Corps of Engineers. The Corps was also responsible for monitoring the performance of the devices, for assessing the results of the program, and for disseminating the findings.

b. Department of Agriculture Role. The authorizing act specified that the Secretary of Agriculture shall cooperate with the Secretary of the Army in carrying out the demonstration program, particularly with respect to vegetation as a device for preventing or controlling shoreline erosion. The Secretary of Agriculture designated the Administrator of the Soil Conservation Service (SCS) as that Department's point of contact, who in turn appointed the Chief Plant Material Specialist as the responsible officer to work with the Chief of Engineers in the Program. A number of plant material specialists in the various SCS regional and state offices served as advisors for the specific demonstration projects, and worked directly with the U.S. Army Engineer District's project engineers. The SCS personnel advised and assisted the Corps, and applied their considerable expertise in helping to develop, demonstrate, monitor, and evaluate the use of vegetation as a shoreline erosion control device. Although the Corps was responsible for the overall conduct of the program, the SCS's effort greatly enchanced the program and enabled, by its continuing advice, the project engineer to satisfactorily plant, fertilize, cultivate, sample, and then evaluate the effectiveness of vegetation as a shoreline erosion control device. In a number of cases, the SCS Plant Materials Center provided the plants. SCS personnel directly supervised the planting and fertilization, and trained people to be effective observers for the monitoring phase.

c. Shoreline Erosion Advisory Panel Role. The legislation also authorized the establishment of a 15-member Shoreline Erosion Advisory Panel (SEAP). Panel members, none of whom were to be Federal employees, were selected by the Corps from various geographical areas, institutions, backgrounds, and organizations, with consideration being given to their interests in different aspects of shoreline erosion. The responsibilities of the panel are to:

(1) Offer general advice to the Chief of Engineers to assist him in performing the provisions of the act;

(2) recommend criteria for the selection and development of demonstration sites;

(3) recommend alternative institutional, legal, and financial arrangements necessary to reach agreeements with non-Federal sponsors of project sites;

(4) periodically review the progress of the program;

(5) recommend means of disseminating results of the program to the program to the public; and

(6) perform other tasks as requested by the Chief of Engineers.

The 1980 SEAP membership is as follows:

*Mr. Joseph M. Caldwell, Chairman: Consultant on Coastal Engineering, (former Technical Director of the Coastal Engineering Research Center and Chief of the Engineering Division in the Office of the Chief of Engineers, U.S. Army) Alexandria, Va.

**Dr. Billy L. Edge, Vice Chairman: Associate Professor of Civil Engineering, Clemson University, S.C.

Mr. Robert C. Baum: National Association of Conservation Districts, Salem, Oreg.

Dr. E. William Behrens: University of Texas, Marine Science Institute, Galveston, Tex.

Dr. Ernest F. Brater: Department of Civil Engineering, University of Michigan, Ann Arbor, Mich.

Mr. John S. Habel: Head, Marine Technology Branch, California Department of Boating and Waterways, Sacramento, Calif.

Prof. Joe W. Johnson: Consultant on Coastal Engineering (Professor of Hydraulic Engineering Emeritus, University of California) Berkeley, Calif.

Dr. Lee E. Koppelman: Executive Director, Nassau-Suffolk Regional Planning Board, Hauppauge, N.Y.

Mr. Omar J. Lillevang: Consulting Engineer (International Consultant on Shore Structures) Whittier, Calif.

Mr. William D. Marks: Chief, Water Development Service Division, Michigan Department of Natural Resources, Lansing, Mich.

Ms. Evelyn L. Pruitt: Coastal Geographer (former Director of Geography Programs, Office of Naval Research, Arlington, Va.)

Dr. Robert A. Sweeney: Director, Great Lakes Laboratory, Professor of Biology, New York State University College at Buffalo, N.Y.

Mr. Arthur R. Theis: Chief Engineer, Department of Public Works, State of Louisiana, Baton Rouge, La.

Dr. William W. Woodhouse, Jr.: (Specialist in Vegetation), Professor of Soil Science, North Carolina State University, Raleigh, N.C.

Dr. Donald J. Zinn: Marine Biologist, Director and past President, National Wildlife Federation, Falmouth, Mass.

*Resigned 5 September 1980 due to ill health; died 22 December 1980.
**Became Acting Chairman effective 6 September 1980.

Among the criteria developed by the panel under item (2) of the panel responsibilities were the definitions of "sheltered areas" and "low cost" for use in the demonstration program. Congress had not defined these terms in the authorizing legislation. Sheltered areas were defined by the panel as shorelines in a coastal or tidewater area or on the Great Lakes where the incident breaking waves do not preclude the use of low-cost protection. Low-cost protection was defined by the panel as either $50 per lineal foot for materials (1975 price levels) if construction is possible without heavy equipment, or $125 per lineal foot for labor, materials, and heavy equipment rental. Every effort was made to select sites that represented categories or classes of coasts which would be good examples of shoreline conditions in large areas. Sites that were unique or that were examples of highly localized conditions were rated low. Other selection criteria included whether the site was free from influence by adjacent shoreline features or structures, whether the site was subjected to a range of physical forces (winds, waves, etc.), and whether the site was suitable for testing several alternative kinds of shore protection devices. The intent of the construction was to demonstrate shore protection devices which, due to cost limitations, should not be expected to survive a more vigorous storm than may be expected to occur on the average of once in any 10-year span.

In selecting sites for new test installations, consideration was also given to a large number of sites at which low-cost shore protection projects had already been built under other programs. These projects provided an opportunity to expand the data base without the added costs of device installation, and many such projects were selected for monitoring under the demonstration program.

d. Execution of the Program. The Chief of Engineers was responsible for the execution of the program and was provided technical assistance by the U.S. Army Coastal Engineering Research Center (CERC). The Corps of Engineers Districts in which the test sites were located were assigned the responsibility for designing and installing the test devices and monitoring their performances. District Engineers were given the options of having the devices installed by (1) a contractor, (2) the district itself, with hired labor and rented equipment, or (3) a combination of both procedures. Each district in which a demonstration project was undertaken implemented a public relations program in conjunction with that project. Press releases concerning the demonstration program and the district projects were issued to the local news media. A sign with data concerning the project was posted at the demonstration site, and, in some instances, a path was constructed to facilitate public access to the site. Some districts prepared fold-out brochures concerning their project that were issued to the public at both the district office and the demonstration site. As noted in subsection b, much of the vegetative work was done by the Soil Conservation Service of the Department of Agriculture.

3. Purpose and Scope of Report.

This report was prepared to document and evaluate data on the functional and structural performances of the various devices and vegetative schemes for low-cost shore protection that were installed in the Shoreline Erosion

Control Demonstration Program or were monitored under it. This document constitutes the final report on the program to be submitted to Congress by the Chief of Engineers, U.S. Army. It is intended to be a reference from which material may later be extracted for the preparation of manuals and guidelines tailored to the requirements of various categories of potential users to fulfill the requirement to disseminate the knowledge gained to the public.

This report was not intended to serve as a manual on low-cost shore protection. It was prepared primarily from an office synthesis of reports and data collected from the demonstration projects throughout the country that were installed or monitored under the program. As a result of delays from various circumstances, many of these demonstration projects were monitored for less than 1 year, and a fair evaluation may not be possible or presented in all cases. However, projects that were previously constructed and were monitored for a short time under this program have a longer experience record. The findings from those projects may have better substantiation and provide credence for evaluations of similar projects more recently installed.

Care should be exercised in using this report for developing general guidelines. The demonstration and monitoring projects were installed at specific sites--each site with specific soils types and exposure to specific weather and wave conditions. Because many of the monitored devices were constructed by local interests prior to inception of the Federal program, cost and construction data concerning these devices were often meager or lacking. As a result, no costs are presented for some devices, and, where available, costs of similar devices vary from site to site. The report therefore gives a history of the site, an evaluation of the performance of structures and materials, and, where possible, historical construction costs. The documentation is then grouped and analyzed in summary fashion to evaluate groups of similar installations and to define bases on which individual shore-front owners might reach specific conclusions concerning them.

This report is presented in an executive summary and five sections. The executive summary briefly describes the program, the results achieved, and significant findings and conclusions. Section I contains background information on the program, identifies the demonstration and monitoring sites, and describes the data collection program and analysis procedures. Section II describes each site and the various devices or systems installed at each, including costs; describes the experience gained at each site from each system, including the vegetation plan, from time of installation to the termination of the monitoring program; and discusses the functional effectiveness of those systems and their adaptability to the site at which they were installed.

Section III summarizes and compares the functional effectiveness and structural adequacy of each different device tested on an overall rather than on a site-by-site basis. The adaptability of each device to various functional systems (revetments, bulkheads, groins, breakwaters, etc.) is examined. Vegetation is evaluated in various climates, soil types, tidal ranges, and degrees of structural protection. Section IV evaluates the effectiveness of various materials used in the tests as to their adequacy to withstand wave and weather environment and to resist damage from floating debris, ice, and vandalism.

Section V is a critique summarizing the significant findings of the program.

4. Site Selection.

The authorizing legislation mandated 6 named sites along the western shore of Delaware Bay and required the selection of at least 10 additional sites, 2 each on, or connected to, the shorelines of the Atlantic Ocean, the Gulf of Mexico, the Pacific Ocean (between Canada and Mexico), the Great Lakes, and Alaska (Fig. 1-1). Eight million dollars was authorized by Congress to carry out this program. Where construction took place on non-Federal land, local interests were required to pay at least 25 percent of the first cost of each new installation, provide the site free-of-cost to the United States, and agree to take responsibility for the project at the end of the demonstration program. The selected sites reflect a variety of geographic and climatic conditions. In addition to the 16 demonstration sites, 21 additional sites, where prior shore protection projects had been implemented, were selected for monitoring. At these sites, data were gathered on the performance of existing structures similar in concept to that of structures at the demonstration sites.

5. Devices and Materials Used.

Various structural materials, used in a variety of shore protection devices and supplemented with vegetation plantings wherever possible, were installed at the 16 sites. Selection of materials was based largely on local availability and costs. Selection of shore protection systems (groins, revetments, fill, breakwaters, etc.) was based on a determination of the types of protection that would probably be most effective at each site, usually with some form of separation between systems to assure that their effects were not interactive. Materials, devices, and systems used are shown in Table 1-1.

6. Vegetation.

The authorizing legislation required the use of various kinds of vegetation in eroding intertidal zones and backshore areas both as a primary shore protection device and as a supplement to the protection afforded by structural devices. Accordingly, the indigenous vegetation at each site was examined to determine which types would be most suitable, and vegetative plantings were incorporated into most of the projects. Attempts were made to relate the beach form (high bluff, low bluff, sandy beach) to types of structures and vegetation that have proved successful in prior shore protection projects. The efficiency of vegetation in retaining beach materials was also studied, and various means of planting (e.g., peat pots, sprigs, seedlings, or plugs), fertilizing, and spacing of plants were compared.

Table 1-2 is a listing of demonstration and monitoring sites and protective systems monitored at each site.

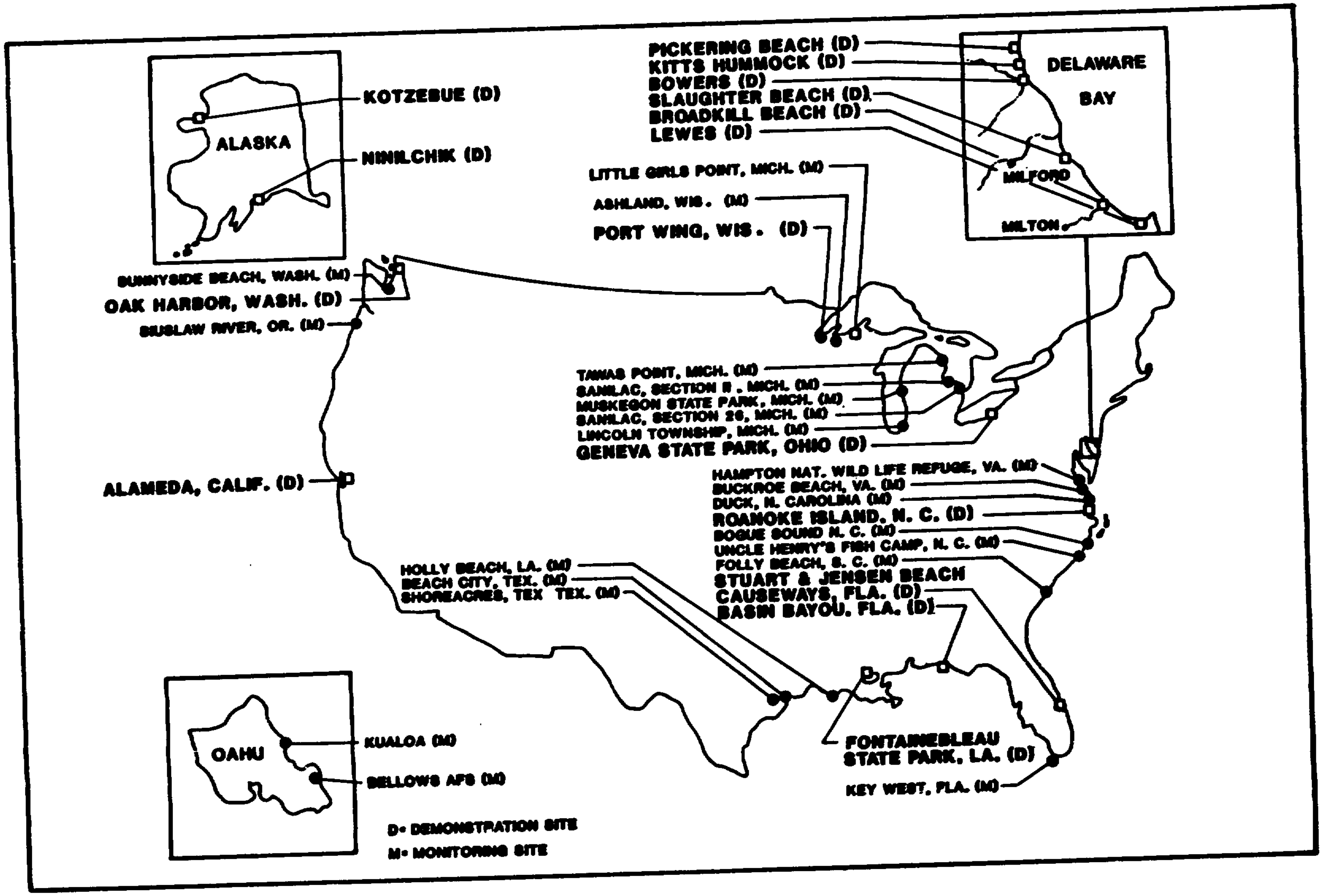


Figure 1-1. Site locations.

Table 1-1. Materials, devices, and systems used.

Material	Device or system
Timber (treated or untreated)	Bulkheads Groins Sills
Concrete	Block revetments Rubble Revetments Z-walls Sta-pods Surgebreaker Sandgrabbers Pipe sections Box breakwaters
Quarrystone and rock	Revetments Bulkhead backfill Filler for gabions Toe protection
Steel	Wire mesh for gabions (PVC-coated and galvanized) Hogwire fencing to retain sand-filled bags Fuel barrels for groins, sills, etc. Wire rope and rods for bulkhead tiebacks Corrugated metal drainpipe for a groin Round bars for tie rods H-beam posts for a bulkhead Bolts, nuts, washers for connections
Rubber	Used tires for floating breakwaters and seawalls Used tires stacked on posts for bulkheads and breakwaters
Burlap	Sandbags for sills, groins, revetments, etc.
Synthetics	Cloth filters (woven and nonwoven), sandbags, tubes, etc. Artificial seaweed
Paper	Bags filled with dry sand-cement for revetments

Table 1-2. Sites and their protective systems.

Location	Type of site[1]	Protective system
Delaware Bay sites		
Pickering Beach, Del.	D	Floating tire breakwater, vegetation
Kitts Hummock, Del.	D	Offshore breakwaters, vegetation
Bowers, Del.	DM	Beach nourishment, groins
Slaughter Beach, Del.	D	Perched beach, vegetation
Broadkill Beach, Del.	DM	Beach nourishment
Lewes, Del.	DM	Beach nourishment
Atlantic Coast sites		
Roanoke Island, N.C.	D	Artificial seaweed
Stuart-Jensen Beach Causeways, Fla.	D	Revetments, groin, floating tire breakwater, vegetation
Hampton Natural Wildlife Refuge, Va.	M	Existing vegetation
Buckroe Beach, Va.	M	Groins, bulkheads, offshore sill
Duck, N.C.	M	Vegetation
Bogue Sound, N.C.	M	Bulkhead, vegetation
Uncle Henry's Fish Camp, N.C.	M	Vegetation
Folly Beach, S.C.	M	Submerged sill, perched beach, groins, bulkheads, revetments
Gulf Coast sites		
Basin Bayou State Recreation Area, Fla.	D	Offshore breakwaters, Surgebreaker, bulkhead, Sandgrabber, vegetation
Fontainebleau State Park, La.	D	Offshore breakwaters, revetments, vegetation
Key West, Fla.	M	Vegetation
Holly Beach, La.	M	Revetment
Beach City, Tex.	M	Bulkhead
Shoreacres, Tex.	M	Bulkhead
Pacific Coast sites		
Alameda, Calif.	D	Beach nourishment, groin, offshore breakwater, revetments, perched beach, vegetation
Oak Harbor, Wash.	D	Revetments, bulkheads, vegetation
Sunnyside Beach, Wash.	M	Bulkhead, beach nourishment
Siuslaw River, Oreg.	M	Groins

Table 1-2. Sites and their protective systems (continued).

Location	Type of site[1]	Protective system
Great Lakes sites		
Port Wing, Wis.	D	Bulkheads, revetments
Geneva State Park, Ohio	D	Offshore breakwaters, vegetation
Ashland, Wis.	M	Seawall, groins
Little Girls Point, Mich.	M	Revetment
Lincoln Township, Mich.	M	Groins
Muskegon State Park, Mich.	M	Beach fill, natural revetment
Tawas Point, Mich.	M	Revetment
Sanilac Section 11, Mich.	M	Seawall
Sanilac Section 26, Mich.	M	Seawall, groins
Alaska sites		
Kotzebue, Alaska	D	Groins, revetments
Ninilchik, Alaska	D	Groins, revetments
Hawaii Sites		
Kualoa, Hawaii	M	Sandgrabber
Bellows Air Force Sta., Hawaii	M	Sandgrabber

[1]Key to type of site:

D = Demonstration site with protective system(s) installed under program.

DM = Site was specified in the Public Law. No devices were constructed, but a beach nourishment program (by others) was monitored.

M = Monitoring site where prior shore protection project(s) had been implemented.

7. Data Acquisition Program.

Monitoring of the installations at the 37 sites and collection of data were accomplished by personnel of the various Corps Districts in which the sites were located. Non-Corps personnel were specially trained by the Corps to make Littoral Environment Observations (LEO). The data collected included the LEO data, topographic and bathymetric surveys, aerial photography, monthly site visit reports, surficial sand samples, ground level photographs, and vegetation planting inventories.

a. LEO Data. At least one LEO station was established at all but six sites and, in general, observations at each station were made twice daily. The data from these observations were transmitted to CERC, where they were analyzed and compiled in summary form. Data were collected on the wave period, wave direction, breaker height, windspeed, and wind direction. At some sites, foreshore slope, width of the surf zone, longshore current speed and direction, and tide levels were recorded. Any unusual conditions or activities were also noted.

b. Topographic and Bathymetric Surveys. Surveys were specified just prior to and after construction, and at 3-month intervals thereafter. At some sites, deviations from this schedule were permitted to allow for special conditions such as winter ice cover. Surveys were planned to adequately define spatial and volumetric changes in the shoreline and nearshore areas in the vicinity of the structures. Additional surveys were conducted after severe storms that caused significant shoreline changes at a site.

c. Aerial Photography. Aerial photography for monitoring at most sites was flown at roughly 3-month intervals for the duration of the monitoring program. The photos had an approximate scale of 1:2400 (1 inch equals 200 feet) on a 9- by 9-inch format. They were exposed with 60-percent overlap to facilitate stereoscopic study of the beach terrain. Flights were made as near to the time of low tide as possible.

d. Sediment Samples. Samples of surface sediments were gathered at selected locations along the range lines during the topographic surveys. These samples were analyzed to describe the size gradation and other characteristics of the beach sediments at the sites.

e. Monthly Site Visits. Each site was visited monthly by a representative of the Corps of Engineers District Office. At each visit, the structural and the functional performances of the devices were evaluated, and the information was noted on standard monitoring forms. These completed forms were then used in analyzing the results of the program.

f. Ground Level Photography. At each monthly inspection, ground level photos were taken of the entire site. On each occasion, a series of standard photos was taken from the same location and in the same direction of view to enable comparisons of beach and structural changes over a period of time. Photos were also taken of a series of randomly located, 1-square meter vegetation test plots. Other items of interest, such as damage due to storms or closeups of incipient failures, not otherwise covered by the standard views, were also photographed.

g. Vegetation Planting Inventories. Vegetation planted at each site was recorded on an Initial Post Planting Inventory form immediately after planting growth was made at the beginning, middle, and end of the growing season. The Initial Post Planting Inventory consisted of a precise count of the number of individuals of each species, the type of plantings used, and the average size of the individuals, using height or number of stems, as available. The areas in which specific types of plantings were made, the spacing, and the fertilizer regime were also recorded at the outset of each planting project and were recorded on the Post Planting Inventory.

The followup visits, made periodically during the growing season, provided information on the number of surviving individuals of each species planted, the area in which they grew, and their performance in a qualitative sense. In addition, the number of stems in randomly placed 1-square meter plots was counted as a measure of density. When the stem density became high and it was no longer possible to distinguish individual plants, only the number of stems per square meter in randomly selected plots was recorded. Plots were identified as to section or area within the overall planting design at each site where appropriate. At the end of each growing season, several randomly selected square-meter plots were counted and then the standing plant material was clipped to the ground. The clipped material was then air-dried and weighed. These data were reported on the end-of-season report form.

The vegetation at each site was photographed at each visit, as were significant damage and structural problems which were pertinent to the survival of the vegetation. Narrative reports were also furnished on particulars of storm damage and vegetation survival.

8. <u>Data Analysis</u>.

a. <u>LEO Data</u>. The LEO data provided a basis for comparison of the normal daily wave climate from site to site. It is a useful way to rank the sites in terms of relative severity of wave action. Table 1-3 summarizes the results of the data analysis for all but six sites, where no LEO stations were established. The categorization as to relative severity of the wave climate was tempered by knowledge of special conditions at some sites. Where the daily wave heights were known to be fairly uniform, the LEO mean height was considered a good indicator of the wave climate. Where short episodes of waves much higher than average are common, a category more severe than that indicated by the data was assigned. Because observations were seldom made during severe storms, the LEO reports provided little information on extreme events. However, the results of such events are described in Section II of this report for the sites at which they occurred, based on special reports by the district monitors.

Where adequate data on wave energy flux were available, they were processed to provide information on two categories of factors concerning longshore transport of littoral material: (1) the energy-weighted resultant direction vectors for waves approaching the site from quadrants right and left of the normal to shore and the net resultant of the two vectors, and (2) the computed potential of the waves, represented by these vectors, to move sand along the beach during the months of record. Longshore transport is always in the direction opposite that from which the waves approach the shore at the breaker line. Therefore, to avoid confusion, the vector angles are expressed in terms of the angle of the breaking wave crest with respect to the shoreline. If the angle opens to the right,

Table 1-3. Summary of LEO data.

Site	Wave Statistics				Energy Flux						
	Average height (ft)	Maximum height (ft)	Average period (sec)	Rel. Sev.[1]	Data Base (mo)	Left[2] Angle (deg)	Left[2] Volume (yd^3)	Right[2] Angle (deg)	Right[2] Volume (yd^3)	Net Angle (deg)	Net Volume (yd^3)
DELAWARE BAY											
Pickering Beach, Del.	0-1	2.5	0-1	I	3	14	3,110	13	14,860	1	11,750(R)
Kitts Hummock, Del.	0-1	2.2	0-1	I	3	19	53,111	12	18,490	12	34,621(L)
Bowers, Del.	1-2	4.0	3-4	I	3	9	14,984	12	49,557	2	34,572(R)
Slaughter Beach, Del.	0-1	3.0	3-4	I	10	19	57,622	11	57,028	2	594(L)
Broadkill Beach, Del.	1-2	4.6	5-6	S	4	13	68,153	11	45,874	2	22,279(L)
Lewes, Del.	0-1	3.0	4-5	I	3	7	5,634	7	11,085	2	5,450(R)
ATLANTIC COAST											
Roanoke Island, N.C.	3										
Stuart and Jensen Beach Causeways, Fla.											
2N	0-1	1.0	0-2	M	9	23	87	76	1,475	61	1,388(R)
2S	0-1	1.7	1-2	M	9	17	153	51	6,601	42	6,447(R)
3S	0-1	1.0	0-1	M	9	16	251	48	2,443	33	2,192(R)
Hampton, Va.	3										
Buckroe Beach, Va.	0-1	4.0	0-1	I	3						
Duck, N.C.	0-1	1.0	2-3	M	2	19	22	16	20	1	1(L)
Bogue Sound, N.C.	0-1	-	0-1	M	3						
Uncle Henry's, N.C.	3										
Folly Beach, S.C.	1-2	5.5	6-7	S	11	8	101,394	8	408,904	2	302,511(R)
GULF COAST											
Basin Bayou, Fla.	0-1	4.0	0-1	I	11	15	174,428	17	114,069	4	60,369(L)
Fontainebleau, La.	0-1	2.5	0-1	M	3	12	3,322	15	20,529	3	17,207(R)
Key West Fla.											
Sec. A	0-1	0.5	0-1	M	5	23	451	62	144	8	307(L)
Sec. B	0-1	1.0	0-1	M	8	10	1,726	24	12,261	16	10,535(R)
Sec. C	0-1	4.0	0-1	M	8	44	58,479	40	2,220	37	56,259(L)
Sec. D	0-1	5.0	0-1	M	8	39	85,394	31	2,786	37	82,608(L)
Sec. E	0-1	2.0	0-1	M	8	52	13,760	37	3,231	43	10,529(L)
Holly Beach, La.	0-1	8.0	0-1	S	3	12	70,898	16	855,035	5	784,137(R)
Beach City, Tex.	0-1	1.4	0-1	I	4	40	26,573	38	14,741	30	11,832(L)
Shoreacres, Tex.	0-1	1.8	0-1	I	4	46	23,262	28	53,835	11	30,572(R)
PACIFIC COAST											
Alameda, Calif.	0-1	7.0	0-1	I	6	22	39,265	22	2,771	19	36,494(L)
Oak Harbor, Wash.	0-1	3.0	0-1	M	6	10	6,594	8	2,913	6	3,682(L)
Sunnyside Beach, Wash.	0-1	2.0	2-3	M	10	24	8,811	15	28,504	3	19,692(R)
Siuslaw River, Ore.	0-1	1.0	0-1	M	4	78	179	61	3,989	52	3,810(R)
GREAT LAKES											
Port Wing, Wis.	0-1	5.0	1-2	S	7	8	39,871	9	235,220	3	195,349(R)
Geneva State Park, Ohio	0-1	4.0	0-1	S	4	23	23,421	12	128,069	4	104,648(R)
Ashland, Wis.	3										
Little Girls Point, Mich.	3										
Lincoln Township, Mich.	3										
Muskegon, Mich.	0-1	7.1	0-1	S	2	24	53,281	11	148,497	8	95,217(R)
Tawas Point, Mich.	0-1	4.3	2-3	S	1	26	23,935	21	23,400	5	535(L)
Sanilac Sec. 11, Mich.	0-1	2.5	0-1	I	3						
Sanilac Sec. 26, Mich.	0-1	4.0	0-1	I	3						
ALASKA											
Kotzebue, Alaska	0-1	6.5	0-1	S	3						
Ninilchik, Alaska	0-1	6.5	0-1	S	8	9	235,314	8	307,146	1	71,831(R)
HAWAII											
Kualoa, Hawaii	0-1	1.7	7.9	I	3	7	591	9	1,274	2	683(R)
Bellows AFS, Hawaii	1-2	3.0	5.6	S	6	9	14,447	21	144,704	15	130,268(R)

[1] Relative Severity: S=Severe; I=intermediate; M=mild.

[2] Breaker angle: when angle opens to the right, longshore transport is to the right and vice versa.

[3] No LEO station established, or insufficient data for meaningful analysis.

longshore transport is to the right and vice versa. The longshore transport potential was computed in accordance with the method outlined by Walton (1980).

The energy-flux data are useful primarily at sites where groins, offshore breakwaters, or beach nourishment are being demonstrated. For a site at which the data base is 8 or more months, the results could be extrapolated to provide a rough indication of the annual rate of transport for a beach fill or to determine the probable configuration of an accretion in the lee of a breakwater. Because the observed data do not document durations of wave events, the energy-flux calculations may vary considerably from actual occurrences.

Also, at most of the sites, the data base comprised observations made during only a few months. Therefore, the tabulated rates for those months may not be indicative of conditions during the months of the year for which data were missing or were inadequate for meaningful analysis. The tabulated transport rates should be verified wherever possible by more positive indicators, such as accretions at existing groins and other barriers. The energy-flux data are good indicators of the probable effectiveness of groins. Groins, for example are most effective where waves approach preponderantly from either right or left of the normal to shore. They are least effective when the net resultant direction is nearly normal to shore. When there is a wide divergence of approach directions to the right and to the left of the normal, groins of sufficient length may prevent the escape of sand from the site area; however, rip currents may be generated along the sides of the groins, which tend to jet large quantities of sand out into deep water. At sites where bulkheads, seawalls, and revetments are demonstrated, the LEO data have relatively little value other than to develop general information as to the wave climate.

b. Survey Data. The survey data were processed to obtain comparative beach and offshore profiles for each range line surveyed. These profiles were examined to determine the functional effectiveness of structural devices and to plot contour changes. Where the devices produced significant erosion or accretion, calculations were made to quantify the volume changes.

c. Aerial Photos. The photos were used individually to interpolate shoreline changes between widely spaced range lines of the instrumental surveys. Stereoscopic viewing of successive pairs along the flight line provided a more reliable determination of bluff and shoreline locations. The photos were also helpful in detecting terrain and structural features not indicated on line drawings nor described in the monitors' reports. Some of the photos were used in this report to illustrate changes resulting from device installations.

d. Sediment Samples. Soils classifications and grain-size data were used to characterize beach and bluff sediments and to compare materials found at the various sites. The soils data were also used to estimate slope-stability properties and to evaluate the effectiveness of filter materials used in conjunction with structural devices.

e. Monthly Site Visits. Monthly reports by the monitors provided the bulk of the data describing the structural and functional performance of the devices at each site during the monitoring period. Observations of structural failures in progress and notations by the monitors as to their exact nature provided the insight needed for an accurate analysis of structural performance and adequacy. Many suggestions for improvements contained in the monitors' reports have been incorporated into the analyses of this report.

f. Ground Level Photos. The abundance of ground level photos taken by the monitors was most helpful in understanding the changes in structure and beach forms described in the monthly reports. Comparison of successive photos of specific devices and their components, and of beach formations affected by the devices, provided excellent insight into the causes of structural failures and the effectiveness of shore protection systems. Many of the photos have been used in this report.

9. Evaluation of Devices.

Based on the analysis of data, each device was evaluated as to its structural and functional performance. Devices were termed either unsuccessful, partially successful, or successful. An unsuccessful device that failed structurally and was deemed to be unrepairable within the scope of the program was considered a "successful failure" in that much had been learned about its performance, even though it did not measure up to expectations. A device that survived but did not protect the shoreline was also considered unsuccessful. A device that survived the monitoring period with significant damage, but protected the shoreline reasonably well, was considered partially successful, and recommendations were made as to conditions that would limit its use or minor changes that would improve it structurally. A device that survived with relatively little or no damage and performed well was considered successful.

A device that was damaged as a result of the failure of an adjacent device that left its flank unprotected, or as a result of a storm that exceeded design criteria, was repaired, and the monitoring was continued. Other devices that deteriorated during the monitoring period but still provided some degree of protection were monitored to evaluate the aging process. None of the devices was "maintained;" thereby, the aging process could take its normal course and the effective service life of each device, as initially installed, could be estimated.

II. PROGRAM ANALYSIS BY SITES

1. Delaware Bay Sites.

a. Common Characteristics.

(1) Geographical Setting. Six demonstration sites are located along a segment of the western shore of Delaware Bay extending 34 miles generally northward from the mouth of the bay at Lewes Beach. The site locations are shown in Figure 2-1. All six sites have the same climate and generally the same physical environment. Shore alinements vary from north-south at Pickering Beach to east-west at Lewes Beach.

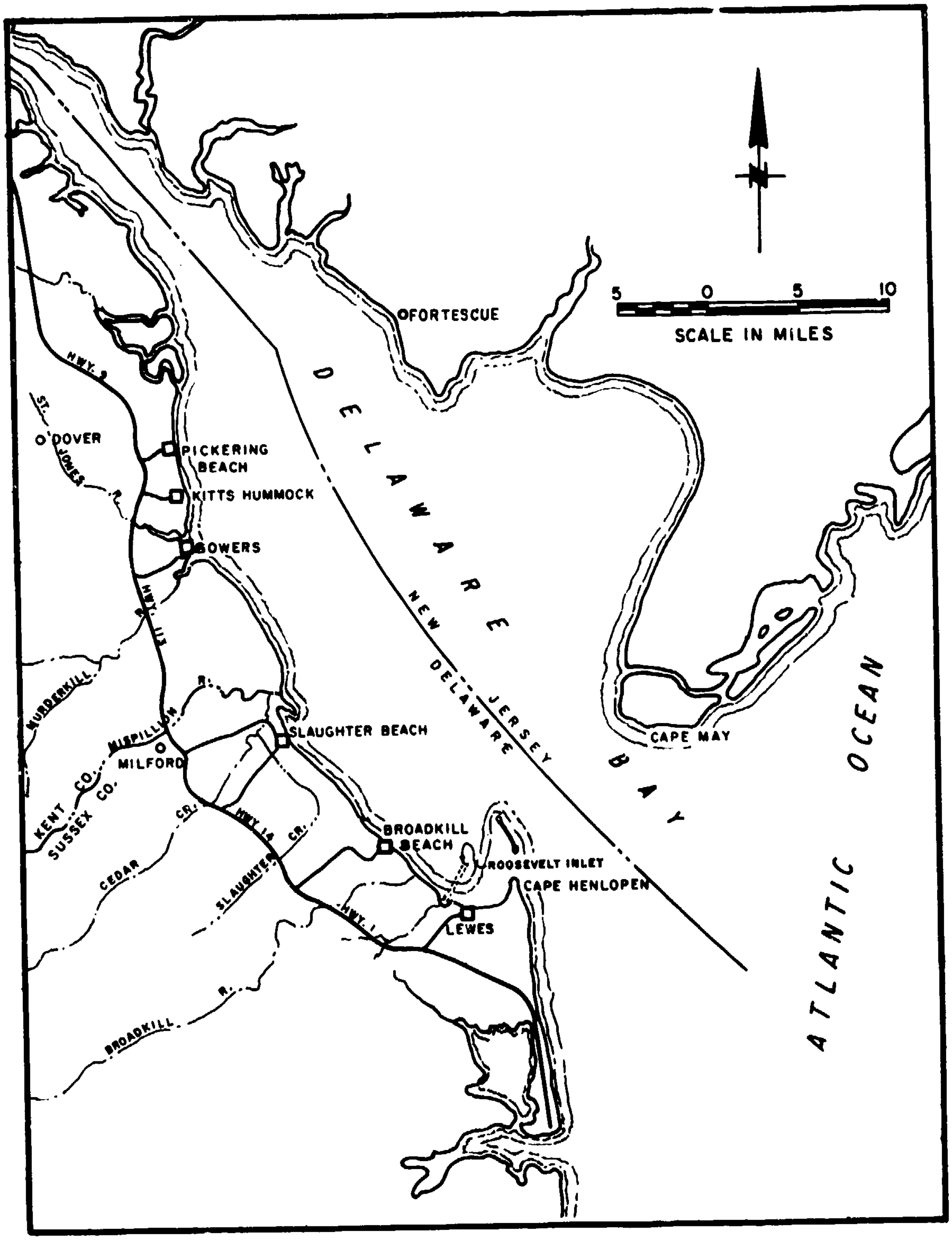


Figure 2-1. Location map of Delaware Bay demonstration sites.

The Delaware Bay section of the Delaware coast is an undeveloped and sparsely populated region, consisting primarily of a narrow strip of sandy beach separating the bay from the marshes and low flat uplands. Small communities that have developed along the beach are accessible individually via narrow roadways through the marsh to interior highways. The coastal marshes of Delaware are considered among the most ecologically productive in the country, providing nutrients which proliferate a large and diverse aquatic population, and are protected as nature preserves.

(2) Climate, Waves, and Tides. The climate of Delaware Bay is moderated somewhat by its proximity to the Atlantic Ocean. In winter, storms may be accompanied by strong gusty winds and rain or snow. Warm spells, sometimes with abundant rain, alternate with cool, dry weather in the spring. In summer, weather is relatively constant but may have frequent unstable showers and thunderstorms, uniform warm temperatures, high humidity, and low windspeeds. The hurricane season typically extends from June to October. Precipitation is moderately heavy (45 inches per year) and well distributed geographically.

The wave climate varies with the shore alinement and exposure to waves entering the bay from the Atlantic Ocean. Ocean waves at the mouth of Delaware Bay are generally closely related to the wind conditions. In the winter months, the predominant ocean waves are from the northeast, and the waves are rougher than during the summer months. In summer, the ocean waves come from southerly and southeasterly directions and the heights are moderate. Storm waves may transmit considerable longer period wave energy into the bay. The LEO data (Table 1-3) indicate that Broadkill Beach has a relatively severe wave climate, with heights averaging 1 to 2 feet and reaching maximum heights of nearly 5 feet. The other five sites have an intermediate wave climate, with heights averaging about a foot or less and reaching maximums of 2 to 3 feet, except at Bowers, where a 4-foot maximum height was observed.

Within Delaware Bay, waves that impinge upon the shoreline are primarily generated by local winds, and have relatively short periods compared to those along the Atlantic coast. Wave heights are also relatively small (not more than 6 feet because of the limited fetch and water depth across the generating area). Sites farther from the bay mouth generally have milder wave climates. Waves generated within the bay approach the project shorelines most frequently from east-northeast. The Great Storm of March 1962 produced a water surface setup of 7.9 feet above National Geodetic Vertical Datum (NGVD), or 9.9 feet above mean low water (MLW) at Lewes, which was the maximum ever recorded. That storm was of unusually long duration, with strong easterly winds that continued over five successive tide cycles, generating waves that destroyed many shore protection structures. Large amounts of ice form in Delaware Bay during severe winters. Ice pressures and impacts by floe ice often damage shore protection structures.

It is difficult to estimate the direction or rate of longshore transport at some of the project sites because beach erosion control measures, changes in the configuration of the shoreline, and the construction of shore structures have altered the nearshore processes. However, the LEO data provide an indication of the probable net direction and rate of longshore transport at each site.

Tides are semidiurnal (two nearly equal high and low tides each day), and their heights vary among the six sites, increasing in range progressively with increasing distance from the bay mouth. Because of the bay's influence, midtide levels are above NGVD. Tide data are given in Table 2-1.

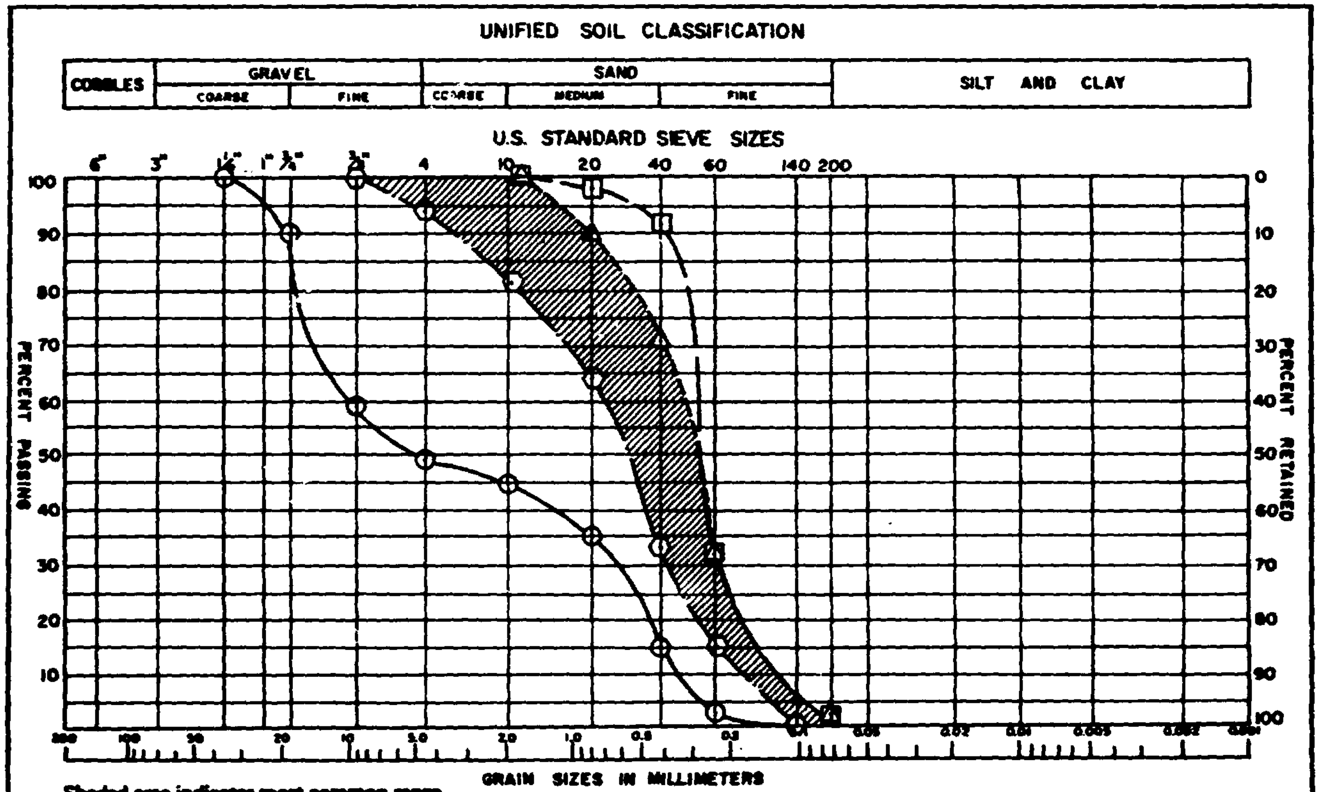
Table 2-1. Tidal ranges for Delaware Bay sites.

Location	Mean range (ft)	Spring range (ft)	Mean tide level (ft above MLW)
Pickering Beach	5.2	6.3	2.7
Kitts Hummock	5.2	6.3	2.7
Bowers	4.8	5.7	2.4
Slaughter Beach	4.6	5.4	2.3
Broadkill Beach	4.4	5.2	2.2
Lewes	4.4	5.2	2.2

(3) Geomorphology, Soils, and Vegetation. All six Delaware Bay sites are in the Delaware Coastal Plain and exhibit common geomorphologic characteristics. The coastal area is typically a low, featureless plain underlain by Miocene and Pleistocene formations. Unconsolidated and semi-consolidated Pleistocene sediments of marine and fluvial origin are present along the shores of the bay. They typically contain glacial debris transported by melt waters and deposited in the Delaware River Valley. The Pamlico Formation, of fluvial and estuarian origin, deposited in Pleistocene time, forms the surface between sea level and about 25 to 30 feet above sea level. It consists primarily of silts, sands, and gravel. Overlying the Pamlico deposits are recent sediments consisting of beach and dune sands and tidal marsh deposits composed of silts, clays, and peat.

The existing soils along the beaches and sand dunes of the Delaware Bay sites generally consist of fine to coarse sands with some gravel. Figure 2-2 is a graph showing four representative grain-size curves selected from beach samples collected at all sites. The shaded area of the graph indicates the range of gradations where most of the samples commonly fell. Figure 2-3 shows the range of gradations from samples collected from the dune areas at all sites.

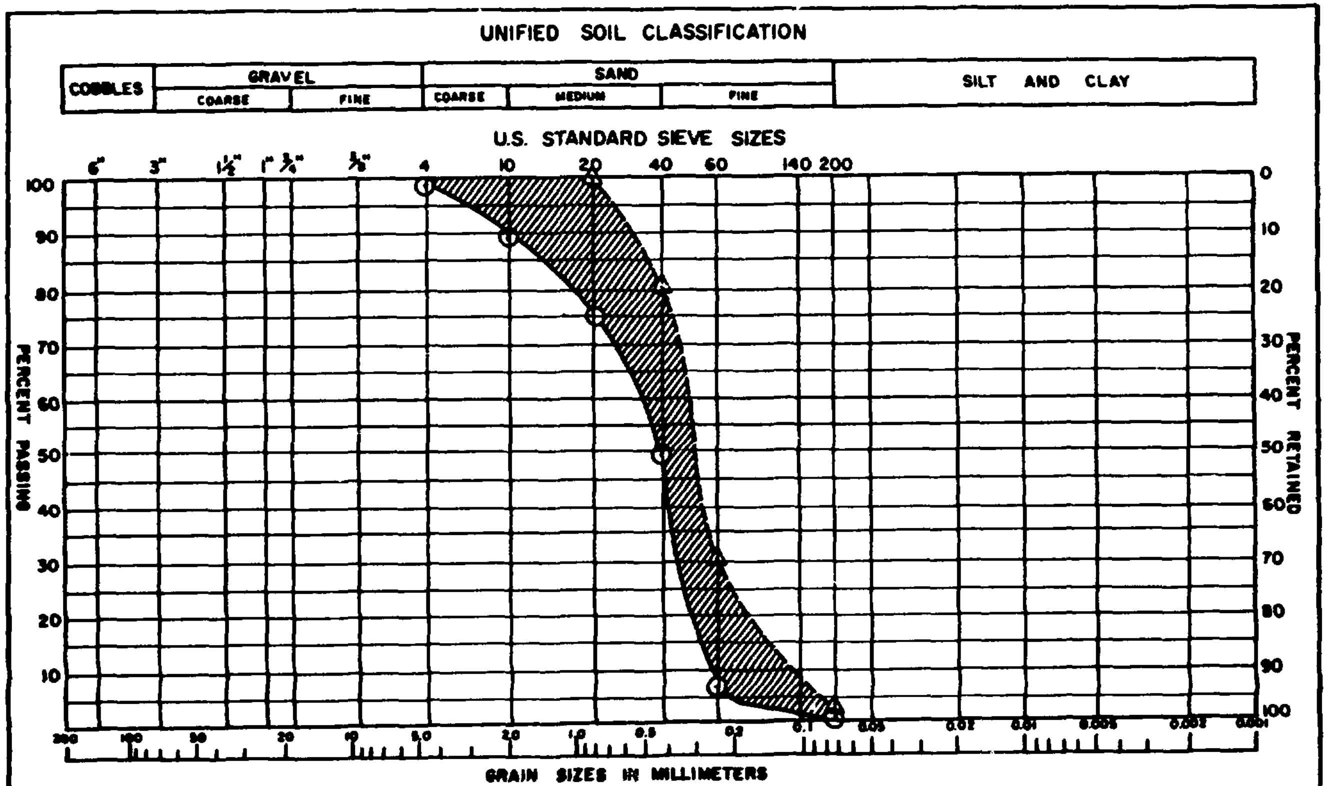
The shoreline is characterized by beaches backed by low sand dunes, which support a sparse growth of native beach and dune grasses. The beaches are nearly continuous and narrow, and range from 10 to 50 feet wide at high water. The sand dunes range from 30 to more than 200 feet wide and 8 to 12 feet high. Extensive tidal marsh areas separate the dunes from the backshore areas. The salt marshes with dense growths of marsh vegetation extend from 0.5 to 2 miles inland and generally follow inland stream channels. Although these streams flow through regions composed basically of sands and silts, stream velocities are too slow to pick up and transport appreciable amounts of sediment. The small volumes of material carried are deposited in the marshes and practically none reaches the shoreline littoral zone to serve as a source of beach nourishment.



Shaded area indicates most common range.
Samples collected on beach at high water line.

LOCATION	STATION	SYMBOL	LL	PI	CLASSIFICATION
Bowers	10 + 00B	⊙——⊙			Sandy gravel to gravelly SAND (GW–SW)
Broadkill Beach	4 + 35N	△– – –△			Medium SAND (SP)
Kitts Hummock	17 + 30S	⊙— · —⊙			Medium SAND (SP)
Slaughter Beach	13 + 25S	⊡— —⊡			Fine SAND (SP)

Figure 2-2. Representative grain-size distribution curves, Delaware Bay sites.



Samples collected on dune.

LOCATION	STATION	SYMBOL	LL	PI	CLASSIFICATION
Slaughter Beach	6 + 656	⊙———⊙			Medium SAND (SP)
Bowers Beach	17 + 008	△- - - - -△			Fine SAND (SP)

Figure 2-3. Representative grain-size distribution curves, Delaware Bay sites.

(4) The Problem. The six Delaware Bay demonstration sites are typical examples of severe beach erosion. At each site, coastal erosion is ruining the beach and threatening to destroy the beach community. Corrective action taken by the State of Delaware and the Federal Government before implementation of the demonstration program was mainly the periodic nourishment of the beach with dredged sand from offshore deposits. Thus, the surficial sediments are not normal beach material. In a few instances, attempts were made to retain the fills with low groins. Structural devices were installed at three sites (Pickering Beach, Kitts Hummock, and Slaughter Beach) to demonstrate their effectiveness under conditions that were typical of other places in the national sense.

(5) Vegetation Planting. At each of the three sites where structural devices were installed, vegetation was planted to demonstrate its effectiveness in controlling erosion. Development of the planting scheme was coordinated with the Plant Materials Research Center, Soil Conservation Service, Cape May, New Jersey, and the Northeast Technical Service Center, Broomall, Pennsylvania. The Cape variety of American beachgrass (*Ammophila breviligulata*) was planted in a 10-foot strip above the tide zone, saltmeadow cordgrass (*Spartina patens*) was planted in a 30-foot strip in the zone above MLW, and smooth cordgrass (*Spartina alterniflora*) was planted in a 30-foot strip below MLW. Separate costs were not kept for each site, but the overall costs for the planting program are given in Table 2-2.

Table 2-2. Combined vegetation planting costs for Pickering Beach, Kitts Hummock, and Slaughter Beach, Delaware Bay.

Item	Cost		
	Cape American Beachgrass	Saltmeadow cordgrass	Smooth cordgrass
Labor	$1,000	$3,000	$3,500
Materials (plants)	1,166	3,467	4,499
Per diem	150	450	600
Loading	-	473	540
Shipping	-	420	420
Fertilizer-material	100	300	952
F[illegible]tilizer-labor	300	300	-
[illegible]ment/machinery	140	420	470
	$2,856	$8,830	$10,981
Cost/lin. ft	$ 1.77	$5.47	$6.81
		Total	$22,667

b. P[illegible]ing Beach, Delaware.

(1) Site Description.

(a) Geographical Setting. Pickering Beach, approximately 10 miles east of Dover and 34 miles from the bay mouth, is a small summer resort extending approximately four-tenths of a mile along the bay shore. The community consists of about 40 summer cottages located about 20 feet behind the landward edge of the barrier dune.

(b) Geomorphology and Soils. The soil at Pickering Beach consists primarily of fine to coarse sands. The shoreline generally is oriented north to south. The beach is approximately 40 feet wide at high water and slopes at about 1 on 10. The offshore zone slopes at approximately 1 on 20 for about 100 feet offshore, then flattens to a slope of about 1 on 800. A sand dune with a base width of 30 feet and a top elevation about 10 feet above MLW parallels the beach. The beach was last nourished in March 1979.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 2.5 feet, and the net longshore transport potential southward was 11,800 cubic yards for the 3 months analyzed.

(2) Demonstration Project. The protective devices initially constructed at Pickering Beach were two types of a floating scrap-tire breakwater: type I, a modified version of the Wave-Maze designed by Noble (1969), and type II, a design developed by the Goodyear Tire and Rubber Company. The Wave-Maze was much more costly than the Goodyear system. After receipt of the construction bids, various changes were considered for reducing the cost of each installation. In the Wave-Maze, automobile tires were substituted for truck tires, galvanized steel bolts were substituted for the costly nylon bolts in all but a 50-foot control segment, ordinary nuts were substituted for locknuts, and fender washers were substituted for rubber patches. In both systems, the total length of breakwater was reduced from 500 to 404 feet: one 20- by 202-foot section and one 40- by 202-foot section. Concrete blocks were used for anchors. The layout plan is shown in Figure 2-4.

Wide separation of the two breakwaters was considered at one time as a means of avoiding overlap of effects on the shoreline. Close spacing of the breakwater sections was selected because separation might permit too much wave energy to be transmitted through the gap and, by diffraction, dilute the attenuation effects of the overall installation at the shoreline. (The flat slopes and shallow nearshore depths did not permit the location of the shortened structures close to the beach where they could be individually effective if widely separated).

The Wave-Maze breakwater has a basic module composed of five tires bolted directly together as shown in Figure 2-5. The full-width section is 15 modules wide, and the half-width section is 7 modules wide. The Goodyear breakwater has a basic module composed of 18 tires strapped together and utilizes an extra tire for joining with the adjacent module on any one of its four sides (Fig. 2-6). The full-width section is six modules wide; the half-width section is three modules wide. The method of placing foam for flotation in the tires of both breakwater types is shown in Figure 2-7. The foam used for flotation was a two-component polyurethane foam manufactured by Witco Chemical Company. The foam expands at a ratio of 30 to 1. The concrete anchor block dimensions and anchorage scheme are also shown in

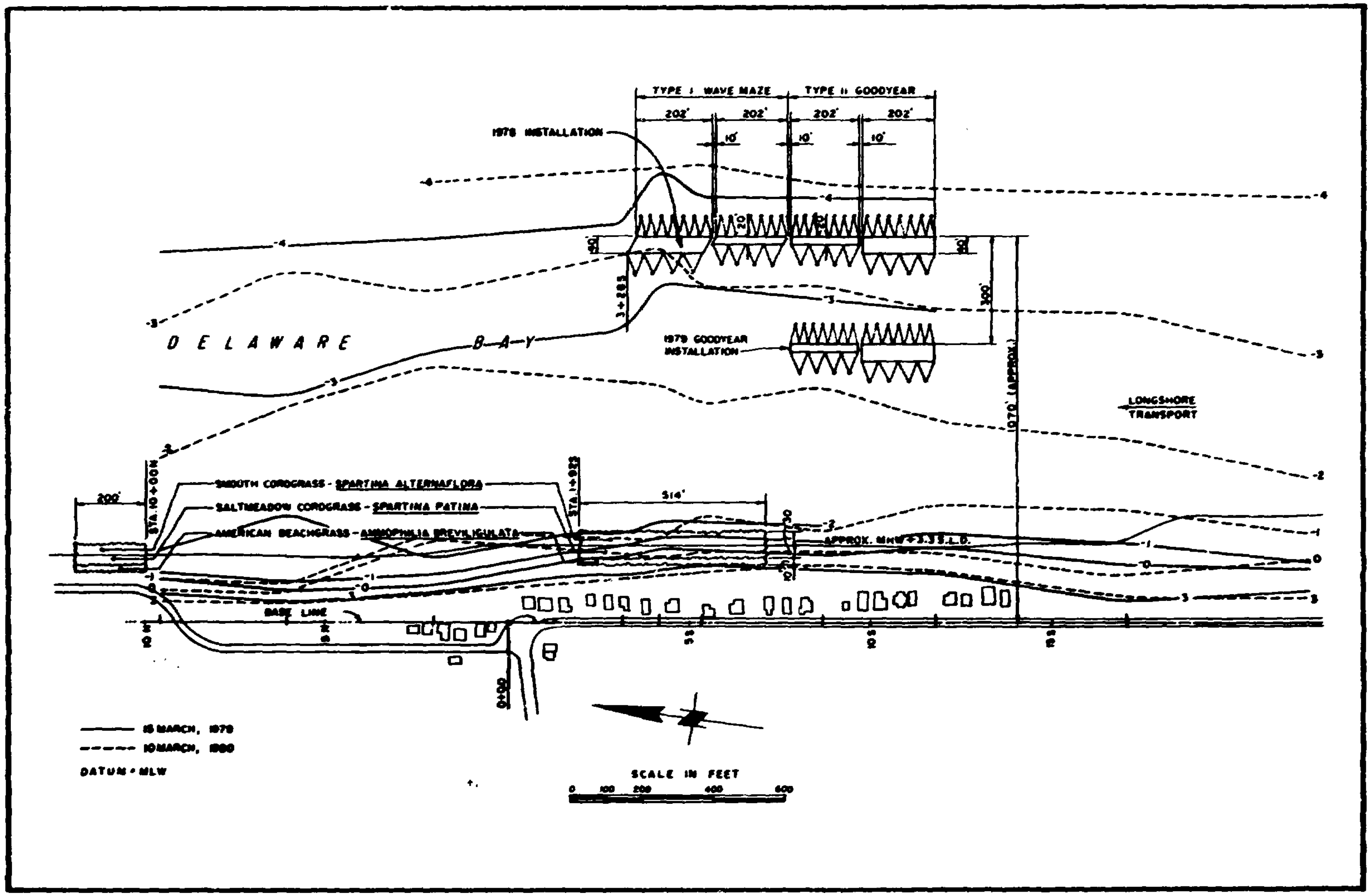


Figure 2-4. General plan of Pickering Beach, Delaware, demonstration site.

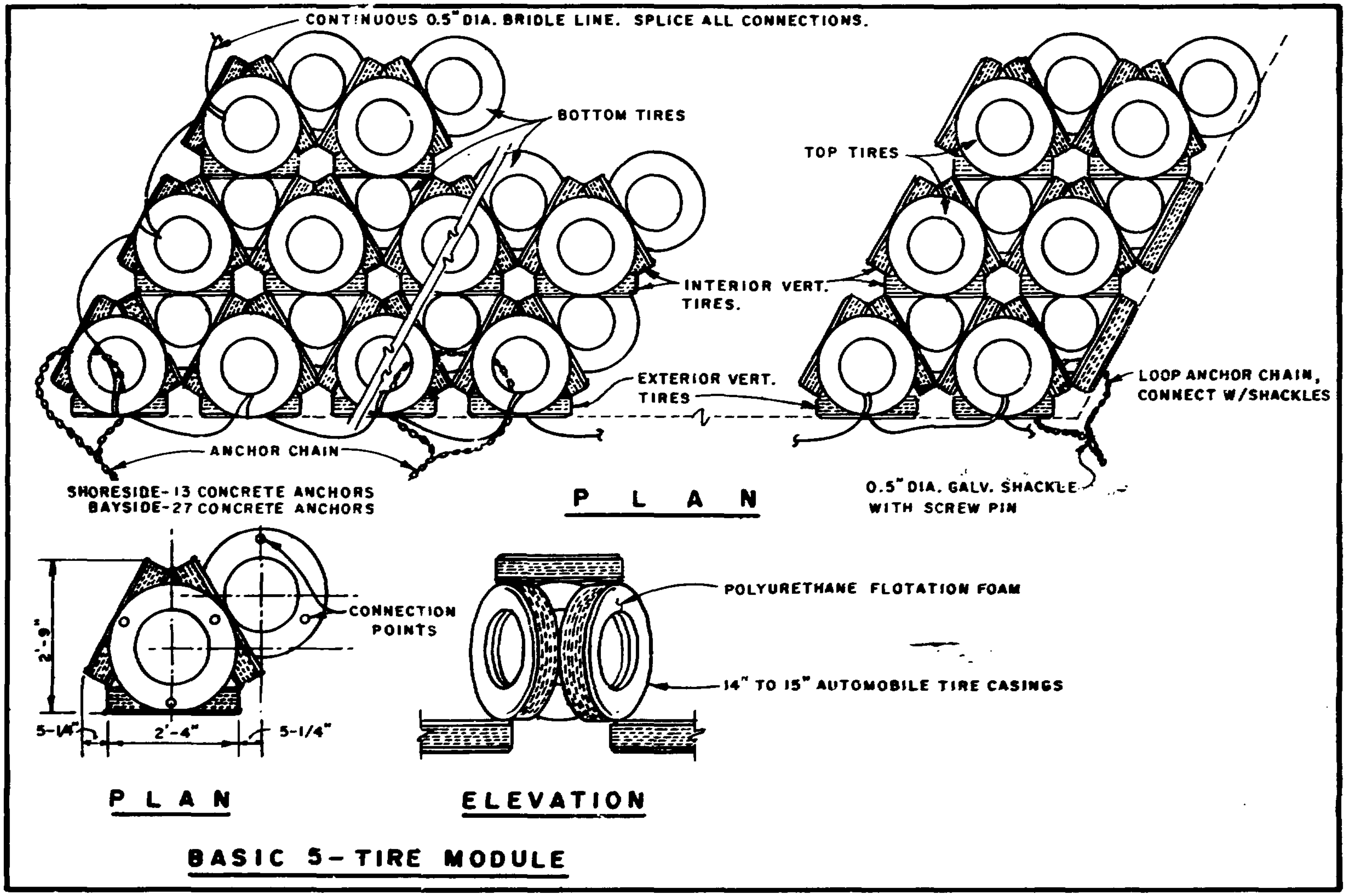


Figure 2–5. Wave–Maze breakwater (type I) at Pickering Beach site.

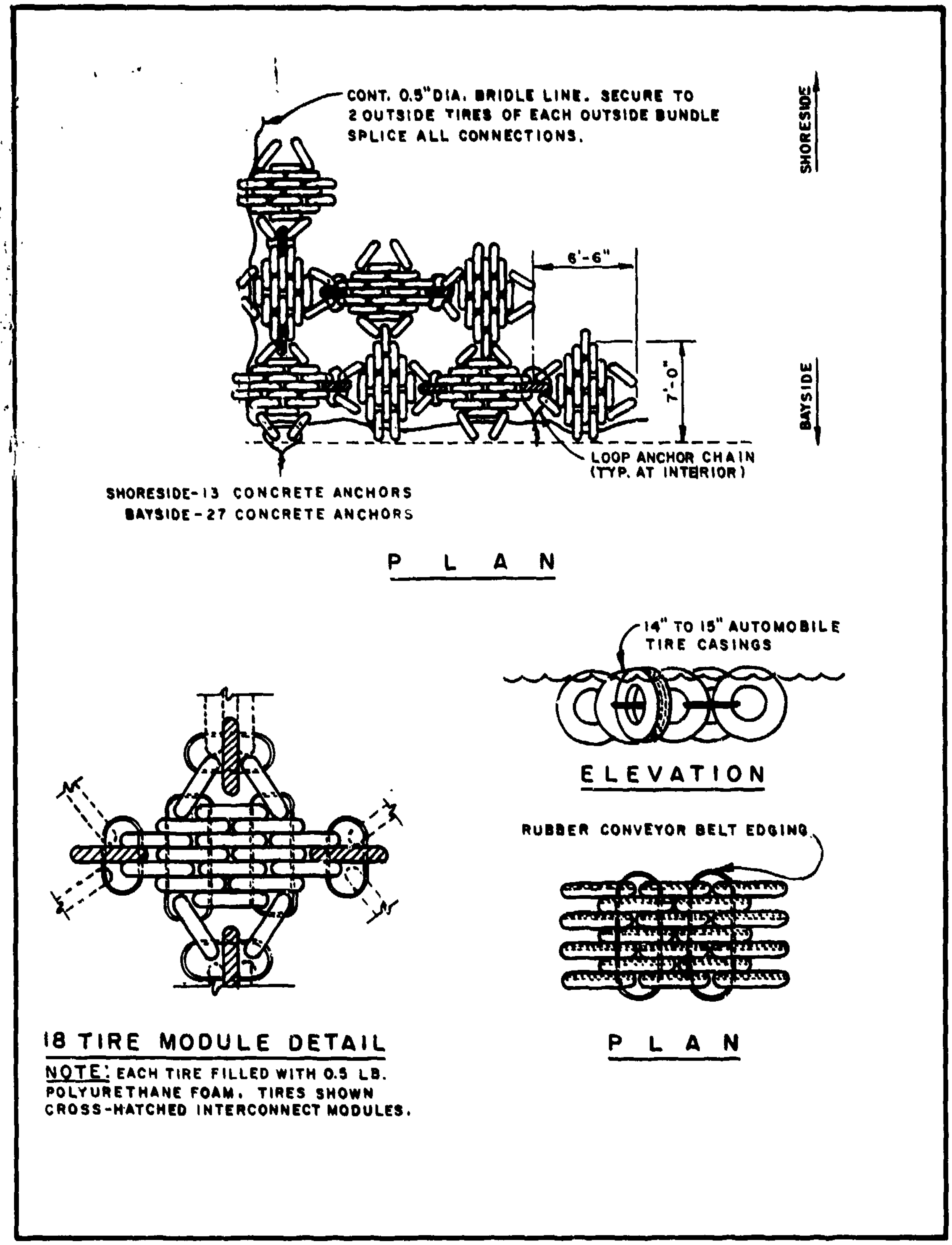


Figure 2-6. Goodyear system breakwater (type II) at Pickering Beach site.

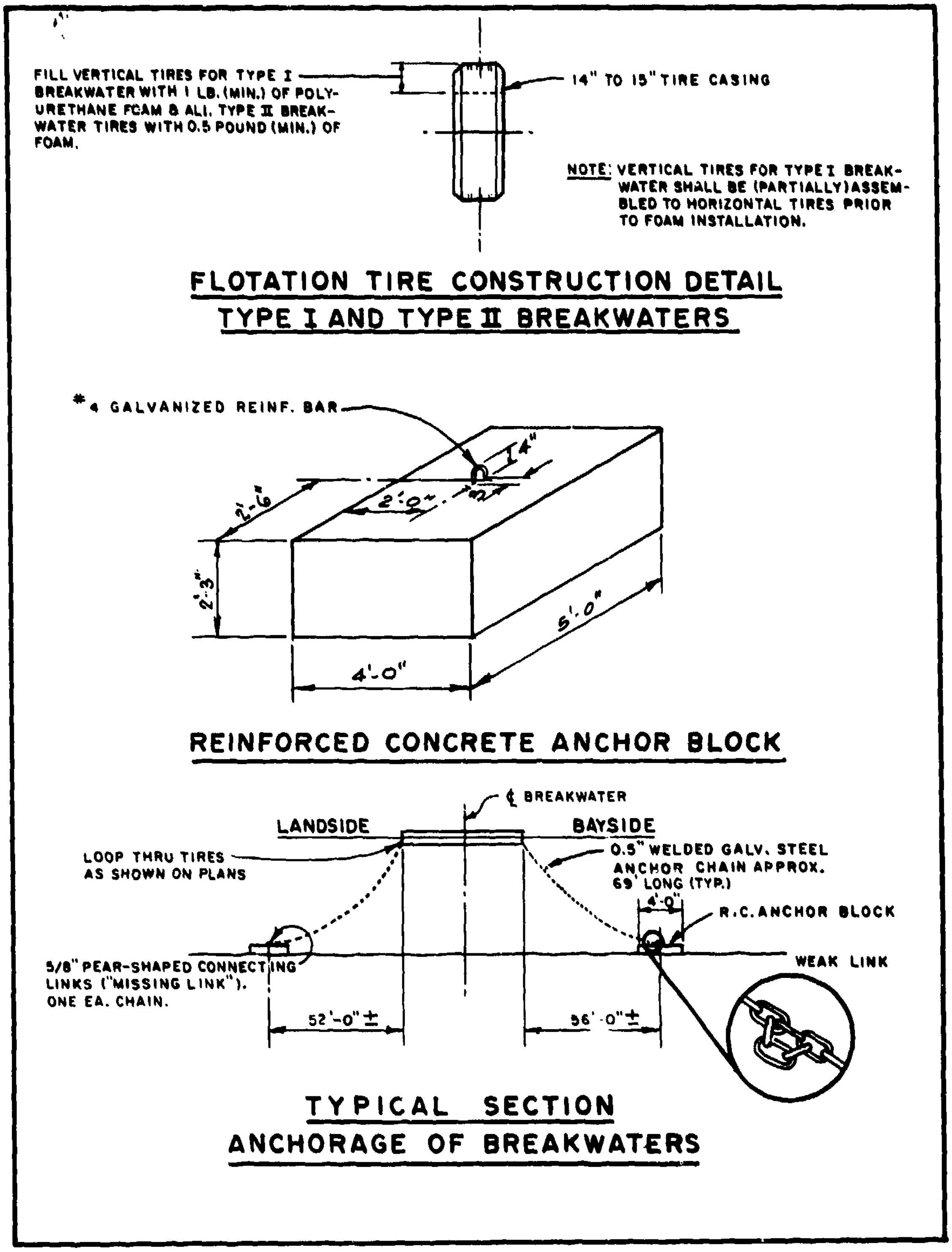


Figure 2-7. Flotation and anchorage of breakwaters at Pickering Beach site.

Figure 2-7. Weak test links were made by fabricating three different sizes of bar stock into split links, which were then tested to determine the force required to open each size. These links were then inserted as indicated in 12 of the 0.5-inch anchor chains to check forces to which the chains were subjected by anchorage stress.

(3) Statistics, Costs, and Construction. The statistics and contract costs for the two types of floating tire breakwaters are given in Table 2-3.

The concrete anchors for the four breakwater sections were cast at Fortescue, New Jersey, and barged across the bay for placement at the site. The used tires were delivered to a stockpile at Pickering Beach within easy haul distance of the site. An open shed or lean-to was erected on the beach to shelter the working area, and a launching ramp 20 feet wide by 50 feet long was constructed for assembly and launching of the tire modules.

Construction of the Goodyear breakwater was undertaken first. A Gusmer polyurethane machine was used to spray 0.5 pound of foam into each tire before assembling the modules. The contractor set the 44 anchors on 17 August 1978. The tops of these anchors were only visible at low tide or when a strong wind kept the tide out. A few days later, the first 30 modules were towed to the site during high tide and secured to the anchors. Bolt holes predrilled in the conveyor belt edging used for strapping the tires into modules were mislocated, and the straps had to be field-punched before bolting to make the modules compact. Some delays in construction resulted from a northeasterly storm on 9 September 1978 which damaged the work boat, from activities of another contractor laying discharge pipe for the dredging operation through the project site, and from rains. The Goodyear breakwater was completed 18 September 1978.

Assembly of the Wave-Maze breakwater began with the punching of boltholes in the tires with a punch press. In this breakwater only the vertical tires were foamed--1 pound of foam each. The 20-foot section was assembled first. As partial sections of breakwater were completed, the assembly was pulled out into the water, leaving the uncompleted end on the ramp. At 50-foot intervals, the completed work in the water and the modules on the ramp were connected with belting, an option permitted by the specifications. During rough waters on 17 November 1978, the partially completed breakwater section broke in three places. Most failures were due to bolts and washers pulling through the holes in the tires, although some were due to nylon bolts snapping. Further explanation was that the section of structure on the shore was held fast while the section in the water was free to rotate. This induced great stress on the bolt connections, which caused them to pull through (or snap in the case of the nylon bolts). Repairs were accomplished in about 3 weeks, and the 20-foot-wide breakwater section was completed and anchored at the site on 15 December. When roughly 50 feet of the 40-foot section of the Wave-Maze was in place and anchored, work was suspended for the winter.

The statistics and costs for the vegetation plantings at Pickering Beach are given in Table 2-4.

Table 2-3. Floating tire breakwater statistics and costs, Pickering Beach, Delaware.

<table>
<tr><td colspan="2"><u>Type I (Wave-Maze) breakwater</u> Contract cost: $99,242</td></tr>
<tr><td>40-ft section</td><td>Modules - 945 (5 tires ea.)
Tires - 4,725</td></tr>
<tr><td>20-ft section</td><td>Modules - 441 (5 tires ea.)
Tires - 2,205
50-ft test section w/rubber patches and nylon bolts
Completed: 15 Dec. 1978.</td></tr>
<tr><td colspan="2"><u>Type II (Goodyear) breakwater</u> Contract cost: $65,450</td></tr>
<tr><td>40-ft section</td><td>Modules - 180 (18 tires ea.)
Tires - 3,240
Completed - 12 Sept. 1978</td></tr>
<tr><td>20-ft section</td><td>Modules - 90 (18 tires ea.)
Tires - 1,620
Completed - 18 Sept. 1978</td></tr>
<tr><td><u>Anchorage</u></td><td>Contract cost (both systems): $45,925</td></tr>
<tr><td>Concrete blocks</td><td>Modules - 44
Block weight - 4 ton (3 ton submerged)
Reinforced w/#4 rebars on 10-in centers
56 ft off bayside
52 ft off landside</td></tr>
<tr><td>Chains</td><td>Welded steel (galvanized) - 6,072 ft, 0.5 in
Test links - galvanized steel.
<table>
<tr><th>Wire diameter (in)</th><th>"Breaking" strength[1] (lb)</th></tr>
<tr><td>5/16</td><td>400</td></tr>
<tr><td>3/8</td><td>780</td></tr>
<tr><td>1/2</td><td>1,980</td></tr>
</table></td></tr>
<tr><td colspan="2"><u>Landside operations</u></td></tr>
<tr><td>Average crew</td><td>3 workmen, Goodyear; 5 workmen, Wave-Maze
1 part-time supervisor</td></tr>
<tr><td>Equipment</td><td>1 small bulldozer
1 front-end loader
1 vacuum cleaner
1 polyurethane foamer
1 punch press (Wave-Maze only)</td></tr>
<tr><td colspan="2"><u>Water area operations</u></td></tr>
<tr><td>Average crew</td><td>2 operators</td></tr>
<tr><td>Equipment</td><td>1 pusher-type work boat
1 outboard motorboat</td></tr>
</table>

[1] Force required to open link to wire diameter.

Table 2-4. Vegetation statistics and costs, Pickering Beach, Delaware.

Item	Cost
Materials	
Smooth cordgrass	$2,843
Saltmeadow cordgrass	1,949
Cape American beachgrass	748
	$5,540
Labor	
Four men; cost given only as combined cost for three sites (Table 2-2)	
Equipment	
One portable, gas-powered 3-in auger; cost given only as combined cost for three sites (Table 2-2)	

Vegetation plantings were made in two sections, north planting and south planting (Fig. 2-4). Smooth cordgrass was planted below MLW, saltmeadow cordgrass on the beach above MLW, and Cape American beachgrass on the dune ridge above the beach. Specifics of planting for each species in the north and south plantings are:

Smooth cordgrass:

Planted area: south planting 514 by 30 feet; north planting 150 by 40 feet
No. of plants: south planting, 6,860; north planting, 2,700
Method: peat pots (both plantings)
Fertilizer: 1 ounce, 8- to 9-month Osmocote 18-6-12 at time of planting
Planting date: 13-14 June 1979

Saltmeadow cordgrass:

Planted area: south planting 314 by 30 feet; north planting 150 by 40 feet
No. of plants: south planting, 4,200; north planting, 2,700
Method: peat pots (both plantings)
Fertilizer: normal mix (10-10-10); 1st fertilization 2-3 weeks after planting; 2d fertilization 6 weeks after initial fertilization at 500 pounds per acre at each application
Planting date: 3-4 May 1979

Cape American beachgrass:

Planted area; south planting 514 by 10 feet; north planting 150 by 12 feet
No. of plants: south planting, 2,401; north planting, 800
Method: sprigs (both plantings)
Fertilizer: as described for saltmeadow cordgrass
Planting date: 26-28 March 1979

The Cape variety of American beachgrass was planted on 1.5-foot centers during 26-28 March 1979. Sprigs, obtained from Environmental Concern, St. Michael's, Maryland, were furnished in flats containing about 1,000 plants and were planted within 4 hours after arrival at the site. All sprigs were planted in holes approximately 4 to 5 inches deep and about 14 inches wide. Fertilizer was placed in the hole when opened and the sprig inserted to a depth of approximately 4 inches. Loose-textured sand covered the base of each sprig; no additional tamping was considered necessary. At the time of planting, each sprig consisted of at least three upright stems; a small rhizomal mass with attached roots subtended each shoot. Although planting was completed on 28 March, the Initial Post Planting Inventory was not recorded until after the cordgrass planting was completed in June 1979.

Saltmeadow cordgrass was planted on 1.5-foot centers during 3-4 May 1979. The 1.75- by 2.25-inch pots were obtained from Environmental Concern. Five environmental strains of saltmeadow cordgrass were also obtained from the Soil Conservation Service and used in 18 rows of the north planting. The plants were about 3 months old at the time of planting. Peat-potted plants were furnished in water-filled plastic-lined trays containing approximately 48 potted plants. Each plant was placed in holes (drilled with a portable power auger) to the height of the pot and was later fertilized with normal mix (10-10-10). Each peat-pot plant contained 3 to 4 upright stems approximately 12 inches in height. Although planting was completed 4 May 1979, the Initial Post Planting Inventory was not completed until June 1979.

Smooth cordgrass was planted during 13-14 June 1979. The planting method was the same as that used for saltmeadow cordgrass. Plants were supplied by Environmental Concern, and were of the same size and age as the saltmeadow cordgrass. The Initial Post Planting Inventory was made on 14 June 1979 for all plantings.

(4) Performance.

(a) Structural. An aerial photo taken 30 January 1979 showed the tire-breakwater structures intact; however, a massive ice buildup (1 to 2 feet) occurred during February. As a result, the galvanized-steel bolted connections in the Wave-Maze breakwater sections were stressed, deformed, and subsequently pulled through the tires. Figure 2-8 (upper photo) shows the incipient breakup of the two Wave-Maze sections. The 50-foot test section with nylon bolts, washers, and nuts, and with additional rubber patches, was displaced but somewhat less damaged. On 13 April 1979, high waves separated two sections (of 30 and 75 modules) from the Wave-Maze and floated them ashore. The larger section drifted 0.25 mile to the north. The Wave-Maze as modified for this installation was considered a successful failure and was removed (Fig. 2-9). The Goodyear breakwater sections did not separate but were deformed by unequal drag of the anchors.

In the summer of 1979, the concrete-block anchors for the Goodyear installation were salvaged. New anchors were provided, consisting of untreated-oak piles, approximately 12 inches in butt diameter and 25 feet long, driven to or cut off just below MLW. Anchor chains were fastened to the piles with galvanized-steel eyebolts, two bolts to a pile, each alined in the pull direction. The entire Goodyear breakwater structure was reinstalled

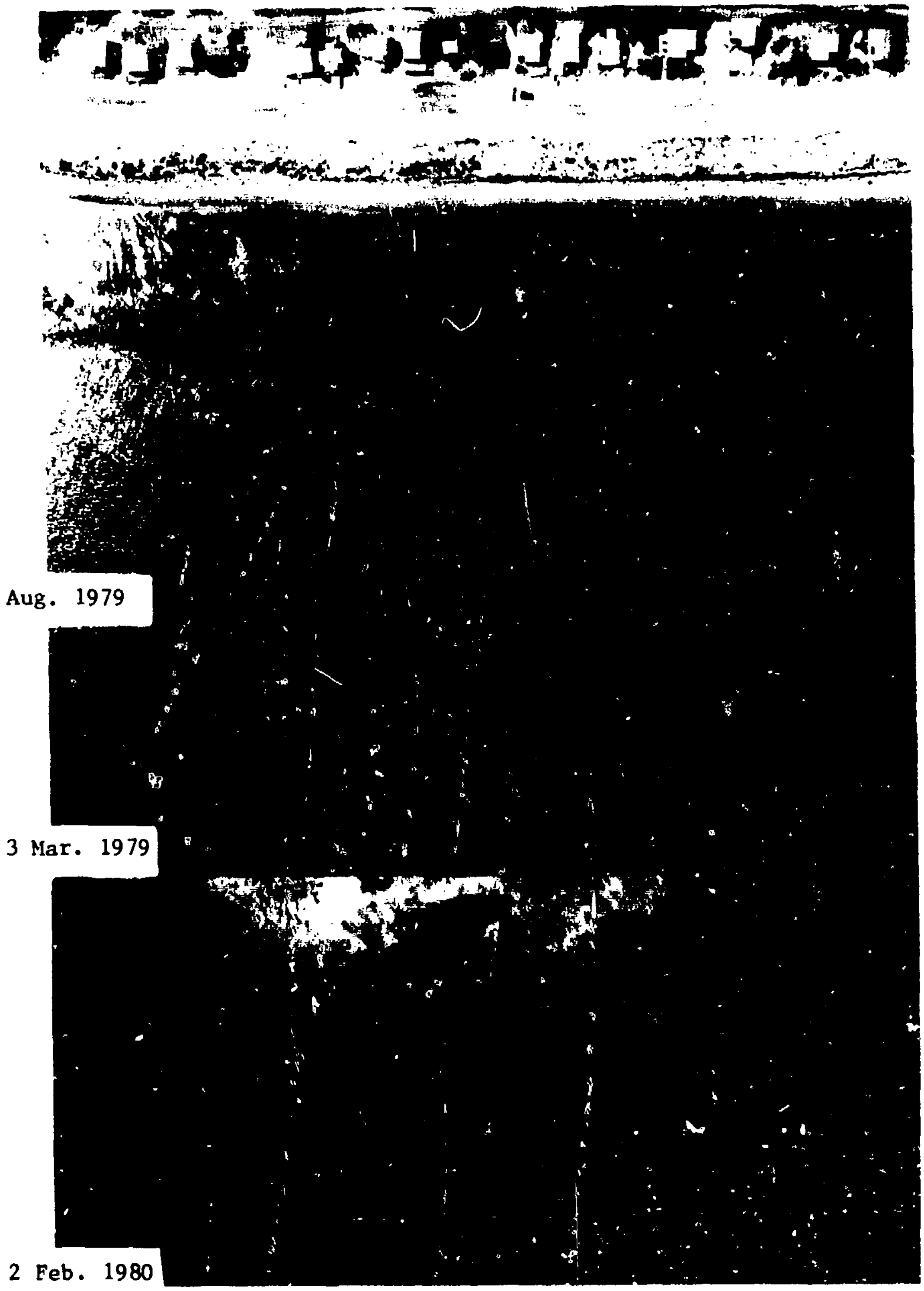


Figure 2-8. Comparative aerial photos of breakwaters at Pickering beach, Delaware.

Figure 2-9. Beached Wave-Maze tires with pulled-through washers, Pickering Beach, Delaware, April 1979.

300 feet closer to shore (Fig. 2-4). Work was completed in August 1979. The anchors held during the fall, but by February 1980 the Goodyear breakwater was deformed as shown in Figure 2-8 (lower photo). By June 1980, the north end of the 20-foot section had apparently broken loose from its mooring and had shifted shoreward to an orientation roughly perpendicular to the main section (Fig. 2-10). The breakwater appeared to have little effect on the shoreline.

(b) Vegetation. Both systems of the vegetation plantings were outside the influence of the breakwaters, having been planted after the Wave-Maze was removed. Survival of smooth cordgrass was poor in both the north and south plantings. At midseason (July 1979), 20 percent of the plants remained in the north planting and 40 percent in the south planting. By October 1979, the end of the growing season, no plants remained in either planting.

Survival of saltmeadow cordgrass gradually decreased over the first growing season, with lowest survival in the north planting. In July 1979, midseason survival was 35 percent of the original plants in the north and 82 percent in the south; by October 1979 this decreased to 12 percent and 37 percent, respectively. In May 1980 only a few individual plants (less than 2 percent) remained in either planting. Evidently growth during the summer of 1979 was not sufficient to allow the plants to survive winter storm waves.

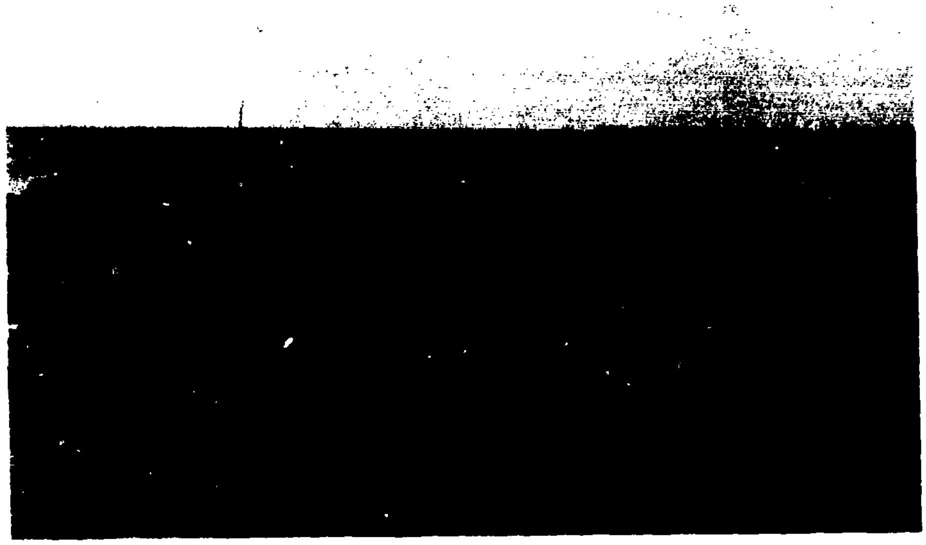


Figure 2-10. Goodyear breakwater misalinement, Pickering Beach, Delaware, 20 June 1980.

American beachgrass did well in both plantings. Midseason survival was lower in the north planting, with 45 percent of the initial plants remaining; approximately 30 percent remained at the end of the growing season in October. In the south planting, 90 percent of the beachgrass was still alive at midseason, but this decreased to 40 percent by the end of the summer. In May 1980, growth of this species was vigorous at both sites, with little or no additional mortality over the winter. Beachgrass appears successfully established at this site. Figures 2-11 and 2-12 show the vegetation plantings at Pickering Beach in September 1979.

The appearance of many plants in the north planting indicated that the root mass was too small at the time of planting and growth too slow to permit establishment during the hot summer months. Additional time in the greenhouse, before planting, may improve the performance of this species.

(5) Analysis.

(a) Tire Breakwater. Throughout the monitoring period, the floating breakwater sections appeared to have little effect on the sand beach. However, the survey profiles indicate about 1 foot of accretion on the offshore bottom between the reinstalled Goodyear breakwater and shore (Fig. 2-13). Bottom shadows in the March 1980 photo (Fig. 2-8) show the outline of this accretion. Figure 2-4 shows two sets of contours of the shore and offshore bottom: one plotted from a survey made just before the Goodyear breakwater was reinstalled in 1979 and one plotted from a survey made a year later. The minus contours also depict the configuration of the underwater accretion, but the shore contours have remained essentially in

Figure 2-11. Vegetation plantings at Pickering Beach, looking south, 12 September 1979.

Figure 2-12. Vegetation plantings at Pickering Beach, looking north, 12 September 1979.

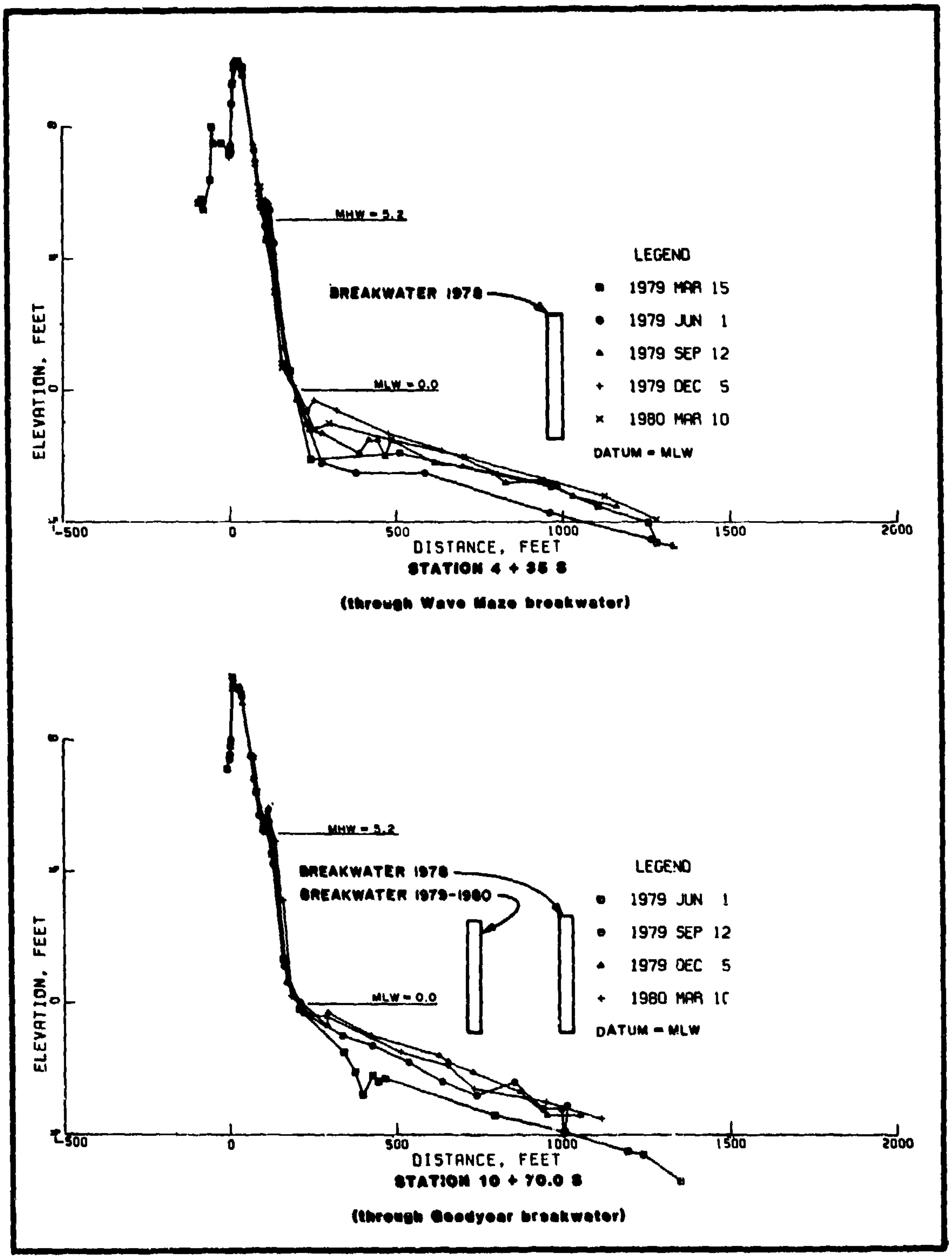


Figure 2-13. Typical profiles, Pickering.

their original locations. Additional profiles are not presented because the contours provide a better depiction of the effect of the breakwater on littoral sediments. Although the accretion probably consists of fine material, its continued growth, together with wave attenuation by the floating structure, should provide considerable beach protection. The monitoring period ended before the test links in the anchor chains were checked to determine the amount of stress to which they had been subjected. The failure of the anchorage system at the north end indicates that the design load may have been exceeded. The demonstration showed that the Wave-Maze, as modified, pulled apart and therefore was unsuccessful. The Goodyear breakwater was partially successful and probably could be made fully successful with improved anchorage.

(b) Vegetation. The two planting sites outside the protective lee of the floating tire breakwaters were fully exposed to bay waves. These sites are apparently unsuitable for smooth cordgrass and are marginal for saltmeadow cordgrass at least on the lower beach. The sandy substrate and the constant wave action appear to preclude successful planting of smooth cordgrass. This problem is compounded by the annual appearance of horseshoe crabs which mate along the shore and lay eggs on the beach. The crabs appear in May and completely cover entire stretches of shoreline; by July this covering extends up to the dune. The crabs burrow in the sand in the water and on the beach. Because of their shell structure and pattern of movement, they are very effective bulldozers, removing all traces of vegetation where they pass. Because this area of Delaware Bay is a traditional mating ground for horseshoe crabs, and because they mate during the same time of year as the growth of cordgrass takes place, it seems unlikely that cordgrass plantings will ever be successful at this site. Beachgrass appears to do well if allowed a period of establishment, and could be expected to aid in controlling erosion of the higher beach areas.

c. Kitts Hummock, Delaware.

(1) Site Description.

(a) Geographical Setting. Kitts Hummock, about 3 miles south of Pickering Beach and 8 miles southeast of Dover, is a small fishing resort community extending approximately 0.5 mile along the bay shore. The community consists of about 100 houses, most of which are summer cottages. About one-half of these cottages are located along the landward edge of the dune.

(b) Geomorphology and Soils. Beach material at Kitts Hummock consists of granular soils ranging from medium- to fine-grained sands to fine gravel. The beach extends approximately north to south, is 40 feet wide from the dune to the high water line, and slopes at approximately 1 on 10. The offshore zone has a slope of approximately 1 on 150. The dune, which parallels the beach, is about 12 feet above MLW at its crest and has a base width of about 20 feet. The beach was last nourished in March 1979.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 2.2 feet, and net longshore transport potential northward was 34,600 cubic yards for the 3 months analyzed.

(2) Demonstration Project. The installation comprised vegetation planting and a series of low, fixed offshore breakwaters utilizing three structural devices: rubble mound, nylon sandbags, and precast concrete boxes, as shown in plan view in Figure 2-14. The rubble-mound and concrete-box breakwaters are each 330 feet long, and the nylon sandbag breakwater is 336 feet long. The breakwaters are separated by 350-foot gaps. This spacing subjects each structure to the same wave climate while avoiding any functional interferences due to reflection or diffraction. The spacing also allowed an independent evaluation of the structural integrity of each segment and its relative functional behavior. Initial crest elevations were midway between the high and low water planes. Because the structures were founded on 2 feet of loose silty material, considerable settlement on the bottom was anticipated. Filter cloth was placed under the entire length of the sandbag section. The rubble-mound segment comprised two types of design: 550- to 950-pound stone on filter cloth; and 800- to 1,200-pound stone on 1 foot of 1- to 40-pound matstone. Typical profiles and sections are shown in Figures 2-15 and 2-16.

The structural work was done between October 1978 and August 1979 by a general contractor under contract with the U.S. Army Engineer District, Philadelphia.

(3) Statistics, Costs, and Construction.

(a) Rubble-Mound Breakwater. The statistics and costs for the rubble-mound breakwater are given in Table 2-5.

The rubble-mound breakwater was constructed between April and August 1979. The stone was placed by a clamshell crane mounted on a work barge anchored on the bayside of the breakwater. The stone transport barges were towed in and anchored on the offshore side of the work barge as construction progressed. The work barge was generally held in place with three anchors and two spuds. The spuds were undersized for the barge's spud wells; on several occasions, the work barge shifted or broke anchor and came to rest on the breakwater.

The construction started at the north end and proceeded southward. The matstone was dumped first, then the armor stone (Fig. 2-16, sec. C-C). On the average, the stones were dropped from a height of 1 to 3 feet above the water surface onto the previously placed stones. No stones were dropped from a height of greater than 5 feet. When the first half was completed, placement of filter cloth for the next 165 feet of armor stone began.

The initial stone placement was done by eye; no optical instruments or line stakes were used. As a result, the initial alinement was poor, the section was irregular, and many stones in the armor layer were undersized and not properly seated. The contractor was instructed to remove the undersized

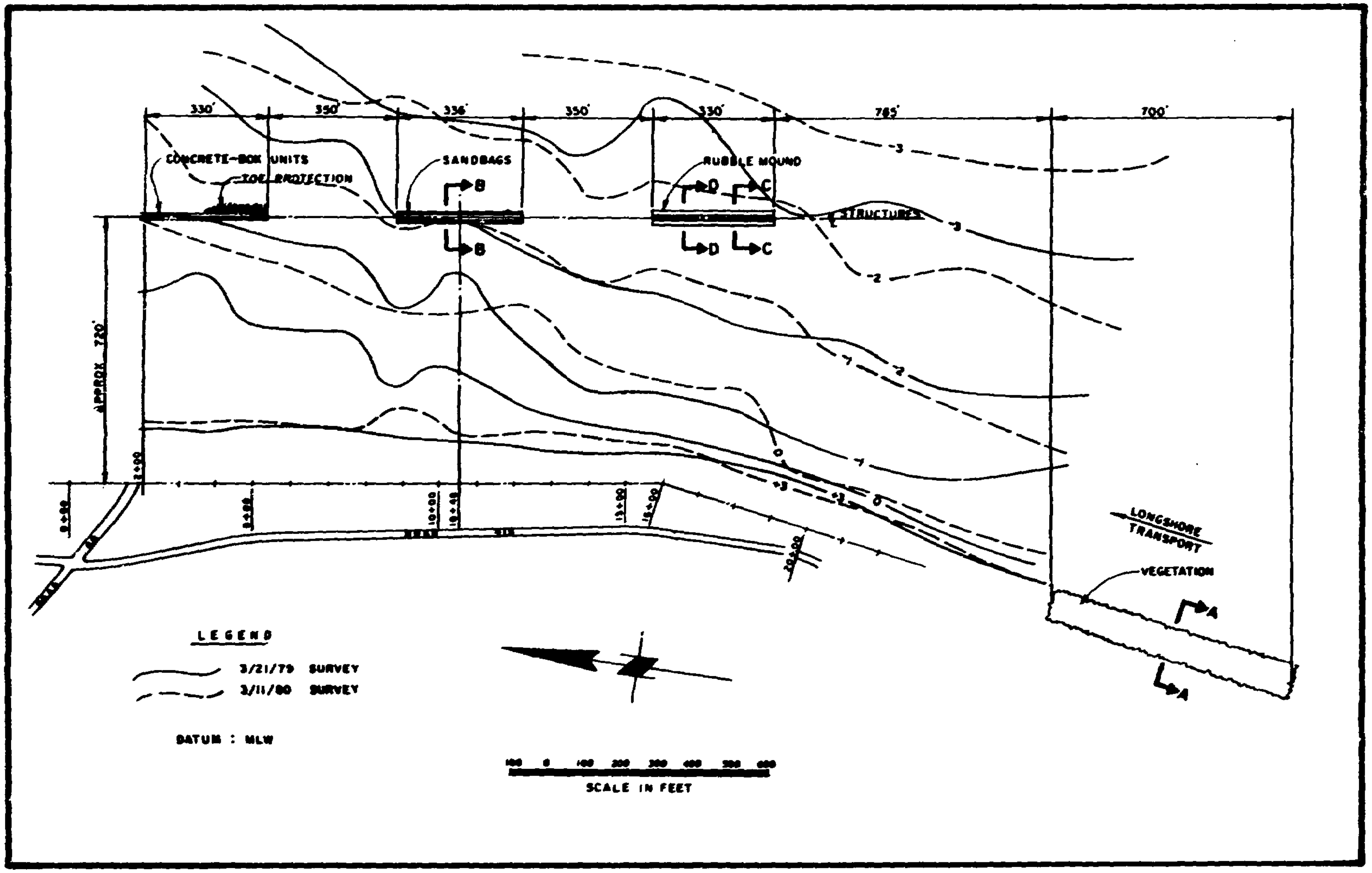


Figure 2-14. General plan of Kitts Hummock, Delaware, demonstration site.

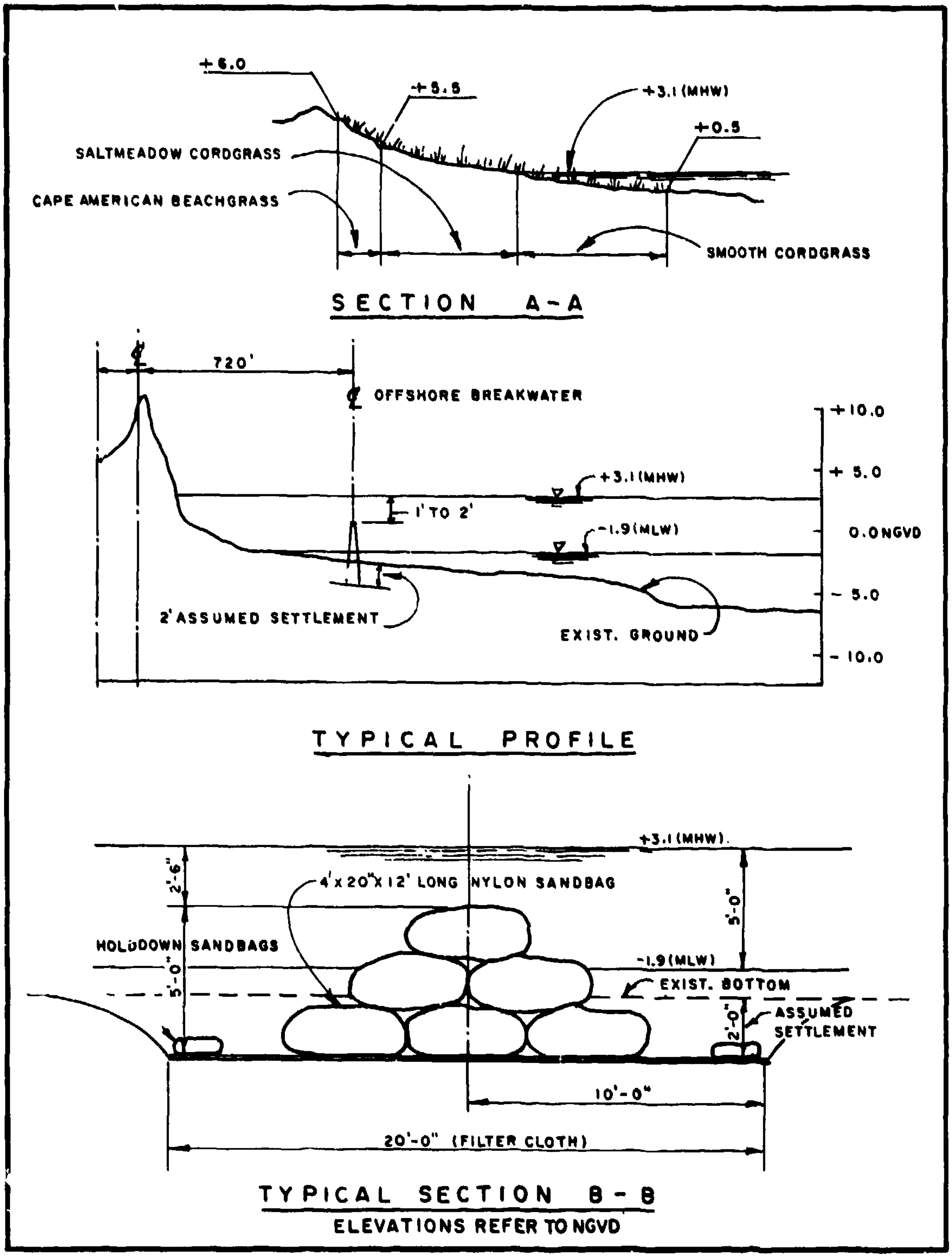


Figure 2-15. Typical sandbag breakwater at Kitts Hummock site.

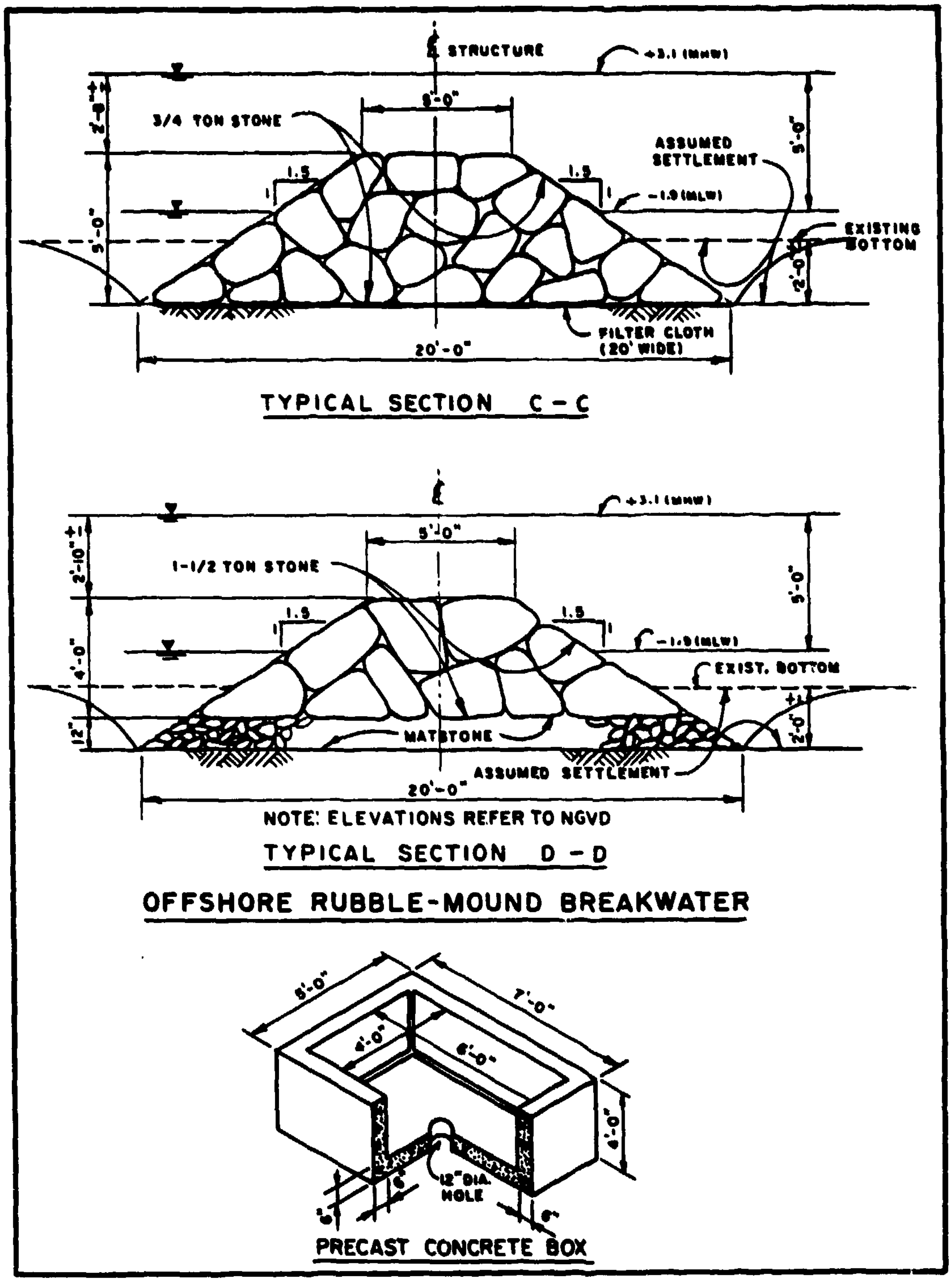


Figure 2-16. Rubble-mound and concrete-box breakwater sections at Kitts Hummock site.

Table 2-5. Statistics and costs for the rubble-mound breakwater and operations.

Rubble-mound breakwater	Contract cost: $69,865	
Item	Description	
	North half	South half
Length	165 ft	165 ft
Rock	800 to 1,200 lb	550 to 950 lb
Quantity	770 ton approx.	770 ton approx.
Cost	$4,793	$4,793
Filter type	1- to 40-lb matstone	Fabric filter cloth
Quantity	221 ton	3,300 ft^2
Cost	$1,370	—
Completed	July 1979	August 1979
	Operations	
Average Crew	5 workmen, 1 supervisor	
Equipment	1 barge crane w/clamshell 1 rock barge 1 tugboat	

stone and rebuild the section to specifications, individually seating the armor stones. In the south half, an attempt was made to place the small, undersized stone on the filter cloth first, thus allowing it to be covered before the breakwater was built to full height with larger stone (Figs. 2-17 and 2-18).

During the construction the following observations were made:

(1) Winds up to 25 miles per hour interfered little with stone placement.
(2) The circumstances of equipment used and work-barge placement made it impossible for the crane operator to see where the stone was being placed.
(3) Some type of sighting instrument was needed to assure proper alinement of the breakwater.
(4) Poorly maintained equipment resulted in a significant amount of lost time.

(b) Nylon Sandbag Breakwater. The statistics and costs are given in Table 2-6.

Construction of the nylon sandbag breakwater began at the north end in October 1978. The operation was carried out utilizing a work barge anchored on the exposed side of the breakwater site. Sandfilled barges were towed in from Bowers and moored to the offshore side of the work barge. Work was initiated by placing a segment of filter cloth and anchoring it with small hand-filled burlap sandbags. Large empty nylon sandbags were then laid out in their intended locations and filled with

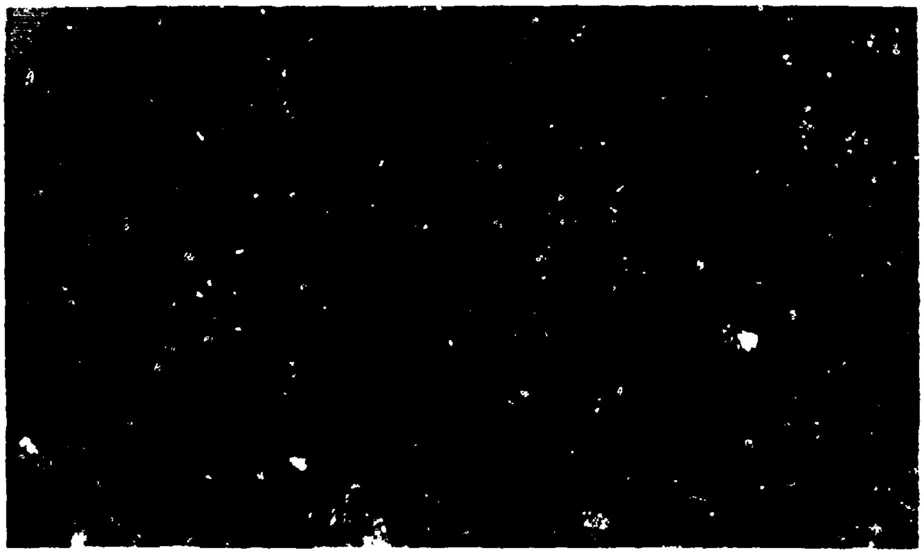


Figure 2-17. Quarrystone breakwater, Kitts Hummock, Delaware, 6 August 1979.

Figure 2-18. Initial construction of quarrystone breakwater, Kitts Hummock, Delaware, 6 August 1979.

Table 2-6. Statistics and costs for the nylon-sandbag breakwater.

Nylon sandbag breakwater (336 ft)		Contract cost: $52,245	
Item	**Description**	**Quantity**	**Cost**
Bag dimensions	4 ft wide by 12 ft long by 1.7 ft high	140 bags	$7,000
Nylon sandbag	Advance Bag	140 bags	7,000
Filter type	Laurel Erosion Control filter cloth	8,232 ft^2	1,317
Sandfill	Commercial grade sand for bags	560.7 ton	3,364
Holddown bag	100-lb burlap sand-bags	--	--

sand using a jet-pump system. A sand and water slurry was pumped into two bags simultaneously, the water escaping through the mesh of the fabric and leaving the bags filled with sand. The crane operator transferred sand from the cargo barge to the mixing tank on the work barge. One worker was stationed at the open top of the tank to regulate the slurry mixture, while two workers in wet suits directed the slurry into the bags. With the work half completed about mid-Decemeber, work was suspended because of bad weather. The remainder of the sandbag breakwater was completed in spring 1979 (Figs. 2-19 and 2-20). Throughout construction operations, the performance of the jet-pump system was "fair" at best. Adjustments to the specifications of the sandbag breakwater were required due to the following:

(1) The shipment of holddown sandbags were undersized and the contractor weighted down the filter cloth with loose sand.

(2) During construction, 1- to 2-foot gaps on the bottom layer of three bags were filled with loose sand to make an even surface for the next layer of bags.

(c) Concrete-Box Breakwater. Statistics and costs are given in Table 2-7.

The concrete-box breakwater was constructed during November 1978. Again the contractor used a work barge anchored on the exposed side of the breakwater site. Transport barges loaded with concrete boxes (Fig. 2-16) were towed from Bowers and moored to the offshore side of the work barge. The work barge crane placed the boxes end-to-end on the breakwater alinement. Gaps between boxes averaged 6 to 12 inches.

Placement of the concrete boxes went smoothly with two exceptions. First, the contractor mistakenly laid out filter cloth before placing the concrete boxes on location. The second difficulty arose when the work barge lost anchor and came to rest on the concrete-box breakwater. This probably caused the misalinement of the structure. On completion of box placement, the boxes were filled with commercial grade sand from Dover by the work barge crane, and the specified rock toe protection was placed along the southern half of the breakwater.

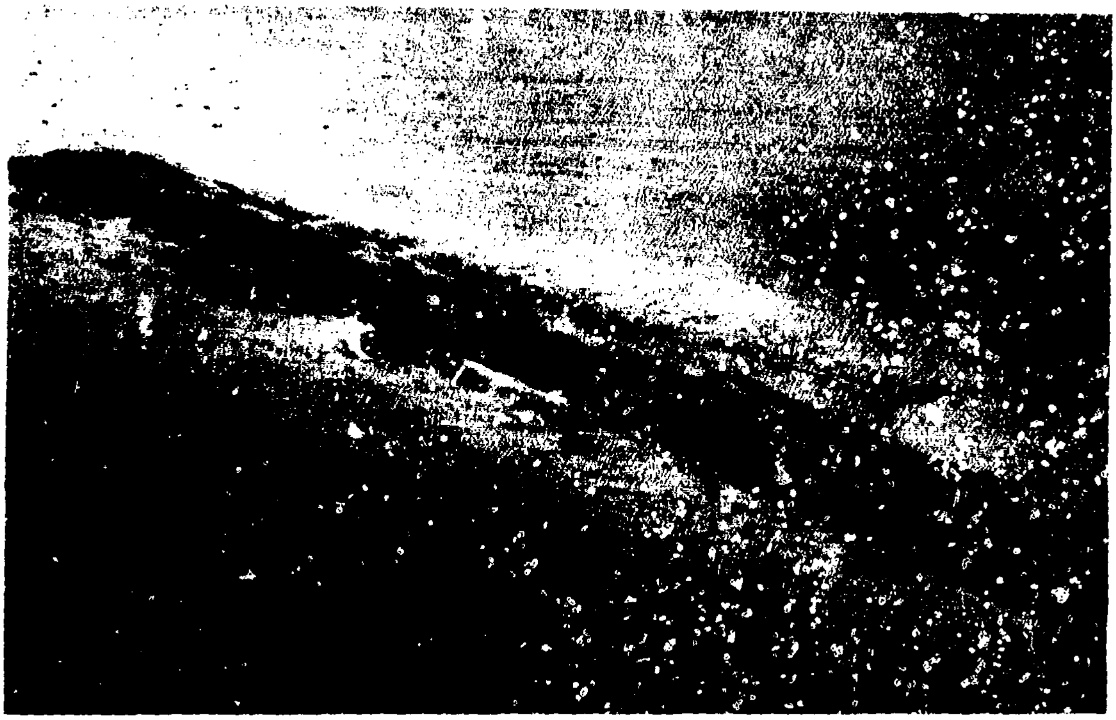


Figure 2-19. Nylon sandbag breakwater at Kitts Hummock, Delaware, 14 May 1979.

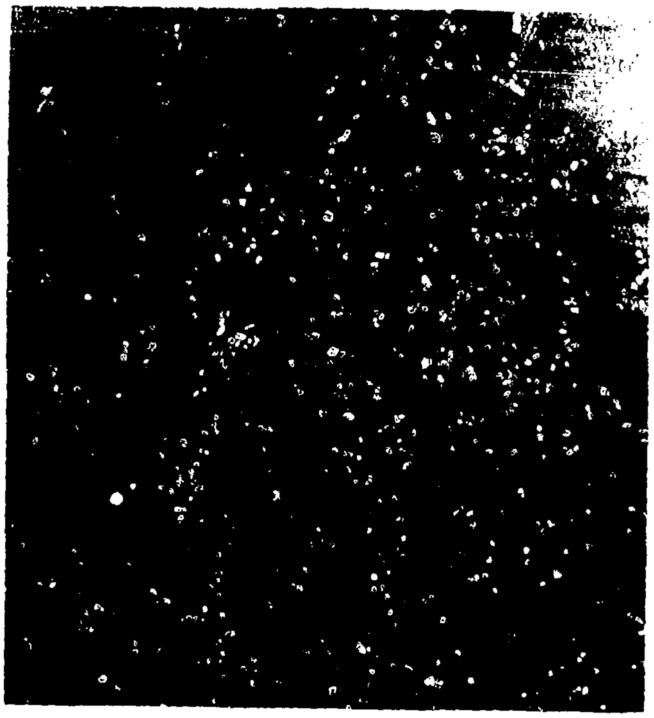


Figure 2-20. Nylon sandbag breakwater, Kitts Hummock, Delaware, 6 August 1979.

Table 2-7. Statistics and costs for the concrete-box breakwater.

Concrete-box breakwater (329 ft)		Contract cost:	$39,490
Item	Description	Quantity	Cost
Concrete boxes	5- by 7- by 4-ft, with 6-in walls; no covers (5,000 lb/in^2, std. 420 bridge-load reinforcement)	47	$23,500
Toe protection	1- to 40-lb matstone	27 yd^3	--
Filter cloth (mistakenly placed)	Laurel Erosion Control filter cloth	1,680 ft^2	269
Sandfill	Washed concrete sand	per box	--
Equipment	Work and transport barges	8 hr	450

(d) Vegetation Planting. Vegetation was planted along the beach as shown in Figure 2-14. Development of the planting scheme was coordinated with the Soil Conservation Service. Although the optimal planting season is in the spring, the vegetation was not planted until July 1979. The statistics and costs are given in Table 2-8.

Smooth cordgrass, saltmeadow cordgrass, and American beachgrass were planted at Kitts Hummock. The position of each species was the same as at Pickering Beach, with smooth cordgrass below MLW, saltmeadow cordgrass on the beach between MLW and MHW, and American beachgrass above MHW. Due to a delay in preparing the site, the planting was delayed between 3 weeks (cordgrass) and 3 months (beachgrass). This delay and other problems resulted in all plantings being located in an unprotected area on the south end (Fig. 2-14), not behind the structures as originally planned.

Table 2-8. Statistics and costs for the vegetation plantings.

Materials	Cost
Smooth cordgrass	$ 828
Saltmeadow cordgrass	759
American beachgrass (Cape variety)	209
	$1,796
Additional costs incurred as a result of delay in planting:	
Pots	$ 187
Labor	768
Maintenance	100
Additional transportation	290
Planting labor	420
	$1,765
Total	$3,561

The statistics for each species are:

Smooth cordgrass:

Planted area:	700 by 30 feet
No. of plants:	9,340
Method:	peat pots
Fertilizer:	1 ounce, 3- to 4-month Osmocote at time of planting
Planting date:	3-6 July 1979

Saltmeadow cordgrass:

Planted area:	700 by 30 feet
No. of plants:	9,340
Method:	peat pots
Fertilizer:	as described for smooth cordgrass
Planting date:	3-6 July 1979

American beachgrass (Cape variety):

Planted area:	700 by 30 feet
No. of plants:	3,269
Method:	potted plants, sprigs
Fertilizer:	as described for saltmeadow cordgrass
Planting date:	3-6 July 1979

All species were planted between 3 and 6 July 1979. A total of 3,269 American beachgrass plants (obtained from Environmental Concern) was planted on 1.5-foot centers. Because of the delay in site preparation, the sprigs were moved to pots and held until the site was ready. The plants were transported by truck and were planted within 3 days of arrival. The planting method and fertilization were the same as described for Pickering Beach, except that 3- to 4-month (release) Osmocote was used rather than the 8- to 9-month type. Planting was completed 6 July 1979, and the Initial Post Planting Inventory was made at this time.

About 9,340 plants each of saltmeadow cordgrass and smooth cordgrass were planted at Kitts Hummock. Materials were supplied by Environmental Concern in peat pots and were planted in the same manner as described for Pickering Beach. The Initial Post Planting Inventory was made 6 July 1979.

(4) Performance.

(a) Rubble-Mound Breakwater. The structure maintained good alinement but the north end, which is founded on matstone, settled 0.5 foot just after construction. No further settlement in either half of the structure occurred after this. There was no loss of stone.

(b) Nylon Sandbag Breakwater. In spring 1979, it was discovered that most of the bottom offshore-side bags in the half-completed section had been displaced seaward, allowing the top bags to settle. Many bags were not completely filled, and some had been torn by ice or floating debris. The damaged bags were repaired when the remainder of the section was completed. A filter neck on a shoreside bag was open. Although the breakwater had settled 1 or 2 feet into the bottom, its functional performance was not considered to be significantly impaired.

The March 1980 inspection revealed that the sandbags had begun to deteriorate. The bags were pulling apart at the seams, allowing the sand to flow out. A sample of the thread used to sew the seams was tested and revealed a breaking strength of from 1.6 to 2.0 pounds. The bags had been in the water for over a year, but the manufacturer's specifications stated that saltwater has no effect. It was not determined whether this applies only to the bag material or also to the thread. Regardless, the thread deteriorated at a rate that would preclude any productive use of these bags in similar environments.

(c) Concrete-Box Breakwater. An inspection following the 1978-79 winter storms revealed that the boxes in the northern half were misalined by a few feet and some were slightly skewed. Some boxes were tilted longitudinally by as much as 18 inches, and only 6 to 12 inches of sandfill remained in each box. None of the boxes were structurally damaged, and the breakwater was considered functionally unimpaired. The missing sand was replaced, and in May 1979 the structure appeared as shown in Figures 2-21 and 2-22.

(d) Functional Effectiveness. Figure 2-14 shows two sets of contours of the shore and offshore bottom, plotted from surveys made in March 1979 and March 1980, respectively. Although the structures were installed in fall 1978, winter ice prevented wave disturbance of littoral sediments most of the time preceding the 1979 survey, and that survey is generally representative of preconstruction conditions. The beach at Kitts Hummock was renourished by the State of Delaware in August 1979 with an unknown quantity of fairly coarse sand. The small advance of the 1980 +3-foot contour in the lee of the structures gives some indication that the breakwaters were at least partially effective in preventing loss of the beachfill. However, the monitoring period was too short for a positive determination of the effectiveness of the structures. The significant advance of the minus contours shows that an underwater accretion, probably of very fine material, has occurred as a result of wave-energy absorption by the breakwaters. Figure 3-77, which was put in Section III for comparison of profiles through similiar devices, shows a series of profiles taken through the sandbag breakwater at Kitts Hummock. These profiles indicate the progressive buildup of the bottom accretion. As was the case at Pickering, the continued growth of this offshore accretion, together with wave attenuation by the breakwaters, should provide considerable beach protection.

(e) Vegetation. None of the species planted at this site became (well) established by the end of the first growing season in October 1979. Smooth cordgrass was completely gone by October, and had only 20-percent survival 3 weeks after planting. Saltmeadow cordgrass had the same 20- percent survival 3 weeks after planting, but only 3 percent of the original planting remained by October. Only a few plants of saltmeadow cordgrass were visible in May 1980.

Beachgrass also did very poorly at this site. Midseason survival was about 40 percent, but the end-of-season count showed only 7 percent of the original plantings remaining. In May 1980, only the uppermost row of beachgrass remained. Figures 2-23 and 2-24 show the vegetation plantings in December 1979.

Figure 2-21. Concrete-box breakwater, viewed from the sout. , Kitts Hummock, Delaware, 14 May 1979.

Figure 2-22. Skewed concrete boxes of breakwater, viewed from the north, Kitts Hummock, Delaware, 24 April 1980.

Figure 2-23. Vegetation plantings at Kitts Hummock, Delaware, looking south, 30 December 1979.

Figure 2-24. Vegetation plantings at Kitts Hummock, Delaware, looking north, 30 December 1979.

(5) Analysis.

(a) Rubble-Mound Breakwater. Though this device performed well, some construction procedures should be noted as critical to the structure:

(1) Proper optical instruments or line stakes should be used when laying stone for a rubble-mound breakwater. Placement by eye resulted in improper alinement, irregular section, and undersized stones in the armor layer.

(2) Proper seating of armor stones is needed to prevent their displacement by wave action. Before the section was rebuilt, the armor layer was not seated properly and the stones were more easily displaced.

(3) Filter cloth was more effective than matstone in preventing settlement of the mound. The north end was constructed with matstone and settled 0.5 foot after construction was complete. If matstone is used at a site where settlement may be a problem, allow for settlement by overbuilding.

The rubble-mound breakwater functioned as intended--to protect the beach from erosion.

(b) Nylon Sandbag Breakwater. Although its functional performance was good, this breakwater did settle 1 to 2 feet and the sandbags suffered small tears from ice and floating debris. Settling of the bags due to displacement and the deflated condition did not affect the structure's performance. Wave action on the breakwater soon washed out the sand used to fill the voids between the bags, causing settling to continue. Therefore, sand proved to be a useless method of compensation for the gaps. Continued monitoring will be required to determine the life expectancy of the bag seams and the bag fabric.

An additional problem encountered was Velcro fasteners of the filling necks. These fasteners were very unstable and tended to loosen and deteriorate from wave or chemical action. A substantially better system is the self-closing system of the Dura-Bag.

(c) Concrete-Box Breakwater. The device performed well, protecting the beach. The only problems were alinement of the structure and tilting of the boxes along their longitudinal axes. One possible solution at future sites would be to use some kind of toe protection on the seaward side so that toe scour is eliminated and tilting is prevented.

(d) Vegetation. Survival of all plantings at this site was greatly reduced due to the lateness of planting and to the lack of protection. The smooth cordgrass apparently could not become established without protection and most plants were uprooted by wave action within a few weeks of planting.

The saltmeadow cordgrass was also insufficiently rooted to survive waves and blowing sand. Many plants suffered from drought, apparently due to midsummer planting. In addition, the soil contains more rocks at this site and may have contributed to shearing off of the tops of the plants.

The beachgrass also apparently suffered greatly from the heat and drought after planting. Although these plants were larger than those planted at Pickering Beach, they were unable to break out of the pots and consequently did not obtain sufficient root growth to sustain them during the summer.

The steep slope of the upper beach at this site, appears to be suitable for beachgrass, as indicated from other nearby plantings. The time of planting should be much earlier however, and should probably involve non-potted sprigs rather than potted plants.

The strong wave action and blowing sand at this site probably preclude the successful establishment of cordgrass species.

d. Bowers, Delaware.

(1) Site Description.

(a) Geographical Setting. The incorporated community of Bowers is located at the mouth of the Murderkill River, about 2 miles south of Kitts Hummock, and extends about 0.5 mile along the bay. Many cottages in the area are occupied by permanent residents. Although designated as a demonstration site in the authorizing act, Bowers is in effect only a monitoring site, as it is protected by a groin-retained beach fill placed in 1973 and periodically renourished thereafter by the State of Delaware. No additional shore protection devices were added under the demonstration program at this site.

(b) Geomorphology and Soils. The beach soils at Bowers generally consist of fine to medium sands with some fine gravel, and the dune consists primarily of fine to medium sands. The shoreline extends in a general northwest-southeast direction. The natural beach has been altered by the earlier shoreline protection project (a groin-retained beach fill). The slope of the beach varies from 1 on 6 to 1 on 8; the offshore zone slopes at about 1 on 150. The 30-foot-wide dune behind the beach consists of fine to medium sand and ranges in height from 9 to 11 feet above MLW.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 1 to 2 feet, with a maximum of 4.0 feet, and the net longshore transport potential southward was 34,600 cubic yards for the 3 months analyzed.

(2) Monitoring Project. The beach berm at Bowers has an elevation of +10 MLW and is approximately 70 feet wide. The foreshore is about 40 feet wide, and slopes about 1 on 8 to the flat nearshore bay bottom, which is at approximately MLW datum. The fill is retained by two nylon Dura-Bag groins, one at the south end of Bowers, 750 feet long, and one at the north end, 400 feet long. Details of the project are shown in Figure 2-25. The State has restored the fill essentially to its original condition twice: with 15,825 cubic yards in the fall of 1973 and with 28,800 cubic yards in the summer of 1974. For monitoring, a base line was established as shown on the plan, and profiles were taken periodically at the range lines indicated.

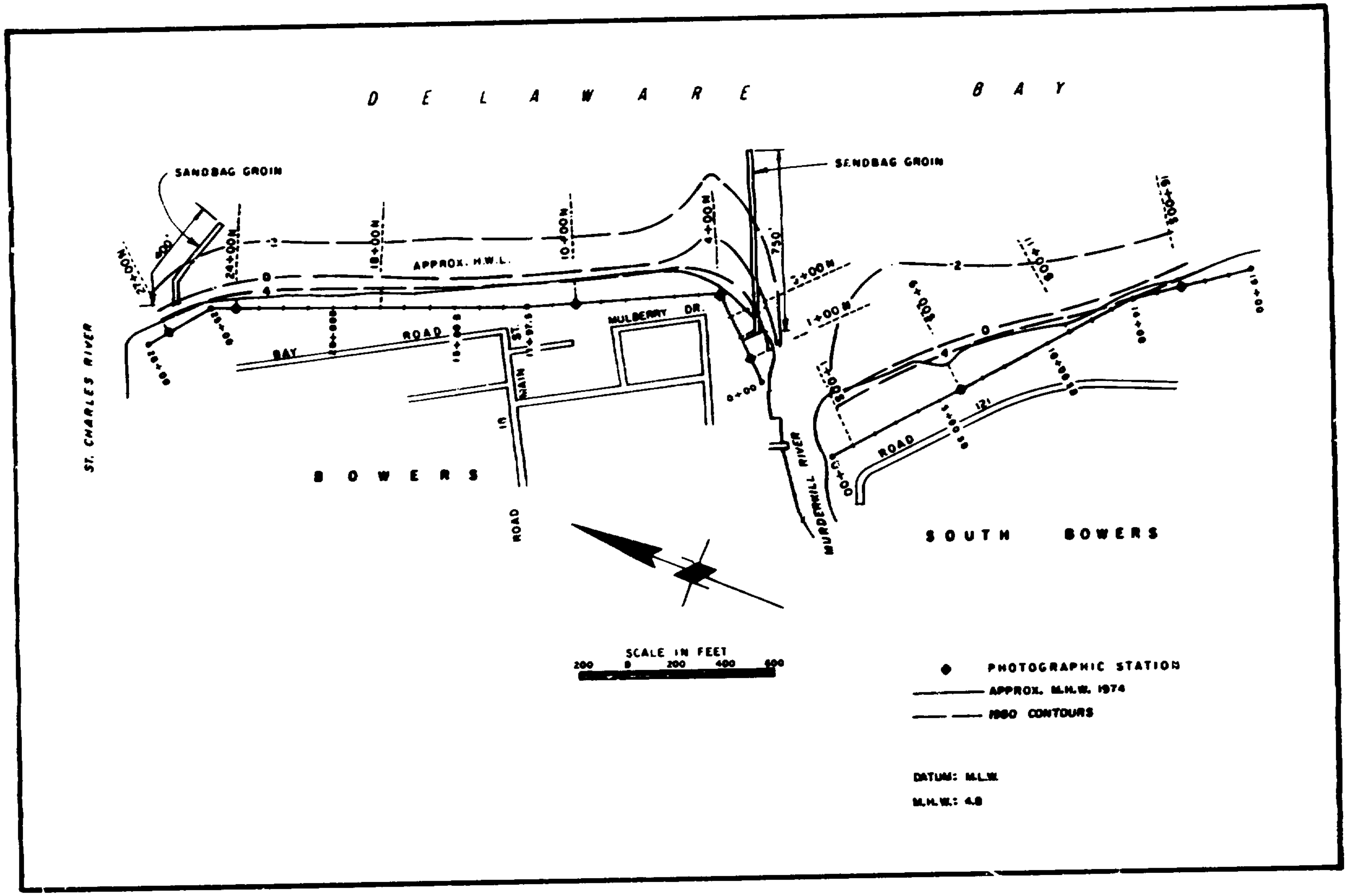


Figure 2–25. General plan of Bowers, Delaware, demonstration site.

(3) Performance. No additional beach nourishment projects have been required at Bowers since the summer of 1974. The groins have been somewhat effective in retaining the beach fill, although considerable erosion has occurred. Figure 2-26 illustrates the performance of the beach fill and groins. Material has accreted on the north side of the south groin, which suggests a dominant northwest to southeast littoral transport. Also, a smaller amount of accretion has occurred just south of the north groin, indicating an occasional change in direction of the littoral transport, i.e., a southeast to northwest transport. The profiles in Figure 2-27 show that about 20 feet of beach fill eroded between April 1979 and March 1980.

(4) Analysis. The combined system of the two nylon Dura-Bag groins and beach fill has performed reasonably well in controlling the shoreline erosion at Bowers. Although the direct protection of the shoreline is attributable to the beach fill, the groins have helped to prevent loss of the fill, and in spite of the erosion that has occurred since 1974, no additional beach nourishment has been required.

e. Slaughter Beach, Delaware.

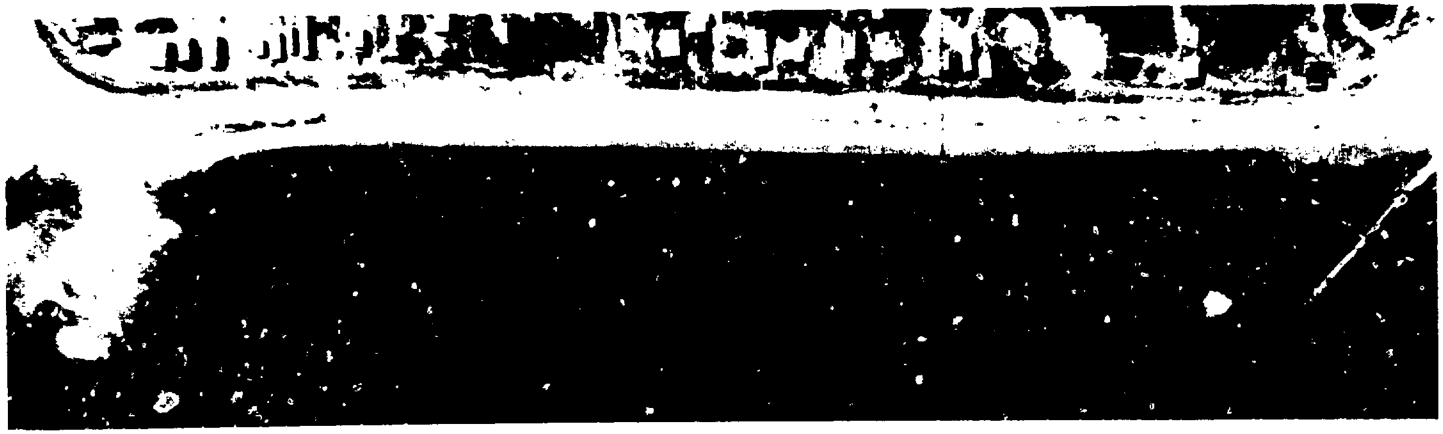
(1) Site Description.

(a) Geographical Setting. Slaughter Beach is approximately 12 miles southeast of Bowers and 13 miles from the bay mouth. It is a small incorporated community, extending for approximately 1.5 miles along the bay shore. About 115 houses are along the beach front, and 35 houses are on the west side of a road paralleling the bay.

(b) Geomorphology and Soils. The beach and berm at Slaughter Beach consist primarily of fine to medium sands. The shoreline at Slaughter Beach is oriented in a general northwest-southeast direction. The foreshore has a slope of approximately 1 on 200. A 70-foot-wide beach, which slopes at approximately 1 on 10, extends from the high water line to a 30-foot-wide berm which has a nearly vertical face on the shoreside. Bulkheads and buildings are situated on the berm. The area behind the berm is mainly marshland, some of which has been drained for farmland.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 3.0 feet. Although a small northward net transport potential is indicated by the energy-flux analysis for the 10-month period of observations, this is the difference between two vastly larger gross potentials in each direction. It is probable that this reach of shoreline is virtually neutral with respect to longshore transport.

(2) Demonstration Project. The device at Slaughter Beach consisted of a "perched beach" using three different types of structural devices for the low sill on which the raised beach "perches": concrete boxes, wood sheet piling, and large nylon sandbags. The structure was installed at the south end of the community (Figs. 2-28 to 2-31). The conventional perched beach has a low-profile retention sill to trap and retain littoral material. Because longshore transport at this site and in the bay is minimal, the conventional design was modified in that the beach was backfilled with sand in lieu of relying on littoral transport for natural deposition.



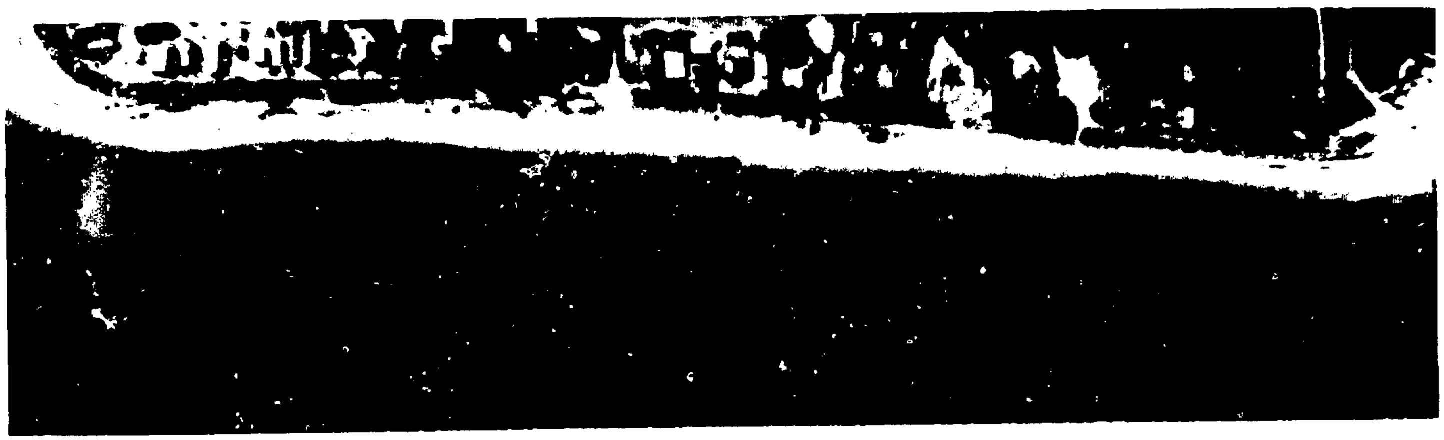


Figure 2-26. Groin-retained beach fill at Bowers, Delaware (top photo, 9 March 1979; bottom photo, 18 October 1979).

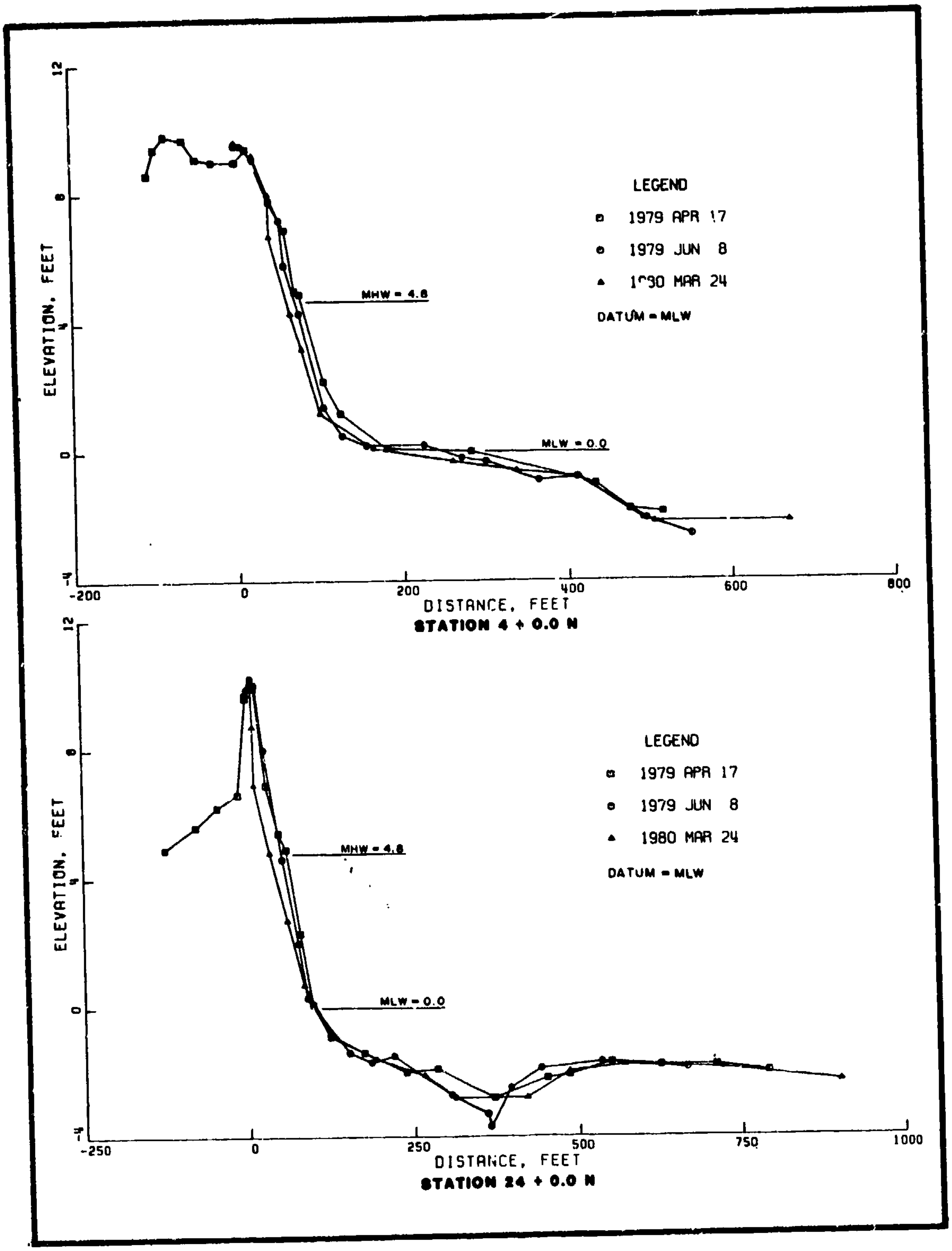


Figure 2-27. Typical profiles, Bowers.

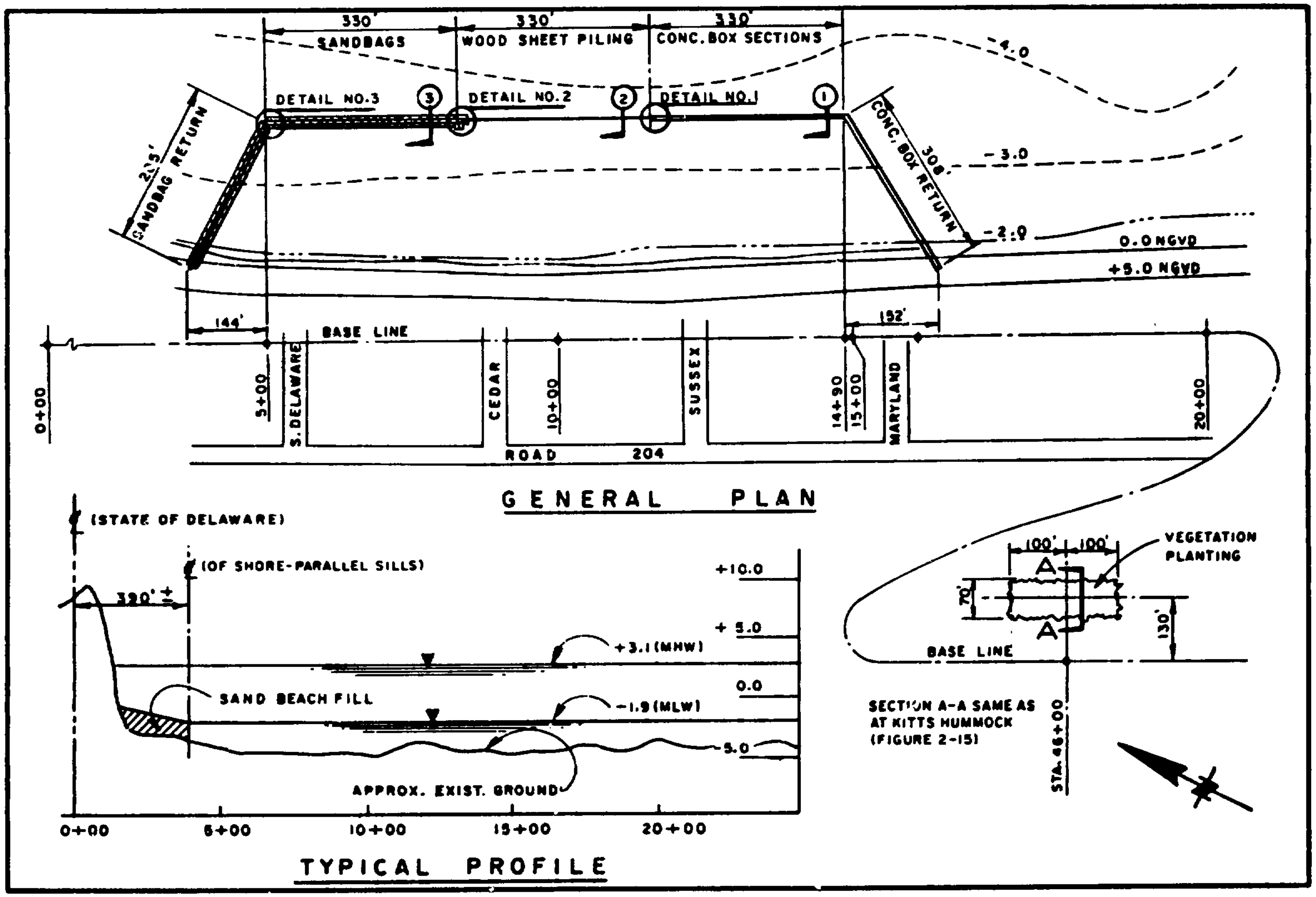


Figure 2–28. General plan and typical profile of Slaughter Beach, Delaware, demonstration site.

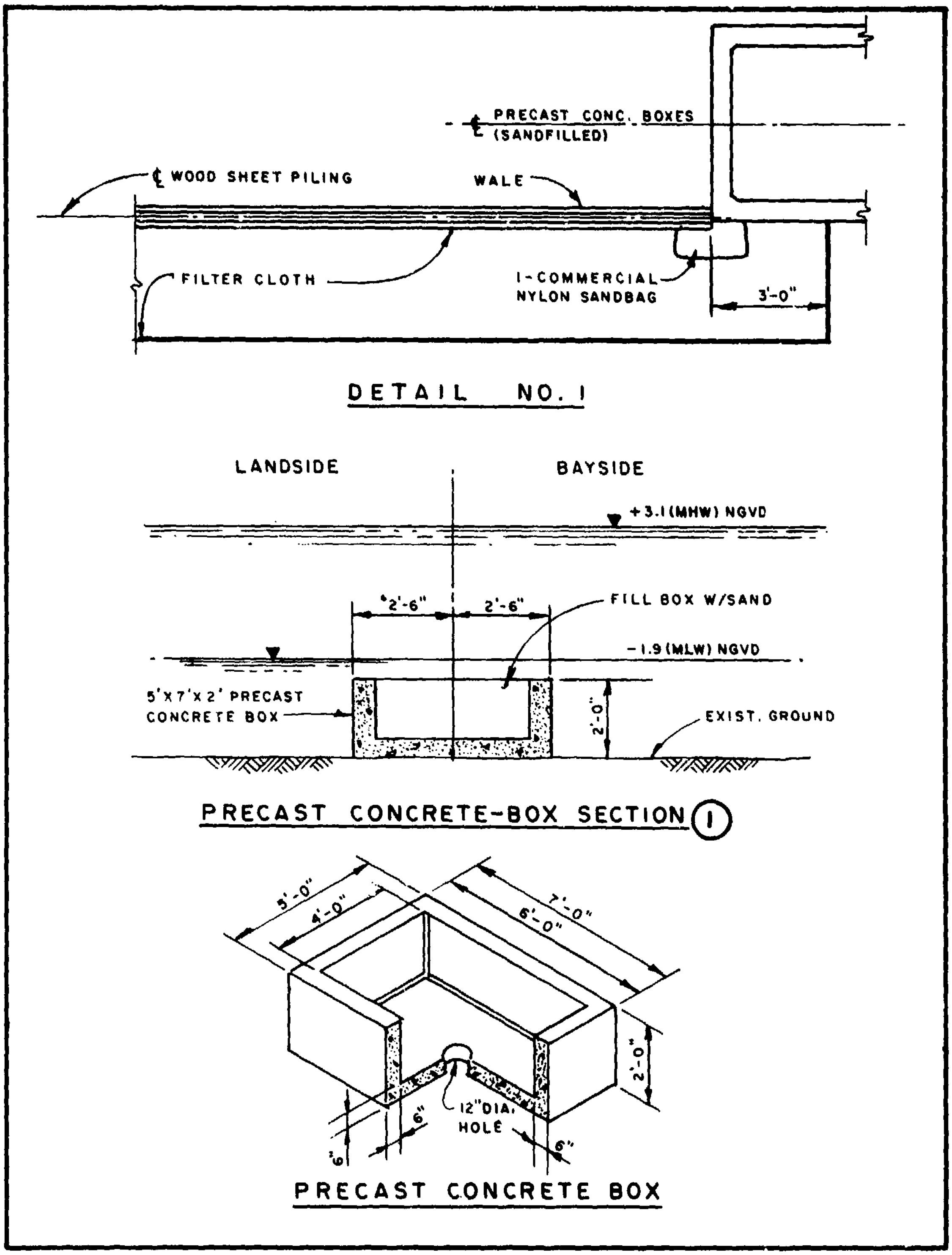


Figure 2-29. Detail No. 1 and concrete-box sill at Slaughter Beach site.

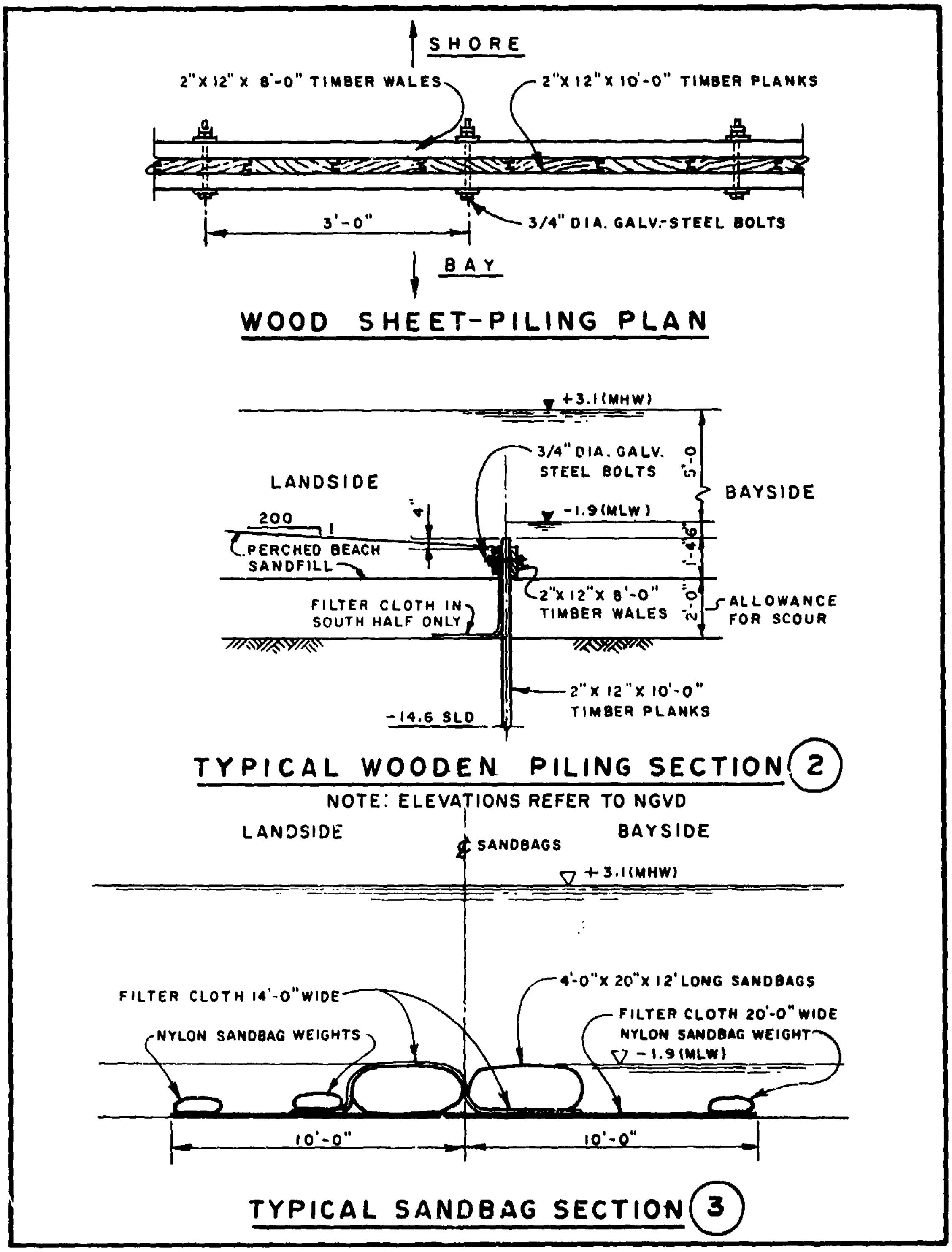


Figure 2-30. Wood piling and sandbag sill at Slaughter Beach site.

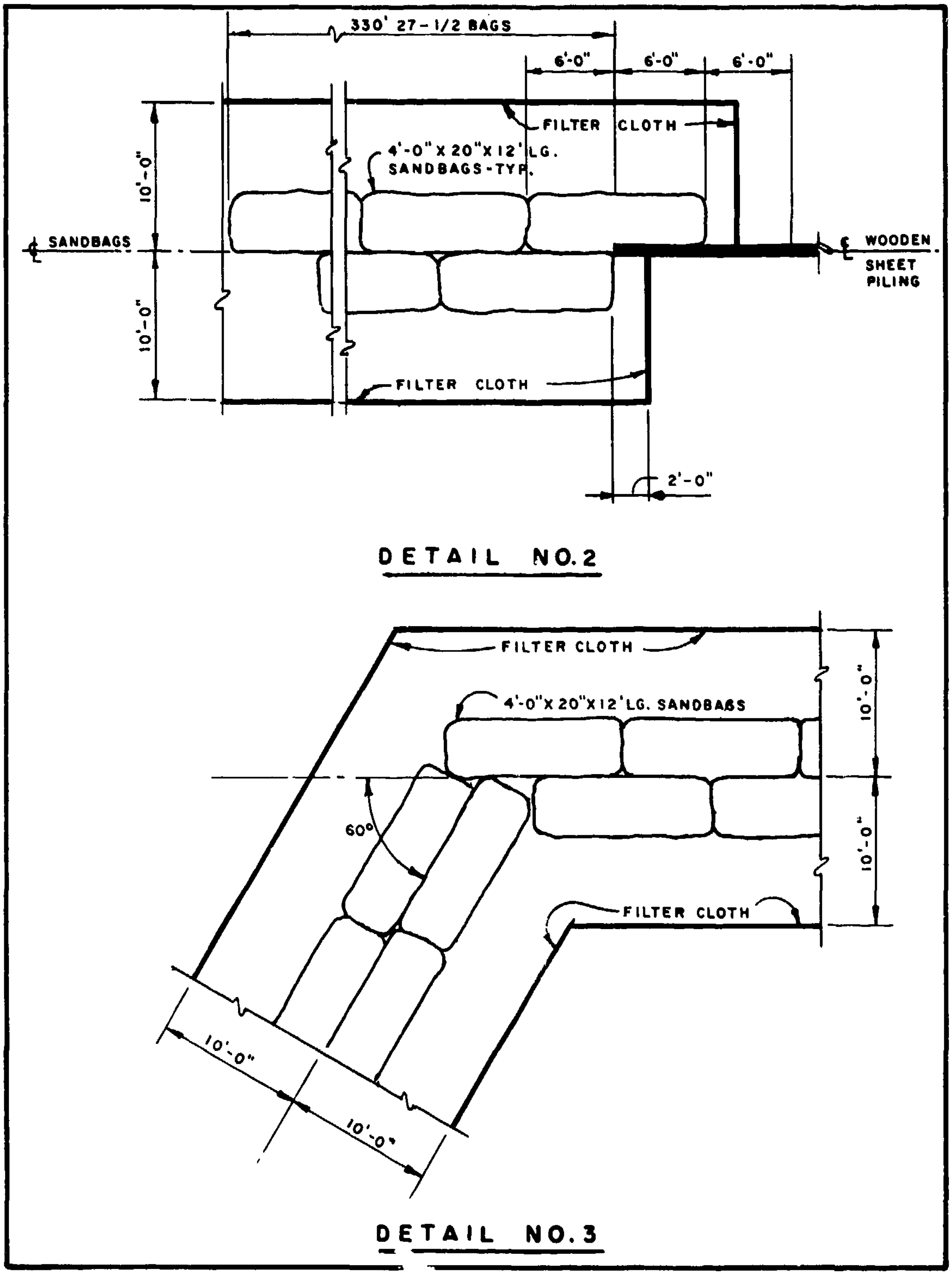


Figure 2-31. Details No. 2 and 3 at Slaughter Beach site.

The sill is 990 feet long, each device consisting of 330 feet. The shore returns comprise 308 feet of concrete boxes at the south end and 285 feet of sandbags at the north end. The top elevation of the sill, as indicated in Figures 2-29 and 2-30, is just below MLW. Because the structure was placed on 2 feet of loose, silty material, filter cloth was placed under the entire length of the sandbag segment and anchored on either side by small commercial sandbags spaced 20 feet apart. Filter cloth was also placed on the shoreward face of half of the sandbag sill segment as a control section to determine if there is a substantial loss of beach-fill sand through the bags in the uncontrolled segment. The structural work was done between April and June 1979 by a general contractor under contract with the U.S. Army Engineer District, Philadelphia.

Vegetation was planted along the beach in March 1979 (Fig. 2-28) to monitor its effectiveness as a complementary erosion control device. Plantings were designed so that half would be protected and the other half unprotected. However, due to an error by the contractor, all plantings were placed in the unprotected area.

Construction data and illustrations for each device installed at the Slaughter Beach project are given below.

(3) Statistics, Costs, and Construction.

(a) Precast Concrete-Box Sill and Return. The statistics and cost of the precast concrete-box sill and return are given in Table 2-9.

Table 2-9. Statistics and costs for the precast concrete-box sill and return.

Total length	637 ft
Sill length	329 ft
Return length	308 ft
Box dimensions	5 by 7 by 2 ft deep with 6-in wall
Sill	47 boxes
Return	44 boxes
Constructed	2 April to 26 April 1979
Contract cost	$54,950

Construction of the concrete-box sill at Slaughter Beach began at the south end and proceeded northward. During construction, the barge was held in place by four spuds. The precast concrete boxes were hoisted from the barge and placed in section by a crane mounted on the barge deck. After a number of boxes were in place they were filled with sand pumped from the beach. When the sill part was completed, the barge moved to the south return and worked toward shore. The work was completed 26 April 1979.

(b) Wood Sheet-Pile Sill. The statistics and cost of the wood sheet-pile sill are given in Table 2-10.

Table 2-10. Statistics and costs for the wood sheet-pile sill.

Total length	330 ft
Sheet piles	2 in by 12 in by 8 ft long (6-ft penetration and Wolman salt treatment)
Wales	2 in by 12 in by 8 ft (Wolman salt treatment)
Bolts	3/4-in galvanized steel
Filter cloth	approx. 1,000 ft^2
Constructed	27 April to 17 May 1979
Contract cost	$32,880

Construction of the wood sheet-pile segment of the sill began with unsuccessful attempts to drive the piles with a 65-pound jackhammer and a sheathing hammer (MKT #2 weighing 340 pounds). The piles would not penetrate more than 3 to 4 feet. The contractor then tried jetting in conjunction with the sheathing hammer and successfully drove eight piles to grade before stopping for the weekend. The following Monday morning the crew discovered that the sheathing had floated up during the weekend. An attempt to redrive the piles failed. Finally, a 2,300-pound drophammer successfully drove 16 sheet piles to grade, each pile driven to at least a 6-foot penetration with an average blow count of 25, using a 1-foot drop. Most of the driving energy was expended during the last 3 feet of penetration, the upper 3 feet of material being relatively loose. To speed operations in the remainder of the timber segment, two piles were driven simultaneously, with an average blow count of 50. At the end of each day, a temporary wale was nailed in place. Later, holes were drilled, the wales and sheet pilings were bolted together (about 36 inches on center), and the filter cloth was stapled to the inside wall in the southern half of the structure to prevent sand passing through the joints between adjacent sheet piles.

(c) Nylon Sandbag Sill and Return. The statistics and cost of the nylon sandbag sill and return are given in Table 2-11.

Table 2-11. Statistics and costs for the nylon sandbag sill and return.

Total length	618 ft
Sill length	330 ft, 54 nylon bags
Return length	288 ft, 48 nylon bags
Sandbags	4 ft by 12 ft by 1.7 ft (Advance Bag and Dura-Bag)
Holddown sandbags	100-lb burlap bags
Filter cloth	approx. 10,000 ft^2
Constructed	16 May to June 1979
Contract cost	$33,410

Construction of the nylon sandbag sill and return began by placing a sheet of filter cloth along the foundation site and anchoring it with small sandbags. Large empty nylon sandbags were then placed in the section and filled with beach sand using a 3-inch trash pump. A slurry mixture of sand and water was pumped from the beach, through a discharge hose, and directed into the nylon sandbags by two workers standing in the water. During most of the operation the sand-water slurry was lean and the water escaped through the bag fabric slowly, requiring an exceptionally long pumping time. An attempt to use a higher sand/water ratio only added to the problem by causing an occasional delay to unstop the discharge hose when it became plugged. The manufacturer has since improved the Dura-Bag filling system, and this should not be a problem in future installations. During construction, the following observations were made:

(1) There were problems with the alinement of the large nylon sandbags, including unacceptable gaps left between many bags.

(2) The contractor ran out of holddown sandbags and anchored the filter cloth with loose sand.

(3) Both Dura-Bags and Advance Bags were used. The Advance Bags were more porous and required less pumping time; the Dura-Bags had a cleaner neck closure.

(4) Velcro fasteners on the Advance Bags did not close well and had to be tied in knots to prevent the filler necks from unfastening.

(d) Beach Fill. The statistics and cost of the beach fill are given in Table 2-12.

Table 2-12. Statistics and cost for the beach fill.

Beach-fill material	34,000 yd^3
Bulldozer (contract)	42.5 hr
Bulldozer (State of Delaware)	47 hr
Contract cost	\$45,581

The beach fill was pumped to the site from an offshore sand deposit. The work was done by the State of Delaware with a hydraulic pipeline dredge. The work period was from 31 July to 19 November 1979.

(e) Vegetation. The statistics and costs for the vegetation plantings at Slaughter Beach are given in Table 2-13.

Table 2-13. Vegetation, statistics, and costs, Slaughter Beach, Delaware.

Materials	Cost
Smooth cordgrass	\$828
Saltmeadow cordgrass	759
American beachgrass (Cape variety)	209
	\$1796
Labor and equipment Cost given only as combined cost for three sites (see Table 2-2).	

Smooth cordgrass, saltmeadow cordgrass, and American beachgrass (Cape variety) were planted at Slaughter Beach according to the same design as used at Pickering Beach and Kitts Hummock. The planted area was unprotected and all plantings took place by mid-June 1979. Specifics of planting for each species are:

Smooth cordgrass:

Planted area:	200 by 30 feet (below MHW)
No. of plants:	2,660
Method:	peat pots
Spacing:	1.5 feet
Fertilizer:	1 ounce, 8- to 9-month Osmocote at time of planting
Planting date:	13-14 June 1979

Saltmeadow cordgrass:

Planted area:	200 by 30 feet
No. of plants:	2,660
Method:	peat pots
Spacing:	1.5 feet
Fertilizer:	normal mix (10-10-10); 1st fertilization 2-3 weeks after planting; 2d fertilization 6 weeks after initial fertilization at 500 pounds per acre at each application
Planting date:	3-4 May 1979

American beachgrass (Cape variety):

Planted area:	200 by 10 feet
No. of plants:	931
Method:	sprigs
Spacing:	1.5 feet
Fertilizer:	as described for saltmeadow cordgrass
Planting date:	26-28 March 1979

Planting took place at the same time and using the same methods at Slaughter and Pickering Beaches. Initial plantings of both species were washed out at Slaughter Beach almost immediately after planting was completed. The site was subsequently replanted, moving the saltmeadow cordgrass up to MHW and planting smooth cordgrass to ajoin the lower margin of the saltmeadow cordgrass. This resulted in smooth cordgrass above the MLW line.

(4) Performance.

(a) Precast Concrete-Box Sill and Return. The alinement of this structure was good and only minor amounts of settlement and structural damage occurred. The most significant settlement took place along the sill in February 1980 (Fig. 2-32). Structural damage was limited to the cracking of a few boxes along the return (Fig.2-33).

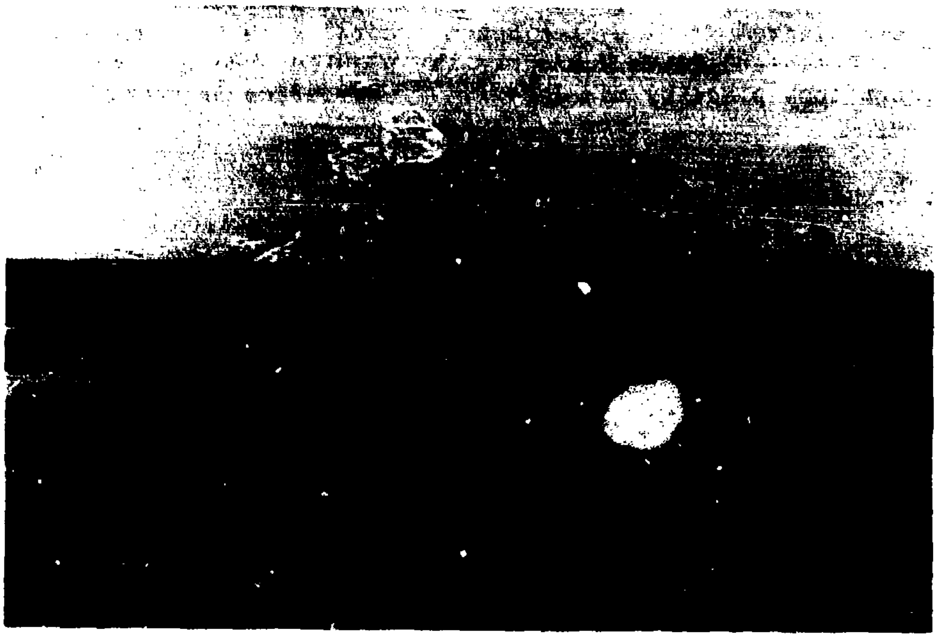


Figure 2-32. Settlement of a few concrete boxes along the sill, Slaughter Beach, Delaware, 29 February 1980.

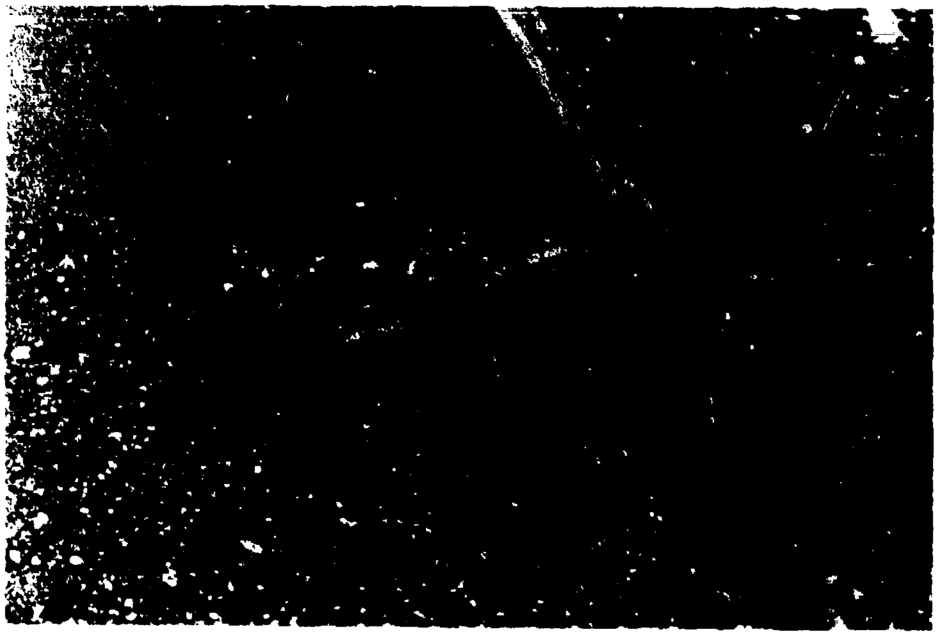


Figure 2-33. Cracked concrete box along the south return, Slaughter Beach, Delaware, 28 January 1980.

(b) Wood Sheet-Pile Sill. This section of the sill remained as initially constructed. There was no settlement or floating of the piles and the alinement did not change (Fig.2-34).

(c) Nylon Sandbag Sill and Return. After construction, the initial site visit revealed many gaps along this part of the sill and return. The contractor plugged most of the gaps with half-bags (Fig. 2-35), but a few small gaps were left open. During the monitoring period some of the sandbags shifted position, causing minor changes in the alinement of the sill and return. The individual sandbags weathered well without any major damage.

(d) Functional Effectiveness. The perched beach at Slaughter Beach was filled with 14,000 cubic yards of material in October 1979 and 20,000 cubic yards in November 1979. Each part of the sill and returns functioned well in retaining the beach fill. There were no major leaks through or transport of sand over the structures. Figure 3-76, which was put in Section III for comparison of profiles through similar devices, shows a series of profiles through the timber sill at Slaughter Beach. The profiles for December 1979 and March 1980 show the effect of the sill in retaining the perched beach. Note that the toe of the fill slopes to the original bottom at the location of the sill, but that little buildup has occurred bayward of the sill. It is probable that overtopping waves create so much turbulence behind the structure that a scour trench develops in the lee of the sill, but the attenuated waves cause the fill to beach out about 100 feet landward of the structure.

In January 1980, the dune behind the perched beach and to the south began to erode (Fig. 2-36). The problem area started at about the midpoint of the sill and extended south approximately 1,000 feet. Movement of the eroded material was from north to south, as evidenced by Figure 2-37. By March 1980, the erosion of the dune had stabilized, and no other problems developed. The monitoring period was too short to determine the effect of the perched beach on the adjacent shore and the offshore bottom. A few widely spaced profiles were surveyed on each side of the installation after it was completed, but not enough to delineate contour changes. For this reason, only the initial contours are shown in Figure 2-28, and no other profiles are presented.

(e) Vegetation. Smooth cordgrass did not survive. Wave action uprooted many of the plants, as did the burrowing of numerous horseshoe crabs.

Saltmeadow cordgrass also showed very high mortality immediately after planting with only 15 percent of the plants remaining by the July midseason inventory. At the end of the growing season in October, about 9 percent of the original planting remained. Those plants that did survive until October also survived the winter and were actively growing in May 1980.

Most of the American beachgrass plantings washed out immediately after planting, with 39 percent remaining at the midseason inventory in July 1979. By the end of the growing season there was only 20-percent survival. The beachgrass plants which did survive through the summer showed signs of

Figure 2-34. Wood sheet-pile sill, Slaughter Beach, Delaware, 29 February 1980.

Figure 2-35. Half-bags used to plug gaps, Slaughter Beach, Delaware, 7 August 1979.

Figure 2-36. Erosion of the dune behind and to the south of the perched beach, Slaughter Beach, Delaware, 27 January 1980.

Figure 2-37. Sand movement along the south return, Slaughter Beach, Delaware, 26 March 1980.

having become well established and were growing into new areas of the upper beach in May 1980. Figures 2-38 and 2-39 show the vegetation plantings in September 1979.

(5) Analysis.

(a) Precast Concrete-Box Sill and Return. The structure performed well in retaining the beach fill. The only problems were localized cracking and settlement of a few boxes, neither of which had a significant impact upon overall performance.

(b) Wood Sheet-Pile Sill. The performance of this device demonstrated that filter cloth is not needed when using tongue and grove sheeting for a retaining sill. The snug fit between adjacent sheets eliminates the need for filter cloth. This structure performed well in retaining the beach fill.

(c) Nylon Sandbag Sill and Return. After the contractor plugged most of the gaps along this part of the sill and return it performed well in retaining the beach fill. As this device illustrated, care should be taken when filling the sandbags to ensure contact between adjacent bags.

(d) Vegetation. The analysis of vegetation planted at Pickering Beach applies to the plantings at Slaughter Beach.

Figure 2-38. Vegetation plantings at Slaughter Beach, Delaware, looking north, 12 September 1979. Note incipient erosion along toe of dune.

Figure 2-39. Vegetation plantings at Slaughter Beach, Delaware, looking north, 12 September 1979.

f. Broadkill Beach, Delaware.

(1) Site Description.

(a) Geographical Setting. Broadkill Beach is 9 miles southeast of Slaughter Beach and about 7 miles from the mouth of the bay. About 150 summer cottages extend along approximately 6,000 feet of bay frontage. Although designated as a demonstration site in the authorizing act, Broadkill Beach is in effect only a monitoring site, as it is protected by a beach fill placed by the Corps of Engineers as a Federal Small Beach Erosion Control Project. No additional shore protection devices were added under the demonstration program.

(b) Geomorphology and Soils. The soils at Broadkill Beach consist primarily of fine to medium sands. The shoreline extends along the bay in a general northwest-southeast direction. The foreshore and offshore zones have slopes of approximately 1 on 20 and 1 on 300, respectively. The beach is 60 feet wide, and extends inland from high water to a sand dune. The dune is 30 feet wide at its base and has a top elevation of 10 to 12 feet above MLW.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 1 to 2 feet, with a maximum of 4.6 feet, and net longshore transport potential northward was 22,300 cubic yards for the 4 months analyzed.

(2) Monitoring Project. A project involving the improvement of 4,500 feet of beach extends from a point 2,700 feet north of an access road (State Route 16) to a point 1,800 feet south of that road by placement of suitable sand to provide a berm 50 feet wide at an elevation 10 feet above MLW with a foreshore slope of 1 on 10, and periodic sand replenishment for 10 years. Details of the project are shown in Figure 2-40. The initial beach fill, placed in the summer of 1976, was 40,300 cubic yards of material. Plans were being prepared to place an estimated 60,000 cubic yards at this site in the fall of 1980. A sand fence authorized for the project was not installed because it would infringe on the active beach and interfere with the recreational use of the beach. The State of Delaware has restored the beach twice -- 18,100 cubic yards the fall of 1973 and 29,500 cubic yards the spring of 1975. For monitoring, profiles of the beach and shore bottom were taken periodically at the range lines indicated in Figure 2-40.

Seven groins were previously placed at the site. Groins at stations 20+80N and 16+40N are concrete rubble groins placed in 1964 by the Delaware Department of Highways. Groins at stations 11+70N and 6+70N are timber and stone groins installed in 1954. Groins at stations 2+00N, 2+65S, and 7+40S are timber groins installed in 1950. The aerial photo in Figure 2-41 shows that the timber groins at stations 2+65S and 7+40S have been covered with sand, apparently during the 1976 beach nourishment program.

(3) Performance. The groin field has retained the beach fill placed in 1976 and no renourishment projects have since been required. The contour changes were too small to show on the General Plan, but Figure 2-41 illustrates the functional performance of the groin-retained beach fill.

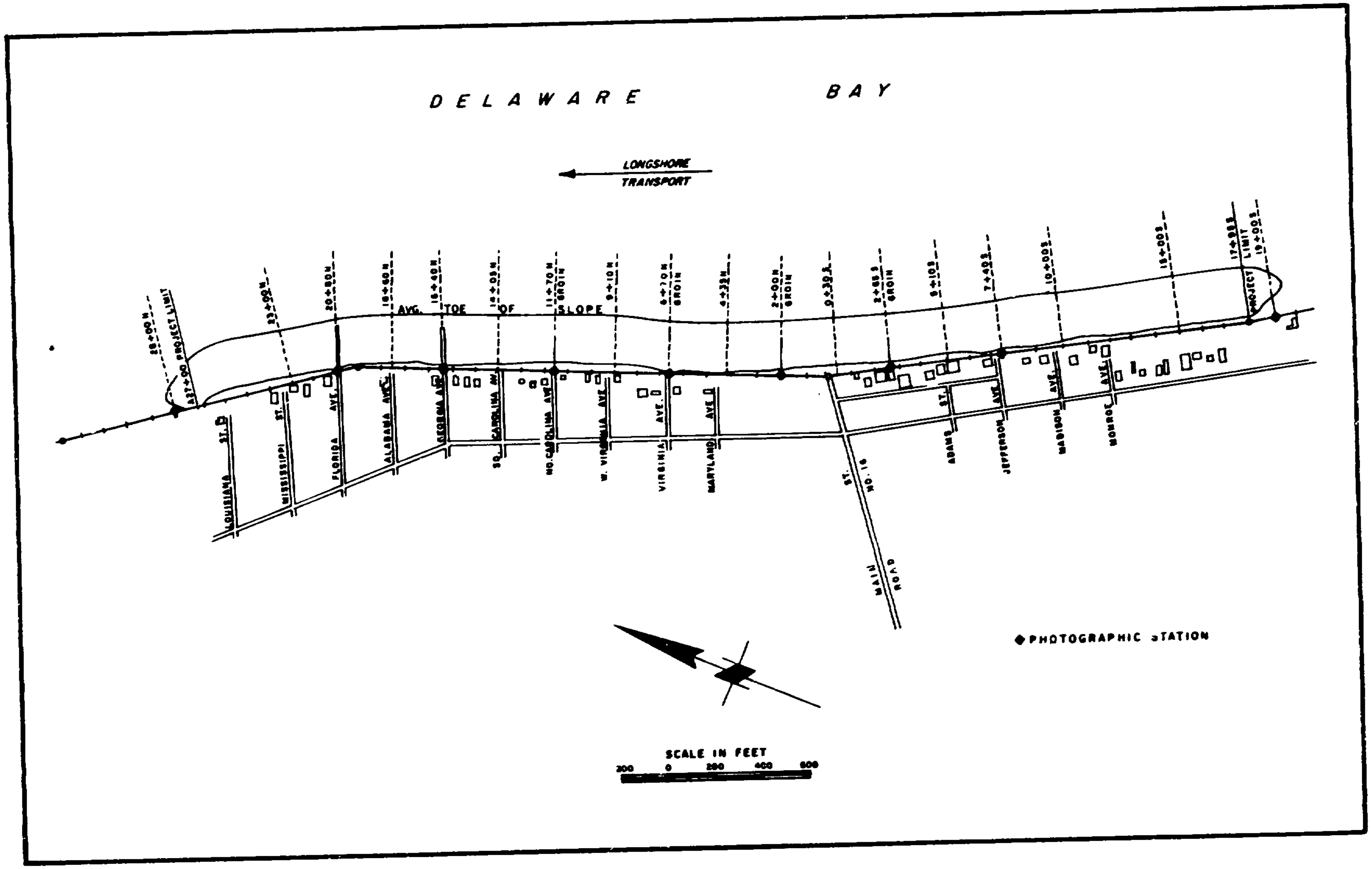


Figure 2-40. General plan of Broadkill Beach, Delaware, demonstration site.

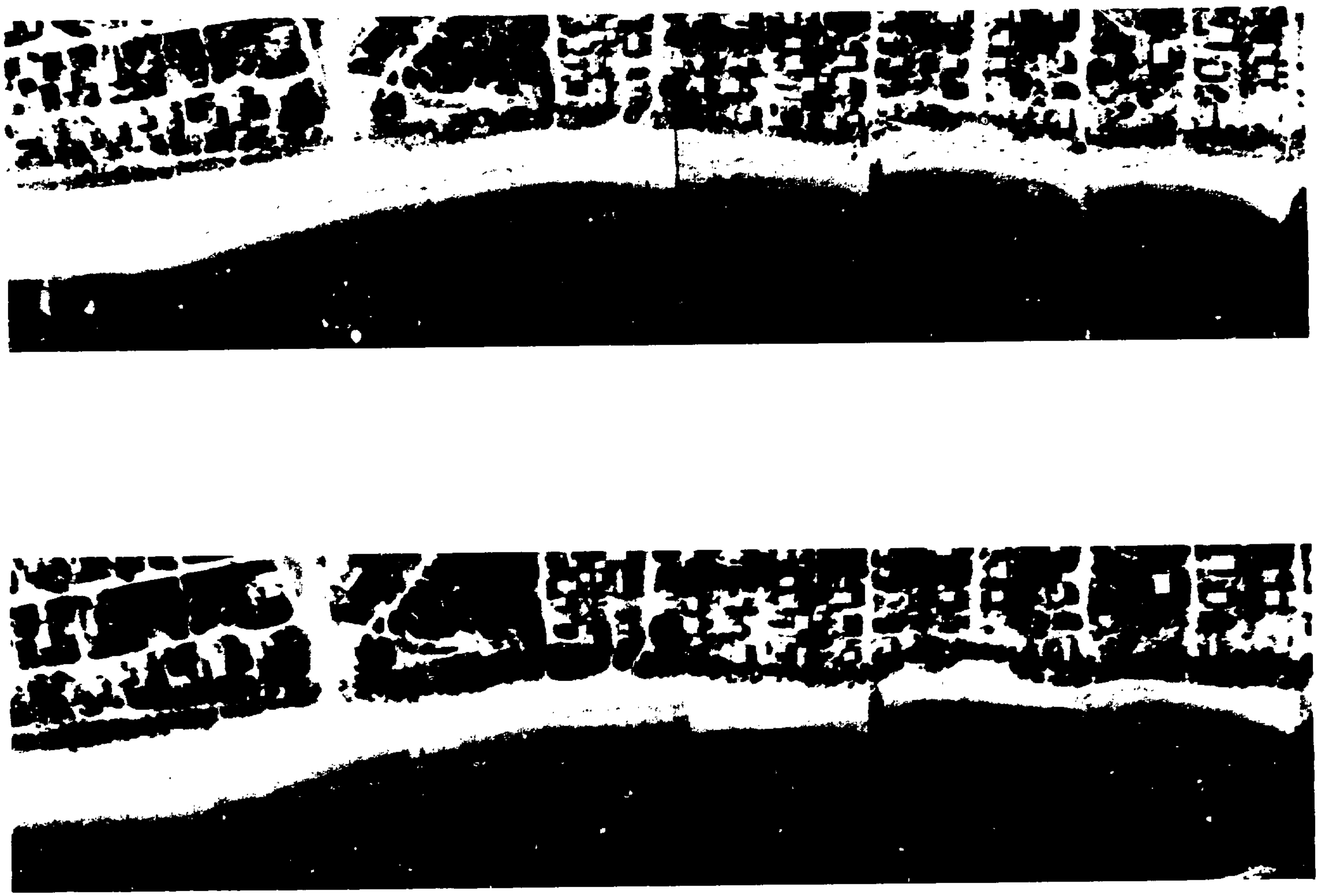


Figure 2-41. Groin-retained beach fill at Broadkill Beach, Delaware (top photo, 9 March 1979; bottom photo, 18 October 1979).

Beach fill has accreted on the south side of each groin demonstrating the dominant southeast to northwest littoral transport. The aerial photo also reveals an important difference between the performances of timber groins and concrete rubble groins. The concrete rubble groin allows sand to pass through it and thereby nourish the downdrift beach; however, sand must pass around the timber groin before moving on to the downdrift beach. The beach alinement next to the timber and rubble groins at stations 16+40 N and 11+70 N, respectively, demonstrates this difference. The profile at station 23+00 N, just north of the groin field (Fig. 2-42), shows about 20 feet of erosion between April 1979 and March 1980. This further demonstrates the southeast to northwest littoral transport and that the groin field is effectively trapping littoral drift.

(4) Analysis. The combined action of the groins and beach fill has protected the shoreline at Broadkill Beach since 1976. As indicated by the aerial photos, there is a strong littoral transport along this reach, and it is doubtful that a beach fill alone could control shoreline erosion without a costly renourishment schedule. The groin field, therefore, has contributed significantly to shoreline protection by stabilizing the beach. Since 1976 only a minor amount of beach fill has eroded and no renourishment programs have been required.

g. Lewes, Delaware.

(1) Site Description.

(a) Geographical Setting. The incorporated municipality of Lewes extends from Roosevelt Inlet, 3 miles southeast of Broadkill Beach, to about 3 miles eastward to Cape Henlopen State Park at the mouth of Delaware Bay.

(b) Geomorphology and Soils. The beach and dune at Lewes consist of fine to medium poorly graded sand. The beach extends along Delaware Bay in a generally east-west direction. The general site conditions have been influenced by removal of parts of an approximately 100-foot-wide dune, and by construction of buildings. The beach foreshore area extends 100 feet inland at a slope of 1 on 11 and meets the base of the dune at about mean high water (MHW) level.

(c) Waves and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 3.0 feet. The energy-flux analysis for the 3-month observation period indicates a 5,450-cubic yard net potential for eastward longshore transport at this site, which extrapolates to about 18,000 cubic yards annually for the ice-free season.

(2) Monitoring Project. A project provides for widening 8,000 feet of beach by placement of suitable sand to provide a beach with a berm 100 feet wide at an elevation 10 feet above MLW and periodic nourishment for 10 years. The initial beach fill, placed in the winter and spring of 1975, comprised 86,710 cubic yards of material. Construction of a sand fence and planting of dune grass authorized for the project were not installed. Details and location of the project are shown in Figure 2-43. The State of

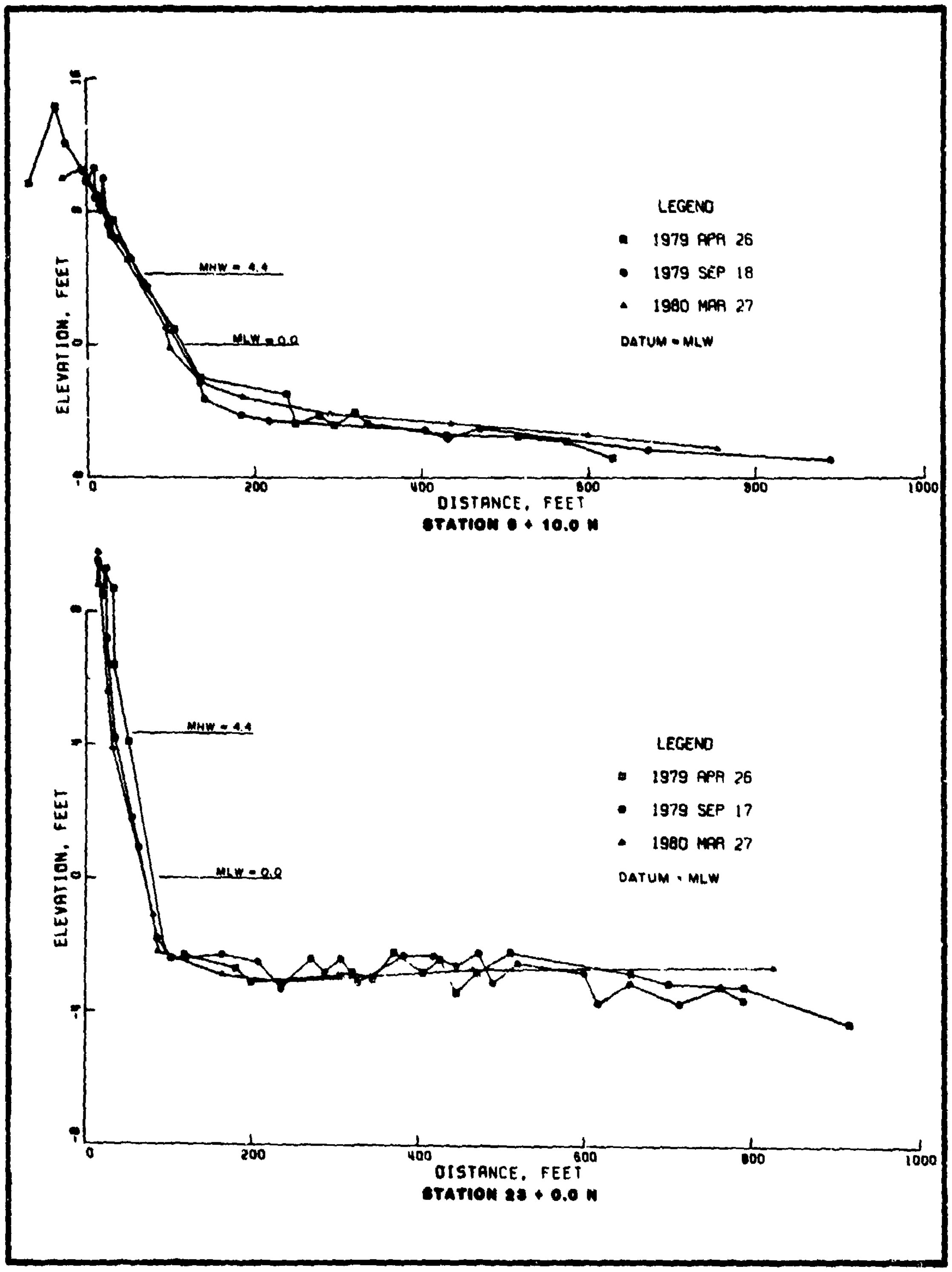


Figure 2-42. Typical profiles, Broadkill Beach.

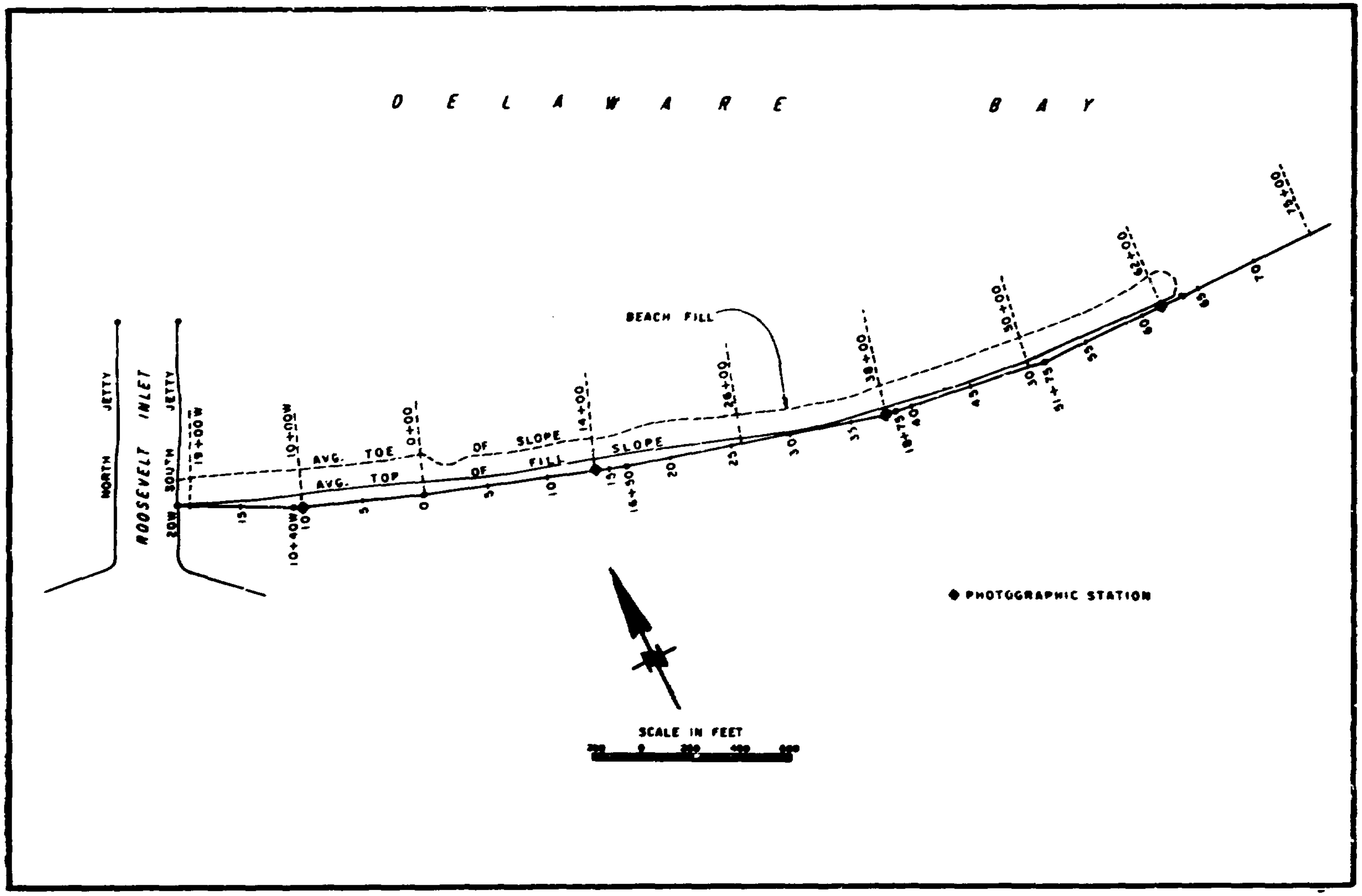


Figure 2-43. General plan of Lewes, Delaware, demonstration site .

Delaware has nourished the beach on three occasions -- 69,800 cubic yards the winter of 1972-73, 11,400 cubic yards the fall of 1977, and 31,000 cubic yards the fall of 1978. The average is about 14,000 cubic yards a year, which agrees fairly well with the energy-flux calculations for the LEO data. An estimated 87,000 cubic yards of beach fill was planned for the fall of 1980. For monitoring, profiles of the beach and offshore bottom were taken periodically at the range lines indicated in Figure 2-43.

(3) Performance. Shoreline erosion along the beach at Lewes has been controlled only by the placement of a protective beach fill. Aerial photos reveal no dramatic changes during the relatively short timespan from March to October 1979 (Fig. 2-44), and the scale of the General Plan is too small to plot contour changes. However, profiles taken from surveys made between April 1979 and April 1980 (Fig. 2-45) show a retreat of the beach berm of about 50 feet near the west and tapering to only a few feet near the east end of Lewes beach. This is indicative of a strong eastward transport of littoral drift along this reach.

(4) Analysis. The initial beach fill to be monitored under this program comprised 86,710 cubic yards placed in 1975. Since then the beach has been nourished twice with 11,400 and 31,000 cubic yards in 1977 and 1978, respectively. Although the shoreline erosion has been controlled, the amount of renourishment has been high, with another 87,000 cubic yards planned for the fall of 1980. These large renourishment quantities seem to indicate that the cost of protection along this reach might be reduced if the beach fill were stabilized. As demonstrated at Broadkill Beach, a groin field would probably stabilize the beach and greatly reduce the amount of required renourishment, thereby reducing the annual maintenance costs.

2. Atlantic Coast Sites.

a. Common Characteristics.

(1) Geographical Setting. Two demonstration sites and seven monitoring sites are located on, or connected to, the Atlantic coast. The site locations are shown in Figure 2-46. Most of this coastal area is composed of elongated barrier islands and sandspits separated from the mainland by tidal estuaries. Most beaches on the open coast are exposed to high-energy ocean storm waves and do not make suitable sites for the demonstration of low-cost shore protection devices. The two demonstration sites are in protected areas fronted by short fetches of open water. Most of the monitoring sites are similarly located, but two are on relatively open coast.

(2) Climate, Waves, and Tides. The climate of the Atlantic coast sites varies from moderate in the northern area to semitropical on the southern end. Considerable rainfall occurs throughout the region, but ice is not a problem. All sites are subject to occasional hurricanes. Wave heights at the interior sites are limited by the open-water fetches over which the waves are generated. Although the exposed sites are frequented by ocean storm waves, the wave heights at the shoreline seldom exceed 7 feet because the higher waves break in the shallow offshore waters. Tides vary throughout the region, ranging from about 2 to 5 feet. The two daily tides are about equal, and datum for all sites is MLW. Because of the wide Continental Shelf

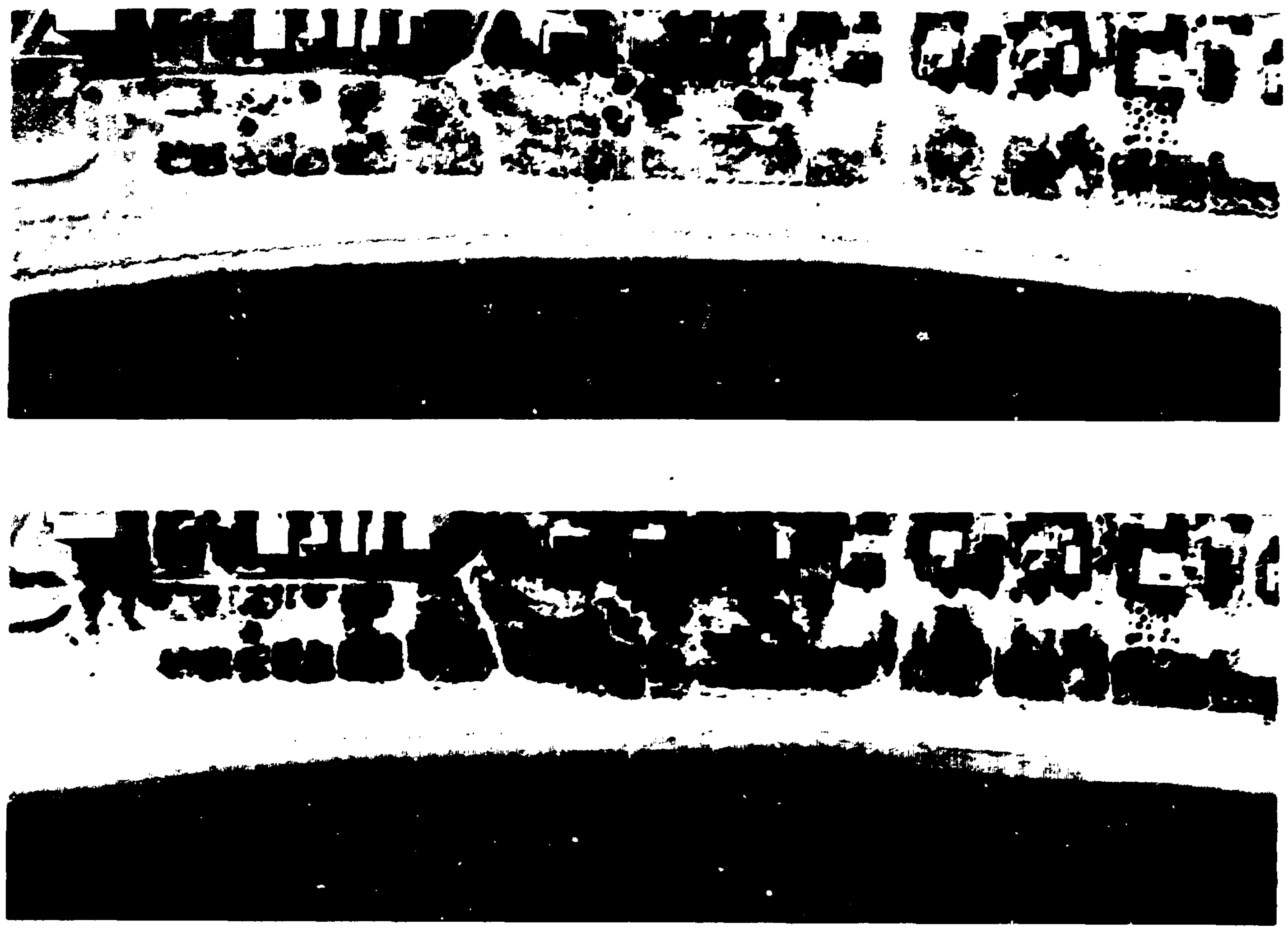


Figure 2-44. Aerial photos of the beach nourishment at Lewes, Delaware (top photo, 9 March 1979; bottom photo, 18 October 1979).

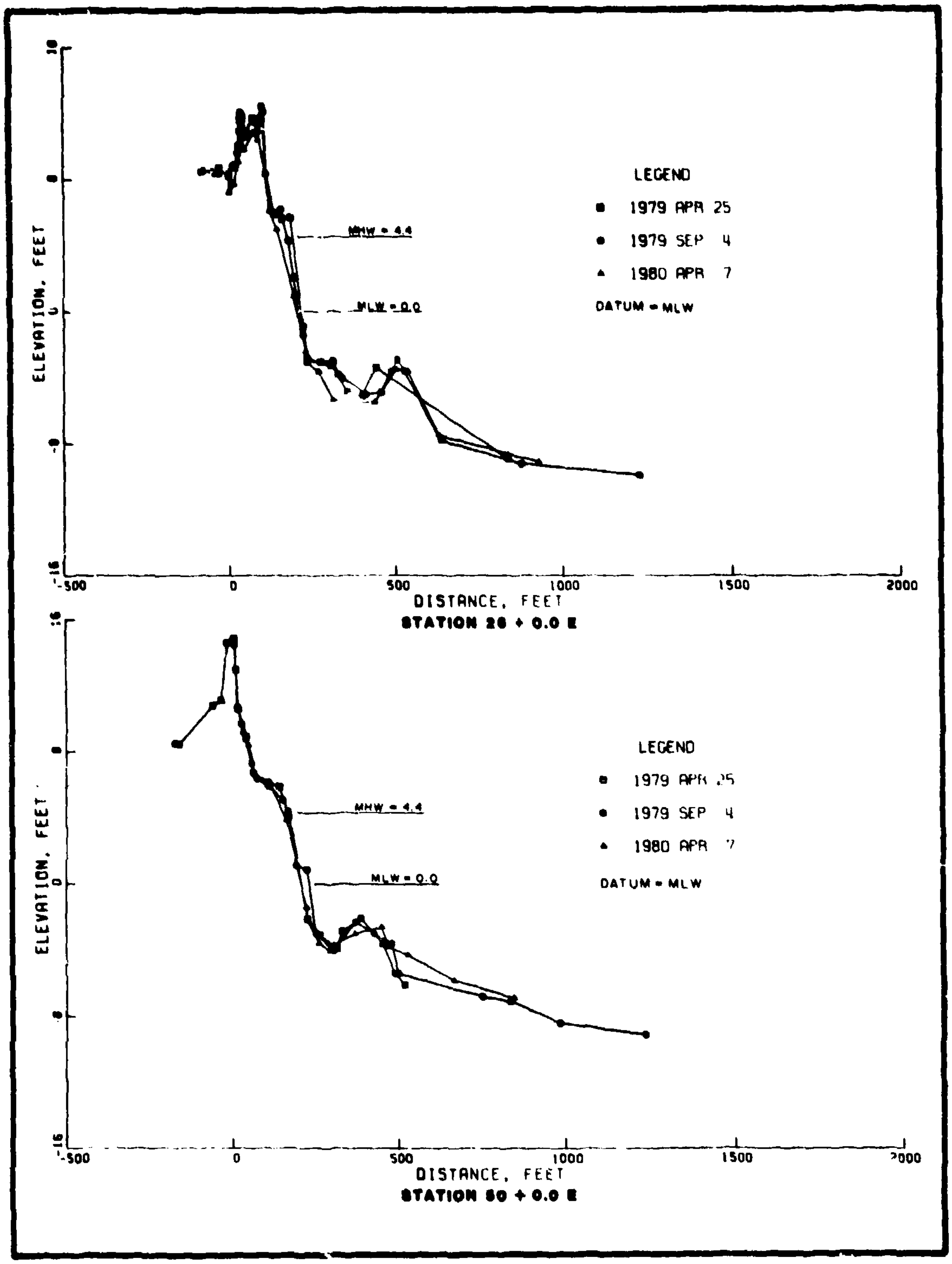


Figure 2-45. Typical profiles, Lewes.

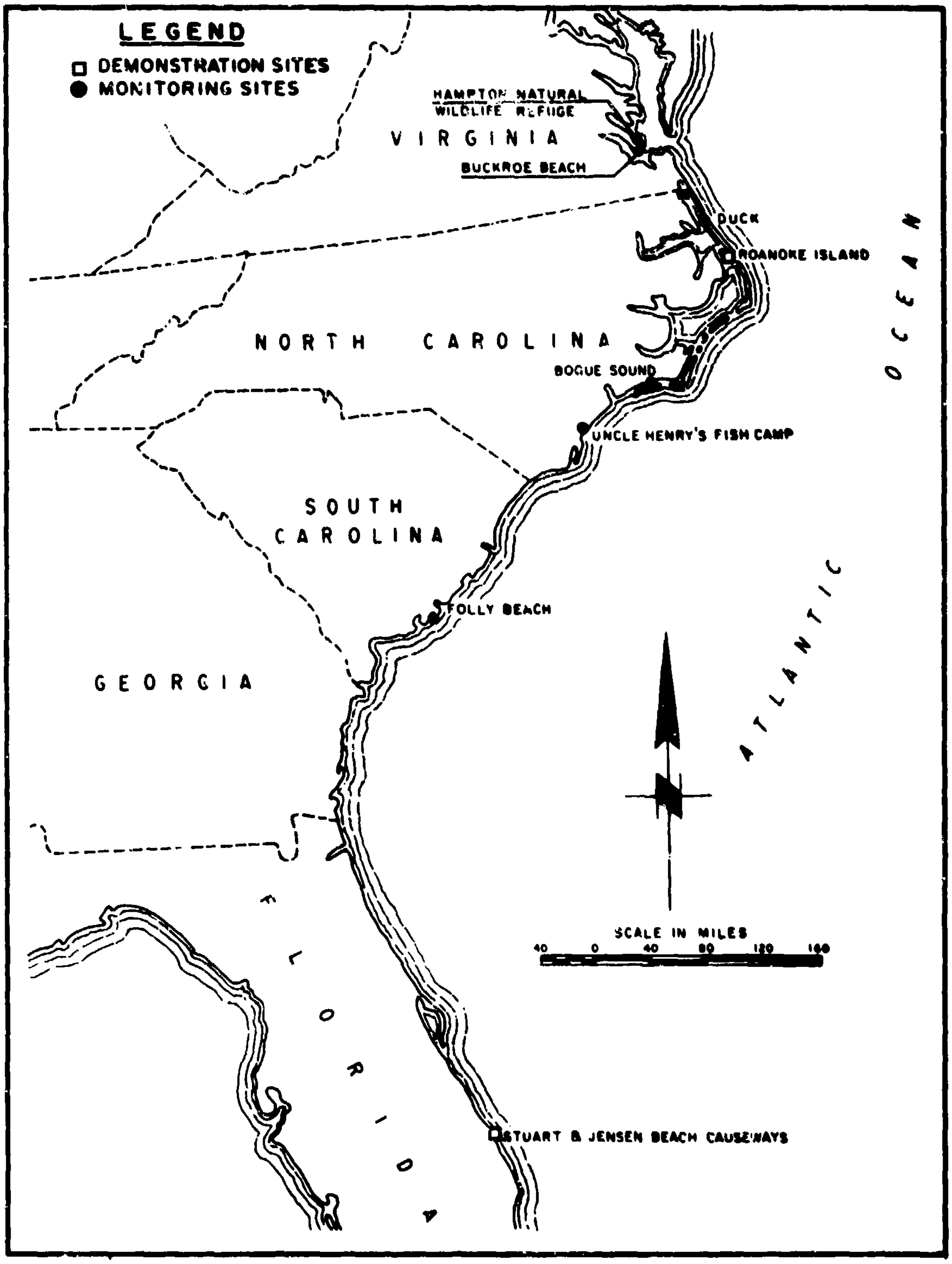


Figure 2-46. Location map of Atlantic Coast sites.

and its shallow waters, wind setup during storms often raises the water surface at the sites well above the level of the astronomical tides.

(3) Geomorphology, Soils, and Vegetation. The Atlantic coast sites lie on the Atlantic Coastal Plain physiographic subdivision. The Coastal Plain is underlain by poorly consolidated sediments, Cretaceous to recent in age, that dip gently seaward. The coastal region is typically low and featureless with occasional hills, ridges, and dunes reaching elevations of 200 to 300 feet. Soils along the coast are predominantly sandy. Relatively resistant erosional remnants form low elongated asymmetrical ridges, or cuestas, which parallel the coastline. The shorelines of bays and estuaries consist mostly of unconsolidated clays and loose sandy silts which are easily eroded. Native grasses grow abundantly in the flat lowlands, and salt-tolerant varieties grow in many of the tidal areas. A wide variety of trees grow in the mainland area where root growth is not inhibited by tidewater penetration. Guidelines for vegetative treatment in the Middle Atlantic States, devised by the Soil Conservation Services, Broomall, Pennsylvania, are given in Table 2-14.

b. Roanoke Island, North Carolina.

(1) Site Description.

(a) Geographical Setting. Roanoke Island lies in the northeast region of North Carolina at the junction of Albemarle and Pamlico Sounds. The demonstration site is located along the northwest corner of Roanoke Island on a stretch of shoreline approximately 1 mile long. The shoreline is part of a national historic site, Fort Raleigh, which occupies approximately 144 acres of upland along the island (Fig. 2-47).

(b) Wind, Waves, and Tides. The site faces directly on Albermarle Sound and is exposed to meterological tides and waves generated over the sound by perennial winds and infrequently large storm fields acting over the northeast-to-northwest atmospheric sector. Northwesterly winds occur with the highest frequency and have the highest mean windspeed, 19.3 knots. The storm winds generate considerable wind setups at the site, a setup of 6 feet occurring on an average of once in 10 years. The effective fetch distance for northwest exposure at Fort Raleigh is approximately 18 miles over an average depth of 10 feet. These fetch characteristics, coupled with winds from the northwest, produced the wave characteristics given in Table 2-15. In Ablemarle and Pamlico Sounds, except near the inlets, the periodic tide has a mean range less than 0.5 foot. The nearshore area fronting the north end of Roanoke Island is a broad, shallow plateau with an average depth of only 3 feet for a distance of about 2,000 feet offshore. Wave transformation over this shallow area was accounted for in the wave computations. No LEO data were obtained for this site; however the wave climate is classified as intermediate. Longshore transport is predominantly eastward.

(c) Geomorphology, Soils, and Vegetation. Roanoke Island lies at the southern end of the embayed section of the Coastal Plain, which is characterized by barrier beaches, drowned river valleys, and swampy alluvial flats. The island itself appears to be a part of the series of

Table 2-14. Vegatative treatment potential for eroding tidal banks in the Middle Atlantic States--directions for use (from Sharp, Belcher, and Oyler (1980).

1. Evaluate each of the first four shoreline variables and match the site characteristics of the variable to the appropriate descriptive category.
2. Place the Vegetative Treatment Potential (VTP) assigned for each of the 4 variables in the right hand column.
3. Obtain the Cumulative Vegetative Treatment Potential for variables 1, 2, 3, & 4, by adding the VTP for each.
4. If it is 23 or more, the potential for the site to be stabilized with vegetation is very good and the rest of the table need not be used. If it is below 23, go to step 5.
5. Determine the VTP for shoreline variables 5 thorugh 9 and obtain the cumulative VTP for variables 1-9.
6. Compare the cumulative VTP score with the Vegetative Treatment Potential scale at the end of this table.

SHORELINE VARIABLES	DESCRIPTIVE CATEGORIES The Vegetative Treatment Potential (VTP) is located in upper left hand corner of each category box					VTP for each variable
1. Fetch: Average distance in miles of open water measured perpendicular to the shore and 45° either side of perpendicular to shore	8 Less than 0.5 miles	7 0.5 thru 1.4 miles	4 1.5 thru 3.4 miles	2 3.5 thru 4.9 miles	0 over 5 miles Do Not Plant	
2. General shape of shoreline for distance of 200 yards on each side of planting site.	8 Coves	3 Irregular shoreline	0 Headland or straight shoreline			
3. Shoreline Orientation General geographic direction the shoreline faces	5 Any orientation less than one-half mile fetch.	3 West to North	2 South to West	1 South to East	0 North to East	
4. Boat Traffic: Proximity of site to recreational & commercial boat traffic	5 None	3 1-10 per week within 1/2 mi. of shore	2 More than 10 per week within 1/2 mile of shore	1 1-10 per week within 100 yards of shore	0 More than 10 per week within 100 yds. of shore	
Sum of VTPs for Variables 1, 2, 3, & 4 If this score is 23 or above the potential for the site is very good and the rest of the table need not be used. - - - If it is below 23, go to step 5 below						———
5. Width of Beach Above Mean High Tide in Feet	3 Greater than 10'	2 10' thru 7'	1 6' thru 3'	0 Less than 3'		
6. Potential Width[1] of Planting Area in Feet	3 More than 20'	2 20' thru 15'	1 14' thru 10'	0 Less than 10' Do Not Plant		
7. On Shore Gradient % slope from MLH to toe of bank	6 Below 8%	3 8 thru 14%	1 15 thru 20%	0 over 20%		
8. Beach Vegetation	3 Vegetation below toe of slope	0 No vegetation below toe of slope				
9. Depth of Sand[2] at Mean High Tide in inches	3 More than 10"	2 10" thru 3"	0 Less than 3"			

Sum of VTPs for Variables 1 through 9 ———

VEGETATIVE TREATMENT POTENTIAL SCALE

VTP	Potential of Site to be Stabilized with Vegetation
40 to 33	Good
32 to 24	Fair
23 to 16	Poor
below 16	Do Not Plant

Footnotes

1. If tidal fluctuation is 2.5 feet or less, measure from MLW to toe of bank. If tidal fluctuation is over 2.5 feet, measure from MW to toe of bank.
2. Refer to depth of sand deposited by littoral drift over the substrate.

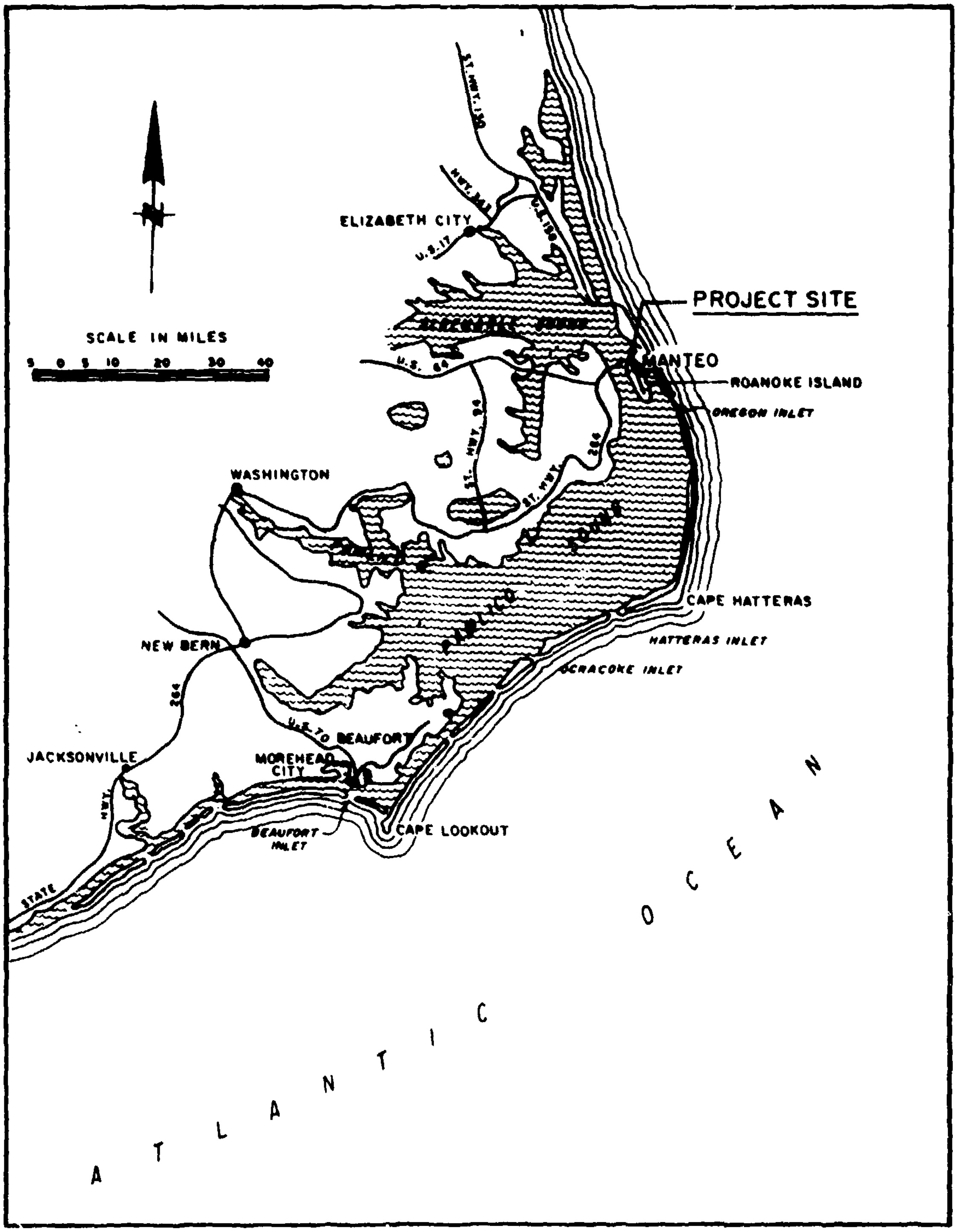


Figure 2-47. Location map of Roanoke Island, North Carolina, demonstration site.

Table 2-15. Computed wave characteristics, Roanoke Island, North Carolina.

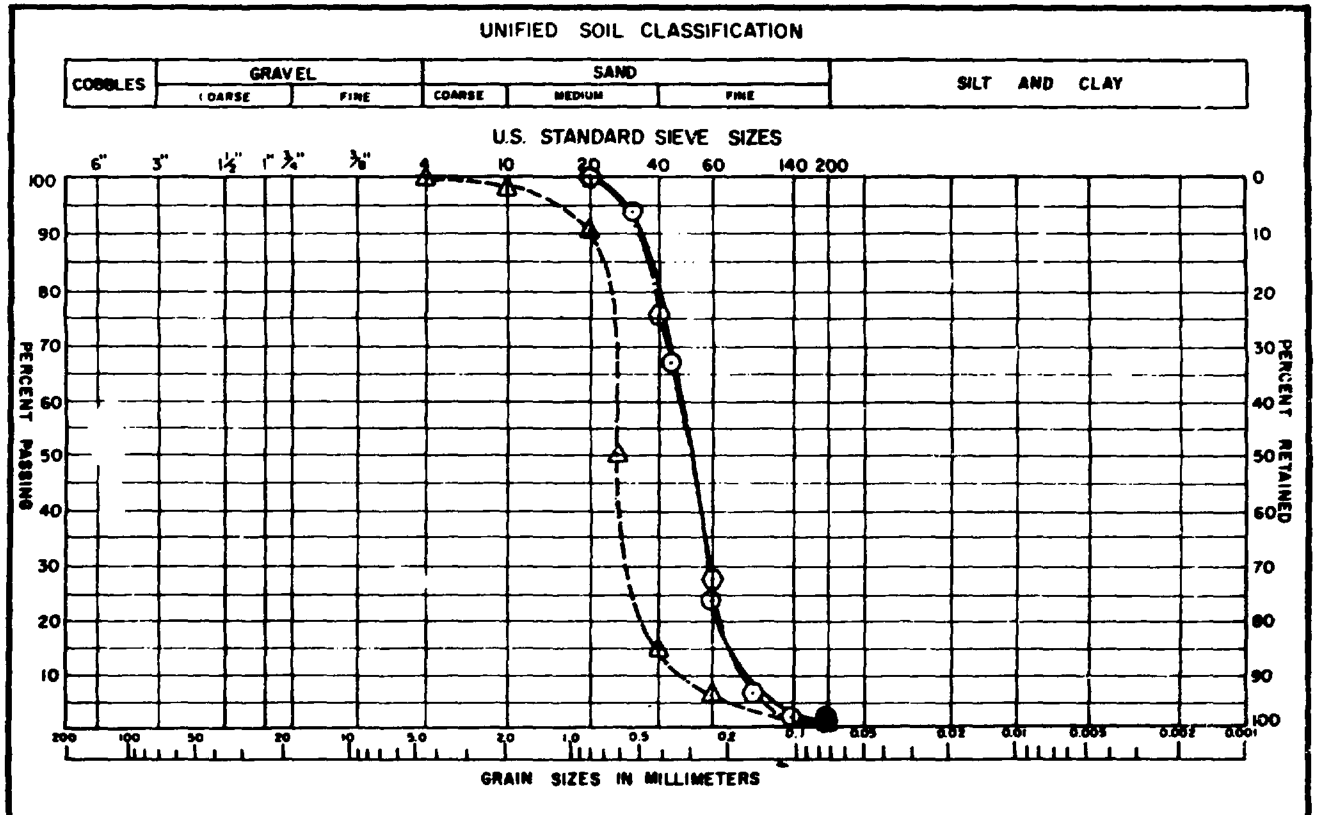
Windspeed (mi/h)	Significant Waves	
	Height (ft)	Period (s)
10	0.8	2.1
20	1.7	2.9
30	2.2	3.4
40	2.6	3.8
50	2.9	4.2

barrier beaches which enclose the Albemarle and Pamlico Sounds. At the demonstration site, soils consist primarily of tan, poorly graded, fine to medium sands. Figure 2-48 shows representative gradation curves selected from samples collected from the bluff, the beach at the waterline, and offshore. The materials are exposed in a bluff from 7 to 30 feet high and on the 20- to 50-foot-wide beach which slopes at approximately 1 on 10. The offshore area slopes seaward at about 1 on 150. The friable bluff material is easily eroded when exposed to waves and wind tides. The shoreline is generally characterized by wooded upland, narrow beach strands, and eroding bluffs.

(d) The Problem. A high erosion potential along the sandy bluff shore of Fort Raleigh is apparent, considering its exposure to the expansive shallow waters of Albemarle Sound and the record of winds applicable to that particular region. Albemarle Sound is about 56 miles long and 10 miles wide, with its long axis approximately oriented east to west. The sound is essentially freshwater, and there is no significant astronomical tide. Generally, the depths within the sound range from 17 to 20 feet, with very shallow depths of 1 to 4 feet along the fringes of land masses and within Currituck Sound, a northeastern appendage of Albemarle sound. The effects of the northwesterly exposure create a predominant eastward transport of littoral materials, which is evident by the shore configurations between the groins at Fort Raleigh and by the sandspit east of the historic site at Otis Cove (Fig. 2-49).

Erosion rates for the period 1851-1970 ranged from 2.3 to 7.2 feet per year, depending on specific shoreline position. However, between 1950 and 1970, accretion, as well as erosion, was measured. Along the eroding area, the average annual rates of erosion ranged from 4.0 to 7.5 feet per year. The accretion, totaling 30 feet, occurred within a groin field along that part of the historic site shore extending between the west end of the "Elizabethan Gardens" and the east boundary of the National Park Service lands at Fort Raleigh (Fig. 2-50).

(2) Demonstration Project. The Roanoke Island demonstration project involves three different sites (A, B, and C) (Fig. 2-50). Three of the devices consist of hand-placed artificial seaweed beds in shallow water just below NGVD, and the planting of natural shore grasses adjacent to the artificial seaweed. At all the sites an artificial seaweed bed was placed



LOCATION	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
Bluff		⊙———⊙			Fine SAND (SP)
Water Line		△------△			Medium SAND (SP)
Offshore		⊙—·—⊙			Fine SAND (SP)

Figure 2-48. Representative grain-size distribution curves, Roanoke Island site.

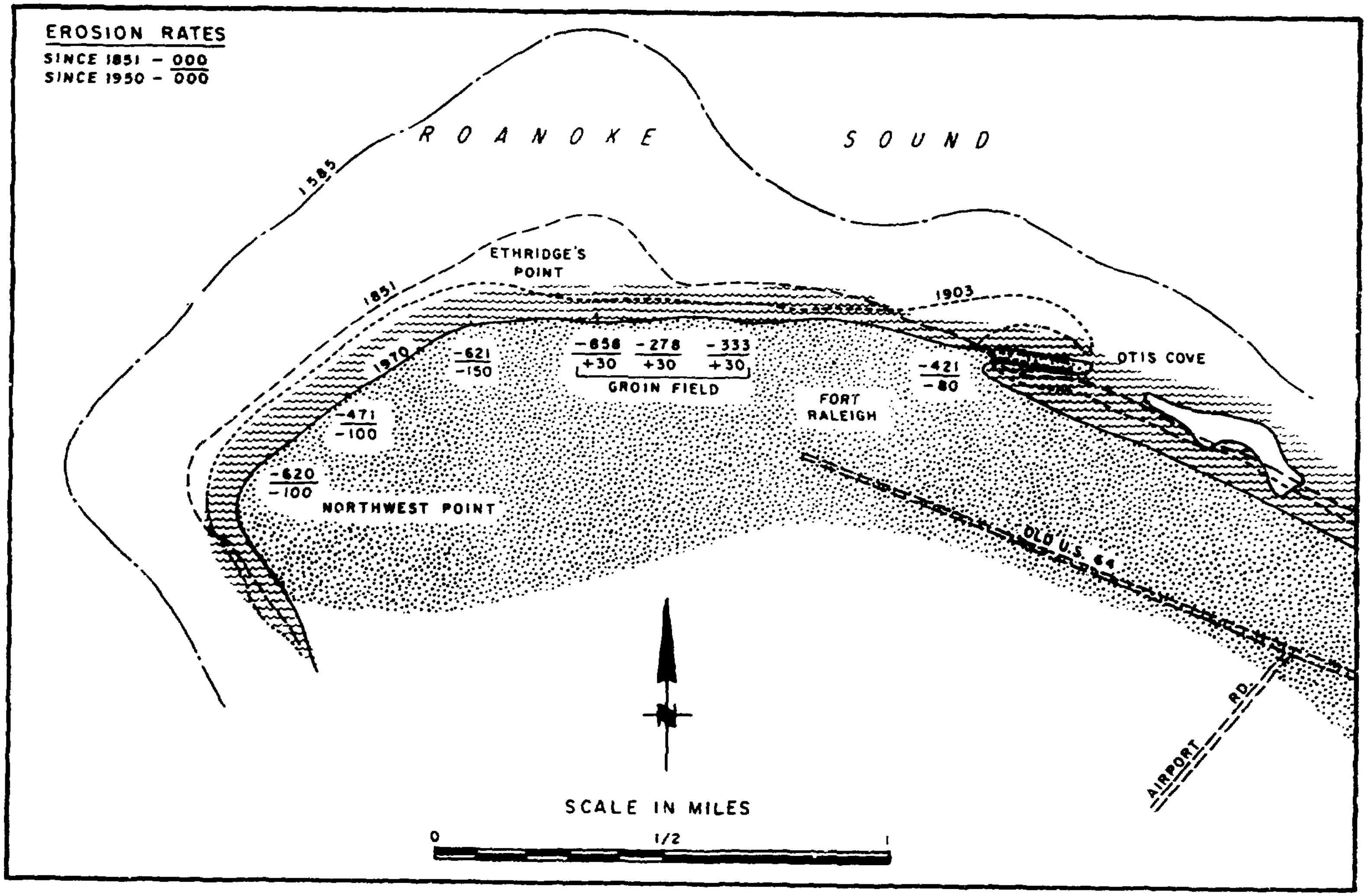


Figure 2-49. Erosion trend of Roanoke Island.

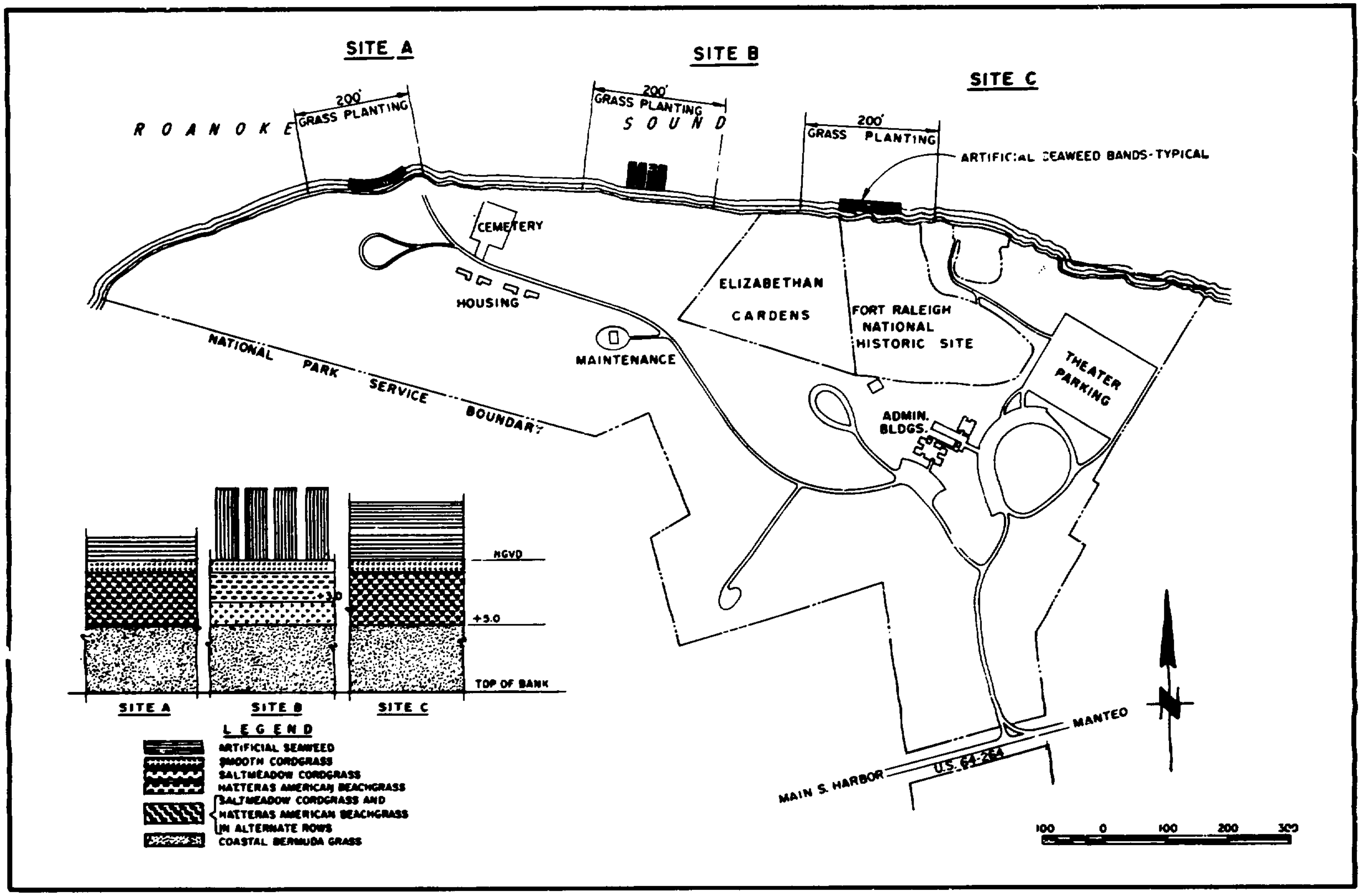


Figure 2-50. General plan of Roanoke Island, North Carolina, demonstration site.

just below NGVD, and two rows of smooth cordgrass were planted at the waterline. The artificial seaweed mat (made by Imperial Chemical Industries Limited, Harrogate, North Yorkshire, U.K.) consists of links of interwoven parawebb mat with bunches of polypropylene fabricated fronds locked into the mat on a 1.64-foot grid. The artificial seaweed mat is intended to provide a plastic curtain that interacts with sediment-bearing currents to cause an accumulation of sand.

(a) Site A. At site A, the Hatteras variety of American beachgrass was planted in alternating rows with saltmeadow cordgrass adjacent to the smooth cordgrass and extending to an elevation of 5 feet above NGVD. Coastal Bermuda grass (*Cynodon dactylon*) was planted above those grasses extending to the top of the bank where conditions permitted. The seaweed bed at site A consisted of a 7.2- by 98.4-foot strip of mat with 1.64-foot fronds placed parallel to the shoreline just below NGVD. Another 7.2- by 98.4-foot strip of mat with 3.28-foot fronds was placed beside and parallel to the mat with the 1.64-foot fronds.

(b) Site B. Saltmeadow cordgrass was planted adjacent to the smooth cordgrass and extended to an elevation 3 feet above NGVD. The Hatteras variety of American beachgrass was planted from 3 to 5 feet above NGVD, and above it to the existing bank where coastal Bermuda grass was planted as at site A. The seaweed bed at site B consisted of four 14.4- by 49.2-foot strips of mat with 3.28-foot fronds. The fronds were cut to water depth after placement. A 6.6-foot space was left between strips.

(c) Site C. The plantings at site C were identical to those at site A except for the placement of the artificial seaweed, which consisted of a 7.2- by 98.4-foot strip of mat with 1.64-foot fronds placed parallel to the shoreline just below NGVD. A 7.2- by 98.4-foot strip with 3.28-foot fronds was placed seaward of and next to the strip of mat near the waterline. A third strip of mat, 14.4 by 98.4 feet with 3.28-foot fronds, was placed 6.56 feet seaward and parallel to the others.

(3) Analysis. At the time of writing, the demonstration devices at the Roanoke Island site were just being installed, and analysis of their performance was not possible.

c. Stuart and Jensen Beach Causeways, Florida.

(1) Site Description.

(a) Geographical Setting. The Stuart and Jensen Beach Causeways are located in the Indian River tidal estuary in Martin County, Florida (Fig. 2-51). The estuary is connected to the Atlantic Ocean by St. Lucie Inlet 3 miles to the south and by Fort Pierce Inlet 16 miles to the north of the causeways. The sites comprise shore segments of four manmade islands constructed as parts of highway causeways, two forming part of the Jensen Beach Causeway, and two forming part of the Stuart Causeway, 4 miles to the south. Each causeway crosses the estuary in an east-to-west direction, connecting the Hutchinson Island barrier beach ridge with the mainland. For reference purposes the sites are numbered as shown in Figure 2-51; e.g., the west island of Jensen Beach Causeway is designated site 1,

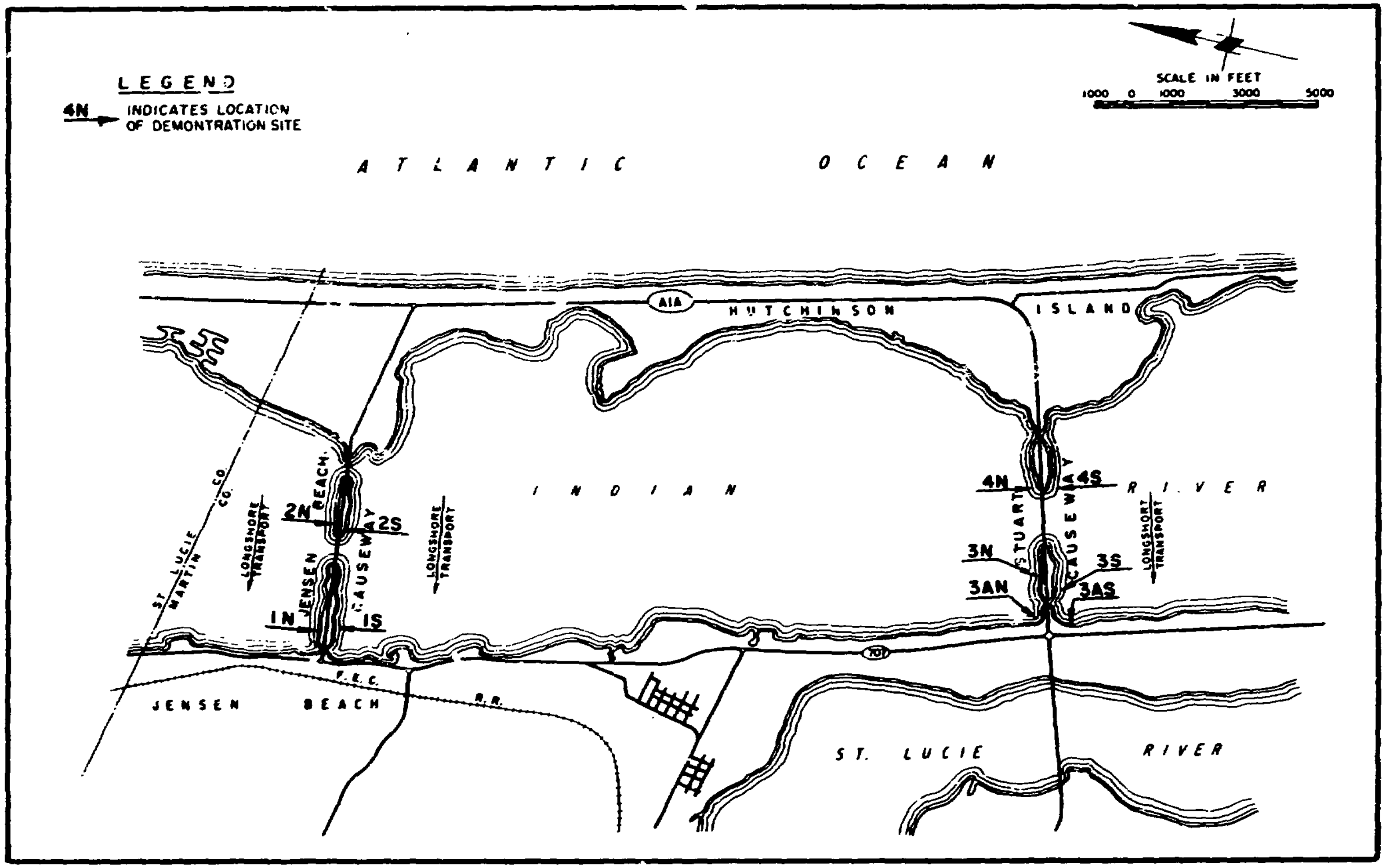


Figure 2-51. General plan of Stuart and Jensen Beach Causeways, Florida, demonstration sites.

with the north and south shores of the island designated as 1N and 1S, respectively.

(b) Climate. The subtropical weather of the south Florida peninsula is generally warm and humid. Average Fahrenheit temperatures range from 60° to 70° in winter and 80° to 90° in summer. Occasional cold airmasses moving down from the north may reduce temperatures to a low of 40° Fahrenheit in winter, but frosts are rare. Annual rainfall averages about 60 inches and is fairly well distributed throughout the year.

(c) Wind, Waves, Tides, and Longshore Transport. Prevailing winds are from the northeast and southeast quadrants, which limits the maximum effective fetch at the sites to about 10,000 feet. The east fetch limitation is Hutchinson Island, which separates the Indian River from the Atlantic Ocean. Occasionally a hurricane passes through or near the sites, increasing the windspeed to many times that of the prevailing wind regime.

The most prevalent waves affecting the sites are wind-generated, but in some locations, boat wakes cause more damage. Wind-generated waves vary from ripples up to heights of about 2 feet, being limited in size by the relatively short fetches and shallowness of the estuary. They are the primary cause of sand losses from the causeway island shorelines. However, the larger waves and wind setup caused by hurricanes are the most damaging.

The mean and spring tidal ranges for the ocean shoreline are about 2.8 and 3.0 feet, respectively. The same tidal ranges for the Indian River estuary at the demonstration sites are 1.0 and 1.2 feet, respectively.

Approximately 19 tropical disturbances passing within 50 miles of the sites since 1830 have reached full hurricane intensity (Hurricane David (1979) passed directly over the site). The 10-year design surge level due to wind setup at the Stuart Causeway is 3.5 feet, and at the Jensen Beach Causeway is 2.9 feet.

A LEO station was established at sites 2N, 2S, and 3S. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot for the three sites, with a maximum of 1.7 feet at site 2S. Periods did not exceed 2 seconds. The wave climate at each site is classified as mild. A preponderance of the waves approached from the left, generating longshore transport to the right at each station. Waves from the predominant direction were strongly skewed from the normal. Net longshore transport potentials westward at sites 2S and 3S were 6,400 and 2,200 cubic yards, respectively, for the 9 months analyzed. Energy-flux analysis indicated a net longshore transport potential eastward at site 2N of 1,400 cubic yards. However, movement of littoral material was observed westward as evidenced by an accumulation of sand on the eastern side of structures at the site.

(d) Geomorphology, Soils, and Vegetation. Hutchinson Island is a segment of barrier beach which evolved in recent geologic time by a tectonic emergence off the Florida peninsula. This east coast emergence is attributed to a rotation of the Florida plateau about its longitudinal axis. During preemergence time, the peninsula was leveled by wave action so that on emergence the entire landmass was very flat. The exposed coquina reefs that thrived in the shallow coastal waters were then also eroded by wave action.

Calcareous sands that are typical to the area were used to construct the islands that form parts of the two causeways. The demonstration sites are dredged fills of this material. An approximately 2-foot-high berm extends behind the narrow beach which ranges from 0 to 10 feet wide at MHW. With the exception of site 1N east, the foreshore and beach areas slope at about 1 on 20. At site 1N east, there is an existing concrete seawall and the immediate foreshore is nearly horizontal at about 1.2 feet below MHW.

The warm climate and frequent rains produce abundant vegetation under natural conditions. In areas where Australian pines (*Casuarina equisetifolia*) were introduced, the trees have flourished and crowded out the native plants and grasses.

(e) The Problem. Although the wave climate is relatively mild, the light, calcareous sands of this region are conducive to erosion by waves and rapid longshore transport by littoral processes. The Stuart and Jensen Beach Causeway sites are typical examples of this type of erosion.

The bluff and narrow beach along both causeways are being eroded by waves generated within the estuary. Waves up to 2 feet high generated by prevailing winds attack the shoreline. The lightweight carbonate sand becomes suspended by wave runup on the beach. Littoral currents then transport the suspended sand westward along the shoreline. Boat wakes also have adverse effects on the shorelines. Although larger waves and wind setup from hurricanes cause the most severe damage, most of the erosion is caused by ordinary wind-generated waves and the associated littoral transport. Also, the Australian pines, whose roots do not effectively retain soil, have prevented the establishment of native vegetation along the shore, which could have retarded erosion.

(2) Demonstration Project. The protection measures examined at the Stuart and Jensen Beach Causeways included both structural and vegetative devices. For native coastal vegetation plans to be effective in protecting the shoreline, the demonstration areas were cleared of Australian pines before planting. The overall plan is shown in Figure 2-51. Details of the various devices are given below.

(a) Site 1N (Revetment). A 900-foot revetment was divided into four segments in which Monoslab blocks, Turfstone blocks, Lok-Gard blocks, and standard concrete masonry units were to be installed. An isometric illustration of each block and corresponding revetment profiles are shown in Figures 2-52 to 2-55. At each end, the revetment was to be tied into the existing seawall. Each of the four revetment devices was to extend into the bottom for toe anchoring and protection. At each segment, splash aprons with and without vegetation were to be provided.

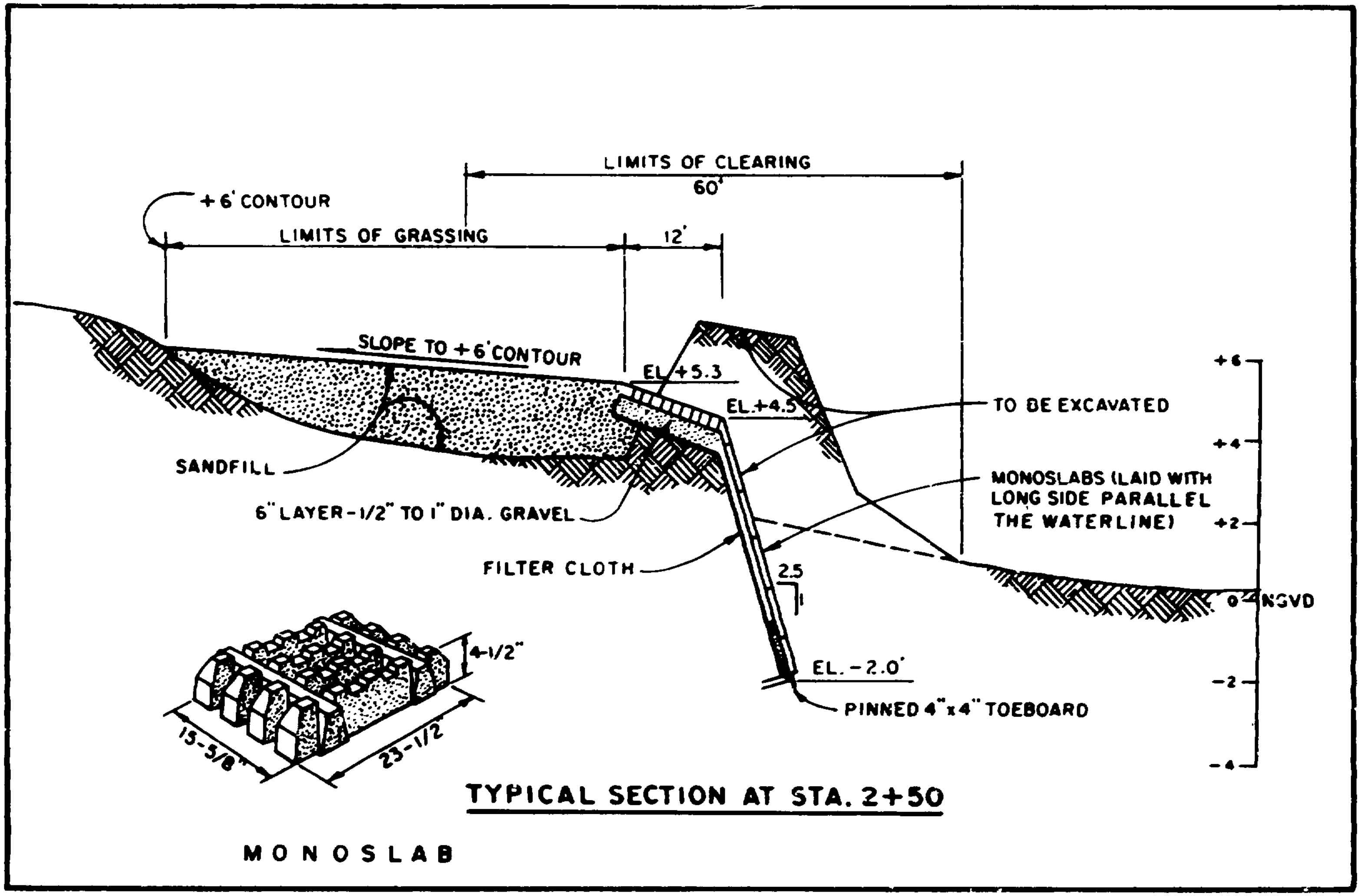


Figure 2-52. Monoslab section at Stuart and Jensen site.

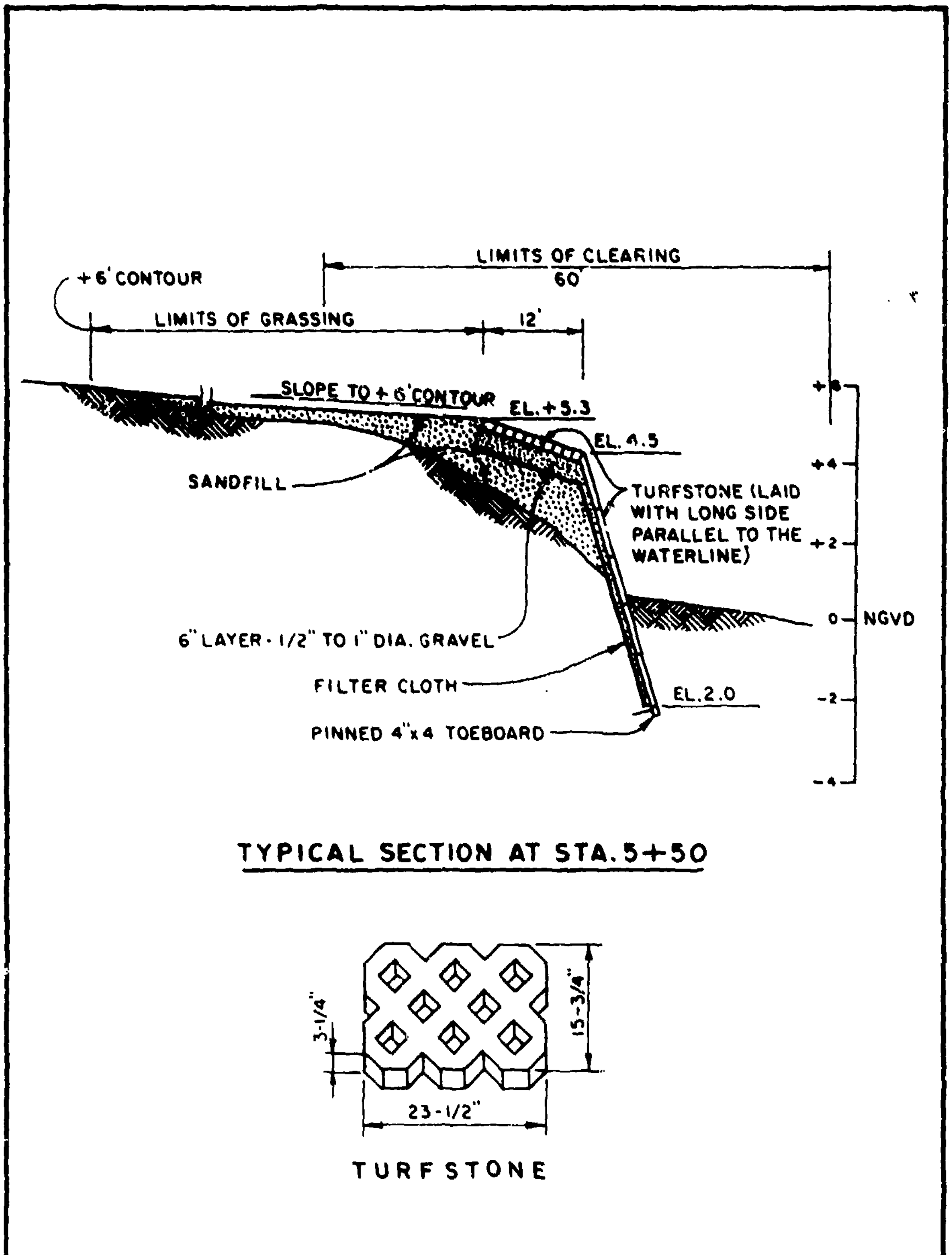


Figure 2-53. Turfstone section at Stuart and Jensen site.

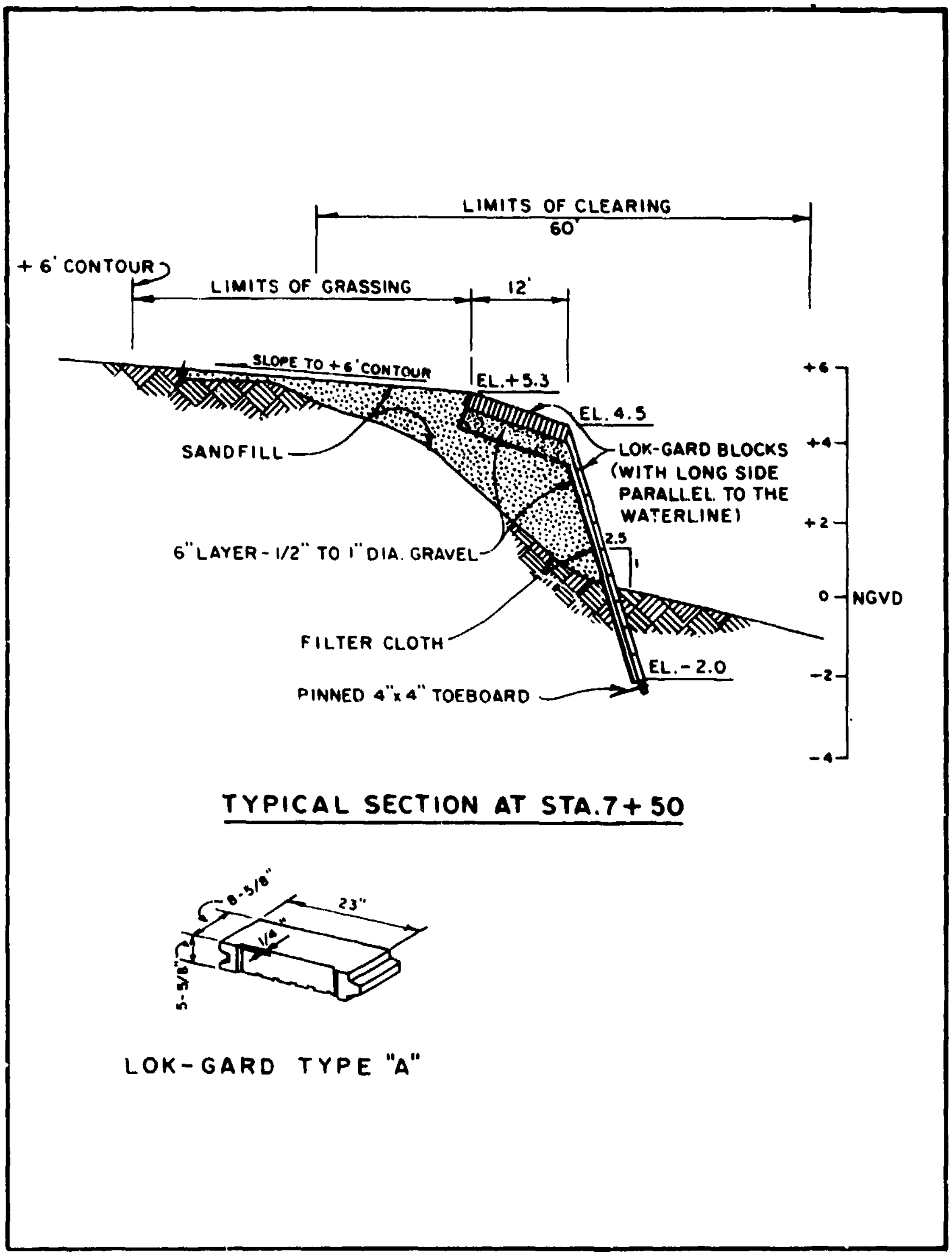


Figure 2-54. Lok-Gard section at Stuart and Jensen site.

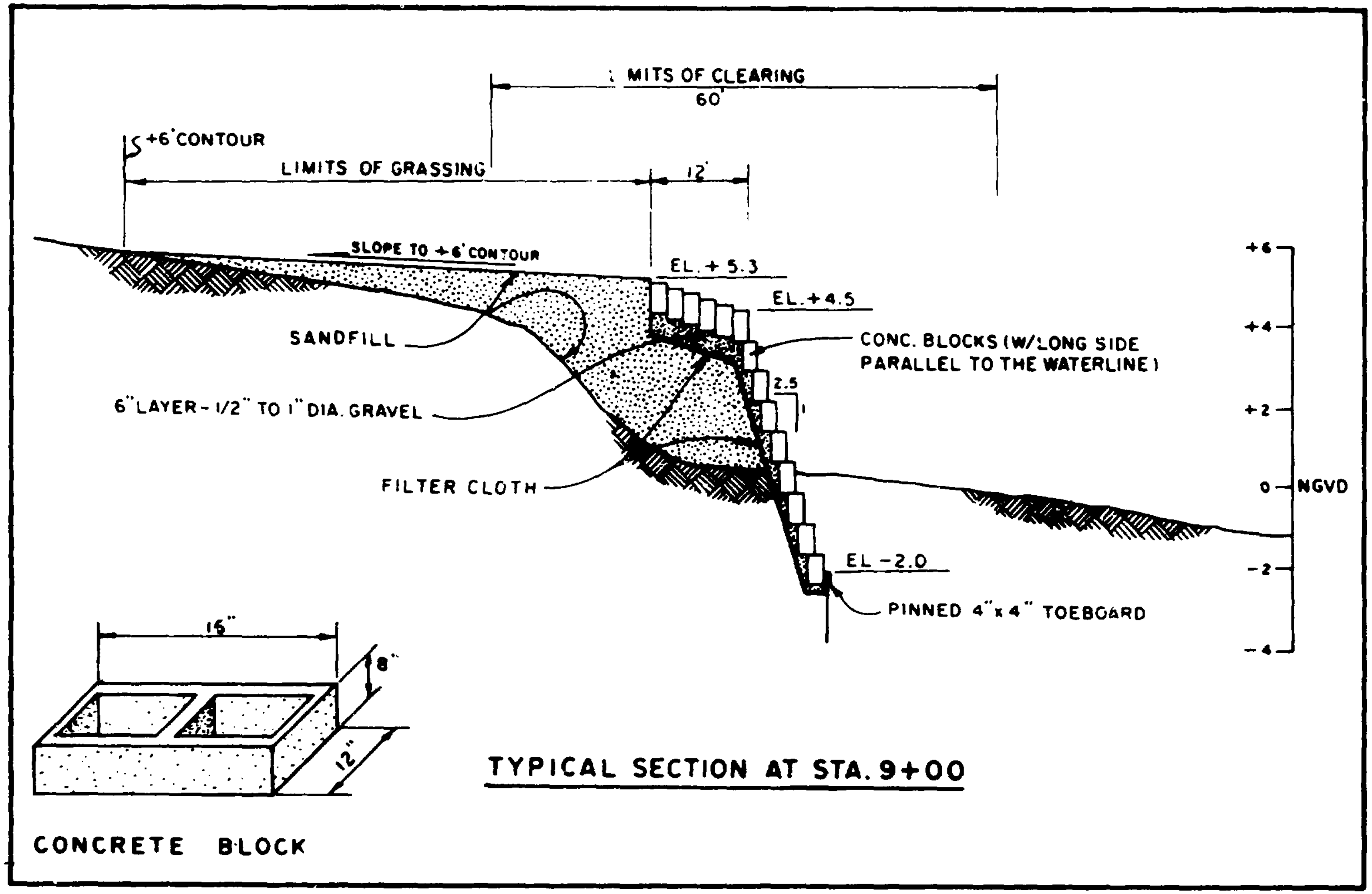


Figure 2-55. Concrete block section at Stuart and Jensen site.

(b) Sites 1S and 4N. These sites were not used.

(c) Site 2N (Sand Recycling and Terminal Groin). To recycle littoral drift, an impoundment basin was created by constructing a rubble-mound terminal groin at the west end of the island. Sand trapped adjacent to the groin was to be dredged and back-passed to a feeder beach approximately 1,500 feet updrift (Fig. 2-56).

(d) Site 2S (Vegetation). The south shore of the island, located in the most protected area on the Jensen Beach causeway, was judged most suitable for demonstrating erosion control using only coastal vegetation; various vegetation schemes were monitored. These plantings were carried out in conjunction with the removal of the existing Australian pines. In some areas the shoreline was allowed to revegetate naturally after removal of the pines.

(e) Sites 3AN and 3AS (Natural Vegetation Control). This site was used to demonstrate vegetation management for erosion control by removing the pines and leaving the shore to vegetate naturally.

(f) Site 3S (Vegetation and Temporary Tire Breakwater). Various vegetation schemes were monitored using marsh grasses and several species of mangrove. These plans were carried out in conjunction with the removal of the Australian pines. The establishment of vegetation was aided by the temporary placement of a floating tire breakwater to reduce the wave energy (Fig. 2-56).

(g) Site 3N and 4S. These sites were left undisturbed for control purposes.

(3) Statistics, Costs, and Construction. The statistics and construction costs for the structural and vegetative protection devices demonstrated at the Stuart and Jensen Beach Causeways are given by site number.

(a) Site 1N (Revetment). This revetment was under construction by Corps personnel at the conclusion of the program.

(b) Site 2N (Sand Recycling and Terminal Groin). The terminal groin was completed in March 1980 with 140 tons of 6-inch bedding stone and 185 tons of 12-inch mound stone.

(c) Site 2S (Vegetation). This site was planted twice, once in June and July 1979 and again in March 1980. A second planting was required because Hurricane David passed directly over the site in September 1979 before the plants became established, removing or destroying most of the plants in place at that time.

Planting at Jensen Causeway was done in 10 sections (110 feet wide) which were each subdivided (A, B, and C) on the basis of tidal range (Fig. 2-57). Species planted and other statistics are given in Table 2-16; their approximate distribution is shown in Figure 2-57. Costs for the first planting are included with construction materials for this site.

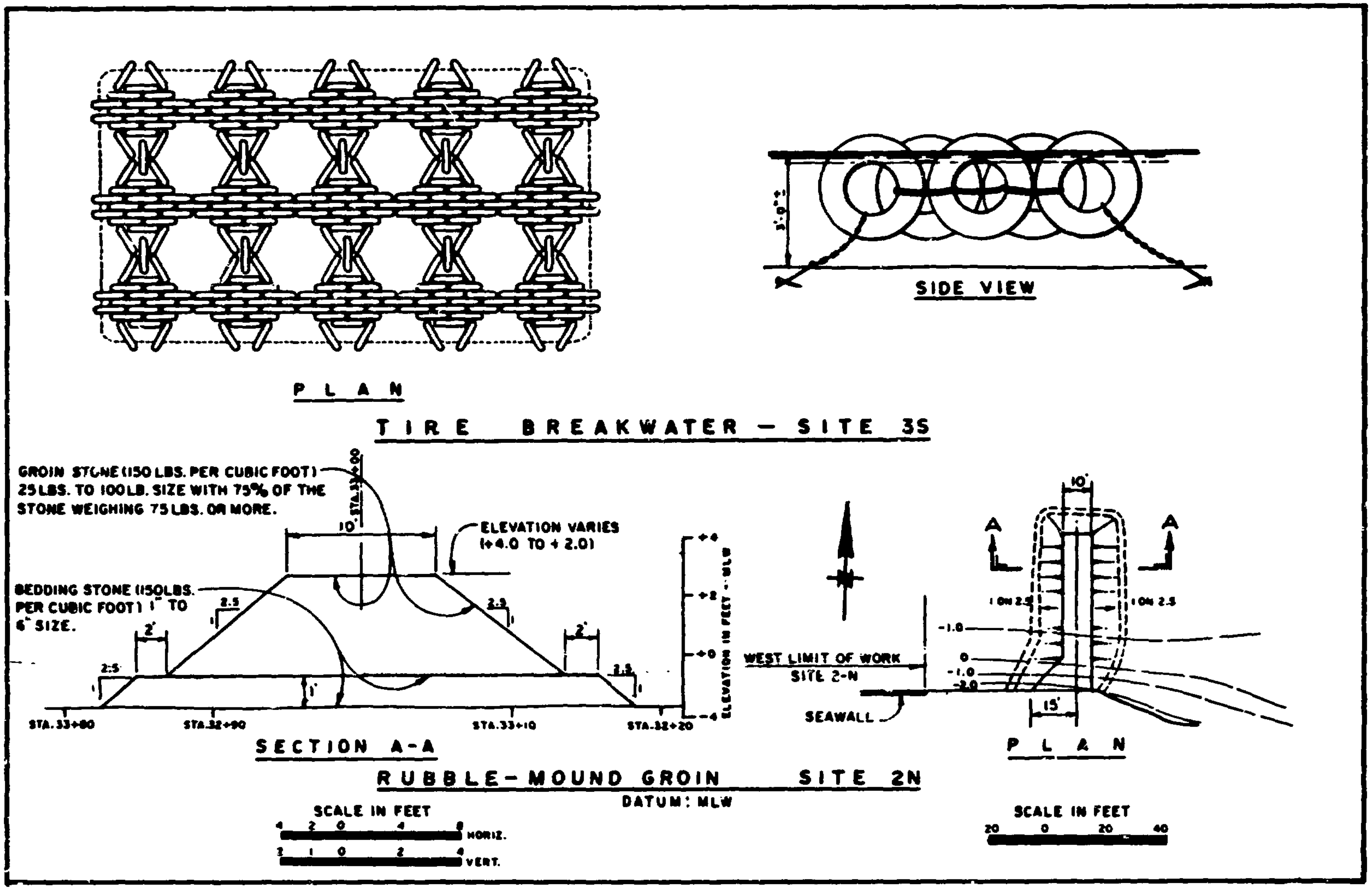


Figure 2-56. Groin and tire breakwaters at Stuart and Jensen site.

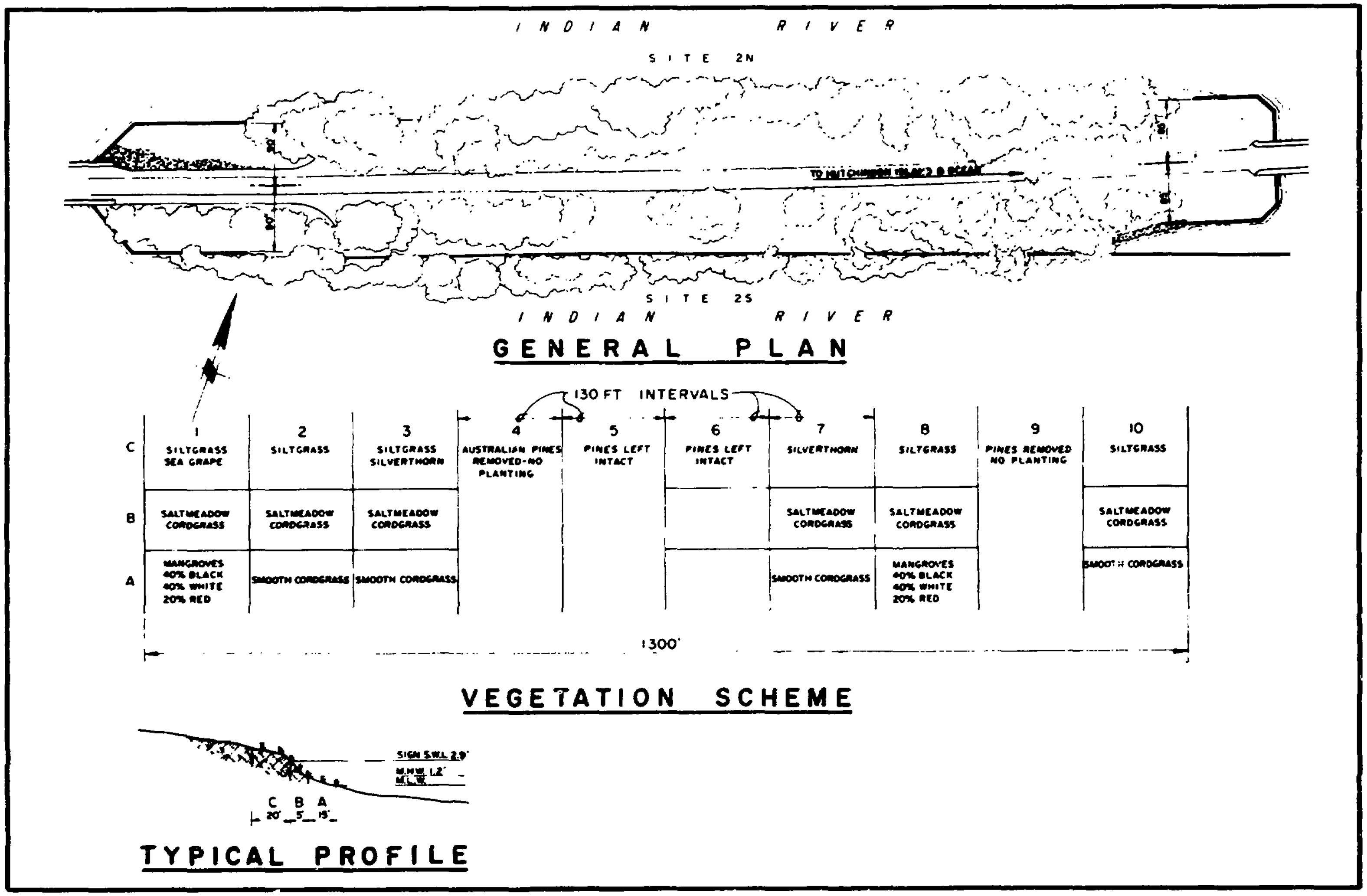


Figure 2-57. Vegetation plantings at Stuart and Jensen site 2S.

Table 2-16. Site 2S statistics, Jensen Beach Causeway.

Materials	Amount
Red mangrove (*Rhizophora mangle*)	200 plants
Black mangrove (*Avicennia nitida*)	390 plants
White mangrove (*Laguncularia racemasa*)	390 plants
Smooth cordgrass (*Spartina alterniflora*)	2,500 plants
Saltmeadow cordgrass (*Spartina patens*)	2,200 plants
Siltgrass (*Paspalum vaginatum*)	1,500 plants
Sea grape (*Coccoloba uvifera*)	50 plants
Silverthorn (*Elaeagnus pungens*)	358 plants
Sandfill	2,500 yd^3
Equipment	
Bulldozer	56 hr
Dump truck	80 hr
Dragline	24 hr
Front-end loader	32 hr
Labor	
Skilled	96 hr
Semiskilled	96 hr
Unskilled	168 hr

The second planting of Jensen Beach (site 2S) took place in March 1980. Sections 1, 2, and 3, and 6B to 10 were replanted. Species planted in each section are given in Table 2-17.

The second planting of all sections took place on 14 March 1980. All species were spaced approximately on 2-foot centers, with a density of approximately 1 plant per 4 square feet. In section 9, rubble prevented precise hole placement; however, holes were dug as close as possible to the original spacing. Smooth cordgrass was planted as plugs (in all but section 6B) which were 4 to 5 inches in diameter with 1 to 5 live shoots in a compact silty-clay soil. Plugs were planted 2 to 6 inches below original growth level in a hole opened with a power auger. Each plug was fertilized at the time of planting with 50 pounds per acre of 3- to 4- month slow-release Osmocote 14-14-14, and 50 pounds per acre of nitrogen from Mag Amp 6-40-6.

In section 6B sprigs of smooth cordgrass were obtained by washing the soil from roots of plugs then separating the root mass into live shoots with attached roots. One live shoot (sprig), about 1 foot high, was used per planting hill. Planting depth and fertilization regime for sprigs were the same as used for plugs.

Saltmeadow cordgrass was planted as either sprigs or seedlings. Sprigs consisted of a clump of 5 to 15 fine stems, more than 10 inches tall, with some attached root material, and were planted about 6 inches deep. Seedlings were held in 4-inch pots, with 1 to 3 seedlings per pot,

Table 2-17. Vegetation planted at site 2S, Jensen Beach Causeway.

Section	Plant	No. of plants
1	Smooth cordgrass Saltmeadow cordgrass Siltgrass	125 125 350
2	Smooth cordgrass Saltmeadow cordgrass Siltgrass	380 120 270
3	Smooth cordgrass Saltmeadow cordgrass Siltgrass Red mangrove Black mangrove	192 128 256 64 64
6B[1]	Smooth cordgrass Saltmeadow cordgrass	108 108
7	Smooth cordgrass Saltmeadow cordgrass	305 122
8	Smooth cordgrass Saltmeadow cordgrass Siltgrass Red mangrove Black mangrove	328 82 246 124 123
9	Smooth cordgrass Saltmeadow cordgrass Siltgrass	175 (no quantity given) 150
10	Smooth cordgrass Red mangrove White mangrove Black mangrove	115 11 17 10

[1] Section 6B was planted to evaluate the use of sprigs of smooth cordgrass after some of the Australian pines in this section were removed by the County.

and were planted at about 32 weeks old in 1- to 3- inch-deep holes. Both sprigs and seedlings were fertilized at time of planting with 50 pounds per acre of nitrogen from Osmocote 14-14-14.

Siltgrass was planted using 14-week-old seedlings raised in plastic pots with 1 to 3 seedlings per pot. The seedlings were planted at 1- to 3-inch depths and fertilized at time of planting with 50 pounds per acre of nitrogen from Osmocote 14-14-14.

Red mangrove plants were either 24 inches tall and 17 months old, or 10 inches tall and 5 months old. Both sizes were used in section 8 plantings, only 24-inch plants in section 9, and only 10- to 12-inch plants in section 10. Red mangrove was planted on alternate hills with smooth cordgrass so that plants remained on a 24-inch spacing. Both sizes of red mangrove plants were planted in auger-opened holes and fertilized at the time of planting with 50 pounds per acre of nitrogen from Mag Amp (medium granule) 7-40-6.

Black mangrove was also planted in sections 8 and 10. In section 8, black mangrove was alternated with smooth cordgrass (still on 24-inch spacing); in section 10, black, red, and white mangroves were planted in rows alternating with smooth cordgrass. Black mangrove seedlings were used in all cases. Seedlings were approximately 15 months old and about 18 inches high. Fertilization regime and planting method were the same as used for red mangrove.

White mangrove was planted in sections 9 and 10 with seedlings about 15 months old and about 18 inches tall. White mangrove was alternated with smooth cordgrass in section 9, retaining 24-inch spacing, and alternated with red and black mangroves and smooth cordgrass in section 10. Seedlings were planted in auger-opened holes and fertilized at time of planting with 50 pounds per acre of nitrogen from Mag Amp 7-40-6. Planting was completed on 14 March 1980 and an Initial Post Planting Inventory completed at this time. The cost of the first planting (including plants and site construction) was $23,789. The cost of the second planting was about $5,000.

(d) <u>Sites 3AN and 3AS (Natural Vegetation Control)</u>. No installation was required at this site. Tree removal was estimated to cost $4,800 for the work performed by the County.

(e) <u>Site 3S (Vegetation and Temporary Tire Breakwater)</u>. Statistics on this site, completed in March 1979 at a cost of $23,501, are given in Table 2-18. The vegetation was $3,541; sandfill, including all equipment and labor, was $10,475; the tire breakwater, including labor and anchors, was $8,125; and the rope for the tire breakwater was $1,360.

The Stuart Causeway plantings were destroyed by Hurricane David (which also caused the floating tire breakwater to beach); replanting was undertaken in March 1980. The site was divided into the west and the east sides. Statistics on these new plantings are given in Table 2-19.

All species were planted using the same type and size of plants as described for all sections (except 6B) of the Jensen Causeway (site 2S); planting method and fertilization were also the same. Plants were spaced 2 feet apart, except for mangroves which were planted at 48-inch spacing and were alternated with smooth cordgrass. No costs were available for the second planting.

(4) <u>Performance</u>.

(a) <u>Site 1N (Revetment)</u>. Construction of the revetments began in January 1980; however, difficulties in excavating the toe of the structures resulted in contract termination. As of May 1980, none of the revetments had been constructed.

Table 2-18. Site 3S statistics, Stuart and Jensen Beach Causeways.

Item	Amount
Materials	
Red mangrove	190 plants
Black mangrove	390 plants
White mangrove	390 plants
Smooth cordgrass	500 plants
Siltgrass	1,364 plants
Sandfill	800 yd^3
Tires	3,000 tires
Rope	6,000 lin ft
Danforth anchors	10 each
Equipment	
Bulldozer	12 hr
Dump truck	16 hr
Hand auger	8 hr
Front-end loader	8 hr
Labor	
Skilled	24 hr
Semiskilled	16 hr
Unskilled	112 hr

(b) Site 2N (Sand Recycling and Terminal Groin). The construction of the terminal groin was completed in March 1980. Sandfill had not yet been placed on the beach.

(c) Site 2S (Vegetation). Most of the initial plantings were destroyed at this site due to damage by Hurricane David and prior drought. Storm surge removed most of the loose fill in which saltmeadow cordgrass and siltgrass were planted. Smooth cordgrass in the formed material in the intertidal zone survived about 25 percent. These plants were beginning new growth at replanting time. The red mangroves were probably smothered by debris before Hurricane David.

Performance of species planted in March 1980 (the second planting) is difficult to evaluate at this date; however, some data are available. As recorded in May 1980, survival of smooth cordgrass plugs varied from a low of 33 percent in section 8 to a high of 73 percent in section 9; sprigs of smooth cordgrass in section 6B showed a 33-percent survival. Saltmeadow cordgrass showed a similar range in survival percentage with an average of about 50 percent of the plants remaining in May 1980. Black mangrove survival was very poor (19 percent) in section 3, but was considerably better in section 10 (60 percent). About 50 percent of the red mangrove was still surviving in all sections where planted in May 1980. White mangrove survival was about 40 percent; however, very few plants of this

Table 2-19. Replanted vegetation at site 3S, Stuart Causeway, Florida.

Plant	Amount
__West side__	
Smooth cordgrass	330 plants
Red mangrove	60 plants
Black mangrove	60 plants
Saltmeadow cordgrass	60 plants
Siltgrass	410 plants
__East side__	
Smooth cordgrass	180 plants
Red mangrove	25 plants
White mangrove	25 plants
__Equipment__	
Hand auger	1 each
__Labor__	
Skilled	12 hr
Semiskilled	16 hr
Unskilled	96 hr

species were planted in either section 9 or 10. Siltgrass survival was better than 90 percent in section 3, but only 40 percent in other sections where it was planted.

(d) __Sites 3AN and 3AS (Natural Vegetation Control)__. Some trees were removed from the sites in April 1979. It is not known how the sites were affected by Hurricane David in September 1979. Apparently, the method was ineffective, as vegetation did not grow naturally. It will take several more years of monitoring to evaluate natural vegetation.

(e) __Site 3S (Vegetation and Temporary Tire Breakwater)__. Assembly of the floating tire breakwater was completed by the Florida Institute of Technology in June 1979. It was assembled in sections onshore, then the sections were combined and anchored 300 feet offshore. The alinement of the structure shifted after being anchored and began orienting itself perpendicular to the approaching wave fronts (Figs. 2-58 and 2-59). Polypropylene rope with a nylon protective cover (0.5 inch) was used to bind the tire clusters together. Once installed, the rope wore through the rubber coating covering the steel band around the inside rim of the tires as a result of wave agitation. The exposed steel band then frayed the rope, eventually causing it to fail. Inspection of the breakwater indicated that a heavy growth of marine organisms (oysters and barnacles), as well as some accumulation of silt, was observed in the submerged parts of the tires. Nevertheless, the structure was effectively attenuating the normal wave activity at the site.

Figure 2-58. Floating tire breakwater at Stuart Causeway, Florida, 4 May 1979.

Figure 2-59. Floating tire breakwater at Stuart Causeway, Florida, 4 May 1979.

In September 1979, Hurricane David broke the breakwater loose and washed it ashore. Several tire bundles (30 tires per bundle) separated from the structure, scattering tires along adjacent beaches. The breakwater was reassembled in spring 1980, using steel cable instead of synthetic fiber rope and screw anchors instead of Danforth anchors.

(5) Analysis.

(a) Site 1N (Revetment). Given the soil conditions at the Stuart and Jensen Causeways, it is not desirable to place a revetment of the types indicated here. The proposed revetments required a toe that extended below the ground surface to prevent the undermining of the structure. During construction, problems were encountered in the excavating, i.e., it was not possible to keep water and soil out of the trench long enough and at the proper slope for the placement of the revetment blocks. It would be necessary to drive sheet piling in order to allow the proper construction of the revetments, which is not a low-cost measure.

(b) Site 2N (Sand Recycling and Terminal Groin). This structure was neither completed nor monitored long enough to analyze in this report.

(c) Sites 2S and 3S (Vegetation). The effectiveness of vegetation in stabilizing shoreline erosion in either causeway site cannot be evaluated. The plantings were not yet established when they were removed, along with much of the beach, by Hurricane David in September 1979. At site 3S, the floating tire breakwater severely damaged many plants that might otherwise have survived when that structure became beached on top of the plants during the hurricane. New plantings, if allowed to become well established, would provide a better indication of the potential role of vegetation in preventing erosion under normal conditions, and perhaps even under an occasional hurricane. The plantings were not intended to resist hurricane forces, however, and it is doubtful that any such device will hold the shoreline firm enough during the excessive wind and wave action. It is not advisable to apply a fill immediately before planting.

(d) Sites 3AN and 3AS (Natural Vegetation Control). This is not a particularly effective way to deal with erosion, and it is generally more advisable to plant vegetation in a region than to allow it to grow naturally.

(e) Site 3S (Temporary Tire Breakwater). The tire breakwater is a useful shore-front protection device when exposed to a moderate wave climate. The device was not installed to withstand the forces generated by a storm as large as Hurricane David.

To reduce the misalinement of the breakwater, it should be oriented perpendicular to the impinging waves when installed. The use of screw anchors reduced shifting and increased the structure's ability to endure larger storms. Also, the rope used for binding tires should be replaced with conveyor belt edging or appropriately sized galvanized-steel chain.

d. Hampton Natural Wildlife Refuge, Virginia.

(1) Site Description.

(a) Geographical Setting. This monitoring site is located on the southwest shore of Chesapeake Bay at the Hampton Natural Wildlife Refuge in the city of Hampton, Virginia (Fig. 2-60), about 3 miles north of Buckroe Beach. An unpaved access road from State Highway 169 leads to the site. The study area is about 2.5 miles of shoreline extending from the Grandview area to the mouth of Back River. The shoreline is curvilinear, changing from a northeast to northwest alinement, and along most of its length, trends approximately N. 60° W.

(b) Geomorphology, Soils, and Vegetation. The small wildlife refuge consists of undeveloped fine- to medium-grain narrow sandy beaches and low sand dunes. Landward of the dunes, vegetation covers about 60 percent of the area. Along station 0, remnants of tree trunks are visible. Large deposits of gravel (1 to 5 inches in diameter) and an abandoned lighthouse exist at station 40. A tombolo has formed between the abandoned lighthouse and the shoreline (Fig. 2-61). Exposed peat beds are present in the swash zone at stations 0, 10, and 70. Sandflats exist at the north end point near station 120.

(c) Waves and Tides. Because of the proximity of Hampton Natural Wildlife Refuge to Buckroe Beach, the wave and tide conditions are similar (see the Buckroe Beach monitoring site).

(d) The Problem. Erosion, narrow beach, low dunes, exposed peat, etc., are in response to sea level rise and shoreward migration of the barrier system. Recession of the shoreline has been evident over a number of years. The rate of shoreline recession is approximately 5 to 10 feet per year.

(2) Monitoring Project. No shore protection devices were installed at this site. It was selected for monitoring sand movement along the shore and determining whether the natural vegetation in the area is effective in controlling erosion. Monitoring consisted of three site visits beginning in March 1979 and three beach profile surveys (Figs. 2-62 to 2-65). The monitoring program was discontinued in the summer of 1979 when no significant change occurred in the rate of shoreline recession.

(3) Analysis. The inability of vegetation to become established in the lower levels of the beach in the Hampton Natural Wildlife Refuge has left the shoreline vulnerable to continuing erosion by waves and winds. As the shoreline recedes, vegetation at higher levels is undermined and washed into the bay. Natural vegetation at this site is obviously ineffective as shore protection. Former plantings by the city have also failed to halt shoreline recession, and a more effective means of shore protection is needed to prevent further loss of upland at this site (Fig. 2-66). Table 2-20 is a volumetric accounting of losses and gains by survey stations from south to north along the site shoreline from the first survey in

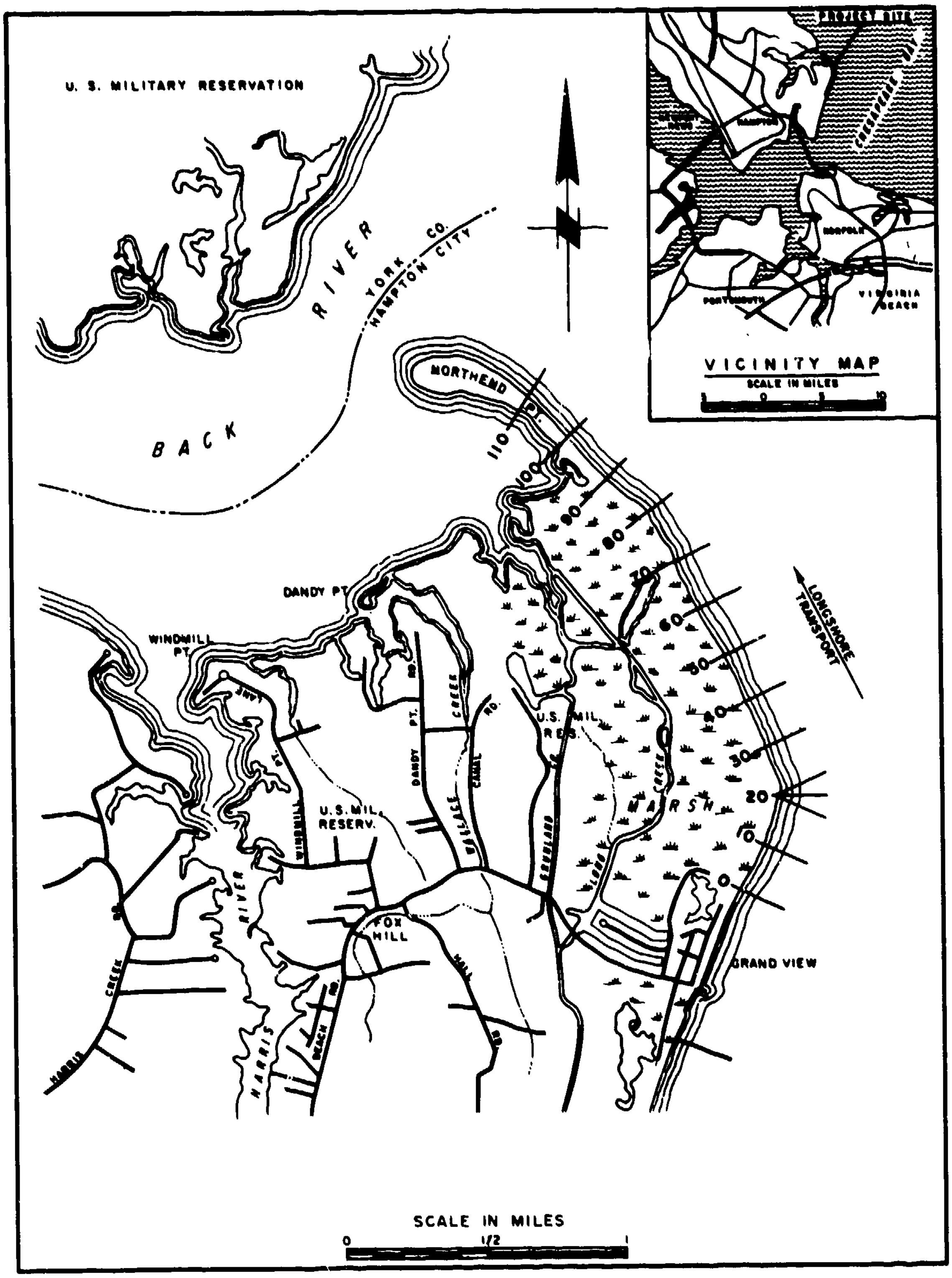


Figure 2-60. Location map of Hampton Natural Wildlife Refuge, Virginia, monitoring site.

Figure 2-61. Tombolo forming behind lighthouse at Hampton Natural Wildlife Refuge, 7 May 1979.

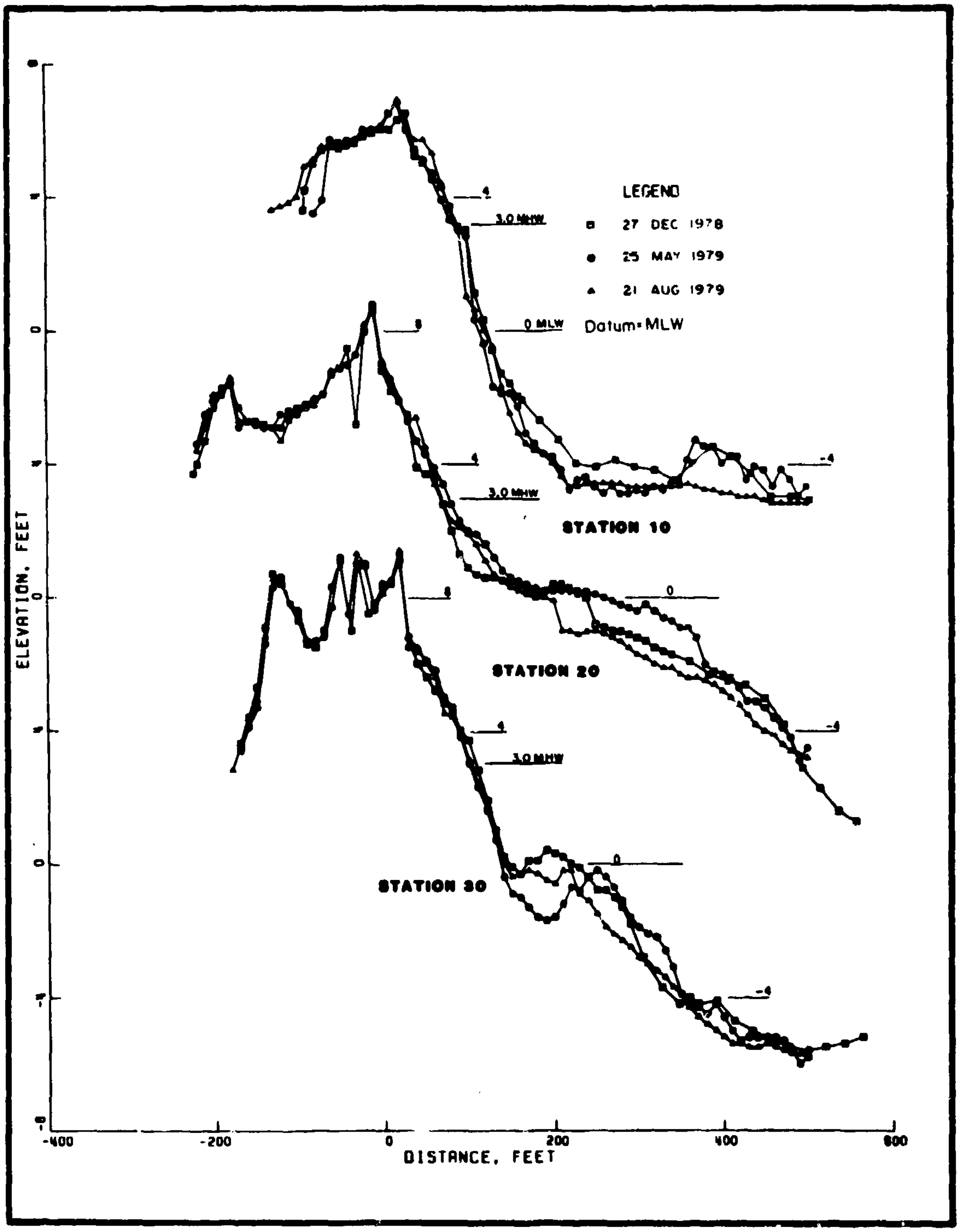


Figure 2-62. Beach profile surveys, Hampton Natural Wildlife Refuge, stations 10, 20, and 30.

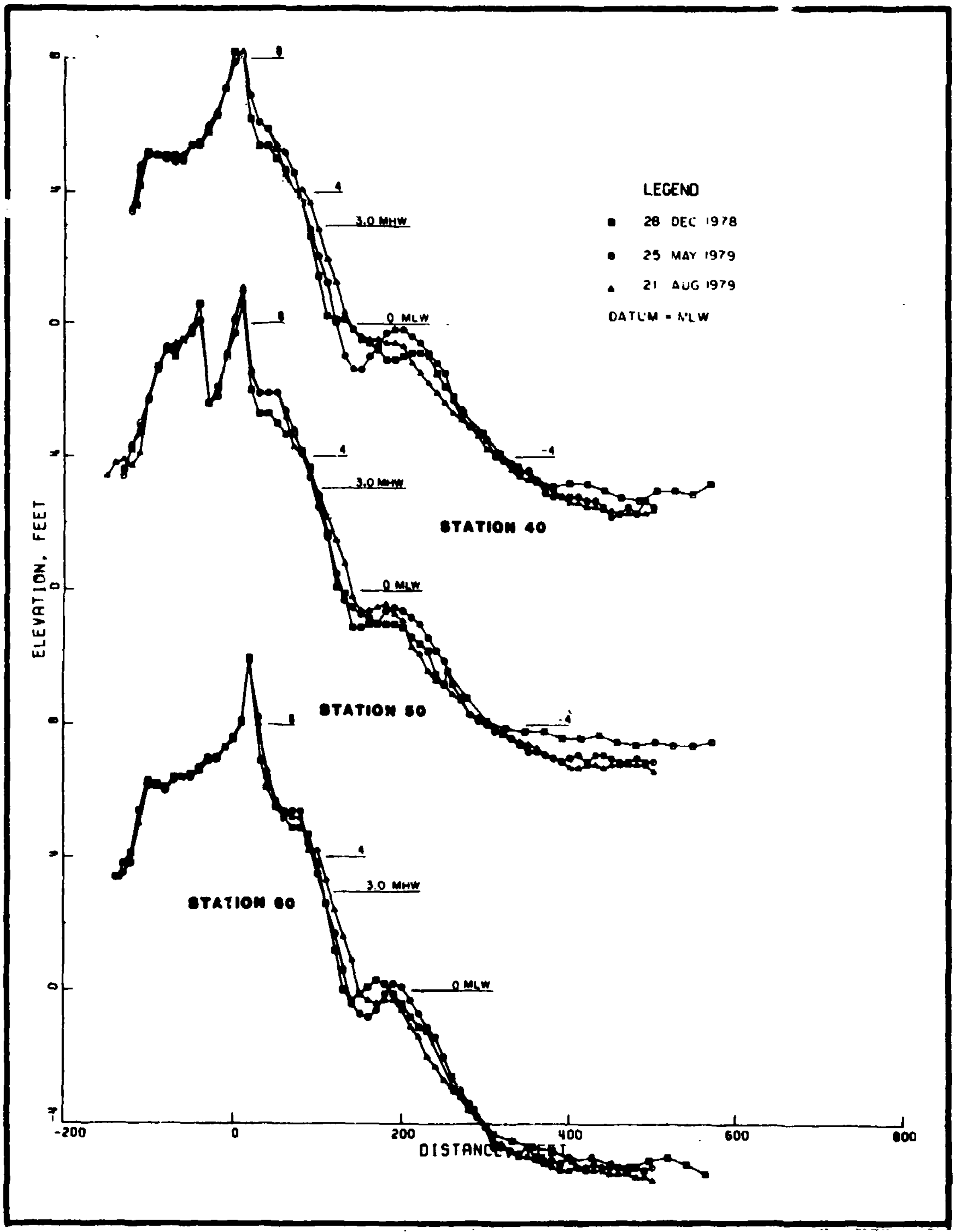


Figure 2-63. Beach profile surveys, Hampton Natural Wildlife Refuge, stations 40, 50, and 60.

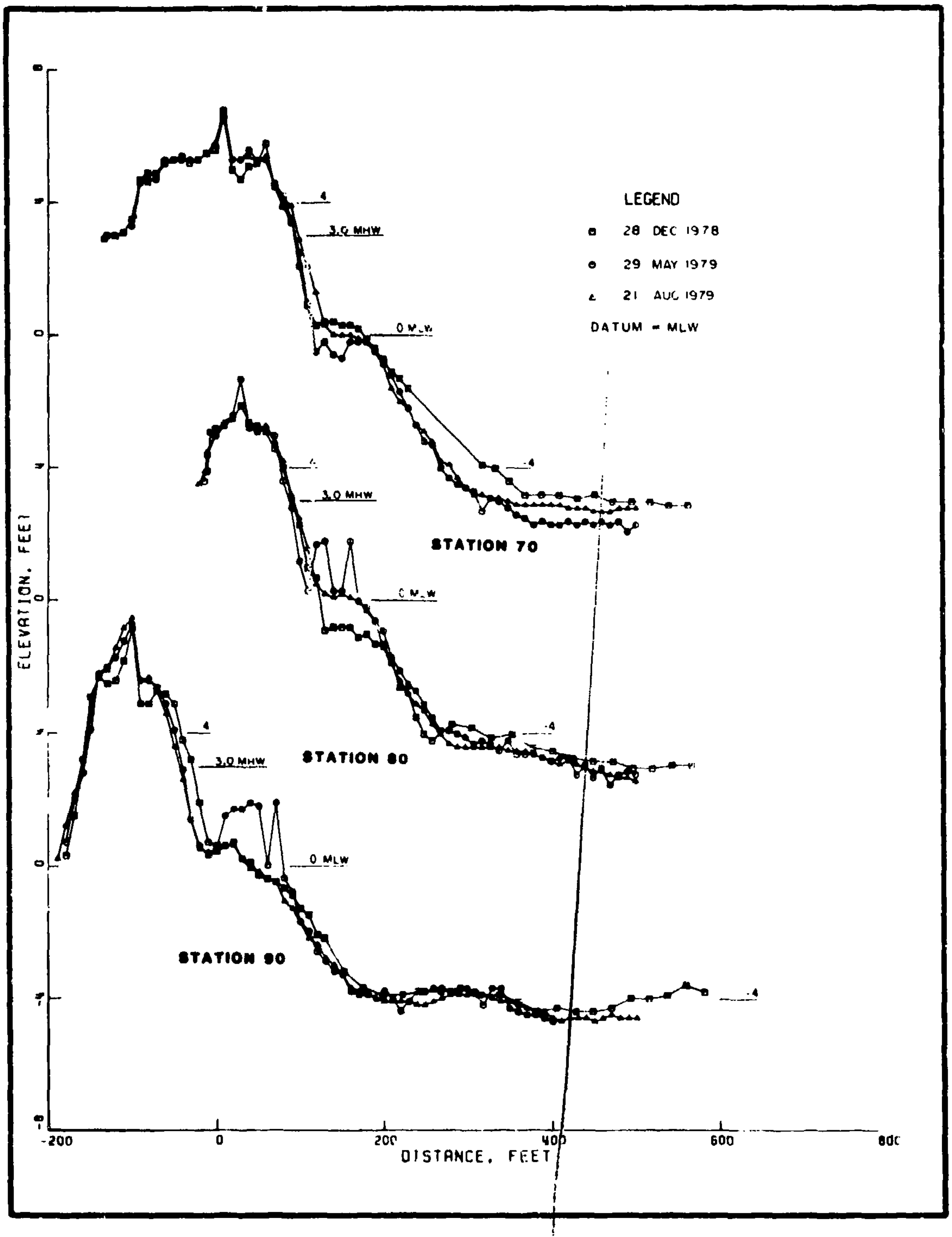


Figure 2-64. Beach profile surveys, Hampton Natural Wildlife Refuge, stations 70, 80, and 90.

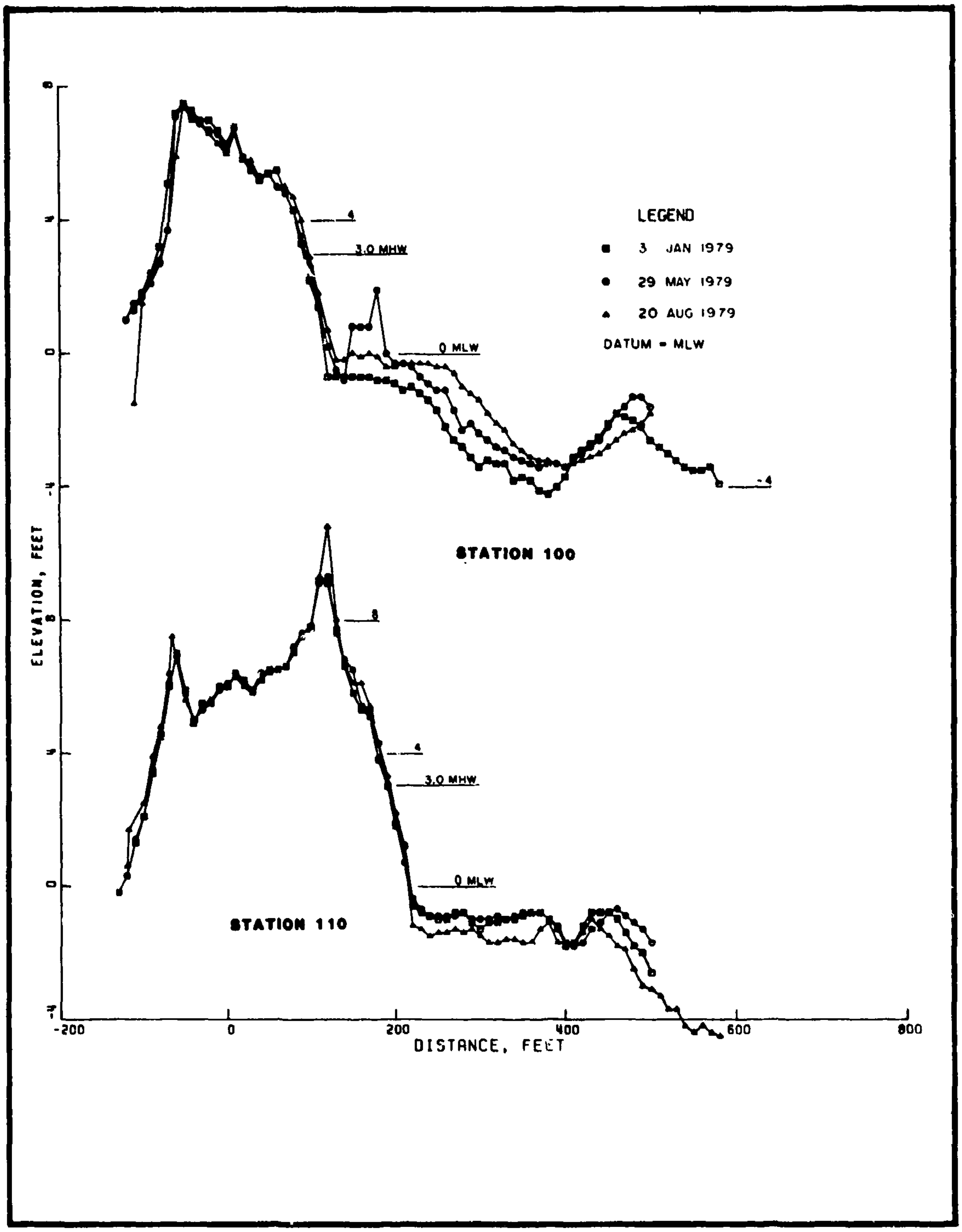


Figure 2-65. Beach profile surveys, Hampton Natural Wildlife Refuge, stations 100 and 110.

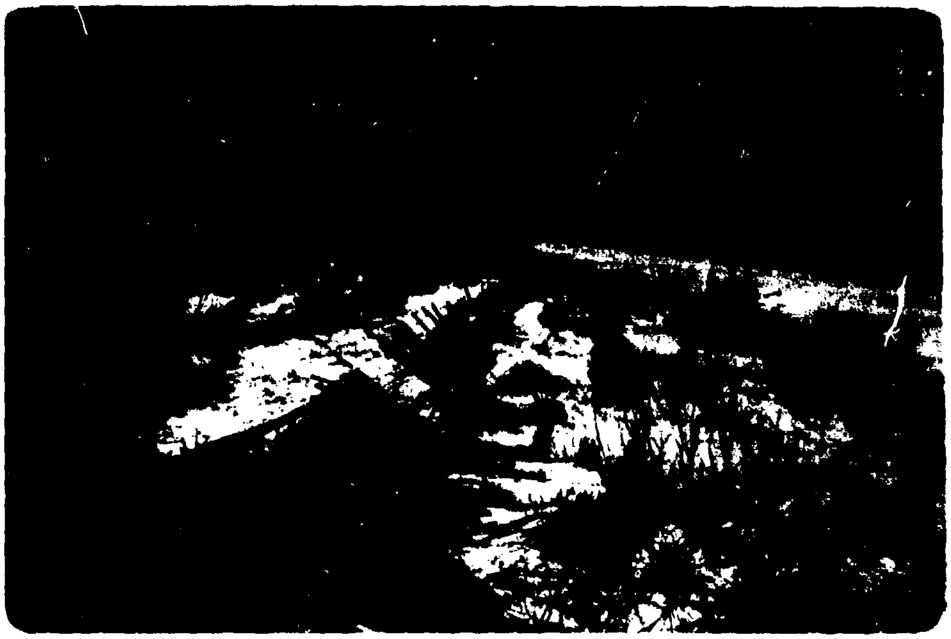


Figure 2-66. Typical beach-dune profile showing vegetation planted by the city and a deteriorating sand fence, Hampton Natural Wildlife Refuge, 12 March 1979.

Table 2-20. Volumetric analysis of beach profiles at Hampton Natural Wildlife Refuge, Virginia (25 May 1979 to 22 August 1979).

Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
30+00			
40+00	4,997.7	3,766.2	-1,231.5
50+00	3,503.3	3,123.7	-379.6
60+00	3,699.6	2,431.1	-1,268.5
70+00	2,286.2	3,842.7	1,556.5
80+00	1,981.7	3,550.2	1,568.5
90+00	4,657.5	1,651.0	-3,006.5
100+00	5,754.9	3,347.5	-2,407.4
110+00	3,361.7	3,602.5	240.7
120+00	985.8	2,884.0	1,898.1
127+46	1,471.7	6,942.4	5,470.7
Totals	32,700.2	35,141.2	2,441.1

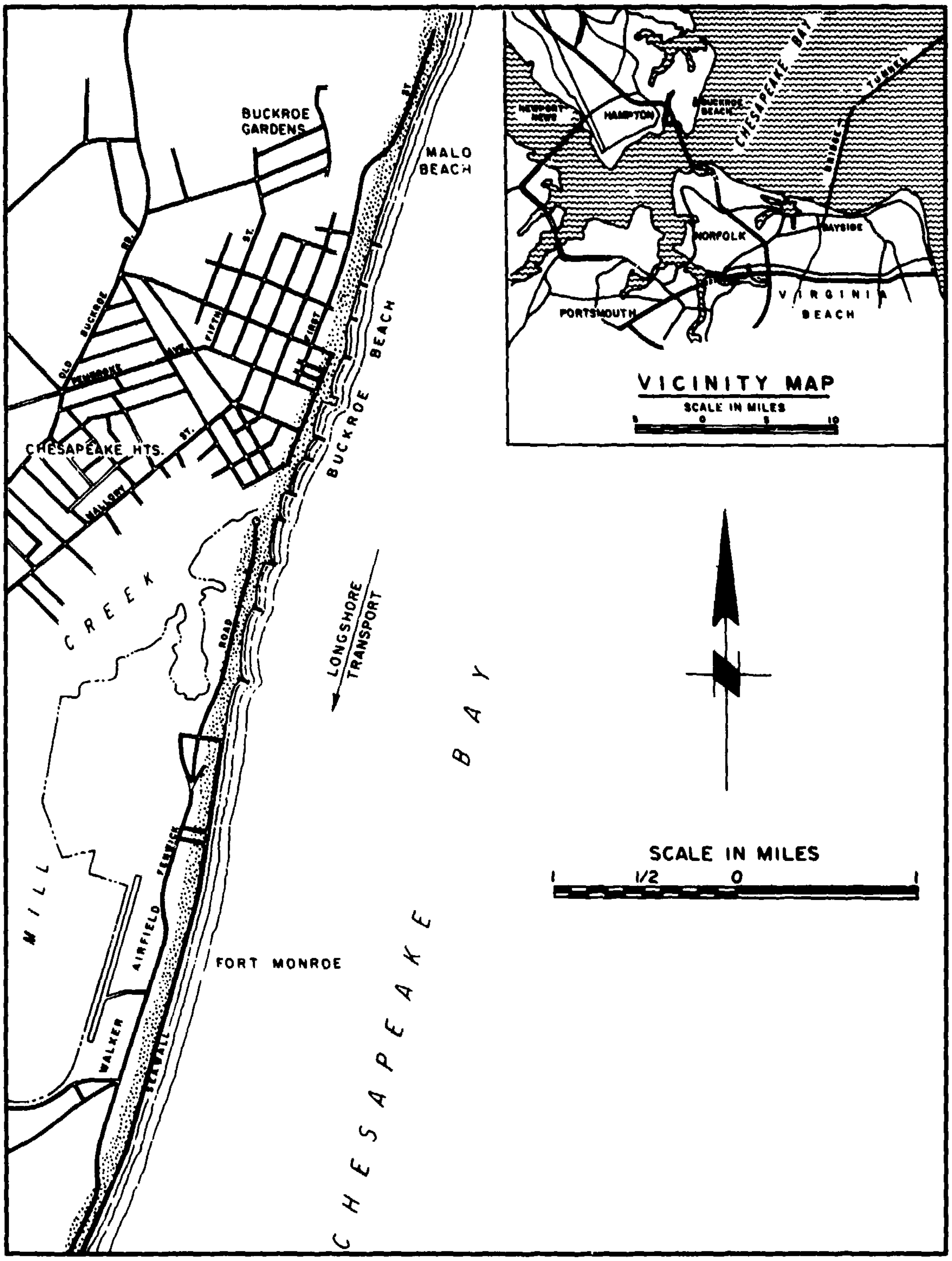


Figure 2-67. Location map of Buckroe Beach, Virginia, monitoring site.

December 1978 to the third survey in August 1979. It shows no distinct pattern other than an accretion trend near the tombolo near station 40. Figure 2-61 shows a widening of the beach to the south of the tombolo, indicating northward littoral transport. The configuration of the north end sandspit also indicates northward transport. The profile indicates local seasonal changes in some areas, but the time interval between surveys was too short to indicate any long-term trends.

e. Buckroe Beach, Virginia.

(1) Site Description.

(a) Geographical Setting. Buckroe Beach, the primary public beach at Hampton, Virginia, is located on State Highway 169 about 2 miles from the Hampton Roads Bridge-Tunnel to Norfolk, Virginia. The site is on the southwest shore of Chesapeake Bay, about 7,000 feet north of the Fort Monroe military reservation. It is approximately 3,300 feet long, trending roughly N. 15° E. and is fronted by a continuous concrete-capped timber bulkhead (Fig. 2-67).

(b) Geomorphology, Soils, and Vegetation. The beach sediment consists of an upper layer of fine to medium sand with traces of pea gravel. Beneath this upper layer are peat beds. Local vegetation is sparse because the beach face ends at the bulkhead, and the area immediately behind the bulkhead and concrete cap is developed with sidewalks, roads, and buildings. Some grass patches grow at the south end of the study area where residential properties begin.

(c) Waves, Tides, and Longshore Transport. The mean tide level at Buckroe Beach is 1.[illegible] feet above MLW. The spring range is 3 feet and the mean is 2.5 feet. Under normal conditions, wave heights are from 1 to 2 feet with periods of about 7 seconds. During storm conditions, wave heights are 4 to 6 feet with periods of about 5 seconds. The fetch distances vary from 5 miles southward to more than 80 miles from the northerly directions. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 4.0 feet. The wave climate is classified as intermediate. The wave data for this site were not considered sufficiently complete for a meaningful energy-flux analysis. However, the large accumulation of sand on the north sides of the groins indicates a net southward longshore transport.

(d) The Problem. Buckroe Beach has been eroding at a rate of 5 to 10 feet per year. To preserve the beach, in October 1967 the city of Hampton constructed eight treated-timber groins of various lengths along the 3,300-foot reach of the monitoring site. Also, the Virginia Institute of Marine Science placed a low offshore sandbag sill about 500 feet long parallel to the shoreline in an effort to hold a perched beach. The sill extends from groin 1 to groin 5 at about the -2.0-foot MLW contour (Fig. 2-68).

(2) Monitoring Project. The structures described above were monitored to evaluate their structural and functional performance.

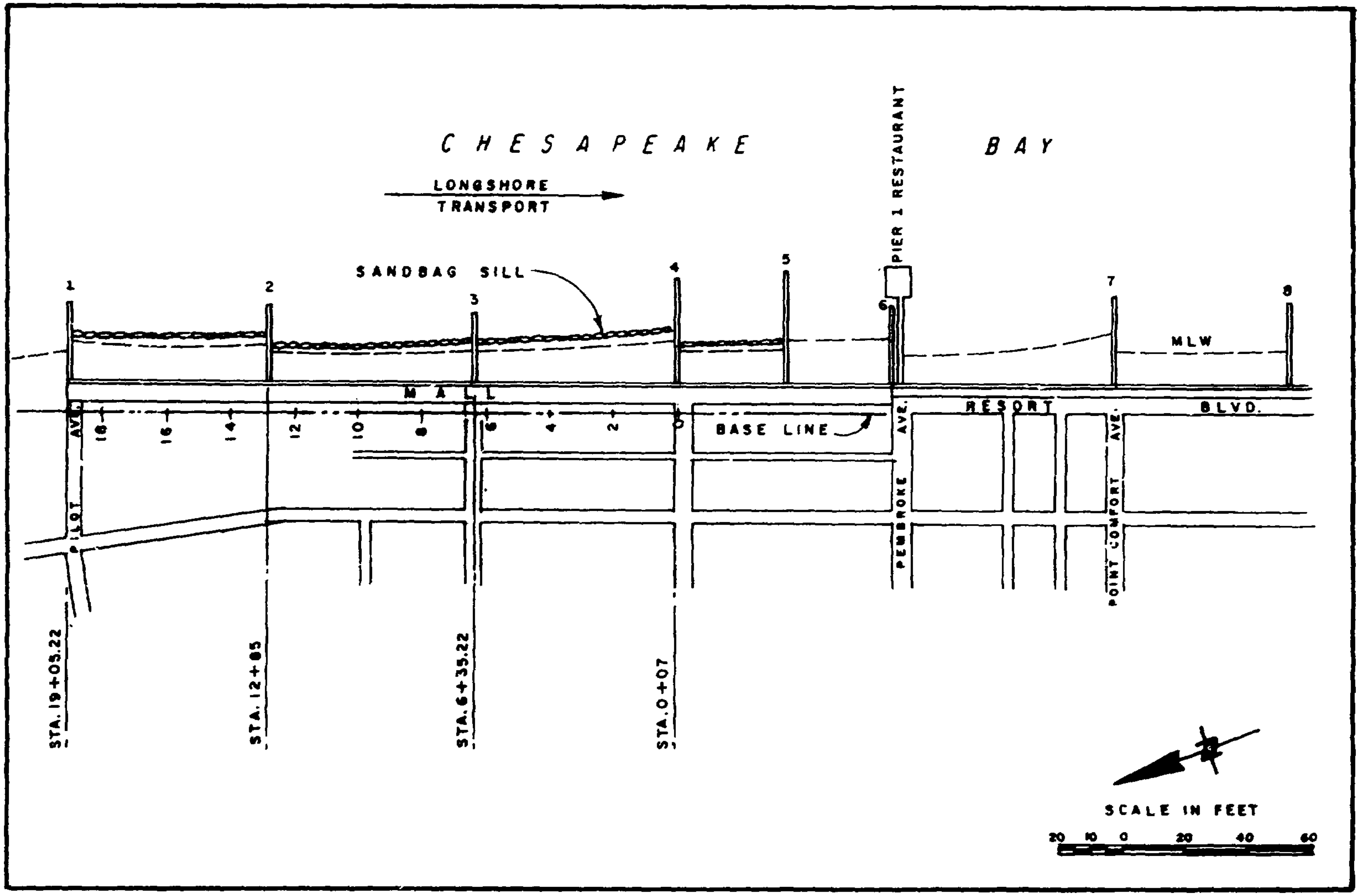


Figure 2-68. General plan of Buckroe Beach, Virginia, monitoring site.

(3) Performance. No loss in the structural integrity of the protective devices monitored at Buckroe Beach has been observed. The groins, bulkhead, and sandbag sill have not been damaged or altered during the study period. There is no evidence of structural material degradation (Fig. 2-69).

The beach has narrowed slightly during the study period, with wider beaches located at the south end of the project (where no surveys were made), narrowing to the north. Littoral transport is assumed to be from north to south as evidenced by the large accumulation of sand on the north sides of most of the groins. At groin 8 the beach is 180 feet wide on the north side of the structure, and 90 feet wide on the south side. The beach narrows to the north to a uniform width near groin 2. Figure 2-70 illustrates these groin effects.

The sandbag sill has not accumulated enough sand to create a perched beach; however, the sill has successfully retained the sand at the existing beach. A series of profiles in the beach segment from groin 1 to groin 4, surveyed four times in 1979, showed almost no movement of the shoreline and only small seasonal changes offshore. Figure 3-77, which was put in Section III for comparison of profiles through similar devices, shows a series of profiles through the stacked bag sill at Buckroe Beach. Profiles through the treated timber bulkhead, compared with those of similar devices, are shown in Figure 3-17 of Section III.

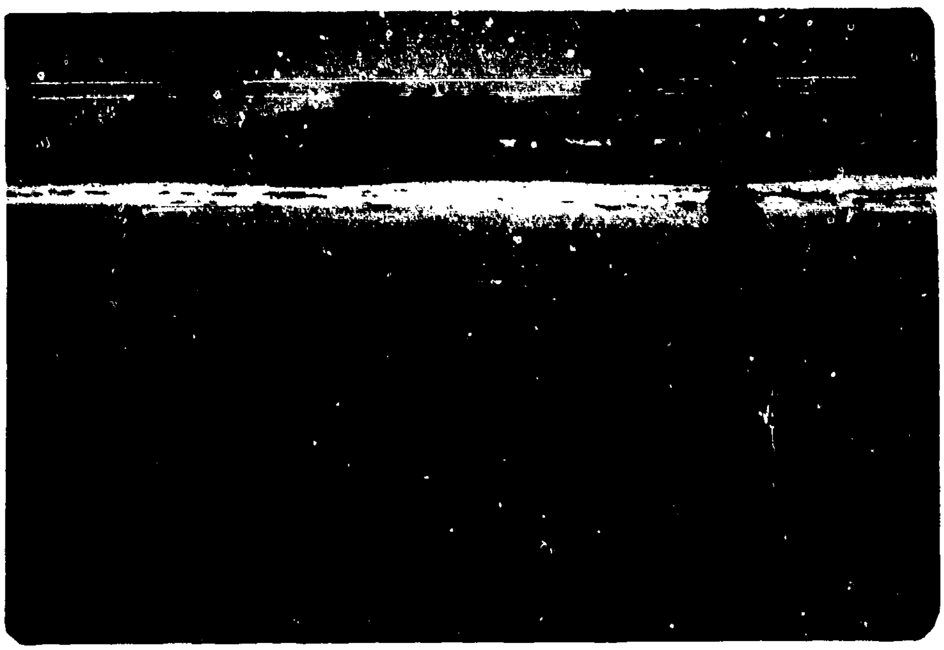


Figure 2-69. Typical condition of structures (bulkhead, groin, and sill), Buckroe Beach, Virginia, 12 March 1979.

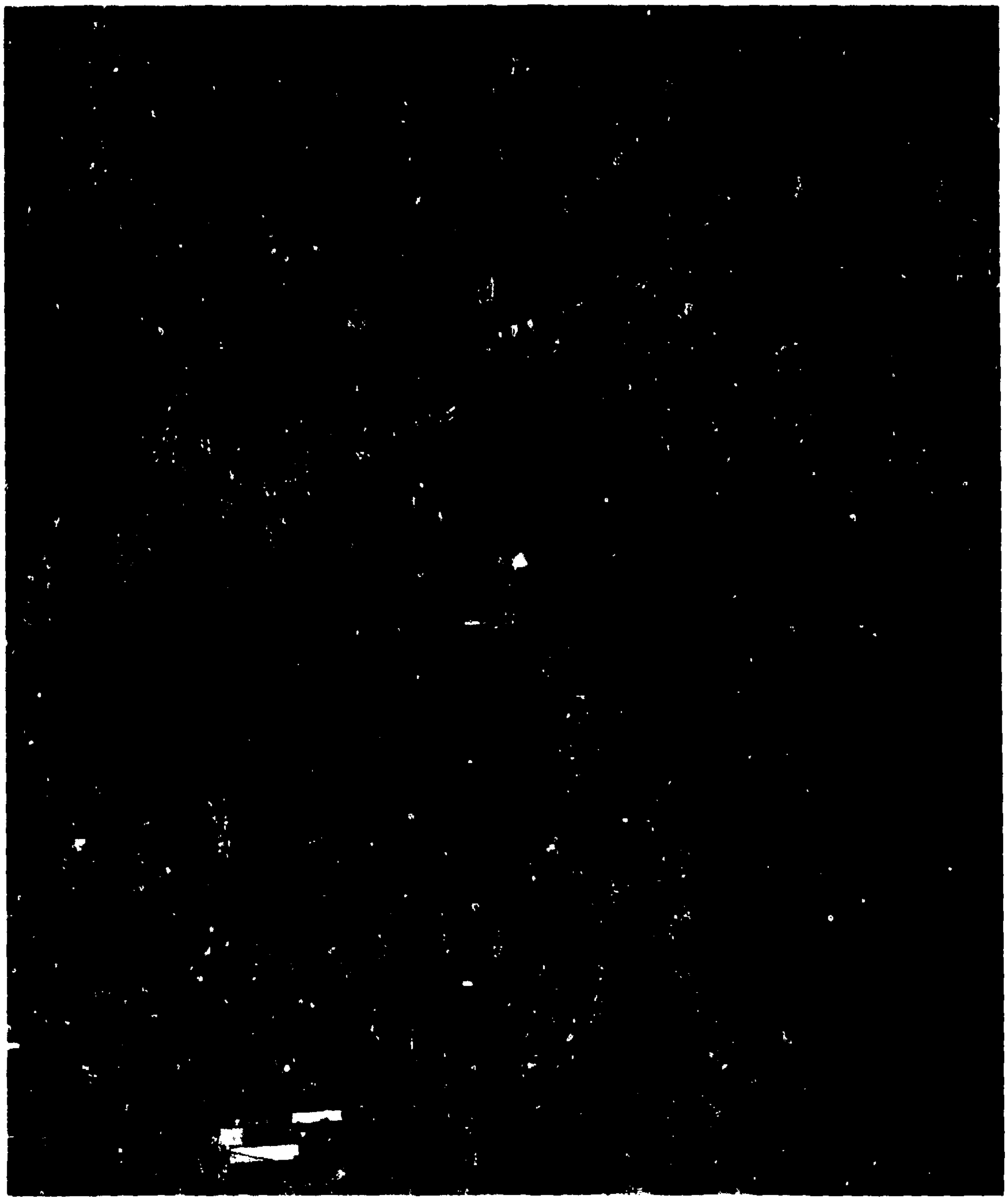


Figure 2-70. Aerial view indicating shoreline trends, Buckroe Beach, Virginia, 7 May 1979.

After a heavy snowstorm in February 1980, the beach was covered with debris, indicating that unusually high water levels occurred during the storm. The eight timber groins and the continuous timber bulkhead were in good condition. In March, another snowstorm with blizzard conditions hit Buckroe Beach. Again, an unusual amount of debris was left on the beach (Fig. 2-71) and there was evidence of above-normal high tides. It was also noted that the beach had been lowered adjacent to the seawall (Fig. 2-72).

(4) Analysis. The installation of a groin system at Buckroe Beach as allowed only slight erosion between groins 1 to 8 in recent months. ›pending on the direction of the littoral transport at any given time, erosion and accretion occur alternately on either side of the groins. The accretion pattern indicates that the predominant direction of littoral transport is southward. Yet, sand still accretes on the south side of groin 1 instead of the north side. This may be due to the influence of the drainage of Mill Creek into the bay from the north.

f. Duck, North Carolina.

(1) Site Description.

(a) Geographical Setting. This monitoring site is located near the Duck Field Research Facility (FRF) of the Coastal Engineering Research Center (CERC) about 17 miles north of the Roanoke Island demonstration site on the east bank of Currituck Sound, North Carolina (Fig. 2-73). It can be reached by going north along the barrier island about 6 miles from the east end of Wright Memorial Bridge. The shoreline is generally straight, with a north-south orientation. The shore is a 4- to 5-foot eroding sand scarp backed by land covered with native grasses and shrubs. The soil is predominantly fine sand with a median diameter of 0.125 to 0.250 mm, with less than 1 percent organic material. Salinities vary from 1 to 5 parts per thousand.

(b) Wind, Waves, Tides, and Longshore Transport. Winds at Duck are predominantly from the northeast and southwest, the northeasterlies having the highest speeds. However, the site is exposed only to waves generated by southwesterly to northwesterly winds blowing across Currituck Sound. As the sound is only 4 miles wide at the site, wave periods seldom exceed 3 seconds. Winds from the westerly directions occasionally have sustained speeds of about 20 miles per hour. Boat wakes are not a factor. The astronomical tides in the sound are negligible, but hurricane winds raise the water level several feet. Mean water level is about 0.9 foot NGVD. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 1.0 foot. The energy-flux analysis indicates that wave energy along this reach of shoreline is so low that no significant longshore displacement of littoral drift should be anticipated. The wave climate is classified as mild.

(c) The Problem. From 1941 to 1965, the Duck site was used as an aircraft bombing range, and most of the formerly existing marsh along the Currituck Sound shore was destroyed. In areas where the marsh growth was destroyed, the east bank of the sound has been eroding at a rate of

Figure 2-71. Unusual amounts of debris were on the beach after a snowstorm, Buckroe Beach, Virginia, 21 March 1980.

Figure 2-72. The sand was cut away from the timber bulkhead, exposing the bulkhead in some areas, Buckroe Beach, Virginia, 21 March 1980.

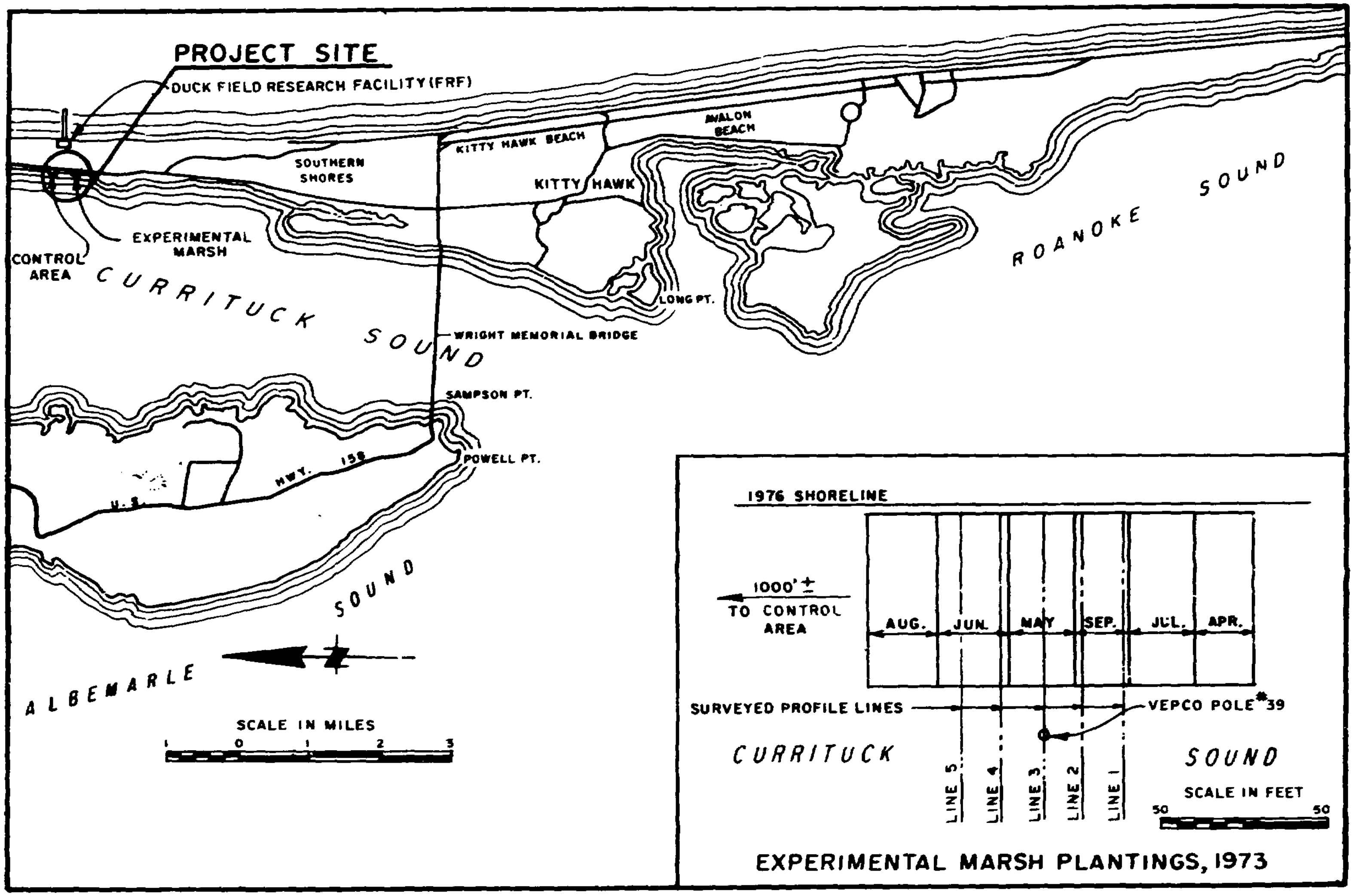


Figure 2-73. Location map and vegetation plan at Duck, North Carolina, monitoring site.

about 8 feet per year. A line of power transmission poles, once on land, is now about 100 feet from the shoreline, and the original road through the area is now under water.

(2) Monitoring Project. In 1973, CERC initiated an experimental marsh project along the sound behind the FRF, then under development on the Atlantic side of the barrier island. A plot about 230 feet long extending from the base of the eroding bluff about 100 feet through the tidal zone into shallow water was planted with four species of marsh plants: narrow- and broad-leaved cattails (*Typha augustifolia* and *T. latifolia*), giant reed (*Phragmites australis*), smooth cordgrass (*Spartina alterniflora*), and black needle rush (*Juncus roemerianus*). These species were randomly planted along the transects to determine zones in which each species would best survive. The site was subdivided into six subplots, as shown in Figure 2-73 (inset) and one subplot was planted each month as indicated during the summer of 1973. Additional plantings were made to the north of the site during the winter, but survival was poor. The planting transects were 3 feet apart and consisted of single culm plantings at 3-foot spacings. The wide spacing was selected to monitor survival and spread of individual plants. The purpose was to develop criteria for planting times and elevations. The cost of building this marsh was $3.00 per foot of shoreline. Locally harvested plants and student labor were used. The marsh has not been fertilized or otherwise maintained since its original planting.

Profiles of the bank and offshore bottom were surveyed in September 1973, September 1978, May 1979, and October 1979, along the five survey lines indicated in Figure 2-73 and along five survey lines in a control area 1,000 feet north of the marsh where no plantings had been made. Figure 2-74 shows the results of these surveys. The May 1979 survey plotted too close to the October 1979 profiles, and therefore, was not included.

(3) Performance. Most of the plantings in the experimental marsh in 1973 survived and multiplied through the ensuing years. With the protection they afforded, volunteer growths of other marsh plants soon appeared among the planted species. In May 1979, randomly located square-meter counts made throughout the area indicated a predominance of certain species by zones roughly paralleling the shoreline (Fig. 2-75). Just offshore in the sedge zone, most of the plantings were crowded out by an invasion of numerous native species. In the midzone, the giant reed dominated all other species. In the outer zone that was subject to deeper inundation and more wave action, the smooth cordgrass was more dominant.

The effect of the experimental marsh in slowing the rate of bank erosion is indicated by the profiles in Fig. 2-74. While the control area shoreline continued to erode at an average rate of 8.8 feet per year, the rate of shoreline recession behind the marsh decreased progressively as the growth of vegetation provided more and more protection. The profiles taken in October 1939 show that bluff erosion behind the marsh has now ceased, while erosion in the control area is continuing unabated. Volume calculations based on the profile data show that some accretion occurred in the marsh area between the May and October surveys in 1979 (Table 2-21).

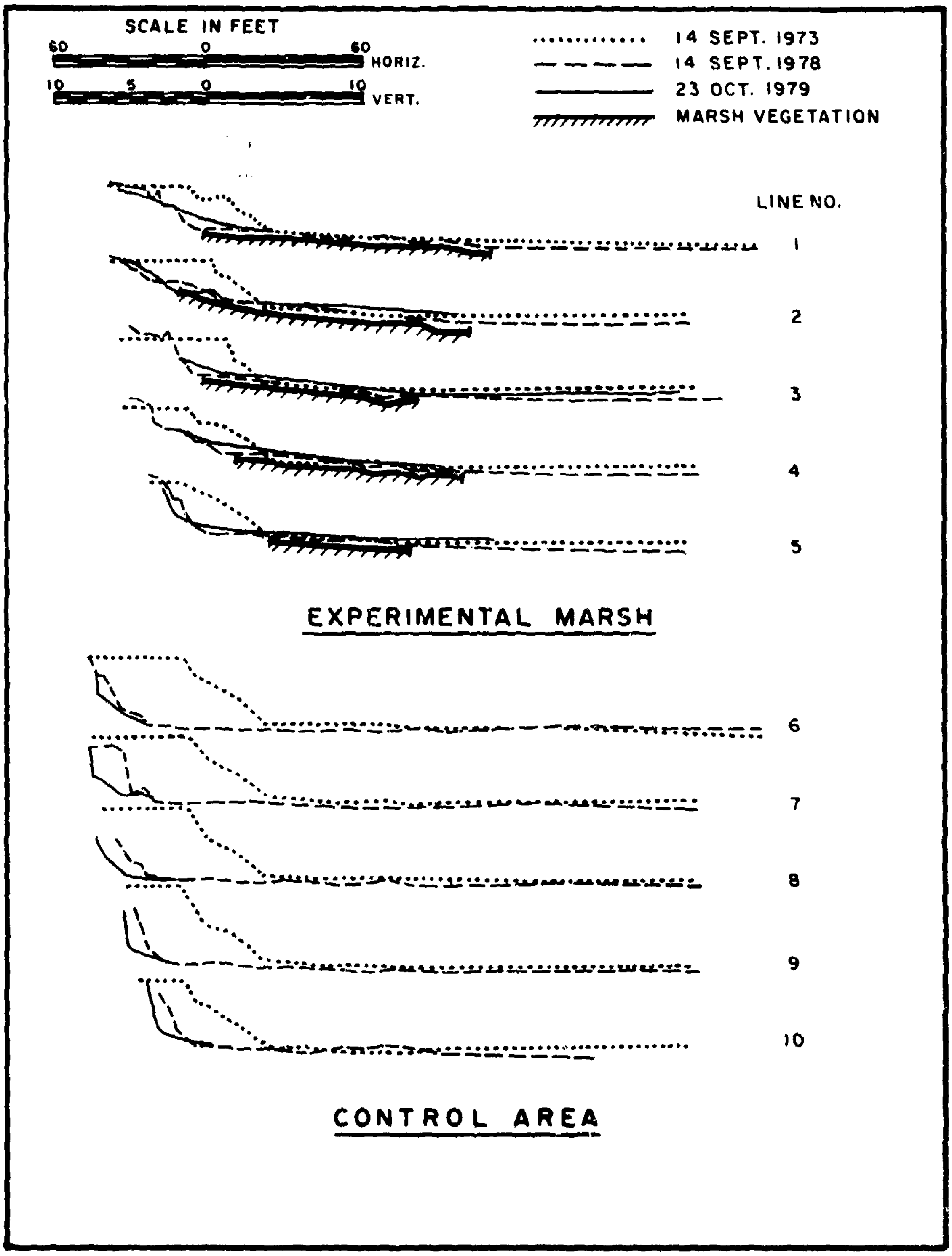


Figure 2-74. Comparative profiles, Duck, North Carolina.

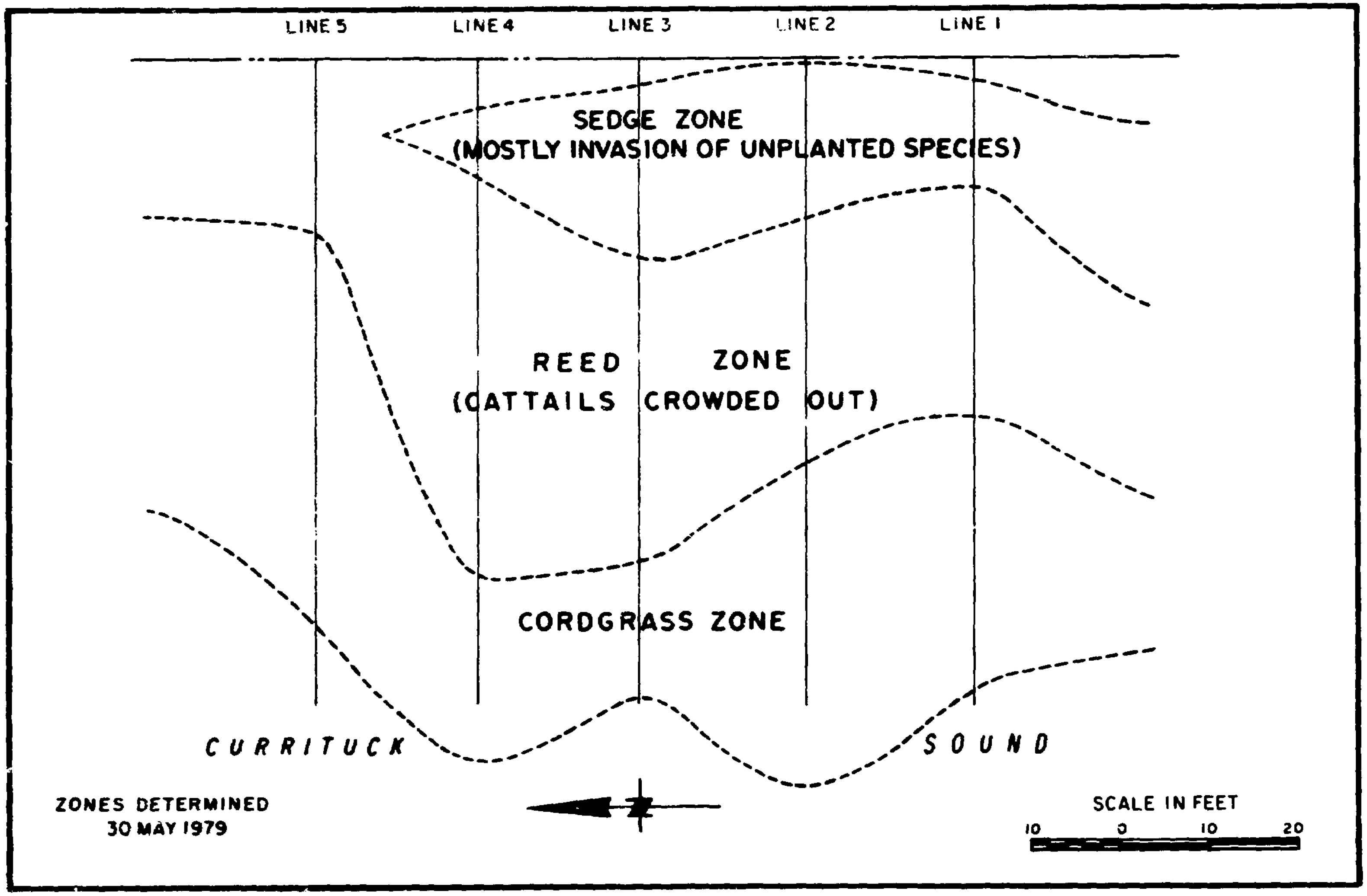


Figure 2-75. Zones of predominant marsh plants, Duck, North Carolina.

Table 2-21. Volume changes between surveys at Duck.

Survey date	Changes in yd^3 per ft of shoreline	
	Experimental marsh	Control area
14 Sept. 73		
	-2.71	-5.85
14 Sept. 78		
	+0.07	-0.40
30 May 79		
	+0.37	-0.88
23 Oct. 79		

The monitor's report for October 1979 stated that the bank behind the marsh appeared stable and that sediment appeared to be trapped in the marsh, although fall dieback of the vegetation had begun. No vegetation had invaded the control area, which was still eroding. The report for April 1980 stated that an unusually large snowstorm in March had buried the marsh under 2 to 3 feet of drifing snow, which melted within 2 weeks. At some time before the snowstorm, the power company had driven trucks through the middle of the marsh to work on the powerlines. The fossil growth from 1979 was broken down, but no other damage was noted.

Figures 2-76 and 2-77 show two views of the experimental marsh as it appeared in June 1979. Figure 2-78, taken during a high water period in April 1980 from near the northeast corner of the marsh, shows the remnants of the 1973 winter plantings in the foreground and the control area in the distance. Figure 2-79 is a closeup of the sedge zone during the clip harvest of October 1979 showing the square-meter frame used to outline the clip area.

(4) Analysis. The active erosion along the east bank of Currituck Sound has been due largely to man's destruction of formerly existing marsh growth along the shoreline. The rapidity of the erosion did not permit natural reestablishment of the marsh growth in the immediate offshore area. Without man's assistance most volunteer growth is soon destroyed by wave action and disturbance of bottom materials. Because protective marshes had formerly existed in this area, it was postulated that the substrate could support new marsh growth if it were given a good start.

Planting of the experimental marsh with species known to take root and become established rapidly in the particular environment of the region was undertaken on the basis of the above postulation. The plantings performed about as expected, and aided by volunteer growths of other species, the marsh has not only halted bank erosion but is now trapping sediment, which will provide added protection as the offshore bottom is built up. Of the four species planted, cordgrass, rush, and reed were effective in stabilizing the shoreline and trapping sediments. These plants are recommended for use in areas of low wave energy and low salinities. However, the giant reed soon crowded out the cattails, which are a more environmentally desirable species. In future experiments, elimination of reed plantings might be

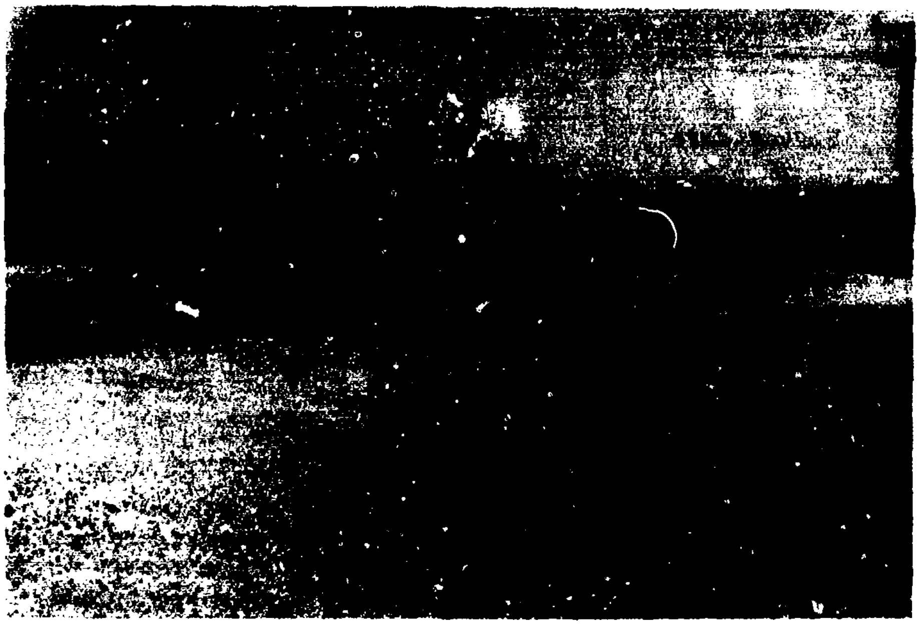


Figure 2-76. CERC experimental marsh from northwest corner, Duck, North Carolina, 20 June 1979.

Figure 2-77. Experimental marsh from power company pole, Duck, North Carolina, 20 June 1979.

Figure 2-78. Remnants of 1973 winter plantings (foreground) and control area (background), Duck, North Carolina, 10 April 1980.

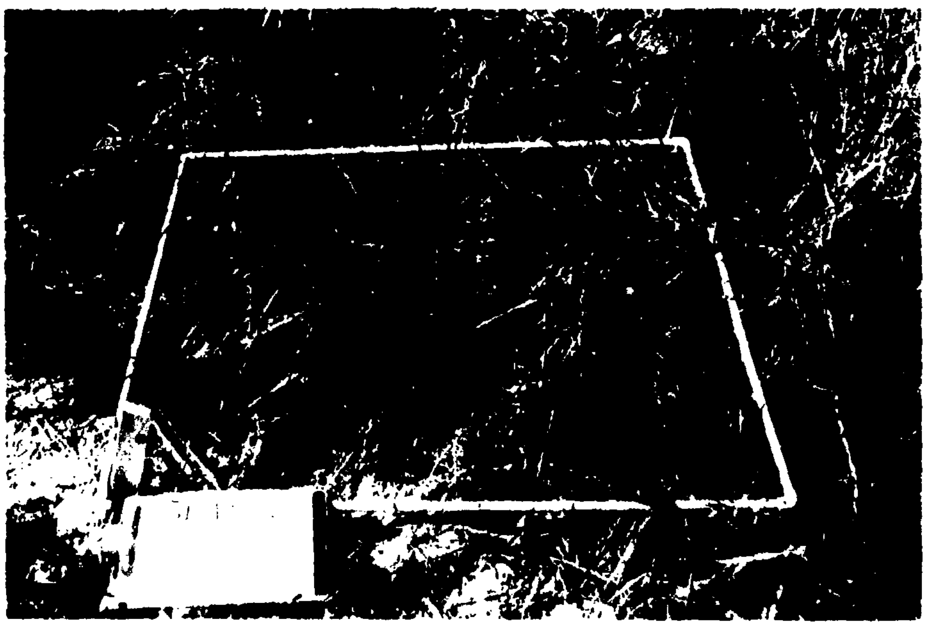


Figure 2-79. Sedge zone, Duck, North Carolina, 24 October 1979.

desirable. At a cost of only $3.00 per linear foot of shore, this experiment has demonstrated a highly successful and economical method of shore protection with vegetation alone that could be used in other areas having similar problems under similar environments and conditions.

g. Bogue Sound, North Carolina.

(1) Site Description.

(a) Geographical Setting. This vegetation monitoring site is located on the south shoreline of Bogue Sound. The site is located in the town of Pine Knoll Shores, a residential development of Bogue Banks, located in Carteret County, North Carolina, about 32 miles southeast of New Bern and 37 miles east of Jacksonville (Fig. 2-80). The shoreline has an east-west orientation. Access to the site is via the Bogue Banks road.

(b) Wind, Waves, Tides, and Longshore Transport. The most prevalent waves affecting the site are caused by storms. Waves are generated by northeast and northwest prevailing winds over fetch lengths of more than 3 miles. The mean tidal level is 1.2 feet (MLW) and MHW is 2.5 feet. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot. The wave data for this site were not considered sufficiently complete for a meaningful energy-flux analysis. However, the wave climate is classified as mild, and longshore transport is not a problem.

(c) The Problem. During development of Pine Knoll Shores, a boat channel was dredged parallel to the shore, and the dredged material was placed on top of the existing marsh fringe to create a sandy beach. The marsh grasses were killed by the fill material, and the beach began to erode. An asbestos sheet-pile bulkhead was then installed to protect the waterfront lots, but continuing erosion threatened to undermine the structure.

(2) Monitoring Project. In April 1974, smooth cordgrass sprigs were planted in the eroding beach in an effort to protect the bulkhead. Ten sections were planted within the experimental area (Fig. 2-81); the specifics for each section are given in the Initial Post Planting Inventory (Table 2-22). Fig. 2-82 shows the beach and bulkhead at the time of the plantings.

(3) Performance. Survival of transplants was moderate to poor in the most open sections, 1, 2, 3, 8, and 9, and good in the more protected areas, sections 4 to 7 and 10, at the end of the first growing season in October 1974. Forty-five percent of the original plantings were still alive in section 1, but only 26 percent in sections 2 and 3 at the end of the first season, and only 39 percent in sections 8 and 9. By contrast in sections 4 to 7 there was 68-percent survival of plantings and 76-percent survival in section 10. From data on stem counts shown in Table 2-23, plantings on 18-inch centers were most successful by the end of the first growing season.

This response changed in subsequent years, with increased growth in number of stems per square meter observed in the 24- and 36-inch centered plantings as shown in Table 2-24.

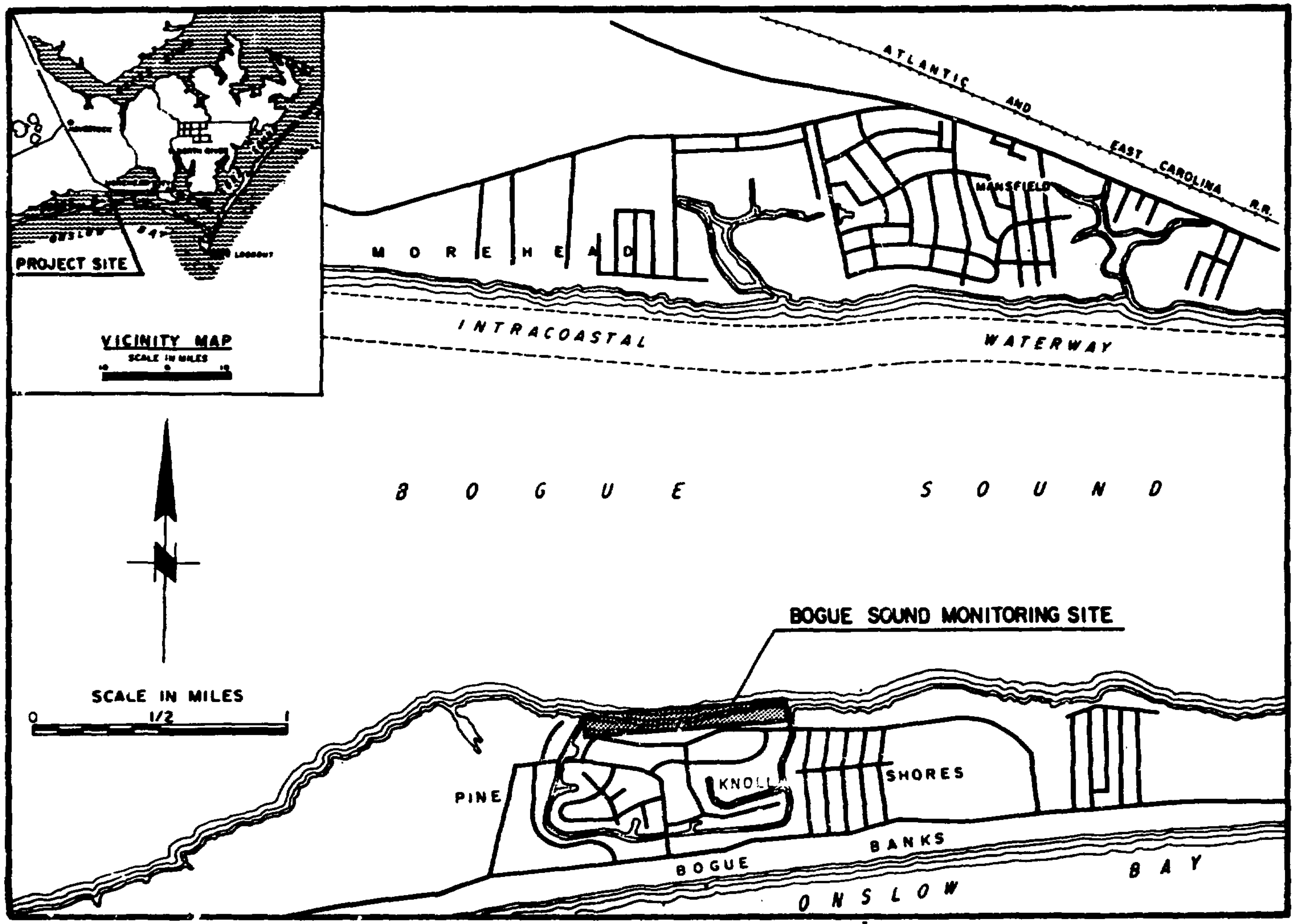


Figure 2-80. Location map of Bogue Sound, North Carolina, monitoring site.

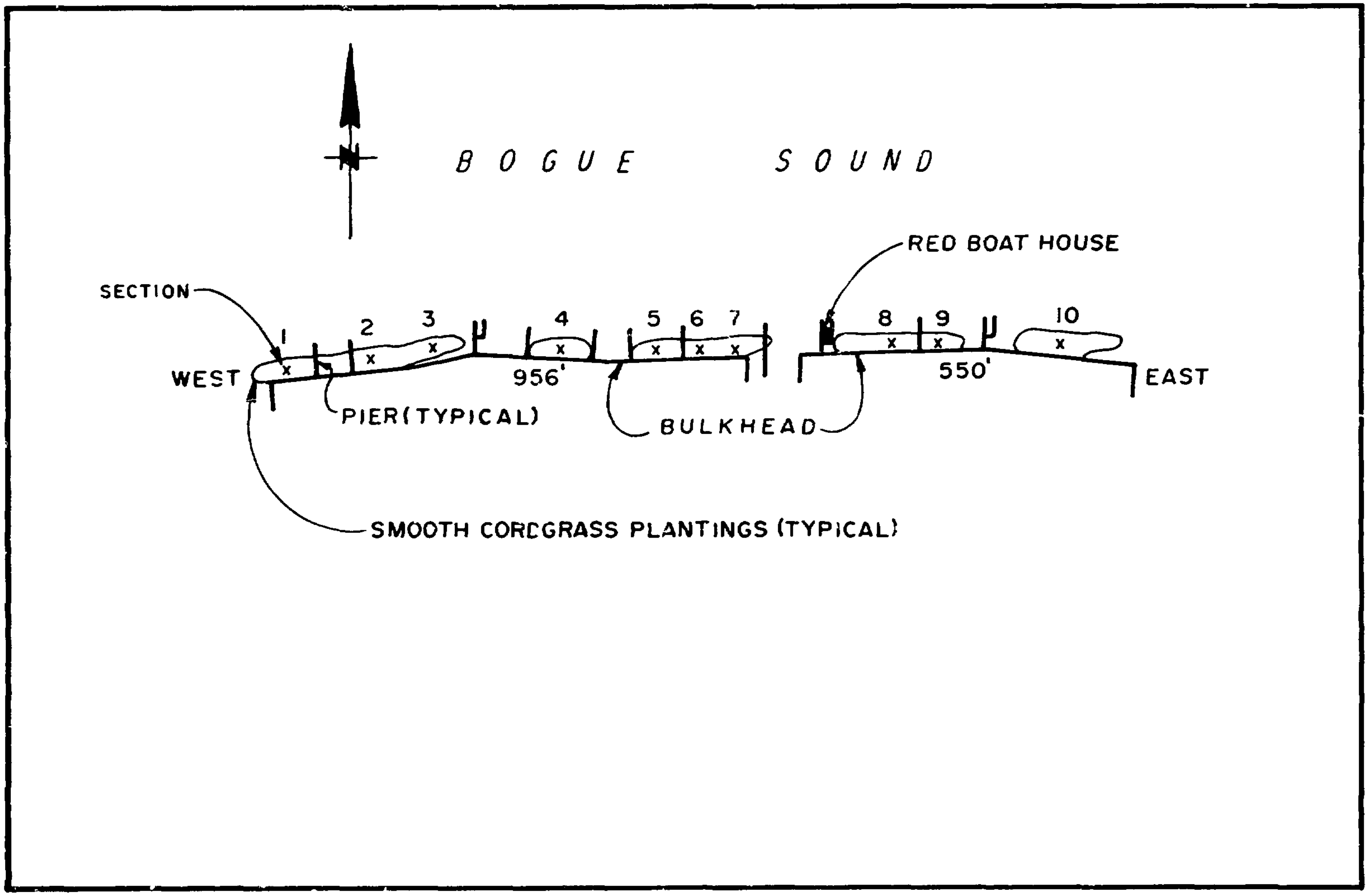


Figure 2-81. Vegetation plan at Bogue Sound, North Carolina.

Table 2-22. Initial Post Planting Inventory of smooth cordgrass at Bogue Sound, 18 April 1974.

Section	No. of hills or rows	Spacing	Total no. of plants	Planting method	Fertilization
1	27 rows 25 hills/row	24 in within rows; rows 36 in apart	675	Machine	None
2,3	54 rows 25 hills/row	24 in within rows; rows 36 in apart	1125	Machine	None
4,5,6,7	2667 hills 1500 hills 667 hills	18 in 24 in 36 in	4834	Machine and hand	None
8,9	36 rows 25 hills/row	24 in within rows; 36-in rows	900	Machine	None
10	36 rows 16 hills/row	24 in within rows; 36-in rows	600	Machine	Experimental variables

Note: The 10 sample plots were spread along 1,500 feet of shoreline where several experiments were transplanted. Experimental variables included plant spacing, source of plants, effect of fertilizer at transplanting, and a comparison of greenhouses and field-grown transplants. At the time of sampling in 1979, the original treatments did not affect the results. (For details, see Woodhouse, Seneca, and Broome, 1976).

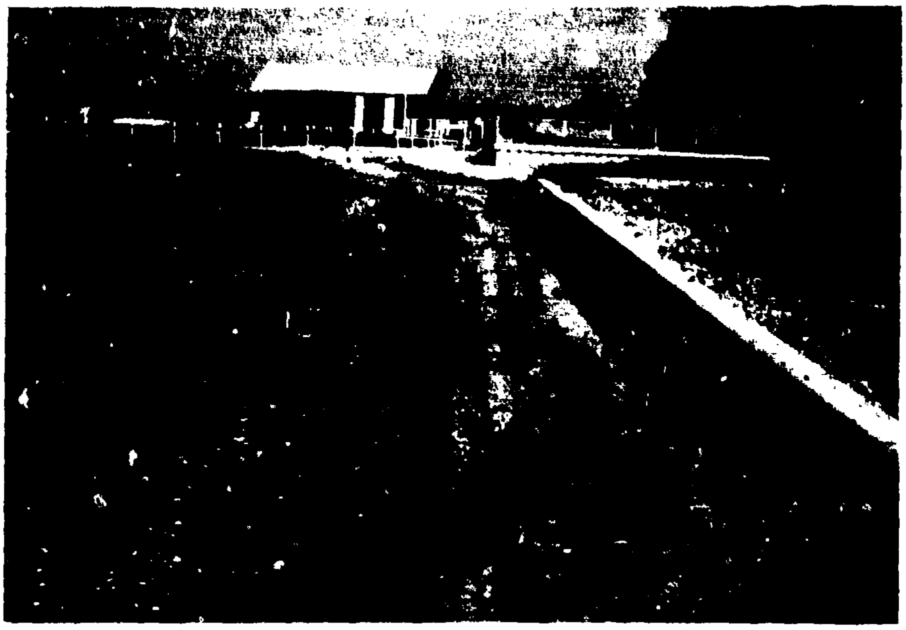


Figure 2-82. Transplanting smooth cordgrass sprigs, Bogue Sound, North Carolina, 11 April 1974.

Table 2-23. Stem counts and dry weights of smooth cordgrass at Bogue Sound after one growing seaston (1974).

Spacing of transplants (ft)	Stems/m^2	Dry wt (g/m^2)
1.5	113	211
2.0	68	126
3.0	44	61

Note.--Each count is the mean of five samples.

Table 2-24. Number of stems of smooth cordgrass at the end of each growing season since 11 April 1974.

Year	Number of stems (stems/m^2)[1]		
	At 1.5- by 1.5-foot spacing	At 2- by 2-foot spacing	At 3- by 3-foot spacing
1974	113	68	44
1975	560	300	348
1976	361	336	380
1977	394	344	421
1978	544	558	710
1979	742	751	850

[1]Each count is the mean of five samples.

Regardless of the spacing, a complete cover had developed on the site by the 1976 growing season. Figure 2-83 shows the site as it appeared in June 1979. Some sand was being trapped at this time, and the marsh was expanding into previously unplanted areas.

Figure 2-83. Good cover which had been established by the end of the 1976 growing season, Bogue Sound, North Carolina, 15 June 1979.

At the time of sampling in May 1979, growth was good (with one exception) and no treatment effects remained from earlier experimental manipulations (see Woodhouse, Seneca, and Broome, 1976,for details). One area along the bulkhead which had originally been planted using 3-foot spacings did not become established after the 1974 plantings, and in this section the bulkhead is still in danger of being undermined.

(4) Anaylsis. The smooth cordgrass planting effectively halted erosion in all but a small area where plantings were too widely spaced and were subsequently washed out. The plants bordering nonplanted areas have trapped sand and have stabilized the planted area and its borders. Planting density affected initial survival of sprigs, with more closely placed sprigs surviving best. However, after two growing seasons this effect was lost and growth was best in sections where sprig plantings were more widely spaced. This may have been due to increased production of rhizomes and the more sheltered location of these plantings. In any case, a full stand of smooth cordgrass was successfully established in all but one small area within 3 years after planting. In the area that was not vegetated, erosion is occurring.

The experimental cordgrass planting at the Bogue Sound site was unique in that the vegetation was used to protect an endangered bulkhead. At all other sites where structures and vegetation were used in combination, the structures were built seaward of the vegetation so as to reduce wave heights in the planted areas. Obviously, the Bogue Sound procedure is applicable only in very mild wave climates where the substrate, tidal regime, and weather conditions are conducive to rapid establishment of the vegetation plantings.

h. Uncle Henry's Fish Camp, North Carolina.

(1) Site Description.

(a) Geographical Setting. This monitoring site is about 6 miles southeast of Wilmington on the Atlantic Intracoastal Waterway just north of the mouth of Whiskey Creek. It is accessible by taking Masonboro Loop road south to Uncle Henry's Fish Camp. The site shoreline is approximately 250 feet long and trends S. 45°W. The Masonboro Boat Yard is on the southwest shore of Whiskey Creek, opposite the fish camp (Fig. 2-84).

(b) Wind, Waves, and Tides. The site is exposed only to boat wake waves from traffic on the Atlantic Intracoastal Waterway and from pleasure boats using the Masonboro boat Yard. The site is sheltered from Atlantic Ocean waves by Masonboro Sound and its outer bank. The mean tide level is +1.9 feet MLW and MHW is +3.8 feet. No LEO station was established at this site.

(c) The Problem. Erosion began occurring along the shoreline after dredging of the Intercoastal Waterway. Increased boat traffic and wake waves are currently the main reason for erosion. The boat channel is within 10 feet of the MLW line at the site.

(2) Monitoring Project. Smooth cordgrass was planted 2 June 1976 along the Intracoastal Waterway in a barren area between existing marsh growths (Fig. 2-85). Sprigs were used, and were planted by machine (Fig. 2-86). All plantings were spaced 18 inches apart along rows spaced 24 inches apart. A total of 2,310 sprigs was planted. Seven weeks after planting, the site appeared as shown in Figure 2-87. The entire planting was 30 feet wide and 250 feet long. In a section about 100 feet long at the southwest end of the planting, survival was very poor (Fig. 2-88). This was apparently due to heavy foot traffic which broke the tops off many plants or caused them to become uprooted. By 1978, the unvegetated section had been reduced to a length of about 85 feet, which was replanted by machine 5 May 1978. Spacing was 12 inches apart. Again, survival was poor, and this area was replanted by hand 21 April 1979 with 576 plants spaced 24 inches apart in both directions. This was a fertilizer experiment, with rates and types of fertilizer materials as variables. Most of this planting also did not survive the heavy foot traffic. Variable fertilization was applied as follows:

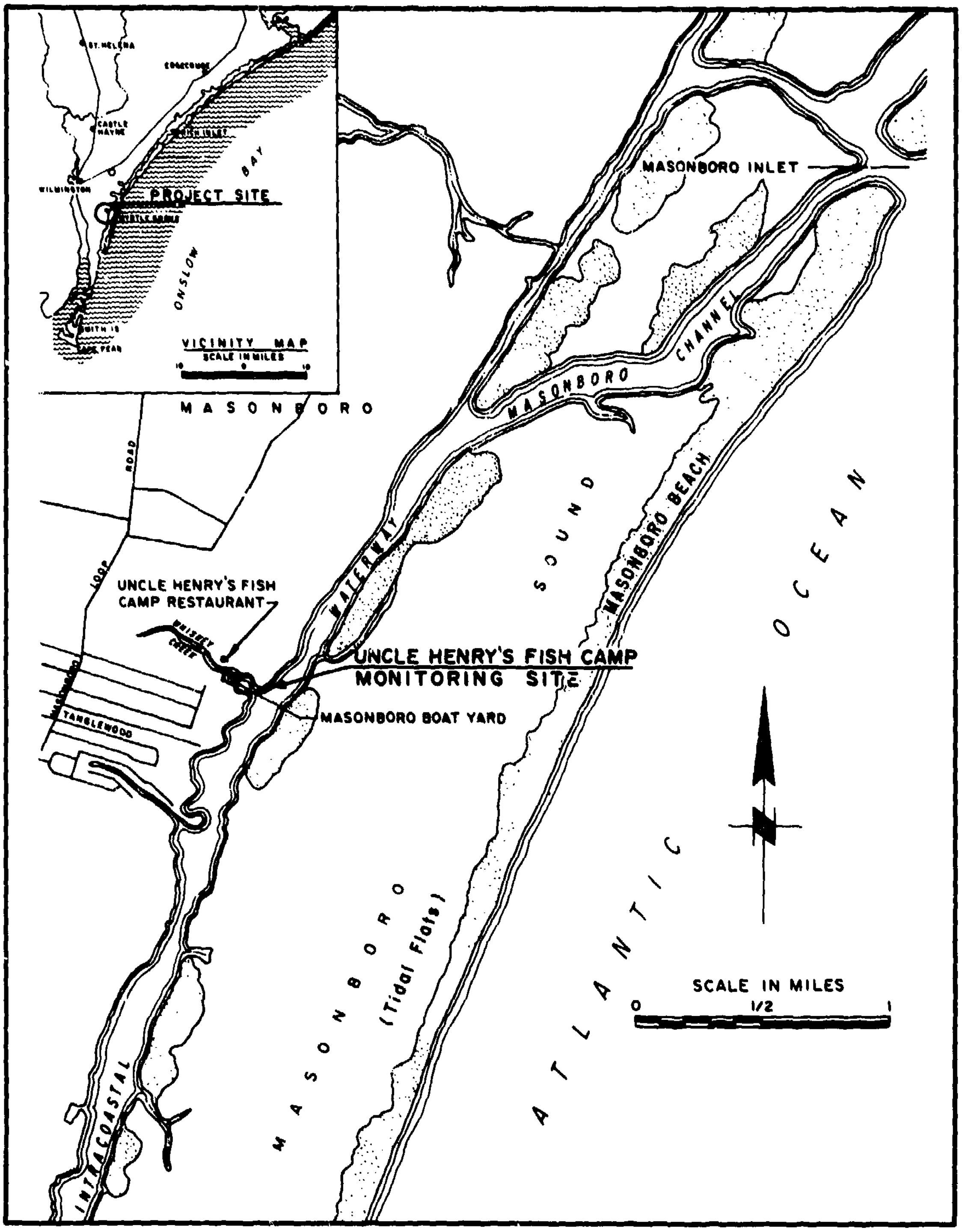


Figure 2-84. Location map of Uncle Henry's Fish Camp, North Carolina, monitoring site.

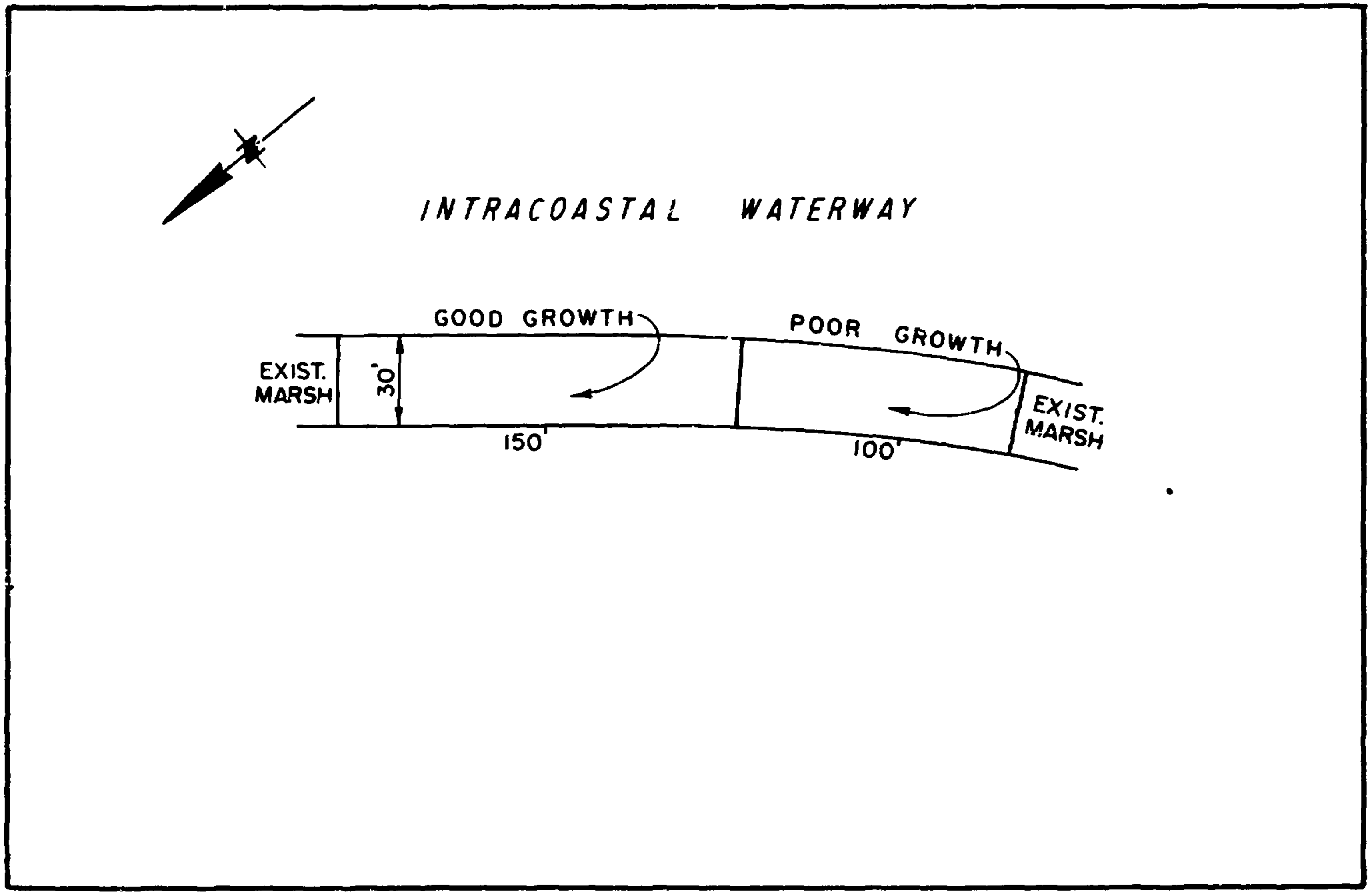


Figure 2-85. Plantings of smooth cordgrass at Uncle Henry's Fish Camp.

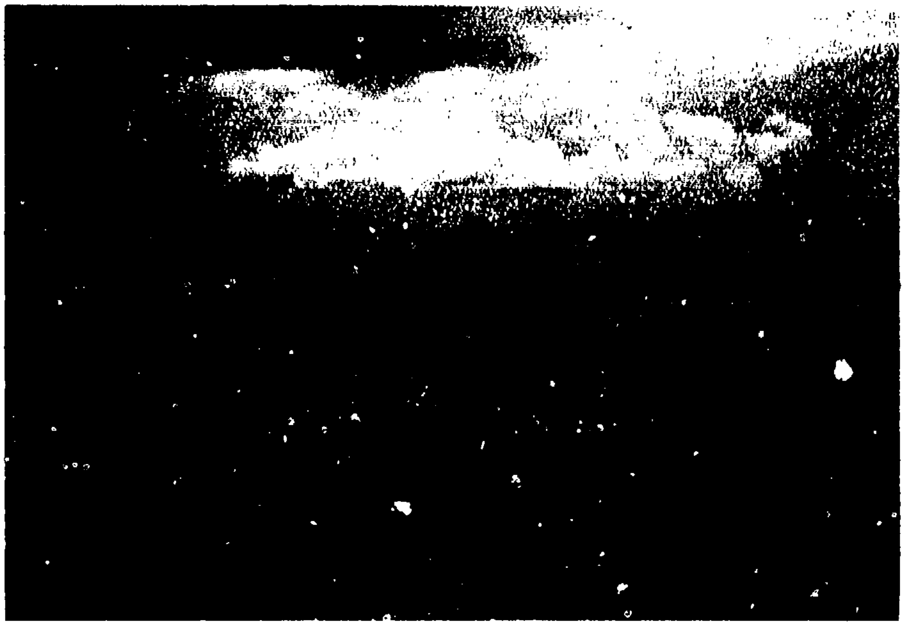


Figure 2-86. Transplanting the smooth cordgrass using a tractor, Uncle Henry's Fish Camp, North Carolina, 2 June 1976.

Figure 2-87. View of cordgrass 7 weeks after transplanting, Uncle Henry's Fish Camp, North Carolina, 22 July 1976.

Figure 2-88. Southwest end of site where growth was very poor, Uncle Henry's Fish Camp, North Carolina, 12 June 1979.

Nitrogen was applied at the rate of 50 pounds per acre on 13 July 1977 and 10 August 1977 using ammonium sulfate as the source of nitrogen. Nitrogen was applied at the rate of 100 pounds per acre (ammonium sulfate) and phosphorus at the rate of 100 pounds per acre P_2O_5 (source was concentrated superphosphate) on 19 July 1979.
Nitrogen was applied at the rate of 100 pounds per acre (ammonium sulfate) on 17 August 1979.

(3) Performance. Survival counts were not taken at the end of the first growing season due to the problems encountered in achieving a successful planting of the area. The number of stems per square meter and dry weight were recorded in October 1976 (the end of the growing season). These data indicate a moderate rate of establishment at the northeastern end where the least damage occurred, and virtually no growth at the southeast end where destruction of the plantings continued throughout the growing season.

After the first growing season, growth increased steadily at the northeast end of the planting area (Table 2-25). Because of the poor survival on the southeast end, no further measurements were taken in this section.

Even at the northeastern end where plants were allowed to grow undisturbed, growth has been fair to moderate during the 3 years since planting. For example, at Bogue Sound not far away, a complete cover of

Table 2-25. Dry weight and number of stems of smooth cordgrass at the end of each growing season since 2 June 1976, Uncle Henry's Fish Camp.

Year	Stems/ m^2	Dry wt (g/m^2)
1976	39	27
1977	179	341
1978	280	234
1979	417	290

Note.--Counts are a mean of five samples from northeast end.

smooth cordgrass had been achieved after 3 years with an average value of 360 stems per square meter over all plots. It is evident from comparison with the above data that growth was much slower at the Uncle Henry's site even after four growing seasons (Fig. 2-89). In some areas of the northeast section, growth has been good (Fig. 2-90). In this area, sand has accumulated. However, in some cases this accumulation has been excessive and has damaged the plantings.

(4) Analysis. Where a good stand of smooth cordgrass has developed some sediment has accumulated and the area appears stable. However, growth has not been vigorous at this site and damage has occurred as a result of foot traffic, boat-wake waves, and excessive sand buildup. In addition, the salinity of the water (35 ppt) and low nutrient supply available in the soil may prevent vigorous establishment of plantings. The failure of the southwest section plantings to become established after three trys and fertilization experimentation indicates that conditions there are too severe for shore stabilization with vegetation alone. It may be necessary to install a submerged sill in that area and to fence it off from foot traffic. The successful establishment of a cordgrass marsh at the northeast end of the site indicates that vegetation might be successful at the southwest end if conditions can be improved.

i. Folly Beach, South Carolina.

(1) Site Description.

(a) Geographical Setting. The Folly Beach monitoring sites are on the Atlantic coast of Folly Island in Charleston County about 10 miles south of Charleston. Folly Island is typical of most barrier islands along the south Atlantic coast, and its alinement is roughly northeast-southwest. Bounded on the northeast end by Lighthouse Inlet, on the east by the Atlantic Ocean, on the southwest end by Stono Inlet, and on the northwest side by Folly River, the island is approximately 6 miles long and at the widest part is about 0.5 mile across. Access to the island from Charleston is by State Highway 171 (Fig. 2-91).

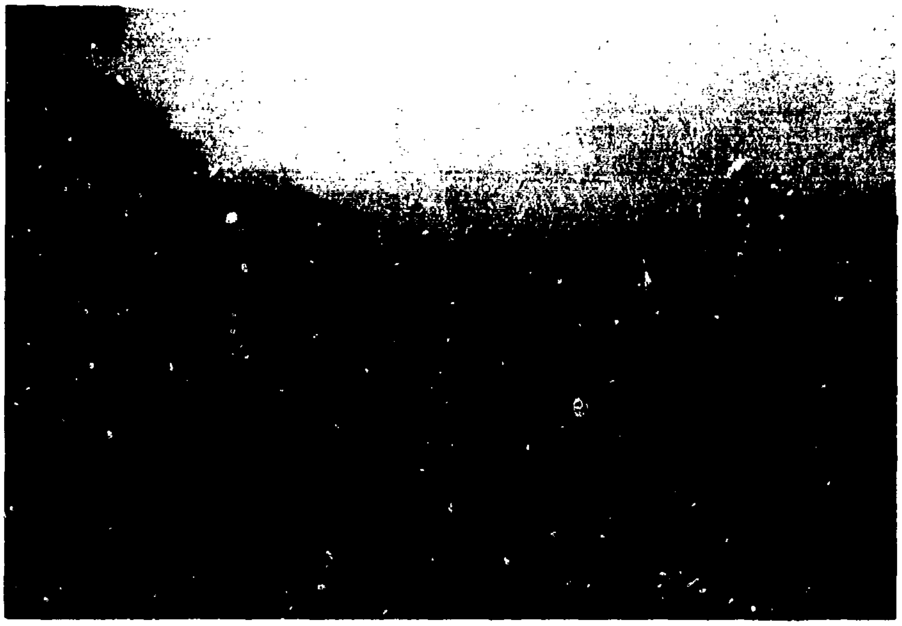


Figure 2-89. Overall moderate growth, Uncle Henry's Fish Camp, North Carolina, 17 September 1979.

Figure 2-90. Good growth at northeast end of planting, Uncle Henry's Fish Camp, North Carolina, 17 September 1979.

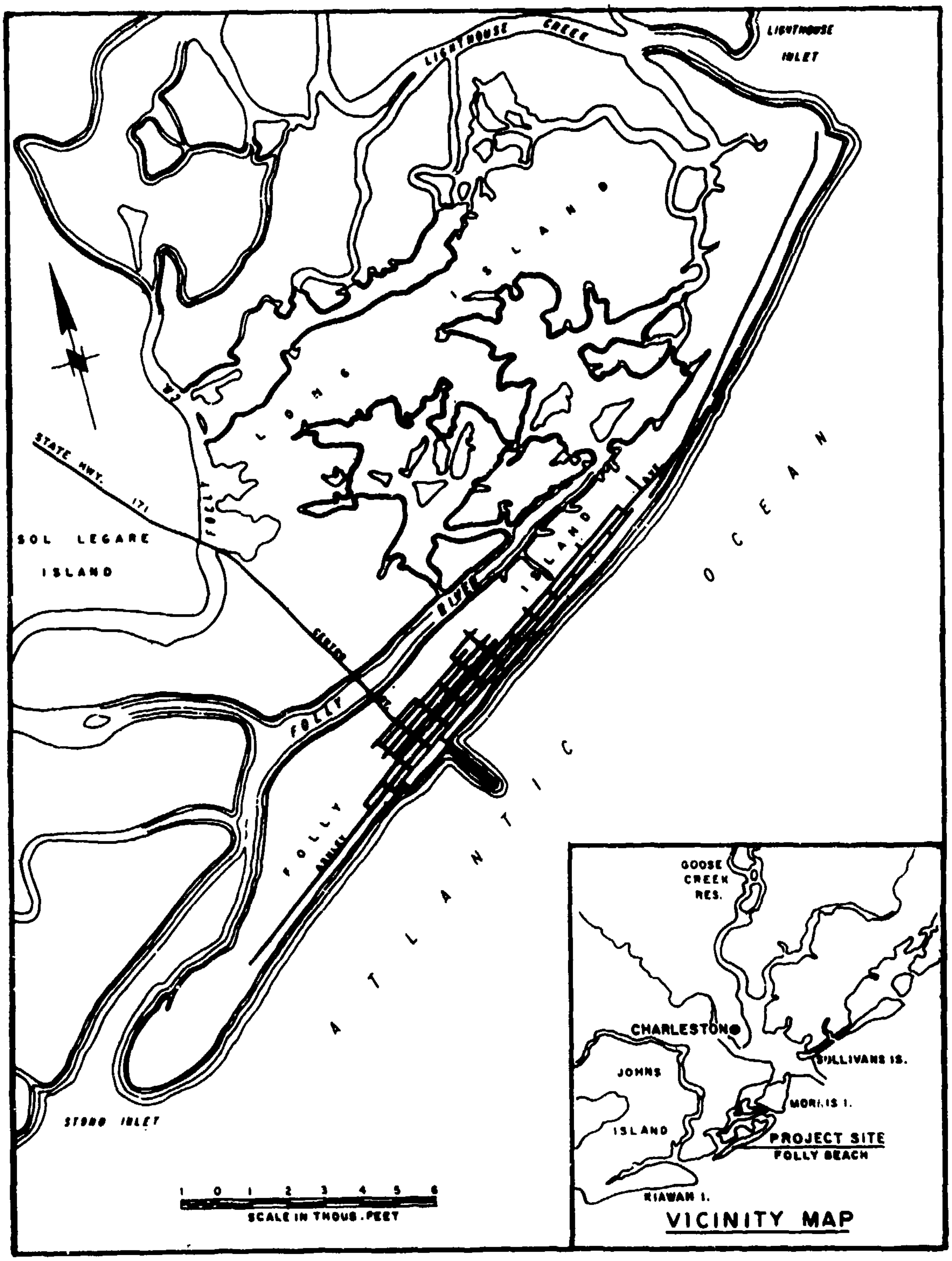


Figure 2-91. Location map of Folly Beach, South Carolina, monitoring site.

(b) Geomorphology, Soils, and Vegetation. The geological formations of this area are comprised of layers of unconsolidated sands and gravel underlain by nearly horizontal layers of loams, clays, and marls of different ages. Typical soil borings have produced fine silty sand to a depth of about 20 feet MLW. Silt contents in the samples increase with the distance of the sampling sites from the ocean. Folly Beach is a flat strand consisting of fine clear sand with a high shell content. An ecologically productive salt marsh area adjoins the Folly River along the northern half of the island. Where not displaced by development, a mixture of palmetto, pine, and deciduous growth covers the uplands behind the beach.

(c) Climate. The climate of the barrier islands is marine subtropical. The mean average annual temperature near Folly Island is 66° Fahrenheit with an average high temperature in July of 81° Fahrenheit, and an average low of 49° Fahrenheit in February. Relative humidity is high, about 75 percent, with the discomforting effects moderated by an afternoon sea breeze. Precipitation is usually rainfall which is fairly well distributed throughout the seasons and averages 50 inches per year. The high percentage of fair-weather conditions in this area promotes a relatively long growing season, nearly 300 days per year.

(d) Waves, Tides, and Longshore Transport. Wind-generated waves of varying magnitude and direction occur at Folly Beach. The coastline is exposed to onshore and alongshore winds from northeast to east, southeast, and south to southwest. Winds from the northeast to east to southeast move over practically unlimited fetches of the Atlantic Ocean. Near Stono Inlet, fetches to the southwest are limited by Kiawah Island, but elsewhere fetches to the south are extensive. The wind data indicate that the predominant winds are from the northeast quadrant. The resulting wind-driven waves break offshore and re-form as they are propagated over the shallow Continental Shelf. Tides are semidiurnal with a mean range of 5.2 feet and a spring range of 6.1 feet. The direction and rate of littoral transport depend primarily on the direction and energy of waves approaching the shore, except on shores adjoining tidal inlets where tidal currents may be dominant. The LEO data (Table 1-3) indicate that wave heights average from 1 to 2 feet, with a maximum of 5.5 feet, and net longshore transport potential southwestward was 302,500 cubic yards for the 11 months analyzed. The wave climate is classified as severe.

(e) The Problem. Since 1849, approximately 560 acres of beach front has been lost from Folly Island. This is equivalent to an average annual erosion rate of 5.9 feet over the entire length of the ocean shoreline. The erosion has not been uniform, varying greatly with both location and time. Currently, the southernmost beach is experiencing net erosion at a maximum rate of 14,000 cubic yards per year. Nearly half of this amount is deposited on northward beaches up to the bend in the island shore just south of Center Street. Net erosion begins again from that point to the U.S. Coast Guard Station with a maximum rate of 20,000 cubic yards per year. According to local residents, about 25 feet of the project beach has eroded in the past 5 years, despite the numerous shore protection devices installed to solve this problem.

(2) Monitoring Project. The South Carolina Highway Department has constructed and is maintaining 41 timber and rock groins along the developed coastline of Folly Beach from the Loran Station to the northeast to within about 4,000 feet of the southwest end of the island (Fig. 2-92). Beach-front property owners have also constructed many different shore protection structures, including concrete sheet pile, asbestos corrugated sheet pile, timber seawalls, rock revetment, rubber tire walls, sand fencing, and Sandgrabbers. The devices being monitored at Folly Beach comprise a number of groins, revetments, and bulkheads that were selected as being representative of the many that have been installed. These devices are grouped by the specific area locations in south to north sequence. Monitoring began in March 1979.

(a) Area 1.

1 Description. One groin (device 1) and 530 feet of riprap revetment (device 2) were monitored in this area. The completion date of the groin is unknown. The revetment was built in two stages; the second stage was completed in February 1979. The groin is of timber sheet-pile construction, 227 feet long, stabilized along the outer half with 200- to 500-pound stone. The revetment is a two-stone-thick layer of 100- to 400-pound armor stone laid directly on Polyfilter-X filter cloth. The revetment extends from MLW to +10 feet on a 1 on 2 slope. The location and configuration of the area 1 devices are shown in Figure 2-93.

2 Performance. No change in structural alinement or cross section occurred from March to July 1979. In July 1979, wave action during high tide displaced many rocks along the revetment, exposing the filter material (Fig. 2-94) at three locations. This deterioration continued through August, but was not as severe. In early September 1979, Hurricane David destroyed the entire beach at device 2, leveling the revetment and dispersing rocks across Arctic Avenue (Fig. 2-95); the timber groin remained intact.

By the end of October the revetment had been reconstructed. Stones were gathered, restacked, and realined. In many locations the crest height of the repaired structure was somewhat lower than the original construction (Figs. 2-96 and 2-97). The beach material was placed behind the revetment. The reconstructed section averaged about 5 feet wide and 4 to 6 feet high (Fig. 2-98). This was the condition of the site in December 1979. The device 2 riprap section parallel to Arctic Avenue (Fig. 2-99) and a small part of the section behind the buildings have been replaced.

Although the revetment slowly deteriorated in the first 3 months of monitoring, it still protected the shoreline and Arctic Avenue from severe erosion. However, an unprotected 70-foot gap between device 1 and the south end of the revetment was severely eroded (Fig. 2-100). The protected segment of bank held until September, when it was destroyed by Hurricane David (Fig. 2-101). As of May 1980, the revetment and groin were still intact.

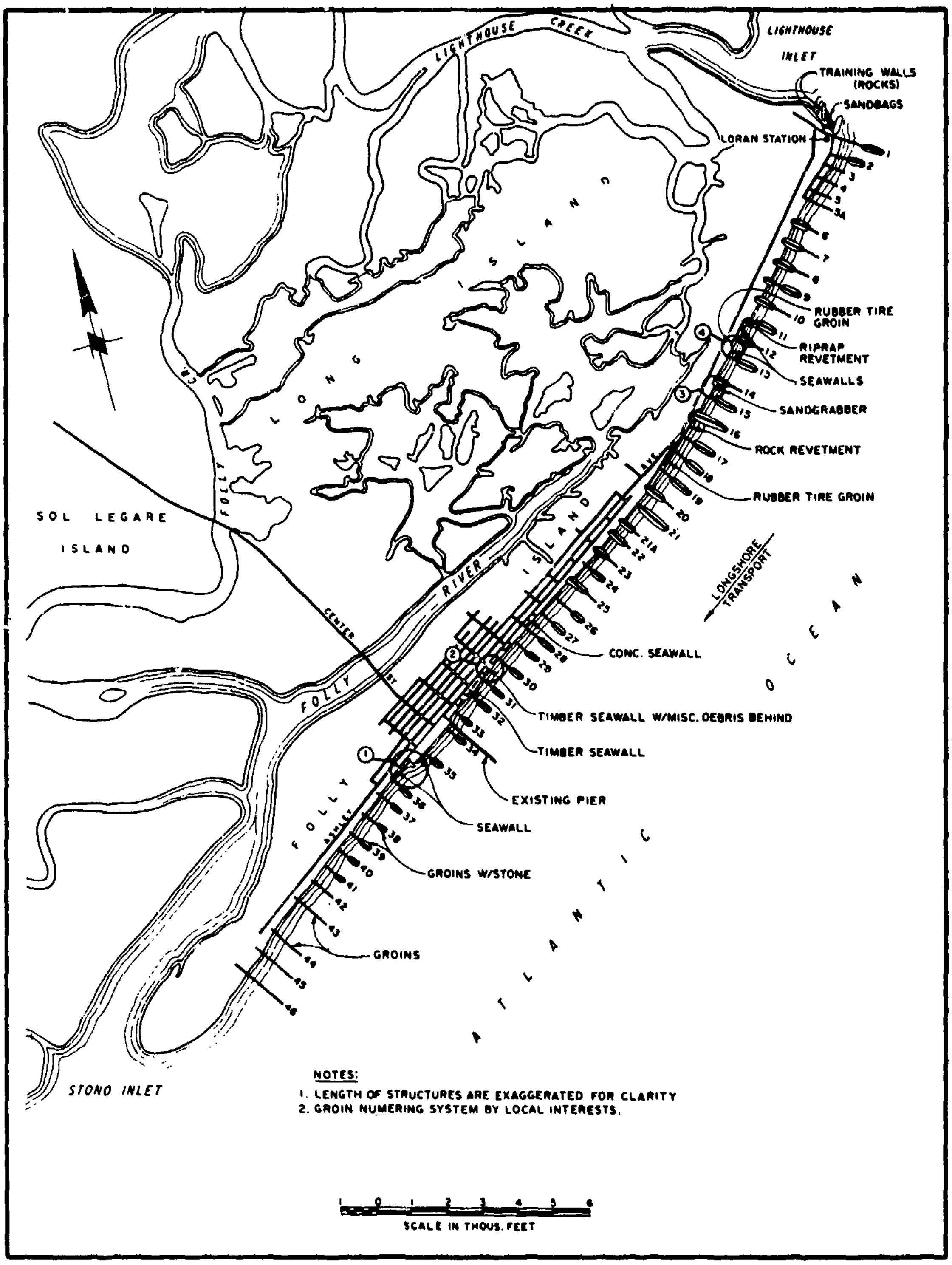


Figure 2-92. General plan of Folly Beach, South Carolina, monitoring site.

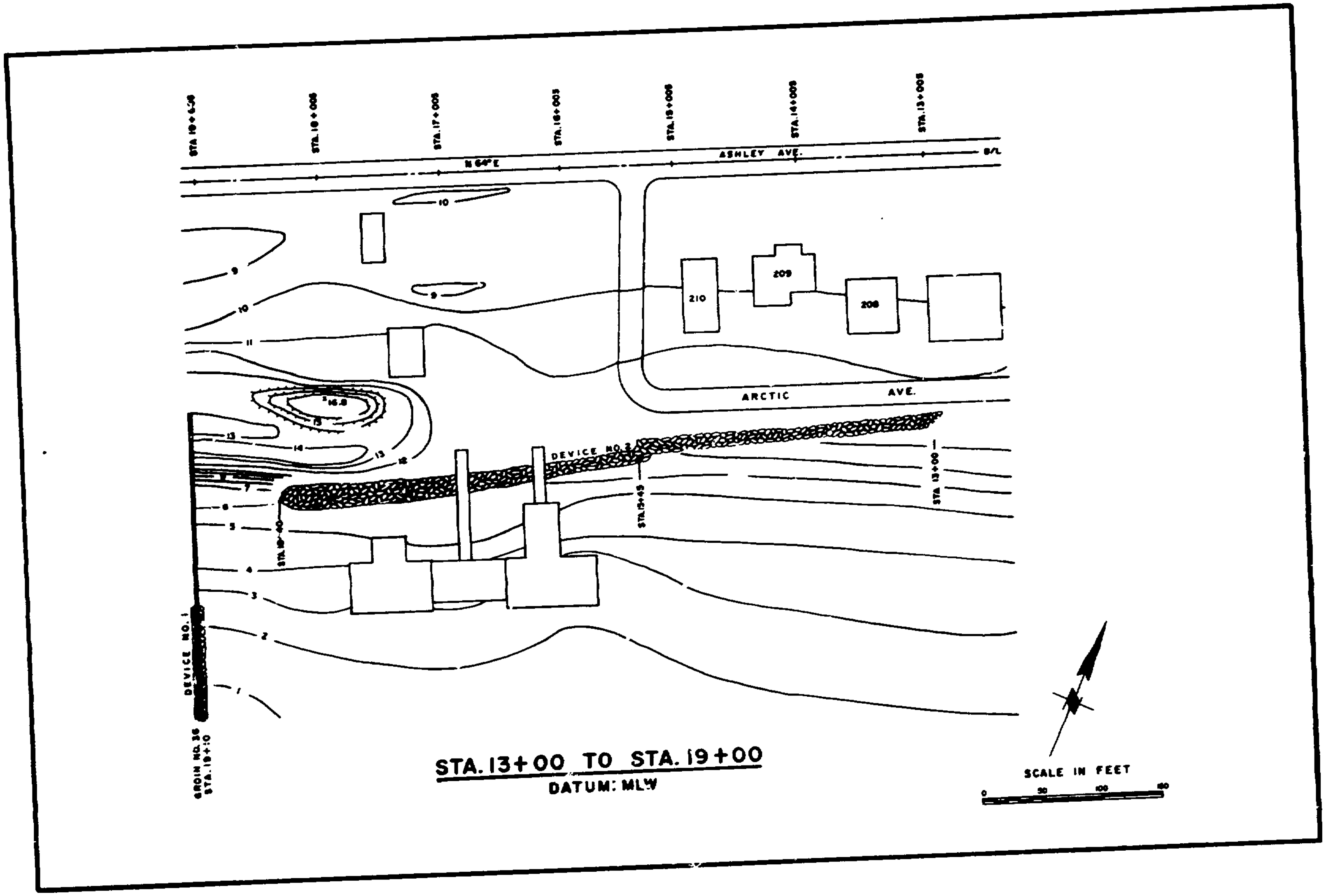


Figure 2-93. Folly Beach area 1.

Figure 2-94. Displaced rock exposing filter material, device 2, Folly Beach, South Carolina, 16 July 1979.

Figure 2-95. Rocks dispersed across Arctic Avenue, Folly Beach, South Carolina, 5 September 1979.

Figure 2-96. Device 2 riprap revetment before storm, Folly Beach, South Carolina, 27 June 1979.

Figure 2-97. Repaired revetment section after storm, Folly Beach, South Carolina, 30 October 1979.

Figure 2-98. Beach fill added at device 2 after Hurricane David, Folly Beach, South Carolina, 28 December 1979.

Figure 2-99. Replaced riprap in area 1 along a small section parallel to Arctic Avenue, Folly Beach, South Carolina, 25 January 1980.

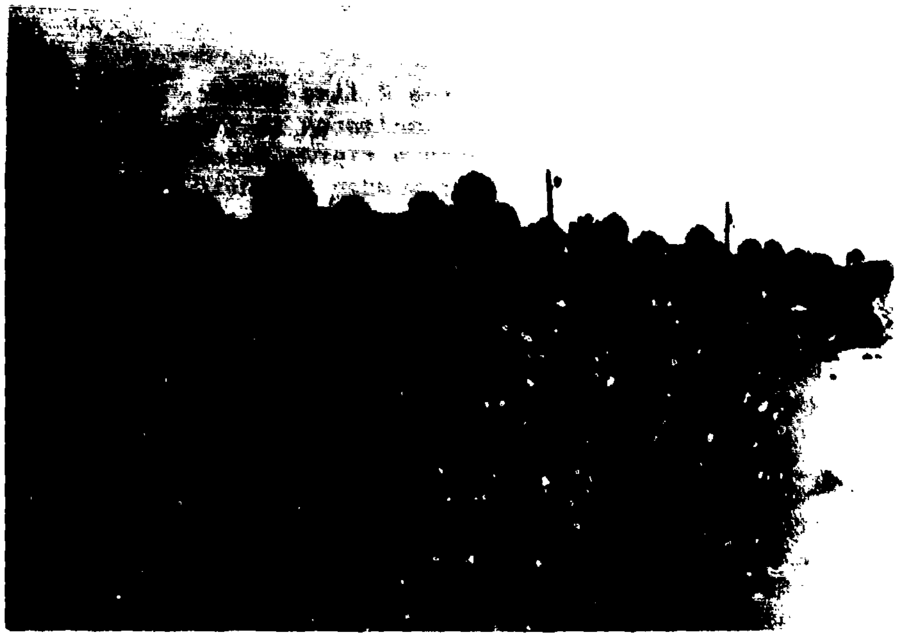


Figure 2-100. Erosion of unprotected berm between devices 1 and 2, Folly Beach, South Carolina, 16 July 1979.

Figure 2-101. Erosion near Arctic Avenue, Folly Beach, South Carolina, 5 September 1979.

(b) Area 2.

1 Description. Area 2 lies between stations 14+10 and 26+70. Five contiguous devices were monitored in this area. Device 3 is a 60-foot timber-pile bulkhead to which a riprap revetment was added in early August 1979. Device 4 is a 75-foot wooden bulkhead supported by timber piles. Device 5 is a 68-foot riprap revetment constructed of 10- to 12-inch stones. Device 6, spanning the remainder of the monitoring site, is a 400-foot concrete bulkhead. Because waves break on this bulkhead at high tide, causing sand to wash out from behind it in one reach, approximately 65 feet of the concrete structure are reinforced with riprap (device 7). Groin 30 forms the northeast boundary of area 2. Figure 2-102 shows the layout of the devices.

2 Performance. Monitoring began in February 1979. No changes in the structural section occurred until riprap was added to the northeast corner of the concrete bulkhead in July 1979. Not related to the fill, cracks developed in the bulkhead as it began parting at the seams. Sandfill and riprap stone were added to the northeast corner of the structure in July. Wave action during August displaced stones from device 7, and in September 1979, Hurricane David destroyed all riprap structures except the lower section along the concrete bulkhead (Fig. 2-[illegible]). The revetments were knocked down, displacing the stones along the [illegible]nd in front of the houses (Fig. 2-104). The concrete bulkhead survived the storm. The only damage was to a small seaward-leaning section near station 24+00. The northeast corner of the wall was completely separated at its seam and a large piece of concrete had broken off exposing the steel reinforcement (Fig. 2-105). The fill and rock dumped at the end of the concrete bulkhead were also lost.

In October 1979 two concrete masonry block walls were constructed perpendicular to the concrete seawall on either side of house 310. Between these walls and in front of house 309, sandfill was deposited to replace the material lost during the storm (Fig. 2-106). In November and December, repairs continued as the riprap from the revetments was replaced. The groins were not damaged during the monitoring period.

The shoreline in area 2 was protected from the waves and tides the first 3 months of study. During Hurricane David severe bank erosion occurred behind the revetments and bulkheads. The bank was eroded by waves breaking through the revetments and overtopping the bulkhead (Figs. 2-104 to 2-110). The last 150 feet of the bank on the north end of area 2 has no protective structures (Fig. 2-111). Figure 2-112 shows the erosion that resulted from the storm.

In December 1979 the rock revetments (devices 3 and 5) were rebuilt to higher levels in front of houses 306 and 308. In January 1980 sandfill was added on top of and behind device 3 (Fig. 2-113), and the rocks on device 5 were displaced by waves, exposing the filter cloth (Fig. 2-114). By March 1980 major structural damage had occurred to house 306 behind device 3. It is not known whether wave action damaged the house; however, adjacent houses were not damaged. By May 1980, device 3 was completely destroyed (Fig. 2-115) and more riprap was placed on device 5. As of May 1980

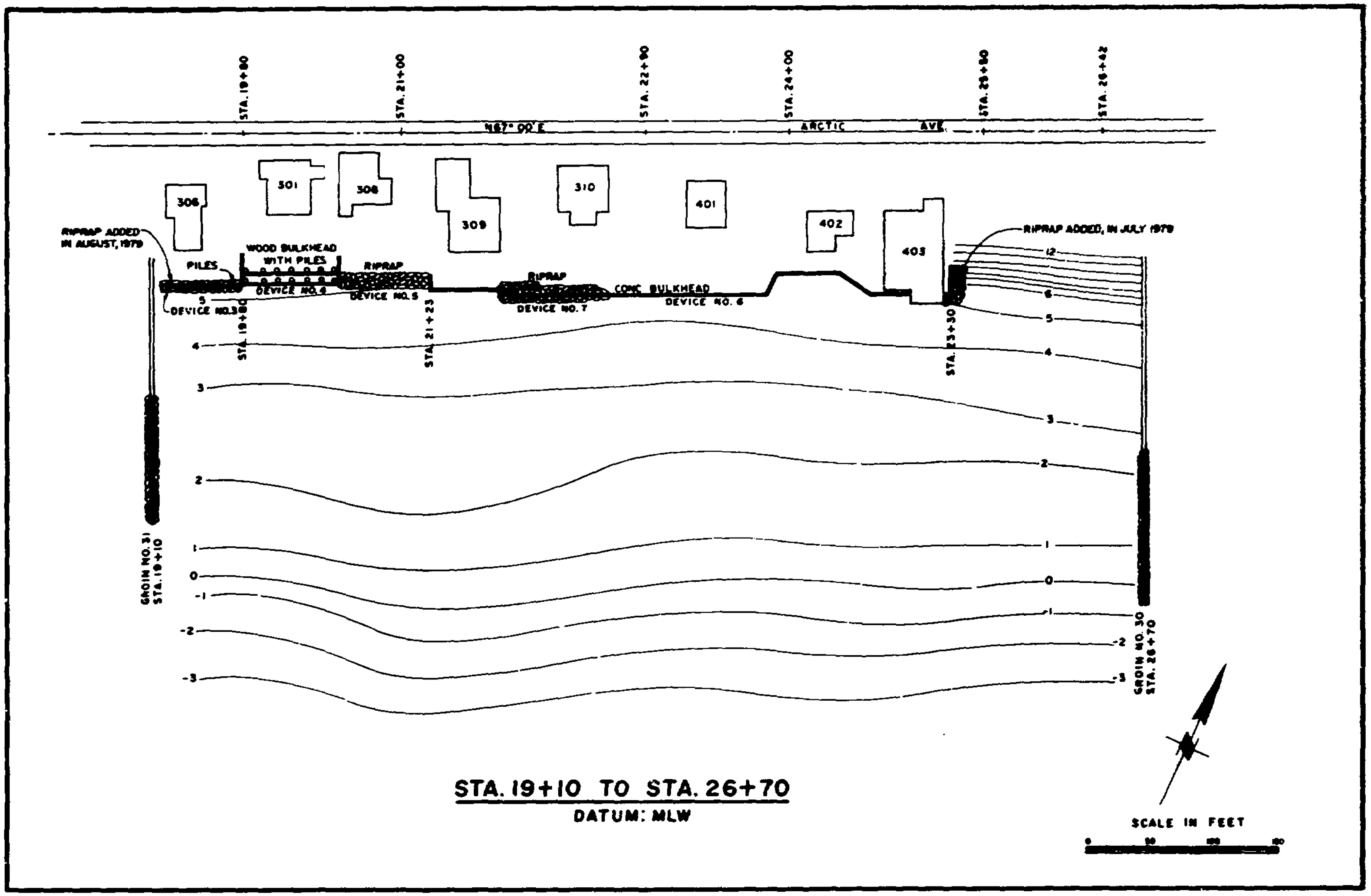


Figure 2-102. Folly Beach area 2.

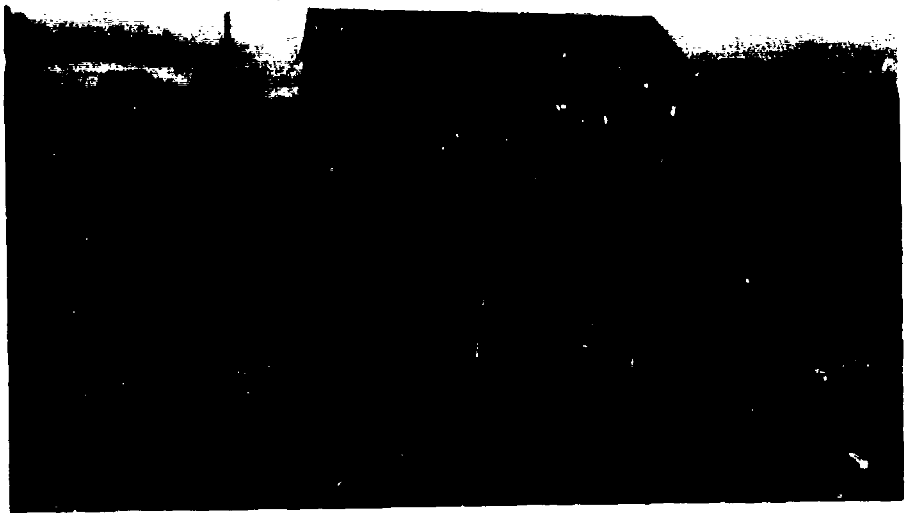


Figure 2-103. Concrete bulkhead with riprap, device 7, Folly Beach, South Carolina, 5 September 1979.

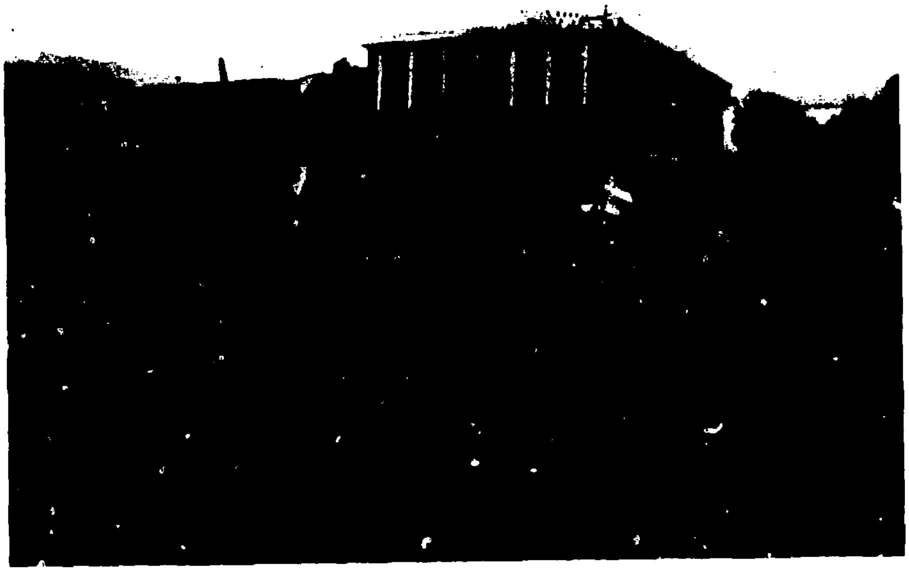


Figure 2-104. Riprap revetment destroyed by Hurricane David, Folly Beach, South Carolina, 5 September 1979.

Figure 2-105. Damage at northeast corner of device 6, Folly Beach, South Carolina, 5 September 1979.

Figure 2-106. Concrete block walls constructed on either side of House 310, Folly Beach, South Carolina, 30 October 1979.

Figure 2-107. Wooden bulkhead before storm, Folly Beach, South Carolina, 27 June 1979.

Figure 2-108. Erosion behind wooden bulkhead, Folly Beach, South Carolina, 5 September 1979.

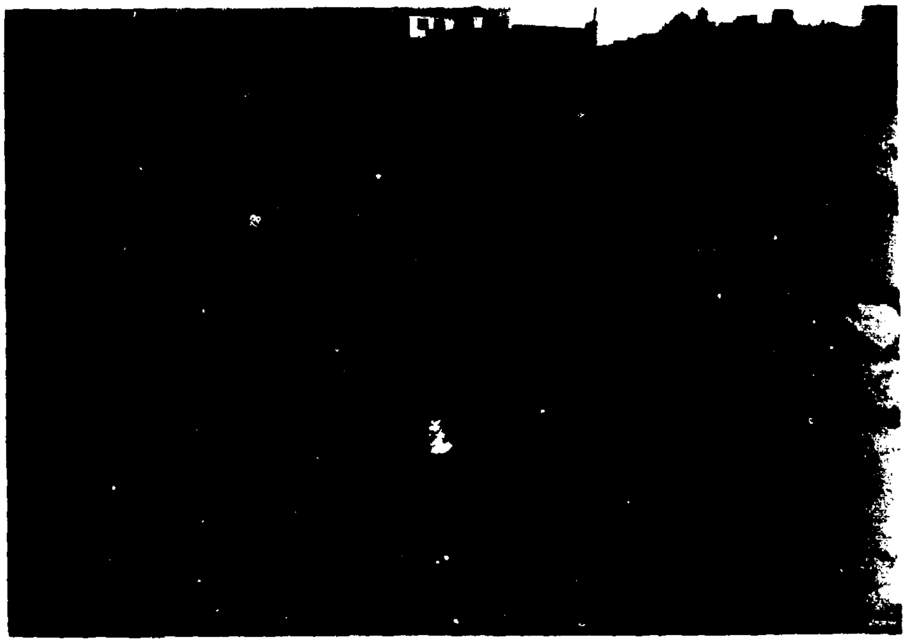


Figure 2-109. Concrete bulkhead and riprap before Hurricane David, Folly Beach, South Carolina, 27 June 1979.

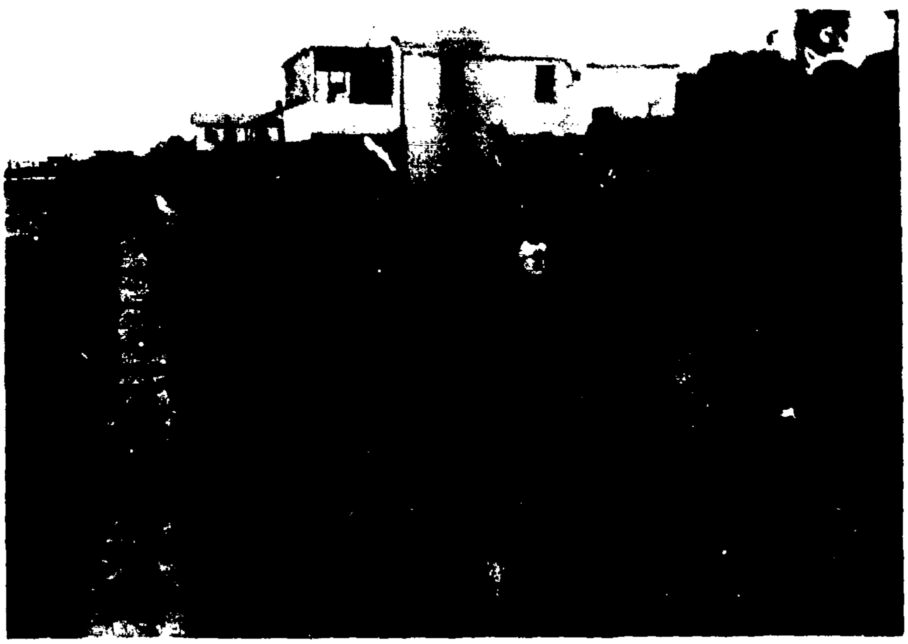


Figure 2-110. Same section as shown in Figure 2-109 after Hurricane David, F lly Beach, South Carolina, 5 September 1979.

Figure 2-111. Unprotected bank at north end of site, Folly Beach, South Carolina, 27 July 1979.

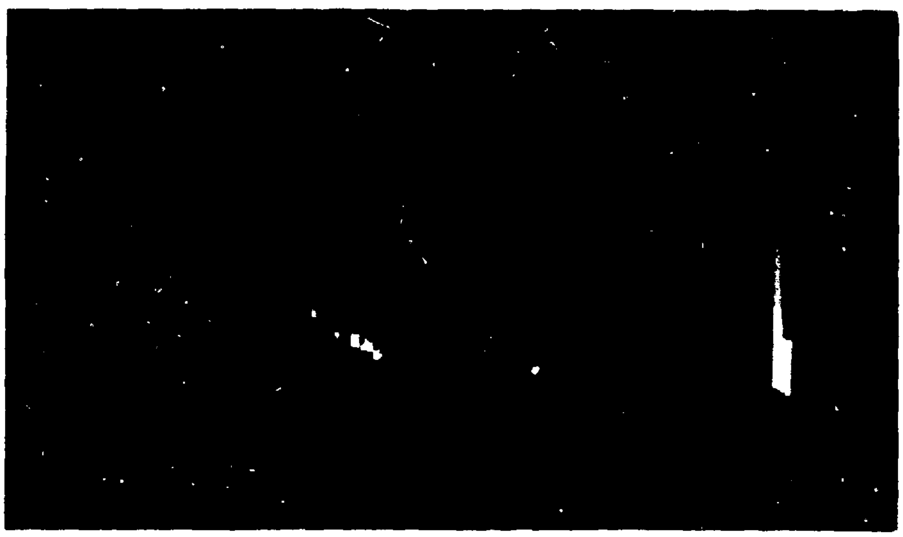


Figure 2-112. Unprotected bank after storm, Folly Beach, South Carolina, 5 September 1979.

Figure 2-113. Sandfill placed on top of heightened device 3 riprap revetment, Folly Beach, South Carolina, 25 January 1980.

Figure 2-114. Device 5 riprap displaced, exposing the filter cloth, Folly Beach, South Carolina, 25 January 1980.

Figure 2-115. Device 3 revetment was completely destroyed, Folly Beach, South Carolina, 29 May 1980.

device 3 had been destroyed, device 4 was still intact, device 5 had been rebuilt, device 6 was still intact, and device 7 had been built up to a higher level. The revetments did not hold up well under normal conditions; the bulkheads were more successful.

(c) Area 3.

1 Description. Area 3 lies between stations 119+57 and 124+80, between device 8, a 170-foot groin, and device 14, a 185-foot groin. The groins are constructed of timber and riprap. Other devices installed in this area include device 9, a 230-foot reinforced concrete bulkhead; device 10, a 220-foot riprap revetment; device 11, a 55-foot concrete-block Sandgrabber; device 12, a 60-foot concrete-block Sandgrabber; and device 13, a 90-foot timber and concrete-block bulkhead backfilled with cinder blocks and other debris, and fronted by large triangular concrete blocks for support. With the exception of the Sandgrabber, many of the devices in area 3 appear in each of the other three monitoring areas. Figure 2-116 shows the location of the devices in area 3.

2 Performance. When the monitoring began in February 1979 the devices were in good condition. By July 1979 the Sandgrabbers were deteriorating rapidly. Many of the blocks on the top row of the seaward Sandgrabber were broken. The center of the landward unit was slumping, but few of its blocks were broken (Fig. 2-117). Sections of the concrete-block seawall between the timber piles were leaning in various

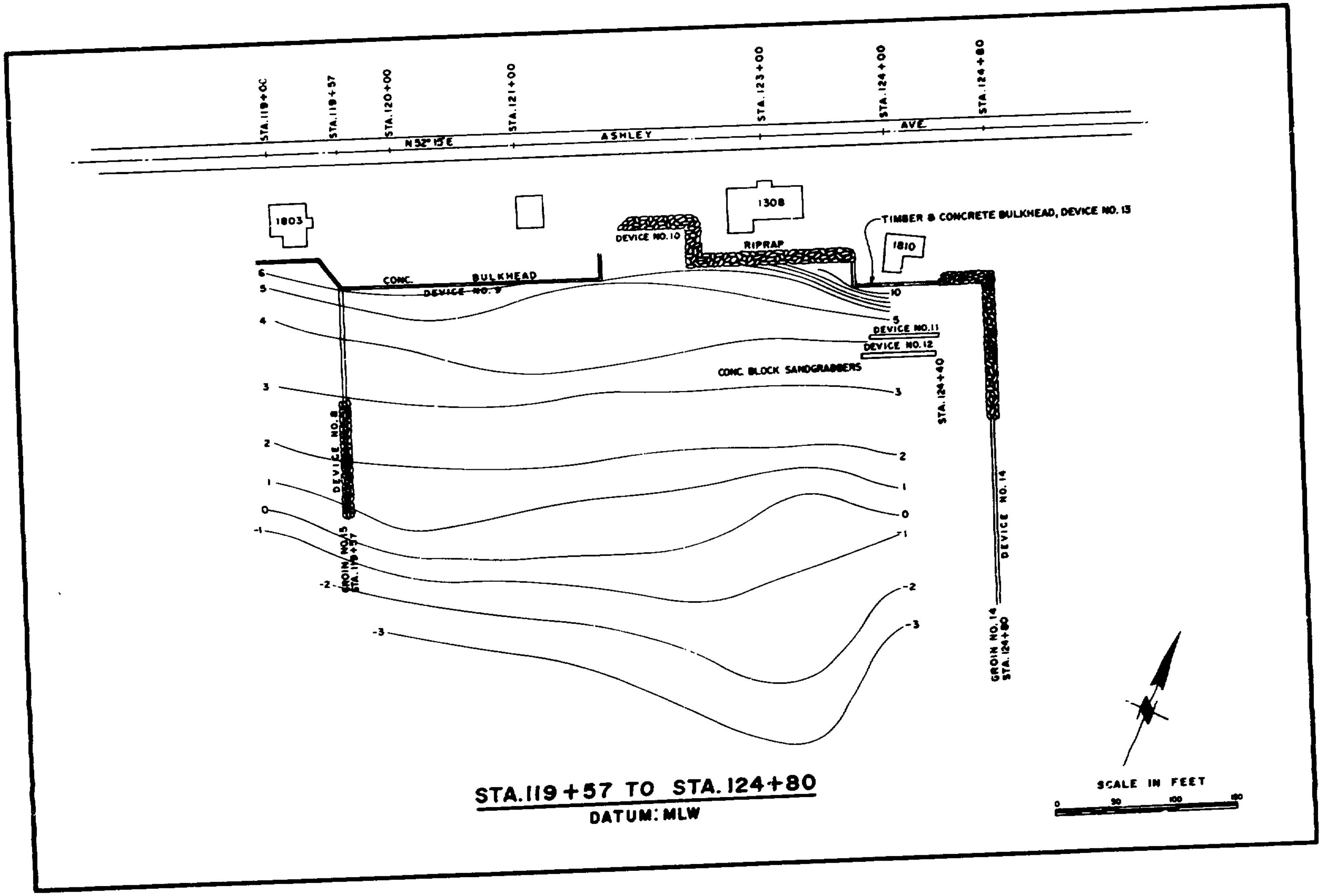


Figure 2-116. Folly Beach area 3.

Figure 2-117. Broken blocks and settlement of Sandgrabbers, Folly Beach, South Carolina, 27 June 1979.

configurations, but the wall remained intact (Fig. 2-118). The condition of the structure did not change much over the next 2 months. The Sandgrabbers continued to deteriorate, and some spalling occurred at the north end of the concrete bulkhead (device 9). The last 60 feet of the bulkhead was destroyed during Hurricane David (Fig. 2-119). The riprap revetment was knocked down but the rocks were not displaced far from the structure (Fig. 2-120). Three sections of the concrete-block bulkhead also fell during the storm (Fig. 2-121). The Sandgrabbers were damaged but held together. The seaward unit was buried up to the top row of blocks. Half of the top row of blocks were missing and many of the blocks that remained were broken. The other Sandgrabber continued to settle and slump, yet only a few of its blocks were missing or broken (Fig. 2-122).

Repairs began in December 1979. Most of the riprap was restacked and the concrete-block bulkhead was resurrected and backfilled. The Sandgrabbers and the concrete bulkhead were not repaired. As in the other monitoring areas, the devices in area 3 performed well under moderate wave conditions. The riprap and bulkheads protected the berm and landward structures from the surging waves and tides, while the Sandgrabbers slowly accreted sand. Figures 2-122 and 2-123 show the sand accumulation from September to December. The seaward Sandgrabber protected the other unit from initial wave contact, as evidenced by the number of broken blocks in the top row of each structure. The tandem positioning of the Sandgrabbers also hastened sand accumulation between them.

Figure 2-118. Timber and concrete-block seawall, device 10, Folly Beach, South Carolina, 27 June 1979.

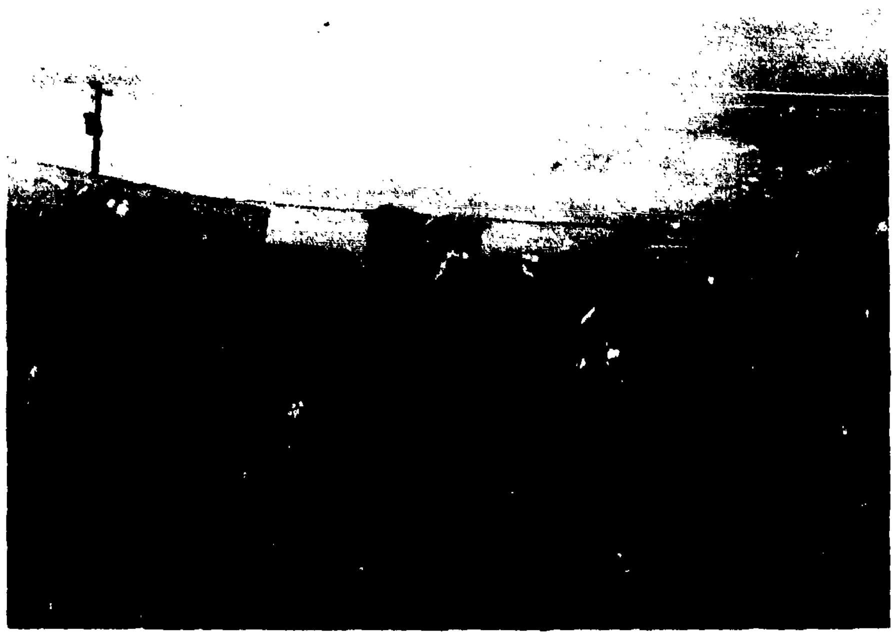


Figure 2-119. Concrete bulkhead section in device 9 destroyed by Hurricane David, Folly Beach, South Carolina, 5 September 1979.

Figure 2-120. Section of device 10 riprap revetment after Hurricane David, Folly Beach, South Carolina, 5 September 1979.

Figure 2-121. Fallen concrete-block bulkhead sections, Folly Beach, South Carolina, 5 September 1979.

Figure 2-122. Sandgrabbers after Hurricane David, Folly Beach, South Carolina, 5 September 1979.

Figure 2-123. Sand accumulation by Sandgrabbers, Folly Beach, South Carolina, 28 December 1979.

Before the hurricane, some sand was accumulating behind the north end of the concrete bulkhead. Local erosion occurred where the revetments and bulkheads had failed (Figs. 2-124 and 2-125). In March 1980, new fill and riprap were placed where device 10 had been in front of building 1308 (Fig. 2-126). No additional changes were made as of May 1980. The two groins, devices 8 and 14, sustained little damage.

(d) Area 4.

1 Description. Area 4 lies between stations 131+95 and 138+00, bounded by about 300-foot-long groins (12 and 13) made of timber and riprap. Device 15 is a 210-foot concrete wall fronted with riprap. Device 16 is a 350-foot rock revetment that extends to groin 12. Figure 2-127 indicates the positions of the two devices monitored in area 4.

2 Performance. No changes in structural alinement or cross section occurred until July 1979, when small sections of riprap fell, exposing the filter blankets near buildings 2000 and 2005 (Figs. 2-128 and 2-129). The concrete wall and riprap section (device 15) was not damaged by Hurricane David. Rock from the revetment and groin 12 was scattered and pulled down during the storm but damage was not extensive (Fig. 2-130). Fill material and broken concrete were placed in front of the concrete wall after the storm in October (Fig. 2-131). The displaced riprap was restacked, and fill was added to the north end of the area near groin 12.

Figure 2-124. Local erosion behind destroyed section of device 9 bulkhead, Folly Beach, South Carolina, 5 September 1979.

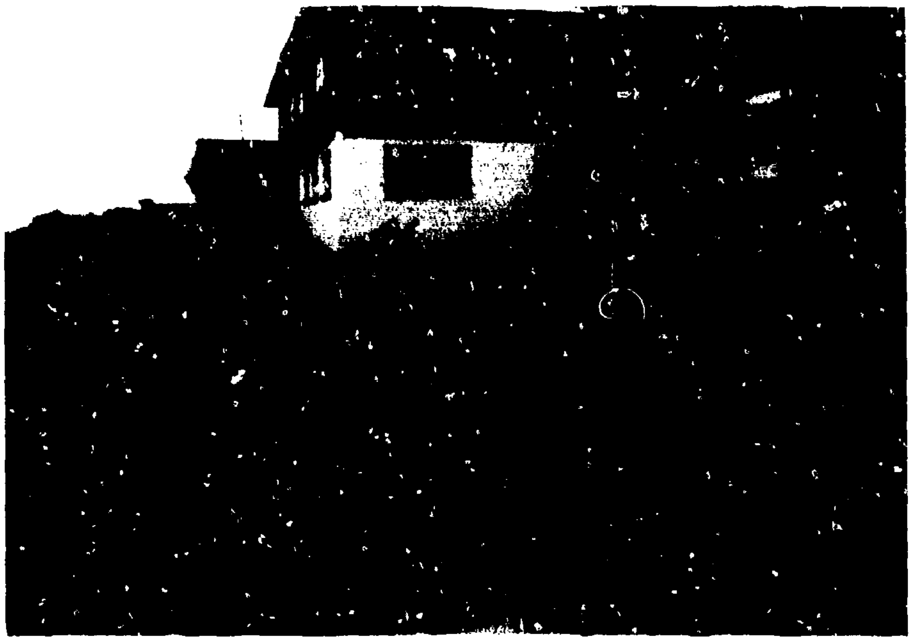


Figure 2-125. Local erosion behind failing device 10 revetment, Folly Beach, South Carolina, 5 September 1979.

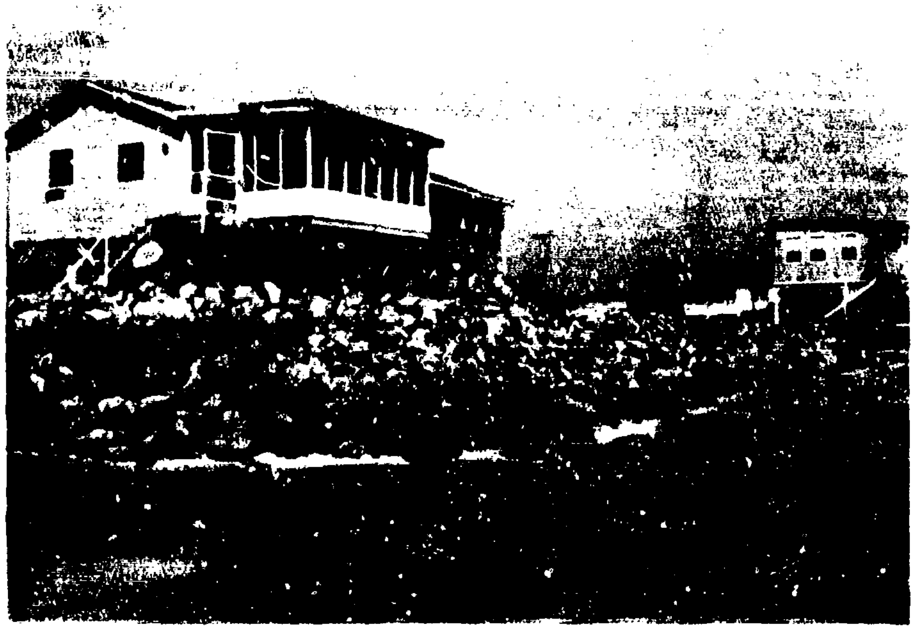


Figure 2-126. New riprap placed in front of building 1308 in device 10, Folly Beach, South Carolina, 5 March 1980.

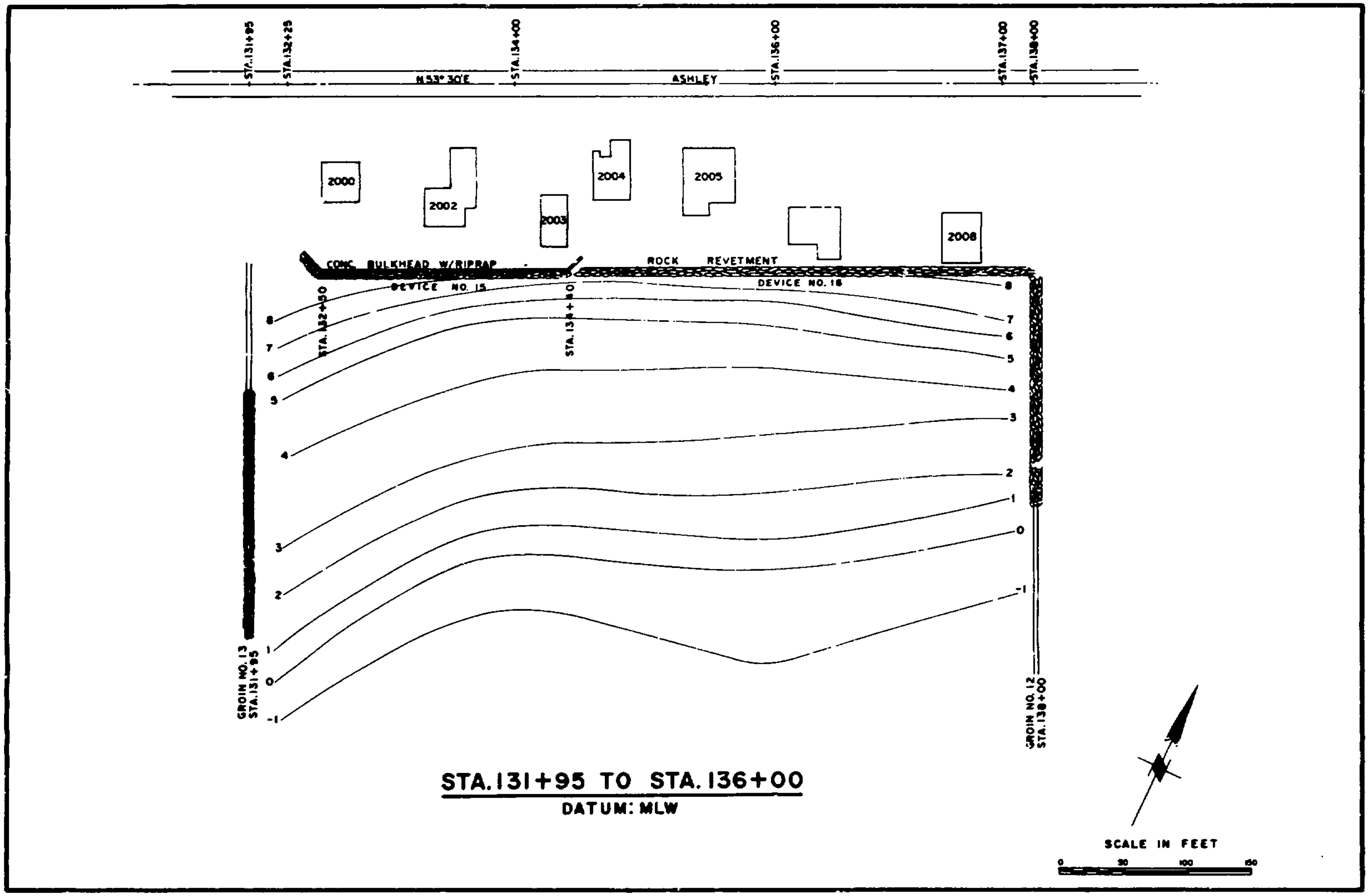


Figure 2-127. Folly Beach area 4.

Figure 2-128. Fallen riprap and exposed filter blanket near building 2000, device 15, Folly Beach, South Carolina, 16 July 1979.

Figure 2-129. Fallen riprap and exposed filter cloth near building 2005, device 16, Folly Beach, South Carolina, 16 July 1979.

Figure 2-130. Fallen device 16 revetment and erosion behind it, Folly Beach, South Carolina, 5 September 1979.

Figure 2-131. Broken concrete placed in front of device 15 bulkhead after storm, Folly Beach, South Carolina, 30 October 1979.

The shoreline in area 4 did not deteriorate severely from the wave forces of Hurricane David. There was no erosion of the bank behind the riprapped concrete bulkhead. Erosion did occur along the revetment where the fallen riprap exposed the berm (Fig. 2-132). The adjacent beach just north of groin 12 was severely eroded after a small section of bulkhead there failed (Fig. 2-133). As of May 1980, devices 15 and 16 remained intact.

(3) Analysis. The revetments and bulkheads at the Folly Beach site were installed to preserve the berm and landward structures behind them. They were not designed and located to influence the littoral system. This is the function of the groin field. Only two groins, devices 8 and 14, were monitored. They sustained little damage, but they failed to provide any significant protection to the shoreline. With these purposes in mind and with the monitoring data, it is concluded that the revetments and bulkheads were moderately adequate in protecting the shoreline under normal conditions. The failure of the riprap revetments (devices 2, 3, 5, and 10) during Hurricane David was expected, in that these structures were not designed to withstand hurricanes. The continual need for reconstruction of devices 3 and 5 probably resulted because the filter behind the rubble was inadequate and because the rocks were placed on too steep a slope. Most of the bulkhead structures maintained their structural integrity, with the exception of devices 9 and 13 in area 3, which were destroyed in Hurricane David. In general, the bulkhead structures more adequately protected the beach against erosion under normal conditions.

Special attention should be given to the modular concrete bulkhead. It protected the berm well during Hurricane David, failing in only one location. The entire length of the bulkhead should be fronted with toe stone (similar to that used in device 7), which would protect the structure from displacement due to toe scouring. The tandem arrangement of the Sandgrabber is unique. More monitoring is needed to determine whether the use of the sacrificial seaward Sandgrabber is cost-effective or whether some other alternative should be devised.

3. Gulf Coast Sites.

a. Common Characteristics. Two demonstration sites and four monitoring sites are on, or in waters connecting with, the Gulf of Mexico (Fig. 2-134). The demonstration sites are on the north shore of Choctawhatchee Bay near Basin Bayou, Florida, and on the northeast shore of Lake Pontchartrain at Fontainebleau State Park, Louisiana. The monitoring sites are Key West, Florida, and Holly Beach, Louisiana, both on the gulf coast, and at Shoreacres and Beach City in Galveston Bay, Texas. The climate is mild to hot and generally humid, but moderated by proximity to gulf waters. Tides are generally minimal, but all six sites are subject to severe hurricanes.

b. Basin Bayou State Recreation Area, Florida.

(1) Site Description.

(a) Geographical Setting. The Basin Bayou demonstration site is in the Basin Bayou State Recreation Area on the north side of Choctawhatchee Bay in Walton County, Florida. It is located about 56 miles east of

Figure 2-132. Erosion of berm behind device 16, Folly Beach, South Carolina, 5 September 1979.

Figure 2-133. Erosion of adjacent beach north of groin 12, Folly Beach, South Carolina, 5 September 1979.

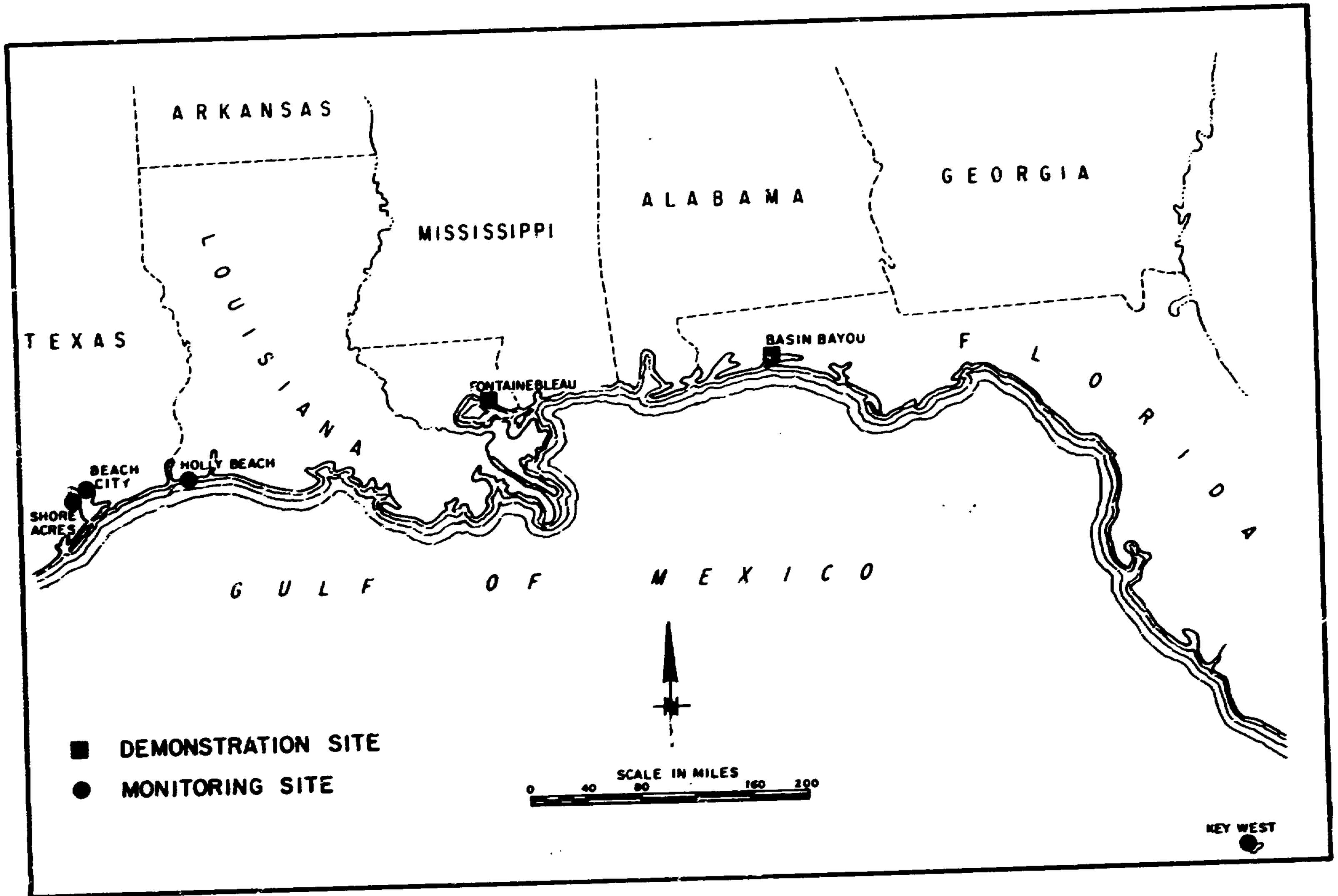


Figure 2-134. Location map of gulf coast demonstration and monitoring sites.

Pensacola in the southern part of the Eglin Air Force Base military reservation (Fig. 2-135). The park is a 287-acre tract leased from the U.S. Air Force by the Division of Recreation and Parks, Florida Department of Natural Resources, and has about 3,000 feet of frontage on the north shore of Choctawhatchee Bay. The demonstration site is within this reach of shoreline. Access to the site is via State Highway 20 and roads within the park.

(b) Winds, Waves, Tides and Longshore Transport. Choctawhatchee Bay is one of the major bays on the northern shore of the Gulf of Mexico, with a total area of about 122 square miles. The bay is 30 miles long (east to west), averages 4 miles wide, and ranges in depth to 40 feet; 16 square miles has greater than a 30-foot depth. In the demonstration area, the bay waters are very shallow. Tides are diurnal, with a mean tidal range of 0.5 foot and an extreme, except during storms, of about 1.5 feet. Prevailing winds are southwesterly. Wave action along the shoreline is normally minimal, but during periods of brisk, southwesterly winds the water level is raised by wind setup, producing higher energy waves, which cause erosion primarily by moving the sand offshore. Structural design for the shore protection devices was based on a 50-mile per hour southwesterly wind, generating waves in the bay that reach the site with a period of 4 seconds and a significant height of 3.6 feet. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 4.0 feet, and the net longshore transport potential was 60,400 cubic yards southeastward for the 11 months analyzed. The wave climate is classified as intermediate.

(c) Geomorphology, Soils, and Vegetation. The site area forms part of the coastal lowland of the east Gulf Coastal Plain physiographic section. This narrow coastal lowland is composed of Quaternary fine clastic sediments, primarily sand. Terracing, caused by Pleistocene sea level fluctuations, forms a line of low subdued bluffs paralleling the beach and marking the extent of former shorelines. A belt of gently southward-sloping cuestas, formed from erosion of Pliocene river and estuarine deposits of gravel, sand, and clay, is located north of the site and parallel to the beach. Longshore sand migration and eroded sediments from the inland elevated Pliocene clastic sediments provide a ready source of beach sand.

Soils in the backshore are classified as silty to fine sands. The thickness of the surface stratum or the existence of any significant substrata have not been defined. The foreshore and offshore surface generally consists of medium dense sands under the influence of littoral transport.

The general topography of Basin Bayou Park is flat and at about an elevation of +10 feet above NGVD. The backshore area typically terminates in a 5- to 10-foot bluff, where it meets a narrow sandy beach. The offshore waters are generally shallow, with depths of less than 4 feet extending more than 1,000 feet offshore.

The park is situated in a coastal pine flatlands dominated by an overstory of slash pine (*Pinus elliottii*) with some small stands of live oak (*Quercus virginiana*) near the bay.

(d) The Problem. This site is a typical example of a low-bluff erosion problem resulting in recession of the shoreline. Although

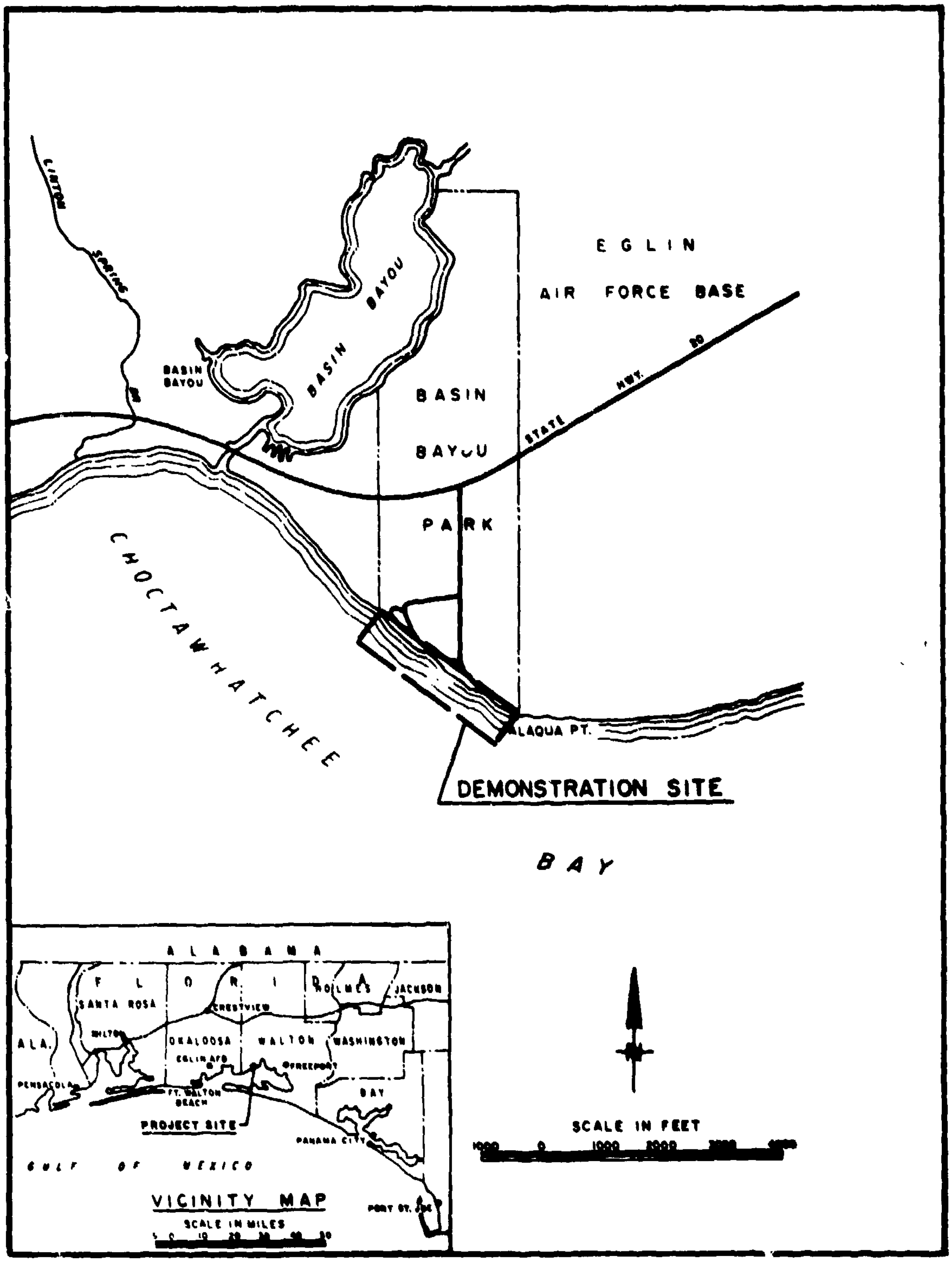


Figure 2-135. Location map of Basin Bayou State Recreation Area, Florida, demonstration site.

southwesterly winds generate a net eastward littoral current along the shoreline, erosion from this process appears minimal under normal weather conditions.

During periods of high winds or storms, the water level is raised due to wind setup and the waves are larger. The narrow beach provides little protection for the bluff during these conditions, and direct wave attack of the shoreline occurs, eroding the bluff and moving the sandy material offshore. As erosion occurs, trees along the shoreline are undermined and wave action prevents growth of other vegetation.

(2) Demonstration Project. The shore protection devices constructed at the Basin Bayou site include structural installations and vegetation plantings. The structural devices comprise a fence-sandbag bulkhead, a sandbag breakwater, a Longard-tube breakwater, a Surgebreaker offshore reef, and a Sandgrabber. Vegetation was used as a primary erosion control device in one location and as a complementary device in conjunction with the Sandgrabber. Much of the work was done by hand. The largest items of equipment were a helicopter for the Surgebreaker installation, dump trucks, and a front-end loader used in loading the hopper of the special slurry mixer and pump that filled the Longard tube. The Mobile District constructed the fence-sandbag bulkhead and the sandbag breakwater but awarded contracts for construction of the Sandgrabber and Surgebreaker devices and for installation of the Longard tube. Figure 2-136 shows the layout plan for the various installations.

(3) Statistics, Construction, and Costs. Table 2-26 provides available data on all devices.

Table 2-26. Statistics and costs, Basin Bayou, Florida.

Contracts	Costs
Longard tube	\$23,800
Sandgrabber	28,000
Surgebreaker offshore reef	25,000
Mobile District Labor Statistics	
Fence-sandbag bulkhead and sandbag breakwater	
Labor (man-hours)	550
Skilled	550
Semiskilled	550
Unskilled	1,850
Materials	
Sand (yd^3)	1,100
Advance Bag sandbags	6,000
Sand Pillow sandbags	4,000
Dura-Bag sandbags	45
36-inch fence (lin ft)	210
4-in-diameter posts	20
Plants	4,600
Equipment (hr)	
Truck (semi)	16
Truck (2-1/2 ton)	40
Truck (1/2 ton)	1,200
Front-end loader	160
Swamp buggy	80
Pump	80

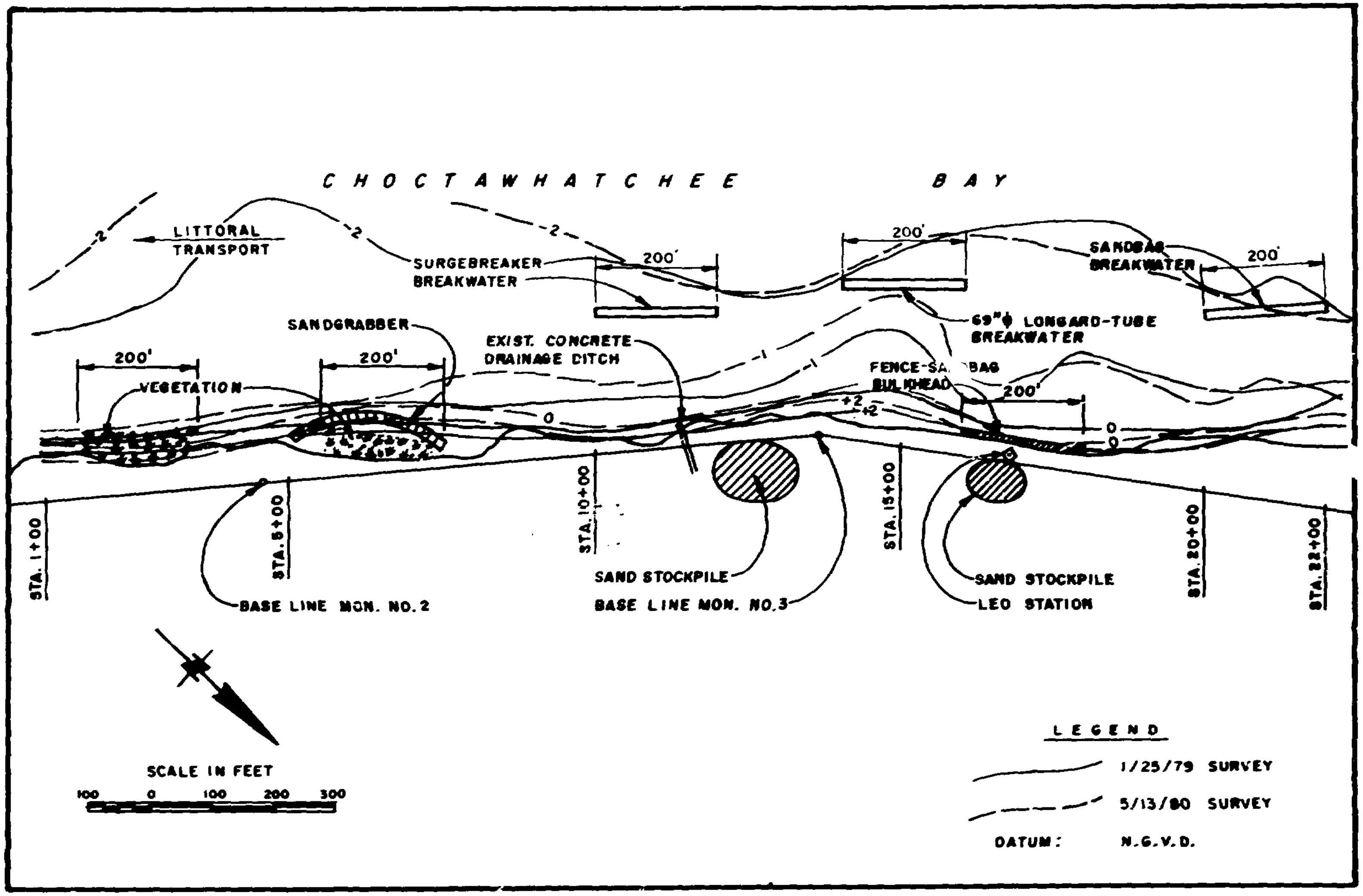


Figure 2-136. General plan of Basin Bayou, Florida, demonstration site.

(a) Fence-Sandbag Bulkhead. A 200-foot section of fence-sandbag bulkhead was used to protect the bluff between stations 16+00 and 18+00 (Fig. 2-137). The bluff in this area is about 7 feet high with a gentle, sandy slope from the toe of the bluff to the shoreline. A 3-foot-high fence was placed parallel to the general alinement of the bluff. Wooden creosoted posts (7 feet long), placed 4 feet in the ground and 5 feet on centers, and 36-inch hogwire fabric were used as cribbing. Two kinds of bags were used: a woven acrylic, ultraviolet-resistant sandbag called "Sand Pillows," manufactured by Monsanto Company, and "Advance Bags," which are spun-bonded polypropylene sandbags manufactured by the Advance Construction Specialities Company. The sandbags, which weigh about 100 pounds when filled, were hand-placed between the fence and the face of the bluff to a height of 3 feet. The bags were folded and sewn closed with nylon thread; factory seams were made with cotton thread. A 100-foot segment of the bulkhead contains Sand Pillows, and the other 100 feet contains Advance Bags. Half of each segment is two bags wide, as measured out from the bluff, and the other half is three bags wide (Fig. 2-137). No filter cloth was used. Construction was completed early in December 1978 (Figs. 2-138 and 2-139).

(b) Sandbag Breakwater. A 200-foot sandbag breakwater was constructed 200 feet offshore in about 3 feet of water between stations 20+00 and 22+00 using three different types of sandbags: Sand Pillows, Advance Bags, and Dura-Bags (Fig. 2-140). When the breakwater was put in place, the cross sections deviated somewhat from section A-A in that the base was wider. The large nylon Dura-Bags required filling in place. A small trash pump (3- to 4-inch diameter) and hopper arrangement (Fig. 2-141) hydraulically filled the bags with commercial sand stockpiled at the site. Sand was transferred from the stockpile to the hopper arrangement by front-end loaders. The hopper was set up onshore and the slurry mixture of sand and water was pumped some 200 feet offshore to the breakwater location. The Sand Pillows and Advance Bags are small and light enough when filled to be handled by hand. They were stacked 5 feet high with the longitudinal axis of each bag laid parallel to the shoreline. Construction was completed in February 1979.

(c) Longard-Tube Breakwater. A 200-foot length of 69-inch-diameter Longard tube was used as a breakwater about 200 feet offshore between stations 14+00 and 16+00 (Figs. 2-142 and 2-143). The Longard tube is a two-ply, flexible tube. The outer tube is a black, high-density, polypropylene woven fabric that is stabilized against deterioration by ultraviolet rays; the inner tube is an impermeable, low-density polyethylene film. First, filter cloth was unrolled and moved into location along the breakwater axis. The 10-inch-diameter Longard tubes attached to each side

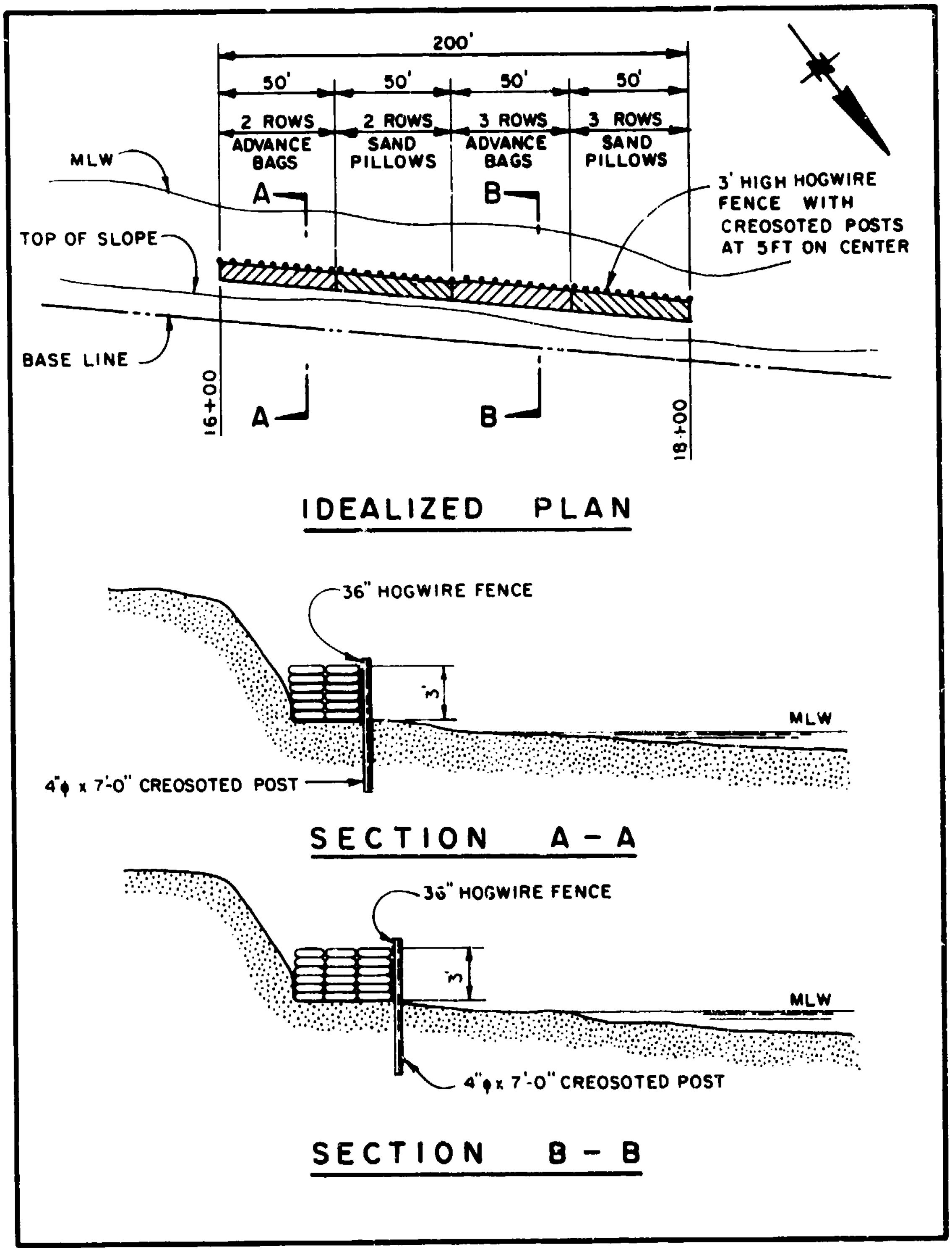


Figure 2-137. Fence-sandbag bulkhead at Basin Bayou, Florida.

Figure 2-138. View of fence-sandbag bulkhead from east end at shoreline, Basin Bayou, Florida, 13 December 1978.

Figure 2-139. View of fence-sandbag bulkhead from west end at shoreline, Basin Bayou, Florida, 13 December 1978.

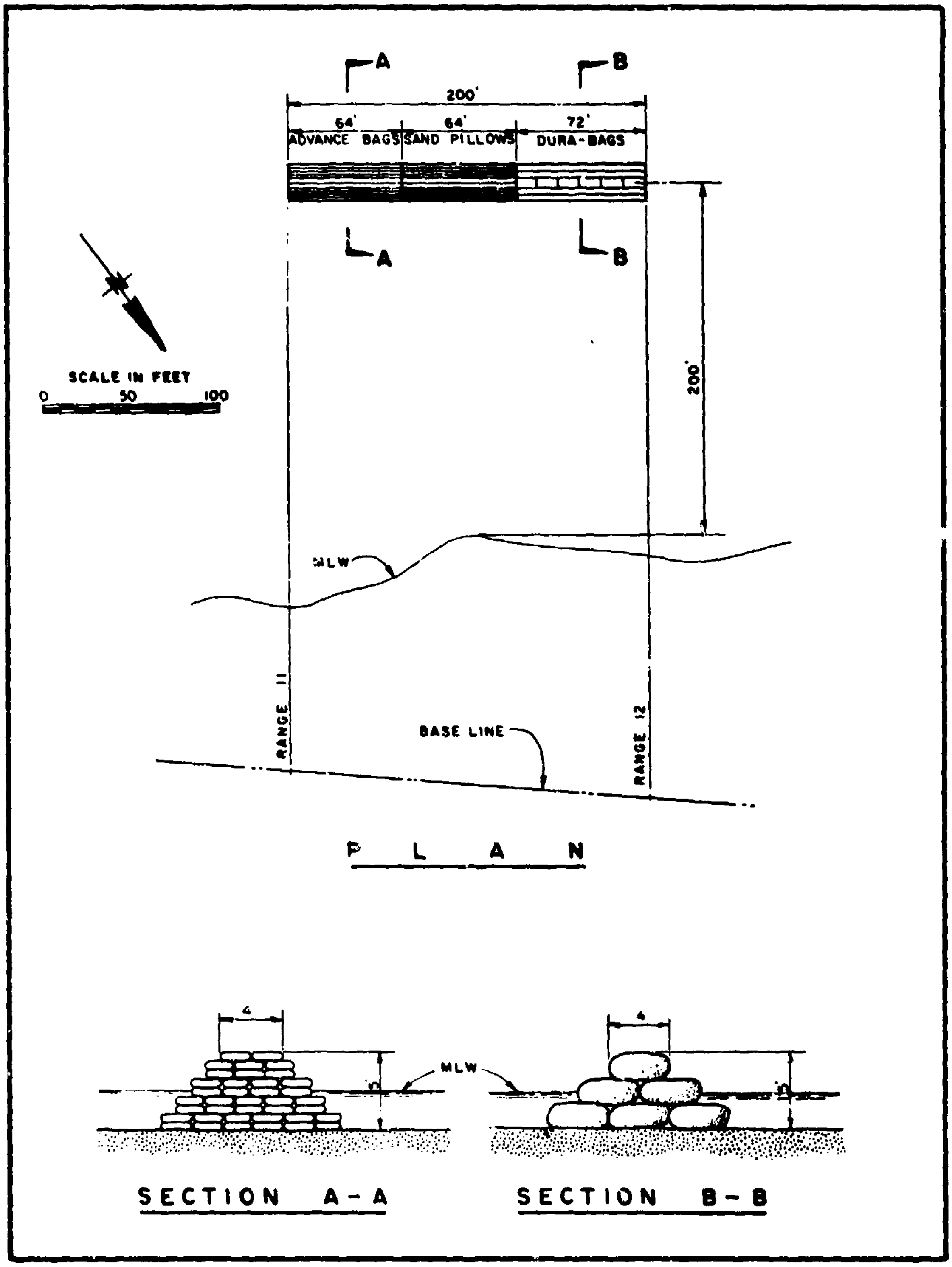


Figure 2-140. Sandbag breakwater at Basin Bayou site.

Figure 2-141. Trash pump and pipeline for filling Dura-Bags, Basin Bayou, Florida, 9 February 1979.

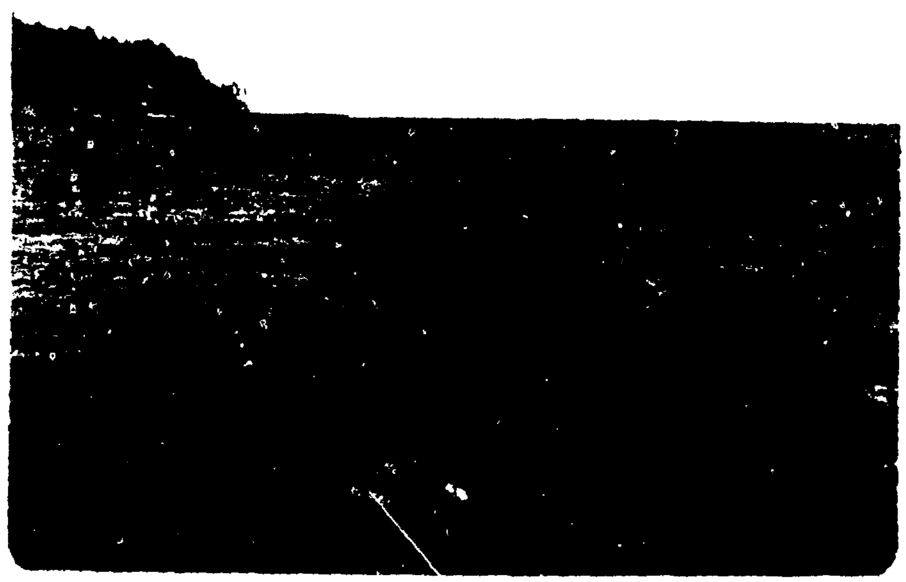


Figure 2-142. Longard-tube breakwater, Basin Bayou, Florida, 5 December 1978.

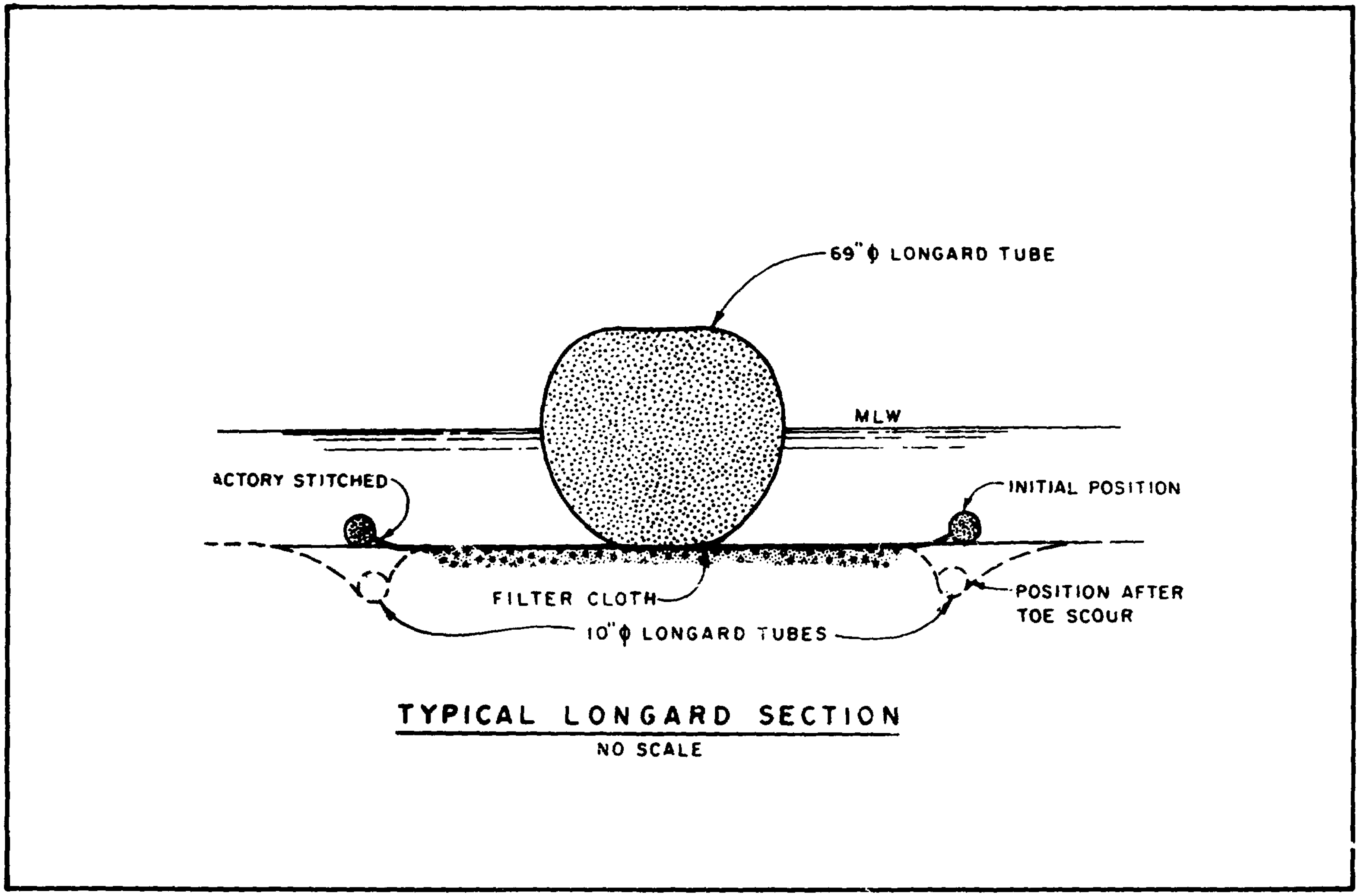


Figure 2-143. Longard-tube installation at Basin Bayou, Florida site.

were hydraulically filled with commercial sand stockpiled at the site. Then, a 69-inch tube was placed on the filter cloth and filled with sand in the same manner. The sand-pumping mechanism is a patented device built expressly for filling Longard tubes (Fig. 2-144). Construction was completed 5 December 1978.

(d) Surgebreaker Offshore Reef System. A Surgebreaker 200 feet long was constructed about 200 feet offshore between stations 10+00 and 12+00 (Fig. 2-145). The Surgebreaker modules consisted of 50 precast concrete modules reinforced with steel fibers and steel reinforcing bars. The modules have ventholes to release wave-pressure buildup and are triangular in cross section, 4 feet high with a base 7 feet wide (Fig. 2-146). The 3,700-pound modules were trucked to a stockpile on the shore, individually airlifted by helicopter to the offshore site at low tide, guided into place by a wading crew, and set end-to-end on the sand bottom (Fig. 2-147). Because of difficulties in contract negotiations, installation was deferred to about a year after installation of the other structural devices at this site. The structure was completed in November 1979 (Fig. 2-148).

(e) Sandgrabber. A Sandgrabber about 240 feet long was constructed between stations 5+00 and 7+50. The Sandgrabber is a patented permeable structure composed of hollow concrete blocks similar to, but larger than, commercial building blocks. In principle, the approaching wave, with its load of entrained sand, loses most of its energy as it passes through the structure, and some of the sand is deposited behind it, building a stabilized back berm. As the openings in the lower courses of blocks are filled in, the waves deposit sand on the seaward side of the structure

Figure 2-144. Filling the Longard tube with hydraulically pumped sand, Basin Bayou, Florida, 2 December 1978.

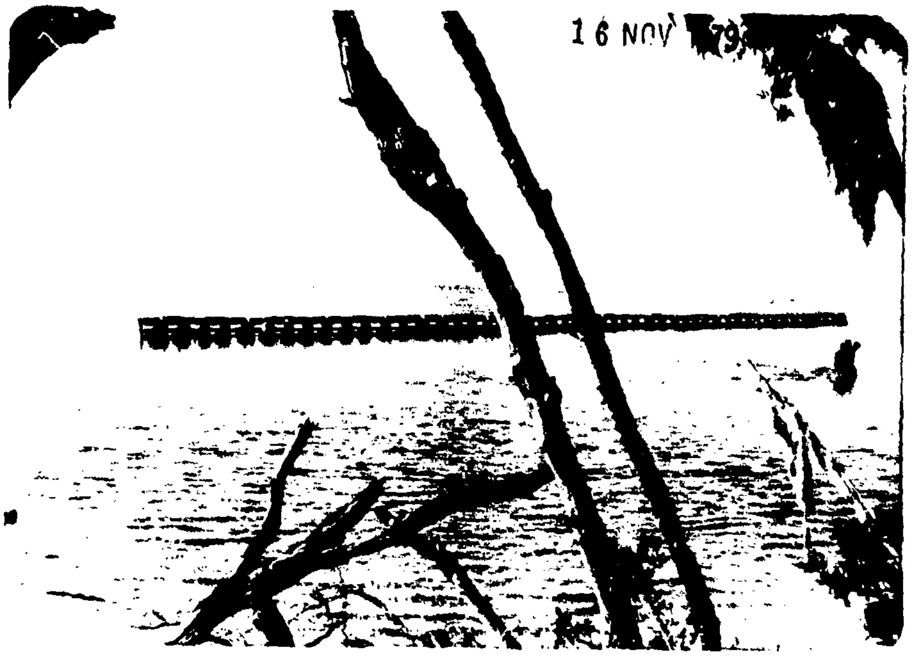


Figure 2-145. View of the Surgebreaker breakwater, just completed, Basin Bayou, Florida, 16 November 1979.

Figure 2-146. Surgebreaker modules, Basin Bayou, Florida, 15 November 1979.

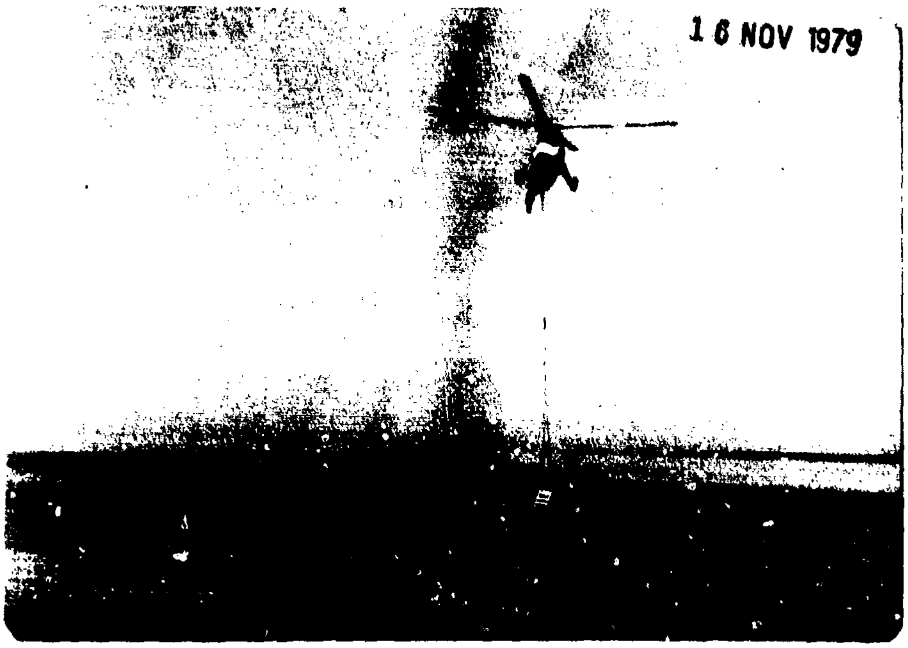


Figure 2-147. Helicopter placing the Surgebreaker modules, Basin Bayou, Florida, 16 November 1979.

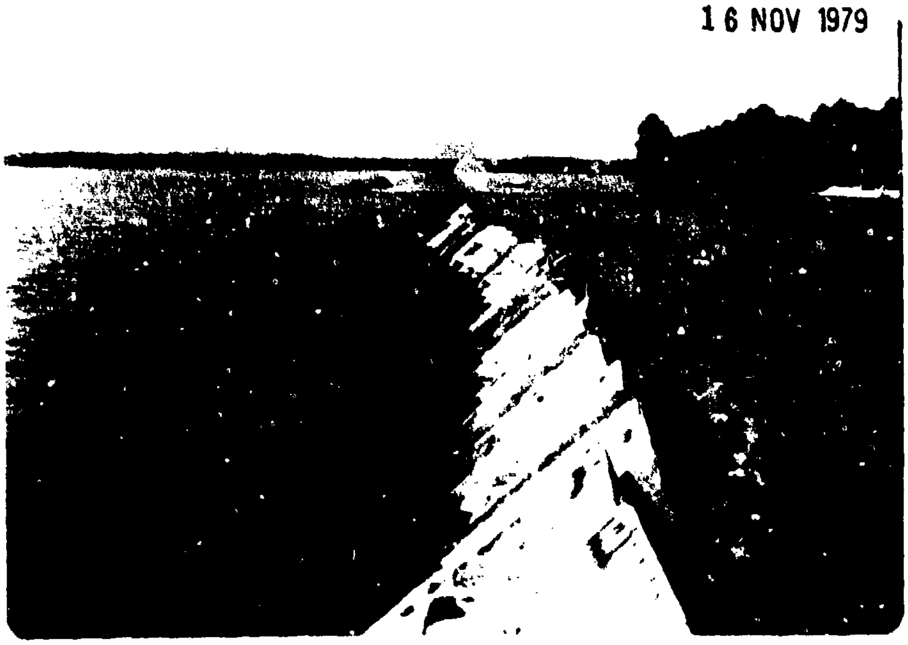


Figure 2-148. View of the Surgebreaker breakwater from the east end, Basin Bayou, Florida, 16 November 1979.

instead of behind it. Because the Sandgrabber is permeable and fairly flexible, the scouring and stability problems common to many coastal structures are theoretically averted. At the Basin Bayou site, the Sandgrabber was built in a flat arc alinement with its ends tying back into the beach. The structure is 64 inches wide by 34 inches high, placed directly on the bottom with no bedding layer or filter. The blocks were placed by hand as in brickwork, with hollows facing seaward and then tied together with U-shaped steel rods passing through the hollows from one side to the other (Fig. 2-149). Construction was completed in September 1978.

(f) Vegetation. Smooth cordgrass and saltmeadow cordgrass were planted in sections A, B, C, and F of unprotected beach and in sections D and E behind the Sandgrabber (Fig. 2-150). In the unprotected sections, smooth cordgrass was planted from MLW, elevation +0.20 to 1.00. Behind the Sandgrabber, smooth cordgrass was planted up to elevation 1.00, but the lower elevation varied with sand deposition behind Sandgrabber blocks. Saltmeadow cordgrass was planted from elevation 0.70 to 1.50 in the unprotected sections, and from approximately elevation 1.00 to 1.50+ behind the Sandgrabber. Much of the area behind the Sandgrabber is above elevation 1.50 but was planted so that a large enough area was available to monitor.

Smooth cordgrass plantings consisted of 3,000 plugs 5 to 6 inches in diameter, 6 to 8 inches high with 1 to 3 stems per plant (stems were 2 to 3 feet high); saltmeadow cordgrass plants (3,000) consisted of 4 to 6 stems, 12 to 18 inches high, in 4-inch plastic pots. Bermuda or Bahia grass was supposed to be planted, but was not, due to the poor survival of the cordgrass. Fertilization was varied on both exposed and protected plantings. Fertilizer regimes were:

Plan 1 - 50 pounds of nitrogen per acre from 8- to 9-month release Osmocote (18-6-12) in planting hole.

Plan 2 - Topdress with 50 pounds of nitrogen per acre from ammonium sulfate and 50 pounds P_2O_5 from super phosphate 3 to 4 weeks after planting; repeat nitrogen application after 4 to 6 weeks.

Smooth cordgrass planting took place approximately between 20 and 30 March 1979. Smooth cordgrass plugs were planted by hand in holes opened with a spade so that the base of the stem was at or 1 inch below ground level. Plantings were placed on 18-inch centers. Section A contained 200 unfertilized smooth cordgrass plants; section B, 300 plants fertilized according to plan 1; section C, 300 unfertilized plants; and section F, 600 plants fertilized according to plan 2. Protected sections D and E behind the Sandgrabber were each planted with 100 plants; section D was fertilized according to plan 1, and section E according to plan 2. Planting was completed 30 March 1979, and an Initial Post Planting Inventory was made at that time.

Saltmeadow cordgrass was planted between 1 and 5 May 1979, approximately. Plants were hand-planted on 18-inch centers and placed so that the base of the stems were at, or 1 inch below, ground surface. Numbers of plants and fertilization regimes varied in each section. In section A, 400 unfertilized

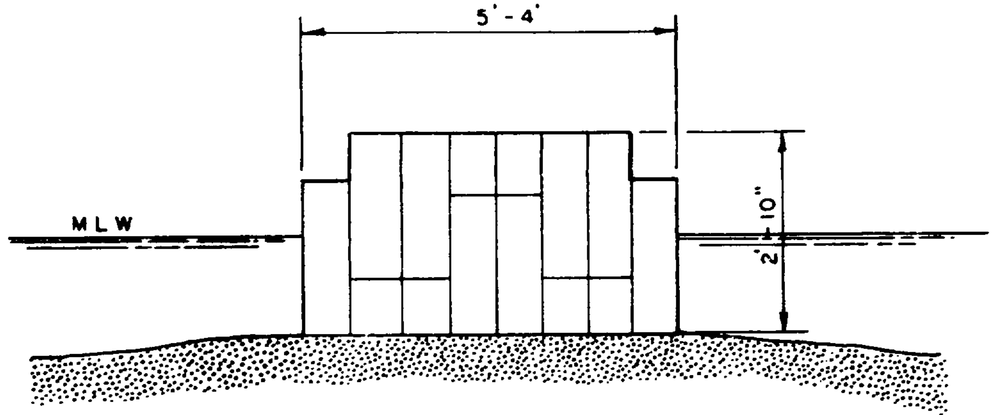


Figure 2-149. Sandgrabber section at Basin Bayou site.

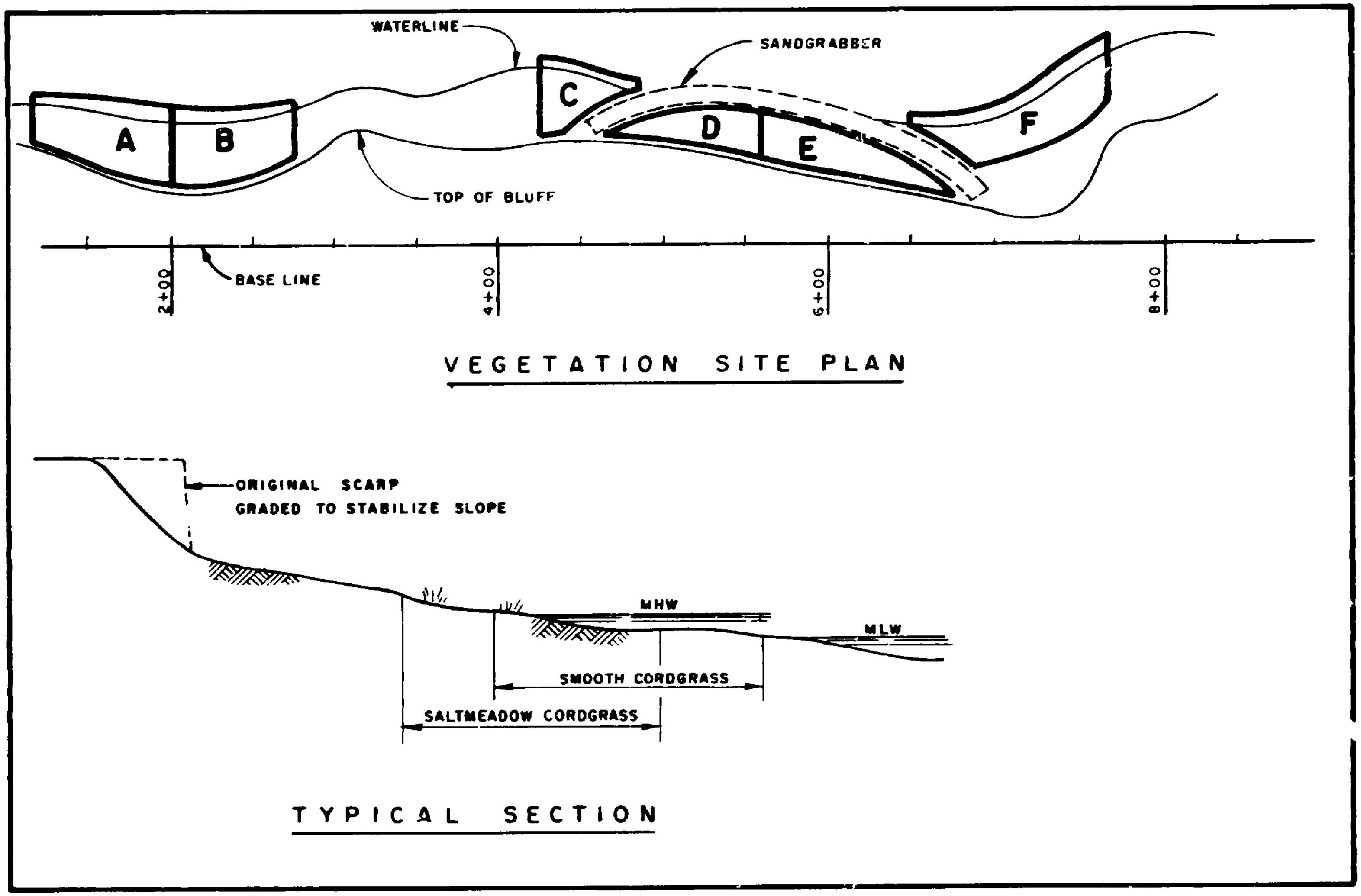


Figure 2-150. Vegetation site plan at Basin Bayou site.

plants were used; in section B, 700 plants fertilized according to plan 1 (at time of planting); in section C, 500 plants fertilized according to plan 2; and section F, 900 plants fertilized at time of planting according to plan 1. Protected sections D and E were each planted with 500 plants. Plants in section E were fertilized at time of planting (plan 1); plants in section D were fertilized later (plan 2). Planting was completed on 10 May 1979 and an Initial Post Planting Inventory made at that time. Costs are given in Table 2-27.

Table 2-27. Vegetation costs at Basin Bayou, Florida.

Item	Cost
Smooth cordgrass (3,000 plants)	$3,000
Shipping	500
Labor (8 man-days)	800
Saltmeadow cordgrass (3,000 plants)	$1,500
Labor (9 man-days)	900
	Total $6,700

(4) Performance.

(a) Fence-Sandbag Bulkhead. Accretion and erosion were noticeable along the bulkhead 2 months after installation. Figure 3-18, which was put in Section III for comparison of profiles through similar devices, shows a series of profiles through the fence-sandbag bulkhead. The profiles show the erosion and accretion trends which have occurred. Sand movement along the bulkhead ranged from a buildup of about 1.5 feet on the east end to a depletion of about 1 foot on the west end, indicating predominance of longshore transport to the east. Where erosion occurred, the sandbags were undermined and settled, then dislodged. No noticeable change was apparent in the adjacent beach to the west, and the adjacent beach to the east was protected by the Longard tube. By June 1979, the sandbags in the eroding west segment had pushed the fence and most of the posts seaward (Figs. 2-151 and 2-152). Although dislodged from their original positions, most of the Sand Pillows in this segment remained intact except where torn by excessive pressure against the fencing. The Advance Bags in the accreting east segment were deteriorating badly, much of the top layer being broken open, leaving the sand completely exposed (Figs. 2-153 and 2-154). In addition, the side seams of the bags came open as the cotton thread deteriorated. Hurricane Frederic passed over the area in early September 1979, destroying the structure and eroding the bank behind it (Fig. 2-155). Fill was placed on the bluff in November 1979 to mitigate the erosion problem. Despite the structural damage in the west half of the fence-sandbag bulkhead, that segment functioned as intended during the first 9 months, preventing erosion of the bank behind it. Toe scour and undercutting was anticipated, and the bags slumped into the scour trench as expected, halting further shore recession. The predominant eastward littoral transport in this area was revealed by the root structure of two trees about 30 feet east of the east end of the revetment, which acted as a groin. Sand eroded from the west

Figure 2-151. Sandbags dislodged from bulkhead, Basin Bayou, Florida, 22 June 1979.

Figure 2-152. View from east end of bulkhead, Basin Bayou, Florida, 22 June 1979.

Figure 2-153. Dislodged Monsanto Sand Pillows, Basin Bayou, Florida, 22 June 1979.

Figure 2-154. Deteriorated Advance Bags, Basin Bayou, Florida, 22 June 1979.

Figure 2-155. Destruction of fence-sandbag bulkhead from Hurricane Frederic, Basin Bayou, Florida, 19 September 1979.

segment of the bulkhead (and perhaps some from farther west) was trapped in a fillet extending from the trees westward to about the center of the bulkhead. This fillet of sand protected the east segment of the structure from erosion damage. The structure was destroyed in September 1979 and was subsequently never replaced. The structure, however, performed adequately under normal conditions.

(b) Sandbag Breakwater. One month after completion, the east one-third of the breakwater built with Advance Bags had lost about 1 foot of height. This was due to the relatively light weight of the bag and to the side seams (factory-sewn) on the bags coming apart. The bags did not deteriorate as much in the water as those on the fence-sandbag bulkhead. By June 1979, the segment had lost 2 to 3 feet of height, mainly because of side seams pulling apart, allowing the sand to spill out, with resultant collapse of structure. The Sand Pillows in the central segment were in good condition, but had acquired a coating of green algae (Fig. 2-156) and were slowly being dislodged, apparently by sliding over each other under wave agitation. The Dura-Bags in the western segment showed little change except for one bag which was torn and lost most of its sand (Fig. 2-157). The structure withstood Hurricane Frederic and remained structurally sound. By May 1980 the structure was still intact.

This structure functioned as intended during the first year by trapping sand along the shore in its lee. Typical profiles, compared with those of similar devices, are shown in Figure 3-76 in Section III. A 10- to 20-foot extension of the beach appeared among the stillstanding trees just offshore prior to project implementation. In fact, the resultant denial of littoral

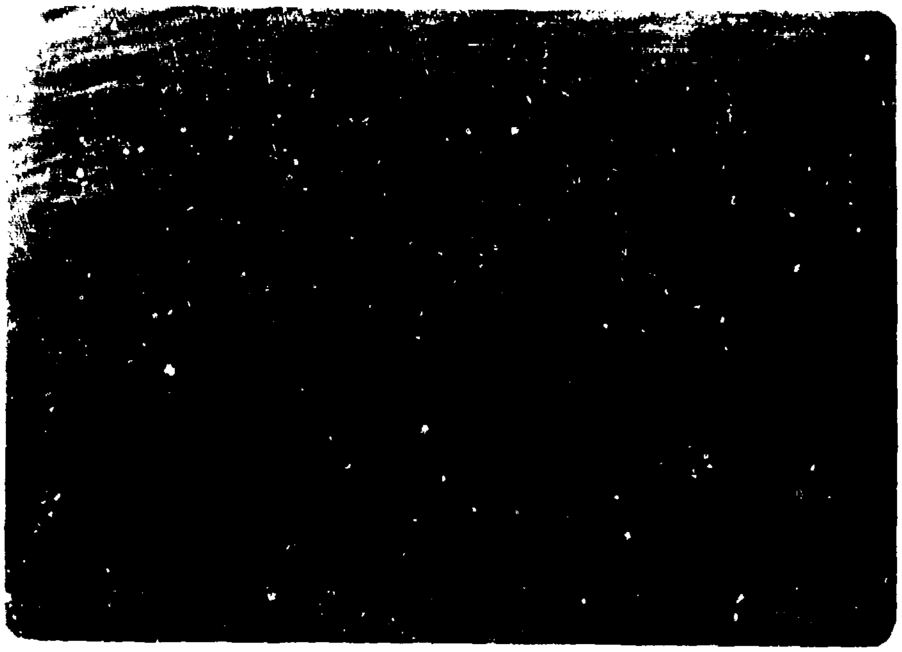


Figure 2-156. Advance Bags underwater on east end of structure, Basin Bayou, Florida, 22 June 1979.

Figure 2-157. View from offshore breakwater, looking east, Basin Bayou, Florida, 22 June 1979.

material to the downdrift shore probably accentuated the erosion at the west end of the fence-sandbag bulkhead. As of May 1980, the structure continued to trap some sand in its lee along the beach.

(c) Longard Tube. The Longard-tube breakwater remained as initially placed, with no visible changes in cross section or alinement until the summer of 1979. In a 26 July 1979 report, the District monitor noted minor vandalism damage which was no threat to the integrity of the tube. However, on 15 August 1979 the Mobile District was notified by the Basin Bayou State Park ranger that the Longard tube had been vandalized. About a 40-foot segment near the center of the tube was damaged, with sand spilled on the bottom. During the subsequent inspection (20 August 1979) it appeared that the tube had been slashed with a knife. The structure continued to deteriorate, losing sand from the middle section until it was almost completely submerged. In May 1980, the Longard tube was replaced with a new aluminum-covered tube and located just landward of the old structure (Fig. 2-158).

After the damage was done to the center of the tube, the tube slowly lost more sand, and the beach behind the breakwater was slowly eroded. This erosion continued until May 1980 when the structure was replaced. It appeared as though the structure was functionally effective as long as it was structurally sound.

Profiles through the Longard tube, compared with those through similar devices, are shown in Figure 3-75 of Section III. The profiles of January 1979 and August 1979 show the buildup of about 20 to 30 feet of beach behind

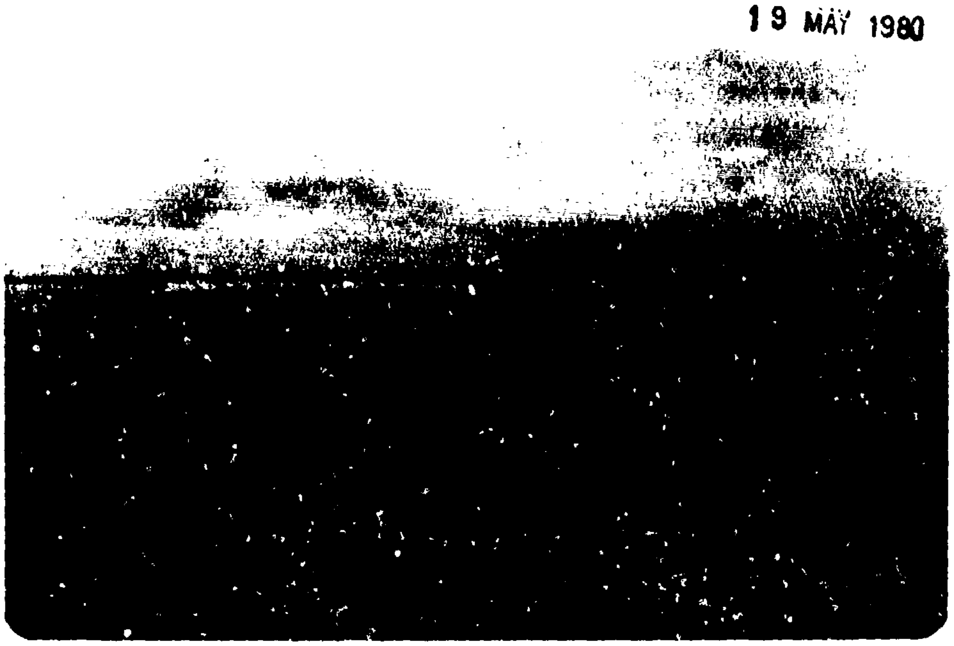


Figure 2-158. New Longard tube with aluminum cover, Basin Bayou, Florida, 19 May 1980.

the structure. The profiles of August 1979 and May 1980 show the subsequent erosion due to the vandalism of the Longard tube.

(d) Surgebreaker. This offshore reef performed well structurally. It remained in place, with no observable changes in cross section or alinement, and no degradation of structural material. Settlement was nominal, and no leaning of the structure from toe scour was apparent. As of May 1980 the structure remained intact and undamaged.

The structure functioned well during the period monitored. There was no noticeable change in the site's beach profile, and the adjacent beach remained unchanged. Erosion or accretion trends were unchanged while the structure was present. Profiles through the Surgebreaker, compared with those through similar devices, are shown in Figure 3-78 in Section III.

(e) Sandgrabber. Accretion of sand in front of and behind the middle third of the structure was noticeable the first month after installation. Erosion of sand from under the seaward side of the structure about 50 feet from the east end was also evident. A few of the steel rods connecting the blocks were beginning to rust. Two months after installation, the middle of the structure appeared to be rotating downward from the quarter points. On the east end, this was attributable to sand erosion from under the toe. A number of cracked and broken blocks were found about 50 feet from the east end, as a result of sand erosion from under the structure. Where earlier accretion had occurred, the beach had started to erode. Structural damage continued; after 4 months there were about 35 broken concrete blocks around the east quarter point and 15 broken blocks around the west quarter point, displacing some of the steel rods. Sand had built up to within 6 inches of the top of the structure about 50 feet from the west end. After about 6 months, the settlement of the central segment ceased and the structure stabilized, with little additional damage. Figures 2-159 and 2-160 show the condition of the Sandgrabber in late August 1979. In March 1980, the eastern end of the structure rotated seaward slightly, yet did not substantially damage the structure. As of May 1980, there was some slight erosion in front of the structure but it remained structurally sound.

This structure performed generally as intended, trapping a considerable amount of sand behind it. This is evident in profiles through the Sandgrabber, shown in Figure 3-79, placed in Section III for comparison with other profiles through similar devices. In the first few months, sand seemed to accumulate both inshore and just seaward of the device during high wave episodes, and to erode in front of and from under it during normal wave activity. The buildup of sand along the west end of the structure was further evidence of eastward littoral transport in this area. The small sand fillet trapped by the seaward bulge of the alinement soon reached saturation, and sand moving along the shore began to bypass the device on the seaward side. No significant erosional or accretional trends were observed as of May 1980.

(f) Vegetation. The survival of both species of plants in the exposed sections (A, B, C, and F) was very poor. The smooth cordgrass was more than half gone only a week after planting, and the midseason report shows less than 10 percent surviving. The short survival rate of these

Figure 2-159. View of Sandgrabber about 50 feet east of structure, Basin Bayou, Florida, 20 August 1979.

Figure 2-160. View while standing on Sandgrabber, looking east, Basin Bayou, Florida, 20 August 1979.

plants was due to sand shifting from one side of the beach cusp to the other and then back again, thus uprooting the plants. The saltmeadow cordgrass in the exposed sections lasted only slightly longer than the smooth cordgrass, probably because of the slightly higher planting elevation. The midseason report shows about 63 percent of saltmeadow cordgrass remaining, but by the August structure inspection (3 weeks before Hurricane Frederic) only about 25 percent remained. At the end of the growing season (October 1979), the only plants, of either species, remaining in any of the exposed sections were 34 saltmeadow cordgrass plants in section A and 58 in section B. These plants were readily identifiable and had about 10 stems per plant. Both species planted in the protected area behind the Sandgrabber (sections D and E) showed a good midseason survival rate of more than 90 percent. As the season progressed, it became apparent that the plants in section D (plan 1 fertilization; both species) were doing better than those in section E (plan 2 fertilization). At the end of the growing season, about 90 percent of both species in section D and about 60 to 70 percent in section E survived. In both sections the plants of both species were still identifiable at the end of the growing season. In section D, saltmeadow cordgrass had increased in height to about 3 feet and the number of stems had increased to about 20 to 30 per plant. The smooth cordgrass plants in section D were also about 3 feet high, with about 10 to 15 stems per plant. Both species of plants in section E were smaller and had fewer stems per plant than those in section D; saltmeadow cordgrass plants were 20 to 30 inches high with 10 to 20 stems per plant, and smooth cordgrass plants were about 30 inches high with 5 to 10 stems per plant.

In May 1980, all of the smooth cordgrass was apparently dead in sections D and E behind the Sandgrabber. A small amount of saltmeadow cordgrass survived in the unprotected sections (A and B) and in sections D and E. Shifting sands had uprooted many plants and in places had covered them completely.

(5) Analysis.

(a) Fence-Sandbag Bulkhead. The installation demonstrated the vulnerability of sandbags to tear open by pressure and sliding against the hogwire fencing when undercut by toe scour. The factory-sewn seams of the Advance Bags also failed, but not as quickly as in the sandbag breakwater. Where shore recession is in progress, toe scour should be anticipated. Damage might have been prevented by placing the bottom bags in a trench excavated to at least the depth of the anticipated scour. This would require a higher wall and therefore entail additional cost but may be necessary to avert failure. In the east segment where no scour occurred, the bags stayed in section as initially placed and successfully protected the bluff. Had the bags been ultraviolet-resistant, they might have survived undamaged.

In the erosion damaged segment, many posts were found leaning seaward from back pressure. This could have been prevented by tiebacks, but the pressure would still have damaged the bags. A smaller mesh fabric could be the answer, and a PVC-coated wire mesh, such as that used for gabion baskets, might suffice. The PVC coating is smoother and will not cut the bags as easily. Another alternative would be to fill the bags with sand-cement to avoid reliance on the bag fabric for structural integrity.

Hurricane Frederic destroyed this structure, clearing away most of the sandbags and eroding the bank behind the structure. This type of structure cannot be expected to withstand hurricanes, and therefore it may be inadvisable for use in hurricane-prone areas.

(b) Sandbag Breakwater. Bursting of the seams of the Advance Bags indicated a basic weakness that might be overcome by an improved bag construction technique. Although the Monsanto Sand Pillows individually held their integrity, the slumping due to sliding lessened their effectiveness. The sliding was probably facilitiated by the stacking pattern and growth of algae the bags acquired, but the sliding might have occurred even if the algae had not formed. Experimentation with other stacking patterns appears advisable. Wave action on the breakwater appeared to be too great for these relatively lightweight bags. The Dura-Bag segment performed well except for tears, which spilled sand. This is one of the hazards of sandbag construction that cannot be avoided. The test demonstrated that where large pieces of floating debris are prevalent, or where vandalism is anticipated, the sandbag breakwater may have a relatively short life.

(c) Longard-Tube Breakwater. This device maintained its integrity well until partially destroyed by vandals. Although the outer fabric is quite strong and resists the elements well, it is easily cut with a sharp knife. The tube size attracts vandals because the tube, when cut, explodes in a cascade of spilling sand. The manufacturer recommends a sand-epoxy coating for the exposed areas of the tube to deter vandals, but this coating can only be applied in the dry. The aluminum covering on the new tube was intended to eliminate destruction by vandals; however, it also proved unsuccessful and eventually had to be removed because it was hazardous to bathers.

The tube functioned exceptionally well until damaged. The sandspit that formed in its lee provided an attractive recreational beach while it lasted. The breakwater may have been sited too close to shore, as the continued extension of the spit could have resulted in a tombolo, which would have interrupted all littoral drift and starved the downdrift coast. The shape of the spit indicated a net resultant wave approach angle strongly skewed to the west. Aerial photos revealed that sand waves on the bay bottom in this region may be contributing to the sandspit.

(d) Surgebreaker. This offshore reef performed well for the period of observation. Beach profiles indicate no noticeable changes along the shore in lee of the structure or along adjacent beaches.

(e) Sandgrabber. This device also performed reasonably well, both structurally and functionally. Contrary to the manufacturer's recommendation, the test indicated that where toe scour is anticipated, the structure should be entrenched, rather than built on the existing bottom and left to settle into the sand to a stable position. One alternative would be to place the structure on a bed of quarrystone to prevent settlement. Other than the central part of the structure dropping to a less functional depth, the settlement does not appear to have lessened its functional effectiveness.

The apparent use of substandard-strength concrete blocks from the manufacturer was claimed by the contractor to be the contributing cause of the cracking of many blocks. Tests by the District Engineer proved that the concrete strength was above standards for concrete building blocks. These tests, however, were conducted 6 to 9 months after installation, leaving more time for the concrete to cure. Nevertheless, cracking occurred during construction when the modules were lifted, and this suggests that substandard-strength concrete may have been used in some blocks. Because of the severe stresses to which a Sandgrabber may be subjected when settling unevenly, block strength should be tested before installing the device at any site.

(f) Vegetation. The sandy soil and exposed location of this site seem to preclude the successful establishment of unprotected cordgrass plantings. Bank and beach erosion are so severe during storms, especially hurricanes which are frequent at this site, that the plants are either uprooted or buried before they can become firmly established. Even if a stand were to persist through a few seasons it seems unlikely that it would significantly halt erosional loss from the nearby banks. The deposition and shifting of large masses of sand depend to a considerable extent on this erosion from the bank.

When protection is provided, as in the Sandgrabber, there is a reasonable possibility for successful establishment of plantings, but apparently only of saltmeadow cordgrass. The growth of these plants was fairly good considering the storms they sustained; however, the plants seem to have little effect in retarding sand movement and thereby controlling erosion. Sand was repeatedly deposited and removed by waves overtopping the Sandgrabber. The plants were consequently exposed, at times to the root level, or buried. It seems unlikely that vegetation even in conjunction with a structure will provide much long-term protection from erosion at this site. Tree-shaded plantings at this site are futile.

(g) Volume Calculations. Table 2-28 is a compilation of volume calculations for changes between profile stations that occurred from the first survey in June 1977 to the last survey in May 1980. The base line for the surveys, which shows the stationing (in east-to-west order), is shown in the general plan (Fig. 2-136). The littoral transport direction is from northwest to southeast. The table shows that sand accumulates in regions behind the offshore breakwaters. The fact that net erosion occurred behind each end of the Surgebreaker is a reflection of the late installation of that structure. The shoreline in the Surgebreaker section eroded during most of the timespan as a result of material removed from the littoral system by the Longard-tube breakwater. Most of the regions between the breakwaters are eroding. The offshore region at the fence-sandbag bulkhead also eroded, probably reflecting the long-term trend at this site. The Sandgrabber adequately protected the beach and induced accretion along that region of the beach. The regions around the vegetation at station 2+00 also accreted during the monitoring period, perhaps because of sand released to littoral transport when the Longard tube was not functioning.

c. Fontainebleau State Park, Louisiana.

(1) Site Description.

(a) Geographical Setting. The Fontainebleau State Park demonstration site is located about 30 miles north of New Orleans and 2

Table 2-28. Volumetric changes at Basin Bayou, Florida (28 June 1977 to 13 May 1980).

Device	Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
--	0+50	344.7	191.8	-152.9
--	1+00	146.4	210.0	63.7
Vegetation	1+50	91.0	302.8	211.8
Vegetation	2+00	74.9	262.2	187.3
Vegetation	2+50	88.0	206.3	118.3
Vegetation	3+00	157.7	204.7	47.0
Vegetation	3+50	268.6	202.6	-66.0
Vegetation	4+00	195.8	226.1	30.3
Vegetation	4+50	84.8	191.3	106.5
Sandgrabber	5+00	84.5	146.4	61.9
Sandgrabber	5+50	53.4	181.9	128.5
Sandgrabber	6+00	52.9	187.7	134.8
Sandgrabber	6+50	56.1	179.8	123.8
Sandgrabber	7+00	99.3	139.2	39.8
Sandgrabber	7+50	81.2	241.1	159.9
--	8+00	58.9	607.8	549.0
--	8+50	126.0	517.1	391.1
--	9+00	193.4	188.0	-5.5
--	9+50	195.3	181.9	-13.4
Surgebreaker	10+00	166.5	215.3	48.8
Surgebreaker	10+50	158.1	236.1	78.0
Surgebreaker	11+00	164.5	210.7	46.2
Surgebreaker	11+50	187.4	148.7	-38.7
--	12+00	207.5	118.2	-89.3
--	12+50	261.5	85.5	-176.0
--	13+00	250.2	40.9	-209.3
--	13+50	125.2	108.0	-17.2
Longard tube	14+00	144.8	244.6	99.8
Longard tube	14+50	166.5	314.0	147.5
Longard tube	15+00	90.0	290.1	200.0
Longard tube	15+50	240.1	131.2	-108.8
Longard tube	16+00	402.5	6.3	-396.2
Fence sandbag	16+50	332.2	46.8	-285.4
Fence sandbag	17+00	216.8	95.5	-121.2
Fence sandbag	17+50	174.1	130.7	-43.4
Fence sandbag	18+00	145.3	122.1	-23.1
--	18+50	276.5	89.6	-186.9
--	19+00	443.6	58.0	-385.6
--	19+50	380.6	59.6	-321.0
Sandbag B/W	20+00	175.5	144.5	-31.0
Sandbag B/W	20+50	62.8	247.6	184.7
Sandbag B/W	21+00	67.4	293.4	-226.1
Sandbag B/W	21+50	58.2	246.1	187.9
Sandbag B/W	22+00	54.1	163.2	109.1
--	22+50	44.5	128.2	83.7
--	23+00	78.6	296.5	217.9
--	24+00	265.5	987.3	721.8
--	26+00			
	Totals	7793.2	9827.2	2034.0

miles east of Mandeville, Louisiana, on the northeast shore of Lake Pontchartrain. The project involves about 3,000 feet of shoreline between the park pavilion and Green Point (Fig. 2-161). The park has 2,605 acres of land, with more than 2 miles of lake shoreline. The shoreline is straight and bears approximately N. 20° W. Existing public facilities include recreational picnic units, a swimming pool, pavilions, and a nature trail.

(b) Climate. The climate is subtropical, having mild winters and hot, humid summers. Normal annual temperatures average about 68° Fahrenheit.

(c) Waves, Tides, and Longshore Transport. Tides in Lake Pontchartrain are diurnal with a mean range of 0.6 foot. Mean low, mean, and mean high tides are +0.9, +1.2, and +1.5 feet NGVD, respectively. Southeasterly winds predominate in the area, generating waves that approach almost parallel to shoreline, causing littoral transport to the northwest. Records of wave data for the site do not exist; however, observations in the vicinity of the site indicate that the breaking wave heights are limited by the water depth and generally do not exceed a height of about 3 feet under normal conditions. During major storms and hurricanes, wind setup often floods the north shore area, and the higher waves generated in the lake reach the site before breaking. The highest observed storm tide in the site area was +7.7 feet NGVD. It is estimated that winds combined with storm tides can produce wave heights of 4 to 6 feet.

The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 2.5 feet. Net longshore transport potential northward was 17,200 cubic yards for the 3 months analyzed. The wave climate is classified as mild.

(d) Geomorphology, Soils, and Vegetation. The site area lies within the Deltaic Plain of the Mississippi Alluvial Plain physiographic section (a flat, low-aggrading alluvial plain which received deltaic sedimentation from the Mississippi River). However, the Mississippi River no longer flows into Lake Pontchartrain except when the Bonnet Carre Spillway is open during floods.

Surface soils in the park generally consist of soft Holocene deposits of dark-gray organic silts and clays. These deposits range from 2 to 13 feet in depth and are overlain by 2 to 5 feet of fine to medium, poorly graded sand. The offshore sediments generally consist of organic clays with occasional lenses of silts and sands near the surface.

Sand beaches are from 30 to 50 feet wide with elevations varying from 3 to 5 feet. The landforms behind the beach are relatively featureless marshes and lie at about an elevation of +1 foot. Water depths are about 1 to 2 feet immediately offshore, and the lake bottom slopes at 1 on 150 for a distance of at least 1,000 feet from the shore.

(e) The Problem. The Fontainebleau site is a typical example of an erosional problem resulting from two processes--direct attack of storm waves and littoral transport. Erosion along the 3,000-foot-long shoreline has been estimated at approximately 3 to 5 feet per year since 1898. Some local beach areas have experienced erosion rates as high as 9

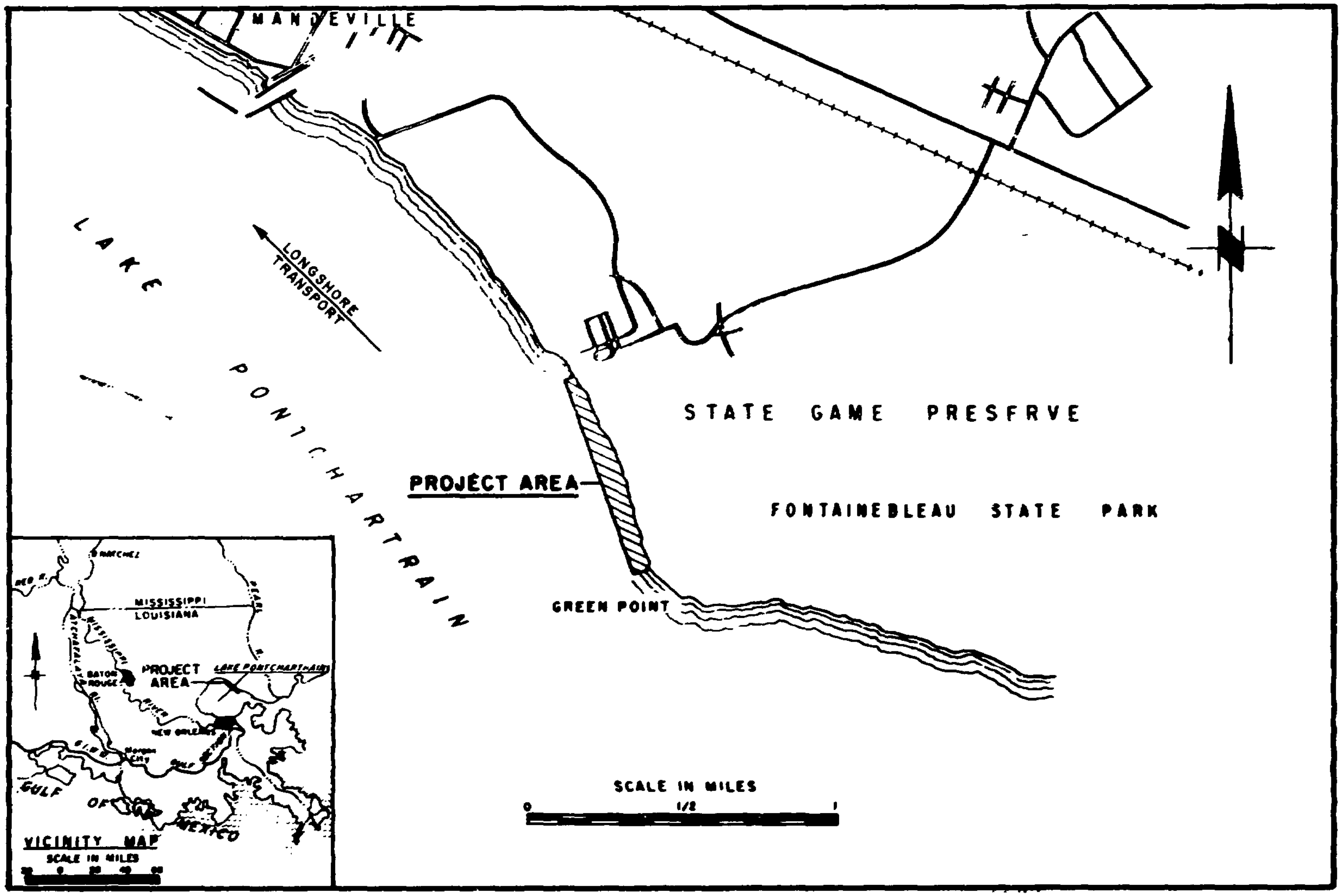


Figure 2-161. Location map of Fontainebleau State Park, Louisiana, demonstration site.

feet per year. An artificial beach constructed in 1942 initially extended approximately 300 feet into the lake from the shore; it was almost entirely eroded by 1968.

Prevailing winds from the southeast generate waves which approach from a direction almost parallel to shoreline. This sets up a strong littoral current transporting beach materials to the northwest. In addition, erosion is aggravated by storm (and hurricane) waves that reach shore before breaking. During storms, beach material may be washed inland to the marsh or offshore into the lake, the latter resulting in offshore bars. During these conditions the underlying organic silts and clays may be exposed and subject to further erosion by wave action. Louisiana has many miles of marsh shoreline protected by fringe sand- shell beaches, where effective low-cost shore protection methods could have many applications.

(2) Demonstration Project. The devices demonstrated at Fontainebleau State Park include structural and vegetative devices located both onshore and offshore. The installations are shown in Figures 2-162 and 2-163. Devices 1 to 5 and 7 to 9 abut one another. Devices 6, 10, 11, and 12 are isolated. Device 13 is a vegetative planting scheme (Fig. 2-163).

(a) Device 1 (Double Erco-Mat Revetment). This double-mat revetment consists of two back-to-back layers of 13-pound Erco blocks, factory-bonded to a common filter-cloth carrier strip. The dimensions of individual Erco blocks are 8 by 8 by 4 inches. A large, presewn 55- by 40-foot filter-cloth panel was placed on the graded sand beach before placement of the double mats. The filter cloth used in this and other revetment devices at the Fontainebleau site is Nicolon 66301. Details of the device 1 revetment are shown in Figure 2-164. The double mats were placed by dragline.

(b) Device 2 (Erco-Block Revetment). The 13-pound Erco blocks (also known as Gobi blocks) in this revetment were placed by hand (Fig. 2-164) on a 55- by 40-foot presewn filter-cloth panel (Nicolon 66424).

(c) Device 3 (Jumbo-Mat Revetment). The blocks used in this revetment were 115-pound Jumbo blocks, similar to Erco blocks but with larger dimensions (24 by 16 by 6 inches) than those used in devices 1 and 2. They were factory-bonded in a single layer to carrier strips of woven Filterweave filter cloth. Details of the revetment are shown in Figure 2-164. The mats were placed by dragline on a 55- by 40-foot filter-cloth panel.

(d) Device 4 (Standard Construction Block Revetment). The dimensions of standard blocks used were 16 by 8 by 8 inches; the blocks were placed by hand with the hollows facing up (Fig. 2-165) on a 55- by 40-foot filter-cloth panel (Nicolon 66301).

(e) Device 5 (Erco-Mat Revetment). This revetment consisted of a single layer of Erco blocks, factory-bonded to 20- by 4-foot woven carrier strips (Fig. 2-165). The mats were placed on a 55- by 40-foot filter-cloth panel (Filterweave 70A).

(f) Device 6 (Tire/Timber-Pile Breakwater). Located 50 feet lakeward of the mean lake shoreline is a tire/timber-pile breakwater

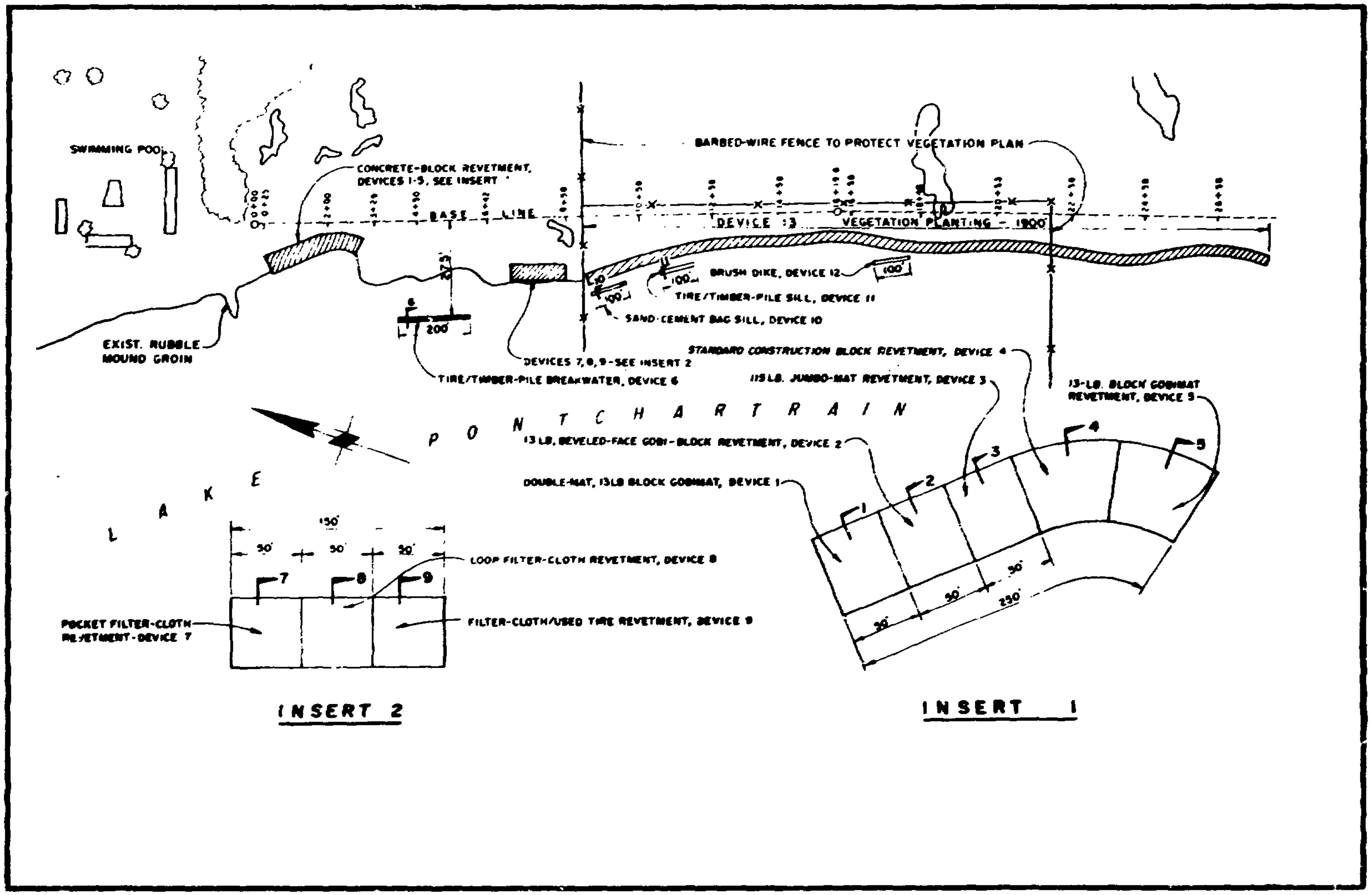
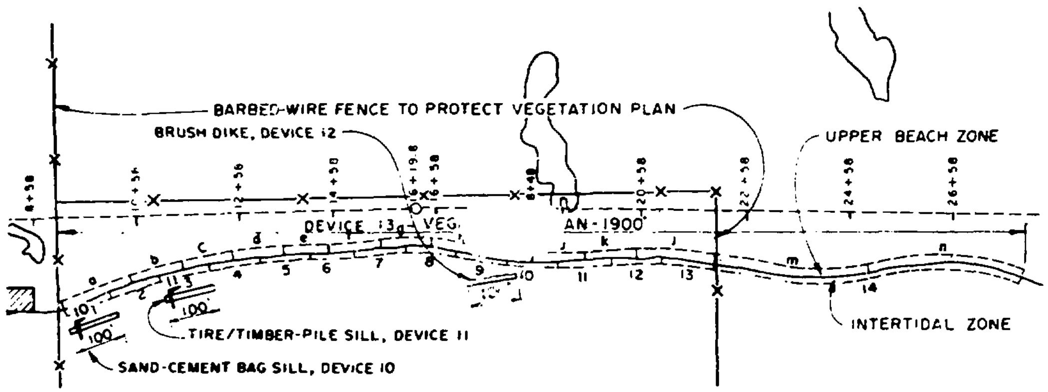


Figure 2-162. General plan of Fontainebleau State Park, Louisiana, demonstration site.



KEY TO DEVICE 13 – VEGETATION PLAN

INTERTIDAL ZONE

SMOOTH CORDGRASS – MLW TO MHW

SECTION NO	*TREATMENT	SECTION LENGTH (ft)
1	STANDARD PLANTING + PROTECTION	100
2	STANDARD PLANTING	100
3	STANDARD PLANTING + PROTECTION	100
4	SPACED 18 INCHES ON CENTER	100
5	SPACED 36 INCHES ON CENTER	100
6	NO PLANTING (UNTREATED CONTROL)	100
7	PLUGS	100
8	STANDARD PLANTING	100
9	STANDARD PLANTING	100
10	STANDARD PLANTING	100
11	STANDARD PLANTING	100
12	STANDARD PLANTING	100
13	PEAT-POT SEEDLINGS	100
14	NO FENCES	600

* STANDARD PLANTING = SPRIGS SPACED 24 INCHES ON CENTER, EXCEPT AS NOTED.

UPPER BEACH ZONE

SELECTED VEGETATIVE SPECIES
MHW TO HIGH SPRING OR STORM TIDE

SECTION NO.	TREATMENT	SECTION LENGTH (ft)
a	COMMON REED	100
b	NO PLANTING	100
c	TORPEDO GRASS **	100
d	SALTMEADOW CORDGRASS	100
e	SALTMEADOW CORDGRASS - SELECTION 1	100
f	SALTMEADOW CORDGRASS - SELECTION 2	100
g	SALTMEADOW CORDGRASS - SELECTION 3	100
h	TORPEDO GRASS **	100
i	COMMON REED	100
j	SALTMEADOW CORDGRASS - SELECTION 1	100
k	NO PLANTING	100
l	TORPEDO GRASS **	150
m	NO PLANTING	300
n	SALTMEADOW CORDGRASS + McCARTNEY ROSE	300

** NATURALLY GROWING

Figure 2-163. Vegetation scheme at Fontainebleau site.

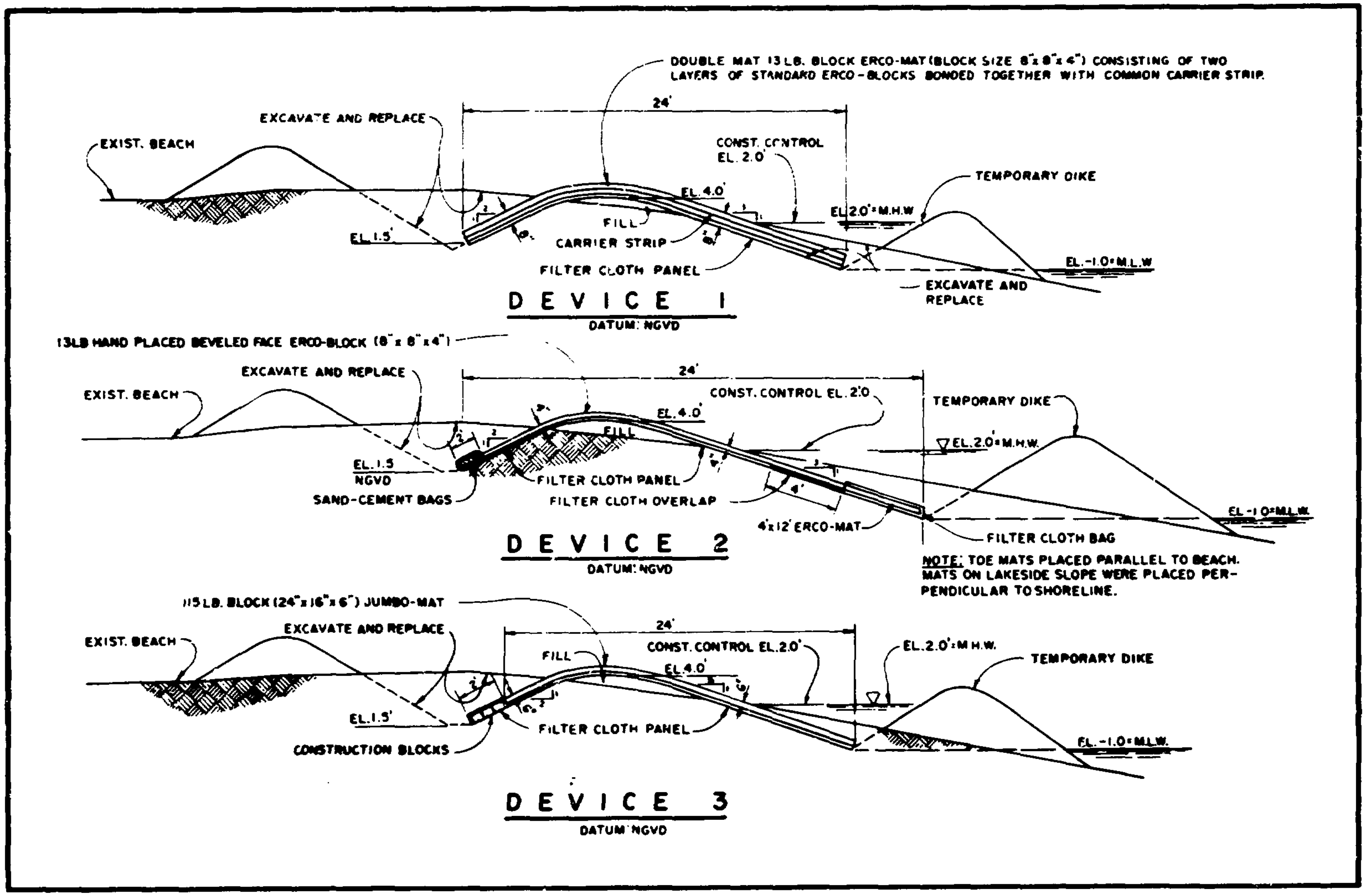


Figure 2-164. Devices 1, 2, and 3 at Fontainebleau site.

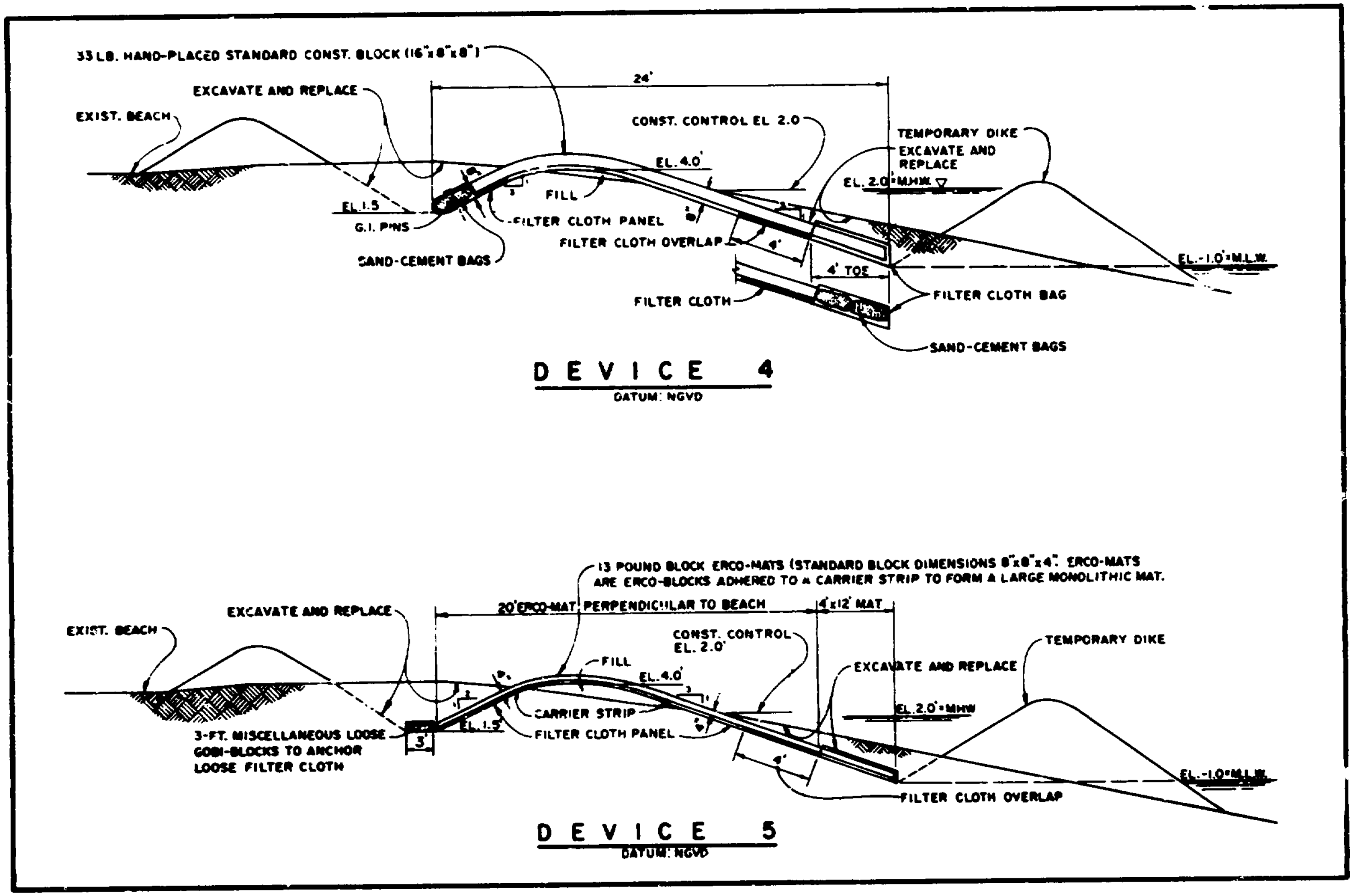


Figure 2-165. Devices 4 and 5 at Fontainebleau site.

(Fig. 2-162). This 200-foot-long device consists of six variations of timber-pile configuration (Fig. 2-166). The vertical piles were stacked with used automobile tires; half of the breakwater had filter-cloth protection against bottom scour. The filter cloth underneath the structure consisted of several types, including Bidim and Nicolon. Alternative methods of connecting the piles with one another (Fig. 2-166) were also used.

(g) Device 7 (Pocket Filter-Cloth Revetment). Device 7, constructed using pocket filter cloth (Nicolon 66424), is shown in Figure 2-162. Pocket filter cloth has large, presewn ballast pockets to anchor the filter-cloth panel. The two outer rows of pockets were filled with sand-cement bags, and the remaining rows were filled with shell. To complete the revetment, the cloth was covered with 6 inches of shells, which in turn were covered with 6 inches of topsoil. The revetment was then seeded with coastal Bermuda grass and fertilized. Details are shown in Figure 2-167.

(h) Device 8 (Loop Filter-Cloth Revetment). Abutting device 7 is a loop filter-cloth (Nicolon 66424) revetment (presewn loops attached at regular intervals along the top surface). Ballast blocks can be attached to the loops to anchor the cloth until it is covered with shells and topsoil, which is then planted, as in device 7. The 55- by 40-foot filter cloth was anchored by four rows of Jumbo blocks (115 pounds each) and rows of sand-cement bags. Instead of attaching the Jumbo blocks to the loops, reinforcing rods (galvanized iron pins) were driven through the holes of the large blocks. Details are shown in Figure 2-167.

(i) Device 9 (Used-Tire/Filter-Cloth Revetment). Immediately adjacent to device 8 is a filter-cloth revetment constructed of grout-filled used tires placed on top of a 55- by 40-foot filter-cloth (Nicolon 66424) panel (Fig. 2-167).

(j) Device 10 (Sand-Cement Bag Sill). Three offshore structures (devices 10, 11, and 12) were installed to reduce wave action along vegetation segments 1, 3, 9, and 10 (Fig. 2-162). Device 10, fronting vegetation segment 1, consists of a sand-cement bag sill. A continuous piece of filter cloth was placed on the existing bottom along the axis of the low sill structure. Details are shown in Figure 2-168.

(k) Device 11 (Rolling Tire Breakwater). A rolling tire/timber-pile breakwater was constructed in front of vegetation segment 3. In contrast to device 6, the piles of device 11 are oriented horizontally and anchored to the bottom by screw anchors and only truck tires are used (Fig. 2-169).

(l) Device 12 (Brush Dike). The structure fronting vegetation segments 9 and 10 is a brush dike (Fig. 2-168).

(m) Device 13 (Vegetation). To compare different methods of planting and types of vegetation, a reach of 1,900 feet was subdivided into sections for the vegetation plan (Fig. 2-163). Initial inspection of the site showed considerable evidence of cattle grazing, so a barbed-wire fence was installed landward of the base line, and enclosing 1,300 feet of the vegetation plot.

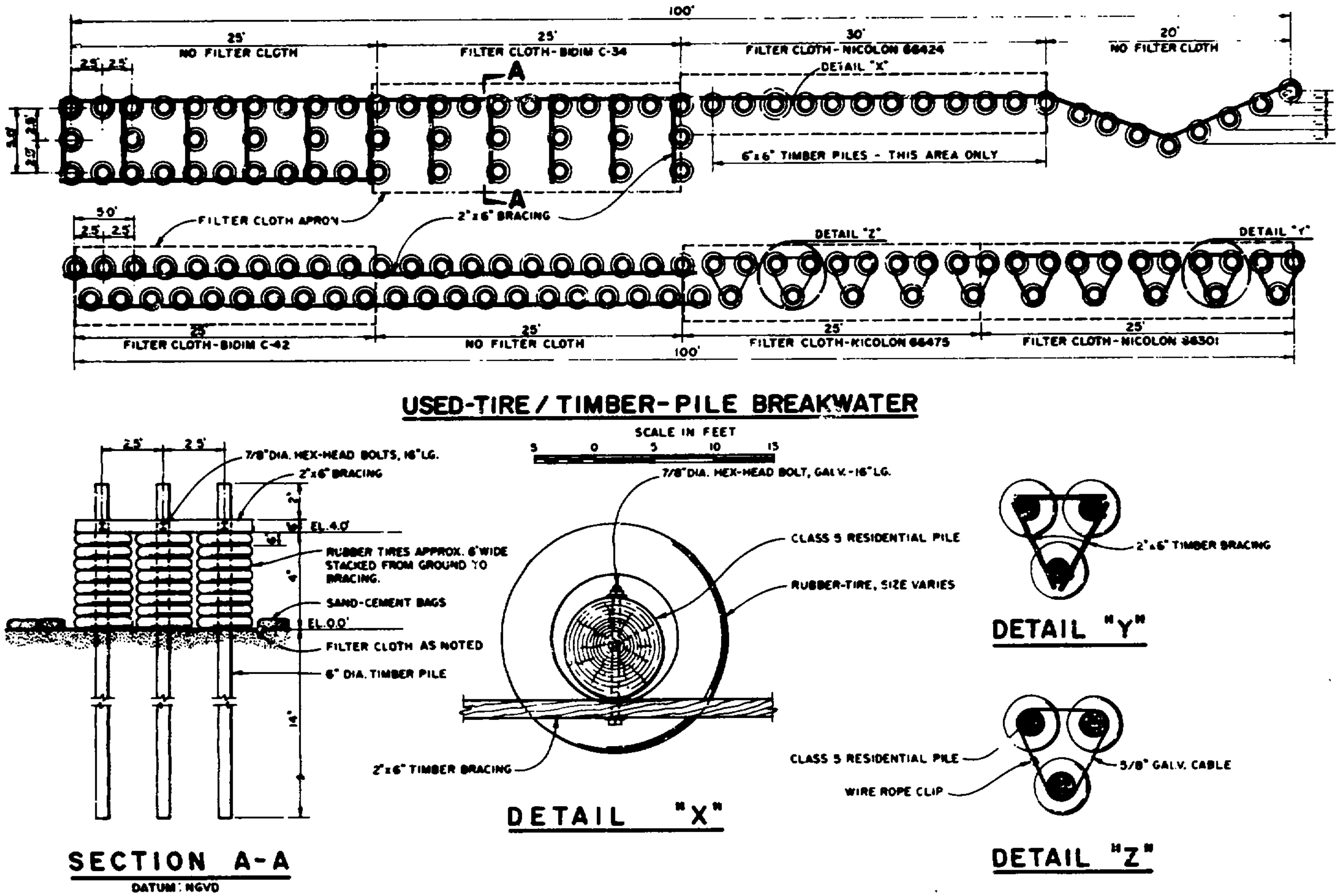


Figure 2-166. Device 6 at Fontainebleau site.

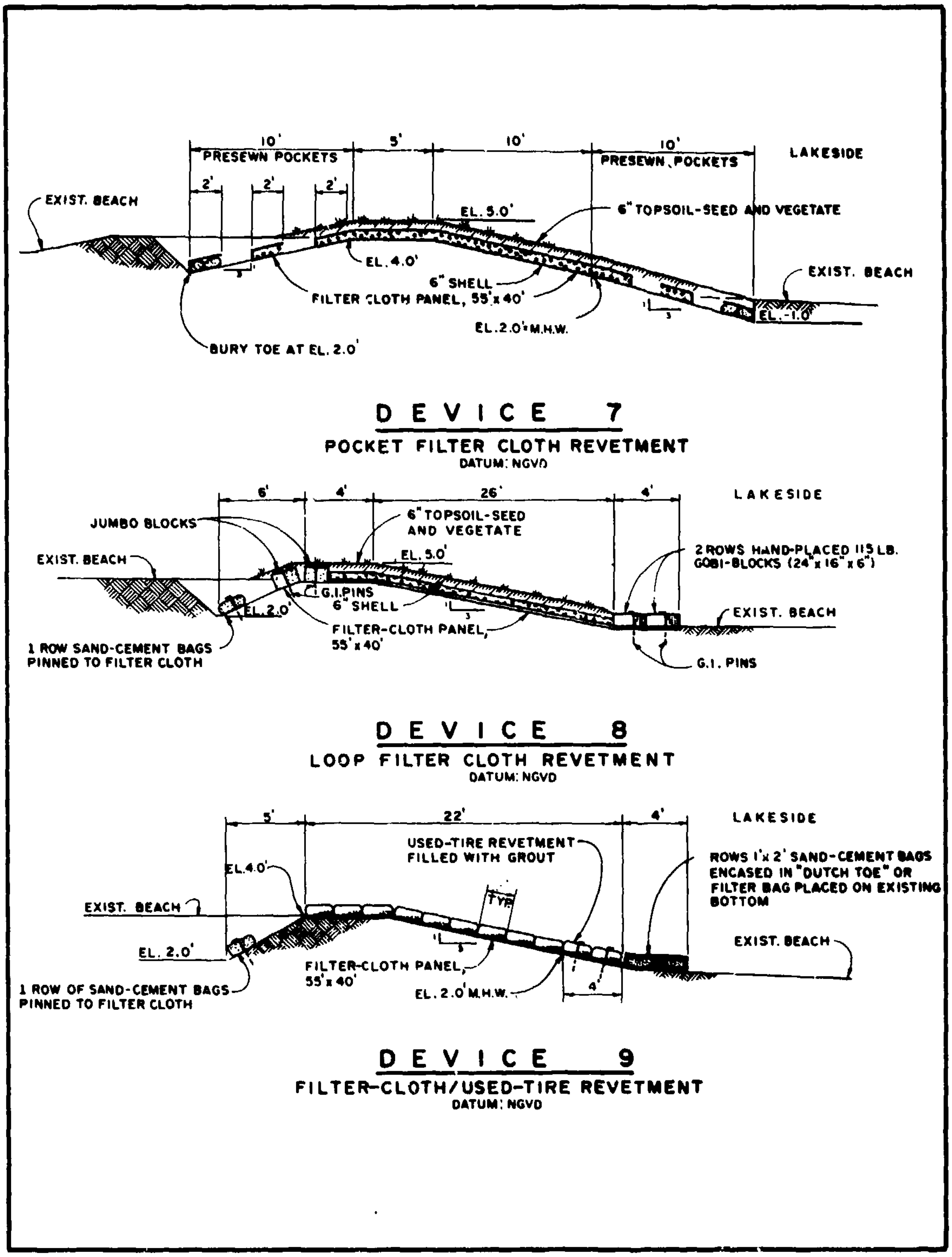


Figure 2-167. Devices 7, 8, and 9 at Fontainebleau site.

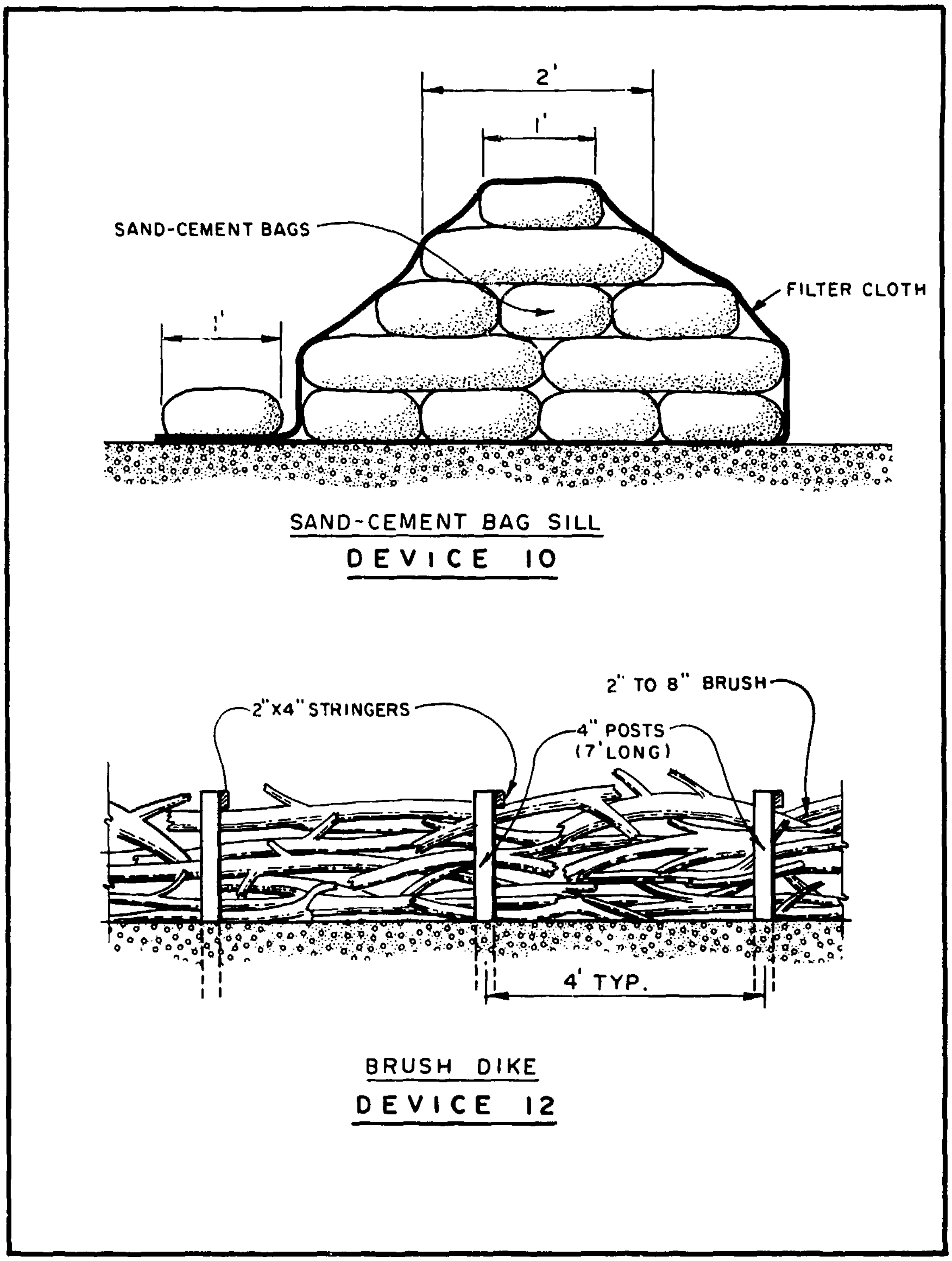


Figure 2-168. Devices 10 and 12 at Fontainebleau site.

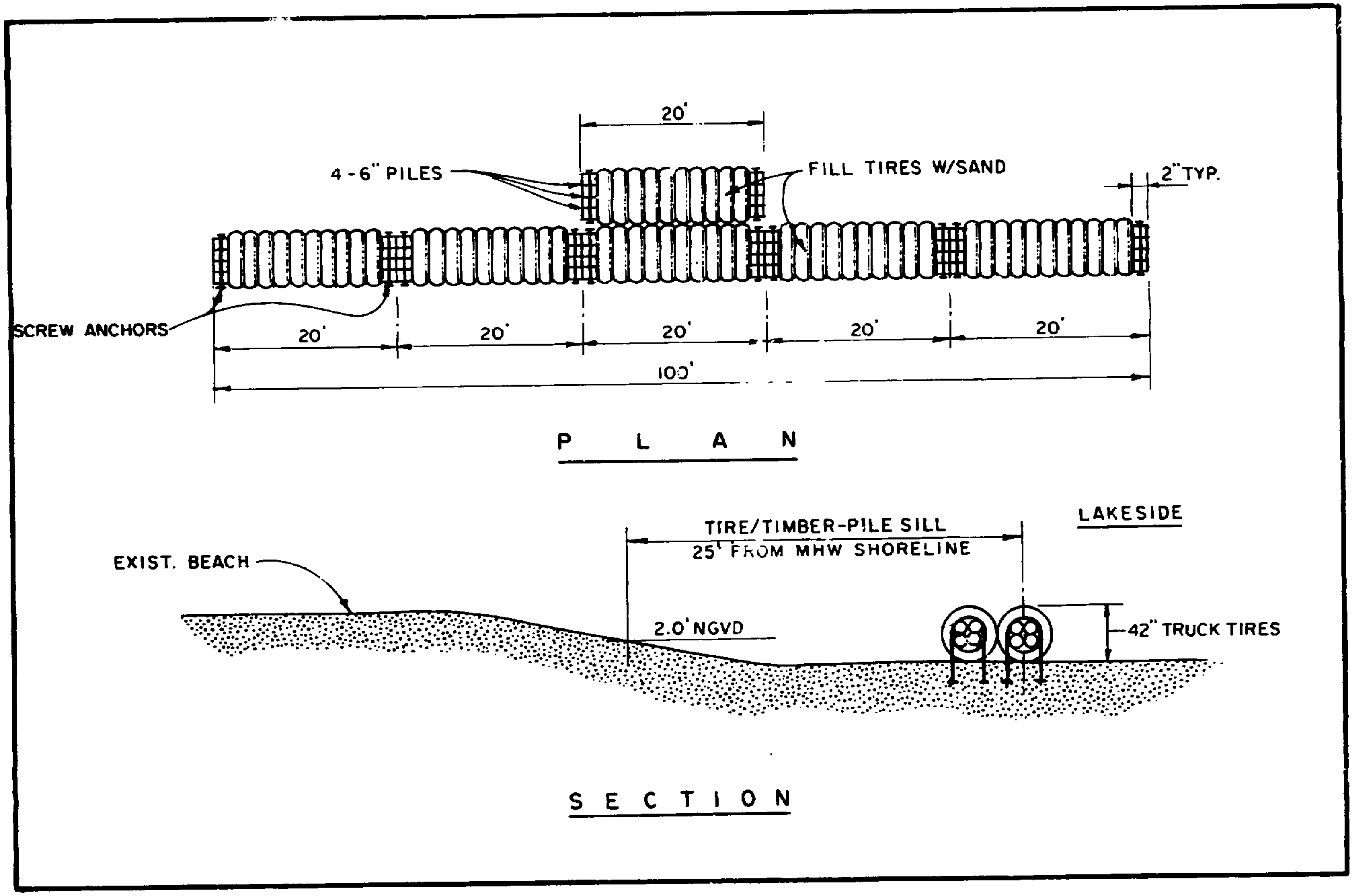


Figure 2-169. Device 11 at Fontainebleau site.

(3) Installation and Performance. Construction data, performance, and costs of each device at Fontainebleau State Park, Louisiana, are given in this subsection. Construction procedures for the revetments (devices 1 to 5 and 7 to 9) were alike in that a bulldozer or crawler tractor was used to prepare the slope before placement of the filter cloth. After the revetments were in place, the bulldozer was used again to place the backfill. All structures except the brush dike were installed during the fall of 1979. The brush dike was constructed in the spring of 1979; major repairs to structures 2 and 4 were made in October 1979 and April 1980, respectively.

(a) Device 1 (Double Erco-Mat Revetment).

1 Statistics, Construction, and Cost. The double Erco-mats (Gobi-mats) were placed over the filter cloth on the prepared slope. The Erco-mats were lifted from a delivery truck after clamping the ends of the carrier strip in a spreader bar. The spreader bar was suspended from a boom from a dragline of sufficient capacity and reach to allow the mats to be placed without additional handling. Construction was completed 9 November 1979. Costs are given in Table 2-29.

Table 2-29. Costs for double Erco-mat revetment.

Item	Quantity	Cost
Materials		
Nicolon 66301 filter cloth	2,200 ft^2	$ 396
4- by 12-ft double mats	24 ea.	3,963
Misc.		942
Equipment		
Crawler tractor (including operator)	12 hr	345
3/4 yd^3 dragline (including operator)	12 hr	150
Labor	34 man-hours	256
Total		$6,052
Cost per foot		$ 121
Cost per foot excluding labor		$ 116

2 Performance. Some settlement occurred during the first winter of the monitoring period but no major changes in cross section were apparent. Soon after construction, the mat settled irregularly as shown in Figure 2-170. On 13 April 1980, a storm hit the area. The structure suffered no damage, although the lakeward edge of the mat was settling as a result of toe scour. The unprotected north end had subsided slightly before the storm, but the storm waves apparently did not aggravate this settlement (Fig. 2-171). The sand that had accumulated on the crown of the device was removed and washed over the access road by the storm. This structure was still intact by June 1980.

Figure 2-170. Photo shows a drop of approximately 10 inches (near sign) of the Erco-mat of device 1 and the protruding filter-cloth carrier strip used for placing the double mats, Fontainebleau State Park, Louisiana, 4 January 1980.

Figure 2-171. Subsidence of the north end of device 1, Fontainebleau State Park, Louisana, 8 April 1980.

(b) Device 2 (Erco-Block Revetment).

1 Statistics, Construction, and Cost. This hand-placed Erco-block revetment featured a Dutch toe. Two smaller panels of pocket filter cloth were substituted for the regular filter panel with dimensions of 55 feet by 40 feet with overlap joints parallel to the shoreline. The Dutch toe was formed by placing 4- by 12-foot Erco-mats inside the bulb formed by the filter cloth along the lakeward edge or toe. The pockets along the landward edge were filled with sand-cement bags. The revetment Erco blocks were placed with the aid of a long straightedge and a string line. First, a line of blocks was placed along the straightedge parallel to the beach. Next, several rows of blocks were positioned adjacent to those already in place to establish a series of lines perpendicular to the beach. The string line was then stretched along one of the perpendicular lines to extend the blocks up the slope. Gaps were later filled in with individual blocks or pieces of block. Construction was completed 8 November 1979. Cost data are given in Table 2-30.

Table 2-30. Costs for beveled-face, Erco-block revetment.

Item	Quantity	Cost
Materials		
Nicolon 66424 filter cloth	2,200 ft^2	$ 440
4- by 12-ft Erco-mats	4 ea.	637
Sand-cement bags	25 ea.	50
Shells	6 yd^3	65
Erco blocks	2,630 ea.	920
Misc. items		456
Equipment		
Crawler tractor (including operator)	12 hr	345
3/4 yd^3 dragline (including operator)	12 hr	150
Labor	72 man-hours	541
Total		$3,604
Cost per foot		$ 72
Cost per foot excluding labor		$ 61

2 Performance. Several blocks were removed from the revetment during the first 6 months of monitoring. First, a 6-square foot section of blocks was removed by vandals, leaving the filter cloth intact (Fig. 2-172). The wave action of the April 1980 storm displaced a 3- by 7-foot area of blocks. By May 1980, approximately 65 square feet of blocks was either loose or missing (Fig. 2-173). Despite the missing blocks in this structure, progressive damage had not occurred to the adjacent blocks as of June 1980. A storm in May 1980, which caused a lake stage of about 4.7 feet NGVD, did not damage the structure.

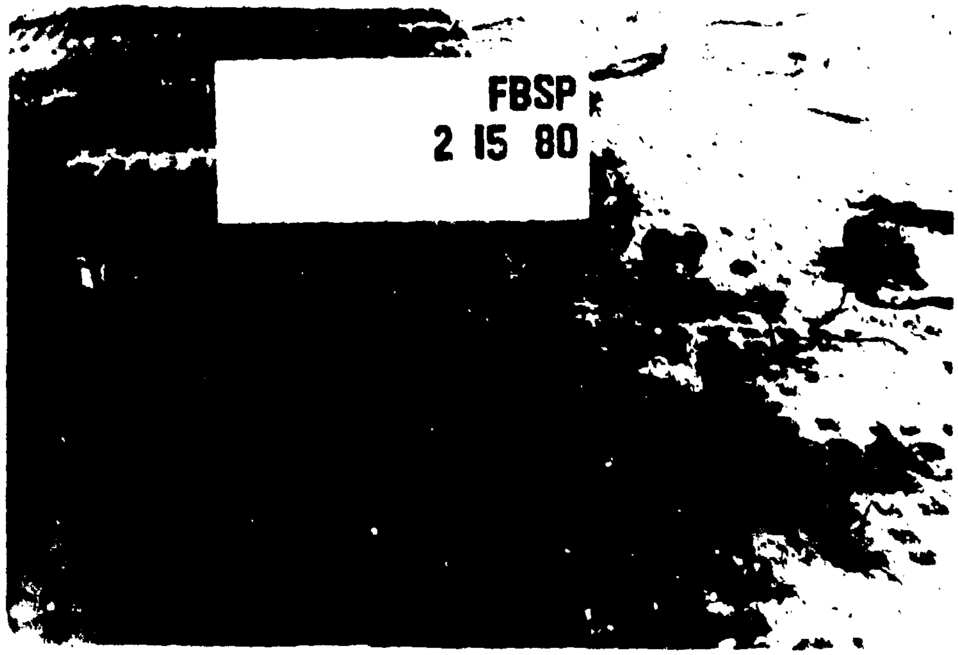


Figure 2-172. A small section of blocks removed from device 2, Fontainebleau State Park, Louisiana, 15 February 1980.

Figure 2-173. Scattered areas of blocks are missing after storm, device 2, Fontainebleau State Park, Louisiana, 9 May 1980.

(c) Device 3 (Jumbo-Mat Revetment).

1 Statistics, Construction, and Cost. The Jumbo-mat revetment consisted of 115-pound Jumbo blocks glued to a filter-cloth carrier strip and placed by dragline on the filter-cloth panel. Placement of the mats was similar to that of the double mats of device 1. Construction was completed on 7 November 1979. Costs are given in Table 2-31.

Table 2-31. Costs for Jumbo-mat revetment.

Item	Quantity	Cost
Materials		
Nicolon 66424 filter cloth	2,200 ft^2	$ 440
4- by 12-ft Jumbo-mats	24 ea.	3,216
Concrete construction blocks	72 ea.	55
Shells	6 yd^3	65
Misc.		815
Equipment		
Crawler tractor (including operator)	12 hr	345
3/4 yd^3 dragline (including operator)	12 hr	150
Labor	34 man-hours	256
Total		$5,342
Cost per foot		$ 107
Cost per foot excluding labor		$ 102

2 Performance. A month after construction, pieces of the carrier-strip filter cloth were protruding up between the mats (Fig. 2-174). The carrier strip is an integral part of the underside of each mat, and each mat has extra strips at each end for placing the mat. The revetment suffered no structural damage during the April 1980 storm, but the southern lakeward toe settled into the lake bottom. The objective of using filter panels and articulated mats is to allow the revetment to settle uniformly. Also, some of the construction blocks used to anchor the landward slope of the filter-cloth panel were exposed as a result of wave action washing the sand toward the marsh. As of June 1980, this structure had performed very well with no structural damage.

(d) Device 4 (Standard Construction Block Revetment).

1 Statistics, Construction, and Cost. The hand-placed construction block revetment also contained a Dutch toe. In this case, the toe was formed by placing sand-cement bags on the filter cloth, which in turn rested on the prepared beach slope and extended lakeward of the bags. The excess cloth was then lapped over the bags to form a Dutch toe. Another row of bags was placed on the filter cloth at the landward

Figure 2-174. Photo shows the Jumbo-mat revetment, device 3, in the foreground. Note the filter-cloth carrier strip used for placing the mats, Fontainebleau State Park, Louisiana, 4 December 1979.

edge of the revetment. Here, because the cloth lap was shorter, steel pins were driven through the bags and the filter cloth. The construction blocks were then placed on the remaining exposed filter cloth with their long axes perpendicular to the beach. Construction was completed 6 November 1979. Cost data are given in Table 2-32.

2 Performance. In January 1980, approximately 250 square feet of blocks was removed by vandals (Fig. 2-175). On 9 April 1980, the structure was repaired; missing blocks were replaced and a minimum of 1 foot of shell was placed over the blocks to fill the void spaces and to seat the revetment. During the 13 April 1980 storm, several blocks of the repaired section were displaced to the landward side of the revetment. The filter-cloth Dutch toe settled unevenly into the lake bottom. The northern section of the filter cloth was below the existing beach, and the southern section was exposed (Fig. 2-176). A minor tear on a seam running perpendicular to the shore was probably due to uneven lateral stresses. In April 1980, the revetment was repaired in the section of displaced blocks; however, the replaced blocks were mistakenly oriented with their long axes parallel to the beach. About half of the blocks used were displaced again, indicating that block orientation may be a factor in the structural integrity of construction block revetments.

Figure 3-42, which was put in Section III for comparison of profiles through similar devices, shows a series of profiles through the construction block revetment.

Figure 2-175. A large area of blocks were removed from device 4, Fontainebleau State Park, Louisiana, 4 January 1980.

Figure 2-176. After the storm, blocks were displaced and the southern end of the filter cloth was exposed, Fontainebleau State Park, Louisiana, 9 May 1980.

Table 2-32. Costs for standard construction block revetment.

Item	Quantity	Cost
Materials		
Concrete construction block	1,360 ea	$1,047
Nicolon 66301 filter cloth	2,200 ft^2	396
Steel pins	80 ea.	138
Sand-cement bags	160 ea	323
Shells	6 yd^3	65
Misc.		425
Equipment		
Crawler tractor (including operator)	12 hr	345
3/4 yd^3 dragline (including operator	12 hr	150
Labor	62 man-hours	466
Total		$3,355
Cost per foot		$ 67
Cost per foot excluding labor		$ 58

(e) Device 5 (Erco-Mat Revetment).

1 Statistics, Construction, and Cost. This Erco-mat revetment was also constructed with a Dutch toe. A two-layer row of Erco-mats was placed along the lakeward edge of the filter cloth, parallel to the beach. To form the Dutch toe, the filter cloth was lapped over this row of Erco-mats. The remaining Erco-mats (20 by 4 feet) were placed perpendicular to the beach. Construction was completed 30 October 1979. Cost data are given in Table 2-33.

2 Performance. After construction, sand began accumulating slowly on the lakeward side of the revetment. Some scour occurred at the north end adjacent to device 4, while sand accumulated near the south end adjacent to the grass-protected dune (Fig. 2-177). After the storm in April 1980, a part of the large filter panel on the north side of the Dutch toe was floating free, and the toe was exposed by erosion. As of June 1980, the structure was still intact and performing very well.

(f) Device 6 (Tire/Timber-Pile Breakwater).

1 Statistics, Construction, and Cost. Construction of the tire/timber-pile breakwater began by placing several different filter cloths and driving stakes through the cloth. Two rows of sand-cement bags were placed on the cloth along the lakeward edge and one row of bags was placed along the landward edge. The piles were driven with a pneumatic hammer suspended from swinging leads. Stacks of tires which had been banded together were delivered to the site and placed on the piles.

Table 2-33. Costs for Erco-mat revetment.

Item	Quantity	Cost
Materials		
4- by 20-ft Erco-mats	12 ea.	$2,256
4- by 12-ft Erco-mats	4 ea.	637
Filterweave 70-A filter cloth	2,360 ft^2	354
Shells	6 yd^3	65
Misc.		715
Equipment		
Crawler tractor (including operator)	20 hr	575
3/4 yd^3 dragline (including operator)	16 hr	200
Labor	32 man-hours	241
Total		$5,043
Cost per foot		$ 101
Cost per foot excluding labor		$ 96

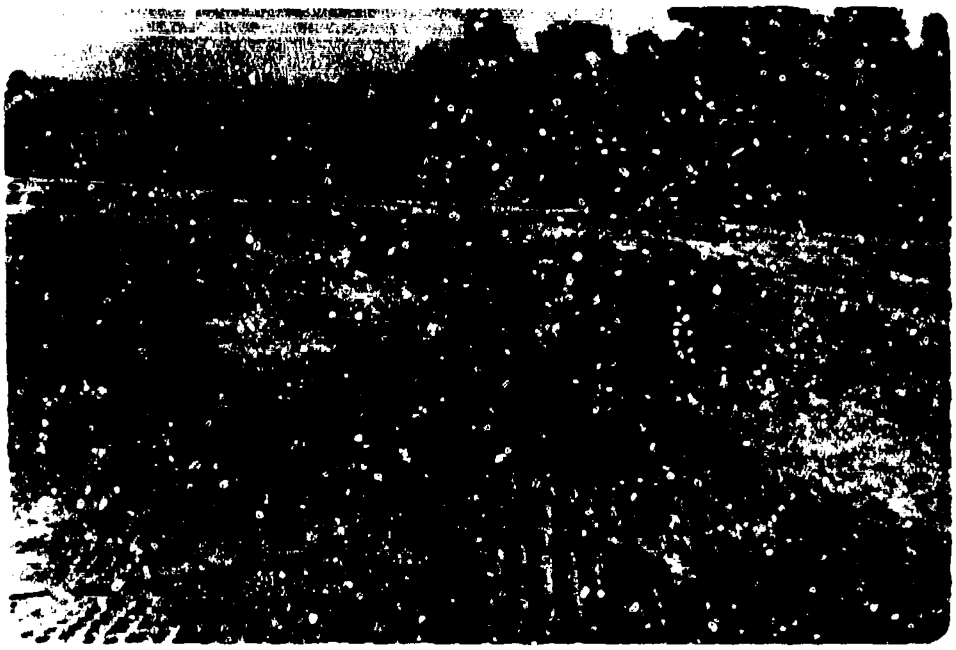


Figure 2-177. Photo shows sand accumulated near the south end of device 5, Fontainebleau State Park, Louisiana, 13 March 1980.

However, the prebanding operation was suspended because of the difficulty of handling the heavy stacks. Tires were then placed singly from a platform built on a crawler tractor. Timber wales and cables, used to fasten the piles together (Fig. 2-178), were installed with the aid of a small aluminum boat. Construction was completed on 2 November 1979. Cost data are given in Table 2-34.

Table 2-34. Costs for tire/timber-pile breakwater.

Item	Quantity	Cost
Materials		
30-ft-long creosoted piles	138 ea.	$4,416
2- by 6-in creosoted lumber	1,076 lin ft	611
Bidim C-42 filter cloth	325 ft^2	72
Bidim C-34 filter cloth	435 ft^2	44
Nicolon 66475 filter cloth		165
Nicolon 66424 filter cloth		
Nicolon 66301 filter cloth		
Used tires	1,060 ea.	-----[1]
Sand-cement bags	150 ea.	303
Shells	20 yd^3	216
Misc.		1,258
Equipment		
Crawler tractor (including operator)	104 hr	2,990
3/4 yd^3 dragline (including operator)	112 hr	1,400
Air compressor	80 hr	130
Labor	572 man-hours	6,452
Total		$18,084
Cost per foot		$ 91
Cost per foot excluding labor		$ 59

[1]No cost assigned to tires; charges insignificant.

2 Performance. One month after construction, the tires began sinking into the sandy bottom of the lake. Some sand was removed from the headland between devices 5 and 6, and the beach to the south receded (Fig. 2-179). The functional performance of the breakwater was very good. Profiles through the tire-on-poles breakwater were placed in Section III with profiles through similar devices for comparison. Figure 3-75 shows accretion in lee of the breakwater. Wave heights on the lakeward side were estimated at 3 to 4 feet, and the breakwater attenuated these heights to 1 to 2 feet in its lee. Wave diffraction around the outer ends combined with the wave attenuation formed a sandbar in the protected area, with sand trapped on both sides of the construction road causeway. By June 1980, a significant amount of sand had been trapped in the lee of the breakwater (Fig. 2-180).

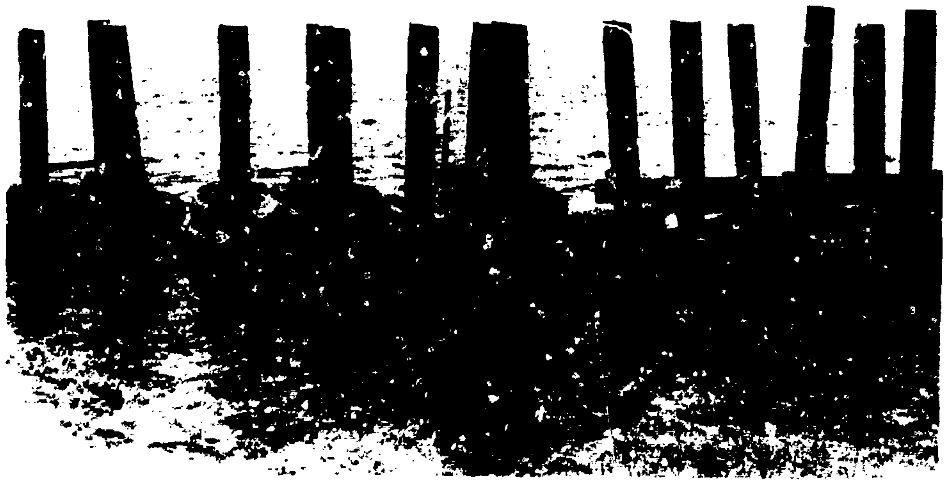


Figure 2-178. Piles fastened together with cables and timber wales, device 6, Fontainebleau State Park, Louisiana, 15 February 1980.

Figure 2-179. The beach south of device 6 has receded, Fontainebleau State Park, Louisiana, 8 April 1980.

(g) Device 7 (Pocket Filter-Cloth Revetment).

1 Statistics, Construction, and Cost. The pocket filter cloth was positioned after the tractor had shaped the beach. Two sand-cement bags were placed in each pocket of the lakeward and landward edges. Shells were placed on the filter cloth and were shoveled into the remaining pockets. Additional shells were placed on the filter cloth and smoothed before topsoil was placed over the shells. The topsoil was then smoothed and seeded, and the backfill on the landward trench was placed by a dragline. Construction was completed 17 October 1979. Cost data are given in Table 2-35.

2 Performance. Two months after completion, wave action removed the shell and topsoil from the bottom 12 feet of the lakeward slope and some of the filter cloth was exposed (Fig. 2-181). During the winter, wave action gradually removed the lakeward half of the soil cover, and the storm of 13 April 1980 removed the remainder. Sections of filter cloth that were not buried below the existing beach were exposed to wave action, debris, sunlight, and possible vandalism. Some sand was lost underneath the panel because the sides were not buried, but the filter cloth was found to be intact (Fig. 2-182). By May 1980, most of the filter cloth was exposed (Fig. 2-183).

Table 2-35. Costs for pocket filter-cloth revetment.

Item	Quantity	Cost
Materials		
Nicolon 66424 pocket filter cloth	2,200 ft^2	$1,100
Sand-cement bags	100 ea.	202
Shells	16 yd^3	173
Topsoil	16 yd^3	104
Misc.	16 yd^3	341
Equipment		
Crawler tractor (including operator)	20 hr	575
3/4 yd^3 dragline (including operator)	16 hr	200
Labor	70 man-hours	526
Total		$3,221
Cost per foot		$ 64
Cost per foot excluding labor		$ 54

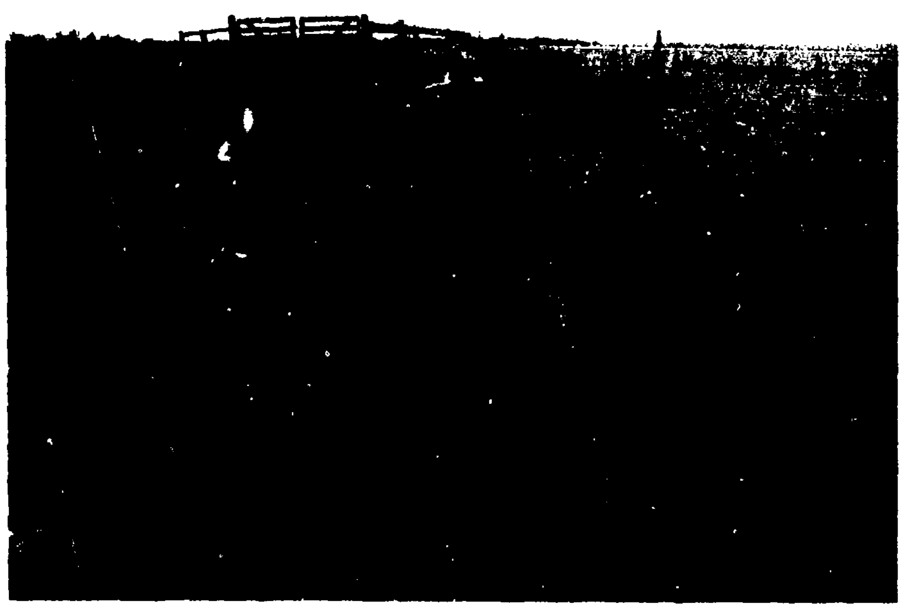


Figure 2-181. Exposed filter cloth on device 7, Fontainebleau State Park, Louisiana, 15 February 1980.

Figure 2-182. Device 7 under wave attack, Fontainebleau State Park, Louisiana, 8 April 1980.

Figure 2-183. Exposed filter cloth on device 7, Fontainebleau State Park, Louisiana, 9 May 1980.

(h) Device 8 (Loop Filter-Cloth Revetment).

1 Statistics, Construction, and Cost. After the filter cloth was positioned, pallets of Jumbo blocks were unloaded and placed alongside the cloth with a dragline. The blocks were lifted from the pallets to the proper position along the lakeward edge of the filter cloth with the aid of lift bars. Two men, each equipped with a lift bar, carried one Jumbo block. While shells were scattered over the remaining exposed filter cloth, sand-cement bags were placed along the back edge of the cloth. Steel pins were driven through the four rows of blocks and one row of sand-cement bags as the shell layer was smoothed with hand rakes. Topsoil was scattered over the shell, using a dragline, and then smoothed and seeded. Backfill was placed with the aid of a dragline to minimize damage to the marsh behind the structure. Construction was completed 17 October 1979. Cost data are given in Table 2-36.

Table 2-36. Costs for loop filter-cloth revetment.

Item	Quantity	Cost
Materials		
Jumbo blocks	155 ea.	$ 736
Nicolon 66392 filter cloth	2,200 ft^2	440
Shells	16 yd^3	173
Topsoil	16 yd^3	104
Steel pins	155 ea.	267
Misc.		713
Equipment		
Crawler tractor (including operator)	24 hr	690
3/4 yd^3 dragline (including operator)	16 hr	200
Labor	148 man-hours	1,113
Total		$4,436
Cost per foot		$ 89
Cost per foot excluding labor		$ 66

2 Performance. A few months after installment, the topsoil and shell were removed from the bottom 10 feet of the lakeward slope. The filter cloth was exposed in some areas at the time of initial visit (Fig. 2-184), but the cloth later appeared to be covered with sand and shells (Fig. 2-185). The steel pins used to fasten the blocks to the filter cloth began to rust after 3 months. The storm of 13 April 1980 damaged the structure considerably. Two rows of blocks along the crest of the revetment were displaced by wave action, causing loss of sand underneath the filter panel and growth of roseau cane. The regrowth of roseau cane was sufficient to lift individual Jumbo blocks; therefore, the southern

Figure 2-184. Filter cloth beginning to expose behind device 8, Fontainebleau State Park, Louisiana, 4 January 1980.

Figure 2-185. Filter cloth no longer exposed, device 8, Fontainebleau State Park, Louisiana, 13 March 1980.

edge of the lakeward toe was higher and less stable. Wave action induced uplift pressures underneath the filter panel and overturned several blocks due to loss of sand from beneath the filter-cloth panel. Shell and topsoil were completely removed after the storm and the filter cloth was largely exposed (Fig. 2-186).

(i) Device 9 (Tire/Filter-Cloth Revetment).

1 Statistics, Construction, and Cost. A 55- by 40-foot presewn filter cloth was positioned on the prepared slope, and two rows of sand-cement bags were placed parallel to the lakeward edge of the cloth. The filter cloth was then lapped over the bags, forming a Dutch toe, before tires were placed on it. Galvanized L-shaped pins were driven through the filter cloth on the interior of the bottom rows of tires. The tires were filled with a dry sand-cement mixture, and a row of bags was placed along the back edge of the filter cloth. Galvanized pins were driven through these bags and the underlying filter cloth. A small gas-powered pump was used to spray water from the lake onto the dry mixture. Construction was completed 4 October 1979. Cost data are given in Table 2-37.

2 Performance. Two months after construction, the spaces between the bottom three rows of tires were filled with sand (Fig. 2-187). Vegetation sprouted in between the tires, but the survival rate was low. The Dutch toe settled as a result of toe scour and wave action. This is the purpose of the Dutch toe. After the storm of 13 April 1980, 50 percent of the tires were displaced by wave action (Fig. 2-188). As of June 1980, the structure was still intact, but a scarp had developed at the shoreline. At this point, the structure still protected the shore despite its damaged condition.

(j) Device 10 (Sand-Cement Bag Sill).

1 Statistics, Construction, and Cost. After the filter-cloth base of the low sill was positioned, pallets of nylon-reinforced paper bags filled with sand-cement were lifted to a convenient position with the aid of a dragline. The bags were stacked by hand during a high tide. The filter cloth on the northern end was lapped over the bags before an additional row of bags was placed on top to hold the cloth. The southern 35 feet of sill had no filter-cloth overl[illegible] or Dutch toe configuration, as shown in Figure 2-168. Construction was completed 28 September 1979. Cost data are given in Table 2-38.

2 Performance. The top row of bags (not covered by filter cloth) was displaced from the sill by wave action. Most of the nylon-reinforced paper bags were degradated from the exposed area (Fig. 2-189) as per design. The 13 April 1980 storm eroded the beach, moving the beach crest approximately 8 feet toward the marsh. This structure was not damaged by the storm, and as of June 1980 the sand-cement bags remained intact.

Figure 2-186. Storm-damaged revetment, with displaced blocks and washed-away topsoil exposing the filter cloth, device 8, Fontainebleau State Park, Louisiana, 9 May 1980.

Table 2-37. Costs for tire/filter-cloth revetment.

Item	Quantity	Cost
Materials		
Used tires	170 ea.	--[1]
Sand-cement bags	330 ea.	$ 667
Galvanized-iron pins	90 ea.	248
Nicolon 66424 filter cloth	2,200 ft^2	440
Misc.		293
Equipment		
Crawler tractor (including operator)	24 hr	690
3/4 yd^3 dragline (including operator)	16 hr	200
Labor	190 man-hours	1,429
Total		$3,967
Cost per foot		$ 79
Cost per foot excluding labor		$ 51

[1]No cost assigned to tires; changes insignificant.

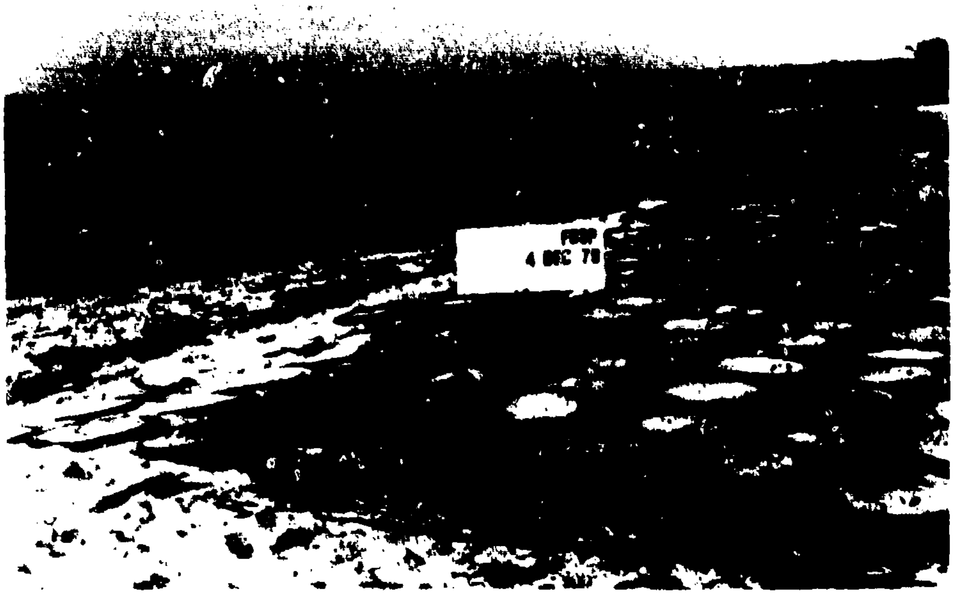


Figure 2-187. Sand had filled the voids between tires along the bottom three rows of device 9, Fontainebleau State Park, Louisiana, 4 December 1979.

Figure 2-188. After the storm, the tires and Dutch toe were displaced, device 9, Fontainebleau State Park, Louisiana, 9 May 1980.

Table 2-38. Costs for sand-cement bag sill.

Item	Quantity	Cost
Materials		
Sand-cement bags (nylon-reinforced)	1,140 ea.	$2,303
Nicolon 66424 filter cloth	1,950 ft^2	390
Carthage Mills G.B. filter cloth	210 ft^2	56
Misc.		886
Equipment		
Crawler tractor (including operator)	12 hr	460
3/4 yd^3 dragline (including operator)	24 hr	300
Labor	60 man-hours	451
Total		$4,846
Cost per foot		$ 48
Cost per foot excluding labor		$ 44

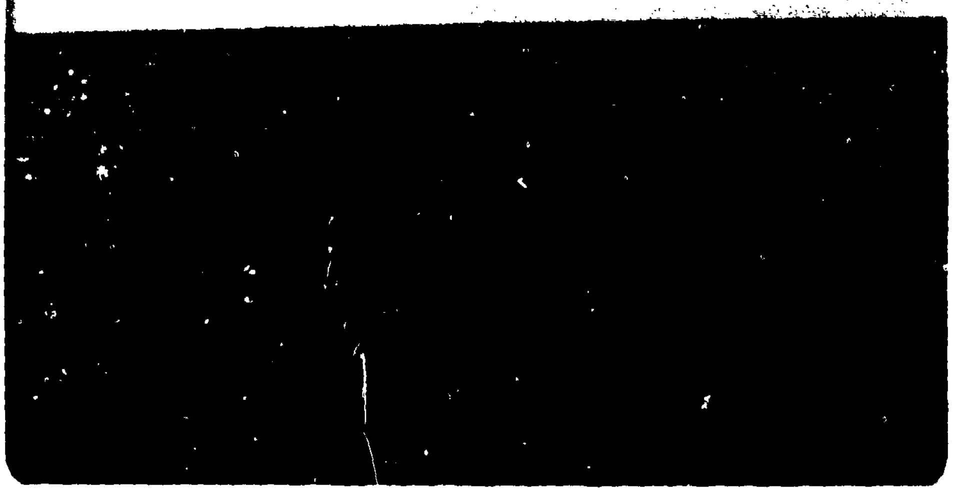


Figure 2-189. The nylon-reinforced bags removed from device 10, Fontainebleau State Park, Louisiana, 4 January 1980.

(k) Device 11 (Rolling Tire Breakwater).

1 Statistics, Construction, and Cost. The six modules (20 feet long) of the rolling tire/timber-pile breakwater were brought to the site preassembled, and positioned with the aid of a dragline. Two galvanized screw anchors were driven into the lake bottom at each end of each element. The anchors were then tied to the elements with 0.5-inch steel cable and clamps. Construction was completed 16 October 1979. Cost data are given in Table 2-39.

Table 2-39. Costs for rolling tire breakwater.

Item	Quantity	Cost
Materials		
Used truck tires (20 in or larger)	120 ea.	\$-----[1]
30-ft-long creosoted piles	24 ea.	768
0.5-in galvanized cable	1,000 ft	296
0.5-in galvanized cable clamps	75 ea.	73
Galvanized screw anchors	12 ea.	82
Misc.		263
Equipment		
Crawler tractor (including operator)	12 hr	345
3/4 yd^3 dragline (including operator)	24 hr	300
Labor	308 man-hours	2,316
Total		\$4,443
Cost per foot		44
Cost per foot excluding labor		21

[1]No cost assigned to tires; charges insignificant.

2 Performance. The units moved shoreward 5 feet from their original positions in the first 7 months. Several of the galvanized-steel anchors were bent. A low sandbar formed behind the breakwater during the first 3 months (Fig. 2-190); another sandbar also formed in the shape of a crescent (Fig. 2-191) as a result of wave diffraction around the end of the breakwater. The wave action of the storm in April 1980 deposited sand inside the tire rims, which prevented flotation of the breakwater. The middle two rows of tires settled into the lake bottom where wave scour and wave diffraction around the ends loosened the screw anchors and caused the breakwater alinement to became concave to the shore. By June 1980, at least three screw anchors had pulled out.

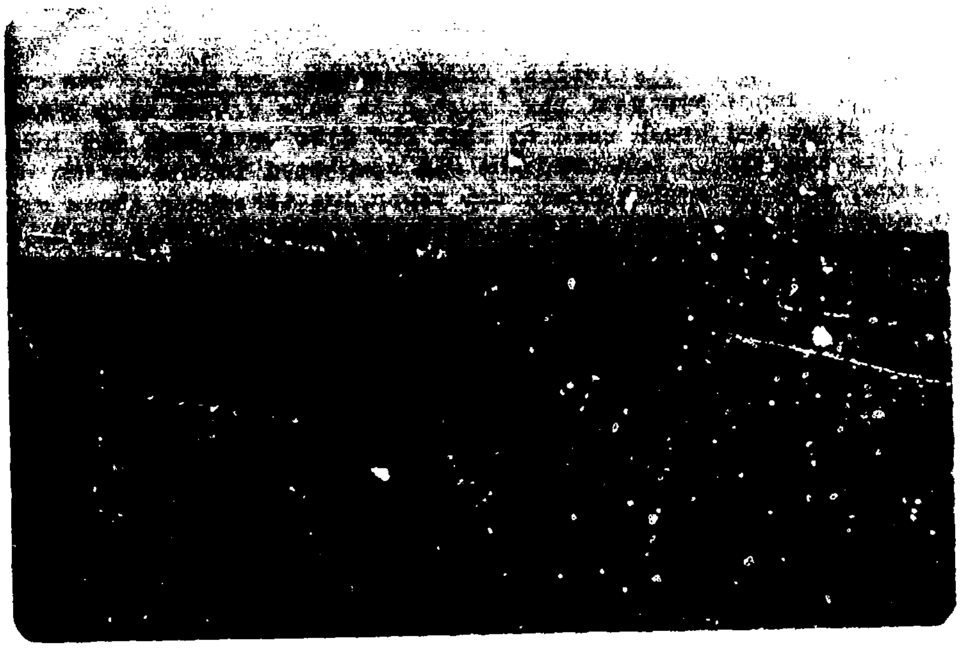


Figure 2-190. A sandbar began forming behind device 11, Fontainebleau State Park, Louisiana, 4 January 1980.

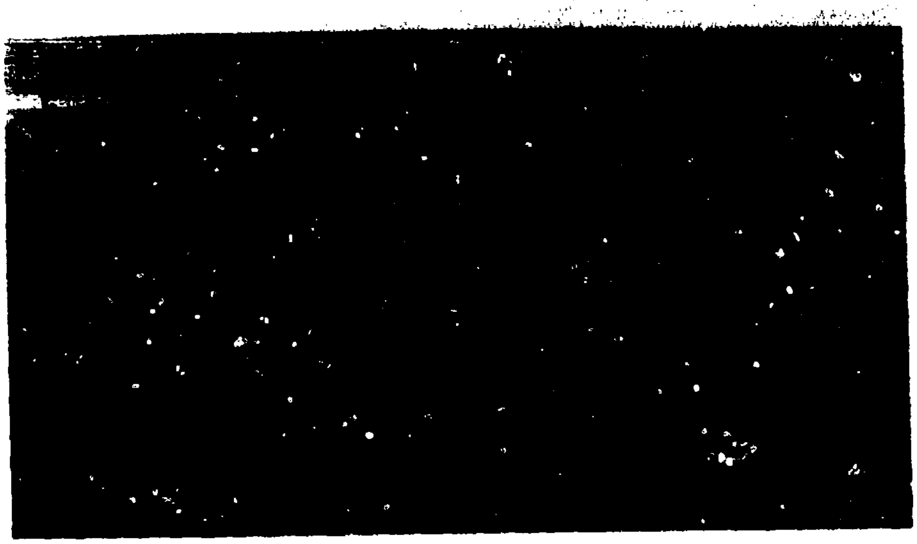


Figure 2-191. A cresent-shaped sandbar formed behind device 11, Fontainebleau State Park, Louisiana, 13 March 1980.

(l) Device 12 (Brush Dike).

1 Statistics, Construction, and Cost. The creosote-treated, 7-foot timber posts of the brush dike were driven with a hand post driver. The piles and brush used to fill the dike were acquired locally and trucked to the site. Construction was completed in the spring of 1979. Major repairs, such as adding brush and replacing stringers where necessary, were completed 18 October 1979. Cost data are given in Table 2-40.

Table 2-40. Costs for brush dike.

Item	Quantity	Cost
Materials		
2- by 6-in-diameter brush	4 tons	Free
Post, 3-in-diameter by 7-ft length	81 ea.	$ 134
2- by 4-in, pressure-treated lumber	54 lin ft	14
Misc.		32
Equipment		
Crawler tractor (including operator)	68 hr	1,955
Labor	144 man-hours	1,083
Total		$3,218
Cost per foot		$ 32
Cost per foot excluding labor		$ 21

2 Performance. The dike performed well (Fig. 2-192), until the April 1980 storm, when most of the brush washed out of the cribs (Fig. 2-193). Many of the timber wales were removed by wave action, but the 7-foot posts remained in place.

(m) Device 13 (Vegetation).

1 Statistics, Construction, and Cost. Smooth cordgrass was planted on the lower beach (intertidal zone) in sections 1 to 5 and 7 to 14 during the spring and summer of 1979 (see Fig. 2-163 for vegetation plan). The original plan called for protection in sections 1, 3, and 9; however, these structures were not installed until late fall 1979 after the planting was completed. Consequently, the 1979 plantings were unprotected. Storm damage by Hurricanes Bob and Claudette required several sections to be replanted.

Saltmeadow cordgrass, common reed (*Phragmites communis*), and McCartney rose (*Rosa bracteata*) were planted on the upper beach during the spring and summer of 1979. Sections 1 to 13 and a-1 were enclosed by a barbed-wire fence to exclude cattle (see Fig. 2-163).

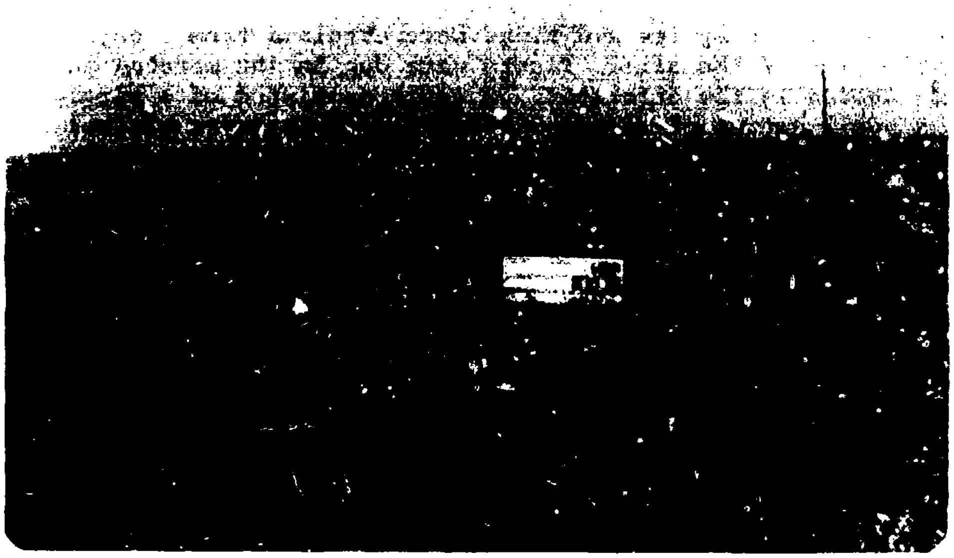


Figure 2-192. The brush dike in good condition, device 12, Fontainebleau State Park, Louisiana, 15 February 1980.

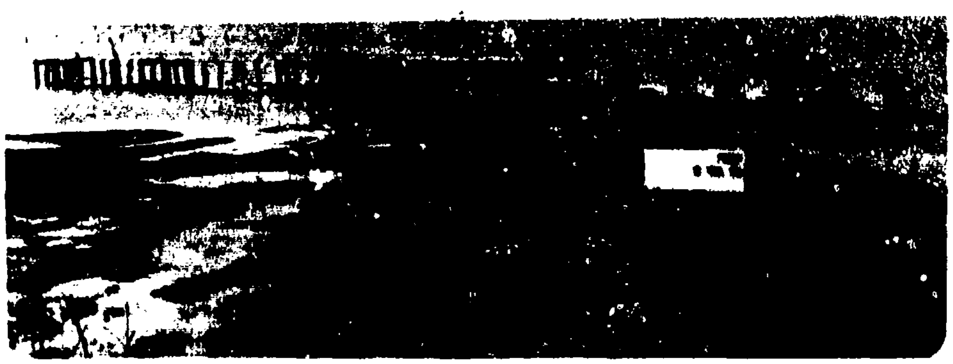


Figure 2-193. The brush dike was severely damaged by the April 1980 storm, Fontainebleau State Park, Louisiana, 9 May 1980.

Specifics of planting for lower and upper beach sections are summarized in Table 2-41.

Smooth cordgrass sprigs and plugs were obtained from a natural marsh about 1 mile west of the site. Plants were dug in the morning and most were hand-planted in the afternoon. A sprig consisted of a single upright stem and a small amount of attached rhizome. Sprigs were planted in 4- to 6-inch holes (opened with a spade or dibble) and covered to the base of the stem. Plugs were 5 to 6 inches in diameter and about 6 inches deep. Each plug contained at least two healthy stems and a large amount of rhizome and root material. Plugs were planted in holes (dug with a spade or dibble) large enough to insert the plug 1 inch below the surface so that it was retained firmly. All plantings were done at low tide when water was off the surface. All sections were planted with locally dug cordgrass except sections 2 and 13. Section 2 was replanted (17 May 1979) using peat-pot seedlings grown at the Plant Material Center, Coffeville, Mississippi. Approximately 75 percent of the peat pots contained one plant of a single stem; 25 percent of the pots contained two plants or stems. Section 13 was also planted with nursery-grown plants in peat pots which were planted with a post-hole digger and were covered to a depth of at least 3 inches. Details of fertilization methods of sections 1 through 14 are shown in Table 2-41.

Saltmeadow cordgrass plants were bare-root seedlings obtained from the Texas Plant Material Center, Knoxville, Texas. Planting methods were the same as with smooth cordgrass. Common reed was also obtained from the Knoxville Center and planted as rhizomes. McCartney rose cuttings were obtained locally, separated, and allowed to root before planting. Torpedo grass (*Panicum repens*) was not planted as scheduled in sections c, h, and l because it was found to be abundant and occurring naturally in these sections. Fertilization details for sections a through n are given in Table 2-41.

Vegetation costs are given in Table 2-42.

2 Performance. The performance of all plantings was greatly influenced by changing lake water levels and by storms. Sections 2 to 5 were planted with sprigs in mid-April. Just as planting was completed (17 April 1979), the Bonnet Carre Spillway was opened, releasing floodwaters from the Mississippi River into Lake Ponchartrain. This, combined with southeast winds, produced a rise in the lake level of about 2 feet, enough to innundate, or produce strong wave action in, the planted areas. Most of the plants were washed out as a result of the wave action and the inland movement of sand produced by the high water. These sections were replanted after the closing of the spillway on 23 May 1979; planting was completed in the other sections by the end of June.

On 11 July 1979, Hurricane Bob swept across the site and removed all the plantings in sections 1 and 2 and damaged plantings in the other sections; on 24 July 1979, Hurricane Claudette also passed over the site, destroying additional plantings. No records were made of mortality after Claudette, but few plants remained. Sections 3, 4, and 5 were again replanted, with plugs rather than sprigs. The site was fertilized in August.

Table 2-41. Planting statistics for lower and upper beach sections.

Section	Variety	Hills or rows (no.)	Spacing (in)	Total plants	Planting method	Date planted (1979)	Fertilization[1]
Lower beach							
1	Smooth cordgrass	5	24	250	Sprigs	21 June	—
2	Smooth cordgrass	5	24	250	Sprigs	17 April	—
		6	24	300	Peat pots	17 May	—
3	Smooth cordgrass	5	24	250	Sprigs	17 April	—
		5	24	250	Sprigs	8 June (replant)	—
		5	24	250	Plugs	10 August (replant)	2
4	Smooth cordgrass	6	18	396	Sprigs	17 April	—
		6	18	396	Sprigs	8 June (replant)	—
		6	36	120	Plugs	16 August (replant)	2
5	Smooth cordgrass	5	36	198	Sprigs	17 April	—
		5	36	198	Sprigs	8 June 28 June (replant)	—
		5	36	No accurate count	Plugs	16 August 30 August (replant)	2
6	Control	Not planted					—
7	Smooth cordgrass	5	24	250	Plugs	21 June	3
8	Smooth cordgrass	5	24	350	Sprigs	21 June	3
9	Smooth cordgrass	5	24	250	Sprigs	28 June	—
10	Smooth cordgrass	5	24	250	Sprigs	28 June	—
11	Smooth cordgrass	5	24	250	Sprigs	28 June	—
12	Smooth cordgrass	5	24	250	Sprigs	28 June	—
13	Smooth cordgrass	6	24	300	Peat-pot seedlings	17 May	—
14	Smooth cordgrass	5	24	250	Sprigs	28 June (completed 19 July)	—
Upper Beach							
a	Common reed	12	18	550	Rhizomes	19 April (completed 5 May)	4,5
b	None[6]	Not planted					4,5
c	Torpedo grass[7]						4,5
d	Saltmeadow cordgrass	12	24	600	Bare-root seedlings	30 July	4,5
e	Saltmeadow cordgrass	12	24	600	Bare-root seedlings	30 July	4,5
f	Saltmeadow cordgrass	12	24	600	Bare-root seedlings	19 July	4,5
g	Saltmeadow cordgrass	12	24	600	Bare-root seedlings	19 July	4,5
h	Torpedo grass[7]						—
i	Common reed	12	18	792	Rhizomes	9-10 May	—
j	Saltmeadow cordgrass	12	24	600	Bare-root seedlings	30 July	4,5
k	None	Not planted					4,5
l	Torpedo grass[7]						4,5
m	None	Not planted					4
n	Saltmeadow cordgrass	1	36	24	Plugs	16 August	4
	McCartney rose	4	48	300	Rooted cuttings	4 April	4

[1]Unless otherwise indicated, no fertilization was used. Numbers refer to footnotes below, which indicate fertilization method when fertilization was used.

[2]Osmocote 18-6-12 placed in hole at planting time (600 lb/acre).

[3]Osmocote 18-6-12 placed next to each plant (600 lb/acre).

[4]13-13-13 broadcast fertilized at 345 lb/acre; ammonium nitrate also broadcast fertilized at 115 lb/acre; both fertilizers applied 10 August 1979.

[5]Refertilized with 13-13-13 at 172 lb/acre on 30 August 1979.

[6]Not planted due to sand conditions.

[7]Not planted because torpedo grass is common to the area and was already present.

Table 2-42. Vegetation costs, Fontainebleau, Louisiana.

Item Item	Quantity	Cost[1]
Materials		
McCartney rose (rooted cuttings)	300 ea.	Free[2]
Smooth cordgrass (sprigs)	3,930 ea.	Free
Smooth cordgrass (peat-pot seedlings)	600 ea.	Free
Smooth cordgrass (plugs)	544 ea.	Free
Saltmeadow cordgrass (bare-rooted)	3,000 ea.	Free
Common reed	1,584 ea.	Free
Osmocote fertilizer	100 lb	$ 51
13-13-13 fertilizer	300 lb	22
Ammonium-nitrate fertilizer	200 lb	14
Labor[3]	592 man-hours	6,633
Total		$6,720
Cost per foot		$ 4
Cost per foot including labor		$ 10

[1]The entire vegetation plan was constructed with the Bogue Chitto Soil and Water Conservation District for $20,000, which included excavating the plants from another site and transporting them to the individual plots.

[2]Transportation for seedlings from the Coffeville Material Center was an additional $105.

[3]Estimate provided by Soil Conservation Service to include time and expenses of supporting crew and maintaining plantings.

Survival was monitored in December 1979, shortly after the completion of the sills. (The brush dike was badly damaged by the winter storms and was not rebuilt.) No plants remained in sections 1 and 2, probably due to the movement of sand away from this area. Survival of the plugs (and few remaining sprigs) in sections 3, 4, and 5 was high, between 60 and 80 percent. The survival of the plugs in section 7 was also high at 70 percent of the original plantings, and in section 13 where peat-potted plants were used (77-percent survival). The other areas had about 20-percent survival, except for section 14 (less than 5-percent survival) which was heavily grazed.

In April 1980, the sites were inspected and survival was generally good. Sections 3, 4, and 5 had a minimum of 70 percent and a maximum of 96 percent of the plants remaining. The precise number of plants was difficult to ascertain since many young stems were sprouting from the 1979 plantings. Growth was good in these areas and the plants appeared healthy. Growth and survival were also good in sections 7, 8, 11, 12, and 13, with 70- to 90-percent survival. The highest survival was in section 7, which was planted with plugs. Sections 9, 10, and 14 had less than 40 percent of the plants remaining, but these all showed renewed growth and a healthy appearance at this time (Fig. 2-194). Planting of smooth cordgrass appears to be reasonably successful at this site, even during rather unusual storm conditions.

Upper beach plantings consisted of saltmeadow cordgrass, common reed, and McCartney rose. Survival of planted saltmeadow cordgrass was very poor in the upper beach plantings (except in section n) by December 1979. Most of the plantings in place before the hurricanes were destroyed by sand movement and wave action by midsummer. Continued movement of sand and dry conditions later in the year caused complete mortality, except where plugs had been used in section n. Survival of plugs was 92 percent as of December 1979. In the spring of 1980 good growth of saltmeadow cordgrass was observed in section e. This species occurs naturally in the marsh behind the beach and may have reappeared from native plants or from planted seedlings that survived, but were undetected in the December count. Section n had experienced considerable losses over the winter months with only two surviving plants.

All of the common reed plants were lost in April and May 1979 due to southeast winds raising the lake level; none had returned as of April 1980. The McCartney rose was also washed cut early and has not become reestablished from natural sources.

(3) Analysis.

(a) Concrete-Block Revetments. Five of the demonstration devices were concrete-block revetments of various types. All the revetment

Figure 2-194. Vegetation in section 9 shows poor overall performance during 1979; replanted in May 1980, Fontainebleau State Park, Louisiana, 28 May 1980.

sections had a 1 on 3 lakeward slope and a 1 on 2 back slope with crowns rounded over at +4.5 feet MSL. Each device adequately prevented beach erosion during the monitoring period. The smaller sized units are light enough to be removed by vandals from parts of devices 2 and 4, which can lead to failure if the exposed filter cloth is torn. A problem with the standard construction block revetment was displacement by wave action when blocks were placed with their long axes parallel to the shoreline.

The revetments were subjected to more than 3-foot waves in April 1980, but the toes and back anchors performed satisfactorily; however, some toe scour was noted at all but one of the Dutch toes. This suggests that care must be taken to provide adequate depth of embedment of toes, to launch the Dutch toes uniformly.

The revetment structures at Fontainebleau were particularly well suited to this site and to other sites with similar wave climate and storm events. When designing a low-cost shore protection device of this type, it is important to (a) place standard construction blocks with long axes perpendicular to the shoreline, (b) provide adequate embedment of toe to prevent undermining, and (c) use small-sized blocks with the mat-type construction or where vandalism can be prevented.

(b) Breakwaters, Dikes, and Sills. The tire and timber-pile breakwater (device 6), despite some initial settling of the tires, performed very well. The piles were driven deep enough (7 to 9 feet) and the tires were adequately fastened to prevent any structural damage. The breakwater attenuated waves well, reducing 3- to 4-foot waves to 1 to 2 feet in its lee. The structure may have been placed too close to shore, however, as an incipient tombolo began to form in its lee. Further growth of this tombolo could interrupt littoral transport to the detriment of the downdrift shoreline. Nevertheless, this structure appears suited to this area and is recommended for similar sites.

The rolling tire breakwater (device 11) essentially consisted of a 100-foot-long low sill, parallel to the beach at about 0.5 foot NGVD. The structure consisted of six individual units, each about 20 feet long. To keep the structure from rolling too far under wave action, the ends of the pile cores were wrapped with steel cable and anchored to the lake bottom by 5-foot screw anchors. Problems encountered with this structure were settlement into the soft lake bottom and the loosening of the screw anchors due to scour, which caused the structure to realine itself in a concave position. The structure performed reasonably well, but longer screw anchors appear to be needed as well as flotation material, which would allow the structure to float and remain effective at high lake stages when it is most needed. The structure was effective in allowing sandbars to form behind it.

The sand-cement bag sill (device 10) consisted of a series of sand-cement bags covered with a sheet of filter cloth. The structure was built to an elevation of approximately 2 feet NGVD with the intention of reducing wav ction along the vegetated segments of beach. A row of nylon-reinforced paper sandbags placed on top of the filter cloth in the initial construction was almost entirely removed by waves within the first 2 months after installation.

The main structure sustained no significant damage during the monitoring period; however, when the lake level exceeds 2 feet NGVD waves transmitted over the structure are not noticeably attenuated. The 13 April 1980 storm did not significantly damage the structure, but it caused shoreline erosion behind it. Although the structure was not successful when subjected to storm conditions, it performed well when the lake level was below 2 feet NGVD. A higher structure is actually needed at this site. The monitoring period was not long enough to indicate the longevity of the structure. Most filter cloth is not strong enough to withstand continuous wave attack, and if it becomes torn, the sandbag structure it covers might soon deteriorate.

The brush dike consisted of a 100-foot-long low sill offshore breakwater structure of cribs made with timber posts and wales. The cribs were filled with small branches, many of which were removed by normal wave action. After the April 1980 storm almost all of the brush and many of the wales were removed. An individual landowner with an ample supply of brush could continually replace brush after significant storms, but using wire or some other material to tie brush together might be a better approach. There was no significant buildup of the beach in the lee of the structure. Until a better design is developed, the brush dike device is not recommended.

(c) Filter-Cloth Revetments. There were three types of revetments where filter cloth was used along with other materials to stabilize a beach: pocket filter-cloth revetment, loop filter-cloth revetment, and a tire/filter-cloth revetment (devices 7, 8, and 9, respectively). The pocket filter-cloth revetment consisted of a filter fabric panel with six rows of ballast pockets presewn to the panel. The two outer rows of ballast pockets were filled with sand-cement bags for positive anchoring. Six inches of shell was placed on the filter and then an additional 6 inches of topsoil was placed for vegetation. By April 1980 the entire soil cover was removed and the filter fabric that was not buried below the existing beach was exposed to wave action, debris, and sunlight. This type of structure did not perform adequately in that it was easily damaged by wave action and lacked sufficient overlaps. The structure, despite the damage (confined to the armor of shell and topsoil), continued to protect the beach from severe erosion. This structure could serve as a means of emergency protection, but it is not well suited for normal use. Perhaps the structure would be more effective if used in conjunction with an offshore breakwater, otherwise it is not recommended.

The loop filter-cloth revetment (device 8) consists of a loop filter cloth anchored with two rows of Jumbo blocks at the lakeward toe, two rows of Jumbo blocks at the crest, and one row of sand-cement bags at the buried landward toe. This structure was disrupted by the resurgence of common reed vegetation and wave action. The lakeward toe blocks were then anchored with reinforcing rod and were not displaced, but the crest row of blocks was displaced. Wave action induced uplift pressures underneath the filter panel and overturned some of the blocks due to loss of sand at the sides of the panel. Because of the thickness of the common reed, the grass could not grow through the voids of the filter fabric. Thus, the filter cloth was pushed up by the vegetation.

This structure was adversely affected by the growth of vegetation underneath the filter cloth, but the structure was also damaged by wave action. Since the structure did not withstand wave action, it is not advisable for a site such as Fontainebleau unless it is built in lee of a breakwater. Again, the loss of sand at the sides was a major factor.

The tire/filter-cloth revetment (device 9) consisted of a filter panel anchored at the lakeward toe with two rows of sand-cement bags encased in a Dutch toe. The landward toe is buried underneath the beach and is anchored with one row of sand-cement bags with reinforcing rods driven through the bags and the filter panel. The rest of the structure is armored with rows of tires which were filled with a dry sand-cement mixture, sprayed with water, and left to harden. The Dutch toe of the structure was undercut by wave action and settled into the scour trench, providing protection against further toe scour, as planned. The tires were displaced by wave action, and waves greater than 3 feet tend to make tires partially bouyant, moving them around on the revetment. The instability under wave attack is attributed to the lack of pressure relief holes through the tire centers. The revetment has continued to prevent bank erosion, and if the tires were secured in place, this structure would be a good low-cost shore protection. Without the tires being secured, the structure will not withstand wave action in a hurricane-prone environment.

(d) Vegetation. Smooth cordgrass appears to be a suitable species for planting in the intertidal zone at this location. The survival and growth of this species depend on the type of planting (i.e., sprigs, peat pots, or plugs), substrate, and weather patterns. In all cases, the best survival and most rapid growth resulted from the planting of plugs. This is probably a result of the greater root system and the deeper planting depth. The plugs apparently are more resistant to being uprooted than are sprigs, and may begin growth sooner after planting. Peat-pot seedlings also survived well and have become successfully established. Plants from peat pots were somewhat smaller after 1-year growth than plants started from plugs. Sprigs at this site have also done well.

At this site the substrate varies from areas of sand 2 to 5 feet deep to areas of peat which have little or no sand covering. Although exposed peat may rapidly erode by wave action, it is apparently the most stable substrate for initial planting of smooth cordgrass. For example, in section 7, which has a peat substrate, growth is extremely good, with rapid colonization and filling in of bare areas between plants (Fig. 2-195). Sandy areas, such as sections 1 and 2, suffered immediate loss of plantings. Peat soil probably provides a firmer substrate for planting, and losses due to washing out are reduced. The higher nutrient content in these soils may also assist plantings in becoming established; once established, the soil becomes stabilized and erosion lessens. The plants which survived storms and winter weather appear to be healthy after 1 year of growth. Growth is now good and large areas are becoming stabilized. Smooth cordgrass appears to be successful at this site.

Weather patterns affect establishment of plants, especially near planting time. Hurricane-force wind and waves cannot be resisted in many cases even by mature vegetation and erosion will occur. However, except for these extremes, plantings do stabilize the substrate and lessen erosional loss.

Figure 2-195. Growth of section 7, Fontainebleau State Park, Louisiana, 28 May 1980.

Saltmeadow cordgrass was unsuccessful as planted, but appears to be reinvading from natural sources in some areas. This species forms extensive meadows above the high tide line. The large amounts of loose sand movement in the upper beach make establishment of plantings very difficult. In this area several replantings would probably be necessary before a successful planting is established.

Common reed and McCartney rose did not do well when planted, but common reed is plentiful elsewhere along the beach. Storms may have removed the common reed before there was enough time for it to become sufficiently established. This plant should not be ruled out, however, since it appears to be resistant to burying and is common in this area.

Torpedo grass appears to be successful in holding the upper beach since it forms thick mats in many places; it is apparently resistant to burying and spreads well in sandy soil. No planting of torpedo grass was necessary as it has good colonizing abilities. Once established, this species would aid in erosion control. (However, use of torpedo grass in the State of Florida would not be possible as it is a prohibited plant in that state: it may not be imported, transported, or cultivated.)

Although not planted, rattlebox (Daubentonia sp.) appeared commonly along the upper beach at this site. No effort has been made to examine its growth, but numerous seedlings were apparent. This species should be considered for planting as it is naturally occurring and vigorous on the upper beach, although it is short-lived.

(e) Volumetric Changes. Table 2-43 gives volumetric changes between profile stations from July to December 1979. The base line for the surveys, which shows the stationing from northwest to southeast (opposite to the direction of littoral drift), is shown in Figure 2-162. The time interval between installation of structures, in fall 1979, and the December survey was too short to reveal any meaningful accretion or erosion trends. A March 1980 survey covered only a few stations that were missed in the December 1979 survey, and those stations were beyond the limits of the installations. The calculations do not reveal significant accretions in the lees of the offshore structures as expected, but they indicate a net accretion of about 1,300 cubic yards in the vegetation area. This may be the result of an influx of littoral drift from the south, but more monitoring is needed to determine the long-term effect of the vegetation and structures on the littoral regimen.

d. Key West, Florida.

(1) Site Description.

(a) Geographical Setting. The vegetation monitoring site at Key West comprises five sections of mangrove plantings, three near the Florida Keys Community College and one at each end of the Boca Chica Causeway (Fig. 2-196). Two exposed sections (B and C) and a protected section (A) are on a filled area northwest of the college. Two other sections (D and E) are on the north side of Boca Chica Causeway off Highway 1; a low-height boulder riprap wall partially protects these sections.

(b) Climate. The climate is subtropical to tropical, with mild winters and hot, humid summers. Average annual temperature is about 70° Fahrenheit.

(c) Geomorphology, Soils, and Vegetation. All of the planting sites are in water not more than 2 feet deep and in a mixture of sand and coral. The dominant vegetation in this area is a mixture of mangroves and seagrass beds. Three species of mangroves are common throughout the Keys, dominating broad stretches of intertidal and supratidal habitats. Five species of seagrasses are common in the waters along the Keys, with different species predominating on the basis of sediment characteristics, water clarity, water depth, and salinity.

(d) Waves, Tides, and Longshore Transport. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot. The mean and spring tidal ranges are 1.2 and 3.0 feet, respectively. The wave climate at each section is classified as mild. Although the wave-energy flux is strongly biased either to the right or to the left at each section, the direction of net longshore transport has little bearing on the effectiveness of the mangrove plantings, and only the survival rates need be considered.

(e) The Problem. In the early 1970's, the Florida Department of Transportation (FDOT) began a reconstruction program on Highway 1. This program involved grading new areas along the existing highway and replacing old bridges and other support structures. Due to the potential loss of

Table 2-43. Volumetric changes at Fontainebleau State Park, Louisiana (18 July 1979 to 21 Dec. 1979).[1]

Device	Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
Concrete-block revetments	0+25			
		256.2	585.8	329.6
	2+00			
		204.5	253.6	49.1
	3+29			
		278.9	313.4	34.5
Tire-pile breakwater	4+50			
		464.1	599.1	134.9
Cloth revetments	6+42			
		547.1	370.8	-176.2
	8+58			
		407.9	169.8	-238.2
Sandbag sill	10+58			
		352.8	103.9	-248.2
Rolling-tire breakwater	12+58			
		279.6	122.3	-157.3
Vegetation	14+54			
		339.3	84.3	-255.1
	16+58			
		246.6	104.4	-142.2
Brush dike	18+48			
		164.7	263.5	98.8
Vegetation	20+58			
		145.2	244.5	99.3
	22+58			
		74.5	585.0	510.5
—	24+58			
		72.8	682.1	609.2
—	26+58			
	Totals	3,834.3	4,482.3	648.0

[1]All devices were installed in fall 1979.

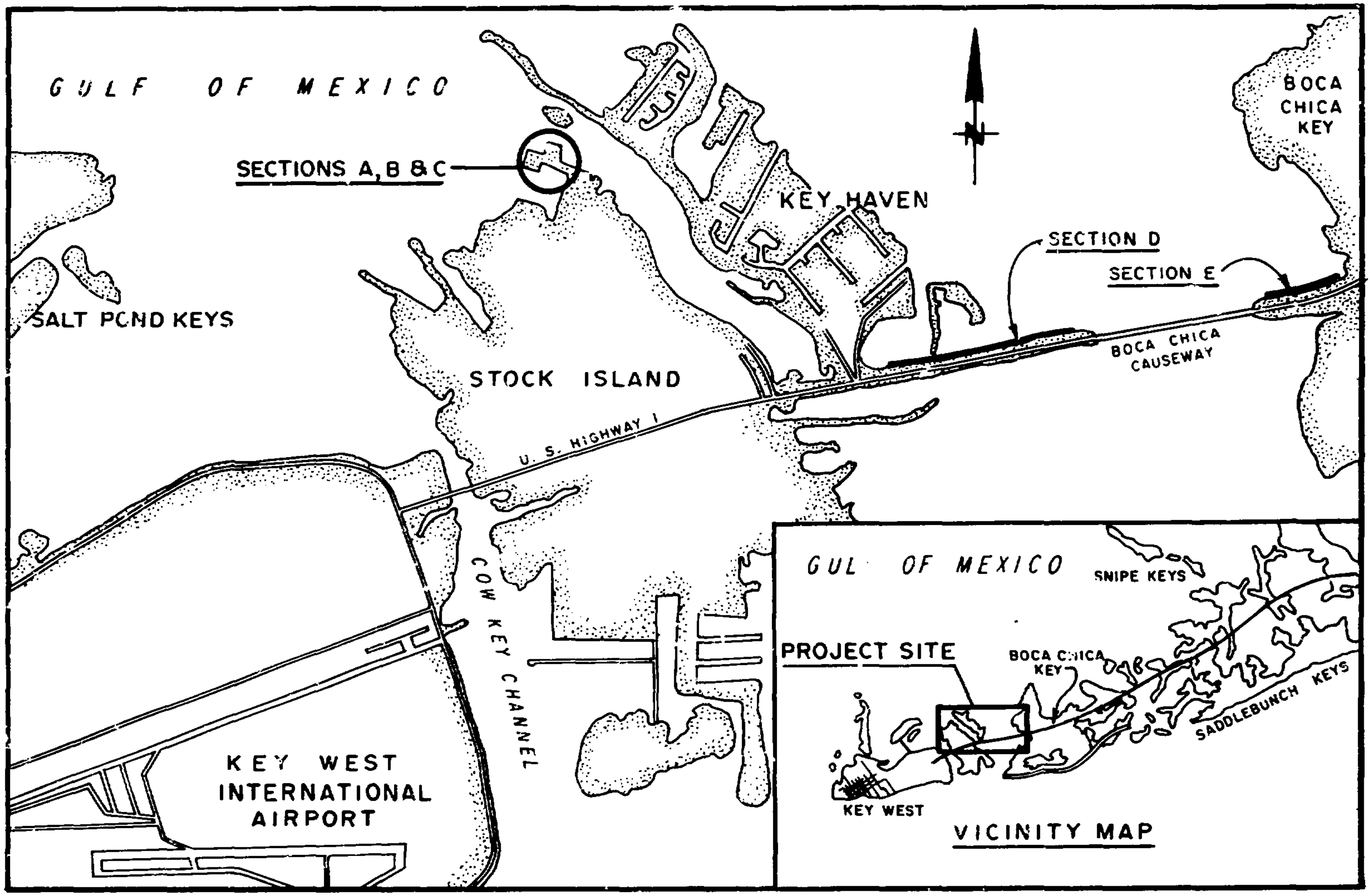


Figure 2-196. Location map of Key West, Florida, monitoring site.

naturally occurring vegetation near the proposed reconstruction, the FDOT proposed to provide mitigation for the projected losses by replanting destroyed vegetation communities. An agreement for replanting was reached between the FDOT and the Florida Department of Natural Resources, Marine Research Laboratory, St. Petersburg, in 1977. Planting of mangroves began in 1977 at several areas, and planting has continued as necessary since that time. The FDOT personnel agreed to the use of these sites being monitored under the demonstration program.

Selection of mangrove planting areas by the FDOT was based on the areas having (a) protection from strong wind and wave action, (b) suitable ("penetrable") substrate, (c) ease of access for the planting crew, (d) ample length of shore available for planting, and (e) Federal, State, or County ownership of adjacent lands. Basic plant materials were seeds, beans, fruits, or "propagules," collected by convict labor and placed in a small shoreline cove holding area behind the FDOT office in Marathon, Florida, pending transplantation.

(2) Monitoring Project. The monitoring sites involved about 8,000 feet of shoreline at Florida Keys Community College, and about 4,650 feet of shoreline along Boca Chica Causeway.

The college area was planted in June 1977. The number of plants and the spacing used in the planting are shown in Table 2-44; a general plan is shown in Fig. 2-197.

Table 2-44. Red mangrove plantings at the Florida Keys Community College area, Key West, Florida, June 1977.

Section	No. of hills or rows	Spacing (in)	Total no. plants	Planting method
A	2	78	14	Propagules
	2	78	14	Seedlings
	2	78	14	Plants with prop roots 2 ft tall
B and C	←——— same as section A ———→			

Note.--No fertilization was attempted at this site; although 100 black mangroves were reported to have been planted at this site, no planting or monitoring data are available.

The Boca Chica Causeway (sections D and E) was planted in 1978 and replanted in 1979. Section D was divided into subsections Dw, Dm, and proposed De; section E was divided into subsections Ew, Em and Ee (Fig. 2-198). A total of 2,000 black mangroves and 1,300 red mangroves were planted in section E, and 4,000 black mangroves and 4,000 red mangroves were planted in section D (all in Dw and Dm). No statistics for spacing or type of planting material are available.

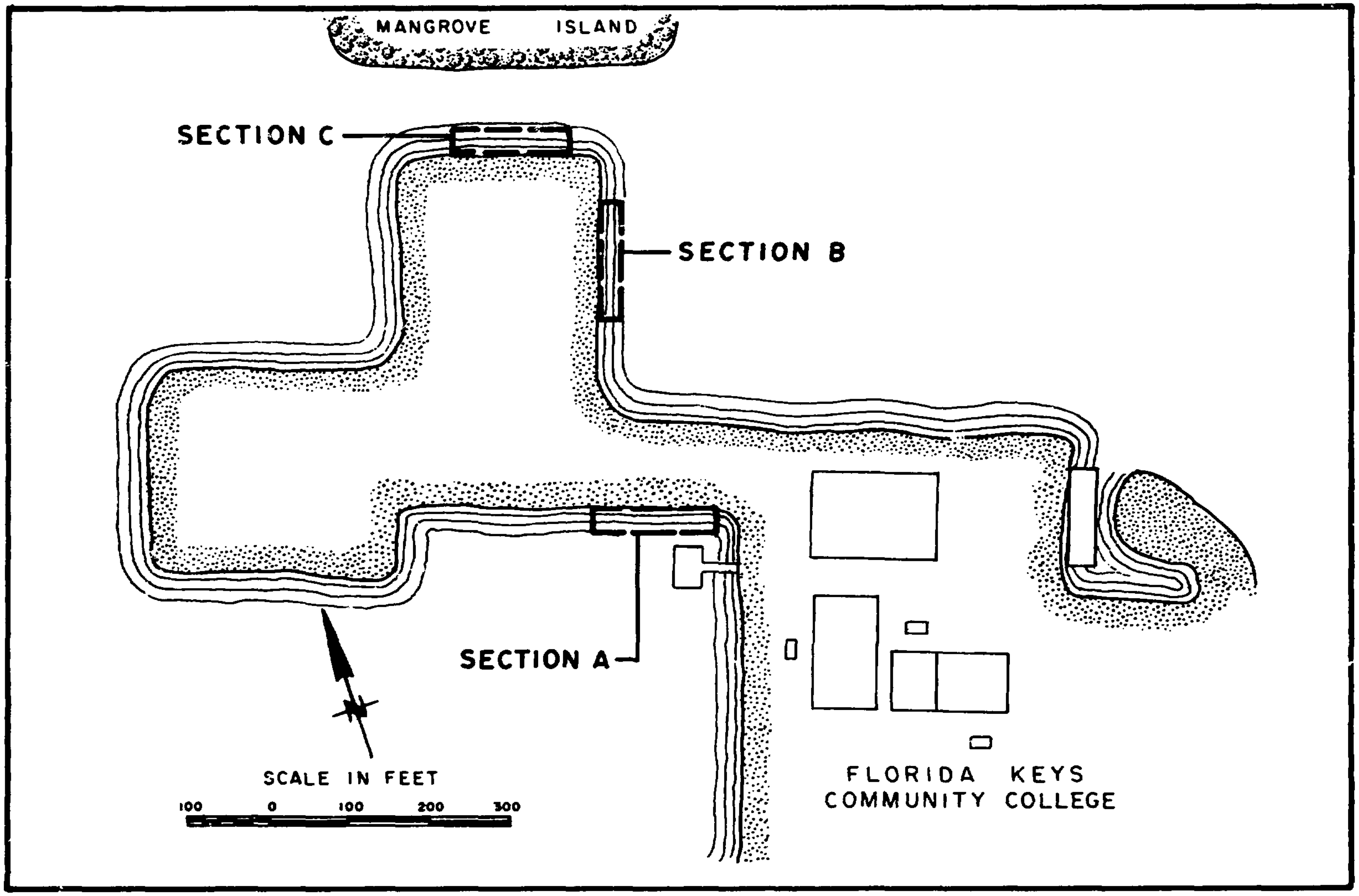


Figure 2-197. General plan of sections A, B, and C, Key West, Florida.

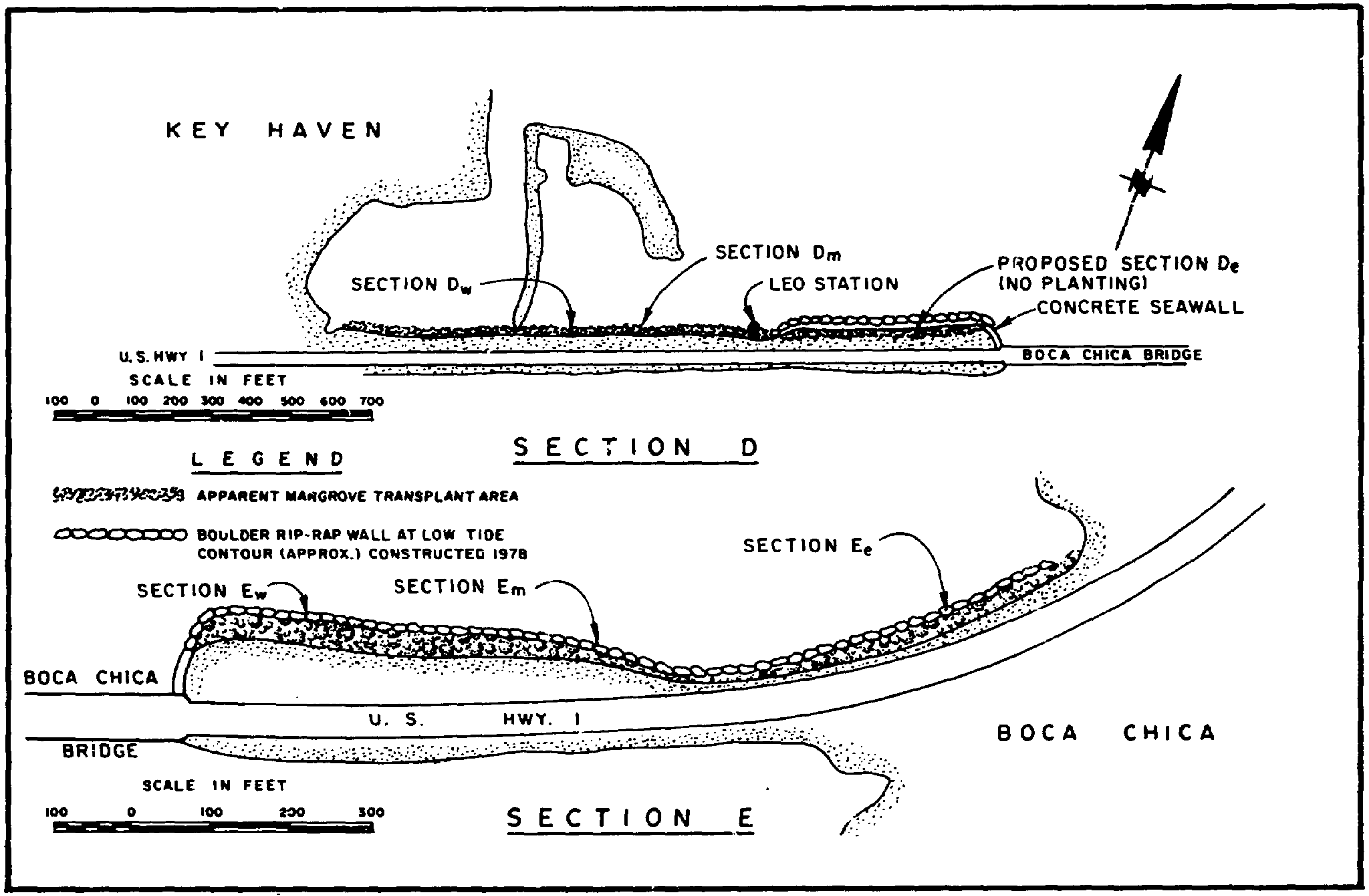


Figure 2-198. General plan of sections D and E, Key West, Florida.

(3) Planting Procedure. The college area was planted in June 1977 with plants placed in 14-inch-diameter holes, which were then filled with a mixture of seagrass debris and commercial peat. Each individual plant was then anchored with a degradable tie to an upright steel rod that was driven into the bottom sediments. As much of the rootball was retained as possible, on both the seedlings and the plants with prop roots. Diagrams were made at this time showing the number of plants in place.

The causeway sections were presumably planted using the same methods. Density counts taken in June 1979, based on randomly placed quadrants, served as the Initial Post Planting Inventory for these sections. Table 2-45 documents this inventory, along with later density counts.

Table 2-45. Density counts for sections D and E, Key West, Florida.

Subsection	Variety	Plants per m^2	
		July 1979	Jan 1980
Proposed De	------------	0	0
Dm	Red mangrove	1	1
Dw	Red mangrove	6	4
Ee	Red mangrove	2	5[1]
	Black mangrove	1	
Em	Red mangrove	2	8[1]
	Black mangrove	1	
Ew	Red mangrove	0	7
	Black mangrove	0	1

[1]Count taken September 1979 did not specify species; no count was taken in 1980.

Plantings are continuing at all sections to replace seedlings washed out by wave action or pulled out by pedestrians, the latter being a major factor in plant loss. No counts are presently available for 1979 or 1980 replantings.

(4) Costs. Labor for gathering, storing, and planting the mangroves to date has been provided by convicts or by volunteer students. No paid labor has been involved. Thus, an accurate estimate of the value of the unpaid labor is difficult to assess. One FDOT estimate is about $3.00 per plant, to include the costs of hauling and placement of breakwaters. Supervisory personnel indicate a cost of $0.50 to $0.75 per plant, not including the breakwaters.

(5) Performance. Survival of red mangroves at the college area varied with the degree of protection and the size of the plant at the planting time. A count of surviving individuals from the initial plantings was made in late May 1979. Figure 2-199 traces the survival of the plants to that date.

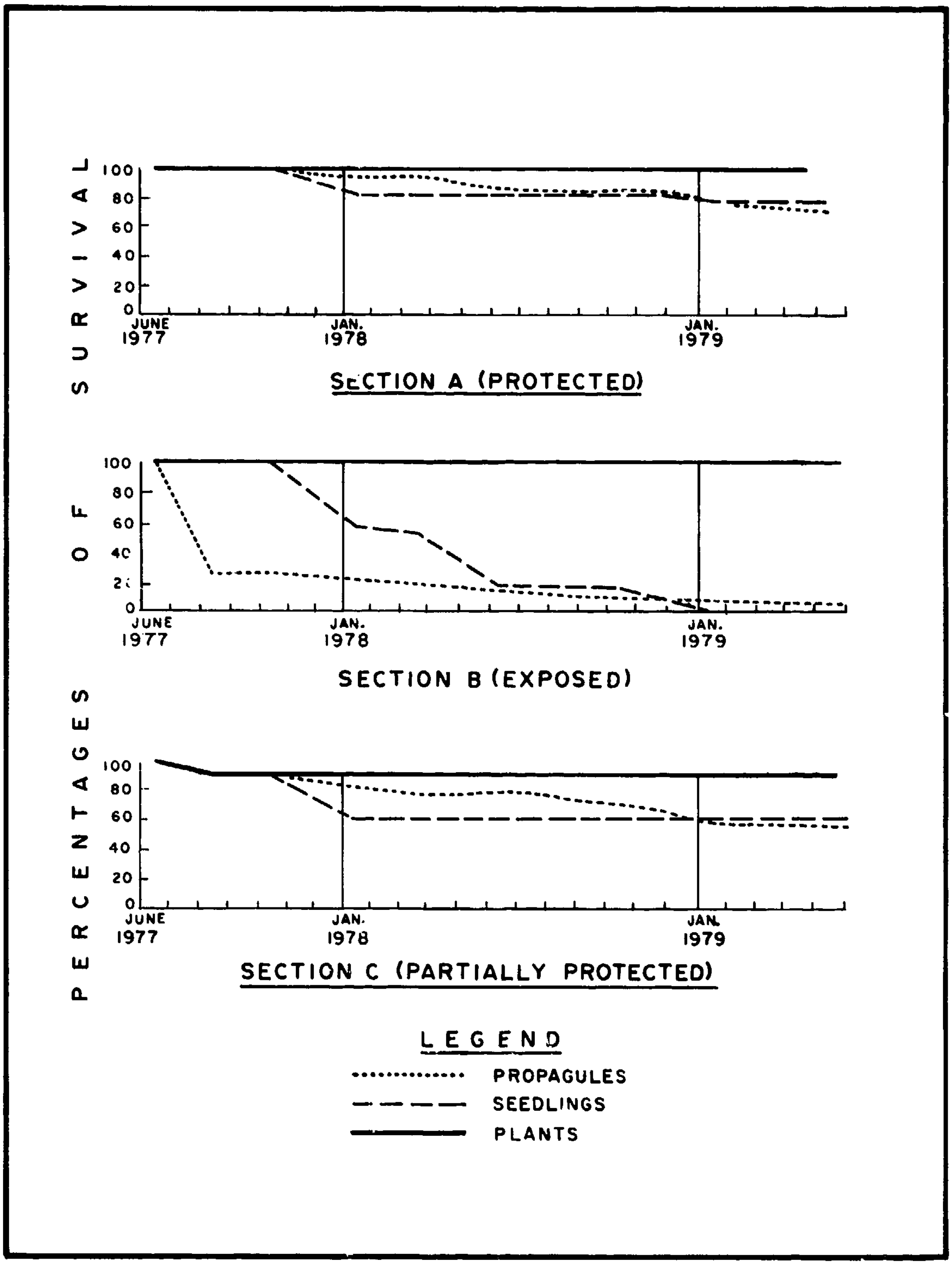


Figure 2-199. Survival of transplanted red mangroves at Florida Keys Commun‹ty College, Key West, Florida, June 1977 to M‹y 1979.

During the period from 31 August through 12 September 1979, two hurricanes threatened Key West and the Florida Keys. While neither storm directly impacted the Keys, higher than normal winds and tides were experienced during this period. On 10 and 11 September 1979, LEO observers recorded highest sustained winds of 18 miles per hour from northeast to east at section B on the college campus, producing waves on that shoreline of 0.7 to 1.0 foot in height. At section A on the campus, the highest windspeed recorded for this period was 2 miles per hour, and waves were less than 0.2 foot in height. These two sections represent the maximum and minimum conditions, respectively. All other sections (C, D, E) ranged between those conditions.

Vegetation observations were made on 16 September 1979. No transplant losses had occurred at any of the sections on the college campus. At sections D and E a replanting of red mangrove propagules was accomplished during August 1979 by representatives of the Florida Department of Transportation. Many of these young plants survived the higher than normal tides. No significant erosion was observed at any of the sections.

Section A, the protected area, had the highest survival, with more than 70 percent of the plants surviving through January 1980. There was 100-percent survival of mangroves that were planted as young trees with prop roots. Section B, the most exposed area, had the greatest mortality. No seedlings survived, and less than 10 percent of the "propagule" plantings were alive in 1979 and 1980. In contrast, the prop root-bearing plants showed 100-percent survival by January 1980. Section C, somewhat more protected than section B but less than section A, showed intermediate survival for propagules and seedlings (60 percent) and a small amount of mortality in the prop-rooted plants. Of these larger plants, 92 percent were alive in 1979; no further mortality was observed. A site visit in February 1980 indicated that mortality may now be higher because of a sewage-treatment plant emptying its effluent into the bay. This has not been conclusively determined, however.

Survival of the causeway area plantings has been variable. Along section Dm, which is unprotected, mortality was very high; only a few plants remained in February 1980. Along section Dw, survival was better, as a mangrove fringe that extends parallel to the shoreline to the north of this section breaks the approaching waves and shelters the plantings. Along section E, which is protected by the boulder riprap wall, many seedlings have survived and appear to have become well established (Figs. 2-200 and 2-201). Figures 2-202 through 2-213 show how those sections appeared at the beginning and the end of the monitoring period.

(6) Analysis. Success of the mangrove plantings has been generally good at this site. Given protection, the plants appear to do better, and survival is higher when larger plants are used. In most cases, plants with prop roots survived better than the other types, perhaps because they were more stable and more resistant to wave action. Mortality has been high in some cases due to vandalism--many of the seedlings were pulled up by pedestrians; however, larger plants were evidently less susceptible to this kind of loss. Another cause of mortality, particularly on the gulf side, has been the accumulation of seagrass leaves behind the breakwaters. These windrows of leaves are several inches deep and reduce the light available to the mangroves.

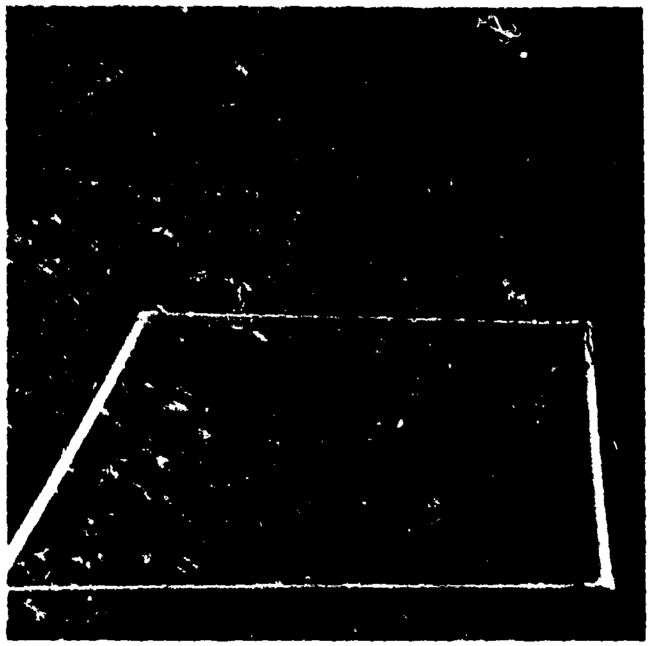


Figure 2-200. Stem counts in January 1980 reported seven stems per square meter of red mangrove and one stem per square meter of black mangrove, section Ew, Key West, Florida, 8 January 1980.

Figure 2-201. Stem counts in January 1980 reported two red mangrove stems per square meter in section E_{leo}, Key West, Florida, 8 January 1980.

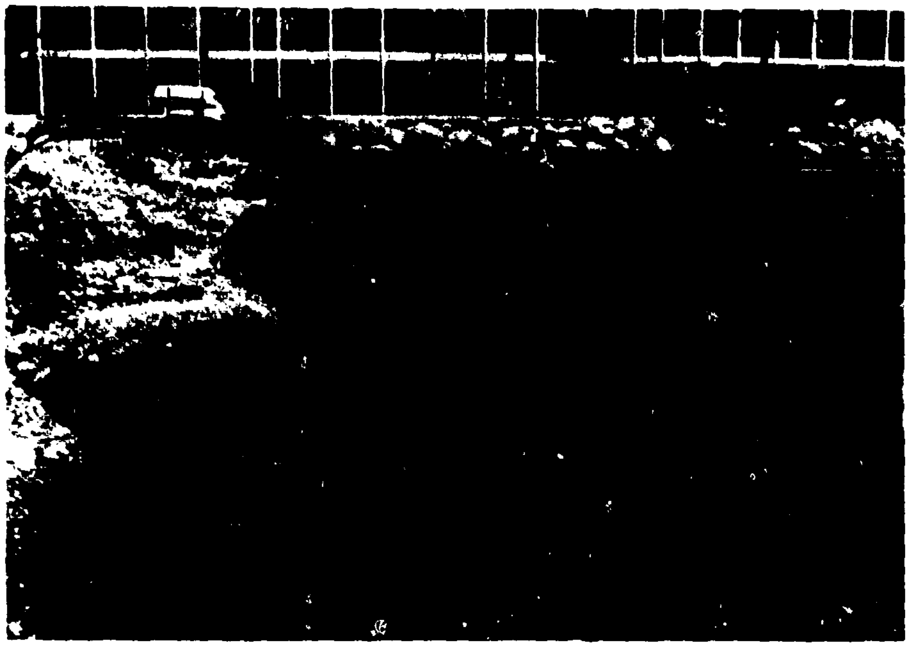


Figure 2-202. Section A at beginning of monitoring period, Key West, Florida, 3 June 1979.

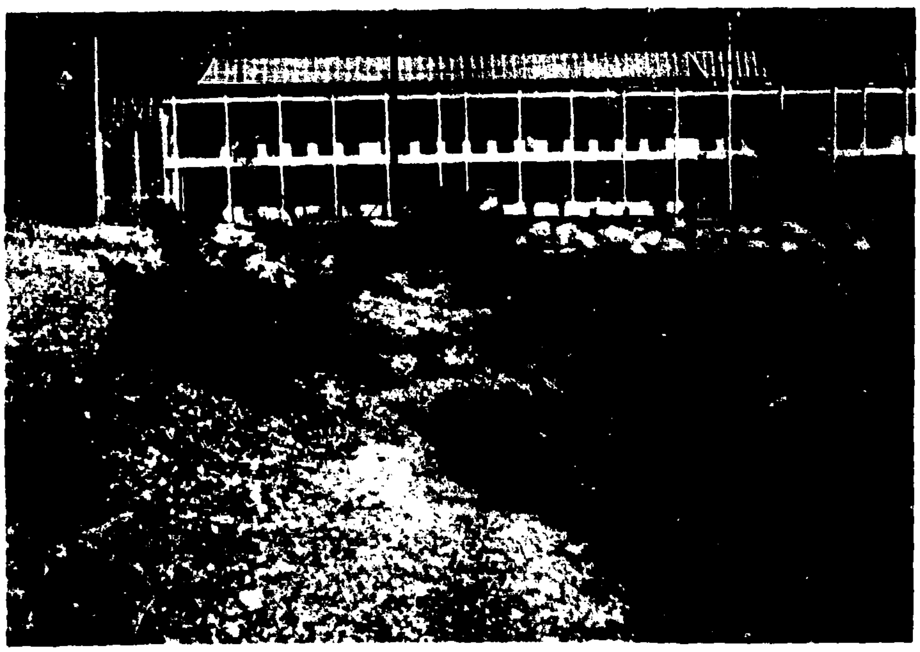


Figure 2-203. Section A, Key West, Florida, 12 March 1980.

Figure 2-204. Section B at initial monitoring visit, Key West, Florida, 3 June 1979.

Figure 2-205. Section B, Key West, Florida, 12 March 1980.

Figure 2-206. Section C, looking east, Key West, Florida, 3 July 1979.

Figure 2-207. Section C, 8 months later than Figure 2-206, Key West, Florida, 12 March 1980.

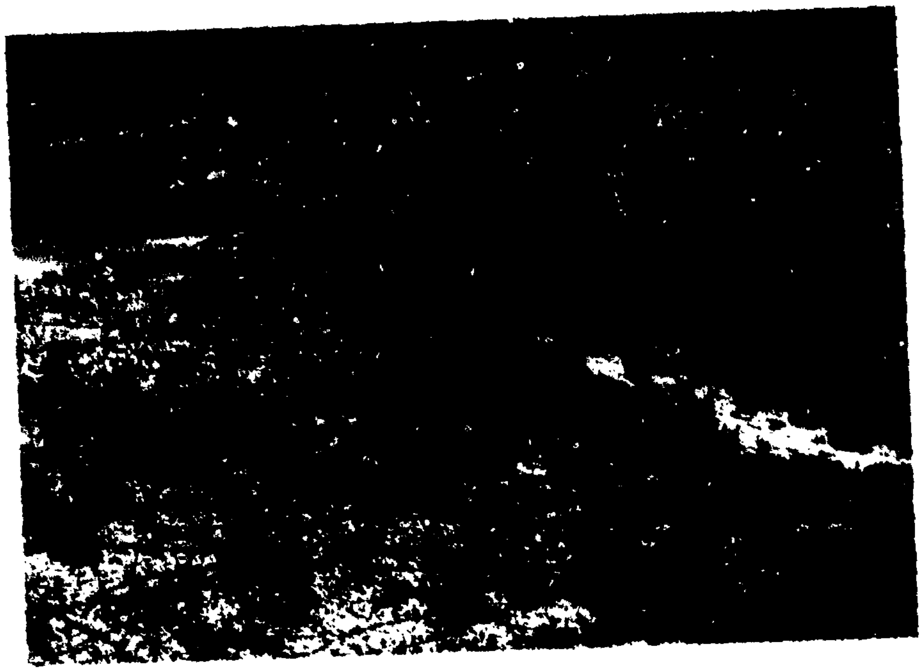


Figure 2-208. Section Dm at the beginning of the monitoring period, Key West, Florida, 3 June 1979.

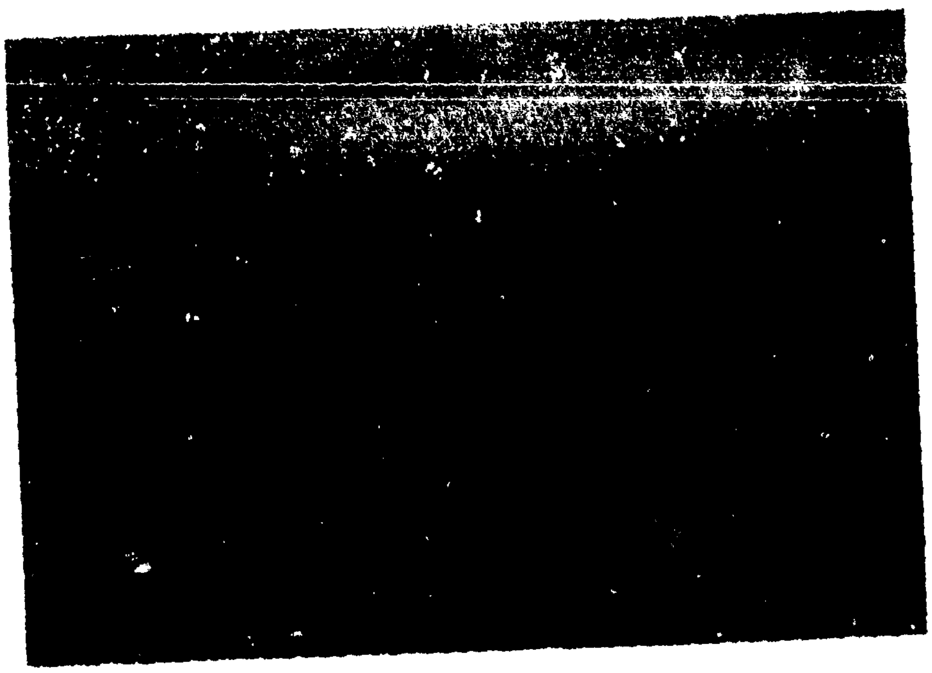


Figure 2-209. Section Dm, Key West, Florida, 12 March 1980.

Figure 2-210. Section Dw along Boca Chica Causeway,
Key West, Florida, 3 June 1979.

Figure 2-211. Section Dw, Key West, Florida, 12 March 1980.

Figure 2-212. View of section E, Key West, Florida, 3 July 1979.

Figure 2-213. Section E, 8 months later, Key West, Florida, 12 March 1980.

The plantings at this site appear to suffer fewer losses after the initial period of transplanting and establishment. High storm tides and even debris have little effect in the long run on the larger plants. Once established, a fringe of mangroves serves to attenuate waves reaching the shore, and therefore protects the vegetation behind it. Since mangroves occur naturally in this area and plantings can become established fairly easily, this type of vegetation seems a useful measure in erosion control.

e. Holly Beach, Louisiana.

(1) Site Description.

(a) Geographical Setting. Holly Beach, Louisiana, is a revetment monitoring site on the Gulf of Mexico (Fig. 2-134). The site is located about 40 miles south of Lake Charles, Louisiana, and 2 miles west of Holly Beach, Louisiana. The shoreline is almost straight, with an east-west orientation, and is easily accessible from State Highway 82, which approximately parallels the shoreline in the vicinity of the site (Fig. 2-214).

(b) Climate. The climate is subtropical, having mild winters and hot, humid summers; the normal annual temperature averages about 63° Fahrenheit.

(c) Geomorphology, Soils, and Vegetation. The foreshore slope is about 1 on 25, flattening to 1 on 100 or less just offshore. Depths are on the order of 3 feet below NGVD for a distance of several hundred feet offshore. The beach is composed of shell, shell fragments, silt, and sand, and varies from 50 to 100 feet wide. In the site area, the land immediately behind the beach has been filled to provide a raised subgrade for the highway, which has a crown elevation of +9.5 feet NGVD. The wave runup area behind the foreshore has been graded and revetted to the shoulder of the highway, completely obliterating the original beach-ridge formation. Behind the highway are extensive, vegetatively productive marshes with an average elevation of +1 to +2 feet NGVD.

(d) Waves, Tides, and Longshore Transport. Tides at the site are diurnal, and the mean tidal range is approximately 2 feet. Extreme tides or storm surges due to wind setup occur during major storms and hurricanes. A high storm surge of +12.1 feet NGVD occurred during Hurricane Audrey in 1957, and a low tide of -3.1 feet NGVD occurred during a winter storm in February 1965. Predominantly, wave heights are 2 to 3 feet and periods are 5 to 8 seconds. The littoral currents during the predominant range of waves are westward at 1 to 3 feet per second; the net annual transport rate is estimated at 62,000 to 100,000 cubic yards per year. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 8.0 feet. The wave climate is classified as severe. Although a strong net potential for eastward transport of littoral material is indicated from the 3-month study period of the energy-flux analysis for this site, there is little material to be moved, and the waves expend most of their energy in breaking and in uprush on the existing revetment.

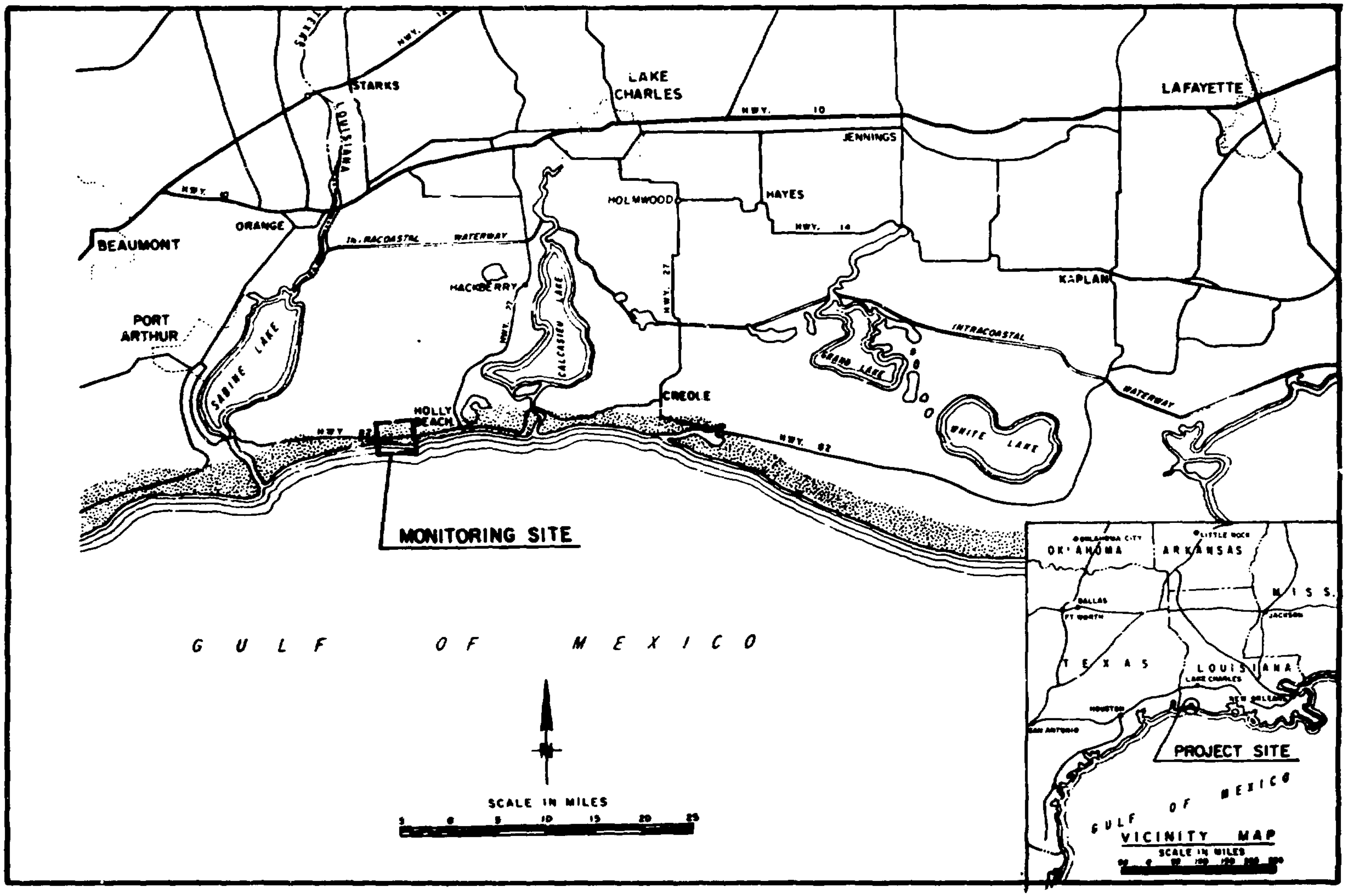


Figure 2-214. Location map of Holly Beach, Louisiana, monitoring site.

(e) The Problem. Records indicate that between 1883 and 1968 the maximum recession of the shoreline was approximately 590 feet. During this 85-year period, the average recession rate was 4.7 feet per year, but from 1957 to 1968 the shoreline receded at a rate of 11.2 feet per year. The later period included two major hurricanes: Audrey in 1957 and Carla in 1961.

During major storms, State Highway 82 is often overtopped, and erosion of the foreshore has undercut and damaged the pavement. The highway has been badly damaged and the embankment partially destroyed at least three times in recent years--1957, 1961, and 1969. The length of shoreline suffering major damage each time varied from 1.5 to 2 miles. Protection of State Highway 82 is important because it serves as the major hurricane evacuation route for coastal residents.

(2) Monitoring Project.

(a) Prototype Test. The prior frequency of damage to State Highway 82 led the Louisiana Department of Highways to investigate the Gobi-block revetment system. A demonstration section of hand-placed Gobi blocks was divided into two reaches, each 100 feet long. In one reach, the blocks were set on a filter cloth with openings of about 170 micrometers (4.6 percent open area), which results in a critically low permeability. In the other reach, the blocks were placed on a filter cloth that has openings of about 350 micrometers and is highly permeable, with more than 30 percent open area. The sand at Holly Beach has a median grain size of 150 micrometers, which requires that a filter cloth have openings large enough to relieve hydrostatic pressures, but small enough to retain the soil. Two weeks after its completion, the demonstration section was subjected to storm waves which overtopped the road and caused severe damage along 2 miles of the highway. Waves were estimated at 3 to 4 feet. The revetment with the low-permeable filter cloth was completely destroyed. The revetment with the highly permeable filter cloth remained intact.

(b) Three-Mile Project (1970). The success of the demonstration encouraged the Highway Department to build a 3-mile Gobi-block revetment along State Highway 82 in November 1970. Construction included 42,000 square yards of Gobi blocks hand-placed on Nicolon 66301 filter cloth (350 micrometers). The filter cloth was anchored in two excavated trenches, each approximately 3 feet deep; one trench was at the toe of the structure and one at the shoulder of the road. The revetment was built in conjunction with an 8-foot-wide, 8-inch-thick soil-cement road shoulder on the gulf side of the highway. A narrower shoulder was built on the landward side of the road. Details of the installation are shown in Figure 2-215; Figures 2-216 and 2-217 show the construction in progress. The shoulder revetment structure was built in lieu of a continuous revetment for two reasons: economy and the prediction that the road would not be overtopped more often than about once in 15 years. It was believed that as long as no hurricanes occurred, the revetment would provide sufficient protection and reduce roadway maintenance requirements.

Wave conditions at the site proved to be more severe than had been anticipated. The revetment suffered little damage in the first 3 years, although it was estimated that in the winter of 1972-73, the road was

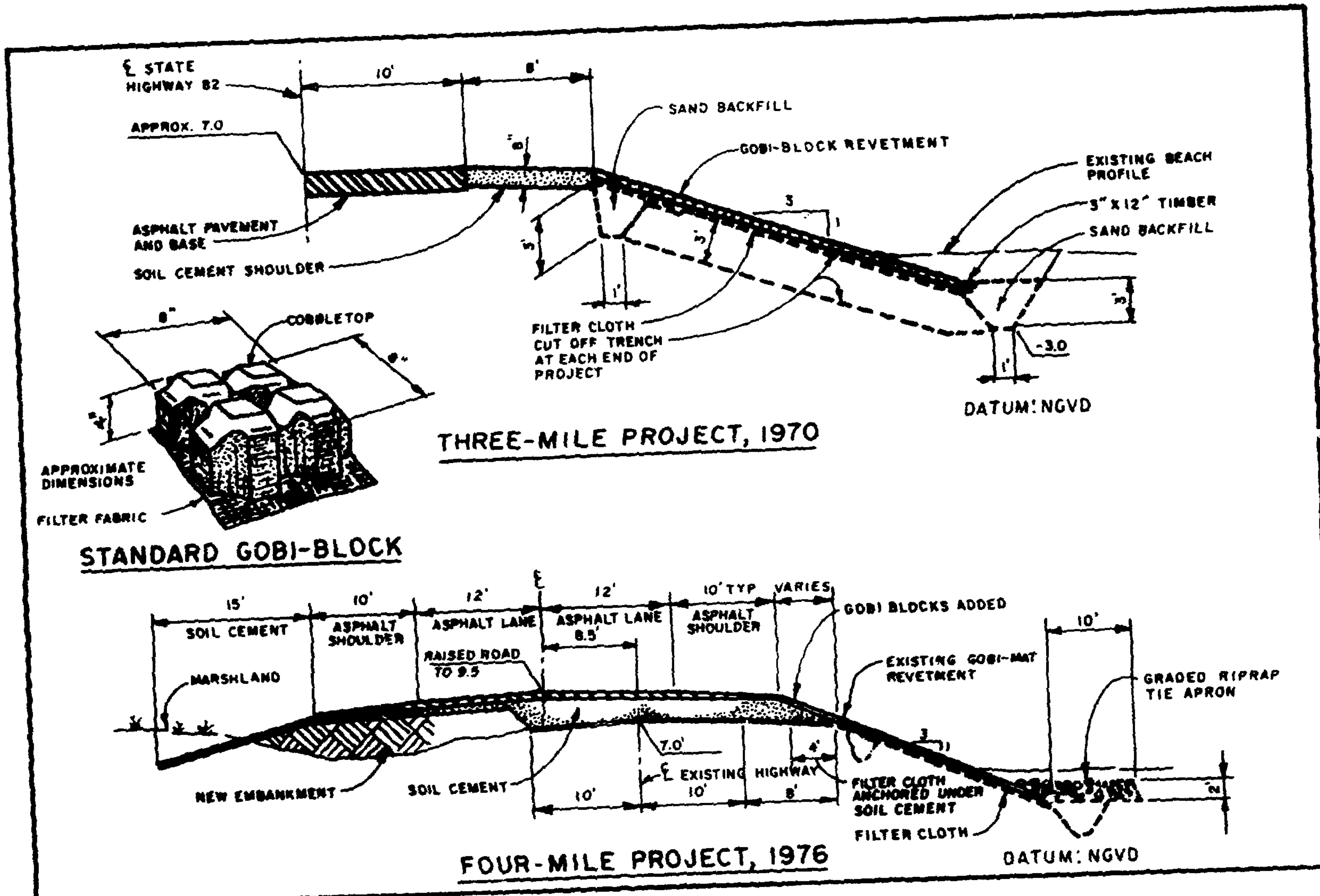


Figure 2-215. Original construction of 3-mile and 4-mile projects at Holly Beach site.

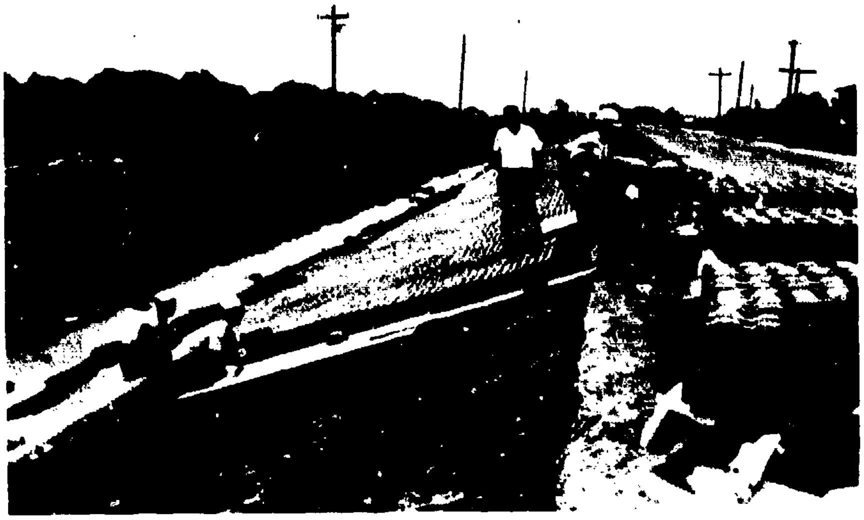


Figure 2-216. Construction at Holly Beach, Louisiana, 5 October 1970.

Figure 2-217. Construction at Holly Beach, Louisiana, 5 October 1970.

overtopped more than 20 times. In September 1973, Hurricane Delia initially headed toward the site but then veered west and hit Galveston, Texas. Although the hurricane was about 100 miles offshore for a period of 4 to 5 days, the winds created abnormally high water levels in the Holly Beach area which overtopped the highway. The local highway patrol estimated the depth of water flowing across the road to be more than 2 feet. As a result of Hurricane Delia, about 1,200 feet of revetment was damaged, but the highway was damaged only to its centerline. This partial protection of the highway was attributed to the revetment, because without it, the road probably would have been completely breached. The landside shoulder beyond the asphalt pavement was also severely damaged by overwash in several places. The remainder of the project shoreline (about 90 percent of the project length) suffered little damage and remained in excellent condition. The integrity of this design was based on having a beach in front of the revetment to absorb the wave energy of the storm waves greater than 3.5 feet.

Damage to the revetment seemed to be initiated at the top where a construction joint connected the pervious concrete Gobi blocks to the impervious cement shoulder. Settlement of the shoulder trench due to inadequate compaction allowed the blocks to drop away from the soil-cement shoulder. Apparently, uplift pressures transmitted through the joint then lifted and destroyed large segments of the shoulder (Fig. 2-218). After the joint and shoulder had failed, loss of sand from underneath the revetment caused the remaining blocks to dislodge under the action of breaking waves and wave runup.

Figure 2-218. Shoulder destruction due to uplift pressures transmitted through joint, Holly Beach, Louisiana, 26 July 1979.

(c) Four-Mile Project (1976). Following Hurricane Delia, the Highway Department attempted to keep the 1,200 feet of damaged roadway open using various methods of maintenance, such as sandfill, large concrete blocks, and broken concrete, but all were unsuccessful. In May 1974, plans and specifications were prepared to improve the roadway by (1) raising the surface of the road 2.5 feet (to +9.5 NGVD) to reduce overtopping, (2) repairing the 1973 storm damage, (3) moving the centerline of the highway 8.5 feet landward, (4) extending the project length to 4 miles, and (5) improving the landside shoulder. The filter cloth underneath the blocks was to be anchored under a soil-cement base on the road shoulder (Fig. 2-215). As an additional feature, the toe of the revetment was to be protected by a 2-foot-thick graded-riprap tie apron 10 feet wide. Figure 2-219 shows the location of the 4-mile project.

Early in 1976, the Highway Department began to implement the improvement plan by first removing the concrete blocks, broken concrete, and other materials from the 1,200-foot reach damaged by Hurricane Delia. Sandfill was then dumped on the embankment where required, and workers dressed the slope to 1 on 3 (Fig. 2-215). Two separate Nicolon filter-cloth panels were installed, the first panel (8 feet wide) was unrolled onto the prepared slope and trenches, with 4 feet of cloth overlapping the 1970 road shoulder and 4 feet overlapping the second panel. Large panels were presewn and workers placed panels so that the seams were parallel to the roadway. To anchor the filter cloth and the revetment, riprap was placed 2 feet thick at the toe of the structure. The sandfill was placed over the toe and bottom of the revetment. (For details see Fig. 2-220.) Gobi blocks were hand-placed on the filter cloth starting at the revetment toe. Finally, the 1,200 feet of storm-damaged revetment was rebuilt to the existing soil-cement shoulder. The additional mile was also revetted with a structure according to Figure 2-220. Graded riprap was trucked to the site and placed by crane along the entire 4 miles of revetment, forming a 2-foot-thick, 10-foot-wide, rock toe apron. The original 3-mile revetment reach was rebuilt to a cross section as in Figure 2-220.

The raised roadway section was then constructed as planned, and additional Gobi blocks were placed on the gulf-side soil-cement slope from the upper side of the revetment up to the edge of the asphalt shoulder. The asphalt was placed last, and was extended past the edge of the shoulder down the 1 on 3 slope, covering the Gobi blocks almost to the edge of the soil-cement underlayer.

(3) Performance. From 1970 to the initial site visit by the New Orleans District monitor in March 1979, only minor maintenance (8 to 10 percent) had been required to repair the road and revetment. No appreciable scour beneath the toe or bottom row of the blocks had occurred nor was the degradation of the concrete blocks significant; however, a few isolated blocks were considerably worn. This was attributed to low cement content, inadequate curing, or poor-quality aggregate. Other minor damages were due to vandals prying the hand-placed blocks loose with crowbars and removing them from the site.

During Tropical Storm Claudette in July 1979, approximately 2 miles of the Gobi-block revetment was displaced or otherwise damaged. In a few

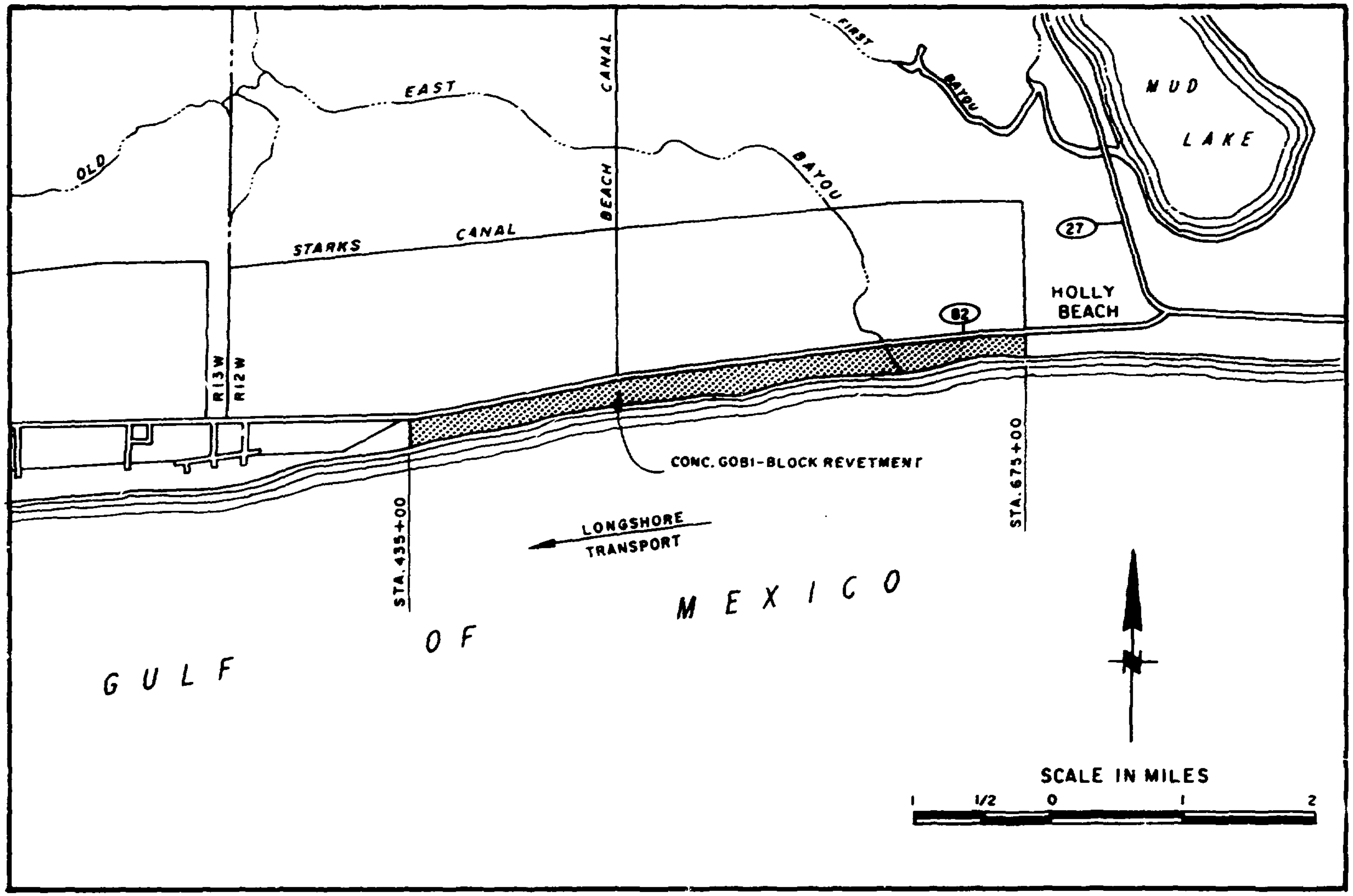


Figure 2-219. General plan of Holly Beach, Louisiana, monitoring site.

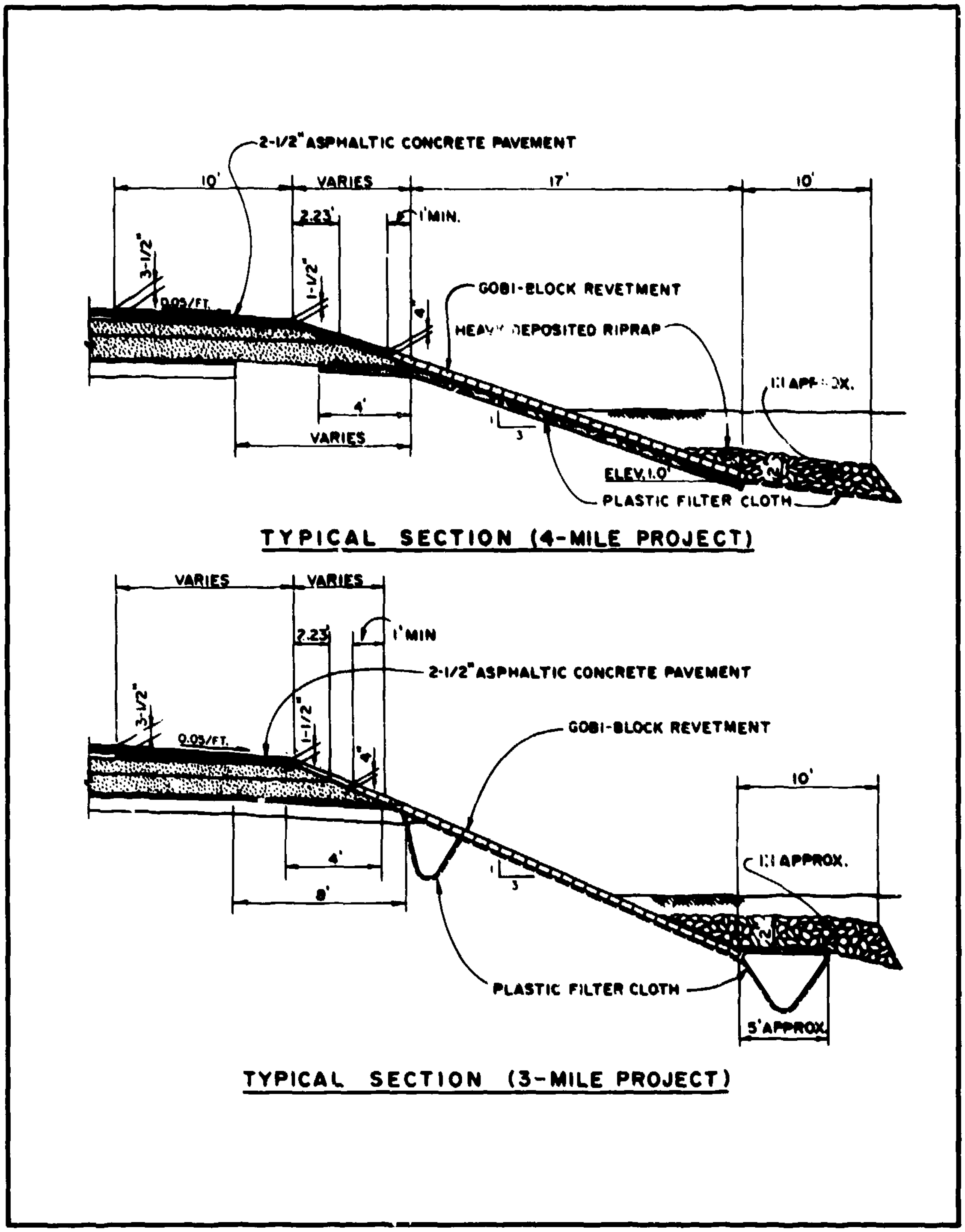


Figure 2-220. Rebuilt 3-mile and 4-mile projects at Holly Beach site.

locations, the blocks were displaced, the filter cloth was ripped, and the 8-foot overlap was uncovered, but the slope and shoulder of the road remained intact (Fig. 2-221). At other locations, where the filter fabric reached the asphalt/soil-cement shoulder, waves had displaced the Gobi blocks which anchored the first filter panel, which was only 8 feet wide. This displacement of blocks allowed the two separate filter-cloth panels to float free, then hydrostatic pressure and wave action removed sand under the shoulder. After the filter fabric loosened, progressive displacement of adjacent blocks was rapid (Fig. 2-222). Along reaches of several hundred feet, the blocks and filter cloth were displaced, the slope was eroded, and the shoulder of the road was damaged (Fig. 2-223). Also, a large quantity of Gobi blocks, riprap, and fill was transported across the highway by wave action (Figs. 2-224, 2-225, and 2-226).

Comparative profiles of similar devices were included in Section III. Profiles through the concrete block revetment are shown in Figure 3-42. Shoreline erosion of 10 to 15 feet is indicated by a comparison of the 9 July 1979 with the 16 January 1980 profiles.

This project was not designed to withstand hurricane attack but rather to reduce maintenance required as a result of damage to the highway and shoulder by less severe storms. To this extent, the Gobi-mat revetment performed its intended function and protected State Highway 82. Had the revetment not been built, the sand chenier on which the road was built could have been severely eroded. Even during Tropical Storm Claudette, when several thousand square feet of shoulder was damaged or displaced, less than 1,000 square feet of roadway was damaged. As of April 1980, damage along parts of the revetment continued toward the traffic lanes and large sections of soil-cement have been displaced by wave action (Fig. 2-227).

(4) <u>Analysis</u>. Failure of the revetment during Tropical Storm Claudette indicated that a substantial amount of damage was due to five features of the revetment. First, although the LEO observer was not present when the revetment failed, it is probable that design conditions were exceeded during the storm. Second, the revetment was inadequately protected from raveling once a few blocks became dislodged. Third, the connection where the pervious block revetment met the impervious asphalt-soil cement shoulder was susceptible to undermining after the blocks holding down the 8-foot-wide overlap of the filter cloth were displaced. Fourth, settlement of the Gobi blocks along the upper filter-cloth anchor trench (while upslope blocks supported by soil-cement held firm) created a zone of weakness where blocks became separated from each other and were then easily displaced by wave action. Fifth, the stitched seams in the filter cloth running parallel rather than perpendicular to the roadway were weaker than the cloth itself and when overstressed, tore apart over long reaches.

In areas with environmental conditions similar to those at Holly Beach, the described Gobi-block revetment should effectively reduce damage due to minor storms. Where tropical storms and hurricanes occur frequently the District Engineer suggests the following modifications to effect a more efficient use of materials and a greater degree of protection. Gobi blocks should be epoxy-glued to the filter cloth to (1) form a large monolithic,

Figure 2-221. Gobi blocks displaced and filter cloth ripped; slope and shoulder intact, Holly Beach, Louisiana, 26 July 1979.

Figure 2-222. Revetment failure (blocks displaced; filter cloth undamaged), Holly Beach, Louisiana, 26 July 1979.

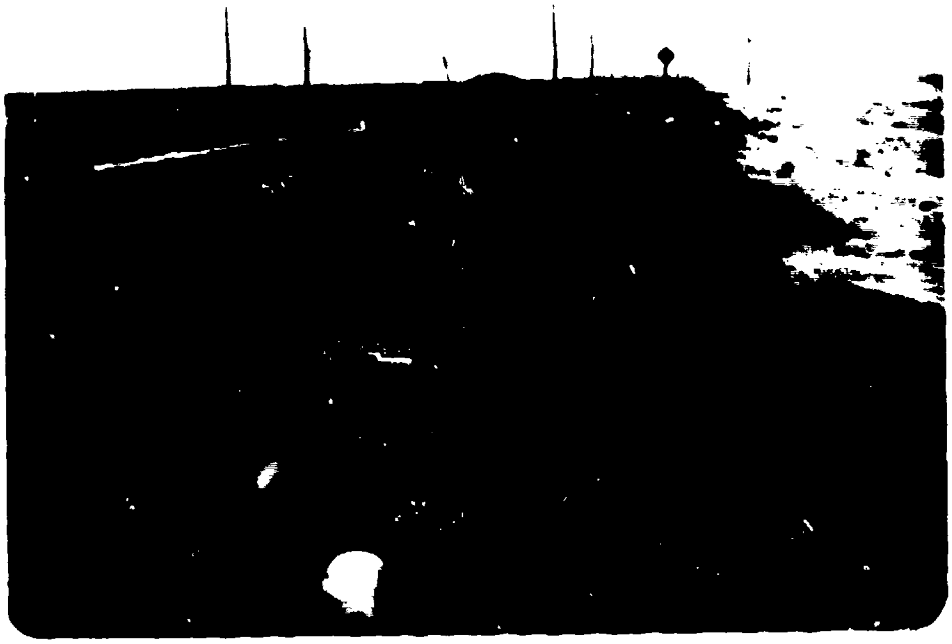


Figure 2-223. Revetment failure and damage to the highway shoulder, Holly Beach, Louisiana, 26 July 1979.

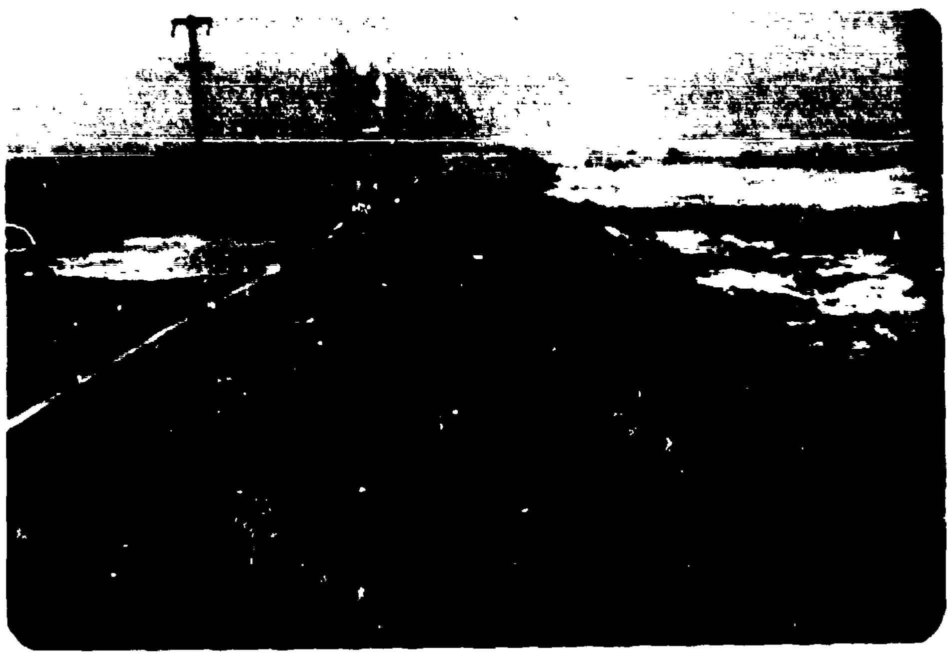


Figure 2-224. Damage to the shoulder and revetment; material transported across highway, Holly Beach, Louisiana, 26 July 1979.

Figure 2-225. Wave runup on highway, Holly Beach, Louisiana, 26 July 1979.

Figure 2-226. Material transported by wave action, Holly Beach, Louisiana, 26 July 1979.

Figure 2-227. During high tides, wave attack and runup undercut road shoulder due to displacement of filter cloth, Holly Beach, Louisiana, 16 April 1980.

articulated mat which deters vandalism, (2) prevent development of gaps between the blocks after any initial settlement or subsequent movement, and (3) prevent raveling after the displacement of a few blocks. Filter cloth should be placed with seams perpendicular, rather than parallel, to the roadway to prevent long reaches of the cloth sections from separating. A larger radius of curvature, or more careful control of construction, is necessary at changes in slope to assure a smooth, continuous revetment.

During significant storms, the raised highway is still overtopped. Without the revetment, major breaches in the road would have occurred at least seven times since 1970. The long-term erosion rate at this site is estimated to be 5 feet per year. Without the road and revetment, the natural sand chenier on which the road was built would have been pushed toward the marsh during storms.

f. <u>Beach City, Texas</u>.

(1) <u>Site Description</u>.

(a) <u>Geographical Setting</u>. The Beach City monitoring site, located along the northwest Trinity Bay shoreline in McCollum County Park within the city limits of Beach City, Texas, is about 8 miles east of Baytown and is accessible from Houston via the U.S. Highway I-10 east to State Road 2354 and then south to the entrance road into McCollum Park (Fig. 2-228). The shoreline at the site is oriented approximately northeast-

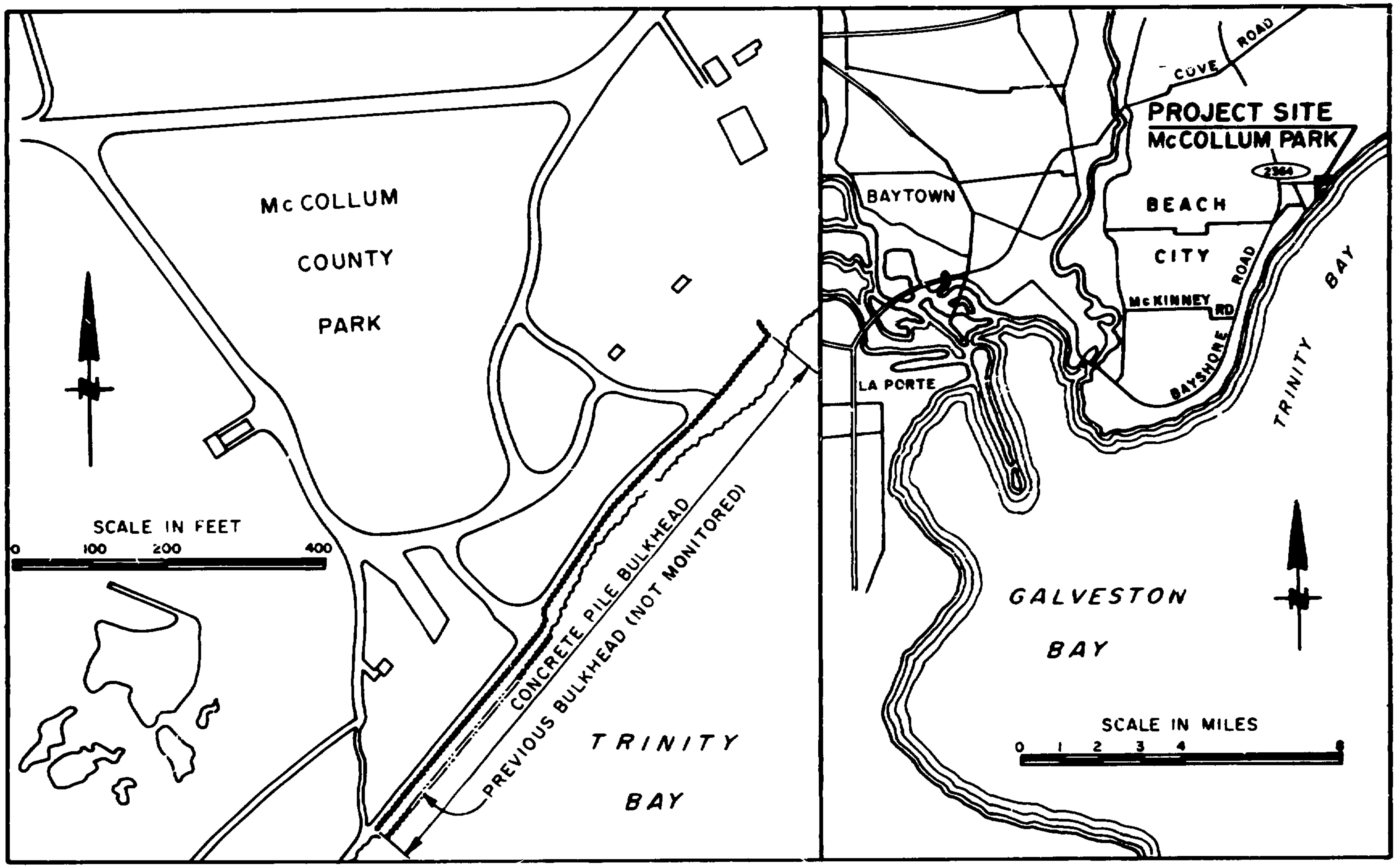


Figure 2-228. Location map of Beach City, Texas, monitoring site.

southwest and is slightly convex to the bay. The shoreline to the northeast consists of a short, gravelly pocket beach.

(b) Geomorphology, Soils, and Vegetation. The site consists of a narrow beach at the base of 15- to 25-foot bluffs. The beach material is mostly clay of high plasticity, with very little sand. There are also some shell fragments along with the silts and clays. Vegetation comprises various grasses, with some shrubbery at the top of the bluff, and covers an estimated 90 percent of the park area. A few small pocket beaches in the area are composed mainly of gravel or shell fragments.

(c) Waves, Tides, and Longshore Transport. The site is exposed to winds from the northeast with a fetch distance of approximately 6 miles across the bay. Waves seldom reach 4 feet in height. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of only 1.8 feet. The wave climate is classified as intermediate. Although the energy-flux analysis for 4 months indicates a moderate net potential to transport littoral material at this site, the problem in this area does not appear to result from longshore transport but rather from direct wave attack at the bluff line.

(d) The Problem. Past records of shoreline changes indicate a 5- to 10-foot-per-year rate of shoreline retreat at the site.

(2) Monitoring Project. To protect the retreating shoreline, Chambers County built an 800-foot-long bulkhead, circa 1976. The bulkhead consisted of a single row of vertically placed concrete pipes, filled with soil. The used pipes were cracked, chipped, or otherwise unsuitable for use as culverts. The pipes were 4 or more feet long with 36-to 90-inch diameters. Previously, an approximately 350-foot-long bulkhead of similar design was constructed of 18- to 36-inch used-culvert pipe at the southern end of the project site. This structure is now about 20 feet in front of the existing structure and it is assumed that failure was a result of Hurricane Carla.

(3) Performance. During July 1979, storm waves caused some degradation of structural materials. The structure was overtopped and a 25-foot-long segment failed, destroying one pipe (Fig. 2-229). Several pipes underwent moderate displacement, and gaps between the pipes allowed some backfill erosion to occur (Fig. 2-230). For the next 4 months, minor deterioration of the structure continued along the segments breached during the storm.

Reconditioning of the damaged section began in November 1979. By December 1979, the entire structure had been totally reconditioned, with spaces between all the pipes filled with concrete mortar, and earthfill placed in the pipes and behind the structure (Figs. 2-231 and 2-232).

During the January 1980 site visit, a rain shower caused damages due to inadequate drainage (Figs. 2-233 and 2-234). In February 1980, capping of the structure with a 2- to 4-inch concrete cap, placed on top of and behind the pipe structure, was begun (Fig. 2-235). At this time, it was noted that a large hole had been washed out near a drainpipe at the southwest end of the shoreline (Figs. 2-236 and 2-237). In March 1980, the washed-

Figure 2-229. Destroyed pipe along bulkhead, Beach City, Texas, 17 August 1979.

Figure 2-230. Displacement of pipes and backfill erosion, Beach City, Texas, 17 August 1979.

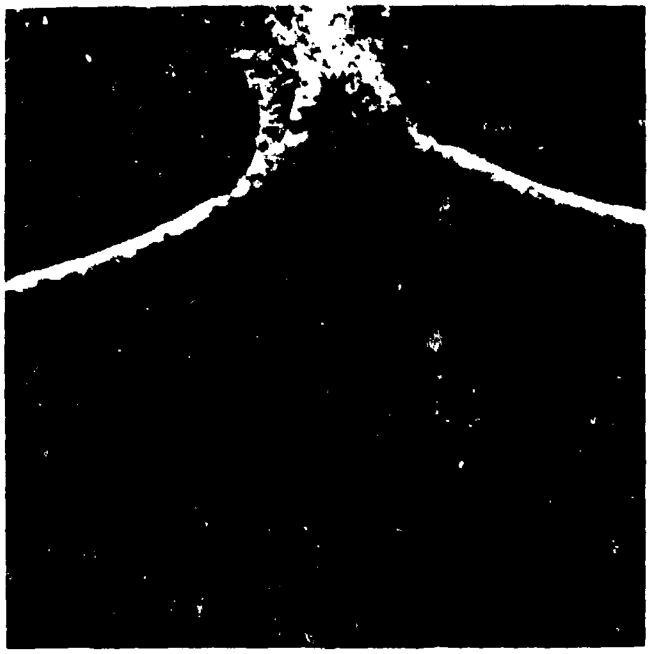


Figure 2-231. Gap filled with mortar, Beach City, Texas, 20 November 1979.

Figure 2-232. Backfill of structure; note seaward row of pipes installed in an earlier project, Beach City, Texas, 21 December 1979.

Figure 2-233. Drainage during rain showers, Beach City, Texas, 22 January 1980.

Figure 2-234. Drainage during rain showers, Beach City, Texas, 22 January 1980.

Figure 2-235. Concrete cap on top of and behind pipe structure, Beach City, Texas, 26 February 1980.

Figure 2-236. Location of pipe drain in Figure 2-237, Beach City, Texas, 26 February 1980.

Figure 2-237. Large hole washed out near drainpipe at southwest end of shoreline, Beach City, Texas, 26 February 1980.

out hole was filled with concrete rubble. Also in March, concrete rubble was placed as toe protection in an effort to control toe scour occurring along some sections of the device (Figs. 2-238 and 2-239). In April 1980, additional broken concrete was placed at the toe of the structure for protection. As of May 1980 no additional structural damage had occurred.

Profiles through the earth-filled pipe bulkhead are shown in Section 3 (Fig. 3-20). Due to a short survey period, the structure's performance cannot be determined from the two surveys. However, the 10 July 1979 survey shows the erosion behind the structure due to a storm earlier in the month.

(4) Analysis. The structure as first installed was inadequate in preventing shoreline erosion because of (a) gaps between pipes which allowed backfill erosion, (b) inadequate toe protection which allowed toe scour, and (c) poor drainage which increased erosion. The use of granular backfill might have served to relieve hydrostatic pressure.

One section of the structure failed during the only major storm that occurred in the monitoring period. After the failed section had been repaired and the gaps between all pipes filled with concrete mortar, the structure functioned well in preventing backfill erosion, but toe scour continued to be a problem. Setting the pipes deeper could have reduced the toe scour problem. Also, poor drainage of runoff from rains and from overtopping waves tended to erode the pipe-fill material, as well as the backfill behind the structure. Capping the pipes and hardening the area immediately behind the structure helped to protect the structure; however,

Figure 2-238. Toe scour, Beach City, Texas, 21 December 1979.

Figure 2-239. Broken concrete placed as toe protection, Beach City, Texas, 31 March 1980.

no relief of hydrostatic pressure was provided. Backfill pressure could cause pipes to lean seaward, as had happened previously, resulting in eventual failure of the structure.

Although inadequate as originally installed, the structure after repairs and changes, prevented further shoreline erosion during the monitoring period. However, signs of incipient failure of the pipes (Figs. 2-234 and 2-240) indicated that the lifespan of the structure may not be as long as expected. A longer monitoring period is needed to adequately assess the performance of the structure.

g. Shoreacres, Texas.

(1) Site Description.

(a) Geographical Setting. The Shoreacres monitoring site is located on the northwest shore of upper Galveston Bay (Fig. 2-241), about 15 miles southeast of Houston, 14 miles southwest of the Beach City monitoring site, and 3 miles southeast of La Porte, Texas. It is accessible by taking State Highway 146 north to the site. The project occupies about 3,100 feet of the shoreline of a shallow bight, concave toward the bay. The shoreline trends approximately northwest-southeast.

(b) Geomorphology, Soils, and Vegetation. The natural shoreline in this area consists of a narrow sand beach with 2- to

Figure 2-240. Start of pipe failure, Beach City, Texas, 22 January 1980.

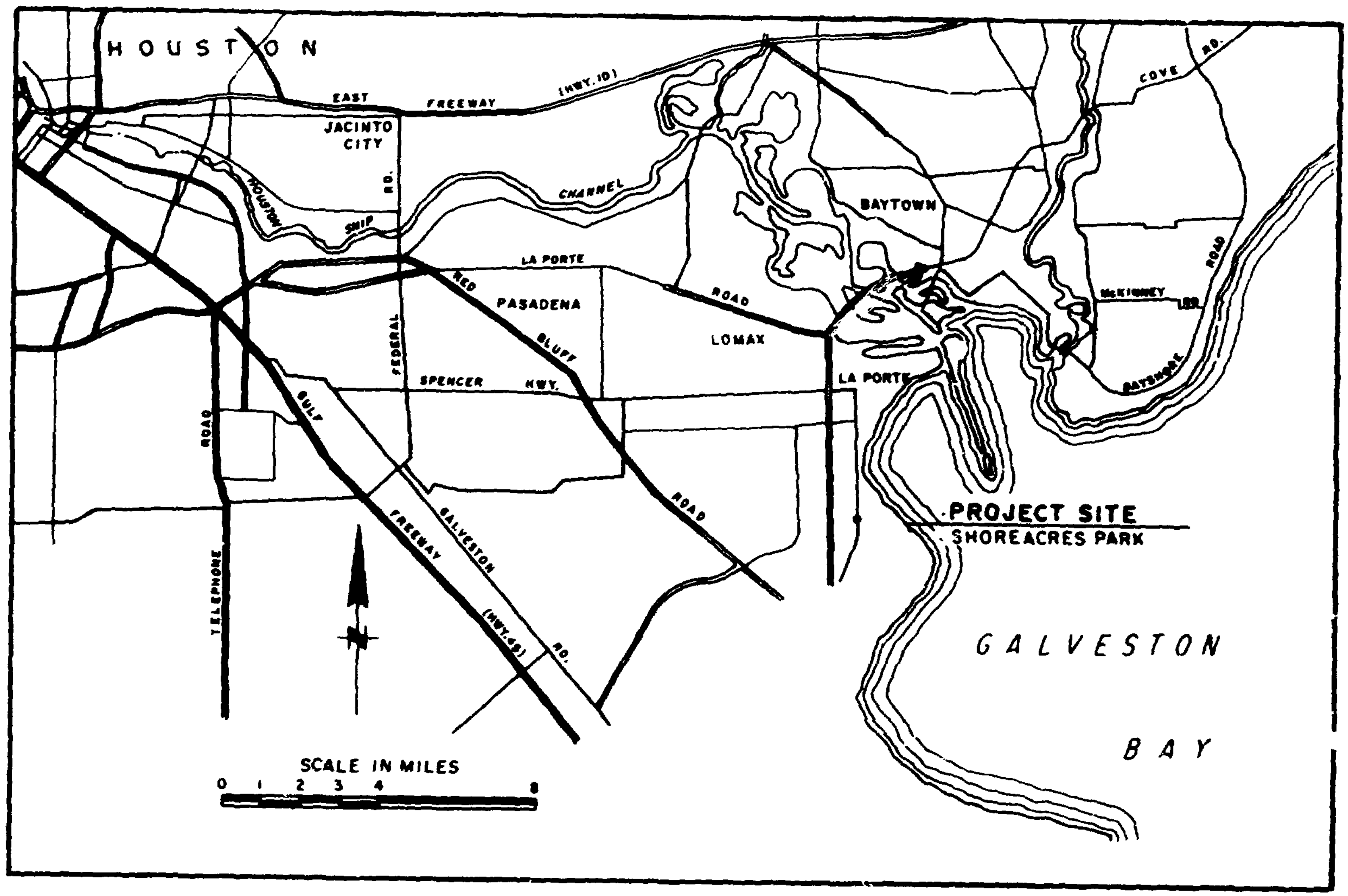


Figure 2-241. Location map of Shoreacres Park, Texas, monitoring site.

5-foot-high bluffs composed of clays and silts. A relatively narrow flatland covered with about 75 percent vegetation lies behind the bluffs.

(c) Waves, Tides, and Longshore Transport. The site is sheltered from waves moving across Galveston Bay by islands created by dredged material placed parallel to the Houston Ship Channel. The fetch distance is about 3 miles, and waves seldom exceed 3 feet in height. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 1.8 feet. The wave climate is classified as intermediate. Although the energy-flux analysis for 4 months indicates a moderate net potential to transport littoral material at this site, the problem in this area does not appear to result from longshore transport but rather from direct wave attack at the bluff line.

(d) The Problem. Prior to 1976, the shoreline at the site had been receding at the rate of 2 to 5 feet per year.

(2) Monitoring Project. In 1976, a riprap revetment of reinforced-concrete rubble was placed along the shoreline. No data are available on construction procedures or whether any restrictions were placed on sizes of rubble used. Figure 2-242 is a photo of a typical segment of the riprap; Figure 2-243 shows the project plan at the site.

(3) Performance. No significant degradation of structural materials has occurred since installation. One significant storm occurred in August

Figure 2-242. Typical segment of the riprap, Shoreacres, Texas, 19 July 1979.

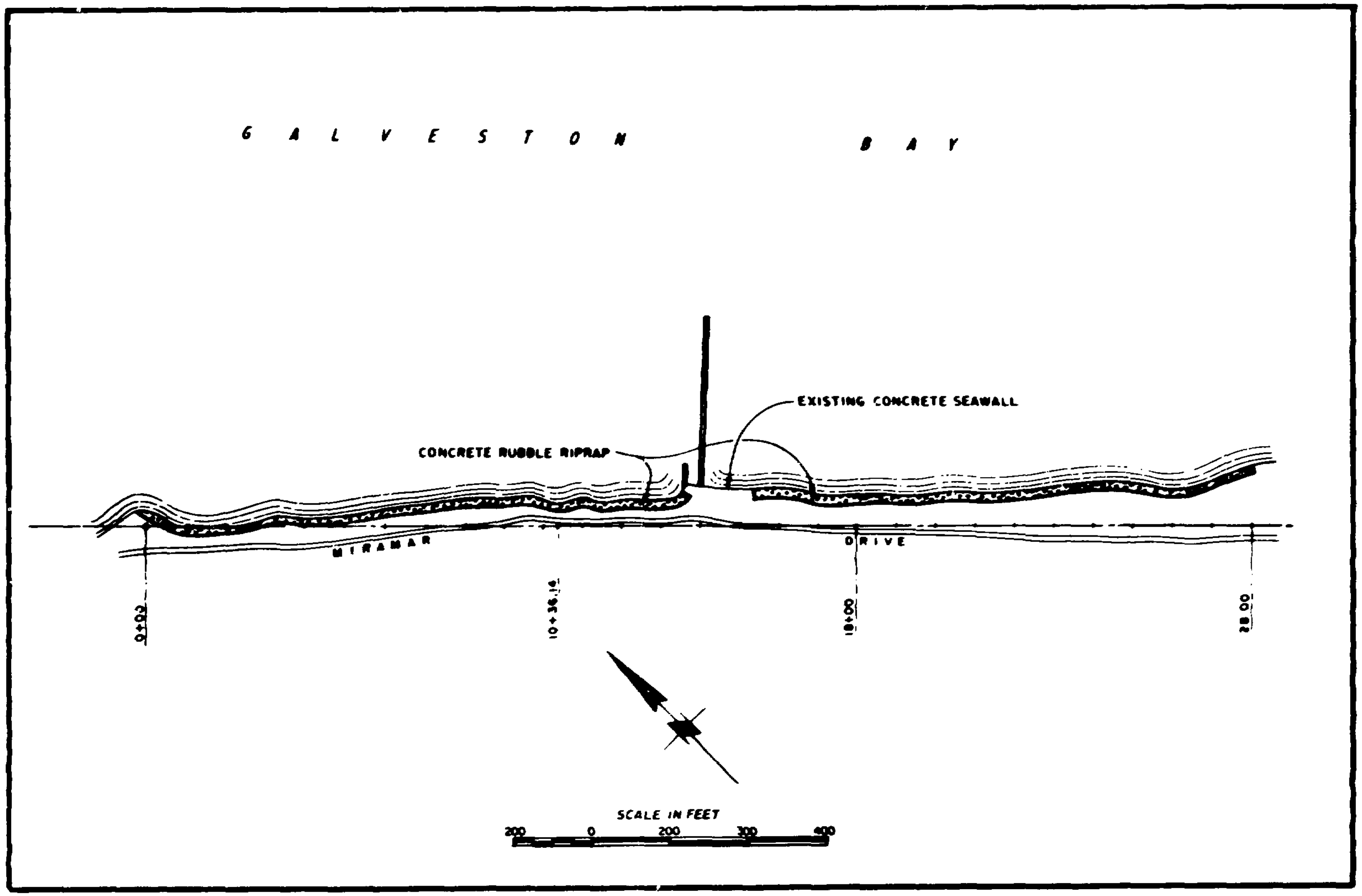


Figure 2-243. General plan of Shoreacres, Texas, monitoring site.

1979; however, no noticeable changes were observed. The riprap has performed well functionally, preventing shoreline recession. In May 1980, the bank sloughed slightly in a few places, but caused no significant damage. Profiles of similar concrete rubble revetments are shown in Section III; the profiles through the revetment at Shoreacres are shown in Figure 3-43. Profiles show no erosion at the site.

(4) Analysis. Although the site is sheltered from high waves, the riprap structure has performed well structurally and functionally. Apparently the rubble contained the proper gradation of sizes to form its own filter layer under wave agitation. A design filter layer of graded rubble or filter cloth might have prevented the bank sloughing in May 1980.

4. Pacific Coast Sites.

a. General. Two demonstration sites and two monitoring sites are on waters connecting with the Pacific Ocean (Fig. 2-244). The demonstration sites are on the San Francisco Bay shore of Alameda, California, and on a small peninsula near Oak Harbor, Whidbey Island, Washington, fronted by an inland passage off Puget Sound. The monitoring sites are at Sunnyside Beach on the east shore of a passage off the south end of Puget Sound near Tacoma, Washington, and along the southwest bank of the Siuslaw River estuary at Siuslaw River, Oregon. The climate is generally cool, with fairly low humidity. Winter snowfalls occur only occasionally at the Washington and Oregon sites. Strong winds blow occasionally in winter at all sites, but none are associated with hurricanes. Pacific Ocean tides have a diurnal inequality, and MLLW is the datum plane generally used in coastal areas. West coast tides usually have a greater range than those on the east and gulf coasts.

b. Alameda, California.

(1) Site Description.

(a) Geographical Setting. The city of Alameda occupies the central and southwest part of an island, in central San Francisco Bay, which is separated from the city of Oakland by the Oakland Inner Harbor and San Leandro Bay. The northwest end of the island is occupied by the Alameda Naval Air Station. Bay Farm Island to the south is separated from Alameda by San Leandro Bay and its outlet, the San Leandro Channel. Figure 2-245 shows the project location. The shoreline is oriented in about a northwest-southeast direction.

(b) Climate. The San Francisco Bay area has a generally pleasant temperate climate that is moderated by its large water area and proximity to the Pacific Ocean. Although summer temperatures often exceed 100° Fahrenheit in the Sacramento Valley a few miles to the east, Alameda usually remains fairly cool as a result of the daily sea breeze. Fog blown in from the ocean often limits visibility and keeps the air moist. Winter temperatures are seldom below freezing, and snowfall is rare. Eastward-moving cyclonic disturbances in the North Pacific Ocean bring considerable rain to the area in winter, but summers are mostly dry.

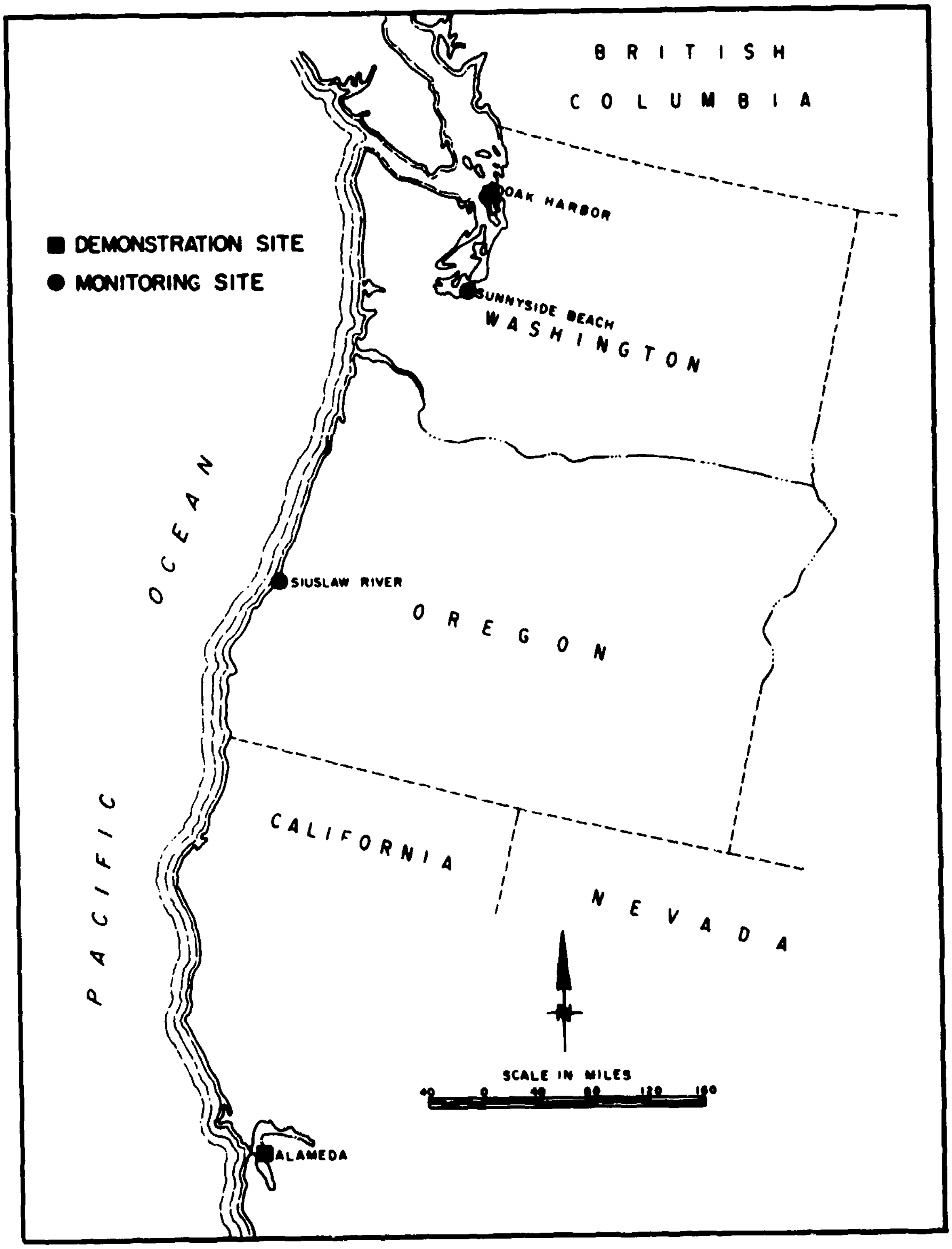


Figure 2-244. Pacific coast demonstration and monitoring sites.

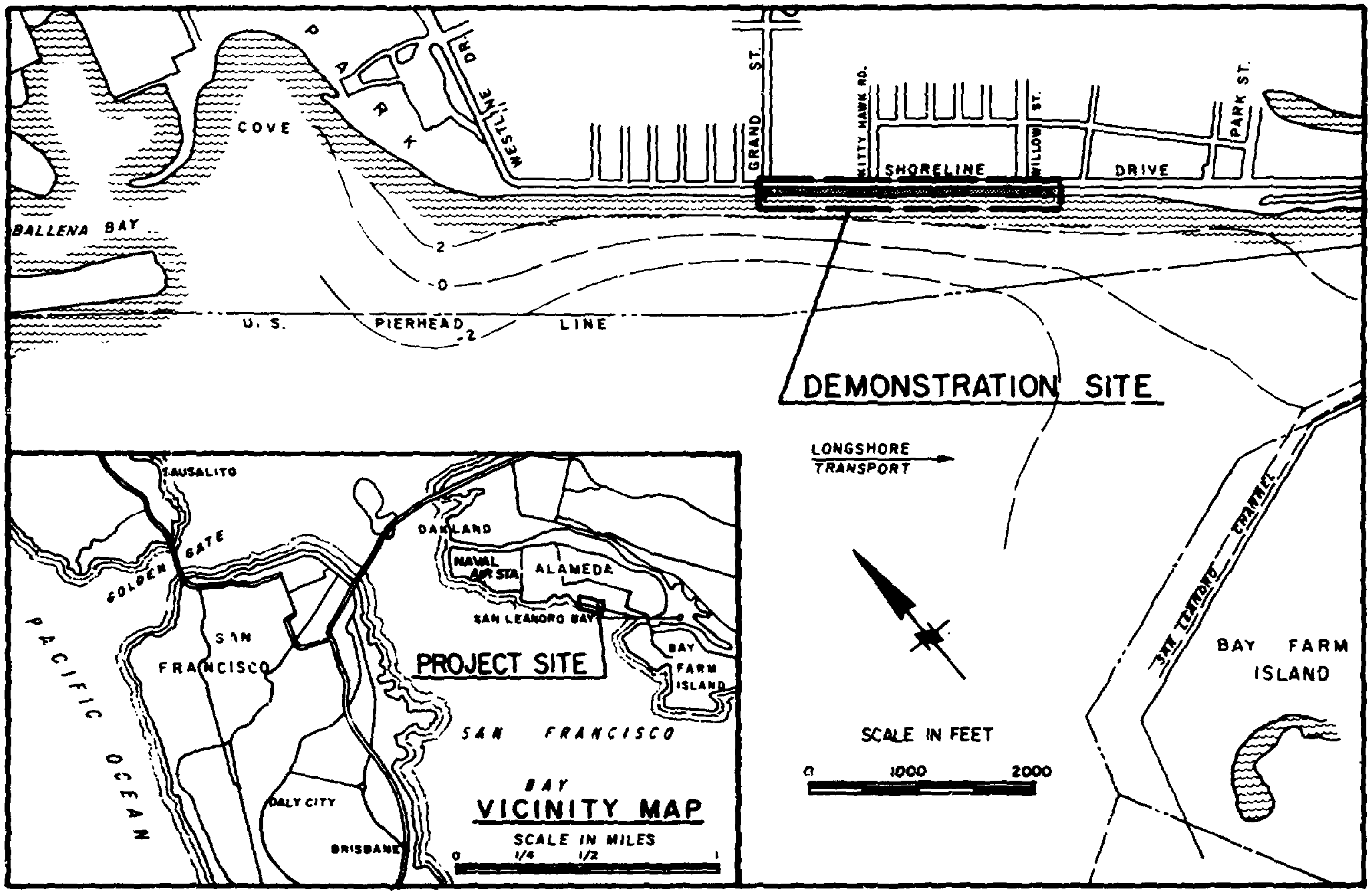


Figure 2-245. Location map of Alameda, California, demonstration site.

(c) Wind, Waves, Tides, and Longshore Transport. The prevailing wind throughout the bay area is from the west, and the average speed at Alameda is about 9 knots. Figure 2-246 is a wave rose developed by hindcasts from the Alameda Naval Air Station wind data. The only significant wave activity affecting the site is that generated by local winds; thus, the prevailing wave direction is also westerly. Wave periods usually range from 2 to 4 seconds, and wave heights exceed 1 foot 30 percent of the time. The tidal range between mean lower low water (MLLW) datum and mean higher high water (MHHW) is 6.4 feet. Extreme high water is +8.8 feet MLLW. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 7.0 feet; the net longshore transport potential southeastward was 36,500 cubic yards for the 6 months analyzed. The wave climate is classified as intermediate.

(d) Geomorphology, Soils, and Vegetation. The site of the demonstration project is located on a hydraulically placed sandfill that was dredged from San Francisco Bay in the late 1950's. The hydraulic fill was placed over recent soft bay mud sediments ranging in thickness from 0 foot at the original shoreline to greater than 60 feet at a distance of 2,400 feet offshore. Mud depth at the present shoreline varied from 20 to 40 feet in thickness before placement of the fill. The recent bay mud is underlain by firm clay and sands of Pleistocene age. Bedrock lies at an estimated depth of between 400 and 1,000 feet below the existing surface. Recent surface sediments consist of a fine to medium sand (grain size 0.2 to 0.3 millimeter) on the beach with mudflats offshore. Mud boils penetrate the sand surface in several areas of the beach.

Sediment samples collected from the beach, tombolo, and offshore areas show very uniform gradations. Figure 2-247 illustrates the gradation of a sample collected from the tombolo formed by littoral deposition. All other samples have similar gradation characteristics.

Natural vegetation along the shoreline is scarce. However, just east of the project site, a fairly dense growth of intertidal vegetation has developed, mainly Pacific cordgrass (*Spartina foliosa*) fringed with pickleweed (*Salicornia* sp.), providing excellent wildlife habitat.

(e) The Problem. Since the artificial shoreline was formed in 1959, beach erosion has been a problem. It has progressed to a critical stage along several areas of the Shoreline Drive (Fig. 2-248). Wave data suggest that more than half of the waves set up by wind and storms approach the beach at an angle of 45° or less. This angle of approach sets up strong littoral currents parallel to shore; these currents are the primary source of erosion at the site. The net longshore transport rate is estimated to be more than 8,000 cubic yards in the southward direction. Theoretical estimates of the potential transport rate, based on unlimited availability of sand, are about 80,000 cubic yards annually. Beach erosion is estimated at 4 to 10 feet annually. Previous attempts to reduce erosion by beach nourishment and use of rubble revetments have been unsuccessful. Longsnore currents have exposed several storm sewer pipes, which now act as groins perpendicular to shore. These pipes have accumulated littoral drift on their updrift sides and therefore increased the erosive capacity of the littoral current on the downstream side. This has accelerated erosion along the beach in some areas southeast of the exposed drains.

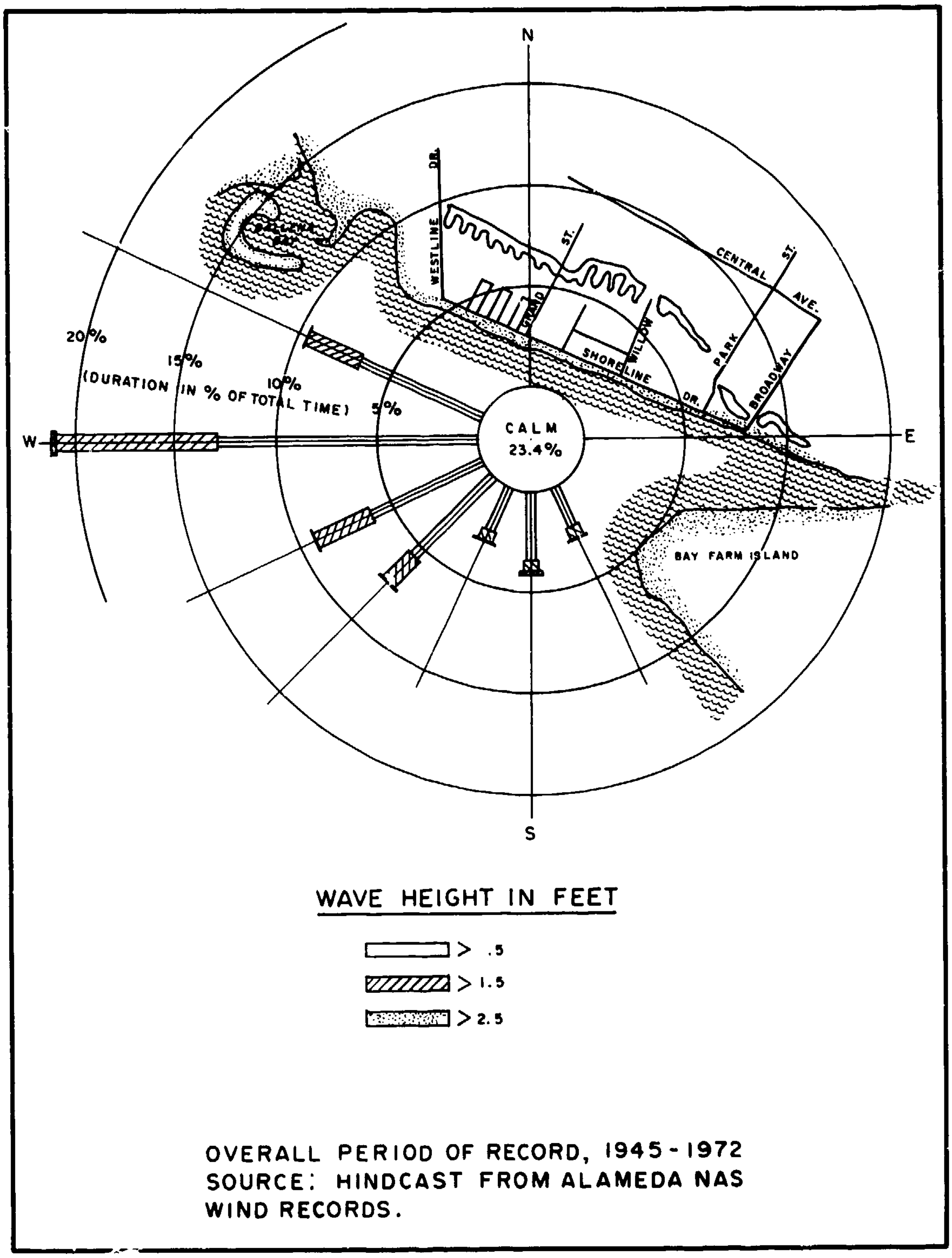
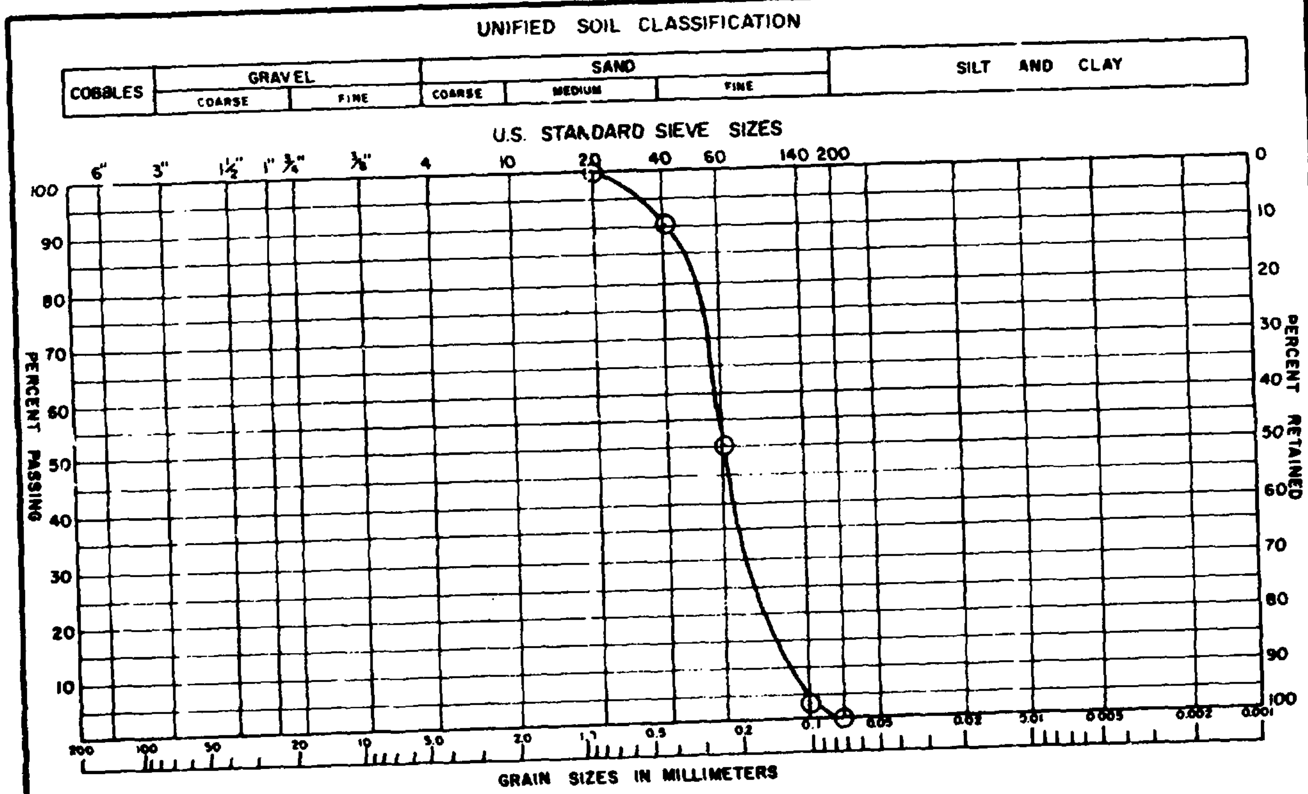


Figure 2-246. Wave rose for Alameda, California.



Typical Grain-Size Curve

LOCATION	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
Tombolo		⊙—⊙			Fine SAND (SP)

Figure 2-247. Representative grain-size distribution curve, Alameda site.

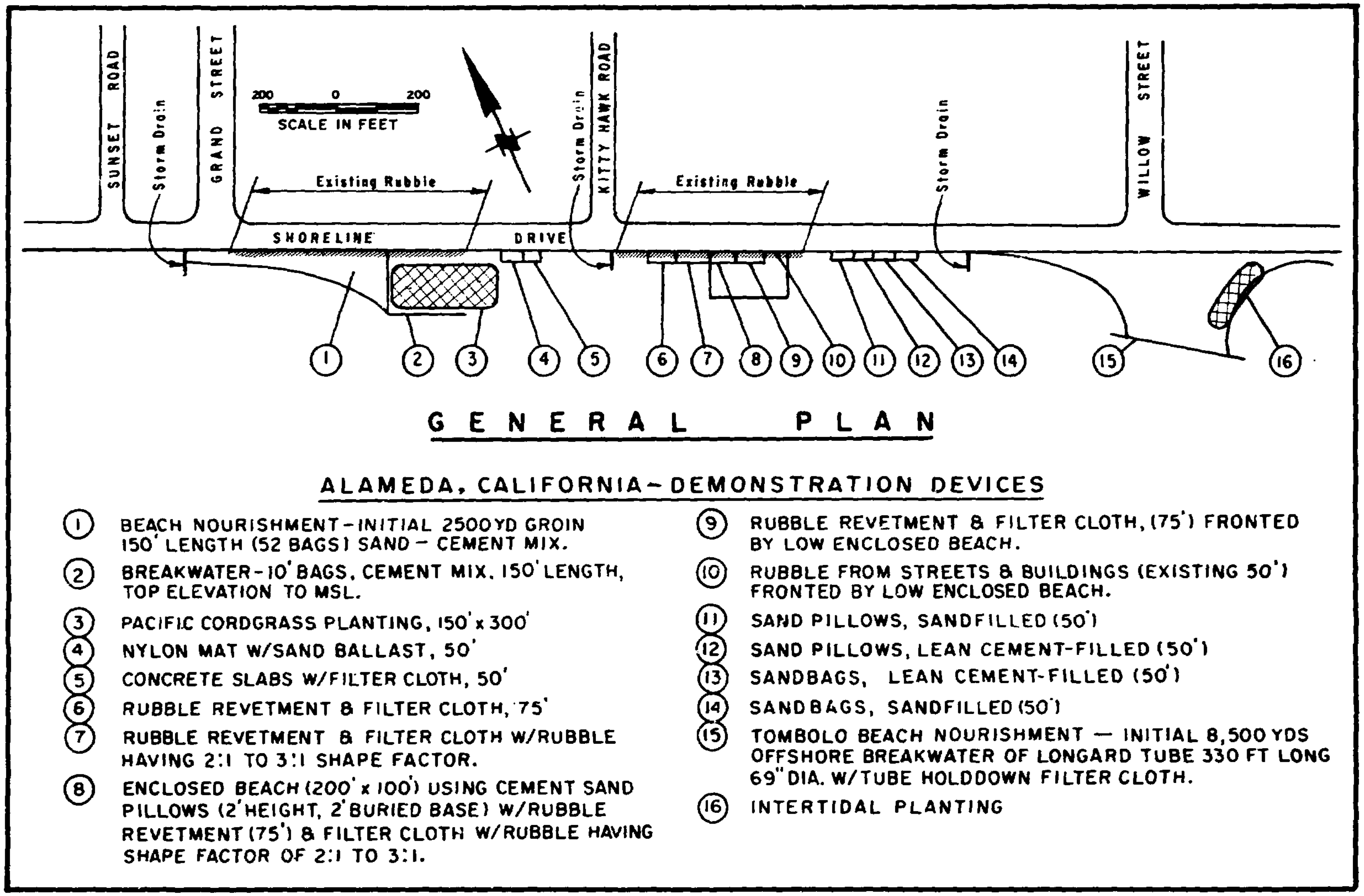


Figure 2-248. General plan of Alameda, California, demonstration site.

(2) Demonstration Project. The installations evaluated at the Alameda site included both structural devices and vegetation plantings. Five categories of devices were selected: beach nourishment, groins, revetments, offshore sills and breakwaters, and vegetation. Beach nourishment was examined in order to monitor it as a category by itself and as a possible source for a downdrift entrapment device. Success of the entrapment device would preclude aggravation of an existing shoaling problem at the southeast end of the beach. Only one groin was tested because the effectiveness of the groin category had already been demonstrated by the groin effect of the existing storm drains. Six types of revetments were evaluated. Rubble previously used at the site to retard bank erosion presented an opportunity to evaluate two revetment materials, shape-sized and existing rubble (building and street debris), both with and without a filter. In addition, each rubble revetment was installed both fully exposed and fronted by a perched beach. The other four revetment devices comprised a sandfilled nylon mat, concrete slabs over filter cloth, Sand Pillows, and sandbags. Low sills to retain beach fill were constructed with sand-and cement-filled bags in two areas, and one offshore breakwater was constructed with a Longard tube. Because of the flatness of the beach, a single tube, exposed at low tide, was used. Beach material was placed behind the breakwater and shaped into a tombolo configuration that wave action was expected to produce. Intertidal vegetation, both exposed to waves and on a perched beach retained by a low sill, was tested for the vegetation category. This included planting intertidal species on the backfilled tombolo behind the offshore breakwater. Figure 2-221 describes and shows the location of each device. Construction data for each type and variation of device for the Alameda project are outlined and presented in this subsection. The structural work began in October 1978 and was completed in August 1979. The U.S. Army Engineer District, San Francisco, personnel supervised the construction. Vegetation was planted in fall 1978 and spring 1979 by the San Francisco Bay Marine Research Center. All elevations refer to MLLW datum.

Many of the devices at Alameda, including some that were still under construction, were damaged during the storm of 26 and 27 March 1979, which brought heavy rains and high winds. Predicted elevations of the two high tides on 27 March were 5.8 and 5.7 feet; the actual recorded tide elevations were 6.4 and 6.2 feet. A LEO observer estimated the breaking wave heights to be in the 2.5- to 3.0-foot range. Damage to the shore protection devices at that time was primarily limited to erosion of backfill and displacement of sand at the toes of the devices. During the succeeding month some of the devices were severely damaged or failed completely; others survived and continued to function.

(a) Device 1 (Beach Nourishment and Groin).

1 Statistics, Construction, and Costs. Device 1 statistics are given in Table 2-46, and details are shown in Figure 2-249.

In early November 1978, construction of device 1 began with the dumping of 2,500 cubic yards of sand on the 540 feet of beach to be retained by the groin. The sand was transported to the site in dump trucks and dumped onto the embankment. Bulldozers then spread and graded the sand to its estimated equilibrium fillet configuration. The groin at the downdrift end of the beach fill was constructed of nylon sandbags filled to about 80-percent

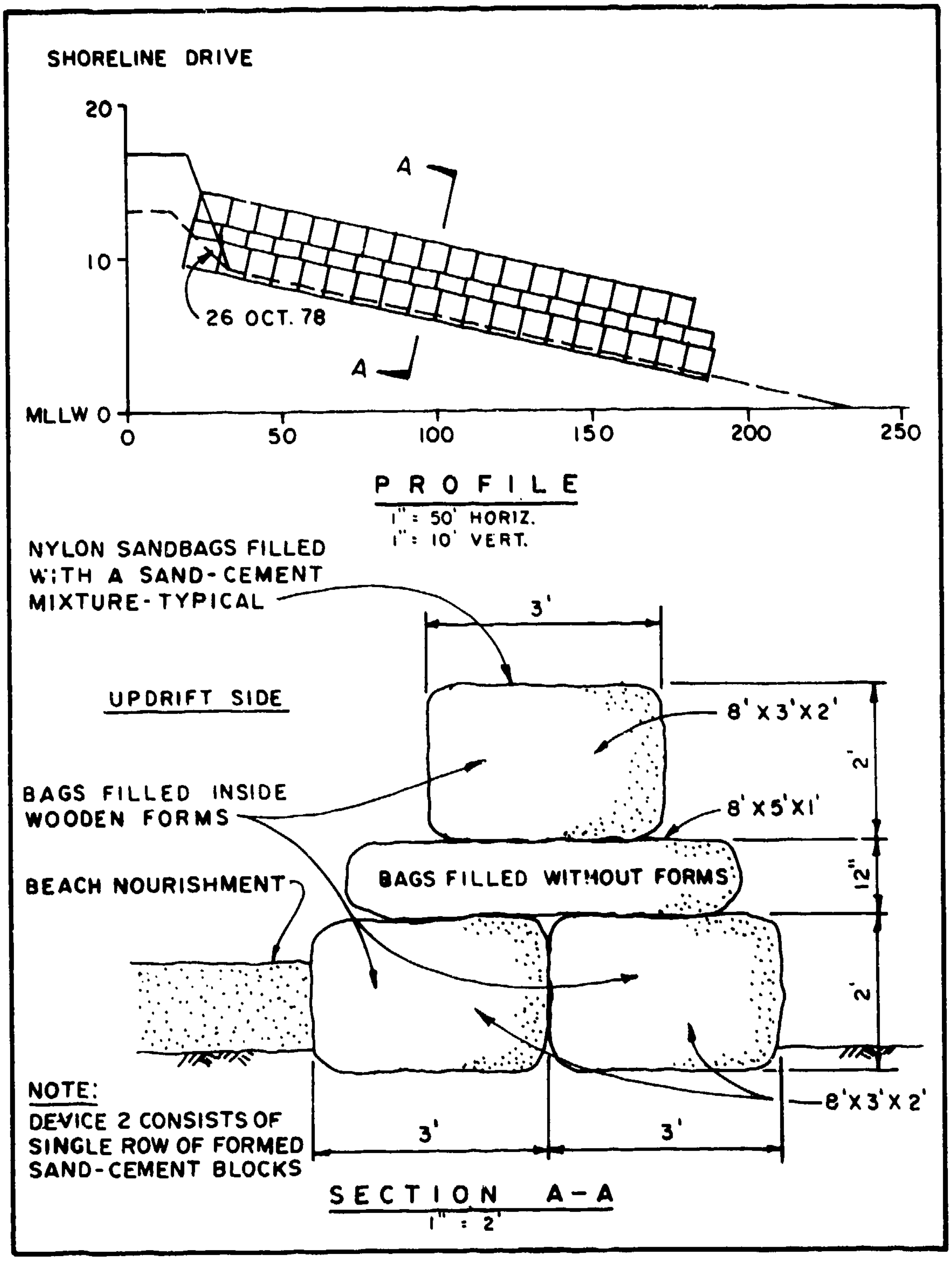


Figure 2-249. Device 1 at Alameda site.

Table 2-46. Device 1 statistics.

	Beach	Groin
Length	540 ft	150 ft
Top elevation	+3.7 ft	+14.0 ft
Toe elevation	+2.5 ft	+2.0 ft
Structure slope	Approx. horizontal	0.08(profile)

capacity from the chute of a ready-mix truck with a lean (8-to-1) sand-cement mixture. Care was taken to assure contact between the bags. The sandbags were supported on each side by wooden forms, which allowed the mixture to set as a 8- by 3- by 2-foot block (Fig. 2-250). The bottom tier consisted of two such blocks, side by side. On the second tier, however, forms were not used, and the sand-cement mixture spread out and set as a 8- by 5- by 1-foot block (Fig. 2-251). As the sand buildup along the west side began to top the groin crest, the District Engineer increased the height of the groin, and in January 1979 a third tier of bags was added using the wooden forms (Fig. 2-252). Construction costs for device 1 are given in Table 2-47.

Table 2-47. Device 1 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft[1]
Materials					$ 9.13
Sand beach fill	2,500	yd^3	$ 0.80	$2,000	
Nylon sandbags	68	ea.	31.00	2,100	
Sandfill	160	yd^3	1.50	240	
Cement fill	385	sacks	4.15	1,600	
Misc. supplies				400	
Subtotal				6,300	
Labor	1,963	hr	6.11	12,000	17.39
Equipment					9.33
Cat. D-4	16	hr	25.00	400	
Cat. D-7 bulldozer	90	hr	33.33	3,000	
Dump truck				240	
Concrete mixer truck	128	hr	12.50	1,600	
Case backhoe	96	hr	12.50	1,200	
Subtotal				6,440	
Total				$24,700	$35.85

[1]Cost/ft is based on the length of beach.

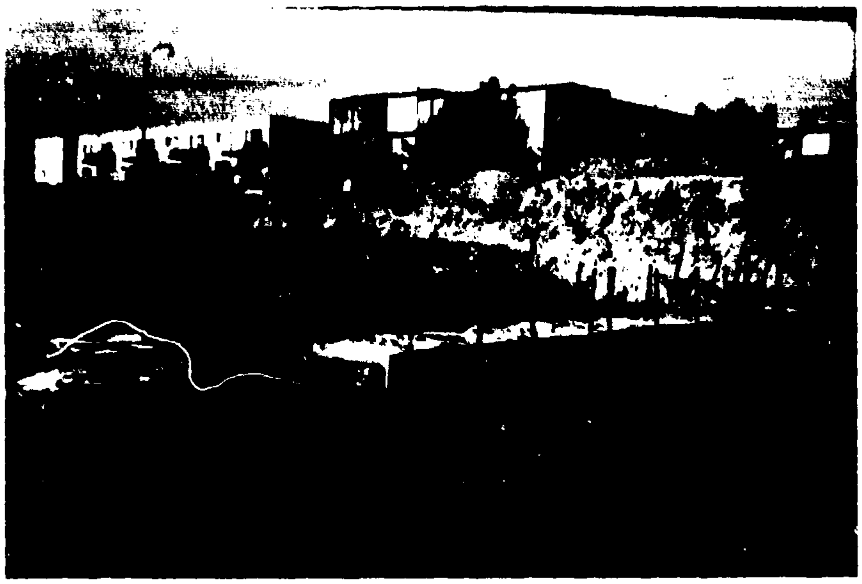


Figure 2-250. Beach nourishment and groin, device 1, Alameda, California, November 1978.

Figure 2-251. Beach nourishment and groin, device 1, Alameda, California, 7 November 1978.

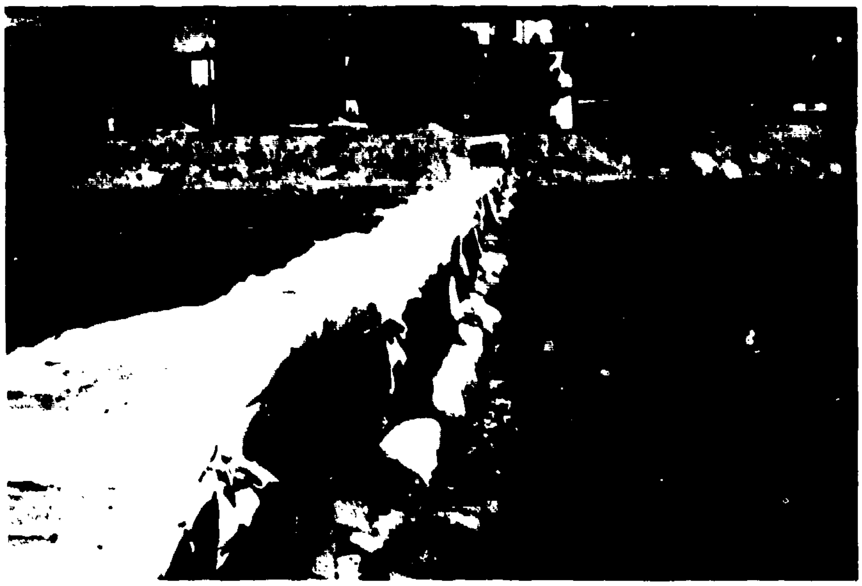


Figure 2-252. Beach nourishment and groin, device 1, Alameda, California, 23 March 1979.

2 Performance. Some of the nylon bags on the top tier deteriorated where exposed to sunlight, but as the fill material was a mixture of sand and cement, the blocks held their shape (Fig. 2-253).

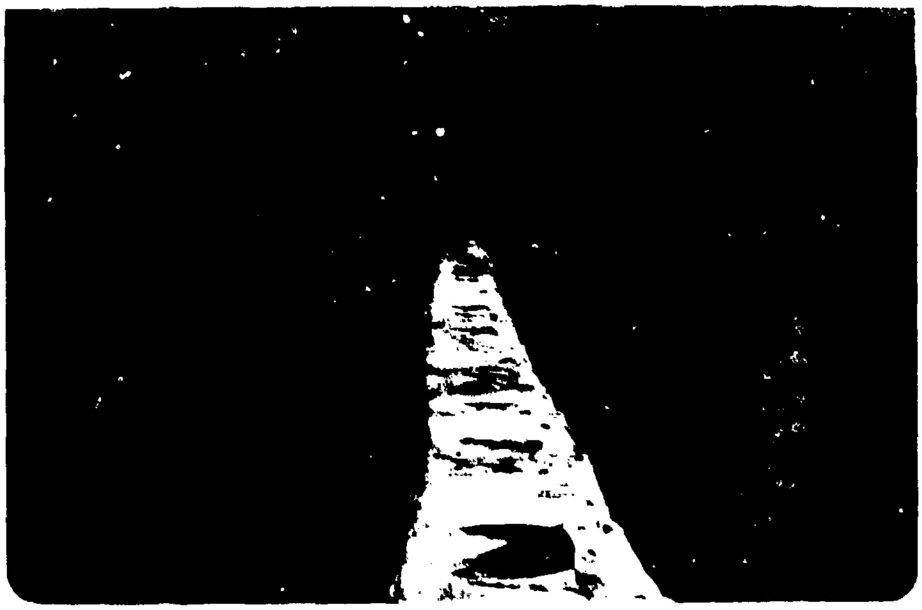


Figure 2-253. Beach nourishment and groin, device 1, Alameda, California, 9 August 1979.

Along the 500 feet of shoreline that received beach nourishment, the general beach alinement remained unchanged, and sand gradually accreted adjacent to the groin. The elevation of the sand buildup, being higher than MHHW (+5.5 feet), gave additional protection to the existing embankment. By September 1979 a buildup of 1 to 3 feet had occurred adjacent to the groin. The initial plan for this installation called for an additional 2,500 cubic yards of beach nourishment after 1 year. However, the predominant eastward littoral transport and the trapping capacity of the groin made the additional beach nourishment unneccessary. The beach fillet trapped by the groin not only prevented further embankment erosion west of the groin, but provided an esthetically pleasing beach area which attracted a large attendance. By May 1980, 2 to 3 feet of accretion had formed adjacent to the groin, but some erosion was observed at the junction of the groin and the bank.

(b) Device 2 (Nylon-Bag Breakwater).

1 Statistics, Construction, and Costs. Device 2 statistics are given in Table 2-48.

Table 2-48. Device 2 statistics.

Length	150 ft
Top elevation	+4.0 ft
Toe elevation	+2.0 ft
Structure slope	horizontal

Construction of the breakwater began in mid-October 1978 using the same procedure as that used in constructing the groin. The breakwater cross section, however, consisted of only one formed sand-cement block (8 by 3 by 2 feet) and the wooden forms used to shape the filled bags into blocks were upgraded as shown in Figure 2-254. The breakwater was completed on 9 November 1978. Construction costs for device 2 are given in Table 2-49.

Table 2-49. Device 2 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$ 6.87
Nylon sandbags	17	ea	$31.00	$ 530	
Sandfill	40	yd^3	1.50		
Cement fill	96	sacks	4.15	400	
Misc. supplies				100	
Subtotal				1,030	
Labor	338	hr	4.73	1,600	10.67
Equipment					5.07
Dump truck				60	
Concrete-mixer truck	32	hr	12.50	400	
Case backhoe	24	hr	12.50	300	
Subtotal				760	
Total				$3,450	$23.00

Figure 2-254. Nylon-bag breakwater, device 2, Alameda, California, November 1978.

2 Performance. The first followup inspection on 28 March 1979 revealed that the structure had subsided an average of about 6 inches (Fig. 2-255). In some segments the crest dropped as much as 12 inches, and the differential settlement caused one block to buckle and crack. The rylon bag material deteriorated along the top face, but as with the groin, the cement mixture provided sufficient strength to maintain the block dimensions (Fig. 2-256).

Despite the subsidence of parts of the breakwater, it continued to function as a low sill, retaining a fillet of sand cast over its crest by waves and protecting the device 3 vegetation plantings from wave damage. Sand accretion was most noticeable in the southwest corner adjacent to the groin. The nonuniform accretion of sand caused the formation of a small tidal pool in the planted area (Fig. 2-255). Considerable erosion occurred along the bank behind devices 2 and 3, attributed to the material being disturbed during construction. Emergency rubble was placed on the bank in November 1979 (Fig. 2-257). By May 1980, considerable shoaling (up to 1 foot or more) had occurred on the bayside of the breakwater (Fig. 2-258).

Figure 3-76, which was put in Section III for comparison of profiles through similar devices, shows a series of profiles through the nylon-bag breakwater at Alameda. The profiles of December 1978 and November 1979 show about 30 to 40 feet of accretion in lee of the sill.

(c) Device 3 (Pacific Cordgrass Planting).

1 Statistics. Plantings of Pacific cordgrass were made in both protected and unprotected areas behind the breakwater (device 2). Areas 1 and 2 were planted in fall 1978 with some rows of plants being

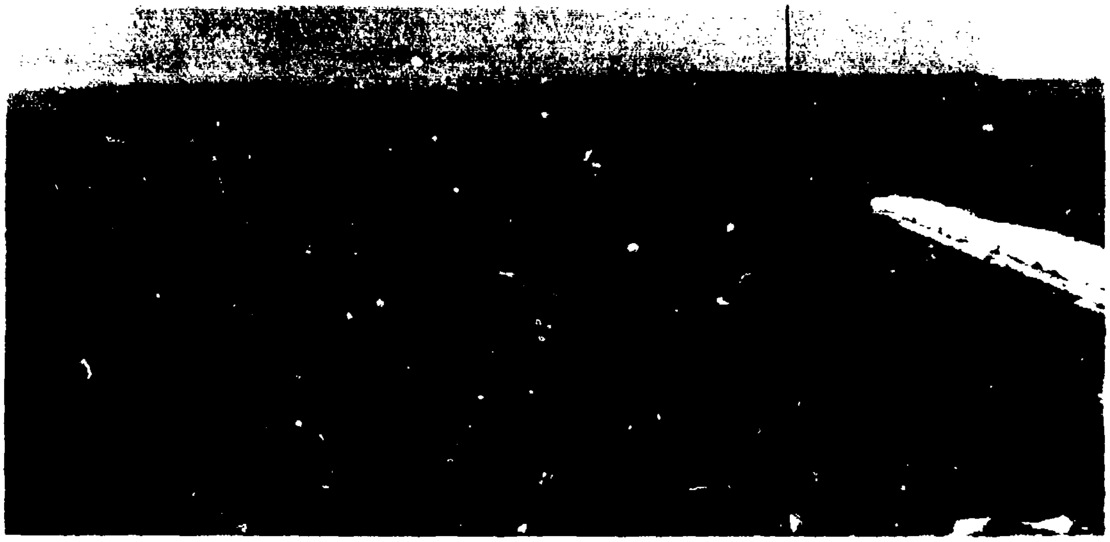


Figure 2-255. Nylon-bag breakwater, device 2, Alameda, California, 28 March 1979.

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5.30.79

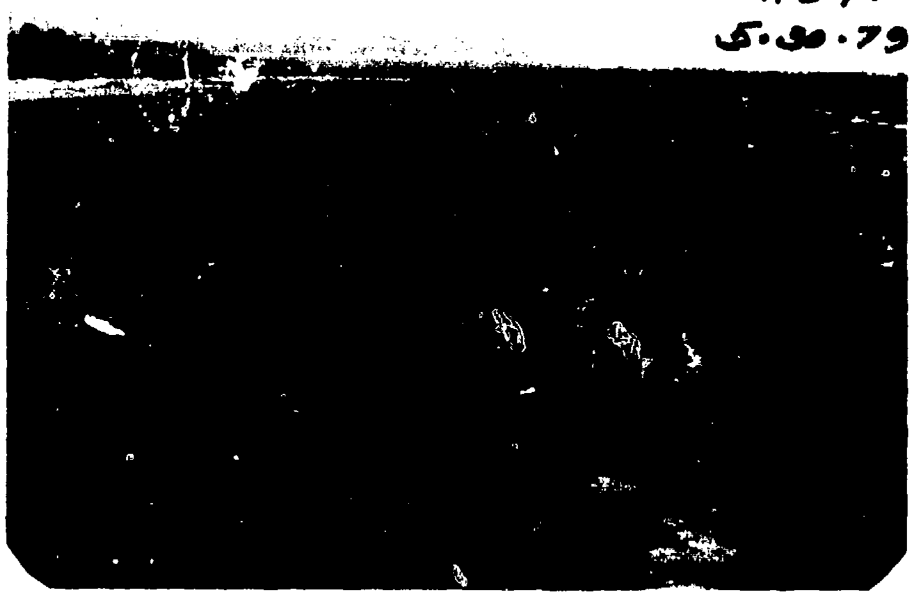

Figure 2-256. Nylon-bag breakwater, device 2, Alameda, California, 30 May 1979.

Figure 2-257. Erosion and emergency rubble on embankment, Alameda, California, 2 May 1980.

Figure 2-258. Accretion on bayside of device 2, Alameda, California, 2 May 1980.

established behind small wave breakers made with wooden roofing shingles. These shingle wave breakers are dug into the sand in front of each planting and extend 8 to 10 inches into the air. Areas 3 to 11 were planted in spring 1979. Particulars of planting in each area are:

(1) Fall 1978. Low-density protected sprig planting; 38 rows of 35 sprigs + 8 rows of 37 sprigs + 1 row of 20 sprigs. Two rows (70 sprigs total) were planted with shingle wave breakers. Total planted: 1,646 sprigs on 2-foot centers (one per 4-square foot density).

(2) Fall 1978. Low-density semiprotected sprig planting; 11 rows of 48 sprigs + 2 rows of 36 sprigs + 5 rows of 27 sprigs + 1 row of 34 sprigs. One row (48 sprigs) planted with shingle wave breakers.

(3) Spring 1979. Low-density protected sprig planting (to complete area 1); 47 rows of 15 sprigs less 6 sprigs not planted due to concrete. Every other group of 5 rows received 1 ounce of Mag-Amp fertilizer; 20 rows (300 sprigs) were fertilized. Total planted: 699 sprigs.

(4) Spring 1979. Low-density semiprotected to exposed sprig planting; 30 rows of 50 sprigs. Total planted: 1,500 sprigs.

(5) Spring 1979. High-density protected sprig planting; 36 rows of 100 sprigs less 5 sprigs "overrun" in the 47th row of area 1. Two rows (200 sprigs) received 1 ounce of Mag-Amp fertilizer per sprig. Total planted: 3,595 sprigs on 1-foot centers (one per 1-square foot density).

(6) Spring 1979. High-density semiprotected sprig planting; 10 rows of 40 sprigs. One-half (5 successive rows or 200 sprigs) received 1 ounce Mag-Amp fertilizer per sprig. Total planted: 400 sprigs.

(7) Spring 1979. High-density exposed sprig planting; 25 rows of 100 sprigs from 88 to 113 feet south of the end of the breakwater. Two rows (200 sprigs) received 1 ounce of Mag-Amp fertilizer per sprig. Total planted: 2,500 sprigs.

(8) Spring 1979. Replanting denuded zone of area 1; 15 rows of 35 sprigs at high density (one per 1-square foot density) less 8 sprigs unplanted due to rocks. Total: 517 sprigs.

(9) Spring 1979. Replanting denuded zone of area 2; 20 rows of 20 sprigs at high density (one per 1-square foot density). Total planted: 400 sprigs.

(10) Spring 1979. Plug planting (plugs with at least 10 times the rhizome mass of sprigs). 100 plugs planted at approximately one plug per 1 square foot; protected area.

(11) Spring 1979. Plug planting. 100 plugs planted at approximately one plug per 1 square foot.

2 Construction. Areas 1 and 2 were planted between 6 and 8 December 1978. A total of 2,700 sprigs were planted at a density of one sprig per 1 square foot on 2-foot centers in both areas (Fig. 2-259). Sprigs were obtained by the San Francisco Bay Marine Research Center personnel from Red Rock Marsh near Alameda. Clumps of Pacific cordgrass were dug from the marsh and transferred to the Marine Research Center laboratory where the clumps were separated into sprigs. The sprigs were placed in plastic flats (about 130 to 170 sprigs per flat) and the rhizomes kept moist until planting. Sprigs were dug 3 to 5 days in advance of planting, transported to the site in the flats, and planted as holes were dug. Sprigs were placed in holes made with hand-held trowels and planted to a depth of 4 to 6 inches, depending on the substrate. In sandy areas where the soil was difficult to pack, plants were planted in holes at least 6 or more inches deep. After the sprigs were placed in the hole, sand or mud-sand was repacked in the holes and tamped down by hand or by walking around the base of each plant. At the time of planting each plant had at least two healthy upright culms (stems), with the average size being 2.8 inches. A small rhizomal mass with attached roots subtended each shoot. A total of 1,646 sprigs were planted in area 1 and 776 sprigs in area 2.

Shingle wave breakers were positioned in front of each of 35 plants in two rows in area 1, and in front of one row (48 plants) in area 2 to see if this would aid in plant establishment. None of the plantings at this date were fertilized because it was the dormant season. Planting was completed and the Inital Post Planting Inventory was made on 8 December 1978.

In April 1979 new high- and low-density plantings were made (one plant per 4 square feet, low density; one plant per 2 square feet, high density) in areas 3 to 11 to compare the effect of spacing. Slow-release fertilizer (Mag Amp) was used on some rows of plantings. Sprigs were again used for most plantings, but plugs were also used in areas 10 and 11 at high density only.

Plantings were done by personnel of the San Francisco Marine Research Center. Three to eight people were involved in the planting which extended from mid-April to early June 1979. Sprigs were again obtained from plants at Red Rock Marsh by the same procedure as that used in December, although in one case a delay of about 10 days followed the digging of the sprigs and planting. Plugs used in areas 10 and 11 (also obtained from Red Rock Marsh) were dug with a spade to make a 7-by 7-inch square block of rhizomal and shoot material, and then transferred intact to the site using plastic trays. Rhizomal masses were again kept damp during the delay between digging and planting. Each plug was at least 10 times larger than the sprigs used in areas 3 to 9.

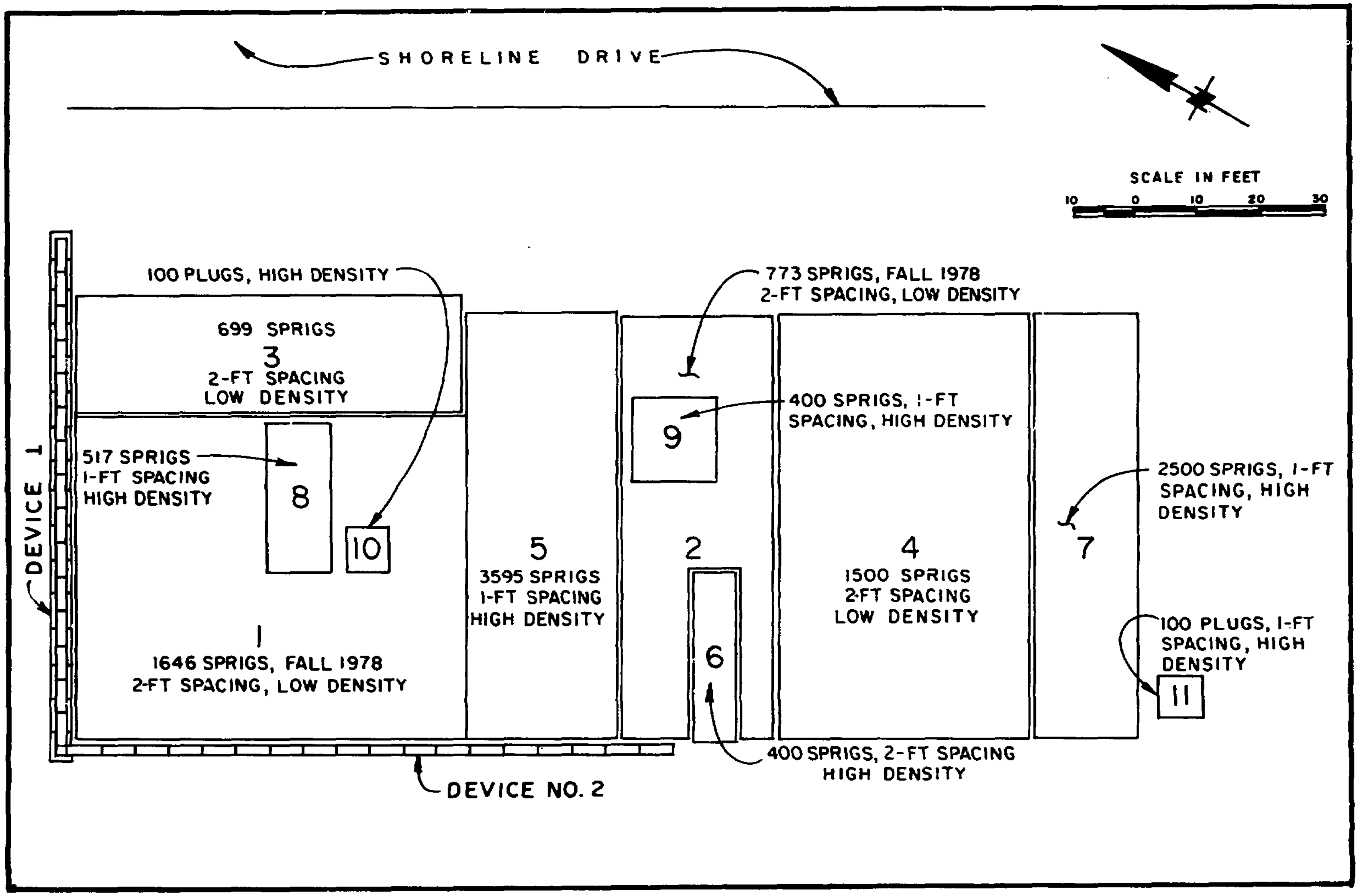

Figure 2-259. Device 3, planting arrangement of Pacific cordgrass at Alameda site.

The planting procedure was the same as in the December planting using trowel-formed holes and planting at depths of about 4 to 6 inches. In those rows to be fertilized, one ounce of Mag-Amp fertilizer (slow release) was placed in the bottom of the hole before putting the sprig in place so that the roots were in direct contact with the fertilizer. No fertilizer was applied to the plugs.

Planting of all areas and the Initial Post Planting Inventory for areas 3 to 11 were completed on 6 July 1979. Planting costs for device 3 are given in Table 2-50.

Table 2-50. Device 3 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					\$ 3.00
Sandfill	125	yd^3	\$0.80	\$ 100	
Pacific cord-grass	12,000	ea.	0.07	800	
Subtotal				900	
Labor	115	hr	8.70	1,000	3.33
Total				\$1,900	\$6.33

3 Performance. Areas 1 and 3 were protected low-density sprig plantings (one plant per 4 square feet) behind the breakwater. Shingle wave breakers were used on some rows in area 1. Some rows in area 3 were fertilized.

A number of plants were lost immediately after planting because of loose sand and the sprigs floated away during high tides. Survival of the sprigs in area 1, planted in December 1978, dropped steadily during the early spring months of 1979 (the storm of 27 March caused considerable damage to an end-of-season (October 1979) survival of about 36 percent of the original plantings). Plants increased in size over the summer months from an average of 2 stems per plant to an average of 28 stems per plant, showing vigorous growth. At the beginning of the next year (February 1980), almost 70 percent of those plants which were counted in October 1979 remained and showed evidence of active growth. The sand has become stabilized around the base of these remaining plants and the plants are beginning to spread out from their rhizomes.

The shingle wave breakers had an adverse effect on sprig establishment in the sandy soil of this area. Waves hitting the shingles were apparently deflected causing the water to swirl around behind them breaking off the sprigs at the base and, in many cases, digging up the plant entirely. This kind of device has been used successfully in muddy areas of the bay, but is evidently unsuitable for a sandy substrate.

Area 3, planted in May 1979, showed better survival of sprigs between initial spring planting and the end of the growing season, with an inventory survival of about 86 percent of the original planting. Stem growth was less in this area for unfertilized plants than for the same type of planting in area 1, with an end-of-season count of 10 stems per plant versus 28 stems per plant in area 1. The fertilized rows of area 3 showed the same amount of growth as in area 1, with 27 stems per plant by October 1979.

Survival of these plantings was reduced by winter and early spring storms but approximately 60 percent of those found at the end of the season were still present in February 1980. These plants showed renewed growth activity in March 1980, and appeared to be well established. Fertilized rows appeared to be doing slightly better in March 1980, but this was not conclusive, since reinitiation of growth was somewhat patchy in all areas.

Areas 5, 8, and 9 were planted from April to June 1979 as protected high-density sprig plantings (one plant per 2 square feet) behind the breakwater. Area 10 was also a high-density protected planting, but plugs were used. Some sprigs in area 5 were fertilized, but no plants in areas 8, 9, and 10 were fertilized. End-of-season survival was more than 90 percent for all of these areas, with only sprig plantings showing any losses. Winter and spring storms removed about half of the plants in areas 5, 8, and 9, but had only a small effect on the plugs in area 10. Growth during the summer months was less for unfertilized sprigs than recorded in area 1, for example, with only 7 to 13 stems per plant by the end of the season. Fertilized sprigs showed good growth with about 27 stems per plant by the end of the season. No counts were made of increased stem growth in plugs. Area 9 showed somewhat lower growth and slightly higher mortality than did the other areas, perhaps due to receiving only a small amount of protection from the end of the breakwater.

Areas 2 and 4 were low-density unprotected sprig plantings (1 plant per 4 square feet) outside the protection of the breakwater; neither area received any fertilizer. Survival of sprigs in area 2, planted in December 1978, was low; only 10 percent of the sprigs originally planted in this area remained by the end of the first growing season. Sixty percent of those which were counted in the end-of-season inventory persisted through the winter and were alive in February 1980. Stem growth was less than in area 1 (13 versus 28 stems per plant), the protected area also planted in December 1978.

Area 4 which was planted in May 1979 showed slightly more than 80-percent survival of sprigs by the end of the growing season. Survival was much less over the winter and spring with only half of the plants remaining in February 1980. Stem growth was not as good in this area by the end of the season as in the others, an average of nine stems per plant. In March 1980 most of the remaining plants were initiating new growth and appeared to be established.

Areas 6, 7, and 11 were high-density (1 plant per 2 square feet) unprotected plantings outside of the breakwater. All plantings were made in late spring 1979. Areas 6 and 7 were sprig plantings with some fertilized rows; area 11 was planted with fertilized plugs.

Survival of sprigs in areas 6 and 7 was greater than 80 percent by the end of the first growing season (October 1979). Survival over the winter months was much less with only 50 percent of the end-of-season plants remaining in area 6 and only 40 percent in area 7. Survival of the plugs in area 11 was 100 percent at the end of the growing season and 96 percent by the following February (1980) count.

Stem growth was moderate, as in the areas previously discussed with 10 to 13 stems per plant by October 1979. No counts of stem growth were made for the plug plantings in area 11, but considerable rhizomal growth appeared by October 1979.

(d) Device 4 (Nylon-Mat Revetment).

1 Statistics, Construction, and Costs. Device 4 statistics are given in Table 2-51, and details are shown in Figure 2-260.

Table 2-51. Device 4 statistics.

Length	50 ft
Top elevation	+10.0 ft
Toe elevation	+1.0 ft
Structure slope	1 on 0.6
Filter type	Mirafi-140 filter cloth

Construction began on device 4 in early November 1978 with the dumping of sand onto the existing embankment. Rough preparation of the embankment slope was accomplished with a bulldozer. The embankment fill was then dressed to a 1.00 on 0.6 slope using hand shovels and rakes. A 24-inch trench was dug into the sand with shovels at the toe of the revetment slope so that the nylon mat could be keyed in. A Mirafi-140 filter cloth was then unwound directly from the supply roll and stretched over the graded embankm . A Fabriform Filterpoint nylon mat was spread out on the filter cloth filled (using dry sand and an air compressor) with sand pumped through a 2-inch galvanized pipe. The pipe branched into a Y-shaped nozzle where it entered the filler hole in the mattress. Although the nozzle was baffled, care still had to be taken to prevent blowing a hole through the fabric. After the mat was filled, the 24-inch trench at the revetment toe was backfilled and packed down by hand. Installation was completed on 22 November 1978. Construction costs for device 4 are given in Table 2-52.

2 Performance. The 50-foot nylon-mat revetment was initially damaged during the storm of 26 and 27 March 1979. However, before t stor our pieces of 3/8-inch-diameter rebar shaped like an inve "u" w e driven through bags at the top of the embankment for stablization. This precaution apparently arrested the sliding caused by persons using the mat as a ramp (Fig. 2-261). The 28 March 1979 inspection revealed that the mat had been torn in several places, causing a loss of sand and exposing the underlying filter cloth (Fig. 2-262). Tears in the nylon fabric and filter cloth behind it allowed wave action during high tides to erode t ackfill being retained by the revetment. Flanking wave action at the west end soon undermined the mat in that area. Thereafter, the progressive failure of the revetment made it no longer functional (Figs. 2-263 and 2-264). Erosion continued until emergency fill and concrete rubble were placed on the bank in February 1980.

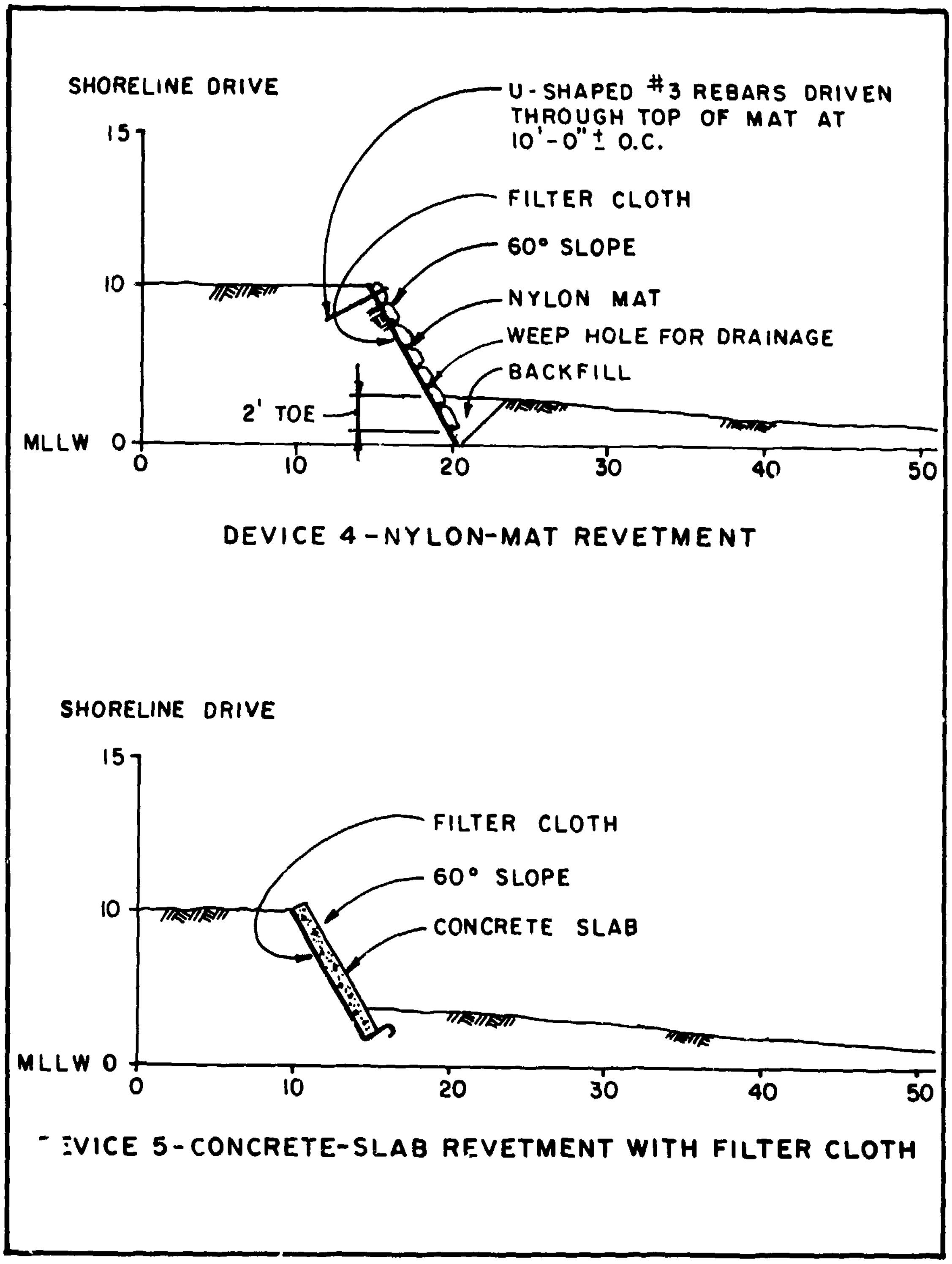

Figure 2-260. Devices 4 and 5 at Alameda site.

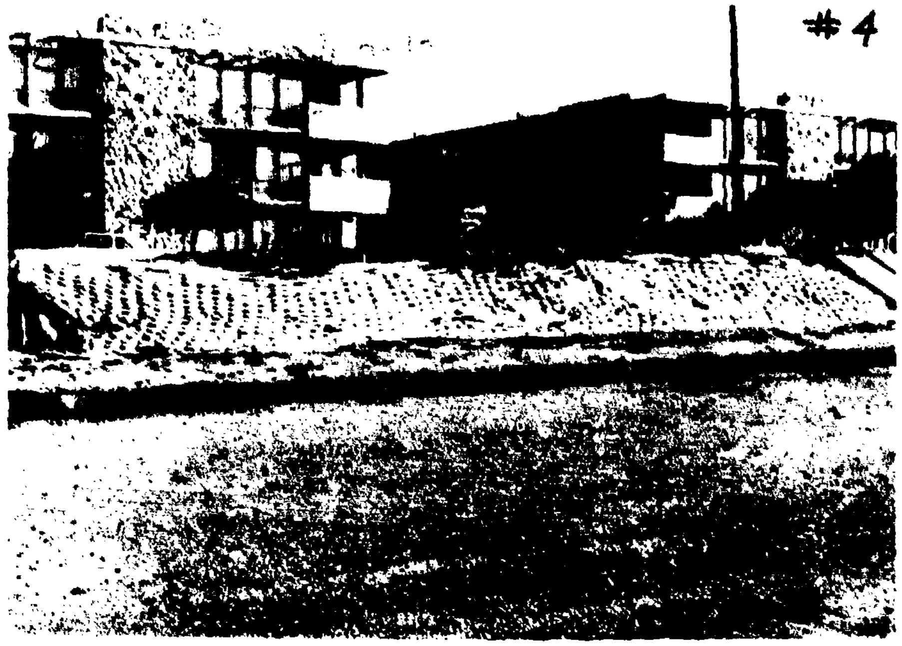

Figure 2-261. Nylon-mat revetment, device 4, Alameda, California, 27 November 1978.

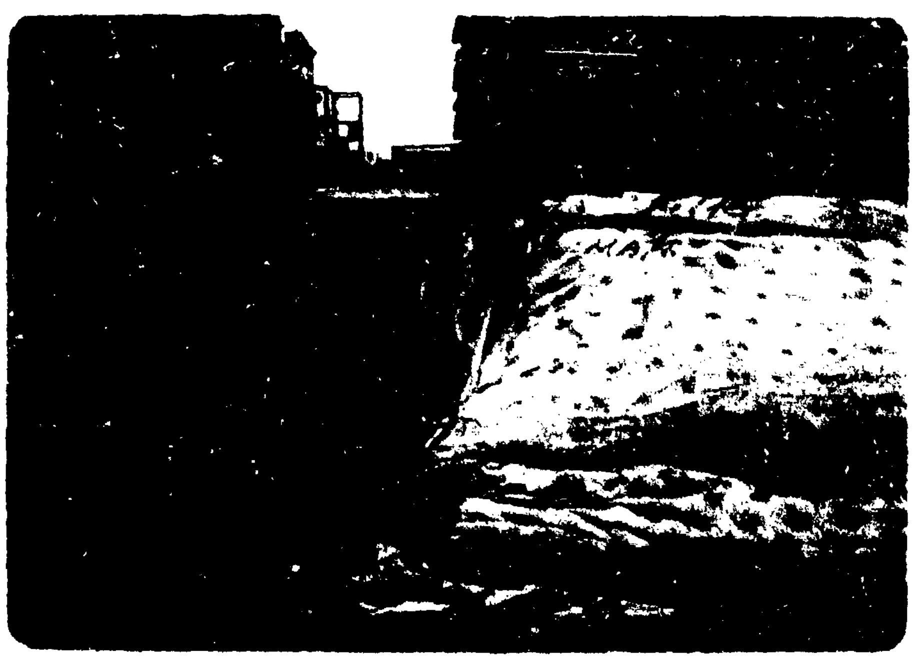

Figure 2-262. Nylon-mat revetment, device 4, Alameda, California, 28 March 1979.

Figure 2-263. Nylon-mat revetment, device 4, Alameda, California, 7 August 1979.

Figure 2-264. Nylon-mat revetment, device 4, Alameda, California, 21 August 1979.

Table 2-52. Device 4 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					$16.60
Sandfill	135	yd^3	$1.50	$200	
Fabriform nylon mat	1	ea.	529.00	530	
U-shaped No. 3 rebars	4	ea.			
Mirafi-140 filter cloth	100	yd^2	0.79	100	
Subtotal				830	
<u>Labor</u>	90	hr	12.22	1,100	22.00
<u>Equipment</u>					8.00
Backhoe	30	hr	13.33	400	
Total				$2,300	$47.00

(e) <u>Device 5 (Concrete-Slab Revetment with Filter Cloth)</u>

<u>1 Statistics, Construction, and Costs</u>. Device 5 statistics are given in Table 2-53, and details are shown in Figure 2-260.

Table 2-53. Device 5 statistics.

Length	50 ft
Top elevation	10.0 ft
Toe elevation	1.5 ft
Structure slope	1 on 0.6
Filter type	Mirafi-140 filter cloth

Sandfill was dumped on the existing embankment. After the rough slope preparation by the bulldozer, hand shovels and rakes were used to dress the embankment to a 1.00 on 0.6 slope. A hand-shoveled trench was dug at the toe of revetment to key in the concrete slabs. Mirafi-140 filter cloth was spread over the prepared embankment. The concrete slabs were salvaged from a building wall, trucked to the site, hoisted by a crane into position above the prepared bed, and guided into place. The placement was uniform; however, one slab was cracked during the unloading operations. Construction was completed on 15 November 1978, when the trench at the revetment toe was backfilled with sand. Construction costs for device 5 are given in Table 2-54.

<u>2 Performance</u>. The 50-foot revetment of concrete slabs over filter cloth suffered bank erosion, but not as much as device 4. Although most of the individual slabs were undamaged during the 26 and 27

Table 2-54. Device 5 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					$10.00
Sandfill	250	yd^3	$0.80	$200	
Concrete slabs	13	ea.	15.38	200	
Filter cloth	100	yd^2	0.79	100	
Subtotal				500	
<u>Labor</u>	136	hr	10.29	1,400	28.00
<u>Equipment</u>					2.00
Backhoe	7.5	hr	13.33	100	
Total				$2,000	$40.00

March 1979 storm, their uneven settlement caused the slab edges to tear the filter cloth and allow the retained sand to erode from behind the structure (Figs. 2-265 and 2-266). With each wave uprush at high tide, some of the fine to silty embankment sand became suspended in the water and drifted into the bay. A change in the cross section also occurred gradually through June 1979 (Fig. 2-267), then rapidly thereafter. After the embankment eroded at the downdrift end, some of the slabs fell flat on the beach. By August 1979, the slabs had been displaced to the extent that the revetment was no longer functionally effective (Fig. 2-268). In subsequent months the downdrift section of the embankment behind the slabs was severely eroded (Fig. 2-269). In January 1980, emergency fill was placed to prevent road damage (Fig. 2-270).

(f) <u>Device 6 (Rubble Revetment with Filter Cloth)</u>.

<u>1</u> <u>Statistics, Construction, and Costs</u>. Device 6 statistics are given in Table 2-55, and details are shown in Figure 2-271.

Table 2-55. Device 6 statistics.

Length	75 ft
Top elevation	+8.5 ft
Toe elevation	+4.0 ft
Structure slope	1 on 3
Filter type	Mirafi-140 filter cloth

The initial plan for device 6 called for a 60° embankment slope; however, the prepared slope was washed out by storm waves before the rubble could be placed. When construction operations resumed, the embankment was dressed to a much flatter slope of about 1 on 3. Along some segments, particularly the updrift end, the upper part of the slope was gently rounded. The Mirafi-140 filter cloth was then rolled over the embankment and secured

Figure 2-265. Concrete-slab revetment with filter cloth, device 5, Alameda, California, 28 March 1979.

Figure 2-266. Concrete-slab revetment with filter cloth device 5, Alameda, California, 28 March 1979.

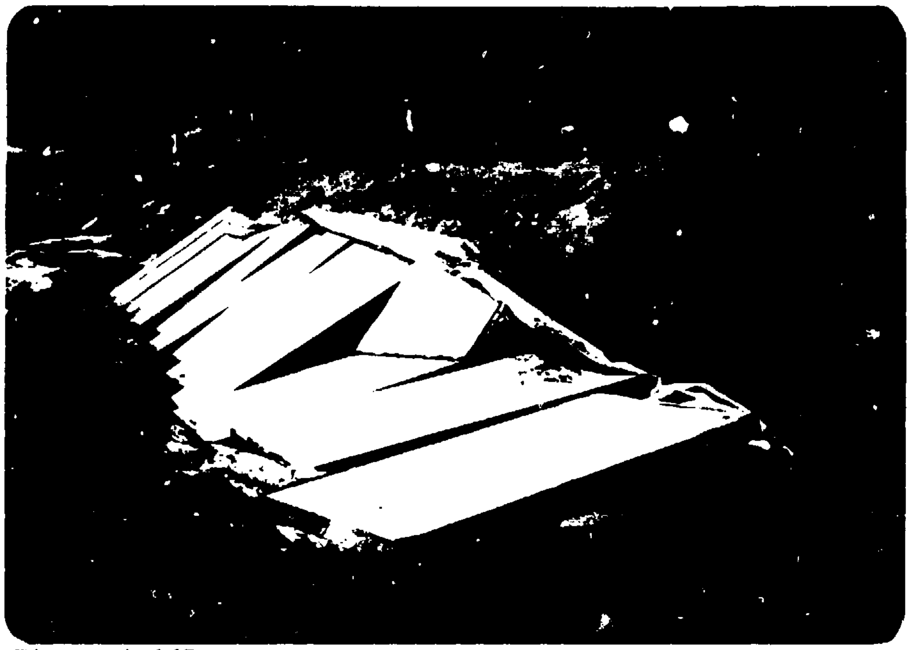

Figure 2-267. Concrete-slab revetment with filter cloth, device 5, Alameda, California, 21 June 1979.

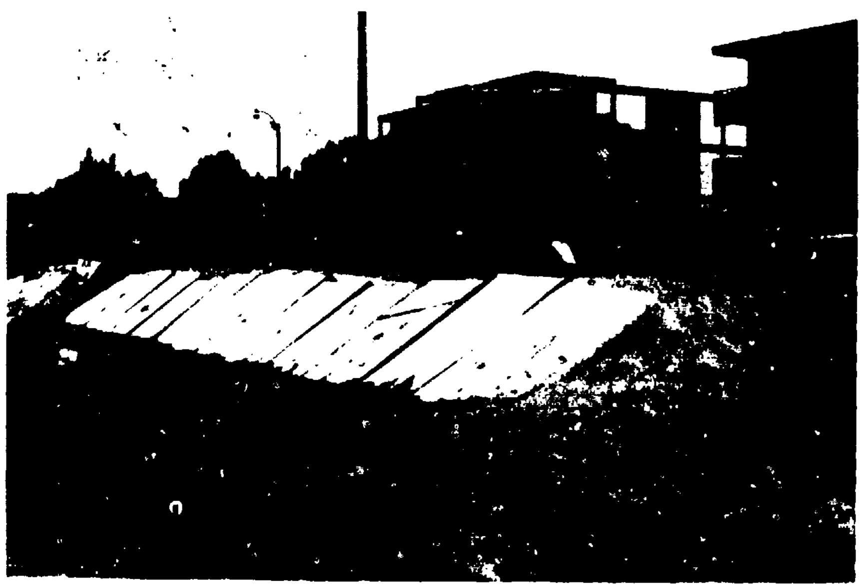

Figure 2-268. Concrete-slab revetment with filter cloth, device 5, Alameda, California, 21 August 1979.

Figure 2-269. Severe erosion on downdrift bank, device 5, Alameda, California, 8 January 1980.

Figure 2-270. Emergency fill placed on bank behind device 5, Alameda, California, 2 May 1980.

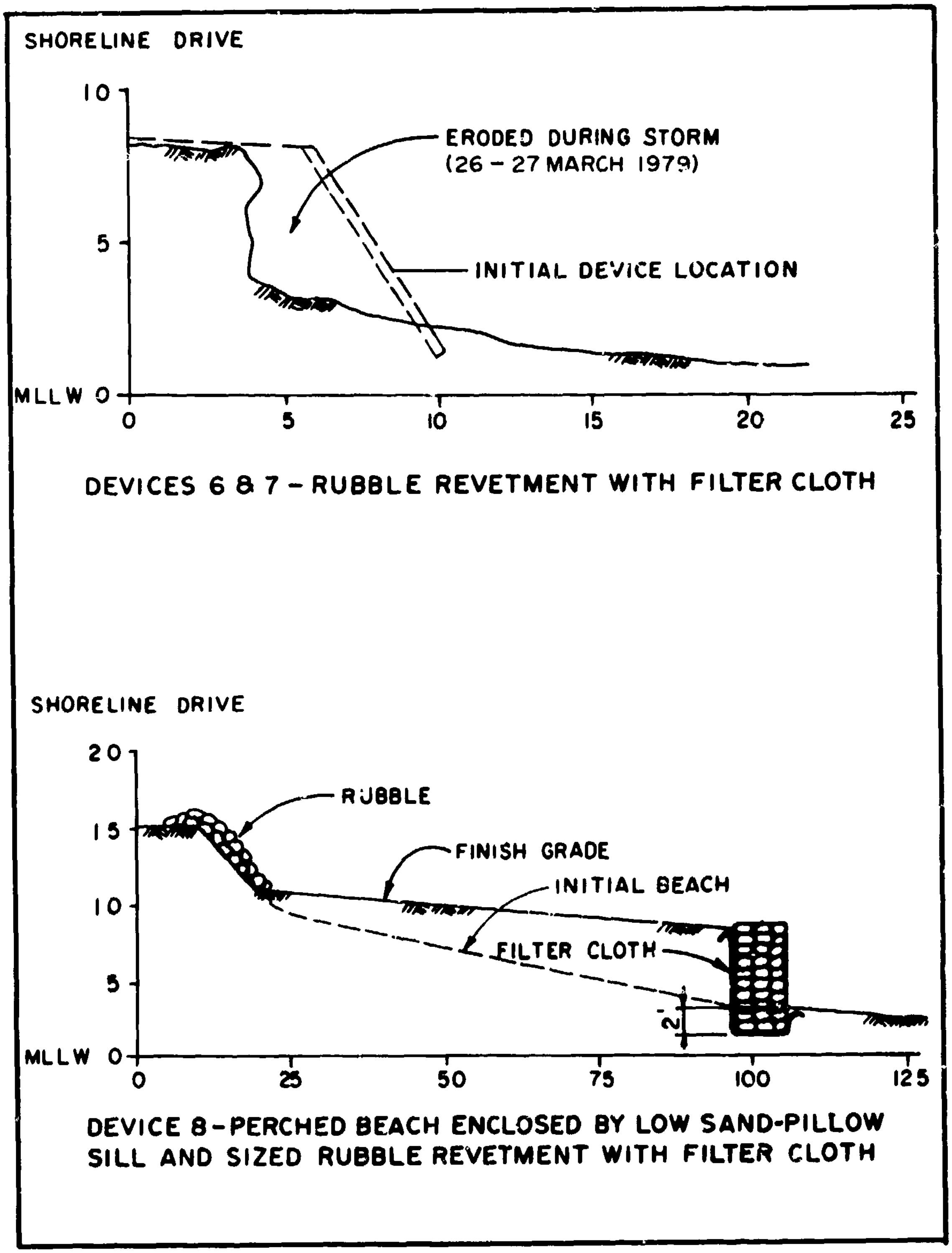

Figure 2-271. Devices 6, 7, and 8 at Alameda site.

in place by a 2-inch layer of sand. Flat slabs of broken concrete were placed on the slope, one layer deep, in a mosaic pattern. The revetment was completed on 23 April 1979. Construction costs for device 6 are given in Table 2-56.

Table 2-56. Device 6 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					$5.60
Sandfill	375	yd^3	$0.80	$300	
Rubble				N/C	
Mirafi-140 filter cloth	150	yd^2	0.79	<u>120</u>	
Subtotal				420	5.60
<u>Labor</u>	360	hr	9.72	3,500	46.67
<u>Equipment</u>					1.33
Backhoe	7.5	hr	13.33	100	
Total				$4,000	$53.36

<u>2</u> <u>Performance</u>. This 75-foot revetment, completed after the March 1979 storm, suffered only minor structural damage. Some of the sand placed over the filter cloth was washed away and some bank erosion occurred on the updrift side, but not enough to cause significant profile changes. Except for erosion of bluff material from behind the unprotected west end of the revetment, the structure adequately protected the bluff from wave erosion (Figs. 2-272 and 2-273) until February 1980 when the structure collapsed as a result of wave attack exposing the filter cloth and displacing most of the rubble (Fig. 2-274). As of May 1980 there was no severe erosion in the area.

(g) <u>Device 7 (Sized Rubble Revetment with Filter Cloth)</u>.

<u>1</u> <u>Statistics, Construction, and Costs</u>. Statistics are given in Table 2-57; a detailed drawing is shown in Figure 2-271.

Table 2-57. Device 7 statistics.

Length	75 ft
Top elevation	+8.5 ft
Toe elevation	+4.0 ft
Structure slope	1 on 3
Filter type	Mirafi-140 filter cloth

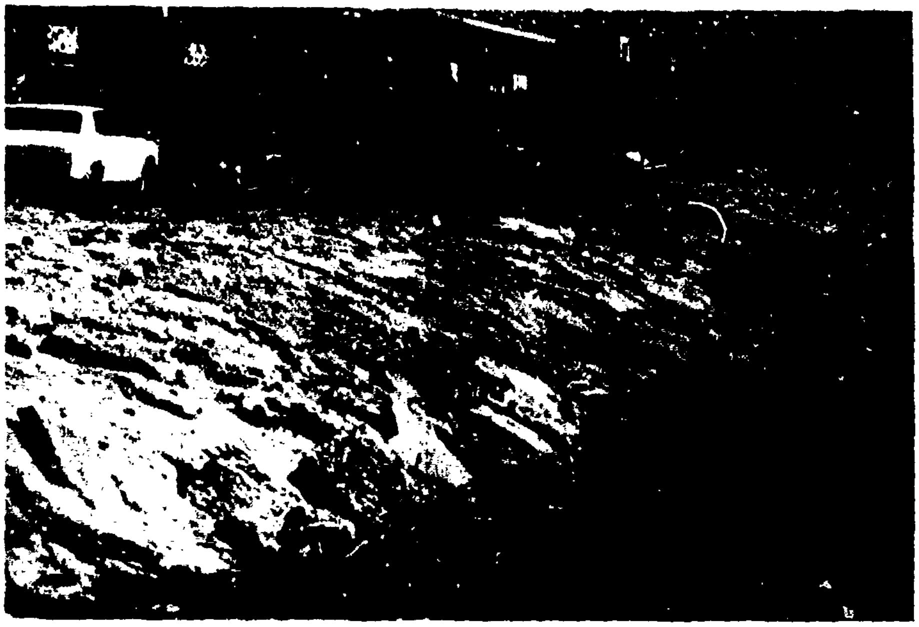

Figure 2-272. Rubble revetment with filter cloth, device 6, Alameda, California, October 1979.

Looking from West To East. #6

Figure 2-273. Rubble revetment with filter cloth, device 6, Alameda, California, 5 September 1979.

Figure 2-274. Structural damage to device 6, Alameda, California, 20 February 1980.

Device 7 lost its prepared embankment due to storm waves. Reconstruction ran concurrently with that of device 6. Sandfill was spread out over the 50-foot-long reach by a bulldozer; shovels and rakes were then used to finish the 1 on 3 slope. After the filter cloth was installed and covered with a layer of sand, placement of the concrete rubble began. Plan specifications called for limiting the aspect ratio of the rubble to 3:1; however, some of the rubble pieces exceeded this limitation and the resulting revetment was almost identical to that of device 6. The work was completed on 23 April 1979. Construction costs for device 7 are given in Table 2-58.

2 Performance. The 75-foot revetment, completed after the March 1979 storm, was undamaged with no significant loss of retained sand (Figs. 2-275 and 2-276). The buildup of sand in front of the revetment due to the groin action of device 8 prevented direct impact of waves on the structure, and wave uprush deposited a considerable amount of sand on its face, nearly blanketing the rubble. This additional sand buildup prevented device 7 from being damaged by the February 1980 storm waves (as at device 6 with no sand buildup) (Fig. 2-277). The profiles through device 7, compared with those through similar devices, are shown in Section III, Figure 3-43. The profiles of March 1979 and September 1979 show the accretion of sand in front of the structure. Some erosion was present at the junction of devices 7 and 8.

(h) Device 8 (Perched Beach Enclosed by Low Sand-Pillow Sill and Sized Rubble Revetment with Filter Cloth).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-59 and a detail drawing in Figure 2-271.

Table 2-58. Device 7 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					$5.60
Sandfill	375	yd^3	$0.80	$300	
Rubble					
Mirafi-140 filter cloth	150	yd^2	0.79	120	
Subtotal				420	
<u>Labor</u>	360	hr	9.72	3,500	46.67
<u>Equipment</u>					1.33
Backhoe	7.5	hr	13.33	100	
Total				$4,020	$53.60

Figure 2-275. Sized rubble revetment with filter cloth, device 7, Alameda, California, 21 June 1979.

Figure 2-276. Sized rubble revetment with filter cloth, device 7, Alameda, California, 21 August 1979.

Figure 2-277. Device 7 as of 20 February 1980, Alameda, California.

Table 2-59. Device 8 statistics.

	Beach	Sill	Revetment
Length	200 ft	400 ft	75 ft
Top elevation	+11.0 ft		+15 ft
Toe elevation	+8.5 ft		+11 ft
Structure height		4 ft	
Structure slope	0.03 (profile)	Vertical	1 on 2.5
Filter type		Mirafi-140 filter cloth	Mirafi-140 filter cloth

Work began on the low Sand-Pillow sill in December 1978 during ebbtides. Mirafi-140 filter cloth was placed on the existing bottom. After the 20- to 30-foot segment of filter cloth had been alined, bag placement began. The small, 100-pound Sand Pillows were filled by hand with a lean (8-to-1) sand-cement mixture, which was prepared on location by a generator-driven concrete mixer. Figure 2-278 shows the overall construction operations. To prevent the Sand Pillows in the shore-parallel segment from being displaced by waves, the landward area directly behind the sill was backfilled with sand (Fig. 2-279). In June 1979 the entire enclosure was filled and graded. Sand was transported from the beach near Park Street to the site, dumped on the existing embankment, and rough-graded with a bulldozer. The completed 200-foot shore-parallel segment of the sill was six pillows high and three abreast; the 100-foot returns were six pillows high and two abreast.

During the grading of the enclosed beach, the bulldozer also dressed the embankment behind the sill to a 1 on 3 slope. A Mirafi-140 filter cloth was spread out over the first 75 feet of embankment and was secured in position with a 2- to 3-inch layer of sand. Concrete rubble was then dumped on the embankment and uniformly arranged by hand. The work was completed on 21 June 1979. Construction costs for device 8 are given in Table 2-60.

4 Performance. The Sand-Pillow sill retaining the 100-by 200-foot beach fill remained essentially intact. Some of the jute material was torn, due either to vandalism or to floating objects striking against it, but in each unit the sand-cement fill held its shape. Some of the top Sand Pillows displaced by storm waves during installation were never replaced (Fig. 2-280), and others were randomly displaced later. The beach sand was generally contained within the perimeter of the sill, although receding waters worked holes through the pillows at the southeast corner, allowing some sand to escape (Fig. 2-281). Considerable sand was washed through tears in the filter cloth and from between the bags, forming a sand blanket over the mud bottom all along the bayside of the sill thick enough to support sunbathers at low tide (Figs. 2-282 and 2-283).

The rubble revetment with filter cloth behind the first 75 feet of perched beach maintained its original cross section with no significant structural degradation.

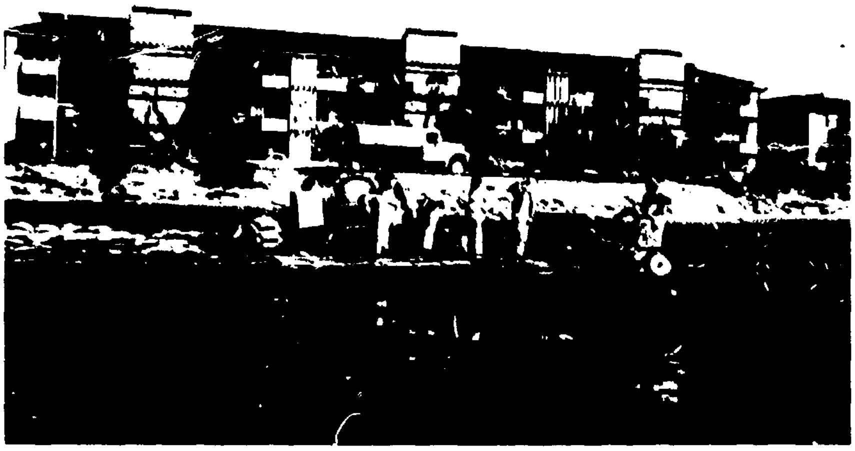

Figure 2-278. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, January 1979.

Figure 2-279. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, January 1979.

Table 2-60. Device 8 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$7.58
Sand	1,500	yd^3	$0.67	$1,000	
Cement	520	sacks	3.85	2,000	
Sand Pillows	150	ea.	0.67	100	
Rubble					
Mirafi-140 filter cloth	660	yd^2	0.79	500	
Subtotal				3,600	
Labor	1,620	hr	4.63	7,500	15.79
Equipment					15.79
Cat. D-4	12	hr	25.00	300	
Cat. D-7	20	hr	50.00	1,000	
Cat. 977	22	hr	50.00	1,100	
Water truck	40	hr	40.00	1,600	
Dump trucks					
Portable mixer	180	hr	11.11	2,000	
Case back-hoe	120	hr	12.50	1,500	
Subtotal				7,500	
Total				$18,600	$39.16

Device #8

Figure 2-280. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, 28 March 1979.

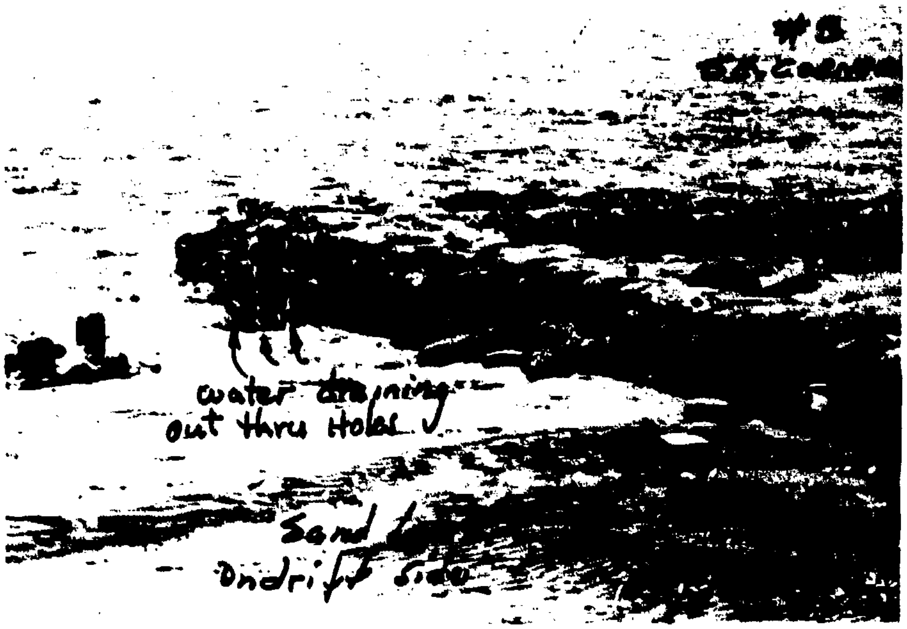

Figure 2-281. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, 5 October 1979.

#8

Figure 2-282. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, 7 September 1979.

Figure 2-283. Perched beach enclosed by low Sand-Pillow sill and sized rubble revetment with filter cloth, device 8, Alameda, California, 5 September 1979.

Despite the segments of lowered sill due to displacement of pillows, the sill was generally effective in retaining most of the imported fill. Some of the fill was moved about within the enclosure, probably as a result of unequal wave energy propagation due to variations in sill height.

The sill and perched beach prevented high waves from attacking the revetment directly. Although the embankment was not eroded, the revetment was not subjected to enough wave action to test it adequately. As of May 1980 the structure still retained the beach sufficiently despite some displacement and deterioration of the sandbags.

(i) Device 9 (Rubble Revetment with Filter Cloth, Perched Beach).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-61 with a detailed drawing in Figure 2-271.

Table 2-61. Device 9 statistics.

Length	75 ft
Top elevation	+15 ft
Toe elevation	+11 ft
Structure slope	1 on 2.5
Filter type	Mirafi-140 filter cloth

This 75-foot concrete rubble revetment fronted by the perched beach was constructed the same as the device 8 revetment. The work was completed on 21 June 1979. Cost data are given in Table 2-62.

Table 2-62. Device 9 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$8.00
Sandfill	600	yd^3	$0.67	$400	
Rubble				N/C	
Misc. supplies				100	
Mirafi-140 filter cloth	125	yd^2	0.79	100	
Subtotal				600	
Labor	380	hr	10.53	4,000	53.33
Equipment					9.33
Cat. 977	6	hr	50.00	300	
Dump truck				200	
Case backhoe	16	hr	21.50	200	
Subtotal				700	
Total				$5,300	$70.66

2 Performance. This 75 feet of rubble revetment just east of device 8 was also fronted by the perched beach. The revetment cross section remained as installed, with no significant structural degradation (Fig. 2-284) until February 1980 when the structure was significantly damaged by storm waves. Although some rubble remained on the bank, a large amount was carried completely off the bank (Fig. 2-285).

(j) Device 10 (Existing Rubble Revetment, Perched Beach).

1 Statistics, Construction, and Cost. Statistics are given in Table 2-63. A detail drawing is not available.

Table 2-63. Device 10 statistics.

Length	50 ft
Top elevation	+15 ft
Toe elevation	+11 ft
Structure slope	1 on 2.5

Figure 2-284. Rubble revetment with filter cloth and perched beach, device 9, Alameda, California, 7 September 1979.

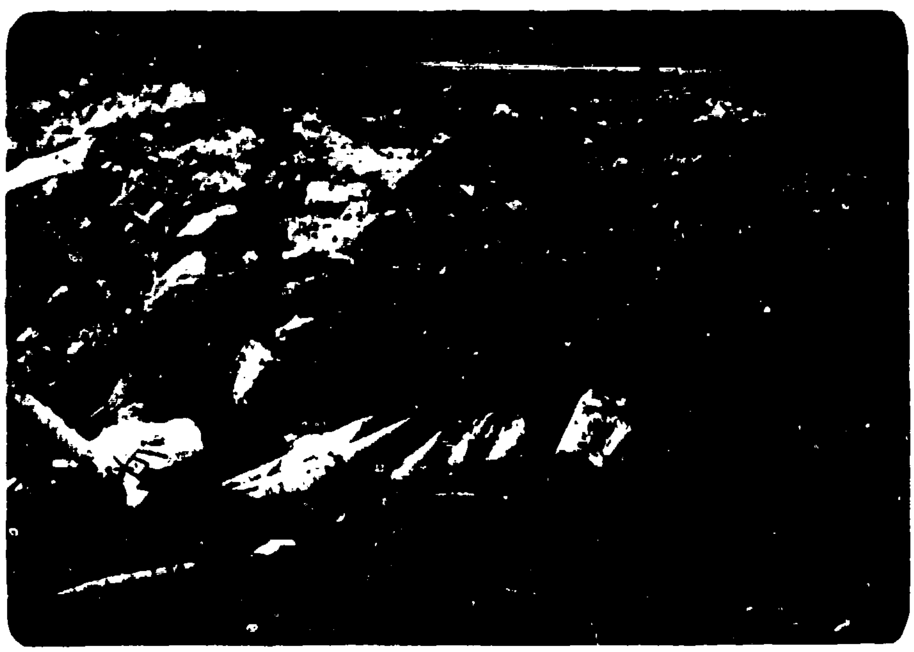

Figure 2-285. Damage to device 9 after February 1980 storm waves, Alameda, California, 20 February 1980.

The rubble for this 50-foot revetment consisted of broken chunks of concrete and asphalt dumped on the existing embankment slope (Fig. 2-286) without any slope grading. The rubble was randomly placed without a filter cloth. Construction was completed on 10 August 1979. Construction costs for device 10 are given in Table 2-64.

Table 2-64. Device 10 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$4.00
Sandfill	300	yd^3	$ .67	$200	
Rubble				N/C	
Subtotal				200	
Labor	240	hr	12.50	3,000	60.00
Equipment					10.00
Cat. 977	4	hr	50.00	200	
Dump truck				200	
Case backhoe	8	hr	12.50	100	
Subtotal				$500	
Total				$3,700	$74.00

2 Performance. This 50 feet of existing dumped riprap fronted by a perched beach was similar to device 9 except that no filter cloth was used. In February 1980, this structure was severely damaged by storm waves and most of the rubble was removed from the bank (Fig. 2-287).

(k) Device 11 (Acrylic Sand-Pillow Revetment, Dry Sandfill).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-65 (see Fig. 2-288 for details).

Table 2-65. Device 11 statistics.

Length	50 ft
Top elevation	+8.0 ft
Toe elevation	+0.0 ft
Structure slope	1 on 0.6

Figure 2-286. Existing rubble revetment and perched beach, device 10, Alameda, California, 5 December 1978.

Figure 2-287. Most of the rubble removed from device 10, Alameda, California, 11 April 1980.

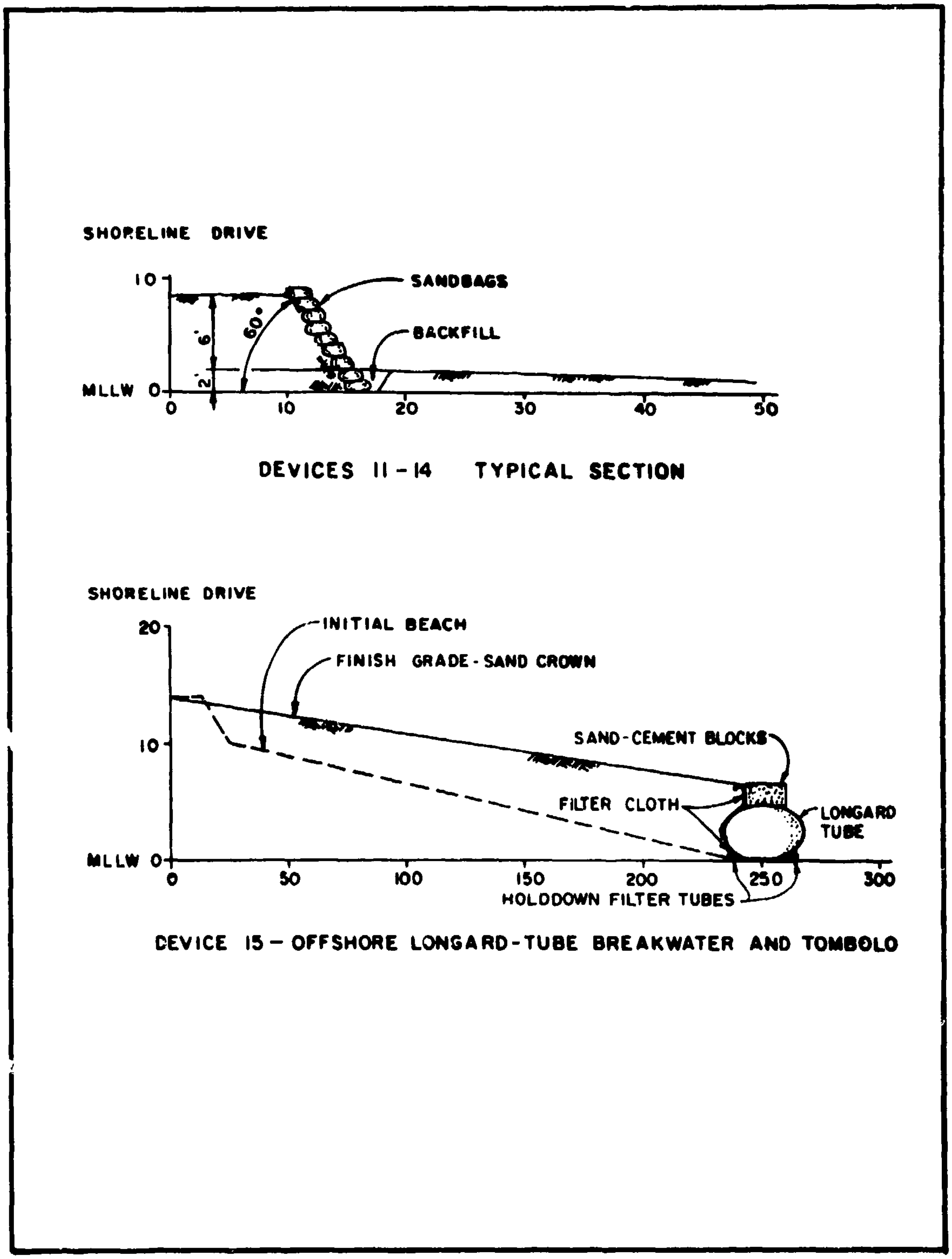

Figure 2-288. Devices 11 to 14 and 15 at Alameda site.

Rough spreading and grading of the embankment fill was accomplished by a bulldozer. Shovels and rakes were then used to dress the slope to 1.0 on 0.6. At the base of the revetment a trench about 2 feet deep was dug with shovels to MLLW so that the bottom rows of Sand Pillows could be keyed in. Pillow placement began at the revetment toe, where the Sand Pillows were filled with sand and sewn shut with garden twine having a 17-pound breaking strength. All work was done by hand. Weighing 100 pounds each when filled, the pillows were placed in section directly against the prepared embankment without any filter material. After the 2-foot toe was constructed, the trench was backfilled and packed down by hand. Work then continued until the revetment reached the crest of the berm. The revetment was completed on 14 December 1978. Construction costs are given in Table 2-66.

Table 2-66. Device 11 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$20.00
Sandfill	60	yd^3	$8.33	$500	
Sand Pillows	330	ea.	1.21	400	
Misc. supplies				100	
Subtotal				1,000	
Labor	580	hr	7.76	4,500	90.00
Equipment					20.00
Backhoe	80	hr	12.50	1,000	
Total				$6,500	$130.00

2 Performance. Devices 11 to 14, lacking the partial protection of a perched beach, were severely damaged. No filter cloth was used behind device 11, and as sand was pumped through voids in the revetment and scoured from under its toe, the Sand Pillows began to slump and slide down the slope (Fig. 2-289). An inspection after the March 1979 storm revealed that the Sand-Pillow fabric was intact but that sand was lost through the mouths of bags because the twine used in sewing the bags shut was broken or cut. During the next few months the cross section continued to change. The Sand-Pillow fabric deteriorated, more bag sand was lost, and by June 1979 the entire structure was washed out (Figs. 2-290 and 2-291). The city of Alameda then placed rubble in the damaged area as an emergency measure to stop further erosion (Fig. 2-292).

Although this revetment was esthetically pleasing and appeared to function well before the March 1979 storm, its structural failure resulted in the complete loss of functional effectiveness.

Figure 2-289. Acrylic Sand-Pillow revetment, dry sandfill, device 11, Alameda, California, 28 March 1979.

Figure 2-290. Acrylic Sand-Pillow revetment, dry sandfill, device 11, Alameda, California, 21 June 1979.

Figure 2-291. Acrylic Sand-Pillow revetment, dry sandfill device 11, Alameda, California, 9 August 1979.

Figure 2-292. Acrylic Sand-Pillow revetment, dry sandfill, device 11, Alameda, California, 7 September 1979.

(1) Device 12 (Acrylic Sand-Pillow Revetment, Sand-Cement Fill).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-67 (see Fig. 2-288 for details).

Table 2-67. Device 12 statistics.

Length	50 ft
Top elevation	+8.0 ft
Toe elevation	+0.0 ft
Structure slope	1 on 0.6

Construction of this revetment was identical to that of device 11 except that a lean (8-to-1) sand-cement mixture was used to fill the Sand Pillows. The revetment was completed on 13 December 1978. Construction costs for device 12 are given in Table 2-68.

Table 2-68. Device 12 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					\$44.00
Sandfill	50	yd^3	\$8.00	\$ 400	
Cement fill	315	sacks	4.13	1,300	
Sand Pillows	330	ea.	1.21	400	
Misc. supplies				100	
Subtotal				2,200	
Labor	770	hr	7.79	6,000	120.00
Equipment					34.00
Portable mixer	120	hr	5.83	700	
Case backhoe	80	hr	12.50	1,000	
Subtotal				1,700	
Total				\$9,900	\$198.00

2 Performance. An inspection of the revetment after the March 1979 storm revealed that about 50 percent of the sand-cement pillows remained (Fig. 2-293). When the embankment gave way, some of the pillows cracked, hastening loss of retained material. Except for this cracking, damage to the revetment units was prevented by the hardened cement in the fill mixture which allowed the pillows to maintain their shape after the fabric had deteriorated (Figs. 2-294 and 2-295).

Although the individual pillows held their shape, displacement of the pillows under wave attack made the entire structure ineffective as a revetment. The mound of units that was ultimately left along the toe of the bluff provided some protection, but not enough to prevent continued bluff recession. In February 1980 the bluff erosion was severe enough to require the placement of emergency rubble behind the pillows (Fig. 2-296).

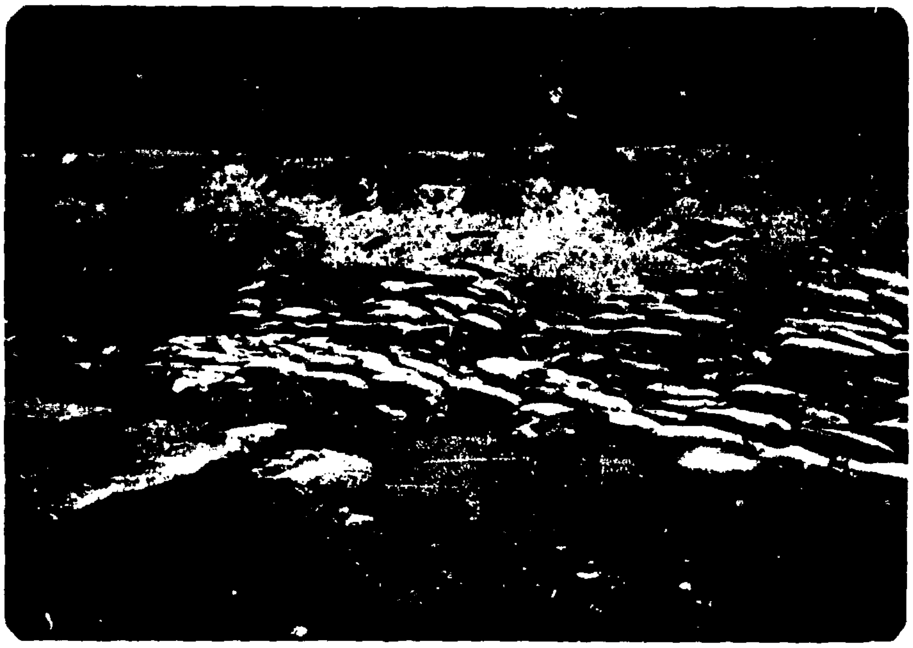

Figure 2-293. Acrylic Sand-Pillow revetment, sand-cement fill, device 12, Alameda, California, 7 August 1979.

Figure 2-294. Acrylic Sand-Pillow revetment, sand-cement fill, device 12, Alameda, California, 7 September 1979.

Figure 2-295. Acrylic Sand-Pillow revetment, sand-cement fill, device 12, Alameda, California, 9 August 1979.

Figure 2-296. Emergency rubble placed behind device 12, Alameda, California, 20 February 1980.

(m) Device 13 (Burlap-Sandbag Revetment, Sand-Cement Fill).

1 Statistics, Construction, and Cost. Statistics are given in Table 2-69, and a detail drawing is given in Figure 2-288.

Table 2-69. Device 13 statistics.

Length	50 ft
Top elevation	+8.0 ft
Toe elevation	+0.0 ft
Structure slope	1 on 0.6

Device 13 was installed to the same specifications as those of the updrift revetment (device 12), except that burlap bags were used instead of acrylic bags. Device 13 was completed on 14 December 1978. Construction costs for device 13 are given in Table 2-70.

Table 2-70. Device 13 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$44.00
Sandfill	50	yd^3		$ 400	
Cement fill	315	sacks		1,300	
Burlap sandbags	330	ea.		400	
Misc. supplies				100	
Subtotal				2,220	
Labor	720	hr	$7.64	5,500	110.00
Equipment					34.00
Portable mixer	120	hr	5.83	700	
Case backhoe	80	hr	12.50	1,000	
Subtotal				1,700	
Total				$9,400	$188.00

2 Performance. The revetment units used in this device were burlap bags filled with sand-cement. As with the other sandbag

revetments, no filter cloth was used. During the March 1979 storm the embankment fill failed, the entire revetment slumped, (Fig. 2-297), and the structure became more susceptible to wave damage. The width of the berm behind the revetment was reduced by as much as 18 feet in severely damaged areas (Fig. 2-298). About 50 percent of the sand-cement bags remained in a stable configuration at the bottom of the slope. The deterioration of the burlap fabric was comparable to that of the Sand-Pillow fabric, but the sand-cement mixture was hard enough to hold its shape even after the bag was completely stripped away (Fig. 2-299).

The performance of this revetment closely paralleled that of device 12--the mound of units that had slumped to the toe of the bluff partially protected the bluff itself. Profiles through device 13 are shown in Figure 3-45, Section III. About 1 to 2 feet of erosion is evident in the profiles of December 1978 and September 1979. Emergency rubble was placed behind device 13 in February 1980.

(n) Device 14 (Burlap-Sandbag Revetment, Dry Sandfill).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-71 (see Fig. 2-288 for details).

Table 2-71. Device 14 statistics.

Length	50 ft
Top elevation	+8.0 ft
Toe elevation	+0.0 ft
Structure slope	1 on 0.6

The burlap sandbags of this revetment were filled with dry sand; otherwise, construction was identical to that of device 13. The work was completed on 29 November 1978. Construction costs for device 14 are given in Table 2-72.

Table 2-72. Device 14 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$20.00
Sandfill	60	yd^3	$8.33	$500	
Burlap sandbags	330	ea.	1.21	400	
Misc. supplies				[illegible]	
Subtotal				1,[illegible]00	
Labor	660	hr	6.82	4,500	90.00
Equipment					20.00
Backhoe	80	hr	12.50	1,000	
Total				$6,500	$130.00

Figure 2-297. Burlap-sandbag revetment, sand-cement fill, device 13, Alameda, California, 28 March 1979.

Figure 2-298. Burlap-sandbag revetment, sand-cement fill, device 13, Alameda, California, 9 August 1979.

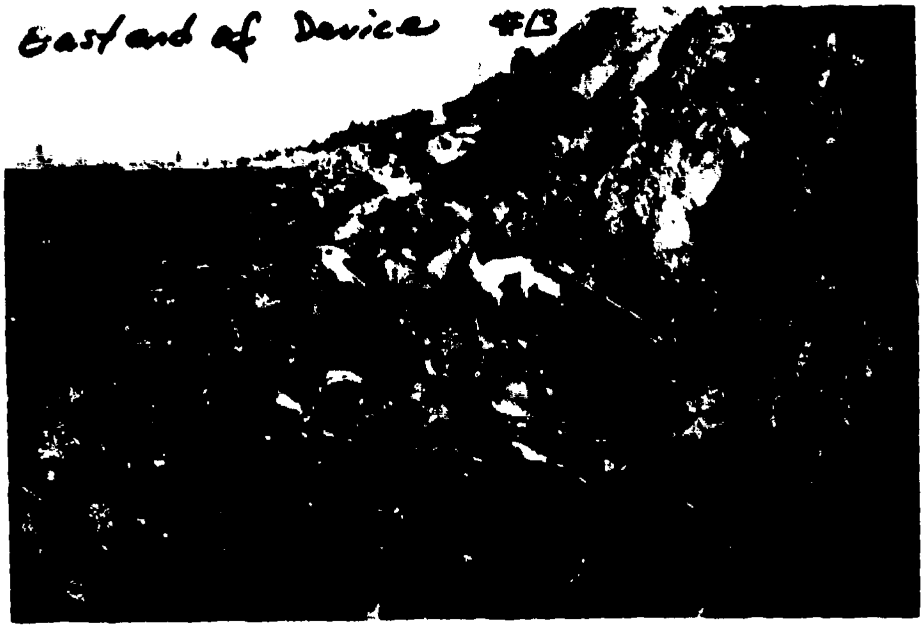

Figure 2-299. Burlap-sandbag revetment, sand-cement fill device 13, Alameda, California, 7 September 1979.

2 Performance. During the March 1979 storm, device 14 suffered less damage than any of the other exposed sandbag revetments, and its cross section remained stable for about 3 additional months. As Figure 2-300 shows, the eastern end of the revetment was still unharmed, but by July 1979 the burlap bags were degrading in strength, ripping open, and losing their sandfill (Fig. 2-301). After the bag material failed, the erosion rate increased rapidly and the revetment was completely washed out in early October 1979 (Fig. 2-302).

The performance of this revetment closely paralled that of device 11, its structural failure resulting in complete loss of functional effectiveness. By 8 January 1980, no evidence of device 14 remained at the site.

(o) Device 15 (Offshore Longard-Tube Breakwater and Tombolo).

1 Statistics, Construction, and Costs. Statistics are given in Table 2-73; a detailed drawing is shown in Figure 2-288.

Table 2-73. Device 15 statistics.

	Tombolo	Breakwater
Top elevation	+14.0 ft	+7.0 ft
Toe elevation	+7.0 ft	+0.0 ft
Structure slope	0.03 (profile)	
Length		330 ft

Figure 2-300. Burlap-sandbag revetment dry sandfill, device 14, Alameda, California, 21 June 1979.

Figure 2-301. Burlap-sandbag revetment, dry sandfill, device 14, Alameda, California, 18 July 1979.

Figure 2-302. Burlap-sandbag revetment, dry sandfill, device 14, Alameda, California, 5 October 1979.

A 69-inch diameter Longard tube (on a strip of filter cloth with 10-inch holddown tubes attached to each side) was used as an offshore breakwater at the head of the tombolo. The Longard-tube system is described in the Appendix. A loader was used to carry the roll of filter cloth; workers spread out the cloth as it was unrolled along the axis of the breakwater (Fig. 2-303). The 10-inch-diameter Longard holddown tubes were hydraulically filled with sand. The 69-inch tube was then unrolled onto the filter cloth and filled with sand in the same manner. The sand-pumping mechanism is a patented device built especially for filling Longard tubes (Fig. 2-304). The sandfill for the tombolo, brought to the site in dump trucks from Harbor Bay Isle, was graded to shape by a bulldozer (Fig. 2-305). In mid-December 1978, it became necessary to increase the height of the breakwater by adding 8- by 3- by 2-foot sand-cement blocks. The blocks were nylon bags filled with the lean sand-cement mixture and formed in the same manner as that used for device 2. To seal the junctions between adjacent sand-cement blocks, additional filter cloth was placed directly behind the breakwater (Fig. 2-306). More sandfill was dumped in the outer part of the tombolo and bulldozed against the breakwater to hold the filter cloth in place. The breakwater was completed on 16 October 1978. Construction costs for device 15 are given in Table 2-74.

2 Performance. The breakwater successfully resisted damage by natural forces but was continually being damaged by vandals. Wave and tidal action then began to destroy vandalized segments of the Longard tube. Before the March 1979 storm, a vandalized section of the tube was

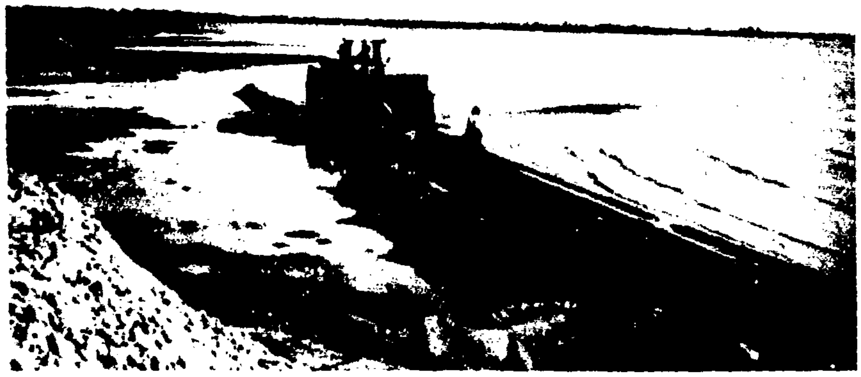

Figure 2-303. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, 27 September 1978.

Figure 2-304. Hydraulic filling of Longard tube, Alameda, California, 2 October 1978.

Figure 2-305. Offshore Longard-tube breakwater and tom' lo, device 15, Alameda, California, October 1978.

Figure 2-306. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, October 1978.

Table 2-74. Device 15 costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					\$143.00
Sandfill	9,500	yd^3	\$1.19	\$11,300	
Cement fill	490	yd^3	4.08	2,000	
Longard tube	1	ea.	33,000	33,000	
Miscellaneous				200	
Nylon bags	22	ea.	31.00	700	
Subtotal				47,200	
<u>Labor</u>	2,300	hr	8.04	18,500	56.06
<u>Equipment</u>					56.67
Loader	120	hr		4,900	
Cat. D-4		hr		300	
Cat. D-6	48	hr		2,400	
Cat. D-7	130	hr	35.38	4,600	
Concrete-mixer truck	80	hr	12.50	1,000	
Portable mixer				300	
John Deere 750 dozer	90	hr	25.26	2,300	
John Deere 450 dozer	16	hr	18.75	300	
Case backhoe	120	hr	12.50	1,500	
Pumps	32	hr	34.38	1,100	
Subtotal				18,700	
Total				\$84,400	\$256.00

plugged with additional sand-cement blocks (Fig. 2-307). However, the storm waves washed out this weakened section (Fig. 2-308), and many of the sand-cement blocks were displaced to precarious positions atop the tube, causing a safety hazard. A week later, all of the blocks were pushed to the bayward side of the tube (Figs. 2-309 and 2-310). The exposed Longard tube was then progressively vandalized, until by midsummer it could not retain the beach material, and the tombolo in its lee began to erode. The tube was so badly damaged and esthetically displeasing that it was removed in early August 1979. The sand-cement blocks were then realined and pushed together as closely as possible, forming a breakwater about 4 feet lower than the original tube and block device (Fig. 2-311).

During its full-height period, the breakwater effectively held the artificially created tombolo in its lee, which in turn acted as a groin, trapping a fillet of sand that extended a few hundred feet to the west. When the breakwater was reduced to a row of blocks in September 1979, overtopping waves scoured the sand in its immediate lee as littoral currents carried the temporarily suspended material in the scour trough eastward. By November, a shallow 100-foot channel had formed behind the structure, but the remaining

Figure 2-307. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, 28 March 1979.

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28 Mar. 79

Figure 2-308. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, 28 March 1979.

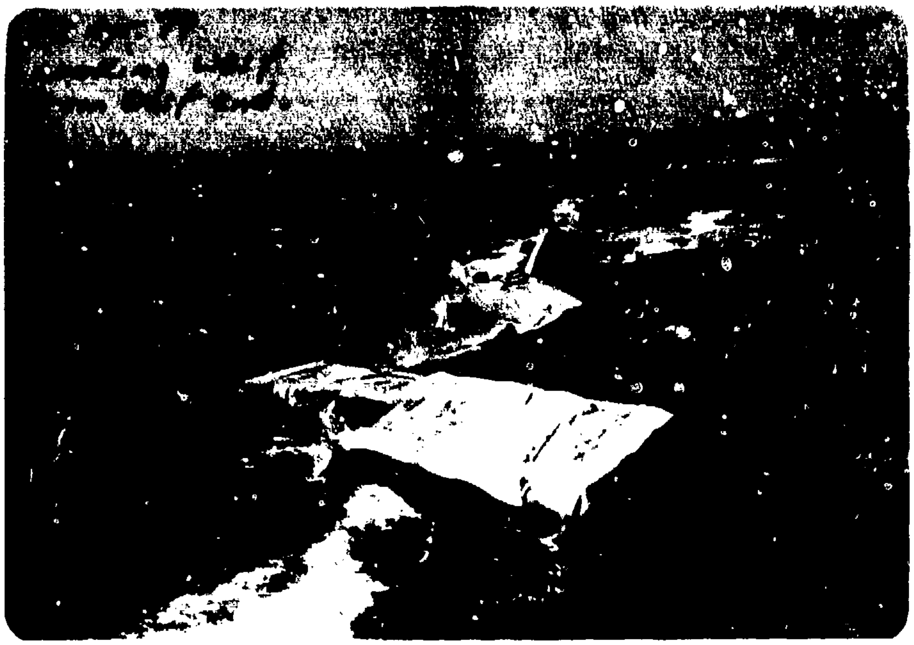

Figure 2-309. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, 2 April 1979.

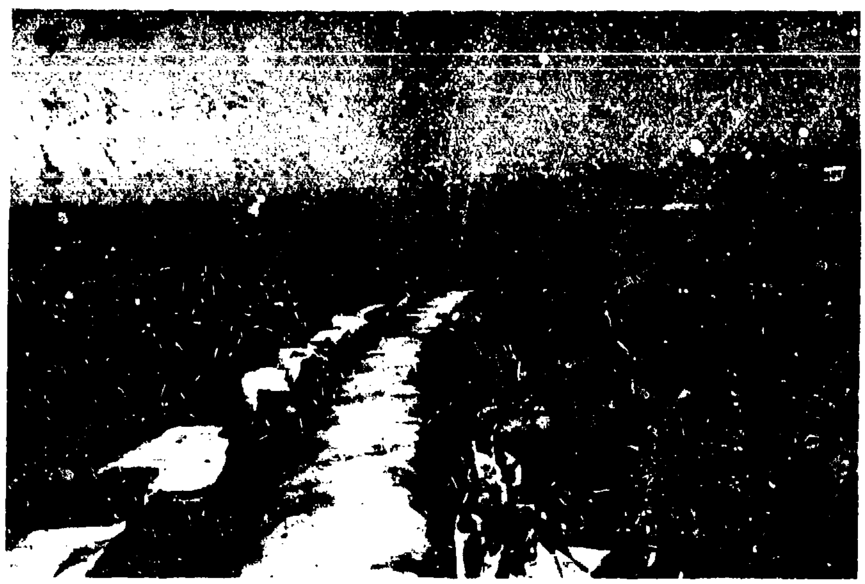

Figure 2-310. Offshore Longard-tube breakwater and tombolo, device 15, Alameda, California, 2 April 1979.

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Figure 2-311. Breakwater of realined blocks fronting the tombolo, Alameda, California, 14 November 1979.

bulge of sand berm acted as a groin, holding the advanced alinement of the beach fill for a distance of about 300 feet east of Willow Street (Fig. 2-312). By May 1980, there was continuing yet slow erosion behind the blocks; however, the individual blocks maintained their position. Profiles through the Longard tube are shown in Figure 3-75, Section III. The profiles show the accretion and erosion trends which have occurred at this location.

(3) Analysis.

(a) Groins and Beach Fill. Three of the devices evaluated functioned as groins--the sand-cement bag structure of device 1, the Sand-Pillow sill of device 8, and the initial tombolo (and later the shoreline bulge) retained by the device 15 breakwater. Each of these structures demonstrated that for the Alameda site, a functionally effective groin would retain a fillet of sand extending about three times its length along the updrift shore. At devices 1 and 15, the initial fills were adequate to form these fillets; at device 8, the lack of initial fill west of the structure was offset by existing sand in the updrift littoral compartment (device 2 to device 8) and by transport (leakage) of sand past device 2. Sand accumulating along the west leg of the device 8 sill overtopped that structure in the fall of 1979 and began to nourish the beach behind the frontal sill (Fig. 2-313). The monitoring did not last long enough to accurately determine the loss rates past the groin structures or the rate at which renourishment would be required. It appeared that renourishment every 5 to 10 years would suffice. The Sand-Pillow sills appeared to be structurally adequate for groin construction at the site.

Figure 2-312. Retreat of tombolo behind device 15 breakwater, Alameda, California, 13 November 1979.

Figure 2-313. Sand accumulating behind device 8 sill, Alameda, California, 13 November 1979.

The protection afforded by the retained sand fillet varied with its width. A beach berm of at least 15 feet appeared to be necessary to prevent major erosion of the bluff behind it. The berm height established by the wave and tide regimen of the area was not adequate to prevent some bluff erosion at extreme high tides under storm conditions, and some type of low-cost revetment appeared necessary, even with the sand berm protection.

The functional effectiveness of the groins was due to (a) the preponderance of waves with a potential for littoral transport in the downdrift direction, (b) the limited fetch at the site, which held ximum wave heights in deeper water to about 5 feet, and (c) the flatness and shallowness of the nearshore bottom, which caused the larger waves to break and limited wave heights near the shoreline to a maximum of about 3 feet.

(b) Breakwaters and Sills. These structures proved to be adequate to retain sandfills or to prevent wave destruction of intertidal vegetation planted in their lee when maintained at a crest elevation of about MSL or higher. Uniformity of crest elevation appeared necessary to prevent the formation of swash channels and displacement of littoral materials in the lee area. The Longard tube functioned well until its destruction by vandals. The vulnerability to this type of damage makes Longard tubes unsuitable for use in areas where vandalism is prevalent and cannot be controlled. A sand-epoxy coating applied after the tube was filled might have deterred vandalism, but in this area it is doubtful that any coating would have prevented the ultimate destruction of the tube. The sand-cement bag construction proved to be structurally adequate, but improvement is needed to maintain crest-elevation uniformity. Further experimentation is needed to find a satisfactory solution to the settlement problem.

Placement of the formed, sand-cement blocks atop the Longard tube to increase the height of the device 15 breakwater proved to be an ineffective expedient. Where Longard tubes can be used in areas safe from vandalism and a single tube does not provide enough height, the three-tube system recommended by the manufacturer should be used. In that system, the two bottom tubes are attached at their sides so as not to spread apart under the load of the top tube.

(c) Revetments. Several lessons were learned by monitoring the revetments tested at the Alameda site. Perhaps the most important was the need for a slope much flatter than the specified 60° angle. The steepness of the structures aggravated the toe scour problem, allowing the revetment materials to slide down the embankment into the scour trench and thereby disrupting their continuity. At points of disruption, erosion of materials from the bank then caused unsupported zones which, under continuing wave attack, resulted in complete structural failure. Even if the toes of the structures had been embedded sufficiently to prevent sliding, the steepness of the slope would have caused instability of the bluff material when it became saturated, and slumping would result, since none of the revetments had the weight or strength to resist the earth pressure. A slope of 1 on 1.5 is about the maximum allowable for granular materials, and flatter slopes are required where cohesionless silts and clays are present.

A revetment of stones, concrete debris, or modules of any type can not survive if it has voids or cracks through which wave action can wash out the retained material. Dumped riprap usually fails for this reason, and a revetment must be designed with a filtering device that is adequate to prevent the fines in the bank on which it is placed from being washed through the structure. Device 5, a revetment of concrete slabs on filter cloth, failed largely because nonuniform displacement of the slabs tore the cloth, and this allowed the retained material to be pumped out from behind the structure. If the slabs had been set on a flatter slope with their toes deeply embedded, they might have remained stable and the filter cloth probably would have prevented loss of material through the cracks between adjacent slabs. Adequate flank protection would, of course, have been necessary. Devices 6 and 7, built on sandfill after the March 1979 storm had washed out the original steep bank, were constructed on a much flatter slope, and because the filter cloth was not disrupted they retained the fill and were functionally effective. The beach fillet formed by the groin action of device 8 prevented higher waves from reaching these two revetments until February 1980. In February 1980, device 6 failed and device 7 was damaged by the same processes that caused device 5 to fail.

The filter cloth used at Alameda was a nonwoven type consisting of a compressed and heat-bonded mat of entangled plastic fibers. This type of cloth has no distinct openings and is easily clogged by silts and clays, leading to saturation of retained soils. Also, it is readily torn by large rubble not carefully placed, and the cloth when stretched by displacement of modules in a heavy revetment, may tear and open up large holes. Use of the cloth is recommended only for horizontal placement under a layer of bedding stone that will protect it from direct contact with heavy superimposed modules.

The sand-cement and sand-filled pillows and bags of devices 11 to 14 failed primarily as a result of excessive slope steepness and consequent embankment slumping. Although no filter was used, the bags were knitted so tightly that no voids were left in the structure, and filter cloth would have served no useful purpose. Only dry sand was used to fill the pillows in device 11 and the bags in device 14, and initial damage was due to loss of sand from bag openings caused by failure of either the closure twine or the bag material. Even if these revetments had been placed on a flatter slope, they would soon have failed through degradation of bag material or closure twine. The lesson learned was that the bags or pillows must be filled with sand-cement so that they will hold their shape regardless of what happens to the fabric.

The nylon-mat revetment of device 4 is not recommended by the manufacturer for long-term use with sandfill alone. Although slope steepness and tearing due to foot traffic were the main cause of failure, the structure might have survived and remained functionally effective had it been placed on a flatter slope and filled with concrete grout. This type of construction has been used elsewhere with success in low wave environments. Its primary advantages are rapidity of installation and conformance to subgrade irregularities, requiring a minimum of fine grading before mat emplacement.

(d) Vegetation. Planting dates seem to significantly affect survival of Pacific cordgrass sprigs. Losses of 70 percent were encountered in those areas planted in December when the plants were dormant; only about 35 percent of the plants were lost from spring plantings. Little growth could be expected during the winter and early spring months, and storm damage can be extensive. Plants which survived the winter (after planting in December) grew more vigorously than plants planted later, indicating that in some cases, sprigs can become established in the winter. Apparently much of the loss in the December plantings was due to the washing out of the sand in which they were planted and the uprooting of the plants. If plants were weighted it is possible that successful plantings might be made at this time of year. Once established through a growing season, survival over the next winter was approximately the same for plants of December or April to June plantings. Apparently, a period of active growth is necessary for the plants to remain intact during their dormant season.

Plantings behind the breakwater versus those in unprotected areas showed little difference after 1 year. Survival rates were initially somewhat higher with protection, but after the first winter total survival was the same in exposed and protected sites. The contour of the beach area behind the breakwater seemed to affect survival of the plantings. In areas that were slightly elevated and in the upper tidal range (i.e., those in the higher contours), high waves, particularly those driven by 10- to 15-knot winds (common in summer), broke or damaged many of the plants and in some cases uprooted them. Also, plants did poorly in very low areas where water accumulated. The breakwater subsided almost 1 foot in some places by March 1979 and probably did not provide the kind of protection from wave action that was initially planned. The exact effect of this subsidence is now difficult to assess since growth is good in both protected and unprotected areas. Growth of December plantings may have been helped by the protection of the breakwater, but since losses were so high, this is also difficult to evaluate.

Density of plantings also appears to have an effect on overall survival. In all cases, high-density plantings survived better than low-density plantings, but overall growth was not significantly affected. In March 1980 good rhizomal spread and top growth appeared in low-density plantings. High-density plantings showed good growth, and individual plants were becoming difficult to distinguish at this time. Survival appears to be more directly affected by time of planting than by density.

Fertilizaton of sprigs did increase the production of stems during the first growing season. Although it is difficult to determine the effect on rhizomal growth, overall performance appears better in these plants. No signs of fertilizer burn were observed even though the fertilizer was in direct contact with the roots of newly planted sprigs. Fertilization with a slow-release fertilizer seems to be beneficial.

Plugs and sprigs were used for high-density planting at this site. Overall survival and growth of the plugs appear to be somewhat better than that of the sprigs, probably due to the initially larger root mass, leaf area, and perhaps the weight. Aboveground growth (stems) did not appear to be as rapid in the plugs as in the sprigs the first growing season, but rhizomal growth was much greater. In March 1980, areas previously planted with plugs had filled in and new shoots were appearing all along the edges of the old area and for some distance into the surrounding sand. Sprig plantings showed some lateral growth, but in no case was this as vigorous as in the plug-planted areas. Mortality was also much less in those areas planted with plugs. This may be in part attributable to the greater weight of the plugs and the difficulty of dislodging them after being planted. The plugs represent about 10 times the biomass of a sprig and are consequently more expensive. It appears that in the long term the plugs establish better and are less subject to destruction than the sprigs and therefore may be the more economical type of planting. It might also be noted that growth of plugs was excellent even when no fertilizer was used.

At the present time, the plantings appear to be established and are growing well. Sand is retained around the base of the established plants and in the rhizomal mass. The plants also provide an esthetically pleasing area. Overall erosion control of this site is probably facilitated by the planting, but the upper bank area supporting the road continues to remain unstable. This area must be stabilized first for the vegetation to have maximum effect.

The device 16 cordgrass plantings were lost prior to the Initial Post Planting Inventory when the breakwater failed and sand drifted over the plot. Therefore, no report on this device was submitted.

c. Oak Harbor, Washington.

(1) Site Description.

(a) Geographical Setting. The Oak Harbor demonstration project is located at the Whidbey Island Naval Air Station on the end of a small peninsula between Oak Harbor and Crescent Harbor. The site faces southward on Saratoga Passage, a tributary of Puget Sound in western Washington. The project occupies 1,175 feet of shoreline at the base of a 30-foot-high bluff between Maylor Point and Forbes Point, about 1 mile southeast of the city of Oak Harbor. A U.S. Navy residential housing project is on top of the bluff behind the project area. The nearest residences are about 50 feet from the edge of the bluff. Figure 2-314 is a regional map showing the project location; Figure 2-315 shows the project site.

(b) Winds, Waves, Tides, and Longshore Transport. The Oak Harbor region has a temperate climate, with cool, dry summers and mild winters. The mean temperature ranges from 39.5° Fahrenheit in January to 71° Fahrenheit in August. The annual rainfall on Whidbey Island averages 20 inches. The waters of Saratoga Passage have average temperatures of about 48° Fahrenheit in winter and 55° Fahrenheit in summer. The salinity is about the same as that of the Pacific Ocean. Prevailing winds are westerly to southeasterly during fall and winter and westerly to northwesterly during spring and summer. Southerly storms are the most severe.

The site is exposed to wind waves from the east, south, and west. Major storm action is from the south-southeast, with maximum forecast deepwater waves of 5.8 feet and periods of 4.8 seconds. However, the breaking height of waves that attack the bluff directly is depth-limited to about 3 feet. Wave action generated by south-southeasterly winds over Saratoga Passage is the primary cause of erosion at the project site.

The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 3.0 feet. Although the energy-flux analysis for 6 months indicates a moderate net potential to transport littoral material at

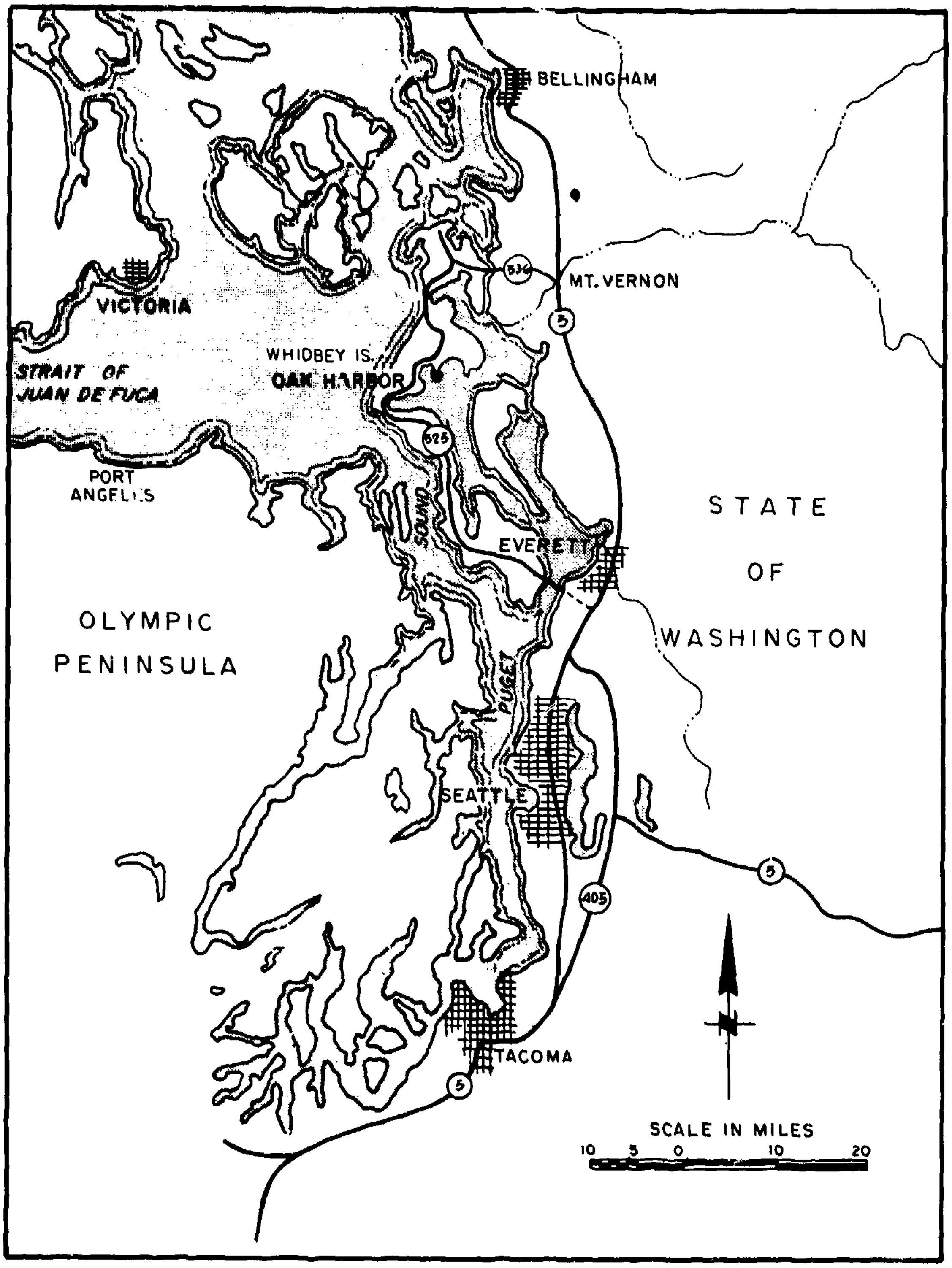


Figure 2-314. Regional map of Oak Harbor, Washington.

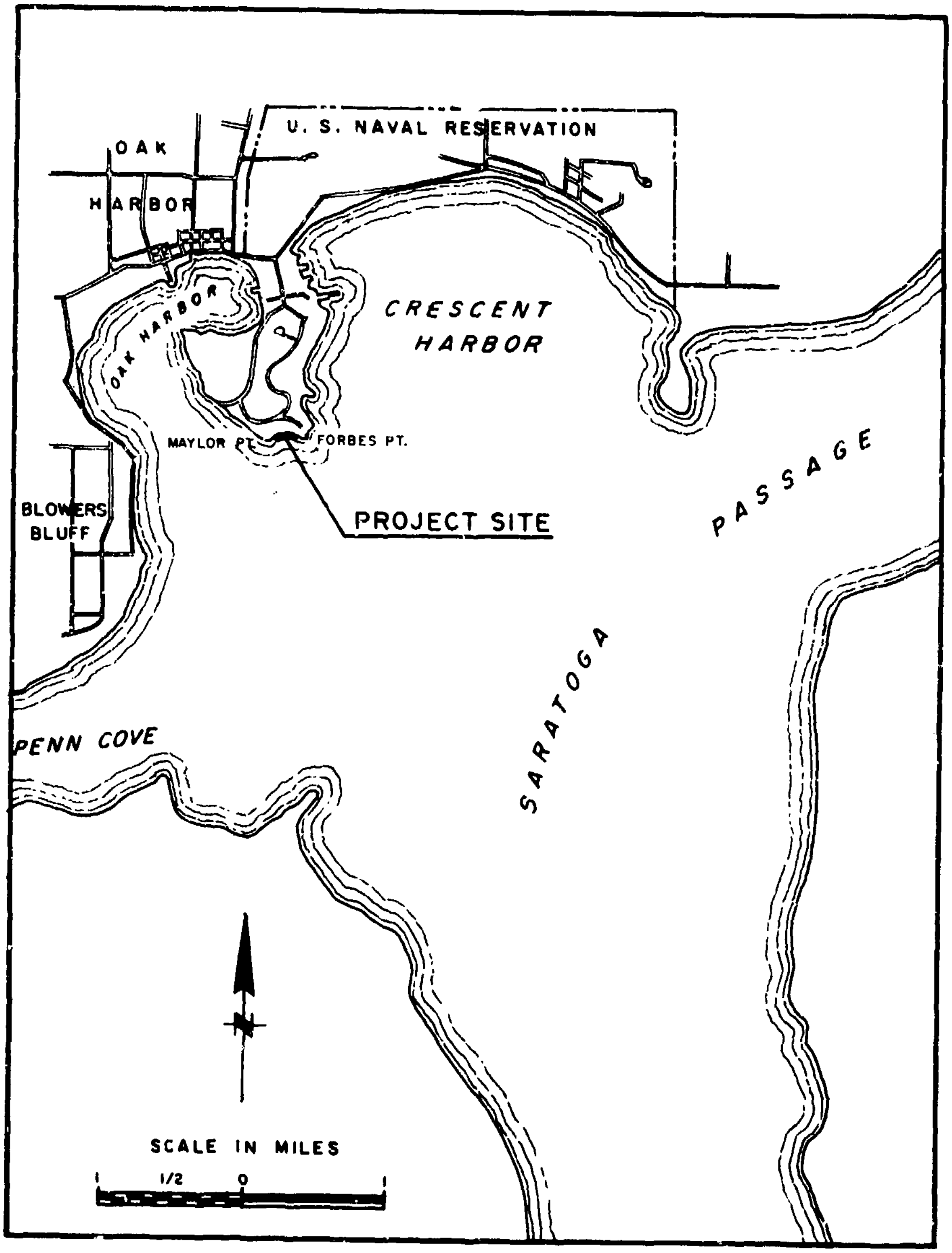


Figure 2-315. Location map of Oak Harbor, Washington, demonstration site.

this site, the problem in this area does not appear to result from longshore transport but rather from direct wave attack.

The datum used at Oak Harbor is MLLW, which is 6.14 feet below NGVD. The mean range of tides is 7.8 feet, the mean diurnal range (MLLW to MHHW) is 11.4 feet, and the estimated extreme high tide is 14.5 feet above MLLW. Tidal currents in the project vicinity are negligible.

(c) Geomorphology, Soils, and Vegetation. The geology of Whidbey Island is characterized by deposits of several glaciations which occurred over the Puget Trough from the north. Most of the exposed materials, deposited by the last glacier 10,000 to 15,000 years ago, consist of compact clay till, sand and gravel outwash, and glaciomarine drift. The present sea cliff developed through direct erosion by waves from the south-southeast which produced undercutting, block calving, and sloughing of till-like sediments as well as individual grain erosion and raveling of the sands. No major slumping or sliding due to soil saturation is evident. The rate of long- and short-term cliff retreat was determined from two lines of evidence. On Maylor Point, erosion has left a concrete drainpipe support (probably built in the early 1940's) looking like a flying buttress, with 7 feet of slope retreat since construction, averaging about 0.2 foot per year. Over a longer period, the bathymetry of the peninsula suggests that the land extended about 3,000 feet farther south at the close of the last glacial period. Thus, the resulting long-term erosion rate of 0.3 foot per year complements that based on more recent historical evidence.

The beach width varies from about 20 feet at MHHW to nearly 200 feet at MLLW. The elevation at the base of the bluff is 1 to 2 feet above MHHW, and at higher tide stages the bluff is occasionally subjected to direct wave attack from the south-southeast. A heavy riprap revetment partially protects about 600 feet of the bluff toe in the vicinity of Forbes Point, but from there to Maylor Point the bluff is unprotected. The littoral mantle on the beach is about 1.5 feet thick and consists of sand and gravel residue from eroded bluff material, the fines having been washed out by wave action. This mantle overlies glacial till of which most of the peninsula is composed. A few larger stones (old riprap), logs, and other debris litter the toe of the bluff along this section. At several places, surface water runoff has scoured gullies that extend from the top of the bluff downward to near the toe. Several of these gullies have undeveloped foot trails used as access to the beach. One such gully has been used for construction of an access trail to the project site for visitors. Native shrubs and grasses grow abundantly at the base of the bluff above MHHW and on the upland behind the bluff; when removed they become reestablished rapidly by natural reseeding (described later in more detail under the discussion dealing with the vegetation plantings at the site).

(d) The Problem. Bluff recession at the project site is not an immediate problem, but the U.S. Navy would eventually have to take corrective action to protect the housing area behind the bluff. The low erosion rate at the Oak Harbor site did not permit a direct assessment of the erosion capabilities of the devices demonstrated. Instead, an emphasis was placed on obtaining information on structure durability, with the assumption that successful designs could be used at other sites to protect the backshore area and prevent wave-caused erosion.

(2) Demonstration Project. Four basic types of shore protection devices were constructed of materials locally available at the Oak Harbor site--a sand-cement bag revetment, a gabion mat revetment, a used-tire bulkhead, and a timber bulkhead. Two variations of each device were used to evaluate the effects of different filters, details in construction materials and methods, and vegetation use. For identification, the device types were numbered 1 to 4, with a and b designating the variation of each type, and E and W designating the east and west halves of each variation where different types of backfill, filter, and vegetation were tested (Fig. 2-316).

Construction methods were by hand labor and by relatively inexpensive equipment including a backhoe, front-end loader, and an auger-drill truck.

To maximize usage of available beach, no space between structures was provided. However, timber-pile bulkheads extending normal to shore were installed between test sections to prevent a failing device from adversely affecting neighboring devices. The length of each device was determined by topographical features. The toe placement for each installation was at or above MHHW, ranging from +11.4 to +13.0 feet MLLW. Therefore, it was expected that the devices would be stressed only when high waves occurred at high tides. Table 2-75 gives specifications for most of the materials used in the various devices.

Table 2-75. Specifications for materials used at Oak Harbor.

Material	Specifications
Sand backfill	Pit-run, 0.5 in minus sandy silt or silty sand with not less than 35 pct by dry weight passing the No. 40 sieve
Gravel backfill, gravel filter	Pit-run, screened 0.5 to 1.5 in, well-graded, 6 in minus clean sandy gravel with 30 to 60 pct by dry weight passing the No. 4 sieve and not more than 10 pct by dry weight passing the No. 200 sieve
Filter cloth	Mirafi-140 and Bidim (nonwoven)
Concrete	Ready-mix, 5.5 sacks portland cement-concrete per cubic yard (gravel included)
Top soil	Local supplier; 12-mi. haul
Toe rock	6- to 10-in shot rock; cobbles or rock containing not more than 5 pct fines passing the No. 200 sieve
Gabion baskets	Bekaert, PVC-clad, 1 ft 8 inches by 3 ft 3 in by 9 ft 9 in
Rock for gabions	15 in minus
Planks and square posts	Douglas fir, creosote pressure treated
Round posts	Douglas fir, 10 feet long with 10-in butt and 8-in tip
Bulkhead logs	Douglas fir (various lengths) with 12-in butt and 10-in tip
Anchor logs	Floating debris washed on beach
Tires (used)	Donated and delivered free by local garage

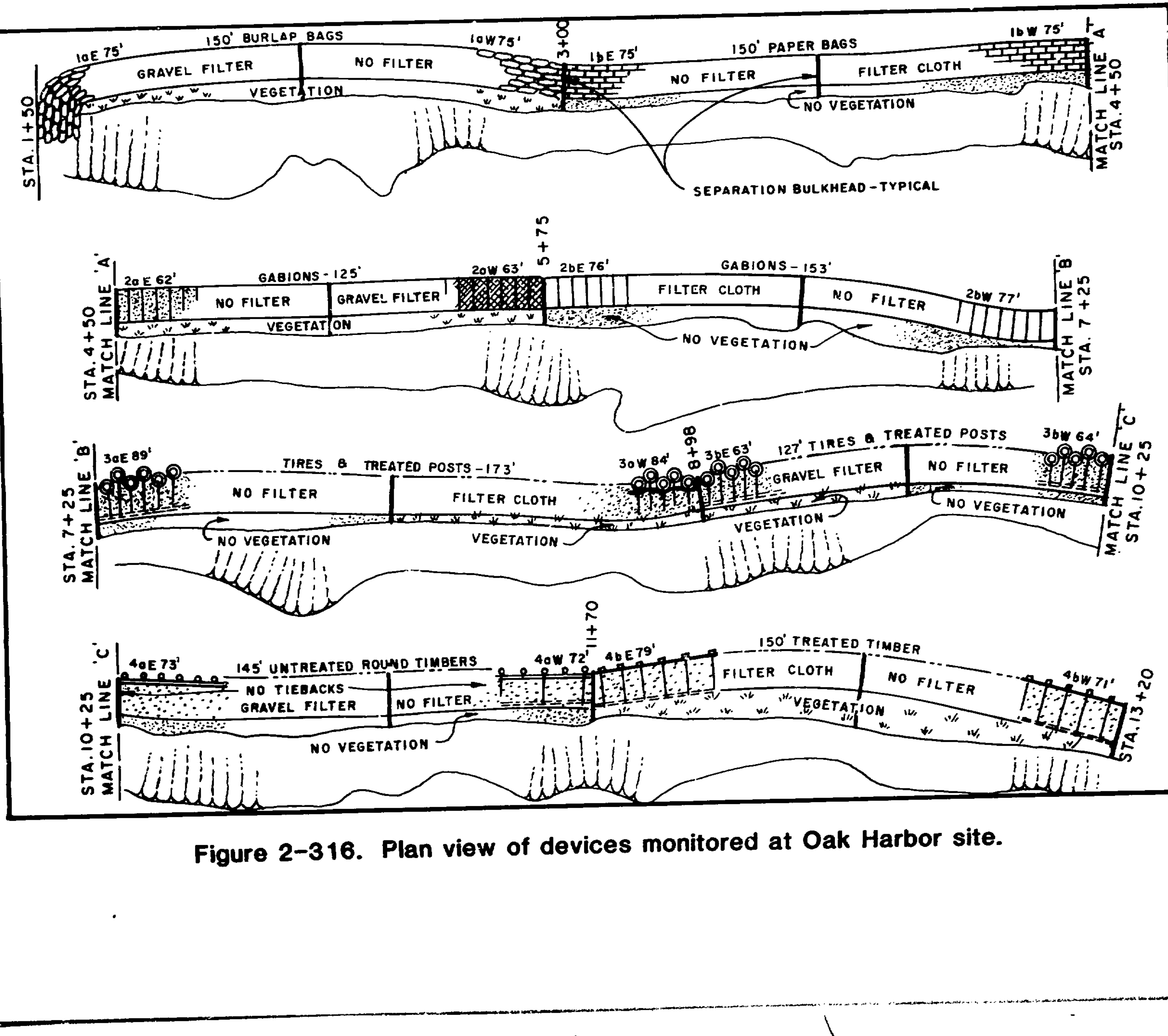


Figure 2-316. Plan view of devices monitored at Oak Harbor site.

(3) Implementation. Construction data for each type and variation of device for the Oak Harbor project are presented in the following subsections. The structural work was done in May and June 1978 by hired labor with rented equipment, and was supervised by Seattle District personnel. The toe rock was placed after completion of the devices. Vegetation was planted in September 1978 by the Soil Conservation Service. All elevations refer to MLLW datum. Design breaker heights for each device are depth-limited by the elevation of the beach at the toe as computed for extreme high stage of tide (+14.5 feet MLLW). Construction details of each device are shown in Figures 2-317 and 2-318. Data on the structural devices follow, succeeded by data on the vegetation program.

(a) Device 1a (Sand-Cement Burlap Bag Revetment).

1 Statistics, Construction, and Costs. Statistics for device 1a are given in Table 2-76.

Table 2-76. Device 1a statistics.

Length	150 ft
Top elevation	+17.5 ft
Toe elevation	+11.5 ft
Estimated breaker height	3.4 ft
Structure slope	1 on 1
Filter type	gravel

The subgrade for the bottom bags was leveled and the sand and gravel backfill placed and dressed to a 1 on 1 slope. In the east 75 feet (1aE), a gravel filter was placed against this slope, before bag placement began. No filter was used for the west 76 feet (1aW). Bags were filled to about 75-percent capacity from the chute of a ready-mix truck with concrete composed of 1 part cement, 2 parts sand, and 3 parts gravel, yielding 5.5 sacks per cubic yard. The bags were then placed in the section in two-bag tiers, each tier stepped back to follow the slope of the filter layer with joints staggered as in brickwork (Fig. 2-319). The tops of the bags were folded under, care being taken to assure full contact between bags, leaving no voids. Two-inch PVC drainpipes were placed between the third and fourth tiers of bags. Filter cloth was placed behind each pipe to preclude pumping of backfill and filter material through the drains. Topsoil was then placed along the top of the revetment by a loader and fine graded. To complete the revetment section, toe rock was dumped along the base of the revetment (Fig. 2-320). Construction costs for device 1a are given in Table 2-77.

(b) Device 1b (Sand-Cement Dry Mix Revetment).

1 Statistics, Construction, and Costs. Statistics for device 1b are given in Table 2-78.

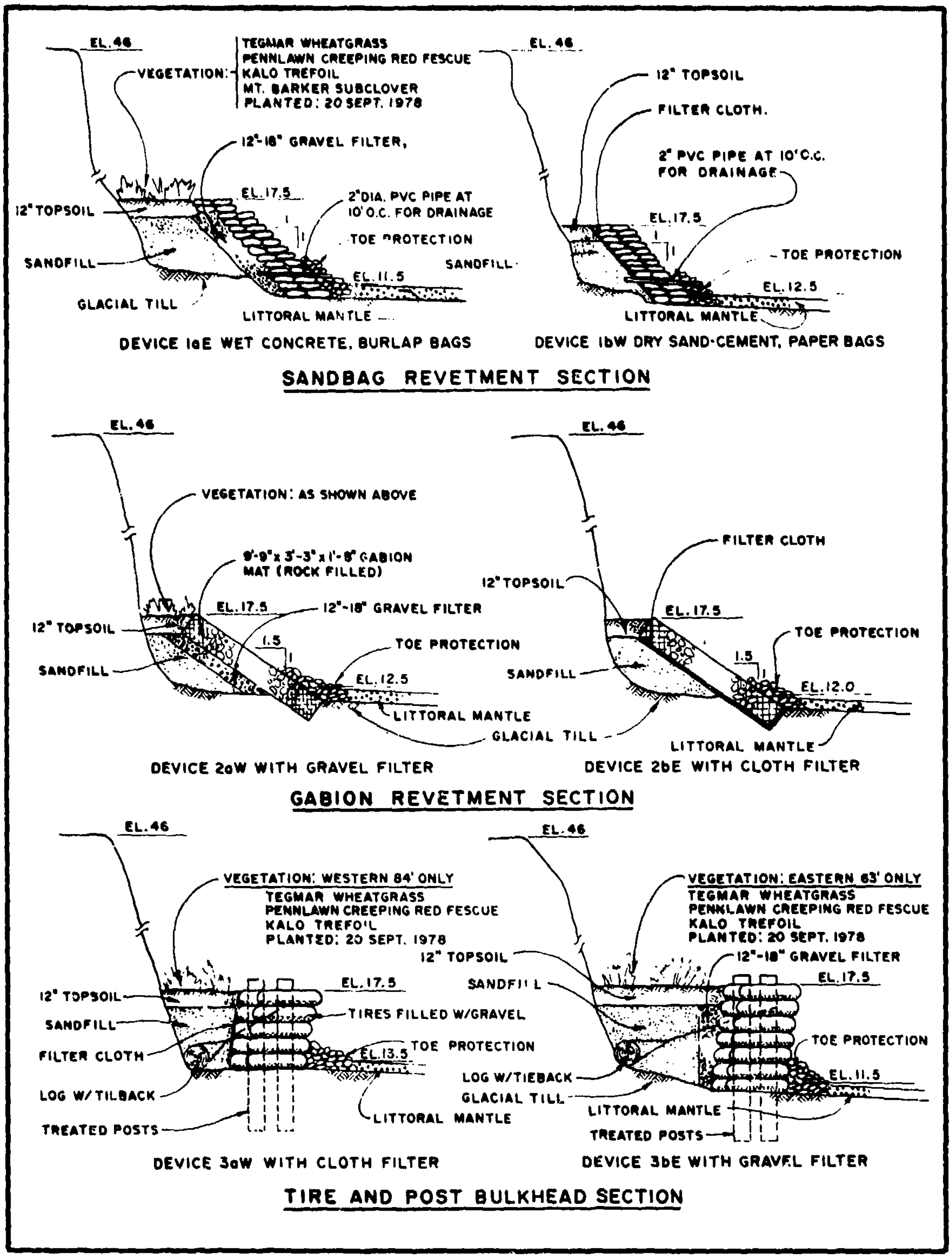


Figure 2-317. Sandbag revetment, gabion revetment, and tire and post bulkhead sections at Oak Harbor site.

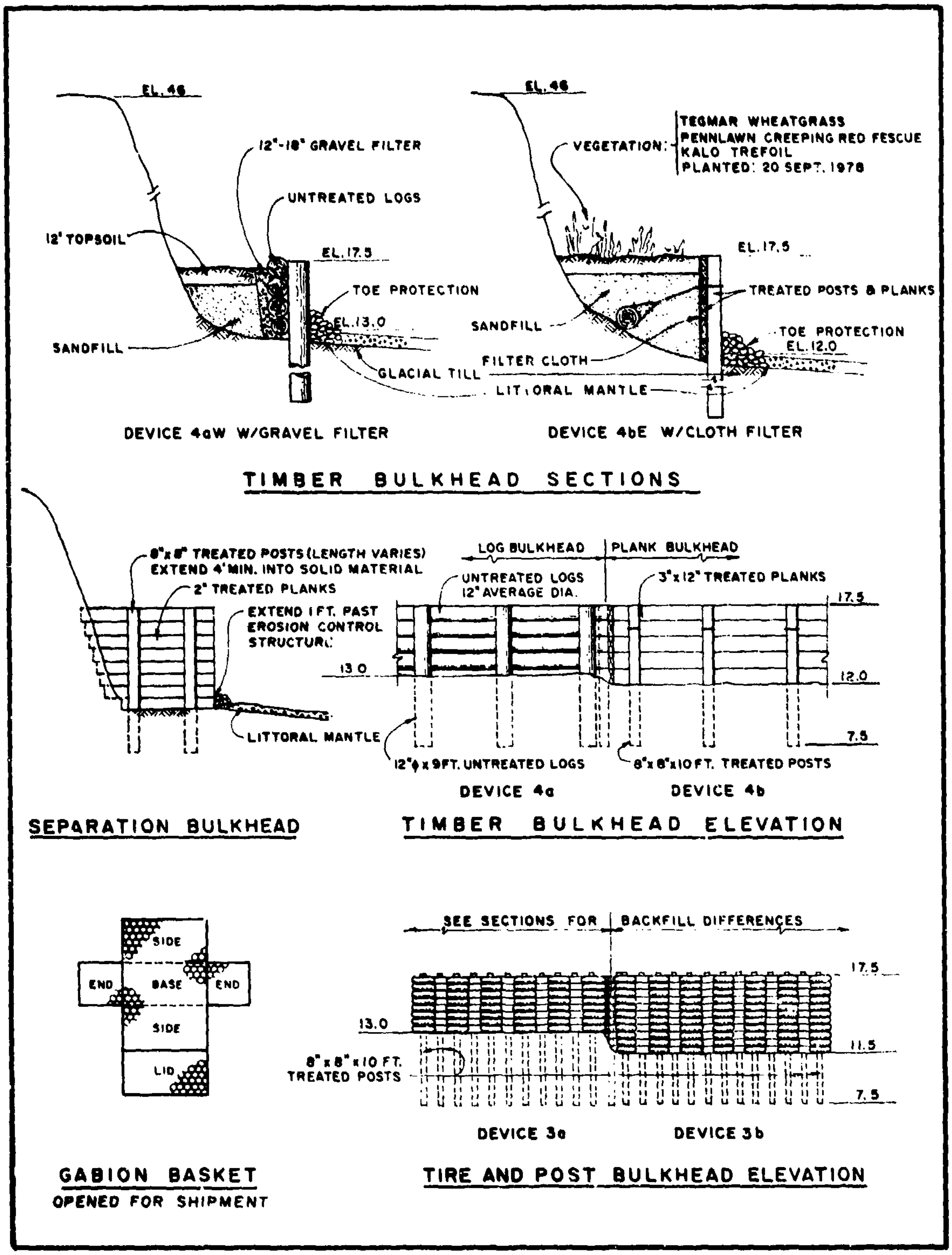


Figure 2-318. Timber bulkhead and tire and post bulkhead sections and elevations at Oak Harbor site.

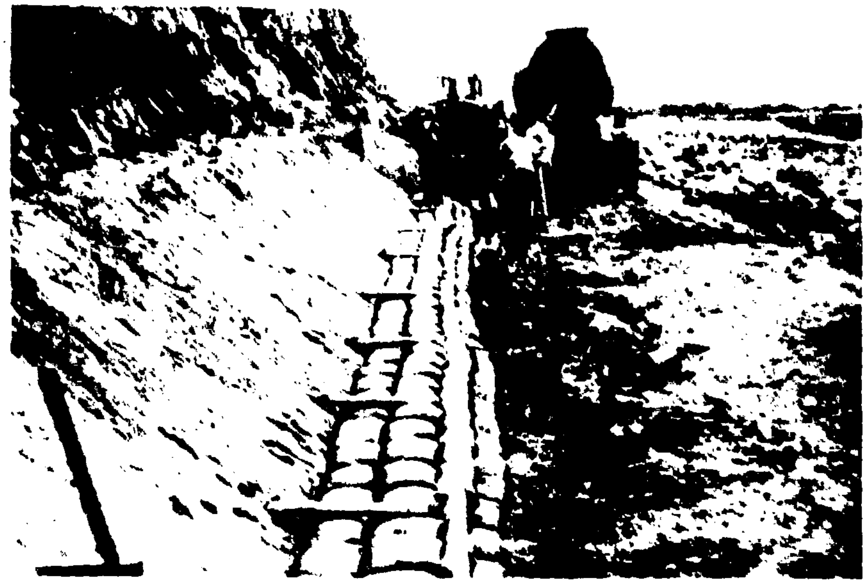


Figure 2-319. Construction of device 1a showing the PVC drains, Oak Harbor, 20 June 1978.

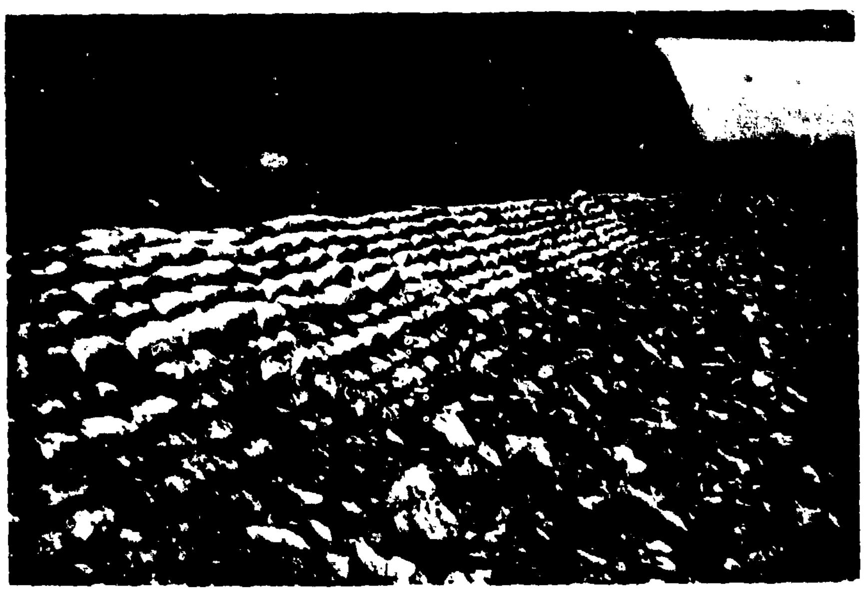


Figure 2-320. Construction of device 1a complete, Oak Harbor, 6 July 1978.

Table 2-77. Device 1a costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$35
Sandbags	4,400	ea.	$0.35	$1,540	
Concrete	66	yd^3	34.25	2,261	
Gravel back-fill	100	yd^3	3.50	525	
Sand backfill	75	yd^3	3.50	263	
Misc. supplies				125	
Filter gravel-(1aE only)	40	yd^3	7.00	280	
Topsoil	10	yd^3	9.75	98	
Toe rock (12 in)	12	ton	10.00	120	
Subtotal				5,212	
Labor	208	hr	10.36	2,155	14
Equipment (loader)	27	hr	36.00	972	6
Total				$8,339	$55

Table 2-78. Device 1b statistics.

Length	146 ft
Top elevation	+17.5 ft
Toe elevation	+12.5 ft
Estimated breaker height	1.8 ft
Structure slope	1 on 1
Filter type	filter cloth

Preparation of the subgrade and placement of backfill proceeded as for device 1a, but only sand was used for backfill. Paper bags filled with dry mix sand-cement were placed as in device 1a, but each tier was punctured with pitchforks and saturated with freshwater by a garden hose before the next layer was placed (Fig. 2-321). No filter was placed behind the eastern half of device 1b, but filter cloth was placed behind the western half (1bw). The inner ends of the PVC drainpipes were placed next to the filter cloth of the western segment, and filter cloth was placed around the inner ends of the eastern segment. Interbag bonding was poor throughout device 1b, with small voids at joints between bags. No vegetation was planned for device 1b because of its close proximity to the bluff; however, toe protection was the same as for device 1a (Fig. 2-322). Construction costs for device 1b are given in Table 2-79.

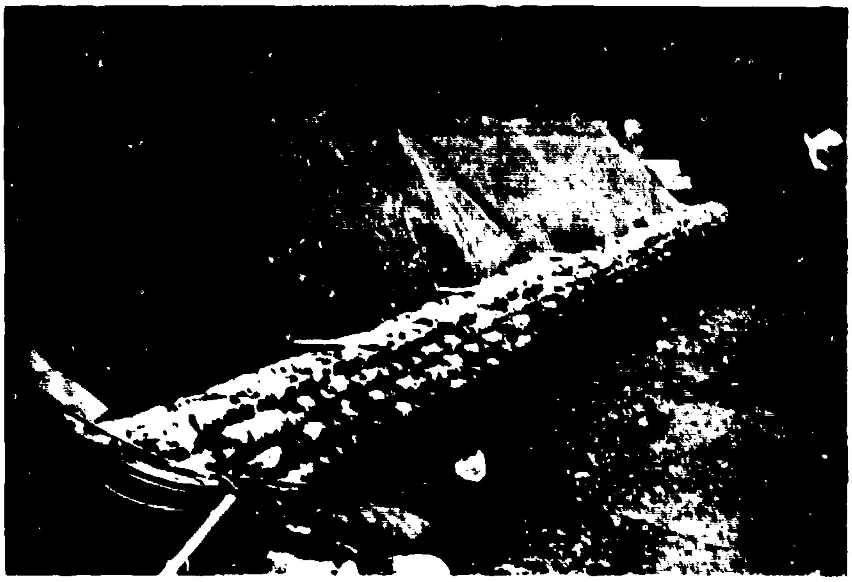


Figure 2-321. Saturating the bags of device 16 to hydrate bond the sand-cement mixture, Oak Harbor, 16 June 1978.

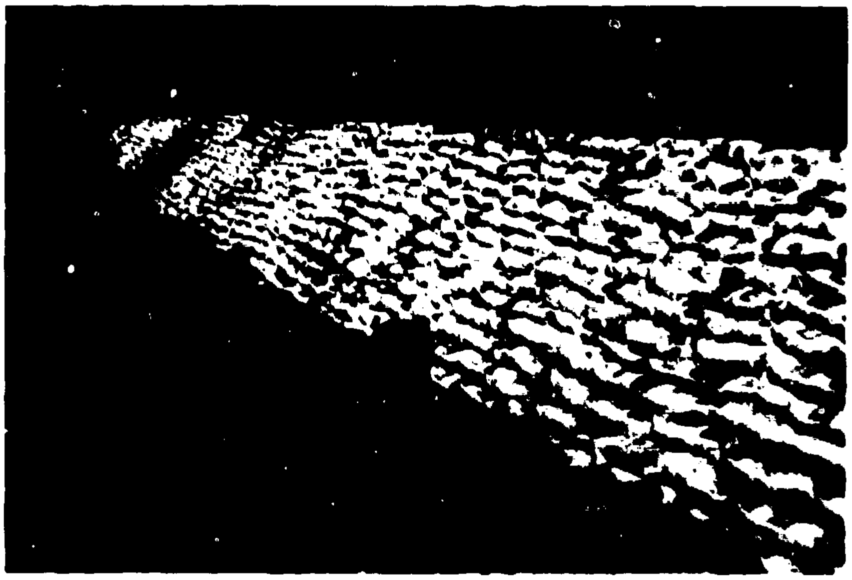


Figure 2-322. Construction of device 1b complete, Oak Harbor, 6 July 1978.

Table 2-79. Device 1b costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$54
Sand-cement-filled bags	4,000	ea	$1.68	$6,720	
Sand backfill	200	yd^3	3.50	700	
Misc. supplies				125	
Filter cloth	850	ft^2	0.08	68	
Topsoil	20	yd^3	9.75	195	
Toe rock (12 in)	12	ton	10.00	120	
Subtotal				7,928	
Labor	143	hr	10.36	1,481	10
Equipment (loader)	80	hr	36.00	2,880	20
Total				$12,289	$84

(c) Device 2a (Rockfilled Wire, Gabion-Mat Revetment).

1 Statistics, Construction, and Costs. Statistics for device 2a are given in Table 2-80.

Table 2-80. Device 2a statistics.

Length	126 ft
Top elevation	+17.5 ft
Toe elevation	+12.5 ft
Estimated breaker height	1.8 ft
Structure slope	1 on 1.5
Filter type	gravel

Rough excavation for the lower edge of the revetment and placement of backfill materials and gravel filter were accomplished with the loader. Fine grading was done by hand shovels and rakes to prepare the bed for the gabions. The wire baskets were placed side-by-side on the bed with all lids opening in one direction, and then filled (one at a time) with stone by the loader (Fig. 2-323). Final grading of stones just under the lid was done by hand; the lid was then closed and wired to the sides of the adjacent basket with twists of wire at about 6-inch intervals to save time, rather than by continuous wire lacing as recommended by the manufacturer. Placement and filling of baskets proceeded from one end of the section to the other. The gravel filter was placed only behind the western half of device 2a. Finally, the topsoil layer was placed behind the upper edges of the baskets, and toe protection was dumped along the base of the revetment (Fig. 2-324). Construction costs for device 2a are given in Table 2-81.

Figure 2-323. Filling the wire baskets of device 2a, Oak Harbor, 31 May 1978.

Figure 2-324. Toe protection along device 2a, Oak Harbor, 25 May 1978.

Table 2-81. Device 2a costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$28
Gabions	38	ea.	$35.00	$1,330	
Rock (12 in)	35	ton	10.00	850	
Gravel backfill	70	yd^3	3.50	245	
Sand backfill	150	yd^3	3.50	525	
Misc. supplies				125	
Filter gravel	30	yd^3	7.00	210	
Topsoil	10	yd^3	9.75	98	
Toe rock (12 in)	10.5	ton	10.00	105	
Subtotal				3,488	
Labor	120	hr	10.36	1,243	10
Equipment (loader)	61	hr	36.00	2,196	17
Total				$6,927	$55

(d) Device 2b (Rockfilled Wire, Gabion-Mat Revetment).

1 Statistics, Construction, and Costs. Statistics for device 2b are given in Table 2-82.

Table 2-82. Device 2b statistics.

Length	153 ft
Top elevation	+17.5 ft
Toe elevation	+12.0 ft
Estimated breaker height	2.6 ft
Structure slope	1 on 1.5
Filter type	filter cloth

Construction of device 2b was the same as for device 2a except that pairs of baskets were placed with lids opening outward so that both could be loaded at the same time, thereby reducing the loading time. The filter cloth was placed only under the eastern half of the revetment, on the prepared bed up the slope to the level of the upper edge of the basket bottom. After the baskets were filled and tops wired down, the backfilling was completed to elevation +17.5 feet. Construction costs for device 2b are given in Table 2-83.

Table 2-83. Device 2b costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
<u>Materials</u>					$24
Gabions	46	ea.	$35.00	$1,610	
Rock (12 in)	100	ton	10.00	1,000	
Gravel backfill	130	yd^3	3.50	455	
Sand backfill	90	yd^3	3.50	315	
Misc. supplies				125	
Filter cloth (2bE only)	910	ft^2	0.08	73	
Toe rock (12 in)	11	ton	10.00	110	
Subtotal				3,688	
<u>Labor</u>	161	hr	10.36	1,668	11
<u>Equipment</u>	49	hr	36.00	1,764	12
Total				$7,120	$47

(e) <u>Device 3a (Used-Tire Bulkhead)</u>.

<u>1</u> <u>Statistics, Construction, and Costs</u>. Statistics for device 3a are given in Table 2-84.

Table 2-84. Device 3a statistics.

Length	173 ft
Top elevation	+17.5 ft
Toe elevation	+13.0 ft
Estimated breaker height	1.0 ft
Structure slope	vertical
Filter type	filter cloth

After leveling the subgrade along the two rows of tires, the locations for post holes were spotted and 12-inch holes drilled to +7.5 feet MLLW by an auger-drill truck. The posts were placed in the holes and held vertical while the gravel backfill was tamped around the posts to secure them in place (Fig. 2-325). Filter cloth was used only in the western half of the section (3aW). The tires were then placed concentrically around the posts and filled with gravel as each tier was completed. When construction reached the next-to-top tier, a 3/8-inch wire cable was looped around each front post and fastened with a cable clip. The free ends were looped around the anchor log, which had been placed near the base of the cliff, and similarly fastened with cable clips. The cables were then tightened by shoving the log as far back as possible before backfilling (Fig. 2-326). A

Figure 2-325. Construction of device 3a, Oak Harbor, 14 June 1978.

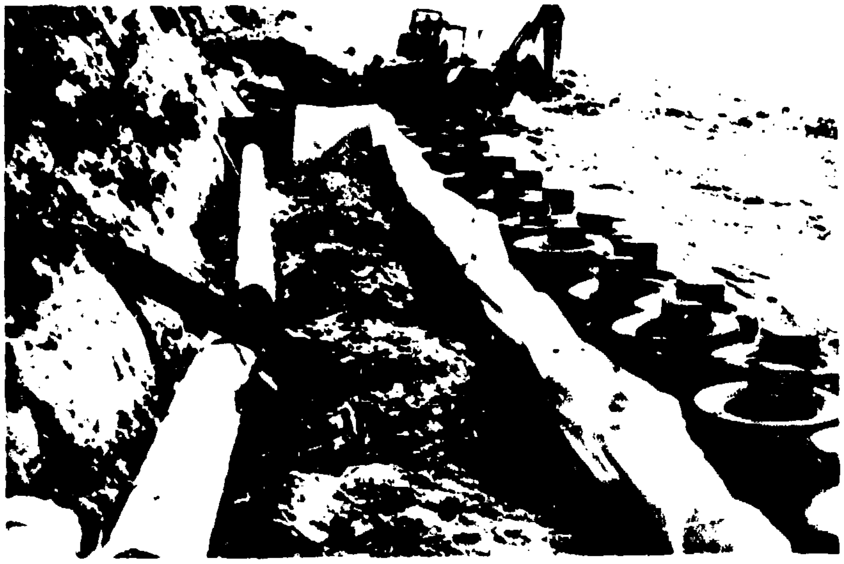


Figure 2-326. Device 3aW before placement of backfill, Oak Harbor, 14 June 1978.

separate loop of cable was also placed around each stack of tires to prevent loss of tires in case of a failure. In the western half, the filter cloth was stretched vertically against the backface of the tires before backfilling. The section was then completed by placing and filling the top tier of tires and placing the remainder of the backfill. To prevent tires from being lifted off, a 5/8-inch round steel bar 30 inches long was driven through a tight-fit hole drilled through each post above the level of the top tire and then bent down against the tire. To complete the bulkhead, topsoil and toe rock were placed using the loader (Fig. 2-327). Construction costs of device 3a are given in Table 2-85.

Table 2-85. Device 3a costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$30
Posts (8 in by 8 in by 10 ft)	85	ea.	$31.00	$2,635	
Used tires	714	ea.	0.00	0	
Gravel backfill	200	yd^3	3.50	700	
Washed gravel	90	yd^3	7.00	630	
Misc. supplies				935	
Filter cloth (3aW only)	760	ft^2	0.08	61	
Toe rock (12 in)	15	ton	10.00	150	
Subtotal				5,111	
Labor	226	hr	10.36	2,341	14
Equipment					13
Drill	39	hr	30.40	1,186	
Loader	30	hr	36.00	1,080	
Total				$9,718	$57

(f) Device 3b (Used-Tire Bulkhead).

1 Statistics, Construction, and Costs. Statistics for device 3b are given in Table 2-86.

Table 2-86. Device 3b statistics.

Length	127 ft
Top elevation	+17.5 ft
Toe elevation	+11.5 ft
Estimated breaker height	3.4 ft
Structure slope	vertical
Filter type	gravel

Figure 2-327. Placement of toe protection fronting device 3a, Oak Harbor, 14 June 1978.

The construction procedure for device 3b was the same as that for device 3a except that a gravel filter was used in the eastern half of the section (3bE). No filter was used in the western half (3bW). Also, the drill truck encountered much harder material in this segment, requiring more drill time. Construction costs are given in Table 2-87.

Table 2-87. Device 3b costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$36
Treated posts (8 in by 8 in by 10 ft)	64	ea.	$31.00	$2,014	
Used tires	575	ea.	0.00	0	
Gravel backfill	100	yd^3	3.50	350	
Sand backfill	50	yd^3	3.50	175	
Washed gravel	90	yd^3	7.00	630	
Misc. supplies					
Gravel filter	30	yd^3	7.00	210	
Topsoil	20	yd^3	9.75	195	
Toe rock (12 in)	11	ton	10.00	110	
Subtotal				4,619	
Labor	292	hr	10.36	3,025	24
Equipment					28
Drill	49	hr	30.40	1,490	
Loader	57	hr	36.00	2,052	
Total				$11,186	$88

(g) Device 4a (Untreated-Timber Bulkhead).

1 Statistics, Construction, and Costs. Statistics for device 4a are given in Table 2-88.

Table 2-88. Device 4a statistics.

Length	145 ft
Top elevation	+17.5 ft
Toe elevation	+13.0 ft
Estimated breaker height	1.0 ft
Structure slope	approximately vertical
Filter type	gravel

After leveling off the base for the bottom log of this bulkhead, a row of holes about 18 inches in diameter and spaced 4 feet on centers was drilled to -7.5 feet MLLW along the bulkhead alinement with the auger-drill truck (Fig. 2-328). Untreated log posts were placed in the holes and held vertical and on line while the holes were backfilled with gravel and tamped (Fig. 2-329). Holes were then drilled almost through the horizontal logs so that they could be spiked to the vertical posts to hold them in place until backfilled. Because the soil was very wet in the extreme western end of 4aW, 3/8-inch cable ties were attached to about every other vertical post between top and next-to-top horizontal log and secured to anchor logs as for device 3. No tiebacks were used in the remainder of device 4a. Backfilling was done by the loader (Fig. 2-330). The gravel filter was used only in the eastern half of the section. Construction costs for device 4a are given in Table 2-89.

Table 2-89. Device 4a costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$30
Posts (untreated)	40	ea.	$30.00	$1,200	
Logs (untreated)	750	lin ft	2.40	1,800	
Gravel backfill	90	yd^3	3.50	315	
Sand backfill	100	yd^3	3.50	350	
Misc. supplies				380	
Gravel filter (4aE only)	30	yd^3	7.00	210	
Toe rock (12 in)	12	ton	10.00	120	
Subtotal				4,375	
Labor	126	hr	10.36	1,305	
Equipment					
Drill	33	hr	30.40	1,003	9
Loader	26	hr	36.00	936	13
Total				$7,619	$52

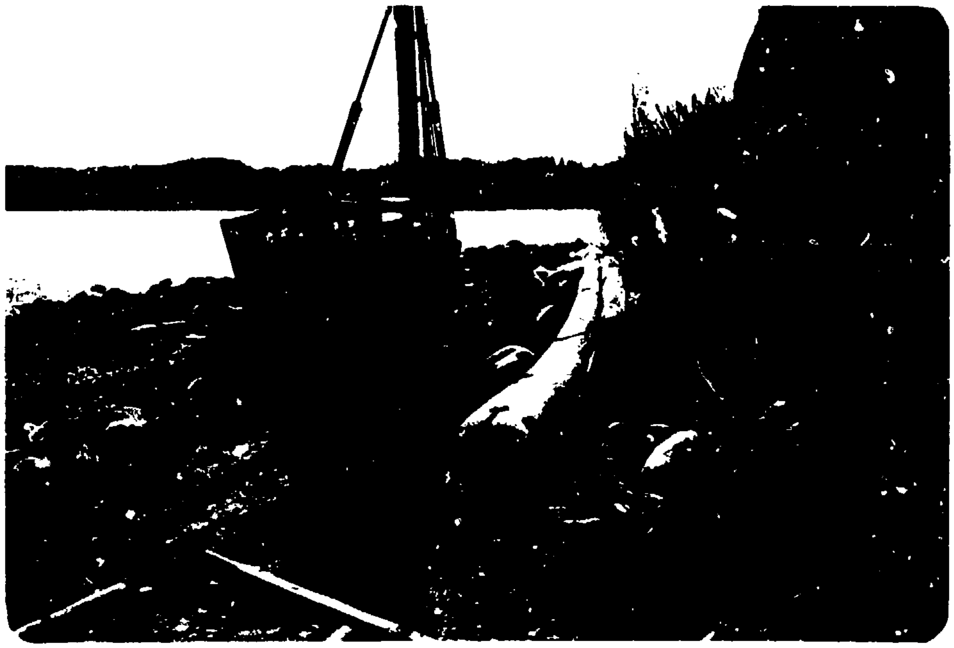


Figure 2-328. Use of an auger-drill truck to place the untreated log posts, Oak Harbor, 4 May 1978.

Figure 2-329. Construction of device 4a, Oak Harbor, 10 May 1978.

Figure 2-330. Placement of backfill for device 4a, Oak Harbor, Washington, 16 May 1978.

(h) Device 4b (Treated-Timber Bulkhead).

1 Statistics, Construction, and Costs. Statistics for device 4b are given in Table 2-90.

Table 2-90. Device 4b statistics.

Length	150 ft
Top elevation	+17.5 ft
Toe elevation	+12.0 ft
Estimated breaker height	2.6 ft
Structure slope	approximately vertical
Filter type	filter cloth

Base preparation and post spacing were the same as for device 4a, but 8-inch square treated posts were used in lieu of untreated round posts, and although the drilled holes were smaller in diameter, much harder material was encountered and drilling time almost doubled. Treated 3- by 12-inch planks were spiked to the posts in place of the horizontal logs used in device 4a. Tiebacks were used at every post, the ties passing through holes drilled in the center of the next-to-top plank on each side of each post. In the eastern half of the section, filter cloth was stapled to the inside of the planks before backfilling (Fig. 2-331). No filter material was used in the western half. One foot of topsoil was placed over the sand and gravel backfill and toe rock was dumped along the base of the revetment (Fig. 2-332). Construction costs for device 4b are given in Table 2-91.

Figure 2-331. Filter cloth stapled to the planks of device 4bE, Oak Harbor, 19 May 1978.

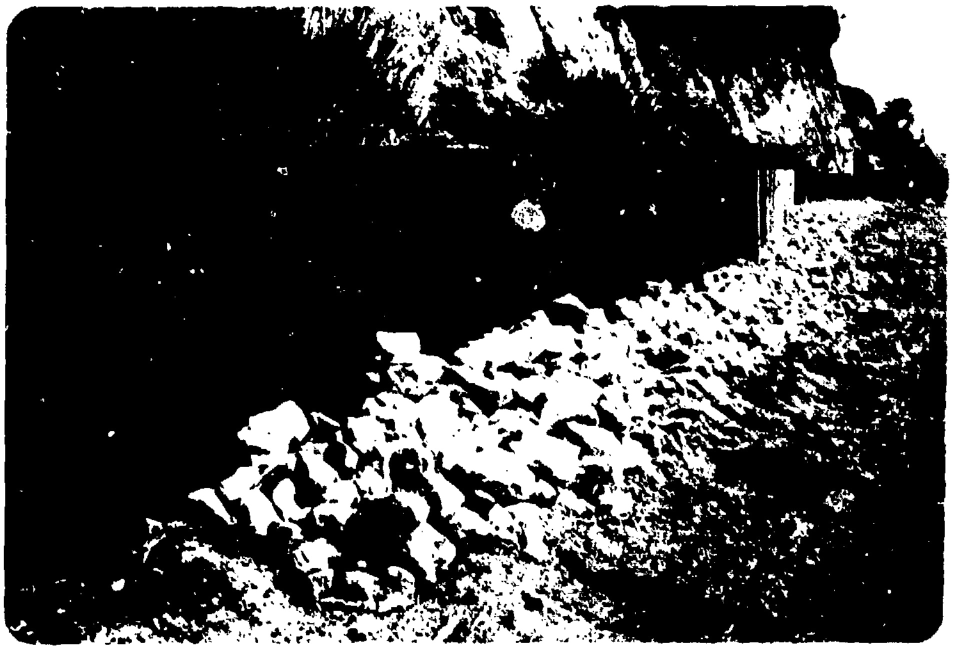


Figure 2-332. Placement of toe protection for device 4b, Oak Harbor, 31 May 1978.

Table 2-91. Device 4b costs.

Item	Quantity	Unit	Unit price	Cost	Cost/ft
Materials					$29
Treated posts (8 in by 8 in by 10 ft)	40	ea.	$31.45	$1,258	
Treated planks (3 in by 12 in by 20 ft)	2,700	fbm	0.60	1,620	
Gravel backfill	110	yd^3	3.50	385	
Sand backfill	100	yd^3	3.50	350	
Misc. supplies				380	
Filter cloth	710	ft^2	0.08	57	
Topsoil	25	yd^3	9.75	244	
Toe rock (12 in)	13	ton	10.00	130	
Subtotal				4,424	
Labor	212	hr	10.36	2,196	15
Equipment					
Drill	60	hr	30.40	1,824	22
Loader	40	hr	36.00	1,440	
Total				$9,884	$66

(i) Separation Bulkheads. These bulkheads were installed essentially in the same manner as device 4b; however, because they were perpendicular to the bluff, tiebacks were not used (Figs. 2-333 and 2-334). The bulkhead was installed between the different types of devices before the devices themselves were installed. Lengths varied with the distance of the seaward face of the device from the bluff. Planks were placed tightly against one another so that no filter was needed to prevent lateral pumping of backfill through cracks from one side to the other in case one device failed and its neighbor did not. No detailed cost estimate is provided for the bulkheads, as they would not normally be used in conjunction with a shore protection project except possibly as end returns to prevent flank erosion. However, cost per foot was probably about the same as for device 4b.

(j) Vegetation. Initial plantings were made as part of the overall erosion control design behind devices 1a, 2a, 3aw, 3bE, and 4b. The area available for planting was a narrow strip of upland between the revetments, bulkheads, and the eroding 30-foot bluff, created when backfill material was added in July 1978.

In September 1978 all planting areas were seeded with a mixture of intermediate wheatgrass (Agropyron intermedium), creeping red fescue (Festuca rubra), and two clovers, Kalo trefoil (Trifolium sp.) and Mt. Barker subclover (Trifolium sp.). Most of these areas were either washed out or covered with debris during the winter months.

In March and April 1979, European beachgrass (Ammophila arenaria) and shrub species Kinnikinik (Arctostaphylos uva-ursi), salal (Gautheria shallon), ocean spray (Holodiscus discolor), Nootka rose (Rosa nutkana), snowberry (Symphoricarpos albus), and willows (Salix sp.) were planted behind the structures. Planting was divided into sections (areas) as shown on Figure 2-335. In late fall 1979 the area behind the gabion revetment was seeded with Largo tall wheatgrass (Agropyron elongatum).

The species, spacing, planting method, and numbers planted for each section are given in Table 2-92. A summary of costs is given in Table 2-93.

Table 2-92. March-April 1979 vegetation plantings, Oak Harbor, Washington

Section	Species	No. of rows or hills	Spacing (in)	No. of plants	Planting method
1	Kinnikinik	2	36	17	4-in pot
	Salal	3	36	24	4-in pot
2	Kinnikinik	2	18	24	4-in pot
	Salal	2	18	24	4-in pot
3	Willow	1	18	131	cuttings
	Ocean spray	1	18	73	bare root
	Nootka rose	1	18	84	bare root
	Snowberry	2	18	163	bare root
4	European beachgrass	-	18	500±	culms
5	Willow	1	12-18	72	cuttings
6	Willow	-	12-18	84	cuttings
	Snowberry	5	24	25	--
	Nootka rose	5	24	25	--
	Ocean spray	5	24	25	--
7	Willow	3	18	127	cuttings
8	Wheatgrass	← seeding deferred to fall 1979 →			seed

Note: All shrubs were fertilized at time of planting with 30 to 40 0.5-cm granules of Mag-Amp (6-40-16) slow-release fertilizer.

Figure 2-333. Separation bulkhead used between devices, Oak Harbor, 25 May 1978.

Figure 2-334. Separation bulkhead perpendicular to bluff, Oak Harbor, 11 May 1978.

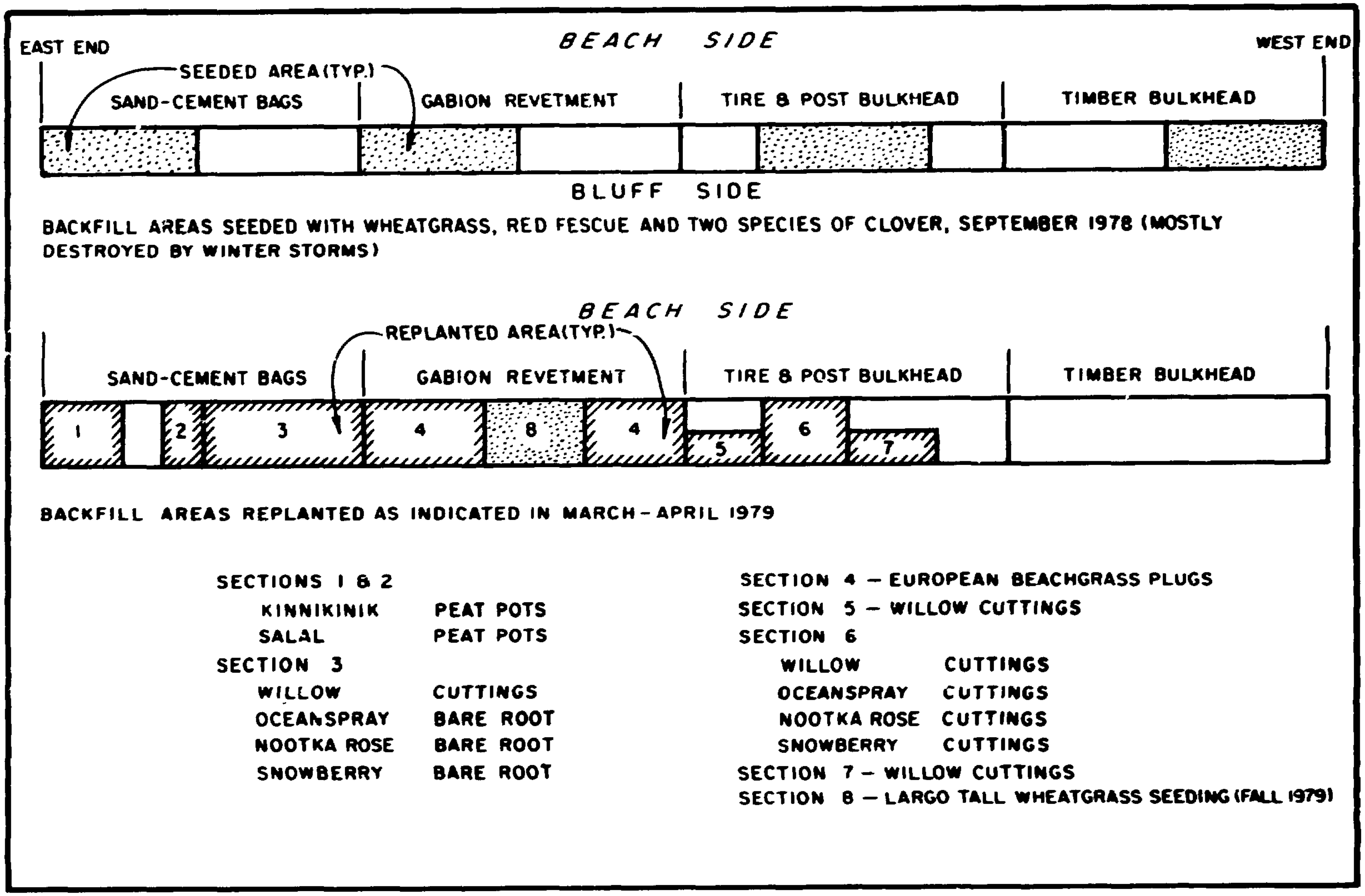


Figure 2-335. Vegetation planting scheme at Oak Harbor; planting date 20-21 March 1979 except where noted.

Table 2-93. Cost of vegetation plantings.

Item	Quantity	Unit Cost	Total Cost
Grass legume mix			
Tegmar intermediate wheatgrass	3 lb		$4.00
Cascade birdsfoot trefoil (inoculant)	1 lb		2.50
Pennlawn creeping red fescue	1 lb		1.50
Mt. Barker sub-clover (inoculant)	1 lb		2.00
Shrubs			
Ocean spray and Nootka rose, contract propagation	350 plants	$0.75 ea.	265.00
Salal	65 plants	1.50 ea.	97.50
Kinnikinik	65 plants	1.50 ea.	97.50
Labor			
Grass planting (broadcasting seed, inoculating legumes, covering seed manually, and fertilizing)	1 man-day	100.00	100.00
Collecting and processing 550 native willow and 550 snowberry hardwood cuttings on site	3 man-days	100.00	300.00
Planting 1,450 cuttings and shrubs, at 60/hr	3 man-days	100.00	300.00
Planting 130 shrubs, at 30/hr (gallon-size material required holes dug)	0.63 man-days		65.00
Fertilizer			
Grass (16-20-0-12)	400 lb/acre		10.00
Shrub			5.00
Downtime (travel)	4 man-days	100.00	400.00
Overhead (5 pct)			80.00
Total			$1,730.00

(4) Performance.

(a) Before Storm Attacks. All devices remained as installed during the summer and fall of 1978. Vegetation plantings appeared healthy and well established but were not yet spreading to achieve the desired density of coverage. No significant volunteer growth appeared in any of the unplanted sections. The beach fronting the test project remained about the same as at installation. No changes were detected in the quarterly ground surveys or aerial photos. However, some bluff erosion due to weathering was noted, and a runoff channel across device 2bE developed. Early in December, some toe rock had also been displaced from in front of devices 1b and 3, and erosion of backfill in device 4aW had occurred.

(b) Storm of 16-17 December 1978. A front moved through the area during this period bringing rain and high winds with gusts over 60 miles per hour. High tides at Oak Harbor for both days were 12.9 feet MLLW. Although no LEO observations were made, breaking wave heights were estimated at 2.5 to 3 feet. Damage to the shore protection devices was primarily restricted to erosion of backfill and displacement of toe protection material. Erosion of backfill was mostly limited to areas constructed without filter cloth or gravel filters.

Devices 1a and 1b (sand-cement bags) suffered no notable damage except for a significant amount of toe rock displaced seaward. Debris (including a 1-foot-diameter log) was noted on top of the structure. Some erosion of backfill occurred behind devices 2a and 2b (Fig. 2-336). A large amount of toe rock was displaced and spread around on the foreshore. Several gabions had pockets where undersized rock had been washed out.

Devices 3a and 3b (used tires) suffered the most severe damage, particularly 3bW which did not have a filter behind it, and a large amount of backfill material washed out from behind the structure (Fig. 2-337). Device 3bE, which had a gravel filter, suffered far less erosion damage as shown in the background of Figure 2-337. Along device 3aW gravel had been washed from many of the tires (Fig. 2-338). In about 10 percent of the tire stacks, all the gravel was missing. Toe rock had been scoured from in front of long sections of the device. Devices 4a and 4b (timber) also evidenced erosion of the backfill, particularly where no filter was present (Fig. 2-339). Much of the toe rock remained in place along devices 4a and 4b.

The separation bulkheads were working well, and the damaged devices did not appear to be endangering any of the adjacent structures at this time. The planted vegetation was unharmed except where the topsoil was undermined by scour holes and trenches where backfill was lost under or through a bulkhead.

(c) Storm of 13 February 1979. This storm occurred concurrently with a recorded high tide of 13.8 feet MLLW at Oak Harbor (0.5 foot higher than at Seattle due to wind setup). Winds in the nearby area, mostly from the south, were measured with gusts in excess of 100 miles per hour. A LEO observer estimated that waves broke at the structures with heights up to about 3.5 feet with a period of 4.3 seconds, 2 hours after peak winds. Considering the concurrence of high waves with the extreme high tide, this event would appear to have about a 10-year recurrence interval. However, because of the extremes of tides and winds common to this region, annual events are only slightly less severe. The waves overtopped most of the devices, severely eroding the backfill and destroying much of the vegetation.

Devices 1a and 1b (sand-cement bags) were undamaged with the exception that most of the toe protection material had been displaced (Fig. 2-340). Drift debris littered the backfill area along most of this reach (Fig. 2-341).

A large amount of debris was deposited on the backfill behind devices 2a and 2b (gabion mat) (Fig. 2-342), and the overtopping waves caused some erosion of the backfill material, particularly behind device 2bW (Fig. 2-343).

Figure 2-336. Device 2bW (foreground) without filter; device 2bE (background) with filter cloth. Oak Harbor, 19 December 1978.

Figure 2-337. Device 3bW (foreground) without filter; device 3bE (background) with gravel filter. Oak Harbor, 19 December 1978.

Figure 2-338. Device 3aW, gravel missing from inside of tires, Oak Harbor, 19 December 1978.

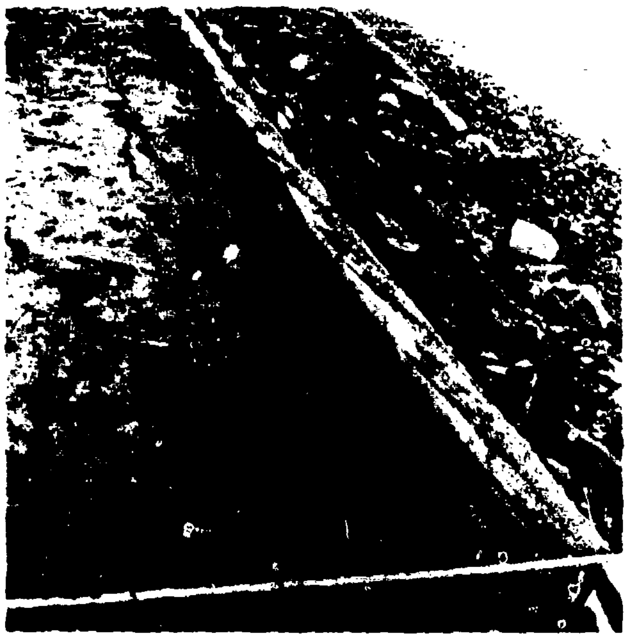


Figure 2-339. Device 4aW without filter, Oak Harbor, 19 December 1978.

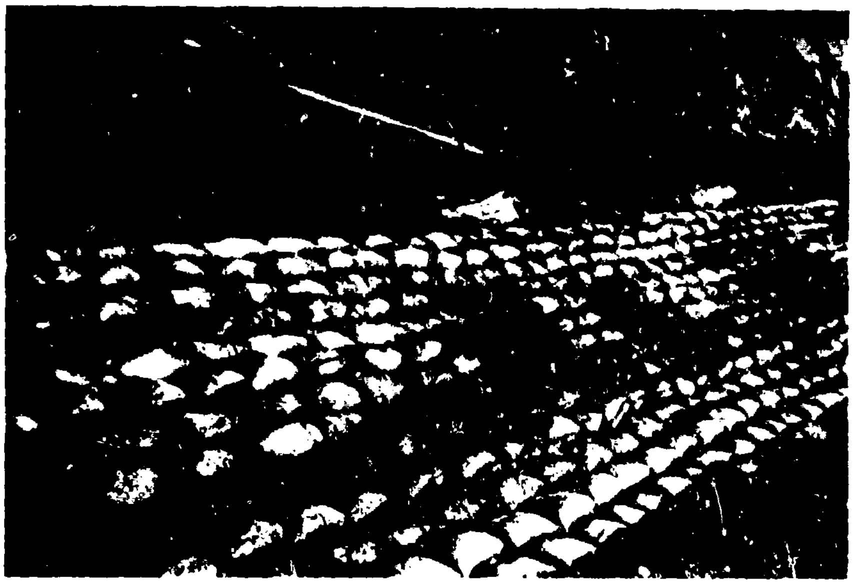


Figure 2-340. Device la intact following the February storm, Oak Harbor, 14 February 1979.

Figure 2-341. The backfill area of device la littered with debris, Oak Harbor, 14 February 1979.

Figure 2-342. Debris scattered along the backfill area behind the gabions, Oak Harbor, 14 February, 1979.

Figure 2-343. Eroded backfill area behind device 2bW, Oak Harbor, 14 February 1979.

Material was washed out along the side of the separation bulkhead between devices 2a and 2b, causing a lowering of backfill for a distance of 20 feet on either side (Fig. 2-344). The tops of the gabions in this area were lowered about 3 feet.

The western part of device 3bW (used-tire bulkhead without filter material) suffered significant erosion of the silty sand backfill material (Fig. 2-345), but the structure did not appear to be in immediate danger of failing. In device 3aE (used-tire bulkhead without filter) the pit-run gravel used for backfill was apparently coarse enough to prevent rapid erosion of the material from behind the tires (Fig. 2-346). Devices 3aW and 3bE (used-tire bulkheads with cloth and gravel filters, respectively) were not damaged except for a 1-foot-deep by 2-foot-wide scour area (topsoil) behind 3bE (Fig. 2-347). Most of the gravel was washed from the tires all along device 3, and a tire settlement of 6 to 10 inches occurred (probably because the tires flattened as the gravel washed out).

Device 4 (timber bulkhead) was damaged extensively; device 4a (horizontal logs) was essentially destroyed. All backfill material was washed out, and a number of the horizontal logs were removed (Figs. 2-348 and 2-349). The western part of device 4a had no filter, and the fine (silty sand) backfill material was easily washed through the logs. Apparently the gravel filter material in the eastern part of device 4a was undersized and was washed through voids between the logs. Once a breach in the filter had been established, the backfill material (pit-run gravel) escaped. A small void under the separation bulkhead between 4aW and 4bE allowed about 20 cubic yards of material to escape from behind 4bE (treated plank bulkhead with cloth filter) (Fig. 2-350). In device 4bW (treated planks without filter material), all of the silty sand backfill (about 110 cubic yards) was eroded (Fig. 2-351). Much of the toe protection was displaced along device 4b, exposing a gap of 1 to 2 inches below the bottom plank, which allowed the backfill to escape. The destruction of devices 4aW and 4bW exposed the separation bulkheads on each end of 4bE.

Most of the water from the U.S. Navy housing development, which overlooks the project, drains off behind devices 2bE, 3bW, 4aE and 4aW. Possibly, the runoff created a preliminary pathway through the devices, and when wave overtopping occurred during the 13 February 1979 storm, the erosion of backfill material was hastened by the drainage route previously formed by the surface runoff.

(d) <u>Vegetation Plantings</u>. Success of plantings at this site was low due to extreme weather and the damage to the structures below the plantings (Figs. 2-352 and 2-353). The severe storm in February 1979 removed the backfilled material in areas behind devices 3b, 4a, and 4b, and these could not be sampled. The area behind device 2a was also not sampled as repair to the structure deposited a layer of gravel cobble over the plantings. It was seeded with large variety tall wheatgrass in September 1979, but sampling was not performed until 1980.

Two months after planting (May 1979) about 20 percent of the shrub plantings remained, and of these plantings, few showed signs of growth or

Figure 2-344. Backfill material washed out from alongside the separation bulkhead of devices 2a and 2b, Oak Harbor, 14 February 1979.

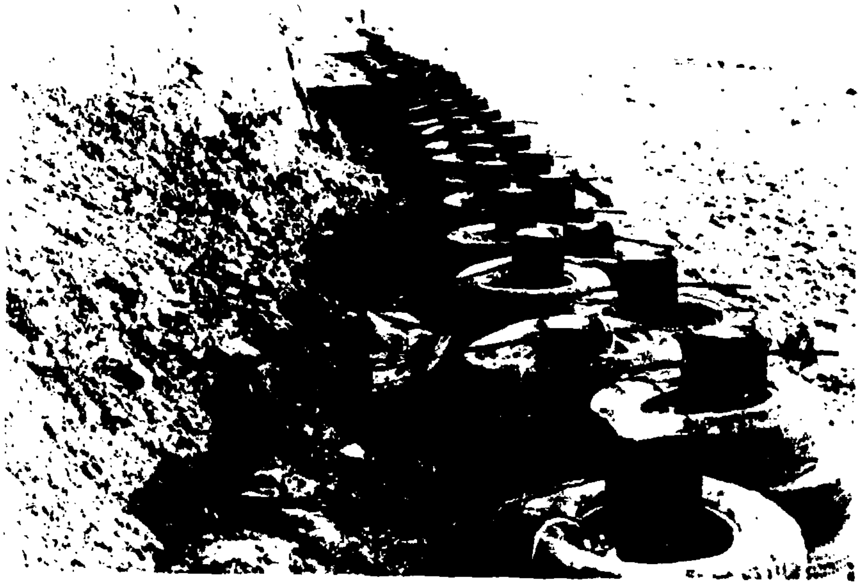


Figure 2-345. Erosion of the sand backfill from behind device 3bW, Oak Harbor, 14 February 1979.

Figure 2-346. Device 3aE with a pit-run gravel backfill following the storm, Oak Harbor, 14 February 1979.

Figure 2-347. Scour trench behind device 3bE (with a gravel filter), Oak Harbor, 14 February 1979.

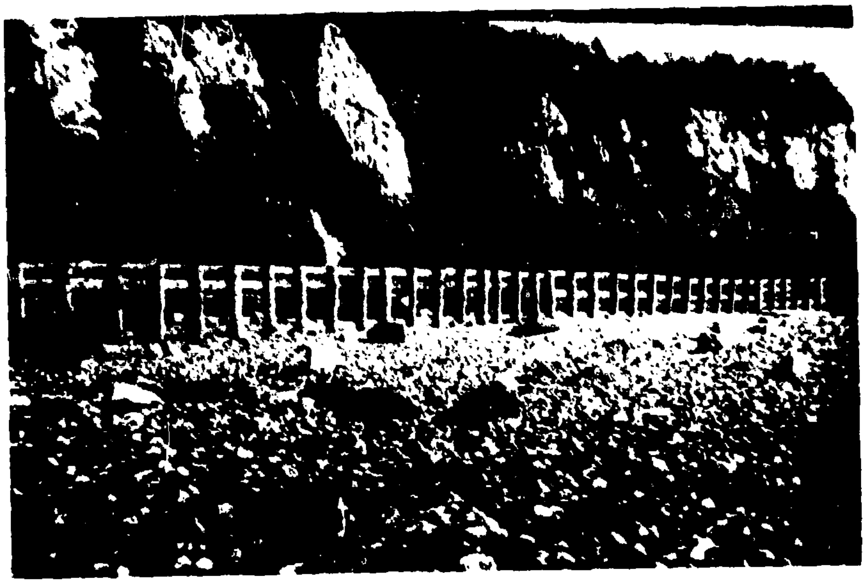


Figure 2-348. Extensive damage to device 4a, Oak Harbor, 14 February 1978.

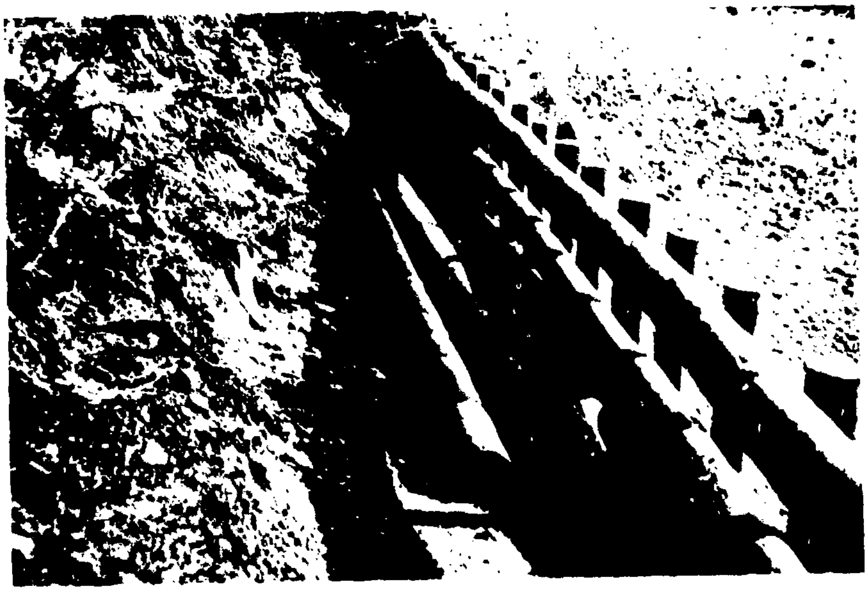


Figure 2-349. Backfill completely washed out from behind device 4a, Oak Harbor, 14 February 1978.

Figure 2-350. Loss of backfill from behind device 4bE under the separation bulkhead, Oak Harbor, 14 February 1979.

Figure 2-351. Complete loss of backfill from behind device 4bW, Oak Harbor, 14 February 1979.

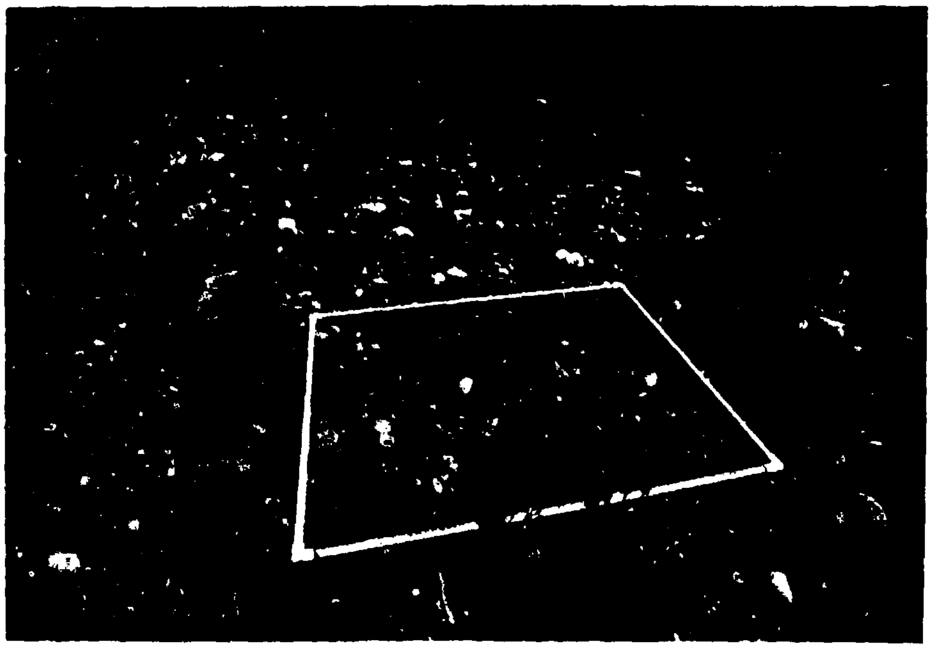


Figure 2-352. Destroyed vegetation area behind the gabion revetment, Oak Harbor.

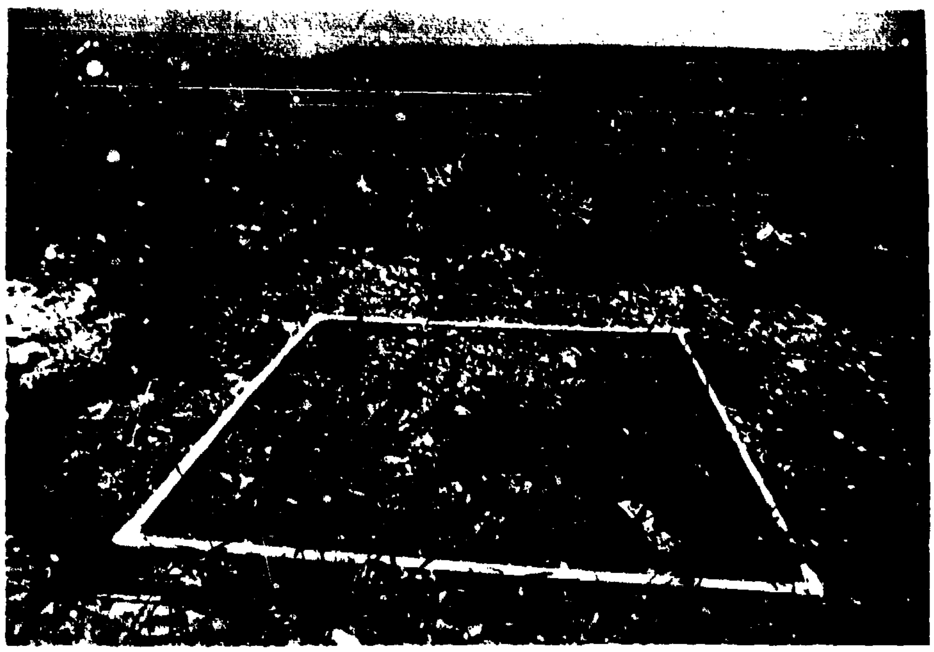


Figure 2-353. Vegetation plantings behind the used-tire bulkhead, Oak Harbor.

establishment. The wheatgrass and red fescue were surviving and flowering. By September 1979 only a few scattered shrubs remained, with mortality more than 90 percent. A few individual snowberry plants had become established by September and the intermediate wheatgrass and creeping red fescue were reseeding naturally. The plant that dominated during most of the summer was lambs quarter (*Chenopodium album*), a weedy species.

In May 1980, about 1 year after planting, a small percentage of salal, kinnikinik, ocean spray, snowberry, and willows remained behind the sand-cement-bag revetments. The grass mixture of wheatgrass, fescue, and trefoils also persisted. Some wheatgrass, trefoil, and red fescue remained behind the gabion-mat revetment, but all shrubs had died. The area behind the tire and post bulkhead showed limited shrub survival, mostly willow and snowberry. The wheatgrass mixture was still present in this area. The area behind the treated timber bulkhead had a few stray grasses, but was otherwise nonvegetated.

(5) Reconstruction. As a result of the damage caused by the two winter storms, the District Engineer made several recommendations. Most of the damage to the shore protection devices resulted from either a filter failure (4aE) or the absence of a filter (3bW, 4aW, and 4bW). In both cases the failures illustrated the importance of proper filter design. No repair of devices 3bW, 4aE, 4aW, or 4bW was undertaken as their condition clearly demonstrated the result of improper or inadequate attention to filter design. However, damage in two areas was caused by leakage of backfill through the separation bulkheads, one between devices 2a and 2b, and the other between devices 4aW and 4bE. Repair of the gabion-mat revetment required removing approximately four gabions on either side of the 2a-2b bulkhead (Figs. 2-354), providing drainage for surface runoff from the bluff (Figs. 2-355 and 2-356), placing an adequate gravel or cloth filter beside the bulkhead, placing 40 cubic yards of backfill, and replacing and refilling the gabions (Fig. 2-357).

Repair of the eastern part of the treated timber bulkhead (4bE) required placing filter cloth on the inside of the eastern end of the bulkhead (Fig. 2-358), replacing 20 cubic yards of backfill material (Fig. 2-359), and replacing toe rock along the base of the bulkhead (Fig. 2-360). The structural repairs were completed in June 1979. As of May 1980 no significant changes had occurred to any of the structures since the reconstruction phase.

(6) Analysis of Structural Devices.

(a) Sandbag Revetment. Of the two variations, the burlap bags nested better, leaving no visible voids through which backfill could escape. The test indicated that no filter was necessary. The cost was also one-third less than that of the dry-mix paper bags, even though 45 percent more labor was required. However, at a remote site that precluded access by a ready-mix truck, the paper-bag variation might be more feasible.

By the spring of 1980 some undermining of device 1aE (sand-cement burlap bags with gravel filter) was evident. Figure 3-45 in Section III shows a series of profiles through device 1a. Profiles of July 1978 and April 1980 show little change; however, the erosion at the toe of the structure can be seen. In an area of progressive long-term shore recession, the toe of the revetment would have to be entrenched deeply enough to preclude any possibility of undermining,

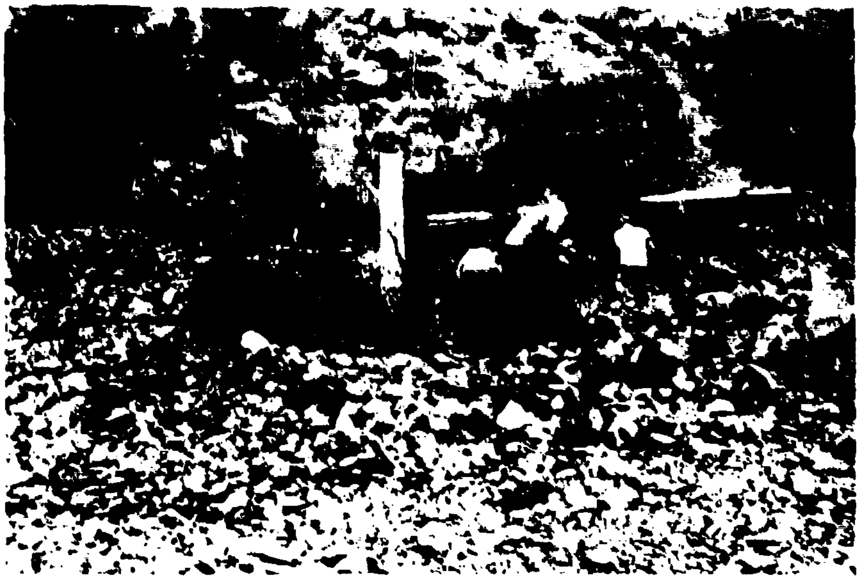


Figure 2-354. Removal of gabions from both sides of the 2a-2b bulkhead, Oak Harbor, 28 June 1979.

Figure 2-355. Providing drainage for surface runoff from bluff, Oak Harbor, 28 June 1979.

Figure 2-356. Rock filter and backfill covering drainage channel from bluff, Oak Harbor, 28 June 1979.

Figure 2-357. Repairs completed on gabion revetment, Oak Harbor, 28 June 1979.

Figure 2-358. Securing the separation bulkhead, Oak Harbor, 28 June 1979.

Figure 2-359. Replacing the backfill material, Oak Harbor, 28 June 1979.

Figure 2-360. Replacement of toe rock fronting device 4bE, Oak Harbor, 28 June 1979.

or larger toe stone would have to be used. Nevertheless, the demonstration proved that the burlap-bag variation is an excellent low-cost revetment under conditions similar to those at Oak Harbor.

(b) Gabion Revetment. Except for the loss of a few stones from some of the baskets and settlement of the top in some areas due to washing of backfill through the revetment (probably because no filter was used at the top edge), the gabions performed about as well as the sandbags. However, more debris was thrown over the gabions, probably because of the flatter slope (1 on 1.5 versus 1 on 1). This indicates a need for a higher crest elevation for the gabion revetment to prevent massive overtopping. The washout of backfill at the separation bulkhead between devices 2a and 2b was not attributable to gabion failure, as both filter types were performing well elsewhere. Discontinuity at the bulkhead may have allowed wave action to initiate washout of backfill. However, loss of backfill from behind device 2bW, which had no filter, indicated a need for some type of filter under the gabion revetment. The difference in cost between the two sections is due primarily to the greater number of loader hours charged to device 2a (63 percent per foot more), the reason for which is attributable to the double-loading technique used in device 2b. If this technique had been used for device 2a, the cost per foot would have been $51 ($41 per foot excluding labor). Structural performance could be improved by filling the baskets with larger stone to preclude leakage through the wire mesh. In an area of progressive long-term shore recession, the flexibility of the gabions should improve their performance over that of the sand-cement bags, as they would conform to toe undercutting. However, monitoring did not continue long enough to determine the validity of this assumption or to detect any deterioration of the wire mesh due to debris impacts, vandalism, or corrosion. The profiles through device 2b are shown in Figure 3-.. (see Sec. III). Comparison of the profiles of July 1978 and April 1980 show that about 2 feet of backfill, including most of the topsoil, was washed from behind the gabions.

(c) Tire and Post Bulkhead. As constructed, this innovative bulkhead performed quite well functionally. The major structural failure which occurred in device 3bW was due to the fine material used for backfill and lack of a filter. Device 3aE also had no filter but was backfilled with gravel only, which acted as a filter. Another lesson learned from the test was the vulnerability of the gravel fill in the tires to wash out. It would appear that this weakness could easily be corrected by filling the tires with a weak, vibrated-in concrete. Use of concrete for tire fill would obviate the need for the safety ties. All of the holddown bars through the tops of the posts remained in place and probably prevented the tires from being lifted off by wave action after the gravel washed out. The need for tiebacks with this system was not demonstrated by performance.

None of the posts were displaced seaward. However, it is questionable whether the anchor logs were far enough behind the bulkhead to secure it effectively in case of bluff slumping. Had slumping occurred, the weight of the surcharge against the bulkhead might have rotated the posts seaward, pulling the anchor logs through the loose backfill. It would have been interesting to see what would have happened if the tiebacks had been omitted in a section where there was no loss of backfill. Elimination of the tiebacks would effect a savings in both labor and materials costs, and it would simplify placement of filter cloth behind the tires.

Where filter cloth was used, it appeared that it was stretched too tightly. The cloth ballooned into the voids between the back row of tires without contacting the front row (Fig. 2-361). Although no failures due to bursting pressure of the backfill were noted, the potential for such failures could be eliminated by allowing the cloth to conform more closely to the irregularities of the tire surfaces. Considering the good functional performance of the tire and post bulkhead where loss of fill materials did not occur, the low cost, and the indications of longevity of the major components in a marine environment, should warrant serious consideration for its use at sites similar to those at Oak Harbor. Profiles through device 3a are shown in Figure 3-18 (see Sec. III). Comparison of the profiles of July 1978 and April 1980 show that about 2 feet of backfill, including most of the topsoil, was washed from behind the tire and post bulkhead.

(d) Timber Bulkhead. The failure of the bulkhead constructed with untreated logs was due primarily to the lack of an adequate filter. The resulting loss of backfill material assured the rapid destruction of the bouyant, free-standing structure under heavy wave attack. The cracks between the horizontal logs were too great even to retain the washed gravel filter in device 4aE. If a filter cloth had been used in this section, the results might have been different. By extending the bottom skirt of the cloth well back under the backfill, loss of backfill by piping under the bottom logs might have been prevented. The lack of tiebacks in this section did not contribute to its failure and would have made the installation of filter cloth

Figure 2-361. Ballooned filter cloth of device 3aW, Oak Harbor, 3 January 1979.

quite simple. Where logs are relatively cheap, this scheme appears to merit further consideration. However, the lack of preservative treatment could lead to early deterioration of such a structure.

The treated-timber bulkhead faired much better, especially in device 4bE where filter cloth was used. If the separation bulkhead between device 4bE and 4aW had not leaked, this section might have weathered the 13 February 1979 storm with little damage. As in the case of the tire and post bulkhead, it would have been interesting to see what would have happened if the tiebacks had been omitted. The 4.5-foot embedment of the posts may have been deep enough to develop enough strength of the posts against overturning by cantilever bending forces exerted by the backfill, as some ties were slightly loose and did not appear to tighten as a result of backfill pressure. Again, lack of tiebacks would have eliminated some labor and material costs and simplified installation of filter cloth. The latter is good assurance against loss of backfill through cracks or under the bottom planks (if the cloth is extended back under the backfill). Preservative treatment of timber for use in the marine environment is necessary to assure longevity of more than 10 years. Where tiebacks are not required, a timber bulkhead can be constructed much closer to the bluff than a revetment, which could be an advantage in some cases. Profiles through device 4a are shown in Figure 3-19 (see Sec. III). Although there was some loss of backfill, it is not evident in the April 1980 profile.

(e) Vegetation Plantings. Establishment of shrub vegetation appears to be difficult at this site. Survival was poor in most areas within

1 month of planting. However, since some of the willows and snowberry did become established in the area behind the sandbag revetments and were able to persist through the winter, it appears that these shrubs might be successful if replanted to maintain the original planting configuration. Failure of the structures, especially the gabion mat and the tire and timber bulkhead, were responsible for much mortality in plantings. Sloughing off of bluff materials with the concommitant burying of the plantings over the winter (1979) also prevented many plants from reaching the light, and therefore surviving.

Plantings of intermediate wheatgrass, creeping red fescue, and clover appeared successful at this site. The establishment of these plants over a period of time could provide a firmer surface and perhaps a better area for growth of shrub seedlings. Grasses are more resistant to burying, and they survive disturbance much better than the shrub species.

For planting of shrub species to be successful here, considerable protection in the form of both structures and a stable ground surface appears to be necessary. A layer of topsoil is also a necessary prerequisite for growth. In addition, replanting and maintenance would probably be required during the first several years to ensure good establishment. The roots of these shrubby species, if well established, would provide considerable soil stability and would probably halt shoreline erosion. However, without good structural protection, and with continued sloughing off of bluff material above the plantings, success would be limited.

It is apparent from observations of plots recorded during the summer of 1979 that certain weedy herbaceous species, such as lambs quarter and gum weed, are better suited to summer conditions than were the planted species. These plants, however, are annuals with a short lifespan and shallow roots. They could not be expected to stabilize the soil or prevent erosional loss in the long run. These species along with the grasses and legumes seeded on the site could provide a temporary surface cover, which might aid shrub establishment.

d. Sunnyside Beach, Washington.

(1) Site Description.

(a) Geographical Setting. The Sunnyside Beach monitoring site is at the south end of Puget Sound within the town of Steilacoom, Washington. The site is approximately 0.5 mile south of Chambers Creek and 5 miles southwest of Tacoma, Washington (Fig. 2-362). The beach, which is 100 feet wide at MLLW, has developed along a 5-acre shoreline fill 1,000 feet long and 180 feet wide near its center, forming a marked bayward bulge in the shoreline. It extends generally in a north-south direction. A municipal sewage treatment plant is located on a 1-acre tract in the southern part of the fill behind the beach. A 4-acre park, developed in the northern part of the fill, is protected from wave erosion by a timber bulkhead 550 feet long that retains the vertical bank fronting the park area. Its top elevation is about +17 feet MLLW.

(b) Geomorphology, Soils, and Vegetation. The shoreline fill fronted by Sunnyside Beach was constructed in the early 1900's with waste sand from a now abandoned gravel pit just east of the site. Erosion of this fill formed the beach initially, but to prevent further loss of the filled land, the town of Steilacoom in 1967 constructed the 550-foot bulkhead along the

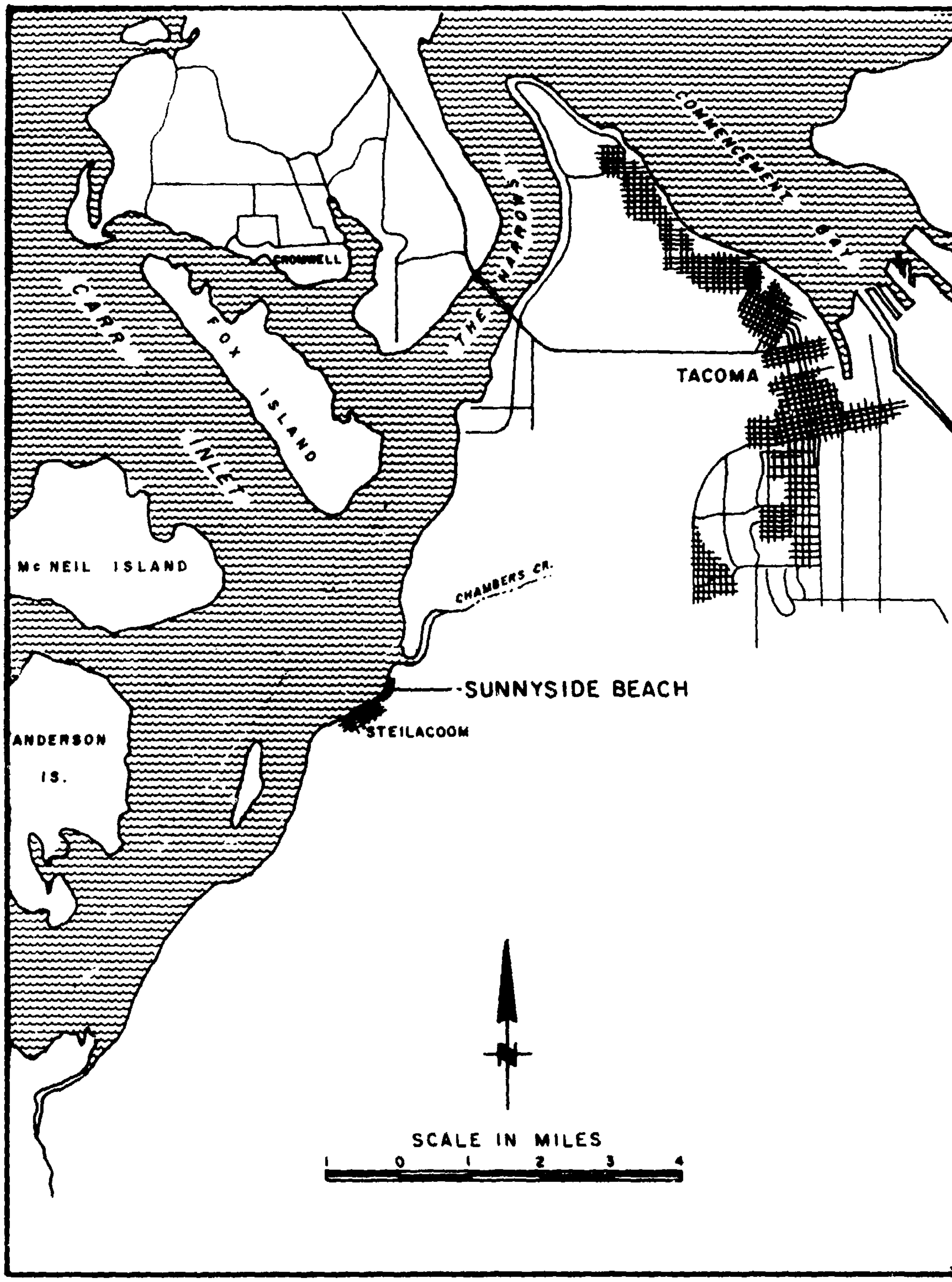


Figure 2-362. Location map of Sunnyside Beach, Washington, monitoring site.

eroding bluff fronting the park area with log piles driven side-by-side and backfilled with cobbles and gravel. Behind the bulkhead is a grassy slope rising about 10 feet to the level of the plateau on which the park facilities are located. The park area is open and grassy, with a few stands of shade trees.

(c) Waves, Tides, and Littoral Drift. The mean tide level at Sunnyside Beach is 7.5 feet MLLW, the diurnal range is 13.1 feet, and the mean range is 9.3 feet. Under normal conditions, the wave height averages 2.1 feet with a period of about 3 seconds. During storm conditions, waves reach heights of 5.0 feet, with periods of about 4.5 seconds. Some northward longshore transport has been observed at the site, but the littoral material moves primarily seaward. In a 1974 study, the Corps of Engineers estimated that offshore transport approached 900 cubic yards per year over the 1,000 feet of beach frontage at the project site. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 2.0 feet. The wave climate is classified as mild.

(d) The Problem. The project beach was in an active state of erosion before 1975. The average shoreline recession was about 1 to 2 feet per year. Exposure to wind waves had damaged the bulkhead to the extent that parklands behind it were beginning to erode. Complete failure of the bulkhead was expected within 5 to 10 years. The unprotected bank behind the beach near the sewage treatment plant was receding about 3 feet each year. Continued erosion of the uplands and the beach would reduce the potential of the beach for recreational use, destroy the town park, and render the sewage treatment plant unusable.

(2) Monitoring Project. In 1974 local interests asked the Corps of Engineers to investigate the erosion problem at Sunnyside Beach. The Seattle District Engineer determined that the lowest cost solution to the erosion problem, and the one with the least environmental impact, was periodic beach nourishment. In December 1975, the town of Steilacoom completed placement of 18,000 cubic yards of beach fill at the Sunnyside site, and in July 1978, nourished the beach with an additional 4,200 cubic yards of waste sand. Longshore transport has extended the beach several hundred feet northward of the park (Fig. 2-363). Monitoring of the beach fill under the Shoreline Erosion Control Demonstration Program was approved by the Chief of Engineers, and the first survey was made in January 1979.

(3) Performance. After 1 year of monitoring, the heavily used recreation beach continues to protect the park and sewage facilities. The beach has also prevented further deterioration of the timber bulkhead protecting the park area (Figs. 2-364 and 2-365). Volume calculations of erosion and accretion show that both trends are present at the site (Table 2-94). Erosion has occurred primarily from station 0+00 to 2+00 and from station 6+00 to 12+50. Major accretion areas range from station 2+00 to 6+00 and from station 12+50 to 19+00. The net accretion of nearly 4,000 cubic yards between the 1979 and 1980 surveys is an anomaly, in view of the long-term record of continuous erosion at this site. A longer monitoring period is needed to develop a better account of littoral movement in this area. Profiles of the eroding sections show a minor

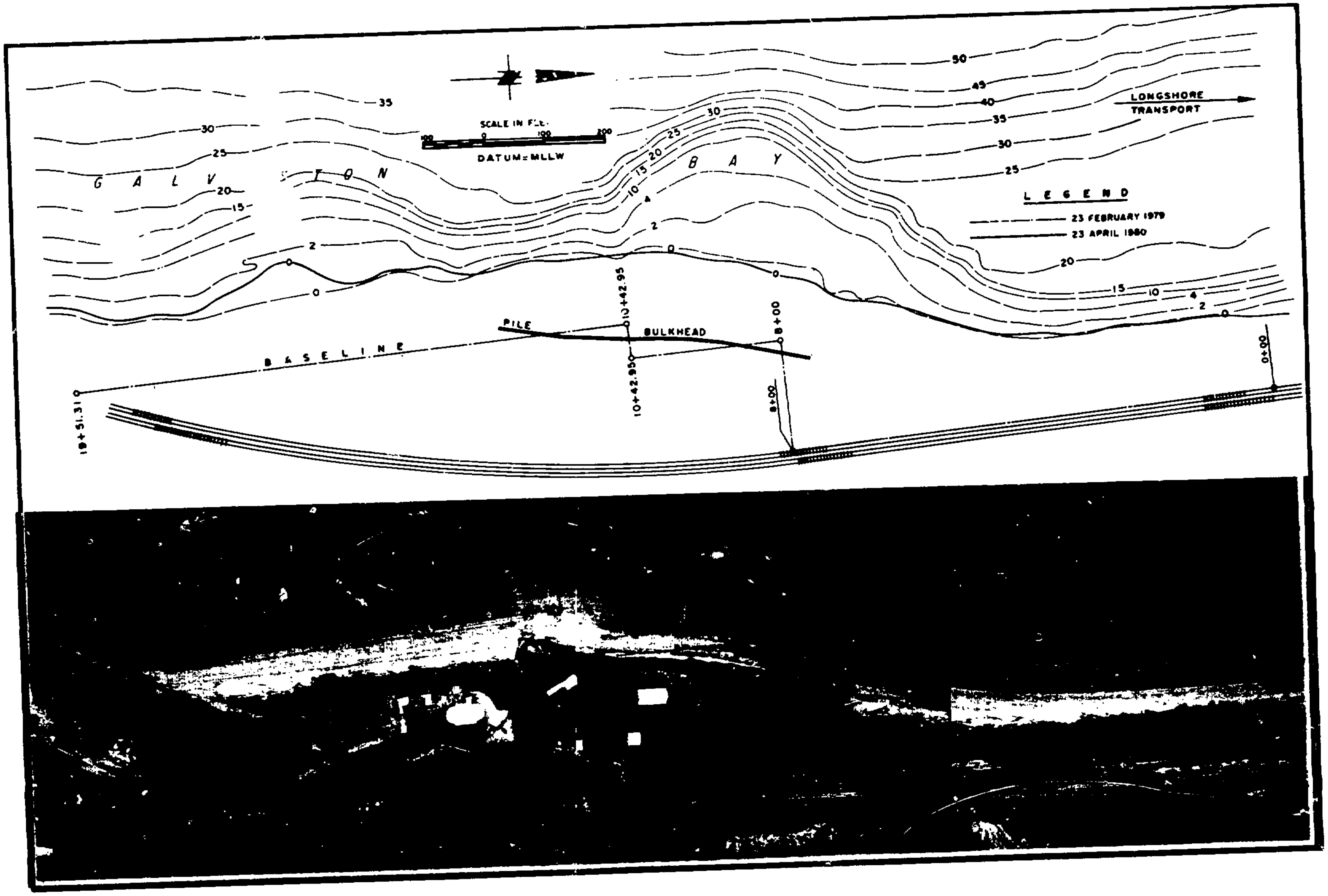


Figure 2-363. General plan and photo of Sunnyside Beach, Washington, monitoring site.

Figure 2-364. Sunnyside Beach from the park center looking north, 29 May 1980.

Figure 2-365. Sunnyside Beach from the park center looking south, 29 May 1980.

Table 2-94. Volumetric analysis of beach profiles at Sunnyside Beach (1 March 1979 to 23 April 1980).

Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
0+00	986.2	127.8	-858.4
0+50	808.8	120.8	-688.0
1+00	559.4	301.7	-257.7
1+50	420.7	328.7	-92.0
2+00	320.4	658.9	338.5
2+50	144.0	741.4	597.3
3+00	147.3	436.7	289.3
3+50	109.9	579.5	469.6
4+00	107.3	527.5	420.2
4+50	111.5	382.9	271.4
5+00	90.5	584.2	493.7
5+50	298.0	471.4	173.4
6+00	1,021.8	191.2	-830.5
6+50	966.8	216.2	-750.7
7+00	619.6	103.2	-516.4
7+50	633.1	43.7	-589.4
8+00	311.4	107.7	-203.6
8+50	239.0	132.5	-106.4
9+00	234.4	82.5	-151.9
9+50	251.9	57.2	-194.7
10+00	167.4	595.0	427.7
10+50	206.7	602.8	396.1
11+00	560.8	38.9	-522.0
11+50	433.0	40.3	-392.7
12+00	229.9	51.9	-178.0
12+50	182.0	319.7	137.7
13+00	53.0	582.4	529.4
13+50	45.9	359.1	313.2
14+00	151.3	154.8	3.5
14+50	183.2	210.5	27.3
15+00	139.7	271.4	131.8
15+50	126.3	487.3	361.0
16+00	58.5	726.7	668.2
16+50	43.1	828.1	785.0
17+00	58.3	888.3	830.0
17+50	51.2	855.2	803.9
18+00	38.5	804.4	765.9
18+50	133.7	1,061.1	927.4
19+00			
Totals	11,244.44	15,073.7	3,829.3

amount of accretion between March and April 1979, but then an overall erosion up to April 1980. The profiles at station 16+00 show continual accretion (Fig. 2-366), and station 4+00 shows a fluctuation between accretion and erosion (Fig. 2-367).

(4) Analysis. The primary movement of beach fill along this 1,000 feet of shoreline appears to be onshore-offshore. Profiles indicate that the fluctuations between erosion and accretion trends at a single station can change as rapidly as 1 month. Apparently, the eroded material is carried offshore, redistributed, and is later carried back onshore. An occasional northward longshore transport has also been observed at the site and is responsible for some of the sand redistribution. In any event, the rate of loss of imported fill at Sunnyside Beach is so low that the renourishment project has proved to be an effective low-cost method of shore stabilization.

e. Siuslaw River, Oregon.

(1) Site Description.

(a) Geographical Setting. This monitoring site is located in western Lane County, Oregon, along the southwest bank of the Siuslaw River estuary (Fig. 2-368). Beginning near Cottage Grove, Oregon, Siuslaw River's 790-square mile drainage basin extends westward about 50 miles to the Pacific Ocean. Terrain of the basin is quite rugged and heavily forested, mainly with various species of conifers. The river becomes tidal in its lower reaches, where it widens to an average width of about 1,000 feet. At the project site, it flows in a north-northwestward direction. Excellent sand beaches line the seacoast adjacent to the river mouth. The site is accessible by taking U.S. Highway 101 to the town of Florence and then along the south jetty road that runs parallel to the site.

(b) Geomorphology, Soils, and Vegetation. Rocks exposed along the Oregon coast consist primarily of sandstone, siltstone, and shales locally interspersed with marine basalts. The Siuslaw River drainage basin is mostly underlain by the Tyee Formation, which is composed of rhythmically bedded sandstones and mudstones. An outcrop of Tyee sandstone forms a rock nose on the south bank of the Siuslaw River about 1 mile upstream of the highway bridge at Florence. No bedrock is exposed adjacent to the channel. Depth of bedrock in the project area is probably at least 25 feet and possibly more than 95 feet below MLLW.

Alluvial deposits in the river flood plain and at the project site vary from fine to coarse grained, are moderately shallow to very deep, and are poorly, to excessively drained. Terrace soils and dunes are mostly medium to coarse textured, are vulnerable to wind erosion when vegetation cover is removed, and are moderately shallow to very deep. Vegetation at the site consists mainly of European beachgrass (*Ammophila arenaria*).

(c) Climate. The climate of the lower Siuslaw River Basin is characterized by mild, wet winters and warm, dry summers with small temperature variations. The area receives a normal average annual precipitation of 65 inches. Approximately 70 percent of the precipitation occurs in November through March and about 30 percent in December and January. Average

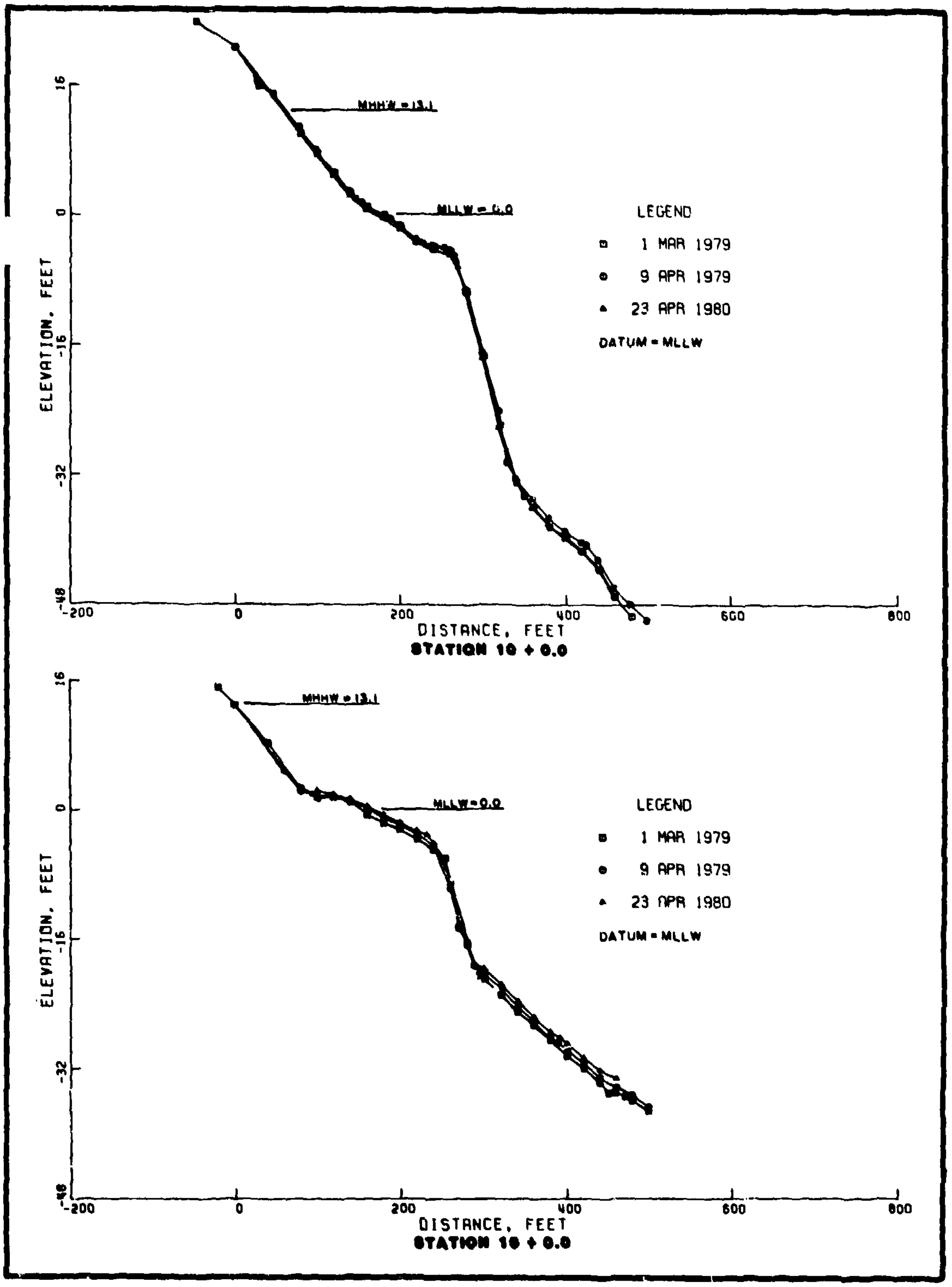


Figure 2-366. Typical profiles, Sunnyside Beach.

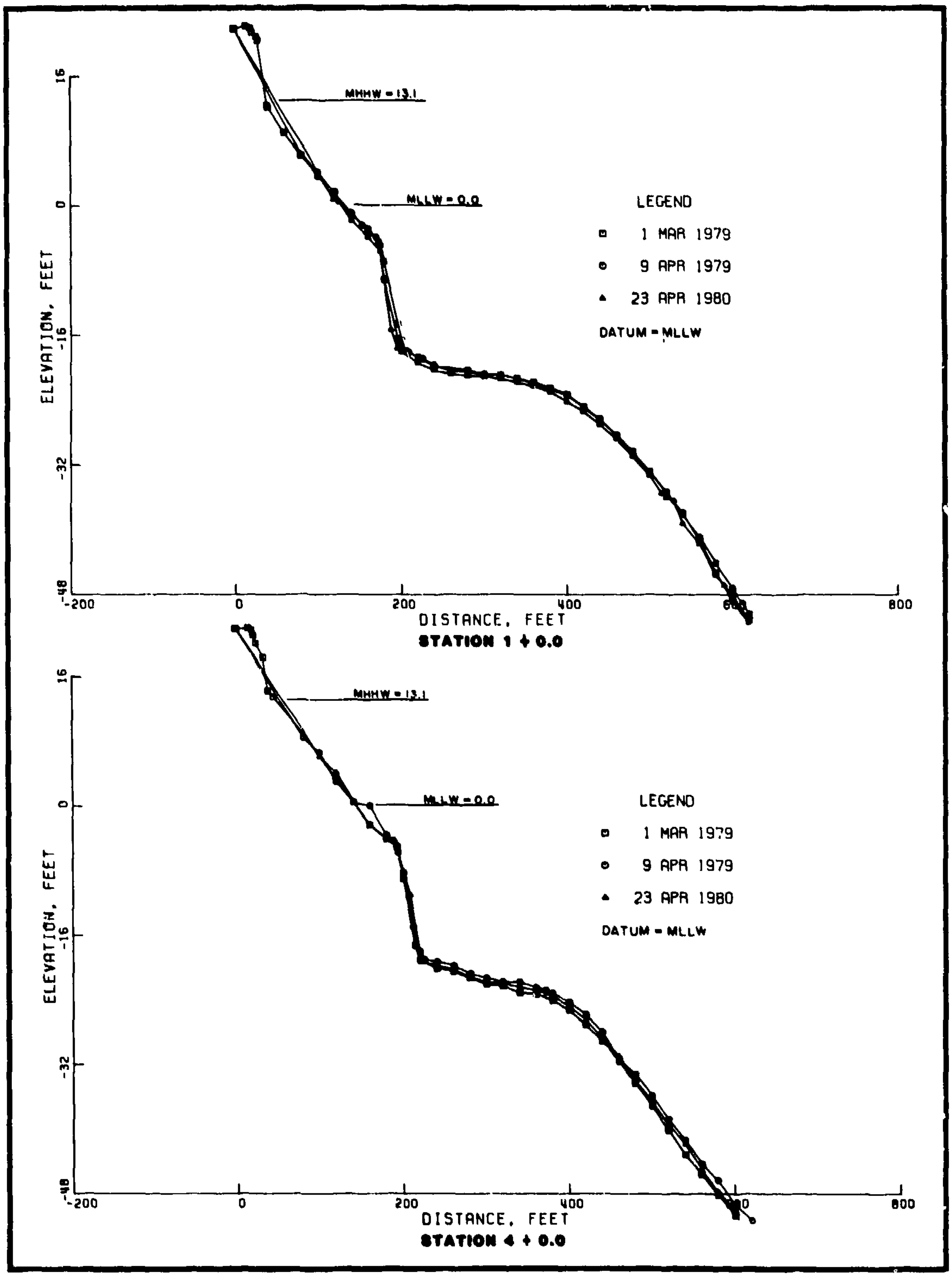


Figure 2-367. Typical profiles, Sunnyside Beach.

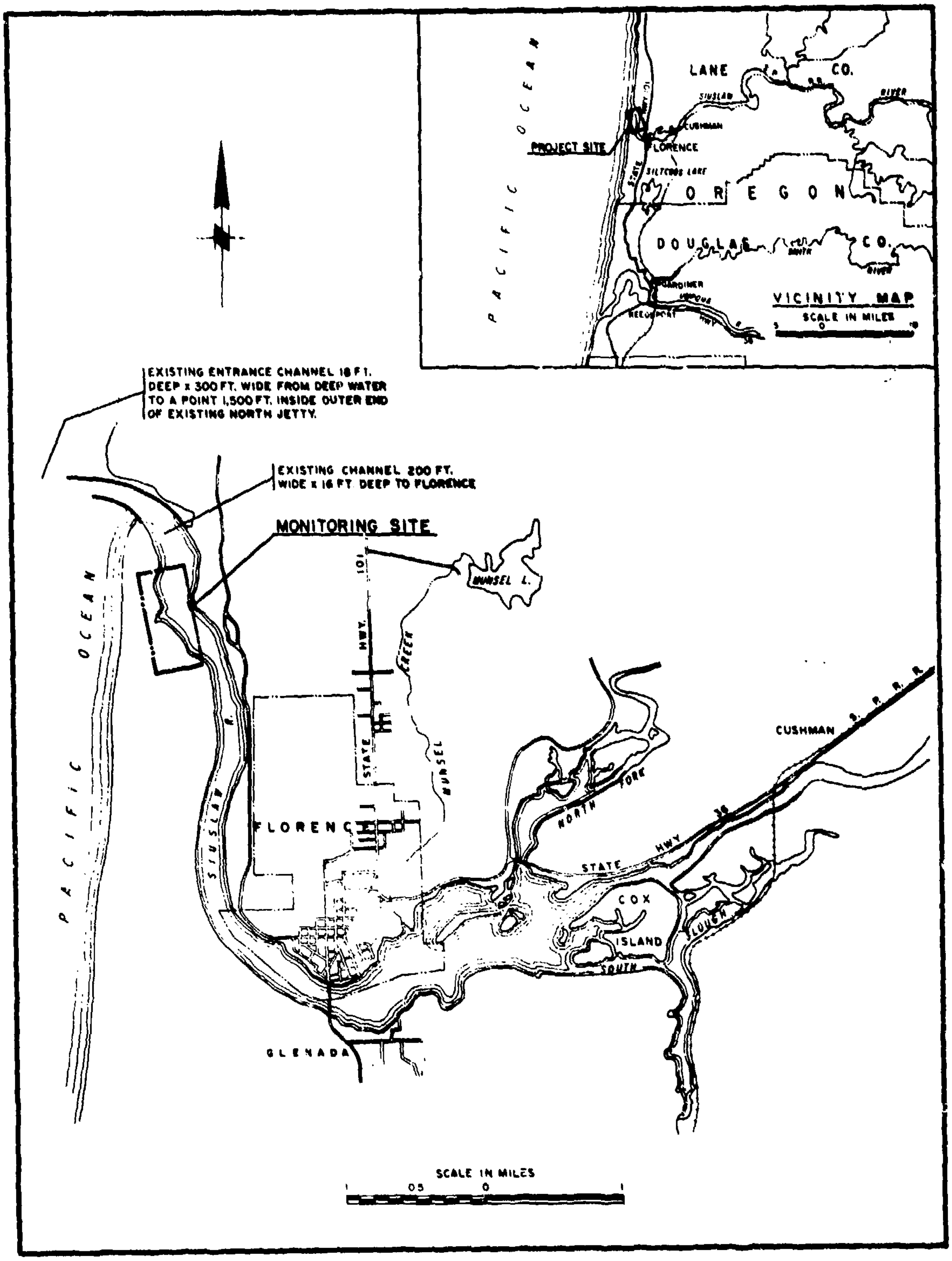


Figure 2-368. Location map of Siuslaw River, Oregon, monitoring site.

temperatures range from 43° Fahrenheit in January to 61° Fahrenheit in August, with a mean temperature of 52° Fahrenheit.

(d) Waves, Tides, and Longshore Transport. Pacific Ocean waves passing through the river mouth at high tide occasionally can reach the site through processes of reflection and refraction with estimated heights up to about 3 feet. Most of the time, waves at the site seldom exceed about 2 feet. Tides at the Siuslaw River mouth have the typical diurnal inequality of Pacific coast tides. The mean tidal range and the mean diurnal range at the entrance are 5.2 and 6.9 feet, respectively. At Florence, about 5.25 miles upstream, the mean ranges are 5.0 and 6.6 feet, respectively. Estimated extreme high water is 11.0 feet above MLLW, and the estimated extreme low is -3 feet. Freshwater discharge of Siuslaw River ranges from about 120 cubic feet per second during low stages to about 40,[illegible]0 cubic feet per second at average freshet stages. Because of the small tidal prism of the estuary, tidal flow alone is not sufficient to keep the entrance bar scoured to the authorized depth of the navigation channel. Although river floodflows often scour the bar to greater depths, in years of low precipitation, river freshets are inadequate to maintain navigable depths in the entrance. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 1.0 foot. Although the energy-flux analysis indicates a moderate net potential to transport littoral material upstream at this site, the action of river currents on riverbed and bank material far outweighs the effects of wave action.

(e) The Problem. Channel stabilization in the lower reaches of the Siuslaw River became a requirement in recent years as a result of the tendency of the river to shift, forming sharp bends that were difficult to navigate. Westward shifting in the project reach also threatened to destroy the south spit area, including the Oregon dunes, and its access road and parking facilities.

(2) Monitoring Project. To prevent further shifting of the channel or a possible breach of the spit and resultant loss of access to the area, a two-phase plan was implemented. Phase I consisted of constructing a 1,200-foot rock revetment on the eroded bank adjacent to the south jetty. The revetment (not monitored) replaced the part of the south jetty which was not repaired in 1962. Phase II consisted of constructing a longitudinal rock groin and four transverse rock groins (Fig. 2-369). Cross sections are shown in Figures 2-370 and 2-371. Construction was completed in August 1974, and monitoring of these groins by the Portland District Engineer under the demonstration program began in December 1978. The total length of the structures is 2,470 feet; the average cost was $147 per foot. Photos of each groin are shown in Figures 2-372 to 2-376. Statistical data on the project are given in Table 2-95.

(3) Performance. Throughout the monitoring period the rock groins and revetment remained structurally sound. No significant changes in structure cross sections or alinements were detected, and there was no apparent degradation of structural materials. The Siuslaw River

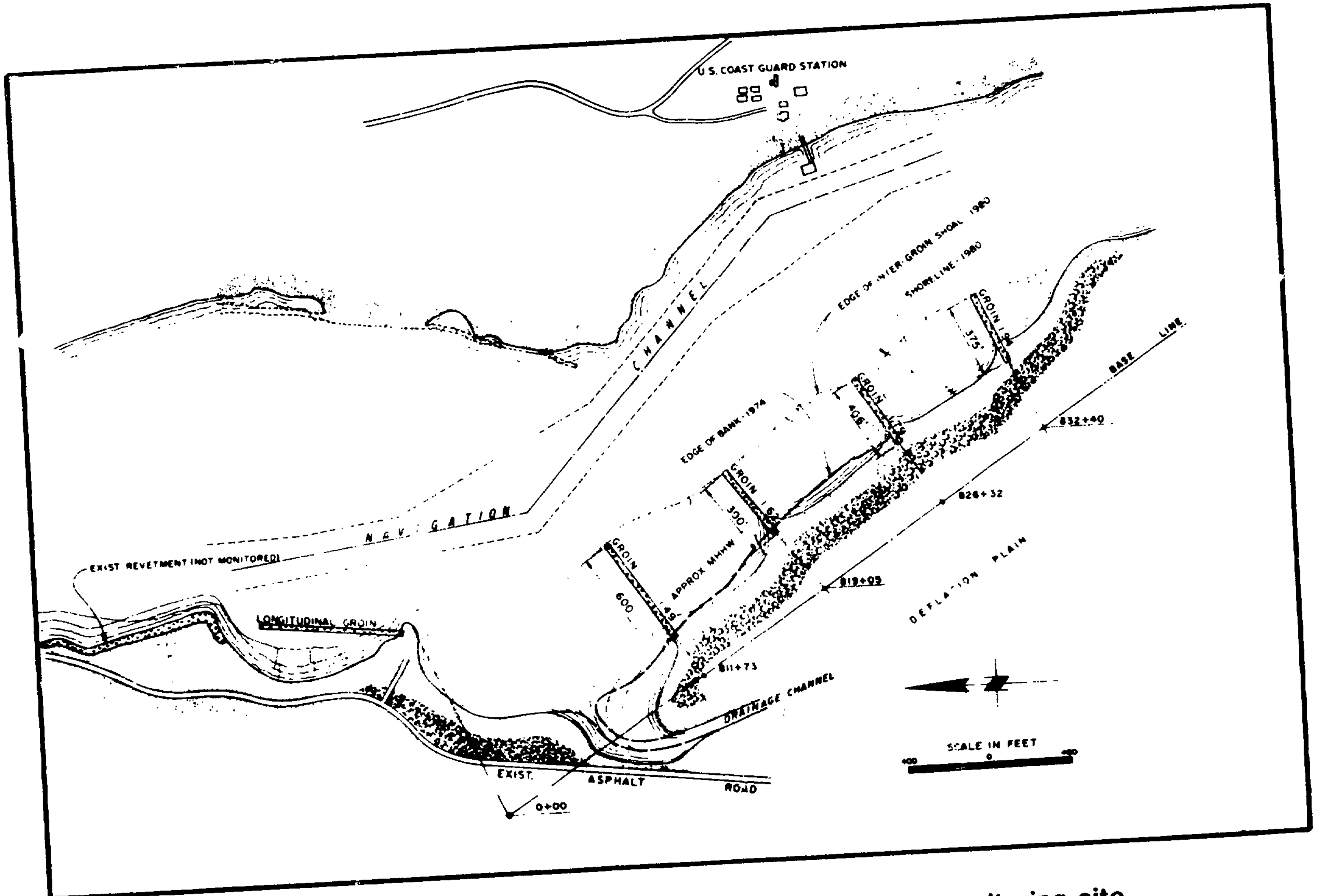


Figure 2-369. General plan of Siuslaw River, Oregon, monitoring site.

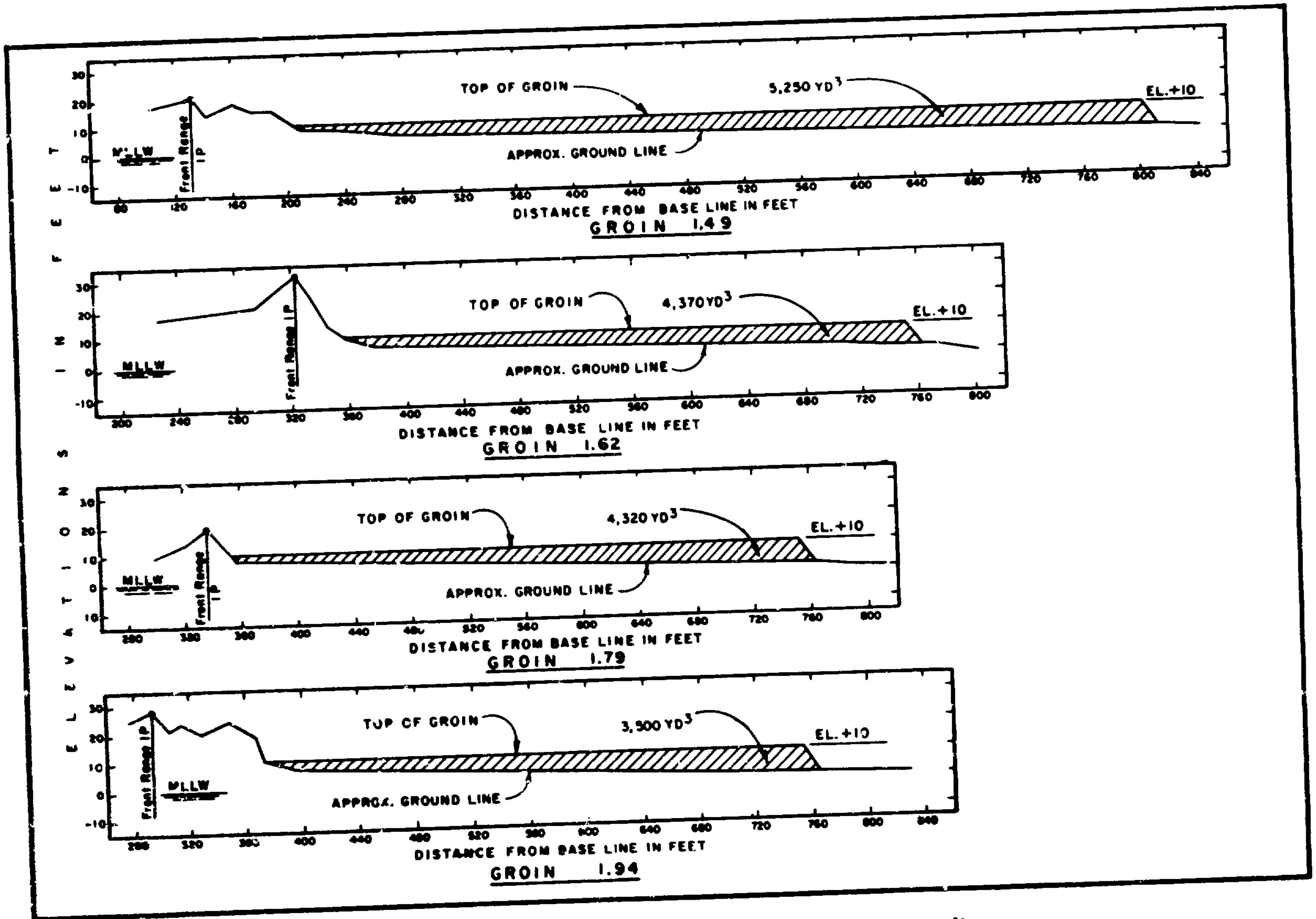


Figure 2-370. Profiles at Siuslaw River site.

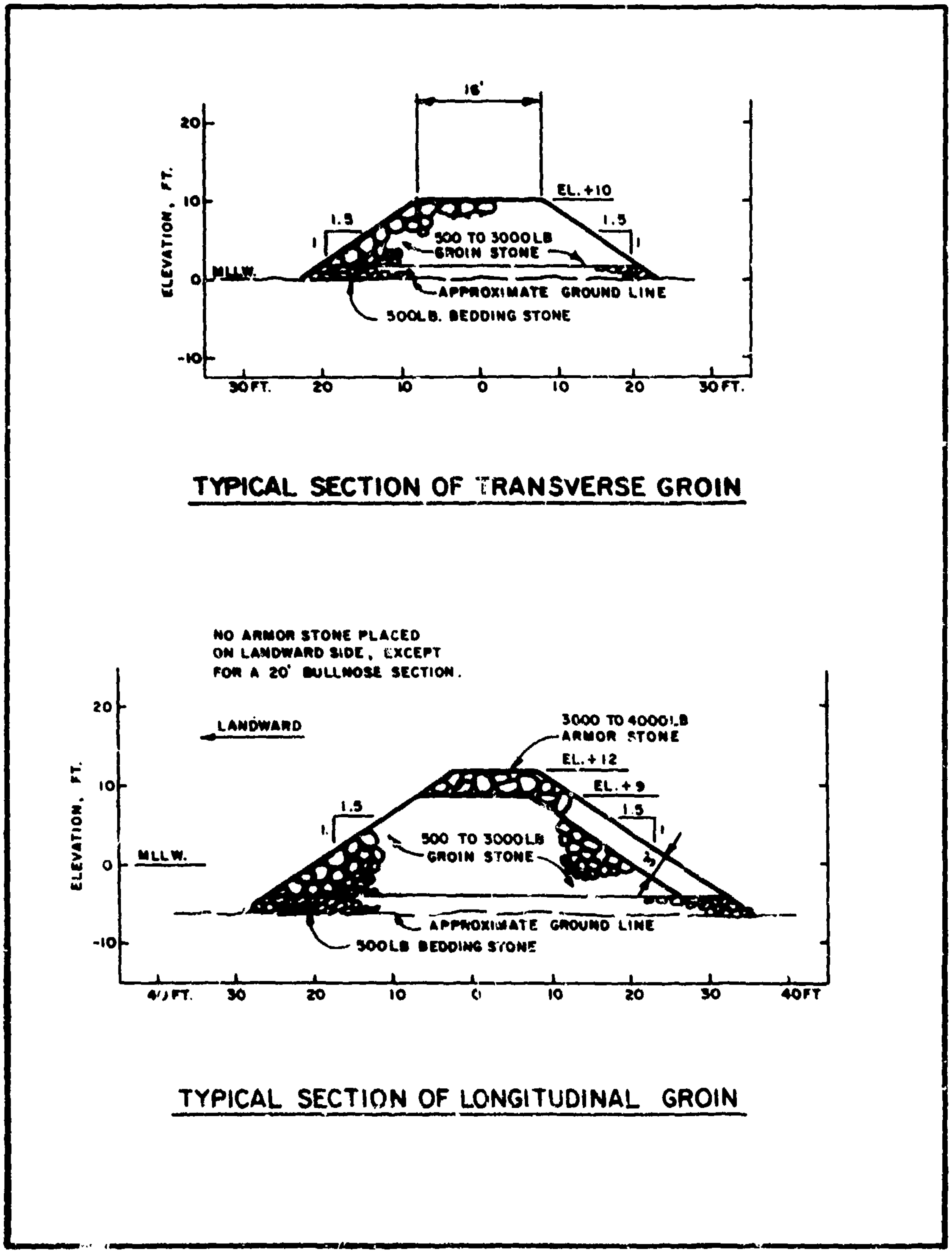


Figure 2-371. Typical sections of transverse and longitudinal groins at Siuslaw River, Oregon.

Figure 2-372. Transverse groin 1.79, Siuslaw River, Oregon, May 1979.

Figure 2-373. Transverse groin 1.94, Siuslaw River, Oregon, May 1979.

Figure 2-374. Transverse groin 1.49, Siuslaw River, Oregon, May 1979.

Figure 2-375. Transverse groin 1.62, Siuslaw River, Oregon, May 1979.

Figure 2-376. Longitudinal groin, Siuslaw River, Oregon, May 1979.

Table 2-95. Statistics and costs at Siuslaw River site.

Materials	Quantity (yd^3)	Weight (lb)	Cost
Bedding stone	6,700	500	$100,500
Groin stone (dumped)	30,000	500 to 3,000	229,500
Armor stone (placed on longitudinal only)	4,000	3,000 to 4,000	33,000
Total			$363,000

channel ceased its westward migration when the five groins were installed. Bank erosion had threatened to breach the spit and form a new outlet to the ocean somewhere along the shoreline where the revetment and longitudinal groin are now located. However, this area is now accreting. During the monitoring period, a large amount of sand accreted on both sides of the base of each transverse groin except at groin 1.49 where erosion occurred on the north side because the dune drainage channel had swung westward against the bank. Figures 2-377 to 2-383 show these trends as of December 1978.

Figure 2-377. Accretion and erosion trends along the shoreline of Siuslaw River, Oregon, 17 March 1979.

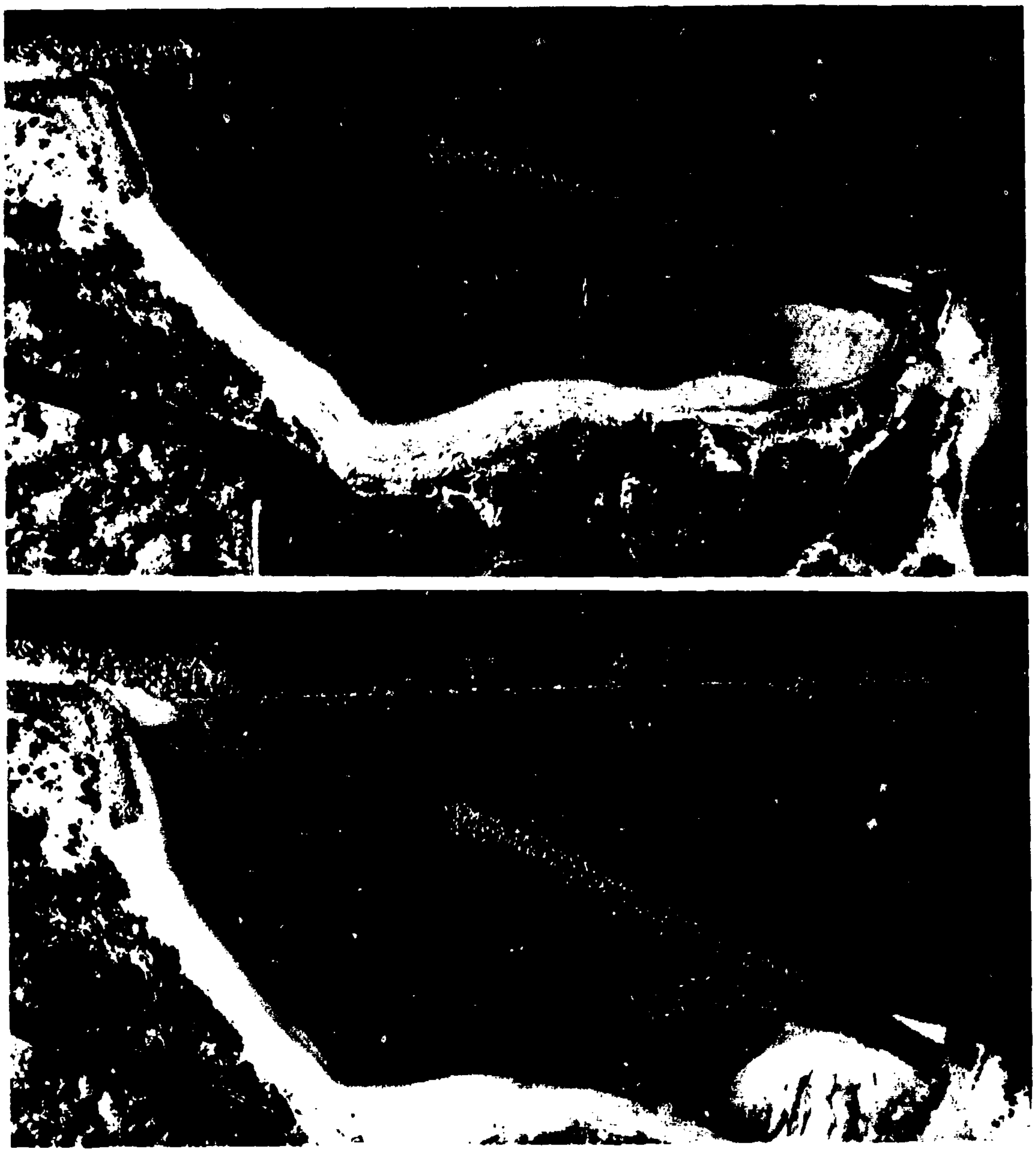


Figure 2-378. Comparison of the area in between the longitudinal groin and the rock revetment, Siuslaw River, Oregon (top photo, 5 January 1979; bottom photo, 11 December 1979).

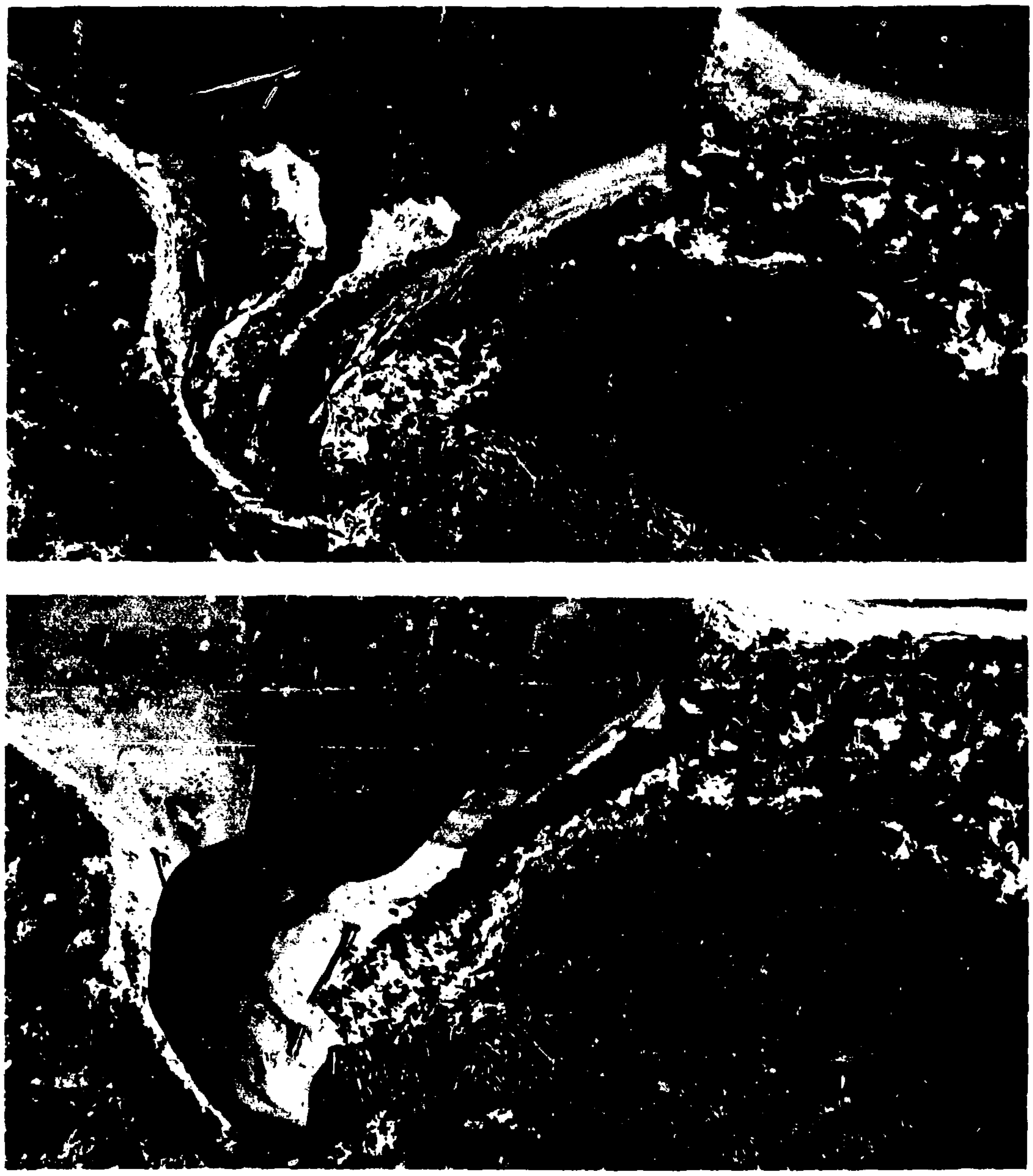


Figure 2-379. Comparison of the area between the transverse groin and longitudinal groin, Siuslaw River, Oregon, (top photo, 5 January 1979; bottom photo, 5 December 1979).

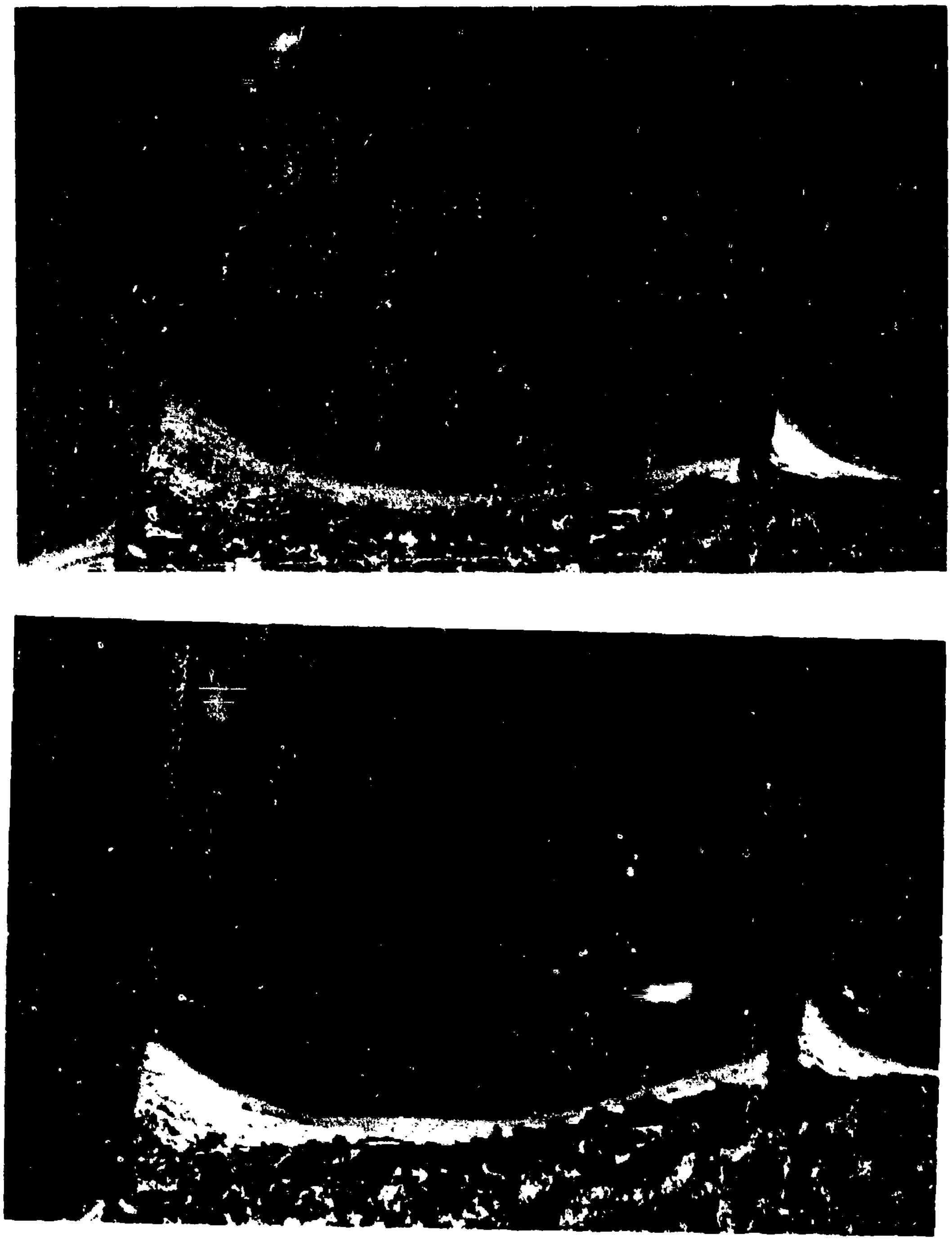


Figure 2-380. A comparison of transverse groins 1.49 (left) and 1.62 (right), Siuslaw River, Oregon, (top photo, 5 January 1979; bottom photo, 11 December 1979).

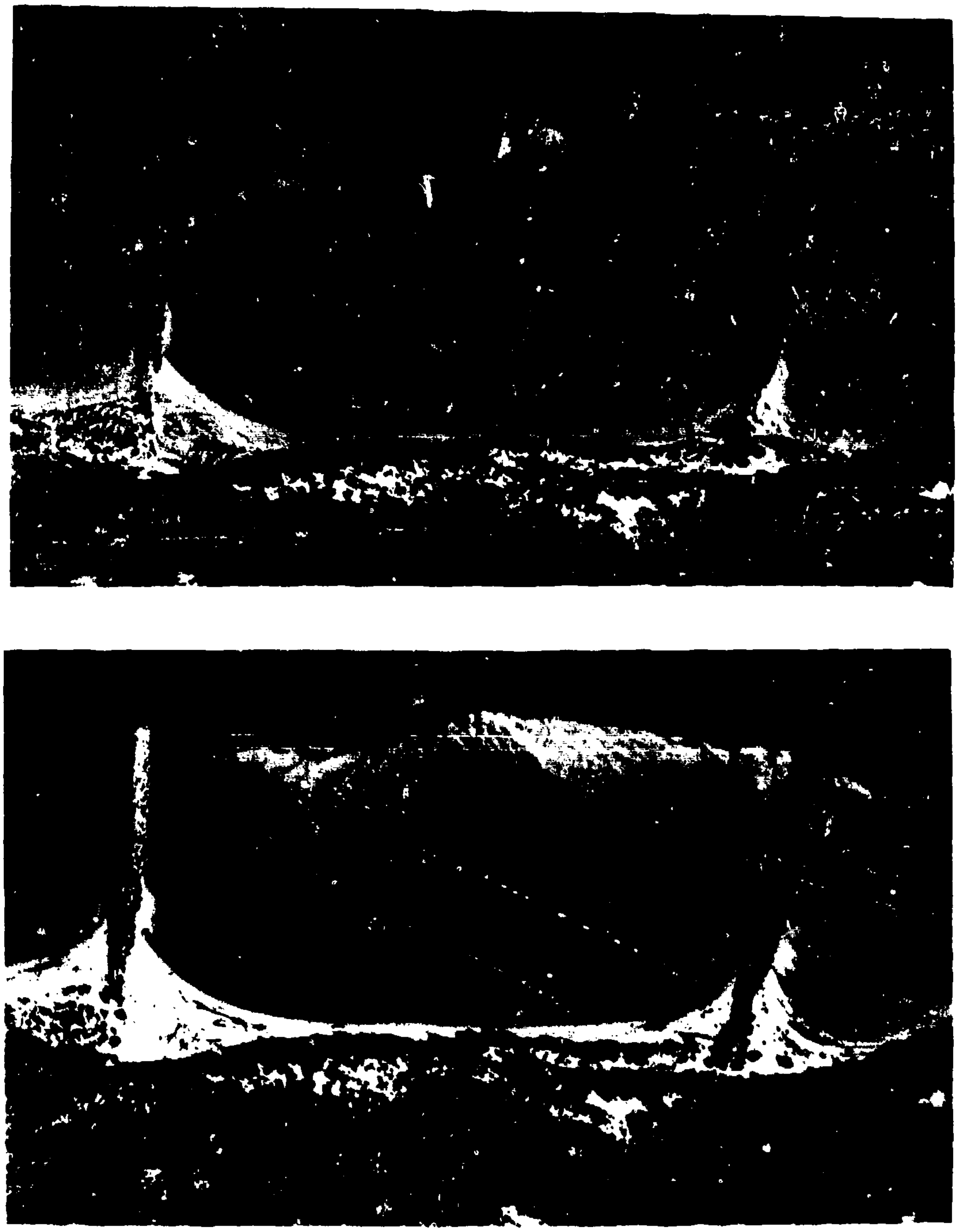


Figure 2-381. A comparison of transverse groins 1.79 (left) and 1.94 (right), Siuslaw River, Oregon, (top photo, 5 January 1979; bottom photo, 11 December 1979).

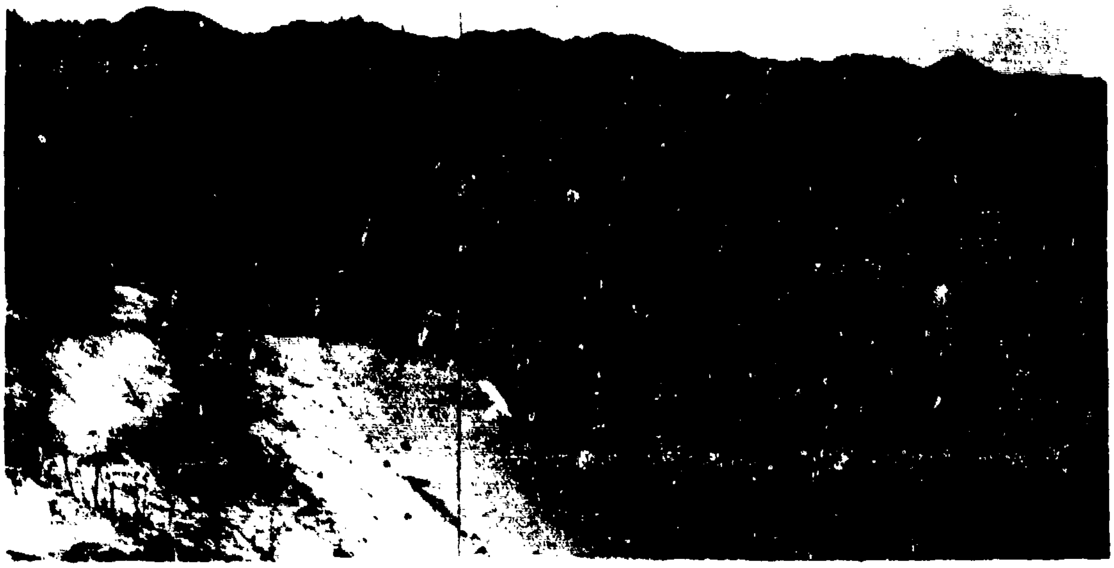


Figure 2-382. Shoreline adjacent to mouth of dune drainage channel, Siuslaw River, Oregon, June 1979.

Figure 2-383. Shoreline recession adjacent to mouth of dune drainage channel, Siuslaw River, Oregon, December 1979.

(4) Analysis. Over the past year, specific areas of erosion and accretion developed through the influence of the groin and revetment system. Accretion is now occurring over most of the formerly eroding riverbank. The existing revetment and groins could be considered successful in that the navigation channel was stabilized and bank erosion was prevented. However, continued monitoring of the dune drainage channel will be necessary to study the movement of the dune drainage channel and the erosion of the adjacent bankline. Based on monitoring data, measures may become necessary to prevent further erosion in that area.

5. Great Lakes Sites.

a. Common Characteristics.

(1) Geographical Setting. Two demonstration sites and seven monitoring sites are located on the shores of the Great Lakes (Fig. 2-384). One demonstration site is on the south shore of Lake Superior at Port Wing, Wisconsin; the other is on the south shore of Lake Erie at Geneva State Park, near Ashtabula, Ohio. Three of the monitoring sites (Tawas Point, Sanilac Section 11, and Sanilac Section 26) are on the west shore of Lake Huron in Michigan, one site (Muskegon State Park) is on the east shore of Lake Michigan in Michigan, and one site (Ashland) is on the south shore of Lake Superior in Wisconsin. At two monitoring sites (Little Girls Point, Michigan, on Lake Superior, and Lincoln Township, Michigan, on Lake Michigan), the previously installed devices were no longer effective, and monitoring was discontinued shortly after the initial site visits. Therefore, coverage of these sites is limited by the small amount of data available. Located on freshwater lakes, the sites are not subject to severe corrosion, marine borer problems, or salinity problems. The sites have no tides, but there are seasonal variations in lake level of about 1 foot or more. However, lake levels change yearly, and a long-term rising trend of lake levels can cause erosion problems equally as severe as those due to daily ocean tides, or long-term sea level changes.

(2) Climate. Although the Great Lakes lie between latitudes 41° and 50°, the region does not have the severe climate normally associated with these latitudes; the large surface areas and depths of the Great Lakes moderate the summer and winter temperatures along their shores. The average annual temperature for the basin is 43° Fahrenheit. The basin has warm summers, with frequent periods of hot, humid tropical air from the Gulf of Mexico. In winter, arctic air dominates the region, with mean daily temperatures below freezing for 3 to 6 months. Annual precipitation (including equivalent snowfall) ranges between 26 and 52 inches, of which more than half occurs during the summer season.

(3) Water Levels. Lake levels fluctuate in response to changes in precipitation on the lake surfaces, inflow from tributary watersheds, evaporation, and outflow from one lake to another and to the Atlantic Ocean via the St. Lawrence River. Since the amount of water entering the Great Lakes system cannot be controlled, lake regulation works can modify but cannot control the seasonal rise and fall of lake levels. Through the years, each lake has intermittently dropped to a low level and then recovered. Long-term annual and recent monthly average levels for each of the lakes on

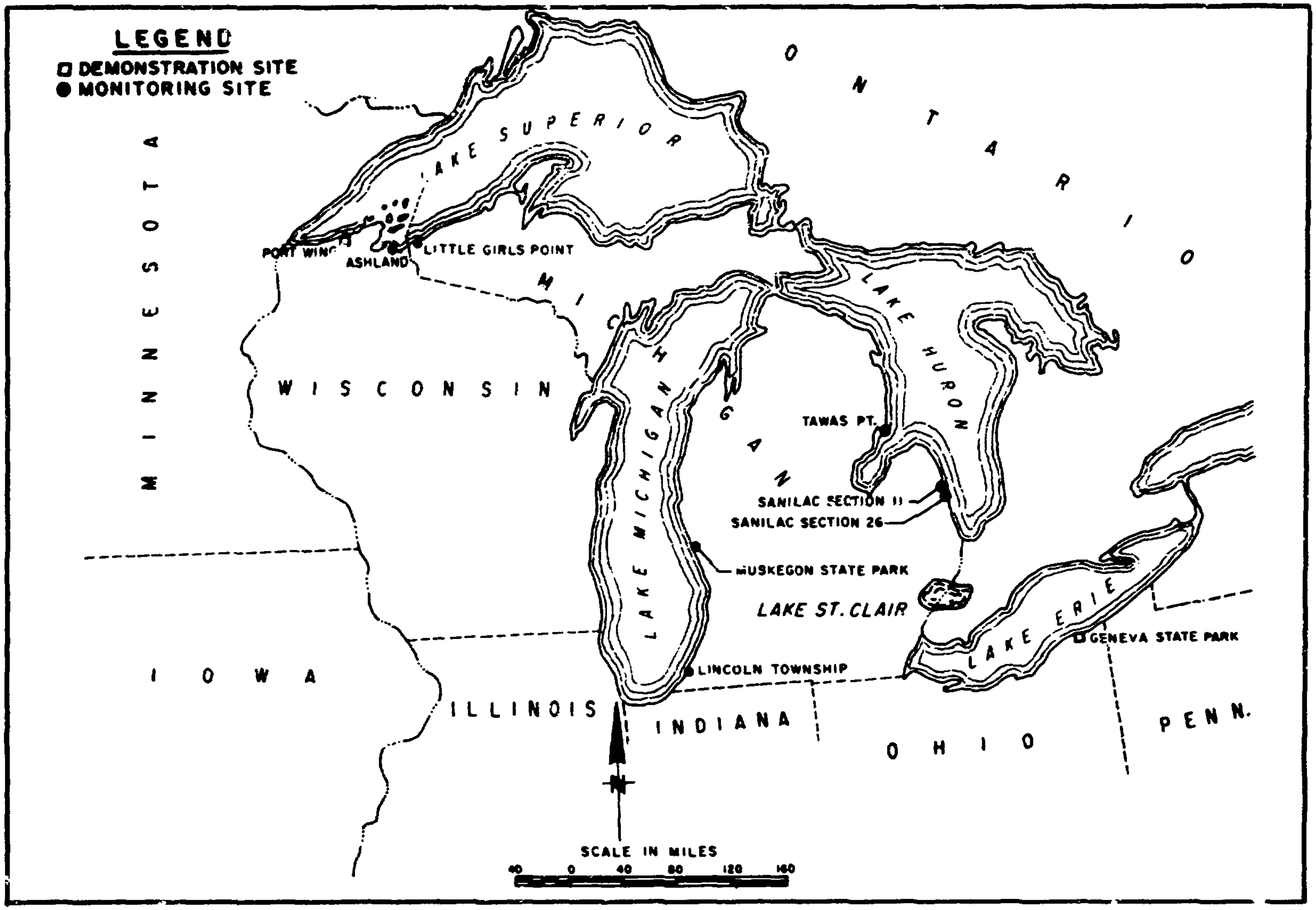


Figure 2-384. Location map of Great Lakes sites.

which demonstration or monitoring sites are located are shown in Figures 2-385 and 2-386. A single datum plane, the International Great Lakes Datum (IGLD), is used for general purposes for all of the Great Lakes. IGLD is the mean water level in the St. Lawrence River at Father Point, Quebec. In addition to IGLD, each lake has its own low water datum (LWD) plane, determined by water level fluctuations through the period of record and expressed in feet above IGLD. LWD is + 600.0 feet IGLD for Lake Superior, + 576.8 feet IGLD for Lakes Michigan and Huron, and + 568.6 feet IGLD for Lake Erie.

(4) Ice. Ice usually begins to form around the shoreline of each lake about the end of December. The initial ice cover is 6 to 8 inches thick and extends for several hundred yards offshore; ice during severe winters may become about 2 feet thick. The ice begins to weaken about the middle of March, forming into moving fields and windrows. The winter ice cover prevents wave action from eroding the shore or affecting littoral processes for about 4 months of the year. Expansion of the ice cover and shifts under wind stress may gouge an unprotected shoreline. During severe winters, ice masses may pile 15 to 20 feet high. The ice does not normally cause appreciable or lasting damage to beaches or riprap, but it may damage timber, steel, or concrete structures by overloading them with excessive horizontal or vertical stresses.

(5) Wind, Waves, and Littoral Transport. Each of the Great Lakes is large enough for winds blowing across it in any direction to generate ocean-size waves that erode the windward shore. The erosion process is aggravated by the friable nature of the perimeter bluffs, most of which are of glacial origin and succumb readily to wave attack. In general, each lake is considerably longer than its width. Material eroded along the perimeter near the minor axis of the lake tends to move in either direction in longshore transport toward the far end of the lake. The farther the material gets from the minor axis the faster it tends to move, both because the fetch distance and therefore the wave heights tend to increase, and because the angle of wave approach tends to become more oblique to the shoreline, increasing the net longshore component of wave energy flux. The result is eroding shorelines along the sides of the lake and accreting shorelines at the ends of the lake. This effect is not universal, as it is modified by prevailing winds, local anomalies in the general trend of shoreline orientation, resistant shore segments, and river-mouth discharges of granular material. However, the geologic trend is to make a round lake out of every large, elongated lake.

Not all of the material eroded from the lake bluffs moves laterally along the shore. The finer grains tend to move offshore by a selective transport process, with some eventually moving into water deep enough that they remain below reach of the waves. In this process, submerged bars often develop just offshore. High waves break at the bars and plunge to the bottom, scouring a trench just shoreward of each bar. The broken waves may re-form into smaller waves and, together with waves that are too small to break on the bar, impinge directly on the shoreline. Because of this process, and as long as the lake level remains relatively constant, all the higher storm waves break offshore, and only the smaller waves and the waves that have already been broken can attack the lakeshore directly. Under these conditions, a low-cost shore protection device may be quite effective in

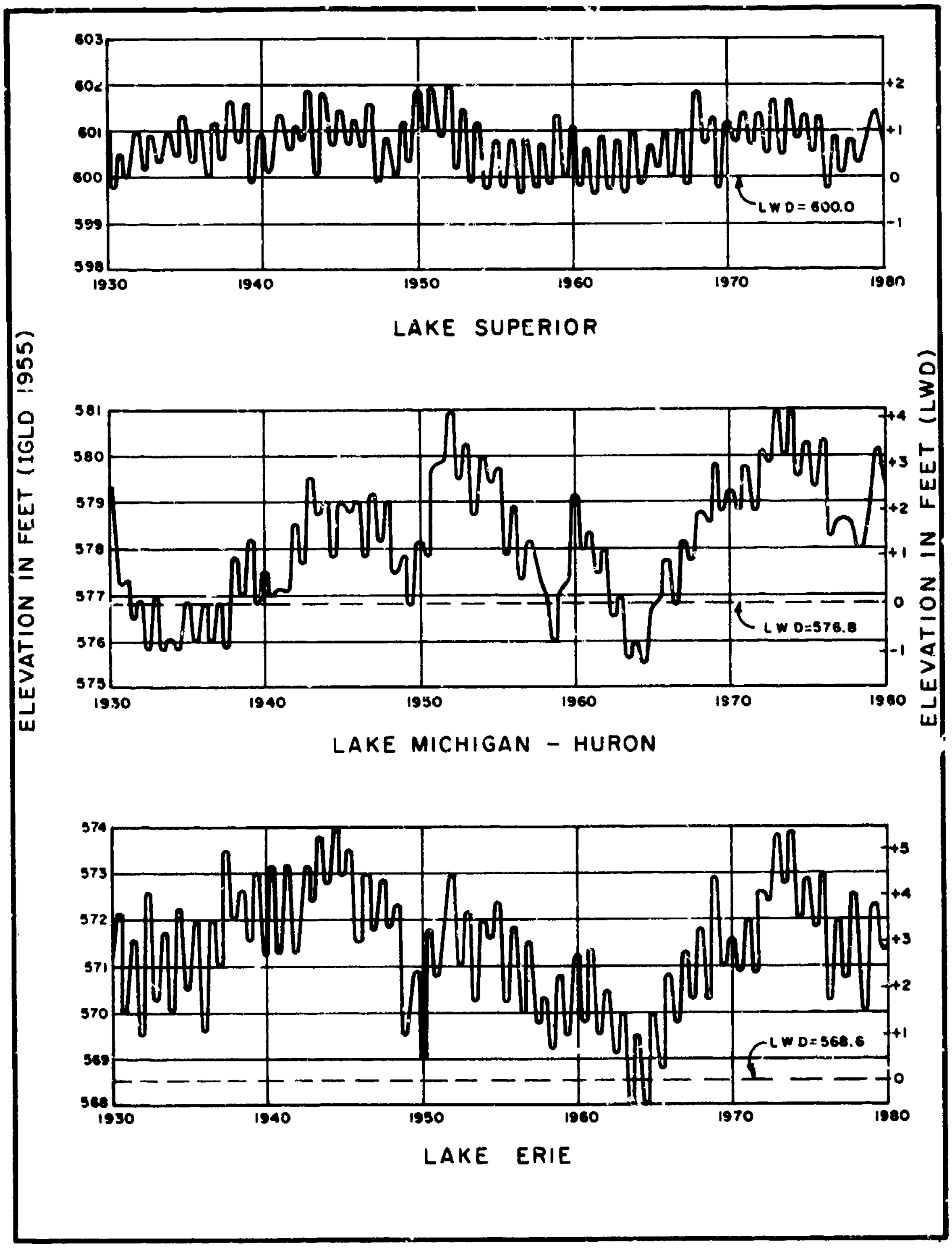


Figure 2-385. Great Lakes surface elevation changes (last 50 years).

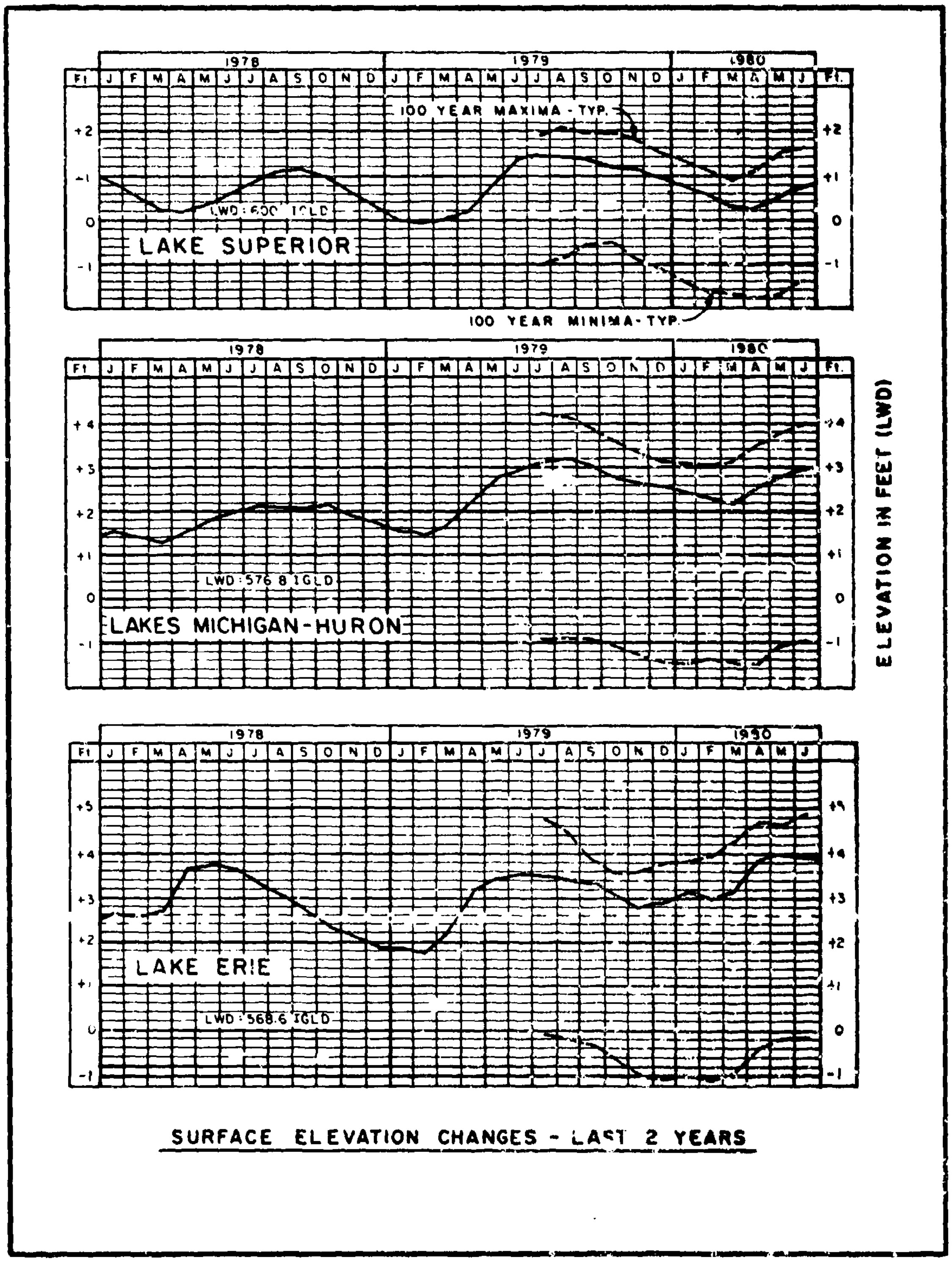


Figure 2-386. Great Lakes surface elevation changes (last 2 years).

preventing bluff erosion. However, during a progressive long-term rise in lake level, the offshore bars become more deeply submerged, allowing the larger waves to reach shore before breaking. At such times, the low-cost devices could not resist the wave forces, and only massive protective structures, commensurate with those used on open seacoasts, would survive.

(6) Geomorphology and Soils. Before the Pleistocene epoc it is believed that the basins now occupied by the Great Lakes were weak rock lowlands that drained eastward to the Gulf of St. Lawrence. The area was characterized by well-drained valleys and divides of several large rivers; the Great Lakes, as they are known today, did not exist. During the Pleistocene epoc, an ice sheet up to 2 miles thick advanced south to what is now St. Louis and Louisville. Temperature changes during the glacial period caused the ice sheet to retreat and advance several times, the effects of the most recent ice advance, the Wisconsin, resulted in the topography of the Great Lakes region as it is today.

Three basic shoreline types exist at the project sites: (a) Bluffs consisting of lacustrian clay or reworked lacustrian clay deposited as till; (b) bluffs or dunes consisting of fine- to medium-grained windblown sand; and (c) bluffs consisting of silty clay till, probably deposited as the glacier retreated. Bluff erosion generally results from slope instability, groundwater seepage, surface runoff, freeze-thaw cycles, wave scouring at the toe, and removal of detritus by littoral currents, all contributing to slope failure in varying degree, depending on specific site conditions. Much of the southern shore of Lake Superior is in an area of high steep bluffs developed by wave action in unconsolidated fine-grained glacial deposits. In many areas, both glaciolacustrian deposits and glacial till form the bluff; lacustrian deposits are typically stratified and contain little or no coarse-grained material. The glacial tills consist mainly of reworked reddish-brown lacustrian silts with varying amounts of coarser material. The beach is typically narrow and the waterline is at or near the base of the bluff. This shoreline is typical of the shoreline at the three sites along Lake Superior.

The eastern shore of Lake Michigan typically consists of sand dunes or sandy bluffs ranging in height from a few feet to several hundred feet. These are generally sands that were shoreline or beach deposits at different lake levels that have been windblown into their present form. In some areas of the coastline, till deposits may be exposed or covered by only a few feet of wind-deposited sand. Beaches of varying width exist, as do offshore sandbars. The sites along the Lake Michigan shoreline fall into this category and are typically low sand dunes or bluffs with narrow beaches.

The sites along the western Lake Huron shoreline and the Lake Erie site can by typified by moderately high bluffs consisting primarily of clay till, but with some areas of lacustrian silt deposits. These deposits have a different origin than those along the Lake Superior shoreline. A narrow beach or no beach is typical of most sites. The Geneva State Park demonstration site along the Lake Erie shoreline is unique in that the lake is very shallow, and minor changes in lake level greatly alter the shoreline because of flat bottom slopes.

(7) The Problem. Bluff erosion is common to all of the Great Lakes sites. Although soil conditions vary from site to site, most of the sites have no beach or a very narrow beach, which allows waves to

attack and remove material from the base of the bluff. The bluff slope then becomes unstable, and large masses of material slide to the bluff toe. The waves then distribute this detritus both laterally along the shore and lakeward into deep water, and the process continues. Mass wasting of high, steep bluffs that are predominantly clay and silt may occur as a result of sloughing of surface material, deeper seated slides, or solifluction. Solifluction and surface erosion may also occur on a flatter slope. Where the bluffs are primarily sandy materials, the mass wasting consists of sloughing of the undercut areas.

During a progressive long-term rising trend of the lake level, higher waves reach the shoreline and intensify the erosion process. Shoreline recession is accelerated both by the intensified erosion and by the landward displacement of the land-water interface. Although the long-term trend has been a lowering of lake levels in recent years, the seasonal spring and summer high levels have remained well above the long-term average, and erosional trends have abated only slightly.

b. Port Wing, Wisconsin.

(1) Site Description.

(a) Geographical Setting. The Port Wing demonstration project is on the southern shore of Lake Superior along State Highway 13 in Bayfield County, Wisconsin, approximately 23 miles east of Superior and 6 miles west of Port Wing (Fig. 2-387). State Highway 13 is the South Shore Scenic Drive of Lake Superior between Superior and Ashland, Wisconsin, and provides an access to the Apostle Islands National Lakeshore area. In the project area, it crosses relatively flat terrain, about 30 feet above lake level, which is traversed at irregular intervals by streams draining into the lake.

(b) Water Level, Wave Conditions, and Longshore Transport. The greatest seasonal lake level fluctuation in Lake Superior, based on the highest and lowest monthly means for a particular year, was 2.67 feet in 1869. In addition to the general lake level fluctuations, short-term variations at the Port Wing site are caused by differential atmospheric pressures and by wind setup. Changes of this type can be more than 2 feet but seldom exceed 1 foot above or below the normal level. The discharge of water from Lake Superior to the lower lakes has been regulated since 1921 in an attempt to maintain the mean monthly level of the lake as closely as possible between 600.5 and 602.0 feet IGLD. The design water surface level of 602.9 feet was obtained by adding to the mean of the monthly levels over the past 10 years the maximum recorded 1-year short-term rise.

The foreshore at the site has a gentle slope of 1 on 75, which causes large waves to break well lakeward of the shoreline. However, smaller waves can and do reach the toe of the bluff. A 5-foot design wave was determined appropriate for demonstration devices. The shoreline orientation at the site is roughly N. 80° E.; the shoreline is exposed to storm waves with approach directions of west to northeast. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 5.0 feet, and the net longshore transport potential northeastward

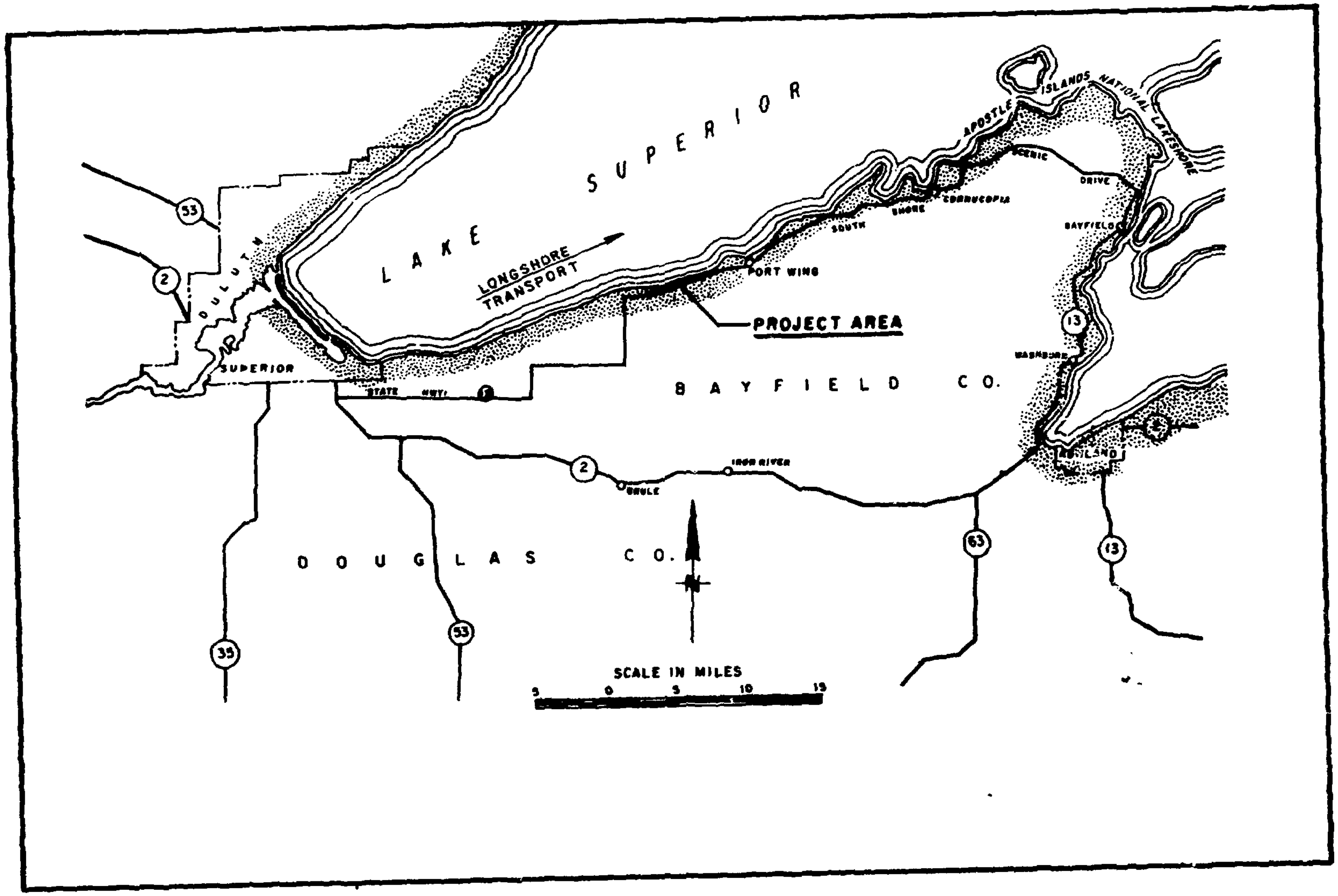


Figure 2-387. Location map of Port Wing, Wisconsin, demonstration site.

was 195,300 cubic yards for the 7 months analyzed. The wave climate is classified as severe.

(c) Geomorphology, Soils, and Vegetation. The south shore of Lake Superior between the town of Superior and the Apostle Islands varies in topography from marsh to dunes and bluffs. At the Port Wing demonstration site, a 20- to 30-foot-high bluff extends along the shoreline, with little or no beach. The bluff is comprised mostly of a medium-stiff red lacustrian clay, deposited during a higher level of glacial Lake Superior. In some areas, the medium-stiff clay is underlain by a stiff brown or gray silty clay with gravel, which is probably an earlier till deposit. The bluffs have fairly steep slopes ranging from 1 on 1.5 at the base, where till exists, to a flatter 1 on 4 in the upper parts of the bluff, where the red lacustrian clays exist. Along the shore, 5 to 10 feet of sand and silty sand overlies very dense, weathered sandstone. The foreshore in this area has a slope of approximately 1 on 75.

The land area adjacent to the site is generally heavily wooded with pine, red maple, trembling aspen, sugar maple, yellow birch, basswood, balsam fir, spruce, and white cedar. In natural clearings and where the trees have been removed, there are native shrubs and herbs which include hazel, bearberry, blueberry, sweet fern, dogbane, columbine, sarsaparilla, strawberry, and bunchberry. Several species of sedges and grasses also provide natural ground cover.

(d) The Problem. The Port Wing site is a typical example of a high-bluff erosion problem common to many shoreline reaches. A 1974 survey indicated that the top of the bluff had retreated 50 feet and that the toe of the bluff had retreated 26 feet since 1962. A more recent survey indicated an additional 7 feet of retreat at the top of the bluff between 1974 and 1977. This yields an annual rate for the last 5 years of about 4 feet per year at the top of the bluff and 2 feet per year at the toe. During this interval, the average annual lake level rose 0.5 foot.

Bluff recession at the site is a combination of two processes--sloughing at the bluff face and wave erosion of the bluff toe. The softer lacustrian clay deposits in the upper part of the bluff are apparently undergoing sloughing, solifluction, and block-calving due to weathering, surface runoff, and undercutting. Erosion at the base is primarily by direct wave attack. The stiffer till deposits at the base of the bluff are more resistant to erosion, which results in relatively steeper lower slopes.

(2) Demonstration Project. The selected plan comprised devices designed to protect about 1,030 feet of eroding shoreline and to stablize the upper bank. The overall plan as constructed is shown in Figure 2-388. The structures initially considered most effective were placed in areas with a critical need for highway protection. To prevent loss of foundation material, granuler fill and a nonwoven filter cloth were placed behind or under each protective structure. To provide slope stability, the toe of each structure was keyed into the lake bottom. The top elevation of each structure was designed to protect the upper slope from waves with a 20-year return frequency. At each end of the project and separating the demonstration devices, 50-foot segments of stone riprap on filter cloth were used to prevent flanking failures in case the adjacent structures failed or flanking

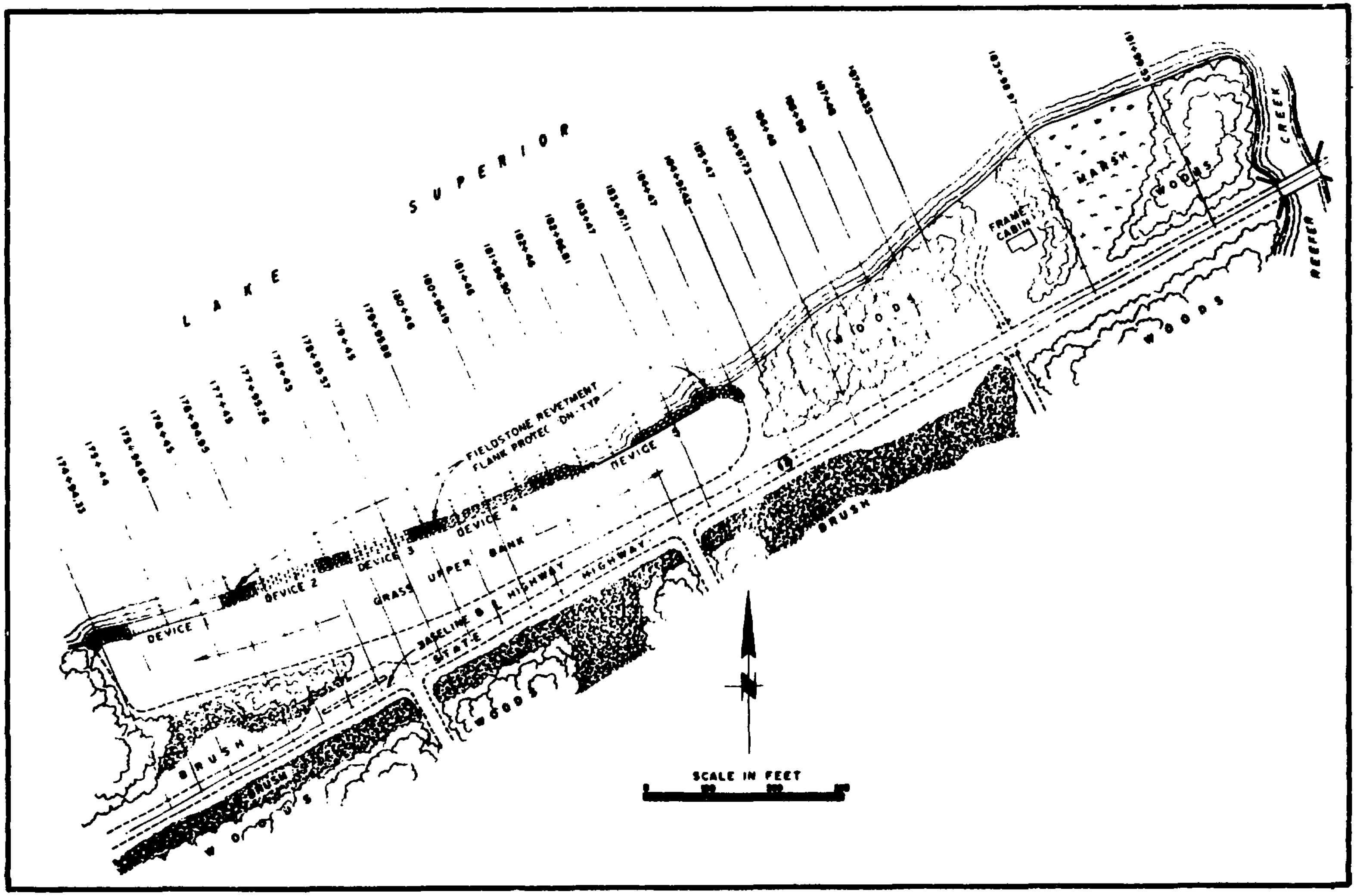


Figure 2-388. General plan of Port Wing, Wisconsin, demonstration site.

bluffs eroded. Although the bank area between the demonstration structures and the highway embankment was seeded, it did not constitute a vegetation planting demonstration device. The structural devices installed at this site are numbered 1 to 5 (from west to east) and are described below. These device numbers do not necessarily agree with the numbering system used by the District, which retained originally assigned numbers although changes made during installation rendered that system nonconsecutive. The riprap segments and the seeding of the upper bank are also described.

(a) Device 1 (Turfblock Revetment). The revetment was constructed of 16- by 24-inch concrete Turfblocks (also called Monoslabs) 4.5 inches thick, cast in a perforated grid pattern. The blocks were laid on a nonwoven filter cloth on a 1 on 3 slope. Granular material was used to build up the subgrade where the design slope was above the existing ground surface. The structure is 150 feet long and has a toe elevation of 596.8 feet (IGLD, 1955) and a top elevation of 607.0 feet. The toe of the structure is about 3 feet below LWD and extends 9 feet out into the lake. Figure 2-389 shows details and a cross section of the Turfblock revetment.

(b) Device 2 (Small Control-Block Revetment). The revetment was constructed of control blocks 8 inches wide, 16 inches long, and 8 inches deep. The blocks are similar to the standard concrete construction blocks used at Fontainebleau State Park, Louisiana, except that protrusions are cast into the block ends to provide a tongue-and-groove interlock between blocks. The blocks were laid on a nonwoven filter cloth on a 1 on 3 slope, as in device 1. The structure is 100 feet long with a toe elevation of 596.8 feet (IGLD, 1955) and a top elevation of 607 feet. Figure 2-389 shows details and a cross section of both device 2 and device 3 structures.

(c) Device 3 (Large Control-Block Revetment). The structure is similar to the small control-block revetment except that it is constructed of blocks 12 inches wide. It also is 100 feet long.

(d) Device 4 (Used-Tire Bulkhead). The bulkhead was constructed of scrap tires of 13-, 14-, and 15-inch inside diameters. The tires are interconnected (both vertically and horizontally) with fortypenny galvanized spikes and pushnuts. Three rows of galvanized-steel anchors on 10-foot centers secure the structure to the beach. The tires are placed flat with their holes vertical; the holes in adjacent layers of tires are staggered. Granular material was used both as backfill in low areas and as fill for the tires. Nonwoven filter cloth was placed under the structure and between the structure and the bank. The structure is 150 feet long with a toe elevation of 598.8 feet (IGLD, 1955) and a top elevation of 607 feet. Figure 2-390 shows details and a cross section of the used-tire structure.

(e) Device 5 (Steel and Timber Bulkhead). The bulkhead was constructed of 26-foot-long steel H-piles (HP 10x42) and 8-inch by 6-inch by 8-foot treated-timber railroad ties. The ties were placed between the flanges of adjacent H-piles. The toe of the structure is protected by riprap. Nonwoven filter cloth and granular fill were placed between the structure and the bank. The 200-foot-long structure has a toe elevation of 598.8 feet (IGLD, 1955) and a top elevation of

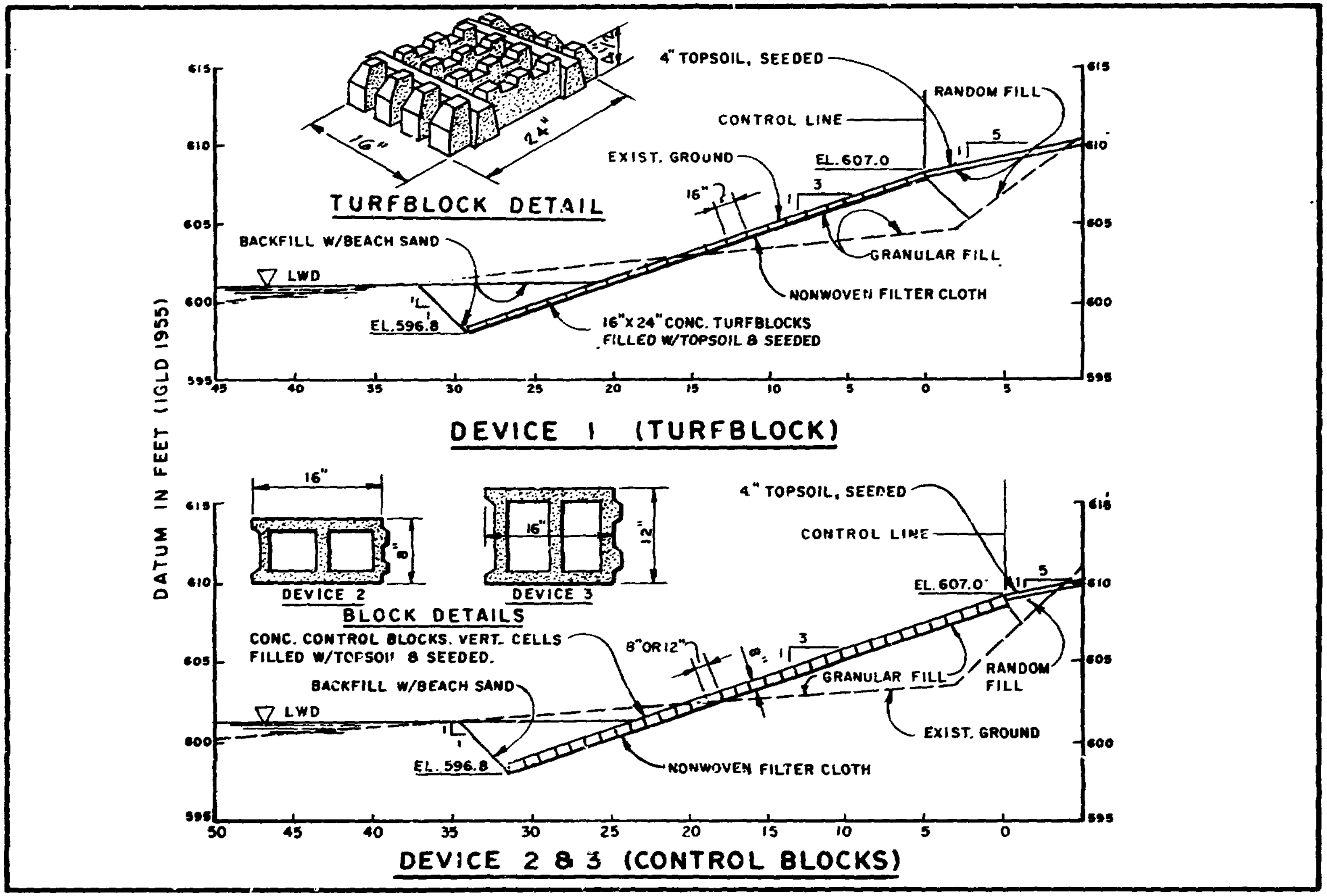


Figure 2-389. Devices 1, 2, and 3 at Port Wing site.

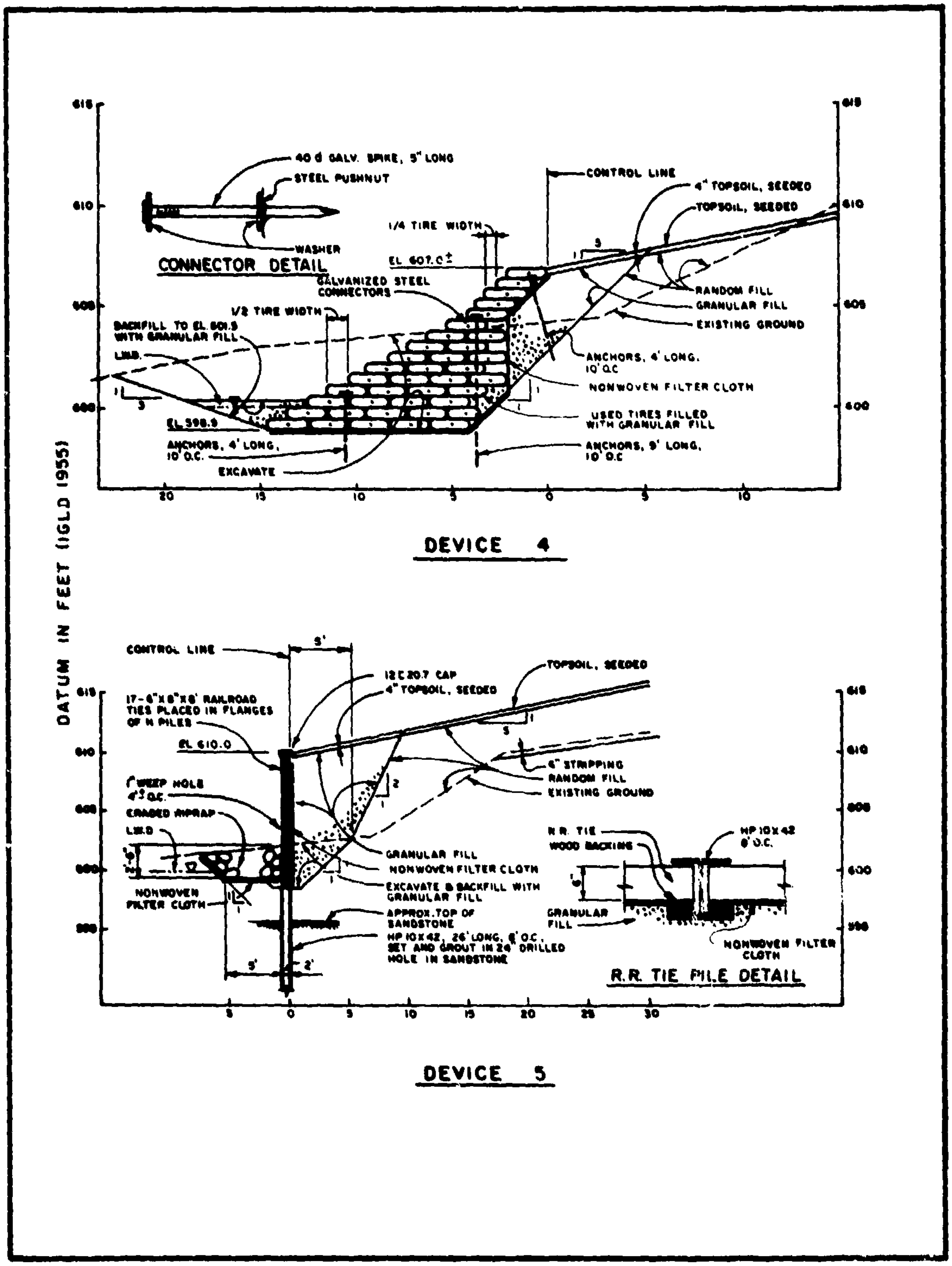


Figure 2-390. Devices 4 and 5 at Port Wing site.

610 feet. The steel and timber structure has a higher top elevation because of the higher wave runup produced by a vertical structure. Figure 2-390 shows details and a cross section of the structure.

(f) Riprap. A total of 328 feet of stone riprap was placed on both ends of the project site and between demonstration structures to prevent erosion from flanking the ends of the project site and to prevent the failure of one structure from contributing to the failure of an adjacent structure. It is also a conventional time-tested method of erosion control and can be used to compare the effectiveness of the demonstration structures. The riprap is 2.6 feet thick and was placed on a 1 on 2.5 slope. The stones (from a deposit of boulders) are rounded and do not key together as well as the more angular quarrystones of most rubble structures. The minimum stone weight is 120 pounds (50 percent of the stones are 550 pounds or heavier) and the maximum weight is 2,000 pounds. The elevation of the toe is 595.8 feet (IGLD, 1955); the elevation of the top is 607.0 feet.

(g) Upper Bank. The bank between the structures and the road was graded to a uniform 1 on 5 slope, covered by 4 inches of topsoil, fertilized, seeded, and mulched. This provided an attractive, stable upper bank. The grass cover provides good erosion control and the structures are readily accessible for observation. The type of vegetation cover and the planting method were established through coordination with the Soil Conservation Service and the Wisconsin Department of Transportation. Grass seeding met the requirements of the Wisconsin Department of Transportation specifications (see Table 2-96 for seed and mixture).

Table 2-96. Seeds and mixtures used for vegetative cover at Port Wing, Wisconsin.

Species	Seeds		Mixture	
	Purity (pct)	Germination (pct)	No. 3 (pct)	No. 5 (pct)
Kentucky 31 fescue (*Festuca* sp.)	97	85	65	--
Kentucky 21 bluegrass (*Poa pratensis*)	85	80	10	--
Creeping red fescue (*Festuca rubra*)	97	80	15	--
Red top (*Agrostis alba*)	92	85	5	--
Perennial ryegrass (*Lolium perenne*)	95	90	5	--
Empire birdsfoot trefoil (*Trifolium* sp.)	95	80	--	35
Crownvetch (*Vicia* sp.)	95	70	--	65

The seed was planted on the upper bank with a hydroseeder in July 1979 after all the structures were completed. Seed was applied at a rate of 3.5 pounds per 1,000 square feet. Fertilizer was uniformly distributed at a rate of 7 pounds per 1,000 square feet and applied into the top 2 inches of soil. Mulch was applied to protect the seedlings. The area was watered when required to ensure germination of the seed and establishment of turf. The turf became established by the fall of 1979; Figure 2-391 is a cross section of the upper bank area. The total cost of grading and seeding the reshaped upper bank was $33,955, or about $33 per foot. A gravel turnout was constructed next to the highway; a sign describing the project and defining each type of structure was erected next to the turnout. This made the site readily accessible and informed the public about the project.

(3) Statistics. Statistics for the five devices and upper bank work are given in Table 2-97.

Table 2-97. Statistics for Port Wing, Wisconsin.

Item	Quantity
Materials	
H-piles	676 lin ft
Railroad ties	3,400 lin ft
Stabilized aggregate	55 yd^3
Granular fill	825 yd^3
Riprap	1,100 yd^3
Filter cloth	2,860 yd^3
Turfblocks	1,800 ea.
8-in concrete blocks	3,600 ea.
12-in concrete blocks	2,400 ea.
Used tires	2,600 ea.
Wood posts	16 ea.
18-in corrugated metal pipe (with end sections)	8 lin ft
Labor (man-hours)	
Skilled	490
Semiskilled	345
Nonskilled	1,665
Equipment (hour)[1]	
Bulldozer	460
Backhoe	390
Tracked excavator	240
Scraper	220
Power auger	120
Dump truck	80
Farm tractor	40
Mulcher	40
Hydroseeder	24
Chipper	200
Bobcat loader	120

[1]Hours the equipment was at the site; no actual accounting of hours used is available, as the project was bid on a lump-sum basis and an inspector was not on site during all working hours.

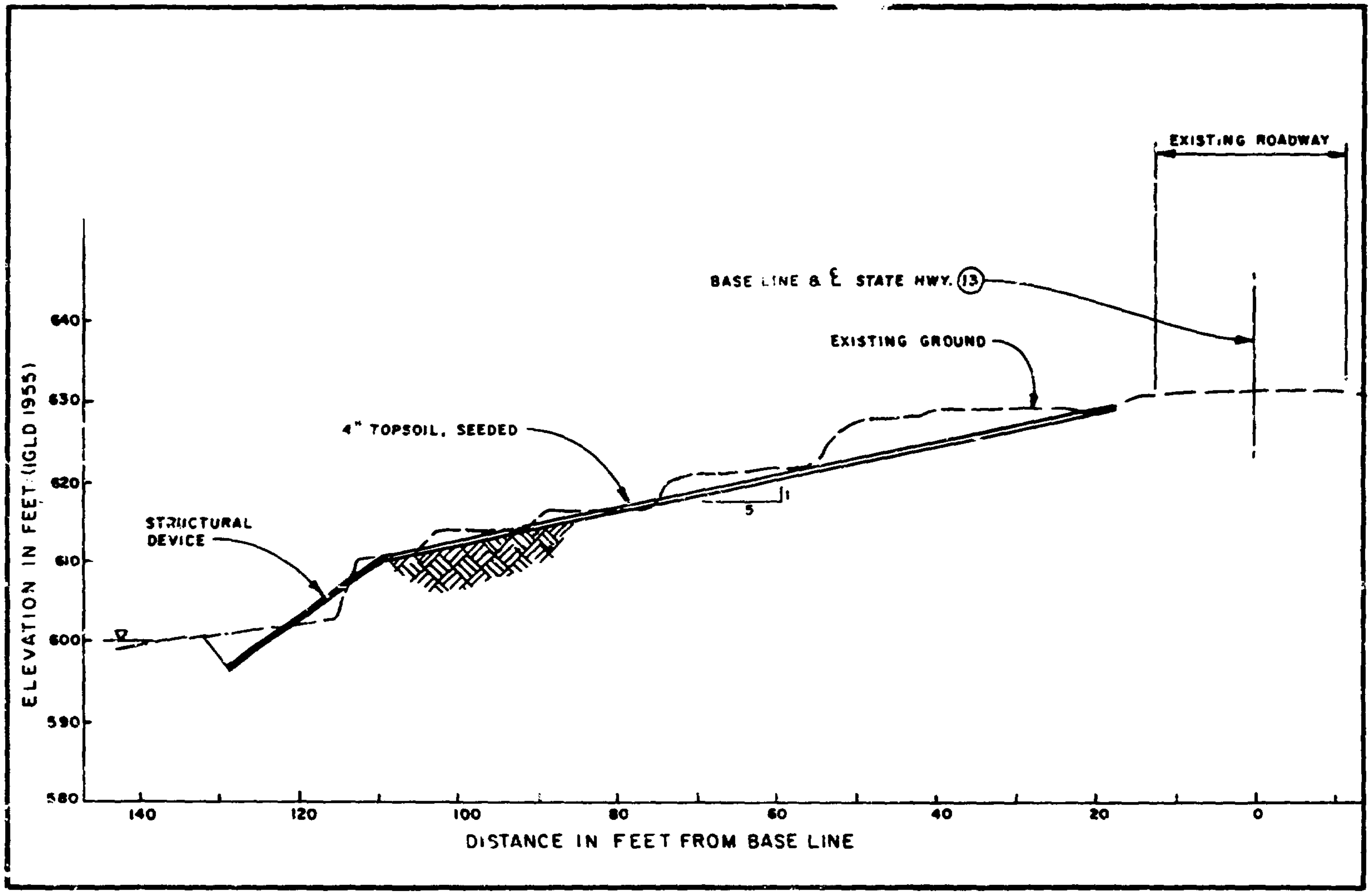


Figure 2-391. Typical profile at Port Wing site.

(4) Construction and Costs.

(a) Device 1 (Turfblock Revetment). A backhoe and loader were used to prepare a uniform 1 on 3 slope. Beach sand was excavated near the toe of the structure and granular fill was placed near the top of the structure to provide the base. Nonwoven filter cloth was then placed on the prepared slope, and Turfblocks were hand-placed on the filter cloth (Fig. 2-392). The blocks were placed with the 24-inch dimension parallel to the waterline. Some difficulty was encountered in preparing the slope and placing blocks near and below the waterline because of wave and weather conditions (Fig. 2-393) (also caused difficulty in placing the control blocks and tires). The toe of the structure was backfilled with beach sand to the LWD, and the top of the structure was covered by topsoil. However, before the soil could be seeded, storm waves removed the topsoil and largely replaced it with sand. Construction was completed 9 November 1978.

(b) Device 2 (Small Control-Block Revetment). The construction procedure was similar to that used for the Turfblock structure. The blocks were hand-placed with their long dimensions parallel to the waterline and the cells vertical (Figs. 2-394 and 2-395). Construction was completed 9 November 1978.

(c) Device 3 (Large Control-Block Revetment). The construction procedure was similar to that used for the small control-block structure. Construction was completed 30 October 1978.

(d) Device 4 (Used-Tire Bulkhead). A backhoe and loader were used to excavate the beach and prepare a base for placement of the tires. The base was about 11 feet wide and 1.2 feet below LWD. A nonwoven filter cloth was placed under the tires; the tires in each layer were then filled with granular fill during placement (Fig. 2-396). The tires were placed flat, and the holes in successive layers of tires were staggered. A row of anchors on 10-foot centers was installed near the structure toe (4-foot-long anchors), near the middle (10-foot-long anchors), and near the top of the structure (4-foot-long anchors). The anchors are galvanized steel with a 4-inch anchor and a 0.75-inch rod, similar to the anchors used for power-pole guy wires (Fig. 2-397). A reinforcing steel rod was run parallel to the waterline between anchors to exert vertical force on the structure and hold it in place. Nonwoven filter cloth was placed on the bank side of the tires, and granular backfill was placed between the filter material and the bank. Granular fill was also placed on the toe of the structure to about LWD. Construction was completed 19 July 1979.

(e) Device 5 (Steel and Timber Bulkhead). A truck-mounted auger drill was used to sink 2-foot-diameter holes about 12 feet into the sandstone bedrock on 8-foot centers. The H-piles were set into the holes and grouted in place (Fig. 2-398 and 2-399). A backhoe and loader were used to excavate the beach to about elevation 598.8 feet (IGLD, 1955). Timber ties were then placed between the flanges of adjacent H-piles. A 12-inch steel channel was welded on top of the H-piles (Fig. 2-400) to serve as a cap to aline the steel H-piles and protect the timber ties. Graded riprap was placed on the lakeside of the wall for toe protection (Fig. 2-401). Nonwoven filter cloth and granular fill were then placed on the bank side. Construction was completed 1 December 1978.

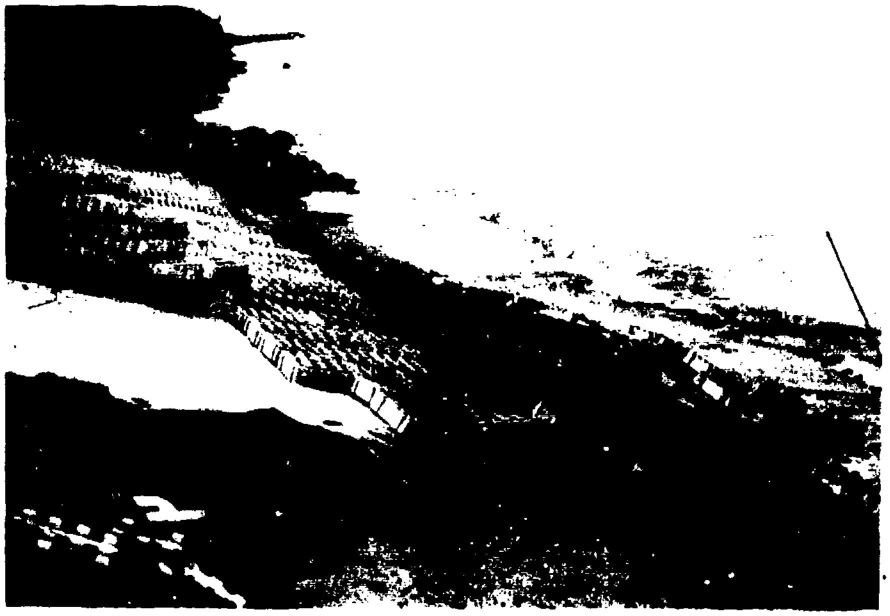


Figure 2-392. Placement of the Turfblocks, Port Wing, Wisconsin.

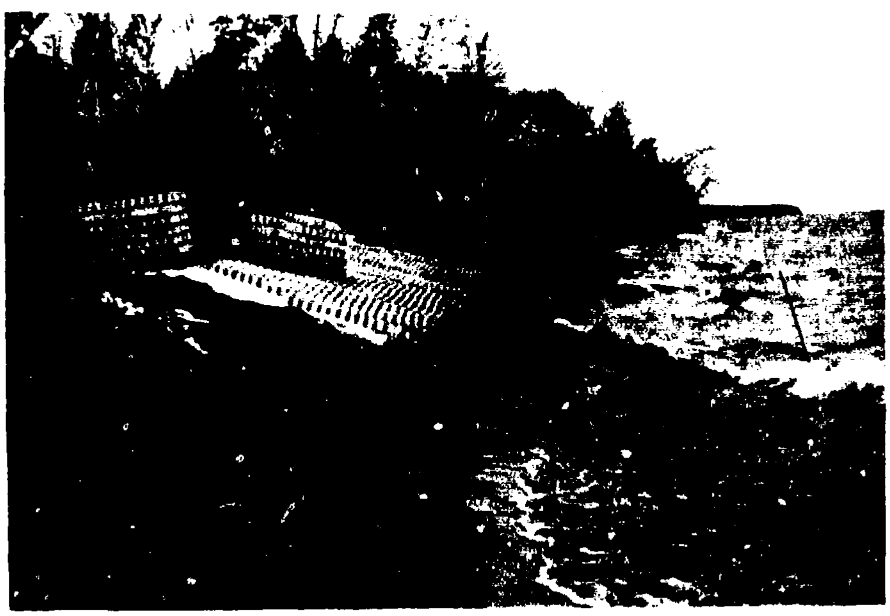


Figure 2-393. Wave and weather conditions causing difficult Turfblock placement, Port Wing, Wisconsin.

Figure 2-394. Placement of control blocks, Port Wing, Wisconsin.

Figure 2-395. Placement of control blocks, Port Wing, Wisconsin.

Figure 2-396. Tires in each layer filled with granular fill, Port Wing, Wisconsin.

Figure 2-397. Installation of anchors, Port Wing, Wisconsin.

Figure 2-398. H-piles in place, Port Wing, Wisconsin.

Figure 2-399. Grouting the H-piles in place, Port Wing, Wisconsin.

Figure 2-400. Steel channel welded on top of the H-piles with filter cloth backing, Port Wing, Wisconsin.

Figure 2-401. Riprap toe protection, Port Wing, Wisconsin.

Project costs for the Port Wing installations are given in Table 2-98.

Table 2-98. Port Wing construction costs.

Materials	Cost	Cost/lin ft
Concrete Turfblocks	$18,315	$122
8-in concrete control blocks	7,400	74
12-in concrete control blocks	7,400	74
Scrap tires	13,650	91
H-pile and timber	36,370	182
Riprap	37,200	113
Reshaped slope (seeding)	33,955	33
Turnout	4,200	
Total	$518,490	

(5) Performance.

(a) Device 1 (Turfblock Revetment). A 2-day storm in early November 1978 generated waves of design height in Lake Superior that damaged the Turfblock revetment. Waves apparently overtopped the structure and displaced many of the individually placed units. The overtopping waves may also have transported material around or through the nonwoven filter fabric. The loss of base material could cause uneven settling of the Turfblocks, making them susceptible to displacement. The resulting rough and irregular cross section increased the likelihood for debris to lodge on the revetment (Fig. 2-402). Figure 2-403 (taken in May 1979) shows that only a small segment at the west end of the revetment remained intact. By September 1979 only minor changes were detectable since the November 1978 storm. Abrasion of the corners and edges of the blocks due to wave-borne debris impact became more obvious, and the filter cloth in exposed areas began to show wear. The runup of waves on this revetment was higher than on any of the others. By May 1980 a significant amount of accretion had occurred along the entire beach. The sand covered most of the abraded Turfblocks that were previously on the foreshore. The bank behind the blocks was considerably more eroded than it was in September 1979 (Fig. 2-404). Some of the exposed blocks deteriorated significantly (Fig. 2-405).

(b) Device 2 (Small Concrete-Block Revetment). By September 1979 this revetment suffered only minimal structural degradation. Some uneven settlement of the blocks occurred and the rows of blocks became misalined (Figs. 2-406 and 2-407); however, the overall revetment remained intact. Abrasion from moving sand and the freeze-thaw cycles had damaged the blocks located in the splash zone. A significant amount of concrete had also been eroded from these blocks. As of May 1980 the revetment was structurally sound and the bank behind it had suffered no erosion (Fig. 2-408). Figure 3-42, Section III, shows a series of profiles through device 2. Comparison of the profiles of June 1979 and October 1979 shows a small amount of accretion lakeward of the toe of the concrete-block revetment. No 1980 survey data were submitted, but the photos adequately depict the performance of the structure.

Figure 2-402. Debris scattered on the damaged Turfblock revetment, Port Wing, Wisconsin, 10 November 1978.

Figure 2-403. Intact western segment of the Turfblock revetment, Port Wing, Wisconsin, 7 May 1979.

Figure 2-404. Accretion of beach and erosion of bank, device 1, Port Wing, Wisconsin, 13 May 1980.

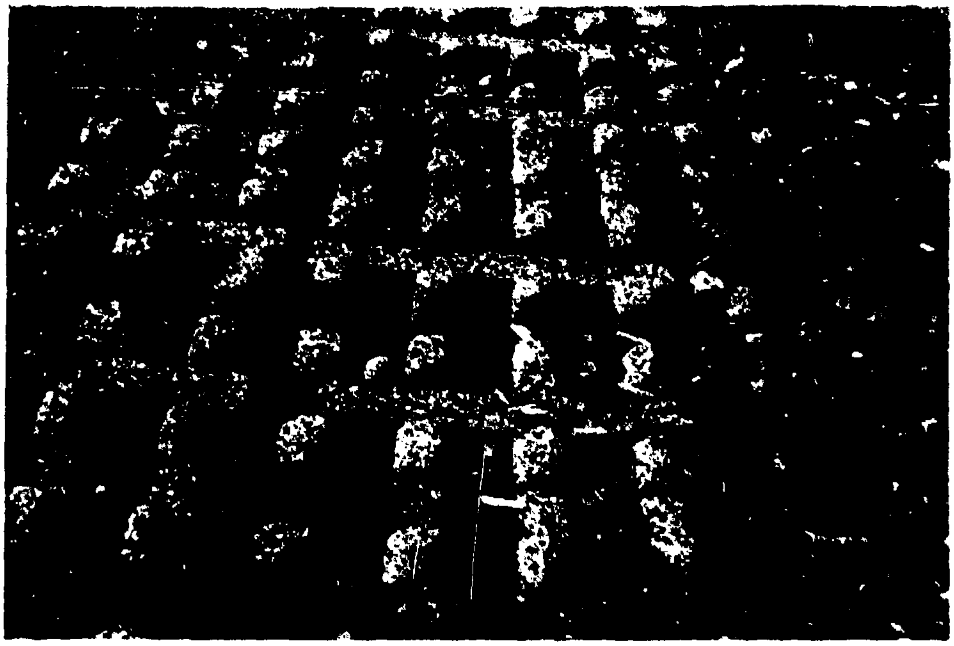


Figure 2-405. Deterioration of Turfblocks, device 1, Port Wing, Wisconsin, 13 May 1980.

Figure 2-406. Uneven settlement of blocks, device 2, Port Wing, Wisconsin, 7 May 1979.

Figure 2-407. Misalinement of blocks, device 2, Port Wing, Wisconsin, 7 May 1979.

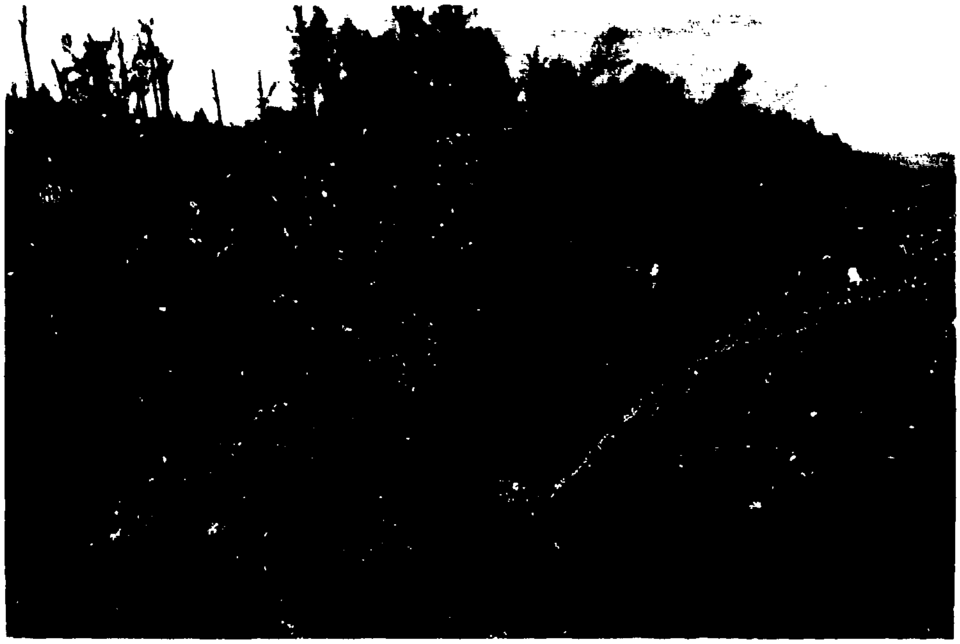


Figure 2-408. Device 2 as of 13 May 1980, Port Wing, Wisconsin.

(c) Device 3 (Large Concrete-Block Revetment). The structural performance of this revetment was quite similar to that of device 2, with the overall revetment remaining intact and relatively undamaged (Figs. 2-409 and 2-410). The flexible nature and low weight of the tires make them difficult to permanently fasten together.

(d) Device 4 (Used-Tire Bulkhead). Although the tires did not deteriorate, the general configuration of the bulkhead underwent considerable change. Within 1 month after completion, many of the tires had separated from the structure and most of the granular fill had been washed out (Fig. 2-411). It became apparent that the tire-interconnecting devices were inadequate for holding the structure together. By the September 1979 inspection, so many tires had broken loose that a major storm could have caused severe structural damage. By May 1980, although the beach had accreted significantly, erosion occurred behind the tires (Fig. 2-412). Profiles through the used-tire bulkhead are shown in Figure 3-18 (see Sec. III), compared with those through similar devices. Comparison of the profiles of June 1979 and October 1979 shows a small amount of accretion in front of the tire stack and minor erosion behind it. No 1980 survey data were submitted, but the photos adequately depict the performance of the structure.

(e) Device 5 (Steel and Timber Bulkhead). At the September 1979 inspection, this bulkhead appeared undamaged and structurally sound (Figs. 2-413 and 2-414). The profiles through the steel and timber bulkhead are compared with those through similar devices in Figure 3-19 (see Sec. III). Comparison of

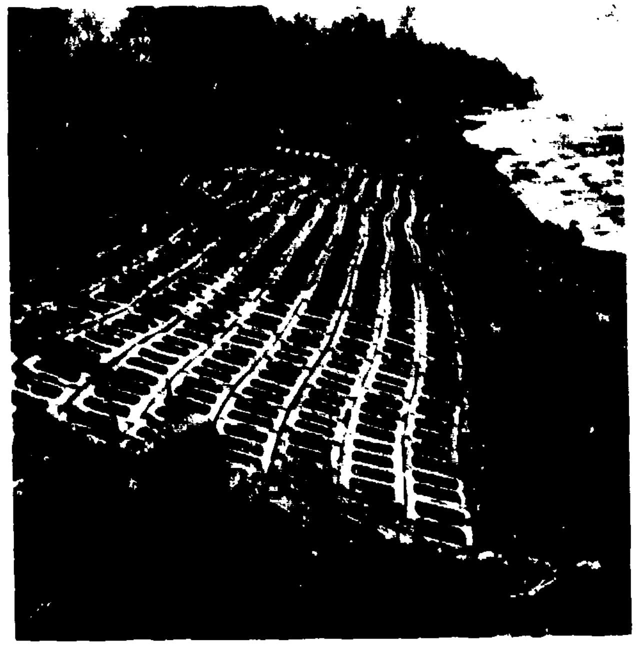


Figure 2-409. Large concrete-block revetment, device 3, Port Wing, Wisconsin, 7 May 1979.

Figure 2-410. Large concrete-block revetment, device 3, Port Wing, Wisconsin, 7 May 1979.

Figure 2-411. Tires separated from bulkhead section, device 4, Port Wing, Wisconsin, 16 August 1979.

Figure 2-412. Device 4 as of 13 May 1980, Port Wing, Wisconsin.

Figure 2-413. Steel and timber bulkhead, device 5, Port Wing, Wisconsin, 16 August 1979.

Figure 2-414. Steel and timber bulkhead, device 5, Port Wing,Wisconsin, 5 September 1979.

the profiles of June 1979 and October 1979 shows a small amount of accretion in front of the st...ture. No 1980 survey data were submitted but the photos adequately depict the performance of the structure. In May 1980 the structure was still intact.

(f) Upper Bank. Good grass cover was achieved by fall 1979 (Fig. 2-415), which stabilized the upper soil surface. However, grass plantings will do little to halt erosion due to overtopping waves because of the shallow nature of the roots.

(6) Analysis. Of the five devices monitored at Port Wing, the steel and timber bulkhead performed best; the control-block revetments performed well; and the Turfblock revetment and used-tire bulkhead were least effective. Offsetting the good performance of the steel and timber bulkhead was its high cost per foot, which exceeded the target maximum for the demonstration program. The control-block revetments were well within the desired cost range and could have matched the steel and timber bulkhead performance except for the abrasion the blocks suffered in the wave-impact zone and their somewhat uneven settlement. Abrasion and settlement caused some blocks to break. Some missing block exposed the filter. The abrasion might be lessened by improving the quality of the concrete, but in a wave and debris climate as severe as that at Port Wing, the best of concrete modules would probably be abraded to some degree. The uneven settlement of blocks was probably due largely to lack of compaction of the granular backfill and the numerous large stones it contained. The interlocking effect of the protrusions in the control blocks undoubtedly prevented more displacement than actually occurred.

Figure 2-415. Grass planting behind structural devices, Port Wing, Wisconsin, 15 October 1979.

The displacement of modules in the Turfblock revetment was probably initiated by uneven settlement of the granular fill, but there was also some evidence of slope failure of the natural bank material due to pore-pressure buildup. This could be due to clogging of the nonwoven filter material, but there was no way to prove that clogging actually occurred. In any event, the demonstration showed that the cheaper control blocks worked better. The deterioration of the used-tire bulkhead indicated the need for a better tire-interconnecting system than the spikes and pushnuts. Also, the use of concrete grout in lieu of granular fill in the tires might have preserved the structural integrity of the system better.

The stone revetments at Port Wing were not intended as demonstration devices; nevertheless, they performed very well and successfully protected the upper bank between the demonstration structures and at the two ends of the project. Although the cost per foot of stone revetment is somewhat high, they showed no signs of deterioration and protected the flanks of the monitored devices. The grading and grassing of the upper bank also was not intended as a demonstration project; however, it performed well, stabilizing the upper soil surface and protecting the highway.

c. Geneva State Park, Ohio.

(1) Site Description.

(a) Geographical Setting. The Geneva State Park demonstration site is located on the south shore of Lake Erie, near the mouth of Cowles Creek about 17 miles east of Fairport Harbor and 12 miles west of Ashtabula Harbor. The park is administered by the Ohio Department of Natural Resources, Division of Parks and Recreation. It has about 8,000 feet of lake frontage, of which the demonstration site occupies about 2,000 feet. The project location is shown in Fig. 2-416.

A large parking lot occupies the part of the park west of Cowles Creek. Between the parking lot and the lakeshore bluff is a large grassy area. The western two-thirds of the grassy area (which contains a bathhouse) is protected by a hinged, interlocking revetment of concrete slabs constructed in 1975. The surfaces of alternate slabs are tilted slightly (Fig. 2-417). The tilted slabs and those paralleling the graded slope (nontilted) are arranged in a checkerboard fashion.

A bridge provides access to the wooded recreation area east of Cowles Creek, where the lakeshore bluff rises about 20 feet above a sand and gravel beach. Erosion of the bluff has concerned park officials, and the demonstration project was devised to study structures that might protect this area and stabilize the mouth of Cowles Creek. Various types of offshore breakwaters were selected as devices to be demonstrated at this site. An existing structure, 600 feet east of Cowles Creek, was constructed before 1949 as a 70-foot-long seawall. The structure, now about 60 feet long, has been outflanked by erosion and lies approximately 90 feet from the bluff, acting as a breakwater (Fig. 2-418). This structure has trapped a considerable amount of littoral drift in its lee; this trapped material has acted as a groin and has thus increased the width of the updrift beach over a distance of about 300 feet.

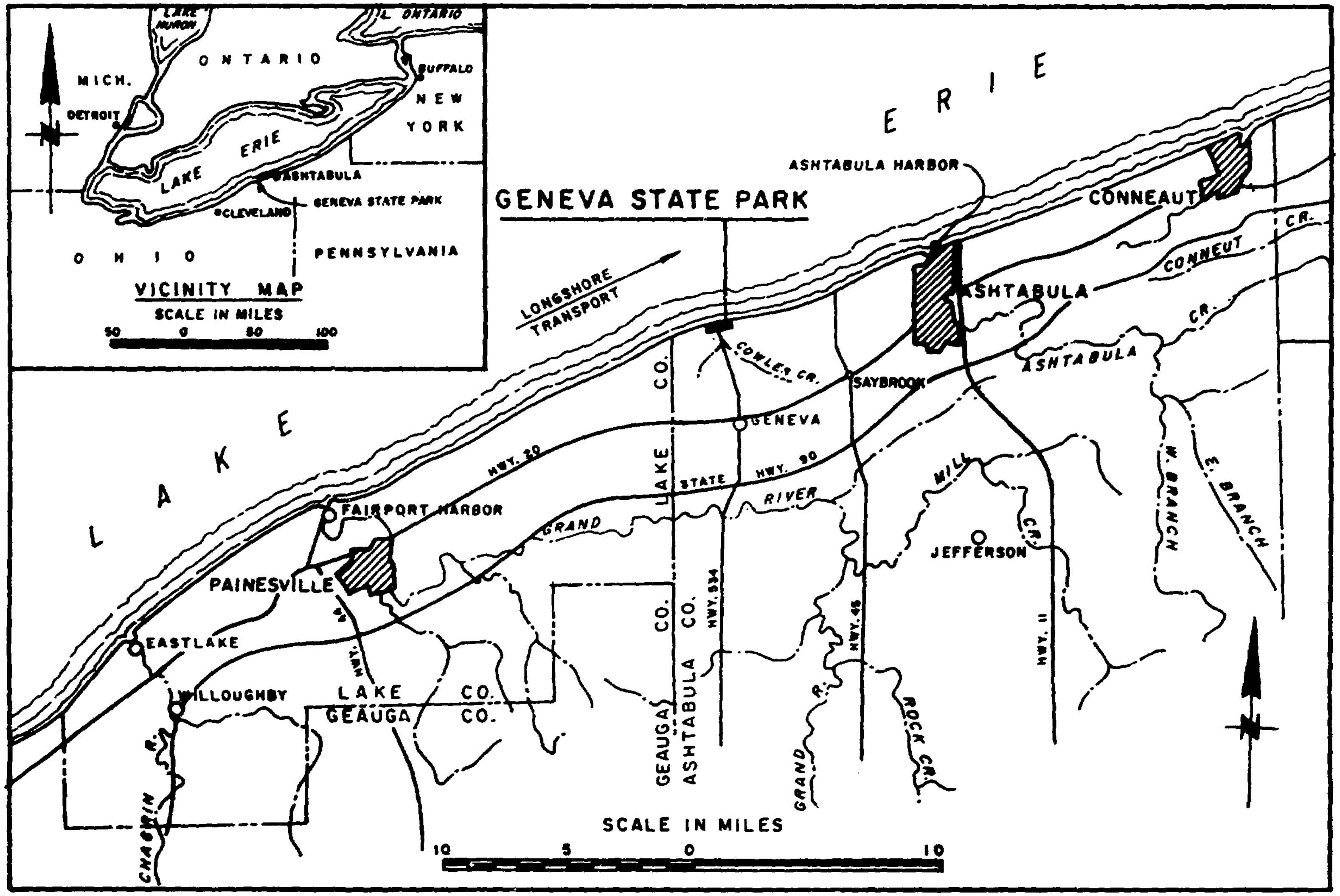


Figure 2-416. Location map of Geneva State Park, Ohio, demonstration site.

Figure 2-417. Existing checkerboard concrete-slab revetment west of Cowles Creek with existing west breakwater in the background, Geneva State Park, Ohio, 3 October 1978.

Figure 2-418. Existing breakwater structure east of Cowles Creek, Geneva State Park, Ohio, 1 November 1978.

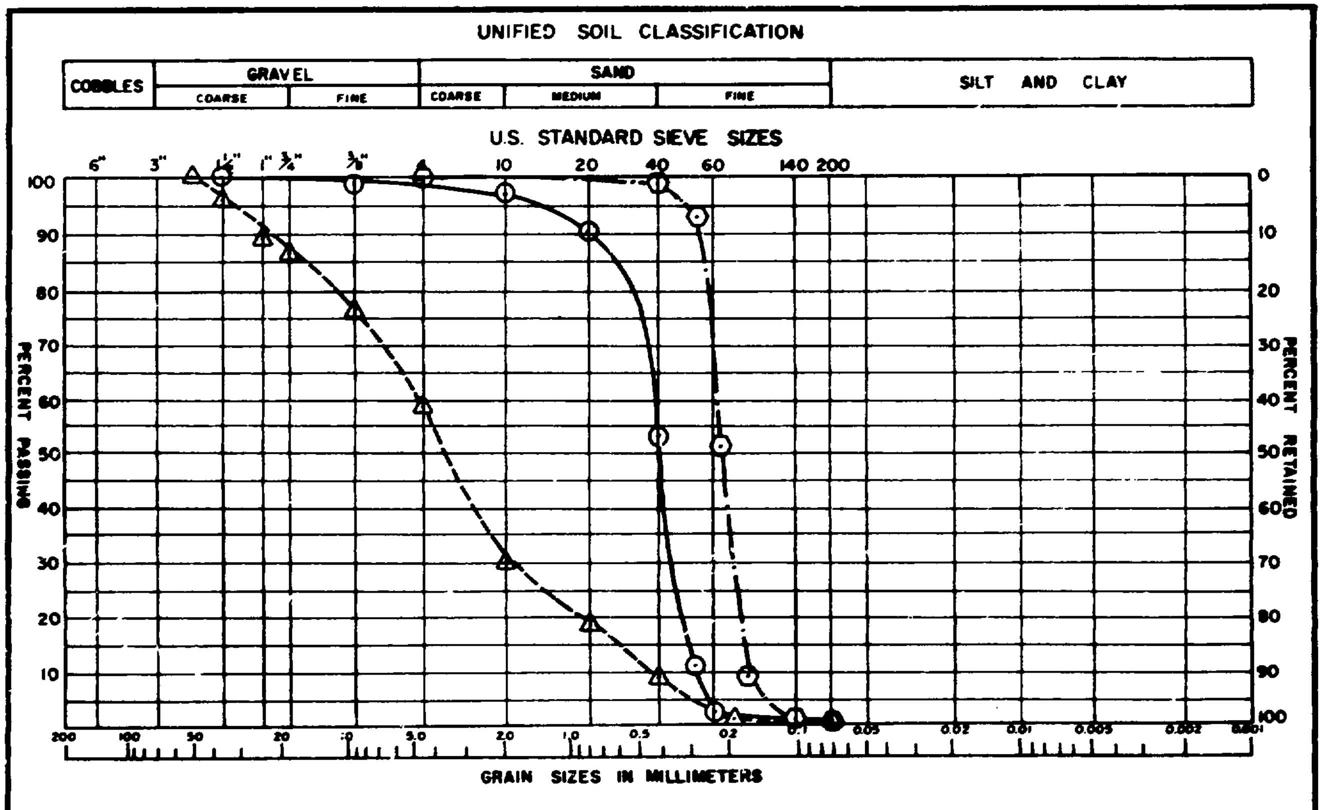
(b) Water Level, Wave Conditions, and Longshore Transport. Water levels in Lake Erie during the monitoring period were about 1 foot above the long-term average and about 3 feet above LWD, which is 568.6 feet IGLD. Seasonal fluctuations average about 1.5 feet. Because the lake is shallower than Lakes Superior, Michigan, and Huron, wind setup tends to be somewhat greater. A 1.5-foot setup occurs about once a year; a 1.9-foot setup occurs about once every 5 years. The park shoreline orientation is about N.70° E., and winds from the west and northwest predominate slightly over those from the northeast. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 4.0 feet, and the net longshore transport potential was 104,600 cubic yards eastward for the 4 months analyzed. The wave climate is classified as severe.

(c) Geomorphology, Soils, and Vegetation. The project is located in an area covered by the Erie Ice Lobe during the Wisconsin Glaciation, and was influenced by the effects of melt-water lakes formed as the glacier retreated and readvanced. Several stages of postglacial lakes existed, creating a series of beach ridges of sand and gravel as the glacial lake levels changed. These beach ridges are present today and parallel the shore up to 5 miles inland from the existing shoreline. In most of the Lake Erie Basin, shales of Upper Devonian age are overlain by glacial till deposited as the ice lobe retreated. After formation of melt-water lakes, readvance of the ice lobe caused lake levels to fluctuate. As a result, much of the till was overlain by lacustrian silt and clay deposits in the offshore areas and sand deposits in the nearshore zones.

In the project area, glacial till, lacustrian silts and clays, and sandy beach ridge deposits exist. Bedrock shales are exposed in the offshore areas from the 40-to 3-foot depth. From the 3-foot depth to the LWD, the rock surface is covered by a thin, transitory layer of fine sand. Shoreward of the LWD, the bedrock is from 0.5 to 4 feet below the surface, and is overlain by medium to very coarse-grained, subangular to subrounded, well-sorted lithic and quartz sand. The average grain size of the sand decreases offshore. The bluff contains intermittent lenses or layers of sand, and in localized areas it is comprised entirely of sand or lacustrian silt. The glacial till forming the the bluff contains about 80 percent silt and clay, 15 percent sand, and 5 percent gravel. The lacustrian silts contain less than 5 percent sand and gravel-sized particles. On the average, 25 to 30 percent of the material comprising the bluff is sand. Figure 2-419 illustrates the gradations of samples collected from the bluff, the beach at the waterline, and the offshore at station 107+00. At this location the bluff is primarily sand. Figure 2-420 illustrates the gradations of samples collected at station 110+00, where the bluff is primarily silt and clay. The two figures show the range of materials that exist in the bluff at different locations; materials on the beach and offshore are similar at both locations.

The width of the beach at Geneva State Park generally is 2 to 60 feet and slopes at 4° to 5°. The bluffs at the site are approximately 20 feet high.

The intermittently wooded area east of Cowles Creek has stands of white oak (*Quercus alba*), basswood (*Tilia americana*), beech (*Fagus* sp.), and ash (*Fraxinus* sp.) Grassy areas provide recreational opportunities for park visitors. The planted grass in the area west of Cowles is becoming well established.



LOCATION	STATION	SYMBOL	LL	PI	CLASSIFICATION
Top of Bluff	107 + 00	⊙———⊙			Medium SAND (SP)
Water Line	107 + 00	△- - - -△			Gravelly SAND (SW)
Offshore	107 + 00	⊙— · —⊙			Fine SAND (SP)

Figure 2-419. Representative grain-size distribution curves, Geneva site.

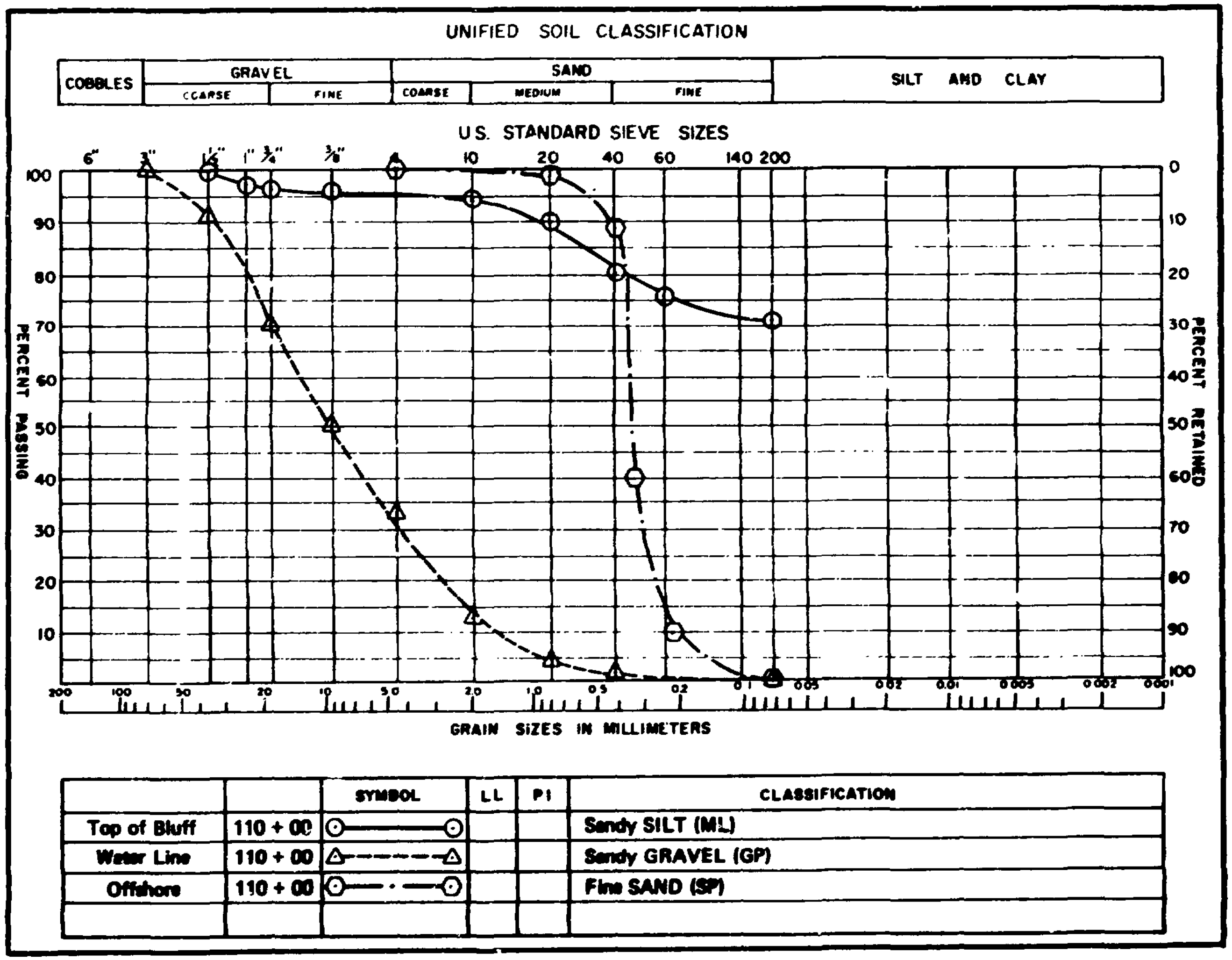


Figure 2-420. Representative grain-size distribution curves, Geneva site.

(d) The Problem. This site is a typical example of erosion caused primarily by wave action and bluff slumping. In general, wave erosion is the more significant process. The eroded material is removed by longshore transport, which promotes additional bluff recession. The nature of the slumping is strongly influenced by the nature of the bluff material. In clayey tills, water percolates down from the overlying ground surface or runs along silty seams, saturating the clay. This results in saturation of the soil mass and creates seepage forces which, along with the steepness of the bluff, reduce its stability. This instability results in slope failure along the bluff face.

The well-sorted lacustrine silts fail mostly as small rotational slumps or by block failure. Tension cracks form behind the surface of the bluff due to the steepness, surface unloading, and soil expansion and contraction. The bluff face deteriorates as downward-percolating water loosens blocks of soil, and gravity causes them to fall. The process is accelerated during high lake levels, when the bluff base is undercut by wave attack and support to the overlying bluff face is lost.

On the average, the bluffs in the Geneva State Park area are 10 to 20 feet high and are receding at a rate of less than 1 foot per year. However, this recession rate varies with time and location along the shoreline. During a year of high lake level, many feet of bluff may be lost and the recession rate may exceed 10 feet per year, whereas during a year of low lake levels, the recession rate may drop to zero. A particularly high, steep bluff may recede quite rapidly, while a nearby low, vegetated bank may show no visible recession over the same period of time.

(2) Demonstration Project. The selected plan comprised three short offshore breakwaters and a vegetation planting scheme (Fig. 2-421).

(a) Device 1 (Gabion Breakwater). A gabion breakwater about 100 feet long was constructed just west of the mouth of Cowles Creek. A plan of the gabion combination is shown in Fig. 2-422; details of the various sections of the breakwater are shown in Figs. 2-423 and 2-424. The wire baskets were vinyl-coated in the western half of the breakwater and galvanized in the eastern half, and all were filled with stone graded in size between 5 and 9 inches. To prevent sand from working up into the gabions, filter cloth was first laid along the bottom in all but the eastern third of the structure, which was left as a control section by which to judge the effectiveness of the cloth. Mattress-type gabions along the lakeside toe and at the ends of the structure were provided to prevent undercutting of the base modules. The gabion breakwater was intended to trap littoral drift and to stabilize the shoreline between the east end of the existing concrete revetment and the mouth of the creek.

(b) Device 2 (Sta-Pod Breakwater). About 300 feet east of device 1, a 96-foot breakwater was constructed of Sta-pods which are special concrete units consisting of four inclined legs attached to a cylindrical trunk (Fig. 2-425). Each Sta-pod is about 5.5 feet high and

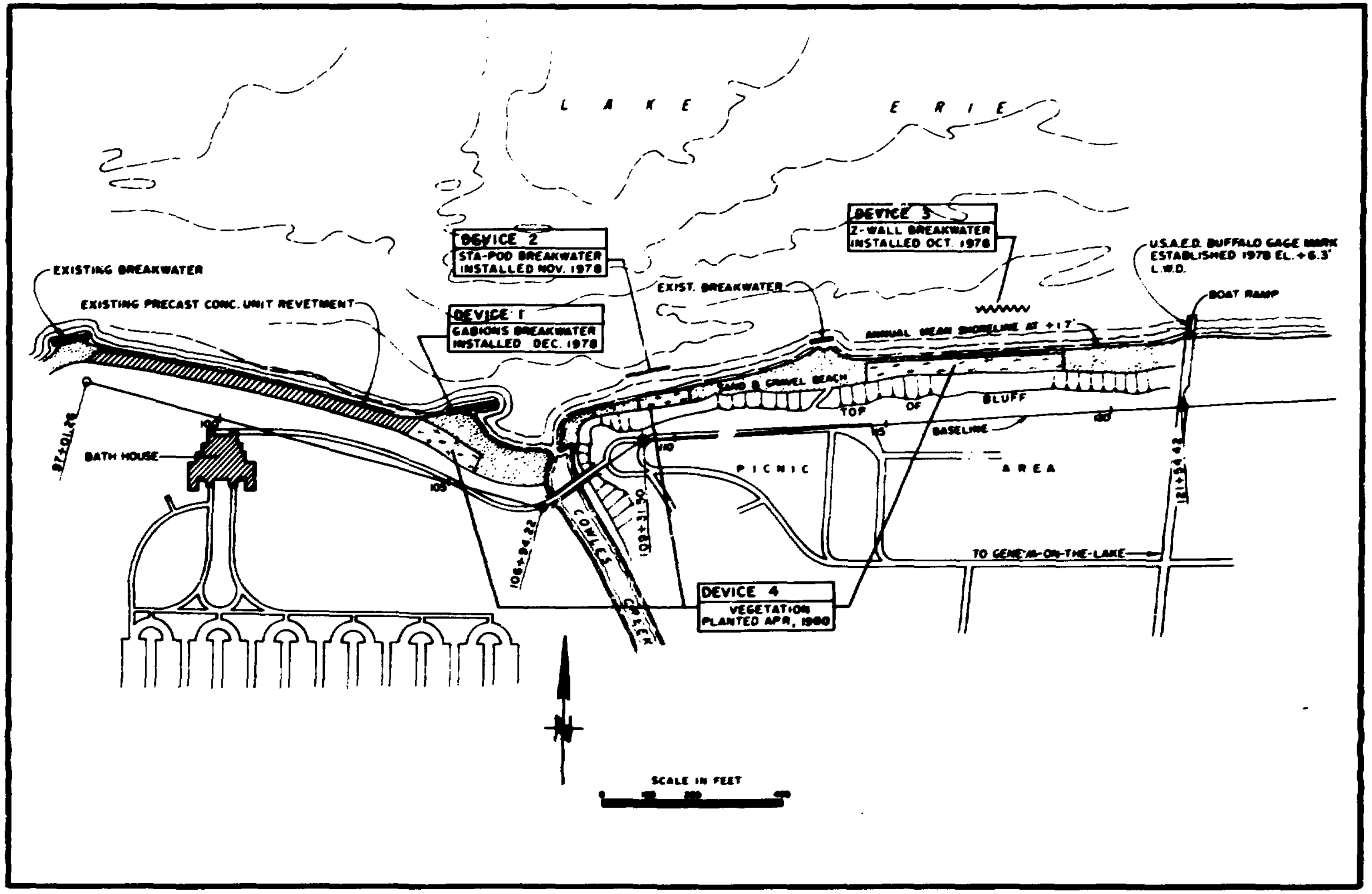
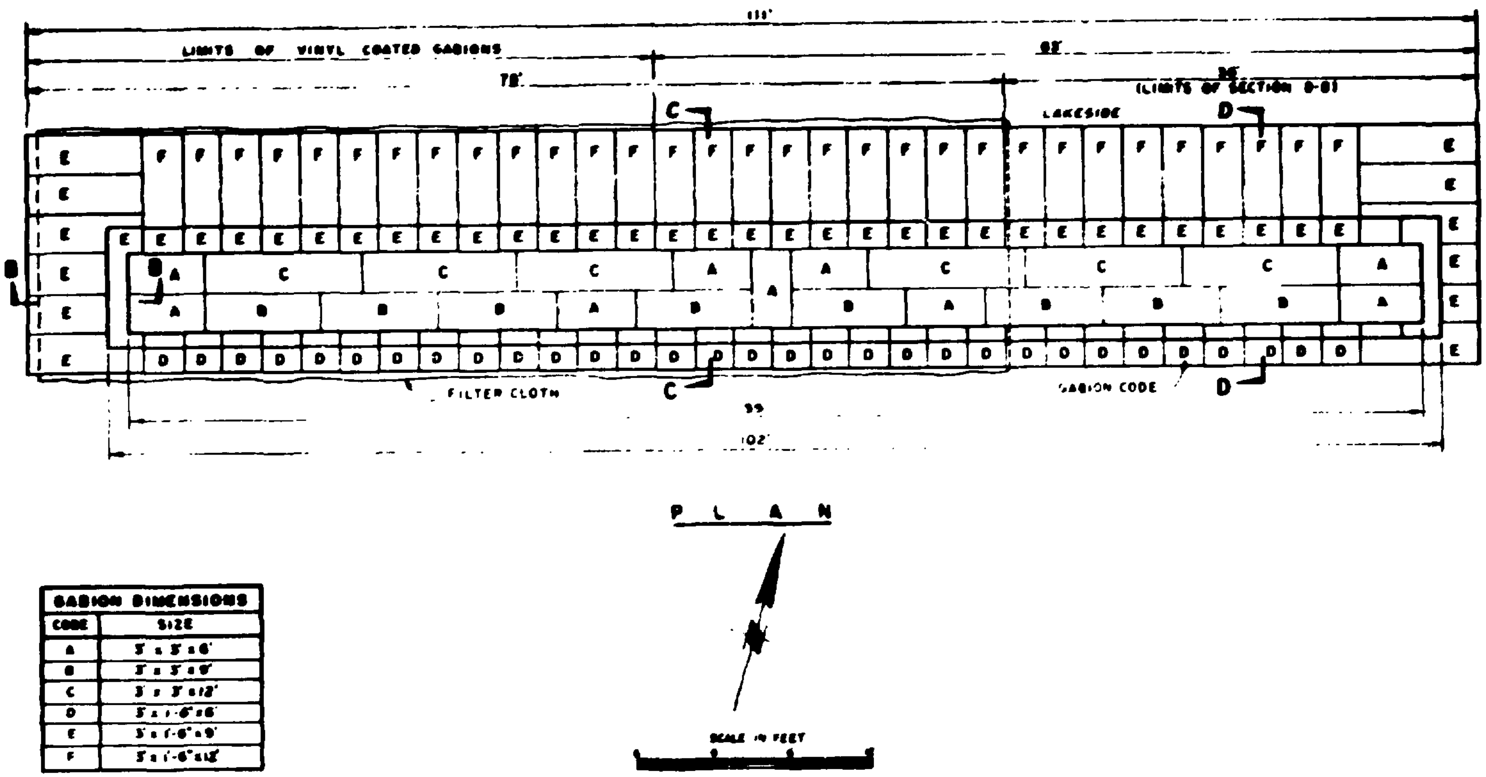


Figure 2-421. General plan of Geneva State Park, Ohio, demonstration site.



GABION DIMENSIONS	
CODE	SIZE
A	3' x 3' x 6'
B	3' x 3' x 9'
C	3' x 3' x 12'
D	3' x 1'-6" x 6'
E	3' x 1'-6" x 9'
F	3' x 1'-6" x 12'

Figure 2-422. Gabion installation at Geneva site.

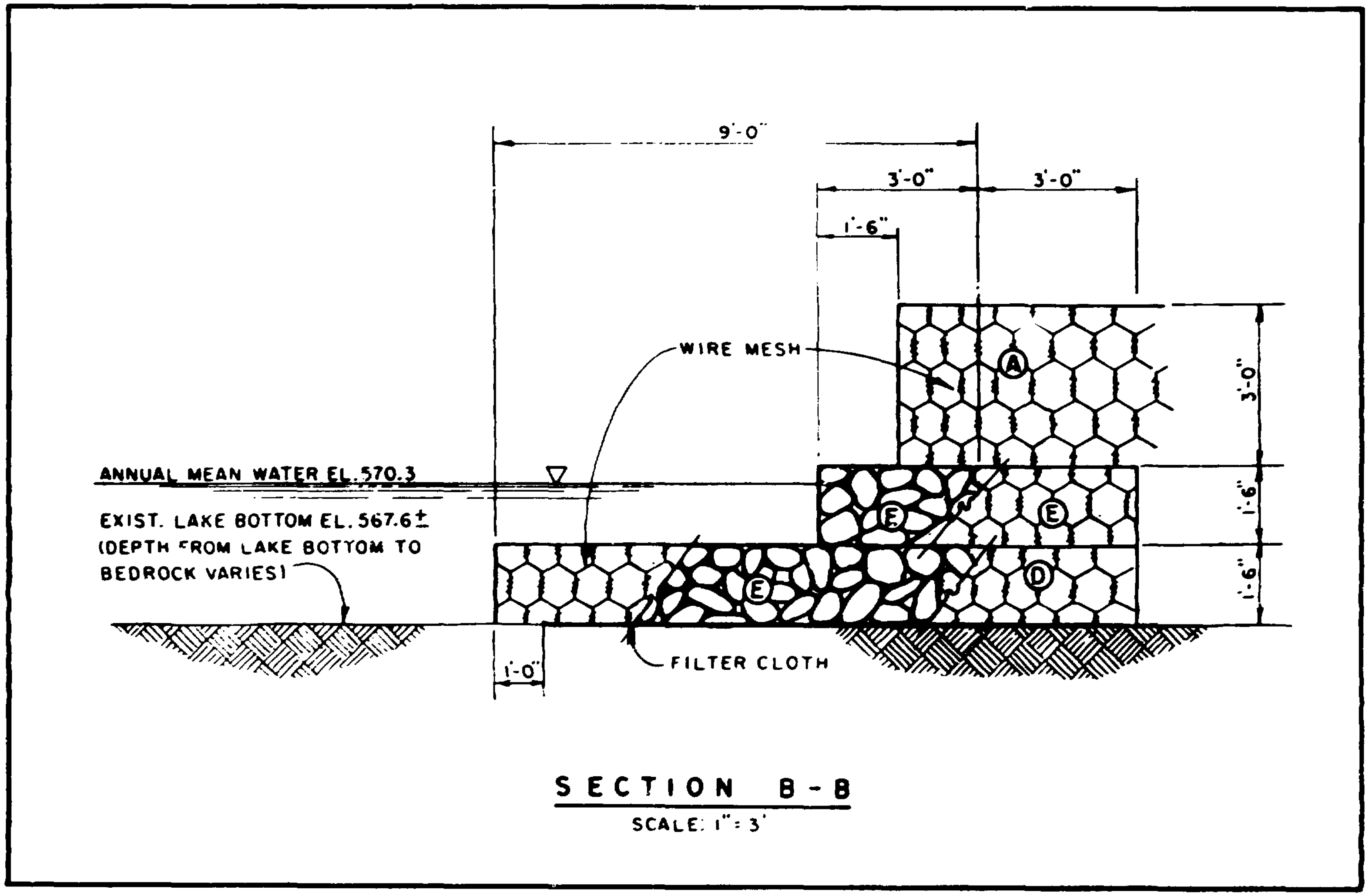


Figure 2-423. Gabion breakwater at Geneva site.

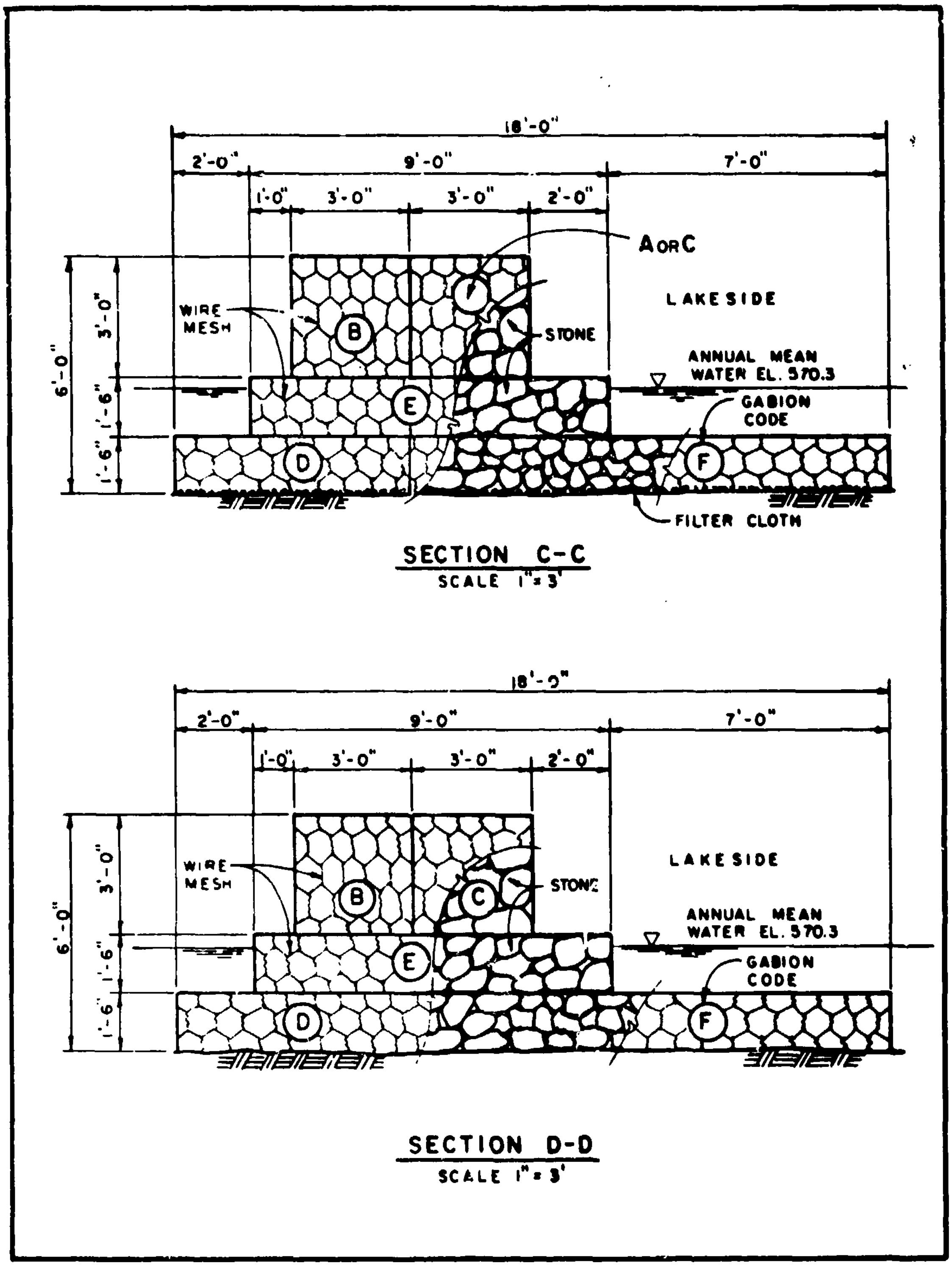


Figure 2-424. Gabion breakwater at Geneva site.

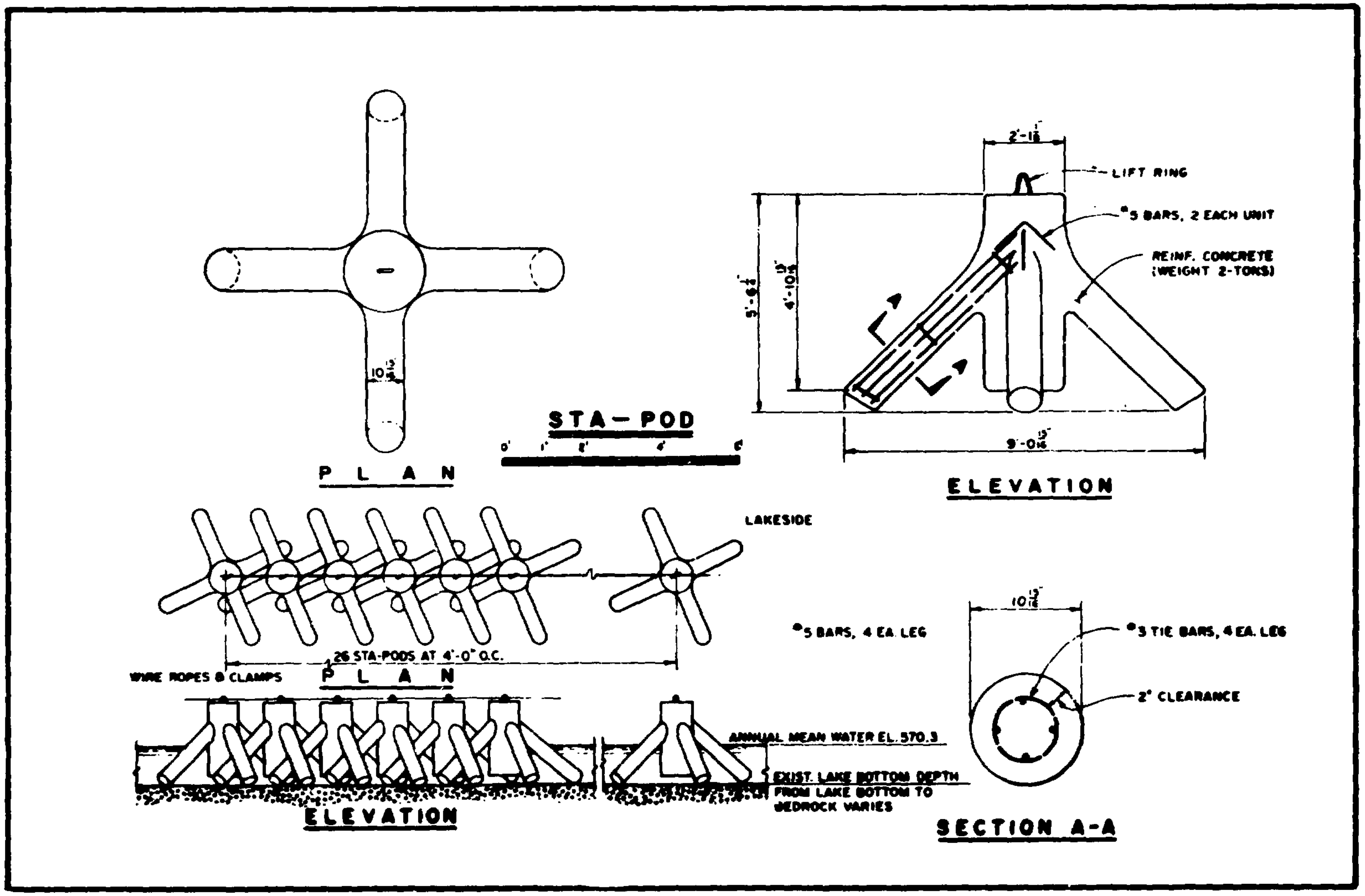


Figure 2-425. Sta-pod breakwater at Geneva site.

weighs about 2 tons. Twenty-six units were placed side-by-side in a row about 60 feet from shore in about 3 feet of water. The westerly 13 Sta-pods are linked together through the lift rings with 5/8-inch galvanized-steel wire rope. This is done to test the individual stability of the Sta-pods. The purpose of device 2 was to reduce wave height in its lee, trap littoral drift, and build cut the beach east of Cowles Creek, thereby protecting the toe of the bluff with a sand berm.

(c) Device 3 (Z-Wall Breakwater). About 700 feet east of device 2, a 98-foot Z-wall breakwater was constructed of steel-reinforced concrete panels set on edge in zigzag fashion and joined together with large hinge bolts (Fig. 2-426). Device 3 is composed of 14 panels, each 6 feet high, 14 feet long, and weighing 6.5 tons. The structure was placed about 100 feet from shore in about 3 feet of water. Device 3 tested the manufacturer's claim that wave uprush into the V-junctions of the panels at the back of the structure would jet sand over the top into the area behind it, building up a back berm.

(d) Device 4 (Vegetation Planting). Combinations of vegetation species were planted behind or near each newly installed breakwater to determine the ability of these plantings to assist in stabilizing sand trapped by the breakwaters (see Table 2-99).

(3) Statistics, Costs, and Construction. Statistics and costs for the Geneva State Park demonstration project are given in able 2-99.

The three demonstration breakwaters were installed in east-to-west order. The Z-wall panels were placed with a crawler crane, which held each new panel in its design position as it was bolted to the previously placed panel. No special preparation was done to the lake bottom before placing the panels. The work has completed in October 1978. The Sta-pods were also placed with a crane, construction being completed in November (Figs. 2-427 and 2-428). The gabion baskets were then wired open, ready for placement in section (Fig. 2-429). After the base was graded and the filter cloth laid, the bottom tier of baskets was placed in sections, filled from the graded-rock stockpile by a crane (Fig. 2-430). The basket was then wired shut. Construction of the second and third tiers followed in order. Work was completed in December 1978. All plant materials for device 4 were planted between early April and early June 1980. The plantings required 560 pounds of Osmocote 18-6-12 fertilizer. Pickerelweed and reed canarygrass were supplied by the contractor for $504 and $476, respectively, and were dug within 2 weeks before planting. The contract cost included staking out and labeling of the three plots.

(4) Performance.

(a) Structural devices. Ice locked in all three structural devices shortly after completion of the gabion breakwater and secured the shoreline against wave attack until the spring thaw in March 1979. The monthly inspection of the site on 5 April 1979 revealed that the gabion baskets had been slightly deformed by the weight of the ice. The

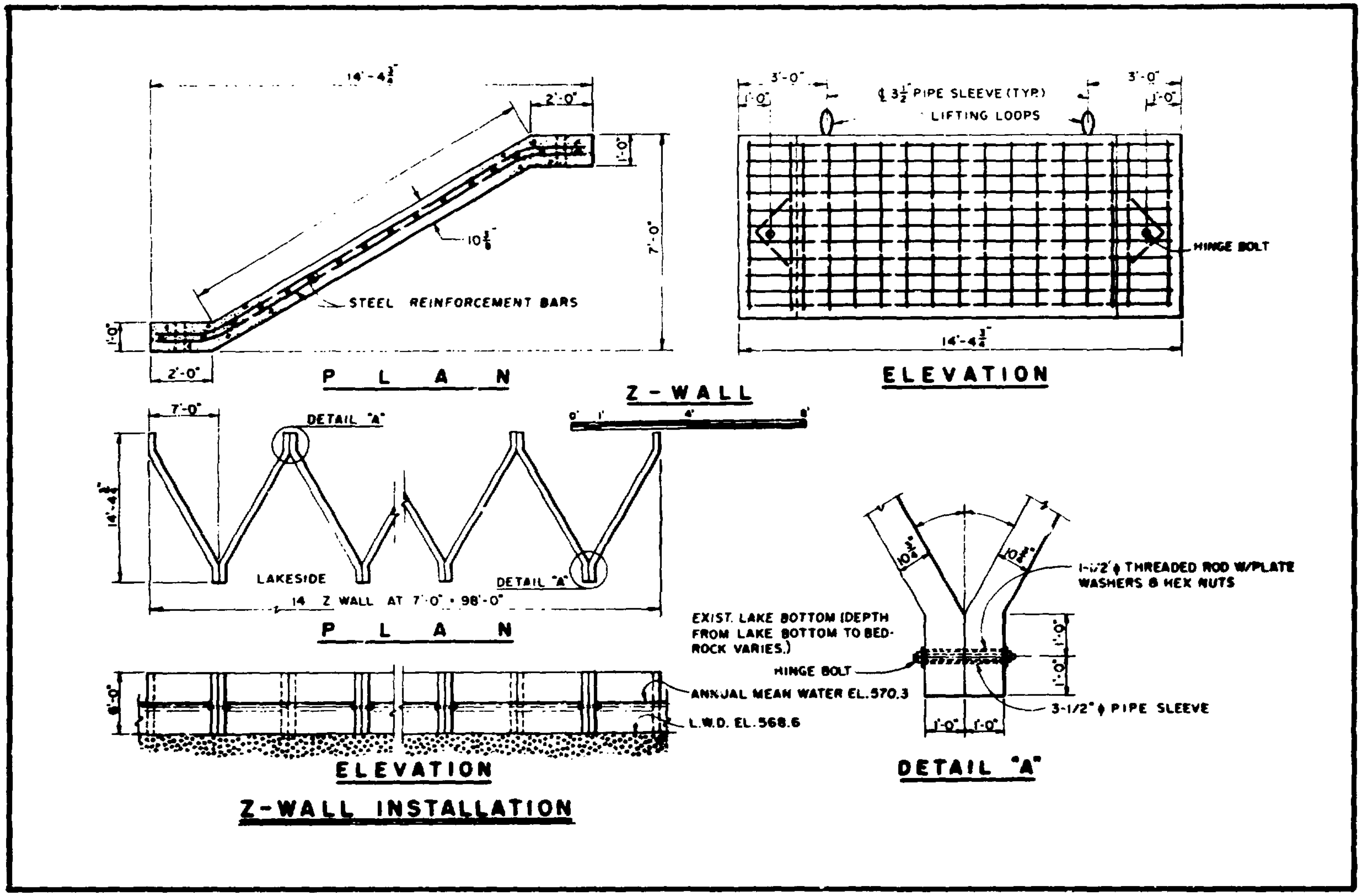


Figure 2-426. Z-wall breakwater at Geneva site.

Table 2-99. Statistics and costs for Geneva State Park, Ohio.

Item	Quantity	Cost/ft
Structures (contract cost $67,200)		
Gabions	230 yd^3	$350
Sta-pods (precast)	26 units	116
Z-wall units (precast)	14 units	206
Labor (man-hours expended in construction)		
Superintendent	310	
Foreman	255	
Surveyor	64	
Operator	320	
Truck driver	24	
Laborer	900	
Diver	60	
Oiler	296	
Equipment (total hours)		
Crane	296	
Loader	24	
Truck	24	
Air compressor	20	
Generator	20	
Vegetation (contract cost $11,688)		
Three-square (*Scirpus americanus*)	1,313 pots (four stems/pot)	
Pickerelweed (*Pontederia cordata*)	504 pots (33 stems per/pot)	
Broadleaf cattail (*Typha latifolia*)	504 hills (three stems/hill)	
Smooth cordgrass	504 pots (four stems/pot)	
Saltmeadow cordgrass	1,388 pots (four stems/pot)	
Reed canarygrass (*Phalans arundinacea*)	1,302 pots (four stems/pot)	
Switchgrass (*Panicum virgatum*)	1,302 pots (four stems/pot)	
American beachgrass	7,444 hills (three stems/hill)	
Osmocote 18-6-12 fertilizer	560 lb	

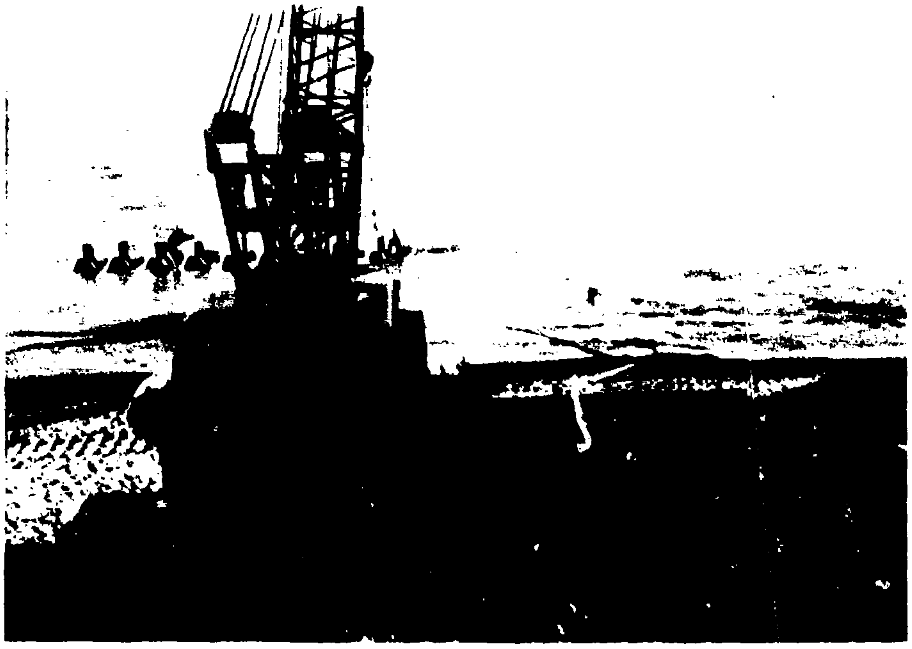


Figure 2-427. Placing Sta-pods, Geneva State Park, Ohio, 1 November 1978.

Figure 2-428. Sta-pod unit, Geneva State Park, Ohio, 1 November 1978.

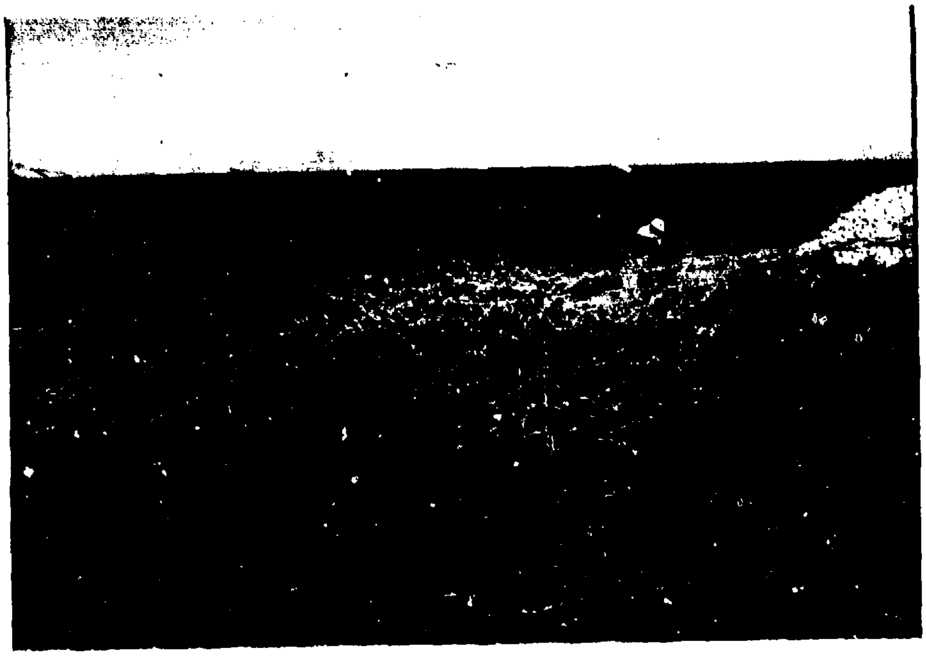


Figure 2-429. Gabion baskets ready for placement, Geneva State Park, Ohio, 1 November 1978.

Figure 2-430. Graded-rock stockpile, Geneva State Park, Ohio, December 1978.

Sta-pods were undamaged, but the easternmost panel of the Z-wall had fallen off shortly after construction during the fall of 1978. On 6 April 1979, 50- to 60-mile-per-hour winds over Lake Erie generated waves that produced 5-foot breakers along the shoreline in the park area. A storm damage inspection on 12 April revealed that some gabion baskets were damaged and rockfill had been lost. Some of the top tier of gabions were misshaped, causing slight depressions in the profile, and the center of the structure had been pushed several inches shoreward. The westernmost panel of the Z-wall was rotated slightly and was leaning on the adjacent panel. No other structural damage was apparent.

The fallen east panel of the Z-wall was reinstalled in mid-May, but it fell off again about 1 week later, apparently as a result of the hinge bolt working loose as waves agitated the panel. The leaning west panel finally fell off in late June during a brief storm that produced near-design waves for a short period of time. Uneven panel settlement caused concrete to spall off the inner ends of the eastern panels, baring the reinforcing steel (Fig. 2-431). In late September 1979, the second panel on the east end dropped off, leaving 11 panels still standing.

During the spring and summer of 1979 the lakeside of the gabion breakwater dropped deeper as a result of deterioration of the top layer of gabions, and the eastern end, lacking a filter-cloth base, settled a few inches. Some of the baskets were cut open at joints, and the wire mesh of others was elongated on the lakeside as the toe was undercut and rotated downward. Three baskets on the lakeside were cut open and lost most of their rockfill (Fig. 2-432). The western end of the Sta-pod breakwater settled from 6 inches to 1 foot, but no units were damaged.

The lakeside baskets of the gabion breakwater deteriorated rapidly during the fall of 1979 as a result of toe scour, wire stretching, and corrosion. The galvanized baskets in the eastern two-thirds of the structure failed first; then the PVC-coated baskets. Before the ice cover in December 1979, all but the exposed westerly end gabions on the lakeside were broken open and the rockfill washed out (Fig. 2-433). The landward side of the structure remained intact except at the eastern end where the lack of filter cloth hastened settlement, resulting in wave destruction of both landside and lakeside gabions (Fig. 2-434).

The performance of the demonstration breakwaters is best shown by the aerial photos in Fig. 2-435, one taken after the 1979 spring thaw, one near the end of summer 1979, and one after the 1980 spring thaw. During the first year, the gabion breakwater effectively controlled erosion at the mouth of Cowles Creek and built the beach out to about 50 feet, leaving a narrow waterway as the spit of sand and gravel accumulated across the creek delta, moving the outlet about 150 feet eastward. For a short time in late summer 1979, this east outlet closed and the creek flowed through the old channel westward scouring a channel behind the breakwater and around its west end into the lake. This west outlet became closed in November 1979 and soon filled with littoral material as the creek reopened its east outlet. However, as the east end of the structure deteriorated, its functional effectiveness decreased. By April 1980, most of the delta material had disappeared.

Figure 2-431. Concrete spalled off Z-wall panels due to uneven settlement; later the panel on the right fell off, Geneva State Park, Ohio, 6 September 1979.

Figure 2-432. Gabion basket cut, 90-percent loss of rockfill, Geneva State Park, Ohio, 6 September 1979.

Figure 2-433. Deterioration of the lakeside gabions, Geneva State Park, Ohio, 11 December 1979.

Figure 2-434. Deterioration of the east-end gabions, Geneva State Park, Ohio, 11 December 1979.

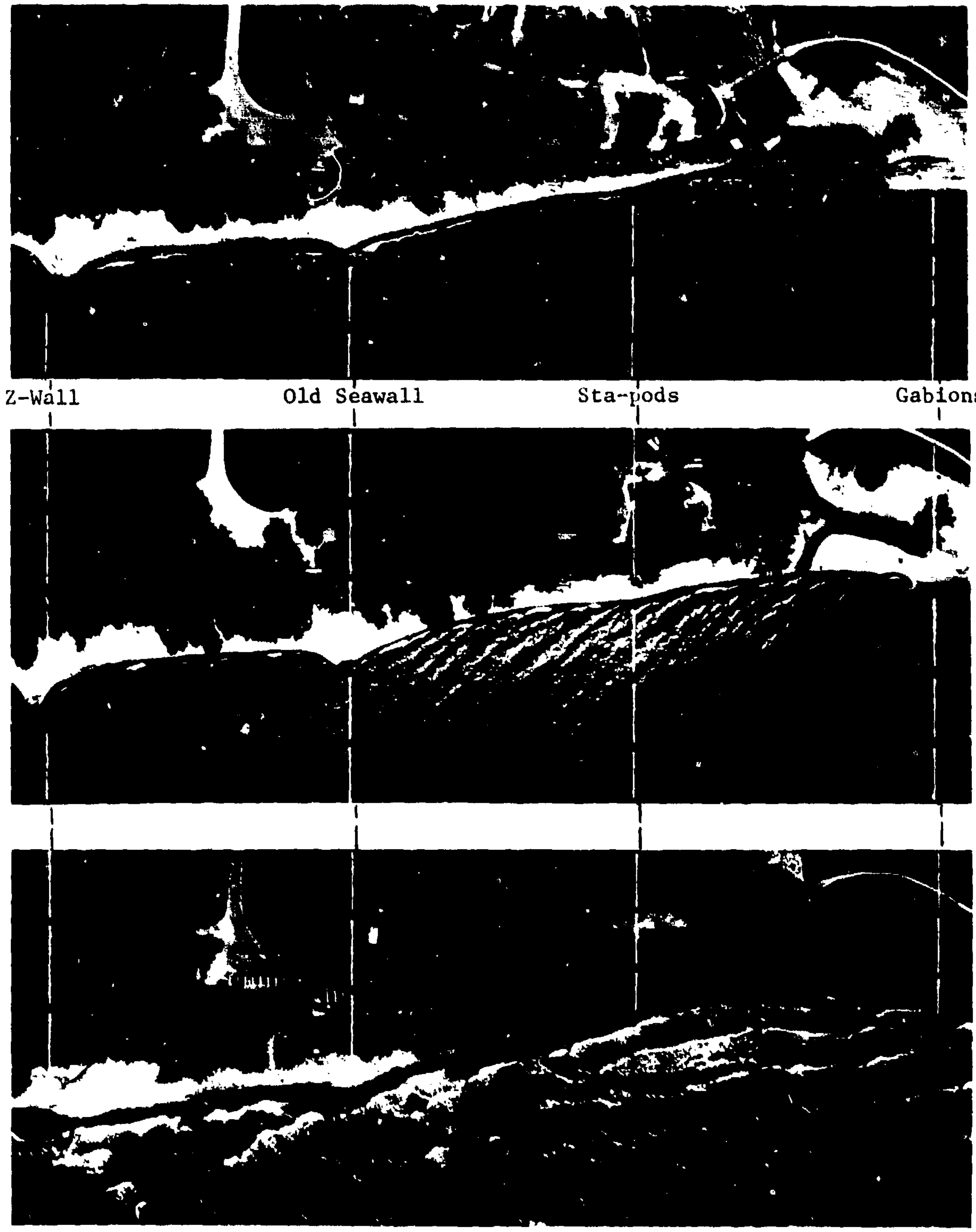


Figure 2-435. Functional performance of the demonstration breakwaters,Geneva State Park, Ohio, (top photo, 15 May 1979; middle photo, 30 August 1979; bottom photo, 23 April 1980).

Not so apparent in the 1979 summer photo is the 20-foot increase of the beach behind the Sta-pod breakwater. Notwithstanding the large void-ratio of this structure, it did absorb some of the wave energy and may have been partially effective in protecting the shoreline. However, the 1980 photo shows waves again lapping at the toe of the bluff. The existing block breakwater remained effective in spite of its deteriorated condition. The Z-wall breakwater was constructed early enough in 1978 to trap considerable material in its lee before the winter freeze. The partial tombolo it created remained relatively stable through the spring of 1980 in spite of the loss of three panels. Early profiles through the Z-wall and Sta-pod breakwaters are shown in Figure 3-78 (see Sec. III). The 1980 survey data were received too late for inclusion in the profile comparison, but the aerial photos adequately depict the performance of all three structural devices.

(b) Vegetation. Photos of the vegetation plantings were received too late for reproduction in this report. However, by June 1980, storms had taken the lower plots of vegetation behind the gabions, as well as all of the beachgrass that had been planted behind the Sta-pods in April. Lakeward vegetation plots of cattails in lee of the Sta-pod breakwater were lost to erosion during May 1980. As of August 1980, approximately 10 percent of the original plantings existed.

(5) Analysis.

(a) Device 1 (Gabion Breakwater). The gabion breakwater was very effective in trapping littoral material and demonstrated an ability to perform well. Even though deformed in places by settlement and by wave forces, the breakwater remained functionally effective for more than 1 year. Its design was well conceived, the toe mattress dropping into the anticipated scour trench and protecting the foundation of the main body of the structure from wave scour. However, the baskets were severely deformed and the wire mesh was strained to the breaking point. The PVC-coated baskets lasted longer, but eventually broke open. The test showed that gabion baskets are not strong enough to withstand the rigors of the wave environment at this site. The sinking of the east-end gabions demonstrated the need for the filter cloth that prevented settlement of the structure elsewhere. The high cost of gabions at this site ($350 per linear foot) was due to the use of contract construction and the need to import the rockfill. Where rockfill of the proper gradation is available, a property owner could erect and fill the baskets himself at a fraction of the cost. However, the demonstration indicated that gabions should not be used in this severe wave climate. In addition, the manufacturer recommends tight packing and repacking of the baskets shortly after construction. These were only partially filled and were therefore more flexible, which could have affected the performance at the gabion breakwater.

(b) Device 2 (Sta-Pod Breakwater). Although the least functionally effective of the three structures installed at this site, the Sta-pod breakwater suffered no damage and was the least costly (only $116 per linear foot). Perhaps some type of interconnecting baffle could be devised for use with such a structure to reduce its permeability. The Sta-pod units can be placed rapidly and would be useful for emergency protection. Access to the site by a crane would, of course, be essential.

(c) Device 3 (Z-Wall Breakwater). This installation demonstrated the vulnerability of the end panels of the Z-wall to design wave action. As noted by the District monitor, a heavier or more stable end panel is needed. The single-hinge bolt by which the panels are interconnected allows adjacent panels to settle nonuniformly but with limited tolerance. Where the tolerance limit is exceeded, adjacent panels tend to lean against or pull apart from each other, causing the concrete to spall in stress zones. The nut on the hinge bolt tends to loosen and unwind under wave agitation. A better hinge or a device to limit the movement of adjacent panels with respect to each other is needed. Near the end of the monitoring period, the District monitor noted that the force of the waves tended to open the landward panel joints and force debris into the gaps, causing stress concentrations. It is probable that continued exposure to these wave forces will cause the successive breakdown of end panel joints and the resultant successive loss of panels from the ends toward the center of the structure. The good functional performance of the Z-wall is worth further consideration if its structural deficiencies can be overcome, but the relatively high cost ($206 per linear foot) is somewhat excessive for a desirable low-cost structure. It is believed that all three structures, especially Sta-pods and Z-walls, are sensitive to bottom conditions with respect to their stability. The relation of bedrock to the structures can greatly influence structural stability.

(d) Vegetation. The early loss of most of the plantings indicates that the wave climate at Geneva State Park may be too severe for effective use of vegetation without considerably more structural complementation than was provided at this site.

d. Ashland, Wisconsin.

(1) Site Description.

(a) Geographical Setting. The Ashland monitoring program was conducted at Madigan Beach on Lake Superior in the Bad River Indian Reservation about 16 miles east of Ashland, Wisconsin. The shoreline along the project site is straight and orientated northwest to southeast. The protective devices were installed along 1,350 feet of shoreline about 2,000 feet northwest of Morrison Creek and 2,000 feet southeast of Nawagen Creek (Fig. 2-436).

(b) Water Level and Wave Conditions. Fluctuations of the water level in Lake Superior are shown in Figures 2-385 and 2-386. Superimposed on the annual fluctuation are the short-term level changes caused by differential atmospheric pressures and by wind setup. These effects can alter the water level along the shore by 2 feet or more, but at this site, by not more than 1 foot above or below the undisturbed lake level. The design water surface level of 602.9 feet was obtained by adding the mean of the monthly levels over the past 10 years the maximum recorded 1-year short-term rise.

Waves that impinge on the project shoreline are wind-generated over long fetches. The Apostle Islands north of the site offer some protection from north-northwesterly storm waves. However, the site is directly exposed to northeasterly storm waves with heights of 4 to 5 feet and periods of about 5 seconds. The shoreline is oriented approximately N. 55° W., and normally, waves heights are about 1 foot with 3-second periods. Although no LEO station was established at this site, the wave climate is classified as severe. Energy-flux analysis could not be provided because of insufficient data. However, sand accumulations on the southeast sides of groins indicate a predominant northwestward longshore transport.

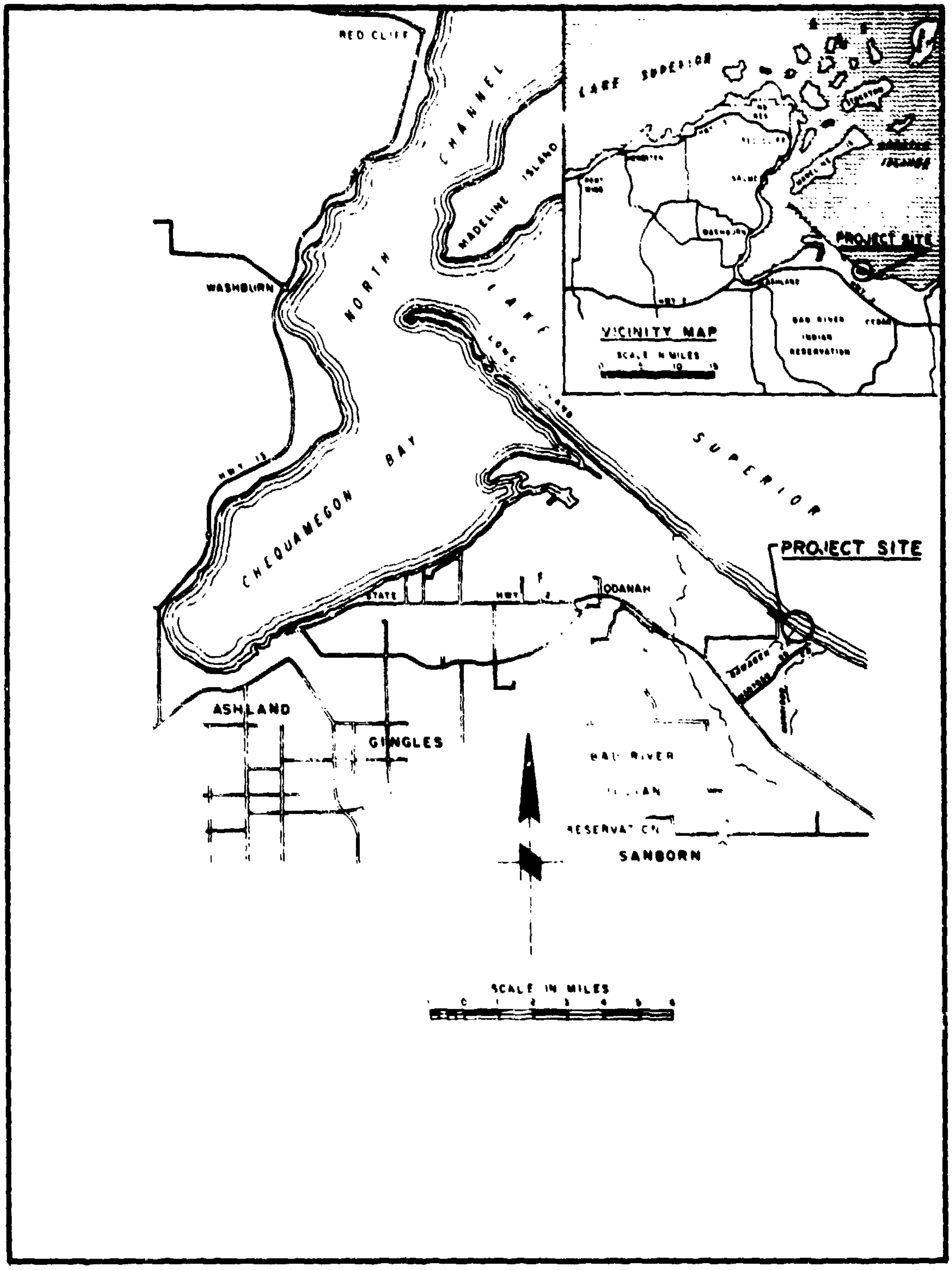


Figure 2-436. Location map of Ashland, Wisconsin, monitoring site.

(c) Geomorphology and Soils. The Ashland monitoring site is characterized by a narrow beach backed by 60- to 80-foot-high clay bluffs. The bluff generally has an inclination of 1 on 1.5 or steeper. The beach is very narrow to nonexistent in the study area. The bluff is composed of three basic materials. A 15- to 20-foot-thick layer of stiff, reddish-brown silty clay of low plasticity at the top of the bluff is underlain by a very dense brown silt layer more than 40 feet thick. Underlying the sandy silt is a stiff, reddish-brown clay of high plasticity. This lower clay layer is below the water surface and is generally not exposed. Grain-size analysis of these materials indicate that the top clay layer is of low plasticity and contains 64 percent silt and 26 percent clay; the cohesionless sandy silt layer is composed of very fine sand and coarse silt, and the lower clay layer is highly plastic and contains 26 percent silt and 64 percent clay.

The offshore lake bottom is composed of well-graded sand and slopes approximately 1 on 50.

(d) The Problem. The site at Ashland is a typical example of lake bluff erosion. The rate of erosion (estimated from 1951 and 1974 aerial photos) varies from a negligible amount to approximately 2 feet per year. The basic erosional process at the site is by direct wave attack on the lower layer of sandy silt. The waves generally approach normal to the shore and there is little or no beach to absorb energy. The silt material also has little strength when wet and is easily eroded by moderate wave action. As the base of the bluff is eroded and undercut, the upper layer of clay sloughs or fails until it again reaches a stable slope. Secondary erosional processes consist of mass wasting due to freezing and thawing, surface runoff, and groundwater seepage.

(2) Monitoring Project. The devices monitored at Ashland were installed between August and October 1977 to protect the eroding shoreline and stabilize the bluff. The devices were installed by the Red Clay Committee, which is a coalition of personnel from Douglas, Bayfield, Ashland, and Iron Counties, Wisconsin, and Carlton County, Minnesota. Financial assistance was provided by the U.S. Environmental Protection Agency. The general plan as constructed is shown in Figure 2-437. Longard tubes were used for all of the devices, which comprised three seawalls and six groins. The Longard tube is made of an inner impermeable polyethylene tube and an outer woven tube of high density polyethylene, as described in the Appendix. The tubes were filled with sand to 95 percent of the theoretical volume. A sand-water slurry was pumped into one end of the tube at a rate which allowed most of the sand to settle out before the slurry discharged at the opposite end of the tube. The ends of the seawall tubes and groins 2, 3, and 4 and the lakeward ends of groins 1, 5, and 6 were closed by clamps made from slotted 3-inch-diameter galvanized pipe. The shore ends of tubes 1, 5, and 6 were closed by a steel plate device which is part of the apparatus for filling the tubes. The exposed surfaces of the tubes that were used for seawalls were sprayed with a sand-epoxy protective coating after the tubes were filled. The devices installed at the site are described below.

(a) Devices 1 to 6 (Groins). Six 100-foot groins were constructed of 69-inch-diameter Longard tubes (Fig. 2-438). Each of the

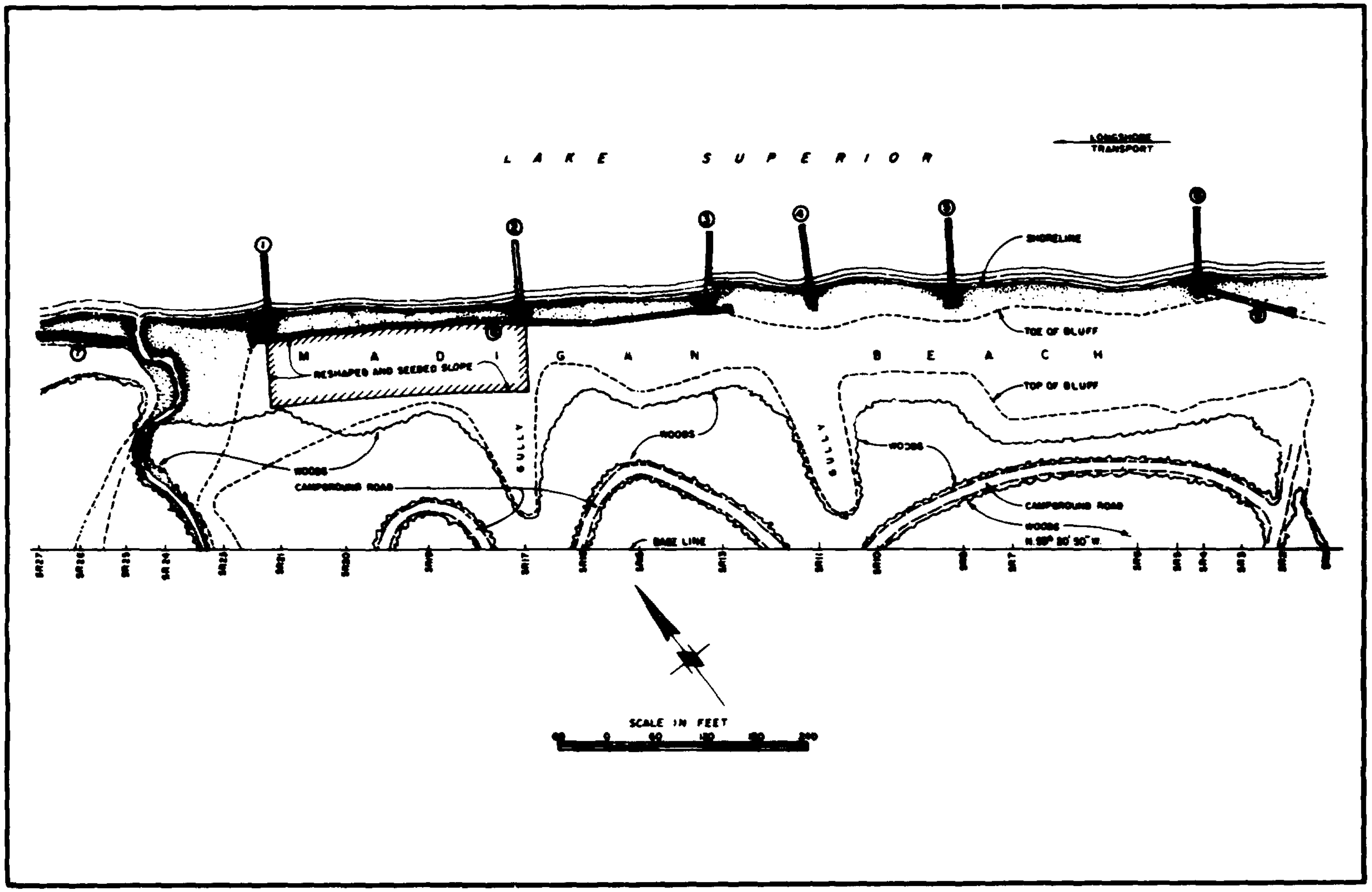


Figure 2-437. General plan of Ashland, Wisconsin, monitoring site.

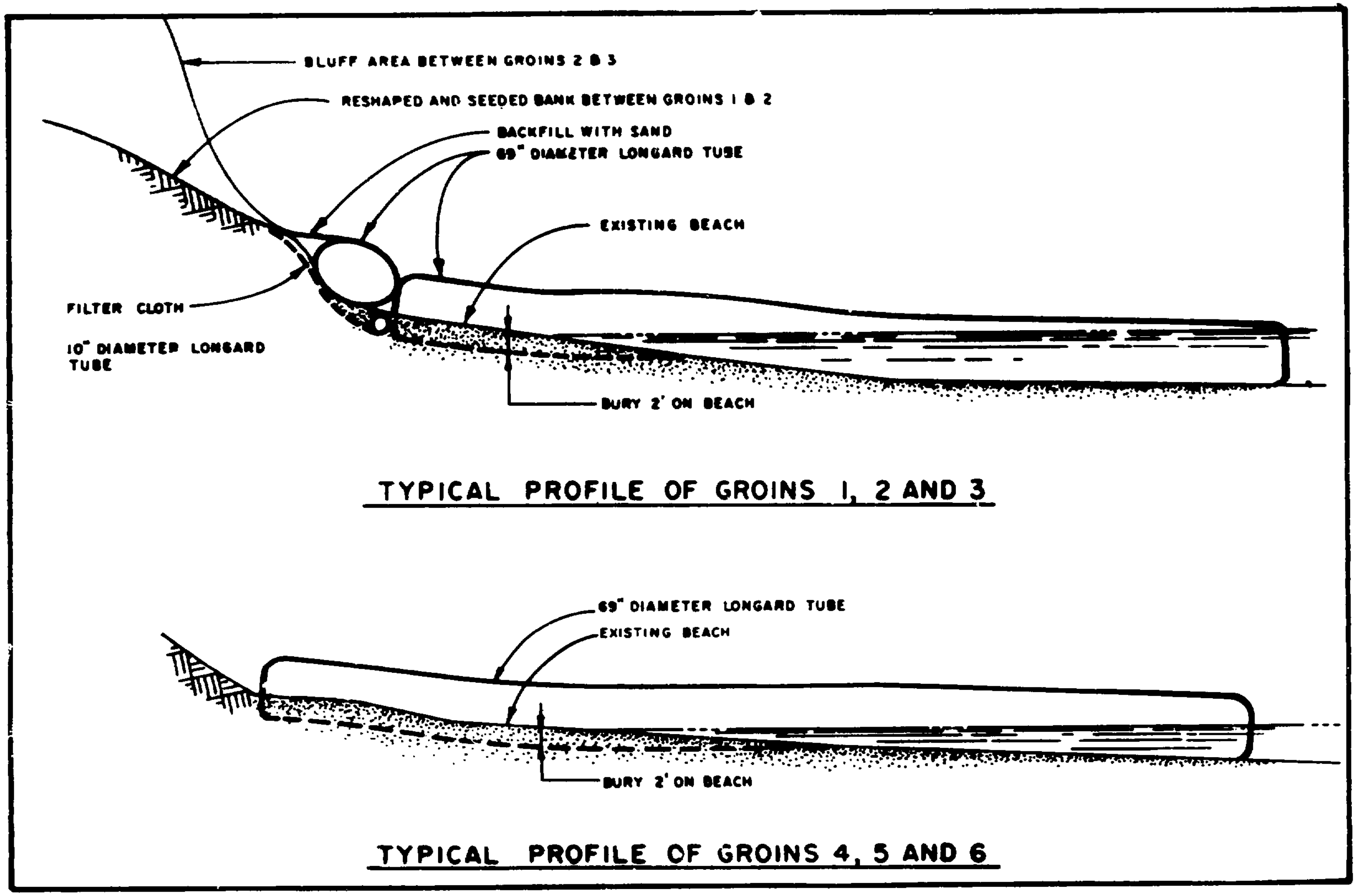


Figure 2-438. Profiles of groins 1 to 6 at Ashland site.

groins was placed on a 7-foot strip of filter cloth with a 10-inch-diameter Longard tube, factory-stitched to each side (Fig. 2-439).

(b) Device 7 (West Seawall). This 100-foot seawall is a 40-inch-diameter Longard tube atop a 69-inch-diameter tube. Filter cloth was used underneath and behind the tubes, and sand backfill was placed between the tubes and the clay bluff. A 10-inch-diameter Longard tube, factory-stitched to the lakeward edge of the filter cloth, provided toe protection for the bottom tube (Fig. 2-440). The east end of the seawall was protected by a timber-pile bulkhead (Fig. 2-441). A concrete grout wedge was placed between the 40- and 60-inch tubes to resist soil forces.

(c) Device 8 (Central Seawall). This 600-foot seawall was constructed of four 69-inch-diameter Longard tubes of varying length, placed end-to-end. Filter cloth and sand backfill were used as in device 7 (Fig. 2-440). The interior tube ends were placed adjacent to each other; however, they were not fastened together. The east and west ends of this seawall were protected by timber-pile bulkheads (Fig. 2-441). The bluff slope directly behind the seawall between groins 1 and 2 was graded to a 1 on 2.5 slope.

The slope was seeded with birdsfoot trefoil, and smooth brome and tall fescue grasses.

(d) Device 9 (East Seawall). This 100-foot seawall is a single 69-inch-diameter Longard tube. Sand backfill and filter cloth were used as in devices 7 and 8 (Fig. 2-440). A timber-pile bulkhead was used to protect the east end of the seawall (Fig. 2-441).

(3) Statistics and Costs. Statistics and costs for the Longard-tube installations monitored at Ashland are given in Table 2-100.

Table 2-100. Statistics and costs, Ashland, Wisconsin.

Construction cost: $134,000	
Material	**Quantity**
Groins	6
69-in Longard tubes	6 ea. (total length 800 ft)
10-in Longard tubes	7 ea. (total length 800 ft)
Filter cloth	20,800 ft^2
Sandfill for tubes	600 yd^3
Seawalls	3
69-in Longard tubes	6 ea. (100 ft long)
40-in Longard tube	1 ea. (100 ft long)
Filter cloth	20,800 ft^2
Sandfill for tubes	820 yd^3
Timber Bulkheads	4
Wood piles	80 ea.
Wales	56 lin ft
Lolts	80 ea.
Washers	80 ea.
Steel corner strips	4 ea.

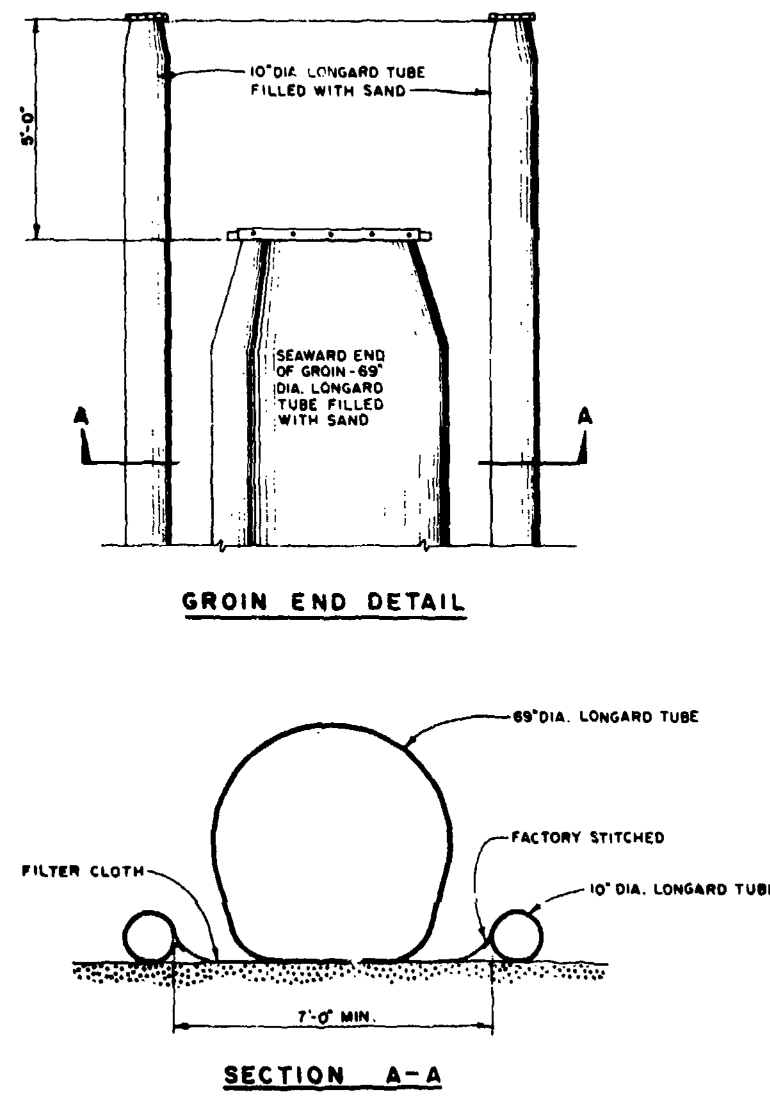


Figure 2-439. Longard-tube groin at Ashland site.

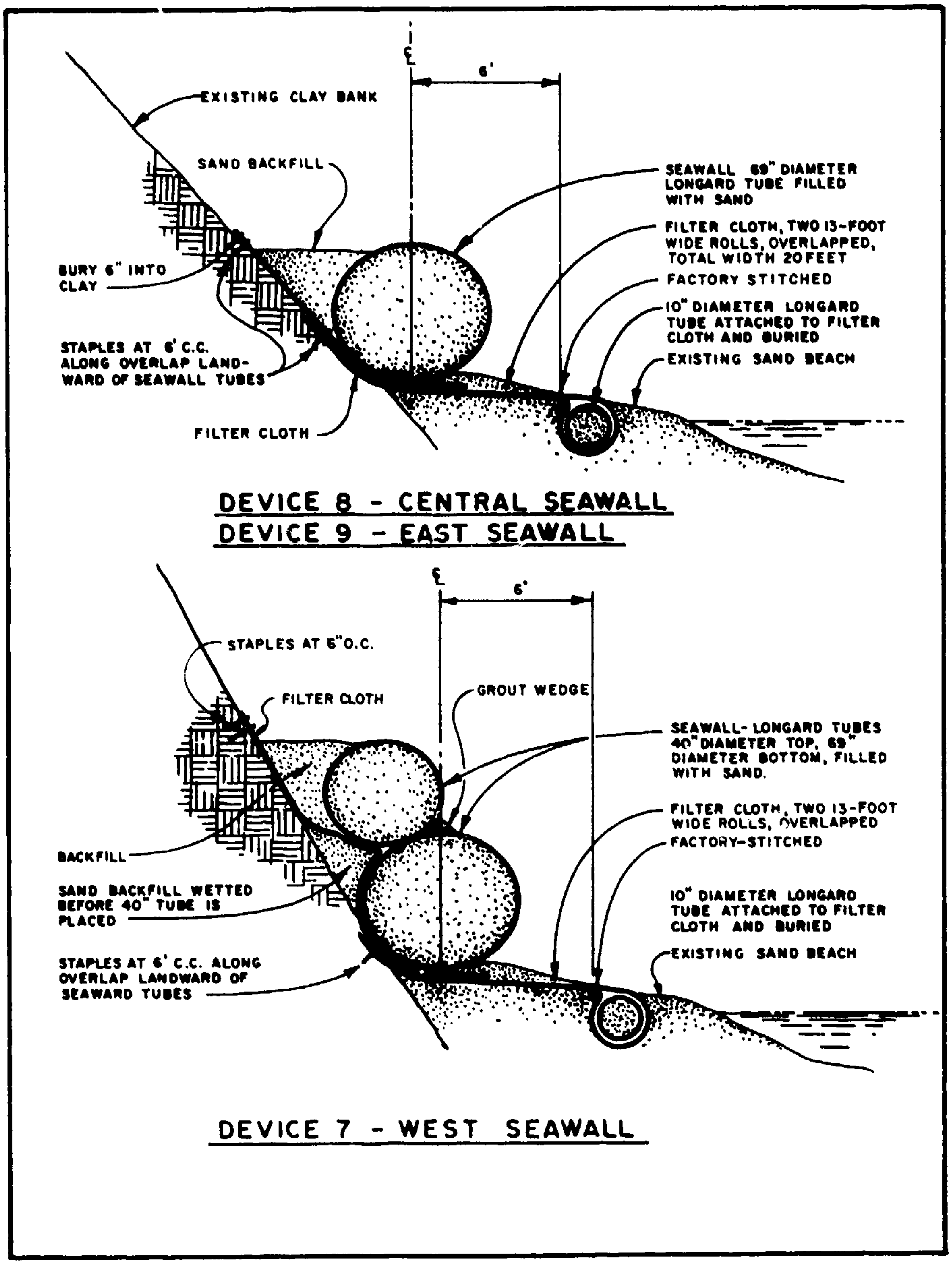


Figure 2-440. Devices 7, 8, and 9 at Ashland site.

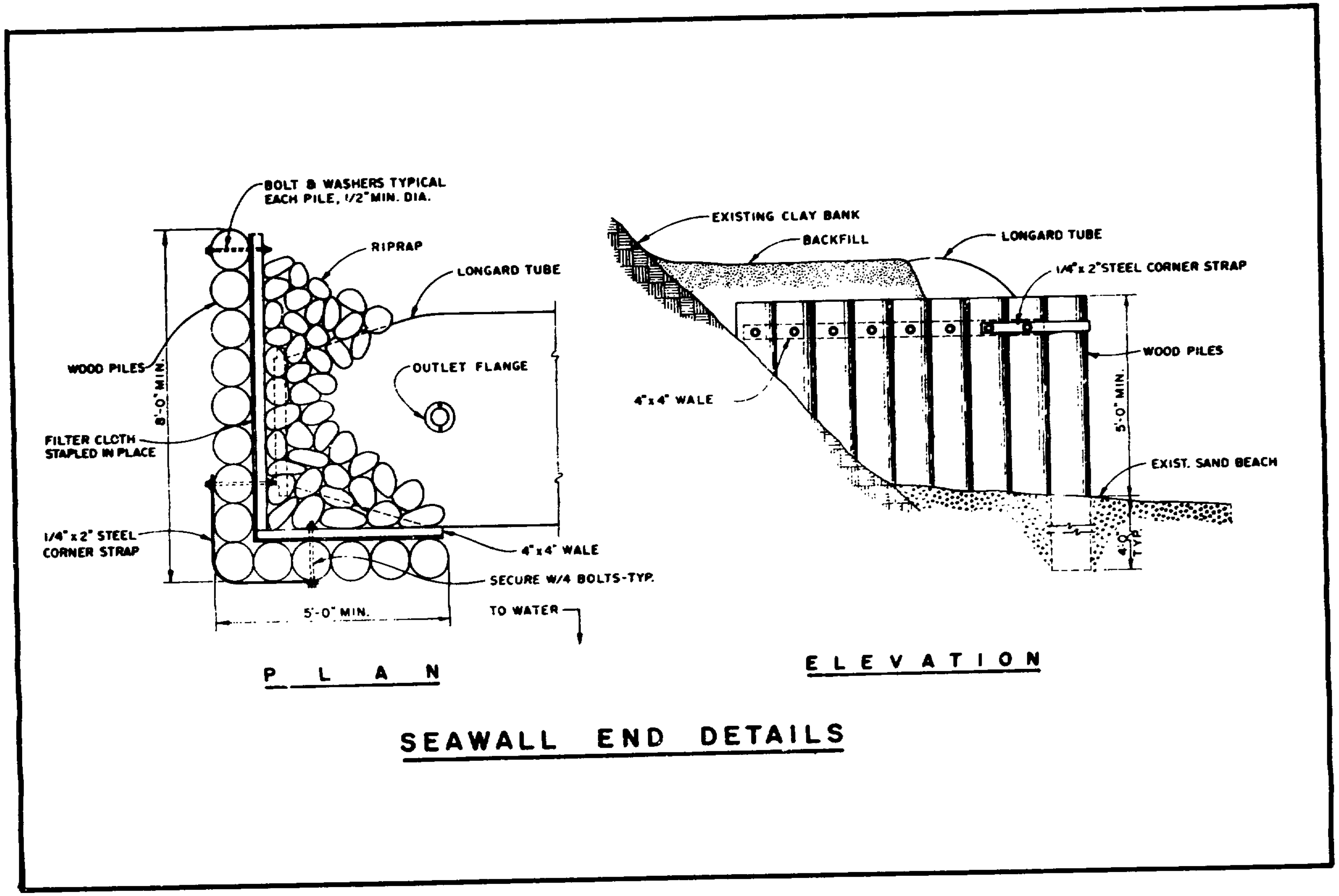


Figure 2-441. Seawall at Ashland site.

(4) Performance.

(a) Device 1 (Groin). The Longard tube remained undamaged from the initial installation in June 1978 until the followup visit in September 1979 (Fig. 2-442). During the October 1979 storms, the tube was damaged by floating debris. The 31 October 1979 site visit revealed that the tube had been damaged near the waterline, and some interior sandfill had been lost (Figs. 2-443 and 2-444). As of May 1980 some sand had been lost from the center of the tube. However, the groin was still performing properly and the loss of sand had a minimal effect (Fig. 2-445). In addition, a considerable amount of accretion was present along the beach.

(b) Device 2 (Groin). Groin 2 was damaged very early in the monitoring program (Fig. 2-446). Floating debris apparently opened a hole in the Longard tube near the landward end. Subsequently, waves continued to act on the structure, causing a considerable loss of interior sand. By October 1978, the lakeward end of the tube had moved eastward and settled to a lower position. The cross section of the tube changed with each monthly visit as the fabric progressively unbraided, allowing the inner liner to tear and lose even more sand (Figs. 2-447 and 2-448). By October 1979, the landward end had completely failed (Fig. 2-449). By May 1980 the tube had completely deflated and ceased to intercept littoral drift (Fig. 2-450).

Figure 2-442. Structurally intact Longard tube, groin 1, Ashland, Wisconsin, 26 September 1979.

Figure 2-443. Damage to the Longard tube by floating debris,groin 1, Ashland, Wisconsin, 31 October 1979.

Figure 2-444. Sand loss from Longard tube, groin 1, Ashland, Wisconsin, 31 October 1979.

Figure 2-445. Groin 1 as of 14 May 1980, Ashland, Wisconsin.

Figure 2-446. Initial damage to the Longard tube, groin 2, Ashland, Wisconsin, 31 October 1978.

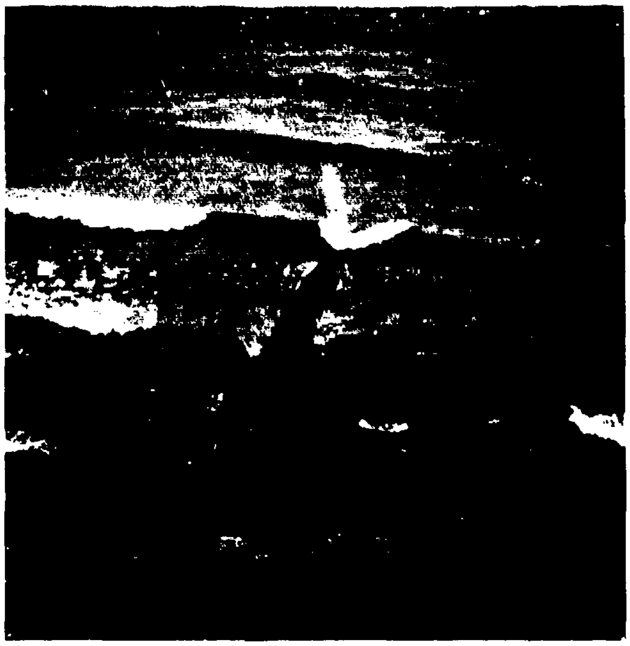


Figure 2-447. Loss of sand from Longard tube, groin 2, Ashland, Wisconsin, 30 May 1979.

Figure 2-448. Continued loss of sand from Longard tube, groin 2, Ashland, Wisconsin, 16 August 1979.

Figure 2-449. Failed landward end of groin 2, Ashland, Wisconsin, 31 October 1979.

Figure 2-450. Deflated condition of groin 2, Ashland, Wisconsin, 14 May 1980.

(c) Device 3 (Groin). The structural performance of groin 3 was almost identical to that of groin 1. Undamaged until the October 1979 storms, this Longard tube maintained its structural integrity for about 1 year (Figs. 2-451 and 2-452). Then floating debris punctured holes in the tube near its landward end, and a significant amount of interior sand was lost (Fig. 2-453). By May 1980 this groin was still intact and acting properly as a groin. A considerable amount of sand accreted on its east side (Fig. 2-454).

(d) Device 4 (Groin). This groin was not damaged as of October 1979 (Figs. 2-455 and 2-456). However, by May 1980 the tube was almost completely deflated and no longer trapped littoral drift (Fig. 2-457). It was noted by the Project Engineer that extensive ice pressure ridges and ice hummocks near the beach in February 1980 may have contributed to the destruction of the groin.

(e) Device 5 (Groin). This groin remained intact through December 1979; however, a slight westward movement occurred at the lakeward end (Figs. 2-458 and 2-459). This groin, like groin 4, was completely deflated when inspected in May 1980 and ceased to control littoral transport (Fig. 2-460). The groin may also have been damaged by ice.

(f) Device 6 (Groin). The lakeward end of groin 6 was vandalized sometime between June and September 1978 by a shotgun blast. Wave action apparently widened the hole, and the sandfill began to washout. By October 1978, only the landward half of the tube remained filled with a substantial amount of sand (Fig. 2-461). After the initial damage and loss of sand, the landward end of the groin seemed to stabilize, without further degradation of the tube fabric (Fig. 2-462). By May 1980, more sand was lost at the landward end of the groin and more fabric was ripped. The structure no longer controls littoral transport (Fig. 2-463).

(g) Device 7 (West Seawall). The initial inspection on 31 October 1978 showed that the alinement of the west seawall had changed significantly. Although the Longard-tube fabric had not deteriorated, earthslides caused the 69-inch tube to slide lakeward, allowing the 40-inch tube to roll backward and drop down behind it (Figs. 2-464 and 2-465). The west end of the device has shown more displacement than the east end. Earthslides have pushed to the west end of the 69-inch tube in the lakeward direction, and the 40-inch tube fell down behind it. The backfill behind the west end of the device has completely eroded away, and the bluff has receded significantly. The west end of the device is buried in the sand, and, because of its displacement, may no longer be on top of the filter cloth. It could not be determined whether the west end of the tube was punctured or whether the sand had escaped before the ends were undermined and buried. The timber bulkhead at the east end was severely damaged both by earthslides and by undercutting due to adjacent streamflow (Figs. 2-466 and 2-467). After the winter of 1978-79, the 40-inch tube continued to drop, exposing the unprotected fabric (Fig. 2-468). No more damage occurred until an October 1979 storm washed out most of the backfill and exposed the

Figure 2-451. Undamaged Longard tube, groin 3, Ashland, Wisconsin, 31 October 1978.

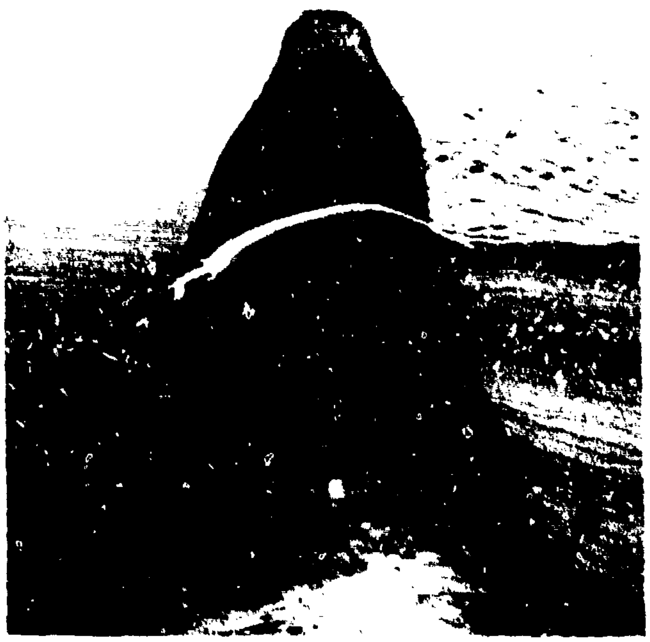


Figure 2-452. Undamaged Longard tube, groin 3, Ashland, Wisconsin, 16 August 1979.

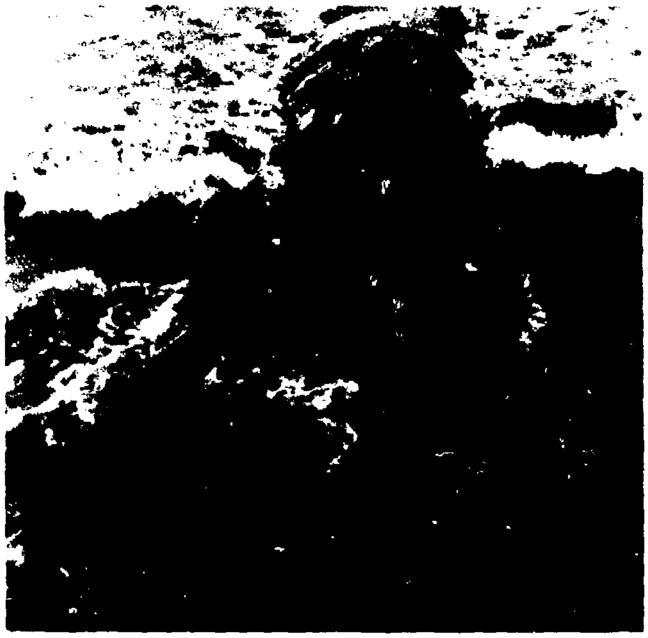


Figure 2-453. Longard tube of groin 3 damaged by floating debris, Ashland, Wisconsin, 31 October 1979.

Figure 2-454. Groin 3 intact, Ashland, Wisconsin, 14 May 1980.

Figure 2-455. Debris scattered around undamaged groin 4, Ashland, Wisconsin, 12 June 1979.

Figure 2-456. Longard tube remained undamaged, groin 4, Ashland, Wisconsin, 31 October 1979.

Figure 2-457. Damaged groin 4, Ashland, Wisconsin, 14 May 1980.

Figure 2-458. Westward shift of the lakeward end of groin 5, Ashland, Wisconsin, 31 October 1978.

Figure 2-459. Unpunctured Longard tube, groin 5, Ashland, Wisconsin, 31 October 1979.

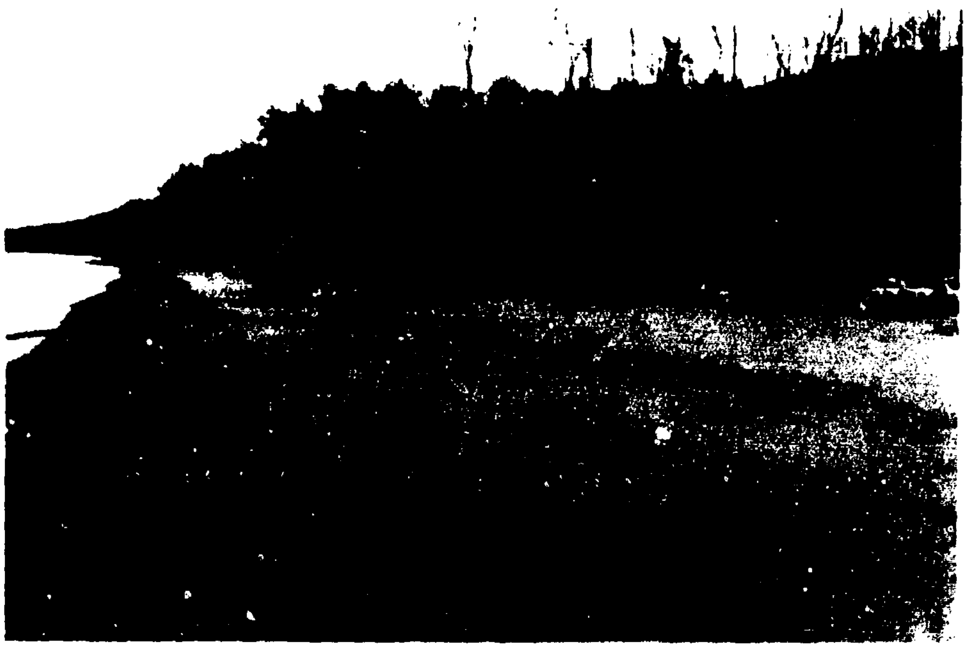


Figure 2-460. Groin 5 damaged as of 14 May 1980, Ashland, Wisconsin.

Figure 2-461. Vandalized Longard tube of groin 6 (upper right) with groins 3, 4, and 5 in foreground, Ashland, Wisconsin, 31 October 1978.

Figure 2-462. Stabilized landward end of groin 6, Ashland, Wisconsin, 31 October 1979.

Figure 2-463. Groin 6 as of 14 May 1980, Ashland, Wisconsin.

Figure 2-464. Displaced Longard tubes of the west seawall, Ashland, Wisconsin, 31 October 1978.

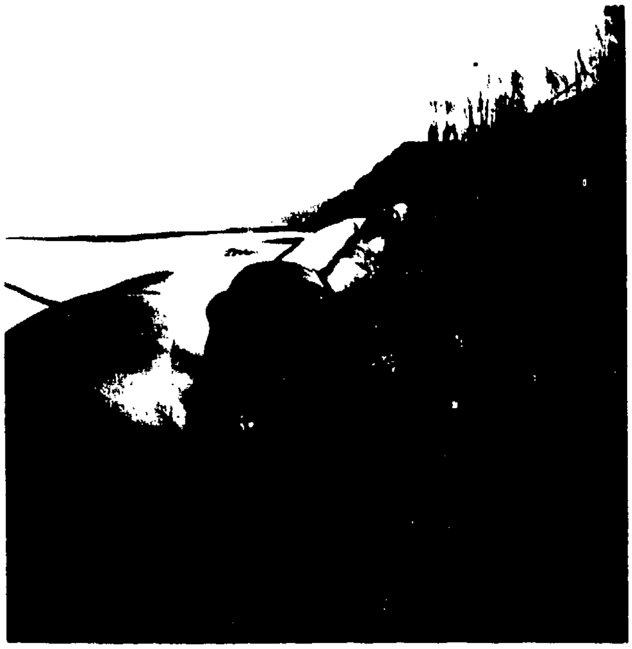


Figure 2-465. West end of the west seawall, Ashland, Wisconsin, 31 October 1978.

Figure 2-466. Damaged timber-pile abutment of west seawall, Ashland, Wisconsin, 31 October 1978.

Figure 2-467. Damaged timber-pile abutment of west seawall, Ashland, Wisconsin, 31 October 1978.

Figure 2-468. Displacement of the 40-inch tube from atop the 69-inch tube of the west seawall, Ashland, Wisconsin, 30 May 1979.

filter cloth. At that time, a piece of floating debris punctured a hole in the top of the 69-inch tube near its east end (Fig. 2-469). By May 1980 much accretion on the beach covered the west end of the seawall (Fig. 2-470). By July 1980, the hole had increased to a rip several feet long, exposing the sandfill.

(h) Device 8 (Central Seawall). Although the central seawall was impacted by earthslides similar to those affecting device 7, the seawall initially held its original position better. The initial site visit on 31 October 1978 revealed some settlement and lakeward bulging of the Longard tubes, but no overall displacement (Fig. 2-471). Although the west-end bulkhead was nearly destroyed (Fig. 2-472), the abutment at the east end remained in good condition. After the 1979 spring thaw, the structural integrity of the seawall was still intact; however, the protective epoxy coating had begun flaking off in many places (Fig. 2-473). By the 25 September 1979 visit, the movement of the seawall segment between groins 2 and 3 had become apparent (Fig. 2-474). Further erosion of the beach during the October storms allowed the tube to roll lakeward, leaving a gap between the seawall and the bluff. Earthslides also pushed the seawall lakeward. This section of the seawall had settled, allowing overtopping waves to erode backfill and slumped bluff material. The erosion rate had increased substantially, exposing the underlying filter cloth (Figs. 2-475 and 2-476). By May 1980 accretion on the beach prevented most waves from reaching the tube, although storm waves had apparently thrown sand on the berm behind it (Fig. 2-477).

(i) Device 9 (East Seawall). This Longard tube with its protective epoxy coating withstood the environmental forces well. An October 1978 photo showed debris scattered along the seawall and a small pile of sand which had spilled from a hole in the side of the tube (Fig. 2-478). However, over the next 9 months, the hole did not enlarge and little additional interior sand was lost. By July 1979 the western end of the tube had rolled slightly lakeward, but there was no overall displacement of the seawall; the timber abutment at the east end remained in good condition (Fig. 2-479). The hole in the Longard tube eventually increased in size, and by August 1979 the loss of interior sand became more apparent (Fig. 2-480). At that time several additional small holes were discovered, apparently caused by vandalism; however, the punctures did not enlarge and therefore presented no problem (Fig. 2-481). As of May 1980 the structure still remained intact.

(5) Functional Performance.

(a) Groins (Devices 1 to 6). The function of the groin field was to trap littoral sand and to build a protective beach, thereby stabilizing the clay bluff. Throughout most of the monitoring program, longshore transport was northwestward. Sand accumulated on the updrift sides of the groins; however, the storms during October 1979 eroded most of the sand fillets and smoothed out the shoreline.

By May 1980, the beach had accreted considerably. This, however, could not be attributed entirely to the presence of the groins, as groins 1 and 3 were the only functioning groins remaining. Instead, the accretion was

Figure 2-469. Backfill washed out, exposing filter cloth; puncture hole in lakeward tube due to floating debris, Ashland, Wisconsin, 31 October 1979.

Figure 2-470. West end of west seawall, Ashland, Wisconsin, 14 May 1980.

Figure 2-471. Longard tube of central seawall between groins 2 and 3, Ashland, Wisconsin, 31 October 1978.

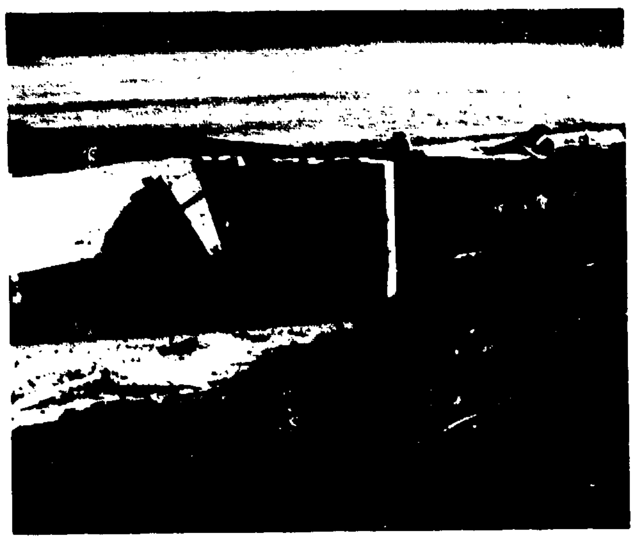


Figure 2-472. Damaged timber-pile abutment on the west end of the seawall, Ashland, Wisconsin, 31 October 1978.

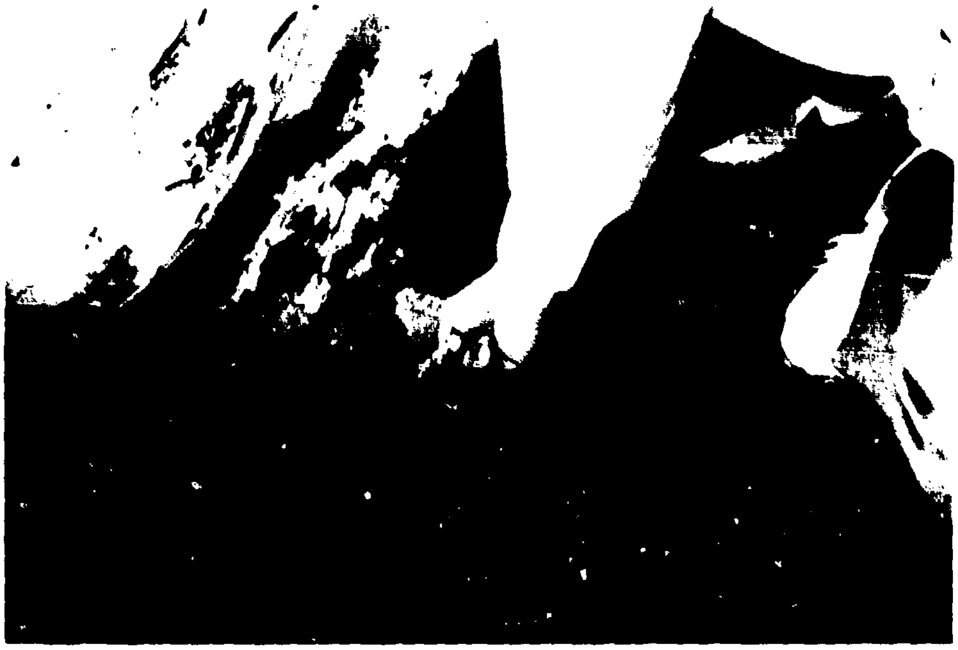


Figure 2-473. Flaking off of protective coating, Ashland, Wisconsin, 31 October 1979.

Figure 2-474. Lakeward movement of Longard-tube between groins 2 and 3 (central seawall), Ashland Wisconsin, 26 September 1979.

Figure 2-475. Lakeward movement of central seawall, Ashland, Wisconsin, 31 October 1979.

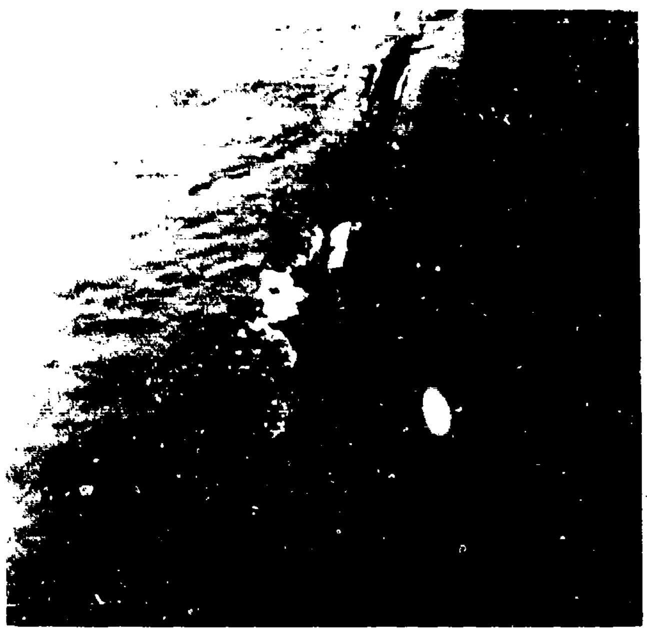


Figure 2-476. Erosion in front of central seawall exposing the underlying filter cloth, and 10-inch diameter Longard tube at toe, Ashland, Wisconsin, 31 October 1979.

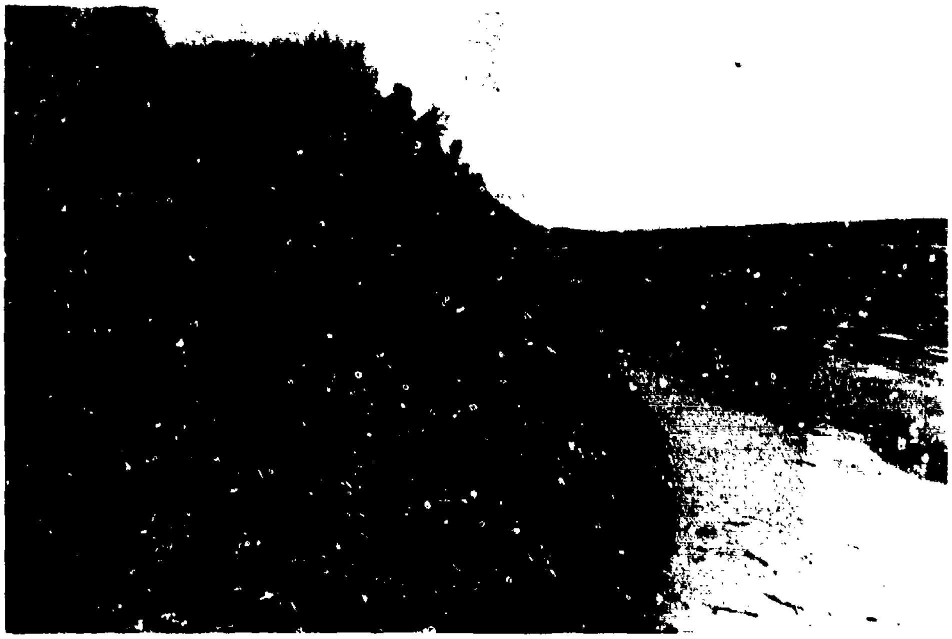


Figure 2-477. Central seawall on 14 May 1980, Ashland, Wisconsin.

Figure 2-478. Longard tube and abutment of east seawall, Ashland, Wisconsin, 31 October 1978.

Figure 2-479. Lakeward movement of the west end of the east seawall, Ashland, Wisconsin, 30 May 1979.

Figure 2-480. Sand loss from puncture hole in the east seawall, Ashland, Wisconsin, 16 August 1979.

Figure 2-481. Inspecting vandalism at the east seawall, Ashland, Wisconsin, 31 October 1979.

primarily due to a seasonal increase in littoral drift (the result of a change in lake level, wave climate, or amount of sediment provided by streams). Figures 2-482 and 2-483 show the sand accumulations. Groins 1 and 3, however, did trap sediment as shown in Fig. 2-484.

(b) Seawalls (Devices 7, 8, and 9). The seawalls were intended to provide bluff-toe protection and to stabilize the highly erodible red clay bluff at the project site. Although the seawalls prevented direct bluff-toe erosion, the bluff material continued to slough down onto the beach, where it was carried away by wave action. An oblique aerial photo taken on 13 June 1979 shows the failure of the seawalls to protect the bluff (Fig. 2-485). Note the bluff slumping behind the central seawall between groins 2 and 3, and the massive bluff slide behind the east seawall. Only where the seawall was used in conjunction with a resloped and planted embankment, as between groins 1 and 2, was it able to stabilize the bluff. Figure 3-20, which was put in Section III for compariaion of similar devices, shows a series of profiles through the seawall. The profiles for June 1979 and October 1979 indicate about 15 feet of erosion behind the longard tube.

(6) Analysis. The results of this monitoring project demonstrate that bluff-toe protection alone may not be sufficient to control the erosion of a highly erodible bluff. In most cases of severe bluff slumping the slope should be graded and planted with vegetation to avoid slumping due to slope instability. In addition, providing a control for runoff water on top of the bluff, such as drainage ditches, would be helpful.

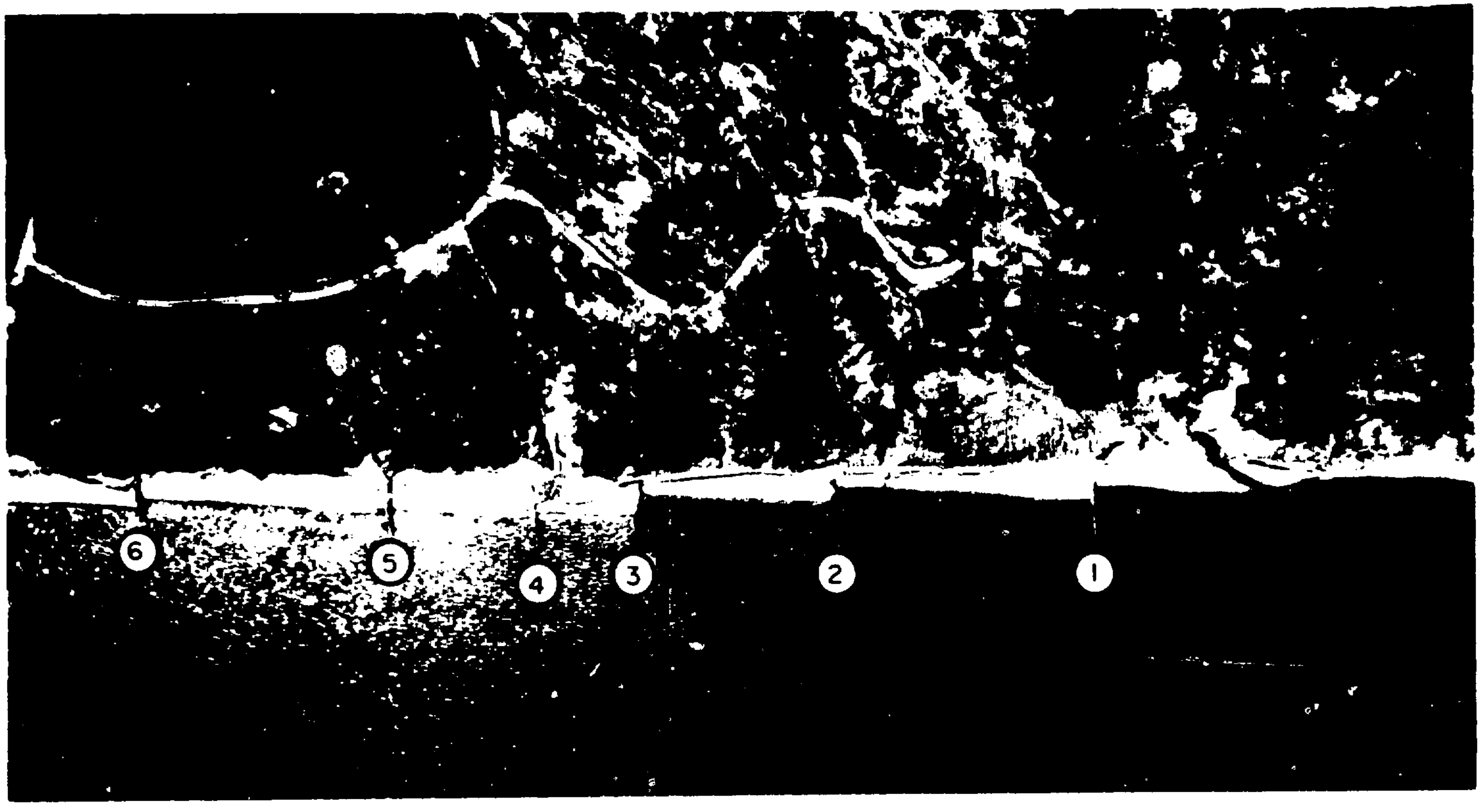


Figure 2-482. Sand accumulation on the updrift sides of the groins, Ashland, Wisconsin, 24 May 1979.

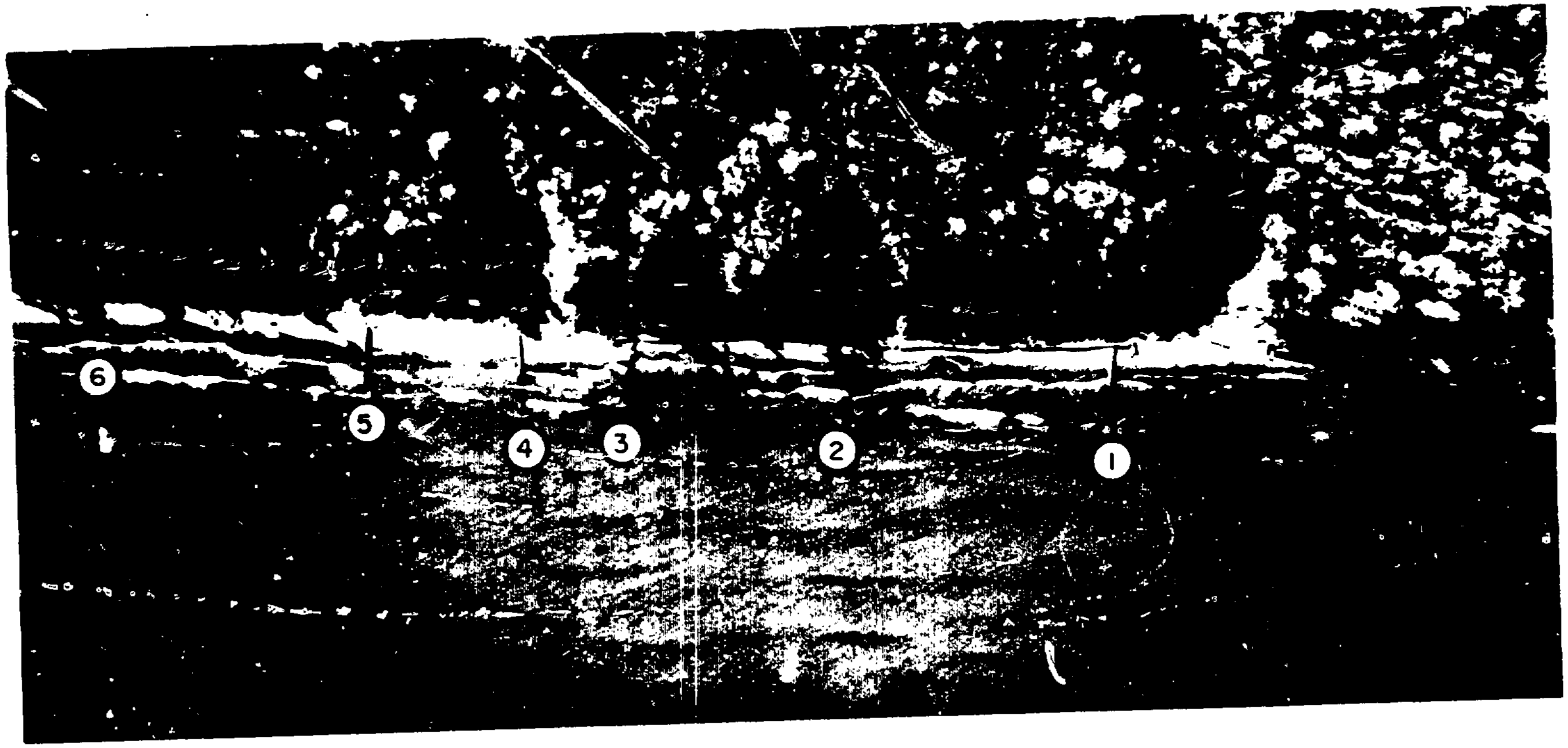


Figure 2-483. Most of the sand fillet material has eroded and the shoreline has smoothed out, Ashland, Wisconsin, 17 October 1979.

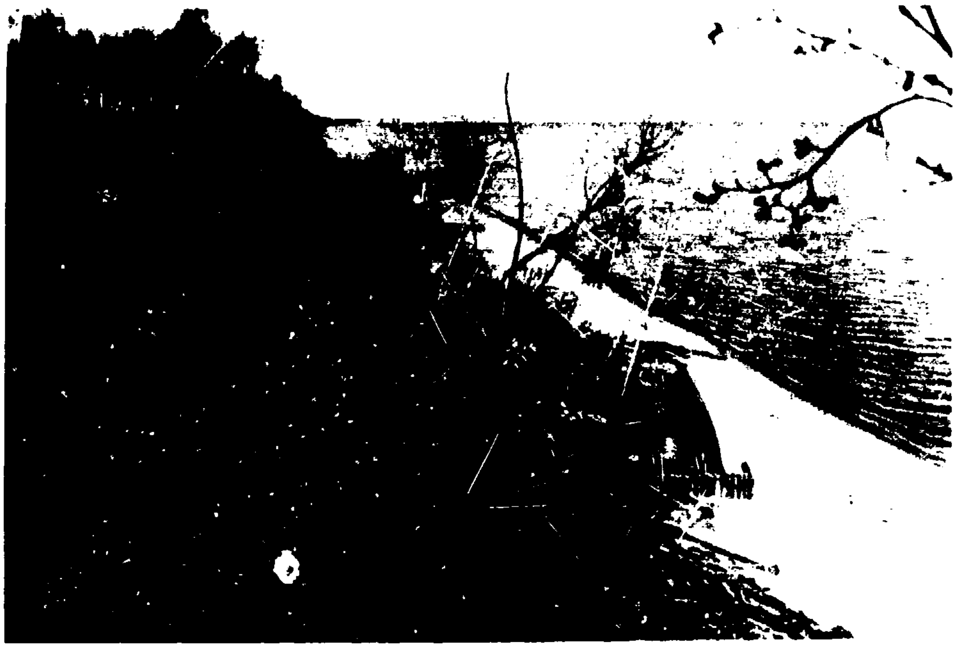


Figure 2-484. Accretion along beach with groins 1 and 3 still intact, Ashland, Wisconsin, 14 May 1980.

They could be graded to be esthetically acceptable. However, some form of toe protection must be provided in conjunction with the prepared slope to prevent undercutting by wave action. At Ashland, the Longard-tube seawall provided the necessary toe protection in some areas, yet overtopping waves apparently eroded the bluff toe in several places. Where overtopping is anticipated, tubes used as seawalls should be placed about 30 feet lakeward of the bluff toe. This permits the formation of a berm of sand behind the tube on which overtopping wave energy is expended without attacking the bluff material.

The use of Longard tubes as groins at Ashland demonstrated their short-term structural adequacy for this purpose. The vandalism of groin 6 and minor damage to the other groins by floating debris, followed by unbraiding of the fabric around puncture holes and progressive sand loss, indicates a possible short life in this environment. Another source of damage may be the ice pressure ridges and ice hummocks near the beach. For this reason it may not be advisable to use Longard tubes in regions where ice forces are a factor. In contrast to the seawalls, the groins were not protected by an epoxy coating. Although the epoxy coating on the seawalls did not prevent punctures, it did prevent unbrading of the fabric. Therefore, holes in the seawalls did not increase in size and little sandfill was lost. If Longard tubes are used in an area subject to floating debris damage or where vandalism may be a problem, a protective coating should be applied; however, this is not possible on tube surfaces that are submerged or continuously washed.

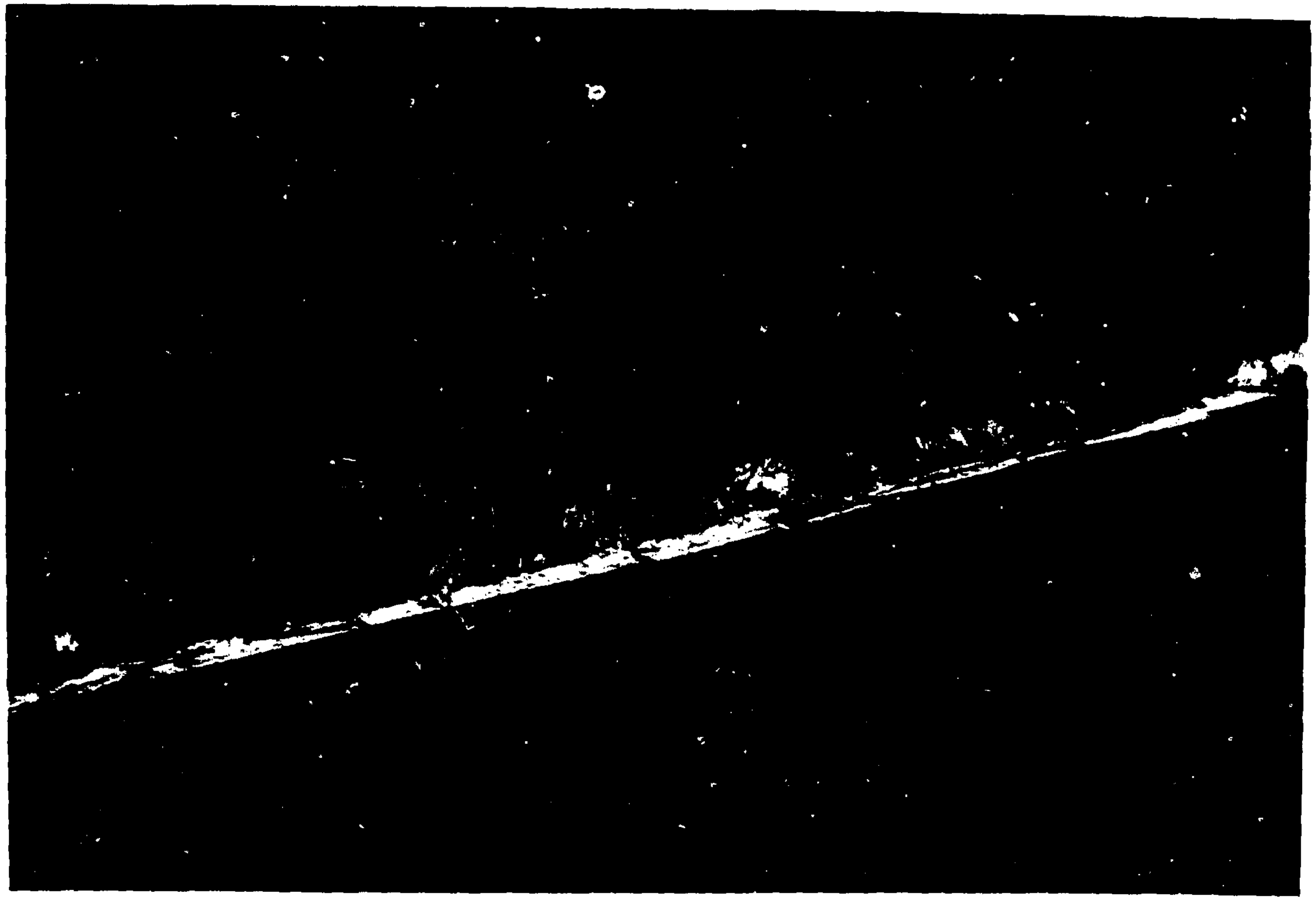


Figure 2-485. Bluff slumping behind the protective devices, Ashland, Wisconsin, 13 June 1979.

The direction of approach of large waves at the site seldom veers far from the normal to shore; therefore, this installation did not provide a good demonstration of the ability of the tubes to resist lateral displacement by angling waves. The displaced outer ends of groins 2 and 5 indicated that severe wave action can move a tube. Also, the buildup of sand along the updrift sides was not as pronounced as it might be in a zone of large longshore transport where groins are most effective. The overturning force of such a buildup, coupled with scour on the downdrift side, has rolled tubes in other areas. Nevertheless, the two tubes that were not damaged at Ashland did perform their function as groins during the monitoring period.

e. Little Girls Point, Michigan.

(1) Site Description.

(a) Geographical Setting. The monitoring site is located on Lake Superior near the western end of upper Michigan, approximately 14 miles northwest of the city of Ironwood in Gobebic County, and about 7 miles east of the Wisconsin State line (Fig. 2-486). At the site, the shoreline trends away from the point in about an east-southeastward direction.

(b) Water Level and Wave Conditions. Long-term fluctuations of the water level in Lake Superior (shown in Figs. 2-385 and 2-386) indicate that lake levels have decreased slightly from a long-term high lake level in 1973. Short-term fluctuation is the result of changing meteorological conditions which can alter the water level along the shore 1 or 2 feet or more. However, water level fluctuations near this site have not been more than 1 foot above or below the undisturbed lake level. No LEO station was established at the site, but the wave climate is classified as severe.

Waves impinging on the shoreline at this site are primarily from the north, developing from fetch lengths of more than 80 miles. The Apostle Islands offer some protection from north-northwesterly storm waves. However, Little Girls Point is fully exposed to northerly storm waves. The net direction of longshore transport at the site is eastward.

(c) Geology, Soils, and Vegetation. The beach site is 300 feet long, has an average width of 30 feet, and has an average slope of about 1 on 10. The basic composition of the beach in 1974 was sand and gravel with a scattering of cobbles varying in size from 2 to 4 inches. A layer of cobbles and coarse gravel now covers the 1974 beach. Some timber debris from the adjacent bluff was also present. The steep bluff is almost 30 feet high and is composed primarily of clay till. Vegetation, covering more than 50 percent of the bluff, consists of a mixture of local grasses, shrubs, and trees (Fig. 2-487). It comprises mainly the survivors of bluff-top vegetation left clinging to fragments of till that have been undercut and are now intermittently sliding down the bluff face.

(d) The Problem. This monitoring site is a typical example of the bluff erosion problem common to many Great Lakes sites. Solifluction can occur in the spring as the saturated soil surface moves downslope over the still frozen underlayer. This process is typical in cold

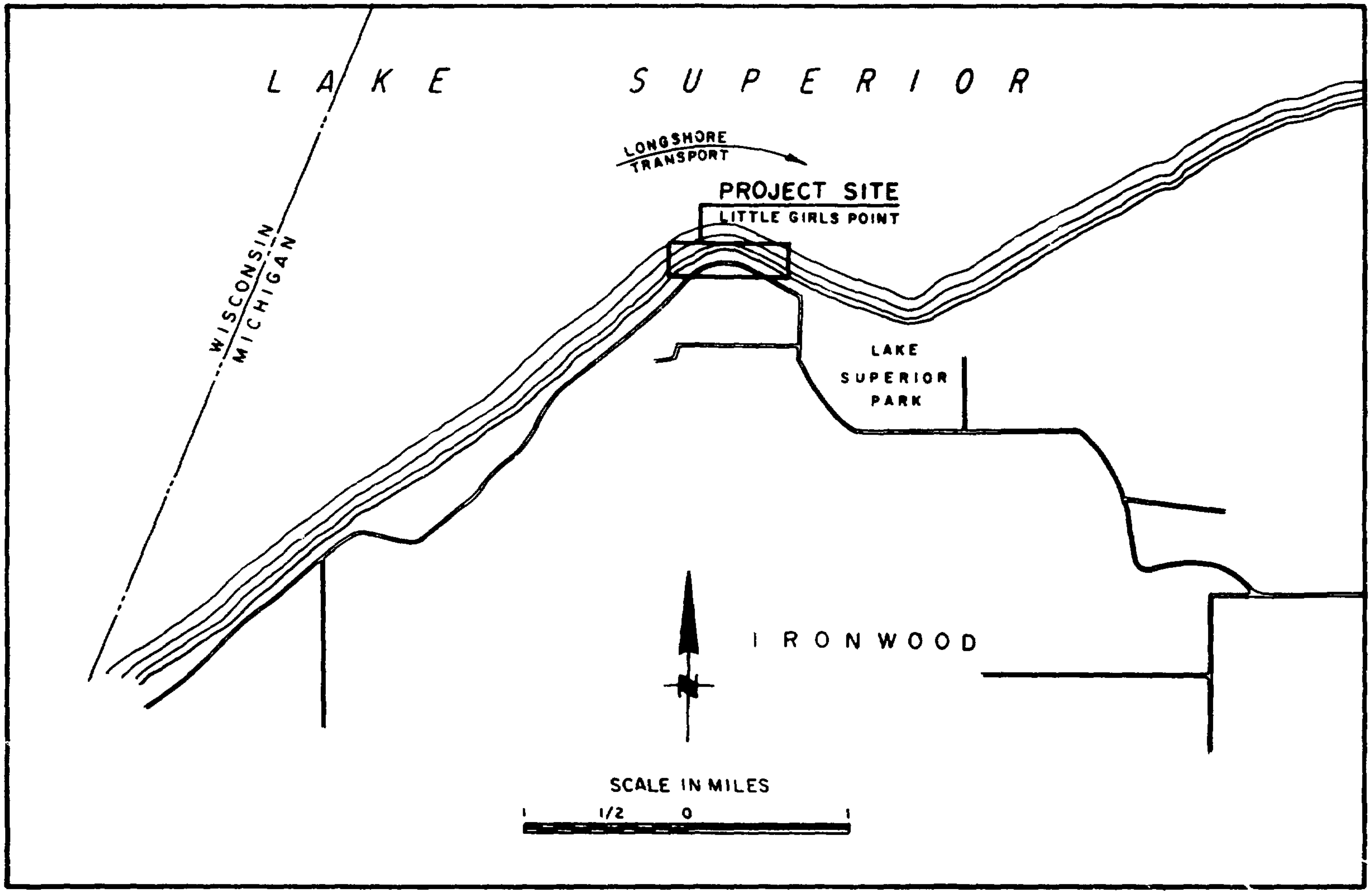


Figure 2-486. Location map of Little Girls Point, Michigan, monitoring site.

Figure 2-487. Note vegetation on bank behind Nami rings, Little Girls Point, Michigan, spring 1979.

areas where unprotected or nonvegetated slopes exist. Slumping or block calving of the slope may also occur as a natural process as the bluff attempts to reach a stable slope angle. This form of mass wasting is aggravated by wave action at the toe of the bluff. Waves breaking on the toe or wave runup can undercut the toe of the bluff or remove waste material at the base. This reduces the stability of the bluff and provokes continued slumping of the bluff. Littoral currents then transport waste material away from the bluff toe.

(2) Monitoring Site.

(a) Description. The installation monitored at this site was a 300-foot-long Nami-ring revetment installed in 1974 under the Michigan Demonstration Erosion Control Program at a cost of about $63 per foot. The Nami ring is a special kind of artificial concrete block made in the shape of a short section of concrete pipe, 2.5 feet in diameter by 1 foot long, weighing 240 pounds. (The use of the Nami ring in revetments is subject to the terms of a U.S. Patent.) "Nami," the Hawaiian word for "wave," was selected by the patent holder as a convenient term to describe the rings used in this system. The rings are placed side-by-side over filter cloth on the slope to be revetted, with a resultant large void ratio. With this arrangement, model tests indicated that prototype waves 6.5 feet high would not displace the rings. Any sand or gravel caught up in the turbulence of the waves tends to be deposited inside the rings and in the voids between adjacent rings, adding to the stability of the section and protecting the filter cloth.

The eastern end of the revetment was placed on a granular filter layer, and the western end on Polyfilter-X filter cloth. Some of the rings on the west end were tied together with steel rods. It had been planned to grade the bluff and beach face to a slope of 1 on 1.5 and then install the revetment at the base of the bluff; however, the bluff was not graded and the revetment was installed along the base of the bluff on the existing beach without excavating the toe to LWD, as planned (Fig. 2-488). Because the number of rings cast was sufficient only for a revetment placed on the graded slope, the revetment as placed on the beach slope did not extend high enough to prevent wave overtopping, with resultant scouring of the bluff toe.

(b) Performance. During the first year, most of the rings remained stable. The eastern segment of the revetment was forced upward along its landward edge, apparently as a result of subgrade upheaval due to surcharge loading imposed by the slumping bluff. However, the western end remained in its original position. Some of the lower rows were covered with beach sand, and rings farther up the slope continued to fill with sand (Fig. 2-489).

Although the site was attacked by a severe storm in November 1975, most of the rings remained in place, but many were displaced and broken. After the storm, the remaining rings were full of sand, and where the rings had been displaced the filter cloth was torn and exposed. In one area a tie rod was also displaced and bent (Fig. 2-490).

Scouring of the bluff continued after the Nami-ring revetment was installed, but the scouring which caused slumping of the bluff was not severe until the November storm. Before the storm it appeared that, except for one short reach, the bluff behind the revetment receded less than in the adjacent areas. The rings accreted sand and coarser debris, as littoral transport deposited large amounts of material. By June 1976 most of the revetment was completely buried. The resultant smoother slope increased wave runup and overtopping of the low-lying revetment, and bluff erosion increased approximately to prerevetment rates. In the next 3 years, littoral debris became progressively coarser as high waves hurled larger stones into the revetment, breaking most of the exposed rings.

Field surveys in the spring of 1979 indicated that the revetment was almost entirely covered with a mantle of cobbles and was no longer performing its intended function. Therefore, monitoring at this site was discontinued.

(c) Analysis. The Nami-ring revetment proved to be a good sand-accreting device, but because of its low height and resultant wave overtopping, it could not provide adequate bluff protection. The rings were displaced to some extent by wave action and ice thrust, but primarily by slumping of the clay bluff and upheaving of the revetment subgrade material. Because the tie rods in the lower west end of the revetment prevented ring displacement until the rings were covered with debris, it appears advisable that all rings be tied with steel rods at

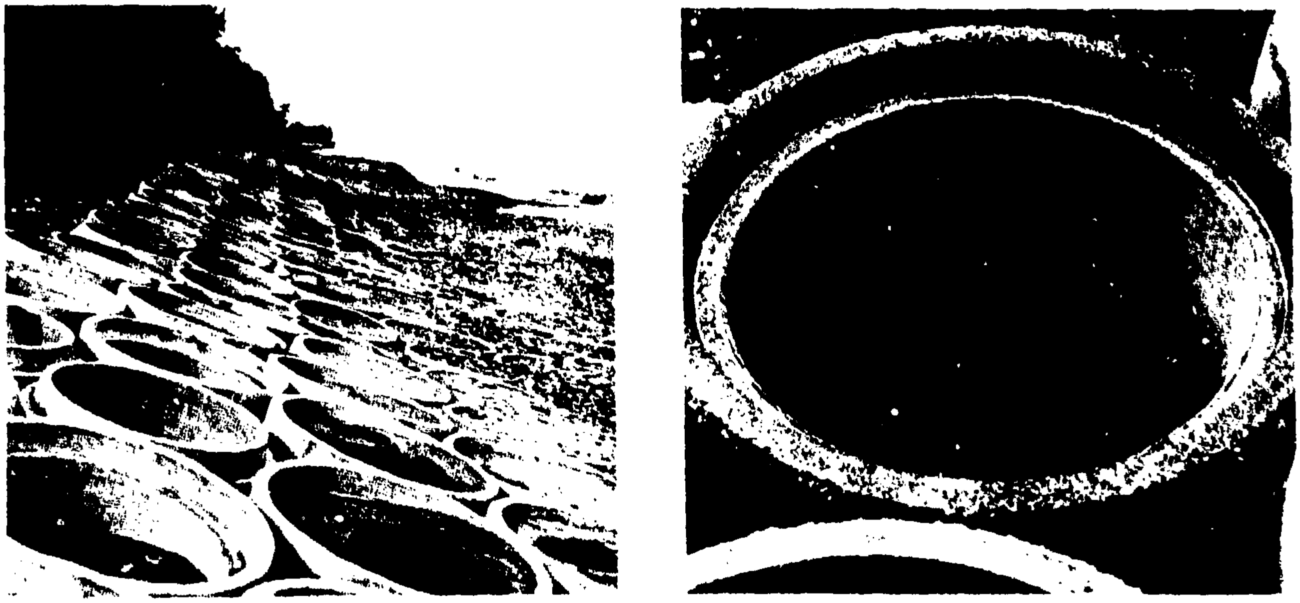


Figure 2-488. Completed Nami rings at Little Girls Point, Michigan, 2 October 1974 (from Brater, 1974).

Figure 2-489. Nami rings after 7 months of service, note displacement and sandfill, Little Girls Point, Michigan, 21 May 1975 (from Brater, Armstrong, and McGill, 1975).

Figure 2-490. Filter cloth was torn and a tie rod was displaced and bent along the Nami-ring revetment, Little Girls Point, Michigan, 3 May 1979.

future installations. Early scour of the bluff toe and slumping might have been avoided if the bluff had been graded to a stable slope as planned. This might also have prevented the displacement of the rings by upheaval due to excessive surcharge loading by the steep bluff. If the revetment had been carried to an elevation high enough to prevent overtopping, performance might have been better, at least until the rings were broken. As installed, the Nami-ring revetment at Little Girls Point did not provide conclusive evidence of the system's inadequacy. It did, however, indicate that cobbles in the littoral mantle can destroy the rings during high wave episodes and that this system should not be used where such conditions are known to exist. On a cobble-free beach, a Nami-ring revetment properly installed might perform well, and more experimentation might reveal the usefulness of the system in an environment that is compatible with its limitations. Provision should be made for higher runup in design, as the revetment becomes smoother when filled with sand, and runup is amplified.

f. Lincoln Township, Michigan.

(1) Site Description.

(a) Geographical Setting. The monitoring site is located in Lincoln Township Park on the southeastern shore of Lake Michigan, about 2 miles northwest of Stevensville, Michigan, and 24 miles from the Indiana State line. Located in Berrier County, the highway route to the site is by U.S. Highway I-94 to the Stevensville exit, west to a service road, then north 1 mile on the service road. The park shoreline is straight and trends approximately N. 30° E (Fig. 2-491).

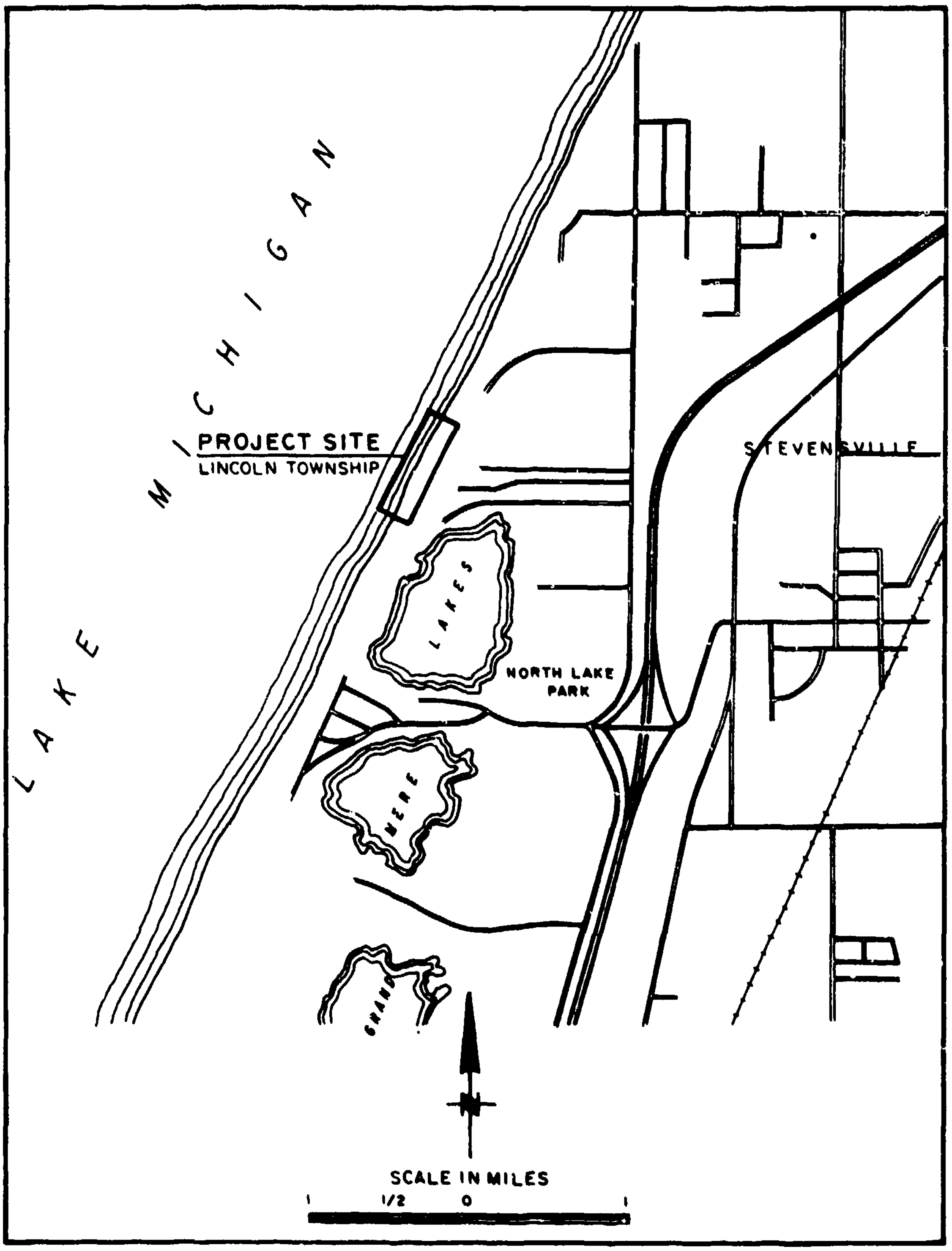


Figure 2-491. Location map of Lincoln Township, Michigan, monitoring site.

(b) Water Level and Wave Conditions. Water level fluctuations for Lakes Michigan and Huron are shown in Figures 2-385 and 2-386. Because the lakes are hydraulically connected by the Straits of Mackinac, the water level fluctuations are applicable to both lakes. The water level of Lakes Michigan and Huron (576.8 feet LWD IGLD) is largely a function of inflow through the St. Mary's River and discharge through the St. Clair River. The highest annual fluctuation of monthly mean lake levels in a 115-year period of record (1860-1974) was 2.20 feet. Lake levels reached a near-record high of +4 feet LWD in 1974, then dropped to about +1.5 in 1977, and has since risen to about +3 LWD. Short-term fluctuations above and below these yearly averages are induced by wind stress and barometric changes associated with meteorological conditions. These effects can alter the water level in some areas by 3 or more feet.

At the site, the highest waves generally approach from the northwest quadrant, developing over effective fetch distances of more than 100 miles. Waves developing from southeasterly winds are lower in height because of the shorter fetch. The net direction of littoral movement is to the south. No LEO station was established at this site, but the wave climate is classified as severe.

(c) Geomorphology, Soils, and Vegetation. The site in Lincoln Township Park extends along the shoreline about 800 feet. Like the other Lake Michigan sites, the project area consists typically of sand dunes of various heights. At the site, a 20-foot-high sand dune is fronted by a 15- to 20- foot-wide beach. The beach and foreshore area slope is approximately 1 on 10. The bluff is covered with less than 25-percent vegetation.

(d) The Problem. This site is a typical example of beach erosion and dune recession, periodic but frequently recurring problems along the southeast Lake Michigan shoreline. The rate of erosion is directly related to lake levels, being more severe when levels are high. Historical records for the site are not available, but estimated erosion rates average 2 to 4 feet annually. The erosion results from wave attack at the base of the dune during storms and periods of high lake levels. Waves breaking or running up to the base of the dune loosen and suspend the sand in shallow water where it is then transported by offshore currents to form bars or is carried from the area by longshore currents.

(2) Monitoring Project. The shore protection structures at this site, installed under the Michigan Demonstration Erosion Control Program, comprise two groins 240 feet apart. Two types of groins were used: a 40-inch- diameter Longard tube, 120 feet long, and a 90-foot-long timber-pile groin (Figs. 2-492 and 2-493). This site was monitored each spring and fall and after most major storms until the Michigan monitoring program ended in fall 1976.

The Longard tube was installed in early fall 1973 (at a cost of $30 per foot of groin) without a filter-cloth foundation to test the effectiveness of setting such a structure directly on a sandy lake bottom. The timber-pile groin was completed under icy conditions in late fall and winter of 1973 (at a cost of $50 per foot of groin).

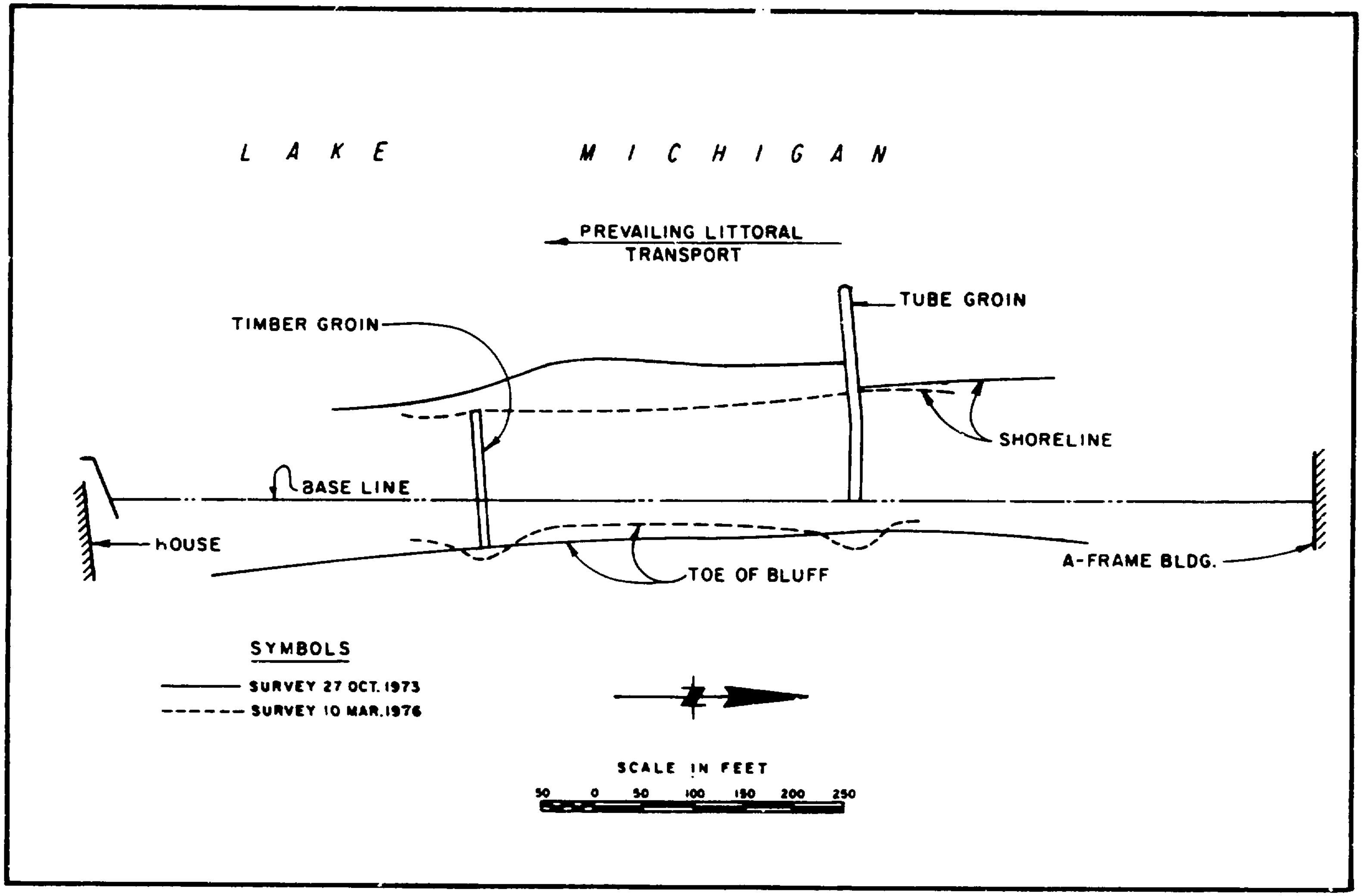


Figure 2-492. General plan of Lincoln Township, Michigan, monitoring site.

Figure 2-493. The Longard tube is shown in the foreground, the timber-pile groin in the background, Lincoln Township, Michigan, 10 October 1973 (from Brater, 1974).

(3) Performance. During the first year after construction, both groins successfully protected the beach by trapping sand. However, the Longard tube suffered moderate damage when about 30 feet of the lake end of the tube was lost and the whole structure settled about 3 feet along the centerline. The rate of settlement increased again in fall 1975 after remaining relatively stable for about 1 year. It has not been determined if this settlement was due to sand washing out of the tube from the lakeward end or to the tube sinking. The tube was still effective as a groin in trapping sand, thereby protecting the bluff, although some bluff recession has been recorded near this structure. However, in less than 3 years after installation its effectiveness was greatly reduced because of tears in the fabric and loss of sand. By summer 1979 a major part of the tube had been lost, and the groin was no longer functional.

The timber groin performed well and showed no sign of deterioration (Fig. 2-494), thus providing additional evidence that wood is an excellent material for groin construction in the Great Lakes. An impervious timber-pile groin is an old and proven means of shore protection in areas where there is adequate littoral drift moving predominantly in one direction. The relatively wide spacing between the groins did not reduce their functional effectiveness, and the system has protected the site by trapping sand and raising the beach profile. Much of the sand retained by the Longard tube before it failed is still on the beach, and it together with that retained by the timber groin is still protecting the bluff, although some minor slumping has occured. This slumping is probably due to factors other than simply wave attack, such as wind erosion or heavy foot traffic on the bluff.

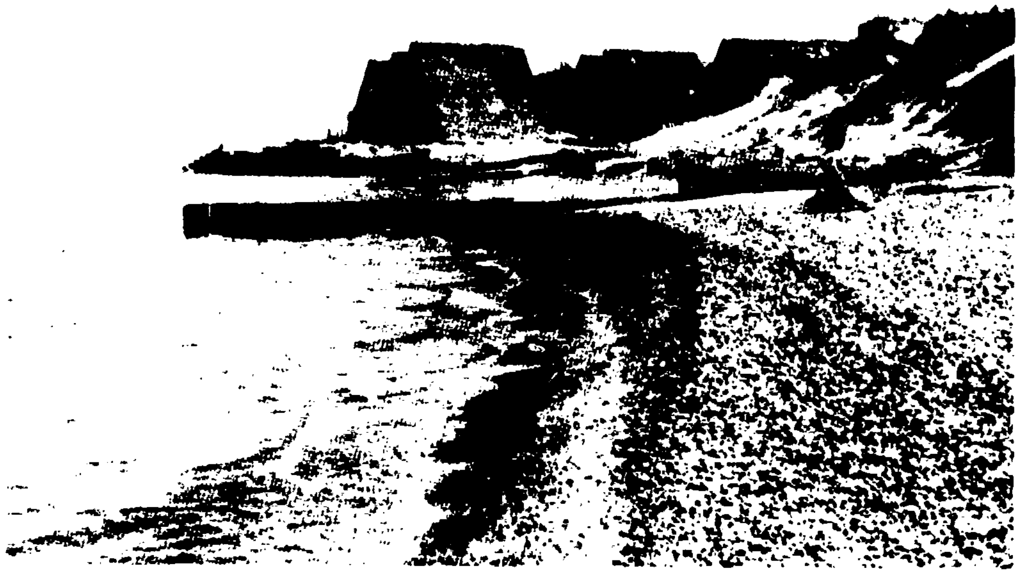


Figure 2-494. Timber groin in good condition, Lincoln, Township, Michigan, 6 May 1976 (from Brater, Armstrong, and McGill, 1977).

It was considered that little would be learned by continuing to monitor the Lincoln Township site, because only the wood-pile groin remained intact. Other similar groins were being monitored elsewhere; therefore, monitoring of the Lincoln Township site was discontinued in October 1979.

(5) Analysis. The Longard tube proved to have a short effective life at the site monitored. The settling of the tube might have been prevented by tying the tube back into the sand bluff. Also, Longard tubes are vulnerable to puncturing and tears by vandals and wave-borne debris. A sand-epoxy coating applied to all exposed dry surfaces of the tube could have detered vandalism and made the fabric less vulnerable to debris impact.

The timber-pile groin performed well and therefore proved to be an adequate shore protection device in this area where adequate littoral drift was moving almost unidirectionally along the shore. There was very little deterioration of the groin, and sand accretion significantly increased the width of the protective beach. Apparently, the rate of longshore transport was adequate to maintain the downdrift beach, as no damage due to downdrift erosion was reported.

g. Muskegon State Park, Michigan.

(1) Site Description.

(a) Geographical Setting. The Muskegon State Park monitoring site is located 12 miles northwest of the city of Muskegon, Michigan, on the northeast shore of Lake Michigan. It is accessible by taking U.S.

Highway 120 west to the site. The project occupies approximately 1,200 feet of the park's 6,000 feet of shoreline (Fig. 2-495). The park shoreline is straight and trends approximately N. 30° W.

(b) Water Levels, Waves, and Longshore Transport . Water level information for Lake Michigan is given in the site description for the Lincoln Township project. Waves approaching Muskegon State Park are predominantly from the southwest because of the 120-mile fetch in that direction. Westerly and northwesterly waves, generated over fetch lengths of 70 and 75 miles, respectively, are also an important part of the wave climate. In fact, as a result of refraction effects and the orientation of the shoreline, the net direction of longshore transport is southeastward. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum 7.1 feet; the wave climate is classified as severe and the net longshore transport potential northwestward was 95,200 cubic yeards for the 2 months analyzed. This indicated reversal of the long-term net direction of longshore transport is probably due to atypical wind conditions during the 2 months of records analyzed.

(c) Geomorphology, Soils, and Vegetation. The Lake Michigan shoreline near Muskegon consists of sand dunes or bluffs common to the west coast of Michigan. A narrow, 20- to 30-foot-wide beach has a slope of about 1 on 25 for a distance of about 200 feet offshore, where the first of a series of four sandbars exist. After the first bar, the slope is more gentle, about 1 on 100, to a distance of 2,000 feet offshore. The beach is backed by a low sand bluff with a slope of about 1 on 1; sand dunes 20 to 40 feet high extend beyond the bluff. The sand dunes are typically well vegetated with red oak, white pine, white birch, alder, maple, hemlock, and juniper.

At the project site, artificial fill was placed to protect the natural bluff. The fill, consisting mostly of dune sand, was placed dry and compacted only by a bulldozer used for grading during placement. The fill is approximately 1,200 feet long, 20 feet high, and extends lakeward 40 feet beyond the natural shoreline. The shoreward side of the fill stands on a slope of about 1 on 4.

(d) The Problem. The Muskegon site is a typical example of erosion due primarily to direct wave attack and littoral transport. Because of the lack of a wave-absorbing beach at the site, waves impinge directly on the toe of the bluff, undermining it and causing large amounts of bluff material with the mature vegetation growing on it to slide into the water.

(2) Monitoring Project. The project comprises the riprapped toe of a high sandfill which extends from Scenic Drive out to the lake some 50 to 125 feet. The sandfill rises to a height of almost 25 feet above LWD; its slope is roughly 1 on 2 (Fig. 2-496). The sandfill contains some riprap-sized stones and small amounts of gravel, wood chips, and cinders (Fig. 2-497). Although the structure now has a riprap toe, it was not placed at the site as part of the original design. The toe actually resulted naturally from the selective sorting under wave action of the fill material which contains large stones with diameters ranging from 6 inches to 3 feet

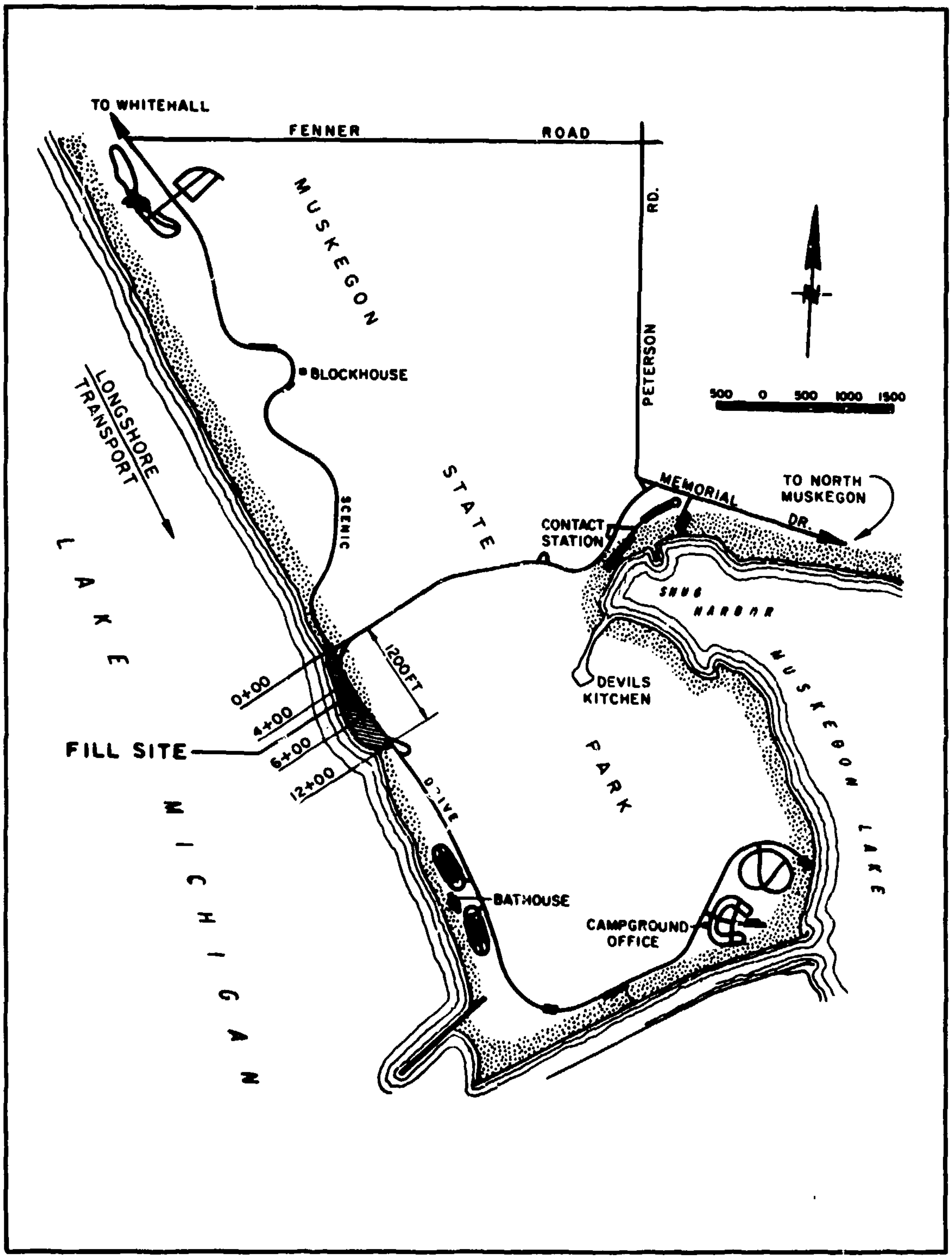


Figure 2-495. General plan of Muskegon State Park Michigan, monitoring site.

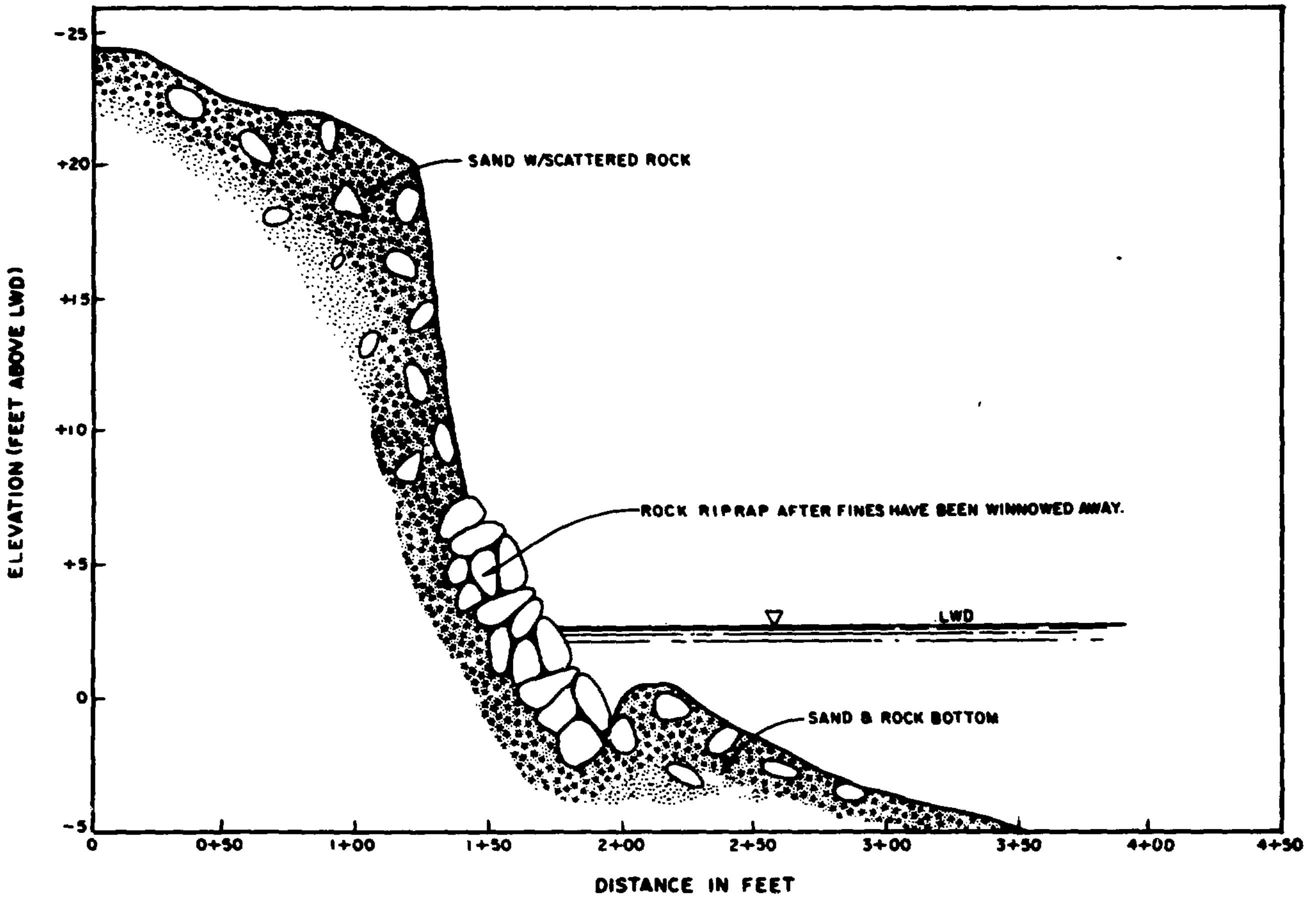


Figure 2-496. Typical riprap section at Muskegon site.

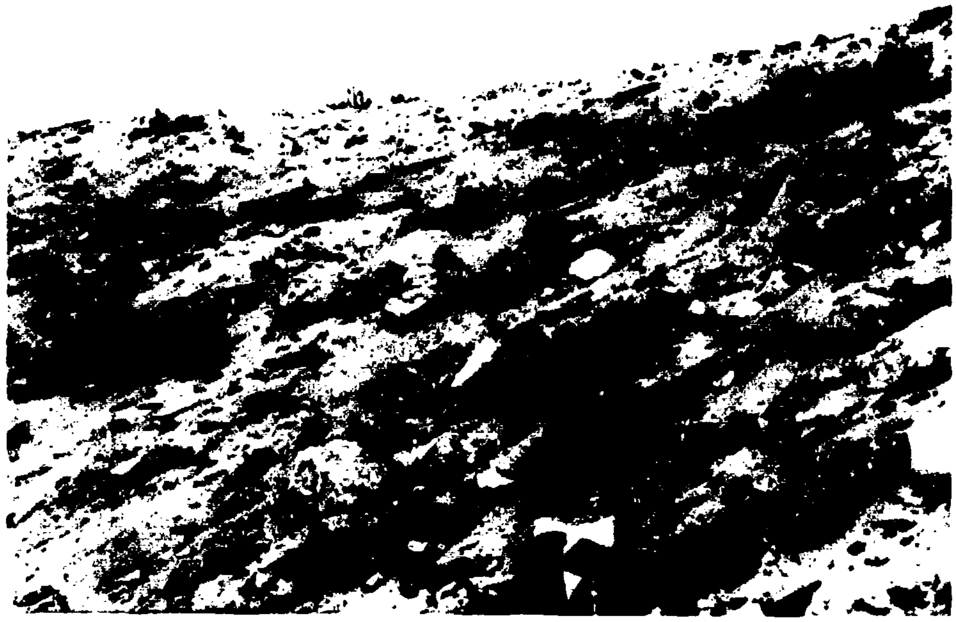


Figure 2-497. Riprap on slope at midpoint, Muskegon State Park, Michigan, August 1979.

(Fig. 2-498). At the north end of the fill, the slope contains tree stumps and pieces of broken pavement (Figs. 2-499 and 2-500).

The original sandfill, installed in 1978 by the Muskegon County Road Commission, was only 800 feet long, but it was extended to 1,200 feet in August 1979. The sandfill was placed at the site for two reasons: (a) to protect Scenic Drive from being undermined, and (b) to provide nearby contiguous shore areas with littoral material. When the original idea of beach nourishment for this area was considered, it was understood that periodic renourishment would be provided as needed. In September 1979, the slope was graded and seeded with Pacific cordgrass (Spartina foliosa).

(3) Performance. When first visited by the Detroit District monitor in August 1979, the fill appeared to be stable and no erosion was evident. There was no significant change in the structure at the second site inspection, but grading of the fill area just before the second visit may have obscured any erosion of fill materials along the slope that might have occurred since the initial visit.

In November 1979 erosion was evident; an indentation had formed along the top of the slope near the south end, apparently caused by the loss of a considerable amount of fill which slid down the slope into the water. The indentation was approximately 100 feet long and extended 10 to 20 feet into the slope (Fig. 2-501). Light vegetation had begun to grow along the top of the fill (Fig. 2-502).

Figure 2-498. Naturally formed riprap looking north along toe, Muskegon State Park, Michigan, August 1979.

Figure 2-499. Looking north at slope containing tree stumps and pieces of broken pavement, Muskegon State Park, Michigan, August 1979.

Figure 2-500. Tree stumps on slope at north end, Muskegon State Park, Michigan, August 1979.

Figure 2-501. Indentation at top of slope, Muskegon State Park, Michigan, November 1979.

Figure 2-502. Settlement of larger material at toe of slope, Muskegon State Park, Michigan, November 1979.

By the end of December 1979, the erosion of the slope had continued at the south end (Fig. 2-503) and additional erosion was evident at the north end. More slides were evident, as the top of the slope had moved landward in three places. Each indentation was about 30 feet long and extended 5 feet into the slope. By May 1980 erosion of the slope had continued at a relatively slow rate with no significant changes. In addition, the vegetation growth was negligible. This sandfill is eroding in the same manner as natural bluffs erode in the Great Lakes region. The waves impinge directly on the toe of the beach, undermining it and causing the slope to be unstable. This occurs despite the large rocks covering the toe which seemingly act as riprap. No noticeable accretion occurred at the beaches in the vicinity of the sandfill as of May 1980 suggesting that the sandfill was not serving its purpose as a supplier of littoral material to nearby beaches. Figure 3-44, which was put in Section III for comparison of similar devices, shows a series of profiles through the stone riprap revetment. It shows considerable fluctuation of offshore depths, probably the effect of varying rates of longshore transport in the nearshore area just lakeward of the revetment.

(4) Analysis. The continued sliding of material down the face of the sandfill indicates that the slope is too steep for this type of material. The instability may be due to the method of placement, as the fill was initially dumped, and apparently it took a 1 on 2 angle of repose. Material on the exposed slope then slid quite readily when rainfall, wave action, freeze-thaw cycles, and other environmental forces altered the slope-stability characteristics of the fill. As a means of protecting

Figure 2-503. Indentation at top of slope along south end of fill, Muskegon State Park, Michigan, December 1979.

Scenic Drive from undermining, the sandfill serves a purpose as long as it is understood that it is necessary to periodically renourish the site. Since this may not be the most practical way to protect the road, perhaps the same sandfill with an adequately designed rock revetment on filter cloth would be more advisable. As a means of supplying littoral material to downdrift beaches the sandfill does not seem to be performing adequately. Although fine material is being washed away from the sandfill, none is appearing downdrift. To be effective, a beach nourishment project requires the selection of fill material of approximately the same grain size (or coarser) and gradation as the native littoral material.

h. Tawas Point, Michigan.

(1) Site Description.

(a) Geographical Setting. Tawas Point monitoring site is on the west shore of Lake Huron at the Tawas Point U.S. Coast Guard Station in Iosco County, Michigan (Fig. 2-504), 15 miles south of Oscoda and 84 miles north of Saginaw. It is accessible by taking State Highway 23 north to Tawas City and Point Road to Tawas Point. The shoreline has an approximate north-south alinement and is slightly curved. The project occupies approximately 400 feet of shoreline in front of the Coast Guard Station.

(b) Water Levels, Waves, and Longshore Transport. Water level data for Lakes Michigan and Huron are contained in the site description for the Lincoln Township project. Waves approaching the site are predominantly northeasterly to easterly and are wind-generated over fetch lengths of 115

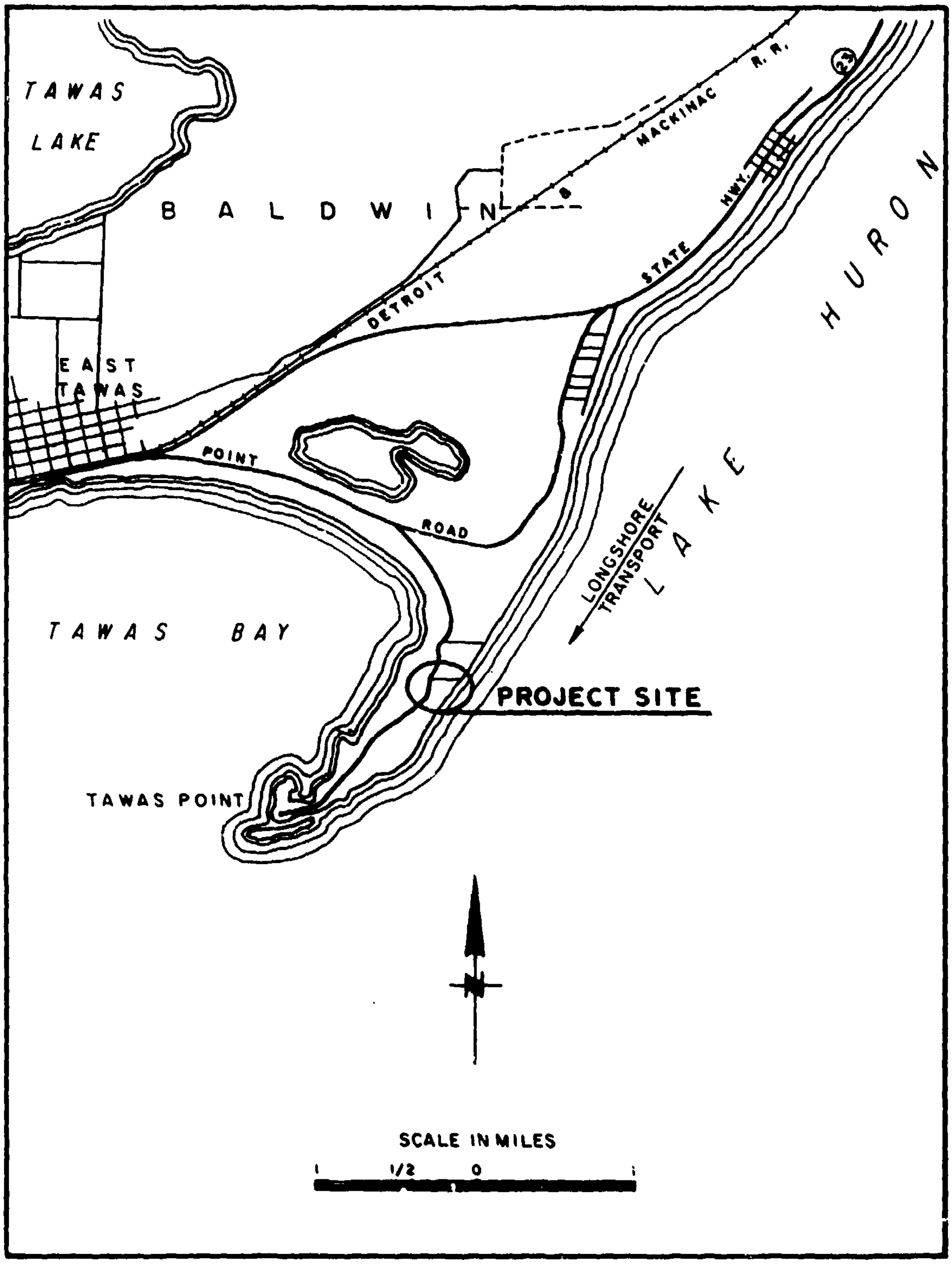


Figure 2-504. Location map of Tawas Point, Michigan, monitoring site.

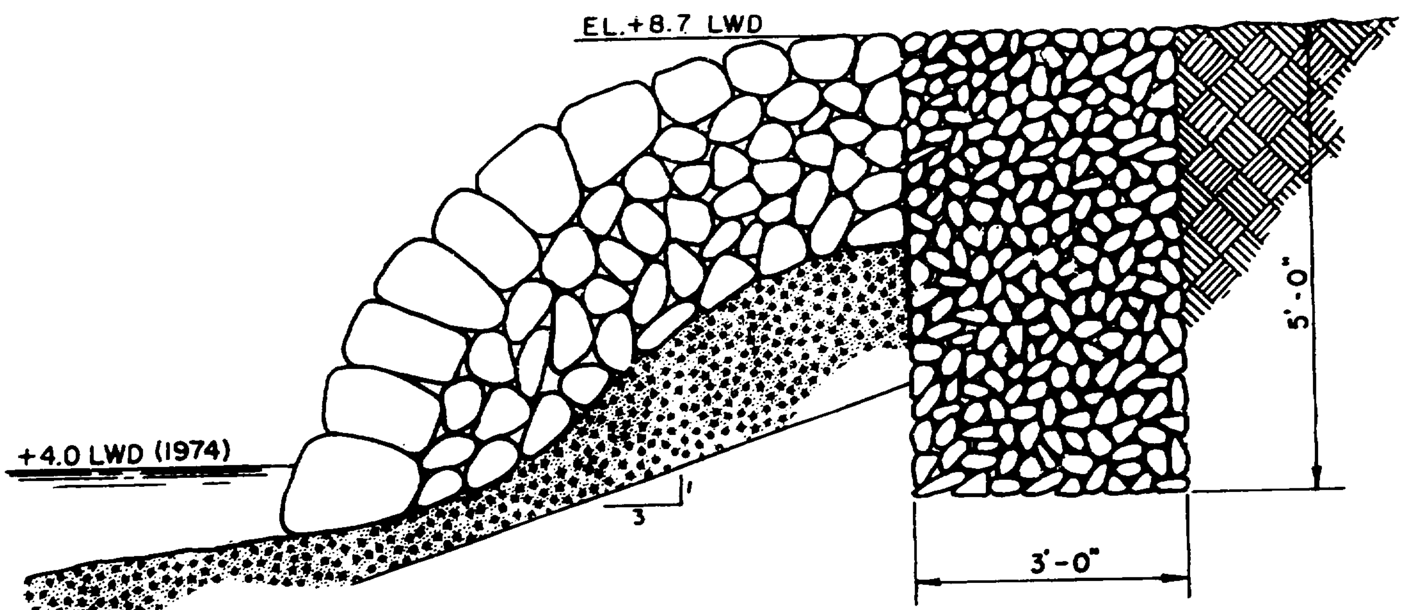
and 85 miles, respectively. Although the prevailing winds are from the southwest, these winds cannot generate high waves at Tawas Point because of the short (6 miles) fetch length. The net direction of littoral transport is to the south. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 4.3 feet; the climate is classified as severe, and the net longshore transport potential was (atypically) 535 cubic yards northward for the 1 month analyzed.

(c) Geomorphology, Soils, and Vegetation. The Tawas Point site is characterized by low bluffs composed of medium-grained sand, with less than 25 percent vegetative cover. Immediately behind the bluffs is a flat grassy area. No beach sediments are exposed at the bluff toe. However, sediments at State Park Beach, a small pocket beach approximately 1/8 mile south of the site, consist of medium- to fine-grained white sand.

(d) The Problem. This site is typical of the beach and bluff erosion problem common to the shoreline of the Great Lakes. Because of the lack of a wave-absorbing beach at this site, the waves impinge directly on the toe of the bluff. As a result, the bluff is undercut and the bluff material slides into the water and is carried away in littoral transport. The erosion process has been accelerated in recent years because of the rising lake levels.

(2) Monitoring Project. In summer 1974, a 400-foot rock revetment was installed at Tawas Point by the State of Michigan for the Michigan Demonstration Erosion Control Program (Fig. 2-505). To install the revetment, the 10-foot-high sand bluffs were graded to a 1 on 3 slope, and then a filter layer of 4- to 10-inch rock was placed on the slope. At the top of the revetment, a trench 3 feet wide and 5 feet deep was dug and filled with 1-to 3-inch rock. On the south half of the revetment, armor stone was placed only along the toe of the structure, to an elevation of about +6 feet LWD (Fig. 2-506). On the north half, armoring of the slope (the average piece weighing about 100 pounds) was carried to the crest of the structure (Fig. 2-507). Thus, the upper slope of the north half of the structure was armored, while the upper slope of the south half was left unarmored.

(3) Performance. Although only minor storms occurred during the first 2 years, slight slumping and shifting of the rock was evident. At the conclusion of monitoring by the State of Michigan in 1976, the structure was considered to have been very effective in protecting the shoreline. Although slight additional movement of rocks occurred after 1976, the rock revetment is still performing well, preventing further shoreline recession at the site. When monitoring under the Federal program began in September 1979, little further shifting of the rocks was apparent; however, in October 1979, a shallow depression (about 3 to 6 inches deep) along the landward edge of the revetment was noted (Fig. 2-506). This was apparently due to a slight settlement or shifting of rocks along the back of the revetment. On 26 November 1979, 39-knot winds from the south-southeast produced waves 8 feet high at the Tawas site (compared to an average sustained wave height of 4 feet during the 2 weeks before the initial visit on 4 September 1979). The 5 December 1979 visit revealed no significant changes since the October visit; this very effectively demonstrated the structural adequacy of the revetment. By 4 January 1980, the depression along the landward edge of the revetment had deepened to about 1 foot (Figs. 2-508 and 2-509). At



NORTH HALF

TYPICAL CROSS SECTION

NO SCALE

Figure 2-505. Rock revetment at Tawas Point site.

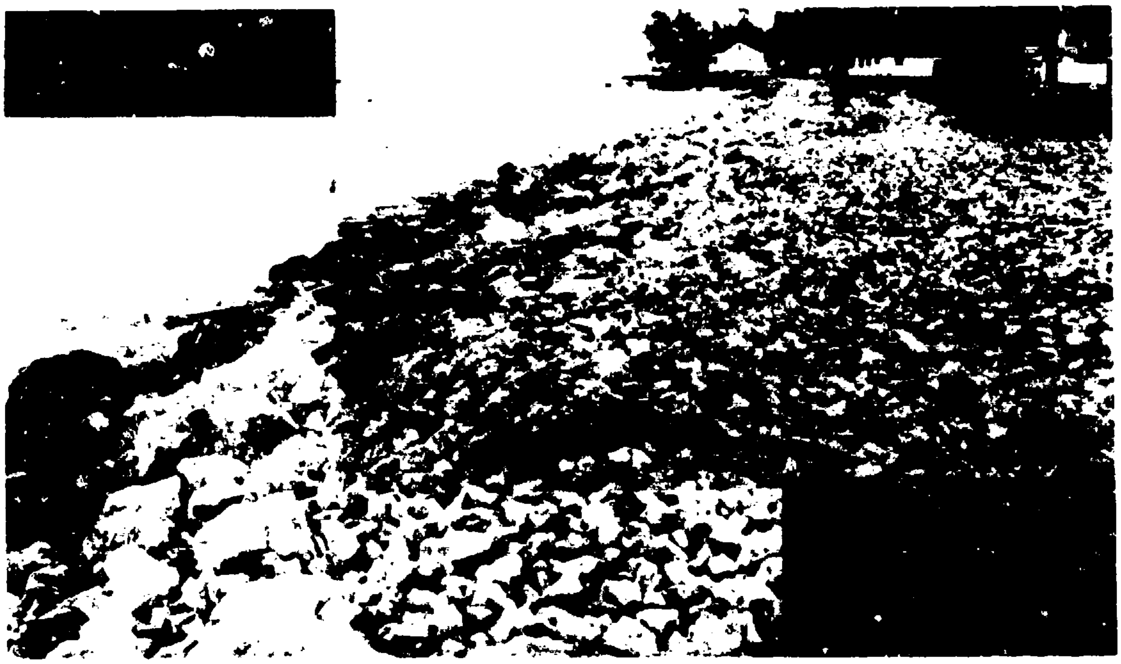


Figure 2-506. South end of revetment, Tawas Point, Michigan, 9 October 1979.

Figure 2-507. North end of revetment, Tawas Point, Michigan, 9 October 1979.

Figure 2-508. Looking south at depression along back of revetment, Tawas Point, Michigan, 4 January 1980.

Figure 2-509. Looking north at depression along back of revetment, Tawas Point, Michigan, 4 January 1980.

this time the depression was observed to extend only along the north end of the revetment.

After the ice breakup in March 1980, the condition of the revetment had changed slightly. The slope along the north end appeared steeper than in previous observations. This change in slope may have resulted from shifting of the larger rocks at the toe of the structure. The depression along the back of the north end of the revetment deepened to approximately 1.5 to 2.0 feet (Fig. 2-510). Figure 3-44, which was put in Section III for comparison of similar devices, shows a series of profiles through the Tawas Point revetment. The accretion occuring in front of the structure and settlement along the back of the revetment are evident in the 1980 profile.

(4) Analysis. The rock revetment at Tawas Point demonstrated an ability to withstand more than 5 years of wave exposure, and it prevented further erosion of the sand bluffs. Although the depression along the back of the revetment was probably a result of the extreme wave heights, the structure sustained no significant damage and remained functionally effective. It is questionable whether the trench filled with filter stone at the top of the revetment was necessary. Otherwise, the design of the revetment section in the north half of the structure closely followed current state-of-the- art practice for good rock revetment construction. Many coastal engineers contend that the stone size in the armor layer and the layer thickness may be made progressively less from the highest point of direct

Figure 2-510. Depression behind the north end, Tawas Point, Michigan, 12 May 1980.

wave impact to the upper limit of wave runup on the structure. The omission of heavy armor stone in the upper part of the south half of the Tawas Point installation was a test of this contention. Although no significant deterioration in the lighter weight section was observed, more time is needed for a better determination of its adequacy for long-term performance.

i. Sanilac Section 11, Michigan.

(1) Site Description.

(a) Geographical Setting. This monitoring site is located at a roadside park in Section 11 of Sanilac Township, Michigan, on the west shore of Lake Huron (Fig. 2-511). The site is located about 3 miles south of Port Sanilac and is accessible by taking U.S. Highway 25 south along the coast to the site. The roadside park is under the jurisdiction of the Michigan Department of State Highways and Transportation. The project occupies approximately 400 feet of shoreline which is curved and irregular but generally on a north-south alinement.

(b) Water Levels, Waves, and Longshore Transport. Water level data for Lakes Michigan and Huron are presented in the site description for the Lincoln Township project. Waves approaching the site are predominantly from the north and northwest, and are generated over fetch lengths of 165 and 140 miles, respectively. Although the predominant winds are from the south to southeast, the waves generated by them have little effect on the Sanilac site because the fetch length is only 12 miles or less. Longshore transport appears to be predominantly southward. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 2.5 feet. The wave climate is classified as intermediate. Although no data were available for wave energy-flux analysis, the problem in this area does not appear to result from undirectional longshore transport but rather from direct wave attack at the bluff line.

(c) Geomorphology, Soils, and Vegetation. The area is composed of very steep, 25- to 35-foot-high clay bluffs with about a 50 percent vegetative cover. The beach is composed of fine-grained, white sand. At the time of project implementation in 1974, the beach was 5 to 10 feet wide. There is no offshore bar at the site.

(d) The Problem. This site is representative of the beach and bluff erosion common along the shoreline of the Great Lakes. Because the site lacks a wide wave-absorbing beach and has no offshore bar, high waves impinge directly on the bluff toe. The undercut bluff then becomes unstable and slides into the lake, where the material is removed by littoral processes. As a result, the bluffs have been actively eroding; the recession rate before 1974 was approximately 1 to 2 feet per year. Sections of the parking lot and driveway at the roadside park are now less than 10 feet from the top of the bluff.

(2) Monitoring Project. In late summer of 1974, the Michigan Department of State Highways and Transportation installed a seawall, comprising two segments of 69-inch-diameter Longard tube, for the Michigan Demonstration Erosion Control Program (Fig. 2-512). The two Longard tubes were placed end-to-end on the beach at the toe of the bluff parallel to

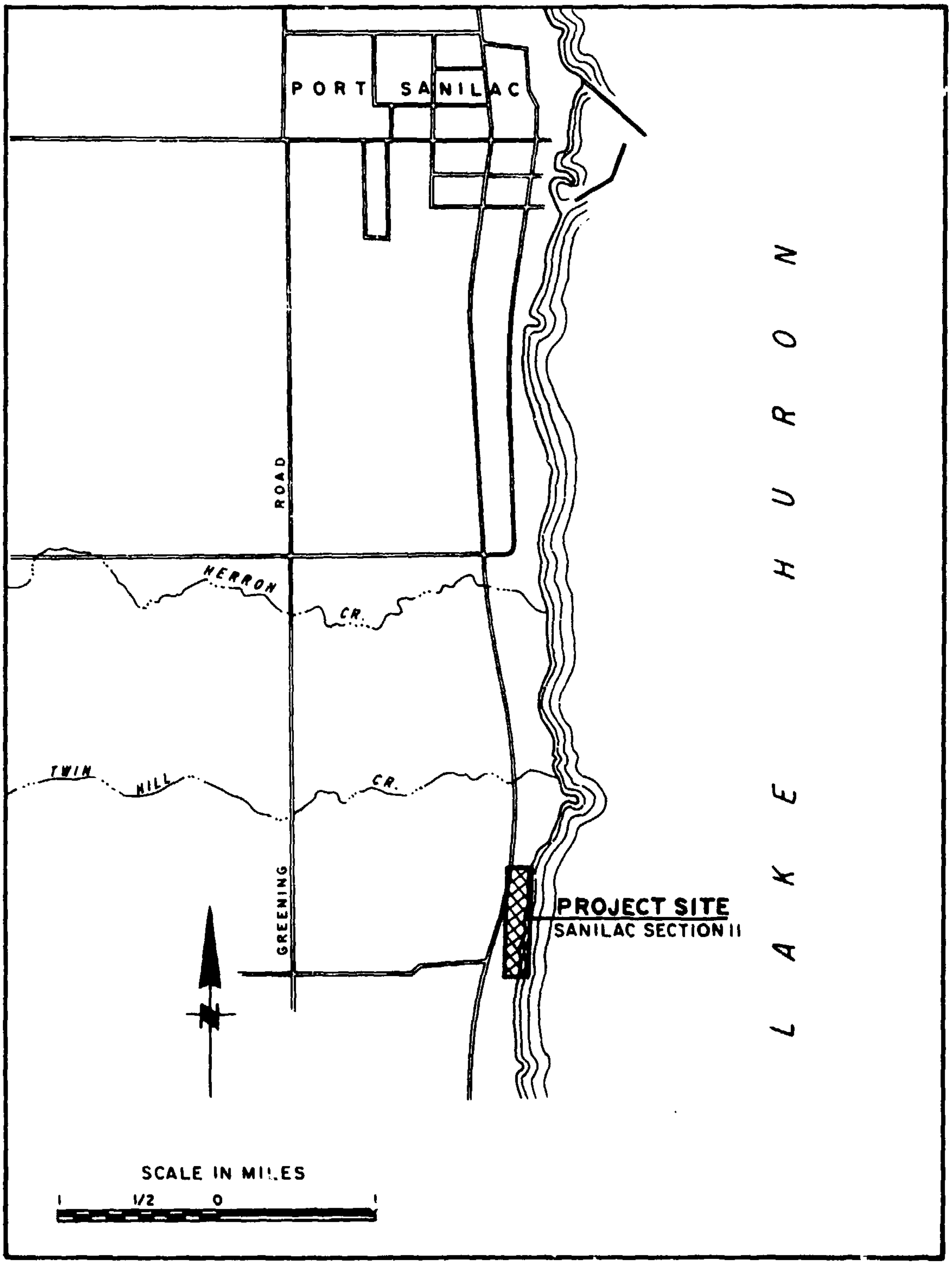


Figure 2-511. Location map of Sanilac Section 11, Michigan, monitoring site.

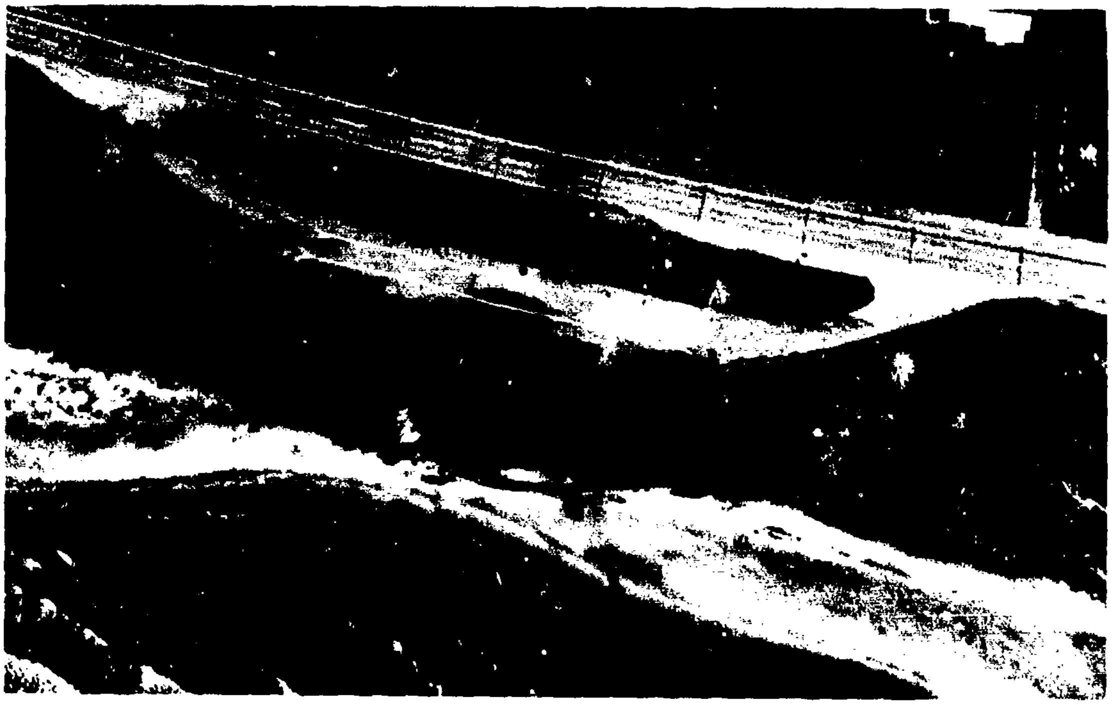


Figure 2-512. Location of Longard-tube seawall, Sanilac Section 11, Michigan, 11 December 1979.

the shoreline where the sandline was at an elevation of +3 feet LWD (Fig. 2-513). The total length of the seawall was approximately 400 feet. A filter layer was not considered necessary because the veneer of sand covering the solid clay lake bottom was only a few inches thick. The cost of construction was approximately $65 per foot.

(3) Performance. Minor shifting of and damage to the tubes were revealed in fall 1975. Some sand had been lost at the center of the structure where the two tubes meet, and the northern end had moved lakeward about 5 feet. A storm severely damaged the structure in April 1976. Material sliding down the steep bluff accumulated against the tube, parts of the tube collapsed, and the entire structure rolled toward the lake.

When first visited by the Detroit District monitor in September 1979, part of the tube was nearly buried (Fig. 2-514). This could have been caused by slumping of the slope or settlement of the tube or both. Sand also began to spill through a large tear near the south end of the tube. This tear (cause unknown) is shown in Figure 2-515 as it appeared 1 month later. By January 1980, the tear had lengthened and a substantial amount of sand had been lost from the tube (Fig. 2-516).

Storm activity was minor during the first 18 months after installation. Photographic records show that, during is period, the bluff recession rates north and south of the structure exceeded the rate at the site. However, during a severe storm in April 1976, the bluff slumped behind the

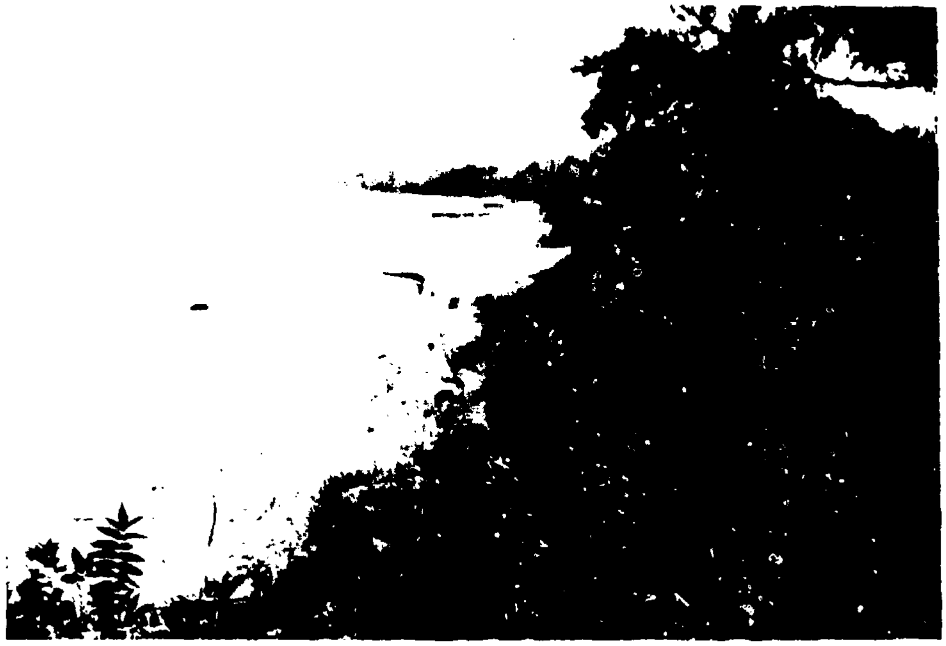


Figure 2-513. Looking south from top of slope, Sanilac Section 11, Michigan, 24 September 1979.

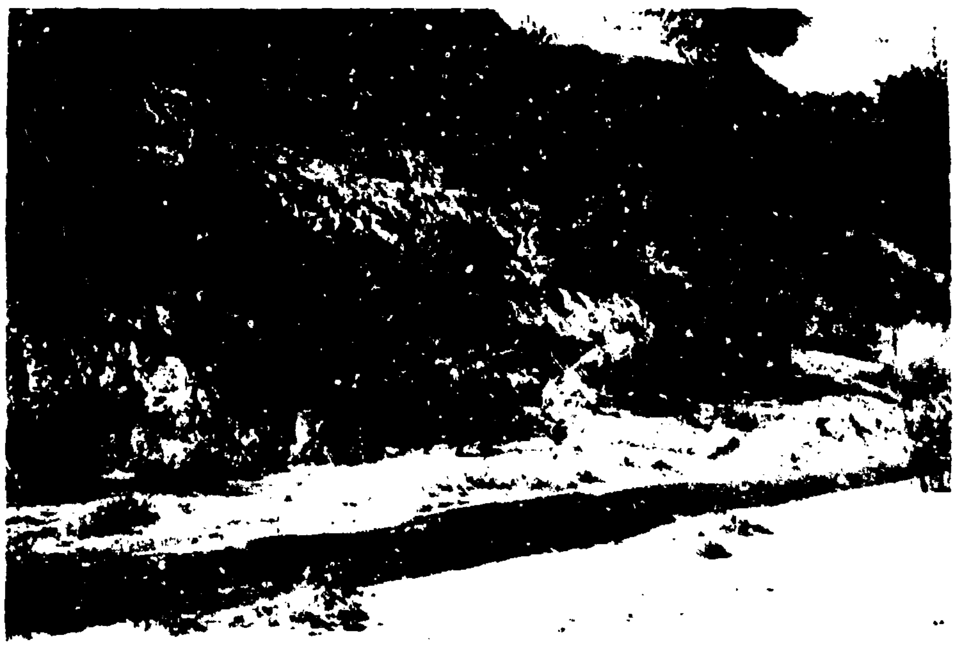


Figure 2-514. Looking northwest from midpoint of structure, showing nearly buried parts of tube, Sanilac Section 11, Michigan, 24 September 1979.

Figure 2-515. Tear in Longard tube at south end, Sanilac Section 11, Michigan, 29 October 1979.

Figure 2-516. Tear in tube 2.5 months after the above photo, Sanilac Section 11, Michigan, 10 January 1980.

tubes. Additional slumping occurred during the succeeding 3.5 years before the initial visit by the District monitor in September 1979. In January 1980, continued slumping was evidenced by fallen trees on the bluff (Fig. 2-517). At that time, a major collapse, behind the tear in the Longard tube, was noted (Fig. 2-518).

In April 1980, water seepage from the bank occurred at several points, probably associated with bank slumping (Fig. 2-519). Although the beach profile was lower than in September 1979, the accretion trend was building up the beach to a normal summer beach profile. High lake water turbidity seemed to indicate this building trend because of the heavy sediment load being carried. The beach immediately south of the site changed to the same extent as at the site, while the beach north of the site did not change as substantially. During the monitoring period, the beach widened, attaining a width from 50 to as much as 100 feet in some places. This widening could be attributed to beach accretion or lowering lake levels. Figure 3-20 (see Sec. III) shows a series of profiles through the longard tube bulkhead. The profiles for July 1979 and November 1979 show about 2 feet of accretion in front and behind the structure.

(4) Analysis. The State of Michigan concluded in a 1976 study that simply placing a single Longard tube at the toe of a steep bluff on an open coast is not an effective method of shore protection. Since that time, the degradation and settlement of the tube have resulted in bluff erosion to the extent that the recession rate approximately equals that of the adjacent unprotected bluffs. The collapse of the bluff behind the tear

Figure 2-517. Fallen trees indicating additional slumping, Sanilac Section 11, Michigan, 10 January 1980.

Figure 2-518. Major collapse near south end of tube, Sanilac Section 11, Michigan, 10 January 1980.

Figure 2-519. Water seepage along the bank. Note bank slumpage. Sanilac Section 11, Michigan, 16 April 1980.

in the Longard tube could be partially attributed to the tear, as the segment of tube near the tear was pushed lakeward; however, the extent to which bluff erosion contributed to the failure of the tube is unknown. It is possible that the collapse occurred first, making it easier for the slumping bluff to push the tube lakeward.

The attempted use of the Longard tube as a bulkhead or seawall to halt bluff erosion at the Sanilac Section 11 site was considered a successful failure. Some degree of success might have been achieved by (a) grading the bluff to a stable slope before tube installation; (b) installing the tube about 30 feet lakeward of the bluff toe so that overtopping waves would build a back berm instead of eroding the bluff; (c) coating the exposed surfaces of the tube with sand-epoxy to deter vandalism and to provide some degree of armoring against tears by w[illegible]borne debris impact; and (d) providing a more secure junction for the [illegible]ubes.

j. Sanilac Section 26, Michigan.

(1) Site Description.

(a) Geographical Setting. The Sanilac Section 26 monitoring site is located 4 miles south of Port Sanilac and 1 mile south of the Sanilac Section 11 monitoring site, on the west shore of Lake Huron, in Sanilac Township, Michigan (Fig. 2-520). The site, located at a roadside park under the jurisdiction of the Michigan Department of State Highways and Transportation, is accessible by taking U.S. Highway 25 south along the shoreline to the park. The project occupies about 2,000 feet of shoreline which trends approximately N. 10° W. and is slightly curved and irr[illegible]gular.

(b) Water Levels, Waves, and Longshore Transport. Water level data for Lakes Michigan and Huron are given in the Lincoln Township section of this report. The wave conditions for Sanilac Section 26 are similar to those of Sanilac Section 11. The LEO data (Table 1-3) indicate that wave heights average from 0 to 1 foot, with a maximum of 4.0 feet. The wave climate is classified as intermediate. Accumulations of material on the north sides of the groins indicate a net southward longshore transport of moderate potential.

(c) Geomorphology, Soils and Vegetation. This site is characterized by a steep, moderately high bluff consisting of clay till covered with less than 25 percent vegetation. The bluff is 30 feet high and slopes at an angle of about 1 on 1.5. A thin mantle of sand and gravel covers the beach, which ranges from 9 feet to about 40 feet in width, depending on the lake level. The offshore area has a hard clay till bottom. Both the beach and the offshore bottom slope at about 1 on 100.

(d) The Problem. This site is a typical example of beach and bluff erosion which has been occurring at this site for many years. By 1973, the bluffs had become very steep, with a potentially unstable slope. Bluff recession has amounted to as much as 6 feet per year, and undermining of the parking lot at the roadside park on top of the bluff had occurred. In addition to erosion by wave action, recession of the till bluff could be

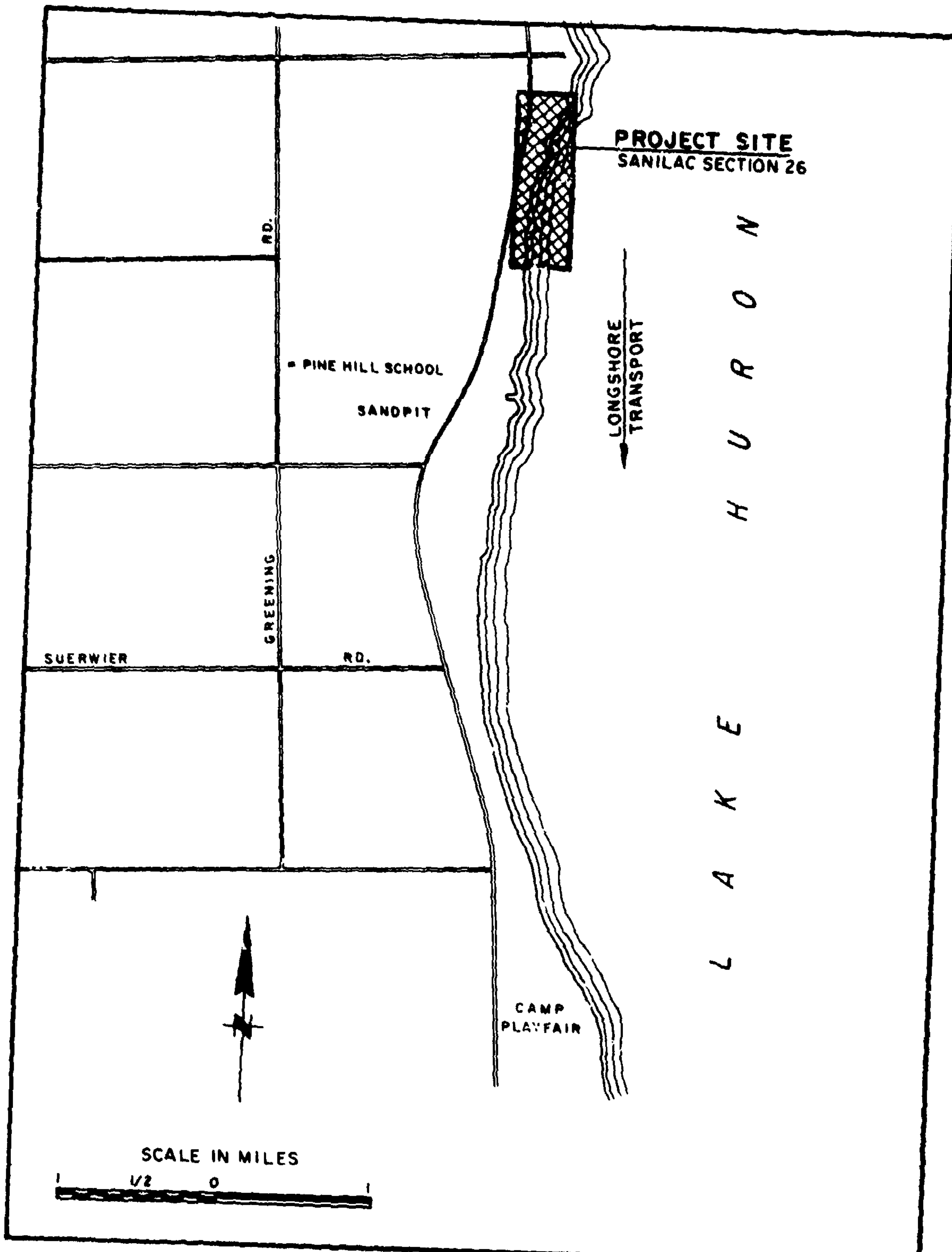


Figure 2-520. Location map of Sanilac Section 26, Michigan, monitoring site.

attributed to clay slumping induced by factors such as wind, rain, and frequency of freeze-thaw cycles in fall and spring.

(2) Monitoring Site. In 1973, the State of Michigan selected this site for testing six different groins for the Michigan Demonstration Erosion Control Program. The materials selected for construction of the groins were, respectively, two 40-inch-diameter Longard tubes placed side-by-side, one 69-inch-diameter Longard tube, gabions, sandbags, rock mastic, and a rock-filled timber crib (Figs. 2-521 and and 2-522).

An added factor in this study was the unusually large spacing between groins. Normally, the spacing between any two groins does not exceed two times the distance that each groin extends into the water. In this study, the groins were spaced at about three times the wetted length. Because groins were known to be effective in this area, evaluation of devices was concerned with their structural adequacy and the 3 to 1 space-to-length ratio. Filter material was not used because little or no settling was anticipated. Figure 2-523 shows the locations of the various groins.

(3) Longard Tubes (40-inch diameter).

(a) Construction. Two 40-inch-diameter Longard tubes each 100 feet long, were installed side-by-side in the fall of 1973. The groin was constructed at a cost of $55 per linear foot of groin. Representing 300 feet of shoreline, the cost is $18 per foot of shoreline.

(b) Performance. At the termination of the Michigan program in 1976, the southern tube had settled about 1 foot and the northern tube had settled about 0.7 foot. It was concluded that the tubes had performed effectively in trapping sand and building up the beach. However, by 1979 the beach profile was lowered about 1.5 to 2 feet, apparently as a result of erosion by water currents generated by a small stream outlet north of the site. Nevertheless, no significant erosion or slumping of the bluff was evident. When first visited by the District monitor in November 1979, the northern tube was not visible and the southern tube was partially buried (Fig. 2-524). At the second visit in December 1979, both tubes were exposed full length from bluff to waterline; the southern tube had settled 1.5 feet lower than the northern tube (Fig. 2-525). There was no apparent change in the alinement of the tubes. A large tear (about 2 by 3 feet) in the northern tube resulted in sand loss (Fig. 2-526); numerous small tears were also evident with sand loss from the lakeward end. The southern tube sustained less damage; however, the lakeward end also lost sand and was partially buried in gravel (Figs. 2-525 and 2-527). By April 1980, the beach appeared to be slightly higher from sediment accretion on the north side of the tubes. The bluff receded at a slow rate due to slumping and erosion. In May 1980, the adjacent beaches were reported to have raised 1 to 2 feet and were about 26 feet wide, or about 10 feet wider than they were in April.

(4) Longard Tube (69-inch diameter).

(a) Construction. The 69-inch-diameter Longard tube was 70 feet long and was installed in the spring of 1974. The groin was constructed at a cost of $71 per linear foot or $24 per foot of shoreline.

Figure 2-521. View of site looking north, Sanilac Section 26, Michigan, 26 December 1979.

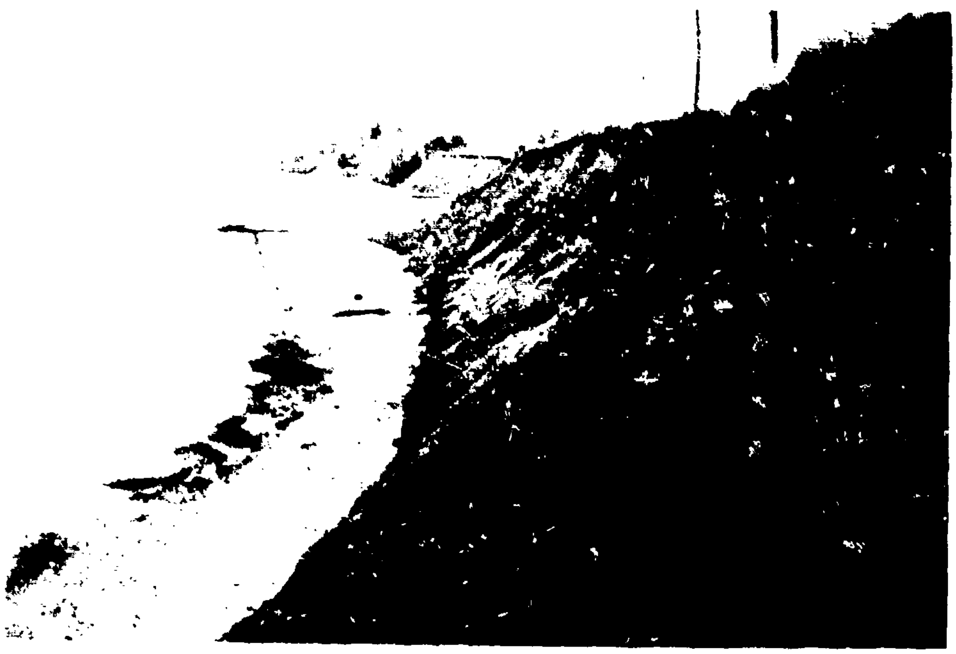


Figure 2-522. View of site looking south, Sanilac Section 26, Michigan, 26 December 1979.

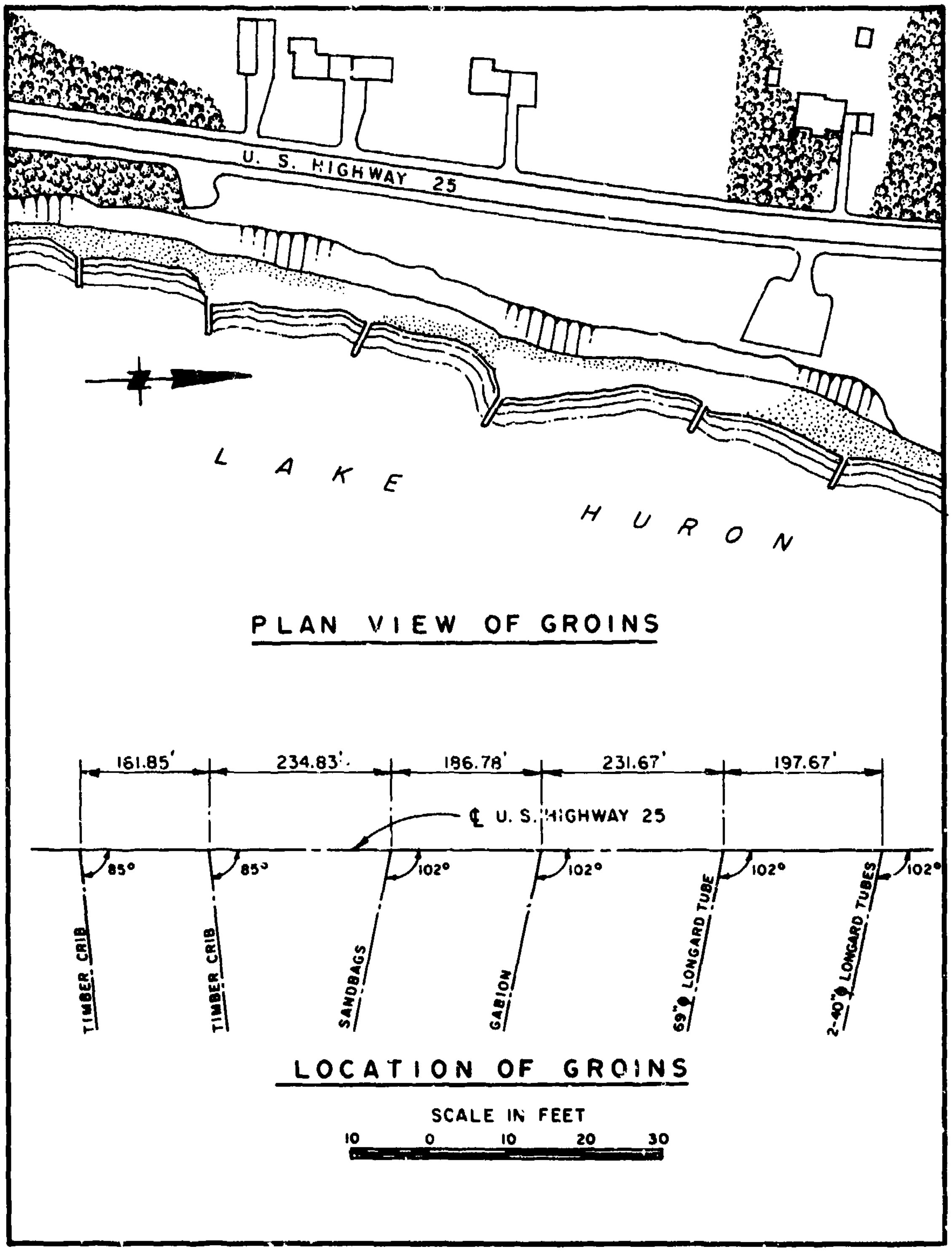


Figure 2-523. General plan of Sanilac Section 26, Michigan.

Figure 2-524. Two 40-inch-diameter Longard tubes; northern tube is not visible (looking north), Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-525. Two 40-inch-diameter Longard tubes, Sanilac Section Michigan, 26 December 1979.

Figure 2-526. Tear in northern 40-inch-diameter Longard tube, Sanilac Section 26, Michigan, 26 December 1979.

Figure 2-527. Damage to the 40-inch-diameter Longard tubes, Sanilac Section 26, Michigan, 26 December 1979.

(b) Performance. Although minor settlement occurred before 1976, the tube sustained little damage. The State of Michigan determined that the groin had effectively trapped sand, built up on the beach, and prevented direct wave attack on the bluff. In November 1979, only the top of the tube near the bluff was visible, and the lakeward end had shifted northward (Fig. 2-528). Although the beach profile appeared to be lower in December 1979, the bluff had remained stable and no slumping was evident. The tube was more visible and a tear approximately 1 foot long was evident about 10 feet lakeward from the bluff end. The lakeward end of the tube was open and a significant amount of sand had been lost (Fig. 2-529). By April 1980, the tube was fully exposed, and a significant amount of sand had been lost (Fig. 2-530). It was also noted that the beach was lower and narrower than the previous inspection, although the accretion trend was in progress. In May 1980, the adjacent beaches were 1 to 2 feet higher than previously. The beach sediment buildup was attributable to seasonal accretion.

(5) Gabions.

(a) Construction. The gabion groin was installed in the late fall of 1974 at a cost of $30 per linear foot or $9 per foot of shoreline.

(b) Performance. The State of Michigan concluded that the gabion groin was effective in trapping sand, despite scouring at the lakeward end of the structure. Slumping of the bluff, which occurred 1.5

Figure 2-528. Northward shift of lakeward end of 69-inch-diameter Longard tube, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-529. Loss of sand from 69-inch-diameter Longard tube, Sanilac Section 26, Michigan, 26 December 1979.

Figure 2-530. The Longard tube is exposed full length at Sanilac Section 26, Michigan, 9 May 1980.

years after installation, was due primarily to the tendency of the bluff material to adjust to a more stable slope. Bottom scour at the lakeward end of the gabion groin increased to the extent that this section of the groin began to collapse in fall 1979 (Figs. 2-531 and 2-532). Settlement of the rockfill inside the baskets was evident at that time. Although the bluff was unstable and slumping was evident at the November 1979 visit, both the beach and bluff were farther lakeward at this structure than on the downdrift side (Fig. 2-523). This was considered indicative of the effective performance of this structure. In December 1979, the beach profile appeared to be about 0.3 foot lower to the north and up to about 0.5 foot lower to the south of the structure than it had been during the previous visits. No additional bluff slumping was evident at the December visit. The first two tiers of baskets were fully exposed, and the third tier was exposed on the south side. In May 1980, the beach profile was 0.5 foot higher to the north and 1.0 foot higher to the south of the structure.

(6) Sandbags.

(a) Construction. The sandbag groin was installed in late fall 1973. Sand was brought from an inland source, dumped at the lake site, then pumped into bags. The large (9 by 3 by 2 feet) bags were held in place by their own weight and tied together with nylon leaders at the corners of each bag. Cost of construction was approximately $109 per linear foot of groin or $36 per foot of beach.

(b) Performance. Within 1 year after installation, the sandbag groin sustained major damage. Approximately 15 feet of the lakeward end of the structure was destroyed, with the deteriorated bags eventually ripping open; other bags were displaced from their original position. By November 1979, the groin was structurally and functionally ineffective. Few landward sandbags were visible, and the lakeward end of the structure appeared to have washed out (Figs. 2-533 and 2-534). The bluff remained unstable. In April 1980, slumping was evident and was most serious just south of the structure. The instability of the bank is influenced by water seepage through the bluff material. May profiles showed that the beach was 1 to 2 feet higher and that it had widened to about 30 to 40 feet due to seasonal accretion.

(7) Rock Mastic.

(a) Construction. The rock mastic groin was installed in fall 1973. The University of Michigan Coastal Zone Laboratory provided the design and supervised construction. Stone was stockpiled on top of the bluff, then pushed down the slope by bulldozers and placed in cuts made in the lake bottom. The groin was formed by piling the rock and pouring a hot asphalt mastic over the groin to fill the voids in the rock and to cap it. The cost of this structure was $154 per linear foot of groin or $43 per shoreline foot.

(b) Performance. The rock mastic groin was structurally and functionally effective in trapping a wide fillet of sand, which halted bluff erosion. By 1976, no movement of the groin was evident. However, a section of the mastic and underlying rock had broken off at the northern corner of the lakeward end (Fig. 2-535). The damage was minor and did not

Figure 2-531. Collapse of lakeward section of gabion groin, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-532. Loss of rock from collapsed section of gabion groin, Sanilac Section 26, Michigan, 26 December 1979.

Figure 2-533. Remains of sandbag groin, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-534. Lakeward end of sandbag groin, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-535. Location of section broken off of rock mastic groin, Sanilac Section 26, Michigan, 27 November 1979.

affect the performance of the groin. The November 1979 visit revealed no change in cross section and alinement, although a few rocks appeared to have been dislodged (Fig. 2-536). The beach at this structure extended farther out into the lake than did the adjacent beach, indicating successful performance. The bluff remained stable at the time of the December 1979 visit. The 16 April 1980 visit revealed that the bank had experienced some localized slumping due largely to bank water seepage. Although the beaches were narrower, the seasonal accretion built up the beach surface 1.5 feet and widened it to about 15 feet to the south and about 40 feet to the north of the groin.

(8) Timber Crib.

(a) Construction. The timber-crib groin was installed in the fall of 1975 at a cost of $30 per linear foot.

(b) Performance. The effectiveness of the timber-crib groin was not assessed by the State of Michigan because of insufficient monitoring time. The November 1979 visit by the District monitor revealed that the rockfill level at the lakeward end was about 1 to 1.5 feet below the top of the crib (Fig. 2-537). This could have resulted from settlement, loss of rock due to wave action, or vandalism. By December 1979, rocks in the lakeward section were about 2 feet below the top of the crib (Fig. 2-538). The crib was exposed only to midbeach in November 1979, but 1 month later it was exposed back to the toe of the slope, indicating a lower beach profile. The bluff appeared to be stable. By April 1980, the beach had narrowed significantly, especially north of the groin. By May the beach

Figure 2-536. Rocks dislodged from rock mastic groin, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-537. Settlement or loss of rockfill from timber-crib groin, Sanilac Section 26, Michigan, 27 November 1979.

Figure 2-538. Continued lowering of rockfill level in timber-crib groin, Sanilac Section 26, Michigan, 26 December 1979.

surface had risen 1.5 feet and the beach had widened to about 45 feet to the north and 25 feet to the south. This widening trend was probably attributable to seasonal accretion trends.

(9) Analysis. In general, the wider spacing between the groins did not appear to have reduced their effectiveness. An adequate fillet of sand was trapped by each structure during its effective life, building up the beach and preventing waves from impinging directly on the toe of the bluff. However, a few sections of the bluff were initially too steep, and some slumping occurred, probably due to adjustment of the bluff slope to a more stable profile.

Of the six structures studied at the Sanilac Section 26 site, the rock mastic groin has been the most effective structurally and functionally; however, this structure is not attractive from an esthetic viewpoint, and was the most costly to construct. Although minor damage occurred (when the section broke off at the lakeward end), the rock mastic groin remained stable. Large amounts of sand were trapped, preventing further shoreline erosion. The sandbag groin sustained the most damage and was structurally and functionally ineffective. Major damage occurred within 1 year after installation, and the structure was completely ineffective after 5 years.

The two Longard-tube groins, performed equally well, although the 69-inch-diameter tube was expected to trap more sand because of its greater height. At this site there appeared to be no advantage in using two Longard tubes side-by-side rather than a single tube. Both groins sustained an equivalent amount of damage; however, the 69-inch-diameter tube settled less.

Thus far, the timber-crib groin (the easiest structure to repair) and the gabion groin have deteriorated significantly, but they have been effective in building a protective beach.

Table 2-101 gives volume calculations for changes between profile stations that occurred from the first survey in July 1979 to the last survey in May 1980. The base line for the surveys is stationed from south to north, which is opposite to the direction of net longshore transport. The table shows that some groins were effective in trapping littoral material, whereas others did not perform adequately. The timber-crib groin did not adequately trap littoral material, as the beach eroded in the area near this structure; however, part of this erosion may have been due to the effectiveness of the rock mastic groin. The rock mastic groin adequately trapped sediment on its updrift side, and erosion was present downdrift of the structure (expected from groins), but this erosion seems to have led to the ineffectiveness of the timber-crib groin. Despite the destruction of the sandbag groin, accretion occurred near the structure; however, this accretion is attributed to the ability of the gabion groin to trap sediment rather than to the presence of the sandbag groin. The shore near the Longard tubes showed erosion throughout, due to the destruction of the tubes. The net loss of about 4,400 cubic yards of sand during the period between surveys is not considered significant, as about 5,400 cubic yards were lost from the entrapment areas of the failing Longard tube groins, whereas the structurally effective groins actually trapped an additional 1,000 cubic yards.

6. Alaska Sites.

a. Common Characteristics. The demonstration sites at Kotzebue and Ninilchik are located on the western and southern coast of Alaska, respectively (Fig. 2-539). Although quite different in many ways, these two sites share a common problem--all shore protection devices must be able to withstand ice action (i.e., static expansion and floe battering). The most significant difference between the two sites is the tidal variation. Tides at both sites have a diurnal inequality. However, at Kotzebue, the tides are minimal, while at Ninilchik the mean tidal range is 16.5 feet. Because of the severe climate, the use of vegetation as a shore-protection device at the Alaska sites was not considered practical.

b. Kotzebue, Alaska.

(1) Site Description.

(a) Geographical Setting. Kotzebue occupies a low-lying gravel spit at the tip of the Baldwin Peninsula which extends into Kotzebue Sound off the Arctic Ocean (inset, Fig. 2-540). The community of Kotzebue is primarily a transportation and service center for the surrounding area. Access is primarily by air and sea, as roads in northwestern Alaska are poor to nonexistent. The project site extends over 3,000 feet of shoreline which trends roughly N. 5° E. at the south end and N. 35° E. at the north end.

Table 2-101. Volumetric analysis of beach profiles at Sanilac Section 26, Michigan (17 July 1979 to 1 May 1980).

Device	Station	Erosion (yd3)	Accretion (yd3)	Net accretion (yd3)
	459+80	1,237.7	1,228.6	-9.1
	462+80	586.4	1,218.7	632.3
	464+80	231.4	457.8	226.5
	465+80	226.9	68.0	-158.9
	466+23	53.3	4.2	-49.0
Timber-crib groin	466+30	70.2	9.1	-61.1
	466+42	186.2	78.3	-107.9
	466+92	211.4	96.9	-114.4
	467+42	202.0	102.5	-99.5
	467+80	65.7	36.5	-29.1
	467+92	47.1	32.0	-15.0
Rock mastic groin	468+04	215.8	227.9	12.1
	468+74	244.2	270.7	26.5
	469+25	156.5	382.8	226.3
	469+76	92.6	272.5	179.9
Sandbag groin	470+15	51.8	61.7	10.0
	470+27	57.0	63.9	6.9
	470+40	51.3	201.9	150.7
	470+61	93.1	485.7	392.6
	471+12	199.2	288.5	89.4
Gabion groin	471+63	179.0	194.0	15.1
	472+07	32.9	12.1	-20.8
	472+14	52.5	8.9	-43.6
	472+23	80.4	18.2	-62.1
	472+41	243.8	69.6	-174.2
	472+92	187.6	146.4	-41.2
	473+43	124.0	170.3	46.3
69-in Longard tube	473+94	193.9	99.0	-94.9
	474+36	55.5	20.9	-34.5
	474+46	56.2	15.5	-40.7
	475+54	260.5	64.6	-195.9
	474+90	291.6	93.5	-198.1
Two 40-in Longard tubes	475+41	152.5	107.7	-44.9
	475+92	1,195.4	121.8	-1,073.5
	476+34	546.6	27.1	-519.6
	476+43	446.4	23.3	-423.1
	476+50	1,501.3	335.0	-1,166.3
	476+93	288.5	657.6	369.1
	477+93	816.0	265.6	-559.4
	479+93	1,669.3	247.4	-1,422.0
	482+93			
	Totals	12,653.4	8,277.9	-4,375.5

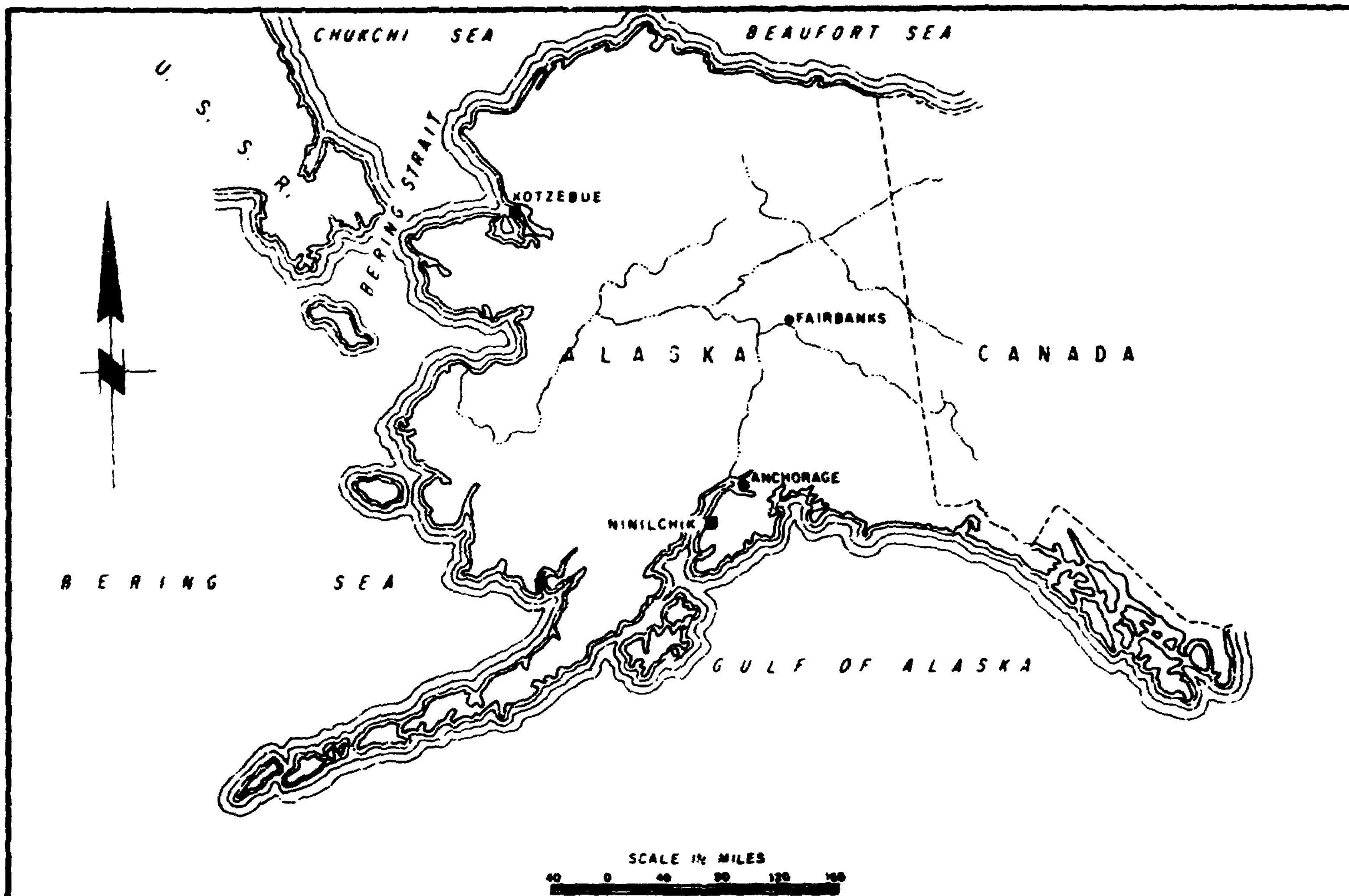


Figure 2-539. Location map of Alaska demonstration sites.

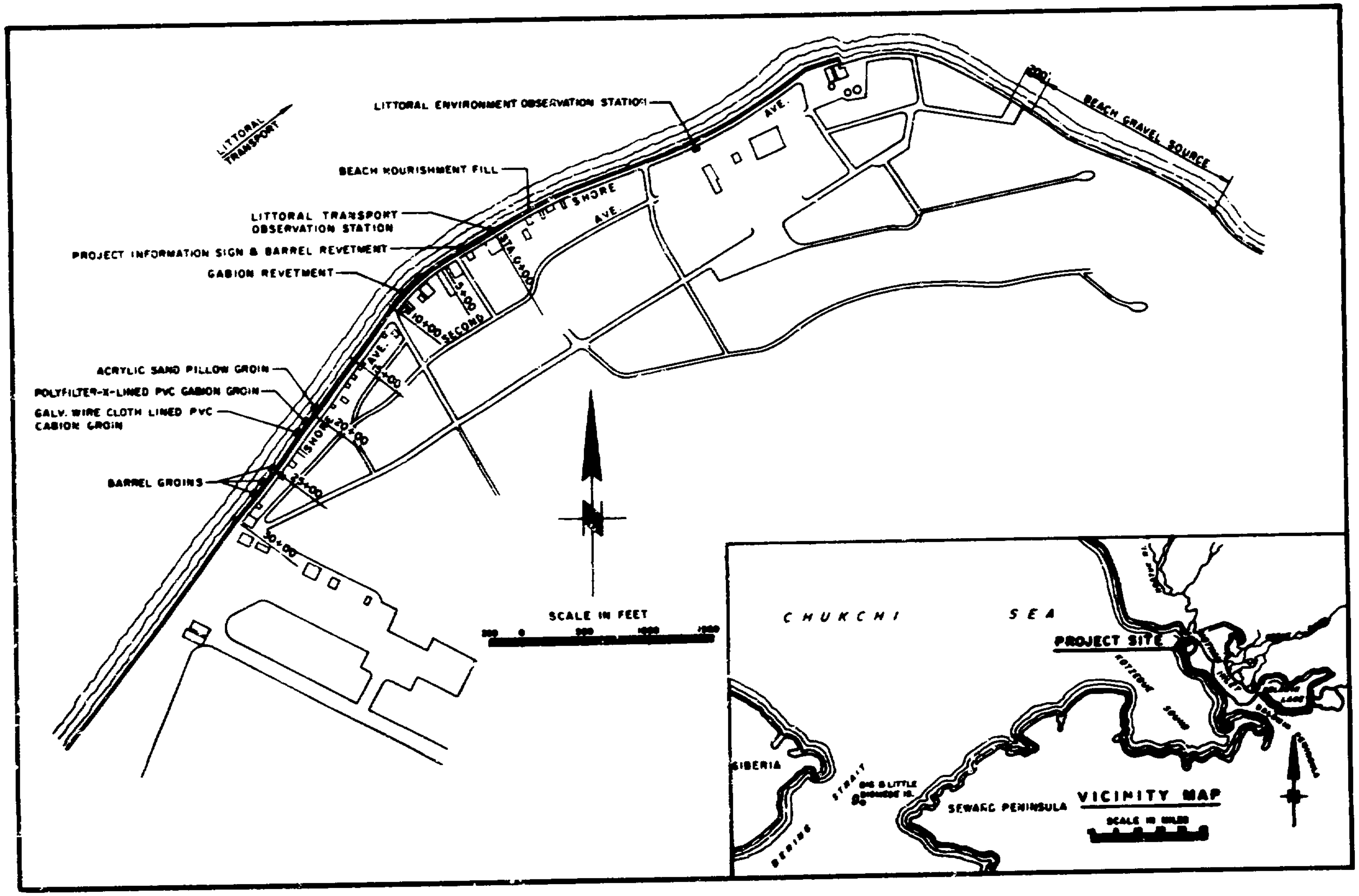


Figure 2-540. General plan of Kotzebue, Alaska, demonstration site.

(b) Climate. Although Kotzebue is located 26 miles inside the Arctic Circle, average winter temperatures are not as severe as might be expected, with normals of 0° to -12° Fahrenheit. During the summer, cloudy skies prevail, fog occurs, daily temperatures are relatively uniform, relative humidity is high, and westerly winds predominate. Normal summer temperatures range from 40° to 60° Fahrenheit. Annual precipitation is very light, about 8 inches during a normal year; snowfall is less than 50 inches per year. Because of the numerous storms and uneven heat radiation of the adjoining land and water areas, the weather is typically windy. Mean hourly windspeeds vary from about 10 miles per hour in May to 15 miles per hour in January.

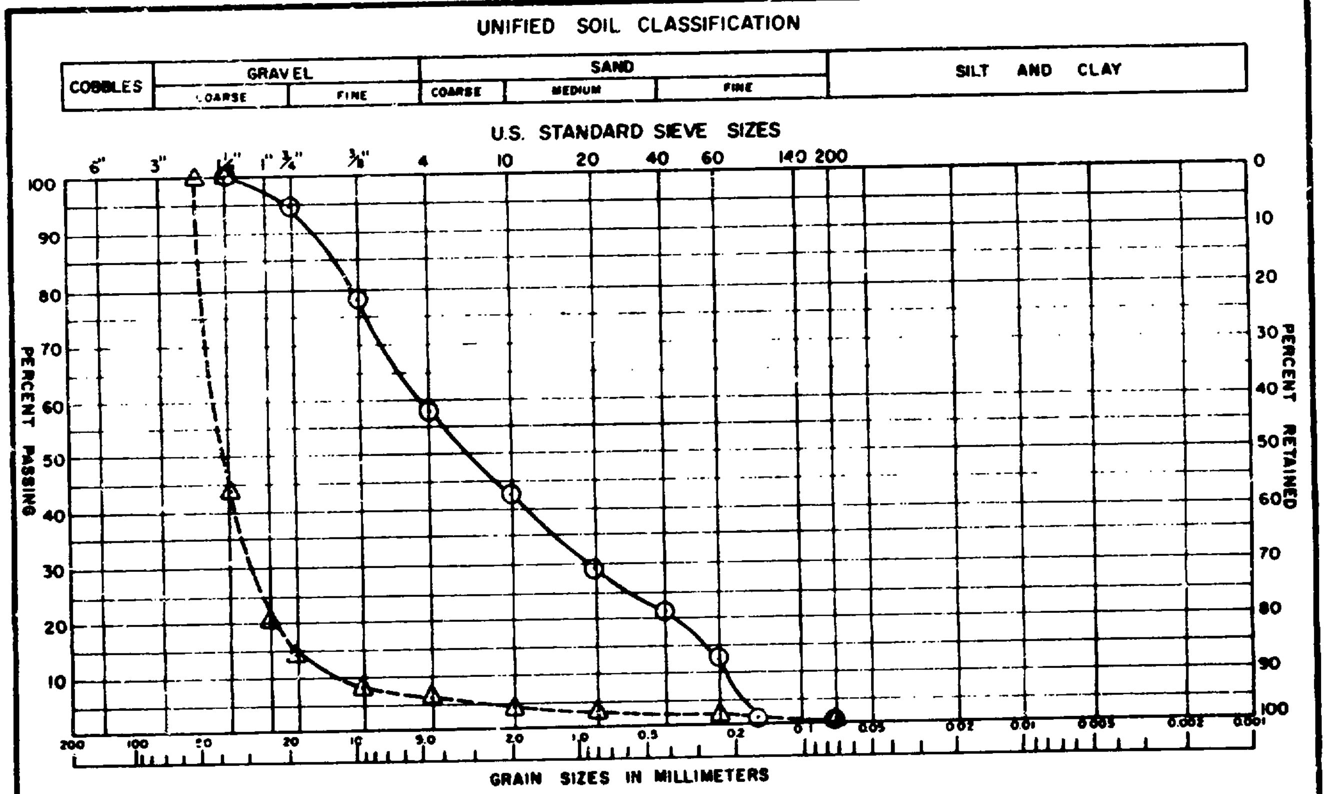
(c) Waves, Tides, and Longshore Transport. Tides are semidiurnal, the mean tide level is 0.55 foot above MLLW, and the spring and mean tidal ranges are 1.1 and 1.0 foot, respectively. The most severe wave action is a result of westerly storm waves generated in the Chukchi Sea between polar ice and shore. These waves break on the offshore gravel and sandbars, then re-form into smaller waves which reach the Kotzebue shoreline with a maximum calculated height of 3.8 feet. The maximum stillwater level (4.5 feet MLLW) was determined by adding the storm wave setup and storm surge to MHHW. Adding the storm wave runup of 2 feet to the maximum stillwater level resulted in a design runup elevation of 6.5 feet MLLW. The elevation of the road fronting Kotzebue is approximately 6.0 feet MLLW; thus, storm waves will overtop the road. The LEO data (Table 1-3) indicate that wave heights average 0 to 1 foot, with a maximum of 6.5 feet. The wave climate is classified as severe. Builtup deposits and eroded areas indicate that longshore transport is predominantly northward.

(d) Geomorphology, Soils and Vegetation. The demonstration site is located on a low-lying gravel spit characterized by a series of ancient beach ridges separated by low-lying muskeg areas. Offshore bars have been formed by the Kobuk River when its sediment load is deposited several miles offshore.

The beach is typically composed of fine to medium grain-size gravel (0.25 to 0.50 inch in diameter) consisting of quartz, limestone, dolomite, and chert schist materials. The offshore bars also consist of sand and gravel. Figure 2-541 shows the range of gradations from samples collected at various locations on the site. From the available data, no relationship between material gradation and location within the site was evident.

Shore Avenue follows the shoreline at approximately +6 ft above MLLW; inland elevations increase to approximately 12 feet above MLLW. A 1- to 2-foot berm exists shoreward of the avenue. The beach averages 30 to 50 feet in width and normal slope is 1 on 8. Narrower beaches with slopes as steep as 1 on 4 exist in the eroded areas of the shoreline. No ice-thrust berm is apparent.

(e) The Problem. The site at Kotzebue is a typical example of an erosional problem caused by wave action and ice floes. The waterfront at Kotzebue extends along approximately 1.5 miles of low-lying beach. During periods of heavy storms from the southwest, storm waves break and run up on the beach. Material which is disturbed by wave attack is primarily transported northward along the shore by littoral currents, as evidenced by



LOCATION	SYMBOL	LL	PI	CLASSIFICATION
El. 1.1 ft, 38.5 ft from base line	⊙———⊙			Gravelly SAND (SP)
El. −1.9 ft, 77 ft from base line	△- - - - -△			Sandy GRAVEL (GW)

Figure 2-541. Representative grain-size distribution curves, Kotzebue site.

shoreline erosion and deposition. It is further hypothesized that the eroded material is eventually carried offshore by flow from the Hotham Inlet and deposited on the offshore bars. About 50 feet offshore, currents from the Hotham Inlet flow southward along the coast at 2 to 3 feet per second. Thus, an active available mechanism for erosion and offshore deposition exists.

Due to the shallow, sloping, offshore topography, ice floes normally lock to the bottom and do not reach the beach. However, during open-water periods, ice floes from offshore have been driven onshore by wind and waves, aggravating erosion and damaging shoreline development. In addition, Shore Avenue is overtopped by storm waves due to its low elevation.

The shore protection devices chosen for this site were selected to demonstrate their effectiveness under conditions typical of those at Kotzebue.

(2) Demonstration Project. The test devices constructed at Kotzebue are two groin fields consisting of three groins each, 200 feet of gabion revetments, and 200 feet of steel-barrel revetment. The updrift groin field was constructed of barrels, and the downdrift field has two gabion groins on the updrift end and a Sand-Pillow groin on the downdrift end (Fig. 2-540).

Kotzebue has an abundant supply of empty 55-gallon steel fuel barrels, which have been used successfully in past shore protection construction. The barrel groin field was constructed by first placing the empty barrels in position, bolting them together, and then filling them with sand and gravel. To test the loss of materials from open barrels, about 70 percent were left uncapped. Bags filled with sand and gravel were used to top off the remaining barrels. The plan and profile of a typical barrel groin are shown in upper Fig. 2-542. The barrel revetment comprised two double rows of barrels, 10 feet apart and parallel to the beach front, connected at 50-foot intervals by a single row of barrels perpendicular to the beach. The barrels were bolted together at the sides and filled with sand-gravel. To prevent downdrift erosion, the 10-foot zone between the shore-parallel barrels was filled to form a perched beach (Fig. 2-543).

Gabion groins and revetments are particularly suited to the Kotzebue site because of the relatively low freight cost of the baskets, their flexibility in conforming to scour holes, and the multiple choices they offer as to types of material that may be used to fill the baskets. Some baskets were lined either with wire-mesh hardware cloth or with Polyfilter-X cloth, as the proper-sized rock for filling unlined baskets was not available. Gravel-filled burlap bag[illegible]ylic sandbags were placed in the unlined baskets. To test the durability of the gabions in the cold marine climate, galvanized and PVC-coated baskets were used (Figs. 2-544 and 2-545).

(3) Statistics and Cost. The cost of construction was $146,956; statistics are given in Table 2-102.

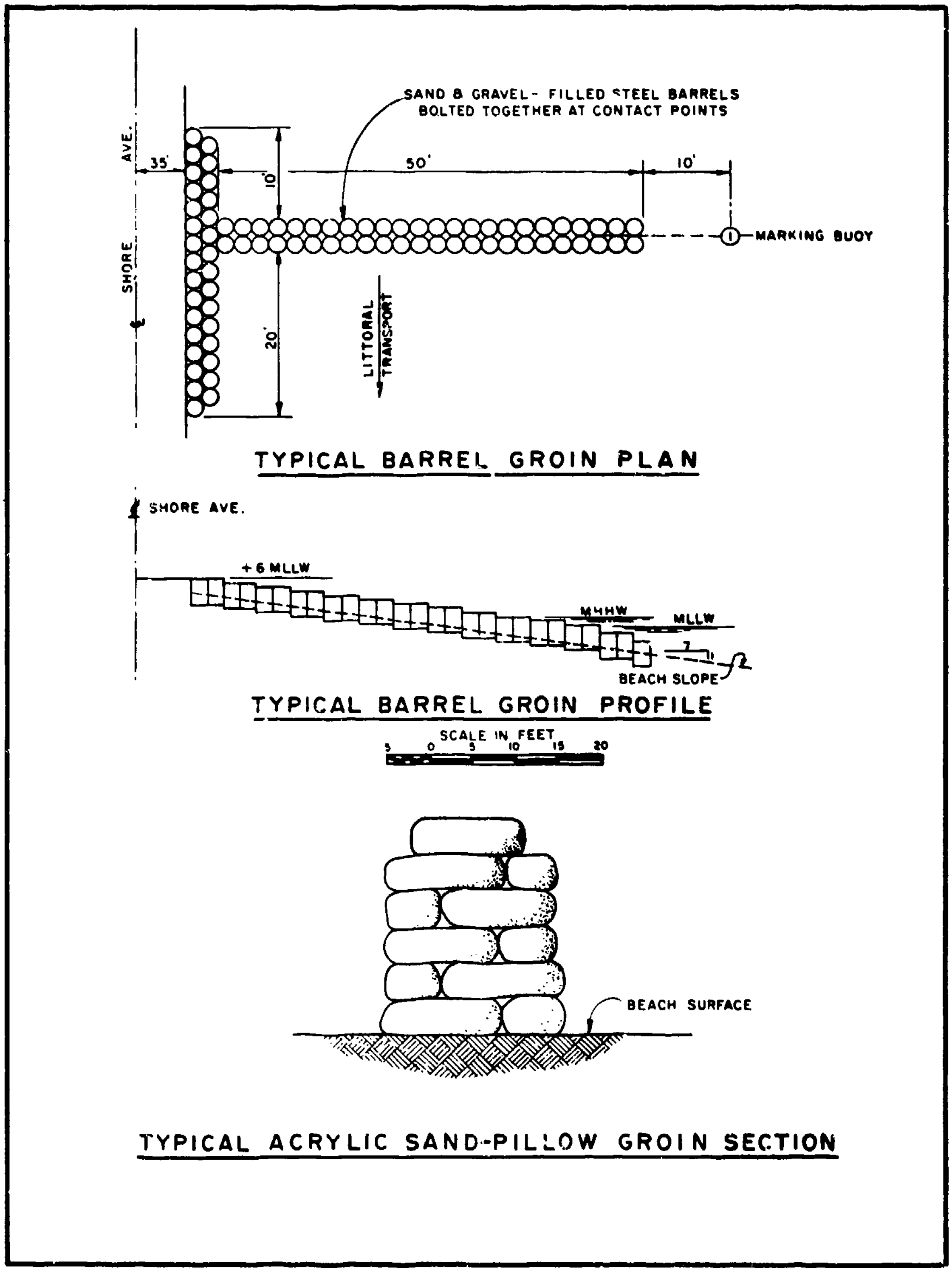


Figure 2-542. Barrel groin and acrylic Sand-Pillow groin at Kotzebue site.

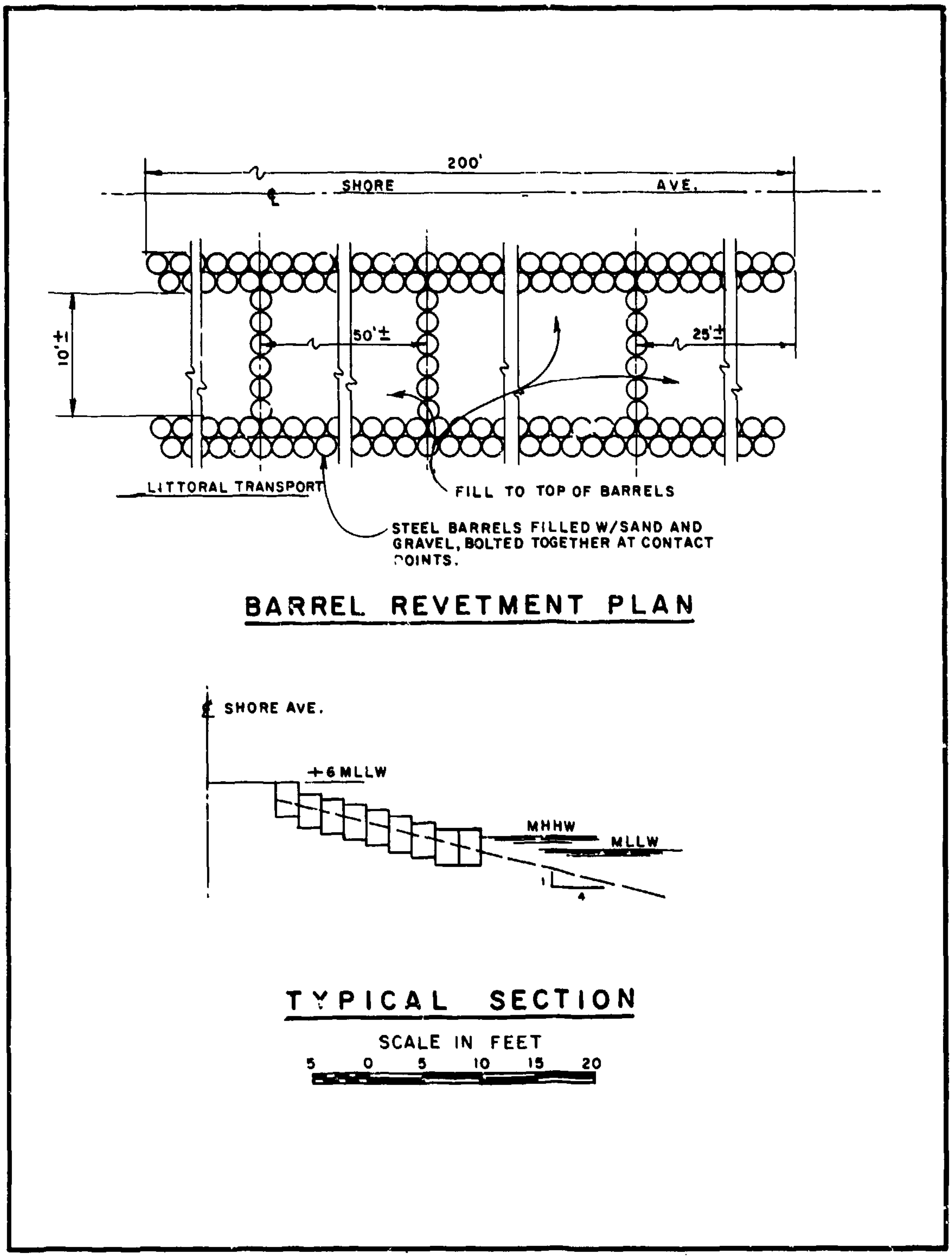


Figure 2-543. Plan and section of barrel revetment at Kotzebue site.

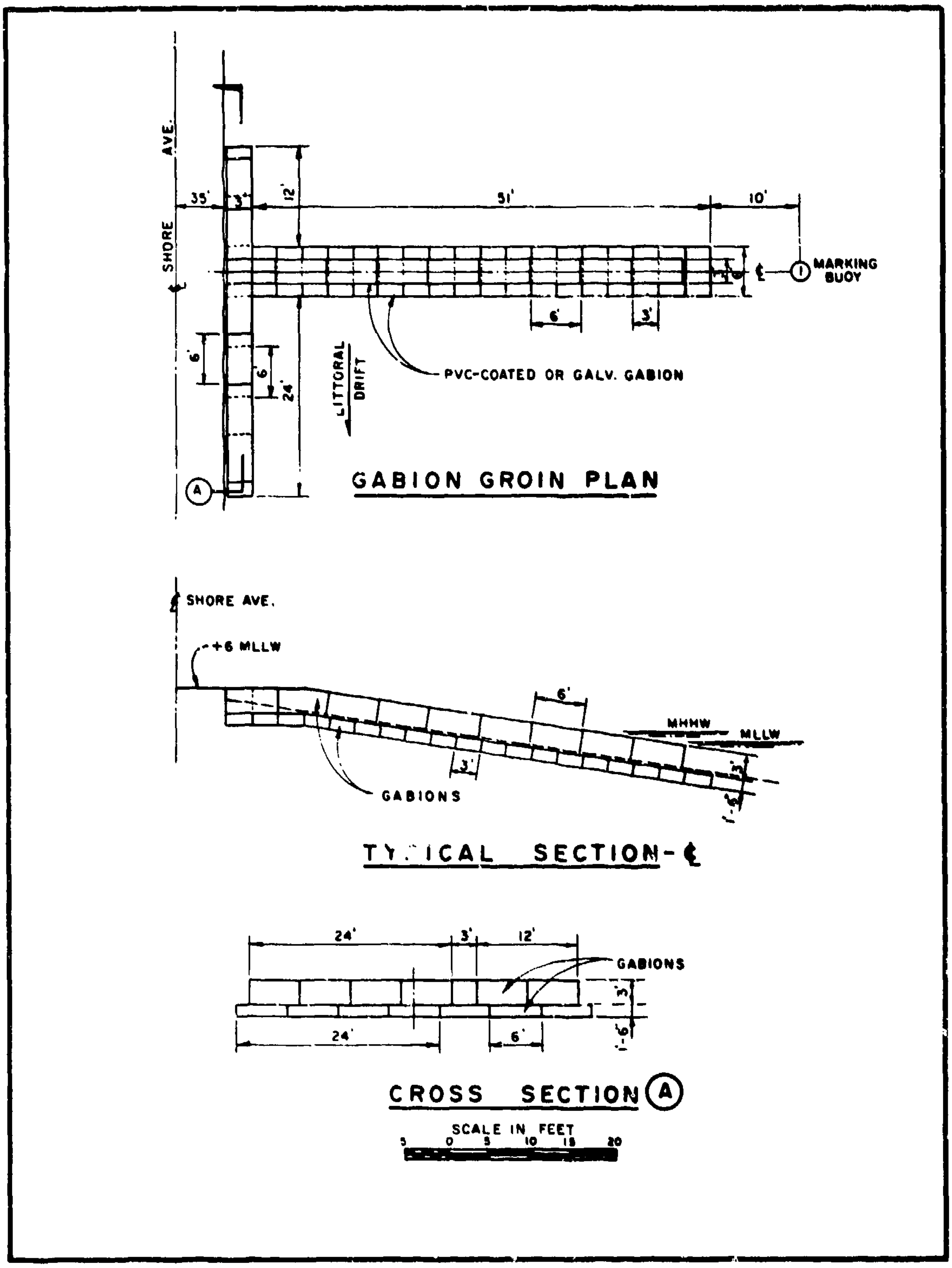


Figure 2-544. Gabion groin at Kotzebue site.

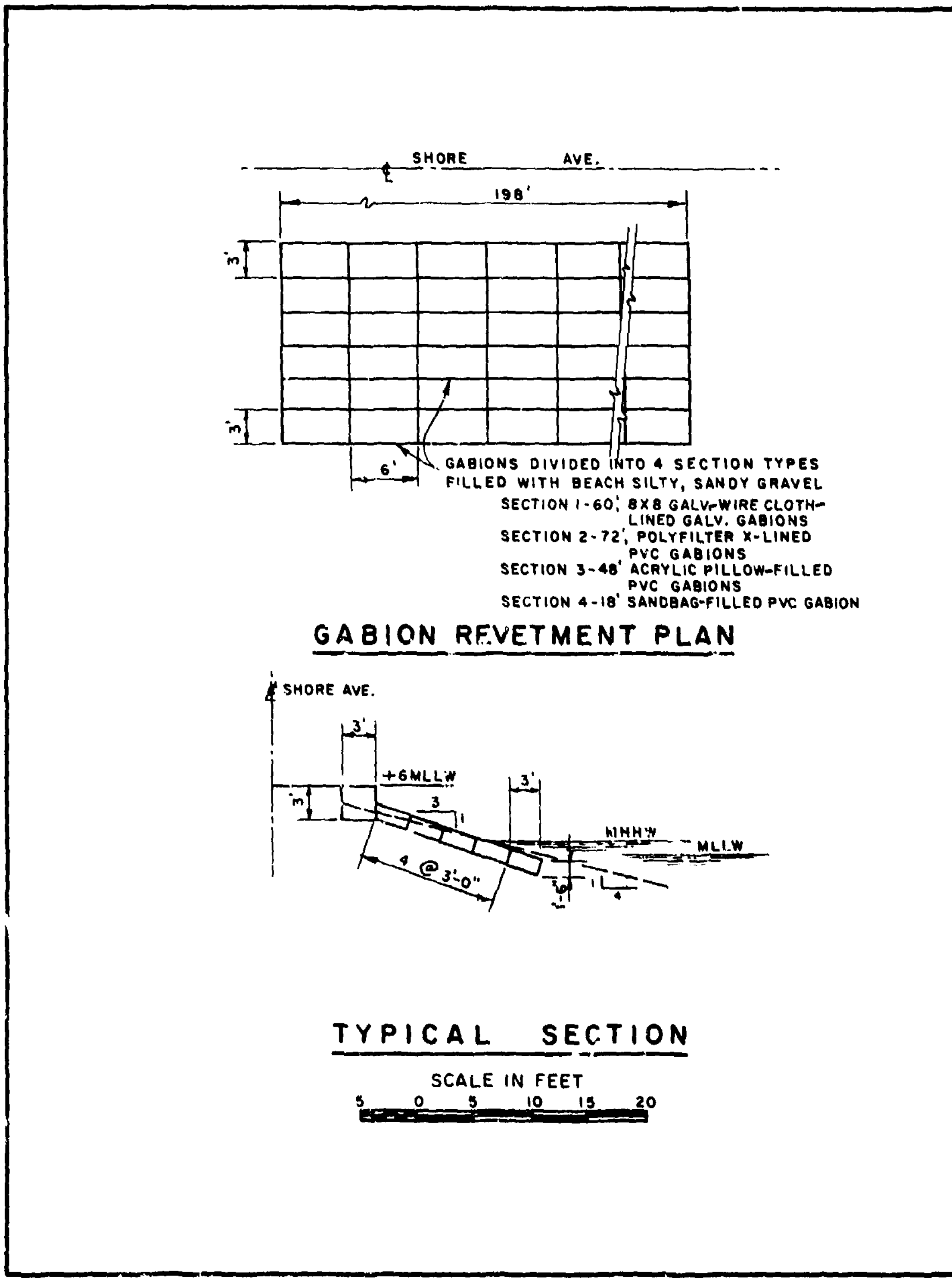


Figure 2-545. Gabion revetment at Kotzebue site.

Table 2-102. Statistics for systems at Kotzebue.

Item	Unit	Quantity
Materials		
Gabion baskets	ea.	110
Steel barrels	ea.	835
Burlap sandbags	ea.	900
Acrylic Sand Pillows	ea.	700
Galvanized hardware cloth	ft^2	13,200
Gravel	yd^3	1,000
Labor		
Skilled	hr	414
Semiskilled	hr	757
Unskilled	hr	1,650
Equipment		
End loader	hr	256
Flatbed truck	hr	85
Caterpillar tractor D6 with dozer blade	hr	115

(4) Construction and Performance.

(a) Barrel Revetment. Construction of the 200-foot barrel revetment was nearly completed in September 1978 when inclement weather forced cessation of work for the winter. Many of the barrels needed to be filled to the top with sand and gravel, or capped on the upper tier (Figs. 2-546 and 2-547). The tiers were connected with four perpendicular diaphragms to create three closed cells with two open ends (upper Figure 2-543). By June 1979, considerable littoral drift was accumulating on the south side of the southern diaphram. Some gravel collected on the south sides of the remaining diaphragms, and minor erosion occurred on the downdrift sides of all diaphragms. The two northernmost cells showed evidence of beach sand and gravel filling in behind the seaward shore-parallel double row of barrels (Fig. 2-548). The tops of three of the uncapped barrels were deformed by ice pressure or debris impact. The barrels were located on the seaward row, two from the north end (Fig. 2-549), and one from the center north diaphragm (Fig. 2-550). The proposed sandbag caps were deleted from the construction schedule, and beach gravel alongside the revetments was used to top off the barrels. The construction was completed in August 1979. No other changes in structural alinement, material, or cross section were observed. The accretion and depletion patterns also remained unchanged through October 1979.

(b) Gabion Revetment. The 198-foot gabion revetment was also incomplete in October 1978 (Fig. 2-551). A storm in late September had destroyed the second row by collapsing the gabion cages in place before they could be filled. After the storm a severe freeze trapped the partially covered gabions creating an expensive removal problem. The 15-foot-wide structure was completed in August 1979 and includes the following gabion types:

Figure 2-546. View of barrel revetment, Kotzebue, Alaska, 12 October 1978.

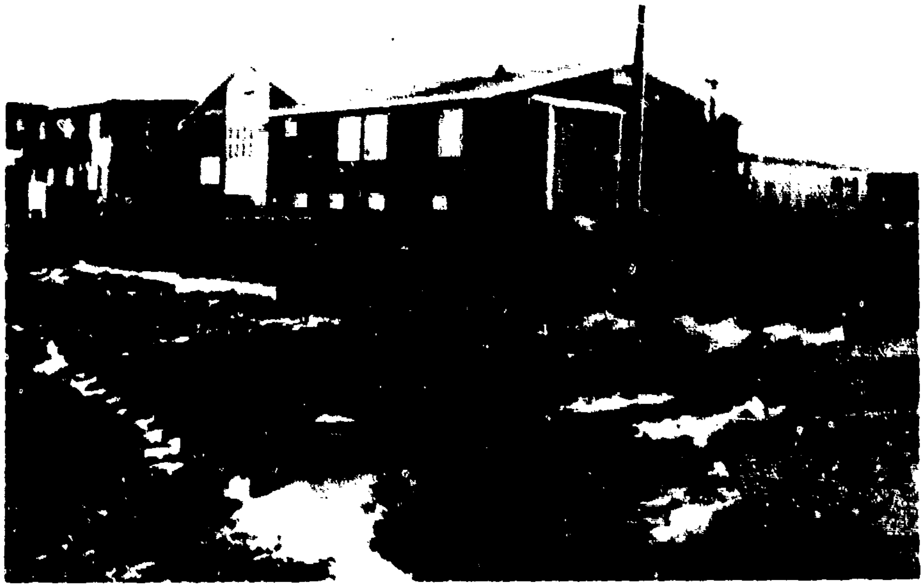


Figure 2-547. Barrel revetment shortly after initial construction, Kotzebue, Alaska, 12 October 1978.

Figure 2-548. Barrel revetment with perpendicular diaphragms, Kotzebue, Alaska, 13 June 1979.

Figure 2-549. Deformed barrels at north end of revetment, Kotzebue, Alaska, 13 June 1979.

Figure 2-550. Deformed barrels in center diaphragm, Kotzebue, Alaska, 13 June 1979.

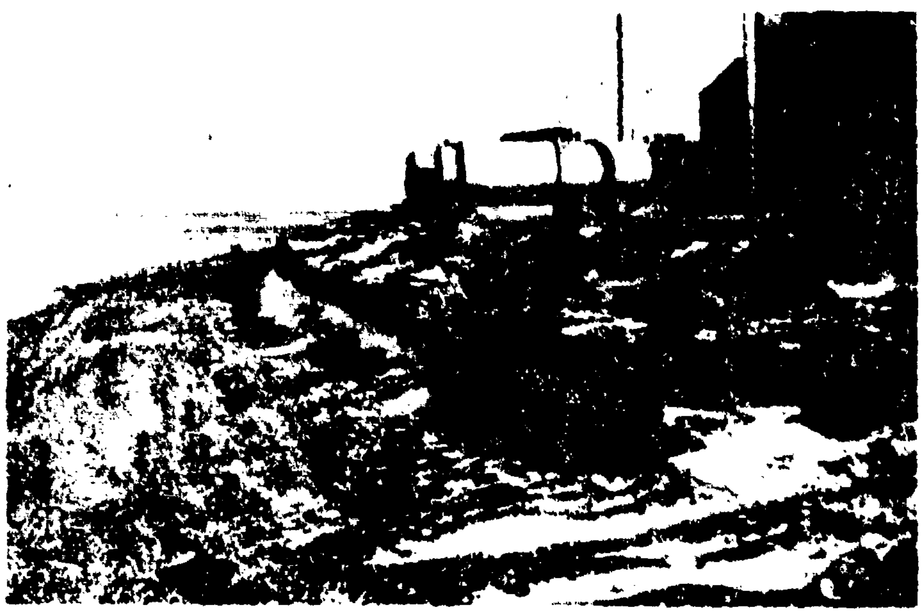


Figure 2-551. Incomplete gabion revetment, Kotzebue, Alaska, 11 October 1978.

(1) 18 feet of PVC-coated gabion baskets of gravel-filled burlap sandbags.

(2) 48 feet of PVC-coated gabion baskets of gravel-filled acrylic bags.

(3) 60 feet of galvanized gabion baskets lined with galvanized hardware cloth and filled with sandy gravel.

(4) 72 feet of PVC and galvanized gabion baskets lined with Polyfilter-X cloth and filled with sandy gravel (Fig. 2-552).

One of the southern gabion baskets was ruptured in July 1979 as a result of residential boaters throwing their anchors in the revetment. A 1-foot-diameter hole developed on the top of the cage but no fill material was lost (Fig. 2-553). In October 1979, there were still no visible changes in structural alinement, cross section, or materials (Fig. 2-554). Comparison of the August and October 1979 photos reveals that the gabion system was protecting the berm and holding the beach well. Figure 3-46 which was put in Section III for comparison of similar devices shows a 1979 profile through the gabion revetment.

(c) Sand-Pillow Groin (Groin A). Construction of the Sand-Pillow groin (groin A), was halted upon the completion of the seaward 30 feet. The acrylic bag material was easily damaged by wave-borne debris, and it failed during construction (Fig. 2-555). The bag-on-bag friction factor was also insufficient to prevent the filled bags from sliding over each other when 3-foot waves attacked the groin. Figures 2-556 and 2-557 show the effect of the September 1978 storm on the seaward Sand Pillows. Wave action dispersed the bags in the lower 8 feet of groin, and although the structural integrity of the landward section was not altered, many bags were torn. By June 1979, more bags were displaced or cut open (Fig. 2-558). The groin was not repaired and continued to deteriorate at this level through October 1979. A nominal amount of drift accumulated on the south side of the structure; however, the incomplete Sand-Pillow groin was largely ineffective.

(d) Polyfilter-X-Lined PVC-Coated Gabion Groin (Groin B). The PVC-coated gabion groin, completed in September 1978, was lined with Polyfilter-X filter cloth and extended 50 feet seaward perpendicular to the beach (Fig. 2-559). October ice, piling up against the south side as a result of tidal movement, shifted the seaward end of the groin northward (Fig. 2-560). The groin was not damaged and this shifting was the only structural or material alteration recorded. The device functions well, trapping the northbound drift on its south side and causing some scouring along the downdrift beach. By June 1979 the sand and gravel accumulation on the south side was 12 to 18 inches deep. Comparison of the June and August photos (Figs. 2-561 and 2-562) illustrates the additional buildup to the top of the groin and the extension of the beach on its south side. Figure 2-563 shows the early northward shifting of the end of the structure as of October 1979.

(e) Wire-Mesh-Lined, Galvanized Gabion Groin (Groin C). This galvanized gabion groin was also completed in September 1978 and was lined with a galvanized hardware cloth (Fig. 2-564). Like gabion groin B, groin C is also about 50 feet long, and ice pressure also forced the seaward

Figure 2-552. Completed gabion revetment, Kotzebue, Alaska, 29 August 1979.

Figure 2-553. Damage to south gabion basket, Kotzebue, Alaska, 27 July 1979.

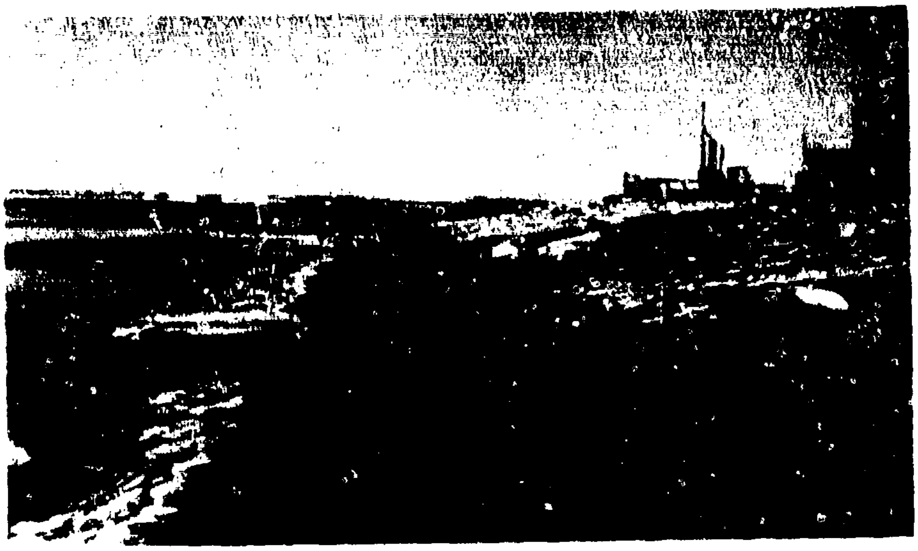


Figure 2-554. Beach and gabion revetment, Kotzebue, Alaska, 24 October 1979.

Figure 2-555. Incomplete Sand-Pillow groin (groin A), Kotzebue, Alaska, 27 July 1979.

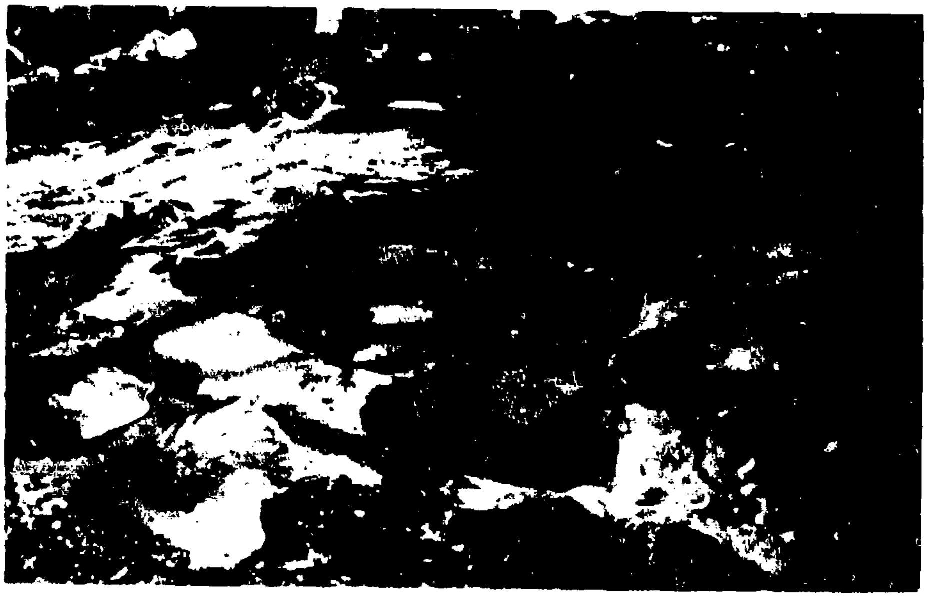


Figure 2-556. Sand Pillows scattered around Kotzebue, Alaska, 11 October 1978.

Figure 2-557. Scattered and torn bags after September storm, Kotzebue, Alaska, 11 October 1978.

Figure 2-558. Sand-Pillow groin, Kotzebue, Alaska, 13 June 1979.

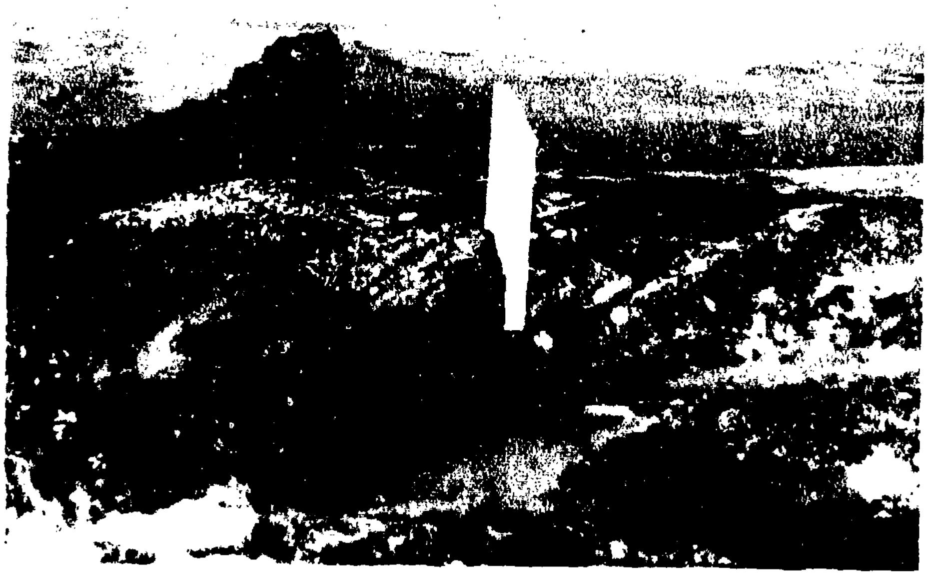


Figure 2-559. Polyfilter-X-lined gabion groin (groin B), Kotzebue, Alaska, 11 October 1978.

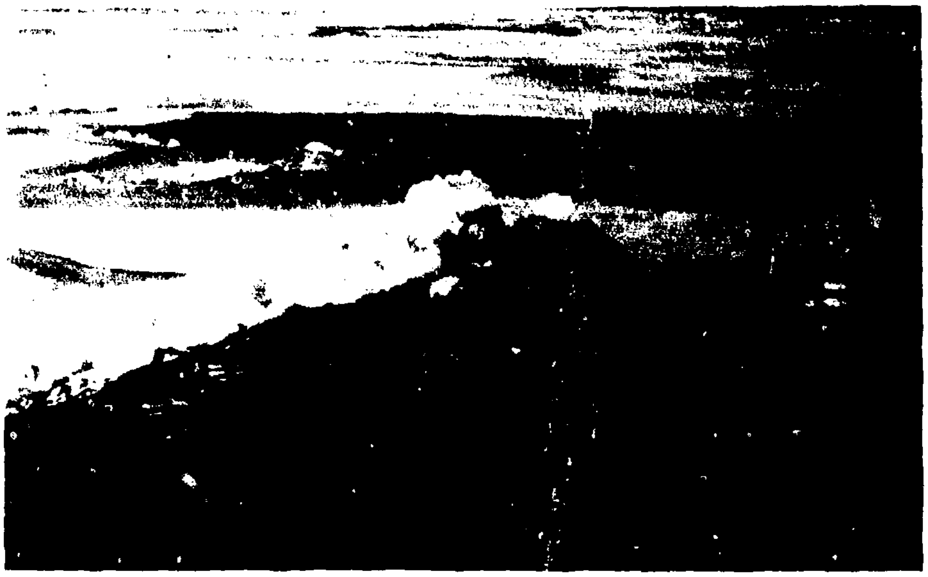


Figure 2-560. Ice piling up at seaward end of groin B,
Kotzebue, Alaska, 11 October 1978.

Figure 2-561. Drift accumulation on south side of groin B,
Kotzebue, Alaska, 13 June 1979.

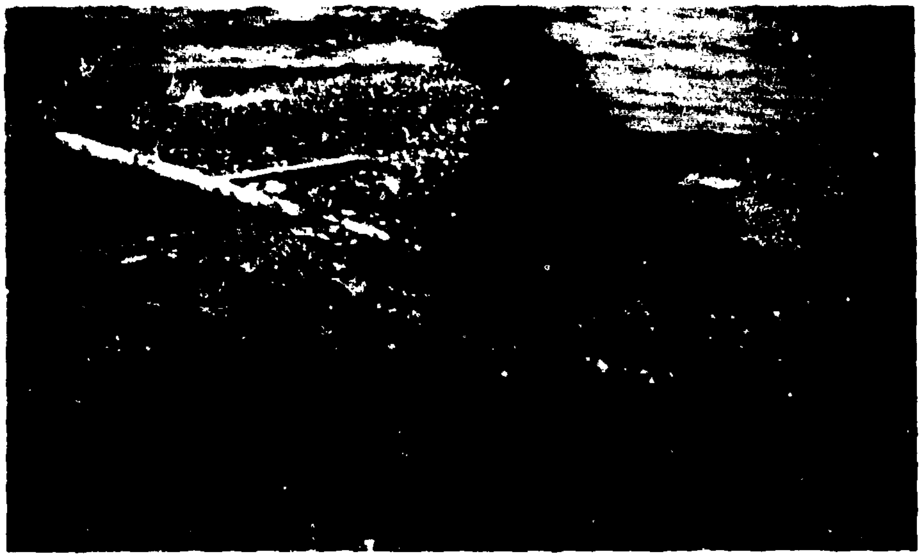


Figure 2-562. Drift accumulation on south side of groin B, Kotzebue, Alaska, 30 August 1979.

Figure 2-563. Shifted PVC gabion groin B, Kotzebue, Alaska, 24 October 1979.

Figure 2-564. Wire-mesh lined, galvanized, gabion groin C,
Kotzebue, Alaska, 11 October 1978.

end of this groin to the north. As a result of this displacement, the wire-mesh lining in groin C separated at its seams, allowing the sand and gravel to escape. Further damage to the seaward gabion was observed during the June 1979 site visit. In July, an abandoned boat found tied near the structure had repeatedly swayed into the side of the groin with the wave motion. The boat caved in the south side, tearing the galvanized cloth and nearly emptying the contents of the baskets (Figs. 2-565 and 2-566). By this time the midsection of the groin had accumulated almost 18 inches of drift on its south side. Accretion continued into August, while a deficiency of beach nourishment was observed near the landward gabions on the north side of the groin (Fig. 2-567). Snow coverage in October precluded a detailed investigation, but no changes were apparent (Fig. 2-568).

(f) Fuel Barrel Groin Field. A field of three groins (groins D, E, and F) was installed in the southern half of the site. Each 50-foot groin is a double row of 55-gallon open-ended steel fuel barrels bolted together and filled with gravel (Fig. 2-542). The construction was completed in September 1978 except for some filling and capping. The September storm deposited substantial littoral drift on the south sides of the groins and eroded the beach on the north sides (Figs. 2-569 and 2-570). By June 1979 the gravel level had climbed to one-half the barrel height along the midsection of the south sides of groins D and E, and some of the sandy gravel had been scoured from underneath a few barrels on the north sides. Groin F was intercepting more drift than the others. The south side had climbed to 80 percent of the barrel height, and on the north side it was even with the bottom of the barrels (Figs. 2-571 and 2-572).

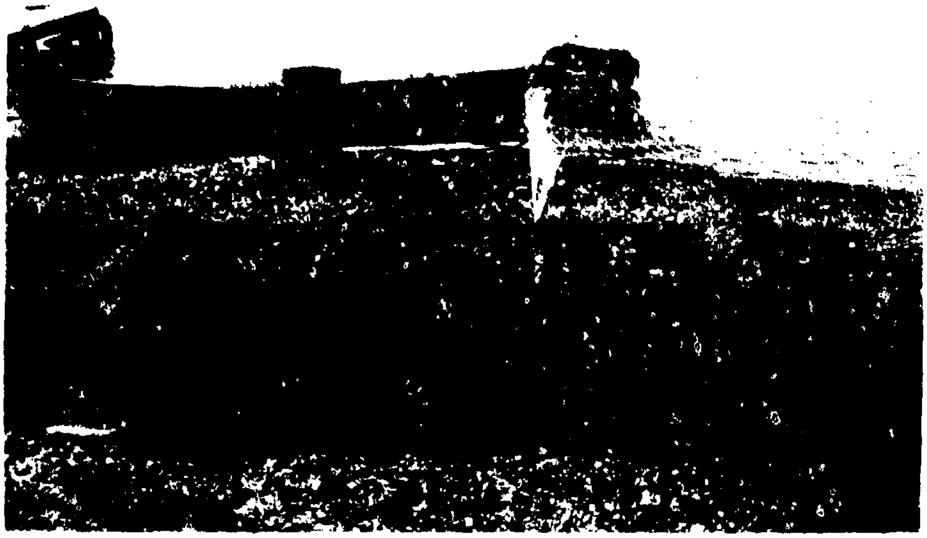


Figure 2-565. Gabion groin C and abandoned boat, Kotzebue, Alaska, 27 July 1979.

Figure 2-566. Damage to seaward end of groin C by north shifting and abandoned boat, Kotzebue, Alaska, 27 July 1979.

Figure 2-567. Effect of groin C on adjacent beaches, Kotzebue, Alaska, 30 August 1979.

Figure 2-568. View of the early shifting of groin C, Kotzebue, Alaska, 24 October 1979.

Figure 2-569. Groin D, Kotzebue, Alaska, 11 October 1978.

Figure 2-570. Groin E, Kotzebue, Alaska, 11 October 1978.

Figure 2-571. Gravel level, north side of groin F. Kotzebue, Alaska, 13 June 1979.

Figure 2-572. Gravel accumulation, south side of groin F, Kotzebue, Alaska, 13 June 1979.

Littoral processes were scalloping the downdrift beach and forming cusped 50-foot gravel beaches between the structures. Groin damage was limited to the seaward barrels. Postconstruction ice floes in September 1978 were responsible for crushing the upper halves of the southern seaward barrels (Figs. 2-573 and 2-574). In summer 1979, gravel from the south sides of the groins was used to top off the barrels and to replace material scoured from the north flanks. This surplus material was also used to complete the filling of baskets in the gabion revetment. In critical areas, barrels were capped with sand-filled bags. Barrel filling and capping were completed in August 1979. Figures 2-575 and 2-576 show the typical condition of the groins and adjacent beaches during the August visit. Before winter ice locked in the groins, some of the seaward barrels were partly crushed but still operational, and some of the northside gravel nourishment had been lost.

(5) Analysis. With the exception of the Sand-Pillow groin the devices at the Kotzebue site functioned well. During the monitoring period, the barrel revetment and groins were particularily effective despite the ice damage each sustained. The northward displacement of the seaward barrels did not hinder groin performance; therefore, this nominal uniform shifting is allowable, considering the small tidal range in this region. However, severe storms could result in larger displacement of groin ends and possibly structural damage. If applicable, a hole pattern could be provided in the seaward barrels to allow wave transmission and reduce the shifting. Filter cloth or larger fill would be necessary to ensure structural integrity. Damage to the upper sections of critical barrels in the devices could have been avoided if the barrels had been initially filled to the

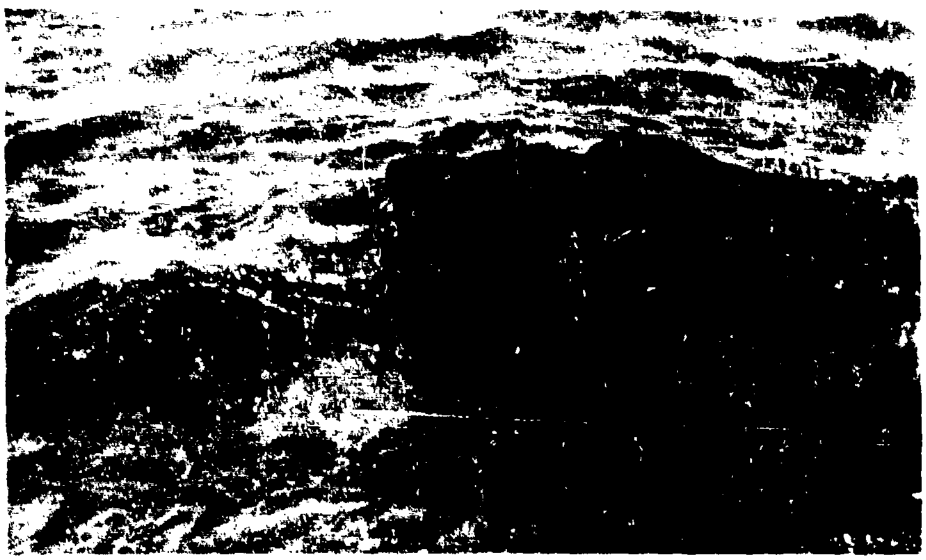


Figure 2-573. Crushed barrels at groin D, Kotzebue, Alaska, 13 June 1979.

Figure 2-574. Damage to seaward barrels at groin E, Kotzebue, Alaska, 13 June 1979.

Figure 2-575. Filled barrels at groin D, Kotzebue, Alaska, 29 August 1979.

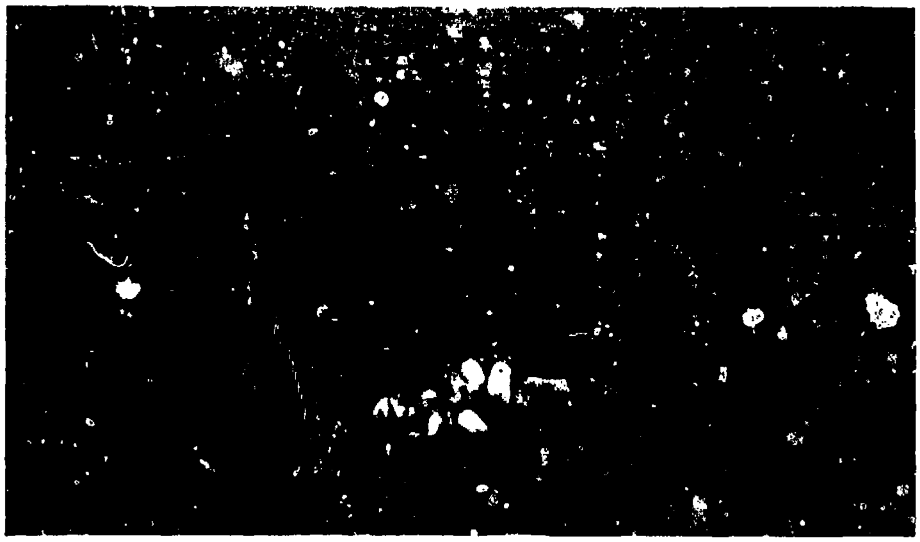


Figure 2-576. Typical beach nourishment on north sides of groins D, E, and F, Kotzebue, Alaska, 30 August 1979.

top. The fill resists the collapse of a barrel as it is being compressed by ice formation and wave action. Concrete caps would provide additional protection.

As expected, groin F at the south (updrift) end of the series of groins trapped the most drift. More monitoring time is needed to determine whether the littoral drift will bypass the groin ends as each groin is filled or whether shorter groins would function better in this area, considering the coarseness of the material.

The Sand-Pillow groin did not perform well because of deterioration of the acrylic bag material. Abrasion of the fabric by littoral drift, ice, debris, and handling appeared to initiate early rupture of the bags. Because the fabric has a low friction factor, many bags were displaced by small waves. The weight of the filled bags is not sufficient to hold them in place. Large bags might have remained in section, but the short-term deterioration of the fabric indicated that textiles are no match for the severe environment of this region.

Further monitoring is required to determine the effectiveness of the gabion revetment. No significant changes in beach or structure cross sections were observed. Items to be examined include durability of PVC coating, corrosion of galvanized baskets and hardware cloth, and performance of inner fabrics by comparison of filler-bag and filter-cloth durability. The Polyfilter-X cloth appears to be more durable than the galvanized-mesh hardware cloth as evidenced by their short-term performance in the gabion groins. The hardware cloth pulled apart at its seams indicating a need for more overlap or some effective method of joining the sheets. More monitoring is needed to determine the survival periods of these devices.

c. Ninilchik, Alaska.

(1) Site Description.

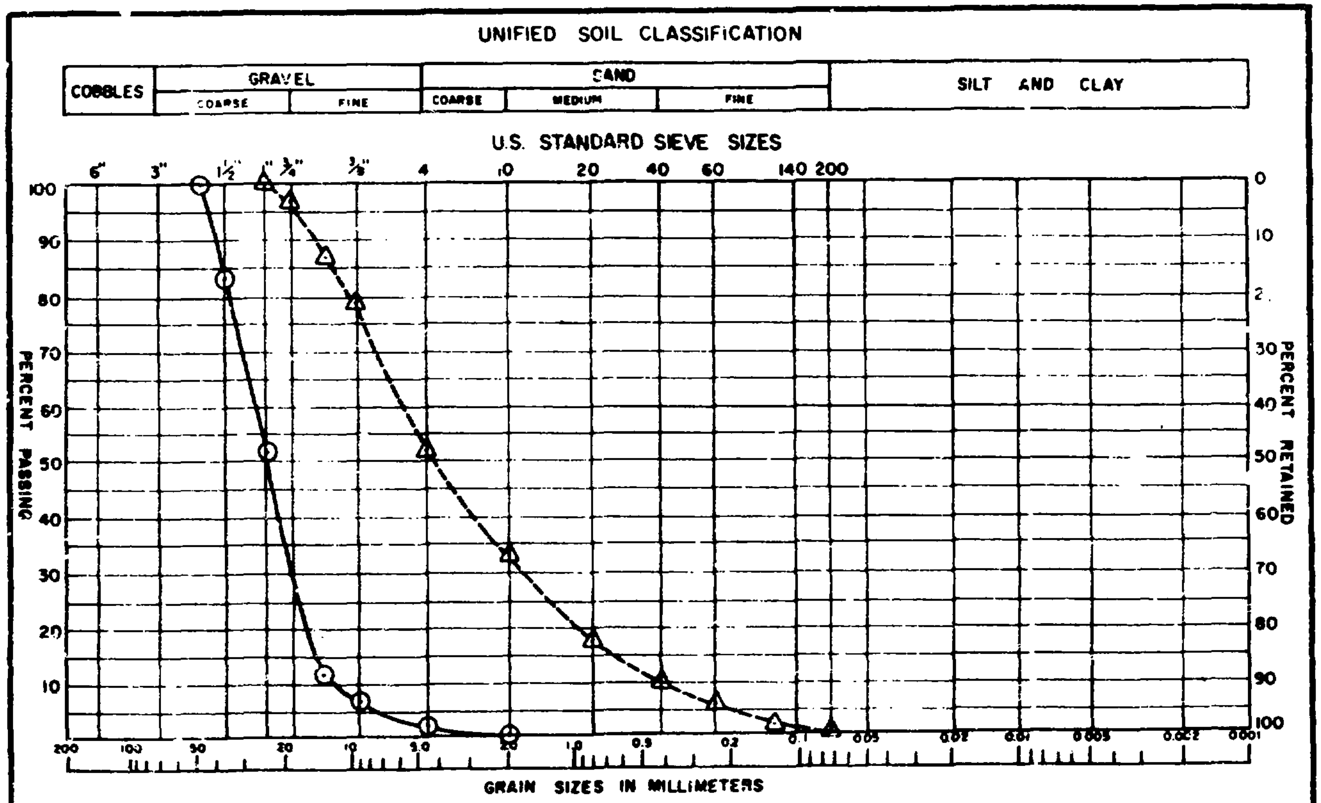
(a) Geographical Setting. The village of Ninilchik is located on the eastern shore of Cook Inlet in south-central Alaska, about 100 air miles southwest of Anchorage and about 600 air miles southeast of Kotzebue. It is readily accessible by paved highway year round, with a driving distance from Anchorage of about 200 miles. The shoreline at the site is oriented in a northeast-southwest direction.

(b) Climate. The Ninilchik area is in a transitional climatic zone, between the harsh climate of the interior Alaska and the moderate maritime climate of the coast. The summers are cool, generally sunny, and fairly dry; however, in late summer and fall, cloudy, rainy weather is dominant. The winters are long and moderately cold.

(c) Geomorphology, Soils, and Vegetation. Ninilchik lies in a region of soft sedimentary rocks overlain by a blanket of unconsolidated glacial deposits. The glacial material consists of till and outwash deposits ranging to thicknesses of a few dozen feet. This material is underlain by the Kenai Formation, a succession of soft, partly consolidated sandstone, siltstone and shale beds, interspersed with thin seams of low-grade coal. The Kenai Formation is extensively exposed on the bluffs along the Cook Inlet shore in the region of the village. Gradation curves are given in Figure 2-577.

The vegetation of the area consists of intermixed forest, muskeg, and grassland. White and Sitka spruce, birch, black cottonwood, aspen, and balsam poplar are the principal tree species in the area. Muskeg vegetation, including mosses, grasses, sedges, heath shrubs, and scattered black spruce inhabit broad, shallow, poorly drained basins. Grasslands, in which bluejoint and other grasses, wild celery, and fireweed predominate, occupy the slopes and broad upland summits. Intermixed with the forest and grassland types are a variety of shrubs, including highbrush cranberry, wild rose, devil's club, and larger red elderberry and alder.

(d) Waves, Tides, and Longshore Transport. The mean tide level at Ninilchik is 10.1 feet above MLLW and the diurnal and mean tidal ranges are 19.1 and 16.5 feet, respectively. Wave action during the spring and summer months is predominantly from the southwest, producing a strong longshore transport northeastward and resulting in aggradation along the spit. Late fall and early winter storms are from the northwest, generating waves that often reverse the flow of littoral material. If these storms occur before formation of protective shore ice, the materials accummulated during the summer are rapidly removed. The LEO data (Table 1-3) indicate that wave heights average 0 to 1 foot with a maximum of 6.5 feet. The wave climate is classified as severe, and the net longshore transport potential northeastward was 71,800 cubic yards for the 8 months analyzed.



SAMPLE NO.	DEPTH	SYMBOL	LL	PI	CLASSIFICATION
LS-1301					GRAVEL (GW)
LS-1314					GRAVELLY SAND (SP)

Figure 2-577. Representative grain-size distribution curves, Ninilchik site.

(e) The Problem. The Ninilchik site is a typical example of beach and bluff erosion due primarily to storm waves. The village is fronted by a barrier sand and gravel spit about 2 miles in length paralleling the mainland. Emptying into Cook Inlet, the Ninilchik River flows through a small boat basin between the mainland and the northern half of the spit. The last 1,200 feet of spit has been particularly susceptible to damage from late fall to early winter storm waves, which have occasionally overtopped the spit and eroded the boat basin slopes. The demonstration project was located along this 1,200-foot reach.

Since 1967 the Alaska District has attempted to prevent waves from overtopping and breaching the spit. Initially, several hundred feet of gravel-filled, steel-barrel revetment was installed to stabilize the beach, but this device proved to be short-lived due to saltwater corrosion and failure of the underlying filter system. The barrel revetment was then replaced by a native spruce log revetment, which has performed quite well, but it also is sensitive to any defects in the underlying filter system. The filter system, consisting of Polyfilter-X cloth, has consistently failed because of inadequate joint overlapping and fastenings. An attempt to stabilize the beach along the toe of the log revetment with a series of low, gravel-filled timber-crib groins was made in 1974. The groin field readily retained littoral materials during the summer months with material accummulating to the tops of the groins on the updrift side, about 4 feet above natural ground. However, the bottom planks of the crib groins extended only about 2 feet below natural ground, and subsequent northwest fall storms quickly eroded the beach below the bottom planks, causing loss of the crib fill as well as the updrift accummulation from under the crib groins.

(2) Demonstration Project. The plan initially proposed called for a construction of five timber groins and five gabion groins. In addition, two gabion-groin ties parallel to the beach were to connect the outer ends and midpoints of three successive gabion groins in an effort to prevent wave downrush from eroding aggraded materials between these groins. Finally, a gabion mattress would be placed along the last 450 feet of the northern end of the log revetment to serve as toe protection. The timber groins were to be constructed of materials salvaged from the existing timber-crib groins. The original plan was modified at the September 1978 SEAP meeting to place more emphasis on the gabion beach revetment and less on the gabion groins. The modified plan is described below.

The installation as constructed is shown in Figure 2-578. Two of the existing timber groins were retained (groins 3 and 4) and gabion toe protection was added on both sides of groin 4 to determine its effectiveness in preventing wave undercutting (Fig. 2-579). Three new groins (groins 2, 5, and 6) constructed of piling salvaged from the other crib groins, and with all new planking, were added (Fig. 2-580). A new gabion groin (groin 1) was also constructed (Fig. 2-581).

The baskets in groin 1 were filled with 4-to 8-inch cobbles available in a limited deposit at the mouth of the Ninilchik River. Along the northern 595 feet of log revetment, a gabion mattress was used for toe protection (Fig. 2-578). Various units of the mattress blanket were lined with either

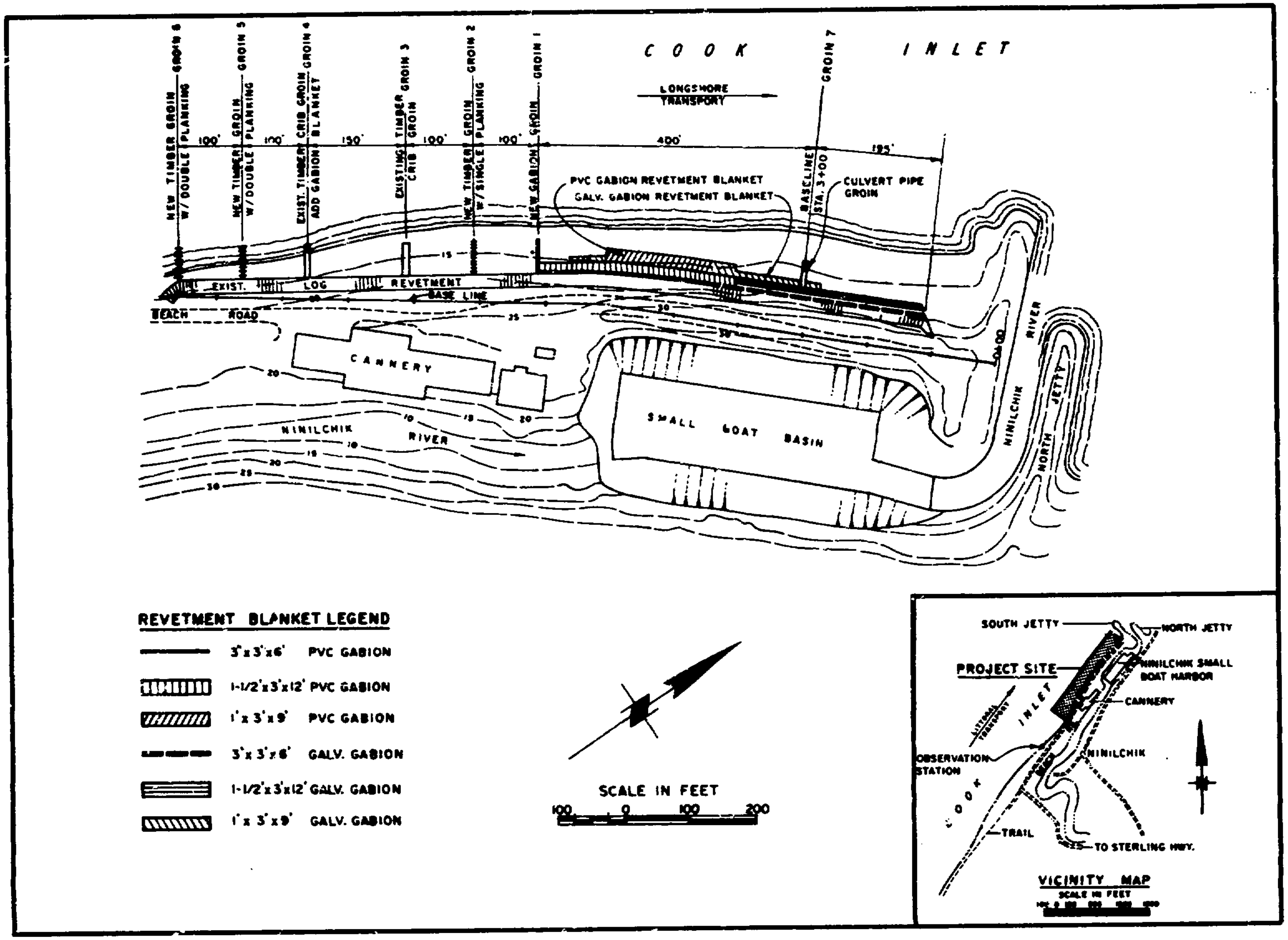


Figure 2-578. General plan of Ninilchik, Alaska, demonstration site.

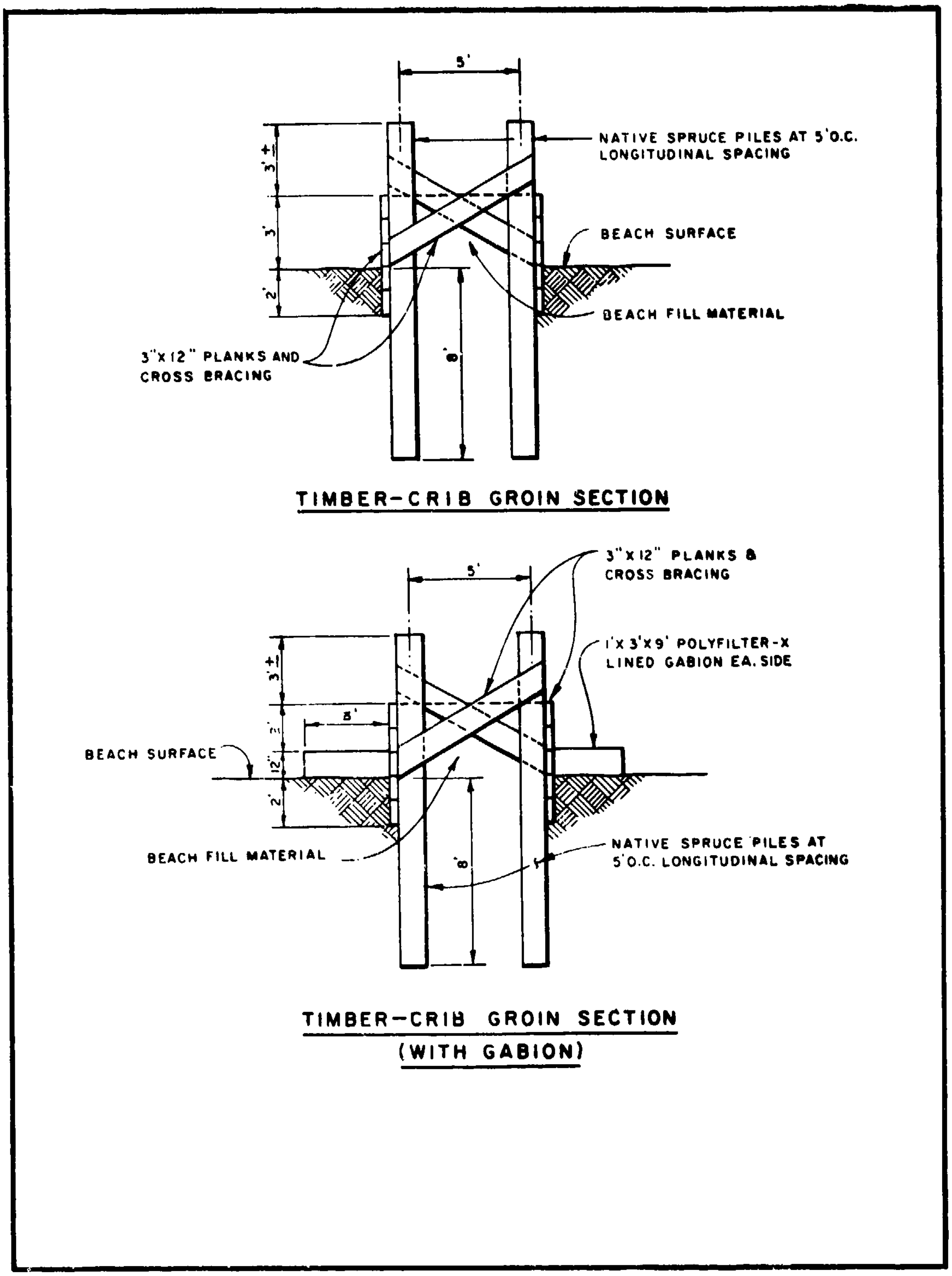


Figure 2-579. Timber-crib groin at Ninilchik site.

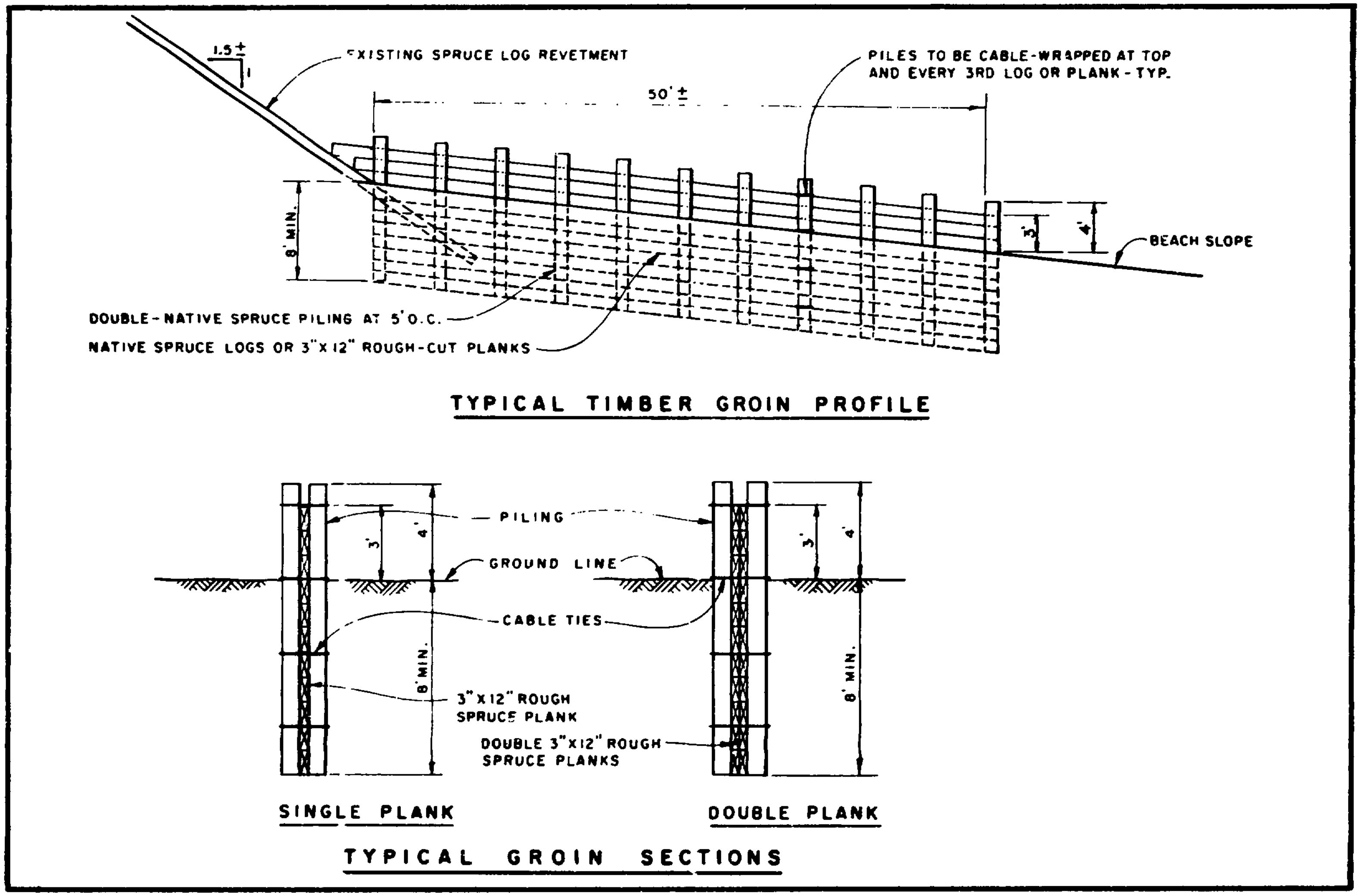


Figure 2-580. Timber groin at Ninilchik site.

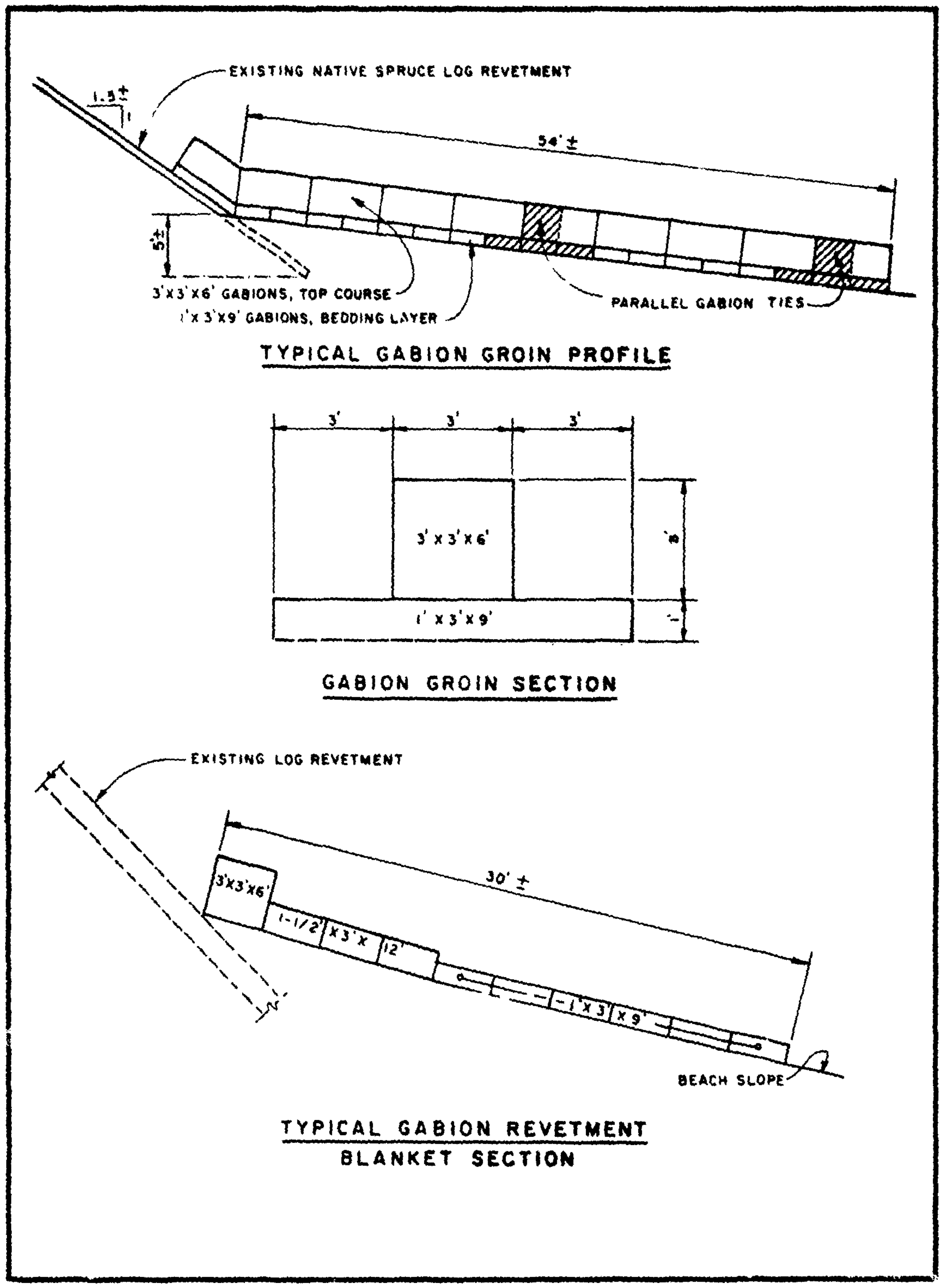


Figure 2-581. Gabion groin and revetment at Ninilchik site.

Polyfilter-X filter cloth or galvanized hardware cloth. All units were then filled with beach gravel in polypropylene bags which were partially filled with concrete. In November 1979 a corrugated metal pipe groin (groin 7) with a 6-inch-thick precast-concrete cap was added (Fig. 2-582).

(3) Statistics, Costs, and Construction. The statistics and estimated costs for the project at Ninilchik are given in Tables 2-103 and 2-104; actual costs are not available.

(a) Gabion Revetment. Construction was started in fall 1978. Gabion baskets were laid out, lined with Polyfilter-X lining, and filled with 4- to 8-inch rubble. Construction was halted in November 1978 because of a subzero cold wave. The Polyfilter-X lining is difficult to work with in freezing weather conditions. The remaining gabion baskets were completed in spring 1979.

(b) Gabion Groin. Construction of this structure was completed in November 1978. Gabion baskets, 1 by 3 by 9 feet on the bottom-bedding layer, were first laid out and filled with 4- to 8-inch rubble. Next, the top layer of 3- by 3- by 6-foot gabion baskets were placed and filled with 4- to 8-inch stone.

(c) Timber-Plank Groins. These groins (2, 5, and 6) were completed in July 1978. During low tide, holes were excavated, the pile-plank structures were put in place, and the holes were backfilled. This type of construction can only be done in regions of high tidal range because the excavated holes would otherwise fill with water and cause difficulties in structure placement.

(d) Timber-Crib Groins. These structures had been previously built, but a gabion toe blanket was added on both sides of groin 4.

(e) Corrugated Metal Pipe Groin. This structure was completed in November 1979. The beach was excavated, the pipes were filled with sand-rubble fill, and backfill was placed around the edges of all but the most landward pipe section that abutted the revetment. A precast-concrete cap was then placed on the top of each of the pipes.

(4) Performance.

(a) Gabion Revetment. The revetment served as toe protection for the existing log revetment behind it. By May 1979, material had deposited on top of the revetment which was in sound condition (Fig. 2-583). In November 1979, the beach had lowered and small rocks of 0.75- to 2-inch diameter had been forced through the Polyfilter-X cloth over about one-third of the structure. At this time the structure had been subjected to [illegible] to 6.5-foot waves; by March 1980 the gabion fabric had deteriorated to the point where almost every gabion basket had holes, and fill material was washing out (Fig. 2-584). As of April 1980, despite damage to filter cloth and baskets, the structure continued to provide protection for the log revetment. Figure 3-46, which was put in Section III for comparison of similar devices, shows a series of profiles through the gabion revetment. The

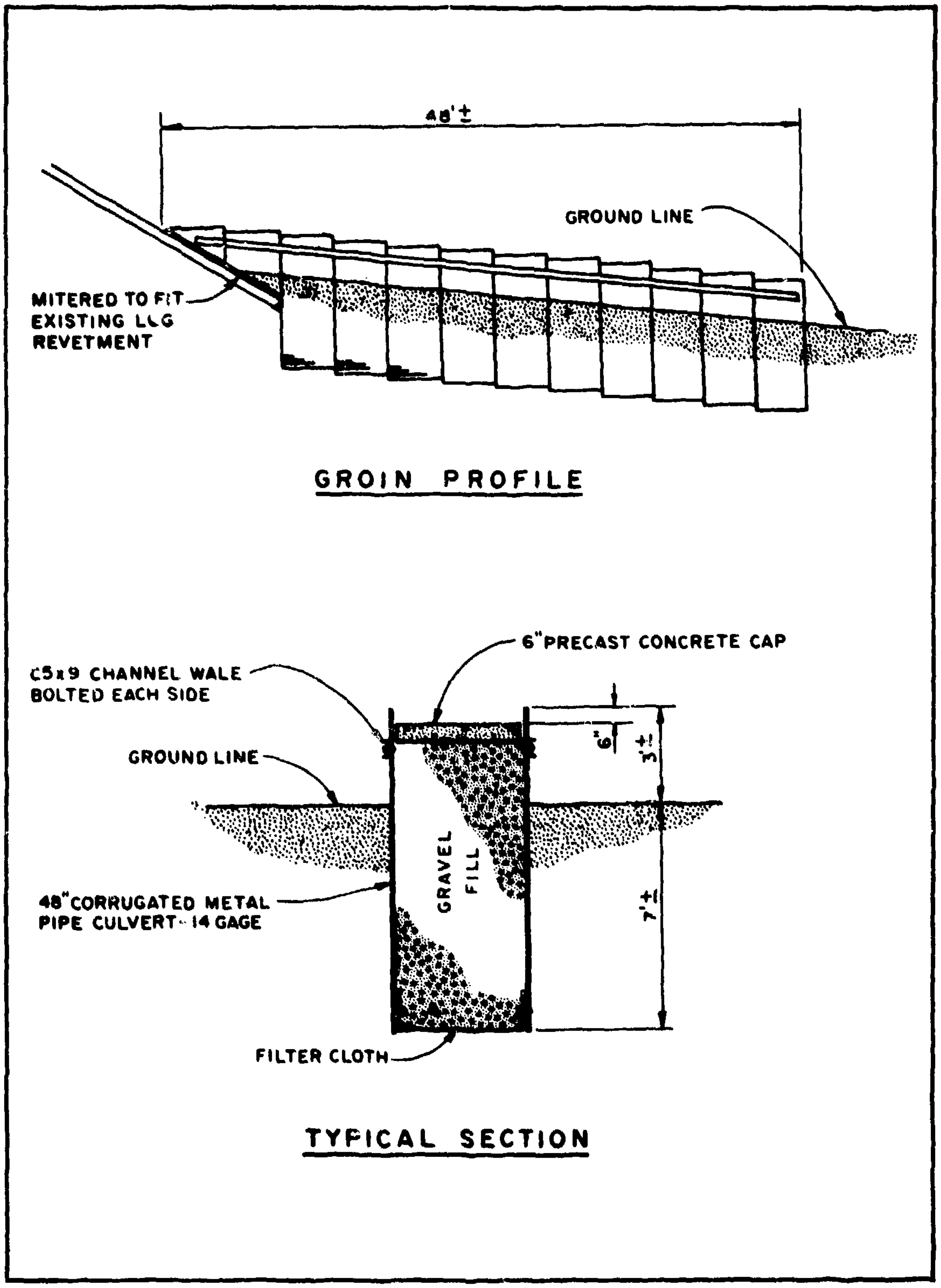


Figure 2-582. Culvert pipe groin at Ninilchik site.

Table 2-103. Statistics for Ninilchik.

Item	Unit	Quantity
Materials		
Gabions	ea.	357
Polyfilter-X cloth	ft^2	64,300
Galvanized hardware cloth	ft^2	2,900
Portland cement	sack	101
Polypropylene sandbags	ea.	2,000
Burlap sandbags	ea.	2,500
Planking (3- by 12-in)	ea.	3,000
Spruce piling (10- by 12-in)	ea.	1,080
Corrugated metal pipes (10-ft by 48-in diam)	ea.	12
5- by 9-in steel channel	ea.	2
Precast-concrete caps (6- by 48-in diam)	ea.	12
Labor		
Skilled	hr	685
Semiskilled	hr	1,394
Unskilled	hr	1,781
Equipment		
End loader	hr	600
J.D. 750 tractor w/dozer blade	hr	160
J.D. 350 tractor w/dozer blade	hr	480
Backhoe	hr	144
Dump truck	day	30
Flatbed truck	day	40

Table 2-104. Estimated construction cost (September 1978).

Structure	Materials	Labor	Equipment	Amount
Existing timber-crib groins (3 and 4)				
Toe protection (1 groin)	$ 600	$ 1,600	$ 400	$ 2,600
Repairs on 2 groins	100	1,600	100	1,800
New timber groins (2, 5, and 6)	3,000	8,000	2,000	13,000
Gabion groin (1)	1,200	2,000	1,000	4,200
Gabion toe protection (630 ft)	46,100	66,800	8,500	121,400
Corrugated metal pipe groin (7)	6,000	7,200	3,400	16,600

Figure 2-583. Littoral material accumulation on top of gabion revetment, Ninilchik, Alaska, 30 May 1979.

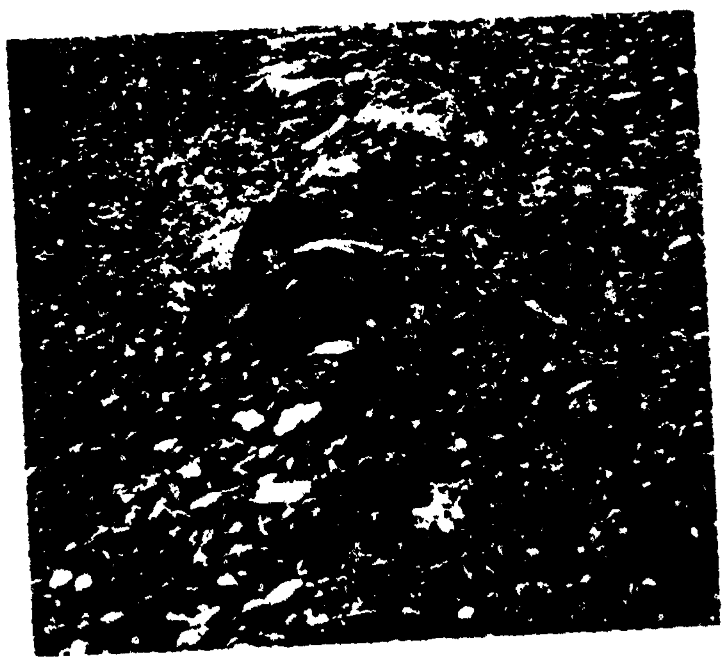


Figure 2-584. Deterioration of filter cloth and gabion baskets, Ninilchik, Alaska, 30 April 1980.

April 1980 profile shows about 2 feet of accretion on the beach below the revetment, extending upslope and covering the toe gabions.

(b) Groin 1. This groin consisted of PVC-coated baskets filled with 4- to 8-inch cobbles. The structure was completed in November 1978; by March 1979 the winter wave activity had moved the top part out of alinement (Fig. 2-585), and the seaward end 3- by 3-foot gabion was destroyed (Fig. 2-586). Apparently, storm waves had lifted the loose cobbles and battered them about inside the baskets, causing damage to the gabion wires. By June 1979, littoral drift had accumulated on the south side of the groin; however, the littoral material periodically washed away during the subsequent summer and fall months. In November 1979 the second 3- by 3-foot gabion basket was destroyed (Fig. 2-587). As of April 1980, the gabion baskets continued to deteriorate, allowing the cobbles to escape. The structure, however, remained intact and still trapped littoral drift.

(c) Groin 2. This structure consisted of a single-plank, timber-pile groin, which performed very well, trapping littoral material according to seasonal trends without any structural degradation (Fig. 2-588). The seasonal trends include accretion on the south side of the groins during the spring and summer months, and the washing away of most of the drift material during the winter and fall months. As of April 1980 this structure was still intact.

(d) Groin 3. This structure was the first of two existing timber-crib groins. However, it did not have toe protection along its sides. Before the beginning of the monitoring period the end pilings had been sheared off; no information is available on how the failure occurred (Fig. 2-589). This groin, like groins 1 and 2, trapped littoral material according to seasonal trends (Fig. 2-590). During November 1979 the cross brace next to the seaward end was shattered where it connected to the north post. By March 1980 some abrasion was apparent on the structure and this was attributed to ice chunks striking the structure (Fig. 2-591). As of April 1980, the structure was intact and functioning.

(e) Groin 4. This structure was built exactly like groin 3, except that a gabion toe blanket was provided along the sides of the structure. The structure also performed well, trapping littoral material in the summer months (Fig. 2-592). The gabion baskets were covered by littoral material almost entirely throughout the monitoring period. Thus, the condition of this part of the structure cannot be determined. As of April 1980, some weathering had occurred to the exposed timber, but no repairs were necessary and the structure was still trapping littoral material.

(f) Groin 5. This structure consisted of a single-plank and double-plank, timber-pile groin. The double planking is on the seaward 35 feet and the single planking is on the shoreward 15 feet. This structure also successfully trapped littoral material in the summer months, but by June 1979 the seaside end of the top plank on the north side had a 2-foot split (Fig. 2-593). This split required no repairs. As of April 1980, the structure was still intact and functioning well.

Figure 2-585. Shifting of top part of groin 1, Ninilchik, Alaska, 22 March 1979.

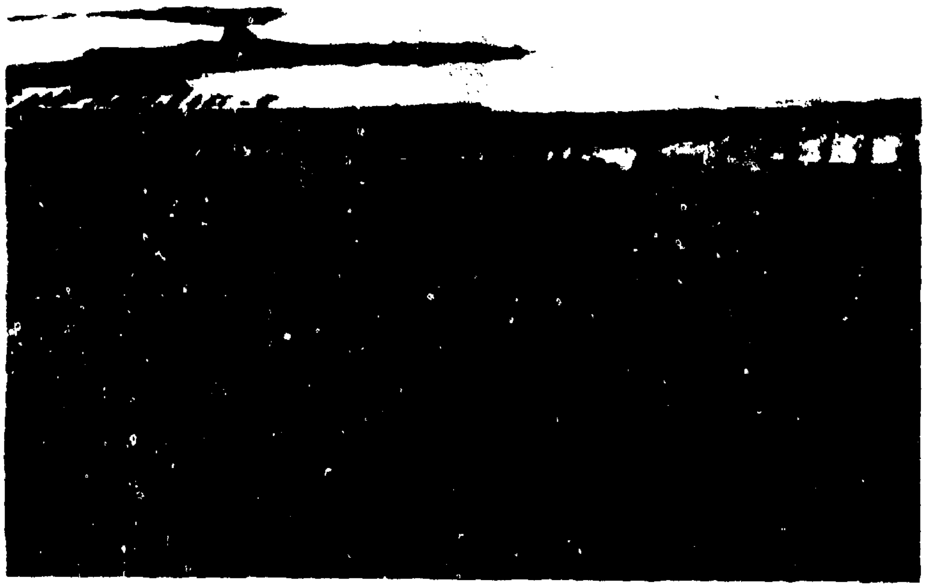


Figure 2-586. Destruction of the seaward 3-by 3-foot gabion of groin 1, Ninilchik, Alaska, 22 March 1979.

Figure 2-587. Destruction of the second 3-by 3-foot gabion of groin 1, Ninilchik, Alaska, 28 November 1979.

Figure 2-588. Groin 2 trapping littoral drift, Ninilchik, Alaska, 30 June 1979.

Figure 2-589. Seaward posts sheared off groin 3, Ninilchik, Alaska, 22 March 1979.

Figure 2-590. Accumulation of littoral material on south side of groin 3, Ninilchik, Alaska, 7 August 1979.

Figure 2-591. Sheared-off cross brace on groin 3, Ninilchik, Alaska, 30 April 1980.

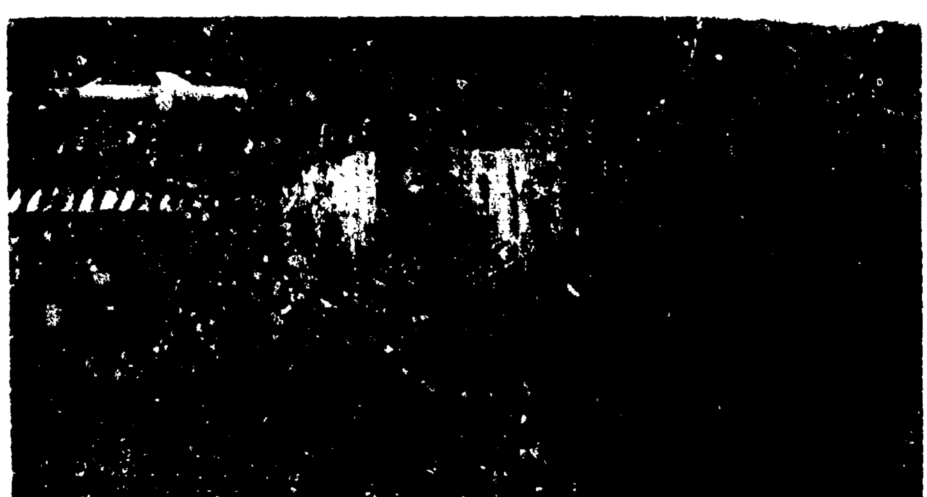


Figure 2-592. Trapping of littoral material by groin 4, Ninilchik, Alaska, 30 June 1979.

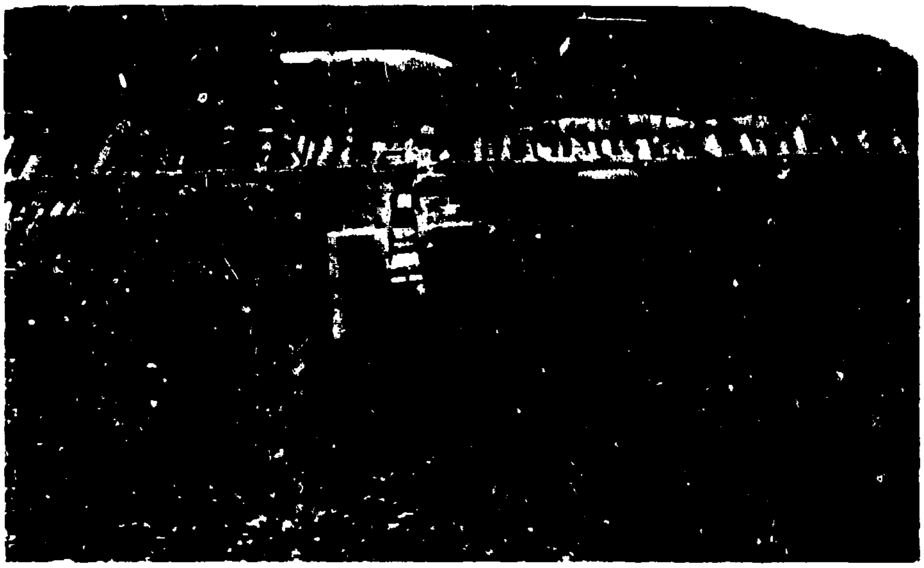


Figure 2-593. Groin 5 trapping littoral drift, and split in outermost plank, Ninilchik, Alaska, 30 June 1979.

(g) Groin 6. Groin 6 was constructed in the same manner as that for groin 5, except that the double planking extended throughout the structure. This structure performed very well, trapping littoral material with no structural damage and only slight weathering of materials (Fig. 2-594).

(h) Groin 7. This structure was completed in November 1979 and consists of a series of corrugated metal pipes filled with fill material and topped with a 6-inch-thick precast-concrete cap. By March 1980, the concrete caps had started to tip due to settlement or loss of fill material, and one pipe section was missing. The missing pipe, which had been cut to fit at the landward end of the groin, abutting the spruce log revetment (Fig. 2-595), had not been buried as the other sections had been. As of April 1980 the structure was trapping some material and no further structural damage occurred; however, the structure had not been monitored long enough to determine its effectiveness.

(5) Analysis.

(a) Gabion Revetment. This structure performed well in providing toe protection for the log revetment. Degradation of the gabions, however, may be a factor to consider before using them in such structures. The filter-cloth lining of the baskets broke, the PVC coating wore off some of the wire, the wire baskets broke in place, and the rubble inside the baskets fell out. This damage would eventually lead to the failure of the structure. Larger stones and stronger filter cloth might have prevented this gabion deterioration. However, this structure adequately protected the log revetment and is recommended for this purpose as long as structural degradation can be kept to a minimum. Repair of this type of structure would not be difficult.

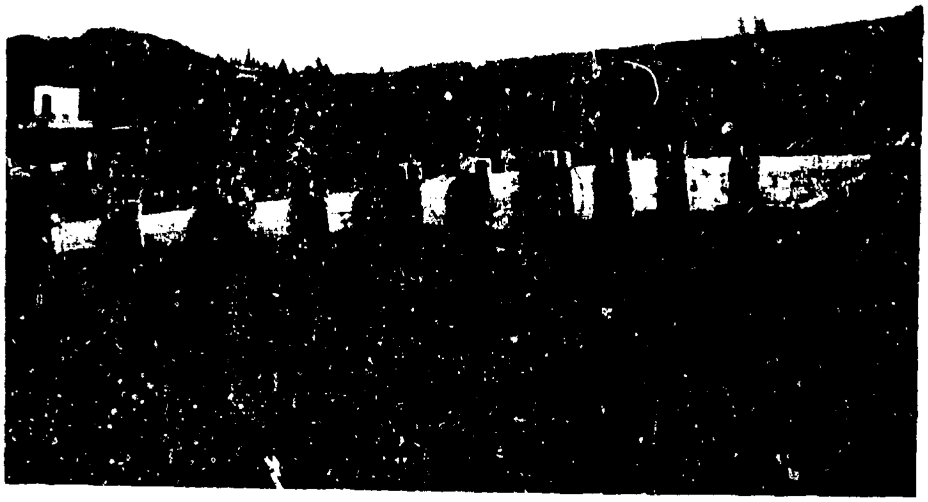


Figure 2-594. Groin 6 trapping littoral drift, Ninilchik, Alaska, 30 April 1980.

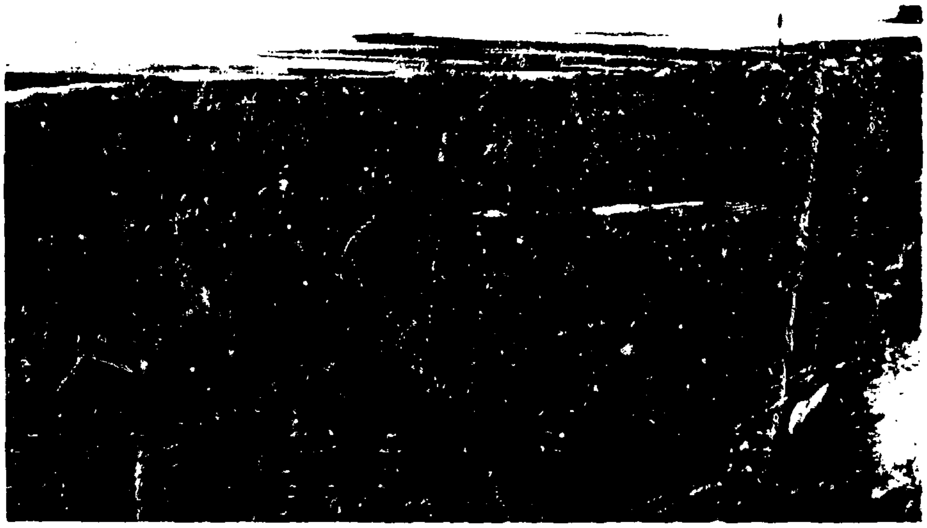


Figure 2-595. Missing corrugated metal pipe section next to log revetment, Ninilchik, Alaska, 22 March 1980.

(b) Gabion groin. This structure, like the gabion revetment, performed well but suffered structural damage. The damage in this case was the destruction of the first two gabion baskets on the top layer of the groin. Larger cobbles would have remained in the baskets and would have been less susceptible to movement. Despite the damage to this structure, the groin functioned effectively. Another problem with the structure was a shifting of the top section with respect to the bottom. This could be prevented by wiring the two parts together. This structure is a good example of effective low-cost shore protection but further experimentation with larger sized fill stone is recommended.

(c) Timber groins. All five of the timber groins were successful at this site. A minimal amount of structural damage was evident, and all the structures adequately trapped littoral drift. There was no timber arrangement that functioned appreciably better than the others. The use of gabion toe protection appears to be unnecessary at this site as littoral material was at a sufficiently high level to prevent toe scour along the groins during the monitoring period. Cost appears to be the governing factor in choosing one system over another.

(d) Corrugated Metal Pipe Groin. This structure suffered minor structural damage within the first 3 months after installation. The pipe that was not buried was torn away, apparently because the sand-gravel fill washed out of the bottom of the structure, leaving a hollow tube which was readily torn away by wave action. In addition, problems were encountered with the tipping of the concrete caps due to settlement or loss of material. The groin, despite the damage, continued to trap littoral drift, although the monitoring period has not been long enough to determine its full effectiveness. Adequate embedment of the pipes is essential for the structure to survive the rigors of this environment.

Table 2-105 provides volume calculations for changes between profile stations that occured from the first survey in June 1979 to the last survey in April 1980. The base line for the surveys is stationed from northeast to southwest, counter to the predominant direction of longshore transport. However, the pattern of gains and losses on opposite sides of groins indicates transport in the direction of the survey stationing, or southwestward. Longshore transport at Ninilchik may be in either direction along the beach, and the results in this table seem to reflect seasonal changes more so than long-term changes associated with the presence of the groin field. The groins, however, do collect sand adequately on their updrift sides as is apparent along the beach, which suggests that the groin field may be serving to hold the beach as intended.

7. Hawaii Sites.

a. Common Characteristics. The monitoring sites of Kualoa and Bellows Air Force Station are located on the northeast windward shore of the island of Oahu (Fig. 2-596). The climate is characterized by a two-season year (summer and winter), mild and uniform temperatures, marked variations in annual rainfall with geographical location, and prevailing northeasterly trade winds. The average annual temperature is 74° Fahrenheit with monthly temperatures varying from 71° Fahrenheit in January to 77° Fahrenheit in

Table 2-105. Volumetric analysis of beach profiles at Ninilchik, Alaska (22 June 1979 to 16 April 1980).

Device	Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
—	0+72	1,093.7	649.8	-443.9
—	2+22	335.7	464.4	128.8
—	2+72	157.1	221.5	64.3
Groin 7	2+97	37.3	46.2	8.9
Groin 7	3+03	163.4	182.2	18.9
Gabion revetment	3+28	159.0	154.8	-4.2
Gabion revetment	3+53	371.6	415.4	43.9
Gabion revetment	4+21	327.0	406.5	79.5
Gabion revetment	4+88	263.8	250.9	-12.9
Gabion revetment	5+34	151.9	115.6	-36.3
—	5+59	72.5	39.8	-32.6
Groin 1	5+74	73.6	8.5	-65.1
Groin 1	5+84	145.8	35.1	-110.7
—	6+03	124.6	80.0	-44.6
—	6+27	99.4	84.0.	-15.4
—	6+51	80.0	113.4	33.4
Groin 2	6+78	34.5	44.6	10.1
Groin 2	6+88	107.0	93.2	-13.7
—	7+09	128.2	101.2	-26.9
—	7+36	92.3	101.3	9.0
—	7+62	51.3	99.4	48.1
Groin 3	7+83	41.5	44.3	2.8
Groin 3	7+93	199.2	115.0	-84.2
—	8+28	143.1	118.8	24.3
—	8+66	89.5	147.6	58.1
—	9+04	59.4	156.6	97.1
Groin 4	9+36	25.7	42.1	16.4
Groin 4	9+46	75.2	63.1	-12.1
—	9+67	75.5	80.4	4.9
—	9+91	53.7	94.1	40.4
—	10+16	42.6	99.9	57.2
Groin 5	10+36	27.8	51.1	23.3
Groin 5	10+46	60.8	82.2	21.4
—	10+66	73.2	101.8	28.6
—	10+94	54.0	82.3	28.3
—	11+18	42.1	64.7	22.6
Groin 6	11+37	32.0	23.1	-8.8
Groin 6	11+47	62.4	50.3	-12.2
—	11+71	39.7	158.2	118.5
—	12+19	38.4	343.7	305.4
—	13+14	156.6	454.8	298.2
—	14+67			
	Totals	5,462.0	6,082.1	620.1

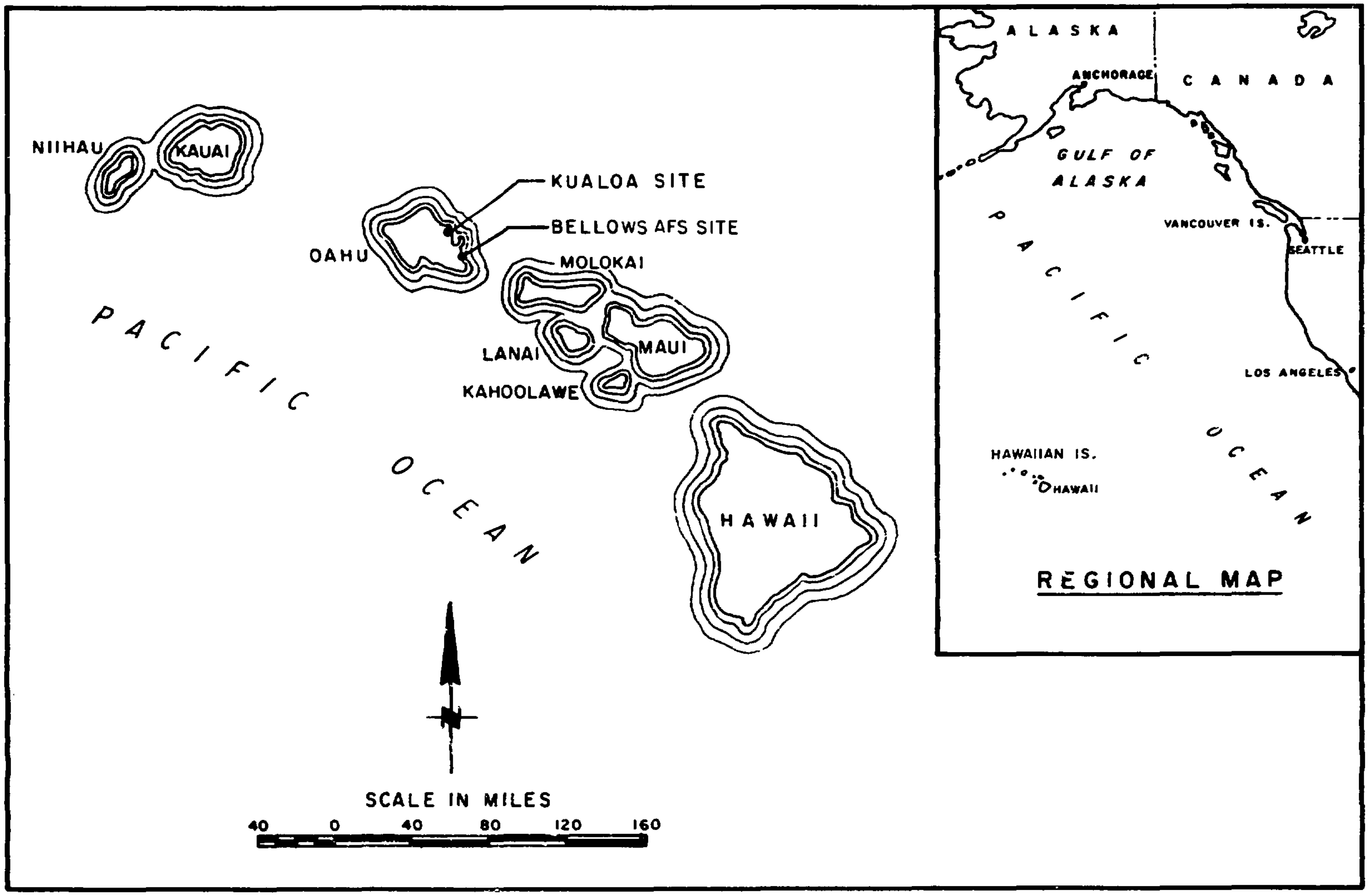


Figure 2-596. Location map of Hawaii monitoring sites.

August. Along the shore, the annual rainfall ranges between 40 and 75 inches per year. The northeasterly trade winds prevail 80 to 90 percent of the time during the summer months and about 50 to 80 percent of the time during the winter months.

The Hawaiian Islands are all of volcanic origin. Shallow coral reefs fringe most of the island shores, protecting the shoreline from the direct forces of ocean waves. These waves impinge on the reefs, breaking them down gradually into a calcareous sand that is carried landward to produce the white beaches that characterize many of the island shores. In the milder wave climate of these reefs, shore protection by means of low-cost devices is often possible.

The northeast shore of Oahu is exposed mainly to winds and waves arriving from the northeast quadrant. Waves generated in the North Pacific Ocean by the prevailing trade winds are of the greatest concern at each of the two monitoring sites. These waves have periods of 5 to 12 seconds and deepwater heights of 4 to 12 feet, and they approach the sites most frequently from the northeast and east. North Pacific swell, generated by storms in the Gulf of Alaska, approaches from the north and northeast with wave periods of 10 to 15 seconds and deepwater wave heights of 8 to 14 feet.

Large waves break on the edge of the reef dissipating much of the wave energy before reaching shore. However, during high tides or times of high water level during storms, a greater amount of wave energy can reach the shoreline as a result of wave regeneration in the shallow-reef waters. Astronomical tides in Hawaii are semidiurnal, with a diurnal inequality, and have a small range. Tidal data taken at Waimanalo Bay are applicable to Bellows (MHHW is +1.8 feet, and the highest estimated tide is +3.0 feet MLLW). Tidal data taken at Waikane, 2 miles southwest of Kualoa, are applicable to Kualoa (MHHW is +2.2 feet, and the highest estimated tide is +3.5 feet MLLW). In addition to wave energy, direct wind-stress currents in the shallow waters of the reef area play an important role in littoral processes. The lightweight coralline sands churned up by waves breaking at the shoreline are readily transported by these wind-driven currents.

b. Kualoa, Hawaii.

(1) Site Description.

(a) Geographical Setting. The Kualoa monitoring program was conducted at the Kualoa Regional Park, located at the northern limit of Kaneohe Bay (Fig. 2-597). The rural residential community of Kaaawa is north of the park. The park itself is at Kualoa Point, bounded by the Kamehameha Highway on the north, the Pacific Ocean on the east, Kaneohe Bay on the south, and Molii Fishpond on the west. The monitoring site is on the eastern side of the park where the shoreline trends about N. 13° W. The park site is generally flat, with an average elevation of approximately +6 feet MLLW. West of the park, the Koolau Mountain Range rises to a peak elevation of about 2,681 feet above MSL. Mokolii Island, commonly referred to as "Chinaman's Hat," is located about 1,800 feet off Kualoa Point and is accessible by foot from the shore at low tide.

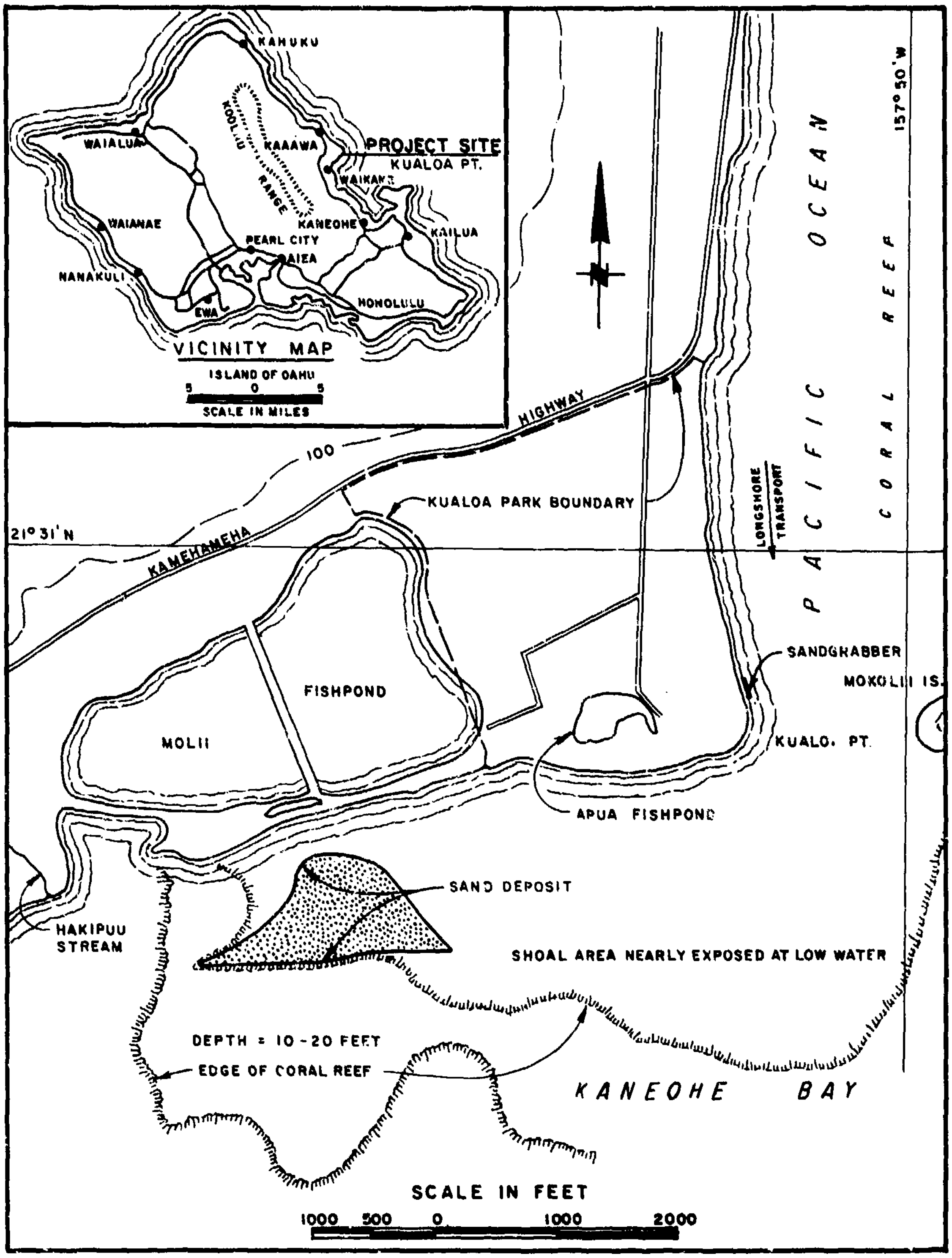


Figure 2-597. Location map of Kualoa, Hawaii, monitoring site.

(b) Geomorphology, Soils, and Vegetation. The Kualoa site is located on the northeast flank of the prehistoric Koolau Volcano. Several periods of marine and subaerial erosion of the volcano and isostatic sinking of the island have contributed to the present appearance of the core and flanks. In addition to the periods of erosion, a geomorphic pattern of fringing, patch, and barrier reefs developed, and the reefs were modified by changes in sea level resulting from the advance and retreat of continental glaciers. A fringing coral reef extends about 2,000 feet seaward of the eastern park shoreline and about 1,800 feet southward into Kaneohe Bay. A large deposit of sand lies about 1,500 feet southwest of the southern park shore, directly offshore Molii Fishpond.

The Kualoa peninsula is a large, dynamic sandpit which was incrementally built in a southward direction into Kaneohe Bay by the net southward movement of sand from the reefs and beaches north of Kualoa. Most of the peninsula consists of unconsolidated marine calcareous sediments, and the very permeable beach sand consists of grains of worn coral, coralline algae, and shells. Vegetation in the immediate area of the monitoring site consists of grass, extending from the top of bluff landward, and a few palm trees randomly spaced behind the bluff.

(c) Waves, Longshore Transport, and Erosion. The LEO data (Table 1-3) indicate that wave heights average 0 to 1 foot with a maximum of 1.7 feet. The wave climate is classified as intermediate. Although the energy-flux analysis indicates a small net potential for southward longshore transport at this site, only 30 observations were made throughout the 3-month analysis period, and the results probably reflect this shortage of basic data. The actual net transport rate appears to be much greater than the analysis indicates. Results of a study of littoral currents at Kualoa Park in August 1979, carried out under the prevailing trade-wind conditions, indicated southward transport at an average speed of 6 feet per minute. On the southern shoreline, the current had a strong westward movement of about 10 feet per minute. Sand-tracing studies concluded that littoral transport of sand moves in a clockwise direction around Kualoa Point. Other causes of erosion at Kualoa Beach have been the building of manmade structures in the area. For example, groins built north of Kualoa Beach may have caused a temporary disruption of the longshore transport of material that provides the area with part of its sand supply.

(d) The Problem. The eastern beach at Kualoa Regional Park has undergone continuous erosion. Resulting shoreline changes are shown in Figure 2-598. The average annual loss of sand from the eastern beach area from 1949 to 1975 was 4,300 cubic yards. During this period, the eastern shoreline had receded at an average rate of about 4 feet per year, and the shoreline near Kualoa Point had receded at about 7 feet per year. Although most of the park's southern shoreline accreted during 1949-75, there was a net loss of 80,000 cubic yards for the whole park. This represents more than 6 acres of parklands lost. Studies have indicated that, from 1949 to 1975, 30,000 cubic yards of sand was lost from the littoral system at Kualoa, probably to the offshore sand deposit south of Molii Fishpond. Beach erosion appears almost continuous during trade-wind conditions, and a period of higher than normal tides under typical trade-wind conditions can accelerate shoreline erosion.

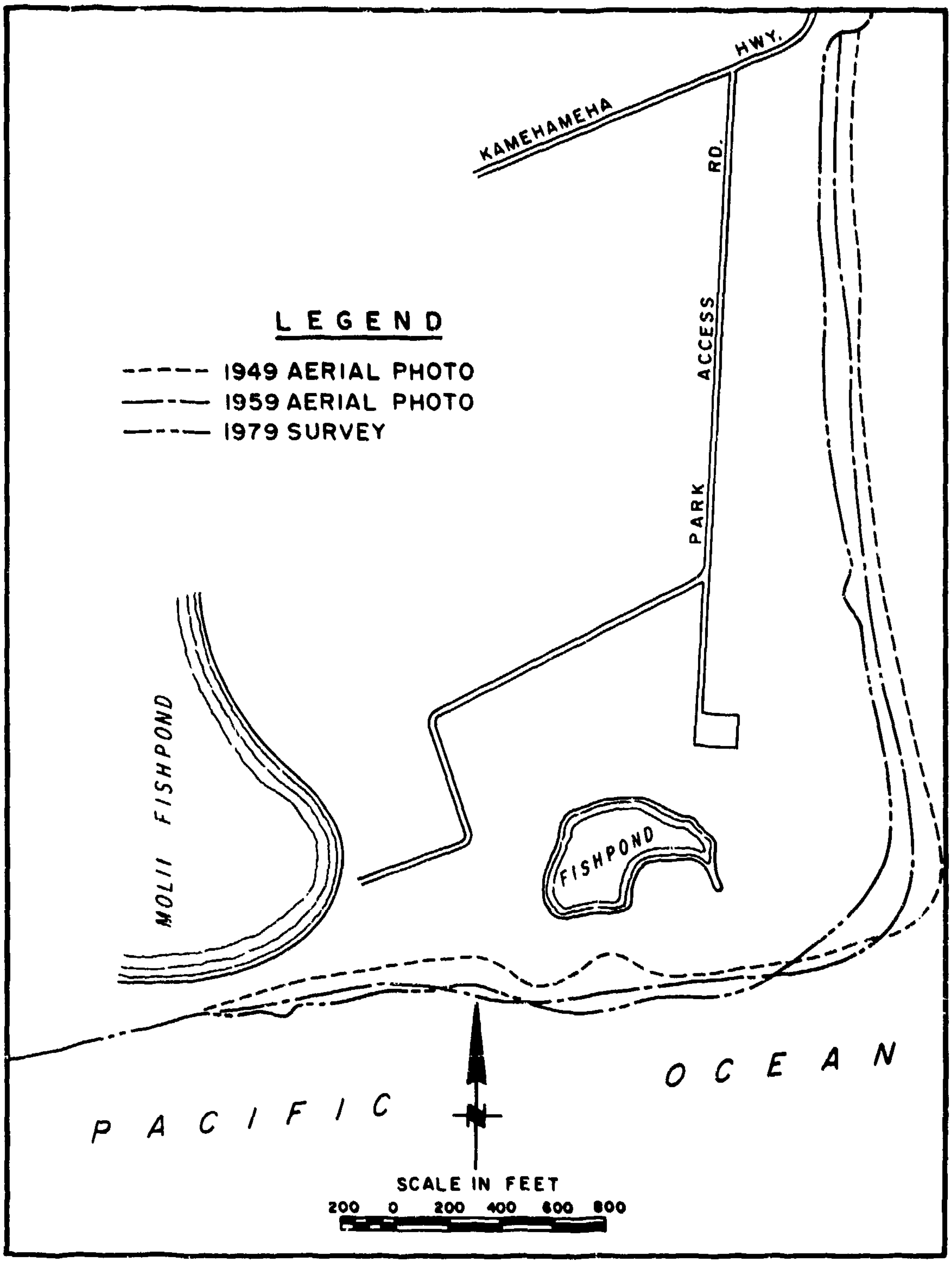


Figure 2-598. Shoreline changes at Kualoa Park.

(2) Monitoring Project. In late fall of 1977, the Department of Parks and Recreation, City and County of Honolulu, had a contractor install a 200-foot-long Sandgrabber just north of Kualoa Point to test its effectiveness in controlling erosion of the park shoreline. A profile of the device is shown in Figure 2-599. After the structure was completed on 5 December 1977, the Department conducted a 2-month study of its performance.

Two problems became apparent almost immediately. The seaward course of blocks consisted largely of red cinder blocks, of a lighter consistency than the blocks used for the main body of the structure. By 26 December 1977, most of these blocks had broken under the effects of wave action, and the structure seemed to be gradually working itself apart. Shortly thereafter, the contractor removed the broken blocks along with the entire seaward course and retightened the loose tie rods. This restored the structural integrity of the Sandgrabber. The second problem which became apparent was that the south end of the Sandgrabber was not curved far enough toward the beach berm to prevent wave attack on the back side of the structure. This southeasterly wave attack was not expected. Aerial photos indicate that it was caused by diffraction of the northeast trade waves around Mokolii Island, and subsequent refraction over the reef. When this occurred, the waves eroded the sand from behind the south end of the Sandgrabber. A protective extension was added to the south end; however, even this extension did not completely solve the problem of wave attack from the southeast. There was continuing evidence that waves were getting in behind the structure.

By the end of the 2-month study, the structure had settled approximately 1 foot into the sand along a 10-foot reach at the south end and along a 15-foot reach near the center. The structure remained intact, and there was a smooth transition from the slumped areas to the rest of the Sandgrabber. Approximately 148 cubic yards of sand accreted both behind and seaward of the structure along its 200-foot length. This was estimated to be about 15 percent of the material available in the littoral transport system. Severe erosion occurred downstream of the structure, extending for a distance of 300 feet. Although the Sandgrabber contributed somewhat to the erosion by retarding littoral transport past the site, it appeared that larger waves than normal, combined with high tides, caused most of the erosion.

In conclusion, the Department of Parks and Recreation felt that longer term observations were needed to allow a more credible determination of the structural stability and functional performance of the installation. The 2-month monitoring period was considered representative only of a typical winter in which the trade winds blow about 63 percent of the time, and Kona winds (from the southwest quadrant), about 20 percent of the time. On a yearly basis, the trade winds blow 82.6 percent of the time. Upon the recommendation of the Division Engineer, Pacific Ocean Division, monitoring of the Kualoa Sandgrabber was continued under the demonstration program.

(3) Performance. Overall, the Sandgrabber remained structurally sound throughout the monitoring period. Differential settlement along its midsection, and of individual blocks, was observed in April 1978 (Figs. 2-600 and 2-601). The entire structure was also rotating downward on the seaward side. The short section on the south end had a seaward slope of 13° (Fig. 2-602). By May, although this short section had continued to settle, the seaward slope was only 8°.

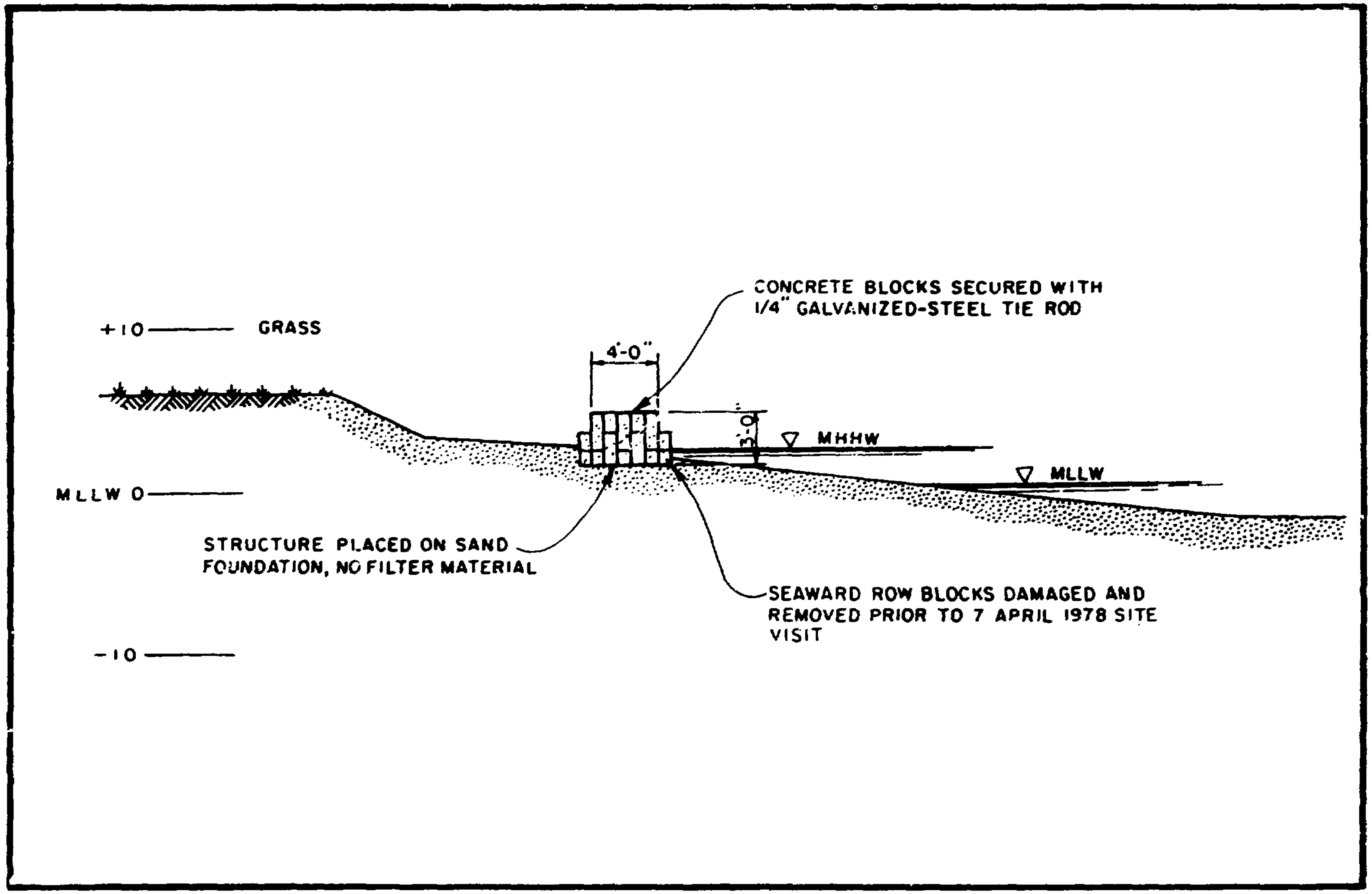


Figure 2-599. Sandgrabber profile at Kualoa site.

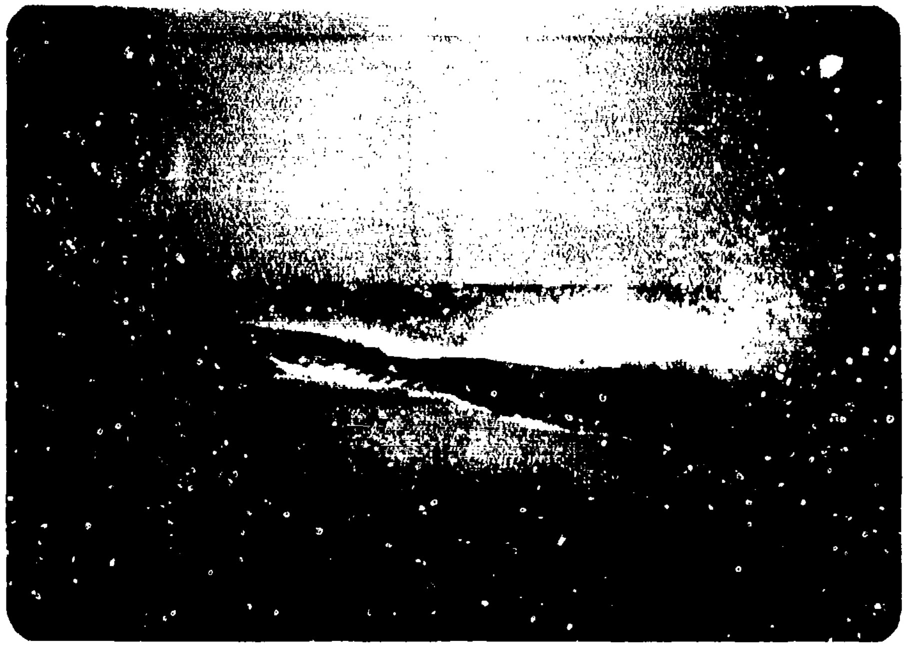


Figure 2-600. Settlement of blocks along midsection, Kualoa, Hawaii, 7 April 1978.

Figure 2-601. Differential settling of individual blocks viewed from structure's midsection, Kualoa, Hawaii, 7 April 1978.

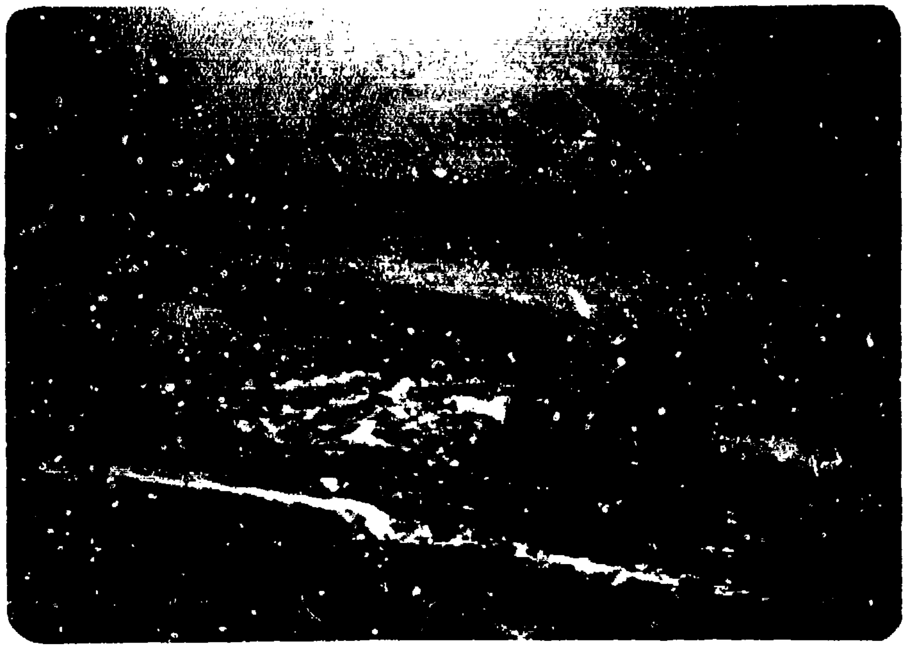


Figure 2-602. Seaward slope of south end of Sandgrabber, Kualoa, Hawaii, 7 April 1978.

Differential settlement of individual blocks continued over the following 18 months. Although the tie rods were loosening and many of the blocks on the seaward face broke, the structure ceased to rotate. By January 1979, four blocks had been damaged--two near the south end were vertically displaced, and two near the center of the structure had broken away (Figs. 2-603 and 2-604). By May 1979 most of the tie rods were rusted and loose, and in November 1979 several blocks in the seaward face were broken apart (Figs. 2-605 and 2-606). Despite this component damage, the Sandgrabber remained functionally effective.

By June 1978, the preliminary effects of the Sandgrabber at the Kualoa site were identified. Sand accumulated on the landside of the structure as expected, but not along the seaward face. The northern and southern beach areas near the Sandgrabber accreted somewhat at first, but erosion of the downdrift bank was accelerated (Fig. 2-607). Between June 1978 and August 1979, erosion of the downdrift bank averaged +1 foot per month. The rate of erosion decreased through November 1979. Figure 2-608 shows the progressive erosion downdrift of the Sandgrabber from November 1978 to November 1979. Between November 1979 and January 1980, +10 feet of downdrift shoreline was lost as a result of winter storms.

In January 1979, there was slight accretion along the seaward face (compare Figs. 2-609 and 2-610); however, this trend did not continue. Accretion of sand landward of the structure continued until September 1979, when minor scouring along the central segment of the landward face was observed.

Figure 2-603. Vertically displaced block toward the south end of the structure, Kualoa, Hawaii, 4 January 1979.

Figure 2-604. Displaced blocks lying in sand along mid-section of the structure, Kauloa, Hawaii, 4 January 1979.

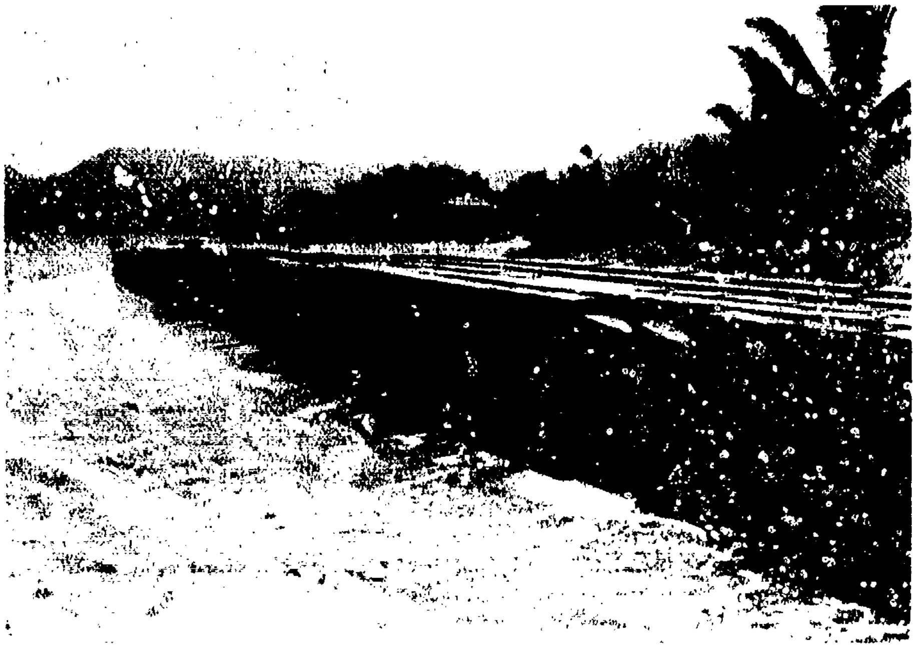


Figure 2-605. Broken blocks viewed from midsection of structure, Kualoa, Hawaii, 8 November 1979.

Figure 2-606. Broken blocks near north end of structure, Kualoa, Hawaii, 8 November 1979.

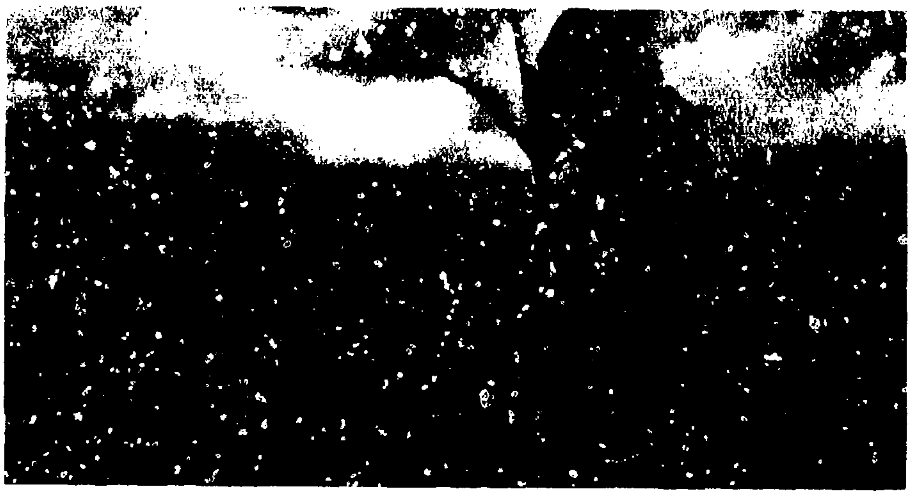


Figure 2-607. Erosion of downdrift bank, Kualoa, Hawaii, 19 June 1978.

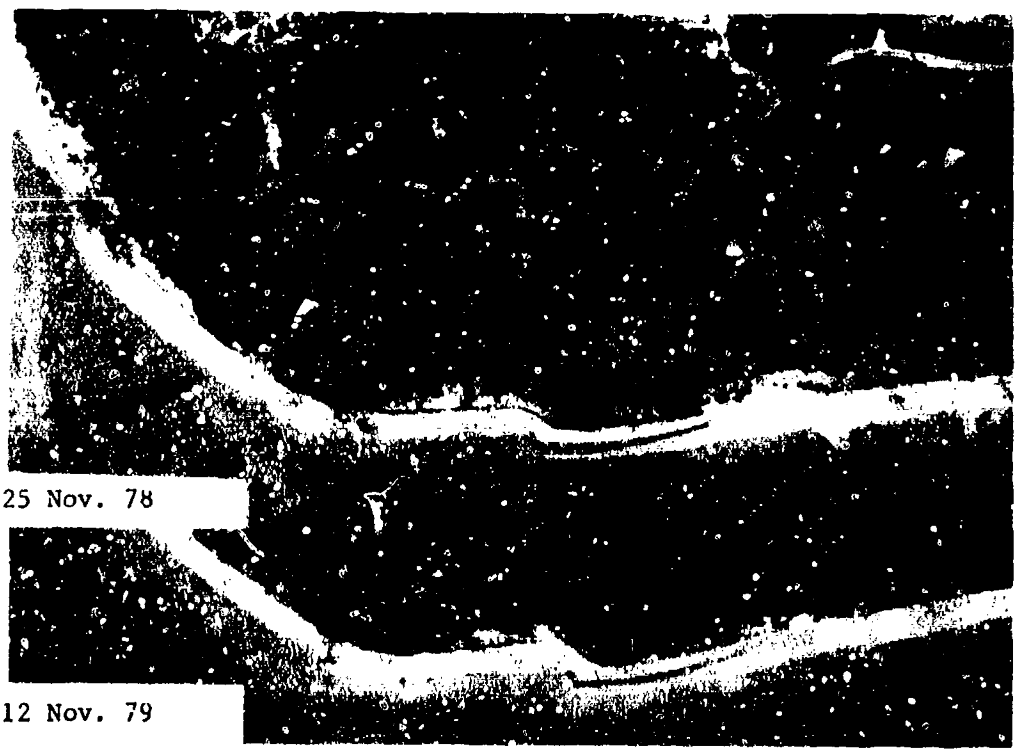


Figure 2-608. Progressive erosion south of Sandgrabber, Kualoa, Hawaii, 25 November 1978 and 12 November 1979.

Figure 2-609. Seaward face of structure, Kualoa, Hawaii, 19 June 1978.

Figure 2-610. Sand accumulation on seaward face of structure, Kualoa, Hawaii, 4 January 1979.

A storm in January 1980 damaged the structure significantly. Four blocks on the landside were broken and 14 were broken on the seaside. Thirteen blocks were missing from the Sandgrabber, five double blocks and eight single blocks from the seaside. Structural damage was progressive. Typically, the bottom half of the seaward double blocks broke, then the remaining unsupported top half worked loose; the single-block second row from seaward was then exposed and became dislodged (Fig. 2-611). Waves eroded the adjacent beach to the south and flattened the slopes. Continued erosion of the beach to the south has occurred (Figs. 2-612 and 2-613).

Figure 3-79, which was put in Section III for comparison of similar devices shows a series of profiles through the sandgrabber. The profiles show the accretion and erosion trends and depict the uneven settlement of the structure.

(4) Analysis. The Sandgrabber demonstrated its usefulness in trapping sand and stabilizing the immediate shoreline at Kualoa. However, it is important, where beach stability depends on nourishment by longshore transport, that the littoral supply to downdrift beaches not be totally cut off by a sand conservation device. The Sandgrabber removed material from the littoral system, causing accelerated downdrift erosion. The lost land area has not been recovered.

The structural integrity of the installation was compromised by differential settlement of the foundation blocks. Some differential settelement is provided for by the tie-rod arrangement, but the allowable limits were exceeded. This resulted in cracked blocks and loosened tie rods, which eventually would hasten structural failure to the extend that the Sandgrabber would no longer function as intended. Two solutions to this problem are suggested: (a) Provide a suitable bedding foundation or initially excavate the foundation to the

Figure 2-611. A damaged section of the Sandgrabber at Kualoa, Hawaii, 17 January 1980.

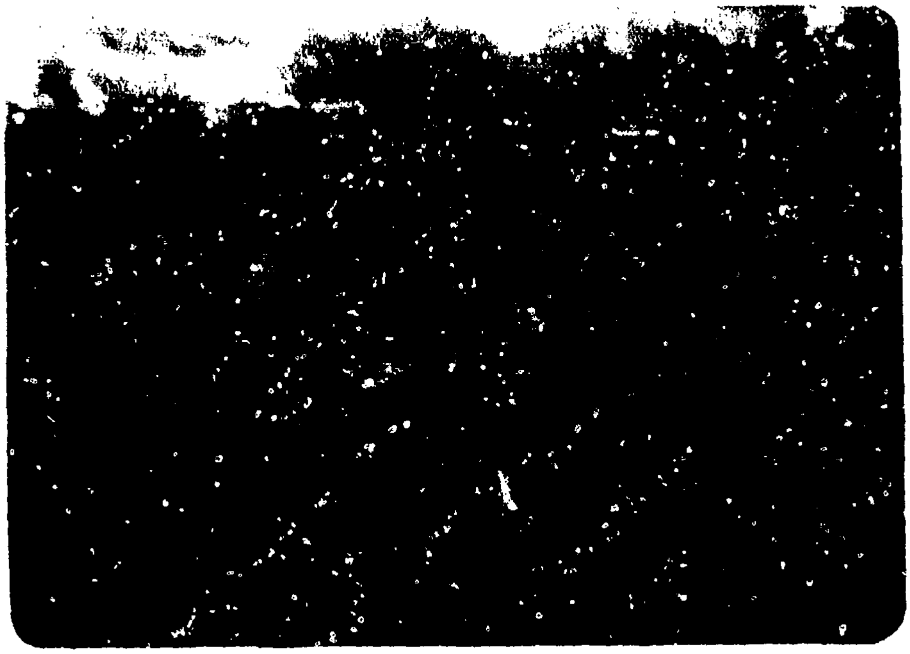


Figure 2-612. The beach south of the structure was eroded by storm waves, Kualoa, Hawaii, 17 January 1980.

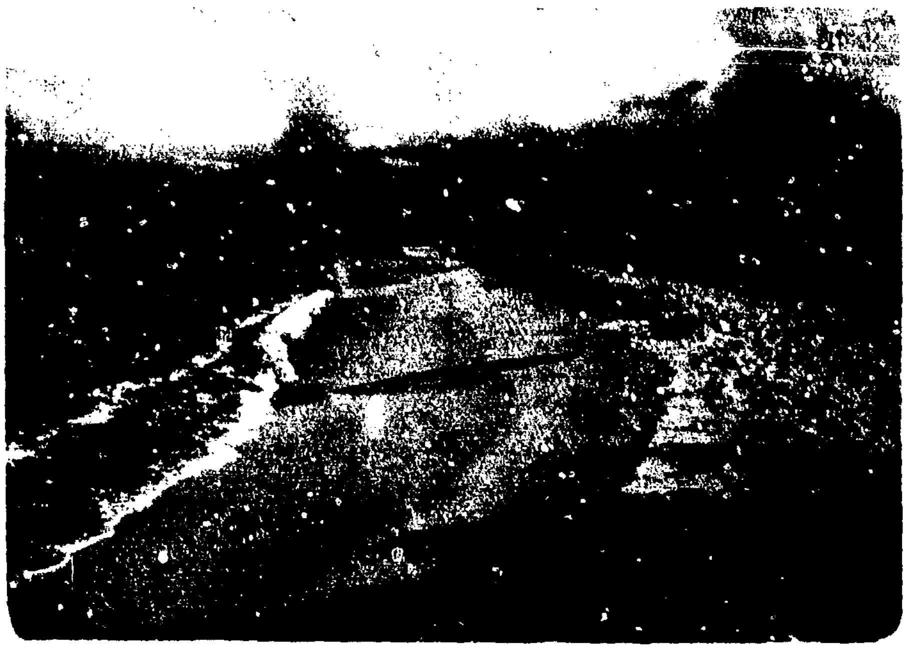


Figure 2-613. Continued erosion of the beach south of the structure, Kualoa, Hawaii, 29 May 1980.

estimated stable depth and tilt angle, and (b) provide a method of interconnecting the blocks that will allow more differential settlement without compromising the structural integrity of the system.

Failure of blocks is also due to wave impact and uplift forces. Strength limits were exceeded, even in the relatively mild wave climate at Kualoa. Use of this structure is recommended for wave conditions in which wave heights do not exceed 3 feet at the structure toe.

Table 2-106 gives volume calculations for changes between profile stations that occurred from March 1978 to May 1980. The base line for the surveys is stationed from north to south, the direction of longshore transport. The table shows that littoral material accreted at the profiles where the Sandgrabber was located. Immediately south of the structure, the downdrift shoreline shows erosion. In this situation, the use of a Sandgrabber (in an area of strong littoral transport) has caused erosion of the downdrift shoreline.

Table 2-106. Volumetric analysis of beach profiles at Kualoa, Hawaii (16 March 1978 to 30 May 1980).

Device	Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
Sandgrabber	25+50			
		6.0	178.8	172.2
Sandgrabber	26+50			
		43.3	180.7	137.4
Sandgrabber	27+50			
		64.7	85.0	20.3
Sandgrabber	28+00			
		119.7	58.3	-61.3
--	28+50			
		120.1	56.0	-64.2
--	29+0			
		196.5	72.4	-124.1
--	29+50			
		372.6	70.6	-302.0
--	30+00			
		543.7	88.4	-455.3
--	30+50			
		725.5	71.1	-654.4
--	31+00			
		725.7	61.4	-664.3
--	31+50			
		1,154.0	110.3	-1,043.6
--	32+50			
		516.4	288.0	-228.4
--	33+50			
	Totals	4,588.1	1,320.3	-3,267.8

c. Bellows Air Force Station, Hawaii.

(1) Site Description.

(a) Geographical Setting. This monitoring site is located at Bellows Air Force Station near the north end of Waimanalo Bay, about 12 miles southeast of Kualoa Point (Fig. 2-614). The shore segment from Wailea Point to the south boundary of the Air Station is known as Bellows

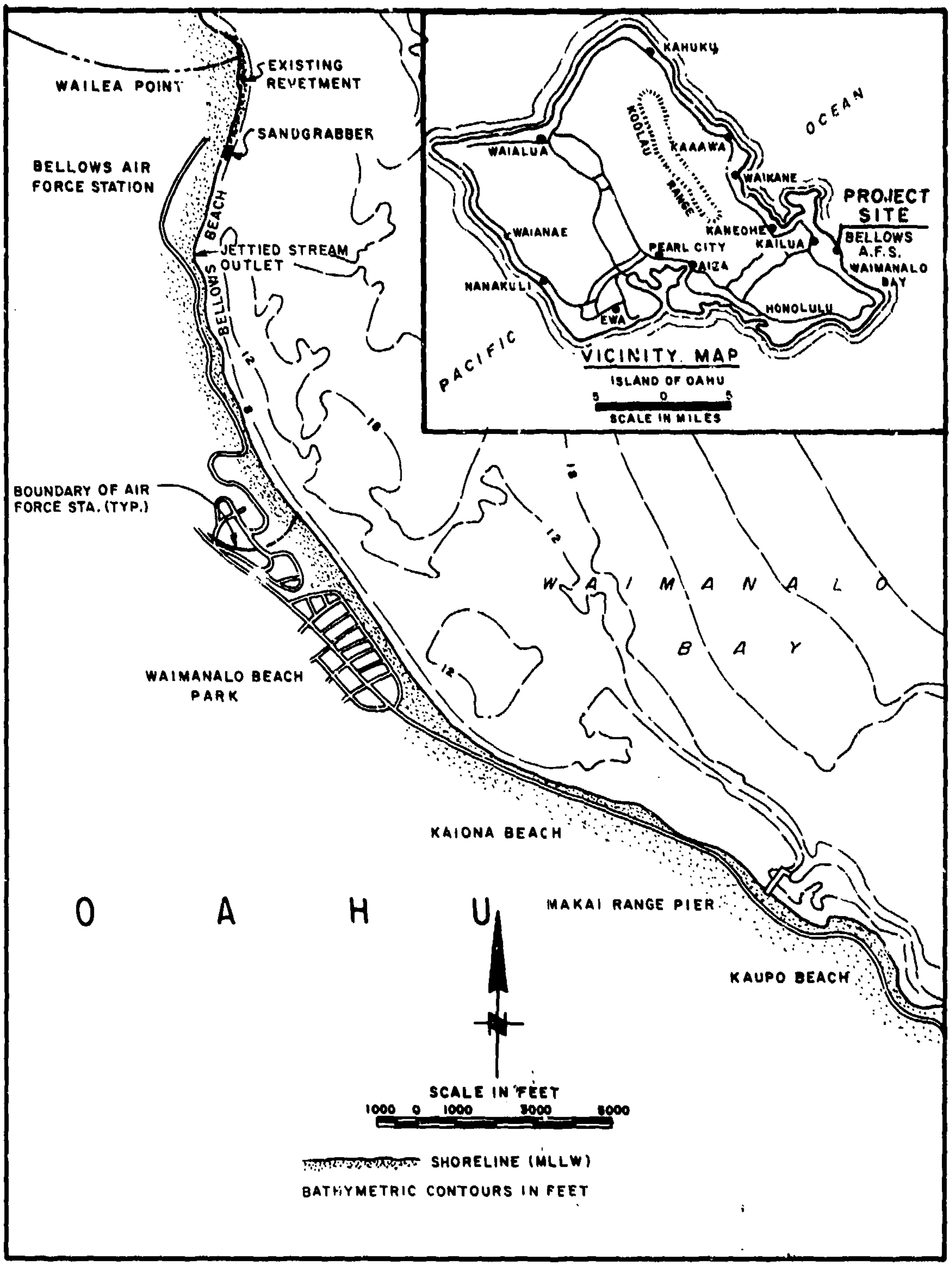


Figure 2-614. Location map of Bellows Air Force Station, Hawaii, monitoring site.

Beach. At the monitoring site, the shoreline trends about N.20°E. Two stream-mouth jetties are located 3,000 feet south of the project site.

(b) Geomorphology, Soils, and Vegetation. A shallow reef extends approximately 3,000 feet offshore. Some patch reefs north of the site are exposed at MLLW. The reef lying to the east and southeast varies in depth from 6 to 12 feet. Beach sand at the site is light colored and fine, except in front of the coral rock revetment just north of the Sandgrabber site, where the sand is coarser. The beach in this area has been eroded, with waves reaching the reveted bank during most tidal stages. The coarser sand may be indicative of selective transport that has removed the finer grains. Vegetation adjacent to the beach is comprised of native ground cover and many ironwood trees (*Casuarina equisetifolia*).

(c) Wave Conditions. Waimanalo Bay is protected from direct deepwater wave approach by the offshore barrier reef. The protection is greatest for waves approaching from the north, which encounter the patch reefs exposed at MLLW. The site is exposed to higher waves approaching from the east and southeast, where the reef is deeper. In a December 1978 study, wave parameters at the site were observed in accordance with the LEO format. The LEO study consisted of six measurement periods over 3 days, thus providing only basic and approximate information. Measured wave periods ranged from 5 to 8.5 seconds, and average breaker heights were estimated to range from 1.3 to 1.7 feet. The maximum estimated breaker height was 2.5 feet. During the study, breaker type was consistently of the spilling-plunging type. The observers estimated that breaking wave heights of greater than 4 feet would be unusual.

(d) Longshore Transport and Erosion. The LEO 3-day study also yielded information on longshore currents. Dye tracing indicated a longshore current moving alternately south and north with almost uniform frequency and no apparent relationship to the tidal stage or to the breaking wave angle. Current speeds ranged up to a maximum of 85 feet per minute. The observed angle of wave approach was usually from the north at angles off the normal-to-shore ranging from 5° to 8°, which would be expected to generate a southward longshore current. However, the dye occasionally moved to the north. This anomaly was attributed to the confused pattern of wave crests in the study area and the presence of rip-current cells, which were apparent in aerial photos taken 7 November 1978. The data base was too meager to determine whether littoral transport was predominantly onshore-offshore or alongshore, nor could the seasonal characteristics of the transport be determined. The causes of the severe erosion problems along Bellows Beach are not readily apparent, and littoral transport processes have not been adequately studied. The LEO data for the monitoring project under the Federal program (Table 1-3) indicate that wave heights average 1 to 2 feet, with a maximum of 5.6 feet. The wave climate is classified as severe. The energy-flux analysis indicated a net longshore transport potential of 130,000 cubic yards southward for the 6 months analyzed.

(e) The Problem. A comparison of 1967 and 1978 aerial photos showed that Bellows Beach had severely eroded during that 11-year period. In 1967, a 350-foot-long coral rock revetment existed 1,200 feet south of Wailea Point. An 80-foot-wide sand beach extended 1,200 feet from the south end of the revetment to the project site, where the beach was 100 feet wide. By 1978, the revetment had extended in both directions until it was continuous from Wailea Point to the project site, and the beach in front of the revetment was narrow or nonexistent. A 1978 visual inspection indicated that erosion was occurring along most of the shoreline between the project site and the stream-mouth jetties, exposing the roots of numerous ironwood trees.

(2) Monitoring Project. In early February 1979, a 100-foot-long Sandgrabber was installed by the U.S. Air Force on Bellows Beach (Fig. 2-615). This structure was selected for monitoring under the demonstration program.

(3) Construction. Construction was accomplished by enlisted U.S. Air Force personnel under the supervision of the Sandgrabber contractor. The blocks were trucked to a stockpile above the site and transferred to the beach with a forklift (Fig. 2-616). As a base for the bottom row of blocks, a structure-wide width of beach along the structure axis was leveled. The blocks were then placed in section and tied together with galvanized tie rods (Fig. 2-617). The north end of the structure was built flush against the existing revetment to provide continuity (Figs. 2-618 and 2-619).

(4) Performance. Generally, the concrete blocks and galvanized-steel tie rods of the Sandgrabber withstood the natural forces of waves and tides. Some settlement was detected about 1 week after installation, but there was negligible differential shifting of individual blocks. One block and one tie rod were broken at the north end as the structure settled against the adjacent revetment stone (Fig. 2-619). Over the next 5 months, the structure continued to settle, rotating downward on the seaward side. This resulted in a few loose tie rods, but the blocks remained intact. During August 1979, the seaward settlement increased, with a seaward slope of 5° on the south end, 11° along the midsection, and 15° on the north end (Fig. 2-620). About eight blocks on the northern landside were broken apart by tension forces exerted by the tie rods as a result of seaward settlement (Fig. 2-621).

The effects of the Sandgrabber on Bellows Beach were detectable within days of its installation. Sand accumulation on the landside of the structure had began, evidenced by the coarser nature of the trapped material relative to the existing beach material. For 2 months after installation, sand continued to accrete on both the landward and seaward sides of the structure. In April 1979, sand accumulation reached a maximum (Figs. 2-622 to 2-625). By June 1979, the sand level on the seaward face of the structure had dropped about 6 inches; the landward sandline was unchanged since April. At that time, some slight scouring on the landward side of the structure along the north end was reported. The July 1979 visit revealed that the scouring along the north end was continuing, but otherwise the sand accumulation was holding (Fig. 2-625). A major change occurred during August 1979. The scouring along the north end worsened, creating a trench landward of the Sandgrabber which worked its way southward along the length of the structure (Figs. 2-620 and 2-626). The north end of the structure was then completely scoured out and rested on the existing revetment stones. Settlement of the structure continued until January 1980, when accretion of the entire beach completely buried it. The structure was still buried in June 1980, and it appears to be in equilibrium with the beach material; i.e., the structure is floating in sand. Figure 3-79 (see Sec. III) shows a series of profiles through the Sandgrabber, depicting its uneven settlement. However, the accuracy of some of the survey data is questionable.

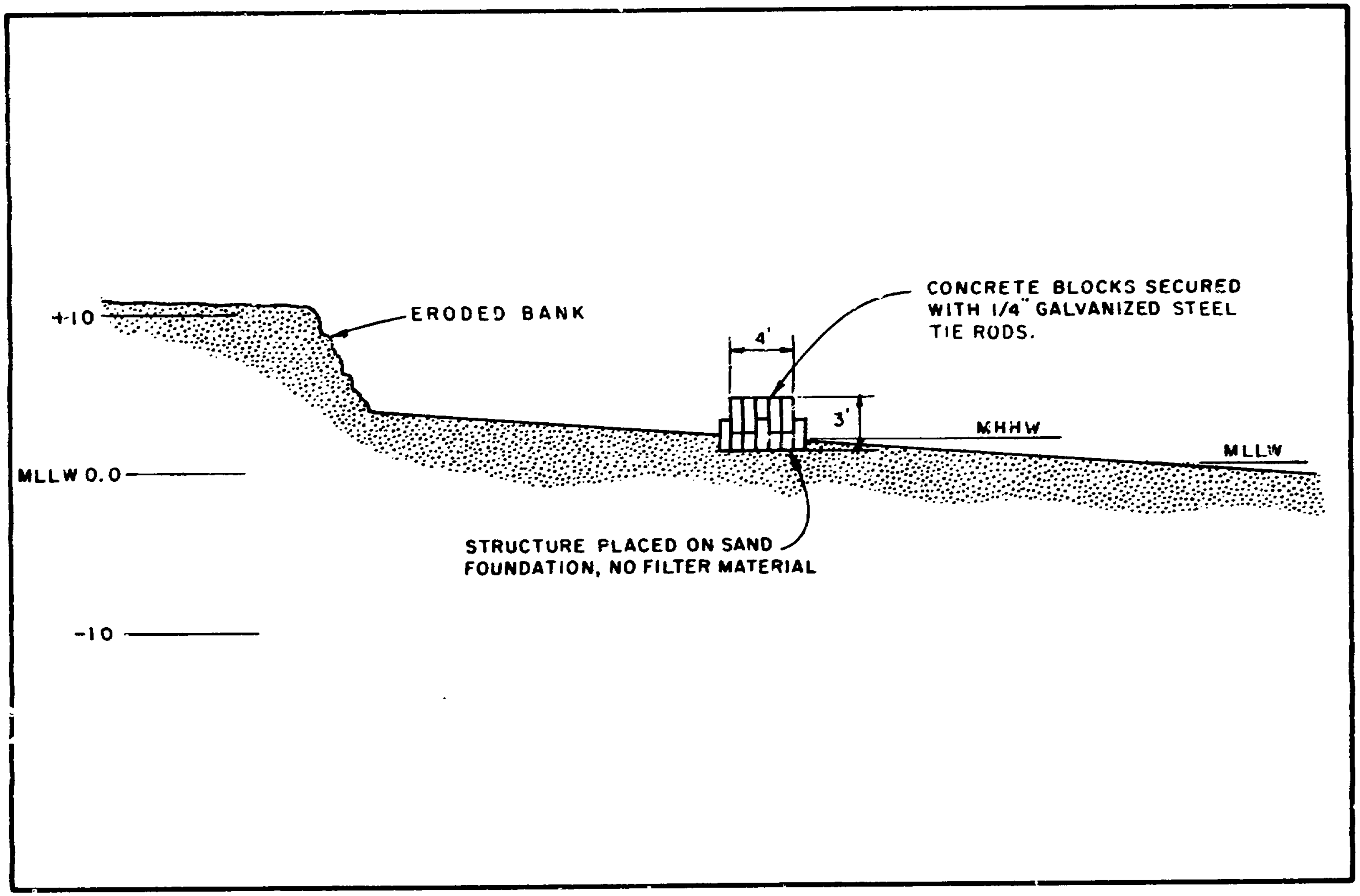


Figure 2-615. Sandgrabber profile at Bellows Air Force Station site.

Figure 2-616. Forklift used to transport blocks, Bellows Beach, Hawaii, 8 February 1979.

Figure 2-617. Placing blocks by hand, Bellows Beach, Hawaii, 8 February 1979.

Figure 2-618. Finishing Sandgrabber construction, Bellows Beach, Hawaii, 8 February 1979.

Figure 2-619. Broken block and tie rod, north end, Bellows Beach, Hawaii, 14 February 1979.

Figure 2-620. Seaward settlement of Sandgrabber, Bellows Beach, Hawaii, 5 September 1979.

Figure 2-621. Broken blocks, landward side, Bellows Beach, Hawaii, 5 September 1979.

Figure 2-622. Sand accumulation, seaward side of south end, Bellows Beach, Hawaii, 12 April 1979.

Figure 2-623. Sand accumulation, landward side of north end, Bellows Beach, Hawaii, 12 April 1979.

Figure 2-624. Sand accumulation, landward side of south end, Bellows Beach, Hawaii, 12 April 1979.

Figure 2-625. Scour beginning at the north end, Bellows Beach, Hawaii, 3 July 1979.

Figure 2-626. Trench working its way south landward of structure, Bellows Beach, Hawaii, 5 September 1979.

(5) Analysis. The Sandgrabber at Bellows Beach demonstrated its short-term ability to trap sand and enhance accretion trends of the immediate beach. Being permeable, this device allowed the beach profile to change with the wave conditions, although not as much as with an unprotected beach. The Bellows Sandgrabber removed considerable sand from the littoral system initially, but erosion of the adjacent beaches was not as apparent as at Kualoa. This points to the hypothesis that sand transport at the Bellows site is predominantly onshore-offshore rather than alongshore.

Some structural performance problems developed at this site which a few comments might help to mitigate at future installations. First, since the sand at Bellows is fine grained, the settlement of the seaward side of the structure due to toe scour should have been anticipated, and bedding should have been provided or the foundation trench should have been excavated to the anticipated scour depth. Second, the Sandgrabber is not sufficiently flexible; it can be damaged by uneven settlement. This was clearly demonstrated at this site by comparing the north and south ends of the installation (Figs. 2-627 to 2-630). At the north end, the Sandgrabber settled on the adjacent, existing coral rock revetment and, due to inflexibility, was unable to conform to the scouring bottom (Fig. 2-630). As a result, the north end of the structure was damaged. This problem might have been avoided had the structure not settled as much, had the structure been more flexible, or had the north end of the Sandgrabber not been constructed such that it settled onto the rock structure. Third, if differential settlement of the Sandgrabber is anticipated, elastic "ties" of some sort might be substituted for the steel tie rods. This would eliminate the breaking of blocks that results from excessive stresses due to inflexibility of

Figure 2-627. North end of Sandgrabber abutting rock revetment, Bellows Beach, Hawaii, 5 September 1979.

Figure 2-628. Scour trench along north end of structure, Bellows Beach Hawaii, 5 September 1979.

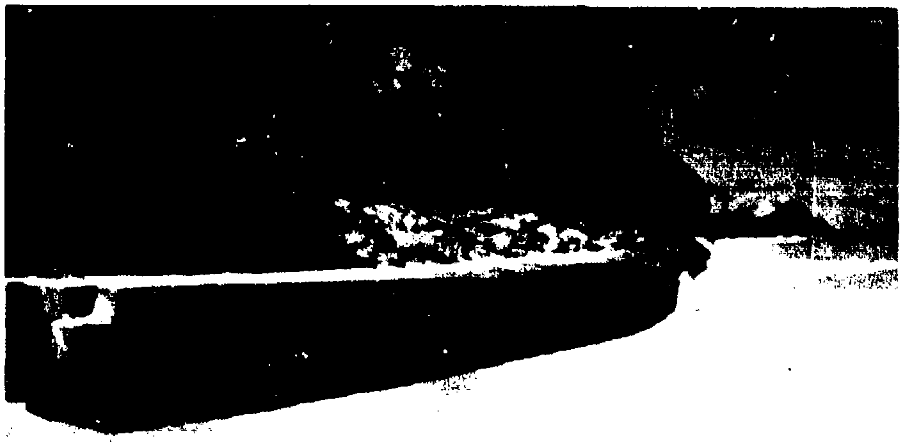


Figure 2-629. South "free end" of the Sandgrabber, Bellows Beach, Hawaii, 5 September 1979.

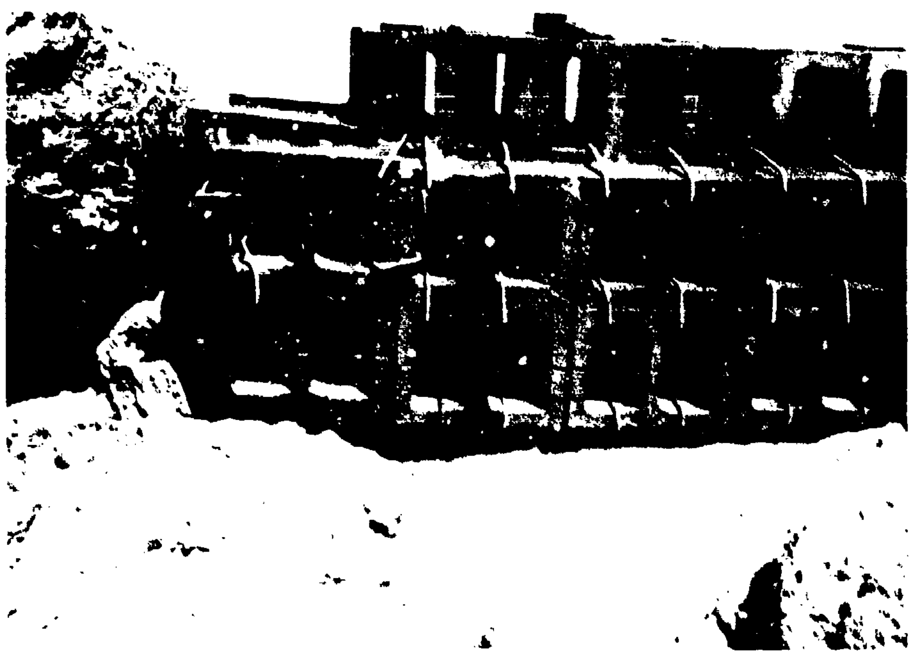


Figure 2-630. Landside of north end of Sandgrabber settling on the existing coral rock revetment, Bellows Beach, Hawaii, 5 September 1979.

the interconnections. A comparision of the Sandgrabber installations at Kualoa and Bellows yields some pertinent observations:

(a) The Kualoa Sandgrabber suffered little scouring and settlement compared to the Bellows Sandgrabber. Scouring and settlement were more localized for Kualoa than for Bellows, where the Sandgrabber settled uniformly over its entire length. The sand is much coarser at Kualoa than at Bellows, therefore providing a better foundation. When the water level is high at Bellows, the sand probably fluidizes, allowing the structure to sink.

(b) Southward longshore transport is much more predominant at Kualoa than at Bellows. There was therefore a significant impact on the downdrift shoreline at Kualoa, whereas virtually no impact on adjacent beaches was discernible at Bellows.

Table 2-107 provides volume calculations for changes between profiles that occurred between October 1978 and May 1980. The base line for this survey is stationed from approximately north to south. The direction of littoral transport is assumed to be onshore-offshore at the Bellows Air Force Station site. The table shows that the beach accreted during the monitoring period. This accretion seems due to the presence of the Sandgrabber, but it may also have been the result of a long-term trend of net accretion. Paradoxically, the volumetric analysis shows less accretion at the Sandgrabber than elsewhere at the Bellows site.

Table 2-107. Volumetric analysis of beach profiles at Bellows Air Force Station, Hawaii (30 October 1978 to 14 May 1980).

Device	Station	Erosion (yd^3)	Accretion (yd^3)	Net accretion (yd^3)
--	0+00	9.8	190.6	180.8
--	0+50	19.0	238.0	218.9
Sandgrabber	1+00	15.1	220.4	205.4
Sandgrabber	1+50	15.9	213.2	197.3
Sandgrabber	2+00	30.8	101.9	71.1
Sandgrabber	2+25	31.6	100.6	69.0
--	2+50	42.5	233.7	191.2
--	3+00	33.2	271.4	238.2
--	3+50	29.3	343.2	313.9
--	4+00	30.6	520.2	489.7
--	4+50			
	Totals	257.8	2,433.3	2,175.5

III. PROGRAM ANALYSIS BY SYSTEMS

The various shore protection systems evaluated at the demonstration and monitoring sites are reviewed in this section to compare their performance and to summarize lessons learned during construction and monitoring. Where performance was poor and possible methods of improving the system were considered too costly, the system was considered unsuccessful in that it demonstrated the futility of further experimentation with that particular system, and the concept was rejected. Where performance was good, the system was considered successful. Where damage occurred but the evidence indicated that the structure might be modified to perform better, within reasonable cost limits, the system was considered partially successful, and suggestions are offered as to possible improvements and methods of decreasing the cost of construction. Where appropriate, suggestions are also made as to (a) limitations of use that might be imposed by specific site conditions, (b) circumstances under which the system should perform best, (c) need for further research and testing, and (d) criteria for future design of similar systems.

The basic information presented in Section II is repeated in this section only to the extent necessary to avoid frequent back-referencing. This section categorizes the various systems under five general headings: bulkheads and seawalls, revetments, breakwaters and sills, groins, and nonstructural devices. Table 3-1 provides a listing of the shore protection systems evaluated and their locations.

1. Bulkheads and Seawalls.

a. Hogwire Fence and Sandfilled Bags. This type of bulkhead was demonstrated at one site only--Basin Bayou, Florida. Half of the structure (Fig. 3-1) was built with ultraviolet-resistant bags, which failed when forced outward into the wire mesh by backfill pressure as the retained embankment slumped from saturation. Concurrently, the bags were undermined by toe scour and dropped downward while being held tightly against the fence. As a result, many bags were torn open, and many fenceposts tilted seaward. The other half of the structure (Fig. 3-2) was built with polyproplylene Advance Bags, which failed in about 6 months, mainly by decomposition of the bag material. No filter material was placed under or behind the bags in either half of the bulkhead, but the methods of failure indicated that this may not have been a contributing cause of structural failure. The demonstration proved rather conclusively that this type of construction cannot survive under conditions encountered at this site without major modification of construction methods. Moreover, the test results were not considered site-sensitive, because failure was attributed to inadequate structural design rather than to unusual site conditions. In fact, the wave characteristics were generally the same as or even less severe than those at many other sites, and the soil at Basin Bayou was no more friable that at most other sites.

The estimated low cost of this structure ($30 per foot) encourages a search for methods of improving the system to perform satisfactorily, considering soil properties and landforms existing at the site. Improvements might include the following:

Table 3-1. Locations and performance of systems investigated.

System	Location
Bulkheads and Seawalls	
Hogwire fence and sandbags	Basin Bayou, Fla.
Treated timber[1]	Oak Harbor, Wash.; Buckroe Beach, Va.; Folly Beach, S.C.
Untreated timber[2]	Oak Harbor, Wash.; Ashland, Wis.
Rubber tire and post[2]	Oak Harbor, Wash.
Rubber tire stack[2]	Port Wing, Wis.
Steel and timber[1]	Port Wing, Wis.
Longard tube[2]	Ashland, Wis.; Sanilac 11, Mich.
Concrete sheet pile[1]	Folly Beach, S.C.
Concrete and timber	Folly Beach, S.C.
Earth-filled concrete pipe[2]	Beach City, Tex.
Revetments	
Concrete blocks[2]	Fontainebleau, La.; Port Wing, Wis.; Stuart-Jensen Beach Causeways, Fla.; Holly Beach, La.; Little Girls Point, Mich.
Concrete rubble[2]	Alameda, Calif.; Shoreacres, Tex.
Stone riprap[1]	Folly Beach, S.C.; Muskegon, Mich.; Tawas Point, Mich.
Concrete slabs	Alameda, Calif.
Sandfilled bags	Alameda, Calif.
Sand-cement-filled bags[1]	Alameda, Calif.; Oak Harbor, Wash.
Fabric	Fontainebleau, La.; Alameda, Calif.
Tire and fabric	Fontainebleau, La.
Gabions[2]	Kotzebue, Alaska; Ninilchik, Alaska; Oak Harbor, Wash.
Steel fuel barrels[2]	Kotzebue, Alaska
Breakwaters and Sills	
Floating tires[2]	Pickering Beach, Del.; Stuart-Jensen Beach Causeways, Fla.
Tires on piles[1]	Fontainebleau, La.
Longard tubes[2]	Alameda, Calif.; Basin Bayou, Fla.
Sandfilled bags	Basin Bayou, Fla.; Kitts Hummock, Del.; Slaughter Beach, Del.; Buckroe Beach, Va.
Sand-cement-filled bags[1]	Fontainebleau, La.; Alameda, Calif.
Timber sheet piles[1]	Slaughter Beach, Del.
Gabions[2]	Geneva State Park, Ohio.
Z-wall[2]	Geneva State Park, Ohio
Sta-pods	Geneva State Park, Ohio
Surgebreaker[2]	Basin Bayou, Fla.
Sandgrabber[2]	Basin Bayou, Fla.; Folly Beach, S.C.; Kualoa, Hawaii; Bellows Beach, Hawaii
Stone rubble[1]	Kitts Hummock, Del.; Siuslaw River, Oreg.
Concrete boxes[2]	Kitts Hummock, Del.; Slaughter Beach, Del.
Brush dike	Fontainebleau, La.

Table 3-1. Locations and performance of systems investigated--Continued.

System	Location
<u>Groins</u>	
Sandfilled bags	Alameda, Calif.; Bowers, Del.; Kotzebue, Alaska; Sanilac 26, Mich.
Sand-cement-filled bags[1]	Alameda, Calif.
Gabions[2]	Kotzebue, Alaska; Ninilchik, Alaska; Sanilac 26, Mich.
Steel fuel barrels[2]	Kotzebue, Alaska
Stone rubble[1]	Siuslaw River, [illegible]
Longard tubes[2]	Ashland, Wis.; Lincoln Township, Mich.; Sanilac 26, Mich.
Timber[1]	Broadkill Beach, Del.; Ninilchik, Alaska; Buckroe Beach, Va.; Lincoln Township, Mich.
Timber and rock[1]	Folly Beach, S.C.; Sanilac 26, Mich.
Rock asphalt mastic[1]	Sanilac 26, Mich.
Concrete rubble[1]	Broadkill Beach, Del.
Corrugated metal pipe[1]	Ninilchik, Alaska
<u>Nonstructural Systems</u>	
Perched beach[1]	Alameda, Calif.; Slaughter Beach, Del.
Artificial seaweed (not monitored)	Roanoke Island, N.C.
Beach fill[2]	Alameda, Calif.; Bowers, Del.; Broadkill Beach, Del.; Lewes, Del.; Muskegon, Mich.; Sunnyside, Wash.
Vegetation alone[2]	Pickering Beach, Del.; Slaughter Beach, Del.; Kitts Hummock, Del.; Fontainebleau, La.; Basin Bayou, Fla.; Roanoke Island, N.C.; Duck, N.C.; Hampton Refuge, Va.; Key West, Fla.; Uncle Henry's, N.C.; Alameda, Calif.; Stuart-Jensen Beach Causeways, Fla.
Vegetation with structure[2]	Alameda, Calif.; Basin Bayou, Fla.; Geneva Park, Ohio; Oak Harbor, Wash.; Key West, Fla.; Roanoke Island, N.C.; Bogue Sound, N.C.; Stuart-Jensen Beach Causeways, Fla.

[1]Systems that proved successful.

[2]Systems that could be made successful with minor changes or that should be used only in special environments or circumstances.

Note.--Unmarked systems are those that failed structurally or functionally.

Figure 3-1. Failure of ultraviolet-resistant sandbag bulkhead at Basin Bayou, Florida, 20 August 1979.

Figure 3-2. Failure of polypropylene sandbags at Basin Bayou, Florida, bulkhead, 20 August 1979.

(1) Use of PVC-coated wire mesh or smaller mesh size to reduce bag cutting due to pressure and sliding;

(2) use of tiebacks or deeper post embedment to prevent tilting due to backfill pressure;

(3) deeper entrenchment of the bottom bags to reduce the damaging effects of toe scour;

(4) battering the seaward face back to reduce outward pressure of bags against hogwire;

(5) filling at least the exterior bags with concrete grout and using less costly (burlap) bags;

(6) inserting plastic pipe weep holes near the toe to relieve pore pressure; and

(7) placing stone riprap at the toe of the bulkhead.

b. Treated Timber. Bulkheads constructed of treated timber were monitored at three sites--Oak Harbor, Washington; Buckroe Beach, Virginia; and Folly Beach, South Carolina. At Oak Harbor, the creosote-treated timber was Douglas fir (*Pseudotsuga menzezii*) (Fig. 3-3). Three-inch-thick planks were spiked horizontally to 8- by 8-inch posts compacted in 4-foot-deep holes and tied back to logs buried in the backfill. The planking was backed by filter cloth over half of the structure, and the silty sand backfill was placed directly against the planks in the other half. Displacement of toe protection, due to scour during a severe storm, exposed a 1- to 2-inch gap below the bottom planks in the filterless segment, allowing all of the backfill to escape. The planks then began to loosen. Although some backfill was lost where filter cloth was used, the loss was attributed to failure of the bulkhead that separated the test sections rather than the structure being tested. At a cost of $69 per linear foot, the treated-timber bulkhead with filter cloth was considered a satisfactory low-cost shore protection device. The loss of backfill from the filterless section demonstrated that a bulkhead of this construction should be provided with an adequate filter and adequate toe protection. The separation bulkhead which protected the flank of the filter cloth-backed bulkhead was repaired, and the section was restored to its original condition. It survived the remainder of the monitoring period (2 years total exposure) with no further damage.

Little information was available concerning the bulkhead at Buckroe Beach, which is a timber sheet-pile structure protecting the seaward side of the coastal road (Fig. 3-4). It appears to have had a creosote treatment and is still in good condition after several years of exposure. However, the groins along Buckroe Beach have maintained a fair width of protective berm, and the bulkhead has not been subjected to direct wave impact. Nevertheless, it appears to be structurally as adequate as the Oak Harbor treated-timber bulkhead, and its sheet-pile construction, extending deeply into the subtrate, provides better protection against toe scour. Sheet-pile construction is usually more costly than post and siding construction, and the latter may be selected for low-cost protection by most shore-front owners in lieu of sheet piling.

A treated-timber sheet-pile bulkhead at Folly Beach performed well until overtopping waves during Hurricane David washed out most of the backfill in September 1979. The bulkhead was not damaged, and the backfill was replaced. The structure was still in good condition in May 1980.

Figure 3-3. Gap below bottom planks of treated-timber bulkhead (filterless) at Oak Harbor, Washington, 5 April 1979.

Figure 3-4. Bulkhead protecting coast road still in good condition, at Buckroe Beach, Virginia, 21 March 1980.

The experience with treated-timber bulkheads at Oak Harbor, Buckroe Beach, and Folly Beach generally reflects the history of treated-timber bulkheads in the United States. Where properly designed for site conditions, they have performed exceptionally well. Failures have been the result mainly of lack of effective filter material, weak connecting devices, insufficient timber thickness for the environment to which the bulkhead is exposed, and overtopping by storm and hurricane waves. One objection to the coal-tar creosote treatment is that it tends to bleed for several years and is unpleasant to work with. Once constructed, however, a creosote-treated timber bulkhead tends to discourage vandalism; few bulkheads of this construction have been destroyed by vandals or would-be timber salvagers. Such a structure can be destroyed by fire, but the oily smoke that results from burning creosote generally discourages such attempts. Other wood treatments have preserved timber structures for 10 or more years, but none has come close to being as effective as creosote in the shore zone, especially in a saltwater environment. Many creosoted timber bulkheads have been known to last 20 or 30 years before deterioration has reduced their effectiveness.

c. Untreated Timber. Bulkheads constructed of untreated timber were monitored at two sites--Oak Harbor, Washington, and Ashland, Wisconsin. At Oak Harbor, Douglas fir logs cut to post length were compacted in 4-foot-deep holes, and full-length logs were attached horizontally to the landward side of the posts. Half of the structure was backfilled without a filter, and a gravel filter was used in the other half. A single storm removed all backfill from both halves of the structure within a few hours, and waves knocked the logs loose from the posts, resulting in complete destruction of the bulkhead (Fig. 3-5). The monitor was not able to determine whether the majority of the backfill was washed through cracks between the logs or through toe scour holes that developed under the structure. The demonstration proved the futility of attempting to retain fill behind the log structure without a better filtering system and adequate toe protection. The use of filter cloth properly placed behind the logs and under the backfill might have prevented loss of backfill material. In fact, the structure appeared to be sturdy enough to withstand considerable direct wave impact as long as it remained backed by fill material. At a cost of $47 per linear foot, this would appear to be a good low-cost bulkhead if leakage could be prevented. More experimentation is needed with this type of construction before it can be recommended for general use. Meanwhile, the user is advised to devise his own filtering and toe scour prevention system and to use bulkheads constructed of logs at his own risk. The low cost may be a reflection of the location of Oak Harbor in the heart of the northwest logging area. Elsewhere, the logs could be much more expensive and other methods of shore protection might be more economical.

At Ashland, untreated logs set upright side-by-side in a trench and compacted in place, then joined by a single top wale, were used to construct four crib-type bulkheads designed to protect the exposed ends of Longard-tube seawalls. These bulkheads were partially wrapped around the ends of the tubes, and the space between the tube and the bulkhead was backfilled with stones and cobbles. After a series of storms, all of the backfill

Figure 3-5. Logs knocked loose from posts of untreated-timber bulkhead (device 4aW) at Oak Harbor, Washington, after loss of backfill, 5 April 1979.

material was washed out from behind each bulkhead, and the posts, not having tiebacks, began to lean precariously (Fig. 3-6). This experience was much the same as at Oak Harbor, adding to the evidence that round-timber bulkheads are of little value without an effective filter system to prevent backfill washout.

d. Rubber Tire and Timber Pile. This type of bulkhead was demonstrated at one site only--Oak Harbor, Washington. Two rows of 8-by 8-inch treated posts were set in staggered fashion in drilled holes and compacted in place. The tires were then strung over the posts and adjusted to abut each other, then filled with gravel. The posts were then tied back to logs which were to be buried in the backfill. Filter cloth was used in one-fourth of the section (device 3aW) and gravel filter was used in another one-fourth (device 3bE). The remaining one-half was left without any filter other than the sand and gravel backfill that was placed behind both sections (devices 3aE and 3bW).

Under wave attack, the gravel soon washed from the tires, allowing the tires to collapse, and resulting in a settlement of the tires (Fig. 3-7). No repairs were made to the system. Even though the tires were donated, the cost per foot was $58 for one section and $88 for the other section, the difference resulting from the greater amount of drilling time required to drill the post holes in the more expensive section, which is underlain

Figure 3-6. Posts leaning after backfill material washed out from behind crib-type bulkhead at Ashland, Wisconsin, 29 November 1979.

Figure 3-7. Collapse of tires of device 3aE (rubber-tire and timber-pile bulkhead) after gravel washed out from tires, Oak Harbor, Washington, 23 April 1980.

with a highly resistant layer of compacted glacial till. The Oak Harbor bulkhead of treated posts and planks backed with filter cloth, which survived the monitoring period well and cost about the same, was considered a superior system. The tire and pile bulkhead functioned nearly as well as the filter-backed treated-timber bulkhead at Oak Harbor.

e. Rubber-Tire Stack. This device was demonstrated at one site only--Port Wing, Wisconsin. It consisted entirely of scrap tires laid horizontally, with the tires staggered so that each tire rested on three below with the seaward side of the device sloped back toward the top. To provide stabilizing weight, each tire was filled with granular material. The tires were fastened together with galvanized-steel spikes and pushnuts, which greatly speeded the assembly process compaired to bolted connections. Preconstruction separation tests indicated that the spikes and pushnuts probably had sufficient holding power. In practice, this did not prove to be the case. Within 1 month, some of the tires were separated by wave action, and some of the granular fill was washed out from inside the tires (Fig. 3-8). The lightened load against the filter-cloth backing then failed to hold the filter cloth in place, and progressive failure of the retained embankment ensued during the remainder of the monitoring period.

Figure 3-8. Tires pulled away from rubber-tire stack revetment, and granular fill washed out from inside tires, Port Wing, Wisconsin, 5 September 1979.

The structure has deteriorated, but is still reducing the rate of erosion. An inexpensive, easy-to-use fastening device is needed, which will prevent the tires from being pulled apart under design wave conditions. Filling the tires with concrete grout might help the stability problem, but the all-tire bulkhead would still be esthetically displeasing.

No further experimentation with the rubber-tire stack bulkhead is recommended and the concept should be rejected.

f. Steel and Timber. This type of bulkhead was demonstrated at one site only--Port Wing, Wisconsin. It involved a row of steel H-piles set vertically into sandstone bedrock, with railroad ties secured between the piles. The toe of the structure was protected with graded riprap. The landward face of the bulkhead was lined with filter cloth and backfilled with granular material. The structure survived the monitoring period with no apparent damage (Fig. 3-9). However, it may be susceptible to toe scour causing settlement of the ties and loss of elevation. If ties settle unevenly, voids would open, the filter cloth may tear, and the backfill would be lost. Because of the size of the H-piles used, the care required to achieve proper spacing, and the thickness of timber sheathing provided by the railroad ties, the cost of the steel and timber bulkhead was $182 per linear foot, well in excess of what could be considered low-cost protection. Nevertheless, where adequate funding is available and no less costly alternatives are available that could provide the same degree of protection, this type of bulkhead appears to fulfill an important need.

g. Longard Tubes. Bulkheads of single 69-inch Longard tubes were evaluated at Ashland, Wisconsin, and Sanilac Section 11, Michigan. A bulkhead comprising a 40-inch tube resting on a 69-inch tube was also evaluated in one area at Ashland. For each bulkhead, a filter-cloth apron with a 10-inch tube attached was used to support the large tube or tubes and to prevent undermining due to toe scour. In each case, the tube tended to roll or be pushed lakeward as a result of the instability of the bluff behind and above the tube, and not because the tube failed to function as a bulkhead. The rounded shape of the tube lessens the ability of the tube to resist rolling due to backfill pressure or bluff slumping (Fig. 3-10). Perhaps this could be avoided by entrenching the bottom of the tube a foot or so below the sandline so that the tube would have to be lifted or rolled up over the lip before it could be moved lakeward. This was not done, however, and the effectiveness of this entrenchment can only be surmised.

Several tears and punctures were noted in the outer fabric and inner liner of the tubes, allowing some sand to spill out (Fig. 3-11). Some of the damage was obviously the result of shotgun blasts, but in other areas it could not be determined whether the tears were due to wave-borne debris impact or to vandalism. However, the tears or punctures that were in epoxy-coated areas did not seem to enlarge with continued exposure. Where the tube rolled and exposed uncoated areas that were then punctured, the tough outer fabric continued to unbraid, the holes became larger, and more sand was lost as exposure to wave action continued.

Where the sandfill can be taken from the existing beach or from a nearby deposit without the added costs of royalties or import charges, the tubes can provide a relatively low-cost bulkhead, but one of limited

Figure 3-9. Successful but costly steel and timber bulkhead at Port Wing, Wisconsin, 5 September 1979.

Figure 3-10. Longard tubes tend to roll lakeward from unstable bluffs backing bulkhead, Ashland, Wisconsin, 26 September 1979.

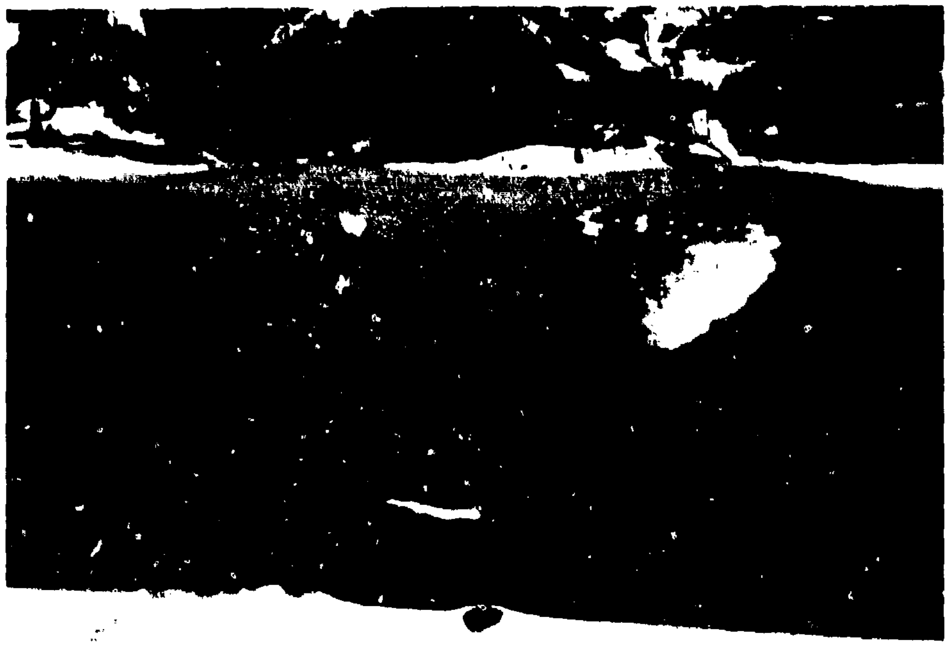


Figure 3-11. Longard tubes are subject to tears and punctures, Ashland, Wisconsin, 27 December 1979.

height. Such a bulkhead will not retain a steep bank, but if the bluff slope is dressed to a stable angle and protected with a good vegetative cover, the tube may be effective in preventing erosion of the bluff toe. However, the tube should be set far enough seaward of the bluff to permit the natural accretion of a back berm on which overtopping waves are attenuated. If set directly against the bluff, overtopping waves may scour the bluff toe and trigger slope failure. The rapidity with which the tubes can be laid out and filled makes them good candidates for emergency installations. Their successful performance, however, will depend on (1) proper entrenchment against undercutting and rolling, (2) sand-epoxy coating all exposed surfaces, (3) control of vandalism, (4) proper drainage of surface and ground water, (5) protection from undermining, and (6) provision of space for a backberm to form between the bluff and the tube.

h. Concrete Sheet Pile. Concrete bulkheads are normally too expensive to be classified as low-cost protection devices; however, one structure of this type was monitored at Folly Beach, South Carolina. It involved a 400-foot-long wall comprised of concrete slabs jetted into the beach sand and tied together at the top with a poured-in-place reinforced concrete cap. The bulkhead was constructed in the mid-1970's by the South Carolina Department of Highways. During Hurricane David in September 1979 the northeast corner of the wall at the shore return separated about 2 inches at a panel point, allowing a piece of the concrete cap to spall (Fig. 3-12); otherwise no damage occurred.

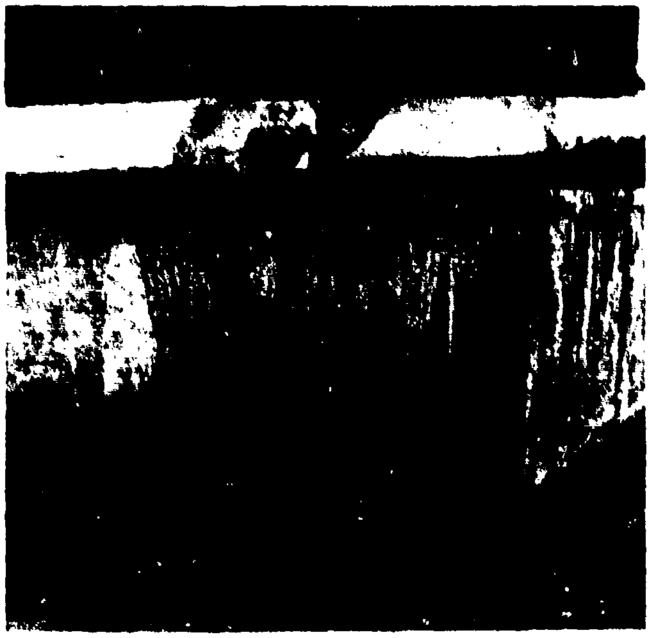


Figure 3-12. In area 2, the concrete sheet-pile bulkhead separated at a panel point, allowing a piece of the concrete cap to spall, Folly Beach, South Carolina, 25 January 1980.

As might be expected, this structure proved to be about as trouble-free a bulkhead as is possible to build. Its sheer mass and relatively indestructible nature make it an excellent shore protection device. However, the cost factor probably places it beyond economic reach of most owners of shorefront property.

Where concrete sheet piling is selected for bulkhead construction notwithstanding its high cost, good performance is best assured by incorporating the following provisions in its construction:

(1) Provide some type of tongue-and-groove edging for the concrete sheets, and set the sheets in a manner that assures sand-tight junctions between sheets.

(2) Provide weep holes in the sheets at 5- to 10-foot intervals along the wall at the proper elevation to relieve pore pressure in the backfill. Provide adequate filters for weep holes to prevent wave action from pumping backfill through them.

(3) Provide an adequate poured-in-place cap to distribute lateral loads due to backfill pressure uniformly along the wall, including tieback stresses if tiebacks are used. Anchors for tiebacks must be beyond slip-circle distance from the wall.

(4) Keep all reinforcing steel at least 3 inches away from any concrete surface, and thoroughly encase tieback steel in a mastic wrap-coating of approved design.

(5) Set the sheets deep enough to prevent wave action from pumping liquified backfill out from under them. This usually requires a toe depth of at least 5 feet below the anticipated maximum scour depth at the face of the wall.

(6) If cantilevered construction is used instead of tiebacks, assure adequate depth of penetration to prevent overturning of the wall due to backfill pressure. Also, assure an adequate thickness and reinforcement to prevent failure of the sheets in cantilever bending.

i. Concrete Block and Timber. A 90-foot bulkhead evaluated only at Folly Beach, South Carolina, was apparently constructed by driving or jetting in a row of untreated-timber posts and then forming a succession of vertical slabs to fill the gaps between the posts. However, the bonds between posts and slabs were so poor that the slabs had no support from the post and were pushed over (seaward) by backfill pressure, probably aided by saturation of the backfill material as a result of wave overtopping (Fig. 3-13). Wave action then progressively removed the backfill, leaving the line of posts and toppled slabs near the center of the newly developing beach profile.

The obvious lesson learned from this experience at Folly Beach was that the concrete-block and timber bulkhead is a very poor shore protection device. With so many other better methods available, no attempt should be made to improve on this basically inadequate design. The concept should be rejected.

j. Earth-Filled Concrete Pipe. At Beach City, Texas, a bulkhead was constructed with various-sized concrete pipes standing on end, side-by-side, and filled with granular soil. Initially, the pipes were not bound together, and the stability of each pipe section against overturning depended on its own base support plus some frictional support afforded by abutting pipes on either side. As a result of backfill pressure, some pipes leaned precariously seaward, resulting in gaps between adjacent pipes (Fig. 3-14). Early in the Corps monitoring program, Chambers County straightened all the pipes, grouted the cracks between adjacent pipes, and restored the backfill material (Fig. 3-15). The pipes were refilled with soil and capped with 2 to 4 inches of concrete.

During the next few months of the monitoring period, the repaired pipe bulkhead performed well and showed no further signs of deterioration. The time period was too short, however, to indicate the true longevity of this device. Its application will be limited to locations where large quantities of used concrete pipe have been salvaged. The use of new pipe would be too costly for low-cost bulkhead construction at any location. In locations where used concrete pipe is available for bulkhead construction, the following precautions are suggested:

Figure 3-13. Slabs pushed seaward by backfill pressure along the concrete-block and timber bulkhead in area 3, Folly Beach, South Carolina, 25 January 1980.

Figure 3-14. The pipes began to lean seaward due to backfill pressure along the concrete-pipe bulkhead at McCollum Park, Beach City, Texas, 28 September 1979.

Figure 3-15. Grout used to close the cracks between pipes and fill was replaced in the concrete-pipe bulkhead, McCollum Park, Beach City, Texas, 21 December 1979.

(1) Set the bottom of the pipes below the anticipated maximum toe scour depth.

(2) If the top of the wall is more than two pipe diameters above the sandline at the toe, some type of tieback system should be used in conjunction with a cap beam designed to distribute lateral loading.

(3) Place a woven filter cloth behind the pipes before backfilling, forcing the cloth deeply into the grooves between pipes to avoid ballooning of the cloth to the burst point. This is less costly than sealing the joints with concrete grout, assures relief of pore pressure, and allows for some differential settlement of the pipes.

(4) Seal the tops of the pipe fills with about 2 inches of concrete grout to prevent washout by overtopping waves (Fig. 3-16).

k. Profiles.

(1) Treated Timber. Profiles were compared at two sites where treated-timber bulkheads were monitored--Buckroe Beach, Virginia, and Folly Beach, South Carolina (Fig. 3-17) The bulkhead at Buckroe Beach was a timber sheet-pile structure which remained structurally sound throughout the monitoring period. This structure acted in conjunction with groins which maintained a protective beach berm that prevented waves from impinging directly on the bulkhead. The profiles show that the beach in front of the

Figure 3-16. Concrete placed on top of and behind the pipe structure, McCollum Park, Beach City, Texas, 26 February 1980.

bulkhead progressively accreted. This accretion is primarily due to the effect of the groins.

At Folly Beach, the bulkhead was also of timber sheet-pile construction, and it also remained structurally sound throughout the monitoring period. It performed well until September 1979 when hurricane waves overtopped and washed large quantities of material from behind the structure. The profiles show the material lost in September 1979, as well as the material replaced by January 1980.

(2) Rubber Tire and Pile, Rubber Tire Stack, and Hogwire Fence and Sandbags. The profiles for these structures are shown in Figure 3-18. The rubber tire and pile bulkhead at Oak Harbor, Washington, despite being placed high in the tidal zone, was damaged by wave attack allowing the tires to collapse and settle and some of the gravel washed out. This damage did not seriously reduce its functional performance, as the profiles show that little material was eroded from behind the structure.

The rubber-tire stack bulkhead at Port Wing, Wisconsin, was progressively damaged by wave action. Despite the damage and the inability of this structure to sustain under wave attack, the profiles indicate that the structure did protect the upper bank from significant erosion.

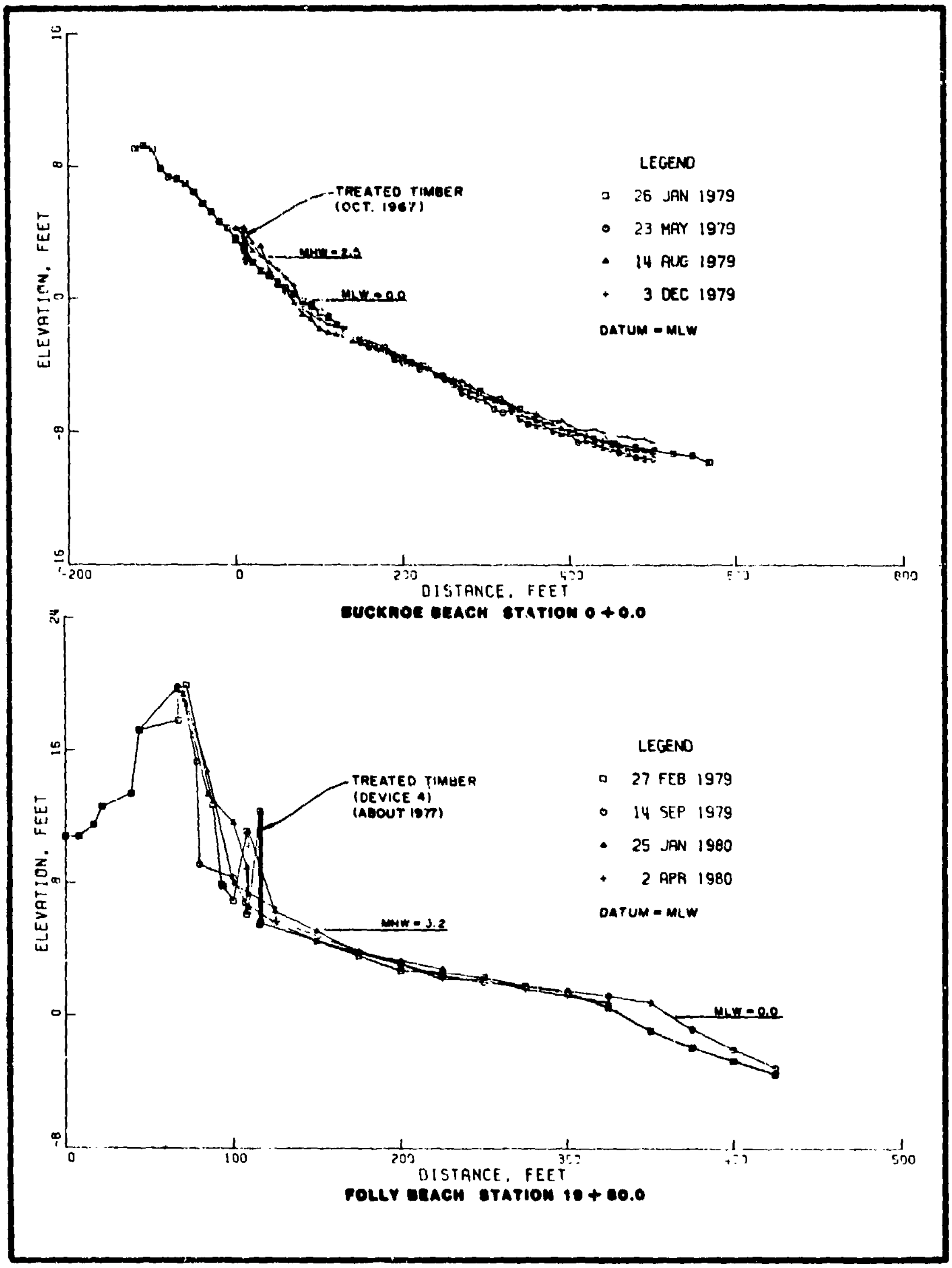


Figure 3-17. Profiles of treated-timber bulkheads.

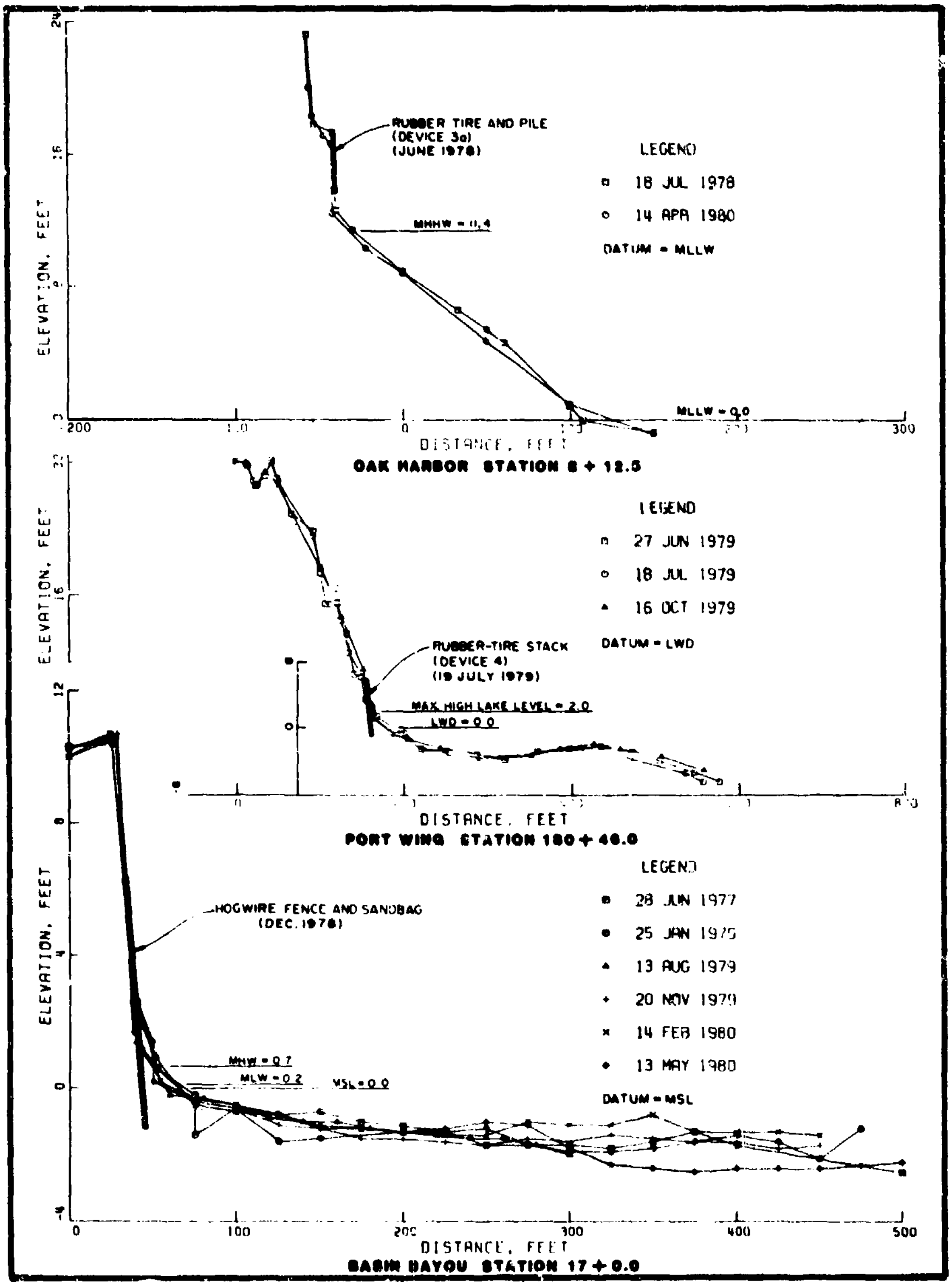


Figure 3-18. Profiles of rubber tire and pile, rubber-tire stack, and hogwire fence and sandbag bulkheads.

The hogwire fence and sandbag bulkhead at Basin Bayou, Florida, was destroyed by Hurricane Frederic in September 1979 and the bank behind the structure was severely eroded. The erosion is not shown by the profiles because replacement fill was added before the profile was surveyed again. Without structural protection, another episode of high waves would surely cause measurable bluff erosion.

(3) Untreated Timber, Steel and Timber, and Concrete Sheet Pile. The untreated-timber bulkhead at Oak Harbor, Washington, was completely destroyed by wave action soon after its construction, and fill material behind the structure was removed. Figure 3-19 shows that although the structure was damaged and fill was lost, serious erosion of the bank behind it did not occur.

The steel and timber bulkhead at Port Wing, Wisconsin, suffered no damage during the monitoring period, and the profiles show that the structure adequately protected the bank, as no erosion was evident. The upper-bank aberrations in the profiles are due to grading work in progress. The bank was seeded in late July 1979.

The concrete sheet-pile bulkhead at Folly Beach, South Carolina, suffered some damage during Hurricane David in September 1979, and material behind the structure was removed by wave action, as shown by the profiles. New fill was later placed behind the structure, as shown by the April 1980 profile.

(4) Longard Tube and Earth-Filled Concrete Pipe. At Ashland, Wisconsin, the profiles are at the west seawall (device 9). Although this structure was damaged by debris and vandals it did not demolish the tube. In addition, the western end of the tube rolled slighly lakeward. The structure otherwise remained intact throughout the monitoring period. Figure 3-20 shows that some erosion occurred behind the structure, probably due to overtopping waves.

At Sanilac Section 11, Michigan, the Longard-tube bulkhead was placed in the middle of the tidal zone. This structure was moved lakeward, damaged, and eventually almost completely buried during the monitoring period.

The earth-filled concrete-pipe bulkhead at Beach City, Texas, was damaged in July 1979, and the bank behind the structure was eroded. The structure was rebuilt in November 1979 and new fill was placed behind it. The 6-month survey period was not long enough to determine the rebuilt structure's performance. The first survey shows the erosion caused by the storm; the second survey shows the profile of the rebuilt section.

(5) General. Bulkheads and seawalls are not intended to change the physiographic characteristics of the beach and offshore area, but to protect the upland from further erosion. The profiles show that most of the structures were effective in that respect. The profiles show little evidence of toe scour, and changes in the offshore area at each site were probably seasonal variations in the littoral regime.

2. Revetments.

a. Artificial Concrete Blocks. Various shapes and sizes of concrete blocks (Fig. 3-21) were used as revetments at four sites--Holly Beach and

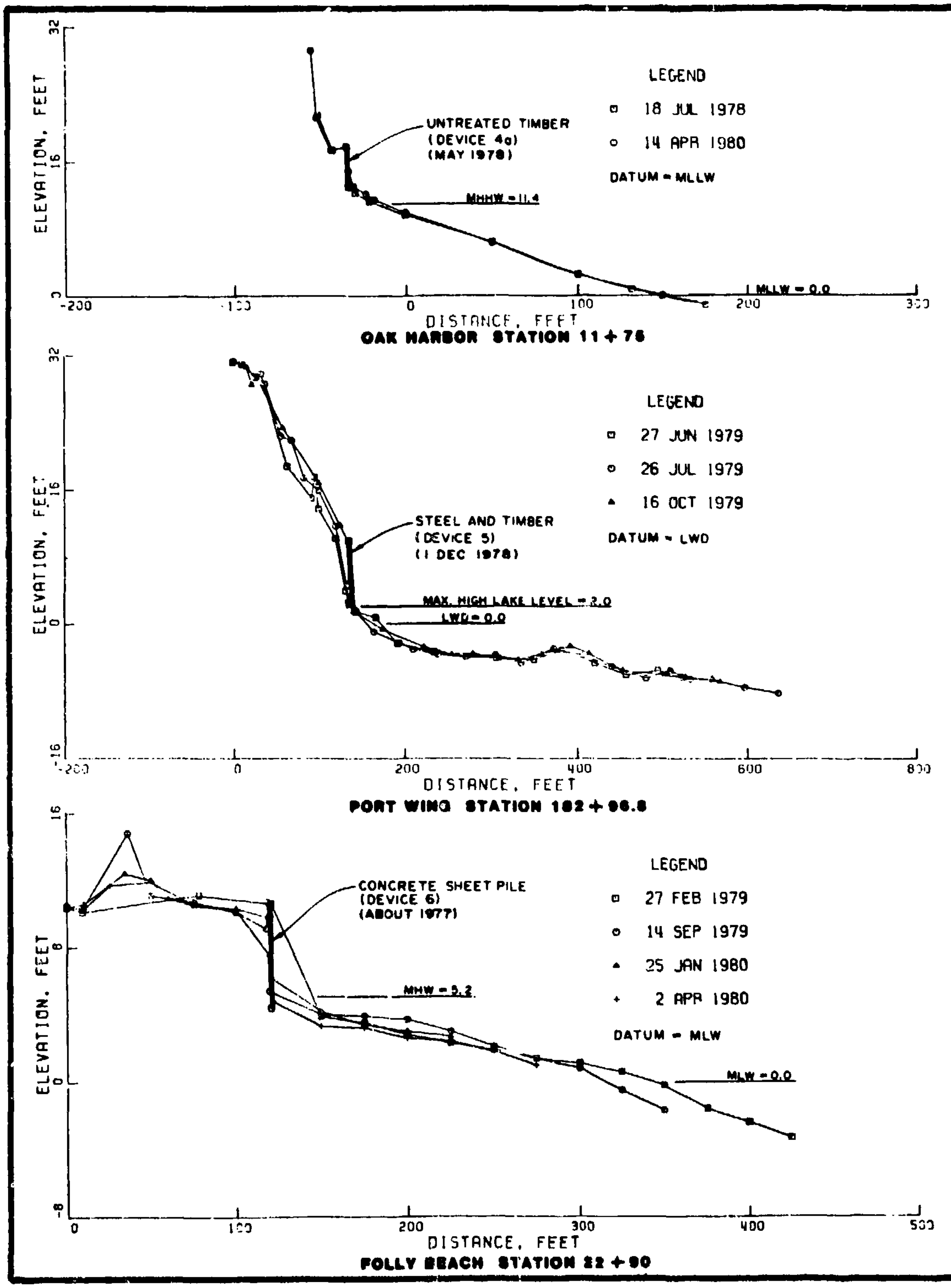


Figure 3-19. Profiles of untreated timber, steel and timber, and concrete sheet-pile bulkheads.

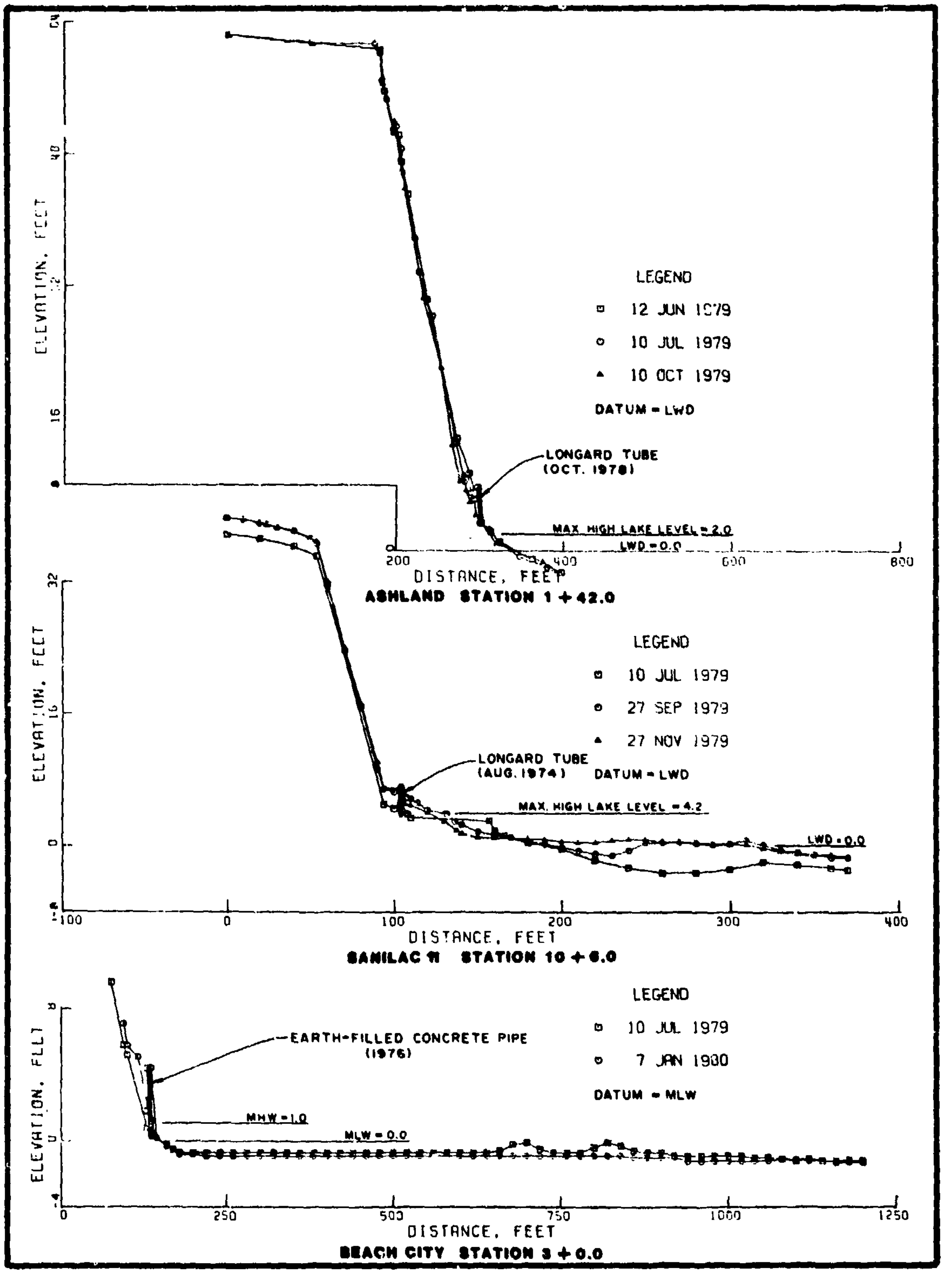


Figure 3-20. Profiles of Longard-tube and earth-filled concrete-pipe bulkheads.

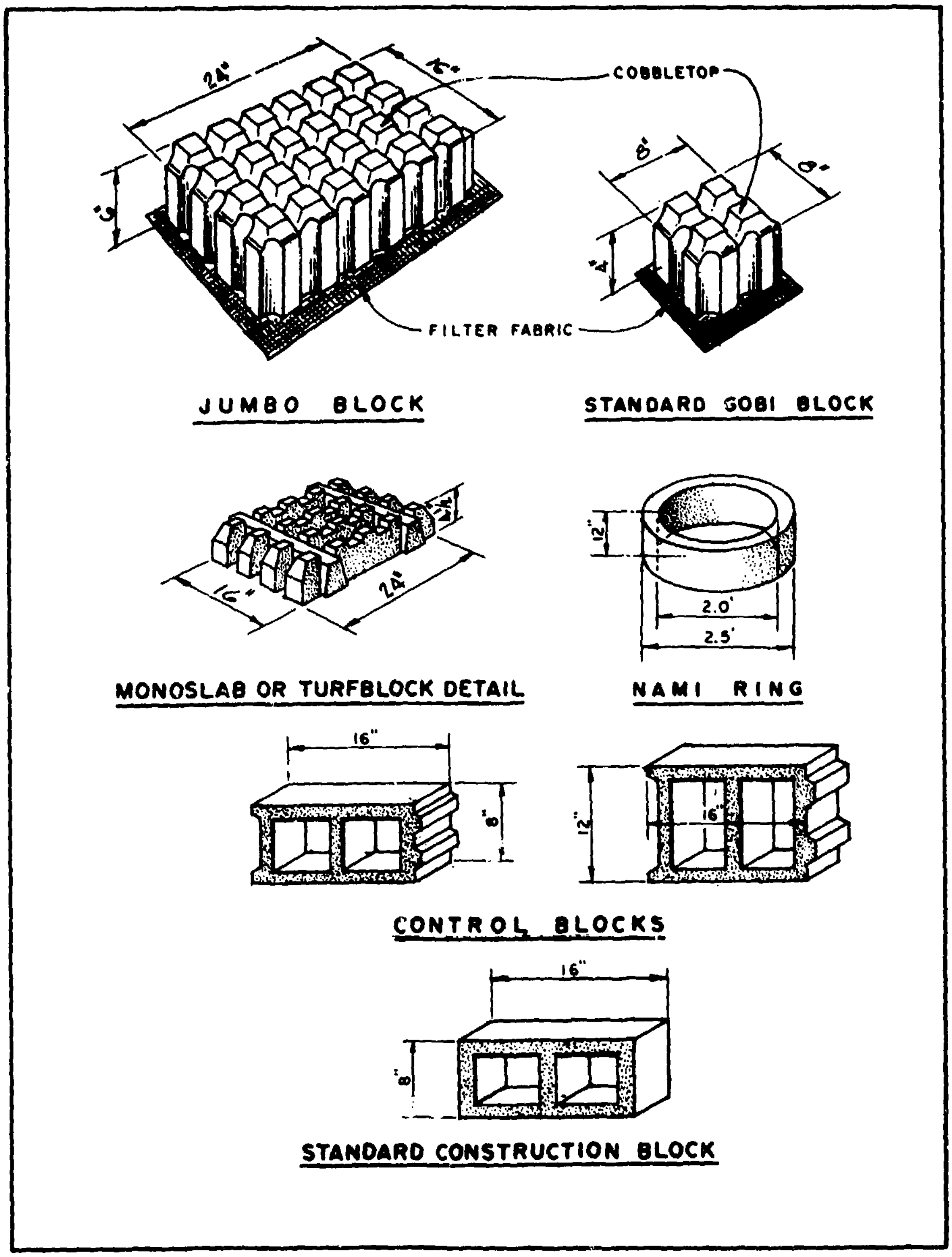


Figure 3-21. Types of concrete revetment blocks.

Fontainebleau State Park, Louisiana; Port Wing, Wisconsin; and Little Girls Point, Michigan.

(1) Gobi Blocks. Gobi blocks (Erco blocks) were used at the Holly Beach monitoring site on the gulf coast and the Fontainebleau State Park demonstration site on Lake Pontchartrain. These blocks come in several sizes; those demonstrated were the smaller 13-pound module and the larger 115-pound module. The latter is called a Jumbo block. The modules are designed to be hand-placed on filter cloth or are factory-glued to carrier strips of filter cloth; in the latter case they are called Gobi-mats or Jumbo mats, depending on the size of module used. If the blocks are glued to both sides of the carrier strip back-to-back, they are called double Gobi-mats or double Jumbo mats.

At Holly Beach, Gobi blocks were first hand-placed by the Louisiana Department of Highways in a test revetment section. Two types of filter cloth were used--one with mesh openings just larger than the median grain size of the embankment material, and the other with mesh openings over twice the median grain size. The revetment on filter cloth with the smaller openings failed as a result of filter clogging, pore-pressure buildup, and bank sloughing. The revetment on filter cloth with the larger mesh openings survived waves 3 to 4 feet high. In 1970, the Louisiana Department of Highways built a 3-mile-long Gobi-block revetment, with a 1 on 3 slope, on the type of filter cloth that had proved successful. This revetment, designed to protect an endangered reach of coast highway, survived 3 years with little damage; in 1973, however, hurricane waves damaged about 10 percent of the structure. In 1976, the revetment was rebuilt, with some alterations, and extended 1 mile; this 4-mile project survived nearly 3 years before being damaged by high waves and overtopping by a tropical storm in September 1979, while being monitored under the demonstration program. Most of the damage was initiated by uneven foundation settlement at the junction of the hardened highway shoulder with the graded embankment extending downslope (Fig. 3-22). This highlighted the need for subgrade uniformity in this type of revetment.

At Fontainebleau, revetment sections were constructed in 1979 with Gobi blocks, Gobi-mats, double Gobi-mats, and Jumbo mats, all on a 1 on 3 lakeward slope and a 1 on 2 backslope with crowns rounded over at +4.5 feet NGVD to avoid abrupt slope changes. The sections were designed for overtopping at high lake levels. Toes were extended down to mean sea level, terminating in various types of Dutch-toe structures, and backslopes were carried down to +1.5 feet NGVD, terminating in various anchorages. Toes and back anchors performed satisfactorily during a storm of near-hurricane intensity, with waves more than 3 feet high in April 1980. Design of toes and back anchors appears to depend on availability of materials; however, good judgment as to depth of the embedment and method of attachment to the main revetment is important. The prior theft of some units of the Gobi-block revetment may have led to loss of additional units during the storm, but the system suffered no major damage. None of the mat-type revetments were damaged, and are preferred to hand-placed revetments. The Gobi-block system proved generally successful under assumed design conditions, but careful preparation of the slope, uniformity of subgrade materials, and adequate toe and shoulder anchorage are essential to good performance.

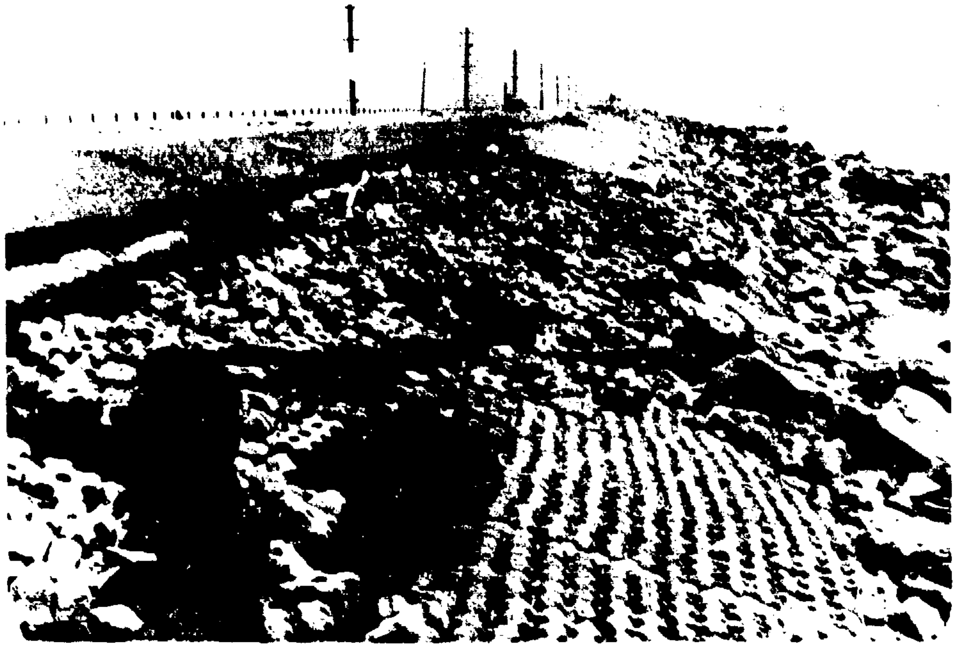


Figure 3-22. Gobi blocks displaced during a 1979 storm at Holly Beach, Louisiana, 16 April 1980.

(2) Turfblocks. Turfblocks (Monoslabs) were used only at the Port Wing, Wisconsin, demonstration site in a revetment constructed in November 1978. The modules were placed over nonwoven filter cloth on a uncompacted granular fill containing large stones graded to a 1 on 3 slope. A few days after installation, the revetment was overtopped and badly damaged by storm waves on Lake Superior. After the storm, the modules were found to have been irregularly displaced and were covered with debris but not broken; the filter cloth was exposed in many places and the embankment slope was irregular, with evidence of sloughing in some areas. During the next 18 months, the revetment sustained little further damage, but displaced modules in the upper levels allowed high waves to erode the upper bank in a few places, and exposed surfaces of modules in the lower levels were severely abraded by wave-borne debris and cobbles (Fig. 3-23). Exposed areas of filter cloth had deteriorated badly. Poor subgrade preparation and possible filter cloth clogging was largely responsible for module displacement, and the abrasion due to cobble bombardment was not necessarily the result of defective concrete in the modules. Therefore, this one demonstration was not considered a fair test of the Turfblock revetment system. With proper installation in a less severe environment, there appears to be no reason why the system should not perform about as well as Jumbo blocks.

(3) Control Blocks. Rectangular blocks similar to hollow masonry units generally used in construction were used for revetments at Port Wing. Two widths of blocks were used. The blocks termed "control blocks" had protrusions cast into their ends to provide a tongue-and-groove

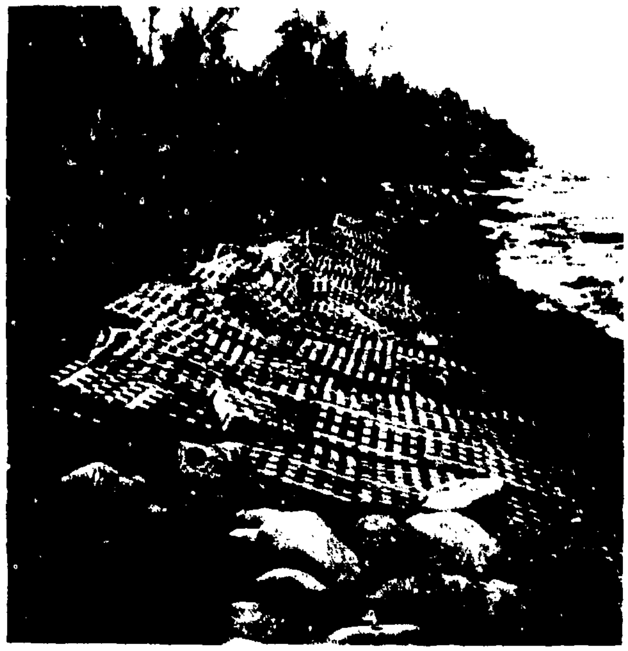


Figure 3-23. Displacement of Turfblocks, Port Wing, Wisconsin, 7 May 1979.

interlock. The control-block revetment was constructed in October 1978 of blocks laid on nonwoven filter cloth with their long axes parallel to the shoreline. The blocks settled irregularly on the uncompacted fill, probably because of the large stones it contained, and rows of blocks were misalined after the November 1978 storm that also damaged the Turfblock revetment. Because of their greater thickness and the interlocks, however, they were less susceptible to overturning by waves. Revetments of both block widths remained essentially intact during the next 18 months and continued to protect the upper bank (Fig. 3-24). Apparently, the width of the block did not affect its performance. As with the Turfblocks, however, exposed control blocks in the lower levels were severely abraded.

(4) Standard Building Blocks. At Fontainebleau, a revetment of standard building blocks (hollow masonry units) was constructed in November 1979 of blocks laid on woven filter cloth with their long axes perpendicular to the shoreline. A large number of these blocks were stolen from the crown of the structure shortly after construction, but the cloth protected the subgrade in this section (Fig. 3-25) until new blocks were placed over it early in April 1980. These replacement blocks were mistakenly laid with their long axes parallel to the shoreline, and most were displaced by overtopping waves during a storm on 13 April 1980 (Fig. 3-26). The remainder of the revetment remained essentially intact, except for one small area, where the blocks had also been mistakenly placed; three of these initially mislaid blocks were also displaced by the storm. The lessons learned at Fontainebleau were (1) that the standard building blocks should be laid with long axes

Figure 3-24. The control blocks holding very well at Port Wing, Wisconsin, 16 August 1979.

Figure 3-25. A large number of control blocks were stolen shortly after construction; however, the filter cloth protected the subgrade, Fontainebleau, Louisiana, 13 March 1980.

Figure 3-26. Most of the replacement blocks, mistakenly placed parallel to shoreline, were displaced during April storm, Fontainebleau, Louisiana, 12 June 1980.

perpendicular to the shoreline, and (2) that they should not be used in areas where their theft seems likely. Although they appeared to be less stable than control blocks, cost and availability are factors that may affect the choice of module type. Both types are thicker and blockier than Gobi blocks and Turfblocks, and therefore should be more stable in a revetment.

(5) Nami Rings. A Nami-ring revetment installed in 1974 at Little Girls Point, Michigan, was monitored briefly under the demonstration program. The rings were laid out on the existing foreshore slope instead of excavating the subgrade to a 1 on 2 slope as originally planned. Enough rings had been cast to carry the revetment from LWD to the estimated limit of uprush on a 1 on 2 slope. Actually, the rings were placed on the existing beach, which sloped about 1 on 10, and the top of the revetment was too low to prevent overtopping. Only half the revetment was placed on filter cloth; the rings in the lower part of the filter cloth-backed revetment section were tied together with steel tie rods. This half of the revetment performed well initially, except for wave overtopping, but eventually failed due to (1) slumping of the steep bluff behind the revetment due to scouring of the bluff toe above the low structure, and (2) upheaval of the subgrade due to excessive surcharge loading.

Failure occurred early in the half of the revetment where neither filter cloth nor tie rods were provided because of displacement of rings by undermining and subsequent ring breakage from wave action. Later, the lower part of the revetment was covered with a mantle of cobbles brought in by littoral processes. Then, wave-borne cobbles and debris broke the upper rings of the revetment which remained exposed but were not displaced.

The evaluation of Nami rings was considered inconclusive in regard to the capabilities of the system when properly installed. It did demonstrate that Nami rings as constructed should not be used where cobbles and debris exist in the littoral mantle. Further testing of the system under more favorable conditions is recommended. These conditions should include the following:

(1) Trim the bluff behind the revetment to a stable slope, as determined by soils analysis;

(2) place the revetment on a prepared 1 on 2 slope extending from the estimated limit of toe scour to the limit of waveuprush, as estimated for the design wave;

(3) use a woven filter cloth of proper opening size to retain the subgrade material;

(4) use tie rods throughout the section; and

(5) increase the resistance of the rings to breakage by increasing the wall thickness, the strength of the concrete, or both.

b. Concrete Rubble. Revetments constructed of concrete rubble were monitored at three sites--Shoreacres, Texas; Alameda, California; and Folly Beach, South Carolina. The Shoreacres installation was a monitoring site; the revetment consisted of 3,100 feet of park shoreline protected from erosion by concrete rubble dumped as a riprap along the base of the low shore bluff (Fig. 3-27). No filter material was used, but the rubble was broken into a good gradation of sizes. Apparently, the thickness of the rubble riprap was adequate to allow it to form its own filter of smaller pieces of rubble retained by the heavier pieces that remained as armor on the exposed slope; the finer pieces in the outer layers were washed out of the section by wave action. This type of construction wastes rubble material, and may not work well if the rubble does not have a good gradation of smaller sizes. In other areas where this type of riprap has been attempted, wave action has washed the retained embankment through the voids in the rubble, leaving a windrow of riprap material deteriorating in the wave zone while shore erosion continued unabated behind the structure. This did not happen at Shoreacres, probably for two reasons: (1) the exceptional thickness of the riprap, and (2) the relatively mild wave climate of the area.

At the Alameda demonstration site, the system used successfully at Shoreacres was attempted by the city of Alameda before implementation of the demonstration project; however, the reasons for the success of the system at Shoreacres apparently were not applied at Alameda. Wave action pumped the retained bank material throught the riprap, leaving the concrete rubble scattered over the beach, and erosion of the embankment continued. Five concrete rubble sections were then constructed for the demonstration program. A discussion of these five devices follows.

Figure 3-27. The concrete riprap is performing well at Shoreacres, Texas, 26 February 1980.

(1) Device 6 (75 feet). The same concrete rubble that had been used by the city was placed over Mirafi-14 filter cloth on a 60° embankment slope. The structure was destroyed by storm waves before it was completed and was later rebuilt on a much flatter slope. Pieces of rubble were placed in mosaic fashion, one slab thick, over the filter cloth. The new structure performed well for a short time, but was badly damaged in 1980.

(2) Device 7 (75 feet). Concrete rubble used by the city, but shape-sized (broken up) so that no piece was longer than three times its smallest dimension, was placed over Mirafi-140 filter cloth on a 60° embankment. This structure was also destroyed by storm waves before it was completed and was later rebuilt on a much flatter slope. The completed structure had almost the same appearance as that of device 6, which led to the conclusion that the shape-sizing (reduction of maximum dimension) had not been achieved in the final section. Determination of the effect of this lack of shape-sizing was rendered academic, however, because the structure was protected by a fillet of sand trapped by the device 8 sandbag sill (Fig. 3-28). Device 7 survived the monitoring period with little damage.

(3) Device 8 (75 feet). In addition to the sill which was built as a feature of this device to retain a perched beach 100 feet wide by 200 feet long, device 8 included a revetment indentical to that of device 7 which extended along the first 75 feet of bluff within the enclosure.

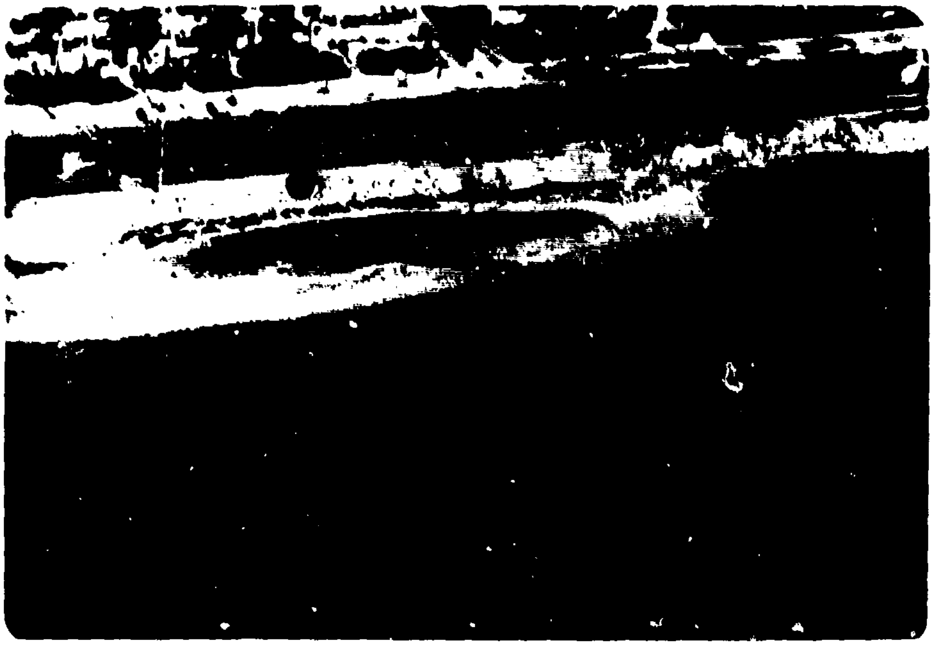


Figure 3-28. The low sill of device 8 provided protection to devices 7 to 10 at Alameda, California, 13 November 1979.

The revetment was completed after the storm that destroyed the initial efforts on devices 6 and 7. The perched beach had the same effect on the device 8 revetment as the sand fillet that protected device 7 (Fig. 3-29). Device 8 survived the monitoring period with virtually no damage.

(4) Device 9 (75 feet). This rubble revetment was constructed in the same manner and at the same time as the device 8 revetment but without shape-sized rubble (Fig. 3-28). Also fronted by a perched beach, device 9 survived the monitoring period with only minor damage.

(5) Device 10 (50 feet). This device consisted of dumped concrete rubble without filter material of any kind. It was placed in the same manner as the city's failing riprap for use as a control section. However, it also was placed in the protective lee of the device 8 perched beach, and as a result, survived the monitoring period with very little damage (Fig. 3-28).

The good performance of four of the concrete-rubble test sections at Alameda was attributable to a protective sand berm fronting each structure. In fact, the berm was so effective in attenuating wave action that almost any type of armoring would have prevented bluff erosion. The Alameda demonstration project was therefore discounted as a valid evaluation of the performance of concrete-rubble revetments, either with or without filter material.

Figure 3-29. The Sand-Pillow sill of device 8 provided protection for devices 7 to 10, Alameda, California, 11 April 1980.

The widespread use of concrete rubble for low-cost shore protection throughout the United States indicates a need for the development of reliable design criteria for such use. Because none of the devices installed under this demonstration program have provided adequate design data, the following procedure is suggested as an interim method for the construction of concrete-rubble revetments until the procedure can be verified or improved by extensive field experimentation. Though unproven, this procedure is based on (1) observations of many failures and a few successes with revetments of this type throughout the Nation, and (2) application of principles that have proven effective and generally reliable in the design of quarrystone revetments. Moreover, the use of concrete rubble as shore protection should be encouraged as a means of utilizing to advantage a waste product that is otherwise difficult to dispose of in an environmentally acceptable manner.

First, to be effective, the rubble to be used in a revetment should be shape-sized by breaking up slabs and beams until no piece is longer than three times its minimum dimension. Shape-sizing prevents waves from picking up individual slabs or beams and "surfboarding" them out of the section; i.e., it reduces the lifting power that turbulent wave action induces on broad areas of rubble surfaces.

Second, a slope must be selected to satisfy the equation:

$$\cot A = \frac{kH^3}{W}$$

where

cot A = slope of embankment expressed as the ratio of the horizontal leg to the vertical leg

k = 12.5 for seawater; 10.7 for freshwater

H = significant height of design wave

W = weight in pounds of median-sized piece of rubble

(This is the solution of the Shore Protection Manual (SPM)(U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1977) formula for breakwater armor stone assuming concrete rubble weighs 150 pounds per cubic foot and the shape factor is 5.)

Third, the embankment, dressed to the cot A slope, is covered with a woven filter cloth of adequate opening size for the retained material. The rubble is spread over the filter cloth to a thickness of three times the average dimension of the median-sized piece of rubble. The toe should be entrenched to a depth well below the anticipated scour depth, or a substantial sacrificial berm of additional rubble over filter cloth should be provided at the toe. The top elevation of the revetment should be at least the design wave height above the maximum anticipated water level to prevent backfill washout by overtopping waves.

c. <u>Stone Riprap</u>. Revetments of stone riprap were monitored or observed at four sites--Tawas Point, Michigan; Folly Beach, South Carolina; Siuslaw River, Oregon; and Port Wing, Wisconsin. The 400-foot revetment at

Tawas Point, armored with 100-pound stone, lies on a 1 on 3 slope with an underlayer of 4- to 10-inch stone acting as a filter. A control section tested the effect of omitting armor stone in the upper part of the revetment, which is subjected only to wave uprush. The entire structure, including the control section, survived storm waves that were reported to have breaking wave heights of up to 8 feet (although it is questionable whether they broke directly on the structure). There was no apparent damage to the structure nor to the bluff behind it (Fig. 3-30).

At Folly Beach, five stone riprap revetments were laid on 1 on 2 graded slopes. Each was constructed of "two-man" stone (about 100 to 200 pounds) in a layer two stones thick, placed on Polyfilter-X filter cloth. The revetments sustained little damage from normal wave action, although some stones were displaced and the filter cloth was exposed. However, each of these five revetments was destroyed by Hurricane David (Fig. 3-31). Stones in the 350-foot revetment in area 4 were not widely dispersed by the hurricane waves, as were the stones in the other revetments. This was because the high, steep bluff behind the revetment prevented the waves from carrying the stones over the top of the structure, and they fell back down the slope, protecting the bluff toe but exposing the upper bluff face.

The 1,200-foot revetment on the Siuslaw River consists of stones ranging from 300 pounds to 1 ton, placed on a gravel filter. It is designed for protection against river currents as well as wave action. Although not monitored under the demonstration program, it has functioned well. However, the stone sizes are substantially larger than at Tawas Point and Folly Beach, so this revetment cannot be classed as a low-cost structure.

At Port Wing, several short sections of fieldstone riprap totaling 328 feet in length were used to separate demonstration devices and to protect the two ends of the installation. The minimum stone weight was 120 pounds, 50 percent was 550 pounds or heavier, and the maximum was 2,000 pounds. The riprap is 2.6 feet thick, and the slope is 1 on 2.5. The fieldstone is rounded, not angular as is quarrystone, and it does not interlock very well; however, it performed well, without apparent damage, throughout the monitoring period.

Under normal conditions, the stone revetments have performed well under moderate wave exposure, and the gravel and cloth filters appeared equally effective. However, at Tawas Point and Folly Beach, the size of the armor stone appeared to be too small for the normal wave exposure according to current design practice. Using the CERC Shore Protection Manual formula and assuming a stone weight of 160 pounds per cubic foot, a slope of 1 on 2, and a shape factor of 5, the following stone weight requirements are found:

Height (ft)	Weight (lb)[1]
3	127
4	303
5	592
6	1,023

[1] 75 percent of the armor should be stones of at least the listed weight.

Figure 3-30. The stone riprap holding very well at Tawas Point, Michigan, 16 April 1980.

Figure 3-31. Destruction of riprap revetment at Folly Beach, South Carolina, 15 September 1979.

Continued monitoring of the Tawas Point and reconstructed Folly beach revetments is needed to assess their adequacy under more prolonged subjection to their particular wave environments.

d. Concrete Slabs. This type of revetment was demonstrated at only one site--Alameda, California. The large slabs, 15 feet long, were laid against a 60° slope covered with a sheet of Mirafi-140 filter cloth. The toe was entrenched about 2 feet into the beach sand. Wave action overtopped the revetment repeatedly, saturating the sand slope and washing out the backfill. Within a few months all 12 slabs were lying almost flat on the beach (Fig. 3-32). No attempt was made to restore the section.

To be effective, a revetment should be flexible, so that it will conform to irregularities of the slope on which it is placed. The slabs in this revetment were too large and tended to bridge over the depressions that developed under them. Also, the slope was far too steep and the slabs did not transmit enough weight to the underlying soil to hold it down and provide a stable armor layer. Hydrostatic pressures lifted and tilted the slabs as the supporting materials gave way. The slabs were eventually displaced down onto the beach. Concrete slope paving, properly designed, is an excellent protection device. Proper slope, weep holes, toe embedment, etc., are necessary and vital considerations. However, the Alameda concrete-slab revetment was not properly designed and therefore failed.

e. Stacked Bags. Revetments constructed with stacked bags were demonstrated at two sites--Alameda, California, and Oak Harbor, Washington. At Alameda, four 50-foot sections were demonstrated; each section used a different type of construction. A discussion of these four devices follows.

(1) Device 11. Ultraviolet-resistant, woven acrylic Sand Pillows by Monsanto Company, filled with sand and tied shut, using a double-lap seam, with 17-pound-test nylon garden twine. Each filled bag weighed about 100 pounds. The bags were stacked in horizontal tiers with the long axis of each bag parallel to shore. Each tier was echeloned back to make solid contact with the prepared 60° slope. The joints between bags in each successive tier were supposed to be offset as in brickwork, but this did not always occur, and many bags were directly above those below. The revetment was about one bag thick extending from the toe at MLLW (about 2 feet below the sandline) to about +8 feet MLLW.

Device 11 was installed about 100 feet downdrift of the device 8 perched beach at Alameda. The sill surrounding and retaining the device 8 beachfill acted as a groin, preventing littoral nourishment from reaching the device 11 area. As a result, the beach fronting device 11 became progressively lower with time. Within a few months, the toe was undercut and the lower tiers of bags slid downward, exposing gaps of unprotected embankment farther up the slope. Also, it was noted that the closure twine in many of the bags had broken, allowing sand to spill out and cause collapse of these bags. Wave action during high tides then removed the embankment material from behind the bags, hastening the collapse of the entire structure. The erosion seemed to occur both at the toe and along the top of the embankment. As the Sand Pillows slid, erosion of the embankment increased. After about 6 months, all of the bags were down on the beach, and most were empty (Fig. 3-33). The bag fabric had not deteriorated.

Figure 3-32. The concrete slabs lying almost flat on the beach, Alameda, California, 11 April 1980.

Figure 3-33. Device 11 (left) failed completely, and subsequent erosion of the bank occurred, Alameda, California, 18 July 1979.

(2) Device 12. This revetment was identical to device 11 except that the Sand Pillows were filled with a lean (8-to-1) sand-cement mixture. As a result, the Sand-Pillow modules generally held their shape (although some did crack), but toe scour and bank erosion behind the modules eventually rendered the revetment ineffective. Within 1 year, all of the modules were in a windrow on the beach, and erosion of the embankment behind them continued virtually unabated, although the modules on the beach did absorb a small amount of wave energy (Fig. 3-34).

(3) Device 13. This revetment was identical to device 12 except that burlap bags were used in lieu of acrylic Sand Pillows. Because they were filled with a lean sand-cement mixture, they held their shape; however, their performance as a revetment was nearly identical to that of device 12.

(4) Device 14. This revetment was identical to device 11 except that burlap bags filled with sand were used in lieu of acrylic Sand Pillows. However, device 14 lasted longer than device 11, being partially protected by the small fillet of sand held by the groin effect of device 15 a few hundred feet downcoast. After about 8 months, however, the bag material began to deteriorate rapidly. Sand leaked out of the bags, and in another month, little evidence of the structure remained (Fig. 3-35).

The following lessons were learned from the stacked-bag revetments at Alameda:

(a) The 60° slope is far too steep for any type of revetment.

(b) The single-bag thickness of the revetment was inadequate. When any one bag moved out of place, the embankment became exposed to erosive forces that could progressively destroy the entire structure.

(c) Sand-cement fill provides some degree of structural safety with sandbag construction. However, if filled only with sand, a nondegrading fabic such as the acrylic material in the Sand Pillows should be used, and closure twine should be sufficiently strong and durable to make the bag closure as secure as the fabric material itself.

At Oak Harbor, two types of sandbag revetment, each 150 feet long, were demonstrated in device 1--one with a full-strength, wet-mix concrete in burlap bags, and the other with commercial dry-mix sand-cement in paper bags which were punctured and saturated with water as construction progressed. With each type, half the section was underlain either with Mirafi-140 filter cloth or with a gravel filter. No filter material was used in half of each revetment. Half of the paper bag dry-mix revetment was placed against Mirafi-140 filter cloth. Both revetments were built to a 1 on 1 slope, two bags thick. The base for each revetment was excavated through about 6 inches of littoral mantle to the underlying clay till, which is at or just below MHW (11.5 feet MLLW) in the project area. Each revetment was

Figure 3-34. Sand-Pillow revetment of device 12 in a windrow while erosion of embankment continued, Alameda, California, 7 February 1980.

Figure 3-35. Device 14 failed after 8 months; sandbag material deteriorated and bags lost sand, Alameda, California, 14 November 1979.

carried to an elevation of +17.5 feet MLLW, and 2-inch-diameter PVC drainpipes were installed at about elevation +13.5 feet MLLW, 10 feet on centers along the revetment.

The performance of the stacked-bag revetments at Oak Harbor was exactly the reverse of that at Alameda. Of the four types of bulkheads and revetments demonstrated at Oak Harbor, the stacked bags sustained the least damage (Fig. 3-36); however, during the monitoring period, waves of design magnitude occurred during extreme high tides at least once, and near-design waves attacked the structures on several occasions. Storm wave heights at Alameda and Oak Harbor were comparable, ranging from 3 to 3.5 feet at point of breaking on structures. However, because of the much greater tidal range at Oak Harbor, the devices installed there were subjected to destructive wave action for shorter periods of time. Also, the devices at Oak Harbor were protected with a continuous toe riprap of 12-inch quarrystone.

In assessing the construction problems at Oak Harbor, the District monitor noted that the intersack bonding of the burlap bags (device 1a) was much better than that of the paper bags (device 1b) in which small voids occurred at joints between sacks. The poor intersack bond in device 1b was due to the paper. Also, the cost per foot was only $56 for burlap bags, but was $84 for paper bags. The burlap bags were not sewn shut at Oak Harbor, but open ends were folded under, with the bags filled only to about 75 percent capacity. This was somewhat wasteful of bag material, but was cost-effective in reducing construction time and eliminating labor expense. Some of the bags were removed from device 1b, apparently by vandals, but enough remained to provide adequate protection for the 40-foot embankment behind the revetment.

The excellent performance of the concrete-filled bags at Oak Harbor demonstrated the effectiveness of this type of low-cost shore protection when properly constructed. The improvement in results at Oak Harbor over results at Alameda was primarily due to the less steep slope, the two-bag thickness of the revetment, the use of weep holes, and the use of toe protection at Oak Harbor. The Oak Harbor demonstration indicated that at least equally good results, at much lower cost, can be obtained by using wet-mix filled burlap bags instead of dry-mix filled paper bags. It also indicated that filter cloth or filter gravel does not necessarily improve the performance of this type of revetment. However, the nature of the backfill material should be assessed before deciding to eliminate filter material with this type of revetment at any given site. The use of a good gravel or cloth filter may be good insurance against the possibility of "piping" or pumping of backfill through small voids in the revetment during periods of high wave attack.

f. <u>Fabric</u>. Revetments of synthetic cloth fabric were demonstrated at two sites--Alameda, California, and Fontainebleau State Park, Louisiana. A Fabriform filter-point nylon mat over Mirafi-140 filter cloth was filled with sand and used as a revetment at Alameda. Fabriform is intended by the manufacturer to be filled with concrete grout, the cloth serving primarily as a form until the grout has hardened to provide a continuous concrete lining for a channel or basin. Even when filled with grout, this device has been subjected to very few tests as a revetment in the shore zone, as less costly materials are usually available which can serve the same purpose. In the Alameda demonstration, the mat was filled with a sand slurry instead

Figure 3-36. Device 1a, stacked-bag revetment, performed well, Oak Harbor, Washington, 23 April 1980.

of concrete grout, as a means of reducing the cost. It was placed on a 60° slope, with the toe entrenched 2 feet into the beach sand. U-shaped steel pins made of No. 3 reinforcing bars were driven through the mat at 10-foot intervals along the top of the slope in an effort to prevent the mat from sliding down.

The fabric revetment was completed in November 1978 and remained in place until a storm the latter part of March 1979, when an inspection revealed that the fabric had been torn in several places causing a loss of sand and exposing the filter cloth. In the succeeding months, the mat deteriorated rapidly. By August 1979, most of the mat was lying on the beach, and the embankment behind it was being eroded (Fig. 3-37). No attempt was made to restore the device. Fabriform was never intended for use as a revetment when filled only with sand. The concept should be rejected.

At Fontainebleau State Park, device 7 is a pocket filter-cloth revetment ballasted with shells. In principle, the ballast in the presewn pockets located at intervals throughout the fabric holds the cloth in place on a flat slope in the wave overtopping zone until it can be covered with about 12 inches of fill, half shells and half topsoil. The topsoil layer is then planted with a quick-growing grass that is known to root well in the local environment and to have a fair tolerance of brackish water. At Fontainebleau, Bermuda grass was used. The root structure of the grass holds the fill in place when inundated by overtopping waves. The filter

Figure 3-37. The Fabriform revetment, device 4, tore and lost most of its sand by August 1979, Alameda, California, 7 September 1979.

cloth limits the depth of any washouts that may occur and prevents progressive unraveling of the revetment until it is repaired or heals itself by natural spread of the grass. Device 7 was completed in October 1979, and during the winter the lakeward half of the soil and shell cover was removed by wave action. In April 1980, a storm removed the remainder of the cover. During the remainder of the monitoring period, wave action was not severe enough to damage the exposed filter cloth; however, without a protective cover, it was not expected to last long. The device 7 demonstration highlighted the need for some type of structural protection lakeward of the revetment to allow the grass time to root and become established. Another method is to completely cover the filter cloth with sand-cement bags.

Device 8 at Fontainebleau is a loop filter-cloth revetment ballasted with concrete blocks. In principle, the ballast is attached to the loops which are located at regular intervals throughout the cloth. As with the pocket filter cloth, the ballast holds the cloth in place until it can be covered with fill and planted. Jumbo blocks were used for ballasting device 8; in addition, sand-cement bags were placed along the back edge of the cloth, and steel pins were driven through the openings in the blocks into the underlying soil. Filling and planting were the same as for device 7. The device 8 experience closely paralleled that of device 7, except that it was in an area where common reed had formerly grown. Regrowth of the cane lifted two rows of Jumbo blocks along the crest, and wave action displaced them. By May 1980, the toe Jumbo blocks were still holding with the aid of rapidly rusting holddown pins, but most of the filter cloth was exposed. Thus, device 8 also highlighted the need for additional structural protection.

g. Tire and Fabric. A revetment constructed of used tires and filter cloth were demonstrated at only one site--device 9, Fontainebleau State Park, Louisiana. A 55- by 40-foot Nicolon 66424 filter cloth was positioned on the prepared 1 on 3 slope. Two rows of sand-cement bags were placed parallel to the lakeward edge of the cloth. The toe was protected by lapping the cloth over the bags, forming a Dutch toe. Used tires were then placed on the cloth, filled with dry sand-cement mixture and sprayed with water. One row of bags was then placed along the back edge of the filter cloth and galvanized pins were driven through the bags and underlying cloth into the sand. Work was completed in October 1979.

The spaces between the bottom three rows filled with sand after the first 2 months of operation, but a storm in April 1980 displaced 50 percent of the tires, and destruction of the revetment by future storms seemed imminent. It appeared that waves higher than 3 feet tend to make the tires bouyant, moving them around on the revetment. Until some method of stabilizing the tires so as to prevent their displacement by waves is devised, this revetment system is not recommended.

h. Gabions. Filled gabion baskets were used as revetments at three demonstration sites--Kotzebue, Alaska; Ninilchik, Alaska; and Oak Harbor, Washington. At Kotzebue, a gabion revetment 18 feet wide was constructed on a 1 on 3 graded slope, in four sections (Fig. 3-38). Two sections used PVC-coated baskets--one containing gravel-filled burlap bags, the other containing gravel-filled acrylic bags. A third section used galvanized gabion baskets containing sandy gravel. The baskets were lined with galvanized wire-mesh hardware cloth before the gravel was added. The fourth section contained both types of baskets, PVC and galvanized, and each type was lined with Polyfilter-X filter cloth and filled with sandy gravel. Only one gabion basket has been damaged to date. The damage resulted not from natural causes, but from a fisherman using the gabion as a dock for his boat.

The gabion revetment at Ninilchik comprised a tiered mattress of seven contiguous tiers of gabion baskets placed on the existing beach slope and extending 240 feet along the shoreline. It uses PVC-coated baskets lined with Polyfilter-X filter cloth and filled with a combination of beach gravel, cobbles, and concrete rubble. The revetment is 21 feet wide, the baskets in the tier measuring 3 by 3 by 9 feet. The revetment serves as toe protection for an existing log revetment in its lee. Accretion and depletion alternately covered and reexposed the upper part of the revetment during the evaluation period, but the lower part remained covered most of the time. The filter-cloth lining in the baskets was ruptured at many locations along the top tier, but without a substantial loss of fill. A number of small rocks punctured the cloth as they were hurled into the gabions by a November 1979 storm, which generated 6- to 6.5-foot waves, and one gabion lid came off during the storm. Some loss of PVC coating was also evident along the top tier. Most of the damage occurred in the exposed 2 feet along the seaward face of the top gabion tier (Fig. 3-39). The revetment could be improved by using a smaller basket size for the top tier so as not to expose a high vertical face, as the log revetment behind it would not be damaged as severely by waves overtopping the gabions.

Figure 3-38. A view of the gabion revetment, Kotzebue, Alaska, 29 August 1979.

Figure 3-39. Torn Polyfilter X-filter cloth bags along the top tier of the gabion revetment, Ninilchik, Alaska, 5 October 1979.

The 278 feet of PVC-coated gabion revetments (device 2) at Oak Harbor were of the mattress type (only 1 foot 8 inches thick) placed side-by-side on a graded 1 on 1.5 slope. The use of gravel and cloth filters versus control areas without filter was evaluated. All sections had a rock riprap toe protection. The revetments were subjected to storm waves of at least design height twice during the monitoring period without suffering major damage. Backfill was lost from behind the top edge of the control areas, apparently the result of its being washed through the gabions (Fig. 3-40). All toe protection was displaced and many undersized stones were washed from the gabion baskets. The filterless areas were reconstructed, half to include gravel and half to include cloth filter. These two types of filters proved equally effective.

The functional effectiveness of gabion revetments is sensitive to slope and tidal conditions. Where the tidal range is large and the beach front is wide, the tiered revetment on a flat slope in the upper tidal zone at Ninilchik served as an accretion device somewhat like a Sandgrabber. Water-borne materials transported with wave runup on the revetment were trapped by the tiering arrangement. The revetments at Oak Harbor and Kotzebue were constructed on steeper graded slopes than was the revetment at Ninilchik. These revetments were being evaluated primarily for bank protection, and they functioned well for that purpose.

The relatively low cost of gabion revetments makes their usage attractive. The baskets are easily handled and readily attained; the fill-material options are diversified. Appropriate rock sizes are usually readily available, which keeps costs low. Also, the flexibility of a gabion installation enables the device to conform to minor subgrade changes without damage from toe scouring and differential settlement. A revetment of gabion baskets is a consistent, low-cost performer that is applicable to varying wave and site conditions where the wave environment is relatively mild. One drawback to gabion-revetment usage is the hazard imposed by broken wires. Care must be exercised to fill the baskets completely, and refill them at least once after initial consolidation has occurred, or they will deform and lid closures will rupture. The monitoring period was too short to determine the longevity of either galvanized or PVC-coated baskets in marine waters and freeze-thaw cycles.

i. Steel Fuel Barrels. The use of 55-gallon steel barrels to form a revetment was demonstrated at one site only--Kotzebue, Alaska. Two shore-parallel double rows of bolted (together) 55-gallon barrels were spaced 10 feet apart, and the barrels were filled with sandy gravel. A diaphragm consisting of a single row of shore-perpendicular barrels spanned the 10-foot gap between these rows every 50 feet. Fill was added between the diaphragms to form perched beaches. Overtopping of the lower seaward barrels deposited more fill between the diaphragms, increasing the functional effectiveness of the revetment (Fig. 3-41). Shoreline recession was halted, and the barrels survived the monitoring period with little damage. The steel fuel-barrel revetment system appears to be an effective shore protection device. Further monitoring is needed to determine whether rusting of the barrels will soon terminate their effectiveness. At Ninilchik the use of fuel-barrel revetments was terminated in 1978 when the District Engineer found that the barrels corroded within a few years, and other shore protection devices were substituted.

Figure 3-40. Device 2bW (foreground) has no filter cloth; device 2bE (background) has filter cloth. Note loss of backfill behind device 2bW, Oak Harbor, Washington, 19 December 1978.

Figure 3-41. Steel-barrel revetment performed well during its short monitoring period. Note the large deposit of fill between the two sets of rows, Kotzebue, Alaska, 29 August 1979.

This system is economical only when discarded fuel barrels are readily accessible, as was the case at Kotzebue. Obviously, it has very limited application, However, if fuel barrels are used in any future installations, the following modifications are suggested:

(1) Completely fill the barrels on installation to limit crushing of the upper half by floe ice and debris;

(2) cap the critical seaward barrels with concrete;

(3) partially bury the barrels to increase their stability; and

(4) enhance the esthetic appearance of a structure of discarded steel fuel barrels by tiering.

j. Profiles.

(1) Concrete Blocks. Profiles of concrete-block revetments were compared at three sites: Fontainebleau, Louisiana; Port Wing, Wisconsin; and Holly Beach, Louisiana (Fig. 3-42).

At Fontainebleau the profiles are at device 3, a revetment of standard construction blocks. The structure was damaged by vandalism and problems were encountered with displacement of blocks with their long axes placed parallel

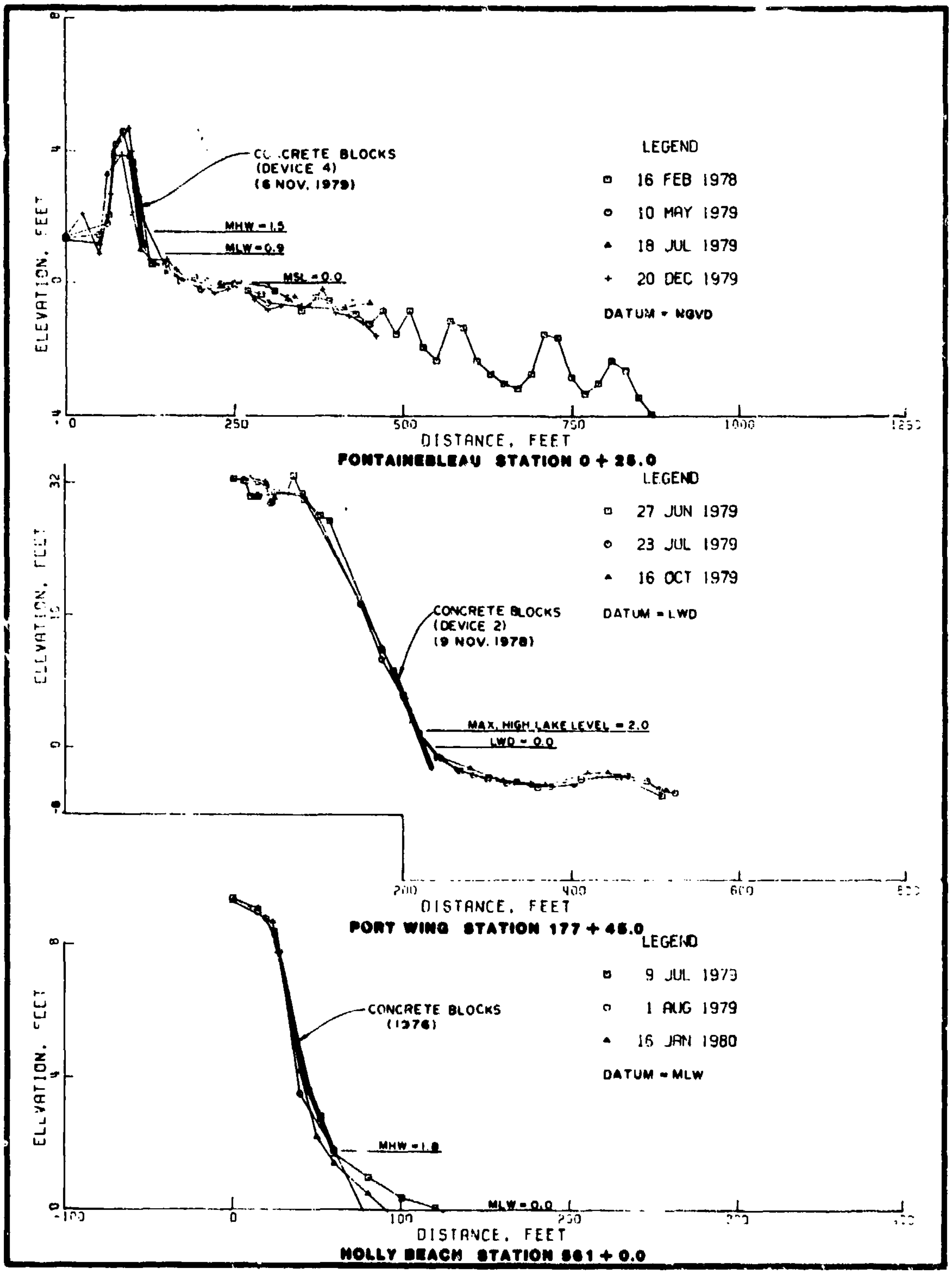


Figure 3-42. Profiles of concrete-block revetments.

to the shoreline. This structure was completed in November 1979. Only one survey was made after the completion of the revetment, thus, the profiles do not give any information as to the effectiveness of the structure in retaining the beach.

At Port Wing, the profiles are at device 2, a revetment of small control blocks. This revetment, despite some settlement and damage from storms, remained essentially intact throughout the monitoring period. The profiles show that this structure adequately prevented bank erosion. The survey data only cover a period of 3 months, not enough time to determine the effectiveness of the structure.

At Holly Beach, the profiles are typical of 4 miles of Gobi-block revetment. This revetment was damaged, rebuilt, and extended after it was first constructed in 1970. The revetment was damaged in September 1979 by storm waves, and some erosion on the bank occurred. The profiles only cover a 6-month period (July 1979 to January 1980), but these dates include the September storm, and some erosion is apparent, particularly in front of the structure.

(2) Concrete Rubble. Profile analysis was done to compare concrete-rubble revetments at two sites--Shoreacres, Texas, and Alameda, California. These profiles are shown in Figure 3-43.

The concrete-rubble revetment at Shoreacres was successful in protecting the beach from erosion. This structure remained intact and undamaged during the monitoring period. The profiles show that no erosion occurred at the site.

The concrete-rubble revetment (device 7) at Alameda survived the monitoring period without much structural damage, but this is attributed to the fact that the structure was protected by a fillet of sand trapped in the lee of a sandbag sill. The profiles show the accretion in front of the revetment which protected it from damage. Other concrete-rubble revetments which were exposed to wave action were destroyed at Alameda. Therefore, the Shoreacres revetment was a more effective device in protecting the beach; however, this effectiveness is site-sensitive.

(3) Stone Riprap. Profile analysis was done to compare stone-rubble revetments at Folly Beach, South Carolina; Muskegon, Michigan; and Tawas Point, Michigan. The profiles are shown in Figure 3-44.

A stone riprap revetment (area No. 3) was demonstrated at Folly Beach. The revetment was destroyed in September 1979 by Hurricane David and erosion of the beach occurred, as shown in the profiles. Subsequently, the revetment was rebuilt and sand was placed behind the structure, also shown in the profiles.

The Muskegon rubble was not a designed revetment, but was a stone layer at the foot of a beach fill which was formed by selective sorting. The result was a layer of stone rubble which covered the toe of the fill. Some erosion and slumping of the bluff occurred at this site, as shown by the profiles.

A stone-rubble revetment was monitored at Tawas Point. This structure remained structurally sound and withstood storm waves with breaking heights of

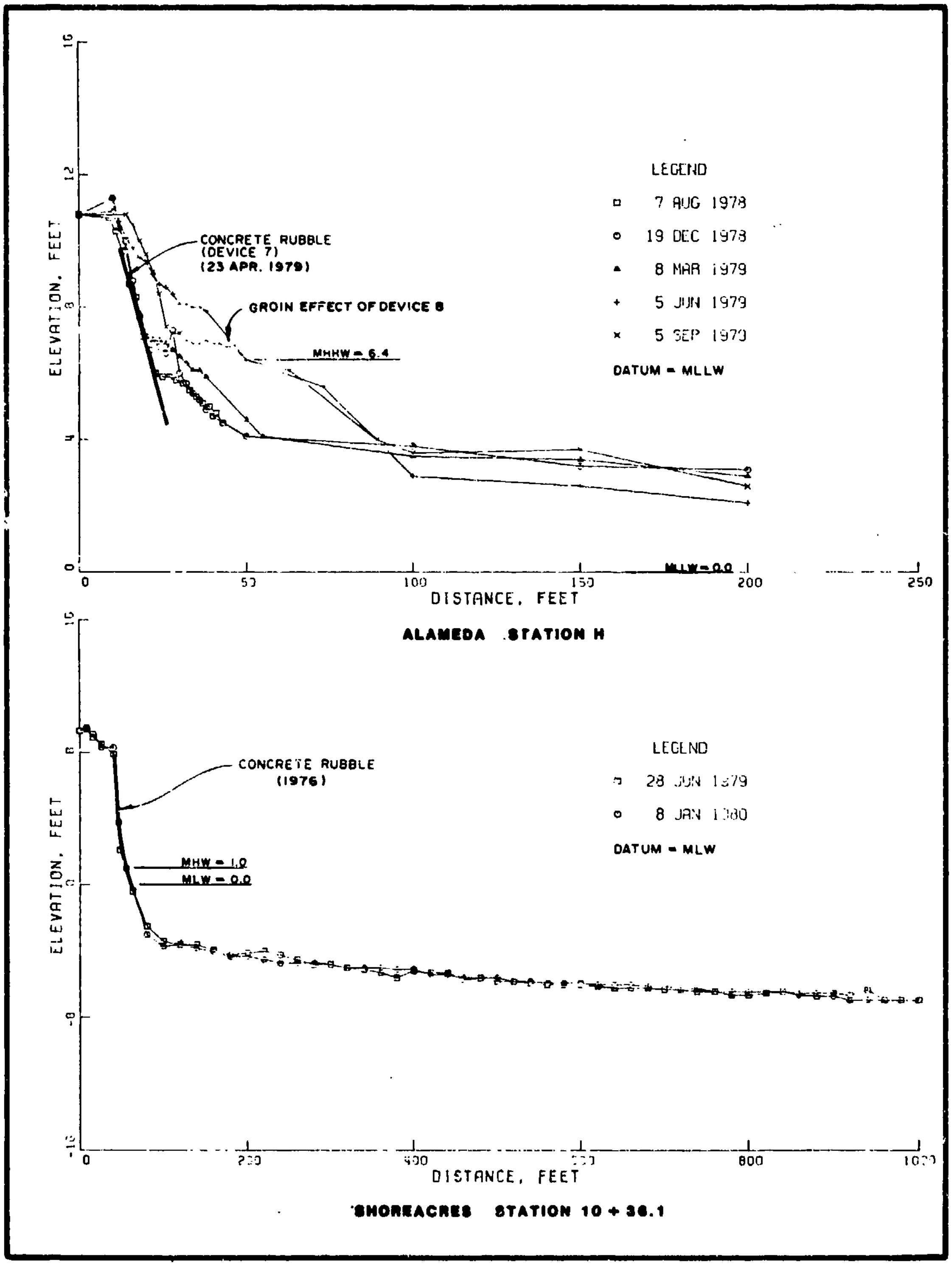


Figure 3-43. Profiles of concrete-rubble revetments.

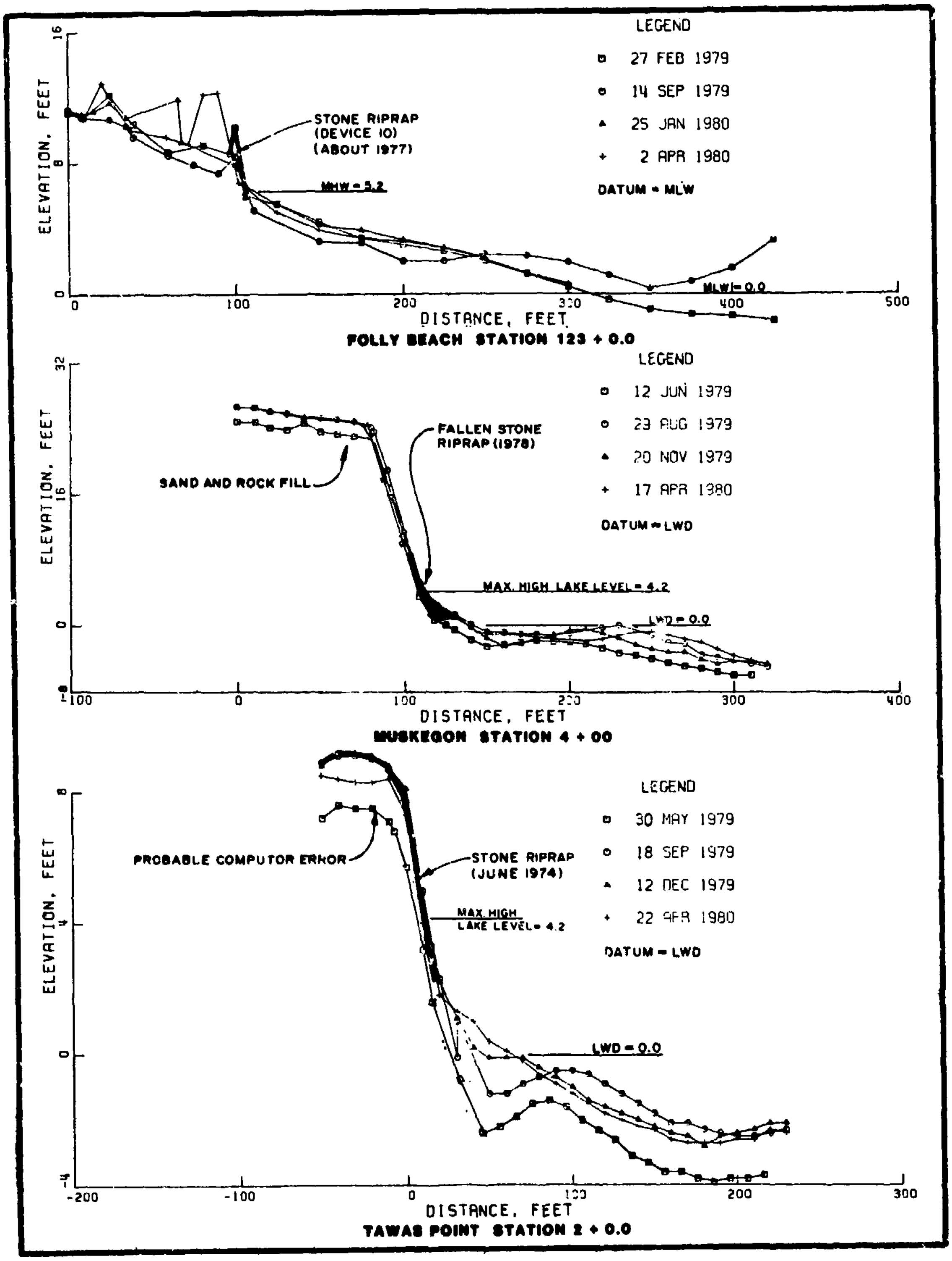


Figure 3-44. Profiles of stone-riprap revetments.

up to 8 feet. The profiles show that significant accretion was present at the structure site.

The stone-rubble revetment at Tawas Point was the most effective in preventing erosion, as the others suffered damage and allowed erosion.

(4) Stacked Bags. Profile analyses were done to compare stacked bag revetments at Alameda, California, and Oak Harbor, Washington. Profiles are shown in Figure 3-45. A revetment of stacked bags was demonstrated at Alameda. The toe of this structure was progressively undermined and the structure eventually failed. The embankment behind the structure also suffered severe erosion. The profiles show erosion on both the beach in front of the revetment and the embankment behind it. (Note: the structure was built about 4 feet higher than the construction drawings called for.)

A revetment of stacked bags was also demonstrated at Oak Harbor. This structure, unlike Alameda, performed well, surviving wave attack with minimal damage. The revetment successfully protected the beach from eroding as shown in the profiles. This structure was clearly more effective than the Alameda revetments.

(5) Gabions. Profile analysis of gabion revetments were done at three sites--Kotzebue, Alaska; Ninilchik, Alaska; and Oak Harbor, Washington. The profiles are shown in Figure 3-46. The gabion revetment at Kotzebue suffered no structural damage to date, and the structure adequately protected the beach. The site, however, has not been surveyed since the initial time in October 1979, and thus the profiles offer no information as to the extent of erosion or accretion at the site.

The gabion revetment at Ninilchik suffered some damage in that some baskets were damaged and rocks were knocked out. Despite this damage, the structure protected the beach from erosion. The profiles show that a significant amount of accretion was present at the site. Both of these structures appear to work reasonably well, but there has not been sufficient survey data to adequately compare the two structures.

3. Breakwaters and Sills.

a. Floating Tire. Floating tire breakwater systems were tested at three demonstration sites--Stuart and Jensen Beach Causeways, Florida; Pickering Beach, Delaware; and Fontainebleau State Park, Louisiana.

The floating tire breakwater installed at the Stuart and Jensen Beach Causeways site, on the south side of Stuart Causeway, was in conjunction with a vegetation planting scheme. It was intended for temporary use to reduce wave energy so that the vegetation could establish. The tire modules, containing about 3,000 tires, were assembled onshore, then floated 300 feet offshore, connected and anchored. The tire modules were bound and interconnected with a 0.5-inch, nylon-covered polypropylene rope; the breakwater was secured with the rope to Danforth anchors set in the bottom. Many binding ropes were soon cut by tire rims, and the anchoring rope gave way during Hurricane David in September 1979, allowing the entire breakwater to drift ashore (Fig. 3-47). The breakwater was reconstructed using screw anchors and steel cable late in the program, but the time left for monitoring was too

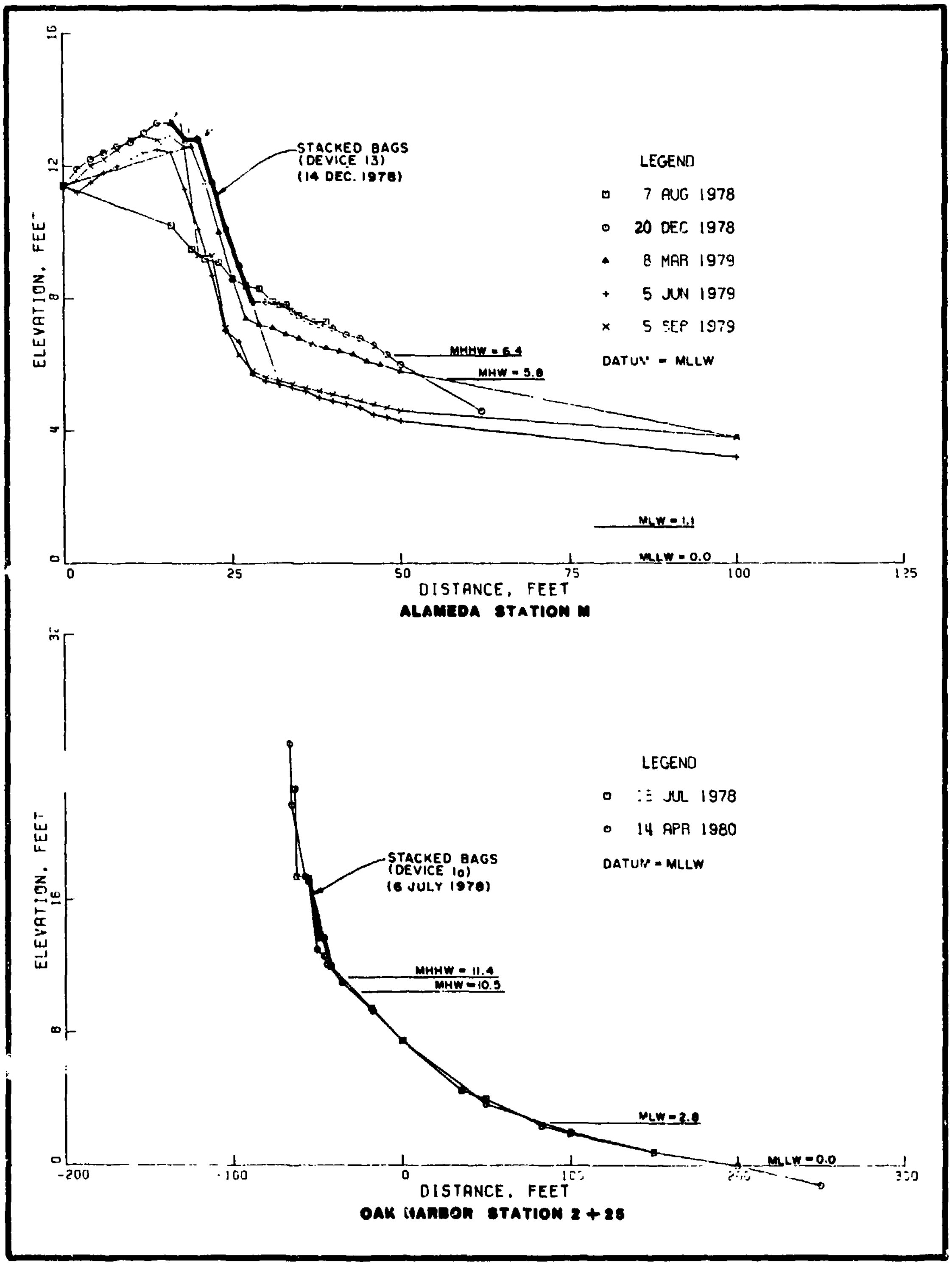


Figure 3-45. Profiles of stacked-bag revetments.

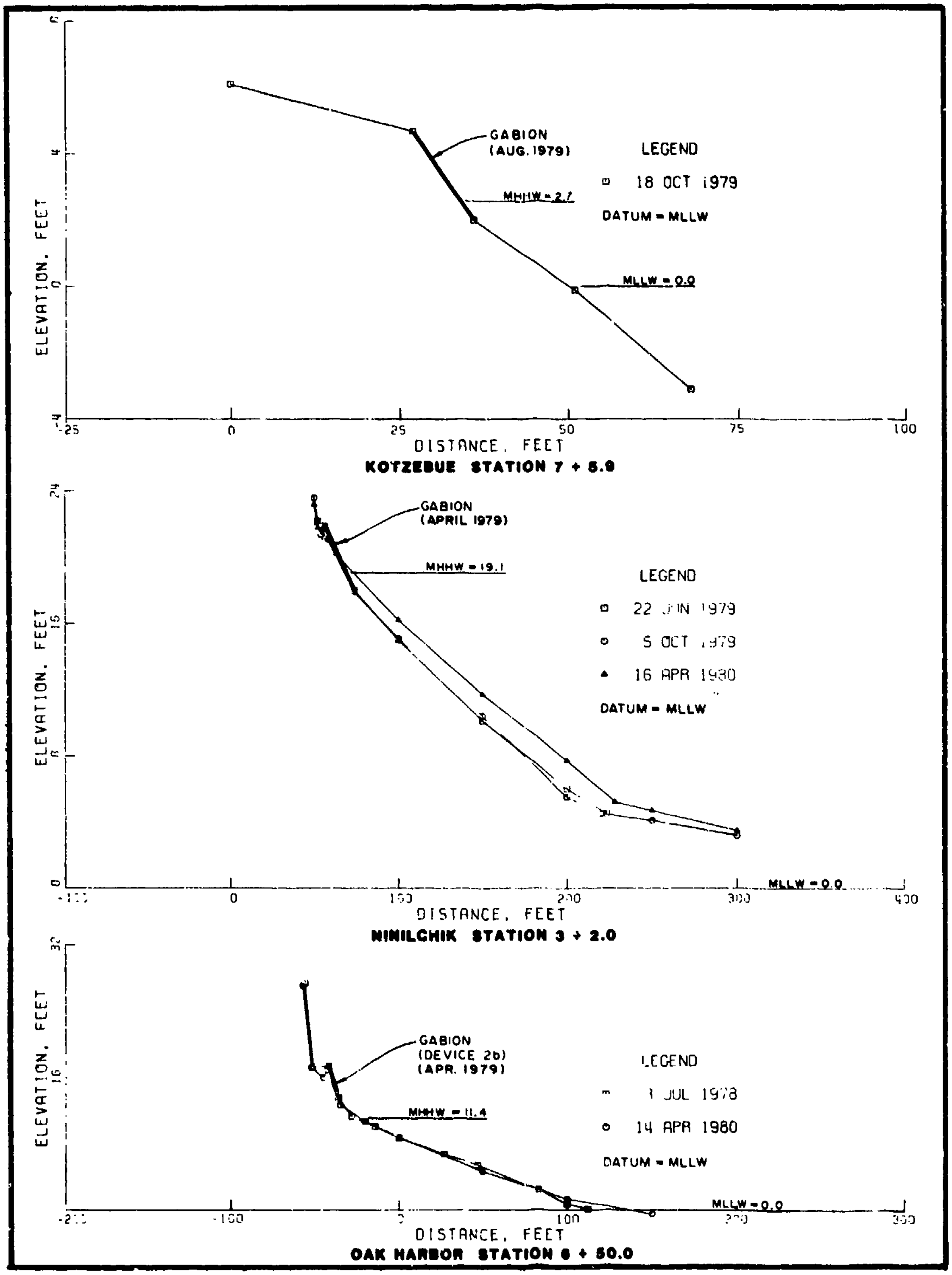


Figure 3-46. Profiles of gabion revetments.

Figure 3-47. Floating tire breakwater at Stuart and Jensen Beach Causeways, Florida, washed ashore with tire bundles scattered along beach, 7 December 1979.

short to evaluate performance.

Two types of floating tire breakwaters, each about 400 feet long, were constructed and evaluated at Pickering Beach. The Wave-Maze, a patented system, consisted of many five-tire modules bolted together with galvanized bolts, nuts, and fender washers. The Goodyear system consisted of a smaller number of 18-tire modules bound together with rubber conveyor belt edging. The two breakwater systems had the same type of anchoring system--a 0.5-inch galvanized anchor chain secured them initially to a number of strategically located 4-ton concrete anchors. Half of each breakwater was 40 feet wide, and half was 20 feet wide. The Wave-Maze had a 50-foot-long control section of more costly rubber patches and nylon bolts (recommended by the patent holder) so that their performance could be compared with that of less costly galvanized washers and bolts. For flotation, 0.5 pound of polyurethane foam was pumped into each tire of the Goodyear system, and 1 pound was pumped into each vertical tire of the Wave-Maze.

Problems developed in the Wave-Maze as the galvanized bolts and washers pulled through the tires, and some of the nylon bolts snapped because of excessive stresses due to wave action and ice buildup in winter. Two large clusters of the Wave-Maze were washed ashore as the result of bolt failure, and many modules broke away. The Goodyear breakwater did not break up, but was deformed by unequal drag of the anchors (Fig. 3-48). The Wave-Maze was dismantled; however, it was considered a partial success because of the demonstrated inadequacy of the more costly washer-bolting technique. The Goodyear assembly was less complicated, thus easier to install. A timber-pile

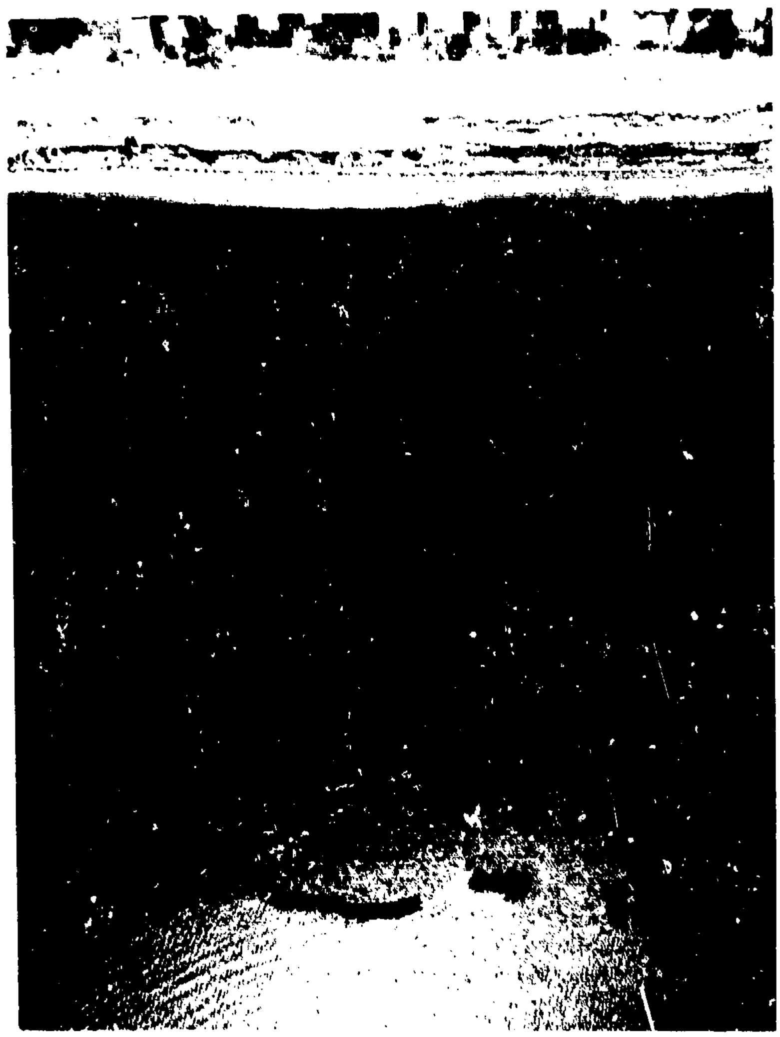


Figure 3-48. The Wave-Maze tire breakwater (at right) broke apart and some modules floated ashore. The Goodyear tire breakwater (at left) was deformed by drag of anchors. Pickering Beach, Delaware, 3 July 1979.

anchoring scheme was implemented for the Goodyear breakwater. The monitoring period was too short to adequately evaluate the performance of the reanchored Goodyear breakwater, but incipient littoral accretions in its lee indicated that it was performing as intended.

This demonstration did not necessarily indicate the superiority of the Goodyear system, as the modules in that system, being strapped together, cannot expand and contract under wave agitation and are therefore less able to attenuate wave energy. The Wave-Maze is intended to be constructed with truck tires, but automobile tires (with less intrinsic strength) were used at Pickering Beach to reduce the cost. The bolted connections of the Wave-Maze allow the modules to expand, contract, and deform under wave action by flexing the sides and treads of the tires. The patent holder claims that this breaks up the orbital rotation of water particles that propagate the waves, and thereby the breakwater attenuates them more effectively than does the Goodyear breakwater system. He also claims that the bolting could be done more rapidly with trained personnel and proper assembling equipment. The use of smaller automobile tires also hindered the bolting procedure and increased the cost.

Pickering Beach proved to be a good site for the demonstration because of its distance from the mouth of Delaware Bay. The wave climate is not severe (less than a 6-foot design wave) with waves characteristically of short period and length. Wave period is critical for a floating breakwater, as similar installations at sites subjected to long-period waves have been unable to adequately attenuate such waves. The monitoring period was too short to determine ice effects on the Goodyear breakwater.

At Fontainebleau, on the northeast shore of Lake Pontchartrain, two types of tire and pile breakwaters were demonstrated. One device was a timber-tire breakwater where piles were driven vertically into the bottom so that every three piles formed a triangular pattern. Used automobile tires were stacked on the piles to a height of 4 feet above the bottom. Just above the level of the top tires, the triangular groupings of piles were interconnected by rope or 2- by 6-inch planks bolted to the piles (Fig. 3-49). This device attenuates the waves effectively, and shows no evidence of deterioration. Sand has accumulated both offshore and on the beach itself.

The other tire and pile breakwater at Fontainebleau was a rolling-tire breakwater, constructed by threading used truck tires on four three-pile cores and tying the pile ends by steel cable to 5-foot screw anchors in the lake bottom, with modules placed end-to-end parallel to shore. The tires were free to roll on the bottom, and the structure to rock back and forth under wave agitation, within the range permitted by the anchorage system. This device effectively attenuates waves and has afforded protection to onshore vegetation. Sand deposited inside the rims of the tires, however, is preventing flotation, and one of the modules has settled at least 1 foot into the soft lake bottom (Fig. 3-50). Scour around the screw anchors, some of which have pulled loose, has allowed the structure to take on a curved shape concave to the beach as waves diffract around its ends. It is recommended that in any future installations this structure be built with longer screw anchors and flotation material which would allow the tires to float at any lake stage.

Both devices were installed where the bottom was about 1 foot below mean tide level. The tides in Lake Pontchartrain have a mean range of only 0.6 foot,

Figure 3-49. Timber-tire breakwater attenuates waves effectively and shows no evidence of deterioration, Fontainebleau, Louisiana, 9 May 1980.

Figure 3-50. Sand deposited in the tire rims of the rolling-tire breakwater is preventing flotation, and one of the modules has settled about 1 foot, Fontainebleau, Louisiana, 9 May 1980.

but wind setup often raises the water level at the site 2 feet or higher. Because the rolling-tire device is limited in height by the diameter of the tires, it might not be as effective as the stacked-tire device where the tidal range is greater than at Fontainebleau.

b. Longard Tubes. Longard tubes were evaluated as breakwaters at two demonstration sites--Alameda, California, and Basin Bayou, Florida. A 69-inch-diameter tube, 330 feet long, was installed at the Alameda site. A continuous 10-inch tube, factory-sewn to a woven synthetic filter cloth, provided protection against undermining of the large tube by toe scour. The filter cloth extended from the small tube back under the large tube and behind it. The structure withstood natural forces well, but began deteriorating when punctured and torn by vandals. A row of 9- by 3- by 2-foot sand-cement blocks was then placed along the crest of the large tube to increase the breakwater height after the filled tube had flattened from loss of sand and sunk deeper than expected into the soft bottom. The blocks were easily displaced and soon had to be removed to the bayside of the structure to eliminate a safety hazard (Fig. 3-51). Thereafter, the tube was progressively vandalized until it could no longer retain the manmade sand tombolo that had been placed in its lee. The Longard-tube breakwater was removed a few months after its installation.

The 69-inch tube at Basin Bayou was also damaged by vandals, but not as severely. A 40-foot length of the 200-foot breakwater was cut open with a sharp instrument, allowing it to spill sand (Fig. 3-52), and halting development of tombolo which had been forming in the lee. This breakwater rested on a layer of synthetic filter-cloth material, with factory-sewn 10-inch Longard tubes on each side, which prevented both toe and heel scour during the monitoring period. This system of toe protection is recommended at future installations. A new Longard tube was installed at the site and cladded with aluminum sheeting in May 1980, and the tombolo began to reform. By June 1980, some of the corners of the sheeting had bent outward, and missing rivets had caused some sheets to loosen. The device was considered a safety hazard, and the District Engineer recommended removal of the aluminum sheeting.

The placement of blocks on top of the Alameda tube was an expedient not recommended by the tube manufacturers. Although the blocks were not displaced by wave action they reduced the stability of the structure. Because of the precarious perch of the blocks, stronger waves might have dislodged them even before the vandalism occurred. Where greater height is needed than that provided by a 69-inch tube, an alternative breakwater system should be used. Additional height was not necessary at the Basin Bayou installation because water depth at the site averages less than 4 feet, and the bottom was sufficiently firm to prevent sinkage of the structure. The vulnerability of the tube to vandalism and to damage by floating debris is the major problem with the Longard-tube. Where vandalism is not a problem, or where it can be thwarted by a resistant coating or cladding, it appears to have a good potential for use as a low breakwater. The rapidity with which it can be installed may be an important factor in its selection, provided that the site conforms to the applicable criteria for its use. However, the filter cloth base is a necessary adjunct to prevent toe scour or undermining, particularly when used as a seawall or as slope toe protection. Longard tubes, once in place, are not very flexible, and will not generally deform or conform to major changes in bottom conditions.

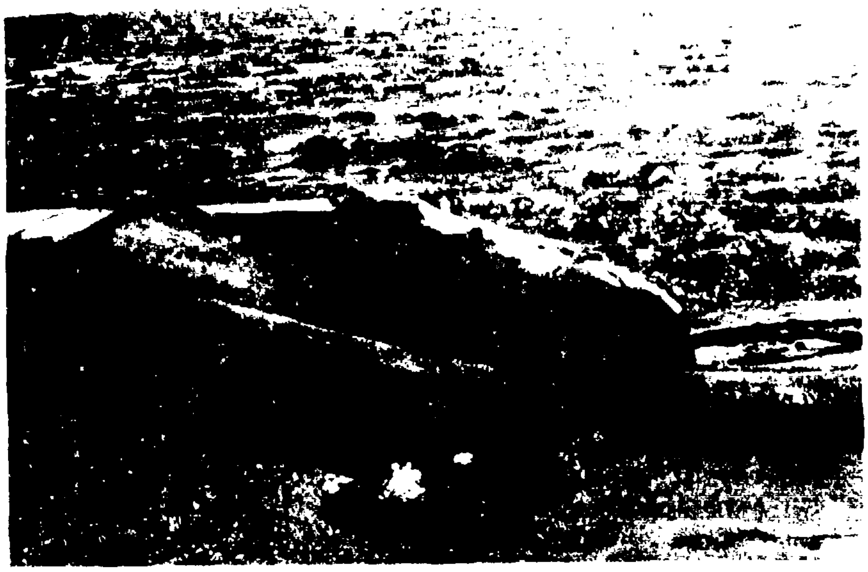


Figure 3-51. Longard-tube breakwater at Alameda, California, after vandalism and removal of concrete blocks, 2 April 1979.

Figure 3-52. Hole in Longard tube at Basin Bayou, Florida, resulting in considerable loss of sand, 20 August 1979.

c. Sandfilled Bags. The performance of assorted sandfilled-bag breakwater and sill systems was evaluated at Buckroe Beach, Virginia; Basin Bayou, Florida; Kitts Hummock and Slaughter Beach, Delaware. At Buckroe Beach, 500 feet of partially submerged sandbag sill was placed about 150 feet from shore. The filled bags measure about 4 by 7 by 1.7 feet and are assumed to be either Dura-Bags or Advance Bags that were originally placed end-to-end to contain a perched beach between flanking groins. No filter cloth was used under the bags. Site photos show that the bags have not been damaged, but some are separated by 2 or 3 feet, and little fill has been retained (Fig. 3-53).

At Basin Bayou, a low breakwater built with 100-pound Sand Pillows, 100-pound Advance Bags, and large nylon Dura Bags was constructed 200 feet offshore in about 3 feet of water. Filter cloth was used under the bags at one half of each section. The 64-foot Advance Bag section was lost within the first few months, as a result of weak seams pulling apart and allowing sand to spill (Fig. 3-54). The pillows in the 64-foot Sand Pillow section remained intact, although some were dislodged by wave action. The displacements were facilitated by slippery algal growth on the bags (reducing the bag-to-bag coefficient of friction) and by the relative lightness of the filled bags (Fig. 3-55). The Dura-Bag section remained unchanged, except for a tear in one unit, but the profiles show that the structure had no effect on the beach in its lee.

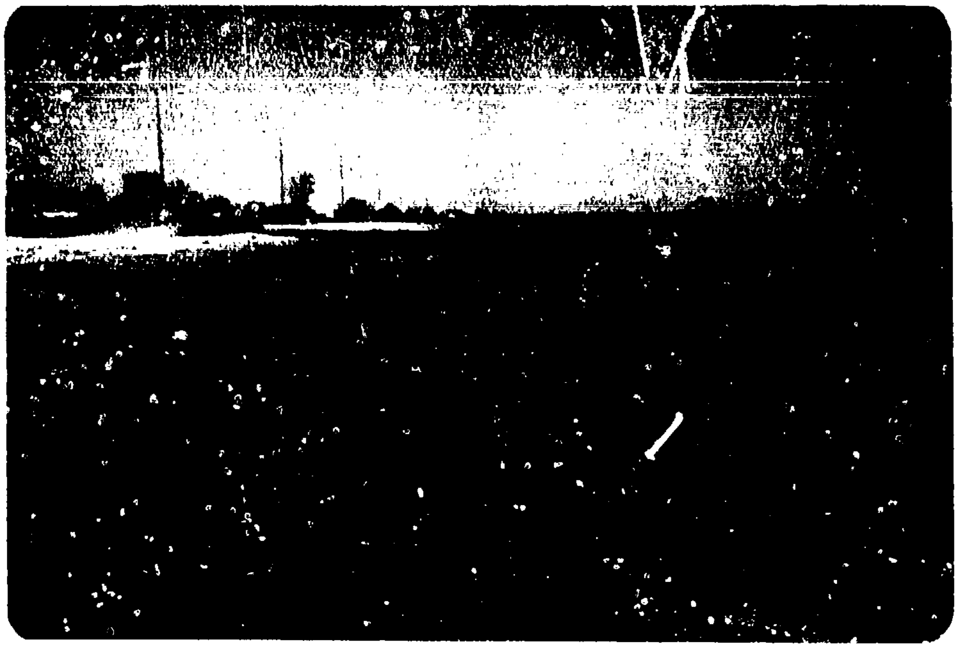


Figure 3-53. Sandbags in sandbag sill extending from groin 5 to groin 4 have separated, Buckroe Beach, Virginia, 12 March 1979.

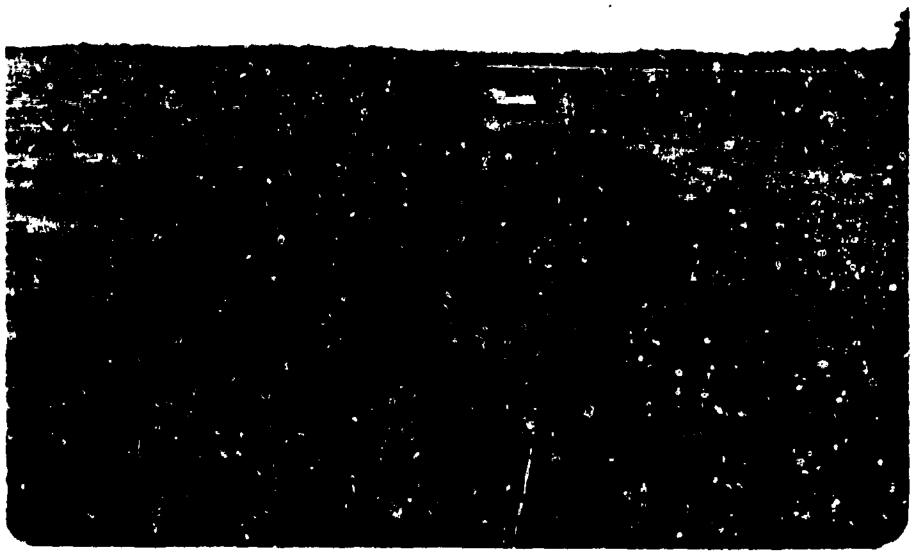


Figure 3-55. The Monsanto Sand Pillows have been displaced by wave action, facilitated by reduced bag-to-bag friction due to algal growth, Basin Bayou, Florida, 19 October 1979.

Large Dura-Bags or Advance Bags underlain with filter cloth were installed at the two Delaware Bay sites--Kitts Hummock and Slaughter Beach. The size of these bags (4 by 12 by 1.7 feet) necessitates filling them in place. Because they are so difficult to handle once filled, some bags in the lower tiers became and remained separated by gaps of 1 or 2 feet. The contractors filled the gaps with sand before placing the upper tiers or bags. At Kitts Hummock, the Advance Bags were stacked in a pyramid section, three bags high, to form a breakwater (Fig. 3-56). The top bags settled 1 to 2 feet, uniformly along the structure. Two of the bags were emptied as a result of seam failure, tears inflicted by floating ice or debris, or by the unraveling of their tied filler necks (Fig. 3-57). The profile surveys showed about 1 foot of accretion on the offshore bottom in lee of the structure but no significant beach changes. Apparently, the breakwater was functionally effective.

Sections two bags across, composed of Advance Bags and Dura-Bags, were used to form part of the sill and the returns at Slaughter Beach, which were built to contain a perched beach (Fig. 3-58). There was some misalinement of the bags during the monitoring period (Fig. 3-59).

Several lessons were learned from observing the construction methods and the performance of materials used in sandfilled-bag breakwater and sill systems:

(1) Displacement problems occur when lightweight 100-pound bags are exposed to even moderate wave climates. The larger Dura-Bag and Advance Bag are recommended for breakwater construction, even though they are more difficult to handle, require filling in place, and may settle.

(2) Care must be exercised during construction to maintain bag-to-bag abutment. Otherwise, gaps between the bags allow wave transmission through gaps and loss of sandfill from perched beaches. The practice of filling gaps between bags in the lower tiers with sand should be disallowed, as the loose sand is soon washed out, and partial collapse of the structure results.

(3) Placement of filter cloth under the bags is recommended to reduce settlement of breakwaters in soft bottoms. For sill construction, the filter cloth should extend up the shoreward face to the top of the sill to prevent the piping of sand through the spaces between bags.

(4) Wherever possible, a sand-epoxy coating should be applied to the outer surface of the bags. The resulting roughness increases the coefficient of friction, which helps prevent sliding of bag over bag. The sand-epoxy coating should also reduce damage from vandalism and from floating ice and debris.

(5) The filling neck of the large Advance Bag requires [illegible]ing off and tucking under for protection. In practice, the tucking under proved very difficult, unless the bags were underfilled. As a result, most necks were left exposed and

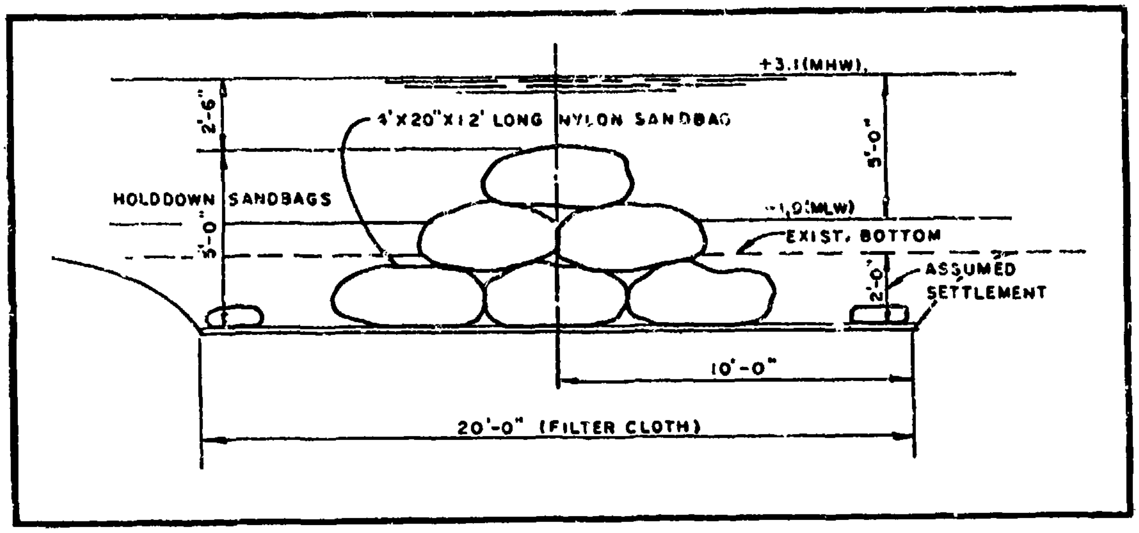


Figure 3-56. Typical sandbag-breakwater section, Kitts Hummock, Delaware.

Figure 3-57. Advance-Bag breakwater deteriorating due to loss of sand through open seams, Kitts Hummock, Delaware, 25 March 1980.

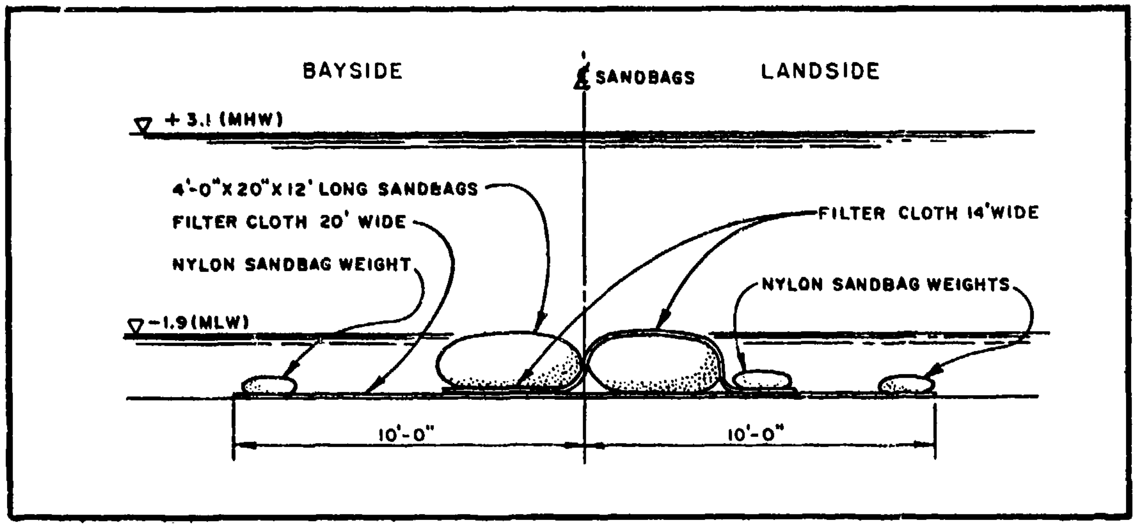


Figure 3-58. Typical sandbag section of sill at Slaughter Beach, Delaware.

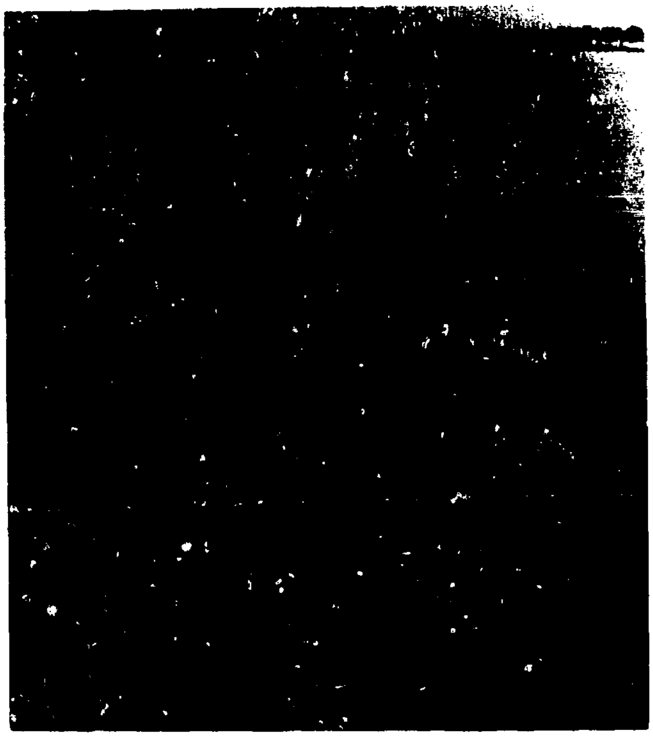


Figure 3-59. Misalinement of bags in sandbag sill at Slaughter Beach, Delaware, 6 August 1979.

soon became torn, spilling large amounts of sand under wave action. Seam failure was a problem with the Advance Bags. The Dura Bag is more expensive than the Advance Bag, but the lap-cover filling system seemed to work better, closing itself after the filling hose was removed and not requiring a tied closure.

d. Sand-Cement-Filled Bags. Sand-cement-filled bag sills or breakwaters were demonstrated at Alameda, California, and Fontainebleau State Park, Louisiana. Two low, sand-cement sills had already been constructed by others at Roanoke Island. Both are continuous, nearly submerged, narrow-crested mounds paralleling the shore, but because they were not monitored, evaluation of their performance was not possible.

At Alameda, devices 2 and 8 are sand-cement-filled bag sills; device 2 bags are nylon and device 8 bags are acrylic Sand Pillows. Device 2 is a single 150- foot-long row of abutted 9- by 3- by 2-foot nylon bags filled with a lean (8-to-1) sand-cement mix. The bags rest directly on the fine sandy bottom at about +2 feet MLLW, approximately 100 feet offshore. The bags were filled in place at low tide and supported by wooden side forms to achieve a rectangular section. Despite settling ranging from 6 to 12 inches (Fig. 3-60), the structure provided significant protection for the planting area in its lee. The differential settlement resulted in only one cracked block.

A 200-foot sill with 100-foot returns retains a perched beach as one of the features of device 8 at Alameda. The 100-pound Sand Pillows in this structure are also filled with a lean sand-cement mixture, but no forms were used. The sill was placed on Mirafi-140 filter cloth in a trench excavated to +1.5 feet MLLW in the fine sandy bottom about 100 feet offshore. The pillows were stacked about 4 feet high, two or three abreast, and then the sill was backfilled. The filter cloth was placed up the shoreward face to the top of the sill before backfilling. The seaward edge of the cloth, as actually installed under the Sand Pillows, extended past the toe of the structure and flapped with the wave action, inducing toe scour. There was some settling of the Sand Pillows and displacement of bags (Fig. 3-61).

During the monitoring period, both sill structures at Alameda experienced differential settlement. This is a frequently occurring defect in any filled-bag system. Differential settlement results in increased wave energy transmission over the low segments, with attendant damage to, or loss of sand from, structures or beach in the lee of the sill. A substantial amount of fill was lost from behind the Sand Pillow sill (device 8), but damage to planted areas behind the large nylon bags (device 2) was relatively small. No effective method of controlling differential settlement in soft-bottomed areas has been devised. A bedding layer of gravel and spalls over filter cloth might be effective, but this modification of the sand-cement bag sill and breakwater system was not explored in the demonstration program.

At Fontainebleau, a low breakwater 100 feet long (device 11) was constructed in September 1979 with 100-pound sand-cement-filled sandbags stacked in a trapezoidal section about 4 feet high and wrapped with Nicolon 66424 filter cloth over two-thirds of its length. An additional row of sand-cement-filled nylon-reinforced paper bags was then placed atop the filter

Figure 3-60. Differential settlement of device 2 (on the right), Alameda, California, 2 November 1979.

Figure 3-61. Settlement of Sand Pillows of device 8, Alameda, California, 14 November 1979.

cloth. Storm waves in April 1980 removed the top bags in the uncovered one-third of the breakwater. No other damage was reported by June 1980, but the profiles showed no accretion of the beach in lee of the structure. The monitoring period was not long enough to determine how long the filter-cloth wrap can survive the wave climate at this site.

The major advantage of using a sand-cement mixture is that the bags hold their shape after the fabric deteriorates. After pouring of the sand-cement, the bags harden into blocks, which are more durable than the conventional sand-filled bags. A sand-cement mixture is essential where the bag material is expected to be subject to excessive abrasion. However, the monitoring period was not long enough to determine whether the 8-to-1 sand-cement mixture at Alameda would remain intact for a reasonable length of time.

It was found to be advantageous to use the larger sand-cement-bags, such as were used in device 2. The larger sand-cement filled bags (device 2) performed better than the lighter bags (device 8). Using the larger size bags reduces the number of bag contact points where openings may develop. Also, the larger sand-cement blocks perform better simply because the additional weight of each module makes them more difficult to displace. Some of the lighter 100-pound bags at Alameda were randomly moved, opening spaces for the perched beach fill to wash through.

e. Timber Sheet Piles. A timber sheet-pile structure was demonstrated at one location only--Slaughter Beach, Delaware. This structure comprised the 330-foot center segment of the low sill constructed at the site to contain a perched beach fill. Interlocking 2-inch by 12-inch by 8-foot tongue-and-groove treated-timber piles were driven to grade and sandwiched between treated 2- by 12-inch top wales; three-quarter-inch galvanized through bolts at 3-foot intervals bind the wales to the sheet piles and provide longitudinal continuity. The top of the structure is only 6 to 10 inches above the water surface at low tide, and waves can overtop the sill at any stage of the tide. A cloth filter was stapled to the shoreward face in the south half of the timber segment before the perched beach fill was placed. Throughout the monitoring period, the structure remained unaltered, and no bolt corrosion was evident (Fig. 3-62). The cost of this structure slightly exceeded that of the concrete-box system and was almost twice that of the sandbag sill. However, it proved to be structurally superior to both, at least during the monitoring period, and functioned as intended. Further monitoring would indicate if the filter cloth is necessary. If not, the cost per linear foot would be reduced.

f. Gabions. A gabion breakwater was demonstrated at one site only--Geneva State Park, Ohio. The 100-foot breakwater, near the mouth of Cowles Creek, comprises baskets filled with 5- to 9-inch stone. Half of the baskets were PVC-coated and half were galvanized. Two-thirds of the structure was placed on filter cloth; the third at the east end was placed directly on the sandy bottom as a control section. The toe of the structure is protected against undermining by a row of mattress-type gabions that extend 7 feet beyond the main section.

Figure 3-62. Timber sheet-pile sill performing well at Slaughter Beach, Delaware, 6 August 1979.

During the monitoring period, the toe gabions were undermined and deflected downward. Many baskets broke open as the wire was stressed beyond design limits (Fig. 3-63). The galvanized baskets failed first, and the PVC-coated baskets followed. At the end of the first year, all but the end baskets in the toe structure had broken open and the stone was washed away. The main section, though somewhat deformed, remained intact except at the east end (Fig. 3-64). There, more than half of the baskets were open and without stone. Despite the extensive damage to that end, the structure performed well, trapping littoral material behind it. The experience at the Geneva site elicits the following comments and suggestions for future gabion-breakwater installations:

(1) The early destruction of more than half of the control section indicates that tight packing of the rockfill and use of filter cloth or a stone bedding course is needed to prevent settlement of a gabion breakwater into a soft bottom.

(2) The design of the toe protection mat is effective. As toe scour occurs, the mattresses drop into the scour trench, protecting main structure. The demonstration showed that the method is adequate but that deterioration of the protective mattress was too rapid.

Figure 3-63. Open baskets of gabion breakwater at Geneva State Park, Ohio, 1 November 1979.

Figure 3-64. Deformation of east end of breakwater at Geneva State Park, Ohio, 1 November 1979.

(3) This particular system should be reserved for milder wave climates and more resistant bottom formations than those found at the Geneva State Park site. In future installations, the following procedures are recommended:

(a) Select baskets with wire diameter and yield strength appropriate for gabion construction at the particular site;

(b) select the PVC-coated wire option to prevent corrosion;

(c) place the toe protection mat in a trench that has been excavated below the anticipated scour depth; and

(d) place a riprap in front of the toe protection mat for additional protection.

(4) The contract cost of $350 per linear foot was unusually high because of high rockfill costs and because of contract labor costs. Also, construction under water greatly amplifies costs; e.g., the breakwater at Geneva was built in 3 feet of water. Where low-cost rockfill is available, the individual property owner may be able to install a gabion breakwater for a fraction of the cost incurred at the Geneva site.

The added cost of these suggested improvements is recognized. If they can not be justified at a given site, other protective devices should be considered.

g. Z-Wall. This type of breakwater was demonstrated at one site only--Geneva State Park, Ohio. The structure is intended to be erected close to shore on the existing bottom without use of a filter material of any kind. When placed on a sandy bottom, the outboard edges of the panels tend to sink deeper into the sand, causing the structure to lean lakeward, sometimes nonuniformly, with excessive stress on the connecting hinge bolts. Pressure of panel against panel results in spalling of the concrete. This occurred at the demonstration site, with progressive loss of end panels (Fig. 3-65). During the monitoring period, however, the structure performed well functionally, and was still maintaining a large accretion in its lee at the termination of monitoring, despite the loss of three outer panels. Just how long the structure will continue to survive is a matter for speculation. At the present rate of loss of end panels, its longevity appears to be only 5 or 6 more years without maintenance. The panels are too heavy for filter cloth to be used, and it is doubtful that a granular filter would prevent the nonuniform settlement that is causing structural degradation. The results of the demonstration were considered site-sensitive only to the extent of the leaning problems induced by the sand and gravel mantle on which the structure rests. Had this mantle been thinner, or had the structure been founded on bedrock, the results might have been different.

The relatively high cost of the structure, $206 per foot, virtually places it out of the range of low-cost construction. One of the reasons for

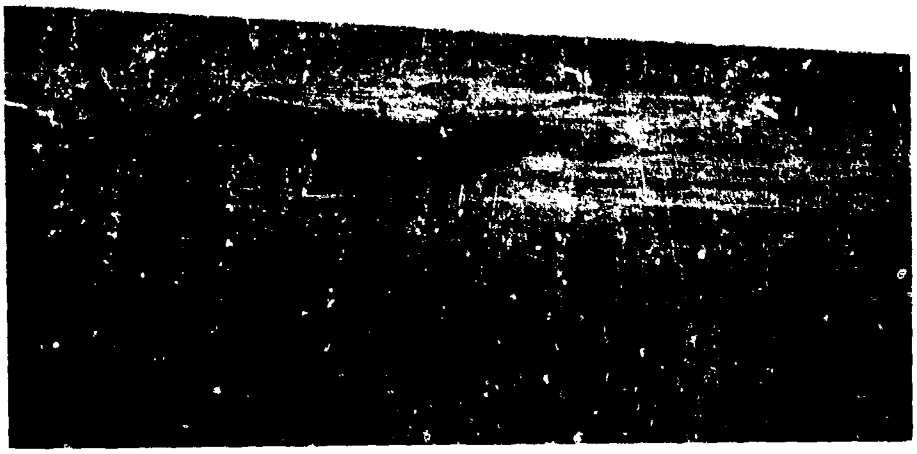


Figure 3-65. Gap between panels at eastern end of Z-wall breakwater, Geneva State Park, Ohio, 6 September 1979.

this higher cost is the need to place the panels by crane and to bolt them together in the water. Also, the 6-foot height of the panels limits their use to relatively shallow water. Nevertheless, the cost is not excessively large, relative to other structures that must be built offshore. Such structures are considerably more expensive than nearshore or onshore structure, and their costs highlight the need to use bulkheads, revetments, and groins instead of breakwaters to keep costs low. However, where an abundance of littoral drift is moving predominantly in one direction, the groin effect of the bulge of sand trapped in the lee of the breakwater may protect the shoreline for a considerable distance updrift of the structure. Thus, the cost of the structure should not be related to its length, but to the length of shoreline it protects.

h. Sta-Pods. Only one Sta-pod breakwater was monitored--one of the three breakwater sections at Geneva State Park, Ohio. At $116 per linear foot, this was the least costly of the three breakwater devices installed at the Geneva site. The reason for the relatively low cost was that the units were cast at the manufacturers site nearby, under controlled conditions, and were delivered by truck. The units were placed in juxtaposition by a crane. No coupling units were required except for a wire rope strung through the lift rings of the westerly 13 Sta-pods on completion of placement to monitor individual stability of the modules.

Performance was rather poor, as the units could not be placed close enough together to screen out a significant amount of wave energy. During high waves, enough wave energy was transmitted through and over the structure to erode the beach severely in its lee. Nevertheless, some accretion did occur in lee of the structure during milder wave periods, and only near the end of the monitoring period did the shoreline recede to its preconstruction position. The units all remained erect (Fig. 3-66) and, after initial settlement, sunk no farther into the bottom. As noted in the site analysis, some type of interconnecting baffle might be devised to reduce the permeability of the system. However, the increased wave force on the structure might then be sufficient to overturn it during high wave episodes. Also, as discussed under site analysis, the rapidity of placement could make the Sta-pods useful for emergency protection. More research and monitoring are needed to evaluate this device as a reliable shore protection system.

i. Surgebreaker. A relatively new modular shore-front device known as Surgebreaker was demonstrated under the moderate wave conditions at Basin Bayou, Florida. It is constructed with 3,700-pound precast concrete triangular modules. A helicopter was used to install the 200-foot breakwater by placing the modules end-to-end on the bay bottom. The manufacturer and designer of the Surgebreaker recommends installation of the device without a bedding layer or filter cloth because he says the design is intended to uniquely obviate the need for either. This recommendation was followed. Construction was completed in November 1979. During the monitoring period, the breakwater remained structurally sound and functioned as intended (Fig. 3-67). Wave action in its lee was significantly reduced. Because the performance was observed for only a short time period, no conclusions regarding its long-term performance can be reached.

j. Sandgrabber. This patented configuration of tie rod-connected construction blocks was used as a breakwater at four sites--Basin Bayou, Florida; Kualoa, Hawaii; Bellows Air Force Station, Hawaii; and Folly Beach, South Carolina. The Sandgrabber is an accretion device that allows for some differential settlement of the blocks by using U-shaped, galvanized-steel connecting rods. The hollow blocks allow waves to wash sand through the structure, trapping the coarser, water-borne particles behind the structure. This action normally maintains a differential of about 2 feet between the elevation of the sand berm behind the structure and the beach fronting it. If the beach is naturally accreting, accumulation continues until the structure becomes buried. If the beach is eroding, the Sandgrabber will be undermined, and its front face will drop to a lower level. The back berm may remain

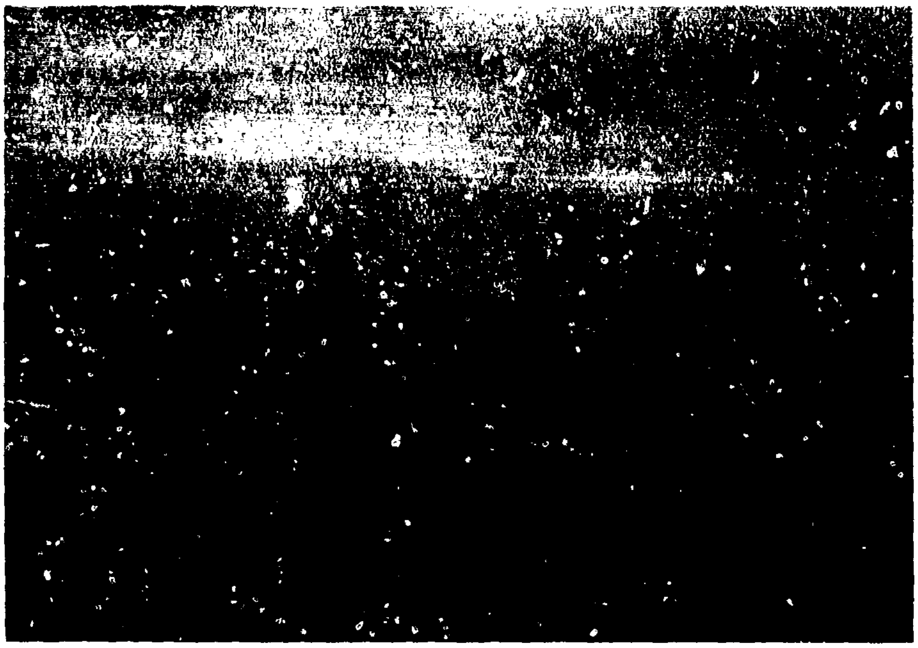


Figure 3-66. The Sta-pods at Geneva State Park, Ohio, remained stable, 1 November 1979.

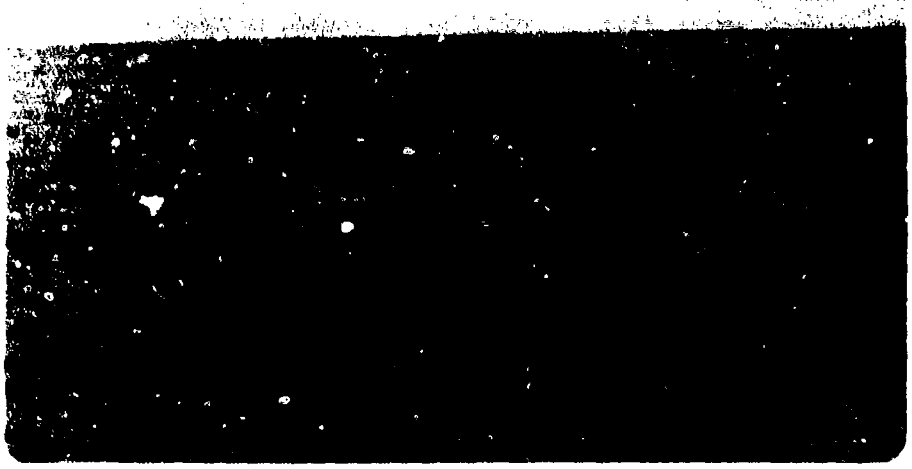


Figure 3-67. The Surgebreaker breakwater functioned well at Basin Bayou, Florida, 20 February 1980.

stable in a mild wave climate, but high waves will overtop the structure and wash sand from the back berm into the sea. Under the naturally eroding condition, the littoral drift first feeds the Sandgrabber, then bypasses it to feed the downdrift beaches. This general pattern was repeated at all of the installations in varying degrees, depending on the nature and intensity of littoral transport at each site.

The Basin Bayou and Hawaii Sandgrabbers accumulated sand quickly (Figs. 3-68 and 3-69), while accretion was slow at Folly Beach. A number of common problems developed at each of the project sites. The first, which deals with downdrift erosion, was readily observed at the Kualoa site. To the extent that the Sandgrabber withdraws sand from the littoral budget, it deprives the downdrift beaches of their normal sand nourishment. This results in accelerated erosion downdrift of the structure, with the rate depending on the volume of sand in littoral transport and its speed in the downdrift direction. The littoral system at Kualoa was already deficient when the Sandgrabber was installed. The structure managed to halt the erosion at the site, but at the cost of excessive loss of downcoast beach (Fig. 3-70).

Structural degradation of the Sandgrabber usually begins with toe scour. As sand is lost, the seaward side of the structure rotates downward. If the toe scour is uniform and the structure rotates as a whole, no damage occurs. If scour is not uniform, the differential rotation from block to block results in some blocks being displaced in the section. The U-ties were designed to allow for a certain amount of differential displacement; however, as the displacement exceeds the allowable, the stress of the U-ties against the concrete cracks or breaks the blocks. As weak concrete will hasten this block breakage, compressive strength tests should be performed on each batch of blocks before construction to assure that they meet building block standards. Block breakage occurred at each of the project sites, but the damage was severe only at Folly Beach, where more than half of the blocks in the seaward row of the top tier were lost (Fig. 3-71).

Several precautions are suggested to reduce block breakage. Periodic tie-bar adjustment might help prevent breakage in zones of differential settlement. Installing the blocks in a trench excavated to the anticipated toe scour depth should prevent structure rotations. Elastic ties instead of the galvanized-steel ties might provide additional flexibility, allowing smooth transitions and conformations to grade changes. Synthetic, elastic ties of some type would also eliminate corrosion problems. Cloth or gravel filters were not tested, but properly installed under a Sandgrabber they might deter scouring and promote uniform settling. The flanking erosion at the south end of the Kualoa installation might have been prevented by gradually curving the end of the structure well back into the bank (Fig. 3-70). This would have necessitated some excavation, but it would have obviated the need for the supplemental tieback structure, which had to be added later.

A tandem placement of Sandgrabbers tested at Folly Beach was effective in accreting sand between the structures. This arrangement works best at locations with large tidal ranges. However, the doubled cost of such an installation probably makes it less economical than a number of available alternatives monitored in this program. The tandem placement resulted in the seaward structure being destroyed by the more turbulent wave action at the

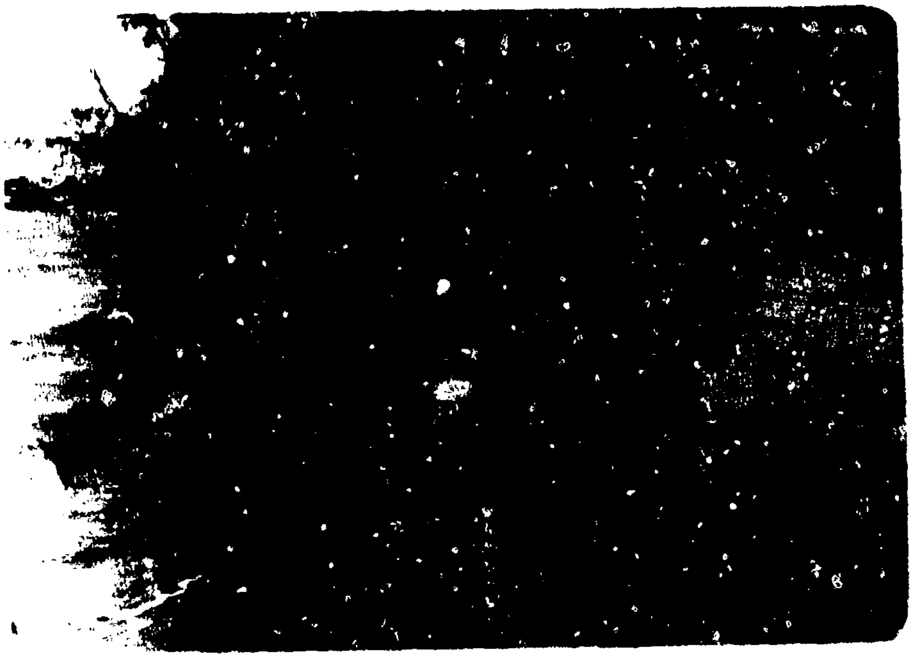


Figure 3-68. A view from middle of Sandgrabber, looking west; note the large accumulation of sand 4 months after construction, Basin Bayou State Recreation Area, Florida, 25 January 1979.

Figure 3-69. Sand accumulation behind Sandgrabber at Bellows Beach, Hawaii, 8 November 1979.

Figure 3-70. The eroded shoreline south of the Sandgrabber at Kualoa, Hawaii, 5 September 1979.

Figure 3-71. The top tier of the seaward row of the Sandgrabber in area 3 lost over half of its blocks, Folly Beach, South Carolina, 5 March 1980.

lower elevation on the beach. The landward structure survived with little damage and performed as intended. It appears that some type of sturdy sill structure would have worked better than the Sandgrabber at the lower elevation. The Sandgrabber appears to work best in mild wave climates where wave heights do not exceed about 3 feet. Impact and uplift forces of higher waves soon destroy the Sandgrabber as currently designed.

k. Stone Rubble. A low breakwater constructed of quarrystone was used at one site only--Kitts Hummock, Delaware. However, a longitudinal groin at Siuslaw River, Oregon, acted as a breakwater and is discussed here also. The 330-foot structure at Kitts Hummock is founded on a soft mud bottom at about -2 feet NGVD with an assumed settlement to -4 feet NGVD. The 5-foot-wide crest is at an elevation about midway between high and low tides. The mean tidal range is 5.2 feet. The southern half of the structure comprises 0.75-ton stone on Mirafi-140 filter cloth.

The northern half of the Kitts Hummock breakwater settled almost 6 inches after construction, but the southern half did not. This indicates that filter cloth was more effective than matstone in preventing structure subsidence at this site. Further monitoring is needed to confirm this. No displacement of stones was apparent during the monitoring period. This is not surprising, as 0.75-ton stone on a 1 on 1.5 slope should be stable for waves under 6 feet high. The profile surveys showed about 1 foot of accretion on the offshore bottom in the lee of the structure but no significant beach changes. Apparently the breakwater was functionally effective. However, the cost of this breakwater, $212 per linear foot, is quite high for a presumably low-cost structure.

The longitudinal groin at Siuslaw River built in 1974 to +12 feet MLLW is in effect a full-height breakwater. Although designed primarily as a training dike to prevent meandering of the river channel, it also prevents bank erosion due to low waves and has performed successfully without structural damage (Fig. 3-72).

l. Concrete Boxes. Concrete boxes were used as low breakwaters and sills at two demonstration sites--Kitts Hummock and Slaughter Beach, Delaware. The walls and bottoms of the precast boxes are 6 inches thick. The boxes at Kitts Hummock (breakwater) measure 7 feet long by 5 feet wide by 4 feet high and the boxes at Slaughter Beach (sill) measure 7 by 5 by 2 feet; the boxes are without covers. After placement the boxes were filled with clean commercial sand. Filter cloth was mistakenly placed under 50 feet of the Kitts Hummock structure, but the boxes were not placed uniformly or symmetrically along the centerline of the cloth. Rubble toe protection was added to the southern half of the 330-foot breakwater which was completed in November 1978. Inspections following winter storms revealed that the boxes in the northern half of the structure had shifted a few feet, and some boxes had tilted bayward as much as 13 inches, probably because of offcenter placement on the filter cloth and lack of toe protection (Fig. 3-73). Only 6 to 12 inches of fill remained in the boxes. The profile surveys showed about 1 foot of accretion on the offshore bottom in lee of the structure but no significant beach changes. Apparently the breakwater was functionally effective.

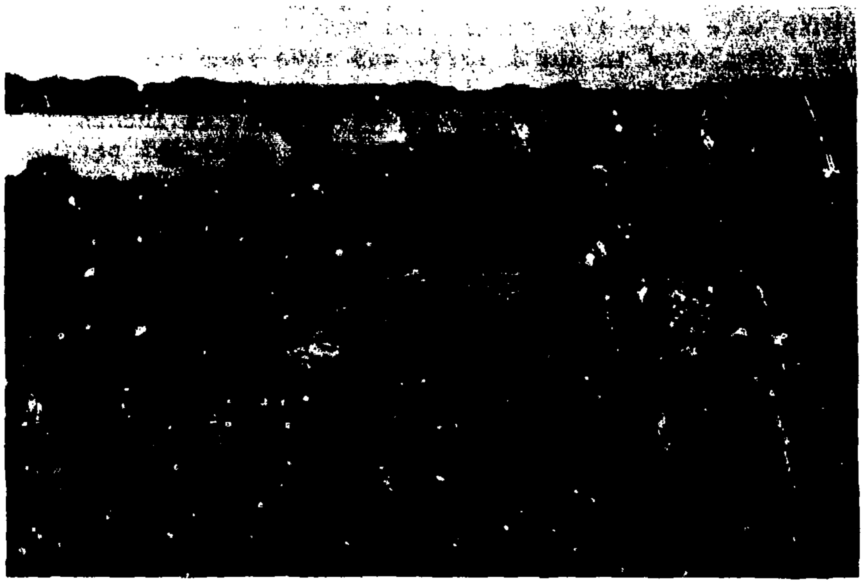


Figure 3-72. Longitudinal groin built as a training dike also acts like a breakwater to prevent bank erosion, Siuslaw River, Oregon, 13 December 1979.

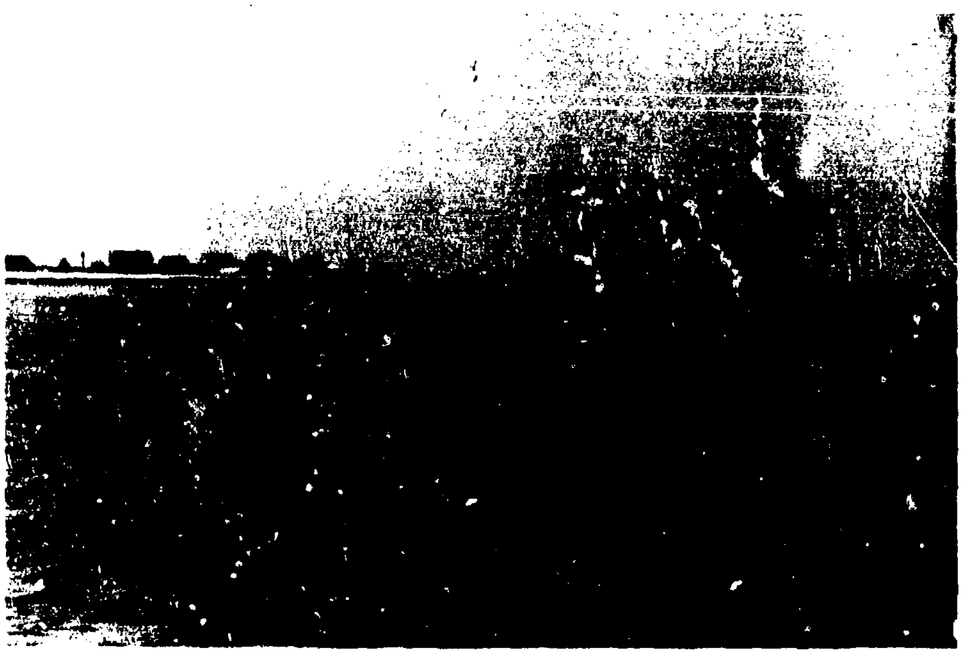


Figure 3-73. Settled concrete boxes at Kitts Hummock, Delaware, 24 April 1980.

At Slaughter Beach, a 330-foot sill and a 300-foot return to the beach were built with concrete boxes to retain a segment of manmade perched beach. The boxes at this site were also sandfilled and no filter cloth was used. Construction was completed in April 1979, and then sand was hydraulically pumped from an offshore deposit to fill the perched beach. By September 1979, before the perched beach was completely filled, 18 inches of berm was lost and a scarp formed. The scarping stabilized after the perched beach was completed. Two boxes at the north end settled slightly below the water surface (Fig. 3-74), exposing the beach in their lee to more wave action at high tides than had been anticipated. A number of conclusions were drawn from the monitoring at these two sites:

(1) Lids or sand-cement fill are needed to prevent sand loss from the boxes by washout.

(2) Better control of box placement is needed to close the gaps between boxes. However, wave transmission through these gaps during the monitoring period was not excessive. The gaps averaged 6 to 12 inches, which was tolerable at the Delaware sites but not where greater protection is needed.

(3) Rubble toe protection is necessary, as evidenced by the seaward tilting of boxes at the northern end of the box sill at Kitts Hummock where toe rock was omitted.

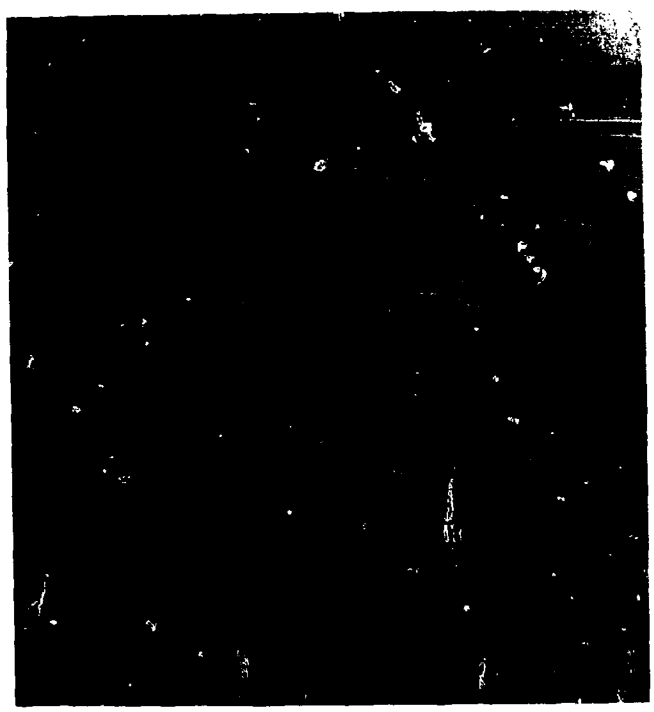


Figure 3-74. The concrete-box sill with returns in the background; the settled boxes allow wave action to penetrate the sill, Slaughter Beach, Delaware, 6 August 1979.

(4) The relative low cost ($130 per linear foot) of sandfilled concrete boxes for low breakwaters and sills appears to make this type of structure attractive for use under moderate wave conditions. Continued monitoring of the Delaware Bay installations is needed to determine the structural adequacy of the modular units and to evaluate the functional performance of concrete-box breakwaters and sills.

(5) The sill structures, much lower in height, allow more wave transmission and are not as functionally effective as breakwaters. They are intended primarily to prevent the seaward movement of retained fill. Breakwaters are intended to attenuate the waves and thereby accrete littoral drift or halt wave erosion in their lees.

m. Brush Dike. A breakwater constructed of creosote-treated posts driven into the offshore bottom, then tied across the top with timber cross-ties and filled with brush was installed at Fontainebleau, Louisiana, in the spring of 1979. By the spring of 1980 most of the brush had been washed out. Although the posts and cross ties remained intact, they had little effect on the beach. The system might be made successful by any or a combination of the following:

(1) Closer spacing of posts;

(2) longer sized pieces of brush; and

(3) sheathing along the insides of the rows of posts.

n. Profiles.

(1) Tires on Piles and Longard Tubes. Profile analyses of these structures were done at Fontainebleau, Louisiana; Alameda, California; and Basin Bayou, Florida. These profiles are shown in Figure 3-75.

At Fontainebleau State Park, Louisiana, a breakwater was constructed of closely spaced timber piles driven into the lake bottom with tires stacked 4 feet high on each pile. This device adequately attenuated waves, and performed without any structural deterioration. The profiles at this site are somewhat erratic, but they show some accretion in lee of the structure.

A Longard-tube breakwater was demonstrated at Alameda, California. This structure was damaged by vandals and eventually could no longer retain the manmade sand tombolo that had been placed in its lee. Construction of this structure was completed in October 1978. The profiles show accretion in lee of the breakwater while the structure was intact, from December 1978 to June 1979. After June 1979, corresponding with the degradation of the structure, erosion of the tombolo continued through the remainder of the monitoring period.

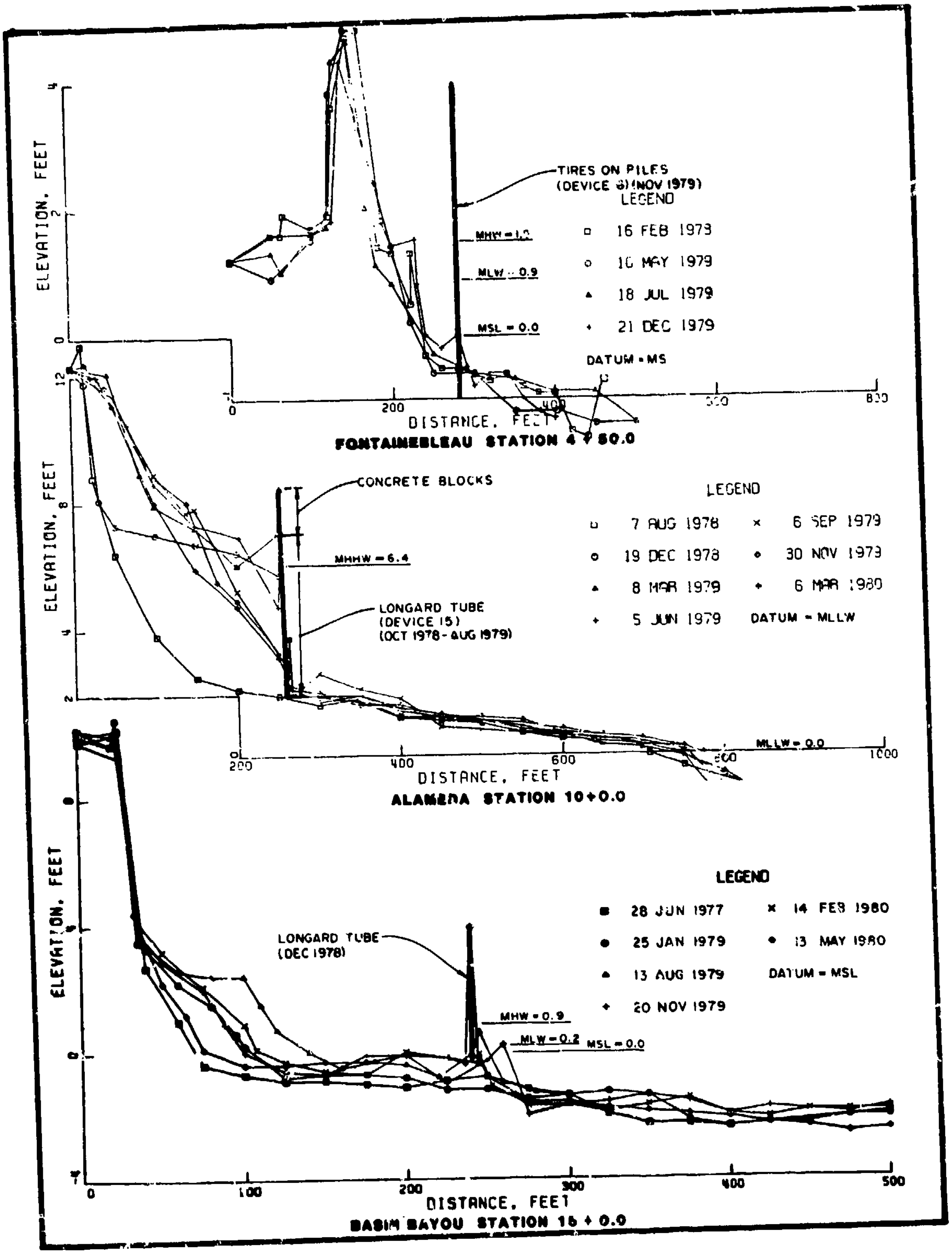


Figure 3-75. Profiles of tires on piles and Longard-tube breakwaters.

Another Longard-tube breakwater was demonstrated at Basin Bayou, Florida. This structure was also damaged by vandals, although not as badly as at Alameda. This damage halted the formation of a tombolo which had been formerly in the breakwater's lee. The profiles for this site show that progressive accretion was evident just behind the structure. On the beach, however, the trend was more erratic with accretion from August 1977 through August 1979 followed by erosion thereafter. A new Longard tube was placed at the site in May 1980, but as this was the end of the monitoring period its effectiveness cannot be determined. Because Longard tubes are susceptible to damage by vandals (which seems to be unavoidable), the stacked tire on pole breakwater appears to be a more effective device in protecting the beach from erosion. Longard tubes work well, but once damaged their lifetime is greatly reduced.

(2) Stacked Bags and Timber Sheet Piles. Profile analyses of these structures were done at Alameda, California; Slaughter Beach, Delaware; and Basin Bayou, Florida. These profiles are shown in Figure 3-76. A stacked-bag sill was demonstrated at Alameda, California, which consisted of sand-cement-filled bags made of nylon. This structure underwent some settlement, but proved to provide protection to the vegetated shoreline. The structure acted in conjunction with a groin. The profiles show accretion in lee of the sill, due to sand traveling over the groin being trapped by the sill.

At Slaughter Beach, Delaware, a timber sheet-pile sill was constructed and monitored. This structure remained structurally sound and unaltered during the monitoring period. The profiles for Slaughter Beach show that the sill stabilized the backing beach for the first 6 months of the survey period. By December 1979 the new profile established by the placement of the perched beach (November 1979) was shown, and the breakwater maintained the perched beach through to March 1980.

A stacked-bag low breakwater was demonstrated at Basin Bayou, Florida. Due to growth of algae on the bags, the bags were displaced. The displacement of the bags did not, however, have any effect on the beach behind the structure. The profiles show minor but progressive accretion behind the breakwater throughout the monitoring period.

All of these structures adequately maintained the beaches in their lee. The stacked bags at both Alameda and Basin Bayou suffered some damage, but this did not affect their performance appreciably.

(3) Stacked Bags. Profile analyses of stacked-bag breakwaters (sills) were done at Kitts Hummock, Delaware; Slaughter Beach, Delaware; and Buckroe Beach, Virginia. These profiles are shown in Figure 3-77.

At Kitts Hummock, the bags deteriorated somewhat, but the breakwater remained functionally effective. The profiles at this site show a progressive accretion in the lee of the breakwater.

The sandbag sill at Slaughter Beach performed reasonably well in retaining a perched beach, despite some misalinement of bags which occurred during the monitoring period. The profiles show that accretion was present behind the structure, particularly in the latter months of the monitoring period.

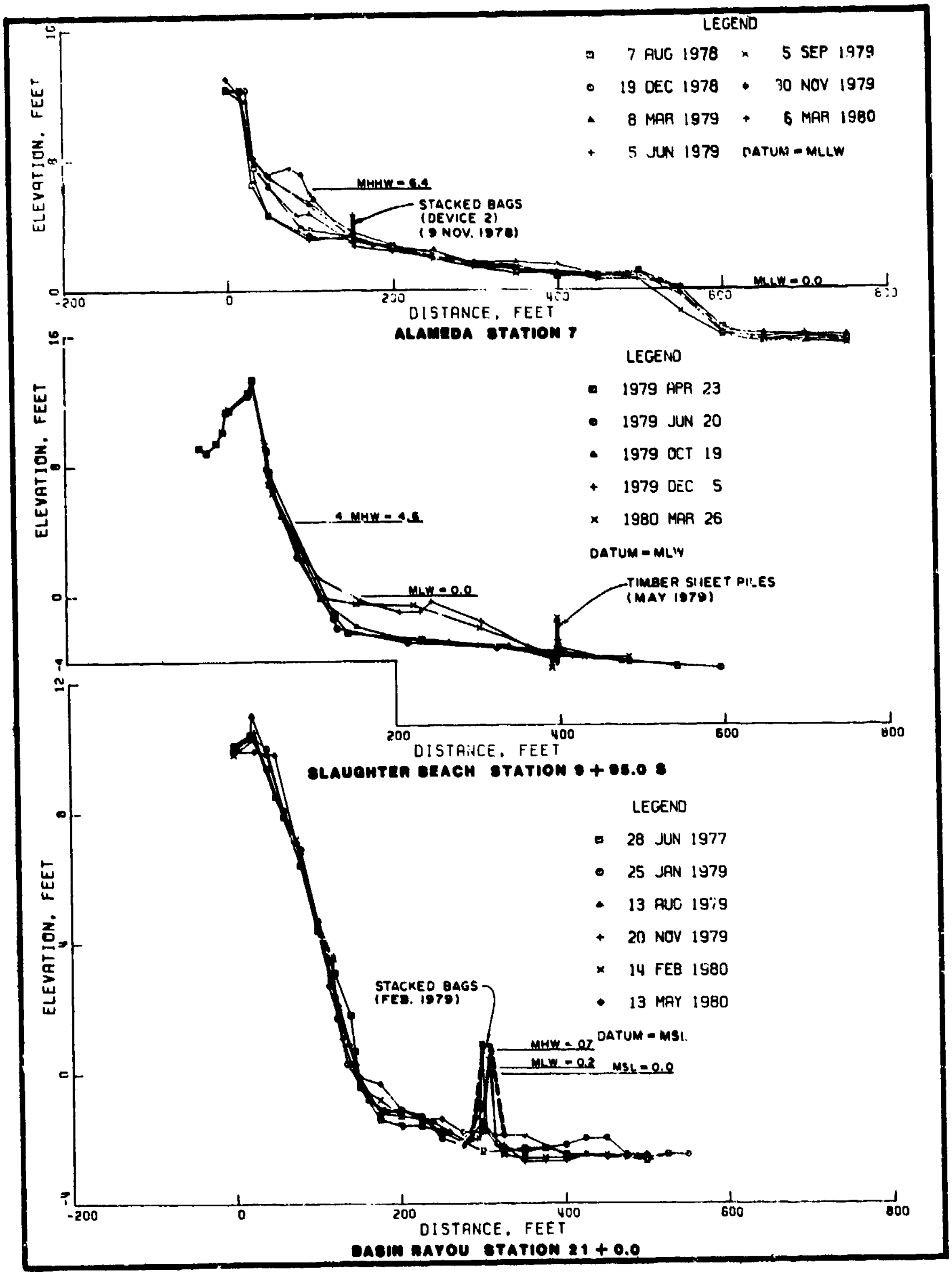


Figure 3-76. Profiles of stacked-bag sill, timber sheet-pile sill, and stacked-bag breakwater.

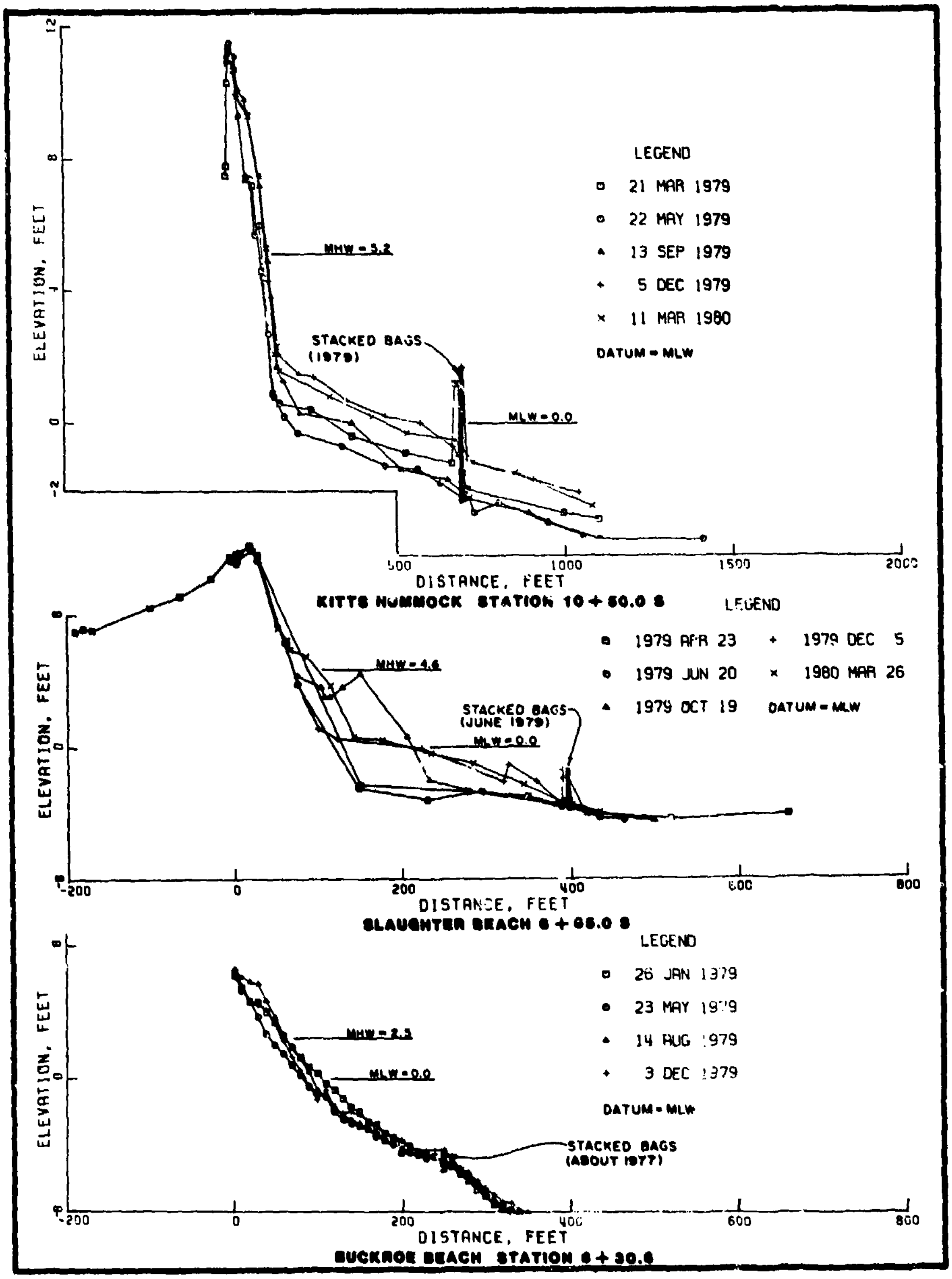


Figure 3-77. Profiles of stacked-bag sills.

At Buckroe Beach, the sandbag sill underwent some slight movement and separation of bags. This sill was placed between flanking groins to contain a perched beach. The profiles show that this structure had little effect on the beach in its lee.

The structures at Kitts Hummock and Slaughter Beach performed well, the latter undergoing deterioration but still containing the perched beach.

(4) Z-Wall, Sta-Pods, and Surgebreaker. Profile analyses of Z-wall, Sta-pod, and Surgebreaker breakwaters were done at two sites--Geneva State Park, Ohio, and Basin Bayou, Florida. The profiles for these structures are shown in Figure 3-78.

The Z-wall breakwater was demonstrated at Geneva State Park. Panels of this structure were broken off due to nonuniform settlement. The profiles show that this structure, despite damage, was successful in trapping sand.

The Sta-pod device was also demonstrated at Geneva State Park, but was not as successful. The problem was that the individual units could not be placed together close enough to screen out a significant amount of wave energy. During high waves, wave energy was transmitted through the structure allowing the beach behind it to be eroded. The profiles show that no significant accretion is present.

The Surgebreaker breakwater was demonstrated at Basin Bayou. This structure remained structurally sound throughout the monitoring period, and as the profiles show, some slight accretion was present.

The Z-wall breakwater at Geneva State Park was the most successful in trapping sand, but is prone to structural damage. The Sta-pod device appeared to be the least successful.

(5) Sandgrabber. A profile analysis was done on Sandgrabbers at three sites--Basin Bayou, Florida; Kualoa, Hawaii; and Bellows Air Force Station, Hawaii. The profiles for these sites are shown in Figure 3-79. At Basin Bayou, the structure suffered some damage, attributed to erosion of sand from under the toe causing seaward rotation of the structure and block breakage. The profiles show that sand has been trapped by the structure, but variations in the sand level behind the structure has also been present.

The Sandgrabber at Kualoa, although structurally damaged, maintained its shape while settling throughout the monitoring period. Although the profiles do not show it clearly, the Sandgrabber trapped sand both in front of and behind the structure throughout the period, but in doing so, caused considerable downdrift erosion.

The Sandgrabber at Bellows Air Force Station also trapped sand at the beach in its immediate vicinity. This structure, like the other Sandgrabbers, also suffered from toe scour and subsequent nonuniform settlement. The accretion present at the Bellows Sandgrabber (shown in the profiles) was not to the detriment of adjacent beaches as at Kualoa. This was due to the predominantly onshore-offshore littoral transport at the site.

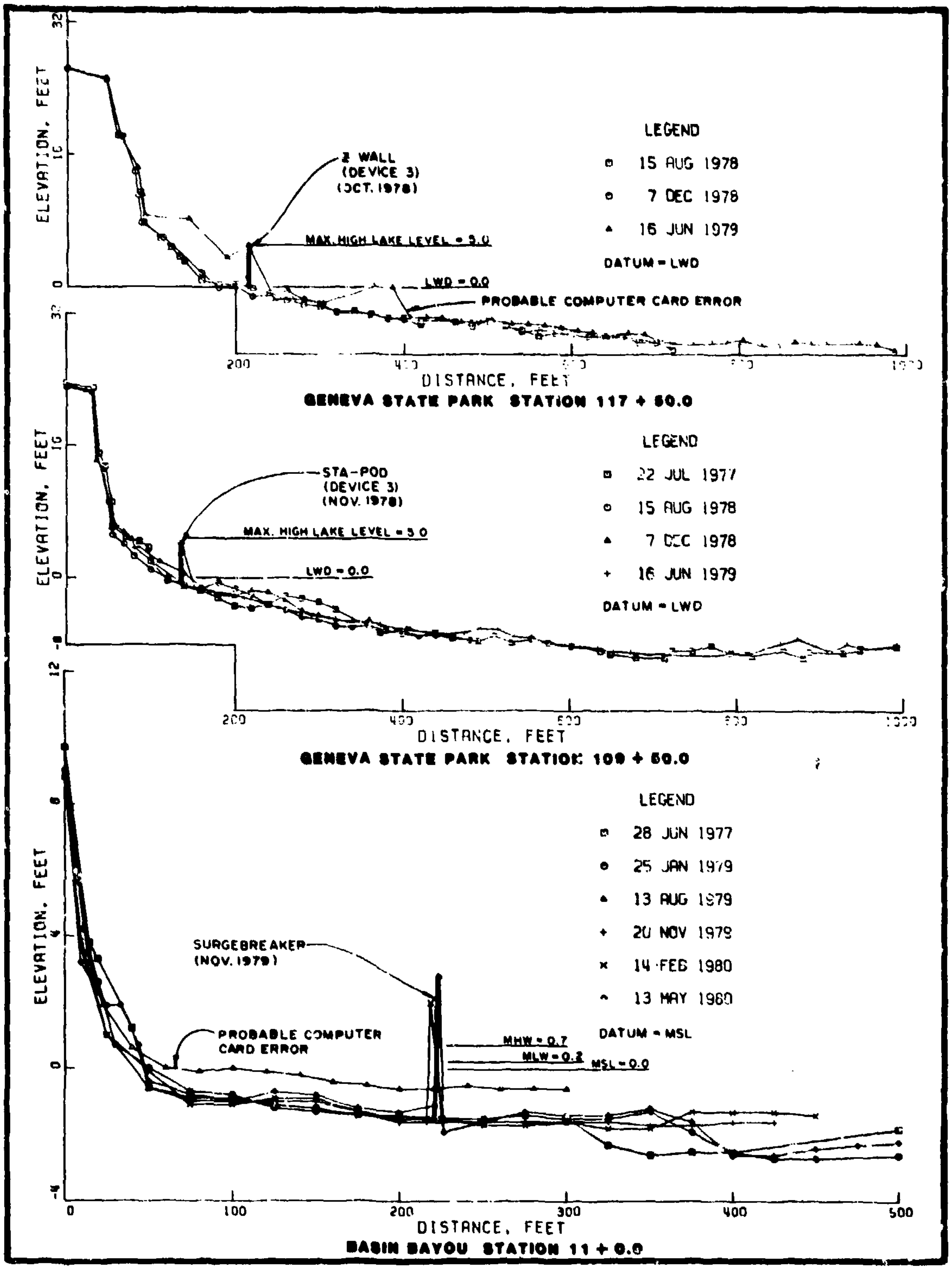


Figure 3-78. Profiles of Z-wall, Sta-pod, and Surgebreaker breakwaters.

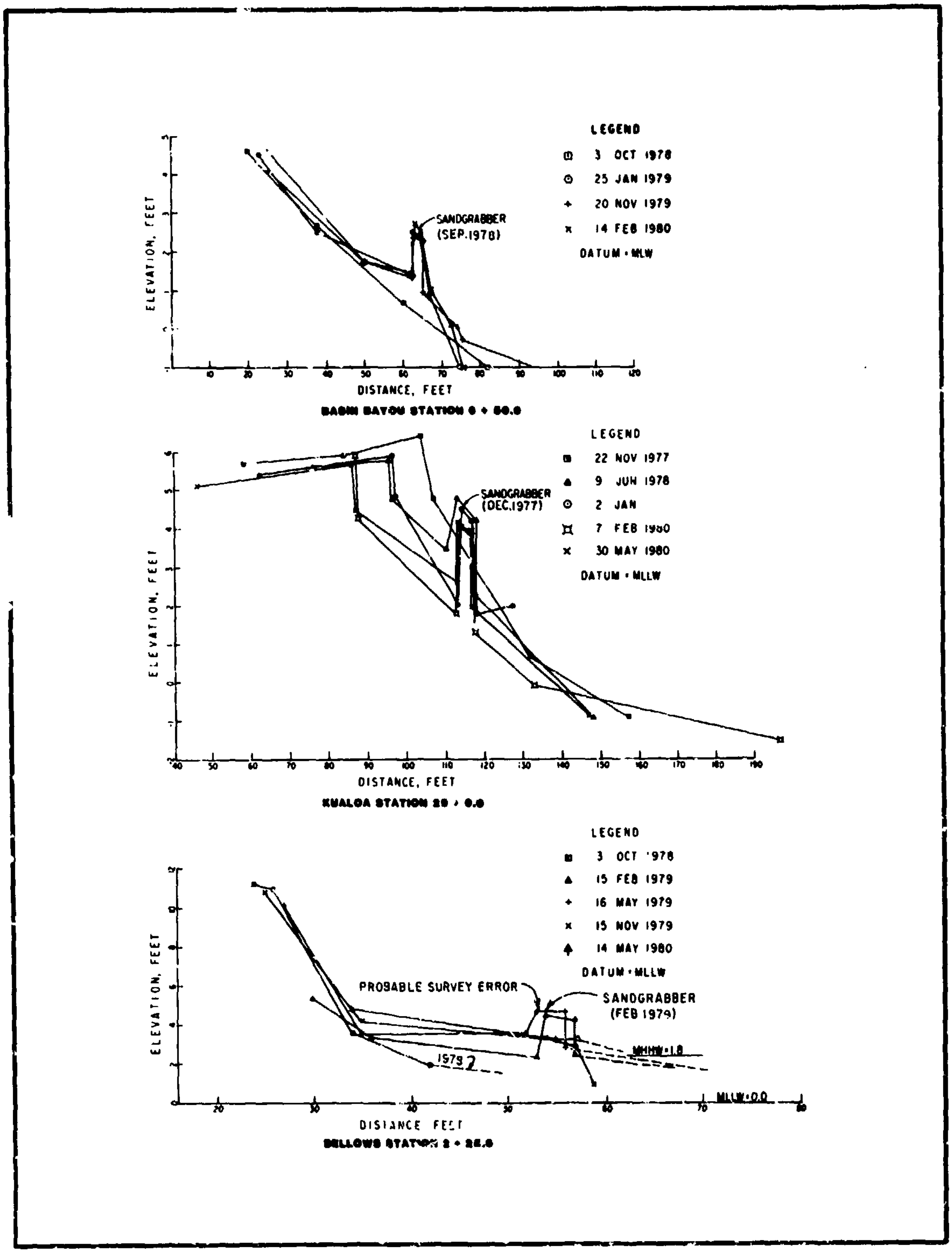


Figure 3-79. Profiles of Sandgrabbers.

The profiles show that the Sandgrabbers all suffered from differential settlement due to toe scour. The Bellows Beach Sandgrabber appeared to be most successful in that it did not cause downdrift erosion.

(6) General. The purpose in building a breakwater at a site is to attenuate waves and thereby induce accretion behind the structure. Sills are used primarily to retain fill material or perched beaches. Profiles taken over long periods of time will indicate the effectiveness of structures in fulfilling their purpose. In general, the profiles have failed to show significant buildups and have shown some trends of perched beaches being lost. The monitoring period has not been long enough to adequately define trends, but if given sufficient time, losses may result.

4. Groins.

a. Stacked Bags. Groins constructed with stacked bags were monitored at four sites--Bowers, Delaware; Sanilac Section 26, Michigan; Kotzebue, Alaska; and Alameda, California. At Bowers a 2,500-foot beach fill was contained between two groins constructed in about 1972 with large, sandfilled Dura-Bags. No data are available on the initial construction or performance, but the groins remain generally intact and were still performing fairly well in the spring of 1980. Some bags were torn and had lost considerable sand, but the cloth in undamaged bags was still in fair condition.

At the Sanilac Section 26 site, large 9- by 3- by 2-foot bags were filled and stacked to form a sandbag groin. The contiguous bags were interlocked by tying adjacent bag corners together. No filter material was used. Adjacent groins built with other materials were spaced three groin lengths from the sandbag structure. Since the construction of the sandbag groin in 1974, 15 feet of its lakeward end has been washed out as deteriorating bags were displaced, torn open, and emptied (Fig. 3-80). By 1980 the sandbag groin was still partially functional, trapping sand to form cusped, adjacent beaches.

At the Kotzebue site, smaller 100-pound acrylic Sand Pillows were stacked 6 feet high to form a short groin. This system failed during construction, as the bags were easily torn by floating debris and ice floes (Fig. 3-81). These lighter bags are also easily displaced by wave action. Smoothness of the bag surface facilitated bag-sliding, which hastened their displacement.

At Alameda, a 100-foot groin was constructed using large nylon bags. The bags were filled with a lean sand-cement mixture while supported between side forms, hardening into 9- by 3- by 2-foot blocks. Wear and deterioration due to exposure opened large holes in the nylon fabric, but the hardened blocks retained the structural integrity of the system (Fig. 3-82). The groin functioned so well that the scheduled beach nourishment was unnecessary.

It is clear that smaller, lighter bags are not appropriate for use as groins. The sand-cement mixture used at Alameda proved effective in preventing the structural deterioration noted at other sites where the bags were filled with sand; however, differential settlement as the result of foundation scouring should be anticipated where wave climates are more severe. The sand-cement bags might be displaced under such conditions and lose their functional

Figure 3-80. Deteriorating bags at Sanilac Section 26, Michigan, showing lakeward end washed out, 26 December 1979.

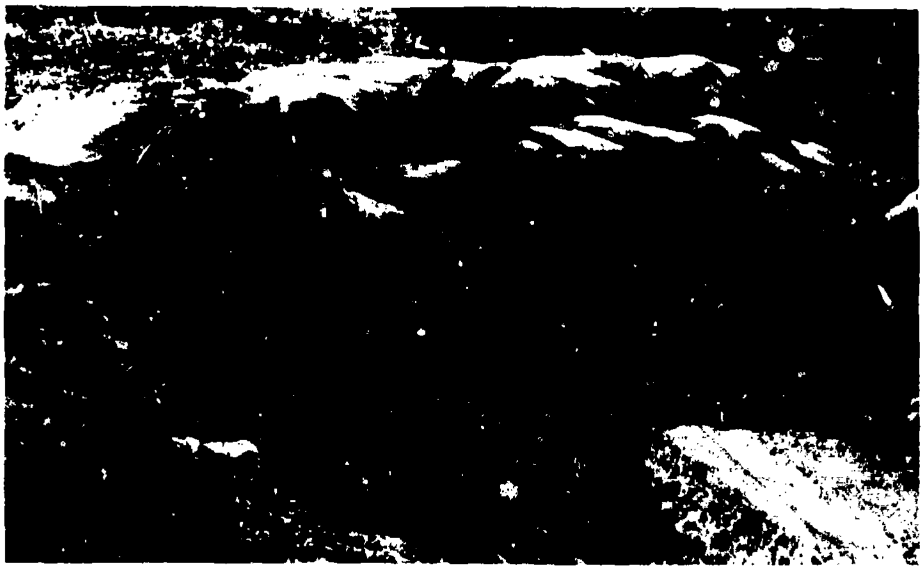


Figure 3-81. Torn bags of groin, Kotzebue, Alaska, 13 June 1979.

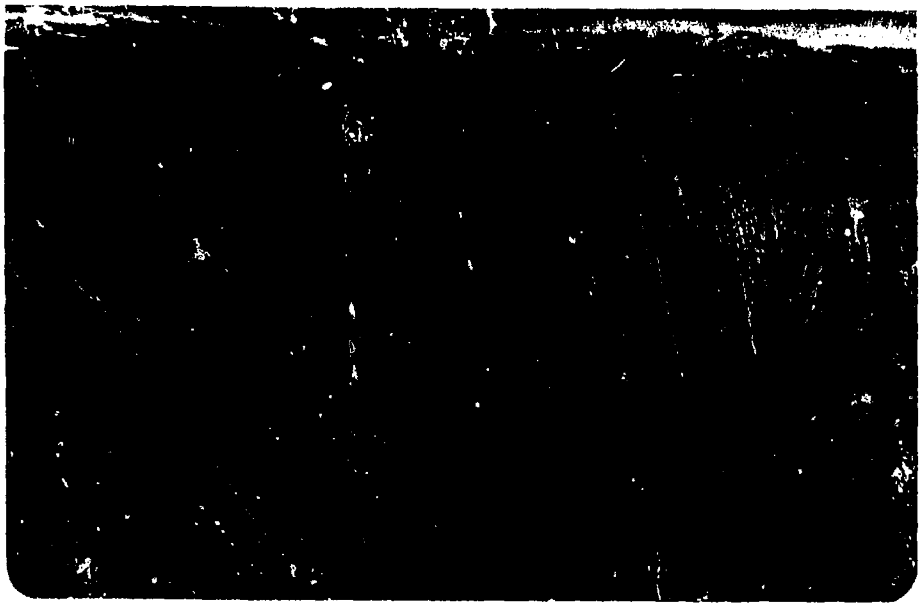


Figure 3-82. Wear and deterioration of material due to holes in the fabric; however, hardened blocks are retaining structural integrity, Alameda, California, 1 March 1980.

capability even when formed into blocks unless seated on a scour-resistant foundation.

b. Gabions. Gabion groins were evaluated at three demonstration sites--Kotzebue, Alaska; Sanilac Section 26, Michigan; and Ninilchik, Alaska. Two groins were built at the Kotzebue site. PVC baskets for the updrift groin were lined with Polyfilter-X filter cloth and filled with beach gravel and cobbles. The only structural damage sustained resulted from ice pressure causing rotation of the seaward end (Fig. 3-83). The baskets were not broken. The downdrift groin suffered the same shifting but damage was more extensive. Its baskets are lined with hardware cloth (galvanized wire screening). The lining separated at its seams, allowing the gravel and cobble fill to escape. This occurred in two of the seaward baskets (Fig. 3-84). No other damage occurred and both groins were proficient at trapping sand and causing sand fillets to form between them.

Of the six groins constructed at the Sanilac Section 26 site, only one used galvanized gabion baskets. Filter cloth was not used beneath the baskets nor was it used to line them when filled with large cobbles. Functionally, the groins performed much the same as those at Kotzebue. Sand was trapped, and beach fillets were formed between groins protecting the bluff toe (Fig. 3-85). Some scour beneath the seaward baskets facilitated the collapse of one of the baskets and subsequent loss of cobble fill.

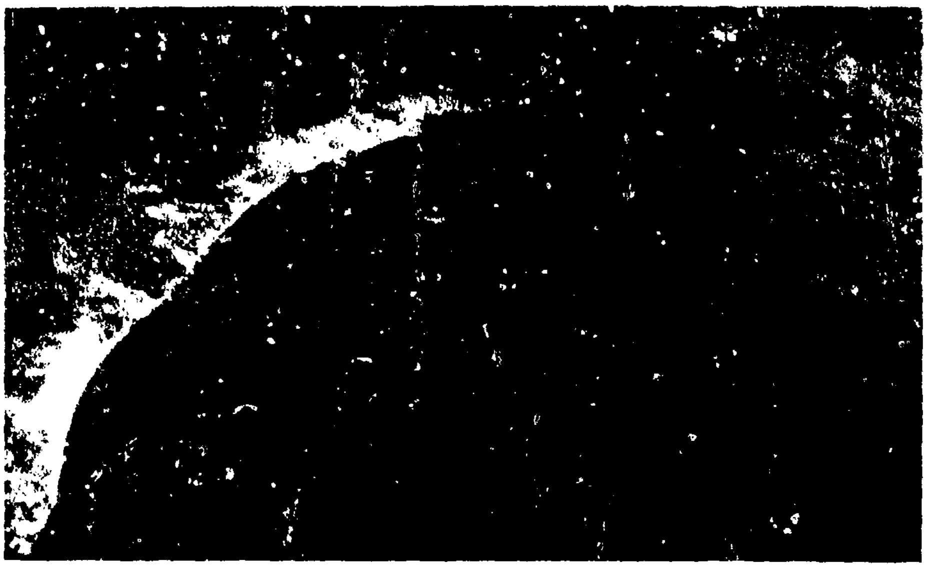


Figure 3-83. Gabion groin with Polyfilter-X filter cloth had its seaward end rotated from ice pressure, Kotzebue, Alaska, 13 June 1979.

Figure 3-84. A seaward basket that lost its fill, Kotzebue, Alaska, 13 June 1979.

Figure 3-85. Galvanized gabion-basket groin at Sanilac Section 26, Michigan, performed well functionally, but lost some fill due to a collapsed basket, 26 December 1979.

The gabion groin at Ninilchik used PVC-coated baskets filled with 4- to 8-inch cobbles. Winter and fall storms claimed the two seaward-end gabions and opened a 1-foot-diameter hole in the next basket (Fig. 3-86). Initially, the baskets were displaced, stretching the basket wires and allowing smaller cobbles to escape. As the void ratio inside the baskets increased, the retained cobbles were free to batter the wire walls, eventually rupturing the basket. Larger cobble sizes are recommended at this installation to prevent this occurrence. Despite the damage, functional performance was satisfactory. Accretion is seasonal, with drift material overtopping the shoreward baskets during the early summer months. The larger gravel washes away with the winter storms.

The gabion groin system is effective and moderately priced ($30 per linear foot at the Sanilac Section 26 site). Some suggestions to improve its performance are:

(1) The structure should be underlain with filter cloth to inhibit basket settlement and eventual basket rupture.

(2) All groin baskets should be PVC-coated to provide resistance to corrosion.

(3) More experience with inner liners (to prevent loss of fill material) is needed. A strong, woven filter cloth appears to be a good possibility. Galvanized

Figure 3-86. Storms claimed two seaward-end gabions of groin, Ninilchik, Alaska, 5 October 1979.

screening is not recommended as a lining because of its susceptibility to corrosion and its deficient seam strength.

(4) Gabion baskets must be fully filled to eliminate excessive deformation.

(5) Tiers of gabion baskets should be tied together with appropriately sized galvanized wire to prevent shifting of upper tiers over lower tiers. Baskets should be securely wired together as recommended by the manufacturer, and not partly so as was done with the demonstration devices.

c. Steel Fuel Barrels. The use of 55-gallon steel fuel barrels filled with gravel to form groins was demonstrated at one site only--Kotzebue, Alaska. Three 50-foot-long groins, each formed with a double row of barrels, were monitored. Each groin is tied perpendicularly to a short bulkhead comprised of a double row of barrels along the bank. These groins trapped a large amount of northward-moving littoral material and caused some erosion on their downdrift sides (Fig. 3-87). In general, the groins functioned as intended with little structural damage. The fuel-barrel groin system survived 3- to 4-foot storm waves and seasonal ice pressures with little damage. This performance, together with the economy realized by using discarded fuel barrels, identifies this groin system as an attractive shore protection device where littoral transport characteristics are suitable for shore stabilization with low groins and where the used barrels are plentiful and have no other salvage value.

Figure 3-87. A large amount of littoral material was trapped by each of the three barrel groins, Kotzebue, Alaska, 27 July 1979.

The following are suggestions for improving the system through lessons learned at Kotzebue:

(1) It is important to ensure that the barrels are topped off with gravel fill to protect them from ice-floe crushing or from damage due to floating debris. This is especially true for the seaward end barrels.

(2) Concrete caps are recommended for additional strength.

(3) The barrels should be entrenched sufficiently to prevent them from being undermined by scour on the downdrift side.

(4) Steel fuel barrels rust out rapidly in more temperate climates and should be used only in arctic regions.

d. Stone Rubble. Stone rubble groins were monitored at only one site--Siuslaw River, Oregon. One 1,200-foot longitudinal groin (actually a training dike) and five shorter transverse groins were installed to control westward river channel migration. Armor stone weighing 1.5 to 2 tons was placed on top of a core of 0.5-ton stone. No structural or material deterioration was observed during monitoring (Fig. 3-88). Functionally,

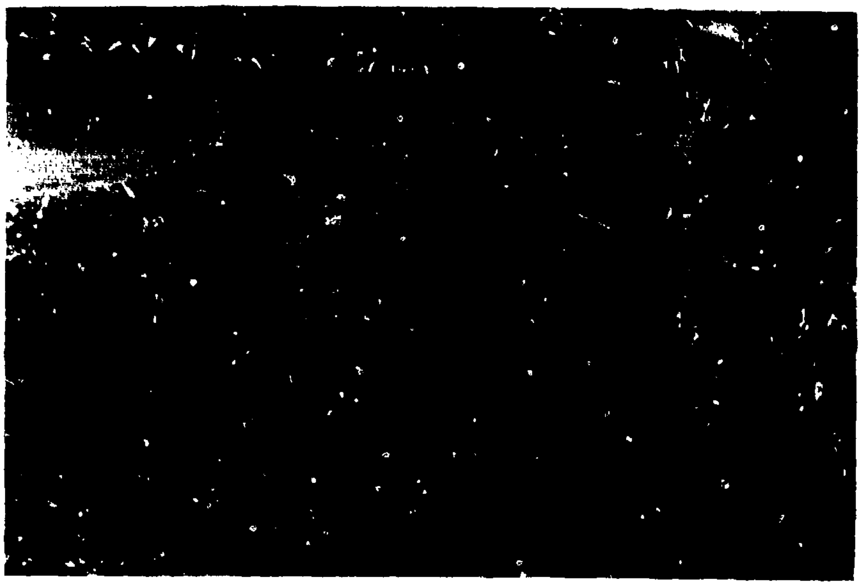


Figure 3-88. No structural or material deterioration was observed for the six stone rubble groins, Siuslaw River, Oregon, 9 April 1980.

the system performed as intended. Channel migration ceased and bank erosion that was threatening to breach the spit behind the longitudinal groin was halted. This area is now accreting sand and fillets between groins are forming. A stone groin was installed at the Stuart-Jensen Causeways site in spring 1980, but its performance was not monitored.

Structurally, stone groin systems are the most durable of those monitored. However, because of higher construction cost, stone rubble should be used only in the more severe wave climates at sites that are usually not included within the scope of this program.

e. <u>Longard Tubes</u>. Various sizes and combinations of sizes of Longard tubes were used as groins at three sites--Sanilac Section 26, Michigan; Lincoln Township, Michigan; and Ashland, Wisconsin. Two 40-inch-diameter tubes and a single 69-inch-diameter tube were evaluated at the Sanilac Section 26 site. The two 40-inch tubes were placed side-by-side, without filter protection, to form one groin. The tubes settled 1 to 2.5 feet, with the southern tube settling 1.5 feet below the northern tube, due to littoral transport. The southward transport direction causes material to accrete on the north side of the tubes while scouring the south side. The southern tube fell into the scour trench, resulting in the additional settlement (Fig. 3-89). Downdrift scour also induces sectional displacements as the tube is free to roll; this was common at all sites. At Lincoln Township, monitoring was discontinued because of tube failure. Tube failure initiates from ruptures and tears in the tube fabric, causing a loss of

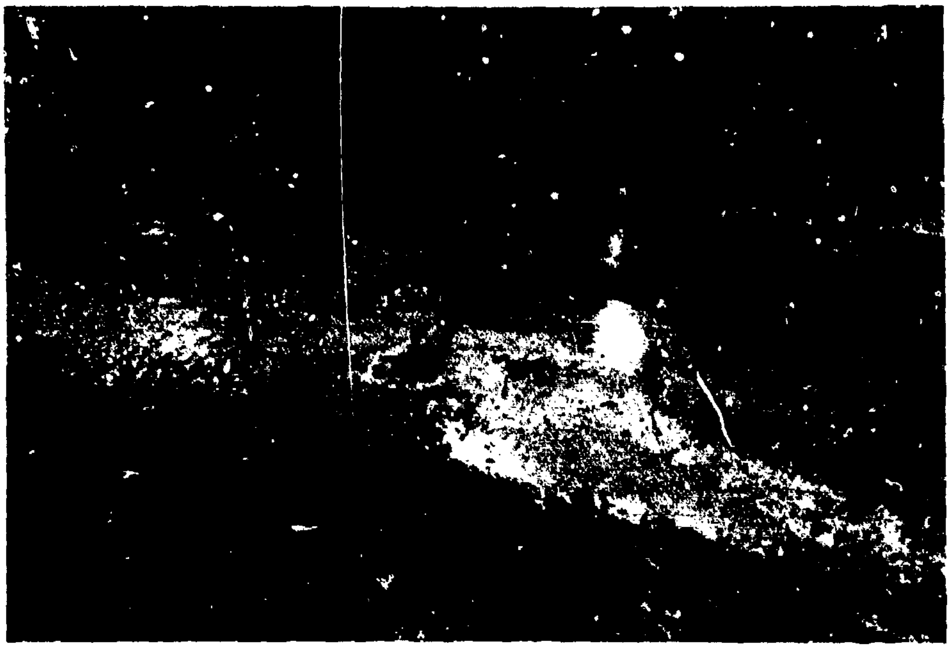


Figure 3-89. Twin 40-inch Longard tubes at Sanilac Section 26, Michigan, 27 November 1979.

sand; the section collapses and is usually buried. Within 3 years, the 40-inch tube at Lincoln Township failed. Tears opened after the tube settled 3 feet along its centerline. The ruptures were the result of collisions from floating debris and timber.

Four of the six 69-inch tubes at the Ashland site (groins 2 to 6) suffered similar damage; all had been torn and lost sand to the extent that they were no longer functioning by May 1980. Half of groin 6 was lost by September 1979 (Fig. 3-90). Total failure is usually prevented as the collapsed section is buried with littoral material; this keeps the tear from spreading. Groin 2 was vandalized by a shotgun blast which ruptured the fabric, thereby leading to the collapse of half of the groin (Fig. 3-91). Groins 1 and 3 lost considerable sand but were still functioning in June 1980.

Functionally, the performance of the Longard tube groins on the Great Lakes can best be described as mediocre. They did cause sand to accrete, forming fillet beaches as long as they remained functional, but the bluff toe was never adequately protected. The larger 69-inch tubes generally trapped more sand because of their additional height. Two suggestions are given to improve performance:

(1) Filter-cloth protection is necessary to inhibit downdrift scour. The method recommended should be similar to that used for the Ashland groin. They were placed on a

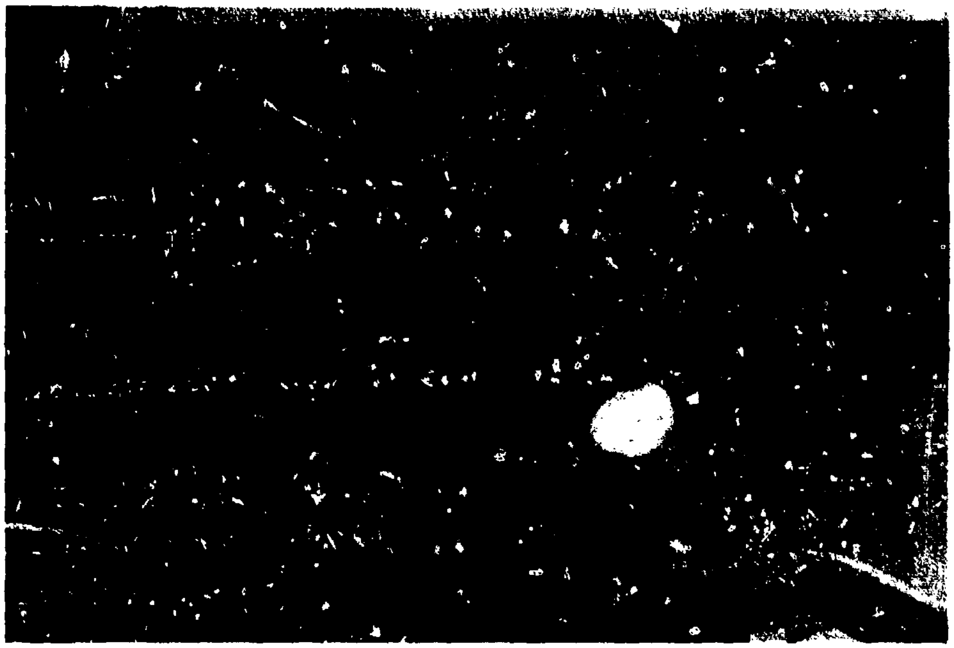


Figure 3-90. Half of groin 6 was lost, Ashland, Wisconsin, 26 September 1979 (structure in foreground is east seawall).

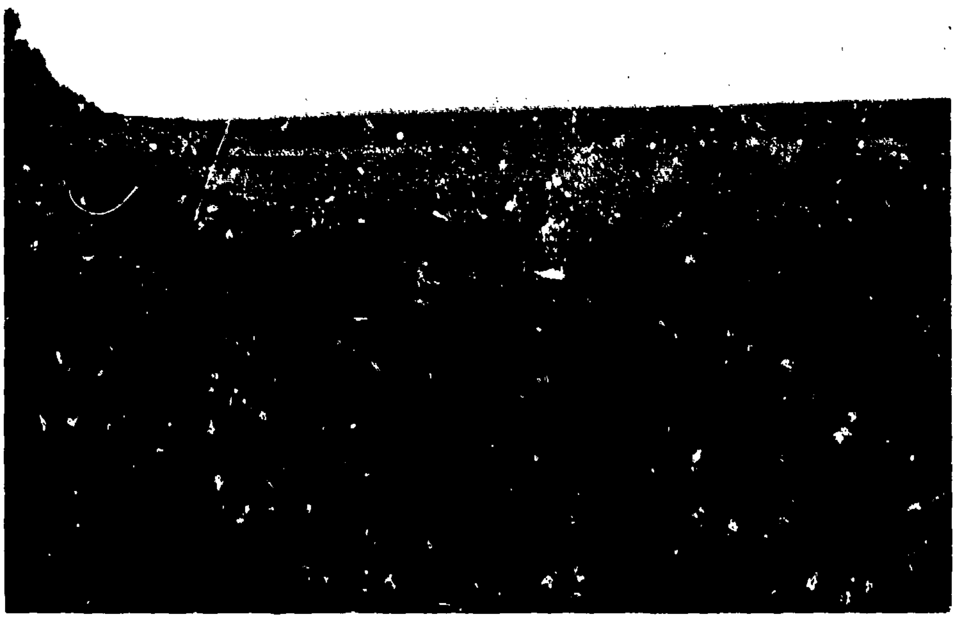


Figure 3-91. Groin 2, in foreground, was ruptured by a shotgun blast and half of the groin was lost, Ashland, Wisconsin, 16 August 1979.

7-foot-wide Polyfilter-X filter cloth with 10-inch sacrificial scour tubes factory-stitched to each side. As the sand beneath these tubes is scoured away, they fall into the trench and are buried, successfully tying down the filter cloth. The settlement problem was not evident at the site.

(2) It is true that failure is initiated by ruptures in the tube fabric; however, failure results from these punctures spreading. None of the groins were treated with sand-epoxy because such coatings must be applied dry. Coatings do not prevent ruptures, but they keep the fabric from unbraiding once it is ruptured. It might be helpful to coat all dry surfaces of the tubes for this reason. However, Longard tubes are not recommended at sites where excessive large pieces of flotsam or ice floes may batter shore structures.

The average cost of a 69-inch Longard tube without filter cloth was $71 per linear foot at Sanilac Section 26, and where groins are effective, each groin will protect about twice its length of shoreline. It is anticipated that the cost of the above improvements would not remove the Longard-tube system from the low-cost category.

f. Treated Timber. Treated-timber groins were monitored at Ninilchik, Alaska; Lincoln Township, Michigan; and Buckroe Beach, Virginia. At Ninilchik, five timber groins were evaluated: two existing timber-crib groins, and three with planking between piles. The timber-crib groins consisted of two parallel lines of piles that were embedded 8 feet deep; then the exposed tops were cross-braced together and sheathed with 3- by 12-inch timber planking. Gabions (1 by 3 by 9 feet) were installed along the sides of the southern groin to prevent erosion below the planking during winter months. The northern groin was left without flanking gabions as a comparative control device. The only damage sustained by either groin was at the seaward end of the northern groin. Late in the monitoring process one of the cross braces was shattered (Fig. 3-92). The coarse littoral material accumulated between the piles of timber-crib groins and added ballast to enhance the structural integrity of these structures. Erosion below the planks of the groins did not occur at either structure.

In the second type of timber groin at Ninilchik, single and double planking sandwiched between two close-set rows of piles were used with equal functional effectiveness (Figs. 3-93 and 3-94). Two groins with double planking were installed at the southern boundary of the project site, and a groin with single planking was built 350 feet downdrift. One seaward plank split; otherwise the groins remained unchanged. Overall, the five Ninilchik groins were equally effective, and the planked groins proved to be structurally as sound as the timber-crib system. Their presence indentified the seasonal nature of the littoral system. During the spring and summer months, the south sides of the groins accreted drift. The accumulation was nonuniform and sometimes overtopped the groins. Larger concentrations of drift also collected near the base of the log revetment behind the structure. Most of the drift was washed away during the winter months; coal and gravel deposits accreted about the groins at this time.

Figure 3-92. Shattered brace of groin 3, Ninilchik, Alaska, 28 November 1979.

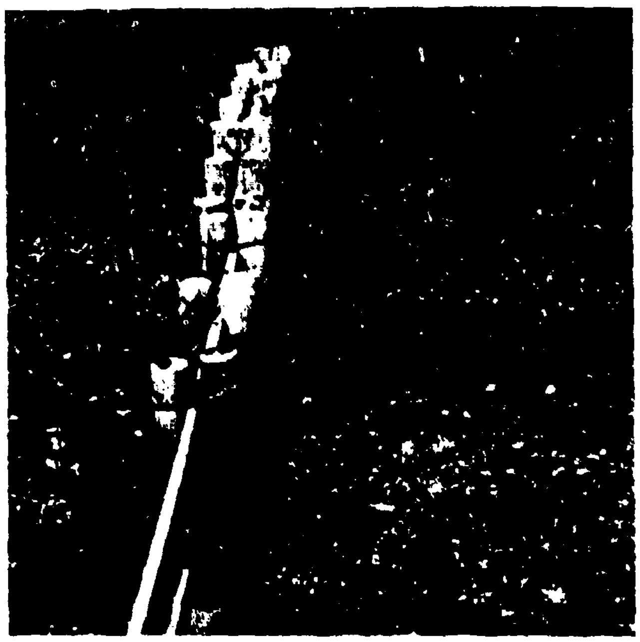


Figure 3-93. The single-planking timber groin, Ninilchik, Alaska, 30 June 1979.

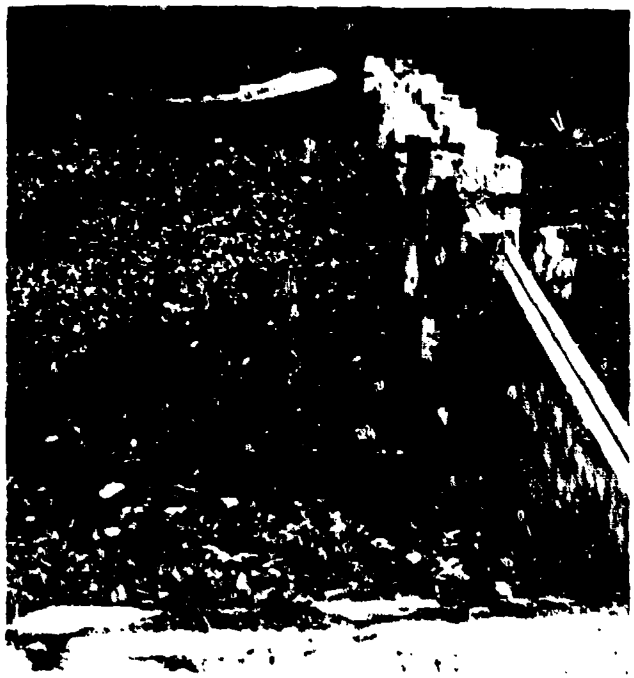


Figure 3-94. Double-planking timber groin, Ninilchik, Alaska, 30 June 1979.

The littoral mechanism is not primarily alongshore, as the log revetments deposits are the result of the onshore-offshore transport.

The timber-pile groins at Lincoln Township and Buckroe Beach also show no signs of material or structural degradation. The 90-foot, impervious, Lincoln Township groin was highly successful at trapping sand and raising the beach profile. It became so successful that monitoring of the 3-year-old device was discontinued in 1976. Eight timber-pile groins were monitored at Buckroe Beach. Again, this system was instrumental in forming protective fillet beaches (Fig. 3-95).

Timber groins are an old and proven means of shore protection in shore segments where littoral transport conditions justify their use. The only cost information available was for Lincoln Township, where in 1973 the groin was constructed for $50 per linear foot. This figure is reasonable even when escalated to prices considering the good performances monitored in the program.

g. <u>Timber and Rock</u>. Groins constructed of timber and rock were evaluated at two sites--Folly Beach, South Carolina, and Sanilac Section 26, Michigan. Seven of the 41 existing timber and rock groins monitored at Folly Beach were built by the South Carolina Highway Department; their lengths vary from 180 to 300 feet. They are treated timber-pile groins, the seaward half of each being flanked on both sides with 200- to 500-pound armor stone. The structures have been monitored since 1979; the

Figure 3-95. One of the timber groins used at Buckroe Beach, Virginia, which were very effective in accumulating sand, 12 March 1979.

structure age is unknown but is assumed to be 10 to 15 years. No structural damage was observed even after the groins were exposed to wave forces generated by Hurricane David. The cusped beaches that developed between the groins have not changed significantly during the monitoring period, and the shoreline has been stabilized by the groin system.

A rockfilled timber-crib groin was installed at the Sanilac Section 26 site. The 4- by 4-inch-thick timber was bolted together to form a frame; the frame was then lined with galvanized wire mesh and filled with 3- to 5-inch stone. No underlying filter cloth was used. Because of settlement into the bottom and washout of small stones, the rockfill level fell almost 2 feet at the lakeward end (Fig. 3-96). Settlement of the timber frame was minor. Accretion and depletion of drift material was seasonal. Early monitoring revealed that, overall, the beach profile was lowering. Continued observation is necessary to verify this trend.

The two types of timber and rock groins, sheet piling reinforced with armor stone and rockfilled timber cribs, have dissimilar applications. Groins of the former type, at Folly Beach, proved to be more sound structurally, after surviving Hurricane David. Although cost information for Folly Beach is not available, it is expected that high construction and material expenses for this type of groin will exceed the low-cost criteria.

While the Folly Beach system is applicable where larger wave forces are anticipated, the timber-crib design is applicable for more

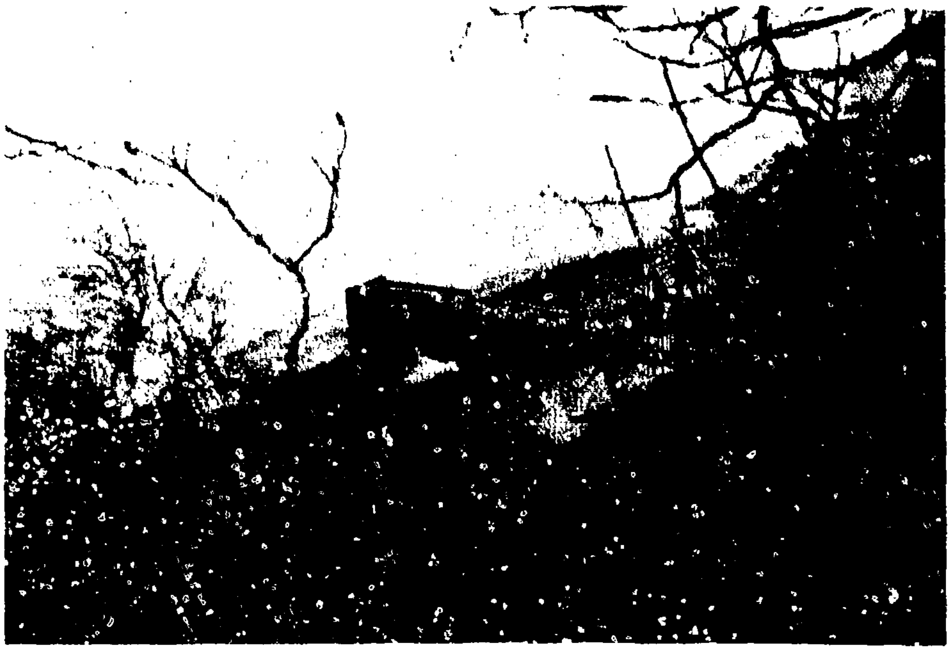


Figure 3-96. Rockfilled timber-crib groin viewed from the top of the bluff; level of rockfill fell almost 2 feet at lakeward end; accretion was seasonal, Sanilac Section 26, Michigan, 26 December 1979.

moderate wave climates, but a gabion groin might be less costly and perform equally well.

h. Rock Asphalt Mastic. Piled rock covered with asphalt was used as a groin at one site Sanilac Section 26, Michigan. Stone was dumped to form the core, and hot asphalt mastic was poured into the voids to seal and bind the structure. Only minor structural damage occurred. Some stones were dislodged and a piece of the northern corner of the lakeward end broke off; however, this did not affect the functional effectiveness of the groin. Performance was good. The groin trapped sand, forming a wide fillet (Fig. 3-97). Adjacent beaches were extended, which aided in halting bluff recession. Anticipated scour about the downdrift side of the groin was not observed and settlement was nominal. The amorphous quality of the asphalt mastic allows the structure to adjust to settlement without loss of structural integrity. However, this device cost $154 per linear foot when built and the use of asphalt in a groin in 1980 would probably be a prohibitive expense.

i. Profiles. Profile comparisons were not made for groin systems, as the effectiveness of groins can best be determined by taking profiles of the beach immediately updrift and downdrift of each groin. The survey programs were not set up to do this, and the best alternative method of determing groin effectiveness is evaluation of successive ground and aerial photos.

Figure 5-97. Rock asphalt mastic groin at Sanilac Section 26, Michigan, 27 November 1979.

5. Nonstructural Devices.

a. Perched Beach. Sandfills retained by low sills to form perched beaches were monitored at two sites--Slaughter Beach, Delaware, and Alameda, California. At Slaughter Beach, the sill was constructed in three types of segments: large sandbags, timber-sheet piling, and concrete boxes. The sheet-pile sill retained the fill best, followed in order of decreasing effectiveness by the concrete boxes and the sandbags. Monitoring did not continue long enough to determine how well the perched beach was being retained, but little loss of sand occurred in the tidal zone, and the system was considered successful.

At Alameda, device 8 was a perched beach retained by sand-cement-filled small sandbags, well entrenched into the bottom and with filter-cloth backing to prevent loss of fill through the structure. Although settlement of the sill was uneven and some top bags were lost, the system was generally effective in retaining the fill and preventing erosion of the bank behind it.

b. Artificial Seaweed. At Roanoke Island, North Carolina, the use of artificial seaweed, made of closely set fronts of polypropylene fibers anchored in the offshore area, was demonstrated. Its purpose was to attenuate waves sufficiently to allow vegetation to become established in the tidal zone in its lee. The installation was completed in late spring 1980, leaving no time to monitor the project.

c. <u>Beach Fill</u>. Beach nourishment programs were conducted at six sites--Bowers, Delaware; Broadkill Beach, Delaware; Lewes, Delaware; Muskegon, Michigan; Alameda, California; and Sunnyside Beach, Washington. The beach fill at Bowers is retained by two nylon Dura-Bag groins, 750 and 400 feet long. The last nourishment project was in the summer of 1974 when 28,800 cubic yards of sand was supplied. Since then the groins have stabilized the fill, and the combined system of groins and fill has controlled shoreline recession.

A previously constructed groin field retains the beach fill at Broadkill Beach. The groin field is comprised of two concrete rubble groins, two timber groins, and one groin constructed of both timber and concrete rubble. Beach nourishment programs have placed 18,100, 29,500, and 40,300 cubic yards of sand along this reach in 1973, 1975, and 1976, respectively. The combined action of the groins and beach fill is controlling the shoreline recession. Although the direct protection of the shoreline is attributable to the beach fill, the groins have been instrumental in stabilizing the fill. The potential for littoral transport at this site is high and it is doubtful that a beach fill alone could have controlled shoreline erosion without a costly renourishment schedule.

Shoreline recession at Lewes has been controlled by the placement of a protective beach fill. The initial program comprised the placement of 86,710 cubic yards in 1975. Since then the beach has been renourished with 11,400 and 31,000 cubic yards in 1977 and 1978, respectively, with another 87,000 cubic yards planned for the fall of 1980. Although the beach fill has controlled shoreline recession, the amount of renourishment has been high relative to the beach-fill programs at Bowers and Broadkill Beach. Comparison of the three sites demonstrates the potential reduction of annual maintenance charges due to renourishment when the beach fill is retained by groins.

The site at Muskegon is protected by a fill mixture of sand, large stones, small amounts of gravel, wood chips, and cinders. The fill rises to a height of almost 25 feet above LWD with a slope of roughly 1 on 2. The fill was intended to serve the dual purpose of supplying the downdrift shoreline with littoral drift and protecting a nearby road from being undermined. Monitoring of the site has revealed that the fill has been only partially successful. Erosion of fill material has been retarded to the extent that none is appearing downdrift. Apparently many large stones contained in the fill have come to rest at the base of the slope to form a toe riprap. This riprap has partially stabilized the fill, preventing erosion of beach-size material for transport to downdrift beaches.

Five hundred feet of shoreline was protected at Alameda by the placement of 2,500 cubic yards of beach fill in November 1978. The fill was retained by a single groin 150 feet long. During the monitoring program the general beach alinement remained unchanged, and sand gradually accreted adjacent to the groin. Initial plans for this installation called for additional beach nourishment after 1 year; however, the predominantly eastward littoral transport and the trapping capacity of the groin eliminated the need for renourishment. The beach fill not only protected the shoreline from further recession but also provided an esthetically pleasing beach which attracted a large attendance.

A protective beach fill of 18,000 cubic yards was placed at the Sunnyside Beach site in December 1975. In July 1978 the site was renourished with an additional 4,200 cubic yards of waste sand. Monitoring of the beach fill under the Shoreline Eosion Control Demonstration Program began in January 1979. After more than 1 year of monitoring, the heavily used recreation beach continues to protect the park and sewage facilities. Although it has eroded at a moderate rate, the beach has remained in good condition and the eroded material is evenly distributed within a few hundred feet of shore.

d. Vegetation. Plantings of various species of vegetation in and immediately behind the tidal zone as a means of shore stabilization were demonstrated at a wide variety of sites, both alone and in conjunction with offshore structures. Since vegetation is essentially a material used only for shore stabilization, this subject is treated in more detail at the end of section IV. A summary of the results of the vegetation plantings at all sites is presented in tabular form at the end of Section V.

IV. EVALUATION OF MATERIALS

1. Filter Cloth.

All types of filter cloth presently on the market are adequately resistant to decomposition due to contact with seawater, soils, and structural materials. However, the nonwoven types made of synthetic fiber mats or machine-punched sheets of synthetic material tend to tear or otherwise lose their filtering capability when stressed nonuniformly. They may be used with good results when laid out on a horizontal or gently sloping bottom to prevent the bottom material from working up into a superimposed structure such as gabion or rubble-mound breakwater, sill, or groin. The failures at Alameda, California, demonstrated the inadequacy of nonwoven filters used behind vertical or sloping bulkheads or revetments.

Woven filters are normally made with polypropylene or polyvinylidene-chloride monofilament yarns that are heat-calendered after weaving to maintain uniform-width openings by fusing the fill and warp yarns. The size of the openings must be appropriate for the soil on which the cloth is to be placed. At Holly Beach, Louisiana, the first filter cloth used with the Gobi blocks was woven too fine and became clogged with silt and fine sand, allowing hydrostatic pressure to build up with resultant slumping of the embankment in some areas. When a coarser weave was used, this problem was eliminated. However, field seams stitched between adjacent sheets of cloth ran horizontally along the slope and failed when stressed by differential settling of the revetment. This indicated that sheets of cloth should be continuous from top to bottom on the slope, with either field seams stitched or generous overlaps running perpendicular to the contours. Failures due to excessive stretching of the cloth may be avoided by pulling it into loose folds after it is unrolled, rather than stretching it out over the slope. Where placed behind a bulkhead with irregular surfaces such as those of rubber tires or logs, the cloth should be pushed (or allowed to yield under backfill pressure) into crevices to prevent ballooning beyond the burst strength of the fabric.

Woven filter cloth made of polyvinylidene chloride is heavier than water and should be specified wherever large amounts are to be laid out below the water surface. It is recognizable by its dark-green color. Woven polypropylene cloth is dark brown, lighter than water, and stiffer, stronger, and less costly than the polyvinylidene-chloride type. It should be specified for use above the water surface or for extending a short distance downslope into the water. Various methods of wrapping the extremities of the sheets around small amounts of cobbles or small quarrystones in the form of Dutch toes have been used successfully to terminate the edges of the cloth or to anchor it underwater.

Filter cloth is seldom kept in stock for resale by materials suppliers because of the large amount of space it would have to occupy to be available in the quantities needed for most projects. If it must be ordered directly from the manufacturer and shipped to the site, 2 to 4 weeks should be allowed for delivery after placing the order.

2. Rubber Tires.

Used rubber tires are well suited for low-cost shore protection because they are inexpensive, and in some areas may even be donated. They are resistant to decomposition by virtually all environmental forces and have several features that make them advantageous for use in shore protection structures. Being quite tough and flexible, they will not break when struck by floating debris or when deformed by exterior loading or settlement of a structure. The size of a tire is near optimum for man-handling, and fairly large structures can be assembled without the use of costly equipment. The specific gravity of a tire is slightly greater than that of seawater and it will sink to the bottom if loosened from a structure, provided it has not trapped air or been buoyed with flotation foam. On the other hand, tires cannot easily be fastened together securely, and if a tire structure is broken up by wave action, the tires or tire modules are readily carried away by currents and distributed along the shoreline in an esthetically unattractive manner.

In the demonstration program, tires have been used in three basic ways: (a) fastened together and buoyed to form floating breakwaters, (b) strung together on posts either set upright side-by-side to resist wave action or placed end-to-end horizontally to form a low sill in shallow water, and (c) laid flat on a slope side-by-side and filled with grout to form a revetment. In each installation where tires were used, the tires themselves functioned as intended, without deterioration. Failures of tire structure resulted from (a) separation due to inadequate fastenings or anchorages, (b) loss of filler material, and (c) scouring of bedding or backfill materials due to inadequate filtering systems.

The separation failures in the demonstration program were due primarily to washers or fastening devices pulling through the walls of the tires. New fastening devices are being developed that may overcome this defect. Strapping with high-strength belting, as in the Goodyear floating breakwater system, proved most effective in holding the tires together. Even that system generated high stresses in the anchor lines during severe wave action, which dragged the concrete block anchors first used at Pickering Beach, Delaware. The Goodyear system was then reinstalled with pile anchors.

Attempts to increase the inertia of tire structures by filling the tires with gravel proved futile. Wave turbulence easily washed out the gravel, allowing the tires to collapse and the tire structure to lose its integrity. Filling the tires with concrete grout proved most effective in adding weight and strength to the structural system.

The annular shape of a tire makes it difficult to fit into a structural system without producing a high void ratio. Safe retention of fill behind tire bulkheads of any design depends on the use of high-strength woven filter cloth and its careful and proper placement.

3. Concrete.

Portland cement concrete is an excellent material for use in the shore zone because, when properly proportioned, mixed, and cured, it virtually has the properties of solid rock. However, the high cost of preparing forms for on-site construction precludes its use in low-cost, monolithic shore protection structures. Its low tensile strength can be overcome by the incorporation of reinforcing steel, but again, the high cost of doing this generally precludes reinforcement of concrete modules used in a low-cost program.

Concrete was used in various modular structures evaluated in the demonstration project, only three of which were reinforced: Surgebreakers, Sta-pods, and Z-walls. Nonreinforced modules included Sandgrabbers, concrete boxes, Nami rings, and various types and sizes and shapes of blocks. Each of the above modules was manufactured away from the site under controlled conditions with special forms designed for long-term use. Transportation of the modules to the site can be a significant cost factor if the casting yard is very distant. Three of the modules were heavy enough to require special handling equipment: Surgebreakers, Sta-pods, and concrete boxes; others could be hand-carried. Some of the modules evaluated broke under stresses due to uneven settlement when the foundation was scoured. Of these broken modules, a number were found to have substandard concrete; others were simply stressed beyond design limits. No internal failure or deterioration of a module resulted from exposure to the elements or to direct wave impact, although many modules were abraded by wave-borne cobbles. The behavior of concrete in grout-filled bags, concrete rubble, and concrete blocks is discussed later under these respective headings.

In general, the monitoring proved that modules constructed with good quality concrete survived without damage in structures that were not subjected to abrasion and were not undermined or otherwise displaced. Some of the smaller block modules were too lightweight for the wave environment to which they were subjected and were displaced by water impact and turbulence. These failures, as well as those due to undermining or filter failure, should not be charged to inadequacy of concrete as a structural material, but rather to inadequacy of structural design or exceedance of design conditions.

The following specifications should apply to all concrete used in shore protection structures: Portland cement should be type II, air-entrained; aggregates should be inert to seawater; and compressive strength should be 3,000 pounds per square inch minimum.

4. Sandfilled Bags.

Ordinary sandbag construction with "100-pound" burlap bags has been used for many years in emergency situations for raising dikes, plugging leaks, and otherwise controlling floodwaters. The use of sandfilled bags for shoreline erosion control is a fairly recent innovation, and reliable data on their performance and longevity in the shore zone have been lacking. Because the burlap deteriorates with exposure to sunlight, it was used only at one site (Alameda, California), in a revetment that failed through bank failure before any deterioration of the fabric occurred.

Acrylic, nylon, and polypropylene bags, both large and small, were used at several sites in revetments, bulkheads, low sills, and breakwaters. The ultraviolet-resistant, woven acrylic bags called Sand Pillows and nylon Dura-Bags survived exposure to sunlight best. The installations with the longest exposure monitored under the program were the two Dura-Bag groins at Bowers, Delaware. The top fabric of these large bags was just beginning to show signs of deterioration about 8 years after construction. Although some bags had been torn and had leaked considerable sand, the groins were still functional. The acrylic Sand Pillows installed for the demonstration projects appeared to be at least as resistant to sunlight deterioration as were the Dura-Bags; however, these were not monitored for a long enough time to verify this contention. The spun-bonded polypropylene sandbags used in the fence-sandbag bulkhead at Basin Bayou, Florida, deteriorated badly in a single season.

None of the small sandfilled bags used for revetments or bulkheads in the surf zone survived, mostly because they were dislodged by wave impact forces, bank slumping, or toe scour. It appeared that the relatively light weight of these small bag modules permitted them to be lifted or slid out of place by the forces of high waves. The larger bags, all of which had to be filled by pumping, resisted dislodgement quite effectively in groins, sills, and breakwaters. Nevertheless, they were quite vulnerable to vandalism and debris-impact tears. Once torn, they lost their sandfill rapidly under wave agitation and collapsed. At Basin Bayou, the deterioration of factory-sewn seams was a primary cause of module failure in one breakwater section. At Alameda, deterioration of the closure twines resulted in bag failure. The large Advance Bags have a filling spout that is difficult to seal off on completion of filling. If the spout is improperly sealed, the sand may leak out.

The lessons learned about sandfilled bags used for shore protection were:

(a) Small bags filled only with sand should not be used in the surf zone.

(b) Large bags have much better stability and can be functionally effective in areas where puncturing by debris or vandalism is not a problem.

(c) Sandfilled bags are only as strong as the bag material. To have reasonable longevity, the fabric must be ultraviolet-resistant, and seams and closure twines must be adequate to develop the same strength as the fabric itself.

(d) The bags must have a stable foundation and a stacking arrangement that will ensure structural stability of the device in which they are used.

(e) The bags should not be overfilled as this decreases their nesting ability and lessens the stability of the structure in which they are used.

5. Grout-Filled Bags.

The inherent weakness of the fabric in sandbag construction can be overcome by filling the bags with concrete grout (sand-cement). As soon as the grout hardens, the strength of the fabric is no longer a factor, and the grout-filled modules are essentially irregularly shaped concrete blocks that nest well together in the structure. The bags merely serve as sacrificial, flexible forms for the concrete and can be made of any reasonably strong fabric. In the demonstration program, large bags filled with grout were not used extensively because the cement tends to separate from the sand in the pumping process and much of it escapes through the bag fabric. Small grout-filled bags were used only at two sites (Alameda, California, and Oak Harbor, Washington), with widely varying results.

At Alameda, grout-filled bags were used in revetments, groins, and sills. The two revetments, one with acrylic bags and the other with burlap bags, failed structurally because of toe scour and bank slumping before the individual units could be properly tested as to their stability and longevity. However, during initial settlement, some of the acrylic units cracked apart. In groins and sills, however, grout-filled bags were more successful. Some of the large nylon bags in devices 1 and 2 were formed into relatively rectangular sections with the use of side forms for better stability and to prevent the escape of cement; other large bags were allowed to spread out without forming. The modules all retained their shape, although some were displaced from the structures either by wave forces or by vandalism. In general, these groins and sills performed well functionally, but the monitoring did not continue long enough to document their longevity. A weak grout, consisting of 8 parts sand and gravel to 1 part cement, was used, which may account for some units in the revetment cracking as the structure settled. No deterioration was observed in hardened modules where the fabric had been torn by debris impact or by vandals.

At Oak Harbor, two revetments similar to the grout-filled bag revetments at Alameda were constructed--one with burlap bags filled with wet concrete grout, and the other with dry-mix concrete in paper bags that were punctured on top and flooded as successive tiers were placed. For each, the grout was a design mix aggregate with a yield of 5.5 sacks of cement per cubic yard. Both these revetments survived storms of about design intensity without damage, even though battered by logs and other debris. The stability of the slope on which the revetments were placed and adequate toe embedment may have accounted for their superior performance over the Alameda revetments.

In general, the evaluations at these two sites proved that grout-filled bags can be used effectively in properly designed and constructed revetments, sills, and breakwaters if exposed to waves not exceeding about 3.5 feet. They were not subjected to more severe conditons. In the Oak Harbor revetments, the wet-filled bags nested better than did the dry-filled bags, and appeared to be more stable in the structure. The higher specific gravity and solidity of grout, as compared to sandfill increases module stability under wave agitation and greatly increases resistance to damage by floating debris and vandalisim. The bag modules compare favorably with formed concrete modules of various types used in shore protection structures.

6. Timber.

In general, timber is a useful material for some types of shore protection structures, either in the form of straight tree-trunk logs or sawed lumber. A number of basic considerations may have a bearing on the selection of a timber device. The relative ease with which it can be cut or shaped for various applications, or with which it can be drilled for bolted connections, makes it a versatile structural element. It does have certain drawbacks, most of which can be overcome with proper design. The strength, the bending characteristics, and the amount of expansion and contraction with moisture content vary with the species of tree from which the timber is cut. Its graininess is a problem that must be considered in design. Most woods are much weaker perpendicular to the grain than parallel with it. Allowable end and edge distances must not be reduced in bolted connections, and spikes or drift bolts must not be driven in such a way that they will split the wood. This is particularly important when the wood is dry. Although fire is a potential hazard, the thickness of the timber elements in most shore structures makes fires hard to start, and a sustained fire that consumes a timber shore protection structure is a rare occurrence.

In a marine environment, untreated timber tends to deteriorate relatively soon as a result of fungus dry rot, termite infestation, or marine borers. At two sites where untreated-timber structures were monitored (Oak Harbor, Washington, and Ashland, Wisconsin), those structures were destroyed by waves as a result of design inadequacies before any evidence of deterioration appeared in the material itself. At Ninilchik, Alaska, the timber structures have not been exposed long enough to show signs of deterioration. Those structures were not treated because the causes of deterioration are much reduced in the cold Alaskan climate, and local experience indicated that untreated-timber construction should survive at least 10 years of exposure if not destroyed earlier by wave action or ice.

At Ninilchik and Oak Harbor, the structures were constructed in or above the upper part of the tidal zone, where destructive marine organisms cannot gain a foothold. At both sites, the large tidal range assured an interval of several hours of nonexposure to either inundation or wave runup between successive high tides. Also, native structural timber was plentiful and relatively inexpensive, and the added cost of pressure treatment may not have been justified for structures not intended to have more than about a 10-year life. Untreated timber placed lower in the tidal zone or continuously submerged in seawater could not be expected to survive more than 2 or 3 years in most regions.

The use of untreated log posts in the seawall structures designed to protect the ends of Longard tubes at Ashland was possible because of the freshwaters of Lake Superior. Although that was the only site on the Great Lakes where a timber structure was monitored for any length of time, timber protective devices have been installed in many areas along the Great Lakes shores. The larger timber structures, mostly in the more exposed shore segments, are pressure-treated for greater longetivity. The smaller untreated structures that were properly designed have generally performed well over periods of 10 or more years. A timber groin installed at the Lincoln Township site on Lake Michigan in 1973 was still in good condition when monitoring was terminated in 1979. That structure probably is not treated, although this was never determined.

Treated-timber devices exposed to seawater were monitored at several sites on the Atlantic, gulf, and Pacific coasts. All have survived well. The treated-timber groins constructed at Buckroe Beach, Virginia, in 1963, the oldest of any timber structure monitored during the program, are still in good condition. Because of the importance of proper design and preservative treatment in constructing a timber shore protection device that will survive and function properly in a marine environment, the following recommendations that have evolved through many years of experience are offered:

(a) Treatment should normally consist of a pressure injection of coal-tar creosote that provides full penetration of sawed lumber up to about 6 inches in thickness. Treatment should be in accordance with applicable Federal specifications or American Wood Preservers Association (AWPA), Standard C18. Various salt preservative treatments are often used in upper parts of structures where bleeding of coal-tar creosote would be a problem. These treatments are not as long lasting or effective as coal-tar creosote in a marine environment, but they do prevent dry rot and termite infestation, and they greatly extend the life of the structure. Many shore structures have creosoted piles and salt-treated wales and planking.

(b) The structural properties of timber piles, wales, struts, and sheathing vary with the types of timber used, the nature of curing to which it has been subjected, its environmental exposure in the structure, and sometimes the type of preservative treatment used. These properties, including allowable stresses under various loading conditions and environments, are discussed in the Timber Construction Manual published by the American Institute of Timber Construction (AITC)(1974). The standards for various types of timber are listed in section 101-65, paragraph 3.5, of the publication; these standards change from time to time, and only the current issues should be used.

(c) Timber structures often fail because of corrosion, abrasion, or fatigue of metal connectors, or because of abrasion of the wood by loose connectors, and not as a result of deterioration of the wood members. For marine exposure, all hardware should be galvanized or coated with coal-tar epoxy. The minimum sizes to be used are a function of design.

(d) In general, boltholes should not exceed the diameter of the bolt by more than 0.062 inch. Driftbolts or spiral bolts should have a driving fit. Washers should bear evenly and fully on the timber, and where the axis of the bolt is not perpendicular to the face of the timber, beveled plates or washers should be used.

7. Concrete Rubble.

In general, good quality concrete rubble from such sources as road pavements, sidewalks, and building demolition is almost the equivalent of similarly graded quarrystone rubble for performance in rubble-mound structures and revetments, provided it is adequately shape-sized to an aspect ratio no greater than 3 to 1. The shape-sizing however may reduce the rubble to a size smaller than that required for an armor layer, and it may be suitable only for use in a mild wave climate. The individual pieces are usually a bit lighter than equally sized pieces of stone, and they may have slightly less hardness and durability. To assure consolidation of the rubble by wave impact, all protruding rebars should be burned off individual pieces before placement in the mound or revetment. A tangle of rebars in the section may leave holes in which water pressure, amplified by wave impact, can lift adjacent pieces of rubble out of the section and trigger progressive raveling of the structure. Also, a stockpile of concrete rubble delivered to the site from a demolition source is apt to contain a large amount of dirt, concrete dust, and small chips. This excess of fines should be separated and used for backfill or otherwise disposed of before placing the rubble in the design section.

The performance of concrete rubble was monitored in revetments at three sites only--Alameda, California; Shoreacres, Texas; and Folly Beach, South Carolina. At Alameda, the rubble consisted primarily of road paving, sidewalk, and building debris, in which the largest pieces were only a few inches thick. When large pieces were used in a single-layer mosaic on filter cloth, too much cloth was exposed, and in places the cloth was torn and considerable backfill was washed out by wave action. Some of the large, flat pieces of rubble were tilted and displaced. Considering the design deficiency, however, the concrete rubble itself performed well. Elsewhere at Alameda, concrete rubble dumped at random was less effective as a riprap revetment because of inadequate filtering effect. Storm waves washed large amounts of the fine embankment sand out through voids in the rubble, leaving individual large pieces and clumps stranded, and surrounding areas of sand exposed to further depletion by wave erosion. The wave climate at is site may be too severe for any structure built with the kind of concrete rubble used in the demonstration project.

At Shoreacres, no filter was used, but the wave climate was milder and the rubble revetment was thicker. It successfully protected the embankment by forming its own filter of finer pieces trapped by the larger pieces in the outer layers of debris.

In area 4 at Folly Beach, large pieces of broken concrete were used as an armor layer over smaller sized quarrystone which had been dumped in front of a failing vertical concrete seawall. Hurricane David had scattered the stone to some extent in September 1979, and after the stone was piled back in the section, it was covered with the broken concrete for additional protection. This composite structure remained undamaged through the winter storms of 1979-80, but whether it can survive hurricane waves remains to be seen.

The lessons learned concerning the successful use of concrete rubble in rubble-mound structures were:

(a) All large, flat slabs and long pieces should be broken (shape-sized) into pieces of lower aspect ratio, generally not more than 3 to 1, before placement in the section.

(b) If shape-sizing so reduces the weight of individual pieces that they are too light for the wave climate of the site, the concrete rubble should be covered with an armor layer of selected larger pieces of concrete or quarrystone.

(c) The thickness of the rubble section in a revetment must be at least three times the thickness of the largest pieces, so that wave shakedown does not separate the rubble into small clumps, leaving surrounding areas of embankment unarmored.

(d) If the debris contains a large percentage of small-sized chunks and particles, the fines should be screened out before the rubble is placed in section.

(e) Adequate filtering of a revetment must be provided to assure that wave turbulence and embankment drainage will not carry fine bank material through the structure. Openings must be large enough to prevent buildup of pore pressure.

(f) The toe depth must be sufficient to prevent undermining of the structure by toe scour.

(g) A revetment must have its crest high enough to prevent overtopping waves from washing out the embankment behind the structure.

8. Concrete Blocks.

Many sizes and shapes of concrete modules were evaluated at several sites representing a variety of environments. The primary advantage of the concrete-block revetment system is the uniformity of its modules with resultant simplicity of installation without need for heavy equipment. In each system, all modules are cast in the same size and shape, designed to provide a uniform pattern of openings and junctions with one another. Dissipation of wave energy by roughness of the exposed surface is the main purpose of some units; others rely on nesting or interlocking patterns to assure better stability in the section. Most are designed for use in a single layer over filter cloth to reduce the section thickness and make them comparable from a cost standpoint to the thicker revetments of rubble structures. Each system allows for some displacement of the embankment under wave shakedown and subgrade consolidation, but when the allowable limits of displacement are exceeded, failure of the system by progressive raveling often ensues. For this reason, it is important that (a) the subgrade be compacted and dressed to a slope that is inherently stable without dependence on the weight of a heavy cover, and (b) the units are heavy enough to resist displacement by design waves. The performance of the various blocks is summarized below.

a. Gobi Blocks (Erco Blocks). Gobi blocks were used at Holly Beach and Fontainebleau State Park, Louisiana. The small-sized standard Gobi blocks, hand-placed at Holly Beach, survived ordinary wave action with little damage, but were severely displaced by tropical storms. A similar installation at Fontainebleau was not damaged during the short monitoring period, but at both sites a large number of blocks were removed by vandals or souvenir hunters. Gobi-mats, both single and double, and Jumbo mats, tested at Fontainebleau were not damaged. One early slope failure problem at Holly Beach resulted from the use of a filter cloth with low permeability which clogged and caused pore-pressure buildup.

b. Turfblocks (Monoslabs). Turfblocks were demonstrated in a revetment at Port Wing, Wisconsin. These units have about the same length and width as Jumbo blocks but are lower in height and are hand-placed, not glued to the filter cloth. The Turfblocks, placed on a 1 on 3 slope, were subjected to 4- to 5-foot waves that overtopped the revetment soon after construction. The units were not widely displaced; most were either overturned or had settled into the embankment. None of the units were broken. The irregular appearance of the slope after the storm indicated that some slope failure may have occurred. This could have been caused by pore-pressure buildup due to clogging and loss of permeability in the nonwoven filter cloth which was used at this site, or by consolidation of the backfill which had not been compacted and which contained large boulders. The exact cause could not be determined. In the summer of 1980, the Port Wing Turfblock revetment was reconstructed, half on woven and half on nonwoven filter cloth, to an elevation 2 feet higher than in the original installation. The monitoring period was too short to reassess the performance of Turfblocks in this modified system.

c. Hollow Concrete Building Blocks. Hollow concrete building blocks were used in revetments at Port Wing, Wisconsin, and Fontainebleau State Park, Louisiana, with generally good success. The Port Wing blocks had interlocks cast into their ends; the Fontainebleau blocks were standard hollow masonry units with flat ends. The storm waves that damaged the Turfblock revetment at Port Wing did not overtop the 8- and 12-inch-wide building block structures; both installations were virtually unharmed. Also wave runup was higher for the Turfblocks than for the building blocks.

The 8-inch-block revetment at Fontainebleau was unharmed by the milder wave climate of Lake Pontchartrain, but many blocks were stolen. There was some evidence that blocks placed with their long axes perpendicular rather than parallel to the shoreline were more stable. However, more monitoring is needed to prove the validity of this contention. Standards for concrete building blocks are much lower than those for poured-in-place concrete (1,000 pounds per square inch versus 3,000 pounds per square inch), and some abrasion of the blocks has been noted. However, because of their rectangular construction, with 8-inch-high sides and ends in full contact with adjacent units, building blocks are not as readily displaced by wave turbulence as are other types of blocks.

d. Sandgrabber Blocks. A modified type of hollow building block was used in the various Sandgrabber installations. The performance of the units in these structures was generally good except where nonuniform undermining occurred, distorting the shape of the structure. The distortion resulted in severe tie-rod stresses, which cracked many blocks. In general, the performance of the Sandgrabber was rather poor, as discussed in Section III. The building blocks performed much better in revetments.

e. Nami Rings. In a revetment constructed in 1974 at Little Girls Point, Michigan, Nami rings were used in much the same manner as other types of revetment blocks. Where tied together with a crisscross pattern of tie rods and placed on filter cloth, they performed reasonably well; however, in 1975, during a severe storm on Lake Superior, many of the rings were broken by wave-borne cobbles and tree stumps. The beach accreted naturally during the ensuing years, covering the lower part of the revetment with a mantle of cobbles, gravel, and remains of trees. In 1979, when monitoring was discontinued, only two or three rows of rings remained visible, and they were badly broken. This revetment had not been installed according to the design; its top was left much too low, and overtopping waves continued to erode the bluff behind it. For this reason, and because of the heavy bombardment of cobbles, this installation was not considered a fair demonstration of Nami-ring performance.

f. General. Of those blocks that were subjected to high wave impacts, the hollow building blocks performed best and had the lowest cost per linear foot of revetment. Lessons learned about the use of concrete blocks in revetments for shore protection were:

(1) The block module should be of such size and shape that it will not be displaced by design waves. Tables of allowable wave heights for each type of block revetment placed on various slopes should be prepared for user guidance. This has not yet been done, as more data on the performance of various modules in various wave climates are needed.

(2) The slope must be flat enough to remain stable under design conditions. Even though the revetment armor is adequate to resist displacement, it may fail if the embankment sloughs.

(3) Adequate filtering must be provided to assure relief of pore pressure without loss of embankment material due to piping. The continuity of filter provided by a filter fabric is good insurance against failure due to slight shifting of block modules under wave agitation.

(4) Environmental considerations are important in selecting the type of block for a given site. Blocks that are useful for other purposes may be stolen. Blocks that are heavy enough to resist displacement by design waves may be dislodged or broken by floating-debris impact or ice thrust.

(5) Factory-gluing of Gobi blocks to filter fabric reduces the danger of the blocks being stolen, and where power equipment is available, may speed construction. The gluing may also resist displacement of blocks by wave action. However, Gobi-mats and Jumbo mats are not as adaptable to placement in nonuniform configurations as are hand-placed blocks.

(6) Wave runup is higher on some blocks than on others. More data are needed on the runup of waves of various heights on block revetments of different types so that they can be designed to prevent overtopping.

9. Gabions.

The gabion basket is a versatile shore protection component that is most useful where large armor stone or other armoring material is not available but where a plentiful supply of smaller stone or rubble exists. Filled baskets of various dimensions may be stacked on top of one another to form bulkheads or low breakwaters, they may be laid end-to-end to form groins, or the low- and wide-mattress type may be laid side-by-side on a slope to form a revetment. At Geneva State Park, Ohio, a breakwater constructed with a mixed pattern of gabion sizes and shapes provided considerable information on gabion performance in a relatively severe wave climate. Gabion groins and revetments were monitored at Sanilac Section 26, Michigan, and at Ninilchik and Kotzebue, Alaska. A gabion revetment was demonstrated at Oak Harbor, Washington. Because the baskets are shipped from the factory unfolded and flat to reduce their bulk, the cost of shipment is low. Where stone fill is plentiful, a gabion structure may be the least costly shore protection option for a do-it-yourself property owner. Lessons learned about gabion construction were:

(a) Where toe scour is anticipated, the baskets at the toe of the device should be entrenched sufficiently to prevent undermining and resultant deformation of the baskets. The stress of deformation at Geneva broke the wire mesh of many baskets, after which wave action removed the stone fill. The flexibility of the gabion basket allows it to adjust to minor changes of bottom configuration, but this adjustment must be kept within reasonable limits.

(b) The baskets should be filled as completely as possible. In some baskets that were partially filled, the stones shifted under wave agitation, causing basket deformation. Some of the smaller stones that would have remained in place, if the outer stones had shifted, were washed through the mesh. Some baskets were ruptured by the deformation.

(c) If fill stone more than 4 inches in diameter is not available, other options for filling the baskets may be used. At the Alaska sites, some baskets were lined with hardware cloth and others with filter cloth to retain gravel fill, and some baskets were filled with sandbags filled with the sandy gravel of the local beach. At Ninilchik, polypropylene sandbags partially filled with concrete were used to fill some of the baskets. The devices were not monitored long enough for a valid analysis of their performance.

(d) The PVC-coated wire lasted longer than the galvanized wire, even in the freshwater of Lake Erie. Thus, "Sea-Type" PVC-coated gabion baskets should be used for all shore protection structures.

(e) Lid closures were made with twists of wire at about 6-inch intervals in the Oak Harbor revetment in lieu of the uniform lacing prescribed by the basket manufacturer. This saved considerable construction time, but a number of ties broke along the toe of the structure. Therefore, this closure method is not recommended for high structures, or for areas where deformation is likely to be an important factor.

(f) Lacing together of adjacent baskets proved to be an important requirement in maintaining structural integrity.

(g) Gabions placed directly on sandy material tended to sink into the bottom as wave agitation worked the bottom material up into the baskets. Unless a gabion structure is founded on a solid base, a filter-cloth foundation should be provided. The cloth should not extend past the gabions, as the flapping cloth protrusion accelerates erosion of adjacent bottom material during high wave episodes.

10. Steel Fuel Barrels.

The use of new steel in a shore protection structure usually escalates the cost beyond the low-price range. Only where an abundance of used fuel barrels with little salvage value exists will they be suitable for use in shore protection devices. Monitoring at Kotzebue, Alaska, proved their usefulness in that arctic outpost region, both in revetments and groins. However, earlier installations at Ninilchik, Alaska, proved that the barrels rust out too soon in that region to be economical. Lessons learned concerning fuel barrel structures at Kotzebue were:

(a) The barrels must be filled with gravel and kept filled to avoid crushing by ice floes and debris.

(b) Critical barrels in the more exposed parts of structures must be capped with concrete to avoid loss of fill material.

(c) Where scour at the base of the barrels is anticipated, the barrels should be entrenched to the estimated scour depth.

(d) Bolting together of adjacent barrels effectively increases the integrity of a fuel-barrel structure.

11. Longard Tubes.

Longard tubes were evaluated as bulkheads, low breakwaters, and groins at several sites and in a variety of environments. Their primary advantage is the speed with which they can be filled after all equipment and materials are in place ready for the filling operation. Within their design limits, they are functionally effective as long as they remain structurally sound and are not displaced. Their chief drawback is a vulnerability to vandalism and to damage by floating debris. This has been well documented for all sites monitored. For construction, the tubes require a large supply of good quality sand, which may not be available at some sites. Also, the patented filling equipment must be mobilized at the site before filling can begin, and only specially qualified personnel can be used in the filling process. Lessons learned concerning Longard tubes at the several sites where they were monitored were:

(a) Proper placement of a filter-cloth foundation with the small 10-inch tubes attached is good insurance against the large tube being displaced by wave scouring of bottom materials. Entrenching the tube in a depression may prevent it from being rolled out of place by wave forces.

(b) In a bulkhead, the tube must not be placed too close to a bluff or overtopping waves will erode the bluff. If placed far enough out on a sandy beach, the tube will cause overtopping waves to deposit a berm of sand between the tube and the bluff. The overtopping wave energy is absorbed on this berm, preventing bluff erosion.

(c) Longard tubes should not be used at high wave-energy sites where large pieces of floating debris may gash the tube fabric. Considerable damage to tubes installed along the shores of the Great Lakes apparently resulted from debris impact and possibly from ice floes.

(d) Application of a sand-epoxy coating to exposed surfaces of a tube will deter vandalism and prevent puncture holes from enlarging. Also, the tube must not be allowed to roll after the coating is applied, as uncoated surface areas will then be exposed, and distortion of the tube fabric may cause the existing coating to flake off.

(e) Placement of other devices on top of a tube to increase the height of the structure did not prove effective. The sand-cement blocks used for this purpose at the Alameda site were easily dislodged by wave forces.

(f) Where tubes are used as groins in an active littoral environment, buildup of sand on one side and scour on the other side tend to cause the tubes to roll and become distorted, ultimately causing the fabric to tear and allowing the sandfill to spill out. The 40-inch Longard tubes can be obtained in pairs, factory-stitched together along their sides. These paired tubes should be used as groins under conditions of active littoral drift, with the base for the downdrift tube being excavated to a lower elevation to avoid excessive distortion due to scour on the downdrift side of the groin.

12. Quarrystone.

Where good quarrystone is competitive in price with other options for shore protection, it is probably the most reliable and long-lasting material that can be used. Stone rubble was used in one demonstration device only, the stone breakwater at Kitts Hummock, Delaware. That structure cost $212 per linear foot, which is not excessive considering that the groin effect of the breakwater protects about three breakwater lengths of shoreline and that its performance indicates that it will last much longer than 10 years. Previously constructed groins monitored at Sanilac Section 26, Michigan, and at Siuslaw River, Oregon, as well as a revetment at Tawas Point, Michigan, all showed equally good performance, but cost data on most of them are unavailable. Quarrystone structures have none of the drawbacks that plague structures of other materials. They are inert to light, temperature changes, and chemical decomposition. They are difficult to vandalize and are usually self-healing when small displacements occur as a result of foundation settlement or wave action. Also, fairly reliable criteria have been developed for the design of rubble-mound structures. However, the weight of adequate sized stone for most structures requires the use of heavy construction equipment not available to most property owners.

Design criteria for stone structures are presented in the Shore Protection Manual published by the U.S. Army Coastal Engineering Research Center. The following specification used in obtaining stone for the Kitts Hummock breakwater is generally applicable throughout the United States:

"Stone for the breakwater shall be durable and of suitable quality to assure permanence in the structure in which it is placed and the climate in which it is to be used. The stone shall be free of cracks, seams, and other defects that would tend to increase unduly its deterioration from natural causes or breakage in handling or dumping. The stone shall weigh, when dry, not less than 150 pounds per cubic foot. The inclusion of objectionable quantities of sand, dirt, clay, and rock fines will not be permitted. Selected granite and quartizite, rhyolite, traprock, and certain dolomitic limestones generally meet the requirements of these specifications."

In addition, it is customary for the owner to require that he approve the source of the stone to be used. Where the quality is questionable, samples should be tested for resistance to abrasion in accordance with specification C535, American Society for Testing Materials (ASTM)(1971).

The gradation of the stone is usually expressed in a table that gives the percentage by weight that must be larger than each of three or four listed sizes. It is often advisable to determine the difficulty of obtaining a given gradation before writing the specifications; otherwise, a premium price may have to be paid when a slightly more liberal gradation at a much lower cost would serve just as well.

The contractor is generally required to place stone of the size categories specified such that the limits of stone in place conform, with reasonable variation, to the design section without continuous underbuilding or overbuilding. A two-stone thickness should always be specified for the armor layer, with the largest stones placed on the outside. A 1 on 1.5 slope is about the steepest that will remain stable for most applications, with 1 on 2 being preferred. Stones in the outer layer should be nudged gently with a backhoe or similar equipment so as to seat them firmly, care being taken not to weaken the stability of adjacent stones in the process.

Lessons learned concerning the use of quarrystone in the Kitts Hummock breakwater were:

(a) Proper equipment and trained personnel are needed for proper and efficient construction of rubble-mound structures.

(b) A bedding course of matstone or its equivalent, or a layer of filter cloth, is needed to prevent the mound from sinking too deeply into a soft bottom.

(c) In estimating quantities of stone needed, the probable settlement of the structure during construction must be taken into account.

13. Asphalt Mastic.

This material was used to seal the voids in a rubble-mound groin at the Sanilac Section 26, Michigan, monitoring site. The hot mastic was poured over the structure after it was completed in an effort to make the groin sand-tight. The extent of penetration of the mastic material could not be determined, but the excellent performance of the groin indicated that it was adequate. Experience elsewhere has shown that the mastic cools too quickly in water to penetrate the submerged parts of a rubble-mound structure. However, where it does penetrate and fills the voids, the amorphous, pliable nature of asphalt mastic when cold assures flexibility of the stone mound, allowing it to adjust when displacements occur as a result of foundation settlement or wave agitation. In this respect, it is much better than a seal of concrete grout. However, more experience is needed for a full evaluation of its potential as a sealant for rubble-mound groins.

14. Vegetation.

a. Smooth Cordgrass. Smooth cordgrass was planted in the intertidal and subtidal areas in sites on the Atlantic and gulf coasts and only intertidal in Florida. This species did best when it was planted early in the growing season, in protected sites (natural or structural protection) on peat or with a peat layer beneath the sand. Plugs were the most successful method of planting, apparently because of their greater weight and stability. Sprigs were successful in some cases. Fertilization probably assists plants in becoming established, however the exact nature of this assistance cannot be determined at this time.

Pacific cordgrass was planted in the same tidal range on the Pacific coast as was the smooth cordgrass in the Atlantic sites. Good substrate stability, a protected location, and large size of plantings (plugs) also comprised the most successful combination for this species.

b. Saltmeadow Cordgrass. This species was planted throughout the Atlantic and gulf coast sites. The saltmeadow cordgrass performed best when there was protection from wind and sand movement, when planted early in the season, and when planted as relatively large plants. In a number of cases plants were uprooted and exposed by shifting sand, or were buried shortly after planting. Once established, the plants were somewhat more resistant to being seasonally buried and then uncovered, and seemed to propagate well. Since the performance of these plantings was evaluated for only a short time, the long-term response remains to be determined.

c. American Beachgrass. Two to three varieties of this species were planted along the Atlantic coast sites on the upper beach and dune. It is resistant to burying and appears to establish readily under a variety of conditions. Planting date affects survival, because plants placed in the ground after late spring suffer drought stress during the summer. This is probably the most successful species on sandy beaches, where it is adapted and can become well established after a relatively short period of time.

d. Red Mangrove. Mangroves were used in Florida on both the Atlantic and gulf coasts. This species appeared to do best when planted as a young tree with prop roots, in soils which allowed penetration of the roots, and where protected from direct wave attack. Once established, a fringe of mangroves along the coast appears to lessen wave force and provide protection for the shoreline. Mangroves occur naturally near these sites, in protected bays and shores. However, the success of the planting program for erosion control is difficult to evaluate fully at this time because hurricane damage was extensive at all sites.

Black and white mangroves were also planted to a limited degree at Key West. They appear to respond best to conditions favorable to red mangroves, although insufficient data were obtained for a conclusive evaluation.

e. Miscellaneous. A variety of other species were planted or allowed to establish at sites on the Atlantic, gulf, and Pacific coasts. Common reed was found to occur naturally in the Delaware Bay sites. It was also planted at the Fontainebleau site. This species is a widespread member of brackish and saltwater communities throughout the eastern United States, and could be expected to invade stabilized coastal habitats, away from direct wave action. Because little monitoring was done on the growth or establishment of this species in either area, its use in shoreline stabilization cannot yet be determined.

Torpedo grass appears to be a successful stabilizer of the upper beach area at Fontainebleau. It occurs naturally in dense patches and was allowed to establish in several sections. Perhaps this species could be used for further erosion control projects, except in Florida where torpedo grass is a prohibited plant to import, transport and cultivate. Shrubby species such as McCartney rose were also tried in gulf sites. Success was poor and little data are available.

A variety of shrub species was also tried at the Oak Harbor site. Success was poor, in large part due to the failure of supporting structures. The future use of this type of planting will have to await longer term records of (successfully) established plants.

V. SUMMARY CRITIQUE

This section summarizes the significant findings of the program, with emphasis on system-to-site adaptability, effectiveness of each system in protecting a bluff or preserving or widening a beach, and suitability and economy of materials used in each system. The critique summarizes by categories the systems as covered in Section III, but with fewer site-specific details of performance and analysis. The year of installation is shown in parentheses immediately after the site designation. For more information on a given system, refer to discussions of that system in Sections II and III. Under each category of devices, the systems are discussed generally in order of decreasing success, economy, or effectiveness.

1. Bulkheads and Seawalls.

a. Purpose and Characteristics. This shore protection system is intended to prevent erosion of a bluff or backshore area, often at some detriment to the beach. It is characterized by a vertical or steeply sloping face, and is generally self-supporting or tied back at the top. The structure usually requires backfilling to the level of its crest. In principle, it absorbs or reflects wave energy that would otherwise attack and erode the material behind it. In doing this, the structure deflects water particles both downward, causing toe scour, and upward, causing wave overtopping. Bulkheads are usually distinguished from seawalls in that they primarily lend structural support to the retained fill; seawalls are primarily built to protect the backshore from wave attack. Common modes of failure are:

(1) The structure is damaged by wave impact or by wave-borne debris;

(2) the structure is damaged by foundation failure due to scour;

(3) backfill is washed out as a result of inadequate filter, allowing waves to pump retained material through voids or cracks in the structure, with resultant continued erosion behind the structure;

(4) bluff erosion continues because the structure is too low, allowing overtopping waves to erode the backshore area;

(5) the structure is pushed out of place or damaged by a slumping bluff or by pore pressure resulting from a clogged filter or lack of weep holes; and

(6) the structure is damaged by vandalism.

b. Successful Systems.

(1) Treated Timber. At Oak Harbor, Washington (1978), Buckroe Beach, Virginia (1967), and Folly Beach, South Carolina (date unknown), posts and sheathing backed with filter cloth and stone toe protection were provided. No damage occurred from waves up to 3.5 feet high. This system is applicable where posts and bottom sheathing can be embedded sufficiently to prevent undermining. At Buckroe Beach, the structure used sheet piling and wales, apparently with no filter. No structural damage occurred. At Folly Beach, the recently installed structure survived Hurricane David, but overtopping waves washed out the backfill. These bulkheads can be used only where timber piles can be driven or embedded solidly in drilled holes.

(2) Steel H-piles and Timber. At Port Wing, Wisconsin (1978), railroad tie "stop-log" fillers were placed between steel H-piles grouted into drilled holes in bedrock. The bulkhead was backed with filter cloth. No damage by ice or by wave-borne debris was noted. This device would be useful where bedrock prevents the driving of sheet piles. The high cost of this type of construction is a disadvantage.

(3) Concrete Sheet Piling. At Folly Beach, South Carolina (date unknown), apparently no filter was used. No damage to the main structure by Hurricane David was noted, but a corner cracked where the return wall started. This system is applicable only where sheet piles can be driven or jetted through sand. High cost is a disadvantage.

c. Partially Successful and Unsuccessful Systems.

(1) Rubber Tire and Post. At Oak Harbor, Washington (1978), gravel-filled rubber tires were stacked on posts, with and without filter material. Gravel fill washed out from inside the tires, the tires collapsed, and the backfill washed out. Grout fill in the top tires might have prevented the washouts. Filter-cloth backing proved to be better than gravel and probably would be needed to retain the backfill if tires were filled so as not to collapse. Costs escalate rapidly with increased difficulty of drilling the many post holes required. The system is not recommended as monitored, in view of the danger of the tires being displaced by high wave impact and the availability of probably more reliable and less costly systems.

(2) Longard Tubes. At Ashland, Wisconsin (1978), and Sanilac Section 11, Michigan (1974), waves overtopped the structures and caused the high clay bluff to slough, pushing and rolling the tubes lakeward. The exposed surfaces of the Ashland tubes were sand-epoxy coated, but the movement exposed uncoated areas which were then torn or punctured by debris and vandals. Holes and tears in uncoated material easily enlarge, spilling sand and lowering the height of the tube. Tubes should be located about 30 feet lakeward of the bluff toe to allow overtopping waves to build a back berm and to expend their energy thereon. The system is not recommended in areas where vandalism and floating debris could be a problem. However, the rapidity with which the tubes can be installed makes the system useful in emergency situations.

(3) Earth-filled Concrete Pipe. At Beach City, Texas (1976), there was no filter backing and pipes began tipping seaward after about 3 years in a relatively mild wave climate. The structure was repaired in 1980, but its performance was not monitored long enough to permit a reliable evaluation. The diameter of the pipe controls the allowable height of the structure. With a controlling diameter of 4 feet, not more than 4 feet of bulkhead can be exposed safely, and some entrenchment of the base is preferable. Although the system could be functional where a low structure would give adequate protection, it would be economical only where an ample supply of salvaged concrete pipe is available.

(4) Rubber-Tire Stack. At Port Wing, Wisconsin (1978), gravel-filled tires were stacked horizontally on filter cloth. The exposed face was stair-stepped up the slope and the entire stack was anchored to the bottom with screw anchors and tie rods at 10-foot intervals. The tires, fastened to each other only with driven spikes and pushnuts, pulled apart quickly under wave action, and many tires drifted away. The system is not recommended in view of better, less costly alternatives.

(5) Untreated Timber (Logs). At Oak Harbor, Washington (1978), filter material washed through cracks between the logs and the waves eventually knocked the logs loose from the posts. Filter cloth may have prevented the loss of backfill. Through-bolting of the sheathing logs to the posts might have prevented a structural failure. This device is cost competitive only where logs are inexpensive.

(6) Hogwire Fence and Sandbags. At Basin Bayou, Florida (1978), small sandfilled acrylic and polypropylene sandbags were stacked two and three rows wide, 3 feet high, behind a wire-mesh fence supported by timber posts. The polypropylene fabric was destroyed by exposure to sunlight in 6 months, spilling sand from the bags. The acrylic bags were not affected by sunlight but were undercut by toe scour and the posts tilted seaward. The bags, sliding downward against the fence, were ripped open, and some bags fell out beneath the fencing. The system is not recommended as designed. Possible improvements are discussed in Section III.

(7) Concrete and Timber. At Folly Beach, South Carolina (date unknown), concrete filler slabs between posts were leaning seaward when first monitored. Hurricane David knocked all slabs onto the beach. The system is not recommended.

d. Recommendations

(1) Check if a less costly system in a different category would serve just as well; e.g., slope revetment, beachfill, groins, etc.

(2) In high wave-energy areas, use a reinforced concrete seawall of adequate design or a steel sheet-pile bulkhead. Such a system can not be kept in the low-cost range.

(3) Where waves do not exceed 5 feet, a timber sheet piling or timber post and sheathing system may be possible.

(4) Where potentially suitable special materials are available at little or no cost, or where they need to be disposed of in any event, design a new system incorporating such materials, but avoiding the design deficiencies and heeding limitations described in Subsection c above.

2. Revetments.

a. Purpose and Characteristics. Revetments are also intended to prevent erosion of a bluff or backshore area, but unlike bulkheads and seawalls, they lie on a slope, which may be either a natural bank or a slope-graded fill along the shore. The slope must be relatively flat and the practical limit is about 1.5 feet horizontal for each foot vertical. In normal practice, slopes are commonly 1 on 2 to 1 on 4. The revetment material serves as an armor layer on which the waves break and runup. Revetments tend to reflect less wave energy than vertical walls and therefore experience fewer difficulties from reflection and toe scour. Wave runup and overtopping are generally greater for revetments than for bulkheads of equal height. Runup can be reduced by increasing the roughness of the revetment surface, and the structural integrity often depends on the weight or interlocking of revetment modules. Also, a slope that might not be stable under wave attack with only lightweight armoring can often be made stable by the additional weight of a heavy revetment. Common modes of failure are:

(1) Revetment modules are displaced by wave impact.

(2) Toe scour undermines the revetment toe, causing the revetment to slide down the slope.

(3) Fine material beneath the revetment is pumped through voids in the armor because of inadequate filtering.

(4) The slope becomes unstable and slumps because of excessive pore pressures resulting from a clogged filter or a lack of weep holes. Such slumps carry the revetment with them.

(5) Cut or fill slopes are too steep and fail, destroying the revetment.

(6) Consolidation or shrinkage of embankment material causes uneven settlement of revetment modules, making them more vulnerable to displacement by wave action.

(7) Modules are stolen.

b. Successful Systems.

(1) Stone Riprap. All systems at Folly Beach, South Carolina (date unknown), Muskegon, Michigan (1978), and Tawas Point, Michigan (1974), survived design-wave episodes with little or no damage. Hurricane waves scattered small-sized armor stone at Folly Beach. High waves or ice displaced some armor stone and pumped bluff fill material through the voids in the Muskegon revetment, causing some sliding of the high sandfill it protected. With adequate filter and armor stone size, this system is recommended wherever stone is available at a reasonable cost.

(2) Sand-Cement-Filled Bags. Oak Harbor, Washington (1978) and Alameda, California (1978). The revetment at Oak Harbor was the steepest successful revetment, being two bags thick on a 1 on 1 slope. There was no filter layer; however, granular material with weep holes was provided. It survived 3.5-foot design waves with virtually no damage, but toe rock helped to protect the structure. Burlap bags filled with wet-mix grout worked better than dry-mix-filled paper bags which were torn open and saturated during placement. A somewhat similar revetment at Alameda was soon washed out as a result of toe scour and bank sloughing, but failure was attributed to the steep, 60° slope, and to the single-bag thickness of the revetment; therefore, the Alameda failure was due to design deficiencies. The Oak Harbor installation performed well and is recommended for low-cost protection. This device is recommended only for low-energy areas.

(3) Concrete Blocks. All systems at Port Wing, Wisconsin (1978), Holly Beach, Louisiana (1974), Fontainebleau, Louisiana (1979), Little Girls Point, Michigan (1973), and Stuart and Jensen Beach Causeways, Florida (1980), performed satisfactorily where placed on a stable slope, provided with an adequate filtering system, and not subjected to waves exceeding recommended design limits. Failure of Turfblocks (Monoslabs) at Port Wing, where the modules were displaced but not broken, resulted from either an unstable slope or filter blockage; the exact cause has not been determined. Gobi blocks at Holly Beach and Fontainebleau survived normal

wave action but were displaced by high storm waves. Some of the blocks were stolen; however, use of Gobi-mats would make theft difficult. Also, Jumbo blocks and Gobi-mats survived higher waves. Nami rings at Little Girls Point were broken by wave-borne cobbles and floating debris. Common hollow building blocks seemed to perform best when laid perpendicular to shore, but many were stolen at Fontainebleau. At Port Wing, control blocks (building blocks with interlocking protrusions in their ends) performed well, but many were abraded by wave-borne cobbles and gravel. Use of concrete-block systems should be limited as follows:

(a) Place blocks only on a stable slope and bury the toe.

(b) Compact fill materials beneath the revetment.

(c) Provide an adequate filter system, preferably a woven filter cloth.

(d) Weight of individual units should be adequate to resist displacement by design waves at the site.

(e) Concrete must be quality-controlled. Standard building blocks are too weak and friable. Special concrete mixes should be used for these units.

(f) Do not use where blocks may either be stolen or damaged by wave-borne cobbles, ice, or debris.

(4) Gabions. Oak Harbor, Washington (1978), Ninilchik, Alaska (1978), and Kotzebue, Alaska (1978). The mattress-type revetment at Oak Harbor was overtopped, the upper end settled because the filter was not extended high enough, and some undersized stones washed out of baskets, but the system still performed well. Settlement and damage were more pronounced in the section without a filter. The Ninilchik revetment of tiered gabions accreted sand, which helped preserve the structure. However, the tops of the exposed baskets were ripped open by cobble bombardment in the wave zone. Lack of larger sized fill stones led to experimentation with filter cloth and wire-screen liners to contain the available gravel. Sand- and gravel-filled sandbags were also used to fill baskets at Kotzebue. Early evidence indicated that these liner devices would not last long under large wave agitation. Use of gabion revetments should be limited as follows:

(a) Gabions may be used where 4- to 8-inch stone is available. Stones less than 4 inches should be removed by processing.

(b) Use of interior liners and sandbags to contain smaller sized material is not recommended.

(c) A filter layer should be used under the gabions, preferably a cloth filter.

(d) A tiered construction should be used to accrete sand and gravel, except at clay-silt shorelines.

(e) Gabions should not be used where bombardment by wave-borne logs or cobbles will result in holes in baskets.

(f) Gabions should not be used where foot traffic over the baskets is anticipated.

(g) The baskets should be filled tightly to prevent movement of the stone, which abraids the wire and leads to premature failure.

(h) Gabions should be refilled as necessary to maintain tight packing.

c. Partially Successful and Unsuccessful Systems.

(1) Concrete Rubble. Shoreacres, Texas (1976) and Alameda, California (1978). At Shoreacres, a large section of dumped rubble without filter and without shape-sizing peformed well in a mild wave environment because of the sheer size of the revetment. At Alameda, a single layer of large flat pieces was laid in mosaic fashion over filter cloth. It performed fairly well because the fronting beach built up as a result of another demonstration structure. For that reason its good performance was discounted. Historically, at Alameda, concrete-rubble revetments without filters have been unable to halt erosion of the fine sandbank. Although not adequately tested in the program, a concrete-rubble revetment designed and constructed as suggested in Section III would be successful.

(2) Steel Fuel Barrels. At Kotzebue, Alaska (1978), a revetment comprised of two double rows of barrels, 10 feet apart, with connecting barrel diaphragms, performed well during the short monitoring period. However, the short life of a similar revetment constructed earlier at Ninilchik, Alaska, due to corrosion of the barrels, ruled out the system as being reliable for any length of time south of the Arctic Circle.

(3) Concrete Slabs. At Alameda, California (1978), a number of salvaged, precast building slabs were laid side-by-side against filter cloth on a 60° slope. Wave overtopping saturated the find sand of the embankment, uneven settlement of the slabs tore the filter cloth, and the embankment was washed out from behind the structure. Although the revetment might have been successful on a flatter slope, the danger of the filter cloth tearing by slight displacement of these large, heavy modules makes the system unsuitable for use in this type of revetment. Where large slabs of concrete are available at low cost, consideration should be given to their use as panels in a seawall.

(4) Sandfilled Bags. The revetment at Alameda, California (1978), had the same design as an adjacent device with sand-cement-filled bags and failed because of other design deficiencies. However, the bag material also failed and spilled sand, indicating that failure was imminent even if the other design factors had been adequate.

(5) Fabric. Alameda, California (1978), and Fontainebleau, Louisiana (1979). At Alameda, a Fabriform filter-point nylon mat filled with sand failed as a result of bank sloughing and tears in the fabric which spilled the sandfill. The slope was too steep, but weakness of the fabric itself, without the concrete-grout fill normally used with Fabriform, was no match for the wave environment. Performance of a mat, filled with concrete as recommended by the supplier, and on a milder slope, would probably have been successful but such a system was not demonstrated. At Fontainebleau, pocket and loop filter cloths were covered with shells and topsoil and then planted with grass. Thus, as long as the grass held the soil in place, the cloth was not exposed. However, used in that manner, it is not really a fabric revetment. The use of any fabric as the primary resisting layer of a revetment is not recommended.

d. Recommendations.

(1) Check the cost of a well-designed stone-rubble revetment before selecting any other type because such structures have proved to be the most reliable and economical over the years.

(2) If other materials are available at lower cost, examine the findings of this report and be guided by suggested improvements over systems monitored in the program when using these materials in a revetment.

(3) Be certain that module weight, slope stability, filter adequacy, and environmental conditions (such as wave heights and tidal range) are all accounted for in the design.

(4) Design for nonovertopping, or provide protection if overtopping is expected.

(5) Make adequate provision for toe scour and potential loss of beach material from in front of the revetment.

3. Breakwaters and Sills.

a. Purpose and Characteristics. The use of breakwaters for shore protection differs from the normal use of harbor-type breakwaters and jetties in that, with shore protection breakwaters, full protection from waves is not required and usually is not desired. Adequate protection can usually be achieved if the structure causes the waves to break, but still transmits some of the wave energy. The transmitted energy should be sufficient to prevent the formation of a tombolo between the natural shoreline and the structure and to bypass littoral drift. A complete tombolo would prevent longshore transport past the structure and cause erosion along the downdrift shoreline. Therefore, the shore protection breakwater is often a low barrier designed for a considerable amount of overtopping, or it may be a permeable structure. When the structure is so low that its crest remains submerged all or most of the time, it becomes a sill. The sill is often used to maintain a perched beach, and for that purpose it is better that it be impermeable from foundation to crest in order to prevent loss of retained sand through voids in the structure.

(1) The structure is damaged by wave impact or wave-borne debris.

(2) Toe scour causes the structure to tip or to be displaced unevenly, with resultant structural damage.

(3) Fixed breakwaters may be too low or too porous, thereby increasing wave transmission and reducing effectiveness.

(4) Floating breakwaters may be displaced because of anchorage failure.

(5) Floating breakwaters may allow too much wave transmission for the structure to be effective.

(6) Sills may be porous and leak their retained fills.

(7) Structures may be damaged by vandalism.

b. Successful Systems.

(1) Stone Rubble. Kitts Hummock, Delaware (1979), and Siuslaw River, Oregon (1974). At Kitts Hummock, the segment founded on matstone settled about 6 inches after construction. A segment founded on filter cloth did not settle. The crest elevation at about mean tide level successfully protected the beach fill in its lee. No damage to the structure was reported after 1 year. At Siuslaw River, a longitudinal groin, which is, in effect, a breakwater, prevented bank erosion in its lee and no structural damage occurred. The system is applicable wherever satisfactory stone rubble is available at a reasonable cost.

(2) Timber Sheet Pile. At Slaughter Beach, Delaware (1979), treated tongue-and-groove sheet piling was used to create a sill. No structural damage has occurred. The sill, with crest elevation just below MLW, retained the perched beach fill during the 9-month monitoring period. The system is applicable only in areas where timber sheet piling can be driven to sufficient depths to guarantee stability.

(3) Rubber Tires on Timber Piles. The structure at Fontainebleau, Louisiana (1979), has caused sand to accumulate in its lee, with attendant erosion of the downdrift shore. Several arrangements of piles were used, but they were not monitored long enough to determine which arrangement works best. No structural damage has been reported. The system is applicable wherever used tires are available at little or no cost and where timber piles can be driven to sufficient depth for stability.

(4) Rolling Truck Tires on Piles. At Fontainebleau, Louisiana (1979), some anchor rods have bent and the tires have filled with sand and will not float at high tide. No other structural damage has occurred. Severe anchor stress has been noted, and screw anchors should be set deeper to prevent pullout. There is little evidence of a tombolo forming, as waves are not adequately attenuated at high lake levels, but sufficient time has not elapsed to reliably evaluate its functional performance. It is applicable only where the tidal range is low and where used truck tires are available at little or no cost.

(5) Sand-Cement-Filled Bags. Fontainebleau, Louisiana (1979), and Alameda, California (1978). At Fontainebleau, five layers of small bags were hand-placed over woven filter cloth and the cloth was wrapped back over the top of the sill. No structural damage occurred, but storm waves displaced the top bags from a test section without the filter-cloth wrap. Sufficient time has not elapsed to assess the rate of deterioration of the filter cloth acting as an armor layer. At Alameda, large bags, filled in place with side forms, created a sill with a rectangular cross section. The structure settled several inches irregularly, but it retained sand cast over its top into the lee area by waves and protected vegetation plantings. No significant structural damage occurred. It is applicable only where the tidal range is moderate and the bottom slope is fairly flat.

(6) Floating Tire. At Pickering Beach, Delaware (1978), the Goodyear system dragged its concrete anchors; pile anchors were then used and they held. Conveyor belt edging used to tie modules together chafed on tire rims. The monitoring period was too short to determine whether the ties would eventually be cut. The system protected the beach in its lee, causing some accretion. It is applicable only where short-period waves prevail, but its effectiveness is not reduced by large tidal ranges.

c. Partially Successful and Unsuccessful Systems.

(1) Floating Tire. Pickering Beach, Delaware (1978), and Stuart and Jensen Beach Causeways, Florida (1979). At Pickering Beach, the Wave-Maze system which broke up after being anchored in place, was considered partially successful. A more secure bolting system would have held the Wave-Maze together. At Stuart and Jensen Beach Causeways, the University of Rhode Island system broke loose and drifted ashore during Hurricane David. The synthetic fiber rope ties were cut by the tire beads. The system was reinstalled in the spring of 1980 but there has not been sufficient time to evaluate its performance.

(2) Longard Tubes. The breakwaters at Alameda, California (1978), and Basin Bayou, Florida (1978), functioned well, but both installations were badly damaged by vandals. The Alameda tube was eventually removed. The Basin Bayou tube was rebuilt using a jacket of aluminum sheathing in the spring of 1980. The aluminum sheathing was also unsucessful and had to be removed, as it posed a danger to bathers. There has been insufficient time for monitoring to permit a fair evaluation of the performance. Longard tubes could be effective if the vandalism problem were overcome.

(3) Gabions. The gabion breakwater at Geneva State Park, Ohio (1978), suffered major structural damage. The toe mattresses were undermined

and deflected downward, stretching and breaking the wire mesh and allowing the stone fill to escape. Upper baskets broke open as wave agitation hurled loose stones about inside the cages. This system is not recommended for breakwater construction at high wave-energy sites. It could be quite successful where waves heights do not exceed about 3 feet.

(4) Concrete Boxes. At Kitts Hummock, Delaware (1978), and Slaughter Beach, Delaware (1978), sills comprising boxes 2 feet high and 4 feet high respectively, were set end-to-end and sandfilled. The boxes settled unevenly and some boxes were displaced. Sandfill was lost and had to be replaced within a few months, but the system functioned reasonably well. This system appears to be effective in mild wave climates. However, the boxes should be capped to prevent loss of sandfill.

(5) Z-Wall. At Geneva State Park, Ohio (1978), the uneven settlement of panels in this structure, coupled with the concentration of wave forces in the landward V-junctions, resulted in progressive loss of end panels. Until a better method of connecting the panels can be devised, this system must be considered structurally deficient, and its use is not recommended.

(6) Sta-Pods. The system at Geneva State Park, Ohio (1978), although structurally undamaged, transmitted too much wave energy to be effective. Until a method is found to reduce its porosity, its use as a breakwater is not recommended.

(7) Sandfilled Bags. At Kitts Hummock, Delaware (1978), Slaughter Beach, Delaware (1978), Buckroe Beach, Virginia (unknown date), and Roanoke Island, North Carolina (unknown date), sills comprising large bags filled in place shrunk in overall dimensions when filled, leaving gaps that leaked retained perched beach fill and allowed too much wave energy to penetrate lee areas. Two types of bags were used: Advance Bags, which had neck-closure problems that eventually leaked sand, and Dura-Bags which performed better; however, the monitoring period was too short to determine how long the bag fabric would last. Sandfilled-bag sills could be effective if placed without gaps, in mild wave climates, where vandalism could be controlled.

(8) Surgebreaker. The device at Basin Bayou, Florida (1979), was placed 200 feet offshore. A helicopter placed the 3,700-pound modules which were guided into place by a wading crew. No structural damage has occurred in 7 months, but it is too soon to evaluate the functional performance of the device. It seems best suited for use in firm, smooth-bottomed areas where the tidal range is moderate.

(9) Sandgrabber. Folly Beach, South Carolina (date unknown), Bellows Air Force Station, Hawaii (1979), Kualoa, Hawaii (1977), and Basin Bayou, Florida (1978). Although not strictly a sill or a breakwater, the Sandgrabber functions in somewhat the same manner. However, the porosity of the structure allows retained material to escape under high wave-energy conditions. At all sites, the Sandgrabber deflected downward unevenly at the toe, causing the steel ties to break blocks. At Folly Beach, where two

Sandgrabbers were placed in tandem, the seaward structure was virtually destroyed in a few years, and the landward structure was seriously damaged. The Bellows Sandgrabber was buried apparently as a result of a change in the wave climate, but remains as a protective device for future erosional trends. The Kualoa structure trapped sand at the expense of the downdrift beach, but broken blocks herald its early demise under continued exposure. In the milder wave climate at Basin Bayou, the Sandgrabber has rotated downward but appears to remain functional more as a revetment than as a breakwater or sill. The system as presently designed is not recommended in view of other breakwater and sill systems that have performed better.

(10) Brush Dike. This system at Fontainebleau, Louisiana (1979), was constructed of driven posts which were cross-tied and filled with brush. Most of the brush washed out within 1 year. The system is not recommended as constructed. It might be improved by closer spacing of posts, larger sized brush, or a sheathing of posts to retain the small brush.

d. Recommendations.

(1) Use fixed breakwaters for shore protection only where the offshore slope is relatively flat and tidal ranges or water level fluctuations are small. If the water is too deep at a distance of 200 feet or more offshore, consider using a revetment or bulkhead at the shoreline or a floating breakwater instead.

(2) Where the depth 200 feet offshore exceeds 3 or 4 feet and wave periods are short, a floating tire breakwater may be more economical than a fixed breakwater.

(3) Use fixed or floating breakwaters where vegetation is to be used as a shore stabilization measure. Design the structures to attenuate the waves adequately to allow the vegetation to become well established.

(4) Examine the bottom substrate to determine whether timber sheet piling can be driven or whether a gravity structure resting on the bottom, such as rubble mound and sandbags, should be used.

(5) Provide adequate filtering under porous gravity structures to prevent their sinking into soft bottom materials.

(6) Provide adequate anchorage for floating tire breakwaters. Examine the findings of this report to determine which types of anchors have failed and which have held.

(7) Examine the findings of this report to determine which breakwater or sill systems have proved structurally adequate.

4. Groins.

a. Purpose and Characteristics. Groins are structures built perpendicular to shore for the purpose of trapping fillets of littoral drift along shorelines where longshore transport is predominantly in one direction and where their possible impedance of longshore transport will not result in unacceptable erosion of the downdrift shore. Each fillet shoreline tends to become oriented parallel to the breaker line, thereby reducing the capacity of the waves to transport littoral drift laterally. A properly designed groin field, when filled to capacity, will allow the drift to be transported around the tips or over the crest of each groin at about the same rate as previously existed without the groins. The fillets of accretion act as buffers on which the waves expend their energy and transport sand without eroding the bluff or backshore area. Common modes of failure are:

(1) Wave action at the site is multidirectional without a predominance of waves with a longshore component of energy in either direction. As a result, the groins do not effectively trap littoral drift, and erosion of the bluff or backshore continues as it did without the groins.

(2) The groins are too long, holding the drift but allowing little of it to be transported to the downdrift coast. The groin field protects the shoreline at the site but aggravates erosion of the downdrift shoreline. Also, rip currents develop at the groins, carrying sand offshore into deep water, where waves cannot return sand to the beach.

(3) The groins are spaced too far apart, so that the fillets of accretion do not extend updrift of each structure to the root of the updrift groin. As a result, erosion of the shoreline for some distance downdrift of each groin continues as it did without the groins.

(4) The groins are too close together and cause bypassing material to be jetted offshore and no accretion in the compartments.

(5) The groins are too porous, allowing wave turbulence to wash large quantities of sand through voids in the structures and preventing them from trapping fillets wide enough to protect the backshore area.

(6) The groins fail structurally, through vandalism or as a result of wave action, and they become ineffective in trapping drift.

(7) The groins do not extend far enough shoreward and become outflanked, allowing the waves to bypass drift around their inshore ends.

b. Successful Systems.

(1) Timber. Lincoln Township, Michigan (1973), Buckroe Beach, Virginia (1967), Broadkill Beach, Delaware (1950; now buried), and Ninilchik, Alaska (1978). At all but the Alaska site, the treated-timber sheet pile groins show little deterioration even though some have had many years of exposure. All are performing well. Use of this system is limited to sites where timber sheet piling can be driven to adequate depths at high wave-energy sites. Round timber brace piles may be needed near the seaward ends. At Ninilchik, two types of timber groins were tested. The first (1974) was a crib type with two rows of spruce posts cross braced with planks and sheathed on the outside with 3- by 12-inch planks. The second (1978) used single 3- by 12-inch plank sheathing sandwiched between pairs of spruce posts cable-tied together. These sturdy groins survived alternate icing-in and wave-borne cobble bombardment with only minor abrasion and weathering. Construction high in the tidal zone enabled placement of sheathing and posts in the dry to depths of 8 feet below the ground line. These horizontally sheathed systems may be limited to installation where the tidal range is very large.

(2) Timber and Rock. Folly Beach, South Carolina (date unknown), Broadkill Beach, Delaware (1954), and Sanilac Section 26, Michigan (1975). At Folly Beach, treated-timber sheet-pile groins were reinforced with stone rubble at their seaward ends to provide lateral stability. They have survived 10 to 15 years with no apparent damage and have functioned well. This combination of timber and rock may reduce costs where rock is relatively expensive and the structures extend into relatively deep water. Its use is limited to sites where timber sheet piling can be driven to adequate depth. At Broadkill Beach, the groins were similar to those at Folly Beach and performed equally well except that the rock was placed only on the downdrift side along the inshore end to brace against overturning by the updrift sand fillet. At Sanilac Section 26, a timber-crib groin with rockfill remained in good condition except for the loss of rock at the lakeward end due to vandalism or wave action. Nonetheless, it was still functional. The system may be useful where piling cannot be driven and the groins only need to extend into about 2 feet of water.

(3) Stone Rubble. Siuslaw River, Oregon (1974), and Sanilac Section 26, Michigan (1973). At Siuslaw River, the groins remained structurally sound and functionally effective as spur dikes, deflecting river currents rather than trapping fillets of littoral drift. At Sanilac Section 26, the use of asphalt mastic to seal the voids in a rubble-mound groin proved to be effective although it is known that the hot mastic will not penetrate voids in the submerged parts of the section. Apparently this was not necessary at this site. Some of the end stones were displaced by wave action, but the groin continued to trap enough sand to protect the bluff and bypassed enough to keep downdrift groins filled to capacity. Rubble-mound groins are recommended for use only in relatively shallow water, as their cost increases greatly with depth. Porosity can be reduced below the waterline by using small stones in the core, and above the waterline by using asphalt mastic, concrete grout, or an upward extension of the small core stone into the crown of the structure. The latter usually requires a wider section, which increases the cost.

(4) Concrete Rubble. At Broadkill Beach, Delaware (1964), construction data are lacking, but the groins are still in good condition and are performing well. These are recommended primarily where large quantities of good quality rubble must be disposed of; however, they are universally applicable. The size of the rubble in the armor layer must be checked for adequacy in the local wave climate.

(5) Sand-Cement-Filled Bags. At Alameda, California (1978), large bags were shaped into blocks, by the use of forms, and stacked 5 feet high. The nylon bag fabric deteriorated, but the blocks held their shape and retained a large fillet of sand. The system is universally applicable but high cost is a disadvantage.

(6) Corrugated Metal Pipe. At Ninilchik, Alaska (1979), sections of 48-inch pipe were embedded upright, 8 feet into the bottom side-by-side, each was filled with gravel and capped with concrete. No structural damage occurred but the system can be used only where embedment of pipe is not difficult. High cost is a disadvantage.

(7) Rock Asphalt Mastic. This rock groin at Sanilac Section 26, Michigan (1973), impregnated with asphalt mastic to seal the voids, functioned very well, despite the loss of rock from the outer end. The high cost of the mastic at current prices makes it uneconomical as a void sealant.

c. Partially Successful and Unsuccessful Systems.

(1) Longard Tubes. At Ashland, Wisconsin (1978), Lincoln Township, Michigan (1973), and Sanilac Section 26, Michigan (1973), the tube fabric was torn by wave-borne debris and possibly by vandals. The sandfill was washed out by wave action. This system is recommended only where large floating debris is not present and vandalism can be prevented.

(2) Gabions. Kotzebue, Alaska (1978), Ninilchik, Alaska (1978), and Sanilac Section 26, Michigan (1974). At the Alaska sites, gabion baskets were lined with wire screening or filter cloth and were filled with beach cobbles. These failed in several ways. First, the liners came apart or tore open, and the small cobbles were washed through the basket mesh. Second, the stones inside the partially filled baskets were hurled about by the waves, which abraded and broke the basket mesh. Settlement and distortion of the baskets broke wires in the mesh. At Sanilac Section 26, the gabion baskets filled with larger stones settled at the outer end. The baskets were distorted and broke open, and the stone fill washed out. The system could be effective in milder wave climates but it is not recommended until more experience is gained.

(3) Steel Fuel Barrels. At Kotzebue, Alaska (1978), barrels placed upright and side-by-side were bolted together at the contact points and filled with beach gravel. Barrels near the outer ends were capped with gravel-filled sandbags. Wave-borne debris and possibly ice crushed or distorted many barrels, indicating a short life in the local environment. The system may be useful in outpost areas where steel barrels have little or no salvage value, but earlier experience at Ninilchik showed that the barrels soon rust out and become ineffective. However, the corrosive environment at Ninilchik is more severe than at Kotzebue.

(4) Sandfilled Bags. Kotzebue, Alaska (1978), Bowers, Delaware (1976), and Sanilac Section 26, Michigan (1973). The small bags used at Kotzebue were displaced by 3-foot waves during construction. The large bags used at Sanilac Section 26 were ripped open by wave-borne debris. The large nylon bags at Bowers survived best and functioned well but the exposed fabric is deteriorating and the sandfill is being lost. Large bags may be used where no large floating debris exists and where vandalism can be controlled, but the exposed fabric may last only a few years.

d. Recommendations.

(1) Use groins for shore protection only where longshore transport is predominantly in one direction.

(2) Use methods outlined in the CERC Shore Protection Manual to determine the proper length and spacing of groins.

(3) If possible, fill the groin compartments completely with material coarser than the native sediments to help prevent erosion of the downdrift beaches.

(4) Where large floating debris is a hazard, use sturdy construction materials and systems.

(5) Check the nature of bottom materials to determine whether piling can be driven or posts and sheathing can be embedded.

(6) Check the availability and costs of materials to determine those most suitable for groin construction under conditions found at the site.

5. Nonstructural Systems.

a. Purpose and Characteristics. Nonstructural systems comprise perched beaches, beach fills with or without recycling, and vegetation. The purpose of each of these systems is to prevent erosion of the backshore area without the use of structural devices or to supplement the protection provided by such devices. Perched beaches are sandfills placed in front of an eroding shoreline and retained by a frontal low sill and by side sills or groins on one or both sides. Beach fills involve the periodic placement of sand on a beach to maintain its width against wave forces that deplete it. The sand may be taken from a drift-collecting area downdrift of the site, from offshore deposits, or from inland sources. With a recycling scheme, the material is trapped at the downdrift end of the project where it is periodically collected and transported back to the updrift side. This may be a low-cost procedure for a long segment of shoreline, but it may not be a do-it-yourself solution in that some technical sophistication may, at times, be required. Vegetation involves the planting and cultivation of grasses or other ground cover in beach and backshore areas as a means of preventing erosion of sand by waves and wind. By itself, vegetation is sometimes effective in mild wave climates, but in more severe exposures, some wave attenuation by structural devices may be needed to allow the vegetation to become established and remain effective.

b. Perched Beach. Both systems at Alameda, California (1978), and Slaughter Beach, Delaware (1979), were effective in retaining sand and providing a wider beach, but some losses occurred because of leakage of sand through voids or over low areas in sills. The short-term monitoring of these two installations did not provide adequate data on their long-term effectiveness or the physical characteristics that make perched beaches effective. Prior model studies at the U.S. Army Coastal Engineering Research Center have indicated that high, steep waves tend to create great turbulence behind the sill and move large quantities of retained sand seaward over its crest. It would appear that the best wave climate for a perched beach is one in which wave height and steepness seldom exceeds that of normal wave events. Also, it is apparent that the retaining sills must be sand tight and of uniform height. The demonstration indicates that perched beaches have excellent potential for low-cost shore protection, especially where it is desired to maintain a recreational beach. Caution should be exercised in their use for recreational beaches because there is a danger that waders may accidentally step off them into deep water. However, more monitoring is required to determine the optimum distance offshore to place the sill, as well as the types of wave climate and the tidal ranges that are compatible with the use of the system.

c. Beach Fill (With or Without Recycling). All the systems at Alameda, California (1978), Bowers, Delaware (1973), Broadkill Beach, Delaware (1973), Lewes, Delaware (1975), and Sunnyside Beach, Washington (1975), were effective in maintaining the beach widths, but the loss rates varied. Long groins used at Bowers, where the longshore transport rate was about equal in each direction, were effective in preventing lateral losses to adjacent beaches. Short groins at Broadkill Beach and at Alameda effectively reduced the downdrift losses where littoral transport was more unidirectional. No groins were at Lewes, but the longshore transport distributed sand from a feeder beach over a mile of shoreline to maintain a relatively uniform width. No groins were at Sunnyside Beach, but the fills maintained beach width against continued slow offshore losses. Fill sand used at Alameda and Sunnyside Beach was trucked to the site from inland sources. Sand used at the Delaware Bay sites was pumped onto the beach from offshore deposits by a hydraulic dredge and pipeline. Beach fills are often more cost-effective and generally environmentally acceptable than structural devices where a medium-to-coarse sand borrow source is nearby and where normal losses of beach material from the site are not excessive. Where the losses are due primarily to longshore transport, the use of groins to reduce the rate of fill loss should be considered.

d. Vegetation.

A summary of vegetation performance is presented in Table 5-1. Data on performance of each species are presented by geographic regions and by individual sites.

Table 5-1. Vegetation summary.

Sites	Species	Success	Comments
Vegetation alone			
<u>Atlantic Coast</u>			
Pickering Beach, Slaughter Beach, and Kitts Hummock, Del.;	Smooth cordgrass	Unsuccessful	Too exposed to waves; soil too sandy; horseshoe crab damage
Uncle Henry's Fish Camp, Bogue Sound, and Duck, N.C.;		Moderate to good	Good in areas of low wave action and little foot traffic; high salinity remains a problem
Stuart-Jensen Beach Causeways, Fla.		Moderate to good	Best with plugs; not uprooted
Pickering Beach, Slaughter Beach, and Kitts Hummock, Del.;	Saltmeadow cordgrass	Poor to moderate	Better than smooth cordgrass; less subject to wave damage
Stuart-Jensen Beach Causeway, Fla.		Moderate to good	Data insufficient; good establishment
Pickering Beach, Slaughter Beach, and Kitts Hummock, Del.; Duck, N.C.	American beachgrass	Good	Some loss; looks like best species for upper beach
Stuart-Jensen Beach Causeways, Fla.	Siltgrass	Good	Naturally occurring; provides good protection
Stuart-Jensen Beach Causeways, Fla.	Red mangrove	Moderate to good	Seems to establish well if allowed time before high waves and winds disrupt
Stuart-Jensen Beach Causeways, Fla.	Black and white mangroves	Moderate	Little data; appears to do well in protected sites
<u>Gulf Coast</u>			
Basin Bayou, Fla.	Smooth cordgrass	Unsuccessful	Too much sand movement; unlikely to ever be successful at this site if unprotected
Fontainebleau, La.		Moderate to good	Plugs and sprigs on peat soils did very well; many plants survived even severe storm damage; not good in loose sand
Basin Bayou, Fla.	Saltmeadow cordgrass	Unsuccessful	Too much sand movement; extensive storm damage
Fontainebleau, La.		Poor to moderate	Loose sand buried plants, although some still remain; abundant in marsh behind site; needs further evaluation
Fontainebleau, La.	Common reed	Unsuccessful	Lost in storm; no further evaluation
Fontainebleau, La.	McCartney rose	Unsuccessful	Apparently buried by sand
Fontainebleau, La.	Torpedo grass	Successful	Natural colonizer; good for upper beach and dune stabilization
Key West, Fla.	Red mangrove	Poor to moderate	Poor survival on very open sites; larger plants survive better
<u>Pacific Coast</u>			
Alameda, Calif.	Pacific cordgrass	Unsuccessful	Destruction of Longard tube left device 16 unprotected, and drifting sand soon destroyed all plants.

Table 5-1. Vegetation summary.--Continued

Vegetation with structure			
Sites	Species	Success	Comments
Atlantic Coast			
Bogue Sound, N.C.	Smooth cordgrass	Very successful	Vegetation was used to protect toe of bulkhead at this site; excellent performance when planted near structure; low wave climate; spacing 24 inches on centers or less proved best; halted erosion in all areas where plants became established
Stuart-Jensen Beach Causeways, Fla.	Mangroves	Moderate to good	Difficult to evaluate in short time; appears to do better than unprotected plants
Gulf Coast			
Basin Bayou, Fla.	Smooth cordgrass	Unsuccessful	Sand movement behind Sandgrabber buried or uprooted plants; probably too high above tide line; unlikely to be successful here
Fontainebleau, La.		No report	All structures were installed after plantings were completed; high water level and storm waves washed out most plants
Basin Bayou, Fla.	Saltmeadow cordgrass	Poor to moderate	Seasonal movement of sand behind Sandgrabber due to storms was extreme; plants are hanging on, but because of sand movement are not doing well; unlikely that plants will become established under such conditions
Key West, Fla.	Mangroves	Good	Appear to do much better behind protection
Pacific Coast			
Alameda, Calif.	Pacific cordgrass	Good	Device 2 breakwater probably helped establishment of device 3 plantings; plugs did best; some loss where shingle wave breakers were used
Oak Harbor, Wash.	Shrub species (kinnikinik, salal, snowberry, ocean spray)	Poor	Only a few shrubs established, partly due to failure of the structures; probably would have to be replanted several times; success doubtful
Oak Harbor, Wash.	Grasses (wheatgrass, red fescue, etc.)	Good	Appears resistant to burying; reseeds or propagates naturally; could serve as groundcover; unlikely to halt bluff erosion
Great Lakes			
Geneva State Park, Ohio	Misc. plantings		Just planted, June 1980; no evaluation possible

6. Summary Analysis of Systems Investigated.

Performance data, summarized for each system, are presented in Table 5-2.

Table 5-2. Systems summary.

System	Comments
Bulkheads and Seawalls	
Treated timber[1]	Excellent performance; treatment extends life of timber; some construction difficulty
Steel and timber[1]	Excellent performance, but high cost; difficult to install
Concrete sheet pile[1]	Excellent performance, but high cost; needs filter and special equipment to install
Rubber tire and post[2]	Fair performance; needs good filter; tire fill material washed out; good way to dispose of used tires
Longard tube[2]	Tube must be away from bluff to prevent displacement by slides; sand-epoxy coating helps protect against vandal and debris damage
Earthfilled concrete pipe[2]	Fair performance; some pipes tipped over; needs stock pile of used pipe
Rubber tire stack[2]	Fair performance, but fasteners failed; system needs improvement; good way to dispose of used tires
Untreated timber[2]	System failed due to filter wash out; useful where logs are plentiful; boring insects could be a problem; needs good filter system
Hogwire fence and sandbags	System failed; could be improved, but short life of bag material is a problem
Concrete and timber	System failed; concrete and timber not compatible
Revetments	
Stone riprap[1]	Excellent performance; stone must be adequate size; filter is essential; recommended wherever low-cost stone is available; needs heavy handling equipment; best suited to large projects
Sand-cement-filled bags[1]	Good performance; easy to install but failed where design and installation were poor; good small project system
Concrete blocks[2]	Good performance where blocks were sized and shaped to match wave environment; easy to install, but subgrade must remain even; good small project system
Gabions[2]	Good performance, but broken basket wires may be a problem; needs proper sized stone fill; good substitute for stone riprap on small projects
Concrete rubble[2]	Good performance but failed where improperly designed; good way to dispose of large amounts of rubble; design criteria in text

Table 5-2. Systems summary.--Continued

System	Comments
Revetments	
Steel fuel barrels[2]	Good performance, but use limited to arctic regions
Concrete slabs	System failed; could be improved but use limited to availability of salvageable building slabs; other systems usually less costly
Sandfilled bags	System failed; not recommended, as bag material is too vulnerable
Fabric	System failed; might work with grout fill, but tests are needed
Tire and fabric	Storm waves displaced tires, and failure seemed imminent; method of stabilizing tires needed
Breakwaters and Sills	
Stone rubble[1]	Excellent performance in breakwaters, but high cost; requires special equipment
Timber sheet piles[1]	Excellent performance in low sills; requires special equipment and substrate must be suitable for driving piles
Tires on piles[1]	Good performance, but requires special equipment; more experimentation needed
Sand-cement-filled bags[1]	Good performance, but filter-cloth encasement at one demonstration site appears vulnerable
Floating tires[2]	Fair performance, but some systems pull apart; better interconnections needed; good way to dispose of used tires; use limited to short-period wave climate.
Longard tubes[2]	Good performance if tubes are not damaged; requires special equipment; vandalism of tubes made demonstrations inconclusive
Gabions[2]	Good performance, but structural failure seemed imminent at demonstration site
Concrete boxes[2]	Fair performance, but requires special equipment; covers needed to keep sandfill in boxes
Z-wall[2]	Good performance, but structure deteriorated; system not recommended until hinging of modules is improved
Sta-Pods	Poor performance, but structure undamaged; system not recommended until improved to attenuate waves better
Sandfilled bags	Poor performance; small bags not stable; large bags require special equipment, tend to pull apart when filled, leaving gaps; vulnerability of bag fabric makes dependability suspect
Surgebreaker[2]	System not monitored long enough to adequately evaluate performance.

Table 5-2. Systems Summary.--Continued

System	Comments
Sandgrabber[2]	System locally effective but depletes downdrift beaches; structures deteriorated; probably could be improved, but other shore protection systems are available that perform better
Brush dike	Brush washed out of cribs and system became ineffective; could be improved, and has potential for areas where large brush is plentiful
Groins	
Timber[1]	Excellent performance, but limited to sites where piling can be driven or embedded deeply; requires special equipment
Timber and rock[1]	Excellent performance; rock used to stabilize sheet piling or fill cribs
Stone rubble[1]	Excellent performance, but requires special equipment; high cost may be a problem
Concrete rubble[1]	Excellent performance at site monitored, but may not be stable with higher waves
Sand-cement-filled bags[1]	Excellent performance at demonstration site, but other types of groins may be less costly
Corrugated metal pipe[1]	Excellent performance at demonstration site, but other types of groins may be less costly
Rock asphalt mastic[1]	Good performance despite loss of groin end; high cost of mastic may make other sealant more economical
Longard tubes[2]	Good performance until structure failure; vandalism and debris damage is a problem
Gabions[2]	Good performance, but deterioration of outer ends exposed to high waves is a problem
Steel fuel barrels[2]	Good performance, but use is limited to arctic regions
Sandfilled bags	Good performance with large bags at one site, but bag fabric is now failing; small-bag groins soon failed
Perched beach[1]	System very effective at demonstration sites; special equipment needed; may be used for small projects
Beach fill[1]	System effective at most sites; special equipment needed; applicable to large projects
Artificial seaweed	Performance not evaluated; installed too late in program.
Nonstructural Systems	
Vegetation alone[2]	System effective in protected reaches; stability of plantings varies with species and substrate; professional guidance is needed in determining where vegetation might be effective and what species to use
Vegetation with structure[2]	System generally more effective than without structure, but environmental conditions control performance

[1]Systems that proved successful.

[2]Systems that could be made successful with minor changes or that should be used only in special environments or circumstances.

Note.--Unmarked systems are those that failed structurally or functionally.

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APPENDIX. GLOSSARY OF TERMS

Source of all definitions (except those marked with an asterisk) is the Shore Protection Manual (SPM) (U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1977). Those marked with an asterisk are definitions by Moffatt & Nichol, Engineers, Webster dictionary, or the Coastal Engineering Research Center staff.

ACCRETION - May be either NATURAL or ARTIFICIAL. Natural accretion is the buildup of land solely by the action of the forces of nature, on a BEACH by deposition of waterborne or airborne material. Artificial accretion is a similar buildup of land by reason of an act of man, such as the accretion formed by a groin, breakwater, or beach fill deposited by mechanical means.

*ADVANCE BAG - A spun-bonded polypropylene sandbag manufactured in large and small sizes by Advance Construction Specialty Company. The large size has a filling tube that must be tied off for closure.

ALONGSHORE - Parallel to and near the shoreline; same as LONGSHORE.

*ASPECT RATIO - As applied to stone or concrete rubble: the ratio of the largest dimension to the least dimension, i.e, the slenderness. The greater the aspect ratio, the more slender the object.

ATTENUATION - A lessening of the height of a wave as it moves across the water from the origin.

*BACKHOE - A power excavator similar to a power shovel except that the bucket faces the operator and is pulled toward him.

*BACKPASS - The transfer of sand, by mechanical means, back up the beach in the direction from which it was driven by the littoral current.

BACKSHORE - That zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water. It comprises the BERM or BERMS (Fig. A-1).

BAR - A fully or partly submerged embankment of sand, gravel, or other unconsolidated material built on the sea floor in shallow water by waves and currents.

BARRIER BEACH - A bar essentially parallel to the shore, the crest of which is above normal high water level. Also called OFFSHORE BARRIER and BARRIER ISLAND.

*BARRIER BEACH RIDGE - The ridge at the shoreward edge of the foreshore, usually the limit of uprush of the highest waves.

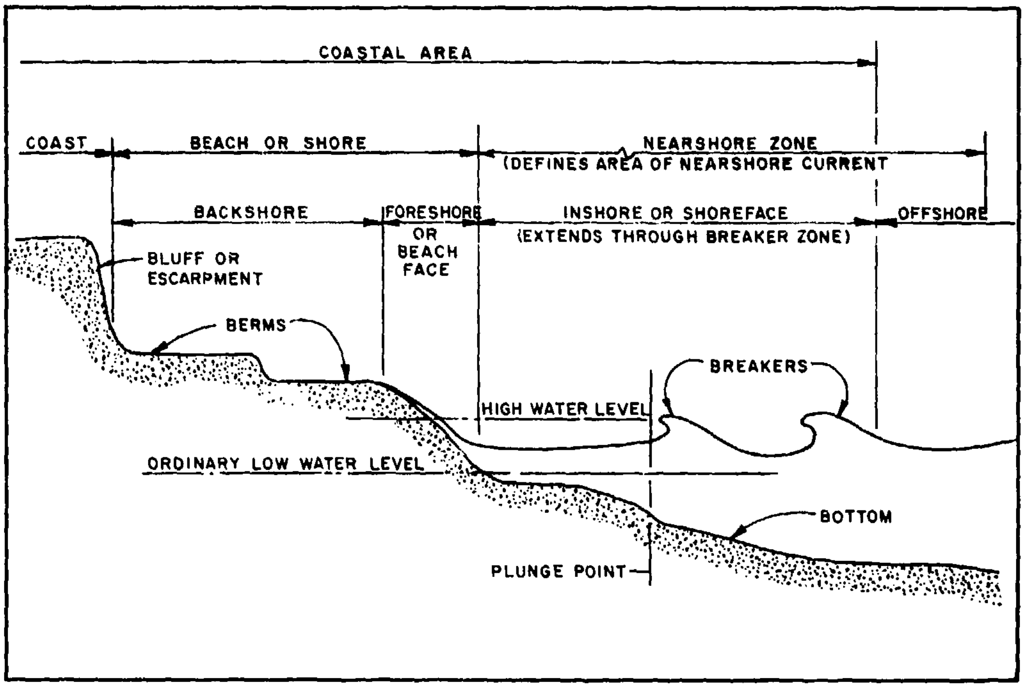

Figure A-1. Beach profile-related terms.

BARRIER REEF - A coral reef parallel to and separated from the coast by a lagoon that is too deep for coral growth. Generally, barrier reefs follow the coasts for long distances, and are cut through at irregular intervals by channels or passes.

BATHYMETRY - The measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

BAYOU - Derived from the French, refers to flat gradient, relatively slow moving coastal area channels.

BEACH - The zone of sand or gravel that extends landward from the low water line to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach - unless otherwise specified - is the mean low water line. A beach includes FORESHORE and BACKSHORE (Fig. A-1).

BEACH BERM - A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several (Fig. A-1).

BEACH EROSION - The carrying away of beach materials by wave action, tidal currents, littoral currents, or wind.

*BEACH-FILL - Sand or sandy material placed on a beach by mechanical means.

*BEACH, PERCHED - See PERCHED BEACH.

*BEDROCK - Ledge or base rock underlying a cover of unconsolidated material.

BENCH - A level or gently sloping erosion plane which is inclined seaward.

BENCH MARK - A permanently fixed surveyed point of known elevation.

BERM, BEACH - See BEACH BERM.

*BLOCK-CALVING - A process in which large blocks of soil fall from a bluff due to undercutting of the bluff base.

BLUFF - A high, steep bank or cliff.

*BLUFF LINE - The line along the top edge of a bluff.

BREAKER - A wave breaking on a shore, over a reef, etc.

BREAKER DEPTH - The stillwater depth at the point where a wave breaks. Also BREAKING DEPTH.

BREAKWATER - A structure protecting a shore area, harbor, anchorage, or basin from waves.

BULKHEAD - A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

CAUSEWAY - A raised road, across wet or marshy ground, or across water.

CHART DATUM - See DATUM, CHART.

*CHENIER - A beach ridge usually built upon alluvial deposits, as at Holly Beach, Louisiana.

CLAY - See SOIL CLASSIFICATION. Generally, fine-grained soils having particle diameters less than 0.002 millimeter.

*CLIFF - A high, steep face of consolidated material or rock.

COAST - A strip of land of indefinite width (may be several miles) that extends from the shoreline inland to the first major change in terrain features (Fig. A-1).

COASTLINE - (1) Technically, the line that forms the boundary between the COAST and the SHORE. (2) Commonly, the line that forms the boundary between the land and the water.

COBBLESTONE - See SOIL CLASSIFICATION. Typically, these are rounded stones having diameters between 64 and 256 millimeter.

CONTOUR - A line on a map or chart representing points of equal elevation with relation to a DATUM. It may be called an ISOBATH if it is a underwater contour.

*CONVERGENCE - A concentration of wave strength evidenced by localized heightening of the waves that may be caused by submerged terrain features over which the wave has passed, or, less frequently recognized, by interaction between waves and water currents across which the waves are moving.

CORAL - (1) (Biology) Marine coelenterates (Madreporaria), solitary or colonial, which form a hard external covering of calcium compounds, or other materials. The corals which form large reefs are limited to warm, shallow waters, while those forming solitary, minute growths may be found in colder waters to great depths. (2) (Hydrography) The concretion of coral polyps, composed almost wholly of calcium carbonate, forming reefs, and treelike and globular masses. May also include calcareous algae and other organisms producing calcareous secretions, such as bryozoans and hydrozoans.

*CRESCENTIC - Smoothly curved like, but not necessarily conforming to, the arc of a circle.

*CREST - As applied to a shore protection device, the upper prominence, edge, or limit of the structure.

CREST OF WAVE - (1) The highest part of a wave. (2) That part of the wave above stillwater level.

*CROSS SECTION - A section in a vertical plane showing profiles of ground surface and underlying material, providing a sectional view of a structure or beach.

*CUESTA - A hill or ridge with a steep face on one side and a gentle slope on the other.

*CULM - A single stem of grass.

CURRENT - A flow of water in a given direction.

CURRENT, COASTAL - One of the offshore currents flowing generally parallel to the shoreline in the deeper water beyond the near and surf zone. They are not related to waves and resulting surf, but they may be related to tides, winds, or distribution of mass.

CURRENT, DRIFT - A broad, shallow, slow-moving ocean or lake current.

CURRENT, LITTORAL - Any current in the breaker zone caused primarily by wave action, e.g., longshore current, rip current.

CURRENT, LONGSHORE - The current in the breaker zone moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shoreline.

DATUM, CHART - The plane or level to which soundings (or elevations) or tide heights are referenced. The surface is called a tidal datum when referred to a certain phase of tide. To provide a safety factor for navigation, some level lower than MEAN SEA LEVEL or NATIONAL GEODETIC VERTICAL DATUM is generally selected for hydrographic charts, such as MEAN LOW WATER or MEAN LOWER LOW WATER. See DATUM, PLANE.

DATUM PLANE - The horizontal plane to which soundings, ground elevations, structural features, or water surface elevations are referred. Also, REFERENCE PLANE. The plane is called a TIDAL DATUM when defined by a certain phase of the tide. The following datums are ordinarily used on hydrographic charts:

> MEAN LOW WATER - Atlantic coast (U.S.), (see MEAN LOW WATER);
> MEAN LOWER LOW WATER - Pacific coast (U.S.) (see MEAN LOWER LOW WATER);
> LOW WATER DATUM - Great Lakes (U.S. and Canada); (see LOW WATER DATUM)

DEEP WATER - Water so deep that surface waves are little affected by the ocean bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.

DELTA - A sediment deposit, roughly triangular or fingerlike in shape, formed at a river mouth.

DEPTH - The vertical distance from a specified tidal datum (e.g., Mean Low Water) to the sea floor.

DEPTH OF BREAKING - The stillwater depth at the point where the wave breaks. Also BREAKER DEPTH.

DEPTH CONTOUR - See CONTOUR.

*DETRITAL (DETRITUS) - Pertaining to a loose material worn or broken away from a mass, as by the action of water, usually carried from inland sources by streams.

*DIFFRACTION - The progressive reduction in wave height that takes place when a wave spreads after passing the end of a barrier, such as a breakwater, into water areas in the shaddow of the barrier.

DIURNAL - Having a period or cycle of approximately one TIDAL DAY.

*DIVERGENCE - A decrease of wave strength evidenced by a lowering of wave heights, usually caused by submerged terrain features over which the w[illegible] has passed. See CONVERGENCE.

DOWNDRIFT - The direction of predominant movement of littoral materials in longshore transport.

*DRAGLINE - A power excavator that uses a drag bucket suspended from the end of its boom by a drum-controlled cable to dig material by pulling the bucket toward the operator.

*DRIFT BOLT - A large, headless spike, usually 0.5 inch or larger in diameter driven through large timber into another as a fastening device.

*DUNE - A hill, bank, bluff, ridge, or mound of loose, wind-blown material, usually sand.

*DURA-BAG - The proprietary name for a large bag made of woven nylon yarns with a lap opening for filling with sand. The lap closes and seals the sand in when the bag is full.

DURATION - In wave forecasting, the length of time the wind blows in nearly the same direction over the FETCH (generating area).

*"DUTCH TOE" - A "Dutch toe" is formed by leaving excess filter cloth at the toe of a revetment and lapping the excess back over a "sausage" of small stones or concrete blocks. The lapped-over filter cloth is then secured by placing the bottom row of armor units over or against it.

EBBTIDE - The period of tide between high water and the succeeding low water; a falling tide.

*EQUILIBRIUM - A state of balance or equality of opposing forces.

*ERCO BLOCK - A patented 8- by 8-inch concrete revetment block, 4 inches high, with a flat bottom, raised cobblelike top, and vertical holes to allow the escape of ground water, manufactured by Erco Systems of New Orleans. Also called GOBI BLOCK.

EROSION - The wearing away of land by the action of natural forces. On a beach, the carrying away of beach material by wave action, tidal currents, littoral currents, or by wind.

ESCARPMENT - A more or less continuous line of cliffs or steep slopes facing in one general direction which are caused by erosion or faulting. Also SCARP (Fig. A-1).

ESTUARY - (1) The part of a river that is affected by tides. (2) The region near a river mouth in which the freshwater of the river mixes with the saltwater of the sea.

FETCH - The area in which waves are generated by a wind having a rather constant direction and speed. Sometimes used synonymously with FETCH LENGTH.

FETCH LENGTH - The horizontal distance (in the direction of the wind) over which a wind generates waves or creates a WIND SETUP.

*FILTER CLOTH - A synthetic textile with openings that allow water to escape, but which prevents the passage of soil particles.

FINES - The smaller particles of a granular material, such as silt and clay in sandy soils or sand in sandy gravel.

*FLUVIAL - Produced or deposited by rivers.

FORESHORE - The part of the shore lying between the crest of the seaward berm (or upper limit of wave wash at high tide) and the ordinary low water mark, that is ordinarily traversed by the uprush and backrush of waves as the tides rise and fall (Fig. A-1).

*FRIABLE - Easily crumbled and eroded by moving water.

FRINGING REEF - A coral reef attached directly to an island or continental shore.

*GABION - A wire basket filled with stone or concrete rubble.

GENERATION OF WAVES - The creation and growth of waves caused by a wind blowing over a water surface for a certain period of time. The area involved is called the GENERATING AREA or FETCH.

GEOMORPHOLOGY - That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface, and the changes that take place in the evolution of landforms.

*GLACIAL - Resulting from the effects of a glacier.

*GLACIAL DRIFT - Applies to all the rock material transported and deposited by glaciers.

GLACIAL TILL - SEE TILL.

*GLACIER - A body of ice, consisting mainly of recrystallized snow flowing over land and persisting year around.

*GLACIO-LACUSTRIAN - Silt- or clay-sized material that has been deposited by settling out of glacial melt-water lakes.

*GOBI BLOCK - See ERCO BLOCK.

*GOBI-MAT - A mat comprising Gobi blocks factory-glued to filter cloth.

GRAVEL - See SOIL CLASSIFICATION.

GROIN - A shore protection structure built (usually perpendicular to the shoreline) to trap littoral drift or retard erosion of the shore.

GROIN FIELD - A series of groins acting together to protect a section of beach. Commonly called a GROIN SYSTEM.

*GROUT - A mixture of portland cement, fine aggregates (usually sand), and water. Usually used to seal openings or to fill bags or other containers.

*H-PILE - A straight length of structural steel, with an H-shaped cross section, designed for driving into earth.

*HARDWARE CLOTH - Wire screening with relatively small openings, as used for window screens.

HEADLAND (HEAD) - A high steep-faced promontory extending into the sea.

HEIGHT OF WAVE - See WAVE HEIGHT.

HIGH TIDE, HIGH WATER (HW) - The maximum elevation reached by each rising tide. See TIDE.

HIGHER HIGH WATER (HHW) - The higher of the two high waters of any tidal day. The single high water occurring daily during periods when the tide is diurnal is considered to be a higher high water.

HIGH WATER - See HIGH TIDE.

HIGH WATER LINE - In strictness, the intersection of the plane of mean high water with the shore. The shoreline delineated on the nautical charts of the National Ocean Survey is an approximation of the high water line. For specific occurrences, the highest elevation on the shore reached during a storm or rising tide, including meteorological effects.

*HOGWIRE - A stout, smooth wire fencing of the type normally used to enclose a pig sty.

*HOLOCENE - The present geological epoch, beginning 11,000 years ago.

HURRICANE - An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 75 mph (65 knots) for several minutes or longer at some points. TROPICAL STORM is the term applied if maximum winds are less than 75 mph.

*HYDROSEEDER (vegetation) - A device that injects seed into a water spray for rapid seeding of large areas.

*IMPERMEABLE - Not having openings large enough to permit passage of appreciable quantities of (1) sand or (2) water.

*IMPOUNDMENT BASIN - A water area in which the water motion has been stilled sufficiently to cause waterborne particles to be deposited.

INLET - (1) A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. (2) An arm of the sea (or other body of water) that is long compared to its width, and may extend a considerable distance inland. See also TIDAL INLET.

INSHORE (ZONE) - In beach terminology, the zone of variable width extending from the low water line through the breaker zone. Also SHOREFACE.

*INTERNATIONAL GREAT LAKES DATUM (IGLD) - The common datum used in the Great Lakes area based on mean water level in the St. Lawrence River at Father Point, Quebec, and established in 1955.

*INTERTIDAL ZONE - The land area that is alternately inundated and uncovered with the tides, usually considered to extend from mean low water to extreme high tide.

*ISOSTATIC - Pertaining to hydrostatic equilibrium of the earth's crust.

JETTY - On open seacoasts, a structure extending into a body of water, and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow. Jetties are built at the mouth of a river or tidal inlet to help deepen and stabilize a channel.

*JUMBO BLOCK - A large-sized Gobi block.

*JUMBO MAT - A Gobi-mat in which Jumbo blocks are used in lieu of the small-sized Gobi blocks.

*LACUSTRIAN - Of or pertaining to lakes. Lacustrian sediments are those deposited by lake wave action rather than by streams or ocean waves.

LEE - Shelter, or the part or side sheltered or turned away from the wind or waves.

LEEWARD - The direction toward which the wind is blowing; the direction toward which waves are traveling.

*LEO (LITTORAL ENVIRONMENT OBSERVATIONS) - A program through which local personnel are trained to make observations daily or more often at a given shore station and report on wave, wind and current conditions at the site.

LEVEE - An embankment or shaped mound for flood control or hurricane protection.

*LITHIC - Of or pertaining to stones.

LITTORAL - Of or pertaining to a shore, especially of the sea.

*LITTORAL COMPARTMENT - A segment of beach out of which or into which no littoral drift is transported by longshore currents.

LITTORAL CURRENT - See CURRENT, LITTORAL.

LITTORAL DEPOSITS - Deposits of littoral drift.

LITTORAL DRIFT - The sedimentary material moved in the littoral zone under the influence of waves and currents.

LITTORAL TRANSPORT - The movement of littoral drift in the littoral zone by waves and currents. Includes movement parallel (longshore transport) and perpendicular (onshore-offshore transport) to the shore.

LITTORAL TRANSPORT RATE - Rate of transport of sedimentary material parallel to or perpendicular to the shore in the littoral zone. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LONGSHORE TRANSPORT RATE.

LITTORAL ZONE - In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.

*LOK-GARD BLOCK - A patented flat concrete block with tongue-and-groove edges. The blocks lock in place when pushed together to form a revetment.

*LONGARD TUBE - A patented two-ply flexible tube manufactured in 10-, 40-, and 69-inch diameters. The outer ply is a tough, black, high-density, polypropylene woven fabric that is resistant to ultraviolet light, oil, rot, and chemicals. The inner ply is an impermeable, low-density polyethylene film. The tube is filled with sand by pumping sand slurry into one end. The sand deposits in the tube and the water exits the far end of the tube until the tube is filled with sand.

LONGSHORE - Parallel to and near the shoreline.

LONGSHORE CURRENT - See CURRENT, LONGSHORE.

LONGSHORE TRANSPORT RATE - Rate of transport of sedimentary material parallel to the shore. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LITTORAL TRANSPORT RATE.

LOWER LOW WATER (LLW) - The level of the lower tide when two low tides occur on a single day. The level of the single low water occurring daily during periods when the tide is diurnal is considered to be LLW.

LOW TIDE, LOW WATER (LW) - The minimum elevation reached by each falling tide. See TIDE.

LOW WATER DATUM (LWD) - An approximation to the plane of mean low water that has been adopted as a standard reference plane. See also DATUM PLANE and DATUM, CHART.

LOW WATER LINE - The intersection of any standard low tide datum plane with the shore.

MANGROVE - A tropical tree with interlacing prop roots, confined to low-lying brackish areas.

*MARINE DRIFT - Glacial drift deposited under seawater.

MARSH - An area of soft, wet, or periodically inundated land, generally treeless and usually characterized by grasses and other low growth.

*MASTIC - Asphalt applied hot to seal voids in a rubble mound.

MEAN HIGHER HIGH WATER (MHHW) - The average height of the daily higher high waters over a 19-year period.

MEAN HIGH WATER (MHW) - The average height of the daily high waters over a 19-year period.

MEAN LOWER LOW WATER (MLLW) - The average height of the daily lower low waters over a 19-year period.

MEAN LOW WATER (MLW) - The average height of the low waters over a 19-year period.

MEAN SEA LEVEL - The average hourly height of the surface of the sea for all stages of the tide over a 19-year period. Not necessarily equal to MEAN TIDE LEVEL.

MEAN TIDE LEVEL - A plane midway between MEAN HIGH WATER and MEAN LOW WATER. Not necessarily equal to MEAN SEA LEVEL. Also called HALF-TIDE LEVEL.

MEDIAN DIAMETER - The diameter which marks the division of a given sand sample into two equal parts by weight, one part containing all grains larger than that diameter and the other part containing grains smaller.

*MIOCENE - The epoch of the Tertiary period between the Oligocene and the Pliocene epochs. The time period postulated to be from 25 to 13 million years before the present.

*MODULE - A structural component, a number of which are joined together to make up the whole.

*MONOSLAB (TURFBLOCK) - A patented concrete block 24- by 16- by 4.5- inches high with slots for filling with topsoil and grass-seeding, manufactured by Grass Pavers Limited, of Royal Oak, Michigan. The blocks have raised surface treads for use in parking lots to prevent vehicular traffic from killing the grass.

*NAMI RING - A patented concrete revetment module in the shape of a short section of pipe. The modules are placed hole-up on the slope with sides in tight contact.

*NATIONAL GEODETIC VERTICAL DATUM (NGVD) - The datum of the United States geodetic level net. Mean Sea Level varies slightly from this datum from place to place along the shores of the nation.

NEARSHORE - In beach terminology an indefinite zone extending seaward from the shoreline well beyond the breaker zone (Fig. A-1).

NOURISHMENT - The process of replenishing a beach. It may be brought about naturally, by longshore transport, or artificially, by the delivery of materials dredged or excavated elsewhere.

OFFSHORE - (1) (Noun) In beach terminology, the comparatively flat zone of variable width, extending from the breaker zone to the seaward edge of the Continental Shelf. (2) (Adjective) A direction seaward from the shore (Fig. A-1).

OVERTOPPING - Passing of water over the top of a structure as a result of wave runup or surge action.

*PEAT - The residual product produced by the partial decomposition of organic matter in marshes and bogs.

*PEAT POT (vegetation) - A pot formed from compressed peat and filled either with soil or peat moss, in which a plant or plants, grown from seed, are transplanted without being removed from the pot.

PERCHED BEACH - A beach or fillet of sand retained above the otherwise normal profile level by a submerged dike or sill.

PERCOLATION - The process by which water flows through the interstices of a sediment.

*PERMEABLE - Having openings large enough to permit passage of appreciable quantities of (1) sand or (2) water.

PHI GRADE SCALE - A logarithmic transformation of the Wentworth grade scale for size classifications of sediment grains based on the negative logarithm to the base 2 of the particle diameter (D) in millimeters, $= \log_2 D$. See SOIL CLASSIFICATION.

PIER - A structure, usually of open construction, extending out into the water from the shore, to serve as a landing place, a recreational facility, etc., rather than to afford coastal protection. In the Great Lakes, a term sometimes improperly applied to jetties.

PILE - A long, heavy timber or section of concrete or metal to be driven or jetted into the earth or seabed to serve as a support or protection.

PILE, SHEET - A pile with a generally slender flat cross section to be driven into the ground or seabed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

PILING - A group of piles.

*PIPING - Fluidizing of backfill or an embankment to the extent that large quantities of material are pumped by wave action through holes under or through a bulkhead or revetment.

*PLACE IN SECTION - Install the components of a structure in their proper place as indicated by a cross-section drawing, applied mostly to rubble-mound structures.

*PLASTICITY - As applied mainly to clay: the relative ease with which the material yields or deforms under pressure.

PLATEAU - A land area (usually extensive) having a relatively level surface raised sharply above adjacent land on at least one side; table land. A similar undersea feature.

*PLEISTOCENE - The geological epoch proceding the Holocene. The period postulated to be between 2.5 million and 11,000 years before the present.

*PLIOCENE - The most recent epoch of Teriary time. The period postulated to be between 13 and 2.5 million years before the present.

*PLUG - A core containing both plants and underlying soil. Usually cut with a cylindrical coring device and transplanted to a hole cut by the same device.

POCKET BEACH - A beach, usually small, in a cove or between two littoral barriers.

POINT - The extreme end of a cape, or the outer end of any land area protruding into the water, usually less prominent than a cape.

PORE PRESSURE - The pressure exerted by the water contained in the spaces between individual soil particles which acts either on the particles themselves, or on structural elements embedded in, or adjacent to, the soil.

PROFILE, BEACH - The intersection of the ground surface with a vertical plane; may extend from the top of the dune line to the seaward limit of sand movement (Fig. A-1).

PROPAGATION OF WAVES - The transmission of waves through water.

*PROPAGULE - A structure (as a cutting, a seed, or a spore) that propgates a plant.

*QUADRAT - A rectangular plot used for ecological or population studies.

*RAVELLING - The progressive deterioration of a revetment under wave attack.

*REACH (Nautical) - A length, distance, or leg of a channel or other watercourse.

*REBAR - A bar of reinforcing steel normally used in reinforced concrete, numbered in accordance with the bar diameter in eighths of an inch. (A No. 4 rebar is 0.5 inch in diameter.)

REEF - An offshore consolidated rock hazard to navigation.

REEF, FRINGING - See FRINGING REEF.

REFERENCE PLANE - See DATUM PLANE.

REFRACTION (of water waves) - (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than the part still advancing in deeper water, causing the wave crest to bend toward alinement with the underwater contours. (2) The bending of wave crests by currents.

REVETMENT - A facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.

RIP CURRENT - A strong surface current flowing seaward from the shore. It usually appears as a visible band of agitated water and is the return movement of water piled up on the shore by incoming waves and wind. With the seaward movement concentrated in a limited band, its velocity is somewhat accentuated.

RIPRAP - A layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

RUBBLE - (1) Loose, angular, waterworn stones along a beach. (2) Rough, irregular fragments of broken rock or concrete.

RUBBLE-MOUND STRUCTURE - A mound of random-shaped and random-placed stones or concrete rubble protected with a cover layer of selected stones, or specially shaped concrete armor units. (Armor units in primary cover layer may be placed in orderly manner or dumped at random.)

RUNUP - The rush of water up a structure or beach on the breaking of a wave. Also UPRUSH. The amount of runup is the vertical height above stillwater level that the rush of water reaches.

SAND - See SOIL CLASSIFICATION. Generally, coarse-grained soils having particle diameters between 0.05 and approximately 5 millimeters.

*SANDBAG - A cloth bag designed to be filled with sand or grout and used as a module in a shore protection device.

*SAND FILLET - An accretion trapped by a groin or other protrusion in the littoral zone.

*SANDGRABBER - A patented permeable structure composed of hollow concrete blocks similar to but larger than commercial building blocks. The blocks are tied together with U-shaped rods passing through the hollows from one side to the other. When the structure is placed on the beach face, waves carry sand through the hollows and build up a berm behind the structure.

*SANDLINE - On a beach profile: the line demarking the top of the sand layer.

*SAND PILLOW - A woven acrylic, ultraviolet-resistant sandbag, holding about 100 pounds of sand, manufactured by the Monsanto Textiles Company.

SCARP, BEACH - An almost vertical slope along the beach caused by erosion by wave action. It may vary in height from a few inches to several feet, depending on wave action and the nature and composition of the beach (Fig. A-1).

*SCARPING - Erosion of a beach in a manner that causes carp formation.

SCOUR - Removal of underwater material by waves and/or currents, especially at the base or toe of a shore structure.

*SCREW ANCHOR - A type of metal anchor that can be screwed into the bottom for holding power.

SEAS - Waves caused by wind at the place and time of observation.

SEAWALL - A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. See also BULKHEAD.

*SEMIDIURNAL TIDE - A tide with two high and two low waters in a tidal day, each high and each low being approximately equal in stage.

SETUP, WAVE - Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

*SETUP, WIND - (1) The vertical rise in the stillwater level on a body of water caused by piling up of the water on the shore due to wind action. Synonymous with WIND TIDE and STORM SURGE. STORM SURGE is usually reserved for use on the ocean and large bodies of water. WIND SETUP is usually reserved for use on reservoirs and smaller bodies of water.

SHALLOW WATER - (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water.

*SHAPE-SIZED - Concrete rubble broken into pieces of no greater than a given aspect ratio.

SHEET PILE - See PILE, SHEET.

*SHOAL - (noun) A rise of the sea bottom due to an accumulation of sand or other sediments. (verb) - (1) To <u>become</u> shallow gradually. (2) To <u>cause</u> to become shallow. (3) To <u>proceed</u> from a greater to a lesser depth of water.

SHORE - The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach (Fig. A-1).

SHORELINE - The intersection of a specified plane of water with the shore or beach (e.g., the high water shoreline would be the intersection of the plane of mean high water with the shore or beach). The line delineating the shoreline on National Ocean Survey nautical charts and surveys approximates the mean high water line.

*SILL - A low offshore barrier structure whose crest is usually submerged, designed to retain sand on its landward side.

SILT - See SOIL CLASSIFICATION. Generally, refers to fine-grained soils having particle diameters between 0.002 and 0.05 millimeter.

*SIZED RUBBLE - Shape-sized rubble.

*SLIP CIRCLE - The curved face along which an unbalanced bluff slides to reestablish equalibrium.

SLOPE - The degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 on 25, indicating 1 unit vertical rise in 25 units of horizontal distance; or in degrees from horizontal.

*SLOUGHING - A mass wasting process similar to block-calving where a weakened mass of soil fails and moves downslope.

*SOIL CLASSIFICATION (size) - The arbitrary division of a continuous scale of soil grain sizes into definite ranges. Soils are described (classified) according to which range the majority of the constituent particles (by weight) belong. (e.g., sand, silt, clay, etc.) There are many classifications; the two most often used are shown in Table A-1. (See GEOMETRIC MEAN DIAMETER, MEDIAN DIAMETER, and PHI GRADE SCALE).

*SOLIFLUCTION - The slow downslope flow of weak saturated soil. The flowing layer is generally underlain by frozen soil.

*SPECIFICATIONS - A detailed description of particulars, such as size of stone, quality of materials, contractor performance, terms, and quality control.

SPIT - A small point of land or a narrow shoal (usually of sand) projecting into a small body of water from the shore.

*SPREADER - A device (as a bar) holding two linear elements (as lines, guys, rails) apart and (usually) taut.

*SPRIG - A single plant with its relatively bare roots, as pulled apart from a clump and used for transplanting.

*STA-POD - A concrete module with a vertical cylindrical body and four legs. When a number of modules are placed closely together in the nearshore zone, they form a permeable wave barrier.

STILLWATER LEVEL - The elevation that the surface of the water would assume if all wave action were absent.

STORM SURGE - A rise above normal water level on the open coast due to the action of wind on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric-pressure reduction as well as that due to wind stress. See SETUP, WIND.

*SUBANGULAR - Angular sand particles that have been abraded by stream transport or wave action only enough to remove sharp edges.

Table A-1. Soil classification.

Wentworth Scale (Size Description)		Phi Units ϕ *	Grain Diameter, D (mm)	U.S. Std. Sieve Size	Unified Soil Classification (USC)	
Boulder					Cobble	
		-8	256			
Cobble			76.2	3"		
		-6	64.0		Coarse	Gravel
			19.0	3/4"		
					Fine	
Pebble			4.76	No. 4		
		-2	4.0		Coarse	Sand
Granule						
		-1	2.0	No. 10		
Sand	Very Coarse					
		0	1.0			
	Coarse					
		1	0.5		Medium	
	Medium		0.42	No. 40		
		2	0.25			
	Fine				Fine	
		3	0.125			
	Very Fine		0.74	No. 200		
		4	0.0625			
Silt						
		8	0.00391		Silt or Clay	
Clay						
		12	0.0024			
Colloid						

* $\phi = \log_2 D$ (mm)

*SUBROUNDED - Sand particles that have been abraded sufficiently to make them nearly spherical, ovate, or oblate.

*SUBSIDENCE - Sinkage of a structure or fill into soft bottom material.

*SURGEBREAKER - A patented breakwater system comprised of precast concrete modules of triangular section 4 feet high, with a base 8 feet wide and 7 feet long. The modules are set end-to-end on the bottom in shallow water, without intermodule bonding. Openings through the blocks dissipate wave energy in a profusion of water jets.

SUSPENDED LOAD - The material moving in suspension in a turbulent fluid.

SWELL - Wind-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and longer period, and has flatter crests than waves within their fetch (See SEAS).

TIDAL PERIOD - The interval of time between two consecutive like phases of the tide.

TIDAL RANGE - The difference in height between consecutive high and low (or higher high and lower low) waters.

TIDE - The periodic rising and falling of the water that results from gravitational attraction of the moon and sun and other astronomical bodies acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as TIDAL CURRENT, reserving the name TIDE for the vertical movement.

TIDE, EBB - See EBBTIDE.

TIDE POOL - A pool of water remaining on a beach or reef after recession of the tide.

TIDE STATION - A place at which tide observations are being taken. It is called a primary tide station when continuous observations are to be taken over a number of years to obtain basic tidal data for the locality. A secondary tide station is one operated over a short period of time to obtain data for a specific purpose.

*TIE ROD - A steel rod used to tie back the top of a bulkhead or seawall. Also, a U-shaped rod used to tie Sandgrabber blocks together, or a straight rod used to tie Nami rings together.

*TILL - Unstratified glacial drift consisting of an unsorted mixture of clay, sand, gravel, and boulders intermingled.

*TOMBOLO - A bar or spit that connects or "ties" an island or a breakwater to shore.

TOPOGRAPHY - The configuration of a surface, including its relief, the position of its streams, roads, building, etc.

*TRADE WIND - An easterly wind in the equatorial zone.

TROPICAL STORM - A tropical cyclone with maximum winds less than 75 mph.

TROUGH OF WAVE - The lowest part of a waveform between successive crests. Also that part of a wave below stillwater level.

TSUNAMI - A long-period wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave."

*TURFBLOCK - See MONOSLAB.

*TURFSTONE - A patented concrete paving-slab module, 3.25 inches thick, with square opening through which grass can grow; sometimes used as a revetment module.

UNDERTOW - Tidal undercurrent generated by wave action similar to riptide or rip current.

UPDRIFT - The direction opposite that of the predominant movement of littoral materials in longshore transport.

UPRUSH - The rush of water up onto the beach following the breaking of a wave. See RUNUP.

*WAKE (boat) - Waves generated by the motion of a vessel through the water.

*WALE - A horizontal beam on a bulkhead used to transfer horizontal loads against the structure laterally along it and to hold it in a straight alinement.

WATERLINE - A juncture of land and sea. This line migrates, changing with the tide or other fluctuation in the water level. Where waves are present on the beach, this line is also known as the limit of backrush. (Approximately the intersection of the land with the stillwater level.)

WAVE - A ridge, deformation, or undulation of the surface of a liquid.

*WAVE CLIMATE - The normal seasonal wave regimen along a shoreline.

WAVE CREST - See CREST OF WAVE.

*WAVE DAMPING - Reduction of wave height by a structure or device through which the wave passes.

WAVE DIRECTION - The direction from which a wave approaches.

*WAVE-ENERGY FLUX - The total amount of wave energy delivered to a given shore segment over a season or year, broken down by direction. The longshore component of the flux on either side of the normal-to-shore is indicative of the gross potential rate of longshore transport in the component direction. The difference between components in each direction is indicative of the net potential longshore transport rate in the predominant direction.

WAVE HEIGHT - The vertical distance between a crest and the preceding trough.

WAVELENGTH - The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

*WAVE-MAZE SYSTEM - The floating tire breakwater system patented by H. Morgan Noble.

WAVE PERIOD - The time for a wave crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point.

WAVE PROPAGATION - The transmission of waves through water.

WAVE REFRACTION - See REFRACTION (of water waves).

WAVE SETUP - See SETUP, WAVE.

WAVE STEEPNESS - The ratio of the wave height to the wavelength.

WAVE TRAIN - A series of waves from the same direction.

WAVE TROUGH - The lowest part of a wave form between successive crests. Also that part of a wave below stillwater level.

*WEEP HOLE - A hole through a solid revetment, bulkhead, or seawall for relieving pore pressure.

WINDROW - A line of detritus, cobbles, or other loose materials left on a beach.

WIND SETUP - See SETUP, WIND.

WIND TIDE - See SETUP, WIND and STORM SURGE.

WINDWARD - The direction from which the wind is blowing.

WIND WAVES - (1) Waves being formed and built up by the wind. (2) Loosely, any waves generated by wind.

*Z-WALL - A patented concrete breakwater system composed of reinforced concrete slabs 6 feet high and 14 feet long set on edge in zigzag fashion and joined together with large hinge bolts.

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1. Breakwaters. 2. Bulkheads. 3. Groins. 4. Hydraulic structures. 5. Revetments. 6. Seawalls. 7. Shore protection. 8. Shoreline changes. 9. United States. 10. Vegetation. I. Shoreline Erosion Control Demonstration Program. II. Coastal Engineering Research Center (U.S.). III. Moffatt & Nichol. IV. Series: Contract (Coastal Engineering Research Center (U.S.)) ; DACW72-79-R-0009.

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